SHEAR BEHAVIOR OF PRETENSIONED GIRDERS CONSTRUCTED WITH SLATE HIGH STRENGTH LIGHTWEIGHT CONCRETE

Karl F. Meyer, PhD, PE, United States Military Academy, West Point, NY
Lawrence F. Kahn, PhD, PE, Georgia Institute of Technology, Atlanta, GA

ABSTRACT

This paper describes the results of shear testing on pretensioned AASHTO Type II composite girders constructed using slate high strength lightweight concrete (HSLC) with f_c' between 8910 psi (61 MPa) and 10980 psi (76 MPa). Each girder was prestressed using 10 0.6-inch (15.2 mm) LOLAX strands tensioned to 75% of strand ultimate stress. Shear reinforcement spacing was different at each end and at the middle of each girder. Results indicated the following for slate HSLC: the current 17th Edition AASHTO Standard specification provides a conservative prediction of concrete and ultimate shear capacity for compressive strengths below 11,000 psi (76 MPa) when steel strength is capped at 60 ksi (414 MPa); the alternate design procedure listed in ACI 318 Section 11.4.2.2 produced some unconservative predictions for concrete compressive strengths over 9,000 psi (69 MPa); the current AASHTO LRFD specification provides a conservative prediction of ultimate strength for compressive strengths below 11,000 psi (76 MPa).

Keywords: Prestressed Concrete, Lightweight Concrete, High-Strength Concrete, Transfer Length, Development Length, Shear Strength, Pretensioned Bridge Girders, Prestressing Strand Slip

INTRODUCTION

Increasing prestressed bridge girder length without increasing section depth has been made possible by advances in concrete quality and strength. Bridge girders over 150 feet (45.7 m) in length are more common, but introduce new construction and transportation challenges. High-strength lightweight concrete (HSLC) can be used to alleviate some of these challenges by reducing member self-weight thus facilitating easier road movement and erection.

Previous research conducted relating to lightweight concrete (LWC) used in prestressed applications involved concrete with compressive strengths less than 7,500 psi (52 MPa). The research presented herein was designed to extend the application of lightweight concrete to design strengths up to 10,000 psi (69 MPa) and to verify the application of current design standards to the higher strength materials. An earlier analytical study showed that the use of HSLC would be beneficial for extending the lengths of bridge girders[1].

EXPERIMENTAL PROGRAM

Six AASHTO Type II composite girders were constructed with ten 0.6-inch (15.2 mm) diameter 270 ksi (1.86 GPa) low relaxation strands as illustrated in Figs. 1 and 2. Three girders were made with an 8,000 psi (55 MPa) design strength expanded slate HSLC and three girders were made with a 10,000 psi (69 MPa) design strength expanded slate HSLC mix. The 8,000 psi (55 MPa) HSLC was termed a Grade 1 mix while the 10,000 psi (69 MPa) HSLC was termed a Grade 2 mix.

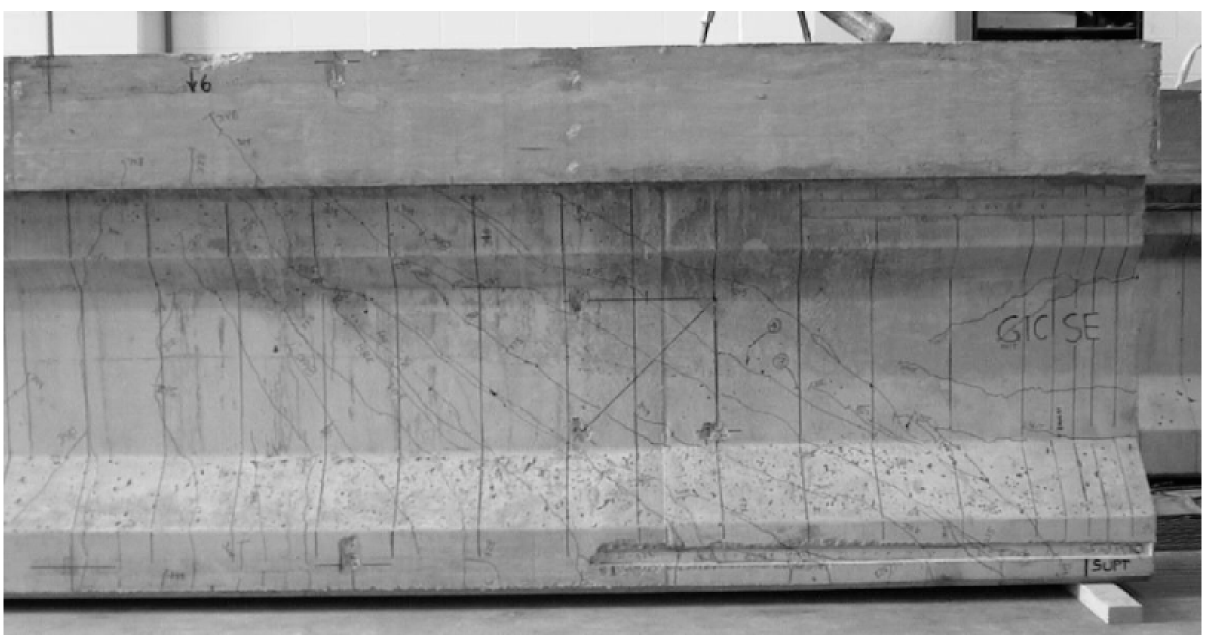

Fig. 1. East End of Girder G1C After Testing

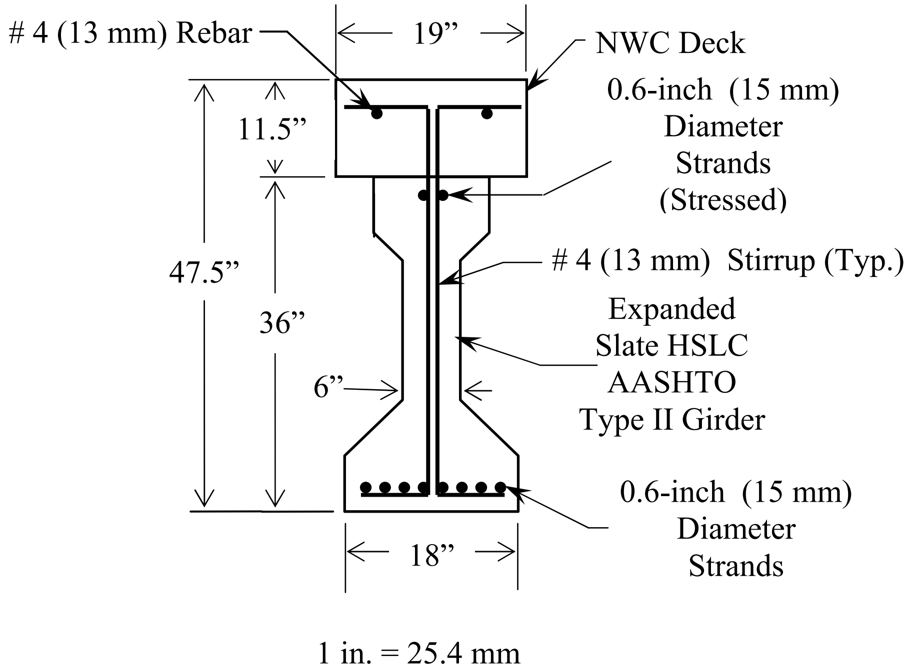

Fig. 2. Composite Girder Cross Section

The mix designs were developed in the laboratory and field-tested at a precast plant earlier in the research project. The coarse aggregate in both mix designs consisted of expanded slate lightweight aggregate from North Carolina. Table 1 provides details on the mix components for each girder, the date of casting, and the 1-day, 56-day, and “on day of test” compressive strengths. The G1 and G2 series girders refer to girders constructed with the 8,000 psi (55 MPa) (Grade 1) and 10,000 psi (69 MPa) (Grade 2) design strengths, respectively. The eight bottom and two top strands were stressed to $0.75f_{pu}$ (203 ksi, 1.40 GPa) prior to release. The girders were cast in pairs on three different days as listed in Table 1.

The 39-foot (11.9-m) long “A” and “B” girder and 43-foot (13.1-m) long “C” girders were designed so that point loads could be placed near each end for development length tests and so that the middle section could be subsequently tested to examine shear and flexural capacity. A composite deck with a thickness of 11.5 inches (292 mm) and width of 19 inches (483 mm) was placed on each girder eight weeks after girder construction. The composite deck was designed to give the same internal flexural moment arm “*jd*” as in a composite bridge with an 8-inch (20.3-cm) thick deck and an effective width of 93 inches (2.36 m), but to ease construction by having the deck with a width as close to the width of the top flange of the girder as possible. A 3,500 psi (24 MPa) normal weight bridge mix was used for the composite deck; its mean compressive strength at date of testing was 5,370 psi (37 MPa). The nine test configurations given in Table 2 were built for both the Grade 1 and 2 mixes. In total, 18 sets of shear data were collected during the testing program. Fig. 3 illustrates the sequence of testing on each girder to allow three tests per girder and dimensions for test set-up.

Table 1. Expanded Slate HSLC Mixes for Girder Placements (Quantities per Cubic Yard)

Material / Girder #	G1A, G1B	G2A, G2B	G1C	G2C
Design Strength (psi)	8,000	10,000	8,000	10,000
Date of Casting	9 Jul 01	12 Jul 01	17 Jul 01	17 Jul 01
Slate Lightweight Aggregate (lbs)	989	979	974	998
Normal Weight Sand (lbs)	1098	1087	1078	1083
Class F Fly Ash (lbs)	142	150	142	150
Silica Fume (lbs)	19	100	19	100
Type III Portland Cement (lbs)	785	740	785	740
Water Reducer (WRDA 35) (fl oz)	57	60	57	60
Air Entrainer (Daravair 1000) (fl oz)	12	10	12	10
HR Water Reducer (ADVA Flow) (fl oz)	48	129	48	129
Water (lbs)	165	153	186	138
Water/Cementitious Materials Ratio	0.28	0.23	0.28	0.23
1-day f_c' (average, accelerated curing) (psi)	7,470	9,640	6,320	8,260
28-day f_c' (average, ASTM curing) (psi)	8,840	10,120	7,600	9,810
56-day f_c' (average, ASTM curing) (psi)	9,350	10,820	8,460	10,510
f_c' on day of girder test (psi)	9,580	10,980	8,910	10,520
Girder age at test (days)	103	123	110	144

Note: 1000 psi = 1 ksi = 6.895 MPa, 1 lb = 0.454 kg, 1 fluid oz = 29.6 ml

Table 2. Girder Test Configurations

Test Configuration	Stirrup Density	Stirrup Spacing (in.)	Shear Span "a" (in.)	Distance "L_1" (in.)	Distance "L_2" (in.)	Distance "L" (in.)
1	Single	7	90	456	0	456
2	Double	3.5	61	316	0	456
3	Double	3.5	75	456	0	456
4	Single	7	85	504	0	504
5	Single	7	61	321	0	456
6	Single	7	75	369	0	504
7	Minimum	24	82	185	140	456
8	Minimum	24	96	210	135	456
9	Minimum	24	120	244	135	504

Note: 1 in. = 25.4 mm

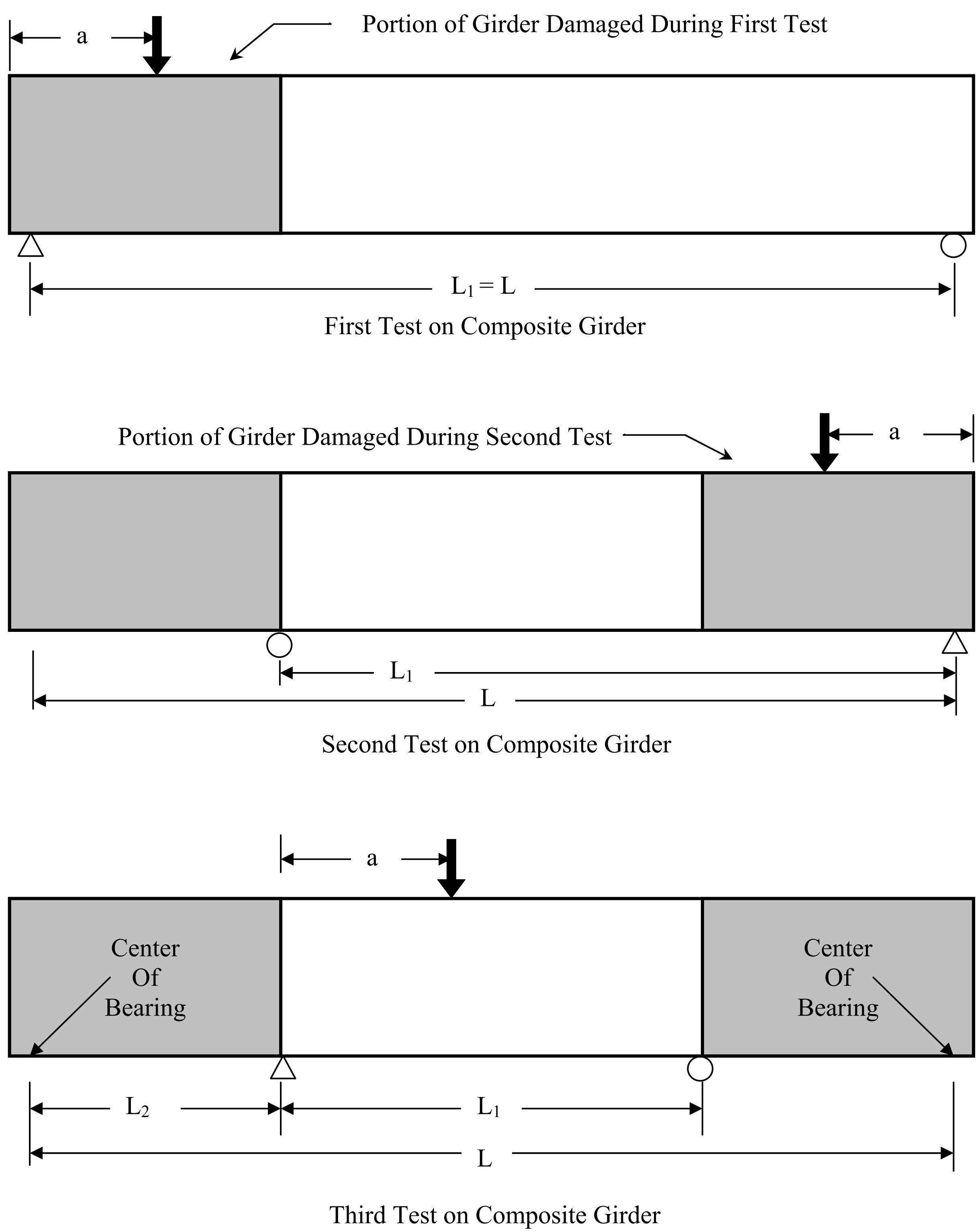

Fig. 3. Girder Testing Sequence and Layout Dimensions

Shear reinforcement in the development length region of each girder end was designed to satisfy AASHTO and ACI provisions and was termed "single density." In some girders, twice this required shear reinforcement was included, termed "double density," in order to study the effect of shear reinforcement on strand development. The stirrup spacing was 24 (610 mm) inches in the center section of each girder to allow specific examination of shear. Stirrups were two "C" shaped No. 4 (13 mm) Grade 60 bars (f_y = 62 ksi, 427 MPa).

SHEAR STRENGTH CALCULATIONS

Three methods of calculating girder shear strength were examined and evaluated for use with HSLC to include the AASHTO Standard Specification 17th Edition[2], the alternate approach in Section 11.4.2.2 of ACI 318-02[3], and the AASHTO LRFD 1st Ed.[4]. Section 11.4.2.2 of ACI 318-02 specifies only an alternate method for calculating V_{cw}; the remainder of the method remains unchanged.

AASHTO STANDARD 17th EDITION SHEAR DESIGN APPROACH

The AASHTO Standard Specification for Highway Bridges 17th ed.[2] specifies the concrete strength, V_c, as the lesser of the flexure shear strength, V_{ci}, and the web shear strength, V_{cw}. Based on the shear spans and characteristics of the girders in this research, the web shear strength, V_{cw}, was always the governing value.

The web shear strength, V_{cw}, was calculated using AASHTO Eq. 9-29 and shown below as Eq. (1). The value of λ was 0.85 signifying the use of sand lightweight concrete.

$$V_{cw} = (3.5\lambda \frac{\sqrt{f_c^{'}}}{1000} + 0.3 f_{pc}) b' d + V_p \tag{1}$$

In the girders tested, there were no draped strands thus V_p was zero. The value f_{pc} accounted for the effect of prestress and the dead load of the girder alone at the centroid of the composite section. The effective prestressing force was reduced when the section being examined was inside a distance equal to the transfer length from the end of the girder.

For the girders tested, the shear area, A_v, was 0.4 in^2 (258 mm^2) for two-leg No. 4 (13 mm) bar stirrups. The value "*s*" was the spacing of the stirrups, which was 3.5 inches (89 mm) for the double density stirrups, 7 inches (178 mm) for single density stirrups and 24 inches (610 mm) for minimum stirrup spacing. The yield strength of the stirrups was 62 ksi (427 MPa) as determined experimentally. The nominal shear strength was calculated using Eq. (2) below.

$$V_n = V_c + V_s \tag{2}$$

ACI ALTERNATE APPROACH FOR CALCULATING WEB SHEAR STRENGTH

ACI 318-02 Section 11.4.2.2[3] provides an alternate technique for calculating V_{cw}. The code states that V_{cw} shall be calculated as the shear force corresponding to dead load plus live load that results in a principal tension stress of $4\lambda(f_c')^{1/2}$ at the centroidal axis of the member. Lin and Burns[5] addressed this alternate technique and provided Eqs. (3) through (5) which result from the application of Mohr's circle:

$$V_{cw-Pred} = f''_{t-Pred}\sqrt{1+\frac{f_{pc}}{f''_{t-Pred}}}\,bd_p \tag{3}$$

The predicted diagonal tensile strength, $f_t''_{-Pred}$, was calculated using Eq. (4).

$$f''_{t-Pred} = 4\lambda\sqrt{f'_c} \tag{4}$$

The value of f_{pc} was calculated at the midpoint of the shear span. The experimental cracking load, V_{cw-Exp}, was recorded when the first shear cracking occurred during the girder test. Based on V_{cw-Exp}, the experimental diagonal tensile strength, $f_t''_{-Exp}$, was calculated using Eq. (5).

$$f''_{t-Exp} = \sqrt{\left(\frac{V_{cw-Exp}}{bd_p}\right)^2+\left(\frac{f_{pc}}{2}\right)^2}-\frac{f_{pc}}{2} \tag{5}$$

The $f_t''_{-EXP}$ value was then normalized using Eq. (6).

$$\xi_t = \frac{f''_{t-Exp}}{\lambda\sqrt{f'_c}} \tag{6}$$

A normalized value, ξ_t, less than 4 was an indication that Eq. (4) provided an unconservative prediction of the tensile stress at which initial web shear cracking occurred.

AASHTO LRFD 1ST EDITION

The 1998 AASHTO LRFD Specification for Highway Bridges 1st Ed.[4] procedure is based on compression field theory and follows a significantly different procedure. The shear strength provided by the concrete, V_c, based on the LRFD procedure resulted in Eq. (7).

$$V_c = 0.0316\lambda\beta\sqrt{f'_c}\,b_v d_v \tag{7}$$

The strength of the shear reinforcing steel was calculated with AASHTO LRFD Eq. 5.8.3.3-1 shown below as Eq. (8) where the crack angle, θ, was determined experimentally and varied between 25 and 38 degrees.

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \tag{8}$$

GIRDER SHEAR TESTING

In order to monitor and measure girder shear behavior, a large strain rosette made with linear variable displacement transducers (LVDT) was placed on the web of each girder at the mid-point of the shear span. Fig. 4 shows a typical plot of applied shear vs. principal strains. The point at which initial shear cracking occurred was the point where the two principal strain plots separated. This value matched within about five percent the value at which initial shear cracking was visually and audibly detected during the test. The plot shows initial shear cracking at an applied shear of 134.2 kips (591 kN). Protractor measurements of the cracking angle, θ, agreed with the rosette measurements within ± 10 percent. Fig. 5 shows a typical center section failing in shear.

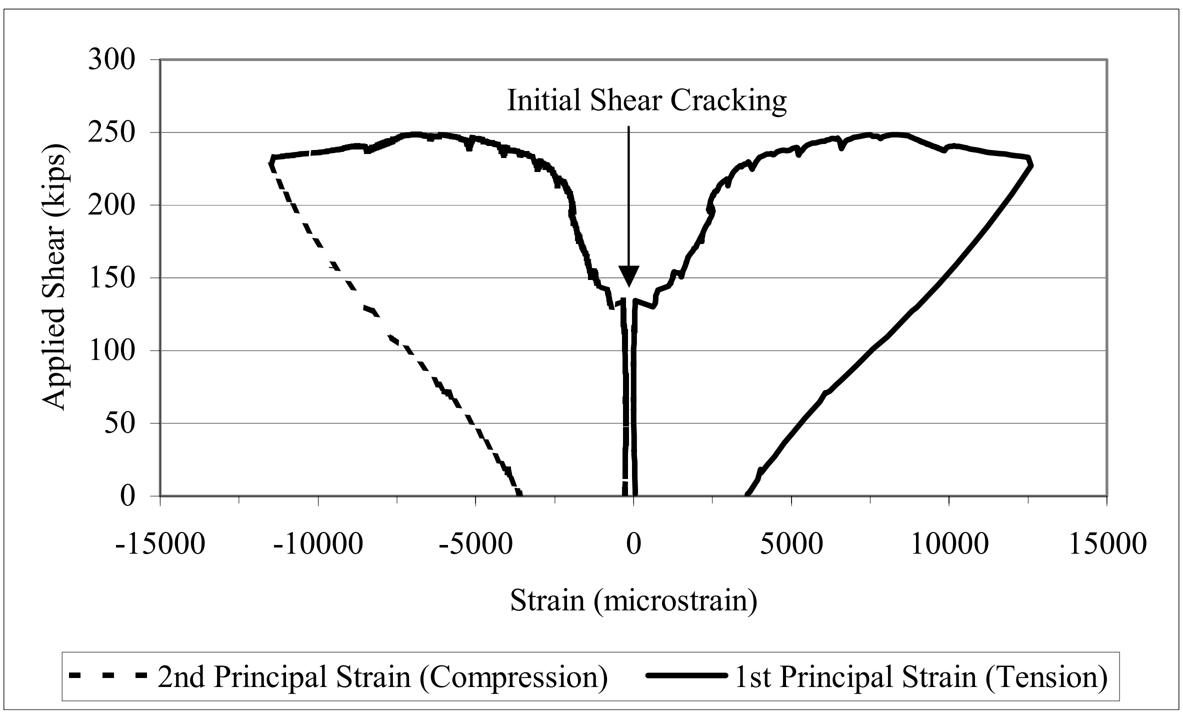

Fig. 4. Plot of Applied Shear vs. Principal Strains for Girder Test G1C-West
(1 kip = 4.445 kN)

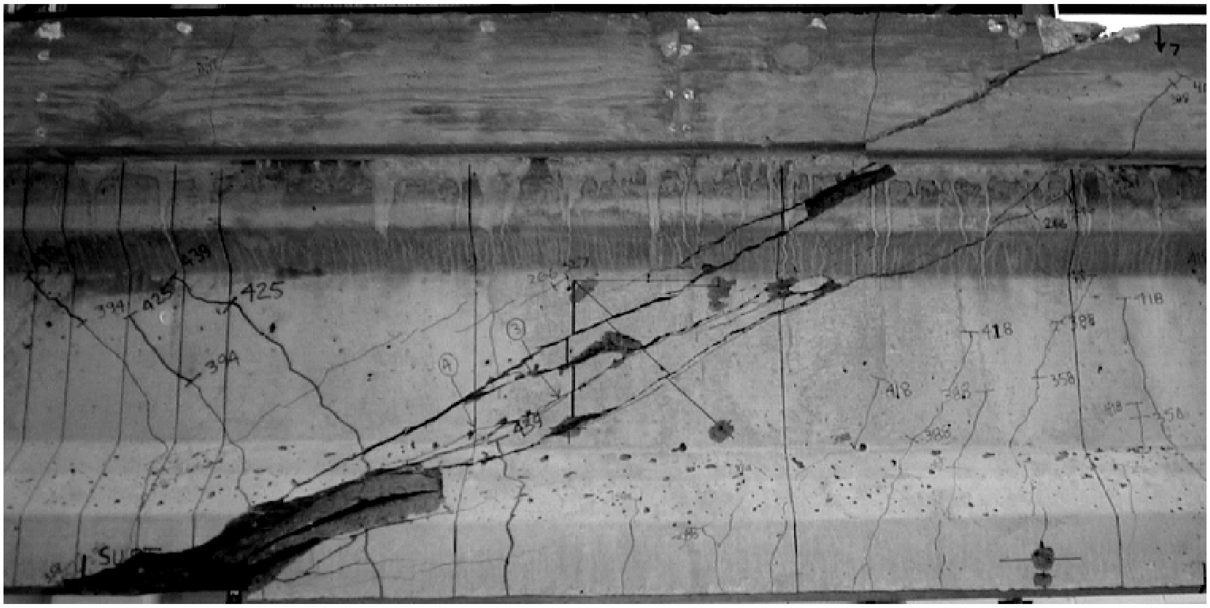

Fig. 5. Girder G2A-Center Section Test Crack Patterns

EXPERIMENTAL RESULTS AND DISCUSSION

INITIAL SHEAR CRACKING

Table 3 provides an overview of initial shear cracking. The predicted values were calculated at the midpoint of the shear span. The experimental value was the applied shear at which cracking was first recorded visually or electronically. At initial shear cracking, the vertical strain was always less than 0.00026. It was assumed that the stirrups resisted no shear ($V_s = 0$) so that the shear force at critical cracking equaled the shear resistance provided by the concrete alone. Examination of Table 3 showed that the AASHTO Standard[2] method of calculating concrete shear strength was conservative overall. Percent difference was calculated using Eq. 9 with a positive percent difference indicating a conservative result.

$$Percent\ Difference = \frac{V_{cw\,(Equation)} - V_{c-EXP\,(Experimental)}}{V_{c-EXP\,(Experimental)}} \times (100) \quad (9)$$

In the case of girder tests G1C-Center and G2C-Center, the prediction was less than 3 percent unconservative.

The ACI alternate approach provided the closest prediction for V_c where the cracking shear was equated to the shear strength provided by the concrete. On average, the predicted V_c values were about 6 percent greater than the experimental values for both the G1 and G2 girder tests. Girder tests G1C-Center and G2C-Center showed the greatest difference from the ACI alternate predicted values; the average was 18 percent less than experimental when the maximum stirrup spacing of 24 inches was used.

Table 3. Overview of Initial Cracking Shears

Test No.	Test Config.	Measured Crack Angle (degrees)	$V_{c\text{-}EXP}$ Exp. (kips)	V_{cw} AASHTO Standard[2] (kips)	V_{cw} ACI[3] Alternate (kips)	Percent Diff. Exp. vs. Standard	Percent Diff. Exp. vs. ACI Alt
G1A-East *	2	31	145.0	104.1	124.0	39.4%	16.9%
G1A-West	1	25	120.0	107.1	127.5	12.1%	-5.9%
G1A-Center *	7	25	134.0	98.5	118.0	36.1%	13.6%
G1B-East *	5	32	140.0	105.5	125.5	32.7%	11.6%
G1B-West	3	38	141.2	103.9	123.6	35.9%	14.2%
G1B-Center *	8	25	136.1	98.9	118.4	37.6%	14.9%
G1C-East *	6	30	123.5	101.5	120.7	21.7%	2.3%
G1C-West	4	30	134.2	104.0	123.3	29.0%	8.8%
G1C-Center	9	25	94.0	96.7	115.8	-2.8%	-18.8%
G2A-East	2	38	178.6	114.4	135.7	56.1%	31.6%
G2A-West	1	34	157.3	118.1	139.5	33.2%	12.8%
G2A-Center *	7	33	140.0	109.4	130.6	27.9%	7.2%
G2B-East	5	35	163.1	114.6	136.1	42.3%	19.8%
G2B-West	3	33	148.0	117.4	138.7	26.1%	6.7%
G2B-Center *	8	26	120.4	110.5	131.9	9.0%	-8.7%
G2C-East	6	27	143.4	112.0	133.0	28.1%	7.8%
G2C-West	4	30	122.4	113.4	134.2	8.0%	-8.8%
G2C-Center	9	33	107.0	107.3	128.1	-0.2%	-16.5%
G1 Avg			129.8	102.2	121.9	26.8%	6.4%
G2 Avg			142.2	113.0	134.2	25.6%	5.8%
G1 Std Dev						14.1%	11.9%
G2 Std Dev						17.8%	15.2%

* Girder failed in shear at ultimate as primary or secondary failure mode.
1 kip = 4.448 kN

INITIAL SHEAR CRACKING CONSIDERING STIRRUP SPACING

Fig. 6 plots the concrete shear strength, $V_{c\text{-}Exp}$, vs. stirrup spacing. The trend lines indicated that $V_{c\text{-}Exp}$ appeared to increase with smaller stirrup spacing. Since the stirrups carried some load prior to concrete cracking, it was understandable that the apparent $V_{c\text{-}Exp}$ value would be higher with closer stirrup spacing. An interesting aspect of Fig. 6 was the convergence of the G1 and G2 trendlines at a stirrup spacing of 24 inches (610 mm). The indication here was that little difference existed between V_c for the G1 and G2 girders. Based on the use of maximum stirrup spacing in these 6 center-span tests, it was likely that these data most truly reflected the V_c value of the girders. The implication of this finding was that an apparent ceiling exists which limits the tension strength of concrete. This same phenomenon has been observed by other researchers in high performance normal weight concrete with f_c' over 10,000 psi (69 MPa)[6]. In order to investigate this possibility, normalization was conducted on the shear results.

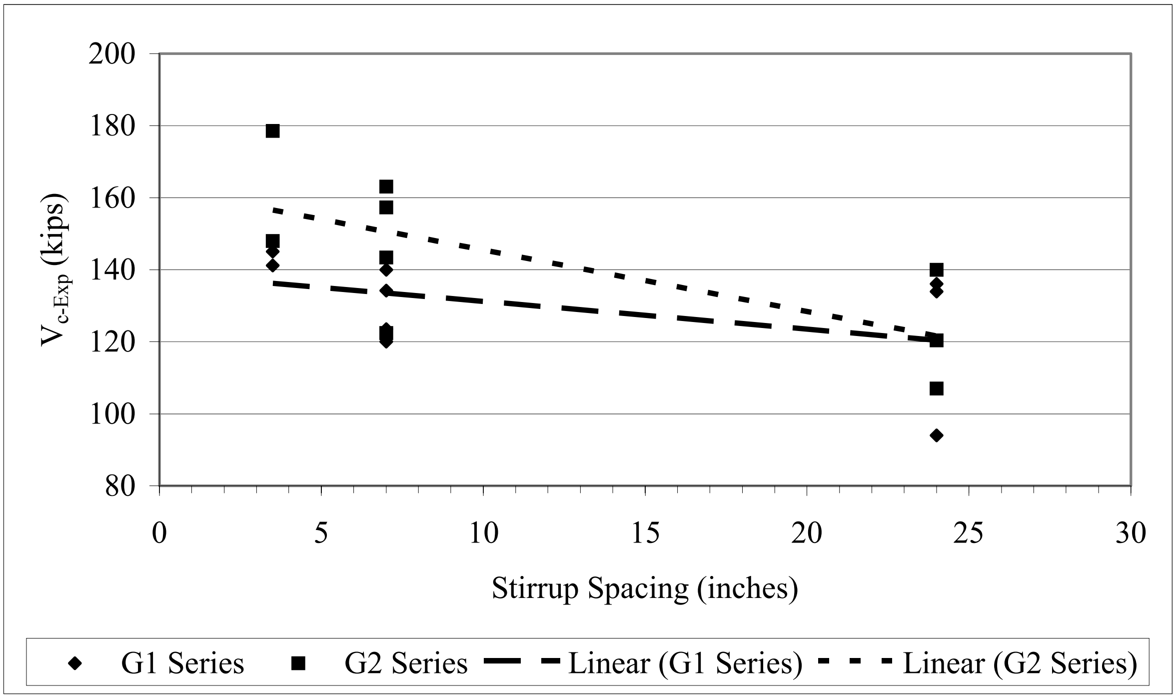

Fig. 6. Concrete Shear Strength, V_c, vs. Stirrup Spacing
(1 kip = 4.445 kN, 1 in. = 25.4 mm)

NORMALIZED DIAGONAL TENSION STRENGTH FACTOR, ξ_t

Fig. 7 plots ξ_t calculated using Eq. (6) vs. stirrup spacing. Values ξ_t of less than four indicate that the actual diagonal tensile stress was less than $4\lambda(f_c')^{1/2}$. Fig. 7 shows that the ACI alternate prediction technique for diagonal tensile strength produced some unconservative results.

ULTIMATE SHEAR CAPACITY, V_u

Table 4 provides an overview of predicted ultimate shear capacity calculated by the AASHTO Standard[2] method and the AASHTO LRFD[4] method with the strength reduction factor, ϕ_s, of 1 and compares them to the experimental maximum shear values for those tests that exhibited a shear (SH), shear-slip (SH-SL) or shear-flexure failure (SH-FL) failure mode. Using the AASHTO Standard method, the stirrup yield strength was capped at 60 ksi (414 MPa) as required by the code. Since the value of f_y was 62 ksi (427 MPa), there was an insignificant difference in predicted values.

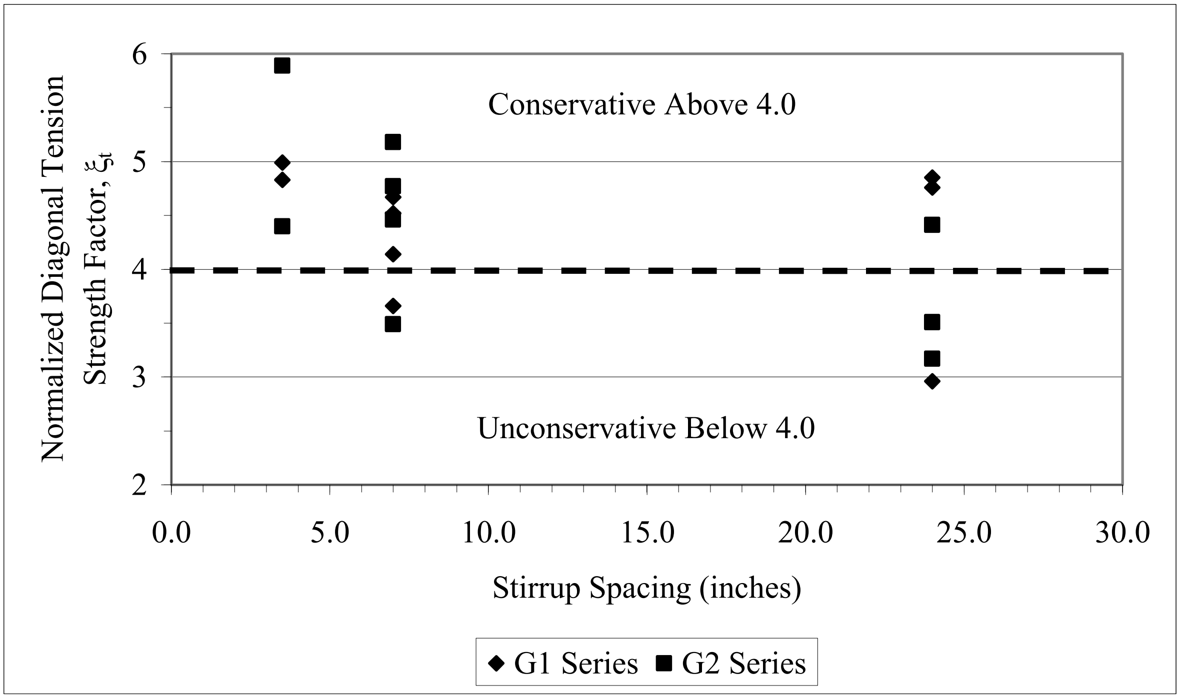

Fig. 7. Normalized Diagonal Tension Strength Factor, ξ_t, vs. Stirrup Spacing (1 in. = 25.4 mm)

Table 4. Predicted vs. Experimental V_u Values for Tests Failing in Shear

Test No.	AASHTO Standard[2] f_y = 62 ksi V_u (kips)	AASHTO Standard[2] f_y = 60 ksi V_u (kips)	AASHTO LRFD[4] f_y = 62 ksi V_u (kips)	Exp. V_u (kips)	Percent Diff. Exp vs Std 62	Percent Diff. Exp vs Std 60	Percent Diff. Exp vs LRFD
G1A-Center	138.7	137.4	103.8	258.0	86.0%	87.8%	148.5%
G1B-East	243.3	238.9	241.1	312.2	28.3%	30.7%	29.5%
G1B-Center	138.6	137.3	104.3	234.1	68.9%	70.5%	124.5%
G1C-East	238.4	234.0	255.5	289.2	21.3%	23.6%	13.2%
G2A-Center	149.5	148.2	82.3	255.9	71.2%	72.7%	211.0%
G2B-Center	150.9	149.6	102.7	246.3	63.2%	64.6%	139.9%

G1 Average	51.1%	53.2%	78.9%
G2 Average	67.2%	68.7%	175.5%
G1 Std Dev	31.3%	31.0%	67.5%
G2 Std Dev	5.7%	5.7%	50.3%

1 ksi = 8.894 MPa

The AASHTO Standard[2] technique produced conservative results overall. Limiting the yield strength to 60 ksi (414 MPa) caused the prediction to be slightly more conservative.

The AASHTO LRFD[4] technique produced more conservative results overall because the girders with 24-inch (610 mm) spacing were predicted to carry much less shear. The AASHTO LRFD penalized those girders with a very low concrete strength relative to the AASHTO Standard procedure.

Fig. 8 provides a plot of the data from Table 4 normalized by dividing the ultimate shear capacity, V_u, by the lightweight concrete factor, λ, and the square root of the compressive strength. Trendlines are depicted in Fig. 8 for the AASHTO Standard and LRFD Specifications. The Fig. shows how the AASHTO LRFD Specification becomes quite conservative at larger stirrup spacings where the contribution of V_s is low. At closer stirrup spacings, the Fig. shows how the results from the Standard and LRFD specifications provide very similar results.

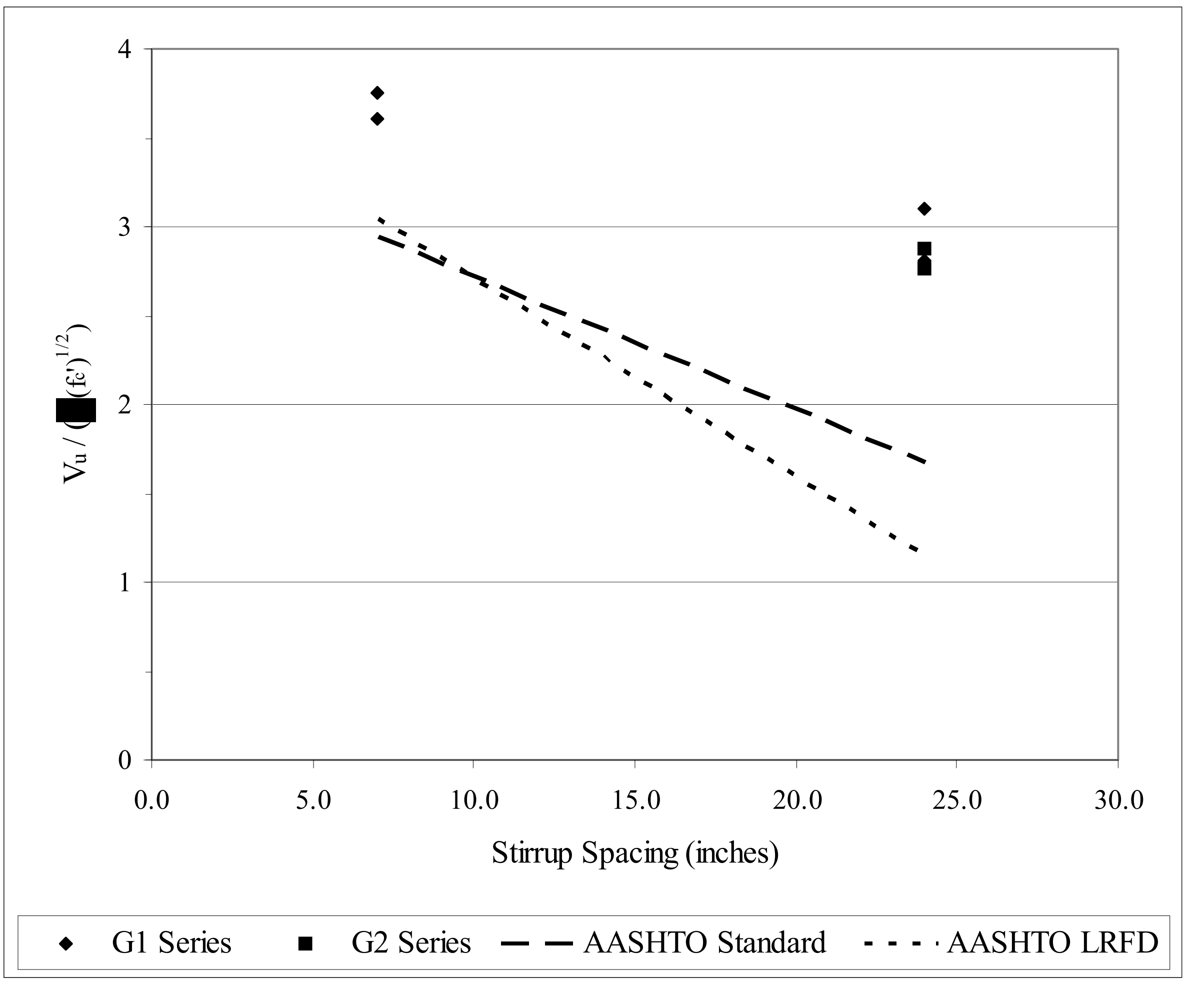

Fig. 8. Normalized Ultimate Shear Capacity, V_u, vs. Stirrup Spacing (1 in. = 25.4 mm)

CONCLUSIONS

The current AASHTO Standard specification provides a conservative prediction of concrete and ultimate shear capacity for pretensioned girders constructed with slate HSLC. The alternate design procedure listed in ACI-318 Section 11.4.2.2 for predicting shear strength produced some unconservative predictions for sand lightweight concrete compressive strengths over 9,000 psi (62 MPa). Further investigations should examine the unconservative nature of ACI 318 Section 11.4.2.2 for high-strength lightweight concrete.

The current AASHTO LRFD specification provides a conservative prediction of ultimate strength.

ACKNOWLEDGEMENTS

This research was sponsored by the Georgia Department of Transportation under Georgia DOT Task Order No. 97-22, Research Project No. 2004. Carolina Stalite Company donated all lightweight aggregate for this project. Grace Construction Products donated chemical admixtures and silica fume. Boral Industries donated all class "F" fly ash. Lafarge Corporation (previously Blue Circle Materials) and CEMEX donated all Type III Cement. INSTEEL donated all 0.6-inch (15.2-mm) diameter prestressing strand. General Steel donated all reinforcing steel. Tindall Corporation, Jonesboro, Georgia, provided Georgia Tech researchers with equipment, manpower, and materials for the field production study. These sponsors and their advice and cooperation are gratefully acknowledged. Mr. Brandon Buchberg, Mr. Adam Slapkus, Mr. Mauricio Lopez and Ms. Natalie Hodges assisted in the experimental work.

The conclusions and opinions expressed herein are those of the authors and do not necessarily represent the opinion, conclusions or policies of the Georgia Department of Transportation or of the other sponsors.

REFERENCES

1. Meyer, Karl F., Kahn, Lawrence F., "Lightweight Concrete Reduces Weight and Increases Span Length of Pretensioned Concrete Bridge Girders," *PCI Journal*, Vol. 47, No. 1, January-February 2002, pp. 68-77.
2. *Standard Specifications for Highway Bridges*, 17th ed., American Association of State Highway and Transportation Officials, Washington, D. C., 2002.
3. *Building Code Requirements for Reinforced Concrete*, ACI 318-02, *and Commentary*, ACI 318R-02, American Concrete Institute, Detroit, 1999.
4. *LRFD Specifications for Highway Bridges*, 1st ed., American Association of State Highway and Transportation Officials, Washington, D.C., 1998.

5. Lin, T. Y., Burns, N. H., *Design of Prestressed Concrete Structures*, Third Edition, John Wiley and Sons, New York, New York, 1981.
6. Dill, J. C., "Development Length of 0.6-inch diameter Prestressing Strand in High-Performance Concrete," Masters Thesis, Georgia Institute of Technology, May 2000, 322 pp.