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SEISMIC RETROFIT OF AN EXISTING HYBRID STRUCTURE BUILDING USING CONCENTRICALLY STEEL BRACED FRAMES

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Abstract

Damage caused by earthquakes is a significant threat for many communities around the world. Existing buildings, designed without any specific seismic requirement or for lower seismic demands (than in force today) are especially vulnerable, and may require structural assessment and extensive interventions. If required, reversible technologies may be applied, allowing both the enhancement of seismic performance and future up-grading intervention.
The building subjected to intervention is composed of hybrid steel-concrete frames and unreinforced masonry walls and is located in a moderate seismicity area. The three story building has been constructed in 1960, before the introduction of the first seismic design code in Romania, in 1963. The study presented in the paper summarizes the main structural features and the results of the numerical simulations carried out to validate the retrofitting intervention.

Keywords: seismic vulnerability, existing building stock, concentric bracing, steel frame, hybrid system.

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1 INTRODUCTION

Seismic vulnerability of existing building stock is an important issue in many parts of the world. In Europe, a large percentage of buildings have been designed and constructed before the introduction of modern seismic design requirements or were designed for lower requirements than those in force today [1]. The problem is even more demanding in higher seismicity areas, e.g., the Balkan and Mediterranean countries, where these buildings are likely to suffer significant damage even from moderate earthquakes. To avoid collapse of these buildings in case of severe earthquakes, structures need to be strengthened, thus creating a more robust structure [2]. If intervention eases the repairing of the structure and the recovery of its functionality, then it becomes more resilient.

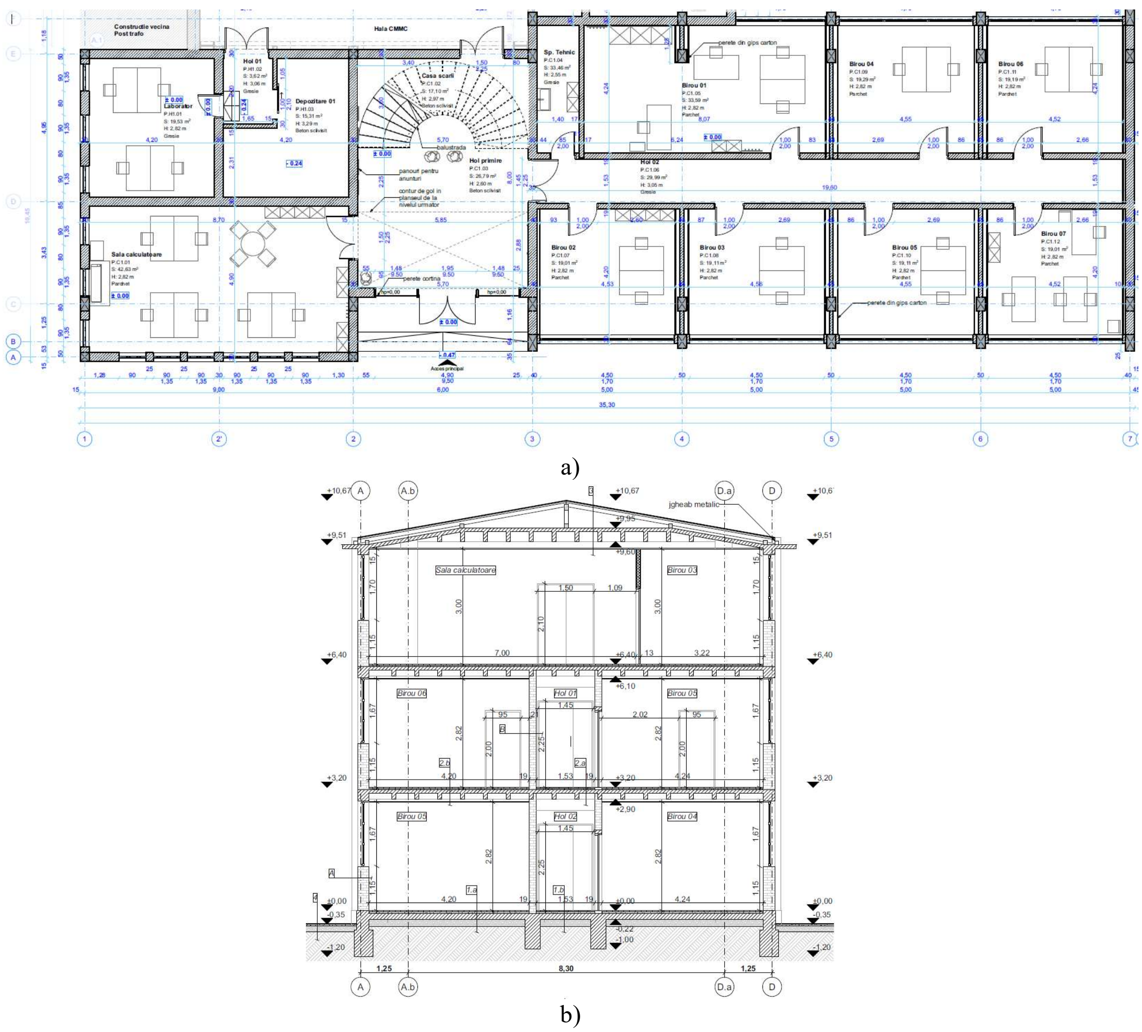
Steel based solutions are largely used for retrofitting/upgrading vulnerable buildings. Their efficiency relies on better control of the contribution to the lateral resistance and stiffness, and of the energy dissipation capacity [1]. If needed, their reversibility (i.e., the system can be easily removed) can be also considered and detailed accordingly [3], [4], [5].

The paper presents the retrofitting interventions on an existing building with a hybrid structural system made of steel-concrete frames and masonry walls. The building has 3 stories and is located in an area characterized by moderate seismicity. The building was completed in 1960 (see Figure 1), before the introduction of the first seismic design regulations in Romania. In order to limit the damage at the interior of the building, the strengthening works were carried out by applying an external steel skeleton (exoskeleton). The main structural characteristics and the results of the numerical simulations performed for the validation of the retrofitting seismic intervention are summarized. The seismic structural assessment was performed using nonlinear response history analyses and three near-fault ground motion records. The analyses were performed in ETABS 2000 [6] program.

Figure 1: View with the building after completion, 1960.

2 STRUCTURAL ASSESSMENT OF THE CASE STUDY BUILDING

The building proposed for intervention is located in Timisoara, Romania, has 3 stories and was completed in 1960. A decade later, an extension of the building was built, but the new construction was separated from the existing one using a seismic joint. Only the initial building is of interest in the present paper. The building hosts several functions, namely the office area, the administrative area and the teaching area. The story height is 3.15 m, the transversal spans are 10.9 m, and the longitudinal bays are 5.00 m, see Figure 2. The structure is composed of transversal steel frames encased in reinforced concrete, and load-bearing masonry walls in transversal and longitudinal direction. The floors are made of reinforced concrete two-way joist slab. The roof floor uses transversal steel beams with variable height. Columns and beams are made with compound sections made from a single hot rolled IPN260 profile (260x113x14.1x9.4 mm) with an additional UPN180 profile (180x70x11x8), and grade of steel S235. They are encased in reinforced concrete sections, but with low longitudinal reinforcement, i.e., four 12 mm longitudinal bars, and 10 mm stirrups placed at 250 mm intervals. The nominal concrete class is C12/16. The poor reinforcement did not guarantee sufficient interaction with the steel elements and was disregarded in the structural assessment.



a)

b)

Figure 2: Plan view (a) and current transversal frame (b).

According to the seismic design code in force today [7], the building is located in a moderate seismicity zone. However, at the time of construction, the design was done without considering any seismic provisions. One may note that, following the strong 1940 earthquake originated from Vrancea source, which is one of the most active seismic zone in Europe ([8]), first seismic instructions were issued in 1941 and 1945, stipulating a lateral seismic force of 5% of the building weight. However, the building site, as well as most part of western Romania were not considered as seismic zones. The last seismic code has been issued in 2013 [7], which followed after several updates.

The structural performance of the existing building was evaluated for the persistent and seismic design situations. The loads considered in the assessment were 3.0 kN/m^2 dead load and 3.0 kN/m^2 live load on the floors, 0.72 kN/m^2 side wind pressure, and 1.2 kN/m^2 snow load on the roof. The seismic assessment was done using modal response spectrum analysis, and parameters defined above, i.e., T_C= 0.7 s and a_g=0.2 g; the q factor (behavior factor) has been limited to 1.5, to account for overstrength but no hysteretic energy dissipation capacity, see Figure 3).

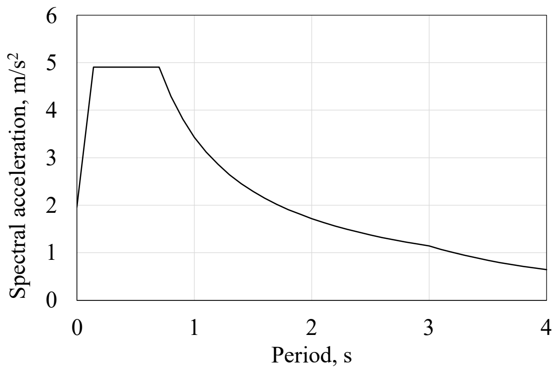


Figure 3: Design response spectrum for the site [7].

The structural analysis was performed using a 3D model in ETABS program, [6], see Figure 4. The columns and beams were modelled as bar steel elements without considering the interaction with the concrete encasement. The concrete slabs were modeled using layered shells and nominal characteristics for C12/16 concrete class. The masonry walls were modeled using layered shells and the following characteristics (see provisions from [9]): compressive strength f_k = 2.3 MPa, short term secant modulus of elasticity E = 2300 MPa, Poisson' ratio ν = 0.2. Note that, the short term secant modulus of elasticity, *E,* for use in structural analysis, was taken as $K_E \times f_k$, where K_E is 1000 (see provisions from [9]). Even not directly specified in [9], and considering also the interaction with the steel frame structure and the share of the seismically induced forces between masonry walls and steel frames, the assessment of the structure has been done considering a "reduced" stiffness. This "reduced" modulus, or long term modulus, is based on the short term value, reduced to allow for creep effects, and results in $E_{long\ term}$ = 1550 MPa (see clause 3.7.2(3), [9]), which is obtained using a value of 0.5 for the final creep coefficient, ϕ_∞. This reduction allows us to consider a possible increase in the seismic demands on the steel frame system.

The perimeter longitudinal walls in the office area were not included in the structural model because they are arranged only on half the height of the floors (parapet to support the windows). The interior longitudinal walls in the office area were also not included in the structural model because they are partially arranged on the floor height and have a reduced thickness (15 cm). However, their contribution to the mass of the structure on the 1st and 2nd level was included in the analysis.

The determination of the seismic effects was done using a modal response spectrum analysis. As the structure is irregular in plan due to the non-uniform arrangement of lateral load resisting components (masonry walls, steel frames), the structure has been analyzed using a spatial model. In addition, according to [7], [10], these irregularities were taken into account by means of an accidental eccentricity, e_{ai}, of $\pm 0.05\ L_i$, where L_i is the floor-dimension perpendicular to the direction of the seismic action.

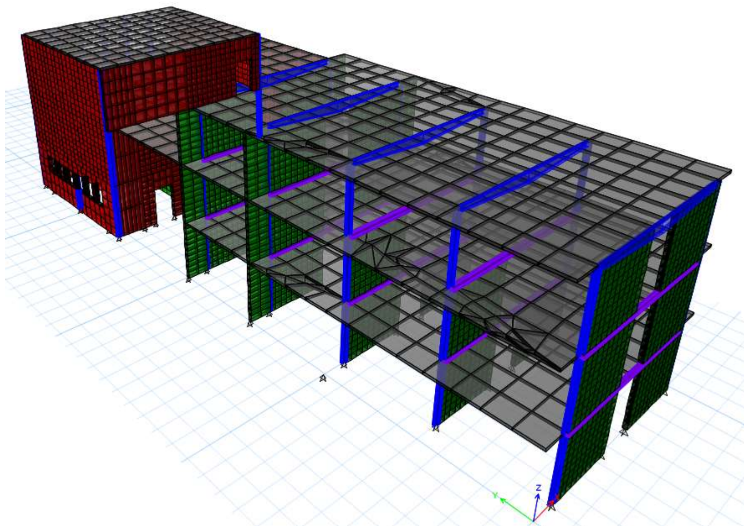
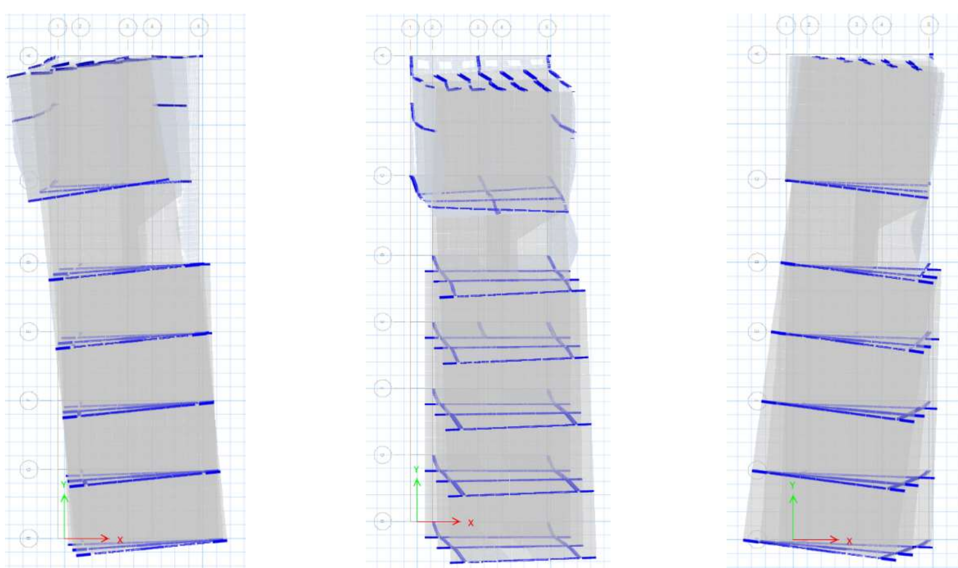


Figure 4: 3D model of the existing structure in Etabs.

The actual structural system has adequate performance under load combinations from persistent design situations, for both Ultimate and Serviceability Limit States. However, in the seismic design situation, the structure does not meet the requirements, with the maximum seismic risk class II for masonry walls. This indicates the building is susceptible to major damage to the action of the design earthquake, corresponding to the ultimate limit state, which may endanger the safety of users, but in which total or partial collapse is unlikely. For the steel frame elements, the maximum seismic risk class was III, indicating constructions which, under the effect of the design earthquake, may present structural degradations that do not significantly affect structural safety, but in which non-structural degradations may be significant. The results of the modal analysis confirmed the structure is torsionally sensitive, with two out of three first modes of vibration with strong torsional components, see Figure 5. Note that the results presented in Figure 5 were obtained considering the long term modulus, $E_{long\ term}$.



$T_1 = 0.34$ s $T_1 = 0.29$ s $T_1 = 0.25$ s

Figure 5: First 3 modes of vibration (top view).

3 RETROFIT STRATEGY

Based on the conclusions of the seismic assessment, a seismic retrofitting strategy has been developed. In order to enhance the response of individual structural elements that show deficiencies, but also to improve the global seismic behavior and correct the torsional effects, a retrofit intervention has been designed.

The intervention was designed considering the application of an external metal skeleton (exoskeleton), see Figure 6. The exoskeleton is placed on both longitudinal ends on transversal and longitudinal directions and is connected with the existing building at each floor level using short steel stubs made of tubular sections. Considering the masonry walls are rather stiff, the decision was to use concentrically braced frames, which are also stiff compared with other structural systems. The steel frames are made of HEB 300 columns, HEB 240 beams, and CHS 193.7x4.2 diagonals, all made of steel S355 J2. The site connections are all bolted, using class 10.9 high strength bolts, see Figure 7.

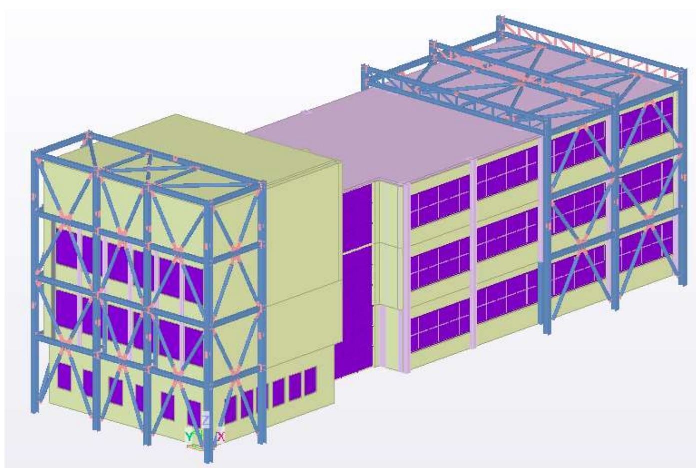


Figure 6: 3D model with the existing building and retrofitting intervention.

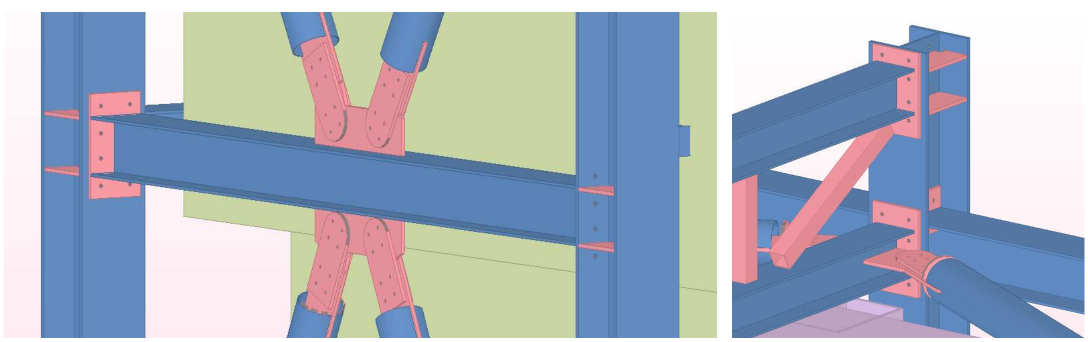


Figure 7: Detailed view with the exoskeleton: a) view with side frame and concentric braces; b) view with the top transversal truss beam and horizontal bracing.

The structural assessment was done using nonlinear response history analyses and three near-fault ground motion records. The analyses were performed in ETABS 2000 [6] program. Three ground motion records, typical for the site and recorded during July 12, 1991, Banat earthquake, were used in the analysis, see Figure 8, left. The earthquake intensity was $M_w = 5.7$, being one of the strongest seismic event in the region in the last decades [11]. The accelerograms were scaled to meet the spectral response accelerations in the range of periods between $0.2 \times T_1$ and $2 \times T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram will be applied [10] (see Figure 8, right). The behavior of masonry walls subjected to cyclic loadings has been modeled using layered shell elements with a thickness of 250 mm and the parameters, see Figure 9 [12], [13]. This modeling approach can provide adequate results with moderate computational demands [14]. Beams and columns were modelled using bar elements with concentrated plasticity at both ends, while concentric diagonals were modeled using bar elements with concentrated plasticity at mid length. Considering the system is hybrid due to the interaction between steel frames and masonry walls, and the share of the seismically induced forces, both short term and long term rigidity characteristics have been considered for the masonry walls. When wall rigidity reduces, the demand on the steel frames increases. The first three modes of vibration for the retrofitted structure are presented in Table 1 and are compared with un-retrofitted structure. As may be seen, the lateral stiffness increases; also, the intervention improves significantly the global behavior, with the first two modes of vibration translational. Note that the results presented in Table 1were obtained considering the long term modulus, $E_{long\ term}$.

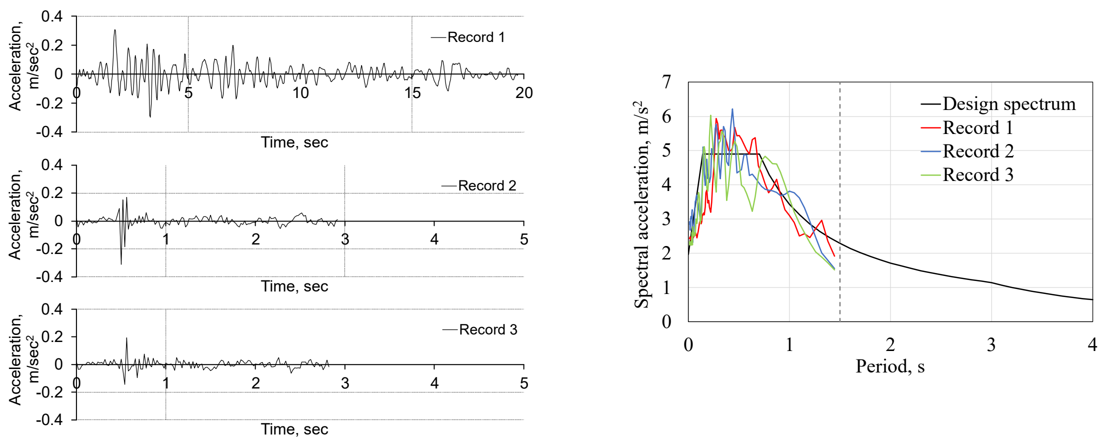


Figure 8: Banat earthquake, July 12, 1991, recorded accelerograms (left), and response spectrums (right).

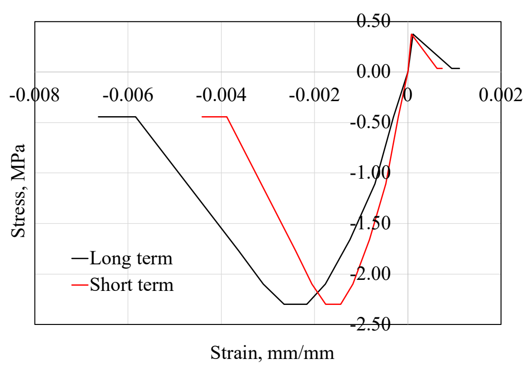


Figure 9: Nonlinear stress-strain behavior of masonry walls.

Retrofitted		Un-retrofitted	
Period (s)	Type	Period (s)	Type
0.28	Transversal	0.34	Torsional + transversal
0.22	Longitudinal	0.29	Longitudinal
0.18	Torsional	0.25	Torsional

Table 1: Results of modal analysis for retrofitted and un-retrofitted structure

Seismic performance of the retrofitted structure was evaluated using an incremental dynamic analysis (IDA), and three performance levels, i.e., Damage Limitation (DL), Significant Damage (SD), and Near Collapse (NC). The accelerograms have been scaled by a factor of 0.5 for DL, and 1.5 for NC, and by a factor of 1.0 for SD, according to provision from [10]. The history of base shear force for the three limit states, considering the maximum values for the three accelerograms, is plotted in Figure 10 for both un-retrofitted and retrofitted structure. For comparison, the base shear force calculated with the response spectrum analysis, and noted with DRS, is also shown. It may be seen the scaling procedure shows good agreement between peak base shear forces from response history analysis at SD Limit state and the reference method for determining the seismic effects, i.e., the modal response spectrum analysis. Also, the retrofitting intervention has a small impact on the total base shear force, with less than 10 % increase. Figure 11 and Figure 12 show the effectiveness of retrofitting intervention. Thus, the maximum compressive stress in masonry walls at Significant Damage (SD) and Near Collapse (NC) reduce after retrofitting to levels lower than the compressive resistance, i.e., 1.45 MPa at SD limit state and 2.05 MPa at NC limit state, respectively. Also, the formation of plastic hinges in the first story columns from the un-retrofitted structure (Figure 12a) at SD and NC are prevented. Note that the capacity is not exceeded in any part of the retrofitting steel system. The contribution of retrofitting system to base shear force is significant on the longitudinal direction, with about 45%, and about 22% on the transversal direction, see Figure 13. These results confirm that the system has a more significant contribution in the longitudinal direction, considered more vulnerable according to the initial seismic structural assessment.

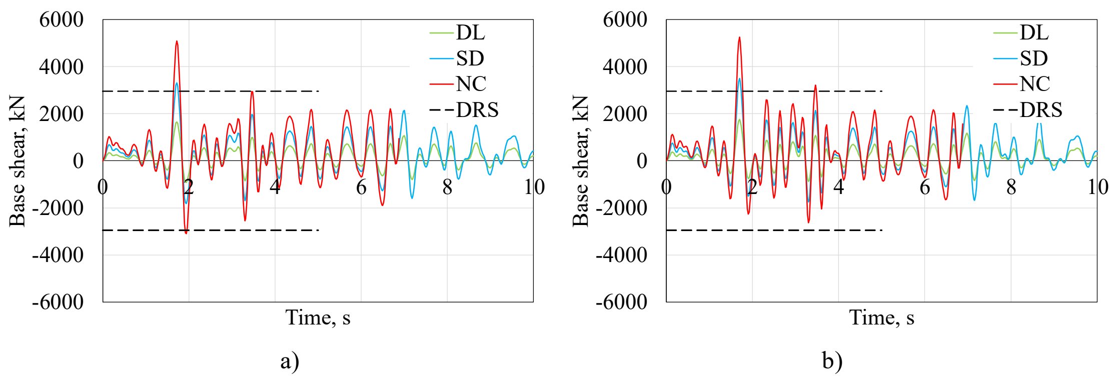


Figure 10: History of base shear force for the three limit states (the maximum values from the three accelerograms), un-retrofitted (a) and retrofitted (b).

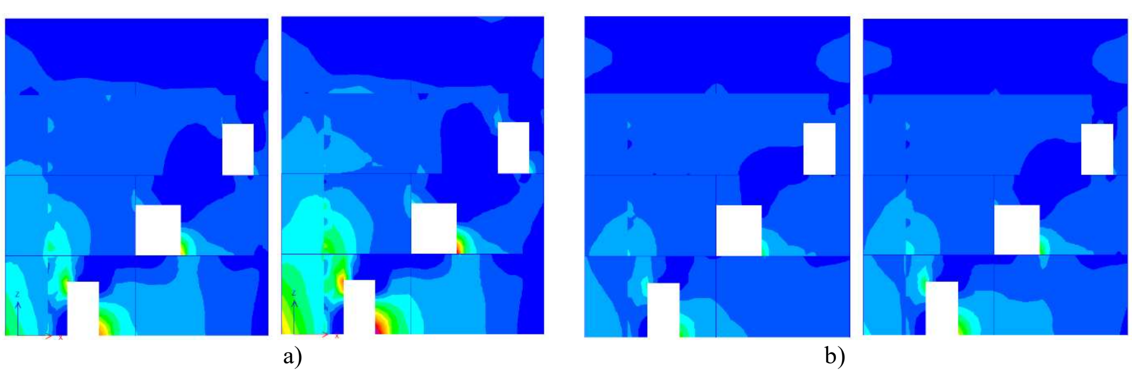


Figure 11: Map of maximum compressive stress in a transversal masonry wall, Significant Damage (SD) (left), and Near Collapse (NC) (right): a) un-retrofitted structure; b) retrofitted structure.

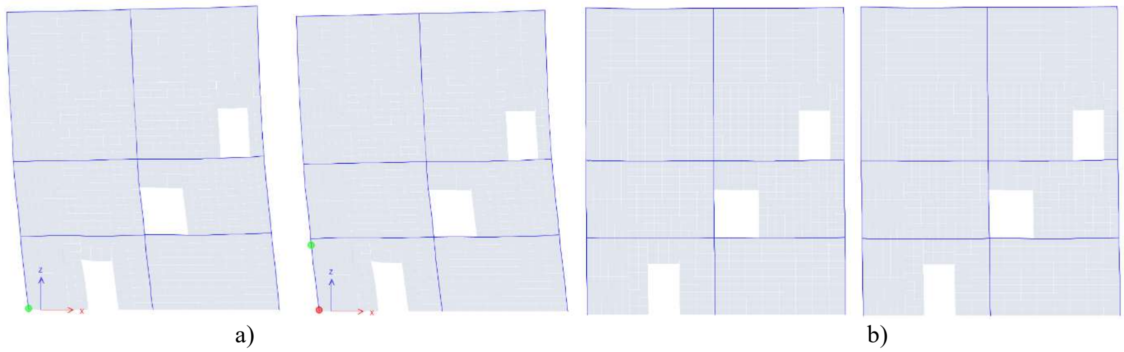


Figure 12: Deformed shape and plastic hinges in a transversal frame, SD (left), and NC (right): a) un-retrofitted structure; b) retrofitted structure.

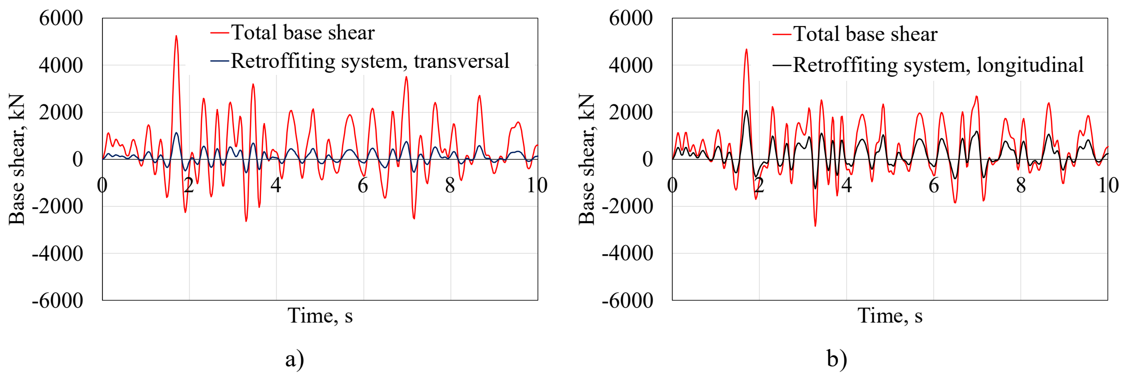


Figure 13: Contribution of the retrofitting system to the base shear force at NC limit state (the maximum values from the three accelerograms): a) transversal direction; b) longitudinal direction.

4 CONCLUSIONS

The building subjected to intervention is composed of hybrid steel-concrete frames and unreinforced masonry walls and is located in a moderate seismicity area. The three story building has been constructed in 1960, before the introduction of the first seismic design code in Romania, in 1963. The main conclusions of the study are the following:

- Damage caused by earthquakes is a significant threat for many communities around the world.
- Existing buildings, designed without any specific seismic requirement or for lower seismic demands than the current provisions, are especially vulnerable, and may require structural assessment and extensive interventions.
- If required, reversible technologies may be applied, allowing both the enhancement of seismic performance and future up-grading intervention.
- The steel base retrofitting system, investigated and described in the paper, demonstrates that the intervention can be done from the exterior of the building, with no or low interventions at the interior of the building.
- The calibration of the retrofitting system with the existing one is of great importance, especially for stiff and low ductility systems, like unreinforced masonry walls.
- In such cases, the two subsystems, i.e., the existing structure and the retrofitting system should be phased to work in parallel, and contribute efficiently in resisting the seismic forces.

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