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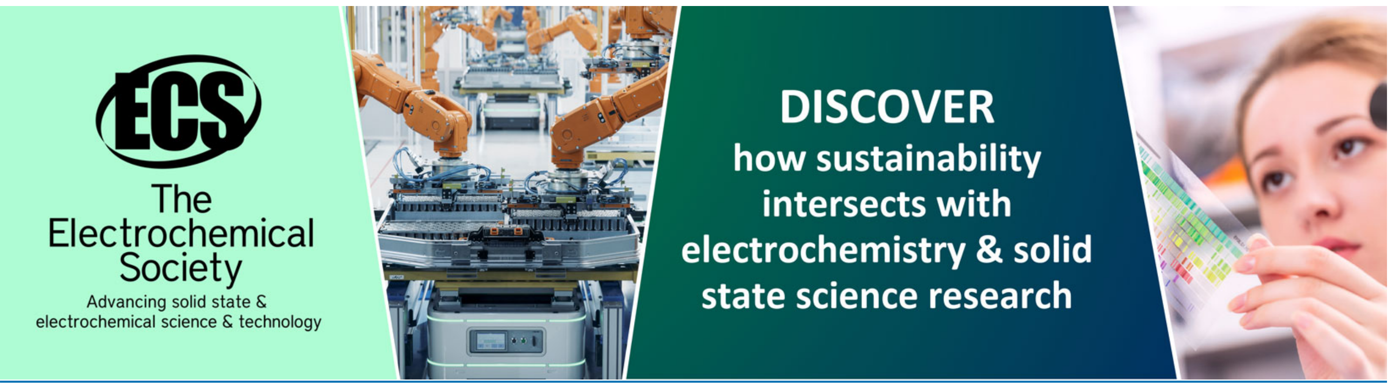

Seismic performance assessment based on displacement capacity of unreinforced masonry structures

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Abstract. The National Geological Museum is a representative building for the typology of historic unreinforced masonry structures built in Romania at the beginning of the XX century. The seismic behaviour was studied with respect to the displacement capacity, by comparing the initial structural configuration of the building with the retrofitted one. Taking into account the irregularities of the structure and the localized reinforced concrete jacketing works done in the 80s', global results were proven to be misleading. Thus, relative floor displacements were used to study the behaviour of individual walls involved in local failures. Limit state thresholds based on relative displacements were used in order to calibrate the initial model with the post-earthquake damage assessment and then they were compared to the results obtained for the retrofitted model to establish the effectiveness of the strengthening works.

1. Introduction

The global structural damage of historic masonry structures subjected to earthquakes is difficult to assess due to layout irregularities that favour the development of localized collapse mechanisms. Moreover, since construction techniques greatly influence the overall behaviour of unreinforced masonry buildings, evaluating the effectiveness of connections between structural elements might also be a difficult task.

Post-seismic damage data are important for better understanding of the structural behaviour, especially in the case of complex buildings, such as the present case study of the National Geological Museum from Bucharest, Romania. Based on the damage recorded after the 1977 Vrancea earthquake, an equivalent frame model was calibrated and its behaviour was analysed from the point of view of relative displacement capacities.

Previous research presented in depth analyses of the difference between the lateral drift capacities and the local drift capacity in case of irregular structures, such as the unreinforced masonry ones [1], [2]. For the present study case, different code provisions for drift limits and maximum displacements will be used in order to establish the damage level corresponding to the building subjected to seismic ground motions.

2. The National Geological Museum

2.1. Structural layout

The National Geological Museum from Bucharest shown in Figure 1 is a historic unreinforced masonry structure built in 1906, following the typical layout of massive palaces. It is composed of two wings with different layouts and storey heights. The secondary wing hosts the exhibition hall with more than

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10 m height that is connected to the main wing through an impressive stair case, without having a proper seismic joint in between.

Figure 1. National Geological Museum

Even though the placement of vertical structural elements allows for a regular load path, several discontinuities are also present. Firstly, the thickness of the masonry walls gradually decreases along the height, leading to 55% reductions of the total wall area, when comparing the semi-basement level to the attic level. Secondly, the openings placement on the front façade vary from one floor to another, both in terms of size and location. One of the most vulnerable structural parts is the front portico above the main entrance, where stone pillars support the attic walls. According to the damage assessment which followed the 1977 earthquake, fractures and excessive fissures were encountered for this part, causing the partial detachment of the portico.

2.2. *Retrofitting works*

In order to ensure the structural safety of the building, retrofitting works were carried out in 1984, according to the knowledge and the practices of that time. For creating proper connections between orthogonal walls, all the original flexible floors were replaced by reinforced concrete slabs. Some of the masonry walls were retrofitted by reinforced concrete jacketing to increase their in-plane strength. The walls strengthened with 20cm layers of reinforced concrete and 4 cm of reinforced mortar are marked on Figure 2 with dark green, respectively light green. Apart from these works, other local repairs were done using grout injections, partial reconstructions and also steel tie rods were added in the secondary wing, as illustrated in Figure 3.

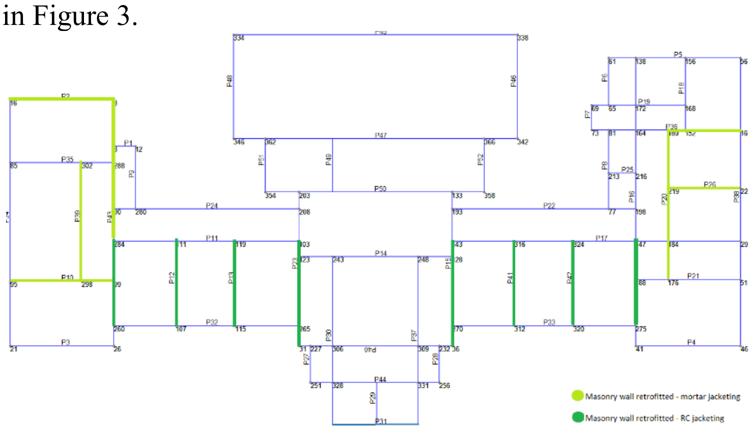

Figure 2. Masonry walls retrofitted by jacketing

Figure 3. Steel tie rods

3. Equivalent frame model

The building was modelled as equivalent frame, using Tremuri software [3]. Thus, masonry walls are meshed in spandrel and piers (deformable macro-elements) linked by rigid nodes. Since the post-damage evaluation indicated diagonal cracking as the main failure mechanism for spandrels and also horizontal fissures in case of slender piers, Mohr-Coulomb failure criteria was used in the analysis.

Considering that no tests on materials were available, the material input characteristics were considered according to the year of construction and also based on previous model calibration of similar buildings from Romania [4].

The equivalent frame model of the building presented in Figure 4 considers several simplifications related to the curved walls of the exhibition hall and also the roof which is not explicitly modelled.

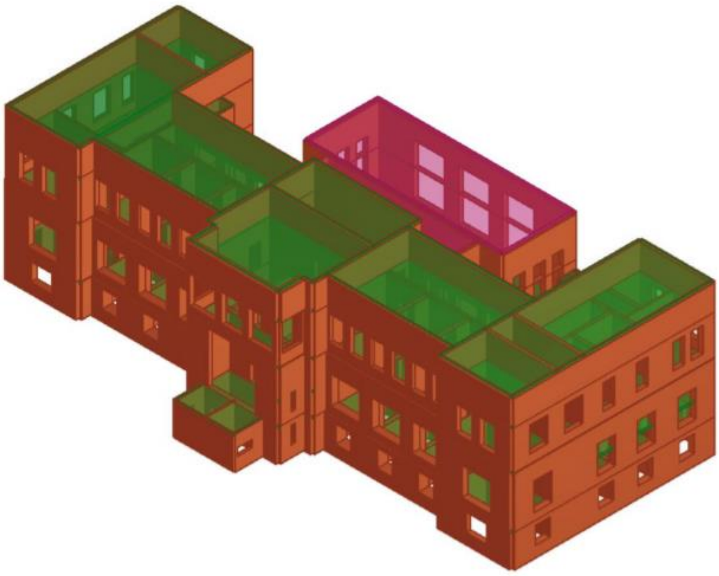

Figure 4. Tremuri model of the National Geological Museum

4. Comparative results

4.1. Nonlinear static analyses

Nonlinear static analyses were used in order to compare the displacement capacity of the historic building, considering two scenarios: initial structure and retrofitted structure. This comparison aims to study the effectiveness of the strengthening works carried out in the 80s' from the point of view of damage states associated with the relative displacement limits proposed in literature.

The results presented in Figure 5 highlight the differences between the analysis for the two orthogonal direction, as well as the differences between the initial and the retrofitted model. Due to the fact that the jacketing works were performed almost exclusively for walls along the transversal direction, the strength increase of up to 150% is recorded only for the Pushover analysis on Y direction. For the two retrofitted models marked with dotted lines, the initial stiffness indicates a similar behaviour, as a result of the rigid slabs, unlike the initial models that present different stiffness characteristics along the two orthogonal directions.

4.2. Target displacement

As it can be observed from the Pushover curves, the ultimate displacement decreases for the retrofitted model, having a larger reduction for the Y direction, where strengthening works are more significant. These results indicate that the retrofitted building is more rigid, even though resisting to larger values of maximum base shear force.

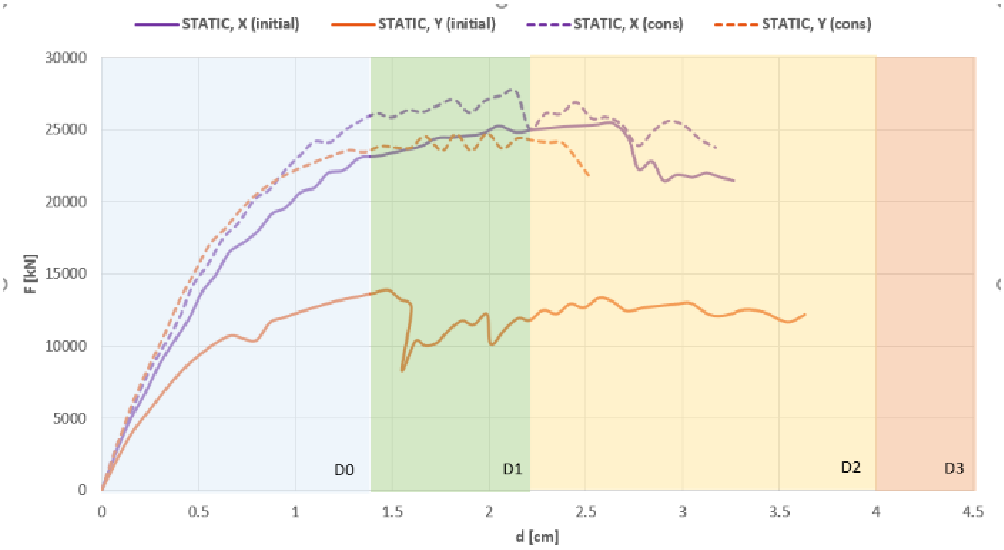

Figure 5. Pushover curves for the initial and the retrofitted model

The coefficient method present in FEMA 356 [5] and FEMA 440 [6] was used in order to evaluate the target displacement for the masonry building. The procedure assumes modifying the linear elastic response of the equivalent single degree of freedom (SFOF) system by means of several coefficients so that the maximum global displacement to be estimated, as shown in Figure 6 .

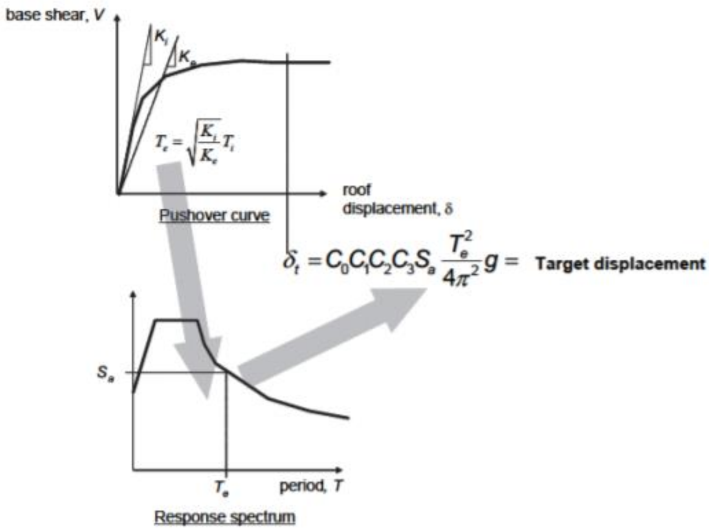

Figure 6. Estimation of target displacement δ_t [5]

The results obtained following the above-mentioned procedure are presented in Table 1, where Te is the effective fundamental period for each loading direction considered. C0 accounts for the change in spectral displacement from SDOF to MDOF, while C1 connects the maximum expected inelastic displacement and the displacements computed for the linear elastic response of the building. C2 is computed for each performance level considered (IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention) and takes into account the corresponding stiffness and strength degradations. C3 marks the influence of second order effects in the estimation of maximum displacement, namely due to geometric nonlinearities.

The target displacements thus obtained indicate a high increase in maximum expected displacement along the direction which was not retrofitted, namely from 4.7 cm (initial model) to 6.7 cm (retrofitted model) for CP. In case of the transversal direction, the difference is less, from 3.8 cm (initial model) to 4 cm (retrofitted model).

Since all target displacements corresponding to collapse prevention performance level are higher than the ultimate displacements from the Pushover curves, it can be concluded that the static nonlinear analyses were stopped before reaching the collapse state, thus indicating the presence of local failure of elements rather than global collapse.

Table 1. Target displacement estimations

		Initial model		Retrofitted model	
		X	Y	X	Y
	T_e [s]	0.31	0.28	0.36	0.29
	C0	1.35	1.35	1.35	1.35
	C1	1.43	1.44	1.41	1.44
IO	C2	1	1	1	1
LS		1.27	1.28	1.27	1.28
CP		1.46	1.47	1.45	1.46
	C3	1	1	1	1
	$S_{a(BUC)}$ [m/s^2]	7.08	6.94	7.36	6.87
IO	δ_t [cm]	3.2	2.6	4.7	2.8
LS		4.1	3.3	5.9	3.5
CP		4.7	3.8	6.7	4.0

4.3. Relative displacement

In order to compare the displacements obtained for the National Geological Museum with other literature references with respect to displacements associated to damage levels, absolute values were converted to drifts. According to FEMA provisions [5], in the case of unreinforced masonry structures, relative displacements associated to performance levels are established based on level drifts, not on global drifts. This is because of the possible local collapse mechanism that might lead to increased localized displacement for weaker floors. The FEMA description of damages that are specific for unreinforced masonry structures under each performance level (CP, LS and IO) are presented in Table 2.

There are also some results presented in literature [1], [2] obtained based on correlating the post-earthquake damage patters with the global displacements obtained from numerical models of unreinforced masonry structures from New Zeeland. The limits are lower than the ones suggested in FEMA, as it can be observed in Table 3. The damage control limit state was added in order to account also for this intermediary state characterized by extensive damage that requires significant repairs and makes the building not occupiable, but without threatening the human life [1]. The colour codes used in Table 3 for each performance level are also marked on the Pushover curves from Figure 5 thus highlighting the damage stages for each step of the static nonlinear analyses.

Table 2. Structural performance levels and damages for URM walls (non-infill) [5]

CP	Extensive cracking; face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.
LS	Extensive cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets.
IO	Minor (<1/8" width) cracking of veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.

Table 3. Drift limits for URM structures

	Performance level	Relative floor displacement limits	Relative global displacement limits
D0	Immediate Occupancy **(IO)**	0 – 0.3%	0 – 0.06%
D1	Damage Control **(DC)**	0.3% - 0.6%	0.06% - 0.1%
D2	Life safety **(LS)**	0.6% - 1%	0.1% - 0.2%
D3	Collapse Prevention **(CP)**	≥ 1%	≥ 0.2 %

According to the global drift limits presented in literature, the Pushover analyses reach an ultimate displacement corresponding to Life Safety (D2), while the Near Collapse state would be triggered by a global relative displacement of 4 cm.

For comparing the relative floor displacements presented in FEMA with the ones obtained for the National Geological Museum, walls drifts will be further on analysed. The same colour scheme was kept in order to highlight the performance level corresponding to each wall, based on its ultimate relative displacement.

Table 4 contains the results for the walls along the longitudinal direction which present the largest relative displacements, most of them located at the second level, meaning ground-floor level. Comparing the initial model to the retrofitted one, it can be observed that for this loading direction (X) there are no significant changes. Even though for the retrofitted model there are more walls that reach the Damage Control state, the redistribution of efforts among walls ensures a slight decrease in terms of relative displacements for each analyzed wall.

Table 4. Relative wall displacements (longitudinal direction)

Initial model			Retrofitted model		
Wall no.	Level	Drift [%]	Wall no.	Level	Drift [%]
44	2	0.39%	44	2	0.33%
40	2	0.38%	40	2	0.33%
3	2	0.38%	33	2	0.33%
33	2	0.37%	3	2	0.33%
32	2	0.36%	26	2	0.33%
4	2	0.36%	10	2	0.33%
10	2	0.35%	19	2	0.33%
21	2	0.34%	25	2	0.33%
14	2	0.33%	36	2	0.33%
17	2	0.32%	4	2	0.33%
26	2	0.31%	14	2	0.33%
11	2	0.31%	21	2	0.33%
25	2	0.31%	32	2	0.33%
22	2	0.30%	5	2	0.32%
36	2	0.30%	22	2	0.32%
24	2	0.29%	17	2	0.32%
50	2	0.28%	11	2	0.32%
19	2	0.27%	50	2	0.32%
35	2	0.26%	24	2	0.32%
1	2	0.26%	35	2	0.32%
2	2	0.25%	1	2	0.32%
5	2	0.21%	2	2	0.31%
			45	2	0.25%
			47	2	0.21%
			1	3	0.17%
			32	3	0.17%

Unlike the results from X direction, for the walls along the transversal direction of the initial model, there are two cases that reach the Near Collapse limit and several others that reach Life Safety limit, as it can be observed in Table 5. Considering that jacketing works were performed only on these walls, the results of the retrofitted model show reduced level of drifts up to 0.67%, slightly above the upper limit of Damage Control of 0.6%. The two walls with drifts that exceed the Near Collapse state: wall 15 and wall 28 are marked in Figure 7 with red, respectively with green. The dis-alignment of these two walls was considered as principal cause for the damaged occurred after 1977 and their important relative displacements from the numerical model at the first-floor level are in accordance with the post-damage information. Wall no. 15 also represented one of the strengthened walls during the retrofitting works from the 80s', thus reaching a lower relative drift in the retrofitted model, namely 0.46%.

Table 5. Relative wall displacements (transversal direction)

Initial model			Retrofitted model		
Wall no.	Level	Drift [%]	Wall no.	Level	Drift [%]
15	3	1.01%	38	3	0.67%
28	3	1.00%	18	3	0.66%
27	3	0.95%	20	3	0.64%
23	3	0.94%	16	3	0.61%
41	3	0.92%	6	3	0.60%
42	3	0.83%	7	3	0.59%
7	3	0.79%	8	3	0.58%
8	3	0.78%	42	3	0.56%
6	3	0.78%	41	3	0.51%
16	3	0.75%	15	3	0.46%
20	3	0.75%	28	3	0.46%
18	3	0.72%	52	2	0.26%
38	3	0.68%	46	2	0.26%
13	3	0.68%	27	3	0.33%
52	2	0.48%	23	3	0.26%
46	2	0.38%	13	3	0.26%
51	2	0.36%	51	2	0.20%
46	3	0.46%	48	2	0.19%
12	3	0.44%	12	3	0.22%
48	2	0.34%	46	3	0.20%
48	3	0.43%			
51	3	0.41%			
52	3	0.35%			
9	3	0.27%			
43	3	0.19%			
0	2	0.13%			

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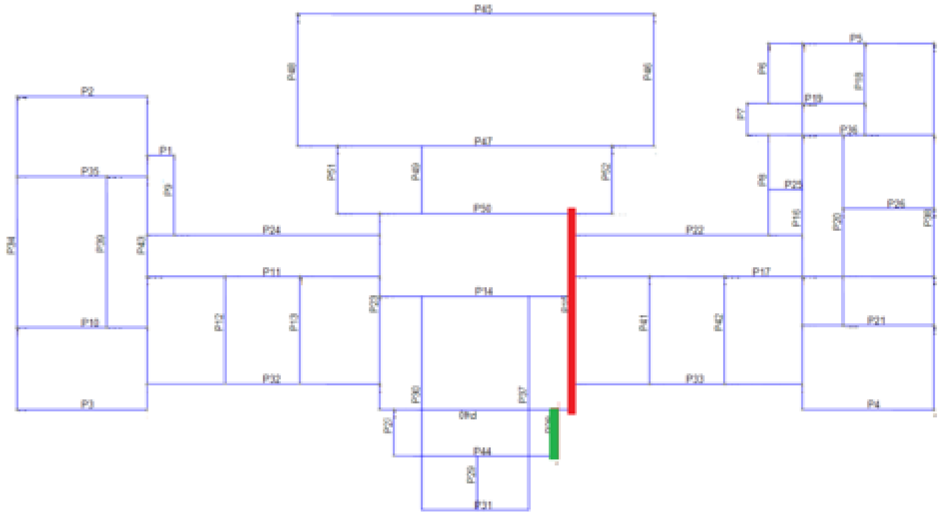


Figure 7. Position of wall 15 and wall 28

For a better understanding of the individual behaviour of wall no. 15, the damage pattern and the relative displacements along the entire height are presented in parallel for the initial and for the retrofitted model in Table 6 and Table 7. From the point of view of damage pattern it can be observed that the even though the initial model has only two elements under bending damaged (marked with pink) and the retrofitted model has most of the macro-elements under shear damage (marked in light yellow), the redistribution of efforts along the height contributes to a more uniform deformation pattern. Thus, the ultimate drift at the portico level (3rd level) is reduced, avoiding in this way local failures.

Table 6. Relative displacement and damage pattern for wall no. 15 (initial model)

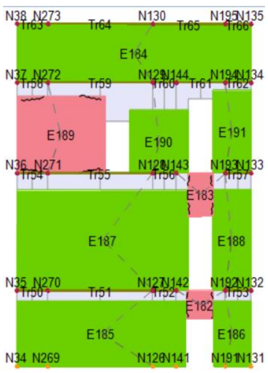
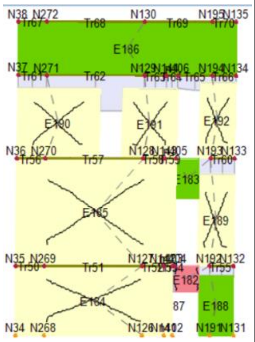
Initial model		
Damage stage of macro-elements	Ultimate displacement per floor [cm]	Ultimate drift per floor [%]
	5.57	0.01
	5.55	1.01
	0.24	0.03
	0.11	

Table 7. Relative displacement and damage pattern for wall no. 15 (retrofitted model)

Retrofitted model		
Damage stage of macro-elements	Ultimate displacement per floor [cm]	Ultimate drift per floor [%]
	2.62	0.02
	2.54	0.27
	1.13	0.15
	0.14	

5. Conclusions

The seismic assessment based on displacement capacity highlighted the importance of taking into account the local failures when evaluating the damage state based on relative ultimate displacements. Firstly, the estimation of target displacement confirmed the results of the nonlinear static analysis that arrived at a final stage before reaching the target displacement, due to local collapses. In order to understand the local collapse that lead to the end of the analysis, the relative displacements at floor level were analysed. The numerical model was thus validated, since the portico level and the connecting walls recorded the greatest relative displacements surpassing the drift threshold for near collapse state. The retrofitted model presented lower values of ultimate global displacements, but it managed to allow for forces redistribution and avoidance of local collapses due to the strengthening works. Even though results show improvements for the transversal direction which contains retrofitted walls, further works are needed in order to ensure the proper level of seismic safety for the entire building. Further studies for the building would require the evaluation of alternative less invasive retrofitting techniques, adequate for historical buildings as well as seismic risk analyses for all the structural configurations considered.

Acknowledgments
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