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Reinforcement of stone masonry walls with carbon fibre textile and tapes

Davor Uglešić[1], **Ante Uglešić**[2]

[1] *Senior Structural Designer*, D&Z doo Zadar, *dz-zadar@zd.t-com.hr*

[2] *Senior Structural Designer*, D&Z doo Zadar, *ante.uglesic@gmail.com*

Abstract

The reconstruction project of an existing masonry building will be presented in this paper. The building was erected in 1911. It has a ground floor and three floors. Its façade is protected as cultural heritage. We will show that the applied solutions meet the basic conservation - restoration requirements: less is more, reversibility and compatibility; and the structural designers: rationality, simplicity and security. Due to the conversion of the building into a hotel, many interventions have to be performed on the existing structure, such as: all the floor structures must be replaced with new reinforced concrete slabs, new greater openings in walls, and new steel roof structure design to provide another floor. This significantly changes the properties of the structure and increases the weight of the building, and thus seismic forces. For structural assessment, the project was preceded by different on site structural tests: reinforced concrete tests, stone masonry tests and structural dynamic characteristics assessment. Soil mechanics and geophysical investigations was also performed. Based on the results of previous tests, a very detailed 3D FEM model with solid elements was created on which the analysis of the structure was carried out. The results of nonlinear analysis for vertical loads, modal (eigenvalue) analysis, response spectrum analysis and push-over analysis (nonlinear construction stage analyse "sequence analysis") will be shown. Only when carbon fibres reinforcement is included in the FEM model after having applied vertical loads, it can be activated for earthquake forces. The comparison between the results for unreinforced and reinforced structure proves an increasing carrying capacity of the stone masonry walls after reinforcement with carbon fibres combined with the masonry walls grouting and deep repointing.

Key words: stone masonry, carbon fibres, cultural heritage, structural assessment, 3D FEM model, push-over

1. Introduction

The former factory Maraska building has a façade protected as cultural heritage (Fig. 1). The building has a main central section and two side asymmetrical wings. The plan dimensions are 60.5 x 10 m, west wing 15.7 x 6 m and the east wing of 12.5 x 17 m. The height of the building to the ridge is 20 m. There are the ground floor, two floors and an attic (Fig. 2). The structure was made up of several different materials. The walls are of roughly dressed stone with two faces and infill of stone fragments in the lime mortar. The foundations are concrete, while the lintels and pillars with beams on the second floor were made of reinforced concrete. Rigid reinforcement was embedded in the lintels on the ground floor. The ceiling structure above the ground floor is a shallow Prussian vault. Other inter-floor structures are wooden as well as the roof structure. The timber is decayed caused by fungi and woodworm. The floors and ceilings are damaged. The steel elements, especially in the walls, are corroded.

Figure 1. Main façade

The drafting of the hotel reconstruction project is finished. Unfortunately, the reconstruction work has not yet begun, and a near start is unlikely. Therefore, all the inter-floor structures will be replaced with new reinforced concrete slabs. Some openings on the ground floor will be enhanced, and a dozen rooms and engine rooms will be built in the attic. Several walls in the wings will be disassembled. The new roof structure will be steel. All this will significantly increase not only the vertical force but the earthquake force also.

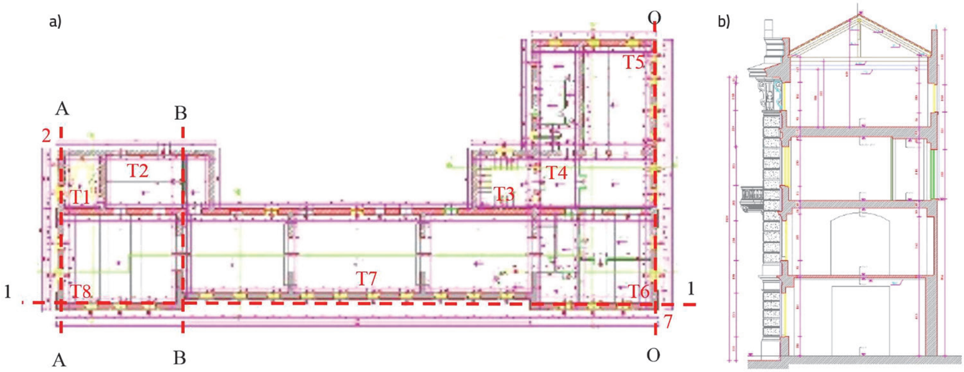

Figure 2. Existing state: a) Floor plan, b) Cross section

2 Structural Assessment

Before starting the project, extensive tests of structure and foundation soil were conducted. Soil mechanics and geophysical investigations of the wider area and foundation testing include: investigation pits, exploratory drillings, multichannel analysis of surface waves, ground penetrating radar profiles and refraction seismic profiles. Structural assessments consist of: reinforced concrete tests, stone masonry tests and structural dynamic characteristics assessment. Reinforced concrete tests include: determining the amount and position of embedded reinforcement in lintels, testing compressive strength of concrete cores (Fig. 3a) and testing compressive strength of concrete by rebound hammer. Stone masonry tests: testing the shear strength of walls (Fig. 3b), testing compressive strains (Fig. 3c) and the strength of masonry and video endoscopy (Fig. 3d). The stone masonry cohesion and angle of internal friction were estimated based on the data of shear strength and compressive stresses in the masonry. Large and numerous cavities in the walls were found by endoscopy examination, which confirmed the necessity of wall grouting. We performed measurements for the assessment of natural frequencies and damping on 32 points in 8 verticals, T1 – T8, (as shown on Fig 2a) using the TROMINO device (Fig. 3e). TROMINO is a device equipped with three orthogonal electrodynamic sensors (velocimeters). The results have been obtained through the standard software Grilla. After the motion was digitized for all the 3 motion components: the acquired signal was divided into windows, each window detrended, tapered with a Bartlett window and padded with zeros. The Fast Fourier Transformation (FFT) was then computed for each window, as well as the amplitude spectrum [1]. We needed this data for calibrating the 3D FEM model of the existing structure, comparing assessed natural frequencies with results of modal analysis.

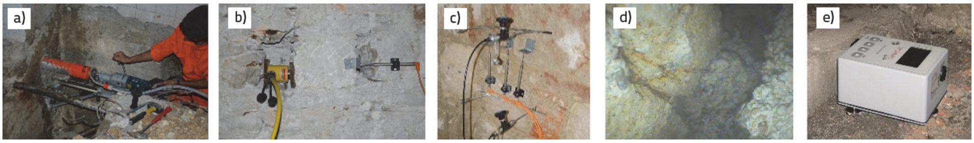

Figure 3. Structural assessments methods: a) Core drilling, b) Shear test, c) Flat jack, d) Endoscopy, e) Tromino

3 Design Solutions

Guided by the fundamental principles of conservation and restoration: less is more, reversibility and compatibility [2]; a priori we opted for the following. The existing masonry will be strengthened by grouting and deep mechanical repointing with mortars based on hydraulic lime. It will thus be possible to partly raise the capacity of the walls, and the rest needs to be compensated with carbon fibres [3]. Carbon fibres will be applied in the following manner: a one-direction textile in two orthogonal layers in the lime mortar based on hydraulic lime, vertical and horizontal strips in epoxy adhesive, and ropes in epoxy adhesive for ensuring the continuity of the strips, interconnecting wythes and preventing delamination. In situ and laboratory tests showed that the lack of connection between wythes caused the splitting breakdown of the wall in core [4, 5]. This can be prevented by wrapping walls around the range [6], which in our case was not possible. We, therefore, decided to connect the wythes with anchors [7]. Reinforcement with carbon fibres increases the carrying capacity of the walls, but does not increase the rigidity of structure, and therefore does not induce an increase in seismic forces, which was another important reason we opted for this solution. The biggest challenge was to activate the carbon fibres to participate in the takeover of horizontal forces in the 3D FEM model. During the analysis of the model, the necessity to trim enhanced openings with RC frames and RC foundations was shown (Fig. 4). Mortars based on hydraulic lime are compatible with the existing masonry, and do not generate harmful minerals ettringite and thaumasite which are incurred in enforcing the cement mortar. Carbon fibres in lime mortar can be easily removed from the wall, as well as fibres in epoxy adhesive remover using industrial fans. Reversibility is thus ensured.

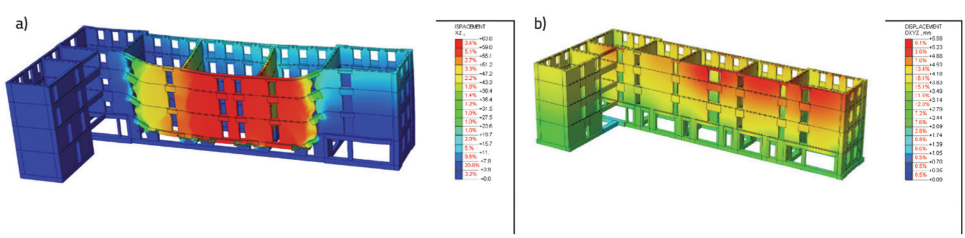

Figure 4. Axis 2 (rear façade) - Deformations of the vertical load: a) without RC frames b) with RC frames and RC foundations

4 3D FEM model

Contacts between structure and foundation soil were modelled as a surface spring elements. Stone walls and RC parts of the structure (beams, columns, floor structure plugs and foundations) were modelled as solid (tetrahedron) elements. New reinforced concrete slabs were modelled as plate elements as well as the steel roof structure with beam elements. Reinforcement in the existing and new parts of RC structure was modelled as grid reinforcement sets, in the same way as carbon fibres walls reinforcement is modelled.

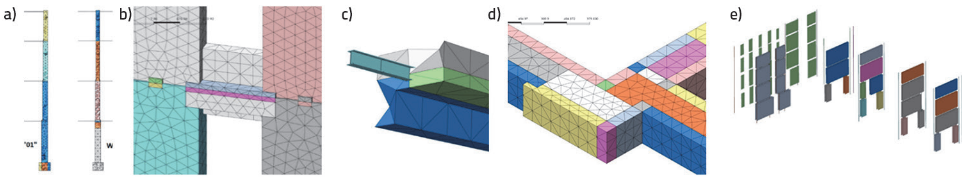

Figure 5. Modelled details: a) Walls, b) Lintels and plugs, c) Embedded INP, d) Foundations, e) Carbon fibres reinforcement

The details we paid attention to while making numerical models are: eccentricity of masonry walls per floors (the walls are asymmetrically thinner on the upper floors, Fig. 5a); existing lintel concrete beams with embedded existing reinforcements (Fig. 5b); lintel cross section is modelled like original "L" shape above the ground, first and second floors; lintels at ground floor are modelled with embedded INP steel profile (without reinforcement) as in the existing state (Fig. 5c); masonry parapet walls are thinner than the wall, with its eccentricity in relation to walls (Fig. 5b); RC plugs through which the floor slabs lean on walls; steel roof structure with its eccentricity in relation to the walls; steel reinforcement (longitudinal and stirrups) of new RC elements (foundations, columns, beams); strengthening of the existing concrete and stone masonry foundation structure and new RC side foundation strips (Fig. 5d).
The question arises as to why we worked on such a complicated, complex and large structural model. This is the second project for the same building, since the investor who determined the new project task changed, which conditioned the change of the architectural project and thus the construction project. The first model was simpler, we used beam elements, with the possibility of hinges opening in vertical and horizontal elements in push-over analyses. We concluded that the simplifications in the first model directed towards greater safety significantly reduce the load-bearing capacity of the structure. As we could not include all the specifics of the existing structure in the simple model, we decided to create the shown model (Fig. 4, 5).

2.1 Material properties

Concerning the different types of materials, material properties and different nonlinear material behaviour laws, which we used in our model, these are shown in the following tables: carbon fibres, hydraulic lime mortar and epoxy bond (Table 1); masonry and concrete (Table 2). For steel, we used Von Mises constitutive model and Tension Function (Hordijk model) "TSC". We used appropriate characteristic tensile strength, from 185 to 500 N/mm2 for different kinds of reinforcement and construction steel.

Table 1. Material properties of carbon fibres, bond and mortar

Material	**Carbon tapes TCU 800/100**	**Carbon ropes G FIOCCO φ12**	**Carbon textile GV 160 UN TFX**	**Epoxy bond**	**Hydraulic lime mortar**
Tensile strength [MPa]	4,900.0	4,900.0	4,700.0	30.0	3.1
Compressive strength [MPa]	/	/	/	/	10.6
Adhesion [MPa]	/	/	/	> 4.0	1.2
Young's modulus [GPa]	240.0	230.0	230.0	4.5	6.8
Elongation at break [%]	2.0	2.1	2.0	0.9	/
Material Constitutive Model	Von Mises		Linear	/	/

Table 2. Material properties of concrete and masonry

Material	**Old concrete (C16/20)**	**New concrete (C30/37)**	**Existent masonry**	**Grouted masonry**
Elastic Modulus [N/mm^2]	27.402	31.938	1.650	2.475
Poisson's Ratio	0.20	0.20	0.40	0.40
Weight Density [kN/m^3]	23.53	2356	20.00	20.00
Material Constitutive Model	Total Strain Crack			
Crack Model	Fixed			
Stiffness	Secant			
Lateral Crack Effect	Vecchio and Colins			
Confinement Effect	None			
Tension Function (Hordijk model) "TSC"				
F_t - characteristic tensile strength [N/mm^2]	1.581	2.912	0.120	0.240
G_{fI} - tensile fracture energy [N/mm]	0.0300	0.0500	0.0030	0.0005
h - crack band width [mm]	147	147	147	147
Compression Function (Thorenfeldt model) "TSC"				
F_c - characteristic compressive strength ([N/mm^2]	20.00	38.00	1.50	3.00
G_c - compressive fracture energy [N/mm]	0.0487351	0.0763783	0.0487351	0.0097470
h - characteristic element length [mm]	147	147	147	147
D_{max} - maximum aggregate size [mm]	16	16	/	/

2.2 Analyses and results

In the Midas FEA we made the next type of analysis: nonlinear for vertical loads (self-weight, dead load, live load); modal (eigenvalue analysis), response spectrum and push over (nonlinear construction stage analysis "sequence analysis"). Using this method of push over analysis, we activated carbon fibres for the absorption of horizontal loads. For the push over analysis, inertial forces were used and obtained from the response spectrum load case for modes (eigenvalues) which participated with more than 1.0% of the total mass. The acceleration on location peak ground (AgR) is: 0.185g for design spectrum Tp = 475 years, with base reaction 11,005 kN or 18.7% of vertical load, and 0.091g for elastic spectrum Tp = 95 years.

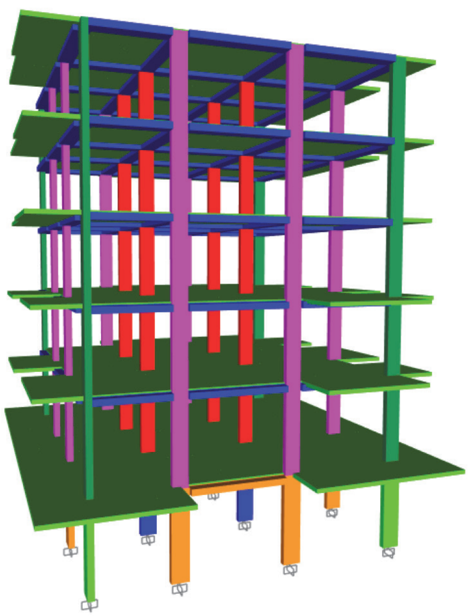

Figure 6. Results of modal analysis: a) 1st mode T=0,3989 [s], b) 2nd mode T=0,2954 [s], c) 3rd mode T=0,2531 [s], d) 4th mode T=0,1903 [s]

The results of modal analysis (Fig. 6) show that the first mode is a combination of translation in Y direction and torsion, the second mode is torsional and the third and the fourth modes are a combination of X and Y translation with torsion. The push over analysis was conducted as follows: in the first stage, vertical loads were applied in twenty steps, in the second stage carbon fibres were activated and horizontal earthquake loads were applied, also in 20 steps. The results of push over analyses show that reinforcement with carbon fibres has significantly reduced horizontal shifts and increased the carrying capacity of the walls. The overall displacements on the 10 controlled points on top of the building were reduced from an average of 84.9 mm to 20.6 whereby for 64.3 mm or

75% (Fig. 6). The biggest difference is in point 7, which amounts to 15.1 mm (83.7%) and the smallest in point 2, which amounts to 27.9 mm (62.3%). Horizontal displacements change linearly with increasing horizontal loads up to 70% of the total horizontal force. Inter-story drifts have also been reduced by an average of 67%, from 85% on the ground floor to 14% on the third floor (Fig. 7). Inter-story drifts change linearly with increasing horizontal loads up to 80% of the total horizontal force.

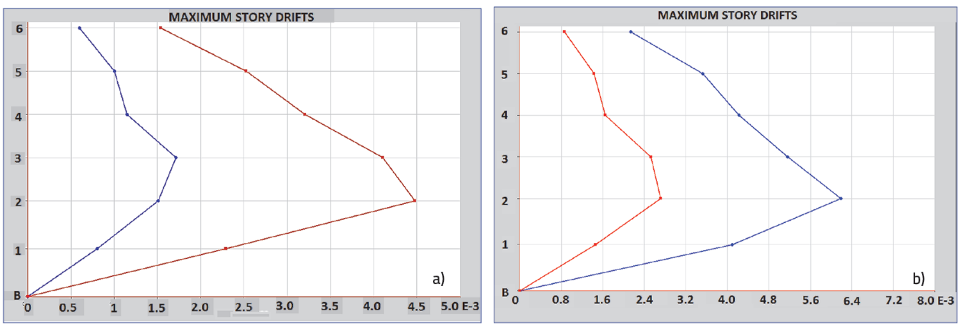

Figure 7.Horizontal displacements for earthquake -1Y-0.3X in points 2 and 7 (as shown on Figure 2a)

The comparison of the fissured state, without and with reinforced structure with carbon fibres, shows a significantly reduced number and size of cracks in reinforced structure. Efficiency is less pronounced on the walls, in line **A** (façade), which are unilaterally strengthened (Figures 8 and 9), compared to those (in line **B**) that are reinforced on both sides with carbon fibres (Figures 10 and 11). On the same figures, we can see that high tensile stresses in carbon fibres correspond with the densest cracks in the walls (Figures 8b to 8d and Figures 10b to 10d). The connection between the longitudinal and transverse walls will be ensured by stainless steel anchors. Observing the growth of the cracks at the characteristic walls (Figures 9c and 9d and Figures 11c and 11d), we notice a significant increase in cracking after reaching 60 % of the horizontal load due to the increase of horizontal loads. Due to the extremely large tensile stresses in horizontal carbon fibres on wall **A**, delamination of reinforcement can be expected.

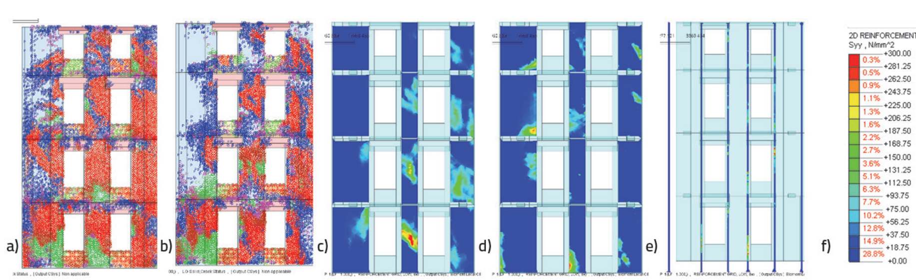

Figure 8. Line A extracted from 3D model (west façade; as shown on Fig 2a) - Cracks in wall and stresses in carbon fibres under 100 % horizontal loading - Legend for cracks as in Fig 9 f: a) Cracks without reinforcement, b) Cracks with reinforcement, c) Tensile stresses in CF horizontal direction, d) Tensile stresses in CF vertical direction, e) Tensile stresses in carbon tapes, f) Legend for tensile stresses in carbon fibers

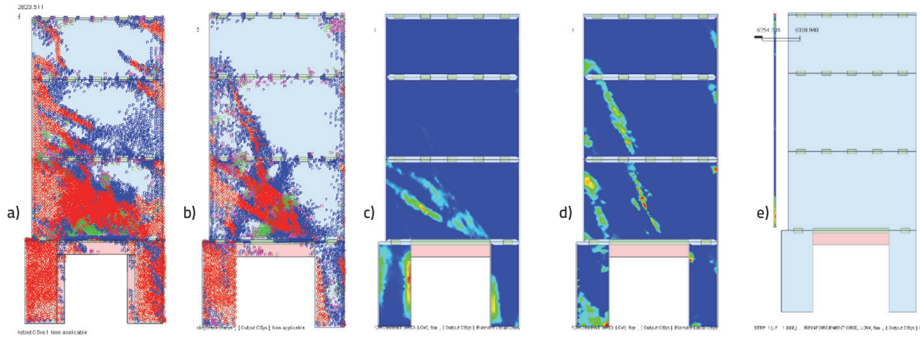

Figure 9. Line A extracted from 3D model (as shown on Fig 2a) - Cracks in reinforced wall for different intensity of horizontal loading: a) 20 % loading b) 40 % loading, c) 60 % loading, d) 80 % loading, e) 100 % loading, f) Legend for cracks in wall

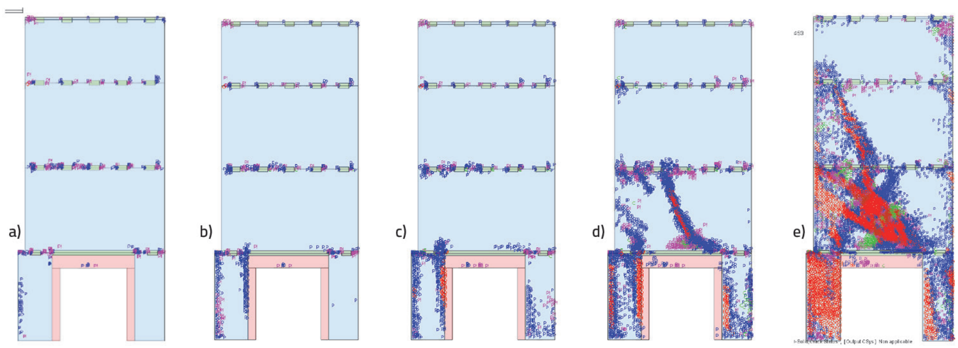

Figure 10. Line B extracted from 3D model (as shown on Fig 2a) - Cracks in wall and stresses in carbon fibres under 100 % horizontal loading - Legend for cracks as in Fig 9 f; Legend for tensile stresses as in Fig 8: a) Cracks without reinforcement, b) Cracks with reinforcement, c) Tensile stresses in CF horizontal direction, d) Tensile stresses in CF vertical direction, e) Tensile stresses in carbon tapesf

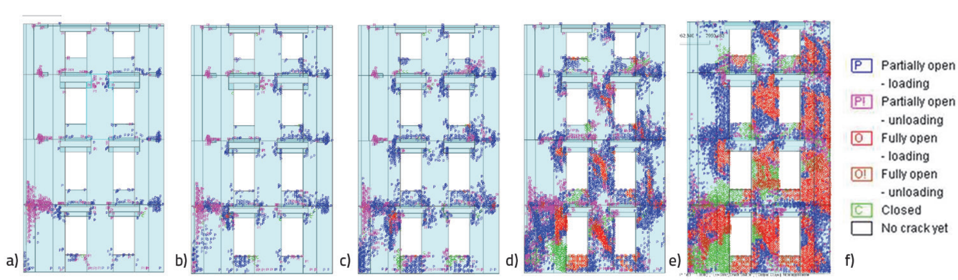

Figure 11. Line B extracted from 3D model (as shown on Fig 2a) - Cracks in reinforced wall for different intensity of horizontal loading - Legend for cracks as in Fig 9 f: a) 20 % loading., b) 40 % loading, c) 60 % loading, d) 80 % loading, e) 100 % loading

An interesting phenomenon is that in the longitudinal wall in axis **1,** more cracks appear due to earthquake in transversal (orthogonal – Y) direction (Fig. 12 a) than due to earthquake in longitudinal (X) direction (Fig. 12 b). Owing to a relatively small percentage of the walls in the Y direction, parts of the walls in the X direction are activated as flanges of **I** profile which take a large part of the tensile stresses. The effect of reinforcing transverse walls on the longitudinal walls is reflected by the difference in the image of the cracks without and with reinforcement (Figures 12d and 12e). The image of the cracks for the load in Y direction changed significantly in contrast to the load in the X direction where the differences are relatively small, relative to the unreinforced masonry.

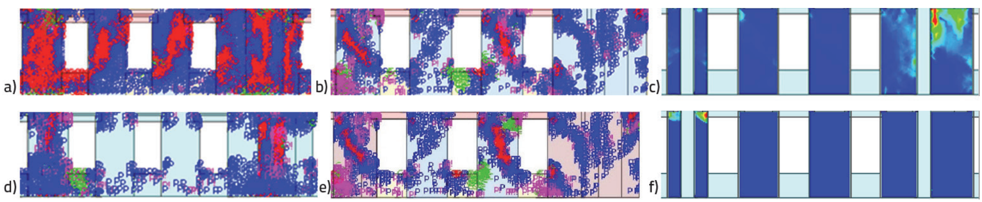

Figure 12. Line 1 (detail) extracted from 3D model (as shown on Figure 2a) - Cracks in unreinforced and reinforced wall for different directions of horizontal loading - Legend for cracks as in Fig 9 f; Legend for tensile stresses in CF as in Fig 8 f: a) 100% Y loading unreinforced, b) 100% X loading unreinforced, c) Tensile stresses in horizontal CF, d) 100% Y loading reinforced, e) 100% X loading reinforced, f) Tensile stresses in vertical CF

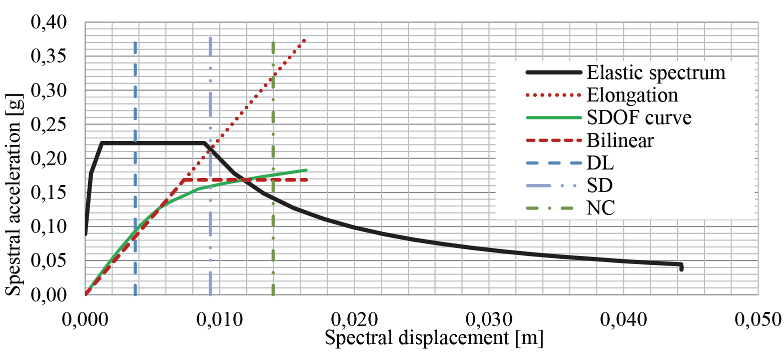

Figure 13. Spectral acceleration vs. spectral displacements

Although the tapes of carbon fibres, by stress state, were not fully utilized, we decided to install them on the outside too: horizontally at the level of floor structures and vertically at the ends of the walls, to ensure the effect of bounded walls. According to EC8 spectral displacement and spectral acceleration, values were calculated for the Limit State of Significant Damage (SD), Damage Limitation (DL) and Near Collapse (NC) [7] (Fig. 13). DL is reached for the spectral displacement of 3.7 mm and the spectral acceleration 0.0934 g. SD was reached for the displacement of 9.3 mm and 92% of the horizontal force. Structure reach NC for 13.9 mm displacement and 96% of the horizontal force. By comparing previous data, it should be noted that the appearance of

DL coincides with the steep increase of stresses in the carbon fibres. SD occurs after a significant increase in the horizontal displacement and inter story drifts.

3 Conclusion

We have shown, with push over analysis on the 3D FEM model of stone masonry structure reinforced with carbon fibres, that masonry buildings height to four stories can successfully take seismic loads in accordance with the norms. It would be good to carry out laboratory tests with which to check the numerical results we acquired. Also, during laboratory tests, it would be interesting to follow stresses change, through embedded strain gages of the carbon fibres reinforcement, mortar and anchors, on both sides and single side reinforced walls with different dispositions and densities of anchors. Based on these results, one might develop a numerical model with which to enable an analysis of the impact of the number and position of the anchors on both sides and single side reinforced walls to delamination of reinforcement and appearance of wythes delamination. The installation of a structure monitoring system, during the reconstruction of this building, could provide very useful data on the behaviour of the reinforced structure due to usual actions, as well as due to earthquakes. Monitoring should include: two sets of triaxle accelerometers, one on the foundation rock and one in the attic; strain gauges in reinforcements whether they are textile, tapes or ropes in the areas where the highest tensions increase is expected, added to the reinforcement in the new RC parts of the structure.

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