Structural, seismic and geotechnical analysis of the Sant’ Agostino church in L’aquila (Italy)

Análisis estructural, sísmico y geotécnico de la iglesia de Sant’ Agostino en L’aquila (Italia)

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Abstract

Existing masonry buildings are often the result of constructions, changes and alterations that have developed over the centuries. Therefore, the adaptation and improvement of existing masonry buildings must be accompanied by an adequate level of knowledge of their history. The tragic consequences of the recent earthquakes, in Italy and abroad, have stimulated the belief that, sometimes, the cause of seismic damage might be just the wrong intervention of improvement or adaptation. In most cases, this is attributable to retrofit works which are carried out without any knowledge concerning the construction rules of the building. The case study presented here regards the seismic analysis of the Sant’ Agostino Church in L’aquila (seriously damaged by the earthquake occurred in April 2009), which takes into account the in-depth knowledge of the factory, respecting all three evaluation levels of seismic safety provided by the Italian seismic Guidelines for Cultural Heritage (Eurocode 8, 2003; NTC, 2008). In particular for LV1 (Evaluation Level 1), all the analyses requested to fill the annexes of the Directive of December 10, 2007 were carried out, starting from Module A, which is the “identifier registry”, passing through Module B, which concerns the study of “sensitivity factors”, and ending with Module C, which considers the “morphology of the elements”. From this examination, we concluded that the ground acceleration during the earthquake of April 6, 2009 surpassed the acceleration value corresponding to the limit state of collapse. For LV2 (Evaluation Level 2), we especially focused on the verification of all macro-elements as a potential source of damage mechanisms. In particular, we found that these mechanisms concerned mostly the facade, the hall, the dome, the apse/presbytery, the roofing, the side chapels, the joints between horizontal and vertical alignment, and finally, the bell tower. Once we collected all data regarding the site, the geometry of construction, the characteristics of the materials, the structure, the soil, etc., we carried out a dynamic FEM modal analysis on the structure, valid for LV3 (Evaluation Level 3). Moreover, these dynamic tests are essential for the evaluation of seismic safety and they also present the clear advantage of being non-destructive. Finally, the analysis provided the most significant vibration modes and suggested the optimal structural interventions to remedy the existing damage and prevent the formation of these mechanisms under the action of future earthquakes.

Keywords: Seismic vulnerability, damage, seismic risk, modality of intervention, seismic analysis

Resumen

A menudo, las construcciones existentes en albañilería son el resultado de construcciones, cambios y modificaciones que se han ido desarrollando durante siglos. Por lo tanto, las adaptaciones y mejoras de las edificaciones en albañilería deben ir acompañadas de un nivel adecuado de conocimiento de su historia. Las trágicas consecuencias de los recientes terremotos ocurridos en Italia y otros países, han llevado a pensar que, a veces, la causa del daño sísmico podría ser justamente una errada intervención de la mejora o adaptación que se les ha realizado. En la mayoría de los casos, este hecho es atribuible a las modificaciones realizadas sin tener los conocimientos necesarios sobre las normas constructivas de esa edificación. El estudio de caso presentado en este trabajo se refiere al análisis sísmico de la iglesia de Sant’ Agostino en L’Aquila (gravemente dañada por el sismo ocurrido en abril de 2009), y considera en profundidad los tres niveles de evaluación de la seguridad sísmica establecidos en el Código Italiano para la evaluación y reducción del riesgo sísmico del patrimonio cultural Eurocode 8: Design of structures for earthquake resistance, (2003), Decreto Legislativo (2004), Norme tecniche per le costruzioni, DM. (2005), Norme tecniche per le costruzioni, DM. (2008), Circolare n. (2010), Direttiva del Presidente del Consiglio dei ministri per la valutazione e la riduzione del rischio sismico del patrimonio culturale con riferimento alle Norme Tecniche per le Costruzioni. G.U. (2008). Así, para el LV1 (Nivel de evaluación 1) se realizaron todos los análisis exigidos en los anexos de la Directiva del 10 de diciembre de 2007, comenzando por el Módulo A que corresponde al “Registro de identificación”, continuando con el Módulo B que se refiere al estudio de los “factores de sensibilidad” y finalizando con el Módulo C que considera la “morfológica de los elementos”. A partir de esta evaluación, hemos concluido que la aceleración máxima del terreno durante el sismo del 6 de abril de 2009 superó el valor de aceleración correspondiente a los estados límite de colapso. Para el LV2 (Nivel de evaluación 2), nos enfocamos principalmente en la verificación de todos los macroelementos como una fuente potencial de mecanismos de daños. Descubrimos que estos mecanismos ocurrieron en mayor medida con la fachada, el coro, el ábside, la torre, las capillas laterales, las juntas entre la alineación horizontal y vertical, y por último, el campanario. Un vez recopilados todos los datos relacionados con el sitio, la geometría de la construcción, las características de los materiales, la estructura, el suelo, etc., realizamos un análisis modal a la estructura utilizando el Método de Elemento Finito Dinámico (FEM, en inglés), válido para el LV3 (Nivel de evaluación 3). Además, estos ensayos dinámicos son esenciales para la evaluación de la seguridad sísmica y poseen la ventaja de no ser destrutivos. Finalmente, el análisis entregó los modos de vibración más significativos y sugirió las intervenciones estructurales óptimas para reparar el daño existente y evitar la formación de los mismos mecanismos bajo la acción de un futuro sismo.

Palabras Clave: Vulnerabilidad sísmica, daño, riesgo sísmico, la modalidad de la intervención, el análisis sísmico

1. Historical background

The proposed case is the synthesis of the structural and seismic analysis for the Church of Sant’Agostino in L’Aquila, which was strongly damaged by the seismic swarm of April 2009, by means of a deep historical and scientific understanding of its construction.

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The construction of the church dates back to the early decades of the ‘700. Today, the church still lies, at least partially, on the area of another church founded in 1282 and dedicated to St. Augustine (Cacciamali et al., 2010). It was heavily damaged by several earthquakes and destroyed by the earthquake of 1703. The project of the present church belongs to the architect Giovan Battista Contini and dates to the last months of 1708. It was probably completed in 1725 (Cacciamali et al., 2010; Gavini I. C., 1926).

In its medieval phase, the church had a Latin cross plan, with three naves and transept, apse at the bottom and main façade to the west, towards the surrounding streets rather than the square. It was the church of the three mendicant orders residing in L’Aquila: the Augustinians, after the Franciscans and the Dominicans (Figure 1).

In the city’s iconography of 1622 and 1680, the obvious presence of the sail façade with straight crown, which was the characteristic of the originally constructed church, supports the theory of persistence of the medieval implant (possibly modified in 1656 with the modification of the main door that was opened towards the square) until the collapse suffered during the earthquake of 1703, when Contini designed a new architecture covered by a dome and open to the public square (Cacciamali et al., 2010; Antonini, 2004; Antonini, 1999; Chiodi, 1988).
2. Church description

The church has a longitudinal plan and the entrance opens onto a small atrium covered by a pitched roof, followed by an elliptical-shaped room surmounted by a dome, and a long apse covered by a barrel vault. On both sides of the room, there are three pairs of chapels: the major chapels are located on the orthogonal principal axes, while the smaller rooms are located on the diagonal axes. Outside there is a buttress system. The dome that surmounts the room has a maximum length of twenty meters; it is ribbed and crowned by a lantern.
The facade is divided into two parts, structured into a lower and upper order. The basement corresponds to the head of the atrium; the upper part is retracted approximately seven meters and consists in one of the faces of the lantern’s octagonal prism. The sculptural articulation of the facade is simple and essential, enhanced by a circular medallion in high relief representing St. Augustine. A trapezoidal tympanum surmounted by a balustrade masks the gable roof that crowns the entrance hall and unifies the two partitions of the facade (Cacciamali et al., 2010; Ceravolo R., 2010; Calderini and Lagomarsino, 2009).

The Church is in masonry. From a visual analysis, we observe that all the vaulted surfaces, the arches and the beams (visible due to loss of plaster) are made of excellent-quality masonry brick. Instead, all vertical structures are made of mixed stone/brick masonry of medium quality, with well-connected cantonal in square-shaped stone. The buttresses seem to be made of excellent quality square-shaped stone (Fiengo and Guerriero, 2008). In some cases, above the lintels of the external openings, wooden reinforcement elements are recognized. Inside the building, there are no visible metal chains. However, some metal chain heads, which report their presence, are visible on the exterior walls. The scheme in Figure 5 shows a hypothetical distribution of the chains in the structure (Ceravolo, 2010; Calderini and Lagomarsino, 2009). We can see that the chain heads are only visible on the free external wall towards Via Sant’Agostino and not on the one facing the Prefecture.
3. Vulnerability and damage analysis

The damage suffered by the Church of St. Augustine following the earthquake of April 6th, 2009, led to its inaccessibility and necessitated a provisional first aid, with insertion of shore and tie rods in the main facade.

For interventions regarding post-earthquake reconstruction and reduction of seismic vulnerability on cultural heritage assets, the Directive of the President of the Council of Ministers for the Evaluation and Reduction of the Seismic Risk in the Cultural Heritage, in Line with the Building Technical Standard, October 12th, 2007 is a good reference.

The seismic behavior of the church can be interpreted through its division into macro-elements, characterized by a structural response substantially independent from that of the whole church (facade, room, apse, side walls, principal arch, etc.), and the different kinds of damages could be synthesized in a fundamental number of failure mechanisms. In this way, the various modalities in which the macro-elements break, differing for proportions and materials, are recognized and associated to specific mechanisms of collapse, which represent the real essence of vulnerability (Calderini and Lagomarsino, 2009; Ceravolo and Demarie, 2009; Boscotrecase and Piccarreta, 2009; Gavarini, 1978; Gavarini, 1994; Gavarini 1991).
For the survey concerning the church’s damage and vulnerability, the II level card 1 about the mapping of damage and the seismic vulnerability of the churches was examined in relation to 28 mechanisms, according to the instructions contained in the Directive.

The processing of the collected data allows us to obtain two separate indexes on a statistical basis, through a simple normalized average of the scores on damage levels and vulnerability indicators for each collapse mechanism:

Damage index: it is a numeric parameter ranging from 0 to 1, evaluated by a normalized average of the detected damage. It is provided by the following expression

\[ i_d = \frac{1}{k} \sum_{k=1}^{28} \rho_k d_k \]  \hspace{1cm} (1)

For the k-th mechanism: \( d_k \) is the damage level (0 to 5), \( \rho_k \) is the weight attributed to the mechanism (0 for mechanisms that could not be activated in the church because of the lack of macro-elements, and between 0.5 and 1 in other cases).

- Vulnerability index: varies between 0 and 1 and it is defined as the weighted average of the behavior of different parts of the church. It is provided by the following expression

\[ i_v = \frac{1}{6} \sum_{k=1}^{28} \rho_k (v_{ki} - v_{kp}) + \frac{1}{2} \]  \hspace{1cm} (2)

For the k-th mechanism: \( v_{ki} \) and \( v_{kp} \) are the scores obtained by the mapping of the vulnerability indicators and the seismic principles respectively (Table 3).

Table 1. Assessment of the vulnerability score for each damage mechanism
Tabla 1. Evaluación del puntaje de vulnerabilidad para cada mecanismo de daño

<table>
<thead>
<tr>
<th>Number of vulnerability indicators or seismic principles</th>
<th>Opinion on the effectiveness</th>
<th>( v_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Número de indicadores de vulnerabilidad o principios sísmicos</td>
<td>Opinión sobre la efectividad</td>
<td>( v_1 )</td>
</tr>
<tr>
<td>at least 1/al menos 1</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>at least 2/al menos 2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>at least 2/al menos 2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>None</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

1From about ten years, in Italy exist three levels of catalogation of the churches and palaces after the seismic event (if we don’t consider the level zero, that is the immediate operations of the civil protection):-
- Level 1: Total post-earthquake inspection
  - evaluation of the macroseismic intensity - assessment of the economic damage - judgment of practicability
- Level 2: Analysis of vulnerability - speditive / territorial
  - definition of vulnerability indexes - forecasts of behavior
- Level 3: Deep vulnerability analysis
  - relief of the masonry system to allow the application of simplified response analysis, based on the identification of the mechanisms of failure in the plane and out of the plane - choice of intervention

The cards were worked out by the National Group for the Defence against Earthquakes CNR-CNRDT and the National Seismic Service SSN Department of Civil Protection D.P.C., developed by the University of Genoa (Prof. S. Lagomarsino), and approved by the Ministry of the Cultural Heritage for the technical activity post-earthquake.
The tab for damage and vulnerability mapping of the churches has been used in seismic emergencies since 1995; the huge amount of collected data made it possible, through statistical analysis, to establish a relationship between seismic action and damage, in terms of a vulnerability parameter for one church, and thus a direct correlation between seismic input and detected vulnerability (Cacciamali et al., 2010; Calderini and Lagomarsino, 2009). In practice, the ground acceleration values corresponding to the damage limit state (DLS) and collapse limit state (ultimate limit state or ULS), are calculated according to functions of the proposals’ correlation:

\[ a_{DLS} = 0.025 \times 1.8^{27.344v} \]  \hspace{1cm} (3)

\[ a_{ULS} = 0.025 \times 1.8^{51.344v} \]  \hspace{1cm} (4)

The seismic safety at ULS is measured by an index Is, defined as the ratio between the acceleration at the ULS and the acceleration corresponding to an exceedance probability in 50 years, different than that accepted for new constructions, and dependent on two factors: the relevance of the construction and the use category. This acceleration is obtained by multiplying the reference seismic acceleration on a stiff soil \( a_g \) (with 10% exceedance probability in 50 years) by a factor of importance \( \gamma_I \), whose mean values are provided by the Directive, on the basis of the above parameters (Calderini and Lagomarsino, 2009; Ceravolo and Demarie, 2009).

- Therefore, the index of seismic safety is:

\[ i_s = \frac{a_{ULS}}{\gamma_I \times a_g} \]  \hspace{1cm} (5)

where the coefficient \( S \) depends on the stratigraphic profile of the foundation subsurface and any morphological effects, and the importance factor \( g_I \) is defined by the Directive as in Table 2.

<table>
<thead>
<tr>
<th>Use category/Categoría de uso</th>
<th>Relevance category/Categoría de importancia</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Limited/Limitada</td>
</tr>
<tr>
<td>Exceedance/F</td>
<td>Exceedance/F</td>
</tr>
<tr>
<td>Occasional or not used/Ocasional o no utiliza</td>
<td>40%</td>
</tr>
<tr>
<td>Frequent/Frecuente</td>
<td>25%</td>
</tr>
<tr>
<td>Very frequent/Muy frecuente</td>
<td>17%</td>
</tr>
</tbody>
</table>

Table 2. Exceedance probability in 50 years of seismic action (P), and importance factors \( \gamma_I \) for the ULS verification of protected Heritage

Tabla 2. Probabilidad de excedencia en 50 años de acción sísmica (P), y factores de importancia \( \gamma_I \), para la verificación del ULS del patrimonio cultural protegido.
The safety index calculated in this manner allows having a comprehensive assessment of the church’s vulnerability, and it is significant only when comparing elements affected by the same event.

In the next section there is an excerpt of the mapping card concerning the damage and vulnerability of the Sant’Agostino Church, summary of results, calculation of the three indexes, and descriptions of the identified mechanisms, concluding with proposals for action (Cacciamali et al., 2010; Ceravolo, 2010; Calderini and Lagomarsino, 2009).

4. Calculation of the Seismic Indexes

The damage and vulnerability indexes for the Church of Sant’Agostino were calculated (Cacciamali et al., 2010; Ceravolo, 2010; Calderini and Lagomarsino, 2009). The damage and vulnerability survey was aimed at identifying the mechanisms of collapse that could be potentially activated, as well as the assessment of the damage level of the actually activated mechanisms, in order to calculate an index that represents a synthesis of the damage level in the different macro-elements of the church.

The latter, evaluated on the basis of equation (1), is equal to:

$$i_d = 0.54$$

(6)

The assessment of the church’s vulnerability was then defined in terms of the 28 damage mechanisms, noting those constructive and typological features that have a key role in the structural seismic response; indicators of vulnerability and seismic safeguards were specially considered, in order to define a vulnerability index that represents a synthesis of the vulnerability mapping for the church’s different macro-elements (Calderini and Lagomarsino, 2009).

The vulnerability index, by equation (2), was equal to:

$$i_v = 0.29$$

(7)

In order to understand if the damage level derived from the seismic event was foreseeable and consequently avoidable, the safety index through the simplified mechanical model LV1 proposed in the Guidelines for the Evaluation and Reduction of Seismic Vulnerability in Cultural Heritage (G.U. January 29th, 2008) was also calculated. This model allows estimating the ground acceleration \((a_{ULS})\), in a simplified way, for the ultimate limit state and the serviceability limit state for damage \((ADLS)\), as well as the safety index \((I_s)\) through equation 5.
For churches, the proposed model is based on a statistical elaboration of the damage suffered by similar buildings during Italy’s significant earthquakes, and it uses quality parameters of the damage mapping card (OJ Nº 55 of March 7th, 2006) and annex C (OJ January 29th, 2008). In this case, the assessment considered the expected dangers of the site, proposed in NTC2008 (DM January 14th, 2008): \( a_g \) (ULS) = 0.27g \( a_g \) (DLS) = 0.07g. The associated spectrum was compared with that resulting from the recordings made at the time of the said earthquake (AQK station – source: Itaca INGV database). Given the importance of the asset and its use frequency, the importance coefficient is assumed to be equal to 1.2. Since geognostic surveys were not available, a ground of B-type \( (S=1.25) \) was adopted and a coefficient of topographic amplification equal to 1. Table 2 shows the values circled in red.

\[ i_z = 0.72 \]  

(8)
and the Peak Ground Acceleration is

\[ \text{PGA} = 0.373 \, g \, (\text{AQK station}) \] (9)

The ground acceleration during the earthquake of April 6th, 2009 has exceeded the acceleration limit state of collapse defined by the LV1 analysis, thus validating the model for estimating the acceleration (Cacciamali et al., 2010; Ceravolo, 2010; Calderini and Lagomarsino, 2009).

5. Seismic Analysis

Once accurate data concerning the geometry and the material properties, the structure and the soil were available, it was possible to implement a finite element numerical model (FEM) for the building (Cacciamali et al., 2010).

In this case, a linear dynamic modal analysis has been used. First, a three-dimensional model was created.

![Three-dimensional model and classification of the various construction techniques in the Church](image)

**Figure 6.** Three-dimensional model and classification of the various construction techniques in the Church

**Figura 6.** Modelo tridimensional y clasificación de diversas técnicas de construcción de la iglesia

Then, the mechanical parameters were chosen and since no in-situ testing was available, reference was made to what is suggested by current regulations [DPCM of December 10th, 2007 (4.1.7); OPCM 3274/2003 (11.d) NTC 2008 (11.10.3.4)].

Later on, first the factory’s discretization into 255624 tetrahedrons (Figure 7), and then the structure’s modal dynamic analysis by FEM was made (Cacciamali et al., 2010; Ceravolo, 2010).
The dynamic tests were essential for the evaluation of seismic safety, with the extra advantage of being part of that category of non-destructive testing. Ultimately, the analysis allowed identifying the most significant vibration modes among the 300 modes under examination (100 along the x axis, 100 along y axis, and 100 along z axis), directing itself towards the realization of structural interventions designed to heal the current damage and prevent the formation of mechanisms under the action of a telluric event.

5.1 Modal analysis on 100 vibration modes (axis X)

![Figure 7. Solid element. Discretization into 255,624 tetrahedrons](image)

Figura 7. Elemento sólido. Discretización en 255.624 tetraedros

![Figure 8. Modal analysis on 100 vibration modes along axis x. a) Mode 1, freq. 2.13 Hz, participating mass 56%; b) Mode 9, freq. 4.81 Hz, participating mass 7%](image)

Figura 8. Análisis modal en 100 modos de vibración a lo largo del eje x. a) Modo 1, frecuencia 2,13 Hz, masa participativa 56%; b) Modo 9, frecuencia 4,81 Hz, masa participativa 7%
5.2 Modal analysis on 100 vibration modes (axis Y)

Finally, the FEM model allowed us to confirm and enrich the findings of the failure mechanisms’ analysis. We find that the observed damage shows the effects of a global response of the Church, mainly in the longitudinal sense (Calderini and Lagomarsino, 2009). In particular, the following damage mechanisms seem all attributable to this type of response:

- The damages on the lateral walls of the atrium, which betray an overturning of the facade (Mechanism 1);
• The damage in the dome (Mechanism 14), with damage on the median parallel to the apse (which is a typical “uncovered” mechanism) and the diagonal damages on the segments, particularly evident in the two diagonal slices into the atrium (where the contrasting elements are not as effective as for the apse);

• The damage showing the overturning of the apse and the detachment of the presbytery of the room (Mechanism 16);

• The damages on the lateral walls and on the vaults of the apse and presbytery (Mechanisms 17 and 18);

• The damages on the vaults of the two major chapels (Mechanism 24).

The prevalence of this response compared to the transept ones seems to be motivated by two main factors: the longitudinal direction is that in which there is major pushing by the dome (major axis); in the transverse direction, more effective contrast elements are present (very rigid buttresses, over each other structural pieces, and chains). The collapse of the lantern and the serious damage in the tower and belfry, have proper peculiarities. The collapse of the lantern, in particular, may be due to the movement of the underlying dome and to the presence of the new metallic cover with rigid curb in reinforced concrete. The damage of the bell cell seems mainly attributable to a torsion phenomenon resulting from its irregular shape, and the presence of asymmetric constraints.

6. Conclusions about the “box-behavior” of the structure’s post-seismic analysis

In the work we intend to evaluate, from the historical and architectural point of view of the ensemble, the “box-behavior” of the whole construction that, once recovered after the specific analysis for macro-elements, must continue to show a harmonious unity between function, form and reinstated static aspect.

The graphic and detailed study of the static elements needed for the yard restoration, which highlights in a comprehensive manner the unity of the structure after the static consolidation, is a method so far rarely applied in combination with the verification to macroelements, and at the same time, it is accessible only by a figure that is formed through skills neighboring both architecture and structural mechanics.

The restorer architect, who has the task of coordinating the capabilities within the static restoration project, cannot leave out a final graphic examination, taking into account the “before and after” that derives from the application of a static and/or dynamic model.
The mere application of the results of finite elements and macro-elements are inadequate and not completely suited to reality, because it is divorced from aspects concerning the material that composes the structure. Conversely, as it is noted, the techniques applied by Structural Mechanics about continuous body in the elastic or non-elastic range, do not allow a fast application in practical cases and especially in an emergency.

Thus, as alternative for the implementation of a method that puts together, on a medium scale, i.e., of structural portion, the computer speed of FEM and macro-element methods, and the meticulousness in the characterization of the material belonging to the traditional analysis of the continuous body in inelastic field, currently the only way to get a complete knowledge of the construction would seem a thorough historiographical analysis thereof; it is the only means that is able to provide and authorize the intervention tools along with the overall graphical feedback.

7. References/Referencias

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