

Seismic Behaviour of Romanian Orthodox Churches, Modeling of Failure Modes by Rigid Blocks

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Abstract—Historic religious buildings located in seismic areas have developed different failure mechanisms. Simulation of failure modes is done with computer programs through a nonlinear dynamic analysis or simplified using the method of failure blocks. Currently there are simulation methodologies of failure modes based on the failure rigid blocks method only for Roman Catholic churches type. Due to differences of shape in plan, elevation and construction systems between Orthodox churches and Catholic churches, for the first time there were initiated researches in the development of this simulation methodology for Orthodox churches. In this article are presented the first results from the researches. The theoretical results were compared with real failure modes recorded at an Orthodox church from Banat region, severely damaged by earthquakes in 1991. Simulated seismic response, using a computer program based on finite element method was confirmed by cracks after earthquakes. The consolidation of the church was made according to these theoretical results, realizing a rigid floor connecting all the failure blocks.

Keywords—Dynamic analysis, failure mechanism, rigid blocks seismic simulation.

I. INTRODUCTION

THE seismic behavior of historic buildings in this period in majority of cases is studied with a computer software. They simulate very good the seismic response of the buildings in elastic lineal domain and in nonlinear domain. The numeric simulation of the masonry does not respect any hypothesis (isotropy, elastic behavior, homogeneity) assumed for other materials. In these conditions, elastic models considering a homogenized continuum, can give an indication on the mechanical behavior in the undamaged range and can only be used to detect the weak parts of the structure and the positions of to come cracks. For ultimate state, nonlinear models, using complex finite elements, based on plasticity theory and considering the joint and interface elements to model the planes of weaknesses, can be used only for simple masonry elements, being inadequate to model a full structure.

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Therefore, the development of a simple model, able to determine the ultimate state for complex masonry structures, is a very expected methodology by the designers. A rapid and efficient method for simulation of the response of buildings with complex shapes in ultimate limit state was developed after examining the cracks after the earthquake. The conclusion that often failures occur by formation of collapse mechanisms involving all the buildings or only some part of them. In this case computational models use some rigid body macro elements and the discontinuities are concentrated only along the borders of these elements.

II. CALCULATION MODELS OF MASONRY BUILDINGS FOR SEISMIC DESIGN

A current trend in seismic design is the incorporation of *performance-based design methodology*. In this methodology, every building is designed to have the desired levels of seismic performances corresponding to different specific earthquake ground motion. To achieve this goal, elastic analysis is insufficient, because this cannot realistically predict the forces and deformations during earthquakes. Inelastic analytical procedures become necessary to identify the mode of failure.

Inelastic time-history analysis is the most realistic approach for evaluating the building performances. However, this inelastic analysis is too complex and time-consuming in the design of most buildings, especially if the spatial behavior is considered. As a compromise, a simplified procedure commonly accepted is the pushover analysis, where a sequence of inelastic static analysis is performed for a set of monotonically increasing lateral loads. For the historical masonry buildings, the pushover methodology is complicated by the definition of mechanical properties of the materials, definition of constitutive laws for decayed materials and structure rigidity degradation due to the cracks formation. The behavior of a masonry building is presented in Fig. 1. In the first stage, the building works as a compact element until the first fissures. In this field, the building's masonry can be characterized as an elastic medium with heterogeneous properties. The first fissures produce a reducing of building's rigidity, but the elastic behavior is not modified very much. At superior level of load, the fissures are turned in a system of cracks, which began to affect very much the building behavior. The increasing of load produces a local failure, where the first very important damage of building occurs as seen in Fig. 1. Behavior of masonry arch for lateral load in the ultimate limit, a collapse mechanism is formed, which, finally, generates the building failure. In the frame of

performance-based design philosophy, until the formation of crack system, the building works without important damage and the safe occupancy and operational usage can be considered. In this field the damage control is the main task of design. The field until the local damage is the precursory phase of the structure failure, while the formation of a global collapse mechanism represents the ultimate limit state. In many cases, between local failure and the limit states considered in EUROCODE 8, the masonry structure behavior until the formation of crack system can be considered as the range of *damage limitation stage*, while the behavior near to formation of collapse mechanism as the *ultimate limit state*.

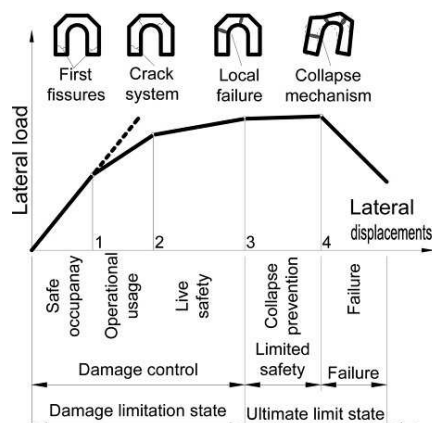


Fig. 1 Behaviour of a masonry triumphal arch

It is well known that masonry structure analysis requires nonlinear modeling accounting for the low tensile capacity and the consequent cracking phenomena. It is well known though that such models cannot be used in the case of very complex structural systems characterized by large number of degree of freedom. In the same time, it is very clearly that the methodologies required for the two limit states differ very much. While for damage limitation state one can use the elastic analysis (with or without considering some fissuration effects), for the ultimate limit state, the methodology must be very different. Therefore, analyzing the behavior of historical buildings, it is need to be adopted a two step procedure as shown in Fig. 1.

i. Global behavior analyses for damage limit state, in the linear elastic range, through a complete and refined FE Method- 3D model. This analysis can give indications on the global behavior in the undamaged range and can only detect the weak part of the structure and the position of to come cracks. In the same time, it can be use to have indication about the efficiency of some strengthening methods.

ii. Global behavior analyses in the ultimate limit state, using the Collapse Mechanisms Method, considering the structure composed by some rigid body macro-elements with discontinuities concentrates only along the borders of these elements, resulted due to seismic action. This methodology is based on the observation in situ or on the models concerning the cracks system of a damaged building, leading to a collapse mechanism. This analysis is very useful to establishing the most efficiently strengthening method.

A. Modeling masonry building by rigid blocks

The use of theory of rigid blocks to determine the limit of historical building has the potential to become a powerful tool in engineering practice. In particular, this approach avoids the use of sophisticated and time-consuming nonlinear finite element technique. The applicability of this theory to masonry structures modeled as assemblage of rigid blocks interacting through joints depends on some basic hypothesis, confirmed by in-site observations and experimental results :

- i. limit loads occur at small displacements, so the linear theory can be used.
- ii. masonry has no tensile strength.
- iii. compression and shear failure at the joints are perfectly plastic.
- iv. hinging failure at joint does not consider the effects of local crushing.

The seismic collapse load corresponding to the ultimate load is determined using a cinematic method. In this method the following steps must be considered:

- i. establishment of the horizontal and vertical loads applied to the structure.
- ii. establishment of possible collapse mechanisms for the structural system.
- iii. determining for each mechanism element the vertical forces and the position of these forces.
- iv. imposing the collapse mechanism of horizontal virtual displacements;
- v. determining the compatible virtual displacement for each element of the mechanism.
- vi. using the principle of the minimum of total potential energy (composed by internal and external parts), the amplification of factors for horizontal forces, corresponding to the all established collapse mechanisms, are determined.
- vii. The collapse mechanism for the ultimate limit state is the minimum value of the determined amplification factors. This methodology was successfully applied for determining the collapse mechanisms and ultimate limit state forces for buildings [1], [3] [4] and Romanesque churches presented in Fig. 2 [5], [6], [7]. and for Orthodox Churches was developed also in Romania [2], [10] –[13].

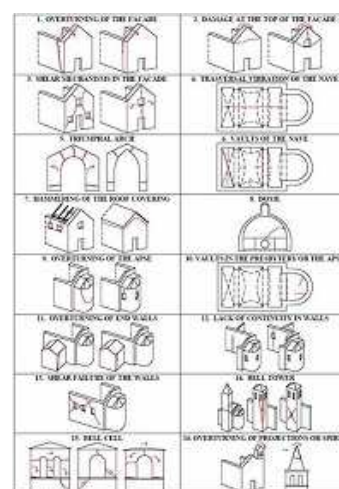


Fig. 2 Romanesque churches. Masonry rigid block [5]

III. SEISMIC BEHAVIOUR OF ROMANIAN ORTHODOX CHURCHES

The buildings of Romanian Orthodox churches are based on the Byzantine style, being characterized by the using pendentives and dome on pendentives as in Fig. 3. It must be pointed that the main structural characteristic of Byzantine architecture is the use of pendentive domes, dome on pendentives, and tower on pendentives. This is unique way of adjusting the circular form of a dome or tower to a square plan. The pendentive dome is derived by trimmer the sides of a circular dome over a square plan as in Fig. 4. The pendentives dome enables to transfer the total load of the roof to the four corners of the building.

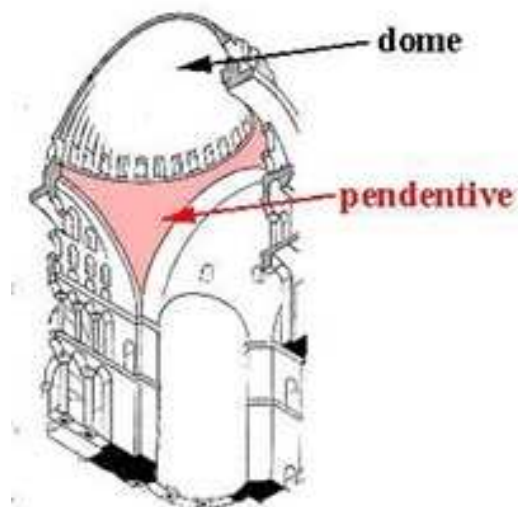


Fig. 3 Pendentive in Byzantine churches

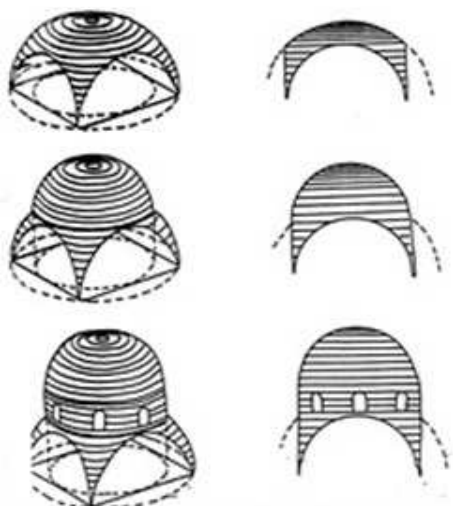


Fig. 4 Pendentives and dome and tower on pendentives

Additionally, the top dome enables to transfer the total load of the roof to the four corners of the building. The top of the pendentive can be trim to introduce another dome on top. The additional dome can further be raised to introduce a cylindrical tower between the pendentive dome and the additional dome.

Windows can then be introduced in the cylindrical tower enabling architects to create interior light effects. The surrounding infilled and exterior masonry walls also contribute to carry out the loads, forming very rigid corner pendentives as in Fig. 5.

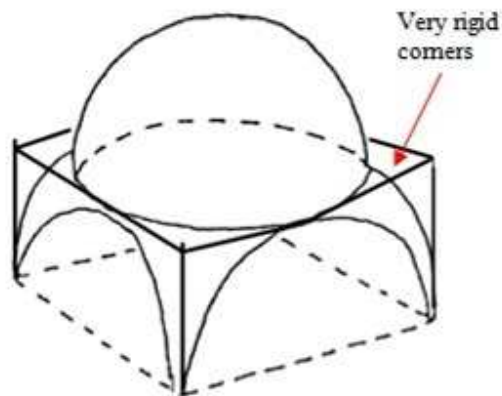


Fig. 5 Rigid corner pendentives, including the surrounding walls

The typical plans of Orthodox churches are presented in Fig. 6: rectangular nave with one lob and three-lobed nave. Unlike the Catholic churches, the Romanian Orthodox churches are relative small in size. The main typical Romanian orthodox churches are the three-lobed plan. This form plays a crucial role in the improving the church behaviour during the earthquakes, because it reduces the distance between stiffness centre and centre of gravity on the longitudinal axis of symmetry [8]. In some cases, some churches were provided with buttresses in order to reduce the distance between these two centers.

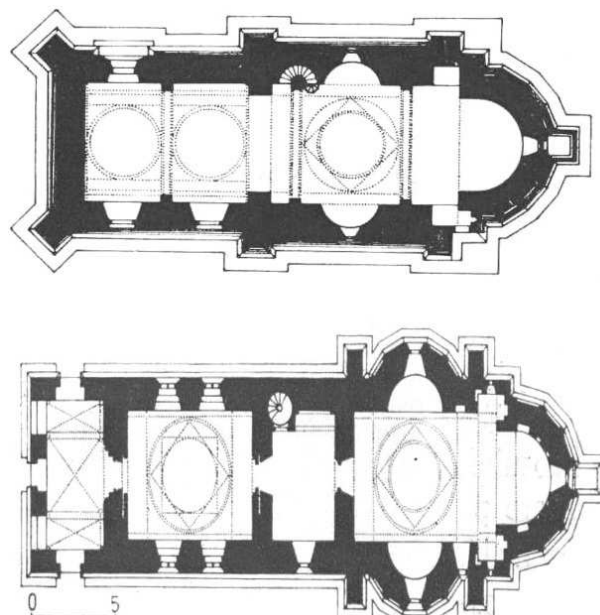


Fig. 6 Typical rectangular and tree-lobed plans [8]

Very many damaged churches were recorded during 1977 and 1986 Vrancea earthquakes and 1991 Banat earthquakes. Among the hundreds of damaged churches during these

earthquakes, Fig. 7 presents the Borzesti (rectangular plan) and Cozia (three-lobed plan) churches [2].

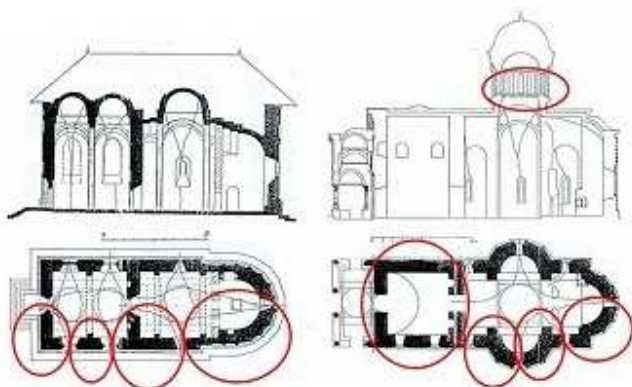


Fig. 7 Damage of Borzesti and Cozia churches due to 1977 Vrancea Earthquake [2]

Analyzing the occurred system of fissures and cracks it is very clear that the spatial collapse mechanism is formed by a longitudinal fracture and multiple transversal fractures which round the pendentives, due to the great rigidity of these ones.

In addition, cracks occurred at the base of tower. The cracks start always from the windows, due to reduced rigidity in these zones. Considering the system of fractures, it is very clear that, in the ultimate limit state, the churches form a block system, working independently each other's as in Fig. 8.

The blocks are formed by the wall delimited by two indows and the corresponding corner pendentives. Due to the seismic actions, the blocks rotate around a basis axis. This rotation is equilibrated by the gravity loads, mainly due to the masonry weight.

The rasion between overturning and stabilizing forces gives the possibility to determine the ultimate limit loads, this one being the minimum of all the values [12], determined for each block [5].

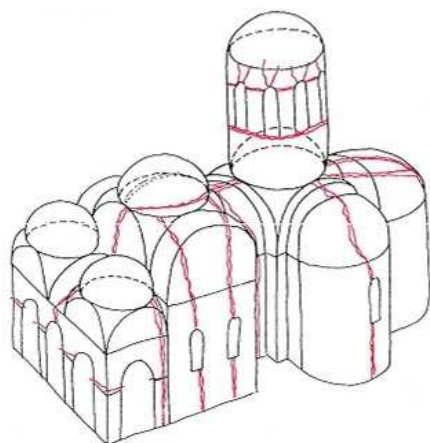


Fig. 8 Blocks system in ultimate limit state [2]

In the Banat region, the earthquake of 1991 has produced significant damages to some churches made of brick. Among these damaged churches is the one of St. George Monastery in the village Manastirea, Timis County. It is attested in the sixteenth century and was originally built as a Byzantine

church. The present form dates from the years 1795-1796 when the church was transformed into a baroque one. Currently it is declared a historical monument.

Hall-type church building is covered with brick arches and wood framing. The church has two towers: the West Tower and East Tower in Fig. 9 and Fig. 10.



Fig. 9 The two towers of the church

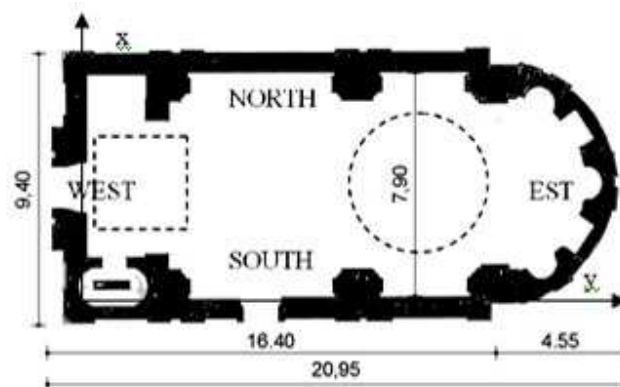


Fig. 10 Horizontal section

Unveiling of foundations in the altar area and in North facade revealed that there are no cracks in the foundation. Because the distances to the two poles are 15 km and 10 km as in Fig.11, this earthquake fits the category of "epicentral earthquake" which is characterized by:

- i. Very short periods of vibration (below 0.2 to 0.3 sec) in which case the massive masonry structures are most affected.
- ii. Pulse action, the first cycle is the most powerful, next alleviating considerably.
- iii. Components perpendicular to the fault rupture are the most important and have vertical components are the same size as the horizontal ones.

According to the seismic Romanian norm P100/1-2006, the maximum acceleration (peak ground acceleration) of the land is 0.16g.



Fig. 11 The distances to the earthquake poles

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In these circumstances, taking into account the position of the church to the two poles, the church's main action direction is longitudinal. Given the longitudinal asymmetry of the church, the west is much stiffer than the east. The center of rigidity of the structure is moved to the west. At the same time, taking into account the different stiffness of the stair area on the opposite side, the center of rigidity is moved in north. Rotation of the church from the center of rigidity is shown in Fig. 12. It can be noticed that the round wall next to altar is the most stressed, being the farthest from the center of rigidity.

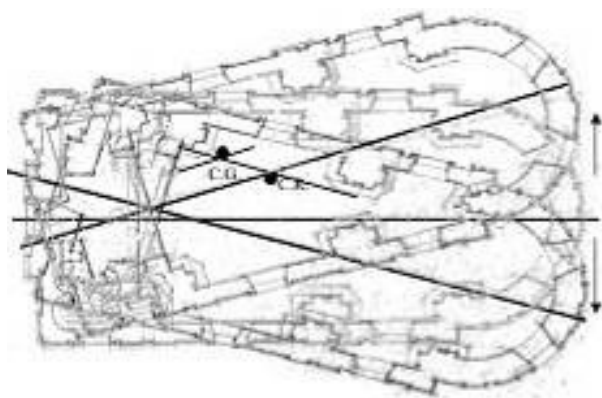


Fig. 12 Rotation of the church

Under these conditions, considering the presence of openings, masonry has a tendency to separate into blocks by splitting of arches above the openings thus forming nine blocks. The most stressed is block 4 of the eastern wall where are the largest lateral deformations in Fig. 13.

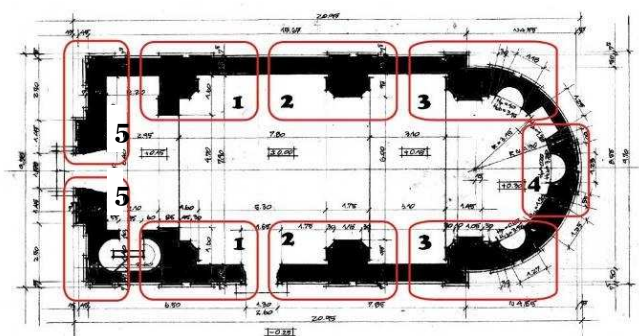


Fig. 13 Mechanism blocks

Brick walls are 75 cm thick, reinforced with brick pillars in front of the roof arches. Masonry structure has a longitudinal symmetry, except the West side where the presence of the stair has introduced an asymmetry characterized by walls with a thickness of 40-45 cm. The window openings on the altar apse and reduced thickness of the wall under the windows have caused cracks in Fig. 14.



Fig. 14 Cracks at the eastern wall

The cover in the central area is made of two domes on brick pendants resting on arches. One of the vaults supports a tower with circular section. Rectangular bell tower section is being supported on one side of the church walls, and on the opposite side of a brick arch that has no tie to the beginning and shows cracks. Under the bell tower windows, X cracks were found on the South and North facades, damages characteristic of seismic actions in Fig. 15.



Fig. 15 Cracks at bell tower

The East Tower is circular and is supported on the middle dome pendants. The tower has the upper closure in a vault made of bricks and has no significant cracks.

Examining the sizes of the cracks after the earthquake in 1990 the followings are found:

- i. The biggest cracks are developed in the eastern wall next to the altar, in full compliance with the structural issues outlined above in Fig.8.
- ii. Next, most important, are the cracks from the southern wall, the northern being much less cracked;
- iii. Asymmetrical turning tendency can be seen on the walls of the western tower, south wall is much more cracked than the northern, while at the western and eastern walls, cracks were insignificant;

Eastern Tower, not being bounded by land and moving freely with the dome, has minor cracks. Inside the church there are cracks at the top of the vaults because arches don't have ties for taking vertical loads in Fig. 15.



Fig. 16 Cracks at arches

IV. MODELING MASONRY BUILDING BY RIGID BLOCKS ROMANIAN ORTHODOX CHURCHES SF. GEORGE BIRDA

Based on the modeling of seismic behavior of Church of St. George Birda, Timis County, Romania, based on the theory of transfer mechanisms, the following assumptions were made:

- i. The church was divided into rigid blocks based on observations recorded after earthquakes cracks in the Banat area since 1991. Transfer blocks are shown in Fig. 17, based on 3D drawing. Global system of axes is shown in Fig. 10.

- ii. Because the greatest damages were recorded in the nave and apse altar, a special attention was given to these areas. They were divided into blocks 1, 2, 3 and 4.

iii. There were not made calculations to determine the peak ground acceleration for the tower named block 5 because the vertical stabilization forces are bigger in this area. The tower has developed cracks in the plan of the masonry in X shape and after earthquakes did not recorded any displacements from verticality as shown in Fig.15. So there is no way to record values of the peak ground acceleration smaller than the peak ground acceleration of blocks 3, 4, 5.

iv. In its calculations they neglected the effect of roof weight because they have very low values.

v. In neither the calculations there was not given any weight to the circular tower of the nave area because it was not participating in the failure mechanism. At failure blocks 2 and 3 was taken in calculation the weight of the arches, the walls and the pendants.

vi. The church has no metal tie to prevent the overthrow of the outside walls.



Fig. 17 St. George church, blocks system in ultimate limit state

vii. Separately was calculated the weight, the position of the center of gravity for each wall, arch, vault from a failure block. Determined position of the center of gravity and of mass for the entire failure block on a 3D model drawing.

viii. The overturning seismic force outside the wall was applied in the center of gravity of the failure block.

ix. The equilibrium relation between overturning moment and stabilisation moment was written for each failure block in relation to point A from the wall as shown in Fig. 18. It is located at the outer end wall. According to [6] the proportion ratio between the overturning moment and the stabilisation moment of the transfer block is called seismic coefficient and is noted by λ . It has subunit value and represents the maximum value of ground acceleration for which the building collapse that occurs outside of the plan of the failure block. Maximum seismic acceleration of ground in the Monastery of St. George is $0.16g$, so $\lambda_{\max \text{ field}} = 0.16g/g = 0.16$. λ values calculated by theoretical modeling for each failure block were compared with this value of $\lambda_{\max \text{ field}}$. Blocks with a value of λ

$< \lambda_{\max \text{ field}}$ are most vulnerable because they will fail and will cause the collapse of the structure. For St. George Monastery was calculated λ value for blocks 2, 3 and 4.

Because the most damaged transfer block after 1991 earthquakes is block 4, in Figure 18 are presented the dimensions and position of the center of gravity and the point of application of seismic force. There are only two centers of gravity for masonry and bolt and one for the entire block 2. The Name of symbols used in Figure 18 are shown in Table I.

TABLE I
 SYMOLS AND UNITS

Symbol	Name	Masurment unit
G	element weight	kN
S	the total seismic force	kN
x, y, z	distances	M
M_o	overturning moment	kNm
M_s	stabilization moment	kNm
γ	specific weight of masonry	20 kN/m ³
V	total masonry volume	m ³
λ	seismic coefficient	
CG	center of gravity of the transfer block	

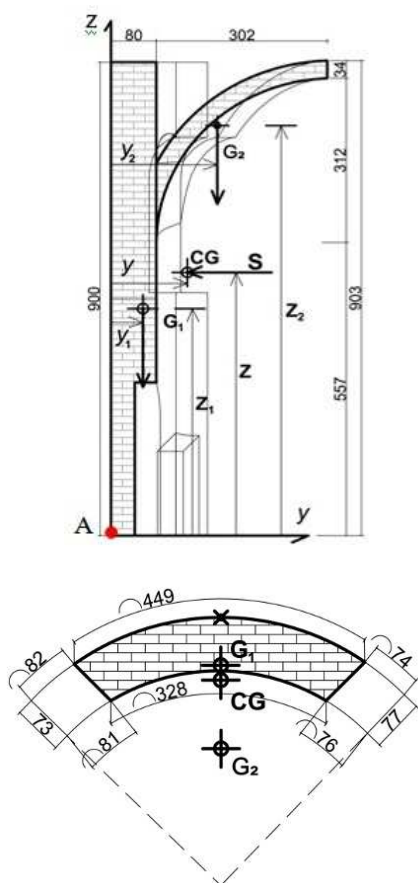


Fig. 18 Dimensions in plan and positions of centers of gravity for block 4

For failure block 4, who was the most damaged after earthquake, λ is determined with (1) [9]. Calculations are presented in Table II. The value of λ is calculated in (2).

$$\lambda = \frac{M_s}{M_o} = \frac{\gamma_1 \times V_1 \times y_1 + \gamma_2 \times V_2 \times y_2}{\gamma \times V \times z} \quad (1)$$

TABLE II
 SEISMIC COEFFICIENT CALCULATION FOR BLOCK NUMBER 4

No. elemen t	γ kN/m ³	V m ³	G kN	x m	$M_s=xG$ kNm	z m	$M_o=zG$ kNm
1	20	32.5	650	0.7	455		
2	20	3.47	69.4	1.87	129.8		
TOTAL		35.97	719.4		584.8	4.84	3482

$$\lambda_4 = 0,168 \quad (2)$$

In Fig. 19 and Fig. 20 are shown the dimensions in plan, the positions of the center of gravity and the point of application of seismic force for failure blocks 2 and 3 in 3D coordinates. There are five centers of gravity for each failure block caused by wall, domes and arches. Values of stabilization and overturning moments are presented in Tables III and Table IV. The values are presented in (3) and (4).

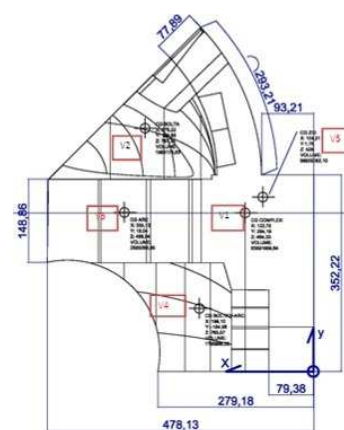


Fig. 19 Dimensions in plan and positions of centers of gravity for block 2

TABLE III
 SEISMIC COEFFICIENT CALCULATION FOR BLOCK NUMBER 2

No. elemen t	γ kN/m ³	V m ³	G kN	x m	$M_s=xG$ kNm	z m	$M_o=zG$ kNm
1	20	63,60	1272	1.22	1552		
2	20	1,55	31	2.75	8.25		
3	20	2,68	107	3.35	358.5		
4	20	1,80	36	1.98	71.2		
5	20	56.62	1132	1.04	1177		
TOTAL		126.25	2578		3167	4.65	11988

$$\lambda_2 = \frac{3167 \text{ kNm}}{11988 \text{ kNm}} = 0,264 \quad (3)$$

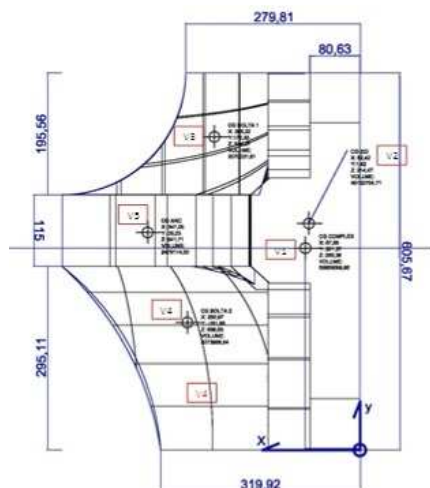


Fig. 20 Dimensions in plan and positions of centers of gravity for block 3

TABLE IV
 SEISMIC COEFFICIENT CALCULATION FOR BLOCK NUMBER 3

No. element	γ kN/m^3	V m^3	G kN	x m	$M_s = xG$ kNm	z m	$M_0 = zG$ kNm
1	20	57	1140	0.87	992		
2	20	49	980	0.62	608		
3	20	2	40	2.65	106		
4	20	3,3	66	2.60	172		
5	20	2.5	50	3.47	174		
TOTAL		113.8	2276		2052	2.66	6054

$$\lambda_3 = \frac{2052 \text{ kNm}}{6054 \text{ kNm}} = 0,338 \quad (4)$$

Comparing relations (2), (3), (4) it is observed that:

- i. $\lambda_4 < \lambda_2 < \lambda_3$ although the theoretical modeling, predicts that the limit state will be achieved by the collapse of block 4, that will yield before blocks 2 and 3 for the same value of the seismic force. This damage is confirmed by the biggest masonry rupture of the apse area.
- ii. These results provided by theoretical modeling are confirmed with the damage and collapse mechanisms developed by the resistance structure of the Church of St. George after the 1991 earthquakes.

V. CONCLUSION

Using structural modeling of brick with finite elements in the elastic lineal domain, determines the main modes of vibration, which detected the weak areas of the structure.

Maximum concentration of efforts identified using computer programs based on finite element method (FEM) occur in the areas, where the structure has developed cracks and was separated into several blocks of brick.

Modeling of failures of buildings through blocks is a method which estimates with good accuracy the maximum ground acceleration at witch structure collapse occurred. In the same time it simulates vet well the seismic response of

building in the ultimate limit state. With this information, you can easily determine building solutions to be taken before an earthquake emergency only in the most vulnerable areas of the building to avoid collapse of the historic building. The theory of mechanisms of failure developed a simple and fast calculation method that has been verified by numerical analysis for the Catholic Church of Roman type. By applying the same principles of modeling and calculation, the theory of yielding mechanisms was verified too for Baroque Orthodox Church of St. George Orthodox, Birda, Banat Region, Romania. The study showed a good concordance between theory Mechanisms of failure and the damaged structure after the Earthquakes of 1991.

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REFERENCES

- [1] Beolcini, G.C. & Milano, L, Antonacci, E. 2006. Definizioni di medelli per l'analisi strutturale degli edifice in muratura. *Protocollo di progettazione per gli interventi di ricostruzioni post-sisma* Vol. II, Parte 1,1-88.
- [2] Crisan, M. 2003. *Structural Restoration of Orthodox Churches* (in Romanian). University "Ion Mincu" Publishing House, Bucharest.
- [3] D'Ayala, D. 2000. Establishing correlation between vulnerability and damage survey for churches. *12th WCEE, Auckland*.
- [4] D'Ayala, D. & Speranta, E. 2004. Definition of collapse mechanism and seismic vulnerability of historic masonry buildings. *Earthquake Spectra*.
- [5] Lagomarsino, S. 2002. Vulnerability of churches. *12th ECEE*, London.
- [6] Binda, L., Saisi, A. 2007. State of the art of research on historic structures in Italy. http://www.arzip.cz/w11/w111_binda.pdf.
- [7] De Luca, A., Mele, E., Romano, A., Paterno, C., Giordano, A. 2004. Valutazione approssimata della capacita portante di elementi murari tipici di chiese a pianta basilicale. *XI Congresso Nazionale L'ingegneria Sismica in Italia. Genova*.
- [8] Sofronie, R. 1983, Post-Seismic Strenghtening of Churches. *IABSE Symposium Venezia*, pp287-294.
- [9] Sorrentino L., Decanini L., Comportamento sismico delle costruzione murarie storiche, 1998-1999, pp 109-124.
- [10] Gioncu V., Mosoarca M., Assessment and mitigation procedures for historical buildings situated in seismic areas, *Recent Advances in Risk Management, Assessment and Mitigation*, RIMA'10, Bucuresti, April 20-22 2010, WSEAS Press, pp.27-33.
- [11] Gioncu V., Mosoarca M., *Ultimate Limit State of Historical Masonry Buildings*, Prohitech WP 9 Datasheet, 2007.
- [12] Gioncu V., Mosoarca M., Anastasiadis A., *Dynamic Modeling of Historical Buildings*, New Trends and Statics and Dynamics of buildings, Bratislava, 20-21 October 2005.
- [13] Gioncu V., Mosoarca M., *Protection of historical buildings*, Proceedings of the International Conference of Protection of Historical Buildings, PROHITECH 09, Rome, Italy, 21-24 June 2009, pp.1153-1158.