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THE CHURCH OF SAN FRANCISCO IN SANTIAGO, CHILE. ANALYSIS OF 400 YEARS OF EARTHQUAKE-RESISTANCE BEHAVIOUR

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Abstract

A research collaboration between the Department of Architecture of the Universidad de Chile and the Department of Architecture of University of Florence (Italy) was carried out on the Church of San Francisco in Santiago, Chile, the oldest building in the city. This research is included in the research project 'Rediscovering Vernacular Earthquake-resistant Knowledge: Identification and analysis of build best practice in Chilean masonry architectural heritage' (2013-2016), funded by the Chilean National Fund for Scientific and Technological Development, FONDECYT.

The Church of San Francisco, located in the historic centre of Santiago, is an architectural heritage and urban symbol of the city. This is the reason why it is recognized as a Historic Monument, protected by the Chilean Law of Monuments since 1951, and is also included in the 'tentative list' to be presented to the UNESCO because of its Outstanding Universal Value.

Another important value, which has not been recognized yet is the earthquake-resistant behavior of the building, considering that it was built in 1586 with fragile building techniques (stone, brick and adobe masonry) and that it has had many constructive transformations throughout its 400 years. In fact, the church of San Francisco was the only surviving building in Santiago from the 1647 earthquake -the most destructive of the Colonial period- and after that, it has faced many earthquakes, exhibiting only local structural damages. Thus, the hypothesis of the research is that San Francisco has some constructive and structural 'secrets' resulting from a long experimentation after earthquakes, which explain the significant structural resilience of the building and which had not been fully investigated before. To this purpose, this research has used a multi-level approach that comprises historical research, in situ surveys, crack pattern analysis and local and global structural analysis, all which is presented in this paper, in order to discover the characteristics that are the basis of good mechanical behavior of the church in such a seismic context as Chile.

Keywords: Chilean earthquakes, seismic vulnerability assessment, stone masonry, Chilean Colonial architecture, heritage

1. The church of San Francisco: origin and constructive transformations

The church of San Francisco (Fig.1) was built in stone-masonry between 1586 and 1618, in a Latin-cross plan with a tower attached. It is the only 'authentic architectural testimony of the sixteenth century preserved in Chile' [1]. Besides, it was the only surviving building in Santiago after the most destructive earthquake of the Colonial period, the 'Magnum earthquake' of 1647 (magnitude 8.5), in which the church lost just its original tower and part of the choir [2], thus, it is the oldest building of the capital.
During the Seventeenth and the Eighteenth-century, four lateral chapels were added to the building, for which big arches were inserted in the stonewalls in order to connect the chapels with the original nave. Despite these transformations, the church survived the earthquake of 1730 (magnitude 8.7) without serious damage, and that of 1751 (magnitude 8.5), in which it lost its second tower built in 1698. As a result of the 1851 seism, the church lost its exterior cornices [2]. Additionally, its third tower built in 1758 was damaged and it had to be replaced in 1857 by the current one. This last tower was built by the famous Chilean architect Fermin Vivaceta, and it was made of wood in order to be lighter and to guarantee a better seismic performance. Vivaceta also replaced the four chapels with two brick aisles segmented by transversal brick arcs, with which the church acquired the

basilica plan [3]; after that, at the beginning of the Nineteen century, the eastern part of the church, behind the altar, was built, an intervention that gave the church its current shape (Fig.2)

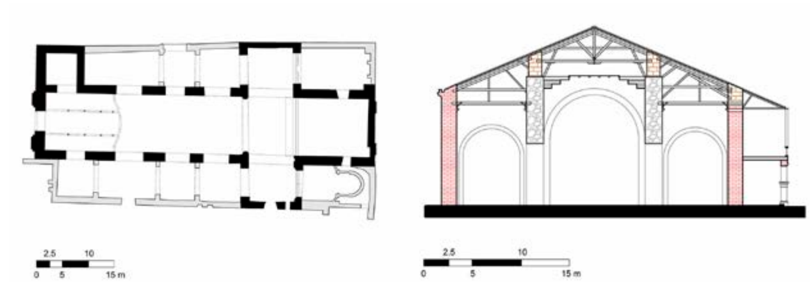

Fig. 1- View of the main façade of the church.

Fig. 2- Current plan and section of the church

Besides the aforementioned earthquakes, along its history the church has survived another ten earthquakes of magnitude between 7.1 and 9.5 [4] [5], most of them with an epicenter far away from Santiago, but still perceived with great intensity in the city, provoking damages in many buildings. In all these earthquakes there was no officially declared structural damage, but the repairs present in the walls of the church put in evidence that the building has suffered some local damage.

After the earthquake of 1985 (magnitude 7.7, with an epicenter at San Antonio near Santiago, and perceived VIII Mercalli intensity in Santiago) the church presented extended damages to its transversal brick-arcs of the lateral aisles, which were then reinforced in 1988 with concrete by the engineer Santiago Arias [6]. This has been the most important structural intervention of the church along its history. After the 2010 earthquake (magnitude 8.8° and VIII Mercalli intensity in Santiago), the church presented significant damage, such as the displacement of the intrados of its arches, some deep cracks in its longitudinal stonewalls, and wall bulging.

Nowadays, the church maintains its original use and the same architectural morphology since the addition of the lateral aisles, and it is one of the main historical monuments of the capital Santiago.

The comprehension of the historical development of the building has been useful to understand its seismic strengths and vulnerabilities.

2. Architectural and constructive features

The church consists of different parts, built at different times with different constructive systems and materials.

The plan is a typical basilica plan, 64.6 m in length, 30.3m wide, and 12m in height.

The original central nave walls are built in stone masonry and are 1.7 m thick; on the top of the wall some courses of adobe masonry were added to increase the wall height. The lateral nave walls are built in brick masonry and are 1.00 m thick on the North and South perimeter. The façade and the wall behind the altar are built in stone masonry and are 1.85 m and 1.7 m thick respectively; changes of materials in their gable are testimonies of ancient earthquake damage and subsequent repairs.

The stone-masonry central walls were bored to create arches and, therefore, they are not able to face dynamic actions well, showing dangerous cracks and deformation of some arches (Fig.3).

Fig. 3- Deformation of an arch and dangerous cracks in its base.

The lateral aisles have three couples of transverse arches built in fired-brick, conceived to reduce its length and to achieve a better transversal stiffness. These were reinforced in concrete, as mentioned.
The stones that constitute the original walls of the church are large blocks of limestone of irregular shapes and variable dimensions, from 40x40cm to 90x90cm in their visible surface. They are joined with a lime mortar, which is filled with small stones (Fig.4), improving the homogeneity and the cohesion of the wall. The resistance of the stone is high (though this is not the only decisive factor in determining the final strength of the masonry); it goes from 430-580 Kg/cm², according to Rebound test and Monotone uniaxial compression test. The fired-bricks of the lateral aisles are regular, with dimensions of 40x22x7cm and their superficial strength is about 140-180 Kg/cm². They are also joined with a lime mortar.

The tower is 46.4m high and is divided in three bodies: the base is built from rubble stone masonry and belongs to the original part of the church; the second part is built from bricks, and the third part is built as a wooden frame.

The oldest part of the church exhibits a system of foundations constituted by round river boulder stones under the walls, contained laterally by an axis of large and hewed stones, which was observed during an archeological excavation. The round river boulders, probably placed under all the original part of the church, could help to partially isolate the building from the seismic action.

Since its origin, the church was equipped with a big horizontal diaphragm, placed on the main nave, constituted by a series of big wooden beams (30x35 cm cross section), placed very close, at an interaxis of 120 cm, and well connected to the walls (Fig.5). This original diaphragm still exists nowadays even if it was bored to create the skylight over the altar.

Fig. 4- Stone masonry.

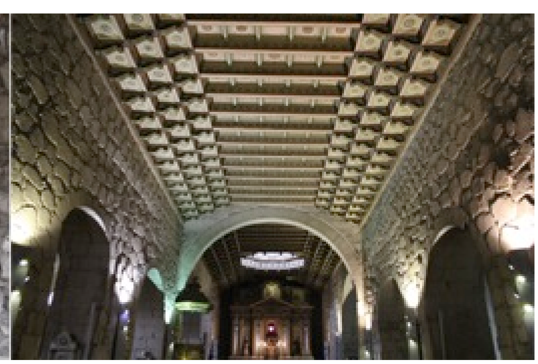

Fig.5. Ceiling diaphragm.

The roof structure is constituted of a sequence of wooden trusses; they are placed above each aisle separately and rest against the raising of the longitudinal walls of the church, which are made of adobe bricks. Together with the trusses there are some transversal adobe walls which act as buttresses, placed in correspondence with the spans in the below church.

According to some historians, both the diaphragm and the roof structure have contributed to the survival of the church during earthquakes [1], as they help with the "box behavior" of the building.

3. Seismic vulnerability assessment

3.1 Assessment of crack patterns and structural criticality

During its 400 years of life, the Church of San Francisco has suffered some local damage due to the combination of two factors: the Chilean earthquake's severe action and the constructive defects of the building due to the numerous transformations in its history, which has given rise to deficiencies both in the in-plane capacity of the walls and in the "box-behavior" determined by disconnections between the walls. Openings in masonry walls, absence of transverse bond and additions with different materials have represented the preferential ways for the damage due to the action of the earthquake. Nonetheless, the stiffness in-plane horizontal of the diaphragm constituted by the timber roof was able to effectively distribute the seismic action between structural elements.

Reconstruction of rear presbytery façade using wooden elements and bricks shows collapse occurred on the top wall. These failure mechanisms have occurred for the following reasons: the high conventional slenderness (λ_c = 14.6) of wall, the significant distance between the transversal walls, the ratio between length and height L/H=0,88 [7], the lack of a connection with the roof covering, and the presence of a wide opening. Similarly, the main façade presents a summit reconstruction of brick and discontinuities with the orthogonal walls with a ratio L/H=0,605 [7]. Moreover, this façade shows an additional vulnerability associated with the eccentricity of the bell tower inducing different inertia than the main block. The longitudinal outside walls have important discontinuities in the proximity of the transept intersection, which belongs to the original nucleus of a Latin cross. This discontinuity represents a weakness to overturn. In the north and south transepts, the reconstruction of the walls with bricks puts in evidence the already activated out-of-plane mechanism.

The foregoing considerations reveal the need to analyze the behavior of the following subparts: the main façade, the rear façade of presbytery, the north and south walls of the transept (Fig. 6).

Fig. 6- Damage to Main Facade (a), Presbytery Façade (b), North Transept (c), South Transept (d).

3.2 Structural analysis

From the results of the integrated studies of constructions features and from assessment of the crack patterns [8] it emerges that the San Francisco Church is the result of transformations over time, more or less coherent with "the rule of art of good building" [9], which implies a number of uncertainties difficult to quantify. For this reason, in order to evaluate the seismic performance of the church, a multi-level approach, that could embrace local and global analysis, was performed. Firstly, in order to define the local behaviors, linear and non-linear kinematic analysis, in the framework of limit analysis, was carried out. Moreover, the control on the global response of the church was addressed through Linear Dynamic Analysis, with a FEM 3D modeling (Straus7 software).

3.2.1. Linear and non-linear kinematic analysis for the out-of-plane capacity

Systematic analysis of the crack patterns of existing masonry buildings [7] have shown that the seismic action selects the most vulnerable masonry portions whose structural behavior, following an out-of-plan action, can be considered independent from the global behavior of the building. The safety assessment must therefore be performed through local analysis which identifies a set of characteristic elements of vulnerability, the macro-elements [10]. The choice of kinematic analysis it due to the awareness that a building's masonry reaches the collapse due to loss of balance of building portions (macro-elements) [9].

The Chilean Standard for Earthquake Resistant Design of Buildings NCh433Of96 [11] does no allow the possibility to verify the seismic behavior of existing unreinforced masonry buildings. For that reason, it was decided to operate by completing the lack of this standard with a combined analysis through the Italian Code NTC2008 [12] and Circ.617/2009 [13]. The study of local mechanisms of damage, based on the limit analysis, was performed through the Kinematic analysis by Italian Code Circ.617/2009 C8A.4.1 and C8A.4.2 [13]

The first step of analysis is the identification of the damage mechanisms that will most likely be activated. The analyses were performed for the current state and the original previous state (Latin-cross plan). In the current state the mechanisms identified are: the gable overturning of the main façade and the rear façade of presbytery, consisting of cuneiform blocks, the overturning out-of- plane around the two oblique cylindrical hinges, and the simple overturning of the north and south transept walls around the cylindrical hinges of the base of building (Fig.7). In the original previous state the mechanisms evaluated are the same but with different materials, with the exception of the mechanism characterizing the main façade. In fact, since the gable was confined both to the contiguous walls and the shear walls, the mechanism considered is the horizontal arch mechanisms (horizontal bending) of confined wall. The horizontal arch inside the wall reached the limit state due to the crushing of the masonry at key stone for the compressive stress at the limit of the material's capacity (considering $f_{m,min}$=2,6MPa,, according with M.Q.I. method) (fig. 8).

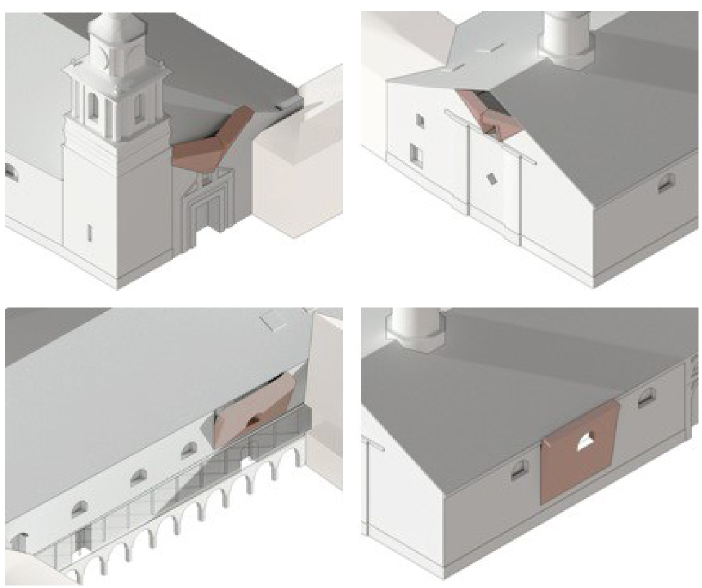

Fig. 7- Damage mechanisms of the Main Facade (a), of the Presbytery Façade (b) and of the South Transept (c) and of the North Transept (d)

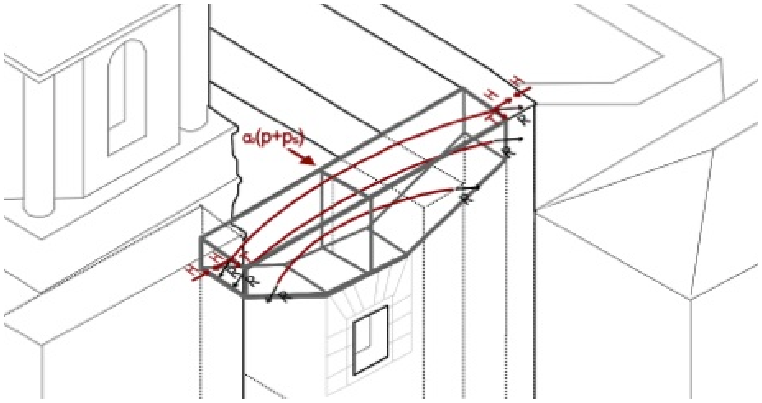

Fig. 8. The horizontal arch mechanisms (horizontal bending) of the Main Façade's confined wall

The second step evaluates the kinematic multiplier $\alpha_{0,}$ converts α_0 into spectral acceleration a^*_0 to get a homogeneous greatness with the demand, and calculates the participating mass as a modal form of vibration:

$$\lambda\left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}\right) = \sum_{i=1}^{n} P_i \cdot \delta_{yi} \qquad a_0^* = \frac{a_0 \sum_{i=1}^{n+m} P_i}{M^* FC} \qquad M^* = \frac{\left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}\right)^2}{g \cdot \left(\sum_{i=1}^{n} P_i \cdot \delta_{xi}{}^2\right)} \tag{1}$$

Where: λ is the kinematic multiplier; *Pi* is the i-th dead and ceiling load; *δxi* is the virtual horizontal displacement of the gravity center of the i-th load *Pi*; *δyi* is the virtual vertical displacement of the gravity centers of the i-th load *Pi*; M^*_0 is the participating mass; a^*_0 is the activation acceleration, *Fc*=1,35 is a confidence factor-related to the knowledge level of building (level 1 LC1, Circ.617/2009 table C8A.1.1).

For the overturning mechanisms a slippage *t* of the cylindrical hinge is considered to take into account the finite compression strength of the masonry: $t = 0.66 \sum_{i=1}^{n} W_i \ (f_d l)^{-1}$
Slippage t depends on i-th selfweight, Wi, design compressive strength, $f_d = f_m$, and width of wall, l.

The Chilean "Earthquake Resistant Design of Buildings" [11] allows defining the effective maximum acceleration value to the ground, with respect to the seismic zone. For Santiago the acceleration coefficient (zone II) is A_0=0,3g. In [14], a probabilistic seismic hazard reassessment of central Chile is proposed considering three different seismogenesis (interplate, intermediate-depth intraplate and crustal seismogenic source). This revaluation leads to ground accelerations of entities greater than those considered by the Chilean Code, but this study still needs to be integrated with the assessment of the amplifications produced by local conditions (site effect). A more accurate definition of the seismic hazard of Santiago will be supported from additional studies on improving seismotectonics and seismic hazard assessment along the San Ramon Fault at the eastern border of Santiago city, Chile [15]. From the mechanics soil analysis, this area is classified as very dense and stable ground (V_{S30} > 500 m/s), whose soil coefficient S is 1,00. The elastic spectrum is defined by (fig.9):

$$S_e = S_a \cdot R^* = I A_0 \alpha$$
(2)

Where S_a is the design spectrum; the R^*=1,54 is the acceleration spectral reduction factor; the values of I=1,2 (building category A) and A_0 are determined by the Code NCh433Of96-6.2.3, in agreement with Eurocode 8 [16]. The amplification factor α is determined for each mode of vibration n. according to the expression:

$$\alpha = \frac{1 + 4{,}5\left(\frac{T_n}{T_0}\right)^p}{1 + \left(\frac{T_n}{T_0}\right)^3} \tag{3}$$

Where T_n is the vibration period of the mode n., T_0 = 0,3s is a parameter that depends on the type of soil (type soil B), p = 1,5 is a parameter that depends on the type of soil (factors- referring to actual characteristic analyzed in D.S. N° 117, (V.Y U.), DE 2010 [16]. The soil type was identified through a soil mechanics report of another nearby building [18].

It is possible to underline that soil parameters are the same for the Italian NTC2008 table 3.2.II [12] and Chilean Code [11].

The first vibration period is calculated by $T1= C_T H^{3/4} = 0{,}05 \cdot 14{,}12^{3/4} = 0{,}37s$. The spectral response in the continuous line highlights that the request acceleration is 0,360g = 3,53 ms^{-2}. The NTC08 verification condition for existing buildings is the following:

$$a_0^* \geq (IA_0\alpha) / R^* = 2{,}31\ ms^{-2} \tag{4}$$

Considering: R*=1,54 acceleration spectral reduction factor, corresponding to structure kind factor referring to actual characteristics analyzed in NTC08 standards.

The mechanisms involving the top of the façades have the input demand at the floor amplified by effect of the height. The Italian Code evaluates this amplification, further verification imposing:

$$a_0^* \geq S_a(T_1)\Psi(Z)\gamma \tag{5}$$

Where: $S_a(T_1)$ is the design spectrum with respect to the period T_1; T_1 is the first vibration period; $\psi(z)=Z/H$ where Z is the height from the foundation to the centroid of the weight forces applied on the rigid bodies, H is the total height of the building from the foundation; $\gamma=3N/(2N+1)$ corresponding to a participation coefficient modal.

The activating accelerations of the mechanisms were compared with the accelerations request, considering both the current state of the building (Table 1) and the state preceding the reconstruction interventions (Table 2).

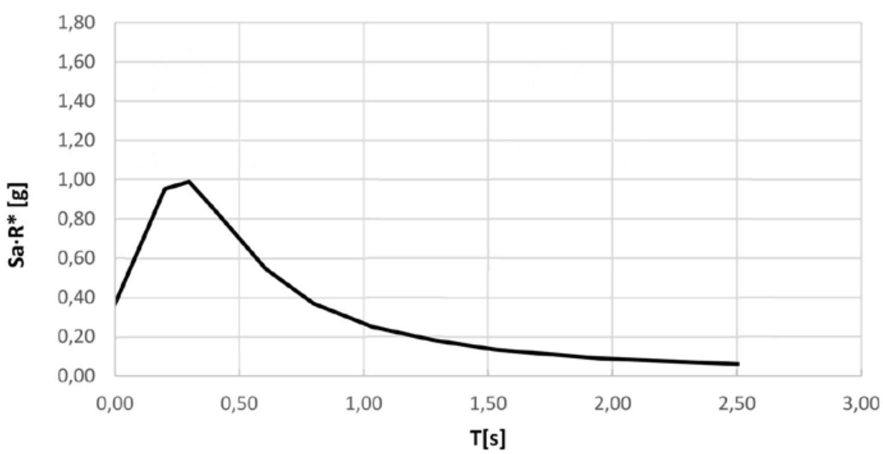

Fig. 9- Elastic Spectral Acceleration (NCh433Of96 and D.S. N° 117, (V.Y U.), DE 2010)

Table 1: Results of Linear Kinematic analysis of the current state: Kinematic multiplier λ, Participating Mass M*, Mechanism Activation Acceleration a_0^*, equation (4) for the Demand Acceleration at ground level, equation (5) for the Demand Acceleration at elevated level.

Block	Mechanism Type	λ	M*	a0*	(IA0α)/R*	Sa(T1)Ψ(z)γ
Behind Presbytery wall	Glabe Overturning	0.286	60	2.468	2.31	4.654
Main Facade	Glabe Overturning	0.336	122	2.478	2.31	4.836
North Transept	Simple Overturning	0.113	172	0.866	2.31	1.318
South Transept	Simple Overturning	0.131	188	1.030	2.31	2.823

Table 2: Results of Linear Kinematic analysis of the previous state of the church: Kinematic multiplier λ, Participating Mass M*, Mechanism Activation Acceleration a_0^*, equation (4) for the Demand Acceleration at ground level, equation (5) for the Demand Acceleration at elevated level.

Block	Mechanism Type	λ	M*	a0*	(IA0α)/R*	Sa(T1)Ψ(z)γ
Behind Presbytery wall	Glabe Overturning	0.222	103	1.71	2.31	4.654
Main Facade	Horizontal arch	0.271	156	2.06	2.31	4.836
North Transept	Simple Overturning	0.141	334	1.063	2.31	1.318
South Transept	Simple Overturning	0.166	303	1.282	2.31	2.823

From the comparative analysis of the current state (fired-bricks blocks) and the original previous state (stone blocks), it shows a significant improvement of resistant behavior of blocks involved in the Main and Presbytery Façades mechanisms. While the walls of the North and South transepts feature a worsening of the seismic behavior due to the reduction of the resisting moment.

Therefore the third step evaluates the Capacity displacement of the North and South transept walls, by means of non-linear kinematic analysis, the kinematic multiplier λ evolutions to growing the displacement d_k of a control point, usually chosen in proximity to the mass center, until reaching the annulment of the Seismic Force. For each analyzed mechanism, the real masonry system is transformed in an equivalent SDOF system, whose capacity in displacement to be compared with the displacement demand (fig. 10). The rotation θ_{k0}, that leads to collapse is given by the expression $Ms = P_i R_i \cos(\beta_i + \theta_k)$ of the stabilizing moment, is:

$$\theta_{k,0} = H_{cp} / Sin(\vartheta_{k,0}) \tag{6}$$

From which follows:

$$d_{k,0} = d_k \{(\Sigma P_i \delta^2_{xi})/[\delta^2_{x,k}(\Sigma P_i \delta_{xi})]\} \tag{7}$$

Where $\delta_{x,k}$ and δ_{xi} are the horizontal virtual displacement of control point and i-th force respectively. The displacement demand is thus obtained:

$$d^*_u \geq \Delta d \tag{8}$$

$d^*_u \geq max \quad S_{De}(T_s); \quad S_{De}(T_1)\Psi(Z)\gamma\{(T_s/T_1)^2/[(1- T_s/T_1)^2+ 0{,}02(T_s/T_1)]^{0,5}\}$

Where: $T^*_s = 2\pi \sqrt{(d_s^*/a_s^*)} \quad d^*_s = 0{,}4\, d_u^* \quad a^*_s = a_0^*(1- d_s^*/d_0^*)$

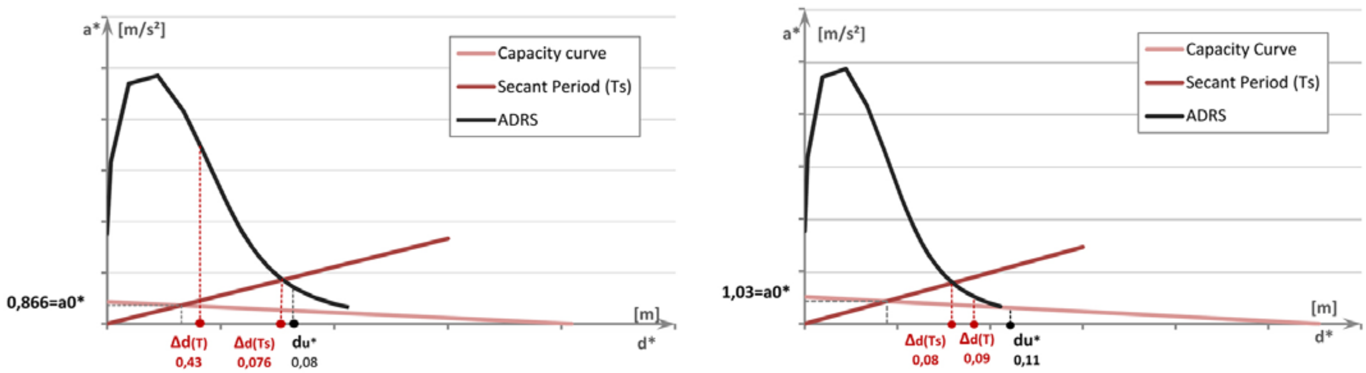

Fig. 10- Capacity and Demand Curves for Macro-blocks: North Transept (a), South Transept (b)

From the comparison between the Capacity and Demand displacement (8) of the transept walls the relations (8) are satisfied (fig.10) and justifies both the absence of the collapse and the activation of the mechanisms detected through linear kinematic analysis.

3.2.2. Global Analysis

In addition to the local analysis, FE models of the church have been developed by using the software Straus 7 [19]. The global structure was modeled assuming: the homogeneous and elastic materials characterized by the proprieties; the decorative elements are not included in the model; the bell tower top and the non-structural loads of roof have not been directly modeled but applied as vertical forces. A linear static analysis for vertical loads was performed, and, subsequently, a natural frequencies analysis for setting the spectral response. All loading configurations have been combined to evaluate the stress and displacement (table 3). In agreement with the NCh433Of96, the analysis included all the modes (100 vibration modes) necessary so that the sum of the equivalent masses, for each of the seismic actions, is higher than 90% of the total mass (table 4). The first 10 vibration modes excite most of the participating mass.

Table 3: Load Case Combinations

Load Combs	S1	S2	S3	S4	C1	C2
Dead	1	1	1	1	1,3	1,3
Ceiling	6	6	6	6	1,5	0
Spectral Case 1	1	-1	0	0	0	0
Spectral Case 2	0	0	1	-1	0	0

Table 4: Natural Frequencies Analysis Results: the first 10 vibration modes.

Mode	Frequency (Hz)	Modal Mass	PX-X%	PX-Y%	PX-Z%
1	3.21	9.39E+05	5.69	21.43	0.05
2	3.76	8.30E+05	10.13	12.81	0
3	4.30	2.36E+06	0.06	30.64	0.01
4	6.13	2.06E+06	1.92	0.05	0
5	6.62	1.30E+05	22	0.043	0.083
6	7.03	3.62E+02	0.226	0.00	0.001
7	7.17	2.55E+04	14	0.427	0.056
8	7.42	1.07E+06	5	7	0
9	7.69	6.51E+05	1	0.042	0.692
10	8.16	1.96E+05	6	2	0,08

Despite the fact that the structural responses obtained through the global finite element analysis are characterized by significant limitations for masonry buildings, the vulnerabilities that have been detected in the kinematic analysis are coherent with the obtained results. In fact the first 9 modes show the intrinsic vulnerability highlights both the crack patterns that kinematic analyzes (Fig. 11). Beyond that it is possible to observe that the distribution of the masses is not prevalent in a single mode but is distributed in numerous modes. This circumstance shows that the structure does not exhibit a well-defined global behavior and that the evaluations based on local analysis are more significant. In fact (from table 4), the structure shows the prevalent local behavior.

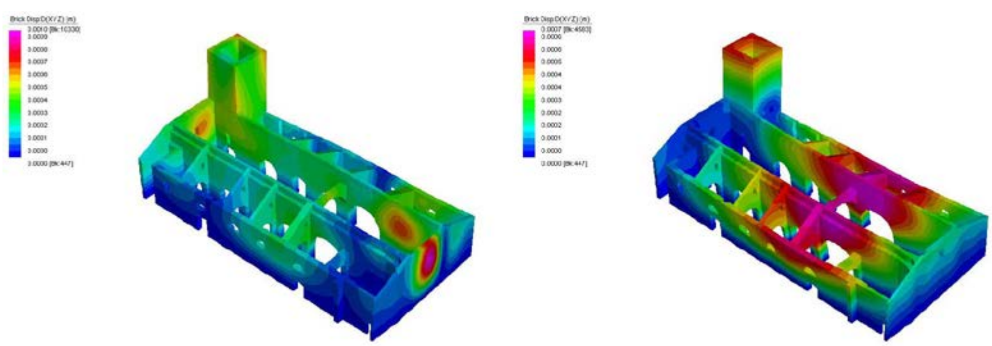

Fig. 11- FEM output: displacement, mode3 and mode7

4. Conclusions

San Francisco church is the result of many additions, subtractions and repairs of local damages suffered in nearly 400 years, and represents an extraordinary example of an earthquake resistant building. The outcomes of this search are the result of a multi-level approach: historical research, direct surveys with special focus on building technique and crack patterns. The set of these analyzes have allowed the bringing to light of the key factors permitting clearer understanding of the surprising capacity of the building to withstand so many destructive earthquakes:

- suitable size ratios of structural and architectural elements
- good constructive technique, in spite of the difficulties in the implementation of the original stone walls
- efficient transverse connection operated by the wooden beam system in the ceiling
- addition of side aisles, operating as buttresses for the native plant and use of triangular buttresses in the arcades extrados to ensure a better transverse response
- uninterrupted maintenance work

Summarizing the results of the cracks pattern assessment, the linear kinematic analysis shows the activation of collapse mechanisms in the North and South transept walls due to the reduction of the resisting transverse section, instead a satisfactory safety assessment in Main and Presbytery Façades. Thus, a limit analysis kinematic approach and not linear static analysis, both highlight not only the necessity of improving the missing connection but also of strengthening some suffering stone piers.

Furthermore the results obtained by global analysis assert that the structure does not exhibit a well-defined global behavior, since the distribution of the masses is not prevalent in a single mode but is distributed in numerous modes, and then the evaluations based on local analysis are more significant.

It is important to mention that there are also some local damages, such as the cracks in the base of some arcs of the central nave, which were not analyzed in depth in the present research, but these have to be considered as they show near to a limit state of crashing for the materials.

5. Acknowledgements

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