FE Analysis of RC Frames with and without Masonry Infill Panels

A. Mohyeddin[1], H. M. Goldsworthy[2], E. F. Gad[3] and S. Maharjan[4]

1. Corresponding Author. Course Coordinator, School of Engineering,
   Edith Cowan University, Joondalup, WA 6027
   Email: a.mohyeddin@ecu.edu.au

2. Associate Professor, Department of Infrastructure Engineering,
   The University of Melbourne, Parkville, VIC 3010
   Email: helenmg@unimelb.edu.au

3. Professor and Chair, Department of Civil and Construction Engineering,
   Swinburne University of Technology, Hawthorn, VIC, 3122
   Email: egad@swin.edu.au

4. Master Student, School of Engineering,
   Edith Cowan University, Joondalup, WA 6027
   Email: shankarm@our.ecu.edu.au

ABSTRACT:

This paper focuses on the FE analysis of reinforced concrete frames with and without masonry infill panels using ANSYS and OpenSees. First, the FE results are compared with those of experimental available from literature, and some of shortcomings of ANSYS associated with concrete/masonry material modelling are investigated. This comparison confirms the validity of a previously proposed indirect methodology to be employed in ANSYS for modelling concrete. The second part of the paper focuses on "Equivalent Single Strut" modelling of infill-frames. Two-dimensional strut models are constructed using OpenSees considering the nonlinear behaviour of the masonry infill panel and concrete frame. The results are compared with those of detailed three-dimensional models created using ANSYS. The effect of some key material properties on the results of strut models is discussed and some inherent limitations of this technique are identified.

Keywords: Finite Element, ANSYS, OpenSees, Infill-Frame, Strut Modelling, Masonry, Concrete

1 Introduction

ANSYS has been used for FE modelling of concrete structures including beams and frames. However, there are some aspects of concrete modelling using ANSYS which need detailed consideration. This is essential when Solid65 element is employed to model the behaviour concrete structures over the full loading range. Most of researchers have assumed that concrete behaves linearly up to the point where it crushes and/or cracks (Barbosa and Ribeiro, 1998; Fanning and Kelly, 2000; Maneetes and Memari, 2009). In fact the material can exhibit considerable nonlinearity in its behaviour prior to crushing. A comprehensive study was conducted on how to combine different material models available in ANSYS to properly consider the nonlinear behaviour of concrete (Mohyeddin et al., 2010, 2013a, 2013b). These studies showed that a combination of two of ANSYS material models and Solid65 element can provide a good prediction of the behaviour of non-ductile frames with and without a masonry infill panel up to 2% and 5% drift, respectively. As a continuation of the studies above, two bare frames are modelled using OpenSees nonlinear concrete material model and the results are compared with those of experimental and ANSYS.

Furthermore, two infill-frames are modelled using the "equivalent strut" technique and some of the inherent shortcomings of this method in representing the nonlinear behaviour of infill-frames are discussed. This concept was first proposed by Onishchik in the late 1930s who suggested that an infill panel can be considered as a "compressed diagonal strut" when the infill-frame is under lateral loading (Mohyeddin, 2011). Holmes (1961) offered an equivalent cross sectional area of such a strut for the first time. The concept of diagonal struts has been widely investigated and a variety of models have been proposed (Stafford Smith, 1962; Mainstone, 1971; Klingner and Bertero, 1978; Zarnic and Tomazevic, 1985; Saneinejad and Hobbs, 1995; Crisafulli, 1997). FEMA 356 (2000) also considers the use of a single strut to represent the infill panel as an acceptable method of analysing infill-frames for both linear and nonlinear range of behaviour.

2 Concrete and Steel Modelling

In the three-dimensional FE models which are developed using ANSYS, reinforced-concrete and masonry materials are modelled using the Solid65 element. This element has smeared reinforcement and smeared cracking capabilities and when used with the built-in concrete material model (Concr) uses the failure surface proposed by Willam and Warnke (1974). The shortcoming of the Concr material model is that it assumes a linear stress-strain relationship. For this reason, another nonlinear model within ANSYS, namely MKIN, is combined with this concrete model to produce the nonlinear stress-strain relationship up to the ultimate stress. Based on a comprehensive discussion presented by Mohyeddin (2011), the failure surface produced as the result of this approach is similar to that taken by Lotfi and Shing (1991), except that in the present model the failure surface is implemented in a three-dimensional stress space. OpenSees, on the other hand, has a better suited material library for concrete. In this study Concrete01 and ConfinedConceret01 models are used for concrete cover and core, respectively, with the latter considering the effect of confinement coming from stirrups. One of the disadvantages of the concrete models of OpenSees the tensile strength of concrete is taken as zero. ANSYS considers a linear stress-strain relationship for the tensile behaviour of concrete. The tensile strength of the concrete and masonry in ANSYS models are considered to be 10% of those of the uniaxial compressive strength. In all the above models the (uniaxial) modified Kent-Scott-Park (Scott et al., 1982) stress-strain relationship is used for concrete. In the two-dimensional models constructed using OpenSees, the “non-linear beam column” element is used. Some trial analyses were conducted using the “beam with hinges” element of OpenSees with 5%, 10% and 20% hinge lengths. It was concluded that “non-linear beam column” element is a more suitable element. This is consistent with the models developed by Hashemi and Mosalam (2006). In all cases a bi-linear model is used for the steel. The modulus of elasticity of steel, E_s, is assumed to be 200 GPa, and the secondary stiffness, E_2 is approximately measured as 2.5% and 1.1% of E_s for the steel used in the experiments by Mehrabi (1994) and Al-Chaar et al. (2002), respectively.

3 Masonry Infill Panel Modelling

For FE modelling of the infill-frame using ANSYS, where the masonry infill panel is explicitly modelled, the mortar joint thickness is halved; each half is attached to the adjacent masonry block from one side and interacts with the other half of the mortar joint through the interface (contact) elements (Figure 1). This method, which is similar to the “simplified micro modelling” based on the classification by Rots (1991) and Lourenco (1996), is selected to reduce the number of interface elements and to decrease the computational costs and convergence problems. This method of modelling represents most of the masonry failure modes including sliding along the joints, cracking of the masonry blocks in direct tension, diagonal cracking of the units (where there is sufficient normal stress to develop friction in the joints) and masonry crushing. For a comprehensive discussion on failure modes considered in this model one can refer to Mohyeddin (2011).

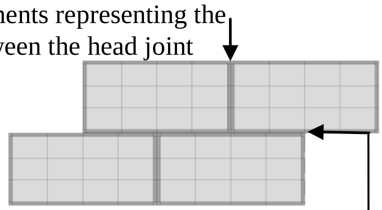

Figure 1: The location of the interface (contact) elements in the masonry (Mohyeddin, 2011).

In the two-dimensional models created in OpenSees, the “equivalent strut” technique is used to represent the masonry infill panel. The single strut is defined as a diagonal connecting windward upper and leeward lower corners of the infill-frame. The cross section area of the strut is calculated using the equation given by FEMA publications (FEMA 273, 1997, FEMA 306, 1998, FEMA 356, 2000), which are all based on the study by Stafford-Smith (1966) and its follow-ons by Mainstone and Week (1970) and Mainstone (1971). The un-reinforced masonry material is defined using Concrete01 material model. The nonlinear stress-strain relationship of the strut (Figure 2) is based on the model proposed by Hashemi and Mosalam (2006) which is very close to those used in ANSYS models using the equations below given by Angel (1994):

$$\sigma_m = \frac{27\,f'_{cm}\,(250\varepsilon_{crm}-1)}{4\,\varepsilon^3_{crm}}\,\varepsilon_m{}^3 + \frac{27\,f'_{cm}\,(1-333.3\varepsilon_{crm})}{4\,\varepsilon^2{}_{crm}}\,\varepsilon_m{}^2 + E_m\varepsilon_m \qquad where \quad E_m = 750 f'_{cm} \qquad 1$$

in which ε_m is the compressive strain, σ_m is the compressive stress of the masonry related to ε_m, f'_{cm} is the maximum compressive strength of the masonry, ε_{crm} is the maximum strain of the masonry before it fails and E_m is the modulus of elasticity of masonry.

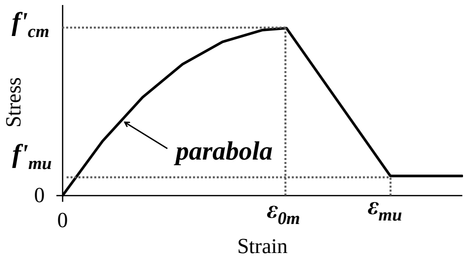

Figure 2: Stress-strain curve used for the equivalent strut created using OpenSees

4 Validation of the models constructed using OpenSees

Two reinforced concrete frames are created using OpenSees. The first model replicates the one used by Al-Chaar et al. (2002), shown in Figure 3. The compressive strength of concrete is 38.4 MPa at the strain of 0.00128, and the yield stress of the steel reinforcing rebar is 438.5MPa. The material properties of the second frame, shown in Figure 4, are given in Table 1. A total gravity load of 294 kN is applied to the columns of this frame to replicate the effect of upper stories (Mehrabi, 1994). This second frame was also used for the verification of the FE models created in ANSYS (Mohyeddin et al., 2010).

Table 1: Concrete and reinforcement properties of the frame tested by Mehrabi (1994)

Concrete (MPa)			Reinforcing steel			
f'_t Modulus of rupture	f'_t Split cylinder	f'_c Cylinder test	Diameter (mm)	Type	Yield Stress (MPa)	Ultimate Strength (MPa)
			6.35	Plain	367.6	449.6
			12.7	Deformed	420.7	662.1
6.76	3.08	30.89	15.9	Deformed	413.8	662.1

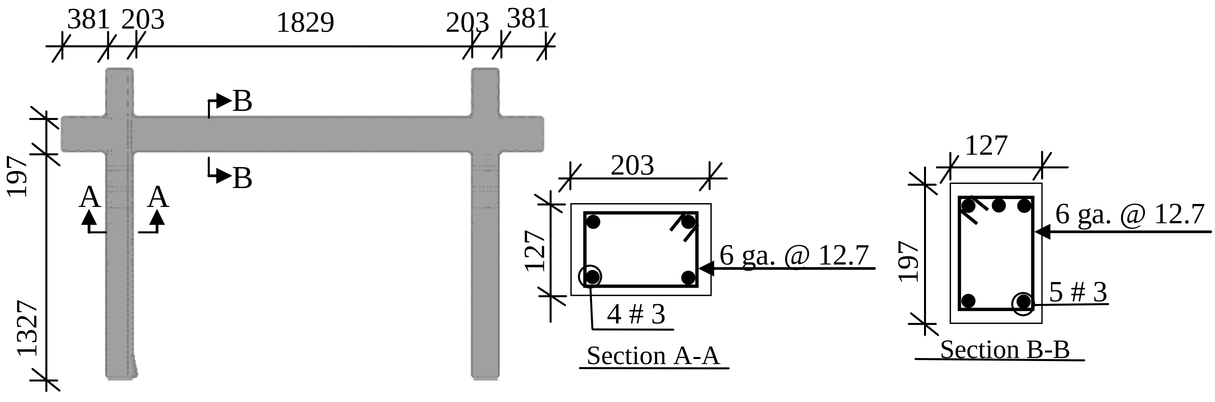

Figure 3: Bare frame geometry and reinforcement details (Al-Chaar et al., 2002). All dimensions in mm.

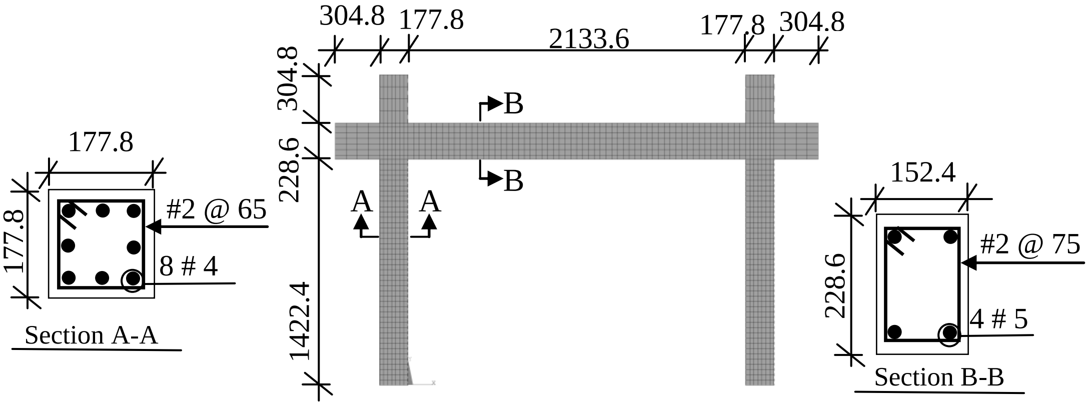

Figure 4: Bare frame geometry, reinforcement details (Mehrabi, 1994) and FE mesh (Mohyeddin, 2011). All dimensions in mm.

Based on the force-deflection curves shown in Figure 5, OpenSees generally provides a very good match with those of experimental. In the case of Al-Chaar's experiment, OpenSees has underestimated the initial stiffness of the frame, whereas in the case of Mehrabi's the initial stiffness has been overestimated. Furthermore, the results of the three-dimensional FE model created using ANSYS perfectly match those of OpenSees (Figure 6a). The fact that the same quantity has been underestimated by OpenSees in one case and overestimated in the other makes it difficult to relate such a discrepancy to errors of the FE model only. In the case of Mehrabi's experiment ANSYS and OpesnSees models, which are created based on two different methodologies, perfectly match. Also the ANSYS model shows a very good match with those of experimental in terms of crack pattern of the frame (Mohyeddin et al., 2010). Therefore, it is concluded that such a disagreement in the initial stiffness is partly associated with the inherent large variations in the concrete mechanical properties and errors in the experimental measurements, and partly with the simplifications involved in the modelling of concrete material in terms of cracking, shrinkage and tensile behaviour, to name a few.

Based on the good agreement between the two FE models, it is further concluded that the methodology applied by Mohyeddin (2011) to overcome the shortcomings of ANSYS regarding concrete modelling can confidently be utilised for similar modelling purposes.

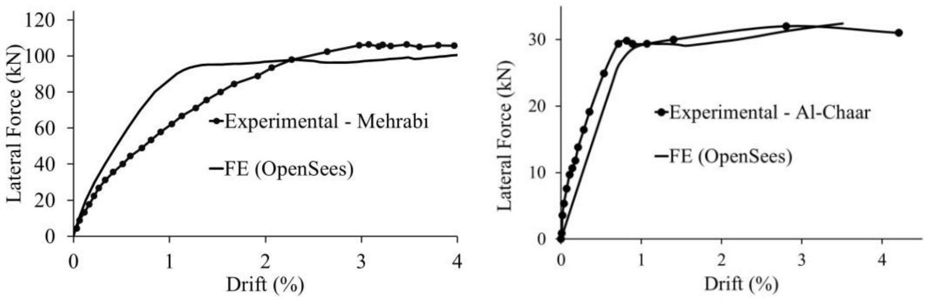

Figure 5: Verification of the bare frame FE results from OpenSees

5 Sensitivity analysis of Mehrabi's bare frame

The same set of sensitivity analyses conducted on Mehrabi's bare frame by Mohyeddin et al. (2010) are replicated here using OpenSees (Table 2).

Table 2: Variables in different analyses

Analysis No.	Concrete Model	E_2 (% of E_s)	Cover (*mm*)	Concrete* (*MPa*)		Large Deflection
				f'_c	f'_t	
1	*Confined*	*2.5*	*25.4*	*30.89*	*6.76*	*off*
2	*Un-Confined*	*2.5*	*25.4*	*30.89*	*6.76*	*off*
3	*Confined*	*5.0*	*25.4*	*30.89*	*6.76*	*off*
4	*Confined*	*0.0*	*25.4*	*30.89*	*6.76*	*off*
5	*Confined*	*2.5*	*25.4*	*23.17*	*5.07*	*off*
6	*Confined*	*2.5*	*25.4*	*38.61*	*8.45*	*off*
7	*Confined*	*2.5*	*12.7*	*30.89*	*6.76*	*off*
8	*Confined*	*2.5*	*25.4*	*30.89*	*6.76*	*on*

Figure 6 compares the FE results from ANSYS with those of experimental and OpenSees. Figure 6a shows a perfect match between the two FE models. When using unconfined concrete stress-strain relationship ANSYS shows a more reduction in the ultimate strength of the frame (Figure 6b) and the analysis terminates at a drift of only 2% (mainly due to convergence issues). Figures 6c and 6d, which investigate the sensitivity of the frame to the secondary stiffness of steel, show a good match between the two analyses when E_s increases to 5%. However, ANSYS shows less sensitivity to E_s when it is taken as zero. While both models produce the same load-deflection curve for a 25% weaker concrete, ANSYS tends to show a larger increase in the ultimate strength of the frame for a 25% stronger concrete (Figures 6e and 6f). As shown in Figure 6g, both models produce the same load-deflection curve as the result of a 50% decrease in the concrete cover. OpenSees demonstrates more sensitivity to the large-deflection effect up to 3% drift, after which the two models converge (Figure 6h).

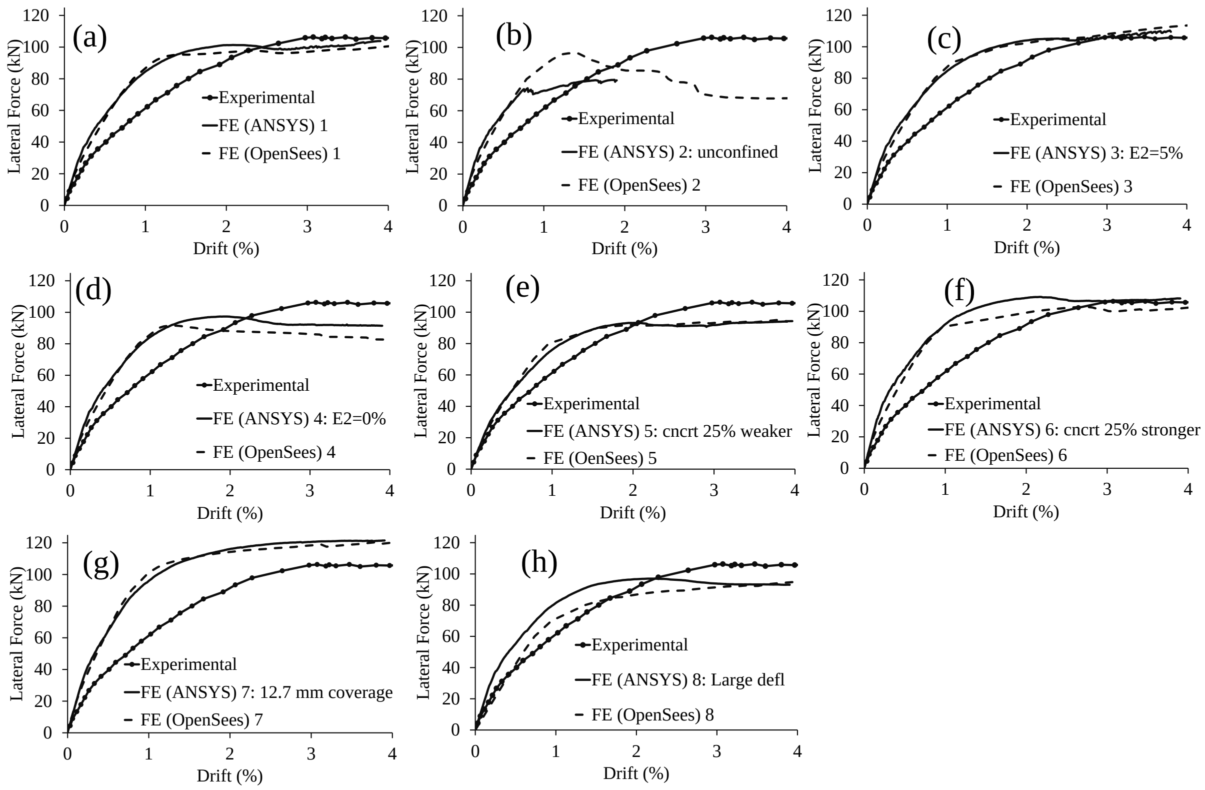

Figure 6: A comparison between load-deflection curves of a bare frame created using ANSYS and OpenSees

6 Equivalent Strut Modelling Technique

Using the "equivalent strut" approach two infill-frames from Mehrabi (1994), namely Specimens 8 and 9, are modelled using OpenSees. In order to consider the nonlinear behaviour of the strut, the stress-strain curve proposed by Hashemi and Mosalam (2006) is used (Figure 2). The width of the strut is calculated based on the equation below which is given by FEMA 356 (2000):

$$a = 0.175\,(\lambda_h h)^{-0.4} d \qquad \lambda_h\, h = \sqrt[4]{\left(\frac{E_m\, t \sin 2\theta}{4 E_f\, I_c\, h_I}\right)}\, h \qquad 2$$

where h is the storey height (height of the column between beam-to-beam centrelines), E_m and t are the modulus of elasticity and the thickness of the infill panel, respectively, θ_d is the angle of the infill diagonal to horizontal, E_f is the modulus of elasticity of the frame material, I_c is the second moment of area of the column, h_I is the height of the infill panel and d is the diagonal length of the panel.

Tables 3 and 4 show the key material properties that are selected for the sensitivity analysis of the two Mehrabi's specimens with the aim of producing the best match with the load-deflection curves of the experimental results. The load-deflection curves from these analyses are given in Figures 7 and 8. The results show that by varying the four main material properties, which govern the stress-strain relationship of the masonry, one can produce a reasonably good match between the analytical and experimental results (Figures 7h and 8h).

Table 3: Variables in analyses of Specimen 8 of Mehrabi (1994)

Analysis	f'_{cm} (MPa)	ε_{0m}	f'_{mu} (MPa)	ε_{mu}
FE (8) 1	9.5	0.0027	2.860	0.0053
FE (8) 2	1.33*9.5	0.0027	2.860	0.0053
FE (8) 3	0.75*9.5	0.0027	2.860	0.0053
FE (8) 4	9.5	1.33*0.0027	2.860	0.0053
FE (8) 5	9.5	0.0027	1.33*2.860	0.0053
FE (8) 6	9.5	0.0027	2.860	1.33*0.0053
FE (8) 7	9.5	0.0027	1.33*2.860	1.33*0.0053
FE (8) 8	1.25*9.5	0.67*0.0027	1.5*2.860	1.50*0.0053

Table 4: Variables in analyses of Specimen 9 of Mehrabi (1994)

Analysis	f'_{cm} (MPa)	ε_{0m}	f'_{mu} (MPa)	ε_{mu}
FE (9) 1	14.21	0.0026	4.263	0.0054
FE (9) 2	1.33*14.21	0.0026	4.263	0.0054
FE (9) 3	0.67*14.21	0.0026	4.263	0.0054
FE (9) 4	14.21	1.33*0.0026	4.263	0.0054
FE (9) 5	14.21	0.0026	1.33*4.263	0.0054
FE (9) 6	14.21	0.0026	4.263	1.33*0.0054
FE (9) 7	14.21	0.0026	1.33*4.263	1.33*0.0054
FE (9) 8	14.21	0.75*0.0026	1.5*4.623	1.5*0.0054

In the case of Specimen 8, the model with a 25% increase in f'_{cm} presents the best match with the experimental result (Figure 7h), whereas in the case of Specimen 9 no change in f'_{cm} is required to get the best match (Figure 8h). In both cases a reduction of 33% and 25% in ε_{0m} for Specimen 8 and 9, respectively, as well as a considerable increase of 50% in f'_{mu} and ε_{mu} is required to match the experimental results. The FE results of detailed three-dimensional models of the same specimens constructed using ANSYS are shown in Figures 7i and 8i (Mohyeddin et al., 2013b). In these analyses f'_{mu} is taken as 2 MPa, which is much less than those of FE (8) 8 and FE (9) 8 (Figures 7h and 8h) and more realistic. While the analytical results shown in Figures 7i and 8i show a very good match with those of experimental, Mohyeddin et al. (2013b) showed that in fact a 25% increase in ε_{0m} leads to a perfect match with experimental results for Specimen 8. This contradicts what is predicted by the equivalent strut models. Also, even though large variations in masonry material properties are normally expected, in the case of the two examples presented here the equivalent strut models estimate a rather unacceptable large variations (e.g. compare f'_{mu} related to Figures 7h and 8h with 2 MPa related to 7i and 8i). In the case of Specimen 8, the equivalent strut model requires a 33% increase in ε_{0m} to produce the best match with experimental results, whereas the detailed model created in ANSYS requires a 25% decrease.

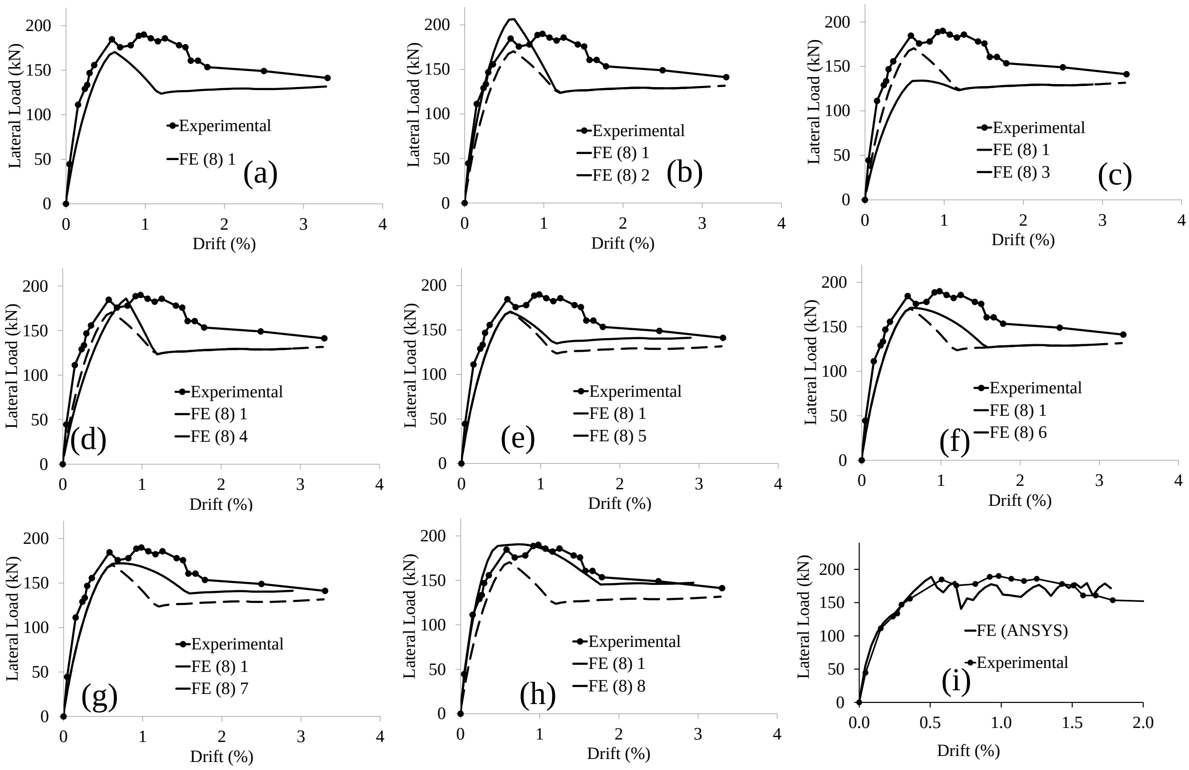

Figure 7: (a) to (h) A comparison between different load-deflection curves created using OpenSees for the Specimen 8; (i) load-deflection curve of the model created in ANSYS

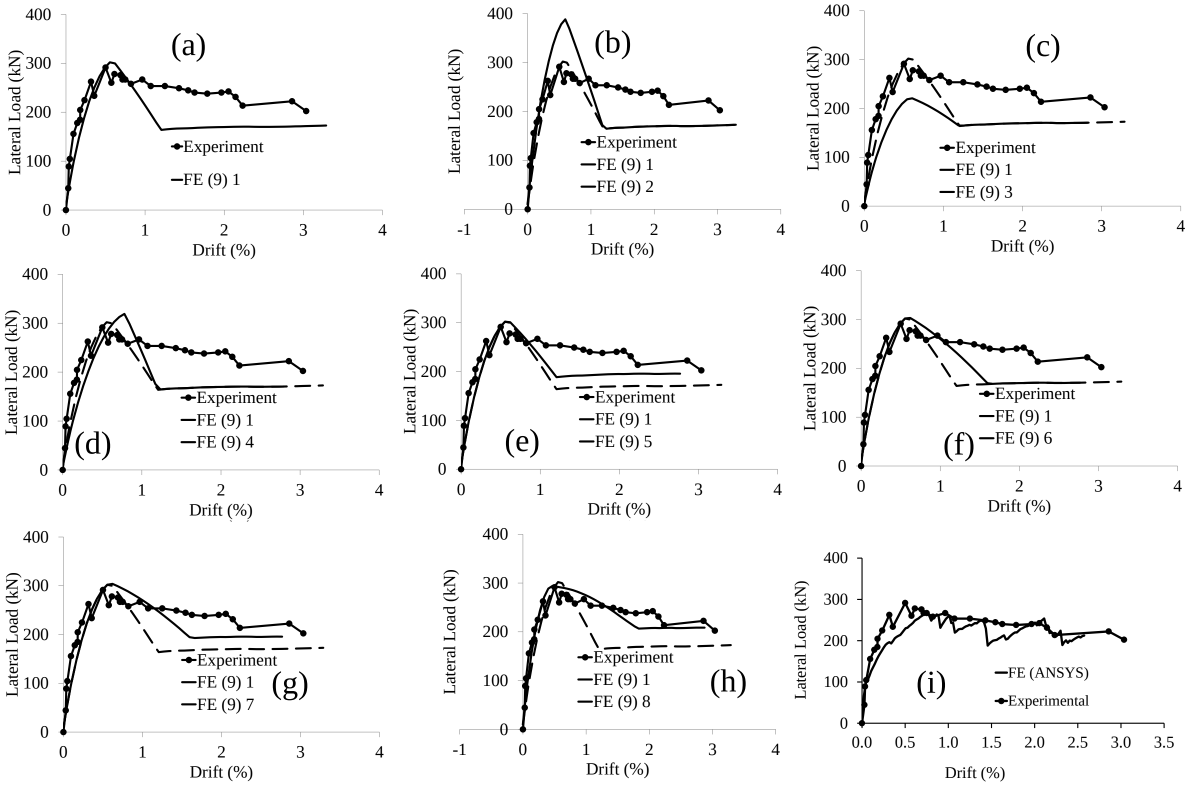

Figure 8: (a) to (h) A comparison between different load-deflection curves created using OpenSees for the Specimen 9; (i) load-deflection curve of the model created in ANSYS

To further investigate the reasons for such discrepancies three snapshots of the principal stresses at three different drift levels are provided in Figures 9 and 10 (Mohyeddin et al

2013a). These figures clearly show that not only a single strut is not a suitable representative of the infill panel, but also the location and number of the struts may vary as the level of drift increases. Such variations may be attributed to a variety of sources such as cracking of the masonry, location and length of the contact surface between the panel and frame at different levels of drift, relative stiffness of the frame and panel at different levels of drift, mortar properties and therefore the friction coefficient at the bed and head joints, and the gravity loads on the frame, to name a few.

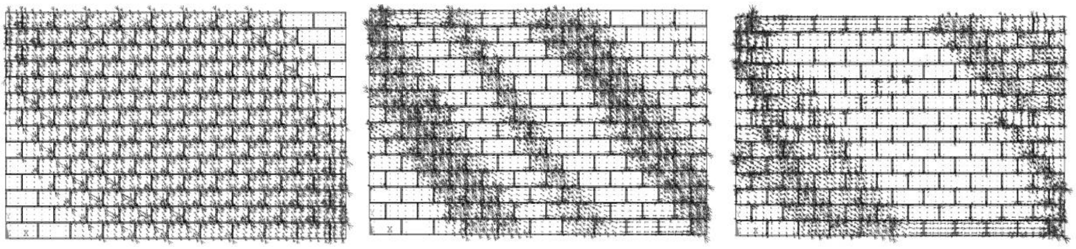

Figure 9: Principal (compressive) stresses in the infill panel of Specimen 8 at 0.08%, 0.76% and 1.73% drift

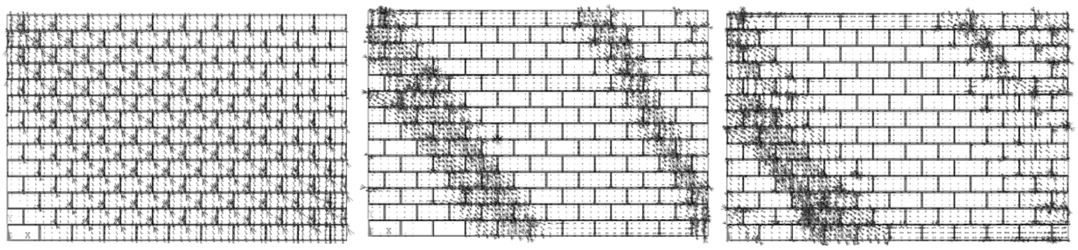

Figure 10: Principal (compressive) stresses in the infill panel of Specimen 9 at 0.02%, 1.06% and 2.53% drift

7 Discussion and Conclusions

The experimental results of four reinforced concrete frames with and without masonry infill panels are compared with those of a two-dimensional model created using OpenSees as well as a sophisticated three-dimensional model created using ANSYS. First the shortcomings of the concrete model of ANSYS are discussed and it is shown that the methodology utilised by Mohyeddin et al. (2013a) can confidently be used for the nonlinear modelling of concrete structures using ANSYS.

It is demonstrated that the equivalent strut model has some inherent limitations so that it cannot be applied for the nonlinear analysis of infill-frames as a generic tool. This is mainly due to the limited number of parameters involved in representing the high nonlinear behaviour of such structures. It is further illustrated that in order to properly replicate the in-plane behaviour of infill-frames the model should incorporate a “dynamic strut model” as the configuration and section properties of the compression strut(s) formed at different drift levels change. Prior research has mainly focused on the evaluation of the width of compression strut(s), and different researchers have proposed models with different numbers of struts and/or different configurations. Nevertheless, it is shown that not only the width, but also the number and location of compression strut(s) change during the application of lateral loads. Also, it is shown that the number of compression struts may vary from one model to the next. Given the inherent variability of the masonry material properties and the large number of parameters influencing the behaviour of an infill-frame, it is concluded that in order to appropriately apply the "strut" modelling approach in a general manner, the number, width and location of compression strut(s) should properly be evaluated at different levels of drift.

8 References

AL-CHAAR, G., ISSA, M. & SWEENEY, S. 2002. Behavior of Masonry-Infilled Nonductile Reinforced Concrete Frames. *Journal of Structural Engineering,* 128**,** 1055-1063.

ANGEL, R.E. 1994. *Behavior of Reinforced Concrete Frames with Masonry Infill Walls.* PhD, University of Illinois at Urbana Champaingn.

BARBOSA, A.F. & RIBEIRO, G.O. 1998. Analysis of Reinforced Concrete Structures using ANSYS Nonlinear Concrete Model. *Computational Mechanics. New Trends and Applications*.

CRISAFULLI, F.J. 1997. *Seismic Behaviour of Reinforced Concrete Structures with Masonry Infills.* PhD, University of Canterbury.

FANNING, P. & KELLY, O. 2000. Smeared Crack Models of RC Beams with Externally Bonded CFRP Plates. *Computational Mechanics,* 26**,** 325-332.

FEMA 273 1997. NEHRP Guidelines for the Seismic Rehabilitation of Buildings, Prepared by Applied Technology Council (ATC). Washington, D. C.

FEMA 306 1998. Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Prepared by Applied Technology Council (ATC). Washington, D. C.

FEMA 356 2000. Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Prepared by American Society of Civil Engineers (ASCE). Washington, D. C.

HASHEMI, A. & MOSALAM, K.M. 2006. Shake-Table Experiment on Reinforced Concrete Structure Containing Masonry Infill Wall. *Earthquake Engineering & Structural Dynamics,* 35**,** 1827-1852.

HOLMES, M. 1961. Steel Frames with Brickwork and Concrete Infilling. *Proceedings of the Institute of Civil Engineers,* 19**,** 473-478.

KLINGNER, R.E. & BERTERO, V.V. 1978. Earthquake Resistance of Infilled Frames. *Journal of the Structural Division, ASCE,* 104**,** 973-989.

LOTFI, H.R. & SHING, P.B. 1991. An Appraisal of Smeared Crack Models for Masonry Shear Wall Analysis. *Computers & Structures,* 41**,** 413-425.

LOURENCO, P.B. 1996. *Computational Strategies for Masonry Structures,* Netherlands, Delf University Press.

MAINSTONE, R.J. 1971. On the Stiffness and Strengths of Infilled Frames. *Proceedings of the Institution of Civil Engineers,* Supplement IV**,** 57-90.

MAINSTONE, R. J. & WEEKS, G. A. The influence of bounding frame on the racking stiffness and strength of brick walls. The second international brick masonry conference, 1970. 165-171.

MANEETES, H. & MEMARI, M. 2009. Finite Element Modeling of Reinforced Concrete Cladding Panels. *Electronic Journal of Structural Engineering,* 9.

MEHRABI, A.B. 1994. *Behavior of Masonry-Infilled Reinforced Concrete Frames Subjected to Lateral Loadings.* PhD, University of Colorado.

MOHYEDDIN, A. 2011. *Modelling and Performace of RC Frames with Masonry Infill under In-Plane and Out-of-Plane Loading.* PhD, The University of Melbourne.

MOHYEDDIN, A., GOLDSWORTHY, H. M. & GAD, E. F. 2010. FE Modelling of RC Frames under In-Plane Lateral Loads. *Australian Earthquake Engineering Society Conference.* Perth.

MOHYEDDIN, A., GOLDSWORTHY, H. M. & GAD, E. F. 2013a. FE modelling of RC frames with masonry infill panels under in-plane and out-of-plane loading. *Engineering Structures,* 51**,** 73-87.

MOHYEDDIN, A., GOLDSWORTHY, H.M. & GAD, E.F. 2013b. Sensitivity analysis of nonlinear behaviour of infill-frames under in-plane and out-of-plane loading. *Advances in Sructural Engineering*, 16 (10), 1729-1747.

ROTS, J.G. Computer Simulation of Masonry Fracture: Continuum and Discontinuum Models. *In:* MIDDLETON, J. & PANDE, G.N., eds. The International Symposium on Computer Methods in Structural Masonry, 1991 Swansea. 93-103.

SANEINEJAD, A. & HOBBS, B. 1995. Inelastic Design of Infilled Frames. *Journal of Structural Engineering,* 121**,** 634-650.

SCOTT, B.D., PARK, R. & PRIESTLEY, M.J.N. 1982. Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates. *ACI Journal,* 79**,** 13-27.

STAFFORD SMITH, B. 1962. Lateral Stiffness of Infilled Frames. *Journal of Structural Division, ASCE,* 88**,** 183-99.

STAFFORD SMITH, B. 1966. Behavior of the Square Infilled Frames. *Journal of Structural Division, ASCE,* 92.

WILLAM, K.J. & WARNKE, E.P. 1974. Constitutive Model for the Triaxial Behaviour of Concrete. *Proceedings of the International Association for Bridge and Structural Engineering, Bergamo, Italy,* 19**,** 1-30.

ZARNIC, R. & TOMAZEVIC, M. Study of the Behaviour of Masonry Infilled Reinforced Concrete Frames Subjected to Seismic Loading. *In:* MCNEILLY, T. & SCRIVENER, J.C., eds. The Seventh International Brick and Masonry Conference, Feb 17-20 1985 Melbourne. 1315-1325.