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Design considerations for interior RC beam–column joint with additional bars

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ABSTRACT

The seismic performance of beam–column connections and joints can be enhanced by using improved details in the joint and ensuring a strong connection behavior. However, specific provision to evaluate quantitatively the increased bar–concrete bond and shear strengths in the beam–column joint is not given in current design codes. In the present study, cyclic loading tests were performed for beam–column connections of which joint detailing was improved by placing additional 45° bent bars and 90° hooked bars. The test results showed that despite small h_c/d_b values, by using the improved details, bond–slip of beam flexural bars and diagonal cracking were substantially decreased in the joint. To address the enhanced performance in the design of beam-column joints, the bond resistance and joint shear strength were quantitatively redefined considering the details of the additional bars. In addition, the relationship between the bond parameter and energy dissipation capacity was proposed. On the basis of the results, considerations for the seismic design and detailing of beam–column joints with additional bars were recommended.

1. Introduction

In reinforced concrete moment-resisting frames, the overall earthquake resistance including stiffness, strength, deformation capacity, and energy dissipation capacity is significantly affected by the beam–column joints. Under cyclic loading, when yielding of the beam flexural bars penetrates into the joint panel and diagonal cracking occurs (Fig. 1), significant bond slip and shear strength degradation occur in the joint [1–6].

To mitigate bar bond–slip in the joint, current earthquake design codes specify the minimum requirement of column depth-to-bar diameter ratio (h_c/d_b): ACI 318-11 [7] and ACI 352R-02 [8] require $h_c/d_b > 20$ and $h_c/d_b > 20f_y/420$, respectively. However, previous test results showed that even when the minimum requirement was satisfied, significant bond slip and shear deformation occurred in the beam–column joints [1–6]. Thus, to secure the structural performance of beam–column joints, NZS 3101:2006 [9] and Eurocode 8 [10] require greater development lengths for the beam flexural bars. However, such large development length frequently causes difficulty in architectural design of low-rise buildings where small columns or shallow depth columns (i.e. wall-columns) are preferred.

Alternatively, additional bars can be used in the joint, to enhance the bond resistance and shear strength [11–18]. Galunic et al. [11] compared the effects of straight bars and 60° bent bars to strengthen the beam–column joints. The bent bars showed better performance in reducing joint shear distortion. Park and Milburn [12] and Joh et al. [13] used 90° hooked bars to relocate the beam plastic hinge zone (i.e. the critical section) to a distance of 0.5–1.0 times the beam depth from the column face. Abdel-Fattah and Wight [14] used additional straight bars in the joint, to reduce yield penetration of the beam flexural bars. Fenwick and Irvine [15] used bond plates welded to the beam and column bars. Yamamoto et al. [16] used U-shaped bars to relocate the slab plastic hinge zone to a distance of 1.8 times the slab depth from the wall face. Pimanmas and Chaimahawan [17] and Shafaei et al. [18] used planar joint enlargement to relocate the beam plastic hinge zone to the edge of enlargement and to reduce shear stress in the joint.

The existing studies showed that the improved details using additional bars efficiently reduced the bond–slip of beam flexural bars and enhanced the shear strength of the beam–column joints. However, current design codes [7,9,10] do not give a detailed provision for the quantitative evaluation of the enhanced bar-concrete

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Nomenclature

A_j	effective joint shear area (mm^2)
E_D	actual energy dissipation per load cycle (kN m)
E_{ep}	idealized elastic-perfectly plastic energy dissipation (kN m)
M_n	beam moment demand (kN m)
M_{nj}	beam moment capacity (kN m)
P_n	lateral load-carrying capacity (kN)
P_u	peak strength (kN)
V_c	shear demand of the column (kN)
V_{jn}	joint shear strength (kN)
V_{ju}	joint shear demand (kN)
b_b	width of beam cross section (mm)
b_c	width of column cross section (mm)
d_b	re-bar diameter (mm)
d_j	distance from the column face to the beam critical section (mm)
f'_c	concrete strength (MPa)
f_y	yield strength of re-bar (MPa)
h	net column height (mm)
h_c	column depth (mm)
h_s	distance between the top and bottom flexural bars of the beams (mm)
l	net beam length between the vertical supports (mm)
l_{dj}	modified development length (mm)
l_s	distance from the column face to the location of zero moment in the beam (mm)
k_s	secant stiffness (kN/mm)
k_y	yield stiffness (kN/mm)
x	smaller perpendicular distance from the longitudinal axis of the beam to the column side (mm)
α	coefficient addressing the effects of material over-strength and cyclic strain-hardening
β	coefficient addressing the joint shear strength enhanced by the use of the additional bars
δ_y	yield drift ratio (%)
δ_u	the maximum drift ratio (%)
γ	coefficient addressing the confinement effect by the beams framed into the joint
γ_j	joint shear deformation
κ	energy dissipation ratio

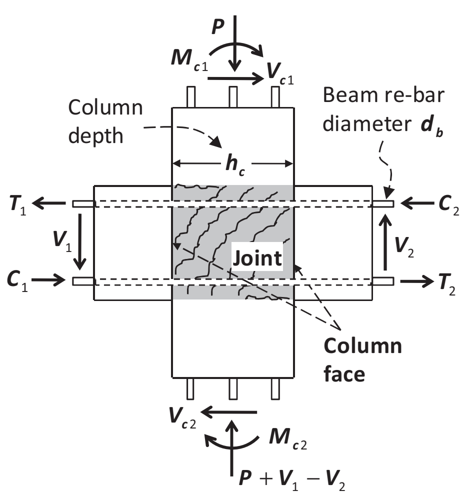

Fig. 1. Load transfer of interior beam–column joint.

bond and shear strengths in the strengthened beam-column joint. Thus, the following studies are required to expand the use of the improved details.

(1) Even with an identical improved joint detail, the structural performance of beam–column connections can significantly vary according to the design parameters. Thus, for performance-based design, the effects of the design parameters on the performance need to be quantified. For example, the variation of energy dissipation capacity depending on the detailing of the joint needs to be studied.
(2) Current design codes specify the detailing of additional bars to ensure a strong connection behavior. NZS 3101:2006 [9] requires that the length of the additional bars be extended to a distance not less than the beam depth or 500 mm from the column face. ACI 318-11 [7] Sec. 12 requires that the development length of the 90° hooked bars be not less than $8d_b$ and 150 mm. However, a large development length may cause inconvenience in the design and construction of additional bars. Thus, the validity of such requirements needs to be verified.
(3) Unless special design provisions are provided, the bond resistance and joint shear strength of beam–column joints with additional bars should satisfy the requirements of the current design codes for conventional beam–column connections. In this case, the requirements of the current design methods or the relevant design parameters need to be modified, considering the enhanced performance and details of the joints.

In the present study, cyclic loading tests for beam–column connections were performed. To verify the effects of the improved details and to examine the relevant code-requirements, the test specimens were intentionally designed with small h_c/d_b ratios less than 20, which is the minimum requirement of ACI 318-11 [7]. For the improved details, 45° bent bars and 90° hooked bars were used in the joints. On the basis of existing test results from the present study and previous studies, design recommendations were proposed for the bar bond- and joint shear-design of beam–column connections with the improved details.

2. Plastic hinge zone relocation design

Fig. 2 shows the concept of the joint improved details using 45° bent bars and 90° hooked bars [9,12,13]. The flexural capacity of the beams at the column face is increased by the additional bars (i.e. the 45° bent bars and 90° hooked bars) (see dotted lines in Fig. 2(a) and (b)). Therefore, the beam critical section and plastic hinge zone are relocated at the distance d_j where the moment capacity of the beam is the same as the demand (see Fig. 2(a) and (b)). Consequently, yielding of the beams does not significantly influence the behavior of the joint.

To prevent early yielding of beam flexural bars at the column face, the beam moment capacity M_{nj} at the column face should be greater than the demand which is developed by the moment capacity M_n at the critical section (Fig. 2):

$$M_{nj} > M_n\left(\frac{l_s}{l_s - d_j}\right) \text{ or } M'_{nj} > M'_n\left(\frac{l'_s}{l'_s - d'_j}\right) \quad (1)$$

where l_s = the distance from the column face to the location of zero moment in the beam; and d_j = the distance from the column face to