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Linear and Non-linear Seismic Analysis of a Hungarian Late Romanesque Church

Eduardo C. Morais*[1], László G. Vigh[2], János Krähling[3]

[1, 2] Budapest University of Technology and Economics (BME), Faculty of Civil Engineering. [3] BME Faculty of Architecture. 1111 Budapest, Műegyetem rkp. 3, Budapest, Hungary.

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Abstract

Compared to commonly applied methods, a better estimation of probable intensities of historical seismic events can be achieved by probabilistic seismic analysis of structures damage due to seismic events. The method usually requires analysing historical data on the damage of several structures and their behaviour, in order to evaluate and identify critical points and probable collapse modes that could lead to the described damage. As a case study, preliminary analysis of the Zsámbék late Romanesque church, in Hungary, which collapsed due to Komárom earthquake in 1763 is discussed in this paper. A 3D finite element linear elastic and a nonlinear macro-model have been built from the descriptions and expected behaviour of the structure, and validated through a sensitivity analysis. The critical points have been identified through modal response spectra and pushover analysis (following EC8-1) and comparing the results with the described damage and other research works on this topic. Using the main stresses, strains and displacements and following a step-by-step collapse simulation the probable collapse mode has been evaluated. The numerical modelling was developed in the FEM code AxisVM. The results support the development of nonlinear FEM and DEM numerical models, relying on modal pushover or time-history analysis, and characterizing the global and the specific behavior of some of its structural elements, thus allowing for a better account for future estimations of the intensity of Komárom earthquake.

Keywords: Historical seismic events. Historical buildings. Zsámbék premonstratensian church. FEM modelling. Modal response spectra analysis. Pushover analysis.

1. Introduction

Nowadays for civil and thus to design purposes the security to seismic hazard can be achieved using EC8 recommendations and following the no-collapse and the damage limitation requirements [1]. These two general criteria connect a reference probability of exceedance (10% in 10 or 50 years) for a hypothetical seismic event (with a return period of 475 or 95 years) to the expected structural safety and thus to a good structural response to that seismic event. From this point on two different paths can be taken, either we resort to available seismic data (seismograms) performing a time-history analysis, or we use the seismic hazard maps (following national standards) to find an appro-

* Corresponding author: Tel: +36 1 463 1825 / Fax: +36 1 463 1784
E-mail address: eduardo.charters@gmail.com Sponsored by CAPES (process no. 9178-13-9)

priate peak ground acceleration to proceed with an equivalent force method, modal response spectrum or even a pushover analysis. The available methodologies can be pretty straightforward and appropriate to modern needs, but what to do when the structure under analysis is a historical structure? Knowing that the available seismograms cover only a certain part of the history (instrumental part, approx.100 years) how representative can they be?

The questioning seems to lead to simpler methods, which sometimes are not conservative plus they usually rely on seismic hazard maps, which despite taking account for a wide data spectra (geological, geophysical, geotechnical, and paleo, historical and instrumental earthquakes data) and a demanding methodology, like in [2], great uncertainties remain. Furthermore its relevancy to historical architecture is low and the behaviour of real structures is totally neglected. Therefore the reliability of design resorting to seismic intensities is often challenged. Thus appears the idea that the determination of seismic intensities should require a stochastic method with the analysis of real structures. The methodology is known and enables the probabilistic estimation of the magnitude of historical earthquakes [3].

For that matter it is usually required to analyse historical data on the damage of several structures and their behaviour, evaluating and identifying critical points and probable collapse modes that could lead to the described damage, which requires a reliable numerical model of the structure under analysis. An early approach to this methodology is here explored. As an historical earthquake is addressed it requires the analysis of historical structures. In this paper a case study is presented that regards the modelling and analysis of the Zsámbék late Romanesque church. It collapsed due to Komárom earthquake in 1763, and ruinous state can be seen in figure 1:

(a) (b)

Figure 1. Zsámbék church: (a) western and (b) northern views (09/06/2014).

The wide usage of the concrete and the steel as construction materials stands between one and two hundred years. Therefore, most of the ancient constructions that constitute our cultural heritage resort to stone blocks, bricks, wood, mortar, or to a combination of these construction materials and to traditional construction and design methods. Therefore, neither the material nor the construction technologies resemble our contemporary design conditions.

Masonry buildings, in which natural stone blocks (or bricks) connected by mortar constitute its structural elements, such as walls, arches and vaults, are very suited to resist to vertical loading (i.e. self-weight) with local low stress levels [4], but the same does not happen when subjected to horizontal loads, which create bending in the walls and columns and therefore tension in elements with low tensile strength (especially true when the building has high and slender walls and columns for which the bending is clearly dominant). In these structures all elements contribute to load bearing capacity this fact creates two main difficulties for a reliable analysis. The first is that any slight settlement, due to ground motion or local failure will cause the change of the equilibrium configuration of the structure [4]. Therefore both the material resistance and the structure equilibrium will be affected by an historical component of difficult estimation and very often neglected. The influence of the structures history evidences its uniqueness as a structure, what leads us to the second point: that becomes very difficult to resort to quantitative simplified mechanical models and/or analysis methods to interpret and assess its actual resistance and behavior. Various sorts of uncertainties blur any presumption of exactness for it is a begging question to know the history of the structure with relative precision. Besides, it is difficult to gather experimental data from the actual structure in a non-destructive way is also an obstacle. These facts are highlighted by Vignoli & Betti *"monumental historical buildings are by definition buildings that is difficult to reduce to any standard structural scheme because of the uncertainties that affect the structural behaviour and mechanical properties"*[5]. Conclusion that supports that both static and dynamic analysis of historical masonry structures should resort on a specific modelling strategy built from reliable historical information such as descriptions of structure, damaging events, restoration interventions, construction methodology and materials, aspects that strongly intervene on the generation of the numerical model [6, 7].

This study specifically addresses the problem of seismic analysis and modelling of historic buildings with the scope on the late Romanesque church of Zsámbék. This structure, as earlier mentioned, is nowadays a ruin and paradoxically a paradigmatic case of monuments preservation in Hungary. From its construction to its actual state it suffered a certain number of interventions and restorations. A description of the structure before the collapse is presented followed by a brief description of the main interventions in the structure, modal response spectra analysis and step-by-step collapse simulation. The present paper extends the previous study, carried out by the authors in [8] as it focus on more detailed features as further developments on the geometrical model and work on the material model, enabling an account of the material nonlinearity and a Pushover analysis. The numerical modelling was developed in the FEM code AxisVM version 12 [9].

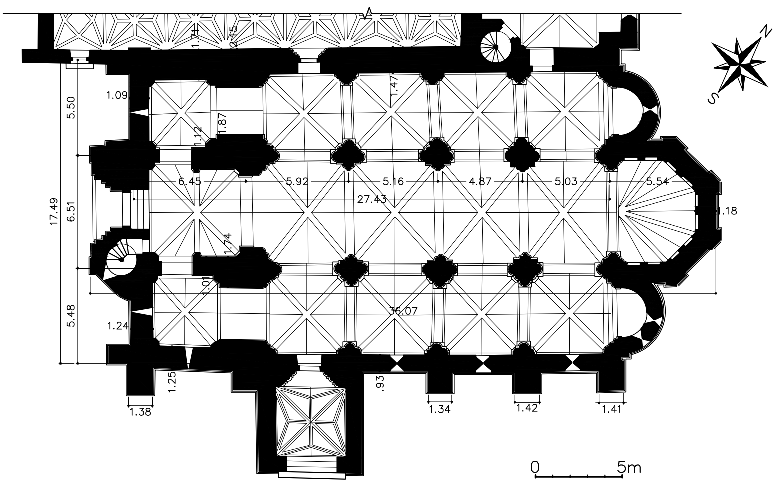


Figure 2. Low plan of Zsámbék church before the Ottoman occupation (reconstruction)[2].

2.The case study

2.1 Description of the case study

[2] The usual orientation of churches' façade is said in history of architecture to be towards the west, although, this study will follow the compass card displayed in the upper right corner of the low plan.

The church under study is located in Zsámbék (Pest County, Hungary) and it was originally built for the *Prémontré* order in the 20's of the 13th century [10, 11]. It follows a typical basilica layout: one central nave, two lateral aisles, two towers on the facade and triple apse (fig. 2). It approximate dimensions are: length of *36 m*, width of *20 m*, wall and towers height of *17 m* and *35,5 m*. The walls thickness varies from *0,90 m* to *1,75 m*. The main central nave is about *27 m* long and *6,50 m* wide and two lateral minor aisles are *5,50 m* wide. The central nave ends in an octahedral apse and the aisles end in lateral in minor circular apses in the north-eastern wall. It is still a begging question whether or not the previous figure represents the structure that fallen in 1763 due to Komárom earthquake, if the monastery was in service or if a side entrance and cloister with star shaped ribbed vaults existed. According to a report of Miklós Jankovics from the beginning of the 19th century, he had been altar server in the monastery church in 1790 [12]. On the contrary, the premonstratensian monk O. Fényi, has found few signs of mass or liturgical action in the monastery church after the expulsion of the Turks in 1686, and deliberately initiated degrading activity resulted stone material for the new parish church after 1749 [13]. The only eyewitness of the demolition caused by the 1763 earthquake on the monastery church was the local priest Kummer who has only reported on the collapse of the northern octagonal pyramidal spire and does not mention the collapse of the north wall of the nave - as this would then certainly not stood [13]. To unlock these sporadic controversial data additional resources would be needed, so it is reasonable to assume that the church was in the middle of the 18th century partially damaged, but some recovery was estimated as usable. Thus despite some lack on information about the actual structure a structural model can be built assuming some symmetries and simplifications: the thickness of the walls can be assumed to be those of the figure; the same goes for the columns, arches, vault ribs and other overall dimensions.

2.2 General damage description

Given the overall dimensions, it is an important issue to individuate damage description associating them to the respective damaging event. For that matter it is important to access systematized surveys on post-earthquake in-situ observations with conclusions on the typical damages. One example of damage survey template was published by the Italian *Dipartimento della Protezione Civile* in 2001 with detailed drawings on the most important crack patterns in churches after earthquakes [14]:

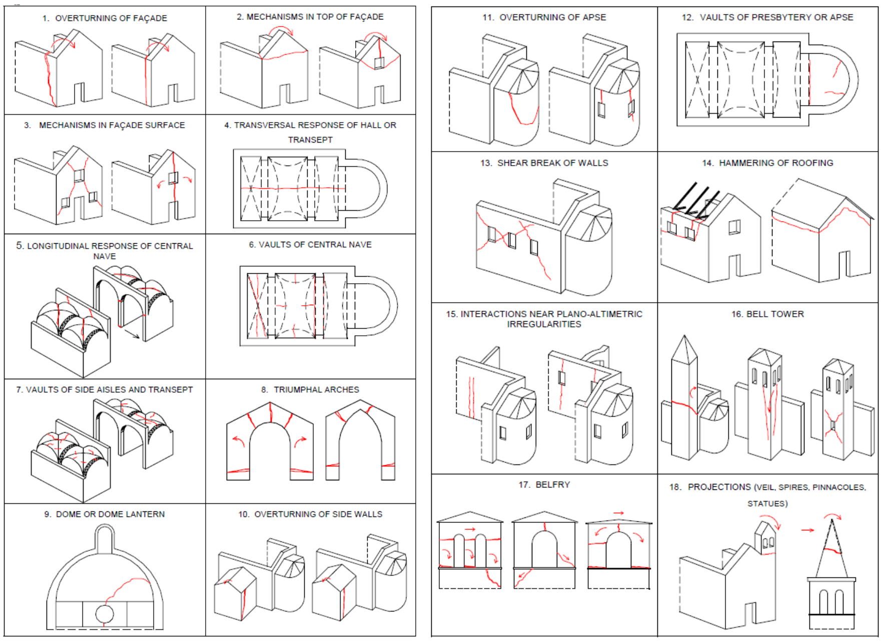


Figure 3. Collapse mechanisms and crack patterns in churches, from [14].

The previous template is particularly relevant for the present case study, but it must be complemented and evaluated regarding the singularity of Zsámbék premonstratensian church as a historical building structure. The following table (chronologically displayed) resumes the most important events in the history of the building:

Dates	Relevant Events
1220's	Beginning of Construction.
1258	Finished construction. Probably not 100%, for the construction work in fault was completed in the following years.
1475-80	Significant reconstruction.
From late 17th cent.	Zsámbék is property of the Zichy family
1749-52	New baroque parish church built in the village; Reconstruction of the ruin ordered by Márton Bíró in 1754.
1763	Collapse of parts of the monastery church due to Komárom earthquake.
1848	Parish priest György Gózon fought for the reconstruction without any result.
1855	Karl Weiss reports on the ruin but with no effect.
1870	New priest tries to gather money to support its reconstruction.
1880	György Klösz takes 8 photos and the restoration is discussed due to a report of the consistence of ruin.
1882	Two engineers, Gusztáv Zsigmondy and Antal Khuen analyzed he ruin and propose to reniew the northern tower, which reveals static problems.
1889	István Möller intervenes to protect the ruin.
1935-39	Géza Lux intervention and renewal.
1957	Collapse of the entry hall vaulting (rebuilt later).
1958	Collapse of the stairs (rebuilt later).
1962	A lightning stroke the western tower (rebuilt in 1963) so two lightning rods were added to both towers.
1960's	Plans on how to definitely intervene in the ruin so that it can be protected and usable.

Table 1. Brief chronology of the interventions in Zsámbék church (from [10, 11]).

Besides the chronology it can be added that much of the destruction and absence of material can be attributed to human causes and environmental degradation: much of the stones were taken either by particulars or to rebuild the new parish church after 1963's earthquake and the remaining ruin of the premonstratensian church was not protected against frost-defrost cycles or burglar actions [10,11]. So far it can be said that its collapse consisted on the collapse of the vaulting system in the central nave and lateral aisles and on the collapse of the north-western (NW) central nave and aisles walls.

3. Analysis methodology

3.1 Structural behaviour and numerical modelling

The general schema in which the structure of a three nave vaulted Romanesque functions can be analogue to nowadays reticular structures: the vaults (similarly to plates) and some walls load the arches and ribs (resembling beams) which in turn load the columns and other walls. The main objective is, after all spanning space in the nave and aisles, and for that matter the vaulting system can be regarded as the basic composed structural unit that can be found to modulate – in smaller or bigger scales – the whole structure (fig. 4).

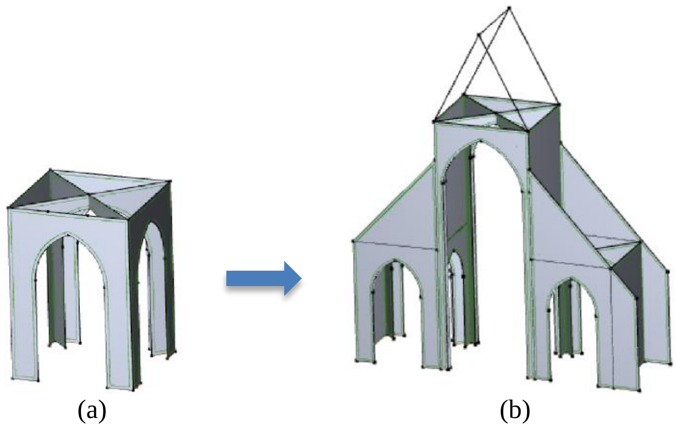


Figure 4. a) Basic vaulting unit and b) transversal section

It can be added that the walls of the central can be laterally unstable (because of the arches and vaults on the supporting peers, or acting wind loading) for which one shall resort to buttresses or dormants or even to transversal walls to assure the stability of the structure. When subjected to an earthquake the lateral resistant elements are walls and dormants helped by the rigidity of the arches and vaults intersecting points and by the diagonal ribs of the vaulting. From the static analysis, with low compression stresses, to the dynamic analysis it is expected to have increased stress values, with appearance of significant tensile stresses. In real structures the zones in which some concentration of stresses is predicted are secured by the use of better quality material (usually carved stone, which for Zsámbék church is limestone): it is the case of the ribs of the arches and vaulting, the columns and the eaves of the windows and towers.

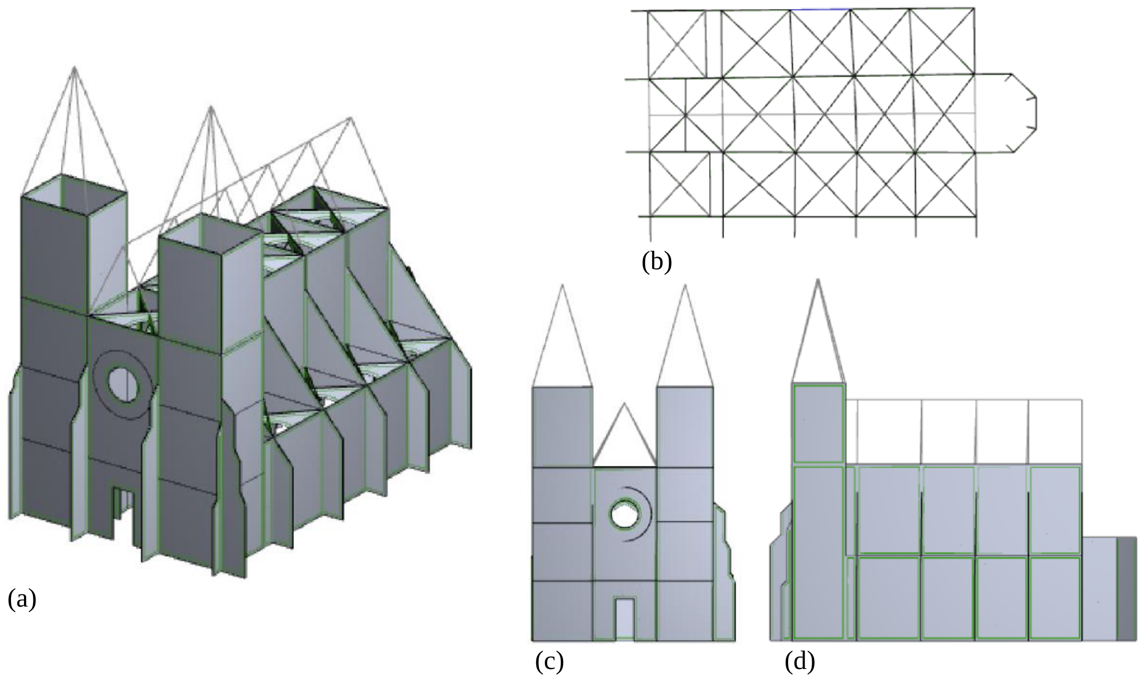


Figure 5. Early model: (a) perspective, (b) top view, (c) façade and (d) lateral view.

The development of the geometrical and successive numerical models was carried out using a macro-modeling strategy [6, 7] in five major modeling phases of increasing complexity: understanding and modeling of the main resisting structural elements (manly in 2D); development of a 3D composed model of shell elements with openings; simulation of the vault rigidity on the 3D model using horizontal shell elements; simulation of the vault rigidity on the 3D model through modeling of the vault ribs and by assigning rigid diaphragms to the openings between the ribs and the arches; and finally the refinement of the geometries and considerable openings. The result of the fourth step is a 3D macro-model (fig. 3) in which the structural strength is not direction independent. The numerical modelling was developed in the FEM code AxisVM version 12 [9].

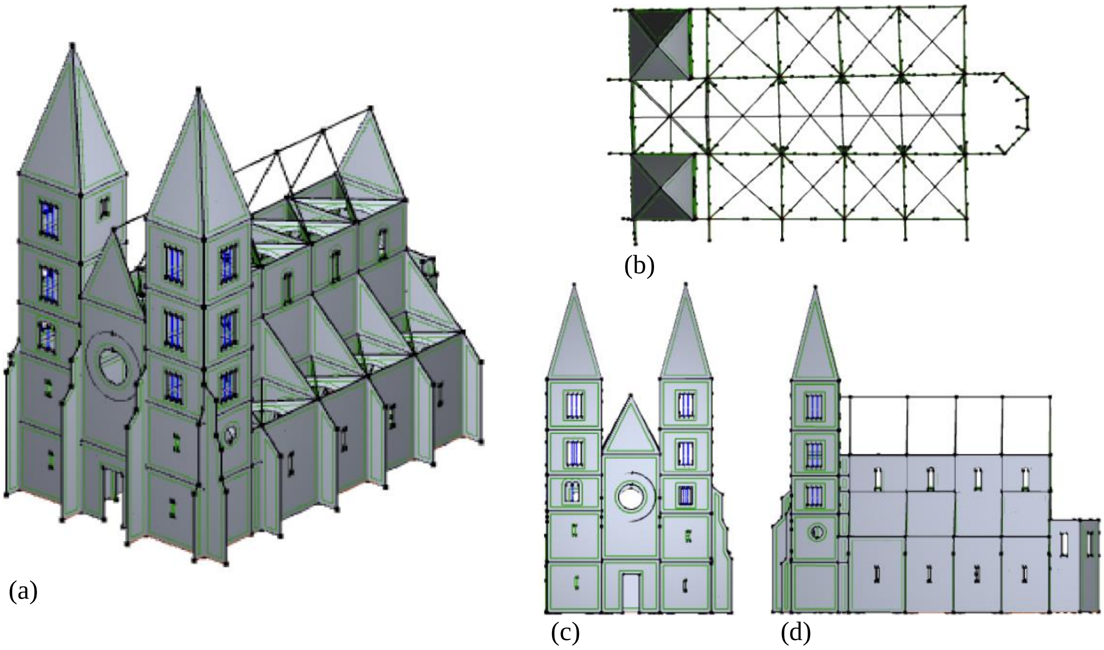


Figure 6. Latest model: (a) perspective, (b) top view, (c) façade and (d) lateral view.

When the ground acceleration affects the structure pushing it from northwest the transversal arches, walls and buttresses will not resist independently. In fact thanks to the diagonal ribs some rigidity will be added to the structure allowing the arches, vaults and ribs to work all together when the structure is excited in some direction. Besides the usual reductions some other assumptions are required to build an initial geometric model, such as symmetry, regularity and continuity. After building the linear elastic Finite Elements (FE) numerical model it requires to be validated so we can assume to be stable for the applied material parameters.

Because in previous steps the pinnacles of the N and S tower pinnacles, façade and back wall tops were object of the low frequency vibration, originating several non-significant vibration modes. Although, with the 5th modelling phase these elements were reintroduced. The final reviewed model can thus be seen.

3.2 Sensitivity analysis and mechanic properties

A sensitivity analysis was ran and the variation of the maximum stresses and displacements was evaluated dependent on the variation of Young modulus (E), Poisson ratio (ν) and density (ρ). The adopted the range of values following limestone and sandstone walls and stone test specimens following [15]:

p_i	$\Delta E/E$	$\Delta p/p$	$\Delta\sigma/\sigma$	$\Delta d/d$
E	3488 to 1770	49,3%	49,7%	94,0%
ν	0,33 to 0,09	72,7%	11,6%	5,3%
ρ	2570 to 1600	37,7%	59,3%	53,3%

Table 2. Relative values for the variation of the maximum stresses and displacements.

As can be seen, both the young modulus and the density variation largely affects the main stresses and displacements, but the same can not be said for Poisson ratio, for which a *72,70%* variation caused a minimal variation. As expected, young modulus seems to largely affect the displacements but the density both stresses and displacements. Furthermore stability of critical points location with parameters and mesh size variation can be observed, and the mechanic parameters that characterize the shell elements can be thus be chosen within values that do not exceed those tested in the sensitivity analysis. Therefore, the assigned material parameters values for the linear elastic material can be chosen within those of table 3 first row. As to the new density and elasticity modulus can be calculated more accurately based on experimental results. For the other parameters they follow average results from experimental data on 3-leaf masonry wallets [15]:

	$\gamma_d (kN/m^3)$	E_{sec} (MPa)	σ_c (MPa)	σ_t (MPa)	ν (-)
Linear model	18,00	2800,00	0,50	0	0,20
Non-linear model	18,47	1925,27	-5,41	0,541	0,12

Table 3. Mechanical properties for the shell elements.

Pushover analysis implies not only geometrical but also material nonlinearity, and therefore, some further considerations shall be made. In an earlier phase the identification of critical points in the structure was enough, although, a more precise material description must follow when more accurate results are needed. To characterize material nonlinearities a bilinear material model was applied (fig. 3 & 4), following the walls properties for limestone walls and the two first simplified formulae tested in [15] (for keyed and straight collar joints), and the average width of the walls for the present case study.

	$\varepsilon(-)$	σ (MPa)
1	-4,00E-03	-5,41
2	-2,81E-03	-5,41
3	0	0
4	5,19E-04	1,00
5	1,709E-03	1,00

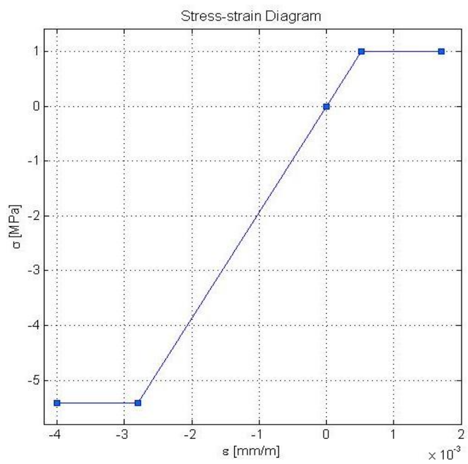


Figure 7. Stress-strain diagram adopted as nonlinear model for pushover analysis.

The process of geometrical and mathematical finitization resulted on a final mesh grid with *0,90 m* average width, both for the linear elastic modal response spectra and non-linear plastic pushover analysis. It was selected a regular mesh instead of an adaptive one, because of the appearance of several distorted elements in the latter case.

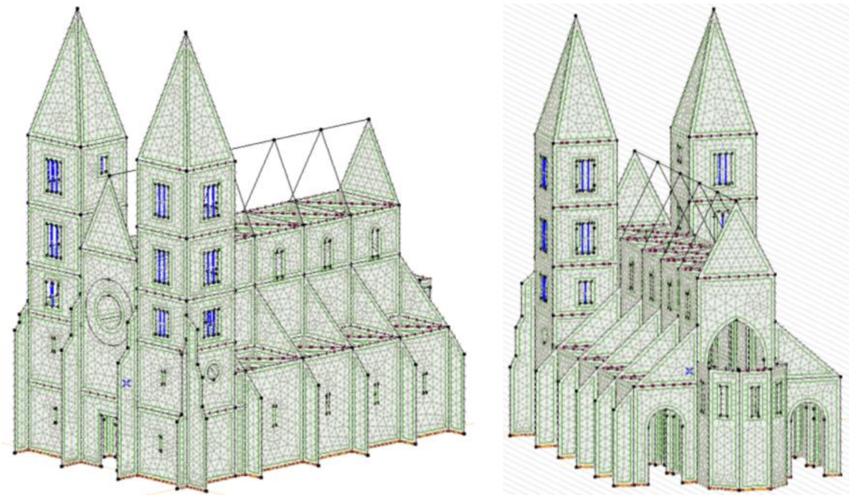


Figure 8. Mesh grid for the final numerical model.

4. Structural and Seismic Analysis

4.1 Critical Points ID

To identify the critical points maximum stresses and displacements resulting from a modal response spectrum analysis were evaluated in the 3D macro-model. As can be seen (fig. 4a), the absolute value of the maximum stress far exceed the ultimate stress of *0,50 MPa* (merely as an early reference value), although, the structure did not collapse in these points. Exception goes to the maximum stresses (originated from torsion probably) in the NW tower, where we can see (fig. 1a) that some intervention roof was required.

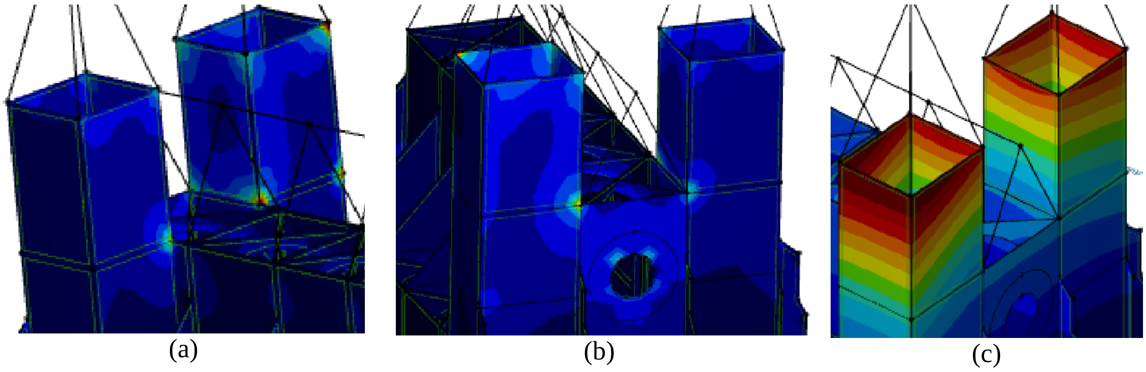
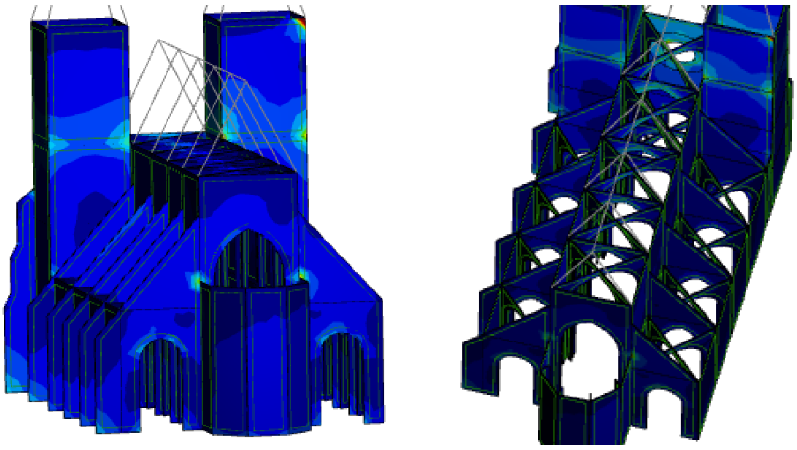


Figure 9. Critical stresses on the (a) façade, (b) back walls and (c) displacements on the tower walls.

These results can be explained by the difference in slenderness of the towers block compared with the nave and aisles block. These two parts act separately as in the first bending is clearly dominant and in the second probably shear is dominant, and as confirms figure 5b, maximum displacements can be expected in the towers top. Thus, being the structure excited those parts behave differently and stresses appear in their connection edges. This result is typical [7].

It must be added though that the arches and vaulting system in the central nave and lateral aisles has a dynamic on its own. Appearing critical points in some arches and vault ribs, not only in parts connected or close to the towers, but also in the last buttresses and back wall (fig. 5).



(a) (b)

Figure 10. Smaller critical stresses on (a) back wall, buttress and (b) arches and vaults.

4.2 Collapse Step-by-step Simulation

The main purpose of this collapse procedure is to validate the model, as a valid and reliable model would show as the critical points the damaged parts of the structure now in ruin. Therefore an evolving procedure in which the critical points signal the damaged parts that are successively being eliminated from the rest of the numerical model should provide at least some insights (despite non taken nonlinearities) on the important elements (within the seismic phenomena) that led to the actual state of ruin. It follows the steps description:

Step no.	**Step Procedure / Collapsed element**
1	NW and SE towers
2	Rigid diaphragms release on the vaulting system
3	2nd vault in the NW aisle
4	2nd and 3rd vaults of the central nave
5	3rd wall panel of the NW aisle
6	4th and 5th vaults of the central nave
7	1st, 3rd and 4th vaults of the NW aisle
8	1st vault of the central nave
9	2nd arch of the central nave
10	3rd and 4th arches of the central nave
11	NW 1st wall panel of the central nave
12	2nd and 3rd buttresses and arches of the NW aisle
13	4th wall panel of the NW aisle
14	Central nave 4th panel NW wall
15	2nd and 3rd panels and 2nd, 3rd and 4th arches of the central nave NW wall
16	NW part of the central nave apse wall
17	5th buttress and respective arch of the NW aisle wall

Table 4. Step-by-step simulation procedure.

As stop criteria for the procedure it was used the apparent ruinous state of nowadays structure. The remaining structure, or ruin, in a practical sense, can be concluded to be very similar to the result of the following model (fig. 6), where some critical points can still be seen to appear in intervened spots, like the corner of the back wall and some buttresses tops.

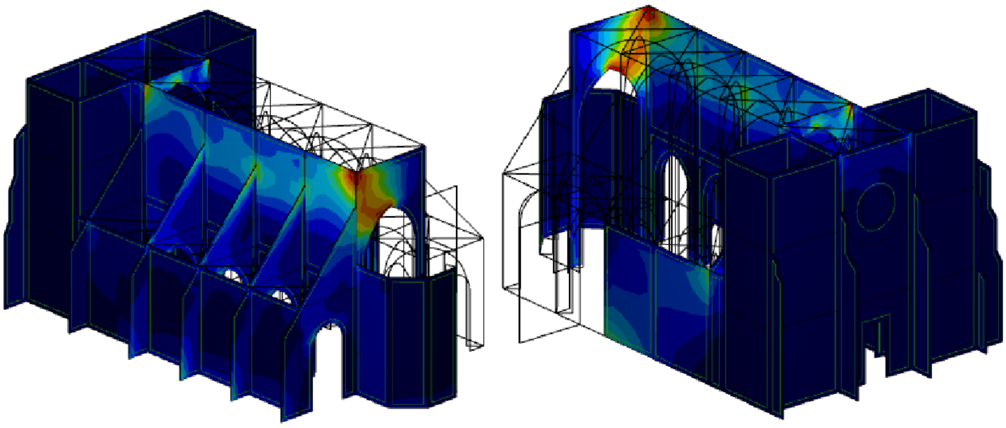
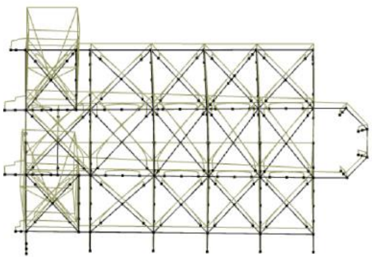
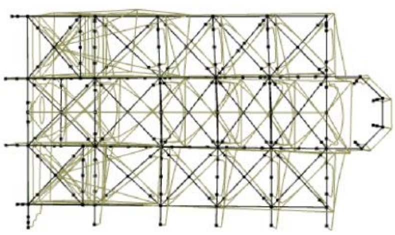


Figure 11. Final result of the step-by-step procedure.

As the verifiable collapse of NW aisle wall (fig. 1) should be explained, it was thought to be essential the addiction of a new wall perpendicular to the NW aisle wall, more precisely in the middle of the 3rd panel wall. It pretends to model the influence of a perpendicular wall from the monastery and cloister parts (*vide* fig. 2). As final comment for this methodology it must be added that the procedure present in table 4 could not be dealt with without some ambiguities, for it required some charity as principle to be performed. Sometimes, from a certain quantity of critical points that successively appeared during the procedure, it was necessary to eliminate some parts that were known to be the collapsed ones in detriment of others that today still stay. Therefore this procedure can be said not to be an exact methodology, but rather a simplified inquiry procedure in which the evaluation of the numerical model accompanies the comparison with the remaining ruin.

4.3. Pushover Analysis

For the same mesh grid, but for the final developed model a pushover analysis will be here presented. Although, and due to the differences in the geometrical and material models, a straightforward conclusion cannot be taken without some previous careful analysis and comparisons. To help settling the statements over the key differences in the analysis procedures, before proceeding to the pushover analysis, the first two vibration modes of three different modelling phases where compared (table 5): the initial model (fig. 5) with linear elastic material properties (table 3), an intermediate model with nonlinear material properties but with the final geometrical configuration (just without the pinnacles and walls tops), and the final model (fig. 6), for which the pinnacles and the walls tops were added. The comparison can be followed in the next table:

	1º Vibration Mode	**2º Vibration Mode**
Initial model		
	$f = 4{,}27$ Hz ; $\varepsilon_x = 0{,}002$; $\varepsilon_y = 0{,}481$	$f = 5{,}31$ Hz ; $\varepsilon_x = 0{,}401$; $\varepsilon_y = 0$

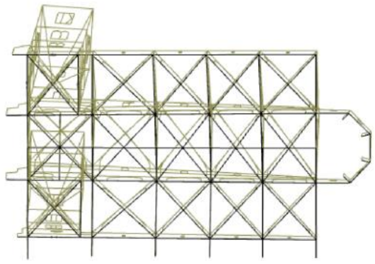
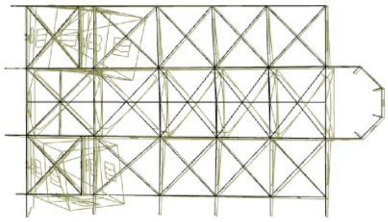
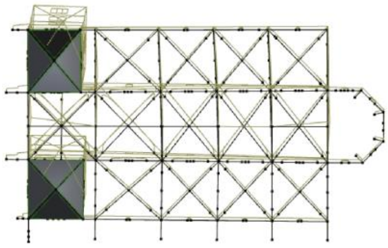
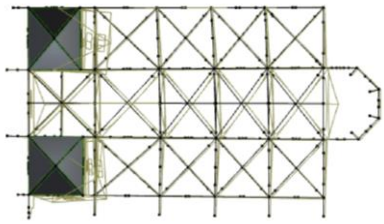
Intermediate model	 $f = 3{,}30$ Hz ; $\varepsilon_x = 0{,}009$; $\varepsilon_y = 0{,}472$	 $f = 3{,}87$ Hz ; $\varepsilon_x = 0{,}335$; $\varepsilon_y = 0{,}006$
Final model	 $f = 3{,}14$ Hz ; $\varepsilon_x = 0{,}009$; $\varepsilon_y = 0{,}406$	 $f = 3{,}54$ Hz ; $\varepsilon_x = 0{,}265$; $\varepsilon_y = 0{,}007$

Table 5. First vibration modes, respective frequencies (f) and modal mass ratios (ε_i).

Comparing the previous results some general conclusions can be drawn. Firstly, that the building is generally more flexible in *x* direction than the *y* direction. Secondly, that the addition of the openings (from the early to the intermediate model) clearly reduced the rigidity of the structure, even despite the change in the mass configuration and an increase in material mass of ~*4,20%* from the early model. There was also a change in the mass of the building from the intermediate to the final model, as the pinnacles and wall tops were added. This small shift may explain the difference in the frequency for the two vibration modes in comparison, and as the loads were also converted to masses the total mass during vibration analysis, the results suggests that not only the total mass but also its distribution is relevant for the vibration modes. As to the difference between the intermediate and the initial models, which lacked these elements (pinnacles and wall tops), it can be explained by the decrease in stiffness (due to *31%* reduction in the young modulus) with not significant but slight decrease of material mass or mass configuration of the building, even with the introduction of openings and the almost unexpressive increase of specific weight (~*2,60%*).

After the vibration analysis and comparison the nonlinear pushover was held for the final model (fig. 6). As in the previous subchapter the critical points were evaluated, based on the displacements and total strains ($\varepsilon_t = \varepsilon_{el.} + \varepsilon_{pl.}$) corresponding to four significant vibration modes (modal mass ratio $\varepsilon_i > 0{,}20$). The capacity curve (fig. 13) considering the 1st vibration mode can be seen to be nonlinear, confirming the appearance of plastic deformations in the structure.

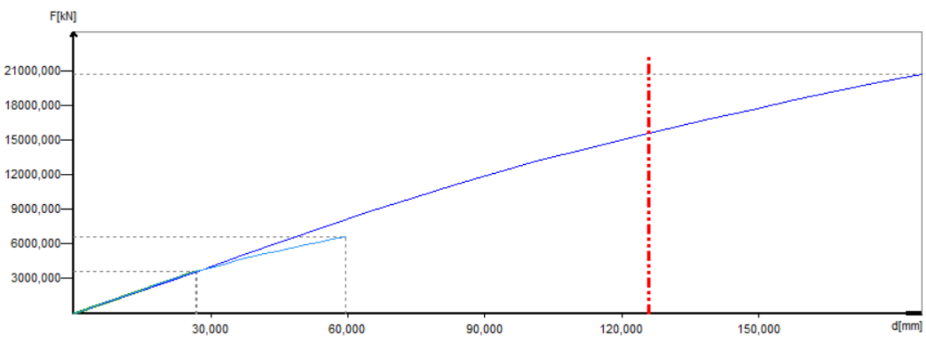
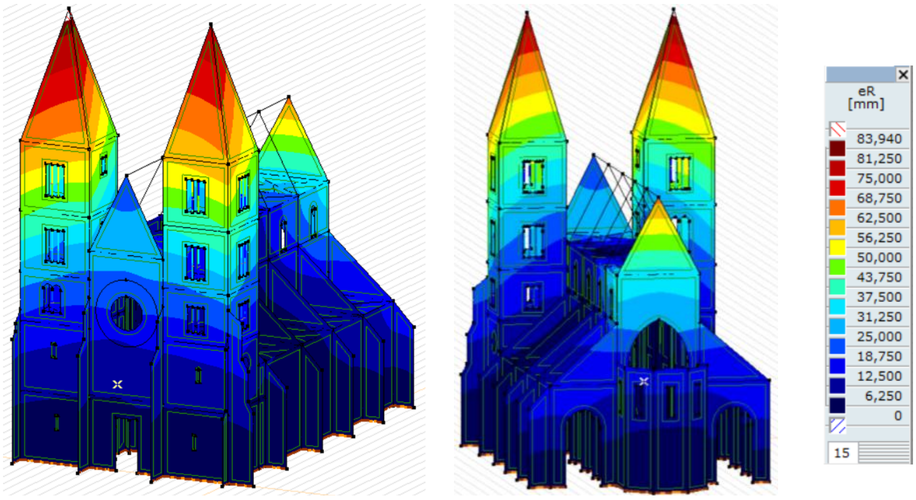


Figure 13. Nonlinear capacity curve for the 1st vibration mode.

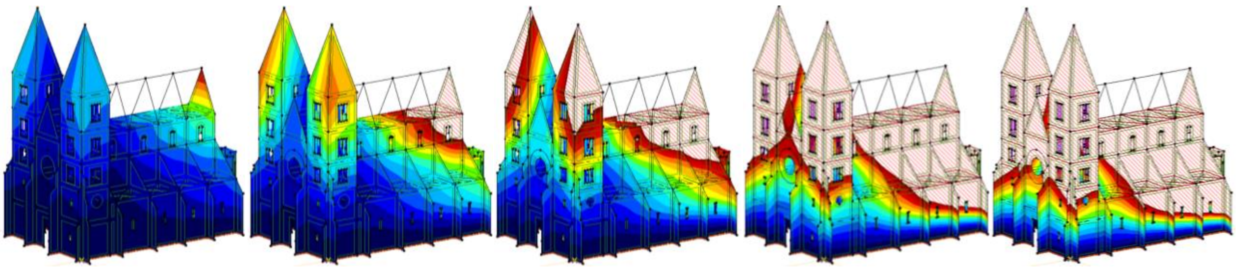
In this paper only the 1st vibration mode which is the most significant one will be here presented and some general tendencies will be mentioned regarding the other three vibration modes. The first two vibration modes develop in *y* and *x* directions, although, the other two are mainly torsional modes, which originate different pushover load configurations.



(a) (b)

Figure 14. Maximum resultant displacements for the 1st vibration mode.

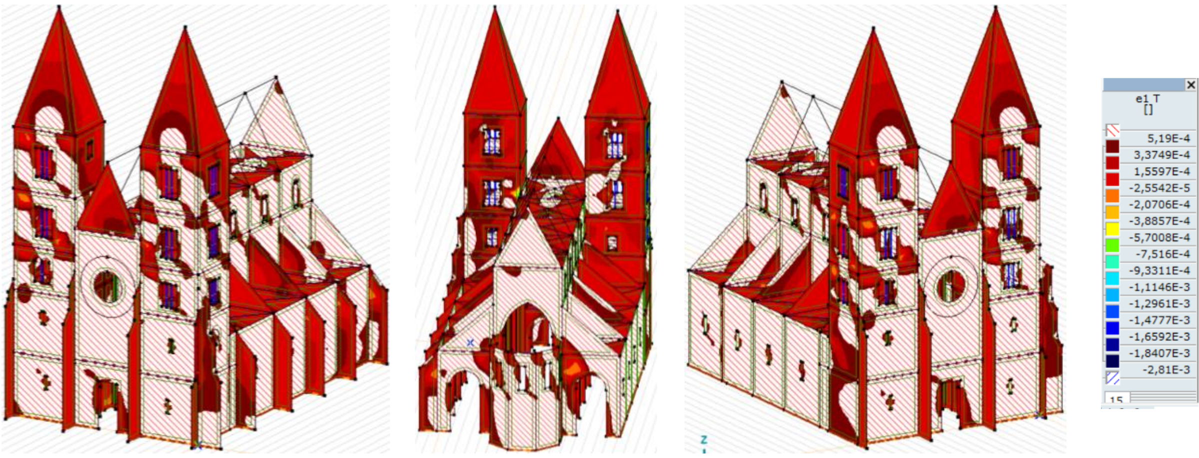
Given the results it can be generally understood that the addition of the pinnacles considerably shifted the behavior of the structure. This corroborates the idea that it is not the mass but its configuration and distribution that is relevant to global dynamics of the structure. The maximum displacements in the structure happen in the recently added SW tower pinnacle and back wall top (fig. 14). Compared with the maximum displacements distribution in the linear elastic model (fig. 9c), some considerable displacements in the main nave last vaults can be observed. Generally speaking the most flexible parts of the structure develop maximal displacements (which happen in different elements considering the different vibration modes): towers, pinnacles, wall tops and back part of the central nave, extending gradually to the central nave vaults and side walls. This tendency can be seen in the selected iterations steps presented in the next figure:



1 2 3 4 5

Figure 15. Development of displacements along the iterative process (limit settled at *83,94mm*).

As to the development of strains in the structure, following figures 16 & 17 show the structural elements where the development of plastic strains in main directions occur.

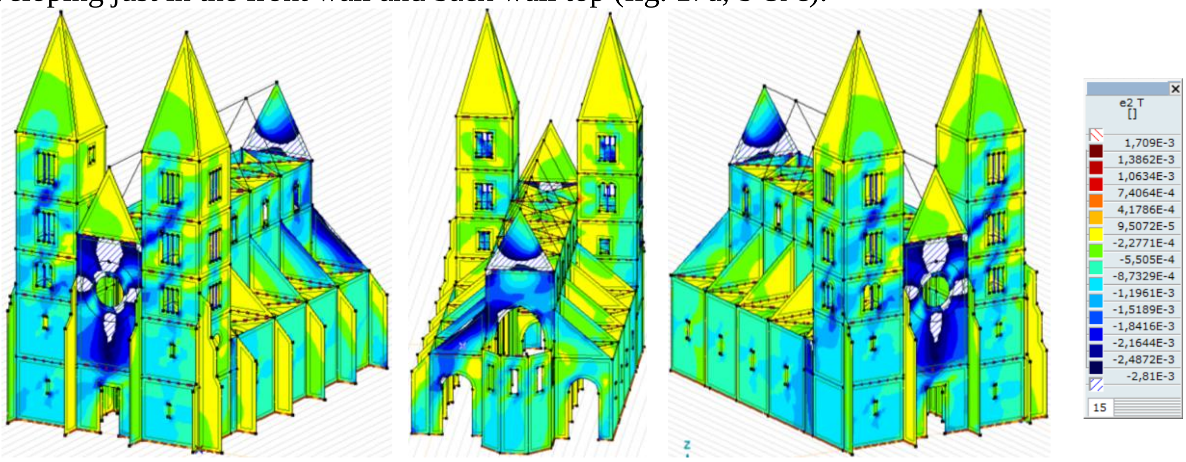


(a) (b) (c)

Figure 16. Total strains in the 1st principal direction ($\varepsilon_{t,1}$).

These results evidence plastic strains occurring from tension (red shaded), mostly in the first main direction. As to their distribution in the structural parts, it can be said that there is a very wide dispersion of tension strains throughout all the structure: walls, main nave and side aisles vaults, dormants and towers (fig. 16a, b & c).

These results evidence plastic strains occurring from compression (blue shaded), mostly in the second main direction. These Compression strains are more restricted than the tension strains, developing just in the front wall and back wall top (fig. 17a, b & c).



(a) (b) (c)

Figure 17. Total strains in the second principal direction ($\varepsilon_{t,2}$).

The following table shows the development of strains for the remaining and significant vibration modes:

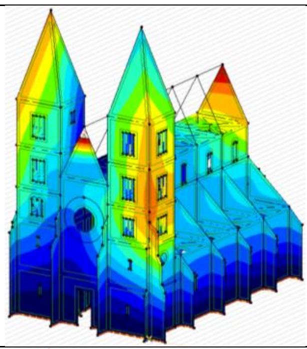
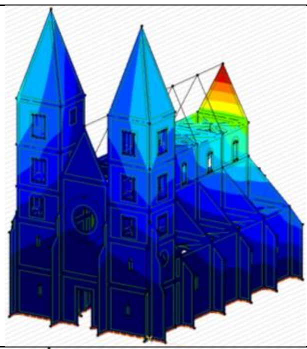
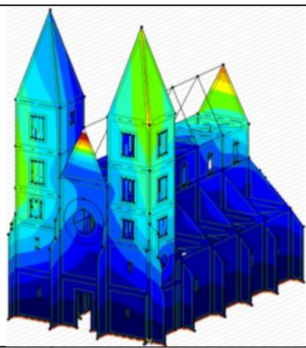
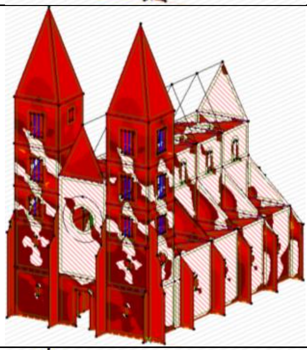
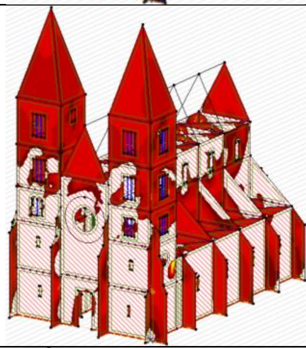
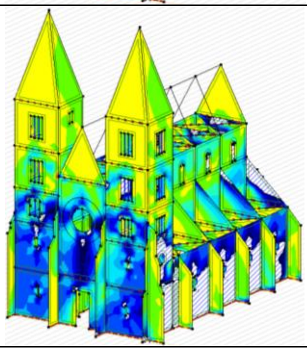
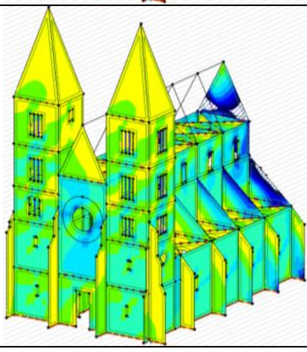
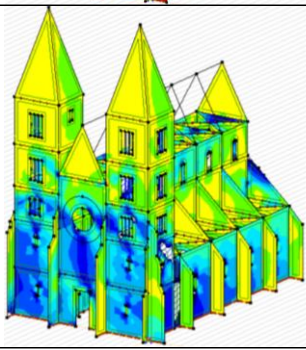
	2nd v. m.	**3rd v. m.**	**4th v. m.**
Displacements			
$\varepsilon_{t,1}$ strains			
$\varepsilon_{t,2}$ strains			

Table 6. Displacements and total strains for the 2nd, 3rd and 4th vibration modes.

The plastic strains in principal directions for the 1st vibration mode (fig. 15 & 16) are consistent throughout the other significant vibration modes (table 6). It can be added that those vibration modes suggest a consistent extension in the development of the plastic strains into other structural elements (in the walls connecting the towers and the nave, side walls, dormants, vaults) both for tension and compression sides, which also provides a possible explanation of the wide damage in the church. This consideration suggests that several vibration modes (with modal mass ratios in *x* and *y* directions, and values $0{,}23 \leq \varepsilon_i \leq 0{,}41$) can be responsible for the global seismic behavior of the structure, and therefore it would be interesting and recommended for further developments to consider them in a modal pushover analysis, enabling their combined analysis.

5. Conclusions

For the linear elastic 3D macro-model, despite its simplified nature, the location of maximum stresses and displacements identified as critical points is typical and coherent with an intuitive interpretation of the structural behavior of the case study. That is probably the reason why a collapse step-by-step simulation could be drawn to a certain level from which the characterization of the non-linear properties of the masonry would be essential. This 3D linear elastic model appears to explain part of the damage in the church, but not all. However the collapse procedure allows the conclusion that the existence of an early monastery with vaulted cloister could have been influential for the collapse. The transition from the early linear-elastic model to the later 3D nonlinear macro-model tried to account for some of the obvious uncertainties in the previous linear elastic model. In the earlier model, the results suggested that a more developed/detailed historical survey could provide a good basis for the development of an advanced non-linear numerical model, one that could characterize the specific behavior of some of its structural elements. The description of the local priest about the demolition of the monastery church and some other descriptions about the structural damages of the new baroque parish church [12, 13] Zsámbék would deserve more attention for a future correlation research between damage survey, damage modelling and intensity estimations. To make a better account for future estimations of the intensity of Komárom earthquake it is important to match the damage descriptions of the different studied structures with the results of their respective numerical models, as they would be more reliable models.

The later non-linear model, as trying to attend the demand for material and geometrical nonlinearities, revealed the wide development of plastic strains both in tension (namely in first principal direction) and compression tension (namely in second principal direction) sides caused by the lateral accelerations imposed to the structure. The occurrence of wide ranged tension strains is important, as masonry structures tend to be sensitive to tension. It also can explain the wide range damage (or most of it) that can nowadays be seen in the intervened ruin. Despite the bilinear material model, using experimental data on similar 3-leaf lime stone walls, there is some room for improvement: material modelling could follow the case study experimenting; the ribs and arches of the roofing system, as well as the columns are made of carved stone, which differs from the 3-leaf walls; and, instead of a continuum FEM modelling, discrete elements method (DEM) could be used to model the structure, attending to both local and global behavior.

There is a certain difficulty in predicting the specific behavior of masonry structures. After the present global evaluation some future developments are expected from the local modelling of particular collapse mechanisms and the application of DEM. Furthermore, as the results suggest, some knowledge on the probable failure modes could also be gained from running a modal pushover, to take into account the various vibration modes simultaneously, and a time-history analysis, that uses actual earthquakes accelerograms. There is, to some extent a general lack of knowledge in these 3-leaf masonry walls behavior, although shear is important and therefore [15] can be followed. Local modelling of some structural elements can also serve the purpose of determining the probable intensities of an earthquake, but careful inspection of the building would also be needed to identify damaged elements, crossing it with historical data, surveys on typical crack patterns and collapse mechanisms evaluation.

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