Behaviour of non-seismically detailed beam-column joints under simulated seismic loading: A critical review

J S KUANG
Associate Professor
Hong Kong University of Science & Technology
Hong Kong

Ho Fai WONG
PhD Student
Hong Kong University of Science & Technology
Hong Kong

Summary

Experimental investigation is presented in this paper on the seismic behaviour of six reinforced concrete beam-column joints with non-seismic reinforcement detailing. The main variables are the amount of transverse reinforcement, axial load ratio and aspect ratio of the joints. Most of the test specimens show satisfactory results in terms of lateral load capacity and displacement ductility. It has been demonstrated that limited ductility level can be reached even it is “empty” within the joint region and addition of limited amount of transverse reinforcement can significantly improve the seismic performance. Axial load ratio does improve the inherent ductility of “empty” joints but no effect on those with transverse reinforcement. The experimental results are compared with the provisions of BS8110 and EC2 in order to evaluate the validity of them in predicting the shear strength of beam-column joints belong to this category.

Keywords: Seismic behaviour ; Non-seismically detailed; Reinforced concrete beam-column joints ; Transverse reinforcement ; Axial load ratio ; Aspect ratio ; Limited ductility level ; Shear Strength

1. Introduction

Presently, building codes in many regions of low to moderate seismicity require only limited or without any consideration for seismic resistance. The staggering numbers of existing structures built in these zones have been designed without any compliance to up-to-date earthquake resistance requirements. The reinforcement in these buildings is adequate for gravity and wind loads. Nevertheless, many of the construction details are felt to be inappropriate for safe transmission of seismically induced inertia forces to the ground [1]. The inherent shortcomings of gravity load design philosophy imply high susceptibility of the frame structures to anticipated seismic risk.

An improved understanding of seismic behaviour of structures with non-seismic reinforcement details is essential to evaluate the existing buildings, which may require retrofitting if necessary, and modify the current codes to achieve adequate seismic performance without resorting to a full seismic design. However, there is a lack of experimental information in this broad class of structures.

This paper aims at providing experimental evident to this category. Evaluation of present codes of practices to the corresponding performance is also demonstrated.

2. Experimental Study

In the experimental study, a total of six full-scale exterior beam-column joints are tested under simulated seismic loading. The design variables include the amount of joint transverse reinforcement, column axial load ratio and aspect ratio of the joints.

2.1. Test Specimens

The length of beam and column for all specimens are 1.7m and 3.1m respectively and they consist of the same column section with dimension of 300mm x 300mm. Two different beam sections are investigated – Type A (400mm) and Type B (300mm). To investigate the effect of moderate amount of transverse reinforcement, two stirrups with diameter of 10mm (A_s = 157 mm^2) are installed within the joint core, which is about 22% (for Type A) and 28% (for Type B) percent respectively of those required by ACI-318-02 [2]. The typical reinforcement details of specimens are shown in Fig.1.

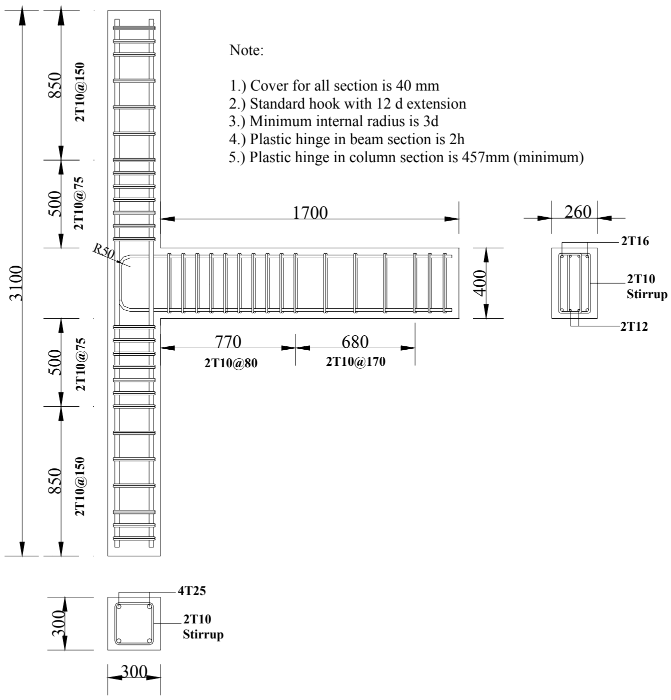

Fig. 1 Typical reinforcement detailing of specimens

Two different axial load ratios are applied and they are chosen as 3% and 15% of the column compression capacity. The higher compression force simulates the situation in real buildings [3] while the lower one acts as a comparison and it can guard the specimen in the test-rid. All specimens are named under the same nomenclature system. The first part is corresponding to the depth of the joint (type A or B). In the middle part, NN, NY means "empty" and reinforced with links respectively. While the final part describes the axial load ratio applied to the column.

For all the specimens, beams and columns are detailed according to ACI-318-02 [2] so that joints become a weak zone among the subassemblage. All reinforcement details are drawn on the surface of specimens for clear indication.

2.2. Experimental Setup and Loading Arrangement

The experimental setup and loading system are shown in Fig. 2. The specimen is rotated 90 degrees such that the column member is in the horizontal position and the beam member is in the vertical position. Proper boundary conditions are given in the test setup to simulate the actual stress inside the test specimens as if it is a part of the frame structure.

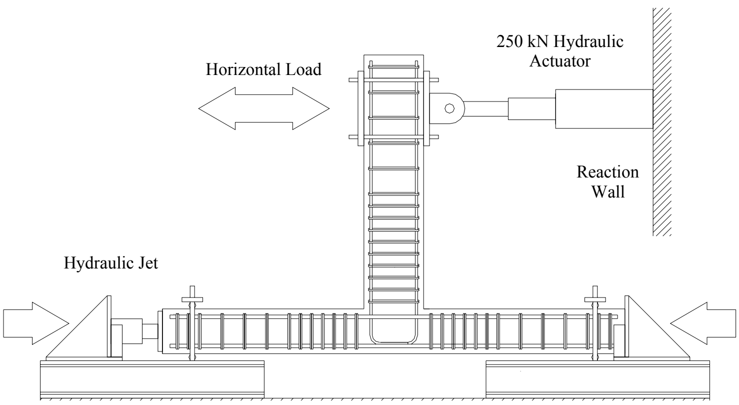

Fig. 2 Test setup for reversal cyclic loading

A 250 kN hydraulic actuator is used to apply cyclic reversal loading at the beam end. Rollers are provided near the ends of the columns to simulate inflection points in the structure and axial load is added to the column through the 1000 kN hydraulic jack located at the steel bearing.

The cyclic loading applied to all specimens is shown in Fig. 3. In the test, both load control and displacement control are utilized at different stages. Two cycles of horizontal loading up to ± 0.5P_i and ± 0.75P_i are initially applied; the load P_i is the horizontal load at the top of the beam associated with the theoretical flexural strength M_i of the beam, which is reached in the critical section of the members

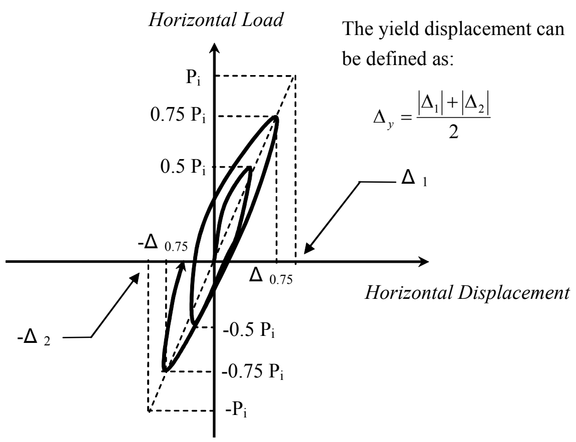

Fig. 3 Definition of yield displacement

and calculated using the conventional compressive stress block for the concrete with an extreme fiber concrete compressive strain of 0.0035.

The yield displacement Δy is extrapolated linearly to Pi based on the stiffness at the inter-story horizontal displacement when the lateral load is ± 0.75Pi (Fig. 3). The cyclic loading applied in the inelastic range is displacement controlled. The test specimens are subjected to two cycles of loading to μ = ± 1, ± 2, ± 3…. where μ is the displacement ductility factor defined as Δ/Δy [4] .

2.3. Experimental Results

The measured horizontal force versus horizontal displacement hysteresis loops and the crack pattern of the specimens at final stage are shown in Fig. 4 and Fig. 5. The experimental results are summarized in Table 1.

2.4. Discussion of Experimental Results

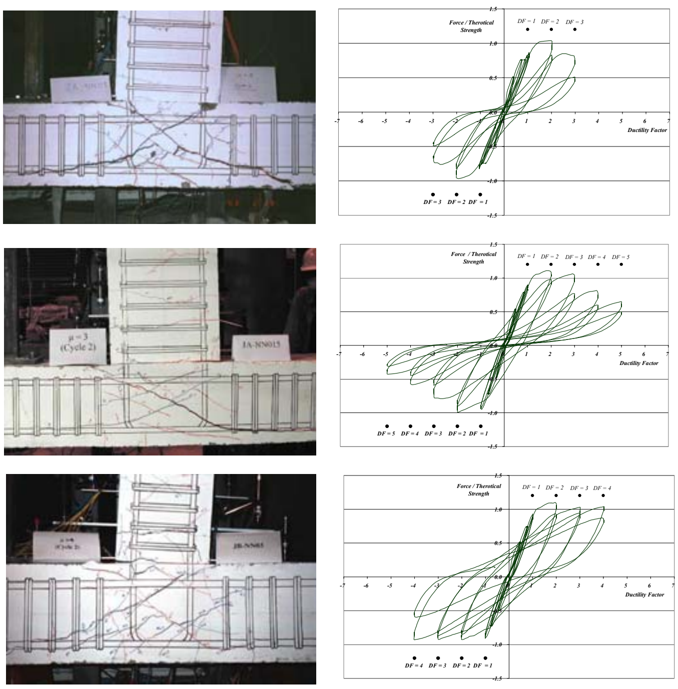

Fig. 4 Crack pattern and hysteresis loops of specimens JA-NN03, JA-NN15 and JB-NN03

In general, all joints failed in shear type failure and two failure modes can be observed: Joint Shear Failure (JS) and Beam Yield – Joint Shear Failure (BY-JS). The failure mode depends on the shear strength of the beam-column joints. For the specimens without joint transverse reinforcements (JA-NN03, JA-NN15 and JB-NN03), it is observed that shear failure of the specimens is due to extensive tension cracking instead of compression failure of joint concrete. Similar failure criterion is also suggested by Calvi and Priestley [5]. However, for JA-NY03, JA-NY15 and JB-NY03, compression failure of concrete is observed. Both diagonal concrete strut and the truss mechanism [6] participate in resisting the joint shear force. It is possible that the presence of transverse reinforcement results in diagonal compression failure of joint while diagonal tension failure occurs when there is not any shear reinforcement. The failure of concrete in tension is similar to the case of a concrete cylinder under splitting test which can resist less compression force.

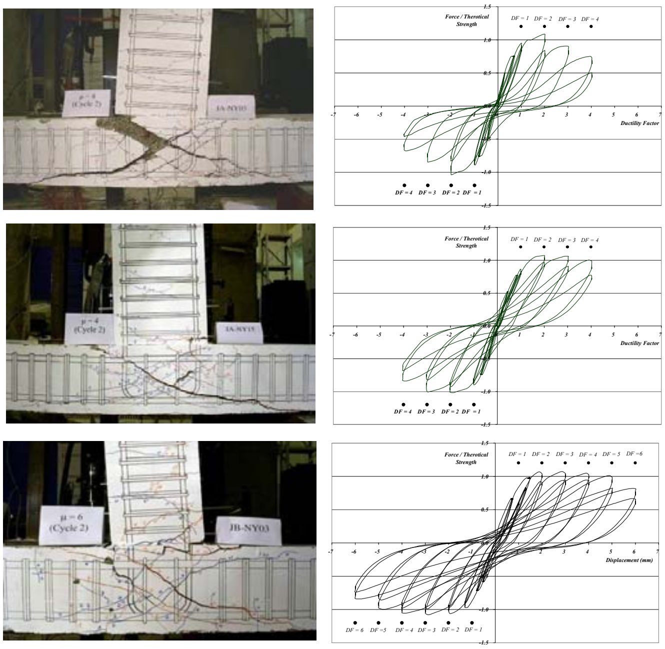

Fig. 5 Crack pattern and hysteresis loops of specimens JA-NY03, JA-NY15 and JB-NY03

Table 1 Summary of experimental results

Specimen	Failure Mode	Ductility Factor	Max. Normalized Shear Stress ($\sqrt{f_c'}$)
JA-NN03	JS	2	0.53
JA-NN15	JS	3	0.56
JB-NN03	BY-JS	4	0.56
JA-NY03	BY-JS	4	0.59
JA-NY15	BY-JS	4	0.59
JB-NY03	BY-JS	6	0.62

From table 1, it can be observed that limited amount of transverse reinforcement (around 25% of ACI requirement) can greatly improve the ductile behaviour of “empty” joints and it is independent of other variables. For the effect of aspect ratio, we can observe that ductility factor is increased from 2 to 4 (JA-NN03 and JB-NN03) and from 4 to 6 (JA-NN03 and JB-NN03). The aspect ratio (beam depth – column depth ratio) is obviously influential to ductile behaviour of beam-column joints. It is believed that this mechanism is analogy to the case of deep beam failure mode. The aspect ratio in joints is similar to shear span – depth ratio of deep beam. The higher inherent shear strength can withstand greater ductility development. However, for the axial load ratio, it is observed that both maximum normalized shear stress and ductility factor of JA-NY03 and JA-NY15 are unaffected. It does increase the ductility and shear strength of those without any transverse reinforcement (JA-NN03 and JA-NN15). The increment of shear strength may be explained in terms of the following equations:-

From Mohr circle, the principal tensile stress (f_t) in concrete can be expressed in terms of shear stress (v_{jh}) and the applied compressive stress ($f_c = N/A_j$) in column:

$$f_t = \frac{-f_c}{2} + \sqrt{\left(\frac{f_c}{2}\right)^2 + v_{jh}^2} \tag{1}$$

By rearranging the terms and assume that tensile strength of concrete: $f_t = k\sqrt{f_c'}$

$$v_{jh} = k\sqrt{f_c'}\sqrt{1 + \frac{N}{k\sqrt{f_c'}A_j}} \tag{2}$$

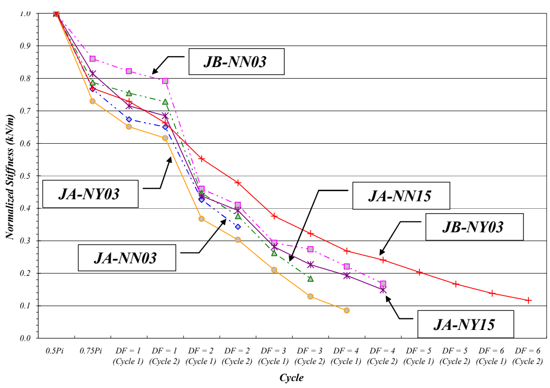

Fig. 6 Stiffness degradation of all specimens

The shear strength of joints increases with axial load provided that the joints fail in tension splitting, which is actually the observed failure mode for “empty” joints, instead of compression failure.

The normalized stiffness of all specimens at different stages is shown in Fig. 6. The values in the figure are all normalized by the initial stiffness at the stage of 0.5P$_i$ for ease of comparison. It can be seen that stiffness degradation occurs when reversal loading is applied. The change of variables is found to have negligible effects on the stiffness degradation of all specimens and they almost degrade at the same rate during the entire test.

(a) *(b)*

Fig. 7 Damage of at the back of Beam-Column Joints: (a) JA-NY03; and (b) JA-NN03

It is observed that there is serious cracking and spalling of cover concrete at the back of the joint (refer to Fig. 7(b)). However, limited amount of transverse reinforcement improve the situation and there are nearly no damage at the same location (refer to Fig. 7(a)). The cracking and spalling of cover concrete at the back is due to dilation of joint after diagonal cracking and the opening of hooks under the action of pulling force from reversal loading acting on the beam [7]. It is believed that longer anchorage length and additional of slight amount of transverse reinforcement can eliminate the damage.

3. Comparison of Experimental Results to BS8110 and EC2

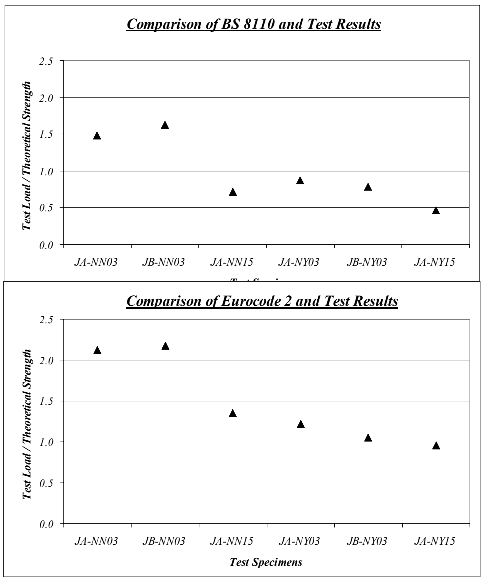

Fig. 8 Comparison of experimental shear strength to prediction in BS8110 and EC2

Comparisons are made between the experimental shear strength of the joint specimens and the value obtained by BS8110 [8] and EC2 [9] as shown in Fig. 8. The purpose of comparison is to evaluate those two codes for prediction of shear strength of beam-column joints with non-seismic and limited seismic reinforcement details.

For non-seismic region, there is not special provision for joint design. Alternatively, the beam-column joint is defined as that portion of the column within the depth of the beams[5]. The corresponding shear strength of joint may be estimated by the provision for column design. From BS8110 [8], the shear strength of a member with axial load is given as:

$$V = \left[v_c' + 1.25 \times 0.6 \frac{N}{A_j} \left(\frac{Vh}{M} \right) \right] bd + A_{jh} f_y \left(\frac{d}{s_v} \right) \tag{3}$$

Where v_c' is shear strength of concrete; N is axial load on columns; Vh/M is taken to be not greater than 1; A_{jh} is the area of joint shear reinforcement.

For Eurocode 2 [9], the shear strength of column is given as follows:

$$V_{R1} = \left[\tau_R k \quad (1.2 + 40\rho_1) + 1.5 \times 0.15 \quad \sigma_{cp} \right] \; bd + A_{jh} f_y (0.9\, d/s) \tag{4}$$

but not greater than $V_{R2} = 0.45 v f_c' bd$ (5)

Where τ_R is shear strength of concrete; ρ_1 is Tension steel ratio on the column; σ_{cp} is longitudinal stress in the column ($= N/A_c$); $\upsilon = 0.7 - f_c'/200 \geq 0.5$.

All of the safety factors have been removed in these provisions in order to make comparison with experimental results. It is observed that both BS 8110 and EC2 underestimate the shear strength of beam-column joints which fail in Joint Shear Failure Mode (JS). Also, they cannot predict the maximum shear stress attainable in beam-column joints with Beam Yield-Joint Shear Failure Mode (BY-JS).

4. Discussion and Conclusion

It is indicated by Park [10] that structures with limited ductility factor (DF = 3) is adequate for regions of low to moderate seismicity. It is demonstrated that limited ductility can be reached even there is no transverse reinforcement for joints under some conditions. Moderate amount of transverse reinforcement, around 25% of ACI requirement in this study, seems to be good enough to guarantee satisfactory performance of beam-column joints for regions of low to moderate seismicity.

Six full-scale exterior beam-column joints with non-seismic and limited seismic reinforcement details are constructed and tested. Based on the experimental investigation and code comparison, the following conclusions are drawn:

1. Moderate amount of transverse reinforcements significantly improve the behaviour of joints in term of strength, ductility and cracking pattern. All specimens with limited shear reinforcement reach to the level of limited ductility.
2. Higher axial load ratio in column is able to increase the ductility factor of joints without shear reinforcement but not obvious for those transverse reinforcement are installed. The possible reason may be due to different failure mechanism at joint (tension or compression failure).
3. Aspect ratio of joint is influential to the shear strength and ductility. The mechanism is analogy to the shear span – depth ratio in deep beam. With small aspect ratio, the joint can reach to D.F. = 4 even there is no transverse reinforcement. This phenomenon is true for all joints no matter which is with or without transverse reinforcement.
4. It shows that BS8110 and EC2 underestimate the shear strength of beam-column joints and further development is required for prediction of shear strength at this critical region.

5. References

[1] Joint ACI-ASCE Committee, *Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-91)*, American Concrete Institute, Farmington Hills, Mich., 1997.

[2] American Concrete Institute. *Building Code Requirements for Reinforced Concrete (ACI 318-02) and Commentary - ACI 318R-02*, Detroit, Michigan, 2002.

[3] HUI, Y. and IRAWAN, P., "Behaviour of Full-Scale Lightly Reinforced Concrete Beam-column Joints", *Civil Engineering Research*, Nanyang University, Singapore, January, 2001, pp. 44 – 45.

[4] PARK. R., "Evaluation of Ductility of Structures and Structural Assemblages form Laboratory testing", *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 22, No.3, 1989, pp. 155 – 166.

[5] PRIESTLEY, M.J.N. and CALVI, G.M., "Toward a Capacity-design Assessment Procedure for Reinforced Concrete Frames", Earthquake Spectra, Vol. 7, No. 3, 1991, pp. 413-437.

[6] PAULAY, T. and PRIESTLEY, M.J.N., *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley and Sons, 1992.

[7] Comite Euro-International Du Beton, *Seismic Design of Reinforced Concrete Structures for Controlled Inelastic Response*, Thomas Telford Ltd, 1997, 174 pp.

[8] BS 8110. *Structural use of concrete*, London, British Standards Institution, 1997.

[9] EC2. *Design of Concrete Structure: Part I, General rules and rules for building*, DD ENV 1992-1-1, London, British Standards Institution, 1992.

[10] PARK. R., "Design Procedures for Achieving Ductile Behaviour of Reinforced Concrete Buildings", *Proceedings of International Workshop on Earthquake for Regions of Moderate Seismicity*, Hong Kong, 1998, pp. 45-50.