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CYCLIC SHEAR BEHAVIOR OF HIGH STRENGTH CONCRETE STRUCTURAL WALLS Susanto Teng and Jimmy Chandra 1

Biography: ACI Member Susanto Teng is an Associate Professor at Nanyang Technological University, Singapore.

He is a member of ACI Committees 445 – Shear and 6

Torsion, 435 – Deflection, and 421 – Design of concrete slabs. His research interests include behavior of structural concrete walls, shear strength of slabs, size effect in shear behavior of concrete members, computational modeling of concrete structures, and durability of marine concrete structures. Jimmy Chandra

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is also a lecturer at Petra Christian University, Indonesia. He received

his Bachelor of Engineering degree from Petra Christian University, Indonesia 37

and his Master of Engineering degree from Asian

Institute of Technology, Thailand. His research interests include behavior and seismic performance evaluation of reinforced concrete 6

structures.

ABSTRACT High strength concrete (HSC) walls, having compressive strength of about **100 MPa, were tested under cyclic lateral loading to investigate their shear behavior. The parameters included were height to length ratio of the walls, vertical and horizontal web reinforcement ratios, and the effects of boundary elements in the form of flanges. The experimental results show that shorter walls exhibit greater shear strength than taller walls. Both vertical and horizontal web reinforcements contribute significantly to increasing the shear strength of the** 1 **walls, with the horizontal web reinforcement being more effective for walls having height to length ratio from 1.0 to 2.0. With** increment **in height to length ratio of walls, the concrete contribution to the shear strength decreases while the web reinforcement contribution increases. The presence of flanges also significantly increases the shear strength of HSC walls. Experimental wall shear strengths from this study as well as from literature were compared with predictions from the ACI Code and Eurocode provisions. It can be seen that both** the **ACI and Eurocode do not give consistent safety factors. The ACI method can be unsafe for low strength concrete walls while the Eurocode is overly conservative in almost all cases.** 1

Keywords: High strength

concrete (HSC) walls; shear behavior; shear strength; height to length **ratio;** 1

web reinforcement ratio; boundary element; building codes. INTRODUCTION Reinforced concrete

(RC) walls are one of the most critical **structural** members **in buildings to** 4

carry lateral loadings from wind, earthquake, as well as gravity. In typical buildings, it can be in the form of shear walls or core walls for lifts or staircases. Despite their importance, the behavior of structural walls is not yet fully understood. The ACI 318 [1] and Eurocode 8 [2] provide guidance for designing structural walls, but their safety factors are still not uniform across relevant ranges of many important design parameters. Previous studies by Cardenas and Magura [3] and Cardenas, Russell, and Corley [4] show that the nominal strength of high rise RC walls with

height to length ratio (hw/lw) greater **than** 2 **.0 is governed** more **by flexural** 17 action while the nominal strength of low rise RC walls with

hw/lw less than 2 .0 is governed more by 17

shear action. It is generally understood that flexural strength can be predicted reasonably accurately using flexural theory while shear strength determination is more complex. 2 There are very few experiments that investigate shear behavior of HSC walls with compressive strength (f'c) of 100 MPa (14500 psi) and higher. As the use of HSC as structural material

becomes more common in engineering practice nowadays, it is necessary to study the 6

behavior of such HSC walls and factors affecting it. This current investigation concentrates on walls failing in shear. The parameters investigated include

height to length ratio (hw/lw) of walls, vertical and horizontal web reinforcement ratios (ρv and ρh), and the effect of wall flanges. The 1

specimens were subjected to vertical axial loading and in-plane cyclic lateral loading which is assumed to simulate typical lateral loading due to earthquake. RESEARCH SIGNIFICANCE This study focuses on experimental investigation of the cyclic shear behavior of

high strength concrete (HSC) walls with compressive strength (f'c) of about 100 MPa (14500 psi). 1

The authors expect that, in addition to providing additional data on HSC walls, this study can also provide useful information for the better understanding of shear behavior of HSC walls subjected to cyclic lateral loading. BUILDING CODE PROVISIONS ACI 318 provisions According to ACI 318 Chapter 18 [1], the nominal

shear strength of RC walls can be calculated as follows 40

(in SI Units): = (√' +)

(1) where: Vn = nominal shear strength 5

(in Newton). Acv =

gross area of concrete section bounded by web thickness and length of 5 section in the direction of shear force considered (in mm2). αc = coefficient defining the relative contribution of

concrete strength to nominal wall shear strength, which 5

may be taken as 0.25 for hw/lw ≤ 1.5,

0.17 for hw/lw ≥ 2.0, and varies linearly between 0.25 and 0.17 for hw/lw between 1.5 and 2.0; where hw/lw is the height to length ratio of 4

the wall. These coefficient values are valid for SI units.

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength. 14

f'c = specified

compressive strength of concrete (in MPa). ρt = ratio of area of distributed transverse (horizontal) reinforcement to gross concrete area perpendicular to that reinforcement. fy = specified yield strength of reinforcement (in MPa). 10

Furthermore, ACI 318 [1] also limits that the value of Vn

shall not be taken larger than 0.83Acw √f'c, where Acw is the area of concrete section of the individual vertical wall segment considered. 13

Eurocode 8 provisions In this study, Eurocode 8 [2] provisions for ductile RC walls

were used to calculate the shear strength of specimens collected. According to the 38

Eurocode 8, the ultimate shear strength (resistance) of RC walls is taken as the minimum shear strength between two failure modes, i.e.

diagonal compression failure of the web due to shear, 8

, , and

diagonal tension failure of the web due to shear 8

(either , or as explained below). The formulations are described as follows: 4 ?

Diagonal compression failure of the web due to shear: 8

, = 1/(cot + tan) (2) where: VRd,max =

design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts. For the critical region, the value 12

is taken as

40% of the value outside the critical region. 55

αcw =

a coefficient taking account of the state of the stress in the compression chord. The recommended value of αcw is 18

as follows: 1.0 for non-prestressed structures (2a) (1.0 +

σcp/fcd) for 0 < σcp ≤ 0.25 fcd (2b) 1.25 for 0.25 fcd < σcp ≤ 0.5 fcd (2c) 2.5 (1. 0 – σcp/fcd) for 0.5 fcd < σcp < 1.0 fcd 3

(2d) σcp =

mean compressive stress, measured positive, in the concrete due to the design axial force. 3

bw = minimum width of wall web

between tension and compression chords. z = inner lever arm, which is 35 taken as 0.8 lw; (lw is wall length). v1 = a

strength reduction factor for concrete cracked in shear. The recommended value is 7

0.6 [1. 0 – **fck/250] (fck in MPa).** 47

fcd = design value of concrete compressive strength (=fck /1.5) fck = **characteristic compressive cylinder strength of concrete at 28 days.** 3

θ = angle between concrete compression strut and wall **axis perpendicular to shear force.** Here, **the** values **of** cot **θ** 7

and tan θ are taken as 1.0. ?

Diagonal tension failure of the web due to shear: 8

If αs = MEd/(VEd lw) ≥ 2.0, the shear strength

is given by , : , 28

= cot / (3)

where: VRd,s = design value of shear force which can be sustained by the yielding shear reinforcement. 20

Asw =

cross sectional area of shear reinforcement. 58

fywd = design yield strength of shear reinforcement. s = spacing of stirrups. 30

MEd =

design bending moment at the base of wall. VEd = design shear force. 29

If αs = MEd/(VEd lw) < 2.0, the shear strength

is given by : = , 28

+ 0.75h,h (4) where: VRd =

shear resistance of a member with shear reinforcement. VRd,c = design shear resistance of a member without shear reinforcement. 26

ρh = reinforcement ratio of

horizontal web reinforcement. fyd, h = design value of the yield strength of horizontal web reinforcement. bwo = width of 27

wall web. LABORATORY EXPERIMENTS Seven

high strength concrete (HSC) structural walls 59

were tested under vertical axial loading and in-plane cyclic lateral loading. 45

All specimens were expected to fail in shear either

by crushing of the web concrete or yielding of web reinforcement. The 21

parameters investigated include

height to length ratio (hw/lw) of walls, vertical and horizontal web reinforcement ratios (ρv and ρh), and the effect of the boundary elements or the flanges. 1

Specimens J1, J2, J3, and J4 had hw/lw of 1.0 whereas specimens J5, J6, and J7 had hw/lw of 2.0. All specimens were cast with flanges except for specimen J4 which had no flange. 6 Specimens J1 and J5 were cast with both ρv and ρh of 0.28%, which satisfies the minimum requirement of ACI 318 code [1] and Eurocode 8 [2]. In specimens J2 and J6, ρv was increased to 0.75% while ρh was kept at 0.28%. In specimens J3 and J7, ρh was increased to 0.75% while ρv was kept at 0.28%. In specimen J4, ρv and ρh were set to be the same as those in specimen J1, i.e. 0.28%,

in order to investigate the effect of the flanges. In all **the** 44

specimens, top and bottom beams were designed to be stiff and strong enough to resist loadings without any significant deformation or damage. Details of specimen dimensions and reinforcements are shown in Figures 1 and 2. Note that the dimensions and reinforcements for specimens J5, J6, and J7 were similar to those of specimens J1, J2, and J3, respectively except for their wall height which is 2000 mm (78.74 in.) instead of 1000 mm (39.37 in.) and the number of horizontal web reinforcement (eleven stirrups instead of six). Materials The concrete mix design was set to achieve cylinder

compressive strength of about 100 MPa (14500 psi). **The** 57

maximum size of coarse aggregate was 10 mm (0.39 in.).

Silica fume and ground granulated blast furnace slag (GGBS) were **used** 33

as mineral admixtures. Superplasticizer was also added

to enhance the workability of the concrete since **the** water **to** 46

binder ratio used was relatively low. The casting of the specimens was done vertically. The reinforcement bars used were all deformed bars with nominal diameter and yield strength listed in Table 1.

Test setup The typical **test setup is shown** in **Figure 3.** 39

Prestressing bars were used to clamp the bottom beam of the specimen to laboratory strong floor to prevent movement or overturning of the specimen. Restraining blocks were also put on both sides of the bottom beam to help prevent movement during testing. The axial load assembly consisted of a vertical loading 7 frame, a 2000 kN (450 kips) hydraulic jack, a 2000 kN (450 kips) load cell, one set of spreader beam subassembly. The lateral load assembly consisted of the reaction wall, two 1000 kN (225 kips) servo-controlled hydraulic actuators, and one set of loading beam subassembly. Each of the

hydraulic actuator was connected to the reaction **wall at** 9 one end and to the loading beam at the other end. The loading beam was attached to the top beam of the specimen through a hinge which was held in placed by four prestressing rods. As shown in Figure 3, the test setup simulated a cantilever RC structural wall that was fixed at the bottom and the loadings were

applied at the top of the wall. The static vertical **loading** 42

from the hydraulic jack was applied to simulate gravity loading whereas the cyclic lateral loading from the hydraulic actuators was applied to simulate earthquake loading. Instrumentation Displacements of each wall specimen

were measured using Linear Variable Displacement **Transformers (LVDTs). The** 6

in-plane lateral displacement of the top beam was measured at the center of the beam which was the point of resultant force from the hydraulic actuators. This data would be used to plot force-drift curve of the wall specimen. Moreover, the out-of-plane lateral displacement of the top beam, if any, was also monitored. The bottom beam was monitored against movement and uplift, if any. All LVDTs were attached to independent steel frames that were erected specifically to hold the LVDTs. Flexural deformations of the wall specimen were measured using a series of LVDTs attached to wall edges, along the wall height. The displacements from these LVDTs were then divided by their gauge lengths to obtain the strains at wall edges. From those strains, curvatures along wall height and, thus, the flexural deformations of the wall specimen could be obtained. Shear deformations of the wall web were measured using diagonally placed LVDTs that were attached to the wall web. The shear deformations could be estimated using the changes in diagonal lengths of the wall web. However, flexural deformation components 8 need to be excluded in order to better estimate shear deformations. Sliding shear deformation was measured using LVDTs that were attached to wall base. Complete LVDTs setup is illustrated in Figure 3.

Strains in the reinforcement bars **were measured using strain gauges** 9

that were installed on the reinforcement bars at certain locations. For vertical bars,

strain gauges were installed at three locations, i.e. 19

bottom, middle, and top of the wall. For horizontal bars,

strain gauges were also **installed at three locations, i.e.** 19

left end, middle, and right end of wall web. Test procedure First, the

axial load was applied gradually **using the hydraulic jack until the** 36 compressive stress in the wall specimen reached 5% of the concrete cylinder compressive strength. This ratio was selected to be within the possible range of axial load for RC structural walls in buildings. Structural walls are normally designed to carry an axial load of up to about 20% of their axial capacity or less. At service load, the axial load will be unfactored and during an earthquake, the axial load drops further as the occupants leave the buildings. The ratio of 5% above considers the capacity of the flanges as well and it is within the acceptable range of axial load during an earthquake. This

axial load was maintained constant throughout the test period. **The cyclic lateral load was** then **applied using** the **hydraulic** 25

actuators by displacement control. Each specimen

was subjected to the same loading history shown **in** Figure 4. **In each** 31

cycle, there were positive and negative drift amplitudes. The amplitudes were increased gradually in subsequent cycles until the specimen failed abruptly or until the lateral load that could be resisted by the specimen dropped to 70% of the peak value or lower. In either case, the peak lateral load was considered to be the failure load. At peak amplitudes in each cycle, crack patterns were marked to capture crack propagations. Displacements of the specimen and strains in the reinforcement bars were monitored and recorded throughout testing. 9 EXPERIMENTAL RESULTS AND DISCUSSIONS Despite clear individual differences in the wall behavior, the general behavior in terms of crack patterns, drift ratios, lateral deformations, and strains in steel bars for specimens J1, J2, and J3 are qualitatively similar to each other. Hence, their discussions can be represented by specimen J3. Similarly, the general behavior of specimens J5 and J6 can be qualitatively represented by specimen J5. The complete results are described in detail in Chandra and Teng [5]. Crack patterns and force-drift relationships The crack patterns are shown in Figure 5 with numerical notes, on the specimens and beside each photograph, explaining crack propagations during testing. The first numbers denote cycle numbers while positive or negative signs denote positive or negative direction. The respective drift ratios and lateral forces are given as well. The force versus drift ratio curves are shown in Figure 6, with notes on some significant stages during testing. The recorded maximum forces as well as their respective story drifts are shown as well. These values are also presented for all specimens in Table 2. For specimen J1, it was decided that the testing would be continued monotonically in the negative direction starting from the sixth cycle (story drift ratio of +0.40%) onwards until the specimen failed in order to avoid too much movement at the bottom of the specimen. Overall, the crack propagations were quite similar in all specimens (Figure 5), except in specimens J4 and J7. Normally, diagonal cracks started to occur in the web of each specimen as early as in the second cycle, and the number of cracks increased in the subsequent cycles. In the flanges, horizontal cracks occurred mostly at stirrup locations starting from the third cycle onwards. Failure of the specimen was sudden after the occurrence of vertical splitting cracks in the compression flange as well as

crushing of concrete at the bottom **of the compression** 51

flange (see Figure 5, notes 9- for specimen J3 and 11+ for specimen J5). For specimen J4, at the ninth cycle (last cycle), the drift ratio was supposed to be increased to +0.80%. However, at a drift ratio of +0. 70%, the applied lateral load for the 22

specimen had already dropped to almost half of its peak value (Figure 6, specimen J4). Thus, further positive drift increment was aborted to prevent severe strength degradation, and the testing was continued in the negative direction until failure. Diagonal cracks started to occur at the third cycle together with horizontal cracks at stirrup locations of edge column that was in tension (Figure 5). Failure of specimen occurred when the concrete at the bottom of edge column was crushed. Furthermore, web crushing was also spotted

and horizontal web reinforcement fracture was observed in the middle of wall 52

web where concrete spalled off. When compared to specimens J1 to J3 and J5 to J6, the crack patterns in specimen J7 were quite different. There was no single major diagonal crack as well as no vertical splitting cracks at the flanges in specimen J7 (Figure 5). This was likely due to the provision of more horizontal web reinforcement in the web that helped limit the diagonal crack width and hence delay the strength degradation to a later stage of loading or nearer to web crushing stage. From the force – drift ratio curves (Figure 6), it can be concluded that

all the specimens failed in brittle shear mode. The 48

lateral force dropped significantly after reaching its peak point for specimens with hw/lw of 1.0 (J1 to J4). However, specimens with hw/lw of 2.0 (J5 to J7) are slightly more ductile. Specimen J7 has a drift ratio at peak lateral load of about 1.17%, which can be due to the combination of higher hw/lw and higher horizontal web reinforcement ratio, ρh. The average strains in the web reinforcements in all specimens did not reach the yield strain (about 0.003) at failure. Only in specimen J4 did some of the flexural reinforcement in the flanges (not in the web) reach yield when the shear failure occurred. The 11 non-yielding of the reinforcement confirmed the brittle shear failure mode of the wall specimens. Thus, the overall

behavior of the seven walls tested is governed by shear and 41

they have low deformation capacity. Note that most of the specimens tested had similar shear strength

in the positive and negative directions, meaning that 24

the diagonal cracks that occurred due to loading in one direction did not affect the shear strength in the other direction as long as web crushing had not occurred. Lateral deformations and strains In general, it can be observed in Figure 7 that as the total wall drift ratio increases, the

contribution of the flexural deformation and shear deformation to the 15 total

wall drift ratio seems to depend on hw/lw of the walls. For wall specimens with hw/lw = 1.0 (specimens J1, J2, and J3), the

contribution of shear deformation **to the** total **wall** drift **was** as **significant** 53

as the

contribution of flexural deformation **to the total** wall drift throughout **the** 15

full range of lateral loads or drift ratios. For specimens with hw/lw = 2.0 (specimens J5, J6, and J7), the flexural deformation was the major contributor to the total wall drift at early stages of loading or before the formation of major diagonal cracks (total drift ratios lower than about 0.70%). At higher loads or higher drift ratios nearer failure, shear deformation became significantly more dominant. For specimen J4 (hw/lw = 1.0 but no flanges), the results seemed to be inconclusive but they could still be categorized to belong to specimens with hw/lw = 1.0. In specimen without flanges (specimen J4), the sliding shear deformations at higher drift ratios or near failure were significantly larger than those sliding shear deformations in similar specimens (hw/lw = 1.0) but with flanges (specimens J1, J2, and J3). This indicates that specimen without flanges is more susceptible to sliding shear failure as compared to those with flanges. Thus, flanges can certainly help to prevent sliding shear failure by providing stiff dowels. 12 The strain distributions in the vertical and horizontal reinforcements are presented in Figures 8 and 9. The strains plotted here were obtained from strain gauges located near major diagonal cracks in order to observe whether the steel bars had yielded during testing. They were also plotted for several drift ratios in either positive or negative loading direction. This was done in order to observe the strain values in the steel reinforcement bars starting from initial loading until the specimens failed. At initial loading stage (drift ratio of 0.10%), the strains in both vertical bars (Figure 8) and horizontal bars (Figure 9) were still low. At this stage, there were only minor cracks in the wall web and major diagonal cracks had not occurred yet. At later stages, the strain values in the steel bars increased with each increment in drift ratio. A sudden increase of strains was normally spotted together with the occurrence of a major diagonal crack at drift ratio of about 0.40% to 0.60%. The strains in those steel bars

continued to increase **until the maximum** lateral **load was reached.** 21

As can be seen in Figures 8 and 9, the strain distributions in the steel reinforcement bars across the length and height of the walls were irregular and they did not follow the typical flexural behavior. At the peak load, while yielding might occur in some reinforcement bars, more horizontal bars than vertical

bars reached the yield strain. The average strains **in the** 2

horizontal bars were higher than in the vertical bars. Barda, Hanson, and Corley [6] investigated low rise normal strength concrete (NSC) walls having hw/lw of 0.25, 0.5, and 1.0. They concluded that for RC walls having hw/lw of 0.25 and 0.5, the average strains in vertical bars were higher than those in horizontal bars; while for RC walls having hw/lw of 1.0, the average

strains in the vertical and horizontal bars **were approximately** 9

equal. Therefore, based on the authors' experimental results and those from Barda, Hanson, and Corley [6], it can be concluded that in RC walls having hw/lw of less than 1.0,

the vertical web reinforcement is more effective than the horizontal web reinforcement. 2

In RC walls 13 having hw/lw equal to or

greater than 1.0, the horizontal web reinforcement is more effective than **the** 2

vertical web reinforcement in resisting lateral forces. Effect of height to length ratio, web reinforcement, and flanges on shear strength Specimen properties and experimental results, such as maximum lateral forces (Vmax) and their respective drifts, experimental wall shear strength (Vexp), and average shear stress are listed in Table 2. Vexp was the maximum lateral force (either positive or negative) before the first shear strength degradation was observed. An average shear stress was calculated by dividing Vexp by the area of wall web (Aw, which is bw x lw). The effects of hw/lw, ρv and ρh, and the presence of flanges to wall shear strengths are discussed below in terms of normalized average shear stress (Table 2, column 9). The effect of hw/lw was investigated by comparing similar specimens but differed only in hw/lw, i.e. specimen J1 against J5, specimen J2 against J6, and specimen J3 against J7. From the data presented in Table 2 and the envelopes of hysteretic curves shown in Figure 10, it can be seen that those walls having lower hw/lw (1.0 rather than 2.0 in this experiment) exhibit greater shear strength. The normalized average shear stresses (Table 2, column 9) of walls having hw/lw of 1.0 are between 1.6-2.0 times of those of walls having hw/lw of 2.0. Barda, Hanson, and Corley [6] found that increasing hw/lw from 0.5 to 1.0 reduced wall shear strength by 20%. The current authors found that increasing hw/lw from 1.0 to 2.0 reduced wall shear strength by about 40%-50%. Therefore, it can be concluded that, for RC wall having hw/lw ranging from 0.5 to 2.0, every increment of 0.5 in hw/lw reduces wall shear strength by 20%. This is valid for normal to very high strength concrete walls. The effect of web reinforcement was investigated by comparing similar specimens that varied only in web reinforcement ratios, i.e. specimen J1 against J2 and J3, and specimen J5 against J6 and J7. For walls having hw/lw of 1.0 (J1, J2, and J3),

both vertical and horizontal 14 **web** reinforcement contributed positively **to the shear strength of** RC **walls.** 1

Increasing ρh and ρv individually (from 0.28% to 0.75%) resulted in increments of 16.81% and 8.40%, respectively, in normalized average shear stresses. This means

that the horizontal **web reinforcement is more effective than the** vertical **web reinforcement** 2

in RC walls having hw/lw of 1.0. In walls having hw/lw of 2.0 (J5, J6, and J7), the contributions of the

vertical and horizontal web reinforcements to shear strength 1

are even more significant compared to those in walls having hw/lw of 1.0. Increasing ρv and ρh individually (from 0.28% to 0.75%) resulted in increments in normalized average shear stresses of 25.42% and 44.07%, respectively. From the above discussion,

it can be concluded that the horizontal **web** (wall) **reinforcement is more effective than the** vertical **web reinforcement. The** 2

effectiveness of horizontal web reinforcement increases from hw/lw of 1.0 up to hw/lw of 2.0. Note, however,

that the vertical web reinforcement is also **effective in** increasing **the** 2

wall shear strength. Based on this experiment and experiments from literature [4, 6], the following conclusion can be made. The

vertical web reinforcement is more effective than the horizontal web reinforcement 2

in RC walls having hw/lw less

than 1.0, while **the horizontal web reinforcement is more effective** 2

in walls having hw/lw equal to or greater than 1.0. Building codes (ACI 318 and Eurocode 8) [1, 2], however, do not consider the contribution of vertical web reinforcement. They prefer to treat it as an extra safety measure. Indeed, the codes seem to be very conservative for walls having both vertical and horizontal web reinforcements (see discussion below). The effect of flanges on shear strength can be seen by comparing specimen J1 (with flanges) and J4 (without flanges) (see Table 2). The presence of large flanges can increase significantly the normalized average shear stress, by 41.67% in this case. This increment is quite significant since the amount of flexural reinforcement and web reinforcements in both 15 specimens are similar. Obviously, specimen J1 with larger compression zone area failed at a significantly higher lateral force than specimen J4. This finding is in agreement with experiment conducted by Corley, Fiorato, and Oesterle [7] on normal strength concrete walls. The size of the flanges determines how much contribution can be provided by the flanges through the available dowel action. COMPARISON WITH BUILDING

CODES In

this study, the methods recommended by the ACI 318- 14 [1] and 4

Eurocode 8 [2] are used to calculate the shear strengths of RC

walls. Experimental wall shear strengths obtained from this study as well as those from literature [4, 6-14] were 1

used to verify the accuracies of the ACI and Eurocode provisions. A total of 84 specimens failing in shear were selected for comparisons, after checking that the specimens satisfied the design requirements of the

ACI 318 and Eurocode 8. The strength of the 11

concrete was not capped. All specimens were provided with web reinforcements, and most of them had flanges. The summary of the comparisons was presented statistically (see Table 3) in terms of experimental wall shear strength (Vexp) normalized by nominal wall shear strength (Vn) from ACI 318 and Eurocode 8. The normalized Vexp/Vn values were plotted against

concrete compressive strength to see the variation of the 56

predictions with respect to

concrete compressive strength (Figure 11). The predictions of the 16

building code provisions [1, 2] seem to be quite conservative with average values of Vexp/Vn of 1.43 for ACI 318 and 2.13 for Eurocode 8. However, their safety factors are not uniform over certain ranges of concrete strengths or wall height to length ratios hw/lw. Eurocode 8 method leads to the predictions of Vexp/Vn

with a minimum value of 1. 21, a maximum value of 7

5.30, and

coefficient of variation of 0. 35. The ACI 318 method gives 50

a minimum value of Vexp/Vn of 0.67, a maximum value of 3.05, and coefficient of variation of 0.38. Figure 11 shows that the ACI 318 predictions may not be conservative 16 enough for lower strength concrete walls (below 60 MPa (8700 psi)). On the other hand, the Eurocode 8 [2] predictions are overly conservative for almost all range of concrete strengths. The Vexp/Vn as

calculated using the ACI 318 and Eurocode 8 11

for HSC walls tested

in this study are listed in Table 2. 11

The non-uniform safety factor of the ACI method can be contributed by the fact that it neglects

the contribution of vertical wall reinforcement to shear strength. 16

The inaccuracy of the Eurocode 8 appears clearly for walls with higher hw/lw. According to Eurocode 8, for walls with moment to shear ratio (equivalent to hw/lw) equal to or more than

2.0, the overall shear strength is determined by the 49

horizontal web reinforcement, while the vertical web reinforcement as well as the so-called concrete contribution are neglected. The neglect of the contribution of vertical web reinforcement (by both ACI 318 and Eurocode 8) and concrete contribution (by Eurocode 8) should have made the ACI 318 and Eurocode 8 methods very conservative. However, the ratio of the applied moment to axial force and the actual contribution of the

vertical and horizontal web reinforcements, as well as the effect of higher concrete 54

strength and hw/lw may play more significant role than expected. Another factor is the dowel action from the boundary element or flanges. It has been found in this study that

the presence of large flanges could significantly increase the shear strength of RC walls. 1

CONCLUSIONS Seven

high strength concrete (HSC) walls, having compressive strength of about 100 MPa (14500 psi), were tested under cyclic lateral loading to investigate their shear strength and shear behavior. The parameters included were height to length ratio of the walls, vertical and horizontal web 1 reinforcement ratios, and the effects of boundary elements in the form of flanges. Based on the results

of this study, the following conclusions can be made: 43

17 1. Six of seven specimens tested in this study have similar shear strengths

in the positive and negative directions. This means that the 24

diagonal cracks that occurred in one direction of loading did not affect the shear strength in the other direction as long as web crushing had not occurred. 2. For wall specimens with hw/lw =

1.0, the contribution of shear deformation to the total wall drift was 34

as significant as the

contribution of flexural deformation to the total wall drift throughout the 15

full range of lateral loads or drift ratios. For specimens with hw/lw = 2.0, the flexural deformation was the major contributor to the total wall drift at early stages of loading or before the formation of major diagonal cracks (drift ratios lower than about 0.70%). At higher loads or higher drift ratios nearer failure, shear deformation became significantly more dominant. 3. Height to length

ratio has significant effect on the shear strength of 22

RC walls. For RC walls having hw/lw ranging from 0.5 to 2.0, every increment of 0.5 in hw/lw reduces the shear strength by approximately 20%. 4. The

vertical web reinforcement is more effective than the horizontal web reinforcement 2

in RC walls having hw/lw less

than 1.0, while the horizontal web reinforcement is more effective 2

in walls having hw/lw equal to or greater than 1.0. 5. The

presence of flanges can **significantly** increase **the shear strength of** RC **wall** 1

failing in web crushing mode due to larger compression area provided by the flanges and through dowel action. 6. ACI 318 predictions may not be conservative enough for lower to normal strength concrete walls (f'c < 60 MPa (8700 psi)) while Eurocode 8 predictions are overly conservative for almost all cases. In general, the accuracies of code

predictions of shear strength can be enhanced **by the** inclusions of **the** 16

contribution of vertical web reinforcement and the dowel action provided by the boundary elements or flanges. 18 ACKNOWLEDGMENTS This research is part of the large Competitive Research Program "Underwater Infrastructure and Underwater City of the Future" funded by the National Research Foundation (NRF) of Singapore. The authors are very grateful for the funding. Support by

Nanyang Technological University, Singapore, through the **School of Civil and Environmental Engineering** 32

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TABLES AND FIGURES List of Tables: Table 1 – Properties of steel bars Table 2 – Specimen properties and experimental results Table 3 – Statistical summary of Vexp/Vn List of Figures: Fig. 1 – Specimens J1-J4 dimensions (Specimens J5-J7 have the same details except that the heights of the wall web are 2000 mm (78.74 in.)). Fig. 2 – Reinforcement details for specimens J1-J4 (Specimens J5-J7 have the same details except that the number of horizontal web reinforcement are eleven pairs). Fig. 3 – Overall test setup and LVDTs setup. Fig. 4 – Loading history. Fig. 5 – Crack pattern at end of test of each specimen. Fig. 6 – Force-drift ratio curves of specimens. Fig. 7 – Contribution of wall deformation components (flexural, shear, and sliding shear) to total drift. Fig. 8 – Strains in vertical bars near major diagonal cracks at various drift ratios. Fig. 9 – Strains in horizontal bars near major diagonal cracks at various drift ratios. Fig. 10 – Envelope curves of specimens. Fig. 11 – Vexp/Vn plotted against concrete compressive strength. Table 1–Properties of steel bars Bar ID (1) Diameter, mm (in.) (2) Yield strength, MPa (ksi) (3) D6 5.94 (0.23) 610 (88.47) D10 9.77 (0.38) 578 (83.83) D10' 9.88 (0.39) 617 (89.49) D13 12.82 (0.50) 592 (85.86) D16 15.72 (0.62) 630 (91.37) D20 19.81 (0.78) 591 (85.72) 24 1 Table 2–Specimen properties and experimental results Wall ID (1) f'c, MPa (ksi) (2) hw/ lw (3) ρ v (4) ρ h (5) Vmax, kN (kips) (6) Drift at Vmax (%) (7) Vexp, kN (kips) (8) Vexp/ [Aw√f'c], MPa (psi) (9) Vexp/Vn (ACI 318) (10) Vexp/Vn (EC 8) (11) J1 103.3 (14.98) 1.0 0.0028 0.0028 +892.29 (+200.59) -1209.60 (-271.93) +0.39 -0.79 1209.60 (271.93) 1.19 (14.28) 2.85 3.25 J2 96.8 (14.04) 1.0 0.0075 0.0028 +1264.75 (+284.33) -1270.82 (-285.69) +0.80 -0.68 1270.82 (285.69) 1.29 (15.48) 3.05 3.48 J3 110.7 (16.06) 1.0 0.0028 0.0075 +1402.76 (+315.35) -1458.85 (-327.96) +0.79 -0.76 1458.85 (327.96) 1.39 (16.68) 2.09 2.36 J4 93.5 (13.56) 1.0 0.0028 0.0028 +810.74 (+182.26) -826.12 (-185.72) +0.54 -0.73 810.74 (182.26) 0.84 (10.08) 1.97 2.35 J5 103.3 (14.98) 2.0 0.0028 0.0028 +595.76 (+133.93) -556.97 (-125.21) +0.70 -0.70 595.76 (133.93) 0.59 (7.08) 1.73 4.36 J6 96.8 (14.04) 2.0 0.0075 0.0028 +724.14 (+162.79) -673.00 (-151.30) +0.80 -0.71 724.14 (162.79) 0.74 (8.88) 2.14 5.30 J7 110.7 (16.06) 2.0 0.0028 0.0075 +894.77 (+201.15) -854.02 (-191.99) +1.17 -0.99 894.77 (201.15) 0.85 (10.20) 1.46 2.58 2 3 Table 3–Statistical summary of Vexp/Vn Statistical parameters (1) ACI 318 (2) EC 8 (3) Minimum value 0.67 1.21 Maximum value 3.05 5.30 Average value 1.43 2.13 Standard deviation 0.54 0.74 Coefficient of variation 0.38 0.35 4 Note: The values listed in Table 3 are for a total of 84 walls, including the authors' 5 specimens. 6 Fig. 1– Specimens J1-J4 (Specimens J5-J7 have the same details except that the heights of the wall web are 2000 mm (78.74 in.)). Note: 5x2D6@200 is five pairs (a total of ten) bars having diameter of 6 mm at 200 mm spacing 1 Fig. 2–Reinforcement details for specimens J1-J4 (Specimens J5-J7 have the same details 2 except that the number of horizontal web reinforcement are eleven pairs). 3 4 5 Fig. 3–Overall test setup and LVDTs setup. 6 2.00 1.50 Drift (%) 1.00 0.50 0.00 -0.50 -1.00 -1.50 -2.00 7 Loading History 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 Cycle Number 8 Fig. 4–Loading history. 1 Vertical splitting cracks 3 Vertical splitting cracks Note: 1 kN = 0.22 kips 4 Negative direction is from left to right. 2+;+0.10%;+275kN indicates second cycle in 5 positive direction, drift ratio, and lateral force at respective drift ratio. 6 Fig. 5–Crack pattern at end of test of each specimen. 7 Yielding of flexural reinforcement First crack First crack Web crushing Yielding of horizontal web reinforcement Web crushing Yielding of horizontal web reinforcement 1 Yielding of horizontal web Yielding of web reinforcement reinforcement First crack First crack Crushing of boundary element Web crushing Wide crack opening 3 Note: 1 kN = 0.22 kips 4 The dots indicate points of maximum forces recorded and their respective drifts. Fig. 6–Force-drift ratio curves of specimens. 6 Sliding shear Sliding shear Sliding shear Sliding shear Shear Shear Shear Shear Flexural Flexural Flexural Flexural 1 Sliding shear Sliding shear Sliding shear Sliding shear Shear Shear Shear Shear Flexural Flexural Flexural Flexural 2 3 Fig. 7–Contribution of wall deformation components (flexural, shear, and sliding shear) to 4 total drift. 6 8 Yield strain Yield strain Yield strain Yield strain 1 Left End Left End Right End Right End Yield strain Yield strain Yield strain Yield strain 2 Right End Left End Right End Left End 3 Note: 1 mm = 0.04 in. Fig. 8–Strains in vertical bars near major diagonal cracks at various drift ratios. 5 Top Top Bottom Bottom Yield strain Yield strain 1 Top Top Bottom Yield strain Bottom Yield strain Note: 1 mm = 0.04 in. 4 Fig. 9–Strains in horizontal bars near major diagonal cracks at various drift ratios. J3 J2 J1 J4 J6 J5 J5 J6 J1 J7 J4 J2 J3 1 2 Note: 1 kN = 0.22 kips 3 Fig. 10–Envelope curves of specimens. J7 J1-J4:

hw/lw = 1.0 J5-J7: hw/lw = 2.0 4 4.00 ACI 318 Trendline 4.00 EC 8 Trendline V exp/Vn 3.00 2.00 V exp/Vn 3.00 2.00 1.00 1.00 5 0.00 0.00 0 30 60 90 120 150 0 30 f'c (MPa) 60 90 120 150 f'c (MPa) 6 Note: 1 MPa = 145.04 psi. 7 Fig. 11–Vexp/Vn plotted against concrete compressive strength. 8 9 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 1 2 3 1 2 3 2 2 5 5 7 4 2 3 5 3 5 10 19 20 21 22 23 24 25 26 27 28 29 30 31