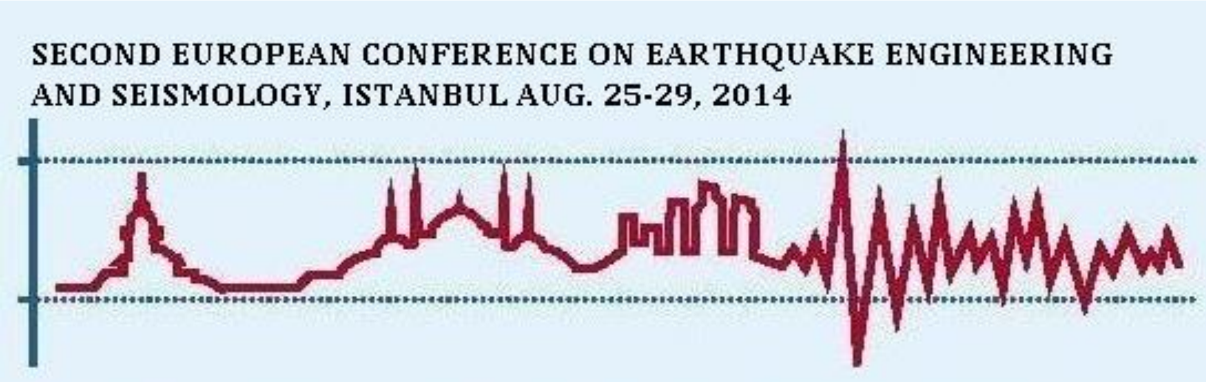


FRAGILITY ANALYSIS OF MID-RISE MASONRY INFILLED STEEL FRAME (MISF) STRUCTURES

Arash NASSIRPOUR[1] and Dina D'AYALA[2]

ABSTRACT

This research looks through the influence of masonry infill walls (solid and with opening) on the seismic behaviour of a dual system steel frame structure. The studied building is a typical 4 storey tall steel framed structure with masonry infill walls covering the entire outside circumference. A detailed three dimensional model of the building, including the infill openings, is generated using fibre based finite element software SeismoStruct. The response of the structures under seismic excitation has been assessed using non-linear static push-over analysis on several cases. The resultant capacity curves are implemented to obtain analytical fragility curves and compared to that of a bare dual system steel frame. Two suites of seven different earthquake records each representing a various intensity and focal mechanism have been selected for the fragility analysis. The comparison of the capacity and fragility curves indicate that the presence of infill has a significant effect on the lateral stiffness, strength and ductility of the entire structural system and ignoring it will be hazardous and uneconomical.

INTRODUCTION

Steel framed structures with unreinforced masonry infill panels make up a significant proportion of residential buildings in seismically active zones (e.g. Japan, China, Turkey, Iran and California). Unreinforced masonry infills tend to interact with the surrounding frame when the structure is subjected to strong earthquake loads, alter the behaviour of the building by increasing the stiffness and strength, and thus can be the dominant cause of structural failure and casualty (D'Ayala & Ellul, 2005). Ignoring this increase in stiffness is not always conservative since higher seismic load is attracted as the building becomes stiffer. Then, if the panel is overstressed and fails partially or wholly, the high forced previously attracted and carried by the stiff panels will be suddenly transferred to the more flexible surrounding frame. Additionally, the change in stiffness distribution may result in higher seismic forces due to torsional effects. However, the influence of infills are generally ignored in the current design practice which leads to uneconomical design of the frame since the lateral stiffness and strength demand of the system can be largely reduced (El-Dakhakhni *et al.*, 2003).

A common way of evaluating the performance of buildings against earthquake loading is through the use of fragility curves. Many studies have investigated bare concrete and steel frame structures, while limited researches have been conducted on developing fragility curves for framed structures with masonry infilled walls. The aim of this study is to derive analytical fragility curves for Masonry Infilled Steel Frame (MISF) structures by considering variation in the intensity of earthquake records. These structures inherit a large amount of non-linear inelastic deformation which is primarily because of material non-linearity. In this contribution, numerical investigation on the seismic behaviour of a steel

[1] PhD Candidate, EPICentre, University College London (UCL), London-UK, a.nassirpour@ucl.ac.uk
[2] Reader in Structures, EPICentre, University College London (UCL), London-UK, d.dayala@ucl.ac.uk

framed structure with masonry infill panels is performed. A typical mid-rise, dual system steel framed building (concentric braced frame + moment resisting frame) and masonry infill is selected as an index building.

The behaviour of the structures under seismic excitation is measured using non-linear static push-over analysis. Several capacity curve for different cases of infills and bracings are presented in terms of force-deflection. By employing simplified fragility analysis method FRACAS (Rossetto & Elnashai, 2004), the fragility functions is generated considering the damage states following the guidelines and two suite of earthquake accelerograms representing the seismicity of the building's location.

MODELLING OF MASONRY INFILL WALLS

Infill panels are widely used as external walls and interior partitions in buildings, however, they are mainly considered as non-structural elements and are commonly not included in the analysis and design process. Since 1950s many studies concluded that the presence of infill has a significant effect on the strength, stiffness and energy dissipation of the structural system. Although recent modern earthquake codes present guidelines for designing the infill walls, they deter the designers from reducing seismic action effects or from relying on the beneficial presence of infill walls. Therefore, infills are usually considered as a source of significant over-strength.

The main reasons are due to complexity of infill reaction (mechanical property of material and construction detailing), the inherent uncertainty associated to the numerous parameters that behaviour of infills depends on and finally the type of interaction between the infill and it surrounding frame. On the other hand, the infilled frame structures need a realistic model and cannot be modelled as elasto-plastic system due to the stiffness and strength degradation especially in short period structures in which the hysteresis loops and energy dissipation capacity have a strong influence on the response. Based on empirical results, the lateral resistance of an infilled frame is not equal to the sum of the resistances of infill and its surrounding frame and can increase between 4 to 20 times the stiffness of a simple bare frame. Furthermore, the location and the dimensions of openings play an important role in the strength and lateral stiffness of single panels and the whole structural system.

In the beginning of 1950s, Polyakov presented one of the first analytical and empirical studies on the seismic response of infilled panels surrounded by reinforced concrete and steel frames (Polyakov *et al.*, 1956). The study discussed the effect of factors such as the material property of masonry units, the mortar type, loading case (uniform or cyclic) and also the influence of openings by undertaking a number of full scale experiments. According to the experimental results and the fact that the frame was resting (leaning) on the infill, the idea of "Equivalent Diagonal Strut" was proposed for modelling of the infill panels. In 1960, Holmes suggested that the equivalent strut should have a width equal to 1/3 of the length of the masonry panel (Holmes, 1960). Later on, Stafford-Smith and Carter (1969) related the width of the equivalent diagonal strut to the infill/frame stiffness parameter (λ). Generally, the interaction of frame and the infill has been mainly studied with simple one bay – one storey frames, from theoretical (El-Dakhakhni *et al.*, 2003; Biondi *et al.*, 2006), experimental (Moghaddam, 2004; Mander *et al.*, 1993; Tasnimi & Mohebkhah, 2011; Mosalam *et al.*, 1997) or numerical (Personeni *et al.*, 2008; D'Ayala *et al.*, 2009) point of view.

In the past recent years, the concept of simulating the infill with a single or multiple diagonal struts under compression is widely accepted as a simple and rational way to describe the influence of the masonry panels on the surrounding frame and has been adopted in many documents and new guidelines, such as S304.1 (CSA, 2004), SEI 41-06 (ASCE, 2006), NZSEE (2006), MSJC (2010).

Following the equivalent strut approach, a numerical macro model proposed by Crisafulli (1997) is adopted for simulating the solid infills and those with openings of this study. Although one diagonal strut results in acceptable values for stiffness and axial forces induced in the frame members, due to lateral load, it underestimates the bending moments and is not capable of estimating the horizontal shear sliding of the masonry panel. Therefore, Crisafulli adopted the double-strut model which is accurate enough to describe the local effects resulting from the interaction between infill panel and its surrounding frame and at the same time less complicated in comparison to the triple-strut.

In Crisafulli’s model, the compressive cyclic behaviour of the masonry is represented by seven hysteresis rules in order to consider different behaviours for loading, unloading and reloading (Figure 1). The four compressive and tensile diagonal struts follow this masonry hysteresis model. The model also considers the local contact effects of the cracked material, the effect of the small inner cycles and the tensile behaviour of masonry. Additionally, a single strut acting across two opposite diagonal corners is implemented to carry the shear from the top to the bottom of the panel which follows a bilinear hysteresis rule. Figure 2, illustrates the arrangement of compressive, tensile and shear struts.

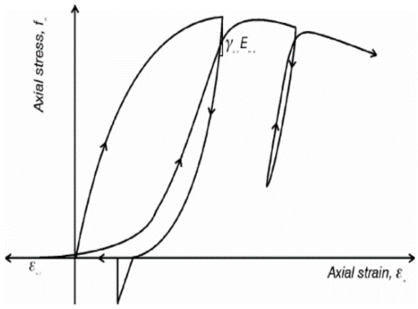


Figure 1 - Crisafulli (1997) hysteresis curve and compressive diagonal strut for masonry infill

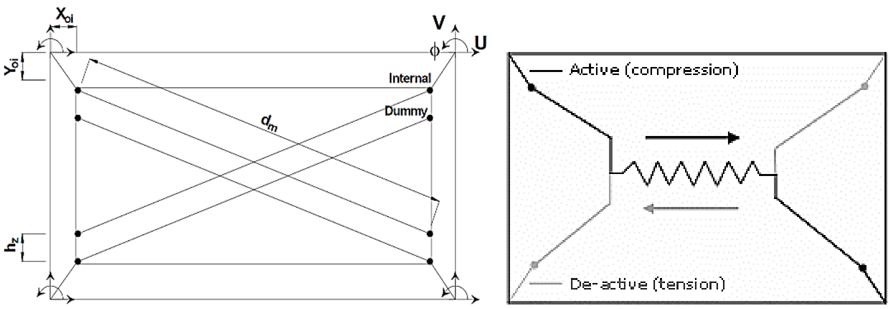


Figure 2 – Arrangement of diagonal compressive and tensile struts and the shear strut of Crisafulli (1997) model

According to Shing and Mehrabi (2002), the main modes of infilled frame failure are corner crushing, sliding shear, diagonal compression, diagonal cracking and also failure of the surrounding frame. The first two modes are the most common ones and Crisafulli’s (1997) analytical model is fully capable of simulating them. It should be noticed that although the model is capable of consideration the out–of–plain failure of infill walls, it has been ignored in this study as it is unlikely due to the arching mechanism.

In order to calibrate the masonry infill models with the identical material used for the index building, a study by Tasnimi and Mohebkhah (2011) is chosen. The study conducts a number of experiments on the behaviour of brick-infilled steel frame with and without opening. The material used for the pseudo-dynamic tests is of great similarly to those available and used for the construction of the selected building. Single-storey, single-bay steel frame specimens were tested under in-plane cyclic loading applied at the top corner of the frame. The masonry infill consists of clay brick and two cases are selected for this study, one with a large window opening located at its centre (ratio=0.176) and another one with solid infill panel. This was followed by a sensitivity study whereby the relative importance of each parameter necessary to calibrate the model is evaluated. As a result, the model is set to be as realistic as possible by considering the location of masonry infills, lateral stiffness, strength of the elements and the effect of any opening (door and windows) on the panels. The calibration results in terms of hysteresis and envelope curve are presented in Figure 3.

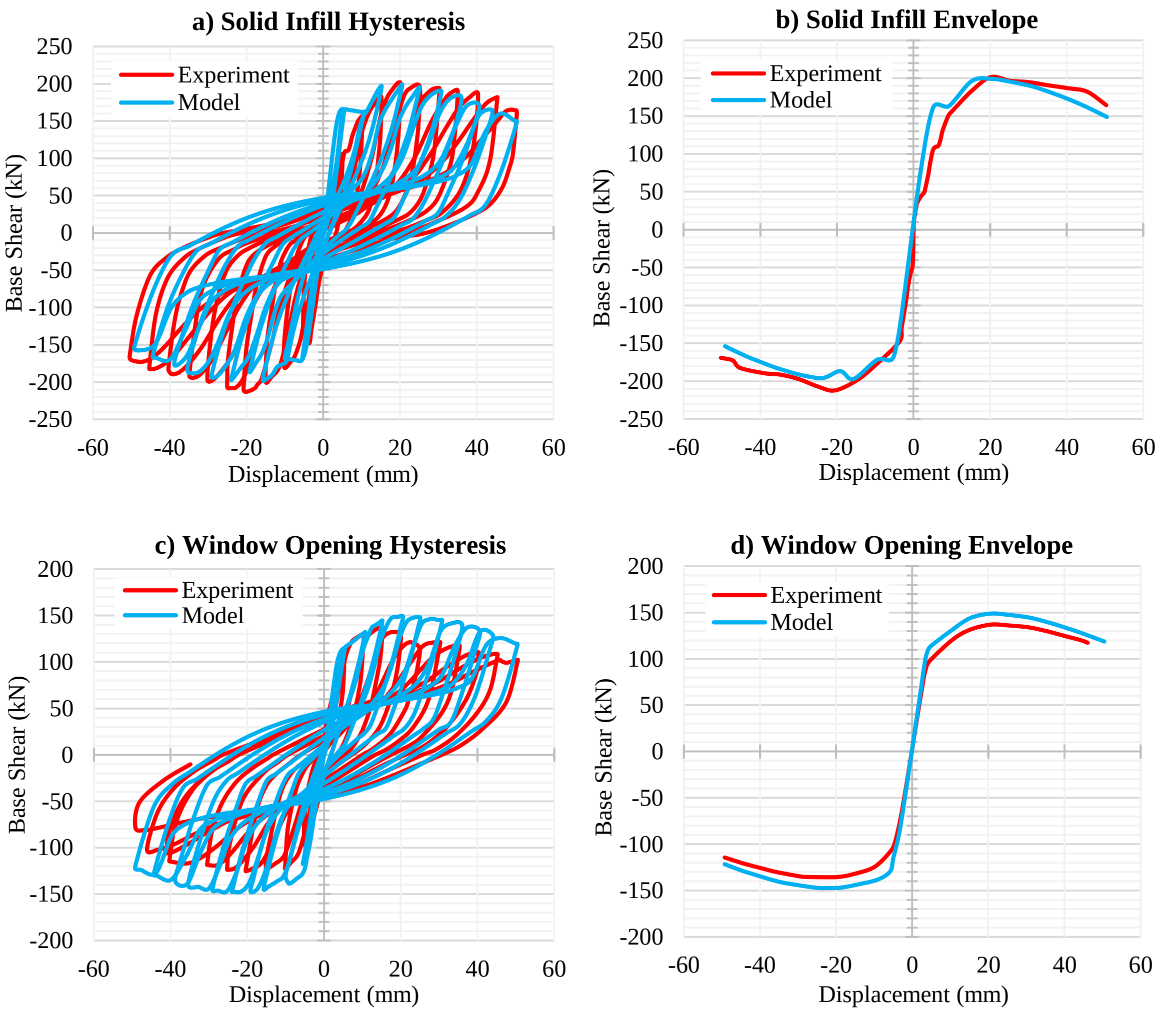


Figure 3 – Calibration outcomes of numerical infill model compared with experimental results of clay brick

INDEX BUIDLING

For this study, the capital city of Iran, Tehran, one of the highly seismic mega cities of the world has been selected. Referring to the 2004 taxation data obtained from the Tehran Municipality Computer Service Organization (TMCSO), steel buildings with masonry infill consist about 27% (254’000) of the total number of the city’s typology. Furthermore, 48% (451’000) of building’s taxonomy include unreinforced masonry. On the other hand, due to the rapid population growth of Tehran in past ten years, the house demand has amplified greatly resulting in an increasing interest in steel framed buildings. Furthermore, motives such as government’s support of industrial buildings, manufactured mainly in the factories, ease and speed of erection and higher quality of construction, have attracted more contractors to build such structures.

As an index building, a residential steel framed structure, 4 storey high, with moment resisting frame in x-direction and concentric bracings in y-direction is chosen. Recently a great number of such buildings has been constructed in different locations of Iran with moderate to extreme seismic activity, exposing a considerable population to the resulting risk from seismic damage. Moreover, the construction method and plan is one of the most common styles of buildings in middle-east region (e.g. Iran, Turkey). Figure 4 and 5 illustrate the building’s plan of different floors, indicating the location and dimensions of beams (IPE), columns (HEB) and bracings. The column sections are given in Table 1 and the beam sections are indicated along the elements in the building’s plan. For the bracings a hollow square section with a cross sectional area of $120 \times 120\ mm^2$ is implemented.

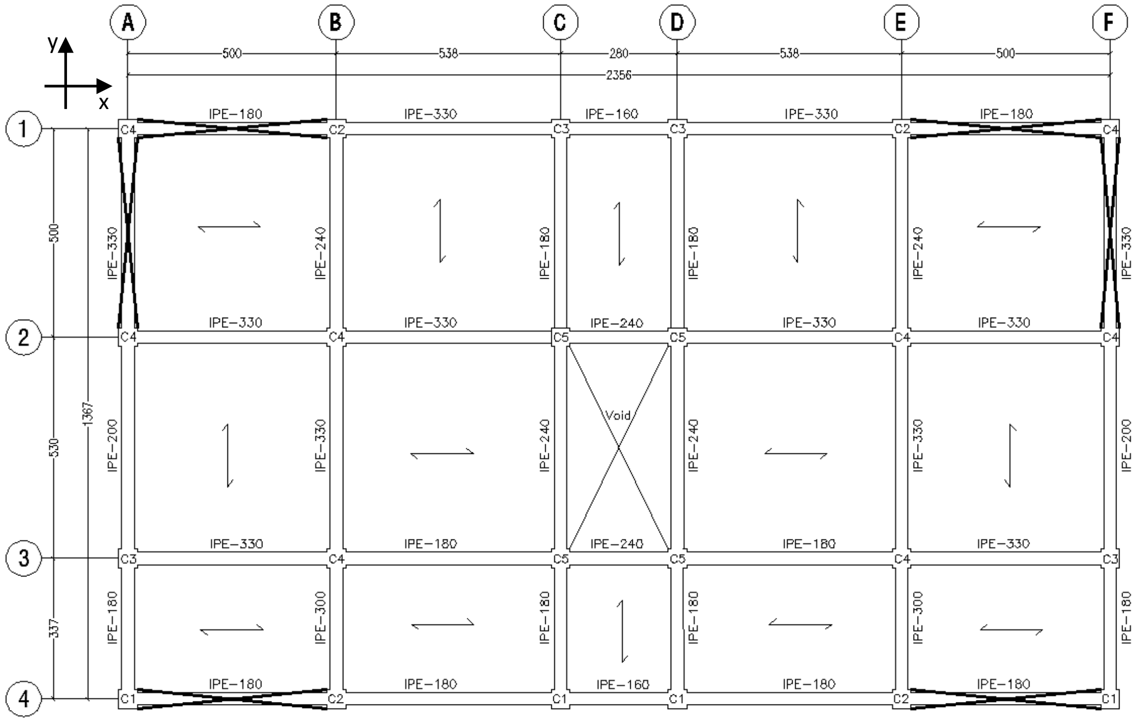


Figure 4 - Plan of the 1st floor of index building (units in centimetre)
IPE: European steel I-beam with parallel flange surfaces

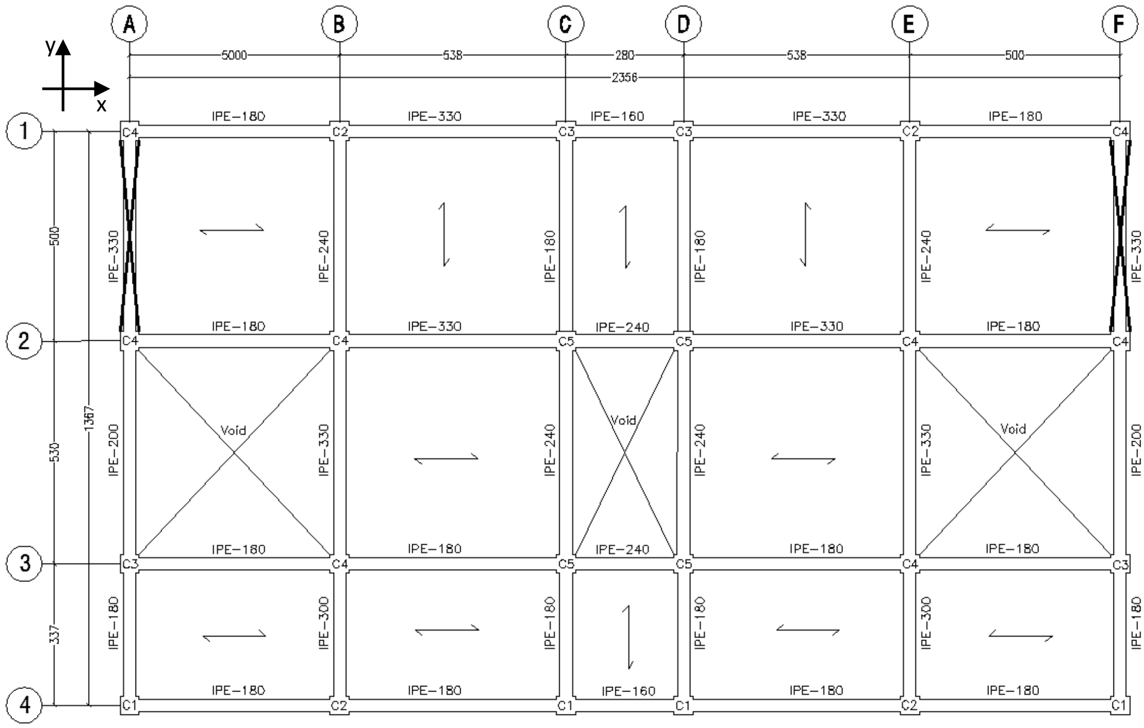


Figure 5 - Plan of the 2nd, 3rd and 4th floors of index building (units in centimetre)
IPE: European steel I-beam with parallel flange surfaces

Table 1 - Column sections used for the index building

	Columns Ref.	C1	C2	C3	C4	C5
Storeys	***4th Floor***	HEB-140	HEB-140	HEB-120	HEB-160	HEB-160
	3rd Floor	HEB-140	HEB-140	HEB-120	HEB-160	HEB-160
	2nd Floor	HEB-140	HEB-140	HEB-140	HEB-180	HEB-160
	1st Floor	HEB-140	HEB-160	HEB-140	HEB-180	HEB-160

HEB: European wide flange beams with parallel flange surfaces and approximate equal width and depth

MODELLING AND ANALYSIS RESULTS

As mentioned, the index building is a 4-storey tall dual system steel frame including 5-bays (y-direction) and 4-frames (x-direction). Lateral resisting system consists of X-bracings in y-direction for all storeys and moment resisting frame in x-direction. Additionally, X-bracings are located at the ground level on both directions.

The building is design based on 2800 Iranian seismic code (version III, issued in 2005 after 2003 Bam earthquake), which is highly identical to UBC-97. Considering the seismicity and the geology of Tehran, a peak ground acceleration (PGA) of 0.35g (highest value in the code) and semi-compact soil condition is chosen for the seismic design of the structure.

A live load of 200 kg/m^2 and 150 kg/m^2 is considered for the floors and roof (4th floor) respectively. The dead load consists of floor finishing, joists and metal decks and is estimated equal to 350 kg/m^2 for the floors and 380 kg/m^2 for the roof (4th floor).

The steel material modelled is S235 based on European standards (f_y = 235 MPa) and follows the Menegotto-Pinto steel model (Menegotto & Pinto, 1973). The infill material as mentioned earlier follow a model proposed by Crisafulli (1997) and are calibrated based on the clay brick material presented in Tasnimi and Mohebkhah (2011) with a panel thickness of 0.11 m. Table 2 presents a summary of the material properties used for the modelling of the index building.

Table 2 - Properties of materials

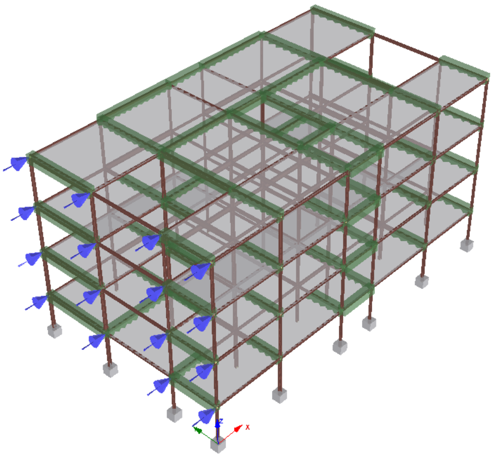
Steel S235	Modulus of Elasticity (E)	2.1×10^5 MPa
	Yield Strength (f_y)	235.0 MPa
	Specific Weight	78.0 kN/m^3
Masonry infill wall	Initial Young Modulus (E_m)	5,194 MPa
	Compressive Strength (f_m)	7.40 MPa
	Tensile Strength (f_t)	0.12 MPa
	Shear Bond Strength	0.48 MPa
	Specific Weight	10.0 kN/m^3
	Panel Thickness	0.11 m

The building is modelled in three dimensions using fibre based finite element software SeismoStruct (2013). The software is capable of predicting large displacement behaviour of space frames under static and dynamic loadings, taking into consideration the geometric nonlinearities and material inelasticity.

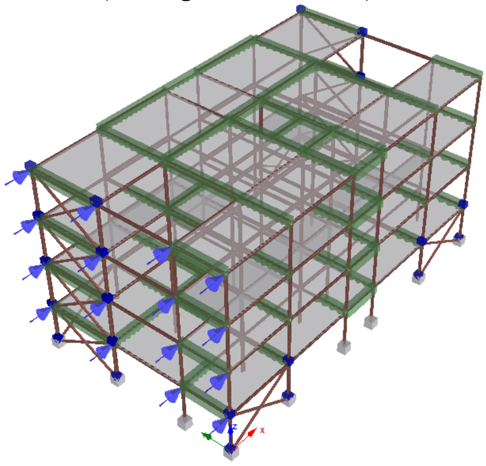
Static push-over analysis with incremental uniform rectangular loading is applied on both sides of the building (x- and y-direction). Response control has been chosen as the loading phase and stops the analysis at a displacement of 0.2 metre of the 4th floor node according to FEMA 356. This loading strategy is able to capture irregular response features (e.g. soft storey), capture the softening post-peak branch of the response and obtain an even distribution of force-displacement curve points. Therefore, this type of loading phase usually constitutes the best option for carrying out non-adaptive push-over analysis.

A number of different hypothesis are analysed based on the removal or addition of bracings or infill panels to the steel frame. In total 6 models are analysed in both directions under static push-over loading. The models are illustrated in Figure 6. In the following models, the incremental loadings are indicated with blue arrows and applied in x-direction. Furthermore, the live and dead loads (represented with green colour) are applied as distributed loading on selected members.

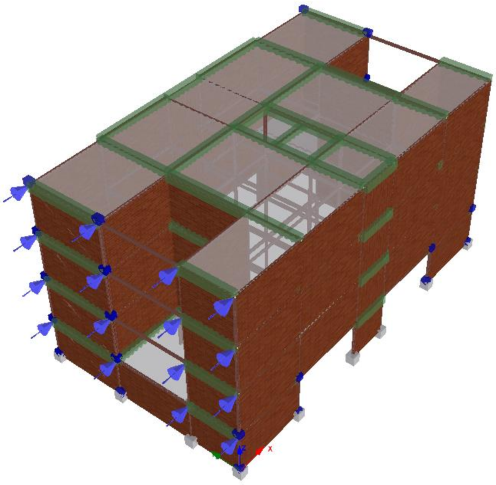
***Model 1* - Bare Steel Frame**
(No Bracings)



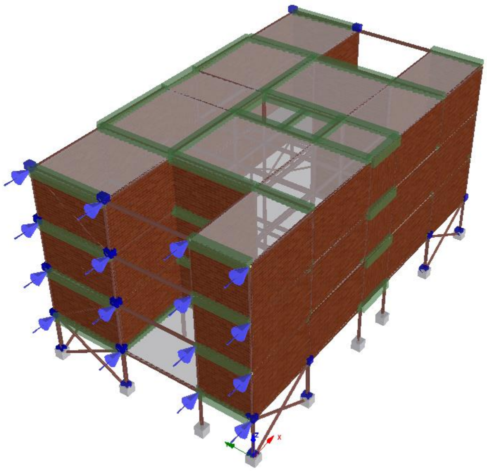
***Model 2* - Bare Steel Frame**
(Bracings in all direction)*



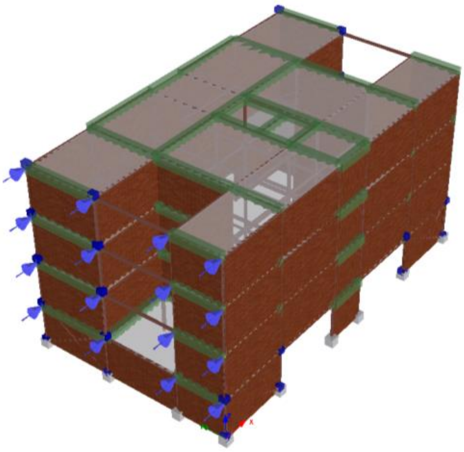
Model 3 - All Infills (opening & solid) - All Bracings in y-direction - No Bracing in x-direction



Model 4 - All Infills (opening & solid) except the ground level - Bracings in all directions



Model 5 - All Infills (opening & solid) - Bracings in all directions*



Model 6 - All Infills (only solid) - Bracings in all directions

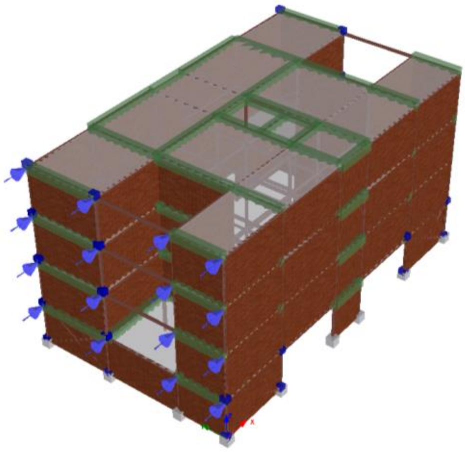


Figure 6 - Illustration of different models of the index building (* Fragility analysis is conducted for the model)

Figure 7, presents a comparison of the resultant capacity curves (force-deflection) for Models 2 and 5 loaded in different directions. The results indicate that in case of bare steel frame (Model 2) the presence of bracing for all storeys in only one direction (y-direction) has a significant influence on the lateral stiffness and capacity of the structure. On the other hand, while the addition of infill panels (Model 5) changes the structural behaviour, the difference in terms of stiffness is very little and again the presence of more bracings in y-direction has increased the total capacity of the structure. Therefore, the fragility analysis is implemented only on the weaker direction (x-direction) of both models.

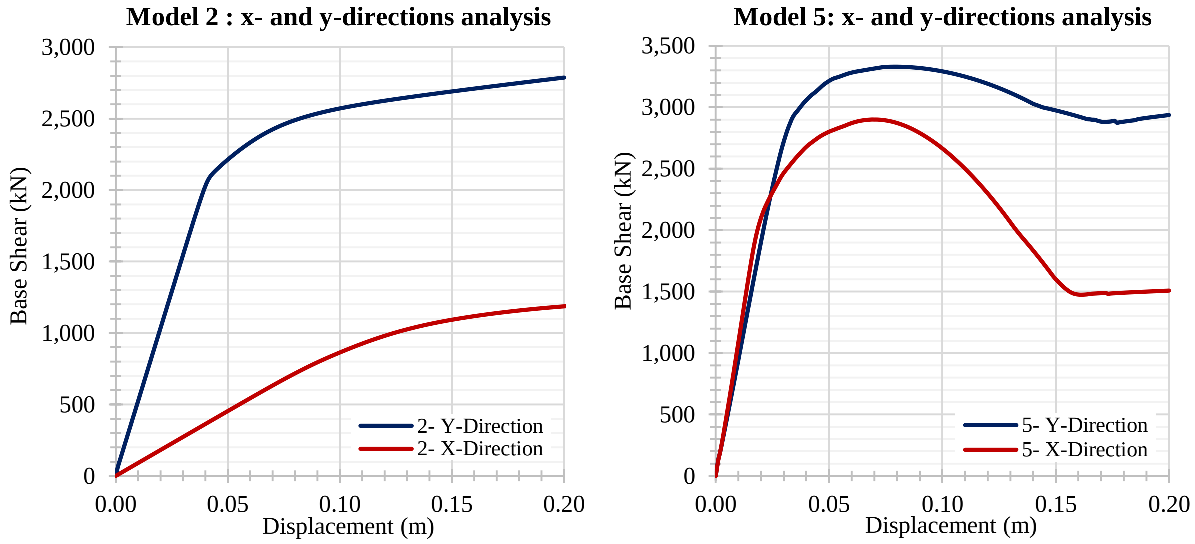


Figure 7 - Comparing the effect of x- and y-direction loading on Capacity Curve

A comparison of capacity curves resultant from the static push-over of all models in x-direction is given in Figure 8. In comparison to bare steel frames, the presence of infill panels causes a considerable increase in the stiffness and strength of the structure. In case of Model 5 (solid & opening infill) and Model 6 (only solid infill), it can be concluded that any opening in infill can noticeably drop the stiffness and capacity of the whole system. Therefore it is important to consider the infill’s opening for the analysis. Furthermore, a comparison between Model 5 and 3 indicate how bracings at first floor can improve the capacity of structure while slightly increasing its stiffness.

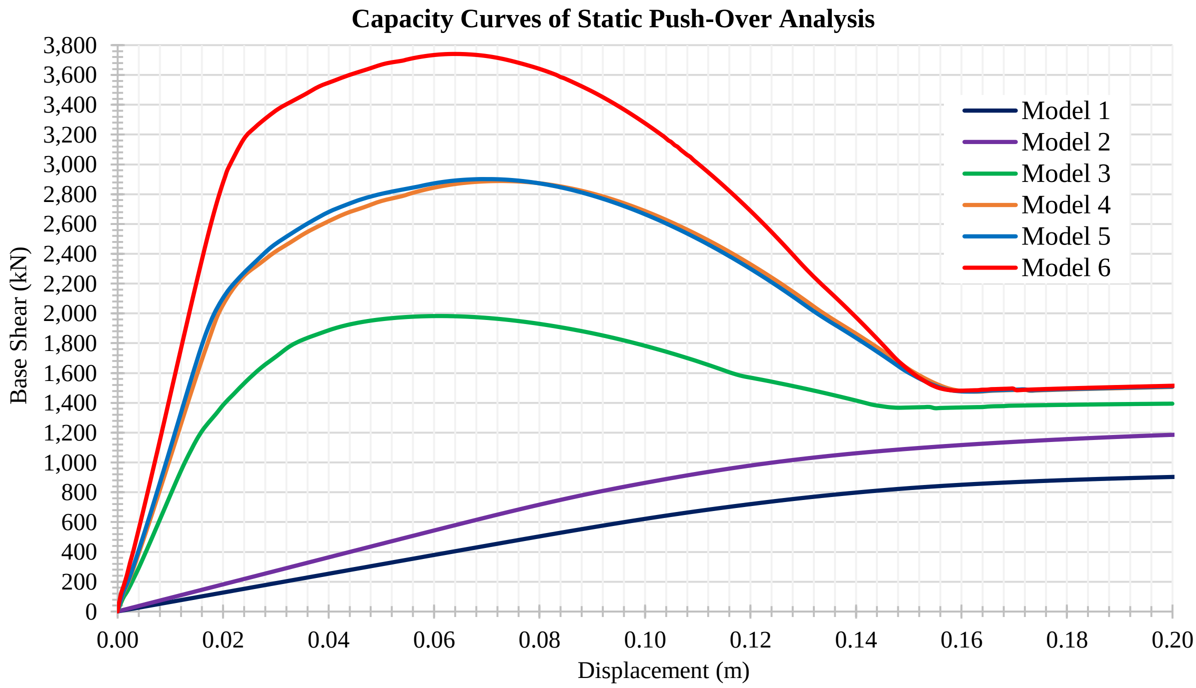


Figure 8 - Comparison of Capacity Curves derived for each model from the static push-over analysis

A deformed shape of Model 5 is shown in Figure 9, indicating a soft storey failure at the 2nd floor as the 1st floor is restrained with steel bracings. On the other hand, the infills can over-strengthen the upper storeys (3rd and 4th) of the structure and induce a soft storey in lower storeys (2nd).

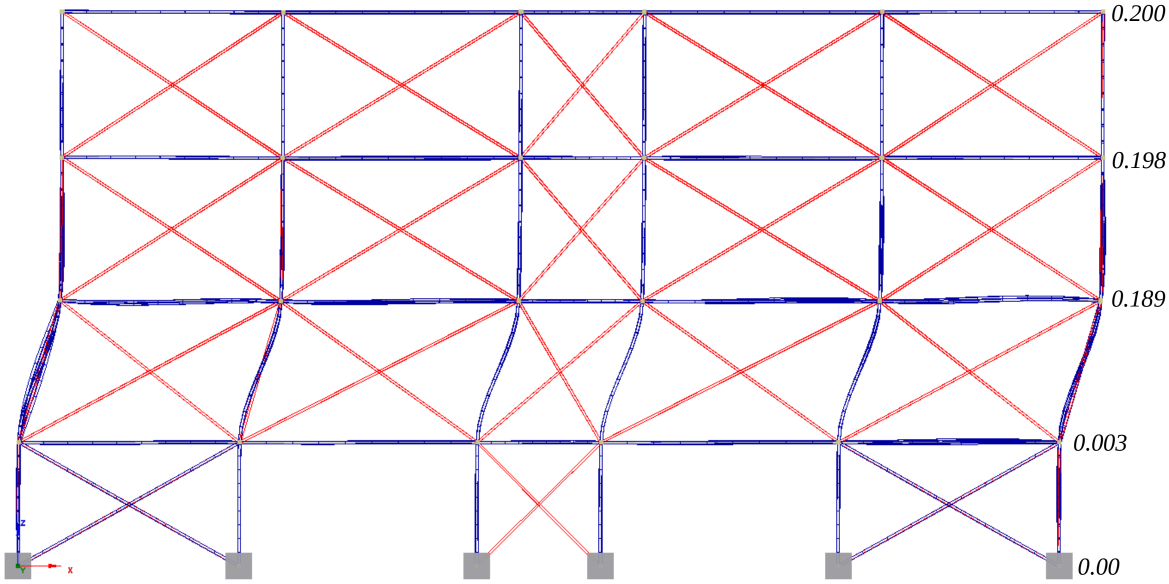


Figure 9 – Side view of deformed shape for Model 5 (units in metre)
(struts representing infills are in red and steel bracings are in blue)

DAMAGE STATES

The inter-storey drift ratio (ISD) is implemented for the fragility calculations of this study. The analysed building has a composite structural system where the initial lateral resistance is provided by the masonry infill walls. As mentioned the steel frame is a dual frame consisting of a moment resisting frame and concentric bracings on both directions at lower level and all floors are braced in y-direction. Upon cracking of the infill walls, the steel frame, which was initially braced by the infills, will provide further lateral resistance. The infill acts as a diagonal compression strut for the surrounding frame. Collapse occurs when the infill walls disintegrate resulting in compression failure of the masonry infills and then the steel frame loses its stability. For developing the fragility curves it is necessary to assign some reasonable "Damage States" for each of the structural elements. Therefore, the damage states defined in Table 5.8 of HAZUS-MH Technical Manual and Table C1-3 of FEMA 356 are compared to the actual relative deformation of the infill panel and its surrounding steel frame presented in Tasnimi and Mohebkhah (2011). Accordingly, three levels of minor, moderate and extensive have been considered for the overall damage of the building based on the performance level of the structure subjected to a given earthquake intensity. Following FEMA 356 these three levels can be described as follow:

1) *Immediate Occupancy:* larger cracks appear on some of the infill walls, some bricks near the beam-column interaction start to break and crush.
2) *Life Safety:* large cracks on most infill walls, a number of bricks dislodged and fall, partial and full collapse of few walls, some walls may bulge out-of-plane, failure at some steel connections, as some critical members may fail, and the structure might undergo a permanent lateral deformation.
3) *Collapse Prevention:* total failure of many infill walls and loss of stability of steel frame, resulting in an imminent or immediate structural collapse.

The ISD values used for the fragility analysis of Model 2 and 5 are given in Table 3. For this purpose exceedance of the selected damage index from the corresponding value associated with each of these performance levels means fragility of the system in that specific performance level.

Table 3 - Damage States assigned for Fragility Analysis

Mid-Rise Dual System Steel Frame (Moment Resisting and Bracing)		
Immediate Occupancy (IO)	**Life Safety (LS)**	**Collapse Prevention (CP)**
0.3 %	0.5 %	2.0 %

Mid-Rise Steel Frame with Un-reinforced masonry Infill Walls		
Immediate Occupancy (IO)	**Life Safety (LS)**	**Collapse Prevention (CP)**
0.3 %	0.7 %	1.6 %

FRAGILITY CURVES

Several approaches exist for deriving analytical fragility functions such as N2 method and HAZUS. For this study, FRACAS (Fragility through Capacity Assessment), a displacement-based procedure, based on the study of Rossetto and Elnashai (2005) is implemented for developing the fragility curves. The method is able to idealise the capacity curves obtained from the static push-over by an elasto-plastic curve. In contrast to other capacity spectrum methods, instead of using a reduction factor or estimating the inelastic response from the elastic one, FRACAS applies a simplified dynamic analysis on the idealised nonlinear SDoF model corresponding to the structures capacity curve. By applying a number of scaled earthquake records with distinct characteristics the inelastic seismic demand is obtained. In this study, the inter-story drift is used as the engineering demand parameter (EDP) and for the intensity measure (IM), PGA is selected.

For the fragility analysis two suites of earthquakes each consisting of seven records and matched to the eurocode 8 spectrum for type I and site class B (semi-compact soil) is selected using Rexel (2010). Suite 1 is matched based on the moment magnitude range of 6 to 7 and epicentral distance of up to 20 km. Suite 2 includes records matched for PGA levels between 0.3g and 0.5g. Each suit of accelerograms are scaled to 35 various PGA levels of 0.03g to 1.12g to create a total of 245 cases of performance points for each fragility curve. The specifications of selected earthquake suits are given in Tables 4 and 5.

Table 4 - *Suite 1* of selected earthquakes based on Moment Magnitude (M_w) and Epicentral Distance (R)

Earthquake Name	**Date**	M_w	**Fault Mechanism**	**Epicentral Distance** (km)	**PGA_X** (m/s^2)	**PGA_Y** (m/s^2)
L'Aquila (mainshock)	06 Apr. 2009	6.3	normal	4.87	6.44	5.35
South Iceland	17 Jun. 2000	6.5	strike-slip	14.56	2.06	4.65
L'Aquila (mainshock)	06 Apr. 2009	6.3	normal	4.39	4.37	4.79
Loma Prieta	18 Oct. 1989	6.9	oblique	7.10	4.70	6.31
Gazli	17 May 1976	6.7	thrust	12.78	7.03	5.96
Gazli	17 May 1976	6.7	thrust	12.78	7.03	5.96
Bam	26 Dec. 2003	6.6	strike-slip	10.16	7.83	6.23

Table 5 - *Suite 2* of selected earthquakes based on Peak Ground Acceleration (PGA)

Earthquake Name	**Date**	M_w	**Fault Mechanism**	**Epicentral Distance** (km)	**PGA_X** (m/s^2)	**PGA_Y** (m/s^2)
Erzincan	13 Mar. 1992	6.6	strike-slip	13.0	3.81	5.02
Montenegro	15 Apr. 1979	6.9	thrust	16.0	3.68	3.55
Montenegro	15 Apr. 1979	6.9	thrust	25.0	4.45	2.99
Friuli (aftershock)	15 Sep. 1976	6.0	thrust	21.0	4.64	4.95
Friuli (aftershock)	15 Sep. 1976	6.0	thrust	14.0	3.39	3.29
Umbria Marche	26 Sep. 1997	6.0	normal	11.0	5.13	4.53
Erzincan	13 Mar. 1992	6.6	strike-slip	13.0	3.81	5.02

For each earthquake suite a fragility curve is obtained by implementing the capacity curve of Model 2 (bare steel frame with concentric bracing) and Model 5 (all infills including opening and solid + all bracings). The fragility curves are presented in Figure 10. Table 6 gives the median and dispersion values obtained for each fragility curve along with dispersion ratios for each model.

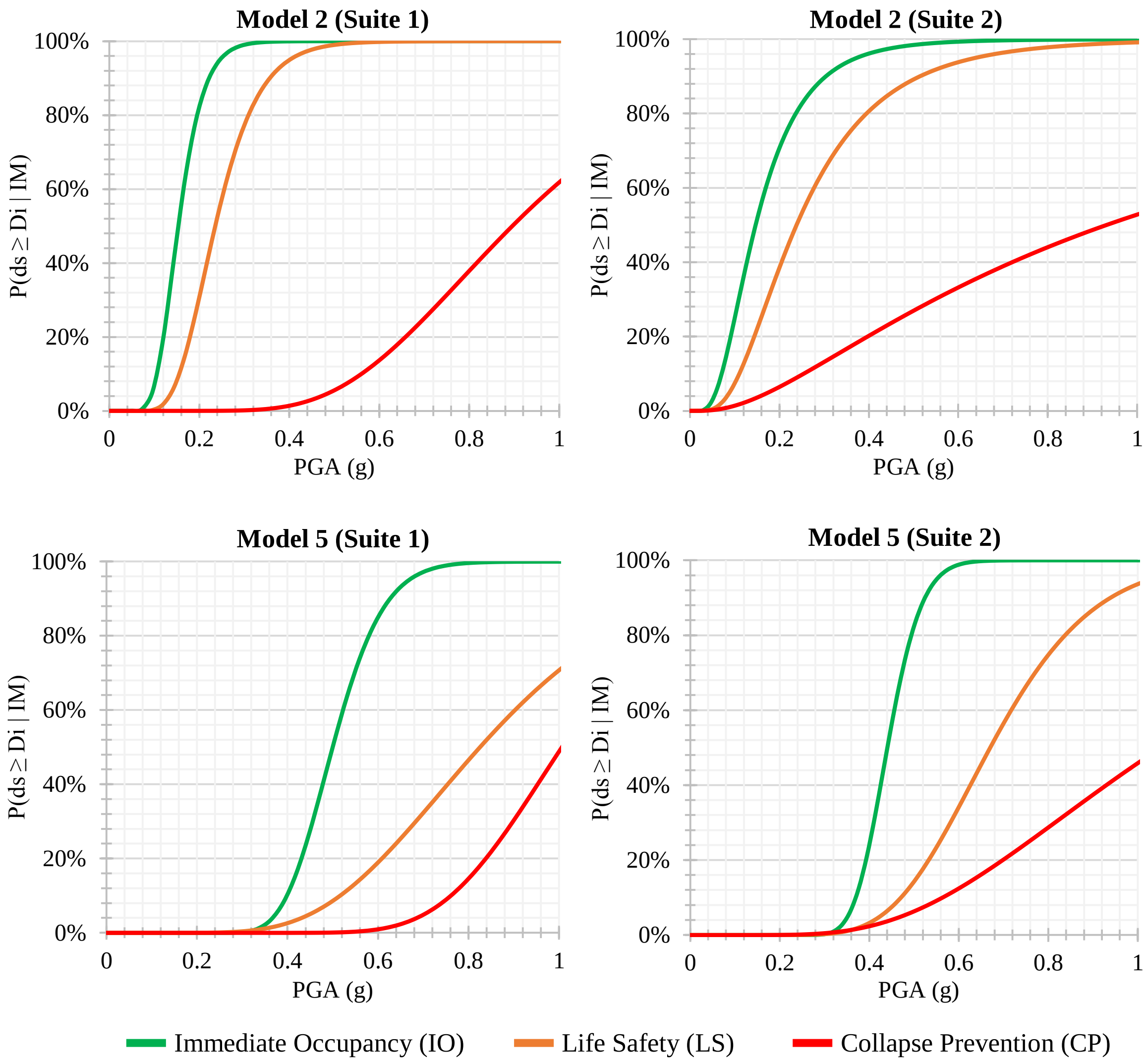


Figure 10 – Fragility Curves obtained for Model 2 and 5, considering Suite 1 and 2 earthquakes

Table 6 – Median (g)[1] and Dispersion[2] values for the obtained fragility curves
Disp. Ratio[3] indicates the ratio of dispersion for suite 2 over suite 1

Damage State	**Model 2 (Suite 1)**		**Model 2 (Suite 2)**		**Model 2**	**Model 5 (Suite 1)**		**Model 5 (Suite 2)**		**Model 5**
	Median[1]	*Disp.[2]*	*Median*	*Disp.*	***Disp. Ratio[3]***	*Median*	*Disp.*	*Median*	*Disp.*	***Disp. Ratio***
IO	0.15	0.288	0.15	0.566	*1.96*	0.40	0.176	0.31	0.136	*0.77*
LS	0.24	0.324	0.24	0.599	*1.84*	0.78	0.317	0.67	0.233	*0.74*
CP	0.90	0.366	0.93	1.011	*2.76*	0.99	0.199	1.05	0.486	*2.43*

It is evident that the fragility performance of a steel framed structure with infill is much higher than the same frame without the infills. Also, Model 5 has a much higher median value compared to Model 2. On the other hand, the dispersion values do not follow a clear path considering any change in the models or earthquake suits and it behaves completely random. In general, the fragility of structure with infill is shifted to higher intensities and reaches the considered damage state at later stages. However, the collapse performance at 1g indicates that at this IM the infills have failed and the structure's behaviour depends solely on the steel frame, therefore both models have quite similar responses. In case of different earthquake suites, both results are inside the confidence bound, while suite 2 causes more damage at a given IM. Furthermore, the shape of Model 5 (Suite 2) IO and LS indicate a more homogeneous distribution of performance points as the curve is more vertical, this is also true according to the dispersion values. While, for the CP case, Model 5 (Suite 1) has a lower dispersion and is more homogeneous.

CONCLUSIONS

The seismic behaviour of a four storey masonry infilled steel frame (MISF) structure is studied by implementing static push-over analysis. A highly accurate 3D model of the index building consisting of all structural elements such as concentric bracings and masonry infill panels (including solid and opening) is created and analysed. Six hypothesis based on the location and presence of structural elements are considered and the resultant static push-over capacity curves are compared. The results indicate that the presence of infills, either solid or with openings, has a significant influence on the lateral stiffness, strength and overall ductility of the whole structure. Therefore, ignoring the effect of masonry infill walls will lead to erroneous calculation of actual response under seismic loading and would be uneconomical. Furthermore, an analytical fragility analysis following a simplified displacement-based procedure is implemented for two of the selected models (Model 2: bare steel frame with concentric bracings ; Model 5: all infills including opening and solid + all bracings). The obtained fragility curves show the effect of infill in increasing the performance of structure at selected intensities in comparison to the bare steel frame. Moreover, it is concluded that selection of applied earthquake suits can also have a significant influence on the resultant fragility curves of the same model.

REFERENCES

ASCE (2007) "Seismic Rehabilitation of Existing Buildings (41-06)", *American Society of Civil Engineers*

BHRC (2005) "Standard No. 2800 Iranian Code of Practice for Seismic Resistant Design of Buildings", *Building & Housing Research Centre*, Tehran, IRAN

Biondi S, Candigliota E, Nuti C (2002) "Tests on different infilled specimens: a critical review of code ultimate state criteria", *Proc. 12th European Conference on Earthquake Engineering*, London, UK, September 9-13, paper 748

Crisafulli FJ (1997) "Seismic behaviour of reinforced concrete structures with masonry infills", *PhD Thesis*, University of Canterbury, New Zealand

CSA (2010) "Design of Masonry Structures (S304.1-04)", 2nd edition, CSA Group

D'Ayala D and Ellul F (2005) "Seismic vulnerability assessment of low engineered masonry infilled reinforced concrete frames", *SECED Young Engineers Conference*, March 1st, Bath, UK

D'Ayala D, Worth J, Riddle O (2009) "Realistic shear capacity assessment of infill frames: comparison of two numerical approaches", *Engineering Structures*, 31:1745-1761

El-Dakhakhni WW, Elgaaly M, Hamid A (2003) "Three-strut model for concrete masonry-infilled steel frames", *Journal of Structural Engineering*, 129:177-185

HAZUS-FEMA (2003) "Technical Manual, Multi-hazard Loss Estimation Methodology, Earthquake Model", HAZUS-MH MR5, Department of Homeland Security, Federal Emergency Management Agency, Mitigation Division Washington D.C., US

Holmes M (1960) "Steel frames with brickwork and concrete infilling", Civil Engineering, *Institute of Technology*, Bradford, 6501:473-478

Iervolino I, Galasso C, Cosenza E (2010) "REXEL: computer aided record selection for code-based seismic structural analysis", *Bulletin of Earthquake Engineering*, 8:339–362

Mander JB, Nair B, Wojtkhowski K, Ma J (1993) "An experimental study of the seismic performance of brick infilled frames with and without retrofit", *Technical Report NCEER-93-0001*, State University of New York at Buffalo

Moghaddam H (2004) "Lateral load behaviour of masonry infilled steel frames with repair and retrofit", *Journal of Structural Engineering*, 130:56-63

Mosalam KM, White N, Gergely P (1997) "Static response of infilled frames using quasi-static experimentation", *Journal of Structural Engineering*, 123:1462-1469

MSJC (2010) "Masonry Standards Joint Committee"

NZSEE (2006) "Assessment and improvement of the structural performance of buildings in earthquakes", *New Zealand Society for Earthquake Engineering*

Personeni S, Di Pilato M, Palermo A, Pampanin S (2008) "Numerical investigations on the seismic response of masonry infilled steel frames", *The 14th World Conference on Earthquake Engineering*, October 12-17, Beijing, China

Polyakov SV (1956) "Masonry in framed buildings", *National Lending Library for Science and Technology*, Boston Spa, Yorkshire, UK

Rossetto T and Elnashai A (2005) "A new analytical procedure for the derivation of displacement-based vulnerability curves for populations of RC structures", *Engineering Structures*, 27:397-409

Seismosoft (2013) "SeismoStruct v6.5, a computer program for static and dynamic nonlinear analysis of framed structures", available from: www.seismosoft.com

Shing PB and Mehrabi AB (2002) "Behaviour and analysis of masonry-infilled frames", *Progress in Structural Engineering and Materials*, 4:320-331

Stafford-Smith B and Carter C (1969) "A method of analysis for infilled frames", *The Structural Engineer*, February 31-48

Tasnimi AA and Mohebkhah A (2011) "Investigation on the behaviour of brick-infilled steel frames with openings, experimental and analytical approaches", *Engineering Structures*, 33:968-980