Seismic analysis and reinforcement of URM substation building

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ABSTRACT

In this paper, it is presented the seismic analysis of unreinforced masonry (URM) substation building. This building is considered as critical for the electricity supply. In terms of performance, it must remain operational after a seismic event. In this way, an appropriate methodology is adopted for the seismic analysis of this building. This methodology dedicated for existing buildings, is based on 3D detailed finite element model (FEM) combined with in-situ investigations in order to reduce the non-necessary conservatism to a reasonable level. Hence, the dynamic properties are obtained from the ambient vibration measurements, and the mechanical properties of URM from in-situ and laboratory tests. This paper presents the nonlinear seismic analysis results and the adopted seismic reinforcement solution for this building.

Keywords: URM, In-situ tests, ambient vibration, nonlinear analysis, seismic reinforcement.

INTRODUCTION

Unreinforced masonry (URM) is one of the most widely used in building construction. This is basically due to numerous advantages such as; aesthetics, heat and sound insulation, fire resistance, etc. Several Hydro-Quebec URM buildings have not been designed to sustain seismic loads and structural walls of these buildings were principally designed to resist gravity and winds loads. This means that these buildings do not possess the ability to dissipate energy through inelastic deformation in an earthquake event. Since these buildings are classified post-disaster facilities, their seismic assessment and, if required, their retrofitting is of prime importance. The aim of seismic retrofitting is to prevent two types of failure commonly observed in load bearing URM walls subjected to seismic loads. These are in-plane failure characterized by a diagonal tensile crack pattern, and out-of-plane failure, where cracks are primarily along the mortar bed joints.

Due to presence of mortar, the URM buildings belong to category of complex structures to model in view of engineering work. Indeed, the mortar is a quasi-brittle material, and its modeling depends on the desired accuracy and the scale of the considered structure (mesoscale or macroscale [1,2]). For real structures (application in engineering), the common practice is to use homogenization techniques [3] and to include the joint representing the mortar in the matrix of the structural element of masonry. This requires prior characterizing work of the homogenized properties of this matrix. Nowadays, mesoscale modelling of URM structures still generally used by researchers involved in academia, and it must be recognized that there is always a significant delay before research developments are transformed in a format suitable for industrial applications and subsequent acceptance by regulatory agencies. Furthermore, this study has to be conducted in limited time frame and within a predefined budget. Therefore, an appropriate methodology should be adopted considering these constraints.

In this paper, it is presented the seismic analysis of URM substation building. This building is considered as critical for the electricity supply. In terms of performance, it must remain operational after a seismic event. In this way, an appropriate methodology is adopted for the seismic analysis of this building. This methodology is based on 3D detailed finite element model (FEM) combined with in-situ investigations in order to reduce the non-necessary conservatism to a reasonable level. Hence, the dynamic properties are obtained from the ambient vibration measurements, and the mechanical properties of unreinforced masonry from in-situ and laboratory tests.

Similarly to concrete, URM walls behaves well under compression, but can only resists low tensile stresses with a brittle post-peak tensile behavior. This is essentially attributed to the weaker tensile properties of the mortar in comparison to those of brick or concrete blocks. Therefore, for engineering work purposes, it seems reasonable to treat the URM as a homogeneous material for which the tensile behavior is controlled by the mortar properties. This approach is based on a macro-modelling continuum model. This latter, predicts cracking and damage distributed all over the continuum.

This paper presents the nonlinear seismic analysis results and the adopted seismic reinforcement solution for this building. The nonlinear analysis is based on the Abaqus [4] Concrete Damage Plasticity "CDP" continuum model, and the homogenized masonry properties are obtained according to FEMA 356 guidelines [5] jointly with in-situ and laboratory tests. One notes that other investigators have used also the CDP continuum model for the analysis of masonry structures [6,7].

NONLINEAIR SEISMIC ANALYSIS

Background

This study is conducted following a methodology dedicated for existing buildings. In fact, it is developed to avoid major reinforcements in the structure that would be close to the cost of replacement, and especially, with a high probability of interruption of service. This is to demonstrate that the building, object of the study, is safe despite it does not comply with the current standards. In this sense, this methodology combines results of in-situ tests: (i) mechanical properties of URM as well as (Figure 1.a) (ii) the dynamic properties of the building obtained from ambient vibration measurement (Figure 1.b) with advanced analysis tools based on detailed 3D FEM and using accelerograms compatible with the uniform hazard spectra "UHS" of the site of study.

a. b.

Figure 1. In-situ tests. a. Direct shear strength test. b. Ambient vibration measurement.

It is important to mention that this methodology follows and enhances the procedure described by the document "Guidelines for the evaluation of existing buildings" of the NRC [7] by using very elaborate calculation tools. Also, a judgment on the seismic behavior is performed based on the results obtained and the recommendations of the FEMA 356 [5] guidelines. This is by stating on the level of nonlinearity of the URM structure. As shown by Figure 1.a, the elements of URM walls for this structure consist of concrete blocks and mortar.

Numerical model preparation

A search on the internal Hydro-Quebec drawing database "Logesdes" was performed to find all the necessarily engineering drawings for this study. Also, a visit to the site was very helpful to be familiar with this substation, to take pictures and to complete the missing data required for preparing the geometry of the numerical model. Hence, Figure 2 describes this geometry which was first prepared using the AutoCad software, and then transferred to FE software Abaqus (2014). As it can be seen in Figures 2 and 3, this model includes many details of the structure. The concrete basement, the URM walls, the top coronation concrete beam, the roof and the corresponding steel joists and beams. In this study one is interested to determine the damage of the URM walls. This damage is obtained from the post-elastic state of stresses. Hence, the 10 node quadratic tetrahedral isoparametric solid elements with a linear strain representation (C3D10) are used for the model (Figure 3), which is preferred to the less accurate linear 4 node tetrahedral solid element (C3D4). One major advantage of tetrahedral element is fast automatic meshing, with regards to the complexity of the structure.

In-situ URM shear strength determination

An external specialised company carried out the in-situ tests to determine the mechanical properties of the mortar and the concrete block. It is important to note that for URM, the properties of the concrete block are found to be much higher than those of the mortar. It is therefore mainly the properties of the mortar that control the structural behavior of the URM walls. In this sense, FEMA 356 [5] classified URM walls into three representative categories based on the quality of the mortar, ie "good" masonry, "fair" masonry, and "poor" masonry (see Table 1). For the shear strength of the mortar, the average value reported by experts of this company is corrected according to the recommendations of FEMA 356 (see section 7.3.2.6 and Equations 7-1 and 7-2) to take into account the favorable contribution of gravity loads to the measurements. As indicated in Table 1, the expected shear strength for the URM walls of this building is 0.47 MPa. It can be observed that the mean tests shear value is roughly 2 times greater than the FEMA good value (0.24 MPa). This demonstrates that the existing URM wall were made with a good quality of mortar, and shows the importance of conducting in-situ tests in order to reduce the conservatism to a reasonable level for the case of existing buildings.

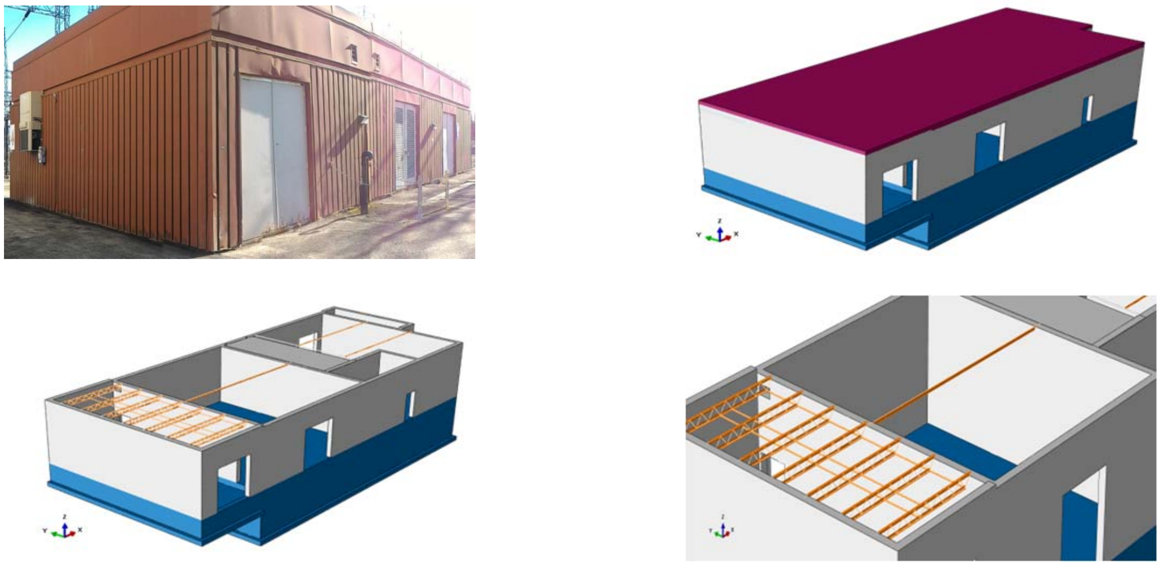


Figure 2. Geometry of the numerical model.

Figure 3. FE meshing of the numerical model.

As the nonlinear analyses are based on the Abaqus CDP continuum model, one major input for theses analyses is the tensile strength of the URM wall. In FEMA 356 (section 7.3.2.5) it is explained how to determine this strength from tests for the critical failure mode corresponding to the out of plane flexure. Since only shear tests are conducted in this study, the tensile strength is obtained by correlation with the shear strength as described in Table 1. Therefore, a conservative value is assumed by considering the tensile strength is set equal to 0.75 times the measured shear strength. This provides a value of 0.35 MPa used for the subsequent nonlinear analyses of this URM building.

Regarding the initial elastic modulus (before calibration with ambient vibration) and the expected compression strength, these values are obtained according to good masonry condition (see tables 7-1 and 7-2 of FEMA 356).

Table 1. Masonry properties.

	Masonry condition according to FEMA			
	Poor	Fair	Good	Tests
Expected tensile strength (TS)	0 MPa	0.09 MPa	0.18 MPa	0.35 MPa
Expected shear strength (ST)	0.12 MPa	0.18 MPa	0.24 MPa	0.47 MPa
Ratio (TS/ST)	0	0.5	0.75	0.75(*)

(*) Conservatively assumed

Numerical model calibration with ambient vibration measurement

The numerical model is first calibrated by adjusting the effective stiffness of structural components with ambient vibrations measurements. Then, the different load conditions are introduced in the model. As indicated in Nour et al. [9], the scope of these measurements is to determine the actual dynamic properties (fundamental frequencies and mode shapes) to be used for calibrating the numerical models and to ensure that the inertia and the stiffness of the numerical model correspond to the real structure. The ambient vibration measurements have been performed using the Granite data acquisition system manufactured by the company Kinemetrics. This system consists of a high performance portable twelve channel data acquisition system with a maximum sampling frequency of 2000 Hz. To get maximum sensitivity for each sensor, the internal jumpers were set to obtain a maximum acceleration range of ±0.25 g using a ±20 volts differential excitation. The data processing has been performed by an external specialised company using the Geopsy software. After filtering the time history records, the technique of Frequency Domain Decomposition (FDD) is used to determine dominant frequencies and the relative mode shapes for the structure. The random decrement technique is used to distinguish between the structural versus the operational modes (machinery vibration). Hence, from the numerical model, the first natural frequencies are computed and compared to results obtained from ambient vibration testing. Figure 4 shows the first natural frequencies obtained from the calibrated numerical model as well as their corresponding mode shapes.

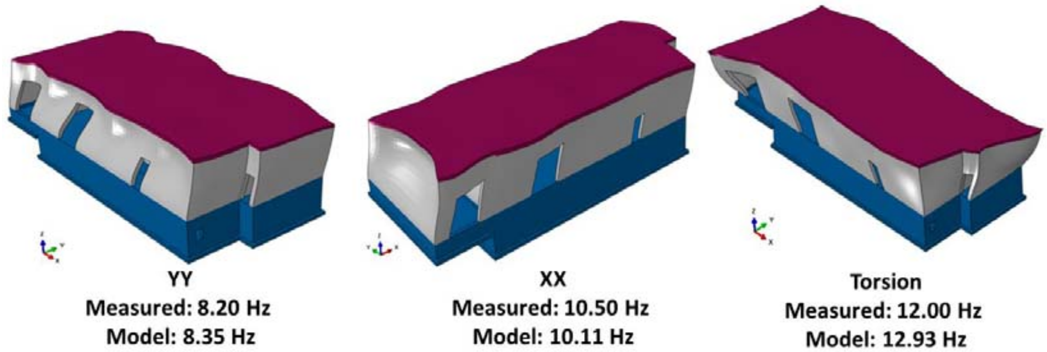
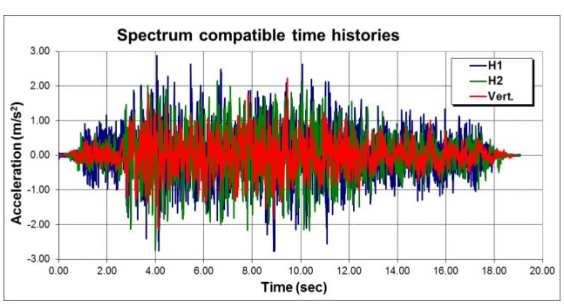


Figure 4. Calibration of the finite element model with ambient vibrations results.

Time history compatible with the UHS spectrum of the site of study

To realise this engineering work based on the detailed FEM presented in the previous section, the nonlinear seismic analyses are very time consuming. With regard the project constraints mentioned above, it was decided to consider only one broad band set of time histories compatible with the UHS spectrum, which is relatively a conservative approach in comparison to the approach based on the selection of at least 05 set of time histories in order to perform statistics [10,11]. Originally, the selected set of time histories was generated by Dr. Atkinson for Montreal at rock site [12] for Hydro-Québec, by modifying the frequency content of the Nahanni earthquake via the spectral concordance technique. The same technique is used to generate the time histories compatible with the UHS spectrum of this substation (see Figure 5) by using a computer program developed by the authors. The UHS spectrum for this substation is determined for soil category C according to the National Building Code of Canada (NBCC-2010) as indicated by the geotechnical study.



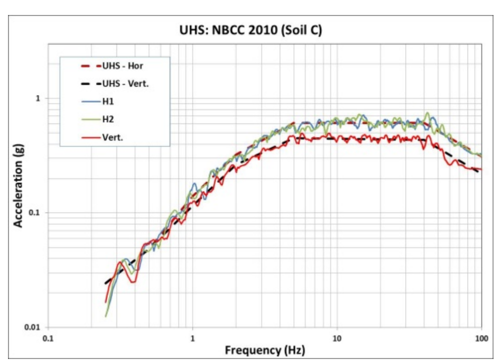


Figure 5. Spectrum compatible time histories.

This seismic action is multiplied by 1.5 because this building is considered a post-disaster facility. It should be mentioned that the vertical UHS spectrum is defined on the basis of H/V ratios as a function of frequency [12]. Furthermore, this seismic event is considered to be the maximum design earthquake. It is defined for a return period of 2500 years or for a probability of exceeding 2% every 50 years (p = 0.0004).

Analyses and results

The seismic nonlinear analyses are conducted using the concrete damaged plasticity model implemented in Abaqus. It is a smeared cracking model and it is appropriate for quasi-brittle materials subjected to cyclic loads. It uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behaviour of the material, where the two main failure mechanisms are the tensile cracking and the compressive crushing of the material. This model assumes that failure of the material can be effectively modelled using its uniaxial tension, uniaxial compression and plasticity characteristics. The uniaxial curves in tension and compression are based on the equations proposed by Feenstra and de Borst [13]. The tension softening is controlled by the fracture energy G_f and the seed size of the mesh h. The fracture energy is estimated from the empirical formula proposal by Vos [14]. The input data required for the nonlinear analyses are summarized in Table 2.

Table 2: Table 1. Adopted URM masonry properties for nonlinear analyses.

Elastic			Tension	Compression	Plasticity (CDP)				
E (MPa)	υ	γ (kg/m^3)	f_t (MPa)	f_c (MPa)	ψ	ε	f_b/f_c	K	ξ
4700	0.2	2000	0.35	8.1	38^o	0.1	1.16	0.667	0

E (MPa): Elastic modulus.
υ: Poisson ratio.
γ (kg/m^3): Density.
f_t (MPa) : Tensile strength.
f_c (MPa): Compressive strength.
ψ: Dilation angle.
ε: Flow potential eccentricity.
f_b/f_c : Ratio of biaxial compressive yield stress to uniaxial compressive yield stress.
K: Ratio of the second stress invariant on the tensile meridian to that on the compressive meridian for the yield function.
ξ: Viscosity parameter (relaxation time).

The nonlinear analyses were successfully realised on the standard solver of Abaqus using the default convergence parameters. In comparison to the explicit solver, the nonlinear analyses based on the standard solver require huge memory resources in order to inverse the equivalent system matrix in the computer RAM. The nonlinear seismic analysis lasts 14 hours on HPZ840 computer having 256 GB RAM and 44 cores.

It is important to mention that preliminary conservative calculations, based on classical engineering calculations, was realised for this building according to CSA-S304 [15] standard. These calculations show, basically, a deficiency of the capacity of several walls regarding the out of plane flexure. The presentation of the detail of these calculations is beyond the scope of this paper. The nonlinear analyses based on the detailed FEM and the measured masonry properties are judged to be more accurate and less conservative than the preliminary calculations.

Unlike linear elastic calculation, the damage of the URM walls caused by the seismic event is accumulated when using a nonlinear seismic analysis. Here, the nonlinear analyses are conducted considering the in-situ measured masonry properties. The tension damage evolution is described by means of a tension damage parameter which represents the degradation of the material properties due mainly to the propagation of cracks. The level of damage of the structure affected by the earthquake is represented by the Cracking index (Cr-I). This Cr-I is a simplification of the representation of the damage parameter displayed by Abaqus "DAMAGET", by indicating the spatial distribution of the post-elastic deformations based on two integers 0 (green color) and 1 (red color). The Cr-I is shown directly as iso-colors. Here 1 means cracked zones and 0 stands for non-cracked zones. The results of the analyses are shown for the URM envelope and partition walls in concrete blocks. Note that the values shown by the red color indicate areas of excessive cracking. Based on the results shown in Figure 6, and from a structural behavior point of view, the URM walls for this building exhibit overall a moderate nonlinear behavior under the effect of the seismic action. Therefore, in order to reduce the post-elastic deformations, under a seismic event for this building, it was decided to enhance the capacity of the URM walls by proceeding to a seismic reinforcement of this building, which is presented in the next section.

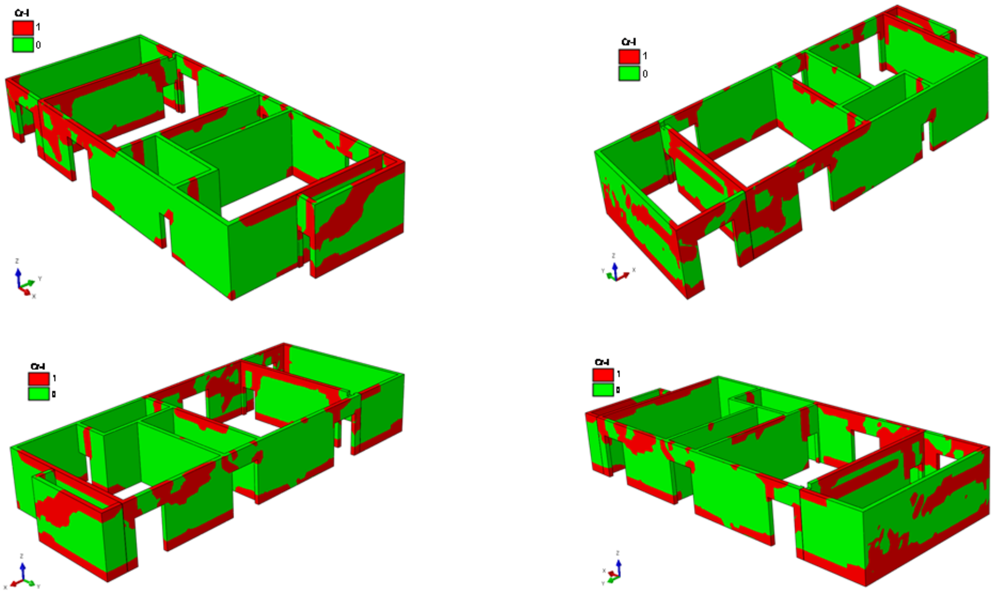


Figure 6. Cracking zone (in red) of the URM walls.

SEISMIC REINFORCEMENT

As stated in the previous section, it was decided to seismically reinforce the URM walls for this building in order to reduce the post-elastic deformations under a seismic event. It is worthy to mention that substation building reinforcement constitutes a huge engineering challenge. In fact, many reinforcement solutions are available in practice. These solutions could be easily implemented in regular residential buildings, schools, etc., but they could not be feasible for the case of critical substation buildings in operation. For this building, several avenues were prospected and proposed. However, based on the coordination with other disciplines (mechanical, electric, environmental, etc.), it was concluded that the solution of filling the concrete cells with grout (mortar) and adding a reinforcement bar to the URM walls would be appropriate for this substation building. This solution is widely used in practice to reinforce structural and non-structural URM walls. This solution allows considerably enhancing the out of plane flexure performance of the wall, as the steel bar is anchored at the bottom of the wall on the existing concrete slab, and at the top to the existing coronation concrete beam (see Figure 7).

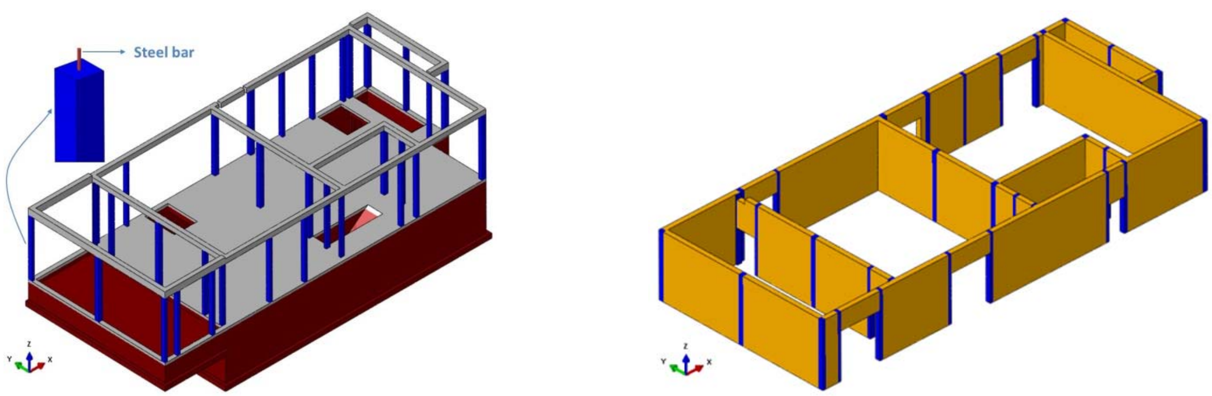


Figure 7. Seismic reinforcement solution.

Ideally is to perform a reel scale shaking table test in order to state on the performance of the proposed solution. Because this is not feasible, then the use of nonlinear seismic analyses based on detailed FEM is an interesting avenue and very helpful to understand the structural behavior of the reinforced structure or to proceed to the optimisation of the proposed solution. To the author’s knowledge, it is the first time that this technique of reinforcement is implemented in an elaborate FEM model to check its validity.

The initial number and the position of the filled cells were determined according to a conservative calculation according to the CSA-S304 [15] standard. This number was optimised based on the nonlinear seismic analyses. The final position of the filled cells is presented on Figure 7, and considers the numerous constraints raised by the other disciplines. For this project, the cells were filled using a commercial grout (mortar) that allows developing at least 40 MPa in compression. Furthermore, a M15 reinforced steel bar was added for each considered filled cell (see Figure 7).

The nonlinear seismic analyses based on the FEM with these reinforcements were realised considering a tensile strength of 2.5 MPa for the grout and 400 MPa as a yielding stress for the steel bar. The grout is modelled as solid element using the CDP of Abaqus, whereas the steel bar is modelled as truss element embedded in the grout mass and follows as elastoplastic behavior. The tension stiffening was neglected between the steel bar and the grout. The obtained results are shown in Figure 8. It is well illustrated that the post-elastic deformations are significantly reduced leading to the improvement of the behavior of the URM for this building under the seismic action, and demonstrating the efficiency of the proposed solution. However, with this solution, the post-elastic deformations (cracks) cannot be totally eliminated. Indeed, the observed cracks are judged to not compromise the integrity of the URM walls.

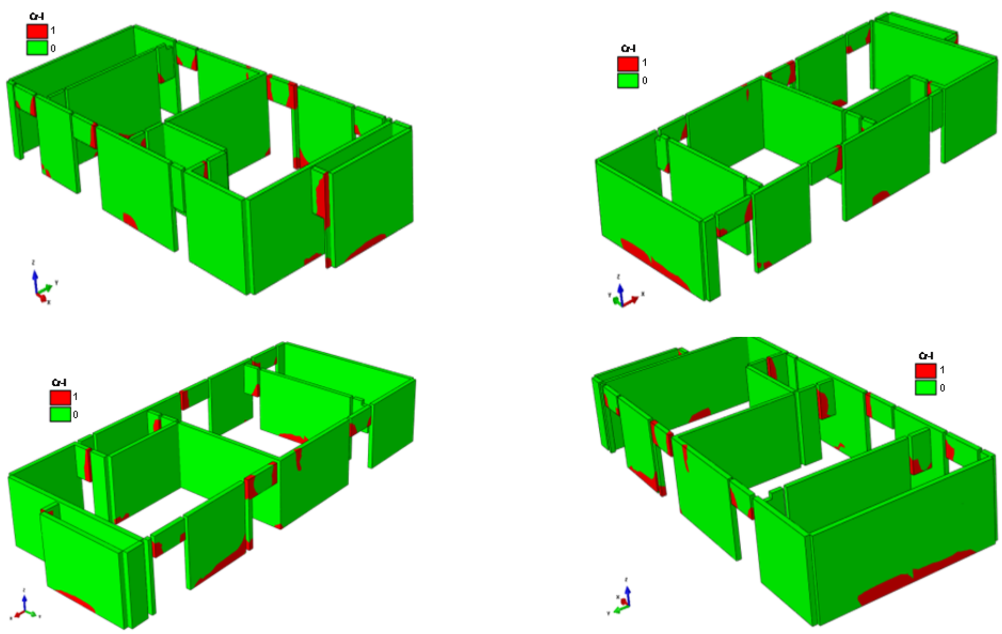


Figure 8. Cracking zone (in red) of the masonry wall after reinforcement.

CONCLUSIONS

In this paper, it is presented the seismic analysis of URM substation building. The nonlinear analyses are based on an elaborated FEM using the CDP model of Abaqus. The results of the analyses make in evidence that the URM walls for this building exhibit overall a moderate nonlinear behavior under the effect of the seismic action. Therefore, in order to reduce the post-elastic deformations (cracks) for the URM walls, it was decided to proceed to a seismic reinforcement. In this study, the reinforcement solution is presented and its efficiency is demonstrated. This solution consists of filling the concrete block cells with grout (mortar) and adding a steel bar reinforcement to the URM walls. In fact, with the proposed solution the post-elastic deformations are significantly reduced leading to the improvement of the behavior of the URM for this building.

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