Seismic vulnerability of Santa Maria Novella Basilica in Florence

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Abstract
This paper presents the evaluation of the seismic vulnerability of Santa Maria Novella Basilica in Florence. Santa Maria Novella is one of the most important historical churches in Italy and, for this reason, different studies on the structural behavior of this monument were conducted during the last decades. Particularly, this work is focused on the dynamic behavior of the church. Mechanical properties of masonries were determined through “in situ” and laboratory tests, according to the National Italian Code (Norme Tecniche per le Costruzioni, 2018). An eigenvalue analysis on a finite element model of the Basilica was performed to obtain the fundamental vibration mode shapes.
Finally, to evaluate the seismic risk index in terms of ratio between the minimum peak ground acceleration which leads to the first collapse of a structural element and the design peak ground acceleration, a response spectrum analysis was carried out to evaluate the stress fields in both columns and walls.

Keywords: masonry, historical building, finite element analysis, seismic vulnerability

1. Introduction
Santa Maria Novella is one of the most important churches in Florence, placed beside the square of the same name (Figure 1). The Basilica, together with its adjacent monastery, represents an important cultural and religious center managed by the Dominican order. Santa Maria Novella is the first Basilica where Gothic architecture elements were used in Florence, particularly for the typical aspects related to the Cistercian Gothic architectural style.


Fig. 1: Santa Maria Novella.

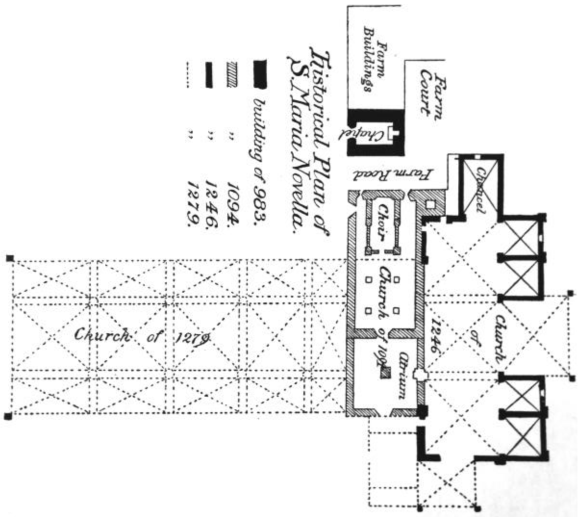

Fig. 2: Historical plan of Santa Maria Novella.

During the centuries, the Basilica was interested by several interventions that had led to different modifications of the original plan (Figure 2).

In the current configuration, the Basilica is characterized by a cross-shaped plan, for a total length of 99.20 m and a width of about 28.20 m (maximum transversal size of 61.54 m in the transept). In the longitudinal direction, the three aisles of the nave are characterized by a variable span between columns, ranging from higher spans close to the main entrance to lower spans near the transept, giving the perception of a greater length than the real one (Figure 3). The nave is 12.00 m wide and 30.00 m high, while the aisles have a width of about 6.00 m and a height of 20.00 m.

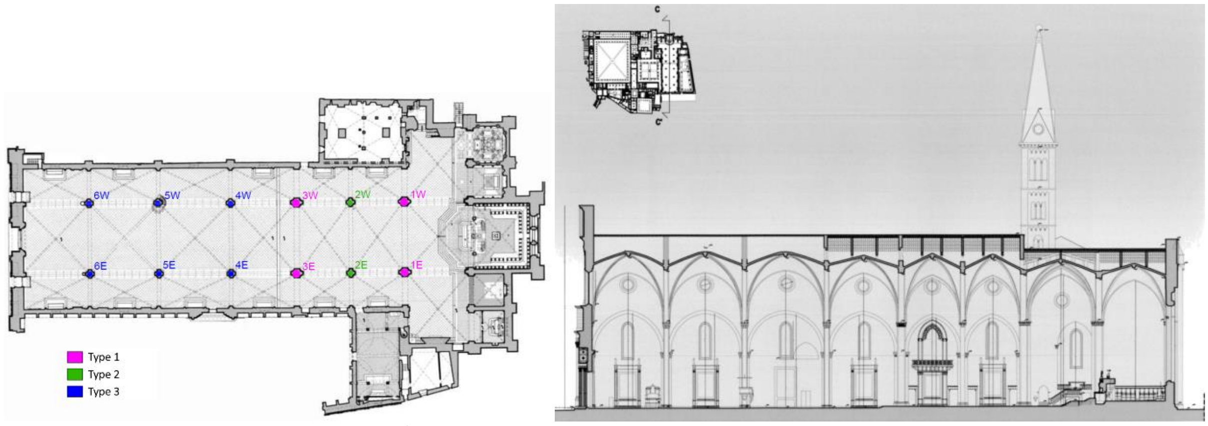

Fig. 3: Plan and longitudinal section of the Basilica.

The pointed arches supporting the ribbed vaults stand on stone columns with different cross sections to hold up, at same time, both the nave vaults and the aisles vaults, positioned at different heights. The columns of the nave are characterized by a maximum height of 15.00 m while those of the aisles have a maximum height of 9.00 m. The transept has three spans with a central square apse and lateral minor chapels. The longitudinal walls along the aisles are characterized by the presence of buttresses and lancet windows.

The Basilica presents three different types of columns: the first has an asymmetric cruciform cross section (maximum dimension oriented in the longitudinal direction of the Basilica), the second and the third have a polylobated shape with different sizes, the third greater than the second one (Figure 4).

2. Numerical Analysis

A Finite Element Model (FEM) of the Basilica was implemented to evaluate its dynamic behavior under seismic actions. MIDAS Gen [1] software was adopted to create the FEM model considering Timoshenko beam elements [2] for columns and arches, four nodes plate elements for walls and the facade and three nodes plate elements for vaults (Figure 5). The maximum size of the generated mesh was generally set to 60 cm. Only close to the intersection between the columns and the vaults, the mesh size was reduced. In the FEM model, all the structural elements are considered fully constrained at the ground level.

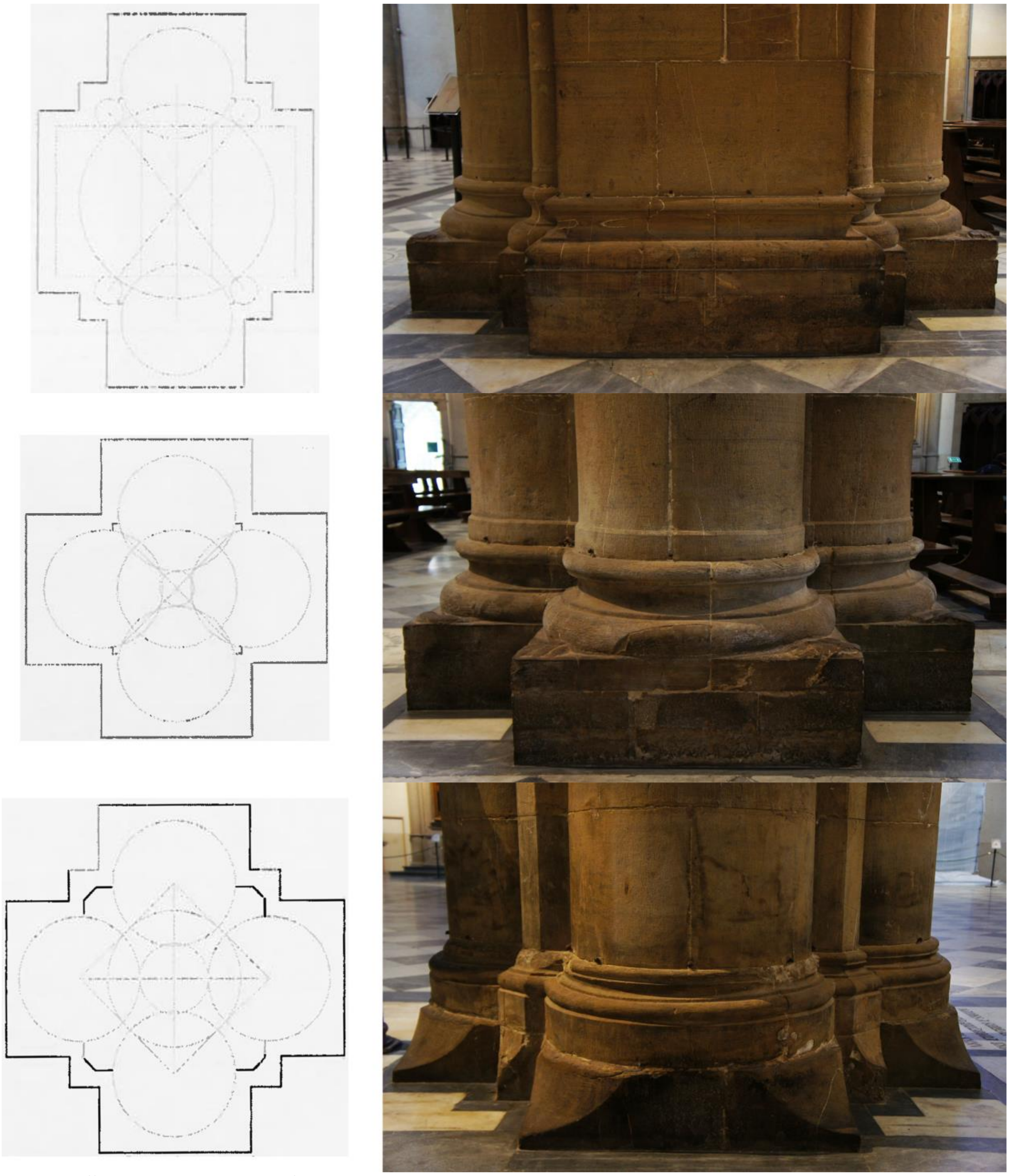

Fig. 4: Different cross sections of the columns.

Three different types of masonry can be recognized in the Basilica, for the different types of construction elements. The adopted mechanical properties of the materials, determined according to the Italian Design Code [3], are summarized in Table 1.

Elements	f_{md} [N/cm^2]	τ_{0d} [N/cm^2]	E [N/mm^2]	G [N/mm^2]	w [kN/m^3]
Columns and arches	386	5.3	4032	1238	22
Walls	107	1.875	1230	410	20
Vaults	107	2.68	1500	500	18

Tab. 1: Mechanical properties of the three considered types of masonry.

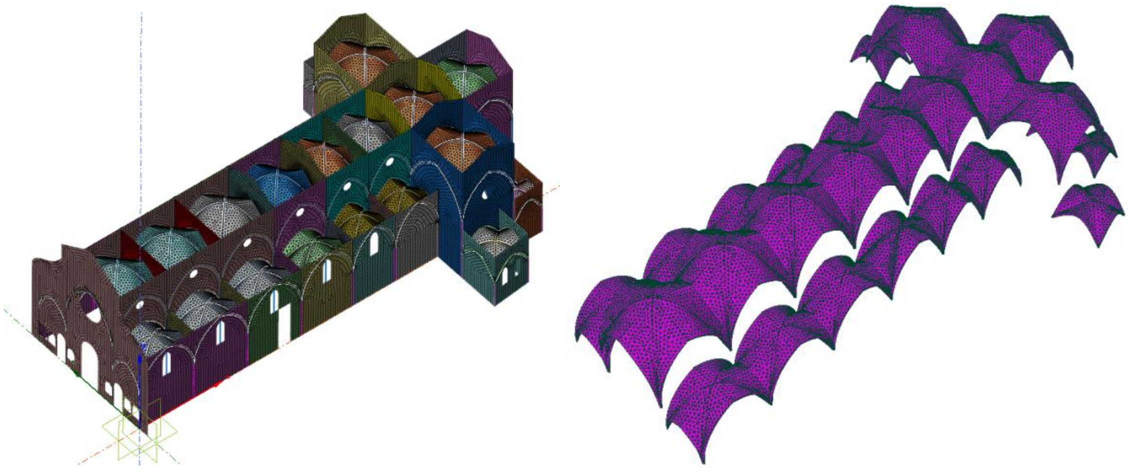

Fig. 5: FEM views.

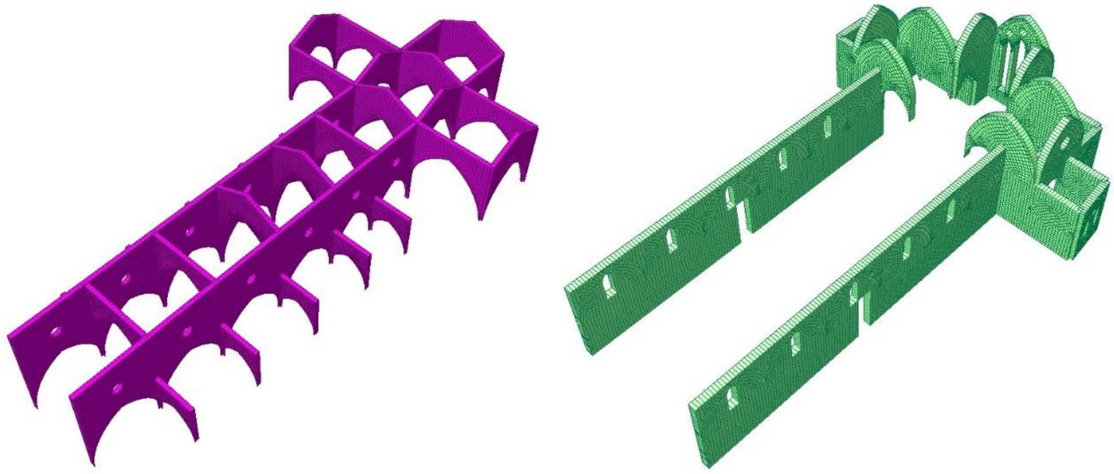

Fig. 6: Walls characterized by thickness of 80 cm (purple) and 150 cm (green).

The walls of the Basilica, including the façade, was considered as elements characterized by the same masonry type and mechanical property. Regarding the thickness, walls can be distinguished as: (i) internal walls, 80 cm thick, (ii) external walls, 150 cm thick and (iii) a façade having an average thickness of 220 cm (Figure 6). Furthermore, the vaults were modelled with an average thickness of 35 cm.

The static loads applied on the model are the self-weight of the structural elements and the dead loads. The seismic action was evaluated according to [3] and the assumed Response Spectrum parameters are listed in Table 2.

Limit State		SLV	
Life of the structure	V_N	[year]	50
Use category	[-]	[-]	III
Coefficient for use category	C_U	[-]	1.5
Reference life	V_R	[year]	75
Probability of exceedance	P_{VR}	[%]	10
Topographic coefficient	S_T	[-]	1
Soil category	[-]	[-]	C
Design ground acceleration	a_g	[g]	0.149
Behaviour factor	q_0	[-]	2.36

Tab. 2: Response Spectrum parameters.

3. Linear Dynamic Analysis

A linear dynamic analysis has been performed to evaluate the seismic behavior of the Basilica, considering the design response spectrum described in paragraph 2. Table 3 shows the vibration mode frequencies of the Basilica, coming from the eigenvalue analysis (in red, all the modes having a participant mass greater than 5% are highlighted).
The vibration mode shapes of modes N. 1, 3, 4 and 5 are shown in Figure 7.

Mode	Frequency	Period	Tran-X		Tran-Y		Rotn-X		Rotn-Y		Rotn-Z	
			Mass	Sum	Mass	Sum	Mass	Sum	Mass	Sum	Mass	Sum
[-]	[cycle/s]	[s]	[%]	[%]	[%]	[%]	[%]	[%]	[%]	[%]	[%]	[%]
1	1.153	0.867	0.00	0.00	42.77	42.77	8.07	8.07	0.00	0.00	6.13	6.13
2	1.651	0.606	0.02	0.02	0.00	42.77	0.00	8.08	0.04	0.04	0.01	6.14
3	1.845	0.543	0.02	0.04	1.96	44.73	0.73	8.80	0.00	0.04	26.80	32.94
4	1.945	0.514	59.25	59.29	0.00	44.73	0.00	8.80	33.22	33.26	0.01	32.95
5	2.138	0.468	0.00	59.29	18.48	63.21	6.43	15.23	0.00	33.26	10.22	43.17
6	2.205	0.454	0.12	59.41	0.09	63.30	0.02	15.25	0.09	33.42	0.15	43.32
7	2.437	0.410	1.12	60.53	0.05	63.35	0.01	15.26	0.00	33.42	0.02	43.34
8	2.469	0.405	0.00	60.53	8.78	72.13	3.22	18.48	0.00	33.42	4.30	47.64
9	2.791	0.358	0.00	60.53	0.19	72.32	0.00	18.48	0.01	33.43	9.30	56.94
10	2.915	0.343	0.70	61.23	0.03	72.36	0.01	18.49	0.01	33.44	0.02	56.96
11	2.938	0.340	1.02	62.25	0.00	72.36	0.00	18.49	0.00	33.44	0.00	56.96
12	3.005	0.333	0.02	62.27	0.00	72.36	0.00	18.49	0.00	33.44	0.01	56.97
13	3.034	0.330	0.00	62.27	0.01	72.37	0.01	18.50	0.00	33.44	4.40	61.37
14	3.098	0.323	2.52	64.79	0.00	72.37	0.02	18.51	0.11	33.55	0.03	61.40
15	3.188	0.314	0.00	64.79	0.62	72.99	11.00	29.51	0.00	33.55	0.67	62.07

Tab. 3: Vibration modes and participant masses of the Basilica.

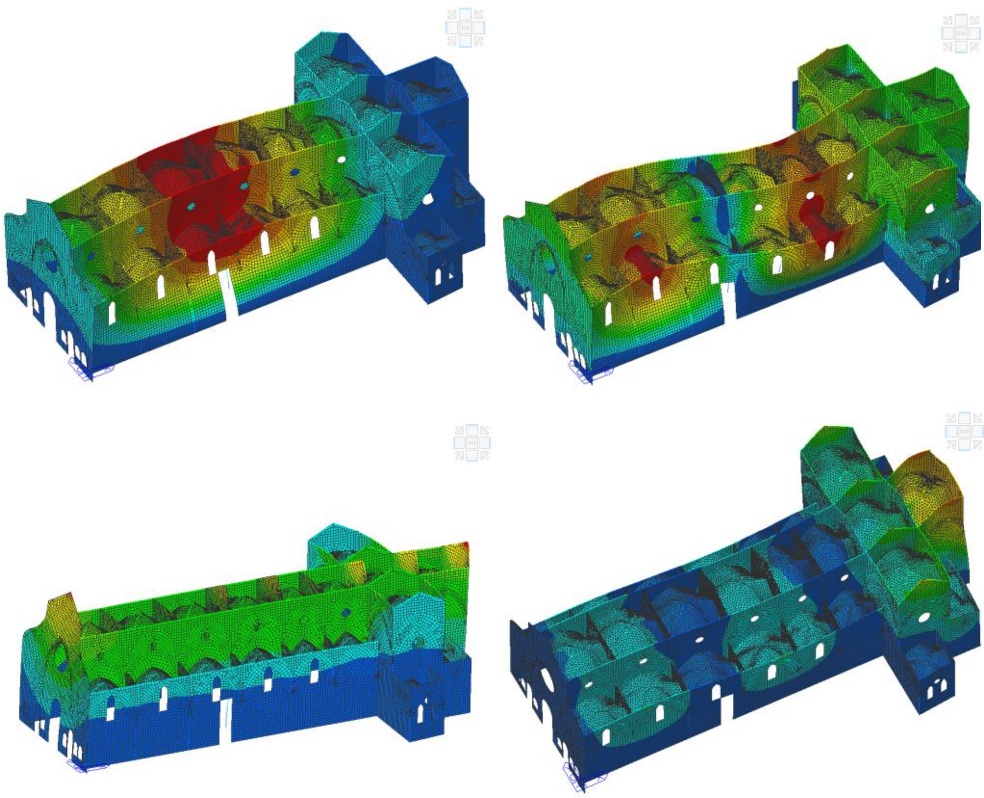

Fig. 7: Vibration mode shapes of modes 1, 3, 4 and 5.

Bending moments, in the longitudinal and transversal directions of the Basilica, are presented in Figure 8 and 9 respectively. These pictures clearly show that the central columns behave like in a shear-type frame. The maximum value of the bending moment occurs in the columns close to the transept, in the longitudinal direction, and in the central part of the nave, in the transversal direction, according to the linear dynamic analysis results [4, 5].

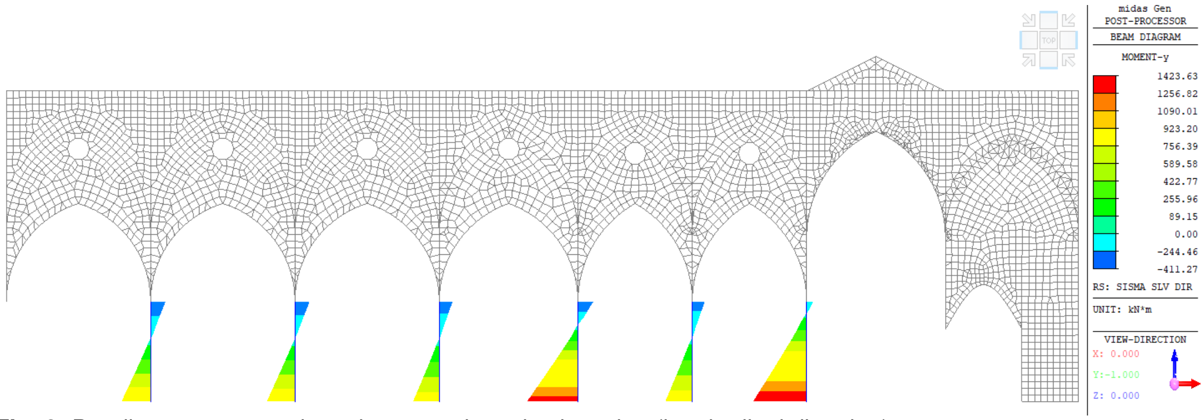

Fig. 8: Bending moment on the columns under seismic action (longitudinal direction).

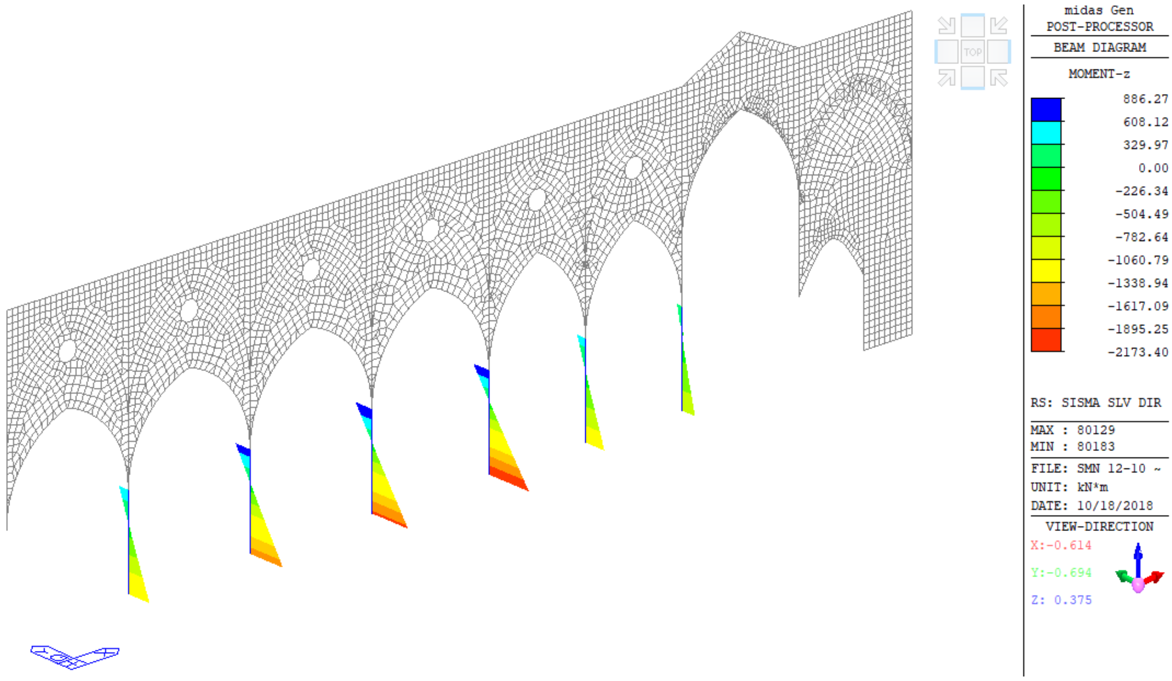

Fig. 9: Bending moment on the columns under seismic action (transversal direction).

The internal action on the structural elements (columns, walls, arches and vaults) obtained from the linear dynamic analysis were combined according to [3] in order to implement the relevant safety verifications. In particular, biaxial bending and shear safety verifications were made for the columns while in plane and out of plane bending, together with shear actions, are checked for walls [6, 7]. Biaxial bending safety verifications for columns were made through Navier's trinomial formula:

$$\sigma_c = \frac{N}{A} + \frac{M_x}{I_x} y - \frac{M_y}{I_y} x$$

by iteratively updating A, I_x and I_y on the cracked section, verifying that:

$$\sigma_c \leq f_{md} = 3.86\ MPa$$

The shear verifications were made, considering the Turnsek-Cacovic criteria [8, 9], checking that:

$$\tau_c \leq \tau_u = 0.36\ MPa$$

For the walls, the in plane and out of plane bending moments are evaluated. These moments should be lower than the corresponding resisting moments evaluated with the following equation:

$$M_{Rd} = \frac{l^2 t}{2} \frac{N_d}{A} \left(1 - \frac{N_d}{A 0.85 f_d}\right) = l^2 t \frac{\sigma_0}{2} \left(1 - \frac{\sigma_0}{0.85 f_d}\right)$$

valid for rectangular section. For shear, the safety verifications are checked against two different failure modes: the diagonal cracking and the bed-joint sliding modes, according to the formulations reported in [10].

4. Evaluation of the risk index

The risk index of the Basilica is obtained considering the ratio between the minimum peak ground acceleration which leads to the first collapse of a structural element:

$$PGA_c = a_g \cdot S = a_g \cdot S_S \cdot S_T = 0.024 \cdot 1.5 \cdot 1 = 0.037g$$

and the design peak ground acceleration (Figure 10):

$$PGA_d = a_g \cdot S = a_g \cdot S_S \cdot S_T = 0.149 \cdot 1.486 \cdot 1 = 0.221g$$

Consequently, the calculated risk index is:

$$I_S = \frac{PGA_c}{PGA_d} = \frac{0.037\ g}{0.221\ g} = 0.167$$

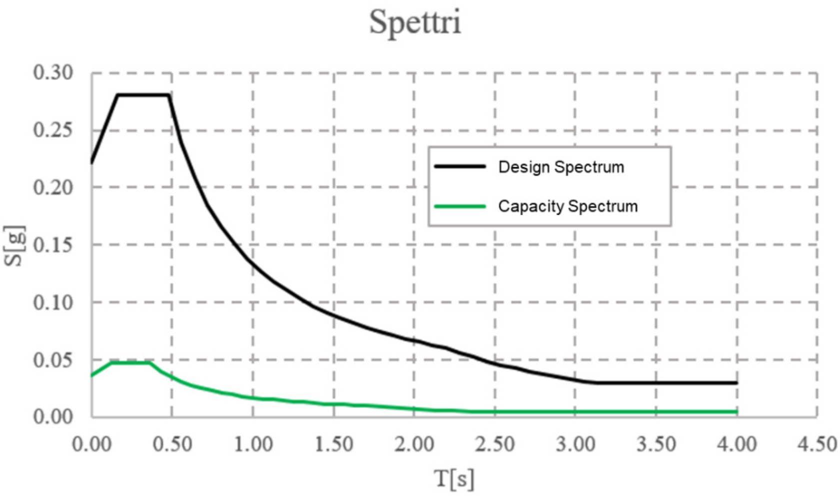

Fig. 10: Design and capacity spectrum.

5. Conclusions

In this paper, the evaluation of the seismic vulnerability of Santa Maria Novella Basilica in Florence is presented. A linear dynamic analysis was conducted to determine the behavior of the most important structural elements (walls and columns) of the structure. The evaluation of the risk index is then expressed in terms of the ratio between the minimum peak ground acceleration which leads to the first collapse of a structural element and the design peak ground acceleration. The analysis showed that the risk index is lower than 1, thus the Basilica is not able to resist the design seismic action.

The eigenvalue analysis shows that the first vibration mode shape in the transversal direction is characterized by a natural period equal to 0.8 s while the fundamental period in the longitudinal direction is close to 0.5 s.

Moreover, the eigenvalue analysis has highlighted the complex dynamic behavior of this structure. In fact, to obtain a modal participation mass equal to 85% in the two principal directions, it was necessary to consider more than 150 modes. This result demonstrates the presence of different local vibration modes, involving only a limited part of the church and giving a difficult interpretation of the global dynamic behavior.

These considerations show that the performed analysis is useful to identify the most vulnerable structural elements of the Basilica.

References

[1] MIDAS Gen Analysis Reference.

[2] LABUSCAGNE, A., VAN RENSBURG, N.F.J., VAN DER MERWE, A.J. *Comparison of linear beam theories*. Mathematical and Computer Modelling, Volume 49, Issues 1-2, January 2009, p. 20-30.

[3] Decreto Ministeriale 17/01/2018, Ministero delle Infrastrutture e dei Trasporti, G.U. Serie Generale n.42 del 20/02/2018 – S.O. 8.

[4] CRESPI, P., FRANCHI, A., GIORDANO, N., SCAMARDO, M., RONCA, P. *Structural Analysis of stone masonry columns of the Basilica S. Maria di Collemaggio*. Engineering Structures, Volume 129, 2016, p. 81-90.

[5] GATTULLI, V., ANTONACCI, E., VESTRONI, F. *Field observations and failure analysis of the Basilica S. Maria di Collemaggio after the 2009 L'Aquila Earthquake*. Engineering Failure Analysis, Volume 34, 2013, p. 715-734.

[6] LA MENDOLA, L., PAPIA, M., ZINGONE, G. *Stability of masonry walls subjected to seismic transvers forces*. Journal Structural Engineering, ASCE, 2005.

[7] LEE, S., PANDE, G. N. *Numerical modelling of brick masonry panels subjected to lateral loadings*. Computer and Structures, Volume 4, 1996.

[8] TURNSEK, V., CACOVIC, F. *Some experimental results on the strength of brick masonry walls.* Proc. of the 2nd Intern. Brick Masonry Conference, Stock-on-Trent, 1971, p. 149-156.

[9] HENRIQUES, J. P., BINDA, L., ANZANI, A. *Testing and modelling of multiple-leaf masonry walls under shear and compression*. Proc. of the fourth international seminar on structural analysis of historical constructions, 2004.

[10] Circolare 2 febbraio 2009, n. 617, Istruzioni per l'applicazione delle "Nuove norme tecniche per le costruzioni".