Collapse Analysis of the Basilica di Santa Maria di Collemaggio, L'Aquila, Italy, 2009

Vincenzo Arcidiacono & Gian Paolo Cimellaro
Politecnico di Torino, Turin, Italy

John A. Ochsendorf
Massachusetts Institute of Technology, Cambridge, Massachusetts, USA

SUMMARY:
On the 8th of April, 2009 an earthquake damaged a large historical area of downtown L'Aquila, Italy. Among the damaged buildings was the Basilica di Santa Maria di Collemaggio. During the earthquake two large transept columns collapsed, leading to the collapse of central dome and adjacent vaults. These columns were responsible for sustaining the dome and two barrel vaults that constituted the roof of the transept. This paper presents three approaches to evaluate the dynamic behaviour of the church: modal analysis performed with analytical solutions and different finite element models; pushover analysis using a discretization of the analytical model; and time history analysis of the previous model. Moreover, to evaluate the effectiveness of the chosen models' hypotheses, the numerical analyses are compared with previous research and with the damage states of the Basilica after the earthquake. In conclusion, these analyses demonstrate that the collapse is due to the excessive forces concentrated on the two failed columns near the transept.

Keywords: Santa Maria di Collemaggio, Collapse, Masonry, Nonlinear Analysis, and L'Aquila Earthquake 2009

1. BASILICA DI SANTA MARIA DI COLLEMAGGIO

The *Basilica di Santa Maria di Collemaggio* is the most celebrated medieval church in the region of Abruzzo, located Piazzale Collemaggio 5 in L'Aquila town (Abruzzo, Italy; LON. 13.404275; LAT. 42.342845 with WGS84 datum; see Fig. 1.1.), at an altitude of 685 meters above sea level.

Pope Celestino commissioned the Basilica, which it is influenced by medieval architecture of Abruzzo. The façade of the basilica is made of white and pink limestone geometrically and chromatically arranged to form crosses bi-chromatic texture. There are three impressive entrances above that stand three rosettes. There is also an octagonal tower attached to the right side of the façade.

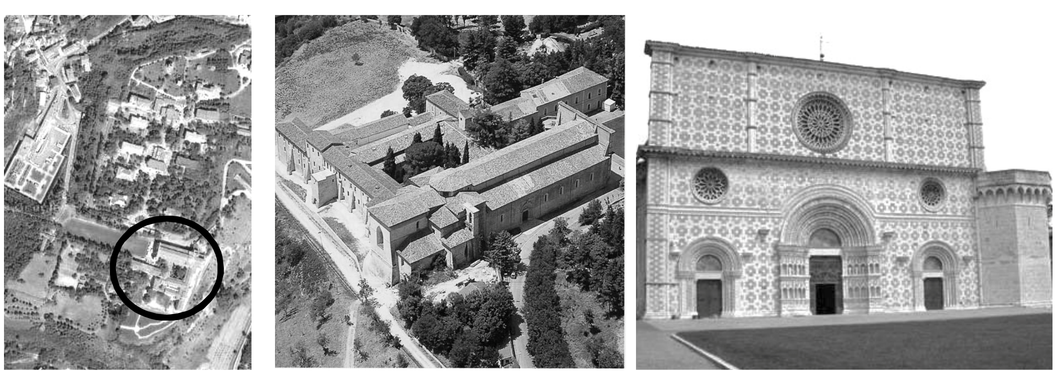

Figure 1.1. Aerial photo of the position of the Basilica (left); Architectonical complex of Santa Maria di Collemaggio (center); Façade of the Basilica (right).

The Romanesque hall is made of three naves: the central spans 11.3 meters and is 18.3 meters tall and also has a steel bracing system on the roof, while, the lateral naves span 8 meters with a height of 12.5 meters. The columns that divide the naves are 5.25 meters tall with a distance of 7.5 meters apart and an octagonal shape with a diameter about one meter. The thickness of the external walls varies between 0.95 and 1.05 meters while the internal walls are about 0.9 meters. The roof of the hall is made of wood trusses and in the central nave there is a horizontal bracing system in steel, which was installed in 1999. The hall is separated from the transept by two large columns, with a floral shape and an equivalent diameter of 1.7 meters, that support the two barrel vaults, the dome and a part of the internal walls. Behind the transept there are three apses made in masonry (see Fig. 1.2.).

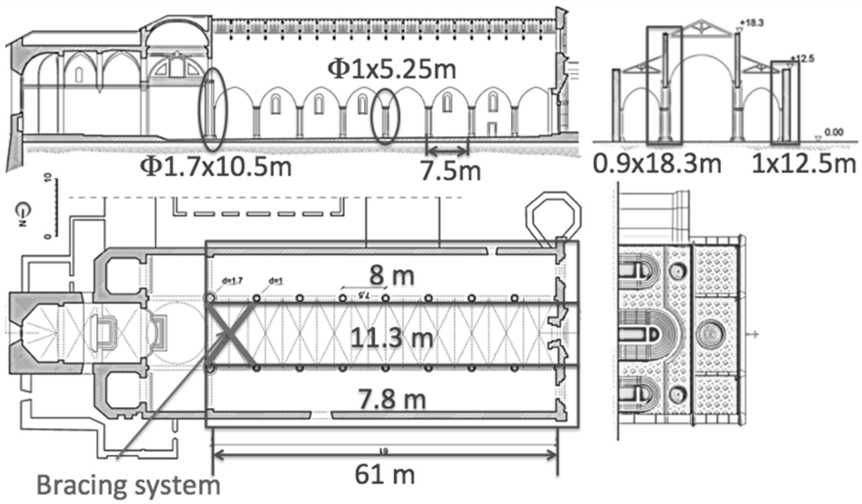

Figure 1.2. Sections of the Basilica of Santa Maria di Collemaggio.

The Basilica was affected by numerous transformations due to earthquake damage or stylistic renovations. Moreover the church, in the last thirty years, has been subjected to diagnostic tests and interventions to improve its mechanical behaviour (see Table 1.1.).

Table 1.1. Significant events in the structural history of the Basilica.

Date	Earthquake	Event
11/10/1287	-	Legal purchase of the land: possible start of construction of the Basilica
25/8/1288	-	Consecration of the Church
1289	-	Completion of work
29/8/1294	-	Coronation of Pope Celestine V
1315	?	Collapse of the apse
1316	-	Construction of the chapel dedicated to San Pietro Celestino
9/9/1349	IX	Serious damage to the church: probably only two rows of arches still standing
1350-1430	-	Modification apsidal part (with three apses and the extension of the central one) and expansion of the lateral naves
1397	-	Affresco of the Holy Door
1380-1450	-	Construction of the lower part of the facade
27/11/1461	IX	Collapse of the chapel and a large part of the monastery, and lesion of the walls
17/12/1461	?	Damaged roof
3-4/1/1462	?	Damage to the tower
1480	-	Demolition of the bell tower
1450-1500	-	Construction of the upper facade, completed in the current rectangular flat shape
1508	-	Completion of the exterior stone facing of the facade, using marble squared
1517	-	Construction of the tomb of San Pietro Celestino by Girolamo da Vicenza
1639	VIII	Superficial damage

1654	VII	Structural damage is not well identified, and start of Baroque transformation
1659	-	Construction of the vaults in the two lateral naves
1669	-	Construction of the ceiling of the central nave
14/1/1703	VIII	Serious damage. The tomb of San Pietro Celestino, the lateral walls, and the façade, remain intact. The walls and ceiling are lowered by about two meters.
1708	-	Completion of the restoration work. The walls and ceiling are lowered by about 2m.
1730	VII	No evident damage
1762	IX	No evident damage
1786	VIII	No evident damage
1791	?	No evident damage
1880	-	Construction of the bell tower to the left of the apse
1891	VIII	No evident damage
1905	VIII	No evident damage
13/01/15	IX	Collapse of the upper-left corner of the façade
1920	-	Reconstruction of the collapsed part of the facade and construction of buttresses
1958	VII	Collapse of the dome and subsequent start of work
1960	VII	Superficial damage
1961	VII	-
1963	VI	-
1967	V	-
1970-1972	-	Restoration and rehabilitation works
1980	-	Analysis of the degradation of the facade performed by ICR, Rome
1988	-	Investigation of the static conditions of the Basilica (DISAT, University of L'Aquila)
1992	-	Analysis of the degradation of the façade (DCICM, University of L'Aquila)
1999	-	Study of seismic behavior of the Basilica: mortar injection and steel bracing system on the roof (DISAT, University of L'Aquila)
2003	-	Study of the dynamic behavior of the façade (DISAT, University of L'Aquila)
06/04/09	?	Collapse of the entire transept

As shown in Table 1.1., on the 8th of April, 2009 an earthquake, with Richter magnitude of 5.8 grade, damaged a large historical area of downtown L'Aquila, Italy. This destroyed the transept, composed of two barrel vaults, one dome and the two large columns below, and also damaged the presbytery and all the internal columns (see Fig.1.3.).

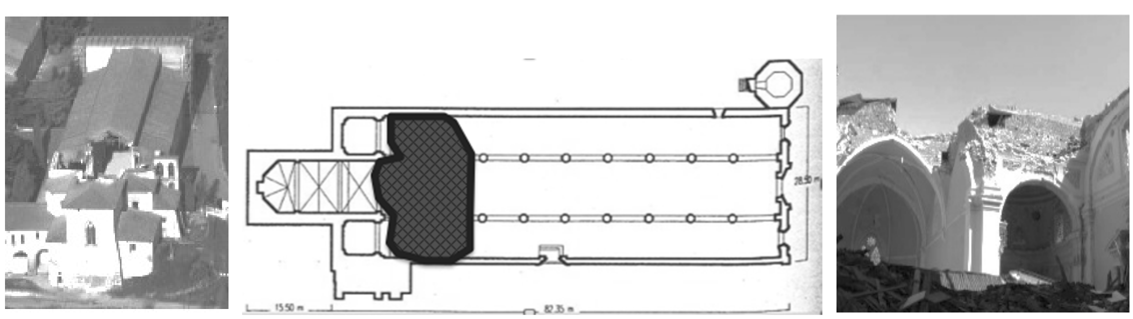

Figure 1.3. Aerial photo of the Basilica after the earthquake (left); Localization of the damages (center); internal view of the transept (right).

2. MODEL ASSUMPTIONS

The aim of this paper is to identify the mechanisms of collapse due to the 2009 earthquake of L'Aquila that caused the collapse of the two large transept columns, leading to the collapse of the central dome and adjacent vaults. Hence, the Basilica di Santa Maria di Collemaggio has been divided in three blocks that are: altar, transept, and naves (see Fig. 2.1.). The analyses are focused on the transept and nave blocks, because the altar block is massive and can be approximate as a rigid block.

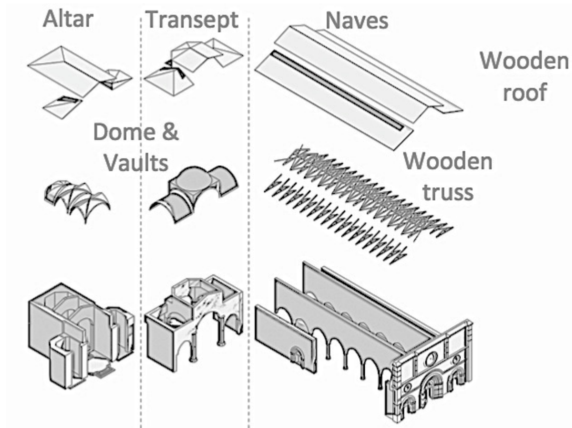

Figure 2.1. Conceptual subdivision of the Basilica.

The bending and axial failures of the two large transept columns – that were responsible for sustaining the dome and two barrel vaults that constituted the roof of the transept – are considered here as two primary collapse mechanisms. Therefore, to evaluate the dynamic behaviour of the Basilica di Santa Maria di Colemaggio two models are developer here: the vertical behaviour of the transept and the horizontal behaviour of the Basilica considering the transept and the nave blocks.

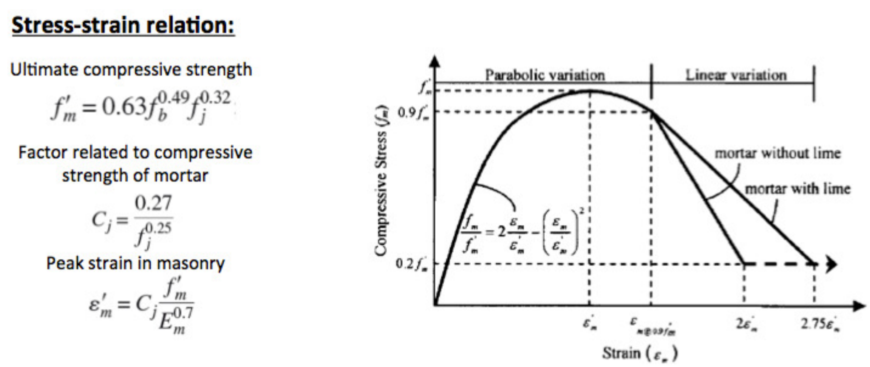

Figure 2.2. Stress-strain relation.

The basilica is made almost entirely by masonry and stone. Figure 2.2. shows the nonlinear behaviour (Hemant et al.) considered for both materials. In detail, it has been assumed a compressive strength equal to 20.8MPa and 20.6MPa, respectively for brick (f_b) and for mortar (f_j) of the masonry; while, for the stone – that constitutes the columns – a compressive strength of 101.7MPa for the "brick" and of 52.6MPa for the mortar. Moreover, the seismic action has been designed according to the Italian Seismic Standard and the peak ground acceleration data – recorded during the 2009 earthquake in the adjacent parking of the church (the station is named AQK, L'Aquila - v. Aterno - Aquil parking.).

2.1. Horizontal behavior of the Basilica (beam model)

The bending failure of the two large transept columns, studying the global horizontal behavior of the Basilica, has been investigated. The altar block has been assumed as a rigid block, because it is stiff and heavy. Initially, three models (Fig. 2.3.), to select the predominant behavior of the church and to check the validity of the chosen hypotheses, have been developed: simple FEM, complex FEM, and an analytical model.

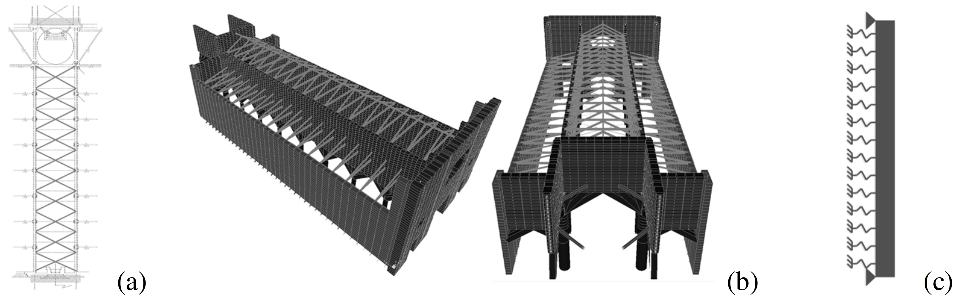

Figure 2.3. Simple FEM model (a); Complex FEM model (b); Analytical model (c).

Considering the above material assumptions, the three models give the following results: the complex FEM – that considers all the three dimensions – has the horizontal behaviour as predominant with a period of 0.79s; the simple FEM model – that considers only the horizontal behaviour with ten degree of freedom – has a period of 0.78s for the first modal shape; while, the analytical model – that is a simply supported beam (with only the shear deformation) on elastic soil – has the first period of 0.80s. Hence, the horizontal behaviour has been considered predominant, and all three models agree on a natural period of approximately 0.8s.

Consequentially, a fourth model – that is an evolution of the analytical model, because it considers also the real stiffness of the transept – has been developed as follows (Fig. 2.4.).

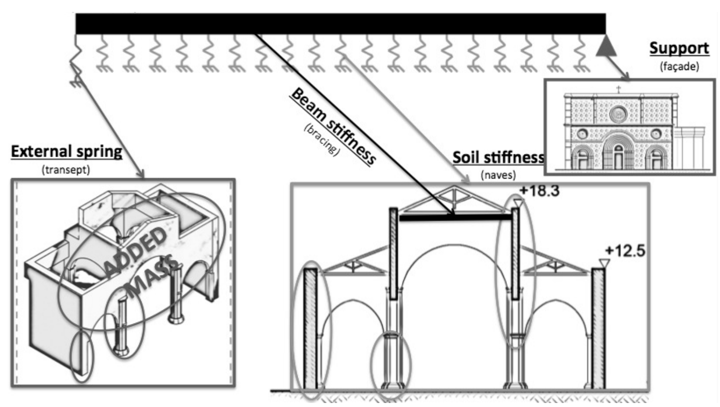

Figure 2.4. Assumptions of the analytical model.

The external spring reproduces the transept. This spring has a damping ratio of 5% and has an added mass of 704.9ton due to the masonry wall that separates the transept from the naves. The stiffness of the spring was modelled as follows:

$$K_{transept} = 2 \cdot \left(K_{EM5.5} + K_{IbC5.5} \right) \tag{5.1}$$

where: $K_{EM5.5}$ and $K_{IbC5.5}$ are the respective stiffness at 5.5 meters of external masonry wall and of internal big column that are considered as cantilevers with flexural rigidity. The shear beam (i.e. allows only shear deformation) simulates the steel bracing system acting on the level of the roof. We assumed a mass per unit length equal to 50.6ton/m, which is the sum of the naves section masses weighted with its deformed shape, and to increase the stiffness of the beam by 50% to consider the wooden roof. This stiffness has been modelled as follows:

$$K_{beam} = E \cdot A \cdot \cos\theta \cdot \sin^2\theta \cdot (1+0.5) \tag{5.2}$$

where: E is the Young modulus of the steel, A is the cross section of the cable of the steel bracing system (710mm^2), and θ is the angle between the cable and the nave axes (61,6°).

The elastic soil simulates the naves section behaviour. The stiffness of the naves was modelled, assuming a damping ratio of 5%, as follows:

$$K_{soil} = 2 \cdot \left(K_{EM12.5} + \left(K_{IsC5.5}^{-1} + \frac{7.5}{K_{IM}} \right)^{-1} \right) \tag{5.3}$$

where: $K_{EM12.5}$ and $K_{IsC5.5}$ are the respective stiffness at 12.5 and 5.5 meters of the external masonry wall and of the internal small column that are considered as cantilevers with flexural rigidity, and K_{IM} is the stiffness of the internal masonry that is modelled with the below assumptions (Fig. 2.5.).

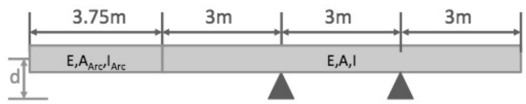

Figure 2.5. Assumptions of the internal masonry.

where: d is the displacement where is evaluated the stiffness, E is the Young modulus of the masonry, A and I are the cross section and the inertia of the whole masonry wall (depth 1m and base of 7.5m), and A_{Arc} and I_{Arc} are the cross section and the inertia of the arch of the masonry wall (depth 1m and variable base from 1m to 7.5m). Finally, the support models the massive façade in the Nord-South plane because it is considered infinite rigid.

2.2. Vertical behavior of the transept (VSDoF model)

The axial failure of the two large transept columns has been investigated by studying the vertical behavior of the transept with a virtual single degree of freedom (VSDoF). The vertical model (Fig.2.6.) is composed of a concentrated mass of 704.9ton (that is proportional to the vertical loads acting on top of the "springs"), by a viscous damper (that has a damping ratio of 5%), and by a vertical spring (that simulates the external masonry walls and the internal large columns).

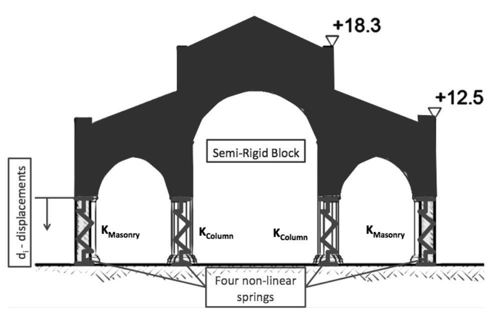

Figure 2.6. Assumptions of the vertical behavior of the transept.

The wall between the transept and naves has been assumed as a semi-rigid block that distributes the displacements proportionally to the loads as follows:

$$\begin{cases} K = \left(\dfrac{K_{EM} \cdot V_{EM} + K_{IC} \cdot V_{IC}}{V_{EM} + V_{IC}} \right) \cdot 2 \\ u = u_{EM} \cdot \dfrac{V_{EM} + V_{IC}}{V_{EM}} = u_{IC} \cdot \dfrac{V_{EM} + V_{IC}}{V_{IC}} \end{cases} \tag{5.4}$$

where: K is the global vertical stiffness; u is the virtual vertical displacement; the K_{EM} and V_{IC}, V_{EM} and K_{IC}, and u_{EM} and u_{IC}, are the respective axial stiffness, vertical load, and vertical displacement of external masonry walls and internal columns.

3. RESULTS

The seismic records of the 2009 L'Aquila earthquake in direction North-South and Up-Down have been analyzed with a Fourier transformation, which shows that the most relevant frequencies are between 0 and 10Hz. Therefore, a return period of 1898yrs (Ultimate Limit State) has been considered to simulate the 2009 L'Aquila earthquake according to the Italian Seismic Standard and the above seismic records.

Moreover, the above-described models have been used to perform the analyses to identify the seismic structural behaviour of the Basilica. Different approaches, such as: the modal analysis (to evaluate the natural frequency and the seismic structural response), the pushover analysis (to estimate the mechanism of collapse), and the time history analyses (to find the mechanism of collapse), have been used and described follows.

3.1. Beam model

This model, as mentioned above, has been used to investigate the bending failure of the two large transept columns. The performed analyses, which are presented below, are:

- Modal analysis;
- Pushover analysis;
- Linear time history analysis;
- Nonlinear time history analysis.

3.1.1. Modal analysis

The modal analysis, according to the Ultimate Limit State (*Stato Limite di Slavaguardia della Vita*-SLV from the Italian Seismic Standard), has been performed and shown in the following figure.

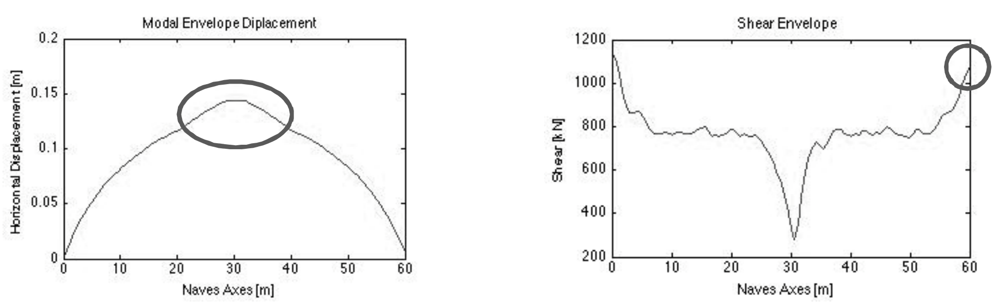

Figure 3.1. Displacement of the naves (left); Shear of the bracing system, beam (right).

According to our model, the maximum predicted horizontal displacement of the roof (12m high) is of 14cm and is located near the centre of the nave. The steel bracing system transfers the load of the naves to the supports increasing the loading on the transept, but the soil system remains strongly loaded. In conclusion, this type of analysis gives an approximation of the initial distribution of the load but says nothing about the mechanism of collapse.

3.1.2. Pushover analysis for naves and transept sections

The pushover analysis has been developed on the naves and transept section considering the geometrical nonlinearity with P-delta effect and material nonlinearity with the bending curvature of each element assuming the stress-strain law defined above. Assuming a distribution of the loads according to the mass distribution and the first modal shape, the results are shown in Figure 3.2..

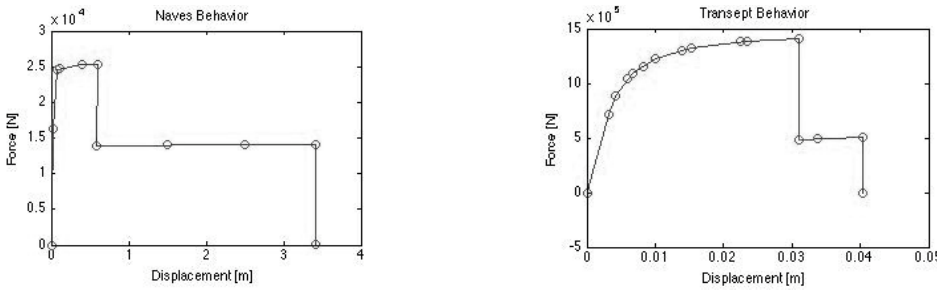

Figure 3.2. Pushover curves of the nave (left); Pushover curves of the transept (right).

The two curves (Fig. 3.2.) consider: the shear resultant at the basement, and the displacement at 12.5m and 5.52m from the basement respectively for naves and transept sections. The naves section has a ductile behaviour (about 0.7m of horizontal displacement before the failure of internal masonry walls) while the transept section – that has a stronger strength than the naves – has a brittle behaviour (about 0.03m of horizontal displacement before the failure of the two large columns).

3.1.3. Linear time history analysis

The linear time history analysis has been performed using the North-South PGA record of the 2009 L'Aquila earthquake. The analysis has been shown that after 32.8s (of the data record) almost of the sections reach the critical yielding displacement and after 37.2s the middle part of the nave reaches the critical collapse displacement. The critical yielding displacement is defined as the minimum elastic displacement necessary to match the elastic stress with the yielding stress recorded in the pushover analysis. The critical collapse displacement is defined as the minimum elastic displacement necessary to match the elastic energy with the collapse energy defined by the pushover analysis (0).

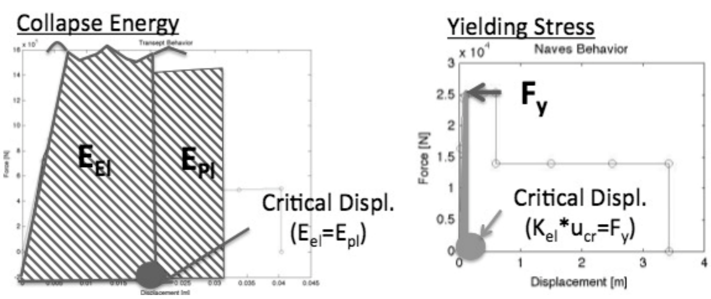

Figure 3.3. Definition of: yielding displacement (left) and collapse displacement (right).

3.1.4. Nonlinear time history analysis

The nonlinear time history analysis gives a better understanding of the problem. The analytical beam model has been transformed in a finite element model able to match the same results of the analytical model up to 20Hz, because the most relevant frequencies of PGA record are between 0 and 10Hz.

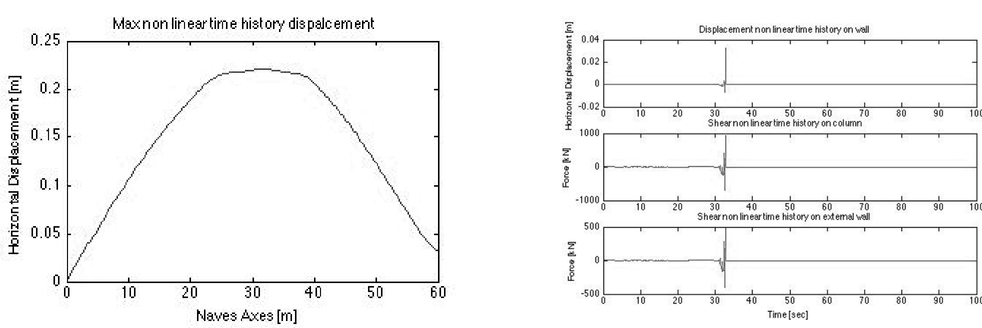

Figure 3.4. Displacement of the naves (left); Actions on the transept (right).

The analysis shows that during the main shock (31s-33s of the record) almost of the naves are plasticized (allowing large displacements, Fig. 3.4.), so the steel bracing system carries the entire load transferring it to the façade and the transept. Moreover, after 32.8s the brittle transept reaches the maximum loading (Fig.3.4.), therefore causing the collapse of the two large columns of the transept. After the collapse of the transept, the finite element model has been rearranged deleting the degree of freedom and the mass of the transept, and, continuing the analysis, no more collapses occurred.

3.2. VSDoF model

This model investigates the axial failure of the two large transept columns. In this case, two analyses have been performed:

- Pushover analysis;
- Linear time history analysis.

3.2.1. Pushover analysis

The pushover analysis has been developed considering the material nonlinearity of stone and masonry that were described above.

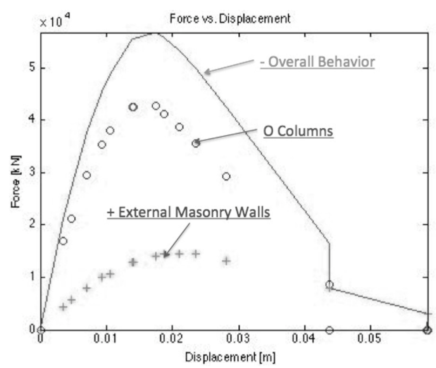

Figure 3.5. Pushover curve.

Figure 3.5. shows the overall load and its repartitions on columns and external masonry walls versus the virtual displacement of the VSDoF model. Moreover, the two large transept columns collapse before the two external masonry walls.

3.2.2. Linear time history analysis

The linear time history analysis is performed using the Up-Down PGA record of 2009 L'Aquila earthquake.

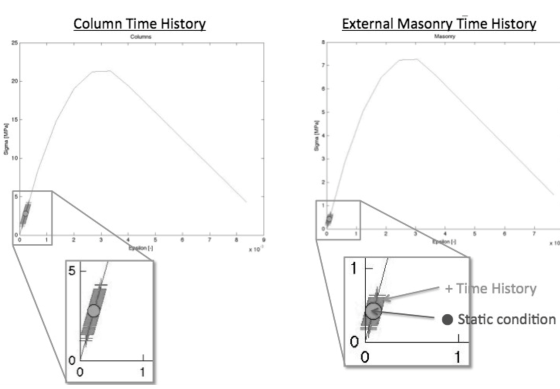

Figure 3.6. Stress-Strain on the column (left); Stress-Strain on the external masonry wall (right).

The analysis results, i.e. stress-strain laws for materials, self-weight condition (with a fill circle), and time history (with the crosses), are shown in Figure 3.6.. The maximum allowed displacements are 15.5mm and 12.1mm respectively for the two big columns and the external masonry walls. The maximum uplifting is equal to 60% of the static displacement that are 1.2mm and 0.5mm respectively for the two large columns and the external masonry walls. The static displacements are very small compared to the maximums, so the vertical collapse mechanism is not plausible, but the strong uplift reduces the bending strength of columns and walls.

4. CONCLUSIONS

To explain the seismic response of the basilica, different analytical and finite elements models are developed and performed, showing the importance of nonlinear behavior for masonry structures. The vertical model (VSDoF) has demonstrated that the vertical seismic action does not cause a collapse mechanism, but reduces the bending strength of columns and walls, because there is a strong uplift of the two large transept columns and the external masonry walls. Instead, the horizontal beam model has demonstrated the collapse of the transept due to the flexural failure of the two columns, which is confirmed by the state of the damage of the basilica after the 2009 earthquake. In conclusion, the main reasons of the collapse are: brittle behavior of the transept and the retrofitted steel bracing system that increases the loading on the transept.

ACKNOWLEDGEMENT
The research leading to these results has received funding from the MITOR PROJECT/MISTI Global Seed Funds of the project Guiding L'Aquila reconstruction after 2009 Earthquake rebuilding a resilient city to multiple hazards.

REFERENCES

Antonacci E., Benedettini F., Di Fabio F., Di Egidio A., Fusco F., Gattulli V., Martinelli A., Vestroni F., *Basilica di S. Maria di collemaggio in l'aquila: Studi e ricerche dell'università*

Antonacci E., Beolchini G. C., Di Fabio F., Gattulli V., Retrofitting effects on the dynamic behaviour of S.Maria di Collemaggio, Tenth International Conference on Computational Methods and Experimental Measurements, which was held in Alicante, Spain, June 4-6, 2001

Antonacci E., Beolchini G., Di Fabio F. and Gattulli V. The dynamic behavior of the basilica S. Maria of Collemaggio, 2nd Internatıonal Congress on Studıes ın Ancıent Structures, Instabul, Turkey, 2001

Antonacci E. & Beolchini G.C., The dynamic behaviour of the façade of the Basilica S.Maria di Collemaggio, STRUCTURAL ANALYSIS OF HISTORICAL CONSTRUCTIONS, IV International Seminar, November 10-13 2004, Padova, Italy.

Bartolomucci C.: Santa Maria di Collemaggio, Interpretazione critica e problemi di conservazione, Palombi Editori, 2004.

Ceci A.M., Contento A., Fanale L., Galeota D., Gattulli V., Lepidi M., Potenza F. (2010). "Structural performance of the historic and modern buildings of the University of L'Aquila during the seismic events of April 2009", Engineering Structures, in press, DOI: 10.1016/j.engstruct.2009.12.023.

CEN – ENV 1996 Eurocodice 6.

CEN – ENV 1998 Eurocodice 8.

Ceradini A. & Grillo F.," Intervento sulla Basilica di S. Maria di Collemaggio all'Aquila – Rapporto sui risultati delle indagini conoscitive relative alle condizioni statiche della basilica. "Risultati della Convenzione tra il D.I.S.A.T. dell'Università dell'Aquila e la Soprintendenza per i Beni Ambientali, Architettonici Artistici e Storici per l'Abruzzo, Comunicazione interna, settembre 1988.

D.M. 14 gennaio 2008: "Nuove norme tecniche per le costruzioni".

Fumo M., Ausiello G. (a cura di), *Il progetto nello spazio della memoria: segni, idee e potenzialità*, atti del convegno internazionale, Napoli 1995.Pasta A., *Restauro conservativo e antisismico. II edizione aggiornata e ampliata*, Flaccovio, Palermo 2006.

Piccirilli C., *Consolidamento critico e sue premesse storico-strutturali*, Multigrafica, Roma 1989.

Volpiano M. (a cura di), *Il consolidamento strutturale degli edifici storici: diagnostica, monitoraggio, materiali per l'intervento compatibile*, L'Artistica, Savigliano 2010.

Zevi L. (a cura di), *Il manuale del restauro architettonico*, Mancosu, Roma 2002.