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Cyclic Behaviour of Wide Beam-Column Joints with Shear Strength Ratios of 1.0 and 1.7

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***Abstract*—**Beam-column connections play an important role in the reinforced concrete moment resisting frame (RCMRF), which is one of the most commonly used structural systems around the world. The premature failure of such connections would severely limit the seismic performance and increase the vulnerability of RCMRF. In the past decades, researchers primarily focused on investigating the structural behaviour and failure mechanisms of conventional beam-column joints, the beam width of which is either smaller than or equal to the column width, while studies in wide beam-column joints were scarce. This paper presents the preliminary experimental results of two full-scale exterior wide beam-column connections, which are mainly designed and detailed according to ACI 318-14 and ACI 352R-02, under reversed cyclic loading. The ratios of the design shear force to the nominal shear strength of these specimens are 1.0 and 1.7, respectively, so as to probe into differences of the joint shear strength between experimental results and predictions by design codes of practice. Flexural failure dominated in the specimen with ratio of 1.0 in which full-width plastic hinges were observed, while both beam hinges and post-peak joint shear failure occurred for the other specimen. No sign of premature joint shear failure was found which is inconsistent with ACI codes' prediction. Finally, a modification of current codes of practice is provided to accurately predict the joint shear strength in wide beam-column joint.

***Keywords*—**Joint shear strength, reversed cyclic loading, seismic codes, wide beam-column joints.

I. INTRODUCTION

WIDE beam system is one of the most popular structural forms in areas of low to medium seismicity due to its superior architectural, constructional and structural advantages such as minimizing the storey height, unifying the depths of beams and slabs, reducing the reinforcement congestion in the beam-column joint core and simplifying the construction of formwork [1]. However, there is no consensus on predicting the wide beam-column joint shear strength under cyclic loading among codes of practice on account of the limited experimental studies and the vacancy of analytical model. In the past decades, researchers mainly focused on investigating the effect of the beam to column width ratio [2]-[5], the percentage of beam reinforcement anchored in the joint core [2], the spandrel beam torsional resistance [1], [3], [6]-[10], the slab participation [11], the different performance between wide beam-column joints and conventional beam-column joints [1] and the different performance between seismically designed and as-built wide beam-column joints [12], [13], whereas the shear strength of the wide beam-column joint has not been investigated thoroughly yet. The main reason is that most of the specimens exhibited beam flexural failure and spandrel beam torsional failure in previous tests, while only a few premature joint shear failure was observed. Previous tests showed that the existence of spandrel beam helped to resist the joint shear through torsion and hence reduced the average shear stress in the joint core. As a consequence, the diagonal joint shear cracks were limited and few joint shear failure was reported [14], [15]. Without any usable test results and simple analytical models to refer to, the most popular seismic design codes, including ACI 318-14 & ACI 352R-02, Eurocode 8 and NZS 3101-06, do not reasonably estimate the upper bound of wide beam-column joint shear strength. Thus, this paper first reviews the joint shear prediction of three popular codes of practice mentioned above. Then the experimental results of two full-scale exterior wide beam-column joints tested at the Structural Lab in the Hong Kong University of Science and Technology are reported and analysed to investigate the upper bound of joint shear strength. Finally, a recommendation is proposed to modify the current ACI codes to accurately estimate the joint shear strength.

II. REQUIREMENTS OF CODES OF PRACTICE

To design a wide beam-column joint, the most important procedures are identifying the design joint shear force, defining the effective joint width and calculating the nominal joint shear strength. An exterior joint is chosen as an example to investigate the differences among three popular codes of practice, which is presented as follows.

A. ACI 318-14 and ACI 352R-02 [16], [17]

The design joint shear force defined by ACI codes consider the most critical combination resulted from the interaction of the multidirectional forces that the members transmit to the joint. For exterior joints, the value V_u can be calculated by:

$$V_u = 1.25 f_y A_s - V_{col} \tag{1}$$

where $1.25f_yA_s$ is the tension force induced by the flexure of beam reinforcement and V_{col} is the shear in the column calculated from overall equilibrium equation. It is obvious that the design joint shear force increases with the increase of beam reinforcement ratio. Then the nominal shear strength of wide

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beam-column joint V_n can be expressed as:

$$V_n = 1.25\sqrt{f_c^{'}}b_j h_c \quad (2)$$

where $f_c^{'}$ is concrete compressive strength from cylinder test, h_c is the column depth and b_j is the effective joint width which is considered as the column width b_c for wide beam-column joints. The ACI codes define a beam-column joint as a portion of the column within the depth of the deepest beam framed into, so only the area of the joint core is taken into account to resist the joint shear.

B. Eurocode 8 (EC8) [18]

The design joint shear force V_u of EC8 is similar to that of ACI codes except for a little difference in overstrength factor:

$$V_u = 1.2 f_y A_s - V_{col} \quad (3)$$

However, their definitions of joint shear strength V_n and effective joint width b_j are quite disparate, as shown in (4) and (5):

$$V_n = 0.8\eta f_{cd}\sqrt{1-\nu_d/\eta}\, b_j h_c \quad (4)$$

$$b_j = \min\{b_w; (b_c + 0.5h_c)\} \quad (5)$$

where f_{ck} and η are related to concrete compressive strength and ν_d is the normalised axial force in the column above the joint. In general, the strength calculated by EC8 is larger than that by ACI codes, mainly due to a larger effective joint width.

C. NZS 3101-06 [19]

The New Zealand standard for designing concrete structures does not provide an exact formula to calculate the design joint shear force, but states that it should be evaluated from a rational analysis considering the effect of all forces acting on the joint. The expression for joint shear strength is relatively simple, as shown in (6). The identification of effective joint width b_j is the same as EC8, as calculated through (5).

$$V_n = \min\{0.20 f_c^{'} b_j h_c; 10 b_j h_c\} \quad (6)$$

III. Experimental Programme

Two full-scale exterior wide beam-column joints were tested under quasi-static reversed cyclic loading. The specimens were mainly designed according to the ACI codes as shown in Fig. 1 and Fig. 2. Both the concrete dimensions and reinforcement details are identical except for the beam reinforcement ratio. S9 has a smaller beam reinforcement ratio compared with S10 which directly leads to a smaller joint shear force as explained in (1).

TABLE I summarizes the main design parameters of the specimens. It can be found that both specimens satisfy strong-column/weak-beam principle as the flexural strength ratios of column to beam are all larger than 1.2 [17]. The main variable is the design shear force $V_{u,ACI}$, which is calculated using ACI's formulation. The prediction of nominal shear strength V_n of different codes of practice are calculated and compared. It appears that the ACI codes give the smallest value while the EC8 provide the largest. According to ACI's prediction, the design joint shear force to shear strength ratios, or shear strength ratios in short, are 1.0 and 1.7 for S9 and S10, respectively, indicating that S9 is at a critical state between beam failure and joint shear failure while S10 should be failed in premature joint shear failure.

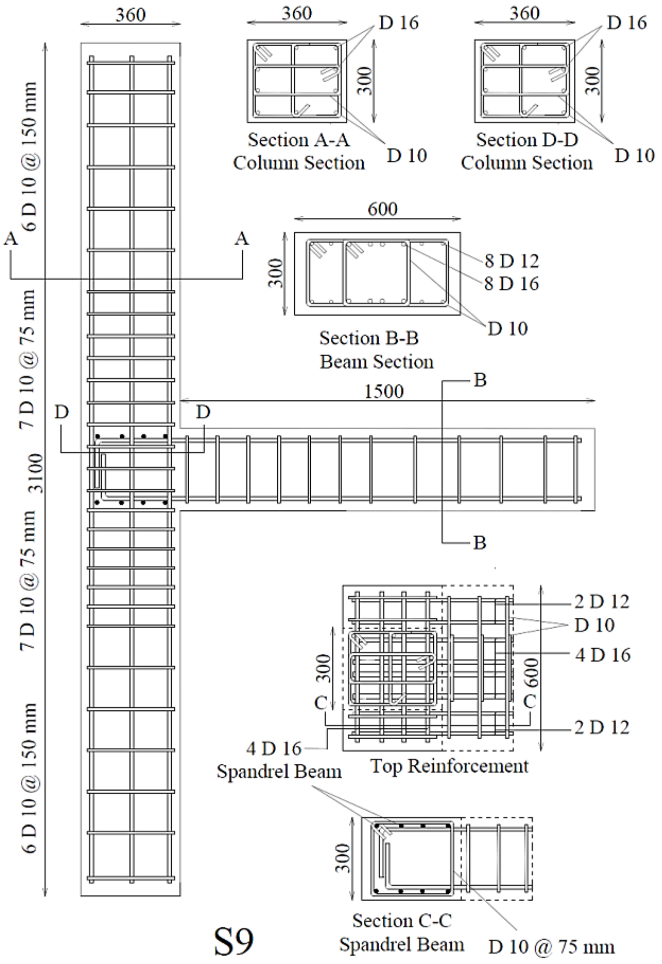


Fig. 1 Dimension and reinforcement details of S9

TABLE I
DESIGN PARAMETERS FOR S9 AND S10

Specimen	S9	S10
Beam reinforcement	4T16+4T12	1T25+2T20+4T16
Column reinforcement	10T16	10T16
M_c / M_b	2.0	1.3
ρ_b %	0.8	1.3
$V_{u,ACI}$ (kN)	778	1245
$V_{n,ACI}$ (kN)	767	750
$V_{n,EC}$ (kN)	1333	1275
$V_{n,NZS}$ (kN)	1123	1075
$V_{u,ACI} / V_{n,ACI}$	1.0	1.7

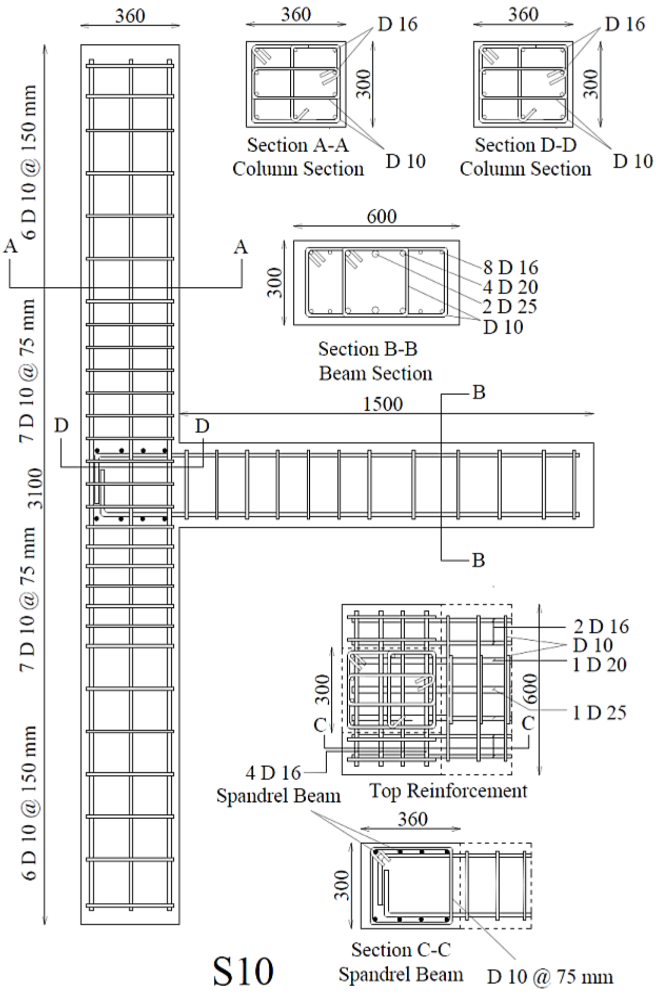


Fig. 2 Dimension and reinforcement details of S10

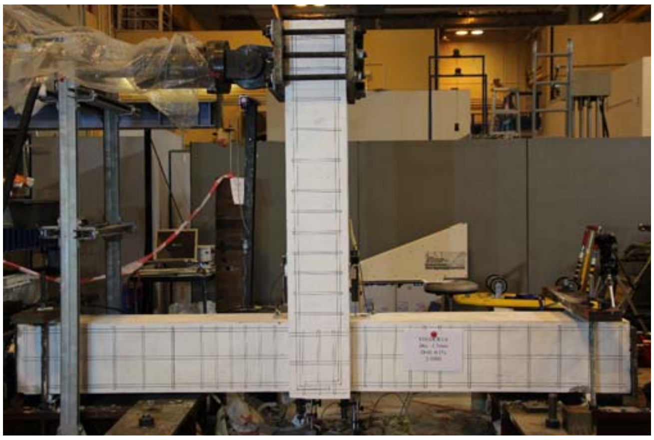


Fig. 3 Testing arrangement

The specimens were tested at the Structural Lab of the Hong Kong University of Science and Technology. The specimens were rotated 90 degrees, with the beam being vertical and column being horizontal, for the convenience of testing as shown in Fig. 3. Free ends of both column and beam are pinned supported to simulate the inflection points at the mid length of original members. In addition, the column was subjected to a 500kN axial load by a hydraulic jack which is approximately 15% of column squash capacity and is considered to be a reasonable value in practice [20]. Finally, the reversed cyclic loading was provided by a servo-controlled actuator at the end of the beam and the loading scheme suggested by ACI committee is shown in Fig. 4 [21].

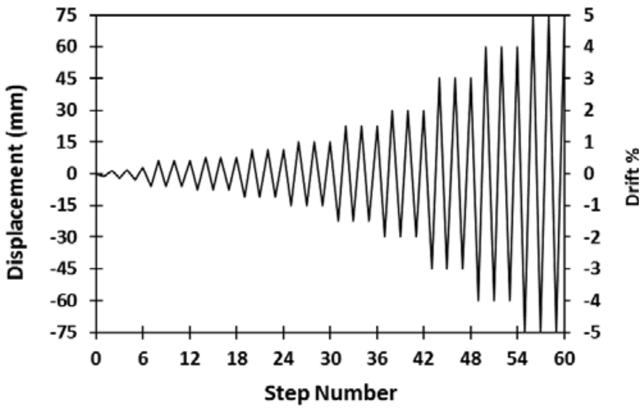


Fig. 4 Loading scheme

IV. Results and Analysis

A. Hysteresis Behaviour

Fig. 5 and Fig. 6 represent the hysteresis curve of S9 and S10, respectively. The dashed lines are the maximum lateral loads of actuator deduced from the beam flexural capacities. It is indicated that both specimens reached their expected beam flexural capacity and no premature joint shear failure occurred. The peak load of S9 did not drop after attaining its maximum load at 1.5% drift and the shape of the curve was full, revealing a beam flexural failure mechanism. S10 reached its maximum strength at 3% drift and the peak loads of the subsequent drifts dropped in both loading directions. It can be explained that the beam flexural failure dominated at small drift followed by post peak joint shear failure. Pinching of the hysteresis curve was visible of S10 due to the occurrence of joint diagonal shear cracks, which limited the total energy dissipation capacity.

B. Cracking Pattern

The cracking patterns of both specimens at loading levels of 0.75%, 2% and 5% drift ratios are presented in Fig. 7 and Fig. 8. Beam flexural cracks dominated in S9 and finally form full-width plastic hinges. It should be pointed out that no diagonal joint shear crack was observed at the joint face and only slight diagonal cracks formed at the outer faces of the spandrel beam after 1% drift. In addition, few cracks formed after 2% drift and only concrete crushing became severer at plastic hinges region. It can be concluded that S9 was under a typical beam flexural failure mechanisms in spite of its shear strength ratio of 1.0.

Beam flexural cracks were developed in S10, but the cracks at the outer faces of spandrel beams were much more severe those that in S9. In addition, the joint diagonal shear cracks occurred at the joint face at 2% drift and propagated in the following cycles, which was an indication of joint shear failure.

Moreover, such cracks only propagated in the joint core region and did not extend to the spandrel beam from the bottom view.

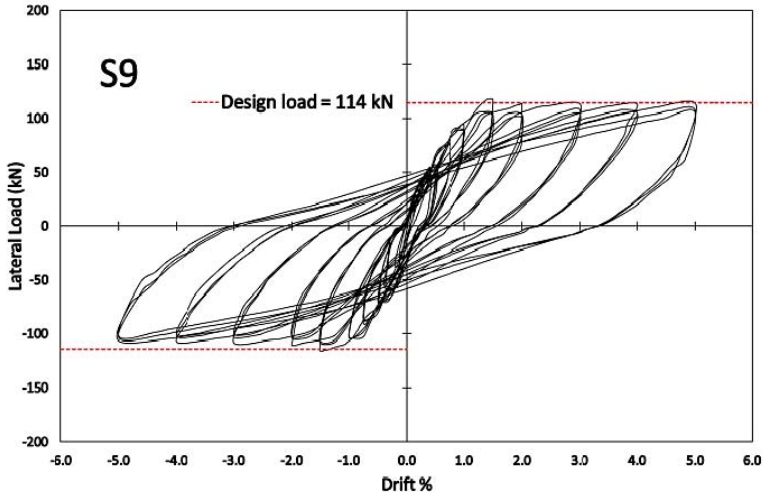


Fig. 5 Hysteresis loop of S9

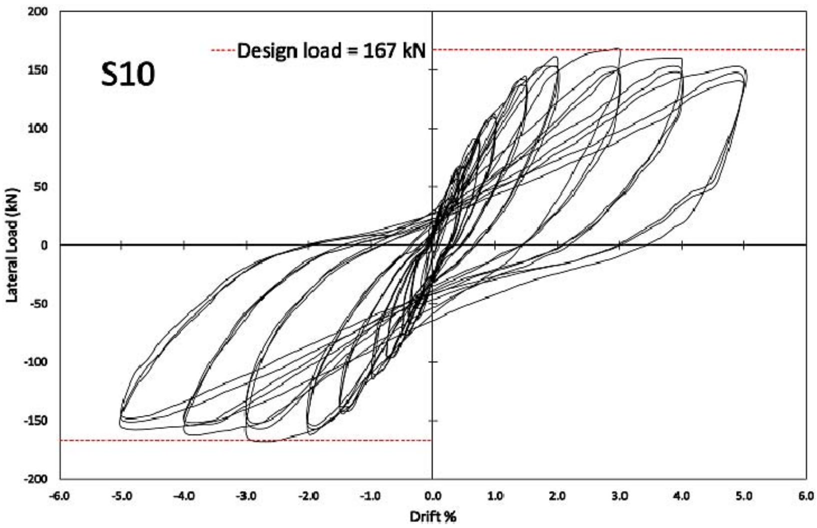
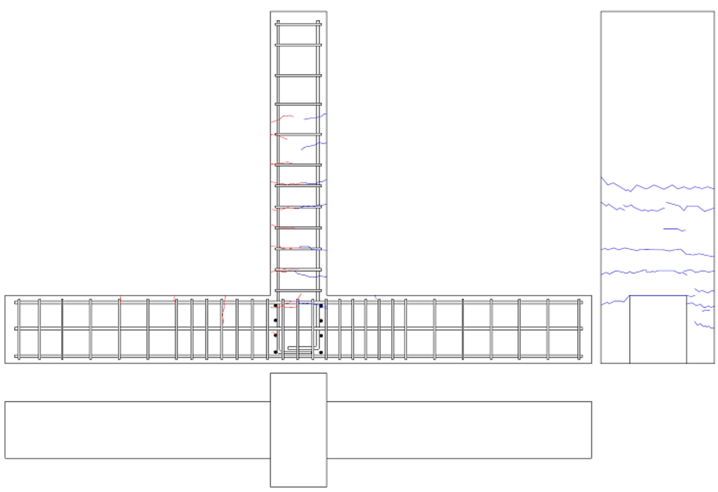
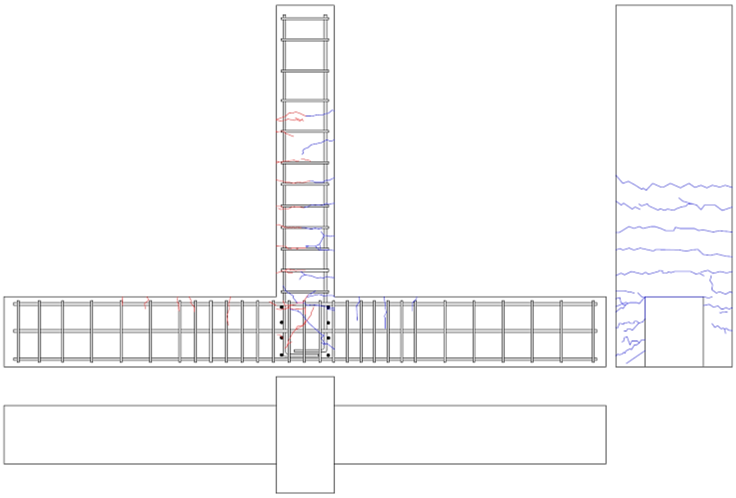


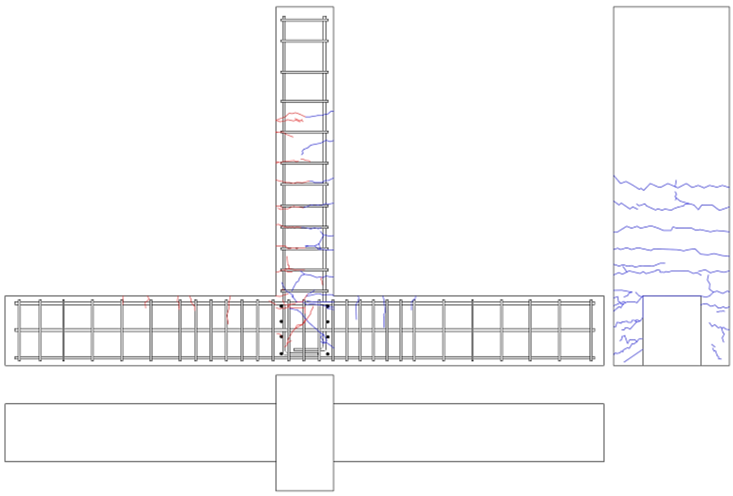
Fig. 6 Hysteresis loop of S10



(a)

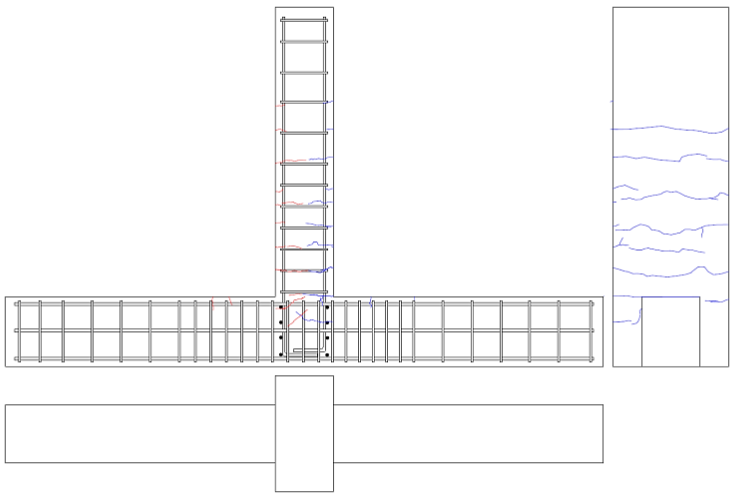


(b)

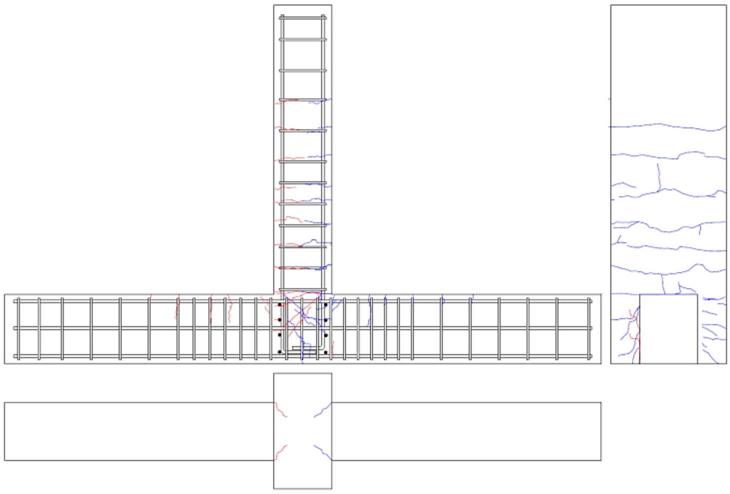


(c)

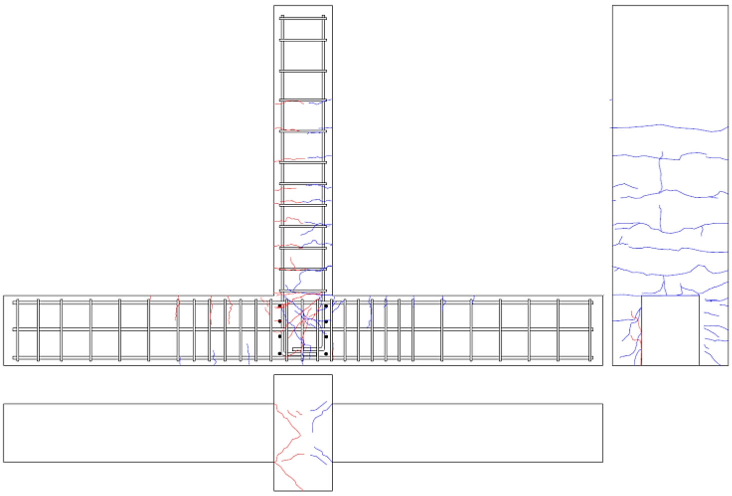
Fig. 7 Cracking pattern of S9 at different loading levels (a) Loading level at 0.75% drift (b) Loading level at 2% drift (c) Loading level at 5% drift



(a)



(b)



(c)

Fig. 8 Cracking pattern of S10 at different loading levels (a) Loading level at 0.75% drift (b) Loading level at 2% drift (c) Loading level at 5% drift

It can be explained by the larger beam reinforcement ratio in the joint core than the outer joint portion. Therefore, more forces were transmitted directly into the joint core and the torque at spandrel beam was reduced accordingly. No premature joint shear failure was found in both specimens revealing that none of the specimens reached their ultimate joint shear strength which was inconsistent to ACI's prediction.

C. Design Recommendations

The inconsistency between the experimental results and ACI's prediction can be explained by the definition of effective joint width b_j. Such value defined by ACI is always equal to the column width b_c for wide beam-column joints which is too conservative. It is suggested that such value be increased, as shown in (7) [15]:

$$b_j = b_c + 0.5h_c \tag{7}$$

The reason is that not only the joint core, but also some of the outside beam portion participates in resisting the joint shear, especially when the spandrel beam is designed to resist the torque, resulted from the beam reinforcement outside the joint core. Table II compares the design shear force to nominal shear strength ratio between codes of practice and proposed recommendations. It can be found that the current requirements of ACI codes and the New Zealand code are over conservative, while the EC8 and the proposed one give reasonably conservative values, based on the experimental result that premature joint shear failure did not occur in S10. Although the current EC8 and New Zealand code are more reasonable than the current ACI codes in predicting the ultimate joint shear strength of wide beam-column joint, they are not suggested for designing, as those codes do not require the torsional detailing for spandrel beam which may lead to spandrel beam torsional failure and joint shear failure at early stage.

TABLE II
NOMINAL SHEAR STRENGTH TO DESIGN SHEAR FORCE RATIO

Specimen	S9	S10
$V_{u,ACI} / V_{n,ACI}$	1.0	1.7
$V_{u,ACI} / V_{n,EC}$	0.6	1.0
$V_{u,ACI} / V_{n,NZS}$	0.7	1.2
$V_{u,ACI} / V_{n,proposed}$	0.6	1.0

V. CONCLUSIONS

This paper has presented the preliminary results of the test on two full-scale wide beam-column joints with shear strength ratios of 1.0 and 1.7, aiming at investigating the beam longitudinal reinforcement ratio to the overall seismic behaviour and joint shear strength. The results have been compared with the predictions of current seismic codes and the following conclusions can be drawn.

1) Specimen S9 with shear strength ratio of 1.0 exhibited beam flexural failure as full-width plastic hinges were developed at the beam end. No sign of joint shear failure or spandrel beam torsional failure was observed. In contrast, specimen S10 with 1.7 shear strength ratio failed in beam flexural failure followed by post-peak joint shear failure.
2) Eurocode provides a more reasonable prediction of the shear strength of wide beam-column joints while ACI codes and New Zealand code are more than conservative.
3) A modification of the current ACI codes is proposed to provide a more accurate prediction to the shear strength of wide beam-column joints.

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