Advanced Materials Research Vols. 133-134 (2010) pp 355-360
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doi:10.4028/www.scientific.net/AMR.133-134.355

Analytical and Numerical Approach to the Romanian Medieval Church Structures

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Abstract Whenever interventions on medieval churches are necessary, they must be approached as a whole, based on a numerical analysis of the load-bearing structure, especially if exceptional actions, like seismic ones, are present. Numerical analysis results can be successfully used in the assessment of quantitative elements for timber load-bearing structures. Nonetheless, masonry load-bearing subunits are more difficult to design and assess, results of the calculations made in case of elastic behaviour of the masonry structure being valid only until the first cracks. Numerical analysis can indicate, from a qualitative viewpoint, the behaviour of load-bearing structures of church buildings.. This paper presents the modelling principles of various types of load-bearing structural units belonging to medieval churches, which are subjected to dead loads and non-gravity actions, including Lutheran Church in Bistrita, Reformed Church in Aiud, Lutheran Church in Drauseni. , drafted within the Built Heritage Conservation Research and Design Centre – Utilitas, Cluj-Napoca City. It applies to three case studies, displaying their structural behaviour and contributing to the selection of appropriate strengthening methods.

Keywords: Medieval church, numerical analysis, masonry load-bearing subunit

Introduction

Historic load-bearing structures must be verified, if possible, at ultimate limit state, by modelling the structure. These modellings are currently only used to a small extent for planning interventions on historic buildings, even if complex programs that were launched at the end of the 20th century and beginning of 21st century make it possible.

Approach Principles

Masonry load-bearing structure is modelled without taking into account the roof structure, which is considered only as loading. The contribution of the roof structures' stiffness is not taken into account, so the gable deformations are exaggerated. Since the wall-plate connection of the roof structures and of the masonry is achieved by friction, the influence of roof structures on masonry rigidity is reduced, difficult to quantify, and therefore negligible. As a general rule, calculations are made for fundamental combination of actions (persistent, imposed, snow and wind actions) or exceptional combinations (taken into consideration seismic actions).

Beforehand, a schematic approach is recommended, demarcating subunits of the main load-bearing unit with different rigidity levels, i.e. (taking as an example the supporting subunit of the load-bearing structure of the Lutheran Church in Bistriţa) (Fig. 1): obviously the tower is stiffer than the nave at the tower-nave interface; therefore overload of leading load is predictable. On the longitudinal direction, a difference of rigidity between the northern area (containing the tower and sacristy) and the southern area is notable. On the cross-line direction, there are four different rigidity subunits: the most rigid is the western side, including the tower, followed by the eastern subunit, with the triumphal arch and the sacristy. Obviously, in the contact zones of the indicated subunits, overload of leading loads is possible, including appearance of cracks. These conclusions help us interpret the results of modelling, as well as at identifying potential modelling errors.

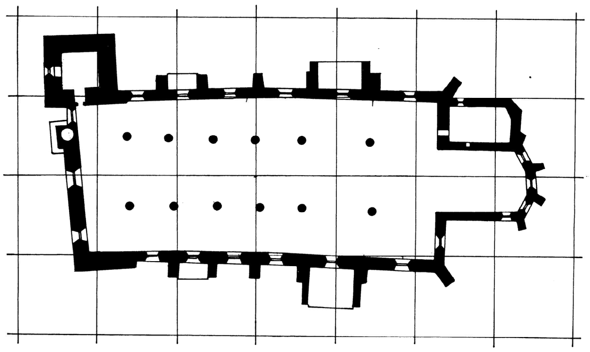

Figure 1 (a): Lutheran Church in Bistrița – Church layout

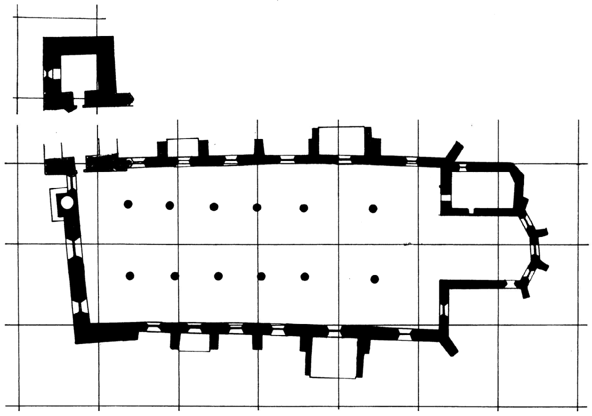

Figure 1 (b): Tower subunit demarcated by the nave and choir subunit

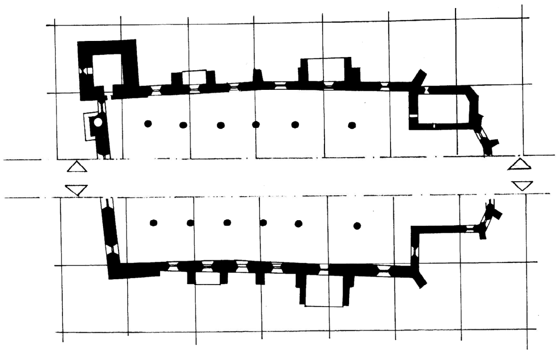

Figure 1 (c): Northern subunit (with the tower and sacristy) demarcated by the southern subunit

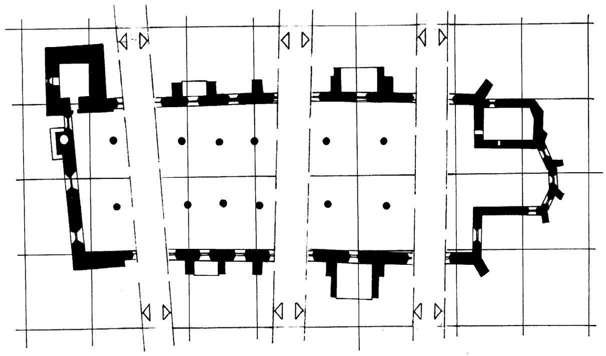

Figure 1 (d): Four different rigidity subunits in cross-line direction

The three-dimensional model (a network of points and lines) is made by graphical design programs: examples are presented in Autocad (Fig. 2).

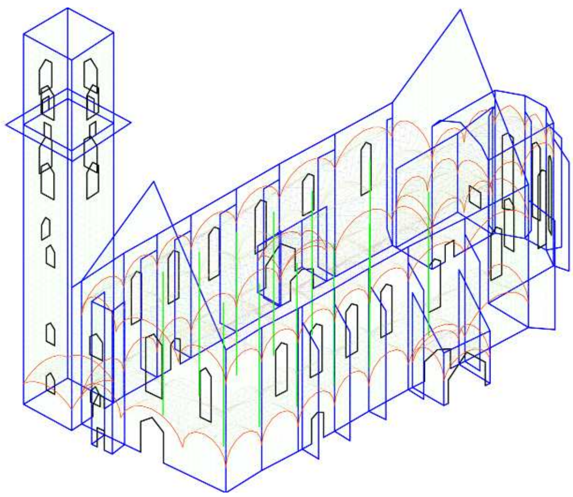

Figure 2: Three-dimensional geometrical model – Lutheran Church in Bistrița

In the geometrical model, the thicker lines represent outlines of walls, vaults and door- and window frames. Thin lines were used to divide the elements into finite elements with corresponding dimensions. Mechanical models contain approximately 9,520-26,200 outward finite elements. Dimensions of finite elements are approximately between 0.50 m x 0.50 m and 1.00 m x 1.00 m for

vertical elements (walls, abutments). Dimension of finite elements is smaller for the vaults, variation depending on the vault curving (Fig. 2).

For a correct interpretation of the results it is necessary to determine the direction of the local system, for every finite element. Thus, for plane surfaces (walls, abutments), the x-direction of the local system is vertical, with orientation from bottom to top; for vaulted surfaces with one curving line, the x-direction of the local system is parallel to the generating line; and for vaulted surfaces with several curving lines, the x-direction of local system points to the vault crown, allowing thus interpretation of bending and axial forces at the vaults.

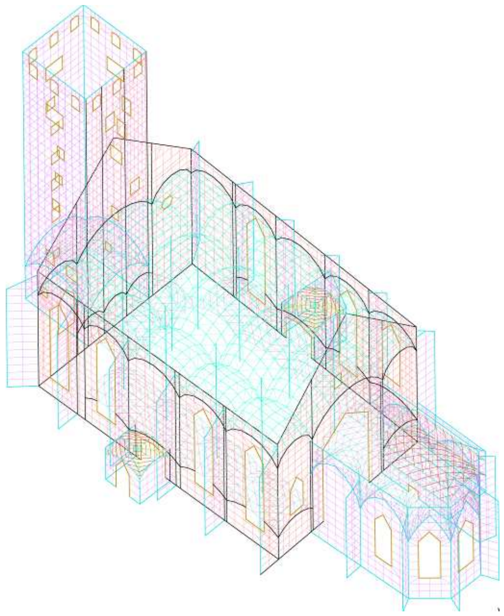

Figure 3: Three-dimensional geometrical model – Calvinist Church in Aiud

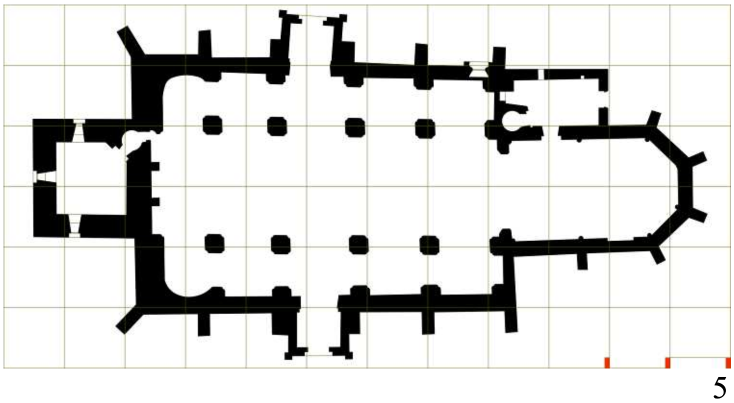

Figure 4: Layout – Calvinist Church in Aiud Romania

Modelling load-bearing structures' behaviour under seismic loads was made through a modal analysis of the first nine vibration modes. From the vibration modes calculated, the application generates the values of equivalent seismic loads and applies them as static loads on the structure. The maximum stress from seismic action results from the values calculated by summing up the results of vibration modes.

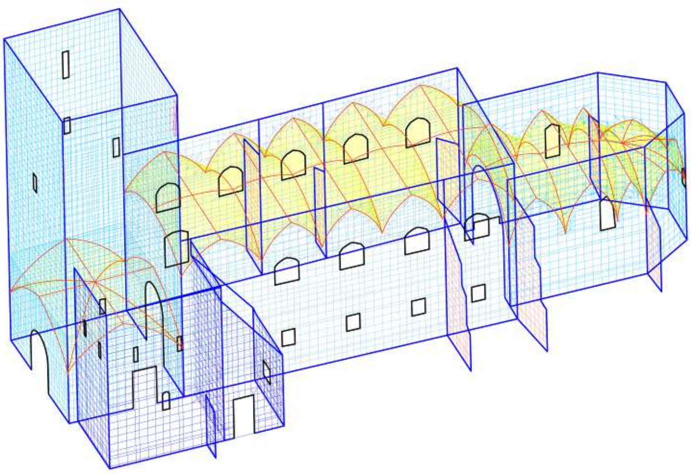

Figure 5: Three-dimensional geometrical model – Lutheran Church in Drăuşeni

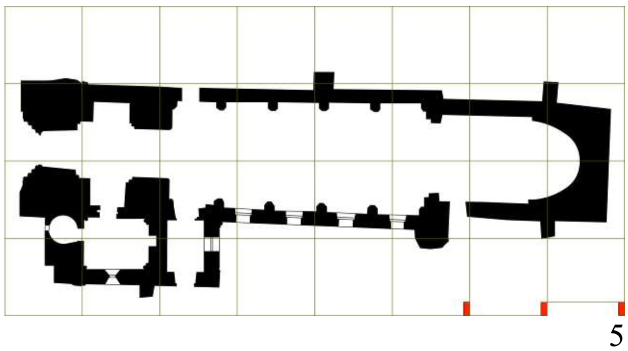

Figure 6: Layout – Lutheran Church in Drăuşeni

Due to the high complexity degree of a load-bearing structure, to potential errors in its survey, to diverse construction materials, to difficulties encountered when modelling a heterogeneous and

anisotropic matter (in the mechanical models presented, modelling was made for homogeneous and isotropic material), the result can be interpreted rather quality-wise than quantity-wise.

Case Studies

Three churches from Transylvania – Romania are presented here: the Lutheran Church in Bistriţa (with tower and sacristy positioned eccentrically), the Calvinist Church in Aiud (with Baroque-style elements: inner columns, intermediary and roof slabs mounted inside the Gothic church) and the Lutheran Church in Drăuşeni (where the early Gothic basilica was transformed in a fortified church by removing the aisles, which had an important reinforcing role for both gravity and non-gravity loads).

Table 1: Case studies general information

No	*Buildings*	*Seismic zone*	*Structural interventions in time*	*Dimension/Span*	*Number and dimension (m)*	*FEM*
1	Lutheran Church in Bistriţa	F ag = 0,08g	Original structure	7.91 m - choir 19.41 m - nave	21700	1.00x 1.00
2	Calvinist Church in Aiud	F ag = 0,08g	Baroque insertion in a Gothic structure	11,34 m - choir 14.51 m - nave	9528	1.00x 1.00
3	Lutheran Church in Drăuşeni	D ag = 0,16g	Partial demolition of the Romanesque basilica	5,63 m - choir 7,40 m - nave	26200	0,50x 0,50

Lutheran Church in Bistriţa

The mechanical model contains approximately 21,700 finite elements on the surface. The dimensions of the finite elements are of approximately 1.00 m x 1.00 m for the vertical elements (walls, abutments), where columns are modelled as lineal elements, with the shape of their cross-section corresponding to the real situation.

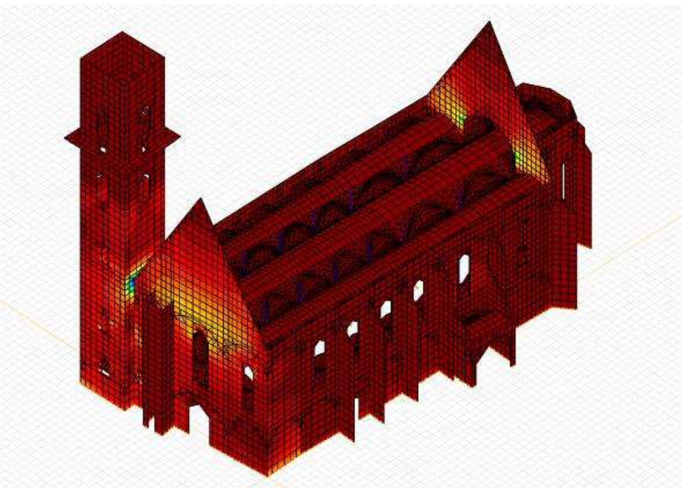

Figure 7.a: Lutheran Church in Bistriţa – Bending diagram under seismic load

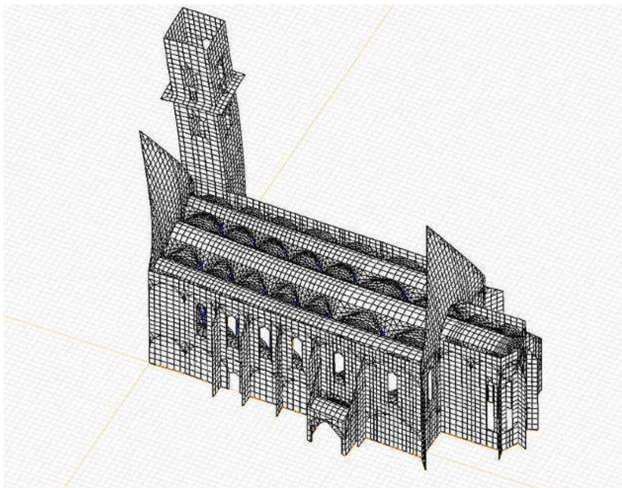

Figure 7.b: Lutheran Church in Bistriţa – Deformation under seismic load

Maximum stress resulting from seismic actions appear next to the fire wall above the triumphal arch, next to the western fire wall (elements acting as cantilever, due to lack of longitudinal bracing frame of the roof structure), at the basis of the tower and at the contact zone between the tower and the nave (due to difference in rigidity between tower and nave). Maximal unit loading exceeds limit values around the tower, and at the central longitudinal axis.

Calvinist Church in Aiud

Two calculation models were set up: the first one for modelling the behaviour of the structure under seismic loads and the second one for determining the stresses appearing in the vaults from gravity loads and from yield of vault abutments on the western gallery, where such phenomena probably occurred, judging from the cracks on the vaults and arches.

Both mechanic models contain 9,528 outward finite elements and columns are modelled as lineal elements, the shape of their cross-section corresponding to the real situation. Dimensions of the finite elements for the vertical elements (walls, abutments) are of approximately 1.00 m x 1.00 m.

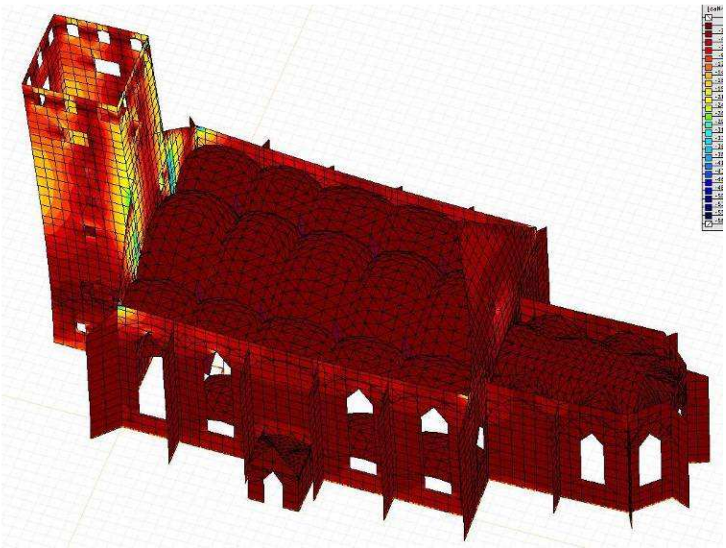

Figure 8.a: Calvinist Church in Aiud – Bending diagram under seismic load

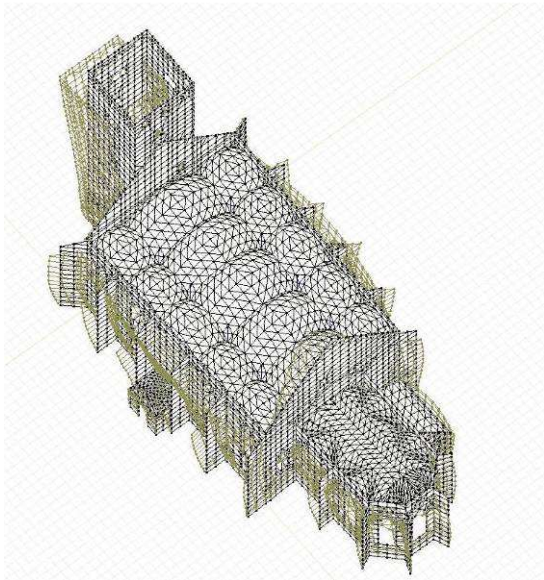

Figure 8.b: Calvinist Church in Aiud – deformation

Maximum stresses resulting from seismic actions appear next to the fire wall between the sacristy and the nave (element acting as cantilever), at the basis of the tower, at the contact zone between the tower and the nave (elements with highly different rigidity levels, due to difference in thickness of tower wall and nave wall) and next to the tower's windows, where there is an area with high moment of flexure, which may account for the cracks visible next to the frames, from tower bottom to top. These phenomena can also be noticed at the displacement diagram, the fire wall edge having a shift of approximately 64 cm on the east-west direction, while the tower a shift of approximately 33 cm.

The interpretation of the results at the fundamental combination (the mechanical model for determining loads in the vaults) shows an outward displacement of the southern and northern nave walls, as well as an inclination in the same direction of the rows of columns which divide the aisles and the nave. The displacement of the columns on the crosswise direction, towards the exterior, reaches 26 mm at the end of the columns, whereas the displacement of the southern and northern walls is approximately 9 mm (the vault subunit over the nave does not offer a representation that makes the result interpreting easier). These phenomena provide explanations for the cracks existing on the vaults of the median vault segment, on the east-west direction. The southern and northern walls, being considerably high, cannot deal appropriately with vault thrust, the vault springing is shifting towards the exterior, therefore cracks are open in the middle of the vaulted slab above the nave.

At the columns on the western gallery, a bearing-yield type of shift of 1.5 cm downwards is recorded. It is also noted that the moment of flexure and the cross force on the vault surface increase considerably upon propping on these columns, which might explain the appearance of concentric cracks on the vault intrados next to the columns. Geotechnical studies show that the foundation of these columns does not descend at requisite depth, so one can assume that settling occurred at the foundation bases, and that most probably the foundations shifted downwards and, at the same time, also rotated at their base, but neither their size nor their direction can be estimated accurately.

Lutheran Church in Drăuşeni

The mechanical model contains approximately 26,200 finite elements on the surface. The dimension of finite elements is approximately 0.50 m x 0.50 m for vertical elements (walls, abutments).

Figure 9.a shows the bending on the main x-direction. The maximum flexure from seismic loads appears at the connection between the tower and the nave (elements with highly different rigidity, due to the difference in thickness between tower wall and nave wall).

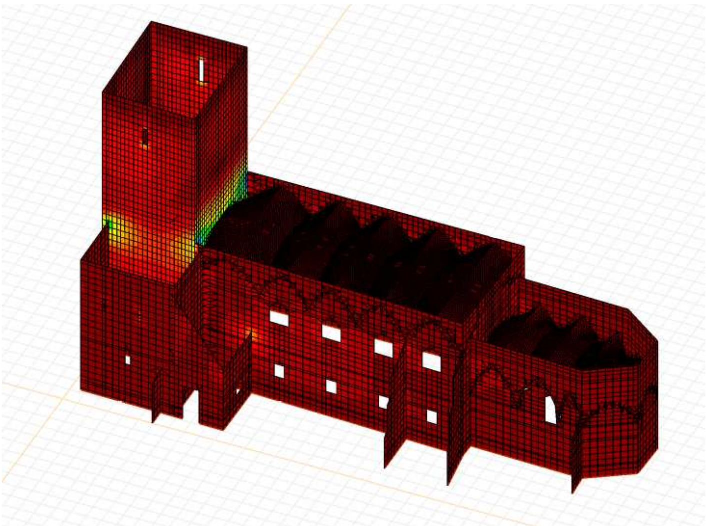

Figure 9 (a): Lutheran Church in Drăuşeni – Bending diagram under seismic load

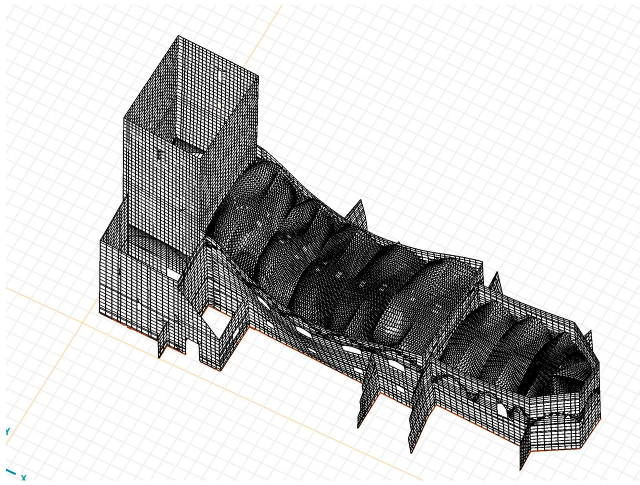

Figure 9 (b): Lutheran Church in Drăuşeni – Deformation in mode #3

Moments of flexure resulting from seismic loads generated from vibration mode #3 indicate a vulnerability of the structure to horizontal forces on the south-north direction. This vulnerability is due to the fact that median areas of the nave's southern and northern walls lack abutments (see Figure 9.b).

Maximum loading in this case is 6.72 daN/cm^2, so below the value of 10daN/cm^2, which stands for the load-bearing capacity of a medium-quality masonry. The value of Young's modulus $E=0.5\alpha R_c k=0.5 \times 1666 \times 20 \times 2 = 33320$ daN/cm^2 used for calculations is close to the values indicated by specialized studies published in the UK (Lourenço, Ramos, Vasconcelos and Peña 2008, Stylianidis and Sextos 2008).

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[2] Stylianidis, K A, and Sextos, A (2008). "Estimation of the seismic history of the city of Thessaloniki through back analysis of its Byzantine land walls," in *Structural Analysis of Historic Construction – Preserving Safety and Significance*, D D'Ayala and E. Fodde, Ed. London, UK: Taylor&Francis Group, 611-619.