EVALUATING SHEAR STRENGTH OF RC INTERIOR WIDE BEAM COLUMN JOINTS UNDER SEISMIC ACTION

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ABSTRACT

In this paper, applicability of the RC beam-column joint nominal shear strength specified in codes of practice, including ACI 318-08, NZS 3101and EN 1998-1, to the interior wide beam-column joints is evaluated. This is particularly important because the code provisions were only developed for conventional beam-column joints that should be very different from those in a wide beam system.

The results of all past published experimental studies on interior wide beam-column joints are collected. The measured joint shear force for all tested specimens are calculated by three codes provision and results are compared with the related specific codes limitation to the joint nominal shear strength. It is shown that the actual joint shear force exceeds the ACI 318-08 limit in most cases, but these joints do not encounter shear failure or shear strength degradation within the joint core. This indicates that the limit specified in ACI318-08 is conservative when applied to wide beam-column joints. A model is proposed based on the strut-and-tie concept to consider the effect of wide beam on the shear capacity of the joint. The joint nominal strengths are calculated using the ACI approach with the proposed effective joint area and then compared with the experimental results. Good agreements are found. The impact of this modification on ACI approach is efficiently on the accurate design of the wide beam-column joints under strong seismic actions.

INTRODUCTION

Adopting a wide beam system for the design scheme provides many advantages from both structural and architectural points of view. Compared to the flat slab floor system, it provides larger spans and cantilevering structures, allowing larger column-free spaces to provide more flexibility for the framing or partitioning of the completed building as well as greater architectural freedoms (LaFave and White 1999). Several examples are noted in which wide beams have been used [Figure 1].

While the design and construction practice of wide beam system have been proven to be efficient and cost effective and the past experience demonstrates a good serviceability record, the resistance of the beam–column joints in this structural system against lateral earthquake load is believed to be largely inadequate. Research focusing specifically on their seismic behaviour is very limited. Even studies dealing with the existing ones that were designed under earlier seismic codes are scarcer.

Current codes of practice have been adopted the same design methodologies for conventional and wide beam-column assemblages except the geometric restriction. They may be over-simplified the actual structural behaviour, such as the complex load transfer mechanisms at regions adjacent to the joint cores and the effect of disturbed stress in this region (D region) on the overall performance, which have been ignored. Since the code provisions were developed for conventional beam-column joints that are much different in dimensions and reinforcement layout, the applicability of these codes to design the wide beam-column joints are questionable.

Figure 1. Different use of wide beam-column in rigid frame systems

In ACI-318-08, the beam width is limited to $b_c + 1.5h_c$, where b_c is the column width and h_c is the depth of column. In New Zealand, the beam width is restricted to the lesser of $b_c + 0.5h_c$ and $2b_c$. In Eurocode 8, the width of primary beams should be smaller than the lesser of b_c+h_b and $2b_c$, where h_b is the beam depth. It can be seen that the restrictions of beam width vary with codes of practice, mainly because of the scattering experimental results (Luk 2013). There are a lot of ambiguity in codes of practice regarding to design and detail of the wide beam frame systems. Two important related issues are how to calculate the beam effective width in bending and how to estimate the joint effective width in shear. To calculate the joint nominal shear strength, the first step is to know the effective area of the joint. Both NZS and EC8 consider the joint effective width in shear equal to the beam effective with in bending (Table 2), while ACI considers the bigger effective width for beam in bending and smaller effective width for joint in shear and ignored the contribution of beam to carry the shear force.

This paper aims at investigating the applicability of the joint nominal shear strength specified in codes of practice, including ACI 318-08, NZS 3101and EN 1998-1 (EC8) for the interior wide beam-column joints.

BACKGROUND OF WIDE BEAM-COLUMN JOINTS

A number of experimental studies have been carried out on RC wide beam-column joints. However, the performance of wide beam-column joints subjected to seismic actions is not well understood since only limited experimental results are available currently as compared with those of conventional frame structures (Hatamoto et al. 1991; Popov et al., 1992; Gentry and Wight 1994; LaFave and White 1999; Benavent-Climent 2007; Benavent-Climent et al., 2009; Li and Kulkarni 2010; Elsori and Harajili 2013).

A database of previous experimental tests on wide beam-column joints have been collected from the literature and presented in Table 1. The main design parameters, applied axial load and lateral load capacities for all specimens, are summarised in Table 1. In Table 1, b_c and h_c are the width and depth of the column cross-section in the loading direction; b_b, b_t and h_b, h_t are the respective width and depth of the wide beam and spandrel beam cross-section. V_{max} represent the maximum attained shear at the top of the column and V_{cal} represent the maximum expected shear calculated by flexural analysis.

Based on the reported results on interior wide beam-column connections, three major types of failure are considered: (1) beam flexural failure (BF), (2) transverse beam (spandrel beam) failure (TF), and (3) column flexural failure (CF).

JOINT NOMINAL SHEAR STRENGTH

The level of shear stress is an important factor affecting both strength and stiffness of the joint. The codes restrict the nominal shear stress depending on the compressive strength of concrete and axial load acting on the column (Kim and LaFave 2007). To prevent joint shear failure before beam hinging, according to the codes of practice, the shear strength V_j computed on a horizontal plane within the joint should be smaller than the joint nominal shear strength V_n,

Table 1. The database of previous experimental studies

No	Ref	specimen	b_c mm	h_c mm	b_b mm	h_b mm	b_t mm	h_t mm	f'_c *MPa*	P/ $A_g f_c$ %	V_{max} kN	V_{cal} kN	Failure Mode
1	Hatamoto et al., 1991	WF-2	400	400	730	250	400	250	24	20	72	73.3	BF
2		WF-3	400	400	730	250	400	250	25	20	72	73.3	BF
3		WF-4	200	800	730	250	800	250	24	20	78	73.3	BF
4		WB-2	200	200	350	125	200	125	23	0	62	66.2	TF
5		WB-2C	200	200	350	125	400	125	23	0	66	66.2	BF
6		WB-3	200	200	530	125	200	125	23	0	82	100	TF
7		WB-3C	200	200	530	125	500	125	23	0	94	100	TF
8		WB-4	200	200	710	125	200	125	23	0	87	135	TF
9	Popov 1992	UCB-1	430	430	750	225	430	225	28	40	115	115	BF
10	Quintero Weight., 1997	IWB1	350	350	875	300	350	300	35.5	0	133	117	BF
11		IWB2	350	350	650	300	350	300	27.5	0	134	113	BF
12		IWB3	325	500	825	300	500	300	25.5	0	146	134	BF
13	Siah et al., 2003	WBB-I1	250	250	1200	200	250	200	32	20	80	108	TF
14		WBB-I2	250	250	1200	200	250	200	32	20	90	108	CF
15	Nishimura et al., 2008	RC-1	240	240	500	160	240	160	34.8	0	112	100	BF
16		RC-2	240	240	500	160	240	160	34.8	0	111	100	BF
17		RC-3	240	240	500	160	240	160	34.8	0	112	100	BF
18		RC-4	240	240	500	160	240	160	34.8	0	94	75	BF
19	Benavent et al., 2010	IWB	230	230	700	165	230	165	21	10	55.7	71.4	CF
20		IL	270	270	480	180	270	180	24.9	15	72	80.6	TF
21		IU	210	210	360	180	210	180	24.9	15	36	51.1	TF
22	Li and Sudhakar 2010	IWB1	300	900	800	300	900	300	64.3	0	272	249	BF
23		IWB3	300	900	800	300	900	300	47.9	0	256	244	BF
24	Elsouri and Harajli 2013	UIJ-F1	250	700	800	250	700	250	40	1	205	168	BF
25		UIJ-F2	700	250	800	250	250	250	37	1	162	158	BF

$$V_j \le V_n \tag{1}$$

The shear demand V_j is calculated as $V_j = T_u + C_u - V_{col}$, where V_{col} is the maximum lateral load capacity attained by the joint; $T_u = A_{s(top)}\, f_{s(top)}$ (A_s is steel area and f_s is steel stress) is the measured tension force in the beam top reinforcement at peak lateral load subjected to negative bending; and C_u is the compression force developed in the beam subjected to positive bending, which is equal to the measured tension force $A_{s(bottom)}\, f_{s(bottom)}$ in the beam bottom reinforcement at the same section. The joint nominal shear strength, V_n as per different codes is given in Table 2.

Table 2. Effective width of joint, b_j

	ACI318-08	NZS	EN
V_n	$\varphi\gamma\sqrt{f'_c}A_j$	$0.2 f'_c A_j$	$\eta f'_{cd}\sqrt{1-\nu_d/\eta}\,A_j$
A_j	$b_c h_c$	$\min(b_b; 2b_c; b_c + 0.5h_c).h_c$	$\min(b_b; 2b_c; b_c + 0.5h_c).h_c$

Where φ denotes the reduction factor on shear strength and is equal to 0.85 according to ACI 318. γ is equal to *1.67* for joints confined on all four sides, *1.25* for joints confined on three faces or two opposite faces, and *1.0* for others. η=0.6(1-f_c/250) denotes the reduction factor on concrete compressive strength due to tensile strains in transverse direction in which f_c is in MPa. f_{cd} is design value of concrete compressive strength and in this paper it take as $0.67f_c$. The shear strength of exterior joints is taken as 80% of the value given by EC8. The effective joint area, A_j is the area resisting the shear within the joint and is contributed by the framing members in the considered direction of loading. The depth of the joint, h_c is taken as equal to the depth of the column. In this paper the value of φ and over strength factor are taken equal to 1.

Results of the measured joint shear force for the various specimens are presented in Table 3 in comparison with the appropriate limits of the nominal shear strength V_n specified in codes.

Table 3. Joint shear strength and joint nominal capacity

No	specimen	A_{s-} mm^2	A_{s+} mm^2	$f_{y\ beam}$ MPa	V_j kN	γ ACI	V_j/V_n ACI	V_j/V_n NZS	V_j/V_n EC8	b_j ACI	b_j NZS	b_j Ave	b_j Prop
1	WF-2	1592	1592	380	1138	1.25	1.37	0.99	0.54	400	600	565	565
2	WF-3	1592	1592	380	1138	1.25	1.34	0.95	0.52	400	600	565	565
3	WF-4	1592	1592	380	1132	1.25	1.36	0.49	0.27	200	400	465	400
4	WB-2	615	615	345	362	1.25	1.78	1.31	0.57	200	300	275	275
5	WB-2C	615	615	345	358	1.67	1.32	1.30	0.56	200	300	275	275
6	WB-3	923	923	345	555	1.25	2.72	2.01	0.87	200	300	365	300
7	WB-3C	923	923	345	543	1.67	1.99	1.97	0.85	200	300	365	300
8	WB-4	1231	1231	345	762	1.25	3.74	2.76	1.19	200	300	455	300
9	UCB-1	1609	1126	420	1321	1.67	0.95	0.85	0.75	430	645	590	590
10	IWB1	1495	1063	420	941	1.67	0.91	0.72	0.33	350	525	612	525
11	IWB2	1495	1053	420	936	1.67	1.03	0.93	0.41	350	525	500	500
12	IWB3	1816	1292	420	1159	1.67	1.00	0.79	0.35	325	575	575	575
13	WBB-I1	2260	904	444	1325	1.25	3.53	2.21	1.26	250	375	725	375
14	WBB-I2	2260	904	444	1315	1.25	3.50	2.19	1.25	250	375	725	725
15	RC-1	570	356	34.8	395	1.25	1.09	0.66	1.77	240	360	370	360
16	RC-2	570	356	34.8	396	1.25	1.10	0.66	1.77	240	360	370	360
17	RC-3	570	356	34.8	395	1.25	1.09	0.66	1.77	240	360	370	360
18	RC-4	570	356	34.8	413	1.25	1.14	0.69	1.85	240	360	370	360
19	IWB	1135	679	500	851	1.25	3.31	2.55	1.21	230	345	465	345
20	IL	1681	1018	404	1018	1.67	1.97	1.87	0.96	270	405	375	375
21	IU	1236	452	404	646	1.67	2.07	1.96	1.00	210	315	285	285
22	IWB1	3024	2160	460	2113	1.67	0.69	0.24	0.13	300	600	550	550
23	IWB3	3024	2160	460	2129	1.67	0.80	0.33	0.16	300	600	550	550
24	UIJ-F1	2400	1600	590	2107	1.25	1.79	0.63	0.30	250	500	525	500
25	UIJ-F2	2400	1600	590	1625	1.25	1.44	1.10	0.51	700	825	750	750

The correlation of the experimental joint shear strength versus the nominal joint shear strength predicted by code provisions are shown in Figure 2. It can be observed from Figure 2 that the actual joint shear force V_j exceeds the ACI 318-08 limits in most of the cases. However, despite exceeding the limits, these joints did not encounter shear failure or shear strength degradation within the joint core. This implies that the limits specified in ACI 318-08 may be conservative when applied to the wide beam. Both NZS and EC8 consider the greater effective width for joint and consequently in most of the cases joint shear strength are lower than joint capacity. An important point is that both of the NZS and EC8 could capture the torsional failure and column failure correctly. It means that for specimens with beam plastic hinge failure the result are in the safe side but for two other failure mode the prediction lead to joint shear.

In Figure 3, each plot represents storey shear strength on a vertical axis normalised by calculated storey shear from the flexural analysis, and joint shear demand normalised by the joint shear capacity in the related code on horizontal axis.

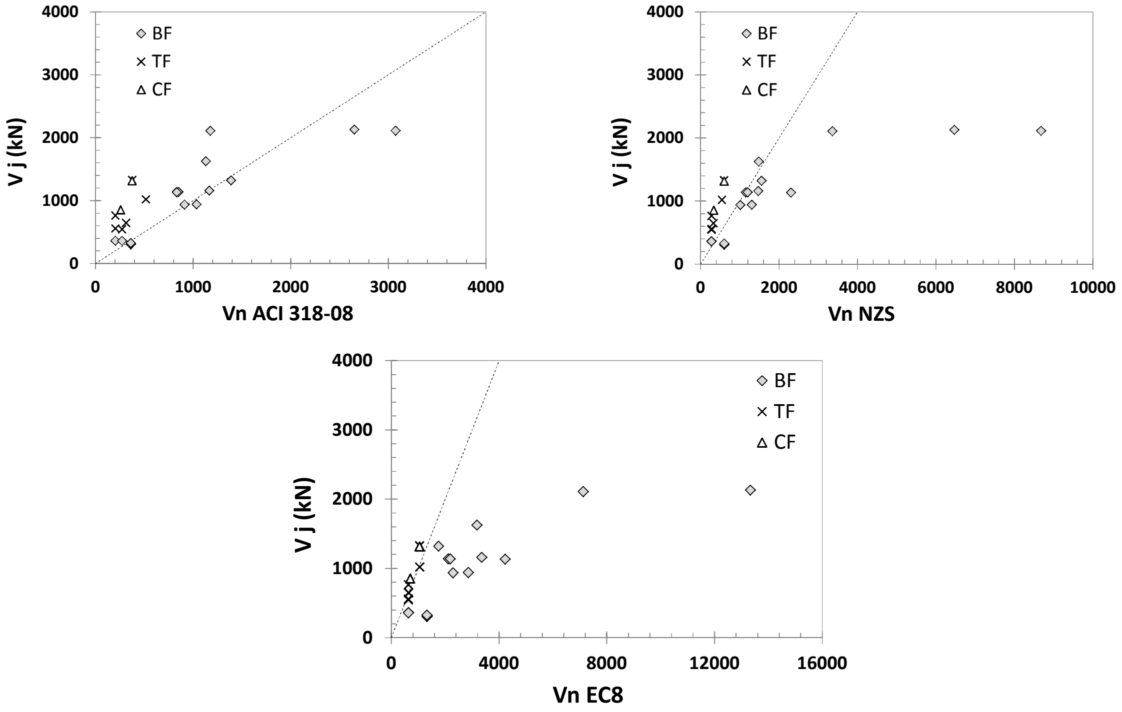

Figure 2. Joint shear strength vs joint shear capacity

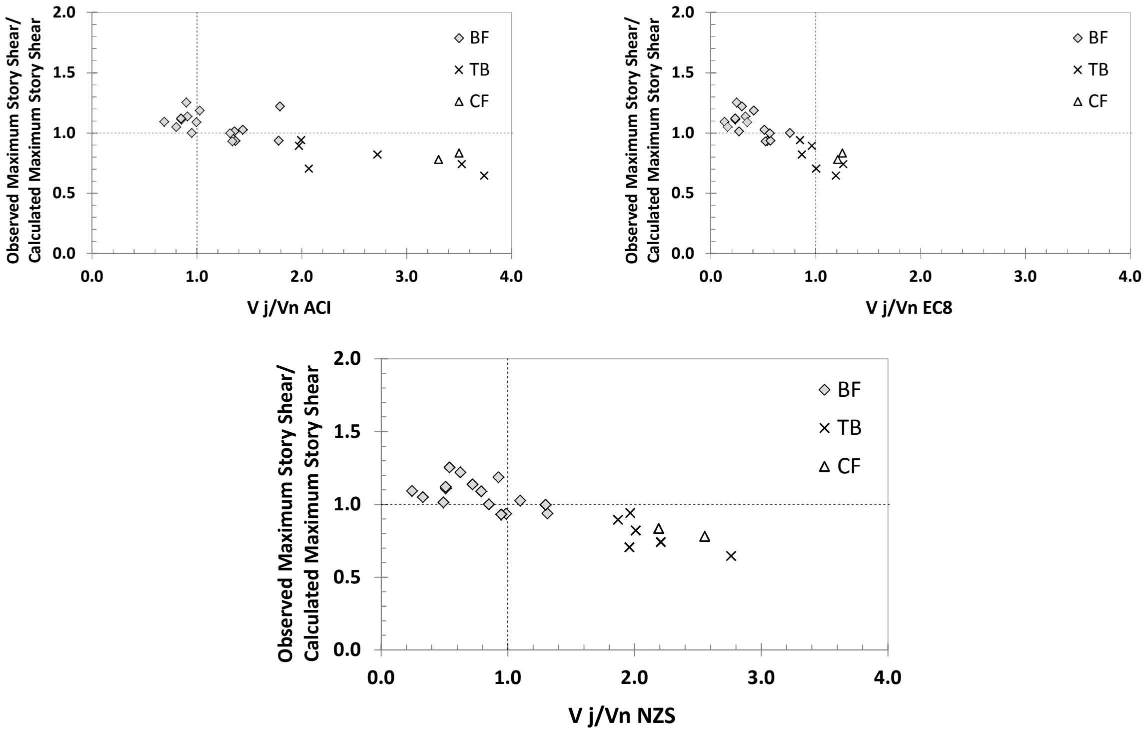

Figure 3. Normalised story shear vs normalised joint shear

From Figure 3, it is certain that even for the joint shear capacity margin larger than 1 there is no specimens with joint shear failure. But a quit number of the specimens can be found that the observed maximum storey shear is smaller than calculated maximum story shear. In some specimens, the strength fell more than 20%. The main reasons for this problem are related to type of failure. In most of these specimens, torsional failure was happened in spandrel beam and consequently part of the reinforcement in outside beam become inefficient. As noted by other researchers (Benavent and CahisVico 2009, Fardis MN 2009, and Elsouri and Harajli 2013) torsional failure results in reduction of the shear strength of the joints and consequently the whole connections.

PROPOSED ANALYTICAL MODEL

All three codes evaluate the joint nominal shear capacity based on the strut mechanism and express it as a function of concrete strength irrespective of the amount of shear reinforcement. However, the nominal shear capacity is influenced by the confinement provided by the adjoining members. The ratio of beam width b_b to the column width b_c may have some influence on the effectiveness of the joint shear area (Gentry TR1992, Li and Kulkarni 2010). From past studies it was found that wide beam construction is one way to effectively limit joint shear cracking because failure of the joint was not observed in previous tests even in the most heavily loaded joint. With respect to conventional beam-column joints, there are two ways to decrease the level of shear stress stresses in the joint. First, the beam wraps around the column at the joint, and this region participates in resisting joint shear. This effectively reduces the average joint shear stress, resulting in less joint diagonal cracking (Gentry and Wight 1994; LaFave and White 1999; Benavent-Climent et al., 2009; Li and Kulkarni 2010; Elsori and Harajili 2013). Second, the wide beam longitudinal reinforcement passing outside the column on each side transfer load through the torsion in spandrel beam. According to the "Space Truss" model, a reinforced concrete beam resists torsion by two mechanisms: concrete struts and steel ties. This means that part of the torsional moment, which changes to the tensile force in the spandrel beams longitudinal steel, reduces the applied shear stress in the joint. Furthermore, strain gage data from past experimental tests indicate that the area resisting the joint shear forces is larger than the column section. This suggests that the joint shear requirements could be relaxed. Thus, the effective width of joint is shown in Figure 5 and is given by:

$$b_j = \min(b_b; 2b_c; b_c + 0.5h_c; (b_c + b_b)/2) \qquad (2)$$

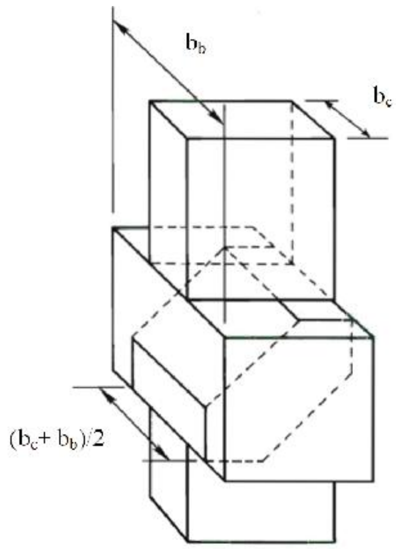

Figure 4. Proposed model for interior wide beam-column joint

This definition of the effective joint width to calculate the ultimate shear strength is based on the same concept of those in the ACI code for concrete joint with column wider than the beam. It is based on the strut and tie concepts .The shear capacity of specimens were calculated by the ACI equation and proposed effective area. In table 2, the values of $(b_c+b_b)/2$ and also the value of effective width according to proposed model are listed. It is found that in most of the specimens the value of $(b_c+b_b)/2$ are smaller than $b_c+0.5h_c$. By this assumption and using the ACI equation on the joint nominal shear strength shown in Figures 5 and 6 are plotted.

Figure 5 shows the correlation of the experimental joint shear strength vs the nominal joint shear strength predicted by ACI provisions using proposed effective width. Figure 6 represents storey shear strength on a vertical axis normalised by calculated storey shear from flexural analysis, and joint shear demand normalized by the joint shear capacity in related code on horizontal axis. It may be concluded that the wide beam improves the joint shear capacity. This is partially because joint shear forces applied at the top and bottom of the joint were distributed across the entire column width by means of the wide beam, hence the effective joint width is enlarged when compared with normal beam column joints.

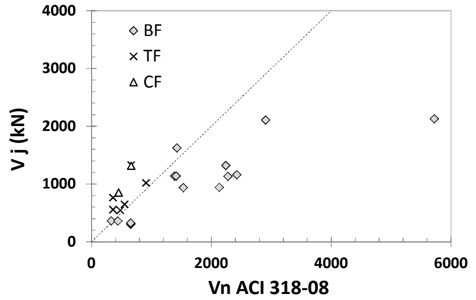

Figure 5. Joint shear strength vs joint shear capacity

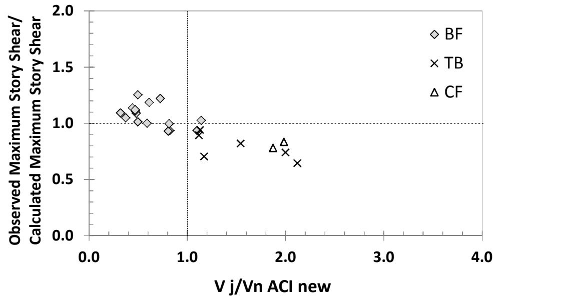

Figure 6. Normalized story shear vs normalized joint shear

It is certain that for the joint shear capacity margin smaller than 1, all the specimens have the beam hinging failure. For those with the joint shear capacity margin greater than 1 the observed maximum storey shear is smaller than calculated maximum storey shear. This trend is very useful for design purpose. If the joint designed according to this method and the joint shear capacity margin is kept smaller than 1, the beam hinging failure became the dominant failure mode. Seismic resistant design concepts require that joint hoops remain elastic and the possible damage happen in the plastic hinge region which is located on beam on the face of the column. For this reason, the required joint reinforcement should be estimated using a proposed effective joint width. From an analysis of tests it is obvious that the value of $(b_b+b_c)/2$ appears to give reasonable equivalent width for shear. Effective widths computed in this manner are necessary to reliably estimate the effect of wide beam wrapping.

CONCLUSION

Applicability of the joint nominal shear strength specified in codes of practice, including ACI 318-08, NZS 3101and EN 1998-1 to the interior wide beam-column joints is evaluated. To this end, results of all published experimental studies on interior wide beam-column joints are collected. The measured joint shear forces for all tested specimens are calculated by the provisions of three codes and results are compared with related specific codes limitation on the joint nominal shear strength. It is found that the ACI 318-08 limitation on joint effective area are conservative when applied to wide beam-column joints. A model is proposed based on the strut-and-tie concept and the joint nominal strengths are calculated based on the ACI approach with the proposed model. The results show very good agreement with the experimental results.

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