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Horizontal shear strength on various PC types of composite beams

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ABSTRACT

The use of composite construction method using precast concrete (PC) and cast-in-place (CIP) concrete became common in modular construction. However, current design codes do not clearly define the horizontal shear strength of various types of CIP concrete. In the present study, 49 specimens were tested to evaluate the horizontal shear strengths of composite beams with or without shear reinforcement. The test variables were types of CIP concrete, the cross-sectional area ratios of PC and CIP, and shear reinforcement ratio. The horizontal shear strengths of the test specimens were compared with the predictions of current design codes. Thus, current design code ACI318 predicts horizontal shear strength of specimens without shear reinforcement conservatively. Additionally, shear strength ratio of SFRC were in non-safety side.

1. INTRODUCTION

Recently, hybrid constructions using precast concrete (PC) and cast-in-place (CIP) concrete became popular, particularly for large parking or storage buildings (Fig. 1). In the PC-CIP hybrid construction, the structural integrity can be improved and the lifting weight of PC can be reduced by using CIP concrete. Usually, pre-tension is applied to the PC for better structural performance.

For the composite members, generally, different concrete strengths are used for PC and CIP concrete: high-strength (HS) concrete is used for PC and low-strength (LS) concrete is used for CIP concrete. Following the previous study of Kim et al. (2013, 2014) and Suh et al. (2015), a series of tests of composite beams which of CIP concrete were RC, PSC, and SFRC were performed to examine the shear strength of composite beams. However, because concrete strength are different, horizontal shear

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stress of interface should be considered depending on current design codes. ACI 318 suggests two horizontal shear mechanisms but horizontal shear strength should be carefully evaluated according to CIP concrete types. On the basis of the test results, the validity of current horizontal shear design methods was verified.

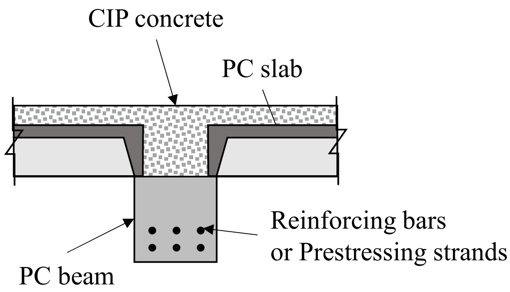


Fig. 1 Composite members using PC and CIP concretes

2. CURRENT DESIGN CODES

2.1 ACI 318

In ACI 318, shear transfer mechanism is used when horizontal shear stress is lower than 3.5 MPa and shear friction method is used when horizontal shear stress higher than 3.5 MPa.

In the case of shear transfer mechanism, where 1) contact surfaces are clean, free of laitance, and intentionally roughened (6mm) or 2) minimum ties are provided and contact surfaces are clean and free of laitance, but not intentionally roughened, the horizontal shear strength of a composite concrete member is defined as follows:

$$v_{nh} \le 0.56 \qquad \text{(MPa, mm)} \tag{1}$$

When transverse reinforcement for horizontal shear is used, the horizontal shear strength is defined as follows:

$$v_{nh} = (1.8 + 0.6\rho_v f_y) \le 3.5 \qquad \text{(MPa, mm)} \tag{2}$$

where ρ_v is the ratio of the cross-sectional area of transverse reinforcement to the interface area, and b_v is the width of the cross section at the interface.

The area of transverse reinforcement shall not be less than the following minimum requirement.

$$A_{v,\min} = 0.0625\sqrt{f_{ck}}\frac{b_w s}{f_{yt}} \ge 0.35 b_w s / f_{yt} \qquad \text{(MPa, mm)} \tag{3}$$

The spacing shall not exceed four times the least dimension of supported element, nor exceed 600mm.

For prestressed members with an effective prestressing force not less than 40 percent of the tensile strength of the flexural reinforcement, the area of transverse reinforcement shall not be less than the smaller requirement value from Eq. (3) and Eq. (4).

$$A_{v,\min} = \frac{A_{ps}}{80}\frac{f_{pu}}{f_{yt}}\frac{s}{d}\sqrt{\frac{d}{b_w}} \geq 0.35 b_w s / f_{yt} \qquad \text{(MPa, mm)} \qquad (4)$$

where A_{ps} is the area of prestressing steel in flexural tension zone, and f_{pu} is the stress in prestressing steel at nominal flexural strength.

If horizontal shear stress exceeds 3.5MPa, shear friction method shall be used.

$$v_{nh} = \mu \rho_v f_y \leq (0.2 f_{ck}, 3.3 + 0.08 f_{ck}) \qquad \text{(MPa, mm)} \qquad (5)$$

where, μ is friction coefficient which depends on state of interface which is presented in Table 1 and $\rho_v f_y$ is clamping force by shear reinforcement.

Table 1. Cohesion (c) and friction factor (*μ*)

Code	State of interface		*c* (MPa)	*μ*
ACI 318	Horizontal shear transfer	Where contact surfaces are clean, free of laitance, and intentionally roughened	0.56	
		Where minimum ties are provided, and contact surface are clean and free of laitance, but not intentionally roughened		
		Where ties are provided, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4in.	1.8	0.6
	Shear friction	Concrete placed monolithically		1.4
		Concrete placed against hardened concrete with surface intensionally roughened		1.0
		Concrete placed against hardened concrete not intensionally roughened		0.6
		Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars		0.7
EC2	Very smooth: a surface cast against steel, plastic or specially prepared wooden mouls		0.25	0.5
	Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration		0.35	0.6
	Rough: a surface with at least 3 mm roughness at about 40mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour		0.45	0.7
	Indented: a surface with indentations		0.50	0.9
AASHTO LRFD	For a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in		1.93	1.0
	For normal-weight concrete placed monolithically		2.76	1.4
	For light weight concrete placed monolithically, or nonmonolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25 in		1.65	1.0
	For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in		1.65	1.0
	For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened		0.52	0.6
	For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars where all steel in contact with concrete is clean and free of paint		0.17	0.7
CSA	For concrete placed against hardened concrete with the surface clean but not intentionally roughened		0.25	0.6
	For concrete placed against hardened concrete with the surface clean and intentionally roughened to a full amplitude of at least 5mm		0.5	1.0
	For concrete placed monolithically		1.0	1.4
	For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars		0.0	0.6

2.2 Other design codes

In the case of Eurcode2, AASHTO LRFD, and CSA as shown in Table 2, however, shear friction mechanism is used regardless of magnitude of horizontal shear stress and they use cohesion of interface(c) and friction coefficient(μ). Where shear reinforcement are provided in and contact surfaces are clean and free of laitance, and intentionally roughened, ACI 318-11, Eurocode2, AASHTO LRFD, and CSA suggests 1.8 MPa, 0.45 MPa, 1.7 MPa, and 0.5 MPa respectively thus these codes suggest different cohesion factors. AASHTO LRFD estimated horizontal shear stress rather unconservative to experimental values whereas these values were in safety side according to ACI 318, Eurocode2, and CSA in condition of same state of contact surface and shear reinforcement.

All design codes suggest different cohesion factors and friction coefficients respectively according to state of interface which can be visible to the naked eye. Whereas ACI 318 suggests three equations depending on horizontal shear stress and shear reinforcement index, EC2, AASHTO LRFD and CSA suggest only one equation to calculate horizontal shear strength using shear friction method. Using ACI 318, it is difficult to calculate feasibly because of discontinuity of equations. Thus clear criteria is needed to estimate horizontal shear strength of PC-CIP composite members.

Table 2. Other design codes

Code	Shear strength model (SI Units)
Eurocode 2	$V_{Rdi} = cf_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu\sin\alpha + \cos\alpha) \le 0.5vf_{cd}$ where: c and μ are factors which depend on the roughness of the interface f_{ctd} is the value of the design tensile strength σ_n is stress per unit area cuased by the minimum external normal force across the interface α should be limited by 45°≤ α ≤ 90° v is a strength reduction factor
AASHTO LRFD	The nominal shear resistance of the interface plane shall be taken as: $V_n = cA_{cv} + \mu(A_{vf}f_y + P_c)$ The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of: $V_{ni} \le K_1 f'_c A_{cv}$ or $V_{ni} \le K_2 A_{cv}$ where: A_{cv} is area of concrete considered to be engaged in interface shear transfer A_{vf} is area of interface shear reinforcement, crossing the shear plane within the area A_{cv} c is cohesion factor specified in Article 5.8.4.3 of AASHTO LRFD μ is friction factor specified in Article 5.8.4.3 K_1 is fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3 K_2 is limiting interface shear resistance specified in Article 5.8.4.3
CSA	The factored shear stress resistance of the plan shall be computed from $V_r = \lambda\phi_c(c + \mu\sigma) + \phi_s\rho_v f_y \cos\alpha_f$ where the expression $\lambda\phi_c(c+\mu\sigma)$ shall not exceed $0.25\phi_c f'_c$ and α_f is the angle between the shear friction reinforcement and the shear plane.

3. VERIFICATION

3.1 Experimental verification

In this study, 49 specimens were tested which are 23 specimens of RC composite beams, 18 specimens of prestressed composite beams and 8 specimens of steel-fiber reinforcement composite beams. All specimens are 260mm×400mm and beam lengths change depending on shear span to depth ratio. And in this test, three parametres were considered: the types of precast members, the area ratio of high-strength (HS: 60MPa) and low- strength (LS: 24MPa) concrete, and spacing of transverse reinforcement (Table 3).

First, three types were considered: RC(Reinforced concrete), PSC(Prestressed concrete) and SFRC(Steel fiber reinforced concrete) members. In PSC, two levels of prestress (f_{pj}), 55% and 70% of f_{pu} were used. In the case of SFRC, steel fiber is Bundrex which are made by Kosteel and aspect ratio ($(l_f / d_b = 30/0.5)$ is 60 and yield strength is 1,336MPa. Fiber volume ratio is 0.75% according to current design code. In all specimens, PC members were made first and after curing 24 hours normal concrete was placed on PC members.

Second, five cross-section types were considered. Composite section C and D were used to evaluate the effect of LS concrete on the overall shear strength. The depth of LS concrete for section C, D, F were 3/8, 5/8 and 7/8 of the overall depth from top, respectively. In composite section E which used the HS concrete in the compression zone, the depth of HS concrete was 3/8 of the overall depth from the top. This condition occurs in the negative bending zone when composite beams are designed as a continuous member.

Third, spacing of shear reinforcement was considered. In the case of RC specimens, non-shear reinforcement, 170mm, 450mm were considered. In PSC, non-shear reinforcement, 200mm, 300mm, and 450mm were considered. And non-shear reinforcement and 450mm spacing of shear reinforcement were considered in SFRC.

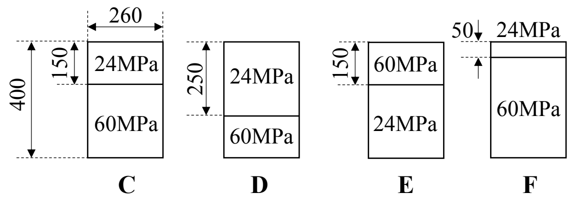


Fig. 2 Section types of composite member (unit: mm)

Two-point vertical loading was applied to the middle of the specimens. Rollers were used at the supporting and loading points. Strain gauges were attached to the strands to check yielding of the strands, and five LVDTs were installed to measure the deflection and curvature of the specimen. In the test results, all specimens occurred shear failure before flexural failure. In specimens which occurred horizontal shear cracks, vertical loads were used at initiating horizontal shear crack. And in specimens which did not occur horizontal cracks, maximum loads were used to calculate horizontal shear stress. However, there is no difference between loads at occurring horizontal shear crack and maximum loads.

Horizontal shear stress can be calculated by two method using vertical shear stress as shown in Fig. 3 and Eq. (6) or actual variation (F_h) of compressive force or tensile force in Fig. 4 and Eq.(7).

$$\tau = \frac{dM}{dx}\frac{1}{Ib}\int ydA = \frac{V}{Ib}\int ydA = \frac{VQ}{Ib} \qquad \text{(MPa, mm)} \tag{6}$$

where, Q is geometric moment of area, I is second moment of inertia, and b is width of beam.

$$\tau = f_h = \frac{F_h}{A_{cr}} = \frac{F_h}{b_v l_{vh}} = \frac{M_{\max}/(0.9d)}{b_v l_{vh}} \qquad \text{(MPa, mm)} \tag{7}$$

where, A_{cr} is area of horizontal interface, l_{vh} is length from inflection point to maximum moment point. In this research, horizontal shear stress was calculated by Eq. (6) which considers actual variation (F_h).

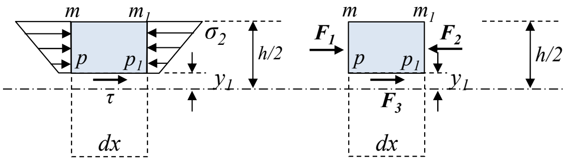


Fig. 3 Horizontal shear stress from vertical shear stress

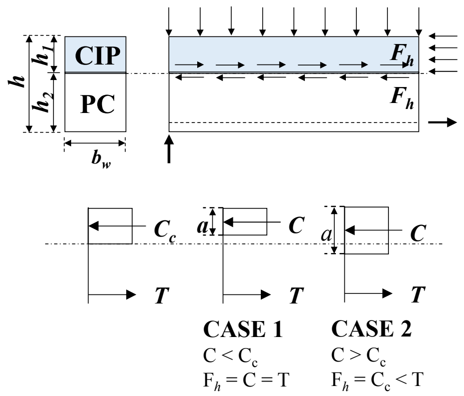


Fig. 4 Horizontal shear stress from actual variation

3.2 Comparison between tests and codes

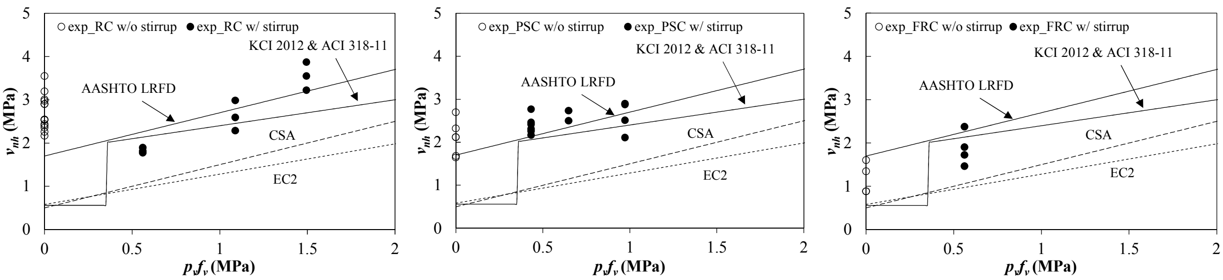
Table 4 presents comparison between experimental values of specimens of three PC types and calculation values using current design codes (ACI 318, EC2, AASHTO LRFD, CSA). And, Fig. 5 shows relationship between test results and codes.

Shear strength ratios (v_{test}/v_{cal}) of RC specimens in Fig. 5(a) were conservative excessively which were 4.11~7.04 of ACI 318, 2.52~7.34 of EC2 and 4.60~7.89 of CSA. However, shear strength ratio of AASHTO LRFD was ranged from 1.35 to 2.32. In case of non-shear reinforcement, ACI 318 and CSA predicted shear strength using only cohesion factor of 0.56 MPa and 0.5 MPa, respectively and these values were rather conservative. Thus additional research is needed through experimental approach. Shear strength ratios (v_{test}/v_{cal}) of SR13, SR14, and SR15 which were 0.56 MPa of shear reinforcement index($\rho_v f_y$) were 0.88~0.94 in ACI 318 and 0.83~0.89 in AASHTO LRFD which were non-safety side.

In PSC specimens (Fig. 5(b)), current codes predicted shear strength conservatively in the case of non-shear reinforcement. Also shear strength ratios of

ACI318 and AASHTO LRFD were 1.17~1.49 and 1.08~1.76 which were predicted reasonably.

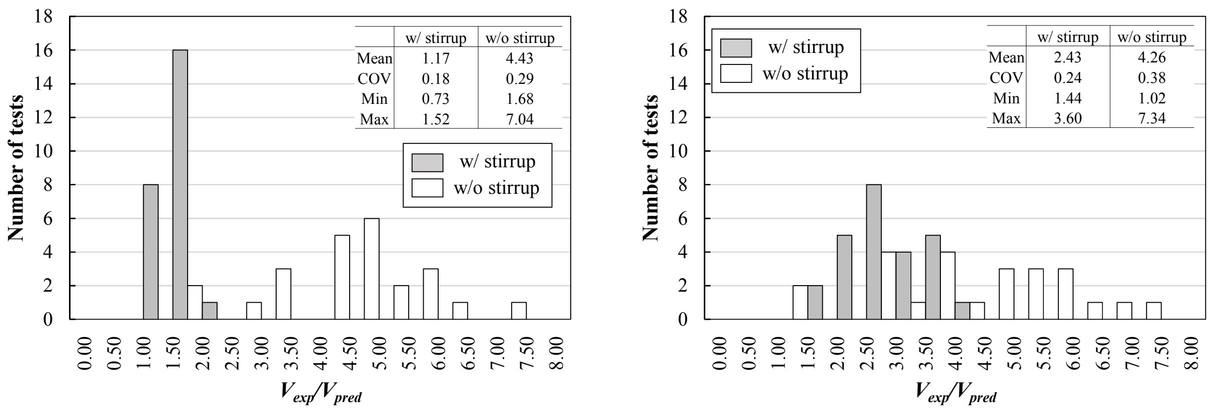
In the case of SFRC (Fig. 5(c)), horizontal shear stress decreased rather than that of RC and PSC specimens. Because current codes do not suggest cohesion factor and friction coefficient in the condition of SFRC, current codes about RC were applied. Horizontal shear stresses of SU-E and SU-F were 0.94MPa. And these values were lower than 1.7MPa presented in AASHTO LRFD and in non-safety side. Also, shear strength ratio of SR-C, SR-D, and SR-F with transverse reinforcement were 0.69~0.95 which were in non-safety side according to ACI 318 and AASHTO LRFD. Thus, material property about steel fibre should be considered.



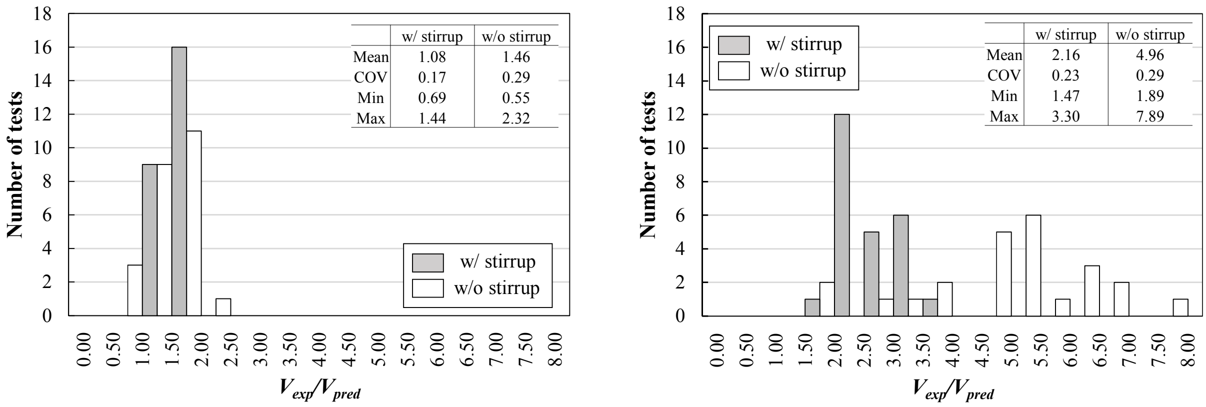
(a) RC specimens (b) PSC specimens (c) SFRC specimens

Fig. 5 Comparison between tests and codes

Fig. 6 shows distribution of shear strength ratio according to shear reinforcement. As shown in Fig. 6(a), (b), (d), distribution according to ACI 318, EC2, and CSA presented separately depending on existence of shear reinforcement and conservatively for non-shear reinforcement. In Fig. 6(c), distribution of shear strength ratio did not be present separately 0.55~2.32. However, 12.5 percent of specimens with non-shear reinforcement and 36 percent of specimens with shear reinforcement shows non-safety side.



(a) ACI 318 (b) EC2



(c) AASHTO LRFD (d) CSA

Fig. 6 Distribution of shear strength ratio according to shear reinforcement

4. CONCLUSION

Composite beams using HS(60MPa) concrete and LS(24MPa) concrete were tested to investigate the horizontal shear strength. The test parameters were types of CIP concrete, the cross-sectional area ratio of PC concrete, and transverse reinforcement ratio. The major findings were summarized as follows.

1) The horizontal shear strengths of the composite beams with or without transverse reinforcement were compared with ACI 318, EC2, AASHTO LRFD, and CSA predictions.
2) In the case of RC and PSC specimens, the shear strength ratios (v_{test} / v_{cal}) of specimens without shear reinforcement were excessively conservative. However, Horizontal shear strengths of SFRC were in non-safety side. Thus, material properties of steel fiber in interface should be considered.
3) Cohesion factor of ACI318, EC2, and CSA is too low to evaluate horizontal shear strength without shear reinforcement. Thus, it should be adjusted higher than current design codes not to lead to separation of distribution of shear strength ratio(v_{test} / v_{cal})

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Table 3. Test variables and predictions of moment and shear capacities of specimens

	Speci-mens	Section types	$f'_{c,CIP}$ (MPa)	$f'_{c,PC}$ (MPa)	a/d	b (mm)	d_1 (mm)	d_2 (mm)	Ties	ρ_v	f_{vy} (MPa)	$\rho_v f_{vy}$ (MPa)	v_{test} (MPa)
RC	3	C	25	55	4.0	260	150	190	-	-	-	-	2.70
	4	D	25	55	4.0	260	250	90	-	-	-	-	2.40
	5	E	55	25	4.0	260	150	190	-	-	-	-	2.30
	6	F	55	25	4.0	260	50	290	-	-	-	-	2.59
	9	C	22	59	4.0	260	150	190	-	-	-	-	3.39
	10	D	23	53	4.0	260	250	90	-	-	-	-	2.55
	11	E	52	23	4.0	260	150	190	-	-	-	-	2.69
	12	F	22	59	4.0	260	50	290	-	-	-	-	3.08
	15	C	25	53	4.0	260	150	175	-	-	-	-	3.94
	16	D	25	53	4.0	260	250	75	-	-	-	-	3.22
	17	E	55	25	4.0	260	150	175	-	-	-	-	3.34
	20	C	27	55	2.5	260	150	190	-	-	-	-	3.15
	21	D	27	55	2.5	260	250	90	-	-	-	-	2.69
	22	E	58	27	2.5	260	150	190	-	-	-	-	2.50
	SR3	C	21	59	2.5	260	150	190	D10@170	0.0032	467	1.49	3.77
	SR4	D	21	64	2.5	260	250	90	D10@170	0.0032	467	1.49	3.42
	SR5	E	52	23	2.5	260	150	190	D10@170	0.0032	467	1.49	4.11
	SR8	C	23	53	4.0	260	150	190	D10@170	0.0032	467	1.09	2.75
	SR9	D	23	53	4.0	260	250	90	D10@170	0.0032	467	1.09	2.43
	SR10	E	52	23	4.0	260	150	190	D10@170	0.0032	467	1.09	3.17
	SR13	C	21	63	4.0	260	150	190	D10@450	0.0012	467	0.56	2.01
	SR14	D	21	63	4.0	260	250	90	D10@450	0.0012	467	0.56	1.89
	SR15	E	58	22	4.0	260	150	190	D10@450	0.0012	467	0.56	1.92
PSC	1-C	C	23	57	3.0	260	150	175	-	-	-	-	2.58
	1-D	D	23	57	3.0	260	250	75	-	-	-	-	2.35
	2-C	C	23	57	3.0	260	150	175	-	-	-	-	3.00
	2-D	D	23	57	3.0	260	250	75	-	-	-	-	2.35
	3-C	C	23	57	4.0	260	150	175	-	-	-	-	1.87
	3-D	D	23	57	4.0	260	250	75	-	-	-	-	1.83
	1-CS	C	26	55	3.0	260	150	175	D10@200	0.0027	360	0.97	3.22
	1-DS	D	26	55	3.0	260	250	75	D10@200	0.0027	360	0.97	2.79
	2-CS	C	26	55	3.0	260	150	175	D10@450	0.0012	360	0.43	2.68
	2-DS	D	26	55	3.0	260	250	75	D10@450	0.0012	360	0.43	2.56
	3-CS	C	22	53	3.0	260	150	175	D10@200	0.0027	360	0.97	3.20
	3-DS	D	22	53	3.0	260	250	75	D10@200	0.0027	360	0.97	2.34
	4-CS	C	22	53	3.0	260	150	175	D10@300	0.0018	360	0.65	3.04
	4-DS	D	22	53	3.0	260	250	75	D10@300	0.0018	360	0.65	2.77
	5-CS	C	26	55	3.0	260	150	175	D10@450	0.0012	360	0.43	3.08
	5-DS	D	26	55	3.0	260	250	75	D10@450	0.0012	360	0.43	2.74
	6-CS	C	22	53	4.0	260	150	175	D10@450	0.0012	360	0.43	2.51
	6-DS	D	22	53	4.0	260	250	75	D10@450	0.0012	360	0.43	2.41
SFRC	SU-C	C	21	56	4.0	260	150	190	-	-	-	-	1.71
	SU-D	D	21	56	4.0	260	250	90	-	-	-	-	1.43
	SU-E	E	56	21	4.0	260	150	190	-	-	-	-	0.94
	SU-F	F	56	21	4.0	260	50	290	-	-	-	-	0.94
	SR-C	C	22	59	4.0	260	150	190	D10@450	0.0012	467	0.56	1.84
	SR-D	D	22	59	4.0	260	250	90	D10@450	0.0012	467	0.56	1.56
	SR-E	E	59	22	4.0	260	150	190	D10@450	0.0012	467	0.56	2.02
	SR-F	F	59	22	4.0	260	50	290	D10@450	0.0012	467	0.56	2.53

Table 4. Calculation value of current design codes and shear strength ratio (v_{test} / v_{cal})

	Speci-mens	Section types	$\rho_v f_{vy}$ (MPa)	① v_{test} (MPa)	v_{cal} (MPa)				Shear strength ratio (v_{test} / v_{cal})			
					② KCI & ACI 318	③ EC2	④ AASHTO LRFD	⑤ CSA	①/②	①/③	①/④	①/⑤
RC	3	C	-	2.70	0.56	0.53	1.70	0.50	4.83	5.06	1.59	5.40
	4	D	-	2.40	0.56	0.53	1.70	0.50	4.29	4.49	1.41	4.80
	5	E	-	2.30	0.56	0.91	1.70	0.50	4.11	2.52	1.35	4.60
	6	F	-	2.59	0.56	0.91	1.70	0.50	4.62	2.83	1.52	5.18
	9	C	-	3.39	0.56	0.49	1.70	0.50	6.06	6.88	2.00	6.79
	10	D	-	2.55	0.56	0.50	1.70	0.50	4.56	5.07	1.50	5.10
	11	E	-	2.69	0.56	0.87	1.70	0.50	4.80	3.08	1.58	5.38
	12	F	-	3.08	0.56	0.49	1.70	0.50	5.50	6.24	1.81	6.16
	15	C	-	3.94	0.56	0.54	1.70	0.50	7.04	7.34	2.32	7.89
	16	D	-	3.22	0.56	0.54	1.70	0.50	5.75	6.00	1.90	6.44
	17	E	-	3.34	0.56	0.91	1.70	0.50	5.96	3.65	1.96	6.68
	20	C	-	3.15	0.56	0.56	1.70	0.50	5.63	5.62	1.86	6.31
	21	D	-	2.69	0.56	0.56	1.70	0.50	4.80	4.79	1.58	5.38
	22	E	-	2.50	0.56	0.94	1.70	0.50	4.47	2.67	1.47	5.00
	SR3	C	1.49	3.77	2.70	1.52	3.19	1.99	1.40	2.48	1.18	1.89
	SR4	D	1.49	3.42	2.70	1.53	3.19	1.99	1.27	2.23	1.07	1.71
	SR5	E	1.49	4.11	2.70	1.92	3.19	1.99	1.52	2.14	1.29	2.06
	SR8	C	1.09	2.75	2.45	1.27	2.79	1.59	1.12	2.18	0.99	1.73
	SR9	D	1.09	2.43	2.45	1.27	2.79	1.59	0.99	1.92	0.87	1.53
	SR10	E	1.09	3.17	2.45	1.63	2.79	1.59	1.29	1.94	1.14	1.99
	SR13	C	0.56	2.01	2.14	0.87	2.26	1.06	0.94	2.31	0.89	1.90
	SR14	D	0.56	1.89	2.14	0.87	2.26	1.06	0.88	2.17	0.83	1.78
	SR15	E	0.56	1.92	2.14	1.34	2.26	1.06	0.90	1.44	0.85	1.81
PSC	1-C	C	-	2.58	0.56	0.51	1.70	0.50	4.60	5.06	1.52	5.15
	1-D	D	-	2.35	0.56	0.51	1.70	0.50	4.20	4.62	1.38	4.71
	2-C	C	-	3.00	0.56	0.51	1.70	0.50	5.35	5.88	1.76	6.00
	2-D	D	-	2.35	0.56	0.51	1.70	0.50	4.20	4.62	1.38	4.71
	3-C	C	-	1.87	0.56	0.51	1.70	0.50	3.33	3.66	1.10	3.73
	3-D	D	-	1.83	0.56	0.51	1.70	0.50	3.26	3.59	1.08	3.66
	1-CS	C	0.97	3.22	2.38	1.23	2.67	1.47	1.35	2.61	1.21	2.19
	1-DS	D	0.97	2.79	2.38	1.23	2.67	1.47	1.17	2.26	1.04	1.89
	2-CS	C	0.43	2.68	2.06	0.86	2.13	0.93	1.30	3.14	1.26	2.88
	2-DS	D	0.43	2.56	2.06	0.86	2.13	0.93	1.25	3.00	1.20	2.75
	3-CS	C	0.97	3.20	2.38	1.18	2.67	1.47	1.34	2.72	1.20	2.17
	3-DS	D	0.97	2.34	2.38	1.18	2.67	1.47	0.98	1.99	0.88	1.59
	4-CS	C	0.65	3.04	2.19	0.95	2.35	1.15	1.39	3.20	1.29	2.65
	4-DS	D	0.65	2.77	2.19	0.95	2.35	1.15	1.27	2.93	1.18	2.42
	5-CS	C	0.43	3.08	2.06	0.86	2.13	0.93	1.49	3.60	1.44	3.30
	5-DS	D	0.43	2.74	2.06	0.86	2.13	0.93	1.33	3.20	1.28	2.93
	6-CS	C	0.43	2.51	2.06	0.80	2.13	0.93	1.22	3.15	1.18	2.69
	6-DS	D	0.43	2.41	2.06	0.80	2.13	0.93	1.17	3.02	1.13	2.58
SRC	SU-C	C	-	1.71	0.56	0.48	1.70	0.50	3.05	3.56	1.01	3.42
	SU-D	D	-	1.43	0.56	0.48	1.70	0.50	2.56	2.99	0.84	2.87
	SU-E	E	-	0.94	0.56	0.92	1.70	0.50	1.68	1.02	0.55	1.89
	SU-F	F	-	0.94	0.56	0.92	1.70	0.50	1.68	1.02	0.55	1.89
	SR-C	C	0.56	1.84	2.14	0.89	2.26	1.06	0.86	2.07	0.81	1.73
	SR-D	D	0.56	1.56	2.14	0.89	2.26	1.06	0.73	1.76	0.69	1.47
	SR-E	E	0.56	2.02	2.14	1.35	2.26	1.06	0.95	1.50	0.90	1.91
	SR-F	F	0.56	2.53	2.14	1.35	2.26	1.06	1.18	1.88	1.12	2.38