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Modelling non-ductile reinforced concrete beam-column joints

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ABSTRACT: It is very common for buildings in Australia to have shear walls and cores as the primary lateral load resisting system and reinforced concrete moment resisting frames as the gravity load resisting system. However, often little to no attention is given to the displacement capacity of these frames and their ability to move with the walls in the event of an earthquake. In particular, there are concerns about the seismic response of the beam-column joints as they are typically poorly detailed and are vulnerable to failure mechanisms which are not accounted for in design. This paper presents interim findings of a study that is aimed to examine practical means of modelling non-ductile reinforced concrete beam-column joints. The two main mechanisms which contribute to joint inelastic behaviour, the shear response of the joint core and the slip of longitudinal beam reinforcement bars, are investigated. The findings of this study will be useful in assessing the seismic performance of buildings with non-ductile reinforced concrete gravity moment resisting frames.

Keywords: Non-ductile reinforced concrete beam-column joints, gravity moment resisting frames, inelastic joint behaviour

1. INTRODUCTION

There has been a growing interest in Australia to assess the seismic performance of vulnerable buildings due to greater awareness of the potential losses that may be endured by cities in the event of an earthquake. This research is primarily interested in the performance of reinforced concrete (RC) moment resisting frames (MRFs) which form part of the gravity load resisting system in buildings that have shear walls/cores as the primary lateral load resisting system. This form of construction is very common in Australia; however, there is a general concern that the gravity frames do not have sufficient displacement capacity to move with the walls under seismic loading due to their non-ductile detailing. This is particularly a concern for older mid-rise buildings constructed prior to the 1990s when no consideration was given to seismic loading. Such buildings which have eccentrically placed cores, due to the preference of providing open floor space for occupants and hence significant displacement demand may be imposed on the perimeter frames due to torsional effects.

In general, there has been lack of research into the performance of frames forming part of the secondary structural system. However, over the past few decades significant research has been conducted on the performance of non-ductile MRFs forming part of the primary lateral load resisting system in regions of high seismicity (Park, 1996; Hakuto, Park & Tanaka, 2000; Ghannoum, Moehle & Bozorgnia, 2008). The deficiencies in the design and detailing of these frames are very similar to those in gravity frames designed in accordance with the current Australian Concrete Structures standard, AS 3600 (2009). Some of the common characteristics are:

- Minimal to no transverse reinforcement (ties) in the beam-column joint region
- Poor anchorage of the longitudinal reinforcement bars in the joint region
- Inadequate transverse reinforcement in beams and columns (for shear strength and confinement)
- Splices located in potential hinge regions
- Inadequate longitudinal reinforcement in columns

Based on the deficiencies highlighted, it is clear that the two greatest concerns of RC frames are the performance of the columns and the beam-column joints. The focus of this paper is the latter.

It is common practice to assume rigid joints in the design of reinforced concrete (RC) moment resisting frames (MRFs) irrespective of the type of detailing that is provided. This assumption is not valid when assessing the seismic performance of frames as there may be a significant reduction in the joint rigidity due to shear deformation of the joint panel and slip of beam longitudinal reinforcement bars. Therefore, the assumption of rigid joints often results in an overestimation of the frames' (and also the buildings') stiffness and an underestimation of the expected drifts (Shafaei, Zareian, Hosseini & Marefat, 2014). Furthermore, the consideration of joint flexibility is particularly important for non-ductile RC MRFs since significant strength and stiffness degradation can occur once the maximum joint shear strength is reached. Therefore, in order to accurately assess the performance of non-ductile RC frames and hence the global response of buildings, it is critical to incorporate the inelastic behaviour of the joints (and hence joint flexibility) in numerical models.

2. LITERATURE REVIEW

The behaviour of beam-column joints under lateral loading is complex; however it is well accepted that two key mechanisms define the joint response: (i) the inelastic shear response of the joint core and (ii) the slip of bottom longitudinal beam reinforcement bars. The consideration of these two mechanisms, which leads to a reduction in joint rigidity, is critical in assessing the performance of non-ductile joints since premature and sudden failure of the joints may occur before full capacity of the frame members have developed.

Various numerical models have been established to incorporate joint behaviour in the modelling of frames. Some of these models are illustrated in Figure 1**.**

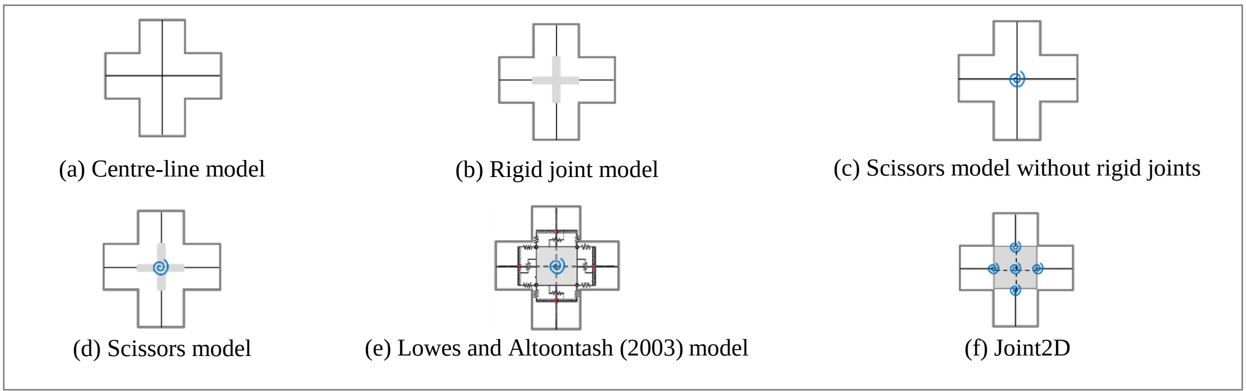

Figure 1: Summary of various joint models

The center-line model and the rigid joint model (Figure 1(a) and 1(b)) ignore the effects of joint flexibility, which is the common practice for the design of gravity frames. In these models it is assumed that the behaviour of the frame will be governed only by the behaviour of the beams and the columns. The explicit modelling of joint response has become possible with the introduction of zero-length rotational spring elements, which also allow the decoupling of the inelastic response of beams and columns. One of the first models to incorporate zero-length rotational spring elements was by El-Metwally and Chen (1988, cited Celik & Ellingwood, 2008) where the spring is located at the intersection of the beam and column members (Figure 1(c)). The inelastic behaviour of the joint is defined through the spring via a load-deformation response. This model is sometimes referred to as the scissors model without rigid joints, and was later improved by Alath and Kunnath (1995, cited in Celik & Ellingwood, 2008) and is better known as the scissors model (Figure 1(d)). The shear deformation of the joint core (panel) is simulated via the zero-length rotational spring element; however, the beams and the columns are connected via rigid links in this model and are capable of rotating independently.

More recently a continuum type of element has been introduced, combined with transition interface elements to allow for compatibility with beam-column line elements. An example of this is the model introduced by Lowes and Altoontash (2003) (Figure 1(e)). The model, which explicitly simulates three inelastic mechanisms of a joint consists of: (i) one rotational spring to model the shear response of the joint core, (ii) eight bar-slip springs to represent the bond failure of the longitudinal bars within the beams and the columns, and (iii) four interface-shear springs to model the loss of shear load transfer at the beam-joint and column-joint interfaces due to crushing of the concrete. While the model provides high control over the various inputs its disadvantage is the increased computational effort. Also there is the lack of availability of detailed response of various components (such as bond-slip). Therefore the model was simplified by Altoontash (2004) and the simplified model is commonly referred to as the Joint2D (Figure 1(f)). Joint2D has a rotational spring to model the shear deformations within the joint core, and it has four zero-length rotational springs at the beam-joint and column-joint interfaces to model bond-slip behaviour of the longitudinal beam and column bars. Both the Lowes and Altoontash (2003) model and Joint2D model have been implemented in OpenSees (2013).

A critical part of any of the models which attempt to simulate joint behaviour is the load-deformation response which is more commonly referred to as the backbone or envelope response. The most accurate method to obtain the backbone curve is from experimental testing in order to capture the complex behaviour of joints. To the knowledge of the authors there is no consensus on one available empirical or numerical model that is capable of providing the backbone curve of various joints with different detailing. However, it is generally accepted that for seismically detailed beam-column joints, the modified compression field theory (MCFT) may be used to provide a good approximation of the expected backbone curve (Altoontash, 2004); although this approach is generally not suitable for non-ductile joints (Celik and Ellingwood, 2008). Instead, strut and tie models are preferred to obtain the joint shear strength since the primary mechanism formed in non-ductile joints to transfer forces is a single compression strut formed between the compression zones of the adjacent beam/s and columns (shown in Figure 2). In addition there is a preference to use strut and tie models because they are easier to implement than the MCFT, thus making it a more practical option for the assessment of numerous MRFs with different joint detailing which can then be used to construct fragility curves.

In this paper the strut and tie model and the approach suggested by Celik and Ellingwood (2008) are adopted to construct the backbone curve for various interior and exterior joints. Once the backbone curve is obtained, hysteresis rules need to be defined to conduct nonlinear cyclic analysis of the joints. This is completed for one of the exterior joints presented in published literature by using the Lowes and Altoontash (2003) model in OpenSees. The results are compared with experimental results to examine the validity of the proposed approach.

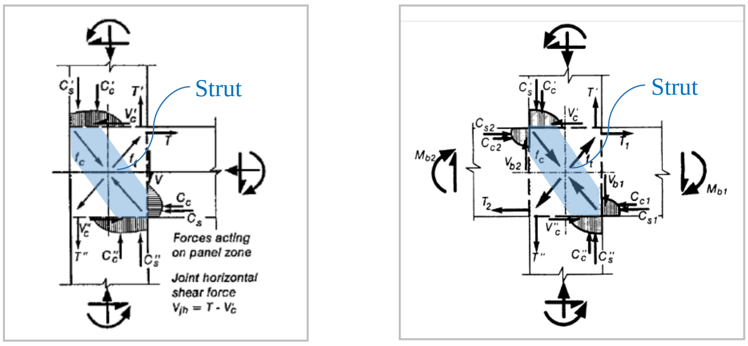

Figure 2: Forces acting on exterior and interior joints (adopted from Hakuto, Park & Tanaka, 2000)

3. METHODOLOGY

The four key steps involved in conducting nonlinear cyclic analysis for non-ductile joints are discussed in this section.

3.1 Defining the backbone curve

The two primary mechanisms which need to be captured by load-deformation response are:

(i) Inelastic shear response of the joint core/panel
(ii) Bond-slip of poorly anchored longitudinal beam bars

The approach suggested by Celik and Ellingwood (2008) to establish the backbone curve combines the effects of the two mechanisms into a single stress-strain joint envelope for positive and negative bending. The effect of bond-slip is taken into account by reducing the yield and moment capacity of the beam under positive bending based on a reduction factor obtained from numerous experiments. The rotation experienced by the joint is approximated as the angular joint shear strain (γ), and a range of values are provided based on experimental tests. The rotation caused by bond-slip is ignored since this additional rotation is usually very small. A brief overview of the Celik and Ellingwood (2008) model is provided in Table 1.

Table 1: Celik and Ellingwood (2008) shear stress-strain backbone model guide

Critical point	Positive envelope	Negative envelope
1. Shear cracking strength, $\tau_{jh.cr}$	$\tau_{jh.cr} = 3.5\sqrt{1 + 0.002\left(\frac{P}{A_g}\right)}$ Where P/A_g is in psi Or $\tau_{jh.cr} = 24\sqrt{\left(\frac{1}{145}\right)\left(1 + 0.002\left(\frac{P}{A_g}\right)\right)}$ Where P/A_g is in MPa $0.0001 \leq \gamma_{cr} \leq 0.0013$	$\tau_{jh.cr} = 3.5\sqrt{1 + 0.002\left(\frac{P}{A_g}\right)}$ Where P/A_g is in psi Or $\tau_{jh.cr} = 24\sqrt{\left(\frac{1}{145}\right)\left(1 + 0.002\left(\frac{P}{A_g}\right)\right)}$ Where P/A_g is in MPa $0.0001 \leq \gamma_{cr} \leq 0.0013$
2. Reinforcement yielding, $\tau_{jh.y}$	$\tau_{jh.y} \leq \tau_{jh.max}$ Where: $\tau_{jh.y}$ is the shear stress corresponding to the stress imposed on the joint due to beam or column yielding reduced by α to account for bond-slip of longitudinal bars. ($0.4 \leq \alpha \leq 0.7$) $\tau_{jh.max}$ is the joint shear strength obtained from strut and tie model. $0.002 \leq \gamma_y \leq 0.01$	$\tau_{jh.y} \leq \tau_{jh.max}$ Where: $\tau_{jh.y}$ is the shear stress corresponding to the stress imposed on the joint due to beam or column yielding. $0.002 \leq \gamma_y \leq 0.01$
3. Ultimate capacity, $\tau_{jh.u}$	$\tau_{jh.u} \leq \tau_{jh.max}$ Where: $\tau_{jh.u}$ is the shear stress corresponding to the stress imposed on the joint due to beam or column reaching ultimate capacity reduced by α to account for bond-slip of bars. $0.01 \leq \gamma_u \leq 0.03$	$\tau_{jh.u} \leq \tau_{jh.max}$ Where: $\tau_{jh.u}$ is the shear stress corresponding to the stress imposed on the joint due to beam or column reaching ultimate capacity. $0.01 \leq \gamma_u \leq 0.03$
4. Residual strength, $\tau_{jh.res}$	$\tau_{jh.res} = \tau_{jh.cr}$ $0.03 \leq \gamma_{res} \leq 0.1$	$\tau_{jh.res} = \tau_{jh.cr}$ $0.03 \leq \gamma_{res} \leq 0.1$

3.2 Strut and tie model

A modified version of the strut and tie model presented by Hassan (2011) has been used in this study to obtain the maximum shear strength of the joints ($\tau_{jh.max}$). A brief overview of the model is provided in Table 2.

Table 2: Adopted strut and tie model

Effective strut compressive strength: $f_{cu} = \emptyset\beta_s . f_c'$	Where: $\emptyset = 0.85$ β_s is the concrete softening coefficient $\beta_s = \frac{1}{1+0.66cot^2\theta}$ as defined in AS 3600 for bottle-shaped strut θ is the strut angle (defined later) f_c' is the concrete compressive strength
Diagonal strut capacity: $D = f_{cu}A_{str}$	Where: A_{str} is the concrete strut area, $A_{str} = a_s . b_j$ b_j is the effective joint width a_s is the strut depth, $a_s = \beta_1\sqrt{a_b^2 + a_c^2}$ a_b is the compression zone depth of the beam $a_b = k. d_b$ $k = \left((\rho + \rho')^2 n^2 + 2(\rho + \frac{\rho' d_b'}{d_b})n\right)^{0.5} - (\rho + \rho')n$ n is modular ratio ($n = \frac{E_s}{E_c}$) ρ is the ratio of longitudinal beam bars in tension ρ' is the ratio of longitudinal beam bars in compression d_b is the distance to the centroid of tensile longitudinal beam bar from the extreme compressive fibre d'_b is the distance to the centroid of compressive longitudinal beam bar from the extreme compressive fibre $\beta_1 = 1 - 0.05 \times 0.145(f_c' - 27.6) \leq 1.0$ a_c is the compressive zone depth of the columns (approximated using the equation proposed by Paulay and Priestley) $a_c = \left(0.25 + 0.85\left(\frac{P}{f_c' A_g}\right)\right)h_c \leq 0.4h_c$ P is the axial load A_g is the gross-section of the columns h_c is the column depth (parallel to the direction of lateral loading)
Joint shear strength: $V_j = D. \cos(\theta)$	θ is the strut angle, $\theta = tan^{-1}\left(\frac{d_b - d_b'}{d_c - d_c'}\right)$ d_c is the distance to the centroid of tensile longitudinal column bar from the extreme compressive fibre d'_c is the distance to the centroid of compressive longitudinal column bar from the extreme compressive fibre
Joint shear stress: $\tau_{jh.max} = \frac{V_j}{A_j}$	A_j is the effective joint cross-sectional area $A_j = b_j h_c$

3.3 Hysteresis response

Once the backbone curve has been defined, it is necessary to define the hysteresis rules for the cyclic response of the joint. Non-ductile joints (and other elements such as columns) typically have degrading envelopes and pinched hysteresis response since significant strength and stiffness degradation takes place once the ultimate capacity is reached. Therefore a suitable hysteresis curve must be multi-linear and allow for a tri-linear unloading-reloading path to represent the pinching behaviour of the joint. The Pinching4 material model developed by Lowes and Altoontash (2003) is able to represent this behaviour and is used in this study (see Figure 3). The following parameters have been adopted for the hysteresis rules as recommended in Shafaei et al. (2014) for exterior joints with poorly anchored longitudinal beam bars:

$$rDispP = rDispN = 0.5 \quad (1)$$
$$rForceP = rForceN = 0.1 \quad (2)$$
$$uForceP = uForceN = 0.01 \quad (3)$$

Where:

rDispP defines the ratio of the deformation at which reloading occurs to the maximum historic deformation demand

rDispN defines the ratio of the deformation at which reloading occurs to the minimum historic deformation demand

rForce defines the ratio of the force at which reloading begins to force corresponding to the maximum historic deformation demand

rForceN defines the ratio of the force at which reloading begins to force corresponding to the minimum historic deformation demand

uForceP defines the ratio of strength developed upon unloading from negative load to the maximum strength developed under monotonic loading

uForceN defines the ratio of strength developed upon unloading from negative load to the minimum strength developed under monotonic loading

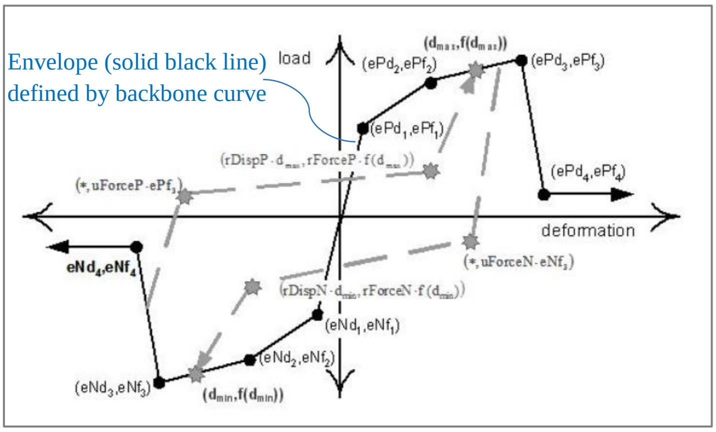

Figure 3: Pinching4 material model in OpenSees (Lowes & Altoontash, 2003)

In addition, there are another 15 parameters which can be defined for the Pinching4 material to control the loading and unloading stiffness and strength degradation. However, for this study these parameters are not utilised.

3.4 Defining rotational spring element

Based on the approach by Celik and Ellingwood (2008) highlighted in Section 3.1, only one zero-length rotational spring element is required to model the joint response. However, instead of implementing the scissors model, the Lowes and Altoontash (2003) model (Figure 1(e)) is adopted in this study since this joint element has been implemented by the developers in OpenSees (unnecessary springs are set to behave as rigid links). In addition, this allows for future work to refine the model (and to provide comparison) for cases where the bond-slip and joint response are modelled separately.

For the Lowes and Altoontash (2003) joint element, the moment imposed at the rotational spring element (M_j) may be calculated as:

$$M_j = \tau_{jh} V_j \qquad \text{(where } V_j \text{ is the joint volume)} \tag{4}$$

And the corresponding joint rotation (θ_j) is set to the angular joint shear strain:

$$\theta_j = \gamma \tag{5}$$

4. RESULTS AND DISCUSSION

4.1 Joint shear stress-strain backbone curves

The backbone curve obtained from the Celik and Ellingwood (2008) model for the lower, mid-range and upper bound of angular shear strains are compared with experimental results (shown in Figures 4-

6) obtained from published literature for interior and exterior joints which have detailing that are similar to Australian beam-column joints. Details of the joints are not provided due to space constraints. However an image of the joint is provided with the results, as well as the axial load ratio $\left(ALR = \frac{P}{f_c' A_g}\right)$.

4.1.1 Interior Joints

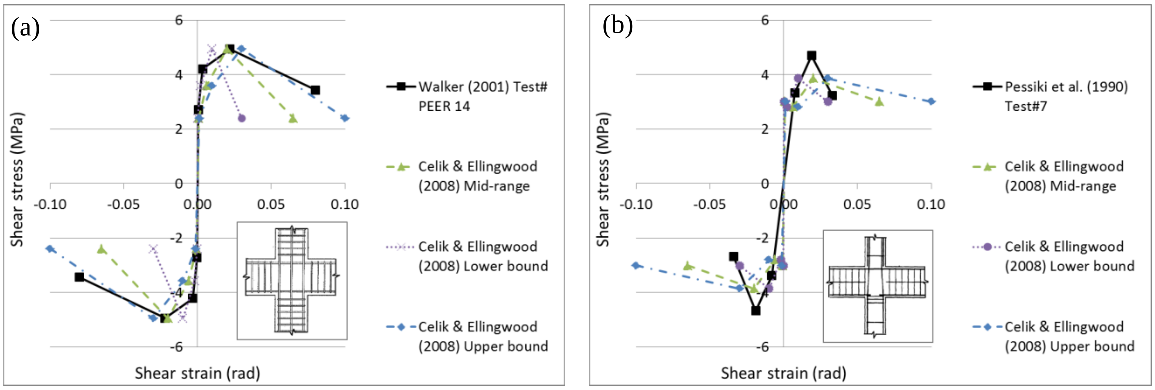

Figure 4: Joint stress-strain backbone (a) Walker (2001), test# PEER 14 with ALR=0.1, (b) Pessiki et al.,(1990), test# 7 with ALR = 0.36.

4.1.2 Exterior Joints

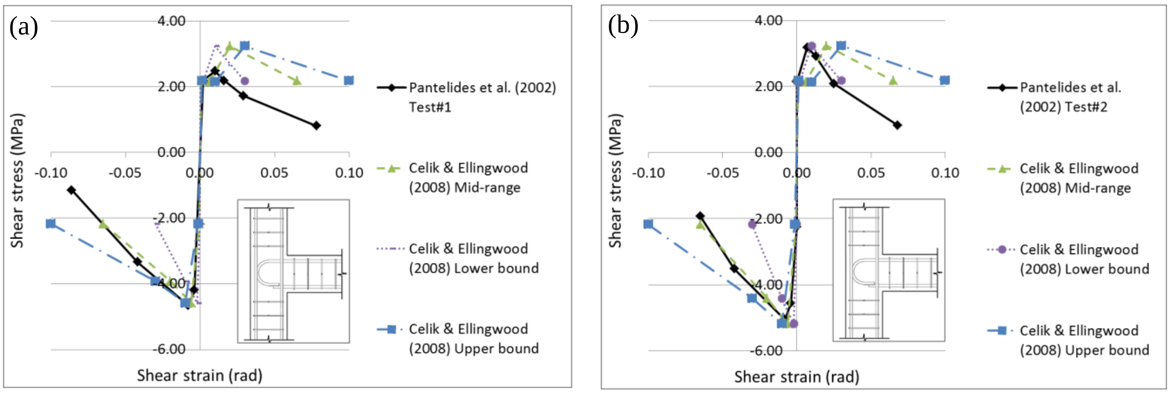

Figure 5: Joint stress-strain backbone, Pentelides et al. (2002) (a) test#1 with ALR 0.1, (b) test #2 with ALR 0.25 (embedment depth of beam bar is 150 mm)

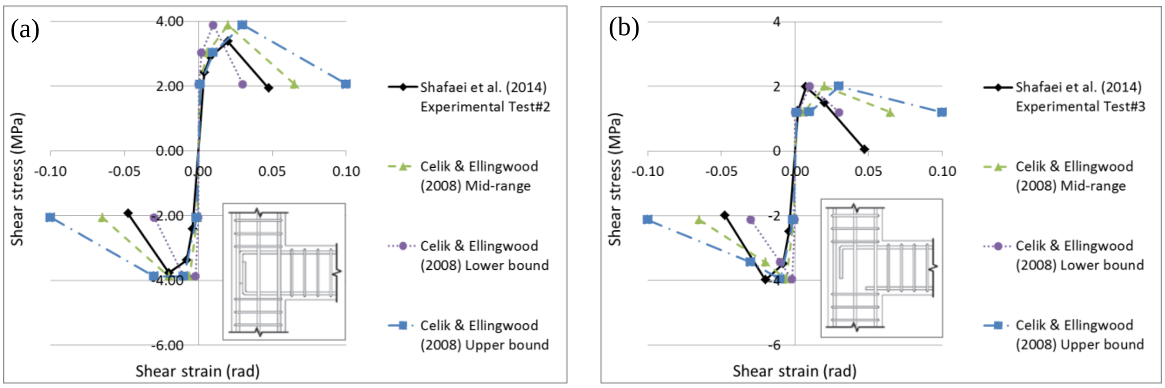

Figure 6: Joint stress-strain backbone, Shafaei et al. (2014) (a) joint test#2 with ALR=0.16, (b) joint test#3 with ALR 0.16 (embedment depth of beam bar is 75 mm)

The results show that in general the model is capable of predicting the ultimate capacity of the joints (except for Pentelides et al. (2002) joint test#1). It is also observed that the residual strength predicted by the model is not representative of the significant degradation which can take place in non-ductile joints after the ultimate shear capacity is reached. It is noted, however, that the residual strength predicted by Celik and Ellingwood (2008) would be suitable for predicting the performance of joints forming part of the primary lateral load resisting system, as failure of the joint is likely to be taken at 20% reduction of the peak capacity. Such an approach cannot be adopted when analysing the performance of joints in frames forming part of the secondary structural system where often the axial load failure is of interest. To be able to determine this point accurately it is necessary to have a model that is able to predict the joint strength at axial failure as well as the expected joint rotation. The determination of this point is the next stage of research which will be conducted by the authors in future work.

4.2 Nonlinear cyclic analysis

Comparison between the simulated results obtained from the nonlinear cyclic analysis for joint#3 presented in Shafaei et al. (2014) and the experimental results is provided in Figure 7. The results show the column shear versus the drift. It is noted that force-based nonlinear beam-column elements were used with five integration points to model the beam and the columns in OpenSees. Figure 7(a) illustrates the importance of considering joint response, as the rigid joint assumption is not capable of capturing the strength and stiffness degradation experienced by the beam-column joint, and hence overestimating the displacement capacity of the joints. Figure 7(b) shows the simulated response obtained in this study via using the experimental backbone curve presented in Shafaei et al. (2014). Figures 7(c) to 7(e) illustrate the response obtained from the three backbone curves established in this study (Figure 6(b)). Considering the simplicity of the adopted approach, the level of accuracy is acceptable and has the potential of being improved through modifications in the future.

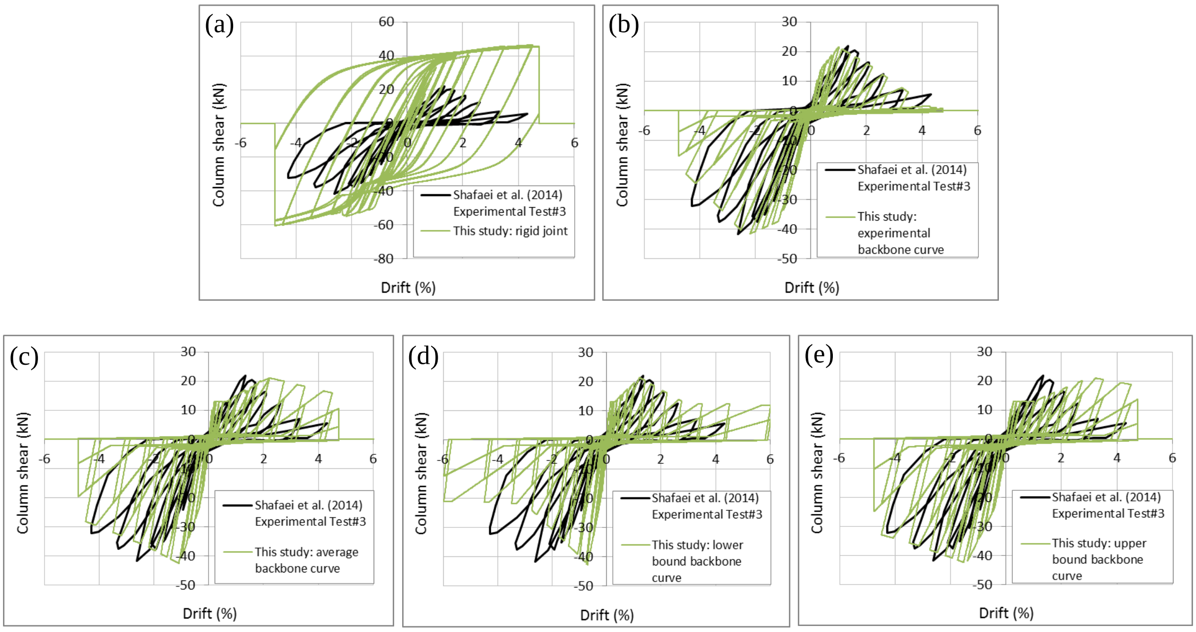

Figure 7: Comparison between simulated results by Shafaei et al. (2014) and this study for various backbone curves.

5. CONCLUSION

This study has investigated a practical method of modelling the response of non-ductile reinforced concrete beam-column joints under various axial load ratios. An applicable model for the assessment of numerous frames (required for the construction of fragility curves) requires simplicity (and hence efficiency) without significant compromise of the level of accuracy provided. The adopted backbone model, suggested by Celik and Elligwood (2008), and the modified strut and tie model originally presented by Hassan (2011) have shown to provide a reasonable degree of accuracy for the interior and exterior joints investigated in this paper. However, the authors intend to improve the proposed approach with particular focus on defining the point at which axial load failure occurs and hence to accurately determine the displacement capacity of non-ductile joints in frames forming part of the secondary structural system.

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