NUMERICAL ANALYSIS OF A REINFORCED CONCRETE FRAME WITH MASONRY INFILL UNDER SEISMIC LOADING

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ABSTRACT

The paper is mainly focused on the investigation of the seismic response of reinforced concrete frames with masonry infills. Numerical results obtained with a strut model built in ETABS 2016 software are compared with the experimental results obtained on a full-size model. Tests were conducted in 2018, at the Technical University of Civil Engineering of Bucharest, on a reinforced concrete frame with masonry infills and on a bare reinforced concrete frame under compressive and shear loading. A static pushover analysis method was selected, and results were fairly compatible, even though some differences in the initial stiffness, as well as in the post-yielding stage, were observed between numerical and experimental results. The model is presented, proposing a simple engineering method to be used in calculations.

Keywords: Seismic response; masonry infills; cyclic loading; reinforced concrete structures.

REZUMAT

Articolul se concentrează în principal asupra investigării răspunsului seismic al cadrelor din beton armat cu umpluturi de zidărie. Rezultatele numerice obţinute cu un model construit în programul ETABS 2016 sunt comparate cu rezultatele experimentale obţinute pe un specimen executat la scară reală. Încercările au fost efectuate în anul 2018, la Universitatea Tehnică de Construcţii din Bucureşti, pe un cadru de beton armat cu umpluturi de zidărie şi pe un cadru gol din beton armat sub încărcare de compresiune şi forfecare. A fost selectată o metodă de calcul static neliniar, iar rezultatele au fost compatibile, chiar dacă s-au observat unele diferenţe în rigiditatea iniţială, precum şi în stadiul inelastic, între rezultatele numerice şi cele experimentale. Este prezentat modelul şi se propune o metodă simplă inginerească pentru a fi utilizată în calcule.

Cuvinte cheie: răspuns seismic; umpluturi din zidărie; încărcare ciclică; structuri din beton armat.

1. INTRODUCTION

Reinforced concrete (RC) frames with masonry infills are very popular worldwide, due to both strength and economic reasons. They have been also used for reconstruction in areas affected by seismic events, such as Haiti after the 2010 earthquake (Miyamoto, 2020), when the investigation teams realized that although most of the buildings were made of RC frames, they were so poorly executed, without respecting any anti-seismic rules, that their behavior produced a humanitarian crisis, killing thousands of people.

Thus, RC frames with infills, although having the potential to resist earthquakes, may be vulnerable if they are not correctly designed and executed.

This reason led to the experimental investigation of such configurations, the way they were executed in developing countries. Within a project carried out at the Technical University of Civil Engineering Bucharest, Romania (UTCB), under the supervision of Dr. Matsutaro Seki (Japan), RC frames with and without masonry infills were executed and tested under combined vertical and lateral loading. A retrofit solution, using wire mesh, was applied as well on one of the frames (Seki et al., 2018).

For the study presented in this paper, within an initial numerical investigation, a numerical model was built in ETABS 2016 to simulate the experiments on RC frames with and without the clay brick masonry infill.

2. LITERATURE OVERVIEW

Masonry infills are usually the first affected, even in case of moderate earthquakes, when it is common to observe a diagonal x-shaped crack pattern between the opposite corners of the infill panels. This is basically due to their large in-plane stiffness, combined with their small in-plane diagonal tensile strength.

The seismic performance of RC frames with masonry infills throughout past earthquakes, like Ecuador 2016 (Lizundia et al., 2016), Albania 2019 (Sextos et al., 2020), Samos Island 2020 (Cetin et al., 2020), showed specific types of damage: x-cracks in the infill panels, flexural and shear cracks in adjoining RC columns, out-of-plane failure of infills, interface cracking at frame-boundary interfaces, concrete cover spalling and buckling of longitudinal reinforcement in columns.

Several researchers studied experimentally, in a cyclic static testing regime, the influence of the masonry infills on the seismic behavior of RC frames. Ning et al., (2019) carried out in-plane tests on RC frames with aerated lightweight concrete (ALC) blocks infills, with and without openings, and found that the infills are shifting the inflexion points towards the top of the columns and, as well, that the openings reduce the influence of the infills on the RC frame. Failures were observed in the column and beam hinges for the frame without infills, while shear failure appeared at column ends in frames with infills. Dautaj, Kadiri, and Kabashi (2018) also tested, for in-plane loads, several specimens of RC frames of different strengths, with and without hollow and solid clay brick masonry. They found that shear failure of the masonry infill and beam column joint was predominant for hollow clay brick masonry infills, while for the solid ones shear failure of the column and the masonry infill was observed. Anić et al. (2021) conducted out-of plane tests on RC frames with and without infills, noticing that neither the infill walls nor the openings significantly affected the overall behavior of the specimens; however, for large drifts, the damage of the infills may threaten life safety. Angel (1994) carried out both in-plane and out-of-plane tests. Among other conclusions, they observed that the shear strength of masonry was affected by the type of mortar used and that the lateral stiffness was directly proportional to the compressive strength of masonry, for the in-plane tests. For the out-of-plane tests, the strength greatly depended on the slenderness ratio and on the compressive strength of the masonry, and not on the tensile strength as one would have expected. In-plane cracking reduced the out-of-plane strength of the slender panels by a factor as high as two.

Simplified evaluation methods and numerical models were proposed for this type of buildings (Madan et al., 1997, Stavridis and Shing, 2012, Hak et al., 2016, Alwashali et al., 2019) and the main governing parameters were investigated. Even if, in the classic evaluation methods, the infills are simply disregarded when considering their influence in the overall behavior, such a method tends to be abandoned, given that - as observed in real earthquakes - the stiffness and ductility are significantly influenced by the presence of infills. Bagnoli et al. (2022) also presented arguments towards this influence of the infill, which can be both positive (adding stiffness) and negative (threatening life, when damaged).

Many researchers attempted to find an optimal solution to retrofit the RC frames with masonry infills that are vulnerable to earthquakes. For example, in (Griffith, 2008), after a thorough literature review, the conclusion was that, at least in the Mediterranean area, the buildings with masonry infills are expected to experience a relative lateral deformation (drift) of maximum 2%. A number of retrofit solutions were studied and recommended, such as: replacing the infill with damped bracing, strengthening the masonry and columns by jacketing, removing the infill and jacketing the columns and joints, and seismically isolating the superstructure. The effectiveness of these solutions also depends on the country where it is applied, since workmanship and the quality control of execution works are very important factors. In developing countries it may be difficult to apply sophisticated solutions, thus other solutions could be considered, such as applying wire mesh on the infills surface, and anchoring it into the RC frame (Sen et al., 2020). Moreover, for these countries, simplified numerical analysis could be a useful tool for the evaluation of existing buildings with RC frames and masonry infills. As in the case of retrofit solutions, if the model is too sophisticated, it could be found as difficult to use by practicing engineers.

3. EXPERIMENTAL PROGRAM

3.1. Introduction

An experimental research program was recently conducted in the laboratory of the Technical University of Civil Engineering Bucharest, in order to identify cost-effective retrofitting solutions that could be easily applied for the retrofitting of weak concrete frames in developing countries (Seki et al., 2018). Some key details of the program are presented in the following, with reference to the two specimens that are further analyzed in the present paper.

The above-mentioned test specimens were constructed of normal strength concrete with an average compressive strength of 14 MPa. The masonry infill was made of lime-cement mortar with 6.7 MPa average compressive strength and of solid burnt-clay bricks (240 × 115 × 63 mm). The construction materials were chosen such as to match as closely as possible the corresponding practice in developing countries.

The experimental program was accompanied by an analytical study, aimed to investigate the best practices for modeling and analyzing the specimens, for an improved agreement with test results.

3.2. Description of test specimens

Specimen layout and dimensions are presented in Figures 1 and 2. Both specimens consisted of a reinforced concrete frame, which for specimen S1-F was left without infill, while for specimen S3-FM was infilled with clay brick masonry. For the columns, the reinforcement ratio was 0.6%, and for the beams, the ratio was 0.3%. This reinforcement ratios were chosen such as to reproduce the low performance of RC frames in Bangladesh (Seki et al., 2018). The masonry infill was done in accordance with the Romanian practice. Both mortar and bricks were tested, more details being given in (Seki et al., 2018).

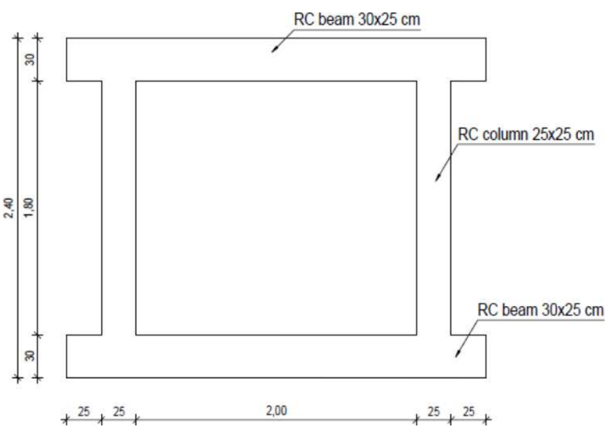

Fig. 1. Bare reinforced concrete (RC) frame specimen (S1-F) (Seki et al. 2018)

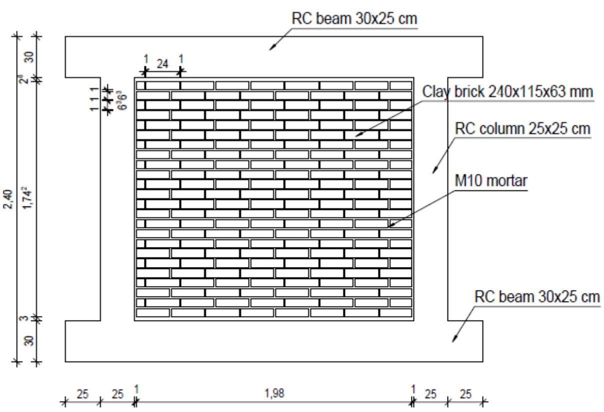

Fig. 1. Reinforced concrete (RC) frame specimen with clay-brick infill (S3-FM) (Seki et al. 2018)

3.3. Test setup

Tests were conducted according to the loading protocol shown in Figure 2. The tests were of cyclic static type, being performed on the reaction frame of UTCB, donated by the Japanese International Cooperation Agency, JICA (Fig. 3).

The axial load was calculated as $0.4F_c$, minus the weight of the pantograph, where F_c is the average compressive strength obtained from cylinders and multiplied by the area on which the axial load is applied. The setup details are fully presented in (Seki et al., 2018).

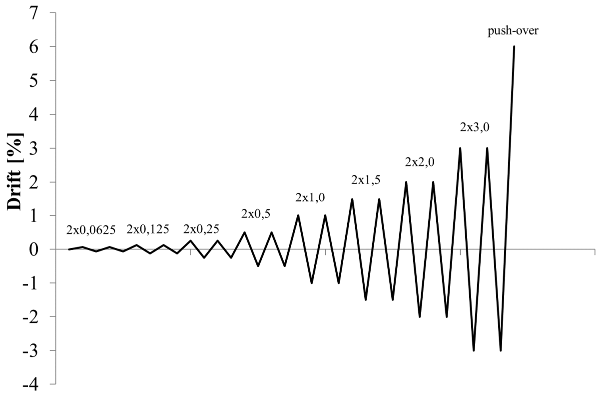

Fig. 2. Loading protocol (Seki et al., 2018)

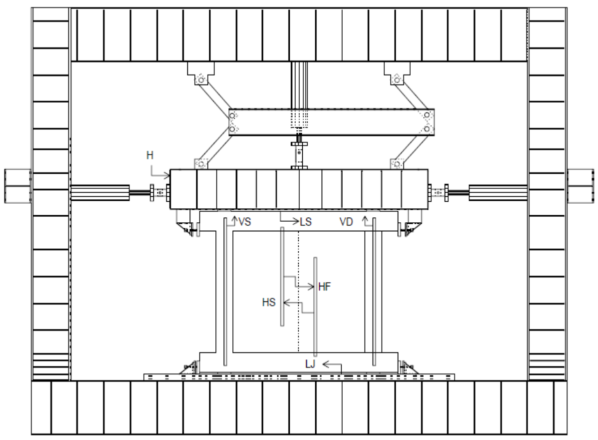

Fig. 3. Test setup on the reaction frame (Seki et al., 2018)

3.4. Test results

The actual experiment was performed by the original authors of the cited paper, on multiple configurations, whereas the analytical model presented in the next section has been done based on a single configuration.

For specimen S1-F (frame without infill), the maximum capacity was 81 kN, a value attained for 1% lateral drift. The failure occurred at -2% drift.

The experimental curve is shown in Fig. 4.

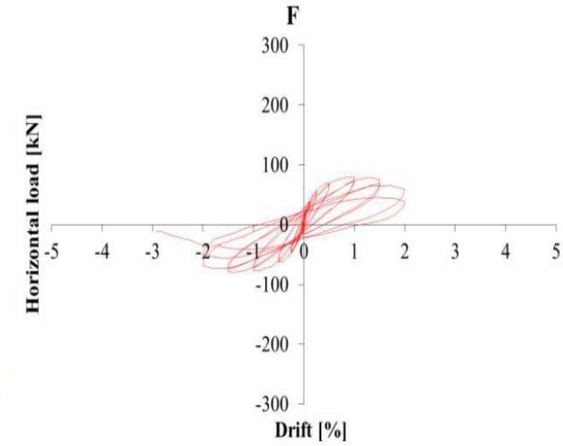

Fig. 4. Hysteretic curve of specimen S1-F (Seki et al., 2018)

The RC frame with masonry infill (S3-FM) specimen failed through shear at the bottom of the columns, followed by the transfer of the axial force to the masonry infill (Fig. 5), at around 3% drift. The capacity of this specimen was 190 kN, corresponding to -1% drift (Seki et al., 2018).

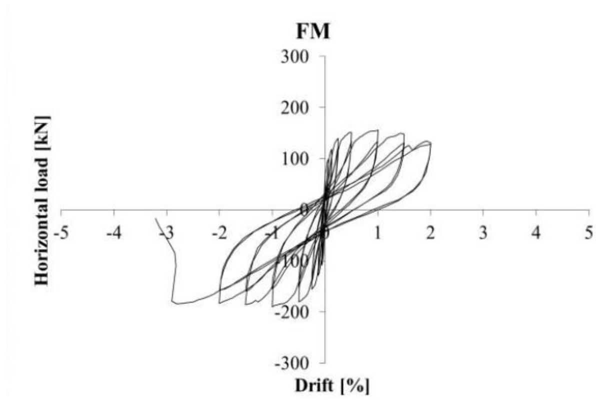

Fig. 5. Hysteretic curve of S3-FM specimen (Seki et al., 2018)

4. MODELLING

Since in the experiment a predominant diagonal tension crack failure mechanism was observed, this phenomenon was considered also in the analytical modeling. Thus, the strut is represented schematically in Fig. 6, corresponding to the failure at peak strength. One can notice how the lateral strength depends on the strut compression force, which in its turn depends on the strut width.

A hand calculation method of the lateral strength, Q_{dia}, is given by the following equation (SATREPS-TSUIB, 2022), which represents the horizontal component of the diagonal compression strut:

$$Q_{dia} = C_{strut} \cdot cos\theta \tag{2}$$

$$C_{strut} = 0.5 \cdot f_{m,\theta} \cdot W_s \cdot t_{mas} \tag{3}$$

$$W_s = 2a_c cos\theta \tag{4}$$

$$a_c = \frac{\pi}{4\lambda} \tag{5}$$

where:

C_{strut} = diagonal force of the infill strut; $f_{m,\theta}$ = compressive strength of masonry along the diagonal (= $0.5f_m$, with f_m = masonry prism strength);

W_s = strut width; a_c is the contact length;

λ = relative rigidity of masonry infilled RC frame;

t_{mas} = masonry thickness;

θ = inclination of loaded diagonal with respect to the horizontal plane.

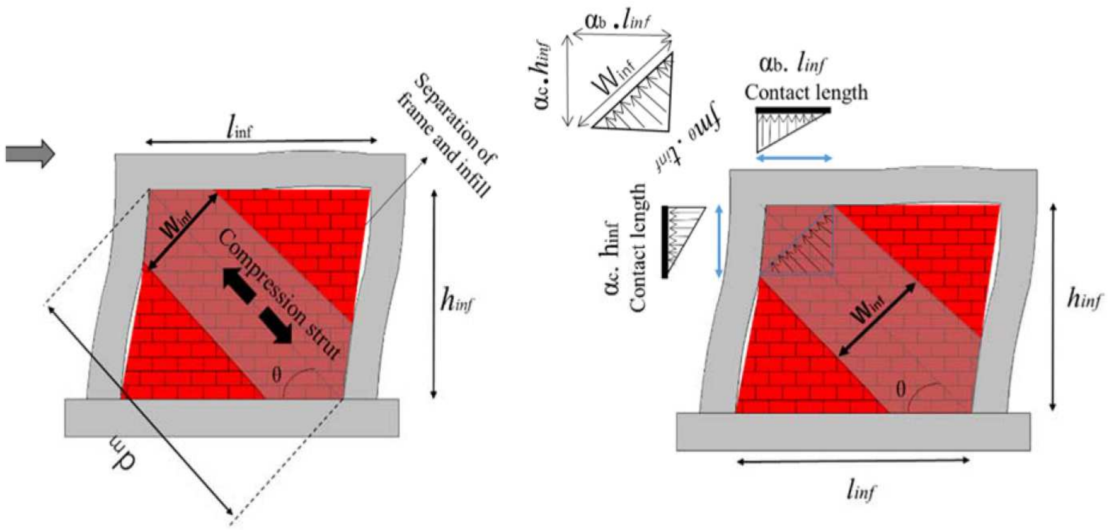

Fig. 6. Diagonal strut mechanism representation (SATREPS-TSUIB 2022)

The compressive stress distribution in the strut is not uniform, having a shape close to a parabolic or triangular distribution, with the maximum stress at the corner (SATREPS-TSUIB 2022).

The material tests values presented in (Seki et al., 2018) were taken into account to calculate Q_{dia}, which resulted as 147 kN.

The analytical model was generated using ETABS 2016 (CSI, 2016). The mechanical characteristics of the concrete, steel and masonry were input as the average values obtained from the experimental tests on materials, presented in (Seki et al., 2018).

Two types of nonlinear hinges were used. The first type, assigned to both ends of the upper beam, was automatically generated by the software as deformation-controlled (ductile) and designed to carry only flexural moment (M3).

The second type of nonlinear hinge was also created and assigned to both ends of the two columns, being deformation-controlled (ductile), but also considering the presence of the compressive axial force applied at the upper end of the columns (P-M3 interaction). The characteristics of this nonlinear hinge were chosen of moment – rotation type, with a symmetry condition (i.e. a symmetric moment - rotation dependence). The scale factor (SF) was taken equal to 1. The option "Load Carrying Capacity Beyond Point E Is

Extrapolated" was used (i.e. not dropping load after the extreme point of the moment-rotation curve). The axial load–displacement relationship was considered to be elastic-perfectly plastic.

The interaction surface data was computed outside ETABS by using the Response 2000 software (Bentz and Collins, 2001), chosen due to its simplicity and ease of use. The classical beam theory was applied to predict the load-deformation response of reinforced concrete sections subjected to bending moments, axial loads and shear forces, as shown in Fig. 7.

The N-M data points of the interaction curve were extracted from Response 2000 (Fig. 8). A number of 19 points (Table 1) were selected along the interaction surface. The values of the moment and axial force corresponding to the interaction surface points (N-M) were input in ETABS, to be used in the setup of the nonlinear hinge properties.

Fig. 9 shows the way in which the axial force/biaxial moment interaction surface for a reinforced-concrete section was generated according to strain compatibility. Subsequently, this was checked to verify its compliance to all the 5 interaction curve requirements. This was meant, finally, to verify that the demand is not exceeding the capacity.

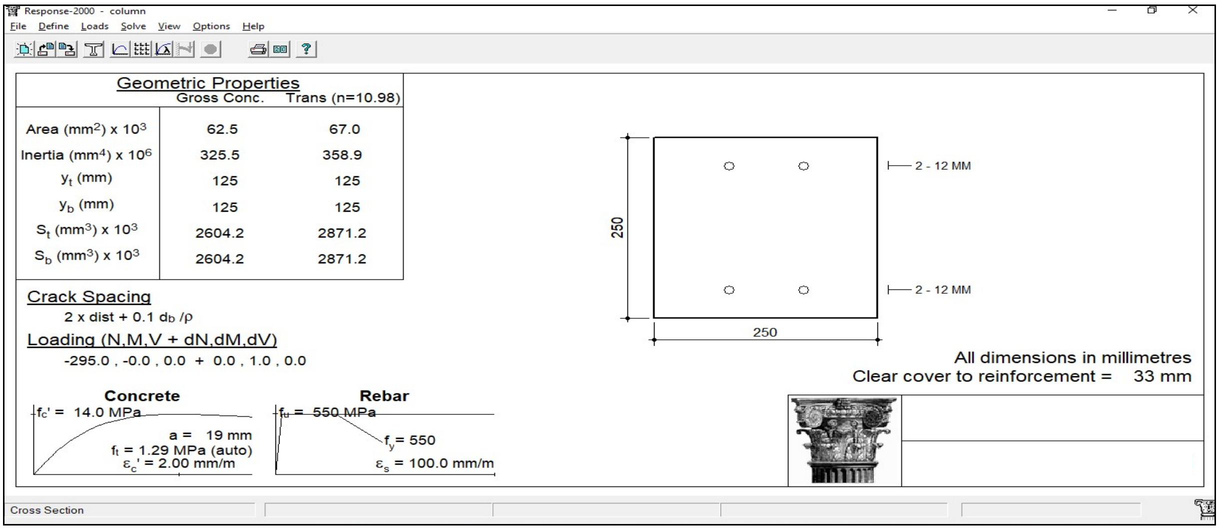

Fig. 7. Interaction surface calculation with Response-2000

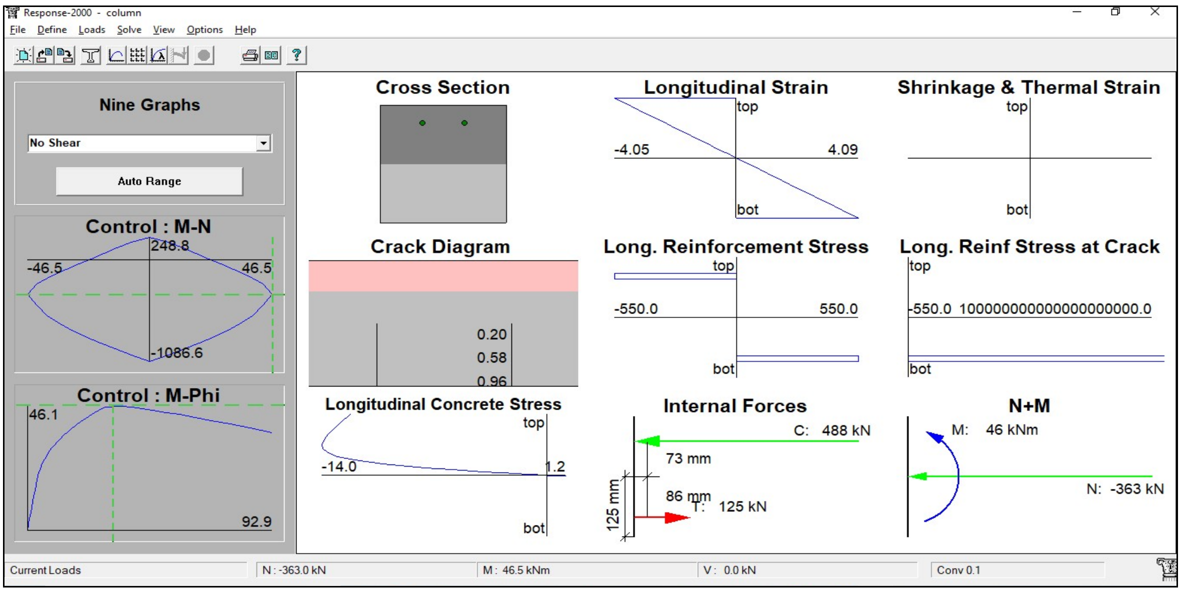

Fig. 8. Interaction surface calculation with Response 2000

Table 1. N-M interaction surface points input into ETABS software

N (kN)	M (kNm)	N (kN)	M (kNm)
-1061.27	0	-321.015	41.3
-983.622	6.42	-225.922	36.969
-921.974	11.776	-151.096	31.754
-866.677	16.557	-87.631	26.763
-813.624	21.102	-31.051	21.941
-757.871	25.637	25.444	17.223
-693.86	30.006	81.839	12.186
-619.6	34.109	138.417	6.545
-533.302	37.904	194.532	0
-432.198	41.463		
-432.198	41.463		

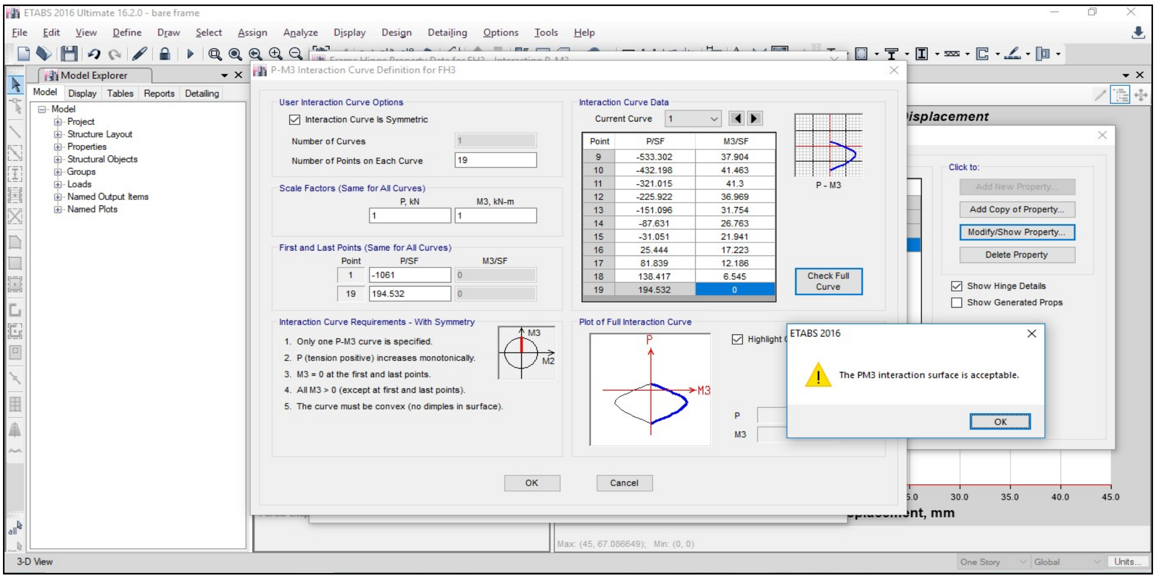

Fig. 9. Input of N-M curve data computed with Respose-2000 in ETABS software

The link used for modeling the masonry infill in the second test specimen was defined as being nonlinear, of "Plastic Wen" type.

The default values of the model were left unchanged, except for the directional properties, which were set to apply in the same direction (U1), i.e. that of the in-plane lateral force. The effective stiffness was set to 20000 kN/m, both for linear and nonlinear properties, being based on iterations performed to estimate an approximate value. The yielding strength was set to 120 kN, to match the experiment.

Four load cases, as follows, were defined to match the loading protocol of the experiment and its sequence:

- Dead - linear static case, to be used as the self-weight load of the frame;
- Axial - linear static case, to represent the loading exerted on both columns;
- LCase1 - nonlinear static case, defining the initial conditions for case Pushx;
- Pushx - nonlinear static case, to represent the accumulation of the previous load case and carry on from that loading state to the final loading stage.

The Axial load pattern was set to simulate the 590 kN load, which was applied by the vertical jacks on the column-beam joints, where each of the two joints carried half of the mentioned load. The Pushx was the load pattern representing the in-plane lateral force that will push the frame until failure.

5. RESULTS AND DISCUSSION

Before getting into details about the results obtained on the masonry infilled frame, it should be noted that the variables of influence are quite numerous. Many combinations which are affecting the outcome of the experiments can be considered, from changing the characteristics of masonry, mortar and concrete, to considering the constructive techniques used to build the structure and their related conditions. Despite the numerous results of experiments, it's quite rare to easily find comparable results and to come up with conclusions about the relations between the different parameters and the behavior of the URM walls, managing, at the same time, to avoid the scattering of data and obtained results.

A total of 108 points along the Lateral Load-Drift Ratio curves were selected when plotting the Envelope plot comparison between model and the actual experiment results, for both the concrete frame with the masonry infills and the bare concrete frame, as shown in the end of this paper.

The angular drift was calculated by Eq. (1). According to the displacements computed with ETABS, the drift ratios have been calculated, tabulated in Table 2 and plotted as shown in Fig. 10.

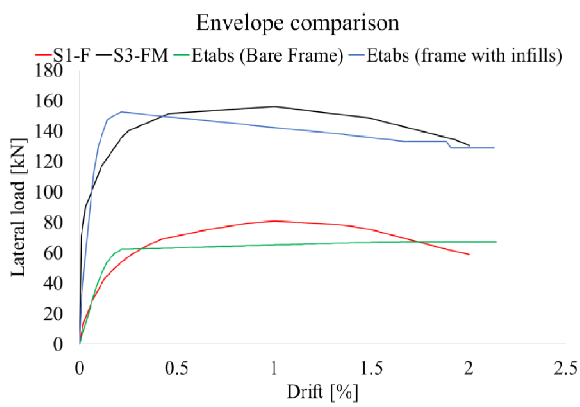

Fig. 10. Envelope plot (pushover curve) comparison between the analytical model and the actual experiment

The hinge formation at the joints of the model during the analysis was quite similar to the hinge formation observed in the actual experiment. Despite the simplicity of the modeling process, the analytical results were fairly compatible with those from the experiment. The difference in terms of the maximum lateral load, when comparing the experimental versus the numerical model was of about 2% for the bare frame and of about 30% for the frame with masonry infills.

Table 2. *Comparison between results provided by the ETABS model and the experiment*

Title	Methodology	Maximum recorded lateral force (kN)	Drift ratio corresponding to the max. force (%)
Concrete frame with masonry infills (S3-FM)	Experiment	156	1
	ETABS	152	0.2
Bare concrete frame (S1-F	Experiment	81	1
	ETABS	62.4	0.23

In the early stage of the testing, the behavior was almost elastic, being mainly influenced by the characteristics of the infill panel. In a simplified approach, in this stage both columns act like tension or compression members and the masonry infill acts like a connecting shear element which makes the entire system to be similar to a cantilever wall. Later, as the hinges are forming, stress concentrations appear at the corners of the frame. The principal stress diagram in Fig. 11 shows the principal trajectories of the stress. Failure patterns and hinge formation are fairly consistent with the stress distribution computed in the URM wall.

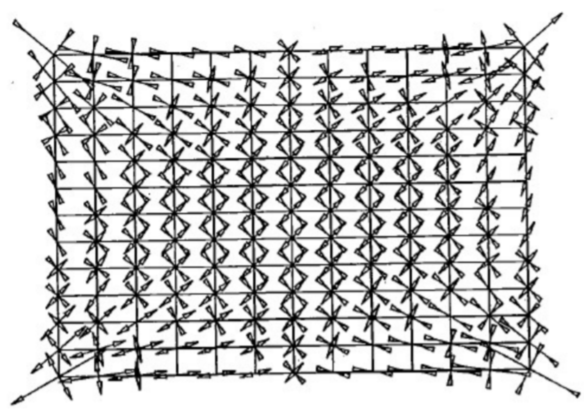

Fig. 11. Principal stresses distribution in the masonry infill panel, obtained with ETABS software

6. CONCLUSIONS AND RECOMMENDATIONS

Several conclusions can be drawn from the comparative study, which are useful for a better understanding of the structural behavior and of the modeling process.

All material details and loading cases were input in the software using its analytical capabilities, although it is quite rare to find comparable results to come up with conclusions about the relations between the different parameters and the behavior of the infilled masonry frames to avoid the scattering of data and the results obtained (Seki et al., 2018).

The masonry infill was introduced as a strut model. For a better understanding of the interaction between masonry and concrete, a more detailed model could be used in the future, possibly including a detailed consideration of masonry units and mortar. The gap effect between the concrete frame and masonry bricks should also be studied.

It can be concluded that, when the demand has exceeded its elastic capacity, the infill wall will experience significant damage. Consequently, for the existing URM buildings in seismic prone areas, assessment and retrofitting analyses should determine whether the residual strength would still provide the capacity of resisting the predicted seismic forces in accordance with the current seismic code. Strengthening is mandatory if the demand forces exceed the capacity forces.

Further experimental work needs to be conducted to investigate in more detail the effect of total lateral displacements resulting from flexure, shear, sliding and rocking by placing more strain gauges along the URM wall height.

Various retrofitting solutions were already proposed and compared to assess their performance such as the use ferrocement (wire mesh) (Seki et al., 2018, Sen et al., 2020) to get more efficiency in preventing the URM walls from diagonal cracking under different load conditions.

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