5-2014

Shear Capacity of Prestressed Lightweight Self-Consolidating Concrete

Micah Alan Wyssmann
University of Arkansas, Fayetteville

This thesis is approved.

Thesis Advisor:

Dr. Micah Hale

Thesis Committee:

Dr. Findlay Edwards

Dr. Kirk Grimmelsman

SHEAR CAPACITY OF PRESTRESSED LIGHTWEIGHT SELF-CONSOLIDATING CONCRETE BEAMS

An Undergraduate Honors College Thesis

in the

Department of Civil Engineering
College of Engineering
University of Arkansas
Fayetteville, AR

by

Micah Alan Wyssmann

ABSTRACT

In prestressed members, self-consolidating concrete (SCC) has in some cases exhibited lower shear capacity than conventional concrete, which has been attributed to decreased aggregate interlock. However, little data is currently available to assess whether use of lightweight aggregate in prestressed SCC beams has further impact on aggregate interlock and shear strength. This study measured the shear capacity of six prestressed beams made from three different concrete mixtures: lightweight self-consolidating concrete (LWSCC) with expanded shale aggregate, LWSCC with expanded clay aggregate, and control normal weight SCC with limestone aggregate. Predicted shear capacities were determined based on current ACI Building Code and compared with measured shear strengths. ACI predictions underestimated shear capacity for all tests and were more conservative for the control normal weight SCC beams than for the LWSCC beams.

INTRODUCTION

Presented in this paper are the results of a research project that examined the use of lightweight self-consolidating concrete (LWSCC) in prestressed beams. Lightweight concrete by definition has a lower unit weight than that of conventional concrete, which reduces dead load and thereby decreases overall material requirements. Self-consolidating concrete (SCC) was developed in response to a need for improved construction and products. The use of SCC eliminates the need for vibration during placement, which is achieved through both a high deformability and a resistance to segregation when in its fresh state. These advantages compared to conventional concrete have been the major driving factor for the prestressed concrete research that has focused on LWSCC and SCC. However, there is currently little data on the shear strength of prestressed LWSCC beams, and likewise there is little understanding regarding their mechanical properties. This experimental study was conducted to determine the shear capacity of prestressed LWSCC beams and to compare these values both to the predicted shear capacity obtained using current ACI Building Code (ACI Committee 318, 2011), referred to hereafter as ACI 318, and to the results for control normal weight SCC beams.

BACKGROUND

The shear strength of prestressed concrete members cast with normal weight self-consolidating concrete (SCC) has been studied on multiple occasions, but the conclusions vary regarding whether or not SCC has lower shear strength than conventional concrete (CC). Multiple studies have displayed similar shear strengths for both SCC and CC in prestressed beams (Burgueño and Bendert 2007; Hamilton and Labonte 2005; Naito et al. 2006), while others have shown slightly lower shear strengths for SCC compared to CC (Hegger et al. 2007; Hassan et al. 2010). Aggregate interlock has long been known as a significant contributing factor to shear strength (Taylor 1970). Khayat and Mitchell (2009) concluded that an observed decrease in shear strength of SCC relative to CC is likely the result of reduced aggregate interlock in SCC because of its lower coarse aggregate content. In addition, aggregate type also affects aggregate interlock in both SCC and CC mixtures, though the effects of aggregate type on shear strength and on other mixture properties of SCC are still not fully understood (Kim et al. 2010).

ACI 318 design provisions use a modification factor (λ) when calculating shear strength to account for the decreased mechanical properties of lightweight concrete (LWC) relative to normal weight mixtures of similar compressive strength. Yang and Ashour (2011) assessed the aggregate interlock properties of LWC in deep beams, which either have an overall span-to-depth ratio equal to or less than 4.0 or have a concentrated load placed within a distance of two times the overall height from the face of a support. Their study determined that the ACI modification factor for LWC led to conservative shear capacity predictions for some but not all concrete mix designs. Dymond et al. (2010) determined the shear strength of a single prestressed LWSCC bridge girder and compared this to the predicted strength from the AASHTO general method (2002), which was found to be conservative. The AASHTO (2002) method analyzed for their study differs from the ACI 318 detailed method only by minor variations in the application of the LWC modification factor. Though cases of decreased shear strength for both LWC and SCC have been demonstrated, little data is available other than that provided by Dymond et al. (2010) to assess the shear capacity of prestressed LWSCC members.

MATERIALS AND METHODS

Test Specimens

Six rectangular beams with dimensions of 6.5 in. x 12 in. x 18 ft. were cast for the non-destructive study of prestress losses at the University of Arkansas (Bymaster 2012). These beams were later used for this study of shear strength. The sample set consisted of two beams made from each of three mixture designs: LWSCC with expanded clay aggregate, LWSCC with expanded shale aggregate, and control normal weight SCC with limestone aggregate. Table 1 gives the nominal maximum size, the specific gravity (SG) and the absorption capacity (AC) for each of the coarse aggregate types. The concrete mixture proportions are given in Table 2 and were developed during another previous research project at the University of Arkansas (Floyd 2012). Two Grade 270 low-relaxation prestressing steel strands of 0.6 in. diameter were placed at 10 in. of depth from the top fiber of each beam. Additional Grade 60 steel reinforcement included two ¾ in. (No. 6) bars placed 2 in. from the top fiber and ¼ in. (No. 2) smooth rebar shear stirrups. Spacing of the steel reinforcement in the beam design is shown in Figure 1.

Table 1. Aggregate Properties

Aggregate Type	Limestone	Clay	Shale
Nom. Max. Size (in.)	3/8	1/2	3/4
SG (ASTM C127)	2.68	1.24	1.40
SG (ACI 211.2)	NA	1.25	1.41
AC (ASTM C127) (%)	0.38	16.3	15.0
AC (ACI 211.2) (%)	NA	15.0	12.9

Table 2. Concrete Mixture Proportions

Material	Clay	Shale	Limestone
Cement [a.] (lb/yd^3)	808	832	825
Fly Ash (lb/yd^3)	142	147	0
Coarse Aggregate (lb/yd^3)	649	703	1392
Fine Aggregate (lb/yd^3)	1242	1270	1403
Water (lb/yd^3)	333	333	330
w/cm	0.35	0.34	0.40
HRWR ADVA 575 (fl oz/cwt)	9.5 - 11	8 - 10	5 - 6
HRWR ADVA 405 or 408 (fl oz/cwt)	26	25	15
Calculated unit wt. (lb/ft^3)	117.6	121.7	146.3

[a.] Lightweight aggregate mixes used Type III cement; normal weight used Type I

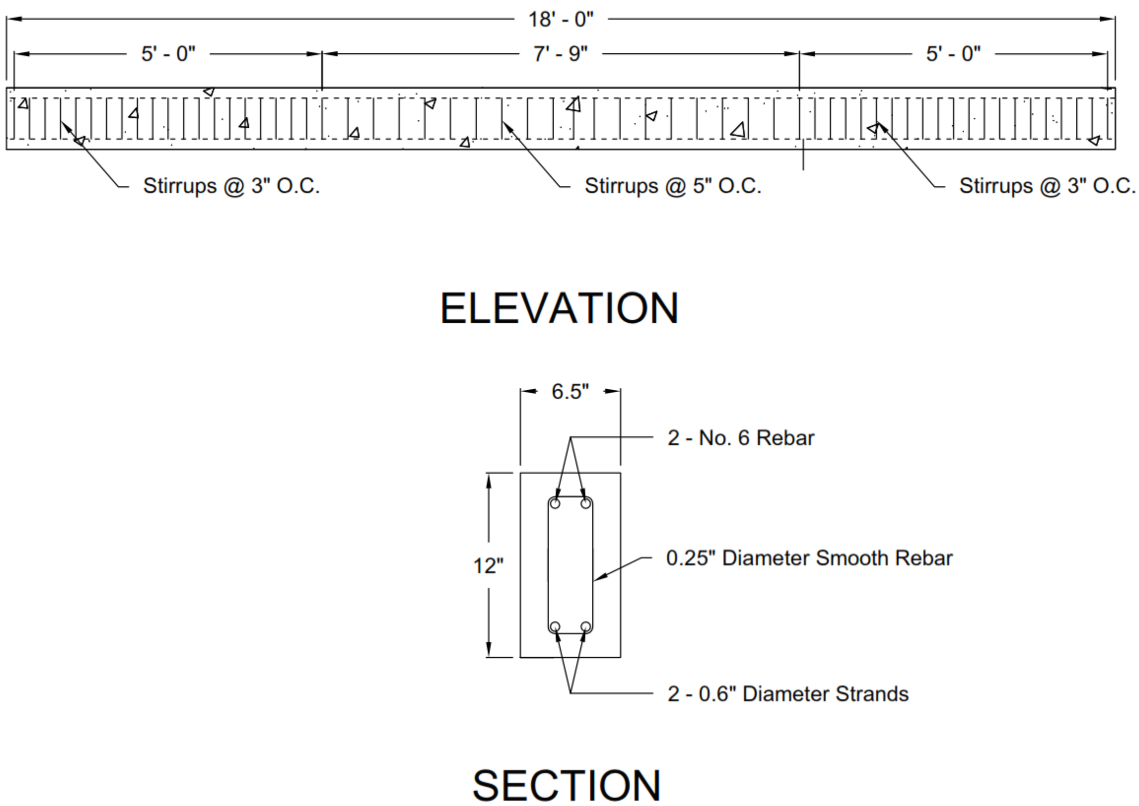

Figure 1. Beam Reinforcement Design

For the study of prestress losses, each beam was loaded approximately five months after casting with about 65 lb./ft. of load (Bymaster 2012). This load met the ACI 318 design requirements of a Class T beam, which has tensile stress in the extreme fiber that is between 7.5 and 12 times the square root of the compressive strength. Class T beam loading is common in bridge girders and did not affect the shear capacity of the beams. These loads were left in place on each beam for a minimum of six months and were removed before shear testing. The average compressive strengths of each beam were obtained from companion cylinders at the time of shear testing and are given in Table 3.

Table 3. Average Compressive Strengths

Beam ID	Compressive Strength (psi)
Shale 1	7,950
Shale 2	7,420
Clay 3	8,670
Clay 4	7,500
Limestone 1	12,070
Limestone 2	12,090

Experimental Setup and Procedure

Each beam end was tested using a simply supported setup with a 7 ft. span between the pin and the roller. The beam was loaded at a single point, the configuration for which is shown in Figure 2. The test ends of each beam are identified as either the live end (end nearest the prestress strand tensioning apparatus) or the dead end (end opposite the prestress strand tensioning apparatus). Floyd (2012) measured transfer lengths for each of these concrete mixtures, none of which exceeded 26 in. by 28 days of age; in addition, it was noted that the transfer length readings had stabilized by approximately 14 days. Therefore, the pin was set no closer than 30 in. from the beam end to ensure that shearing occurred after the transfer length. For the LWSCC tests, the pin was located 30 in. from the test end and the load was an additional 18 in. from the pin, or 48 in. from the beam end. For the normal weight SCC tests, the pin was located 33 in. from the test end and the load was an additional 15 in. from the pin, or 48 in. from the beam end. All tests had approximately 9.5 ft. of overhang past the roller to ensure that testing on one end would not affect the shear capacity of the other. Because of these loading configurations, all tests are considered deep beams, as defined previously.

Load was applied to each beam by a manual hydraulic jack that was attached to an anchored load frame. Load was transferred from the jack onto the beam by a 6 in. steel block that was modeled as a point load. Loading was conducted in 5,000 lb. increments until cracking was observed. Once cracking occurred, 2,500 lb. load increments were used until failure. At each load increment, shear and tension cracks were marked (if necessary) and manual deflection readings were taken.

During testing, two linear voltage differential transformers (LVDTs) were attached to the prestressing strands on the beam end being tested. These LVDTs monitored strand movement during loading. Additionally, a linear encoder was used to measure deflection at the point load location throughout testing. Load, strand slip and deflection data were obtained via instrumentation and data points were taken and recorded continuously by a computer program. For the limestone beams, which failed at higher point loads, the recording capacity of the programmed load variable was exceeded at approximately 86,000 lbs. and manual load readings were taken from the hydraulic jack pressure gauge at loads exceeding this capacity. Figure 2 displays the testing setup with instrumentation in place.

Figure 2. Beam Test Setup

RESULTS

Two modes of shear failure were observed: web-shear failure and flexure-shear failure. Web-shear cracking is characterized by diagonal cracking within the web of a beam and is the result of a shear induced principal tension force. Web-shear cracks occurred suddenly between the pin and the load for all tests, though not always before flexural cracks. All beams that failed in web-shear did so along this initial observed web-shear crack. Flexure-shear cracking results from a combination of shear and moment. The beams that failed in flexure-shear did so as a result of flexure cracks that began in the longer shear span for the loading condition (between the load and the roller) and then turned to diagonal shear cracks that angled toward the load. Flexure-shear failures were also characterized by some amount of moment induced crushing in the top fiber, but web-shear failures did not exhibit this crushing. Figure 3 shows the Clay 4 dead end test, which displays a typical web-shear failure observed in this study. Figure 4 shows the Limestone 1 live end test, which is a typical flexure-shear failure that exhibits crushing in the top fiber.

Figure 3. Clay 4 Dead End Test – Typical Web-Shear Failure

Figure 4. Limestone 1 Live End Test – Typical Flexure-Shear Failure

No significant strand slip was measured during any tests and only one notable slip occurred, which was measured at 0.05 in. during the Limestone 1 dead end test. This indicates that the loading had no overall effect on the strand bond within the transfer zone. Load-versus-deflection graphs were created for all the LWSCC beam tests. Since the capacities of the normal weight SCC beams exceeded the capacity of the program, there are no load versus deflection graphs for these beams. These graphs confirmed shear failure because minimal deflection was experienced after the peak load and before failure. Flexure-shear failure generally exhibits slightly more deflection at a constant load before failure than web-shear failure does, which was reflected in these graphs. The load versus deflection graph for the Shale 1 live end test is a representative sample for all LWSCC beam tests and is given in Figure 5.

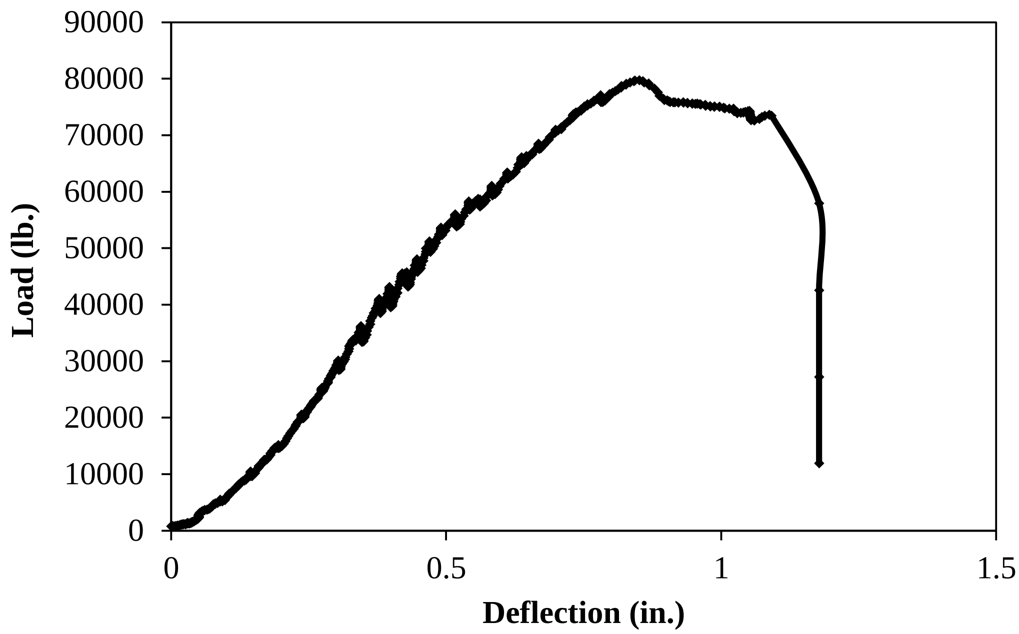

Figure 5. Shale 1 Live End Test – Load-vs.-Deflection

Table 4 outlines the pertinent data for all tests, including the shear span-to-depth ratio (*a/d*). Laskar et al. (2010) investigated the *a/d* ratio for over a hundred prestressed girders available in literature and concluded that this ratio had a direct affect on the concrete contribution to shear strength. For the normal weight SCC beams, the load was shifted closer to the pin support to ensure failure for these relatively stronger beams. This yielded a slightly smaller *a/d* ratio for these tests.

Table 4. Test Data

Beam Test	Max Load (lb.)	Max Deflection (in.)	a/d	Failure Type
Shale 1 – D	79,300	1.3	1.8	Flexure-Shear
Shale 1 – L	79,700	1.1	1.8	Web-Shear
Shale 2 – D	75,700	0.9	1.8	Web-Shear
Shale 2 – L	78,800	1.2	1.8	Web-Shear
Clay 3 – D	75,300	0.8	1.8	Flexure-Shear
Clay 3 – L	77,200	1.9	1.8	Flexure-Shear
Clay 4 – D	72,600	0.9	1.8	Web-Shear
Clay 4 – L	77,200	1.0	1.8	Web-Shear
Limestone 1 – D	100,000	0.9	1.5	Flexure-Shear
Limestone 1 – L	113,000	1.0	1.5	Flexure-Shear
Limestone 2 – D	111,000	1.0	1.5	Web-Shear
Limestone 2 – L	110,000	1.1	1.5	Web-Shear

Note: D = dead end of specimen; L = live end of specimen

Shear Capacity Comparison

The maximum point load for each beam was used to determine the maximum shear stress that occurred. These values were compared with the predicted shear capacity computed from ACI 318 provisions, which is based on the shear resistance contributions of both the concrete and the steel shear reinforcement as given by:

$$V_n = V_c + V_s \qquad \textbf{Equation 1}$$

where:
V_n = nominal shear strength (lb.)
V_c = nominal shear strength provided by concrete (lb.)
V_s = nominal shear strength provided by shear reinforcement (lb.)

ACI 318 provides both a general method and a detailed method for computing the concrete contribution to shear capacity in prestressed members. The general method computes the nominal shear strength of concrete (V_c), while the detailed method states that the lesser of the nominal concrete flexure-shear strength (V_{ci}) and the nominal concrete web-shear strength (V_{cw}) should be taken. The design method employed to create these beams for the study of prestress losses did not require either a factored moment (M_u or M_{max}) or a factored shear force (V_u or V_i), which are two necessary components for computing the general concrete shear strength and the flexure-shear strength discussed above. Thus, this study was limited to use of the web-shear strength (V_{cw}) to calculate the concrete contribution to shear strength as given by:

$$\boldsymbol{V_{cw}} = \left(3.5\lambda\sqrt{\boldsymbol{f_c'}} + 0.3\boldsymbol{f_{pc}}\right)\boldsymbol{b_w d_p} + \boldsymbol{V_p}$$ **Equation 2**

where:
V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web (lb.)
λ = modification factor reflecting the reduced mechanical properties of lightweight concrete
$\sqrt{f_c'}$ = square root of specified compressive strength of concrete
f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads (psi)
b_w = web width (in.)
d_p = distance from extreme compression fiber to centroid of prestressing steel (in.)
V_p = vertical component of effective prestress force at section (lb.) (ACI Committee 318 2011)

The steel contribution to overall shear capacity is given by:

$$\boldsymbol{V_s = A_v f_{yt} d / s}$$ **Equation 3**

where:
V_s = nominal shear strength provided by shear reinforcement (lb.)
A_v = area of shear reinforcement within spacing s (in.^{2})
f_{yt} = specified yield strength f_y of transverse reinforcement (psi)
d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement (in.)
s = center-to-center spacing of transverse reinforcement (in.) (ACI Committee 318 2011)

The calculated web-shear strengths were compared with the maximum and minimum concrete shear strengths outlined in the ACI 318 general method, which are given by $5\lambda\sqrt{\boldsymbol{f_c'}}\boldsymbol{b_w d}$ and $2\lambda\sqrt{\boldsymbol{f_c'}}\boldsymbol{b_w d}$, respectively. After this analysis, it was determined that the calculated values for V_{cw} exceeded the maximum concrete contribution in all cases. While this maximum value gives the true prediction of concrete shear strength according to ACI 318 provisions, the overall shear capacity was also calculated using V_{cw} to assess the accuracy of this equation. A comparison of the measured and calculated shear capacities is given in Table 5.

Table 5. Shear Capacity Comparison

Beam Test Identification	ACI Shear Capacity (From V_{cw}) (lb.)	ACI Shear Capacity (From Max V_c) (lb.)	Measured Shear Strength (lb.)	V_{test}/V_{code} (From V_{cw})	V_{test}/V_{code} (From Max V_c)
Shale 1 – D	55,100	44,600	62,100	1.13	1.39
Shale 1 – L			62,500	1.13	1.40
Shale 2 – D	54,600	43,800	59,300	1.09	1.35
Shale 2 – L			61,700	1.13	1.41
Clay 3 – D	55,900	45,700	59,000	1.05	1.29
Clay 3 – L			60,500	1.08	1.32
Clay 4 – D	54,600	43,900	56,900	1.04	1.30
Clay 4 – L			60,500	1.11	1.38
Limestone 1 – D	62,900	55,700	82,000	1.30	1.47
Limestone 1 – L			92,700	1.47	1.66
Limestone 2 – D	62,900	55,700	91,000	1.45	1.63
Limestone 2 – L			90,200	1.43	1.62

Note: D = dead end of specimen; L = live end of specimen

CONCLUSIONS

The goal of this study was to assess the shear capacity of prestressed LWSCC beams and compare the measured values with current ACI 318 provisions. The following conclusions and recommendations have been drawn from this research:

- Conservative estimates of shear capacity were provided for the LWSCC beams with a 29% - 41% margin of safety. Comparatively, conservative estimates were found for the control SCC beams with a 47% - 66% margin of safety.
- For web-shear failures only, use of V_{cw} provided conservative estimates of shear capacity with a 4% - 13% margin of safety for the LWSCC beams and a 43% - 45% margin of safety for the control SCC beams. Inclusion of flexure-shear failures yielded a 4% - 13% margin of safety for the LWSCC beams and a 40% - 47% margin of safety for the control SCC beams.

- ACI 318 predictions of shear capacity were adequate for the prestressed LWSCC beams studied. Estimates for LWSCC beams were less conservative than those given for control SCC beams, though the change in shear span-to-depth ratio may have influenced this result.
- Further research is needed to assess the accuracy of the ACI 318 general shear and flexure-shear equations to predict the nominal concrete shear strength in prestressed LWSCC beams. Additionally, investigation of aggregate interlock and its affect on the shear strength differences between LWSCC and normal weight SCC in prestressed beams is needed.

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