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Web-shear capacity of prestressed hollow-core slab unit with consideration on the minimum shear reinforcement requirement

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Abstract. Prestressed hollow-core slabs (HCS) are widely used for modern lightweight precast floor structures because they are cost-efficient by reducing materials, and have excellent flexural strength and stiffness by using prestressing tendons, compared to reinforced concrete (RC) floor system. According to the recently revised ACI318-08, the web-shear capacity of HCS members exceeding 315 mm in depth without the minimum shear reinforcement should be reduced by half. It is, however, difficult to provide shear reinforcement in HCS members produced by the extrusion method due to their unique concrete casting methods, and thus, their shear design is significantly affected by the minimum shear reinforcement provision in ACI318-08. In this study, a large number of shear test data on HCS members has been collected and analyzed to examine their web-shear capacity with consideration on the minimum shear reinforcement requirement in ACI318-08. The analysis results indicates that the minimum shear reinforcement requirement for deep HCS members are too severe, and that the web-shear strength equation in ACI318-08 does not provide good estimation of shear strengths for HCS members. Thus, in this paper, a rational web-shear strength equation for HCS members was derived in a simple manner, which provides a consistent margin of safety on shear strength for the HCS members up to 500 mm deep. More shear test data would be required to apply the proposed shear strength equation for the HCS members over 500 mm in depth though.

Keywords: prestressed; hollow-core; slab; web shear; cracking; shear stress distribution

1. Introduction

The prestressed hollow-core slab (HCS) is a factory-manufactured precast member with

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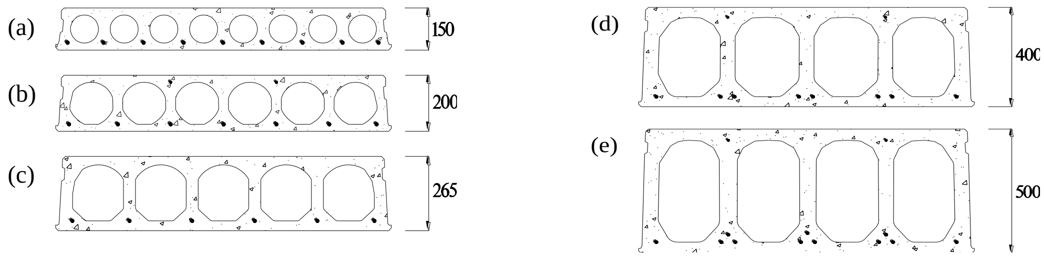


Fig. 1 Various types of section of prestressed hollow-core slab

circular or semicircular hollow-cores in its web section, as shown in Fig. 1. Due to its hollow-cores and replacement of conventional reinforcement by high strength tendons, it is lighter and more cost-effective than conventional reinforced concrete (RC) slabs, and it also has excellent flexural strength and enhanced performance in crack or deflection control by introducing of prestress (Lee and Kim 2011, Kim and Lee 2012(a), (b), Lee *et al*. 2013). Thus, the HCS units have been widely used for lightweight floor structures in Europe and North America, and their demand also increases gradually in other regions (Becker and Buettner 1985, Buettner and Becker 1998, Hawkins and Ghosh 2006). The HCS members are generally manufactured using the extrusion method or the slip-form method. The extrusion method is a dry-cast method that produces slabs by extruding the low-slump concrete on a long-line casting bed using the extruder without any mold, whereas the slip-form method produces the slab units by casting normal- or high-slump concrete onto a slip form. Both methods form the hollow-cores using a tube or auger. Particularly, the extrusion method can reduce facility-related costs because it does not require any steel mold and can reduce the labor costs by automated production process, which makes it better for the mass production of members. For this reason, many precast factories have adopted the extrusion method for the production of HCS units. In the extrusion method, however, it is difficult to provide the shear reinforcement in the HCS unit because it uses very low-slump concrete and extrude it in the longitudinal direction of the member (Buettner and Becker 1998, Hawkins and Ghosh 2006).

The shear failure modes of prestressed concrete (PSC) members without shear reinforcement can be generally categorized into web-shear and flexure-shear failure. The web-shear failure occurs immediately after diagonal cracking when the principle tensile stress in the web reaches its tensile strength, whereas the flexure-shear failure is caused by the diagonal-tension crack changed from flexural cracks (Collins and Mitchell 1991, Nilson 1987, Nawy 2006). Shear strength of HCS members is generally governed by the web-shear cracking capacity because they have multiple thin webs, similar to the prestressed I-beams. A recent study (Hawkins and Ghosh 2006) reported that the web-shear strength equation for PSC members specified in ACI318-05, could overestimate the web-shear strengths of HCS members. Accordingly, the ACI318-08 design code (ACI Committee 318 2008) introduced the minimum shear reinforcement requirement for deep HCS members, in which the HCS members exceeding 315 mm in depth should provide the minimum shear reinforcement unless the half of their factored web-shear cracking strength($0.5\phi V_{cw}$) is greater than the applied shear force(V_u). This means that the web-shear strength of the deep HCS members without the minimum shear reinforcement should be halved. Such a limitation on the deep hollow-core members, however, may provide excessively conservative results, and greatly affects HCS manufacturing industry because it is very difficult to provide the shear reinforcement in the HCS member due to the unique manufacturing process as aforementioned. Thus, a more deliberate investigation and review on the web-shear strength of the HCS members are necessary (Palmer and Schultz 2011). This study carefully reviewed the current web-shear strength equations for the HCS members (or prestressed concrete members) specified in Asian, European, and North American design codes, including the ACI building code, and proposed the web-shear strength equation for the HCS members, which provides a sufficient margin of safety and can help us overcome the issue on the current codes.

2. Review of previous research

The shear capacity of PSC members is generally governed by the web-shear capacity. In general, the following two assumptions are introduced to estimate the web-shear strength of the PSC members in a simple manner. (CEN 2004, ACI Committee 318 2005 and 2008) Firstly, as shown in Fig. 2, the normal stress in the y direction (σ_y) perpendicular to the longitudinal direction of the member is assumed to be negligible. Secondly, the critical section (or the critical point), i.e., the point at which the web-shear failure occurs, is located at the centroid of the section that is ($l_b - l_c$) apart from the face of support in Fig. 2, where l_b and l_c are the distance from the member end to the face of support and to the critical section, respectively. Thus, the normal stress in the longitudinal direction (σ_x) and the shear stress (τ_{xy}) at the critical section can be expressed, respectively, as follows:

$$\sigma_x = \alpha \frac{f_{pe} A_{ps}}{A_c} \tag{1}$$

$$\tau_{xy} = \frac{Q}{b_w I_g} V_y \tag{2}$$

where, α is the coefficient that indicates the actual level of prestress at the critical section, which is typically given by the ratio of the distance between the member end and the critical section (l_c) to the transfer length (l_t). Also, f_{pe} is the effective prestress, A_{ps} is the cross-sectional area of prestressing tendons, and A_c is the corss-sectional area. Q is the first moment of area about centroidal axis, I_g is the moment of inertia of the gross section, b_w is the sum of the web widths, and V_y is the shear force at the critical section. Based on the theory of elasticity (Ugural and Fenster 2003), the principal tensile stress at the critical section (σ_1) can be calculated as follows:

$$\sigma_1 = \frac{\sigma_x}{2} + \sqrt{\frac{{\sigma_x}^2}{4} + {\tau_{xy}}^2} \tag{3}$$

where, σ_x is the normal stress in longitudinal direction of the member. The web-shear cracking occurs when the principal tensile stress (σ_1) in the web reaches the cracking strength of concrete. Although the web shear cracking strength may be slightly different from the web-shear strength, their difference is generally negligible, and it is generally considered as the web-shear strength of a PSC member. Thus, this study assumes that they are identical. Accordingly, the web-shear strength (V_{cw}) can be expressed by substituting Eqs. (1) and (2) to Eq. (3), as follows:

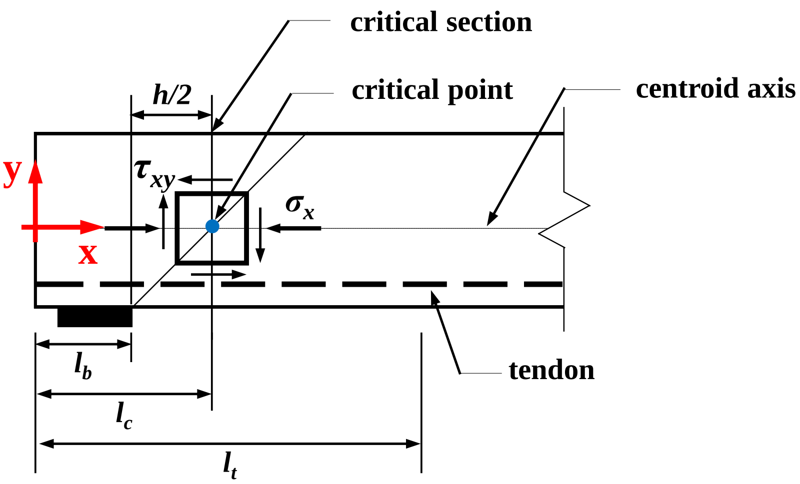
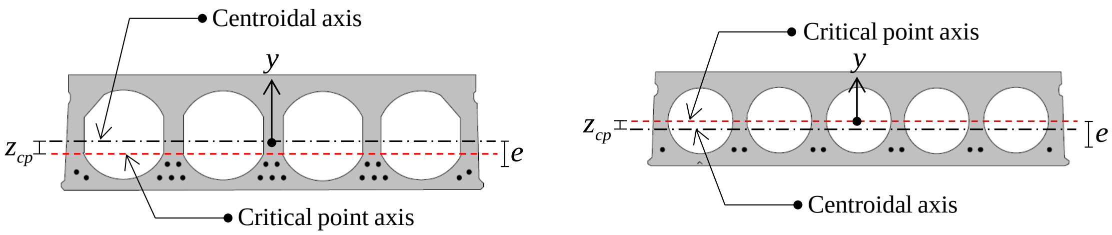


Fig. 2 Description of transfer length, bearing length, critical section and point



(a) Non-circular section (b) Circular section

Fig. 3 Critical section of HCS members (Yang 1994)

$$V_{cw} = V_y = \frac{I_g b_w}{S}\sqrt{f_{ct}^{\ 2} + \alpha f_{pe} f_{ct} \frac{A_{ps}}{A_c}} \tag{4}$$

The code equations in Europe (Eurocode2 2004, British Code of Practice 1972, BS8110 1997, FIP recommendation 1988) use Eq. (4) as a basic form of the web-shear strength for PSC members without shear reinforcement, which is governed by the concrete tensile strength (f_{ct}) and α factor that is determined by the ratio of the distance between the member end and the critical section (l_c / l_t) to the transfer length (l_t). Based on Eq. (4), Eurocode2 (2004) presents the web-shear strength ($V_{Rd,c}$) of PSC members, as follows:

$$V_{Rd,c} = \frac{I \cdot b_w}{S}\sqrt{f_{ctd}^{\ 2} + \alpha_l \sigma_{cp} f_{ctd}} \tag{5}$$

where, f_{ctd} is $\alpha_{ct} 0.7 f_{ctm} / \gamma_c$, in which α_{ct} is a coefficient taking account of long term effect on the tensile strength and of unfavorable effects, resulting from the way the load is applied, and the recommended value, 1.0, is used in this study. γ_c is the partial safety factor for concrete that is 1.5, f_{ctm} is the mean value of the concrete cylinder compressive strength that is $0.3(f_c')^{2/3}$ for $f_c' < 50\,\text{MPa}$ and $2.12\ln\left[1+\left(f_c'+8\right)/10\right]$ for $f_c' \geq 50\,\text{MPa}$; α_l is l_x / l_{pt2} that should not exceed 1.0, where l_x is the distance of the critical section from the starting point of the transfer length, and l_{pt2}, the upper-bound value of the transfer length, is $\alpha_1\alpha_2 d_b f_{pi} / \eta_{p1}\eta_1 f_{ctd}$, wherein α_1 and α_2 are 1.0 and 0.19, respectively, η_{p1} and η_1 are 3.2 and 1.0, respectively, d_b is the diameter of the tendon, f_{pi} is the tendon stress right after the release; σ_{cp} is the concrete compressive stress at the centroidal axis due to the axial load or prestressing forces.

The British Code of Practice (1972) presents the web-shear strength, considering the sectional properties of the web concrete only instead of those of the gross section properties, as follows:

$$V_{cw} = \frac{I_w b_w}{S_w}\sqrt{f_{ct}^{\,2} + \alpha\sigma_{cp} f_{ct}} \tag{6}$$

where, f_{ct} is $0.3\sqrt{f_{cu}}$, f_{cu} is the compressive strength of the cubic concrete specimen, Q_w and I_w are first moment of area and moment of inertia of web section about centroidal axis, respectively, and α is 0.8.

Walraven and Mercx (1983) applied the overall reduction factor of 0.75 into Eq. (5) to consider 5% lower bound for shear-tension capacity, and proposed an equation for web-shear strength (V_{cw}) of HCS members, as follow

$$V_{cw} = 0.75\frac{Ib_w}{S}\sqrt{f_{ct}^{\,2} + \alpha\sigma_{cp} f_{ct}} \tag{8}$$

where, α is

$$\alpha = 1 - \left[\frac{(l_t - l_b)}{l_t}\right]^2 \tag{9}$$

where, f_{ct} is $0.05 f_{cu} + 1$, l_t is the transfer length, and l_b is the bearing length. The FIP recommendation (1998) also adopted the Equation (5) to estimate the web-shear strength of PSC members (V_{Rd12}) with an additional reduction factor of 0.9, as follows

$$V_{Rd12} = \frac{Ib_w}{S}\sqrt{f_{ctd}^{\,2} + 0.9\alpha\sigma_{cp} f_{ctd}} \tag{10}$$

where, f_{ctd} is $0.3(f_c')^{2/3}$ as presented in the CEB-FIP Model Code (1990), and α is determined using Eq. (9).

In the case of the HCS members with non-circular hollow-core section, the web-shear cracking often occurs at the bottom of the web where sectional area is changed rapidly, which means that the critical point is located below the centroidal axis as shown in Fig. 3. For this reason, Yang (1994) considered the eccentric moment induced by tendons in calculating the normal stress in the longitudinal direction, and proposed the web-shear strength (V_{cw}) of HCS members by assuming that the critical point is located at the bottom web where the 35 degree angle line from the center of the support intersects, as follows

$$V_{cw} = \frac{bI_y}{S_{cp}} \left\{ \begin{array}{l} \dfrac{b}{2S_{cp}} f_{ct} x_{cp} y_{cp} + \dfrac{S_{cp}}{b}\left(\dfrac{e}{I_z} - \dfrac{A_{cp}}{AS_{cp}}\right)\dfrac{dN_p}{dx} \\ + \left[\left(\dfrac{b}{2S_{cp}} f_{ct} x_{cp} y_{cp} \right)^2 + \left(\dfrac{e}{I_y} - \dfrac{A_{cp}}{AS_{cp}} \right) f_{ct} x_{cp} y_{cp} + \left(\dfrac{1}{A} - \dfrac{y_{cp} e}{I_y} \right) N_p f_{ct} + f_{ct}^{\,2} \right]^{1/2} \end{array} \right\} \tag{11}$$

where, A_{cp} is the sectional area above the critical point, S_{cp} is the first moment of area of the section above the critical point, x_{cp} is the horizontal distance between the critical point and the center of the support, z_{cp} is the vertical distance between the critical point and the centroidal axis, e is the distance from the centroid of the prestressing tendons to the centroid of the cross section, and N_p is the effective prestressing force in the tendons at the critical point. Pajari (2009) also proposed a web-shear strength equation similar to that of Yang (1994), considering the case in which tendons are provided in the compression area of the section.

The Japanese design standard, JSCE (2007), provides the web-shear strength equation for PSC members without shear reinforcement (V_{wcd}) in a very simple form, as follows

$$V_{wcd} = f_{wcd} b_w d / \gamma_b \tag{12}$$

where, f_{wcd} is $1.25\sqrt{f_c'}$, f_c' is the compressive strength of concrete, γ_b is the member factor that is generally taken to the value of 1.3, and d is the effective member depth.

The North American concrete design standards, such as ACI 318-08 (2008) and AASHTO-LRFD bridge design specification (2007), use the average shear stress (τ_{xy}), simplifying the parabolic shear stress distribution expressed in Eq. (2), as follows

$$\tau_{xy} = \frac{V_y}{b_w d_p} \tag{13}$$

and the web-shear strength (V_{cw}) of PSC member was linearized in ACI 318-08 (2008) and AASHTO-LRFD bridge design specification (2007), respectively, as follows

$$V_{cw} = \left(0.29\sqrt{f_c'} + 0.3 f_{pc}\right) b_w d_p + V_p \tag{14}$$

$$V_{cw} = \left(0.16\sqrt{f_c'} + 0.3 f_{pc}\right) b_w d_v + V_p \tag{15}$$

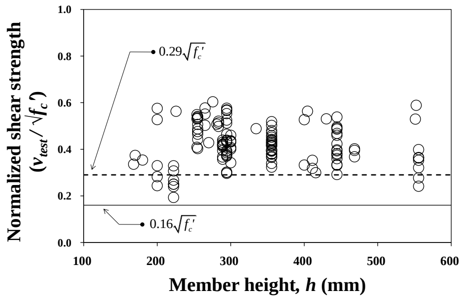
where, d_p is the distance from the extreme compression fiber to the centroid of the prestressing steel, d_v is the effective shear depth, f_c' is the compressive strength of concrete, and V_p is the vertical component of prestressing force. Also, the transfer length is estimated by $50d_b$ in ACI 318-08 (2008) and $60d_b$ in AASHTO-LRFD (2007), respectively, and the effective prestress (f_{pe}) at the section within the transfer length should be reduced properly in calculation of web-shear capacity. The vertical component of the prestressing force (V_p) is typically zero in HCS units because tendons are placed straightly.

3. Evaluation and modification of code equation

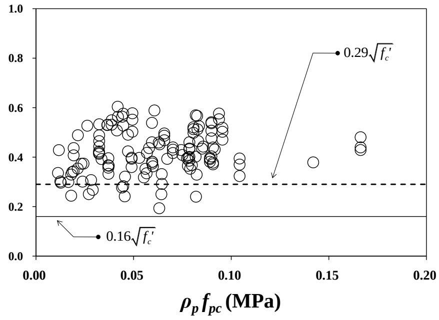
Table 1 shows the database collected from shear test results of HCS members reported in previous studies (Walraven and Mercx 1983, Becker and Buettner 1985, Pajari 2005, TNO 2005, Bertagnoli and Mancini 2009, Celal 2011, Rahman *et al*. 2010). The database consists of 145 shear test results on HCS members, including slender and relatively deep prestressed hollow-core members ranging from 190 to 558 mm in depth with circular or non-circular sections. Their shear span-to-depth ratio (a/d) ranges from 1.66 to 6.73, the compressive strength of the concrete ranges from 40 MPa to 114 MPa, and the prestressing tendons ratio (ρ_p) ranges from 0.26% to 2.12%. All HCS members in this database failed in shear near the support region, and web-shear failure was their dominant failure mode.

3.1 Effect of key parameters

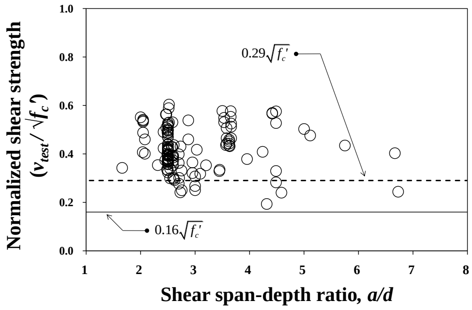
Fig. 4 shows the normalized average shear strength of the HCS members ($v_{test}/\sqrt{f_c'}$) obtained from experimental test data according to the key influential parameters. Fig. 4(a) shows the effect of the member depth (h) on the normalized average shear strength ($v_{test}/\sqrt{f_c'}$), which gives an almost uniform trend of $v_{test}/\sqrt{f_c'}$ along the section height. This indicates that the size effect is at least minimal or negligible in the HCS members under 600 mm in depth, and such a result was also reported by Palmer and Schultz (2010). Due to the scant data on the HCS members thicker than 500 mm, however, additional experimental efforts are still required. Figs. 4(b) to Fig. 4(d) show the effects of the prestress ($\rho_p f_{pc}$), shear span-to-depth ratio (a/d), and concrete compressive strength (f_c'), on the normalized shear strength ($v_{test}/\sqrt{f_c'}$). While the values of $v_{test}/\sqrt{f_c'}$ against a/d did not show any bias along the shear span-to-depth ratio, it tended to increase slightly as the prestress ($\rho_p f_{pc}$) or the concrete compressive strength (f_c') increased. There are, however, only few test results on HCS members with high-strength concrete over 90 MPa in compressive strength available in literature, which indicates that additional tests are necessary on these HCS members.



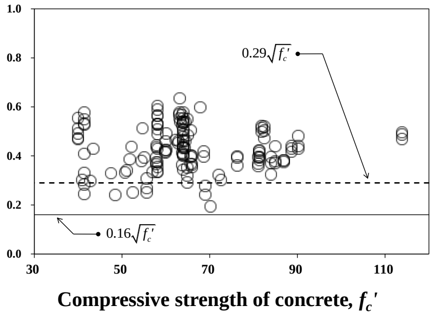
(a) Effect of member height



(b) Effect of prestress

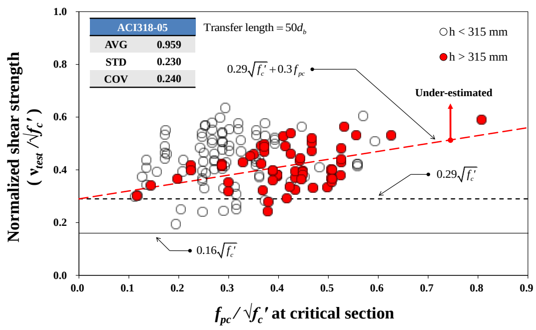


(c) Effect of shear span-depth ratio

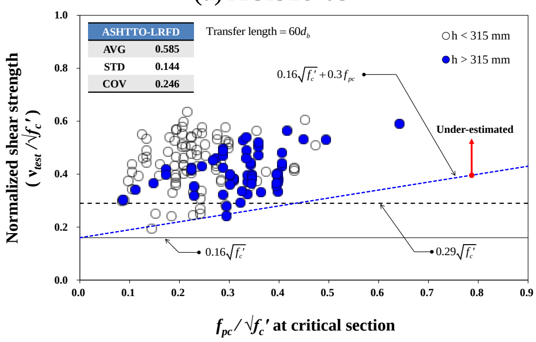


(d) Effect of concrete compressive strength

Fig. 4 Parametric study on shear strengths of HCS specimens

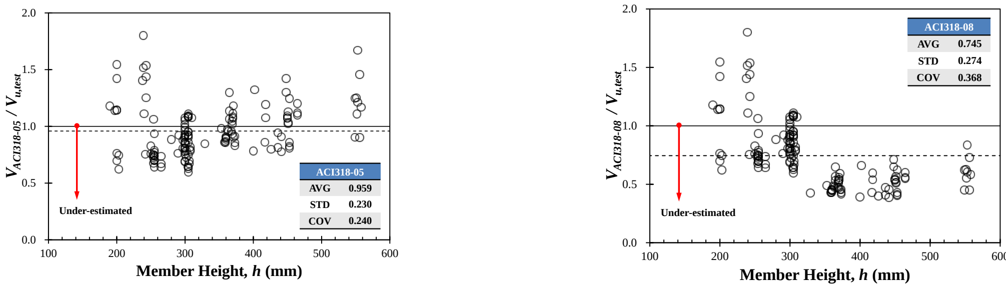


(a) ACI318-05



(b) AASHTO-LRFD simplified model

Fig. 5 Evaluation of U.S. code equations on shear strength of HCS members



(a) ACI318-05 (2005)

(b) ACI318-08 (2008)

Fig. 6 Evaluation of web-shear cracking strengths of HCS members as specified in ACI318

Table 1 Details of collected shear test data

Specimens name	Void shape	Dimensions					Materials		Prestress and Loading geometry			Test result
		dp (mm)	bw (mm)	Ag (mm2)	Aps (mm2)	I·bw/S (mm3)	fc' (MPa)	fpu (MPa)	fpc (MPa)	a/d (-)	L (mm)	Vtest (kN)
15a†	NC	225.0	294.0	171000	564.0	59601	64.0	1860.0	3.79	3.58	6000	234.2
15b†	NC	225.0	294.0	171000	564.0	59601	64.0	1860.0	3.79	2.04	6000	258.3
16a†	NC	225.0	294.0	171000	564.0	59601	64.0	1860.0	3.79	3.58	6000	245.9
16b†	NC	225.0	294.0	171000	564.0	59601	64.0	1860.0	3.79	2.04	6000	282.0
4a†	NC	225.0	294.0	171000	940.0	60490	64.0	1860.0	6.32	2.04	6000	284.3
4b†	NC	225.0	294.0	171000	940.0	60490	64.0	1860.0	6.32	3.58	6000	268.3
5a†	NC	225.0	294.0	171000	940.0	60490	64.0	1860.0	6.32	2.04	6000	286.3
5b†	NC	225.0	294.0	171000	940.0	60490	64.0	1860.0	6.32	5.11	6000	252.1
6†	NC	225.0	294.0	171000	940.0	60490	64.0	1860.0	6.32	6.67	6000	213.3
7a†	NC	265.0	250.0	199000	470.0	56070	64.0	1860.0	2.72	2.04	6000	216.0
7b†	NC	265.0	250.0	199000	470.0	56070	64.0	1860.0	2.72	3.57	6000	231.5
8b†	NC	265.0	250.0	199000	470.0	56070	64.0	1860.0	2.72	1.66	6000	181.6
10a†	NC	260.0	250.0	199000	940.0	57356	64.0	1860.0	5.43	2.08	6000	208.5
11a†	NC	260.0	250.0	199000	940.0	57356	64.0	1860.0	5.43	3.63	6000	224.6
11b†	NC	260.0	250.0	199000	940.0	57356	64.0	1860.0	5.43	2.08	6000	239.3

12†	NC	260.0	250.0	199000	940.0	57356	64.0	1860.0	5.43	5.75	6000	226.2
18†	C	230.0	260.0	178000	676.0	52611	64.0	1860.0	4.37	5.00	6000	240.6
19a†	C	230.0	260.0	178000	676.0	52611	64.0	1860.0	4.37	3.50	6000	276.3
19b†	C	230.0	260.0	178000	676.0	52611	64.0	1860.0	4.37	2.00	6000	263.9
8512††	NC	170.0	431.8	140645	449.0	43658	41.4	1722.5	2.94	4.48	4575	155.8
8512††	NC	170.0	431.8	140645	449.0	43658	41.4	1722.5	2.94	6.73	4575	115.3
8614††	NC	170.0	431.8	140645	722.5	43658	41.4	1722.5	4.54	4.48	4575	271.9
8614††	NC	170.0	431.8	140645	722.5	43658	41.4	1722.5	4.54	4.48	4575	249.2
8614††	NC	170.0	431.8	140645	722.5	43658	41.4	1722.5	4.54	4.48	4575	133.5
10614††	NC	215.9	337.8	165806	722.5	44528	41.4	1722.5	3.91	3.53	4575	249.2
10614††	NC	215.9	337.8	165806	722.5	44528	41.4	1722.5	3.91	3.53	4575	257.7
10620††	NC	215.9	337.8	165806	1032.2	44528	41.4	1722.5	5.30	4.24	10363.2	191.8
31.2†††	C	200.0	239.0	119000	651.0	36425	48.5	-	6.02	4.59	6643	80.0
33.2†††	C	200.0	238.0	119000	651.0	36333	47.5	-	6.02	3.45	4998	108.0
40.2†††	C	200.0	293.0	126000	651.0	43378	70.2	-	5.68	4.32	6257	95.0
63.2†††	C	200.0	262.0	126000	651.0	39492	52.5	-	5.17	2.76	4006	95.0
74.265†††	C	265.0	219.0	172000	465.0	43742	72.6	-	2.97	2.71	5199	149.0
98.265†††	C	265.0	228.0	172000	930.0	45782	64.4	-	5.95	2.73	5253	209.0
104.265†††	C	265.0	244.0	172000	372.0	48930	41.0	-	2.16	2.60	5004	125.0
107.265†††	C	265.0	239.0	172000	372.0	47927	42.8	-	2.16	2.60	5007	123.0
109.265†††	C	265.0	242.0	172000	930.0	48271	51.8	-	5.41	2.60	4997	178.0
110.265†††	C	265.0	220.0	164000	744.0	44242	52.2	-	4.54	2.61	5015	184.0
113.265†††	C	265.0	226.0	163000	481.7	45575	57.8	-	2.89	2.60	4997	170.0
114.265†††	C	265.0	226.0	163000	853.7	45575	57.8	-	4.71	2.60	4997	179.0

115.265†††	C	265.0	215.0	163000	558.0	43356	55.1	-	3.42	2.58	4963	166.0
501.265†††	C	265.0	224.0	171000	930.0	44920	63.2	-	5.98	3.65	5501	272.0
502.265†††	C	265.0	224.0	171000	930.0	44920	63.2	-	5.98	3.65	5501	261.0
505.265†††	C	265.0	216.0	172000	930.0	43372	63.7	-	5.14	3.66	5998	240.0
507.265†††	C	265.0	218.0	172000	930.0	43542	54.7	-	5.14	3.66	5995	219.0
509.265†††	C	265.0	217.0	172000	930.0	43400	62.3	-	5.14	3.66	5996	211.0
511.265†††	C	265.0	221.0	170000	930.0	44437	63.7	-	5.20	4.42	5999	265.0
512.265†††	C	265.0	223.0	170000	930.0	44839	63.0	-	5.20	4.42	6001	267.0
146.32†††	C	320.0	256.0	205000	558.0	61867	51.1	-	2.72	2.54	5890	199.0
148.32†††	C	320.0	263.0	203000	1132.7	63661	43.5	-	5.53	2.58	5985	238.0
151.32†††	C	320.0	270.0	207000	837.0	65394	57.8	-	3.84	2.59	6003	240.0
133.32†††	C	320.0	261.0	237000	1023.0	60757	68.7	-	4.32	2.53	6995	275.0
134.32†††	C	320.0	243.0	237000	1023.0	56966	68.7	-	4.32	3.03	6990	269.0
513.32†††	I	320.0	258.0	181000	1407.0	62490	54.5	-	8.33	3.95	9594	231.0
514.32†††	I	320.0	309.0	190000	1209.0	74100	57.9	-	7.00	3.63	7198	333.0
515.32†††	I	320.0	311.0	190000	1209.0	74580	57.9	-	7.00	3.63	7200	329.0
516.32†††	I	320.0	289.0	180000	1023.0	69938	60.0	-	5.68	3.63	5800	329.0
517.32†††	I	320.0	287.0	180000	1023.0	69524	62.8	-	5.68	3.63	5800	329.0
160.37†††	I	370.0	272.0	243000	1116.0	74395	64.9	-	5.05	2.59	7002	286.0
161.37†††	I	370.0	276.0	243000	1116.0	75489	64.9	-	5.05	3.09	6993	262.0
162.4†††	I	400.0	284.0	213000	1116.0	86375	57.0	-	5.76	3.45	10017	287.0
178.4†††	I	400.0	286.0	218000	1209.0	86388	64.9	-	6.10	2.62	7626	269.0
188.4†††	I	400.0	285.0	219000	1209.0	86463	50.7	-	6.07	2.75	7626	269.0
518.4†††	I	400.0	293.0	210000	1209.0	88720	64.5	-	5.76	2.88	8390	433.0

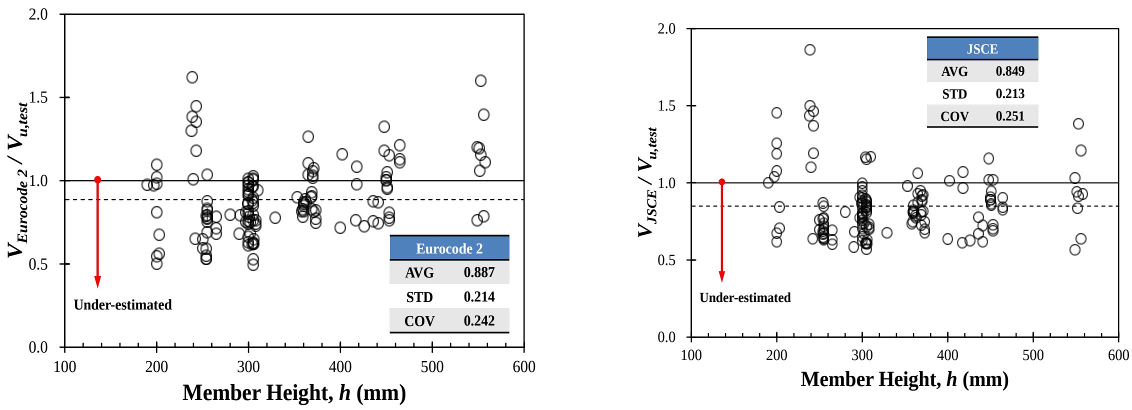
519.4†††	I	400.0	293.0	210000	1209.0	88720	64.5	-	5.76	2.88	8390	507.0
520.4†††	I	400.0	291.0	210000	1023.0	88317	60.1	-	5.36	2.42	5504	443.0
521.4†††	I	400.0	291.0	210000	1023.0	88317	60.1	-	5.36	2.42	5504	382.0
191.5†††	I	500.0	325.0	300000	1488.0	122076	69.0	-	4.96	2.73	10995	326.0
193.5†††	I	500.0	335.0	300000	1488.0	125833	69.0	-	4.96	2.70	8470	386.0
194.5†††	I	500.0	312.0	300000	1674.0	117193	63.7	-	5.58	2.70	8499	452.0
198.5†††	I	500.0	324.0	300000	1488.0	121701	72.1	-	4.96	2.95	8492	442.0
199.5†††	I	500.0	326.0	300000	1953.0	122452	66.0	-	6.51	2.70	8516	528.0
200.5†††	I	500.0	327.0	300000	1953.0	122828	66.0	-	6.51	2.95	8510	485.0
201.5†††	I	500.0	322.0	300000	1953.0	120950	66.0	-	6.51	3.20	8512	462.0
260-1‡	I	255.0	347.0	174000	976.5	68713	81.9	-	7.31	2.50	4000	399.0
260-2‡	I	255.0	347.0	174000	976.5	68713	81.9	-	7.31	2.50	4000	417.0
260-3‡	I	255.0	347.0	174000	976.5	68713	81.9	-	7.31	2.50	4000	411.0
320-1‡	I	320.0	317.0	203000	1228.9	76202	82.5	-	7.88	2.50	4800	434.0
320-2‡	I	320.0	317.0	203000	1228.9	76202	82.5	-	7.88	2.50	4800	463.0
320-3‡	I	320.0	317.0	203000	1228.9	76202	82.5	-	7.88	2.50	4800	478.0
400-1‡	I	400.0	308.0	243000	1228.9	93828	113.9	-	6.58	2.50	6000	652.0
400-2‡	I	400.0	308.0	243000	1228.9	93828	113.9	-	6.58	2.50	6000	616.0
400-3‡	I	400.0	308.0	243000	1228.9	93828	113.9	-	6.58	2.50	6000	640.0
260-1W‡	C	265.0	247.0	172000	853.7	49073	85.0	-	6.95	2.50	4000	228.0
260-2W‡	C	265.0	247.0	172000	853.7	49073	85.0	-	6.95	2.50	4000	224.0
260-3W‡	C	265.0	247.0	172000	853.7	49073	85.0	-	6.95	2.50	4000	265.0
320-1W‡	C	320.0	241.0	206000	1373.5	58019	90.2	-	9.33	2.50	4800	352.0
320-2W‡	C	320.0	241.0	206000	1373.5	58019	90.2	-	9.33	2.50	4800	314.0

320-3W‡	C	320.0	241.0	206000	1373.5	58019	90.2	-	9.33	2.50	4800	322.0
260-1EBM‡	I	260.0	449.0	186000	930.0	89561	81.1	-	4.65	2.50	4000	416.0
260-2EBM‡	I	260.0	449.0	186000	930.0	89561	81.1	-	4.65	2.50	4000	386.0
260-3EBM‡	I	260.0	449.0	186000	930.0	89561	81.1	-	4.65	2.50	4000	376.0
320-1EBM‡	I	320.0	378.0	206000	930.0	90865	59.9	-	4.20	2.50	4800	396.0
320-2EBM‡	I	320.0	378.0	206000	930.0	90865	59.9	-	4.20	2.50	4800	387.0
320-3EBM‡	I	320.0	378.0	206000	930.0	90865	59.9	-	4.20	2.50	4800	391.0
260-1B‡	I	260.0	394.0	181000	1039.7	79011	88.7	-	6.91	2.50	4000	415.0
260-2B‡	I	260.0	394.0	181000	1039.7	79011	88.7	-	6.91	2.50	4000	402.0
260-3B‡	I	260.0	394.0	181000	1039.7	79011	88.7	-	6.91	2.50	4000	424.0
320-1B‡	I	320.0	325.0	184000	1225.7	79592	81.4	-	7.56	2.50	4800	372.0
320-2B‡	I	320.0	325.0	184000	1225.7	79592	81.4	-	7.56	2.50	4800	368.0
320-3B‡	I	320.0	325.0	184000	1225.7	79592	81.4	-	7.56	2.50	4800	358.0
400-1B‡	I	400.0	318.0	222000	1225.7	94768	86.9	-	6.18	2.50	6000	444.0
400-2B‡	I	400.0	318.0	222000	1225.7	94768	86.9	-	6.18	2.50	6000	452.0
400-3B‡	I	400.0	318.0	222000	1225.7	94768	86.9	-	6.18	2.50	6000	452.0
260-1V‡	I	260.0	305.0	178000	1684.4	58095	81.3	-	10.41	2.50	4000	302.0
260-2V‡	I	260.0	305.0	178000	1684.4	58095	81.3	-	10.41	2.50	4000	300.0
260-3V‡	I	260.0	305.0	178000	1684.4	58095	81.3	-	10.41	2.50	4000	295.0
320-1V‡	I	320.0	318.0	209000	1419.8	78000	84.0	-	7.47	2.50	4800	345.0
320-1V‡	I	320.0	318.0	209000	1419.8	78000	84.0	-	7.47	2.50	4800	368.0
320-1V‡	I	320.0	318.0	209000	1419.8	78000	84.0	-	7.47	2.50	4800	302.0

400-1V‡	I	400.0	387.0	261000	1341.3	114927	76.3	-	5.65	2.50	6000	487.0
400-2V‡	I	400.0	387.0	261000	1341.3	114927	76.3	-	5.65	2.50	6000	532.0
400-3V‡	I	400.0	387.0	261000	1341.3	114927	76.3	-	5.65	2.50	6000	538.0
3‡‡	I	200.0	413.0	141000	594.9	61809	55.7	-	4.03	3.00	4100	165.0
4‡‡	I	200.0	444.0	141000	594.9	66448	55.7	-	4.03	3.00	4100	166.0
5‡‡	I	200.0	425.0	141000	594.9	63605	55.7	-	4.03	3.00	4100	195.0
N-42-01‡‡	I	421.5	354.0	238000	1435.0	130090	65.7	-	8.04	2.49	6400	478.0
N-42-02‡‡	I	421.5	343.0	238000	1435.0	126047	65.7	-	8.04	2.49	6400	431.0
N-42-03‡‡	I	421.5	335.0	238000	1435.0	123107	65.7	-	8.04	2.49	6400	460.0
H150‡‡	C	162.0	414.0	134000	375.5	48300	58.1	-	3.82	2.31	4200	181.0
H150‡‡	C	153.0	406.0	118000	375.5	45402	58.1	-	4.02	2.45	4200	177.0
H150‡‡	C	151.0	406.0	116000	249.1	44961	58.1	-	2.77	2.48	4200	157.0
H200‡‡	C	203.0	296.0	134000	498.4	45445	58.1	-	5.06	2.46	4200	258.0
H250‡‡	C	253.0	247.0	157000	548.4	47673	58.1	-	4.72	2.47	4200	242.0
H250‡‡	C	243.0	223.0	142000	265.8	41425	58.1	-	2.38	2.57	4200	177.0
H250‡‡	C	248.0	240.0	148000	548.4	45490	58.1	-	4.54	2.52	4200	274.0
H300‡‡	C	301.0	215.0	170000	438.7	50117	58.1	-	3.16	2.49	4700	241.0
H360‡‡	I	360.0	264.0	212000	744.0	72964	58.1	-	4.74	2.50	5600	241.0
H360‡‡	I	360.0	244.0	186000	591.4	68152	58.1	-	3.90	2.50	5600	353.0
H360‡‡	I	364.0	234.0	187000	760.7	66036	58.1	-	4.94	2.47	5600	366.0
H400‡‡	I	387.0	320.0	225000	870.4	96640	58.1	-	5.21	2.58	6200	501.0
H500‡‡	I	496.0	320.0	293000	1123.1	122826	58.1	-	5.16	2.52	7000	641.0
H500‡‡	I	497.0	320.0	293000	1440.2	122826	58.1	-	6.69	2.52	7000	714.0
300-P1-A*	C	255.0	219.0	180221	888.3	40412	67.9	1860.0	6.88	2.99	-	275.0

300-P1-B*	C	255.0	219.0	180221	888.3	40412	65.7	1860.0	6.88	2.99	-	228.0
300-P2-A*	C	255.0	231.0	180414	888.3	42359	63.2	1860.0	6.87	2.99	-	297.0
300-P2-B*	C	255.0	231.0	180414	888.3	42359	63.8	1860.0	6.87	2.99	-	194.0
250-P2-A*	C	207.0	239.0	154047	652.8	34246	63.3	1860.0	6.56	3.07	-	212.0
250-P2-B*	C	207.0	239.0	154047	652.8	34246	65.1	1860.0	6.56	3.07	-	193.0
200-P2-A*	C	158.0	283.0	132887	502.5	35031	64.9	1860.0	5.41	3.80	-	198.0
200-P2-B*	C	158.0	283.0	132887	502.5	35031	62.9	1860.0	5.41	3.80	-	163.0
S11**	C	212.0	315.7	151812	592.2	47304	40.0	1960.0	4.74	2.83	-	216.5
S12**	C	212.0	315.7	151812	592.2	47304	40.0	1960.0	4.75	2.83	-	198.5
S13**	C	262.0	325.4	179115	789.6	60307	40.0	1960.0	4.03	2.29	-	264.5
S14**	C	262.0	325.4	179115	789.6	70400	40.0	1960.0	4.04	2.29	-	298.9
S15**	C	262.0	325.4	179115	789.6	70400	40.0	1960.0	4.04	2.29	-	254.0
Total 145 Specimens												

Cf. void type : NC (noncircular), C (circular), I (I shape)
f_{pu} : specified tensile strength of prestressing steel
f_{pc} : compressive stress in concrete at the centroid of the section due to effective prestres
Ref. †: Walraven and Mercx (1983), ††: Becker and Buettner (1985), †††: Pajari (2005), ‡: TNO building and Constructions Research (2005), ‡‡: Bertagnoli and Masini (2009), * : Celal (2011), ** : Rahman *et al*. (2010),



(a) Eurocode 2 (2004) (b) JSCE (2007)

Fig. 7 Evaluation of shear strength estimated by European and Japanes codes

3.2 Evaluations of code equations

As aforementioned, according to the recently revised provisions on the minimum shear reinforcement in ACI318-08 building code, the web-shear strength of HCS members without the minimum shear reinforcement exceeding 315 mm in depth should be reduced in half. This revision was based on the study by Hawkins and Ghosh (2006), in which they investigated the shear strengths of 28 deep prestressed hollow-core slabs manufactured by three US precast concrete manufacturers that ranged from 300 mm to 400 mm. They reported that the web-shear capacity of HCS members thicker than 300 mm could be overestimated by web-shear strength equation specified in ACI318-05, i.e., Eq. (14) in this paper. To examine it in more detail, Fig. 5 shows the comparison of the web-shear strength estimated by ACI318-05 equation and AASHTO-LRFD simplified method, i.e., Eq. (15) in this paper, with the test results collected from literature. The larger the ratio of the prestress to the concrete tensile strength ($f_{pc}/\sqrt{f_c'}$) was, the greater the HCS member shear strength ($v_{test}/\sqrt{f_c'}$) was, which both equations captured well. Both equations provided relatively accurate results with a similar COV of about 0.24. The ACI318-05 equation, however, overestimated the shear capacities of a large number of specimens, whereas ASHTTO-LRFD provided safe results for all the specimens. This is because, while the two equations equally consider the effect of prestress, the shear strength contributions of concrete are very different, and their transfer lengths are also differed by about 20%.

Fig. 6(a) shows the ratio of the web-shear strengths estimated by ACI318-05 and the test results with respect to the member depths, and Fig. 6(b) also shows the same but for ACI318-08, in which the web-shear strength of the HCS members thicker than 315 mm were reduced in half according to the revised minimum shear reinforcement requirement. It can be seen that the web strength ratios by ACI318-05 are indeed unconservative for deep HCS members, which is the reason for the revision on the minimum shear reinforcement in ACI318-08. Fig. 6(b) shows that, however, ACI318-08 are excessively conservative for deep HCS members. On the other hand, ACI318-08 still provides unconservative web-shear capacities for many slender HCS members, and its accuracy is also lower than ACI318-05. Therefore, rather than reducing the shear strength of the deep PHC members without shear reinforcement as in the current ACI code, it is reasonable to revise the code equation for the safe estimation of web-shear capacity, maintaining the accuracy of web-shear strength at least similar to that of ACI318-05 throughout the whole range of member depths.

Fig. 7 compares the web-shear strengths estimated by Eurocode2 (2004) and JSCE (2007) with the test results. Both Eurocode2 (2004) and JSCE (2007) showed similar COVs to that of ACI318-05. There are, however, a considerable number of test results that are unsafe in the web-shear strength when estimated by those two codes.

3.3 Modification of web shear strength based on parabolic shear stress distribution

The average shear stress concept adopted in North American design standards is based on the tooth model proposed by Kani (1964), as shown in Fig. 8 (MacGregor 2005). The force equilibrium between adjacent flexural cracks in the concrete member shown in Fig. 8(a) can be expressed as Fig. 8(c), and from the free body diagram cutting on the upper surface of the bottom tension reinforcement shown in Fig. 8(b), the tensile force increase (ΔT_s) in the reinforcement between the adjacent flexural cracks should be equilibrated to the sum of the shear flow developed on the upper surface of the element between the cracks. Therefore, as shown in Fig. 8(d), the magnitude of shear stress on the section along the height of the crack is the same and is the average shear stress. The HCS members without shear reinforcement, however, is generally failed in shear simultaneously with the diagonal cracking in the web, and only few discrete flexural cracks may be developed when web-shear cracking occurs. (Im *et al*. 2012) Therefore, the average shear stress analogy may not be applicable to estimate the web-shear strength of the HCS members. Rather, it is more reasonable to consider it as the elastic state up to the diagonal cracking in the web. Thus, the parabolic distribution of shear stresses, as adopted in Eurocode2, is more suitable for the HCS members with respect to the web-shear failure. Fig. 9 shows the ratio of the average to the parabolic shear stress, that is, $I_g/\left(Qd_p\right)$, at the sectional centroid of the specimens listed in Table 1. The ratios of $I_g/\left(Qd_p\right)$ tend to be consistent with an average value of about 0.76. The web-shear strength equation in ACI318-05 that is based on the average shear stress, therefore, should be reduced as much as the relative ratios of $I_g/\left(Qd_p\right)$. Then, the web-shear strength (V_{cw}) can be expressed, as follows:

$$V_{cw} = \left(0.29\lambda\sqrt{f_c^{'}} + 0.3 f_{pc}\right)\frac{b_w I}{Q} + V_p \tag{15}$$

Fig. 10(a) shows the comparison of the calculated web-shear strengths ($V_{proposed}$) and the test results ($V_{u,test}$). The web-shear strength ratios by Eq. (15) ($V_{proposed}/V_{u,test}$) show an enhanced accuracy with a COV of 0.23, compared to those from the existing code equations, and provided proper margin of safety for most of the specimens. Considering that there were only few HCS specimens that were more than 500 mm deep and that their safety margins were somewhat low, however, the web-shear strengths of such deep members are better to be reduced as stipulated in

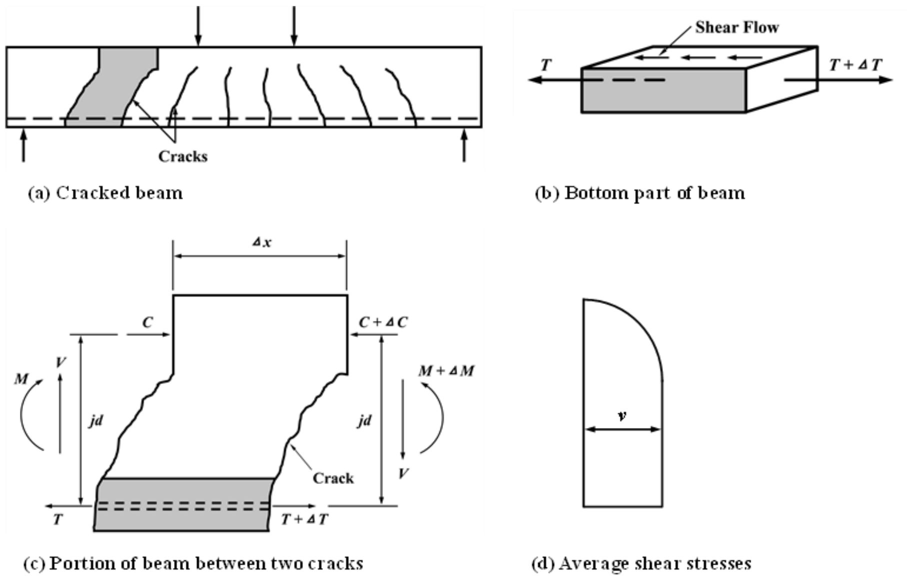


Fig. 8 Concept of average shear stress assumption (MacGregor 2005, Kani 1964)

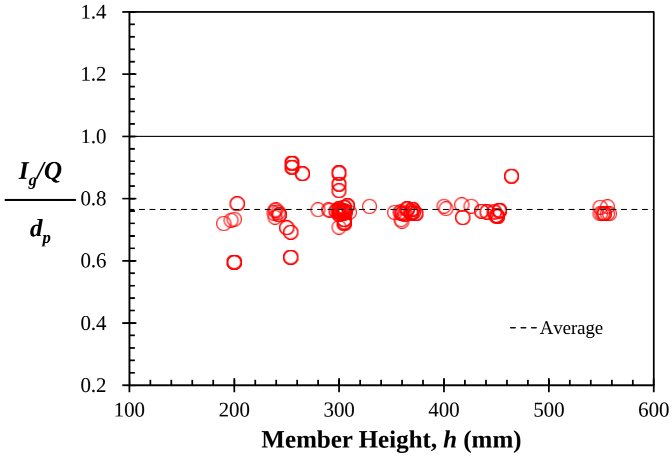
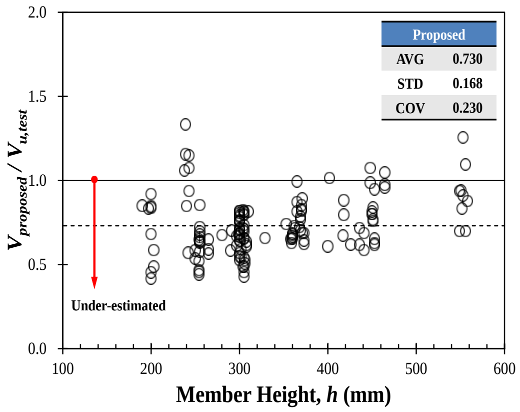
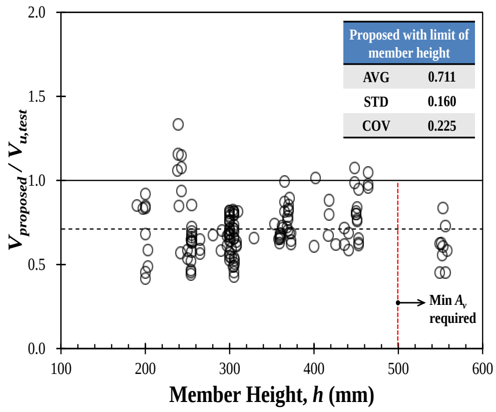


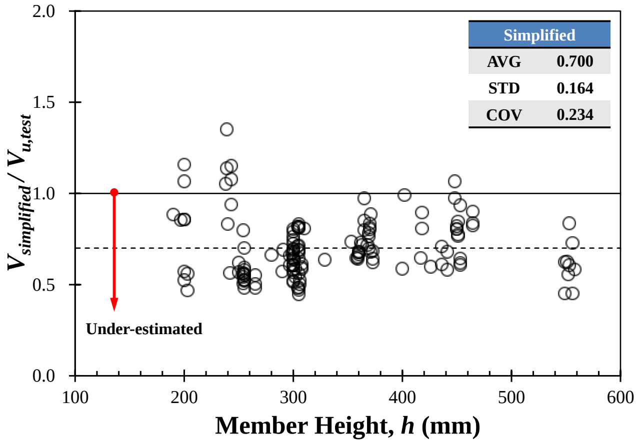
Fig. 9 Ratio of average to parabolic shear distribution



(a) Web-shear strength equation for HCS members



(b) Web-shear strength equation for HCS members considering deep units



(c) Simplified equation

Fig. 10 Performance of the proposed web-shear strength equations

ACI318-08 provision for the deep HCS members without shear reinforcement. Therefore, until further investigations are made, and thereby more test results are available, the safer and more accurate web-shear strengths can be obtained by reducing the web-shear strength in half for the HCS members that are more than 500 mm deep when applying the minimum shear reinforcement regulation, as shown in Fig. 10(b). In other words, due to the lack of test results on the HCS members that are more than 500 mm deep, the web-shear strength reduction in half shall be applied to the HCS members that were more than 500 mm in depth, instead of more than 315 mm deep as in ACI318-08, which also can be an alternative way to consider the size effect in such deep concrete members without shear reinforcement.

In order to use of Eq. (15), however, it is somewhat cumbersome to calculate the sectional constants, such as first area moment and moment of inertia. Adopting the average $I_g/\left(Qd_p\right)$ ratio of 0.76, as shown in Fig. 9, Eq. (15) can be further simplified to give the web-shear strength equation (V_{cw}), as follows:

$$V_{cw}=\left(0.22\sqrt{f_c^{'}}+0.23f_{pc}\right)b_w d_p \tag{16}$$

Fig. 10(c) shows the comparison of the web-shear strengths calculated by Eq. (16) ($V_{simplified}$) and the test results ($V_{u,test}$), wherein the web-shear strength is halved when the HCS members are more than 500 mm deep. It is shown that the proposed Eq. (16) is simple to use but also provides a considerable accuracy with a consistent margin of safety.

5. Conclusions

In this study, the test results on the HCS members failed in web-shear were collected, and a parametric analysis on web-shear strength was performed using the collected test data. Also, the code equations on web-shear strength and related regulations were evaluated thoroughly, and a simple and reasonable web-shear strength equation for HCS members was proposed. Based on this study, the following conclusions can be drawn:

1. Based on the parametric analysis on the shear test results of the HCS members, the size effect was not clearly shown in their web-shear strength. For the HCS members with more than 500 mm in depth, however, it is difficult to affirm any observation on the size effect in web-shear strength at this point due to the lack of test data, and additional experimental investigation on deep HCS members would be required.
2. The web-shear strength equation for HCS members in ACI318 showed a reasonably good level of accuracy. As previous studies have also pointed out, however, it provided unconservative estimation on the shear strength of many HCS members, and such a tendency was also shown in the web-shear strength equations presented in European and Japanese.
3. The simplified shear equation in AASHTO-LRFD had a similar accuracy to the equation in ACI318, but it was excessively conservative.
4. The revised ACI318-08 estimated the web-shear strength of HCS members with more than 315 mm in depth very conservatively when the minimum shear reinforcement requirement is applied. On the other hand, however, the web-shear strength equation in ACI318 provided unconservative estimation for slender HCS members as well.

5. In this study, a simplified web-shear strength equation was proposed, which is easy to use and provides a good accuracy with a proper margin of safety.

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