Lee et al. Int J Concr Struct Mater (2018) 12:70
https://doi.org/10.1186/s40069-018-0303-2

International Journal of Concrete Structures and Materials

RESEARCH Open Access

Simplified Design Equation of Minimum Interior Joint Depth for Special Moment Frames with High-Strength Reinforcement

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Abstract

To avoid excessive slip of beam longitudinal bars at the joints of an earthquake-resisting moment frame, ACI 318 Building Code set a minimum joint depth of 20 times the diameter of the largest longitudinal beam bars passing through the joint, which is based on prior experimental verification of beam-column joints with Grade 420 MPa reinforcement. In view of that the 20-bar-diameter criterion cannot be simply extended for concrete frame joints with higher grade reinforcement, this paper summarizes international existing design criteria and proposes a simplified equation for the minimum joint depth. The equation applicability is assessed by evaluating the cyclic testing results of beam-column joints conducted in East Asian and Pacific Countries, where Grade 490, 590, and 690 MPa reinforcement have been used for earthquake-resistant concrete structures. Beam-column joints that satisfy the proposed equation can demonstrate satisfactory hysteresis behavior at an interstory drift of 4%.

Keywords: beam-column joint, bond, cyclic testing, high-strength reinforcement, slip

1 Introduction

A well-designed beam-column joint in a special moment-resisting frame should be able to transfer beam and column moments and anchor the beam and column longitudinal reinforcement. Under the design basis earthquake (DBE) or the maximum considered earthquake (MCE), beam hinging adjacent to the faces of beam-column joints is anticipated, which results in a severe bond stress demand along the straight beam bars extending through the joint. As shown in Fig. 1, the straight beam bars in the joint are subjected to tension at one face of the joint and compression at the opposite face. If the column dimension or joint depth is relatively short, the beam bar tension cannot be fully developed in the joint because of excessive bond demands, but instead the beam bar must be anchored within the beam on the opposite side of the joint. This results in an increase in flexural compression at the opposite beam end, leading to crushing of the concrete. Figure 2 shows consequences of excessive bond demands at an interior joint with adequate transverse reinforcement (Lee et al. 2007). Moehle (2015) indicated that such bond deterioration along the beam bars in the joint and severe damage at the beam ends enable the bars to slip almost freely, resulting in excessive deformation of the beam-column joint for small lateral force changes (very pinched hysteresis behavior), and thereby reducing the energy dissipation capacity. From analytical and experimental observations, Hakuto et al. (1999) and Shiohara (2001) also indicated that the bond deterioration could cause degradation in beam flexural strength. The occurrence of bond deterioration or joint core anchorage failure in a DBE event may render the frame flexible and prone to large lateral drift in the later moderate earthquakes. Because severe bond deterioration is difficult to repair, and it should be avoided in the design of a special moment frame at the drift demand from the DBE event.

ACI 318 Building Code (ACI Committee 318 2014) has set a minimum joint depth of 20 d_b, where d_b is the diameter of the largest longitudinal beam bars extending through a joint of a special moment frame. This 20

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Full list of author information is available at the end of the article
Journal information: ISSN 1976-0485 / eISSN 2234-1315

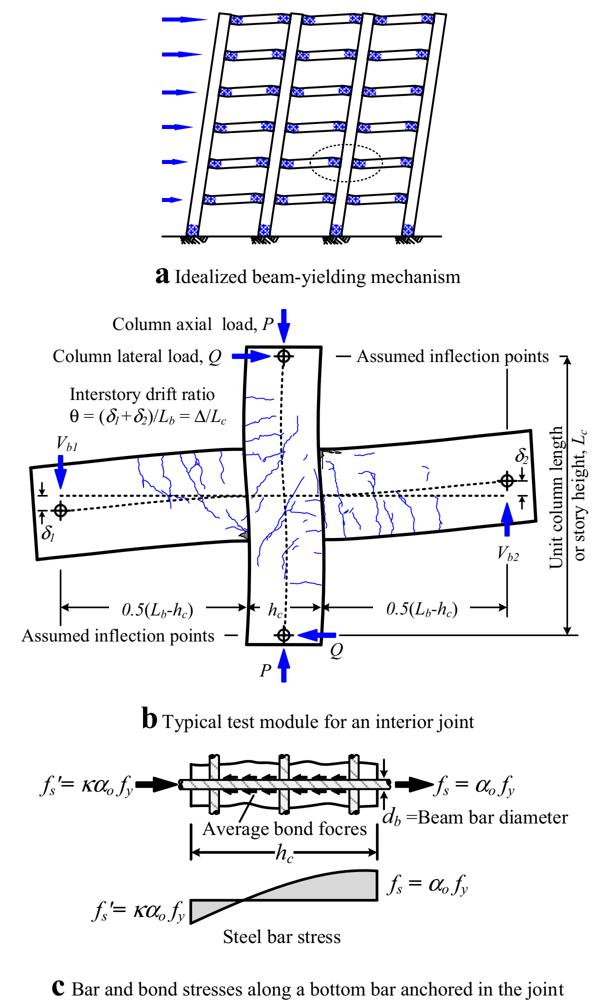


Fig. 1 Moment-resisting frame joints with plastic hinges at the joint faces.

d_b criterion is based on an evaluation (Zhu and Jirsa 1983) of cyclic loading response for 18 beam-column joints made with normal-strength concrete and Grade 420 MPa reinforcement. Zhu and Jirsa (1983) concluded that a minimum column depth of 20–22 d_b is appropriate to avoid excessive bar slip at an interstory drift of 3% for Grade 420 MPa reinforcement.

Currently, several reinforcement producers in the United States are capable of producing Grade 550 and 690 MPa reinforcement with similar manufacturing costs per unit weight (Kelly et al. 2014). Therefore, the use of higher grade reinforcement has advantages of cost and labor savings in steel fabrication. However, ACI 318-14 (2014) only permits Grade 420 MPa for primary reinforcement of special seismic systems because of insufficient data to confirm the applicability of existing code provisions for structures using higher grades. On the other hand, Grade 490 or 500 MPa reinforcement has been widely used as primary reinforcement of earthquake-resistant concrete structures in Japan (AIJ 2010), Taiwan (Ministry of the Interior 2017), and New Zealand (NZS 3101 2006). Even higher Grade 590 and 690 MPa reinforcement can be used in high-rise buildings with peer review and special approval in Japan (Aoyama 2001; Nishiyama 2009). Whenever a higher grade reinforcement is used, the anchorage at a beam-column joint become a critical issue.

To address concerns about the excessive slip of higher grade bars at beam-column joints, the ACI-ASCE Joint committee 352 recommended a multiplier to the 20 d_b criterion as follows (ACI-ASCE Committee 352 2002).

$$\frac{h_c}{d_b} \geq 20\left(\frac{f_y}{420}\right) \quad (1)$$

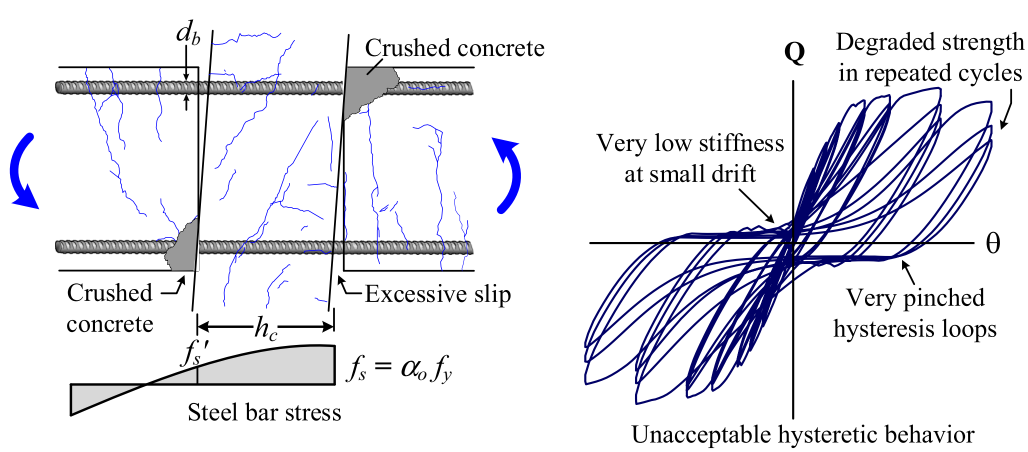


Fig. 2 Consequences of bond deterioration along the beam bars passing through a joint with Grade 420 reinforcement, 31-MPa concrete, and 20 d_b column depth (Lee et al. 2007).

More recently, Hwang et al. (2014) conducted cyclic loading tests of three cruciform beam-column joints made with Grade 600 MPa am bars and 32 MPa concrete. The column depths ranged from 20 d_b to 25 d_b, which is less than 29 d_b required by Eq. (1). The test results exhibited very pinched hysteresis behavior due to excessive slip of beam bars in the joint core. The multiplier of $f_y/(420\text{MPa})$ in Eq. (1) seems necessary for the joints made of normal-strength concrete and higher grade reinforcement; however, a greater h_c/d_b ratio needs either a large column depth or a small permissible diameter of beam bars, which would make the design or construction difficult. In practice, higher grade reinforcement is used together with high-strength concrete, particularly for columns with limited architectural dimensions and high axial load at the lower levels of high-rise buildings (Aoyama 2001). Basically, Eq. (1) does not account for the beneficial effects of high-strength concrete and high axial load. Somewhat more complicated equations of minimum h_c/d_b ratios can be found in other international concrete design codes (AIJ 2010; CEN 2004; NZS 3101 2006).

This paper summarizes international existing design criteria and proposes a simplified design equation for the minimum joint depth of special moment frames. The applicability of the proposed equation is assessed using a test database of beam-column joints made with Grade 490, 590, and 690 MPa reinforcement. Hysteresis performance in terms of strength degradation, residual stiffness, and energy dissipation capacity of each joint test is evaluated according to the ACI standards for special moment frames. Referable design equations and experimental verification are provided. Areas needing further research experiments are also indicated in this paper.

2 Existing Design Codes and Recommendations

2.1 Generic Formula

During the formation of beam hinging adjacent to the joint faces, the beam bar may be subjected to a tensile stress of $\alpha_o f_y$ at one face of the joint and a compressive stress of $\kappa\alpha_o f_y$ at the opposite face of the joint, as shown in Fig. 1c. By assuming an average bond stress on the beam bar along the column depth, the bond demand should not exceed the available bond resistance in the joint to avoid excessive bar slip, as expressed below.

$$\frac{\pi d_b^2}{4}\left(\alpha_o f_y + \kappa\alpha_o f_y\right) \le \pi d_b h_c \alpha_p u_b \tag{2}$$

Rearranging the equation, one can obtain:

$$\frac{h_c}{d_b} \ge \frac{(1+\kappa)}{4}\frac{\alpha_o f_y}{\alpha_p u_b} = \frac{\alpha_s \alpha_o f_y}{4\alpha_p u_b} \tag{3}$$

The design criteria of minimum h_c/d_b ratios or maximum d_b/h_c limits on the basis of Eq. (3) can be found in international concrete design codes and standards such as the NZS 3101 (2006), AIJ (2010), and Eurocode 8 (CEN 2004). Table 1 compares these design equations together with additional two sets of equations from Brooke and Ingham (2013) and Li and Leong (2015), both of which were amendments to the NZS 3101 (2006). These existing design criteria have been widely used for earthquake-resisting concrete buildings made with Grade 300–500 MPa reinforcement and normal strength concrete. Unfortunately, the expressions of α_s, α_p, and u_b listed in Table 1 are inconsistent. Notably, the design equations of Eurocode 8 (CEN 2004) and AIJ (2010) are similar, the difference being the application of reduction factors of 0.75 for the reinforcement compressive stress, 0.80 for the bond strength, and 0.80 for the column axial stress in the case of Eurocode 8. On the other hand, NZS 3101 (2006) uses a basic 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.

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 minimum joint depth. The research concluded that the existing criteria cannot reflect the bond deterioration observed in experiments. Brooke and Ingham (2013) recommended to modify the basic bond strength to 1.25 $\sqrt{f_c'}$ MPa and the corresponding equations of α_s and α_p for NZS 3101 (2006), as shown in Table 1. In addition, Li and Leong (2015) performed experimental and numerical investigations on eight interior beam-column joint specimens and proposed somewhat different modifications. More detailed comparisons of the design equations listed in Table 1 are summarized elsewhere (Brooke 2011; Chen 2017).

2.2 Comparison of Existing Equations for Minimum Column Depth

In common design practice, the beam reinforcement ratio $A_{s,bot}/A_{s,top}$ usually ranges between 0.5 and 1.0. If the top reinforcement area ($A_{s,top}$) exceeds the bottom reinforcement area ($A_{s,bot}$) at a beam-column joint, relatively larger flexural compression would be developed at the bottom beam ends in order to balance the greater flexural tension developed from the top beam bars at the same section. Thus, the compressive stress of the bottom beam bars at the joint face would be greater than that of the top beam bars at the opposite face. In other words, the stress gradient along the bottom bars is larger than

Table 1 Comparison of existing design equations for the minimum joint depth.

Design criteria	Factor for bar stresses at the joint faces $\alpha_s = 1 + \kappa$	Basic bond strength u_b, MPa (psi)	Factor for column axial stress on bond α_p
AIJ (2010)	$1 + \frac{A_{s,bot}}{A_s}$	$0.7 f_c'^{\frac{2}{3}}$ $\left(3.7 f_c'^{\frac{2}{3}}\right)$	$1 + \frac{P}{A_g f_c'}$
Eurocode 8 (CEN 2004)	$1 + 0.75\frac{A_{s,bot}}{A_s}$	$0.56 f_c'^{\frac{2}{3}}$ $\left(2.9 f_c'^{\frac{2}{3}}\right)$	$1 + 0.8\frac{P}{A_g f_c'}$
NZS 3101 (2006)	$1 + 1.55 - \frac{A_s}{A_{s,top}} \leq 1.8$	$\alpha_f \alpha_t 1.5\sqrt{f_c'}$ $(\alpha_f \alpha_t 18\sqrt{f_c'})$	$0.95 + 0.5\frac{P}{A_g f_c'} \leq 1.25$
Brooke and Ingham (2013)	$1 + \frac{0.7}{\alpha_o}\frac{A_{s,top}}{A_s} \leq 1 + \frac{1}{\alpha_o}$	$\alpha_f \alpha_t 1.25\sqrt{f_c'}$ $(\alpha_f \alpha_t 15\sqrt{f_c'})$	$0.9 + 2.0\frac{P}{A_g f_c'} \leq 1.20$
Li and Leong (2015)	$1 + \frac{0.6}{\alpha_o} + \frac{0.8}{\alpha_o}\left(1 - \frac{A_s}{A_{s,top}}\right)$	$\alpha_f \alpha_t 1.25\sqrt{f_c'}$ $(\alpha_f \alpha_t 15\sqrt{f_c'})$	$0.95 + 0.5\frac{P}{A_g f_c'} \leq 1.10$

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 = bar overstrength factor; 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, $\alpha_t = 1.0$ for all other cases. P = axial compression force on column; A_g = gross area of column; f_c' = concrete compressive strength.

that along the top bars in the joint, indicating a relatively severe bond demand for the bottom bars.

For a common beam reinforcement ratio of $A_{s,bot}/A_{s,top} \leq 0.75$ and overstrength factor $\alpha_o = 1.25$, the outcomes of the stress gradient factor $\alpha_s = (1 + \kappa)$ for bottom beam bars are equal to 2.0 per AIJ (2010), 1.75 per Eurocode 8 (CEN 2004), 1.80 per NZS 3101 (2006), 1.75 per Brooke and Ingham (2013), and 1.64 per Li and Leong (2015). Notably, the bar compressive stress $\kappa\alpha_o f_y$ is unlikely to reach $\alpha_o f_y$ as for the tensile stress, because of the limited compressive strain of the beam bar in the flexural compression zone at the beam ends. To conclude, taking $\alpha_s = 2.0$ (or $\kappa = 1.0$) is the most conservative but unrealistic condition, and thereby $\alpha_s = 1.8$ (or $\kappa = 0.8$) may be taken as a physically reasonable assumption.

To compare the minimum column dimensions required by various design codes and recommendations, a reference cruciform beam-column joint is assumed to have beam hinging adjacent to the joint faces, a typical overstrength factor $\alpha_o = 1.25$, a minimum axial compression P=0.2 $A_g f_c'$, a common beam reinforcement ratio $A_{s,bot}/A_{s,top} = 0.75$, and an equal bar diameter d_b for the top and bottom beam bars. Because of the larger stress gradient (α_s) along the bottom bars due to the unequal reinforcement ratio $A_{s,bot}/A_{s,top} = 0.75$, the minimum column dimension given by Eq. (3) and Table 1 would be determined by the bottom beam bars passing through the joint.

For the reference beam-column joint, Fig. 3 shows the ratio of Eq. (3) to Eq. (1) for various values of f_c' using the five sets of design equations listed in Table 1. Notably, Eq. (1) requires minimum h_c/d_b ratios of 20, 26, and 33 for Grade 420, 550, and 690 MPa reinforcement, respectively. The minimum h_c/d_b ratios for bottom beam bars per Eurocode 8 (CEN 2004) are the most conservative (Fig. 3) because of lower bond strength (Table 1). The requirements of the AIJ (2010) are also conservative at taking $\alpha_s = 2.0$. As shown in Fig. 3, the recommendation of Li and Leong (2015) is more conservative than that of NZS 3101 (2006) because of reduced bond strength (Table 1). Brooke and Ingham (2013) also suggested a lower bond strength but allowed taking more advantage of the column axial load (Table 1), thus resulting in h_c/d_b ratios about 2% larger than those of NZS 3101 (2006) for P=0.2 $A_g f_c'$. For very low or high axial load, such as P=0.05 $A_g f_c'$ or 0.50 $A_g f_c'$, the h_c/d_b ratios recommended by Brooke and Ingham (2013) would be 17% larger than those of NZS 3101 (2006). For the normal-strength concrete of $f_c' \leq$55 MPa, the minimum column dimensions required by the five sets of design equations are well above that obtained from Eq. (1) per ACI 352R-02 (ACI-ASCE Committee 352 2002). By contrast, the requirement of Eq. (1) may be too conservative for a concrete compressive strength exceeding 70 MPa.

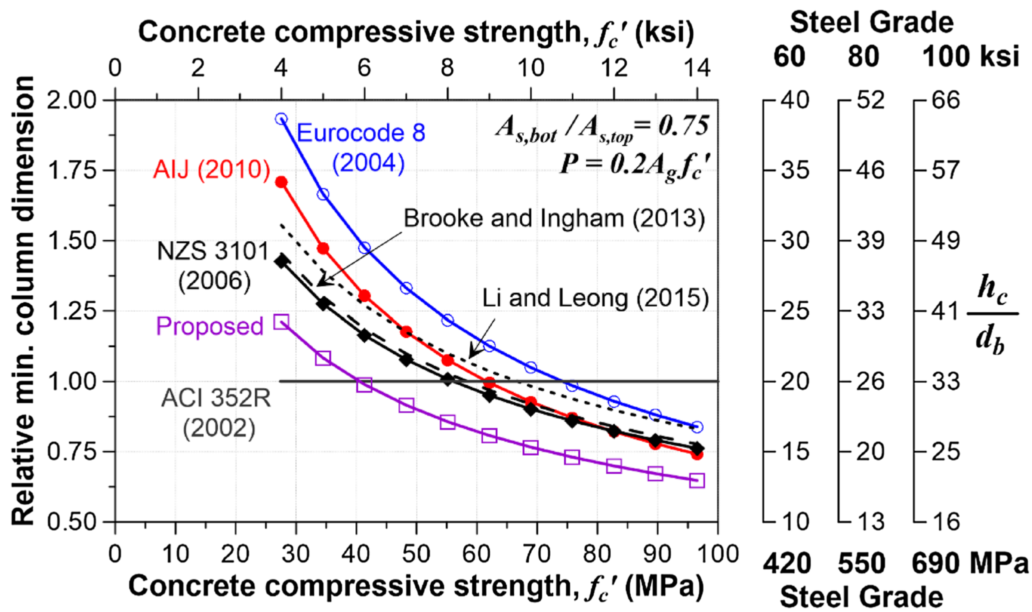


Fig. 3 Comparison of minimum column dimensions required by different codes and recommendations for a reference beam-column joint having a column axial load of 0.2 $A_g f_c'$, a reinforcement ratio of $A_{s,bot}/A_{s,top} = 0.75$, and various concrete strengths.

3 Simplification of Design Equation for the Minimum Joint Depth

Due to the long-standing use of $\sqrt{f_c'}$ in NZS3101 (2006) and ACI 318 (2014) for bond and shear strengths, this paper also recommends simplifying the design equation from the criterion of NZS 3101 (2006), which sets the basic bond strength $u_b = 1.5\sqrt{f_c'}$ MPa for Eq. (3). As listed in Table 1 and discussed in the prior section, the stress gradient factor α_s is affected by beam reinforcement ratios and bar overstrength factor α_o. Except for the AIJ (2010), all existing criteria give a α_s factor not exceeding 1.80 for bottom beam bars with $\alpha_o = 1.25$. For simplicity and conservativeness, $\alpha_s = 1.80$ is taken to simplify Eq. (3) as follows.

$$\frac{h_c}{d_b} \geq \frac{1.8\alpha_o f_y}{\alpha_p 6\sqrt{f_c'}} \tag{4}$$

where a typical value of $\alpha_o = 1.25$ is usually used in practice, and this paper recommends to use $\alpha_p = 0.9 + 2.0\frac{P}{A_g f_c'} \leq 1.20$, which is proposed by Brooke and Ingham (2013).

All design equations listed in Table 1 allow designers to increase the reliable bond strength in the joint with different rates and limits as the column axial load increases. The effect of column axial load on the reliable bond strength in the joint was extensively investigated by Brooke and Ingham (2013). The data support the common view that increasing axial load increases the bond strength. However, tests on isolated bars embedded in concrete suggest that there is an upper limit to the bond strength enhancement that can be achieved by increasing the transverse compression stress acting on an anchorage. Therefore, Brooke and Ingham (2013) proposed to set an upper limit of 1.20 on the α_p factor.

The seismic forces acting on a moment frame generally do not make large contributions to the axial load at interior columns (Moehle 2015). For a typical interior column in a multistory building, a minimum column axial load of 0.15 $A_g f_c'$ is a rational assumption. Substituting P = 0.15 $A_g f_c'$ into the equation of α_p recommended in Eq. (4) results in a constant $\alpha_p = 1.20$. Accordingly, Eq. (4) can be further simplified as below.

$$\frac{h_c}{d_b} \geq \frac{\alpha_o f_y}{4\sqrt{f_c'}} \tag{5}$$

The proposed Eq. (5) gives a relatively short depth for the reference beam-column joint shown in Fig. 3 and verified with the database presented later.

4 Database Investigation

4.1 Assessment of Anchorage Performance of Beam Bars in the Joint

The quasi-static reversed cyclic loading test is the most commonly used method for testing components of earthquake-resistant structures in laboratory. Numerous cyclic loading tests of reinforced concrete beam-column joints have been extensively reviewed and assembled in several databases (Brooke and Ingham 2013; Kim and LaFave 2007; Lee and Hwang 2013) for various evaluations of design equations or analytical models. Brooke and Ingham (2013) assembled a database of 93 interior beam-column joint tests to evaluate the minimum joint depth; however, only 32 specimens had reinforcement grades of 490 or higher. The anchorage performance of higher grade bars has not been fully clarified. For high-strength reinforcement and concrete, Lee and Hwang (2013) also assembled a more comprehensive database consisting of 202 cruciform beam-column joints to assess current ACI 318 design provisions for the joint strength and confinement.

For cyclic testing of a cruciform beam-column joint, the consequences of bond deterioration along the beam bars in the joint core includes excessive bar slip, severe concrete crushing at beam ends, strength degradation at peak displacement, very low stiffness at small displacement, and reduced energy dissipation capacity (hysteretic pinching). In the database of Lee and Hwang (2013), some test results with aforementioned characteristics were reported as “BJa” failure [joint core anchorage failure after beam yielding, or “bond” failure as called by Brooke and Ingham (2013)] by the original researchers. However, some test results may also have bond deterioration but not subjectively indicated as “BJa” failure because of the difficulty in detecting such damage in the joint core.

Therefore, this paper proposes to assess how well the beam bars are anchored in the joint by evaluating the hysteresis behavior of a beam-column joint. For acceptance, as illustrated in Fig. 4, the test results of the third complete cycle to a limiting drift ratio of 3.5% at least should satisfy the following acceptance criteria for testing components of special moment frames given by ACI 374.1-05 (2005).

1. Strength degradation at the peak displacement of the limiting drift cycle shall not exceed 25% of the maximum load resistance in the same loading direction;
2. Residual secant stiffness between ± 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 dissipated in the limiting drift cycle shall not be less than 12.5% of the idealized elastoplastic energy of that drift ratio.

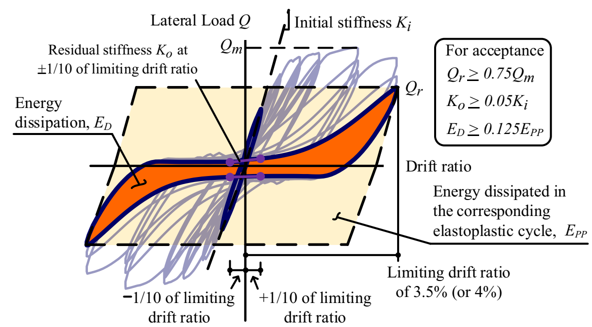


Fig. 4 Acceptance criteria for testing components of special moment frames given by ACI 374.1-05.

Unfortunately, only a few tests had three cycles at the limiting drift ratio. More recently, ACI 374.2R-13 (ACI Committee 374 2013) reported that a minimum of two cycles at each drift ratio is sufficient to consider the damage associated with the number of cycles at a given drift ratio. To evaluate the hysteresis performance for each test specimen, the second (or third, if available) cycle at a drift ratio of 3.5% or 4% was used. Therefore, this study omitted the test data that did not have a minimum of two cycles at a drift ratio of 3.5% or 4%. In addition, the beam-column joints that failed in joint shear (so-called “J” failure) without yielding of beam bars were excluded, because the “J” failure is primarily dominated by the joint shear stress and indirectly related to the permissible bond stress of the beam bars in the joint. Finally, to evaluate the minimum joint depth, this study assembled a database of available test data with the following conditions.

1. Reinforced concrete cruciform beam-column joints without transverse beams and slabs. The confinement of continuous transverse beams and slabs may enhance the bond strength of the joint core concrete.
2. For each joint, all the beam longitudinal bars extended through the joint core are confined by transverse reinforcement.
3. The hysteresis loop of the second (or third, if available) cycle at a drift ratio of 3.5% or 4% can be extracted data values from published graphs by a plot digitizer.
4. Yielding of beam bars occurred at the joint faces, followed by beam flexure, joint shear, or anchorage failure (“B”, “BJ”, or “BJa” failures).

Table 2 Key parameters of the joints with Grade 490 or 500 MPa beam bars.

First author	Specimen	Failure mode	Test parameters								Performance evaluation[c]			
			f'_c MPa	f_{ya} MPa	h_c mm	$\frac{h_c}{d_b}$	$\frac{A_{s,bot}}{A_{s,top}}$	$\frac{P}{A_g f'_c}$	$\frac{V_{jh\text{-}m}}{V_n}$[a]	$A_{sh,ratio}$[b]	$\frac{Q_r}{Q_m}$	$\frac{K_o}{K_i}$	$\frac{E_D}{E_{PP}}$	Rating
Teraoka (1994)	HNO. 9	BJ	93	599	400	18.0	1.00	0.20	0.98	1.03	0.87	0.11	0.35	o
Nakachi (1995)	NO. 1	BJa	45	493	400	20.9	1.00	0.20	0.70	2.33	0.69	0.02	0.21	x
	NO. 2	BJ	48	493	400	20.9	1.00	0.20	1.02	2.18	0.69	0.02	0.23	x
	NO. 3	BJa	31	493	400	20.9	1.00	0.20	0.80	3.36	0.63	0.00	0.17	x
	NO. 4	BJ	32	493	400	20.9	1.00	0.20	1.13	3.21	0.62	0.03	0.25	x
	NO. 5	BJa	60	493	400	20.9	1.00	0.20	0.94	1.72	0.54	0.01	0.29	x
	NO. 6	BJ	65	493	400	20.9	1.00	0.19	1.18	1.61	0.67	0.05	0.30	x
Hosoya (2003)	NO. 1	BJa	47	535	450	23.6	0.83	0.19	0.75	0.61	0.62	0.02	0.23	x
Maruta (2004)	CC-3	BJ	185	543	400	18.0	1.00	0.07	0.98	0.29	0.62	0.14	0.35	x
Hori (2004)	B1	BJ	89	542	400	20.9	0.75	0.27	0.88	0.33	0.79	0.13	0.35	o
Brooke (2006)	1B	B	31	552	800	32.0	1.00	0	0.77	2.01	0.92	0.18	0.40	o
	2B	B	41	552	800	32.0	1.00	0	0.70	1.29	0.67	0.17	0.51	x
	3B	BJa	45	537	675	27.0	1.00	0	0.75	1.13	0.80	0.18	0.42	o
	4B	BJa	43	537	675	27.0	1.00	0	0.74	1.19	0.81	0.12	0.36	o
Umemura (2006)	PJN	BJ	76	554	500	22.5	1.00	0.15	0.94	0.87	0.83	0.11	0.35	o
Kimoto (2006)	I6C	BJa	71	518	400	20.9	1.00	0.20	0.87	0.39	0.76	0.09	0.28	o
	I6P	BJa	71	518	400	20.9	1.00	0.20	0.87	0.39	0.80	0.11	0.29	o
	I1P	BJ	105	518	400	20.9	1.00	0.20	0.97	0.33	0.81	0.15	0.35	o
Yagenji (2009)	JU-S	BJa	55	541	400	20.9	0.67	0.10	0.91	0.55	0.63	0.03	0.26	x
Li (2015)	NS1	B	60	510	450	37.5	1.00	0	1.06	0.83	0.90	0.33	0.43	o
	NS2	B	61	508	450	28.1	0.50	0	0.65	0.58	0.92	0.13	0.24	o
	NS3	B	61	508	450	28.1	1.00	0	0.76	0.82	0.95	0.29	0.42	o
	NS4	B	60	513	450	22.5	0.64	0	0.53	0.64	0.87	0.09	0.28	o
	AS1	B	61	510	450	37.5	1.00	0.30	1.02	0.82	0.87	0.13	0.44	o
	AS2	B	61	508	450	28.1	0.50	0.30	0.55	0.58	0.64	0.00	0.09	x
	AS3	B	61	508	450	28.1	1.00	0.30	0.73	0.82	0.78	0.13	0.42	o
	AS4	B	62	513	450	22.5	0.64	0.30	0.53	0.62	0.69	0.04	0.37	x
Alaee (2017)	IN80	B	80	564	450	28.1	0.50	0	0.45	0.99	0.79	0.09	0.29	o
	IN100	B	100	564	450	28.1	0.50	0	0.40	1.01	0.83	0.21	0.36	o

Failure mode: "B", beam flexural failure; "BJ", Beam yielding followed by Joint shear failure; "BJa" Beam yielding followed by Joint anchorage failure.

[a] Experimental-to-nominal joint strength ratio per ACI 318.

[b] Provided-to-required confinement reinforcing ratio per ACI 318 without limiting f_{yt}.

[c] Joints with hysteresis performance satisfying the three acceptance criteria (Fig. 4) given by ACI 374.1-05 are evaluated to be "Acceptable" and denoted by symbol "o". The rest joints are evaluated to be "Unacceptable" and denoted by symbol "x".

Finally, this paper presents the evaluation results of 61 cruciform beam-column joints reinforced with beam bars of Grade 490 MPa or higher, as listed in Tables 2, 3, and 4. Only nine specimens had three cycles at a drift ratio of 3.5% or 4%. The details and cyclic loading response for each beam-column joint are summarized elsewhere (Chen 2017).

4.2 Joints with Grade 490 or 500 MPa Beam Bars

Table 2 shows the evaluation of the data subset of joints with Grades 490 or 500 MPa beam bars. The hysteresis loops of these tests were obtained from references (Teraoka and Kanoh 1994; Nakachi and Tabata 1995; Hosoya et al. 2003; Maruta and Sanada 2004; Hori et al. 2004; Brooke et al. 2006; Umemura et al. 2006; Kimoto et al. 2006; Yagenji et al. 2009; Li and Leong 2015; Alaee and Li 2017) and evaluated according to the ACI standards. The joints having hysteresis performance that fully satisfy the three criteria of strength, stiffness, and energy dissipation ($Q_r \geq 0.75Q_m$, $K_o \geq 0.05K_i$, and $E_D \geq 0.125E_{pp}$) given by ACI 374.1-05 (ACI Committee 374 2005) are evaluated to be "acceptable", while the others are evaluated as "unacceptable". Figure 4 illustrates an unacceptable 4% drift cycle for a cruciform beam-column

Table 3 Key parameters of the joints with Grade 590 MPa (85 ksi) beam bars.

First author	Specimen	Failure mode	Test parameters								Performance evaluation[c]			
			f_c' MPa	f_{ya} MPa	h_c mm	$\frac{h_c}{d_b}$	$\frac{A_{s,bot}}{A_{s,top}}$	$\frac{P}{A_g f_c'}$	$\frac{V_{jh\cdot m}}{V_n}$[a]	$A_{sh,ratio}$[b]	$\frac{Q_r}{Q_m}$	$\frac{K_o}{K_i}$	$\frac{E_D}{E_{PP}}$	Rating
Oka (1992)	J-1	BJ	81	638	300	23.6	0.78	0.11	1.15	0.26	0.70	0.12	0.28	x
Teraoka (2004)	HJ-5	BJ	54	645	400	20.9	1.00	0.18	0.92	0.66	0.76	0.07	0.25	o
	HJ-8	BJ	93	599	400	18.0	1.00	0.20	0.97	0.27	0.91	0.10	0.33	o
	HJ-12	BJ	89	604	400	18.0	1.00	0.17	1.55	0.58	0.71	0.10	0.35	x
Abe (2006)	MJIS	BJ	86	626	475	21.4	1.00	0.15	0.98	0.55	0.80	0.11	0.36	o
Hori (2006)	B15-4	B	152	616	380	19.9	0.67	0.33	0.74	0.70	0.96	0.22	0.40	o
	B15-5	B	146	616	380	19.9	0.67	03	0.75	0.73	0.93	0.22	0.36	o
Takamori (2007)	HNO. 18	BJ	147	646	700	24.5	1.00	0.10	0.96	0.62	0.96	0.14	0.31	o
	HNO. 19	BJ	165	657	700	24.5	1.00	0.10	1.04	0.57	0.93	0.19	0.34	o
	HNO. 20	BJ	140	640	700	24.5	1.00	0.10	1.10	0.68	0.93	0.18	0.35	o
Hwang (2014)	C2-600	BJa	32	710	550	24.8	0.60	0	0.89	1.70	0.71	0.03	0.15	x
	C3-600	BJa	32	710	450	20.3	0.60	0	1.09	1.46	0.73	0.11	0.19	x
	C4-600	BJa	30	635	550	21.7	0.50	0	0.79	1.81	0.73	0.06	0.17	x

Failure mode: "B", beam flexural failure; "BJ", Beam yielding followed by Joint shear failure; "BJa" Beam yielding followed by Joint anchorage failure.

[a] Experimental-to-nominal joint strength ratio per ACI 318.

[b] Provided-to-required confinement reinforcing ratio per ACI 318 without limiting f_{yt}.

[c] Joints with hysteresis performance satisfying the three acceptance criteria (Fig. 4) given by ACI 374.1-05 are evaluated to be "Acceptable" and denoted by symbol "o". The rest joints are evaluated to be "Unacceptable" and denoted by symbol "x".

joint with bond deterioration initiating in the prior 3% drift cycles.

For various failure modes, the relations of the h_c/d_b ratio, measured concrete compressive strength f_c', and evaluated hysteresis performance are shown in Fig. 5, wherein the symbols "o" and "x" denote the "acceptable" and "unacceptable" test results, respectively. The proposed minimum h_c/d_b ratios of Eqs. (1) and (5) for Grade 490 MPa are plotted for comparison. Ideally, the test data having h_c/d_b ratios above the minimum joint depth should demonstrate acceptable performance unless other design parameters do not conform to the code. In contrast, the test data having h_c/d_b ratios below the minimum joint depth would exhibit unacceptable performance.

From the comparison shown in Fig. 5, the minimum h_c/d_b ratio from Eq. (1) is obviously too conservative for test data with $f_c' \geq 70$MPa. Comparatively, the proposed Eq. (5) is less conservative. Most of the test data having h_c/d_b ratios above the curve of Eq. (5) are evaluated as acceptable. Three "B" failure data [Table 2, Specimen 2B (Brooke et al. 2006); Specimens AS2 and AS4 (Li and Leong 2015)] performed well up to 3% drift but the strengths rapidly degraded in the later drift cycles due to buckling of the beam bars in the plastic hinge regions. Therefore, the unacceptable evaluation of three "B" failure specimens can be primarily attributed to beam flexural failure rather than bond deterioration.

Excluding "B" failure specimens, the remaining four unacceptable data had h_c/d_b ratios slightly above the curve of Eq. (5). Figure 6 shows the reproduced cyclic responses of Specimens NO. 5 and NO. 6 (Nakachi and Tabata 1995), Specimen NO. 1 (Hosoya et al. 2003), and Specimen JU-S (Yagenji et al. 2009). The strength and stiffness of Specimen NO. 6 gradually degraded in 2%, 3%, and 4% drift cycles. This degradation can be attributed to the excessive joint shear stress ($V_{jh,m}/V_{n,ACI} = 1.18$) of Specimen NO. 6, as listed in Table 2. Note that either joint shear failure or bond deterioration would result in hysteretic strength and stiffness degradation; however, the joints with bond deterioration would typically display very low residual stiffness at small displacement (very low K_o/K_i). As shown in Fig. 6 and Table 2, Specimens NO. 5, NO. 1, and JU-S exhibited very low K_o/K_i values in the evaluation of the 4% drift cycle because of the bond deterioration initiated in the previous 3% drift cycles. Since the limiting drift cycle evaluated in this study is 4% drift, which is slightly beyond the 3.5% drift given by ACI 374.1-05 (ACI Committee 374 2005), the performance of Specimens NO. 5, NO. 1, and JU-S could be considered as marginally acceptable.

Notably, four "BJa" failure specimens [Specimens 3B and 4B (Brooke et al. 2006); Specimens I6C and I6P (Kimoto et al. 2006)] are still evaluated as acceptable (Table 2) at the 4% drift cycle, in which the excessive bar slip or joint core anchorage failure has not occurred. In

Table 4 Key parameters of the joints with Grade 690 MPa (100 ksi) beam bars.

First author	Specimen	Failure mode	Test parameters								Performance evaluation[c]			
			f'_c MPa	f_{ya} MPa	h_c mm	$\frac{h_c}{d_b}$	$\frac{A_{s,bot}}{A_{s,top}}$	$\frac{P}{A_g f'_c}$	$\frac{V_{jh\text{-}m}}{V_n}$[a]	$A_{sh,ratio}$[b]	$\frac{Q_r}{Q_m}$	$\frac{K_o}{K_i}$	$\frac{E_D}{E_{PP}}$	Rating
Noguchi (1992)	OKJ-1	BJ	70	718	300	23.6	0.78	0.12	1.21	0.47	0.59	0.12	0.31	x
	OKJ-4	BJ	70	718	300	23.6	0.78	0.12	1.22	0.95	0.73	0.13	0.33	x
Watanabe (2005)	NO. 2	B	107	724	400	20.9	1.00	0.15	1.15	1.01	0.93	0.20	0.33	o
	NO. 3	BJ	107	724	400	20.9	1.00	0.15	1.48	1.01	0.85	0.17	0.39	o
	NO. 4	BJ	157	724	400	20.9	1.00	0.15	1.28	0.69	0.91	0.20	0.38	o
	NO. 5	B	107	748	400	18.0	1.00	0.15	1.18	1.01	0.86	0.14	0.37	o
Hori (2006)	B15-1	BJ	189	690	380	19.9	0.67	0.23	0.66	0.21	0.56	0.08	0.55	x
	B15-3	BJ	186	690	340	17.8	1.00	0.23	0.74	0.19	0.65	0.09	0.48	x
Kuo (2011)	X100	BJ	120	744	600	23.6	0.50	0.03	0.86	0.97	0.94	0.18	0.20	o
Lee (2016)	CG1	BJ	81	738	600	23.6	0.67	0.05	0.86	0.65	0.82	0.16	0.21	o
	CG3	BJ	85	738	600	23.6	0.67	0.05	0.86	0.62	0.90	0.18	0.20	o
	CG4	BJ	83	738	600	23.6	0.67	0.05	0.91	0.64	0.91	0.21	0.20	o
Lee (2014)	A24	BJ	116	725	600	23.6	0.75	0.04	0.94	0.86	0.78	0.13	0.25	o
	B24	BJ	104	725	600	23.6	1.00	0.04	1.11	0.95	0.81	0.13	0.25	o
Alaee (2017)	IH80	B	80	712	450	28.1	0.50	0	0.62	0.97	0.96	0.20	0.25	o
	IH80A	B	80	712	450	28.1	0.50	0.30	0.62	0.97	0.86	0.36	0.46	o
	IH100	B	100	712	450	28.1	0.50	0	0.54	1.01	0.95	0.22	0.28	o
	IH60	BJa	60	707	450	23.7	1.00	0	0.60	1.29	0.80	0.04	0.22	x
	IH60A	BJa	60	707	450	23.7	1.00	0.30	0.56	1.29	0.77	0.04	0.18	x

Failure mode: "B", beam flexural failure; "BJ", Beam yielding followed by Joint shear failure; "BJa" Beam yielding followed by Joint anchorage failure.

[a] Experimental-to-nominal joint strength ratio per ACI 318.

[b] Provided-to-required confinement reinforcing ratio per ACI 318 without limiting f_{yt}.

[c] Joints with hysteresis performance satisfying the three acceptance criteria (Fig. 4) given by ACI 374.1-05 are evaluated to be "Acceptable" and denoted by symbol "o". The rest joints are evaluated to be "Unacceptable" and denoted by symbol "x".

other words, the "BJa" failure could be acceptable if the failure drift ratio is large enough.

4.3 Joints with Grade 590 or 600 MPa Beam Bars

To date, no experiment has been conducted for beam-column joints with ASTM A706 Grade 550 MPa reinforcement, for which the specified yield strength ranged from 550 to 675 MPa. Therefore, the available test data of beam-column joints with Grade 590 or 600 MPa beam bars were collected from literature (Oka and Shiohara 1992; Teraoka et al. 2004; Abe et al. 2006; Hori et al. 2006; Takamori et al. 2007; Hwang et al. 2014) and assembled in Table 3, where 11 out of 13 data had actual yield strength f_{ya} meeting the specification of ASTM A706 Grade 550 MPa. Thus, this subset of the database could be referred for the application of ASTM A706 Grade 550 MPa bars.

Table 3 and Fig. 7 show the relations of the evaluated hysteresis performance and the key test parameters. Obviously, the minimum h_c/d_b ratio of 28 obtained from Eq. (1) is too conservative for the eight acceptable data with $f'_c \geq 55\text{MPa}$ and $h_c/d_b \leq 25$. Notably, although the three "BJa" failure specimens tested by Hwang et al. (2014) also used h_c/d_b ratios ranging from 20 to 25, they did not perform well within 3.5% drift cycles because of the low f'_c value of 32 MPa. The last two data [Specimen J-1 (Oka and Shiohara 1992); Specimen HJ-12 (Teraoka et al. 2004)] had significant strength losses at the 4% drift cycle due to excessive shear stress and insufficient transverse reinforcement in the joint (Table 3). Excluding Specimens J-1 and HJ-12, only three "BJa" failure specimens had h_c/d_b ratios much smaller than that from Eq. (5) and exhibited unacceptable hysteresis performance. However, there are no test data with lower f'_c values and larger h_c/d_b ratios in Fig. 7, neither for higher f'_c values with smaller h_c/d_b ratios. Therefore, it is difficult to conclude whether the proposed Eq. (5) is conservative for various concrete strengths. More beam-column joint tests are needed for these areas.

4.4 Joints with Grade 690 MPa Beam Bars

The available test data of beam-column joints with Grade 690 MPa (100 ksi) beam bars are collected from

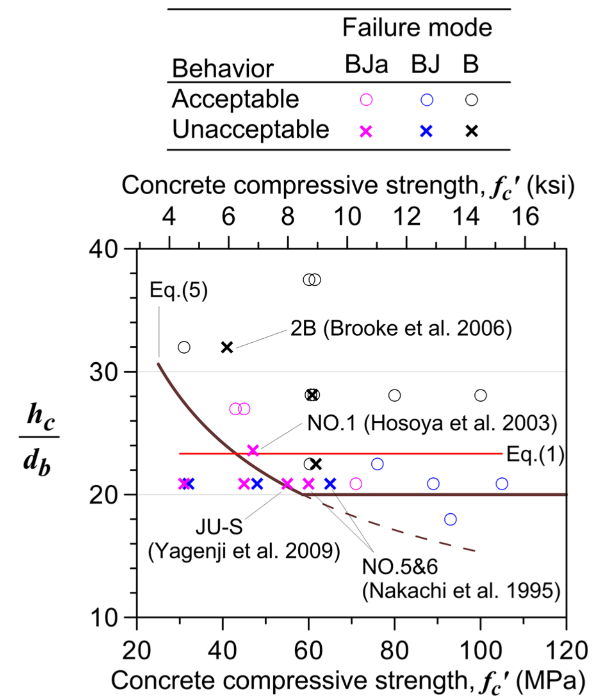


Fig. 5 Relations of the h_c/d_b ratio, measured concrete compressive strength, and evaluated hysteresis performance for joints with Grade 490 or 500 MPa beam bars.

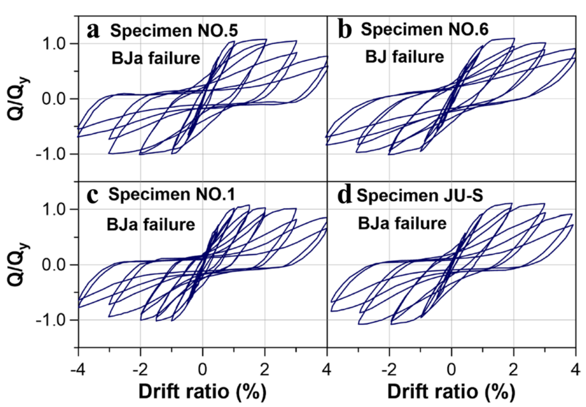


Fig. 6 Normalized column shear versus story drift for beam-column joints with h_c/d_b ratios close to the proposed equation: **a** Specimen NO. 5; **b** Specimen NO. 6 tested by Nakachi and Tabata (1995); **c** Specimen NO. 1 tested by Hosoya et al. (2003); and **d** Specimen JU-S tested by Yagenji et al. (2009).

literature (Noguchi and Kashiwazaki 1992; Watanabe et al. 2005; Hori et al. 2006; Kuo 2011; Lee et al. 2014, 2016; Alaee and Li 2017) and listed in Table 4. All the specimens were made with higher strength concrete. Figure 8 shows the relations of the evaluation results to the h_c/d_b ratio and measured concrete compressive strength f_c'. Definitely, the minimum joint depth from Eq. (1) is far above the acceptable test results with smaller h_c/d_b ratios. As listed in Table 4, six specimens did not perform well at the limiting drift ratio. Specimens OKJ-1 and OKJ-4 (Noguchi and Kashiwazaki 1992) were subjected to excessive joint shear stress ($V_{jh,m}/V_{n,aci} \geq 1.2$). Thus, the lateral strength degraded rapidly due to the occurrence of joint shear failure. Specimens B15-1 and B15-3 (Hori et al. 2006) used ultra-high strength concrete for the joint but the amounts of transverse reinforcement ($A_{sh,ratio}$, Table 4) were far below the code requirement. Thus, both joints failed in shear with a stress level below the nominal joint shear strength. Specimens IH60 and IH60A (Alaee and Li 2017) demonstrated severely pinched hysteresis loops due to the unmatched use of 60 MPa concrete with a joint depth of 24 d_b, which is less than the minimum joint depth obtained from Eq. (5) for Grade 690 MPa beam bars.

On the other hand, the first author and his colleagues (Kuo 2011; Lee et al. 2014, 2016) also tested several cruciform beam-column joints at the laboratory of the National Center for Research on Earthquake Engineering (NCREE) in Taiwan. As shown in Fig. 9, each joint specimen had a unit column length of 3.2 m and a unit beam length of 6 m to simulate an interstory beam-column joint under reversed cyclic loading (Fig. 1b). The test specimens satisfied most of the ACI 318 (2014) and ACI 352R-02 (ACI-ASCE Committee 352 2002) seismic provisions for special moment frames, except the limitations on the joint shear stress, bar f_y, and h_c/d_b ratio. As listed in Table 4, six "BJ" failure specimens (X100, CG1, CG3, CG4, A24, and B24) were reinforced with 25-mm Grade 690 MPa beam bars passing through a joint depth of 600 mm, resulting in a h_c/d_b ratio of 24. The design concrete strength was 100 MPa but the measured concrete compressive strength ranged from 80 to 120 MPa. Figure 10 shows the normalized cyclic loading response of Specimens X100, CG1, A24, and B24 with various ratios of $A_{s,bot}/A_{s,top}$ and $V_{jh,m}/V_{n,aci}$. The cyclic responses of Specimens CG3 and CG4 are not shown here, because they were almost identical to that of Specimen CG1 with the same reinforcing details, except the use of mechanical couplers for bottom beam bars. As demonstrated in Fig. 10, all tested specimens performed well up to the 4% drift ratio, although eventually they failed in joint shear in the later 6% drift cycles. Notably, the residual stiffness around zero drift of the 4% drift cycles were not too low. Excessive bar slip was not observed in these experiments within 4% drift cycles.

Based on Figs. 8 and 10, the proposed Eq. (5) seems adequate for concrete strength f_c' ranged from 80 to 120 MPa, but it may be inadequate for higher strength

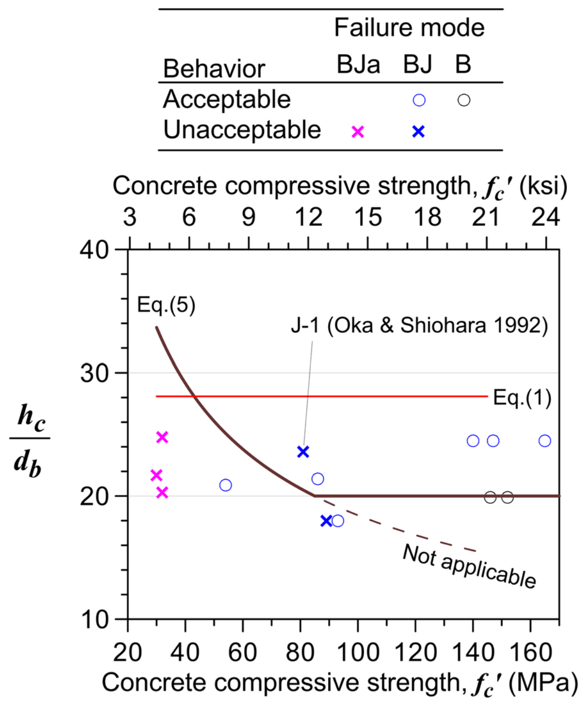


Fig. 7 Relations of the h_c/d_b ratio, measured concrete compressive strength, and evaluated hysteresis performance for joints with Grade 590 or 600 MPa beam bars.

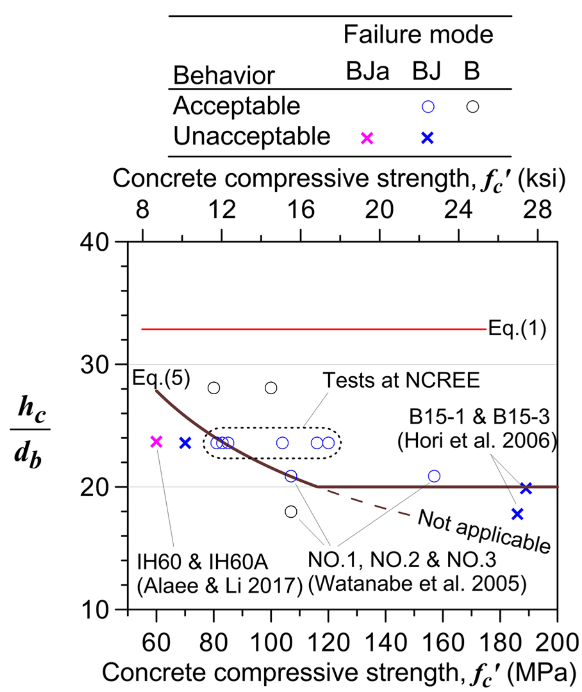


Fig. 8 Relations of the h_c/d_b ratio, measured concrete compressive strength, and evaluated hysteresis performance for joints with Grade 690 MPa beam bars.

Fig. 9 Typical test setup for cruciform beam-column joints at the NCREE laboratory.

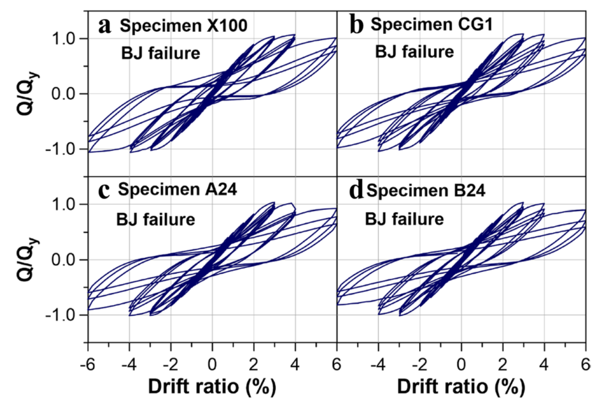
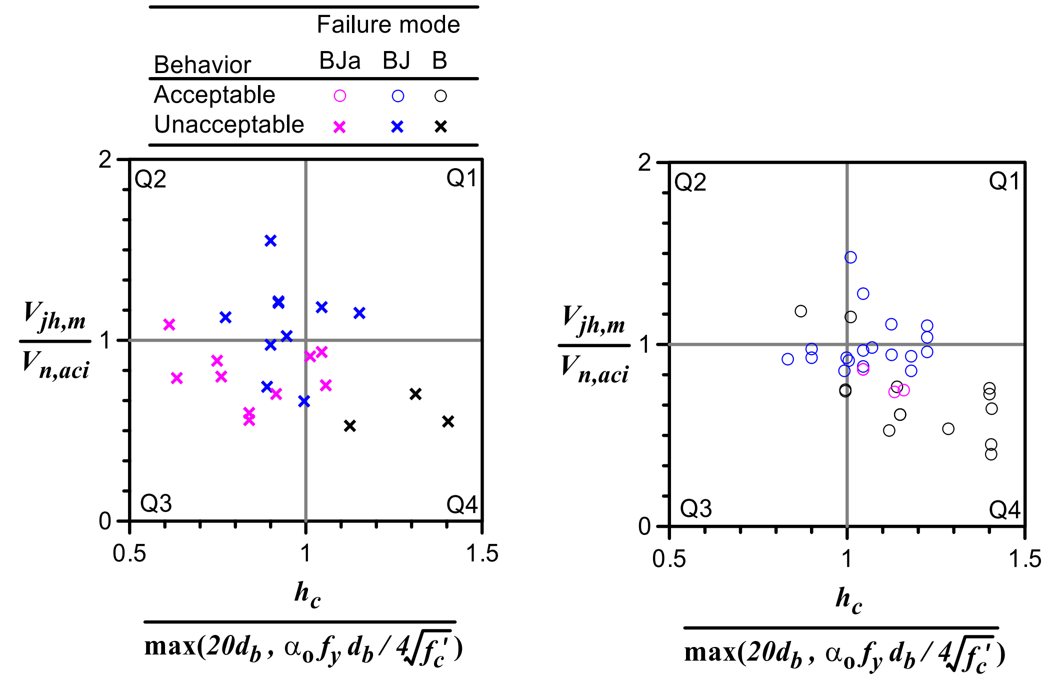


Fig. 10 Normalized column shear versus story drift for beam-column joints tested at NCREE with $h_c/d_b = 24$.

concrete, where the bond strength of ultra-high strength concrete is still questionable. Due to the limited test data with $h_c/d_b \leq 20$, this study also suggests to set a lower limit of 20 for Eq. (5). In other words, this study recommends the minimum joint depth to be the larger of 20 d_b and $\alpha_o f_y d_b \Big/ \left(4\sqrt{f_c'}\right)$. This recommendation is conservative for cruciform beam-column joints reinforced with reinforcement grades not exceeding 690 MPa.

4.5 Overall Observation

For all the test data listed in Tables 2, 3, and 4, Fig. 11 displays the data distributions of unacceptable and acceptable data with respect to the ratio of experimental-to-nominal shear strength and the provided-to-required column dimension ratio (i.e., the provided column depth divided by the minimum joint depth). Attention shall



a *Unacceptable data* **b** *Acceptable data*

Fig. 11 Relations of performance evaluation to the shear strength ratio and provided-to-required joint depth ratio.

be drawn to the test data that fall in Quadrants 3 and 4, where $V_{jh,m}/V_{n,aci} \leq 1.0$ means the experimental shear stresses are below the nominal value of 1.25 $\sqrt{f_c'}$ MPa specified in ACI 318-14 (2014), and thus, the premature joint shear failure data were precluded. The horizontal axis of Fig. 11 represents the ratio of the column depth provided in experiments to the proposed minimum joint depth. Therefore, the test data in Quadrant 4 of Fig. 11 are requirement-conforming joints and are expected to perform well up to 4% drift ratio. Ideally, the joints with excessive bar slip or anchorage failure should not appear in Quadrant 4 of Fig. 11a, where only three BJa-failure specimens (Specimens NO. 5, NO. 1, and JU-S, as shown in Fig. 6) had marginally-acceptable hysteresis performance. The other three "x" data in Quadrant 4 of Fig. 11a are B-failure specimens (Specimens 2B, AS2, and AS4 listed in Table 2), which had significant strength loss at the limiting drift cycle due to bar buckling in the beam hinging zone. In other words, these three B-failure data can be precluded for bond assessment.

Notably, the recommended minimum h_c/d_b ratio is smaller and less conservative than those required by other design codes listed in Table 1. However, most of the test data that fall in Quadrant 4 of Fig. 11 had acceptable hysteresis performance, indicating that the recommended minimum h_c/d_b ratio could be acceptable, on the basis of the present database investigation. Notably, setting the basic bond strength $u_b = 1.25\sqrt{f_c'}$ MPa as suggested by Brooke and Ingham (2013) for Eq. (4) could give a relatively conservative h_c/d_b ratio for the presented data.

A larger h_c/d_b ratio and a lower joint shear stress (Quadrant 4 of Fig. 11b) can ensure better performance of joints under cyclic loading. Higher joint shear stress may result in premature joint shear failure, which may cause building collapse, and therefore should be avoided. A smaller h_c/d_b ratio may result in excessive bar slip in the joint core and pinching hysteretic performance, but it would not lead to any local collapse. Therefore, the minimum h_c/d_b ratios specified for special moment frames by codes and standards are based on the judgment of the expected hysteresis behavior at a design interstory drift.

5 Summary and Recommendations

This paper reviews existing design criteria in different codes for the minimum joint depth of special moment frames and proposes a simplified equation by omitting some minor variables. The applicability of the proposed equation is verified against a database assembling available experiments on the use of higher grade reinforcement in reinforced concrete beam-column joints subjected to quasi-static reversed cyclic loading. The hysteresis behavior of each beam-column joint was evaluated according to the ACI standards for special moment frames. Based on this review and database assessment, the following conclusions and recommendations can be drawn:

- Relatively pinched hysteresis behavior can be observed in the beam-column joints with bond deterioration along the beam bars passing through the joint. Such damage in the joint core is unlikely to be easily repairable and therefore should be avoided in a design basis earthquake event.
- The ACI 318 (2014) requirement of a minimum joint depth of 20 d_b is based on test data with Grade 420 reinforcement and may be too short for bar f_y exceeding 420 MPa. ACI 352R-02 (ACI-ASCE Committee 352 2002) recommends a simple multiplier of $f_y/420$ on the 20 d_b criterion without accounting for the various concrete strength, which is very conservative for high-strength concrete and unconservative for low-strength concrete, according to the presented database investigation.
- Based on the evaluation of the assembled test data, this study recommends that the joint depth, or the column dimension in parallel to the beam bars extending through the joint should not be less than the larger of 20 d_b and $\alpha_o f_y d_b/\left(4\sqrt{f_c'}\right)$. Cruciform beam-column joints with column dimension meeting this criterion can demonstrate acceptable hysteresis performance up to a limiting drift ratio of 3.5% at least.
- The proposed design equation is empirical and should be used with limitations of f_y not exceeding 690 MPa and f_c' not exceeding 100 MPa.
- The simplified design equation is proposed for typical interior beam-column joints with column axial load exceeding 0.15 $A_g f_c'$. If the seismic forces lead to significant variation of the column axial load, the detailed design equations including the effects of column axial load and other parameters should be used.

Abbreviations

A_g: Gross sectional area of column; A_s: Area of the bar group $A_{s,top}$ or $A_{s,bot}$ containing the bar for which the development length is being calculated; $A_{s,bot}$: Area of bottom beam bars; $A_{s,top}$: Area of top beam bars; $A_{sh,ratio}$: Provided amount of joint transverse reinforcement divided by the amount required in ACI 318; d_b: Diameter of largest beam longitudinal bars extending through the joint; E_D: Area of hysteresis loop or energy dissipated in the limiting drift cycle; E_{PP}: Area of the elastoplastic loop for the limiting drift cycle; f_c': Compressive strength of concrete; f_s: Tensile stress in reinforcement; f_s': Compressive stress in reinforcement; f_y: Minimum specified yield strength of reinforcement, or reinforcement grade; f_{ya}: Actual yield strength of longitudinal reinforcement; f_{yt}: Yield strength of transverse reinforcement; h_c: Column depth or joint depth; K_o: Secant stiffness around zero drift, obtained for positive and negative loading directions between ± 1/10 of the limiting drift ratio in the hysteresis loop; K_i: Initial secant stiffness of the first drift cycle; L_b: Unit beam length of test module, or distance between the assumed inflection points; L_c: Unit column height of test module, or assumed story height; P: Axial compression on the column; Q: Lateral load or column shear force of test module; Q_m: Maximum lateral load of test module; Q_r: Peak lateral load of test module in the repeated cycle of the limiting drift ratio; Q_y: Yield strength of test module with beam yielding at both joint faces, determined using actual bar yield strength, measured concrete compressive strength, and nominal beam moment strength at the joint face; u_b: Basic bond strength on the beam bar over the column depth; V_b: Beam shear force; $V_{jh,m}$: Experimental shear force acting on the joint, back-calculated using the force couples in the beam resisting Q_m; $V_{n,aci}$: Nominal joint shear strength per ACI 318; α_f: 0.85 for bi-directional loading; 1.0 for uni-directional loading; α_o: Overstrength factor of the beam bars; α_p: Factor accounting for the effect of column axial compression on the bond strength; α_s: $1+\kappa$, stress gradient factor accounting for the bar stress developed at the joint faces; α_t: 0.85 for a top beam bar where more than 300 mm of fresh concrete is cast below the bar; 1.0 for all other cases; δ: Beam deflection; Δ: Interstory drift; θ: Drift ratio, or angular rotation between the beam and column centerlines at the joint; κ: Ratio of the bar compressive stress to bar tensile stress developed at the joint faces.

Authors' contributions

HJ reviewed all the data analyzed by HC and then wrote the manuscript. TC helped to review Japanese articles. All authors read and approved the final manuscript.

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Acknowledgements

The authors are grateful for the financial support from the Ministry of Science and Technology in Taiwan.

Competing interests

The authors declare that they have no competing interests.

Availability of data and materials

All data analyzed during this study are available in the Ph.D. thesis of Dr. H.-C. Chen at National Yunlin University of Science and Technology in 2017.

Funding

Ministry of Science and Technology of Taiwan, Grant No. MOST 105-2625-M-224-001-.

Publisher's Note

Received: 27 March 2018 Accepted: 22 August 2018
Published online: 20 November 2018

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