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Structural Analysis of the Masonry Envelope of Ica Cathedral, Peru

ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master’s Thesis

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University of Minho

UNIVERSITAT POLITÈCNICA DE CATALUNYA

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"as much as necessary, but as little as possible"
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This thesis not only represents my work on the structural analysis of the Ica Cathedral in Peru. It represents a milestone culminating in one of the most productive years of my life: the year I have spent doing the SAHC masters.

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Abstract

Built in 1759, the Ica Cathedral is one of the four prototype buildings considered as a part of the Seismic Retrofitting Project (SRP) initiative of the Getty Conservation Institute. Under this project, the Ica Cathedral is considered representative of religious buildings built in coastal cities with a structural system consisting of thick mud brick walls and quincha vaults and domes. The Cathedral was also declared as a national monument of Peru in 1982. The Ica Cathedral currently exists in a state of disuse and in a heavily damaged state as a result of the 2007 Pisco earthquake and another subsequent earthquake in 2009.

This thesis primarily addresses the structural analysis of the masonry envelope of the Ica Cathedral. Based on existing information, a 3D model of the walls, base courses and bell towers of the structure was constructed. Consequently this model was used for creating a FE model in Midas FX+ for DIANA software in order to investigate various hypothesis of structural damage. A review of the results of the experimental campaign carried out under the banner of the SRP and existing literature was performed in order to input material properties into the numerical model. The created FE model was then updated in terms of material properties on the basis of ambient vibration tests performed on the cathedral recently. The nonlinear behaviour of the structure was investigated in the light of various static and dynamic simulations. Additionally the validated numerical model was also used to evaluate the effectiveness of a proposed strengthening scheme.

The present work was able to produce a numerical model validated in terms of existing damage. It was also able to make assessments of the seismic capacity of the cathedral and the effectiveness of the proposed strengthening in this regard. Nevertheless, further numerical modelling, including a combined model of the timber and masonry structures and subsequent structural analysis needs to be performed in order to have a more conclusive assessment on the behaviour of Ica Cathedral.
Resumo


Esta tese aborda principalmente a análise estrutural das paredes de alvenaria exteriores da Catedral. Com base nas informações existentes foi desenvolvido um modelo 3D das fundações, paredes e torres. Este modelo foi depois utilizado para a criação de um modelo de elementos finitos (EF) em Midas FX + para o programa DIANA a fim de investigar várias hipóteses de danos estruturais. Uma revisão dos resultados da campanha experimental realizada no âmbito do SRP e da literatura existente foi efetuada a fim de determinar as propriedades dos materiais a utilizar no modelo numérico. O modelo EF foi então atualizado em termos de propriedades dos materiais com base em testes de vibração ambiental realizados na catedral recentemente. O comportamento não linear da estrutura foi investigado em função de diferentes simulações estáticas e dinâmicas. Além disso, o modelo numérico validado foi igualmente utilizado para avaliar a eficácia de um esquema de reforço proposto.

O presente trabalho foi capaz de produzir um modelo numérico validado em termos de danos existentes. Foi também capaz de fazer uma avaliação da capacidade sísmica da catedral e da eficácia da solução de reforço proposta. Novos modelos numéricos, incluindo um modelo combinado das estruturas de madeira e alvenaria e subsequente análise estrutural deveria ser executada, a fim de ter uma avaliação mais conclusiva sobre o comportamento de Catedral de Ica.
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Chapter 1

Introduction
Structural analysis of historical masonry buildings represent some of the most challenging problems in the field of structural engineering. This thesis is focussed on the structural analysis and in particular the numerical modelling of such an construction: The Ica Cathedral in Peru.

Built in 1759, the Ica Cathedral is located in Ica, one of the most important cities on the coast of Peru. The cathedral is located in the corner of the city's historic centre and main plaza, at the intersection of two important streets: Jirón Libertad and Jirón Bolivar. Though currently in disuse after the 2007 Pisco earthquake, which severely damaged the structure, the cathedral has been used as a place of worship in the city under the ownership of various religious orders, changing hands numerous times with history.

The church was originally built by the Society of Jesus but following their expulsion in 1767 from the Viceroyalty of Peru, the Mercedarian Order took over the church in 1780. However the church is presently owned by the Roman Catholic Diocese of Ica which has been in possession of the church since their establishment in 1946.

Like most churches established by Jesuits, the Ica Cathedral follows the Jesuit typology modelled on the Church of Gesù in Rome. The basic layout of the cathedral consists of: a single central nave with four structural bays, a transept, an altar, and a choir loft-all covered by barrel vaults. Aisles flank the main nave on both sides and these aisles are covered with a number of small domes. Another large dome exists in the structure at the crossing of the nave and the transept. The domes and the barrel vaults are constructed with wooden arches and quincha (a local word used to indicate mixed timber-earthen construction). The lateral walls of the church are thick and constructed in mud brick masonry over courses of fired brick and rubble stone—the thickness of these layers vary in various parts of the church. The aisles are separated from the central nave by quincha pillars and arches covered with layers of gypsum plaster and mud. These structural elements are discussed in more detail in 2.2.

Many similarities also exist with the Cathedral of Lima but these were incorporated mostly after the reconstruction works carried out in the late eighteenth century. Reconstruction and repair works were carried out in the cathedral after earthquakes which occurred in 1813.
and 1942, and these are discussed in latter sections of this chapter. The structure is currently in a heavily damaged situation after the 2007 earthquake. The current damage includes primarily: partial collapse of the central dome, collapse of the barrel vaults near the facade, total collapse of the roof over one of the bays near the southern aisle and also loss of materials from various pillars and pilaster bases in the structure.

The following section of this chapter introduce the objectives that were considered before dealing with the case study being considered. The next section outlines how the work carried out to fulfil these objectives are organized in the main body of the thesis that follows.

1.1 Objectives

The main objectives of this thesis are summarized below:

- To understand the geometrical and mechanical characterization of the structure.
- To perform a review of material properties from the experimental campaign conducted on the same structure, existing literature in order to extract material parameters to be input into a finite element model of the structure.
- To build a 3D finite element model of the masonry envelope.
• To update the parameters of the model with results and observations from the site visit to the structure carried out by the team from the University of Minho.
• To perform non linear finite element analyses on the numerical model: self weight loading, mass proportional pushovers and non linear dynamic loading in order to assess the safety and seismic capacity of the structure.
• To validate the model in terms of its ability to reproduce the existing observed damages in the structure.
• To use the validated model in order to simulate the proposed strengthening and evaluate its effect on the seismic capacity of the structure.

1.2 Organization

The work carried out in order to meet the objectives of the thesis, is organized into six chapters:

Chapter 1 introduces the motivation behind the thesis and its objectives. Also the organization of the work is presented here.

Chapter 2 presents both a historical and current survey of the cathedral. This chapter includes detailed information on the structural elements modelled in this thesis. The damage survey of the cathedral is also included.

Chapter 3 deals with three important parts of the thesis. Initially the finite element model is presented. The second part presents the strengthening proposed for the structure for which also another numerical model was constructed. The third part of this chapter deals with a review of all the material data used in the numerical model: constitutive law, mechanical characterization etc.

Chapter 4 presents the results of the dynamic identification tests carried out on the structure and how they were used to calibrate the model. The chapter also presents results of the eigenvalue analysis of the numerical model.
Chapter 5 presents the results of nonlinear analysis carried out on numerical model. The validated numerical model was also used to assess the proposed strengthening of the structure.

Chapter 6 presents conclusions from the works carried out. Additionally a few recommendations for further work are also mentioned.
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Chapter 2

The Ica Cathedral
2.1 History

2.1.1 Historical Background

The city of Ica of which the cathedral is one of the most important buildings, has been inhabited since pre-Inca and Inca times. However, historically the city was claimed to be found by the Spanish conquerors in 1563. The first members of the Society of Jesus (Compañía de Jesus) who built the cathedral arrived in Peru in between 1567-1568. It is to be noted that the Jesuits were neither monks nor friars. Established in 1534 at the University of Paris, they were devoted to educational purposes. They often operated and lived in colleges. To maintain these educational establishments they often invested in farms which served for the purpose of money and also allowed them to interact with the rural population. Likewise in Peru they were associated with a number of farms in the Ica region at places like Mamacona, Belén, Caucato and San Jerónimo.

The Jesuits arrived in Ica in 1739 to open the College of San Luis Gonzaga. The building housing the cathedral at present was in fact originally intended to house this college. Work on the structure started in 1746 and was completed in 1759 if the inscription on the dome of the cathedral is to be believed. However the construction continued even after this date if other historical sources are to be believed, as master mason Gaspar Urrunaga was apparently working at the Jesuit college in Ica from 1762 to 1767, when the Jesuits were eventually expelled from Viceroyalty of Peru (Harth-Terré, 2003).

2.1.2 Administrative Changes

After the expulsion of Jesuits from Peru, the Ica Cathedral changed hands a number of times among various religious orders who not only occupied and renamed the cathedral but also altered and repaired the structure.

Notable among these are the Mercedarians who took over the church in 1780. The college was renamed as Colegio Mercedario de San Jose and the church was dedicated to Our Lady of the Mercy. The main altarpiece was also elaborated in 1802 by them. The church along
with a number of other buildings was damaged in an earthquake in 1813. The church was repaired by the Mercedarians following this. Notably the Neoclassical facade that exists now was built following the damage in this earthquake. The choir loft that exists presently was also possibly built at that time as such choir lofts are very uncharacteristic of Jesuit churches.

The Mercedarians also had to leave Ica during 1821-1824 when Peru gained its independence and many religious orders were being suppressed. The college took back its former name of San Luis Gonzaga and the church took the name of Iglesia Matriz de San Jeronimo from a neighbouring church, which used to be the city's main church previously but was damaged in the earthquake of 1813. Several alterations were carried out in the nineteenth century but notable among these was the repairing of damages in 1874 which occurred during another earthquake in 1868.

The year of 1946 finally saw the establishment of the Roman Catholic Diocese of Ica with the former Jesuit College as its cathedral. The cathedral functioned as a place of worship for the diocese till the 2007 earthquake when it was damaged extensively. Even now the cathedral is still used by the priests of the diocese to celebrate mass occasionally in the damaged structure.

### 2.1.3 Church of Gesù

The floor plans of most Jesuit churches are based on the Church of Gesù in Rome and the Ica Cathedral is no exception. Typically Jesuit churches have floor plans which are in the form of a cruciform or a rectangle. The nave of such churches is often covered by very high barrel vaults and is flanked by low side aisles. These are divided into a number of square spaces, each covered by a dome. They continue to form the shallow transept which in turn is also covered with a barrel vault. The crossing of such churches are typically covered with a large dome holding up a lantern. However the Ica Cathedral shows a marked deviation from the floor plan of the Church of Gesù in the fact that the altar terminates in a flat wall instead of a semi-circular apse. The similarities in their floor plan are better elucidated in Figure 2.
2.1.4 Seismic History

The Nazca and South American tectonic plates meet near the Peruvian coast and earthquakes due to thrust faulting make Peru a very seismic zone. Historically, the Ica Cathedral has suffered damage due to a number of earthquakes which affected Peru after it was built. These earthquakes are tabulated in Table 1. It is to be noted that the damages incurred after these historical earthquakes were repaired and the church is currently in a state of disuse after the 2007 Pisco earthquake. Even more recently the already existing damages after the 2007 earthquake were aggravated by another earthquake in 2009.

Table 1: Historic seismic events affecting the Ica Cathedral

<table>
<thead>
<tr>
<th>Date of Occurrence</th>
<th>Magnitude</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>2009</td>
<td>5.8</td>
<td>Central coast of Peru</td>
</tr>
<tr>
<td>2007</td>
<td>8.0</td>
<td>Near the coast of central Peru</td>
</tr>
<tr>
<td>1942</td>
<td>8.2</td>
<td>Off the coast of central Peru</td>
</tr>
<tr>
<td>1868</td>
<td>9.0</td>
<td>Africa, Peru (now Chile)</td>
</tr>
<tr>
<td>1813</td>
<td>7.5</td>
<td>Ica, Peru</td>
</tr>
</tbody>
</table>
2.2 The Structure

2.2.1 Urban Context

Located at the intersection of two important streets in the historic centre of Ica: Jirón Libertad and Jirón Bolivar, the Ica Cathedral is located in the corner of an urban block rather than the middle of a city block. This makes the building more vulnerable in the event of an earthquake. On the east, near the front facade there is a 280m² walled forecourt. On the southern side, the church is an adjacent partly to a cloister and single storey mud brick construction which previously housed the Jesuit College and now houses a university. A three storey concrete structure and fired brick structure lie right next to the cathedral from the west.

Figure 3: Location and surroundings of Ica Cathedral in the historic centre of Ica

2.2.2 Structural Description

The cathedral has rectangular 22.5×48.5m floor plan oriented along the east-west axis. The total plan area of 1,075m² is organised into ten functional spaces which are separated by changes in floor level, interior pillars, pilasters and piers. More important than functionally
the Ica Cathedral can be divided into two substructures from the point of structural response:

- The masonry envelope and the bell towers
- The timber framed interior structure

This thesis is focused on the numerical modelling of mostly the first substructure, being carried out in parallel with another thesis on the timber structure (Ciocci, 2015). Consequently, the sections that follow contain a more detailed structural description of the substructure being modelled here. A brief overview of structural details of the latter substructure is also provided.

Structural details of the masonry envelope and the bell towers which are very important for the finite element model developed are organised into the following sections: Foundation and Base Course; Fired Brick Facade; Load Bearing Lateral Walls; Bell Towers; Openings in the masonry envelope/ Entrances to the cathedral.

*Foundation and Base Course*

The Ica Cathedral is constructed over a base course constructed in fired brick masonry, rubble stone masonry or a combination of both. The base course in turn lies over foundations consisting of rubble stone masonry walls. The configuration and dimensions of the base course as well as the foundation vary considerably in the structure. However information about this is available only from the very limited number of wall prospections performed in the masonry. Information available from such prospections (Figure 4) are elucidated in the detailed description of various structural components that follows this section.
Figure 4: Floor plan elucidating where wall prospections were performed (Drawing: José Garcia Bryce and Mirna Soto for the GCI)

Fired Brick Facade

The 21 m long facade is constructed in fired brick masonry with lime mortar. The facade which was constructed in Neoclassical style after getting extensively damaged in 1813 due to an earthquake, largely defines the exterior appearance of the cathedral. The facade is characterised by the presence of large arched doors, Corinthian columns flanking them and a pediment on top. It is very interesting to note that the front facade reduces in its thickness and henceforth its slenderness ratio with height. The base is the thickest with a thickness of almost 2.25m while at the top it is about 0.30m thick. The facade rests on a foundation of rubble stone that is wider than the wall above the ground projecting almost 0.15m beyond the face of the wall and extends for a depth of 1m. A lack of proper connection is seen between the pediment and the lower part of the facade. The weak connection along with the highly changing slenderness ratio along with height of the facade makes the pediment highly susceptible to lateral overturning. Then, a choir loft, which was also added most probably to the cathedral at the same time as the facade was constructed, has a 0.10m embedment of its timber joists into the facade. The bases of the bell towers which are also constructed in brick masonry project from the two ends of the brick facade.
**Lateral Walls**

The two lateral walls in the church are both constructed in masonry of mud bricks and mud mortar. They have a slenderness ratio of approximately 3.35 and can be considered very thick. In fact their high thickness and consequently slenderness ratio (less than 5) make them very resistant to lateral overturning during earthquakes (Tolles et al., 2002). Though only a single wall prospection was performed per each of the lateral wall during previous investigations, it is known that the configuration of the foundation and base course is different in the lateral walls. The northern lateral wall rests over a 0.90m high fired brick masonry base course on top of a 0.40m rubble stone masonry wall. The height of the base course in southern lateral wall is 0.60m while the rubble stone wall is 0.48m deep. The lateral walls have a series of piers also constructed of mud bricks which lie behind each of the wooden pilasters in side aisles. Fired brick reinforcement is used at the connections between the piers and the lateral walls. These piers also have brick bases, but the bricks of these brick bases do not interlock with the brick base course of the adjacent lateral walls.

**Bell Towers**

The cathedral possesses two bell towers each approximately $3.80 \times 3.80$ m in plan. The bell tower structure consists of timber framed upper part which rests on a fired brick base projecting from the facade. The upper timber structure consists of vertical wooden posts which are connected to wooden plates that are embedded in the brickwork of the base. Horizontal wooden joists connected to the posts with half lap joints are used to create intermediate levels within the upper structure. Diagonal wooden framing is used to reinforce the posts. The timber framing of the bell tower is wrapped with a wire mesh lath covered with cement plaster finish. Originally however there used to be a gypsum plaster finish on canes which wrapped the voids in between the posts. In fact the entrance to the choir loft and the roof is provided by an opening in the southern bell tower which leads to a spiral staircase. It is also interesting to note that the nature of base course and foundation has been discovered to be markedly different at the connection of the bell towers to the walls. Detailed information is available only about the nature of this at the connection between
the northern bell tower and wall, where the rubble stone base course is interspersed with courses of bricks and extends up to a height of 1.75m above the floor level above. Then, there exists fired brick masonry for another 3m. Constructive details of the bell tower structure are explained in more detail in Figure 5.

![Figure 5: Southern bell tower structure (Drawing: José Garcia Bryce and Mirna Soto for the GCI)](image)

**Openings in the masonry envelope/ Entrances to the cathedral**

There are a total of three entrances to the cathedral. The main entrance is a pair of large arched doors present in the fired brick front facade facing the east. Access is also provided directly to the nave of the church by another set of large doors present in the northern lateral wall along Jirón Cajamarca. A third entrance also exists along the northern lateral wall which provides direct entrance to the sacristy. Access to the roof and the choir loft is provided through an entrance in the south eastern bell tower.
The interior timber framed structure of the cathedral comprise of: Quincha Pillars and Pilasters; The Roof Structure.

**Quincha Pillars and Pilasters**

The main nave of the cathedral is separated from the side aisles by a number of wood-framed quincha pillars. These pillars spaced 5m apart from each other support not only the barrel vault over the nave but also the beams and joists which support the domes over the side aisles. Four similar pillars are present at the crossing of the nave and the transept, and these support the main dome present at the same location.

These pillars basically consist of eight 0.20×0.20 m and 0.20×0.10 m vertical wood posts connected either to an arcade plate or a beam with the help of dowels on the top; and nailed to a sill plate embedded in the fired brick base course. These posts are additionally reinforced by 0.07×0.07 m horizontal and diagonal wooden members also connected by dowels. A huarango tree trunk having diameter approximately 0.33m is located in the centre of each of these pillars, supporting but not connected to the arcade plate on its top and connected to embedded sill plate below. These pillars are finished with flattened cane reeds or cana chancada, typical of quincha constructions which are nailed to wooden battens, which in turn are nailed to the posts. The cana chancada are finally coated with a layer of plaster.
A number of similar but smaller pilasters exist along lateral walls adjacent to the mud brick piers reinforcing the lateral walls. These pilasters are constructed of four vertical wooden posts and they do not have the tree trunk in their centre. The pillars in the nave are connected to each other as well as to the pilasters near the lateral walls by wooden arches spanning in both the east-west direction as well as the north-south direction. Both the pillars and pilasters are supported on bases made of fired brick. More details of the structure of the *quincha* pillar are seen in Figure 7.

**Figure 7: Typical constructive details of the *quincha* pillars (Drawing: José Garcia Bryce and Mirna Soto for the GCI)**

**Roof Structure**

The roof structure of the Ica Cathedral is quite complicated and consists entirely of *quincha*. The main components of the roof structure are as classified below: Barrel vaults over the central nave, transept and altar; A large dome at the crossing of the nave and transept; Smaller domes over the side aisles; Five perpendicular lunette vaults intersecting the main
barrel vault on each side of the nave corresponding to the location of windows in the upper nave wall. Construction details of the timber roof are elucidated in Figure 8 and Figure 9.

Figure 8: Roof structure at the central nave and side aisles (Ciocci, 2015)

Figure 9: Roof structure at the crossing and transept (Ciocci, 2015)

2.2.3 Damage

The following sections of the thesis deal with the damage and other structural irregularities that exist currently in the structure. Like the previous section, more emphasis is laid on the damage seen in the part of the Ica Cathedral that is, mostly, numerically modelled in the scope of this thesis, i.e. the masonry envelope and the bell towers.
The damage and irregularities that exist currently are again organised as: Front Facade and Bell Towers; Lateral Walls; Altar and Roof Structure and other important damages.

**Front Facade and Bell Towers**

The front facade is characterised by horizontal cracks which exist at the connection between the pediment and the lower facade. Diagonal cracks suggesting pounding of the bell towers during seismic events also exist between the upper cornice and the base of the bell towers. In addition to this damage, the timber framed structural part of the bell towers is deteriorated and this poses a risk to the roofing system of the cathedral to the west. The deterioration seen includes failure of the connections between the timber members, change in position of a few timber elements from their original position (especially in the southern tower) and also termite damage.

![Figure 10: Damage map of the front facade and bell towers (Drawing: Claudia Cancino for the GCI)](image-url)
**Lateral Walls**

The damages and irregularities that exist in the northern and lateral walls are summarised here:

- Horizontal and vertical cracks in the northern lateral wall along the perimeter of piers and arches adjacent to the choir loft.
- An irregularity in construction seen is the fact that the bricks at base of the *quincha* piers do not interlock with the bricks of the base course of the lateral walls. This might be a possible cause behind the vertical cracking seen at the connection between the piers and the lateral walls (especially the southern lateral wall).
- The southern lateral wall also suffers from humidity problems. This is most probably the result of improper drainage in the cloister of the Jesuit College which exists right next to this wall. Moisture problems have led to considerable degradation of the mortar and consequently of the masonry in the stone foundations of the southern wall, especially in the lower east end.
- Horizontal cracks also exist at the upper and lower levels of the northern lateral wall in all the altarpiece spaces.
- Both the southern and northern walls show horizontal cracking in their lower parts in the vicinity of the transept.

![Figure 11: Horizontal and vertical cracking seen in the northern lateral wall](image-url)
Altar

The altar, sacristy and chapels of the Ica Cathedral are located at its western end. Though no in-depth damage survey was carried out in the sacristy, the damages located in the altar and the chapels are well documented. These include:

- Cracking, disconnection and out of plane movement of the upper end of the western wall of the altar and the southern chapel.
- Vertical cracking in the corners of the walls of both chapels.
- The upper parts of the northern and southern walls of the altar exhibit horizontal cracking. Similar cracking is exhibited by the southern and western walls of the northern chapel.

Figure 12: Damage map of the northern lateral wall (Drawing and photograph: PUCP for the GCI)

Figure 13: Location of altar and chapels in the Ica Cathedral
Roof Structure and other important damages

The main damages seen in the Ica Cathedral are mostly in the roof structure, namely: Partial collapse of the central dome; Collapse of the barrel vaults near the facade above the choir loft; Total collapse of the roof over one of the bays near the southern aisle. In addition to these main damages extensive cracking, displacements, disconnection among structural members and other damages are seen in the interior, as well as the roof structure of the Ica Cathedral.

Figure 14: Damages in the roof structure of Ica Cathedral
Chapter 3

Finite Element Model and Material Data
This thesis is focussed on structural analysis of the masonry envelope of the Ica Cathedral using a finite element representation with a macro-model approach (Figure 15). The model is based on geometric and inspection surveys performed by the Getty Conservation Institute as part of the Seismic Retrofitting Project. Additional information was obtained by a site visit to the structure conducted during May-June 2015, by a team from University of Minho. Emphasis has been laid on reproducing the existing structural damage, the safety assessment of the structure in its current condition, the evaluation of the dominant mode shapes of the structure as predicted by the model, and also the comparison with the dynamic identification performed during the on site visit. This has been done by performing nonlinear static, eigenvalue, pushover and time history analyses, which will be addressed in subsequent Chapters.

Figure 15: Representation of the FE Model: (a) 3D view; (b) plan view

In addition to assessing the current condition of the church, a finite element model was also constructed based on the strengthening of the cathedral proposed by PUCP Peru on behalf of the Getty Conservation Institute. A detailed description of both finite element models can be found in this chapter.

### 3.1 Adopted Geometry

A 3D finite element model of the Ica Cathedral in its current state was constructed in Midas FX+ Version 3.3.0 Customized Pre/Post processor for DIANA software. This model was generated from the geometric survey carried out by the Pontificia Universidad Católica del Perú for the Seismic Retrofitting Project of The Getty Conservation Institute. The model as seen in Figure 15 includes the fired brick masonry facade, the bell towers, and the northern and southern lateral walls. The model however stops at the altar and the adjoining lateral
chapels. The regions in the cathedral housing the sacristy, reception and offices are not included in the model as from the visit conducted to the structure during May 2015, they were observed to be new structures with light weight roofing systems, not acting as a part of the masonry envelope, as seen in Figure 16. Therefore, these elements are likely to exhibit an independent behaviour from the parts of interest to this thesis.

![Figure 16: Light weight roofing system seen in the sacristy, reception and other new constructions present in the Ica Cathedral not included in the model.](image)

As per boundary conditions, all the nodes at the base of the structure were considered to be pinned, i.e. all the nodes have their translational degree of freedoms in three directions fixed to zero. All intersecting walls are assumed to have full connectivity. Full connectivity is also assumed between the fired brick base course and the adobe masonry parts of the wall and between the fired brick base course and the rubble stone base course foundation. It is also important to note that all the timber elements in the bell tower are also assumed to be rigidly connected, meaning that the stiffness and strength of the connections among these timber members are not considered in this model.

All the openings in the adobe masonry were assigned timber elements. Though no detailed geometrical information is available about their presence, the conducted damage survey shows that no significant damage is present near the openings. Thus the openings were provided with 30cm thick linear elastic timber lintels each having a bearing length of 50cm into the walls. This is illustrated in Figure 17.
Figure 17: Main entrance to the cathedral in the numerical model showing arched entrance (only for a small part of the cross section) and timber lintel (for the remaining part of the cross section).

Though the finite element model constructed does not include it, the constraining effect of the cloister which is adjacent to the southern lateral wall was taken into account. The presence of this cloister and consequently its stiffening effect on the southern lateral wall was simulated by the presence of 1-noded spring-dashpot elements. These spring elements were provided along the entire length of the southern lateral wall. The stiffness of each of these springs was calculated by calculating the stiffness of each of the columns present in the cloister. The stiffness hence calculated was multiplied with the number of columns present in the cloister to obtain the entire stiffness provided by the cloister. This was increased further to account for the presence of other vertical elements and lateral walls. The number finally obtained was divided by the number of nodes present on the mesh on the lateral wall corresponding to the height and thickness of the vault of the cloister present.

Figure 18: Cloister present adjacent to the southern lateral wall and 1 node springs simulating it in the model.
The created FE mesh for the model of the Ica Cathedral in its current state is composed of 353,866 isoparametric linear elements TE12L, 345 one-noded translation spring dashpot elements SP12R and 81,236 nodes in total (TNO DIANA, 2014).

The FE mesh for the model of the Ica Cathedral considering the strengthening suggested for the masonry envelope is composed of 337,488 isoparametric linear elements TE12L, 338 one-node translation spring dashpot elements SP12R and 83,993 nodes in total (TNO DIANA, 2014).

### 3.2 Proposed Strengthening

As stated above, a FE model was also constructed considering the strengthening proposed by PUCP Peru on behalf of the Getty Conservation Institute. This section deals with the strengthening proposed only for the substructure of the Ica Cathedral being modelled in this thesis.

The proposed strengthening of the masonry envelope of the Ica Cathedral involves basically two procedures: replacing sections of the adobe masonry in the lateral walls with new brick masonry and reinforcing a part of both the walls with the help of geogrid nets. More information on how these strengthening procedures are assumed to change the material properties of the assigned sections of the structure in the numerical model are mentioned in.

It can be seen from Figure 19 that the proposed strengthening scheme involves only replacing and reinforcing portions of the adobe masonry and not the fired brick base course. The red portions in Figure 19 indicate regions to be replaced with new fired brick masonry while the shaded green areas in the crossing of the transept and main nave refers to regions of the walls which are proposed to be reinforced with geogrid.
3.3 Masonry Constitutive Behaviour

Masonry is a quasi-brittle heterogeneous material. Different typologies of masonry result from differences in geometry, consistency and construction procedures. All the different typologies of masonry however share the common feature of having a very low tensile strength. The structural behaviour as well as the failure of masonry is mostly governed by the bond between unit and mortar (Lourenço, 1996).

3.3.1 Fracture Process

Numerical models aim to accurately simulate this behaviour of masonry: the transition from elastic behaviour to cracking leading to failure. This behaviour of masonry is often referred to as the softening of masonry. Softening can be defined as the decrease in mechanical resistance of masonry under constant the increase of deformation and gradual progress of...
cracking. Concentrations of tensile and compressive stresses are released through a process evolving from a diffused micro crack pattern to localized macro cracks (Figure 20) (Lourenço et al., 1998).

![Figure 20: Stress-displacement diagrams of quasi-brittle materials under tensile (left) and compressive (right) loading (Lourenço et al., 1998).](image)

The cracking phenomenon is quantified by integral under the stress displacement diagram, denoted as fracture energy $G_f$ for tension and $G_c$ for compression. Additionally, failure can also take place at the unit mortar interface and this depends on the shear resistance of the unit-mortar interface. This mode of failure is quantified by the mode II fracture energy denoted as $G_{fII}$ and is calculated as the area under the shear stress and displacement diagram under the absence of confined loading (Lourenço, 1996).

This physical nonlinear compressive and tensile behaviour of masonry is described in the numerical model of the Ica Cathedral through the Total Strain Rotating crack material model which is available in DIANA. More details about the Total Strain Rotating crack model are presented in the following section.

### 3.3.2 Total Strain Rotating Crack Model

In total strain based models, a stress-total relation is defined for the continuum. This relation can be either defined in fixed or rotating axes and the material model used in the numerical model of the Ica Cathedral it is defined in the rotating axes as suggested by the name. The total strain based rotating model uses an implicit shear term which provides co-axiality of the rotating principal stress and strain. The constitutive relationship in such a model is
evaluated in a rotated local coordinate system defined at a location where the crack is assumed to have initiated.

Globally the strain is updated as:

\[ \varepsilon_{xy} = \varepsilon_{xy} + \Delta \varepsilon_{xy} \]  

(1)

The local strain vector is determined as:

\[ \varepsilon_{ns} = T(\tau \phi)\varepsilon_{xy} \]  

(2)

Here \( \phi \) is the angle between the global coordinate system and the local coordinate system, \( ns \). The local coordinate system is fixed for a certain instance of time \( \tau \). This is calculated in correspondence to the time of occurrence of first occurrence of violation of failure condition in tension and is constantly updated.

If the angle between the global and local coordinate system is known, the strain transformation matrix is calculated as:

\[
T = \begin{bmatrix}
\cos^2\phi & \sin^2\phi & \sin\phi\cos\phi \\
\sin^2\phi & \cos^2\phi & -\sin\phi\cos\phi \\
-2\sin\phi\cos\phi & 2\sin\phi\cos\phi & -\cos^2\phi\sin^2\phi
\end{bmatrix}\]

(3)

The stress vector in the local coordinate system can be computed as:

\[ \sigma_{ns} = D(\varepsilon_{ns})\varepsilon_{ns} \]  

(4)

Assuming co-rotationality between the local strain and local stress vector, the stress vector in the global coordinate system can be updated as:

\[ \sigma_{xy} = T(\tau \phi)^T\sigma_{ns} \]  

(5)

Thus it is seen that the constitutive law has to be input only locally. Thus this model is very attractive in terms of the fact that material model parameters can be defined very easily from stress strain relationships. There is no need to input complicated functions describing
yielding or laws describing cracking. Non-orthogonal multi directional cracking cannot be included in these models and only orthogonal cracking can be formulated. It is to be however noted that there is a difficulty in choosing parameters to formulate non-orthogonal cracking. Moreover a large number of engineering problems involve just orthogonal cracking (Das, 2008).

For the numerical model of the Ica Cathedral, the Total Strain Rotating Crack model in DIANA has been used to characterize adobe masonry, fired brick masonry, rubble stone masonry, new fired brick masonry and geogrid reinforced adobe masonry. The values that need to be input to describe this material include basic properties like specific weight, Poisson's ratio, modulus of elasticity and also separately the behaviour of the model in tension and compression. In this model tensile softening is defined by an exponential curve which is based on fracture energy which is a predefined function in DIANA. Similarly the compressive behaviour is defined by a predefined parabolic function dependent again on fracture energy. This material model is graphically represented in Figure 21.

![Stress-Strain curve showing material model of masonry implemented in the numerical model.](image)

**Figure 21**: Stress-Strain curve showing material model of masonry implemented in the numerical model.

### 3.4 Review of Material Properties

The main materials present in the part of the structural system of the Ica Cathedral modelled in this thesis are: adobe masonry, walls of the structure; fired brick masonry, facade of the cathedral, base of the bell towers and also base course of the walls of the structure; rubble
stone masonry, base course foundation of the walls; timber, the upper part of the bell tower structure and timber lintels above all openings.

The material properties used for the numerical model presented in this report were derived primarily from the extensive experimental campaign carried out by the Pontificia Universidad Católica del Perú for the Seismic Retrofitting Project of The Getty Conservation Institute. Various national technical building standards (NTC-2008, ASTM, Eurocode 6, FEMA 306) and other bibliographic resources have also been used. The following sections explain in detail the methodology and reasoning behind the material properties adopted in the numerical model.

### 3.4.1 Adobe Masonry

Mechanical properties exhibited by adobe masonry often vary across a very wide range of values greatly affected by the quality of soil used and the workmanship associated. Thus a large number of samples needs to be tested from a site for a statistically accurate characterisation of the properties of such masonry. This is however complicated by the high level of fragility and low number of samples that can be extracted from historical sites (Angelillo M., 2014). The factors primarily influencing the compressive strength of such masonry are the quality of adobe units and thickness of mortar joints (Paulay & Priestley, 1992).

Compression tests were performed on adobe wallets constructed with adobe units from the Ica Cathedral as a part of the experimental campaign (GCI & PUCP, 2014). The results of these tests are seen in Table 2, where E and MOE both indicate the modulus of elasticity.

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Area(mm²)</th>
<th>Compressive Strength</th>
<th>MOE</th>
<th>E*(MPa)</th>
<th>Avg. E (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a(mm)</td>
<td>b(mm)</td>
<td>h(mm)</td>
<td>Load(KN)</td>
<td>Stress(MPa)</td>
<td>Avg. Stress(MPa)</td>
</tr>
<tr>
<td>200</td>
<td>150</td>
<td>435</td>
<td>14</td>
<td>0.468</td>
<td>0.46</td>
</tr>
<tr>
<td>200</td>
<td>150</td>
<td>430</td>
<td>14</td>
<td>0.479</td>
<td>0.47</td>
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<tr>
<td>250</td>
<td>150</td>
<td>435</td>
<td>13</td>
<td>0.443</td>
<td>0.44</td>
</tr>
</tbody>
</table>

*The modulus of elasticity E reported in the table are calculated from the slope of the stress strain curve taking into account values up to one third of the ultimate stress achieved in the tests.
The average stress obtained from the three tests can be adopted as the compressive strength for adobe to be used in the numerical model. Thus a compressive strength of 0.46 MPa is adopted for adobe masonry.

According to existing bibliography, the compressive strength of masonry can be related to its modulus of elasticity by the relationship:

\[ E_m = \alpha f_c \]  \hspace{1cm} (6)

According to Tomazevic (1999) the value of \( \alpha \) ranges from 200-1000. On the other hand FEMA 306 (1998) suggests a value of 550 while Eurocode 6 suggests a value of 1000. On choosing a value closer to the lower limit of the range of the suggested values, which is presumably more suitable for traditional masonry a modulus of elasticity of 93 MPa is obtained. This is very close to the values obtained experimentally and hence a Young's Modulus of 93 MPa is adopted for the adobe masonry.

Shear compression tests were also performed on three adobe triplets constructed with adobe blocks extracted from the Ica Cathedral and mortar to which straw had been added. These tests gave an average value of 44.5KPa for the cohesion and a value of 28.65° for the friction angle. The Mohr-Coloumb yield condition can be used to calculate the tensile strength of adobe masonry using the relationship:

\[ f_t = \frac{2c_o \cos \phi}{1 + \sin \phi} \text{ (MPa)} \]  \hspace{1cm} (7)

The tensile strength is calculated as 0.05 MPa. The calculated tensile strength has a ratio of 0.10 with the compressive strength which is in the range of values in accordance with existing literature. Thus the tensile strength of adobe masonry to be used in the numerical model is adopted as this value.
3.4.2 Fired Brick Masonry

No tests were performed on fired brick masonry specimens extracted from Ica Cathedral though tests were performed on adobe wallets extracted completely from Hotel El Comercio as well as piles constructed with units from Hotel El Comercio and a new lime/sand (1/2) mortar. The piles extracted from Hotel El Comercio showed a much lower strength than the piles reconstructed using the new mortar. This is due to the fact that even though the brick units were in good condition the masonry was very weak due to the fragile nature of the sand lime mortar in the piles extracted completely from Hotel El Comercio.

Considering the fact that most of the fired brick constructions in Ica Cathedral were reconstructed in the nineteenth century and are of the same age as in Hotel El Comercio, it is assumed that the sand lime mortar in fired brick masonry in Ica Cathedral has undergone the same deterioration. Thus it seems very reasonable to assume properties from these tests whose results are tabulated in Table 3.

Table 3: Results of compression tests performed on brick masonry from Hotel El Comercio (GCI & PUCP, 2014).

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Area(mm²)</th>
<th>Compressive Strength</th>
<th>MOE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a(mm)</td>
<td>b(mm)</td>
<td>h(mm)</td>
</tr>
<tr>
<td>130</td>
<td>150</td>
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<td>160</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>160</td>
<td>300</td>
<td>300</td>
<td>140</td>
</tr>
<tr>
<td>140</td>
<td>300</td>
<td>300</td>
<td>140</td>
</tr>
</tbody>
</table>

*The modulus of elasticity reported in the table are calculated from the slope of the stress strain curve taking into account values up to one third of the ultimate stress achieved in the tests.

The average stress obtained from the three tests can be adopted as the compressive strength for fired brick masonry to be used in the numerical model. Thus a compressive strength of 1.70 MPa is adopted for fired brick masonry.

According to existing bibliography, the compressive strength of masonry can be related to its modulus of elasticity by (6). Once again on choosing a value closer to the lower limit of the range of the suggested values, which is presumably more suitable for traditional masonry, the value of 340 MPa is obtained as the modulus of elasticity. The results of elastic modulus

Erasmus Mundus Programme
obtained from the experimental campaign show a very high variation and hence they are not considered and the former value is adopted in the numerical model.

The failure of masonry occurs at the unit mortar interface and hence is governed by the strength of this bond. Factors affecting this bond depend on- the units: material, size, perforation, size, air dried or pre-wetted etc; the mortar: composition, water content etc; workmanship adopted: filling of joints, vertical loading etc. The dependence on so many factors makes a recommendation of this value just on the basis of unit type or mortar type or a combination of both very difficult though Eurocode-6 does make an indication of these values. The value is typically very low ranging between 0.1-0.2 MPa (Pluijm, 1999)(Rots, 1997). A value towards the lower end of this range of values was adopted for fired brick masonry and a tensile strength of 0.1 MPa was assumed.

It is to be noted that shear compression tests were also performed on nine fired brick triplets constructed with fired bricks extracted from the Hotel El Comercio and new sand/lime mortar. These tests gave an average value of 111KPa for the cohesion and a value of 57.3° for the friction angle. From the Mohr-Coloumb yield condition(7) the tensile strength can be calculated as 0.065. It is unclear if these tests can be used to characterize the actual nature of fired brick masonry present in the Ica Cathedral because of the use of new mortar in tested specimens.

3.4.3 Rubble Stone Masonry

No testing was performed as a part of the experimental campaign on the rubble stone base course foundation present in the Ica Cathedral. Hence the values used in the numerical model are taken from existing literature. The minimum values are taken from the range prescribed by Italian Technical Building Norm (Table 11.D.1 of OPCM 3431. 2005) for rubble stone masonry. A value of 0.6 MPa is chosen for the compressive strength and the Young’s modulus is calculated as 300 MPa using(6). A ratio of 0.10 is assumed between the tensile strength and...
compressive strength and the tensile strength is calculated as 0.06 MPa. The material properties input into the numerical model for different types of masonry are tabulated in Table 4.

### Table 4: Material properties of different types of masonry input into the numerical model.

<table>
<thead>
<tr>
<th>Type of Masonry</th>
<th>Mechanical Properties</th>
<th>Adobe</th>
<th>Fired Brick</th>
<th>Rubble Stone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength (MPa)</td>
<td>0.46</td>
<td>1.70</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>93</td>
<td>340</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>0.05</td>
<td>0.1</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>Fracture Energy Mode I (Compression) (N/mm)</td>
<td>1</td>
<td>3.5</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Fracture Energy Mode I (Tension) (N/mm)</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td></td>
</tr>
<tr>
<td>Specific Weight (kN/m³)</td>
<td>19</td>
<td>19</td>
<td>19</td>
<td></td>
</tr>
</tbody>
</table>

### 3.4.4 Timber

Timber elements are also present in the structural system of the Ica Cathedral being modelled in two locations: timber framed structure of the bell tower structure and in timber lintels assumed to be present above the openings. From the experimental campaign and inspection of the structure carried out by PUCP, it is known that the timber framed structure of the bell towers is constructed of Huarango timber species. The material properties obtained characterising the same are tabulated in Table 5.

Additionally the timber lintels also assumed to be constructed of Huarango timber species, since it is the species with the highest Young’s modulus and these were inserted to prevent the excessive concentration of stresses near the openings since no damages were seen there.
Table 5: Properties of Huarango timber species (GCI & PUCP, 2014).

<table>
<thead>
<tr>
<th>Property</th>
<th>Huarango</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic Density (g/cm$^3$)</td>
<td>0.9</td>
</tr>
<tr>
<td>Elasticity module in bending MOE (x1000MPa)</td>
<td>16.8</td>
</tr>
<tr>
<td>Compression parallel to grain test (MPa)</td>
<td>92.2</td>
</tr>
<tr>
<td>Compression perpendicular to grain test (MPa)</td>
<td>22.5</td>
</tr>
<tr>
<td>Shear strength parallel to grain test (MPa)</td>
<td>20.8</td>
</tr>
<tr>
<td>Tension parallel to grain test (MPa)</td>
<td>61.9</td>
</tr>
<tr>
<td>Tension perpendicular to grain test (MPa)</td>
<td>1.9</td>
</tr>
</tbody>
</table>

The timber members of the bell tower are also known to be constructed of Huarango. Hence they were assigned the same material properties. However, these members were assigned a higher material density in the numerical model to account for the weight of wire mesh and cement plaster finish wrapping them as mentioned in 2.2.2.

### 3.5 Material Properties for Strengthening

The proposed strengthening by Pontificia Universidad Católica del Perú for the Seismic Retrofitting Project of The Getty Conservation Institute of the masonry envelope of the Ica Cathedral involves the use of two additional materials: geotextile for the strengthening of the adobe masonry and also the use of new brick masonry for strengthening the lateral walls of the cathedral. The proposed strengthening of the cathedral and the location of these materials in the numerical model has already been discussed in detail in 3.2.

This section of the thesis deals with how the material properties to simulate the behaviour of the proposed strengthening were determined. As it will be explained in more detail, the properties were calculated considering primarily the improvement in mechanical response (increase in strength) of the various material properties input into the numerical model to simulate the cathedral in its current condition.

Emphasis was laid on determining the homogenised properties of adobe masonry reinforced with geogrid rather than a detailed characterisation of the geogrid itself. A similar approach was adopted for masonry constructed of new fired bricks. It is to be noted that as previously
discussed in 3.2, strengthening of the timber framed structure of the bell towers with additional timber elements was also proposed but this was not considered in the strengthened model of the Ica Cathedral.

### 3.5.1 Geogrid-reinforced Adobe Masonry

As per existing literature, geogrid reinforcement leads to an increase in both compressive strength, tensile strength as well as the ductility of adobe masonry. In fact the increase is compressive strength and tensile strength maybe up to 50 % compared to adobe masonry without any reinforcement (Torrealva et al., 2011). Thus both the compressive strength as well as the tensile strength of geogrid-reinforced adobe masonry is assumed to increase by 50 % to values of 0.70 MPa and 0.07 MPa respectively (from 0.46 MPa and 0.05 MPa respectively). The value of tensile strength of such masonry again maintains a ratio of 0.10 with the compressive strength in accordance with existing literature.

Using (6) the modulus of elasticity of geogrid-reinforced adobe is calculated using the value of α obtained as a ratio between the modulus of elasticity and compressive strength of adobe masonry without the reinforcement as 140 MPa.

The reinforcement of adobe masonry with geogrid also increases its ductility in tension. The same is accounted for by increasing the ductility index (i.e. the ratio between strength and fracture energy) which is used for calculating Fracture Energy Mode 1 in tension by 50 % as compared to the ductility index for unreinforced masonry. The ductility index in compression and consequently Fracture Energy Mode 1 in compression is calculated using the same ductility index as in compression as for unreinforced adobe masonry. The mechanical properties of reinforced and unreinforced masonry input into the numerical model input are compared in Table 6.
Table 6: Comparison of mechanical properties between Geogrid-reinforced adobe and adobe masonry.

<table>
<thead>
<tr>
<th>Mechanical Properties</th>
<th>Geogrid-reinforced adobe</th>
<th>Adobe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength (MPa)</td>
<td>0.70</td>
<td>0.46</td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>140</td>
<td>93</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>0.07</td>
<td>0.05</td>
</tr>
<tr>
<td>Fracture Energy Mode I (Compression) (N/mm)</td>
<td>1.5</td>
<td>1</td>
</tr>
<tr>
<td>Fracture Energy Mode I (Tension) (N/mm)</td>
<td>0.05</td>
<td>0.01</td>
</tr>
<tr>
<td>Specific Weight (kN/m$^3$)</td>
<td>20</td>
<td>19</td>
</tr>
</tbody>
</table>

3.5.2 New Brick Masonry

The properties for the mechanical characterisation of new brick masonry to strengthen the Ica Cathedral are done by assuming that brick units similar to the existing bricks from the cathedral will be used with a new sand lime mortar. Hence the mechanical properties used to characterise this material are extracted from the tests carried out on brick wallets constructed using fired brick units from the Hotel El Comercio and a new sand lime mortar as a part of the experimental campaign of the Seismic Retrofitting Project for the Getty Conservation Institute. It is to be noted once again that the fired brick constructions and consequently the brick units from Hotel El Comercio and Ica Cathedral are approximately of the same age. Moreover the properties for fired brick constructions used in the model of the Ica Cathedral in the current state are also extracted from the tests performed on wallets extracted from Hotel El Comercio with the original mortar. The results obtained from the compression tests performed on such wallets are presented in Table 7.

Table 7: Results of compression tests performed on brick masonry with new mortar from Hotel El Comercio (GCI & PUCP, 2014).

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Area(mm$^2$)</th>
<th>Compressive Strength</th>
<th>MOE</th>
</tr>
</thead>
<tbody>
<tr>
<td>a(mm)</td>
<td>b(mm)</td>
<td>h(mm)</td>
<td>Load(KN)</td>
</tr>
<tr>
<td>255</td>
<td>128</td>
<td>420</td>
<td>32640</td>
</tr>
<tr>
<td>260</td>
<td>130</td>
<td>420</td>
<td>33800</td>
</tr>
<tr>
<td>260</td>
<td>130</td>
<td>420</td>
<td>33800</td>
</tr>
</tbody>
</table>

*The modulus of elasticity reported in the table are calculated from the slope of the stress strain curve taking into account values up to one third of the ultimate stress achieved in the tests.
The compressive strength of such masonry is hence assumed to have a value of 6 MPa. To account for an increase in the tensile strength of new brick masonry over old brick masonry existing in the structure the value assumed from this parameter is from the higher end of the range suggested by existing literature i.e. values recommended for the strength of the unit mortar interface. Thus the tensile strength of such masonry is assumed as 0.25 MPa. The modulus of elasticity is calculated using (6) using the same value of $\alpha$ used for existing fired brick masonry as 1200 MPa. The fracture energy mode 1 in compression is calculated assuming the same ductility index as existing brick constructions. The mechanical properties between new and existing brick masonry input into the numerical model are compared in Table 8.

<table>
<thead>
<tr>
<th>Mechanical Properties</th>
<th>Type of Masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength (MPa)</td>
<td>New brick</td>
</tr>
<tr>
<td></td>
<td>Existing brick</td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>6.00</td>
</tr>
<tr>
<td></td>
<td>1.70</td>
</tr>
<tr>
<td>Modulus of Elasticity (MPa)</td>
<td>1200</td>
</tr>
<tr>
<td></td>
<td>340</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
</tr>
<tr>
<td>Fracture Energy Mode I (Compression) (N/mm)</td>
<td>12.4</td>
</tr>
<tr>
<td></td>
<td>3.5</td>
</tr>
<tr>
<td>Fracture Energy Mode I (Tension) (N/mm)</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>0.01</td>
</tr>
<tr>
<td>Specific Weight (kN/m$^3$)</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>19</td>
</tr>
</tbody>
</table>
Chapter 4

Dynamic Identification and Model Updating
The chapter presents results from the dynamic identification of the Ica Cathedral which was carried out during the visit by a team from the University of Minho on behalf of the Getty Conservation Institute in May 2015. The latter part of the chapter explains how these results were used in updating the material properties input into the numerical model of the masonry envelope. More details about the experimental testing done during the visit by the team from the University of Minho can be found in Greco et al., 2015.

### 4.1 Dynamic Identification

Many experimental procedures are available to characterize the behaviour of a full scale system. Among the methods available, dynamic identification tests represent one of the most complete and efficient procedures. These tests are used to evaluate a set of parameters which represent the dynamic behaviour of the system. These parameters commonly are natural frequencies, mode shapes and modal damping values (Belmonte, 2010).

Historical structures often have high mechanical and physical diversity. This makes assigning accurately specific properties like stiffness, strength and boundary conditions very difficult (Ramos, 2007). The presence of mostly inhomogeneous quasi brittle materials like masonry, the restrictions on testing implementation in structures of cultural value, the accumulation of damage and alterations over time, and the large diversity of connections and confinements make the uncertainty related with results obtained from conventional forms of testing very high. Ambient vibration tests are highly efficient, non-destructive and have low performance cost. Results obtained from identification are related to the dynamic behaviour and are very useful tools for validating, updating and calibrating numerical models, such that numerical dynamic properties from the model match the experimental ones obtained from identification tests. In addition, these tests can also be used to quantify and localize damage, deterioration of mechanical properties and other factors that affect the behaviour of the structure (Ramos, 2007).
4.1.1 Dynamic Identification Techniques

Dynamic Identification tests can be broadly classified as: input-output vibration tests, where the exciting forces are induced externally; free vibration tests, where the structure is subjected to an initial deformation which is then quickly released to excite the structure; and output-only identification technique where only the response of the structure is measured. Output-only techniques, or ambient vibration techniques as they are more popularly known, have a number of features which make them attractive over other methods for the study of historical structures (Garaygordobil, 2005). The main advantages of these techniques over the others in assessing historical constructions include: avoidance of artificially exciting such structures faced in all other techniques and also their entirely non-destructive nature. This makes them widely applied in the context of historical masonry structures to estimate dynamic parameters. In the ambient vibration technique the dynamic response of the structure is measured during service conditions assuming that the excitations, i.e. the ambient vibration, are of random nature in time and in space. A schematic of how this technique is used in dynamic identification can be seen in Figure 22.

![Figure 22: Schematic of the output-only technique of dynamic identification](Ramos, 2007)

The main assumption in this technique is that the ambient excitation \( u_k \) is considered as a stationary Gaussian white noise stochastic process in the frequency range of interest (Ramos, 2007). It is important to define white noise as a random signal with constant power spectral density. Thus a discrete signal sample which has a normal distribution with zero mean and finite variance can be defined as Gaussian white noise. In this way the structural response can be obtained with no damage on the structure. The response obtained includes the modal contribution of the ambient forces, the contribution of the structural system and the contribution of noise signals from undesired sources. Thus signal processing techniques

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adopted should clearly distinguish between noise and the structural response. As seen in the schematic of the output only technique in Figure 22, $u_k$ represents the excitation vector (ambient vibrations here); $w_k$ is noise from input perturbations and modelling inaccuracy; $v_k$ is noise in measurement by transducers and in data acquisition systems; $x_k$ is the discrete-time state vector with the sampling measured parameters and $y_k$ is the sampling response vector.

Signal processing or identification techniques used to process results from such tests can again be broadly classified into two groups: frequency domain methods and time domain methods, depending on the type of data being processed. Frequency domain methods which are user friendly and faster to process, are based on the signal analysis of each measured point in the frequency domain by applying Fast Fourier Transforms and on the correlation between the signals. These methods are also referred to as non-parametric methods. The time domain or parametric methods are based on model fitting by correlation functions or time history series of every measured point in the time domain. Although faster, frequency domain methods pose difficulties in identifying close frequency values. This arises due to the limitation on the frequency resolution from the Fast Fourier Transformation process. Thus despite being faster, the results from these methods may be inaccurate after the first few mode shapes, due to the difficulties in recognizing the peaks in the frequency values. On the contrary, the time domain methods and in particular the Stochastic Subspace Identification (SSI) method which is used in this thesis, is more complex than a frequency domain method and can be considered one of the most accurate procedures to estimate the modal parameters. It is a robust procedure which can estimate modal parameters with a high frequency resolution and thus poses no problems in recognizing close frequencies. It is based on the State-Space formulation that enables taking into account the white noise as an input of the system. A parametric model is fitted directly with the raw time series data and the model parameters can be adjusted to minimize deviation between predicted model response and the measured experimental one (Ramos, 2007).
4.2 Dynamic Identification of the Ica Cathedral

4.2.1 Test Parameters and Setups

The dynamic identification test results reported in this thesis were performed with piezoelectric accelerometers having a sensitivity of 10V/g and a frequency range of 0.15 to 1000 Hz (measurement range ±0.5g). These transducers have advantages over other transducers that can be used for the same purpose as: they do not use any external power source, are stable, have a good signal-to-noise ratio and are linear over a wide frequency and dynamic range (Ramos, 2007). These were connected to a data acquisition board of 24-bit resolution, which has the function to record the signals given by the accelerometers. The results were processed using the software package ARTeMIS software (Structural Vibration Solutions A/S, 2014).

Both time and frequency domain methods were used to process the data and hence obtain the dynamic properties of the structure. Specifically the Enhanced Frequency Decomposition Domain Method (EFDD), a frequency based method; and the Stochastic Subspace Identification Method (SSI), a time domain method were employed. The results obtained with SSI and EFDD methods were then compared using the Modal Assurance Criterion (MAC).

The tests on the Ica Cathedral were carried out through five setups with one reference accelerometer and three accelerometers positioned in the areas of interest. The reference accelerometer for all these tests, as seen in Figure 23 and Figure 24, was positioned on the upper part of the longitudinal wall adjacent to Jirón Libertad where a concentration of damage is seen. The location of some of the other accelerometers in the structure can be seen in Figure 23. The five different setups used for performing the dynamic identification tests are shown in Figure 24.
Figure 23: Measuring equipment: (a) accelerometer on the tower, (b) reference accelerometer on the top of the northern longitudinal wall, (c) accelerometer on the pillar of the timber structure (Greco et al., 2015).

Figure 24: Configuration of accelerometers in various test setups (Greco et al., 2015).

4.2.2 Results

The poles selection and the link of modes from all five setups, for both the EFDD and SSI-UPC methods, are presented in Figure 25 and respectively. The test results were used to estimate only the first three modes of vibration of the structure as these have the most significant contribution to the dynamic behaviour of the structure. Moreover calibration of a numerical
model on the basis of experimentally obtained modes as performed in this thesis is much more difficult if a very high number of modes are considered (Lourenco et al., 2012). Moreover, the first few modes provide sufficient information to characterize the dynamic behaviour of a structure (Chopra, 2012). The values of frequencies and damping ratios of the modes obtained from the SSI and EFDD Methods are presented in Figure 25 and Table 10 respectively.

![Figure 25: Data driven with the poles selection through the several test setups of the SSI-UPC method (Greco et al., 2015)](image)

![Figure 26: Data driven with the poles selection through the several test setups of the SSI-UPC method (Greco et al., 2015)](image)
Table 9: Frequencies and damping ratios of the first three modal shapes – SSI-UPC Method (Greco et al., 2015).

<table>
<thead>
<tr>
<th>Modes</th>
<th>Frequency (Hz)</th>
<th>Std. Deviation Frequency</th>
<th>Damping Ratio (%)</th>
<th>Std. Deviation Damping Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode 1</td>
<td>2.84</td>
<td>0.04</td>
<td>2.05</td>
<td>0.60</td>
</tr>
<tr>
<td>Mode 2</td>
<td>3.21</td>
<td>0.03</td>
<td>1.84</td>
<td>0.67</td>
</tr>
<tr>
<td>Mode 3</td>
<td>3.92</td>
<td>0.03</td>
<td>2.32</td>
<td>0.64</td>
</tr>
</tbody>
</table>

Table 10: Frequencies and damping ratios of the first three modal shapes – EFDD Method (Greco et al., 2015).

<table>
<thead>
<tr>
<th>Modes</th>
<th>Frequency (Hz)</th>
<th>Std. Deviation Frequency</th>
<th>Damping Ratio (%)</th>
<th>Std. Deviation Damping Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode 1</td>
<td>2.85</td>
<td>0.03</td>
<td>1.90</td>
<td>0.38</td>
</tr>
<tr>
<td>Mode 2</td>
<td>3.22</td>
<td>0.02</td>
<td>1.34</td>
<td>0.37</td>
</tr>
<tr>
<td>Mode 3</td>
<td>3.94</td>
<td>0.12</td>
<td>1.44</td>
<td>1.49</td>
</tr>
</tbody>
</table>

The frequencies of the modes obtained from the SSI Method lie between 2.84 Hz to 3.92 Hz. The standard deviation associated with these values is very low indicating a good estimation of these values. The value of damping ratio for these modes ranges between 1.84 and 2.32 % and a high standard deviation is observed for these values. In fact these values give an average value of 2.07% for the damping ratio. It is very common to obtain low values of damping ratio when output only techniques are used for dynamic identification tests (Ramos, 2007). Moreover it is to be noted that for masonry structures like the Ica Cathedral, the damping ratio is a very sensitive parameter, which is difficult to estimate experimentally (Mendes, 2012).

The Modal Assurance Criterion is a mathematical tool used to compare mode shapes obtained experimentally with those obtained from numerical models or analytically. In this case however a comparison is being made between mode shapes obtained experimentally from two dynamic identification techniques as seen in Figure 27. It is calculated as the normalized scalar product of the two set of vectors being compared as seen in Equation (12). The results of this equation are bounded between 0 and 1, where 1 indicates fully consistent mode shapes. The results of Equation (12) are graphically presented in Figure 27.
where the red value represents values closer to 1. This seems to indicate that only mode shape 1 and mode shape 2 have been clearly identified.

$$MAC_{(r,q)} = \frac{|\{\varphi_A\}_r^T\{\varphi_X\}_q|^2}{(\{\varphi_A\}_r^T\{\varphi_A\}_R)(\{\varphi_X\}_q^T\{\varphi_X\}_Q)}$$  (8)

![Figure 27: Comparison between the SSI-UPC and the EFDD results using the Modal Assurance Criterion (MAC)](image)

The mode shapes of the natural modes of vibration of the structure obtained by dynamic identification and calculated using the SSI-UPC method are presented in Figure 28. The first mode (2.84 Hz) corresponds to the first mode in the transversal direction of the longitudinal side walls of the Cathedral. Both the longitudinal walls of the nave experience a first order out-of-phase excitation with higher intensity in the northern wall on the transept area. The second mode (3.21 Hz) identifies the second order curvature of the northern wall, with a deflection point near the middle span, between the two measured points. The third mode (3.92 Hz) corresponds to a complex mode, which is mainly characterized by the fact that it is the first movement of the main façade.
Figure 28: Mode shapes obtained from the SSI-UPC Method, of the first three modes (Greco et al., 2015).
4.3 Model Updating and Eigenvalue Analysis

4.3.1 Model Updating

Once the dynamic properties of a structure are known experimentally, the mechanical properties of the numerical model: mass, stiffness and damping should be tuned such that the parameters obtained numerically (mode shapes and frequencies) resemble closely the experimentally observed ones. The process followed to this effect is known as model updating or more specifically finite element model updating (Lourenco et al., 2012).

In order to perform the model updating a modal response analysis or an eigenvalue analysis is performed in order to determine the natural frequencies and mode shapes of vibration which contribute to the dynamic response of the structure. The free vibration of a system with N degrees of freedom without any damping is governed by Equations (13) and (14). Under free vibration, the motion of each degree of freedom in a N-DOF system is not a simple harmonic motion. However these systems have characteristic deflected shapes which vibrate in simple harmonic motion and are known as the natural modes of vibration of such structures. Equations (9) and (10) can be reduced to the matrix eigenvalue problem as seen in Equation (15), which can be solved to obtain both the natural frequencies or eigenvalues and the mode shapes or eigenvectors can be estimated for the structure (Chopra, 2012).

\[ m \ddot{u}(t) + k u(t) = 0 \]  \hspace{1cm} (9)

\[ u(t) = \varphi_n(t) q_n(t) \]  \hspace{1cm} (10)

\[ k \varphi_n = \omega_n^2 m \varphi_n \] \hspace{1cm} (11)
In these equations: $m$ is the $N \times N$ mass matrix; $k$ is the $N \times N$ stiffness matrix; $u(t)$ and $u(t)$ are the acceleration and time vectors dependent on time; $\varphi_n$ is the deformed shape vector; and $q_n(t)$ represents the modal amplitude. It is also possible to calculate the amount of mass participating in each mode of natural vibration of the structure. Hence, the higher the percentage of mass participating in a mode, more is the contribution of the mode in characterizing the dynamic response of the structure.

An eigenvalue analysis was performed on the model with the material properties as mentioned in section 4.3.1, with the help of finite element software DIANA (TNO DIANA, 2014). A first quantitative comparison of the mode shapes obtained numerically and experimentally was performed and results of the comparison are presented in Figure 29. In the subsequent section, full modal analysis is presented.

It was found that the second mode shape obtained numerically corresponds to the first mode obtained experimentally, as this is the first mode of the northern longitudinal wall of the cathedral. Similarly the fourth numerical mode also involves the second order curvature of the northern wall and thus corresponds to the second experimental mode. The ninth numerical mode is the first mode that involves activation of the brick facade and can be assumed to correlate to the third mode obtained experimentally. It is to be noted here that the first mode obtained experimentally shows movement in both the longitudinal walls while the numerical mode does not. This can be due to the presence of the timber structure, boundary conditions assumed for the connection to the cloister or other reasons. Still, experimental mode shape 2 seems to indicate that the masonry and timber structure have mostly independent dynamic behaviour. Additionally, the first mode obtained from the numerical model shows movement only in the southern longitudinal wall adjacent to the cloister (Figure 30). It is possible that considering the close lying values of frequency for the first and second mode obtained from the numerical model, these were picked up as one single mode experimentally during the dynamic identification tests, or that the boundary conditions with respect to the cloister part could be changed in order to obtain a unique mode. It is noted that both stiffness and mass boundary conditions would be of relevance.
and, it is believed, that this exercise would not bring additional relevant information for the present thesis.

Figure 29: Comparison of mode shapes obtained experimentally and selected numerical mode shapes.
Figure 30: First numerical mode showing activation of the southern longitudinal wall.

From Figure 29 it can be observed that there is a good consistency between the deflected configurations of the experimental and numerical modes of vibration. In order to evaluate the quality of the model and to perform the process of finite element model updating, these modes are compared in terms of frequency in Table 11. The error reported in Table 11 is calculated with respect to the experimental values.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Freq. Numerical (Hz)</th>
<th>Freq. Experimental (Hz)</th>
<th>Error (%)</th>
<th>Avg. Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.65</td>
<td>2.84</td>
<td>42.01</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.93</td>
<td>3.21</td>
<td>39.98</td>
<td>39.29</td>
</tr>
<tr>
<td>3</td>
<td>2.51</td>
<td>3.92</td>
<td>35.88</td>
<td></td>
</tr>
</tbody>
</table>

It is seen that though these modes are alike in their shapes there is a significant error (about forty percent) for each mode when their frequencies are compared. Thus, as mentioned at the beginning of this section, there is the need to perform finite element model updating in order to reduce this error. Several advanced optimization processes are possible to minimize the error which exists between the mathematical and the experimental response, usually by adjusting the stiffness of the numerical model with selected criteria. However considering the constraint of time while developing this thesis, a very simple process in which the modulus of elasticity of the different kinds of masonry input into the numerical model are calibrated to minimize this error. An iterative process is followed and the error is reduced to a large extent, as it can be observed from Table 12 which compares the response of the calibrated numerical model with the experimental results. An average error of approximately 5% is obtained between the frequencies of the calibrated model and the
experimental frequencies opposed to the average error of approximately 40% which was obtained for the un-calibrated model. A comparison of the material properties for different types of masonry before and after the process of model updating is provided in Table 12.

In this respect, a simple approach was used by affecting all Young’s modulus by the same value, as the differences in frequencies for the three modes considered are similar. Finally, it is noted that the other mechanical properties of the materials will be kept the same, as the most important mechanical values are the tensile strength and the tensile fracture energy, as these are not correlated with the Young’s modulus.

### Table 12: Comparison of numerical (calibrated) and experimental modal frequencies

<table>
<thead>
<tr>
<th>Mode</th>
<th>Freq. Numerical (Hz)</th>
<th>Freq. Experimental (Hz)</th>
<th>Error (%)</th>
<th>Avg. Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.56</td>
<td>2.84</td>
<td>9.90</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3.01</td>
<td>3.21</td>
<td>6.49</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3.94</td>
<td>3.92</td>
<td>-0.63</td>
<td>5.67</td>
</tr>
</tbody>
</table>

### Table 13: Comparison of Young’s modulus before and after model updating

<table>
<thead>
<tr>
<th>Type of Masonry</th>
<th>Mechanical Properties</th>
<th>Modulus of Elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Adobe</td>
<td>Fired Brick</td>
</tr>
<tr>
<td>Un-calibrated</td>
<td>92.60</td>
<td>340</td>
</tr>
<tr>
<td>Calibrated</td>
<td>220</td>
<td>850</td>
</tr>
<tr>
<td>Rubble Stone</td>
<td>Un-calibrated</td>
<td>Calibrated</td>
</tr>
<tr>
<td>Un-calibrated</td>
<td>300</td>
<td>720</td>
</tr>
<tr>
<td>Calibrated</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 4.3.2 Eigenvalue Analysis

The eigenvalue analysis was once again performed for the model with calibrated material properties. The modal participation masses of the first thirty modes in each direction are presented in Table 14. It can be seen that the modes which were used for model updating process have a high participation in terms of mass and thus contribute significantly to the dynamic behaviour of the structure. The structure experiences a lot of local modes with low values of participating masses. It is to be noted that structure reaches a cumulative percentage of mass participation of 90% around the value of 60 Hz. The modes with high participation masses are shown in Figure 31. It can also be seen that after mode 9, the
facade starts to be involved in most modes, and no experimental readings exist for this part of the structure.

Table 14: Modal participation masses for the first thirty modes in each direction

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Modal Participation Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>X-X (%)</td>
</tr>
<tr>
<td>1</td>
<td>2.37</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>2.56</td>
<td>0.01</td>
</tr>
<tr>
<td>3</td>
<td>2.57</td>
<td>0.01</td>
</tr>
<tr>
<td>4</td>
<td>3.00</td>
<td>0.01</td>
</tr>
<tr>
<td>5</td>
<td>3.03</td>
<td>0.00</td>
</tr>
<tr>
<td>6</td>
<td>3.40</td>
<td>0.01</td>
</tr>
<tr>
<td>7</td>
<td>3.59</td>
<td>0.06</td>
</tr>
<tr>
<td>8</td>
<td>3.68</td>
<td>0.53</td>
</tr>
<tr>
<td>9</td>
<td>3.94</td>
<td>0.00</td>
</tr>
<tr>
<td>10</td>
<td>4.10</td>
<td>0.00</td>
</tr>
<tr>
<td>11</td>
<td>4.13</td>
<td>0.12</td>
</tr>
<tr>
<td>12</td>
<td>4.34</td>
<td>0.65</td>
</tr>
<tr>
<td>13</td>
<td>4.42</td>
<td>0.06</td>
</tr>
<tr>
<td>14</td>
<td>4.70</td>
<td>2.31</td>
</tr>
<tr>
<td>15</td>
<td>5.04</td>
<td>0.12</td>
</tr>
<tr>
<td>16</td>
<td>5.35</td>
<td>0.02</td>
</tr>
<tr>
<td>17</td>
<td>5.53</td>
<td>0.06</td>
</tr>
<tr>
<td>18</td>
<td>5.58</td>
<td>0.30</td>
</tr>
<tr>
<td>19</td>
<td>5.66</td>
<td>1.00</td>
</tr>
<tr>
<td>20</td>
<td>5.97</td>
<td>1.45</td>
</tr>
<tr>
<td>21</td>
<td>5.98</td>
<td>0.87</td>
</tr>
<tr>
<td>22</td>
<td>6.20</td>
<td>0.27</td>
</tr>
<tr>
<td>23</td>
<td>6.39</td>
<td>0.40</td>
</tr>
<tr>
<td>24</td>
<td>6.64</td>
<td>0.04</td>
</tr>
<tr>
<td>25</td>
<td>6.85</td>
<td>2.16</td>
</tr>
<tr>
<td>26</td>
<td>7.16</td>
<td>0.01</td>
</tr>
<tr>
<td>27</td>
<td>7.40</td>
<td>0.05</td>
</tr>
<tr>
<td>28</td>
<td>7.71</td>
<td>1.97</td>
</tr>
<tr>
<td>29</td>
<td>7.94</td>
<td>0.01</td>
</tr>
<tr>
<td>30</td>
<td>8.23</td>
<td>2.67</td>
</tr>
<tr>
<td>TOTAL CUM. PERCENTAGE (%)</td>
<td>28.46</td>
<td>48.79</td>
</tr>
</tbody>
</table>
Figure 31: Selected modes from modal analysis
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Chapter 5

Nonlinear Structural Analysis
The main focus of the thesis was to evaluate the structural behaviour of the masonry envelope of the Ica Cathedral in Peru with the help of the finite element model of the same. The previous chapters till now have described the assumptions adopted, the parameters input and how the model was calibrated, as its quality improved with the help of testing performed during the in-situ inspection of the structure. This section of the thesis reverts back to what the title of the thesis stands for, and presents the methods adopted and results of various nonlinear structural analyses performed on the finite element model. In particular, this section presents the results of: structural nonlinear analysis under dead load, mass proportional pushover analysis in the direction of primary axes of the model and nonlinear dynamic analyses with time integration. It is to be noted that in this section three different numerical models are adopted: the model with material properties reviewed in 3.4, referred to as the un-calibrated model; the strengthened model with material properties reviewed in 3.5; referred to as the strengthened model and the calibrated model with material properties updated corresponding to the dynamic identification test and reviewed in 4.3.1; referred to as the calibrated model. It should also be noted here that the properties of adobe, fired brick and rubble stone masonry used in the strengthened model correspond to the improvement in the mechanical properties of the un-calibrated model. This is because the results of dynamic identifications tests used to calibrate the material properties of the model were processed in a latter part of the period in which this thesis was performed.

All these analyses were performed using the finite element software DIANA (TNO DIANA, 2014). DIANA offers a lot of iterative solution methods for nonlinear problems: Newton-Rhapson, Modified Newton-Rhapson, Linear, Secant etc. which have been used in obtaining solutions to the nonlinear structural analysis performed. A more detailed description of these solution methods can be found in (TNO DIANA, 2014). Convergence criteria can be specified on the basis of force, displacement or energy norm in DIANA and the energy norm was used with convergence based on a tolerance of $10^{-3}$. This means that convergence is assumed when the ratio of the work of the internal forces and relative displacements of the previous iteration and the first prediction of that step is less than $10^{-3}$ (TNO DIANA, 2014).
Together with the iterative solution methods used for these analyses, arc length control and the line search algorithm are used while carrying out all these analyses. Masonry structures exhibit strong nonlinear behaviour. The line search algorithm stabilizes the convergence rate and behaviour in analysis of such structures by automatically scaling incremental displacements in the iterative process. The arc length method on the other hand prevents the solution from predicting a very large increment in displacements for a given increment in forces by constraining the norm of incremental displacements to a prescribed value and simultaneously adapting the size of the increment. This is needed to represent the softening behaviour of the structure in the load displacement relationship of the structure.

5.1 Self-Weight

As a starting point to understand the structural behaviour of the masonry envelope of the Ica Cathedral, a nonlinear analysis is performed for vertical loading on the calibrated model under its self weight. The dead weight of the structure is applied in ten steps.

The deformed shape of the structure and distribution of vertical displacements under its own self weight is seen in Figure 32. It is seen that the maximum vertical displacement occurs in the back wall of the sacristy of the cathedral. However the maximum value of displacement observed is approximately 1mm and can be neglected.

Figure 32: Distribution of vertical displacements under self weight on the deformed mesh and location of maximum vertical displacement.
The distribution of minimum principal stresses can be seen in Figure 33. It is seen that high values of compressive stresses occur near the base of the structure. The highest value of compressive stress occurring in the structure is in the brick base of the bell towers and is of magnitude 0.36 MPa which is much lower than the value of compressive strength adopted for brick masonry in the model.

![Figure 33: Distribution of minimum principal stresses under self weight on the deformed mesh and location of maximum compressive stress.](image1.png)

The distribution of maximum principal stresses can be seen in Figure 34. It can be seen that the highest values of tensile stresses occurring in the structure under its own dead weight are near the openings. The tensile stresses reach a maximum value of 0.03 MPa near the opening in the longitudinal wall adjacent to the street. This is lower than 0.05 MPa, the tensile strength of adobe masonry adopted.

![Figure 34: Distribution of maximum principal stresses under self weight on the deformed mesh and location of maximum tensile stress.](image2.png)
5.2 Seismic Analysis

Masonry buildings exhibit a very complex behaviour under seismic loading. Though the heterogeneous nature of masonry and complicated geometrical features of historical masonry structures make their response very difficult to characterize, the mechanical response of masonry in general is governed by some common features: high values of material density, quasi-brittle behaviour, and low tensile and shear strength. Among these, low tensile strength is a very important parameter that has determined the shape of ancient and modern unreinforced masonry construction (Lourenco, 2013).

Under horizontal loading, the nonlinear behaviour of masonry is triggered in very early stages of the loading. This makes linear elastic analysis unsuitable for evaluating the seismic response of masonry constructions such as the Ica Cathedral. In this thesis two methods have been used: the mass proportional pushover analysis and the nonlinear time history method. While pushover analysis is commonly recommended by building codes for seismic assessment of masonry constructions, time history analysis is a complex and time consuming method used mostly for the purposes of research (Lourenco, 2013). More details about these methods and the results obtained from them are explained in the following sections.

5.2.1 Pushover Analysis

The capacity of the structure under lateral loading is evaluated by performing mass proportional pushover analysis. These analyses helped to evaluate not only the capacity of the structure under lateral loads in terms of load displacement curves but also the progression of tensile and compressive damages, and also corresponding failure modes. Failure modes are identified in terms of reaching the capacity of the structure in the load-displacement diagram and high values of principal tensile strains in this thesis.

These analyses were performed in the XX and YY axis of the numerical model in both positive and negative directions. This section of the thesis initially presents the results of mass proportional pushover analysis performed on the calibrated and un-calibrated models.
validity of these models is evaluated in terms of crack correlation between the in situ observations and numerical models under mass proportional horizontal loading. This validated model is then used to simulate the preliminary strengthening proposed by PUCP on behalf of the Getty Conservation Institute and to evaluate the seismic performance of the strengthened masonry envelope.

**XX Direction**

The pushovers for the XX direction of the numerical model were carried out only for the un-calibrated and calibrated models, not for the strengthened model. As seen in the strengthening proposed by PUCP Peru for the Getty Conservation Institute explained in detail in 3.2, strengthening was proposed only for the longitudinal walls of the cathedral and is not going to affect the seismic load capacity of the cathedral in XX direction.

The capacity of the structure under horizontal loading in the XX+ direction of the model for the un-calibrated and calibrated models are summarized in Figure 35 in terms of load displacement diagrams. For the calibrated model the maximum lateral load applied is 0.30g while for the un-calibrated model it is 0.26g. The numerical model enters post peak behaviour after these loads are applied with high value of displacements recorded in the back wall of the sacristy. It is to be noted that this wall adjoins the reception and other new light weight roof construction present in the Ica Cathedral. The deformed shape of the numerical model is also presented in Figure 35. The load displacement diagrams are calculated for a node at the top of the wall showing high values of displacements.

The failure mechanism consists of vertical cracks developing along the height of the wall at the corners. As the tensile damages propagate along the thickness of the wall, this wall becomes independent from the rest of the structure (Figure 36). Large jumps in incremental displacements indicate the out of plane movement of this wall. The progress of flexural cracks seen at the rubble stone base course foundation confirms the out of plane failure of the wall.
Figure 35: Response of the structure in XX+ direction: a) load displacement diagram and b) deformed shape of the numerical model.

Figure 36: Crack pattern depicting the failure mechanism: a) global crack pattern; close up view of b) vertical corner separation cracks c) base course flexural cracks
This part of the structure was not covered in detail during the damage assessment survey of the Ica Cathedral mentioned in Cancino et al. (2012). Hence, an in depth correlation of damage observed in the numerical model and in the actual structure is not possible. However the survey did observe out of plane movement, disconnection and cracking of the upper end of the western wall of the altar as mentioned in 2.2.3(Cancino et al., 2012). Such damages are also observed in the numerical model in the pushover analysis in XX+ direction as seen in Figure 37. It is also possible that the seismic input was not sufficient to trigger this damage.

![Figure 37: Disconnection and cracking of the upper end of the western wall of the altar (also observed in situ)](image)

The capacity of the structure under horizontal loading in the XX- direction of the model for the un-calibrated and calibrated models is summarized in Figure 38 in terms of load displacement diagrams. For the calibrated model the maximum lateral load applied is 0.36g while for the un-calibrated model it is 0.33g. The numerical model enters post peak behaviour after these loads are applied with high value of displacements recorded in the front facade. The deformed shape of the numerical model is also presented in Figure 38. The load displacement diagrams are calculated for a node at the top of the facade.

The failure mechanism consists of out of plane failure of the entire front facade of the Ica Cathedral including the bell towers again indicated by large jumps in the incremental displacements of the facade. Tensile damage can be observed along the connection of the brick masonry bases of the bell towers with adobe longitudinal walls. Additionally flexural
cracks develop also along the base of the facade and the bell towers. In the post peak part of the load displacement diagram presented in Figure 38 this part of the structure separates and behaves independently from the rest of the structure.

Figure 38: Response of the structure in XX- direction: a) load displacement diagram and b) deformed shape of the numerical model.

Figure 39: Crack pattern depicting the failure mechanism: a) global crack pattern; close up view of b) vertical separation cracks and c) flexural cracks at the base.
Pushover analyses were carried out in the YY+ direction and YY- direction of the numerical model. These analyses were carried out for the calibrated, un-calibrated as well as the strengthened model, in a subsequent section. The results of the analyses and the associated load-displacement curves, failure mechanisms and damage correlation with damage observed in situ are presented initially for the calibrated and un-calibrated model to validate the numerical models.

The capacity of the structure under horizontal loading in the YY+ direction of the model for the un-calibrated and calibrated models is summarized in Figure 41 in terms of load displacement diagrams. For the calibrated model, the maximum lateral load applied is 0.25g while for the un-calibrated model is 0.22g. The numerical model enters post peak behaviour after these loads are applied with high value of displacements recorded in longitudinal wall...
adjacent to the street. The deformed shape of the numerical model is also presented in Figure 41. The load displacement diagrams are calculated for a node at the top of this wall.

![Deformed shape of the numerical model](image)

**Figure 41: Response of the structure in YY+ direction: a) load displacement diagram and b) deformed shape of the numerical model.**

The failure mechanism consists of out of plane failure of the external longitudinal wall of the cathedral into the interior of the cathedral. Extensive tensile damage can be observed in the north-western corner of the cathedral. Additionally flexural cracks develop also along the base course of this wall. The wall fails in an arch shaped segment near one of the entrances of the cathedral located on this wall, given the strong constraints imposed by the tower. The main mode of failure seen in the cathedral in the peak of the load displacement curve in terms of distribution of tensile strains (i.e. cracks) is shown in Figure 42.

Other regions of high concentration of tensile strains and correspondingly cracks are also observed in the model in this peak of the load displacement diagrams. These include vertical separation cracks between the front facade and the southern bell tower, and also in the corners between the walls of the altar and sacristy and the other longitudinal wall. Additionally, flexure induced cracks can also be observed at the base course of the other longitudinal wall. These regions are shown in Figure 43. It is noted that significant damage was experienced by the cloister adjacent to the cathedral and that a backing reinforced concrete structure exhibiting severe separation from the church was reported in the south-western corner of the cathedral by the University of Minho team.
Figure 42: Crack pattern depicting the failure mechanism: a) global crack pattern; close up view of concentration of tensile strains in b) north-western corner of the cathedral and c) near the northern bell tower.

Other regions of high concentration of tensile strains and correspondingly cracks are also observed in the model in this peak of the load displacement diagrams. These include vertical separation cracks between the front facade and the southern bell tower, and also in the corners between the walls of the altar and sacristy and the other longitudinal wall. Additionally, flexure induced cracks can also be observed at the base course of the other longitudinal wall. These regions are shown in Figure 43. It is noted that significant damage was experienced by the cloister adjacent to the cathedral and that a backing reinforced concrete structure exhibiting severe separation from the church was reported in the south-western corner of the cathedral by the University of Minho team.
The cracks occurring in the numerical model in the post peak phase of the load displacement diagram of this pushover analysis can be correlated to the cracks that can be observed in situ in the structure. A correlation of these cracks is provided in Figure 44.

The capacity of the structure under horizontal loading in the YY-direction of the model for the un-calibrated and calibrated models are summarized in Figure 45 in terms of load displacement diagrams. For the calibrated model, the maximum lateral load applied is 0.22g
while for the un-calibrated model is 0.18g, which is the controlling failure mode of the structure in terms of pushover analysis, as expected.

![Figure 45: Response of the structure in YY- direction: a) load displacement diagram and b) deformed shape of the numerical model.](image)

The numerical model enters post peak behaviour after these loads are applied with high value of displacements again recorded in the longitudinal wall adjacent to the street. The deformed shape of the numerical model is also presented in Figure 45. The load displacement diagrams are calculated for a node at the top of this wall.

The failure mechanism under horizontal loading in the YY- direction is again the out of plane failure of a section of longitudinal the wall which is adjacent to the street. This is indicated by the jump in out of plane incremental displacements observed just after the peak of the load displacement diagram. The crack pattern describing the failure mechanism is presented in terms of distribution of principal tensile strains in Figure 46.
Thus the failure mechanism observed under horizontal loading in this direction is associated with extensive cracking observed in the north-western corner of the cathedral and flexural cracks developing in the base course under. Additionally, concentration of tensile strains and correspondingly cracking is also seen in other parts of the model under pushover analysis in this direction. These regions include the connection between the northern bell tower and the same longitudinal wall, and also in the base course of the longitudinal wall of the same region as seen in Figure 47.

In the post peak region of the load displacement curve when mass proportional pushover analysis is performed for the numerical model in this direction, a number of cracks observed in the real structure are again predicted by the numerical model. But these cracks have already been correlated with in situ evidence in this section.

Erasmus Mundus Programme
5.2.2 Evaluation of Proposed Strengthening

The pushover analyses carried out on the calibrated and un-calibrated model of the masonry envelope of the Ica Cathedral could efficiently reproduce a large number of the existing cracks in the structure, which are attributed to its seismic history. The validated model is used in this section to assess the seismic performance of the proposed strengthening scheme. It is to be noted here that the results of the strengthened numerical model are here compared with the results obtained for the un-calibrated model as the properties input for the strengthened model were an improvement in the mechanical characteristics of the properties input in the un-calibrated model as explained in detail in 3.5. Moreover these pushovers were performed only in the YY direction of the numerical model as the proposed strengthening explained in 3.2 was directed at improving the seismic capacity of the cathedral in this direction.

The capacity of the strengthened structure under horizontal loading in the YY+ direction of the model is compared with the un-calibrated model in Figure 48 in terms of load displacement diagrams. It can be seen from Figure 48 that the maximum lateral load applied for both the un-calibrated and un-strengthened model is 0.22g. The numerical model enters post peak behaviour after these loads are applied with high value of displacements recorded in longitudinal wall adjacent to the street. It seems that there is no increase in the load capacity of the structure despite the strengthening.
The reason behind there being no increase in the seismic load capacity of the structure despite the proposed strengthening is investigated further by studying the failure mechanism of the strengthened model in this direction. The failure mechanism again consists of out of plane failure of the external longitudinal wall of the cathedral into the interior of the cathedral and is exactly the same as that of the un-strengthened construction, as seen in Figure 49. The principle tensile damages contributing to this out of plane failure take place in the region of adobe masonry near the north-western corner of the cathedral and near the northern bell tower. Moreover flexural damages can also be observed in both these similar failure mechanisms in the base course of this wall.

As it has been already explained in detail in 3.2, the proposed strengthening does not address specifically these regions. It is noted that the model does not consider any connection with the timber structure and effective connection to the timber structure might improve the response.

Figure 48: Response of the strengthened structure in YY+ direction: a) load displacement diagram and b) deformed shape of the numerical model.
Similarly in the YY-direction the load capacities of the un-calibrated and strengthened construction are calculated to be the same, having a value of 0.18g. In this direction also the failure mechanism for both the strengthened and un-calibrated model is due to the out of plane failure of the section of the longitudinal wall adjoining the street near the north-western corner of the cathedral: a region that can also be further considered in the strengthening plan. This is illustrated in Figure 50.
5.2.3 Observations and Recommendations

It is observed from Table 15 that the masonry envelope has a better seismic performance in the XX direction of the numerical model than YY direction, as expected. Transversal directions are usually the weakest in this type of buildings. The maximum lateral load capacity of the masonry structure is in the XX-direction of the numerical model which is a value of 0.36g. The minimum capacity of the masonry structure is in the YY-direction of the numerical model which is a value of 0.18g, which is the controlling failure load and mechanism.
Table 15: Comparison of lateral load capacity of the calibrated and un-calibrated models in all four directions

<table>
<thead>
<tr>
<th>Direction</th>
<th>Lateral Load Capacity</th>
<th>Diff. in Load Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>XX+</td>
<td>0.30g</td>
<td>0.27g</td>
</tr>
<tr>
<td>XX-</td>
<td>0.36g</td>
<td>0.33g</td>
</tr>
<tr>
<td>YY+</td>
<td>0.25g</td>
<td>0.22g</td>
</tr>
<tr>
<td>YY-</td>
<td>0.22g</td>
<td>0.18g</td>
</tr>
</tbody>
</table>

It is observed that the increase in load capacity between the calibrated and un-calibrated model is moderate (about 10%) despite the change in material properties being only in terms of the modulus of elasticity of the different materials. The increase in lateral load capacity for the calibrated model is almost constant in all directions of the numerical model. This is despite the fact that the failure in each direction is controlled by progression of tensile damage in a different material or at the connection between different types of masonry. The results stress the moderate relevance of a proper mechanical characterization of the materials present in the structural subsystems of the Ica Cathedral, as this partly affects the results obtained from pushover analysis. Additional characterization of the masonry was performed during the inspection by the team from the University of Minho in terms of sonic tests, more details of which can be found in Greco et al. (2015). However, considering the high variation in values obtained and the fact that processing of results from these tests were carried out in the last period of the research period of the thesis, the model was only partly updated in accordance to the experimental results.

Full connectivity was assumed between various structural elements in this numerical model. The complex and deteriorated nature of connections present between the various structural elements such as between the pediment and the facade, the northern bell tower and the adjoining longitudinal wall have been observed during the various in situ inspections carried out in the structure and are reported in 2.2.2. The true nature of these connections could be incorporated in the numerical model with the use of interface elements and other numerical modelling approaches and is covered in the complete process of finite element model updating mentioned in the previous section. The assumption of full connectivity leads to
some increase in the load capacity of the structure as all the observed failure modes involves the connections between various structural elements, but this should again only provide moderate changes in the response as most failures found are indeed out-of-plane long wall failure mechanisms, which are effectively 2D and can be addressed at cross section level.

The failure modes in all the directions of the numerical model involves the development of flexural cracks at the base course of the masonry enevelope. However information about the configuration of the base course throughout the masonry envelope is available from only a limited number of wall prospections carried out as mentioned in 2.2.2 and seen in Figure 4. The nature and characterization of the base course can have some influence in these analyses, again expected to be moderate.

The results obtained from the pushover analyses carried out on numerical model of the proposed strengthening of the masonry envelope suggest that the proposed strengthening is not improving significantly the seismic performance, even if it is noted that the timber structure is not included in the FEM model. It is certain that the timber structure is rather complex and the connections between the two substructures (masonry and timber) play a relevant role in the response. From the results obtained in the masonry structure alone, it seems that the FEM model is representing well the global structural response.

Seismic performance is not improved in terms of increasing the load capacity of the structure or in the prevention of the failure mechanisms affecting the structure without strengthening. The strengthening does not seem to address a rather vulnerable region of the structure identified by the pushover analysis, i.e. the wall near the north-western corner of the cathedral. Reduction of the out-of-plane vulnerability of the external cathedral wall might be proposed by tying, additional buttressing or other measures. Additionally mechanical characterization of the geogrid proposed to be used could be beneficial.
5.2.4 Time History Analysis

The finite element model of the masonry envelope was also subjected to nonlinear dynamic loading with time integration. As previously mentioned, the cathedral currently stands in a heavily damaged situation after the 2007 and 2009 earthquakes. The first part of this section of the thesis discusses the methodology and tools used to carry out this analysis. The latter part of the thesis is focused on validating the model in terms of damage and collapse mechanisms seen in the structure after the earthquakes it has been subjected to.

Adopted Parameters

Seismicity of Peru

The national territory of Peru is divided into three seismic zones. This is done on the basis of factors like general characteristics of past seismic activity, spatial distribution of observed seismicity, attenuation with respect to epicentral distance and also on neotectonic information. Each of these zones is associated with a factor which corresponds to a value of PGA with a ten percent probability of being exceeded in fifty years. The seismic zones of Peru are illustrated in Figure 51 and the associated PGAs are tabulated in Table 16.

![Figure 51: Seismic zones of Peru (Peruvian Code) and Ica location](image)

<table>
<thead>
<tr>
<th>ZONE</th>
<th>PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>ICA</td>
<td></td>
</tr>
</tbody>
</table>

Table 16: PGA values associated with seismic zones in Peru
As it can be seen in Figure 51, the Ica Cathedral is located in Zone 3 which corresponds to a peak ground acceleration of 0.4g. Soil effects are addressed below.

**Loading**

A single earthquake in the form of two acceleration time histories was applied to the structure to study its dynamic behaviour. These two accelerograms were applied to it in the X and Y directions of the numerical model as base excitation. The accelerograms applied are both compatible with the elastic response spectrum specified for Peru and defined by (12) and (13).

\[
0 \leq T \leq T_b : S_a = 2.5ZS \tag{12}
\]

\[
T_b \leq T \leq T : S_a = 2.5ZS \left(\frac{T_b}{T}\right) \tag{13}
\]

where:

- \(Z\) is the zone factor defined in Table 16
- \(S\) is the soil factor which depends on the soil type and is defined in Table 17.
- \(T_b\) also depends on the soil type/soil factor and is defined in Table 17.

<table>
<thead>
<tr>
<th>SOIL PARAMETERS</th>
<th>SOIL TYPE</th>
<th>(S)</th>
<th>(T_b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(S1)</td>
<td>1</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>(S2)</td>
<td>1.2</td>
<td>0.6</td>
<td></td>
</tr>
</tbody>
</table>

Table 17: Soil Parameters for calculation of elastic response spectrum of Peru
Since the Ica Cathedral is located in Zone 3 of the seismic hazard map of Peru as seen in Figure 51, the value of Z is taken as 0.4. Moreover assuming soil type to be S1, the elastic response spectrum for this case is calculated as seen in Figure 52.

Using the calculated elastic response spectrum, artificial accelerograms are generated using the software SeismoArtif v2.1 (SEiSMOSOFT, 2013). They were generated using a trapezoidal envelope shape having rise and fall times of 5s each and a level time of 15s. A time step of 0.01s was assigned over their total duration of 20s. The accelerograms were further subjected to linear baseline correction and the Butterworth filter was applied using the software SeismoSignal v5.1 (SEiSMOSOFT, 2013). This was done to eliminate noise and to force the time history to start and end with zero. The accelerograms that were finally applied as loading to the structure are presented in Figure 53. These were applied in two orthogonal directions to each other (X and Y). It can be seen in Figure 53 that they are uncorrelated to each other i.e. peaks in acceleration do not occur at the same time with respect to each other and the model does the maximum acceleration is about 0.4g, the design peak ground acceleration at the site of the cathedral.
Damping of the System

Damping of the structure considered in the nonlinear dynamic analyses was simulated as Rayleigh viscous damping. Similar damping mechanisms are assumed to be distributed throughout the structure and classical damping is a good approach to idealize the dissipation of energy throughout the structural system in such cases. Rayleigh viscous damping calculates the classical damping matrix $C$ of the structure as a linear combination of mass $M$ and stiffness $K$ as seen in (14) (Chopra, 2012).

$$C = \alpha M + \beta K$$ (14)

Here $\alpha$ and $\beta$ are two coefficients which weight the contribution of the mass and stiffness matrices of the structure to the viscous damping matrix $C$. The value of these coefficients are related to the damping ratios associated with the natural modes of vibration of the structure. Generally the value of these coefficients are calculated considering two modes of frequency with known values of damping ratios with the help of (15), (16) and (17).

$$\xi_n = \frac{1}{2} \left( \beta \omega_n + \frac{\alpha}{\omega_n} \right)$$ (15)
The natural frequencies of the structure used to determine the viscous damping through (14)-(17) should preferably lie in the linear range (without damage). The $i$ th and $j$th modes should be chosen carefully in order to ensure a reasonable estimation of values of damping ratio for all modes lying in between them and contributing to the response of the structure (Chopra, 2012). In order to approximate the Rayleigh damping parameters of the masonry envelope of the Ica Cathedral the modes selected as the $i$ th and $j$ th mode are presented in Table 18.

### Table 18: Modes used for calculation of Rayleigh damping parameters

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Cumulative Mass Participation (%)</th>
<th>$\omega_i$ (rad/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>1</td>
<td>1.80</td>
<td>0.00</td>
<td>13.06</td>
</tr>
<tr>
<td>180</td>
<td>16.75</td>
<td>76.83</td>
<td>80.06</td>
</tr>
</tbody>
</table>

The choice of these modes is governed by the fact that more than 80% of the mass of the structure participates in the modes lying between these two modes as seen in Table 18. A damping coefficient of 3% is assumed for these modes and the Rayleigh damping parameters obtained are $\alpha = 0.6124715700$ and $\beta = 0.0005147286$. The variation of damping ratio along various modes for these parameters is graphically represented in Figure 54, which is in between 1.5 and 3%. The experience at University of Minho regarding time history analysis shows that the structural response is not typically sensitive to these small damping variations.
Structural Analysis of the Masonry Envelope of Ica Cathedral, Peru

Strong dynamic loading creates damage in the structure. Subsequently this damage causes a change in the value of frequencies and damping ratios associated with the natural frequencies of the structure. Totally damaged integration points in a numerical model contribute significantly to the increase of damping forces in a nonlinear dynamic analysis. Hence, ideally, the damping of the structure could be updated at the end of each time step. But this often leads to an increase in the computation time associated with such analyses especially with large numerical models and was not performed in the nonlinear dynamic analysis of the model of the masonry envelope of the Ica Cathedral.

**Time Integration Method**

An important choice to be made while performing nonlinear dynamic analysis is the choice of the time integration method to use. Time integration methods need to be used in such analysis as the analytical solution of a structural system to which an earthquake has been applied is not possible due the high variation in the excitation and due to the nonlinear behaviour of the structure. Time integration methods solve this problem through the numerical integration of the various differential equations involved. These methods can be of two types: implicit and explicit. In explicit methods, the unknown, which here is the response of the structure at a time $t_{i+1}$, depends on $t_i$. Hence all unknown parameters can be explicitly evaluated without solving the equation of motion at time $t_{i+1}$. However the
solution by such methods can become unstable for large time steps $\Delta t$ where $\Delta t = t_{i+1} - t_i$. Implicit methods on the other hand compute the structural response at time $t_{i+1}$ by solving the equations of motion at time $t_{i+1}$. With a proper choice of control parameters associated these methods become unconditionally stable. Implicit time integration methods were used in the nonlinear dynamic analysis carried out in this thesis.

The choice of this method while performing such analysis for historical masonry structures is also governed by the quasi-brittle behaviour of masonry. Numerical noise is introduced by the abrupt change from linear elastic to fully cracked state involving zero stiffness. The propagation of high frequency spurious vibrations can be attributed to the quasi-instantaneous changes in the displacement field (Cervera et al., 1995).

With all these considerations, an appropriate choice for such analyses is the Hilber-Hughes-Taylor method (also called the $\alpha$ method) which introduces numerical damping of these vibrations without diminishing the accuracy of the solution (Faria, 1994). The finite difference equations involved in the HHT method are:

$$u^{t+\Delta t} = u^t + [(1 - \gamma)u^t + \gamma u^{t+\Delta t}]$$  \hspace{1cm} (18)

$$u^{t+\Delta t} = u^t + u^t\Delta t + \left[\left(\frac{1}{2} - \beta\right)u^t + \beta u^{t+\Delta t}\right]\Delta t^2$$  \hspace{1cm} (19)

where

$$\gamma = \frac{1}{2}(1 - 2\alpha)$$  \hspace{1cm} (20)

$$\beta = \frac{1}{4}(1 - \alpha)^2$$  \hspace{1cm} (21)

Here $u, u^t, u^t$ represent displacement, velocity and acceleration respectively, $\gamma$ and $\beta$ are parameters associated with the Newmark’s method, $\alpha$ is a parameter associated with the
HHT method and $\Delta t$ is the time step. Thus at a time $t + \Delta t$, for a multi-degree-of-freedom (MDOF) system the equation of motion using the HHT method reduces from (22) to (23).

\[
M\ddot{u}^{t+\Delta t} + C\dot{u}^{t+\Delta t} + Ku^{t+\Delta t} = f_{ext}^{t+\Delta t} \tag{22}
\]

\[
M\ddot{u}^{t+\Delta t} + (1 + \alpha)C\dot{u}^{t+\Delta t} - \alpha Cu^{t} + (1 + \alpha)f_{int}^{t+\Delta t} - \alpha f_{int}^{t} = (1 + \alpha)f_{ext}^{t+\Delta t} + \alpha f_{ext}^{t} \tag{23}
\]

where

\[
f_{int} = Ku \tag{24}
\]

Here $f_{int}$ and $f_{ext}$ are vectors for restoring and external loads respectively.

The HHT method is second order accurate and unconditionally stable. It is to be noted that for a value of $\alpha = 0$, the finite difference equations of the HHT method reduce to those of the Newmark’s method. The value of $\alpha$ adopted is determinant of the amount of numerical damping adopted. Acceptable range of values of $\alpha$ are between $-\frac{1}{3}$ to $\frac{1}{2}$. The value of $\alpha$ is indirectly proportional to the amount of numerical damping adopted. Generally higher damping is adopted for high frequency modes and lower damping for low frequency modes (Mendes, 2012). In the solution of the nonlinear dynamic analyses performed in this thesis, a value of $\alpha = -0.1$ has been adopted (TNO DIANA, 2014).

A very important parameter that needs to be chosen for nonlinear analysis with time integration is the time step $\Delta t$ to be adopted. The choice of an appropriate $\Delta t$ is based on two considerations:

- $\Delta t$ should be chosen such that it is much smaller than the total duration of the analysis: $\Delta t < t_d$
For the contribution of the mode with the lowest frequency and highest time period of vibration to be taken into account with an error of less than 5% involved, $\Delta t$ must be defined as

$$\Delta t = \frac{1}{20} T_i$$  \hspace{1cm} (25)$$

As seen in Table 18, the mode with the lowest frequency and highest time period considered in this analysis is the 180th mode. Hence $\Delta t$ is calculated as $0.00298454s$. For the time stepping procedure this value is rounded off as $0.0025s$.

Iterative solution methods are used to achieve equilibrium in each step of the nonlinear analysis, and in this thesis the secant method is used. A convergence criterion based on internal energy with tolerance equal to $10^{-3}$ was used.

Results

Under the application of the two accelerograms seen in Figure 53 as base excitation (a rather severe earthquake), a lot of damage is seen in the structure in terms of cracks arising due to tensile strains. This is illustrated in Figure 55.

Figure 55: Cracking in terms of principal tensile strains present globally in the structure
A more comprehensive understanding of the dynamic behaviour of the structure can be understood only by performing more than one nonlinear dynamic analysis with time integration and then comparing the response of the numerical model under different dynamic loading scenarios. However the required computational effort to perform such an analysis made it impossible in the time period in which the thesis was performed. The response of the numerical model to the dynamic loading was studied in terms of:

- Relative displacements in elevation (along vertical alignments) in different parts of the structure
- Time history of displacements in different parts of the structure
- Hysteretic response of masonry in the structure
- Co-relation of cracks observed in the numerical model with cracks which can be observed in situ.

The structure was hence organized into ten different regions of interest and the results from each region was analysed separately. Figure 56 shows the different parts into which the structure was organized in order to understand their behaviour under dynamic loading. The organization was done quantitatively on the basis of: capturing the response of different structural elements and on the basis of regions in which concentration of tensile damages are observed.

It is to be noted before the results are presented in this section that the term relative displacements referred to here in this section refers to the displacement of the node relative to the node situated directly vertically below it at the base of the structure. This is done in...
order to understand the exact movement of the considered node during the analysis as the excitation in this analysis is applied as a base excitation.

Relative Displacements in Elevation

Initially from each of the regions defined in Figure 56, the values of relative out of plane displacements of a few nodes along the same vertical alignment for the entire time history of loading are extracted. The maximum value of relative out of plane displacement occurring at each node along the vertical alignment considered is then extracted from its entire time history of displacements. The displacements obtained for each node are then plotted with respect to their height from the base. The graphs obtained can be assumed to give some idea of the movement of the corresponding structural element on which the vertical alignment of nodes are located during the entire period of loading. As such the maximum out of plane displacement during the entire time history, in the regions considered, was observed to occur along the vertical alignment considered in Region 1. The maximum out of plane displacement shown by the highest considered node along the vertical alignment of Region 1 during the entire time history of loading is 14 cm, which is certainly unacceptable from a structural perspective and seems to indicate collapse. Of the regions considered, the second highest values of relative out of plane displacement were observed in region 4. These results are plotted in Figure 57.

Figure 57: Relative displacements in elevation for selected regions of the structure (The direction mentioned corresponds to the direction of the numerical model)
It is interesting to note that of all the regions for which these calculations were carried out, region 1 and region 4 also show quantitatively a higher concentration of tensile strains and correspondingly cracking in the numerical model.

**Time history of displacements**

It is also interesting to study how a few points in the structure are actually displacing during the time history of loading. Figure 58 shows how nodes situated on the top of the wall in region 1 and region 4 are moving in a direction out of plane of the wall on which they are located i.e. here in the Y direction of the numerical model. As expected the displacements measured for the node in region 1 show a higher amplitude in displacements throughout the loading history compared to the node considered in region 4.

![Figure 58: Time history of out of plane displacements of nodes on the top of the walls in region 1 and 4 (Y direction of the numerical model)](image)

**Hysteretic Response of Masonry in the Structure**

The loading and unloading of structure when subjected to dynamic loading can be understood through the evolution of base shear in the structure with respect to horizontal displacements. To understand this, the base shear in the two orthogonal directions of the numerical model in which the artificial accelerograms were applied are plotted with respect to horizontal relative displacements in the same directions. The horizontal displacements and base shear in the Y direction of the model are plotted for a node on top of the wall in region 4 while for the X direction of the model, the same are plotted for a node in region 2 i.e. on the facade. The curves obtained are shown in Figure 59.
Figure 59: Base shear-horizontal displacement relationship in directions of dynamic loading of the model.

From Figure 59 it can be understood that for the given seismic amplitude of the dynamic loading applied to the structure (0.4g), the structure has reached its maximum load capacity and has undergone severe damage. In fact even for a lower seismic amplitude the structure might reach its maximum load capacity in both the directions of loading. This can be studied by performing other nonlinear dynamic analyses on the structure with higher and lower seismic amplitudes and studying the relationship between base shear and horizontal displacement. This was not performed once again due to the high computational effort required for performing such analyses.
Co-relation of Cracks

The nonlinear dynamic analysis applied to the masonry envelope of the Ica Cathedral is able to validate most of the damage seen in the structure currently as a result of the earthquakes it has been subjected to. This section of the thesis is focused on correlating the damage seen in the structure during numerous visual inspections carried out with the damage seen in the model after this analysis. These damages have already been explained in detail in 2.2.3.

The resulting damage observed in the numerical model is studied in terms of principal tensile strains. The images shown here were obtained by scanning the steps of the time-history analysis in intervals of four steps in order to find the most severe conditions of damage. Scanning is not performed for every step in the analysis carried out in this thesis due to much higher computational effort required (and the expected marginal differences).

The correlation of cracks obtained in this part of the thesis is carried out with detailed damage inspection survey of the Ica Cathedral present in Cancino et al.(2012). A large number of cracks observed in situ on the facade are also observed in the numerical model. These are graphically presented below in Figure 60. These cracks include the separation crack between the pediment and the rest of the facade. The crack progressing from the top of the pediment to its connection with the facade is also observed in the numerical model.

Additionally on the wall adjoining the cloister a large number of horizontal and vertical cracks are observed and were reported in the damage assessment carried out during one of the in situ inspections. This damage was also reproduced by the numerical model subjected to dynamic loading. It has to be noted here that, according to Cancino et al.(2012), the vertical cracks occurring between the adobe piers near the southern bell tower and lateral wall adjacent to them are due to a lack of connection between the fired brick base courses...
of the two elements. However these cracks occur in the numerical model under dynamic seismic loading even though full connectivity is assumed between them, due to the difference in elastic properties and impact. Also horizontal and vertical cracking is present in the northern and southern walls of the transept. The horizontal cracks are present at the lower levels of these walls and these cracks are also reproduced by the numerical model. Graphically these cracks are correlated in Figure 61.

A set of important cracks which were not validated by the numerical model under mass proportional pushover analysis were the parallel cracks present in the adobe masonry in the wall adjacent to the street. Additionally cracks present in the upper part of the corners of the walls in the chapels and altars were also reproduced (Figure 62).

Figure 60: Correlation of cracks observed from the numerical model and observed in situ on the façade (Drawing:Cancino et al., 2012).
Figure 61: Correlation of various cracks in the interior part of the masonry envelope between the numerical model and in situ observations: a) global crack pattern; b) near the southern bell tower; c) in the transept. Drawings: Cancino et al.(2012)
Figure 62: Correlation of various cracks between the numerical model and in situ observations: a) northern lateral wall; b) altar and chapels (Images: Cancino et al., 2012).
Observations and Recommendation

The numerical model reproduces most of the cracks which exist in situ on the structure under dynamic loading. Additionally it can be observed from Figure 59 that the structure has entirely lost its load carrying capacity when subjected to the applied dynamic loading having a seismic amplitude of 0.4g. A very interesting possibility would be to perform more nonlinear dynamic loading analyses on the model to assess accurately estimate better its load capacity.
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Chapter 6

Conclusions and Recommendations
Within the thesis, a detailed study of the masonry envelope of the Ica Cathedral was carried out. This chapter summarises the main conclusions which were reached while carrying out this entire body of work.

### 6.1 Conclusions

- From the pushover analyses performed it can be concluded that the masonry envelope has a better seismic performance in the XX direction of the numerical model than YY direction. The minimum capacity of the masonry structure is in the YY direction of the numerical model which is a value of 0.18g, and this is the controlling failure load and mechanism.

- Only a moderate increase in load capacity is observed between the results obtained for the un-calibrated and calibrated models. This gives rise to the question of how important is the mechanical characterisation of material properties in the structural subsystems of the Ica Cathedral.

- Connectivity assumed between various structural elements was complete connectivity. It is known that some of the connections present in the numerical model are deteriorated and cannot be accurately simulated by this assumption. Moreover it is seen that the failure modes identified by the pushover analysis are mostly occurring at the connections. Though the true nature of these connections could be incorporated with the help of various approaches in numerical modelling, the improvement in accuracy of results is highly doubtful as most of the failure mechanisms observed are out of plane long wall mechanisms which can be addressed at a cross section level.

- The assessment of the proposed strengthening shows that it does not significantly improve the seismic performance of the structure. Also it is seen that the strengthening does not seem to address the vulnerable regions of the structure identified by the pushover analysis.

- The structure seems to show severe damage for the applied dynamic loading having a seismic amplitude of 0.4g.
• A good co-relation of damages observed in-situ and in the numerical model is noticed. It can be concluded reasonably that the FEM model is representing well the global response of the structure.

6.2 Recommendations for further work

• The results of the pushover analyses present the base course to be vulnerable region of the structure. However very less information is available about this part of the structure. Additional characterization of this part of the structural subsystem could be beneficial though the contribution of this to the seismic capacity of the structure could be moderate.

• More detailed characterization of the geogrid proposed for strengthening could be beneficial to assess more accurately the effect of the proposed strengthening on the seismic behaviour of the Ica Cathedral.

• The numerical model considers only the masonry envelope of the Ica Cathedral. It is certain that the timber structure is complex and that the connections between the two substructures play a significant role in the seismic response of the cathedral. It could be very interesting to construct a combined model of the timber and masonry structures and assess the seismic capacity by performing pushover and time history analyses and then compare the responses.
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Further Development

The combined timber and masonry structures

This section presents additional integrated research carried out on the basis of the results obtained from the two thesis developed in parallel by Ciocci (2015) and Sharma (2015). The former is focused on the internal timber structure, while the latter on the external masonry envelope. The aim of this collaboration is to investigate how both the structural systems interact and influence the global behaviour of Ica Cathedral. The obtained results represent the foundation work for further developments.

1.1 Introduction

The results obtained from both the theses point out that the interaction between the timber and masonry structures is a critical factor that greatly influences the structural response of Ica Cathedral. A summary of these results which provide the motivation to proceed with this challenging endeavour are summarised below:

- Nonlinear seismic analysis on the masonry envelope shows much more extensive damage in the masonry than seen in the real structure (Sharma, 2015). Moreover, such damages are observed at lateral loads lower than or equal to the design peak ground acceleration.
corresponding to the location of Ica Cathedral by the Peruvian Code. This is despite the fact that the performance of the Ica Cathedral globally can be considered acceptable during the 2007 Pisco earthquake (Cancino, et al., 2012).

- Eigenvalue analysis of the masonry envelope results in no mode of vibration in which both the longitudinal walls of the cathedral are activated simultaneously (Sharma, 2015). On the other hand, experimental results obtained from the ambient vibration tests report most modes to involve both the walls together (Greco, et al., 2015). This can be safely assumed to be due to the connection of both these walls by the timber structure.

- The transversal beams close to the longitudinal masonry walls do not comply with the criteria specified by Eurocode (2004) for the Serviceability Limit State, as mentioned in (Ciocci, 2015). Along all the possible causes, this is likely due to ignoring the influence of the masonry.

- Eigenvalue analysis of the complete timber structure performed in (Ciocci, 2015) shows a correlation with the existing damage in Ica Cathedral due to the experienced earthquakes. Performing an eigenvalue analysis on the combined model with the timber and the masonry structure can provide a more in-depth co-relation.

- As mentioned in (Ciocci, 2015), the results obtained from the mass proportional lateral loading of the complete timber structure show clearly that it is difficult to evaluate a global stiffness value of the timber structure as the displacement of each point differs in value and direction. It is interesting to evaluate the stiffness of the combined model and compare the results with those obtained from the timber and masonry structures considered independently.

1.2 Definition of the numerical model

The finite element model of both timber and masonry structural systems is constructed in Midas FX+ for DIANA software, as shown in Figure 1. The numerical model is primarily based on the individual models initially prepared for them. All the details regarding the FE mesh, material properties, special features and considerations of these numerical models can be found in (Ciocci, 2015) and in (Sharma, 2015). In particular, Class–I beam elements are used to model the complete timber structure taking into account the shear deformation according to the theory of Timoshenko, while 3D isoparametric solid linear elements are used to model the masonry structure. In addition, one-noded translation
spring dashpot elements are used to simulate the influence of the adjoining cloister (TNO DIANA, 2014).

Figure 1. Numerical model of the combined model with both timber and masonry structures.

However, few geometrical adjustments are necessary in order to integrate both the models together. The crucial aspect that has to be considered with particular attention is the connection existing between the two structural systems. It should be mentioned that very little information is available in literature regarding the geometry of these connections (Cancino, et al., 2012). In order to perform this study, the connections are assumed to be between (1) the wooden beams in the upper part of the sotacoro and the fired brick façade and the longitudinal masonry walls, (2) the transversal wooden beams of the bays and the longitudinal masonry walls and (3) the wooden beams supporting the barrel vaults of the chapels and altar and the masonry walls (Figure 2). These connections are modelled in the numerical model by merging the nodes of the beam elements and the corresponding ones generated by imprinting them on the masonry. It should be mentioned that 0.3 m timber plates are assigned to the upper part of the masonry wall flanking the altar, as shown in Figure 2. This is
necessary to avoid the high concentration of stresses – that occurs even performing the analysis under self-weight – that was not observed during the experimental campaign (Greco, et al., 2015).

Finally, it should be mentioned that few elements are added when non-linear dynamic analysis is performed. These elements are added in the barrel vaults of the transept, of the altar as well as of the chapels in order to avoid multiple local modal modes in the model and to simulate the restraining effect provided by the cane and rendering in the real structure. The bracing elements are assumed made of cedar as the elements that they are connecting and they have a cross section 0.15 x 0.15 m², as assumed in (Ciocci, 2015). Moreover, Class-II beam elements are used for this analysis as they can be used for nonlinear analysis (TNO DIANA, 2014).

The material properties assigned to the timber structure of the combined model are assumed to be the same as those considered in (Ciocci, 2015). Concerning the material characterization of the
masonry structure, the values obtained after calibrating the numerical model with dynamic identification tests in (Sharma, 2015) are assumed in this study.

Regarding the boundary conditions, the base of the posts composing the timber structure are pinned as well as the nodes at the bottom of the masonry walls. It should be mentioned that all the timber joints are modelled as rigid connections.

1.3 Structural analysis

In order to understand the behaviour of the combined structures of Ica Cathedral linear and nonlinear structural analyses are performed on this model. This section includes nonlinear analyses under self-weight, mass proportional lateral loading and nonlinear dynamic loading.

1.3.1 Eigenvalue analysis

An eigenvalue analysis is performed on the combined model of the timber and masonry structures. The modal participation masses of the first thirty modes in each direction are presented in Table 1. Additionally, Table 2 shows the modes having modal mass participation percentage above 2%. It should be noted that some of the first modes are very similar to those obtained when only the complete timber structure is considered individually. However, it should be noted that the masonry is always activated. It can also be observed that both the longitudinal walls get activated simultaneously in the modes of vibration. Thus, the results of this analysis correspond better to the results obtained experimentally from the ambient vibration tests carried out on Ica Cathedral (Greco, et al., 2015). Selected mode shapes contributing significantly to the dynamic behaviour of the combined structure are shown in Figure 3.

The deformed mode shapes obtained from the eigenvalue analysis identify the main dome, the central part of the barrel vault with lunettes, the upper part of the barrel vault covering the chapels and the altar as the most likely vulnerable regions of the structure. It should be mentioned that these regions correspond to the mainly damaged ones present in the structure as a result of the experienced earthquakes (Cancino, et al., 2012). For instance, this can be clearly observed in the second mode of vibration – which corresponds to a frequency of 1.50 Hz and a modal mass participation of 3.6% in the XX direction (longitudinal) of the numerical model (Figure 3).
Finally, it should be noted that the combined model of the timber and masonry structures reaches a cumulative percentage of mass participation of 80% around 19.00 Hz. The same is reached for the timber structure and masonry envelope around 4.00 Hz and 17.00 Hz, respectively.

### Table 1. Modal participation masses for the first thirty modes in each direction (left).

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Modal Participation Mass</th>
<th>X-X (%)</th>
<th>Y-Y (%)</th>
<th>Z-Z (%)</th>
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| Total Cum. Percentage (%) | 8.30 | 21.04 | 0.13 |

### Table 2. Modes with the modal participation mass percentage higher than 2% (right).

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<th>Mode</th>
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<th>Y-Y (%)</th>
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Structural Analysis of Ica Cathedral, Peru

Mode 1 (1.42 Hz)
Mode 2 (1.51 Hz)
Mode 5 (2.09 Hz)
Mode 6 (2.20 Hz)
Mode 8 (2.31 Hz)
Mode 72 (3.04 Hz)
Mode 102 (4.02 Hz)
Mode 106 (4.17 Hz)
Mode 108 (4.17 Hz)
Mode 461 (8.60 Hz)

Figure 3. Selected mode shapes of the combined structures of Ica Cathedral.
1.3.2 Nonlinear analysis under self-weight

A nonlinear analysis is performed under self-weight applied in ten steps.

The deformed shape and distribution of vertical displacements in the combined model of the masonry and timber structure can be observed in Figure 4. It can be observed that the higher values of vertical displacement occur throughout the timber structure. In particular, the maximum vertical displacement is calculated in the upper part of the barrel vault covering the altar with a value of 2.0 cm, approximately. Significant values of displacement can also be observed in the transept including the main dome and the barrel vaults with lunettes. It should be noted that negligible change is observed in the vertical displacements occurring in the timber structure with and without the presence of the masonry envelope.

As shown in Figure 5, the distribution of the maximum principal stresses is studied in the masonry envelope in presence of the timber structure – the latter is hidden in Figure 5 to simplify the reading of the results. It is observed that the highest value of tensile stresses occur near the openings and at the location of the connections between the timber structure with the masonry envelope.
Similarly, the distribution of the minimum principal stresses is investigated throughout the masonry envelope in presence of the timber structure (Figure 6). It should be noticed that the highest value of compressive stresses occur at the base of the walls, as expected. Local concentrations of high values of compressive stresses are observed in the regions of the masonry envelope on which the timber structure rests upon, i.e. the barrel vaults covering the altar and the chapels.

1.3.3 Mass proportional lateral loading

In order to assess the seismic capacity of the structure, two mass proportional lateral loading analyses are performed. The lateral loads in these two analyses are applied in the XX- and YY-directions of the numerical model as the failure mechanisms of the masonry envelope identified in these two directions are the most interesting in (Sharma, 2015). The following section presents the
results of these analyses in terms of lateral load capacity and capacity curves, failure mechanism in the masonry and co-relation with cracks observed in-situ. The failure mechanism in the timber structure is not possible to be studied as timber is assumed to be a linear elastic and isotropic material. Comparison in terms of elastic stiffness is carried out with the results obtained from the timber structure and masonry envelope studied individually in (Ciocci, 2015) and in (Sharma, 2015). The load capacity of the combined model is also compared with respect to the load capacity of only the masonry envelope.

It should be mentioned that a number of problems are encountered in order to converge at a numerical solution to the combined model under mass proportional lateral loading analyses. For instance, a sudden change occurs in the value of stiffness matrices while applying lateral loading after vertical loading. These problems are resolved by trying out various iterative solution procedures and other techniques to improve convergence in nonlinear structural analysis provided by DIANA software (TNO DIANA, 2014).

**XX- (longitudinal) direction**

The maximum lateral load that can be applied to the numerical model of the combined timber and masonry structures in the XX-direction is 0.45g. The deformed mesh of the model when this load is applied is presented in Figure 7.

*Figure 7. Deformed mesh of the numerical model under applied loading in XX– direction.*
plane failure of the facade and the bell towers. This is studied in terms of high values of incremental displacements recorded at that part of the structure. The failure mechanism develops as tensile damages between the base of the bell towers and the adjoining longitudinal walls. It is to be noted that in the northern bell tower these damages progress from the connection between the two structural elements through the base of the bell tower. On the other hand, cracks leading to the failure of the southern bell tower progress from the connection between the bell tower and the wall to the adobe masonry of the longitudinal wall. Flexural damages also develop at the base of the facade. More damages can also be seen in the connection of the choir loft to the facade. Additionally, local failures can also be observed in the connection of the transversal walls of the altar with other adjoining walls (Figure 8).

![Diagram of Ica Cathedral](image)

**Figure 8.** Crack pattern depicting the failure mechanism: A) global crack pattern and the close up view of the tensile strains in B) the connection between the bell towers and facade and C) in the proximity of the altar (XX-direction).

As the structure enters the post-peak part of the capacity curve, the façade and the bell towers behave independently from the rest of the structure. This is verified by high values of incremental strains.
displacements observed in that part of the numerical model. Additionally, tensile strains and correspondingly damages progress extensively in the region. This is also confirmed by the high concentration of tensile strains observed in the region of the connection between the choir loft and the façade (Figure 9). The concentration of tensile strains in this region is very interesting as the separation of the pediment from the facade was also observed in-situ (Cancino, et al., 2012).

Figure 9. Cracking occurring in correspondence to the connection between the pediment and the facade observed in A) the numerical model and B) in-situ (Cancino, et al., 2012).

The capacity of the numerical model of the combined timber and masonry structures is presented in terms of a load displacement diagram in Figure 10. In particular, the curves MTS and MS refer to the numerical model of the combined structure and of the masonry envelope, respectively. The other legends refer to various structural parts of the model of the complete timber structure TS: in particular MD, BV, AD2 and TR refer to one node of the main dome, of the barrel vault with lunette close to the transept, of the small dome close to the cloister covering the bay adjacent to the façade and of the transept. More details can be found in (Ciocci, 2015).

Load displacement diagrams of only the masonry envelope and various components of the timber structure are also plotted along with it. It can be observed that the combined structure presents an increase in lateral load capacity to 0.45g in this direction compared to 0.36g observed when only the masonry envelope is considered. No appreciable difference can be observed in the stiffness presented by the masonry envelope and the combined structure under this direction of loading. It is also interesting to note the relationship between the stiffness of the various wooden structural components and that of the combined model. It should be noted that the stiffness value obtained for
TS_AD2 – i.e. for one node of the small dome close to the cloister covering the bay adjacent to the façade – is the most similar to that obtained from the combined model.

**Figure 10.** Comparison of the load displacement diagrams of the numerical models of the combined masonry and timber structures, the masonry envelope and various parts of the complete timber structure (XX-direction).

### YY- (transversal) direction

The maximum lateral load that can be applied to the numerical model of the combined masonry and timber structure in the YY-direction is 0.28g. The deformed mesh of the model when this load is applied is presented in **Figure 11**.

**Figure 11.** Deformed mesh of the numerical model under applied loading in YY-direction.

It can be observed that high values of displacements are recorded in the longitudinal wall adjacent to the street, in the transept and in the barrel vaults covering the altar and the chapel on the side of the...
street. The failure mechanism in the masonry under this loading is identified as the out of plane failure of this entire longitudinal wall. This failure mode also includes out of plane failure and separation of the northern bell tower from the brick facade and a portion of the transversal wall of the altar. Flexural cracks also occur throughout the length of the longitudinal wall: while these cracks are present at the rubble stone base in correspondence to the north–western corner of the cathedral, they are observed at the interface between adobe masonry and the fired brick base course in the part close to the northern bell tower. As previously analysed, the failure mode is studied in terms of high values of principal tensile strains and correspondingly cracking occurring in these regions (Figure 12).

Figure 12. Crack pattern depicting the failure mechanism: A) global crack pattern and close up view of the tensile strains in B) inner face of longitudinal wall and C) in the proximity of the northern bell tower.
As the structure enters the post-peak part of the capacity curve, this longitudinal wall and the northern bell tower behave independently from the rest of the structure. This is verified by high values of incremental displacements observed in that part of the numerical model. Also tensile strains and correspondingly damages progress extensively in damaged regions.

The capacity of the combined numerical model of masonry and timber structures is presented in terms of the load displacement diagram in Figure 13. It can be observed that in this direction the numerical model of the combined structures presents an increase in the lateral load capacity to 0.28g from 0.22g obtained considering only the masonry envelope. Unlike the XX- direction, an appreciable decrease can be observed in the stiffness presented by the combined masonry and timber structure with respect to the timber and the masonry structures considered independently. In this case, it should be noted that the stiffness value obtained for TS_TR – i.e. for one node of the upper part of the main dome – is the most similar to that obtained from the combined model.

**Figure 13.** Comparison of the load displacement diagrams of the numerical models of the combined masonry and timber structures, the masonry envelope and various parts of the complete timber structure (YY-direction).

### 1.3.4 Nonlinear dynamic loading with time integration

The finite element model of the combined timber and masonry structures is also subjected to nonlinear dynamic loading with time integration. A similar analysis is carried out by Sharma (2015) on the masonry envelope and the parameters in terms of loading, damping of the system, time integration method used in this analysis are maintained to be the same. It is also to be noted that unlike the other nonlinear analyses mentioned in this section, the material properties of masonry
used in this analysis correspond to the values input to the numerical model before calibration with respect to ambient vibration tests was carried out. This facilitates in comparing the dynamic response of Ica Cathedral with and without the presence of the complex timber structure and with the same mechanical characterization of the masonry as in (Sharma, 2015).

Concerning the loading, the same artificially generated uncorrelated accelereograms used by Sharma (2015) are applied on the numerical model of the combined timber and masonry structures in the form of base excitation in two orthogonal directions. These accelerations correspond to a maximum peak ground acceleration of 0.4g (Figure 14).

![Figure 14. Acceleration – Time Histories applied on the numerical model of the combined structures.](image)

Rayleigh viscous damping is also considered for the combined structure. The Rayleigh damping parameters for the combined structure are calculated as $\alpha = 0.389297260$ and $\beta = 0.0007876488$. They are calculated considering the natural modes of this numerical model between which there is 80% cumulative mass participation of the structure.

The time-integration method used is the Hilber-Hughes-Taylor method or the HHT method. In particular, $\Delta t$ adopted for this analysis is calculated as 0.0045s, corresponding to the highest time period of the natural mode of vibration considered.

The values of relative out of plane displacements along vertical alignments is studied in various regions of the structure. The north–western corner of the masonry envelope is identified as a very vulnerable region. In order to compare the out of plane movements with respect to the masonry envelope considered individually, the same results are collected for a vertical alignment in the same region of the model of the combined timber and masonry structures. As shown in Figure 15, the maximum out of plane displacement shown by the highest considered node along the vertical
alignment during the entire time history of loading is near 0.20 m while the same node shows a displacement of 0.14 m in (Sharma, 2015). Both of these are certainly unacceptable from a structural perspective and seem to indicate collapse.

![Figure 15. Relative out of plane displacements in elevation for a selected region of the structure.](image)

The loading-unloading diagram plotted for the masonry of the combined model in the two orthogonal directions in which the accelerograms are applied are presented in Figure 16. These diagrams indicate that masonry loses its loading–unloading capacity entirely. This corresponds to severe damage to the structure under the applied dynamic loading.

![Figure 16. Load Factor – Horizontal displacement (m).](image)

Critical cracks occurring in the structure are more well defined in the numerical model of the timber and masonry structure when subjected to nonlinear dynamic loading as compared to when only the numerical model of the masonry envelope is subjected to the same. This is illustrated in Figure 17.
1.4 Conclusion

On the basis of the results obtained from Ciocci (2015) and Sharma (2015), the numerical model of the combined timber and masonry structures was constructed. Linear and nonlinear analyses were performed in order to better understand the interaction of the two structural systems and how the global behaviour of Ica Cathedral is influenced by this interaction. The integration of the two structures in the combined model pointed out clearer correlation between numerical and experimental dynamic response of the structures. Moreover, an increase in seismic capacity of about 25% was observed in both orthogonal directions considered with respect to only the masonry envelope.

Due to the limitation of time and significant computational effort involved in running each analysis on this numerical model, several areas that need to be addressed by further research have been identified. These include the following:

- The timber joints were assumed to be rigid in the numerical model of the combined structures, hence the less conservative model was considered. This was due to the
non-complete geometrical information and mechanical characterization of the most critical joints of the timber structure that did not allow to consider the semi–rigid behaviour of the timber connections (Ciocci, 2015). However, it should be mentioned that even with these information the introduction of the nonlinear behaviour for each timber connection could have been difficult to carry out in the complete timber structure considering their large number throughout the model. A simplified approach can be used considering reduction factors to decrease the linear elastic stiffness. In particular, this reduction factor can be assumed in accordance with the recommendations provided by Eurocode or resulting from numerical simulations of local parts of Ica Cathedral. Running structural analyses considering these reduction factors would be essential in understanding the upper and lower bound of the combined structure in terms of seismic capacity and elastic stiffness.

- A simple process of model updating has been performed in (Sharma, 2015) where the MOE of various materials in the model was updated such that the modal shapes and frequencies obtained numerically match the ones obtained experimentally. This was done using an optimised iterative procedure. It seems sensible to carry out the updating process on this model as both during the ambient vibration tests as well as during the post processing of the data from such tests, the timber structure was assumed to be present (even though in a much simpler form in the latter). Additionally, further dynamic identification tests with the location of more sensors on the timber structure would be even more accurate in this regard. Mass proportional lateral loading analyses carried out on the masonry envelope showed a 10-20% percent increase in the seismic capacity in each direction between the calibrated and un-calibrated models. It would be very interesting to observe the change in seismic capacity of the combined structure after the same process is carried out for this model.

- The damage in the beam elements representing the timber structure can also be studied in terms of principal tensile strains. However, the number of integration points in the cross section of the beam elements (2x2) present by default is not enough to capture their nonlinear behaviour. This needs to be increased in order to capture the damage pattern. Each integration point in elevation corresponds to a
surface while the top and bottom surfaces of the beam elements are defined by the local axes.

- It is certain that the timber structure is rather complex and the connections between the two structures play a relevant role in the response. Thus there is a need for calibrating and fine tuning the connection adopted in the numerical model between the two structures. Further research into the characterisation of this connection is thus of the utmost interest and importance.

References


