FE Modelling of RC Frames under in-Plane Lateral Loads

Alireza Mohyeddin[1], Helen M Goldsworthy[2] and Emad F Gad[3]

1. PhD student, Dept of Civil and Environmental Engineering, The University of Melbourne, VIC 3010, Australia. Email: amk@civenv.unimelb.edu.au
2. Senior Lecturer, Dept of Civil and Environmental Engineering, The University of Melbourne, VIC 3010, Australia. Email : h.goldsworthy@civenv.unimelb.edu.au
3. Associate Professor, Dept of Civil and Environmental Engineering, The University of Melbourne and Swinburne University of Technology, Victoria, Australia. Email: e.gad@civenv.unimelb.edu.au

Abstract

This paper summarises the results of FE analyses of a reinforced concrete (RC) bare-frame under gravity and push-over lateral loads using the SOLID65 finite element in ANSYS. A macro input file is produced which is capable of generating a FE model of a generic one-bay single-storey RC frame in ANSYS. The modelling strategy is discussed in detail and the FE results are compared with results available from experimental work conducted by Mehrabi (1994). This comparison shows a good agreement between the two. A sensitivity analysis concludes this paper and demonstrates the effect of some key parameters on the behaviour of the structure under consideration.

Keywords: Finite Element, ANSYS, Reinforced Concrete, Failure Criteria, Concrete Cracking, Confinement, Lateral Loads, Drifts

Introduction

ANSYS has been used by some researchers for FE modelling of concrete beams (Barbosa and Ribeiro 1998; Fanning and Kelly 2000; Fanning 2001; Maneetes and Memari 2009). However, there are some aspects of concrete modelling when using the ANSYS SOLID65 element which need detailed understanding and careful consideration. This is essential when this element is employed to model the behaviour over the full loading range, i.e. for a performance-based structural analysis. Most of the above researchers have assumed that concrete behaves linearly up to the point where it crushes and/or cracks. In fact the material can exhibit considerable nonlinearity in its behaviour prior to crushing. The generic FE model constructed here in ANSYS gives a good prediction of the behaviour of a non-ductile frame up to a drift level of 5%. An extensive range of parametric analyses were carried out in order to examine the effect of different ANSYS features on the model. The outcomes from these analyses are intended to assist with the development of accurate models without convergence problems.

Concrete Material Constitutive Model for Compression

The stress-strain curve of the uniaxial behaviour of concrete under compression has been proposed by many researchers (Kent and Park 1971; Scott, Park et al. 1982; Mander, Priestley et al. 1988a; Mander, Priestley et al. 1988b) and a number of concrete design standards (CEB-FIP Model Code 1990 1993; Eurocode 2 2005). One of the early equations for the ascending part of the stress-strain curve was proposed by Hognestad (1951) in the form of a Ritter's parabola. Many of the equations later proposed by other researchers, including those for confined concrete and/or masonry, have basically followed the same form of mathematical relationship.

In order to further apply the stress-strain relationships to FE analysis where the concrete is confined by reinforcement, and/or to consider the effect of confinement resulting from a biaxial and/or triaxial stress state, the stress-strain relationships must be modified such that the strength and ductility improvements are reflected. Such models have been proposed by different researchers (Kent and Park 1971; Scott, Park et al. 1982; Mander, Priestley et al. 1988b). A comparison between some of these models is shown in Figure 1. The modified Kent and Park model has been incorporated in the FE model constructed here since it shows a good agreement with the experimental results from the literature (Kent and Park 1971; Scott, Park et al. 1982) and offers a good balance between simplicity and accuracy (Taucer, Spacone et al. 1991).

The most notable shortcoming of the SOLID65 (concrete) finite element of ANSYS is that it assumes a linear stress-strain relationship for concrete (ANSYS Inc. 2009d). The assumption was also made when a commonly used failure criterion was developed by William and Warnke (1974). Various failure criteria, including the William-Warnke one are illustrated in Figure 2a. The assumption of a linear stress-strain relationship, although able to facilitate the derivation of a failure surface, is not adequate to represent the concrete material in a nonlinear analysis.

In Figure 2b experimental load-deflection curves are compared against three FE models which vary solely based on the material models. The dotted line shows the results of an analysis where a nonlinear stress-strain curve is defined. Along with that, both compressive and tensile strengths of concrete are specified. This means that a complete Willam-Warnke failure surface is created by ANSYS (Figure 2a). Nevertheless, this analysis significantly underestimates the strength of the frame. This analysis shows that once the stress state in an element reaches the compressive strength defined by the Willam-Warnke failure surface, the element is eliminated and therefore, the rest of the nonlinear stress-strain curve becomes ineffective. The end result is that the concrete does not follow the softening branch of the stress-strain curve. The dashed line shows the results of a similar analysis where just the tensile strength of concrete is defined. This analysis has successfully replicated the experimental force-deflection curve; although for the compressive stress state the Willam-Warnke failure surface is suppressed. For comparative purpose only, another analysis for which no cracking/crushing is specified is shown by the solid line. In this analysis, the concrete material is hypothetically assumed to behave as a ductile material, with a Huber-von Mises failure criterion, following the same nonlinear stress-strain relationship for both compression and tension (which is not acceptable).

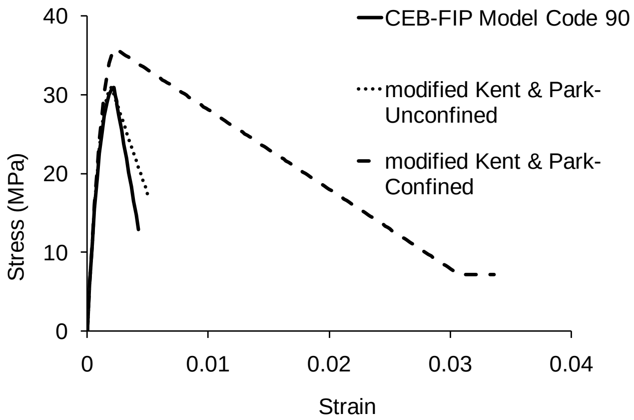

Figure 1: A comparison between different constitutive models for concrete

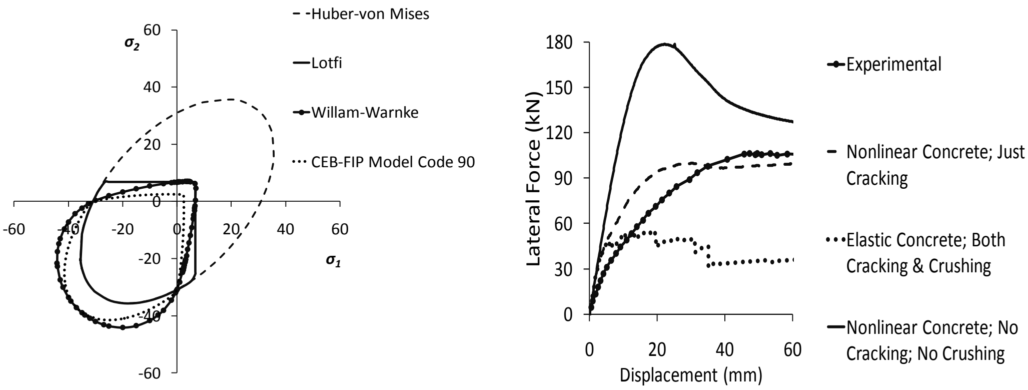

Figure 2: A comparison between different failure surfaces for plane-stress state in concrete (a), and different analytical results in ANSYS using different material modelling techniques (b)

As can be seen in Figure 2b the combination of a nonlinear compressive stress-strain relationship for the concrete and another concrete material model to represent the tensile strength of the concrete gives results that match well with the experimental ones. Since the Willam-Warnke failure surface cannot be constructed if only the tensile strength (f'_t) is given, it appears that ANSYS implements a Huber-von Mises failure criterion and uses the tensile strength as a tension cut-off. This approach is similar to that taken by Lotfi and Shing (1991). This method has been subsequently implemented in the FE modelling presented in this paper.

A further investigation has been carried out to determine whether large deflection effects can be included. The ANSYS Element Reference (2009c) recommends that when either cracking or crushing is activated in the FE model and significant rotations are involved, considering large deflection effects may create convergence issues and the results may be incorrect. In response to personal communications with the developer (ANSYS HQ via LEAP Australia, 2009 and 2010), the authors were informed that “there are no specific reasons why restrictions were added”. This has been explored in the analytical work which indicates that the large deflection effect can be accommodated successfully.

Shear behaviour of SOLID65 is controlled by two shear transfer coefficients for open and closed cracks that need to be defined. Preliminary analyses of a simply supported RC beam showed that the results are not sensitive to the shear coefficient for an open crack. Different values for this parameter did not change the maximum applied load on to the beam. However, the displacement at which yielding occurred did vary slightly. In some cases a very low/high value of this coefficient led to difficulties in convergence. Since the effect of this parameter on overall behaviour of the beam was not significant, it is suggested that this parameter can be used as a tuning tool in cases where convergence becomes an issue. For the FE models here, the shear coefficient for open cracks has been calculated based on the transverse reinforcement of the sections.

Concrete Material Constitutive Model for Tension

Although the concept of fracture energy is widely used in analytical models for concrete cracking (van Mier 1986; CEB-FIP Model Code 1990 1993; Bazant and Becq-Giraudon 2002; Wittmann 2002; Eurocode 2 2005), in ANSYS tensile cracks are not directly related to the fracture energy, and cracking is defined by a single material parameter i.e. the tensile strength of concrete, f'_t. ANSYS considers a bi-linear softening branch for the concrete tensile stress-strain curve. Failure to explicitly consider the fracture energy is one of the shortcomings of ANSYS with regard to the concrete material model in comparison to other commercial programs such as DIANA (2002). Moreover the user has no control over the strain at which the tensile stress becomes zero and it is taken to be at $6\varepsilon_{cr}$, where ε_{cr} is equal to the strain corresponding to f_c'. This means that the fracture energy encompassed by the curve is controlled by the tensile strength of concrete and ε_{cr}. Since the slope of the ascending branch is already considered to be equal to the modulus of elasticity, E_c, ε_{cr} is also defined by f'_t. However, by knowing mode I fracture energy (G^I_F) and f'_t one can still define the tensile stress relaxation such that the energy dissipated under the stress-strain curve approximates that recorded in experimental works. On the other hand, for a better convergence during solution, it is sometimes beneficial to eliminate the stress relaxation term which results in the bilinear softening branch being downgraded to a linear curve.

Material Constitutive Model for Reinforcing Steel

A bi-linear stress-strain relationship is assumed for the steel reinforcement. This is well-established in the literature as being compatible with the actual behaviour of structural steel. The modulus of elasticity, E_s, is assumed to be equal to 200*GPa*. The secondary stiffness, sometimes referred to as the “tangent stiffness”, which is here denoted by E_2, varies based on the steel grade and ductility, rebar diameter and the yield stress and can be found in different references (Eurocode 2 2005; Reynolds, Steedman et al. 2008). As it is applicable to most metals, the Huber-von Mises failure criterion with a total stress range of twice the yield stress (Bauschinger effect) is used here for the reinforcing steel. All steel rebars are modelled as smeared reinforcement.

Therefore no explicit element representing reinforcing rebars is defined here. The drawback of this method is that the debonding between the reinforcing rebar and the concrete is not represented in the FE model.

FE Model for a Reinforced Concrete Frame

A 3D FE model has been constructed in ANSYS 12.1 in such a manner as to facilitate the implementation of a series of parametric analyses. The FE results are benchmarked against experimental results of an RC bare-frame tested by Mehrabi (1994). Since this particular frame was designed for wind load only and no special provisions for seismic design were considered, its detailing is relevant to design requirements in Australia. The structure is first analysed under gravity loads including loads coming from upper storeys and subsequently exposed to the lateral loads (displacements). The experimentally determined material properties for the frame are given in Table 1. The frame specifications can be found in Mehrabi (1994), some of which are also shown in Figure 3g. In constructing the FE model some assumptions had to be made. The cross sections of the beam, columns and their connections are meshed such that the longitudinal rebars are smeared over a limited number of elements rather than the whole cross-section. This is to locate the reinforcement in the appropriate locations. No longitudinal reinforcement is smeared across the elements of the cross-section representing the plain concrete cover and concrete core. The same strategy is used for the transverse rebars along the members (i.e. columns, beams and joints). They are smeared across a limited number of elements (i.e. limited length of the member) which in turn, represent the spacing between the stirrups.

There are other assumptions made with regard to the concrete cover and the material properties. For example, the exact dimensions of the concrete cover are not given in the experimental report (Mehrabi 1994). Based on personal communications (A. Stavridis 6 Aug. 2009), this information is not available. Accordingly, a cover of 25.4*mm* to the outer edge of the longitudinal rebars is generally assumed. Furthermore, a sensitivity analysis is carried out using 12.7*mm* cover to investigate the effect of concrete cover on the results.

Axial stress-strain test results from two steel rebars are given in Mehrabi (1994) based on which the secondary slope of the stress-strain curve, E_2, is approximately measured to be 3% of E_s. However, more tests would be required for a statistically valid value of E_2. Therefore, E_2 is selected as one of the parameters to be investigated in a sensitivity analysis.

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*

FE Results

A series of sensitivity analyses are conducted to investigate the effect of different parameters on load-deflection curves. These analyses are listed in Table 2. Figures 3a to 3e show the results of these analyses in the form of load-deflection curves.

Table 2: Variables in different analyses

Analysis No.	Concrete Model	E_2 (% of E_s)	Cover (*mm*)	Concrete (*MPa*) f'_c	Concrete (*MPa*) f'_t	Large Deflection
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 3a represents the analytical results from Analyses *1* and *2* and shows why using a confined constitutive model is essential. Despite the presence of smeared transverse reinforcement in the model, the modelling is such that this reinforcement is not capable of providing a confining effect to the concrete. Hence, the confinement effect must be incorporated in the concrete material model in order to appropriately track the load-deflection curve and ductility of the frame.

The initial stiffness is generally overestimated by the FE model. This is mainly related to the fact that initial cracks due to shrinkage are not considered in the FE model and the frame is assumed to be uncracked at the onset of loading. Furthermore, as shown in Figure 2, the FE model tends to overestimate the strength in tension-compression regions i.e. when cracking initiates. Since the initiation of nonlinearity is mainly because of cracking than crushing, the model shows a relatively stiffer behaviour at the beginning of the analysis. This discrepancy, however, diminishes as soon as other actions such as yielding of the reinforcement or crushing/softening of concrete start to take effect. That being said Figures 3g and 3h compare the crack pattern of the actual test specimen with the crack pattern of the frame after 60*mm* of imposed lateral displacement from Analysis *1*, and they are in good agreement, indicating that the model is giving reasonable results.

The influence of the steel secondary stiffness (E_2) on the load-deflection curve can be observed from Figure 3b (Analyses *1*, *3* and *4*). As expected, the effect of this parameter can enhance/weaken the strength of the frame, while it does not have any influence on the stiffness on the ascending branch of the curve. In contrast to the steel secondary stiffness, any change in the properties of the concrete material has a direct effect on the behaviour over the full range of the load-deflection curve. This is illustrated in Figure 3c where the results of analyses (*1*, *5* and *6*) are shown to be significantly different. Figure 3d, in which Analysis *1* is compared with Analysis *7*, shows the effect of the size of concrete cover on the behaviour of the frame. A significant difference is observed which is mostly attributed to the lever-arm of longitudinal reinforcement. Although this effect is well-known for design purposes, it is worth emphasising that when assessing existing buildings it is important to have a good estimation of the location of the reinforcement. Since the columns are subject to extra gravity loads to represent the effects of upper storeys, the analysis in which the effect of large-deflections is taken into account (Analysis *8*) is expected to show less stiffness and strength. This is illustrated in Figure 3e.

The analyses are concluded by a supplementary analysis of the model used in Analysis *1* in which the specimen is subjected to two complete loops of lateral displacement of 60*mm*. The hysteresis loops in Figure 3f show the degradation of the load-deflection curves.

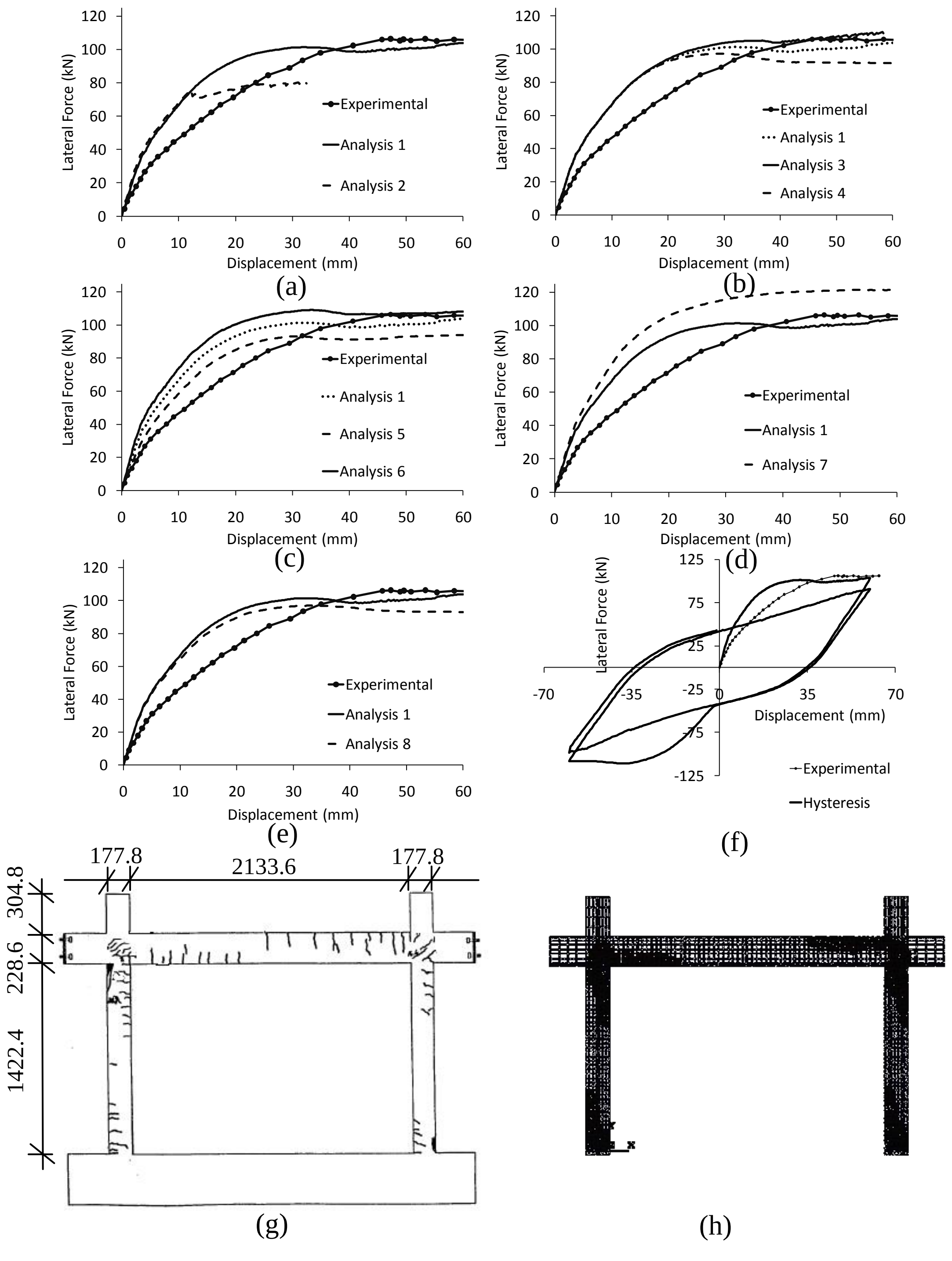

Figure 3: A comparison between different load-deflection curves (a) to (e); hysteresis loops (f); and a comparison between test (g) and FE (h) crack patterns (dimensions in mm)

Conclusions

A generic 3D FE model has been constructed in ANSYS which can be utilised for studying one-bay single-storey RC frames with rectangular cross-sections for the beam and columns. The model is validated against experimental results from the literature and is in good agreement with test results. This generic model can be utilised for parametric nonlinear analyses of RC frames to investigate their performance at different levels of loading e.g. performance-based analysis. These parameters include frame span and height, column and beam dimensions, amount of reinforcement, concrete and steel material properties and stress-strain relationships. The results of some of these parametric analyses have been reported. Despite uncertainties regarding some of the aspects of concrete material modelling in ANSYS, the material model developed using the available material library is shown to give reasonable results. A comprehensive series of analyses have been conducted to determine the best way to represent the concrete material behaviour in ANSYS, and a detailed discussion of the conclusions from this work has been presented. Load-deflection curves, concrete crack patterns and modes of failure of the model correctly predict the test results. The model has been extended to include hysteretic behaviour and hence the results can be used to predict cyclic behaviour such as the amount of hysteretic damping. This model will be further employed in the context of infill-frame analysis.

References:

ANSYS Inc. (2009c). Element Reference (Release 12.1). Canonsburg, PA.

ANSYS Inc. (2009d). Theory Reference for the Mechanical APDL and Mechanical Applications (Release 12.1). Canonsburg, PA.

Barbosa, A. F. and G. O. Ribeiro (1998). "Analysis of Reinforced Concrete Structures Using Ansys Nonlinear Concrete Model." Computational Mechanics. New Trends and Applications.

Bazant, Z. P. and E. Becq-Giraudon (2002). "Statistical Prediction of Fracture Parameters of Concrete and Implications for Choice of Testing Standard." Cement and Concrete Research **32**: 529-556.

CEB-FIP Model Code 1990 (1993). Comite Euro-International Du Beton, Thomas Telford.

DIANA (2002). DIANA Finite Element Analysis, User's Manual, Material Library (Release 8.1), TNO Building and Construction Research

Eurocode 2 (2005). "Design of Concrete Structures - Part 1-1 General Rules and Rules for Buildings."

Fanning, P. (2001). "Nonlinear Models of Reinforced and Post-Tensioned Concrete Beams." Electronic Journal of Structural Engineering **2**: 111-119.

Fanning, P. and O. Kelly (2000). "Smeared Crack Models of Rc Beams with Externally Bonded CFRP Plates." Computational Mechanics **26**(4): 325-332.

Hognestad, E. (1951). A Study of Combined Bending and Axial Load in Reinforced Concrete Members, University of Illinois Engineering Experiment Station.

Kent, D. C. and R. Park (1971). "Flexural Members with Confined Concrete." Journal of the Structural Division, ASCE **97**(ST7): 1969-1990.

Lotfi, H. R. and P. B. Shing (1991). "An Appraisal of Smeared Crack Models for Masonry Shear Wall Analysis." Computers & Structures **41**(3): 413-425.

Mander, J. B., M. J. N. Priestley and R. Park (1988a). "Observed Stress-Strain Behavior of Confined Concrete." Journal of Structural Engineering **114**(8): 1827-1849.

Mander, J. B., M. J. N. Priestley and R. Park (1988b). "Theoretical Stress-Strain Model for Confined Concrete." Journal of Structural Engineering **114**(8): 1804-1826.

Maneetes, H. and M. Memari (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. Department of Civil, Environmental and Architechtural Engineering. Boulder, University of Colorado. **PhD**.

Reynolds, C. E., J. C. Steedman and A. J. Threlfall (2008). Reynolds's Reinforced Concrete Designer's Handbook, Taylor and Francis.

Scott, B. D., R. Park and M. J. N. Priestley (1982). "Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates." ACI Journal **79**(1): 13-27.

Taucer, F., E. Spacone and F. C. Filippou (1991). A Fiber Beam-Column Element for Seismic Response Analysis of Reinforced Concrete Structures. Berkeley, Earthquake Engineering Research Center, University of California at Berkeley.

van Mier, J. G. M. (1986). "Fracture of Concrete under Complex Stress." HERON **31**(3): 1-90.

Willam, K. J. and E. P. Warnke (1974). Constitutive Model for the Triaxial Behaviour of Concrete. Proceedings of the International Association for Bridge and Structural Engineering, Bergamo, Italy **19**: 1-30.

Wittmann, F. H. (2002). "Crack Formation and Fracture Energy of Normal and High Strength Concrete." Sadhana **27**(4): 413-423.