Analysis of seismic design criteria of Santo Domingo Church, a Colonial Heritage of Santiago, Chile

Análisis de los criterios de diseño sísmico de la Iglesia de Santo Domingo, Patrimonio Colonial de Santiago, Chile

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Abstract
Santo Domingo church is analyzed as part of a broader research with the goal of reporting earthquake-resistant features in the masonry architectural heritage of the historic center of Santiago, Chile. Considering that Chile is one of the most seismic countries in the world, it is interesting how some historic buildings built between the XVI and the XVIII century still remain, despite construction with vulnerable building techniques such as unreinforced masonry. Among those buildings, Santo Domingo church is the only one built in stone ashlar and one of the few that has never suffered strong structural damages after earthquakes. Built between 1747 and 1771, this church has withstood around 11 earthquakes of magnitude over 7 without serious mechanical failures. Constructive and structural characteristics of Santo Domingo Church are assessed through historical research, field analysis and tests, damage assessments and finite element analysis. This analysis identifies early earthquake-resistant design criteria of the church, and determines vulnerable areas and their behavior during the last seismic events.

Key words: earthquake-resistant features, Colonial heritage, historical buildings, constructive-structural analysis, seismic vulnerability.

Resumen
El análisis de la iglesia de Santo Domingo forma parte de una investigación mayor, cuyo objetivo fue descubrir las estrategias de diseño sismorresistente presentes en el patrimonio arquitectónico construido en albañilería del centro histórico de Santiago. Considerando que Chile es uno de los países más sísmicos del mundo, es interesante constatar cómo algunos edificios históricos construidos entre los siglos XVI y XVIII aún existan, a pesar de haber sido construidos con técnicas vulnerables como lo son las albañilerías sin refuerzos. Entre aquellos edificios, la iglesia de Santo Domingo es la única construida en sillería de piedra y una de las pocas que nunca ha presentado daños estructurales severos durante los terremotos. La actual iglesia, construida entre 1747 y 1771, ha soportado alrededor de 11 terremotos sobre magnitud 7 sin presentar mecanismos de daño serios. A través de una investigación histórica, trabajo de campo y realización de ensayos, evaluación de daños y análisis de elementos finitos, las características constructivas y estructurales de la iglesia han sido determinadas. El análisis ha permitido identificar los tempranos criterios de diseño sismorresistente de la iglesia, así como determinar sus áreas vulnerables y su comportamiento durante los últimos eventos sísmicos.

Palabras clave: características sismorresistentes, patrimonio colonial, edificios históricos, análisis constructivo-estructural, vulnerabilidad sísmica.

Introduction and Description of the Problem

As part of the research 'Rediscovering Vernacular Earthquake-resistant Knowledge: Identification and analysis of best built practices in Chilean masonry architectural heritage' (2013-2016) (Research funded by the Chilean National Fund for Scientific and Technological Development- FONDECYT under Project Nº 11130628), this article presents a complete analysis of Santo Domingo church, one of the oldest masonry and structurally wellpreserved buildings in Santiago, with the particularity of having survived more than ten earthquakes over magnitude 7 without presenting any significant structural damage.

Chile (33°27'S, 70°40'W) is one of the most seismic places in the world, because of its location on the Pacific Ring of Fire where the subduction zones between tectonic plates concentrate major seismic and volcanic activity (Rauld, 2011). The capital city Santiago, also experiences high levels of seismicity, with *PGA* values of 55% gravity acceleration (*g*) for a return period of 475 years (Leyton, Ruiz & Sepúlveda, 2010), with sixteen earthquakes over magnitude 7 with epicenter near Santiago registered since the Spanish foundation of the city in 1541 (Chilean National Seismological Center, 2015). This seismicity caused considerable damage, especially to the first buildings erected by Spanish conquerors with unreinforced masonry techniques, which were destroyed by the first earthquakes. Nonetheless, some of those unreinforced masonry Colonial buildings still remain today, and serve as a testimony to the effort involved in erecting earthquake-resistant structures. The present research is the first to consider the characteristics of these surviving buildings. The hypothesis then, was that despite the intrinsic vulnerability of the unreinforced masonry techniques, the historical buildings of Santiago must have some special geometrical earthquake-resistant features in common, resulting from a long process of trial and error experimentation following each earthquake. Some of these characteristics have been touched upon by historians, such as the preference of 'massive forms, very thick walls and low ceilings' (Villalobos et al., 1990: 39) and were analyzed in-depth in the first two phases of the investigation through the use of simplified geometric parameters, including structural density, planned length-width and height-width ratios, percentage and location of wall openings, and vertical slenderness and free length of walls (Jorquera, Vargas, Lobos, & Cortez, 2016). However, while geometry is a factor that can be similar in many buildings, constructive and structural features of masonry buildings vary, because dynamic behaviour depends on many factors such as the connections between structural and non-structural elements, stiffness of horizontal diaphragms (Lourenço et al., 2011) and 'wall solidity' resulting from workmanship skills (De Felice, 2011).

In this context, Santo Domingo church must share some geometrical features with other historic churches in Santiago, but it also must have some particular constructive and structural characteristics. Combined, these elements can explain its good structural behaviour and its seismic design criteria. These characteristics were analysed through collaboration between the Department of Architecture of the Universidad de Chile and the Higher School of Architecture of the University of Malaga.

Methodology

The present research was divided into three phases:

- Phase 1 consisted of 'identification' considering approximately 70 buildings in the historic center of Santiago, sharing the following characteristics: built with unreinforced masonry; absence of serious damages due to earthquakes; absence of serious structural interventions that may alter structural behavior; and built between 1541 and 1860 (Jorquera et al., 2016).

- Phase 2 consisted of 'classification of buildings into typologies and comparative analysis' aimed to identify general rules of good structural behavior, through assessment of the aforementioned geometric parameters (Jorquera et al., 2016). Notably, all buildings considered are located inside the Colonial historic center of Santiago. This area was selected due to having the oldest buildings in the city, as well as a consistent type of soil which serves as a common denominator for comparative analysis. The soil is composed of sandy gravels from the Mapocho River with a particular firmness that reduces or limits amplification of seismic signalling from crust rock failures (Leyton et al., 2011).

- Phase 3 consisted of 'analysis of those buildings considered the best in terms of earthquake-resistant performance'. Three Colonial houses, six Colonial public buildings and seven churches were evaluated (Jorquera et al, 2016). Among the latter, the church of San Francisco of Santiago was given priority for being the oldest building in the city and thus has withstood the most earthquakes (Jorquera, Misseri, Palazzi, Rovero, & Tonietti, 2017; Jorquera & Soto, 2016). Santo Domingo church was also selected for analysis during this final phase, because it is one of the oldest buildings in the city, has never presented strong structural damages after earthquakes and is the only building in the city constructed with ashlar stone masonry.

It is important to mention that different analyses were made according to the special constructive and structural features of each building.

The analysis of Santo Domingo started by considering the historical development of the building and directly observing architectural and constructive features. Measuring instruments including a thermographic camera and metal detector were used to verify or discard the presence of reinforcements or other hidden constructive elements. Consequently, it was discovered that despite the good structural behaviour of the church, its two towers were retrofitted with concrete and the roof structure was replaced following a fire. Next, structural characterization and state of conservation were assessed. Direct observation and laser level were used to verify or discard geometric deformations, and the Schmidt Rebound Hammer test (RHT) was used to obtain some mechanical parameters of the walls 'based on the principle that the compressive strength of an elastic mass depends on the hardness of the surface against which sclerometer hits' (Chavez et al., 2014: 36). Finally, structural analysis with finite element modelling (FEM) was done, and subsequently compared with the whole analysis in order to draw conclusions regarding seismic design strategies of the church.

Historical background and typological characteristics

Santo Domingo is one of the few colonial churches still existing in Santiago, as well as 'one of best architectural confections' (Benavides, 1995: 55).

The church was initially built with fired brick and lime mortar in 1557, coinciding with the arrival of the Dominicans to Santiago, however was destroyed by the earthquake of 1595. The second construction began in 1606 and is considered a magnificent example of colonial architecture (Waisberg, 1975), however was totally ruined by the 'Magnum earthquake' of 1647, the strongest seism registered in Santiago. The third construction with 'fired brick and adobe masonry' (Secchi, 1941: 78) started around 1650 and was completed in 1677, but was destroyed by the earthquake of 1730, the second strongest of the Colonial period. Construction of the present fourth version started in 1747 under direction of the famous stonemason Juan de los Santos Vasconcellos, who decided to build it completely in stone ashlar in order to better withstand earthquakes. During construction, another earthquake in 1751 forced the original design proportions to be reduced to 'three naves the plan and two doors the elevation, while the central pinion was suppressed' (Pereira, 1965: 136). The church was inaugurated in 1771 without the two towers, which were finished in 1808. The façade was the work of the famous Italian architect Joaquín Toesca.

Santo Domingo has typological characteristics in common with the other seven colonial churches analysed (San Francisco, San Agustín, the Cathedral, La Merced, Santa Ana and Las Agustinas), such as:

- A basilica plant, with three naves where the central one is bigger than the lateral ones.
- Massif load bearing perimeter wall, while the separation between central and lateral naves is through columns and arcs.
- Campaniles towers often lighter than the rest of the building.
- Main façade wall thicker than the other perimeter walls, in order to avoid out-of-plane mechanism in case of seism.
- Roof structures made of timber trusses or false timber vaults (Jorquera et al., 2016).

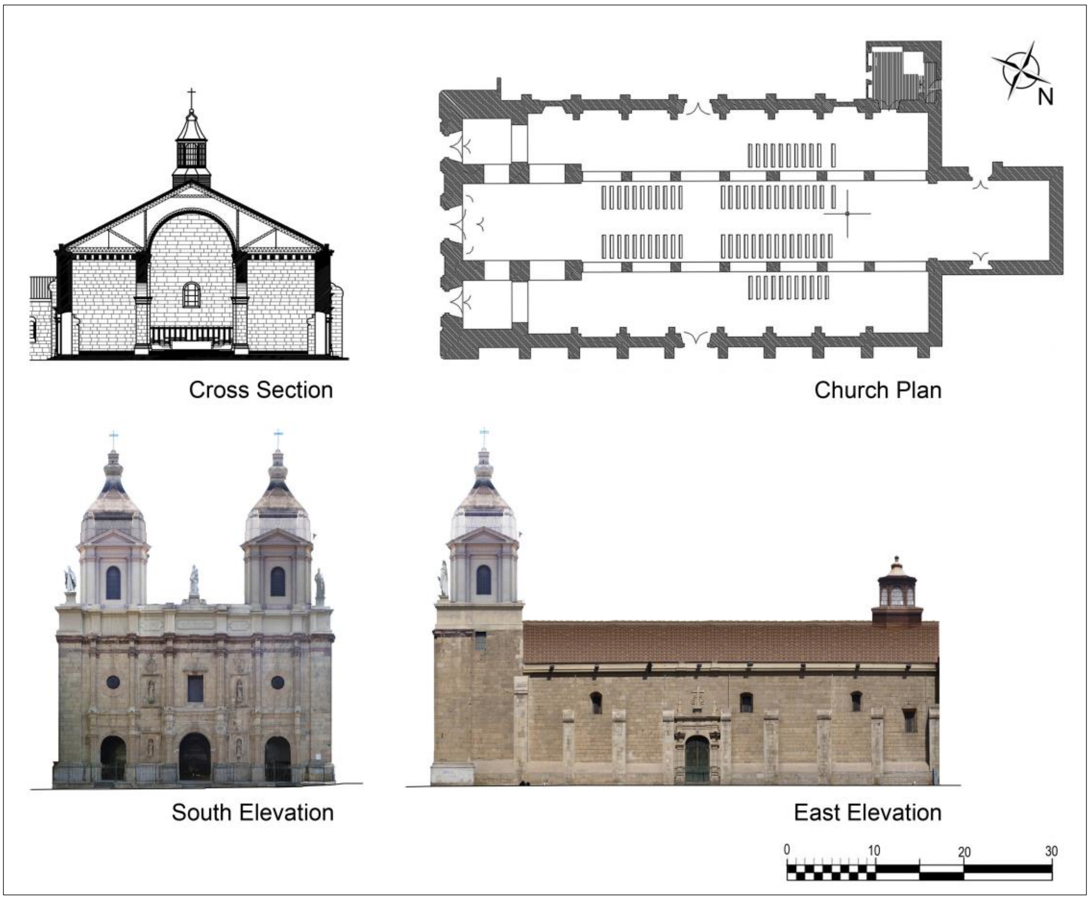
These architectural features of Santo Domingo have remained nearly the same over the centuries, except for the following changes:

- Part of the wooden ceiling was modified after a fire in 1895.
- After the earthquake of 1927, the architect Ricardo Larraín Bravo reinforced the two brick towers with concrete (Sahady, 2015).
- In 1963, a big fire destroyed the wooden structure of the roof, which was then replaced by a steel structure. In the same year, interior wall plasters were removed and so, the stone ashlar remained visible.
- In 2002, an underground parking was built under the eastern part of the church, provoking some damages to the base soil of the building.

Since its inauguration in 1771, the church has survived the earthquakes of 1822, 1850, 1851, 1906, 1909, 1927, 1965, 1971, 1985 and 2010, without strong damages (just small and superficial cracks) and without structural retrofitting in its stonewalls (just some mortars repairs).

Santo Domingo church consists of three bodies, built at different times and with different building systems (Figure 1).

Figure 1. Santo Domingo's church, section, plan and main elevations. Source: Ministerio de Obras Públicas de Chile (plans) & authors (photogrammetry).



The 'box' parallelepiped volume

The 'box' is formed by a limestone masonry volume of 67.1 m length, 27.8 m width, and 12.1 m height, with a higher part of 16.1 m under the towers. The perimeter wall is 1.5 m thick and 2.0 m thick in the main façade under the two towers. The volume is divided by two longitudinal axes of arcades that separate the central nave from the lateral ones (Figure 1 and Figure 2) and constitute the intermediate support of the roof structure. Currently, a concrete top collar beam is present on the top of the 'box', as part of the roof being changed after the fire in 1963.

The longitudinal walls have external and internal stone buttresses (Figure 2) of 1.1 m x 1.3 m in plan and 8.3 m high (they are lower than the perimeter wall), which prevent out-of-plane flexing of the walls during seismic thrusts.

A base plinth is present along all longitudinal walls, which help lower the center of gravity and reduce the height of the walls subjected to out-of-plane mechanism (Figure 2).

Among the seven churches analyzed, Santo Domingo and the church of La Merced were the only to present buttresses and base plinth in the original structure. These special devices were conceived from a long process of trial and error after the big earthquakes of 1647, 1730 and 1751.

Figure 2. Arcades and buttresses with base plinth of the church. Source: authors.

Vertical volumes

Two bell towers were built between 1771 and 1808, of 6.8 m x 6.8 m in plan and 20.5 m in height from the base.

At the beginning 'before 1750 [the towers] were projected with a third body between the base and the top, [but] they were modified in the way they are today, for fear of earthquakes' (Valenzuela, 1991: 270).

Unlike the rest of the church, the towers were built in fired brick masonry with lime mortar, with an outer plaster to provide protection from environmental conditions. This change of materials is probably given the need to simplify the work at height and reduce weight in the upper part of the building, to improve seismic response.

Concrete reinforcement of the towers starts at 9.6 m high, in the same position as the old choir. The reinforcement consists of beams, pillars and diagonals, semi embedded into the brick walls by their interior face, so they are not visible (Figure 3).

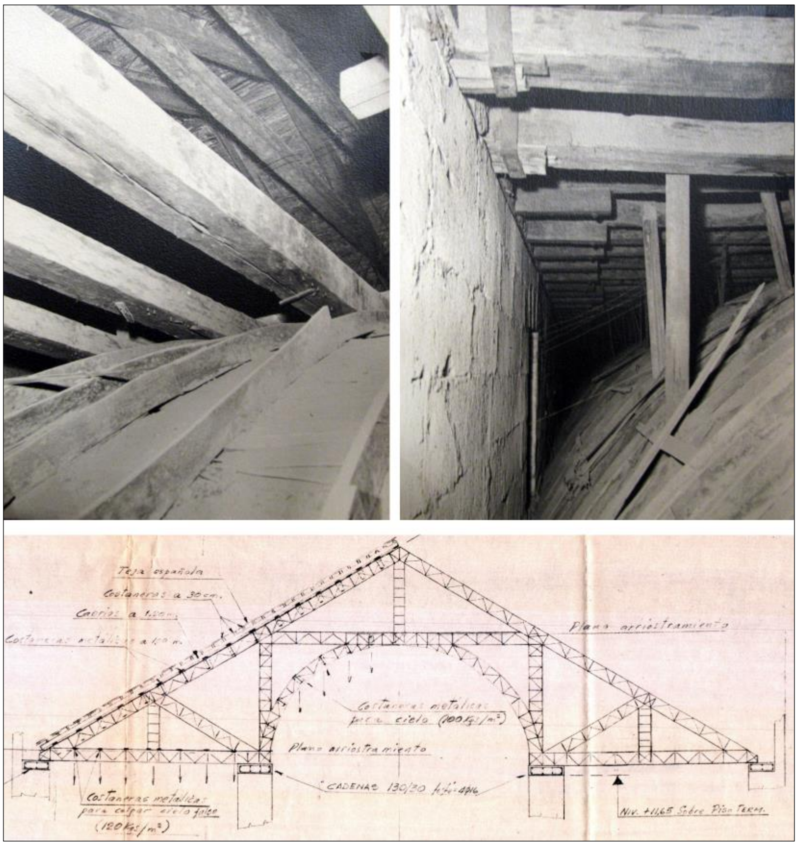
Figure 3. Scheme and image of towers reinforcements. Source: authors based on Ministerio de Obras Públicas de Chile plans.

Horizontal elements

The original roof structure was a truss made of native wood (oak, laurel and cinnamon), which stayed on top of the arcade walls, helping to counter horizontal thrusts. It was covered by a cane and clay tiles and had inferior wooden beams from which a timber false vault was hung. The junctions between wooden structure and stonewall were made of metal plates.

After a fire in 1963, the entire wooden structure was replaced by a new reticulated metal truss composed by steel bars forming an arch, tied longitudinally with beams of similar characteristics (Figure 4). The metal truss is a light structure with highly flexible behaviour and more strength than the old wooden structure. It rests on the concrete collar beam on the top of the stone walls and it is fixed by anchoring plates and profiles. The ceiling and the central skylight over the altar are also made of steel bars. Under current conditions, the entire roof structure acts as a diaphragm that transmits transverse thrusts of the sidewalls to the arcade walls.

Figure 4. Comparison between the original wooden roof structure and the steel new truss Source: Santo Domingo's Church library.



Qualitative approach to the Church earthquake-resistant design

As many authors affirm, 'seismic behavior of ancient masonry buildings is particularly difficult to characterize and depends on several factors, the materials properties, the geometry of the structure, the connections between structural and non-structural elements, the stiffness of the horizontal diaphragms and building condition' (Lourenço et al., 2011: 369). Thus, with the objective of having the first approach to earthquake-resistant strategies present in Santo Domingo church, and according to the aforementioned methodology, geometry and constructive features were analyzed.

Earthquake-resistant geometry

Analysis of the geometric simplified parameters that influenced seismic response of the masonry buildings (Bazán and Meli, 1985; Arnold and Reitherman, 1991; Cruz, 1995; Lourenço et al., 2013; Mendes and Lourenço, 2013; Jorquera et al., 2016) indicates that Santo Domingo has a geometry that has positively influenced its dynamic behavior, both in global capacity and local in-plane and out-of-plane capacity of walls. Santo Domingo church is symmetric and has a simple and regular very horizontal shape; length/width ratio of the building in plan is 2; height/width ratio of the building in façade is 0.6; structural density of the building in plan is 22%; the perimeter walls are 10.2 m in height (from the base-plinth) and 1.5 m thick, resulting in a slenderness of 6.8; the distance between the buttresses (4.2 m) and perimeter walls results in a free length of 2.8. Openings represent just 8% of the total wall area and are well located, far away from corners. All these values indicate a good global capacity of the building to face seismic thrusts, as well as a big local in-plane capacity of walls and an adequate capacity to avoid the out-of-plane failure mechanism of walls. Table 1 compares these values with the average values of the other six analyzed Colonial churches in Santiago (Jorquera et al., 2016), and refers to values taken from the mentioned literature. From this, it is possible to see that Santo Domingo shares geometric characteristics with the other churches –which are also inside the reference values-, but has a better capacity to affront the out-of-plane mechanism. This is on account of its slow slenderness and free-length of walls, and because of the base plinth and buttresses.

Concerning the horizontal thrust that could be created by the longitudinal arcades dividing the three naves, it is important to affirm that they present thick walls in their extremes that control the thrusts.

The towers and skylight are the elements most vulnerable to seismic actions, as they function like an independent appendix from the rest of the building, with a different vibration period given their higher location, increasing their susceptibility to out-of-plane overturning. Moreover, the skylight breaks the continuity of the roof structure and in its vibration generates tensions at its base.

Table 1. Geometric earthquake-resistant values of Santo Domingo. Source: authors.

Parameters	Santo Domingo values	Average values of other Santiago's Colonial churches	Reference from literature
Ratio length/width of the building in plan	2	2.32	≤2
Ratio height/width of the building in façade	0.6	0.47	≤4
Density of structure	22%	20.4%	around 20%
Thick of principal structural walls	1.5m	2.03 m	>0.35 m
Percentage of openings in walls	8%	7.56%	<40%
Vertical slenderness of walls (height/thickness)	6.8	8.09	≤9
Free length of the wall (length/thickness)	2.8	7.8	≤7

Constructive features

Santo Domingo church was built mainly with ashlar stone pulled from a quarry at Cerro Blanco hill in Santiago, with large blocks measuring 50 cm x 50 cm x 100 cm, that present homogeneity, regularity of shape, and courses unique among all the analyzed churches. The big stone blocks are laid both in stretchers and headers in order to provide an adequate connection between orthogonal walls and ensure monolithic behavior in front of perpendicular thrusts. Furthermore, by analyzing the brickwork of external and internal surfaces, it is possible to see the presence of headers that provide an appropriate transverse bond for the walls.

According to laboratory tests, the stones are composed of limestone with around 2.35 g/cm^3 density, joined with a very thin mortar of lime. Stone resistance varies from 380-550 Kg/cm^2 in the interior walls and 220-600 Kg/cm^2 in the exterior walls, according to Schmidt Hammer test. Thermography and steel detector tests indicate no metal reinforcements in the stone walls.

Besides, the church presents adequate connections between perpendicular walls and adequate wall-to-floor connections. Furthermore, although the original wooden truss and ceiling timber vault did not provide adequate wall-to-roof connections, the current steel truss of the roof and the concrete top collar beam are well connected. These characteristics contribute to the 'box' behavior of the building.

Structural analysis

3D Structural model

At a constructive level, historical masonry structures are mainly heterogeneous. The presence of materials with different mechanical properties, such as stone or mortar joints, implies analysis must be developed with an adequate strategy to represent the masonry structural behaviour. Therefore, the masonry building is analysed using three main modelling strategies (Lourenço, 2002): micro-modeling, where units and mortar in the joints are represented by continuum elements, whereas the unit mortar interface is represented by discontinuous elements (Lourenço & Rots, 1997; Orduña & Lourenço, 2005); macro-modelling where all components are smeared in a homogeneous continuum (either isotropic or orthotropic) being material properties obtained through available experimental data (Lourenço, de Borst & Rots, 1997); homogenized model by using a fictitious homogeneous orthotropic material at a structural level, deducing mechanical properties from a suitable boundary value problem solved on an appropriate unit cell (Luciano & Sacco, 1998; Milani, Lourenço & Tralli, 2006).

Since other strategies are more suitable for small size models, macro-modelling procedure was considered applicable mainly due to the large scale of the building considered in the present study (Lourenço & Pina-Henriques, 2006). Moreover, this approach has proved successful among many heritage case studies, allowing to obtain accurate results used in the static and dynamic calculations, as well as linear and non-linear ranges (Betti & Vignoli, 2011; Milani et al., 2012; Pintucchi & Zani, 2014).

In order to analyze structural behavior under static and dynamic loads, a 3D FE model was made, considering the different resistant planes of masonry (ashlar), the lateral buttresses, the lineal elements which configure the roof steel frame, and the concrete reinforcement of both towers.

Seismic assessment of existing buildings under dynamic earthquake actions at ultimate state can be performed based on nonlinear modelling of structural behaviour. Considering nonlinear behaviour, it is possible to evaluate the capacity of the structure to dissipate energy in the post-elastic field. Nevertheless, for structures with low geometrical complexity, it is also possible to perform linear analysis, supplying this calculation reliable information about the failure mechanism of the structure (DPCM, 2007; Jury et al., 2017). Additionally, this is validated because after a first iteration under self-weight actions, the general modes of the building, including towers, are not independent, verifying no elements have distinctly different behaviour (Ceroni, Pecce, & Manfredi, 2009).

Consequently, structural dynamic analysis is developed considering a linear modal approach based on the inelastic response spectrum defined by Chilean standard NCh-433 (INN, 2009). The analysis was performed using the commercial software SAP2000 (Computers & Structures Inc., 2015). This software, based on the finite element method, provides accurate and reliable results for dynamic structure analysis (Bangash, 2011; Pasticier, Amadio & Fragiacomo, 2008).

The whole model was divided into 2144 elements, which allowed obtaining sufficiently accurate results and simultaneously avoiding excessive calculation processes. Definition of the elements considered the need to analyze wall behaviour against bending, as well as the shear effect (Milani et al., 2012).

Material properties

As cited previously, macro-models are applicable when a building has sufficiently large dimensions as well as stresses distributed homogeneously along structural elements. Wall structure of Santo Domingo presents sufficiently large dimensions to ensure stress is substantially uniform across elements and will not suffer sudden changes due to their intrinsic properties (Modena, Lourenço & Roca, 2005). The error in global analysis of the behavior of these types of buildings under macro-modelling approach with respect to the micro-modelling does not exceed 10% (Binda et al., 2006).

Mechanical characterization of in situ materials is one of the most important issues in safety assessment of existing buildings, especially for masonry structures. To obtain a deeper knowledge of the constructive and structural components of the church, a survey based on non-destructive-testing was developed, as described in a previous section. From that survey, intrinsic properties of wall elements were defined considering different constituent materials such as mortar joints and blocks (Lourenço, 1995). This procedure was applied to define the properties of stone and brick masonry, in the main walls and on top of the main façade and towers, respectively.

In addition, to describe mechanical properties of the roof steel frames built after the 1963 fire, typical values related to the steel used in the bars and described in usual standards have been used, considering its properties analogous to S235. Similarly, the concrete reinforcement structure of the towers, built after 1927 earthquake, were defined by typical values related to the materials used and described in technical data of the project, specifically, f_{ck}=17.5 N/mm^2.

Properties used to define different structural elements of the building are described in Table 2.

Table 2. Materials properties used to FEM calculation. Source: authors.

	E_x (kN/m^2)	E_y (kN/m^2)	E_z (kN/m^2)	G_x (kN/m^2)	G_y (kN/m^2)	G_z (kN/m^2)	ϑ	ρ (kN/m^3)
Stone masonry	5515340	5566320	4027100	702240	729260	852440	0.30	23.5
Brick walls	938550	952710	1151800	427910	436550	505250	0.25	18.0
Roof steel framework		210000000			81000000		0.30	78.5
Concrete reinforcement structure (towers)		28000000			10769231		0.20	25.0

Soil properties where the building is located were obtained from a geotechnical survey made for the construction of the underground parking '21 de Mayo' in 2002, next to the eastern wall of the church. Geotechnical profiling characterized the soil as moderately dense or firm (type D) (INN, 2009). This presents a speed of transverse elastic waves or shear (Vs30) between 180 and 350 m/s. This data was used to determine the adequate response spectrum.

Load cases and boundary conditions

To assess the safety of Santo Domingo church, vertical loads (self-weight and live loads) and seismic horizontal loads were considered.

Self-weight vertical loads from structural elements included in the model were defined as a function of the corresponding specific weight of materials (Table 2), applied as a gravity load. In addition, relevant vertical gravity loads coming from constructive elements were added to the model: lightweight roof over nave (1.5 kN/m^2); false vault over the main nave and false ceiling over lateral naves (1.2 kN/m^2).

Seismic load was obtained from the probabilistic scenario defined by the specifications of the current Chilean standard NCh-433 (INN, 2009) and the Rules for seismic design of buildings in Chile (MINVU, 2011) that modified the previous Standard after the 2010 earthquake. This standard establishes two basic methods of calculation: static analysis and dynamic analysis (response spectrum analysis). Specifically, the response spectrum analysis method can be applied to structures that present classical normal modes of vibration, with modal damping representing around 5% of critical damping. As an alternative, this standard enables a static analysis where seismic action is equated with a system of horizontal forces applied at the center of mass for each element in the system, according to NCh433 (INN, 2009). Given the particularities of the evaluated building, a dynamic analysis of the structure based on the definition of the response spectrum is considered (Augusti, Ciampoli & Giovenale, 2001; Barbat, Oller & Vielma, 2005). Therefore, a spectrum defined by NCh433 is obtained and programmed into the FEM software, with its effect considered along the transversal (X) and longitudinal (Y) directions.

Considering the determined set of load cases, load combinations were defined according to provisions established by NCh3171 Chilean Standard 'Structural design – General provisions and load combinations' (INN, 2010). Specifically, the following have been studied: a) 1.4 D; b) 1.2 D + 1.4 E + L; c) 0.9 D + 1.4 E, being D (dead loads), L (live loads), E (earthquake loads) respectively. Additionally, according to the criteria established in different seismic standards (i.e. Spanish Standard for Earthquake Resistant Buildings) (Ministry of Development, 2002), a building must withstand the horizontal action of the earthquake in all directions. Consequently, seismic actions must be considered as acting in multiple directions, which generally implies analysis of two orthogonal directions. In this context, it is required to combine the effects of vertical loads and X seismic action, as well as vertical loads and Y seismic action. To adequately combine its influence on structural behavior, the effects of seismic loads as obtained from the multi-directional analysis are combined with 30% of the seismic loads of the other direction.

Regarding support conditions, it is possible to consider the fixture of the building to the ground, as well as the soil-structure interaction, accounting for the direct influence of soil behavior under seismic loads. This second option could be done by including a sufficient portion of the soil under the building into the FEM model, creating the condition of deformable foundation soil. Likewise, this can be done in a simplified but effective way by replacing the fixed supports with springs, as spring stiffness values depend on soil properties (Kramer, 1996;

Veletsos, 1975). Analysis was performed considering the effects of soil-structure interaction into the models generate larger displacements than those models considered fixed. On the other hand, fixed support hypothesis produces higher stress values on structural elements because of the solid-rigid behavior of the building. This implies reducing deformations, while on the contrary increasing the value of the stresses that structural elements must support (Díaz Guzmán et al., 2012; Kuhlemeyer & Lysmer, 1973; Stewart & Fenves, 1998). Consequently, the damage scenarios obtained in a condition of deformable foundation soil are different from those obtained in the hypothesis of a fixed base support. The condition of deformable foundation soil produces suitable and accurate results especially for non-linear dynamical models. However, as mentioned before, the dynamic structural analysis of the building is developed considering a linear modal approach based on the response spectrum. In this case, the hypothesis of fixed base support is considered appropriate to achieve accurate results regarding the structural behavior of the church.

Structural analysis

A linear static analysis considering vertical (dead and live) loads and modal analysis are performed using different combinations of loading configurations. This allows evaluation of the global response of the building structure, thereby obtaining both the stress and displacements.

Once the calculations were completed, a preliminary analysis of modal behavior was performed. Table 3 lists the periods, frequencies, mass participating ratio, modal masses and modal participation factors of the 10 first eigenmodes obtained. The fourth column shows that 75% of the church mass is activated by the fifth eigenmodes (acting along the x-axis), reaching 78% at tenth. Considering the action along the y-axis, the fifth column shows that, until the fourth eigenmode, just 52% of the mass is activated, obtaining 80% at the sixth and 80% at the tenth.

Table 3. Natural periods (T), natural frequencies (f), mass participating ratio (M), modal mass (MM) and modal participation factors (F) for the first 10 modes. Source: authors.

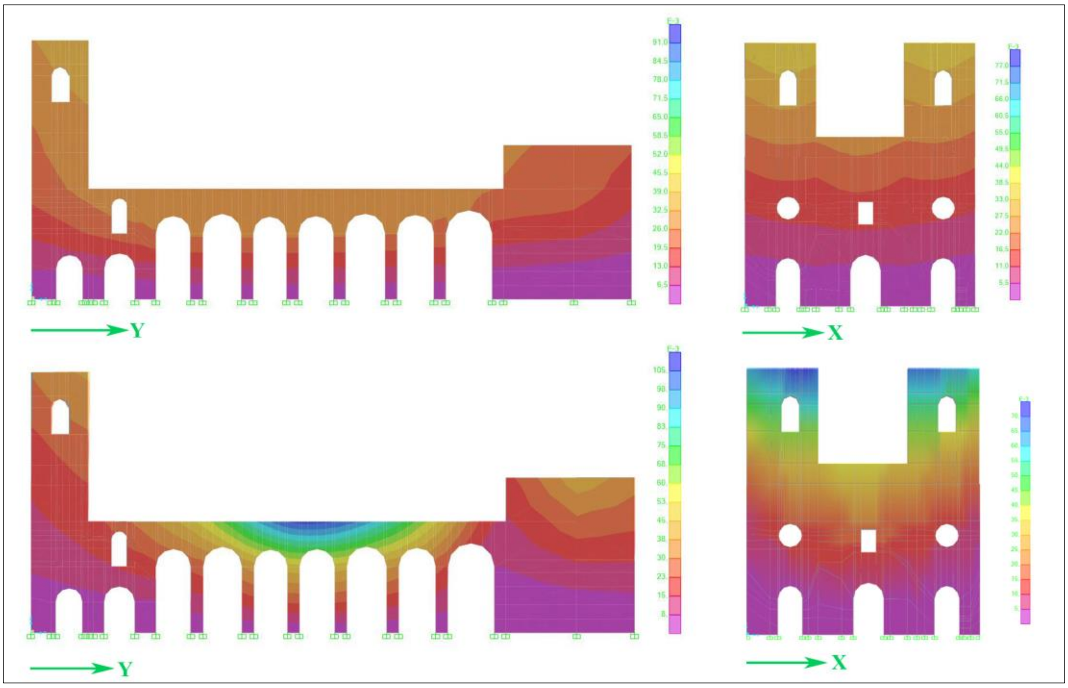
Mode	T (sg)	f (Hz)	Mx (%)	My (%)	MMx (kN)	MMy (kN)	Fx (%)	Fy (%)
1	0.579	1.72	73.57	0.0003705	8.18E+04	4.12E-01	-273.88	0.61
2	0.473	2.11	0.03	45.179	4.11E+01	5.02E+04	6.14	214.61
3	0.463	2.15	0.15	7.385	1.67E+02	8.21E+03	-12.36	86.76
4	0.393	2.54	0.70	0.001493	7.80E+02	1.66E+00	-26.75	-1.23
5	0.352	2.83	1.52	0.001846	1.69E+03	2.05E+00	-39.38	-1.37
6	0.328	3.04	0.001716	27.458	1.91E+00	3.05E+04	-1.32	167.31
7	0.291	3.42	0.00001967	0.273	2.19E-02	3.03E+02	-0.14	-16.68
8	0.285	3.49	0.001703	0.006135	1.89E+00	6.82E+00	1.31	2.50
9	0.270	3.70	2.211	1.694E-06	2.46E+03	1.88E-03	-47.47	-0.04
10	0.256	3.90	0.192	0.0002971	2.13E+02	3.30E-01	-14.00	0.55
			78.39	80.3				

Although participating mass along x and y axis is similar, the first ten vibration modes report a maximum participating mass of 80% (y-axis). In this context, a local analysis of the building should be developed to obtain a more comprehensive evaluation, especially considering the elements that affect global behavior results, such as the skylight over the transept. However, this is beyond the scope of the present study since, as mentioned, the main objective was to evaluate seismic strategies of Santo Domingo church.

Therefore, as a main objective is to estimate the seismic-resistant effect of different constructive elements present in the design of the church, a global analysis was performed to obtain the complete response of the church considering different load combinations.

In order to identify areas where seismic loads produce maximum stress, the first step was to analyze the locations of maximum displacements. As expected, highest displacements were verified in the upper part of the towers (39 mm – X), in the central area of side walls and central arcade (33 mm - X), as well as on the steel structure of the skylight over the transept (315.7 mm - X). The high deformation shown by this element reinforces the idea of its local analysis. Moreover, to compare the bracing effect of elements such as the steel roof, concrete tower reinforcement or sidewall buttresses, the structure has also been analysed without taken them into consideration. Maximum displacements on the tower vary from 39 mm to 70 mm. Similarly, the displacement on the upper part of central arcade increases from 33 mm to 105 mm (Figure 5).

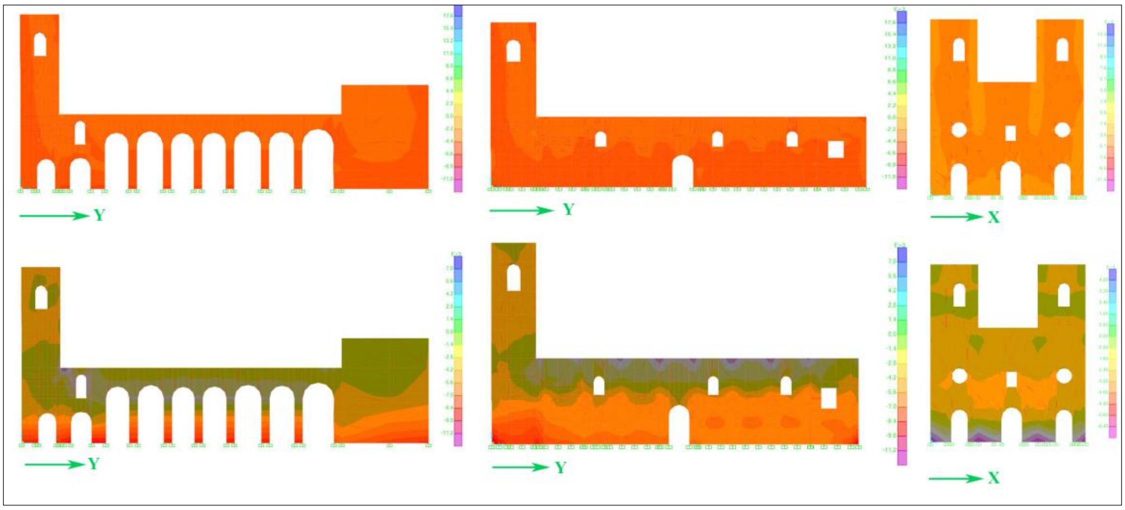
Figure 5. Displacements central arcade of the temple and main façade. Up: Displacement taking into account the bracing of the roof. Left: X Displacement under X seismic action; Right: Y displacement under Y seismic action. Down: Displacement without the bracing effect of the roof. Left: X Displacement under X seismic action; Right: Y displacement under Y seismic action. Source: authors



Maximum compression and tensile stresses caused by seismic actions are shown in Figure 6. By analyzing stress on the masonry it was possible to verify maximum admissible compression stress is not exceeded at any point of the structure. Conversely, there are several points where the value of admissible tensile stress is exceeded. In a probabilistic scenario, as defined by the NCh433-2009 Chilean standard (INN, 2009), an earthquake could produce cracking on these points, due to the appearance of tensile stresses. These points are located mainly at the base of the towers, at the intersection of the West and East walls façade. These tensile tensions appear even considering the concrete reinforcements. The stiffness of these elements reduces tower displacements but also causes distortion at a stress level, creating additional tensile stresses due to the disconnection effect between the masonry and concrete reinforcements (Vicente et al., 2012). This is especially significant on the main façade of the church where the effect of concrete columns is particularly meaningful. Moreover, it is relevant to highlight the interaction between old and new structural elements (since the XXth century): highest tensile stress appears on the capping of the West and East façade walls, showing the interaction between the anchorage of steel trusses of the roof and the concrete beam located over the walls. This effect can also be observed over the central arcade walls.

From the perspective of structural configuration, it is important to emphasize the positive effect of the bracing applied by the interior and exterior buttresses on the sidewalls. Its position reveals the limit where tensile stresses appear, additionally reducing wall displacement. Moreover, the arrangement of the walls at the head of the church, as well as in the chancel and transept, imply reinforcement against horizontal forces. The plan of the church (Figure 1) reveals an increasing thickness on the first two bays at the head of the naves where the height of the arches is also reduced, in comparison with the rest of the arcade. The thickness is maintained along the height of the walls. Similarly, the thickness of the walls increases at the end of the lateral naves in the “L” at the transept and chancel intersection. These solutions, together with the effect of the main chapel walls as a blind extension of the arcade, imply an increase of stiffness in weaker places against horizontal actions (Ruiz, Jaramillo & Mascort, 2014). This effect is augmented by the volume of the sacristy located next to the north-west transept, although this element was incorporated recently, in 1972. Removing the effects of these elements on the model, considering the same thickness as the remaining walls, it is possible to verify both the increase of displacements and stresses on the walls. This consequence is especially relevant regarding the head wall, which is one of the weakest points of the structure on account of the weakness of the façade wall, considering the thrust of arcade walls and the tower slenderness. The global consideration of these elements directly related to the design of the church allows to confirm their contribution to the box-behaviour of the building (Lourenço et al., 2011).

Figure 6. Left: Central arcade main stresses (S11, up y S22, down) under X seismic action; Centre: East façade main stresses (S11, up y S22, down) under X seismic action; Right: main façade stresses (S11, up y S22, down) under Y seismic action (right). Source: authors.



State of conservation assessment

Santo Domingo church presents little significant damage, considering its age, the frequent earthquakes, context changes and interventions it has suffered.

According to direct analysis using a vertical laser level, the perimeter walls are perfectly plumb, which demonstrates its good strength and stability. Besides, there are no cracks or separation between perpendicular walls or between walls and buttresses, demonstrating the good constructive connections of the stone brickwork. Furthermore, there are no shear cracks or out-of-plane deformations. Just some little fissures are present in the mortar joints near the openings, as a consequence of the movement of the upright stones lintels.

Only the pillars of the arcades present displacements (1.5-2 cm), mainly in the eastern axis, which cannot be considered serious as it represents just 1.5% of the width and 0.37% of the height of the pillars. This can be provoked by the horizontal thrusts of the roof that extend from the walls to the arcs.

There are some cracks in the church pavement due to the differential settlement generated during construction of the underground parking garage on the side of the building. Besides, diagonal cracks can be seen in the centre of the nave and in the altar area, evidencing that the new pavement, which acts a rigid diaphragm, has caused torsion twists during earthquakes. The concentration of cracks in the pavement coincides with the displaced pillars in the northeastern part of the church. Therefore, that corner of the building concentrates the major seismic tensions.

Regarding the tower, despite the concrete reinforcement, some fissures appeared in the first section of the wall of brick masonry after the 2010 earthquake. However, the fissures do not cross the wall and can be assumed as minimal damage, considering both towers are vertical plumb without cracks or separations between perpendicular walls.

Finally, the roof structure has remained stable without presenting damage or structural deformation. However, in the lower perimeter of the skylight, there are some fissures in the plaster as a result of the varying rigidities of materials, as seen in the FEM analysis.

After the last earthquake in 2010, there were no structural damages to the church, and so, just some minor repairs were made, such as the incorporation of mortars based on epoxy resins in lintels, arches, beneath the towers and part of the walls, with a similar resistance according to the characteristics of the limestone. Besides, some metal plates were incorporated in the lintels of the upper part of the church, to prevent displacement of the stones.

Conclusions

The current Santo Domingo church is the result of a long process of experimentation, including three failed attempts and thus, the present fourth version is an accumulation of all the empirical knowledge provided by past earthquakes. All its analyzed characteristics have helped the building adequately resist the seismicity of Santiago, as demonstrated by the absence of serious damages after earthquakes and its good conservation status. Consequently, and following analysis, Santo Domingo church can be considered as having an 'earthquake-resistant' design, based on:

- An adequate geometry (suitable size ratios of structural and architectural elements), best among the analyzed Santiago churches, and aligning with recommendations in the literature.
- Efficient construction techniques: the big stone ashlars, good brickwork and adequate connections, make it possible to consider the church as a continuous solid, presenting 'box behaviour' during earthquakes. For this reason, no damages have been registered related to separation of perpendicular walls. This continuity is only interrupted at the openings –most significantly around the lintels- where big tensions are created and appear as micro-cracks.
- Presence of special earthquake-resistance devices, such as the buttresses and base plinth reinforcement, reduce the free-length and slenderness of sidewalls and help avoid out-of-planes mechanisms. The FEM analysis evidenced the important role of these elements.
- Interventions, such as concrete reinforcement of the towers, have improved connection to the base. Furthermore, the structure of the steel roof seems to contribute to the general seismic performance. Nevertheless, the most relevant tensile stresses appear at the contact points between these reinforcements, as well as at the masonry walls on account of the differences between the Young's modulus and stiffness of these materials, which could generate structural disconnections. This does not occur in the stone walls of the façade where the reinforcements start, as stone is able to contain the stresses.

All mentioned features have helped the church survive an important number of earthquakes without substantial damages. In effect, the FEM analysis affirms the good design of the church and the importance of certain elements, such as buttresses, in its structural behavior. Therefore, these analyzed earthquake-design features should not be changed and should serve as a basis for generating guidelines and preventive measures in case of future earthquakes. Nonetheless, the church presents some vulnerable elements that must be monitored, such as the skylight, which is the part of the structure that suffers the most flexion against seismic thrusts. The arcades and some lintels have also presented displacements.

Finally, it must be mentioned that uninterrupted use and maintenance of the church is another important aspect that has influenced its good state of conservation, and thus, the seismic response of the building.

The analysis of Santo Domingo church confirms that 'the value of architectural heritage is not only in its appearance, but also in the integrity of all its components as a unique product of the specific building technology of its time' (ICOMOS, 2003: 1). The geometric, constructive and structural characteristics of Santo Domingo church are a testimony to the effort of Chilean builders to erect structures able to withstand strong earthquakes, and therefore, this is a dimension that must be enhanced as part of the cultural heritage of Chile.

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