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SEISMIC VULNERABILITY ASSESSMENT OF A ROMANIAN HISTORICAL MASONRY BUILDING UNDER NEAR-SOURCE EARTHQUAKE

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Abstract. *In the present research work, the effect of ground motion vertical component in case of near-source excitations on masonry buildings has been analysed. To this purpose, an investigation has been made on the Banloc Castle, a historical masonry construction damaged by the Banat-Voiteg earthquake occurred on December 1991 in the Romanian Region of Banloc. A FEM model of the building, setup using DIANA FEA analysis software, has been analysed in the non-linear dynamic field. In particular, the records have been referred to the Banloc site, characterized by a moment magnitude of 5.5 and a focal depth of 9 km. In order to estimate the influence of the vertical seismic motion on the case study building in terms of both displacement and internal forces, two different scenarios have been examined. The first scenario has taken into account the horizontal component only, whereas the second one has studied the simultaneous effects of three components of the seismic action. The comparison between the two scenarios has shown that the vertical ground motion significantly modifies the structural behaviour of the inspected building. Finally, numerical damage patterns have been plotted and compared to the real cracks detected in the case study after the considered seismic event.*

1 INTRODUCTION

Typically, the effect of the vertical component of earthquake ground motion on constructions has been ignored in the current studies of Earthquake Engineering. Gioncu and Mazzonlani have developed an important research activity concerning the influence of ground motion vertical component for the load-bearing masonry structures in the Banat seismic region [1, 2, 3]. In particular, the seismic design and assessment of Unreinforced Masonry (URM) structures is limited to the effects of the horizontal accelerations, neglecting the vertical component of the design ground motion [4, 5, 6]. The damages caused by disastrous earthquakes highlighted the brittle behaviour of several structural systems especially under seismic actions characterised by not negligible vertical components, which produced significant effects in cases of near-field events.

Generally, the ground motion vertical component is mainly associated to the arrival of vertically propagating compressive P-waves and secondary shear S-waves. However, the propagation direction of volume waves (P and S) is not influenced by the site geology, but by the site distance, the fault type and the magnitude. In these circumstances, it is evident that the effects induced by the ground motion vertical component are prominent in areas characterized by high hazard level and that they should be appropriately considered in the seismic behaviour analysis of existing masonry buildings, which have very low tensile resistance [7].

Generally, earthquakes occurring near an active seismogenic source contain pulse-like records (one or more consecutive peaks), characterised by a long period and a short duration [8]. These impulses, caused by the sliding of faults, release a large amount of seismic energy, which is transmitted abruptly to the structures. For this reason, the vertical component of these earthquakes is one of the determining factors to be taken into consideration for the correct safeguarding of buildings and design preventions.

In fact, as reported in [9, 10], it was shown that the ratio between vertical (V) and horizontal (H) components of ground motion in the near-source regions with high hazard earthquakes is considerably different than that expected at a smaller magnitude and greater distances; moreover, it was suggested that the V/H (empirical) limit of 2/3 should be re-evaluated in order to better foresee the effects of ground motion vertical component for design purposes. In particular, it was observed that these ratios near the seismogenic source can exceed the unitary value. The effects produced by the vertical component of the ground motion significantly influence the dynamic response of structures. In fact, as detailed in [11], the vertical component on masonry buildings provokes a reduction of load-bearing capacity of the wall panels and, consequently, an increase of lateral displacements. In particular, during the time history cycle, the vertical component acts alternately with compression and decompression peaks, that lead to a degradation of the stiffness of the masonry under dynamic action with the consequent increase of the local or global vulnerability.

Other studies [12, 13] inspected the behaviour of masonry buildings subjected to vertical seismic actions, asserting that the vertical component of the ground motion in epicentre areas characterized by a *Joyner and Boore* distance (R_{jb} <30 km) causes a variation of the axial stress regime in the walls. Consequentially, the dynamic amplification due to soil stratigraphy leads to a reduction in terms of the shear capacity of structural resistant elements, which show a deficit in terms of resistance, stability and ductility.

In this framework, focusing on the Banat-Voiteg seismic event occurred on December 1991, the Castle of Banloc has been selected as a case study. In order to take into account the influence of vertical accelerations on the seismic capacity of the examined building, non-linear dynamic analyses have been carried out considering the ground motion records representative of the earthquake occurred in the region of Banat in Romania.

The displacements and foces regime of the investigated castle have been analysed in order to point out the influence of vertical component. Finally, the damages achieved from numerical modelling have been presented, discussed and compared to those really detected under the 1991 earthquake.

2 SEISMICITY OF BANAT REGION

The Western and South-Western part of Romania, well known as the Banat Region, is undoubtedly one of the most seismically active regions of the country, characterized by crustal earthquakes. Due to the presence of multiple vulnerability factors, such as old houses, areas with high population density, unprotected historic buildings, presence of factories, etc., the seismic risk of the region is very high.

The seismic history of this region is characterized by a high dispersion and variability of the epicentres and, therefore, of the events observed. Generally, the earthquakes in the Banat region occurred at the contact between the Carpathian and Pannonian plates (from Timisoara to S-W Jebel and Banloc and north of the Bega canal) and at the contact among irregular structures straddling Sânnicolau Mare, Nădlag-Jimbolia, Arad-Vinga- Calacea and Timis Valley in Faget. Moreover, the distributions of focal mechanisms associated to the earthquakes occurred in the South-West of Romania showed a reverse and strikes faults or the combination of the two. The focal depths of earthquakes in the western areas of Oltenia and Banat-Danube were between 5 and 33 km deep, as indicated in Figure 1 [14].

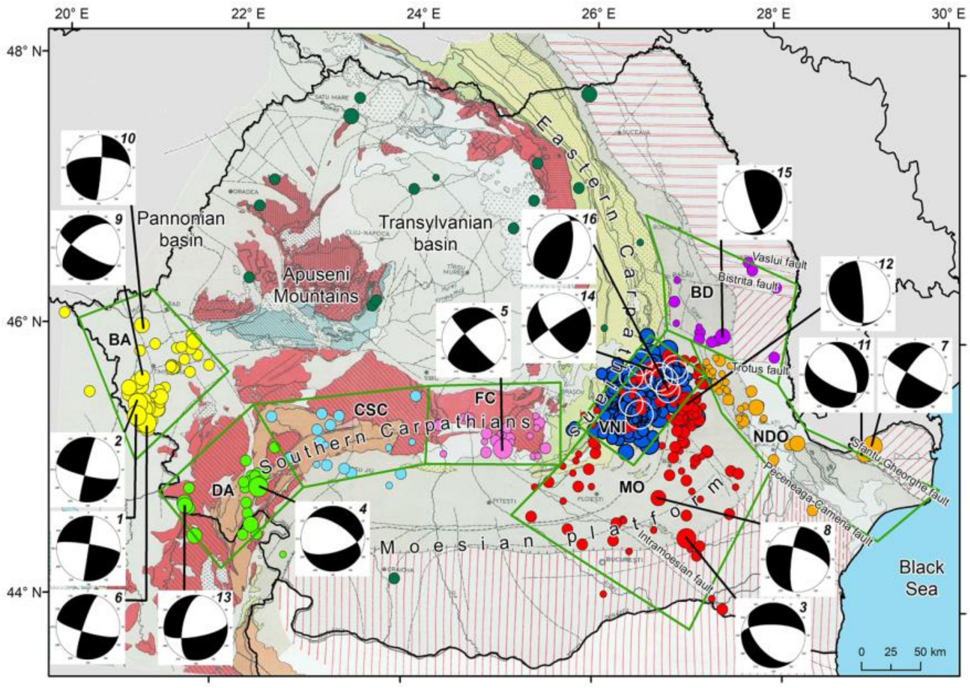


Figure 1: Focal mechanisms occurred in Romania [14].

The most significant earthquakes occurred in the Banat area were the two seismic events detected in Banloc in 1991 and denoted by moment magnitude of 5.5 and 5.6. These main records have been acquired by the INCERC seismic network (Banloc Town-Hall and Timisoara stations). In particular, the accelerometer records taken by INCERC for the cities of Timişoara and Banloc provided earthquakes with the following characteristics: (i) relatively short duration of 9-24 s and (ii) significant spectral values in the range 0.1-0.3 s, which extends up to 0.7 s for the Timişoara record and up to 1.2 s for the Banloc one.

Focusing on the event recorded in the city of Banloc, it is characterized by a reduced site-source distance (7.01 km far from the epicentre), which makes it as a near-source event.

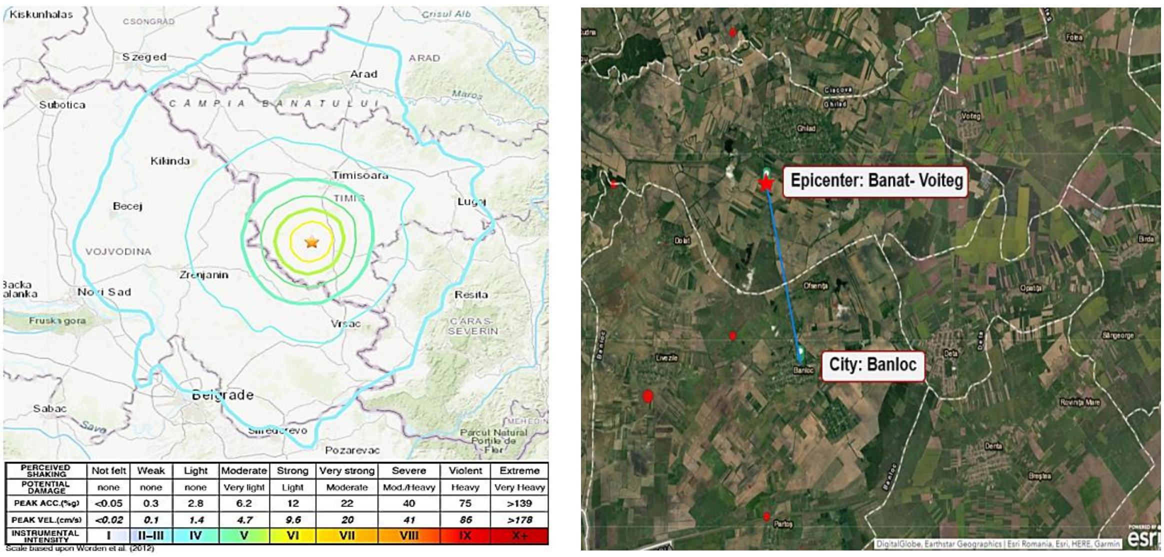
This event showed an impulsive and high vertical peak acceleration of 1.2863 ms^{-2}, a focal depth of 9 km, a moment magnitude, M_w, of 5.5 and an intensity, I, of VIII according to Mercalli Intensity Scale (MCS).

The data provided by the SM-ROM-GL Earthquakes DB [15] related to the 1991 event occurred in the Banat Region are shown in Table 1.

Earthquake	Date	LatN	LongE	Depth - *h* (km)	M_w	Station
Banat-Voiteg	1991/12/02	45.45	21.12	9	5.5	BNL1

Table 1: Reference event occurred in the Banat seismogenic zone in December 1991.

Furthermore, the ShakeMap of the event occurred and the geolocation of the city of Banloc are shown in Figure 2.



(a) (b)

Figure 2: ShakeMap of the Banat-Voiteg event (a) and geolocation of the city of Banloc (b).

3 PRELIMINARY ANALYSIS OF THE CASE STUDY BUILDING

3.1 Historical overview on the Banloc Castle

The municipality of Banloc is attested for the first time in May 1400, when the name "*By-allak*" was reported in a document provided to the clerk of Cenad.

For almost two centuries (1552–1716) Banloc was the summer residence of the Ottoman pasha of Temeşvar Eyalet. In 1716, the Banat region, where Banloc is located, was conquered by the Austrians and, in the census of the year later, the municipality was registered as “*Panloch*” in the district of Ciacova with 85 inhabited houses.

In 1759 the construction of the castle of Banloc was completed.

The castle (Figure 3) is the most important monument of the municipality. It is a massive building with a "U" in-plane shape built in the early nineteenth century. The main façade faces south, while on the opposite side the two wings border a terraced courtyard.

The castle is composed of very heavy perimeter brick and internal walls having thickness of 0.90 m and 0.80 m, respectively at ground level.

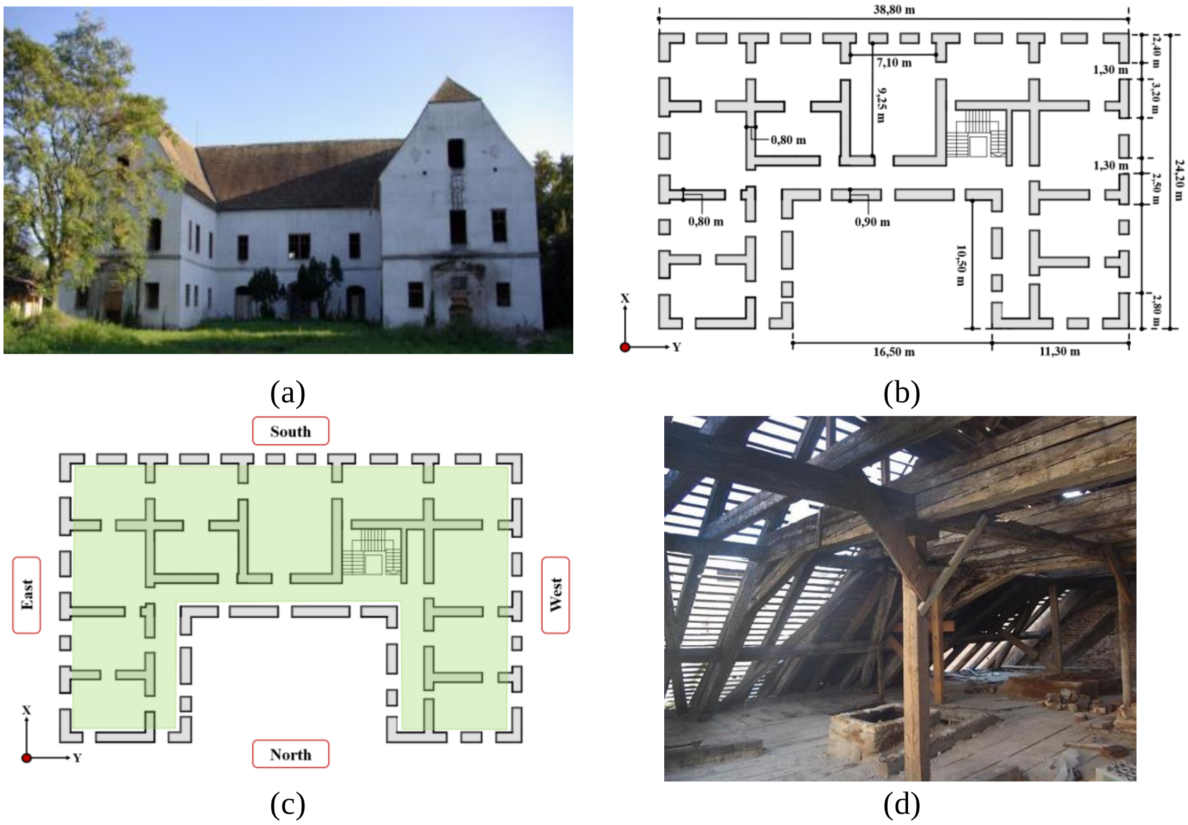


Figure 3: Castle of Banloc: (a) external view, (b) architectural plan layout, (c) geographical orientation and (d) timber roof structure.

On the top, vaulted ceilings with thickness of 0.25 m are present. The inter-storey heights of first and second floors are 4.00 m and 5.70 m, respectively. The roof is made of a solid timber supporting structure, whose construction technique was frequently used in the Banat region (Figure 3d).

3.2 Field evidence after the Banloc earthquake

The Banloc earthquake, occurred on 1991 December 2nd, was a very significant event in the seismic historicity of the Banat region. It was characterized by a focal depth of 9 km with the epicentre localized in Banloc. The released seismic energy produced a magnitude (M_w) of 5.5 on the Richter scale [15]. This earthquake can be classified as a medium-deep geological event. It caused slight damages to new designed structures, but several masonry buildings in the area were characterized by very serious damages, also with partial collapses. Referring to the case study building, the damage was detected through on-site inspections after the earthquake occurred. The achieved post-earthquake damages are presented in Figure 4 [16].


(a)


(b)


(c)


(d)

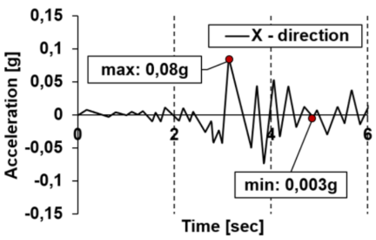
Figure 4: Detected damages on the Banloc Castle after the 1991 earthquake: (a) North façade; (b) West façade; (c) first floor room; (d) second floor room [16]

As it is seen from the previous figures, the highest level of damage was localized in the vaults, which are elements characterized by large spans and resistance lower than that of vertical structures (Figures 4c). In particular, the cracks in the vaults were attributable to the impulsive vertical component, which produced very extensive damages. Contrary, large number of walls were affected by shear mechanisms induced by horizontal seismic components with widespread cracks [16].

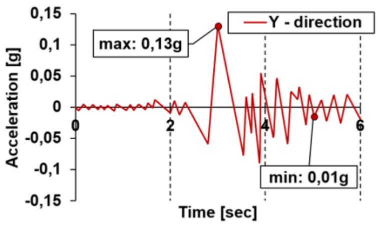
4 STRUCTURAL ASSESSMENT

4.1 Analysis of the ground motion records

The accelerograms adopted are representative of the event occurred in Banloc in December 1991 considering the BNL1 seismometric station [15]. The accelerations are characterized by maximum components in X direction of 0,08g, in Y direction of 0,13g and in Z direction of 0,13g (Figure 5). The seismic phenomenon can be defined of near-source type, since the city of Banloc was located approximately 7 km far from the epicentre.



(a)



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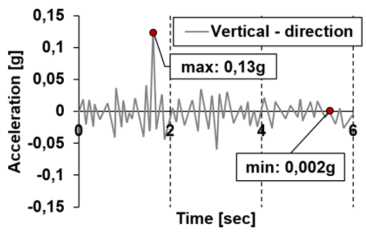


(c)

Figure 5: Accelerograms of the Banat-Voiteg seismic event.

As can be seen from the previous Figure, for numerical analyses, a 6 seconds time history has been considered and elaborated by means of the Seismosignal software [17]. This choice is dictated from the fact that, on one hand, in the first 6 seconds all the seismic energy input deriving from vertical component is manifested and, on the other hand, the analysis elaboration time is short.

The spectral ratios between vertical accelerations, V, and the corresponding horizontal ones, H, for the considered near-source ground motion, have been depicted in Figure 6. These results have shown how the vertical component effects are very important for earthquakes characterized by small site-source distances, where the V/H limit of 2/3 proposed in [9] is exceeded. In particular, Figure 6 shows that for short periods (less or equal than 0,2 seconds), the V/H ratio is greater than 1; this means that the rule imposed in [9] underestimates considerably the vertical component effects for low periods.

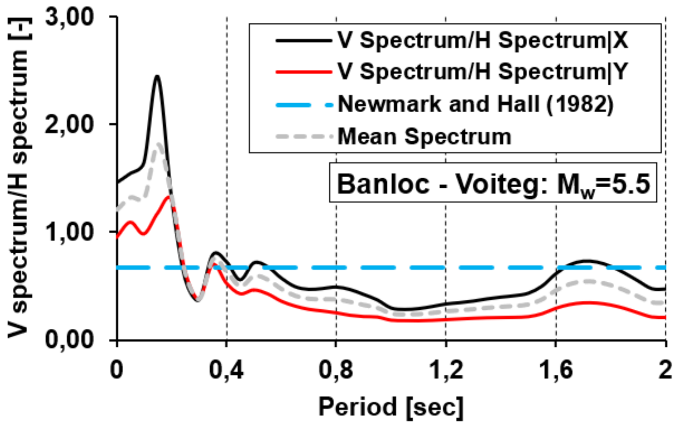


Figure 6: V/H spectral ratios for the Banloc-Voiteg event.

4.2 Non-linear dynamic analysis

Time-history analysis provides a global assessment of the castle dynamic structural response varying specified ground motion accelerograms. The implementation of the model has been set-up using the DIANA FEA software [18], where the structure has been conceived as set of shell elements (Figure 7).

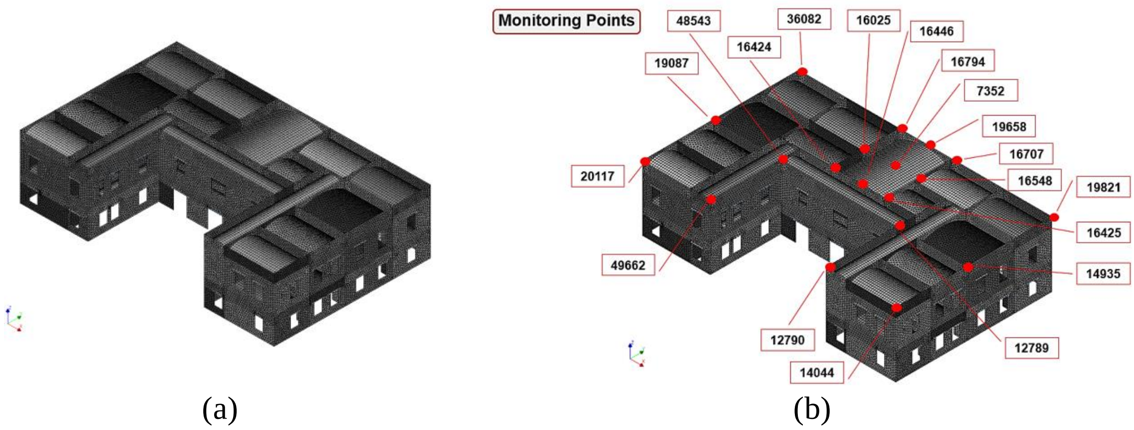


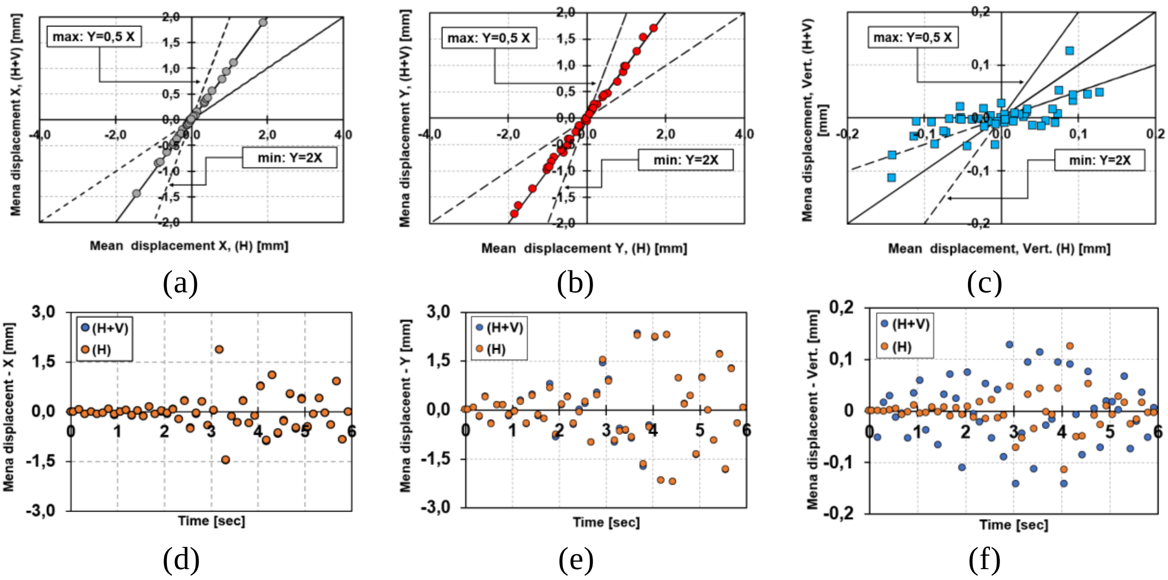
Figure 7: 3D-shell element model of the castle (a) and selected monitoring points (b).

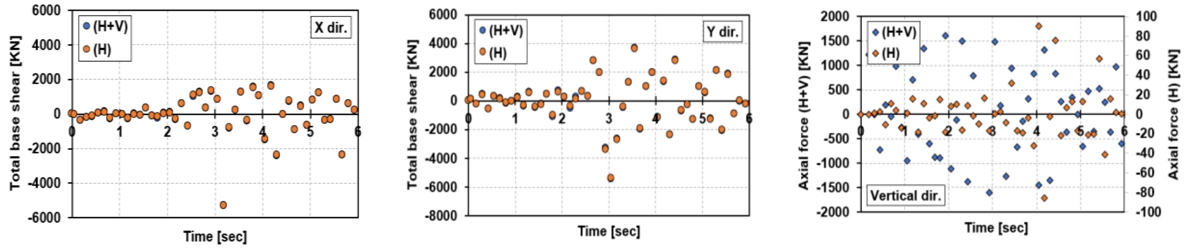
The masonry constitutive law is characterized by a non-linear response with a nonductile post-peak softening behaviour; tensile stresses are assumed to diminish exponentially, while the compression performance combines hardening and softening parabolic phases, as established in [19, 20]. Based on these assumptions, in Table 2 the masonry mechanical properties adopted in the analysis for the case study building have been reported according to NTC18 [21].

Mechanical Properties		
Modulus of elasticity	E [N/mm^2]	1500
Shear modulus	G [N/mm^2]	500
Compressive strength	f_m [N/mm^2]	2.40
Tensile strength	τ_0 [N/mm^2]	0.24
Specific weight	w [KN/m^3]	18
Poisson ratio	ν [-]	0.20
Fracture energy (compression)	G_{fc} [N/mm]	4.64
Fracture energy – Mode I (tensile)	G_{ft} [N/mm]	0.012

Table 2: Mechanical properties of masonry.

The results obtained have been represented, for the two scenarios considered, namely (H) and (H+V), in terms of dispersion of both mean displacements and resistances. They are referred to the monitoring points at the second floor (see Figure 7) and are summarized in Figure 8. Concerning displacements and strengths regime, positive and negative values of the points cloud denote the change of direction induced by the seismic input with respect to the straight line with equation x=y (bisector). Furthermore, in order to better highlight the influence of the vertical component, the upper (y=0,5x) and lower (y=2x) bounds of the dispersion range have been considered. In this way, the greater scatter with respect to the bisector, the larger effect of the ground motion vertical component [11].





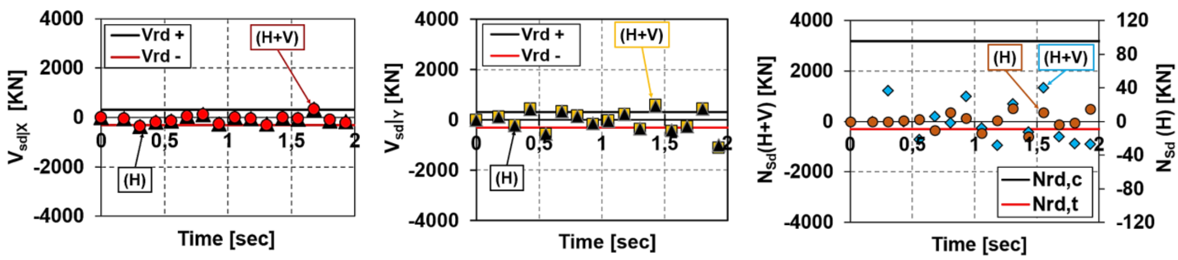
(g) (h) (i)

Figure 8: Scatter results in terms of displacements (a-f) and forces (g-i).

From the gotten results, it is worth noting how the average dispersions associated with the two main directions X and Y, in terms of displacements and base reactions, are not affected by the influence of the vertical component. However, in vertical direction, there is a marked average dispersion (Figures 8c and 8f) with respect to the case in which the vertical component is neglected (H).

It can be seen that in the first two seconds (in which the vertical component reached its maximum acceleration peak), the displacements are increased by about 6 times compared to the case in which the vertical component is neglected (H). Furthermore, in terms of axial stress, the same condition can be found by comparing the two scenarios, (H) and (H+V) mentioned above (Figure 8i).

Consequently, a time step of 2 seconds has been considered in order to consider the effect of ground motion vertical component in terms of both shear [22] and axial demands for one of the most stressed pier of the West façade. So, for the sake of representation, (H) and (H+V) scenarios have been compared and plotted according to the recommendations provided by EC8 [22], as reported in Figure 9.



(a) (b) (c)

Figure 9: Safety checks for shear stresses (a-b) and axial loads (c) in case of (H) and (H+V) scenarios.

Firstly, in Figures 9a and 9b, it is possible to notice how the seismic vertical component in the two main directions, X and Y, does not produce any significant stress variation for both considered scenarios. Secondly, in Figure 9c, it is shown how the vertical component produces a significant increase in terms of axial load. In fact, at time step of 1.55 seconds (which corresponds to the maximum peak vertical acceleration), the difference between (H) and (H+V) is very high (more than two times). Moreover, it is detected that the axial load demands exceed averagely the corresponding tensile capacities ($N_{rd,t}$) of 2 times.

Subsequently, the force reduction factor, q, prescribed in almost all seismic design codes without taking into account the influence of the ground motion vertical component [23, 24], has been predicted for the examined building located in epicentral area of Romania.

The behaviour factor has been assessed both in terms of mean ductility values and areas equivalence criterion between the elastic system and the elasto-plastic one. Based on the energy dissipation criteria, the hysteretic cycles have been defined for the monitoring points mentioned in Figure 7b, and, thus, the mean capacity curves, representative of the global behaviour of the structure in the two main directions, X and Y, have been considered (Figure 10a and 10b). Furthermore, with the same method, the behaviour factor has been calculated when the vertical component is neglected.

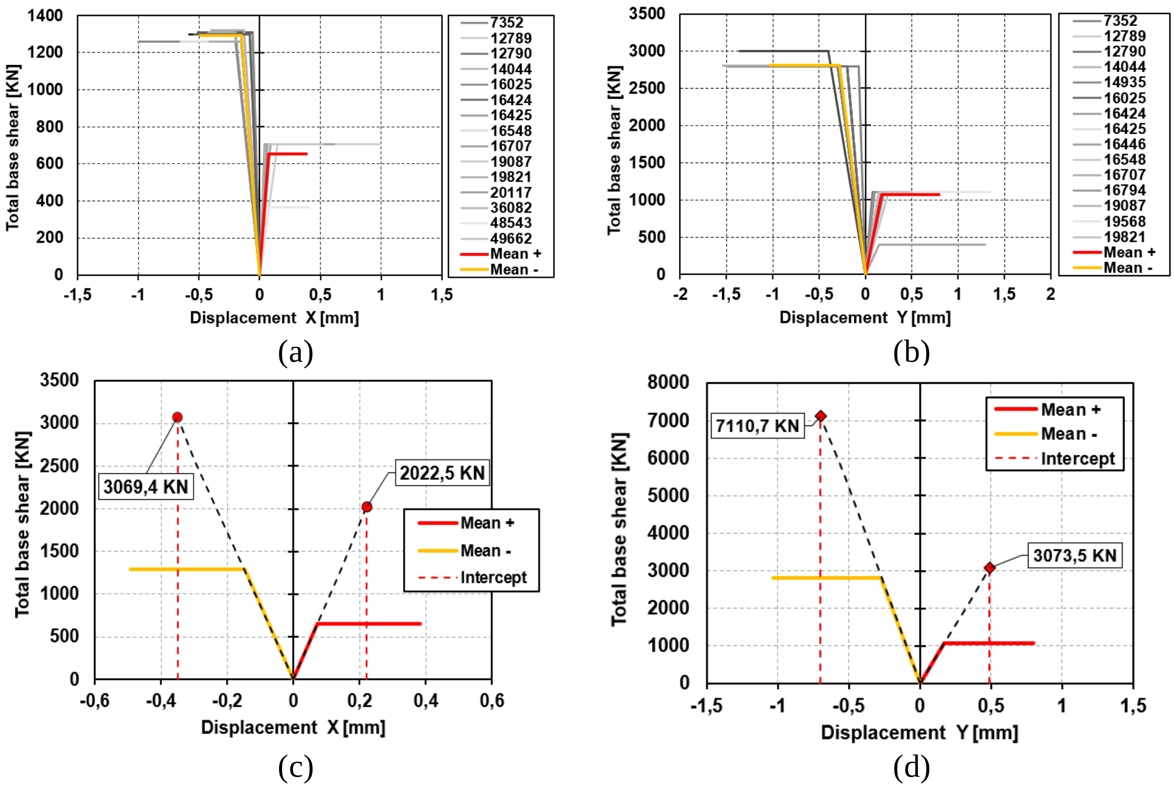


Figure 10: Mean capacity curves for (H+V) scenario in the two main directions X and Y in terms of energetic ductility (a, b) and equivalence criterion of areas (c, d).

From the results obtained, the proposed q-factor is intended as the absolute minimum of the values deriving from the scenarios (H) and (H+V) considered. It has been estimated as equal to 1.94, when the vertical acceleration associated to the case of maximum decompression reached at about 3 seconds in the time history is considered. This choice is based on the fact that, during the decompression phase, the structure presents the highest global vulnerability. Moreover, it is important to observe how the proposed value is contained in the range (1.5-2.5) established by EC8 [22]. Finally, the achieved value shows an increase of about 30% with respect to the value of 1.5 suggested by the Romanian Code [25] for masonry structural systems in case of design spectrum considering vertical acceleration components only.

4.3 Damage assessment

The damages detected in the castle after the Banloc earthquake were represented by a widespread distribution of cracks, that affected the vertical and horizontal structures, producing globally in the time the structure deterioration.

In particular, several failure mechanisms closely related to the seismic event were identified. More specifically, in the first two seconds of the time-history, the maximum vertical peak acceleration produced an extensive damage in the vaults, characterized by a brittle behaviour (Figure 11) [16].

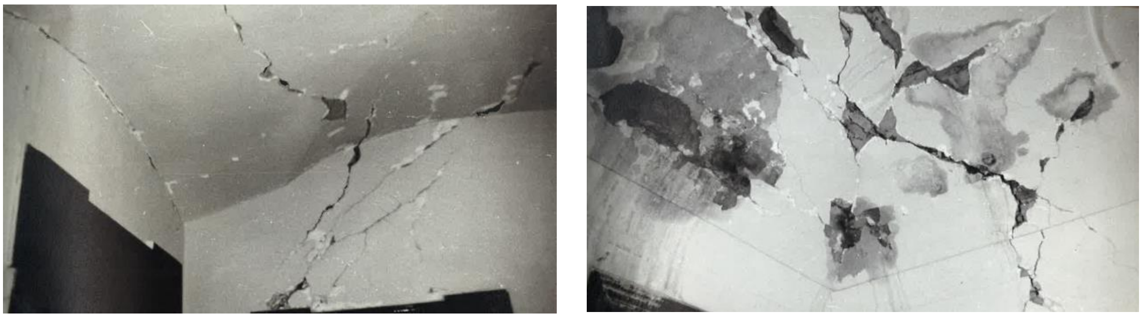


Figure 11: Damages detected in the second-floor vaults mainly due to ground motion vertical component [16].

Successively, following the impulsive action, the arrival of the horizontal components produced damage to the vertical structures, mainly represented by shear cracks in the spandrel beams, as shown in Figure 12 [16].

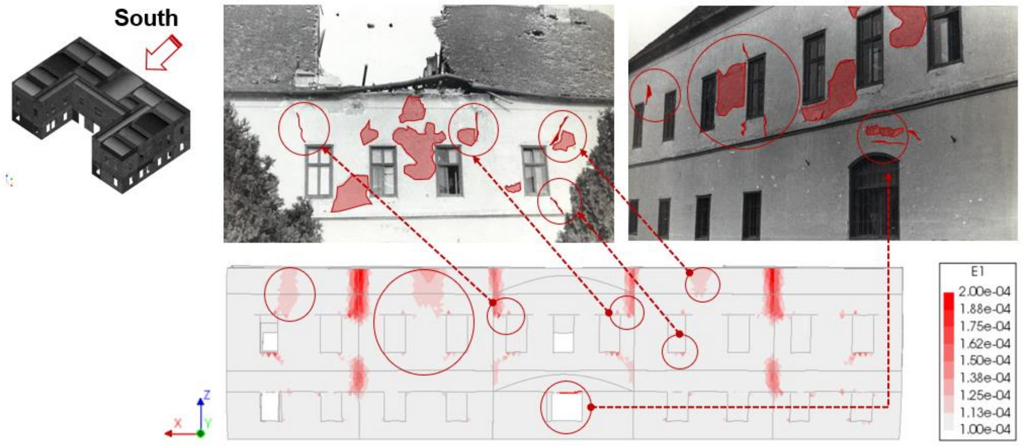
Figure 12: Vertical and diagonal shear cracks detected in the South façade of the case study building [16].

Finally, in the internal court of the building, a partial collapse of the roofing structure was observed. This collapse mechanism was triggered by the impulsive seismic action, as depicted in Figure 13 [16].

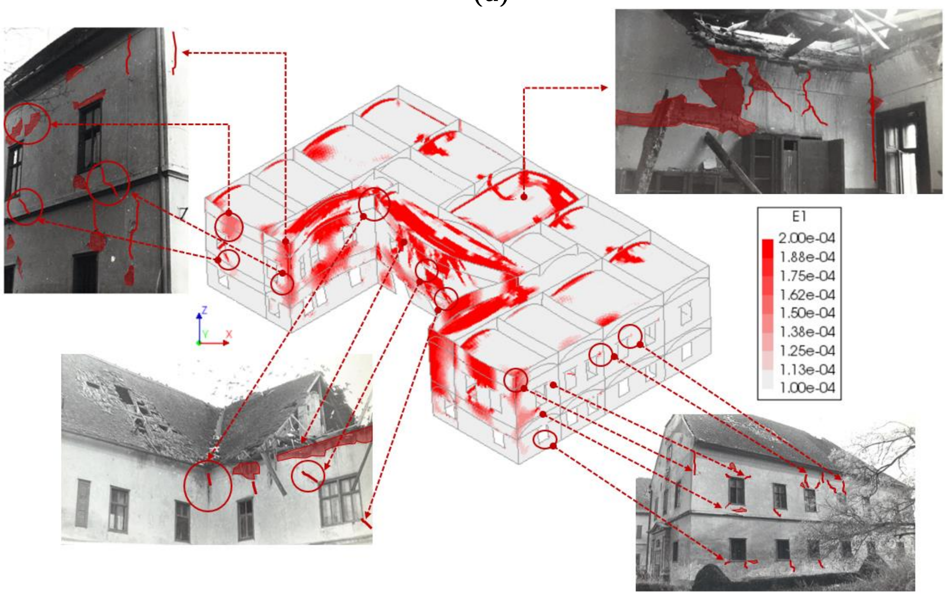
(a) – left view (b) – right view

Figure 13: Failure mechanisms observed in the internal court of the case study building [16].

The damages achieved from numerical analyses have been presented and compared to experimental ones in Figure 14.



(a)



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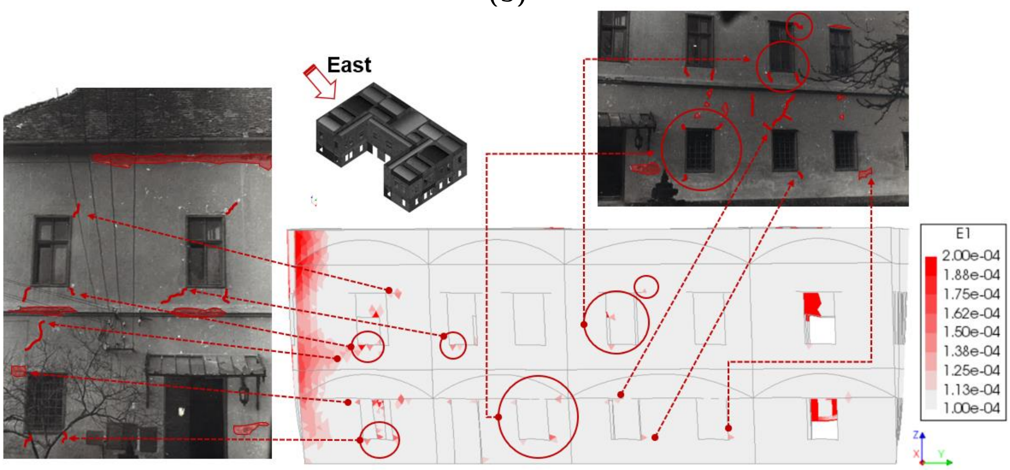

(c)

Figure 14: Numerical damages obtained simulating the Banloc-Voiteg event and comparison with experimental evidences.

From the above comparison, it appears that the non-linear analysis identified local damages very similar to those produced by the Banat-Voiteg earthquake.

The identification parameter used to evaluate the damage state in the numerical analysis is the *tensile fracture energy* (G_f), according to which, when the maximum principal stress reaches the defined cracking criterion, the fracture begins to distribute in the structural element. In particular, the numerical model catches in a satisfactory way the detected local damages and failure mechanisms, providing, as depicted in Figure 14, the right distribution of *Principal Total Strain* (E1).

5 CONCLUSIONS

The proposed work has analysed the effects induced by the vertical earthquake component on a historic masonry building located in the Romanian region of Banloc. The seismic event, occurred in December 1991 with a moment magnitude of 5.5, was characterised by a small (about 7 km) site-source distance. This impulsive event was identified by low frequency content and maximum PGAs comparable in vertical and horizontal directions. The accelerometric records were used to perform non-linear dynamic analyses on the case study building modelled with the DIANA FEA software using shell elements.

The effects of the ground motion vertical component were evaluated considering the comparison between two different scenarios: the first, named H, where only horizontal acceleration components were considered, and the second, called H+V, where also the vertical components were taken into account. The main results are summarized as follows:

- The considered record, due to the combination of medium magnitude and short site-to-source distance, had high energy in terms of vertical component in the first two seconds.
- There was a dispersion of the displacements induced by the vertical component in the first two seconds of the time-history, since the impulsive action releases a high input energy in that time period. The displacements increased about 6 times compared to the scenario in which the vertical component is neglected.
- In the first two seconds of time history, where the maximum vertical energy content of the seismic motion occurred, the comparison of the two scenarios (H and H+V) in terms of stress regime was done. In particular, in terms of shear strength the vertical component did not produce any significant variation of results in the two analysis directions. Conversely, referring to the axial stress regime, the vertical component induces a considerable increase of axial tensile loads, which exceeded averagely the corresponding tensile capacities of about 2 times.
- The q-factor for the study castle was predicted taking into account the influence of the ground motion vertical component. The achieved value was enclosed in the range established by EC8 and it is 30% greater than that provided by the Romanian Code for masonry structures in case of vertical acceleration design spectrum. Moreover, other research activity in this field will be study in future.
- The numerical damage assessment was carried out considering the *Principal Total Strains*, which take into account the overcoming of the tensile fracture energy in the structural elements. The numerical analysis results provided with satisfactory accuracy the same damage mechanisms really detected in the castle, namely cracks in the vaults (produced by vertical components) and vertical structures (mainly caused by horizontal components), as well as the out-of-plane horizontal bending mechanism in the façade facing the courtyard, which was produced by the partial collapse of the roof structures.

REFERENCES

[1] V. Gioncu, FM. Mazzolani, Influence of earthquake types on the design of seismic-resistant steel structures. Part 1: Challenges for new design approaches. Part 2: Structural response for different earthquake types. Behavior of Steel Structures in Seismic Areas. STESSA 2006 (eds. F.M. Mazzolani and A. Wada), Taylor & Francis, London, 113-120, 121-127.

[2] V. Gioncu, FM. Mazzolani, Earthquake Engineering for Structural Design, Spon Press, London, ISBN 0-203-84889-6 Master e-book ISBN, 2011.

[3] V. Gioncu, FM. Mazzolani, Ductility of Seismic Resistant Steel Structures, Spon Press, London, ISBN 9780367865313, 2002.

[4] M. Mosoarca, V. Gioncu, Failure mechanisms for historical religious buildings in Romanian seismic areas, *Journal of Cultural Heritage* 14, e65-e72, 2013.

[5] N. Chieffo, A. Formisano, The influence of geo-hazard effects on the physical vulnerability assessment of the built heritage: An application in a district of Naples, *Buildings* 9, 26.

[6] M. Mosoarca, I. Onescu, E. Onescu, B. Azap, N. Chieffo, M. Szitar-Sirbu, Seismic vulnerability assessment for the historical areas of the Timisoara city, Romania. *Engineering Failure Analysis* 101, 86-112, 2019.

[7] CJ. Collier, AS. Elnashai, A Procedure for Combining Vertical and Horizontal Seismic Action Effects, *Journal of Earthquake Engineering* 5, 521-539.

[8] PG. Somerville, Magnitude scaling of the near fault rupture directivity pulse, *Physics of the Earth and Planetary Interiors*, 137,201-12, 2003.

[9] N. Newmark, W. Hall, Earthquake Spectra and Design, EERI Monographs 1982.

[10] KW. Campbell, Y. Bozorgnia, Updated near-source ground-motion (attenuation) relations for the horizontal and vertical components of peak ground acceleration and acceleration response spectra, *Bulletin of the Seismological Society of America* 93, 1872.

[11] F. Di Michele, C. Cantagallo, E. Spacone, Effects of the vertical seismic component on seismic performance of an unreinforced masonry structures, *Bulletin of Earthquake Engineering* 18,1635–56, 2020.

[12] AJ. Papazoglou, AS. Elnashai, Analytical and field evidence of the damaging effect of vertical earthquake ground motion, *Earthquake Engineering and Structural Dynamics* 25, 1109-37, 1996.

[13] AS. Elnashai, L. Di Sarno, Fundamentals of Earthquake Engineering. 2008.

[14] A. Bala, D. Toma-Danila, M. Radulian, Focal mechanisms in Romania: statistical features representative for earthquake-prone areas and spatial correlations with tectonic provinces, *Acta Geodaetica et Geophysica* 39, 309-314, 2019.

[15] IS. Borcia, I. Craifaleanu, Examples of use of the SM-ROM-GL Database, *In 5th National Conference on Earthquake Engineering and 1st National Conference on Earthquake Engineering and Seismology–5CNIS & 1CNISS*, Bucharest, Romania, (2014), 1:165-172.

[16] IPROTIM TIMISOARA, Expertiza cu principii de consolidare (In Romanian), Pr. Nr. 35353/330, (1990).

[17] Seismosoft, SeismoSignal, User's Manual. 2018.

[18] DIANA FEA, Diana User's Manual, Release 10.2. DIANA FEA BV 2017.

[19] PB. Lourenço, Computations on historic masonry structures, *Progress in Structural Engineering and Materials* 4, 2002.

[20] PB. Lourenço, Structural Masonry Analysis: Recent Developments and Prospects, *J Proceedings of 14th International Brick and Block Masonry conference. University of Newcastle. Australia* 53, 2008.

[21] MD. 17 January 2018, Updating of Technical Codes for Constructions, Official Gazette n. 42 of 20/02/18, Ordinary Supplement n. 8, 2018.

[22] EN 1998-3 - Eurocode 8, Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings, CEN 2004:1–97.

[23] M. Tomaževič, P. Weiss, Displacement capacity of masonry buildings as a basis for the assessment of behavior factor: An experimental study, *Bulletin of Earthquake Engineering* 8, 1267-1294, 2010.

[24] D. Zonta, G. Zanardo, C. Modena, Experimental evaluation of the ductility of a reduced-scale reinforced masonry building, *Materials and Structures* 34, 636–44, 2001

[25] Technical University of Civil Engineering Bucharest, Cod de Proiectare Seismica-Partea I- Indicativ P100-1, 2013.