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SHEAR RESISTANCE OF LIGHTWEIGHT SELF -CONSOLIDATING CONCRETE BEAMS

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ABSTRACT

This paper presents the shear behavior of lightweight self-consolidating concrete (LWSCC) beams without shear reinforcement compared to those made with normal weight self-consolidating concrete (SCC). The variables in this experimental and Code based study was shear span to depth ratio, concrete types and longitudinal reinforcement. The performance of LWSCC was compared with normal SCC beams based on load-deformation response, stress-strain development, and shear strength and failure modes. LWSCC beams showed lower post-cracking shear resistance and the shear strength of LWSCC/SCC beams increased with the decrease of shear span to depth ratio. LWSCC beams showed higher number of cracks and wider crack width at failure than their SCC counterparts. American, Canadian and British Codes were conservative in predicting shear strength of LWSCC beams.

1. INTRODUCTION

One of the latest innovations in self-consolidating concrete (SCC) technology is lightweight SCC (LWSCC) (Okamura and Ouchi 2003). For over 100 years, structural lightweight concrete (LWC) has been widely used as a building component (Hossain 2004a-b; Hossain 1997). LWC may be produced by using either natural lightweight aggregates such as pumice, scoria, diatomite and palm oil clinker or with artificial lightweight aggregates such as expanded clay, shale, slate, perlite, vermiculite and blast-furnace slag (ACI 211.2 1981; Topcu 1997; Bai et al. 2004; Hossain and Lachemi 2007; Hossain et al. 2011; Hossain 2004a-b, 2009a-b; Curcio et al.1998).

Using lightweight aggregates in concrete has several advantages including lower thermal connectivity, maximized heat and sound insulation properties due to air voids. Furthermore, it is reported that reducing the dead load of a building by using lightweight concrete could lead to a considerable decrease in the cross-section of steel-reinforced columns, beams, plates and foundations, reducing the need for steel reinforcement and leading to increased cost savings (Hossain 2004a-b; Topcu 1997; Mor 1993).

Despite all advantages associated with the use of SCC in structures, its use is limited sometimes because of its high self-weight compared to other construction materials. In this regard, the development of new types of high performance concretes, such as lightweight self-consolidating concrete responds to some of the urgent needs of the construction industry (Bentur et al. 2001; Kiliç et al. 2003; Aïtcin 1998). The development of SCC offers also limitless advantages in terms of reduction in the labor cost, better compaction and finish-ability in confined and restricted areas where compaction is difficult and faster construction completion.

LWSCC combines the favorable properties of LWC and SCC. These LWC advantages can be greatly utilized by incorporating lightweight aggregates in SCC mix design. Provided that the strength, mechanical and durability characteristics are comparable to normal weight SCC, LWSCC can be prompted as a new generation of high performance concrete in construction. More recently, Lotfy et al. (2014, 2015a-b) and Hossain & Anwar (2015)

developed LWSCC mixtures with furnace slag (FS), expanded clay (EC), expanded shale (ESH) aggregates and volcanic materials through comprehensive investigation on fresh state (slump flow diameter, V-funnel flow time, J-ring flow diameter, J-ring height difference, L-box ratio, filling capacity, density and sieve segregation resistance), mechanical (compressive/flexural/split tensile/ bond strength) and durability (freeze-thaw, chloride permeability, drying shrinkage, water sorptivity, electrical resistivity, corrosion and acid resistance) properties.

This specific studies (a timely initiative) concentrating on the shear resistance of LWSCC beams can contribute significantly to the application of LWSCC technology in the construction industry. This paper presents the shear behaviour of LWSCC beams without shear reinforcement compared to their SCC counterparts based on test results as well as performance of Code based equations in predicting the shear resistance.

2. EXPERIMENTAL PROGRAM

The experimental program was designed to evaluate shear behavior of LWSCC beams and estimate concrete contribution to overall shear resistance (V_c). Total of six shear beams without shear reinforcement were cast and tested. All LWSCC and SCC beams were designed only for adequate flexural reinforcements without shear reinforcement. SCC beams were similar to the LWSCC beams and served as control specimens.

2.1 Geometric Descriptions

LWSCC and SCC beams had different height/depth (h) of 150, 200, and 300 mm while the width (b) was kept constant at 100 mm. The total length of all the beams was at 1100 mm with an effective span of 800 mm. The shear span (a) to effective depth (d) ratio was kept between 1.05 and 2.14 to ensure the shear failure. Geometric dimensions and reinforcement details of the experimental beams are summarized in Table 1 and shown in Figures 1 and 2. The beam code was denoted by concrete type, total beam height. For example, LWSCC beam having a total height of 150 mm is coded as: LWSCC-150
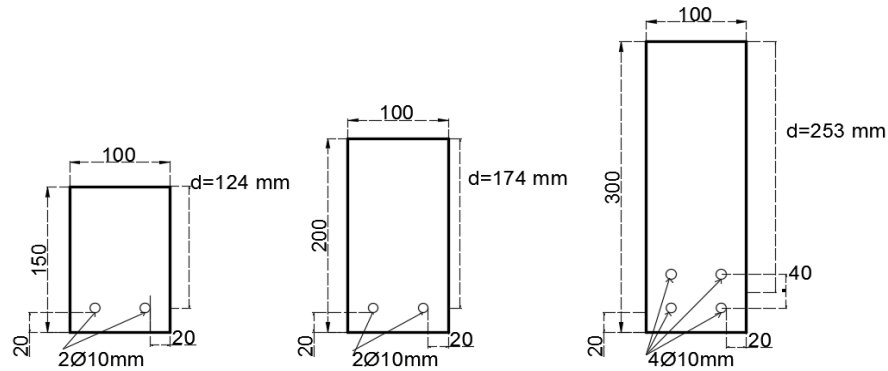

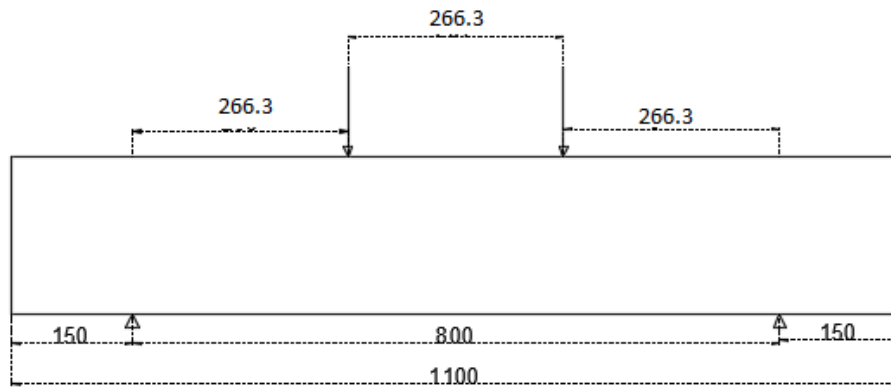Figure 1: Beam cross-sections (dimensions in mm)

Figure 2: Beams showing four point loading (dimensions in mm)

Table 1: Beam Geometry and reinforcement configuration

Beam code	Effective Depth (d) mm	Total height/depth (h) mm	Shear span (a) to depth (d) ratio a/d	Flexural reinforcement ratio, ρ (=$100A_s/bd$)* %
LWSCC-150	124	150	2.14	1.6
LWSCC-200	174	200	1.53	1.15
LWSCC-300	253	300	1.05	1.57
SCC-150	124	150	2.14	1.6
SCC-200	174	200	1.53	1.15
SCC-300	253	300	1.05	1.57

*Beams had a clear cover of 20mm and 10mm diameter deformed bar was used as flexural reinforcement

2.2 Materials

Two types of concretes namely LWSCC and SCC were used in this study. Mix designs of LWSCC and SCC are presented in Table 2. CSA Type 10 or the ASTM Type 1 normal Portland cement with specific gravity of 3.17 was used. Class F fly ash according to CSA classification with a calcium oxide (CaO) content of less than 8%, a typical bulk density value of 540 ~ 860 kg/m^3 and specific gravity of 2.6 was used. A dry-densified silica fume (SF) powder was used to develop a cohesive but flowable mixture to enhance segregation resistance.

Lightweight blast furnace slag aggregates were used to develop the LWSCC mixtures. The slag aggregates having nominal size of 10 mm and 4.75 mm were used as coarse and fine aggregates. Gradation and physical properties of fine and coarse lightweight furnace slag aggregate satisfied the ASTM C330 (2009) requirement. Normal weight crushed gravel with a nominal size of 10 mm and sand were used as coarse and fine aggregate, respectively for SCC.

During the preparation of LWSCC, coarse and fine slag aggregates were pre-soaked for a minimum of 72 hours due to higher water absorption. Excess water in the aggregate was drained out without losing the fine particles. Saturated surface dry aggregate was used for the mixing and proper water adjustment was made according to the water absorption of the aggregate and the moisture content of the aggregate at the time of mixing.

Table 2: Concrete Mixture proportions (by weight of cement)

Material	LWSCC	Material	SCC
Type 10 Cement	1	Type 10 Cement	1
Fly ash	0.156	Crushed gravels -Coarse aggregate	1.59
Silica fume	0.094	Sand - Fine aggregate	2.31
HRWRA	0.89%	Water	0.41
Water	0.438	HRWRA (high range water reducing admixture)	0.63%
Slag coarse aggregate	1.18		
Slag fine aggregate	1.67		

2.3 Casting and Instrumentation

Immediately after the LWSCC/SCC mixing, beam specimens were cast in wooden molds without any vibration and segregation. Total volume of 100 liter of concrete (one batch) was necessary to cast the three shear beams. One of LWSCC and SCC were required to cast the 6 shear beams. Visual observation show that LWSCC properly filled the forms with ease of movement and same was the case for normal weight SCC.

Control specimens in the form cylinders were also cast to determine strength of concrete and cured under similar conditions as beam specimens until testing. Beam moulds were removed after 24 hours of casting and the beams were moisture cured for five days and then air cured until 28 days of testing. The compressive strength of LWSCC and SCC were determined from 100 x 200 mm control cylinders for each batch according to ASTM C39 (2003).

2.4 Testing Procedures

All specimens were tested as simply supported beam under four-point loading condition. LVDT (Linear variable displacement transducer) was fixed at mid span to measure the central deflection. A hydraulic jack was used to apply the load incrementally with 5kN for each increment and the load was kept constant for some minutes after each increment to observe the crack pattern. All strain gauges, load and LVDT were connected to a computer control data acquisition systems. The initiation and development of shear and flexural cracks and cracking loads at various stages were recorded. Test also provided information on the overall behavior of the beam including development of crack, crack patterns, load transfer mechanism and failure modes.

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Load Deflection Behaviour

Experimental load deflection curves for the tested SCC/LWSCC shear beams are shown in Figure: 3.The slope changes of the curve indicates a reduction in the stiffness of the beam. The initial straight line segment of the curve shows that prior to flexural cracking, stiffness of the beam remained constant. Crack development during loading is indicated by abrupt changes (formation of kinks) in the load-deflection curves. After formation of inclined/diagonal crack, stiffness of the beams suddenly decreased in both LWSCC and SCC beams. When the load reached the ultimate shear capacity, a sudden brittle shear failure was occurred. Immediately after the shear failure, a significant reduction in the load carrying capacity was observed. The ultimate loads/shear capacities for SCC beams were higher than corresponding LWSCC beams as per Figure 3. On the other hand, LWSCC beams showed higher deflection evolution compared with their SCC counterparts.
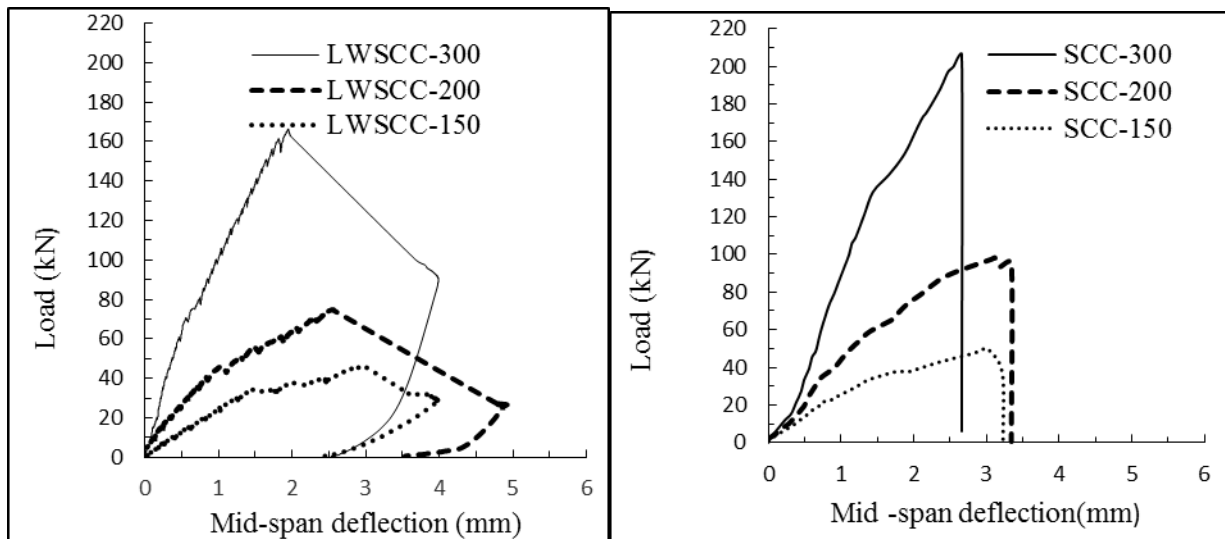

Figure 3: Load deflection response

3.2 Failure Mode and Cracking Behavior

During loading, fine vertical flexural cracks were formed within the mid span of all beams (zero shear region). With increase of load, new flexural cracks were formed within the zero shear regions and in the shear span prior to the formation of first shear cracks. The inclined shear crack initially formed near the support, as expected. With further increase in load, diagonal shear cracks propagated towards the loading point of the beam with the formation of additional shear and flexural cracks along the beam. Finally sudden shear failure was occurred immediately after dominant diagonal shear cracks formed within one or two side of the shear span as shown in Figure 4. The volume of sound at shear failure was identifiably louder in high depth beams than the small depth ones. Table 3 indicates the experimental summary for shear beams without shear reinforcement showing concrete compressive strength, failure modes, shear loads at first flexure/diagonal crack, deflection at first diagonal crack, peak shear load, peak load deflection and angle of diagonal crack.

Formation of the first flexural crack was observed at lower loads in LWSCC beams when compared to the SCC beams. This observation is an indication of lower bending/flexural strength of LWSCC. The angle of dominant diagonal crack was approximately within the range of 50-65 degree for LWSCC beams and 40-60 degree for SCC beams. Angle of diagonal shear crack tends to increase with the increasing of height of the LWSCC and SCC beams. Diagonal shear crack loads varied from 48.1 to 68% of ultimate loads for LWSCC beams and 51.8 to 69.5% of ultimate loads for SCC beams. LWSCC beams had about 14 to 17 cracks at failure and SCC beams had around 6 to 9 cracks. So LWSCC beams developed more crack than SCC beams at failure.

Table 3: Summary of experimental results

Beam code	a/d	f'c (MPa)	Failure pattern	Shear at first flexure V_{fl} (kN)	Deflection at first diagonal crack D_c (mm)	Shear at first diagonal crack V_c (kN)	Failure shear V_u (kN)	Deflection at peak shear load D_u (mm)	Diagonal crack angle (Degree)
LWSCC-150	2.14	33.5	Shear	3.0	0.7	16.0	23.5	2.8	50
LWSCC-200	1.53	33.5	Shear	5.0	0.9	22.5	37.5	2.5	55
LWSCC-300	1.05	33.5	Shear	10.0	0.9	40.0	83.0	1.9	65
SCC-150	2.14	53.0	Shear	8.8	0.6	16.5	25.0	2.9	40
SCC-200	1.53	53.0	Shear	17.0	0.6	27.5	53.0	3.1	46
SCC-300	1.05	53.0	Shear	22.0	1.1	48.0	103.0	2.5	60

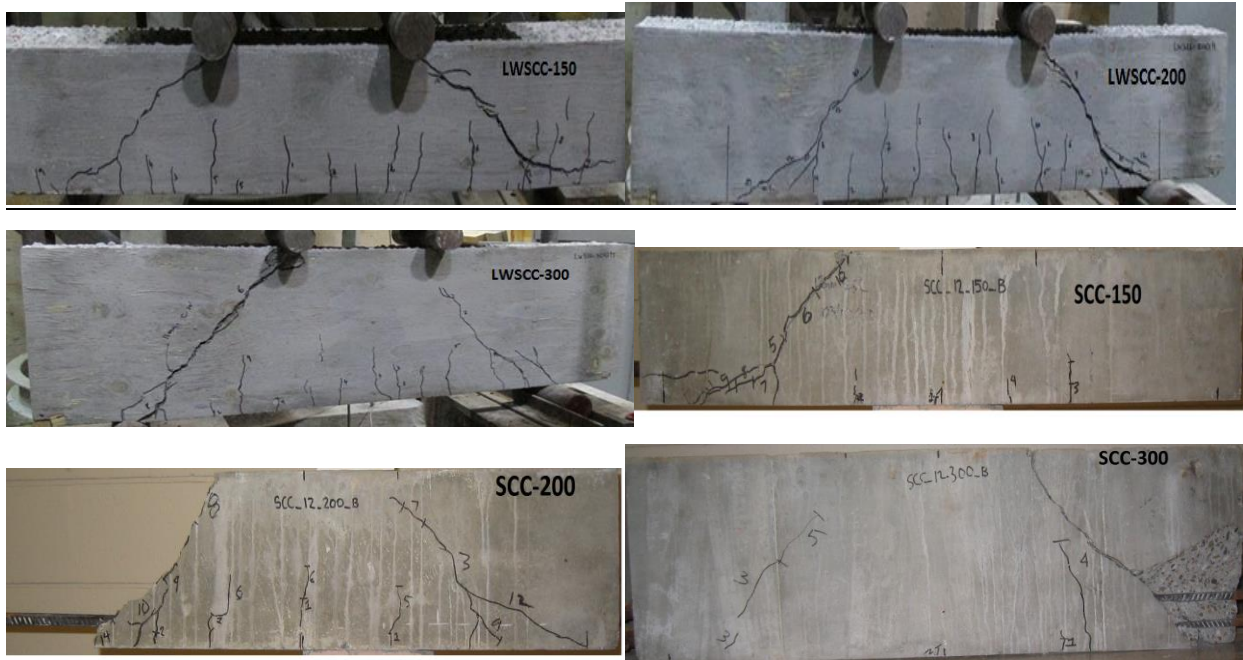a/d :shear span to effective depth ratio, f'c : concrete compressive strength

Figure 4: Failure modes of beams

3.3 Influence of the Shear Span to Depth Ratio (a/d) on Concrete Shear Resistance (V_c)

The influence of shear span to depth ratio (a/d) on the concrete shear resistance capacity (V_c) defined as the shear load at first diagonal crack of LWSCC and SCC beams were investigated. LWSCC and SCC beams had a compressive strength of 33.5 MPa and 53 MPa, respectively. Figure 5 shows the influence of a/d on the concrete shear resistance capacity of LWSCC and SCC beams. As expected, shear resistance capacity of LWSCC and SCC beams decreased with the increase of a/d. The shear resistance capacity of SCC beams was higher than

corresponding LWSCC beams. Shear resistance capacity difference between these two concretes increased with the decrease of a/d.
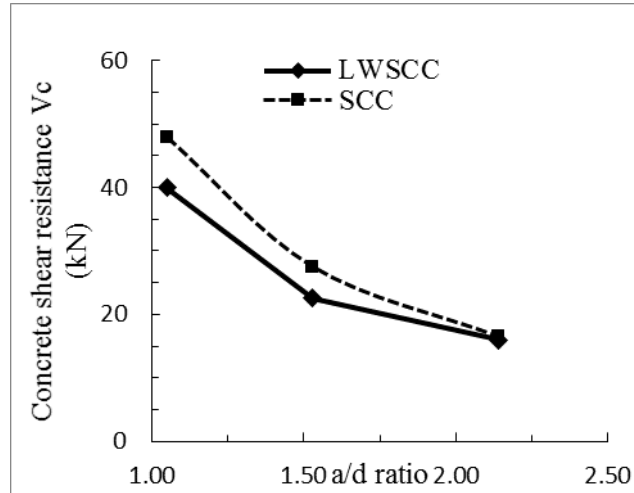

Figure 5: Influence of shear span to effective depth ratio on concrete shear resistance (V_c)

3.4 Post Cracking Shear Resistance, Ductility and Energy Absorption

Aggregate interlock mechanism and dowel action play significant roles in the increase of shear resistance from V_c (shear resistance at the formation of inclined crack) to V_u (ultimate shear resistance or peak load). In this study, the shear at the first diagonal crack is denoted as concrete shear resistance (V_c) and it was identified from the visual observation during the testing of LWSCC and SCC beams. The ultimate shear resistance (V_u) was identified from the maximum load (peak load) that a beam can carry before failure. To characterize the performance of LWSCC and SCC, it is important to analysis the post cracking shear resistance of concrete beams due to aggregate interlock and dowel action. Similar analysis was carried out by previous researchers, Lachimi et al. (2005) and Hassan et al. (2010), by introducing a shear resistance factor (SRF). SRF is defined as the ratio of the failure load to the load at the first diagonal crack (SRF = V_u/V_c).

To investigate and compare the post cracking shear resistance of LWSCC and SCC beams, the ultimate shear load and diagonal cracking shear load are normalized to account for the difference in compressive strength between LWSCC and SCC using Equations 1 and 2 to calculate normalized ultimate shear load (V_{nu}) and normalized inclined cracking shear load (V_{nc}). Since the shear strength is proportional to the square root of the compressive strength of concrete (f'_c) as per CSA A23.3 (2004) and ACI 318 code (2005) based equations, normalization was done accordingly. SRF values were calculated using Equation 3.

[1] $V_{nu} = \frac{V_u}{\sqrt{f_c}}$

[2] $V_{nc} = \frac{V_c}{\sqrt{f_c}}$

[3] $SRF = \frac{V_u}{V_c} = \frac{V_{nu}}{V_{nc}}$

The post cracking shear ductility was defined as the ratio of the deflection at failure load to the deflection at first diagonal crack load by previous researcher Hassan et.al (2010). In this study, ductility of the shear beam is also defined by the ductility factor (DF) as per Equation 4 where D_u and D_c are the deflection at first diagonal crack and peak/failure load, respectively as presented in Table 3.

[4] DF = D_u/D_c

Table 4: Shear resistance and ductility factor

Beam code	Shear span to depth ratio (a/d)	Concrete compressive strength (f'c)	Normalized inclined cracking shear load (V$_{nc}$)	Normalized ultimate shear load (V$_{nu}$)	Shear resistance factor (SRF)	Ductility factor (DF)	Energy absorption J/MPa$^{1/2}$
LWSCC-150	2.14	33.5	2.8	4.1	1.4	4.0	13.6
LWSCC-200	1.53	33.5	3.9	6.5	1.6	2.5	14.7
LWSCC-300	1.05	33.5	6.9	14.3	2.0	2.4	27.7
SCC-150	2.14	53	2.3	3.4	1.5	4.7	11.5
SCC-200	1.53	53	3.8	7.3	1.9	4.5	22.4
SCC-300	1.05	53	6.6	14.1	2.1	2.3	36.1

Normalized shear loads, shear resistance factor and ductility factor for the shear beams without shear reinforcement are shown in Table 4. Main portion of the shear is transferred through aggregate interlock mechanism and dowel action in the post-cracking stage. When considering the aggregate interlock mechanism, coarse aggregate content and it's quality affect the post-cracking stage shear transfer capacity. Table 4 show that SCC beams had a higher SRF than their LWSCC counterparts to weaker aggregate interlock mechanism in the LWSCC beams. It is also noted that SRF increased with the decrease of a/d for both SCC and LWSCC beams.

Shear ductility (defined by DF) of SCC beams was found to be higher than corresponding LWSCC beams except for 300 mm height beam. This can be attributed to the brittle nature of porous lightweight aggregate compared to normal weight aggregate as suggested by Gerritse (1981). Overall, shear ductility increased with the increase of a/d for both SCC and LWSCC beams (Table 4).

To investigate and compare the energy absorption of SCC and LWSCC beams, the shear load is normalized to accommodate for the difference in compressive strength between SCC and LWSCC. Equation 1 is used to normalize the shear loads. Normalized shear load - deflection curves was used. Energy absorption was calculated by area under the normalized shear deflection curve up to the post peak shear of 85 % of the ultimate shear load (V$_u$) and presented in Table 4.

Energy absorption capacity increased with the decrease of beam a/d for both SCC and LWSCC beams. That can be attributed to the louder sound at failure for the higher depth beams compared to smaller depth ones. SCC beams exhibited higher energy absorption capacity compared to LWSCC beams for higher depth beam (height of 200 and 300 mm) or d/b or lower a/d. But higher a/d (height of 150 mm), energy absorption capacity was found higher for LWSCC beam than SCC beam.

4. COMPARISON OF EXISTING ANALYTICAL MODELS

An accepted rational physical method of shear resistance does not yet exist due to the complex nature of the shear failure mechanism in reinforced concrete beams therefore most design codes use empirical equations to calculate the shear capacity of the reinforced concrete beams. The formation of diagonal tension cracks is taken by design codes to be the ultimate shear capacity of the beams without shear reinforcement.

ACI 318-05 (2005) presents the following basic equation 5 (in SI units) for the shear resistance of concrete (V$_c$).

$$[5] \quad V_c = \left(\sqrt{f'_c} + 120\, p_w \frac{V_u\, d}{M_u} \right) \frac{b_w d}{7} \leq 0.3\sqrt{f'_c}\, b_w$$

Where b$_w$ is the width of the cross-section, d is the effective depth, p$_w$ is flexural reinforcement ratio ,Vu and M$_u$ are the ultimate shear force and moment capacity of the section, respectively, f$_c$' is the cylinder compressive strength of concrete.

According to Canadian Code (CSA A23.3-04, 2004) based on modified compression field theory, V_c can be obtained from the following Equation 6 and The β shall be determined from Equations 7,8 and 9.

[6] $\quad V_C = \beta \sqrt{f_c'} \, b_w \, d_v$

[7] $\quad \beta = \dfrac{520}{[(1+1500\,\varepsilon_x)(1000+S_{ze})]}$

[8] $\quad S_{ze} = \dfrac{35 s_z}{15} + a_g \le 0.85 S_z$

[9] $\quad \varepsilon_X = \dfrac{\frac{M_f}{d_v}+V_f}{2 A_s E_s}$

where b is the width of the cross section, d is the effective shear depth which can be taken as the greater of 0.9 of the beam depth or 0.72 of the beam height, f_c' is the cylinder compressive strength -square root of the compressive strength should be less than 8 MPa. According to clause 11.3.4, ε_x is the longitudinal strain at mid-depth of the member due to factored loads, Mf is the factored moment at section, Vf is the factored shear force at section, Es is the modulus of elasticity of non-prestressed reinforcement, S_z and d_v represent crack spacing parameter dependent on crack control characteristics of longitudinal reinforcement and a_g is maximum size of aggregate in the concrete. For high-strength concrete with f_c' greater than 70 MPa, a_g shall be taken as zero.

According to British standards (BS8110-part1, 1997), V_c can be calculated from Equation 10.

[10] $\quad V_C = 0.79 \left(\dfrac{100 A_s}{bd}\right)^{\frac{1}{3}} \left(\dfrac{400}{d}\right)^{\frac{1}{4}} \left(\dfrac{f_{cu}}{25}\right)^{\frac{1}{3}}$

This Code limits the maximum allowable concrete compressive strength to 40 MPa with an alternative table used for values of compressive strength below 25 MPa depending only on the amount of longitudinal steel provided. In BS8110, b_v is the width of the cross section, d is the effective depth, f_{cu} is the cube compressive strength, A_s is the tension reinforcement area in mm^2. f_{cu} should be less than or equal to 40 MPa for calculation purpose only.

ACI 318-05 and CSAA23.3-04 shear strength equation use the cylinder's compressive strength, but BS8110 shear strength equation adopt cube's compressive strength in the shear strength calculation.

The ACI 318-05 (2005) code uses a reduction factor equal to 0.75 for all-lightweight concrete, 0.85 for sand lightweight concrete and 1.0 for normal weight concrete. CSA A23.3-04 (2004) code uses reduction factor equal to 0.75 for low density concrete (with an air dry density between 1850 and 2140 kg/m^3) and 1.0 for normal weight concrete (with an air dry density between 2150 and 2500 kg/m^3)

Table 5: Experiment and code based prediction

Beams	V_c -concrete shear resistance contribution				Ratio of experimental to Code Predicted shear		
	Experiment (Exp)	Code-based predictions			Exp/ACI	Exp/CSA	Exp/BS8110
		ACI	CSA	BS8110			
LWSCC-150	16.0	10.2	10.7	12.5	1.5	1.4	1.2
LWSCC-200	22.5	13.3	13.4	14.4	1.6	1.6	1.5
LWSCC-300	40.0	20.8	21.4	21.3	1.9	1.8	1.8
SCC-150	16.5	16.3	17.9	17.9	1.0	0.9	0.9
SCC-200	27.5	21.4	20.3	20.6	1.2	1.3	1.3
SCC-300	48.0	33.1	33.5	30.4	1.4	1.4	1.5

Shear resistances of LWSCC beams from current experiments and various code based predictions are compared in Table 5. It can be observed that all design codes were conservative in predicting the ultimate shear strength of LWSCC beams. ACI 318 provided the highest safety margin (ratio ranged between 1.5 and 1.9 for all tested

LWSCC beams compared to CSA A23.3 (ratio ranged between 1.4 and 1.8) and BS8110 (ratio ranged between 1.2 and 1.8). CSA-A23.3 and BS8110 codes estimated the shear capacity of SCC-200 and SCC-300 beams reasonably but overestimated SCC-150 beam. In both type of beams (LWSCC and SCC), all codes predictions were conservative and conservativeness increased with the increase of beam depth or decrease in shear span to depth ratio (a/d). This can be attributed to the small a/d of experimental beams which lead strut and tie mechanisms (rather than beam shear) to govern the shear strength especially as no bearing plates were used at the loading and support points (as a part of the research objective). Conservativeness of code predictions was expected since strut-and-tie mechanisms result in higher experimental shear strengths.

It should be noted that conservativeness was higher of LWSCC beams compared to normal weight SCC beams even after the use of reduction factors specified in the Codes. However, the predicted shear capacity differences for similar beams between the Codes were not significant. For the calculation of lightweight concrete shear capacity, ACI 318 and CSA A23.3 Codes use the reduction factor of 0.75 but BS8110 use the reduction factor of 0.8, Therefore, BS8110 predictions were higher than those of CSA A23.3 and ACI 318.

5. CONCLUSIONS

The following conclusions are drawn from the study:

1. The shear resistance capacity of SCC beams was higher than their LWSCC counterparts. Shear resistance capacity difference between these two types of concrete beams increased with the decrease of shear span to depth ratio (a/d).
2. SCC beams had higher post-cracking shear resistance (defined by shear resistance factor 'SRF) than their LWSCC counterparts. SRF increased with the decrease of a/d for both SCC and LWSCC beams. This was attributed to the weaker aggregate interlock mechanism resulting from partially fractured coarse aggregate along the failure surface, higher number of cracks and wider final crack width at failure than normal weight SCC beams.
3. All structural design codes found to be conservatively predicted the shear capacity of the LWSCC beams. For all design codes, experimental to predicted shear strength ratios were high and these ratios ranged from 1.2 to 1.9 for LWSCC beams. This was attributed to the small a/d ratio of experimental beams which lead to strut and tie mechanisms causing higher shear strength.
4. In both type of beams (LWSCC and SCC), all code predictions were conservative except SCC-150 and conservativeness increased with the increase of beam depth or decrease in shear span to depth ratio (a/d). It should be noted that overestimation was higher of LWSCC beams compared to normal weight SCC beams even after the use of reduction factors specified in the Codes. However, the predicted shear capacity differences for similar beams between the Codes were not significant.
5. Overall, current reduction factors suggested by the Codes for lightweight concrete can be increased for the prediction of shear resistance of LWSCC beams. This is reasonable considering the lower volume of weak lightweight aggregate (hence higher volume of strong paste) in LWSCC compared to traditional lightweight concrete.

ACKNOWLEDGEMENTS

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