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Qualitative Seismic Evaluation of the Existing UR Masonry Shear Walls – Reinforced Concrete Frames System

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Abstract

The dual system buildings consisting of Unreinforced Masonry (URM) Shear Walls and Reinforced Concrete (RC) Frames are part of the existing buildings patrimony in Romania. Being a seismic zone, the qualitative evaluation of this type of structural system represents both a specific and a holistic approach. Thus, the in-depth knowledge of the initial situation of the building, including the level of degradation as well as the causes that produced these damages represents the present study. Also, notions such as: the level of degradation importance, the level of damage (destruction), the correction coefficient of the bending stiffness, the level of knowledge, the evaluation methodology are interpreted, in order to establish a score of the degree of seismic configuration of the lateral system and degree of structural damage. This information leads to the establishment of the seismic risk class of the analyzed building and to the establishment of the intervention works necessity.

Keywords: unreinforced masonry (URM), RC frames, dual system, level of damage, seismic assessment

1 Introduction

The dual structures formed by Unreinforced Masonry (URM) Shear Walls and Reinforced Concrete (RC) Frames represent an important part of buildings located in seismic zones [1,2], including in Romania. Their seismic evaluation becomes a national obligation in the conditions of buildings "aging", as the level of damage increases annually [3-5]. These buildings are inhabited, which implies a major assurance of the evaluation quality.
In current practice, there are two types of seismic evaluations:

- qualitative seismic assessment;
- quantitative seismic assessment.

Qualitative seismic assessment of the building is based on:

- in-situ investigations (including identification of the types of damage through the visual inspection [4]);
- a holistic approach to the existing problems in the building;
- identifying the Level of Degradation Importance (LDI);

- identification of Level of Damage (Destruction) in Percentage (LDP) [6,7];
- establishing the impact of the damage level to lateral stiffness of the structural elements [8];
- establishing the level of knowledge;
- establishing the evaluation methodology;
- establishing the degree of seismic configuration of the lateral system (by agreeing a score);
- establishing the degree of structural damage (by agreeing a score);
- the preliminary classification of the building in a seismic risk class;
- establishing at a general level the need for intervention works.

The current study only approach this type of assessment. The quantitative seismic assessment of the building is based on the qualitative seismic assessment (and in-situ information) to be able to perform numerical analyzes of the superstructure and the infrastructure [9-11]. The current study does not approach this type of evaluation.

2 Preliminary data regarding the analyzed building

The analyzed building is located in Romania, Suceava County, and falls into importance category C and earthquake importance and exposure class III in accordance with HG. 766/1997 [12] and P100-1/2013 [13] norms.
The structural system consists of Unreinforced Masonry (URM) Shear Walls and Reinforced Concrete (RC) Frames, with the height regime GF+1F (Ground Floor + 1 Floor). The wall thickness varies between 30 and 50 cm. The foundation system is composed of a RC grid beams. The height of the foundation beams is 2.00 m and their width is 0.60 m.
Centralized information regarding the analyzed building is specified in Table 1. The main facades of the building are represented in Figure 1.

Table 1: Centralized information regarding the analyzed building

The purpose of qualitative seismic evaluation	Establishing the conditions for additional floor construction and thermal retrofitting [14]		
County and Country	Suceava County, Romania		
Importance category [12]			C
Earthquake importance and exposure class [13]			III
Year of completion of the building construction works:	1952		
Building function	Commercial space/ offices		
Total height of the structure (m):	+10.14	Number of floors:	Ground Floor+1 Floor (GF+1F)
Built area (sqm):	404.00	Unfolded area (sqm):	804.00
Structural system:	The structural (lateral) system of the building is made up of UR (Unreinforced) Masonry Shear Walls (MSW) and Reinforced Concrete Moment Frames (RCMF). The outward MSW are 47 cm thick on the GF and 47 cm/ 42 cm		

<table>
<tr><td></td><td colspan="5">thick on the 1F. The internal MSW are 40 cm thick on the GF and 30 cm/ 40 cm thick on the 1F. The Reinforced Concrete Columns (RCC) have a cross section of 35x56 cm and the Reinforced Concrete Beams (RCB) have a section of 30x60 cm.
The Reinforced Concrete Slabs (RCS) above the GF and 1F are 10 cm thick.
The roof of the building is composed of the timber frame system with metal sheet covering.</td></tr>
<tr><td>Foundation system</td><td colspan="5">The foundation system is composed of a Reinforced Concrete grid beams. The height of the foundation beams is 2.00 m and their width is 0.60 m.</td></tr>
<tr><td>Non-structural components:</td><td colspan="5">-</td></tr>
<tr><td>Seismic action (probability of exceedance in 50 years):</td><td>*SLS:</td><td>80%
(*ARI 30)</td><td>*ULS:</td><td colspan="2">40%
(*ARI 100)</td></tr>
<tr><td colspan="2">Seismic evaluation methodology used (in accordance with P 100-3 norm) [15]</td><td colspan="2">1 ☐</td><td>2 ☒</td><td>3 ☐</td></tr>
<tr><td colspan="6">*where: SLS – Service Limit State; ULS – Ultimate Limit State; ARI – Average Recurrence Index.</td></tr>
</table>

(a) (b) (c) (d)

Figure 1: Representation of the: (a) main facade of the analyzed building; (b) right side facade; (c) left side facade; (d) rear facade

The building was built in 1952 and has experienced earthquakes over 6 degrees M_w until this moment (see details in Table 2).
The main phenomena affecting the site of the analyzed building are (see details in Table 3):

- Earthquake action;
- Snow action;
- Wind action;
- Freezing effect;
- Presence of underground water etc.

Table 2: The list of the most important earthquakes in Romania that the analyzed building went through

Date	Magnitude	Intensity	Epicenter
March 4, 1977	7.20 M_w	XI. Extreme	Nereju, Vrancea
August 30, 1986	7.10 M_w	VIII. Destructive	Gura Teghii
30 May, 1990	6.9-7.00 M_w	VIII. Destructive	Vrancea

Table 3: The main phenomena affecting the site of the analyzed building

Analysed phenomenon	Results
Seismic area [13]	*ARI = 225 years, *a_g = 0.20g and the period *T_C =0.7 sec from the design spectrum
Snow load (action) [16]	*s_k=2.5 kN/mp
Wind load (action) [17]	*q_b=0.60 kPa
Maximum freezing depth [18]	100 -110 cm deep of the natural soil
*Soil stratification	0.00 - 1.00 m – Fills from topsoil and construction material scraps; 1.00 - 7.00 m – Brown loamy sandy dust, with calcareous intercalations, with medium plasticity, plastic stout, with high compressibility
The presence of underground water	Groundwater was intercepted at -2.00 m and stabilized at -1.10 m from *L.S.L.
Plastic pressure (P_{pl}) and Critical pressure (P_{cr}) of the soil	P_{pl} =169.00 (kPa) P_{cr} =183.00 (kPa)
*where: ARI – Average Recurrence Index; a_g – horizontal ground acceleration; T_C – the period from the design spectrum; s_k – characteristic snow loading on the ground; q_b - characteristic wind pressure; Soil stratification – according to the geotechnical study; L.S.L. – Landscaped Soil Level.	

3 Identification of damages

3.1 Identification of the types of damage through the visual inspection

Following the visual investigation of the dual structural system, a series of local structural and non-structural degradations of low, medium and high importance were observed (Figure 2, Figure 3 and Table 4).

The observable Non-Structural Degradations (NSD) are represented in Figure 2 and are explained as follows:

- NSD.1 - non-structural physical degradation of mortar and masonry through the capillary rise of water, freeze-thaw effects, and mortar degradation on exterior masonry shear walls (see Figure 2 (a), (b); Table 4, Table 5);
- NSD.2 - sidewalks deterioration, defective presence of concrete sidewalks or their absence favored the appearance and development of vegetation, which facilitates the stagnation of rainwater in the foundation area (see Figure 2 (c), (d); Table 4, Table 5);
- NSD.3 - the deficient presence of gutters and downspouts or their non-existence favored the flow of rainwater on the structural masonry walls and respectively on the socle as well as on the partially destroyed sidewalk (see Figure 2 (e), (f), (g); Table 4, Table 5);
- NSD.4 - cracks and dislocations of the finishes (Table 4, Table 5);
- NSD.5 - local degradation of the ceiling and interior walls (Table 4, Table 5).

(a) (b) (c)

(d) (e)

(f) (g)

Figure 2: Representation of the non-structural degradations: (a), (b) physical degradation of mortar and masonry through the capillary rise of water, freeze-thaw effects, and mortar degradation on exterior masonry shear walls; (c), (d) The defective presence of concrete sidewalks or their absence; (e), (f), (g) the deficient presence of gutters and downspouts or their non-existence

The observable Structural Degradations (SD) are represented in Figure 3 and are explained as follows:

- SD.1 - structural degradation and micro/ macro-cracks of the external masonry shear walls (see Figure 3 (a); Table 4, Table 5);
- SD.2 - horizontal cracks in the masonry shear walls (between the gaps) caused by tensile stresses from bending moments resulting from seismic or non-seismic actions (see Figure 3 (b), (c); Table 4, Table 5);
- SD.3 - inclined cracks (at 45°) above the masonry coupling beams caused by ”vault” action of the masonry in that area (see Figure 3 (d), (e); Table 4, Table 5);
- SD.4 - cracks under the support of the beams (the crushing effect of the local area) caused by compression stress (see Figure 3 (f); Table 4, Table 5);
- SD.5 - local separation cracks between RC floor and masonry shear wall (see Figure 3 (g); Table 4, Table 5);
- SD.6 - inclined macro-cracks (at 45°) in the field (plain) of masonry shear walls caused by axial tensile stresses from seismic or non-seismic actions (see Figure 3 (h), (j), (k); Table 4, Table 5);
- SD.7 - multiple cracks in the RC slab above the ground floor from excessive deformations, vibrations and differential vertical displacements of the structure (see Figure 3 (i); Table 4, Table 5).

(a)

(b)

(c)

(d)

(e)

(f)

(g)

 

(h) (i)

 

(j) (k)

Figure 3: Representation of the structural degradations: (a) degradation and micro/ macro-cracks of the external masonry shear walls; (b), (c) horizontal cracks in the masonry jamb between the gaps; (d), (e) inclined (45°) cracks above the masonry coupling beams; (f) cracks under the support of the beams (the crushing effect of the local area); (g) local separation cracks between RC floor and masonry shear wall; (h), (j), (k) inclined (45°) macro-cracks in the field (plain) of masonry shear walls; (i) multiple cracks in the RC slab above the ground floor

Table 4 shows the Level of Degradation Importance (LDI) for structural and non-structural elements which can be Low (L), Medium (M) or High (H). In Table 5 it is represented the Level of Damage (Destruction) in Percentage (LDP) of the structural and non-structural elements in accordance with type of observable structural or non-structural degradation. The range of LDP is wide for every Type of Degradation (TD) because the lateral system dispose a multiple number of structural and non-structural elements which are affected differently. Thus, an average percentage in extensive line of the damage (destruction) was chosen.

Table 4: Level of Degradation Importance (LDI) in accordance with type of degradation

		Level of Degradation Importance (LDI)		
		Low (L)	Medium (M)	High (H)
Type of Degradation (TD)	Non-structural (NSD)	NSD.4; NSD.5	NSD.1; NSD.2; NSD.3	It is not the case
	Structural (SD)	It is not the case	SD.1; SD.2; SD.7	SD.3; SD.4; SD.5; SD.6

Table 5: The Level of Damage (Destruction) in Percentage (LDP) representation

Type of observable structural or non-structural degradation	Percentage level of damage (destruction) (%)									
	10	20	30	40	50	60	70	80	90	100
*NSD.1										
*NSD.2										
*NSD.3										
*NSD.4										
*NSD.5										
*SD.1										
*SD.2										
*SD.3										
*SD.4										
*SD.5										
*SD.6										
*SD.7										
*where: NSD – Non-Structural Degradation; SD – Structural Degradation										

3.2 Establishment the impact of the damage level to lateral stiffness of the structural elements

To check the lateral displacements at the Service Limit State (SLS) and at the Ultimate Limit State (ULS), the corrected stiffness of the component materials (concrete and masonry) $K_{SLS}{}^{c}$ and $K_{ULS}{}^{c}$ with the correction coefficient μ according to Equation 3, Equation 4 and Table 6 – which takes into account the degradation degree of structural elements before the occurrence of the seismic event. It is mentioned that the corrected lateral stiffness at both limit states is based on the stiffness calculated according to the norm P100-3/2019 [15], with Equation 1 and Equation 2, being affected by the coefficients k_{SLS} and k_{ULS} predetermined by the norm - which take/do not take into account the cracking of the material (masonry; concrete) during the seismic event.

Table 6: The establishment of the correction coefficient μ of the lateral (bending) stiffness K_{SLS} and K_{ULS} at SLS and ULS and the corrected lateral stiffness $K_{SLS}{}^{c}$ and $K_{ULS}{}^{c}$

<table>
<tr><th>Type of observable structural degradation</th><th>Type of structural element with damages</th><th>*k_{SLS}</th><th>*k_{ULS}</th><th>*μ</th><th>*K_{SLS}</th><th>*K_{ULS}</th><th>*$K_{SLS}{}^{c}$</th><th>*$K_{ULS}{}^{c}$</th></tr>
<tr><td>*SD.1</td><td>*EMSW</td><td rowspan="7">1</td><td rowspan="7">0.5</td><td>0.6</td><td rowspan="7">EI</td><td rowspan="7">0.5EI</td><td>0.6EI</td><td>0.3EI</td></tr>
<tr><td>*SD.2</td><td>*MSW</td><td>0.4</td><td>0.4EI</td><td>0.2EI</td></tr>
<tr><td>*SD.3</td><td>*MCB or *MS</td><td>0.2</td><td>0.2EI</td><td>0.1EI</td></tr>
<tr><td>*SD.4</td><td>*RCB and *MSW</td><td>0.4</td><td>0.4EI</td><td>0.2EI</td></tr>
<tr><td>*SD.5</td><td>*RCS and *MSW</td><td>0.3</td><td>0.3EI</td><td>0.15EI</td></tr>
<tr><td>*SD.6</td><td>*MSW</td><td>0.2</td><td>0.2EI</td><td>0.1EI</td></tr>
<tr><td>*SD.7</td><td>*RCS</td><td>0.5</td><td>0.5EI</td><td>0.25EI</td></tr>
<tr><td colspan="9">*where: k_{SLS} – bending stiffness (EI) correction coefficient at SLS that takes into account material (masonry; concrete) cracking during the seismic event (established by the P100-3/2019 norm [15]); k_{ULS} - bending stiffness (EI) correction coefficient at ULS that takes into account material (masonry; concrete) cracking during the seismic event (established by the P100-3/2019 norm [15]); μ - correction coefficient of the bending stiffness K_{SLS} and K_{ULS} at SLS and ULS that takes into account the degradation state of structural elements prior to the seismic event; K_{SLS} – lateral stiffness calculated with k_{SLS} coefficient at SLS in accordance with P100-3/2019 norm [15]; K_{ULS} - lateral stiffness calculated with k_{ULS} coefficient at ULS in accordance with P100-3/2019 norm [15]; $K_{SLS}{}^{c}$ – corrected lateral stiffness calculated with μ coefficient and K_{SLS} value at SLS; $K_{ULS}{}^{c}$ – corrected lateral stiffness calculated with μ coefficient and K_{ULS} value at ULS; EI – bending stiffness; SD – Structural Degradation; EMSW – External Masonry Shear Walls; MSW – Masonry Shear Walls; MCB - Masonry Coupling Beams or MS - Masonry Spandrels; RCB – Reinforced Concrete Beams; RCS – Reinforced Concrete Slabs.</td></tr>
</table>

The correction coefficient μ was established based on the information in Table 5 and Table 4 as well as on the experience of the qualified personnel who performed the in-situ inspection. Thus, it can be calculated with Equation 5 the transverse modulus of elasticity G_c for masonry and concrete related to each type of degraded structural element.
It is mentioned that the use of the coefficient μ in the checks of the lateral stiffness of the structure at SLS and ULS, implies in the numerical calculation stage, the concrete assignment of each structural element the imposed value, so that the same type of structural element positioned differently in the structure has different stiffness, so that the type of degradation is different.

$$K_{SLS} = k_{SLS} \cdot EI \tag{1}$$

where: K_{SLS} - lateral rigidity at SLS calculated in accordance with the P100-3/2019 norm [15]; k_{SLS} – bending stiffness (EI) correction coefficient at SLS that takes into account material (masonry; concrete) cracking during the seismic event; EI – bending stiffness.

$$K_{ULS} = k_{ULS} \cdot EI \tag{2}$$

where: K_{ULS} - lateral rigidity at ULS calculated in accordance with the P100-3/2019 norm [15]; k_{ULS} – bending stiffness (EI) correction coefficient at ULS that takes into account material (masonry; concrete) cracking during the seismic event; EI – bending stiffness.

$$K_{SLS}^{c} = \mu \cdot K_{SLS} \tag{3}$$

where: $K_{SLS}{}^{c}$ - corrected lateral stiffness at SLS; μ – correction coefficient of the bending stiffness K_{SLS} and K_{ULS} at SLS and ULS that takes into account the degradation state of structural elements prior to the seismic event; K_{SLS} - lateral rigidity at SLS calculated in accordance with the P100-3/2019 norm [15].

$$K_{ULS}^{c} = \mu \cdot K_{ULS} \tag{4}$$

where: $K_{ULS}{}^{c}$ - corrected lateral stiffness at ULS; μ – correction coefficient of the bending stiffness K_{SLS} and K_{ULS} at SLS and ULS that takes into account the degradation state of structural elements prior to the seismic event; K_{ULS} - lateral rigidity at ULS calculated in accordance with the P100-3/2019 norm [15].

$$G_c = 0.4 \cdot E_c \tag{5}$$

where: G_c - corrected transverse modulus of elasticity for masonry/ concrete; E_c - corrected longitudinal modulus of elasticity for masonry/ concrete (see Equation 6).

$$E_c = \mu \cdot E \tag{6}$$

where: E_c - corrected transverse modulus of elasticity for masonry/ concrete material; E - longitudinal modulus of elasticity for masonry/ concrete material.

3.3 Establishment the level of knowledge

The design values of the material characteristics of the existing structure are established according to the values of the confidence factors, CF. The confidence factor is unique to a building, characterizing the level of overall knowledge. The values of the confidence factors are chosen according to the level of knowledge achieved, as follows:

- Limited knowledge level, KL1: CF=1.35;
- Normal knowledge level, KL2: CF=1.20;
- Complete knowledge level, KL3: CF=1.00.

In the case of the analyzed structure, the limited level of knowledge KL1 and a confidence factor CF = 1.35 were chosen, since:

- the geometry of the building was established from a complete survey;
- the detailed project was established on the basis of the simulated design in accordance with the practice at the time of the building construction and on the basis of a limited inspection in the field;
- the mechanical properties of the materials were established on the basis of the values from the valid standards and on the basis of the construction practices from the period of the construction of the building as well as from limited tests in the field.

3.4 Establishment the evaluation methodology

In the case of the analyzed structural system, the Level 2 evaluation methodology was chosen because this methodology can be applied to buildings with any type of structural system, belonging to any earthquake exposure-importance class.
Thus, Level 2 evaluation methodology involves:

- the qualitative assessment of the building based on the criteria of structural configuration, composition and detailing of the constructions and the level of degradation, according to the annexes related to the type of structural system from P100-3/2019 [15];
- quantitative assessment based on a linear static structural analysis and behavior factors.

It is mentioned that the current research study addresses only the qualitative assessment of the structural system.

4 Degree of seismic configuration of the lateral system

In accordance with the Level 2 methodology, compliance with the configuration requirements specified in Table 7 for the UR Masonry structural system is verified, as per the P100-3/2019 code:

Table 7: Configuration requirements for the UR Masonry lateral system in accordance with P100-3/2019 code

No.	Criterion	*Score	Calculation
1	The quality of the structural system	5	Table 8

2	The quality of the masonry	6	-
3	The type of slabs	8	-
4	Horizontal configuration	5	Table 9
5	Vertical configuration	8	-
6	Distances between walls	3	-
7	Elements that provide lateral thrusts	10	-
8	The type of foundation soil and foundation system	7	-
9	Possible interactions with adjacent buildings	9	-
10	Non-structural elements	10	-
	*Sum ∑ =	71	**$^*R_1$= 71**
*where: Score – Points are attributed based on the achievement, partial achievement, or non - achievement of the criteria. For full compliance with the criterion, the maximum value of 10 is attributed; for partial compliance with the criterion, a score between 1 and 9 is attributed; for non-fulfillment of the criterion, a score of 0 is given; Sum – the sum of points for all criteria; R_1 – indicator correlated to the degree of seismic configuration of the lateral system.			

Table 8: Checking the density of UR Masonry walls

Level	Global direction of the structure	*$A_{m,net}$ (mm^2)	*A_{sl} (mm^2)	*p (%)	*p_{min} (%)	*Verification
Ground floor (±0.00m)	X direction	20285200	398963200	5.084479	5.5	Not OK
Ground floor (±0.00m)	Y direction	22629000	398963200	5.671951	5.5	OK
First floor (+3.30m)	X direction	26542200	398963200	6.652794	4.5	OK
First floor (+3.30m)	Y direction	21819000	398963200	5.468925	4.5	OK
*where: $A_{m,net}$ - the total net area of structural UR Masonry walls in the global X or Y direction of the structure (see Equation 7); A_{sl} - the total slab area (see Equation 8); p - density of the structural walls (see Equation 9); p_{min} - minimum density of structural walls for UR Masonry buildings with GF+1F height regime and a_g=0.20g at the Ground Floor level (±0.00m) – (see Equation 10) or First Floor level (+3.30m) – (see Equation 11) in accordance with P100-1/2013 norm [13]; Verification – see Equation 12 and Equation 13.						

$$A_{m,net} = \sum A_i \ (mm^2) \tag{7}$$

where: $A_{m,net}$ - the total net area of structural UR Masonry walls in the global X or Y direction of the structure; A_i – the cross-sectional area of each UR Masonry pier.

$$A_{sl} = L \cdot l \ (mm^2) \tag{8}$$

where: A_{sl} – the total slab area; L – floor length; l – floor width.

$$p = A_{m,net}/A_{sl} \cdot 100\ (\%) \tag{9}$$

where: p – the density of the structural walls; $A_{m,net}$ - the total net area of structural UR Masonry walls in the global X or Y direction of the structure (see Equation 7); A_{sl} – the total slab area (see Equation 8).

$$p_{min} = 5.5\ (\%) \tag{10}$$

where: p_{min} – minimum density of structural walls for UR Masonry buildings with GF+1F height regime and a_g=0.20g at the Ground Floor level (±0.00m) in accordance with P100-1/2013 norm [13].

$$p_{min} = 4.5\ (\%) \tag{11}$$

where: p_{min} – minimum density of structural walls for UR Masonry buildings with GF+1F height regime and a_g=0.20g at the First Floor level (+3.30m) in accordance with P100-1/2013 norm [13].

Density verification of the UR Masonry walls:

$$if: p > p_{min} \rightarrow \text{OK} \tag{12}$$

$$if: p < p_{min} \rightarrow \text{NOT OK} \tag{13}$$

where: p – the density of the structural walls (see Equation 9); p_{min} – minimum density of structural walls for UR Masonry buildings (see Equation 10 and Equation 11).

Table 9: Horizontal geometric compactness check

*L (m)	*B (m)	*L/B	*$(L/B)_{max}$	*Verification
35.12	11.36	3.091549	2.0	Not OK

*where: L - the dimension of the building's long side; B - the dimension of the building's short side; L/B - the ratio between long side and short side of the analyzed building; L/B_{max} - the maximum ratio between long side and short side of the analyzed building; Verification – see Equation 14 and Equation 15.

Horizontal geometric compactness verification of the structure:

$$if: L/B > (L/B)_{max} \rightarrow \text{NOT OK} \tag{14}$$

$$if: L/B < (L/B)_{max} \rightarrow \text{OK} \tag{15}$$

where: L/B – the ratio between long side and short side of the analyzed building; $(L/B)_{max}$ – the maximum ratio between long side and short side of the analyzed building.

5 Degree of structural damage

The degree of structural damage is calculated in accordance with P100-3/2019 norm [13] with Equation 16 depending on the type of material existing in the structural system. In the case of the analyzed building, masonry and concrete were considered as the fundamental types of structural material.

$$R_2 = A_h + A_v \tag{16}$$

where: R_2 – the indicator corresponding to the degree of structural damage; A_h - the score appointed to the damage state of the horizontal structural elements (see Table 10); A_v - the score appointed to the damage state of the vertical structural elements (see Table 10).

Table 10: Calculation of the degree of structural damage for horizontal and vertical elements [13]

Damage category	Vertical elements (A_v)			Horizontal elements (A_h)		
	The affected area			The affected area		
	≤ 1/3	1/3÷2/3	>2/3	≤ 1/3	1/3÷2/3	>2/3
Minor	70	70	70	30	30	30
Moderate	65	60	50	25	**20**	15
Critical	50	**45**	35	20	15	10
Highly critical	30	25	15	15	10	5

Using the information collected in situ, the following values are obtained (see Table 10): A_h = 20; A_v = 45 and $\boldsymbol{R_2}$ = 20+45 = **65**.

6 Discussion

The seismic risk determination for a certain building is done by placing it in one of the following 4 risk classes (in accordance with P100-3/2019 norm [13]):

- RsI - Seismic risk class which includes buildings with total or partial collapse susceptibility to the earthquake action corresponding to the Ultimate Limit State (ULS);
- RsII - Seismic risk class which includes buildings susceptible to major damage to the earthquake action corresponding to the Ultimate Limit State (ULS), which expose to danger the safety of users, but where total or partial collapse is improbable;
- RsIII - Seismic risk class which includes buildings susceptible to moderate damage to the earthquake action corresponding to the Ultimate Limit State (ULS), which may expose to danger the safety of users;
- RsIV - Seismic risk class which includes buildings whose expected seismic response under the effect of the earthquake, corresponding to the Ultimate Limit State (ULS), is similar to that expected seismic response for modern buildings designed with the current technical norms.

Thus, the seismic risk class is established for the R_1 and R_2 indicators based on the score obtained and based on the requirements imposed by the P100-3/2019 norm [13] (see Table 11 and Table 12). The seismic risk class in which the building has been classified, R_s, is established according to Equation 17.

Table 11: R_1 values associated with seismic risk classes [13]

Seismic risk class			
I	II	III	IV
Values			
$R_1<30$	$30\leq R_1<60$	$60\leq R_1<90$	$90\leq R_1\leq 100$

Table 12: R_2 values associated with seismic risk classes [13]

Seismic risk class			
I	II	III	IV
Values			
$R_2<50$	$50\leq R_2<70$	$70\leq R_2<90$	$90\leq R_2\leq 100$

$$R_s = min(R_1; R_2) \quad (17)$$

where: R_s – seismic risk class in which the building has been classified; R_1 - indicator correlated to the degree of seismic configuration of the lateral system; R_2 – indicator corresponding to the degree of structural damage.

The centralized information of the qualitative seismic evaluation results of the investigated dual structural system can be found in Table 13.

Table 13: The results of the qualitative seismic evaluation of the investigated dual structural system

Degree of seismic configuration of the lateral system, R_1 :	71			
The seismic risk class associated to the R_1 indicator:	III			
Degree of structural damage (affectation), R_2:	65			
The seismic risk class associated to the R_2 indicator:	II			
The seismic risk class in which the building has been classified, R_s (see Equation 17):	I ☐	II ☒	III ☐	IV☐
Ultimate Limite State check:	The existing dual system building follow the Rs II seismic risk class, after ULS examination.			
Description of the seismic risk class [13]:	Rs II - Seismic risk class which includes buildings susceptible to major damage to the earthquake action corresponding to the Ultimate Limit State (ULS), which expose to danger the safety of users, but where total or partial collapse is improbable.			
Check at Service Limit State (SLS):	It is checked the lateral displacements requirements at SLS.			

Conclusions:	According to the obtained results, intervention works on the structural system of the building are needed to increase the degree of seismic structural resistance. In these conditions, it is possible to carry out work for additional floor construction and thermal retrofitting.			
The necessity of the intervention works:	☒ Yes		☐ Not	
Seismic risk class after intervention works, R_s:	I ☐	II ☐	III ☐	IV ☒

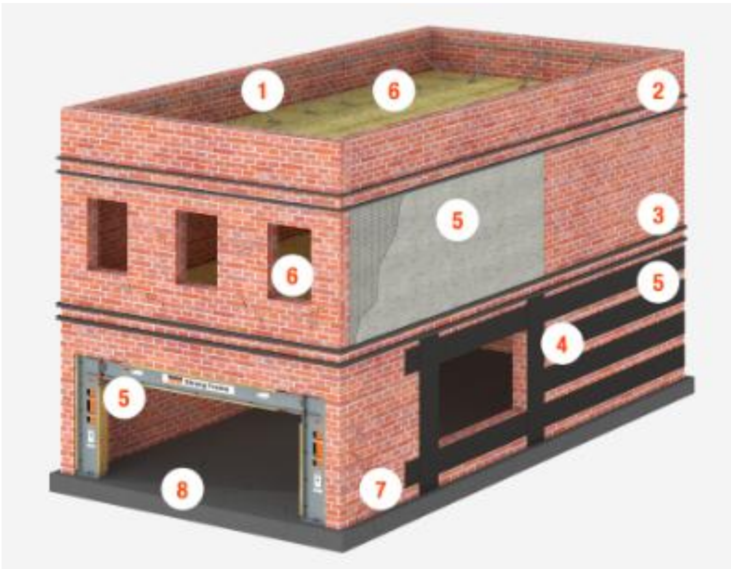

Figure 4: Representation of the retrofit applications for URM buildings [20]: (a) (1) parapet bracing; (2) wall-to-roof anchorage; (3) wall-to-floor anchorage; (4) out-of-plane wall bracing; (5) lateral shear resistance; (6) diaphragm strengthening; (7) veneer anchoring and stitching; (8) concrete foundation

7 Conclusion

The analyzed building with the dual structural system consisting of UR Masonry shear walls and Reinforced Concrete Frames requires intervention work on the lateral structure. Following retrofitting works, additional floor construction and thermal rehabilitation can also be carried out.

The basic approach to improving the seismic performance of URM building is to [19-21] (see Figure 4):

- secure all unrestrained parts that represent falling hazards to the public (e.g. chimneys, parapets and ornaments);
- improve the wall-diaphragm connections or provide alternative load paths; improve the diaphragm; and improve the performance of the face-loaded walls (gables, facades and other walls) by improving the configuration of the building and in-plane walls;
- strengthen specific structural elements, and consider adding new structural components to provide extra support for the building.

The intervention works concern the following rehabilitation techniques for analyzed UR Masonry building [19], [21] (see Figure 4):

- in-plane lateral resistance strengthening with: (a) FRP; (b) FRCM; (c) Reinforced Concrete Shear Walls; (d) Concrete or CMU Infill; (e) Steel special moment frame;
- out-of-plane wall bracing; out-of-plane wall strengthening with: (a) FRP; (b) FRCM;
- diaphragm strengthening;
- concrete foundation for new concrete shear walls.

The necessary intervention works will be executed in compliance with the architectural and structural proposals according to the technical execution project, respectively the measures from the energy audit and the installation proposals.

After the execution of the intervention works, the analyzed building falls under seismic risk class IV. It is also mentioned that the qualitative seismic assessment is a complex and laborious analysis that requires qualified personnel in the field, and an advanced knowledge of the seismic response of the building as well as the rehabilitation methods specific to the analyzed building.

References

[1] Khattak, N., Derakhshan, H., Thambiratnam, D.P., Perera, N.J. (2022). Typological characterisation of vintage unreinforced masonry buildings of Queensland, Australia. *Structures*. 41, pp. 99-116. DOI: https://doi.org/10.1016/j.istruc.2022.04.095.

[2] Khattak, N., Derakhshan, H., Ferreira, T.M., Perera, N.J., Thambiratnam, D.P. (2024). Seismic vulnerability assessment of pre-1945 unreinforced masonry buildings located in Queensland, Australia, using an index-based approach. *Structures.* 67, 106900. DOI: https://doi.org/10.1016/j.istruc.2024.106900.

[3] Buonocunto, V., Acconcia, E., Parisi, F. (2023). Seismic fragility modelling of unreinforced masonry buildings in Campania region. *Procedia Structural Integrity*. 44, pp. 67-74. DOI: https://doi.org/10.1016/j.prostr.2023.01.010.

[4] Yon, B. (2021). Identification of failure mechanisms in existing unreinforced masonry buildings in rural areas after April 4, 2019 earthquake in Turkey. *Journal of Building Engineering*. 43, 102586. DOI: https://doi.org/10.1016/j.jobe.2021.102586.

[5] Cima, V., Tomei, V., Grande, E., Imbimbo, M. (2023). Fragility curves for residential unreinforced masonry buildings prone to out-of-plane mechanisms: the case of the historical center of Sora. *Procedia Structural Integrity*. 44, pp. 211-218. DOI: https://doi.org/10.1016/j.prostr.2023.01.028.

[6] Kaya, A., Adanur, S., Bello, R. A., Genc, A. F., Okur, F. Y., Sunca, F., Gunaydin, M., Altunisik, A. C., Sevim, B. (2023). Post-earthquake damage assessments of unreinforced masonry (URM) buildings by shake table test and numerical visualization. *Engineering Failure Analysis*. 143, 106858. DOI: https://doi.org/10.1016/j.engfailanal.2022.106858.

[7] Shabani, A., Zucconi, M., Kazemian, D., Kioumarsi, M. (2023). Seismic fragility analysis of low-rise unreinforced masonry buildings subjected to near- and far-field ground motions. *Results in Engineering.* 18, 101221. DOI: https://doi.org/10.1016/j.rineng.2023.101221.

[8] Zarzour, N., Avila, M. P. S., Mercerat, E. D., Lenti, L., Oggero, M. (2023). Behavior factor estimation for seismic design of unreinforced masonry buildings. *Case Studies in Construction Materials.* 19, e02483. DOI: https://doi.org/10.1016/j.cscm.2023.e02483.

[9] Mezzina, M., Sollazzo, A., Uva, G. (2023). Torsional response of unreinforced masonry buildings. *Procedia Structural Integrity*. 44, pp. 566-573. DOI: https://doi.org/10.1016/j.prostr.2023.01.074.

[10] Bracchi, S., Rota, M., Penna, A. (2023). Comparison of refined and code-compliant modelling strategies for the seismic analysis of unreinforced masonry buildings. *Procedia Structural Integrity*. 44, pp. 442-449. DOI: https://doi.org/10.1016/j.prostr.2023.01.058.

[11] Kayirga, O. M., Altun, F. (2021). Investigation of earthquake behavior of unreinforced masonry buildings having different opening sizes: Experimental studies and numerical simulation. *Journal of Building Engineering*. 40, 102666. DOI: https://doi.org/10.1016/j.jobe.2021.102666.

[12] Guvernul României. (1997). *Hotărârea nr. 766/1997 pentru aprobarea unor regulamente privind calitatea în construcții*. Monitorul Oficial al României.

[13] Ministerul Dezvoltării Regionale și Administrației Publice (MDRAP). (2013). *Cod de proiectare seismică – Partea I – Prevederi de proiectare pentru clădiri.* Indicativ P100-1/2013. Romania.

[14] Besen, P., Boarin, P. (2023). Integrating energy retrofit with seismic upgrades to future-proof built heritage: Case studies of unreinforced masonry buildings in Aotearoa New Zealand. *Building and Environment*. 241, 110512. DOI: https://doi.org/10.1016/j.buildenv.2023.110512.

[15] Ministerul Dezvoltării Regionale și Administrației Publice (MDRAP). (2013). *Cod de proiectare seismică – Partea a III-a – Prevederi pentru evaluarea seismică a clădirilor existente.* Indicativ P100-3/2019. Romania.

[16] Ministerul Dezvoltării Regionale și Turismului (MDRT). (2012). *Cod de proiectare. Evaluarea acțiunii zăpezii asupra construcțiilor.* Indicativ CR 1-1-3/2012. Romania.

[17] Ministerul Dezvoltării Regionale și Turismului (MDRT). (2012). *Cod de proiectare. Evaluarea acțiunii vântului asupra construcțiilor.* Indicativ CR 1-1-4/2012. Romania.

[18] Institutul Român de Standardizare (IRS). (1977). *Adâncimi maxime de îngheț.* Indicativ STAS 6054-77. Romania.

[19] Federal Emergency Management Agency (FEMA). (2006). *Techniques for the Seismic Rehabilitation of Existing Buildings: FEMA 547*. U.S., Washington D.C.: U.S. Department of Homeland Security.

[20] SIMPSON Strong-Tie Company. (2024). *https://www.strongtie.com/solutions/unreinforced-masonry-seismic-retrofit/out-of-plane-wall-bracing*. Date of access: 24.09.2024

[21] Ministry of Business, Innovation and Employment and the Earthquake Commission. (2017). *Unreinforced Masonry Buildings. Part C – Detailed Seismic Assessment*. C8 – Part C. New Zealand.