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EVALUATION OF THE VULNERABILITY OF THREE CHURCHES IN VALPARAISO AND NUMERICAL CALCULATIONS

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PREFACE

During the last two missions at Valparaíso (May and October-December 2007), many local Organizations strongly cooperated to the *in situ* work of the experts coming from Italy: above all, the Municipality of Valparaíso (mainly the Heritage Office, “Oficína de Gestión Patrimoniál OGP”, thanks to the director Paulina Kaplan Depolo, providing logistic and technical support of about fifteen people, but also to Arch. Sotero Apablaza Minchel and all the OGP professionals); the Ministry of Culture (“Consejo Nacional de la Cultura y Las Artes”); the Regional Authority (“Intendencia V Region Valparaíso”); the Regional Civil Defense (“OREMI”); the SHOA (“Servicio Hidrográfico y Oceanográfico de la Armada de Chile”); Valpomío (“Programa de Recuperación y Desarrollo Urbano de Valparaíso”; the Firemen (“Bomberos” and “Bomba Italia”) and the Sea Rescue (“Bote Salvavidas”) Corps of Valparaíso; city organizations (“Junta de Vecinos” of the Cerro Cordillera and “Gerencia Barrio Puerto”, which is the historical district of the City); the Board of Architects of Valparaíso and other professionals; the Police (“Carabineros de Chile”); Church Authorities and other Universities (“Pontificia Universidad Catolica de Valparaíso”, “Universidad de Valparaíso”); the Valparaíso Italian Community. Finally, important was the contribution of the Geocom Santiago team, which provided the laser-scanner equipment and contributed strongly to the survey work.

Furthermore, after an agreement with the Istituto Italo Latino Americano (IILA), thanks to the General Secretary Ambassador Paolo Bruni and Dr. Eugenia Fedeli, and also with the authorization of the OGP Director Arch. Paulina Kaplan Depolo, 4 Chilean experts (two still belonging to the OGP, and other 2 working for OGP at the time of the Italian missions) have been entrusted of short bursaries in Italy (Spring 2008), specifically targeted on the “MAR VASTO” project activities. They have been:

Arch. Claudia Andrea Zuñiga Jara, OGP;
Arch. Mauricio Sebastian Gonzalez Loyola, OGP;
Arch. Cristian Ignacio Palma Valladares, Chilean expert;
Arch. Carolina Avalos, Chilean expert.

Moreover, the above said expert functionary of OGP, Arch. Sotero Apablaza Minchel, officially entrusted by the OGP Director, reached Italy in the same period and contributed in an excellent way to the “MAR VASTO” project and for the identification of future cooperation.

The Chilean experts have been involved in several activities:

a) contribution to the “MAR VASTO” project

- data transfer and elaboration of the laser scanner results of the three churches (San Francisco del Baron, Hermanas de la Providencia, La Matriz), with the aim to build up structural models for seismic analysis and identification of future strengthening interventions;
- preparation of hazard maps;
- definition of the seismic input for structural calculations to be performed on the above said churches;
- the organization of the final MAR VASTO project conference, planned in Valparaiso on next September 29-30, 2008, with the support of Local Authorities.

b) future cooperation

- an overall discussion on a future project regarding the intervention on the San Francisco Church, discussing a possible organization of a Chilean-Italian "join venture" (design and structural restoration work);

Reference documents are the general progress reports **[01-03]** and the specific task reports **[04-11]**.

1 INTRODUCTION

The problem of seismic vulnerability assumes particular characteristics when the historical and architectural heritage is concerned: the need of guaranteeing the originality of the structural elements, the geometric proportions of the architectural components, some artistic masterpieces preserved inside like frescoes, canvases, altars etc., increases the complexity of the problem and makes the solution more difficult to be achieved. It is a matter of paramount importance because a reliable evaluation of the seismic vulnerability of a historical building allows to define "a priori" the most effective actions to undertake in order to prevent the occurrence of structural damages and to guarantee, in such a way, the safety of the building itself and of anything inside.

Churches represent usually one of the most recurrent architectural typologies in historical urban nuclei. They are characterised by very peculiar aspects. In fact, contrary to what happens usually in ordinary buildings, the fabric is made up with few structural components (front and lateral walls, arches, columns, vaults, dome, etc.), the most of which can be so seriously damaged by an earthquake as to compromise the safety of the whole construction: as a matter of fact, religious buildings are characterised by remarkable dimensions and masses, weak and limited horizontal connections, high and slender masonry walls with no horizontal stabilizing elements, thrusting vaults, arches and domes that can amplify and worsen the effects of seismic events. By the way, structural damages in churches can be detected even in the presence of low/moderate earthquakes and their occurrence denotes the existence of safety problems actually related to the structural typology itself. Moreover, the economic damage is not always strictly proportional to the structural damage, because the costs for the restoration of frescoes, paintings, decorations in general may be very high even in presence of a limited crack pattern**.**

Since the first survey campaigns following the significant seismic events occurred in the last decades (Friuli 1976, Irpinia 1980, Modena and Reggio Emilia 1987, Lunigiana 1995, Umbria e Marche 1997, Piedmont 2000, Garda lake 2004), several studies were carried out in Italy with the aim of better understanding the failure modalities in the damaged buildings and improving the consolidation techniques. The direct observation of the failure modes usually recurring in masonry buildings showing similar characteristics enabled to hypothesize a set of failure mechanisms evolving from the first crack appearance up to the total collapse of the assembly of rigid bodies the building is transformed into after being cracked. The data surveyed were transferred to several *abaci* in which the principal failure modalities for the different structural typologies were reported in the form of interpretative graphical schemes; such *abaci* allowed to underline the difficulty to apply the same analytical procedures to buildings presenting pronounced typological differences. In particular, two main collapse mechanisms were considered, named mode I failure (out-of-plane masonry wall failure) and mode II failure (in-plane masonry wall failure). The first mode is the most dangerous and represents the response of the masonry wall to horizontal actions perpendicular to its plane: the collapse occurs by partial or total overturning of the wall. The latter collects the in-plane collapse modes corresponding to shear and/or overturning failures of masonry walls. Not uncommon is the occurrence of combined principal failure modes in consequence of the arbitrariness of the direction of the seismic action.

The concepts of *macroelement, typical and specific vulnerability* were defined.

The first is intended as a *specific portion of the building, with homogeneous characteristics from a structural point of view, coinciding or not with an architectural or a functional part*. The decomposition of the building into sub portions derives from the impossibility to define particular correlations to interpret the overall behaviour of the structure, and it is far more functional to generalisation, to the description of specific damage mechanisms and to successive applications of the limit analysis concepts. Moreover, in most cases, the damage observed indicates a *local* failure of some building portions which however does not cause the *global* collapse of the structure. The *macroelements* present a mutual interaction with visible cracks in correspondence of their contact surfaces or influence area; such boundary regions are characterized by scarce connections or damage patterns previously occurred between the facing macroelements. The macroelements approach for the structural evaluation of the seismic behaviour consists in the use of *local* simplified models (kinematic mechanisms), based on limit analysis and applied to single structural elements, rather than in the conventional evaluation of the overall structural behaviour; however, this approach can lead to a *global* evaluation of the structure, by successive application of the method to the different macroelements that compose the building.

The *seismic vulnerability* is defined as the tendency of a building to be damaged in the presence of a seismic event. The *typical vulnerability* is related to the above mentioned memory of already observed damage patterns in buildings presenting a similar typology; the concept of *specific vulnerability* can be referred, on the contrary, to the individual characteristics, structural details and weaknesses which appear in the structure and give rise to particular damage patterns. The interpretation of the observed data makes it possible to understand the general behaviour of the monumental building in the presence of horizontal seismic accelerations and in relation with its geometrical configuration which is partially responsible for the damage mechanisms **[12]**. As for the typical vulnerability of churches, the data collection and systematic observations of the structural damages started in Italy in 1976 immediately after the Friuli earthquake. The studies of the structural damages occurring in churches continued with the research work of Lagomarsino and Doglioni on some churches located in the central Regions of Umbria and Marche, hit by the 1997 earthquakes, in the districts of Lunigiana and Garfagnana and in the city of Catania. These studies allowed to define and implement the macroelement approach to churches: in fact the seismic response of churches may be described according to a precise recurrent phenomenology which can be related to the damage modes and the collapse mechanisms of the different macroelements; typical examples of macroelements in curches, which almost autonomous structural behaviours, are the facades, the bell towers, the apses and the side chapels. The macroelements approach allows a very effective qualitative interpretation of the seismic damage of churches. In fact, the main kinematic collapse mechanisms in the different macroelements were summarised in a limited number of damage mechanisms. Therefore, a methodology for damage and vulnerability assessment was established, based on a form for churches: the former version of the survey form implemented 18 indicators, and was essentially aimed at the definition of the seismic damage. The form analyzes the main church elements and their possible collapse mechanisms for seismic actions, according to the macroelement subdivision already elaborated. Successive versions of the form considered an increased number of damage mechanisms (28), allowing at the same time the definition of the seismic vulnerability of the analyzed building. The combined assessment of the damage level and of the construction characteristics allows to quantify, through an index, the damage produced by the earthquake and to define a vulnerability index of the church, characterising the response with regard to other seismic events.

For churches different kinds of damage can be identified:

- *Structural damage*: represents the reduction of the original capacity of the building to resist to an earthquake or to other actions;
- *Economic damage*: is the cost of repair of the architectural damage produced by the earthquake, including the costs for additional works to improve the structural response of the building to seismic actions and, therefore, to guarantee an adequate seismic prevention against further seismic events;
- *Cultural damage*: is the cost of restoration of the artistic assets.

The assessment of the damage in terms of macroelements and collapse mechanisms helps in a clear distinction among the above mentioned aspects of the damage, because the result is a preliminary diagnosis of the seismic response of the fabric. In fact the method does not require to survey the cracks, which may be connected with a non structural damage but it needs only the activation of different structural collapse mechanisms; moreover, previous damages due to other actions or instabilities of the fabric may not be classified in the 16 mechanisms. The diagnostic approach allows to formulate, automatically from the data collected in the form, a preliminary design for the damage repair and the seismic rehabilitation; moreover, these data form the basis for an estimation of the costs of intervention related to the economic damage **[13]**.

2 THE FORM FOR THE DAMAGE SURVEY AND THE CHURCH VULNERABILITY

The damage assessment is addressed to establish the feasibility of the works, to face eventual situations of danger for public safety, to avoid damage to the architectural heritage and to protect or shelter the artistic assets in a safe place. These inspections are also necessary to carry out a first approximate economic estimate of the damage, in order to allocate the resources for their rehabilitation. However, the first assessment of damage should represent a first moment of diagnosis of the sustained damage for the understanding of the building vulnerability, and a starting point for the project of rehabilitation.

The application in the assessment of damage in the Italian churches represents a remarkable test that evaluated the effectiveness of the form and the connected methodology. In fact the number of the checked churches was very high and the investigated area was very wide, corresponding with different levels of the macroseismic intensity.

The form used for the damage survey and the vulnerability assessment of churches is arranged in different sections. The heading contains the name of the building, its location and some general data on the site and the connections with the neighbouring buildings; the other sections take into account:

1. Typological and dimensional data: contains information about the typology and the dimensions of the church, subdivided into the different architectural elements (hall, presbytery, apse, transept chapel, roof covering, dome, crypt, facade, bell tower, vestry); particular attention is paid to the structural elements responsible for the seismic response of the building (buttresses, chains etc.).
2. Damage to elements of artistic value: the presence of artistic assets is noted inside the church and possible damage produced by the earthquake is indicated, without mention of its value.
3. Damage index and vulnerability index: in this section 28 possible damage and collapse mechanisms and characteristics of the different macroelements to be found in churches are identified. The form to be filled in is schematically illustrated, with an example for some mechanism, in Figure 1, together with the abacus, which illustrates the typical damage. For each mechanism it is indicated: a) the presence of a macroelement; b) the damage level (the method of assessing damage refers to the subdivision into five damage levels, provided by the European Macroseismic Scale EMS-98, which considers the following descriptions: 1- negligible to slight damage; 2- moderate damage; 3- substantial to heavy damage; 4- very heavy damage; 5- destruction); c) the intrinsic vulnerability of the building to that mechanism, through two indicators linked to specific construction weaknesses.
4. Characteristics of masonry: the different masonries in the various macroelements are described in attached forms, with reference to the characteristics of bricks/stones and mortar, to the external and internal masonry *apparatus*.
5. Safety: in this section the assessor is asked to judge the safety of the structure, choosing among four possible alternatives: safe, safe with actions of first aid, partially safe, unsafe.
6. Notes: are used to signal the necessity for urgent intervention to protect the asset or public safety and to highlight particular situations that might not be presented in the description of the typology and damage made in sections 1 and 3.
7. Illustrations: plans, prospects, sections and sketches to understand better the structural forms or particular damage mechanisms activated.

It is important to take into account different aspects. For example, in the case of mechanism 2 (damage to the gable of the façade), the damage can be caused by the stiffening of the roof covering (mechanisms 19-20-21) as a consequence of the laying of a reinforced concrete slab and of a concrete tie beam on the top of the walls: this intervention can increase the mass of the roof covering, with a consequent increment of the seismic forces, and the greater hardness of the crowning cannot allow the masonry below to deform naturally.

Bell towers are not elements that show a particular vulnerability to seismic action because their slenderness means a reduction of the seismic action, taxing on elements with a low oscillation period. By the contrary, in the case of outstanding elements, in overturning outside the plane of the spire the thickness of the spire can be significant and, therefore, greater loads and stiffness can be found with a consequent shearing of the masonry. If the spire is instead more slender, the

overturning mechanism can occur in the plane of the same, aided for example by the asymmetry of the piers or by a poor masonry fabric.
It is interesting to verify the correlation between the damage in the different macroelements and their geometric and construction typology, with particular reference to those technological devices identified for the assessment of the intrinsic vulnerability. As an example, for both the overturning mechanism of the facade and the transversal response of the nave (mechanisms 1 and 5) one of the vulnerability indicators is represented by the lack of tie rods: in the first case one refers to longitudinal ties anchored to the facade; in the case of mechanism 5 the transversal ones are considered, usually collocated in correspondence with stiffening arches.
In the analysis of the damage to mechanism 1 (overturning of the facade) the presence of longitudinal ties which connect the facade represents an efficient protection in the face of overturning outside the plane of the same: however, the tie rods do not prevent the occurrence of overturning, for they represent a local device of connection, that cannot work beyond certain limits of the action (the tie rod breaks or piercing of the masonry takes place). In this sense the absence of tie rods is a good indicator of vulnerability.
The transversal response of the nave means the oscillation of the longitudinal walls of the nave together with the arches which support the roof covering or the vault of the hall. This macroelement represents an extension of the arch-pier system and the mechanisms connected are characterised by cracks at the base of the walls, for the formation of hinges, and in the large arches, with possible continuation in the vault. In this case the presence of transversal ties imposes lateral movements at the top of the walls in phase, with consequent reduction of the damage especially in the vaults, which do not appear distorted by failing of the springers. For these reasons, the seismic improvement due to the tie rods is even more evident: but even in this case, they cannot always prevent the occurrence of overturning for strong earthquakes, because the tie is an element of local connection, that cannot work beyond certain limits of the seismic action **[14]**.

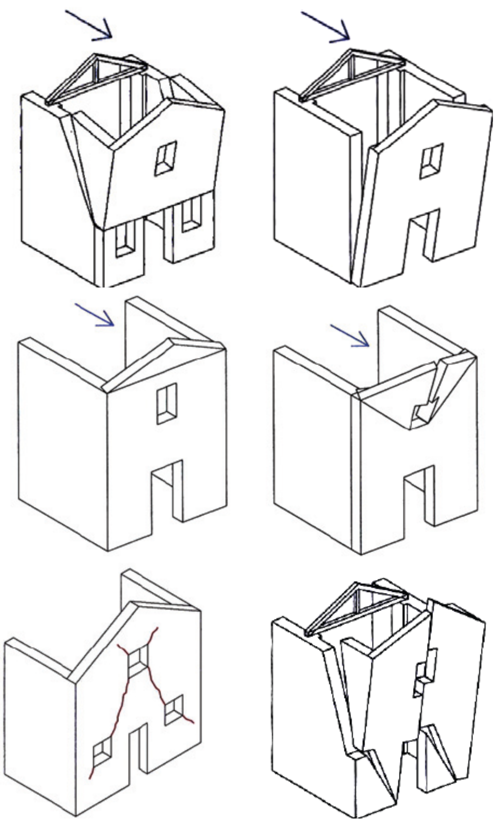
Figure 1: Schematic illustration of some of the 28 damage and collapse mechanisms contained in the form and related abacus.

THE FAÇADE

1. OVERTURNING OF THE FACADE

damage:
separation of the facade from the lateral walls, in proximity to the corner or with curved cracks in the lateral walls

vulnerability:
1. bad connection between the facade and the lateral walls
2. lack of longitudinal tie rods or buttresses



2. OVERTURNING OF THE GABLE

damage:
separation of the top of the facade in parts

vulnerability:
1. presence of wide openings that weaken the facade (rose window)
2. lack of connection with the roof covering (hammering of the ridge beam)

3. SHEAR MECHANISMS IN THE FACADE

damage:
cracks in the facade with X trend; central vertical crack; arched crack near to thcorner

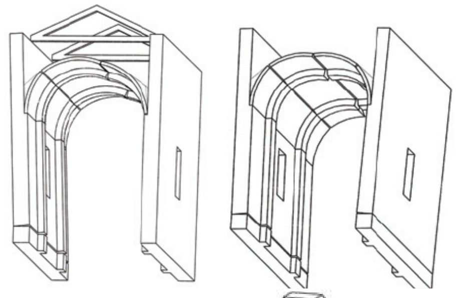
vulnerability:
1. presence of wide openings, even if closed with masonry
2. roof thrusting on the lateral walls and lack of transversal tie rods

THE NAVE AND THE TRANSEPT

5. TRANSVERSAL VIBRATION OF NAVE

damage:
cracks in the structural arches; rotation of the lateral walls, with crushing or opened cracks near to the base of the pillars

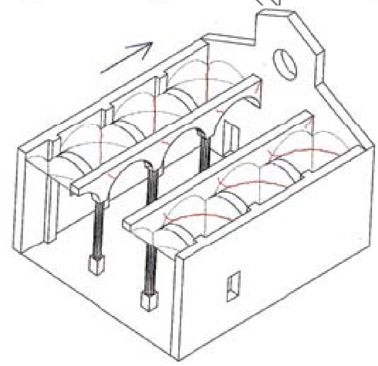
vulnerability:
1. lateral walls too slender
2. lack of transversal tie rods or buttresses



7. LONGITUDINAL VIBRATION OF THE CENTRAL NAVE

damage
cracks in the longitudinal arches; crushing or opened cracks at the base of the column; diagonal shear cracks in the vaults of the lateral naves

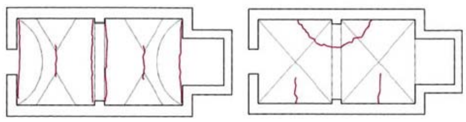
vulnerability:
1. slender columns and central nave very high with respect to the lateral ones
2. lack of longitudinal tie rods



8. VAULTS OF THE CENTRAL NAVE

damage
damage in the vaults, with disjointedness from the stiffening arches

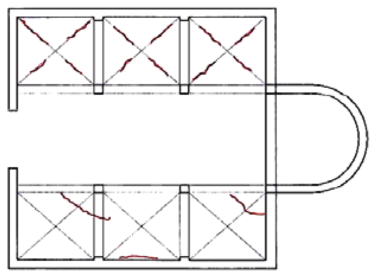
vulnerability:
1. vaults too lowered and/or too thin
2. presence of concentrated actions from the roof covering (wooden prop)



9. VAULTS OF THE LATERAL NAVES

damage
damage in the vaults, with disjointedness from the stiffening arches

vulnerability:
1. vaults too lowered and/or too thin
2. presence of concentrated actions from the roof covering (wooden prop)

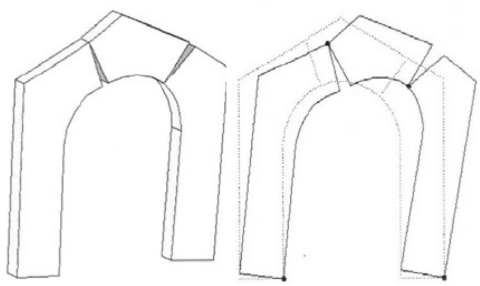


THE TRIUMPHAL ARCH

13. KINEMATISM IN THE TRIUMPHAL ARCHES

damage
formation of hinges in the arch, with opened cracks, crushing of masonry and sliding of stone ashlar

vulnerability:
1. thickness of the arch too thin or presence of masonry of bad quality
2. lack of tie rods or bad positioned; insufficient propping from the lateral walls

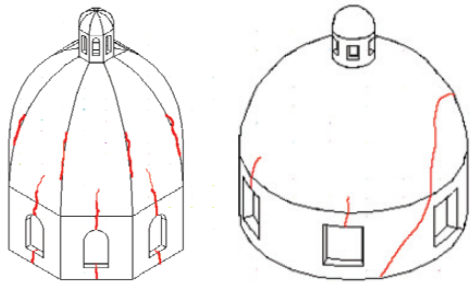


THE DOME AND THE TIBURIO

14. COLLAPSE OF THE DOME AND THE TIBURIO

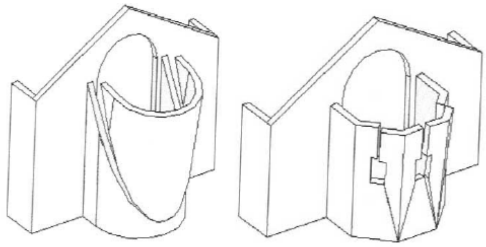
damage
formation of a continuum arched crack in the dome; cracks in the tambour

vulnerability:
1. tambour very high and slender (with big openings)
2. lack of ringing tie rods and of external buttresses



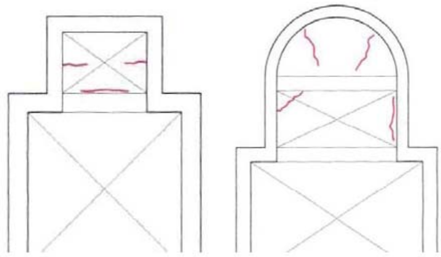
THE APSE

16. OVERTURNING OF THE APSE



damage
vertical cracks in correspondence of the windows; inclined arched cracks
vulnerability:
1. lack of ringing tie rods or of longitudinal tie rods
2. hammering roof covering or weakening for the presence of big openings

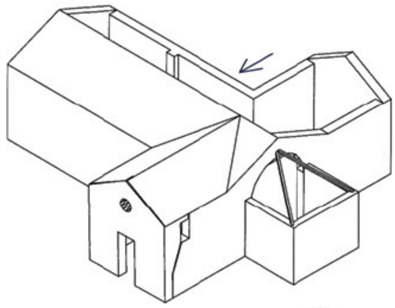
18. VAULTS OF THE APSE AND OF THE PRESBYTERY



damage
damage in the vaults
vulnerability:
1. vaults too lowered and/or too thin
2. presence of concentrated actions from the roof covering (wooden prop)

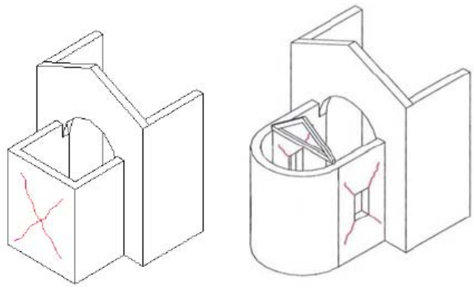
WIDESPREAD MECHANISMS

10-22. OVERTURNING OF OTHER WALLS (TRANSEPT FAÇADE, CHAPELS)



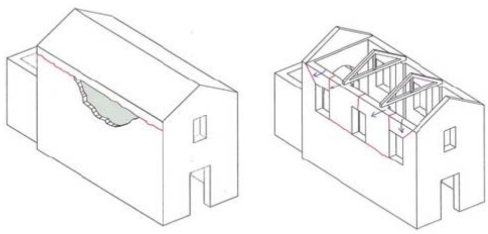
damage
separation of the end wall from the orthogonal walls
vulnerability:
1. bad connection between the wall and the orthogonal walls
2. lack of tie rods or buttresses

11-17-23. SHEAR FAILURE OTHER WALLS (TRANSEPT, CHAPELS, APSE, PRESBYTERY)



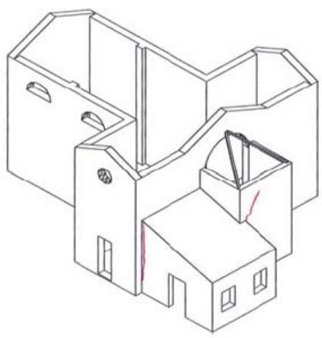
damage
inclined cracks in masonry; disjointedness in the lacks of continuity (closed windows)
vulnerability:
1. masonry of poor quality or too thin
2. weakening for the presence of too many openings

19-20-21. HAMMERING AND DAMAGE IN THE ROOF COVERING



damage
cracks in proximity to the support of the beam; disjointedness from the r.c. ring beam and the masonry
vulnerability:
1. hammering roof, with absence of a link from the wooden beam and the masonry
2. increasing of the weight and the stiffness of the roof (substitution with a r.c. slab)

25. INTERACTION BETWEEN ELEMENTS OF DIFFERENT BEHAVIOUR



damage
cracks due to the hammering between different parts
vulnerability:
1. significant difference in the global stiffness of the two parts of the fabric
2. lack of a good connection between the masonry in the two parts or of tie rods

BELL GABLE, SPIRES AND PROJECTIONS

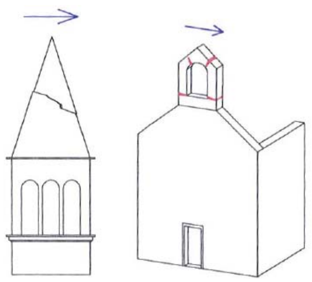
26. OVERTURNING OF STANDING OUT ELEMENTS

damage

global permanent rotations or sliding; cracks at the base of the element

vulnerability:

1. lack of an effective connection with the church
2. element too slender



THE BELL TOWER

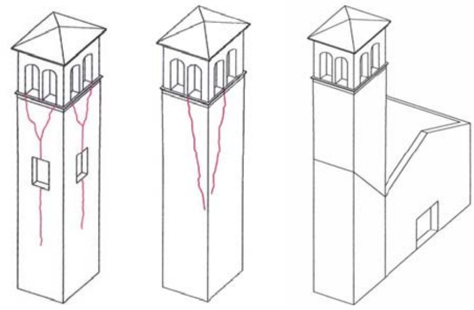
27. GLOBAL COLLAPSE OF THE BELL TOWER

damage

cracks near to the connection with the church; vertical cracks below the bell cell

vulnerability:

1. bell tower too slender and made of walls of limited thickness
2. masonry of poor quality and lack of connection between the walls



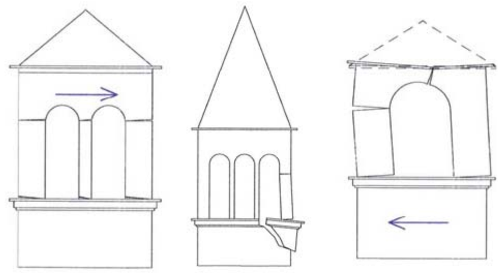
28. MECHANISMS IN THE BELL CELL

damage

cracks in the arches; rotations and sliding of the pillars

vulnerability:

1. lack of tie rods or hooping ties
2. pillars too slender and roof too heavy and/or thrusting



3 THE DAMAGE INDEX AS A MEASURE OF THE STRUCTURAL DAMAGE

The application of the form for churches has pointed out the effectiveness of the approach for macroelements and damage mechanisms: during inspection operations, the association of the seismic damage observed (cracks and deformations) was well fitted to a particular kinematic collapse mechanism and the exclusion of the earthquake as cause in the case of damage already existing and of other nature was possible.

The most of the damage mechanisms are related to actual macroelements, well visible in the churches; the remaining damage mechanisms may be activated in different parts of the church or refer to widespread cracking. It is worth noting that the concept of macroelement is aimed at a more effective understanding of the seismic response of the fabric, but it is not strictly necessary; the landmark of the methodology is the singling out of the collapse mechanisms.

Contextually with indexing of the damage, a qualitative judgement of the functioning of each macroelement is supplied, by pointing out the weaknesses of the fabric due to the absence of some structural details, usually present with the aim to prevent from seismic instability. The damage assessment carried out in this way represents, therefore, a real preliminary diagnosis of the effects of the earthquake on the building. In this sense also a light damage, found in a low stricken area, is particularly interesting because such cracks, that do not compromise the structure, warn of the vulnerability to that collapse mechanism; this is important in the zones less affected by the earthquake because it allows the highlighting of the vulnerability of the building. In certain situations the damage was worsened by a particularly poor quality of the masonry or by an evident lack of maintenance; in these cases the interpretation for macroelements must be integrated with the data in section 4.

The elaboration of data collected supplies, through the simple average of the levels of damage in the actual macroelements and the vulnerability scores, two indexes:

– Damage index: is a number between 0 and 1 which measures the average level of damage to the church, defined by the equation:

$$i_d = \frac{1}{5}\frac{\sum_{k=1}^{N}\rho_k d_k}{\sum_{k=1}^{N}\rho_k}$$

where: ρ_k is the weight associated to each mechanism, d_k is the damage in the k^{th} mechanism (from 0 to 5); N is the number of mechanisms that can be potentially activated in the church (N = 28).

– Vulnerability index: is linked to the propensity of the church to be damaged by the earthquake and is obtained through the equation:

$$i_v = \frac{1}{6}\frac{\sum_{k=1}^{28}\rho_k\left(v_{ki} - v_{kp}\right)}{\sum_{k=1}^{28}\rho_k} + \frac{1}{2}$$

where: ρ_k is the weight associated to each mechanism, v_k are the number of indicators of vulnerability and of the aseismic elements (a booleian variable) associated to the k^{th} mechanism (from 0 to 2); N is the number of mechanisms that potentially may be activated in the church.

The damage index is particularly useful, for it is a synthetic parameter which allows the definition of a hierarchy in the seriousness of damage. The vulnerability index, linked to the intrinsic characteristics of the building, is a parameter aimed at forecasting the damage in the church, due to the expected macroseismic intensity: in this way it represents a characteristic parameter of the structure, useful for vulnerability analysis of churches, even in areas not struck by recent earthquakes **[15]**.

4. SEISMIC HAZARD

Chile is one of the most seismic country in the world. Major earthquakes interesting the City of Valparaiso are reported in Table 1 and Fig. 2. "State-of-the-art" information has been provided by Chilean partners and stakeholders. Specific studies on seismic hazard, made by Italian partners and used for numerical analyses, are reported in **[08]**. The definition of the seismic input for numerical calculation is discussed in a separate paragraph (see paragraph 6.).

Table 1: Strong earthquakes interesting Valparaiso

date			location	Magnitude M
year	month	day		
1730	07	08	Valparaiso, Chile	8.7
1906	08	17	Valparaiso, Chile	8.2
1965	03	28	Near Santiago, Chile	7.1
1971	07	09	Valparaiso region, Chile	7.5
1985	03	03	offshore Valparaiso, Chile	7.8

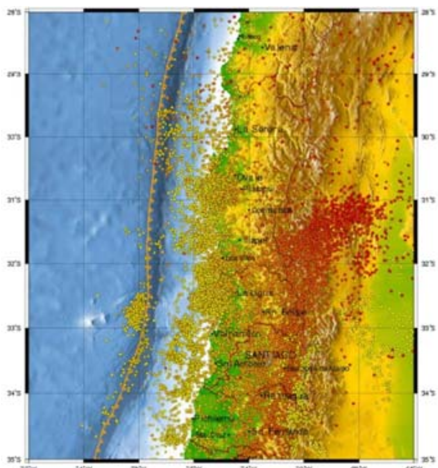


Figure 2: Earthquake in Chile (courtesy of UC).

5. NUMERICAL ANALYSIS AND THE FINITE ELEMENT METHOD

In spite of the intrinsic difficulties presented by the evaluation of the structural behaviour of historical structures, the choice of a numerical approach to assess the seismic response seems to be currently quite appealing, due to the possibilities offered by the continuative improvement of computational tools. The availability of suitable constitutive laws that take into account the non linear behaviour of different materials, implemented in several software packages, boosted this process. For instance, in the last decade, a substantial increment in the use of Finite Element Method (FEM) codes for the analysis of existing structures was noticed.

FEM is a powerful tool to study stresses and displacement in solids. A mathematical description of the material behaviour, which yields the relation between the stress and strain tensors in a material point of the structural element, is necessary for this purpose. This mathematical description is commonly named a constitutive model. Constitutive models of interest for practice are normally developed according to a phenomenological approach in which the observed mechanisms are represented in such a fashion that simulations are in reasonable agreement with experiments. It would be not realistic to try to formulate constitutive models which fully incorporate all the interacting mechanisms of a specific material because any constitutive model or theory is a simplified representation of reality.

The reliability of data emerged from the numerical simulation heavily depends on the closeness of the parameters implemented by the numerical model with the material properties experimentally defined, and on the similarity of the model - tested sample structural response.

The geometry can be idealised in different ways, namely, by considering the structure to be made of linear elements, two-dimensional elements or fully three-dimensional elements. Since the definition of the *structural* layout of an historical structure is not straightforwardly definable as in the case of “modern” buildings, this affect the choice of the most appropriate geometrical idealisation. Usually, the geometry of historical buildings is rather complex and in many cases there is no distinction between decorative and structural elements. As a first impression it would seem reasonable the use of three-dimensional elements, but also this strategy could be misleading in the sense that, implementing a too much detailed model, the feeling of the effective “bearing” structure is somehow lost. Also for computational reasons, the geometric idealisation should be kept as simple as possible, as long as it can be considered adequate for the problem being considered. In the geometric idealisation the following principles should be considered a priori:

- fully three-dimensional models are usually very time consuming with respect to preparation of the model, to perform the actual calculation and to analyse the results. Additionally, in the case of the widely spread FEM, many authors have been using solely one element (8 nodded) over all the thickness of the walls. The errors associated with such a discretization may be very large even in the case of a linear elastic analysis;
- the results of models incorporating shell elements are reasonably difficult to analyse due to the variation of stresses along the thickness of the elements. In addition, the large thickness of the structural elements might yield a poor approximation of the actual state of stress;
- increasing the details and size of the model might result in a large amount of information that may blurs the important aspects.

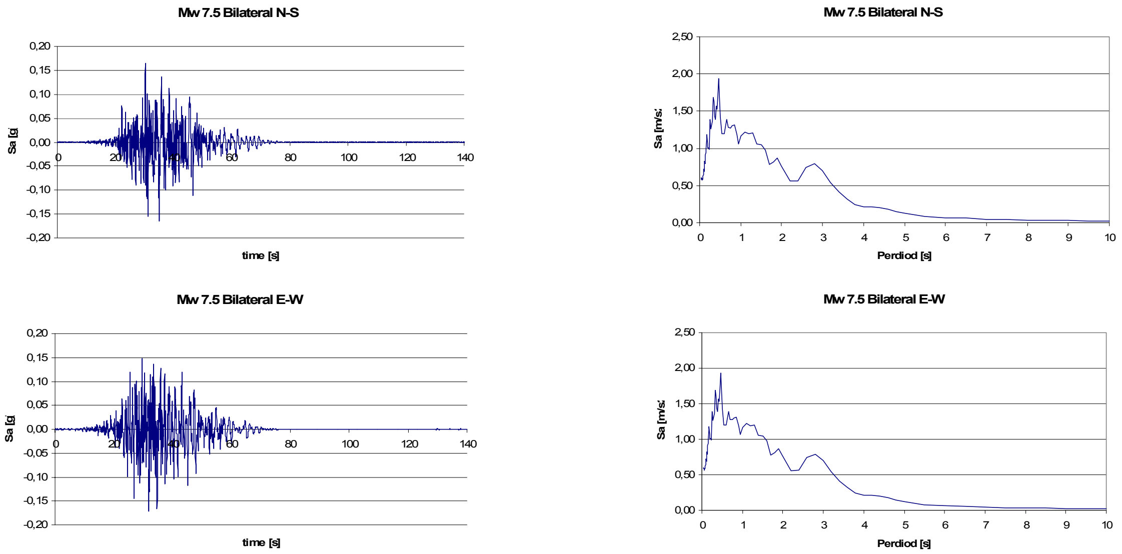
From a general point of view, a numeric representation can be achieved by separately modelling the basic constituents or following a global approach in the sense that the whole structure is schematized with finite elements implementing the assembly constitutive law. The first modelling strategy is known as micro-modelling, the second as macro-modelling. The first is again divisible into *detailed* micro-modelling and *simplified* micro-modelling.

The macro-modelling approach does not consider the different behaviour of unit and joint, treating the structure as a homogeneous continuum (implementing an isotropic or orthotropic constitutive law).

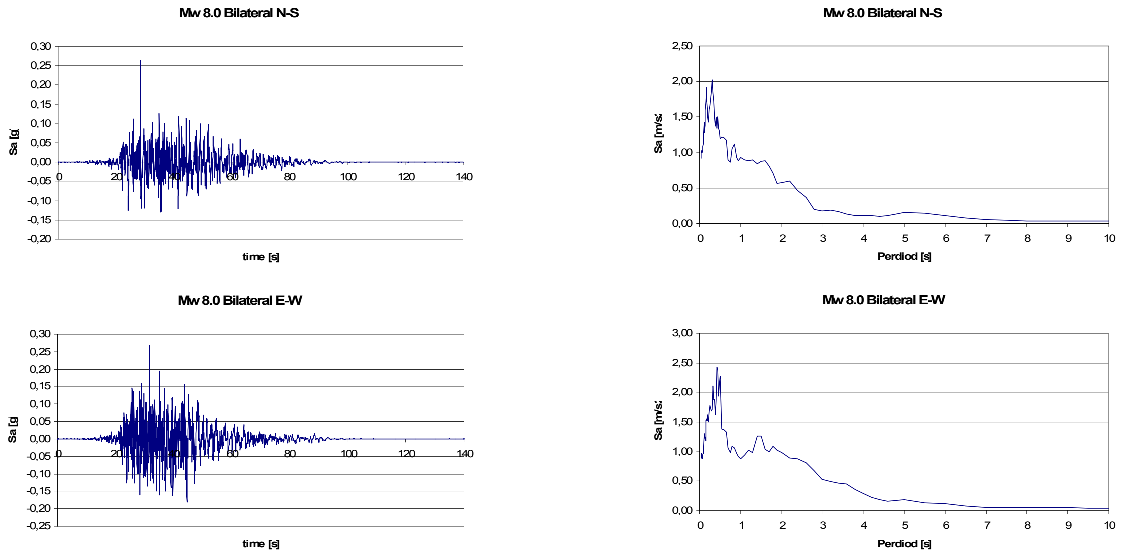
The equilibrium equations are solved within the finite element and thus the results in terms of stresses and strains are averaged on the dimension of the element. Generally speaking, the bigger is the element, the rougher is the "local" solution. In this formulation, it is implicit that the constitutive law adopted should be calibrated on the basis of experimental results referred to appropriate size samples in order to consider an average behaviour of the structural elements. Such different modelling approaches are applicable to solve different problems; for this reason a method can not be advised instead of another. In general, micro-models are used to numerically simulate the behaviour of laboratory specimens experimentally tested, and in general for small structures, subjected to states of stress and strain that are strongly heterogeneous. The use of micro-models is hence useful to the comprehension of the local structural behaviour. On the contrary, such strategy is not suitable for simulating the global behaviour of buildings, since the computational effort is in many cases excessive and the control on the overall structural behaviour is generally too limited to make this strategy attractive. For this purpose, macro-models are generally used, giving averaged values, being a compromise between accuracy and efficiency. The methodology adopted to skip from micro to macro level is the homogenization technique.

6. SEISMIC INPUT

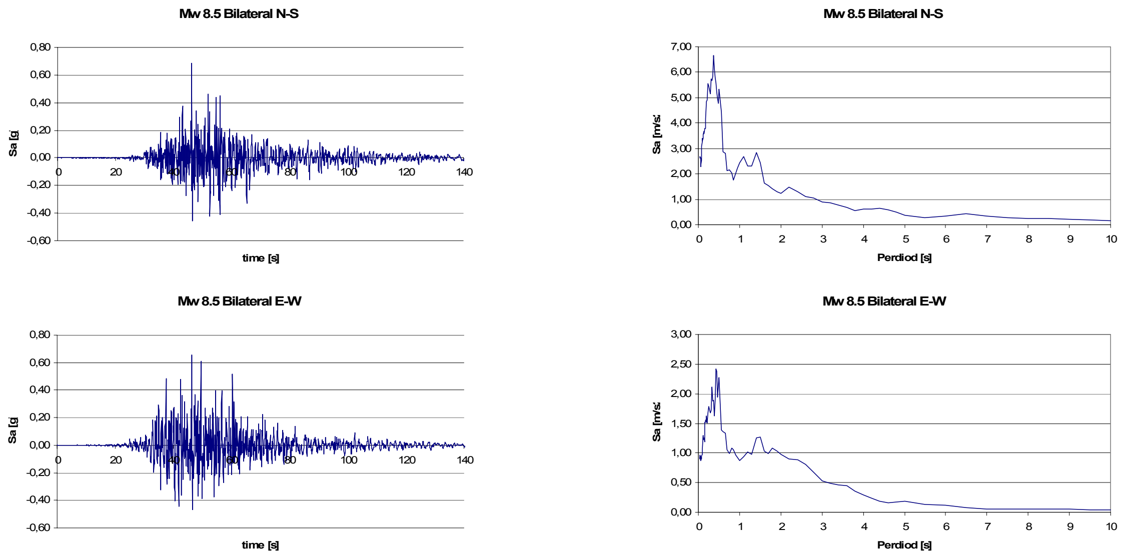
The seismic classification often relies upon the standard Probabilistic Seismic Hazard Analysis (PSHA) approach; that is acceptable, as a general indication of the hazard in terms of probability of excedance of an acceleration value, but that has been proven to be not fully satisfactory in several instances.
Case studies indicate the limits of the PSHA currently used methodologies, deeply rooted in engineering practice, supplying indications that can be useful but not sufficiently reliable to characterize seismic hazard: recent examples Kobe (17.1.1995), Bhuj (26.1.2001), Boumerdes (21.5.2003) and Bam (26.12.2003) events. In other words, the problem with PSHA is that its data are inadequate and its logic defective. Furthermore, for the cultural heritage protection, the concept of return period is of little value; in fact, such kind of patrimony, which must be handed down intact to posterity as far as possible, cannot be exposed to the roulette of the PSHA approach.
An innovative deterministic procedure has been developed and widely applied that supplies realistic time histories from which it is possible to retrieve peak values for ground displacement, velocity and design acceleration at bedrock level, in correspondence of earthquake scenarios. A proper evaluation of the seismic hazard, and of the seismic ground motion due to an earthquake, can be accomplished by following a deterministic or scenario-based approach, coupled with engineering judgment. This approach allows to incorporate all available information collected in a geological, seismotectonic and geotechnical database of the site of interest, as well as advanced physical modeling techniques to provide a reliable and robust basis for the development of a deterministic design basis for cultural heritage and civil infrastructures in general **[19]**.
It is to worth noting that the deterministic approach has been followed in the "MAR VASTO" project to evaluate the seismic input in the Valparaiso area for certain earthquake scenarios (in general), and in some sections underneath the churches locations (in particular) **[08]**.
In fact, another topic question is the identification of site amplification effects, due to geologic and topographic factors. Therefore, local seismic motion should be identified with accuracy, also performing micro-zoning surveys (to be improved in the Valparaiso urban area).
Another relevant aspect is the implementation of a reliable and widespread network, able to record in several points the seismic input (displacement, velocity and acceleration), not only in the field, but also at various levels of important structures, in order to gather indispensable data to enhance numerical analysis. For this reason, a logic step is to foresee the instrumentation of above said churches through monitoring systems, especially after a restoration intervention.
In the numerical analyses, seismic excitation input data related to the bilateral failure, taken from **[08]**, have been used; in fact, they show a higher excitation of the ground (Fig. 3).



Earthquake scenario: MW 7.5 Bilateral Rupture



Earthquake scenario: MW 8.0 Bilateral Rupture



Earthquake scenario: MW 8.5 Bilateral Rupture

Figure 3: Time histories and related acceleration Spectrum Curve for Damping Ratio of 5.0% (vibrational period vs spectral acceleration) for each expected magnitude and for each main direction used for the spectral response analysis of the Las Hermanas de la Providencia Church.

7. INVESTIGATION ON THREE IMPORTANT CHURCHES IN VALPARAISO

In the framework of the mission of the Italian team in Valparaiso (Autumn 2007) three important churches, located in different sites and made by different materials, have been investigated, in agreement with the OGP of the Valparaiso Municipality: La Matriz, San Francisco del Baron, Las Hermanas de la Providencia.

About each church, the following steps have been carried out:

- collection of historical data;
- laser scanner survey **[10]**;
- photographic survey;
- visual investigation and evaluation of maintenance and damage;
- vulnerability evaluation;
- execution of numerical calculations, if necessary;
- indication of rehabilitation actions.

Among the historical and architectonic information gathered at Valparaiso, a book regarding the Religious architecture was a fundamental reference **[16]**. Moreover, the Italian Guidelines for evaluation and mitigation of seismic risk to cultural heritage **[17]** have been taken into account as another important reference for our work.

8. IGLESIA DEL SALVADOR, MATRIZ DE VALPARAISO

8.1 General description

Periodically destroyed by earthquakes, tsunamis and fires, the present fourth version of the "Iglesia del Salvador, Matríz de Valparaíso" was constructed from 1837 to 1842 (and modifications after 1897), in the same place of the original first chapel, built after the discovery of the Valparaíso Bay in 1559, in the ancient nucleus of the "Puerto".

Figure 4: Location of the La Matriz in the Valparaiso urban tissue.

The church, in simple neoclassic style, is made by adobe perimetral walls (height 12 m and thickness 1.30 m), masonry façade, with a roof by clay tiles. The bell-tower (height 40 m), modified at the end of the XIX century, is wooden made and presents an iron spiral staircase inside. The internal colonnades, forming the naves, are also wooden made. In the XX century a certain damage occurred, due to seismic activity, scarce maintenance and termite attacks. Partial interventions have been done between 1971 and 1988. A specific report **[08]** focuses the seismic input also in the church's place. Fig. 4 shows the location of La Matriz in the Valparaiso urban tissue.

8.2 Laser scanner, geometric, photographic and damage survey

Figs. 5-8 show some pictures of the church. Thanks to the laser scanner **[10]**, geometric, photographic and damage surveys, carried out by the Italian team (Autumn 2007), a lot of information was obtained, in order to provide data useful for the following analyses.

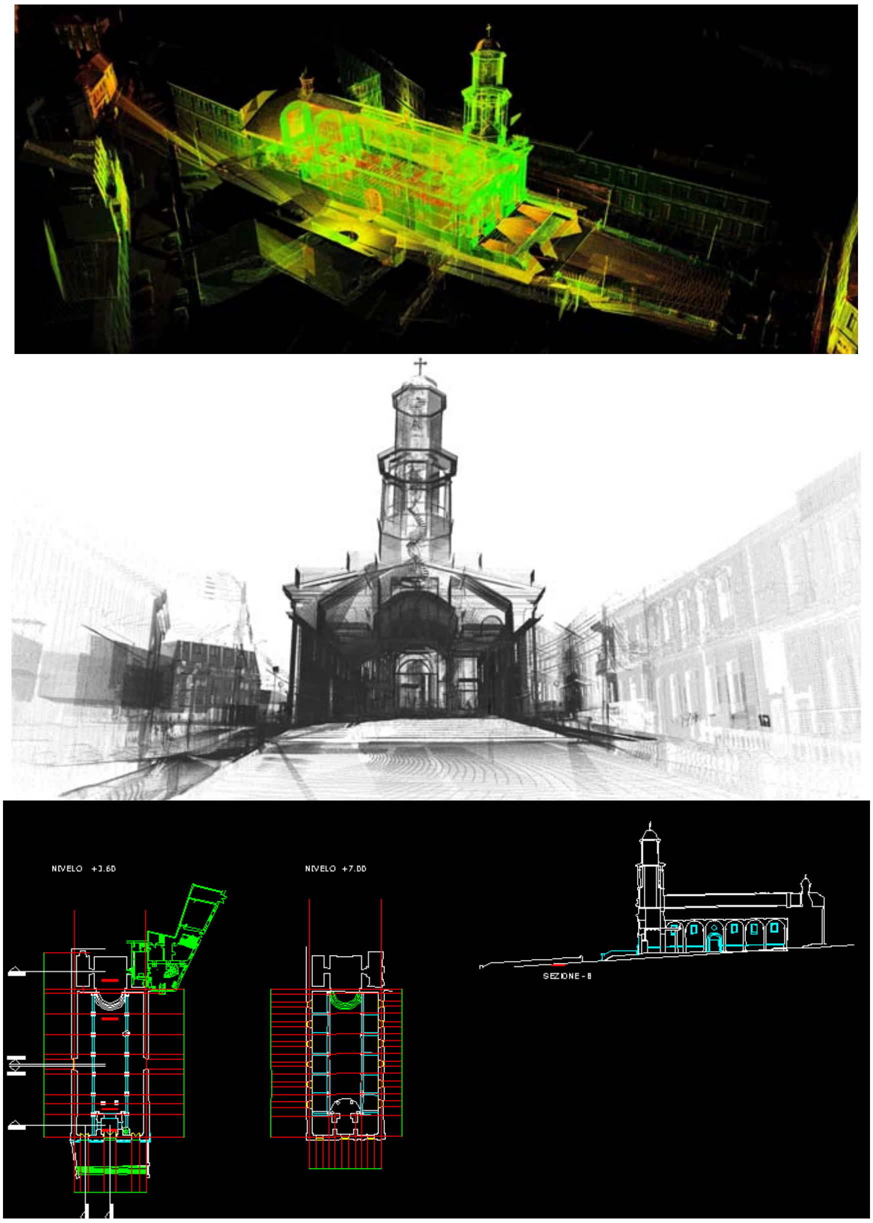


Figure 5: Laser scanner at "La Matríz" and extraction of geometric drawings.

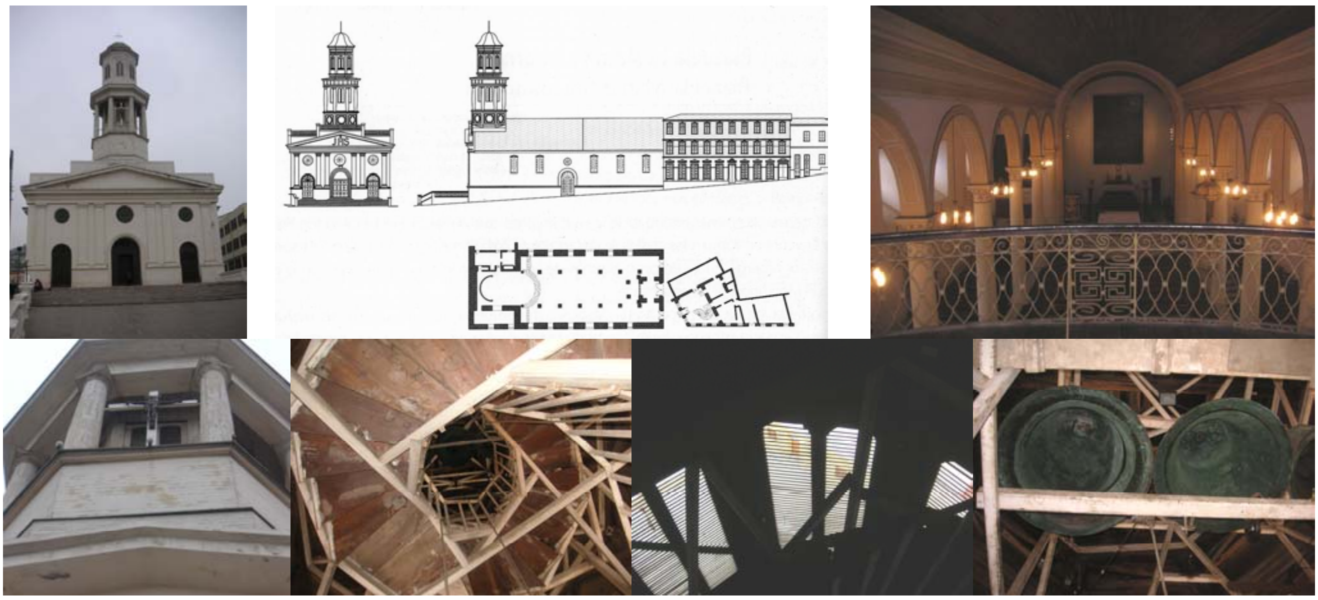


Figure 6: Pictures of the "La Matriz" Church.

Figure 7: Other pictures of the "La Matriz" Church.

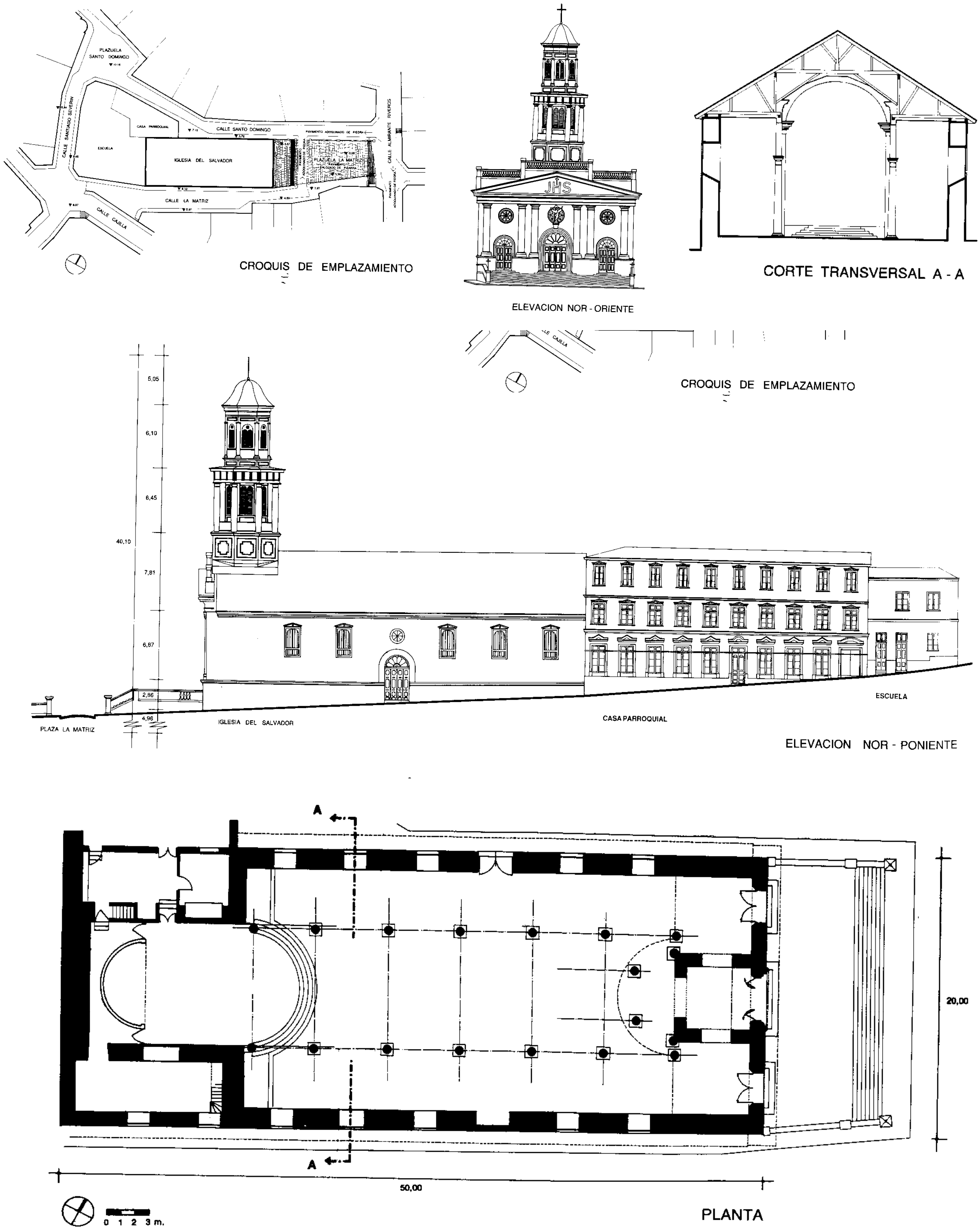


Figure 8: Drawings of the "La Matriz" Church.

8.3 Evaluation of the structural vulnerability

For the evaluation of the structural vulnerability, the active failure mechanisms have been marked in red in Fig. 9. The global damage index speaks about 8%, which is a very low value. It has been calculated as shown in the separate Appendix 1.

Failure mechanism	
1 OVERTURNING OF THE FACADE	16 OVERTURNING OF THE APSE
2 OVERTURNING OF THE GABLE	18 VAULTS OF THE APSE AND OF THE PRESBYTERY
3 SHEAR MECHANISMS IN THE FACADE	10-22 OVERTURNING OF OTHER WALLS (TRANSEPT FAÇADE, CHAPELS)
5 TRANSVERSAL VIBRATION OF NAVE	11-17-23 SHEAR FAILURE OTHER WALLS (TRANSEPT, CHAPELS, APSE, PRESBYTERY)
6-7 LONGITUDINAL VIBRATION OF THE CENTRAL AND LATERAL NAVE	19-20-21 HAMMERING AND DAMAGE IN THE ROOF COVERING
8 VAULTS OF THE CENTRAL NAVE	25 INTERACTION BETWEEN ELEMENTS OF DIFFERENT BEHAVIOUR
9 VAULTS OF THE LATERAL NAVES	26 OVERTURNING OF STANDING OUT ELEMENTS
13 KINEMATISM IN THE TRIUMPHAL ARCHES	27 GLOBAL COLLAPSE OF THE BELL TOWER
14 COLLAPSE OF THE DOME AND THE TIBURIO	28 MECHANISMS IN THE BELL CELL
Global index damage = 8%	Local index damage -

Fig. 9: Global damage index in "La Matriz" Church".

8.4 Final remark

The situation of La Matriz is enough good from the seismic point of view; for this reason numerical calculations didn't seem particularly necessary, at this preliminary investigation state. On the other hand, this building needs surely an improvement of fire protection, together with preservation measures against materials degradation and termite attack (in particular for wooden elements).

Very simple strengthening interventions can be done, as the insertion of new elements compatible with existing ones, eliminating local vulnerability of certain parts of the construction and improving the overall functionality in terms of resistance or ductility. The traditional technique can be used, as the insertion of tie-rods (placed in the two horizontal directions of the structure, at the level of floors and in correspondence to bearing walls) anchored to the masonry; in our case, horizontal tie-rods connecting façade and nave should be foreseen, in order to minimize out-of plane overturning.

9. IGLESIA SAN FRANCISCO DEL BARON

9.1 General description

The "Iglesia San Francisco del Barón" was constructed when the Franciscans moved from the "Puerto" to the Barón Hill, from 1845 to 1851 (thick adobe walls, wood colonnades, clay tiles later replaced by galvanized iron plates). Later, adjacent buildings and cloisters were added. The neo-baroque tower and façade were erected in 1890-92, thanks to the project of the architect Eduardo Provasoli (brick masonry connected by lime, without effective reinforcements). The church faced several earthquakes (mainly 1906 and 1985) without collapse, but a severe damage was found mainly in the bell-tower and the arcades during the investigation. A specific report **[08]**, dedicated to the evaluation of the seismic hazard in Valparaiso, focuses the seismic input also in the church's place. Fig. 10 shows the location of San Francisco del Baron in the Valparaiso urban tissue.

Figure 10: Location of San Francisco del Baron in the Valparaiso urban tissue.

9.2 Laser scanner, geometric, photographic and damage survey

Figs. 11-15 show pictures and drawings of the church. Thanks to the laser scanner **[10]**, geometric, photographic and damage surveys, carried out by the Italian team (Autumn 2007), a lot of information was obtained, in order to provide data useful for the following analyses.

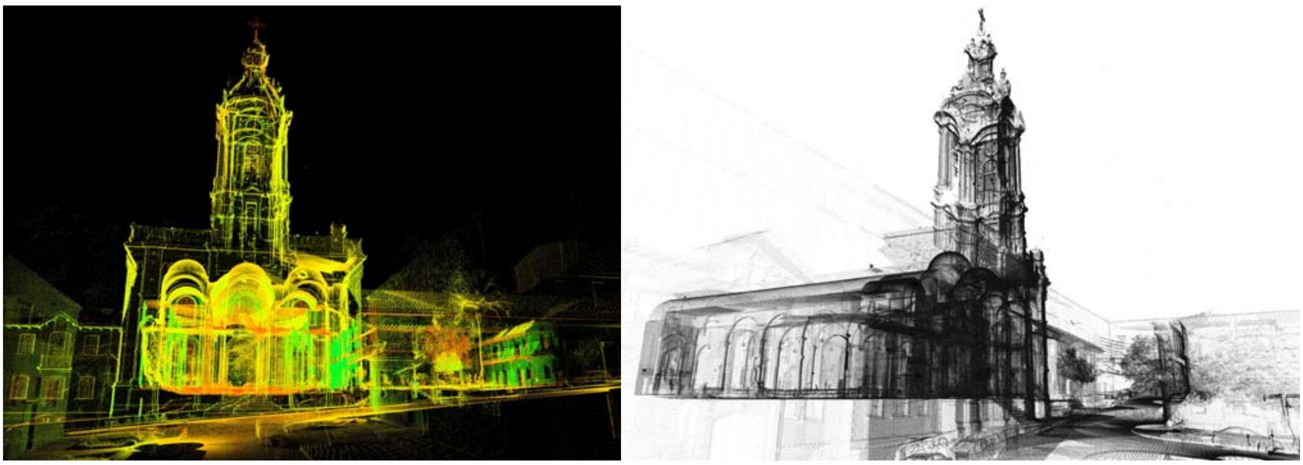
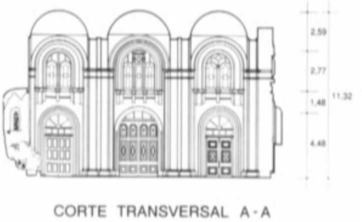


Figure 11: Laser scanner at "San Francisco".

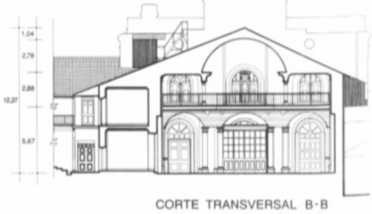
Figure 12: Pictures of "San Francisco del Barón"Church and Monastery.

Figure 13: Masonry bricks of "San Francisco del Barón"Church.

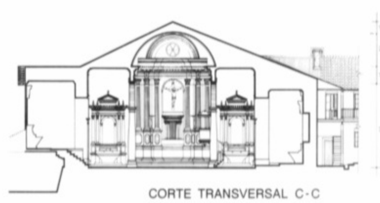
The masonry bricks (40 x 19 x 0.7 cm, Fig. 13) were measured by Arch. Claudia Zuñiga (OGP).



a) transversal section A-A



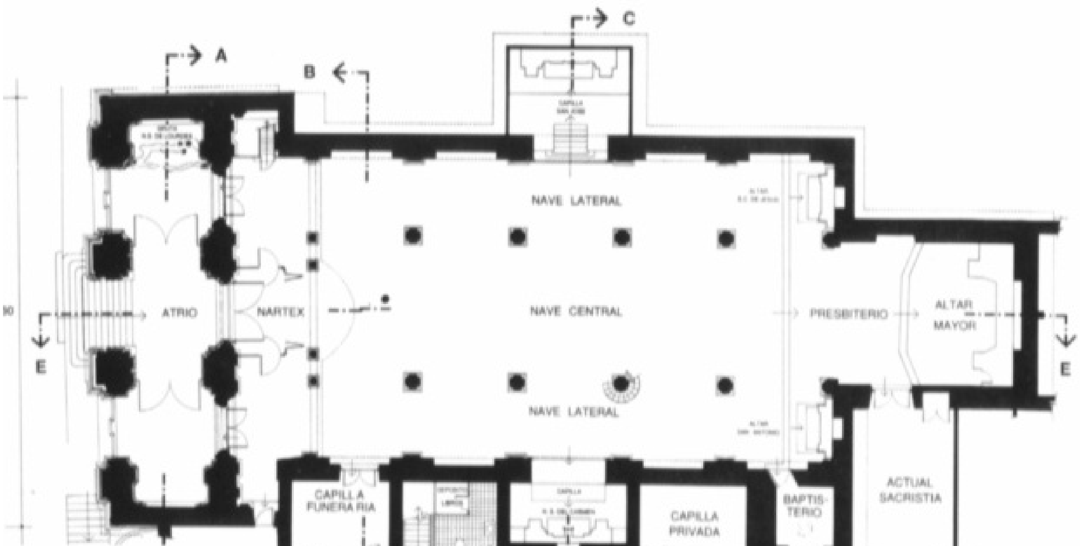
b) transversal section B-B



c) transversal section C-C

d) prospect

e) longitudinal section E-E



f) plan

Figure 14: Sections, prospect and plan of San Francisco del Baron.

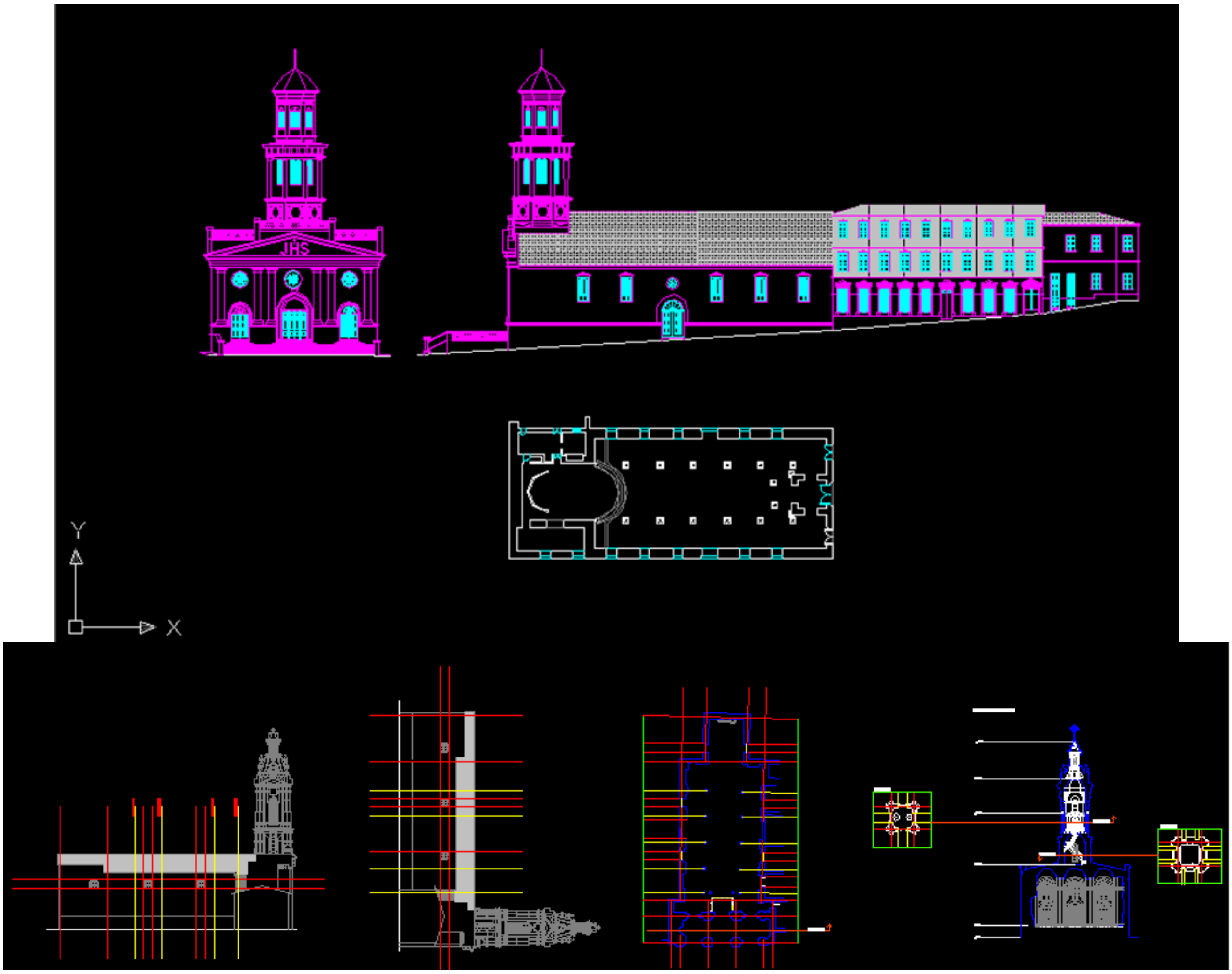


Figure 15: Output drawings from the laser scanner suvey of San Francisco del Baron.

In 1983, the church naves were burned by a fire (Fig. 16) and later reconstructed using similar techniques.

Figure 16: Fire damage in 1983 at San Francisco.

A summary of the damage assessment (thanks also to a contribution of Claudia Zuñiga, OGP), is shown in detail by Fig. 17. The most worrying situation is clearly in the façade and the bell-tower.

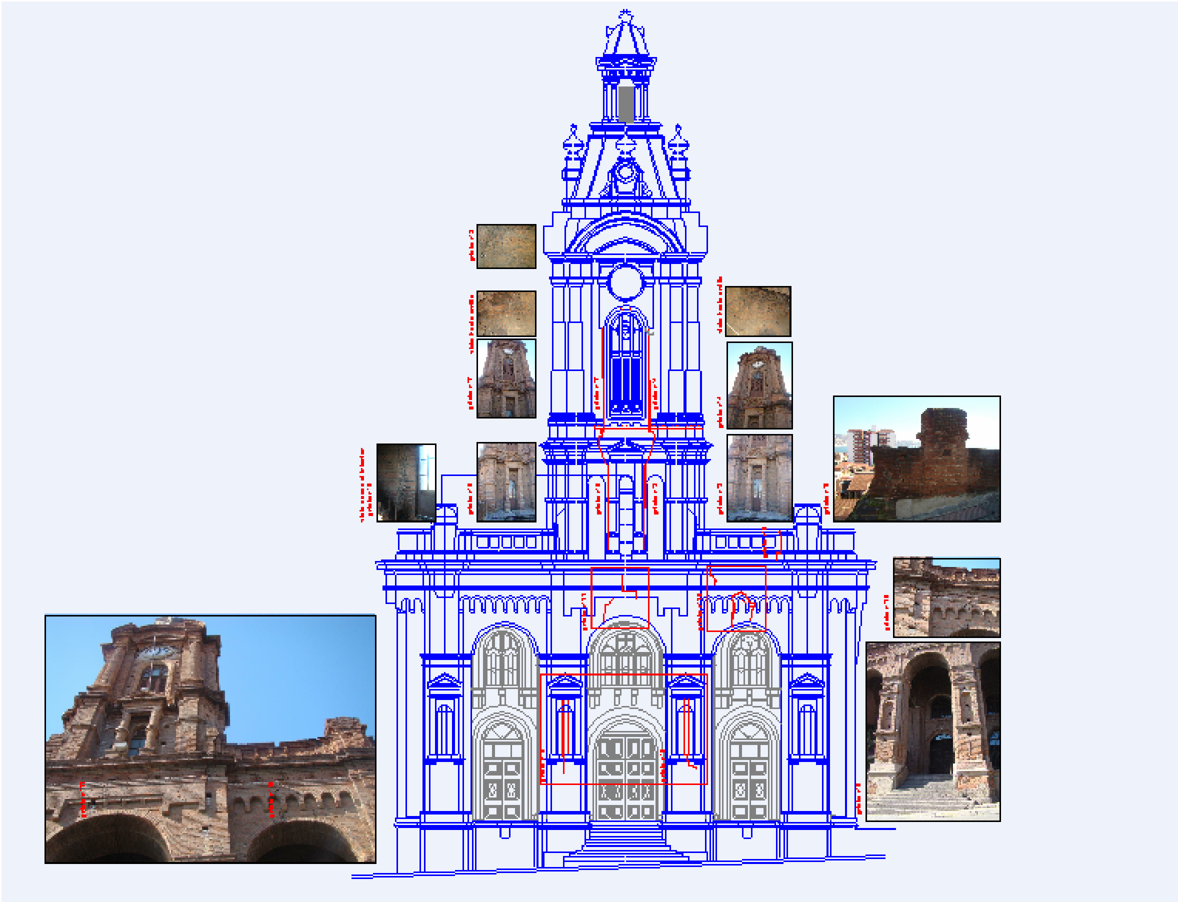


Figure 17:Summary of damage assessment at San Francisco, made by Arch. Claudia Zuñiga (OGP).

Damage in façade.

The damage assessment is shown by Fig. 18. Pictures *Fa* and *Fb* are overall views of the church façade. Pictures from *Fc* to *Ff* highlight the damage present in the vault key of the lateral nave arch (hinge and shift); the crack runs upwards along almost all the façade. Same damage is evident in the vault key of the central nave (pictures from *Fg* to *Fj*). Pictures *Fk* and *Fl* show vertical cracks in a bearing column. Damage in the battlements at the façade top is reported by *Fm* and *Fn* (diagonal cracks and material detachment).

Damage in the bell-tower

The damage assessment is shown by Fig. 19. Pictures *BTa* and *BTb* are overall views of the bell-tower. Pictures from *BTc* to *BTh* highlight very worrying vertical cracks starting from the bell-tower basis, crossing all the masonry width, as shown by *BTi* and *BTj*.

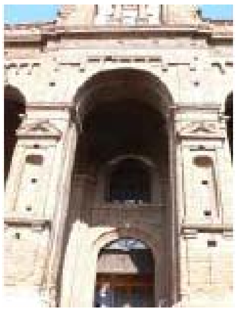
Damage in the internaldomes

The damage assessment is shown by Fig. 20. Pictures *Da* and *Db* show the entire central dome, while pictures from *Dc* to *Dh* summarize the worrying crack status along the whole dome meridian; being not possible to check if this crack is passing through the masonry thickness, anyway its depth measures at least a couple of cm (*Dg*).

Damage in the lateral prospect

The damage assessment is shown by Fig. 21. Pictures *LPa* and *LPb* give an overall view, while *LPc* and *LPd* show the damage. Also in this case, a vertical crack along the wall upper part is evident (with a façade detachment from the lateral walls), that can drive to an out-of-plane overturning mechanism in case of seismic event.

Fa)

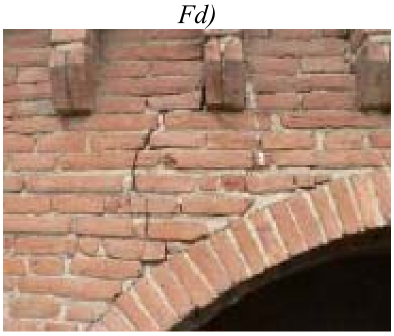


Fb)

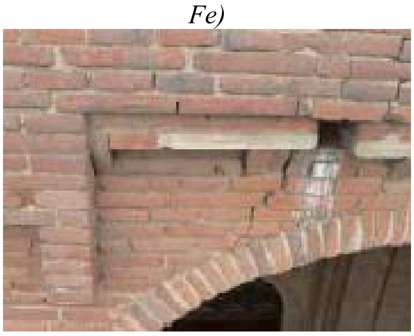
Fc)

Fd)

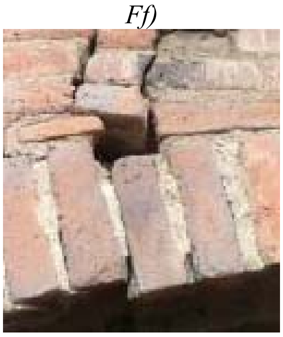
Fe)



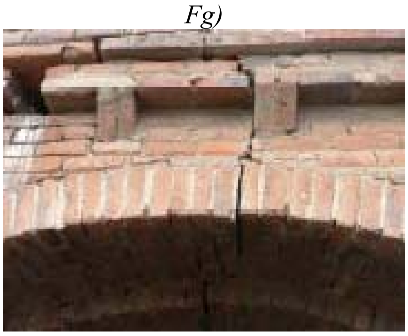
Ff)



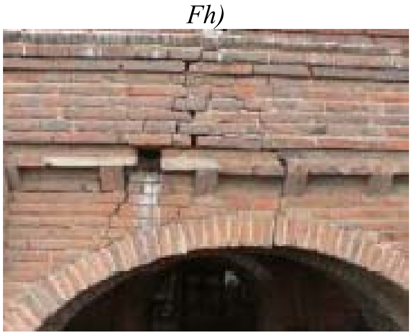
Fg)



Fh)

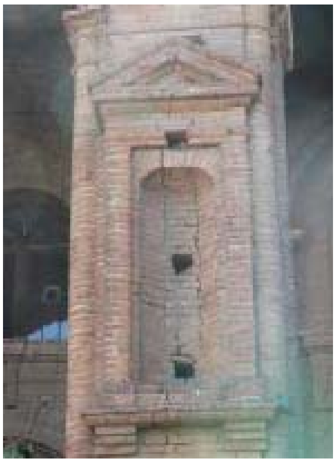


Fi)



Fj)

Fk)



Fl)

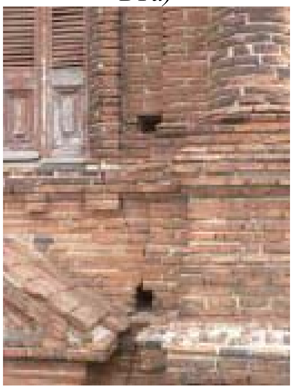
Fm)

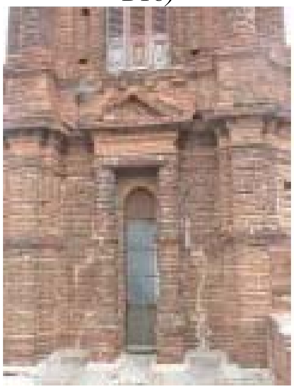
Fn)

Figure 18: Damage in the façade.


BTa)


BTb)

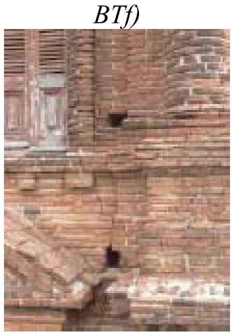

BTc)


BTd)


BTe)


BTf)


BTg)


BTh)


BTi)


BTj)

Figure 19: Damage in the bell-tower.

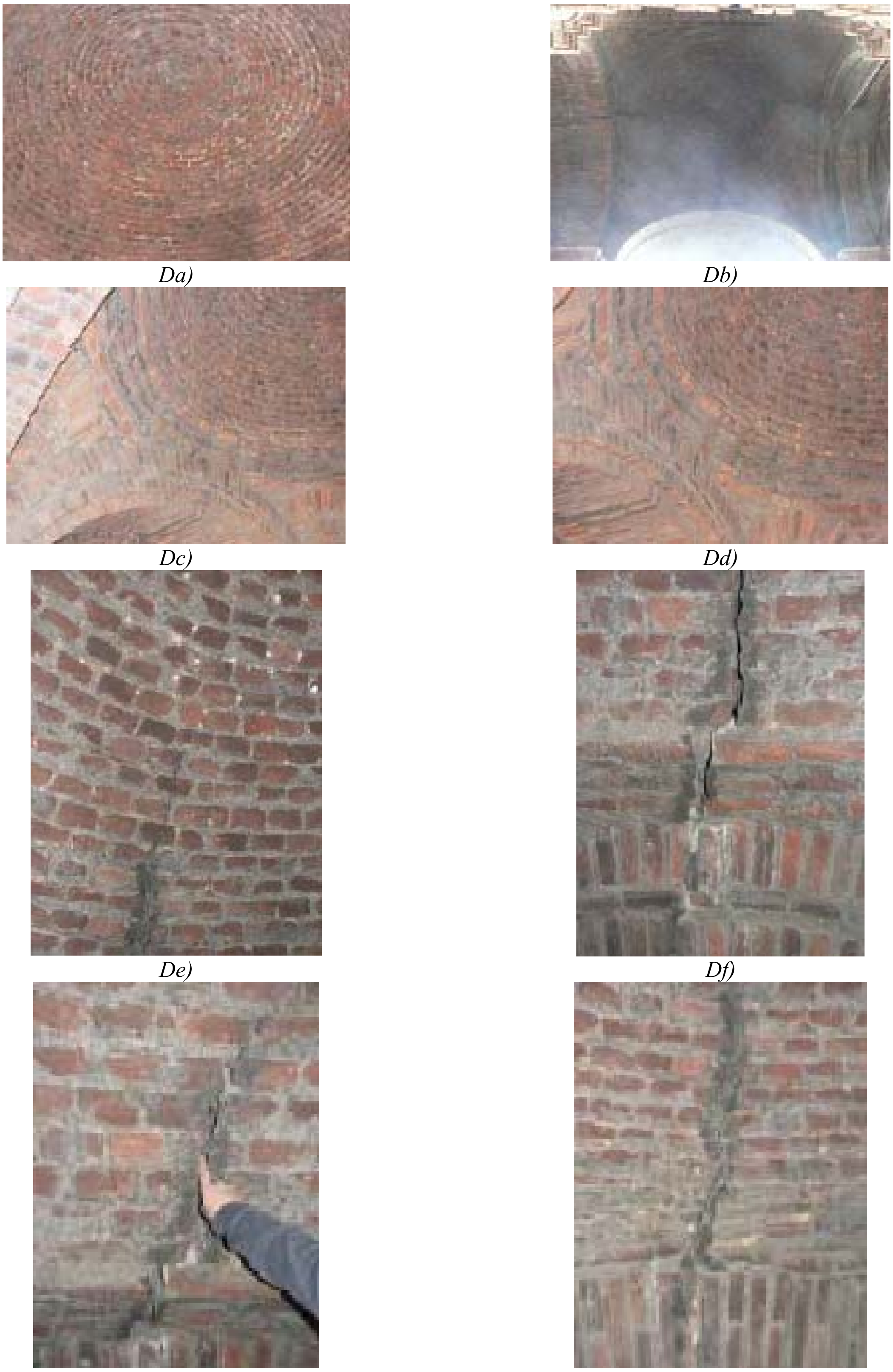
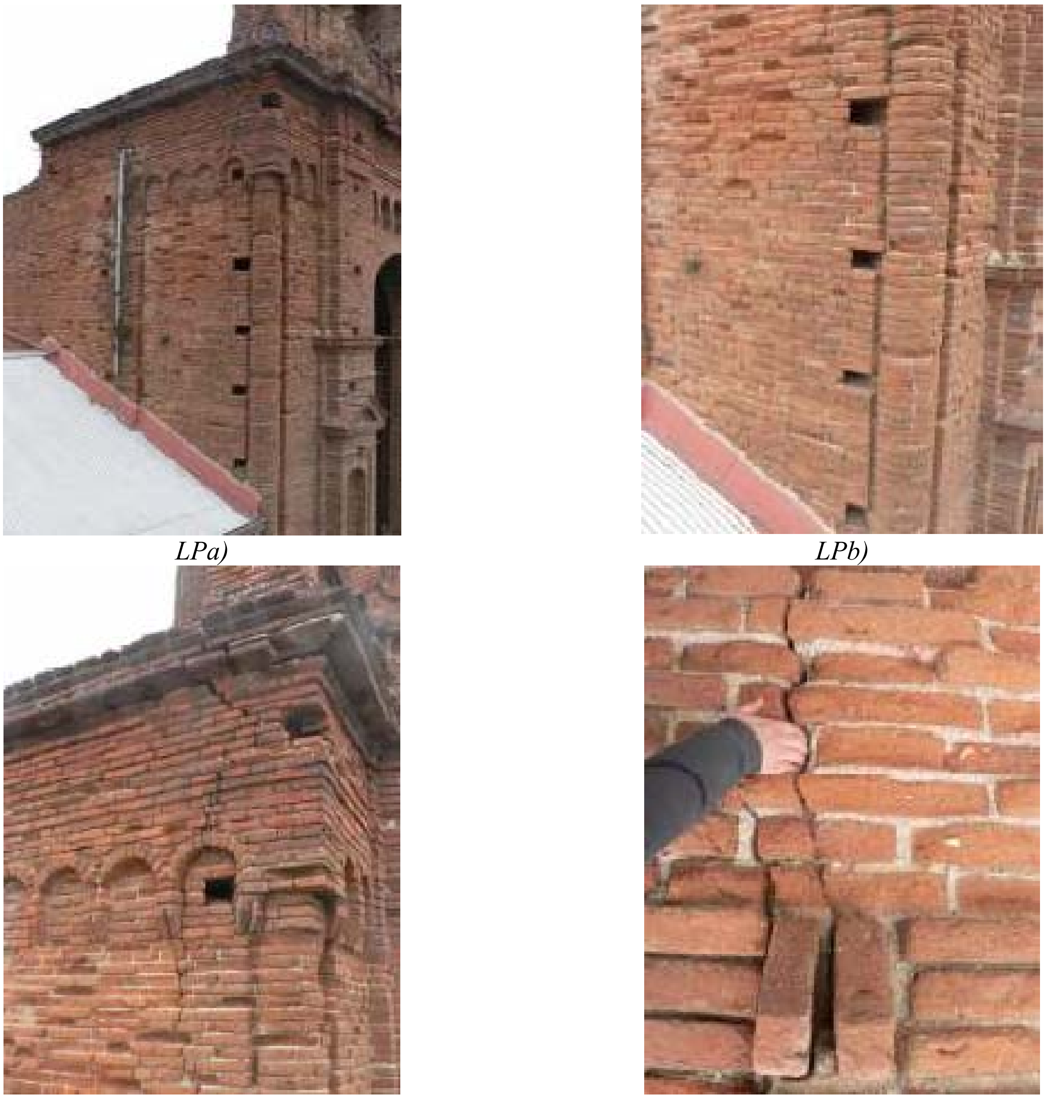


Figure 20: Damage in the internaldomes.



LPa) LPb)

LPc) LPd)

Figure 21: Damage in the lateral prospect.

9.3 Evaluation of the structural vulnerability

For the evaluation of seismic damage, the active failure mechanisms have been marked in red in Fig. 22. The global damage index speaks about 33%, but the local damage index in the façade (66%) is very high. It has been calculated as shown in the separate Appendix 2.

Failure mechanism	
1 OVERTURNING OF THE FACADE	16 OVERTURNING OF THE APSE
2 OVERTURNING OF THE GABLE	18 VAULTS OF THE APSE AND OF THE PRESBYTERY
3 SHEAR MECHANISMS IN THE FACADE	10-22 OVERTURNING OF OTHER WALLS (TRANSEPT FAÇADE, CHAPELS)
5 TRANSVERSAL VIBRATION OF NAVE	11-17-23 SHEAR FAILURE OTHER WALLS (TRANSEPT, CHAPELS, APSE, PRESBYTERY)
6-7 LONGITUDINAL VIBRATION OF THE CENTRAL AND LATERAL NAVE	19-20-21 HAMMERING AND DAMAGE IN THE ROOF COVERING
8 VAULTS OF THE CENTRAL NAVE	25 INTERACTION BETWEEN ELEMENTS OF DIFFERENT BEHAVIOUR
9 VAULTS OF THE LATERAL NAVES	26 OVERTURNING OF STANDING OUT ELEMENTS
13 KINEMATISM IN THE TRIUMPHAL ARCHES	27 GLOBAL COLLAPSE OF THE BELL TOWER
14 COLLAPSE OF THE DOME AND THE TIBURIO	28 MECHANISMS IN THE BELL CELL
Global index damage = 33%	Local index damage on the facade = 66%

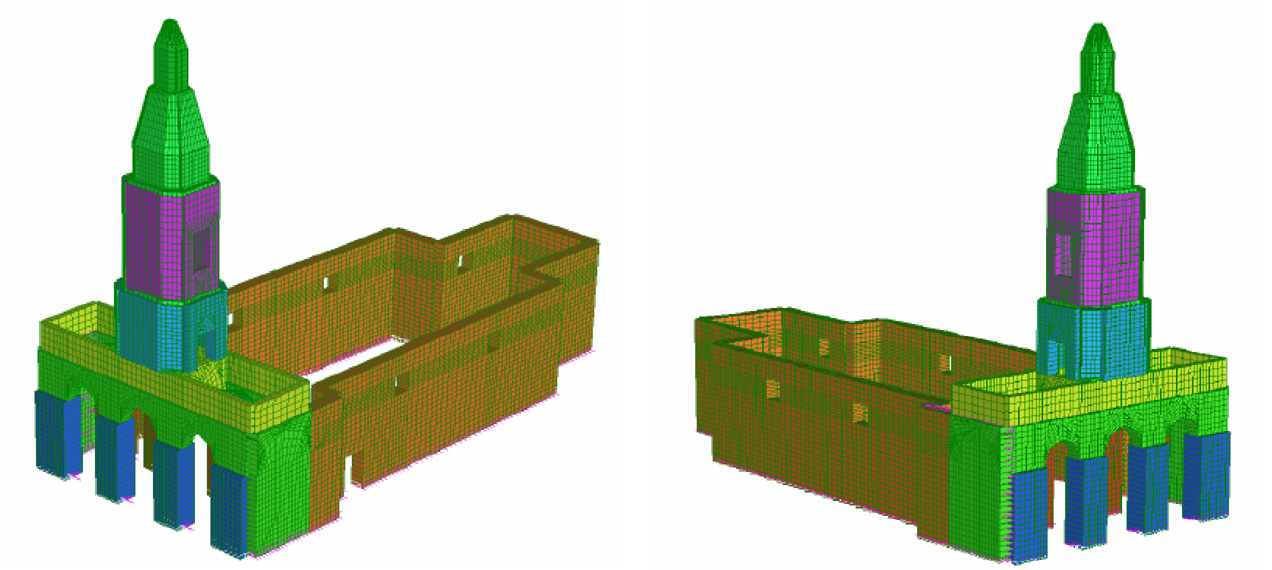
Figure 22: "San Francisco del Barón" Church and Monastery: Damage Index

9.4 Results of the numerical analyses

The FEM model principal features are shown by Fig. 23 and Table 2. Modal analysis by response spectrum method has been used. About the brick masonry, lacking experimental tests, the following data have been considered:

- low resistance to compression ($f_k = 2$ N/mm^2);
- Young Modulus E equal to 1000 x f_k, or E = 2000 MPa;
- Tangential Elasticity Modulus equal to 0.4 x E.

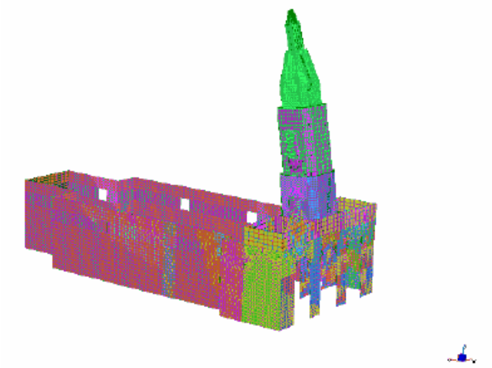
For each principal horizontal direction taken into account, vibration modes with a significant contribution to the dynamic response (Fig. 24) have been considered (mass participation factors > 1%; antiseismic codes are usually less conservative: > 5%). In order to calculate displacements and actions on the structure, thanks to the modal combination method, the Complete Quadratic Combination (CQC) has been utilized.
Seismit input has been chosen as reported in paragraph 6.



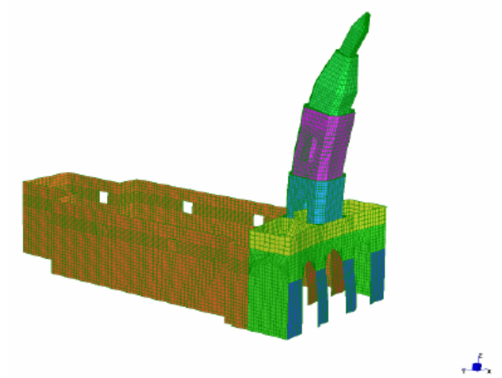
a) *b)*

Figure 23: Visualization of the FEM model of the San Francisco Church.

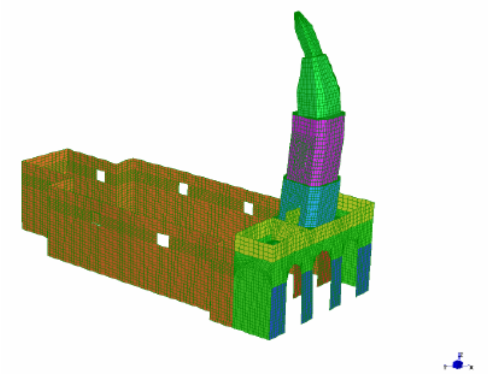
Table 2: Features of the FEM model
16813 nodes
----- beams
16440 plates
----- bricks
----- links
Number of equations: 98620
Number of modes: 20
Number of iterations: 40
Iteration tolerance: 10-5



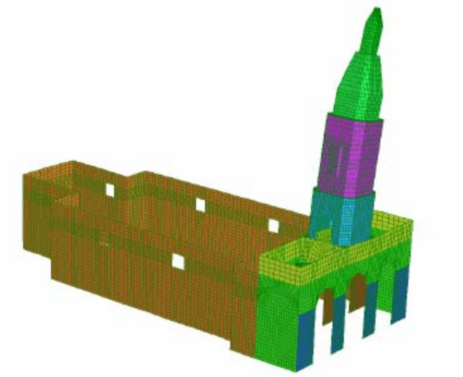
a) Frequency: 2.059 Hz; Mass in Y direction: 12.41%



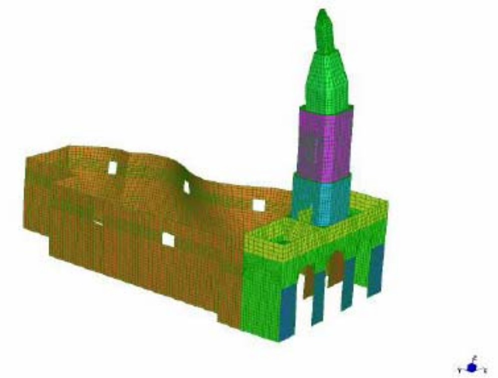
b) Frequency: 4.500 Hz; Mass in Y direction: 11.88 %



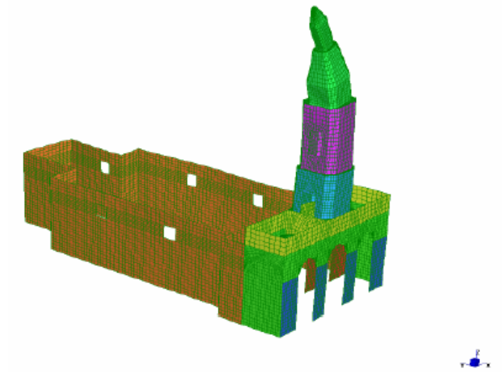
c) Frequency: 7.630 Hz; Mass in Y direction: 2%



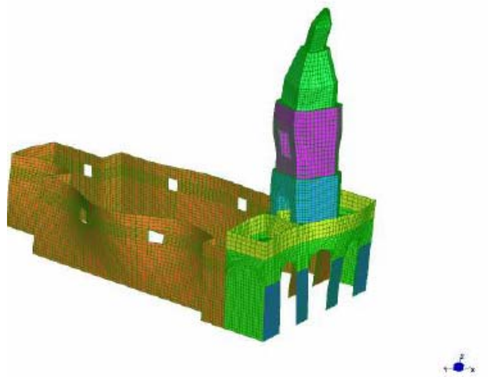
d) Frequency: 2.700 Hz; Mass in X direction: 6.82 %



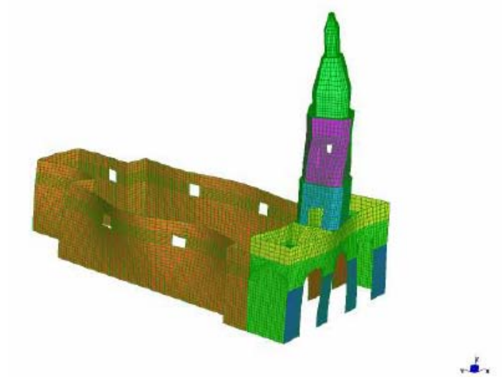
e) Frequency: 6.910 Hz; Mass in X direction: 11.89%



f) Frequency: 7.370 Hz; Mass in X direction: 4%



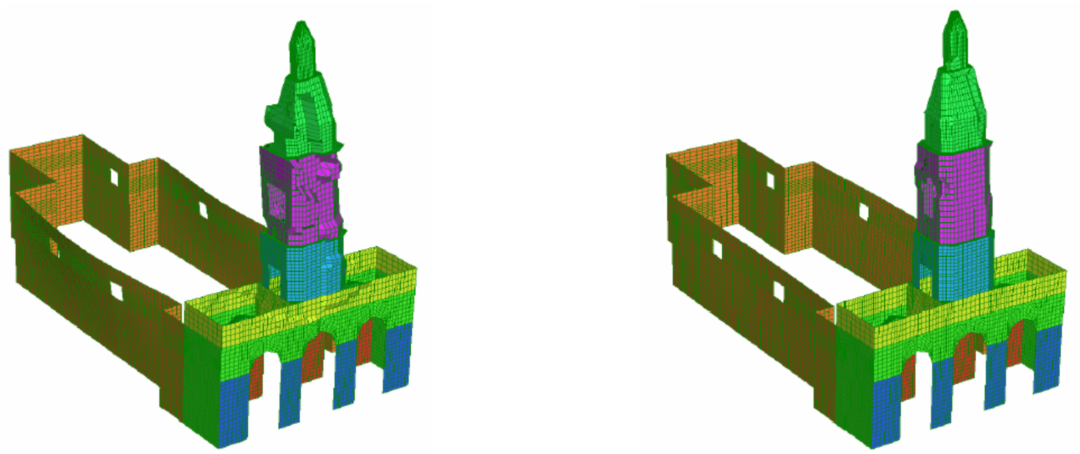
g) Frequency: 7.750 Hz; Torsional Mode



g) Frequency: 7.800 Hz; Torsional Mode

Figure 24: Vibration modes with a significant contribution to the dynamic response.

Following the Eurocode 8 **[18]**, the seismic response can be evaluated separately for each of its components, as written below:
AEx "+" 0.3AEy
0.3AEx "+" AEy
where:
AEx action directed along the longitudinal direction,
AEy action directed along the transversal direction.



a) AEx "+" 0.3AEy combination *b) 0.3AEx "+" AEy combination*

Figure 25: Deformation shapes.

Taking into account only the significant mass participation factors (in our case > 1%), the deformation shapes for the two selected load combinations have been carried out (Fig. 25). It is evident that the most affected structural part is really the bell-tower (due to flexural and torsional actions), with important top displacements, worsened by the damage found during the investigation.

9.5 Final remarks

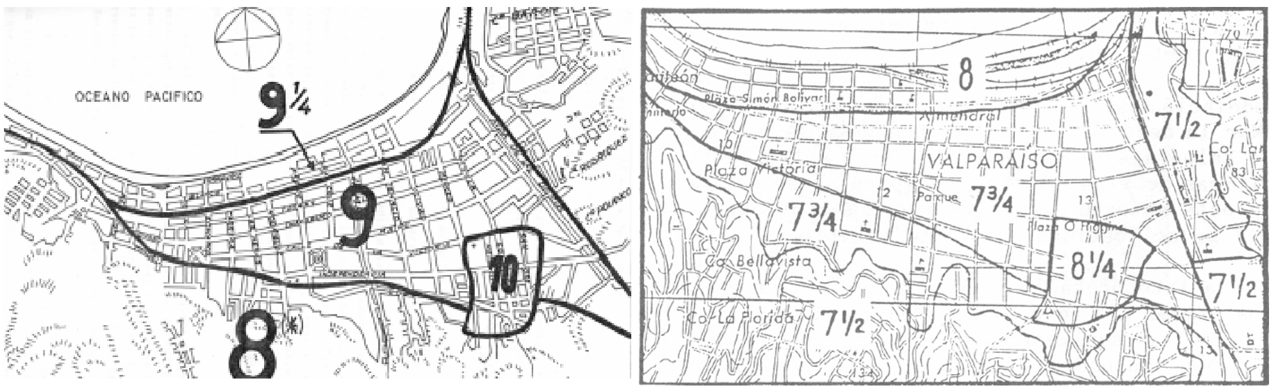
The present damage situation of the San Francisco Church must be considered very worrying, because partial or total collapse (especially in the bell-tower and in the façade) can occur in case of earthquake (i.e. medium to high magnitude seismic excitations, as expected in the Valparaiso area); in fact, the church is unsafe and urgently must be closed partially of totally, planning a strengthening intervention as soon as possible.
The construction seems to be (in the façade and in the bell-tower) a very regular masonry brickwork, but diagnostics testing is strongly recommended. The building shows heavy widespread structural damage and lack of effective antiseismic protections.
The main intervention steps can be foreseen as follows:
- reinforcement of part or all the resistant elements, increasing selectively resistance, stiffness, ductility or a combination of these (always paying careful attention to induced modifications to the structural scheme); it can be done: increasing the strength of masonry, trough local repairs to cracked or deteriorated parts; reconstructing masonry unity in the most weak or deteriorated parts, utilizing materials with analogous physical-chemical and mechanical properties; common non-invasive techniques used in Italy are *rip and sew*, *injections of mixed bonding agents*, *redrafting the junctions*; the insertion of post-tightened vertical tie-rods is applicable only in specific cases and when the masonry has been proven to be able to support the increase in vertical load;
- insertion of new elements which are compatible with existing ones, eliminating local vulnerability of certain parts of the construction and improving the overall functionality in terms of resistance or ductility; it can be done mainly through the traditional techniques, as the insertion of tie-rods (placed in the two horizontal directions of the structure, at the level of floors and in correspondence to bearing walls) anchored to the masonry; arches and vaults can be strengthened also using tie-rods (normally placed at the rear), put in place with adequate pre-solicitation; other methods (jaketing by concrete or strips of composite materials) should be evaluated with care.

10. CAPILLA DE LAS HERMANAS DE LA PROVIDENCIA

10.1 General description

The congregation of "Las Hermanas de la Providencia", originated in Canada in 1844, extended its work in Chile in 1853 in Valparaiso in 1858. The congregation, established initially to Puerto, built after 1867, a chapel, which undergoes various modifications until the fire of 1880.
Then, a second version was erected on the Merced Hill (1880-1883), but collapsed almost completely due to the 1906 earthquake and later demolished.
The present building (designed by the architect Victor Auclair in a neo-renaissance style but made by a rare primitive reinforced concrete) began in 1907.
Las Hermanas Chapel is located in the Almendral at the Merced foothill (Fig. 26), exactly where the 1906 and 1985 earthquake Intensities reached the maximum value (Fig. 27).

Figure 26: Location of the Las Hermanas de la Providencia Chapel in the Valparaiso urban tissue.



1906 earthquake *1985 earthquake*

Figure 27: Earthquake Intensities in the 1906 and 1985 seismic events.

Figs. 28-29 show structural and damage details of the church. The church was severely damaged by the 1985 seismic event: because the church still presents the cracks caused by the 1985 earthquake without any rehabilitation, it is now declared unsafe and closed to the public. A specific report **[08]**, dedicated to the evaluation of the seismic hazard in Valparaiso, focuses the seismic input also in the church's place.

10.2 Laser scanner, geometric, photographic and damage survey

The geometrical and damage survey has been carried out during the mission performed in Valparaiso by the Italian team (Autumn 2007). The output elaboration of the laser scanner survey (see Fig. 30 and also the specific report **[10]**) provided exhaustive internal and external details of all the geometric, architectonic, decorative, and structural aspects: the plan, front and section drawings (Fig. 31) of the building are in fact indispensable data for the preparation of the structural models.

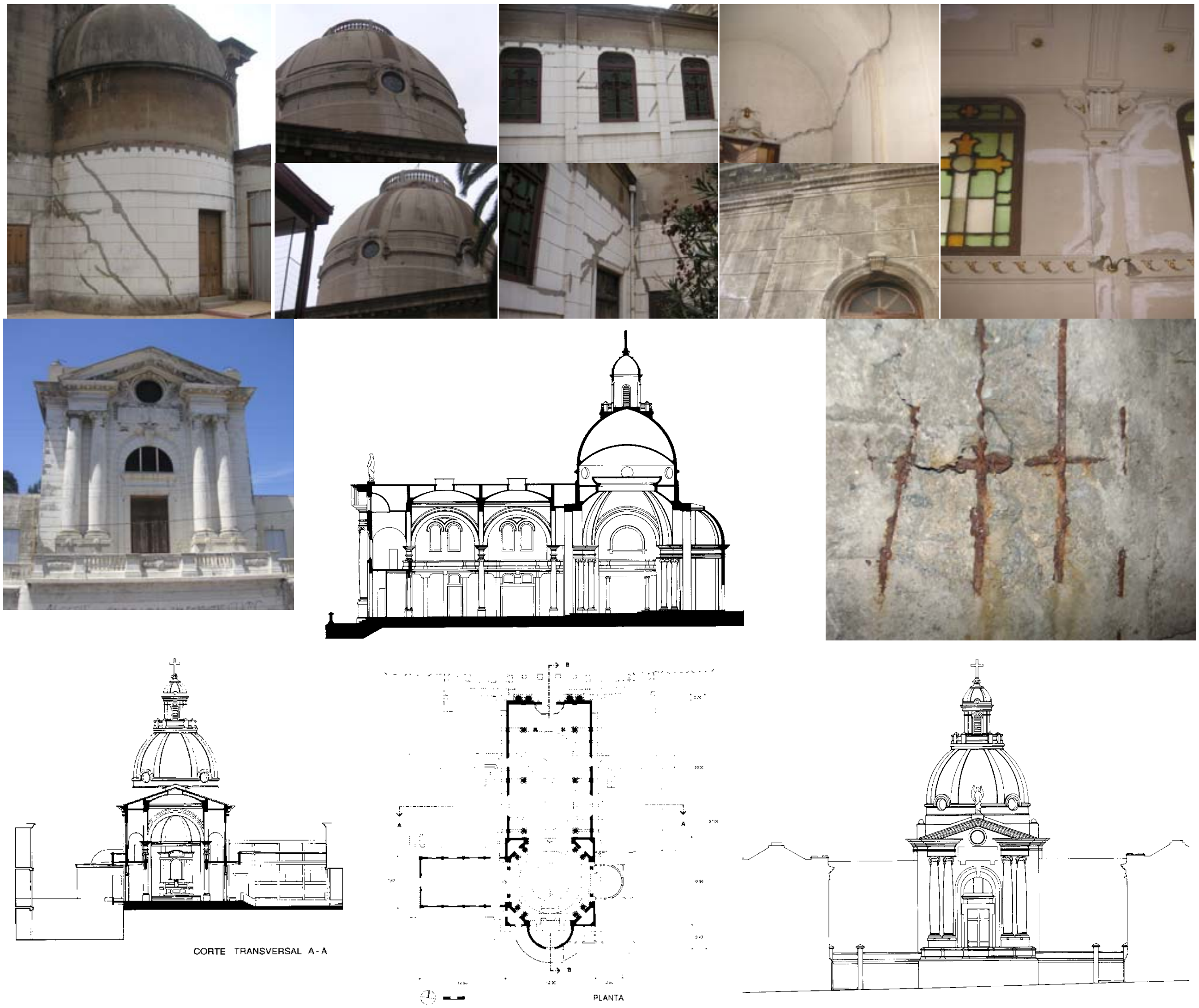


Figure 28: Same pictures and drawings of the Las Hermanas Chapel.

Figure 29: Some pictures of the interior of the Las Hermanas Chapel.

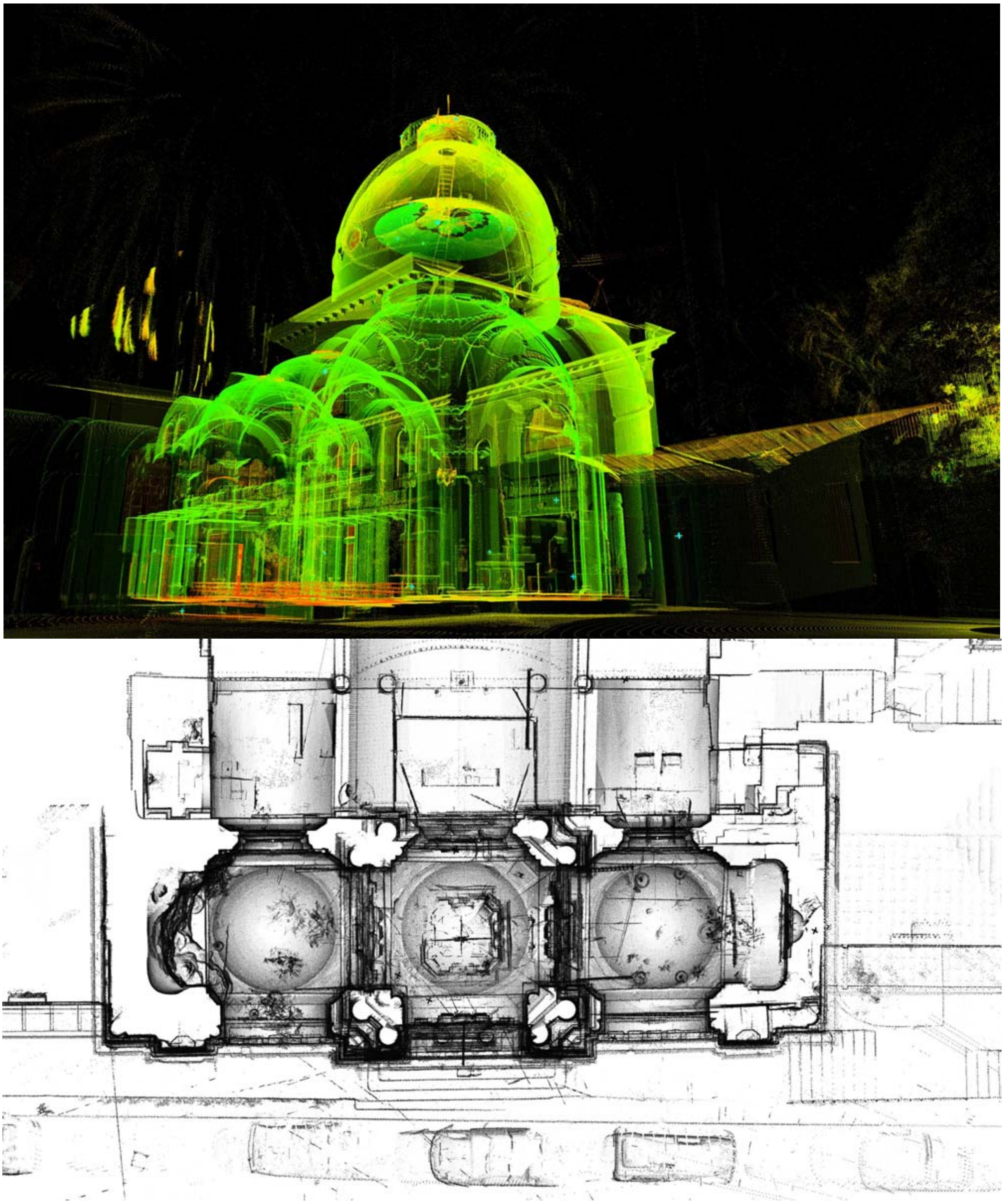


Figure 30: laser scanner at "Las Hermanas de la Providencia".

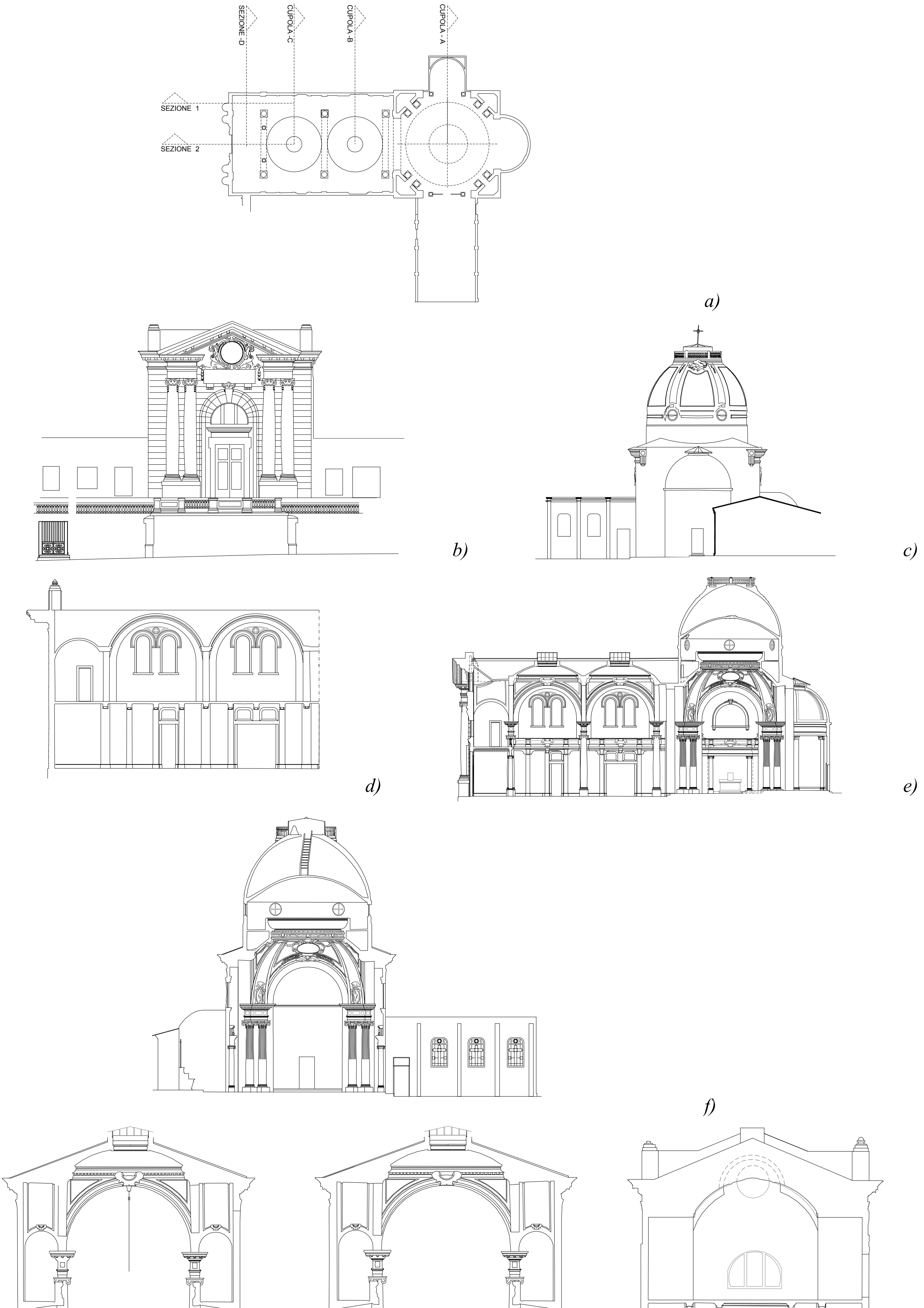


Figure 31: Plan, front and section drawings of the Las Hermanas de la Providencia Church: a) plan; b) North front; c) South front; d) section 1; e) section 2; f) section A; g) section B; h) section C; i) section D.

A photographical survey (also carried out during the mission), with a short description of the main features and damages of the church, is shown in Fig. 32.

View of the main façade

View of the West side of the Church

View of the main dome of the church: some diagonal cracks are visible near the round openings, probably due to torsional effects

View of the South apse: the diagonal cracks at the basis of the walls are probably related to overturning mechanisms associated to torsional effects

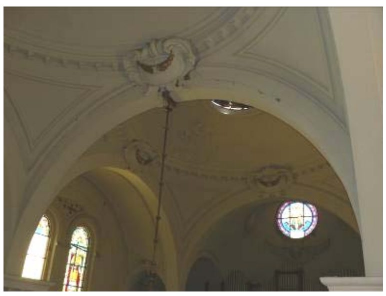
View of the West apse (chapel): some diagonal shear cracks are well visible in the lateral walls, between the openings

View of the East side of the church

View of the roof covering

View of the main nave and of the area of the presbytery

View of one of the two domes of the main nave: no particular cracks are visible on these structural elements



View of the triumphal arch between the two domes of the main nave: some longitudinal and radial cracks, related to an in-plane seismic action, are present

View of the lateral walls of the main nave: some diagonal cracks are present, starting from the openings

View of one of the lateral arches of the main nave: some radial cracks are present

View of the triumphal arch between the main nave and the presbytery: some longitudinal and radial cracks, related to an in-plane seismic action, are present

View of the triumphal arch between the presbytery and the South apse: some longitudinal and radial cracks, related to an in-plane seismic action, are present

View of the East apse: some diagonal shear cracks are present, starting from the openings area

View of the presbytery's columns that support the main dome: no particular cracks are visible on these structural elements

View of the West apse: some diagonal shear cracks are present, starting from the openings area

Internal view of the West apse: some cracks are present even in the covering vault

Internal view of the West apse: some cracks are present even in the covering vault

View of the wall that separate the main nave and the presbytery: some diagonal cracks are present in this corner

Internal view of the main dome: the horizontal cracks are probably related to a rigid torsion of the upper part of the dome

Internal view of the main dome: the steel bars of the primitive reinforced concrete are clearly visible

View of one of the decorative elements of the main dome of the church: some reinforcement steel bars are visible

Internal view of the façade area

Internal view of the façade wall: a vertical crack is present in the lateral wall nearby the façade wall, probably related to an overturning mechanism of the façade wall

Internal view of the façade area: each column support different horizontal forces related to the different arches and vaults it support

Figure 32: Main structural characteristics and damages of the Las Hermanas de la Providencia Church.

10.3 Evaluation of the structural vulnerability

For the evaluation of seismic damage, the active failure mechanisms have been marked in red in Fig. 33. The global damage index speaks about 58%, which is a high value. It has been calculated as shown in the separate Appendix 3.

Failure mechanism	
1 OVERTURNING OF THE FACADE	16 OVERTURNING OF THE APSE
2 OVERTURNING OF THE GABLE	18 VAULTS OF THE APSE AND OF THE PRESBYTERY
3 SHEAR MECHANISMS IN THE FACADE	10-22 OVERTURNING OF OTHER WALLS (TRANSEPT FAÇADE, CHAPELS)
5 TRANSVERSAL VIBRATION OF NAVE	11-17-23 SHEAR FAILURE OTHER WALLS (TRANSEPT, CHAPELS, APSE, PRESBYTERY)
6-7 LONGITUDINAL VIBRATION OF THE CENTRAL AND LATERAL NAVE	19-20-21 HAMMERING AND DAMAGE IN THE ROOF COVERING
8 VAULTS OF THE CENTRAL NAVE	25 INTERACTION BETWEEN ELEMENTS OF DIFFERENT BEHAVIOUR
9 VAULTS OF THE LATERAL NAVES	26 OVERTURNING OF STANDING OUT ELEMENTS
13 KINEMATISM IN THE TRIUMPHAL ARCHES	27 GLOBAL COLLAPSE OF THE BELL TOWER
14 COLLAPSE OF THE DOME AND THE TIBURIO	28 MECHANISMS IN THE BELL CELL
Global index damage = 58%	Local index damage -

Figure 33: Active failure mechanism in the Hermanas Church.

10.4 Results of the numerical analyses

The numerical simulation of Las Hermanas de la Providencia church considered a linear elastic Finite Element Model representing the global church's structure, since the overall dynamic response can not conveniently be evaluated by considering *partial* models. It is worth noticing that all of the numerical models implemented lie on the concept of *macro-modelling*.

The performed analysis (Spectral Response Analysis) have the aim of calculating the maximum element stresses due to the gravity load and the earthquake loads.

The structural elements in the model are defined by plate/shell linear bi-dimensional elements, implementing both membrane and bending behaviour, as defined within the software code used (Straus7, ©2003 G+D Computing Pty Ltd): only the columns are modelled using beam elements. Model geometry was defined with a compromise between accuracy and efficiency, considered the complexity of the three dimensional structure of the church; roofs were not modelled (Fig. 34). All structural members are made of the same material (concrete).

Once defined the frame geometry and the section and material properties, the static analysis of the gravity load can be performed. Some views of the results in terms of maximum displacements (Fig. 35) and element stresses (Fig. 36) follow. The deformed shape of the model and the magnitude of the displacements and of the stresses suggest that the material properties and loads are correct and that no errors have been made with elements: the two central columns of the main nave present a high compressive-bending stress.

An earthquake excitation of the ground can be given in the form of a time history of the ground acceleration, or in the form of a Spectrum. The spectral response approach is more common, and it is utilised by almost every modern design code. The horizontal axis represents the frequency content of the earthquake and the excitation is applied according to the natural period of the structure (the units for the horizontal axis are seconds).

It is important to distinguish between Response Spectrum curve and Spectral curve: both are of similar shape and both have the same units, but the meanings are different. The Response Spectrum curve is used in design and represents the maximum response of a single degree of freedom system, defined with its natural period and its damping: for different damping there will be different response curves This type of curve is used in the design codes: the values of the Response Spectrum curves are usually normalised using gravitational acceleration and is given in g's (gravitational acceleration): there is a scaling factor or multiplier, which multiplies the values on the vertical axis of the curve. After scaling, these values usually have the dimension of acceleration (i.e. m/s^2). The other type, known as Spectral curve, is a Fourier Transform of time process. When the time process is an acceleration record of an earthquake, the Spectral curve is only a mathematical transformation of this data from the time domain into the frequency domain: this type of Spectral curve provides information about the frequency content of the time process.

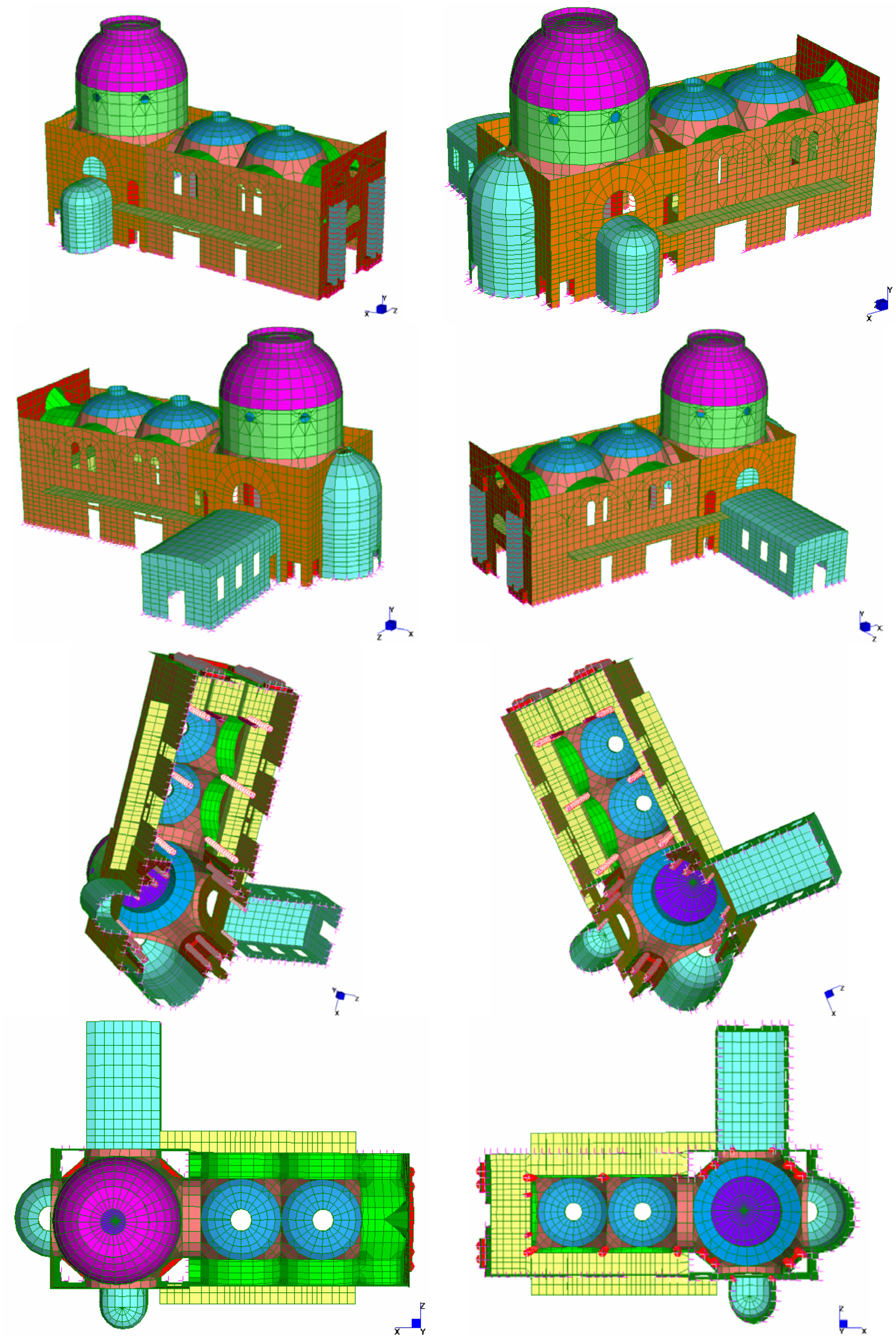
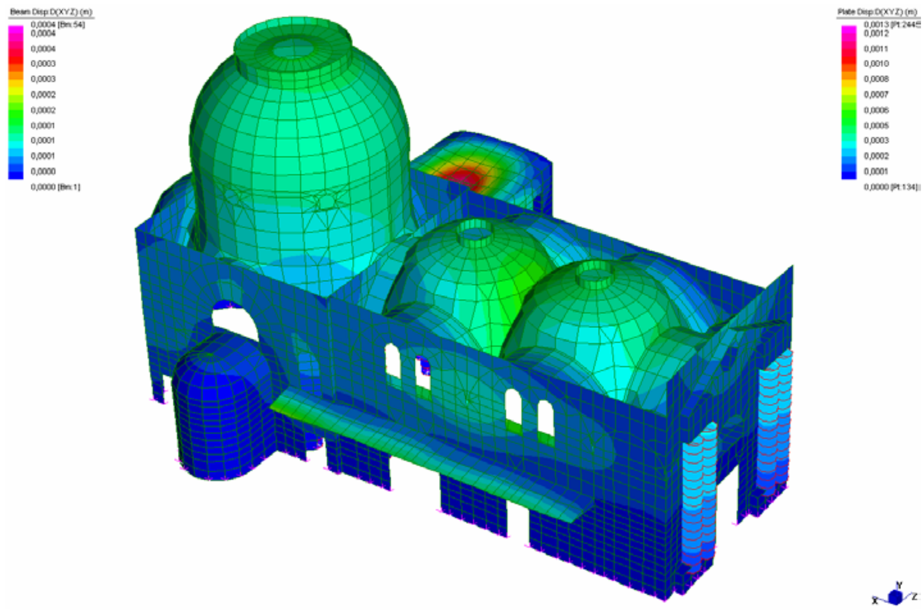
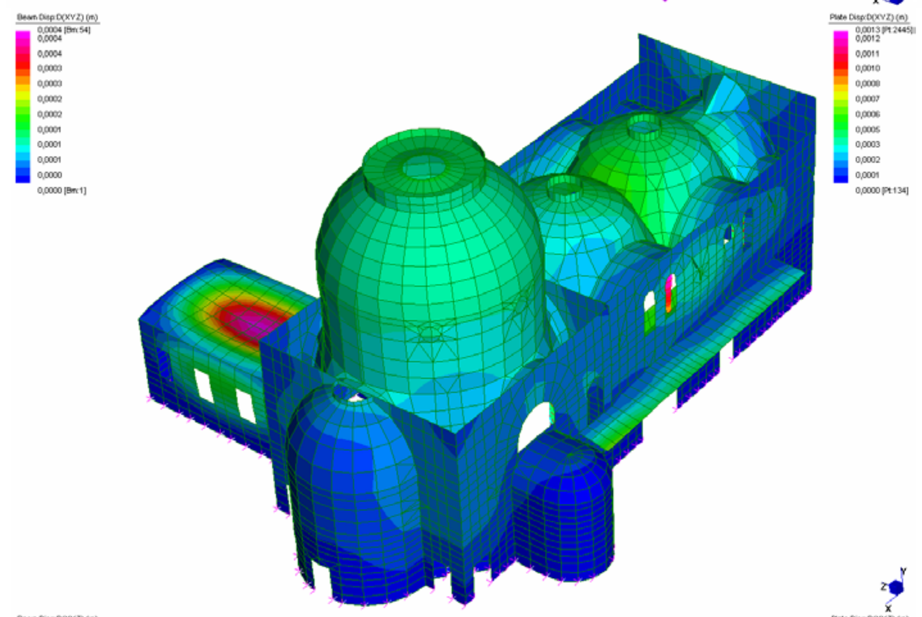
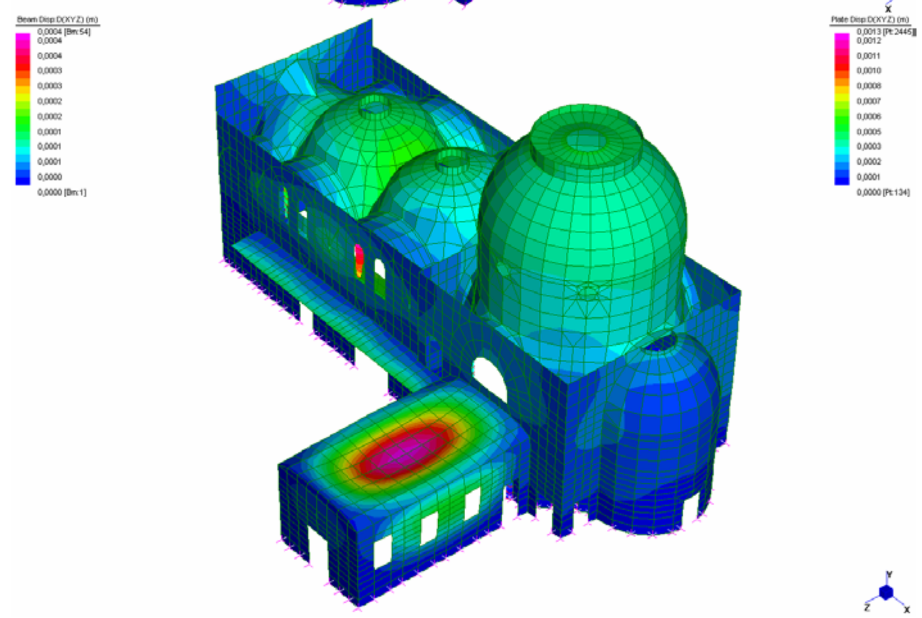


Figure 34: View of the FEM model of the Las Hermanas de la Providencia Church.


Beam Disp:D(XYZ) (m)
0,0004 [Bm:54]
0,0004
0,0004
0,0003
0,0003
0,0002
0,0002
0,0001
0,0001
0,0001
0,0000
0,0000 [Bm:1]
Plate Disp:D(XYZ) (m)
0,0013
0,0012
0,0011
0,0010
0,0008
0,0007
0,0006
0,0005
0,0003
0,0002
0,0001
0,0000 [Pt:134]


Beam Disp:D(XYZ) (m)
0,0004 [Bm:54]
0,0004
0,0004
0,0003
0,0003
0,0002
0,0002
0,0001
0,0001
0,0001
0,0000
0,0000 [Bm:1]
Plate Disp:D(XYZ) (m)
0,0013 [Pt:2445]
0,0012
0,0011
0,0010
0,0008
0,0007
0,0006
0,0005
0,0003
0,0002
0,0001
0,0000 [Pt:134]


Beam Disp:D(XYZ) (m)
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0,0004
0,0004
0,0003
0,0003
0,0002
0,0002
0,0001
0,0001
0,0001
0,0000
0,0000 [Bm:1]
Plate Disp:D(XYZ) (m)
0,0013 [Pt:2445]
0,0012
0,0011
0,0010
0,0008
0,0007
0,0006
0,0005
0,0003
0,0002
0,0001
0,0000 [Pt:134]

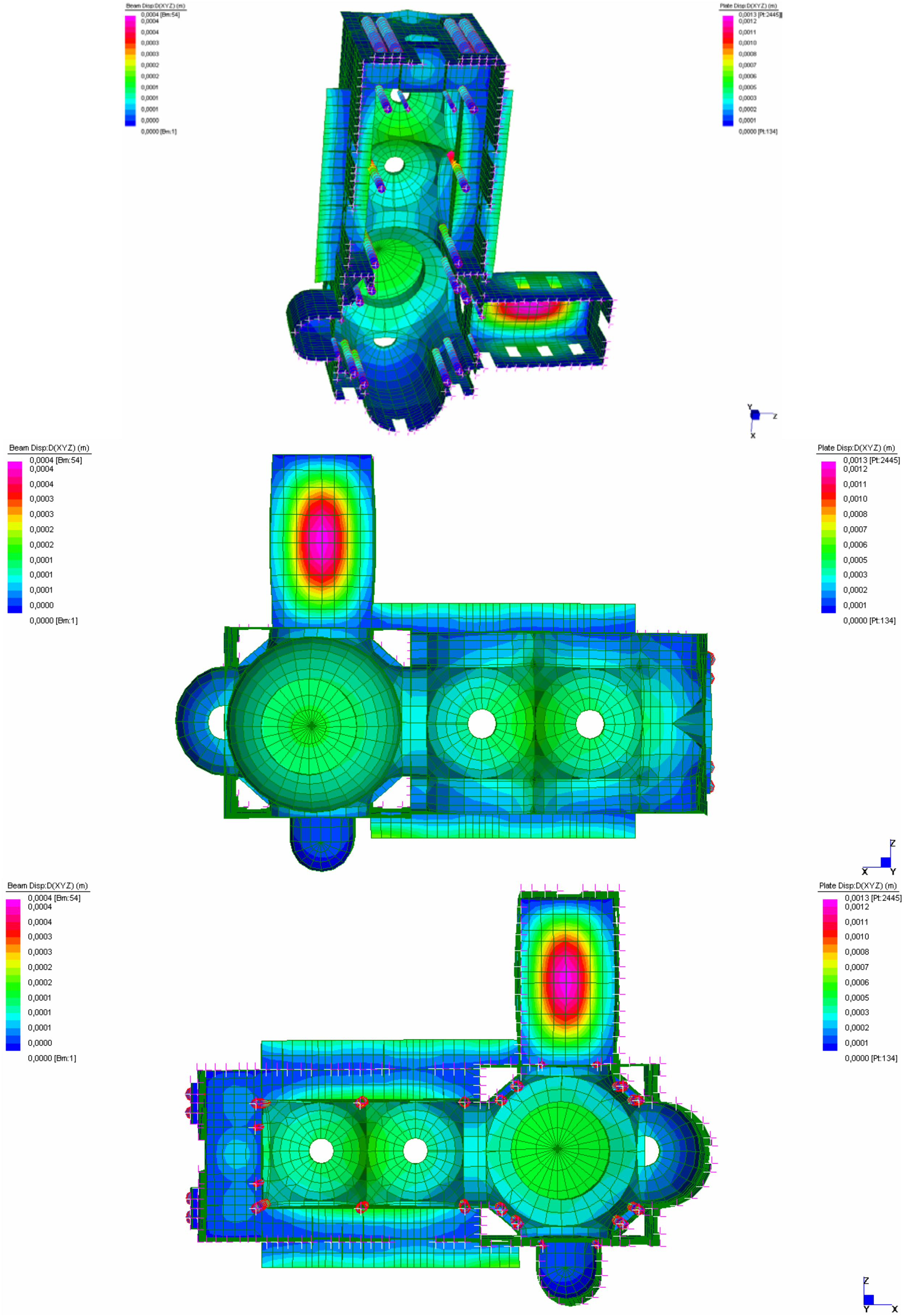
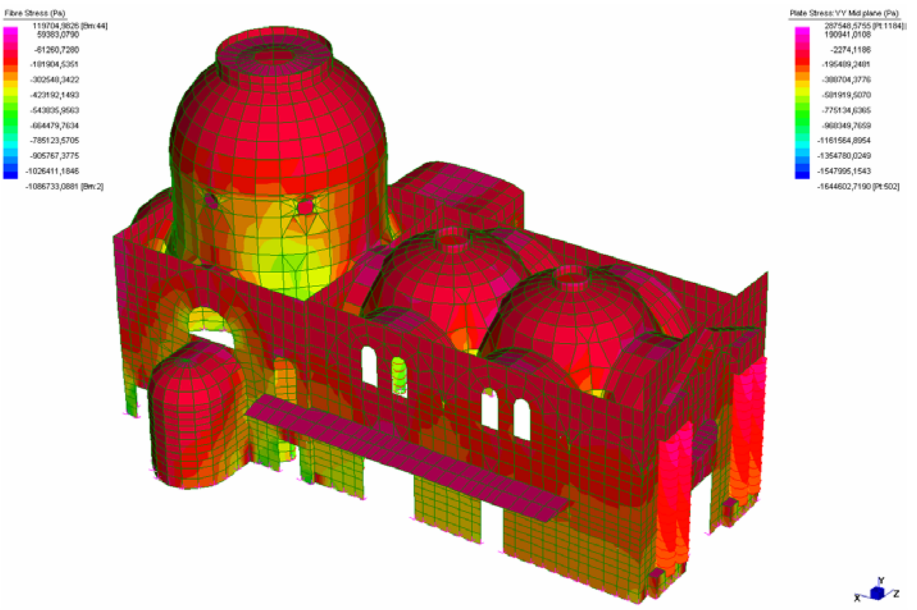
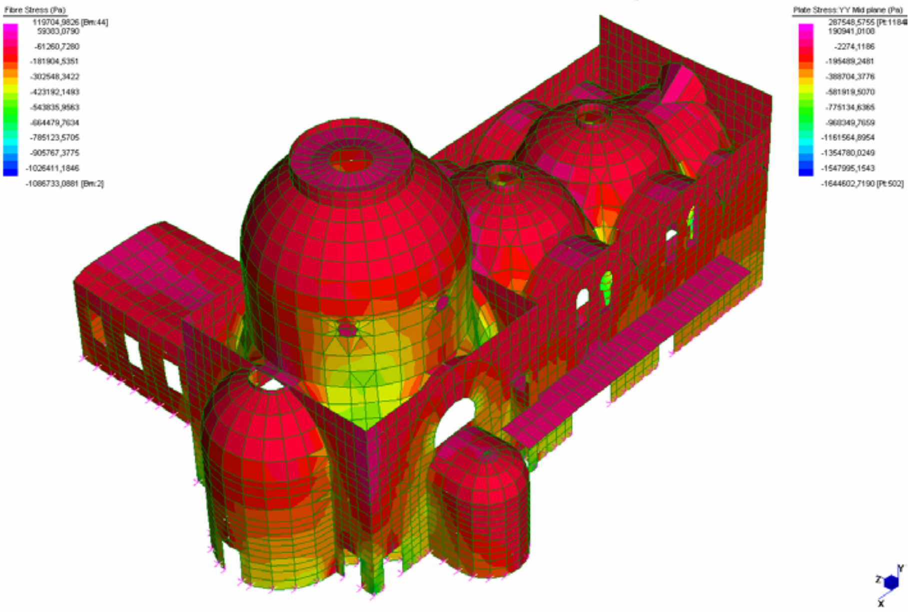


Figure 35: Displacements under the gravity load of the Las Hermanas de la Providencia Church FEM model.


Fibre Stress (Pa)
119704,9826 [Bm:44]
59383,0790
-61260,7280
-181904,5351
-302548,3422
-423192,1493
-543835,9563
-664479,7634
-785123,5705
-905767,3775
-1026411,1846
-1086733,0881 [Bm:2]
Plate Stress:YY Mid plane (Pa)
287548,5755 [Pt:1184]
190941,0108
-2274,1186
-195489,2481
-388704,3776
-581919,5070
-775134,6365
-968349,7659
-1161564,8954
-1354780,0249
-1547995,1543
-1644602,7190 [Pt:502]
Y
X
Z


Fibre Stress (Pa)
119704,9826 [Bm:44]
59383,0790
-61260,7280
-181904,5351
-302548,3422
-423192,1493
-543835,9563
-664479,7634
-785123,5705
-905767,3775
-1026411,1846
-1086733,0881 [Bm:2]
Plate Stress:YY Mid plane (Pa)
287548,5755 [Pt:1184
190941,0108
-2274,1186
-195489,2481
-388704,3776
-581919,5070
-775134,6365
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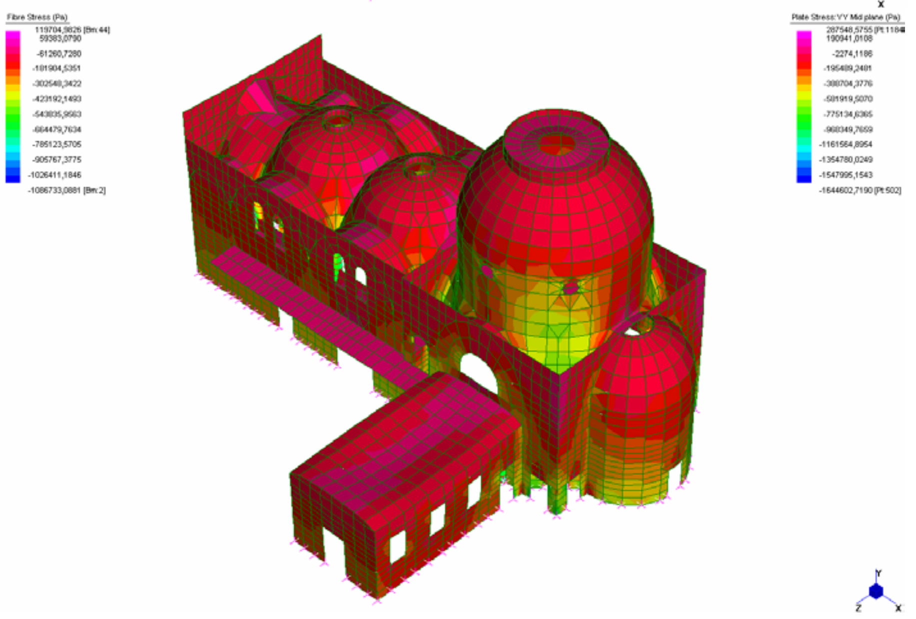
Z
Y
X


Fibre Stress (Pa)
119704,9826 [Bm:44]
59383,0790
-61260,7280
-181904,5351
-302548,3422
-423192,1493
-543835,9563
-664479,7634
-785123,5705
-905767,3775
-1026411,1846
-1086733,0881 [Bm:2]
Plate Stress:YY Mid plane (Pa)
287548,5755 [Pt:1184
190941,0108
-2274,1186
-195489,2481
-388704,3776
-581919,5070
-775134,6365
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Y
Z
X

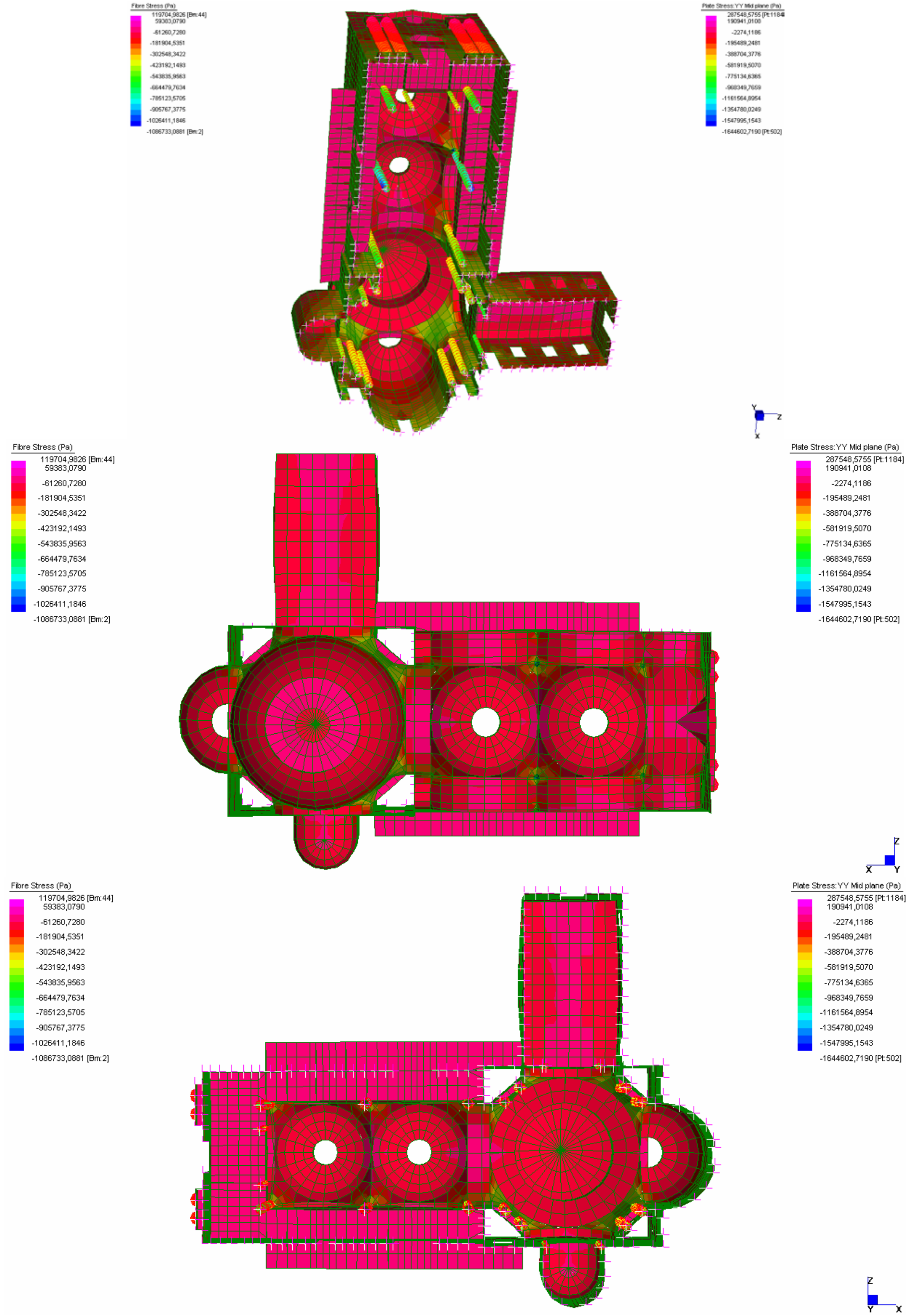


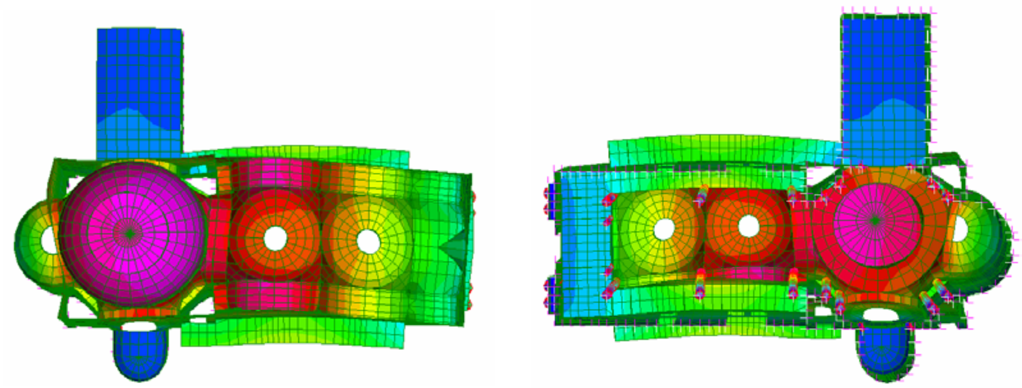
Figure 36: Elements stresses (beams: total fibre stress; plates: YY mid plane stress) under the gravity load of the Las Hermanas de la Providencia Church FEM model.

The Spectra used in the present analyses are the results of the studies of the seismic hazard of Valparaiso performed in the *Mar Vasto* Project: for different Magnitudes of the expected seismic events, the geological models gave two different time histories (acceleration record of the earthquake), according to two different kind of failure of the soil (monolateral and bilateral), for each main direction (North-South and East-West) and the related Spectra for a 5% damping ratio (usual value adopted for buildings) for the site of Las Hermanas de la Providencia Church. Seismic input has been chosen as reported in paragraph 6. The Spectra are already multiplied by the above mentioned scaling factor (the assumed value is 0.33, that correspond to a "behaviour factor" of 3) and Gravity (g): after this multiplication, the values of the curve will be in m/s^2. These units are consistent with all other units used in the analysis.
The earthquake was considered acting in the horizontal X-X and Z-Z directions of the FEM model of the Church separately and is given as an Acceleration Spectrum. To consider the X direction, the spectral values from the North-South graph will multiply the masses and they will apply the seismic action in the X direction of the FEM model; the earthquake will not be considered acting in the Y and Z directions. To consider the Z direction, the spectral values from the East-West graph will multiply the masses and they will apply the seismic action in the Z direction of the FEM model; the earthquake will not be considered acting in the X and Y directions.
Before executing the Spectral Response Solver, a Natural Frequency Analysis must be performed: in fact, the Spectral Response Solver uses a mode superposition technique that requires the mode shapes and corresponding natural frequencies and participating masses of the structure. The results for the twenty natural frequencies are displayed in Table 3 and in Fig. 37. After the 20th mode, the mass participation factor becomes lower than 2%, denoting local modes.

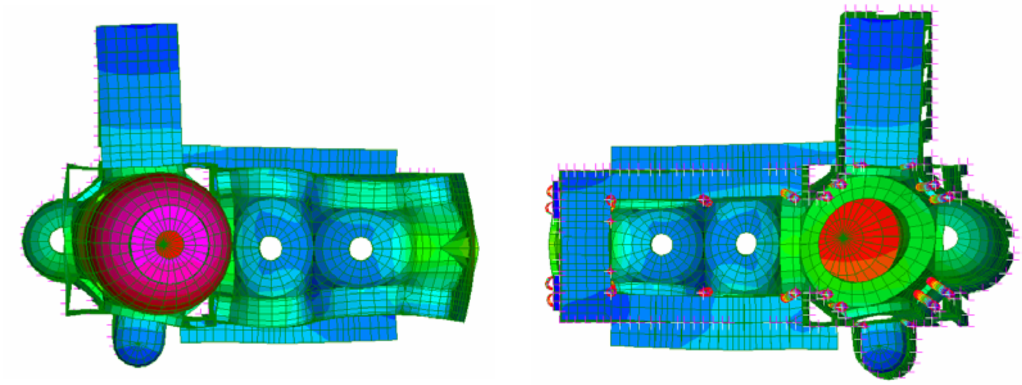
Table 3: Mode frequencies and Participation factors of the first twenty natural shapes of the Las Hermanas de la Providencia Church.

Mode	Frequency [Hz]	PF-X [%]	PF-Z [%]	PF-Y [%]
1	9.550E+00	0.000	62.429	0.006
2	1.317E+01	47.912	0.024	0.000
3	1.360E+01	1.167	0.015	0.225
4	1.394E+01	0.823	1.965	0.006
5	1.586E+01	0.002	0.000	1.296
6	1.664E+01	0.361	0.001	0.019
7	1.901E+01	0.058	0.302	0.000
8	1.930E+01	0.220	4.739	0.000
9	1.998E+01	21.236	0.025	0.008
10	2.090E+01	0.000	0.001	1.078
11	2.146E+01	1.771	0.062	0.516
12	2.156E+01	0.017	2.538	0.003
13	2.198E+01	0.007	0.232	0.024
14	2.242E+01	0.009	0.053	0.105
15	2.302E+01	7.006	0.048	0.002
16	2.371E+01	0.204	0.069	0.098
17	2.422E+01	0.016	0.149	0.185
18	2.451E+01	0.009	0.110	4.256
19	2.478E+01	0.402	0.410	0.685
20	2.506E+01	0.212	1.037	1.785
Total Mass Participation Factors		**81.432**	**74.208**	**10.297**

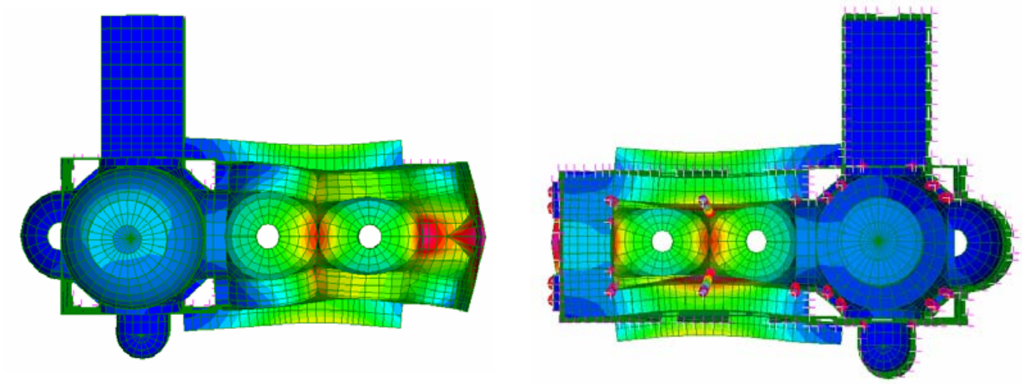
1 9.550E+00 Hz



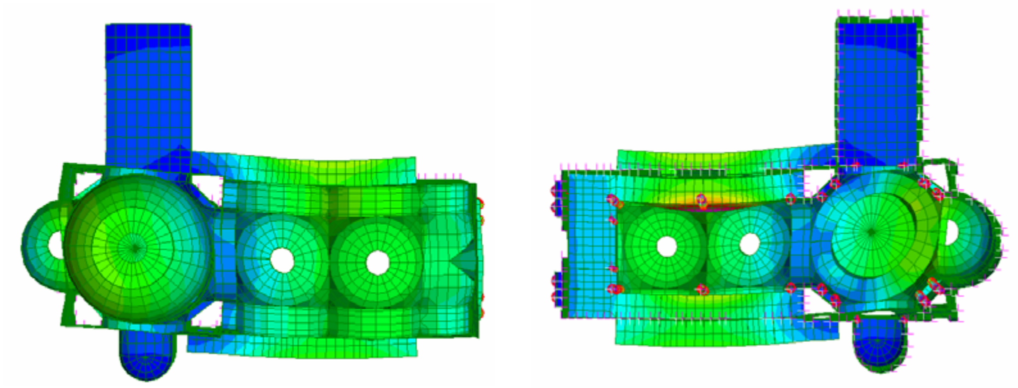
2 1.317E+01 Hz



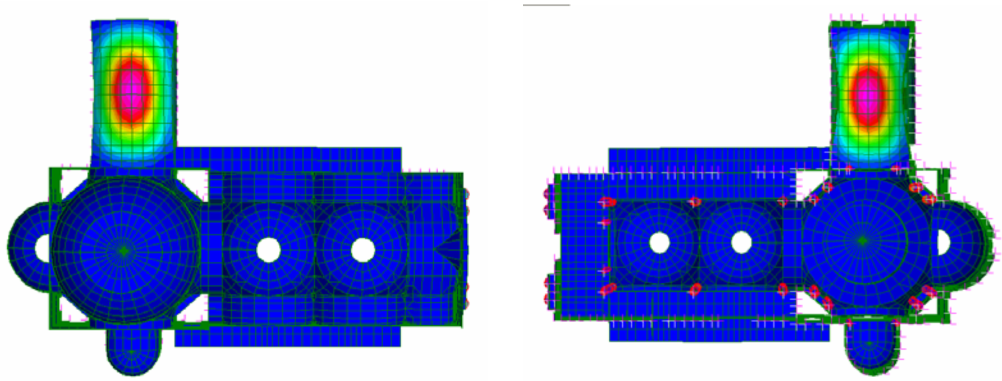
3 1.360E+01 Hz



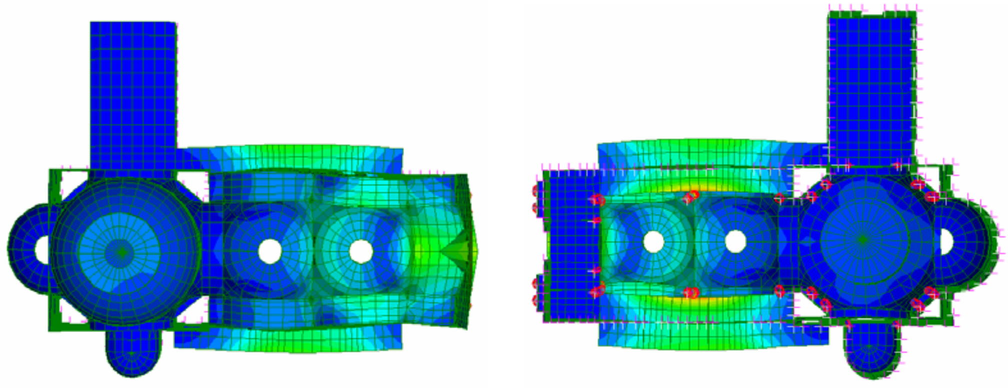
4 1.394E+01 Hz



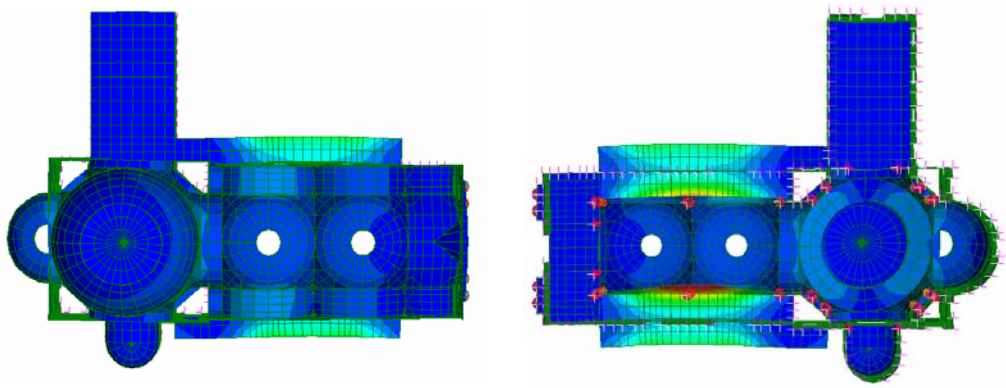
5 1.586E+01 Hz



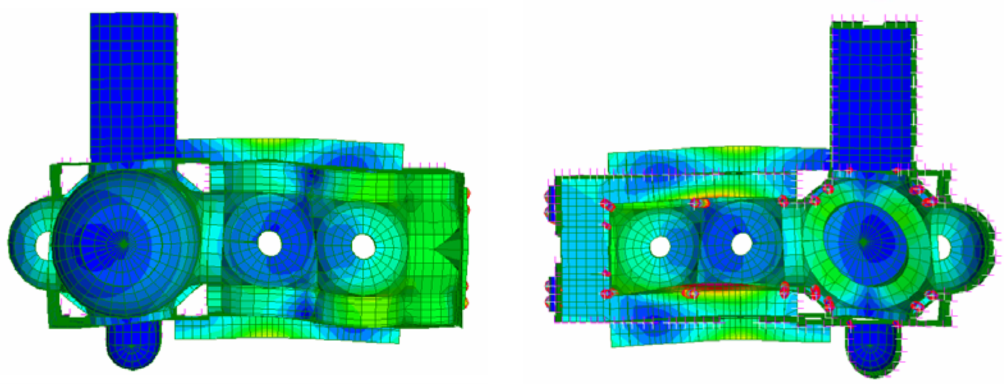
6 1.664E+01 Hz



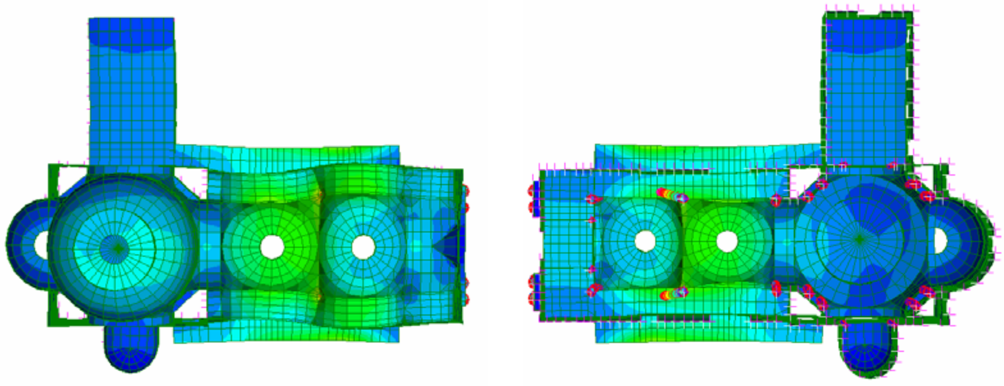
7 1.901E+01 Hz



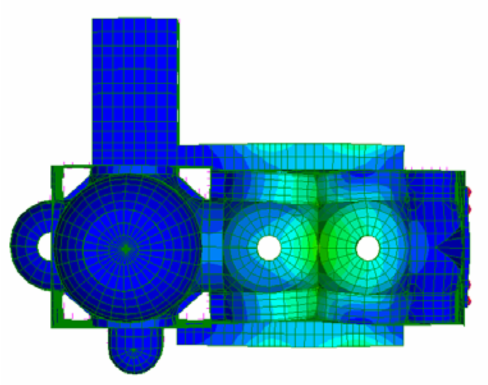
8 1.930E+01 Hz

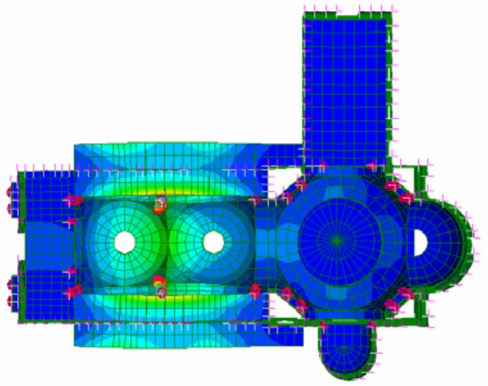


9 1.998E+01 Hz

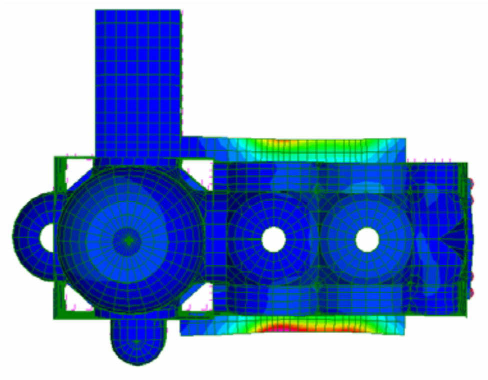


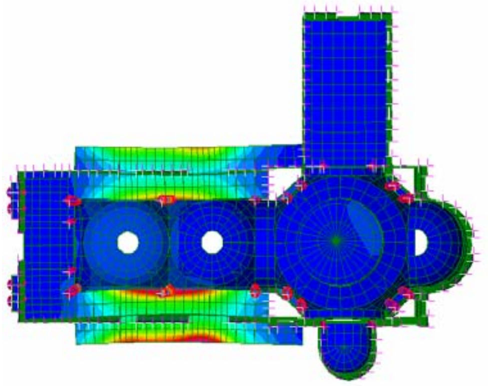
10 2.090E+01 Hz



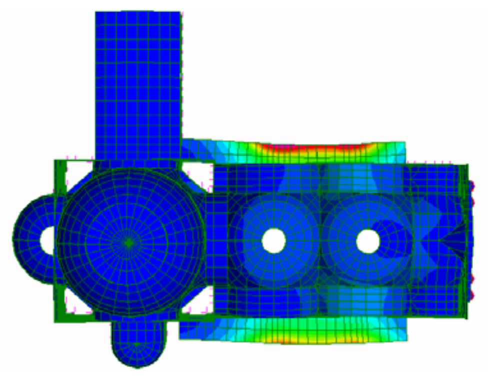


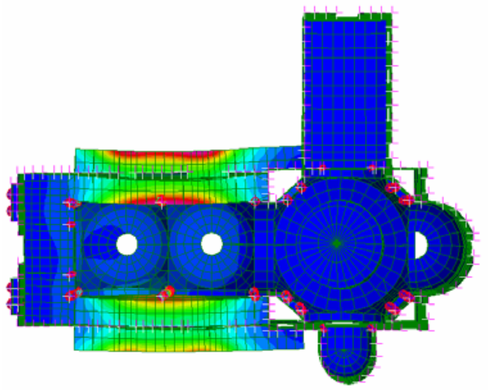
11 2.146E+01 Hz



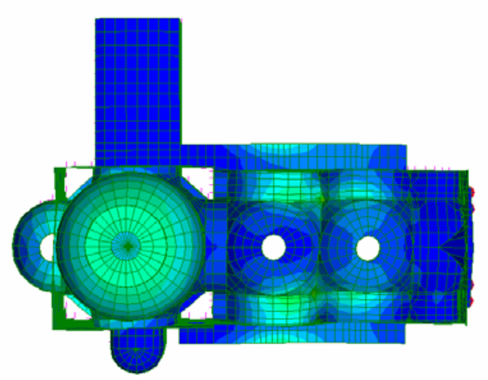


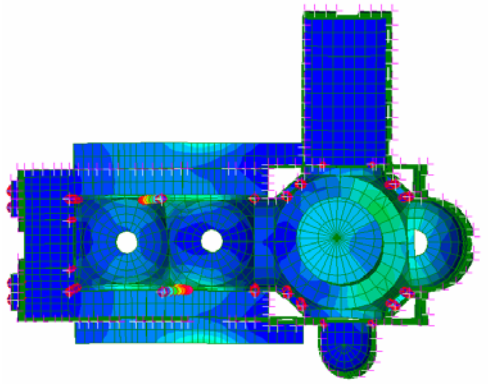
12 2.156E+01 Hz



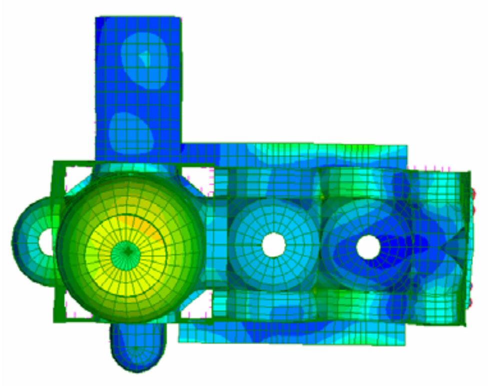


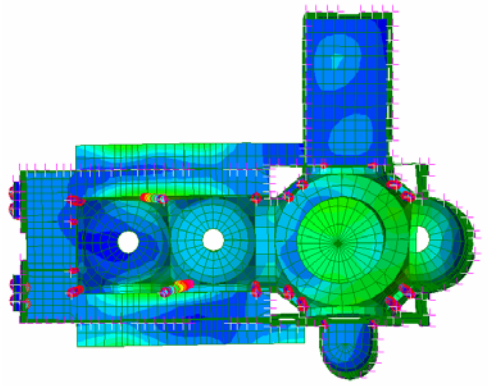
13 2.198E+01 Hz



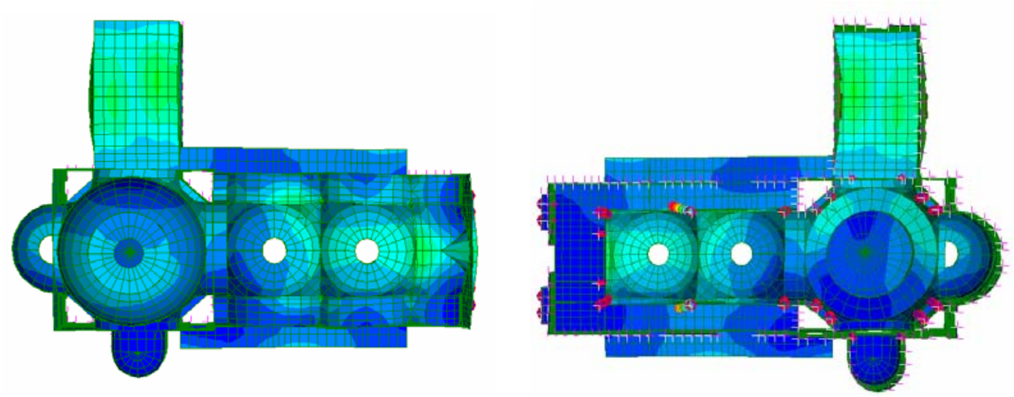


14 2.242E+01 Hz

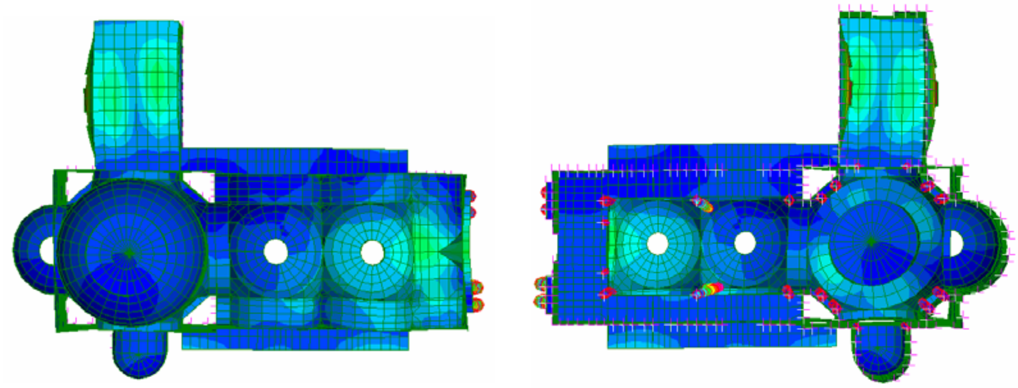




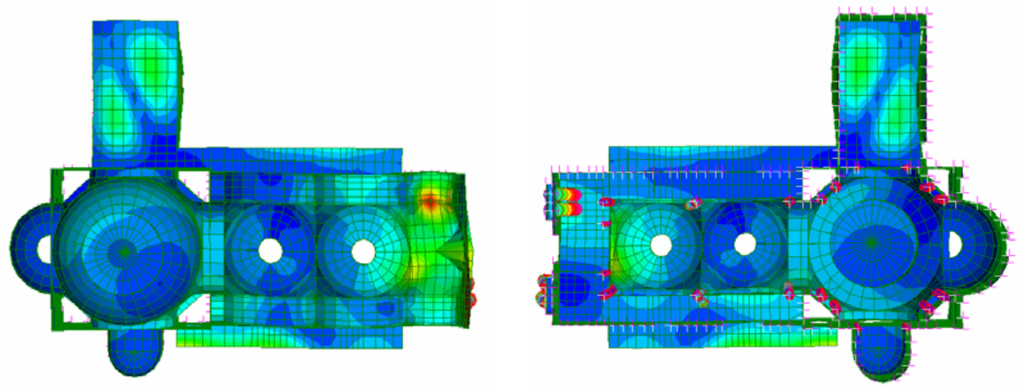
15 2.302E+01 Hz



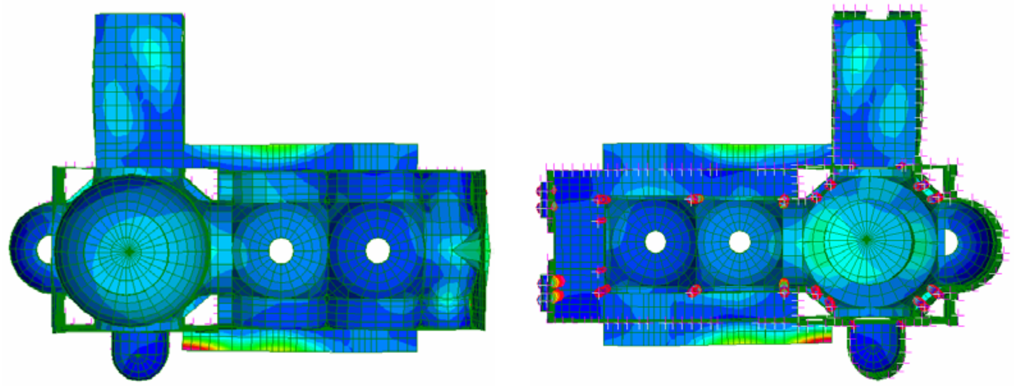
16 2.371E+01 Hz



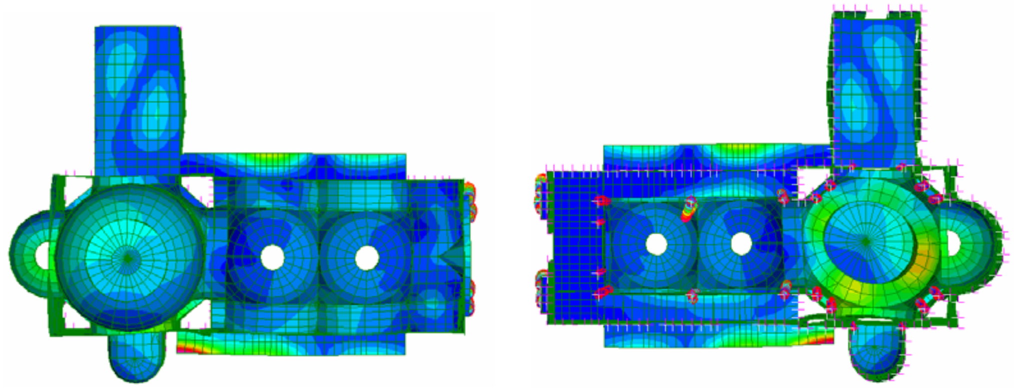
17 2.422E+01 Hz



18 2.451E+01 Hz



19 2.478E+01 Hz



20 2.506E+01 Hz

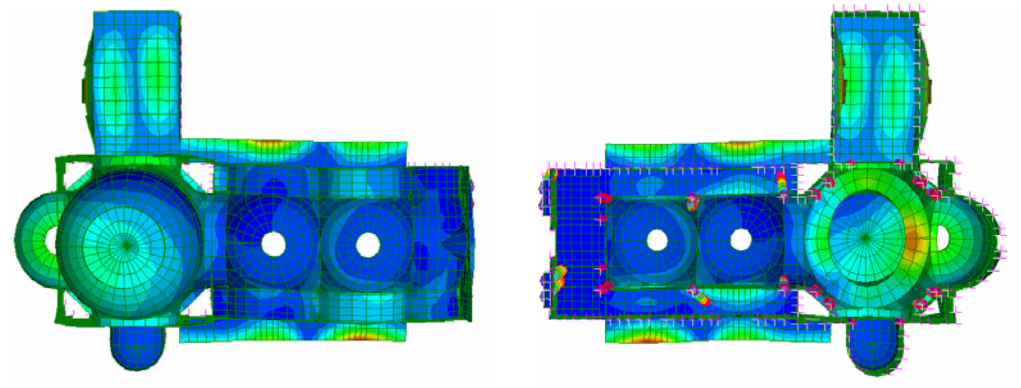


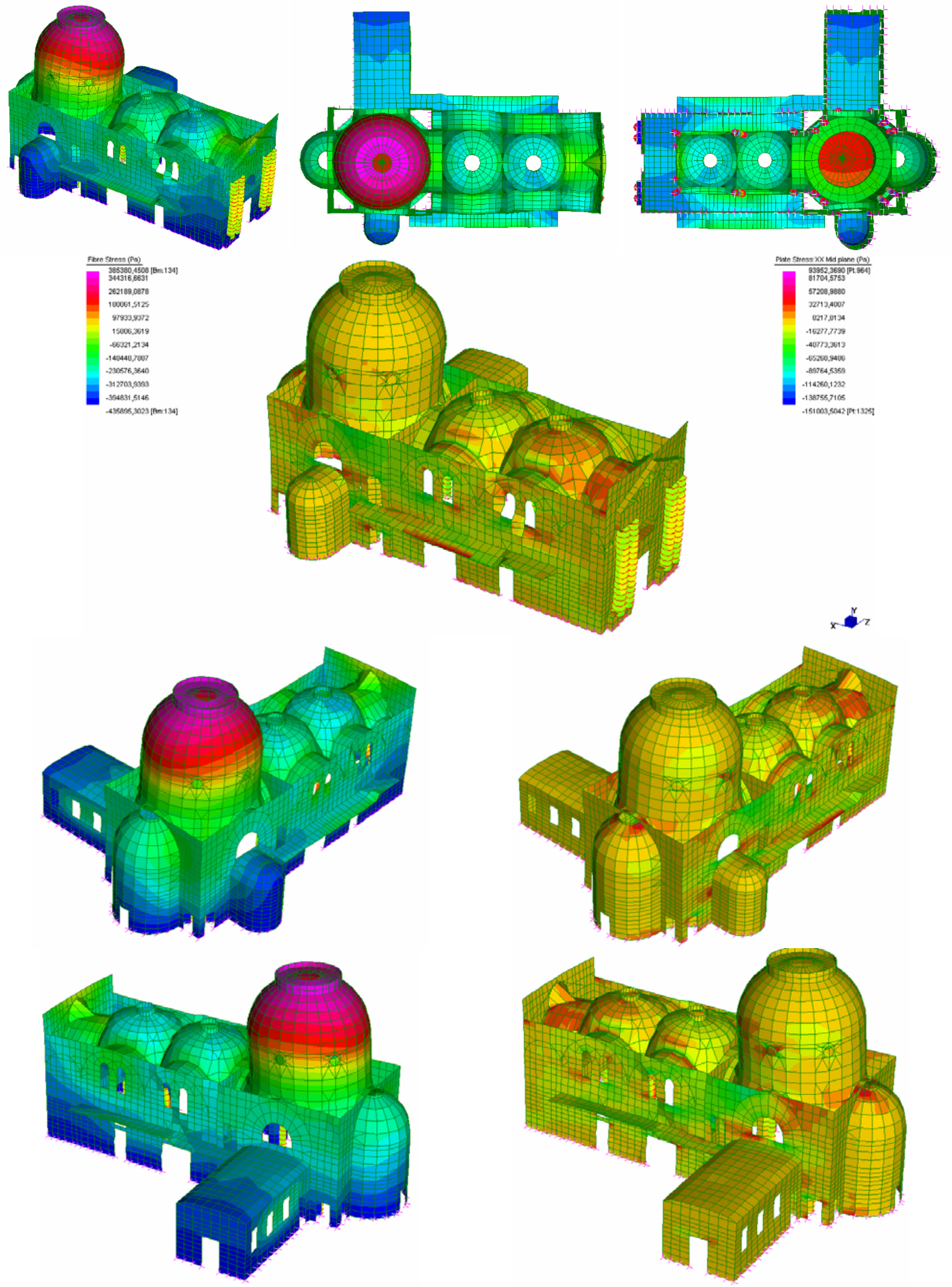
Figure 37: First twenty mode shapes of the Las Hermanas de la Providencia Church.

The results of the Spectral Response Analysis in terms of deformed shape, maximum displacements and maximum element stresses are given in Fig. 38. The Total Mass Participation factor (Table 4) indicates that the number of included natural modes is sufficient enough to adequately represent the dynamic behaviour of the structure.

Table 4: Spectral Response Analysis of the Las Hermanas de la Providencia Church.

Analysis	Magnitude	Direction	Total Mass Participation factor
1	7.5	X (North-South)	81.432%
2	7.5	Z (East-West)	74.208%
3	8.0	X (North-South)	81.432%
4	8.0	Z (East-West)	74.208%
5	8.5	X (North-South)	81.432%
6	8.5	Z (East-West)	74.208%

Case 1: Magnitude 7.5 Bilateral – X (North-South) direction



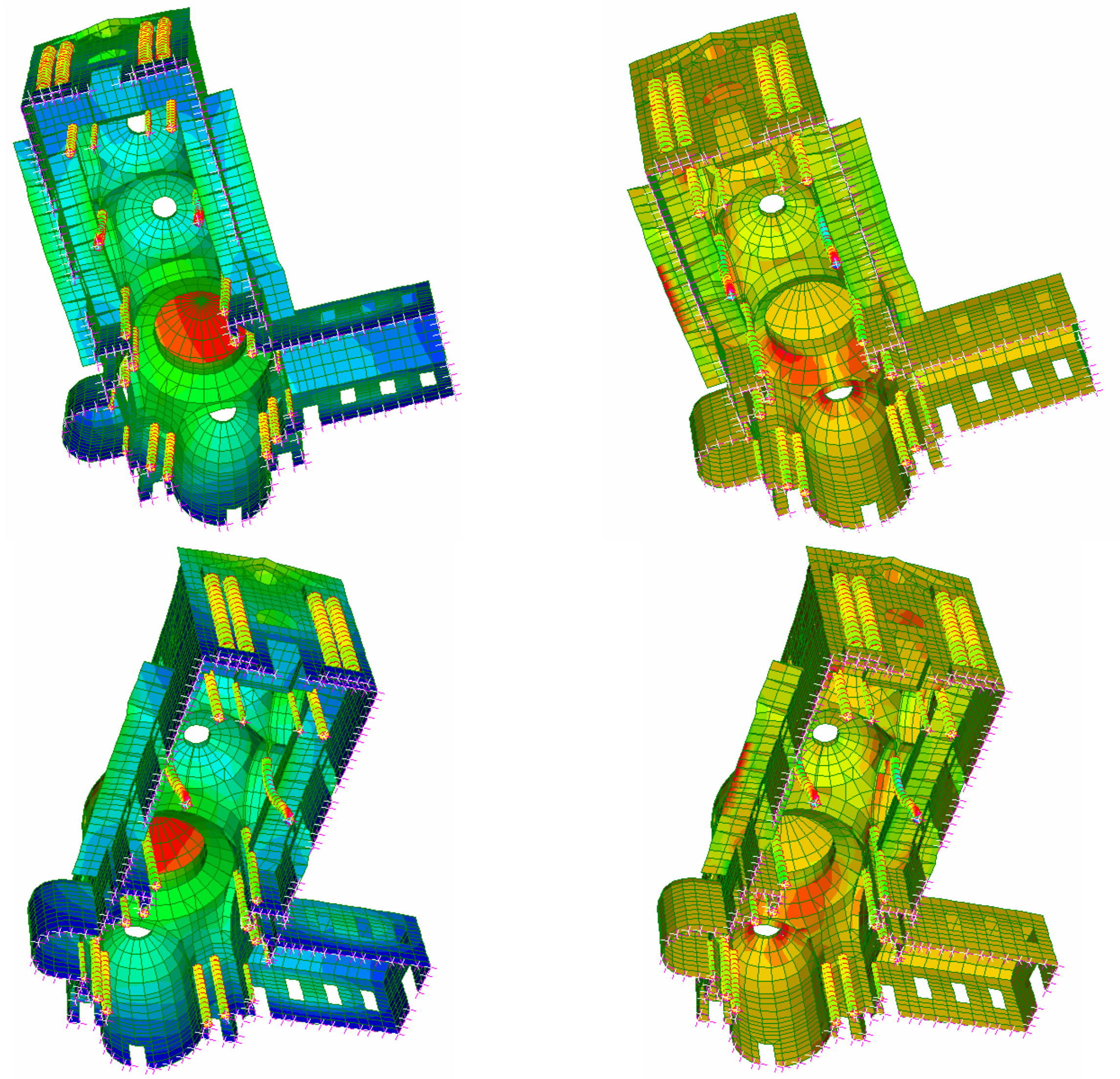
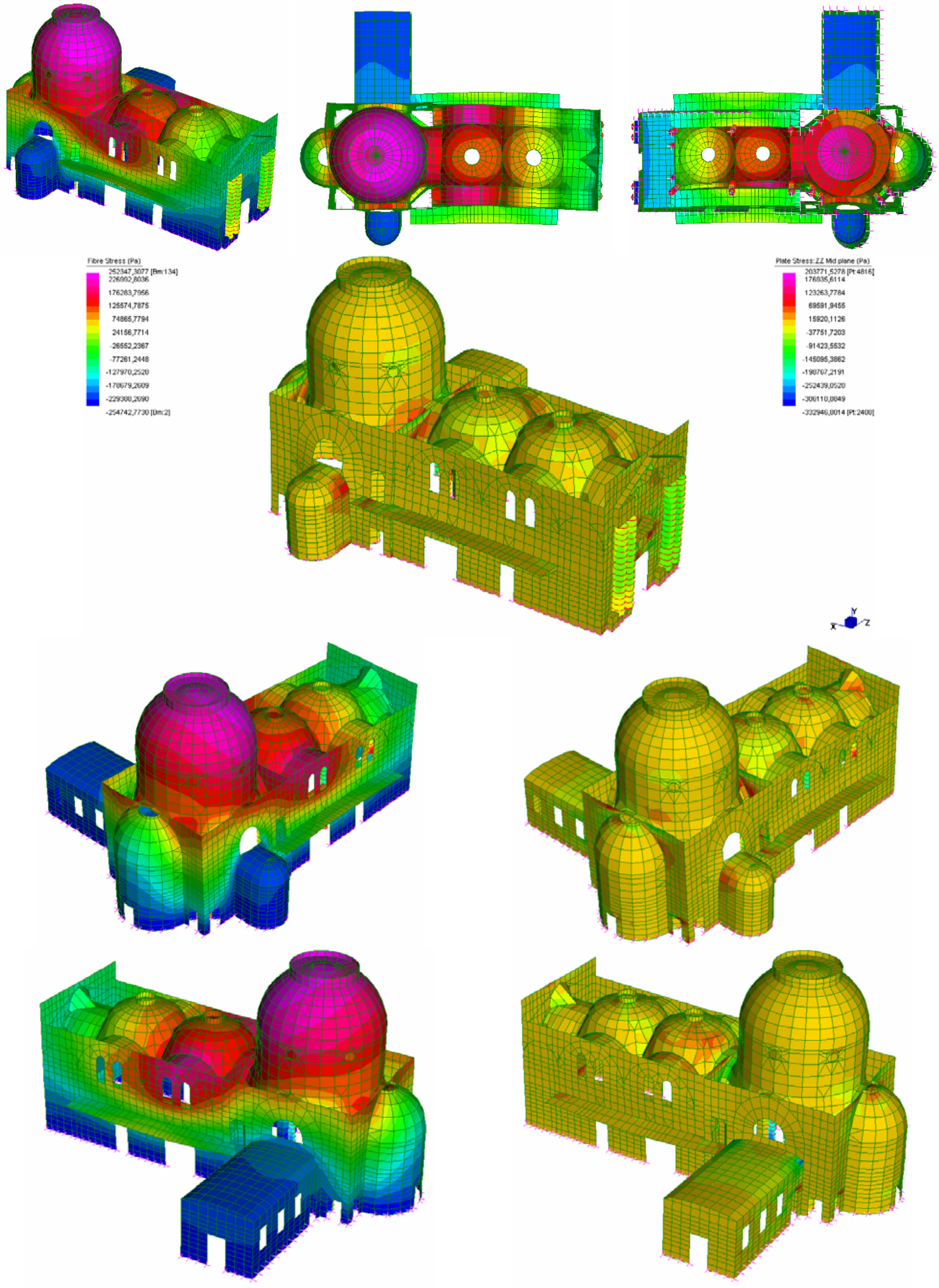


Figure 38a: Case 1, Magnitude 7.5 Bilateral – X (North-South) direction.

<u>Case 2:</u> Magnitude 7.5 Bilateral – Z (East-West) direction



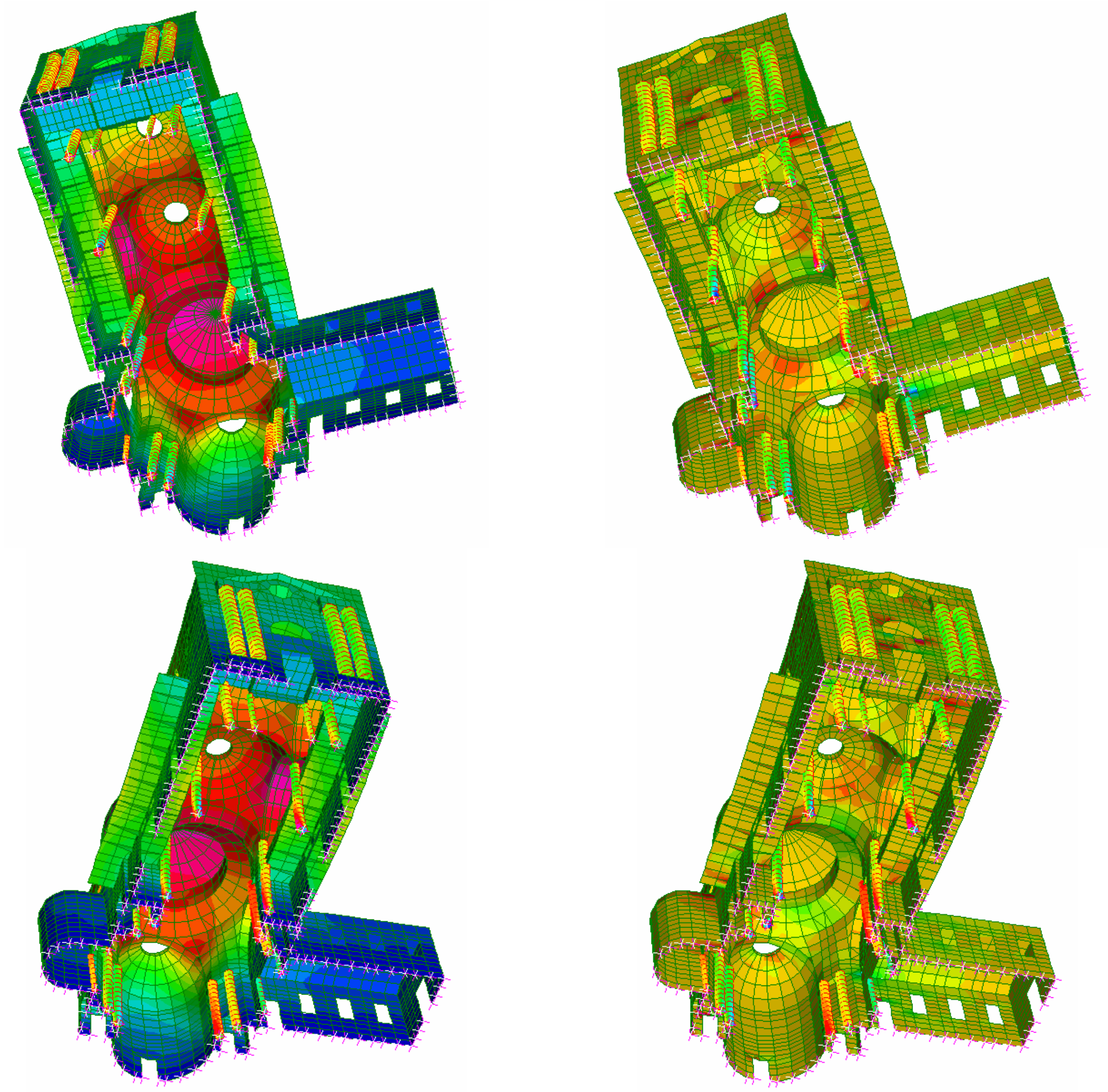
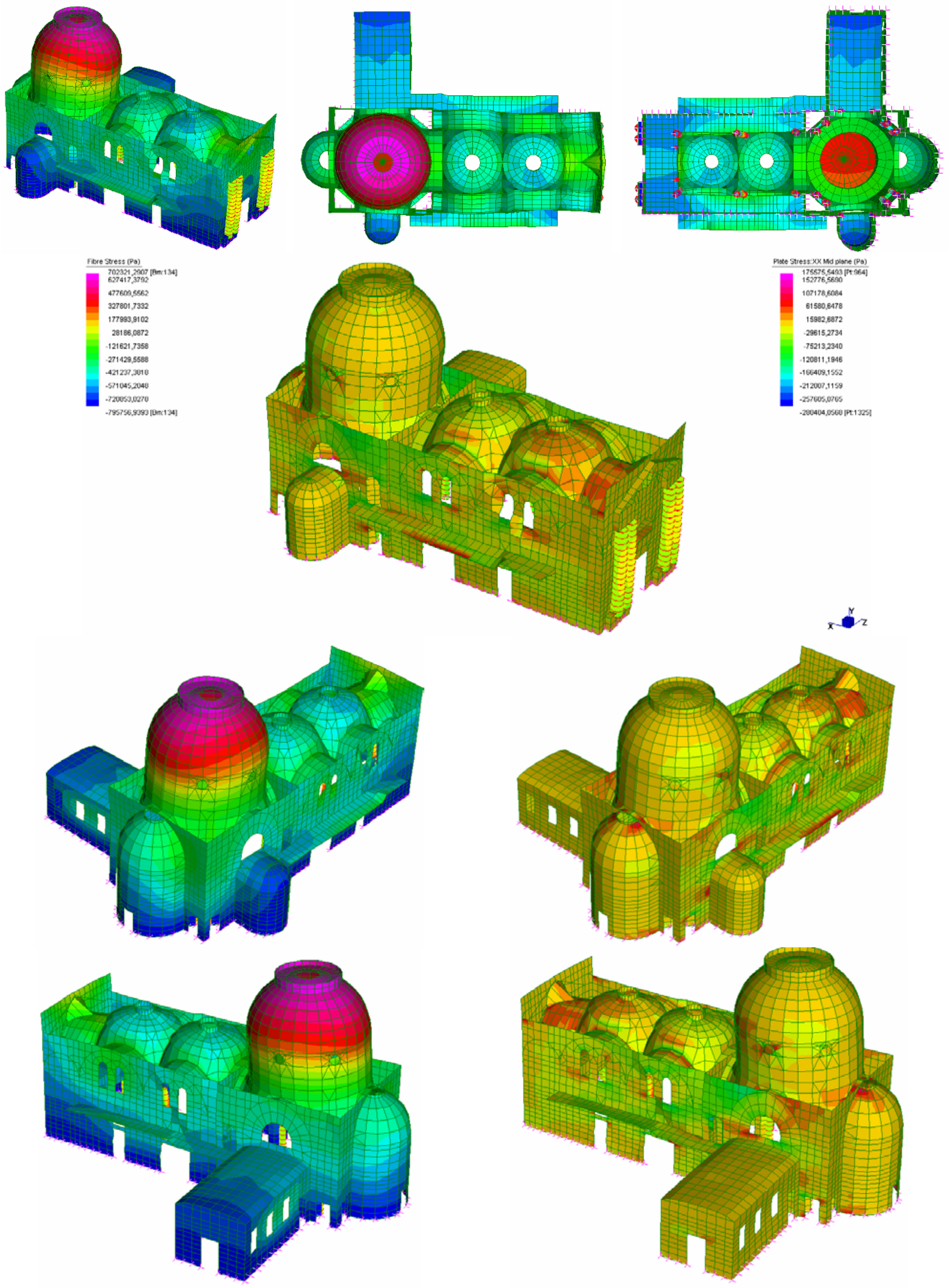


Figure 38b: Case 2, Magnitude 7.5 Bilateral – Z (East-West) direction.

<u>Case 3:</u> Magnitude 8.0 Bilateral – X (North-South) direction



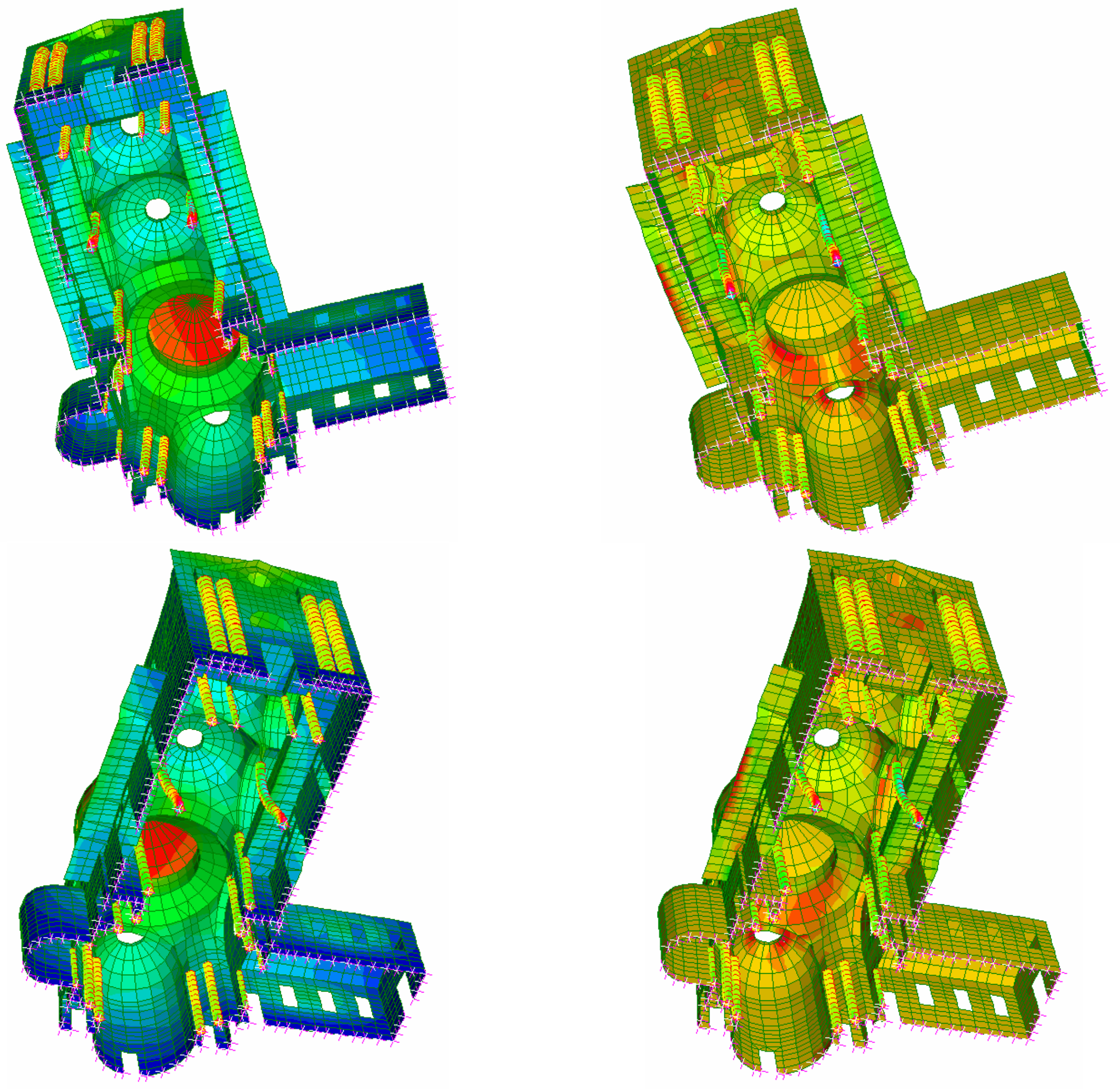
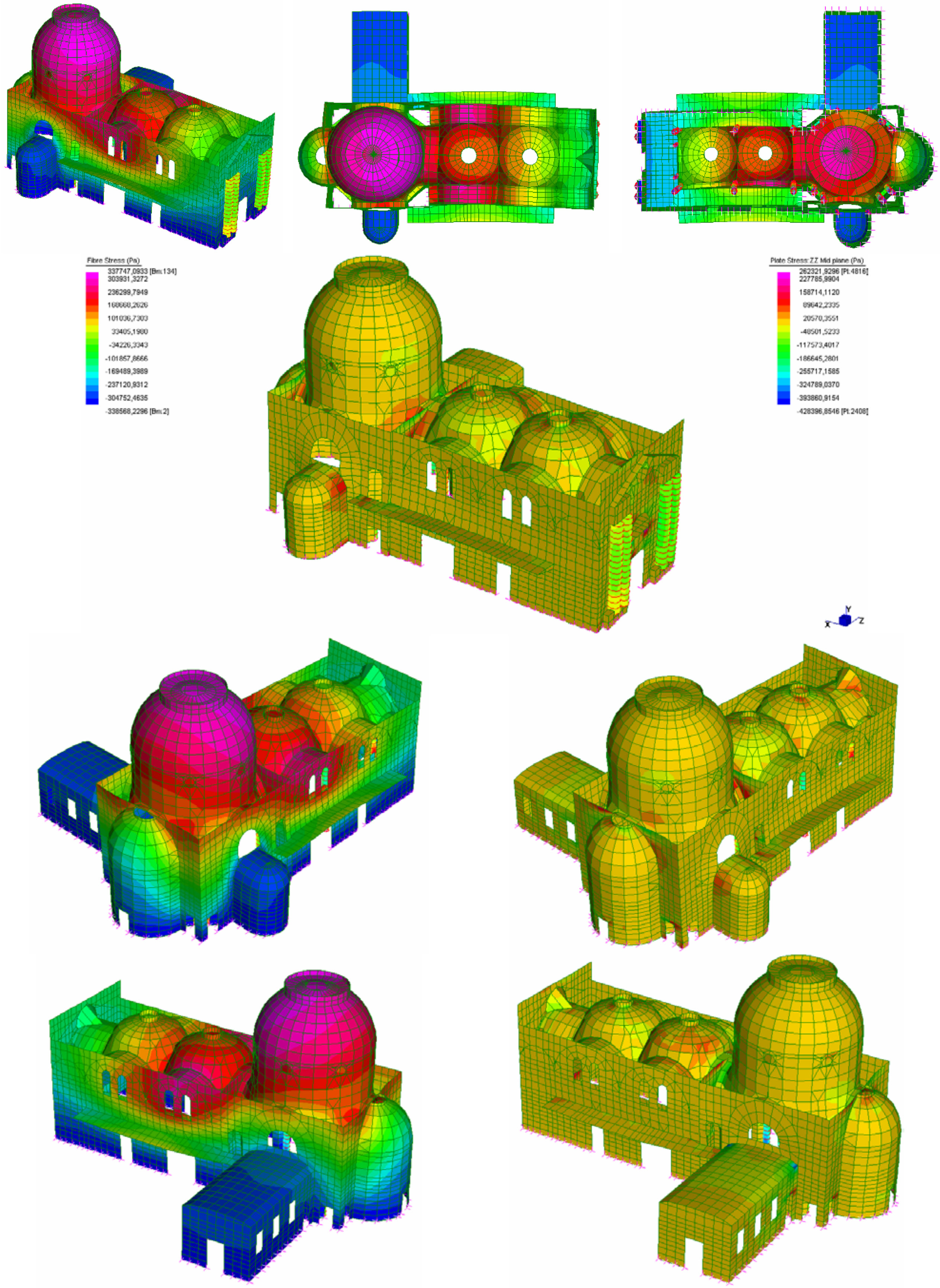


Figure 38c: Case 3, Magnitude 8.0 Bilateral – X (North-South) direction.

Case 4: Magnitude 8.0 Bilateral – Z (East-West) direction



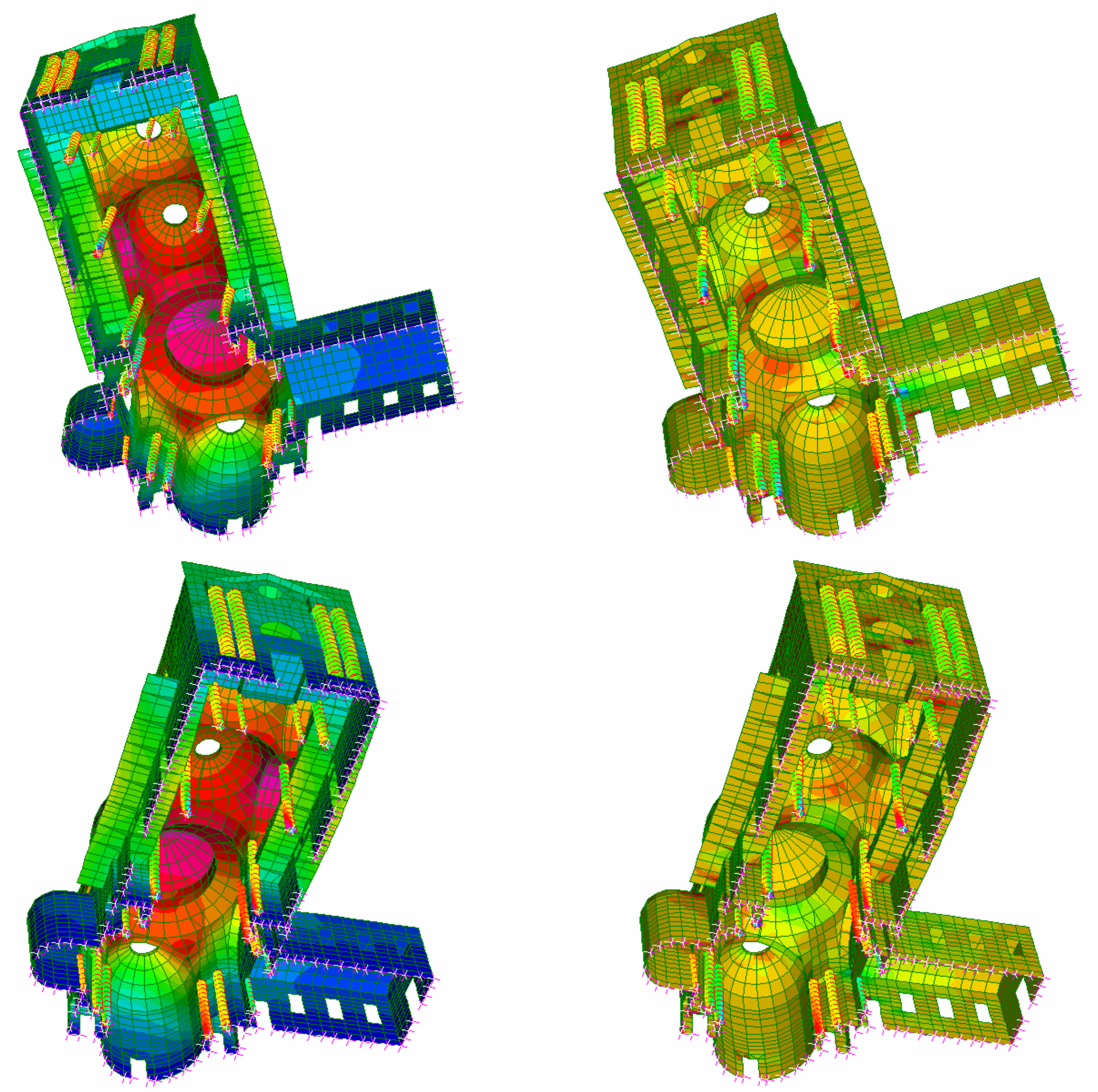
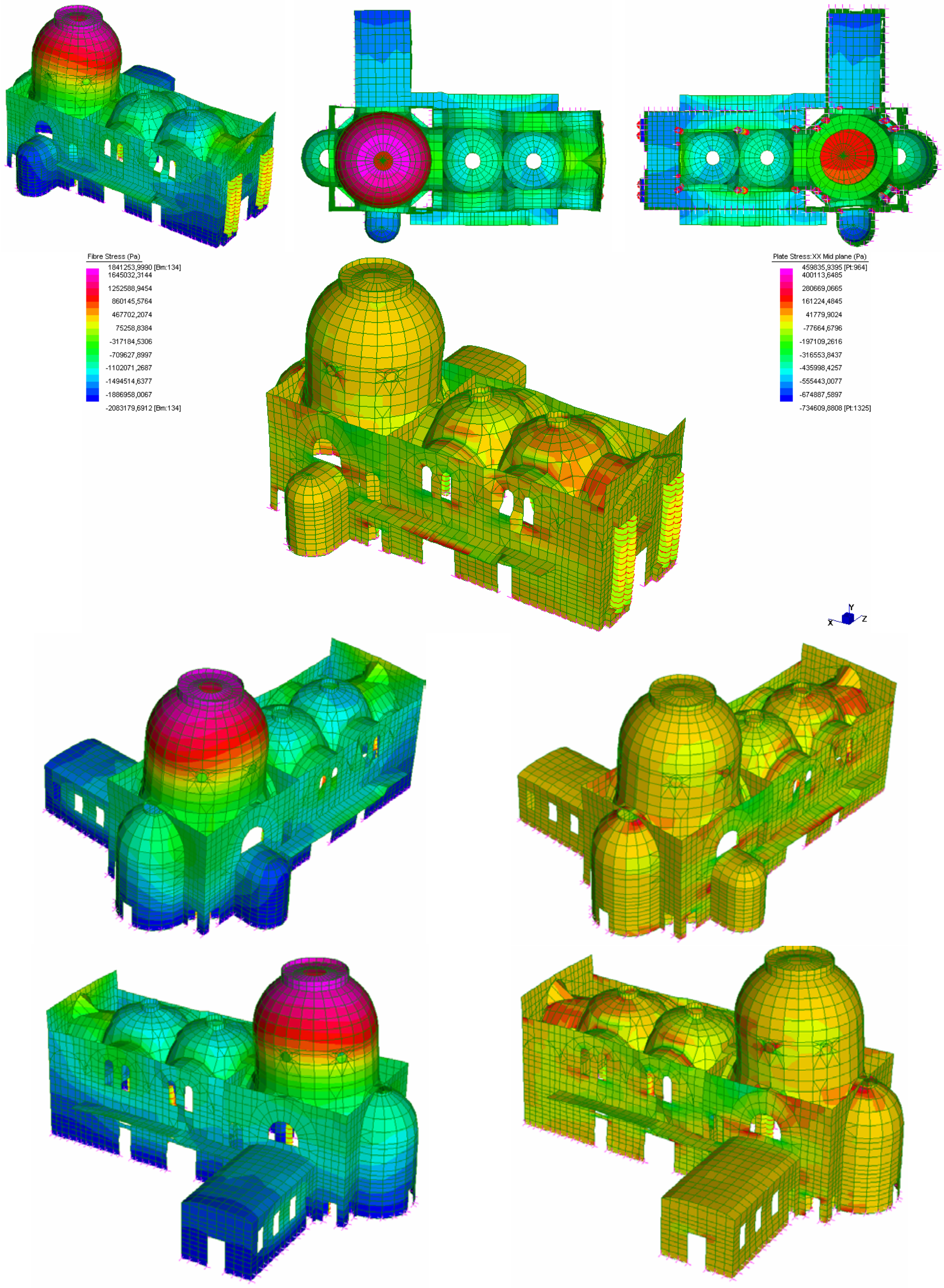


Figure 38d: Case 4, Magnitude 8.0 Bilateral – Z (East-West) direction.

Case 5: Magnitude 8.5 Bilateral – X (North-South) direction



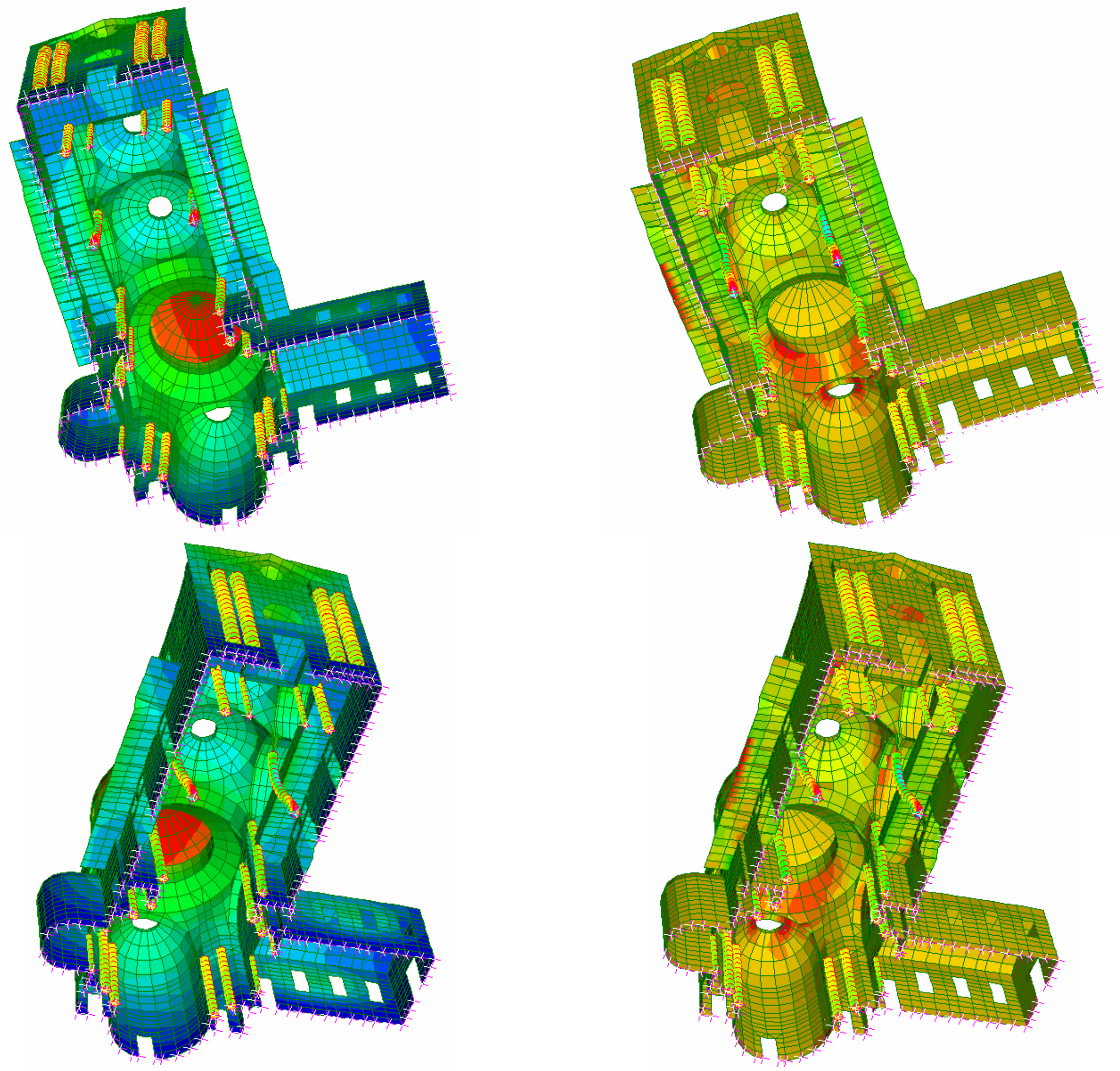
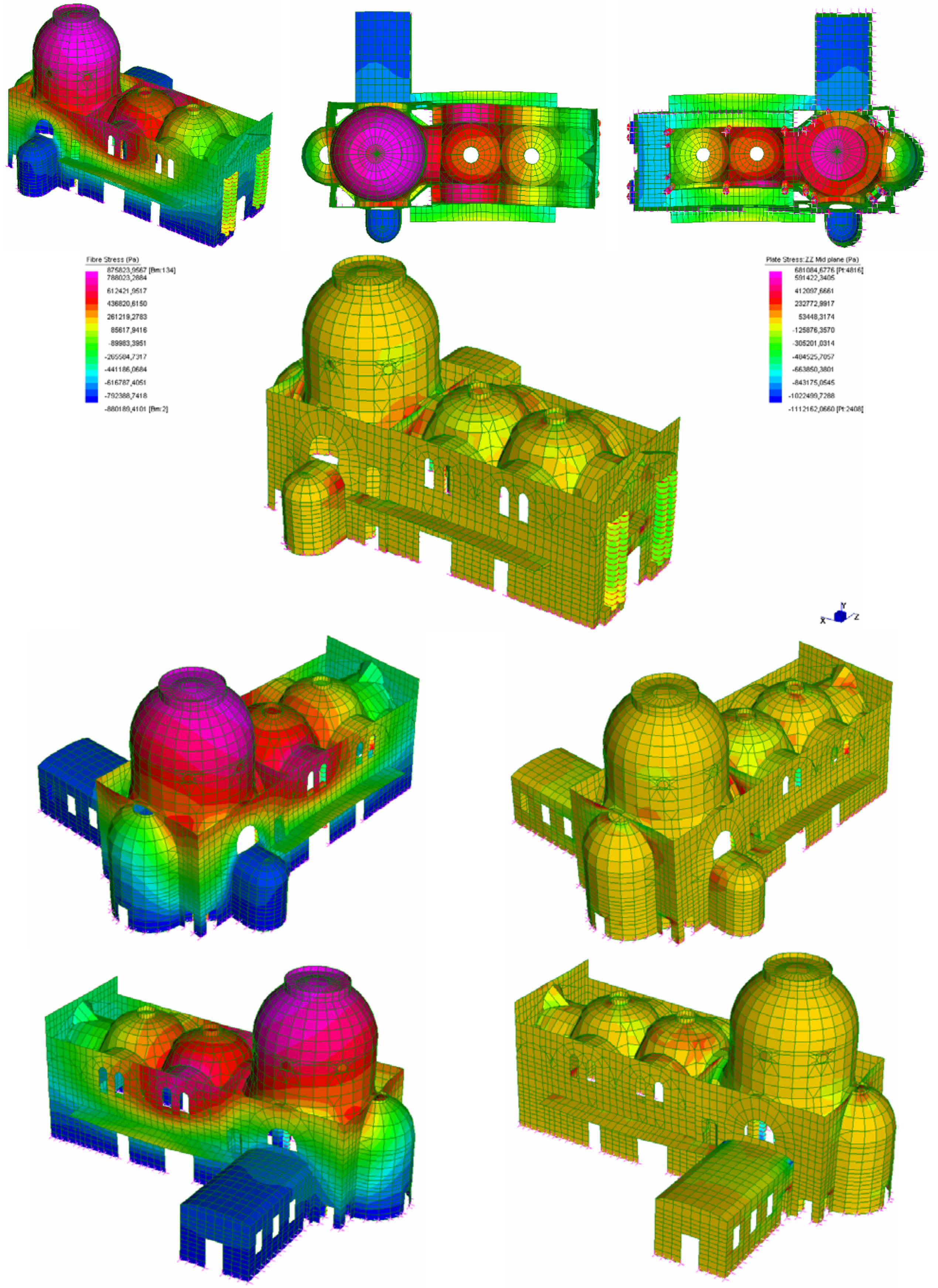


Figure 38e: Case 5, Magnitude 8.5 Bilateral – X (North-South) direction.

Case 6: **Magnitude 8.5 Bilateral – Z (East-West) direction**



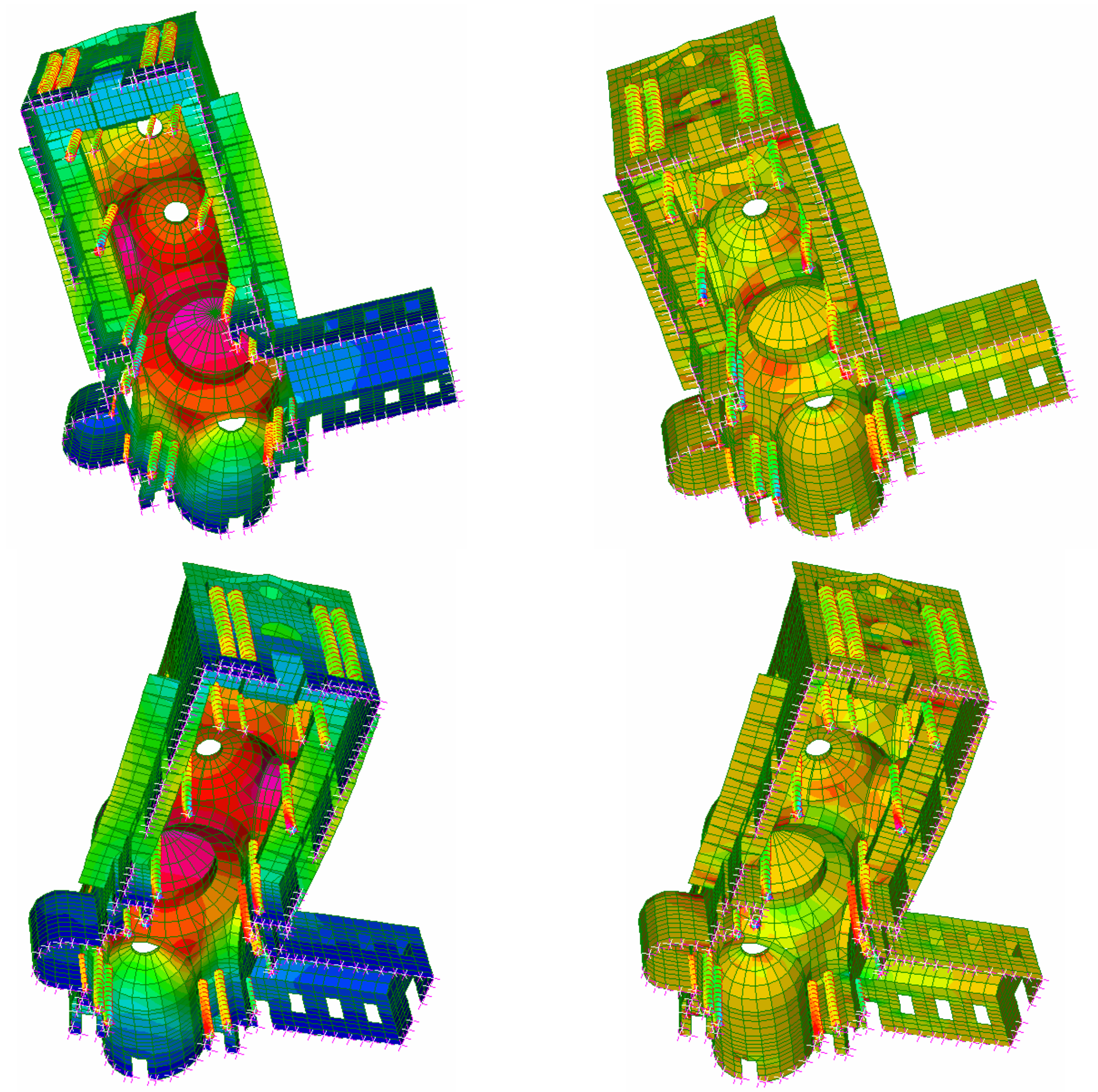
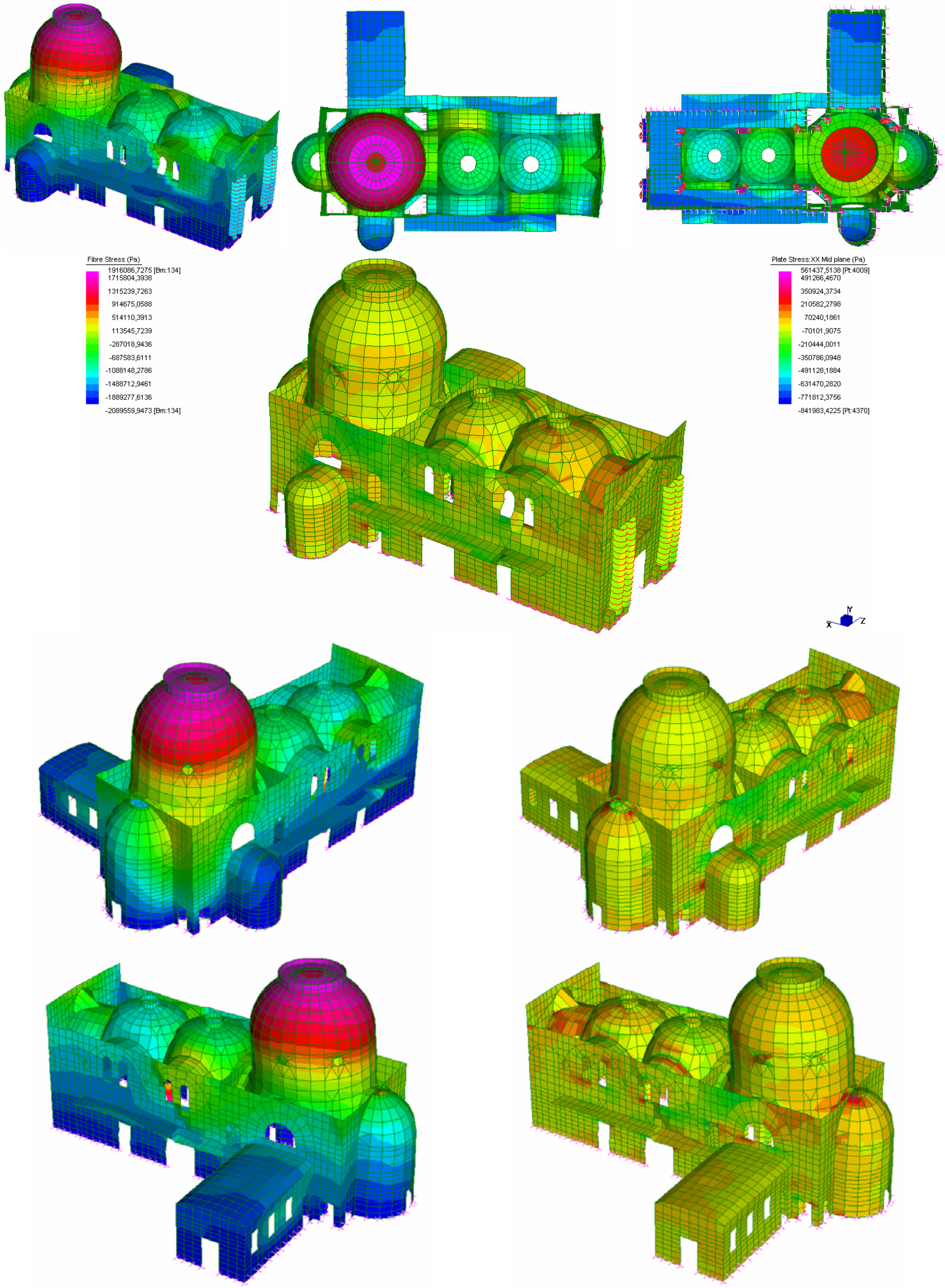


Figure 38f: Case 6, Magnitude 8.5 Bilateral – Z (East-West) direction.

Two other cases have been taken into account, combining the results of Case 5 and Case 6: in Combination 1 the 100% of the actions in the horizontal X-X direction of the FEM model of the Church (Case 5) and the 30% of the actions in the horizontal Z-Z direction (Case 6) were applied; in Combination 2 the 100% of the actions in the horizontal Z-Z direction of the FEM model of the Church (Case 6) and the 30% of the actions in the horizontal X-X direction (Case 5) were applied. The earthquake will not be considered acting in the Y direction.
The results of the Spectral Response Analysis in terms of deformed shape, maximum displacements and maximum element stresses are shown by Fig. 39.

<u>Combination 1:</u> Magnitude 8.5 Bilateral
100% X (North-South) direction + 30% Z (East-West) direction



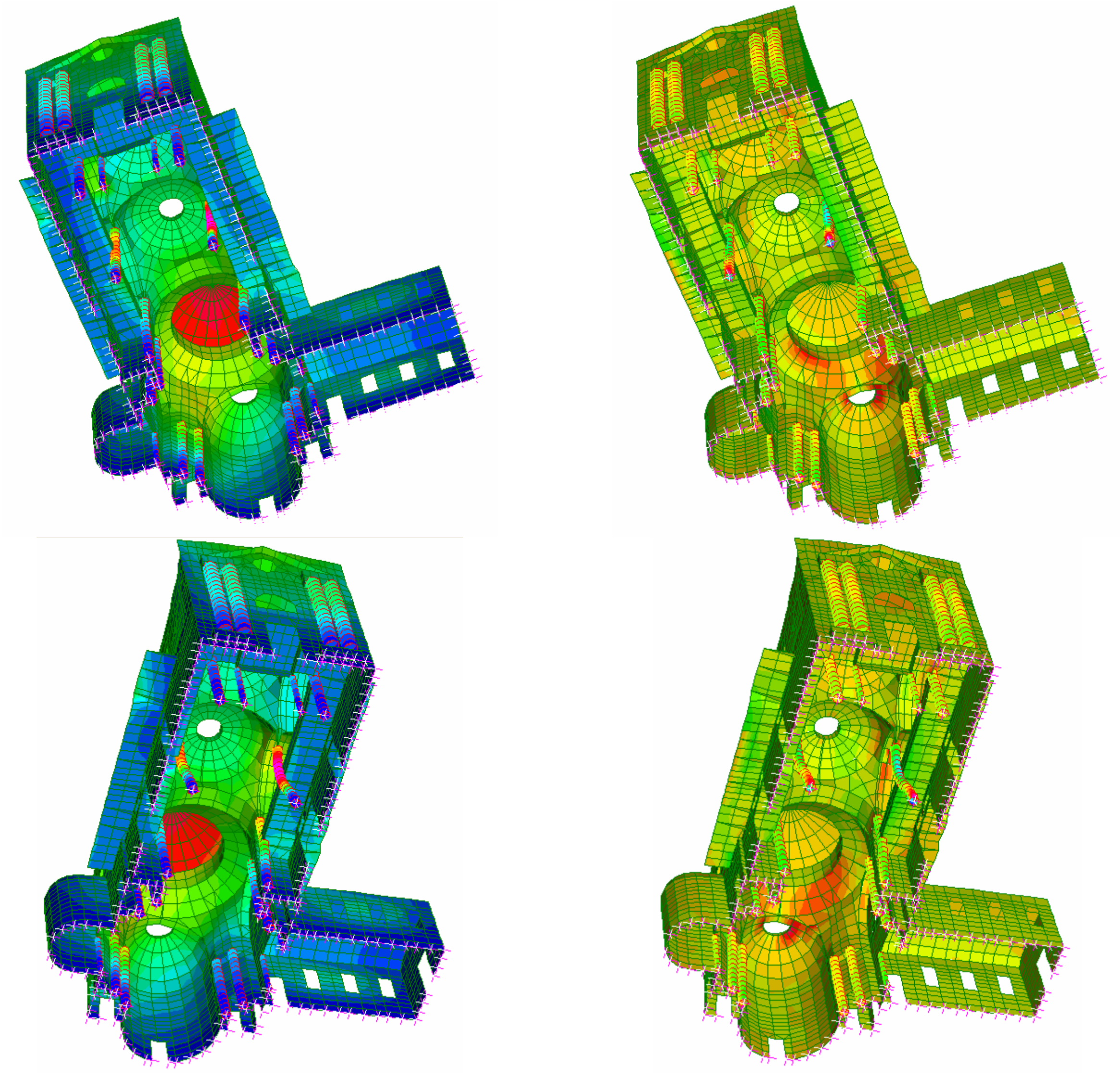
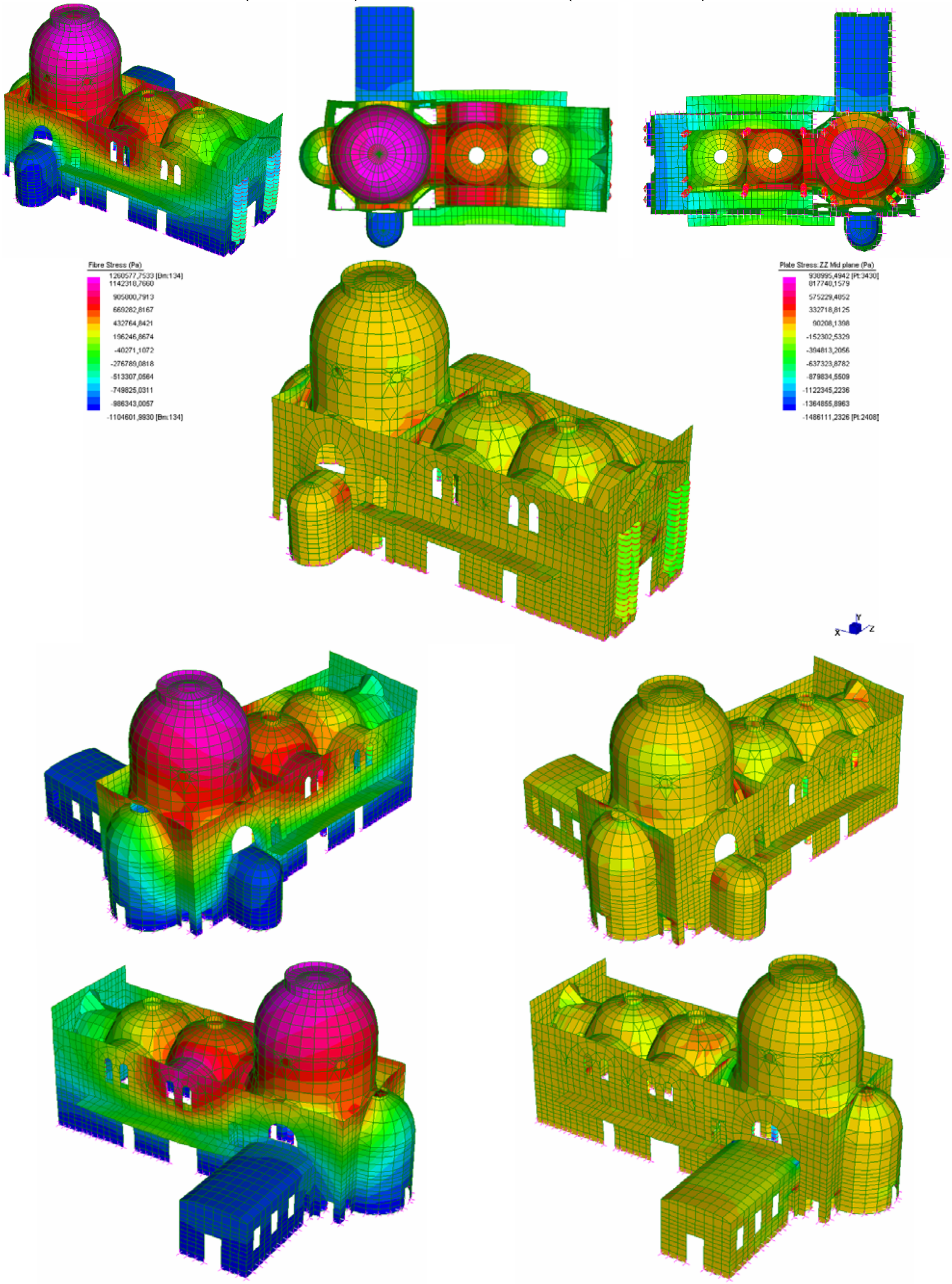


Figure 39a: Combination 1, Magnitude 8.5 Bilateral, 100% X (North-South) direction + 30% Z (East-West) direction.

<u>Combination 2:</u> Magnitude 8.5 Bilateral
100% Z (East-West) direction + 30% X (North-South) direction



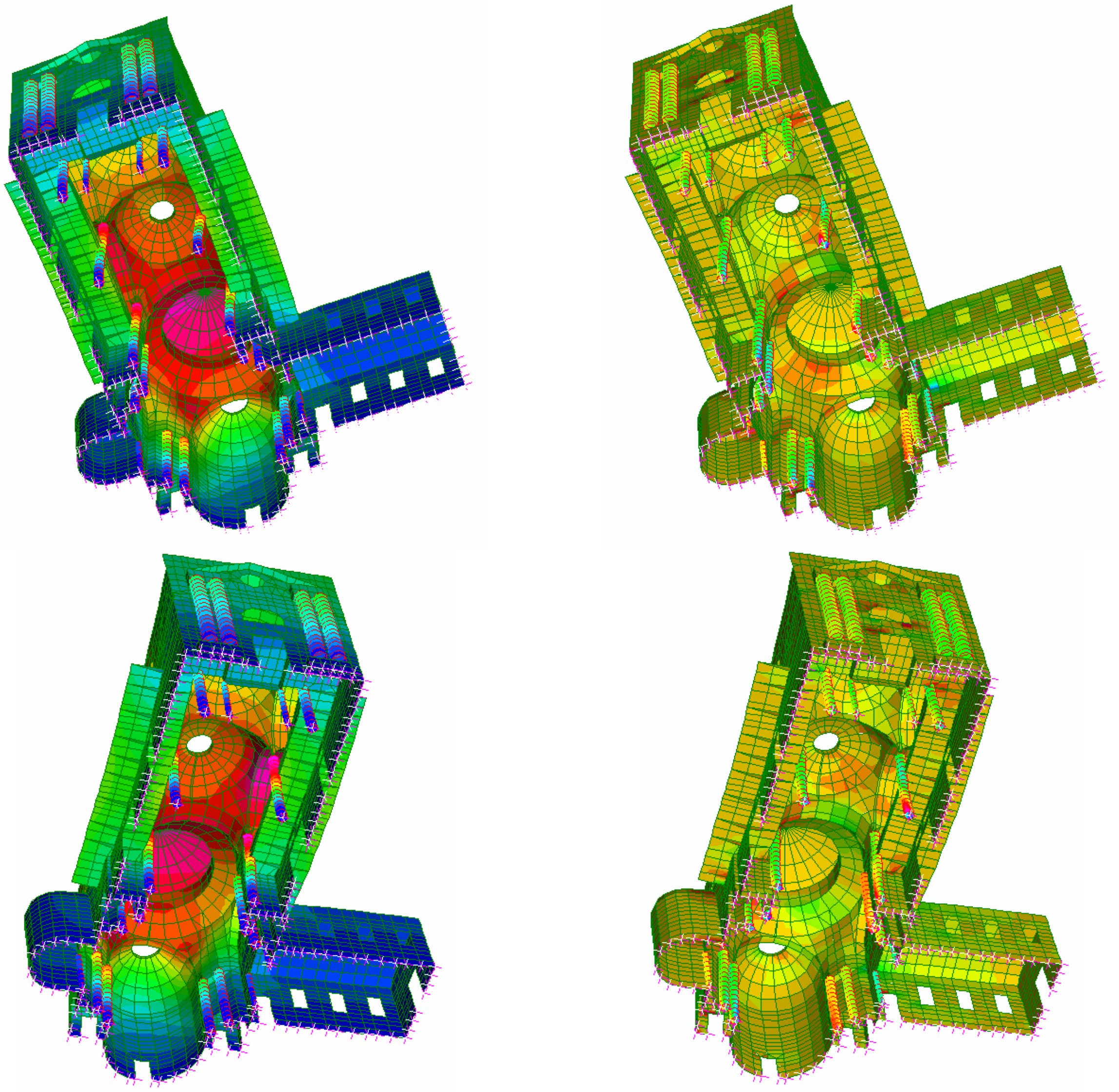


Figure 39b: Combination 2, Magnitude 8.5 Bilateral, 100% Z (East-West) direction + 30% X (North-South) direction.

10.5 Final remarks

The numerical simulation of Las Hermanas de la Providencia church considered a linear elastic Finite Element Model: the performed analysis had the aim of calculating the maximum element stresses due to the gravity load and the earthquake loads. For what concerns the static analysis of the gravity load, any structural element present a particularly high stress: the basis of the lateral walls and the arches and columns that support the dome have a higher vertical stress. Only the two central columns of the main nave present the maximum compressive-bending stress.
The Natural Frequency Analysis has been performed excluding local modes. The mode shapes and corresponding natural frequencies and participating masses of the structure denote a certain stiffness of the structure: they have been used for the Spectral Response Solver.
Taking into account the results of the heaviest considered seismic actions (Case 5, Case 6, Combination 1, Combination 2), the Spectral Response Analysis denote that the number of included natural modes is sufficient enough to adequately represent the dynamic behaviour of the structure. Some portions of the structure of the Church present a high status of stress, that is related to the observed cracks: in particular, it is worth noticing the tensile stresses that result in the West and South apse, near the round openings of the main dome, near the openings of the East and West apses and of the lateral walls of the main nave, in the triumphal arch between the presbytery and the South apse, between the main nave and the presbytery and between the two domes of the main nave, in the lateral arches of the main nave.

However, the results of the Spectral Response Analysis of the Las Hermanas de la Providencia Church are not always fully complying with the actual damage status: this is essentially due to the lack of knowledge about the Church's structure and materials. For example, the position of the steel bars in the reinforced concrete is not known and for this reason an reliable analysis of the resistance of the structural elements is not possible now.

For this reasons, the study accomplished in this report must be considered as a starting point, consistent with the reached level of knowledge, to be developed in the future: in particular, further development of the analysis of the Church can be

- an experimental campaign of investigation, that can include non destructive (NDT) and minor (MDT) destructive in situ tests (radar test for the position of the bars, dynamic identifications, etc.);
- the calibration of the full model on the basis of the results of the in situ tests;
- the seismic analysis of significant portions of the full model, that can be extracted and separately analyzed with non linear models and software codes.

In any case, the present damage situation of Las Hermanas de la Providencia must considered very worrying, because partial or total collapse (in several structural parts, due to widespread weakness) can occur in case of earthquake (i.e. medium to high magnitude seismic excitations, as expected in the Valparaiso area; moreover, the church is located in the X highest Intensity area, as shown by the 1906 seismic event); the church (declared unsafe after the damage subjected by the 1985 seismic event) is now almost completely closed.

Due to the very particular typology of the construction material (a primitive reinforced concrete very rare in the world), a strengthening intervention with conventional techniques can be ineffective or very invasive, but a strengthening solution should be planned only after a detailed design work. As a suggestion, an innovative solution can be imagined, in order to reduce drastically the seismic input, as the introduction of a base isolation system (with all the due precaution, avoiding elevation and foundation wall cutting, by means of the insertion of a new subfoundation system), that seems possible due to the apparent absence of a crypt.

11. CONCLUSIONS

11.1 Status of the churches

The work carried out on the three churches (San Francisco del Baron, Hermanas de la Providencia, La Matriz), even if it can be considered as a first work step to be deepened in the future throughout specific rehabilitation projects, allows to say the following:

- the present damage situation of the San Francisco Church must be considered very worrying, because partial or total collapse (especially in the bell-tower and in the façade) can occur in case of earthquake (i.e. medium to high magnitude seismic excitations, as expected in the Valparaiso area); in fact, the church is unsafe and urgently must be closed partially of totally, planning a strengthening intervention as soon as possible;

- the present damage situation of the Hermanas de la Providencia Church must be considered very worrying, because partial or total collapse (in several structural parts, due to widespread weakness) can occur in case of earthquake (i.e. medium to high magnitude seismic excitations, as expected in the Valparaiso area; moreover, the church is located in the X highest Intensity area, as shown by the 1906 seismic event); the church (declared unsafe after the damage subjected by the 1985 seismic event) is now almost completely closed;

- the situation of La Matriz Church is enough good from the seismic point of view; on the other hand, this building needs surely an improvement of fire protection, together with preservation measures against materials degradation and termite attack (in particular for wooden elements).

11.2 References for cultural heritage rehabilitation

In order to avoid a possible conflict between the conservation requirements prescribed for cultural heritage structures (integrity, compatibility, reversibility and durability) and the antiseismic improvement, the philosophical approach can be summarized in these following simple statements:

a) because cultural heritage structural rehabilitation problems are much more difficult to solve than those related to modern r. c. or steel structures, interventions can derogate from the antiseismic design criteria foreseen for ordinary buildings;
b) in relation to the state limit analysis, the intervention must be defined as a “controlled structural improvement”, i.e. accepting an antiseismic protection level lower than required, in order to reduce invasivity, but depending on the category of use and importance;
c) for each limit state, the improvement effectiveness must be quantified, evaluating the PGA (Peak Ground Acceleration) levels which generate the local collapse mechanisms, before and after the intervention;
d) because the cultural heritage structures characteristics (history, material properties, construction details, quality of connections, state of integrity and maintenance, etc.) are frequently not well known, detailed survey, damage assessment and diagnostic campaigns must be carried out, in order to reach a knowledge level as deeper as possible; moreover, each cultural heritage structure is different: therefore, it is necessary to undertake the rehabilitation design in a specific way, use of standardized procedures being not possible;
e) the observance of the “regola dell’arte”, i.e. the unwritten construction rules for masonry elaborated by architects and bricklayers in centuries of work practice, is fundamental for protection (good overall static and dynamic behavior), conservation (durability in after years) and restoration (avoiding irreversible mistakes); the use of modern techniques and materials can be very useful to reduce seismic vulnerability, but it must be philologically correct, compatible and mechanically effective.

Specific antiseismic guidelines and codes for the cultural heritage protection should be used; in particular, the following references are suggested for cultural heritage structural restoration:

- guidelines for evaluation and mitigation of seismic risk to cultural heritage, recently edited by the Italian Ministry for Cultural Heritage and Activities (July 21, 2006) **[17]**;
- International Standard ISO 13822, Assessment of Existing Structures;

- ICOMOS-ISCARSAH (International Scientific Committee for Analysis and Restoration of Structures of Architectural Heritage, UNESCO), 20051: Recommendations for the analysis, conservation and structural restoration of cultural heritage.

The application of the above said references for cultural heritage structural improvement is, in our opinion, mandatory. Thus, the choice of some emblematic projects, to be exploited in the framework of an International Chilean - Italian partnership, seems highly desirable, also with the aim to disseminate knowledge and experience.

11.3 Identification of the seismic input

Takin into account what said in paragraph 6., it is to worth noting that the deterministic approach has been followed in the "MAR VASTO" project to evaluate the seismic input in the Valparaiso area for certain earthquake scenarios (in general), and in some sections underneath the churches locations (in particular) **[08]**.
Another topic question is the identification of site amplification effects, due to geologic and topographic factors. Therefore, local seismic motion should be identified with accuracy, also performing micro-zoning surveys (to be improved in the Valparaiso urban area).
Another relevant aspect is the implementation of a reliable and widespread network, able to record in several points the seismic input (displacement, velocity and acceleration), not only in the field, but also at various levels of important structures, in order to gather indispensable data to enhance numerical analysis. For this reason, a logic step is to foresee the instrumentation of above said churches through monitoring systems, especially after a restoration intervention.

11.4 Knowledge of the structure

As anticipated before, cultural heritage structures characteristics (history, material properties, construction details, quality of connections, state of integrity and maintenance, etc.) are frequently not well known, due to their intrinsic complexity. On the other hand, a well done rehabilitation project should need basic data on geometry, structural features, construction details, damage, conservation, mechanical properties of materials, etc., in order to reach a knowledge level as deeper as possible.
The first steps to foresee are the execution of a detailed geometric survey and a reliable damage assessment, by using conventional or innovative (laser scanner) methods.
Diagnostic campaigns requires non destructive (NDT) or minor destructive (MDT) techniques, in order to avoid invasive tests, as follows:
- single flat jack tests, allowing to evaluate the in-situ stress level of the structural material;
- double flat jack tests, used to evaluate the deformability characteristic;
- shear pull out tests, consisting in the insertion of a tensile element (usually a steel bar) into a larger borehole; if used on different material portions, they aim to investigate the sliding behavior of the walls, identifying a local shear value "marking" the wall out-of-plane mechanism;
- borehole with video endoscopy, performed on elevation and foundation walls, giving a general stratigraphy of the wall section;
- sonic pulse velocity tests, based on the generation of sonic/ultrasonic impulses at a point of the structure, useful for different purposes, i.e. to qualify the material through the investigation of the wall section morphology, detect the presence of voids, and find crack and damage patterns.

- absorption tests, to be used to compare different products for mortar injections, aiming to set up the consolidation process parameters;
- mortar analyses, oriented to evaluate the mortar conservation state, identifying composition, resistance and degradation;
- construction details critical survey, which provides important data regarding the connection quality of bearing walls, effectiveness of wall-floor nodes, presence or lack of steel ties, stability of vaults and arches; similar results can be also carried out through the analysis of a generic transversal wall section, aiming to evaluate the voids percentage.

In-situ experimental campaigns for dynamic characterization (performed through ambient vibrations or impulse produced by an impact of a mass dropped on the ground close to the structure), are also recommended, in order to examine the motion in terms of modal shapes.

Both diagnostics and dynamic characterization tests are fundamental to calibrate the Finite Element Model, with the aim to obtain accurate outputs in structural calculations **[20]**.

In the case of the three churches, we can consider satisfactory the geometric survey, sufficient the damage assessment, while it was not possible to perform experimental tests (due to lack of resources and time). They must be done in any case if a rehabilitation project will be foreseen in the future.

The scarcity of experimental information was replaced by data taken from literature. A supplemental difficulty has been encountered for the Las Hermanas, due to the unicity of the constituent material (a primitive reinforced concrete); in this case, a conservative approach has been followed.

In relation to the depth of the structural knowledge, it is possible to assign a *confidence factor* F_C to be used in the numerical analyses **[17]**. In our case, due to the speedy level of knowledge reached, a penalizing F_C has been choosen.

11.5 Reasonable anticipations about future rehabilitation projects

a) San Francisco del Baron

This construction seems to be (in the façade and in the bell-tower) a very regular masonry brickwork, but showing heavy widespread structural damage and absence of antiseismic protections. The main intervention steps can be foreseen as follows:

- reinforcement of part or all the resistant elements, increasing selectively resistance, stiffness, ductility or a combination of these (always paying careful attention to induced modifications to the structural scheme); it can be done: increasing the strength of masonry, trough local repairs to cracked or deteriorated parts; reconstructing masonry unity in the most weak or deteriorated parts, utilizing materials with analogous physical-chemical and mechanical properties; common non-invasive techniques used in Italy are *rip and sew*, *injections of mixed bonding agents*, *redrafting the junctions*; the insertion of post-tightened vertical tie-rods is applicable only in specific cases and when the masonry has been proven to be able to support the increase in vertical load;

- insertion of new elements which are compatible with existing ones, eliminating local vulnerability of certain parts of the construction and improving the overall functionality in terms of resistance or ductility; it can be done mainly through the traditional technique, as the insertion of tie-rods (placed in the two horizontal directions of the structure, at the level of floors and in correspondence to bearing walls) anchored to the masonry; arches and vaults can be strengthened also using tie-rods (normally placed at the rear), put in place with adequate pre-solicitation; other methods (jaketing by concrete or strips of composite materials) should be evaluated with care.

b) Las Hermanas de la Providencia

Due to the very particular typology of the construction material (a primitive reinforced concrete very rare in the world), a strengthening intervention with conventional techniques can be ineffective or very invasive. In this case, an innovative solution can be imagined:

- introduction of a base isolation system (with all the due precaution, avoiding elevation and foundation wall cutting, by means of the insertion of a new subfoundation system), that seems possible due to the apparent absence of a crypt.

c) La Matriz

Very simple strengthening interventions can be done:

- insertion of new elements which are compatible with existing ones, eliminating local vulnerability of certain parts of the construction and improving the overall functionality in terms of resistance or ductility; it can be done mainly through the traditional technique, as the insertion of tie-rods (placed in the two horizontal directions of the structure, at the level of floors and in correspondence to bearing walls) anchored to the masonry; in our case, the horizontal tie-rods connecting façade and nave should be foreseen, in order to minimize out-of plane overturning. In addition, this building needs fire protection, preservation from materials degradation and termite attack.

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APPENDIX 1
Vulnerability sheets for La Matriz

Denomination of the church

Matriz

Terms of Use

Daily	■	weekly	□	Saltuario	□	Abandoned	□	Busy hours	□

Position

Isolated	□	In aggregate	□	Low-rise buildings	□	Extremities or corner	■

Level of maintenance.

Awful	□	Expiring	□	Reasonabl e	□	Good	■

Typological and dimensional data

Damage and vulnerabilty index

1- OVERTURNING OF THE FACADE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of longitudinal chains	□	□	□		
	□	■	Presence of effective elements of contrast (buttress, other building)	□	□	□		
	■	□	Good quality of scarf between the facade and the walls of the nave	■	■	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of thrusted elements	□	□	□		
	□	■	Presence of large openings in the side walls near the cantonal	□	□	□		
Damage	Current		Detach of the facade from the walls or clear off lead	■	■	■	□	□
	Old		Detach of the facade from the walls or clear off lead	■	■	■	□	□

2- OVERTURNING OF THE GABLE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of links with timely coverage	□	□	□		
	□	■	Presence of groundwater braces	□	□	□		
	□	■	Presence of orizzontal ledge	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (rosette)	□	□	□		
	□	■	Presence of a top sailing with a large size and weight	□	□	□		
	□	■	Rigid beam filled in c.a. Heavy coverage in c.a.					
Damage	Current		Inclined crack pattern (shear) - vertical crack pattern - Rotation of main couple	□	□	□	□	□
	Old		Inclined crack pattern (shear) - vertical crack pattern - Rotation of main couple	□	□	□	□	□

3- MECHANISMS IN PLAN OF FACADE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains in counter	□	□	□		
	□	■	Side contrast provided by bodies or smeli facade inserted in aggregate	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (also curtain wall)	□	□	□		
	□	■	High slenderness (ratio height / width	□	□	□		
Damage	Current		Inclined crack pattern (shear) - vertical crack pattern - Other cracks	■	■	□	□	□
	Old		Inclined crack pattern (shear) - vertical crack pattern - Other cracks	■	■	□	□	□

4- PROTHYRUM - NARTHEX

(The prothyrum is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains	□	□	□		
	□	■	Presence of columns, piers with adequate stiffness	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of pushing elements (arch, vault)	□	□	□		
Damage	Current		Crack pattern in the entablature for rotation of the columns - Detach comprehensive from the facade - Pounding of the protiro- Arches damaged	□	□	□	□	□
	Old		Crack pattern in the entablature for rotation of the columns - Detach comprehensive from the facade - Pounding of the protiro- Arches damaged	□	□	□	□	□

5- TRANSVERSAL ANSWER OF THE HALL

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of external buttresses	□	□	□		
	□	■	Presence of adjacent bodies annexes	□	□	□		
	□	■	Presence of transversal chains					
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of walls with high slenderness	□	□	□		
	□	■	Presence of vault and arches					
			(The vault is pretended)					
Damage	Current		Crack pattern in the arches (with the possible continuation in the vault) - Rotazioni wall - shear carck pattern in the vault - Out of lead and crushing	□	□	□	□	□
	Old		Crack pattern in the arches (with the possible continuation in the vault) - Rotazioni wall - shear carck pattern in the vault - Out of lead and crushing	□	□	□	□	□

			6- SHEAR MECHANISMS IN SIDE WAALS (LONGITUDINAL ANSWER)					
Vulnerability	yes	no	*Aseismatic presidi*					
	■	☐	Masonry uniform (the only constructive phase) and of good quality (Adobe)	■	■	☐		
	☐	■	Presence of good openings architraves	☐	☐	☐		
	☐	■	Presence of orizzontal ledge (reticular metal, masonry army)	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of large openings (buffered), wall with restricted thickness	☐	☐	☐		
	☐	■	Orizzontal ledge in AC, heavy coverage in AC.					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐
	Old		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐
			7- LONGITUDINAL ANSWER OF THE COLONNADE OF CHURCHES WITH SIDE AISLE					
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of longitudinal chains	☐	☐	☐		
	☐	■	Presence of buttresses in front or body appendages	☐	☐	☐		
				☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of large openings (buffered), wall with restricted thickness	☐	☐	☐		
	☐	■	Orizzontal ledge in c.a, heavy coverage in c.a.					
Damage	Current		Crack pattern in the arches - shear crack pattern in the side vault - Crush and / or crack on the base of the columns	☐	☐	☐	☐	☐
	Old		Crack pattern in the arches - shear crack pattern in the side vault - Crush and / or crack on the base of the columns	☐	☐	☐	☐	☐
			8- VAULT OF CENTRAL AISLE **(The vault is not carryng)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of chains into effective place	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of concentrated loads transmitted from the coverage	☐	☐	☐		
	☐	■	Presence of lunette with high dimension	☐	☐	☐		
Damage	Current		Crack pattern in the vault or disconnections	☐	☐	☐	☐	☐
	Old		Crack pattern in the vault aisle or disconnections	☐	☐	☐	☐	☐
			9- VAULT OF SIDE AISLES **(The vault is not carryng)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of chains into effective place	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of concentrated loads transmitted from the coverage	☐	☐	☐		
	☐	■	Presence of lunette with high dimension	☐	☐	☐		
Damage	Current		Crack pattern in the vault or disconnections	☐	☐	☐	☐	☐
	Old		Crack pattern in the vault or disconnections	☐	☐	☐	☐	☐
			10- OVERTURNING OF THE END WALLS OF TRANSEPT **(The transept is not present)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of longitudinal chains	☐	☐	☐		
	☐	■	Presence of effective elements of contrast (buttresses, smeli bodies, other buildings)	☐	☐	☐		
	☐	■	Good connection with the coverage (beam-chains, controventi)	☐	☐	☐		
	☐	■	Good interaction between the front wall and side walls	☐	☐	☐		
	☐	■	Presence of orizzontal ledge (reticular metal, masonry army, about subtle)	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence, beams filled AC, heavy coverage	☐	☐	☐		
	☐	■	Presence of large openings in the front wall (rosette), or in those side	☐	☐	☐		
	☐	■	Presence of a gable wall with a great size and weight	☐	☐	☐		
Damage	Current		Detach of the front wall from the side walls or overtuning in the top	☐	☐	☐	☐	☐
	Old		Detach of the front wall from the side walls or overtuning in the top	☐	☐	☐	☐	☐
			11- SHEAR MECHANISM IN THE WALL OF THE TRANSEPT **(The transept is not present)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Masonry uniform (the only constructive phase) and of good quality	☐	☐	☐		
	☐	■	Presence of good openings architraves	☐	☐	☐		
	☐	■	Presence of orizzontal ledge (reticular metal, masonry army)	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of large openings (buffered), wall with restricted thickness	☐	☐	☐		
	☐	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐
	Old		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐

	yes	no	12- VAULT OF THE TRANSEPT **(The transept is not present)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	□	□	□	□	□
	Old		Crack pattern in the vault or disconnections	□	□	□	□	□

	yes	no	13- TRIUMPHAL ARCH **(The triumphal arch is not present)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Effective enforcement walls (ratio light/width nave)	□	□	□		
	□	■	Presence of chains	□	□	□		
	□	■	Good ashlar or appropriate thickness	□	□	□		
	□	■	Presence of tympanum	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage in AC	□	□	□		
	□	■	Presence of dome or lantern	□	□	□		
Damage	Current		Crack in the arch - creep of ashlar - Crush on the base of piers	□	□	□	□	□
	Old		Crack in the arch - creep of ashlar - Crush on the base of piers	□	□	□	□	□

	yes	no	14- DOME AND THE TIBURIO **(We consider the 3 dome under the bell tower)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, even at multiple levels	□	□	□		
	□	■	Presence of external buttresses in the drume	□	□	□		
	□	■	Dome directly set on triumphal arches (no drum)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage in AC	□	□	□		
	□	■	Presence of large openings in drum	□	□	□		
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
Damage	Current		Crack in the dome with possible prosecution in drum (Crack alog the meridian of central dome)	□	□	□	□	□
	Old		Crack in the dome with possible prosecution in drum (Crack alog the meridian of central dome)	□	□	□	□	□

	yes	no	15- LANTERN **(The lantern is not present)**					
Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Presence of external reinforcement ring (internal reinforcement)	□	□	□		
	□	■	Presence of external buttresses	□	□	□		
	□	■	Size contained compared to those of dome	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Lantern with high slenderness, with large openings and small pillars	□	□	□		
Damage	Current		Crack in the dome of lantern - Rotazioni of piers	■	■	□	□	□
	Old		Crack in the dome of lantern - Rotazioni of piers	■	■	□	□	□

	yes	no	16- OVERTURNING OF THE APSE					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, of chains	□	□	□		
	■	□	Presence of external buttresses in the drume	■	■	□		
	■	□	Presence of braced coverage not pusher	■	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of strong weakening for the presence of openings in the walls	□	□	□		
	□	■	Presence of pushing vault	□	□	□		
	□	■	Presence of heavy coverage, strut of pitch in AC	□	□	□		
Damage	Current		Inclined or vertical crack in the wall of the apse	■	□	□	□	□
	Old		Inclined or vertical crack in the wall of the apse	■	□	□	□	□

	yes	no	17- SHEAR MECHANISM IN THE WALL OF THE APSE					
Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Masonry uniform (the only constructive phase) and of good quality	■	■	□		
	□	■	Presence of good openings architraves	□	□	□		
	□	■	Presence of orizzontal ledge (reticular metal, masonry army)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	□	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□
	Old		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□

18- VAULT OF APSE
(The vault is not carryng)

	yes	no						
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	□	□	□	□	□
	Old		Crack pattern in the vault aisle or disconnections	□	□	□	□	□

19- MECHANISMS OF ELEMENTS IN COVERAGE - SIDE WALLS SIDE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of orizzontal ledge in AC	□	□	□		
	■	□	Presence of local link between beams and walls	□	□	□		
	□	■	Presence of braced pitch	□	□	□		
	■	□	Presence of good connections between the elements of warping coverage	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of pushing coverage	□	□	□		
	□	■	Presence of orizzontal ledge, heavy coverage	□	□	□		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□

20- MECHANISMS OF ELEMENTS IN COVERAGE - TRANSEPT

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of orizzontal ledge in AC	□	□	□		
	□	■	Presence of local link between beams and walls	□	□	□		
	□	■	Presence of braced pitch	□	□	□		
	□	■	Presence of good connections between the elements of warping coverage	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of pushing coverage	□	□	□		
	□	■	Presence of orizzontal ledge, heavy coverage	□	□	□		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□

21- MECHANISMS OF ELEMENTS IN COVERAGE - APSE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of orizzontal ledge in AC	□	□	□		
	□	■	Presence of local link between beams and walls	□	□	□		
	□	■	Presence of braced pitch	□	□	□		
	□	■	Presence of good connections between the elements of warping coverage	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of pushing coverage	□	□	□		
	□	■	Presence of orizzontal ledge, heavy coverage	□	□	□		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□

22- OVERTURNING OF THE CHAPEL
(The chapel is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, of chains	□	□	□		
	□	■	Presence of external buttresses in the drume	□	□	□		
	□	■	Presence of good connections between frontal wall and side walls	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of strong weakening for the presence of openings in the walls	□	□	□		
Damage	Current		Detach from the frontal wall to the side walls	□	□	□	□	□
	Old		Detach from the frontal wall to the side walls	□	□	□	□	□

23- SHEAR MECHANISM IN THE WALL OF THE CHAPEL
(The chapel is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Masonry uniform (the only constructive phase) and of good quality	□	□	□		
	□	■	Presence of good openings architraves	□	□	□		
	□	■	Presence of orizzontal ledge (reticular metal, masonry army)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	□	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□
	Old		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□

24- VAULT OF THE CHAPEL
(The chapel is not present)

	yes	no						
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	□	□	□	□	□
	Old		Crack pattern in the vault or disconnections	□	□	□	□	□

25- INTERACTIONS NEAR OF PLANO-ALTIMETRIC IRREGULARITIES

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains	□	□	□		
	□	■	Presence of an adequate connection between the walls of different stages	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of high stiffness difference between the two bodies	□	□	□		
	□	■	Possible actions transmitted from Relay	□	□	□		
Damage	Current		Movement of the joint or crack in the masonry for hammering - vertical crack in the less rigid body - Rotation in the highest body	□	□	□	□	□
	Old		Movement of the joint or crack in the masonry for hammering - vertical crack in the less rigid body - Rotation in the highest body	□	□	□	□	□

26- OVERHANG (GABLE - SPIRES - PINNACLES - STATUTES)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of pins link with masonry or elements of restraint	□	□	□		
	□	■	Elements with limited importance and size	□	□	□		
	□	■	Monolithic masonry (a squared or otherwise of good quality)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Elements with high slenderness	□	□	□		
	□	■	Asymmetric location respect to the underlying element	□	□	□		
Damage	Current		Elevated permanent rotations or slide	□	□	□	□	□
	Old		Elevated permanent rotations or slide	□	□	□	□	□

27- BELL TOWER

Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Stell internal structure (the only constructive phase) and of good quality	■	■	□		
	■	□	Presence chains	■	■	□		
	□	■	Presence of adequate distance from the walls of the church (where adjacent)	□	□	□		
	□	■	Presence good connection with the walls of the church (if incorporated)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of significant openings on multiple levels	□	□	□		
	□	■	Constraint on asymmetrical walls to the base station (tower incorporated)	□	□	□		
	□	■	Masonry low to the ground on some sides (for porch), tower building on pillars	□	□	□		
Damage	Current		Crack near the detachment from the body of the church - shear crack or slider - vertical crack (expulsion of one or more corners)	□	□	□	□	□
	Old		Crack near the detachment from the body of the church - shear crack or slider - vertical crack (expulsion of one or more corners)	□	□	□	□	□

28- BELL CELL

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of piece piers	□	□	□		
	■	□	Presence of chains or reinforcemenet ring	■	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage or other significant masses	□	□	□		
	□	■	Presence coverage Pusher	□	□	□		
Damage	Current		Crack in the arches - rotation or sliding of piers	■	■	■	□	□
	Old		Crack in the arches - rotation or sliding of piers	■	■	■	□	□

NON SEISMIC DAMAGE

A- FOUNDATION SETTLEMENT								■			
Damage	Inclined crack 45° - vertical crack - rotation					■	■	□	□	□	
macroelement	front wall	□	side walls	■	transept	□	apse	□	bell tower	□	
B- CRUSHING OF WALLS								□			
Damage	Detach of masonry walls - extended vertical crack for crushing stress					□	□	□	□	□	
macroelement	front wall	□	side walls	□	transept	□	apse	□	bell tower	□	
C- ROTATION OF WALLS								■			
Damage	Out of plumb					■	■	□	□	□	
macroelement	front wall	□	side walls	□	transept	□	apse	□	bell tower	■	

APPENDIX 2
Vulnerability sheets for San Francisco del Baron

Denomination of the church

S. Francisco

Terms of Use

Daily	□	weekly	■	Saltuario	□	Abandoned	□	Busy hours	□

Position

Isolated	□	In aggregate	■	Low-rise buildings	□	Extremities or corner	□

Level of maintenance.

Awful	□	Expiring	□	Reasonabl e	■	Good	□

Crack pattern in the arches - shear crack pattern in the side vault - Crush and / or crack on the base of the columns

Typological and dimensional data

Damage and vulnerabilty index

			1- OVERTURNING OF THE FACADE					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of longitudinal chains	□	□	□		
	□	■	Presence of effective elements of contrast (buttress, other building)	□	□	□		
	□	■	Good quality of scarf between the facade and the walls of the nave	■	■	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of thrusted elements	□	□	□		
	□	■	Presence of large openings in the side walls near the cantonal	□	□	□		
Damage	Current		Detach of the facade from the walls or clear off lead	■	■	■	□	□
	Old		Detach of the facade from the walls or clear off lead	■	■	■	□	□

			2- OVERTURNING OF THE GABLE					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of links with timely coverage	□	□	□		
	□	■	Presence of groundwater braces	□	□	□		
	□	■	Presence of orizzontal ledge	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (rosette)	□	□	□		
	□	■	Presence of a top sailing with a large size and weight	□	□	□		
	□	■	Rigid beam filled in c.a. Heavy coverage in c.a.					
Damage	Current		Inclined crack pattern (shear) - vertical crack pattern - Rotation of main couple	■	■	■	□	□
	Old		Inclined crack pattern (shear) - vertical crack pattern - Rotation of main couple	■	■	■	□	□

			3- MECHANISMS IN PLAN OF FACADE					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains in counter	□	□	□		
	■	□	Side contrast provided by bodies or smeli facade inserted in aggregate	■	■	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of large openings (also curtain wall)	■	□	□		
	□	■	High slenderness (ratio height / width	□	□	□		
Damage	Current		Inclined crack pattern (shear) - vertical crack pattern - Other cracks	■	■	■	■	□
	Old		Inclined crack pattern (shear) - vertical crack pattern - Other cracks	■	■	■	■	□

			4- PROTHYRUM - NARTHEX					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains	□	□	□		
	□	■	Presence of columns, piers with adequate stiffness	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of pushing elements (arch, vault)	□	□	□		
Damage	Current		Crack pattern in the entablature for rotation of the columns - Detach comprehensive from the facade - Pounding of the protiro- Arches damaged	□	□	□	□	□
	Old		Crack pattern in the entablature for rotation of the columns - Detach comprehensive from the facade - Pounding of the protiro- Arches damaged	□	□	□	□	□

			5- TRANSVERSAL ANSWER OF THE HALL					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of external buttresses	□	□	□		
	■	□	Presence of adjacent bodies annexes	■	□	□		
	□	■	Presence of transversal chains					
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of walls with high slenderness	□	□	□		
	□	■	Presence of vault and arches					
			(The vault is pretended)					
Damage	Current		Crack pattern in the arches (with the possible continuation in the vault) - Rotazioni wall - shear carck pattern in the vault - Out of lead and crushing	□	□	□	□	□
	Old		Crack pattern in the arches (with the possible continuation in the vault) - Rotazioni wall - shear carck pattern in the vault - Out of lead and crushing	□	□	□	□	□

6- SHEAR MECHANISMS IN SIDE WAALS (LONGITUDINAL ANSWER)								
Vulnerability	yes	no	*Aseismatic presidi*					
	■	☐	Masonry uniform (the only constructive phase) and of good quality	☐	☐	☐		
	☐	■	Presence of good openings architraves	☐	☐	☐		
	☐	■	Presence of orizzontal ledge (reticular metal, masonry army)	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of large openings (buffered), wall with restricted thickness	☐	☐	☐		
	☐	■	Orizzontal ledge in AC, heavy coverage in AC.					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	■	■	☐	☐	☐
	Old		Inclined crack (single or cross) - Crack through local discontinuity	■	■	☐	☐	☐

7- LONGITUDINAL ANSWER OF THE COLONNADE OF CHURCHES WITH SIDE AISLE								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of longitudinal chains	☐	☐	☐		
	☐	■	Presence of buttresses in front or body appendages	☐	☐	☐		
				☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of large openings (buffered), wall with restricted thickness	☐	☐	☐		
	☐	■	Orizzontal ledge in c.a, heavy coverage in c.a.					
Damage	Current		Crack pattern in the arches - shear crack pattern in the side vault - Crush and / or crack on the base of the columns	☐	☐	☐	☐	☐
	Old		Crack pattern in the arches - shear crack pattern in the side vault - Crush and / or crack on the base of the columns	☐	☐	☐	☐	☐

8- VAULT OF CENTRAL AISLE (The vault is not carryng)								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of chains into effective place	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of concentrated loads transmitted from the coverage	☐	☐	☐		
	☐	■	Presence of lunette with high dimension	☐	☐	☐		
Damage	Current		Crack pattern in the vault or disconnections	☐	☐	☐	☐	☐
	Old		Crack pattern in the vault aisle or disconnections	☐	☐	☐	☐	☐

9- VAULT OF SIDE AISLES (The vault is not carryng)								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of chains into effective place	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of concentrated loads transmitted from the coverage	☐	☐	☐		
	☐	■	Presence of lunette with high dimension	☐	☐	☐		
Damage	Current		Crack pattern in the vault or disconnections	☐	☐	☐	☐	☐
	Old		Crack pattern in the vault or disconnections	☐	☐	☐	☐	☐

10- OVERTURNING OF THE END WALLS OF TRANSEPT (The transept is not present)								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of longitudinal chains	☐	☐	☐		
	☐	■	Presence of effective elements of contrast (buttresses, smeli bodies, other buildings)	☐	☐	☐		
	☐	■	Good connection with the coverage (beam-chains, controventi)	☐	☐	☐		
	☐	■	Good interaction between the front wall and side walls	☐	☐	☐		
	☐	■	Presence of orizzontal ledge (reticular metal, masonry army, about subtle)	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence, beams filled AC, heavy coverage	☐	☐	☐		
	☐	■	Presence of large openings in the front wall (rosette), or in those side	☐	☐	☐		
	☐	■	Presence of a gable wall with a great size and weight	☐	☐	☐		
Damage	Current		Detach of the front wall from the side walls or overtuning in the top	☐	☐	☐	☐	☐
	Old		Detach of the front wall from the side walls or overtuning in the top	☐	☐	☐	☐	☐

11- SHEAR MECHANISM IN THE WALL OF THE TRANSEPT (The transept is not present)								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Masonry uniform (the only constructive phase) and of good quality	☐	☐	☐		
	☐	■	Presence of good openings architraves	☐	☐	☐		
	☐	■	Presence of orizzontal ledge (reticular metal, masonry army)	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of large openings (buffered), wall with restricted thickness	☐	☐	☐		
	☐	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐
	Old		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐

			12- VAULT OF THE TRANSEPT (The transept is not present)					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
				□	□	□		
				□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	□	□	□	□	□
	Old		Crack pattern in the vault or disconnections	□	□	□	□	□

			13- TRIUMPHAL ARCH (The triumphal arch is not present)					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Effective enforcement walls (ratio light/width nave)	□	□	□		
	□	■	Presence of chains	□	□	□		
	□	■	Good ashlar or appropriate thickness	□	□	□		
	□	■	Presence of tympanum	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage in AC	□	□	□		
	□	■	Presence of dome or lantern	□	□	□		
Damage	Current		Crack in the arch - creep of ashlar - Crush on the base of piers	□	□	□	□	□
	Old		Crack in the arch - creep of ashlar - Crush on the base of piers	□	□	□	□	□

			14- DOME AND THE TIBURIO (We consider the 3 dome under the bell tower)					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, even at multiple levels	□	□	□		
	□	■	Presence of external buttresses in the drume	□	□	□		
	■	□	Dome directly set on triumphal arches (no drum)	■	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage in AC	□	□	□		
	□	■	Presence of large openings in drum	□	□	□		
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
Damage	Current		Crack in the dome with possible prosecution in drum (Crack alog the meridian of central dome)	■	■	■	■	■
	Old		Crack in the dome with possible prosecution in drum (Crack alog the meridian of central dome)	■	■	■	■	■

			15- LANTERN (The lantern is not present)					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of external reinforcement ring	□	□	□		
	□	■	Presence of external buttresses	□	□	□		
	□	■	Size contained compared to those of dome	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Lantern with high slenderness, with large openings and small pillars	□	□	□		
Damage	Current		Crack in the dome of lantern - Rotazioni of piers	□	□	□	□	□
	Old		Crack in the dome of lantern - Rotazioni of piers	□	□	□	□	□

			16- OVERTURNING OF THE APSE					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, of chains	□	□	□		
	□	■	Presence of external buttresses in the drume	□	□	□		
	■	□	Presence of braced coverage not pusher	■	■	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of strong weakening for the presence of openings in the walls	□	□	□		
	□	■	Presence of pushing vault	□	□	□		
	□	■	Presence of heavy coverage, strut of pitch in AC	□	□	□		
Damage	Current		Inclined or vertical crack in the wall of the apse	■	■	■	□	□
	Old		Inclined or vertical crack in the wall of the apse	■	■	■	□	□

			17- SHEAR MECHANISM IN THE WALL OF THE APSE					
Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Masonry uniform (the only constructive phase) and of good quality	■	■	□		
	□	■	Presence of good openings architraves	□	□	□		
	□	■	Presence of orizzontal ledge (reticular metal, masonry army)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	□	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□
	Old		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□

18- VAULT OF APSE (The vault is not carryng)								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of chains into effective place	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of concentrated loads transmitted from the coverage	☐	☐	☐		
	☐	■	Presence of lunette with high dimension	☐	☐	☐		
Damage	Current		Crack pattern in the vault or disconnections	☐	☐	☐	☐	☐
	Old		Crack pattern in the vault aisle or disconnections	☐	☐	☐	☐	☐

19- MECHANISMS OF ELEMENTS IN COVERAGE - SIDE WALLS SIDE								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of orizzontal ledge in AC	☐	☐	☐		
	■	☐	Presence of local link between beams and walls	☐	☐	☐		
	☐	■	Presence of braced pitch	☐	☐	☐		
	■	☐	Presence of good connections between the elements of warping coverage	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of pushing coverage	☐	☐	☐		
	☐	■	Presence of orizzontal ledge, heavy coverage	☐	☐	☐		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	☐	☐	☐	☐	☐
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	☐	☐	☐	☐	☐

20- MECHANISMS OF ELEMENTS IN COVERAGE - TRANSEPT								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of orizzontal ledge in AC	☐	☐	☐		
	☐	■	Presence of local link between beams and walls	☐	☐	☐		
	☐	■	Presence of braced pitch	☐	☐	☐		
	☐	■	Presence of good connections between the elements of warping coverage	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of pushing coverage	☐	☐	☐		
	☐	■	Presence of orizzontal ledge, heavy coverage	☐	☐	☐		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	☐	☐	☐	☐	☐
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	☐	☐	☐	☐	☐

21- MECHANISMS OF ELEMENTS IN COVERAGE - APSE								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of orizzontal ledge in AC	☐	☐	☐		
	☐	■	Presence of local link between beams and walls	☐	☐	☐		
	☐	■	Presence of braced pitch	☐	☐	☐		
	☐	■	Presence of good connections between the elements of warping coverage	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of pushing coverage	☐	☐	☐		
	☐	■	Presence of orizzontal ledge, heavy coverage	☐	☐	☐		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	☐	☐	☐	☐	☐
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	☐	☐	☐	☐	☐

22- OVERTURNING OF THE CHAPEL (The chapel is not present)								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Presence of reinforcement ring, of chains	☐	☐	☐		
	☐	■	Presence of external buttresses in the drume	☐	☐	☐		
	■	☐	Presence of good connections between frontal wall and side walls	■	■	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of strong weakening for the presence of openings in the walls	☐	☐	☐		
Damage	Current		Detach from the frontal wall to the side walls	☐	☐	☐	☐	☐
	Old		Detach from the frontal wall to the side walls	☐	☐	☐	☐	☐

23- SHEAR MECHANISM IN THE WALL OF THE CHAPEL (The chapel is not present)								
Vulnerability	yes	no	*Aseismatic presidi*					
	☐	■	Masonry uniform (the only constructive phase) and of good quality	☐	☐	☐		
	☐	■	Presence of good openings architraves	☐	☐	☐		
	☐	■	Presence of orizzontal ledge (reticular metal, masonry army)	☐	☐	☐		
	yes	no	*Indicators of vulnerability*					
	☐	■	Presence of large openings (buffered), wall with restricted thickness	☐	☐	☐		
	☐	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐
	Old		Inclined crack (single or cross) - Crack through local discontinuity	☐	☐	☐	☐	☐

24- VAULT OF THE CHAPEL

(The chapel is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
				□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	□	□	□	□	□
	Old		Crack pattern in the vault or disconnections	□	□	□	□	□

25- INTERACTIONS NEAR OF PLANO-ALTIMETRIC IRREGULARITIES

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains	□	□	□		
	□	■	Presence of an adequate connection between the walls of different stages	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of high stiffness difference between the two bodies	□	□	□		
	□	■	Possible actions transmitted from Relay	□	□	□		
Damage	Current		Movement of the joint or crack in the masonry for hammering - vertical crack in the less rigid body - Rotation in the highest body	□	□	□	□	□
	Old		Movement of the joint or crack in the masonry for hammering - vertical crack in the less rigid body - Rotation in the highest body	□	□	□	□	□

26- OVERHANG (GABLE - SPIRES - PINNACLES - STATUTES)

Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Presence of pins link with masonry or elements of restraint	■	□	□		
	□	■	Elements with limited importance and size	□	□	□		
	□	■	Monolithic masonry (a squared or otherwise of good quality)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Elements with high slenderness	□	□	□		
	□	■	Asymmetric location respect to the underlying element	□	□	□		
Damage	Current		Elevated permanent rotations or slide	□	□	□	□	□
	Old		Elevated permanent rotations or slide	□	□	□	□	□

27- BELL TOWER

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Masonry uniform (the only constructive phase) and of good quality	■	□	□		
	□	■	Presence chains	□	□	□		
	□	■	Presence of adequate distance from the walls of the church (where adjacent)	□	□	□		
	□	■	Presence good connection with the walls of the church (if incorporated)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of significant openings on multiple levels	□	□	□		
	□	■	Constraint on asymmetrical walls to the base station (tower incorporated)	□	□	□		
	□	■	Masonry low to the ground on some sides (for porch), tower building on pillars	□	□	□		
Damage	Current		Crack near the detachment from the body of the church - shear crack or slider - vertical crack (expulsion of one or more corners)	■	■	■	■	■
	Old		Crack near the detachment from the body of the church - shear crack or slider - vertical crack (expulsion of one or more corners)	■	■	■	■	■

28- BELL CELL

Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Presence of piece piers	□	□	□		
	■	□	Presence of chains or reinforcemenet ring	■	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage or other significant masses	□	□	□		
	□	■	Presence coverage Pusher	□	□	□		
Damage	Current		Crack in the arches - rotation or sliding of piers	■	■	■	□	□
	Old		Crack in the arches - rotation or sliding of piers	■	■	■	□	□

NON SEISMIC DAMAGE

A- FOUNDATION SETTLEMENT □

Damage	Inclined crack 45° - vertical crack - rotation							□	□	□	□ □
macroelement	front wall	□	side walls	□	transept	□	apse	□	bell tower	□	

B- CRUSHING OF WALLS □

Damage	Detach of masonry walls - extended vertical crack for crushing stress							□	□	□	□ □
macroelement	front wall	□	side walls	□	transept	□	apse	□	bell tower	□	

C- ROTATION OF WALLS ■

Damage	Out of plumb							■	■	□	□ □
macroelement	front wall	■	side walls	□	transept	□	apse	□	bell tower	□	

APPENDIX 3
Vulnerability sheets for Las Hermanas de la Providencia

Denomination of the church									
Les Hermanitas									
Terms of Use									
Daily	□	weekly	□	Saltuario	□	Abandoned	■	Busy hours	□
Position									
Isolated	□	In aggregate	□	Low-rise buildings	□	Extremities or corner	■		
Level of maintenance.									
Awful	■	Expiring	□	Reasonable	□	Good	□		
Typological and dimensional data									
Damage and vulnerabilty index									

1- OVERTURNING OF THE FACADE

	yes	no						
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of longitudinal chains	□	□	□		
	□	■	Presence of effective elements of contrast (buttress, other building)	□	□	□		
	□	■	Good quality of scarf between the facade and the walls of the nave	■	■	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of thrusted elements	□	□	□		
	□	■	Presence of large openings in the side walls near the cantonal	□	□	□		
Damage	Current		Detach of the facade from the walls or clear off lead	□	□	□	□	□
	Old		Detach of the facade from the walls or clear off lead	□	□	□	□	□

2- OVERTURNING OF THE GABLE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of links with timely coverage	□	□	□		
	□	■	Presence of groundwater braces	□	□	□		
	□	■	Presence of orizzontal ledge	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (rosette)	■	□	□		
	■	□	Presence of a top sailing with a large size and weight	■	□	□		
	■	□	Rigid beam filled in c.a. Heavy coverage in c.a.					
Damage	Current		Inclined crack pattern (shear) - vertical crack pattern - Rotation of main couple	■	■	□	□	□
	Old		Inclined crack pattern (shear) - vertical crack pattern - Rotation of main couple	■	■	□	□	□

3- MECHANISMS IN PLAN OF FACADE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains in counter	□	□	□		
	□	■	Side contrast provided by bodies or smeli facade inserted in aggregate	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (also curtain wall)	□	□	□		
	□	■	High slenderness (ratio height / width	□	□	□		
Damage	Current		Inclined crack pattern (shear) - vertical crack pattern - Other cracks	■	■	■	□	□
	Old		Inclined crack pattern (shear) - vertical crack pattern - Other cracks	■	■	■	□	□

4- PROTHYRUM - NARTHEX
(The prothyrum is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains	□	□	□		
	□	■	Presence of columns, piers with adequate stiffness	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of pushing elements (arch, vault)	□	□	□		
Damage	Current		Crack pattern in the entablature for rotation of the columns - Detach comprehensive from the facade - Pounding of the protiro- Arches damaged	□	□	□	□	□
	Old		Crack pattern in the entablature for rotation of the columns - Detach comprehensive from the facade - Pounding of the protiro- Arches damaged	□	□	□	□	□

5- TRANSVERSAL ANSWER OF THE HALL

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of external buttresses	□	□	□		
	□	■	Presence of adjacent bodies annexes	□	□	□		
	□	■	Presence of transversal chains					
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of walls with high slenderness	□	□	□		
	□	■	Presence of vault and arches					
			(The vault is pretended)					
Damage	Current		Crack pattern in the arches (with the possible continuation in the vault) - Rotazioni wall - shear carck pattern in the vault - Out of lead and crushing	■	■	□	□	□
	Old		Crack pattern in the arches (with the possible continuation in the vault) - Rotazioni wall - shear carck pattern in the vault - Out of lead and crushing	■	■	□	□	□

6- SHEAR MECHANISMS IN SIDE WAALS (LONGITUDINAL ANSWER)

Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Masonry uniform (the only constructive phase) and of good quality	□	□	□		
	■	□	Presence of good openings architraves	□	□	□		
	■	□	Presence of orizzontal ledge (reticular metal, masonry army)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	■	□	Orizzontal ledge in AC, heavy coverage in AC.					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	■	■	■	□	□
	Old		Inclined crack (single or cross) - Crack through local discontinuity	■	■	■	□	□

			7- LONGITUDINAL ANSWER OF THE COLONNADE OF CHURCHES WITH SIDE AISLE					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of longitudinal chains	□	□	□		
	□	■	Presence of buttresses in front or body appendages	□	□	□		
				□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	■	□	Orizzontal ledge in c.a, heavy coverage in c.a.					
Damage	Current		Crack pattern in the arches - shear crack pattern in the side vault - Crush and / or crack on the base of the columns	■	■	■	□	□
	Old		Crack pattern in the arches - shear crack pattern in the side vault - Crush and / or crack on the base of the columns	■	■	■	□	□

			8- VAULT OF CENTRAL AISLE					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	■	■	■	□	□
	Old		Crack pattern in the vault aisle or disconnections	■	■	■	□	□

			9- VAULT OF SIDE AISLES					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	■	■	■	□	□
	Old		Crack pattern in the vault or disconnections	■	■	■	□	□

			10- OVERTURNING OF THE END WALLS OF TRANSEPT					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of longitudinal chains	□	□	□		
	□	■	Presence of effective elements of contrast (buttresses, smeli bodies, other buildings)	□	□	□		
	□	■	Good connection with the coverage (beam-chains, controventi)	□	□	□		
	□	■	Good interaction between the front wall and side walls	□	□	□		
	□	■	Presence of orizzontal ledge (reticular metal, masonry army, about subtle)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence, beams filled AC, heavy coverage	□	□	□		
	■	□	Presence of large openings in the front wall (rosette), or in those side	□	□	□		
	■	□	Presence of a gable wall with a great size and weight	□	□	□		
Damage	Current		Detach of the front wall from the side walls or overtuning in the top	□	□	□	□	□
	Old		Detach of the front wall from the side walls or overtuning in the top	□	□	□	□	□

			11- SHEAR MECHANISM IN THE WALL OF THE TRANSEPT					
Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Masonry uniform (the only constructive phase) and of good quality	□	□	□		
	□	■	Presence of good openings architraves	□	□	□		
	■	□	Presence of orizzontal ledge (reticular metal, masonry army)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	■	□	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	■	■	■	□	□
	Old		Inclined crack (single or cross) - Crack through local discontinuity	■	■	■	□	□

			12- VAULT OF THE TRANSEPT					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
				□	□	□		
				□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	■	■	■	□	□
	Old		Crack pattern in the vault or disconnections	■	■	■	□	□

13- TRIUMPHAL ARCH

	yes	no						
Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Effective enforcement walls (ratio light/width nave)	□	□	□		
	□	■	Presence of chains	□	□	□		
	□	■	Good ashlar or appropriate thickness	□	□	□		
	□	■	Presence of tympanum	□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of heavy coverage in AC	□	□	□		
	■	□	Presence of dome or lantern	□	□	□		
Damage	Current		Crack in the arch - creep of ashlar - Crush on the base of piers	■	■	■	□	□
	Old		Crack in the arch - creep of ashlar - Crush on the base of piers	■	■	■	□	□

14- DOME AND THE TIBURIO

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, even at multiple levels	□	□	□		
	■	□	Presence of external buttresses in the drume	□	□	□		
	■	□	Dome directly set on triumphal arches (no drum)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage in AC	□	□	□		
	□	■	Presence of large openings in drum	□	□	□		
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
Damage	Current		Crack in the dome with possible prosecution in drum (Crack alog the meridian of central dome)	■	■	■	□	□
	Old		Crack in the dome with possible prosecution in drum (Crack alog the meridian of central dome)	■	■	■	□	□

15- LANTERN
(The lantern is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of external reinforcement ring	□	□	□		
	□	■	Presence of external buttresses	□	□	□		
	□	■	Size contained compared to those of dome	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Lantern with high slenderness, with large openings and small pillars	□	□	□		
Damage	Current		Crack in the dome of lantern - Rotazioni of piers	□	□	□	□	□
	Old		Crack in the dome of lantern - Rotazioni of piers	□	□	□	□	□

16- OVERTURNING OF THE APSE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, of chains	□	□	□		
	□	■	Presence of external buttresses in the drume	□	□	□		
	□	■	Presence of braced coverage not pusher	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of strong weakening for the presence of openings in the walls	□	□	□		
	■	□	Presence of pushing vault	□	□	□		
	■	□	Presence of heavy coverage, strut of pitch in AC	□	□	□		
Damage	Current		Inclined or vertical crack in the wall of the apse	■	■	■	□	□
	Old		Inclined or vertical crack in the wall of the apse	■	■	■	□	□

17- SHEAR MECHANISM IN THE WALL OF THE APSE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Masonry uniform (the only constructive phase) and of good quality	□	□	□		
	□	■	Presence of good openings architraves	□	□	□		
	□	■	Presence of orizzontal ledge (reticular metal, masonry army)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	□	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	■	■	■	□	□
	Old		Inclined crack (single or cross) - Crack through local discontinuity	■	■	■	□	□

18- VAULT OF APSE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	□	□	□	□	□
	Old		Crack pattern in the vault aisle or disconnections	□	□	□	□	□

19- MECHANISMS OF ELEMENTS IN COVERAGE - SIDE WALLS SIDE

	yes	no						
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of orizzontal ledge in AC	□	□	□		
	□	■	Presence of local link between beams and walls	□	□	□		
	□	■	Presence of braced pitch	□	□	□		
	□	■	Presence of good connections between the elements of warping coverage	□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of pushing coverage	■	■	□		
	■	□	Presence of orizzontal ledge, heavy coverage	■	■	□		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	■	■	■	□	□
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	■	■	■	□	□

20- MECHANISMS OF ELEMENTS IN COVERAGE - TRANSEPT

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of orizzontal ledge in AC	□	□	□		
	□	■	Presence of local link between beams and walls	□	□	□		
	□	■	Presence of braced pitch	□	□	□		
	□	■	Presence of good connections between the elements of warping coverage	□	□	□		
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of pushing coverage	■	■	□		
	■	□	Presence of orizzontal ledge, heavy coverage	■	■	□		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	■	■	■	□	□
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	■	■	■	□	□

21- MECHANISMS OF ELEMENTS IN COVERAGE - APSE

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of orizzontal ledge in AC	□	□	□		
	□	■	Presence of local link between beams and walls					
	□	■	Presence of braced pitch					
	□	■	Presence of good connections between the elements of warping coverage					
	yes	no	*Indicators of vulnerability*					
	■	□	Presence of pushing coverage	■	■	□		
	■	□	Presence of orizzontal ledge, heavy coverage	■	■	□		
Damage	Current		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□
	Old		Crack near to the heads of wooden beams, sliding them - connectionless between beam and masonry - Movements significant mantle - connectionless	□	□	□	□	□

22- OVERTURNING OF THE CHAPEL
(The chapel is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of reinforcement ring, of chains	□	□	□		
	□	■	Presence of external buttresses in the drume	□	□	□		
	□	■	Presence of good connections between frontal wall and side walls	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of strong weakening for the presence of openings in the walls	□	□	□		
Damage	Current		Detach from the frontal wall to the side walls	□	□	□	□	□
	Old		Detach from the frontal wall to the side walls	□	□	□	□	□

23- SHEAR MECHANISM IN THE WALL OF THE CHAPEL
(The chapel is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Masonry uniform (the only constructive phase) and of good quality	□	□	□		
	□	■	Presence of good openings architraves	□	□	□		
	□	■	Presence of orizzontal ledge (reticular metal, masonry army)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of large openings (buffered), wall with restricted thickness	□	□	□		
	□	■	Orizzontal ledge in AC, heavy coverage in AC					
Damage	Current		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□
	Old		Inclined crack (single or cross) - Crack through local discontinuity	□	□	□	□	□

24- VAULT OF THE CHAPEL
(The chapel is not present)

Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains into effective place	□	□	□		
				□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of concentrated loads transmitted from the coverage	□	□	□		
	□	■	Presence of lunette with high dimension	□	□	□		
Damage	Current		Crack pattern in the vault or disconnections	□	□	□	□	□
	Old		Crack pattern in the vault or disconnections	□	□	□	□	□

	yes	no	25- INTERACTIONS NEAR OF PLANO-ALTIMETRIC IRREGULARITIES					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of chains	□	□	□		
	□	■	Presence of an adequate connection between the walls of different stages	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of high stiffness difference between the two bodies	□	□	□		
	□	■	Possible actions transmitted from Relay	□	□	□		
Damage	Current		Movement of the joint or crack in the masonry for hammering - vertical crack in the less rigid body - Rotation in the highest body	□	□	□	□	□
	Old		Movement of the joint or crack in the masonry for hammering - vertical crack in the less rigid body - Rotation in the highest body	□	□	□	□	□

	yes	no	26- OVERHANG (GABLE - SPIRES - PINNACLES - STATUTES)					
Vulnerability	yes	no	*Aseismatic presidi*					
	■	□	Presence of pins link with masonry or elements of restraint	□	□	□		
	□	■	Elements with limited importance and size	□	□	□		
	□	■	Monolithic masonry (a squared or otherwise of good quality)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Elements with high slenderness	□	□	□		
	□	■	Asymmetric location respect to the underlying element	□	□	□		
Damage	Current		Elevated permanent rotations or slide	□	□	□	□	□
	Old		Elevated permanent rotations or slide	□	□	□	□	□

	yes	no	27- BELL TOWER (The bell tower is not present)					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Masonry uniform (the only constructive phase) and of good quality	□	□	□		
	□	■	Presence chains	□	□	□		
	□	■	Presence of adequate distance from the walls of the church (where adjacent)	□	□	□		
	□	■	Presence good connection with the walls of the church (if incorporated)	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of significant openings on multiple levels	□	□	□		
	□	■	Constraint on asymmetrical walls to the base station (tower incorporated)	□	□	□		
	□	■	Masonry low to the ground on some sides (for porch), tower building on pillars	□	□	□		
Damage	Current		Crack near the detachment from the body of the church - shear crack or slider - vertical crack (expulsion of one or more corners)	□	□	□	□	□
	Old		Crack near the detachment from the body of the church - shear crack or slider - vertical crack (expulsion of one or more corners)	□	□	□	□	□

	yes	no	28- BELL CELL (The bell tower is not present)					
Vulnerability	yes	no	*Aseismatic presidi*					
	□	■	Presence of piece piers	□	□	□		
	□	■	Presence of chains or reinforcemenet ring	□	□	□		
	yes	no	*Indicators of vulnerability*					
	□	■	Presence of heavy coverage or other significant masses	□	□	□		
	□	■	Presence coverage Pusher	□	□	□		
Damage	Current		Crack in the arches - rotation or sliding of piers	□	□	□	□	□
	Old		Crack in the arches - rotation or sliding of piers	□	□	□	□	□

NON SEISMIC DAMAGE

	A- FOUNDATION SETTLEMENT								□		
Damage	Inclined crack 45° - vertical crack - rotation						□	□	□	□	□
macroelement	front wall	□	side walls	□	transept	□	apse	□	bell tower	□	

	B- CRUSHING OF WALLS								□		
Damage	Detach of masonry walls - extended vertical crack for crushing stress						□	□	□	□	□
macroelement	front wall	□	side walls	□	transept	□	apse	□	bell tower	□	

	C- ROTATION OF WALLS								□		
Damage	Out of plumb						□	□	□	□	□
macroelement	front wall	□	side walls	□	transept	□	apse	□	bell tower	□	