Database investigation on bond performance of interior beam-column joints with high-strength reinforcement

H.C. Chen

Graduate School of Engineering Science and Technology, National Yunlin University of Science and Technology, Yunlin, Taiwan.

H.J. Lee & T.C. Tsai

National Yunlin University of Science and Technology, Yunlin, Taiwan.

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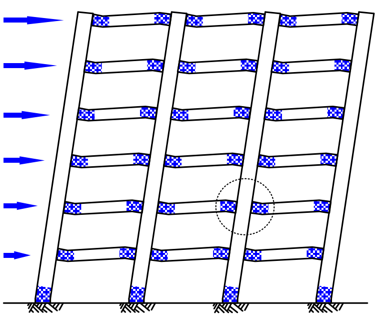
ABSTRACT: The use of high-strength reinforcement in concrete structures has many advantages such as labor and cost savings. Whenever higher strength reinforcement is used, the bond and development length becomes critical problems for design of concrete structures, especially for joints of moment-resisting frames. This paper reviewed existing design criteria of development length for the straight beam bars within beam-column joints and recommended to extend the bond requirements of NZS 3101 for the use of Grade 690 reinforcing bars. According to an extensive database investigation, the validity of the design equations of NZS 3101 is assessed by hysteresis performance of beam-column joint tests collected from laboratories in Unites States, Japan, New Zealand, and Taiwan. Practical design recommendations are drawn for Grade 690 reinforcing bars being use as longitudinal reinforcement passing through joints of moment-resisting frame.

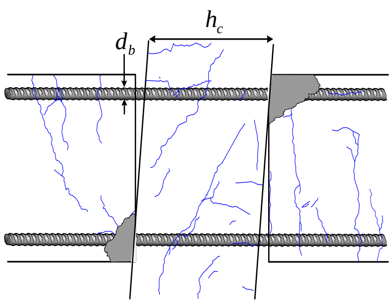
1 INTRODUCTION

Special moment-resisting frames are widely used for the design of reinforced concrete building structures in moderate to high seismic zones. If properly detailed, the plastic hinge can be arranged to develop at the beam regions adjacent to the joint when the frame subjected to large lateral loads, as shown in Figure 1(a). During the formation of these beam plastic hinges, extremely high bond stresses can be developed along the straight beam bars passing through the joint, because these bars may be forced to yield in tension at one column face and be close to yield in compression at the opposite column face. Once certain degree of bond deterioration occurred within the joint, these beam bars may slip within the joint under large load reversals.

Significant bond slip is not desirable because it reduces the stiffness and energy dissipation capacity of beam-column connections. Some bond deterioration is inevitable and should be accepted. However, if the bond deterioration is severe, the bar tension will penetrate through the joint and develop in the beam compression zone on the opposite side. This means that both top and bottom beam bars are in tension at the column face and then large compression forces will transfer to the concrete of the beam compression zone. As shown in Figure 1(b), concrete crushing at beam ends may occur consequently and followed by significant reduction on beam flexural strength and ductility. Hakuto et al. (1999) ever demonstrated the detrimental effect of bond deterioration by analytical studies and concluded that bond deterioration should be considered in the design of beam-column joints.



(a) Beam hinging mechanism



(b) Bond slip and concrete crushing at joint faces

Figure 1. Bond failure at interior beam-column joints for earthquake resistance.

Although bond performance of beam bars passing through interior joints have been extensively studied since 1980s, the development length requirements for beam-column joints still differ remarkably among the current ACI 318 Code (ACI 318 2014) in United States, the AIJ Guideline (AIJ 1999) in Japan, and the NZS 3101 Standard (NZS 3101 2006) in New Zealand. For seismic design of interior beam-column joints in special moment-resisting frames with normal weight concrete, the ACI 318 Code requires a minimum column dimension of 20 times the largest diameter of beam bars parallel to that column dimension. This criterion was based on an evaluation of available tests in 1980s. Zhu and Jirsa (1983) reviewed cyclic loading response of 18 interior beam-column joints with normal strength concrete and reinforcement. While ACI 318 Code set a simple criterion of 20-bar diameters, both the AIJ Guideline and the NZS 3101 Standard establish the minimum ratios of column dimension to beam bar diameter as a function of material strengths and the column axial stress. The philosophy behinds these requirements are based on elaborate studies on the energy dissipation capacities of beam-column joints in Japan and New Zealand.

High-strength concrete has been used in many building structures in Japan (Aoyama 2001), particularly, for columns with limited architectural dimensions and high axial load at the lower levels. Whenever high-strength reinforcement is used for beam longitudinal reinforcement passing through a beam–column joint, either a large column depth or a small permissible diameter of beam bars would make design and proportion difficult. To provide a promising solution, this paper compares existing bond requirements in international concrete design codes and then validates proper design equations using a large database of beam–column joint tests. Laboratory testing performances such as strength, stiffness, and energy dissipation capacity of each beam–column joint specimen are evaluated according to ACI standards (ACI 374 2005) for special moment frames. Finally, a viable set of design equations for the development length in beam–column joints is recommended to achieve acceptable bond performance for special moment frames.

2 EXISTING DESIGN CRITERIA

2.1 Genetic formula

For the use of Grade 500E reinforcement in New Zealand, Brooke and Ingham (2013) reviewed existing design criteria for the reinforcement anchorage length at interior beam-column joints. During the formation of the adjacent beam hinging, the stresses on the beam bar may achieve $\alpha_o f_y$ in tension at one face of the joint and $\kappa\alpha_o f_y$ in compression at the opposite face of the joint, as shown in Figure 2.

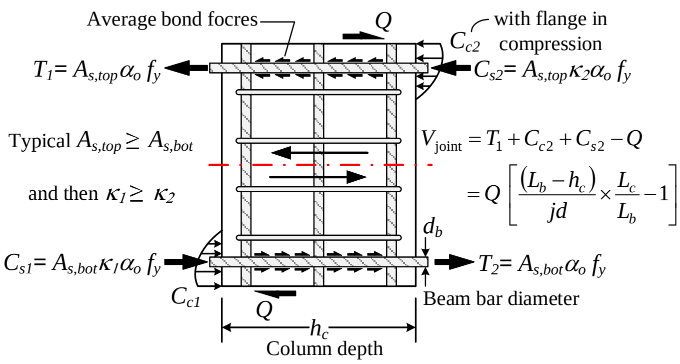


Figure 2. Horizontal shear and bond forces acting on the joint concrete.

By assuming an average bond stress on the beam bar along the column depth, the bond requirements for preventing excessive bond slip of beam bars in joints are given as follows.

$$\pi d_b h_c \alpha_p u_b \geq \frac{\pi d_b^2}{4}\alpha_o f_y (1+\kappa) \quad \text{or} \tag{1a}$$

$$\frac{h_c}{d_b} \geq \frac{(1+\kappa)}{4}\frac{\alpha_o f_y}{\alpha_p u_b} \tag{1b}$$

where h_c = column depth; d_b= maximum bar diameter of the beam bars passing through the joint; f_y=is specified yield strength of the reinforcement; u_b= average bond stress on the beam bar in the joint; α_p= factor accounting for the benefit effect of column axial compression on bond strength; α_o= overstrength factor of the beam bars; and κ= ratio of bar compressive stress to the bar tensile stress.

2.2 Existing design equations

Bond requirements on the basis of Eq. (1) can be found in AIJ Guideline (AIJ 1999) and NZS 3101 (NZS 2006). Table 1 compares above design criteria with the design recommendations proposed by Brooke and Ingham (2013) and this paper. Among the existing design criteria, differences can be found for the $(1+\kappa)$ term of bar stress being developed in the joint, average bond strength u_b along the bar, and the factor α_p of the column axial stress on bond strength. Originally, NZS 3101 uses an average bond strength of $1.5\sqrt{f_c'}$ MPa, which is 60% of the peak local bond strength of $2.5\sqrt{f_c'}$ MPa observed by Eligehausen et al. (1983), and two additional modification factors, α_f and α_t, to consider the bidirectional loading and the top bar effects, respectively. Paulay and Priestley (1992) described detail development of above α factors.

Recently, Brooke and Ingham (2013) assembled a database of 93 interior beam-column joint tests to assess the suitability of existing design criteria for the bond development length in joints and concluded that the existing criteria cannot reflect the bond failure observed in experiments. They proposed to modify the basic bond strength from $1.5\sqrt{f_c'}$ to $1.25\sqrt{f_c'}$ MPa and the corresponding equations of κ and α_p for updating NZS 3101, as shown in Table 1. The α_p factor proposed by Brooke and Ingham (2013) is relatively conservative with a upper limitation of 1.20 for high axial load conditions. Following prior investigation, this paper recommends to omit the α_f and α_t terms and set the bond strength $u_b=1.5\sqrt{f_c'}$, which are demonstrated with satisfactory bond performance in laboratory testing.

Table 1. Comparison of existing design equations for the development length in interior joints.

Design criteria	Bar stress factor $\mathbf{1+\kappa}$	Bond strength $\boldsymbol{u_b}$ (MPa)	Axial stress factor $\boldsymbol{\alpha_p \geq 1.0}$
AIJ 1999	$1+\frac{A_{s,bot}}{A_s}$	$0.7(f_c')^{2/3}$	$1+\frac{P}{A_g f_c'}$
NZS 3101	$1+1.55-\frac{A_s}{A_{s,top}} \leq 1.8$	$\alpha_f \alpha_t 1.5\sqrt{f_c'}$	$0.95+0.5\frac{P}{A_g f_c'} \leq 1.25$
Brooke & Ingham 2014	$1+\frac{0.7}{\alpha_o}\frac{A_{s,top}}{A_s} \leq 1+\frac{1}{\alpha_o}$	$\alpha_f \alpha_t 1.25\sqrt{f_c'}$	$0.9+2.0\frac{P}{A_g f_c'} \leq 1.20$
Recommended	$1+\frac{0.7}{\alpha_o}\frac{A_{s,top}}{A_s} \leq 1+\frac{1}{\alpha_o}$	$1.5\sqrt{f_c'}$	$0.9+2.0\frac{P}{A_g f_c'} \leq 1.20$

Note: With limitation of $A_{s,top} \geq A_{s,bot}$, where $A_{s,bot}$= area of bottom beam bars; $A_{s,top}$= area of top beam bars; A_s= area of the bar group, $A_{s,top}$ or $A_{s,bot}$, containing the bar for which development length is being calculated; α_o= overstrength factor for beam bars; α_f= 1.0 for a beam bar passing through a joint subjected to unidirectional loading, and α_f= 0.85 for bi-directional loading; Bar location factor α_t= 0.85 for a top beam bar where more than 300 mm of fresh concrete is cast below the bar, α_t= 1.0 for all other cases. P= axial compression force on column; A_g= gross area of column; f_c'= concrete compressive strength.

For many years, Grade 420 (f_y=420 MPa) steel reinforcement has been the standard for reinforced concrete construction in Taiwan as well as in the United States. Whenever high-strength steel reinforcing bars are used in the concrete structures, bond requirements become critical issues for design and proportion. ACI-ASCE Committee 352 (2002) recommends a reasonable multiplier of $f_y/420\ MPa$ for the minimum column depth of $20d_b$ for higher grade reinforcement.

$$\frac{h_c}{d_b} \geq 20\frac{f_y}{420} \tag{2}$$

For comparison of the existing design equations, a reference cruciform beam-column joint is assumed to have beam hinging adjacent to the joint faces, a least axial compression of $0.2A_g f_c'$, a practical beam reinforcement ratio $A_{s,bot}/A_{s,top}$ of 0.75, equal bar diameter for top and bottom reinforcement, and bar f_y of 420 or 690 MPa. Under such conditions, the minimum column depths with respect to the bottom bar diameter are compared in Figure 3. Clearly, the bond requirements of AIJ Guideline (1999) are very conservative, and those of NZS 3101 are relatively less conservative. The recommendations of Brooke and Ingham (2013) still results in a column depth similar to those of NZS 3101 (2006). The minimum column depth recommended by ACI 352R (2002) is also displayed in Figure 3, which may be conservative for normal strength concrete but be too conservative for high strength concrete.

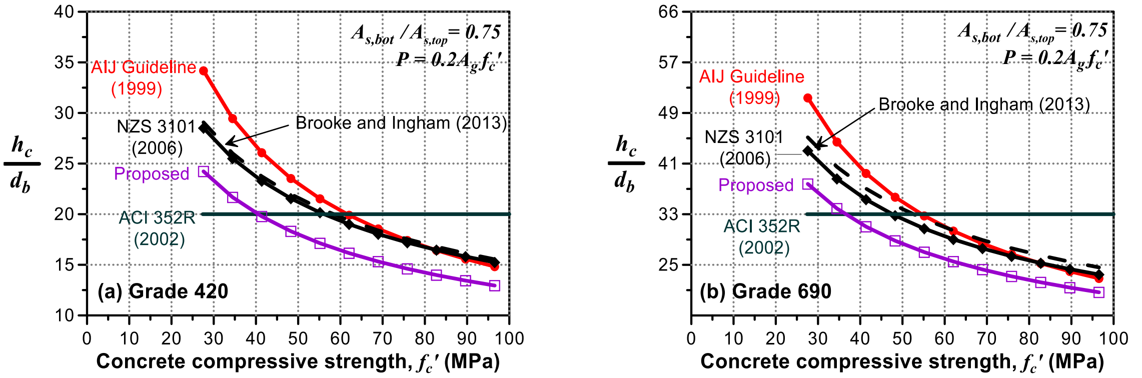


Figure 3. Comparison of minimum column depth for (a) Grade 420 bars with $\alpha_o = 1.25$; (b) Grade 690 bars with $\alpha_o = 1.15$.

3 DATABASE INVESTIGATION AND PERFORMANCE EVALUATION

3.1 Database of beam-column joint tests

Lee and Hwang (2013) presented a database for reverse cyclic tests of reinforced concrete beam-column joints in special moment frames by extensively reviewing the related papers published in Japan, United States, New Zealand, and Taiwan. About 200 interior joints were assembled in this database. All specimens were reinforced concrete concentric beam-column subassemblages isolated from inflection points of beams and columns, and tested under quasi-static cyclic lateral loading (typical repeated cycles for each drift ratio ranged from one to three) to simulate the earthquake-introduced forces acting on the joints.

Test results of beam-column joints were classified in three basic failure modes including: Beam flexure failure ("B" failure), Joint shear failure without yielding of beam bars ("J" failure), Joint shear failure with yielding of beam bars ("BJ" failure). The modes of B-, BJ-, and J-failures are well-accepted in Japan for the development of design guidelines for beam-column joints (Kitayama et al. 1991). Besides above three basic failure modes, some joint specimens were reported as BJa failure, which is refer to bond or anchorage failure along the beam bars in the joint.

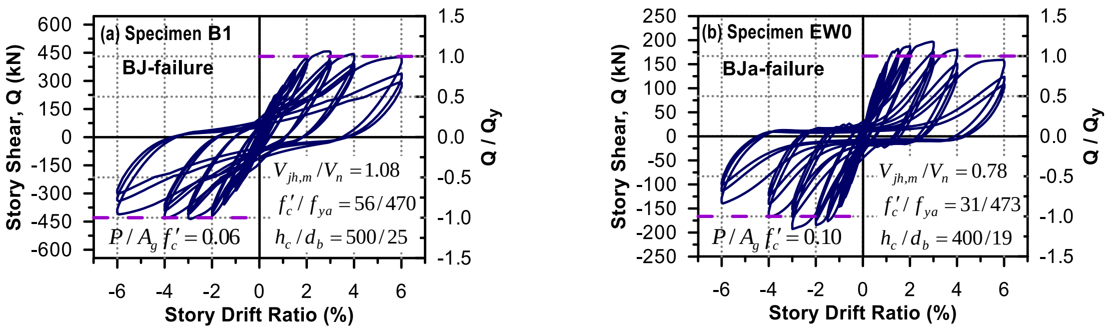


Figure 4. Cyclic loading response of (a) BJ-failure and (b) BJa-failure joint specimens tested in YunTech.

Figure 4 shows two interior beam-column joints tested by first authors and his colleagues at YunTech in Tawian. Both specimens reached beam yielding at about 1.5% to 2% drift ratio, but Specimen B1 eventually failed in joint shear at 6% drift ratio while Specimen EW0 exhibited a very pinched hysteretic curve since the bond failure occurred in the 3% drift cycles. Notably, both specimens had a column depth of 20 bar diameters and similar bar yield strength, but hysteresis performance of the Specimen B1 is better than that of Specimen EW0, although the later had a lower target shear stress in the joint. Obviously, the key parameter make Specimen B1 perform better is attributed to its higher concrete strength (greater bond resistance in the joint).

It seems the hysteresis performance of beam-column joints is also related to the bond stress of the beam bar in the joint. Although the consequence of bond failure is not as severe as that of shear failure in joints, it is still preferred to prevent the bond failure within the design earthquake level, which is about 3.5% drift capacity for the structural testing. This paper intends to determine a viable set of design equations to cover the use of Grade 690 reinforcement in joints of special moment frames. Therefore, a subset of available test data from the database of Lee and Hwang (2013) was investigated in this paper according to ACI 374.1-05, the acceptance criteria for moment frames (ACI Committee 374, 2005). Hysteretic performances including strength, stiffness, and energy dissipation capacity for each test specimen were evaluated for assessing existing design codes and recommendations for the development length in beam-column joints.

3.2 **Acceptance criteria for testing performance of beam-column joints**

ACI Committee 374 (2005) reported a testing protocol and acceptance criteria for structural components of special moment frames. For acceptance, test results of the third complete cycle to a limiting drift ratio not less than 3.5% should satisfy:

1. Strength at peak displacement shall not be less than 75% of the maximum peak strength in the same loading direction;
2. Secant stiffness between drift ratios of -1/10 and +1/10 of the limiting drift ratio shall not be less than 5% of the initial stiffness obtained from the first cycle; and
3. Energy dissipation in the third cycle of limiting drift ratio shall not be less than 12.5% of the idealized elastoplastic energy of that drift ratio.

For Specimen EW0 shown in Figure 4(b), the third 4% drift cycle had a peak strength equal to 64% of the maximum peak strength, a secant stiffness between $\pm$0.4% drift ratios equal to 1% of the initial stiffness, and an relative energy dissipation ratio of 19%. Obviously, hysteresis performance of Specimen EW0 is not acceptable because it does not satisfy the three acceptance criteria given by ACI 374.1-05. The poor hysteresis performance of Specimen EW0 can be attributed to the bond failure along beam bars in the joint occurred in 3% drift cycles.

The selection of number of cycles at each drift ratio depends on the judgment of the researchers and the particular degradation characteristics of the system being tested. More recently, ACI 374.2R-13 (ACI Committee 374 2013) reported that a minimum of two cycles at each deformation level is sufficient to consider the damage associated with the number of cycles at a given drift level. For the assessment of the bond performance for each test specimen, therefore, the second (or third, if available) cycle at a drift ratio between 3.5% and 4% were reproduced. Therefore, this paper selected 59 interior joints from the database of Lee and Hwang (2013) according to following conditions:

- BJ or BJa failure specimens;
- straight beam bars passing through the joint with bar f_y exceeding 400 MPa;
- cyclic loading response have a minimum of two cycles at a drift ratio exceeding 3.5%.

After screening, a total amount of 65 interior joint specimens (46 BJ and 19 BJa data) are evaluated in this paper. Figure 5 displays the range of measured concrete strength and bar yield strength for the selected BJ and BJa failure specimens. Obviously, bond failure is likely to occur for the combination of higher strength reinforcement and normal-strength concrete. Within the test database, there is no BJa-failure specimens available for concrete strength exceeding 100 MPa.

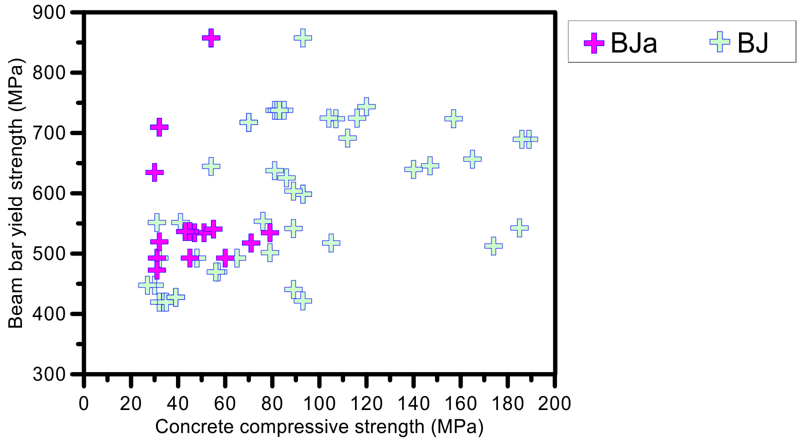
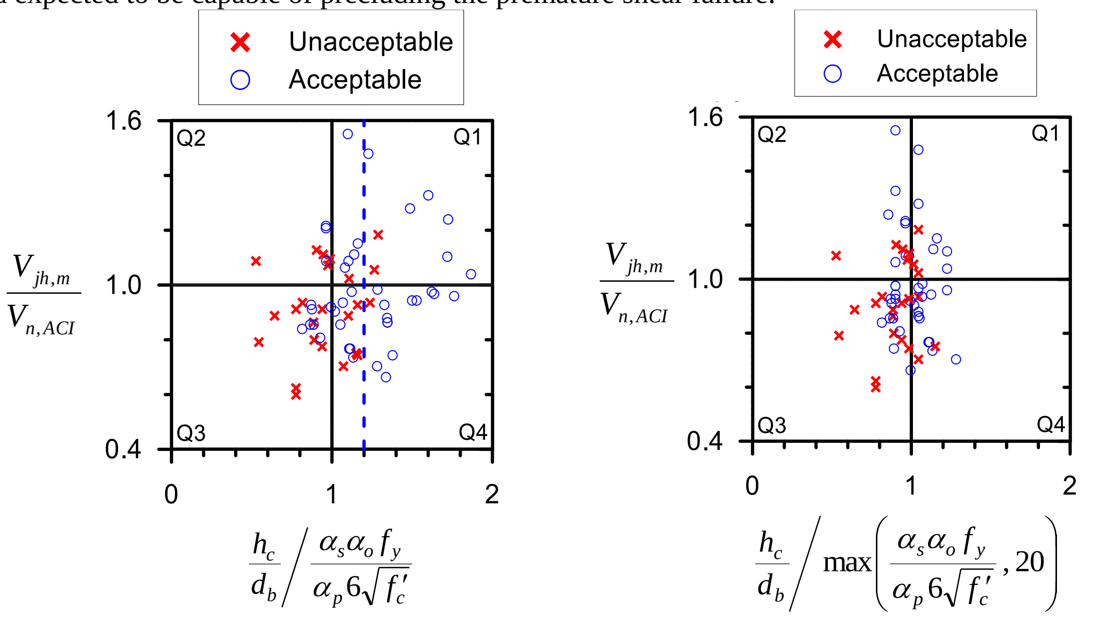


Figure 5. Range of joint concrete strength and bar yield strength in the selected subset of test data.

3.3 Assessment of bond requirements

According to the three acceptance criteria given by ACI 374.1-05, the second (or third, if available) cycle at a drift ratio about 4% for each test specimen is evaluated and classified as "acceptable" or "unacceptable" performance. The acceptable test data satisfy the aforementioned three acceptance criteria while the unacceptable may only meet one or two of the three acceptance criteria.

Figure 6(a) shows the relations of hysteresis performance to the ratio of experimental-to-nominal joint shear strength and the ratio of provided-to-required column depth. The vertical axis of Figure 6 is the maximum experimental joint shear force ($V_{jh,m}$), which can be back-calculated from the beam moments in equilibrium with the peak maximum lateral loads, divided by the nominal joint shear strength ($V_n = 1.25\sqrt{f_c'}A_j$) specified in ACI 318 (2014) for interior joints without transverse beams. Therefore, test data fall in Quadrat 3 and 4 had experimental joint shear stresses below the permissible value of $1.25\sqrt{f_c'}$ MPa and expected to be capable of precluding the premature shear failure.



(a) Eq.(2) with $u_b = 1.5\sqrt{f_c'}$ MPa

(b) Set $u_b = 1.5\sqrt{f_c'}$ MPa and a minimum column depth of $20d_b$

Figure 6. Relations of hysteresis performance to the ratio of experimental-to-nominal joint shear strength and the ratio of provided-to-required column depth.

The horizontal axis of Figure 6(a) is provided column depth in experiments divided by the required column depth recommended in this paper. In other words, test data fall in Quadrat 1 and 4 had column depth exceeding the minimum column depth, calculated using a basic bond strength of $1.5\sqrt{f_c'}$ MPa combined with the factors $(1 + \kappa)$ and α_p proposed by Brooke and Ingham (2013), and expected to be capable of precluding premature bond failure. Ideally, test data with unacceptable performance shall not appear in Quadrat 4 because both the joint shear and bond stress are kept within permissible limits. However, as shown in Figure 6(a), there are six unacceptable data in Quadrat 4, indicating that the use of basic bond strength of $1.5\sqrt{f_c'}$ MPa may be unconservative for certain conditions.

One of the solutions to improve conservation is to reduce the basic bond strength from $1.5\sqrt{f_c'}$ to $1.25\sqrt{f_c'}$ MPa, as proposed by Brooke and Ingham (2013), and then increase 20% of the required column depth, as shown by the vertical dash line in Figure 6(a). Clearly, only one unacceptable test data exceeds 1.20 times the proposed bond development length (vertical dash line). However, many acceptable test data also fall between 1.0 and 1.20 times the proposed bond development length, indicating the reduction of basic bond strength from $1.5\sqrt{f_c'}$ to $1.25\sqrt{f_c'}$ MPa may also be too conservative for some conditions.

Another simple promising way is to improve the safety by setting a minimum column depth of 20 bar diameters instead of reducing the basic bond strength. As shown in Figure 6(b), the limitation of minimum $20d_b$ criterion moves those test data with low $f_y/\sqrt{f_c'}$ ratios toward the left quadrants and substantially improves the distribution of the unacceptable data. According to data observation in Figure 5, it is concluded that either the minimum column depth of $20d_b$ or the reduction of basic bond strength from $1.5\sqrt{f_c'}$ to $1.25\sqrt{f_c'}$ MPa would give similar safety to preclude unacceptable hysteresis performance.

Finally, the minimum column depth proposed herein are based on test data obtained from relatively conservative bond conditions of the beam bars, which extend through an isolated cruciform beam-column joints without transverse beams and slabs. There is lack of experimental evidence of bond failure observed from indeterminate frame with floor slabs, which may counter the slip of beam bars passing through the frame joints. Definitely, a larger h_c/d_b ratio could reduce the potential beam bar slip within the interior joints during a major earthquake, but it would also lead to larger columns and/or smaller diameter bars in groups, which makes design and construction difficult. Therefore, the minimum h_c/d_b ratios specified for special moment frames by codes and standards are based on the judgment of how well the hysteresis behaviour expected at a design interstory drift is.

After a design earthquake attack, a moment frame with bond failure in joints may be too flexible under a moderate earthquake. Because it is unlikely to be repair bond failure, a beam-column joint has better be well-proportioned to avoid bond failure occurred during a design-based earthquake. Although bond performance of beam bars passing through interior joints have been extensively studied since 1980s, the development length requirements for beam-column joints still differ remarkably among the international design codes and standards. This paper only provides recommendations for joints of special moment frame designed and detailed according to ACI 318 code.

4 DESIGN RECOMMENDATIONS AND CONCLUSIONS

Based on database investigation, this paper recommends a viable set of design equations for the development length of straight beam bars passing through the joints of special moment frame. The proposed design equations are validated by the testing performance of beam-column joints at a drift ratio about 4%, where the hysteresis behaviour is evaluated by the acceptance criteria specified in ACI Code and standards. For achieving an acceptable bond performance, a minimum column depth-to-beam bar diameter ratio can be related to available bond strength in the joint, reinforcement ratio in beams, and column axial loads. Using common bond design equations, it is recommended to use a basic bond strength of $1.5\sqrt{f_c'}$ MPa with two modification factors proposed by Brooke and Ingham (2013), accounting for effects of reinforcement ratio and column axial loads. Besides, the minimum column depth-to-beam bar diameter ratio should not be taken less than 20.

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