Multi-Platform Simulation of Infilled Shear-Critical Reinforced Concrete Frames Subjected to Earthquake Excitations

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Abstract

This paper proposes a multi-platform simulation method for seismic performance assessment of masonry infilled reinforced concrete (RC) frames, especially for those who tend to fail in shear during an earthquake due to inadequate reinforcing details. The method is based on a micro-macro modeling approach where a detailed finite element (FE) model of the RC frame is incorporated with a strut model of the infill wall. It takes advantage of the strut model in terms of computational efficiency and the FE model based on the Modified Compression Field Theory (MCFT) to capture the nonlinear behaviour of the RC frame with explicit modeling of the beam-column joint failure and shear failure of the frame. The proposed method is validated against previously tested frames subjected to lateral loads, and its advantages over the conventional strut models are demonstrated through both quasi-static and dynamic analyses.

1 Introduction

Reinforced Concrete (RC) frame structures with infill walls are widely used all over the world. The infill walls were usually neglected during the structural design process since they are considered non-structural elements. However, many studies indicate that the infill walls increase the structure's stiffness, decrease its natural period, frequently increase its capacity, and reduce its ductility. Moreover, the interaction of the RC frame with the infill wall during the lateral deformation may change the failure mode of the frame compared to a bare frame.

1.1 Frame failures during earthquakes

Partial infill walls are those that are not built over the entire columns' height or walls that include openings beside the columns. If these walls are not adequately isolated from the surrounding frame, they have a negative effect on the lateral response of the structure. Specifically, the presence of the partial infill wall constrains the lateral deformation of the columns by decreasing their clear heights, i.e., the distance from the top of the infill wall to the bottom of the upper beam or the floor above. Consequently, it significantly increases the shear demands of the columns, and a shear failure typically occurs, as shown in Figure 1. This behavior is well known as the short column effect.

Even if the infill wall is built all over the column's height, shear failure of the frame may still happen, as shown in Figure 2. During earthquakes, the structure deforms laterally, and parts of the infill wall detach from the surrounding frame when other parts are in contact. The frame is subjected to not negligible interfacial interaction stresses at the frame and infill wall contact regions. These stresses increase the flexural and the shear demands on the RC frame elements. If the frame columns were not designed for these additional loads from the infill wall, they tend to fail during strong earthquakes (Figure 2). A detailed field survey performed after the 2015 Gorkha Earthquake (Brzev et al. 2017) revealed that this type of shear failure of the RC columns is devastating. It leads to a loss of the column's gravity load-carrying capacity and a progressive collapse of the structure, as shown in Figure 3.

1.2 Challenges in modeling infilled frame failures

The recurring failures of infilled frames during earthquakes led to changes in the current seismic codes' provisions, e.g., ASCE 41 (American Society of Civil Engineers 2017). These codes typically adopt a strut model, which replaces the infill wall with a single compression-only diagonal strut. They suggest connecting the diagonal at a certain distance from the beam-column joint to account for the infill-frame interaction. While the equivalent strut approach can well represent the increased stiffness of the frame and thus more reliably represent the period of the structure, its ability to represent the failure mode of the structure is still limited. This is due to the challenges associated with predicting the response of infilled RC frames that involves the nonlinear behavior of the frame, the infill wall, and the interaction between the infill wall and the frame.

1.2.1 Infill-frame interaction

The infill wall and the frame interact with each other during the loading/deformation process. (Brodsky et al. 2018) investigated the infill-frame interaction using a dedicated experimental facility and found that the contact regions vary during the deformation process. In particular, significant infill cracks change the infill-frame contact length and the stress distribution over the contact regions.

1.2.2 Infill wall

Various approaches have been developed to represent the response of the infill wall, as illustrated in Figure 4. They range from very detailed models, such as micro-scale models where each block and mortar layer are modeled (Asteris et al. 2013; Wararuksajja et al. 2020; Erdem et al. 2021), to simplified models, such as models based on homogenization of the infill wall, e.g. (Anthoine 1995; Kawa et al. 2008; Zucchini and Lourenço 2009; Casolo and Milani 2010), or the use of an equivalent strut or multiple struts, e.g. (Chrysostomou et al. 2002; Cavaleri and Papia 2003; Uva et al. 2012; Motovali Emami et al. 2017).

One challenge of the detailed micro-scale models is defining all the mechanical parameters of the block and the mortar joints. But even when all the required material tests have been performed, the use of sophisticated micro-scale models cannot always ensure accurate prediction of the wall's nonlinear behavior and cracking pattern. It is due to high uncertainties of the material properties and the complex infill-frame interaction. Moreover, this technique requires high computational resources. Thus, it is mainly limited to small-scale frames and unsuitable for multi-story structures. The equivalent strut models, on the other hand, are more widely used due to their simplicity and efficiency.

1.2.3 RC frame

The equivalent strut models of the infill wall are usually incorporated in a RC frame modeled by fiber-section elements with either distributed or lumped plasticity (Burton et al. 2014a; Sattar and Liel 2016a; Mohammad Noh et al. 2017; Huang et al. 2020). The models allow capturing the global behavior of the entire structure. However, the complex two-dimensional stress state at the infill-frame interaction regions and the deformability of the beam-column joints cannot be well represented in these simplified models.

The main limitation of the fiber-section element is that it is based on the assumption that the plain-section remains plane, which is incapable of capturing shear-related behaviour of RC frame components during earthquakes. Therefore, additional efforts should be made to use these simplified models for shear-critical frames. For example, (Burton and Deierlein 2014) and (Sattar and Liel 2016b) added several lumped shear springs to the ends of frame columns to account for the shear deformation and failure of the frame. The backbone curve representing the spring response is based on a combined shear limit surface proposed by (Sezen and Moehle 2004) and (Elwood 2004). In this study, the shear-axial failure interaction of the columns was not simulated, i.e., the axial failure and collapse were conservatively assumed to occur when the shear strength reduces to zero. (Verderame et al. 2019a) used the ASCE/SEI 41 (2017) code and the Simplified Modified Compression Field Theory (SMCFT) (Bentz et al. 2006) equations to predict the shear failure of the frame. In the study, they calculated the shear capacity of the frame column as a part of the post-processing procedure of the numerical analysis, which was carried out in OpenSees (McKenna 2011) with a fiber-based element. They found that the SMCFT shear failure predictions are in good agreement with the test results.

The SMCFT is the bases for shear provisions of the Canadian concrete code (Canadian Standards Association 2014) and the AASHTO US bridge design code (AASHTO LRFD Bridge Design Specifications 2014). But this simplified version of the Modified Compression Field Theory (MCFT) (Vecchio and Collins 1986) is a procedure that allows computing the shear strength of an element under a basic assumption that the perpendicular compressive stress (the “clamping stress”) is negligibly minor (Bentz et al. 2006). In other words, the method is only applicable to the non-disturbed sections. However, the shear-critical section is typically located at the infill-frame interaction zone where the clamping stresses are significant. Thus, the use of these equations stands in contradiction to the theory's assumptions, and it is necessary to implement the original set of MCFT equations for the case investigated in this study.

Another common assumption in fiber-section element models is to model the beam-column joints by rigid link elements, e.g. (Al-Chaar 2002; Di Trapani et al. 2021). Therefore, the models cannot explicitly account for the potential deformability and any failure of the joints. Other studies modeled the joint by zero-length artificial rotational springs, e.g., (Liel et al. 2010; Karayannis et al. 2011; Grande et al. 2021). However, most lumped spring models involve substantial approximations and calibrations as addressed by (Ghannoum and Moehle 2012), (Mohammad Noh et al. 2017), and (Verderame et al. 2019b). There is still no widely accepted method for fiber-section models to accurately define the location and behaviour of the lumped springs for modeling of shear-related behaviour of RC elements and the nonlinear behaviour of the frame joints.

1.3 Multi-scale model

To overcome the limitations of the fiber-based element models to represent the complex two-dimensional stress state in the interaction regions, the shear deformation of the frame, and the deformability of the joints, a micro-macro approach was proposed by (Brodsky 2021). Specifically, the frame is modeled by two-dimensional (2D) continuum elements, such that the 2D stress state of the RC frame elements can be explicitly considered. In addition, the constitutive behavior of continuum elements is based on the MCFT

(Vecchio and Collins 1986). Therefore, the nonlinear behavior of the frame that involves cracking, crushing, tension stiffening, and softening of the concrete and aggregate interlocking along the cracks can be fully considered. While the RC frame is modeled by 2D elements, the infill wall is represented by a struts model considering the complexity and uncertainty of the micro-scale models as described above. In other words, this modeling technique combines two detailing levels: a macro-scale level for the infill wall and a micro-scale level for the RC frame. This micro-macro method allows the integration of various strut models with the detailed frame model. More details about the methodology and validation of the method can be found in (Brodsky 2021). In this study, the micro-macro model is employed by modeling the infill wall with three parallel struts: one diagonal and two off-diagonal struts to account for the increase of the flexural and shear demands on the frame elements due to the infill-frame interaction.

1.4 Multi-platform simulation

The multi-platform simulation method allows the decomposition of a structural system into several substructures, each of which can be modeled independently using different analysis tools (Mortazavi et al. 2017; Huang and Kwon 2020). When applying the method to the micro-macro model of infilled frames, at least two numerical models can be developed, one for the RC frame and the other for the struts.

OpenSees (McKenna 2011) is a frequently used tool for the analysis of RC infilled frames subjected to earthquake excitations. (Burton et al. 2014b) applied the peak-oriented hysteretic model developed by (Ibarra et al. 2005) and implemented it in OpenSees to describe the axial response of the equivalent struts. (Cavaleri and Di Trapani 2014a) modeled the infill wall as a diagonal strut described by the hysteretic pivot model. (Noh et al. 2017) compared three different constitutive strut models available in OpenSees: *Concrete01*, *Hysteretic*, and *Pinching4*. They found that the *Hysteretic* and *Pinching4* materials more accurately predicted the hysteretic response of the infill wall.

As discussed earlier, while OpenSees model may predict the global response, it has a limited ability to model the shear failure of the columns and the deformability and damage of the beam-column joints. To deal with shear-related cracks and failures, the RC frame elements can be modeled according to the MCFT (Vecchio and Collins 1986). This theory showed an excellent prediction ability for shear critical RC members. In this study, the 2D FE software VecTor2 (Wong et al. 2013) that employs the constitutive relationships of the MCFT is used.

The multi-platform method allows to combine the two tools (VecTor2 and OpenSees) and take advantage of both programs. The benefits of the method in terms of capturing the global behaviour of the infilled wall-frame system and the failure mode of the RC frame are demonstrated through both quasi-static and time history analyses. In addition, the potential of the method for accurate and efficient modeling of infilled RC frame buildings is also presented.

2 Multi-platform Simulation Method

In the present study, the multi-platform simulation method for seismic performance assessment of infilled frames is demonstrated by modeling the RC frame and struts with VecTor2 and OpenSees, respectively (Figure 5). The two models are interacting with each other by transferring data at the interface nodes that are shown in the figure. These interface nodes include those at the interface boundary between the struts and the frame, the nodes at the fixed base, as well as those where external loads are applied. More specifically, the OpenSees model includes the struts and the external loads, and it works as the main program to solve the incremental form of the system's equilibrium equation at a certain step as:

$$(K_{str} + K_{Int}) \Delta u_{Int} = P - (R_{str} + R_{Int}) \quad (1)$$

where K_{Str} is the stiffness matrix of the struts modeled in OpenSees; Δu_{Int} is the displacement increment vector of all interface nodes. P is the external force vector; R_{str} is the restoring force vector of the struts; K_{Int} and R_{Int} are the stiffness matrix and restoring force with respect to the interface nodes of the RC frame modeled in VecTor2 (see Figure 5(b)). The VecTor2 model, on the other hand, is a slave program that computes K_{Int} and R_{Int} , and sends them to the OpenSees model to formulate Eq. (1). For nonlinear problems, an iterative solution scheme, such as Newton-Raphson's method, should be used to make the unbalance force $P - (R_{str} + R_{Int})$ close to zero. In each iteration, trial displacements at the interface nodes are sent to the VecTor2 model to update R_{Int} . Due to the difficulty in retrieving the frame's tangent stiffness matrix K_{Int} from the VecTor2 model for each iteration, the initial stiffness of the frame is mostly used for K_{Int} . More details about the algorithm and its implementation in OpenSees and VecTor2 can be found in references (Huang and Kwon 2020; Brodsky and Huang 2021). It should be noted that the above method is not limited to OpenSees and VecTor2 programs but is also implementable to other simulation packages as long as a similar communication scheme is established between the two programs.

3 Quasi-static Analysis

This section demonstrates the use of the multi-platform simulation method for the modeling of infilled frames subjected to quasi-static cyclic loading. Two infilled frames tested in previous studies are considered. One frame (S1A-2) failed in a flexural manner and the other (UB1) failed in shear.

3.1 Flexure-critical RC frame

Figure 6 shows the details of the specimen S1A-2 tested by (Cavaleri and Di Trapani 2014b). The frame was infilled with calcarenite masonry. Its dimensions and material properties are summarized in Table 1. The frame was fixed to the ground and loaded with a vertical load of 200 kN at the top of each column followed by a lateral load applied at the top of the frame. The concrete compressive strength and elastic Young's modulus were 25 MPa and 25,500 MPa, respectively. The rebars had an ultimate strength of 450 MPa and an elastic modulus of 200 GPa. Since the yield strength of the rebars was not documented, it

was assumed as 400 MPa. Both the simplified and multi-platform simulation methods presented above were employed for the modeling of the specimen.

Table 1
Properties of masonry

Masonry properties	Unit	Value
Length×width×height ($l \times b \times h$)	mm	360×210×160
Horizontal modulus of elasticity (E_1)	MPa	7,408
Horizontal compressive strength (f_{m1})	MPa	3.08
Vertical modulus of elasticity (G_{12})	MPa	3,933
Vertical compressive strength (f_{m2})	MPa	2.67
Modulus of rigidity (G_{12})	MPa	2,937
Poisson's ratio (v_{12})	-	0.10

3.1.1 Simplified OpenSees model

The simplified model of the S1A-2 specimen is shown in Figure 7. The beam and columns were modeled with fiber-section elements in OpenSees with rigid offset at the beam-column joints. The uniaxial behavior of each fiber was modeled with Concrete02 material. To account for the confinement effect, the concrete strength and concrete strain at the maximum strength of the core concrete were increased based on the method proposed by (Mander et al. 1989). The longitudinal rebars were modeled as independent layers to be superimposed into the concrete fibers. The infill wall was represented as truss elements with *Hysteretic* material model in OpenSees to capture the nonlinear cyclic response, including pinching and strength/stiffness degradation. Specifically, the behavior of each truss element was characterized by the following parameters:

- Equivalent width of the infill wall, w . Based on the method proposed by (Papia et al. 2003), the equivalent width of the wall can be determined from

$$w = dk \frac{c}{z} \frac{1}{(\lambda^*)^\beta} \quad (2)$$

- where d is the diagonal length of the strut; k is a coefficient for the effect of the vertical load on the frame columns. c and β are parameters based on the diagonal Poisson's ratio (v_d) of the infill wall; z

is a shape coefficient and it equals to 1.0 for a square infill wall; λ^* is a parameter related to the material and geometry of the frame. For the three-strut model considered in this study, half of the calculated width was assigned to the diagonal truss element and quarter to each off-diagonal truss element, as shown in Figure 7.

- Contact length, L_c . This parameter indicates the locations of the off-diagonal struts as shown in Figure 7 and allows considering the interaction between the infill wall and the surrounding frame. It can be estimated using the method proposed by (Paulay and Priestley 1992) as 0.25 times the diagonal length of the infill wall.
- Envelope parameters, $S_1, S_2, S_3, \delta_1, \delta_2$, and δ_3 , as shown in Figure 8. The struts are compression-only elements, and the envelope of their response is idealized as a trilinear curve, including the initial elastic branch ($S_1 - \delta_1$), Post-yielding branch ($S_2 - \delta_2$), and degradation branch ($S_3 - \delta_3$). These envelope parameters can be estimated according to (Cavaleri and Di Trapani 2014b).
- Hysteretic material model parameters, $p_x, p_y, d_1, d_2, \beta$. These parameters were calibrated against the experimental results to capture the pinching and degradation behaviors (Wijaya et al. 2020).

The above parameters that defined the truss elements in the simplified OpenSees are listed in Table 2.

Table 2
Modeling parameters for infill wall (S1A-2)

Parameters	Unit	Value
w	mm	882.5
L_c	Mm	320.6
S_1	MPa	0.798
S_2	MPa	1.338
S_3	MPa	0.933
δ_1	mm/mm	2.209E-04
δ_2	mm/mm	5.126E-03
δ_3	mm/mm	1.228E-02
p_x	-	0.8
p_y	-	0.1
d_1	-	0
d_2	-	0.05
β	-	0.3

3.1.2 Multi-platform OpenSees-VecTor2 model

The multi-platform model of the S1A-2 specimen is the same as the simplified OpenSees model except that the RC frame was modeled in detail with VecTor2 software. Specifically, two-dimensional continuum elements were employed to model the beams, the columns, and the joints. As shown in Figure 5, the longitudinal rebars were modeled with truss elements while the transverse reinforcement was smeared with the concrete elements. Table 3 lists the material models used in the VecTor2 model for the concrete, reinforcement, and their bond. These are default models that are determined based on validations

against static and dynamic tests (Vecchio and Shim 2004; Palermo and Vecchio 2007), and they are found to be able to provide reasonable predictions without much calibration effort. As the main integration module, the OpenSees model included the strut elements for the infill wall as well as the external loads applied to the frame. The VecTor2 model acted as the substructure module to compute the restoring forces based on the displacements predicted from the OpenSees model. Data were exchanged between the two models through the interface nodes, as indicated in Figure 5. Each interface node had two translational degrees of freedom.

Table 3
Material models in VecTor2

	Material Property	Model
Concrete model	Concrete Compression Pre-Peak Response	Hognestad Parabola (Vecchio and Collins 1982)
	Concrete Compression Post-Peak Response	Modified Park-Kent (Park et al. 1982)
	Concrete Compression Softening	Vecchio (1992-A) (Vecchio 1992)
	Concrete Tension Stiffening	Modified Bentz (2003) (Bentz 2005)
	Concrete Tension Softening	Linear
	Concrete Confined Strength	Kupfer/Richard Model (Richart et al. 1928; Helmut Kupfer et al. 1969)
	Concrete Dilation	Variable isotropic (Montoya et al. 2006)
	Concrete Cracking Criterion	Mohr-Coulomb (Stress)
	Concrete Crack Slip Check	Walraven (Walraven 1981)
Reinforcement model	Hysteretic Response	Elastic-Hardening
	Dowel Action	Tassios (Crack Slip) (He and Kwan 2001)
	Buckling	Akkaya (Akkaya et al. 2019)
Bond model	Concrete Bond	Eligehausen (Eligehausen et al. 1982)

3.1.3 Simulation results

Figure 9 shows the experimental and numerical cyclic responses of the S1A-2 specimen. Both the simplified OpenSees and the multi-platform OpenSees-VecTor2 models, are in good agreement with the experimental result. As observed in the test, the RC frame developed flexural (i.e., horizontal) cracks at the columns at large lateral displacements. Such flexural behavior can be properly captured by both fiber-section and continuum elements. In addition, diagonal cracks were observed at the beam-column joints in the cyclic test. Such damage can only be captured by the VecTor2 model. This increased flexibility due to

the damage at the beam-column joints was not taken into account in the simplified OpenSees model due to the rigid beam-column joints assumption. However, the deformations of the joints had a minor impact on the base shear when the frame was imposed with the same lateral displacements. As will be addressed in Section 4, the rigid beam-column assumption could lead to inaccurate prediction in seismic response of the same specimen during real earthquake excitations.

Figure 10 presents the patterns and widths of the main cracks at different displacement stages during the analysis. The cracks were plotted on a deformed shape of the frame with a magnification factor of five. This figure shows the evolution of the cracks at the peak positive displacements (i.e., the frame deforms to the right) of the last four loading cycles (+10, +20, +30, +40 mm). The deformed shape along with the cracking pattern for negative displacements is roughly a mirror image of Figure 10. The widest crack developed at the location between the top end of the left column and the beam-column joint. At each loading cycle, the width of this crack increased from 4.4 mm at a lateral displacement of +10 mm to around 21 mm of the final cycle, at a displacement of 40 mm. Additional noticeable cracks are diagonal at the upper beam-column joint, which is 2.3 mm wide at a lateral displacement of +10 mm and opens to 4.5 mm at a displacement of +40 mm. In addition, both columns cracked at their connections to the bottom beam. At a displacement of +40 mm, the width of these cracks on the tension side was about 14 mm. Thus, at this stage, plastic hinges developed at the ends of the columns.

3.2 Shear-critical RC frame

Figure 11 illustrates specimen UB1 tested by (Gao et al. 2018) at the University of Buffalo. The specimen's columns were designed with insufficient transverse reinforcement. Therefore, the columns tend to fail in shear due to the lateral load. The frame was filled with regular double-wythe clay bricks. The material properties of the frame and infill wall are summarized in Table 4. Before imposing the lateral load, the frame was loaded with a distributed load along the top surface of the beam to simulate the gravity load from the above stories. The total applied vertical load was 338.1 kN.

Table 4
Material properties (UB1)

	Parameter	Unit	Value
Concrete	Compressive strength	MPa	30.34
	Elastic modulus	MPa	24,130
Reinforcement	Yield strength	MPa	413
	Ultimate strength	MPa	600 ¹
Masonry	Compressive strength	MPa	15.17
	Elastic modulus	MPa	24,130
	Shear modulus	MPa	9,650
Mortar	Tensile strength	MPa	1.38
	Cohesion	MPa	0.62
	Coefficient of friction	-	1.0
¹ The ultimate strength assumed as 600 MPa ($\sim 1.5f_y$)			

3.2.1 Multi-platform OpenSees-VecTor2 model

To demonstrate the capability of the multi-platform simulation method to predict the shear damage of the RC frame, a pushover analysis was conducted and compared with the cyclic backbone curve obtained from the experimental test. Since the load was applied in one direction, a single set of three struts were included to simulate the response of the infill wall (Figure 12). The RC frame was modeled with VecTor2 where the longitudinal rebars were modeled with truss elements and the transverse reinforcement was smeared within the concrete elements. The interface nodes for data exchange between the OpenSees and VecTor models include those at the ends of the struts, at the loading points, as well as at the fixed base of the frame. The monotonic behavior of the struts was controlled by the effective length (w), contact length (L_c), and the envelope parameters (S_1 , S_2 , S_3 , δ_1 , δ_2 , and δ_3 in Figure 8). The values of these parameters are determined based on (Bertoldi, S.H.; Decanini, L.D.; Gavarini 1993) model and are summarized in Table 5.

Table 5
Modeling parameters for infill wall (UB1)

Parameters	Unit	Value
w	mm	622
L_c	mm	383
S_1	MPa	3.662
S_2	MPa	4.577
S_3	MPa	1.602
δ_1	mm/mm	1.611E-04
δ_2	mm/mm	8.054E-04
δ_3	mm/mm	2.698E-02

3.2.2 Simulation results

The predicted capacity of the OpenSees-VecTor2 model is 486 kN that is achieved at a drift ratio of 0.27%. Meanwhile, the transverse reinforcement at the left column starts yielding. The difference between the predicted and the observed capacity is less than 6.5%. The longitudinal reinforcement starts yielding at a drift value of about 0.47% at the bottom of the right column and 0.55% at the top of the left column. One of the main reasons to use the multi-platform analysis is to capture the shear failure of the frame without any tedious calibration of the concrete material model or lumped-plasticity models. Figure 13 presents the observed cracking pattern at the end of the experimental test and the cracking pattern predicted by the analysis. As can be seen, the shear failure mode is well captured by the numerical analysis. At a drift ratio of 1%, the main cracks are diagonal shear cracks in the columns close to the beam-column joints. It includes cracks at the top of the left column and the bottom of the right column. In addition, horizontal flexural cracks are observed at the bottom of the columns. The diagonal cracking width is about 7.5 mm and the width of the horizontal crack is about 3.9 mm on the tension side.

4 Time History Analysis

The application of the proposed multi-platform simulation method to performance assessment of infilled RC frames subjected to ground motions is demonstrated in this section. To address the importance of capturing the nonlinear hysteresis and the failure mode of the infilled RC frame for the time history

analysis, two specimens are considered. One is the S1A-2 specimen in Section 3.1, which is adopted to represent the infilled RC frames with potential flexural failure of the columns due to earthquake loading. The second specimen S1A-R, on the other hand, is used to represent those with shear-related failure at the frame columns due to the interaction between the RC frame and the infill wall. The S1A-R frame is designed with the same dimensions, materials, and reinforcement details as the S1A-2 specimen, except that the frame columns have a much lower transverse reinforcement ratio, 0.05% compared with 0.14% of specimen S1A-2. The spacing of the columns' transverse reinforcement is increased from 100 mm to 280 mm and the diameter of the stirrups remained the same. This reduction was intentionally made to trigger shear-related mechanisms of the columns. The axial load applied to each beam-column joint is accordingly reduced from 200 kN (S1A-2) to 20 kN (S1A-R).

The inertial forces of the two frames are modeled by adding lumped masses to the beam-column joints. As indicated by (Huang and Kwon 2015), the numerical model of a RC structure should be able to capture the realistic structural hysteresis, especially for those with a short structural period. Therefore, to show the benefit of the multi-platform simulation method, the lumped masses are specified to obtain a relatively short natural period of the infilled frame, i.e., 0.24 seconds. It is worth noting that the period is determined from the multi-platform OpenSees-VecTor2 model, and the computed masses are used consistently in the simplified OpenSees model for a fair comparison. Both frames are subjected to the Kobe earthquake ground motion at Fukushima station with scale factors of 5.5 and 4.5 for S1A-2 and S1A-R frames, respectively. The ground motion interval between 5–95% of the total Arias intensity is shown in Figure 14. Rayleigh's damping is employed with 5% damping ratio for the first and second modes.

Both simplified and multi-platform modeling methods, as presented in Section 3.1 are used to simulate the seismic response of the two frames. In structural dynamics, the implicit Newmark method with a nonlinear solution scheme is mostly used to solve the system's equation of motion (e.g., Newton-Raphson's method). But in most multi-platform or hybrid simulations, the non-iterative numerical integration schemes are preferred. This is because the response of the structural component modeled in the substructure module is not only controlled by the final converged displacement but also influenced by the trial displacements from the integration module for nonlinear iterations. Thus, if each time step involves too many iterations and the predicted displacements for the iterations do not increase or decrease monotonically, additional loading cycles will be imposed on the structural component modeled in the substructure module. This will lead to unrealistic cumulative damage and incorrect results (Del Carpio et al. 2017). The use of the non-iterative integration schemes is not always suitable for all analysis cases. For example, the non-iterative explicit schemes, such as the Central Difference method and explicit Newmark method, requires the use of a small-time step to ensure their numerical stability. This requirement significantly increases the total number of analysis steps, especially for short period structures like the S1A-2 and S1A-R frames considered in this study. To avoid such limitation, the non-iterative implicit schemes can be used as they can be unconditionally stable under a certain circumstance. One of the examples is the α -Operator Splitting (Alpha-OS) method (Combescure and Pegon 1997), which is unconditionally stable for softening structures when α varies between -1/3 and 0. However, as addressed by (Del Carpio et al. 2017), the accuracy of the method depends on the time step,

and the method is less accurate for highly nonlinear problems when compared with the implicit Newmark method with a fixed number of iterations using the same time step. To balance the accuracy, stability, and computational efficiency, the implicit method with a fixed number of iterations is used in this study. To avoid the unrealistic cumulative damage of the substructure module (the RC frame model), the minimum number of iterations was defined. A parametric study was carried out and revealed that S1A-2 and S1A-R converge to a solution within two and five iterations per analysis step, respectively. As expected, the shear-critical frame requires additional computational cost. The analysis results are presented and discussed in the following sections.

4.1 S1A-2 infilled frame

Figure 15 shows that the lateral deformation histories of the S1A-2 frame predicted from the simplified OpenSees model and the multi-platform OpenSees-VecTor2 model are quite different even at the very beginning of the earthquake. Such difference is mainly due to the flexibility of the beam-column joint that is not considered in the simplified OpenSees model as rigid beam-column joints are assumed. The predicted cracks at the beam-column joints in the detailed OpenSees-VecTor2 model are shown in Figure 16. This figure illustrates the deformed shape and the cracking pattern at the final stage of the multi-platform analysis. This explains the increase of discrepancies between the two models subjected to the ground motion. This comparison implies that models with similar hysteresis curves for a certain predefined loading protocol as shown in Figure 9 not necessarily ensure consistent time history responses under earthquake excitations. This is an important finding because numerical models for seismic performance assessment frequently validated through cyclic quasi-static tests. Then, the so-called “validated” models are directly applied to dynamic time history analysis. This simulation approach implies the assumption that the numerical model validated for a certain predefined loading protocol in the quasi-static test can also represent the hysteretic response under earthquake loading. However, the comparison between the simplified and the detailed models reveals that such assumption is not always true.

To examine the effect of the rigid link elements in the simplified OpenSees model, a new OpenSees model was developed. The rigid link elements were removed in the new model, such as the beams and columns being extended to the joints. Figure 17 compares the OpenSees models with and without the rigid link elements during cyclic quasi-static analysis (similar to Figure 9). The rigid joint assumption has a minor impact on the hysteretic behavior of the frame due to a predefined displacement history. However, in time history analysis, only the OpenSees model with the flexible beam-column joints is in good agreement with the detailed OpenSees-VecTor2 model, as shown in Figure 15. Figure 16 shows the deformed shape and frame's crack pattern at the maximum deflection obtained from the VecTor2 substructure model. Large horizontal cracks can be observed at the ends of the two columns, and the frame tends to fail in flexure failure of the columns. The similar seismic responses shown in Figure 15 indicate the accuracy of the OpenSees-VecTor2 model and the efficiency of the simplified OpenSees model for modeling flexure-critical infilled RC frames.

4.2 S1A-R infilled frame

The deflection histories of the S1A-R frame are compared in Figure 18. The simplified OpenSees model without rigid links at the beam-column joints is used here for comparison. It can be seen that the two models predicted similar responses up to 3.255 s, and the hybrid OpenSees-VecTor2 model predicts more considerable deflections from thereafter. This is because of the large shear cracks (maximum shear crack width of 4.25 mm) that developed at the top end of the right column, as shown in Figure 19(a). This response cannot be captured by the fiber-section elements in the simplified OpenSees model. In addition to the shear cracks, flexural cracks are developed throughout the columns. After 10 s, the differences between the two responses become even more significant. The analysis results reveal that the seismic behavior of the infilled frame will be underestimated if the numerical model fails to capture the considerable shear damage of the frame, particularly for shear-critical elements.

In terms of computing time, it took about 21 hours for the multi-platform OpenSees-VecTor2 model to complete the time history analysis of the S1A-R frame. It should be noted that this result is based on a time step of 0.005 seconds for a ground motion of 29.6 seconds. Also, an implicit integration scheme with five iterations for each analysis step was employed. Computing time can be shortened if a non-iterative integration scheme is used. To further extend the method for system-level assessment of RC infilled frame buildings, the computational efficiency can be significantly improved by only including the critical portions of the frame, such as the shear-critical sections of the beams or columns and the frame joints in the detailed VecTor2 model. The rest may be modeled in the OpenSees model using simplified elements, such as truss and fiber-section elements. For illustration, a more efficient multi-platform model of the S1A-R frame (referred to as simplified OpenSees-VecTor2 model) was developed, as shown in Figure 20. Specifically, the columns and upper beam portions between the joint and their connection to the off-diagonal strut are considered critical. Thus, these regions were modeled in the VecTor2 model. Meanwhile, the other portions of the beam and the columns that tend to fail in a flexure manner (Figure 19a) were modeled with fiber-section elements in OpenSees. In addition, a series of rigid link elements are connected to the interface nodes between the VecTor2 and OpenSees models to account for the inconsistent number of degrees of freedom between the 2D continuum elements in VecTor2 and the fiber-section frame elements in OpenSees. The VecTor 2 model includes two translational degrees of freedom per node, while the fiber-section frame elements in OpenSees contain two translational and one rotational degrees of freedom per node.

Figure 21 and Figure 19b show that such simplified OpenSees-VecTor2 model predicted similar deflection histories and failure mode of the critical sections as the original OpenSees-VecTor2 model. The minor difference in the deflection of the frame, as shown in Figure 21, is mainly due to the use of the rigid link elements, which add extra transverse confinement to the connected beam and columns. In the future study, these rigid link elements can be replaced with more advanced beam-membrane coupling methods, e.g., (Sadeghian et al. 2018). Compared with the original OpenSees-VecTor2 model, the simplified multi-platform model almost shortened the computational time by half (i.e., 10 hours).

5 Conclusions

A multi-platform simulation approach for infilled RC frames subjected to earthquake excitations is discussed and validated in this paper. The paper shows the ability of the multi-platform models to capture both flexural and shear failures within the RC frame elements. The infill wall is represented by equivalent struts and modeled in OpenSees, while the RC frame is modeled with MCFT-based VecTor2 software using two-dimensional elements.

The comparison of the multi-platform modeling method with the simplified fiber-based modeling method indicates that flexural-critical frames do not require a detailed representation of the frame. Both modeling approaches show good agreement, even when the beam-column joints are slightly cracked. However, this conclusion is based on a simplified fiber-based model where rigid link elements do not model the beam-column joints. The rigid beam-column assumption typically used in the simplified frame models may underestimate the response under earthquakes, even if the models have been successfully validated against cyclic quasi-static tests.

A detailed model of the frame is crucial for shear-critical frames. These frames were mainly built with inadequate reinforcement detailing before the seismic code became mandatory. The results reveal that the hysteretic behavior of such infilled frame cannot be adequately modeled with fiber-section elements since they fail to capture the significant shear damage of the RC frame. The presented multi-platform simulation provides a solution to overcome this shortcoming. In addition, by properly identifying the critical portions of the structure, the method applies to system-level performance assessment of large RC infilled frame structures.

Declarations

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Figures

Peru 2001

India 2004

Peru 2007

China 2008

Figure 1

Short column effect (Haldar et al. 2013)

Turkey 1998

Turkey 1999

India 2005

China 2008

Nepal 2015

Figure 2

Frame failure because of its interaction with the infill wall (Feng et al. 2014; Fabio Di Trapani and Cavaleri 2015)

Gorkha Earthquake, Nepal 2015

Figure 3

Collapse of the infilled RC structure followed by a shear failure of the frame (Brzev et al. 2017; Karmacharya et al. 2018)

Figure 4

Collapse of the infilled RC structure followed by a shear failure of the frame

Figure 5

Multi-platform model of an infilled frame

Figure 6

Specimen S1A details (dimensions in cm) (Cavaleri and Di Trapani 2014b)

Figure 7

Figure 8

Monotonic envelope of the infill wall element

Figure 9

Comparison of the experimental result and the simulations

Figure 10

Cracking pattern, deformed shape (x5), and the width of the main cracks at different lateral displacements

Figure 11

The layout of the specimen UB1 (unit: mm)

Figure 12

The layout of the specimen UB1 (unit: mm)

Diagonal cracks observed in the simulation

Columns' diagonals cracks at the end of the test

Figure 13

The layout of the specimen UB1 (unit: mm)

Figure 14

Kobe earthquake ground motion

Figure 15

Predicted lateral deflection by the hybrid (OpenSees-VecTor2), OpenSees with rigid links (left), and OpenSees without rigid links (right) simulations

Figure 16

Deformed shape and crack pattern of the S1A-2 model (VecTor2 submodel)

Figure 17

OpenSees models with flexible and rigid beam-column joints

Figure 18

Lateral deflection history of S1A-R frame

Figure 19

Deformed shape and crack pattern of S1A-M frame at 3.255 s

Figure 20

Figure 21

Predicted deflection history. Original vs. simplified OpenSees-VecTor2 models