PSEUDO-DYNAMIC TESTING OF EXISTING STEEL FRAMES WITH MASONRY INFILLS: ASSESSMENT AND RETROFITTING WITH BRBs

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Abstract

Many existing steel multi-storey frames in Europe were designed before the introduction of modern seismic design provisions and often exhibit low performances under earthquake loads due to their insufficient stiffness, strength and energy dissipation capacity. In this context, there is a significant need for advanced assessment procedures able to quantify the seismic performance of these structures and to evaluate the need for retrofitting. However, current procedures for the assessment of existing steel structures in Europe, included in the Eurocode 8 Part 3 (EC8-3), has demonstrated to be inadequate and should be revised. Amongst others, particular attention should be paid to the contribution from masonry infill walls as they significantly affect the modal properties and the lateral strength and stiffness of structures. To this end, the HITFRAMES (*i.e.,* HybrId Testing of an Existing Steel Frame with Infills under Multiple EarthquakeS) SERA project, funded under the H2020-SERA Program, experimentally evaluated the seismic performance of a case study structure representative of non-seismically designed steel frames in Europe including the effects of the masonry infills. A retrofitted configuration of the structure, based on the use of buckling restrained braces, is also tested in order to provide information about the effectiveness of this retrofit strategy. This paper illustrates the analyses performed for the design and the assessment of the case study structure and the preliminary results of the tests on the infilled non-retrofitted structure. Non-linear finite element models of the frame have been developed to complement the experiment design and to forecast the outcome of the tests. The building structure is assessed as a bare and infilled frame under the EC8-3 framework by non-linear static analysis and comparisons are made between the two configurations to estimate the influence of the infills. Then, non-linear time history analyses are performed on the infilled non-retrofitted frame, which focus on the forecast of the experimental outcomes with special attention paid to the response of masonry infills. Preliminary comparison between the numerical predictions and experimental outcomes is also performed for assessing the accuracy of the finite-element model.

Keywords: Existing steel frames, masonry infills, buckling restrained braces, seismic response, pseudo-dynamic tests

1. Introduction

Many existing steel multi-storey frames in Europe were designed before the introduction of modern seismic design provisions. Therefore, they often exhibit high vulnerability to earthquake loads as a result of their insufficient stiffness, strength and energy dissipation capacity. In this context, there is a significant demand for developing advanced assessment procedures, which are able to quantify the seismic performance of existing structures and to evaluate the necessity of retrofitting. Amongst others, a recent post-earthquake study [1] on a steel frame in the area hit by the 2016 Central Italy earthquake has highlighted several failure modes on existing steel frames, including significant yielding at beam-column connections, in-plane and out-of-plane failure of masonry infills, and the soft story failure mechanism. The steel frame was found to suffer large residual drifts after the earthquakes with enormous cracks on infill walls and partial collapse of claddings.

Despite the large effort that has been put into the research on the seismic behaviour of steel frames, there are very few studies addressing the seismic performance of existing steel frames with masonry infills. In fact, most of the research studies focusing on the influence of the infills considered reinforced concrete (RC) structures [*e.g.*, 2, 3, 4, 5 for RC frames and 6, 7, 8 for steel frames] and allowed the development of several modelling strategies to numerically simulate the presence of the infill walls. A common way of modelling the infill walls is through the use of single-strut model, which consists of a single strut in each diagonal direction to simulate the infill wall panels. This model is easy to implement in finite element software and is capable of concentrating the infill wall-frame contact area to the corners. However, currently the property of such model was all calibrated based on RC frames rather than steel frames, which are usually more flexible. Therefore, the reliability of those models for estimating the performance of steel frames remains unclear and requires further justification.

To this end, the H2020-SERA project HITFRAMES (HybrId Testing of an Existing Steel Frame with Infills under Multiple EarthquakeS) focuses on the seismic behaviour of existing steel frames with infills through hybrid tests of a case study steel building, providing insights of the effects of masonry infills on the overall structural performance, and to provide design recommendations for retrofit with buckling restrained braces (BRBs). The objectives of the HITFRAMES project include:

- To experimentally assess the seismic performance of non-seismically designed steel frames with masonry infills under an earthquake sequence evaluating also the effect of the cumulative damage;
- To experimentally evaluate the case study structure retrofitted with BRBs;
- To calibrate numerical models for the bare, infilled and retrofitted frame allowing the definition of modelling recommendations and the assessment of the influence of other aspects influencing the seismic performance of the frame, *e.g.*, record-to-record variability;
- To develop a reliable framework of assessing existing steel frames with masonry infills under earthquake sequence;
- To develop appropriate design procedures of retrofitting existing steel frames with BRBs that account for the effects of masonry infills and earthquake sequences.

To achieve the first two objectives, a series of pseudo-dynamic (PsD) tests have been performed on the case study steel frame at the Structures Laboratory of University of Patras in Greece.

The present paper will focus on the first objective listed above and demonstrate the numerical assessment of the HITFRAMES case study steel moment-resisting frame. Finite-element model of the infilled non-seismically designed steel frame was built, and its seismic performance was firstly evaluated using pushover analysis with special attention paid to the effects of masonry infills. The pushover analysis was performed following the code-based procedure in Eurocode 8 Part 3 (EC8-3) [9]. Then, non-linear time-history analysis was also performed on the case study building in order to predict the experimental outcomes.

Preliminary comparison has also been done between the numerical predictions and experimental outcomes, which allows for assessing the accuracy of the finite-element model and further improvement of the model.

2. Description of the HITFRAMES project

2.1 Case study building

The case study steel frame that was tested is a two-storey one-bay steel moment resisting frame with masonry infills, whose geometric details are summarized in Fig.1. The frame is a non-seismically designed structure, which has been demonstrated in the previous assessment of the structure [10, 11]. The external and internal beams are IPE200 and IPE140, respectively and the columns are HE180A. The steel grade is S355 with a nominal yield strength of 355 MPa. All external beams were connected to the columns through full penetration welding with stiffeners. Stiffeners were also placed at each column base to strengthen the base connections. Fig.2 shows pictures of the stiffeners at beam column connections and column bases. Lastly, regarding the masonry infills, each infill wall consists of two layers of perforated brick, leading to a total thickness of 58 mm.

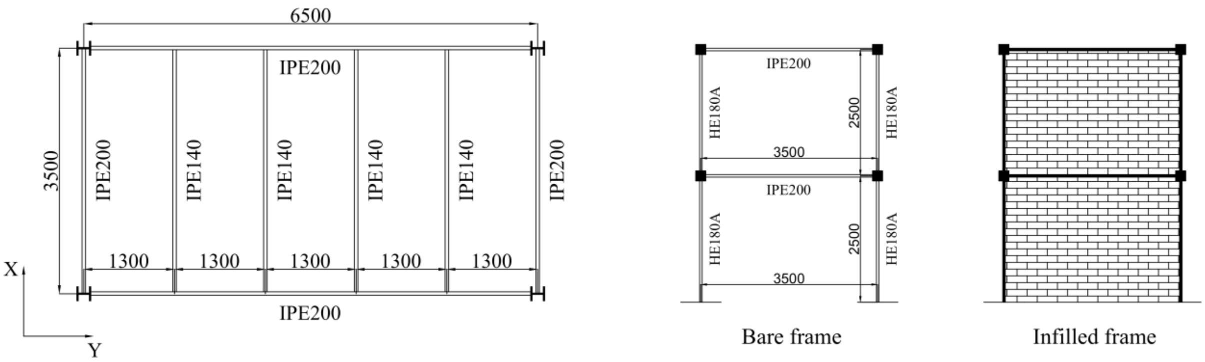


Fig. 1 – Floor plan and side views of the case study steel frame (unit: mm)

(a) (b)

Fig. 2 – Locations of stiffeners: (a) at beam column connection and (b) at column base

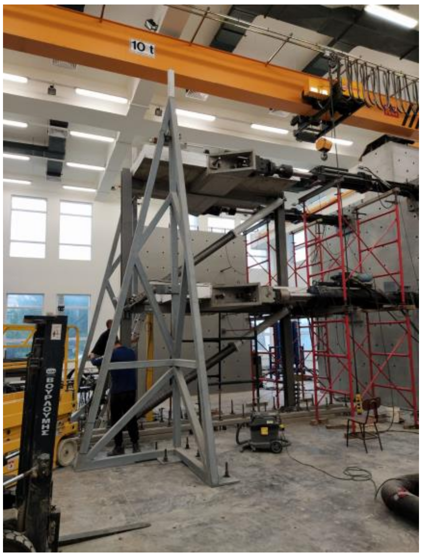
2.2 Test procedure

The experiments carried out for HITFRAMES project consist of two phases, where the first phase included a 3D specimen of the infilled frame and the second phase included a 2D specimen of retrofitted frame. In both phases the PsD tests were performed in the direction along the weak axis of columns. Fig.3 presents the test setup in the two phases.

In the first phase, snap-back free vibration tests were initially carried out on the steel frame before and after the construction of real masonry infill walls. It should be noted that only the infill walls that are parallel to the test direction were accounted for in this study. The results obtained from the free vibration tests were then used for the calibration of the numerical model in terms of its lateral stiffness. Then, PsD tests were performed on the infilled frame subjected to the selected earthquake sequence with scaling factor (SF) of 1.0 and 3.0, respectively. For safety issues, the last test with SF of 3.0 was terminated when the inter-storey drift ratio (IDR) reached 5%.

In the second phase, a plane (2D) frame was tested before and after the installation of BRBs, as shown in Fig.3b. In this phase, the plane frame was a sub-structure of the 3D frame in the previous tests parallel to the test direction and the masonry infill walls were not included. Before the BRBs were installed, the plane frame was subjected to the unscaled foreshock only, which caused some damage on the structure. Then after the installation of the BRBs, the frame was subjected to the foreshock and mainshock with SF= 1.0 and SF=1.5, respectively. The final test with the scaled mainshock record was also terminated when significant torsion was observed and the plane stability of the plane frame was undermined.

(a)



(b)

Fig. 3 – Test setup in the Structures Lab of University of Patras: (a) 3D frame and (b) 2D frame.

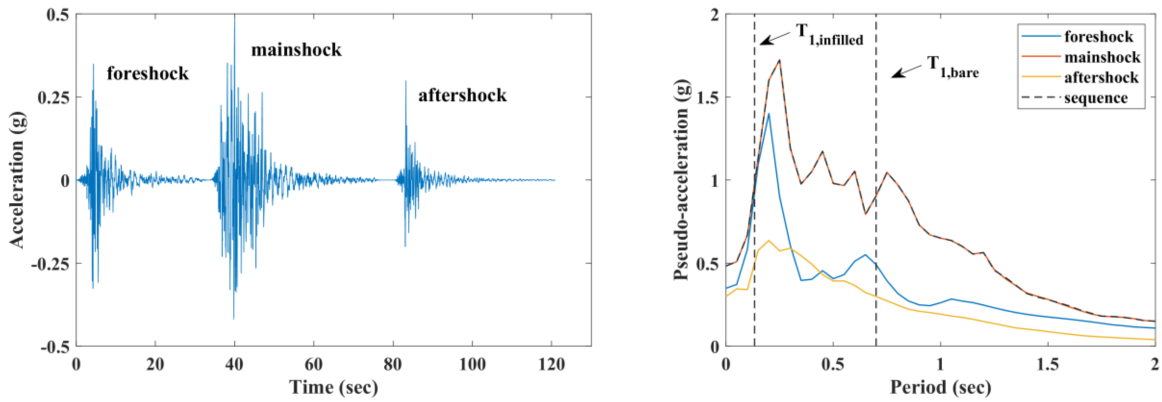
2.3 Selection of earthquake sequence record

The records of three ground motions were selected for this study to form an earthquake sequence, which includes a foreshock, a mainshock and an aftershock, based on their peak ground acceleration (PGA). The records were required to be able to reflect the moderate-to-high seismicity in some areas of the Southern Europe and to have large spectral acceleration in the region near the natural period of the case study frame. All three ground motions occurred during the 2016 Central Italy earthquakes; the basic data for the selected seismic sequence are summarized in Table 1. The foreshock has a PGA of 0.35g, the mainshock has a PGA of 0.48g, which is the largest PGA of the whole sequence, while the aftershock has the smallest PGA of 0.30g. Fig.4 displays the time history of the earthquake sequence.

The response spectra of each individual ground motion listed in Table 1 are also shown in Fig.4, as well as the response spectrum of the earthquake sequence. It is evident that for periods ranging between 0.2 and 1.7 sec, the spectral acceleration corresponding to the mainshock is much larger than that corresponding to the foreshock or aftershock. However, for an infilled structure which usually has an initial period of less than 0.2 sec, the mainshock does not result in a much higher spectral acceleration. Besides, it is also found that the spectrum of the whole sequence is roughly identical to the spectrum of the mainshock.

Table 1 – Summary of the selected ground motions

Event	Date	M_W	R_{epi} (km)	PGA (g)
foreshock	24/08/2016	6.0	15.3	0.35
mainshock	30/10/2016	6.5	4.6	0.48
aftershock	26/10/2016	5.4	9.4	0.30



(a) (b)

Fig. 4 – (a) Time history of the earthquake sequence and (b) corresponding response spectrum.

3. Numerical modelling of the case study steel frame

3.1 Modelling details

The numerical assessment of the case study steel frame was carried out by using the OpenSees platform [12]. For both the 3D and 2D cases, the columns were modelled by force-based elements with ten integration points and the *Steel01* uniaxial material with 2% post-yield hardening. Conversely, the beams were modelled by elastic elements with lumped plasticity. The plastic hinges were modelled by zero-length elements, whose properties were initially calibrated using the moment-rotation relationship proposed by Lignos and Krawinkler [13] and were further modified based on Zareian and Medina [14]. Fig.5 summarises all the materials that were defined in this study. Connections were considered as fully rigid in this study due to the stiffeners that were placed at beam-column joints. In addition, since the tests were performed in the direction along the weak axis of columns, the contribution from column panel zones can be neglected in this case. In terms of the modelling of masonry infills, the single-strut model was adopted in the numerical simulation due to its simplicity and its accuracy in the reinforced concrete structures. The material property of the struts was defined based on the model by Mohammad Noh *et al.* [5] and Liberatore and Decanini [15], as also shown in Fig.5c. It is worth noting that further calibration of the property of infills will be conducted based on the experimental results to achieve higher accuracy. Finally, although the retrofitted frame will not be evaluated in this paper, the response of BRBs is also presented in Fig.5d for completeness. The numerical simulation of BRBs was achieved by the material model developed by Zona and Dall'Asta [16].

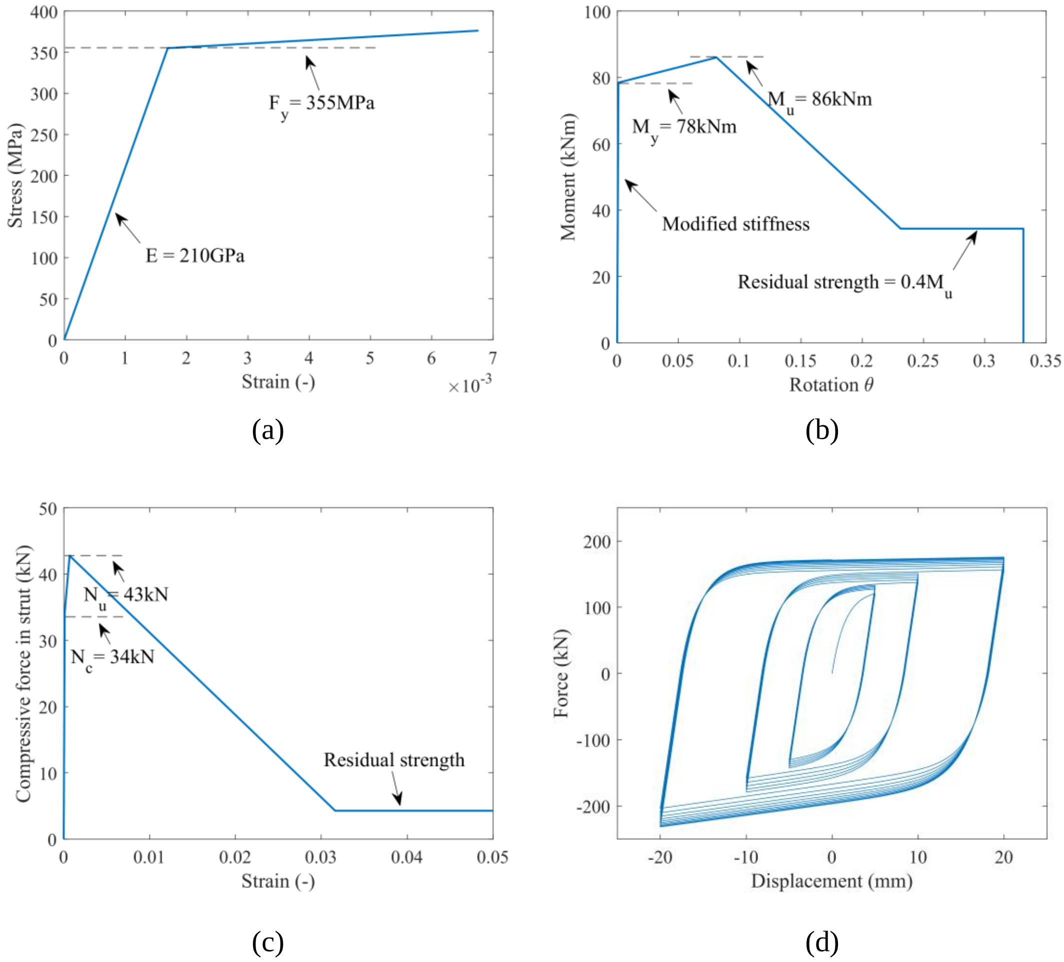
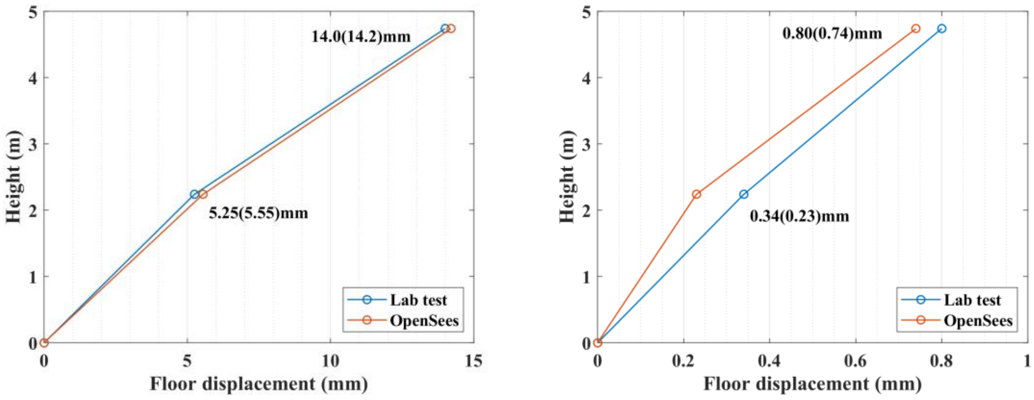


Fig. 5 – Material properties: (a) structural steel, (b) beam plastic hinges, (c) masonry infill struts and (d) buckling restrained braces.

3.2 Calibration of the numerical model

The calibration of the numerical model is twofold according to the results of snap-back free-vibration tests on the 3D steel frame in the first phase of the experiment: comparison of the initial floor displacements induced by the actuators and comparison of the natural periods.

Fig.6 shows the initial floor displacements obtained from the OpenSees model and the free-vibration tests, when a force of 26 and 62 kN was applied to the top floor of the bare and infilled frame, respectively. It is clearly shown in Fig.6a that good agreement on the lateral stiffness of the bare frame has been achieved. Although the results are almost identical, the lab specimen is slightly stiffer than the numerical model, possibly due to the fact that stiffeners at the column base and beam-column connections were not accounted for in the numerical model. However, the discrepancies become larger when the masonry infills were included, as demonstrated in Fig.6b. As mentioned before, the reliability of the adopted modelling method of infills remains unclear and the behaviour of infills will be further adjusted based on the test results. Table 2 shows the comparison of natural periods obtained from the lab tests and numerical models. The results are consistent with the findings in Fig.6. Despite the discrepancy in the displacements of the infilled frame, the model is still able to provide fundamental estimates of the overall response of the steel frame.



(a) (b)

Fig. 6 – Initial displacements induced by actuators: (a) bare frame and (b) infilled frame.

Table 2 – Natural periods obtained from the test and numerical analysis

	Bare frame		**Infilled frame**	
	Lab	OpenSees	Lab	OpenSees
1st Period (sec)	0.6993	0.6987	0.1333	0.1096
2nd Period (sec)	0.2445	0.2424	N/A	N/A

4. Preliminary numerical assessment of the case study frames

To facilitate the assessment of the case study frame, the IDR limits for steel frames in ASCE41-06 [17] was adapted to comply to the limit states in EC8-Part 3 [9], which are 0.7, 2.5 and 5.0% respectively for the damage limitation (DL), significant damage (SD) and near collapse (NC) limit state.

4.1 Results of pushover analysis

In order to study the non-linear monotonic behaviour of the tested structures, pushover analyses were performed on the numerical model, as shown in Fig.7, considering two configurations: bare frame and infilled frame. Also, for this analyses, two load scenarios were considered: a lateral force proportional to the mass and first modes of vibration; and a lateral load distributed uniformly along the height, regardless of the mass, as indicated by EC8-3. For this paper, only the results based on the first load case are shown, since they are slightly less conservative. As it can be observed, the infilled frame presents a significant increase on both stiffness and strength, when it is compared to the bare frame structure. The maximum strength of the infilled frame is 307 kN while the maximum strength of the bare frame is around 215 kN, indicating a nearly 50% increment in the lateral strength. Once the maximum strength is reached, the infilled capacity curve drops to become parallel to the bare frame capacity curve, but with a higher strength due to the residual strength of the infills.

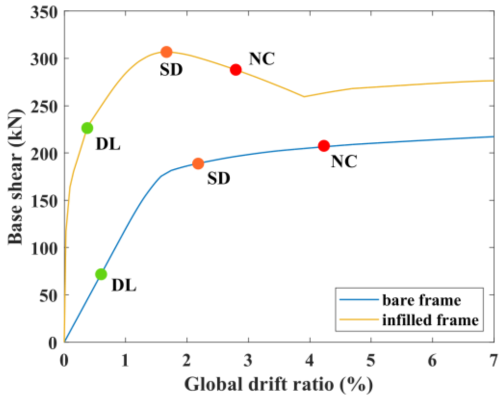


Fig. 7 – Capacity curves of the bare and infilled frame

Fig.7 also presents the capacity points of the bare and infilled frame for each limit state. It is found that the infilled frame reaches its capacity at around 0.3, 1.9 and 2.9% global drift ratio for DL, SD and NC limit state, respectively, while the capacity points of the bare frame is at 0.6, 2.2 and 4.4% global drift ratio. Therefore, it can be concluded that although the presence of infills significantly increases the lateral strength and stiffness of the steel frame, it also reduces the displacement capacity of the framed structure.

4.2 Results of time-history analysis

This section presents the numerical assessment of the 3D steel infilled frame for predicting the structural behaviour during the tests with particular attention paid to the response of infill walls. Preliminary comparison between the numerical predictions and experimental results is also available hereafter.

Fig.8 to 11 summarize the results obtained from the numerical assessment. Fig.8 shows the IDR of the first storey slab of the infilled frame, which experienced the larger lateral displacement during the earthquake sequence. It is evident that with SF=1.0, the peak drift is around 1.1%, which narrowly exceeds the DL limit state, suggesting that the infills are not significantly damaged while the structural elements are not yielded according to the definition of the DL limit state. However, when the SF was increased to 3.0, the peak drift is 5.8%, which violates the NC limit state, indicating severe damage or even partial collapse of infill walls, and significant yielding in structural elements, especially in columns, with little residual strength of the case study structure. Fig.9 presents the response of columns on the ground floor, which is consistent with the observations in Fig.8. It is clear that with SF=1.0 the columns are still in the elastic range, but with SF=3.0 the largest rotation in columns reaches five times the yield rotation and permanent deformation is also found.

Fig.10 and 11 show the response of infill struts on both the ground and first floor. In each figure, the behaviour of the two diagonal struts in the same infill panel is presented, which forms the complete response of an infill wall. It is found that with SF=1.0, the infills on both floors are damaged and stiffness degradation occurred during the earthquake sequence, although the damage is believed to be limited. However, when the SF was raised to 3.0, the damage on infills become more significant, in particular the infills on the ground floor, which are close to reaching the residual strength as shown in Fig.11a, indicating that total or partial collapse of infills may occur. Finally, it is also noted that in both cases (SF=1.0 and 3.0), the infills on the ground floor suffer more severe damage than the infills on the first floor.

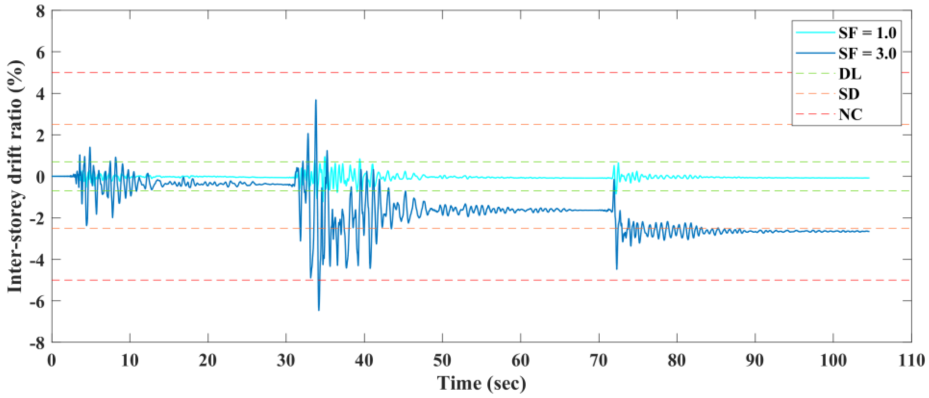
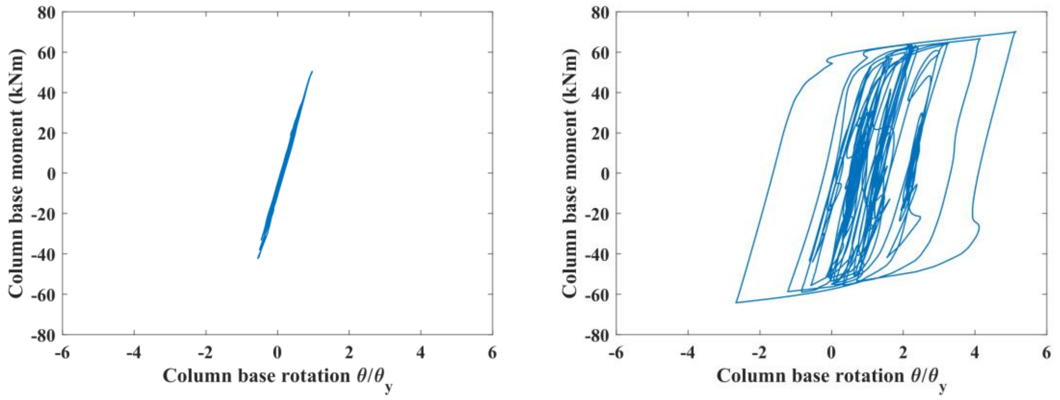
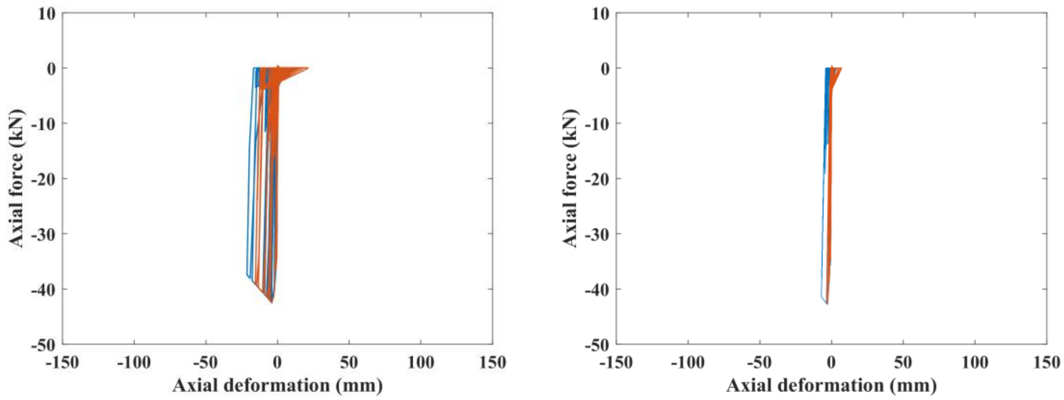


Fig. 8 – Inter-storey drift ratio of the first floor of the infilled frame



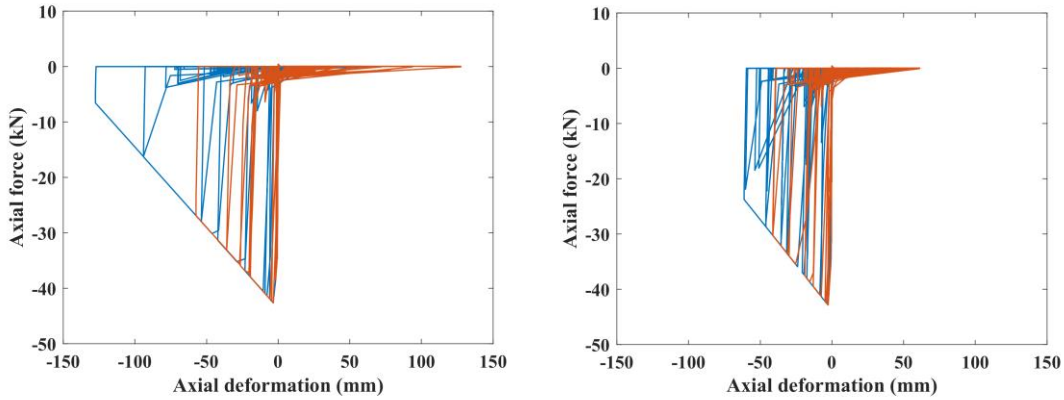
(a) (b)

Fig. 9 – Response of column on the ground floor: (a) SF = 1.0 and (b) SF = 3.0.



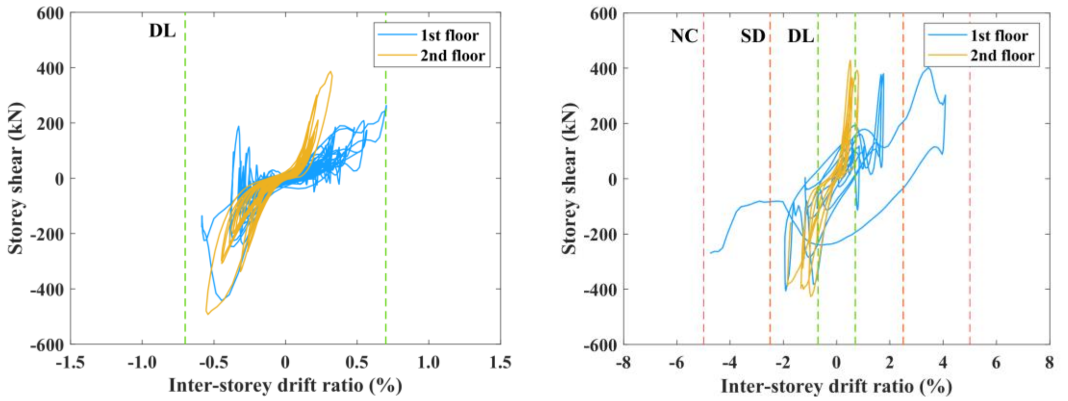
(a) (b)

Fig. 10 – Response of diagonal infill struts on the (a) ground floor and (b) first floor with SF = 1.0.



(a) (b)

Fig. 11 – Response of diagonal infill struts on the (a) ground floor and (b) first floor with SF = 3.0.



(a) (b)

Fig. 12 – Response of 3D steel infilled frame obtained experimentally with: (a) SF = 1.0; (b) SF = 3.0.

(a) (b)

Fig. 13 – Damage on the masonry infills: (a) SF = 1.0 and (b) SF = 3.0.

Fig.12 demonstrates the storey shear-IDR response of both floors obtained experimentally. It is clearly shown in Fig.12 that the cyclic response measured experimentally at both floors for the infilled frame exhibited significant pinching during the test, which is probably due to the open and closing of cracks on the infill walls. Such response increases as the lateral drifts increase for higher values of peak ground acceleration of the earthquake input. Besides, Fig.12 also shows that the experimental specimen experienced 0.7% IDR for SF = 1.0 and 4.8% IDR for SF = 3.0, which are both smaller than the 1.2% and 6.9% IDR obtained from the numerical model, as demonstrated in Fig.12, indicating that the real steel frame in the lab has a higher lateral stiffness than the numerical model. However, Fig.6b suggests that the steel frame in the lab had a lower initial lateral stiffness than the numerical model. This is probably due to the fact that the currently adopted model of infills are not adequate to reliably predict the response of steel infilled frames. Despite this, the current model of infills still correctly predict the damage state of infills during the tests, as shown in Fig.13. It is found during the tests that the first damage occurred at the corners of infill walls, probably due to the bolts at the beam splice connections. Diagonal cracks and shear slides were also observed on all infill walls. As shown in Fig.13b, partial collapse occurred on the ground floor infills at the end the test with SF=3.0.

5. Conclusions

This paper presents the results of preliminary numerical assessment of the case study steel frame adopted in the HITFRAMES project. In terms of model calibration, the natural periods and lateral stiffness of the numerical model well matched the test results, however, when infills were included in the numerical model, large discrepancies were observed between numerical and experimental results.

From the pushover analysis, it is found that the infills increase the lateral strength of the steel frame by nearly 50%. Besides, the time-history analysis suggests that the peak IDR is 1.2 and 6.9% for SF=1.0 and SF=3.0, which are larger than the results obtained from PsD tests. In terms of the masonry infills, it is found from numerical analysis that the infills on the ground floor suffered more severe damage than the first floor infills, which well match the observations during the test. Also, it is estimated that the infills on the ground floor may experience partial collapse when SF=3.0, which was also observed during the tests.

Therefore, the conclusions that can be drawn from the currently available test results are as follows:

- Masonry infills augment significantly the lateral stiffness and strength in steel bare frames;
- The steel infilled frame in the lab shows a lower initial lateral stiffness than the numerical model but a higher lateral stiffness during after damage occurred on the infills;
- The response of double-layered infills in steel frames differ from the response of the counterpart infills in reinforced concrete (RC) framed buildings. As a result, existing models used to simulate the response of infills in RC structures are not adequate to reliably predict the response of steel infilled frames;

6. Acknowledgements

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