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INFLUENCE OF CENTRAL VERTICAL BAR ON THE BEHAVIOUR OF HIGH STRENGTH CONCRETE TRANSFER BEAM-COLUMN JOINTS

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ABSTRACT

Beam column joint (BCJ) specimens tested under monotonic loading were compared with joint shear failure predicted according to EC8-NA [1] and ACI352 [2].

Inadequacy of these design codes for accurate estimations of the shear stresses at BCJ are identified. A design rule for the prediction of BCJ failure for high strength concrete (HSC) is given. The proposed method offer better accuracy when the results are compared with design rules from the above codes and research results.

Finite element numerical models for BCJ specimens were compared with the experimental ones. Furthermore parametric investigations of the influence of Central Vertical Bar, CVB, on the shear capacity of HSC- BCJ were conducted.

Strut and tie model for BCJ with CVB was developed to guide the designers towards using the proposed design rule to calculate the amount of shear CVB and stirrups required in order to resist the excessive joint shear.

Keywords: high strength concrete, transfer beam column joint, central vertical bar

1. INTRODUCTION

The shear design of beam-column joint (BCJ) is normally assessed in seismic countries where ACI [2], AIJ [3&4], IKU [5], AETL [6] and EEFIT [7] reports following earthquakes have identified BCJ as critical part of the reinforced concrete frame structure. The joint shear design for BCJ has been the subject of numerous research projects in the past three decades. This paper investigates the shear behaviour of external beam column joints of HSC column and transfer beam (see Figure 1) exposed to monotonic loading.

Many tall reinforced concrete frames are built with transfer beams to provide clear spaces in their entrance halls. With the advantages of HSC, such buildings usually have HSC columns. The external BCJ (see Figure 1) made of transfer beam and HSC column has unique shear behaviour, which has not been investigated fully by other researchers.

The authors' investigations on 12 beams [8,9], Figure 2, indicated that:

- When the shear span to depth ratio a/d = 3 then HSC beams shear resistance may be less than that of NSC beams (Figure 3a).
- When adding CHB in the beams then shear resistance of HSC beams

significantly improves and become greater than that of NSC.

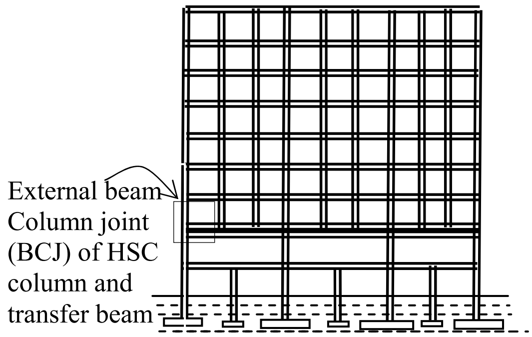
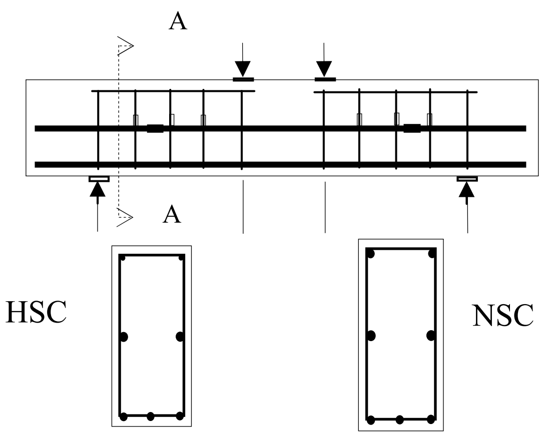


Figure 2. A multi storey RC frame with transfer beams of 3≥ hb /hc≥2.5



SectionA-A
Figure 2. Tweleve HSC and NSC beams tested by the authors

The reasons for such behaviour are due to (i) the stabilising arching affect in the beam as the result on the presence of CHB (Figure 3b) (ii) the double-strut action produced by the presence of central bar in addition to the main reinforcement (Figure 3c).

The ratio of beam depth to column depth is defined as the aspect ratio (h_b/h_c) has significant influence of BCJ behaviour (Figure 3a). Taylor's [10] demonstrated that shear behaviour of short beam is analogous to the behaviour of BCJ when aspect ratio ≤ 2. Similarly Motamed [8&9] has shown that the shear resistance in HSC beams with CHB produces stabilising arching affect, due to the dowel action as well as double strut action, is comparable to short beams shear behaviour, hence HSC beam with CHB will behave similar to BCJ with central vertical bar (CVB). Therefore, since the short beam behaviour is analogues to the behaviour of BCJ thus, it can be assumed that $h_b/h_c \approx a/b$ (Figure 3a). Similarly, BCJ shear resistance with vertical central bars in the column with aspect ratio $3 \geq h_b/h_c \geq 2$ is analogous to HSC beam, $3 \geq a/d \geq 2$ with CHB (Figure 3b).

Following review of the EC8-NA [1] and ACI352 [2] design methods and the past experimental research on BCJ, an empirical design equation for the joint shear is introduced which is proportional to the joint concrete strength, the shear resisting contribution of the dowel action from the vertical central bars and the confinement stirrups.

The proposed design rule for joint shear allows for prediction of quantity of the vertical central bar in the column as shear reinforcement in high strength concrete BCJ with large aspect ratio.

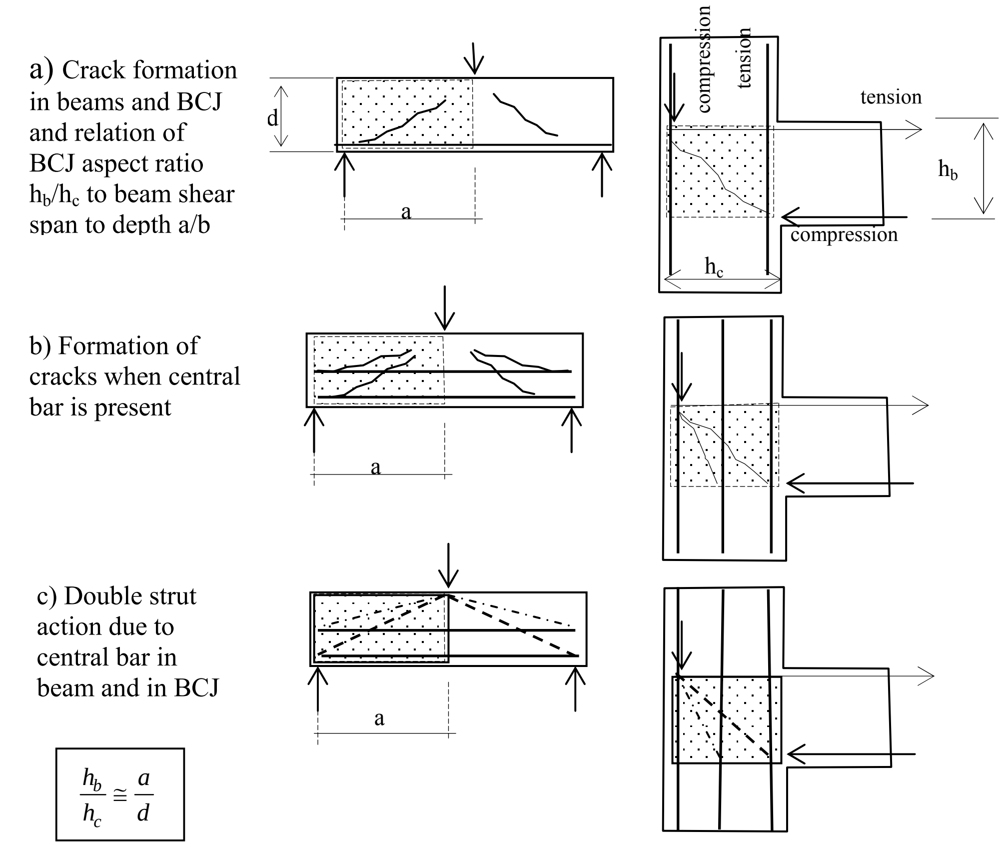


Figure 3. Cracks formation and strut and tie action in short beams and BCJ

2. CALCULATION OF JOINT SHEAR FORCE

A brief review is that $V_{u,joint}=T_n-V_{col}$ (Figure 4) , where $V_{u,joint}$ is the joint shear, V_{col} is the horizontal shear force across the column and T_n is tension force in the tension reinforcement of the beam which is given by $T_n = M_n/z$, where M_n is the beam moment at the column face and z is the flexural lever arm. The theoretical joint shear force is dependent on the assumptions used to calculate M_n and z. M_n is taken as $M_n = P\,(L + d')$ as shown in Figure 4, where L is the distance from the load P to the face of the column and d' is the distance from the face of the column to the centroid of the of column reinforcement as shown in the Figure.

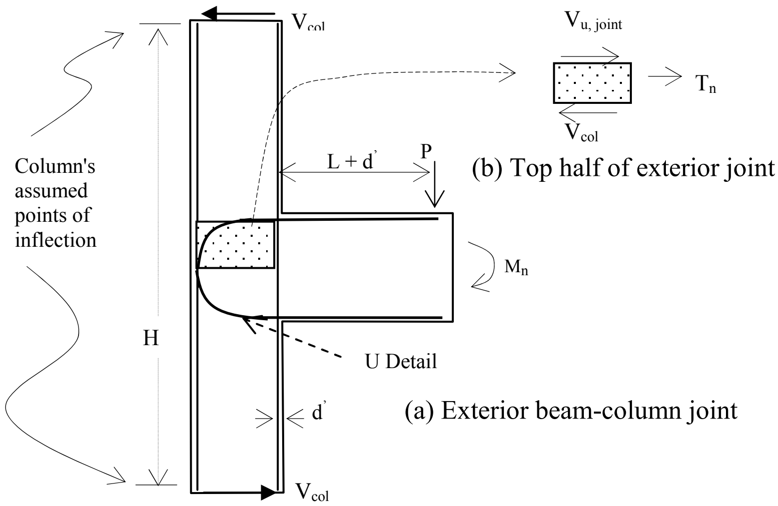


Figure 4. Forces acting on external beam column joints

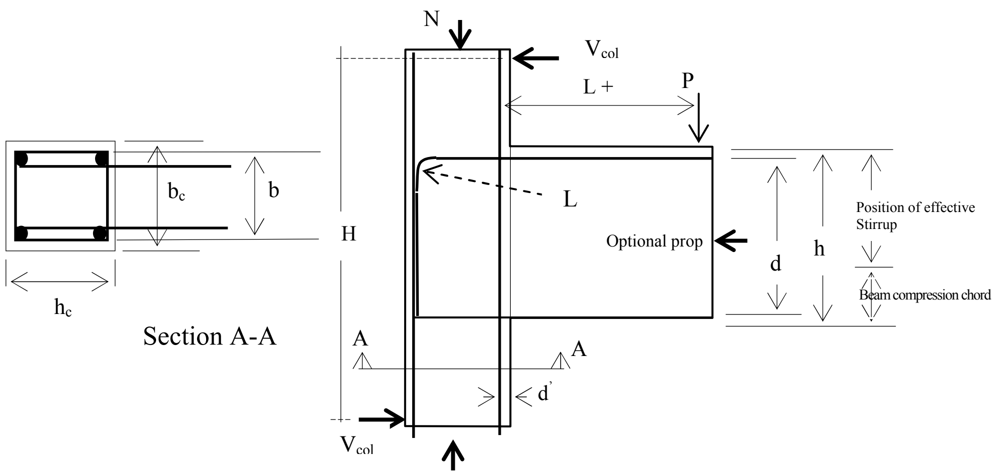


Figure 5: Typical elevation of external beam-column joint specimens used in the tests listed in Table 1

The tensile force in the beam reinforcement is calculated by section analysis assuming that plane section remains plane. The rectangular-parabolic stress block defined in EC2 [11] is used for the concrete.
The stress is assumed to reach a maximum value of $0.8f_{cu}$ at a compressive strain of 0.002. The width of the compressive stress block is taken as the beam width in the analysis of the beam-column joints. An elasto-plastic stress-strain response is assumed for the reinforcement with an elastic modulus of 200 GPa. No material factors of safety are applied.

2. ANALYSIS OF THE AVAILABLE TEST DATA

There is a general lack of agreement among researchers over the influence of

variables such as concrete strength, column loading, joint aspect ratio, joint stirrups, beam thrust, beam reinforcement and column vertical bars on the joint shear behaviour of the external BCJ. Furthermore numerous tests on BCJ under cyclic loading simulating earthquakes behaviour have been performed; however, these researches have a number of shortcomings such as lack of detail investigation of the influence of shear stress at BCJ due to incremental strain development in the reinforcement.
In order to investigate these factors, available data from tested BCJ specimens were statically used to develop the parametrical values for the proposed equation of concrete contribution to joint shear at BCJ.
A finite element model, Figure 8, has been developed for typical specimens and loaded with incremental monotonic loading condition. FE model was used to study the above influences as well as to compared its results with those of the test results and with the predications of the proposed design equation.
Research on monotonically-loaded, external beam-column joints were carried out in the UK, by Ortiz [12], Taylor [14], Scott [15], Scott Hamill [16], Parker & Bullman [17], Wilson [20] and Vollum [19]; similarly in Germany by Kordina [13]. Test data from these experiments are shown in Table 1.
The relationship between shear index and stirrups index, Figure 6, show that there is a linear increase of shear in the joint as the amount of stirrups increases. However this occurs after all the concrete contribution to resist the joint shear has been taken into account. Neither of the above two equations make provision for this behaviour even though both design methods specify minimum stirrup requirements. Furthermore neither of the two equations predicts the degree of dependency of joint shear strength on joint aspect ratio, Figure 7.
A shear analysis is carried out to develop a relationship between concrete strength and the joint shear strength for the specimens shown in table 2. These analyses show that the joint shear strength has a closer relationship to $(f_c')^{2/3}$ of EC8-NA rather than $(f_c')^{1/2}$ of ACI/ ASCE Committee 352. This is because the variance of shear index of specimens without stirrups is 0.29 (0.54-0.25=0.29) for EC8 and 0.43 (0.94-0.51=0.43) for ACI, Figure 6.

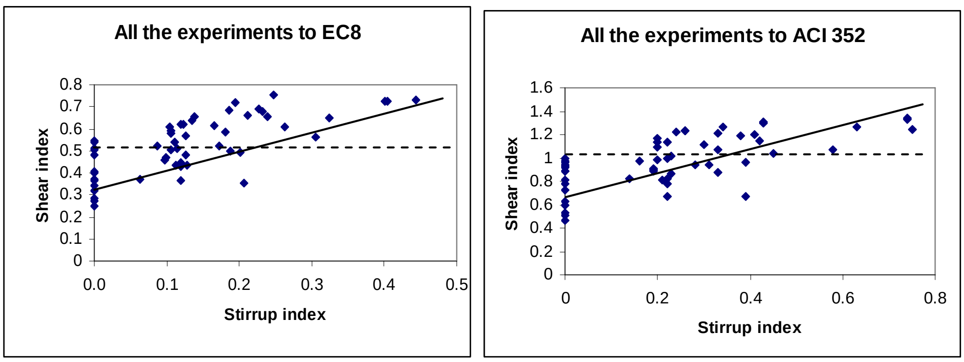


Figure 6. Relationships between Joint Shear index and Stirrup index according to EC8-NA and ACI 352

Table 1: Specimens geometry and beam reinforcement for beam-column joints; HSC joints are shown bold in shade

Researcher	Identity	H_c mm	L mm	h_c mm	d_c mm	b_c mm	h_b mm	d_b mm	b_b mm	ρ_b
Ortiz[12]	BCJ1	2000	1050	300	267	200	400	367	200	0.011
	BCJ2	2000	1100	300	267	200	400	367	200	0.011
	BCJ3	2000	1100	300	267	200	400	367	200	0.011
	BCJ4	2000	1100	300	267	200	400	367	200	0.011
	BCJ5	2000	1100	300	267	200	400	367	200	0.011
	BCJ6	2000	1100	300	267	200	400	367	200	0.011
	BCJ7	2000	1100	300	267	200	400	367	200	0.011
Kordia[13]	RE2	3000	1000	200	167	200	400	365	200	0.009
	RE3	3000	1000	200	167	200	400	265	200	0.018
	RE4	3000	1000	200	167	200	400	265	200	0.012
	RE6	3000	1000	200	167	200	400	265	200	0.012
	RE7	3000	975	230	217	230	350	315	230	0.013
	RE8	3000	975	230	217	230	350	315	230	0.013
	RE9	3000	975	230	217	230	350	315	230	0.013
	RE10	3000	975	230	217	230	350	355	230	0.012
Taylor [14]	P1/41/24	1290	470	140	110	140	200	170	100	0.024
	P2/41/24	1290	470	140	110	140	200	170	100	0.024
	P2/41/24A	1290	470	140	110	140	200	170	100	0.024
	A3/41/24	1290	470	140	110	140	200	170	100	0.024
	D3/41/24	1290	470	140	110	140	200	170	100	0.024
	B3/41/24	1290	470	140	110	140	200	170	100	0.024
	C3/41/24BY	1290	470	140	110	140	200	170	100	0.024
	C3/41/13Y	1290	470	140	110	140	200	173	100	0.024
	C3/41/24Y	1290	470	140	110	140	200	170	100	0.024
Scott [15]	CIAL	1700	750	150	117	150	210	179	110	0.011
	C4	1700	750	150	117	150	210	177	110	0.021
	C4A	1700	750	150	117	150	210	177	110	0.021
	C4AL	1700	750	150	117	150	210	177	110	0.021
	C7	1700	750	150	117	150	300	267	110	0.014
	C3L	1700	750	150	117	150	210	177	110	0.021
	C6	1700	750	150	117	150	210	177	110	0.021
	C6L	1700	750	150	117	150	210	177	110	0.021
	C9	1700	750	150	117	150	300	267	110	0.014
Scott& Hamil [16]	C4ALN0	1700	750	150	117	150	210	177	110	0.021
	C4ALN1	1700	750	150	117	150	210	177	110	0.021
	C4ALN3	1700	750	150	117	150	210	177	110	0.021
	C4ALN5	1700	750	150	117	150	210	177	110	0.021
	C4ALH0	1700	750	150	117	150	210	177	110	0.021
	C6LN0	1700	750	150	117	150	210	177	110	0.021
	C6LN1	1700	750	150	117	150	210	177	110	0.021
	C6LN3	1700	750	150	117	150	210	177	110	0.021
	C6LN5	1700	750	150	117	150	210	177	110	0.021
	C6LH0	1700	750	150	117	150	210	177	110	0.021
	C6LH1	1700	750	150	117	150	210	177	110	0.021
	C6LH3	1700	750	150	117	150	210	177	110	0.021
Parker & Bullman [17]	4b	2000	850	300	245	300	500	445	250	0.009
	4c	2000	850	300	245	300	500	445	250	0.009
	4d	2000	850	300	245	300	500	445	250	0.009
	4e	2000	850	300	245	300	500	445	250	0.009
	4f	2000	850	300	245	300	500	445	250	0.009
	5b	2000	850	300	245	300	500	445	250	0.009
	5f	2000	850	300	245	300	500	445	250	0.014
Sarsam[18]	EX2	1536	1422	204	172	157	305	272	152	0.010
Vollum [19]	EBCJ6	2000	450	200	167	200	300	257	200	0.008
	EBCJ8	2000	450	200	167	200	300	257	200	0.012
Wilson[20]	J1	3000	850	300	269	154	300	257	154	0.017

Table 2: Table for Shear indices: $V_j/b_ch_cf_c^{2/3}$ (average value=0.525) and $V_j/b_ch_c\sqrt{f_c}$ and Stirrup indices: $A_{sje}fy/b_ch_cf_c^{2/3}$ and $A_{sje}fy/b_ch_c\sqrt{f_c}$. Shaded and bold specimens are in HSC

Researcher	Identity	Bar Detail	Fc MPa	f_{yb} (MPa)	P (kN)	$A_{sje}f_y/b_ch_c\sqrt{f_c}$ √MPa	$V_j/b_ch_c\sqrt{f_c}$ √MPa	$A_{sje}f_y/b_ch_cf_c^{2/3}$ MPa2/3	$V_j/b_ch_cf_c^{2/3}$ MPa2/3
	BCJ1	L Bar	34	720	118	0	0.92	0.0	0.51
	BCJ2	L Bar	38	720	125	0.16	0.97	0.1	0.52
	BCJ3	L Bar	33	720	118	0	0.99	0.0	0.55
Ortiz	BCJ4	L Bar	34	720	130	0.33	1.07	0.2	0.59
	BCJ5	L Bar	38	720	115	0	0.89	0.0	0.48
	BCJ6	L Bar	35	720	115	0	0.93	0.0	0.51
	BCJ7	L Bar	35	720	170	0.74	1.33	0.4	0.73
	RE2	L Bar	25	420	67	0	0.94	0.0	0.54
	RE3	L Bar	40	420	80	0.26	1.23	0.1	0.66
	RE4	L Bar	32	420	51	0.19	0.91	0.1	0.50
	RE6	L Bar	32	463	66	0.38	1.19	0.2	0.66
Kordia	RE7	L Bar	26	448	117	0.43	1.31	0.2	0.75
	RE8	L Bar	28	464	105	0.42	1.15	0.2	0.65
	RE9	U Bar	28	454	110	0.41	1.2	0.2	0.68
	RE10	U Bar	24	459	100	0.45	1.04	0.3	0.61
	P1/41/24	L Bar	33	500	35	0.3	1.11	0.2	0.61
	P2/41/24	L Bar	29	500	35	0.33	1.21	0.2	0.68
	P2/41/24	L Bar	47	500	47	0.26	1.23	0.1	0.64
	A3/41/2	L Bar	27	500	35	0.34	1.26	0.2	0.72
Taylor	D3/41/2	L Bar	53	500	50	0.24	1.22	0.1	0.62
	B3/41/2	L Bar	22	500	30	0.75	1.24	0.4	0.73
	C3/41/2	U Bar	32	500	29	0.31	0.94	0.2	0.52
	C3/41/1	U Bar	28	500	27	0.33	0.88	0.2	0.50
	C3/41/2	U Bar	60	500	45	0.23	1.02	0.1	0.51
	CIAL	L Bar	33	540	22	0.23	0.87	0.1	0.48
	C4	L Bar	41	540	30	0.2	1.09	0.1	0.58
	C4A	L Bar	44	540	32	0.2	1.13	0.1	0.59
	C4AL	L Bar	36	540	28	0.22	1.14	0.1	0.62
Scott	C7	L Bar	35	540	32	0.22	0.78	0.1	0.43
	C3L	U Bar	35	540	22	0.22	0.82	0.1	0.45
	C6	U Bar	40	540	22	0.21	0.81	0.1	0.43
	C6L	U Bar	46	540	26	0.19	0.9	0.1	0.47
	C9	U Bar	36	540	28	0.22	0.67	0.1	0.36
	C4ALN	L Bar	42	522	27	0	0.96	0.0	0.51
	C4ALN	L Bar	46	522	34	0.2	1.17	0.1	0.61
	C4ALN	L Bar	42	522	35	0.43	1.3	0.2	0.69
	C4ALN	L Bar	50	522	40	0.63	1.26	0.3	0.65
	C4ALH	L Bar	104	522	43	0	0.89	0.0	0.40
Scott&	C6LN0	U Bar	51	522	24	0	0.78	0.0	0.40
Hamill	C6LN1	U Bar	51	522	25	0.19	0.89	0.1	0.46
	C6LN3	U Bar	49	522	29	0.39	0.96	0.2	0.50
	C6LN5	U Bar	37	522	34	0.74	1.34	0.4	0.73
	C6LH0	U Bar	101	522	36	0	0.81	0.0	0.37
	C6LH1	U Bar	102	522	37	0.14	0.82	0.1	0.37
	C6LH3	U Bar	97	522	41	0.28	0.94	0.1	0.43
	4b	L Bar	39	570	138	0	0.46	0.0	0.25
	4c	L Bar	37	570	170	0	0.59	0.0	0.32
Parker &	4d	L Bar	39	570	150	0	0.51	0.0	0.27
Bullman	4e	L Bar	40	570	160	0	0.53	0.0	0.28
	4f	L Bar	38	570	183	0	0.63	0.0	0.34
	5b	L Bar	43	485	236	0.39	0.67	0.2	0.35
	5f	L Bar	43	515	322	0.58	1.07	0.3	0.56
Sarsam	EX2	L Bar	52	500	37	0	0.72	0.0	0.37
Vollum	EBCJ6	U Bar	26	540	100	0.22	0.99	0.1	0.57
	EBCJ8	U Bar	33	540	120	0.2	0.98	0.1	0.54
Wilson	J1	L Bar	32	520	76	0	0.97	0.0	0.54

The dotted lines in the graphs represent the empirical values 0.525 and 1.058 of equations (1) and (2) respectively. The results of the specimens below the dotted lines indicate over estimation of the joint shear. Ignoring the minimum reinforcement requirements; the numbers of joint failures which are within the safe prediction of EC8-NA [1] are 28 out of 56 tests i.e. 50% of total specimens. Whereas for ACI 352, the numbers of safe prediction of joint failures are 23 out of 56 test i.e. 41%. (These are shown above the horizontal dotted line, Figure 6).
A linear relationship between shear index and stirrup index can be plotted when the shear indices are above 0.35 and 0.7 for EC8-NA and ACI 352 respectively. From the graph, Figure 6, it can be noted that the upper limit of stirrup index for EC8 ≤ 0.4 and for ACI ≤ 0.75
From table 2 it can be concluded that for EC8-NA [1], the mean values for shear index in BCJ for L-reinforcement, Figure 5, is 0.54 and for U-reinforcement, Figure 4, is 0.49.

3. PROPOSED DESIGN EQUATION FOR EXTERNAL BEAM COLUMN

Both design codes ACI 352[2] and EC8-NA [1] specify minimum shear stirrup requirements, however, they do not give provision for the joint strength to be increased by the stirrups.
The design recommendations of these codes fail to predict the observed dependence of joint shear strength on the joint aspect ratio, as well as the influence of HSC and detailing of the anchorage on the behaviour of BCJ. Also they do not provide any recommendation if the amount of stirrups is not adequate in order to provide sufficient shear strength at BCJ when the shear forces are high.
As noted above the HSC beams may be weaker in shear than NSC beams when span depth ratio is 3, it can also be deduced that HSC-BCJ will be weaker than NSC-BCJ when the joint aspect ratio exceed 2.5.
Past research work by Motamed [7] on 12 beams demonstrated that for the design of HSC beams with a/d=3, CHB produced superior shear capacity due to the development of dowel action which in turn enhanced the stabilising arching affect in the beams.
Using Baumann's [21] dowel cracking expression, the dowel force causing cracking is:

$$V_{du} = D_{cr} = 1.64\, h_c d_b\, f_{cu}^{1/3} (n)^{1/4} \quad \text{(for n number of bar in the beam)} \tag{3}$$

$$V_{du} = D_{cr} = 1.95\, h_c d_b\, f_{cu}^{1/3} \quad \text{(for n = 2 i.e. bar at mid-depth, BCJ with CVB)} \tag{4}$$

Where d_b= diameter of the dowel bars and n is number of bars,
V_{du} = dowel force, f_{cu}= cube crushing strength of concrete of 150 mm cubes in N/mm^2.
The stabilising arching effect in the beam with a/d = 3 makes the beam perform like a short beam 2≤a/d≤3 and is analogous to BCJ shear (Figure 3a).
BCJ with central vertical bar in column, the dowel shear resistance is

$$V_{jd} = V_c + 1.95\, h_c\, d_b\, f_{cu}^{1/3} \quad (5)$$

$$V_c = \gamma(f_c)^{2/3}\, b_e.h_c \text{ is joint shear resistance due to concrete} \quad (6)$$

$$V_{jd} = \gamma(f_c)^{2/3}\, b_e.h_c + 1.95\, h_c\, d_b\, f_{cu}^{1/3} \quad (7)$$

Where γ = 0.54 or 0.49 for L-type, or U-type detail connections shown in Figures 4 and 5.
Proposed design rule is based on refining EC8-NA [1] design rule by using γ factor for beam detailing and including the dowel action from the central bar within the depth of the column.
The proposed method for designing shear stirrups in BCJ adopted from Fip Recommendation [22] for short beams is:

$$F_{nw} \approx \left(\frac{2 \times \frac{5}{8} \times h_b / h_c - 1}{3 - N_n / F_n} \right) . K . F_n \quad (8)$$

where K= 2/3 for all perimeter BCJ or K = 1 for corner BCJ, described in para 2 of page 5, F_{nw} is the yield force in the beam reinforcement or $F_{nw} = \sum A_{st} f_y$, A_{st} is total area and f_y is the yield stress of stirrups, F_n is the shear force $V_{u,joint}$ at BCJ (Figure 4), N_n is the axial force acting on the column, if any. The value 5/8 is portion of depth of beam where the stirrups are effective.
As the angles Θ_1 and Θ_2 between the struts and ties decrease, Figure 7, the aspect ratio increases, it is therefore desirable to introduce vertical central bar when $f_{cu} \geq 60$MPa and $\Theta_1 \leq \tan^{-1}0.5$.
Looking at equation (8), when $h_b/h_c \leq 1.25$ no joint stirrups would be required, this is checked with Wilson's experimental results which has h_b/h_c=1, table 2. Without stirrups the shear index is 0.54, which is the same as the predicted Figure to design proposal rule of equation (7) when no central vertical reinforcement, dowel bars, are used because $\Theta_1 \leq \tan^{-1}0.5$.

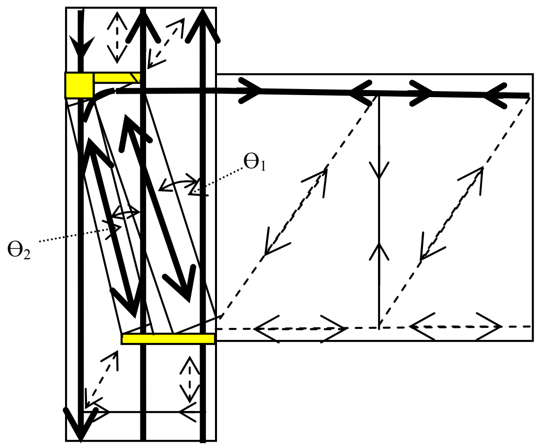


Figure 7. Strut and tie model for BCJ with central vertical reinforcement

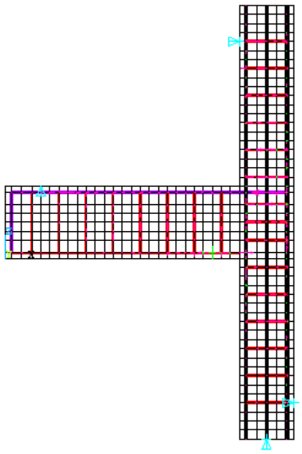


Figure 8. FEM parametric model of Ortiz BCJ4 with central vertical bar

The reason $(f_c)^{2/3}$ $b_e.h_c$ was taken for concrete contribution in the proposed rule is the result of the comparison with $(f_c)^{1/2}$ $b_e.h_c$. The accuracy of EC8-NA for predictions compared to experiments was 50% as compared to ACI352 [2] which was 41%, Figure 6.

VC joint shear from the concrete compression strut action is $Vc=0.54fc^{2/3}b_eh_c$ for L detailing shown in Figure 5, and $Vc = 0.49\ fc^{2/3}b_eh_c$ for U detailing shown in Figure 4.

4. CONCLUSION

1. A design method has been developed, based on statistical data of published 56 test results of BCJ, to calculate the shear resistance in HSC and NSC beam-column joint.
2. The proposed equation is a function of aspect ratio and the magnitude of shear force in BCJ and lower-bound theorem of plasticity maintained.
3. The results given by the proposed design equation are 79% of the total actual experimental data while the results produced from EC8 provided only 21% of the actual experimental results (assuming the experimental results is equal 1).

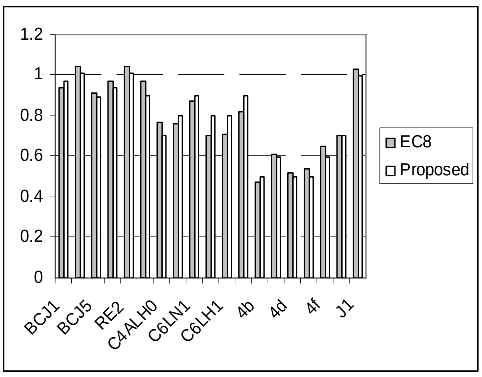


Figure 9. Design rule for all specimens without shear reinforcement

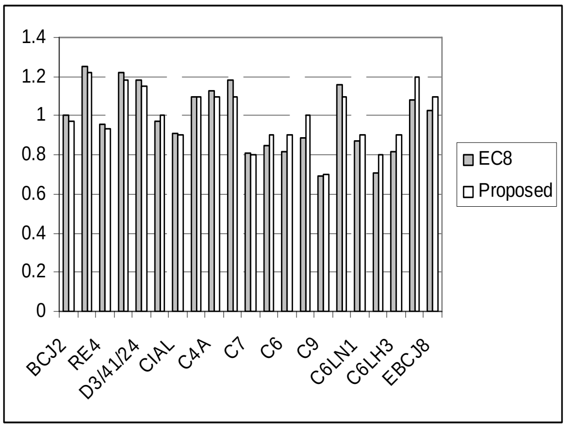


Figure 10. Design rule for all specimens with minimum shear reinforcement

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