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A new empirical model for shear strength of reinforced concrete beam - column connections

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Abstract

A new empirical model to estimate the joint shear strength of both exterior and interior beam-column connections is proposed. In the model, four parameters that have the most influence on joint shear strength are considered. Among these four, a new parameter is introduced to consider the bond condition and the possibility of beam bars transferring joint shear force into the columns. Consideration of this parameter in the model significantly improves the accuracy of the predicted joint shear strength. To calibrate the model, a large database of 98 reinforced concrete (RC) exterior and 73 RC interior beam-column connections displaying joint failure mode was compiled from the literature. A parametric study was also carried out to evaluate the dependence of the predicted to tested joint shear strength ratio on the four influence parameters using the database. The proposed model showed superior performance over existing models. Moreover, comparisons of the predicted joint shear strength with experimental results and with four existing models showed the accuracy of the proposed model.

Keywords

empirical, beam, concrete, reinforced, connections, strength, column, shear, model

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A new empirical model for shear strength of reinforced concrete beam–column connections

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A new empirical model to estimate the joint shear strength of both exterior and interior beam–column connections is proposed. In the model, four parameters that have the most influence on joint shear strength are considered. Among these four, a new parameter is introduced to consider the bond condition and the possibility of beam bars transferring joint shear force into the columns. Consideration of this parameter in the model significantly improves the accuracy of the predicted joint shear strength. To calibrate the model, a large database of 98 reinforced concrete (RC) exterior and 73 RC interior beam–column connections displaying joint failure mode was compiled from the literature. A parametric study was also carried out to evaluate the dependence of the predicted to tested joint shear strength ratio on the four influence parameters using the database. The proposed model showed superior performance over existing models. Moreover, comparisons of the predicted joint shear strength with experimental results and with four existing models showed the accuracy of the proposed model.

Notation

A_{jh}	effective joint area
A_{sb}	greater area of top or bottom beam bars
A_{sjh}, A_{sjv}	total area of horizontal and vertical shear reinforcement respectively
A_{str}	effective area of the diagonal strut
a_i	influence coefficients
BI	beam reinforcement index
b_b, b_c	width of the beam and the column sections respectively
b_j	effective joint width
c	intercept value
d_{sb}	average diameter of beam tensile reinforcement
f'_c	cylinder compressive strength of concrete
f_{jhy}, f_{jvy}	yield strength of horizontal and vertical shear reinforcement respectively
h_b	height of beam cross-section
h_c	height of column cross-section
JI	joint transverse reinforcement index
K	factor of horizontal and vertical joint shear reinforcement
k	total number of collected experimental results
k_1, $k_{2,}$	joint shear factors
N	column axial load
n	number of influence parameters
n_b	maximum number of the top and the bottom beam bars
V_{ch}	contribution of concrete strut on joint shear strength
V_{jh}	horizontal joint shear strength
$V_{jh,model}$	predicted joint shear strength
$V_{jh,test}$	experimental shear strength
V_{sh}	contribution of concrete truss on joint shear strength
x_i	input influence parameters
α	inclination of diagonal strut from the column longitudinal direction
α_{NZS}	factor reflecting the influence of joint geometry and column axial load
α_t	factor describing in-plane geometry
β	factor equal to 1·0 and 0·8 for interior and exterior joints respectively
β_t	factor describing out-of-plane geometry
γ_{ACI}	joint shear factor in ACI 352R-02 (ACI, 2002)
γ_1, γ_2	$\gamma_1 = 0.81$, $\gamma_2 = 0.14$ for interior joints; $\gamma_1 = 0.0.34$, $\gamma_2 = 0.22$ for exterior joints
ζ	softening coefficient
η_t	parameter to account for the influence of beam eccentricity
θ	inclination of the diagonal compression strut from the beam longitudinal direction
ρ_{sb}	beam reinforcement ratio

σ_y column axial stress
χ_b beam bar index
χ_j normalised joint shear reinforcement
χ_{jh} normalised horizontal joint shear reinforcement
χ_{jv} normalised vertical joint shear reinforcement

Introduction

Beam–column connections are known to be one of the most critical regions of reinforced concrete (RC) structures as failure of the connections under seismic loading often leads to partial or total collapse of the whole structure. To improve the safety of RC structures under seismic load, designers have to carefully consider the shear strength and the ductility performance of beam–column connections to ensure that brittle shear failure at the joint region is avoided.

To understand the behaviour of beam–column connections, numerous experimental and analytical studies have been conducted since the mid-1960s. The first studies on beam–column connections were carried out by Hanson and Connor (1967). These were then developed by Zhang and Jirsa (1982), Sarsam
1 and ~~Phillips~~ (1985), Pantazopoulou and Bonacci (1992), Hwang and Lee (1999). The latest studies on beam–column connections include those of Kim *et al.* (2009), Choi and Kim (2011), Joyklad *et al.* (2012), Kim and Yu (2012), Najafian *et al.* (2013) and Patel *et al.* (2013). Although many efforts have been made, the research community has not yet understood the full behaviour of RC beam–column connections because of the large variations in both geometry and distribution of forces that occur in a relatively small volume at the joint region (Pantazopoulou and Bonacci, 1992). This point of view is evident when considering the inconsistencies in existing design standards for predicting the shear strength of RC beam–column connections (ACI, 2008; AIJ, 1999; BSI, 2004; SANZ, 1995).

In general, to carry the joint shear forces, resisting mechanisms, including diagonal struts and/or trusses, are developed. The truss mechanism is justified when the bond between the concrete and the beam and column reinforcement is perfect. In this case, the joint core is considered as a uniform plane zone subjected to shear stress and joint shear reinforcement is thus required to prevent shear failure of the joint core by diagonal principal tension stress. The diagonal strut mechanism is developed by the internal forces generated in the concrete and thus joint shear reinforcement is required to provide sufficient confinement for improving the compressive strength of the concrete diagonal strut(s). Different from the truss mechanism, in the diagonal strut mechanism, the bond between the beam bars and the concrete is allowed to deteriorate. Based on the above resisting mechanisms, numerous models have been developed for predicting the shear strength of beam–column connections. These theoretical models were developed based on either the average stress approach with compatibility of strains and stress equilibrium, or the strut-and-tie approach. In addition, because the mechanism of the joint is complicated and depends on many parameters, empirical and semi-empirical models have also been developed based on experimental data.

Most of the theoretical and empirical models currently available in the literature were summarised by Lima *et al.* (2012). Their summary showed that, in total, 11 parameters (including geometric and mechanical parameters) were taken into account by the available capacity shear strength models. Interestingly, these 11 parameters did not include the number and diameter of beam longitudinal bars, although these factors are the most important parameters that control the mechanism of bond forces transferring from the beam reinforcement to the concrete at the joint area. Analysis of a large experimental database of 171 beam–column connections introduced in the following section showed that, in addition to parameters such as concrete compressive strength, joint shear reinforcement and column axial stress, the diameter and number of beam bars are important factors for the shear strength of a beam–column connection. Therefore, in this work, the number and diameter of beam bars were considered in order to develop an empirical shear strength model for predicting the shear strength of beam–column connections. Development of the model was based on regression analysis using a large database collected from published works. The superiority of the proposed model was evaluated by comparing the predicted joint shear strengths with 171 test results from the literature and with four existing analytical models.

Experimental database

A database of 171 experimental RC beam–column connections (98 exterior and 73 interior) was compiled from the published literature (Alva *et al.*, 2007; Antonopoulos and Triantafillou, 2003; Attaalla, 2004; Au *et al.*, 2005; Chalioris *et al.*, 2008; Chun *et al.*, 2009; Chutarat and Aboutaha, 2003; Clyde *et al.*, 2000; Dhakal *et al.*, 2005; Durrani and Wight, 1985; Ehsani and Alameddine, 1991; Ehsani and Wight, 1985; Ehsani *et al.*, 1987; El-Amoury and Ghobarah, 2002; Fisher and Sezen, 2011; Ghobarah and El-Amoury, 2005; Ghobarah and Said, 2001, 2002; Hwang *et al.*, 2004, 2005; Ishibashi, 1993; Kaku and Asakusa, 1991; Karayannis and Sirkelis, 2005, 2008; Karayannis *et al.*, 2008; Kitayama *et al.*, 1991; Kuang and Wong, 2006; Le-Trung *et al.*, 2010; Lee *et al.*, 2010; Leon, 1990; Liu, 2006; Lu *et al.*, 2012; Megget, 1974; Meinheit and Jirsa, 1977; Morita *et al.*, 1999; Murty *et al.*, 2003; Noguchi and Kashiwazaki, 1992; Noguchi and Kurusu, 1988; Oka and Shiohara, 1992; Otani *et al.*, 1984; Pantelides *et al.*, 2002; Park and Paulay, 1974; Shrestha *et al.*, 2009; Supaviriyakit and Pimanmas, 2008; Tsonos, 2007; Tsonos *et al.*, 1992; Vatani-Oskouei, 2010; Wang and Hsu, 2009; Wong and Kuang, 2008).

All of the specimens were subjected to quasi-static cyclic lateral loading and were at least one-third scale. The final failure modes of the collected specimens were either joint shear or joint shear with yielding of beam reinforcement. All of the specimens had no out-of-plane members (slabs and/or transverse beams) and no eccentricity between beams and columns. Specimens that failed in terms of weak column–strong beam were excluded from the collected data because, in these cases, the column flexural capacity is relatively low and thus failure of the column caused by flexural moment can occur before the shear strength of the connection is reached. For interior connections, the column and beam reinforcement continuously passed through the joint panel. For exterior joints, only those specimens with conventional reinforcement anchorage were included (i.e. the longitudinal bars of the beams were anchored by hooks towards the core of the exterior joints). In the collected database, the test shear forces, $V_{jh,test}$, were either collected from the reported values or derived using the maximum applied load measured from the test. In this calculation, the moment arm of the beam cross-section was assumed to be 80% of the total beam height h_b.

The collected database covered a broad range of various parameters, including joint reinforcement ratio, reinforcement yielding stress, concrete strength, column axial average stresses and beam height to column height ratio (h_b/h_c), as summarised in Table 1 for exterior joints and Table 2 for interior joints. Of the exterior connections, 46 specimens failed through joint shear and 52 specimens failed in joint shear with yielding of beam reinforcement; the corresponding numbers for the interior connections were 41 and 32 respectively.

The definitions and the ranges of the main parameters in the collected database are summarised in Table 3, in which N is the column axial load, A_{sjh} and f_{jhy} are the area and yield strength of the horizontal joint shear reinforcement placed between the top and the bottom beam reinforcement respectively, A_{sjv} and f_{jvy} are the area and yield strength of the intermediate vertical reinforcement passing through the joint respectively, $b_j = (b_c + b_b)/2$ is the effective joint width (b_c and b_b are the section widths of the beam and the column respectively), n_b is the maximum number of the top and the bottom beam bars and d_{sb} is the corresponding average beam bar diameter.

A new parameter χ_b, referred to as the beam bar index, is proposed. From its definition in Table 3, it can be seen that χ_b is a function of the number of beam bars n_b and their average diameter d_{sb}. In addition, geometrical properties are also integrated into χ_b to form a dimensionless parameter reflecting the normalised contact area between the beam reinforcement and the surrounding concrete. As the concrete–reinforcement contact area has a direct influence on the magnitude of the bond forces transferring from the beam reinforcement into the concrete at the joint core, χ_b is proposed as a parameter that can affect the joint shear strength. By considering its definition, it is easy to
2 see that the joint aspect ratio ($\cancel{\alpha} = h_b/h_c$), which is considered as
3 an influence factor of the joint shear strength, is integrated in χ_b. Moreover, the conventional influence factor, the beam reinforcement ratio ($\rho_{sb} = A_{sb}/b_b h_b$, where A_{sb} is the greater of area of the top or bottom beam bars) is also incorporated to a certain degree via the term $n_b d_{sb}/b_b h_b$ in this parameter. Details about the influence of both the beam bar index and other parameters on the joint shear strength are evaluated in the following sections.

Parameters of influence in beam–column connections

The 'key' influence parameters for the joint shear behaviour of beam–column connections were investigated by Kim and LaFave (2007). They found that concrete compressive strength f'_c, in-plane geometry (interior, exterior or knee connections), dimensions of the beams and columns (h_b, b_b, h_c, b_c), joint transverse reinforcement and beam reinforcement were among the influence parameters for the shear strength of beam–column connections. Besides the above parameters, bond condition – which is strongly influenced by the number and diameter of reinforcement bars – and column axial stress are also known to affect joint shear strength.

Beam longitudinal bars passing or anchored in a joint core transfer a fraction of bond stress into the joint core and the remainder into the upper and lower columns. When the bond force of the beam bars is low, the fraction of bond force transferred into the columns is low, thus most of the shear force is claimed by the joint and slippage of the beam bars also occurs. The combination of these two unfavourable factors leads to a reduction in joint shear strength. Conversely, if the beam bars' bond force is high, the fraction of bond force transferred into the columns is significant; thus, a relatively small fraction of shear force is claimed by the joint and slippage of the beam bars would not occur, leading to an improvement in joint shear strength.

Among the parameters of influence, concrete compressive strength, in-plane geometry and the dimensions of the beam and column are the strongest, so their roles in joint shear strength have been mostly evaluated. The remaining parameters are still being debated. Some researchers (Bakir and Boduroğlu, 2002; Hegger *et al.*, 2003; Kim and LaFave, 2008; Parker and Bullman, 1997; Paulay and Priestley, 1992; Sarsam and Phipps, 1985; Vollum and Newman, 1999) have proposed inconsistent contributions of joint shear reinforcement in the shear strength of beam–column joints while some (Marques and Jirsa, 1975, ~~1977~~; Pantazopoulou and Bonacci, 1992) have indicated that column axial load has no coherent effect on joint shear strength. Clyde *et al.* (2000) reported that column axial load helped to improve the joint shear strength while Park and Mosalam (2012) showed that joint shear strength is not clearly affected by column axial stress up to $0{\cdot}2 f'_c$.

Models for shear strength of beam–column connections

As mentioned earlier, strut and truss mechanisms are developed to resist joint shear forces, thus the joint shear strength V_{jh} is usually proposed to be composed of two components

$$V_{jh} = V_{ch} + V_{sh} \tag{1}$$

Specimen	b_c: mm	h_c: mm	b_b: mm	h_b: mm	f'_c: MPa	$\frac{N}{b_c h_c f'_c}$	n_b	d_{sb}: mm	$V_{jh,test}$: kN	$V_{jh,model}$: kN	$\frac{V_{jh,model}}{V_{jh,test}}$	Failure mode[a]
Ehsani and Alameddine (1991)												
HL8	356	356	318	508	56	0·07	4	29	986	869	0·88	J
LL11	356	356	318	508	74	0·03	4	25	768	865	1·13	J
HL11	356	356	318	508	74	0·06	4	29	967	954	0·99	J
Wong and Kuang (2008)												
BS-L-300	300	300	260	300	34	0·15	3	20	505	374	0·74	J
BS-L-450	300	300	260	450	31	0·15	3	20	316	313	0·99	J
BS-L-600	300	300	260	600	36	0·15	3	20	284	316	1·11	J
BS-L-V2	300	300	260	450	33	0·15	3	20	399	355	0·89	J
BS-L-V4	300	300	260	450	28	0·15	3	20	403	367	0·91	J
BS-L-H1	300	300	260	450	33	0·15	3	20	389	342	0·88	J
Tsonos (2007)												
E1	200	200	200	300	22	0·18	3	14	232	195	0·84	J
G2	200	200	200	300	22	0·18	3	14	222	186	0·83	J
Clyde *et al.* (2000)												
Test 2	305	457	305	406	46	0·10	4	29	947	946	1·00	J
Test 4	305	457	305	406	41	0·25	4	29	982	1029	1·05	J
Test 5	305	457	305	406	37	0·25	4	29	941	982	1·04	J
Test 6	305	457	305	406	40	0·10	4	29	927	886	0·96	J
Kuang and Wong (2006)												
BS-L	300	300	260	450	31	0·14	3	20	316	307	0·97	J
BS-U	300	300	260	450	31	0·14	3	20	341	308	0·90	J
Tsonos *et al.* (1992)												
S6′	200	200	200	300	29	0·40	4	14	202	239	1·18	J
Pantelides *et al.* (2002)												
Test Unit 1	406	406	406	406	33	0·10	4	29	872	733	0·84	J
Test Unit 2	406	406	406	406	30	0·25	4	29	833	836	1·00	J
Test Unit 3	406	406	406	406	34	0·10	4	29	826	743	0·90	J
Test Unit 4	406	406	406	406	32	0·25	4	29	927	855	0·92	J
Test Unit 5	406	406	406	406	32	0·10	4	29	770	717	0·93	J
Test Unit 6	406	406	406	406	31	0·25	4	29	851	847	1·00	J
Chalioris *et al.* (2008)												
JA-0	200	300	200	300	34	0·05	4	12	218	255	1·17	J
JB-0	200	300	200	300	32	0·05	6	10	201	251	1·25	J
JB-s1	200	300	200	300	32	0·05	6	10	219	258	1·18	J
JCa-0	100	200	100	200	21	0·10	3	8	66	66	1·01	J
JCb-0	100	200	100	200	23	0·10	3	10	84	76	0·90	J
JCb-s1	100	200	100	200	23	0·10	3	10	97	83	0·86	J
JCb-s2	100	200	100	200	23	0·10	3	10	88	91	1·03	J
Karayannis and Sirkelis (2008)												
B0	200	200	200	300	32	0·05	6	10	199	141	0·71	J
B1	200	200	200	300	32	0·05	6	10	215	153	0·71	J
C0	200	200	200	300	32	0·05	4	12	209	150	0·72	J
Alva *et al.* (2007)												
LVP4	200	300	200	400	25	0·15	4	16	327	360	1·10	J
Antonopoulos and Triantafillou (2003)												
C1	200	200	200	300	16	0·12	3	14	116	98	0·85	J
C2	200	200	200	300	19	0·10	3	14	115	105	0·91	J
S-C	200	200	200	300	15	0·12	3	14	123	103	0·84	J

Table 1. Database of 98 exterior joints (continued on next page)

Specimen	b_c: mm	h_c: mm	b_b: mm	h_b: mm	f'_c: MPa	$\frac{N}{b_c h_c f'_c}$	n_b	d_{sb}: mm	$V_{jh,test}$: kN	$V_{jh,model}$: kN	$\frac{V_{jh,model}}{V_{jh,test}}$	Failure mode[a]
El-Amoury and Ghobarah (2002)												
T0	250	400	250	400	31	0·20	4	20	420	536	1·28	J
Ghobarah and Said (2001)												
T2	250	400	250	400	31	0·10	4	20	502	483	0·96	J
Ghobarah and El-Amoury (2005)												
T-BS3	250	400	250	400	30	0·20	4	20	421	534	1·27	J
Shrestha *et al.* (2009)												
UC1	300	300	300	450	26	0·08	4	24	293	304	1·04	J
Fisher and Sezen (2011)												
B-1-RC	152	152	152	203	30	0·00	3	13	130	109	0·84	J
Murty *et al.* (2003)												
Q1	200	250	200	400	26	0·00	2	20	156	156	1·00	J
R1	200	250	200	400	30	0·00	2	20	173	167	0·97	J
S1	200	250	200	400	28	0·00	2	20	163	161	0·99	J
1B	300	300	259	480	34	0·06	6	21	575	510	0·89	BJ
2B	300	300	259	439	35	0·07	6	21	587	540	0·92	BJ
5B	340	340	300	480	24	0·13	6	22	679	625	0·92	BJ
4	300	300	259	439	67	0·05	5	20	736	691	0·94	BJ
LL8	355·6	355·6	317·5	508	56	0·04	4	25	860	790	0·92	BJ
LH8	355·6	355·6	317·5	508	56	0·04	4	25	837	861	1·03	BJ
HH8	355·6	355·6	317·5	508	56	0·07	4	29	985	939	0·95	BJ
HH11	355·6	355·6	317·5	508	74	0·06	4	29	1020	1027	1·01	BJ
LL14	355·6	355·6	317·5	508	94	0·02	4	25	877	936	1·07	BJ
LH14	355·6	355·6	317·5	508	94	0·02	4	25	890	1005	1·13	BJ
HH14	355·6	355·6	317·5	508	94	0·04	4	29	1032	1092	1·06	BJ
0T0	420	420	320	450	67	0·02	4	25	997	1048	1·05	BJ
3T3	420	420	320	450	69	0·02	4	25	1132	1124	0·99	BJ
2T4	420	420	320	450	71	0·02	4	25	1080	1155	1·07	BJ
1T44	420	420	320	450	73	0·02	4	25	1039	1166	1·12	BJ
Wong and Kuang (2008)												
BS-L-H2	300	300	260	450	42	0·15	3	20	479	399	0·83	BJ
JCa-s1	100	200	100	200	21	0·10	3	8	71	73	1·04	BJ
JCa-s2	100	200	100	200	21	0·10	3	8	71	81	1·14	BJ
JC-2	500	500	350	500	60	0·03	8	22	1320	1635	1·24	BJ
JC-No.11	650	520	450	505	31	0·00	3	36	1179	1279	1·08	BJ
Karayannis and Sirkelis (2008)												
A1	200	200	200	300	36	0·05	2	10	76	112	1·48	BJ
A2	200	200	200	300	36	0·05	2	10	74	112	1·51	BJ
Karayannis *et al.* (2008)												
A0	200	200	200	300	32	0·05	2	10	83	105	1·26	BJ
C2	200	200	200	300	32	0·05	4	12	209	175	0·84	BJ
Hwang *et al.* (2004)												
28–0T0	550	550	380	500	33	0·02	4	25	1138	1356	1·19	BJ
LVP2	200	300	200	400	44	0·15	4	16	514	438	0·85	BJ
LVP3	200	300	200	400	24	0·15	4	16	364	383	1·05	BJ
LVP5	200	300	200	400	26	0·15	4	16	380	392	1·03	BJ
Unit RC-1	230	230	200	330	19	0·07	6	10	140	143	1·03	BJ
AJ1s	200	200	200	300	33	0·05	2	10	87	120	1·39	BJ

Table 1. Database of 98 exterior joints (continued on next page)

Specimen	b_c: mm	h_c: mm	b_b: mm	h_b: mm	f'_c: MPa	$\frac{N}{b_c h_c f'_c}$	n_b	d_{sb}: mm	$V_{jh,test}$: kN	$V_{jh,model}$: kN	$\frac{V_{jh,model}}{V_{jh,test}}$	Failure mode[a]
NS	167	167	134	200	37	0·00	4	10	120	96	0·80	BJ
SD	167	167	134	200	37	0·00	4	10	114	101	0·88	BJ
T1	250	400	250	400	31	0·19	4	20	527	537	1·02	BJ
1st	350	350	350	400	24	0·00	4	18	400	463	1·16	BJ
2nd	350	350	350	400	20	0·00	4	18	425	429	1·01	BJ
Fisher and Sezen (2011)												
C-2-RC	152	152	152	203	30	0·00	3	10	102	102	1·00	BJ
E-1-RC	152	152	152	203	30	0·00	3	12	113	107	0·95	BJ
UNIT A	330	380	255	460	22	0·07	3	28	547	548	1·00	BJ
Q3	200	250	200	400	27	0·00	2	20	211	222	1·05	BJ
S3	200	250	200	400	30	0·00	2	20	198	229	1·16	BJ
Specimen I	406	406	356	457	28	0·00	4	25	1040	846	0·81	BJ
Kaku and Asakusa (1991)												
NO 3	220	220	160	220	42	0·00	4	13	217	208	0·96	BJ
NO 4	220	220	160	220	45	0·17	4	13	239	254	1·06	BJ
NO 5	220	220	160	220	37	0·09	4	13	221	210	0·95	BJ
NO 6	220	220	160	220	40	0·00	4	13	209	196	0·94	BJ
NO 9	220	220	160	220	41	0·00	4	13	236	232	0·98	BJ
NO 11	220	220	160	220	42	0·08	4	13	231	248	1·08	BJ
NO 12	220	220	160	220	35	0·00	4	13	207	210	1·01	BJ
NO 13	220	220	160	220	46	−0·04	4	13	209	236	1·13	BJ
NO 14	220	220	160	220	41	0·08	4	13	226	226	1·00	BJ
NO 15	220	220	160	220	40	0·08	4	13	230	229	0·99	BJ
Park and Paulay (1974)												
S4	330	381	254	457	21	0·00	2	29	317	369	1·17	BJ
Average											1·00	
CoV											0·147	

[a] J, joint shear failure; BJ, joint shear failure with yielding of beam reinforcement

Table 1. (continued)

where V_{ch} and V_{sh} are the contributions of the concrete strut and truss mechanisms respectively. The first component is related to the concrete strength f'_c and the second term relates to the horizontal and vertical joint reinforcement.

ACI 352R-02 (ACI, 2002) and Architectural Institute of Japan design guidelines AIJ (1999) ignore the contribution of V_{sh}, thus the joint shear strength is expressed as a function of concrete compressive strength and the joint geometry. The form of the expression is

$$V_{jh} = k_1 k_2 (f'_c)^m A_{jh} \tag{2}$$

where k_1 and k_2 are joint shear factors that depend on the joint geometry (in ACI 352R-02, $k_1 \times k_2$ is equal to the joint shear strength factor, γ_{ACI}), A_{jh} is the effective joint area and $m = 0{\cdot}5$ in the ACI model and $m = 0{\cdot}7$ in the AJI model.

In NZS-3101 (SANZ, 1995), the contribution of the truss mechanism is somehow considered but a clear contribution of V_{sh} on joint shear strength is not available. The formulation of joint shear strength in NZS-3101 is

$$V_{jh} = \frac{f_{jhy} A_{sjh}}{6\alpha_{NZS} f_{by} A_{sb}} (f'_c)^m A_{jh} \leqslant 0{\cdot}2(f'_c) A_{jh} \tag{3}$$

in which $m = 1$, f_{jhy} and A_{sjh} are the yield strength and the total cross-sectional area of the horizontal joint transverse reinforcement respectively, f_{by} and A_{sb} are the yield strength and the greater of the area of top or bottom beam bars respectively and α_{NZS} is a factor that reflects the influence of both joint geometry and column axial load.

Paulay and Priestley (1992) proposed a theoretical joint shear strength model in which the contribution of the concrete strut V_{ch} depends on the area and yielding strength of the beam's long-

Specimen	b_c: mm	h_c: mm	b_b: mm	h_b: mm	f'_c: MPa	$\frac{N}{b_c h_c f'_c}$	n_b	d_{sb}: mm	$V_{jh,test}$: kN	$V_{jh,model}$: kN	$\frac{V_{jh,model}}{V_{jh,test}}$	Failure mode[a]
Meinheit and Jirsa (1977)												
I	330	457	279	457	26	0·40	3	25	1090	1223	1·12	J
II	330	457	279	457	42	0·25	3	25	1597	1477	0·92	J
III	330	457	279	457	27	0·39	3	25	1228	1448	1·18	J
IV	457	330	406	457	36	0·30	3	25	1455	1197	0·82	J
V	330	457	279	457	36	0·04	3	25	1530	1210	0·79	J
VII	457	330	406	457	37	0·47	3	25	1646	1594	0·97	J
XIII	330	457	279	457	41	0·25	3	25	1468	1361	0·93	J
XIV	457	330	406	457	33	0·32	3	25	1948	1528	0·78	J
Oka and Shiohara (1992)												
J-2	300	300	240	300	81	0·11	8	13	1557	1528	0·98	J
J-10	300	300	240	300	39	0·12	9	13	1539	1199	0·78	J
J-11	300	300	240	300	39	0·12	9	19	516	506	0·98	J
Attaalla (2004)												
SHC1	127	178	127	203	57	0·05	2	12	536	502	0·94	J
SHC2	127	178	127	203	60	0·04	2	12	576	517	0·90	J
SOC3	127	178	127	203	47	0·05	2	12	503	564	1·12	J
Kitayama *et al.* (1991)												
J1	300	300	200	300	26	0·08	8	13	491	497	1·01	J
A1	300	300	200	300	31	0·06	8	13	840	911	1·08	J
Au *et al.* (2005)												
E-0·0	300	300	250	300	41	0·00	4	16	853	931	1·09	J
H-0·0	300	300	250	300	41	0·00	4	16	629	807	1·28	J
E-0·3	300	300	250	300	40	0·35	4	16	1042	1190	1·14	J
Lee *et al.* (2010)												
J10	400	400	300	600	27	0·19	3	25	1119	1306	1·17	J
Noguchi and Kashiwazaki (1992)												
OKJ3	300	300	200	300	118	0·12	10	13	1075	1166	1·08	J
OKJ5	300	300	200	300	78	0·12	10	13	1204	1178	0·98	J
OKJ6	300	300	200	300	59	0·12	8	13	1111	1137	1·02	J
Dhakal *et al.* (2005)												
C1PD	350	500	300	550	32	0·12	5	32	888	1139	1·28	J
C4PD	400	400	300	550	33	0·12	6	32	1261	1180	0·94	J
Morita *et al.* (1999)												
No 1	350	350	250	350	22	0·31	4	25	795	873	1·10	J
No 2	350	350	250	350	22	−0·31	4	25	941	857	0·91	J
No 4	350	350	250	350	23	0·30	4	25	156	186	1·19	J
No 5	350	350	250	350	22	0·31	7	16	164	193	1·18	J
No 6	350	350	250	350	22	−0·31	7	16	163	174	1·07	J
Ishibashi (1993)												
D19-S1	400	400	260	400	44	0·00	5	19	493	573	1·16	J
D19-S2	400	400	260	400	43	0·00	3	19	429	573	1·34	J
D25-S3	400	400	260	400	48	0·00	3	25	628	575	0·92	J
D25-S4	400	400	260	400	48	0·00	3	25	545	568	1·04	J
D29-S5	400	400	260	400	48	0·00	3	29	826	662	0·80	J
Wang and Hsu (2009)												
Ko-JI1	300	300	300	500	32	0·14	4	25	750	679	0·91	J
Ho-JI1	400	400	300	400	26	0·00	4	19	869	763	0·88	J
Au *et al.* (2005)												
E-0·0	300	300	250	300	37	0·00	4	16	814	855	1·05	J

Table 2. Database of 73 interior joints (continued on next page)

Specimen	b_c: mm	h_c: mm	b_b: mm	h_b: mm	f'_c: MPa	$\frac{N}{b_c h_c f'_c}$	n_b	d_{sb}: mm	$V_{jh,test}$: kN	$V_{jh,model}$: kN	$\frac{V_{jh,model}}{V_{jh,test}}$	Failure mode[a]
E-0·3	300	300	250	300	39	0·26	4	16	1038	991	0·96	J
H-0·0	300	300	250	300	43	0·00	4	16	1081	974	0·90	J
H-0·3	300	300	250	300	38	0·26	4	16	378	384	1·02	J
Meinheit and Jirsa (1977)												
VI	330	457	279	457	37	0·48	3	25	414	451	1·09	BJ
XII	330	457	279	457	35	0·30	3	25	876	940	1·07	BJ
Otani *et al.* (1984)												
J1	300	300	200	300	26	0·08	3	19	1180	1086	0·92	BJ
J2	300	300	200	300	24	0·08	3	19	1458	1328	0·91	BJ
J3	300	300	200	300	24	0·08	3	19	1180	1086	0·92	BJ
J4	300	300	200	300	26	0·23	3	19	1180	1113	0·94	BJ
J5	300	300	200	300	29	0·07	3	19	1031	964	0·94	BJ
Durrani and Wight (1985)												
X1	362	362	279	419	34	0·05	4	22	798	635	0·80	BJ
X2	362	362	279	419	34	0·06	4	22	660	635	0·96	BJ
X3	362	362	279	419	31	0·05	3	22	844	630	0·75	BJ
Oka and Shiohara (1992)												
J-1	300	300	240	300	81	0·11	9	13	737	596	0·81	BJ
J-4	300	300	240	300	73	0·13	10	13	1273	1436	1·13	BJ
J-5	300	300	240	300	73	0·13	9	13	1045	1203	1·15	BJ
J-6	300	300	240	300	79	0·12	9	13	871	988	1·14	BJ
J-7	300	300	240	300	79	0·12	7	13	822	683	0·83	BJ
J-8	300	300	240	300	79	0·12	9	19	920	993	1·08	BJ
Kitayama *et al.* (1991)												
C1	300	300	200	300	26	0·08	12	10	865	991	1·15	BJ
B1	300	300	200	300	25	0·08	8	13	810	686	0·85	BJ
B3	300	300	200	300	25	0·08	8	13	1509	1318	0·87	BJ
Lu *et al.* (2012)												
J1–1	400	400	250	400	30	0·04	4	20	973	1151	1·18	BJ
J1–4	400	400	250	400	30	0·04	3	25	1542	1272	0·82	BJ
Leon (1990)												
BCJ2	254	254	203	305	30	0·00	4	13	1053	1272	1·21	BJ
BCJ3	254	305	203	305	27	0·00	4	13	1309	1322	1·01	BJ
Noguchi and Kashiwazaki (1992)												
OKJ1	300	300	200	300	78	0·12	9	13	411	501	1·22	BJ
Noguchi and Kurusu (1988)												
OKJ4	300	300	200	300	78	0·12	9	13	467	537	1·15	BJ
NO.1	300	300	200	300	34	0·06	12	10	484	560	1·16	BJ
NO.2	300	300	200	300	34	0·06	10	10	759	606	0·80	BJ
NO.3	300	300	200	300	34	0·06	6	13	893	868	0·97	BJ
NO.4	300	300	200	300	34	0·06	5	13	750	644	0·86	BJ
Supaviriyakit and Pimanmas (2008)												
J1	200	350	175	300	26	0·11	6	12	814	803	0·99	BJ
J2A	200	350	175	300	29	0·10	6	12	869	770	0·89	BJ
J3B	200	350	175	300	24	0·12	6	12	744	854	1·15	BJ
Average											1·00	
CoV											0·140	

[a] J, joint shear failure; BJ, joint shear failure with yielding of beam reinforcement

Table 2. (continued)

	Definition	Exterior joints		Interior joints	
		Minimum	Maximum	Minimum	Maximum
Concrete compressive strength f'_c: MPa		15	94	22	118
Normalised column axial stress	$\frac{N}{b_c h_c f'_c}$	−0·04	0·40	−0·31	0·48
Normalised horizontal joint shear reinforcement	$\chi_{jh} = \frac{A_{sjh} f_{jhy}}{b_j h_c (f'_c)^{0\cdot5}}$	0·00	1·11	0·00	1·23
Normalised vertical joint shear reinforcement	$\chi_{jv} = \frac{A_{sjv} f_{jvy}}{b_j h_c (f'_c)^{0\cdot5}}$	0·00	1·63	0·00	3·20
Normalised joint shear reinforcement	$\chi_j = \chi_{jh} + \chi_{jv}$	0·00	2·45	0·00	3·50
Column depth to beam height ratio	h_c/h_b	0·50	1·13	0·67	1·00
Beam bar diameter to column depth ratio	d_b/h_c	0·03	0·09	0·03	0·08
Beam bar index	$\chi_b = \frac{n_b d_{sb}}{b_b} \frac{h_c}{h_b}$	0·07	0·51	0·13	0·71

Table 3. Definitions and ranges of parameters in the collected database

itudinal reinforcement, the concrete compressive strength, column axial load and the joint geometry. The contribution of the truss mechanism is simply determined as $V_{sh} = A_{sjh}\ f_{jhy}$. A similar contribution of the truss mechanism on joint shear strength was proposed by Vollum and Newman (1999), but their model additionally accounted for the influence of h_b/h_c.

Bakir and Boduroğlu (2002) proposed an empirical model in which V_{ch} depends on concrete strength, details of the beam reinforcement and joint aspect ratio h_b/h_c. In their model, the contribution of the truss mechanism is determined as $V_{sh} = \alpha_j A_{sjh}\ f_{jhy}$, where α_j depends on the joint shear reinforcement ratio. A similar form was proposed by Hegger *et al.* (2003) for determining V_{sh}, but the parameter α_j was supposed to be influenced by the pattern of the beam bars being anchored and details of the joint shear reinforcement. Different from other models, this model accounted for the influence of the column reinforcement ratio.

Using the Bayesian parameter estimation method, Kim *et al.* (2009) developed an empirical joint shear strength model. The distinctive feature of this model is that the joint shear strength is given in terms of multiplication of the influence parameters

$$V_{jh} = 1{\cdot}31\alpha_t\beta_t\eta_t(\mathrm{JI})^{0\cdot15}(\mathrm{BI})^{0\cdot3}(f'_c)^{0\cdot75}A_{jh} \quad (4)$$

in which α_t and β_t are parameters for describing in-plane and out-of-plane geometry respectively, η_t is a parameter to account for the influence of beam eccentricity, JI is the joint transverse reinforcement index (depending mostly on the volumetric joint shear reinforcement ratio) and BI is the beam reinforcement index (depending mostly on the beam reinforcement ratio ρ_{sb}). A limitation of this model is that column axial load was not taken into account as a possible influence parameter for joint shear strength during development of the model. This is because, in the Bayesian parameter estimation model, all parameters should be non-zero (Kim and LaFave, 2008) and some experimental tests had zero column axial load.

Based on the strut-and-tie approach, Hwang and Lee (2002) developed a theoretical joint shear strength model in which the joint shear strength was determined as

$$V_{jh} = K\zeta f'_c A_{str} \cos\theta \quad (5)$$

where K is a factor accounting for the contribution of horizontal and vertical joint shear reinforcement, ζ $(= 3{\cdot}35/(f'_c)^{0\cdot5} \leqslant 0{\cdot}52)$ is a softening coefficient, A_{str} is the effective area of the diagonal strut depending on column axial load and dimensions of the joint and θ is the angle of inclination of the diagonal compression strut with respect to the longitudinal direction of the beam. The model of Hwang and Lee (2002) can clearly illustrate the strut-and-tie mechanism in the joint core but the joint shear prediction is known to be very sensitive to A_{str}, which is difficult to determine accurately.

Based on the average stress approach, Tsonos (2007) developed a theoretical model for the shear strength of beam–column joints. In the model, a biaxial compression–tension failure envelope was adopted to predict the joint shear strength. However, this model exaggerated the role of joint shear reinforcement in confining the concrete at the strut, while its influence on the truss mechanism was ignored. Using a similar method, Wang *et al.* (2012) developed a theoretical model for the shear strength of beam–column joints. However, the limitation of their model is that the role of joint shear reinforcement was exaggerated by assuming that the tensile strength of the concrete and the joint shear reinforcement is reached simultaneously. The model of Wang *et al.* (2012) is

$$V_{\mathrm{jh}} = \beta \frac{1-(\sin^2\alpha/f_{\mathrm{t,n}} - 0{\cdot}8\cos^2\alpha/f'_{\mathrm{c}})\sigma_{\mathrm{y}}}{(1/f_{\mathrm{t,n}} + 0{\cdot}8/f'_{\mathrm{c}})\sin 2\alpha} A_{\mathrm{jh}} \tag{6}$$

where $\beta = 1{\cdot}0$ and $0{\cdot}8$ for interior and exterior joints respectively, α is the inclination of the diagonal strut, $f_{\mathrm{t,n}}$ is the nominal tensile strength of concrete with contributions from joint shear reinforcement and σ_{y} is the column axial stress.

The proposed model

From the above analysis, it can be concluded that there is inconsistency in the research community about the contribution of strut and truss mechanisms in resisting joint shear force. The role of some parameters related to geometry, column axial load, joint shear and beam reinforcement is still under debate. In most of the joint shear strength models, the role of beam reinforcement was considered in terms of its total cross-sectional area and yield strength. Joint performance is also known to be influenced by the reinforcement–concrete bond condition, especially the bond between beam bars and the concrete at the joint core. Therefore, a new model is proposed in which the bond condition of beam bars is considered via the new parameter χ_{b}. The number of beam bars and their diameters are thus examined instead of their total cross-sectional area and yield strength.

In the proposed model, the general form of joint shear strength is assumed to be a function of influence parameters as illustrated by

$$V_{\mathrm{jh}} = A_{\mathrm{jh}}(f'_{\mathrm{c}})^{0{\cdot}5}\left(\sum_{i=1}^{n} a_i x_i + c\right) \tag{7}$$

where x_i are the input influence parameters, n is the number of influence parameter, a_i are the influence coefficients and c is the intercept. The determination of a_i and c was based on regression analysis of the collected database.

The proposed model considers eight parameters, as shown in Table 3. Of the eight parameters, the normalised joint shear reinforcement is the sum of the normalised horizontal joint shear reinforcement and the normalised vertical joint shear reinforcement. This parameter is taken into consideration to verify whether the horizontal, the vertical or their total significantly influence joint shear strength. Table 4 illustrates the level of significance of the considered parameters on the shear strength of exterior and interior joints. The table shows the variation of the average absolute error (AAE) of the model-to-test shear strength with variation of the considered parameters. The model joint shear strength was calculated using Equation 7 with the number of influence parameters ranging from six to one. The AAE of the model to the test joint shear strength indicates the accuracy of the proposed model and was calculated using

$$\mathrm{AAE} = \frac{1}{k}\sum_{i=1}^{k} \frac{\left|V^i_{\mathrm{jh,model}} - V^i_{\mathrm{jh,test}}\right|}{V^i_{\mathrm{jh,test}}} \tag{8}$$

where k is the total number of collected experimental results and $V^i_{\mathrm{jh,model}}$ and $V^i_{\mathrm{jh,test}}$ are the predicted and experimental joint shear strengths respectively.

Table 4 shows that both the horizontal joint shear reinforcement

Parameter	Number of considered parameters							
	6	5	4	3	3	2	2	1
f'_{c}	✓	✓	✓	✓	✓	✓	✓	✓
χ_{j}	✓	✓	✓	✓	—	—	✓	—
χ_{jh}	—	—	—	—	✓	✓	—	—
χ_{jv}	—	—	—	—	✓	—	—	—
χ_{b}	✓	✓	✓	✓	—	—	—	—
$N/b_{\mathrm{c}}h_{\mathrm{c}}f'_{\mathrm{c}}$	✓	✓	✓	—	—	—	—	—
$\cancel{d_{\mathrm{b}}}/h_{\mathrm{c}}$	✓	✓	—	—	—	—	—	—
$h_{\mathrm{c}}/h_{\mathrm{b}}$	✓	—	—	—	—	—	—	—
AAE, exterior joints	0·102	0·105	0·108	0·143	0·182	0·191	0·182	0·213
AAE, interior joints	0·117	0·117	0·117	0·122	0·132	0·155	0·132	0·162

Table 4. Variation of average absolute error of the model-to-test joint shear strength with the considered parameters

and the column intermediate longitudinal reinforcement influence the joint shear strength. The proposed new parameter, the beam bar index χ_b significantly influences the joint shear strength. Consideration of this parameter in the model helps in reducing the AAE from 18·2% to 14·3% for exterior joints and from 13·2% to 12·2% for interior joints. The two parameters with the least influence include the column depth to beam height ratio and the beam bar diameter to column depth ratio. It is worth mentioning that the beam bar index was limited to 0·4 in all the calculations, that is

$$\chi_b = \frac{n_b d_{sb}}{b_b} \frac{h_c}{h_b} \leqslant 0{\cdot}4$$

4 As the parameters d_b/h_c and h_c/h_b are the two least important factors, these factors were ignored to give a simple formation of the joint shear strength. Moreover, for simplicity, the influence coefficients a_i and the intercept c for exterior and interior joints were adjusted to give a unique formulation that can be applied for both types of connections. The final equation for joint shear strength is

$$V_{jh} = \left(\gamma_1 + \frac{N}{b_c h_c f'_c} + 1{\cdot}2\chi_b \right) A_{jh} (f'_c)^{0{\cdot}5} + \gamma_2 (A_{sjh} f_{jhy} + A_{sjv} f_{jvy}) \qquad 9.$$

in which $\gamma_1 = 0.81$ and $\gamma_2 = 0.14$ for interior joints and $\gamma_1 = 0.34$ and $\gamma_2 = 0.22$ for exterior joints.

Model verification

The results from tests on exterior and interior joints in the compiled database were used to verify the proposed model. The test shear strength $V_{jh,test}$ and the predicted shear strength $V_{jh,model}$ calculated from Equation 9 are shown in Table 1 for exterior joints and Table 2 for interior joints. The tables show that the predicted joint shear strengths are in very close agreement with the experimental values. The proposed model predicts joint shear strength with an average model-to-test shear strength ratio of 1·00 and with coefficients of variation (CoV) of 14·7% for exterior joints and 14·0% for interior joints.

Figure 1 shows the results from the proposed model and results from the joint shear strength models proposed by ACI (2002), Kim *et al.* (2009), Hwang and Lee (2002) and Wang *et al.* (2012), corresponding to Equations 2, 4, 5 and 6 respectively. The ACI model (ACI, 2002) was chosen for the comparison because it is the simplest model and the only parameter considered is concrete compressive strength. The model of Kim *et al.* (2009) was chosen because it is of the same level of complexity as the proposed model and was also developed based on an empirical approach. The models of Hwang and Lee (2002) and Wang *et al.* (2012) were also chosen for comparison because they represent theoretical models developed based on the strut-and-tie and average stress approaches respectively. The average value of the predicted-to-test joint shear strength ratio (AVG) and its coefficient of variation (CoV) are shown in Figure 1 to compare accuracies in predicting the joint shear strength. The figure shows that the model proposed in this paper gave the best predictions of joint shear strength.

Figures 2 and 3 show comparisons of model accuracy for exterior and interior joints respectively; the figures indicate that the proposed model can predict the shear strength of both exterior and interior connections with a great improvement in accuracy. The proposed model predicts the shear strength of exterior joints with an AAE of 11% and CoV of 15%; these values for the most recent model (Wang *et al.*, 2012) are 17% and 22% respectively. The ACI 352R-02 model (ACI, 2002) is the least accurate in predicting the shear strength of exterior joints, with AAE = 27% and CoV = 34%. For interior joints (Figure 3), the proposed model predicts the joint shear strength with AAE = 12% and CoV = 14%; the next most accurate model is ACI 352R-02, with AAE = 13% and CoV = 19%.

Parametric study

The proposed model considers four 'key' parameters – concrete compressive strength f'_c, the normalised column axial stress $N/(b_c h_c f'_c)$, the normalised joint shear reinforcement $\chi_j = \chi_{jh} + \chi_{jv}$ and the beam bar index χ_b. These parameters were double checked and compared with existing models to evaluate the fitness of the influence coefficients as well as the accuracy of the proposed model. Figure 4 illustrates the variation of the predicted-to-test joint shear strength ratio with these four parameters. Linear regression lines were added to the figure to demonstrate clearly the dependency of the joint shear strength ratio on each of the parameters. In general, the figure shows that the proposed joint shear strength ratio has no clear dependence on these four parameters despite their large range variation. Figure 4 also reveals that the proposed model gives the predicted-to-test joint shear strength ratio with less scatter than the other models. In particular, the figure shows that the predicted-to-test joint shear strength ratio of the proposed model varies around 1·00 in a small range from 0·71 to 1·51; this range for the models of ACI (2002), Kim *et al.* (2009) and Wang *et al.* (2012) is 0·57–1·77, 0·61–1·83 and 0·51–1·66 respectively.

Figure 4(a) compares the dependence of joint shear strength ratio on concrete compressive strength for the proposed model and the model in ACI 352R-02. In both models, the power term for concrete compressive strength is 0·5. However, the joint shear strength ratio of the ACI model tends to reduce with an increase in concrete strength while there is no clear dependency in the proposed model. This indicates that consideration of the remaining three parameters effectively improves the reliability of the proposed model.

Figure 4(b) compares the variation of joint shear strength ratio

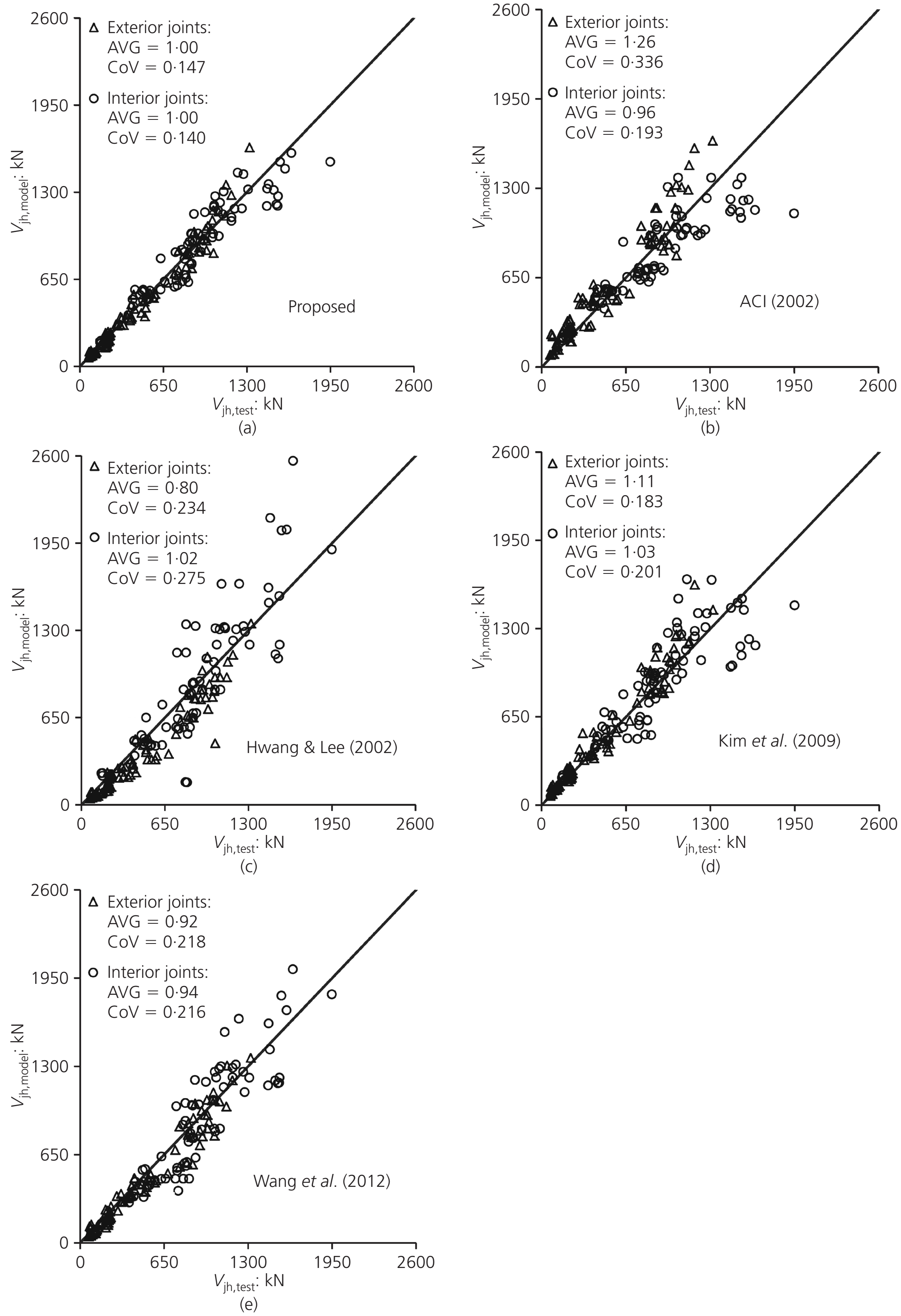

Figure 1. Performance of the proposed model (a) and the models of ACI (2002) (b), Hwang and Lee (2002) (c), Kim *et al.* (2009) (d) and Wang *et al.* (2012) (e)

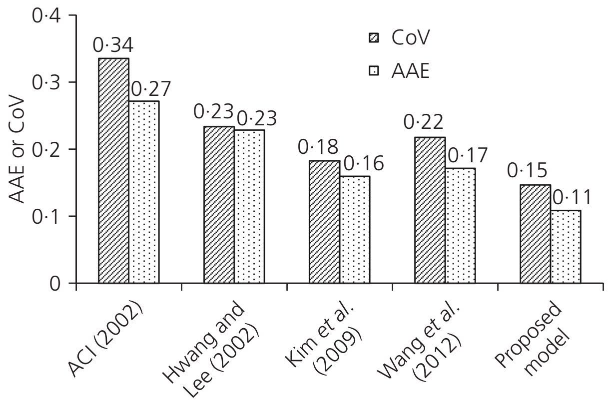

Figure 2. Accuracy comparison of the models for shear strength prediction of exterior joints

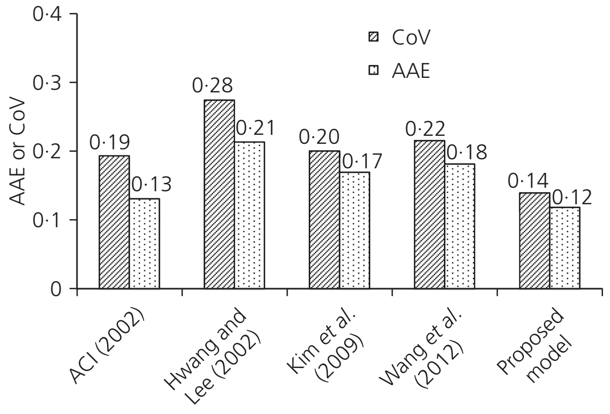

Figure 3. Accuracy comparison of the models for shear strength prediction of interior joints

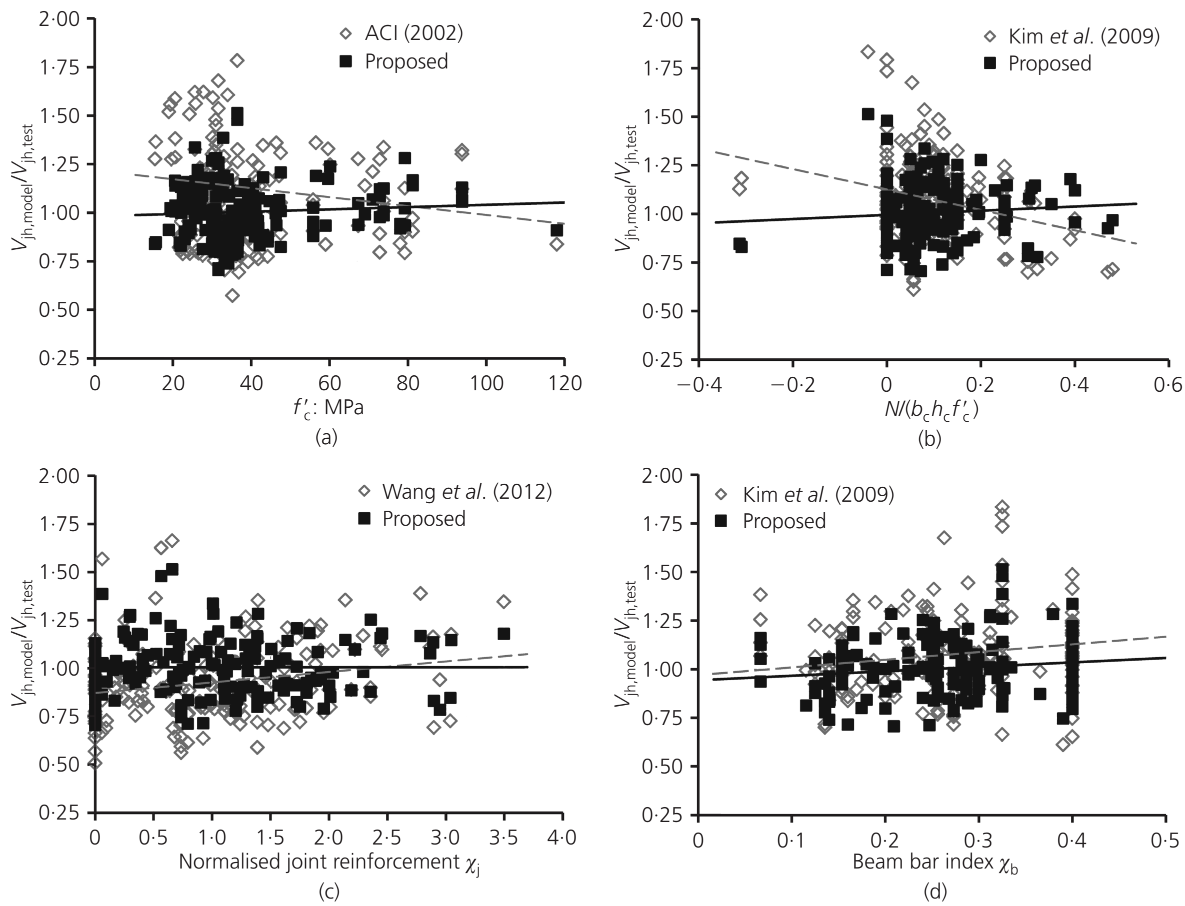

Figure 4. Variation of joint shear strength ratio with each of the influence parameters

with the normalised column axial stress for the proposed model and the model of Kim *et al.* (2009). Both models were developed based on an empirical approach, but the model of Kim *et al.* neglects the influence of column axial load and thus its joint shear strength ratio is significantly influenced by the normalised column axial stress.

The dependency of the joint shear strength ratio on the normalised joint shear reinforcement in the proposed model and the model of Wang *et al.* (2012) is shown in Figure 4(c). Once again, it can be seen that the proposed model has the better performance. The independence of the proposed joint shear strength ratio on the normalised joint shear reinforcement indicates that the proposed coefficient γ_2 in Equation 9 is justified.

Figure 4(d) shows a comparison of the dependence of joint shear strength ratio on the beam bar index. For the proposed model, there is very little dependency of the joint shear strength ratio on the beam bar index, while significant dependency is observed for the model of Kim *et al.* (2009). Moreover, as there was no limitation in the variation range of the amount of beam steel bars, it is believed that the model of Kim *et al.* overestimates the joint shear strength when the amount of beam reinforcement is high. It is noted that, in both models, the role of the beam steel bars was considered in different manners: Kim *et al.* considered the beam bars in terms of their total area together with their yield strength while the proposed model considers their contact area with the surrounding concrete. The better performance of the proposed model compared with that of Kim *et al.* in this respect indicates that the reasons for proposing the parameter χ_b to account for the influence of beam longitudinal reinforcement are rational.

Conclusions

A new empirical model for joint shear strength of RC beam–column connections has been introduced. The model was developed based on regression analysis using a large database collected from the literature. The influence of some 'key' parameters on joint shear strength was analysed and four parameters were chosen to generate the proposed model. These four parameters included a new parameter to reflect the beam bar bond condition as well as the possibility of the beam bar transferring a fraction of the joint shear force into the column. The regression analysis showed that consideration of this parameter helps to significantly improve the reliability of the model when compared with test results.

A parametric study to illustrate the better performance of the proposed model when compared with other models was also conducted. This showed that the proposed coefficients in the equation for joint shear strength are justified and that the proposed equation for joint shear strength is reliable despite the large variation in the parameters of influence. Results from the proposed model were compared with those from four existing joint shear strength models to demonstrate the effectiveness of the new model. Due to its accuracy, stability and simplicity, application of the proposed model for predicting the shear strength of beam–column connections in practical design is expected.

REFERENCES

ACI (American Concrete Institute) (2002) ACI 352R-02: Recommendations for design of beam–column joints in monolithic reinforced concrete structures. ACI, Farmington Hills, MI, USA.

ACI (American Concrete Institute) (2008) ACI 318R: Building code requirements for structural concrete and commentary. ACI, Farmington Hills, MI, USA.

AIJ (Architectural Institute of Japan) (1999) Design guideline for earthquake resistance reinforced concrete buildings based on inelastic displacement concept. AIJ, Tokyo, Japan.

Alva GMS, Debs ALHDC and Debs MKE (2007) An experimental study on cyclic behaviour of reinforced concrete connections. *Canadian Journal of Civil Engineering* **34(4)**: 565–565.

Antonopoulos CP and Triantafillou TC (2003) Experimental investigation of FRP-strengthened RC beam–column joints. *Journal of Composites for Construction* **7(1)**: 39–49.

Attaalla SA (2004) General analytical model for nominal shear stress of type 2 normal and high-strength concrete beam–column joints. *ACI Structural Journal* **101(6)**: 881–882.

Au FTK, Huang K and Pam HJ (2005) Diagonally-reinforced beam–column joints reinforced under cyclic loading. *Proceedings of the Institution of Civil Engineers – Structures and Buildings* **158(1)**: 21–40.

Bakir PG and Boduroğlu HM (2002) A new design equation for predicting the joint shear strength of monotonically loaded exterior beam–column joints. *Engineering Structures* **24(8)**: 1105–1117.

BSI (2004) Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings. BSI, London, UK.

Chalioris CE, Favvata MJ and Karayannis CG (2008) Reinforced concrete beam–column joints with crossed inclined bars under cyclic deformations. *Earthquake Engineering and Structural Dynamics* **37(6)**: 881–897.

Choi H and Kim J (2011) Progressive collapse-resisting capacity of RC beam–column sub-assemblage. *Magazine of Concrete Research* **63(4)**: 297–310.

Chun SC, Oh B, Lee SH and Naito CJ (2009) Anchorage strength and behavior of headed bars in exterior beam–column joints. *ACI Structural Journal* **106(5)**: 579–590.

Chutarat N and Aboutaha RS (2003) Cyclic response of exterior reinforced concrete beam–column joints reinforced with headed bars–experimental investigation. *ACI Structural Journal* **100(2)**: 259–264.

Clyde C, Pantelides CP and Reaveley LD (2000) Performance-based evaluation of reinforced concrete building exterior joints for seismic excitation. *Earthquake Spectra* **18(3)**: 449–480.

Dhakal RP, Pan TC, Irawan P *et al.* (2005) Experimental study on the dynamic response of gravity-designed reinforced concrete connections. *Engineering Structures* **27(1)**: 75–87.

Durrani A and Wight J (1985) Behaviour of interior beam-to-column connections under earthquake-type loading. *ACI Journal* **82(3)**: 343–349.

Ehsani MR and Alameddine F (1991) Design recommendations for type 2 high-strength reinforced concrete connections. *ACI Structural Journal* **88(3)**: 277–291.

Ehsani MR and Wight JK (1985) Exterior reinforced concrete beam-to-column connections subjected to earthquake-type loading. *ACI Structural Journal* **82(4)**: 492–499.

Ehsani MR, Moussa AE and Vallenilla CR (1987) Comparison of inelastic behavior of reinforced ordinary and high-strength concrete frames. *ACI Structural Journal* **84(2)**: 161–169.

El-Amoury T and Ghobarah A (2002) Seismic rehabilitation of beam–column joint using GFRP sheets. *Engineering Structures* **24(11)**: 1397–1407.

Fisher MJ and Sezen H (2011) Behavior of exterior reinforced concrete beam–column joints including a new reinforcement. *Structural Engineering and Mechanics* **40(6)**: 867–883.

Ghobarah A and El-Amoury T (2005) Seismic rehabilitation of deficient exterior concrete frame joints. *Journal of Composites for Construction* **9(5)**: 408–408.

Ghobarah A and Said A (2001) Seismic rehabilitation of beam–column joints using FRP laminates. *Earthquake Engineering* **5(1)**: 113–129.

Ghobarah A and Said A (2002) Shear strengthening of beam–column joints. *Engineering Structures* **24(7)**: 881–888.

Hanson NW and Connor HW (1967) Seismic resistance of reinforced concrete beam–column joints. *Journal of Structural Division ASCE* **93(5)**: 533–559.

Hegger J, Sherif A and Roeser W (2003) Nonseismic design of beam–column joints. *ACI Structural Journal* **100(5)**: 654–664.

Hwang J and Lee HJ (1999) Analytical model for predicting shear strengths of exterior reinforced concrete beam–column joints for seismic resistance. *ACI Structural Journal* **96(5)**: 846–857.

Hwang SJ and Lee HJ (2002) Strength prediction for discontinuity regions by softened strut-and-tie model. *Journal of Structural Engineering ASCE* **128(12)**: 1519–1526.

Hwang SJ, Lee HJ and Wang KC (2004) Seismic design and detailing of exterior reinforced concrete beam–column joints. *Proceedings of 13th World Conference on Earthquake Engineering, Vancouver BC, Canada*, paper 0397.

Hwang SJ, Lee HJ, Liao TF, Wang KC and Tsai HH (2005) Role of hoops on shear strength of reinforced concrete beam–column joints. *ACI Structural Journal* **102(3)**: 445–453.

Ishibashi K (1993) *Experimental Study on Interior Beam-to-*
5 *column Joints of Reinforced Concrete Frames*. ~~???~~ thesis
Tokyo University, Tokyo, Japan (in Japanese).

Joyklad P, Pimanmas A and Dhakal RP (2012) Cyclic performance of beam–column joints with extended column fixed at base. Part I: experimental investigation. *Magazine of Concrete Research* **64(9)**: 807–807.

Kaku T and Asakusa H (1991) Ductility estimation of exterior beam–column subassemblages in reinforced concrete frames.
6 ~~*On-line Journals Search Results* **123(xxx)**: 167–186,~~

7 Karayannis CG and Sirkelis GM (2005) Response of column and joints with spiral shear reinforcement. ~~*Proceedings of CMEM*~~
~~*Conference on Computational Methods and Experimental*~~
8 ~~*Measurements, Xxxxxxxxxxxx, Malta*,~~

9 Karayannis CG and Sirkelis GM (2008) Strengthening and rehabilitation of RC beam–column joints using carbon-FRP jacketing and epoxy resin injection. *Earthquake Engineering and Structural Dynamics* **37(5)**: 769–790.

Karayannis CG, Chalioris CE and Sirkelis GM (2008) Local retrofit of exterior RC beam–column joints using thin RC jackets – an experimental study. *Earthquake Engineering and Structural Dynamics* **37(5)**: 727–746.

Kim J and LaFave JM (2007) Key influence parameters for the joint shear behaviour of reinforced concrete (RC) beam–column connections. *Engineering Structures* **29(10)**: 2523–2539.

Kim J and LaFave JM (2008) Probabilistic joint shear strength models for design of RC beam–column connections. *ACI Structural Journal* **105(6)**: 770–780.

Kim J and Yu J (2012) Analysis of reinforced concrete frames subjected to column loss. *Magazine of Concrete Research* **64(1)**: 21–33.

Kim J, LaFave JM and Song J (2009) Joint shear behaviour of reinforced concrete beam–column connections. *Magazine of Concrete Research* **61(2)**: 119–132.

Kitayama K, Otani S and Aoyama H (1991) Development of design criteria for RC interior beam–column joints. *Design of Beam–Column Joints for Seismic Resistance*. American Concrete Institute, Detroit, MI, USA, ACI SP-123, pp. 97–123.

Kuang JS and Wong HF (2006) Effects of beam bar anchorage on beam–column joint behaviour. *Proceedings of the Institution of Civil Engineers – Structures and Buildings* **159(2)**: 115–124.

Le-Trung K, Lee J, Lee DH, Lee K and Woo S (2010) Experimental study of RC beam–column joints strengthened using CFRP composites. *Composites Part B* **41(1)**: 76–85.

Lee WT, Chiou YJ and Shih MH (2010) Reinforced concrete beam–column joint strengthened with carbon fiber reinforced polymer. *Composite Structures* **92(1)**: 48–60.

Leon RT (1990) Shear-strength and hysteretic behavior of interior beam–column joints. *ACI Structural Journal* **87(1)**: 3–11.

Lima C, Martinelli E and Faella C (2012) Capacity models for shear strength of exterior joints in RC frames: state-of-the-art and synoptic examination. *Bulletin of Earthquake Engineering* **10(3)**: 967–983.

Liu C (2006) *Seismic Behaviour of Beam–Column Joint Subassemblies Reinforced with Steel Fibres*. Masters thesis, University of Canterbury, Christchurch, New Zealand.

Lu X, Urukap TH, Li S and Lin F (2012) Seismic behavior of interior RC beam–column joints with additional bars under cyclic loading. *Earthquakes and Structures* **3(1)**: 37–57.

Marques JLG and Jirsa JO (1975) A study of hooked bar anchorages in beam–column joints. *ACI Structural Journal* **72(5)**: 198–209.

Megget LM (1974) Cyclic behaviour of exterior reinforced concrete beam–column joints. *Bulletin of The New Zealand National Society for Earthquake Engineering* **7(1)**: 27–47.

Meinheit DF and Jirsa JO (1977) *The Shear Strength of Reinforced Concrete Beam–Column Joints*. University of Texas, Austin, TX, USA, CESRL Report 77-1.

Morita S, Kitayama K, Koyama A and Hosono T (1999) Effects of beam bar bond and column axial load on shear strength in

reinforced concrete interior beam–column joints. *Transactions*
10 *of the Japan Concrete Institute* **21~~(xxx);~~** 453–460.

Murty CVR, Rai DC, Bajpai KK and Jain SK (2003) Effectiveness of reinforcement details in exterior reinforced concrete beam–column joints for earthquake resistance. *ACI Structural Journal* **100(2)**: 149–156.

Najafian HA, Vollum RL and Fang L (2013) Comparative assessment of finite-element and strut and tie based design methods for deep beams. *Magazine of Concrete Research* **65(16)**: 970–970.

Noguchi H and Kashiwazaki T (1992) Experimental studies on shear performances of RC interior column–beam joints with high-strength materials. *Proceedings of 10th World*
11 *Conference on Earthquake Engineering,* ~~*Xxxxxxxxxxxxx,*~~
12 ~~*Xxxxxxxxxxxxxxx,*~~

Noguchi H and Kurusu K (1988) Correlation of bond and shear in RC beam–column connections subjected to seismic forces. *Proceeding of 9th World Conference on Earthquake*
13 *Engineering,* ~~*Xxxxxxxxxxxxx, Xxxxxxxxxxxxxxx,*~~ pp. 597–602.

14 Oka K and Shiohara H (1992) Test of high-strength concrete interior beam-to-column joint subassemblages. *Proceedings of 10th World Conference on Earthquake Engineering,*
15 ~~*Xxxxxxxxxxxxx, Xxxxxxxxxxxxxxx,*~~ pp. 3211–3217.

16 Otani S, Kobayashi Y and Aoyama H (1984) Reinforced concrete beam–column joints under simulated earthquake loading.
17 *Proceedings of 1st US–NZ–Japan Seminar~~, Monterey,~~*

18 Pantazopoulou S and Bonacci J (1992) Consideration of questions about beam–column joints. *ACI Structural Journal* **89(1)**: 27–36.

Pantelides CP, Hansen J, Nadauld J and Reaveley LD (2002) *Assessment of RC Building Exterior Joints with Substandard Details*. ~~University of Utah, Publisher location, PEER Report~~
19 ~~2002/18,~~

Park R and Paulay T (1974) Behaviour of reinforced concrete external beam–column joints under cyclic loading. *Proceedings of 5h World Conference on Earthquake*
20 *Engineering, Rome, Italy~~,~~*

Park S and Mosalam KM (2012) Parameters for shear strength prediction of exterior beam–column joints without transverse
21 reinforcement. *Engineering Structures* **36~~(xxx);~~** 198–209.

Parker DE and Bullman PJM (1997) Shear strength within reinforced concrete beam–column joints. *Structural Engineer* **75(4)**: 53–57.

Patel PA, Desai AK and Desai JA (2013) Evaluation of RC and SFRC exterior beam–column joint under cyclic loading for reduction in lateral reinforcement of the joint region. *Magazine of Concrete Research* **65(7)**: 405–414.

Paulay T and Priestley MJN (1992) *Seismic Design of Reinforced Concrete and Masonry Buildings*. Wiley, New York, NY, USA.

SANZ (Standards Association of New Zealand) (1995) NZS-3101: Concrete structures standard. Part 1: the design of concrete structures. SANZ, Wellington, New Zealand.

Sarsam KF and Phipps ME (1985) The shear design of in situ reinforced concrete beam–column joints subjected to monotonic loading. *Magazine of Concrete Research* **37(130)**: 16–28.

Shrestha R, Smith ST and Samali B (2009) Strengthening RC beam–column connections with FRP strips. *Proceedings of the Institution of Civil Engineers – Structures and Buildings* **162(5)**: 323–334.

Supaviriyakit T and Pimanmas A (2008) Comparative performance of sub-standard interior reinforced concrete beam–column connection with various joint reinforcing details. *Materials and Structures* **41(3)**: 543–557.

Tsonos AG (2007) Cyclic load behavior of reinforced concrete beam–column subassemblages of modern structures. *ACI Structural Journal* **104(4)**: 468–478.

Tsonos AG, Tegos IA and Penelis GG (1992) Seismic resistance of type 2 exterior beam–column joints reinforced with inclined bars. *ACI Structural Journal* **89(1)**: 3–12.

Vatani-Oskouei A (2010) Repairing of seismically damaged RC exterior beam–column connection using CFRP. *Journal of Reinforced Plastics and Composites* **29(21)**: 3257–3274.

Vollum RL and Newman JB (1999) Strut and tie models for analysis/design of external beam–column joints. *Magazine of Concrete Research* **51(6)**: 415–425.

Wang GL, Dai JG and Teng JG (2012) Shear strength model for RC beam–column joints under seismic loading. *Engineering*
Structures **40~~(xxx);~~** 350–360. 22

Wang YC and Hsu K (2009) Shear strength of RC jacketed interior beam–column joints without horizontal shear reinforcement. *ACI Structural Journal* **106(2)**: 222–232.

Wong HF and Kuang JS (2008) Effects of beam–column depth ratio on joint seismic behaviour. *Proceeding of the Institution of Civil Engineers – Structures and Buildings* **161(2)**: 91–101.

Zhang L and Jirsa JO (1982) *A Study of Shear Behavior of Reinforced Concrete Beam–Column Joints*. University of Texas, Austin, TX, USA, PMFSEL Report 82–1.

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