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THE ASSESSMENT OF THE SEISMIC BEHAVIOUR OF THE CATHEDRAL OF MODENA, ITALY

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ABSTRACT

The Cathedral of Modena, built at the end of the XI century, is one of the most important examples of the Romanesque art in Italy. In 1997, the monument was declared as "UNESCO World Heritage" site.

Due to its cultural importance and with the aim of the conservation of the monument (symbol of the town), the structural behaviour of the Modena Cathedral has been deeply studied, during the last decade, through a multidisciplinary integrated approach. This approach is aimed at collecting all the information obtained from the different fields in order to develop structural analyses and, in particular, a vulnerability assessment of the actual state of the monument.

The objective of this paper is to assess the seismic behavior of the monument through a multilevel approach, characterized by a number of different analyses. First, the dynamic properties of the monument (natural periods and mode shapes) have been identified through a natural frequency analysis performed on 3D finite element models. The fundamental periods are in the range of 0.25–0.35 s. Then, specific hazard analyses have been developed in order to identify the most probable earthquake scenarios which (i) occurred in the past and (ii) is likely to occur in the future in the site of the monument. Both global (i.e. numerical simulations with reference to the finite models of the whole structure) and local analyses (i.e. local collapse mechanisms of the substructures) have been performed.

The results of the local analyses reveal that the main local vulnerabilities are the facade mechanisms and the failures of the cross vaults. The results of the global analyses reveal vulnerabilities of the perimeter walls with respect to out-of-plane overturning. These numerical results have been confirmed by the experimental evidences of the damages observed after the recent 2012 "Emilia Earthquake", which shook the North of Italy.

INTRODUCTION

Historical buildings are complex structures, built and modified in the course of centuries. Therefore, to perform structural (static or seismic) analyses consistent with the real structural behaviour is essential to have a thorough knowledge of these buildings. The Cathedral of Modena has been therefore studied

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through a multidisciplinary integrated approach, which makes use of the “survey” as a tool able to provide a comprehensive knowledge of the building.

The “survey” is here intended as the integration between: (i) the historical reconstruction of the main interventions and modifications of the structural system; (ii) the materials characterization (typologies and mechanical properties); (iii) the topographic survey of the geometry of the superstructure and the evolution of the foundation settlements; (iv) an accurate description of the actual state of degradation (main cracks, tilts of the external walls).

The information obtained from the “survey” have been used to identify the structural functioning, i.e. to recognise the structural elements and the actual load paths to the ground and the characteristics of the materials. As many historical buildings, from a structural point of view, the entire monument can be subdivided into substructures. Each substructure has been analysed through simple limit schematizations in order to obtain a robust evaluation (order of magnitudes) of the internal forces acting on the elements. The results of the static analyses on the substructures have been used to interpret the cracking patterns as obtained from in situ surveys and the deformations related to changes in the geometrical configuration. In addition to the simple limit schematizations, finite element models with increasing complexity (2D models, 3D models, models with fixed base, models accounting for the soil-structure interaction) have been developed and their responses compared with those of the simple models. Different finite element models, validated through simple patterns, are then compared with the “survey” of the actual state of the building in order to identify the most representative one of the structural behaviour of the Cathedral of Modena. It is the one that considers soil-structure interaction. The soil is modelled by a system of linear springs able to consider the different level consolidations caused by the pre-existing cathedrals.

On this model, various analyses have been carried out in order to identify the seismic behaviour of the monument. Specific hazard analyses have also been developed in order to identify the most probable earthquake scenarios for the site of the Cathedral of Modena.

THE CATHEDRAL OF MODENA

The Cathedral of Modena, whose construction began in 1099 through the instrument of architect Lanfranco and finished in 1184, is one of the most important examples of the Romanesque art in Italy (Fig. 1). In 1997, it was declared “UNESCO Cultural Heritage” site. The Cathedral has a basilica plan with one nave and two aisles culminating in three apses. The Cathedral is connected to the contiguous Ghirlandina Tower (a tower of about 88 meters height) through two masonry arches. During the centuries, the monument experienced various interventions and transformations (Sandonni 1983). The structural system of the Cathedral is based on heavy walls and sturdy pillars that support the weight of the imposing domes and of the wood roof system.

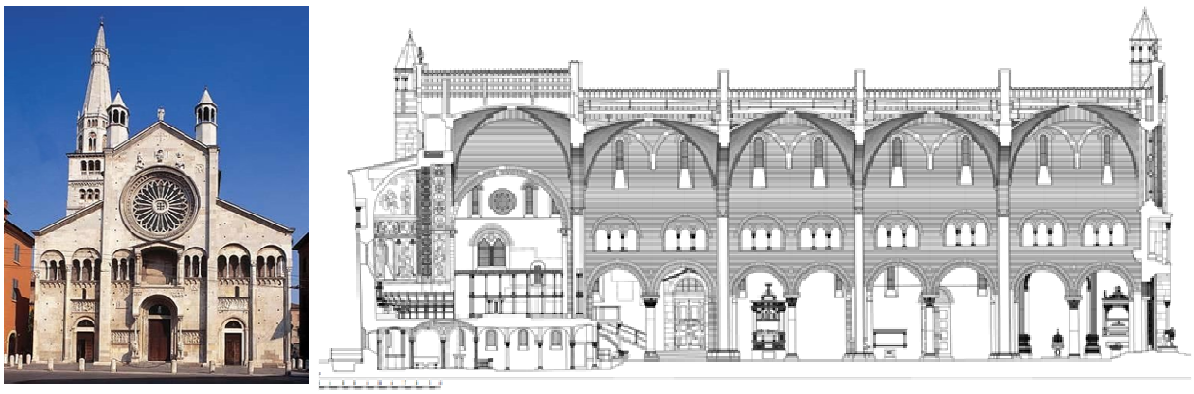


Figure 1. Cathedral of Modena

NATURAL FREQUENCY ANALYSIS

The dynamic properties of the Cathedral (natural periods and mode shape) have been identified through a natural frequency analysis performed on the 3D finite element model, which was considered the more representative of the structure, as identified in the static analysis by continuous comparison with the objective data (experimental evidences) obtained from survey. The finite element models have been developed using the commercial software SAP2000. Since the stress-strain constitutive of masonry structures is yet non-linear for small values of deformation, the reliability of the modes of vibration is to be taken with caution. The common design codes, such as the Italian D.M. 14/01/2008 (Norme Tecniche per le Costruzioni 2008), prescribe that the participating mass must exceed 85%; therefore, in the consecutive seismic analyses 20 mode shapes have been considered in order to satisfy this requirement. The fundamental periods are in the range of 0.25-0.35 s. Fig. 2 shows the first five mode shapes. This analysis shows that the first mode shape is characterized by a translation in the transverse direction of the Cathedral more pronounced in the area of the heavy apses than the area of the nave and the facade. The third mode shape is characterized by a translation along the longitudinal direction.

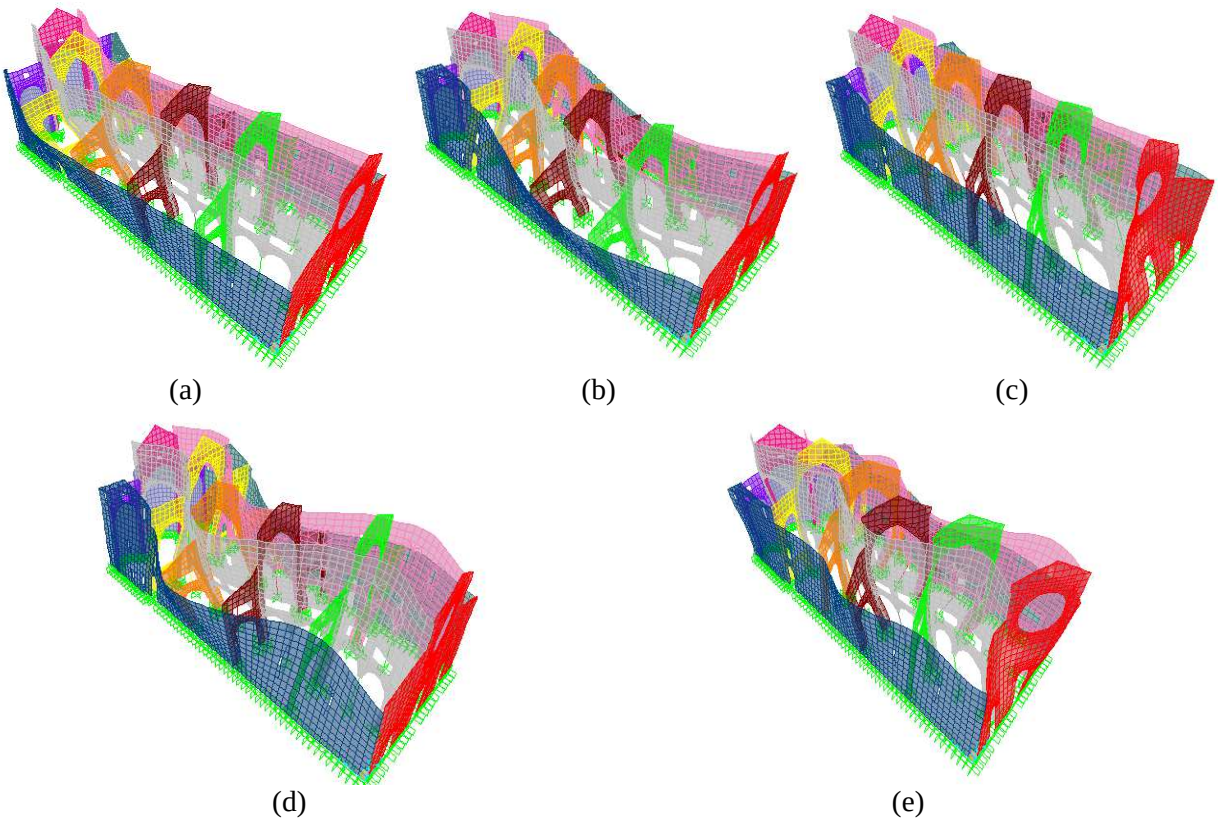


Figure 2. Mode shapes; (a) T=0.35 sec ;(b) T=0,31sec;(c) T=0,28sec;(d) T=0,28sec; (e) T=0,26sec

SEISMIC HAZARD ANALYSIS

The objective of the seismic hazard analysis is to compute, for a given site over a given observation time, the probability of exceeding any particular value of a specified ground motion parameter (commonly the Peak Ground Acceleration, PGA).

In the case of monumental buildings, seismic hazard analysis does not allow only to predict the characteristics of possible future earthquakes, but also to obtain information on the characteristics of already occurred past earthquakes.

The past seismic input has been studied through the reconstruction and the position of the historical earthquakes that have affected the Cathedral. This analysis allows to collect information useful for the identification of the historical periods of specific cracks and failures or interventions and

for the reconstruction of the history of the building. The possible future seismic input has been studied through probabilistic and deterministic seismic hazard in order to identify the most probable earthquake scenarios which can shake the site of the monuments.

Typical probabilistic seismic hazard analysis (as performed according to the approach suggested by Cornell in 1968) assume that, in each point of the seismic zone area, the probability of occurrence of an earthquake is uniform. Thus this approach is suitable for designing new buildings and for regional planning. However, it is not suitable for the identification of the seismic input to be adopted in the studies of monumental buildings, where the consequences of failure are intolerable and protection is needed against the worst that can be reasonably expected to occur. In these cases, the deterministic method is strongly recommended (Krinitzsky 1995). Two kinds of deterministic seismic hazard analyses have been performed for the site of the Cathedral of Modena:

1. Historical Deterministic Seismic Hazard Analysis (HDSHA);
2. Maximum Historical Earthquake Analysis (MHEA);

These analyses have been based on the following data:

- the ZS9 zoning (subdivision of the Italian Territory): the Cathedral of Modena is located in the zone 912 (http://zonesismiche.mi.ingv.it/);
- the CPTI04 earthquake catalogue (http://emidius.mi.ingv.it/CPTI04/);
- the Sabetta-Pugliese attenuation law (Sabetta and Pugliese 1987);
- the Gutenberg-Richter recurrence law (Gutenberg and Richter 1949).

HDHSA has the objective to reconstruct the intensity of historical earthquakes that have actually affected the Cathedral of Modena in the past centuries. Significant historical earthquakes have been selected from the CPTI04 earthquake catalogue, through the following criteria:

- earthquakes that occurred within 20 km from the Cathedral;
- earthquakes characterised by the greater magnitude that occurred in the ZS9 seismogenetic zones near to the site of the Cathedral;
- significant earthquakes in relation to the historical information.

Table 1 shows these significant earthquakes of the past and the reconstruction of their Peak Ground Accelerations, in correspondence of the site of the Cathedral, as obtained using the Sabetta-Pugliese attenuation law. On the basis of the 5 past earthquakes with epicentre in Modena (4 earthquakes with epicentre in Modena respectively in the years 1249, 1474, 1660, 1850 and the earthquake of the Appennino Modenese of 1501), it can be stated that the cathedral might have been hit by accelerations around 0.15 g. The earthquake of 1249 was the most violent and might have rocked the Cathedral with an acceleration of approximately 0.20 g.

Fig. 3 shows the reconstruction of the median of the PGA, obtained considering the epistemic uncertainty associated to the Sabetta-Pugliese ground motion prediction model, for all earthquakes of the CPTI04 earthquake catalogue. Inspection of Fig. 3 indicates that, looking at the past, the earthquake with acceleration between 0.15 g and 0.20 g is characterized by a return period of about 200-250 years.

Table 1. Reconstruction of peak Ground Acceleration (PGA) in correspondence of the site of the Cathedral of Modena for the selected earthquakes

Selection criteria	N.	Year	Location Name	Seismogenetic zone (ZS9)	R [Km] (distance)	Msp (magnitude, as defined by Sabetta-Pugliese 1987)	PGA mode	PGA median	PGA mean value	PGA percentile 80%
Earthquakes that occurred within 20 km from the Cathedral	53	1249	**Modena**	912	0.65	4.80	0.200	0.245	0.270	0.360
	171	1474	**Modena**	912	0.12	4.61	0.170	0.211	0.232	0.310
	195	1501	Appennino modenese	913	16.37	5.82	0.140	0.170	0.187	0.250
	279	1586	Spilamberto	913	10.86	4.53	0.070	0.083	0.091	0.120
	362	1660	**Modena**	912	0.12	4.25	0.130	0.156	0.172	0.230
	374	1671	Rubiera	912	14.26	5.23	0.100	0.117	0.129	0.170
	720	1811	Sassuolo	913	23.49	5.09	0.050	0.066	0.072	0.100
	871	1850	**Modena**	912	5.66	4.53	0.110	0.131	0.144	0.190
	984	1873	Reggiano	913	25.29	4.93	0.040	0.053	0.059	0.080
	1739	1923	Formiggine	913	15.20	5.05	0.080	0.095	0.105	0.140
	1808	1928	Carpi	912	17.83	4.54	0.040	0.054	0.059	0.080
	1859	1931	Modenese	913	15.80	4.54	0.050	0.060	0.066	0.090
	1897	1934	Vignola	913	19.38	4.06	0.030	0.033	0.037	0.060
	2237	1967	Formiggine	913	9.21	4.09	0.050	0.065	0.072	0.100
Earthquakes characterised by the greater magnitude that occurred in the ZS9 seismogenetic zones near to the site of the Cathedral	393	1688	Romagna	912	116.68	5.85	0.020	0.025	0.028	0.390
	30	1117	Veronese	906	82.03	6.49	0.050	0.062	0.068	0.090
	776	1828	Valle dello Staffora	911	209.68	5.55	0.010	0.011	0.012	0.050
	195	1501	**Appennino modenese**	913	16.37	5.82	0.140	0.170	0.187	0.250
	278	1584	Appennino tosco-emiliano	914	147.54	5.99	0.020	0.023	0.025	0.230
	1708	1920	Garfagnana	915	88.64	6.48	0.050	0.057	0.062	0.090
	988	1873	Liguria orientale	916	73.43	5.47	0.020	0.029	0.032	0.060
Significant earthquakes in relation to the historical information	47	1222	Basso bresciano	906	96.77	6.05	0.030	0.036	0.040	0.060
	202	1505	Bologna	913	40.57	5.41	0.040	0.050	0.055	0.080
	1499	1909	Bassa Padana	912	85.20	5.48	0.020	0.026	0.028	0.400
	1684	1919	Mugello	915	99.06	6.18	0.030	0.040	0.043	0.060
	2509	1996	Correggio	912	30.95	5.26	0.050	0.058	0.064	0.090
		2012	Finale Emilia (MO)	912	43.42	5.90	0.060	0.071	0.078	0.110
		2012	Medolla (MO)	912	28.97	5.80	0.080	0.097	0.107	0.150

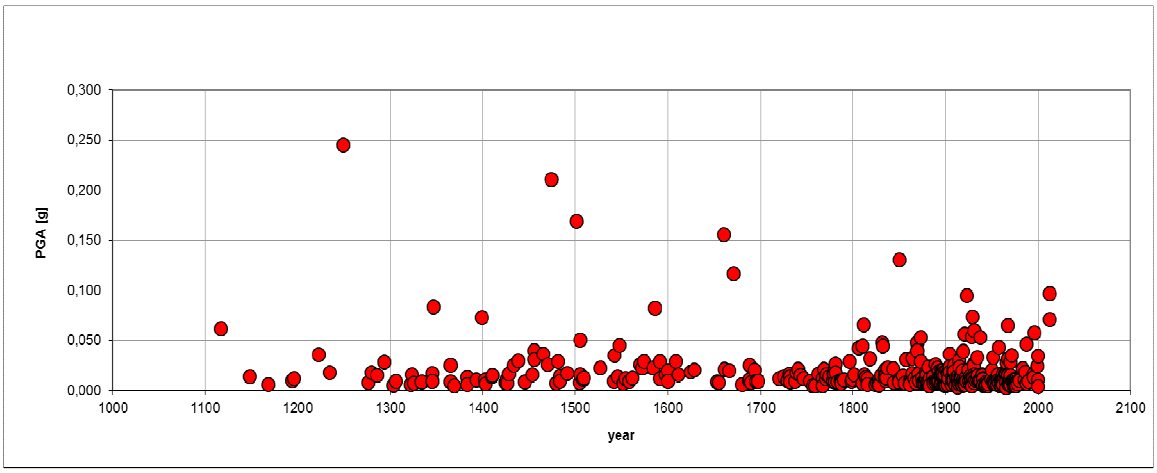
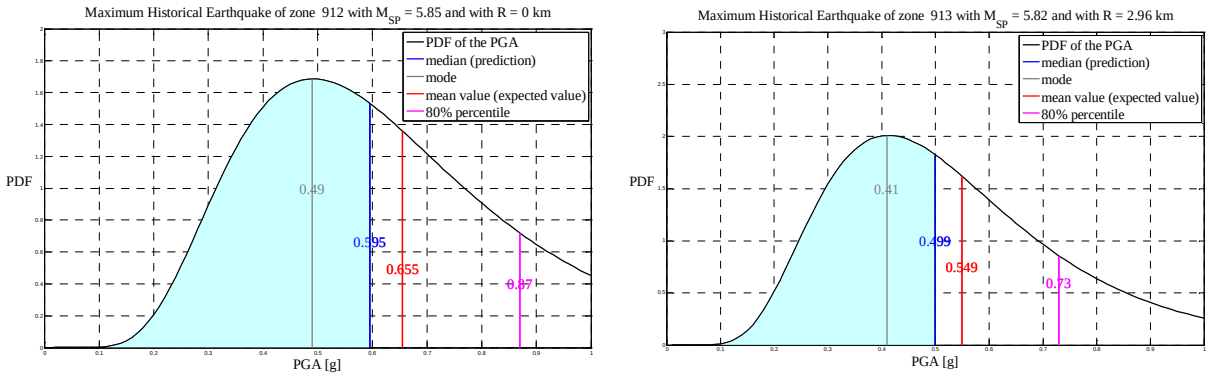


Figure 3. Reconstruction of the median of the PGA, obtained considering the epistemic uncertainty associated to the Sabetta-Pugliese ground motion prediction model, for all earthquakes of the CPTI04 earthquake catalogue.

The MHEA is aimed at estimating the most violent earthquake that could occur in the future on the specific site of the Cathedral. The PGA recorded in a specific site during an earthquake depends on two factors: the magnitude and the distance between the epicentre and the site. Therefore, the worst seismic scenario for a specific site occurs with the combination of the high magnitude and null epicentre-site distance. The maximum magnitudes recorded in the past in the seismic zone (912) of the Cathedral and also in the adjacent zones (913, 914, 915, 916, 911 and 906) were obtained from the earthquake catalogue. Then, it is assumed that earthquakes of such magnitudes could occur at zero distance from the Cathedral, and the intensity of the earthquake worse future is reconstructed considering the epistemic uncertainty associated to the Sabetta-Pugliese ground motion prediction model. Table 2 shows the list of the highest magnitudes occurred in all the considered zones and the reconstructed median, mode, mean values and 80% percentile values of the PGA variable. According to seismic activity of the two areas 912 and 913, it can be stated that a future earthquake with acceleration of about 0.50 g can occur, as shown in the Fig. 4.

Table 2. Estimation, through MHEA, of the PGA that can occur in the future in the site of the Cathedral of Modena.

ZS zoning	Rmin from Cathedral	Mas max	Msp max	Mode	Median	Mean value	80% percentile
912 (zone of Cathedral)	0.00	5.85	5.85	0.49	0.60	0.65	0.87
913	2.96	5.82	5.82	0.41	0.50	0.55	0.73
914	67.50	5.99	5.99	0.04	0.05	0.05	0.08
915	54.67	6.48	6.48	0.08	0.09	0.10	0.14
916	75.48	5.32	5.47	0.02	0.03	0.03	-
911	99.35	5.55	5.55	0.02	0.02	0.03	-
906	72.12	6.49	6.49	0.06	0.07	0.08	0.11



(a) (b)

Figure 4. Probability density function (PDF) of the PGA in the site of the Cathedral of Modena as a result of seismic activity of zones: (a) zone 912, (b) zone 913.

SEISMIC VULNERABILITY ANALYSIS

The historic buildings are characterized by a high seismic vulnerability due to the organization of the structural elements, the characteristics of the materials (masonry) and the construction techniques. In general, the horizontal seismic forces cause damages and/or collapses mainly in the following specific elements: large space without structural walls, arches, vaults, domes, ... , which are common in the churches.

The analysis of the main damages suffered by Italian churches due to the recent earthquakes (L'Aquila 2009 and Emilia 2012) has shown a number of common collapse mechanisms which may involve: (i) the local response of single structural elements and (ii) the global response of the whole structure, and which can be basically divided into: (i) out-of-plane mechanisms and (i) in-plane mechanisms. Thus, both the local collapse mechanisms and the global seismic response of the Modena Cathedral have been studied and presented separately in the next two sections.

LOCAL COLLAPSE MECHANISMS

The local collapse mechanisms are strongly dependent on the construction techniques and on the connection details between orthogonal masonry walls and between the masonry walls and the possible restraining horizontal elements, such as tie-beams, well connected floors, ...

The cathedral has been divided into sub-elements, i.e. structural elements characterized by an autonomous structural behaviour: the façade, the nave, the aisles, the vaults, the longitudinal perimeter walls, the columns, the transept, the triumphal arch and the apses.

For each one of these sub-elements, when applicable, out-of-plane mechanisms and in-plane mechanisms have been considered. As far as the out-of-plane mechanisms are concerned, the limit analysis approach has been applied. Each sub-element is assumed to be composed by a number of stiff, incompressible and infinitely-resistant blocks, and the limit load multiplication coefficient (λ) is calculated by means of equilibrium equations. Limit load is the maximum seismic horizontal load that the structure can safely carry. In general, the limit analysis of masonry structures involves the following assumptions (Heyman 1995): (i) masonry has no tensile strength, (ii) stresses are so low that masonry has effectively an unlimited compressive strength, (iii) sliding failure does not occur.

The results of these studies indicate that the most vulnerable structural elements are the façade and the vaults. This was confirmed by the damages observed after the 29 May 2012 Emilia earthquake, which caused some masonry blocks falling down from the nave and aisles vaults, especially next to the facade and the apses. Fig.5 shows the cracks detected in the vaults of the nave after the earthquake sequence of May 2012.

For sake of brevity, this paper presents only the local analysis performed on the façade. The relief of the cracking patterns shows lesions in the orthogonal longitudinal walls next to the facade (Fig.6a) that suggest a good connection between these elements. Therefore, assuming this good connection, the following mechanisms have been taken into account:

- overturning of the whole façade (Fig. 6b and Fig. 7 – mechanism 1);
- overturning of the left portion of the façade (Fig. 7 – mechanism 2);
- overturning of the central portion of the façade (Fig. 7 – mechanism 3);
- overturning of the right portion of the façade (Fig. 7 – mechanism 4).

Figure 5. Cracks in the nave vaults after the the 29 May 2012 Emilia earthquake.

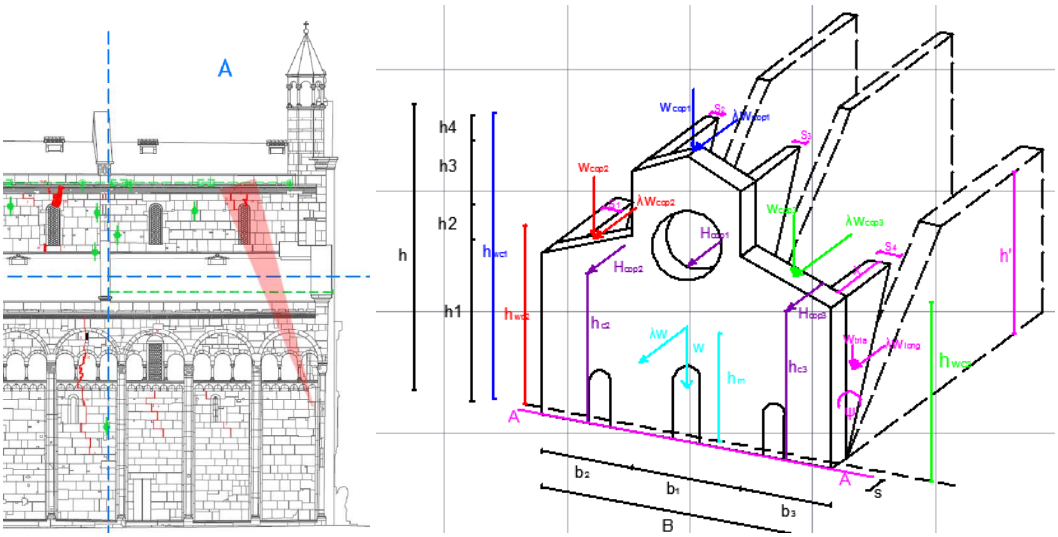


Figure 6. (a) Cracks in longitudinal walls of Cathedral, (b) Overturning of global facade around the base dashed straight line.

The behaviour of the wall in limit equilibrium conditions has been simulated by applying the principle of virtual works, i.e. equating the overturning moment (due to horizontal loads) and the stabilizing moment (due to self weight):

$$M_{overturning} = M_{stabilizing}$$

$$\left[\left(\left(W - W_{holes}\right)\cdot\frac{s}{2} + W_{cop}\cdot d_c - H_{cop1}\cdot h_c + W_{tria}\cdot\left(s + \frac{1}{3}x_i\right)\right)\right] - \lambda\cdot\left[W_{wall}\cdot h_m - W_{holes}\cdot h_f + W_{cop1}\cdot h_{wc1} + W_{long}\cdot\left(\frac{2}{3}\cdot h_i^{'}\right)\right] = 0 \quad (1)$$

The limit load corresponding to the spectral acceleration that activates the local mechanism of collapse has been obtained from equation 1.

$$\lambda = \frac{\left[\left(W - W_{holes}\right)\cdot\frac{s}{2} + W_{cop1}\cdot d_{c1} + W_{cop2}\cdot d_{c2} + W_{cop3}\cdot d_{c3} - H_{cop1}\cdot h_{c1} - H_{cop2}\cdot h_{c2} - H_{cop3}\cdot h_{c3}\right]}{\left[W_{wall}\cdot h_m - W_{holes}\cdot h_f + W_{cop1}\cdot h_{wc1} + W_{cop2}\cdot h_{wc2} + W_{cop3}\cdot h_{wc3}\right]} = 0,22 \quad (2)$$

$$a_0 = \lambda\cdot g = 0,22g \quad (3)$$

Fig. 7 shows the acceleration values that activate overturning mechanisms of the different portions of the façade. These values are higher than the acceleration reference values for the past earthquakes obtained from HDSHA (0.15-0.20 g), but lower than the acceleration estimates for the possible future earthquakes obtained from MHEA (0.50 g).

In general, the study of the local mechanism of other sub-elements reveal that the main local vulnerabilities are relevant to the façade (as described above), the top façade (with trigger accelerations around 0.06 g), the cross vaults (with trigger accelerations around 0.12 g), the triumphal arch (0.07 g), the transversal response of the columns (0.14 g) and the out-of-plane behaviour of the apse walls (0.13 g).

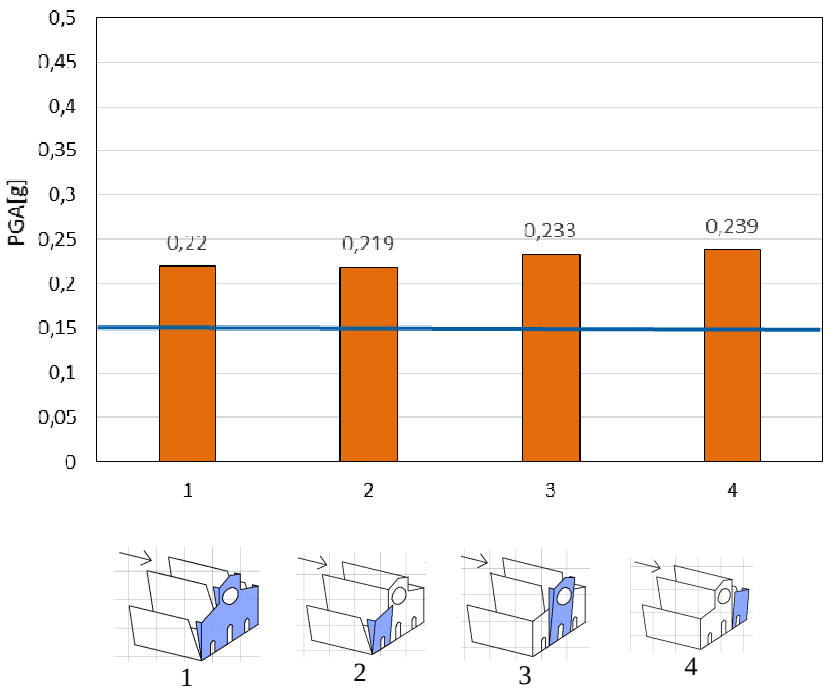


Figure 7. Comparison between the accelerations that activate the 4 mechanisms of collapse for the facade.

GLOBAL SEISMIC RESPONSE

Subsequently, both response spectrum and time history analysis have been performed on the 3D finite element model. The analysis have been devoted to the identification of the criticalities in terms of:

- in-plane mechanisms caused by high shear force (causing possible diagonal cracks or horizontal sliding);
- out-of-plane mechanisms caused by high eccentricity, defined as the ratio between the bending moment and the axial force (causing possible stress concentration at the base or overturning of the wall).

The study of the in-plane mechanisms has been conducted by evaluating the tensile stresses in the walls (diagonal cracking check) and the shear stresses at the base of the walls (sliding check).

Fig. 8 shows the comparison between the tensile stresses and the cracking patterns for the wall 1 (façade wall). In general the results obtained from tensile stresses show a high validation with the cracking patterns. The majority of the lesions seems to be caused by the accumulation of damage over time caused by various earthquakes. The sliding check at the base is performed as follows:

$$\tau < \tau_{di} \tag{4}$$

where

τ is the tangential mean stress and τ_{di} is the shear strength of the masonry, as evaluated according the two diagonal cracking and (friction) sliding mechanisms:

$$\tau_{di} = \tau_{od}\sqrt{1+\frac{\sigma_{oi}}{1,5\tau_{od}}} \tag{5}$$

$$\tau_{di} = \tau_{0d} + 0,4\sigma_{0i} \tag{6}$$

where:

τ_{od}= shear strength of the masonry (τ_{od}=1kg/cm^2);

σ_{oi}= mean compressive stress.

Figure 9 shows the tangential stresses calculated for wall 1 and Table 3 reports the values obtained for the sliding check. The results highlight that the greatest criticalities are related to the in the internal transversal walls.

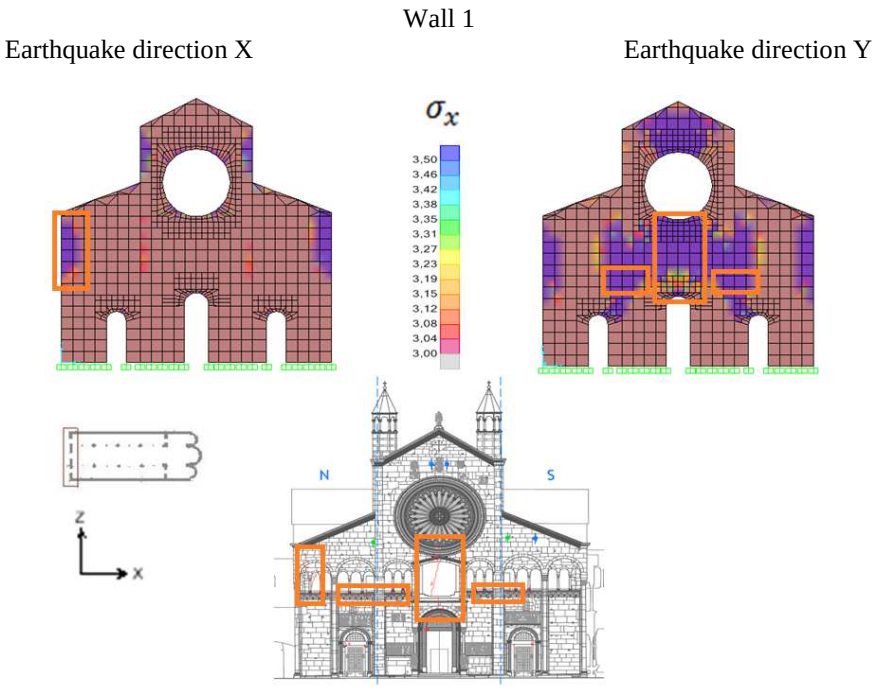


Figure 8. Comparison between tensile stresses and the cracking patterns

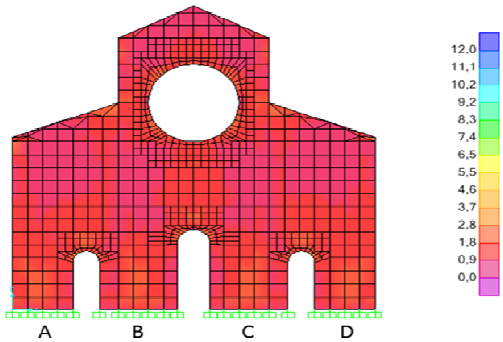


Figure 9. Tangential stresses

Table 3. Verification of the shear strength for the wall 1

	Compressive mean stress	Shear strength Eq. (5)	Shear strength Eq. (6)	Tangential mean stress (Demand)	Demand/ Capacity ratio Eq. (5)	Demand/ Capacity ratio Eq. (6)
	[Kg/cm^2]	[Kg/cm^2]	[Kg/cm^2]	[Kg/cm^2]	[]	[]
Section cut 1A	6.95	2.37	3.78	1.25	0.53	0.33
Section cut 1B	4.66	2.03	2.86	1.36	0.67	0.47
Section cut 1C	4.66	2.03	2.87	1.37	0.68	0.48
Section cut 1D	7.06	2.39	3.82	1.25	0.52	0.33

Out-of-plane mechanisms have been identified by first evaluating the eccentricity at the base of the walls, as defined as the ratio between the bending moment and the axial force in seismic conditions, and then checking that: (i) the eccentricity is below the usual reference values $s/6$ and $s/2$ (with s indicating the thickness of the wall) and (ii) the lateral shear stresses developed on the two vertical lateral sides of the considered wall are below some reference values (i.e. connection capacity).

In detail, the eccentricity at the base of each wall has been calculated by considering both the static loads (self weight and dead loads) and the seismic actions (that, in the case of a dynamic time-history analysis, are function of time) for each section (Figure 10):

$$e_{(t)} = \frac{M_{static} + M_{seismic}(t)}{N_{static} + N_{seismic}(t)} \tag{7}$$

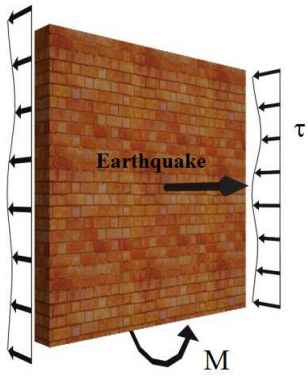
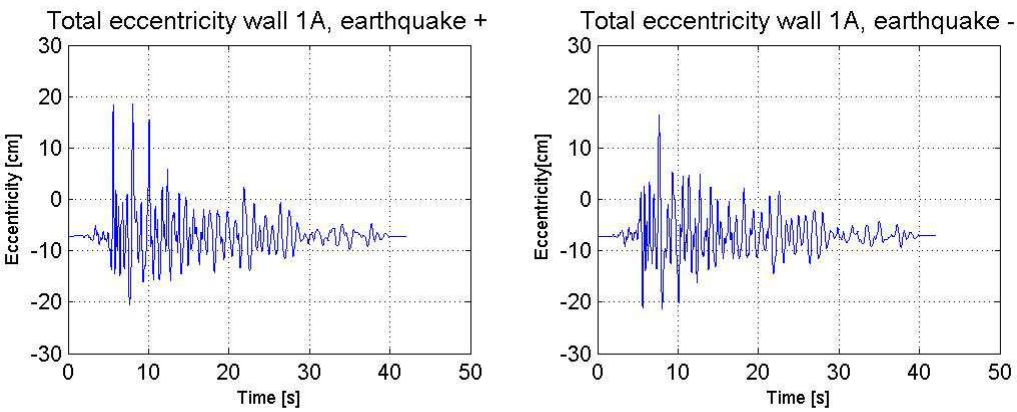


Figure 10. A schematic representation of a single wall with the indication of the out-of-plane seismic action, the base moment and the transversal actions due to the interaction between the orthogonal walls

The time history of the eccentricity has then been evaluated using 9 recorded accelerograms (selected from the P.E.E.R. strong motion database) consistent with the results of the seismic hazard analyses. Then the maximum absolute eccentricities have been used to check the out-of plane stability of the walls.

Two limit cases regarding the quality of the connection between orthogonal walls have been considered to compute the eccentricities: good connections (perfect continuity between orthogonal walls) and bad connections (partial continuity between orthogonal walls, modelled by inserting more flexible elements).

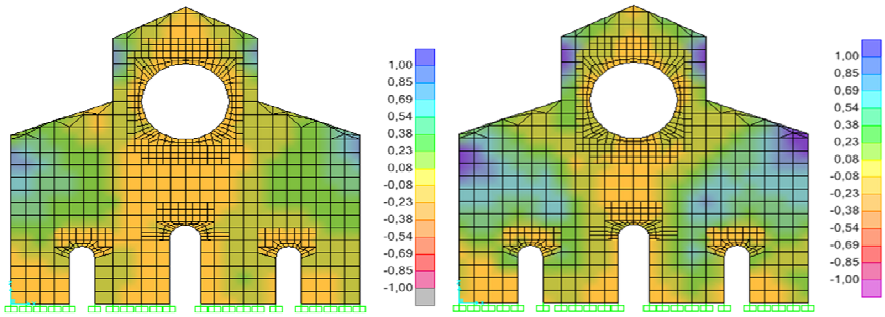
Fig. 11a shows the time history of the eccentricity for the 1A section of the wall 1 whereas Fig.11b shows the shear stresses, exchanged between the considered and the adjacent walls, obtained from the time history analysis for the wall 1. Table 4 shows the values of the eccentricity calculated for the various sections of the wall 1 and verify that these values are lower than s/2. The study of the out of plane collapse mechanisms showed criticality in the transversal walls especially in the control sections B-C. However, the values of the shear stresses for each wall suggest that, even leading to cracked conditions at the base of the walls, the connections are able to keep the wall in a stable configuration. In two longitudinal walls, instead, the results indicate both cracked conditions at the base and values of shear stresses greater than the shear strength of the masonry in the absence of vertical loads.



(a)

Wall 1, thickness s= 145 cm

Good connection Bad connection



(b)

Figure 11.(a) The time history of the eccentricity on the 1A section (b) Tangential stresses for the wall 1

Table 4. Verification of the eccentricity for the wall 1

	Good connection	Bad connection		Good connection	Bad connection	Central core of inertia	s/2
	Mean + [cm]	Mean + [cm]		Mean – [cm]	Mean – [cm]	[cm]	[cm]
Section cut 1A	50,10	120,04		-26,15	-51,02	24,17	72,5
Section cut 1B	1183,40	412,82		-28,70	-99,75	24,17	72,5
Section cut 1C	1386,93	470,64		-28,60	-102,97	24,17	72,5
Section cut 1D	57,03	151,55		-26,20	-53,51	24,17	72,5

CONCLUSIONS

The Cathedral of Modena has been studied through a multidisciplinary integrated approach that allows to obtain a comprehensive knowledge of the building in order to develop sound structural analyses. In particular, this paper presents the study of the seismic behaviour of the Cathedral of Modena through the following steps: natural frequency analysis, seismic hazard analysis, local collapse mechanisms analysis, global seismic analysis.

The results of the local analyses reveal that the main local vulnerabilities are the facade mechanisms and the failures of the cross vaults. The results of the global analyses reveal vulnerabilities of the perimeter walls with respect to out-of-plane overturning. These numerical results have been confirmed by the experimental evidences of the damages observed after the recent 2012 Emilia Romagna earthquake sequence, which struck the northern regions of Italy.

REFERENCES

Sandonni T., (1983) “Cronaca dei restauri del Duomo di Modena, 1897 – 1925”, *a cura di O. Baracchi*

Norme Tecniche per le Costruzioni (2008). Italian Ministerial Decree of 14 January 2008. Published in the Gazzetta Ufficale n. 29 of 04 Feb 2008, Suppl. Ordinario n. 30 (in Italian).

Cornell C.A, (1968) “Engineering seismic risk analysis”, *Bulletin of the Seismological Society of America*, 58, 1583-1605

Krinitzsky E.L., (1995) “Deterministic versus probabilistic seismic hazard analysis for critical structures”, *Engineering Geology* 40:1-7

Sabetta F., Pugliese A., (1987) “Attenuation of Peak Horizontal Acceleration and Velocity from Italian Strong - motion Records”, *Bulletin of the Seismological Society of America*, 77 (5), 1491-1513.

Gutenberg B., Richter C. F.,(1949) ”Seismicity of the Earth”, *Princeton University Press, N.J.* 1954

Heyman J., (1995) “The stone skeleton” *Cambridge University Press*

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