DAMAGE ASSESSMENT ON THE MODELING OF A LARGE SCALE CONFINED MASONRY BUILDING

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ABSTRACT

The seismic behavior of a representative medium-rise building of Mexico City has been evaluated using the capacity spectrum method. This method is widely used nowadays in the seismic assessment of buildings, since it allows obtaining fragility curves which permit evaluating the ability of a building to resist earthquakes.

A real full-height multi-story model is proposed to test the capabilities of the algorithm herein exhibited. The model is outlined through structural drawings; sized and structured following the building code regulations for masonry structures in Mexico City. Computational requirements for the analysis of large structures are indicated, in addition to the improvements to a non-linear computing code for a better performance in terms of memory management and execution times.

Finally, a comparison between obtained results and the building code regulation is carried out, highlighting differences in the obtained results.

The need to handle meshes with high amount of finite elements pushed us to develop a new layered finite element (*FE*), that can reproduce the non-linear behavior of its constituent materials when there are out-of-plane stresses without having to introduce additional degrees of freedom. The proposed FE has been compared with standard FE, presenting different kinematics, and excellent results have been obtained.

This work emerges as the need to combine and improve existing technologies in the field of finite element analysis. One of such technologies is the numerical simulation of the behaviour of composite materials. That is why it has also been necessary to develop a computing program capable of reading both finite element meshes and patterns of fibers represented with convex polygons, and as a result of areas intersections between polygons returns volumetric participation of fiber and matrix of constituents materials for each layer, in addition had to return the fiber orientation with respect to the local axis of the finite element.

Keywords: Laminated element, Composite materials, large reinforced concrete (RC) structures, Masonry structures, Mexican building code

INTRODUCTION

Since the last half of 20th century, masonry building construction has been widely used in countries and regions of extremely high seismic activity – used for construction from one-story single-family housing to medium-rise apartment buildings up to six-story high. The persistence of masonry building construction, in these regions, has a context in two main areas: economics and its structural performance.

From the economical point of view, a low-cost construction is the most significant aspect that makes masonry structures reliable. Hence, construction of housing buildings made out of reinforced masonry bearing walls is highly promoted by governmental entities. Regarding its structural performance, masonry structures, designed in compliance with current code requirements, present a high performance even in cases of significant seismic activity (Meli 1990; Marques and Lourenço

2014). Despite such wide range of application for this construction method, few resources have been put in structural masonry research while compared with steel or reinforced concrete (RC) construction.

The numerical simulation in the field of civil engineering, while widely used in structural analysis, has not benefited from the full potential offered by the new technologies for the analysis of composite materials. Within the framework of the finite element method, this technology is fully adopted in industries such as automotive, aerospace and shipbuilding. The present work emerges from the need of improving the existing computational technologies in the field of structural behavior of reinforced concrete buildings with masonry in-fills using finite element analysis for composite materials.

The need to handle meshes with high amount of finite elements pushed us to develop a new layered finite element (FE), that can reproduce the non-linear behavior of its constituent materials when there are out-of-plane stresses without having to introduce additional degrees of freedom (Escudero et al. 2017). The proposed FE has been compared with standard FE, presenting different kinematics, and excellent results have been obtained. The robustness and efficiency of the proposed methodology in the analysis of confined masonry (CM) buildings is conditioned by the ability of using different patterns of steel reinforcement, which are typically present in such structures.

The objective of this work is to present a more exhaustive contribution on the numerical modeling of CM using a *One-phase material model*, (Lourenço 1996; Rots 1997), to reproduce the overall behavior of masonry structures, using both classical and enhanced mixing theories, (Martínez 2008; Rastellini et al. 2008), on laminated composite materials (Escudero et al. 2016) within the finite element method framework for real case buildings.

A real full-height multi-story model is proposed to test the capabilities of the algorithm herein exhibited. The model is outlined through structural drawings – sized and structured following the building code regulations for masonry structures in Mexico City. Computational requirements for the analysis of large structures are indicated, in addition to the improvements to a non-linear computing code (CIMNE 2014) for a better performance in terms of memory management and

78 execution times. Finally, a comparison between obtained results and the building code regulation
79 is carried out, highlighting differences in the obtained results.

80 NUMERICAL FORMULATION

81 The constructive process of CM involves the interaction of several materials; in this work, it
82 is fundamental to establish the mechanical behavior and constitutive models, from a mathematical
83 point of view, of three different simple materials – steel, concrete, and masonry – which are briefly
84 summarized in this section.

85 Steel

86 The classical macro-mechanical theories of plasticity are based on the notions of a yield surface,
87 a hardening rule, and on the stress-plastic strain relations of the given material (Simo and Hughes
88 1998). Such notions are used to formulate a material model for the calculation of its response
89 during plastic deformation.

90 Let us consider Fig. 1.(a), where the idealized uniaxial *Strain-Stress* relationship of steel is
91 depicted. Both Elastic ($\overline{OA}$) and Plastic ($\overline{AB}$) ranges can be clearly distinguished. Elastic range can
92 be mathematically modeled by generalized Hooke's Law for restoring forces, where the stress is
93 linearly proportional to the strain. However, stresses larger than the elastic limit, or yield strength
94 σ_y (point A) cause a permanent deformation known as plastic or plasticity. On the other hand, a
95 typical representation, in the Westergard stress space, of the von Mises yield criterion is given in
96 1.(b). Within this work, the mechanical behavior of steel will be reproduced following a plasticity
97 model, combined with this yield function. The governing equations of classical rate-independent
98 plasticity follows the formulation presented in Ref. (Simo and Hughes 1998).

99 The general formulation follows the additive decomposition of the strain tensor ε into an elastic
100 ε^e and plastic part ε^p . For more details see references (Simo and Hughes 1998; Escudero et al.
101 2017).

Concrete

Concrete is a composite material, since it is produced of a granular material (*aggregate*) embedded in a hard matrix of material (*cement*); however, it is a common practice to model it as a simple *quasi-brittle geo-material*, even though its high non-linear performance achieved due to the formation of micro-cracks and slipping among the aggregate particles (Oller 1998). Although *Continuum Damage Mechanics (CDM)* was settle first as a scheme to model *quasi-brittle* materials (Chaboche 1988a; Chaboche 1988b; Ju 1989; Oliver et al. 1990), it has been used for the last thirty years to model concrete materials (Mazars and Pijaudier-Cabot 1989; Lubliner et al. 1989; Oller 1998; Jason et al. 2006; Tao and Phillips 2005).

Continuum Damage Models are defined by a degradation, as an internal variable, of both resistance and elasticity modulus – this variable being either a scalar or a tensor; for an isotropic damage model, such internal variable is represented by a scalar (Simo and Ju 1989; Oliver et al. 1990). For this case, the micro-cracking orientation is neglected, which leads to skip the effect of anisotropic damage, or the effect of material's anisotropy in the direction of the micro-crack.

An idealization of the concrete expected behavior, during a cycling loading process, is represented in Fig. 2.(a). The loading process goes from traction to compression, starting in an elastic range $\overline{OA}$, once it reaches the maximum traction resistance A , a degradation of stiffness is activated in $\overline{AB}$, finally stress state B is reached and it starts an unloading process $\overline{BO}$. Now, let us consider the unloading process has reached point O again, which shows the advantage of damage models with two scalar damage variables. Whereas $d^+ \neq 0$, $d^- = 0$ (since they are independent from each other), and loading process along $\overline{OC}$ has an elastic behavior again, and until it reaches the maximum compression stress C compression degradation will not be activated. As expected, once loading process reaches stress C degradation of stiffness in compression is activated $\overline{CD}$ until it reaches D and it starts an unloading process again.

Fig. 2.(b) shows a Westergard stress space where two yield surfaces are superimposed. Several yield functions, appropriated for concrete, have been proposed. For instance, in Ref (Faria et al. 1998) was used a yield surface based on energy combined with a modification of the Drucker-

Prager yield function for the only-compression case; finally, the obtained results by (Faria et al. 1998) were compared with the experimental results reported in Ref. (Kupfer et al. 1969) showing good accuracy among the results.

Damage models for concrete are based in the fracture energy which is assumed to be a material property (Oliver et al. 1990). This work follows a damage model given in Ref. (Paredes 2013) which is a modification of the damage model presented by Faria et al (Faria et al. 1998) where two scalar damage variables are used: a variable for traction d^+ and another for compression d^- . Such modification raises for *quadrants* II and IV of a given yield surface with traction and compression combined, see Fig. 2.(b), consisting on:

- Additive decomposition of the stress tensor σ . In the case that a tensional state is either in *quadrant* II or IV, the weight between traction σ^+ and compression stresses σ^- is computed to obtain a *dominant* stress.
- Stresses σ^+ and σ^- will not be treated separately. The constitutive equation will be integrated using σ taking into account the *dominant* stress.
- *Constitutive Damage* conditions detailed in (Paredes 2013) are applied.

A more in-depth review about the topic, for the reeder, is given in Ref. (Faria et al. 1998).

Masonry

Masonry is a composite material resulting of individual units laid (brick or rock) bound together by mortar. From now on, it will be implied that whenever it is referred to *masonry* it means *brick masonry*.

In order to settle a damage criteria to predict non-linear behavior of masonry, it is necessary to determine its basic failure mechanisms, under the most elemental loading conditions, and its mechanical properties through experimental tests. Large amount of test over masonry materials, mortar and units, have been performed to study their interaction once they are bond together as a masonry prism.

Masonry is stated as an anisotropic material; depending on the accuracy and the simplicity

desired, masonry can be modeled using the following strategies (Lourenço 1996):

- **Detailed Micro-Modeling:** Units and mortar in the joints are represented by continuum elements whereas the unit-mortar interface is represented by discontinuous elements.
- **Simplified Micro-Modeling:** Expanded units are represented by continuum elements whereas the behavior of the mortar joints and unit-mortar interface is lumped in discontinuous elements.
- **Macro-Modeling:** Units, mortar, and unit-mortar interface are bonded in the continuum.
- **Homogenized Modeling:** A midway strategy between micro and macro-modeling. This method deals with the global problem of composite material in a two-scale context. The macroscopic scale uses the composite materials to obtain the global response of the structure; composites are treated as homogeneous materials in this scale. The microscopic scale corresponds to an elemental characteristic volume in which the microscopic fields inside the composite are obtained; this scale deals with the component materials. Homogenization theory assumes a periodical configuration of the composite material to relate these two scales (Oller 2016).
- **Mixing Theory:** Based in two main hypothesis: all composite constituents have the same strains and each constituent collaborates to the composite behavior according to its volumetric participation. The main problem of this approach is the iso-strain condition, which forces a parallel distribution of the constituents in the composite. Some improvements to the original formulation can be found in reference (Oller 2016).

Advantages and disadvantages of each approach are discussed in (Lourenço et al. 2007; Lourenço 1996; Pelà 2009; Lopez et al. 1999; Quinteros et al. 2012). Herein, a *Macro-Modeling* approach is followed due to the reduced time and memory requirements as well as for the implied user-friendly mesh generation. This modeling scheme is the most valuable when a compromise between accuracy and efficiency is needed.

Several approaches have been suggested for the determination of Young's modulus. This

parameter is rather variable even for nominally identical specimens, and, as an approximation, several researches have derived expressions assuming linear elastic behavior for both materials and equating the compressive deformation of masonry to the sum of the deformation of the brick and mortar joints. For more details see references (Lopez et al. 1999; Quinteros et al. 2012).

Extension to this work is given in (Lourenço 1996; Pelà 2009) where composite yield criteria, suitable for modeling anisotropic materials under plane stress conditions, are presented. In such cases, individual yield criterion for both tension and compression have been considered, according to different failure mechanisms. Although both cases miss to reproduce the uniaxial compressive strength parallel to the bed joints, globally, good accuracy can be notice from such yield criteria.

In this work, it is adopted the scheme to model the masonry as an orthotropic material – which is an *implicit* definition of the orthotropic yield function based on the transformed-tensor method proposed by Oller *et al.* (Oller et al. 2003). The reader may abound in the subject consulting reference (Pelà 2009), where an extensive review of the state of the art has been carried out; also, the underlying principles and basis of the numerical implementation follows the same reference.

The objective of the approach, proposed by Oller *et al.* (Oller et al. 2003), is to adjust an arbitrary isotropic yield criterion to the real behavior of an anisotropic material using a mapping method based in the existence of a *real anisotropic space* of stresses σ_{ij} and a conjugate space of strains ϵ_{ij} , such that each of these spaces has its respective image in a *fictitious isotropic space* of stresses $\bar{\sigma}_{ij}$ and strains $\bar{\epsilon}_{ij}$ respectively, Fig. 3.

NUMERICAL DESCRIPTION OF THE STUDIED MODEL

In order to show the potential of the proposed method, the model of a real-life masonry building has been proposed, which corresponds to the geometry, plan distribution, and elevation of typical buildings of Mexico City. Such building will be referred, from now on, as the B-SSC model, and, fictitiously, located in the seismic zone ZIIIIa (placed at the former *lake* area of Mexico City) according the classification depicted in reference (Gobierno del Distrito Federal 2004a).

Building description

The proposed housing building is a six-story construction, with two apartments for each level – a total of twelve (12) apartments per building with an area of $58m^2$ each. Architectural drawings, showing the distribution in plan and elevation of the structural members (Fig. 4 and Fig. 5), demonstrate the complexity of the structural system which is analyzed .

The general aspect of the designing process, for structural masonry, is summarized as follows:

1. Definition of the location of the building, in order to define seismic hazard.
2. Bearing masonry walls, with structural purpose, have to be distinguished – discarding walls with only architectural purpose, like dividing walls or *short* walls.
3. Loading conditions such as dead loads, additional dead loads, and live loads have to be defined using regional codes – for this case ([Gobierno del Distrito Federal 2004d](#)).
4. The first performed design is through gravity loads, paying special attention to walls in the ground floor, since those are the ones subjected to the highest vertical stresses.
5. An iterative design due to lateral loads is performed for each orthogonal direction. The starting point of such analysis is the assumption that all walls are made of confined masonry.
6. Computation of in-plane bending moment of the masonry walls, where the confining columns forming the masonry wall have to be endowed with the required amount of steel reinforcement.
7. Structural documentation is generated using the information obtained throughout the designing process.

The structural design based in the regional construction code was obtained, for comparison purpose, using the commercial software *ANEMgcW*, ([G.C. Ingeniería y Diseño, S. C. 2007](#)). The structural analysis program *ANEMgcW* follows the steps mentioned above and it has been used as an aid for analysis and design process, and consequently, to generate the structural drawings shown in figure 6. Hence, this structural layout become the basis of the upcoming comparisons among the designing process using a code regulation and the analysis scheme proposed in this work. Obtained

results using ANEMgcW are presented in next section.

The stiffness obtained, through an elastic analysis, using ANEMgcW for each of the stories of model B-SSC are:

$$\begin{aligned} {}^1\mathbf{K}_X &= 3,027,012.84 \text{ kN/m} & {}^4\mathbf{K}_X &= 3,111,957.63 \text{ kN/m} \\ {}^2\mathbf{K}_X &= 3,227,568.48 \text{ kN/m} & {}^5\mathbf{K}_X &= 3,310,423.74 \text{ kN/m} \\ {}^3\mathbf{K}_X &= 3,359,669.94 \text{ kN/m} & {}^6\mathbf{K}_X &= 3,373,992.94 \text{ kN/m} \end{aligned} \quad (1)$$

Such values have been assembled into the global stiffness matrix of a concentrated mass model (Fig. 7). Then, a displacement at the top (story 6th), δ_{top} , has been imposed and the corresponding reaction at the bottom $V_{\delta_{top}}$ has been evaluated solving the corresponding linear system.

The value corresponding to the stiffness $\mathbf{K}_{dc}$ is:

$$\mathbf{K}_{dc} = \frac{V_{\delta_{top}}}{\delta_{top}} = 538,316.78 \text{ kN/m} \quad (2)$$

The stiffness obtained can be easily evaluated for the elastic range as

$$\mathbf{K}_{pw} = \frac{\Delta V_j}{\Delta \delta_j} = 476,580.49 \text{ kN/m} \quad (3)$$

where $\mathbf{K}_{pw}$ stands for the total stiffness of model B-SSC, $\Delta \delta_j$ and ΔV_j are an increment of displacement and an increment of shear force within the elastic range, respectively.

Structural elements for numerical model B-SSC

The structural plan, shown in Fig. 6, corresponds to a view for the proposed building. In this section, it is carried out a description of the structural elements that conform model B-SSC.

Masonry Walls

Masonry walls have been structured to meet the minimum thickness required to avoid out-of-plane buckling problems. The walls are confined with reinforced concrete elements along the whole perimeter (Gobierno del Distrito Federal 2004c).

A layer discretization was performed following criteria fully described in (Escudero et al. 2017); the schematic representation of the layers is shown in Fig. 8. Regarding the layers distribution for the masonry walls, they are discretized using an arrangement of 12 layers with different thickness (Table 1) for a total thickness of 12 centimeters. As it is observed, a finer layer distribution is given in the farthest zones away from the geometric axis of the shell, this refinement obeys to the fact that the higher bending stresses appear in such zones.

Reinforced Concrete Walls

Fig. 6 reports a designation of sections that goes from CW-01 to CW-10 (CW stands for concrete wall). Walls from CW-03 to CW-10 has been used for constructive purposes; this is, since they are considerably small, it is preferable to make them of reinforced concrete, whereas CW-01 and CW-02 provide stiffness that the building requires to prove compliance with the requirements for mechanical strength, stability, and safety according Ref. (Gobierno del Distrito Federal 2004a).

RC walls or, simply, concrete walls are built to modify the dynamical properties of the building. Due to their high in-plane stiffness they are used as a solution to lower the period of vibrations for a given structure reducing the deformation other walls undergone and, consequently, the stress levels they are subjected to.

Typical arrangement for the steel reinforcement pattern, used in concrete walls, is two grids on each bed of the wall. In this case, a vertical reinforcement of steel rods, with $3/8in$ ($9.5mm$) of diameter with $20cm$ of separation among themselves, is used; whereas for horizontal reinforcement have been used steel rods of $1/4in$ ($6.4mm$) of diameter with $15cm$ among themselves. All concrete walls have been reinforced with columns C-01 placed at both ends. A depiction of the steel distribution for a concrete wall is shown in Fig. 9. The thickness of the layers regarding the concrete walls cross sections is presented in Table 2; the same arrangement will be used for columns and beams.

Confinement columns

Only two different sections of columns have been used while designing the model B-SSC, which are shown in Fig. 10; both sections of $12 \times 12cm$.

Columns C-01 have been used for the two first levels of the building ($L + 0.00$ and $L + 2.63$), with a main steel reinforcement of 4 rods with $1/2in$ ($12.7mm$) of diameter; whereas their secondary steel reinforcement consist in rods of $1/4in$ ($6.4mm$) of diameter, placed with a separation of $15cm$ among themselves.

On the other hand, section type C-02 have been used from third level to sixth level of the building (levels $L + 5.26$, $L + 7.89$, $L + 10.52$, and $L + 13.15$). The main steel reinforcement is given by 4 steel rods of $3/8in$ ($9.5mm$) of diameter with the same secondary steel reinforcement of section C-01.

Confinement beams

Figure 11 shows the three different sections used in B-SSC model. Two of them have been used to endow horizontal confinement to masonry walls, whereas one of them has been used with the purpose of shorten concrete slabs.

Section RW-01 beams (RW stands for *rib over wall*, technically not a beam) are used to infer confinement on *regular* masonry walls. The cross section is $12 \times 25cm$, and the main steel reinforcement consists in 4 rods with $3/8in$ ($9.5mm$) of diameter. Beams with section type RW-02 are used to endow horizontal confinement to *short* masonry walls (under the window openings), and although both sections RW-01 and RW-02 have different dimensions, they have the same steel reinforcement arrangement (figure 11).

On the other hand, beam section B-01 reduces the dimension of the slab in construction axis 2, from **B-E** and **G-J**, see Fig. 6. The cross section is the same than for RW-01 with a main steel reinforcement of 4 rods with $1/2in$ ($12.7mm$) of diameter.

Finally, it has to be pointed out that the three section types of beams have the same secondary steel reinforcement, which consist in rods of $1/4in$ ($6.4mm$) of diameter placed with a separation of $15cm$ among themselves.

Slabs

Thickness of slabs have been designed to meet requirements regarding serviceability loads stated in reference ([Gobierno del Distrito Federal 2004b](#)). Although in some cases, for structural

purpose, it is necessary to shorten concrete slab boards with a concrete beam, see figure 12.

Slabs have been discretized along the thickness using a pattern of 20 layers for a total thickness of 15cm. Their layer distribution is given on Table 3, where it is noticed the steel reinforcement pattern in both beds and in both orthogonal directions. In all cases, the diameter of steel reinforcement rods remain the same, this is 3/8in(9.5mm), which varies is the separation among themselves. This steel reinforcement gap, for the lower bed, is 30cm, whereas for the upper bed, a separation of 40cm is used.

Simple Material Properties

The mechanical and constitutive properties, of steel, concrete, and masonry, are given below. The constitutive equation for concrete and steel in the slab are set to be elastic justified by the fact that in such area no damage is expected.

The concrete reinforcement is given by corrugated steel rods of ($f_y = 4200kg/cm^2$). The concrete for confined masonry has a compressive strength, ranging from 200 to 250 kg/cm^2 (20 to 25 MPa). In this work the compressive strength is assumed equal to $f'_c = 250kg/cm^2$. In this work, it is assumed a simple compressive strength f'_{mcy} of the combination of masonry units and mortar equal to $f_m^* = 120kg/cm^2$, whereas the maximum shear strength is equal to $v_m^* = 3.5kg/cm^2$. Mechanical and constitutive properties used to model the steel, concrete, and masonry elements are given in Table 4.

Structured Composite Materials (SCM)

In this section, a brief description of SCM used to generate volumetric participation of composite materials within each layer of the FE is carried out.

The procedure herein used is fully described in (Escudero et al. 2017). In all cases, it is only depicted one story of the building, implying that the process has been repeated for the six stories.

Fig. 13 shows the SCM, for model B-SSC, of constructive axis 3. The fibers of longitudinal steel reinforcements (main reinforcement) are shown by a thicker line, whereas the hatched area has been used to represent the transversal steel reinforcement (secondary reinforcement). It is convenient to point out that Fig. 13 shows a superposition of all layers within the SCM and in no case FE

belonging to the SCM have been displayed.

The others SCM shown in this section correspond to the constructive axis **A** and **K** illustrated in Fig. 14; as in the previous example, the information of all layers has been superimposed, and finite elements has not been shown, this due to visibility matters. The purpose of displaying SCM from constructive axis **A** and **K** (as in the previous example) is to remark the existence of the reinforced concrete wall and their steel reinforcement pattern.

Finally, Fig. 15 displays the steel reinforcement pattern of structural elements belonging to construction axis **1** (rear façade), **7** (font façade) and **F**.

Boundary Conditions

The analysis process has been performed using three (3) different loading phases. It is convenient to point out that, in no one of the loading phases it has been used any loading factors which are commonly included in code regulations due to the uncertainty in the assessment of loading forces.

In all three loading phases, the foundation of the structure has been set to remain rigidly fixed; hence, neither displacements nor rotations would be allowed.

First Stage Loading Condition: Dead Load

Herein are given a summary of structural (walls, beams, columns, slabs) and non-structural elements (in-fills, plaster) forces acting due to gravity.

Fig. 16 shows a typical loading analysis for the walls of the current building. A small sketch (on the left) shows both structural and non-structural components of a masonry wall, whereas in the right, a table displays in detail the thickness and weight of all components involved. Additional loads due to uncertainties regarding *real* thickness of components or in the assessment process of the weights are contemplated ([Gobierno del Distrito Federal 2004d](#)). In the case considered, only will be necessary to add $20\text{kg}/\text{m}^2$ due the presence of mortar. Finally, figure reports the total weight for the masonry wall to be equal to $W_D = 260\text{kg}/\text{m}^2$.

Similarly, Fig. 18 shows the story system used in the building; as it can be noticed, it consist in only a vinyl tile to cover the concrete slab. The thickness of the vinyl tile has a uniform weight equal to $5\text{kg}/\text{m}^2$ (included the resin to bond it to the concrete slab). Due to the presence of concrete,

360 20 kg/m^2 has been added to the total weight. Hence, the total weight of the storey systems is equal
361 to $D_L = 385.0\text{ kg/m}^2$.

362 Finally, Fig. 18 displays the loading analysis for the roofing system. The sketch on left shows
363 an average in-fill of aggregate material considered in the whole area of the roof. Such in-fill is
364 necessary to provide a slope to the roof, so the water does not become stagnant when it rains. For
365 thicknesses and weights of elements forming the roof, it is added 40 kg/m^2 due to the presence of
366 mortar and concrete. Hence, the total weight of the roofing systems is equal to $D_L \approx 525.0\text{ kg/m}^2$.

367 *Second Stage Loading Condition: Live Loading*

368 Live loads are based upon published regional regulations; in this case the *Building Regulations*
369 *for the Federal District* (Gobierno del Distrito Federal 2004d). Such code regulation, in its chapter
370 V article 199.V.g, remarks:

- 371 • An uniform live load equal to 100 kg/m^2 has to be used in the roof system of residential
372 houses with an slope not higher than 5% (section 199.V.g)
- 373 • An uniform live load equal to 170 kg/m^2 has to be used in the storey system of residential
374 houses (section 199.V.b).

375 *Third Stage Loading Condition: Pushover*

376 The third loading stage corresponds to a standard pushover applied normal to the building
377 elevation axes generally set as x direction. Hence, a displacement is imposed to a set of nodes
378 located along an edge of the slab of the sixth story. The reason of conducting the pushover analysis,
379 in this way, is to avoid a high concentration of forces to only few FE.

380 The region where displacements in x direction have been applied is shown in Fig. 19. The pur-
381 pose of imposing such displacement is to predict the force-displacement response of the structure.
382 Obtained results for model B-SSC are presented in following section.

383 **RESULTS OBTAINED FOR STUDIED BUILDING**

384 The computation of the analysis for the numerical model were carried on by an in-house code
385 called PLCd (CIMNE 2014). This is a state of the art implicit finite element algorithm written

in Fortran developed to treat a large variety of composite materials through the use of Rules of Mixtures. The obtained results for model B-SSC applying the pushover analysis, described in previous section, are summarized and discussed in this section with the aid of figures described down below.

The displacement-force response of model B-SSC is given in Figure 20, where a schematic representation of the structural stiffness, set by equations 2 and 3, is shown. In dashed lines, the stiffness of both models are shown, and as can be seen, a good approximation has been obtained between them since the difference is only 12.95%.

Figure 20 also shows us separately the stiffness of walls placed perpendicular to the application of displacement (bending walls) and walls parallel to the application of displacement (membrane walls), which confirms reference (Gobierno del Distrito Federal 2004c) regarding the low stiffness exerted by the bending-acting walls, while their stiffness is compared with the membrane-acting walls, showing that their stiffness could be neglected for analysis and design purposes. The overall response of the structure is depicted by the continuous red line.

Fig. 21 to Fig. 24 correspond to a set of deformation stages (amplified 250 times) within the analysis process, where the damage variables of the element is also shown. In all cases, images on the left represent the confining elements, whereas images on the right represent the masonry elements. The selected stages for the displacement have been a) $\delta = 2.00mm$, b) $\delta = 6.00mm$ and c) $\delta = 8.20mm$, respectively.

Fig. 21 presents the damage evolution of walls W-01 and W-02, located along constructive axis 1 (rear façade). The evolution of the damage variables is clearly computed as the displacement is increased in the expected stress areas – near the joints of the confining elements.

Fig. 22 and Fig. 23 corresponds to the damage evolution of walls W-05, W-06 and W-07, located along constructive axis 3 and walls W-08, W-09, W-10 and W-07, located along constructive axis 4, respectively. Due to the eccentric distribution of the plan, it is observed a stronger evolution of the damage variable in axis 3.

Fig. 24 presents the damage evolution of walls W-15, W-16, W-17 and W-18, located along

constructive axis 7 (front façade). A large value of damage is found in the short masonry walls, coinciding with the theory reported in the structural aids for seismic reinforcement buildings.

For all four cases, the confining elements show some degree of damage through this variable; this is interpreted as the cracks and micro-cracks developed in these structural elements as the response for the pushover analysis.

Damage Assessment

In this section, the damage scenarios the structure B-SSC could undergo in a zone with high seismic activity are characterized; therefore, two main assumptions are taking into account:

1. The capacity spectrum and the bilinear capacity spectrum, used to characterize the damage scenarios and depicted in Fig. 25, correspond to the displacement-force response obtained from the pushover analysis, previously described.
2. For comparative purposes, two *locations* have been selected for the model B-SSC. The first location (location L1 from now on) is somewhere in Mexico City within the seismic zone ZIIIIa according to micro zone described in reference ([Gobierno del Distrito Federal 2004a](#)). The second location (location L2 from now on) is somewhere within the European Union where the soil is **D**, type **1** and has a peak ground acceleration equal to 0.20_g according to classification of Eurocode 8 ([de Normalisation 2004](#)). The corresponding design spectra transformed to a $sd - sa$ are shown in Fig. 28.

For the computation of the capacity spectrum shown in Fig. 25, two main simplifications to model B-SSC had to be performed ([Alzate 2013](#)). The first one consists in converting it into a model with equivalent stiffness and *one-degree-of-freedom* per story, where the mass of the corresponding story is concentrated. The second simplification sets the mass-concentrated model as a *one-degree-of-freedom* model; this is achieved through a modal decomposition, using the vibrational mode 1.

The construction of the bilinear capacity spectrum is based on the three hypothesis described next,

1. The area under the spectral capacity curve must be equal to the equivalent bilinear capacity curve.
2. Coordinates of the point with maximum displacement (D_u, A_u) match in both curves.
3. The initial slope in both graphs must be the same.

which led to the corresponding values for points (D_y, A_y) and (D_u, A_u) shown in equation 4.

$$\begin{aligned} D_y &= 1.870mm & D_u &= 3.640mm \\ A_y &= 0.375_g & A_u &= 0.487_g \end{aligned} \tag{4}$$

Once the spectral capacity curve has been identified, the bilinear capacity spectrum is plotted; which is essential to estimate the probabilistic damage, since all damage scenarios are based upon values of points (D_y, A_y) and (D_u, A_u). Risk-EU (Mouroux et al. 2004) distinguishes four possible damage scenarios on a structure, namely, *slight* (ds_1), *moderated* (ds_2), *extensive* (ds_3) and *complete* (ds_4), whose definitions depends on points (D_y, A_y) and (D_u, A_u) of the bilinear capacity spectrum according to equation 5.

$$\begin{aligned} ds_1 &= 0.7 D_y \\ ds_2 &= D_y \\ ds_3 &= D_y + 0.25(D_u - D_y) \\ ds_4 &= D_u \end{aligned} \tag{5}$$

Fragility curves represent the probability in which a damage scenario ds on a structure could be reached or exceeded, as a function of the parameter that represent the intensity of the seismic action. Those values, for the current structure, are given in equation 6, leading to Fig. 26 and Fig. 27.

$$\begin{aligned}
ds_1 &= 1.309mm & ; & \quad \beta_{ds_1} = 0.280 \\
ds_2 &= 1.870mm & ; & \quad \beta_{ds_2} = 0.240 \\
ds_3 &= 2.313mm & ; & \quad \beta_{ds_3} = 0.300 \\
ds_4 &= 3.640mm & ; & \quad \beta_{ds_4} = 0.350
\end{aligned} \tag{6}$$

The seismic demand that the structure undergoes, is obtained through its point of performance. Its computation is given by a linear equivalent approach. To do so, let us consider figure 28, corresponding to the design spectra for location L1 and location L2; both spectra have been plot in a $sd - sa$ representation in order to link them together with the bilinear capacity spectrum shown in Fig. 25.

Fig. 29 represent the superposition of both, the design and the bilinear capacity, spectra. The point of performance corresponding to the location L1 is defined by point **A** ($sd_{L1} = 0.89mm$), which is the crossing of the bilinear capacity spectrum and the design spectrum for L1 – point **A** corresponds to a null damage state, since the seismic demand is low. On the other hand, the crossing point of the prolongation of the first segment of the bilinear capacity spectrum, and the design spectrum for location L2 correspond to the point of performance for location L2 ($sd_{L2} = 2.78mm$).

Finally, using the fragility curves depicted in figure 26, and the point of performance shown in figure 29, for both locations, it is possible to assess probabilistically the damage scenarios that model B-SSC could undergo. Results are summarized in Table 5.

Table 5 shows a very low probability of occurrence of a *low* damage scenario, equal to $P_{(ds_1)} = 8.65\%$; whereas the remaining scenarios have no possibility of occurrence, this is obvious, since the model B-SSC has been designed to withstand the seismic conditions of location L1. From this values, the expected damage can be represented using the damage index $DI = 0.021$.

For location L2, however, since the seismic demand increases, so the probabilistic damage does, which was the reason of using such fictitious location. Results of model B-SSC at location L2 reports a damage index $DI = 0.7246$, being $DI = 1.0$ the index corresponding to the complete damage scenario.

Fig. 30 is a graphical representation of the meaning of fragility curves. It is represented the spectral displacement $ds_e = 2.78mm$, which correspond to the point of performance of structure SSC at location L2; the crossing lines represents the probability of occurrence of the given damage scenarios, which already have been summarized in table 5.

COMPUTATIONAL REQUIREMENTS

The model B-SSC is a *real-life* masonry structure requiring a great amount of computational resources.

In this section are described the improvements carried out in PLCd to perform analysis of large structures in a reasonably short amount of time, and consequently, to be able to analyze structures such as model B-SSC.

Memory-Consuming Based Optimization

Let us consider model B-OSC, which is fully described in Ref. (Escudero et al. 2017), and model B-SSC, herein explored, for comparative purposes in terms of computational resources. Table 6 shows a comparison among the Random Access Memory (RAM) required for different process in the analysis of both models. From left to right: column 2 shows the amount of FE within the used mesh; column 3 shows the total amount RAM required; column 4 depicts the required memory to store information of the simple materials such as internal variables, stresses and strains; and column 5 shows the memory required for the PARDISO solver (Schenk et al. 2010).

Expressed in term of percentages, the required memory to handle the information of simple materials is equal to 45.78%, whereas the solver uses 42.50% of the total. Analyzing such percentages makes clear to where should be directed the efforts on optimizing the consumed memory.

From the assumption that not all elements reach a non-linear behavior, the proposed programming scheme (shown below) was implemented in PLCd.

- It was stored an internal variable for every non-damaged FE with the same composite information.

- Once a component within a FE has reach a non-linear behavior, then allocation of memory to store information of the FE was required (internal variables, stresses and strains).
- Allocating at every loading step or iteration may be time consuming, however, it will be justified for large structures where a considerable amount of RAM would be required.
- For output and visualization purposes (writing information in GiD format) the information of non-damaged finite elements is evaluated using their local displacements, since is the only information stored. This would lead to a major drawback.

The results obtained, once the previously described scheme has been implemented in PLCd, are shown in table 7. There is a considerable reduction on the amount of consumed memory; however, the 19.00Gb required by model B-SSC seems excessive to manage in a *conventional* desktop computer, and maybe to perform the analysis of such models would be required the node of a cluster.

In this section, it has been carried out a comparison only in terms of consumed memory, see Table 8; however, further comparisons in terms of running time while using an iterative solver are required, to fit the best alternative.

Parallelization of PLCD Using OMP directives

This section covers the parallelization of the module used to solve mechanical problems with shell elements, which has been performed using OpenMP ([Barney et al. 2010](#)) directives in three different sections of the code. Such sections are shown below.

1. Loop over elements while evaluating the generalized strains $\hat{\epsilon}$ and the generalized stresses $\hat{\sigma}$. The first section to be parallelized is the loop over elements where the generalized strains $\hat{\epsilon}$ and generalized stresses $\hat{\sigma}$ are evaluated. Since the loop in this case is only over the elements is less time-consuming than the portion of code parallelized shown in next section.
2. Loop over elements while integrating the constitutive equations (plasticity, damage, etc.).
3. Loop over elements while writing/reading information to perform a restart operation.

Results of the implementation of the previously described concepts can be summarized with the use of graph on Fig. 31, that reports the time needed to perform 10 steps for each of the 3 different loading phases, for a total of 30 steps.

Have to be taken into consideration that the process of reading the input information, and allocation of the main variables has not been parallelized, hence, it was also necessary the use of graph on Fig. 32, where the time spent along one simple iteration is reported, omitting the time it takes to write output information, write information to perform restart operations, allocate the variables at the beginning of the analysis process, and so on.

CONCLUSIONS

Using the proposed scheme it has been possible to obtain the force-displacement response for a confined masonry structure of a six stories high building. Results have been compared with the current code regulation for construction in masonry and concrete of Mexico City finding good agreement among them. With the results obtained, it enables the characterization of damage scenarios that such structures could undergo in high seismic activity zones. To achieve this evaluation, it has been used the methodology proposed by Vargas (Alzate 2013) to assess a reliable seismic demand, and consequently, to state a probabilistic damage a structure would undergo given such seismic demand.

The implementation of a macro-modeling technique combined with *2-D shells* finite elements has proven been effective for modeling the full-scale building. On the other hand, the possibility of using a large strain formulation approach for the studied structures seems rather inefficient, due to the small amount of deformation they undergo when cracking occurs.

A computational tool was developed in order to reproduce the steel reinforcement pattern of *real-life* constructions. And so, been able to perform a more precisely numerical simulation of composite materials, which permit deducting the behaviour of the composites based on the constitutive equation of the component materials.

Due to the computational cost of the model considered in this work, it was necessary to adopt a programming strategy that allow to reduce the execution time of analysis and the computational

resources required in terms of RAM.

The analysis scheme for confined masonry buildings proposed in this work, stand on the basis that it is possible to obtain parameters to define the mechanical and non-linear response of masonry elements; this, in order to plot an anisotropic yield function.

ACKNOWLEDGEMENTS

This work has been supported by the European Commission, under the Marie Curie program, at the IRSES agreement 612607 (TCAINMAND project), by the European Research Council through of the Advanced Grant: ERC-2012-AdG 320815 COMP-DES-MAT *Advanced tools for computational design of engineering materials*, by the European Community under grant: NMP-2009-2.5-1 246067 *Multiscale Reinforcement of Semi-crystalline Thermoplastic Sheets and Honeycombs*, by the Spanish Ministerio de Economía y Competividad through the project MAT2014-60647-R *Multi-scale and multi-objective optimization of composite laminate structures (OMMC)*, and by the the Mexican government through the grant provided by CONACyT to complete PhD studies. All this support is gratefully acknowledged.

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Layer Number	Thickness(mm)
1, 12	2.00
2, 11	4.00
3, 10	8.00
4, 9	12.00
5, 8	14.00
6, 7	20.00

TABLE 1. Thickness of layers (mm) for masonry walls.

Layer Number	Thickness(mm)
1, 18	4.00
2, 17	12.90
3, 16	4.80
4, 15	4.70
5, 14	6.40
6, 13	6.30
7, 12	4.80
8, 11	4.70
9, 10	11.40

TABLE 2. Thickness of layers (mm) for columns, beams and concrete walls.

Layer Number	Thickness(mm)
1, 20	2.00
2, 19	8.00
3, 18	11.20
4, 17	5.00
5, 16	5.00
6, 15	4.75
7, 14	4.75
8, 13	9.50
9, 12	12.40
10, 11	12.40

TABLE 3. Thickness of layers (mm) for slabs.

Property	Steel	Concrete	Masonry
Elasticity modulus	$E = 2.1 \times 10^5 MPa$	$E = 2.5 \times 10^4 MPa$	$E_m = E_1 = E_2 = 7.06 \times 10^3 MPa$
Poisson's ratio	$\nu = 0.30$	$\nu = 0.20$	$\nu = 0.00$
Yield criterion	<i>von Mises</i>	<i>Mohr Coulomb</i>	<i>Mohr Coulomb</i>
Damage thresholds	$\sigma_c = 420 MPa$	$\sigma_c = 25 MPa$	$f'_{mtx} = 0.35 MPa$
	$\sigma_c = 420 MPa$	$\sigma_c = 3.5 MPa$	$f'_{mcx} = 5.15 MPa$
			$f'_{mty} = 0.60 MPa$
			$f'_{mcy} = 12.0 MPa$
Fracture energy	$G_c = 2 \times 10^3 kPa \cdot m$	$G_c = 50.0 kPa \cdot m$	$G_c = 20.0 kPa \cdot m$
	$G_t = 2 \times 10^3 kPa \cdot m$	$G_t = 5.0 kPa \cdot m$	$G_t = 0.5 kPa \cdot m$
Self-weight	$\gamma = 7845 kg/m^3$	$\gamma = 2500 kg/m^3$	$\gamma = 1500 kg/m^3$

TABLE 4. Mechanical properties of steel

ds	Location L1	Location L2
$P_{(ds_1)}$	8.65%	99.65%
$P_{(ds_2)}$	0.00%	95.02%
$P_{(ds_3)}$	0.00%	73.04%
$P_{(ds_4)}$	0.00%	22.12%

TABLE 5. Probability of damage occurrence.

Model	F.E.	Initial Memory Req.	S.M. Info	Solver
B-OSC	30,868	572.70Mb	262.22Mb	243.62Mb
B-SSC	1'105,804	28.599Gb	9.82Gb	14.599Gb

TABLE 6. Memory comparison between models B-OSC and B-SSC.

Model	Initial Memory Req.	Final Memory Req.
B-OSC	572.70Mb	329.4Mb
B-SSC	28.599Gb	19.00Gb

TABLE 7. Memory comparison between models B-OSC and B-SSC using the proposed scheme and the direct solver PARDISO.

Model	Initial Memory Req.	Final Memory Req.
B-OSC	572.70Mb	132.51Mb
B-SSC	28.599Gb	4.75Gb

TABLE 8. Memory comparison between models B-OSC and B-SSC using an iterative solver of library FEMT.

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Fig. 3. Relation between the fictitious isotropic and the real anisotropic space.

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