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Numerical simulation of earthquake response of multi-storey steel frame with SIM infill panels

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ABSTRACT

A new masonry system has been developed at the University of Newcastle. It uses masonry panels made of dry stack semi-interlocking masonry (SIM) units capable of relative sliding in-plane and interlocked to prevent sliding out-of-plane. The major objective of the system is to improve the earthquake performance of framed structures with masonry panels acting as energy dissipation devices (EDD). An experimental program was conducted to evaluate the behaviour of different framed masonry panels. It was found that SIM panels have significant energy dissipation capacity due to friction between the masonry units. This paper presents the results of a numerical simulation of earthquake vibrations on a multi-storey steel frame with three-dimensional finite elements.

1. INTRODUCTION

Masonry is one of the most popular building materials. It has many excellent material properties and proven durability. Over time masonry has evolved from a material used for massive structural walls, which work mainly in compression, to more slender walls, which can also be subjected to tension and shear. However, modern slender unreinforced masonry (URM) walls have poor earthquake resistance due to their high mass and stiffness combined with low tensile and shear strength. The design

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of masonry with improved earthquake resistance presents a challenge for structural engineers.
SIM is a conceptually new type of framed masonry built of dry stack semi-interlocking units. Those used in this study were developed at the University of Newcastle (Totoev 2011). When located in a panel, the SIM units are capable of relative sliding in-plane and locked against relative movement out-of-plane. This could be achieved in two different ways: through topological interlocking of specially shaped bricks and also through mechanical interlocking of conventionally shaped bricks using dowels (Wang 2014), as shown in Figure 1.

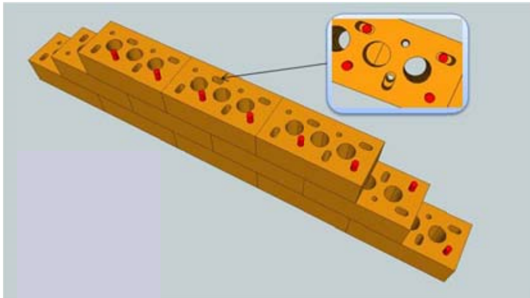
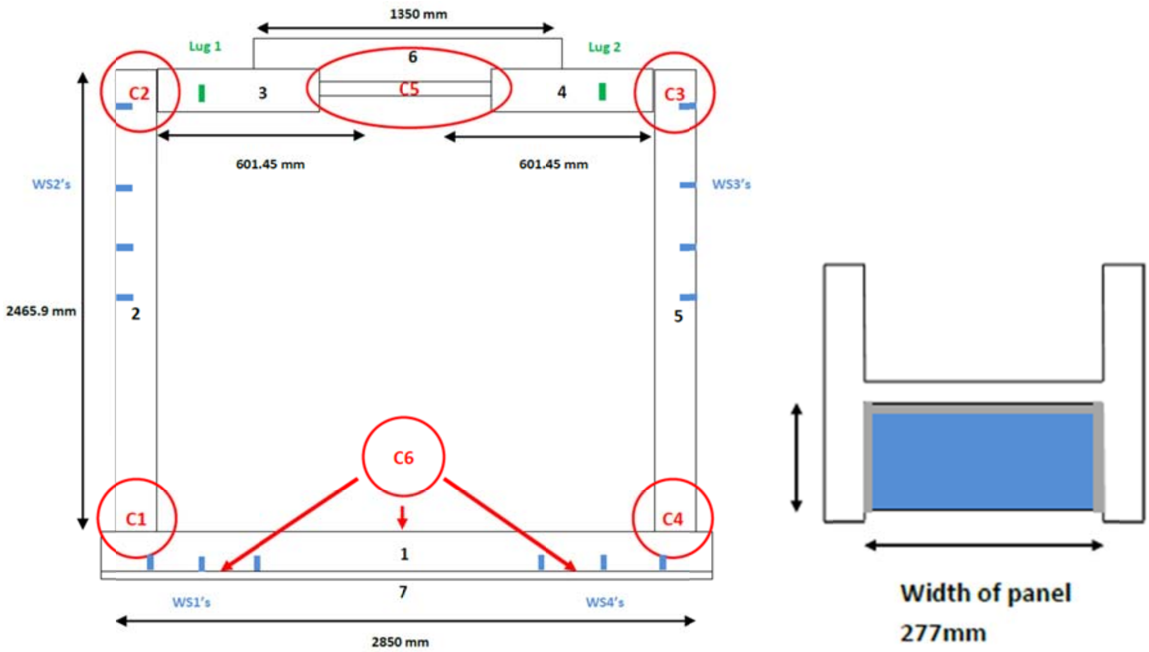
a)  b) 

Fig. 1 Semi-interlocking masonry: a) Topological, b) Mechanical

Previous research on SIM includes in-plane cyclic tests on a reinforced concrete (RC) frame with a SIM infill panel (Lin 2011, Lin 2011, Lin 2012, Lin 2012), and with a steel frame (Totoev 2013), out-of-plane airbag tests on a SIM panel (Totoev 2013), and other supplementary tests (Lin 2011, Lin 2012, Totoev 2012). A finite element numerical model for SIM was developed using DIANA software (Lin 2011) and SeismoStruct software (Totoev 2014).
This paper extends research on the behavior of SIM from the modeling of a single storey, single bay steel frame to the modeling of a multi storey steel frame infilled with SIM. Models with and without a gap between the top of the panel and the frame are considered. Numerical simulations include a non-linear static (push over) analysis and a dynamic (earthquake time history) analysis of such frames.

2. SUMMARY OF EXPERIMENTAL PROGRAM

The cyclic displacement test was performed to evaluate the in-plane behavior of the topologically interlocking SIM panel in a steel frame. The plan and elevation of the steel testing frame is shown in Fig. 2 (Healy 2011). The columns consisted of 310UC137 sections. The base beam consisted of a 310UC137 section and a 23mm plate welded along the underside of the base. The top beam consisted of two 310UC137 sections and a T section using a 250mm by 30mm plate and a 300mm by 20mm plate welded together. The corners were connected by Class 8.8 M20 bolts with in-plane bending.



Schedule of Members		
Member Label	**Member type**	**Length of Member (mm)**
1	310UC137	2850
2	310UC137	2465.9
3	310UC137	601.45
4	310UC137	601.45
5	310UC137	2465.9
6	T section	1350
7	Flat bar	2850 (23mm thick)

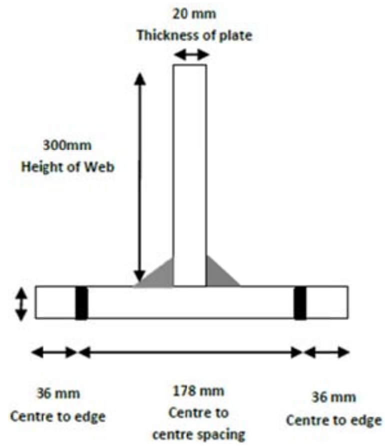


Fig. 2 Details of in-plane test frame (Healy 2011)

The SIM infill panel was built of topologically interlocking bricks (Fig. 1a). The bricks were 230x110x76 mm, and the SIM infill panel was 2400x2400 mm.
All in-plane tests were displacement controlled according to the displacement history shown in Fig. 3. A vertical load of 80kN was applied to the bare steel frame test and the steel frame with SIM infill with gap test. For the steel frame with SIM infill without gap test, the vertical load was increased to 100kN and a thin wood plate and dental plaster was placed between the panel and the top of the frame to completely close the gap.
The force and displacement was recorded using linear variable displacement transducers (LVDTs) at the top of the steel frame.

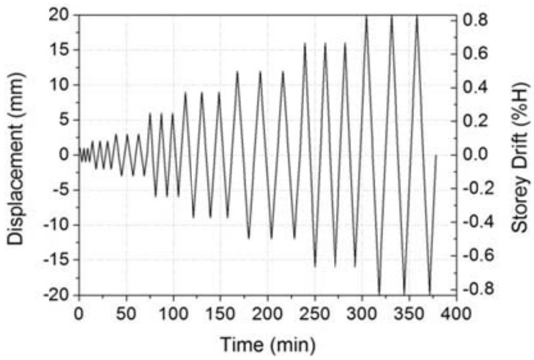
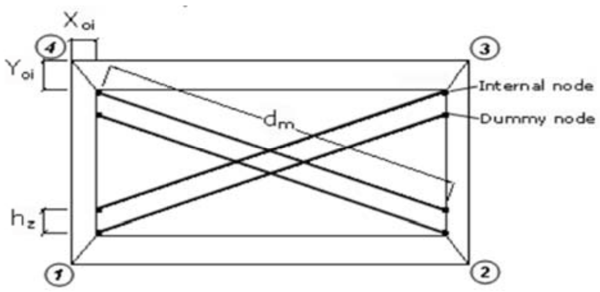


Fig. 3 Applied displacement history

3. MODELLING

Previous modelling of SIM in concrete frames was conducted using SeismoStruct software (Totoev 2014). The same software was used to model SIM in steel frames. In SeismoStruct, each panel is represented by two diagonal masonry strut elements and a shear strut element (Fig. 4) that is activated in the direction of the compressed diagonal strut and transfers shear from the top to the bottom of the panel (Seismosoft 2014). The masonry strut hysteresis model is used for the diagonal struts and the special bilinear hysteresis model is used for the shear strut element. Material damping of various kinds can be specified for the masonry infill panel.



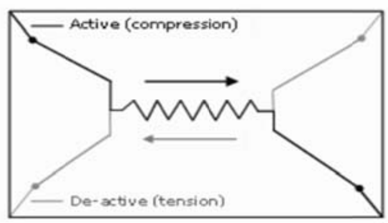


Fig. 4 Infill panel element: a) Compression/Tension struts, b) Shear strut (Seismosoft 2014)

3.1 Model verification of experimental bare steel frame

The first step of model verification was the eigenvalue analysis of the bare steel frame. The natural frequency was 14.17 Hz, compared to 14.75Hz in the experimental bare frame test. The cyclic displacement history at the top of the frame is shown in Fig. 3. SeismoStruct input parameters for this frame are presented in Table 1.

Table. 1 Input parameters for bare steel frame

Material Name	Property	
Beam Column	• Modulus of elasticity • Yield strength • Strain hardening parameter • Fracture/buckling strain • Specific weight	200000MPa 340MPa 0.005 0.1 7.8E-005N/mm^3
Bolt	• Modulus of elasticity • Yield strength • Strain hardening parameter • Fracture/buckling strain • Specific weight	210000MPa 640MPa 0.03 0.35 7.85E-005N/mm^3

The experimental hysteretic curves were compared to the numerically simulated ones in terms of their envelope curves, as shown in Fig. 5c - Fig. 5f. The match between the experiment and the simulation was good in terms of the maximum force, the maximum displacement, and the area of the envelope curve. It was confirmed that the FE model well represents the real experimental testing steel frame.

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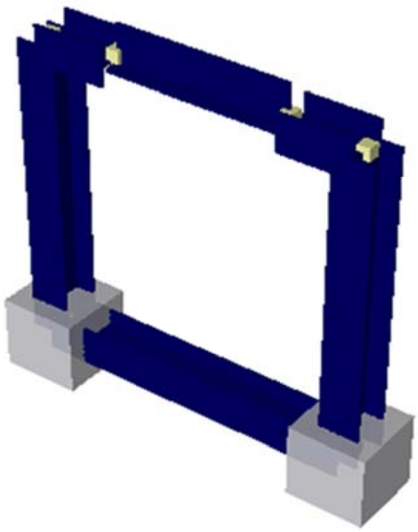
b) 

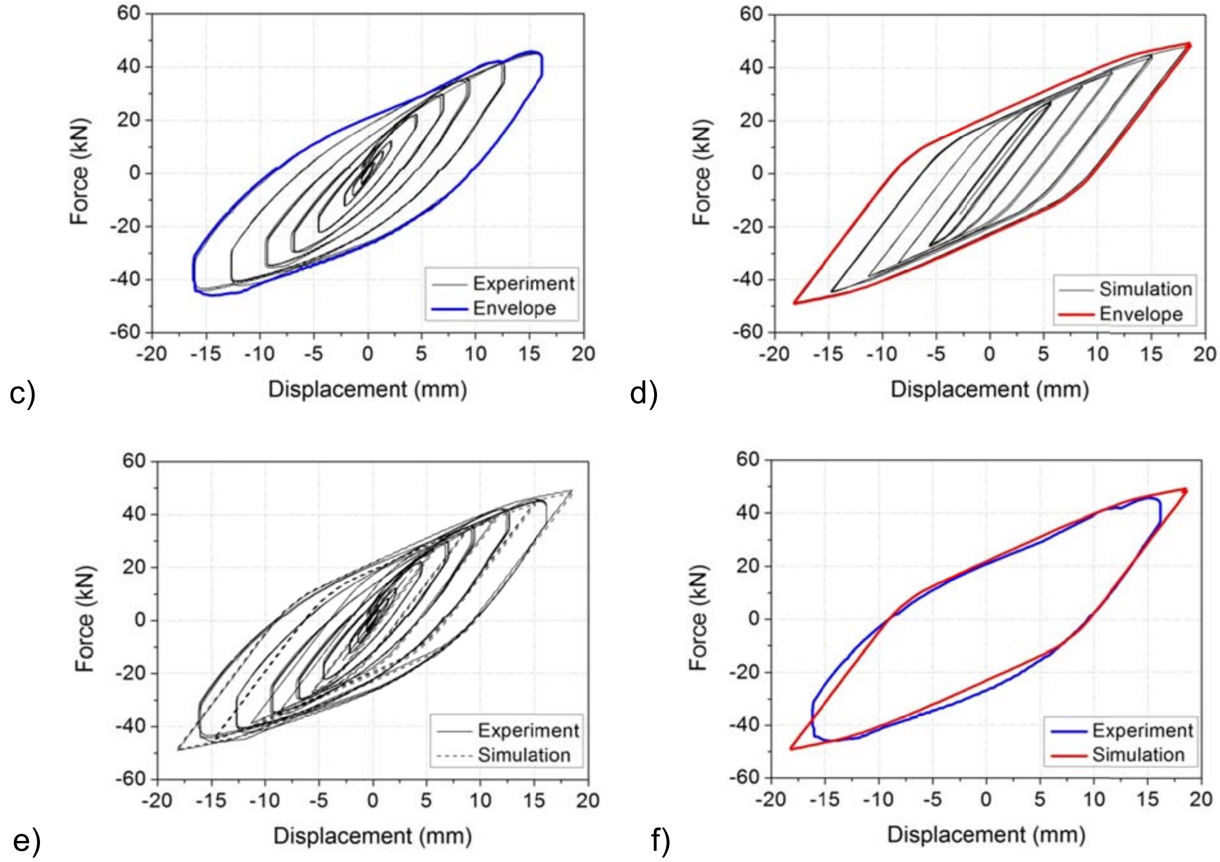


Fig. 5 Model verification: a) Experimental photo of bare steel frame, b) FE model of bare steel frame, c) Experimental hysteretic curves and envelope curve, d) Simulated hysteretic curves and envelope curve, e) Experiment and simulation comparison, f) Envelope curve comparison.

3.2 Model verification of experimental steel frame with SIM panel

The steel frame infilled with SIM panel was controlled with displacement history as shown in Fig. 3. Two panels were modelled: one with a gap between its top and the frame (gap case), and the other with no gap between the panel and the frame (no gap case). SeismoStruct input parameters for the steel frame are the same as those shown in Table 1. The input parameters for the SIM panels are listed in Table 2.

The experimental hysteretic curves were compared to the numerically simulated ones in terms of their envelope curves, as shown in Fig. 6c - Fig. 6j. The match was found to be very satisfactory in terms of the maximum force, the maximum displacement, and the area of the envelope curve. It is concluded that both SIM panel modes, with and without gap, have been verified and adequately represent real SIM panels.

Table. 2 Input parameters for SIM panels model

Property	Value (with gap)	Value (no gap)
Panel Thickness	110mm	110mm
Out-of-plane failure drift (% of vert. panel side)	7.6%	7.6%
Strut Area 1	$4666mm^2$	$18664mm^2$
Strut Area 2 (% of strut Area 1)	50%	100%
Equival. Contact length hz (% of vert. panel side)	21.8%	21.8%
Horiz. Offset xo (% of horiz. panel side)	0.3%	0.3%
Vert. Offset yo (% of vert. panel side)	0.3%	0.3%
Proportion of stiffness assigned to shear (%)	5%	70%
Specific weight	$0.0000235N/mm^3$	$0.0000235N/mm^3$
Initial Young modulus-E_m	2500MPa	2500MPa
Compressive strength-$f_m\theta$	2.5MPa	2.5MPa
Tensile strength-f_t	0.0MPa	0.0MPa
Strain at maximum stress-ε_m	0.006	0.002
Ultimate strain-ε_u	0.024	0.024
Closing strain-ε_{cl}	0.004	0.004
Strut area reduction strain-ε_1	0.0006	0.0006
Residual strut area strain-ε_2	0.001	0.001
Starting unload. stiffness factor-γ_{un}	2.5	2.5
Strain reloading factor-α_{re}	0.2	0.2
Strain inflection factor-α_{ch}	0.7	0.7
Complete unloading strain factor-β_a	2.0	2.0
Stress inflection factor-β_{ch}	0.9	0.9
Zero stress stiffness factor –γ_{plu}	1.0	1.0
Reloading stiffness factor-γ_{pr}	1.5	1.5
Plastic unloading stiffness factor-e_{x1}	3.0	3.0
Repeated cycle strain factor-e_{x2}	1.0	1.0
Shear bond strength (MPa)	0.03MPa	0.1MPa
Friction coefficient	1.2	1.2
Maximum shear resistance (MPa)	0.30MPa	1MPa
Reduction shear factor	1.5	1.5

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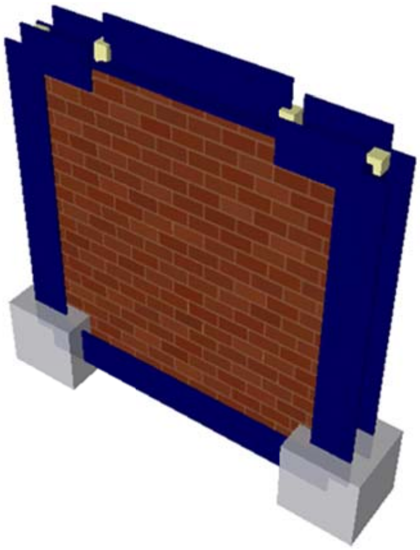

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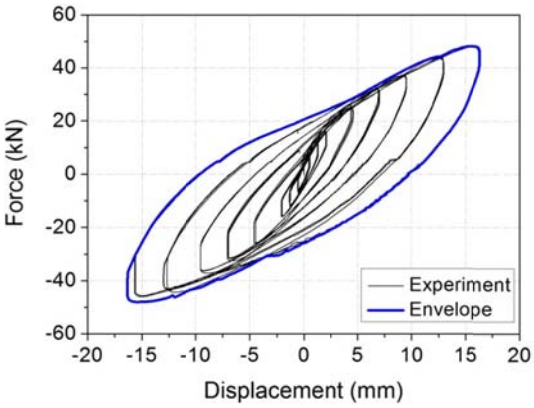
Force (kN)
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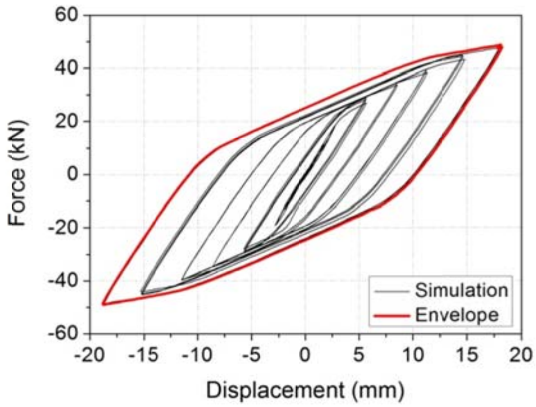
Force (kN)
Displacement (mm)
Simulation
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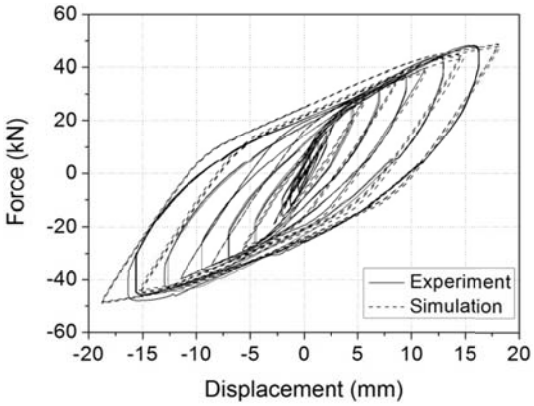
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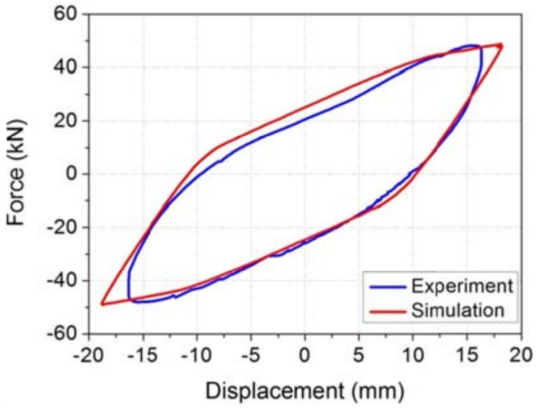
Force (kN)
Displacement (mm)
Experiment
Simulation

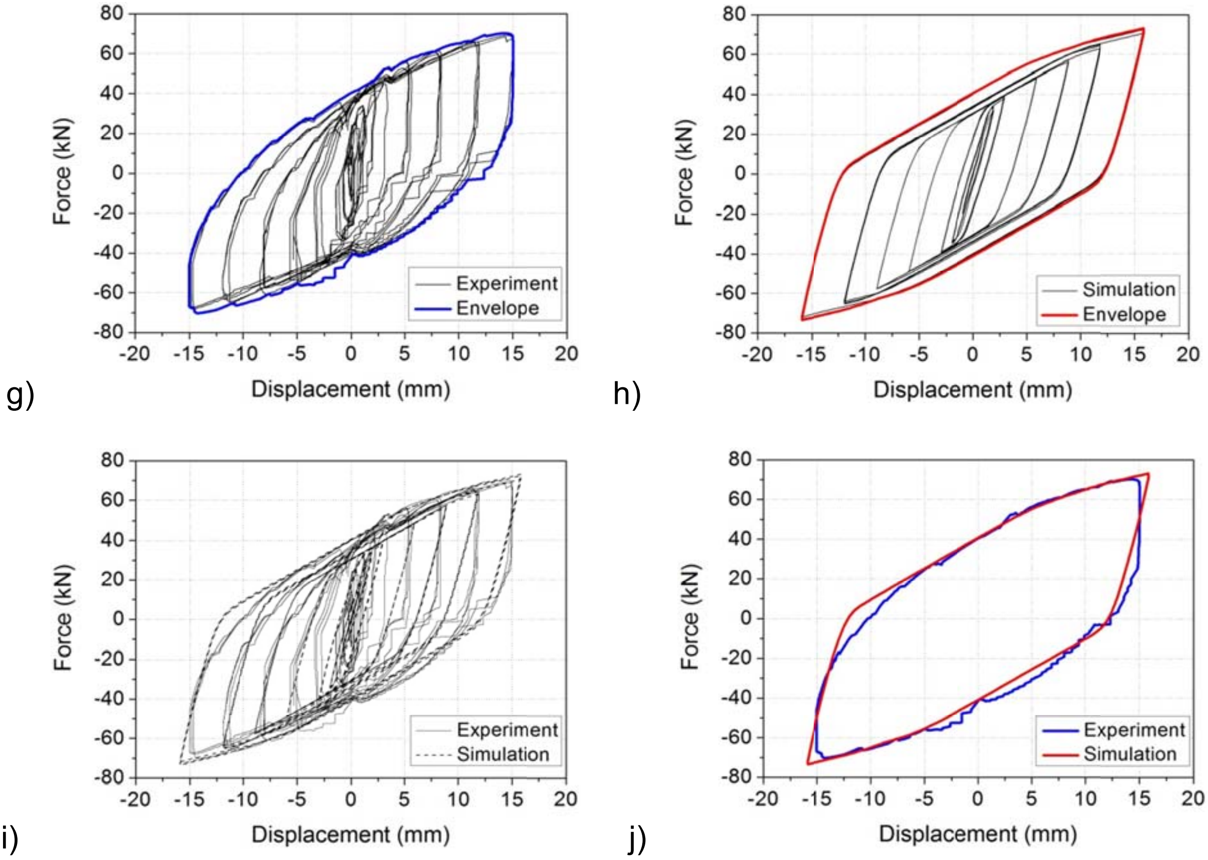

Fig. 6 Model verification: a) Experimental photo of steel frame with SIM panel, b) FE model of steel frame with SIM panel, c) Experimental hysteretic curves and envelope curve in gap case, d) Simulated hysteretic curves and envelope curve in gap case, e) Experiment and simulation comparison in gap case, f) Envelope curve comparison in gap case, g) Experimental hysteretic curves and envelope curve in no gap case, h) Simulated hysteretic curves and envelope curve in no gap case, i) Experiment and simulation comparison in no gap case, j) Envelope curve comparison in no gap case.

3.3 Model verification of reinforced concrete (RC) frame with a traditional URM panel

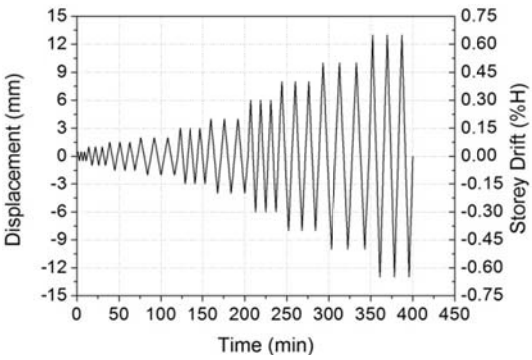

Fig. 7 Applied displacement history (adopted from Lin 2011)

The bare RC frame has been verified elsewhere (Totoev 2014). The response of a RC frame with URM panel to the monotonic cyclic displacements at the top of the frame was simulated as shown in Fig. 7. The SeismoStruct input parameters for the RC frame is shown in Totoev (2014). The input parameters for the URM panel are listed in Table 3.

Table. 3 Input parameters for URM panel model

Property	Value
Panel Thickness	113mm
Out-of-plane failure drift (% of vert. panel side)	0.5%
Strut Area 1	46759mm^2
Strut Area 2 (% of strut Area 1)	40%
Equival. Contact length hz (% of vert. panel side)	11.4%
Horiz. Offset xo (% of horiz. panel side)	5.3%
Vert. Offset yo (% of vert. panel side)	10%
Proportion of stiffness assigned to shear (%)	5%
Specific weight	0.0000235N/mm^3
Initial Young modulus-E_m	4000MPa
Compressive strength-$f_m\theta$	5.5MPa
Tensile strength-f_t	0.575MPa
Strain at maximum stress-ε_m	0.0012
Ultimate strain-ε_u	0.024
Closing strain-ε_{cl}	0.003
Strut area reduction strain-ε_1	0.0003
Residual strut area strain-ε_2	0.0006
Starting unload. stiffness factor-γ_{un}	1.7
Strain reloading factor-α_{re}	0.2
Strain inflection factor-α_{ch}	0.7
Complete unloading strain factor-β_a	2.0
Stress inflection factor-β_{ch}	0.9
Zero stress stiffness factor –γ_{plu}	1.0
Reloading stiffness factor-γ_{pr}	1.1
Plastic unloading stiffness factor-e_{x1}	3.0
Repeated cycle strain factor-e_{x2}	1.0
Shear bond strength (MPa)	0.3MPa
Friction coefficient	0.7
Maximum shear resistance (MPa)	1.0MPa
Reduction shear factor	1.5

The experimental hysteretic curves were compared to the numerically simulated ones in terms of their envelope curves, as shown in Fig. 8c - Fig. 8f. The difference in the maximum displacement is due to the fact that cyclic displacements in the experiment were applied at the top of the frame but the displacement response was recorded at the top of the panel. The simulated applied and recorded displacements were both at the neutral axis of the girder. The difference in the area inside the envelope curves is due to existing cracking at the top and bottom of the columns in the RC frame itself before the in-plane test for the RC frame with URM panel. The match between the experiment and the simulation is considered adequate although not perfect and confirms that the URM panel model satisfactorily represents the real URM panel.

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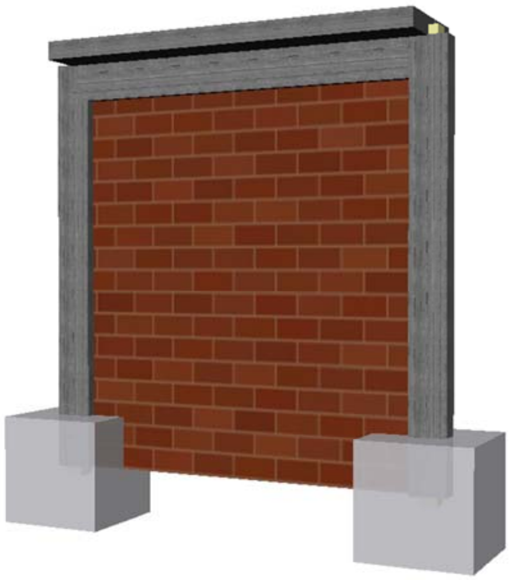

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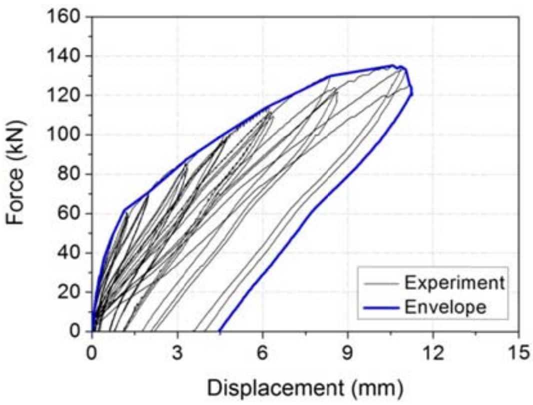

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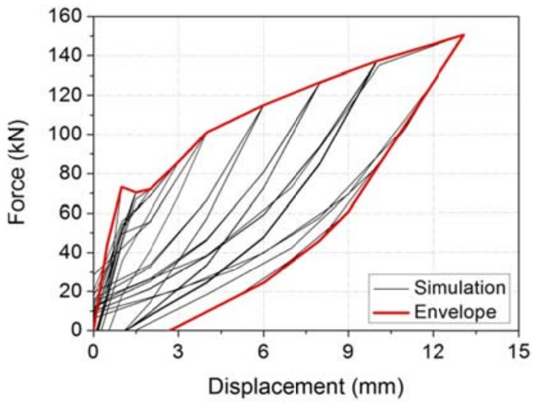

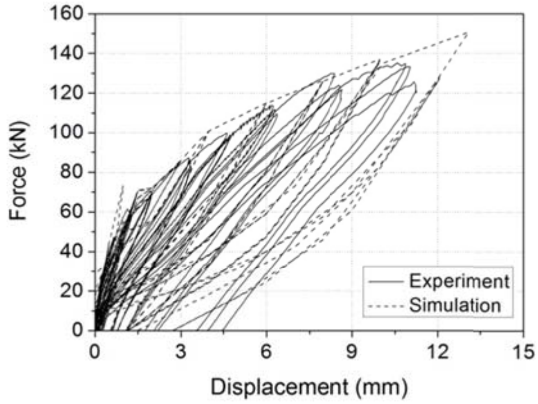

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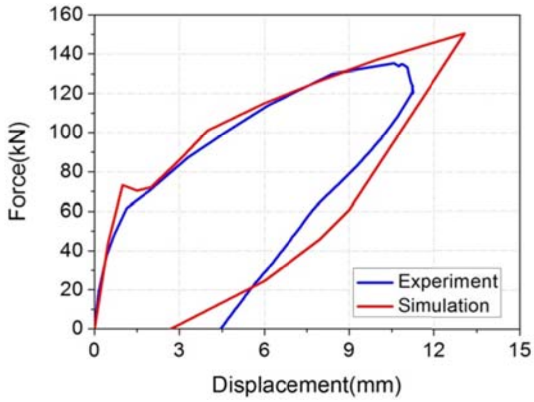


f)

Fig. 8 Model verification: a) Experimental photo of RC frame with URM panel, b) FE model of RC frame with URM panel, c) Experimental hysteretic curves and envelope curves, d) Simulated hysteretic curves and envelope, e) Experiment and simulation comparison, f) Envelope curve comparison.

3.4 Model verification of multistorey bare steel frame

The structure was a three storey two bay steel moment frame tested at the University of Kyoto. Two of these frames are shown side by side in Fig. 9 (Nakashima 2006). Details of the frame are presented elsewhere (Matsumiya 2004, Liu 2004, Nakashima 2006).

For a quasi-static cyclic test, two quasi-static jacks were arranged for horizontal loading. Each jack was placed at the middle of the third storey in each frame. The two frames acted nearly independently. The applied history displacement is shown in Fig. 10.

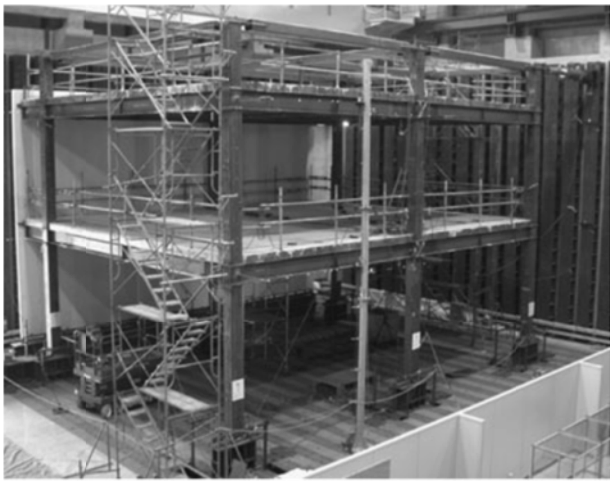


Fig. 9 Three storey two bay steel frame (Nakashima 2006)

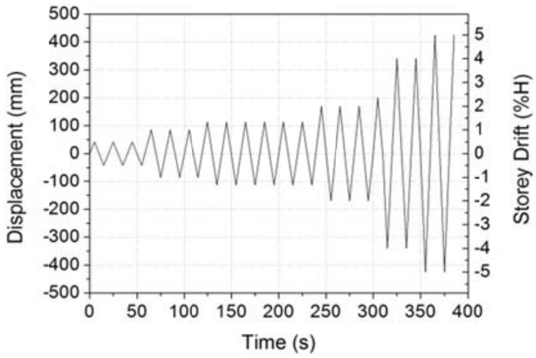


Fig. 10 Applied displacement history (adopted from Nakashima 2006)

The numerical model for the multistorey steel frame was created using SeismoStruct with the input parameters reported in Nakashima (2006). An overview of the finite element model in SeismoStruct is shown in Fig. 11. The nonlinear static time-history numerical simulation was compared to the experimental results shown in Fig. 12a. The simulated pushover curve was compared to the experimental hysteretic curve results shown in Fig. 12b. Failure of the experiment tests and the simulated pushover analysis happened at the same displacement. The match is satisfactory, confirming that the FE model for the multi storey steel frame adequately represents the real tested frame.

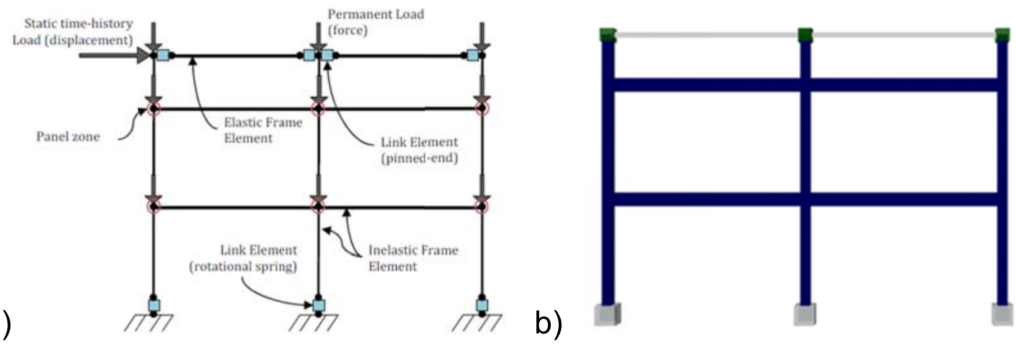


Fig. 11 Overview of finite element model in SeismoStruct (Seismosoft 2014)

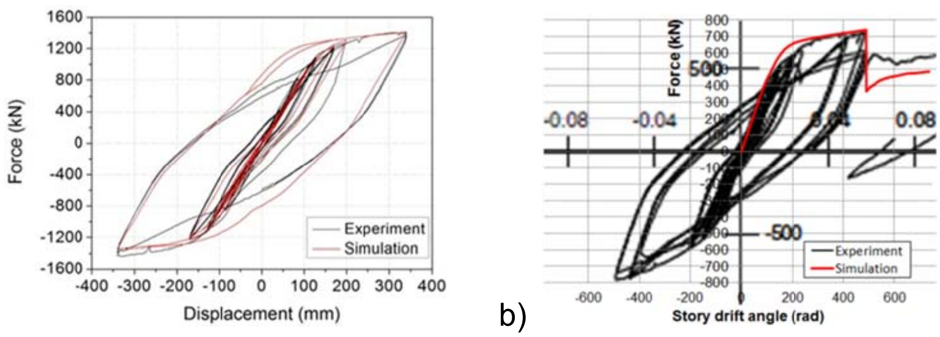


Fig. 12 Model verification: a) Experimental and simulated hysteretic curve comparison, b) Experimental hysteretic curve and simulated pushover comparison.

4. NUMERICAL SIMULATION

As the case study of the three storey, two bay steel moment frame includes link elements and springs. In order to keep the structure stable, the third storey with link elements and springs at the base points were deleted. All the other properties of the frame remained the same. The two storey, two bay steel moment frame with rigid connections between beams and columns was chosen for numerical simulation.

Four numerical models have been created for numerical simulations and comparison:

- Multistorey bare steel frame
- Multistorey steel frame infilled with SIM panels with gap between the panels and the frame;
- Multistorey steel frame infilled with SIM panels without gap between the panels and the frame;
- Multistorey steel frame infilled with traditional unreinforced masonry panels.

The models for the bare multi storey steel frame and the multi storey steel frame with different infill panels are shown in Figure 13.

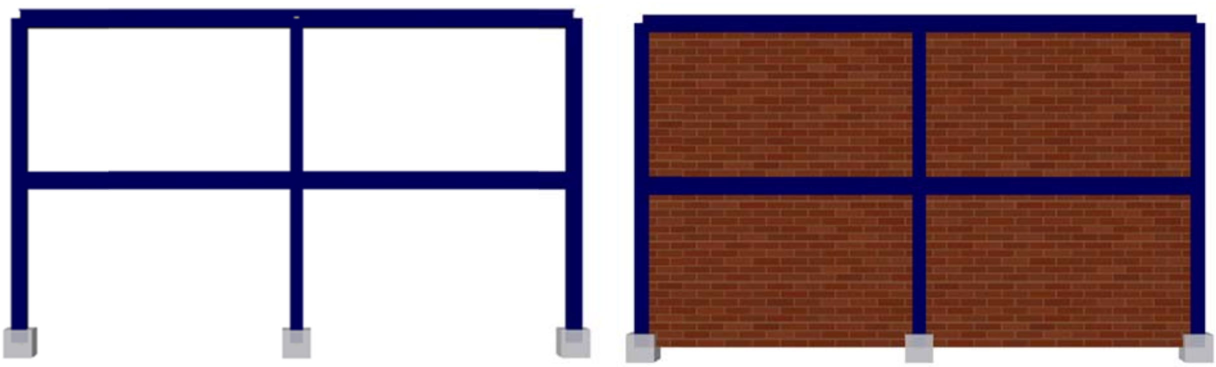


Fig. 13 Finite element model: a) Bare multistorey steel frame, b) Multistorey steel frame with infill panels

Two types of numerical simulations were performed:

- Non-linear static (pushover) analysis under monotonic loading;
- Non-linear response history analysis under synthetic earthquake ground motion.

4.1 Non-linear static analysis

This simulation was displacement controlled with displacement applied at the top of the frame in 1mm increments. The pushover curves and their elastic-plastic approximations for all models are summarized in Fig. 14. The displacement ductility capacities can be calculated by Eq. (1), and the comparison of displacement ductility for all models is shown in Table 4.

$$\mu_{\Delta} = \frac{\Delta_u}{\Delta_y} \tag{1}$$

Where, Δ_y is yield displacement, Δ_u is ultimate displacement, and μ_Δ is displacement ductility capacity.

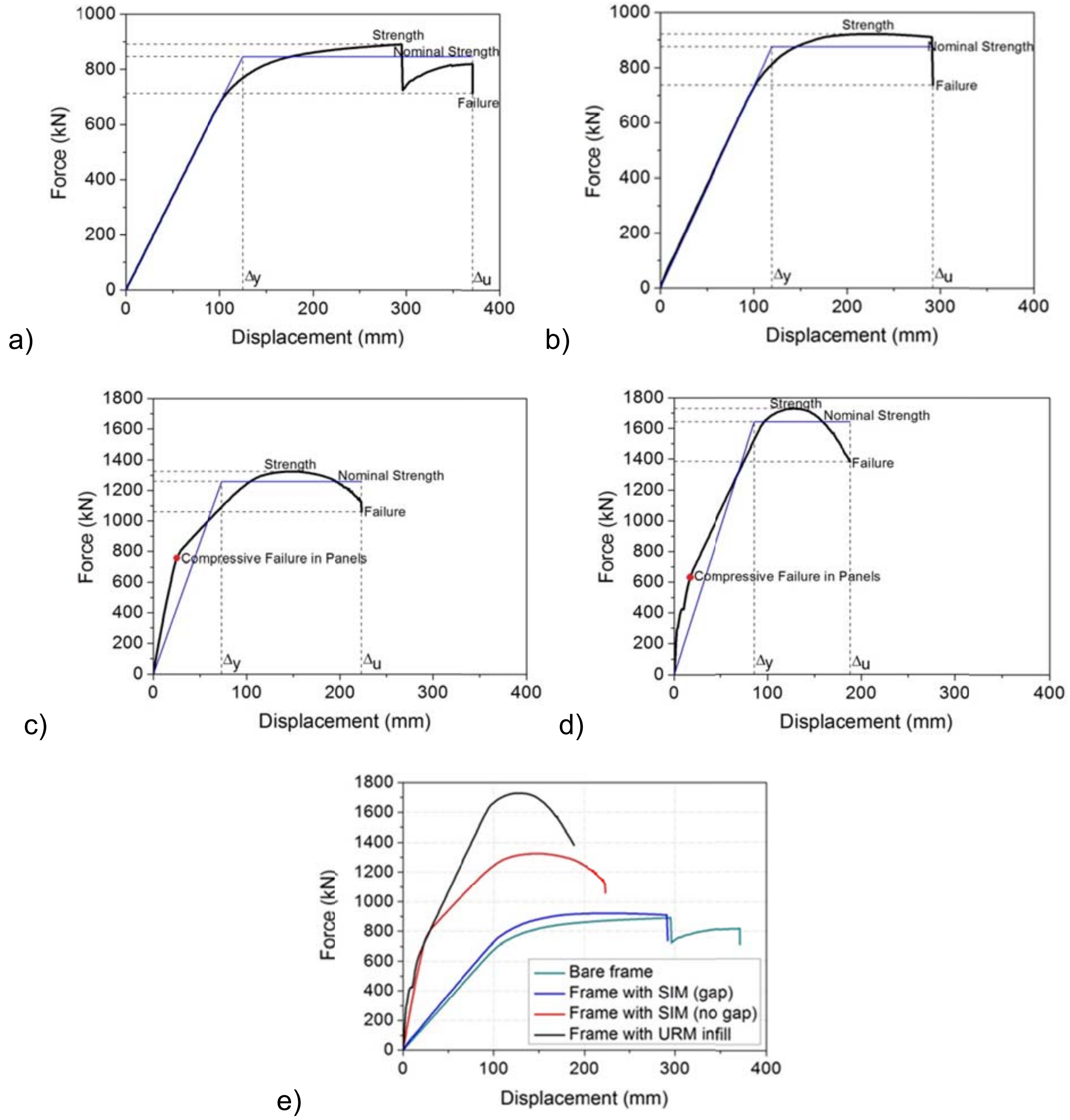


Fig. 14 Push over simulation results: a) Bare steel frame, b) Steel frame with SIM infills with gaps, c) Steel frame with SIM infills without gaps, d) Steel frame with URM infills, e) Comparison of push over curves.

Table. 4 Comparison of displacement ductility

	Δ_y (mm)	Panel failed in compression at (mm)	Δ_u (mm)	μ_Δ
Bare frame	125	—	371	2.97
Frame with SIM infill (gap)	119	—	292	2.45
Frame with SIM infill (no gap)	73	26	223	3.04
Frame with URM infill	86	17	188	2.19

It was assumed that failure occurs after 20% degradation of the peak strength and the nominal strength is 5% lower than the peak strength. There was no panel failure happened in the bare frame and in the frame with SIM infill with gap, thus these two structures remain safe until ultimate displacement limit. However, the frame with SIM infill without gap and the frame with URM panels suffered compressive failure of infill panels even before they reached yield displacement limit. The ultimate displacement is high because the frame is very strong relative to the panels. When the panels failed, the steel frame still resisted load. However, after the fail of infills, these structures are no longer safe.

It is important to note that replacing traditional URM panels with SIM panels could increase (from 2.19 to 2.45) the displacement ductility capacity of the frame. This also reduces the stiffness of the structure, which is not critical because the URM panels are assumed to be non-structural elements and the steel frame is designed to carry all loads. On the other hand, replacing URM panels with SIM panels with gap results in the infilled frame having significantly improved yield displacement. Because of similar pushover curves, it is less likely that SIM panels, particularly with the gap, would negatively interfere with the natural vibrations of the frame during earthquake.

4.2 Response history dynamic analysis

This simulation was performed for two synthetic earthquake ground motions: (i) corresponding to a "Rare" earthquake event with 475 years return period (see Fig. 15a), and (ii) corresponding to a "Very Rare" earthquake event with 975 years return period, shown in Fig. 15b. The damping ratio of 0.08 was assumed for SIM based on previous analysis (Totoev 2012).

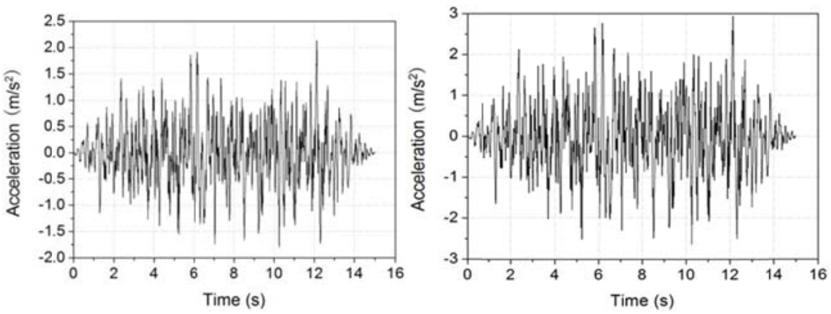
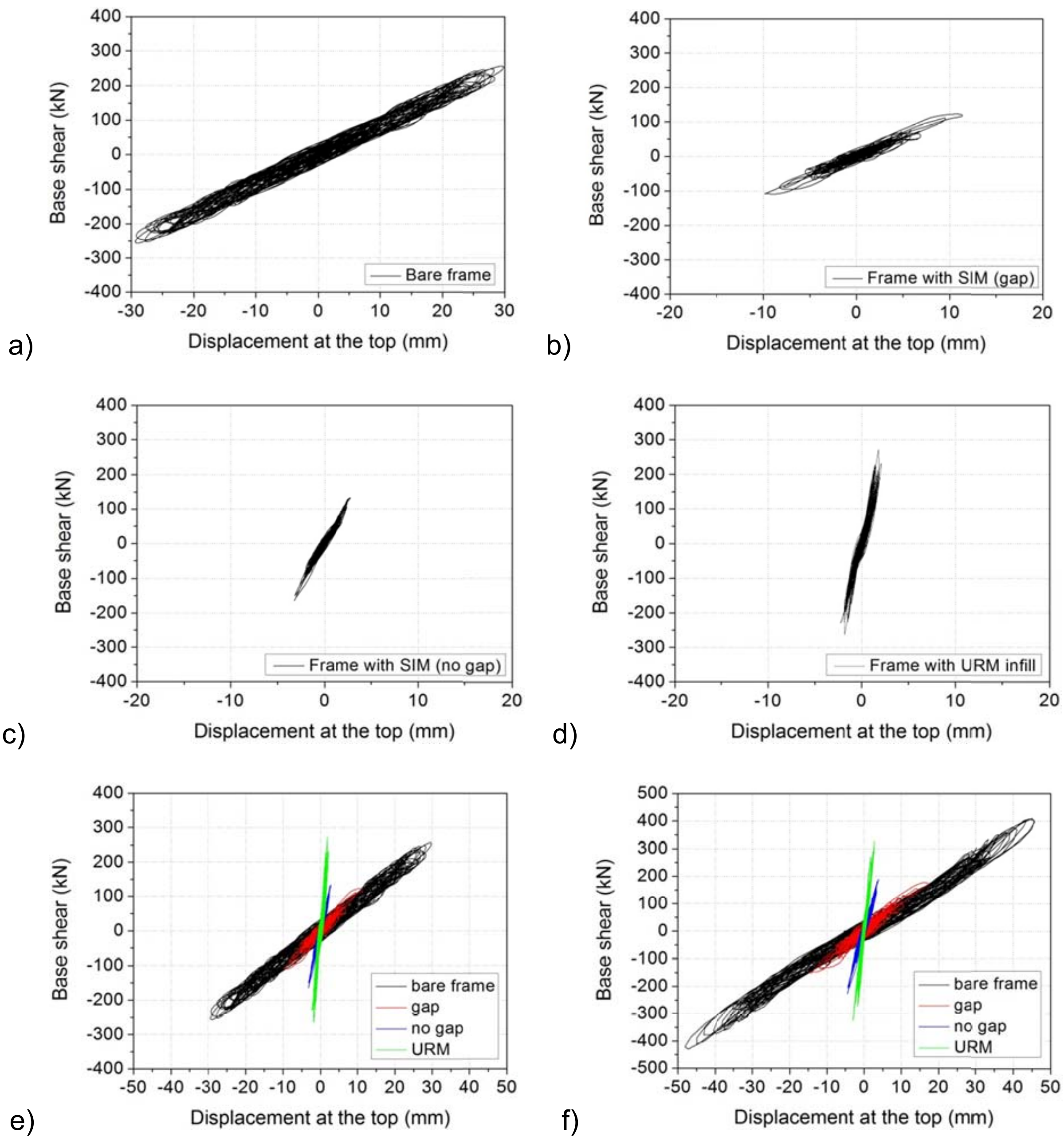


Fig. 15 Ground motion acceleration: a) 475 year return period earthquake, b) 975 year return period earthquake

The earthquake response of the bare two storey steel frame, the frame infilled with SIM panels with and without gap, and the frame infilled with traditional URM panels is presented by the base shear versus global displacement hysteretic curves in Fig. 16 for "Rare" and "Very Rare" seismic events.



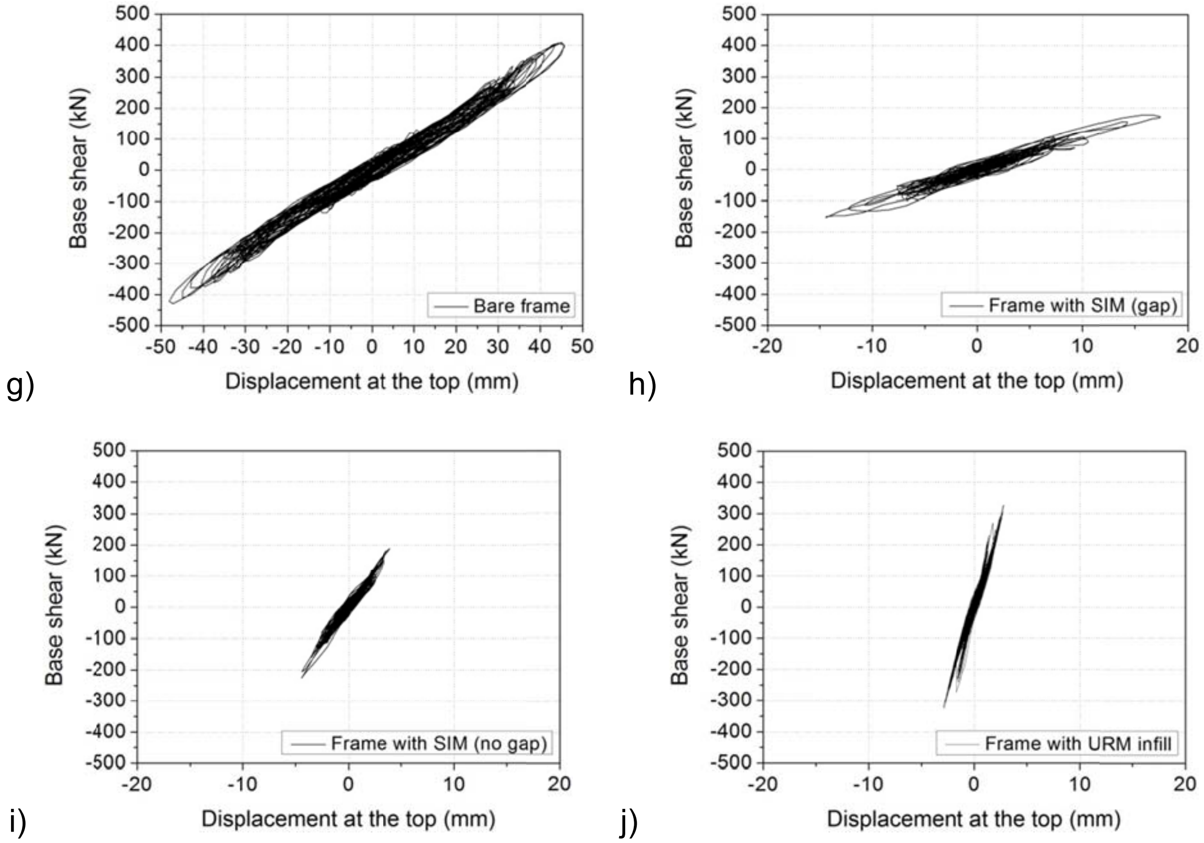


Fig. 16 Simulation of earthquake vibrations: a) Bare frame during “Rare” earthquake, b) Frame with SIM infills with gaps during “Rare” earthquake, c) Frame with SIM infills without gaps during “Rare” earthquake, d) Frame with URM infills during “Rare” earthquake, e) Simulated comparison during the “Rare” earthquake, f) Simulated comparison during the “Very Rare” earthquake, g) Bare frame during “Very Rare” earthquake, h) Frame with SIM infills with gaps during “Very Rare” earthquake, i) Frame with SIM infills without gaps during “Very Rare” earthquake, j) Frame with URM infills during “Very Rare” earthquake

During the “Rare” earthquake, the bare frame vibrated entirely within its yield displacement limits of 125mm with a maximum base shear of 250kN. The vibrations of the frame infilled with SIM with gap were about 10mm, which is well below its yield limit. The vibrations of the frame infilled with SIM without gap were also less than 26mm. The frame with traditional URM infill completely stayed within 17mm. Therefore, there was no damage in all case study structures during the “Rare” earthquake.

In the “Very Rare” earthquake, the bare frame went well below its yield limit and strength. The vibrations of the frame infilled with SIM with gap were less than 20mm, which is well below its yield limit. The vibrations of the frame infilled with SIM without gap were also less than 26mm. The frame with URM infills vibrated entirely within 17mm. Thus, there was no damage in all case study structures during the “Very Rare” earthquake.

5. CONCLUSIONS

The following conclusions can be made based on the numerical study of the multi storey steel frame with different types of infill panels:

- The steel frame infilled with SIM has lower structural stiffness compared to the frame with traditional URM panels;
- The frame with SIM infills with gaps has a higher energy dissipation compared to the frame with URM infills;
- The SIM infilled frames have a lower force capacity than the frame with URM panels;
- The SIM infilled frames have a higher displacement ductility capacity than the frame with URM panels;
- For the SIM infilled frame, the amplitude of structural vibrations and induced internal forces during an earthquake remains within its strength and displacement capacities. SIM panels acted as EDDs;
- The frame infilled with SIM has structural properties, which in combination help it reduce the base shear force during extreme earthquake events;
- SIM infill panels could be a variable alternative to the traditional URM panels in seismic areas.

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