Seismic Analysis of Existing Masonry Structures Reinforced with "SYSTEM DC90" Dampers

V. Gocevski
Hydro-Quebec, Montreal, Canada

Z. Petraskovic
System DC90, Belgrade, Serbia

ABSTRACT:
The paper presents the results of seismic analyses of unreinforced brick masonry walls of a two storey building located in Montreal, Canada. The analysis incorporates a continuum formulation in which the anisotropic properties of the masonry are described using the critical plane approach. First, the specification of material functions/parameters is addressed. The approach involves numerical simulations of representative elementary volume (REV) using a mesoscale approach, which accounts for the onset and propagation of localized deformation. A general methodology is outlined and the results are compared with the available experimental data. The second part of this paper deals with the masonry building. A series of dynamic analyses have been conducted that include a study of the impact of seismic retrofit of the masonry walls on their overall stability. A simplified method using COSMOS/M and SAP2000 software's is also presented and the effect of using "System DC90" dampers on the overhaul structural behavior is examined.

Keywords: *building, unreinforced, masonry, nonlinear, seismic, analysis*

1. INTRODUCTION

Assessment of the mechanical response of existing masonry buildings exposed to seismic loading is a complex and challenging task (Gocevski, 2008). This applies also for the recently proposed design of masonry structures constructed without reinforced concrete shear walls and/or frames and reinforced with steel bracings combined with system of energy dissipating dampers (Petraskovic 2010). An earthquake can have a devastating effect in particular on unreinforced masonry structures. It is therefore desirable to design an adequate reinforcement to enhance their seismic resistance. The analytical methods proposed by structural engineers are often based on simplistic numerical procedures which cannot realistically address the seismic response of existing masonry structures or the new proposed systems of masonry structures without reinforced concrete walls or frames. This is primarily due to the fact that masonry is a complex composite material, which is anisotropic on the macro scale and has a large number of possible modes of failure. The mechanical response is further complicated due to variability in the mechanical properties of its constituents (i.e. bricks and mortar) as well as in the quality of workmanship. Thus, a rational approach to the problem should incorporate advanced nonlinear formulations that account for the diversity of mechanical characteristics.

The use of steel bracings with dampers capable to dissipate energy without allowing large deformations, such as the dampers "SYSTEM DC90", complicates further the numerical procedure. The energy dissipating system in cases like this combines the existing masonry walls and the newly added steel bracings with dampers. The responses of these two systems are governed with entirely different yield/failure criterion.

- The dampers follow the plastic flow rule of uniaxial compression/tension response of homogenous material (steel).
- The behaviour of both constituents, i.e. bricks and mortar, is assumed to be elastic-brittle in tension regime, while for compressive stress trajectories, plastic-brittle characteristics are

employed. Thus, in tension domain the yield and failure surfaces coincide with each other. However, in compression regime a distinct yield surface is introduced a priori, whose evolution is attributed to accumulated plastic distortions.

The issue of seismic retrofit is particularly relevant to the buildings designed to ensure public safety, which are located in seismic zones of southern and eastern Quebec. These are structures of strategic importance and their analysis requires an appropriate methodology which goes beyond the present engineering practice. The work presented here is focused on the analysis of masonry walls of two story public safety building equipped with important instruments that must continue operating after an earthquake occurs. It is located in Montreal, Canada. The building was built in 1947 and, over the last few decades, it has suffered a minor damage due to uneven thermal expansion and contraction. As required by the National Building Code (NBC) of Canada any building constructed before 1970 must be evaluated under the NBC specified seismic loading. The building has non-bearing double layer masonry/concrete block walls that serve as enclosures. Since the main operational equipment is attached to the walls that must not collapse, the primary interest of this study is the assessment of the stability of these walls and the entire building under a seismic excitation.
The behavior of masonry structures, such as the building analyzed here, should be examined by employing a macro-level formulation. The number of elements in a nonlinear dynamic analysis has a limit in order to execute the calculation in reasonably acceptable time. Therefore modeling each constituent (i.e. bricks and mortar) using a meso-scale approach is impossible for large models. In this paper, the masonry is described as a continuum whose average properties are identified at the level of constituents taking into account their geometric arrangement.

Over the last decade, a number of different approximations have been developed for assessing the homogenized properties of structural masonry. Those include, among others, the micropolar Cosserat continuum models (e.g. Sulem & Muhlhaus, 1997) and theory of homogenization for periodic media (e.g. Anthoine, 1997).

At the macro-level, a significant work has been undertaken with regards to the development of phenomenologicaly-based failure criteria for structural masonry. Examples include the studies of Lourenço et al. (1998) and Ushaksaraei & Pietruszczak (2002).

This paper consists of two main parts. The first one deals with the meso-scale approach and its application in studying the mechanical characteristics of structural masonry. Here, the results of numerical simulations are provided for full-size brick masonry panels subjected to various loading histories. The objective is to derive the macro-scale characteristics of masonry form the properties of their constituents. Various methodologies are reviewed which include predictions based on strain-hardening plasticity that address both pre- and post-localization behaviour, elasto-perfect plasticity as well as limit analysis. In part two, this methodology is applied to identify the material functions/parameters of a continuum formulation using the properties representative of the masonry structure. In addition a simplified dynamic analysis of the structure incorporating equivalent Link elements replacing the full-size brick masonry panels is presented. The results (hysteresis of shear at the base to horizontal displacement at the top) for six (6) representative full-size masonry panels subjected to dynamic loading was used for defining equivalent Link elements (as defined by SAP2000 computer program). The building reinforced with steel bracings combined with System DC90 dampers was extensively analyzed. The main objective is to examine the stability of the unreinforced masonry walls of the structure under seismic excitation typical for the region and to evaluate the proposed refurbishing strategy and the requirements of adequate energy dissipating dampers.

2. MODELLING OF THE RESPONSE OF STRUCTURAL MASONRY; MESO-SCALE APPROACH

Assessment of the mechanical response of structural masonry is a complex and challenging task. Apparently, the most direct approach involves the experimental testing of masonry panels. One of the

most comprehensive set of experimental data for in-plane loading of masonry at various angles of bed joint is that obtained by Page (1981, 1983). The experimental data of Page have already been used for partial verification and validation of various constitutive theories. One of the major problems associated with using experimental data for the purpose of a numerical analysis of boundary value problems is that of incompleteness of data. Most investigators determine material parameters which they think may be relevant. This may not be in accordance with the requirements of a specific framework, which may entail the use of a different set of parameters. Thus, it is unlikely that all material data would be available from a single set of experiments on a specific type of masonry and additional tests may have to be conducted. This indeed is not only expensive but also time consuming.
A pragmatic alternative to experimental testing is the use of numerical/analytical tools to predict the response of structural masonry based on properties of constituents, which can be identified from standard material tests. Such an approach is more flexible in terms of providing the information for specification of material parameters in a macro-scale approach. In what follows, a methodology is reviewed which is primarily based on a numerical homogenization.

2.1. Outline of the formulation

The behavior of both constituents, i.e. bricks and mortar, is assumed to be elastic-brittle in tension regime, while for compressive stress trajectories, plastic-brittle characteristics are employed. Thus, in tension domain the yield and failure surfaces coincide with each other. However, in compression regime a distinct yield surface is introduced a priori, whose evolution is attributed to accumulated plastic distortions. Thus,

$$f_1 = \sigma_1 - \sigma_0 = 0; \qquad f_2 = \frac{\bar{\sigma}}{g(\theta)} + \eta(\xi)\sigma_m - \mu = 0 \tag{1}$$

In Eq.**Error! Reference source not found.**, σ_m, $\bar{\sigma}$ and θ represent a set of invariant measures of the stress tensor, σ_0 is the tensile strength of the material and ξ is an internal variable whose evolution is a function of deviatoric plastic strain history, i.e. $\dot{\xi} \propto \dot{e}_{ij}^{p}\dot{e}_{ij}^{p}$. Moreover, $\eta(\xi)$ is a monotonically increasing variable and for $\xi \to \infty$ there is $\eta \to \eta_f$, where η_f refers to the conditions at failure.

Prior to the onset of localization, the deformation characteristics in compression domain are described by employing a non-associated flow rule (Shieh-Beygi & Pietruszczak, 2008). The loading/unloading criteria are established using the classical Kuhn–Tucker conditions and the elastoplastic operator is obtained following the standard plasticity procedure, i.e. invoking consistency condition and the additivity postulate.

The conditions at failure are said to be associated with formation of a macro crack, whose direction is consistent with either Rankine's or Mohr-Coulomb representation, viz. Eq.**Error! Reference source not found.**. In the post-localized range, a simple volume averaging procedure is employed based on the work reported by Pietruszczak (1999). The procedure incorporates the stress/strain rate averaging and the deformation within the fractured zone is defined in terms of velocity discontinuities across the interface. Detailed description of the formulation is presented in the papers by Pietruszczak, S.,& Gocevski, V., (2009) and Gocevski, V., (2008).

2.2. Simulation of tests conducted by Page

In order to illustrate the methodologies for specification of material characteristics of masonry, the experimental tests conducted by Page (1981, 1983) are considered. The tested specimens consisted of square 360×360mm panels with half-scale bricks. The samples were subjected to a series of biaxial load-controlled tests that were conducted at five different orientations of the bed joints, viz. 0°, 22.5°, 45°, 67.5°, and 90°. The results from all orientations were then collected to obtain a comprehensive picture of the directional strength characteristics of brick masonry.

The key results are given in Figs.1b and 1c, which present the directional strength characteristics. It is seen from Fig.1b that the experimental data is quite scattered; the numerical predictions, however, are in a fairly good agreement with the respective mean values. The failure modes corresponding to different configurations are quite diversified and the details, once again, are discussed in the original article.

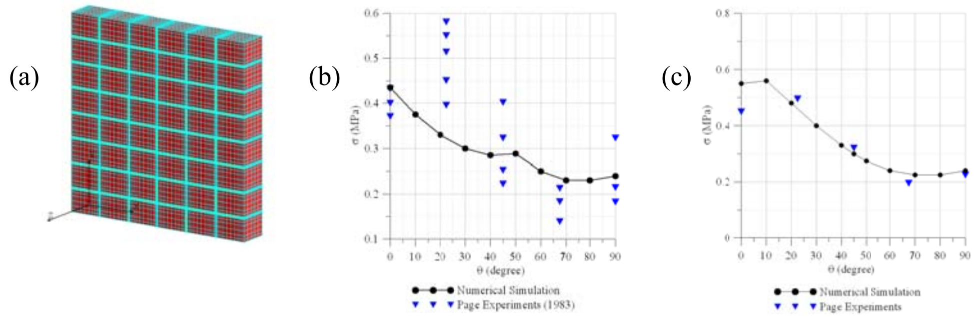


Figure 1. Numerical simulations of panels tested by Page (1983); (a) FE discretization; (b) & (c) failure envelopes for uniaxial tension and biaxial compression-tension (σ_c/σ_t=1.0), respectively

An important aspect of the solution is the periodicity of stress/displacement field for any given orientation. As an example, Fig.2 shows the distribution of the principal stresses for θ=10°. It is evident here that the stress field is periodic within the entire domain, except for the regions adjacent to the boundaries. The notion of periodicity can be exploited by introducing an approach based on numerical homogenization. Thus, rather than considering the entire panel, the boundary value problem can be solved over a suitably chosen Representative Elementary Volume (REV), subjected to periodic boundary conditions. As an illustration of this approach, the key results reported by Shieh-Beygi & Pietruszczak (2008) are provided here. The adopted REV was discretized using approx. 3800 8-noded solid elements, Fig.2c. Again, a load-controlled scheme was used in the analysis and the ultimate load was identified with the onset of global instability.

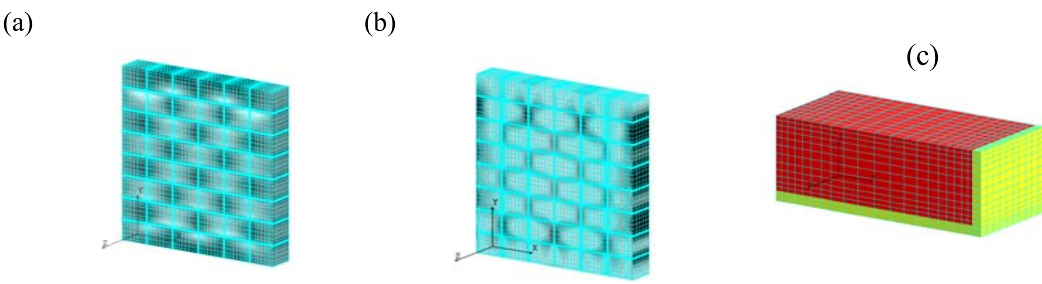


Figure 2. The distribution pattern of (a) maximum principal stress; (b) minimum principal stress in biaxial tension-compression $\theta = 10^0$; (c) REV and its finite element discretization

Fig.3 shows the strength characteristics for different orientations of the bed joints. It can be seen that the results for RVE are close to those obtained from the full-scale tests. The difference stems mainly from the influence of boundary conditions that affect the local stress/strain fields. Apparently, the difference between the full-scale simulations and the REV will decrease by increasing the size of the test panels.

As mentioned earlier, the methodology outlined above can be employed to generate the data on the directional strength properties of masonry. This information can then be used for the purpose of identification of material functions that appear in a macroscopic formulation of the problem.

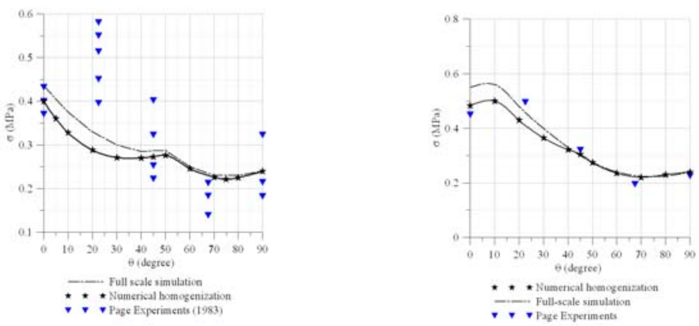


Figure 3. The distribution pattern Directional strength characteristic for uniaxial tension and biaxial compression-tension; comparison between the results for the full-scale and REV simulations

3. MACROSCALE APPROACH; SPECIFICATION OF MATERIAL FUNCTIONS

The numerical simulations for the masonry structures have been conducted by incorporating a continuum approach. The mathematical framework employs an elastic-brittle idealization. The argument here is that under a seismic excitation the predominant failure mode is a tensile fracture, which is of a brittle nature. The onset of localization and the orientation of the failure plane are derived from a functional form of a macroscopic failure criterion, by solving the constrained optimization problem. The response in the post-localized regime is modeled by employing the volume averaging procedure.

In formulating the problem, the conditions at failure at the macro-level are defined following the framework based on the critical plane approach. In particular, a linear Coulomb failure function with a cut-off in tension domain is adopted, which is analogous to representation Eq. **Error! Reference source not found.** used at the meso-level, i.e.

$$F_1 = \sigma_n - \sigma_0; \qquad F_2 = \tau + \sigma_n \tan(\phi) - c \tag{2}$$

where $\tau = \sigma_{ij} n_i s_j$; $\sigma_n = \sigma_{ij} n_i n_j$ are the shear and normal components of the traction vector on the plane with unit normal n_i and $s_i n_i = 0$. In Eq.(2), the material parameters σ_0, ϕ and c are defined in terms of orientation-dependent functions

$$\begin{aligned}
\sigma_0 &= \sigma_{01}(1+\Omega_{ij}^{\sigma_0} n_i n_j) + \sigma_{02}(\Omega_{ij}^{\sigma_0} n_i n_j)^2 + \sigma_{03}(\Omega_{ij}^{\sigma_0} n_i n_j)^3 + \ldots\ldots. \\
\phi &= \phi_1(1+\Omega_{ij}^{\phi} n_i n_j) + \phi_2(\Omega_{ij}^{\phi} n_i n_j)^2 + \phi_3(\Omega_{ij}^{\phi} n_i n_j)^3 + \ldots\ldots. \\
c &= c_1(1+\Omega_{ij}^{c} n_i n_j) + c_2(\Omega_{ij}^{c} n_i n_j)^2 + c_3(\Omega_{ij}^{c} n_i n_j)^3 + \ldots\ldots.
\end{aligned} \tag{3}$$

Here, the parameters $\sigma_{01} \ldots, \phi_1, \ldots c_1$ are the distribution coefficients and Ω's are symmetric traceless tensors which describe the bias in the spatial variation of the strength parameters. The orientation of the localization plane can be determined by maximizing the failure functions F_1 and F_2, Eq.(2), with respect to n_i and s_i, subject to constraints $n_i n_i = 1$; $s_i s_i = 1$; $n_i s_i = 0$. For the given orientation, the conditions at failure correspond to $\max\{F_1, F_2\}=0$.

4. NUMERICAL ANALYSIS OF THE BRICK MASONRY STRUCTURE

The building comprises of a basement, a ground floor and one floor above ground. It is composed of reinforced concrete frames and slabs. Unreinforced two layers non-bearing masonry walls serve as enclosures. The outside layer is continuously constructed of bricks for the entire height of the building while the internal layer build of bricks (bottom half of the story height) and concrete blocks (upper half of the storey height) is incorporated as infield panels between the columns. The laterally unsupported height of the walls is the entire storey height of 6.5 m. The outside view, the geometry of the building and the finite element (FE) model employed in the non-linear dynamic analysis using the above described procedure are presented in Fig. 4. The results of the elaborated analysis are shown in Fig. 4 as well.

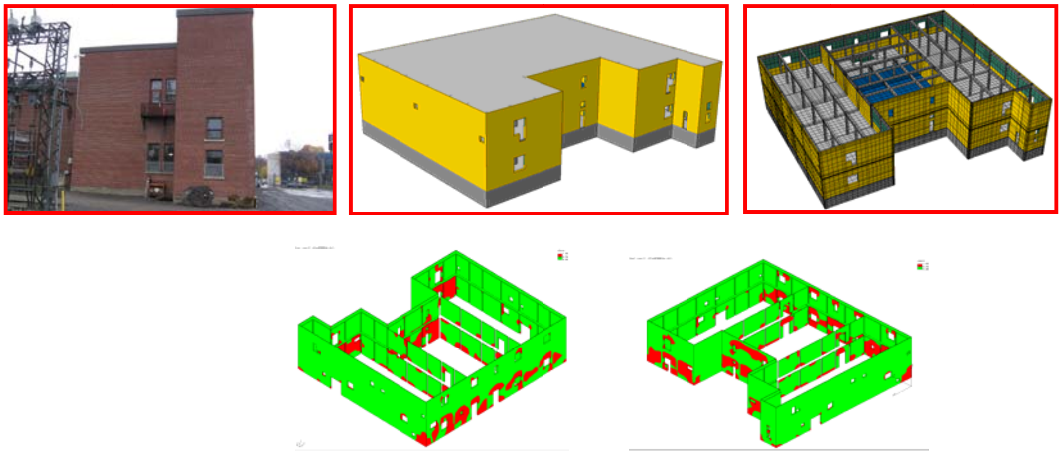


Figure 4. The geometry of the building, and the results of the elaborate FE dynamic analysis. The red areas indicate distribution of cracks in the masonry walls.

In order to specify the material properties, a series of in-situ experimental shear tests was performed on the brick walls. The procedure involved extracting two alternate bricks from a single row of the brickwork and subjecting the remaining brick to a horizontal load. In this way, the in-situ shear strength of the mortar can be assessed which, in turn, can give an indication of its tensile strength. In addition, the bricks extracted from the wall were tested for compressive and tensile strengths. Based on the experimental results, the basic material properties were established for constituents. Using those values, the average macroscopic properties of masonry were assessed, as discussed in Section 3. An important step of the validation process was the evaluation of the dynamic response of the analysed structure. The in-situ measurements of ambient vibrations were performed and the obtained natural frequencies and mode shapes were compared with those obtained from the numerical analysis.

The results of the numerical analysis showed that the story drifts as well as the max structural deformation are greater than the tolerances required by the NBC of Canada for structure used as a public safety building. Hence, strengthening of the building was required. Steel bracing combined with "System DC90" dampers are employed. The adequate choice of dampers depends on the in-plane rigidity of the masonry walls of the building and their capacity to participate with the dampers in the seismic energy dissipation. A simplified numerical analysis is proposed in order to reduce the time requirements of the elaborated FE non-linear dynamic analysis. The dynamic characteristics of typical masonry walls (panels) are evaluated following the macro-scale approach as described in Section 3 using the FE program COSMOS/M and then replaced with equivalent "Link" elements of SAP2000 software. For the analyzed building six typical panels were defined as shown on Fig. 5a. For each panel

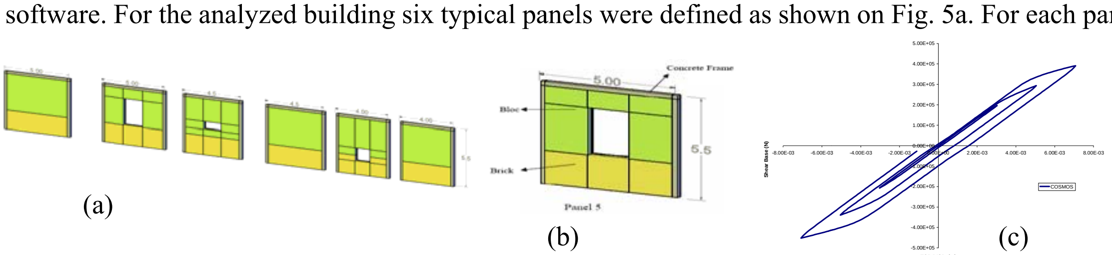


Figure 5. Typical masonry panels (a); panel #5 (b) and its shear force-deformation hysteresis (c)

(such as panel #5 shown on Fig. 5b) the force-deformation hysteresis diagram Fig, 5c was defined by elaborate FE panel analysis using COSMOS/M software. The equivalent "Link" element having the force-deformation hysteresis diagram closely matching the one obtained for the masonry panel is used in the global building model replacing the masonry panel Fig. 6.

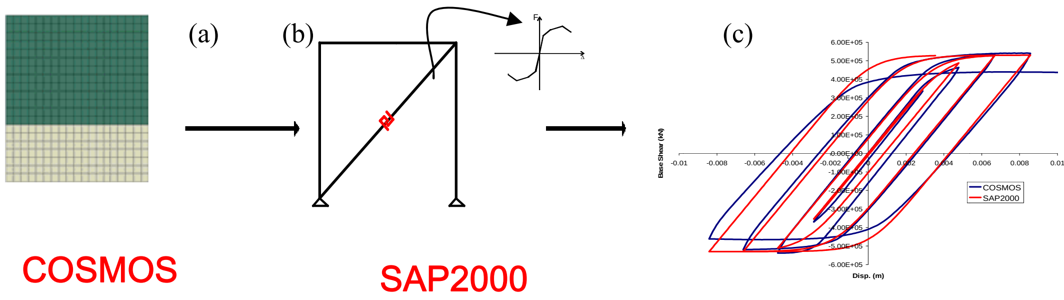


Figure 6. Typical masonry panel (a); replaced by “Link” element (b); having equivalent hysteresis (c)

The numerical model including the “Link” elements is shown on Fig. 7a. Six (6) sets of bracings Figs. 7b and 7c incorporating twelve (12) dampers were added. The required mechanical characteristics of the dampers and their optimal location in the building were obtained numerically by performing several analyses. The procedure of finding an adequate number and suitable type of dampers able to dissipate energy together with the masonry walls is the most important part of the analysis.

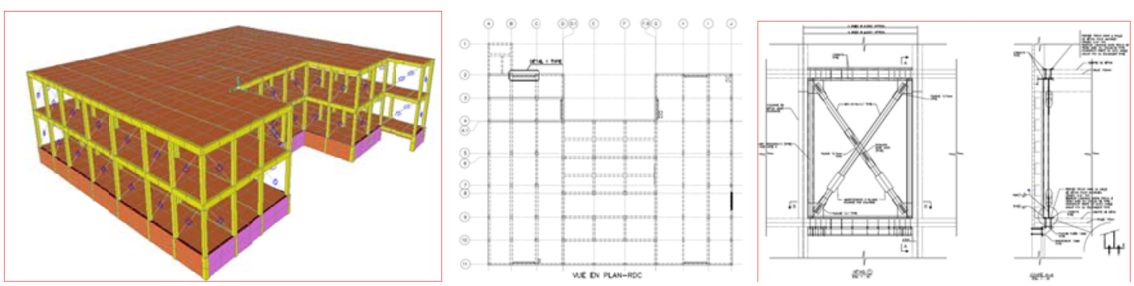
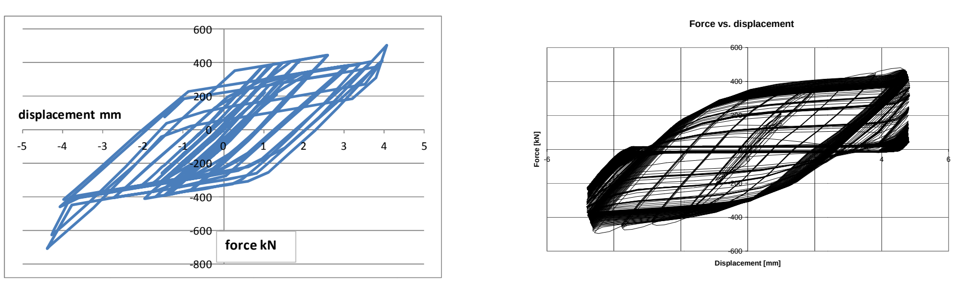


Figure 7. Numerical model including the “Link” elements in SAP2000 (a); location of the bracings with dampers on the ground floor of the building (b); and unit of bracing with dampers (c)

The selection of dampers and the bracing struts, with their stiffness characteristics, is function of the rigidities and the performance of the masonry walls. Hence, they are different for each building and are therefore usually custom made. Fig. 8a represents an optimized damper and for the analyzed building structurally adequate, numerically obtained, damper characteristics. Based on the calculated hysteresis diagram the supplier (System DC90) has fabricated and tested the dampers. Fig. 8b presents the results of their test.



(a) **D-98 Damper 1.5 to 4.0 mm; 320 to 420 kN** (b) **Damper 1.5 to 4.5 mm; 450 kN (test)**

Figure 8. Hysteresis obtained from the analysis (a); Results from the test (b)

The strengthening of the building with steel bracings, anchored to the concrete frame, combined with

“System DC90” dampers for energy dissipation was sufficient to fulfill the requirements of the NBC of Canada for drift limitation and maximum total deformation of the unreinforced masonry public safety building. The hysteresis diagrams of a typical masonry panel before Fig. 9a and after the added reinforcement Fig. 9b, indicates reduction of the deformations to a level acceptable by the NBC. It can be seen that the dampers are dissipating large portion of the seismic energy and in the same time preventing the masonry of excessive deformation and cracking.

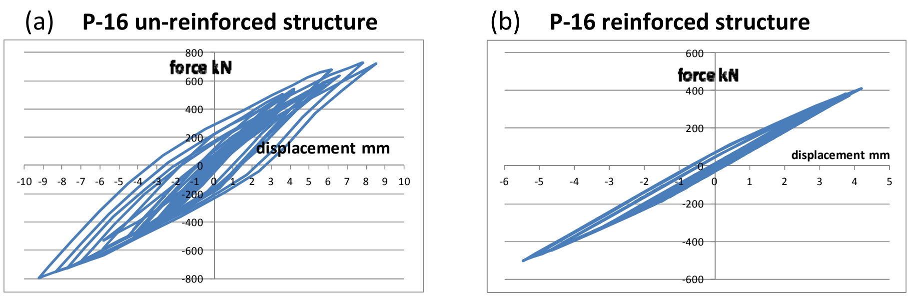


Figure 9. Hysteresis for a typical masonry panel obtained from the analysis: (a) before and (b) after the structural reinforcement with bracings and dampers

5. FINAL REMARKS

The work reported here presents a simple and effective strategy for analysing and reinforcing the non-bearing masonry walls in case of a seismic event. This study clearly demonstrates that, given the complexity of the structure, a conventional approach, based on simplistic standards/guidelines adopted by consulting engineering offices, would not be adequate here. In this case, an appropriate finite element analysis is required, examining the history of loading, to asses the efficiency of the proposed refurbishing strategy.

The macroscopic failure criterion applied to evaluate the areas of potential damage (rather than an intuitive judgement based on the values of individual stress components) represents an additional step in the proper assessment of the destructive nature of the seismic forces. The authors trust that the obtained results will be of valuable insight in assessing the methodology of strengthening the walls in the upcoming refurbishing works.

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