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Effect of Web Reinforcement on High-Strength Concrete Deep Beams




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Results of an experimental investigation on the behavior and ultimate shear strength of 18 high strength concrete deep beams are summarized. The concrete cylinder compressive strength f_c' ranges from 55 to 86 MPa (approximately 8000 to 12,500 psi). The test specimens are divided into three series based on the shear-span-to-overall-height ratio a/h. Each series consists of six beams with different arrangements of horizontal and vertical web reinforcements, i.e., the main variables are the horizontal and the vertical web steel ratios. Observations are made on mid-span deflections, crack widths, failure modes and ultimate strengths. The test results show that for deep flexural members with a/h exceeding 1.00 (or shear-span-to-effective-depth ratio a/d ≥ 1.13), the vertical web reinforcement is more effective than the horizontal web reinforcement. It is also shown that orthogonal web reinforcement comprising both vertical and horizontal reinforcements is the most efficient in increasing the beam stiffness, restricting the diagonal crack width development and enhancing the ultimate shear strength.

The test results are then compared with the ultimate strength predictions obtained using the current ACI Code, the Canadian Code, and the UK CIRIA Guide. The deep-beam provisions in the ACI Code overestimate the contribution of the horizontal web steel to shear strength. Based on the test results, a revision to ACI Eq. (11-31) for web steel contribution is suggested. The Canadian Code shows the most consistent and yet conservative predictions of the test beams with different web reinforcements, while the UK CIRIA Guide is unconservative for beams with horizontal web reinforcement.

Keywords: building codes; cracks; deep beams; deflections; diagonal cracking; high-strength concrete; shear span; shear strength; ultimate strength; web reinforcement.

INTRODUCTION

The design of reinforced concrete deep beam is a subject of considerable interest in structural engineering practice. It has various structural applications ranging from pile-caps and wall foundations, to transfer girders in tall buildings. Despite its wide structural applications, many national codes do not include the design of deep beams. For instance, the British Standards BS 8110[1] explicitly states that "for the design of deep beams, reference should be made to specialist literature". That design document may well refer to the CIRIA Guide-2,[2] issued by the UK Construction Industry Research and Information Association. Besides the UK CIRIA Guide-2, the ACI Building Code,[3] and the 1984 Canadian Code[4] also provide guidance for the deep beam design.

In the ACI Code, the empirical equations governing deep beam design are based on low-strength concrete specimens with f_c' in the range of 14 to 40 MPa (2000 to 6000 psi). The same can be said of the design provisions in the UK CIRIA Guide-2 and the Canadian Code. As high-strength concrete (HSC) is becoming more and more popular, it is timely to evaluate whether these design documents can still provide safe design for HSC deep beams; HSC in this context refers to concrete with f_c' greater than 55 MPa (8000 psi). Previous work[5] shows that the ACI design equations and the CIRIA Guide-2 are applicable for deep beams with nominal web reinforcement and with f_c' exceeding 40 MPa. This paper further investigates the applicability of the codes for HSC deep beams with a significant amount of web reinforcement. There has been relatively limited information on this aspect; most reported investigations are on HSC beams without web reinforcement.[6-9] In the literature on HSC beams with web reinforcement,[10,11,12] the emphasis is on the shallow beam category (with $a/d \geq 2.50$), with only limited test data in the short/deep beam category. The present investigation seeks to supplement such information since the effect of vertical and horizontal web reinforcements on the behavior of low-strength concrete deep beams has already been shown[13-15] to be significant. Therefore, in the test program, the vertical and the horizontal web steel ratios ρ_v and ρ_h are considered as two main variables. The test results are then compared with the code predictions.

RESEARCH SIGNIFICANCE

High-strength concrete is being used more and more widely in the construction industry in recent years. Experimental results described in this paper give further empirical evidence on the behavior of HSC deep beams with f_c' greater than 55 MPa (8000 psi). In particular, the test program investigates the individual and combined effects of vertical and horizontal web reinforcements on HSC deep beams. The study reveals that increasing the horizontal or vertical web steel ratio can considerably increase the ultimate shear strength of the beams. In addition, when a/h exceeds 1.00 (equivalent to $a/d \geq 1.13$), the vertical web steel is more effective than the horizontal web steel. The current ACI Code 318-89 and the UK CIRIA Guide-2 may yield uncon-

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servative strength predictions for HSC deep beams with significant amount of horizontal web steel. A revision to the ACI Code Eq. (11-31) is suggested, based on the test results. The Canadian Code gives the most consistent and safe estimations of the ultimate shear strengths of these 18 beams, as the Code does not take account of the contribution of web reinforcement to shear strength.

EXPERIMENTAL PROGRAM

Beam notation

This paper addresses the effect of different web reinforcements and a/h ratios on the shear behavior of HSC deep beams. In the beam notation under Table 1 the series number is given first; this is followed by the type of web reinforcement and shear-span-to-overall-height ratio a/h. For example, Beam II-5/1.00 refers to a specimen in Series II, with Type 5 web reinforcement and an a/h ratio of 1.00. Figure 1 shows the different types of web reinforcement investigated.

It should be noted that the web reinforcement details in the two shear spans of Type 2 and Type 6 specimens are different (Table 1). The shear span provided with lower strength plain vertical web steel is designated as the "N" span while the other shear span with high strength deformed vertical web steel is designated as the "S" span. For example, I-2N/0.75 refers to the "N" shear span of the specimen I-2/0.75, while I-2S/0.75 refers to the "S" shear span of the same beam.

Beam details

This program consisted of 18 rectangular beams of 500 mm (19.50 in.) height and 110 mm (4.29 in.) width. Full details are given in Table 1. Each beam had a longitudinal main steel ratio ρ of 2.58 percent, consisting of four 20-mm (0.78-in.) diameter high strength deformed bars. At each location of loading or support point, a built-in reinforcement cage was placed to prevent premature bearing failure (Fig. 1). The effective span l_e varied from 1750 to 2500 mm (68 to 98 in.). The shear span a varied from 375 to 750 mm (14.6 to 29 in.), resulting in three a/h ratios. Based on the a/h ratios, the 18 beams were divided into three series of six beams each: Series I for a/h = 0.75 (a/d = 0.85), Series II for a/h = 1.00 (a/d = 1.13) and Series III for a/h = 1.50 (a/d = 1.69).

Three types of steel bars were used as reinforcements (Table 1): (i) 20 mm (0.8 in.) diameter high strength deformed bars with yield strength f_y = 498.9 MPa (72 ksi) as the main longitudinal reinforcement, (ii) 10-mm (0.4-in.) diameter high strength deformed bars with f_y = 446.7 MPa (65 ksi) as vertical or horizontal web bars, and (iii) 10 mm (0.4 in.) diameter plain mild steel round bars with f_y = 353.2 MPa (51 ksi) as vertical or horizontal web bars. Within each series, each beam had different web reinforcement details, as shown in Fig. 1.

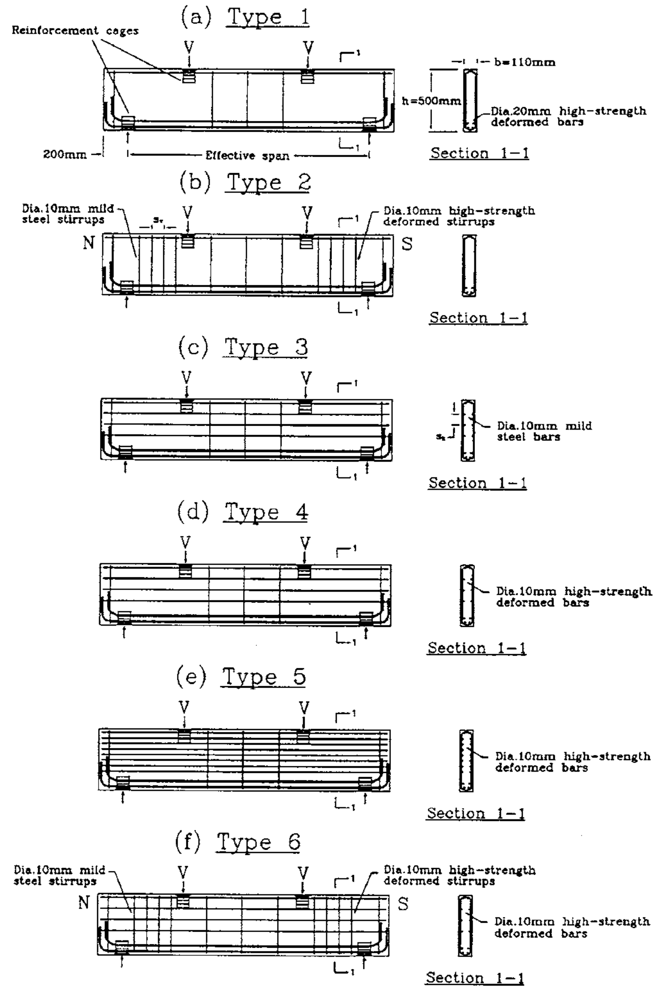

Fig. 1—Different types of web reinforcements.

Type 1: This specimen with no web reinforcement in either shear span served as a *control beam*.

Type 2: Vertical web steel consisting of lower strength plain bars was placed in the "N" shear span whereas vertical web steel of high strength deformed bars was placed in the "S" shear span. The vertical web steel ratio in specimens II-2N/1.00, II-2S/1.00, III-2N/1.50 and III-2S/1.50 was kept at 1.43 percent, while specimens I-2N/0.75 and I-2S/0.75 had ρ_v of 2.86 percent due to the fact that Series I specimens had the shortest shear span. In this manner, both the effects of vertical web reinforcement ratio ρ_v and its yield strength f_{yv} could be investigated.

Type 3: Horizontal web reinforcement consisting of mild steel bars was provided, giving ρ_h = 1.59 percent. The vertical spacing between each layer of the bars was 90 mm (3.50 in.).

Type 4: Horizontal web reinforcement was identical to Type 3 specimen, except that high strength deformed bars were used instead of lower strength plain bars.

Type 5: Horizontal web steel ratio using high strength deformed bars was doubled to ρ_h = 3.17 percent; the vertical

Table 1—Specimen details of Series I, II, and III

Beam*	Effective span l_e, mm	Shear span a, mm	a/h ratio	Effective depth d mm	a/d ratio	f_{cu} N/mm^2	f'_c N/mm^2	f_t N/mm^2	f_y N/mm^2	ρ, percent	f_{yv}, N/mm^2	s_v, mm	ρ_v, percent	f_{yh}, N/mm^2	s_h, mm	ρ_h, percent	Age of beams
I-1/0.75	1750	375	0.75	442.5	0.85	99.4	56.3	4.2	498.9	2.58	0	—	0	0	—	0	146
I-2N/0.75						86.8	56.2	6.4			353.2	50	2.86	0	—	0	162
I-2S/0.75						86.8	56.2	6.4			446.7	50	2.86	0	—	0	162
I-3/0.75						78.8	59.2	6.6			0	—	0	353.2	90	1.59	160
I-4/0.75						92.3	63.8	4.0			0	—	0	446.7	90	1.59	149
I-5/0.75						92.7	57.6	3.7			0	—	0	446.7	45	3.17	155
I-6N/0.75						80.9	59.7	5.4			353.2	50	2.86	446.7	90	1.59	168
I-6S/0.75						80.9	59.7	5.4			446.7	50	2.86	446.7	90	1.59	168
II-1/1.00	2000	500	1.00	442.5	1.13	88.2	77.6	3.3	498.9	2.58	0	—	0	0	—	0	57
II-2N/1.00						88.2	77.6	3.3			353.2	100	1.43	0	—	0	56
II-2S/1.00						88.2	77.6	3.3			446.7	100	1.43	0	—	0	56
II-3/1.00						80.5	78.0	3.4			0	—	0	353.2	90	1.59	70
II-4/1.00						89.4	86.3	3.3			0	—	0	446.7	90	1.59	63
II-5/1.00						89.4	86.3	3.3			0	—	0	446.7	45	3.17	63
II-6N/1.00						91.1	75.3	5.8			353.2	100	1.43	446.7	90	1.59	180
II-6S/1.00						91.1	75.3	5.8			446.7	100	1.43	446.7	90	1.59	180
III-1/1.50	2500	750	1.50	442.5	1.69	88.2	77.6	3.3	498.9	2.58	0	—	0	0	—	0	56
III-2N/1.50						88.2	77.6	3.3			353.2	100	1.43	0	—	0	56
III-2S/1.50						88.2	77.6	3.3			446.7	100	1.43	0	—	0	56
III-3/1.50						80.5	78.0	3.4			0	—	0	353.2	90	1.59	70
III-4/1.50						89.4	86.3	3.3			0	—	0	446.7	90	1.59	63
III-5/1.50						89.4	86.3	3.3			0	—	0	446.7	45	3.17	63
III-6N/1.50						91.1	78.9	5.6			353.2	100	1.43	446.7	90	1.59	182
III-6S/1.50						91.1	78.9	5.6			446.7	100	1.43	446.7	90	1.59	182

*Beam notation—The series number is given before the hyphen; this is followed by the different type of web reinforcement and then by the a/h ratio. In Type 2 and Type 6 beams, the shear span provided with mild steel stirrups is designated as the "N" span while the other span with stirrups made of high strength deformed bars is designated as the "S" span.

Table 2—Summary of concrete mix design

Characteristic cube strength	80 MPa
Cement type	ordinary portland cement
Aggregate type	crushed granite and natural sand
Slump for concrete	150 to 200 mm
Free water content	150 kg/m^3
Cement content	525 kg/m^3
Coarse aggregate content	950 kg/m^3
Fine aggregate content	690 kg/m^3
Water-cement ratio (w/c)	0.27
Aggregate/cement ratio (a/c)	2.90
Admixture I	350 cc per 100 kg of cement
Admixture II	1700 cc per 100 kg of cement

Note: Admixture I = POZZOLITH 300N, a water-reducing plasticizing admixture for concrete; Admixture II = Rheobuild 1000.

spacing of the bars was reduced from 90 to 45 mm (3.50 to 1.75 in.).

Type 6: Here, Type 2 vertical web steel and Type 4 horizontal web steel were combined to investigate the effect of an orthogonal web reinforcement.

Since the purpose in this program was to evaluate the effectiveness of vertical and horizontal web steel (A_v and A_h respectively), the minimum requirements specified by the ACI Code for A_v and A_h were waived. In addition, the width of the bearing block was 150 mm (5.85 in.) at the two loading points and 100 mm (3.90 in.) at the two support points. Thus, the clear shear spans x_e of Series I, II, and III beams were 250, 375 and 625 mm respectively (9.75, 14.63, and 24.38 in.).

Concrete mix design

The concrete for Series I, II, and III specimens were cast from the same batch. The concrete mix design is given in Table 2. The design strength was specified in terms of the characteristic concrete cube strength, i.e., f_{cu} = 80 MPa (11600 psi). Ordinary portland cement was used, with an aggregate to cement ratio of 2.90 and a water-cement ratio of 0.27. To avoid congestion in reinforcement, 10-mm (0.39 in.) chippings were used as aggregates. Two types of water-reducing plasticizing admixture were added to maintain workability at a design slump of 150 to 200 mm. In Table 1, the cube and cylinder compressive strengths f_{cu} and f'_c were obtained from the averages of four 100 mm (3.90 in.) concrete cubes and four 150 mm (5.85 in.) diameter concrete cylinders respectively. The f'_c values of Series I were lower than those of Series II and III, probably due to poor preparation of the capping material. On the other hand, the f_{cu} tests (which did not require any capping) were consistent for all three series. Two other cylinders were used to evaluate the splitting tensile strength f_t. Series II and III specimens were tested first, followed by Series I. Clearly, from Table 1, all the specimens were made of HSC with f'_c > 55 MPa (8000 psi).

Test procedure

Vertical deflections were monitored by the LVDTs. At each load increment, the test data were captured by a data logger and automatically stored. All the beams were tested to failure under two-point symmetric top loading. Specimens with Type 2 (only ρ_v) and Type 6 (combined ρ_v and ρ_h) web reinforcements were tested twice. After the first shear span had failed, the beam was externally clamped by a steel yoke. The specimen was then re-loaded to failure.

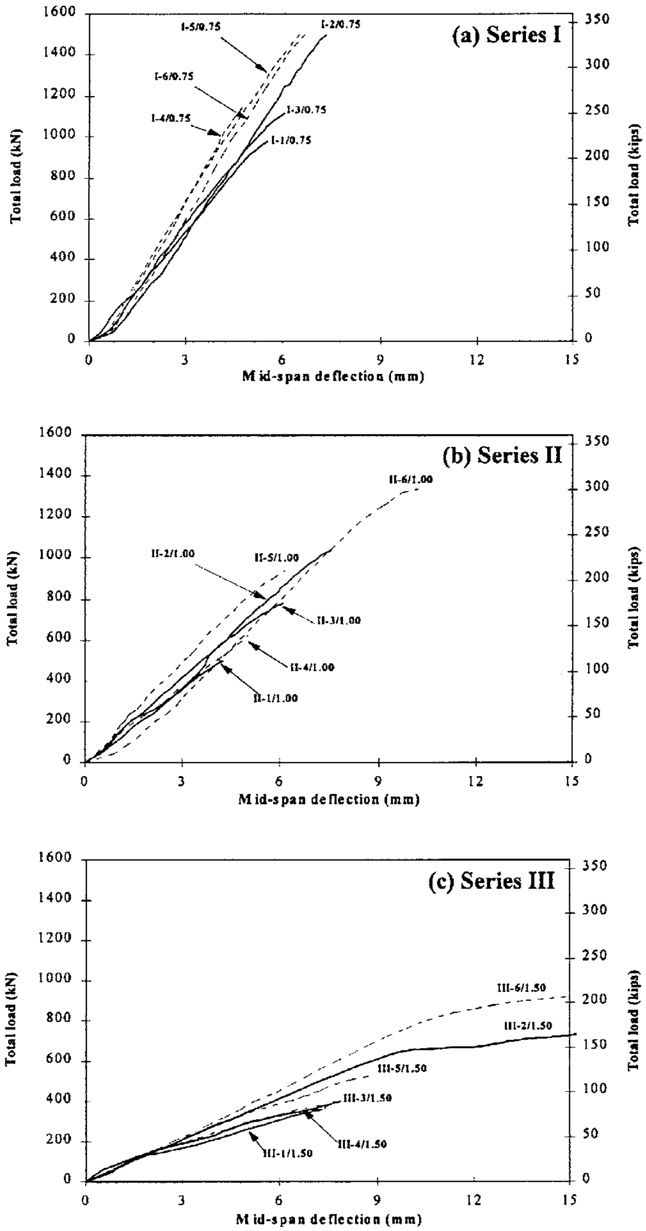

Fig. 2—Mid-span deflection of Series I, I, and III specimens.

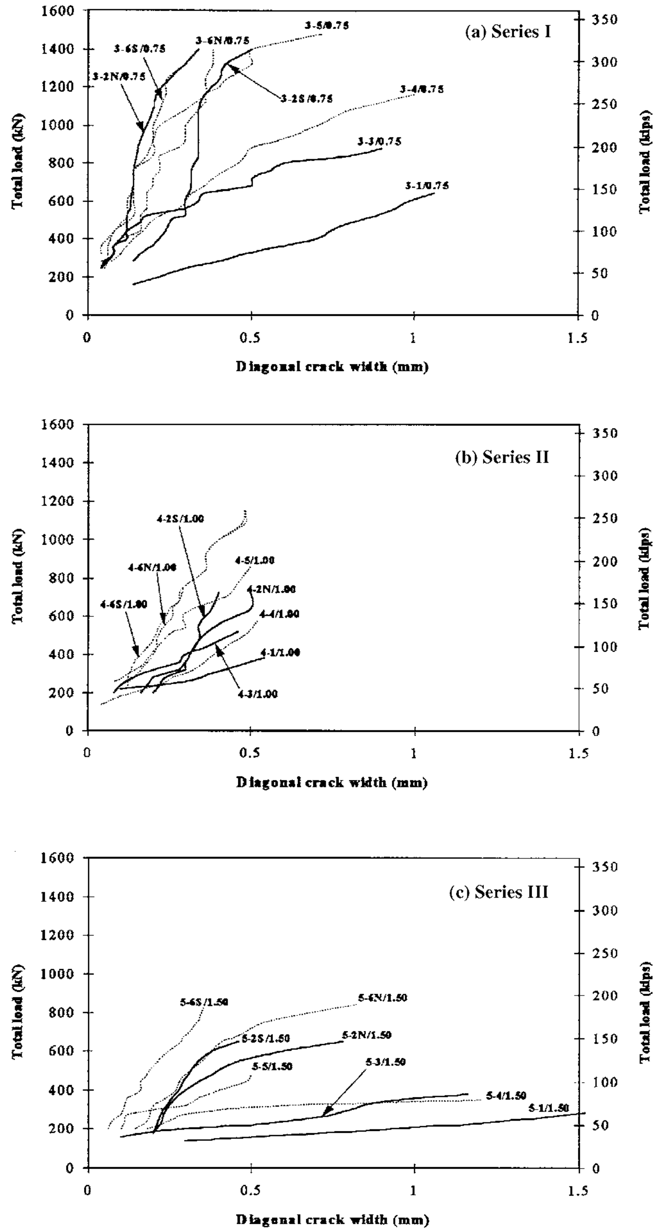

Fig. 3—Diagonal crack width development: Series I, II, and III beams).

Both surfaces of the beam were white-washed to aid observations of crack development during testing. Initial loading was applied at an increment of 20 kN (4.5 kips) for each jack until the first crack occurred. Subsequently, the load increment was increased to 40 kN (9 kips) for each jack.

TEST RESULTS AND DISCUSSIONS

Deflections

The mid-span deflection curves for Series I, II, and III beams are shown in Fig. 2(a), (b) and (c) respectively. It is observed that in Series I (a/h = 0.75), beams with high strength horizontal web reinforcement (Types 4 and 5) appear to be stiffer than beams with lower strength horizontal web bars (Type 3) or with only vertical web steel (Type 2). However, this is not so in Series III (a/h = 1.50); Beam III-2/1.50 is stiffer than the corresponding beams III-4/1.50 and III-5/1.50. Thus, it is deduced that the effect of vertical web reinforcement on beam stiffness is more significant when a/h exceeds 1.00 ($a/d \geq 1.13$). The effect of orthogonal web reinforcement on stiffness is also more dominant at $a/h \geq 1.00$.

Crack widths

In all the tests, the first crack in a specimen was a flexural crack of 0.02 to 0.04 mm (0.008 to 0.016 in.) wide initiated from the beam soffit in the mid-span region. Compared to the diagonal cracks, the flexural cracks were narrower in width although some of them exceeded the serviceability limit of 0.33 mm (0.013 in.) (specified by the ACI Code) towards failure.

Figures 3(a) through (c) present the diagonal crack development for Series I, II, and III beams. Among the three series, the fastest development rate of the diagonal crack occurred in Series III specimens with the highest a/h. Within each series, different web reinforcements led to different rates of diagonal crack development. Diagonal cracks propagated rapidly in Type 1 control beams with no web reinforcement. They also developed more quickly in Type 3

Table 3—Test results of Series I, II, III specimens

Beam*	$2V_{cr}$	$2V_{ser}$	$2V_n^{TEST}$	$\frac{V_{cr}}{V_n^{TEST}}$	$\frac{V_{ser}}{V_n^{TEST}}$	$2V_s$, kN	Failure mode
	kN						
I-1/0.75	160	240	1000	0.16	0.24	0	Diagonal-splitting
I-2N/0.75	240	1320	1520	0.16	0.87	520	Crushing of strut
I-2S/0.75	280	520	—	—	—	—	—
I-3/0.75	240	560	1120	0.21	0.50	120	Crushing of strut
I-4/0.75	280	600	1160	0.24	0.52	160	Shear-compression
I-5/0.75	280	920	1550	0.18	0.59	550	Crushing of strut
I-6N/0.75	240	1080	—	—	—	—	—
I-6S/0.75	320	1320	1550	0.21	0.85	550	Diagonal-splitting
II-1/1.00	220	260	510	0.43	0.51	0	Diagonal-splitting
II-2N/1.00	200	360	1040	0.19	0.35	530	Diagonal-splitting
II-2S/1.00	200	320	—	—	—	—	—
II-3/1.00	200	400	780	0.26	0.51	270	Diagonal-splitting
II-4/1.00	140	300	660	0.21	0.45	150	Diagonal-splitting
II-5/1.00	260	540	940	0.28	0.57	430	Diagonal-splitting
II-6N/1.00	240	720	1340	0.18	0.54	830	Crushing of strut
II-6S/1.00	240	760	—	—	—	—	—
III-1/1.50	140	140	370	0.38	0.38	0	Diagonal-splitting
III-2N/1.50	180	400	670	0.27	0.60	300	Diagonal-splitting
III-2S/1.50	180	460	800	0.23	0.58	430	Shear-compression
III-3/1.50	160	200	400	0.40	0.50	30	Diagonal-splitting
III-4/1.50	200	280	380	0.53	0.74	10	Diagonal-splitting
III-5/1.50	200	320	530	0.38	0.60	160	Diagonal-splitting
III-6N/1.50	200	480	920	0.22	0.52	550	Shear-compression
III-6S/1.50	200	680	—	—	—	—	—

*Beam notation as in Table 1

(ρ_h = 1.59 percent) and Type 4 (ρ_h = 3.17 percent) specimens than in Type 2 (ρ_v = 1.43 percent or 2.86 percent) beams, suggesting that the vertical web steel was more effective than horizontal web steel. In all the three series, Type 6 orthogonal web reinforcement was the most effective in controlling the diagonal cracks.

Table 3 presents the diagonal cracking strengths ($2V_{cr}$), serviceability loads ($2V_{ser}$) and ultimate strengths ($2V_n^{TEST}$) of Series I, II, and III specimens. Generally, from Table 3, the diagonal cracking strength remains more or less constant within the same series, relatively undisturbed by the type of web reinforcement. It is also observed that V_{cr} decreases with increasing a/h.

Serviceability limit

In this context, the serviceability load V_{ser} is defined as the load corresponding to an observed crack width of 0.33 mm (0.013 in.) as specified in the ACI Code. Figure 4 shows the

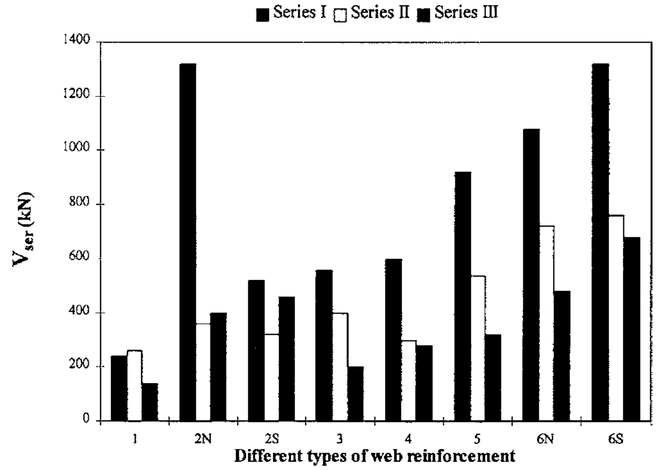

Fig. 4—Serviceability strengths of Series I, II, and III specimens.

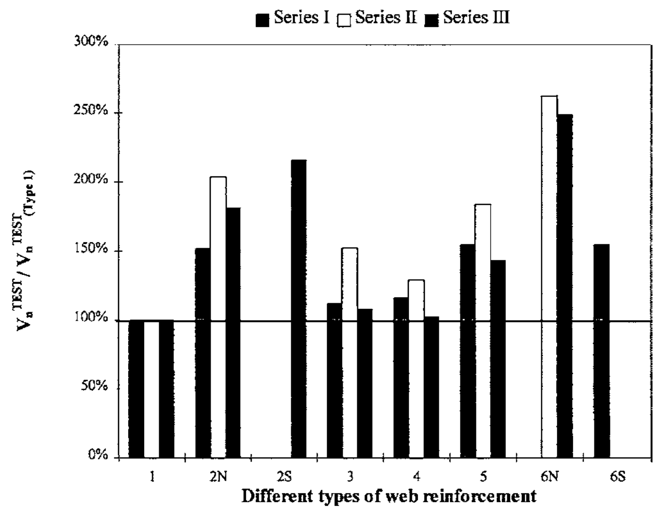

Fig. 5—Ultimate shear strengths of Series I, II, and III specimens.

total serviceability loads of all 18 specimens. On the whole, V_{ser} is around 24 to 87 percent of the ultimate failure loads V_n^{TEST} (Table 3). Beams with different web reinforcements attain different serviceability limits. The greatest V_{ser} is associated with Type 6 orthogonal web reinforcement, whereas the least is V_{ser} with Type 1 control beams. Note that in Series II and III, the combined web reinforcement ratios of Type 6 specimens (ρ_v and ρ_h) add up to 3.02 percent, which is less than ρ_h of 3.17 percent in Type 5 beams in the two series. This confirms that orthogonal web reinforcement offers the most effective restraint on diagonal crack development of HSC deep beams. On the other hand, the diagonal cracks in control beams without any web reinforcement can exceed the serviceability limit very quickly after the first cracking. Thus, to satisfy the serviceability requirement, it is prudent to provide nominal web reinforcement in the shear span, even though it may not be required by calculations.

Ultimate strength

Figure 5 compares the percentage ratios of the ultimate strengths (V_n^{TEST}) of different beams to that of Type 1 beam ($V_{n(Type1)}^{TEST}$) within each series; for the three control specimens, $V_n^{TEST}/V_{n(Type1)}^{TEST}$ = 100 percent. This comparison is valid as within each series there is no significant variation in the concrete strengths (Table 1). The term $V_{n(Type1)}^{TEST}$ represents the pure concrete contribution to shear strength V_c. To obtain the web steel contribution to shear strength V_s, $V_{n(Type1)}^{TEST}$ is subtracted from V_n^{TEST} of each beam in a series. The values of V_s for different types of web reinforcement are shown in the seventh column of Table 3. Clearly, the web steel contribution associated with Type 6 orthogonal web reinforcement is the highest, followed on by the Type 2 vertical web steel and the Type 5 heavy horizontal web steel. Thus, the effective contributions of different types of web reinforcements can be ranked in a descending order as follows: Type 6 > Type 2 > Type 5 > Type 3 ≈ Type 4. The failure loads of specimens with high strength horizontal web reinforcement (Type 4) were about the same as those of beams with smooth horizontal bars (Type 3). But beams with double the horizontal web reinforcement ratio (Type 5) achieved significantly greater load as compared with either Type 3 or Type 4 specimens. However, when $a/h \geq 1.00$, Type 5 specimens (ρ_h = 2.58 percent) with high strength deformed horizontal bars, had smaller V_s as compared with Type 2 beams (ρ_v = 1.43 percent) with low strength smooth bars, showing that the vertical web reinforcement was more effective. Thus, it is confirmed that the effect of horizontal web reinforcement on shear strength diminishes when a/h exceeds 1.00.

There was a total of six specimens with two different web reinforcements in both shear spans (Type 2 and Type 6 specimens from the three series in Table 1). The initial objective was to compare the shear contributions from high strength deformed bars and plain mild steel bars. In the six tests, after the first shear span had failed, the beams were externally clamped with steel yokes and re-loaded to failure. Unfortunately, only Beam III-2/1.50 was successfully retested; two values of V_n^{TEST} were obtained for both shear spans (Table 3). Re-testing was not successful for the other five beams as the concrete strut linking the loading and support points in the failed shear span was severely damaged.

Apart from Beam I-6/0.75, the other five specimens failed first in the "N" spans which were reinforced with lower strength vertical web steel, implying that the ultimate strengths of the "S" spans with high strength deformed bars were higher. This indicates that the shear strength contributions of high strength deformed bars are greater than that of mild steel bars.

Modes of failure

The failure modes of the 18 specimens are indicated in the last column of Table 3. Three failure modes are identified, i.e. crushing of strut failure, diagonal-splitting failure, and shear-compression failure. In the crushing of strut failure, there normally exist more than one inclined cracks. The concrete portion between the inclined cracks is in compression, forming a concrete compression strut, which crushes under high compression. This mode of failure is brittle and sudden. An equally brittle failure mode is the shear-compression mode in which after the appearance of the inclined crack, the concrete portion above the upper end of this crack experiences high compression. When the inclined crack further propagates upward, the concrete above the crack fails by crushing, accompanied by a loud noise. The diagonal splitting mode is a less brittle mode in comparison, characterized by a critical diagonal crack joining the outside edge of the bearing block at loading point and the inside edge of the bearing block at support point. No explosive sound was heard for this mode of failure, which was akin to the splitting failure of a concrete cylinder in a tensile splitting test.

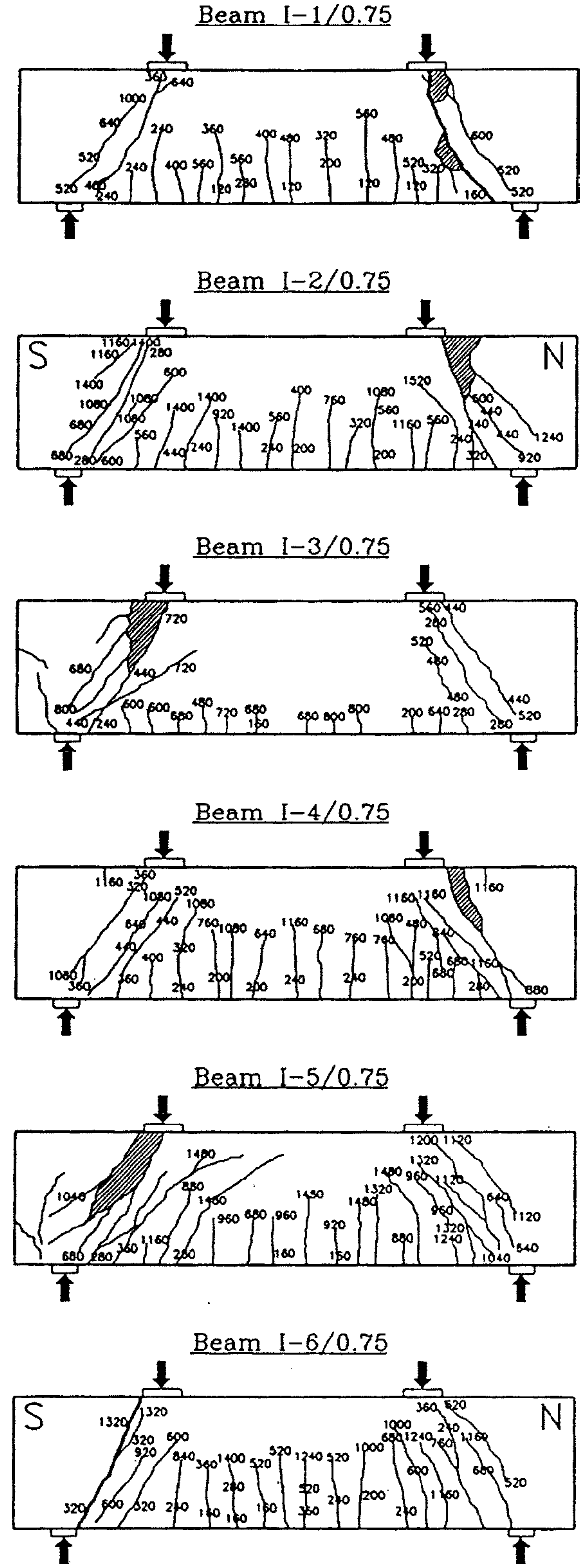

Fig. 6(a)—Crack patterns of Series I specimens.

The crack patterns at failure of Series I, II and III specimens are shown in Fig. 6(a), (b) and (c) respectively. For Series I specimens, Beams I-1/0.75 and I-6/0.75 failed in the diagonal-

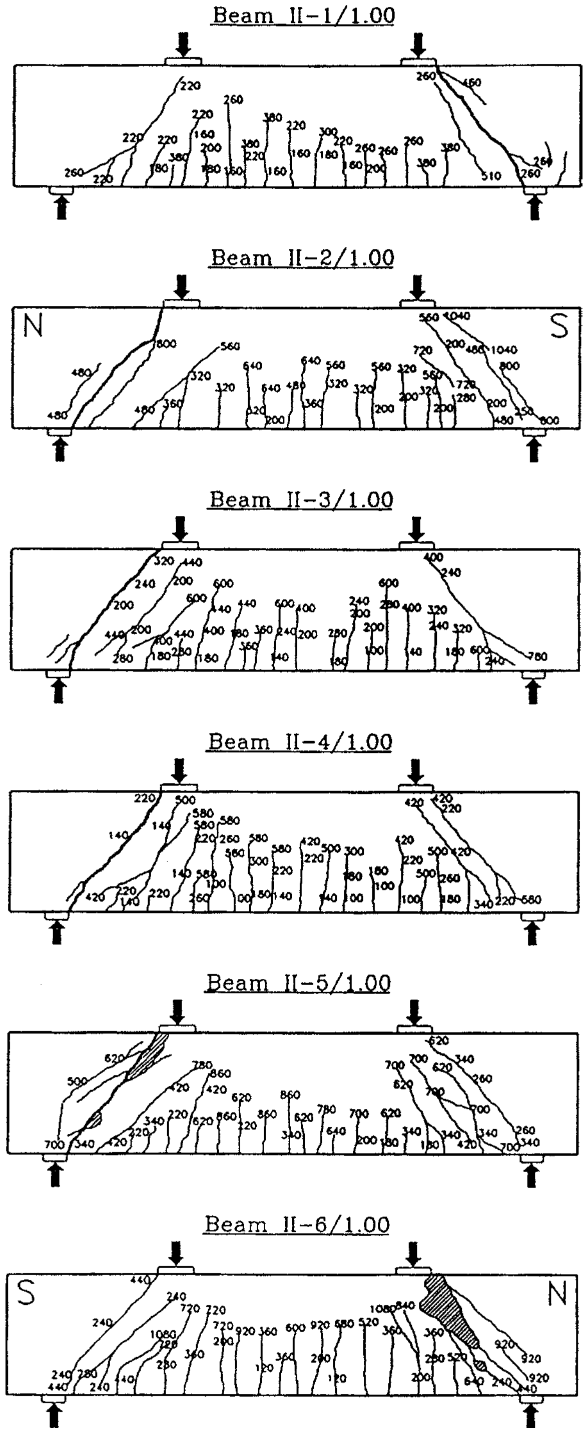

Fig. 6(b)—Crack patterns of Series II specimens.

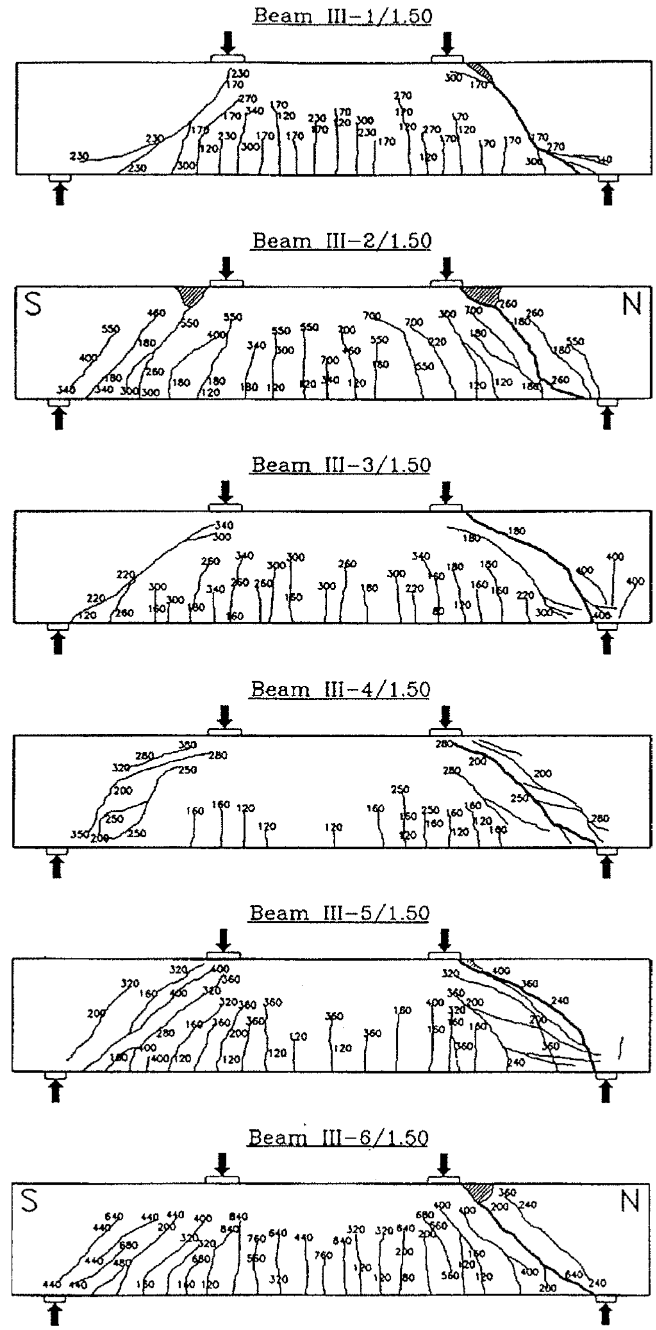

Fig. 6(c)—Crack patterns of Series III specimens.

splitting mode. The failure was not as explosive as the crushing of strut mode experienced by Beams I-2/0.75, I-3/0.75, and I-5/0.75 in which loud noise could be heard. An equally abrupt failure was that of Beam I-4/0.75, which failed in shear-compression mode. The failure modes of Series II and III specimens are indicated in Table 3.

COMPARISON OF TEST RESULTS WITH SHEAR DESIGN EQUATIONS

Three design methods, namely, the ACI Code 318-89,[3] the Canadian CSA Code,[4] and the UK CIRIA Guide-2[2] are used to estimate the ultimate shear strengths of the 18 specimens. The results are given in Table 4 and Fig. 7. Only the relevant deep beam design equations are included in this paper. The meanings of the notations used are explained under the list of symbols.

ACI Building Code (ACI 318-89)

These design equations are applicable to beams with l_n/d less than 5. However, for Series III beams in which l_n/d exceeds 5, the same two equations are used for comparison. This is because the shear span-depth ratio a/d of Series III beams is 1.69 (Table 1), much less than 2.5 which is generally[16] taken to be the transition point between a deep beam and a shallow beam. The shear design at the critical section (at distance $a/2$ from the support) is based on:

$$V_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(1.9\sqrt{f_c'} + 2500\rho_w\frac{V_u d}{M_u}\right)b_w d \quad \text{(ACI Eq. [11-30])} \qquad (1)$$

$$V_s = \frac{A_v}{s}\left[\left(\frac{1 + l_n/d}{12}\right) + \frac{A_{vh}}{s_2}\left(\frac{11 - l_n/d}{12}\right)\right]f_y d \quad \text{(ACI Eq. [11-31])} \qquad (2)$$

The usual restrictions on V_c, V_s, and V_n are imposed. It should be noted that in ACI Eq. (11-31), the coefficients in parenthesis are weighting factors[16] for the relative effectiveness of the vertical and horizontal web steel. At $l_n/d = 5$ (the limit of deep beams according to the ACI Code), vertical and horizontal web steel are taken to be equally effective, i.e. the vertical web steel contribution V_{sv} is equal to the horizontal web steel contribution V_{sh} provided A_v/s is equal to A_{vh}/s_2. As l_n/d decreases below 5, the horizontal web steel becomes more effective.

CIRIA Guide-2 "Supplementary Rules"

The CIRIA Guide-2 method was based on Kong's work[13,14] in the seventies. Basically, it is applicable for the range of $0 \le x_e/h \le 0.7$. In this comparison, it is applied to specimens with $0.5 \le x_e/h \le 1.25$.

$$V_c = \lambda_1\left(1 - 0.35\frac{x_e}{h_a}\right)\sqrt{f_{cu}}\,b h_a \quad \text{(CIRIA Guide-2: Cl 3.4.2)} \qquad (3)$$

$$V_s = \lambda_2\left(\sum_{i=1}^{n}\frac{100 A_r y_r \sin^2\alpha_r}{b h_a^2}\right)b h_a \quad \text{(CIRIA Guide-2: Cl.3.4.2)} \qquad (4)$$

where $\lambda_1 = 0.44$ for normal weight concrete, and $\lambda_2 = 0.85$ MPa for plain round bars and 1.95 MPa for deformed bars. The meanings of the various notations are explained in the list of symbols.

Canadian CSA Code (CAN3-A23.3-M84)

The 1984 Canadian Code recommended a strut-and-tie design approach for deep beams. In a single-span deep beam with two-point top-loading, concrete compression "struts" would lie in between the upper nodal zone and the lower nodal zone. Each strut is inclined at an angle α_s to a horizontal tension tie, which represents the main longitudinal reinforcement. The compressive force of the inclined concrete strut is balanced at the lower nodal zone by the support reaction and the tension tie force, and at the upper nodal zone by the external load and the horizontal thrust. The Canadian Code stipulates that concrete compressive stresses in the nodal zones may not exceed certain stress limits.[4] The compressive stress f_2 in the inclined concrete strut should not exceed f_{2max} given by:

$$f_{2max} = \frac{\lambda\phi_c f_c'}{0.8 + 170\varepsilon_1} \le \lambda\phi_c f_c' \quad \text{(CAN3-A23.3-M84: Eq.11-19)} \qquad (5)$$

where f_{2max} is the diagonal crushing strength of concrete, $\lambda = 1.0$ for normal weight concrete, ε_1 is the principal tensile strain crossing the inclined concrete strut, and ϕ_c is set to unity.

Figure 7(a) shows that the ACI predictions are conservative for $a/h = 0.75$; the conservatism disappears for higher a/h and for beams with Type 4 and Type 5 horizontal web steel. Thus, it is deduced that the beneficial effect of horizontal web steel is overestimated by the current ACI Code. The reason for the discrepancy is that in ACI 318-89 Eq. (11-31) for web steel contribution, the threshold for l_n/d at which both horizontal and vertical web steel are equally effective, is probably unrealistic. By modifying the threshold l_n/d from 5 to 2.50 (thereby reducing the weighting factor from 11 to 6 for V_{sh}), a more consistent result is obtained, as shown in Fig. 7(b). The proposed revision for Eq. (11-31) is as follows

$$V_s = \left[\frac{A_v}{s}\left(\frac{1 + l_n/d}{12}\right) + \frac{A_{vh}}{s_2}\left(\frac{6 - l_n/d}{12}\right)\right]f_y d \qquad (6)$$

The revision of the threshold of web steel effectiveness is in line with the experimental observation for the 18 specimens. It has been shown that the horizontal web steel contribution is most effective in Series I beams (equivalent to $a/d = 0.85$); this effect diminishes for Series III beams (equivalent to $a/d = 1.69$). From linear interpolation, the threshold works out to be at $a/d = 1.27$ or thereabouts. If we assume the clear span l_n to be twice the shear span a (as in the case of central point load), then the value of the threshold l_n/d is about 2.5. From the last column of Table 4, it

Table 4—Summary of predictions: Series I, II, and III

Beam*	V_n/V_n^{TEST}, ACI Code	V_n/V_n^{TEST}, Canadian Code	V_n/V_n^{TEST}, CIRIA Guide-2	V_n/V_n^{TEST}, Modified method
I-1/0.75	0.36	0.79	0.78	0.36
I-2N/0.75	0.36	0.52	0.48	0.37
I-2S/0.75	—	—	—	—
I-3/0.75	0.51	0.73	0.62	0.43
I-4/0.75	0.51	0.75	0.65	0.45
I-5/0.75	0.36	0.52	0.49	0.36
I-6N/0.75	—	—	—	—
I-6S/0.75	0.37	0.53	0.46	0.37
II-1/1.00	0.78	1.36	1.45	0.78
II-2N/1.00	0.59	0.66	0.71	0.59
II-2S/1.00	—	—	—	—
II-3/1.00	0.87	0.89	0.90	0.61
II-4/1.00	1.08	1.13	1.13	0.78
II-5/1.00	0.76	0.79	0.79	0.65
II-6N/1.00	0.50	0.51	0.56	0.50
II-6S/1.00	—	—	—	—
III-1/1.50	0.61	0.90	1.39	0.61
III-2N/1.50	0.73	0.50	0.87	0.73
III-2S/1.50	0.70	0.42	0.84	0.70
III-3/1.50	1.20	0.84	1.29	0.63
III-4/1.50	1.47	0.95	1.45	0.71
III-5/1.50	1.46	0.68	1.13	0.57
III-6N/1.50	0.80	0.37	0.68	0.57
III-6S/1.50	—	—	—	—
Mean	0.74	0.73	0.88	0.57
SD	0.35	0.25	0.33	0.14
COV	0.47	0.35	0.38	0.25

Note: V_n refers to the predicted ultimate strength by various methods whereas V_n^{TEST} is the experimental failure load of the beam.
*Beam notation as in Table 1.

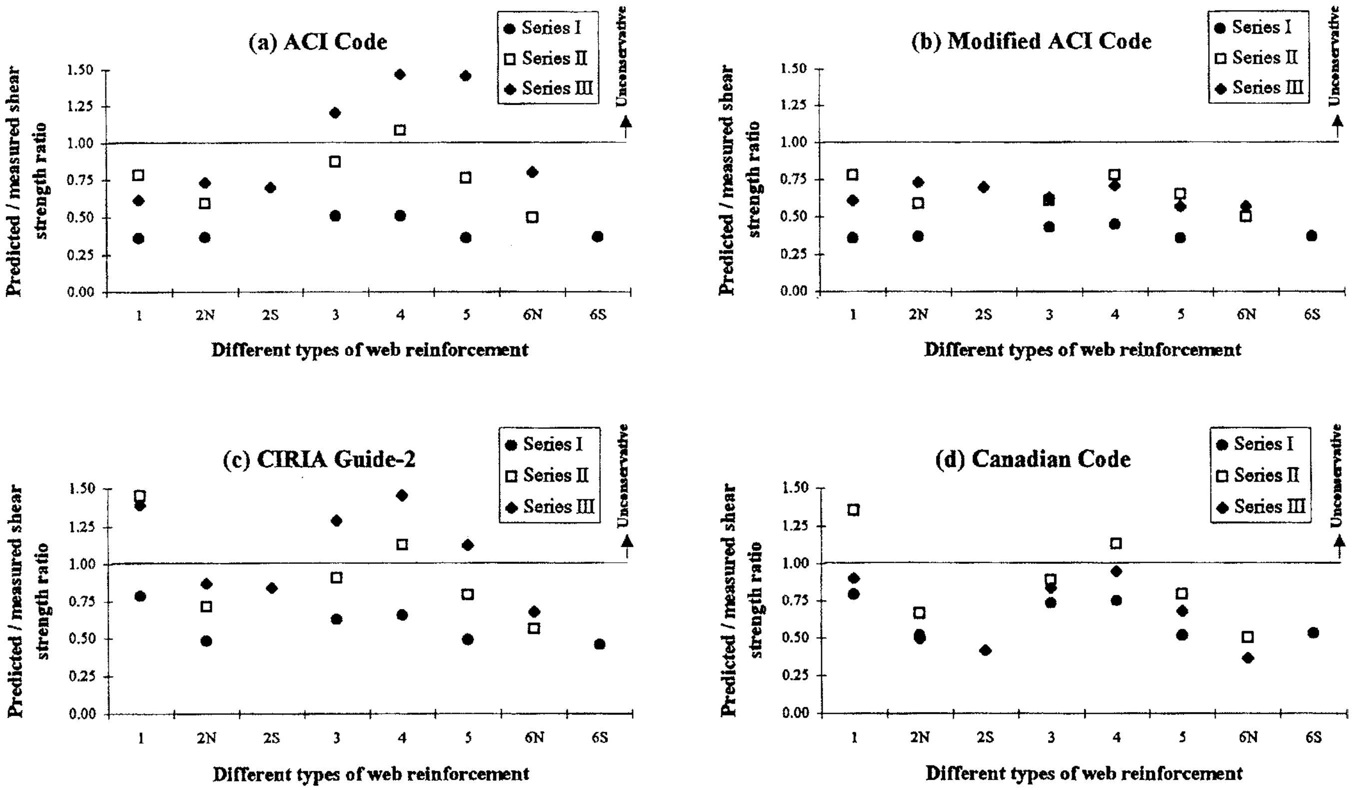

Fig. 7—Ultimate shear strength predictions by different design equations.

is shown that the modified ACI Eq. (11-31) (Eq. [6]) yields the least standard deviation among the three methods considered, and its conservatism is consistent for different types of web reinforcements.

The CIRIA Guide-2 predictions (Fig. 7[c]) also overestimate the benefit associated with ρ_h; the conservatism reduces with increasing a/h. Besides, the Guide seems to overestimate the ultimate shear capacity of Type 1 control beams without any web reinforcement, suggesting that the CIRIA Guide-2 may probably overestimate V_c for HSC deep beams. As for the 1984 Canadian Code, which is essentially a strut-and-tie approach, the shear strength estimations for these 18 specimens are generally conservative and consistent. This is expected as the Canadian Code does not take the contribution of web steel into account.

CONCLUSIONS

From the study of 18 HSC deep beams with six different web reinforcement details, the following conclusions are made:

1. It is evident that web reinforcement can play an important role for HSC deep beams. The most favorable pattern is the Type 6 orthogonal web reinforcement; it is the most effective in increasing the beam stiffness, restricting the diagonal crack width development (thereby increasing the serviceability load) and in increasing the ultimate shear resistance. At a/h = 1.50, the effectiveness of orthogonal web reinforcement is greater than the combined individual contributions of the horizontal and vertical web reinforcements (Column 7 in Table 3: $2V_s$ = 550 kN (Type 6) c.f. $2V_{sv}$ = 300 kN (Type 2) and $2V_{sh}$ = 10 kN (Type 4)).

2. For deep beams with $a/h \geq 1.00$ (equivalent to $a/d \geq 1.13$), the vertical web steel has greater effect on restraining the diagonal crack width and increasing the ultimate shear resistance of HSC deep beams than the horizontal web steel of the same steel ratio.

3. It is also confirmed that the web steel contribution of high strength deformed bars is significantly greater than that of lower strength plain mild steel bars.

The ACI Code overestimates the contribution of horizontal web reinforcement for Series III beams (a/h = 1.50). With a suitable revision to the threshold of web steel effectiveness (Eq. [6]), the conservatism of the predictions is maintained.

4. The UK CIRIA Guide-2 also gives unconservative predictions for specimens with high percentage of horizontal web bars. The Guide also overestimates the concrete contribution from high strength concrete.

The Canadian Code gives conservative predictions for most of the 18 specimens as the method does not take the web steel contribution into account.

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CONVERSION FACTORS

1 mm = 0.039 in
1 mm^2 = 0.00152 in^2
1 kN = 0.2248 kips
1 MPa = 145 psi

NOTATION

a	=	shear span measured from center of support to center of loading point
A_{vh}	=	area of shear reinforcement parallel to flexural tension reinforcement
A_r	=	area of reinforcing bar
A_s	=	area of main longitudinal reinforcement
A_v	=	area of shear reinforcement perpendicular to flexural tension reinforcement
b, b_w	=	beam thickness

d = effective depth
h = overall height of beam
h_a = active beam height (the lesser of h or l_e)
f_{cu} = concrete cube compressive strength
f_c' = concrete cylinder compressive strength
f_t = concrete cylinder tensile splitting strength
f_y = yield strength of reinforcement
f_{yv} = yield strength of vertical reinforcement
f_{yh} = yield strength of horizontal web reinforcement
l_e = effective span as measured from center to center of support points
l_n = clear span measured face-to-face of supports
s_2 = spacing of horizontal web reinforcement
s = spacing of vertical web reinforcement
V = shear force
V_c = calculated nominal shear strength provided by concrete
V_{cr} = measured diagonal cracking strength
V_n = nominal shear strength (= $V_c + V_s$)
V_n^{TEST} = measured ultimate shear strength
V_s = calculated nominal shear strength provided by shear reinforcement
V_{ser} = measured serviceability load (ACI 318-89: Clause 10.6.4)
V_{sh} = calculated nominal shear strength provided by horizontal web steel
V_{sv} = calculated nominal shear strength provided by vertical web steel
ρ = longitudinal main steel ratio (= A_s/bd)
ρ_v = vertical shear reinforcement ratio (= A_v/bs)
ρ_h = horizontal shear reinforcement ratio (= A_{vh}/bs_2)
x_e = clear shear span measured from face of support to face of loading point
y_r = the depth at which a typical web bar intersects a critical diagonal crack

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