36th Conference on

OUR WORLD IN CONCRETE & STRUCTURES

14 – 16 August 2011, Singapore

Conference Theme:

"Recent Advances in the Technology of Fresh Concrete"

Conference Documentation

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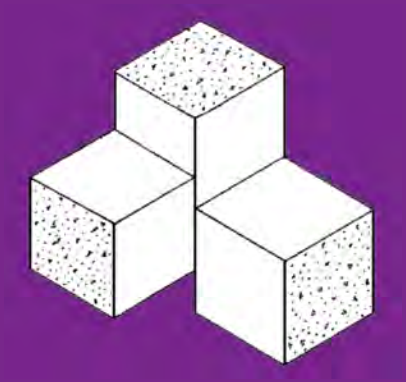

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FOREWORD

The 36th Conference on Our World in Concrete and Structures (OWICS11) is themed "Recent Advances in the Technology of Fresh Concrete". This has always been a major area of focus in this series of conference. Over the years many papers have been presented in this area of concrete research. The intention this year is to bring together all those who share a common interest in this subject area to promote the sharing of new ideas and to sharpen the focus on the significant development and innovation that has taken place in recent years.

OWICS11 is also very special as we are dedicating it to Professor Olafur H Wallervick of the Innovation Centre Iceland for his support of this conference series and for his acknowledged contributions to concrete technology. He will deliver the OWICS11 Conference lecture.

The number of eminent and world renown speakers we have this year have exceeded all our expectations and I would like to thank all speakers, authors and participants for their contributions. Thanks are also due to the OWICS Honorary Chairmen, the OWICS Advisors, our Sponsors and the Organizing Committee.

Khim Chye Gary ONG
&
Min-Hong ZHANG
Conference Chairpersons

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ANALYTICAL STUDY ON HIGH STRENGTH CONCRETE SHEAR WALLS

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Keywords: high strength concrete shear walls, nominal wall strengths, building code formulas

Abstract. *This paper presents an analytical study on the behavior of high strength concrete (HSC) shear walls. Several experiments on HSC shear walls with concrete strength above 60 MPa have been selected to be studied. Data from various experiments were collected and nominal wall strengths have been calculated using several building code formulas, such as those of the ACI (American), AIJ (Japanese), and EC (Eurocode). Subsequently, nominal wall strengths from the building code formulas were compared with actual wall strengths from experiments. Moreover, normalized actual wall strengths over nominal wall strengths and the average shear stresses were also plotted against some significant factors such as shear span ratio, axial load ratio, ratio of longitudinal and transverse reinforcements, etc., in order to observe the behavior of HSC shear walls as influenced by various parameters. The analysis results show that most of the building code formulas underestimate HSC wall strengths for low shear-span ratio (below 2.0) but they predict more accurately for high shear-span ratio (above 2.0). Furthermore, from the results, it seems that axial load up to 0.15 ($f'_c A_g$) does not contribute much to the wall strengths. In addition, the comparative study shows that the contribution of longitudinal reinforcement to wall strengths is more significant than that of the transverse reinforcement. This phenomenon is not accounted for in most building code formulas. Thus, there is a need to develop an expression that can take into account this phenomenon and that can yield better predictions of the strength of HSC walls.*

1 INTRODUCTION

Nowadays, the use of high strength concrete as a structural material has become more common in engineering practice. Compared to normal strength concrete, high strength concrete has many advantages including higher stiffness, higher durability, lower permeability, lower porosity, etc. These advantages make high strength concrete able to cope with modern architectural and structural needs. One of the benefits of high strength concrete is the reduction in the size of structural members such as columns, walls, etc. which can provide more space for other purposes. Furthermore, high strength concrete with low permeability and low porosity can provide better protection for steel reinforcement when corrosion is a major issue. This is a very important aspect when durability of the structure is a concern.

#Nanyang Technological University, Singapore

Shear walls have been used widely in many structures since they provide good resistance to lateral loadings. Moreover, not only for resisting lateral loadings, many structural walls are optimized to resist gravity loading as well. This study presents a review on the behavior of high strength concrete shear walls (HSC walls). Experiments on HSC walls reported by researchers from different countries[1-6] were collected and studied. Data from those experiments were used to calculate nominal wall strengths using building code formulas, such as those formulas recommended by the American Concrete Institute (ACI 318)[7], Architectural Institute of Japan (AIJ)[8], and Eurocode (EC 8)[9]. Subsequently, the nominal wall strengths calculated using the building code formulas were compared with actual wall strengths obtained from experiments. The predicted failure modes were also compared with the actual failure modes. In addition, to investigate further the behavior of HSC walls with various parameters, normalized actual wall strengths and normalized average shear stresses were plotted against shear span ratio, drift ratio, axial load ratio, longitudinal and transverse reinforcement ratios, and concrete strength. Finally, general conclusions based on this analytical study were made regarding the behavior of HSC walls which might be used as a basis for further experimental studies about HSC walls in the future.

2 HSC WALLS EXPERIMENTS

As mentioned before, several experiments about HSC walls reported by researchers from different countries[1-6] were collected and studied. These experiments were reported by Kabeyasawa and Hiraishi[1], Gupta and Rangan[2], Yun et al.[3], Farvashany et al.[4], Yan et al.[5], and Deng et al.[6]. Data from these experiments were collected in terms of concrete strength (f'_c), shear span ratio (H/L; where H is the wall height and L is the wall length), axial load ratio ($P/(f'_cA_g)$; where P is the axial load in wall, and A_g is the gross cross sectional area of wall), longitudinal and transverse reinforcement contributions (ρ_l f_{yl} and ρ_t f_{yt}; where ρ_l and ρ_t are longitudinal and transverse reinforcement ratios of wall, f_{yl} and f_{yt} are the yield strengths of longitudinal and transverse reinforcements), maximum wall strength (lateral load) obtained from experiment, and drift ratio (%). These data are presented in Table 1.

3 ANALYTICAL STUDY

The nominal wall strengths were then calculated according to the methods of the ACI 318[7], AIJ[8], and EC 8[9]. The flexural strength of the walls was calculated based on flexural theory for members subjected to bending and axial loads whereas the shear strength was calculated using formulas given in the codes. The smaller value of the flexural strength and the shear strength was then taken as the nominal strength of the walls as well as the respective predicted failure mode. Shear strength formulas according to various building codes[7-9] are given as follows.

3.1 ACI 318 (Chapter 21)

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f'c} + \rho_t f_{yt}\right) \tag{1}$$

where:

V_n = nominal wall shear strength (N)

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered (mm^2)

α_c = coefficient defining the relative contribution of concrete strength to nominal wall shear strength, which may be taken as 0.25 for H/L ≤ 1.5, 0.17 for H/L ≥ 2.0, and varies linearly between 0.25 and 0.17 for H/L between 1.5 and 2.0

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength

No.	Ref No [No]	Specimen ID	Concrete Strength (f'_c) (MPa)	Shear Span Ratio (H/L)	Axial Load Ratio [$P/(f'_c A_g)$]	Longitudinal Reinforcement Ratio ($\rho_l f_{yl}$) (MPa)	Transverse Reinforcement Ratio ($\rho_t f_{yt}$) (MPa)	V_{max} (kN)	Drift (%)
1	[1]	NW-1	87.6	2.00	0.11	5.34	5.34	1062	1.970
2	[1]	NW-2	93.6	1.33	0.10	5.34	5.34	1468	1.490
3	[1]	NW-3	55.5	2.00	0.13	2.01	2.01	717	0.990
4	[1]	NW-4	54.6	2.00	0.16	2.01	2.01	784	0.930
5	[1]	NW-5	60.3	2.00	0.12	4.02	4.02	900	1.520
6	[1]	NW-6	65.2	2.00	0.13	4.02	4.02	1056	1.340
7	[1]	W-08	103.3	0.67	0.09	5.75	5.75	1670	0.729
8	[1]	W-12	137.5	0.67	0.09	5.75	5.75	1719	0.776
9	[1]	No. 1	65.1	1.33	0.13	1.58	1.58	1101	0.710
10	[1]	No. 2	70.8	1.33	0.12	2.75	2.75	1255	0.700
11	[1]	No. 3	71.8	1.33	0.12	4.22	4.22	1379	0.760
12	[1]	No. 4	103.4	1.33	0.14	4.22	4.22	1697	0.720
13	[1]	No. 5	76.7	2.00	0.11	4.22	4.22	1159	1.000
14	[1]	No. 6	74.1	1.33	0.12	9.31	9.31	1412	0.720
15	[1]	No. 7	71.5	1.33	0.12	7.92	7.92	1499	0.740
16	[1]	No. 8	76.1	1.33	0.11	11.52	11.52	1639	0.760
17	[1]	M35X	62.6	2.00	0.15	6.48	6.48	1049	1.490
18	[1]	M35H	68.6	2.00	0.15	6.48	6.48	1055	1.500
19	[1]	P35H	66.5	2.00	0.15	6.48	6.48	959	1.500
20	[1]	M30H	61.4	2.00	0.13	6.48	6.48	1020	1.450
21	[1]	MW35H	59.7	2.00	0.15	6.48	6.48	1012	1.500
22	[1]	MAE03	58.3	0.60	0.03	3.83	3.83	1460	0.623
23	[1]	MAE07	58.1	0.60	0.03	6.42	6.42	1676	0.592
24	[1]	W48M6	82.3	0.80	0.02	4.44	4.44	1516	0.601
25	[1]	W48M4	82.3	0.80	0.02	4.12	4.12	1479	1.005
26	[1]	W72M8	82.3	0.80	0.02	7.24	7.24	2066	1.014
27	[1]	W72M6	82.3	0.80	0.02	6.65	6.65	2015	1.023
28	[1]	W72M8	101.8	0.80	0.02	7.24	7.24	2128	1.005
29	[1]	W96M8	101.8	0.80	0.02	9.41	9.41	2483	1.022
30	[1]	SMZ01	83.6	0.65	0.00	2.10	2.10	1154	0.865
31	[1]	SMZ03	83.3	0.65	0.00	2.10	2.10	2081	0.809
32	[1]	W8N18	72.7	2.00	0.15	11.31	11.31	882	1.500
33	[1]	W8N13	79.0	2.00	0.10	11.31	11.31	762	1.500
34	[1]	W8N8H	79.4	2.00	0.06	11.31	11.31	689	1.500
35	[1]	TAK01	62.3	1.80	0.11	4.78	4.78	971	1.500
36	[1]	TAK02	62.3	1.80	0.11	6.91	6.91	987	1.500
37	[1]	TAK03	62.3	1.20	0.11	4.78	4.78	1288	1.000
38	[2]	S-1	79.3	1.00	0.00	5.45	2.89	428	1.607
39	[2]	S-2	65.1	1.00	0.07	5.45	2.89	720	1.114
40	[2]	S-3	69.0	1.00	0.13	5.45	2.89	851	0.559
41	[2]	S-4	75.2	1.00	0.00	8.00	2.89	600	1.213
42	[2]	S-5	73.1	1.00	0.06	8.00	2.89	790	0.784
43	[2]	S-6	70.5	1.00	0.13	8.00	2.89	970	0.735
44	[2]	S-7	71.2	1.00	0.06	5.45	5.45	800	0.935
45	[2]	S-F	60.5	1.00	0.04	5.45	2.89	487	2.060
46	[3]	HW1	69.0	1.80	0.06	3.50	3.50	442	2.386
47	[3]	HW2	69.0	1.80	0.03	3.50	3.50	375	2.441
48	[3]	HW3	69.0	1.80	0.00	3.50	3.50	234	2.063
49	[3]	HW4	69.0	1.80	0.03	3.50	7.00	363	2.225
50	[3]	HW5	69.0	1.80	0.03	3.50	1.78	372	2.908
51	[4]	HSCW1	104.0	1.25	0.04	6.74	2.51	735	0.968
52	[4]	HSCW2	93.0	1.25	0.09	6.74	2.51	845	1.125
53	[4]	HSCW3	86.0	1.25	0.09	4.01	2.51	625	0.928
54	[4]	HSCW4	91.0	1.25	0.22	4.01	2.51	866	0.763
55	[4]	HSCW5	84.0	1.25	0.09	6.74	4.01	801	1.318
56	[4]	HSCW6	90.0	1.25	0.05	6.74	4.01	745	1.342
57	[4]	HSCW7	102.0	1.25	0.08	4.01	4.01	800	1.265
58	[5]	SW-1	90.0	2.36	0.15	1.66	1.66	260	1.594
59	[5]	SW-2	90.0	2.36	0.15	6.97	6.97	367	2.836
60	[5]	SW-3	60.0	2.36	0.15	1.66	1.66	200	3.642
61	[5]	SW-4	90.0	2.36	0.25	1.66	1.66	255	2.061
62	[5]	SW-5	90.0	1.50	0.15	1.66	1.66	350	1.714
63	[6]	HPCW-01	61.4	2.10	0.16	2.10	3.36	326	2.019
64	[6]	HPCW-02	73.6	2.10	0.14	2.10	3.36	333	2.480
65	[6]	HPCW-03	75.3	2.10	0.13	2.10	5.09	379	2.448
66	[6]	HPCW-04	86.0	2.10	0.12	2.10	5.09	370	2.677

Table 1: Details of HSC walls reported by researchers[1-6]

3.2 AIJ

$$V_n = tL\rho_t f_{yt} cot\phi + 0.5\, tan\theta\, (1-\beta)\, tL\nu f'c \qquad (2)$$

where:

V_n = nominal wall shear strength (N)
t = thickness of wall panel (mm)
$\cot \phi$ = 1.0
$\tan \theta$ = $\sqrt{(H/L)^2 + 1} - H/L$
β = $(1 + cot^2\phi)\rho_t f_{yt} (\nu f'c)^{-1}$
ν = $0.7 - (f'c/2000)$

3.3 EC8

For diagonal compression failure of the web due to shear:

$$V_n = \alpha_{cw} t z \nu_1 f'c (cot\theta + tan\theta)^{-1} \qquad (3)$$

where:

V_n = nominal wall shear strength (N), which for the critical region, it may be taken as 40% of the calculated value
α_{cw} = coefficient taking account of the state of the stress in the compression chord, which may be taken as 1.0 for non-prestressed structures; $[1 + P/(f'_c A_g)]$ for $0 < P/(f'_c A_g) \leq 0.25$; 1.25 for $0.25 < P/(f'_c A_g) \leq 0.5$; or $2.5\, [1 - P/(f'_c A_g)]$ for $0.5 < P/(f'_c A_g) < 1.0$
t = thickness of wall panel (mm)
z = inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration, which may be taken equal to 0.8L
v_1 = strength reduction factor for concrete cracked in shear, which is $0.6\,(1.0 - f'_c/250)$
$\cot \theta$ = $\tan \theta$ = 1.0

For diagonal tension failure of the web due to shear:

If M/(VL) ≥ 2.0: $$V_n = td\left[C_{Rd,c} k (100\rho_l f'c)^{1/3} + k_1 \sigma_{cp}\right] + zt\rho_t f_{yt} cot\theta \qquad (4)$$

If M/(VL) < 2.0: $$V_n = td\left[C_{Rd,c} k (100\rho_l f'c)^{1/3} + k_1 \sigma_{cp}\right] + 0.75\, t\rho_t f_{yt} M/V \qquad (5)$$

where:

V_n = nominal wall shear strength (N)
t = thickness of wall panel (mm)
d = effective depth of a cross section (mm)
$C_{Rd,c}$ = $0.18/\Upsilon_c$, which Υ_c is taken as 1.0 for nominal strength without reduction factor for material
k = $1 + \sqrt{(200/d)} \leq 2.0$
k_1 = 0.15
σ_{cp} = $P/A_g < 0.2 f'_c$ (MPa)
z = 0.8L
$\cot \theta$ = $\tan \theta$ = 1.0
M = applied bending moment in wall
V = applied shear force in wall

The minimum value of V_n from diagonal compression failure and diagonal tension failure is taken as the nominal shear strength of walls according to EC8[9].

4 ANALYSIS RESULTS

Results are presented in terms of actual wall strengths obtained from experiments normalized by nominal wall strengths, actual mode of failures versus predicted mode of failures, and average shear stresses (shear force divided by wall web area, A_c) normalized by square root of concrete strength. These values are then plotted against various parameters such as shear span ratio, drift ratio, axial load ratio, longitudinal and transverse reinforcement ratios, and concrete strength to investigate further the relationship between these values and those parameters. The analysis results are presented in Tables 2 and 3 as well as Figures 1 to 6.

No.	Specimen ID	Actual Mode of Failure	Vexp / Vcal						Vexp / ($A_c\sqrt{f'_c}$)
			ACI 318	Predicted Mode	AIJ	Predicted Mode	EC 8	Predicted Mode	
1	NW-1	FLEXURE	1.16	FLEXURE	1.16	FLEXURE	1.17	SHEAR	0.83
2	NW-2	FLEXURE	1.39	SHEAR	1.07	FLEXURE	1.58	SHEAR	1.12
3	NW-3	FLEXURE	1.48	SHEAR	0.95	FLEXURE	1.23	SHEAR	0.71
4	NW-4	FLEXURE	1.63	SHEAR	0.91	SHEAR	1.30	SHEAR	0.78
5	NW-5	FLEXURE	1.18	SHEAR	0.97	FLEXURE	1.22	SHEAR	0.85
6	NW-6	FLEXURE	1.36	SHEAR	0.97	FLEXURE	1.35	SHEAR	0.96
7	W-08	SHEAR	1.48	SHEAR	0.61	FLEXURE	2.58	SHEAR	1.21
8	W-12	SHEAR	1.46	SHEAR	0.54	FLEXURE	2.40	SHEAR	1.08
9	No. 1	SHEAR	2.25	SHEAR	0.90	SHEAR	2.58	SHEAR	1.00
10	No. 2	SHEAR	1.90	SHEAR	0.89	SHEAR	2.15	SHEAR	1.10
11	No. 3	SHEAR	1.60	SHEAR	0.89	SHEAR	1.78	SHEAR	1.20
12	No. 4	SHEAR	1.84	SHEAR	0.83	SHEAR	1.89	SHEAR	1.23
13	No. 5	SHEAR	1.41	SHEAR	0.87	SHEAR	1.36	SHEAR	0.97
14	No. 6	SHEAR	1.45	SHEAR	0.70	SHEAR	1.69	SHEAR	1.21
15	No. 7	SHEAR	1.57	SHEAR	0.80	SHEAR	1.82	SHEAR	1.30
16	No. 8	SHEAR	1.66	SHEAR	0.73	SHEAR	1.93	SHEAR	1.38
17	M35X	FLEXURE	1.17	SHEAR	1.08	FLEXURE	1.35	SHEAR	0.97
18	M35H	FLEXURE	1.13	SHEAR	1.04	FLEXURE	1.28	SHEAR	0.94
19	P35H	FLEXURE	1.04	SHEAR	0.96	FLEXURE	1.18	SHEAR	0.86
20	M30H	FLEXURE	1.15	SHEAR	1.13	FLEXURE	1.35	SHEAR	0.96
21	MW35H	FLEXURE	1.16	SHEAR	1.07	FLEXURE	1.34	SHEAR	0.96
22	MAE03	SHEAR	1.46	SHEAR	0.64	SHEAR	3.00	SHEAR	1.10
23	MAE07	SHEAR	1.52	SHEAR	0.68	SHEAR	2.38	SHEAR	1.26
24	W48M6	SHEAR	1.10	SHEAR	0.98	FLEXURE	1.98	SHEAR	0.81
25	W48M4	SHEAR	1.12	SHEAR	1.05	FLEXURE	1.96	SHEAR	0.79
26	W72M8	SHEAR	1.33	SHEAR	0.98	FLEXURE	1.88	SHEAR	1.10
27	W72M6	SHEAR	1.30	SHEAR	1.01	FLEXURE	1.92	SHEAR	1.08
28	W72M8	SHEAR	1.23	SHEAR	1.01	FLEXURE	1.91	SHEAR	1.02
29	W96M8	SHEAR	1.44	SHEAR	0.95	FLEXURE	1.80	SHEAR	1.19
30	SMZ01	FLEXURE	1.30	SHEAR	1.00	FLEXURE	3.31	SHEAR	0.62
31	SMZ03	FLEXURE	2.35	SHEAR	0.90	FLEXURE	5.96	SHEAR	1.13
32	W8N18	FLEXURE	1.11	SHEAR	1.06	FLEXURE	1.29	SHEAR	0.92
33	W8N13	FLEXURE	1.07	FLEXURE	1.07	FLEXURE	1.11	SHEAR	0.77
34	W8N8H	FLEXURE	1.03	FLEXURE	1.03	FLEXURE	1.04	SHEAR	0.69
35	TAK01	FLEXURE	1.11	FLEXURE	1.11	FLEXURE	1.11	FLEXURE	0.76
36	TAK02	FLEXURE	1.04	FLEXURE	1.04	FLEXURE	1.10	SHEAR	0.77
37	TAK03	FLEXURE	1.22	SHEAR	0.99	FLEXURE	1.43	SHEAR	1.01
38	S-1	SHEAR	1.11	SHEAR	0.89	FLEXURE	1.22	SHEAR	0.64
39	S-2	SHEAR	1.96	SHEAR	0.90	SHEAR	1.25	SHEAR	1.19
40	S-3	SHEAR	2.28	SHEAR	1.01	SHEAR	0.98	SHEAR	1.37
41	S-4	SHEAR	1.58	SHEAR	0.84	FLEXURE	1.21	SHEAR	0.92
42	S-5	SHEAR	2.10	SHEAR	0.90	SHEAR	1.24	SHEAR	1.23
43	S-6	SHEAR	2.59	SHEAR	1.13	SHEAR	1.76	SHEAR	1.54
44	S-7	SHEAR	1.52	SHEAR	0.91	FLEXURE	2.52	SHEAR	1.26
45	S-F	FLEXURE	1.34	SHEAR	1.20	FLEXURE	2.53	SHEAR	0.83
46	HW1	FLEXURE	1.22	FLEXURE	1.22	FLEXURE	2.37	FLEXURE	0.52
47	HW2	FLEXURE	1.25	FLEXURE	1.25	FLEXURE	2.63	FLEXURE	0.44
48	HW3	FLEXURE	0.98	FLEXURE	0.98	FLEXURE	2.78	FLEXURE	0.28
49	HW4	FLEXURE	1.21	FLEXURE	1.21	FLEXURE	1.85	FLEXURE	0.43
50	HW5	FLEXURE	1.24	FLEXURE	1.24	FLEXURE	1.87	FLEXURE	0.44
51	HSCW1	SHEAR	2.20	SHEAR	0.83	SHEAR	2.60	SHEAR	1.09
52	HSCW2	SHEAR	2.60	SHEAR	1.04	SHEAR	2.70	SHEAR	1.33
53	HSCW3	SHEAR	1.96	SHEAR	0.82	SHEAR	2.10	SHEAR	1.02
54	HSCW4	SHEAR	2.68	SHEAR	1.09	SHEAR	2.19	SHEAR	1.38
55	HSCW5	SHEAR	1.93	SHEAR	0.99	SHEAR	1.99	SHEAR	1.32
56	HSCW6	SHEAR	1.77	SHEAR	0.87	SHEAR	2.00	SHEAR	1.19
57	HSCW7	SHEAR	1.85	SHEAR	0.85	SHEAR	2.03	SHEAR	1.20
58	SW-1	FLEXURE	1.13	SHEAR	1.10	FLEXURE	0.98	FLEXURE	0.39
59	SW-2	FLEXURE	0.95	FLEXURE	0.95	FLEXURE	1.00	FLEXURE	0.55
60	SW-3	FLEXURE	1.10	FLEXURE	1.10	FLEXURE	0.99	FLEXURE	0.37
61	SW-4	FLEXURE	1.11	SHEAR	0.73	FLEXURE	1.03	SHEAR	0.38
62	SW-5	SHEAR	1.24	SHEAR	0.94	FLEXURE	1.10	SHEAR	0.53
63	HPCW-01	FLEXURE	0.98	FLEXURE	0.98	FLEXURE	0.95	FLEXURE	0.42
64	HPCW-02	FLEXURE	1.00	FLEXURE	1.00	FLEXURE	1.10	FLEXURE	0.39
65	HPCW-03	FLEXURE	0.99	FLEXURE	0.99	FLEXURE	0.74	FLEXURE	0.44
66	HPCW-04	FLEXURE	1.03	FLEXURE	1.03	FLEXURE	1.09	FLEXURE	0.40

Table 2: Values of normalized actual wall strengths and normalized average shear stresses; actual mode of failure and predicted mode of failure

Statistical Parameters	Vexp / Vcal			Vexp / ($A_c\sqrt{f'_c}$)
	ACI 318	AIJ	EC 8	
Minimum Value	0.95	0.54	0.74	0.28
Maximum Value	2.68	1.25	5.96	1.54
Average (Mean Value)	1.46	0.96	1.77	0.91
Standard Deviation	0.44	0.15	0.80	0.32
Covariance	0.30	0.16	0.45	0.35

Table 3: Statistical parameters of normalized actual wall strengths and normalized average shear stresses

As observed, in general, the ACI and EC8 methods underestimate the actual wall strengths. It is understood that most of building codes tend to give lower predictions on strengths such that the design formulas are safe enough to be used for practical design. The AIJ method seems to be the most accurate, with the lowest coefficient of variation compared to the other methods. However, the AIJ method may overestimate wall strengths in some cases.

Comparing the actual mode of failure versus predicted mode of failure, the AIJ method gives the least false predictions of modes of failure among the three codes. The AIJ method fails to predict modes of failure of 13 specimens out of 66 specimens whereas the ACI-318 method fails in 17 specimens and the EC8 method fails in 20 specimens. Further investigation on the failure modes shows that most of the time, the AIJ method gives nominal wall shear strengths higher than the actual ones, which results in the tendency of predicting flexural failures instead of shear failures. On the other hand, the ACI-318 and the EC8 methods tend to give lower nominal wall shear strengths compared to the actual ones, and hence resulting in the tendency of predicting shear failures instead of flexural failures.

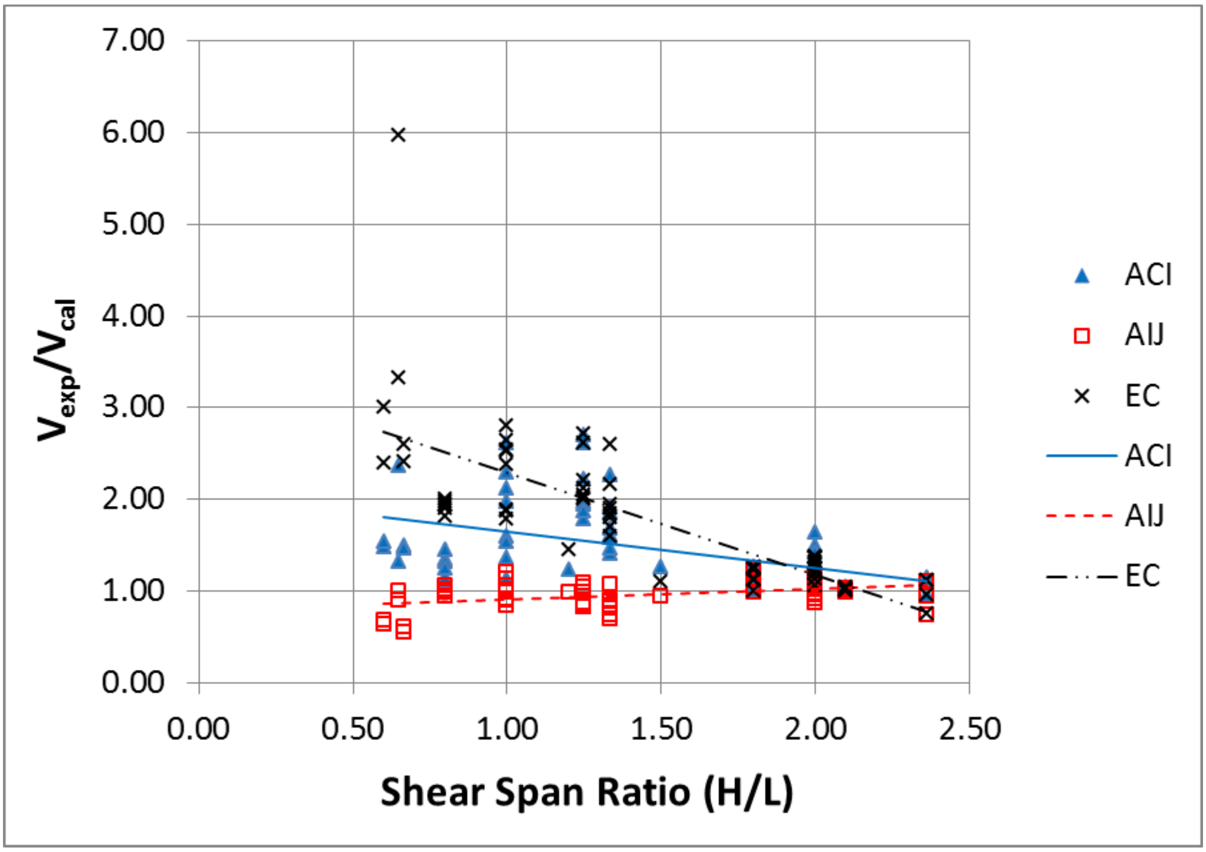

Figure 1: Normalized actual wall strengths plotted against shear span ratio

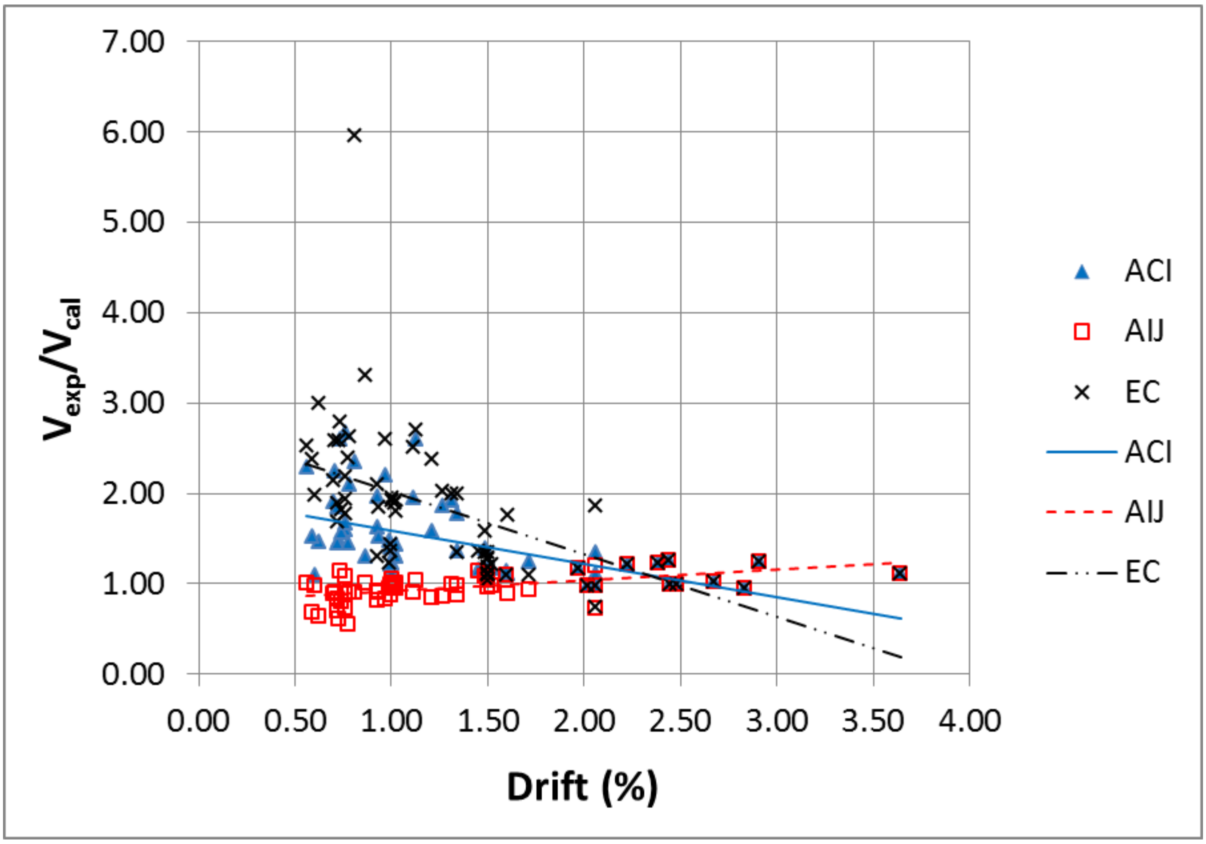

Figure 2: Normalized actual wall strengths plotted against drift ratio

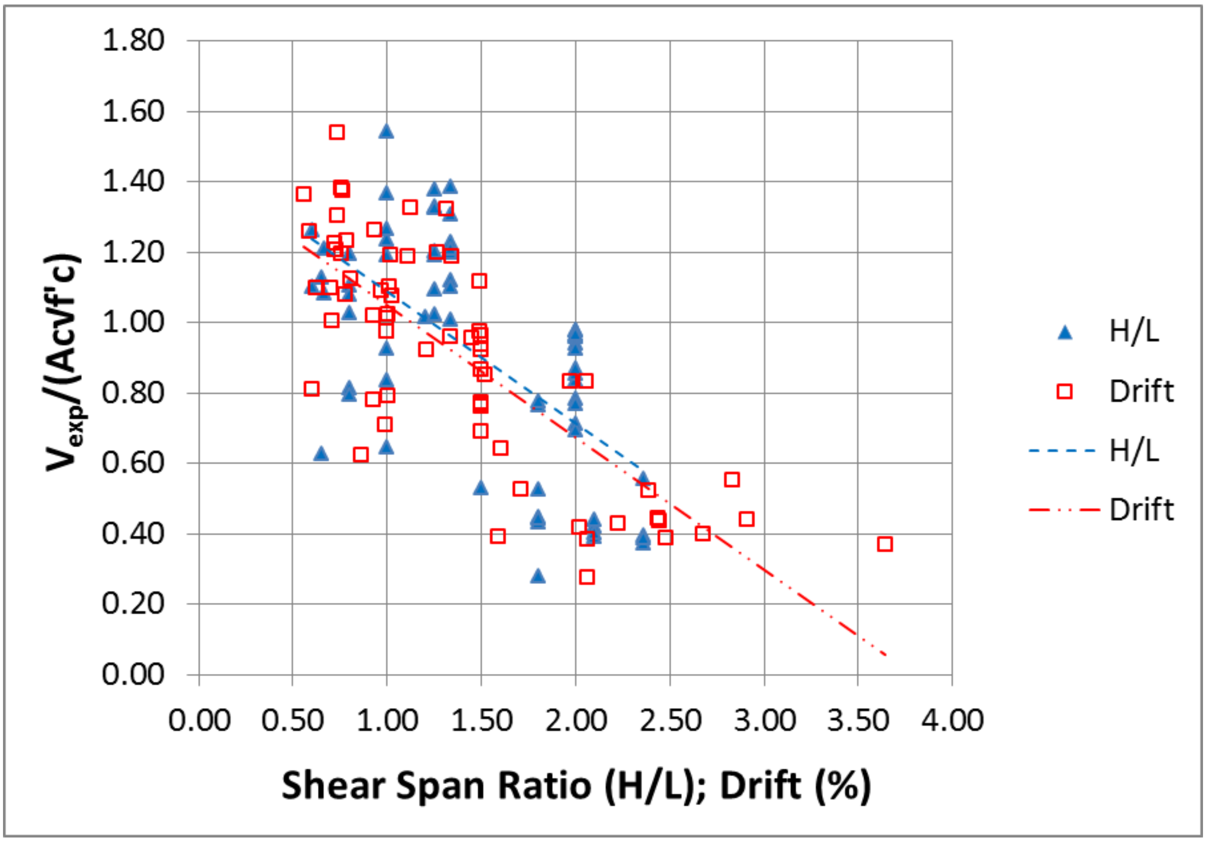

Figure 3: Normalized average shear stresses plotted against shear span ratio and drift ratio

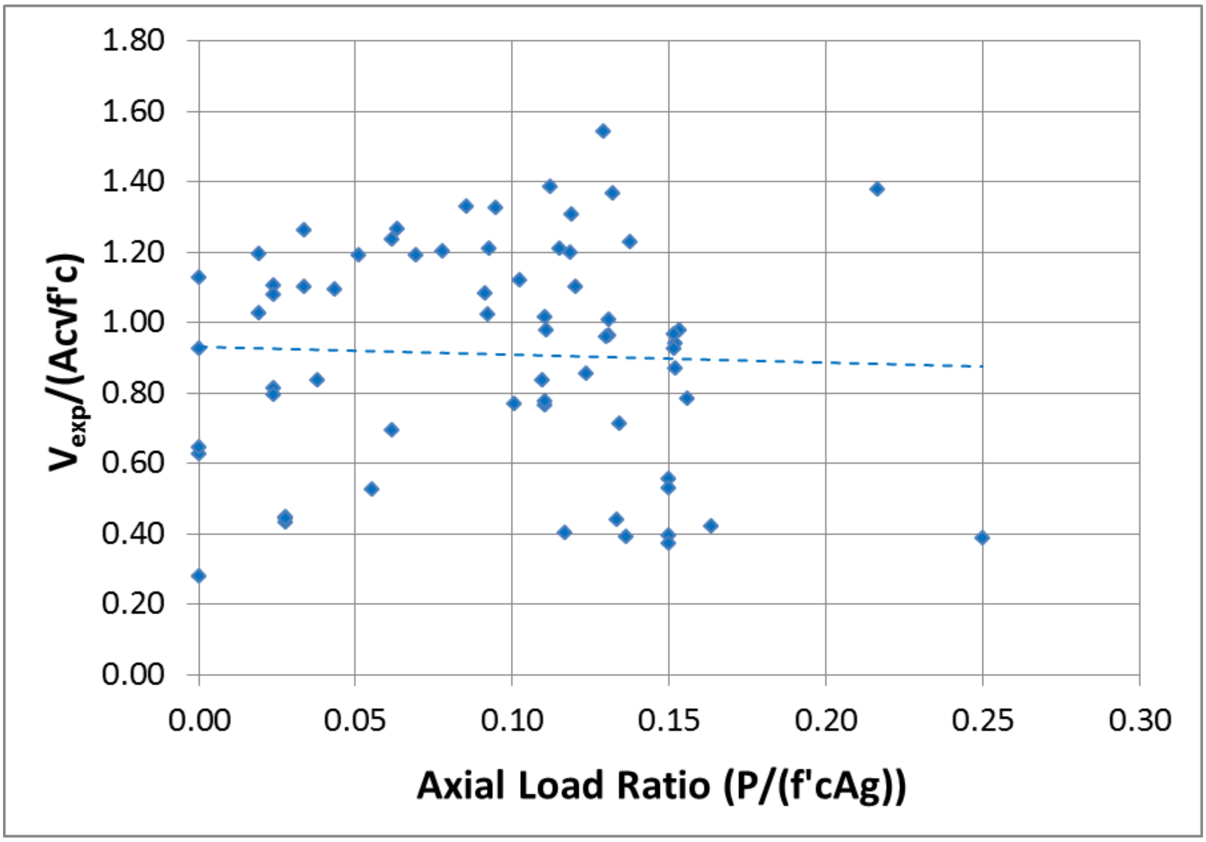

Figure 4: Normalized average shear stresses plotted against axial load ratio

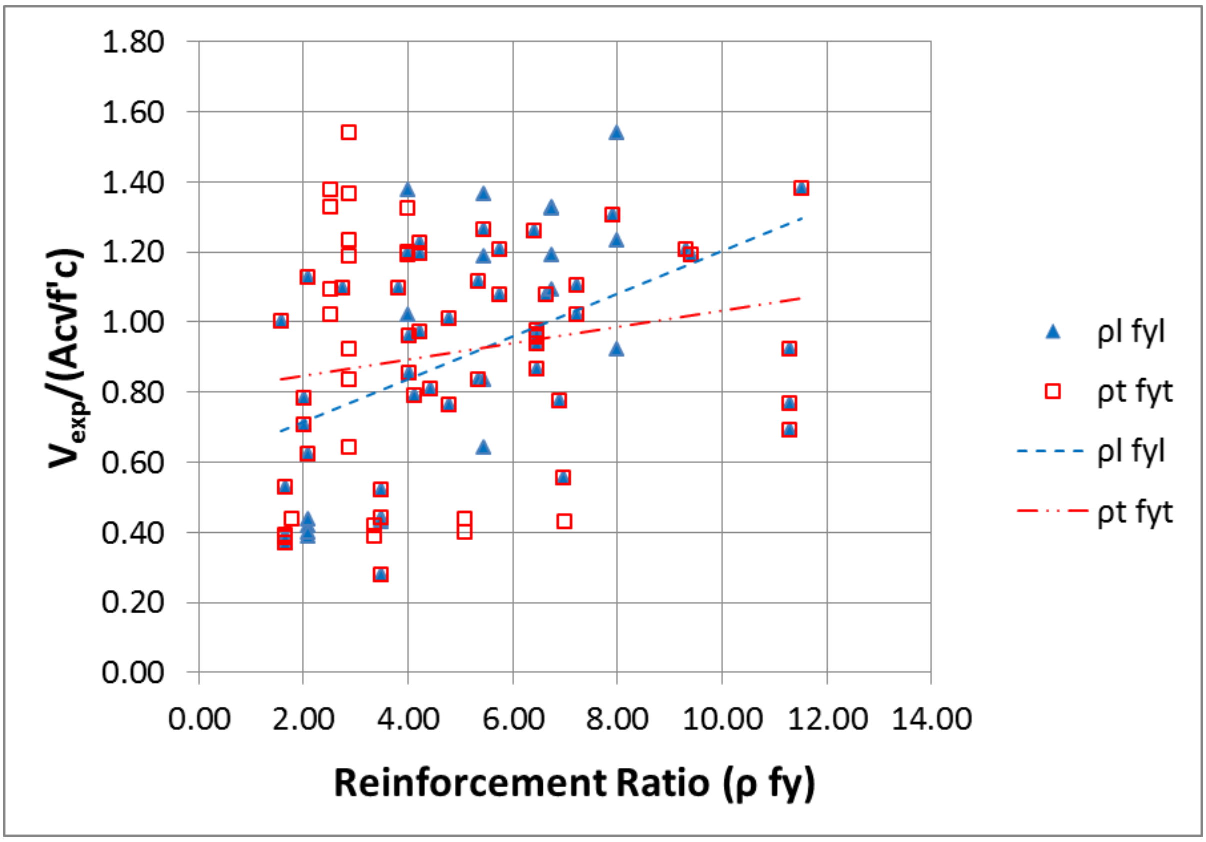

Figure 5: Normalized average shear stresses plotted against reinforcement ratio

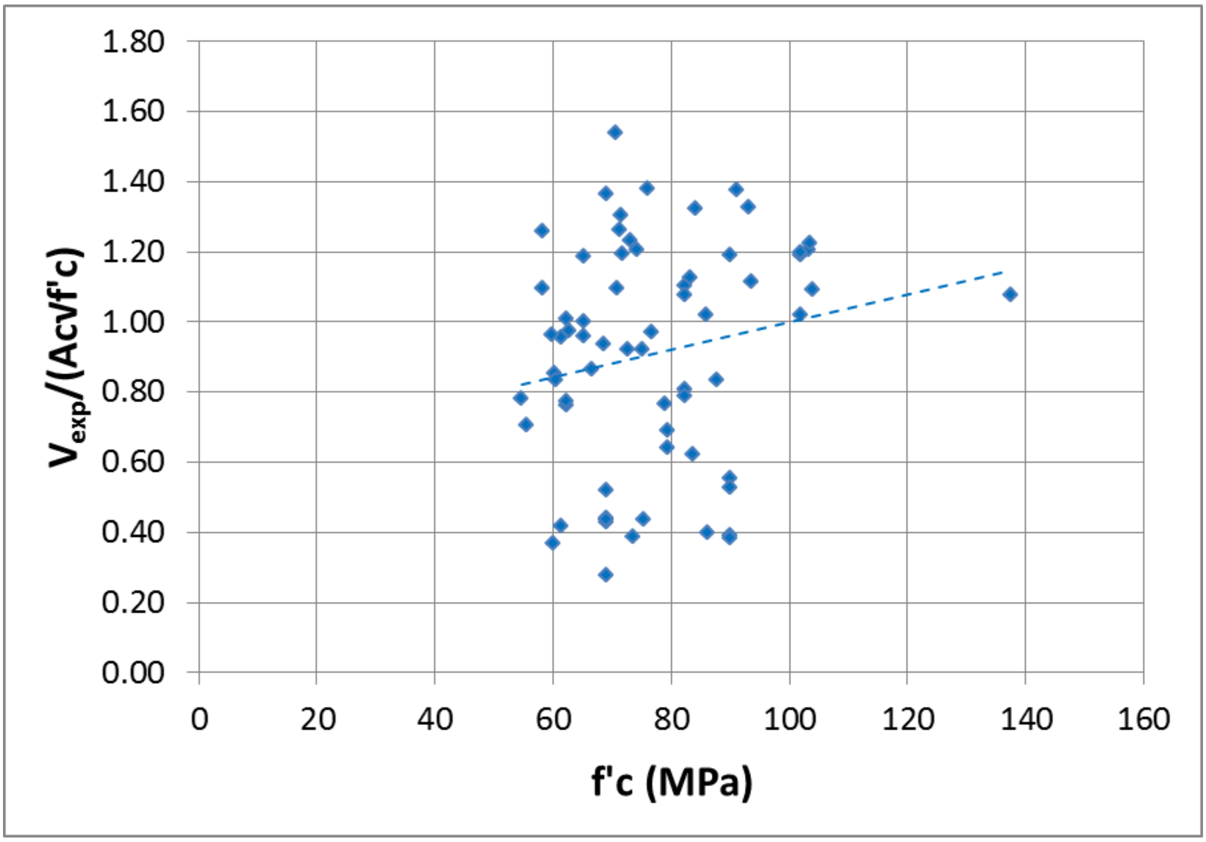

Figure 6: Normalized average shear stresses plotted against concrete strength

From Figure 1, it can be concluded that both the ACI and EC8 formulas underestimate the wall strengths in low shear span ratios (i.e., shear behavior dominates) whereas in high shear span ratios (i.e., flexural behavior dominates), they predict relatively close to the actual wall strengths. Similar phenomenon can also be observed (Figure 2) from the drift ratio, which shows that both the ACI and EC8 formulas underestimate the strength of walls failing in low drift ratio (i.e., shear failure) whereas they predict more accurately the strength of walls failing in high drift ratio (i.e., flexural failure). This means that for the flexural strength, the flexural theories given in those building codes are quite reasonable for predicting the actual flexural strength of HSC walls. For the shear strength, those building code formulas, which are mostly empirical, do not give accurate prediction of the actual shear strength of HSC walls. Moreover, from Figure 3, another conclusion that can be drawn is that as the shear span ratio and the drift ratio increase, the normalized average shear stresses tend to reduce. This means that the flexural behavior is more dominant in high shear span ratio and high drift ratio. Furthermore, according to Figure 4, it seems that an axial load of up to 0.15 (f'_c A_g) does not affect much the wall strengths. The trend line of normalized average shear stresses is nearly flat regardless of the changes in the axial load ratio.

Figure 5 shows that the longitudinal reinforcement affects wall strengths more than the transverse reinforcement. The normalized average shear stresses (i.e., wall strengths) increase more when the longitudinal reinforcement ratio increases, as compared to the increment due to the increase in the transverse reinforcement ratio. This phenomenon, however, is not taken into account in most building code formulas for calculating wall shear strengths. Most of building code formulas only take into account the contribution of the transverse reinforcement while neglecting the contribution of the longitudinal reinforcement. Thus, there is a need to develop a general expression to take into account the contribution of the longitudinal reinforcement when calculating wall shear strengths in order to obtain better predictions. Furthermore, from further investigation of the data, in some cases the ACI-318 and EC8 methods underestimate wall shear strengths because of the upper limit that is imposed on the maximum wall shear strength. For example, in ACI-318, there is a limitation on the value of the maximum shear stress in walls, which is set at $0.83\sqrt{f'_c}$. From the analysis results, it is shown that the average shear stresses in HSC walls is about $0.91\sqrt{f'_c}$ which exceeds the maximum limit provided by ACI-318. Hence, the ACI-318 method can underestimate wall shear strengths because of the upper bound values of the nominal wall shear strength.

The effect of concrete strength is shown in Figure 6. It can be seen that the normalized average shear stresses seems to increase with an increase in concrete strength. Note, however, that the shear strength of walls also depends on factors, such as reinforcement ratios, shear span ratio, etc.

5 CONCLUSIONS

An analytical study on the behavior of HSC walls (above 60 MPa) based on various available experimental results is presented in this paper. Several general conclusions can be drawn. These conclusions are as follows.

Most of building code formulas underestimate the strength of HSC walls failing in shear while they can predict relatively accurately for the ones failing in flexure. The underestimation of the shear strength of HSC walls can be caused by a few inaccuracies in the shear strength formulas, but two factors are especially important. One is the neglected contribution of longitudinal reinforcement to wall shear strengths, and another one is the limitation on the maximum shear strength values, which seems to be quite conservative for HSC walls.

This paper discusses 66 HSC wall specimens that the authors can find in the literature. More experimental studies on the behavior of HSC walls (above 60 MPa), especially those failing in shear, are needed to provide more data which can be used to develop a general expression for predicting the strength of HSC walls. That kind of research is currently being done by the authors.

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Statlstlcal Parameters ACI 318 Ve•n/ V cal EC B AIJ Vexp / (A.vi'.) Minimum Value 0.95 0.54 0.74 0.28 Maximum Value 2.68 1.25 5.96 1.54 Averaae IMean Valuel 1.46 0.96 1.77 0.91 Standard Deviation 0.44 0.15 0.80 0.32 Covariance 0.30 0.16 0.45 0.35 Table 3: Statistical parameters of normalized actual wall strengths and normalized average shear stresses As observed, in general, the ACI and EC8 methods underestimate the actual wall strengths. It is understood that most of building codes tend to give lower predictions on strengths such that the design formulas are safe enough to be used for practical design. The AIJ method seems to be the most accurate, with the lowest coefficient of variation compared to the other methods. However, the AIJ method may overestimate wall strengths in some cases. Comparing the actual mode of failure versus predicted mode of failure, the AIJ method gives the least false predictions of modes of failure among the three codes. 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"a. x x ACI AIJ EC >"' 3.00 2.00 1.00 '!!I - AIJ -EC 0.00 0.00 0.50 1.00 1.50 2.00 2.50 Shear Span Ratio (H/L) Figure 1: Normalized actual wall strengths plotted against shear span ratio 7.00 6.00 5.00 i.:<"x>.'. 43..0000 ACI AIJ)(> x EC 2.00 AIJ LOO 0.00 0.00 0.50 1.00 1.50 2.00 2.50 3.00 3.50 4.00 Drift (%) Figure 2: Normalized actual wall strengths plotted against drift ratio 1.80 1.60 1.40 - v 1.20 -9C>ui;(<">'. 00L.O.86O00 H/L Drift H/L 0.40 Drift 0.20 0.00 0.00 0.50 1.00 1.50 2.00 2.50 3.00 3.50 4.00 Shea r Spa n Ratio (H/l); Drift (%) Figure 3: Normalized average shear stresses plotted against shear span ratio and drift ratio 1.80 1.60 1.40 ū 1.20 :::::::: 0.80 <(x 1.00 • c.. >" 0.60 0.20 0.00 0.00 0.05 0.10 0.15 0.20 0.25 0.30 Axial Load Ratio (P/(fcAg)) Figure 4: Normalized average shear stresses plotted against axial load ratio 1.80 1.60 1.40 pl fyl pt f yt pl fyl 0.40 pt fyt Ill 0.20 0.00 0.00 2.00 4.00 6.00 8.00 10.00 12.00 14.00 Reinforcement Ratio (p fy} Figure 5: Normalized average shear stresses plotted against reinforcement ratio 1.80 1.60 - 1.40 u 1.20 -"S<>au&;;"'":_" 001...860000 0.40 • • 0.20 0.00 0 20 40 60 80 100 120 140 160 f c (MPa} Figure 6: Normalized average shear stresses plotted against concrete strength From Figure 1, it can be concluded that both the ACI and ECB formulas underestimate the wall strengths in low shear span ratios (i.e., shear behavior dominates) whereas in high shear span ratios (i.e., flexural behavior dominates), they predict relatively close to the actual wall strengths. Similar phenomenon can also be observed (Figure 2) from the drift ratio, which shows that both the ACI and ECB formulas underestimate the strength of walls failing in low drift ratio (i.e., shear failure) whereas they predict more accurately the strength of walls failing in high drift ratio (i.e., flexural failure). This means that for the flexural strength, the flexural theories given in those building codes are quite reasonable for predicting the actual flexural strength of HSC walls. For the shear strength, those building code formulas, which are mostly empirical, do not give accurate prediction of the actual shear strength of HSC walls. Moreover, from Figure 3, another conclusion that can be drawn isthat as the shear span ratio and the drift ratio increase, the normalized average shear stresses tend to reduce. This means that the flexural behavior is more dominant in high shear span ratio and high drift ratio. Furthermore, according to Figure 4, it seems that an axial load of up to 0.15 (fc Ag) does not affect much the wall strengths. The trend line of normalized average shear stresses is nearly flat regardless of the changes in the axial load ratio. Figure 5 shows that the longitudinal reinforcement affects wall strengths more than the transverse reinforcement. The normalized average shear stresses (i.e., wall strengths) increase more when the longitudinal reinforcement ratio increases, as compared to the increment due to the increase in the transverse reinforcement ratio. This phenomenon, however, is not taken into account in most building code formulas for calculating wall shear strengths. Most of building code formulas only take into account the contribution of the transverse reinforcement while neglecting the contribution of the longitudinal reinforcement. 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