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Evaluation of Minimum Shear Reinforcement Requirements for Higher Strength Concrete

by Guney Ozcebe, Ugur Ersoy, and Tugrul Tankut

This paper presents an evaluation on the minimum shear reinforcement requirements given in the ACI, Canadian, and Turkish codes for high-strength concrete. Thirteen beams having the minimum shear reinforcement required by ACI 318-83, the Turkish Code, and the equations proposed in this paper were tested. Concrete strength varied between 60 and 80 MPa (8700 and 11,600 psi). For high-strength concrete (f'_c > 69 MPa), the minimum shear reinforcement requirements of the Turkish Code and ACI 318-95 are not very different from one another. Similarly, requirements of the 1994 Canadian Code (CSA A23.3-94) are not too different from the proposed equation. In light of the test results, the adequacy of code requirements are discussed. Emphasis is given to reserve strength, ductility, and cracking.

Keywords: beams (supports); cracking (fracturing); ductility; high-strength concretes; shear strength; stirrups.

INTRODUCTION

In general, design codes provide requirements for minimum shear reinforcement for beams. Most of the expressions given in codes for minimum shear reinforcement are empirical in nature and not based on well-established, accepted criteria. Therefore, the requirements are revised frequently whenever additional new data become available.

It is generally agreed that reinforced beams should have adequate shear reinforcement to prevent sudden and brittle failure after formation of the diagonal crack, and also to keep crack width at an acceptable level. However, there is no established quantitative criteria for reserve strength required beyond cracking strength and limits for the crack width. The minimum shear reinforcement is also required to provide somewhat ductile behavior prior to failure.

While the ACI Building Code (ACI 318-83)[1] and Canadian Code (CSA A23.3-M84)[2] require minimum shear reinforcement when $V > 0.5V_c$, some other codes, such as the Turkish Code (TS500-83),[3] require minimum shear reinforcement regardless of the level of the design shear. The 1983 ACI Building Code and 1984 CSA Standards specify minimum shear reinforcement as a function of yield strength of shear reinforcement only. Concrete strength is not included in the equations given for minimum shear reinforcement.

The use of high-strength concrete raised some doubts on the validity of the equations given for minimum shear reinforcement, since these equations were based on tests of beams made of normal strength concrete. Recent tests on beams with high-strength concrete indicated that reserve strength beyond diagonal cracking strength decreased as concrete strength increased.[4-7] Tests also showed that beams having the minimum shear reinforcement required by the 1983 ACI Building Code had limited reserve strength when higher-strength concrete (f'_c > 69 MPa) was used.[8-10] As a result, the 1989 ACI Building Code (also ACI 318-95)[11] included some revisions to correct the minimum shear reinforcement equation for concrete strength higher than 69 MPa (10 ksi).

The equation given in the 1983 Turkish Code (TS500-83) for minimum shear reinforcement was derived by equating the diagonal cracking strength of the beam to the shear strength of the same beam with shear reinforcement. The diagonal cracking strength was magnified by a factor of 1.5. Since the minimum shear reinforcement derived included the tensile strength of concrete, no additional revision was made for high-strength concrete.

The revised expression formulating the minimum shear reinforcement in the 1994 Canadian Code (CSA A23.3-94)[12] is the same as the one given in TS500-83, except a smaller constant is used.

Since limited test data are available related to minimum shear reinforcement in beams of high-strength concrete, a test program was initiated to study the adequacy of the code requirements. Thirteen beams were tested[13,14] to investigate the adequacy of the minimum shear reinforcement requirements given in ACI 318-83 and TS500-83. It was initially intended to use 60- and 90-MPa (8.7- and 13-ksi) concrete in the test specimens.

The shear reinforcements of the test beams were designed according to the minimum requirements given in ACI 318-83, TS500-83, and the equation proposed by the authors.

The proposed equation is similar to the equation recommended by Ersoy.[15] Ersoy equated the factored diagonal cracking strength of the beam to the ultimate strength and solved for the shear reinforcement required to satisfy this equality.

RESEARCH SIGNIFICANCE

Minimum shear reinforcement required in the codes aims to provide adequate reserve capacity and reasonable ductility beyond diagonal cracking and to minimize crack width. The equations given for minimum shear reinforcement are based on previous experimental data from beams of normal strength concrete. Limited test data is available for high-strength concrete.

In this paper, the adequacy of the minimum shear reinforcement requirements given in the design codes for beams of high-strength concrete is investigated. Thirteen beams having the minimum shear reinforcement required by ACI 318-83, TS500-83, and the equation proposed in this paper, were tested.

In the present paper, the behavior of test beams is reviewed, emphasizing the reserve strength beyond diagonal cracking, ductility, crack pattern, and mode of failure. Code requirements are discussed in light of the test results.

The minimum shear reinforcement requirements of CSA A23.3-94 and the equation proposed by the authors are not very different from each other. The same thing can be said for the ACI 318-95 and TS500-83 codes when high-strength concrete is used. Therefore, although test beams were designed in accor-

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Guney Ozcebe *is an associate professor of civil engineering at the Middle East Technical University, Ankara, Turkey. He received his PhD in 1987 from the University of Toronto, Toronto, Canada. His research interests include structural use of high-strength concrete, seismic repair and strengthening of reinforced concrete structures, and precast joints under seismic actions.*

ACI member **Ugur Ersoy** *is a professor of civil engineering at the Middle East Technical University. He received his PhD in 1965 from the University of Texas at Austin, Tex. He is the recipient of the 1969 ACI Wason Medal. His research interests include structural use of high-strength concrete, seismic repair and strengthening of reinforced concrete structures, and precast joints under seismic actions.*

Tugrul Tankut *is a professor of civil engineering at the Middle East Technical University and is in charge of the Civil Engineering Division of the Turkish Scientific and Technical Research Council. He received his PhD in 1969 from Imperial College, London, UK. His research interests include structural use of high-strength concrete, seismic repair and strengthening of reinforced concrete structures, and precast joints*

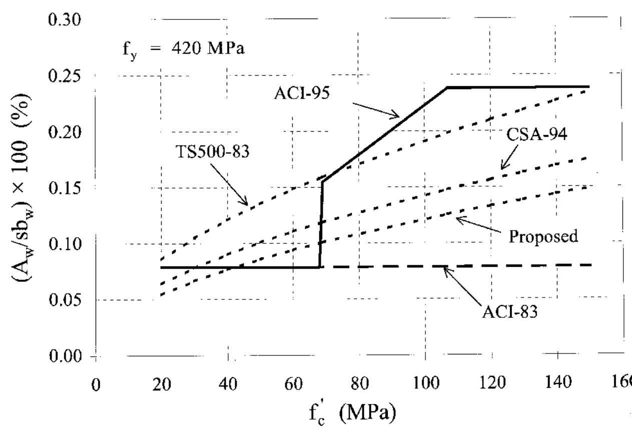

Fig. 1—Comparison of minimum shear reinforcement require-

dance to ACI 318-83, TS500-83, and the proposed equation, test results could also be used to evaluate CSA A23.3-94 and ACI 318-95 requirements.

MINIMUM SHEAR REINFORCEMENT

The beams in the test program were designed to have the minimum shear reinforcement required in ACI 318-83, TS500-83, and the equation proposed by the authors.

The minimum requirements in different design codes and the proposed equation are given below.

ACI 318-83 (ACI 318M-83)[1]

$$\left(\frac{A_v}{sb_w}\right)_{min} = \frac{1}{3f_y} \quad \text{(in SI units)} \tag{1}$$

Minimum shear reinforcement is required when design shear exceeds $0.5V_{c5}$

$$V_c = 0.167\sqrt{f'_c}\, b_w\, d \quad \text{(in SI units)} \tag{2}$$

ACI 318-95 (ACI 318M-95)[11]

Eq. (1) and (2) also appear in the 1995 ACI Building Code. However, for concrete strength greater than 69 MPa (10 ksi), it is required to multiply the right hand side of Eq. (1) by f'_c /35, leading to the following shear reinforcement. In the code, the upper limit is set as $1/f_y$

$$\left(\frac{A_v}{sb_w}\right)_{min} = \frac{1}{3f_y} \times \frac{f'_c}{35} \approx 0.0010\frac{f'_c}{f_y} \le \frac{1}{f_y} \tag{3}$$

Turkish Code 1983 (TS500-83)[3]

The TS500-83 requirement for minimum shear reinforcement is given below. As stated before, the equation given was derived by equating the magnified diagonal cracking strength to the strength of the beam with shear reinforcement

$$\left(\frac{A_v}{sb_w}\right)_{min} = 0.3\,\frac{f_{ctd}}{f_{yd}} \tag{4}$$

In the previous equation, f_{ctd} is the design tensile strength of concrete and f_{yd} is the design yield strength of the shear reinforcement. The design tensile strength of concrete and the design yield strength of the shear reinforcement are given in Eq. (5)

$$f_{ctd} = \frac{f_{ct}}{1.5} = \frac{0.35\sqrt{f'_c}}{1.5} = 0.233\sqrt{f'_c} \quad \text{(in SI units)} \tag{5}$$

$$f_{yd} = \frac{f_y}{1.15}$$

If Eq. (4) is expressed in terms of f'_c and f_y, the following equation is obtained for minimum shear reinforcement

$$\left(\frac{A_v}{sb_w}\right)_{min} = 0.0805\frac{\sqrt{f'_c}}{f_y} \quad \text{(in SI units)} \tag{6}$$

In the TS500-83, minimum shear reinforcement is required regardless of the level of shear.

Canadian Code (CSA A23.3-94)[12]

CSA requires minimum shear reinforcement when the design shear exceeds $0.5V_c$. The required minimum shear reinforcement is given below

$$\left(\frac{A_v}{sb_w}\right)_{min} = 0.06\frac{\sqrt{f'_c}}{f_y} \tag{7}$$

As can be seen, the equations given in Turkish and Canadian codes are the same except for the constants used. The TS500-83 results in approximately 34 percent more minimum shear reinforcement.

Minimum shear reinforcement requirements of different codes are compared in Fig. 1. In this comparison, yield strength of shear reinforcement is assumed to be f_y = 420 MPa (60.9 ksi).

Proposed equation

To prevent brittle failure upon first diagonal cracking, the shear strength of the beam with shear reinforcement should be greater than the diagonal cracking strength

$$V_u > V_{cr} \tag{8}$$

$$(V_s + V_c) > V_{cr} \tag{9}$$

If, for safety, V_{cr} is assumed to be $1.3V_c$, and V_s is expressed as $A_v f_y d/s$, Eq. (9) can be rewritten as

$$\left(\frac{A_v}{sb_w}\right)_{min} = 0.3\frac{V_c}{b_w d f_y} \tag{10}$$

under seismic actions.

Table 1—Properties of test specimens

Beam		f'_c, MPa	A_v/sb_w Required, percent	A_v/sb_w Supplied, percent	s, mm	(A_v/sb_w) required by ACI95 [Eq. (3)], percent	CSA94 [Eq. (7)], percent	TS500 [Eq. (4)], percent	Proposed [Eq. (10)], percent
1		2	3	4	5	6	7	8	9
SERIES56 a/d = 5 ρ_w = 0.0346†	ACI56	58	0.132(1)*	0.139	120	0.132	0.183	0.245	0.160
	TH56	63	0.166(10)	0.167	100	0.132	0.190	0.256	0.166
	TS56	61	0.251(6)	0.239	70	0.132	0.187	0.251	0.164
SERIES59 a/d = 5 ρ_w = 0.0443	ACI59	82	0.132(1)	0.139	120	0.309	0.217	0.292	0.192
	TH59	75	0.184(10)	0.187	90	0.283	0.208	0.279	0.184
	TS59	82	0.292(6)	0.279	60	0.309	0.217	0.292	0.192
SERIES36 a/d = 3 ρ_w = 0.0259	ACI36	75	0.132(1)	0.139	120	0.283	0.208	0.279	0.184
	TH36	75	0.184(10)	0.167	100	0.283	0.208	0.279	0.184
	TS36	75	0.279(6)	0.239	70	0.283	0.208	0.279	0.184
SERIES39 a/d = 3 ρ_w = 0.0307	ACI39	73	0.132(1)	0.139	120	0.275	0.205	0.275	0.185
	TH39	73	0.185(10)	0.170	80	0.275	0.205	0.275	0.185
	TS39	73	0.275(6)	0.279	60	0.275	0.205	0.275	0.185
SERIES26 a/d = 3 ρ_w = 0.0193	ACI26	70	0.132(1)	0.139	120	0.264	0.201	0.269	0.181

*Numbers in parenthesis are equation numbers (as used in paper), identifying equations used in specimen design.
†In all specimens, 4-mm-diameter stirrups with f_y = 255 MPa (37.0 ksi) were used as shear reinforcement.

Eq. (10) is the minimum shear reinforcement required. V_c can be taken from the ACI Building Code[11] (in SI units)

$$V_c = \left(0.16\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}\right) b_w d \tag{11}$$

or simply as

$$V_c = 0.17\sqrt{f'_c}\, b_w d \tag{12}$$

TEST PROGRAM

Test specimens

Thirteen beams having 150 x 360-mm (6 x 14-in.) rectangular cross sections were tested under two-point loading. In two series, a/d ratio was 3, while in the other two series, it was 5. One beam was tested with a/d = 1.9. The properties of test specimens are summarized in Table 1.

Test specimens were grouped in five series according to concrete strength and a/d ratio. Each series, except for Series 26, consisted of three beam specimens. Beams in each series were intended to have the minimum shear reinforcement required by ACI 318-83, TS500-83, and the proposed equation. Series 26 consisted of only one beam and had the minimum shear reinforcement required by ACI318-83. It was intended to use two different concrete strengths, f'_c = 60 MPa (8.5 ksi) and f'_c = 90 MPa (12.8 ksi).

Although there were no beam specimens designed to comply with the 1995 ACI minimum shear reinforcement requirements, the minimum shear reinforcement required by ACI 318-95 for concrete strength ranging from 70 to 90 MPa is not very different from that required by TS500-83 (Fig. 1). In this range, the ratio of TS500-83 minimum shear reinforcement to that of ACI 318-95 varies between 0.85 and 0.96.

In the test program, the main variables were concrete strength, shear reinforcement ratio, and ratio of shear span-to-effective depth. The dimensions and the reinforcement details of the test specimens are shown in Fig. 2.

Adequate tension reinforcement was provided in all specimens to insure shear failure prior to flexural failure. Each longitudinal bar was extended 100 mm beyond the support. To prevent premature bond failures, steel plates were welded to the end of bars for better anchorage.

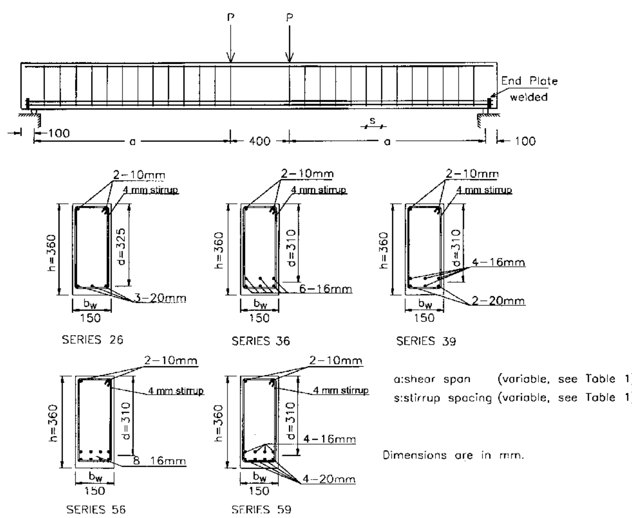

Fig. 2—Dimensions and reinforcements of test specimens.

Each series is defined by two numbers, the first indicating the a/d ratio, and the second the intended concrete strength (1/10 of the strength in MPa). For example, beams of Series 59 are tested with an a/d ratio of 5, and the intended concrete strength is 90 MPa.

Each specimen is identified by two or three letters. These letters indicate the requirement according to which the minimum shear reinforcement is designed. ACI refers to ACI 318-83, TS refers to TS500-83, and TH refers to the proposed equation.

There were some problems with the concrete mix; therefore, the concrete strengths obtained were different from the intended values as shown in Table 1. Since the minimum shear reinforcement of beams was calculated using the intended concrete strength, the shear reinforcement of test specimens was somewhat different from the minimum required values, as a result of differences in concrete strength. In Table 1, the minimum $A_v/(sb_w)$ ratios required using the actual concrete strength are given

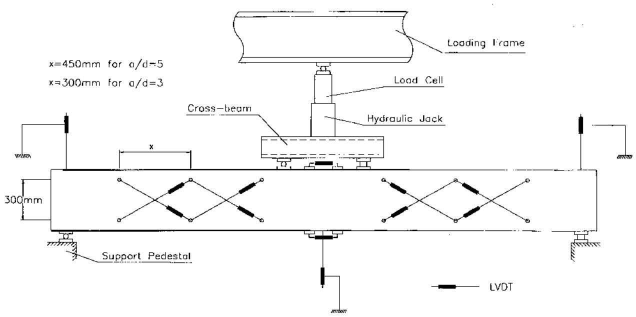

Fig. 3—Test setup and instrumentation (external).

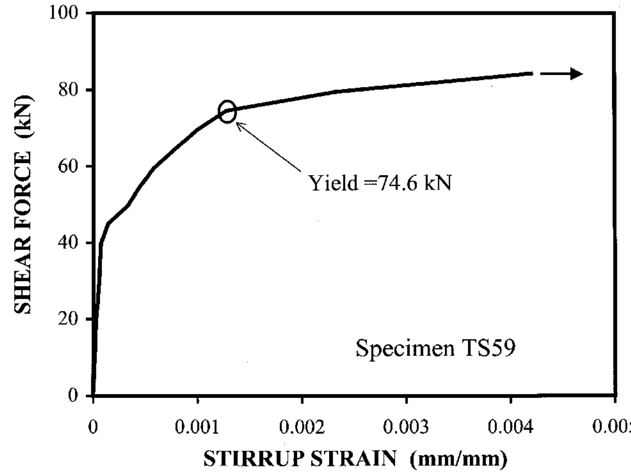

Fig. 4—Load-stirrup strain curve for TS59.

Table 2—Mix proportions

Material	f'_c = 60 MPa	f'_c = 90 MPa
Cement, kg	615	570
Water, kg	160	135
Silica fume, kg	95	113
Superplasticizer, kg	25	50
0- to 3-mm aggregate, kg	475	450
3- to 7-mm aggregate, kg	240	240
7- to 15-mm aggregate, kg	890	875

Table 3—Properties of reinforcement

Bar size, mm	Yield strength, MPa	Ultimate strength, MPa
4	255	360
10	410	560
16	450	720
20	425	680

in Column 3 (shown as "required"). Following this column, ratios provided are given in Column 4 (shown as "supplied"). Since provided values are calculated using the intended concrete strength, required and provided values are different. However, the differences are not very significant.

In the last four columns of the table, minimum $A_v/(sb_w)$ ratios required by ACI 318-95, CSA A23.3-94, TS500-83, and the proposed equation, based on actual concrete strength, are given. These columns are included in the table to enable comparison of shear reinforcement in the test beams with the required minimums.

Materials

Two different concrete mixes were used. These mixes were designed to get 28-day concrete strength of 60 and 90 MPa (8.5 and 12.8 ksi). Concrete mix proportions for 1 m^3 of concrete are given in Table 2 for each mix.

Beams of each series (three specimens) were cast together. Concrete strength was determined from 150 x 300-mm cylinder samples taken for each beam. Concrete compressive strengths given for the test specimens in Table 1 represent the average of three uniaxial tests.

Hot rolled steel bars were used both for longitudinal and shear reinforcement. Three different sizes of bars were used as longitudinal reinforcement. Table 3 shows the strength properties of the reinforcement.

Instrumentation and test procedure

Test specimens were instrumented to measure the applied loads, displacements, strains in extreme fibers at the constant moment region, and diagonal strains on the web. Strains in longitudinal reinforcement and stirrups were measured by electrical strain gages. The instrumentation (except strain gages) is shown in Fig. 3.

The net deflection of the beam at each load stage was calculated using the readings from linear variable displacement transducers (LVDTs) placed at the supports and at the midspan. Top and bottom strains measured by LVDTs placed in the constant moment region were used to calculate the curvature. Diagonal strains measured at four locations on the web (two orthogonal directions) were used to calculate shear deformations and diagonal strains.

Beams were tested in a specially built test frame. Loads were applied by a 500-kN hydraulic jack and measured by load cells. The test setup is schematically shown in Fig. 3. LVDTs, strain gages, and the load cell were connected to a data-logger that fed the data directly to a personal computer where the preliminary data processing was immediately completed to produce a load-deflection diagram.

TEST RESULTS

General

The main objective of the test program was to evaluate the adequacy of the code requirements related to minimum shear reinforcement. As stated previously, the adequacy of the shear reinforcement was investigated considering the reserve strength beyond diagonal cracking load, ductility, crack pattern, and crack width.

In this paper, reserve strength is defined as the difference between the ultimate shear and the cracking shear. Ultimate shear is the maximum shear measured during the experiment. It is not so easy to define the shear that causes the first diagonal cracking. If based on observations during the test, different observers can end up with different values. In the literature, researchers have adopted different techniques to determine the diagonal cracking shear.

In this study, diagonal cracking shear was determined from shear-stirrup strain and shear-diagonal strain curves. The shear at which the slope of these curves changed significantly was taken as the diagonal cracking shear. Stirrup strains measured in Specimen TS59 are shown in Fig. 4. The shear at which the curve became almost horizontal corresponds to yielding of the stirrup. Diagonal cracking shear predicted from stirrup strains and diagonal strains agreed quite well with each other. Ultimate and cracking shear determined experimentally are designated as V_u and V_{cr}, respectively.

It is even more difficult to define ductility for beams that fail in shear. In this study, the ratio of midspan deflection at the ultimate to the deflection corresponding to first diagonal cracking defined previously is called shear ductility index. In the following discussion, these definitions and notations are used.

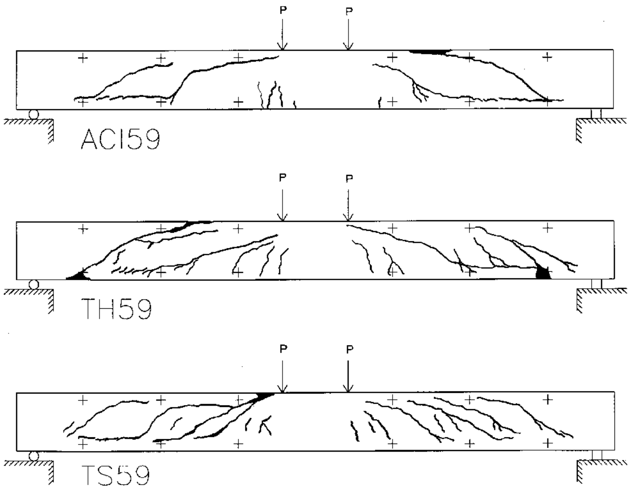

Fig. 5—Crack pattern; beams of Series 59.

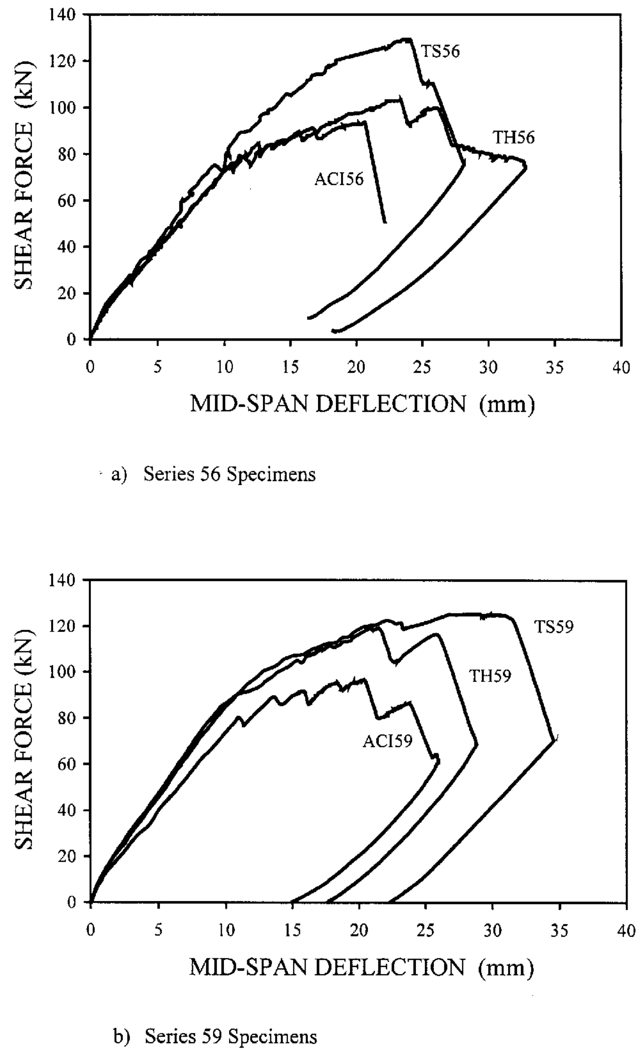

Fig. 6—Load-deflection curves; beams of Series 56 and 59.

Behavior of test specimens

The mode of failure of beams with $a/d = 5$ (Series 56 and 59) was typical diagonal tension. The yield strength of the longitudinal reinforcement was not reached in any of the beams. Flexural cracks were observed first in the maximum moment region. As the load increased, flexural cracks spread into the shear span. Some of these cracks gradually became inclined towards the load point.

Crack patterns observed at the ultimate stage of beams of Series 59 are shown in Fig. 5. Beams having the minimum shear reinforcement required by the ACI 318-83 (Specimen ACI56 and ACI59) had fewer and wider cracks compared to the others.

At the stage when a full diagonal crack developed, crack width was approximately 0.7 mm in the ACI beams. At this stage, the crack width in TS beams was approximately 0.10 mm. This difference in crack width is very significant and raises serious questions about the adequacy of ACI 318-83 requirements.

Load-midspan deflection curves for beams of Series 56 and 59 are shown in Fig. 6. Since all the beams failed in shear, no significant ductility was observed. However, comparisons were made using the shear ductility index as defined previously. This index was lowest in the ACI beams. In Series 56, the index was 2.0 for ACI56 and 2.6 for TS56. In Series 59, the index was about 1.85 for ACI59 and 3.3 for TS59.

The mode of failure of beams with $a/d = 3$ was, in general, diagonal tension. Beam ACI36 was the only test specimen in Series 36 and 39 that failed in shear-compression. First cracks observed were typical flexural cracks. At later load stages, some of these cracks inclined towards the load point. Final failure took place by opening up of one of the diagonal cracks. At this stage, some horizontal cracks appeared at the level of the tension reinforcement. However, these horizontal cracks did not extend to the end of the beam. Steel plates welded to the end of the bars seemed to be effective in preventing the anchorage failure.

Crack patterns of the specimens in Series 39 are shown in Fig. 7. ACI beams had fewer and wider cracks as compared to TH and TS beams. Crack width in ACI beams was approximately 0.42 mm, compared to 0.10 mm in TS beams.

Load-midspan deflection curves of beams of Series 36 and 39 are shown in Fig. 8. The ductility of test specimens having an a/d ratio of 3 is somewhat better when compared to those with $a/d = 5$. While the shear ductility index was 2.0 for the ACI beams, the index was approximately 4.0 for TS beams.

Beam ACI26, which had an a/d ratio of 1.9, failed by shear compression. However, the flexural reinforcement yielded just before the shear failure.

Strength of test specimens

Shear forces corresponding to diagonal tension cracking (V_{cr}) and failure (V_u) are given in Column 2 and 3 of Table 4. In Column 4, ratios of V_u-to-V_{cr} are given. This ratio is a measure of the reserve strength beyond diagonal cracking. The experimental results are compared with the calculated values in Column 7, 8, and 9. The concrete contribution V_c and the shear capacity $V_n = V_c + V_s$ were calculated using the Eq. (11.2) and (11.5) given in ACI 318-95.[11]

V_{cr}/V_c ratios given in Column 7 of Table 4 show that ACI Eq. (11.5)[11] for predicting the shear corresponding to first diagonal cracking, in general, agrees quite well with the experimental values. However, as pointed out previously, experimental prediction of the cracking shear V_{cr} is open to discussion.

Adequacy of ACI Eq. (11.2)[11] for predicting the shear capacity can be discussed by comparing the experimental and calculated capacities, i.e., V_u and V_n. For each specimen, the ratio of V_u-to-V_n is given in Column 9 of Table 4. V_n is calculated using Eq. (11.2), (11.5), and (11.15) in ACI 318-95. [11]

It seems that shear capacity V_n calculated using the ACI Building Code equations underestimates the shear capacity. This might be due to high ratios of flexural reinforcement used in the test beams. Researchers who tested high-strength concrete beams without shear reinforcement have concluded that ACI Eq. (11.5)[11] underestimates the concrete contribution V_c when the tension reinforcement ratio is higher than approxi-

Table 4—Experimental results and comparisons

Beam	Experimental			Calculated*		Comparison		
	V_{cr}, kN	V_u, kN	V_u/V_{cr}, kN	V_c, kN	V_n, kN	V_{cr}/V_c, kN	V_u/V_c, kN	V_u/V_n, kN
1	2	3	4	5	6	7	8	9
ACI56	75.0	93.6	1.25	62.1	78.6	1.21	1.51	1.19
TH56	75.4	103.5	1.37	64.5	84.3	1.17	1.60	1.23
TS56	75.6	129.2	1.71	63.6	91.8	1.19	2.03	1.41
ACI59	80.4	96.5	1.20	74.4	90.9	1.08	1.30	1.06
TH59	75.0	119.3	1.59	71.4	93.4	1.05	1.67	1.28
TS59	74.6	125.4	1.68	74.4	107.4	1.00	1.69	1.17
ACI36	80.0	105.3	1.32	71.3	87.8	1.12	1.48	1.20
TH36	83.7	140.9	1.68	71.3	91.1	1.17	1.98	1.55
TS36	85.2	155.9	1.83	71.3	99.5	1.20	2.19	1.57
ACI39	73.1	111.8	1.53	71.7	88.2	1.02	1.56	1.27
TH39	71.6	142.9	2.03	71.7	96.4	1.00	1.99	1.48
TS39	78.0	179.2	2.30	71.7	104.7	1.09	2.50	1.71
ACI26	—	343.8	—	73.7	90.9	—	4.67	3.78

*V_c and V_n are calculated using ACI-95 Eq. (11.5) and (11.2),[11] respectively.

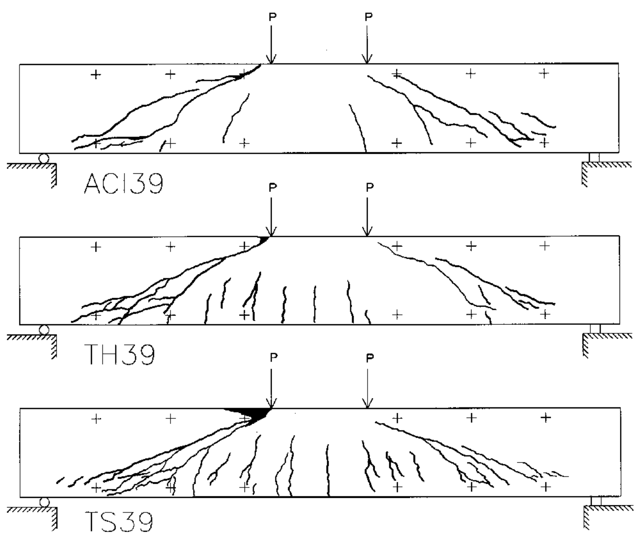

Fig. 7—Crack pattern: beams of Series 39.

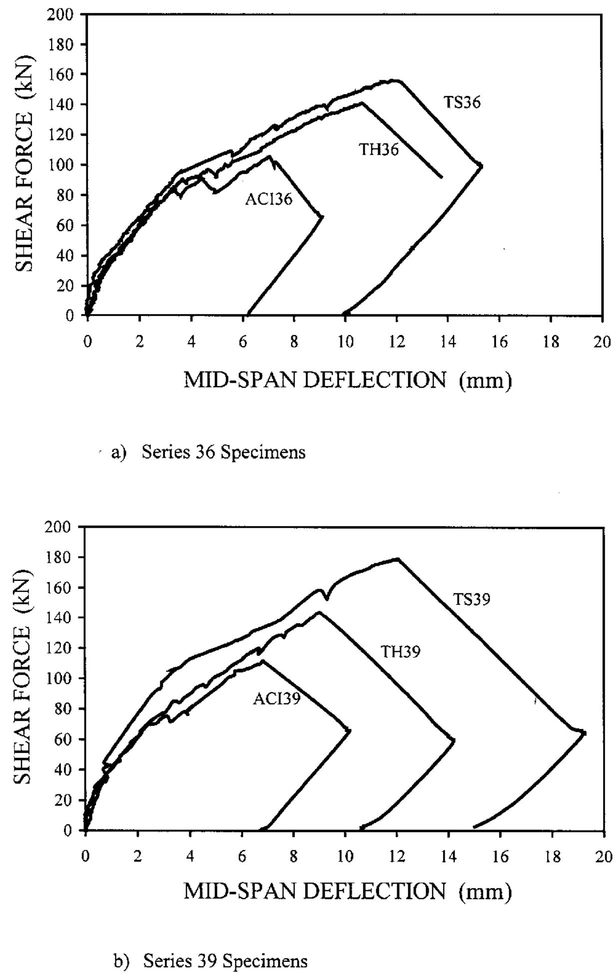

Fig. 8—Load-deflection curves: beams of Series 36 and 39.

mately 0.010.[4,5] If specimen ACI26 is excluded, this ratio in the test specimens varied between 0.026 and 0.044.

The ratios of ultimate-to-cracking shear (V_u/V_{cr}) obtained from tests are given in Column 4 of Table 4. These ratios can be used as a measure of reserve strength beyond first diagonal cracking. As can be seen from the table, even the beams designed in accordance with the minimum requirements of ACI 318-83 had 20 percent or more reserve strength. The reserve strength of beams designed in accordance with the minimum required in TS500-83 (TS specimens) was much higher, 68 to 130 percent. It should be pointed out that all TS beams had slightly less shear reinforcement than required since the concrete strengths were different from target strength.

The reserve strength of TH beams designed using the proposed equation [Eq. (10)] varied between 37 and 103 percent. Lower reserve strength corresponds to beams having $a/d = 5$.

The variation of V_u/V_{cr} with $R = r f_y / \sqrt{f'_c}$, using the test data reported in this paper, is shown in Fig. 9. In this figure, r is the shear-reinforcement ratio $A_v/(sb_w)$. To generalize the shear-reinforcement ratio, r was multiplied by f_y and divided by $\sqrt{f'_c}$, and resulting R was called the shear reinforcement index.

From Fig. 9, it can be concluded that, in general, specimens with $a/d = 3$ had higher reserve strength as compared to the ones with $a/d = 5$. CSA-94, TS500-83, and the proposed minimums are also marked on the figure. Reserve strength increased with increasing R.

In Fig. 10, the variation of V_u/V_c ratio with R is demonstrated, including the test results for high-strength concrete reported by other researchers. Since researchers do not agree on the definition of V_{cr} (diagonal cracking shear determined experimentally), it was decided to use V_c instead of V_{cr}. V_c is the concrete contribution calculated using the ACI Building Code (Eq. 11.5).[11]

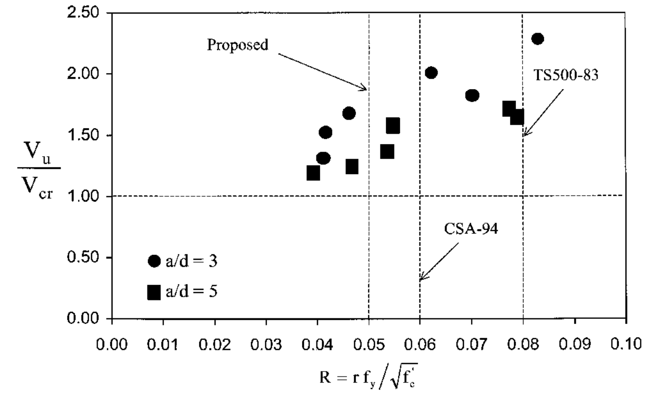

Fig. 9—Variation of V_u/V_{cr} *with shear reinforcement index.*

From Fig.10, it can be concluded that, in general, the reserve strength increases as the shear reinforcement index increases. This may be due to excessive tension reinforcement required to prevent flexural failure in specimens with high shear reinforcement index. The minimum shear reinforcement required by TS500-83, CSA-94, and the proposed equation are also marked on the figure.

In Fig. 11, to test the adequacy of Eq. (11.2)[11] given in the ACI Building Code ($V_n = V_c + V_s$) for high-strength concrete, ratios of shear capacities observed in tests (V_u) to the capacities calculated using the ACI Building Code (V_n) are plotted against R using the available test data.

From this figure, it can be concluded that ACI Eq. (11.2)[11] can be used with confidence to predict the shear capacity of beams with high-strength concrete.

Evaluation of code requirements

To evaluate the adequacy of different code requirements and the proposed equation, decisions have to be made on the acceptable limits of reserve strength, ductility, and crack width. In the absence of established limits and understanding, the authors have chosen the following criteria.

Crack width—Since crack widths greater than 0.3 mm (0.012 in.) are considered to be unacceptable for serviceability, it may be reasonable to limit the crack width to 0.3 mm at the stage when a diagonal crack fully develops. Some other researchers[10] suggest that the crack width to be considered for comparison should be the crack width corresponding to 60 percent of the nominal shear permitted by the ACI 318-95.[11]

Ductility—To make a similar proposal for the shear ductility index is not that easy because the ductility of beams failing in shear is very limited. Herein, 2.5 will be proposed as a lower limit for this index.

Reserve strength—To propose a limit for the reserve strength (V_u/V_c), one has to consider possible variation in concrete strength. The factored strength of concrete in CSA-94 is given as $\phi_c f'_c$, with $\phi_c = 0.6$. In TS500-83, the corresponding value for concrete is of the same order, $\phi_c = 1/1.5 = 0.67$. In light of ϕ values given previously, it will be reasonable to assume that one should consider the possibility of having concrete strength 50 percent greater than the specified value. Since the minimum shear reinforcement required by CSA-94, TS500-83, and by the proposed method are all expressed in terms of $\sqrt{f'_c}$, it will be reasonable to require a reserve strength of 50 percent ($V_u/V_c \geq 1.5$).

In conclusion, the authors believe that the evaluation of the minimum shear reinforcement requirements in the design codes, using the test data, should be based mainly on crack width and reserve strength criteria. Shear ductility index does not seem to be a sound criterion.

When the evaluation of the code requirements is based on these two criteria, behavior of the test specimens having ACI 318-83 minimum shear reinforcement cannot be considered as satisfactory. In these beams, crack limits have been exceeded ($0.7 > 0.3$, $0.42 > 0.3$) and, in some cases, the V_u/V_c ratio is lower than 1.5.

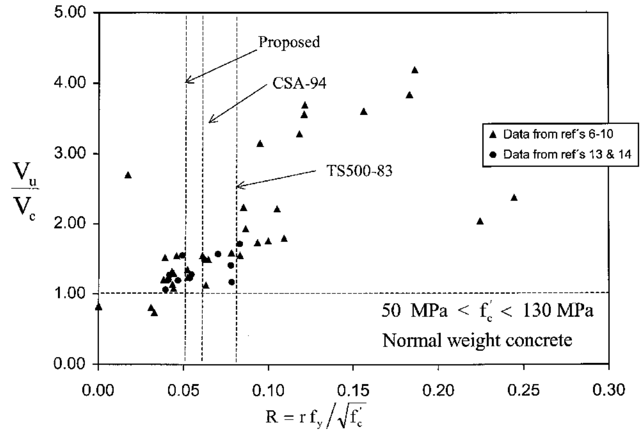

Fig. 10—Variation of V_u/V_c *with shear reinforcement index.*

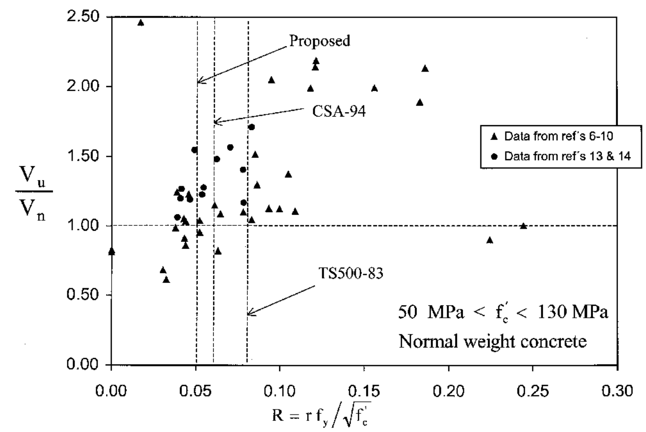

Fig. 11—Variation of V_u/V_n *with shear reinforcement index.*

Specimens that had the minimum shear reinforcement required by the TS500-83 (TS beams) and the proposed equation (TH beams) all satisfied both criteria. In these beams, the crack width was approximately 0.1 to 0.20 mm, and V_u/V_c ratios were greater than 1.5.

As can be seen from Table 1, for concrete strength higher than 69 MPa (10 ksi), the minimum shear reinforcement required by CSA A23.3-94 and ACI 318-95 are higher than the reinforcement provided in all TH beams. Therefore, it can be concluded that beams designed using the ACI 318-95 and CSA A23.3-94 requirements will also satisfy both criteria. It should be pointed out that the minimum shear reinforcement required by ACI 318-95 is much higher than that of CSA A23.3-94.

Finally, TS500-83 and CSA A23.3-94 both have one equation for all ranges of concrete strength. This can be considered as an advantage over the ACI 318-95 requirements that exhibit a rather undesirable discontinuity (Fig. 1).

CONCLUSIONS

Considering only the test results reported by the authors, the following conclusions seem to be appropriate.

1. The ACI 318-83 requirements for minimum shear reinforcement are not satisfactory when high-strength concrete is used. Test beams having ACI 318-83 minimum shear reinforcement had less reserve strength compared to the others, and the crack width observed at the stage of shear cracking was beyond the permissible serviceability limits.

2.The minimum shear reinforcement required by TS500-83 seems to be satisfactory for high-strength concrete. The reserve strength of beams designed according to TS500-83 requirements was more than 60 percent, i.e., $V_u/V_c > 1.6$. The crack width in these beams was approximately 0.1 mm when the diagonal crack fully developed.

3. The equation proposed in this paper requires approximately 37 percent less minimum shear reinforcement than TS500-83. Beams designed using this equation had at least 40 percent reserve capacity ($V_u/V_{cr} > 1.4$), and crack width remained below 0.20 mm when the diagonal crack fully developed. Specimens having the minimum shear reinforcement in accordance with the proposed equation satisfied the criteria set in this paper.

4. The equation given in CSA A23.3-94 for minimum shear reinforcement is the same as the one given in TS500-83 and the proposed equation except for the constant. CSA A23.3-94 requires 25 percent smaller minimum shear reinforcement as compared to TS500-83, but 20 percent higher than the proposed equation. Therefore, it can be concluded that beams designed using CSA A23.3-94 requirements for minimum shear reinforcement will also satisfy the criteria set in this paper.

5. Since the minimum shear reinforcement required by ACI 318-95 is higher than that required in TS500-83, it is reasonable to assume that the ACI requirements are also satisfactory. However, the discontinuity at $f'_c = 69$ MPa is neither natural nor desirable.

When an evaluation is made including the test results reported by others, the following conclusions seem to be valid.

1. Test results indicate that the ACI Building Code underestimates the concrete contribution V_c in beams having high shear reinforcement index R.

2. The reserve strength V_u/V_c increases with increasing shear reinforcement index (Fig. 9 and 10).

3. ACI Eq. (11.2)[11] for predicting the shear capacity can be used with confidence for members with higher-strength concrete (Fig. 11).

4. Reserve strength is the most important parameter in determining the minimum shear reinforcement. Then, instead of using empirical expressions, it would be more reasonable and rational to derive the equation by equating the magnified cracking shear strength to the ultimate shear strength. When this is done, the following equation is obtained

$$\min\frac{A_v}{sb_w} = C\frac{\sqrt{f'_c}}{f_y}$$

5. The coefficient C is related to reserve strength required and is 0.05, 0.06, and 0.08 in the proposed equation, CSA A23.3-94, and TS500-83, respectively (SI units). From Fig. 10, it is obvious that TS500-83 requirements result in adequate reserve strength. Further tests are needed to justify lower limits given in CSA A23.3-94 and the proposed equation.

ACKNOWLEDGMENTS

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CONVERSION FACTORS

1 mm	=	0.0394 in.
1 kN	=	0.225 kips
1 MPa	=	0.145 ksi
1 kN-mm	=	8.85 lb-in.

NOTATIONS

a	=	shear span, mm
A_v	=	area of shear reinforcement located within spacing s, mm^2
b_w	=	web thickness of beam, mm
d	=	effective depth of beam, mm
f'_c	=	compressive strength of concrete, MPa
f_{ct}	=	tensile strength of concrete, MPa
f_{ctd}	=	design tensile strength of concrete = $f_{ct}/1.5$, MPa
f_y	=	yield strength of reinforcement, MPa
f_{yd}	=	design yield strength of reinforcement = $f_y/1.15$, MPa
M	=	flexural moment, kN-mm
r	=	shear reinforcement ratio, A_v/sb_w
R	=	shear reinforcement index, $rf_y/\sqrt{f'_c}$, $\sqrt{\text{MPa}}$
s	=	stirrup spacing, mm
V_c	=	cracking shear and shear force carried by concrete (calculated using ACI Code), kN
V_{cr}	=	shear force corresponding to first diagonal cracking (experimental), kN
V_n	=	calculated shear strength (ACI) = $V_c + V_s$, kN
V_u	=	shear strength (experimental), kN
V_s	=	shear force carried by stirrups (calculated using ACI Code), kN
ρ_w	=	ratio of tension reinforcement, A_s/b_wd

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