COMPARATIVE SHEAR BEHAVIOR OF AASHTO GIRDERS WITH A1035 AND A615 STIRRUPS

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ABSTRACT

An experimental program compared the shear behavior of prestressed, AASHTO Type I girders having ASTM A1035 high strength stirrups in one shear span and standard ASTM A615 stirrups in the other. The girders were prestressed with 0.6-in. diameter strand and cast from 10 ksi concrete; each had a 4 ft x 7.5 in, 5 ksi composite slab. A total of 4 specimens were tested. Two specimens were designed to meet the requirements of the AASHTO LRFD Specifications and had approximately the same stirrup strength on each shear span. Nearly equal strengths were achieved by using smaller stirrups on the A1035 end. Both specimens failed in flexure. The third specimen used the maximum stirrup spacing for the A615 stirrups and failed in shear on the A615 end. The final specimen was designed to fail in shear on the A1035 end and did. The study concluded that as long as the calculated stirrup strength, V_s, is approximately the same, there is no discernable difference in crack patterns, crack widths, or failure modes between the shear spans with A1035 stirrups and those with A615 stirrups, even though the A1035 stirrups are smaller and/or have greater spacing. For the specimens failing in shear, the failure load was approximately 70% greater than V_n calculated using the sectional (compression field) model of the AASHTO LRFD Specifications and was approximately 50% greater than that predicted by a fundamental plane sections approach.

Keywords: A1035 Steel, High Strength Steel, Prestressed Girders, Shear Strength

INTRODUCTION

Congestion is a common problem with precast, prestressed concrete beams, especially in regions near beam ends where shear stirrups and bursting stirrups tend to be closely spaced. The use of ASTM A1035 reinforcing steel could help alleviate this problem. A1035 reinforcing steel has a specified yield strength of 100 or 120 ksi, which is 67 or 100% larger than conventional ASTM A615, Grade 60 steel. In theory, stirrups made of A1035 steel could have twice the spacing (provided spacing limits are not violated) or have half the area of stirrups made of A615 steel. The question is whether A1035 stirrups with larger spacing and/or smaller areas perform the same as A615 with the same theoretical strength.

This paper details the testing of four, AASHTO Type I girders made with A615 stirrups in one shear span and A1035 stirrups in the other. The beams were designed so that, as nearly as possible, both shear spans had the same shear strength, calculated using the Modified Compression Field Theory (MCFT). While MCFT is the method used in the Sectional Model in the AASHTO LRFD Specifications[1], the procedure presented in AASHTO is simplified. In this study the capacity was calculated using the computer program RESPONSE 2000[2], which directly calculates shear strength using the MCFT.

PREVIOUS RESEARCH

Shear strength in a concrete beam is assumed to be made up of two parts: the contribution of the concrete web (V_c) and the contribution of the reinforcing stirrups (V_s). Efforts to find a method of accurately calculating total shear strength have had limited success. This challenge is largely due to the difficulty in calculating the strength of the web concrete, which is in a complex state of biaxial stress, and that the experimental data on shear strength show a large amount of scatter.

The current design methods have not been verified for members having high-strength steel reinforcement. The concern is that to fully engage high-strength stirrups requires higher stress levels; which may cause excessive cracking and/or excessive crack opening in the concrete, resulting in decrease of the concrete component of shear resistance.

Sumpter[3] tested beams having shear span/depth ratios of approximately 3 reinforced with either A615 or A1035 longitudinal and transverse steel. The stirrup spacings used were the minimum and maximum allowed, with an additional intermediate spacing between these limits. There was little difference between specimen behaviors at service loads, probably due to the low shear span/depth ratios. Members having A1035 shear reinforcement had a marginally greater shear capacity than specimens reinforced with A615 stirrups. Sumpter concluded that concrete dominated behavior, and that stress in the shear reinforcement in any specimen never exceeded 80 ksi. While this is beyond yield for the A615 stirrups, it was still within the elastic limit for the A1035 stirrups, so the A1035 stirrups never reached their full strength. A study by Florida DOT[4] drew similar conclusions with respect to the stress that may be developed in shear reinforcement.

Sumpter reported that shear crack widths at service loads were less than 0.016 in., regardless of the reinforcement grade or details. He did report smaller crack widths in comparable members having A1035 stirrups as opposed to those with A615 stirrups. He attributes this behavior to enhanced bond characteristics of A1035 steel, but offers no support of this assumption.

SPECIMEN DETAILS

This research program tested both prestressed and non-prestressed beams. Only the tests of the prestressed beams are described here; details of the tests of the non-prestressed beams can be found in Shahrooz, et al.[5] A total of four prestressed specimens, designated SP1 through SP4, were tested. All specimens were Type I AASHTO I-girders with a 10 ksi nominal concrete strength. A 7-in. thick, 4-ft wide deck, with a nominal strength of 5 ksi, was cast on top of each girder. The deck was made composite with the beam by extending the shear stirrups from the beam into the deck. The deck was reinforced with #4, A615 Grade 60 bars. Figure 1 shows details of each specimen and Table 1 summarizes details and nominal and measured concrete strengths.

In each specimen, A1035 stirrups were used in one shear span and A615 stirrups were used in the other. The A615 steel was Grade 60 and the A1035 was Grade 100, so the nominal yield strengths were taken as 60 and 100 ksi, respectively. Table 2 shows the measured properties of the steel. A1035 steel does not have a yield plateau, so three different definitions of yield strength are provided. The first value, stress at a strain of 0.0035, is the definition given in the AASHTO LRFD Specifications (5.8.2.8) for transverse reinforcement with yield strengths exceeding 75 ksi.

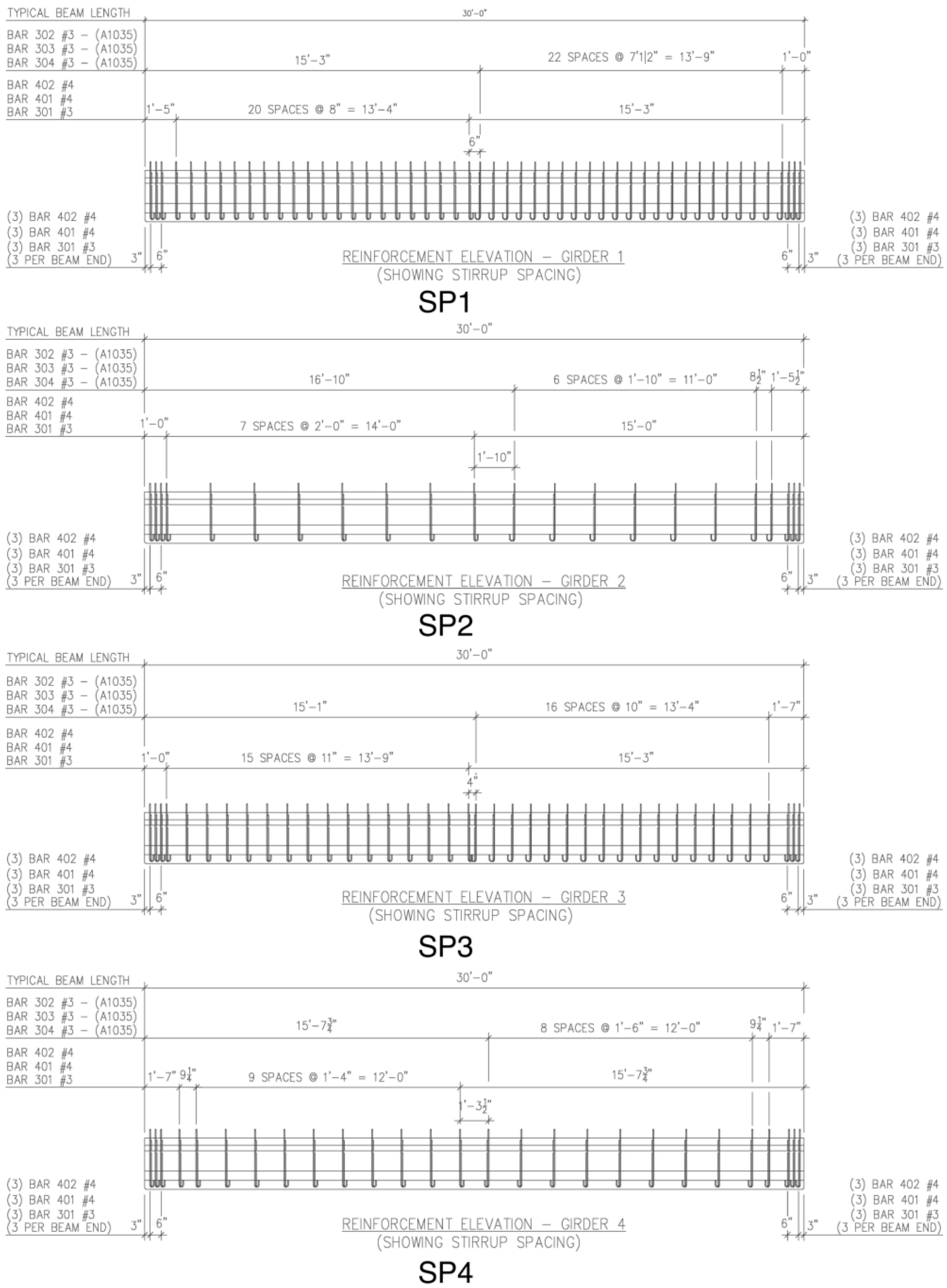
Specimens SP1 through SP3 were designed so that stirrup strength in each shear span was as nearly equal as possible.

$$V_s = \frac{A_v f_y d_v}{s} \cot \theta$$

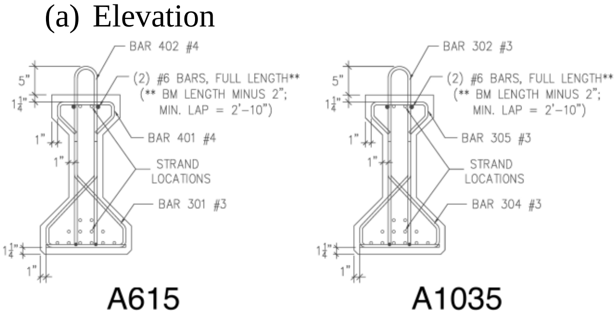
Where:

V_s = stirrup force (kips)
A_v = stirrup area crossing the shear crack (in.^{2})
f_y = stirrups yield stress, assumed as 60 ksi for A615 and 100 ksi for A1035
d_v = shear depth (in.). (Assumed the same for both shear spans.)
s = spacing of the stirrups (in.)
θ = angle of the shear crack (assumed the same for both shear spans)

Specimen SP4 was designed after specimens SP1 through SP3 were tested. Since none of the first three specimens failed in shear in the A1035 reinforced shear spans, SP4 was designed so that the A1035 reinforced shear span had a slightly lower strength than the A615 reinforced shear span in an attempt to force failure of the A1035 stirrups and investigate this behavior.



(a) Elevation



(b) Cross Section

Fig. 1 Specimen Details

All prestressing steel was 0.6-in. diameter, Grade 270, low relaxation strand. The flexural and shear capacity was calculated using the program RESPONSE 2000[2] and the amount of prestressing steel was selected such that the program predicted a shear failure would occur. In spite of this effort, two specimens still failed in flexure at applied loads well above their predicted shear strengths.

Table 1 Shear Specimens (AASHTO Type I Girders)

Specimen ID	Transverse Reinforcement	Girder f'_c (ksi)		Slab f'_c (ksi)		Design Criterion
		Design	Measured	Design	Measured	
SP1	#4 A615 @ 8 in. #3 A1035 @ 7.5 in.	10	11.9	5	7.2	As Needed to Resist V_u
SP2	#4 A615 @ 24 in. #3 A1035 @ 22 in.	10	12.4	5	9.9	A615 at Maximum. Allowable Spacing of 24 in.
SP3	#4 A615 @ 11 in. #3 A1035 @ 10 in.	10	13.1	5	10.1	As Needed to Resist V_u
SP4	#4 A615 @ 16 in. #3 A1035 @ 18 in.	10	10.5	5	6.3	Under-designed A1035

Table 2 Measured Properties of Transverse Reinforcement

Stirrup	Specimens	Rupture Strain	Calculated Modulus of Elasticity (ksi)	Ultimate Strength (ksi)	Yield Strength (ksi)		
					@ strain = 0.0035	@ strain = 0.0050	@ 0.2% offset
#4 A615	SP1-SP3	n.r.	27,596	105.4	86.3	88.2	88.2
#4 A615	SP4	n.r.	23,945	105.0	83.4	92.9	90.2
#3 A1035	SP4	0.070	27,740	164.1	93.0	117.2	131.9

n.r. Not reported.

No data are available for the #3 A1035 stirrups used in Specimen SP1, SP2, and SP3.

TESTING AND INSTRUMENTATION

The testing frame is shown in Figure 2. Load was applied at two points so that there was an 11-ft long zone of constant moment and two 93 in. long shear spans. The shear span-to-overall depth ratio for all specimens was 2.66. A pair of 300-kip hydraulic jacks was used to apply the load. Since the intended load points were not under the loading frame, a spreader beam with elastomeric bearing pads was used (Figure 2).

Bonded strain gages were attached to the prestressing strands (after initial tensioning) and to the shear stirrups. Figure 3 shows the positions of the strain gages on the longitudinal and transverse steel. Figure 4 shows additional strain gages bonded to the concrete surface. Deflection was measured using wire potentiometers. DEMEC targets were glued onto the shear span and were used in an attempt to measure strain and/or crack opening along and perpendicular to the shear struts. Finally, crack widths were measured using visual crack gages.

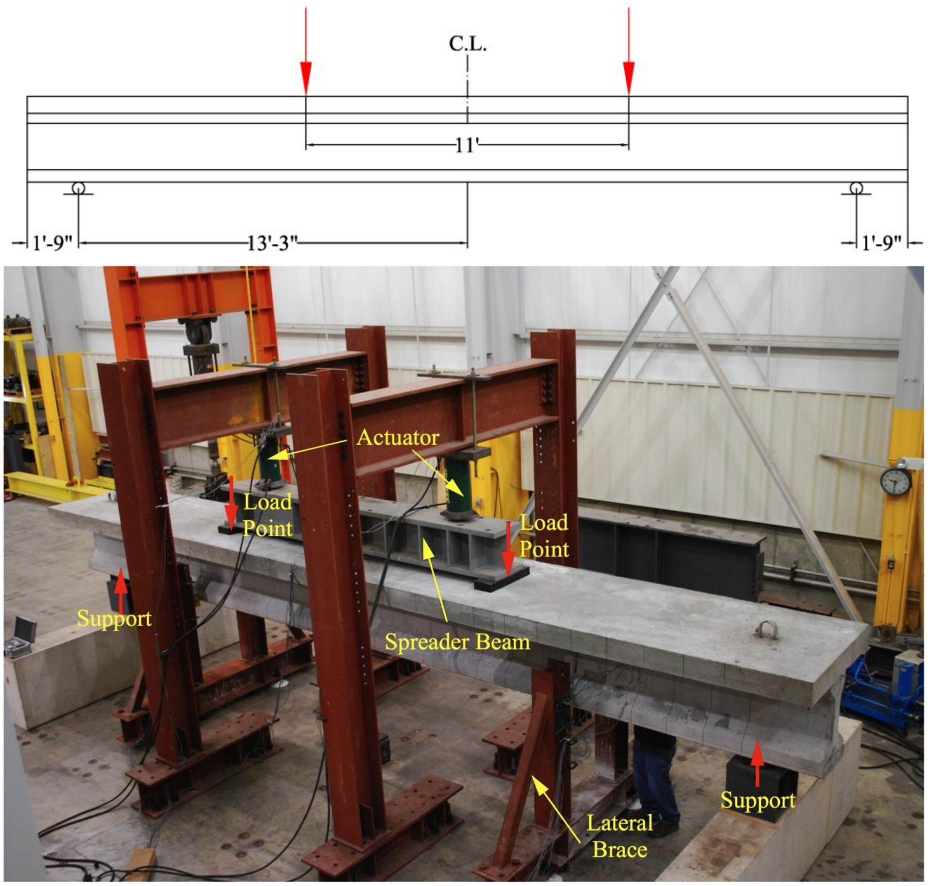


Fig. 2 Loading and Test Setup

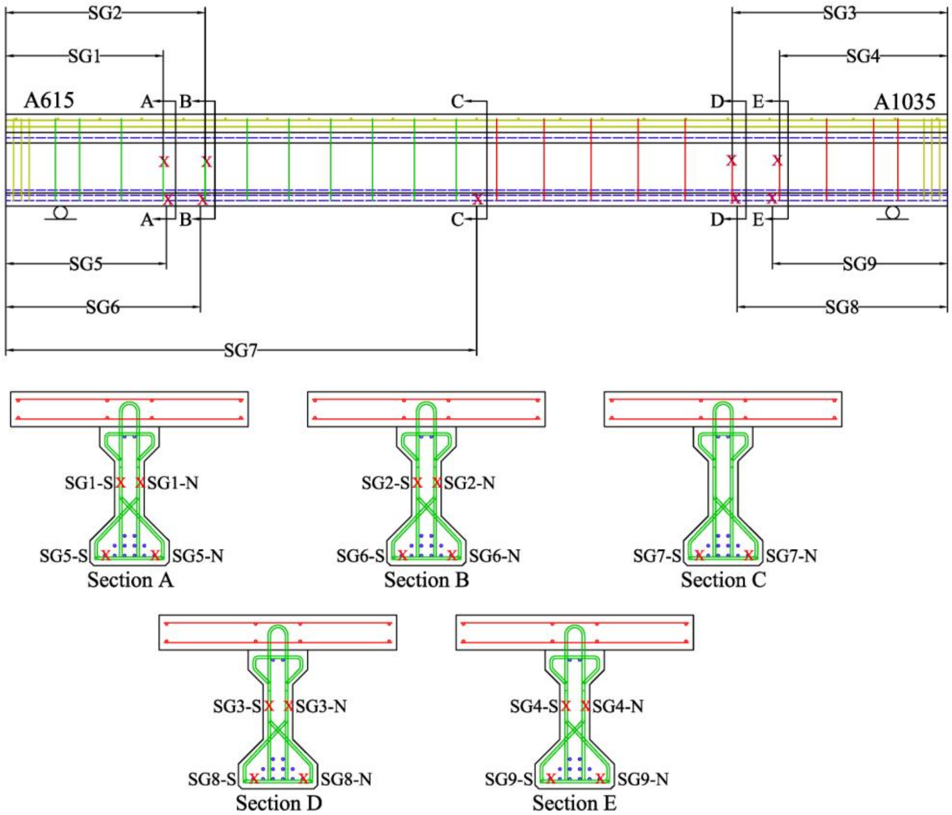


Fig. 3 Strain Gage Locations

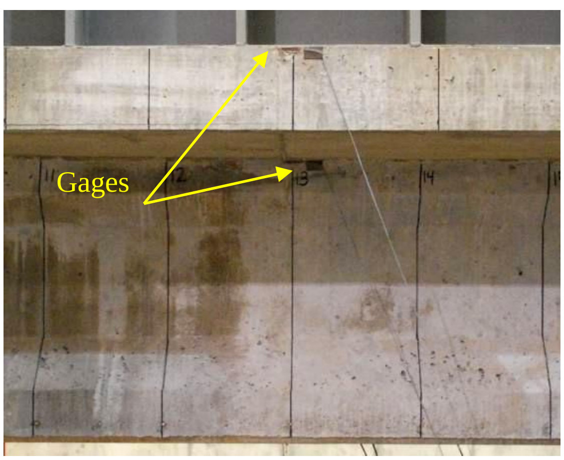


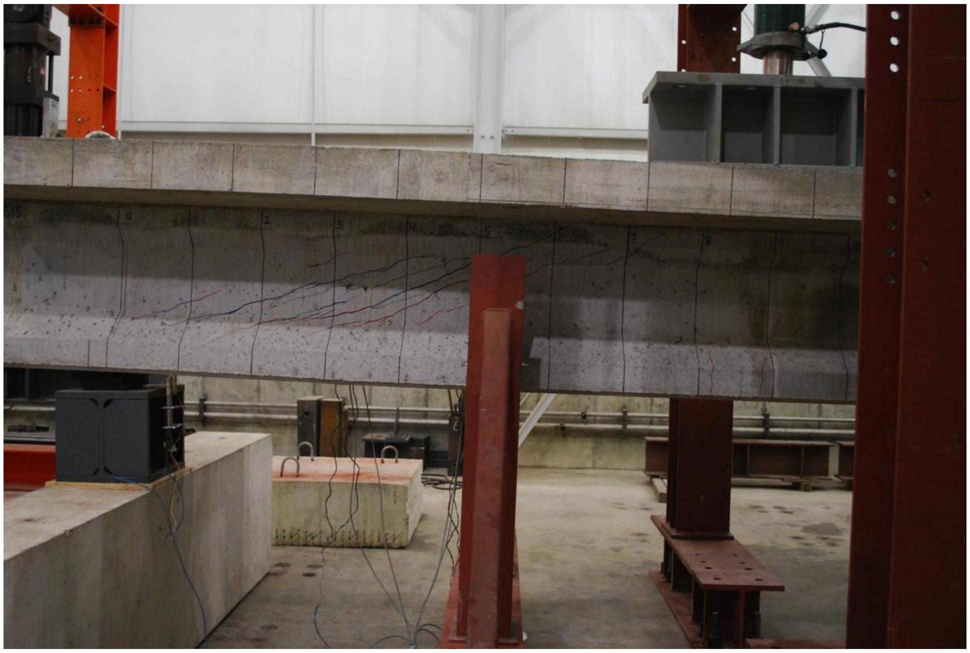
Fig. 4 Externally Bonded Strain Gages

RESULTS

The intent was to have all four specimens fail in shear, but due to difficulty in predicting accurate shear strengths, specimens SP1 and SP3 failed in flexure at loads which exceeded their predicted shear strengths. Specimen SP2 failed in shear in the shear span with A615 stirrups. SP4 was designed to fail in the shear span with A1035 stirrups and did.

As the specimens were loaded, diagonal shear cracks formed. The first diagonal cracks formed at both ends of the beams at the same load in every test. As nearly as could be determined, there was no significant difference in the crack pattern, crack spacing, or crack angle, θ, between the shear spans reinforced with A615 stirrups and those reinforced with A1035 stirrups. Representative crack patterns for specimen SP2 are shown in Figure 5. The similarity of crack patterns, crack spacing, and crack angles was not unexpected as the only difference between the two shear spans was the stirrups. Since both shear spans were relatively lightly reinforced with stirrups, the presence of the stirrups should not have had a major impact on the cracking load, crack spacing, or crack angle.

Figure 6 shows the maximum crack widths for SP2, SP3, and SP4 as a function of shear stress. The cracks in SP1 were too small to measure. The data suggest that cracks in shear spans with the A1035 stirrups were slightly larger than those in the corresponding spans with A615 stirrups. This was not unexpected. At the same applied shear force, the A1035 stirrups are under higher stress because they are at about the same spacing, but of a smaller diameter. In theory, the A615 stirrups should eventually yield and the cracks should begin to open more widely, but the actual yield strength of the A615 stirrups was over 80 ksi, so it is possible that the stirrups did not yield for specimens SP1, SP3, and SP4; hence, the crack openings remained small. It is important to note that the differences between the crack opening values for the A1035 stirrups and the A615 stirrups are small and the crack opening data shows a large scatter, so any difference may be scatter rather than behavior. Moreover, the diagonal cracks became measurable at total shear forces exceeding $0.0316(6)\sqrt{f'_c}b_vd_v = 0.190\sqrt{f'_c}b_vd_v$. In effect, this means measurable cracks did not form until the β value, used in the MCFT, exceeded 6, which is about as large as β can be. At such stress levels, the magnitude of crack width is less of a concern because ensuring adequate load-carrying capacity is the primary design objective. The crack angles were approximately 20 to 25^o. These angles are consistent with expected prestressed beam behavior and consistent with high values of β.



(a) #4 A615 stirrups

(b) #3 A1035 stirrups

Fig. 5 Representative Crack Patterns in the Shear Span – SP2 @ 97% of Ultimate Load

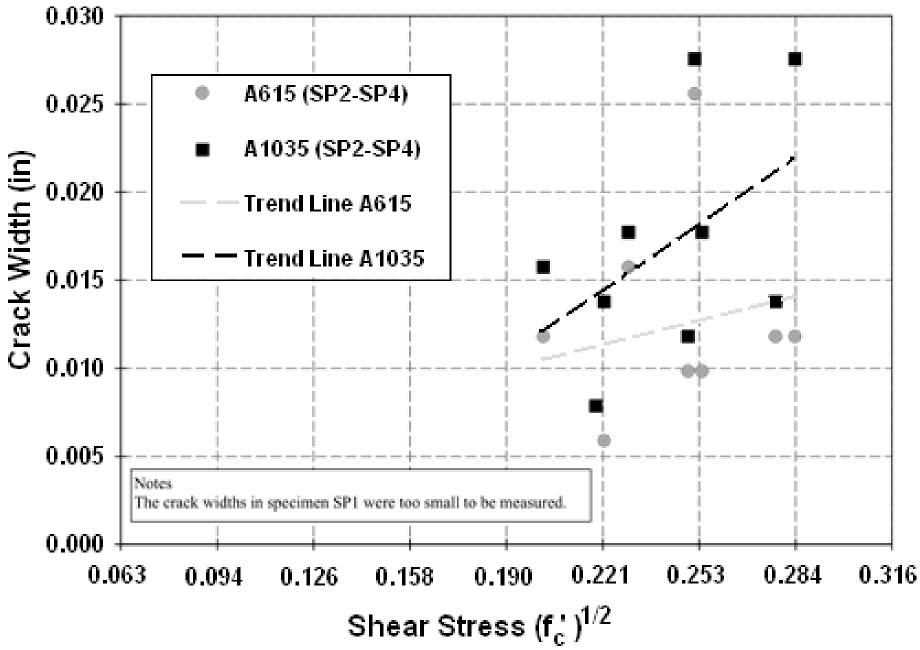


Fig. 6 Maximum Shear Crack Width Versus Shear Stress (ksi)

Table 3 shows the actual applied loads at failure and the shear capacity calculated using the Sectional Model of the AASHTO LRFD Specifications and the Response 2000[2] program. Specimens SP1 and SP3 failed at the predicted shear capacity using Response 2000, but failed in flexure, not shear. Specimens SP2 and SP4 failed in shear at capacities for the side that failed that were 70% greater than the AASHTO design capacity and 50% greater than that predicted by Response 2000. Using the capacities based on AASHTO LRFD Specifications, the shear at the nominal flexural capacity (M_n) is 239 kips. For specimens SP1 and SP3, the measured capacities are larger than this value indicating the flexural failure mode that was observed. For the other two specimens, the measured capacities are slightly below 239 kips, confirming the observed shear failure mode.

Table 3 Shear Capacity of Prestressed Concrete Specimens

Specimen I.D.	Stirrup Type	Failure Mode	Measured Capacity (kips)	AASHTO Capacity		Response Capacity	
				Computed (kips)	Measured/ Computed	Computed (kips)	Measured/ Computed
SP1	A615	Flexure	242	199	1.22	244	0.99
	A1035			170	1.42	244	0.99
SP2	A615	Shear, A615 side	238	139	1.71	157	1.52
	A1035			130	1.83	149	1.60
SP3	A615	Flexure	250	175	1.43	243	1.03
	A1035			154	1.62	239	1.05
SP4	A615	Shear, A1035 side	231	153	1.51	188	1.23
	A1035			132	1.75	164	1.41

Note: Dead load shear has been subtracted from of the computed capacity.

Figure 7 shows the strain in the stirrups of specimen SP2 (failed in the A615 reinforced shear span). The strain in the A615 stirrups is approximately the same as the strain in the A1035 until the A615 stirrups clearly yield. However, when strain is converted to stress (using measured stress/strain behaviors) and the force in the stirrups is calculated, the force in both sets of stirrups is the same (Figure 8). This verifies the design assumption that the two different stirrup configurations still provide the same shear resistance.

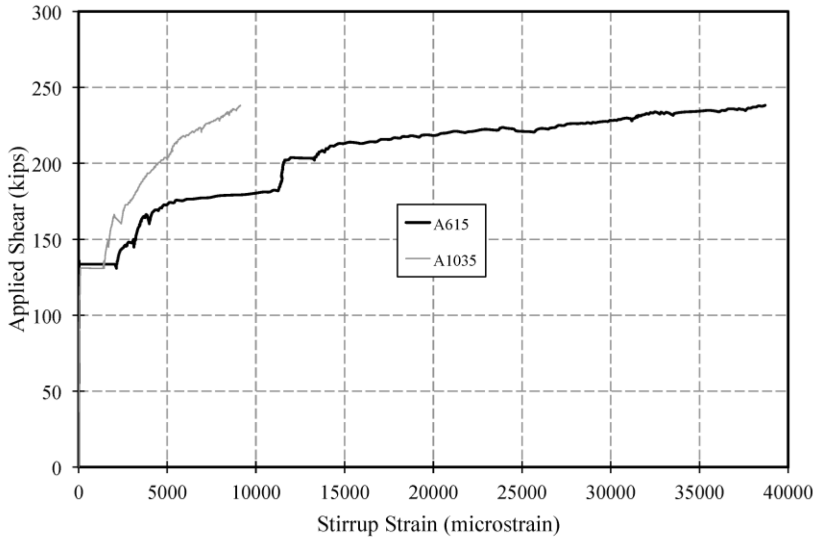


Fig. 7 Strain in Stirrups versus. Applied Load

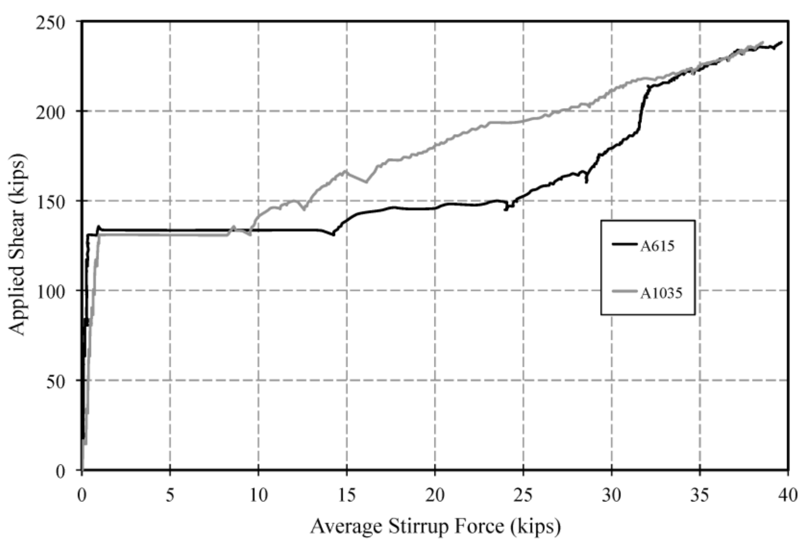


Fig. 8 Total Stirrup Force versus Applied Load

SUMMARY AND CONCLUSIONS

Four prestressed concrete beams were each reinforced with A615 stirrups in one shear span and A1035 stirrups in the other. The two shear spans had stirrups at approximately the same spacing, but the A1035 stirrups were #3 bars while the A615 stirrups were #4 bars. Assuming the yield strength of the A615 stirrups to be 60 ksi and the A1035 stirrups to be 100 ksi, the two shear spans should have had nearly the same stirrup strength. The flexural reinforcement was sized such that the beams would reach their calculated shear capacity before the calculated flexural capacity. The beams were loaded with two, equal point loads so that the shear in each shear span would be equal and constant. Based on the test results, the following observations are made:

1) For a given specimen, the load that caused diagonal cracking in the web was the same for both shear spans.
2) Under load, for a given specimen, there was no discernable difference in the crack spacing or the crack angle of the two shear spans.
3) The A1035 stirrups were smaller and had larger stress levels during the test. The smaller size translated into slightly larger crack openings.
4) The stirrup forces in both shear spans were approximately equal, confirming the design assumption.
5) Although designed to reach shear capacity before flexural capacity, two specimens still failed in flexure. In both cases, the calculated shear capacity of both shear spans was achieved.
6) Two specimens failed in shear, one in the A615 reinforced shear span and one in the A1035 reinforced shear span. In both cases, the measured shear capacity exceeded the capacity calculated by the AASHTO LRFD Specification Sectional Model by 70% and exceeded the capacity calculated by a more exact computer method by 50%.

Thus, it can be concluded that, except for slightly larger crack openings, there is no discernable difference between the behavior of beams reinforced with A1035 stirrups and A615 stirrups where the nominal yield of the stirrups is assumed to be 100 and 60 ksi, respectively. It can also be concluded that the sectional design method in the AASHTO LRFD Specifications is adequate for design using A1035 stirrups with yield strength up to at least 100 ksi. While it is true that cracks in shear spans utilizing the A1035 stirrups were wider, diagonal cracks are not expected to form under service loads (which was observed during the reported tests) so larger crack widths would not cause a serviceability issue.

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The research team dedicates this work to the memory of Panel Member Marcus H. Ansley (Florida DOT).

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