COMPDYN 2015
5th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Crete Island, Greece, 25–27 May 2015

A *MULTISCALE* APPROACH FOR MODELING OF INFILLED FRAMES

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Keywords: Existing Building, Seismic Assessment, Reinforced Concrete, Infilled Frame, Equivalent Strut, Meso-scale, RBSM model.

Abstract. *The behavior of existing reinforced concrete buildings towards to the seismic actions is strongly affected by the infill walls. Although they assume an important role in the global response of the buildings, modeling of infilled frames is quite complex as a result of the many variables involved. These factors (such as the various constructive solutions; the non-homogeneous and anisotropic mechanical behavior of the masonry texture; the interaction between masonry panel and surrounding frame; the in-plane and out-of-plane behavior) make uncertain the structural response.*

The technical literature on the issue suggests approaches that ranging from the macro-scale analysis to micro-scale analysis. The different proposals confirm the complexity to simulate the kinematic mechanism between elements of the panel (mortar and bricks) and between infill panel and surrounding frame. Furthermore, these different models cannot be applied in general. They are most of the times calibrated on experimental tests carried out on samples packaged with materials and executive methods that are specific of the test place. Therefore, there is the need to define simplified models that take into account the geometry, mechanical properties of the masonry walls and elements of the surrounding frame.

The present study proposes a meso-scale approach (a "Rigid Blocks Method") for simulating of the infilled frames in order to define the main parameters that characterize the simplified "Equivalent Strut model" (i.e. the macro-modelling of the infill walls). With the meso-scale model the infill wall is supposed as a mechanism consisting of rigid blocks connected by axial and shear elastic-plastic springs. The model is usually used for in-plane dynamical analysis of masonry structures. The use of an equivalent rod for modeling the infill panels is a simplified approach purely numeric. It is simple and reduces the computational costs when it is need to evaluate the global behaviour on building scale. The results achieved with the proposed approach are as result of extensive numerical tests on samples of infilled frames, with or without openings, which have geometries and material properties within significant ranges.

1 INTRODUCTION

Over the years, the variability exhibited by the infilled frames (in particular for buildings designed under vertical loads only) has strongly stimulated researchers to investigate the interaction between masonry panels and reinforced concrete elements [1,2]. These studies are important in order to establish the retrofit interventions needed to ensure the seismic safety to the deficient buildings [3,4]. The role of the experimental tests is undoubtedly fundamental for the knowledge of the actual response. However, the great variability of the significant parameters involved in the interaction frame-infill does not allow to carry out tests whenever they are needed because the tests are expensive and usually limited to few cases. A valid alternative for the knowledge is provided by research study based on numerical modelling of infilled frame. The several studies available, however, present a great degree of uncertainty due to the high variability of the significant involved parameters. Among these, the mechanical properties of the constitutive materials (mortar and bricks) and of the global masonry texture (intended as a continuous material) are crucial. In addition, another fundamental aspect is represented by the model adopted for simulating the infills within the computational model of the building [5,6,7,8].

Consequently, on the basis of the framework above outlined, the present thesis has the aim to reduce the influence of some variables by proposing a simplified procedure for the calibration of the *Equivalent Strut* model. In the field of the approaches of macro-modelling, this model is the most used to evaluate and simulate the effects of the infills in the global response of the frame. It describes the response of infilled frame under a horizontal load observed from the experimental test. In particular, by increasing of the thrust action, it is observed the detachment of the masonry from the boundary reinforced concrete frame, and the panel exhibits a behaviour no more in line with a two-dimensional element (that it resists to shear stress) but as an "*one-dimensional equivalent rod*". The rod follows the distribution of the compression forces along the diagonal of the panel acting as a *strut* in opposition to the horizontal action. The equivalent strut is defined once known the geometrical and mechanical characteristics and the constitutive law (force-displacement) that regulates its behaviour during the nonlinear phase (post-elastic). The above parameters are dependent by experimental results, therefore, especially in the case of existing structures, they can not be generalized to all possible cases and are reliable only in some cases that are comparable with the characteristics of the test from which derive.

The aim of the present work is to define the Force-Displacement constitutive law of the *equivalent strut*. The law is obtained by a simplified procedure that use the results of a simplified discrete model implemented within common finite element software (*SAP2000*). In order to define the model into software are needed few input data as the geometry of the infilled frame and mechanical parameters of the masonry panel. The model numerically simulates an experimental test by which to evaluate the actual variations in stiffness and strength of the infill-frame system. The purely numerical findings allow to define the characteristic branches of the law, which is the force-displacement relationship (F_w - d_w) that regulates the elastic and plastic behaviour of the strut. Although the transition from *meso-scale* to *macro-scale* analysis could seem a loss in the description of the global response of the infilled frame, it is actually offset by the possibility to study three-dimensional structures, which usually become, with a *meso-scale* approach, more computationally burdensome. Moreover, the simplification to insert simple beam elements for simulating the infill panel, involves a low degree of uncertainty. This advantage is due to use of constitutive laws that are defined on the actual masonry texture and not based on the experimental tests from the literature conducted on panels mechanically different. In conclusion, the procedure is proposed with the aim to improve the *equivalent strut method* which is able to assess the effects of interaction between the infills and primary RC elements on building-scale analysis.

2 A SIMPLIFIED PROCEDURE TO ASSESSING THE EFFECTS OF INFILL PANELS

2.1 The *meso*-modelling

The "*meso-scale*" models are usually referred to as "*discrete models*". If they are applied to structural systems that require a limited number of elements, they allow to reduce the computational burden required by the continuous or discontinuous approaches. The *Rigid Body Spring Model* (RBSM), used in the proposed procedure, belongs to this type of models.

The basic idea of the method (Figure 1) consists in describing the masonry (intended as a composite periodic material denominate RVE - *Representative Volume Element*) as a mechanism composed of the unitary cells constituted by of rigid blocks and elasto-plastic springs [9,10,11] The elastic features of the springs are defined by means of a specific identification procedure with the objective of transferring the information about the main characteristics of the masonry texture from the "*meso-scale*" to "*macro-scale*".

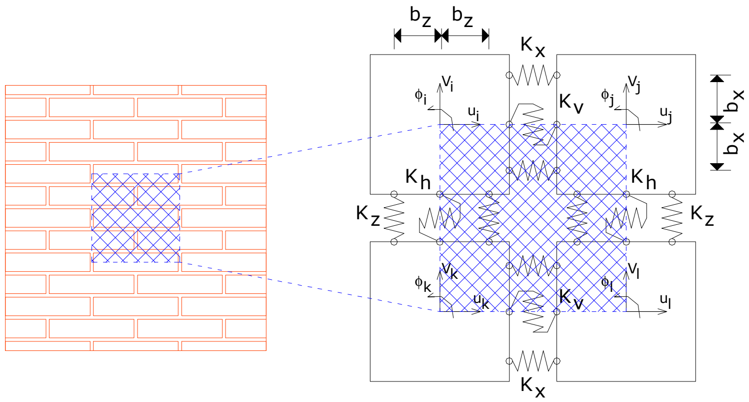

Figure 1: Scheme of a regular masonry texture and an example of the unit cell consists of four rigid blocks connected by elasto-plastic springs.

The connection between the rigid blocks occurs by means of two axial springs (indicated with K_x or K_z, depending on the direction) and a shear spring (K_h, K_v , depending on configuration) arranged along each of the common sides between the blocks at a distance equal, respectively, to b_x and b_z. The hysteretic behaviour of the springs is obtained on the basis of the mechanical parameters of the constitutive elements of the panel (mortar and blocks) and takes account of the mechanical degradation of mortar joints in the cycles of loading and unloading.

2.2 Constitutive law of the springs

In the original version of the model, the stiffness of the springs and the distance between the connection points are assigned with the criterion to approximate the average strain energy in correspondence of the reference volume of each spring. Assigning to the shear springs different stiffness in horizontal and vertical direction, together with a calibration of the distances between the connections, it is possible to model the effects of some masonry texture (for example, stone blocks interlocking often present in the case of significant degradation of the masonry that significantly changes the resistance of the panel). The constitutive laws for elastic-plastic behaviour are assigned using a phenomenological approach [12], based on the results of cyclic tests available in the technical literature [13,14,15].

For the purposes of this thesis, the use of a *meso-scale* modelling is an easy and quick step within the proposed procedure, aimed at defining some characteristic features of the global response of the infilled frame. For these reasons the properties of the springs are defined according to a simplified approach. The elastic stiffness of each spring is estimated through the geometrical and mechanical properties of the infill (Figure 2), such as the *modulus of elasticity* of the masonry panel in horizontal direction (E_{wh}), *modulus of elasticity* of the masonry panel in vertical direction (E_{wv}), *shear modulus* (G), areas of influence of the axial spring (A), areas of influence of the shear spring (A_T), thickness of the head joint (g_t), thickness of the bed joint (g_l) and thickness of the masonry panel (t_w)

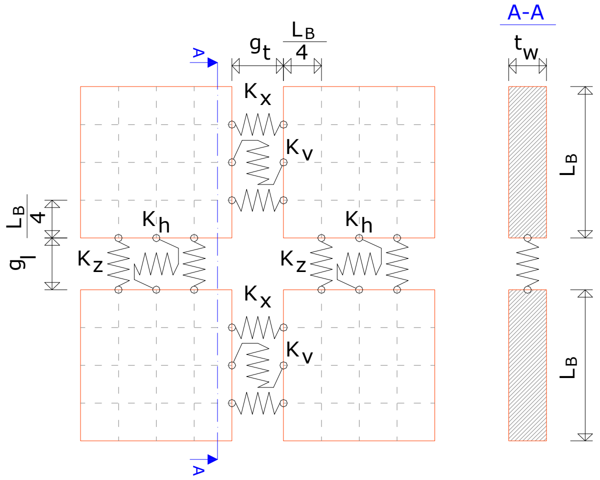

Figure 2: Simplified periodical cell describing the masonry texture.

When the kinematic of the model is such as to engage the springs over the elastic response, force-displacement laws regulate the plastic behaviour of the system. As with the elastic behaviour, also for the decreasing behaviour of the stiffness has been adopted a simplified approach. Figure 3 shows the constitutive laws of the springs, respectively, axial and shear, used in the present work.

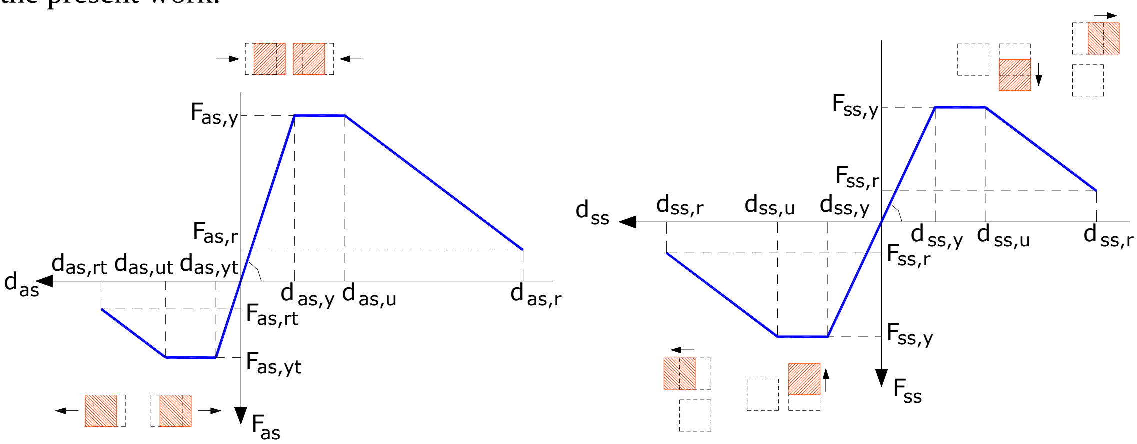

Figure 3: Constitutive laws of the axial and shear springs.

For both, the force-displacement law is constituted by *3* branches. In the figures, the subscript "*as*" indicates the axial spring, while “*ss*” the shear spring. The fundamental variables of the laws are calibrated with an approach depending on the material strengths: compression strength of the masonry in horizontal direction (σ_{wh}), compression strength of the masonry in vertical direction (σ_{wv}), tensile strength of the masonry (σ_{wt}), shear strength of the masonry along the bed joints (τ_{wh}), shear strength of the masonry along the head joints (τ_{wv}).

2.3 Description of the simplified procedure

First step: *"meso-analysis of the infilled frame"*

The first step of the procedure is to assess the response of the infilled frame by a nonlinear static analysis (pushover). The behaviour of the masonry panel is simulated by the model described in the previous section. The reinforced concrete frame is instead modelled with *beam* elements whose response is described by a *lumped-plasticity model*. According to this, the post-elastic behaviour is modelled by introducing plastic hinges, in which all non-linearity is localized at the ends of the elastic beams. The non-linearity of the plastic hinge is defined by a M-ϕ or M-θ relationship, depending on the shear span L_V, which identifies the distance between the end section of the element and the inflection point of the deformed shape (which varies during the incremental pushover loading). The relation adopted in the present work is contained within US standards FEMA 273 [16] which defines a relationship between the dimensionless strength (Q / Q_{CE}) and rotation θ (or displacement Δ) for respectively beams and columns. In the case of existing structures, the evaluation of plastic hinges have to take into account the effective strength values of materials in according to numerical approaches (in this context interesting are the research developed by authors [17,18]. Assuming as a *control point* of the pushover analysis the application point of the F_h load, the two main results at this *step* are, respectively, the *capacity curve* of the configuration with infill (indicated hereinafter with the acronym "INF") and the response of the springs in terms of forces and displacements.

Second step: *"evaluation of the infill conribution without RC frame"*

The F_h-d_h curve of the INF configuration is intended as the "*sum*" of two distinct contributions, respectively, due to the bare frame (B) and the masonry panel (MP). The first contribution is evaluated using a nonlinear static analysis on the model without infill panel. The second, however, is obtained by subtracting (in correspondence with each displacement value) from the response of the INF configuration, the share due to the bare frame B (Figure 4).

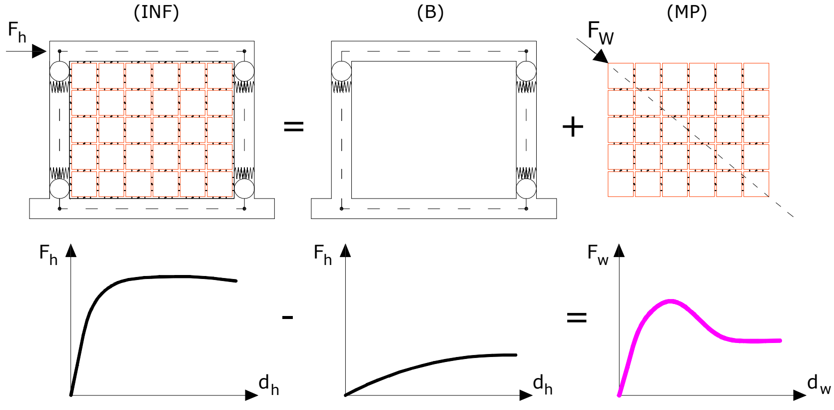

Figure 4 - Graphical description of the simplified evaluation of the masonry panel response.

With the above assumption is obtained the relationship between the horizontal thrust load ΔF_h^{MP} (equal to F_h^{INF}- F_h^{B}) and the horizontal displacement d_h of the wall panel only. The ΔF_h^{MP} - d_h curve is successively expressed according to F_w^{MP} and d_w^{MP} components along the d diagonal of the panel, by means of numerical expressions based on the geometry of the infilled frames.

Third step: *"Evaluation of the constitutive law of the Equivalent Strut"*

Once obtained the response F_w^{MP}- d_w^{MP} for masonry panel, it's possible to evaluate the constitutive law of the equivalent strut on the backbone of MP curve in order to define the ESM model. The law that will be defined consists of four branches delimited by four characteristic points identified on the MP curve. The first branch describes the elastic response up to the yield point "*S*". The second branch describes the behaviour of the equivalent strut from when begins the plastic deformation up to achieve the point of maximum strength capacity "*M*". This is followed by the descending branch due to the propagation of crack in the panel such that induce a significant decrease of resistance until to the point "*D*". Finally, the fourth branch describes the force-displacement relation until the failure (point "*R*") with a residual resistance maintained in order to ensure stability and convergence to the numerical solution (Figure 5).

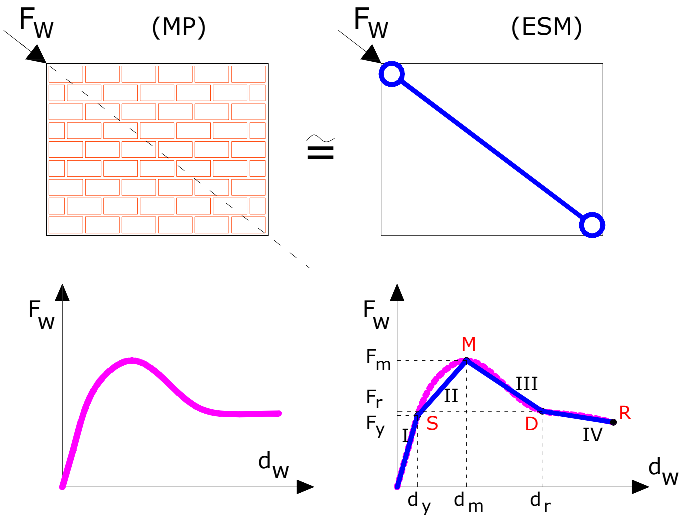

Figure 5: Graphical equivalence between MP and ESM models for the evaluation of F_w-d_w law.

The *M* and *R* points are preliminarily identified and coincide, respectively, with the maximum point and last point of the MP curve. While, *S* and *D* points are directly evaluated by an iterative approach that minimizes the difference between the subtended areas of the MP curve and the defined constitutive law.

The width (*w)* of the *equivalent strut* is evaluated neglecting the degradation of stiffness and the uncracked panel. The used equation derives from a purely linear elastic relation considering the yield point *S* on the MP curves. The width *w* is obtained by the following balance equation:

$$\frac{F_y}{d_y} = \frac{E_{w\theta} t_w W}{d} \tag{1}$$

where $E_{w\theta}$ is the modulus of elasticity of the masonry in the diagonal direction [19].

$$E_{w\theta} = \left[\frac{\cos^4\theta}{E_{wh}} + \frac{\sin^4\vartheta}{E_{wv}} + \cos^2\theta\sin^2\theta\left(\frac{1}{G} - 2\frac{\nu}{E_{wv}}\right)\right]^{-1} \tag{2}$$

3 CASE STUDY

3.1 Characteristic of the sample and set-up of experimental test

The case study of the present work, indicated by the abbreviation "SIB", is part of a broad experimental campaign conducted by Cavaleri and Di Trapani at the laboratory of the University Palermo, Italy [20]. The experimentation involved eight infilled frames (S1 series) with slenderness $L_w/h_w = 1$ designed to represent a typical configuration recognizable in existing buildings designed for gravity loads and without any seismic detail. Two specimens identical of the S1B sample, respectively, S1B-1 and S1B-2, were packed. The Figure 6 reports the geometrical details of the sample.

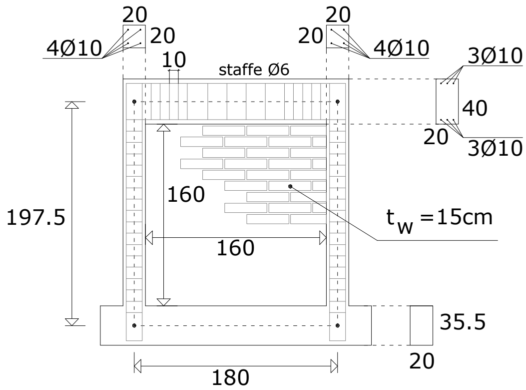

Figure 6: Geometry and reinforcement of the sample.

The mean concrete strength, measured after *28* days, was *25MPa*, while the elastic Young modulus was about *25500MPa*. The reinforcement steel bars had a medium strength of *450MPa*. The mechanical characteristics of the infill is defined on the basis of compression tests, lateral and diagonal preliminarily performed, while further loading tests were performed on bricks and mortar. The results of this preliminary experimental campaign are widely discussed in Cavaleri *et al.* (2012) [21]. The infill panel were constructed by placing the bricks with vertical holes and parallel to the direction of the static loads acting on the columns. The thickness of the mortar joints is equal to *15mm*. Significant data for implementation of the meso-model of the sample are listed in Table 1.

E_{wv}	6401	MPa
E_{wh}	5038	MPa
G	2560	MPa
σ_{wv}	8.66	MPa
σ_{wh}	4.18	MPa
τ_{wv}	0.30	MPa
τ_{wh}	0.31	MPa

Table 1: Mechanical properties of masonry employed for S1B-1 specimen.

3.2 Experimental results

The specimens of infilled frames were tested by increasing the displacement at each cycle up to a drift about *2.5 % - 3.0 %*. Damage mechanisms were monitored during the tests in order to detect propagation of cracks on infills and frames. Stiffness, strength, and ductility evaluations were carried out by observation of force-displacement cycles. A low strength degrading after peak reaching was observed for all specimens, demonstrating an efficient confinement effect produced by the frames on the infill panels. The first cracking, which have involved both frames and infills of both specimens, have arisen from a displacement of *10 mm* (drift of about *0.63%*), in particular, diagonal cracks occurred at the beam-column joints, while along the mortar joints the cracks are distributed according a typical propagation "*stair-step*". For larger displacements, over *20 mm* (drift of about 1.25%), more evident cracking propagation, corresponding to the beginning of strength decay was observed accompanied by the formation of sub-horizontal cracks in the middle of columns and more severe damage at beam column joints. However, both specimens exhibited a significant stiffness degradation at each cycle, especially after the peak strength was reached. On the other hand the peak strength (in particular for S1B-1 specimen) did not significantly decrease until large displacements occurred. By ideally enveloping the force-displacement cycles measured by the tests with a monotonic curve (in red) global ductile behaviour was observed (Figure 7).

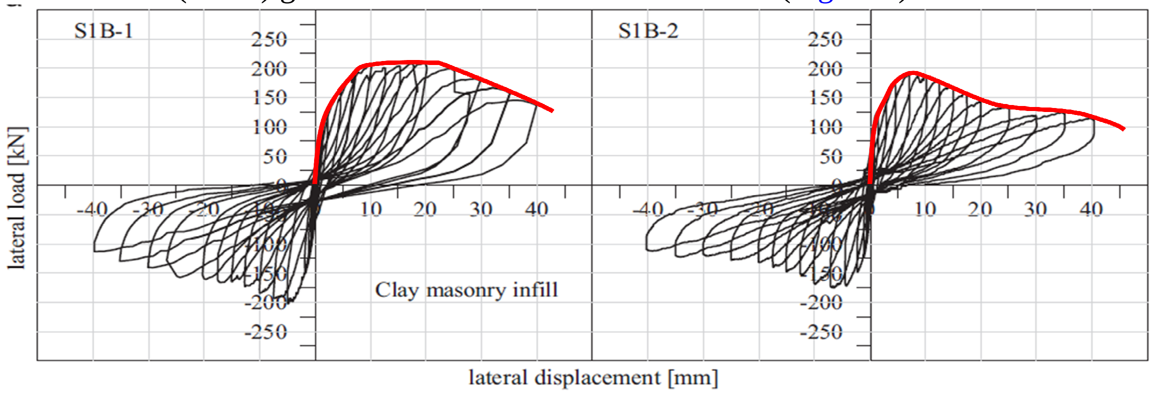

Figure 7: Force–displacement cyclic test results for specimens of S1B series.

3.3 Numerical Modeling

In order to simulate the experimental tests, has been implemented in *SAP2000 software* a finite element model of the S1B sample. The numerical model is constituted by a set of discrete elements (blocks and spring) enclosed by a contour of the beam elements having the function of describing the behaviour of the reinforced concrete frame. The rigid blocks spring model (RBSM) is constituted by a regular discretization of 6 x 6 two-dimensional plane elements (*4*-nodes *shell* elements). The total degrees of freedom are 108. The behaviour perfectly rigid of blocks is guaranteed by assigning to *4*-nodes of each *shell* a "*Diaphragm*" internal constraint with Y rotational axis perpendicular to the OXZ reference system. The internal constraint connects the nodes of the shell element by means of ideal rigid rods devoid of mass and stiffness. The connections between rigid blocks is with "*NLlink*" elements of "*multilinear plastic*" typology, i.e. nonlinear springs (on the right of the Figure 8 are highlighted, respectively, the axial spring - in red- and the shear spring - in blue). At the interface between frame and masonry panel (i.e. between beam and *NLink* elements along the sides of the rigid blocks adjacent to the frame), are arranged *Rigid* typology. The *rigid NLink* maintain unchanged the relative distances between the nodes on axis line of the reinforced concrete element and the boundary line of the concrete, in order to simulate the material between them. The model is constrained to the ground by means of interlocking constraints at the base columns, while for simulating the position of specimen in the test device, the hinge constraints at the end nodes of the *rigid NLink* elements were arranged.

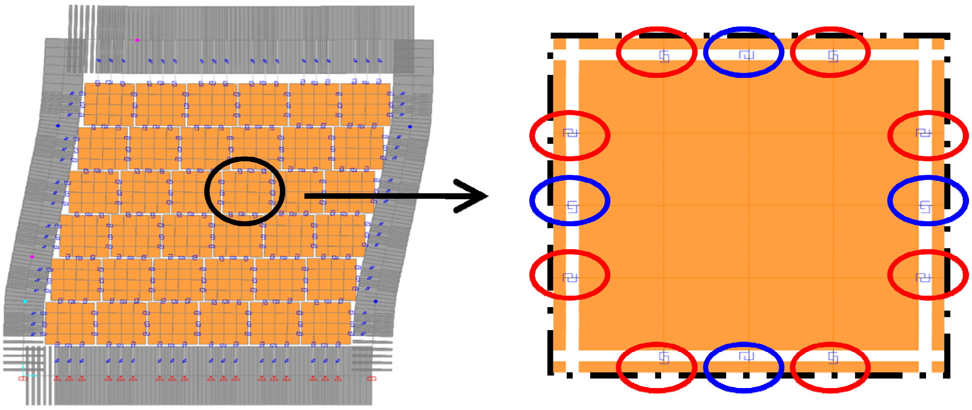

Figure 8: Response of the Finite Element Method (left); extrude view of the single rigid bloc and his connections.

4 RESULTS

The simplified procedure described in *§ 2* is applied to the S1B sample. By assuming the application point of the horizontal load as the *control point*, nonlinear static analysis (push-over) on the discrete model with blocks and springs and the frame model without infills were performed. The capacity curves of two configurations are indicated hereinafter with the abbreviations, respectively, INF and B. Applying the first two steps of the procedure is obtained the relationship between the horizontal thrust load ΔF_h^{MP} and the horizontal displacement dh for the building panel MP only (Figure 9). In particular, ΔF_h^{MP} was evaluated with a simplified approach by the followed numerical difference: F_h^{INF}- F_h^B. The F_h^{MP} - $d_h^M P$ relation is then expressed according to the F_w^{MP} and d_w^{MP} components along the *d* diagonal of the panel.

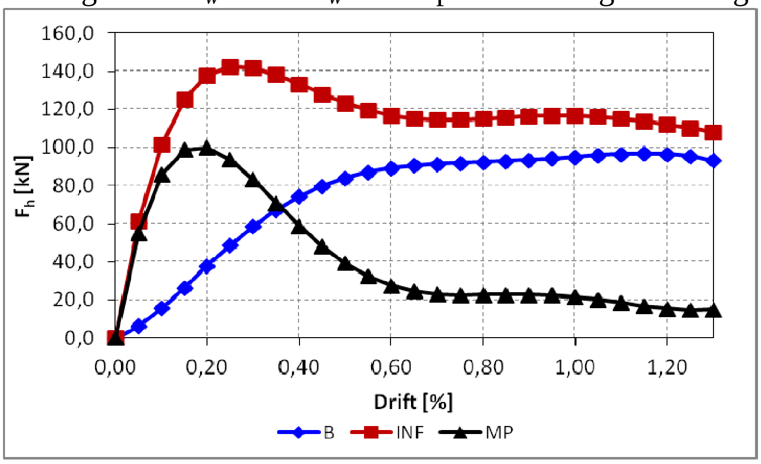

Figure 9: The assessment of the Force-Displacement Capacity Curve of the Masonry Panel (MP) - first and second step of the proposed procedure.

The *4* branches of the constitutive law of the equivalent strut are directly built on the backbone of the MP curve (Figure 10). In order to determine the law more easily two points are directly fixed on the MP curve. In particular, M and R points are, respectively, equal to the maximum point and last point of the MP curve. The yield strength (S point) and the end point of the first degrading branch (D point) are defined by iteration by minimizing the difference between the areas subtended by the MP curve and the constitutive law. The length of the strut is evaluated by Eq. 1 and is equal to *245mm*.

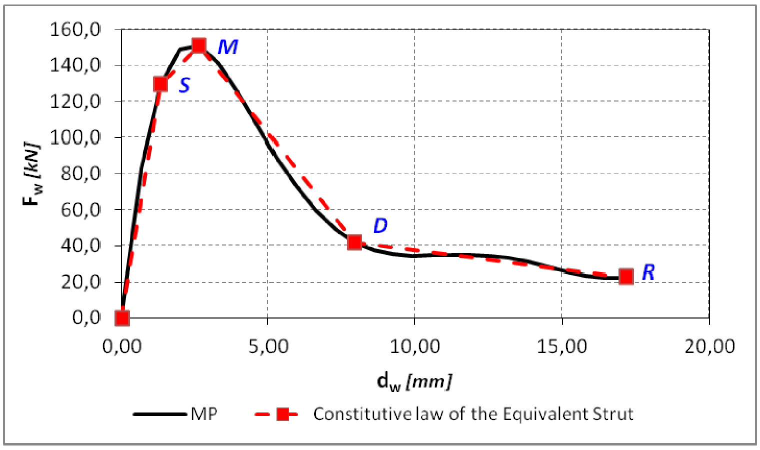

Figure 10 - Graphical definition of the Constitutive Law on the MP curve.

Figure 11 shows the force-displacement curves of the B and EMS configuration (respectively without infill and with equivalent strut), together with the monotonic envelopes of the cyclic responses obtained by tests carried out on both specimens of the S1B sample.

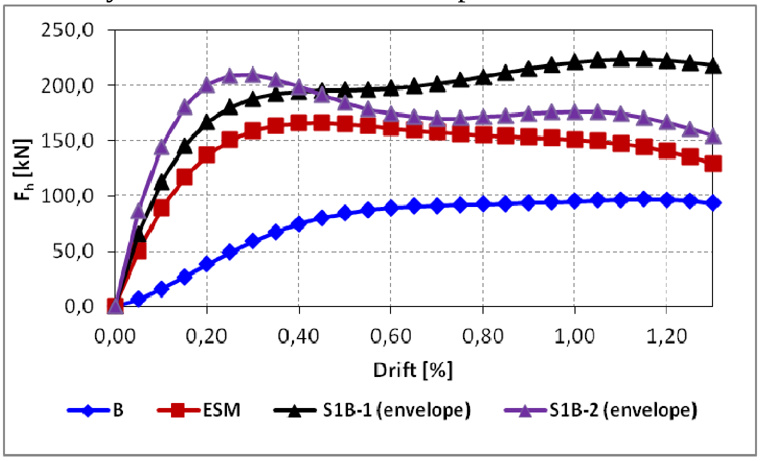

Figure 11 - Comparison between numerical and experimental capacity curves

The comparison shows the expected increases in resistance due to the contribution of the infill. In order to quantify this increase, the following *incremental factor* is defined:

$$\delta_S{}^{(i)} = \frac{F_h^{(i)}}{F_h^{(B)}} \tag{3}$$

with the *i* apix for indicating the *i*-th curve reported in the comparison.

The function $\delta_S = \delta_S(drift)$ for the S1B sample (intended as the function of the average between the increments of both specimens) and for the *Equivalent Strut Model* (EMS) are inversely proportional to the increase of drift. Specifically, starting from a drift about of *0.4%*, i.e. when the specimens still show an elastic behaviour, the increases of the strength than the bare frame, are constant and tend to value equal to 2. The average percentage error in assessing the response of the S1B sample with the ESM model is above 21%

5 CONCLUSIONS

It should be highlighted some aspects which characterize the whole conceptual framework of the proposed procedure.

- The definition of the constitutive laws of the springs is essential to achieve similar results to those observed experimentally. The use of the simplified modelling RBSM, and of the entire procedure proposed, is more reliable if the initial data relating to the mechanical parameters of the materials are sufficiently detailed (elastic modulus, compressive strength ...). The degree of reliability increases further when the above parameters are derived directly from experimental laboratory tests on specimens preliminarily packaged for the purpose (tests on walls), as well as observed in the case study.
- The calibration of the simplified model RBSM becomes less "reliable" when are little the information about the mechanical properties and resistance parameters of the infill walls. In particular, it is more difficult to define the laws of the springs. The springs greatly influence the results obtained by the procedure, therefore, if the information required for their definition are few and uncertain, definitely has an impact on the model.
- The results obtained, both preliminarily on the RBSM in its simplified form, both on the model with equivalent strut, show the significant contribution in strength provided by infills.

ACKNOWLEDGEMENTS

The research presented in this article was partially funded by the Department of Civil Protection, Project ReLUIS-DPC 2014-2018

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