Effect of Longitudinal Steel, Shear Span to Depth Ratio on the Shear Strength of High Strength Concrete Beams

ATTAULLAH SHAH*, AND SAEED AHMAD**

RECEIVED ON 11.08.2007 ACCEPTED ON 17.12.2007

ABSTRACT

The shear strength of NSC (Normal Strength Concrete) has been extensively researched. However, very little data is available on the shear strength of HSC beams for fc'> 6000 psi (40MPa). In this research, 35 beams with five values of longitudinal steel ? and seven values of shear span to depth (a/d) ratio have been tested, to study the effect of longitudinal steel and shear span to depth ratio. The results have been compared with the provisions of ACI 318-06 as well.

The results have shown that shear failure is anticipated when the longitudinal steel ratio for ρ>1%. The shear strength of HSC normally increases with the increase of longitudinal steel and decreases with the increase in shear span to depth ratio, as exhibited in normal strength, but the failure is more sudden and brittle. The ACI 318 provisions for shear strength of beams are normally good predictors but for a/d >5.

Key Words: Shear, Strength, Longitudinal Steel, Shear Span, Beams.

1. INTRODUCTION

Historically Ritter [1] idealized the shear cracks in reinforced concrete beams due to diagonal tension stresses as parallel chords truss with compression diagonals inclined at 45° and later Morsch [2] used the concept of truss model for the design of shear reinforcement in RC beams. The allowable shear strength of concrete was thus mainly based on 45° truss model. ACI standard specification No.23 permitted the maximum shear stress of concrete as 0.025fc', but not more than 0.41MPa [3] for beams without web reinforcement and no mechanical anchorage of longitudinal steel. However, for anchorage of 1800 hooks, the value was increased up to 0.03fc' and max of 0.62 MPa. ACI318-51, however, adopted the uniform values of max shear stress as 0.03fc' with the conditions that plain bars must be hooked and deformed bars must meet the requirements of ASTM A 305 [4]. Earlier Talbot [5] in 1909 had pointed out on the basis of tests of 106 beams, that the shear stress of concrete also depends on the longitudinal steel, shear Span to depth ratio but the same couldn't reflect in the codes at that time. Kani [6] developed a rational model called Kani's Tooth Model in 1964. However, extensive research has been carried out on the shear properties of NSC in the last three decades.

* Ph.D. Student & Project Director, Allama Iqbal Open University, Islamabad.

** Professor, Department of Civil Engineering, University of Engineering & Technology, Taxila.

The shear strength of concrete beams mainly depends on the following variables [7]:

- Depth of member or size effect.
- Shear span to effective depth a/d or moment to shear ratio.
- Longitudinal Reinforcement or dowel action.
- Axial Force.
- The tensile Strength of concrete.
- Crushing strength of the Beam web.
- Yielding of stirrups.
- The aggregates sizes leading to aggregates interlocking.
- Failure of Tension chord.
- Failure of Stirrups anchorage.
- Serviceability failure due to excessive crack width at Service load.

The loading position and pattern has direct relationship with the shear strength of RC beams. Brown, et. al. [8] has studied the effect of loading conditions on the shear strength of RC beams and have reported on the basis of 1200 tests data of beams, that the shear strength of beams subjected to uniformly distributed loads is more than the beams subjected to concentrated loads. In this research, the concentrated load at the main point has been selected mainly to study the shear span to depth ratio a/d >3.

More recent approaches for the shear design of RC beams include CFT (Compression Filed Theory), MCFT (Modified Compression Field Theory), Rotating-angle Softened Truss Model, shear friction theory, truss approaches including the STM (Truss and Tie Model) [9]. These approaches have been adopted by different building codes around the world and based on experimental evidence; empirical equations have been developed to address the problem of shear design of NSC beams. But most of the test data relates to the normal strength concrete beams [10]. Hence the use of these provisions for high strength concrete beams raises many questions. It has been reported in some research [11], that most of these provisions are un-conservative for beams with large sizes and lower values of longitudinal reinforcement. The sudden and brittle failure of HSC beams further influences the degree of reliability and safety of HSC in actual practice. Reduction in the shear capacity of NSC beams has also been reported in some latest research for slender beams with a/d>3.

Zararis [12] researched on the shear strength of slender beams with shear span to depth ratio a/d >3 and proposed a new equation to determine the shear strength of RC beams, both for the cases without and with web reinforcement.

Eric, et. al. [13] studied the influence of beam size, longitudinal reinforcement and stirrups on the shear strength of RC beams and reported that the provisions of ACI-318 are un-conservative for such beams particularly where the longitudinal steel ratio is less than 1% .i.e (ρ <1%).

Bentz, et. al. [14] worked on further rationalizing and simplifying of the shear design concept to make it more convenient for the designers. They suggested Simplified MCFT for quick use. The simplified version of the MCFT, also gave reasonably closed results when compared with the actual test data.

In recent years, STM has been used as an alternative for the design of deep beams, pile caps, corbels and brackets usually referred as disturbed (D-region) in concrete structures. This concept visualizes the RC structures as composed of compression struts and tension ties and then the structures is analyzed, to check the capacity of these struts and ties against the external loads. Stephen, et. al. [15], Hamed, et. al. [16] and Shyh-Jiann Hwang, [17] worked on rationalizing the concept of STM.

Ramirez, et. al. [18], has reported that for a constant low shear reinforcement, the overall reserve shear strength after diagonal cracking diminishes with increase in the compressive strength of concrete mainly due to loss in aggregate interlocking. But this fact according to Duthin and Carino [19] is not well acknowledged by most of the building codes, which makes these provisions less conservative and rather unsafe, particularly for practical design purposes.

Earlier the contribution of the longitudinal steel also called dowel action was assumed as independent of shear reinforcement, but later research by S.Sarkar, O.Adwan and Bose [20] proved that it was an incorrect assumption.

The stress strain curves of normal and high strength concrete exhibit very differently as shown in Fig. 1 [20].

The main variations in these curves are illustrated as follows:

(1) The stress strain curve is getting more linear with the increase of compressive strength of concrete.

(2) A relatively higher strain is observed for corresponding values of stress, which is more pronounced at the maximum stress.

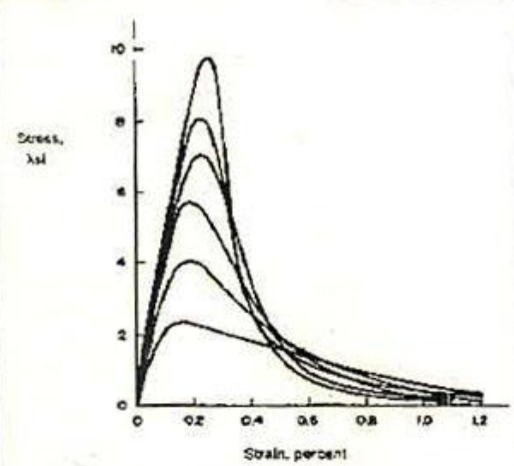

FIG 1. COMPLETE STRESS STRAIN CURVES FOR NORMAL AND HIGH STRENGTH CONCRETE [19]

(3) The descending part of the curve after the peak is steeper in case of HSC.

The high strength concrete thus exhibits ductile behavior as compared to normal strength concrete, as already explained, which can be explained as follows:

(1) The difference in rigidity between cement paste and aggregates leads to concentration of stresses at the contact zones of the two ingredients of HSC and at certain overall stress level, a distributed micro crack pattern forms at the contact points.

(2) When the overall stress level further increases, a substantial part of the increased energy is used in developing a clearer crack pattern. The stress strain curve at this stage tends to deviate from linear elastic line.

(3) With further increase of stresses, the micro crack pattern will provide an efficient re-distribution of the stress and a tough and brittle failure is obtained.

As the compressive strength of concrete increases, the difference in the strength of aggregates and cements matrix decreases and the HSC behaves like a homogenous material and the stress-strain curve becomes more linear as compared to NSC. This relatively uniform re-distribution of stresses leads to sudden failure of HSC.

Duthinh, D., and Carino, N.J., [21] while dealing with the problem of shear design of high strength concrete, pointed out the following facts:

(i) The current provision and empirical equations used for the shear design are mostly based on the research carried with concrete of 40 MPa or less. Again these equations proposed by researchers are both complex and difficult to understand. Hence there is a need to further simplify these equations for better understanding and easy application by the designers.

(ii) The minimum shear reinforcement for HSC beams needs to be rationalized to avoid brittle failure of the beams and adequate control of the shear cracks.

(iii) The relatively little role of the aggregate interlocking in HSC due to stronger matrix, the shear friction of HSC can be expected 30-35% less than the NSC.

(iv) The compression capacity of the cracked web is reduced due to transverse tension, which is sometimes referred to as "Softening of concrete", and depends on the concrete strength.

The shear capacity of concrete beams without stirrups as given by ACI-318-06 is as follows [20]:

$$Vc = \left[1.9\sqrt{fc'} + 2500\rho w \frac{Vud}{Mu}\right] bwd \leq 3.5\sqrt{fc'}bwd \quad (1)$$

or simply

$$Vc = \left(2\sqrt{fc'}\right)bwd \text{ in-lbs units} \quad (2)$$

However, no special treatment has been proposed by the ACI-318 to deal with the above cited problems.

In this research, the effect of shear span to depth a/d ratio, the longitudinal reinforcement ρ, on the shear properties of high strength concrete has been studied. 35 beams have been tested in this research with five values of longitudinal steel and seven values of shear span to depth ratio (ρ = 0.0033,0.0073,0.01, 0.015, 0.020 and a/d = 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0) The beams have been tested under concentrated load at the mid span. The details of beams and loading arrangements have been shown in Fig. 2. The test results and beam failure mechanism of beams has also been observed. The effect of longitudinal steel and shear span to depth ratio has been studied and theoretical failure values are compared with the with the provision of ACI-318 .

2. RESEARCH OBJECTIVES AND SIGNIFICANCE

The research is aimed at addressing the following questions.

(1) To study the effect of longitudinal steel and shear span to depth ratio (a/d) on the shear strength of high strength concrete beams.

(2) To check the adequacy of the ACI-318 Building Code Provisions for the shear strength of HSC beams.

3. MATERIAL

Deformed steel of 60 grade (410 MPa) of nominal sizes #3 (9.5mm), #4 (12.70mm), 6 (19 mm) and #7 (22mm) have been used and all beams were cast from the same

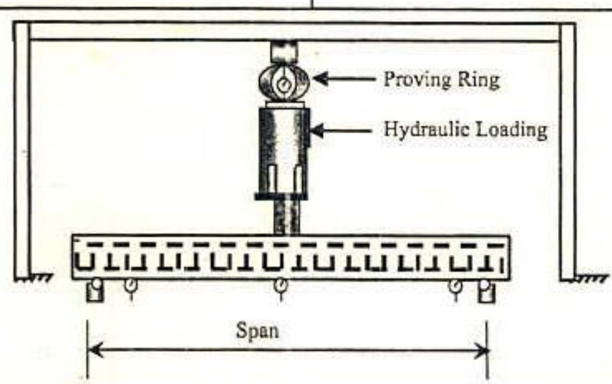

FIG 2. TYPICAL SECTION OF BEAMS AND LOADING ARRANGEMENT ARRANGEMENT

mix design shown in Table 1. The High Range water Reducers Conforming to ASTM C-494 type F standards was used at 1.70% by weight of cement to control the water cement ratio. The design strength of 7500 psi (51.71 MPa) was used; however the average concrete strength was achieved as 8200 psi (56.54 M Pa). The details of Mix design of Concrete are given in Table 1.

4. OBSERVATIONS OF STRUCTURAL BEHAVIOR OF BEAMS

The typical section and loading arrangements have been shown in Fig. 1. When tload is applied and gradually increased, flexural cracks appear in the beams which are more or less vertical in nature and mostly concentrated at the second middle third region of the beam. With further increase of load, secondary branch of crack appears from the primary cracks developed near the supports, which extends further across the section and meeting the load point at the mid span, leading to the Collapse of the beam. The nominal shear stress at the diagonal tension cracking at the development of the second branch of inclined crack is taken as the shear capacity of the beams. The typical shear failure has been shown in Fig. 3.

5. RESULTS AND DISCUSSION

The failure of the beams is caused either by flexural failure or shear failure; hence besides the generation of secondary failure crack, we also need to check the theoretical flexural capacity of the beams as well. For fc' = 7500 psi, fy = 60 ksi.

As the load P is applied at the midspan, therefore the midspan bending moment M=Pl/4, and

$$P = \frac{4M}{Span} \qquad (1)$$

TABLE 1. MIX PROPORTIONING/ DESIGNING OF HIGH STRENGTH CONCRETE

Material	Proportion
Type-I Cement	1058 lbs/yd^3 (628 kg/m^3)
Fine Aggregates	815 lbs/yd^3 (484 kg/m^3)
Coarse Aggregates	1900 lbs/yd^3 (1128 kg/m^3)
HRWR @ 1.7% by weight of cement	18 lbs/yd3(10.70 kg/m^3)
Water @ 0.25 w/c ratio	164.5 lbs/yd3(97.65 kg/m^3)
Design Compressive Strength (28 days)	7500 psi (51.71 MPa)
Actual Average Strength	
(28 days)	8200 psi) (56.54 MPa)

FIG 3. THE SHEAR FAILURE OF BEAM BS-6, WITHOUT WEB REINFORCEMENT. THE BRICK HAS BEEN PLACED UNDER THE BEAM TO AVOID THE SPLITTING OF BEAM. THE FAILURE IS MORE BRITTLE AND SUDDEN (p=1.5 %, A/D=5.5 SPAN=8FT-2")

For given section and area of steel provided, the flexural capacity of the beam

$$a = \frac{Asfy}{0.8 fc'} b \qquad (2)$$

$$My = Asfy\left(d - \frac{a}{2}\right)$$

Substituting the value of M in equation (1), we get the ultimate load

$$P = 4Asfy\left(d - \frac{\frac{a}{2}}{Span}\right) \qquad (3)$$

When the actual failure loads are compared with the theoretical flexure strength of the beams, as given in Tables 2-3, it appears that the failure of 21 beams for ρ<1%, the failure is mainly caused by the, whereas for 14 beams of ρ>1%, the failure has been caused due to shear cracks. It is also witnessed from the crack pattern in the failed beams shown in Figs. 4-5.

Effect of longitudinal steel and shear span to depth ratio on the shear stress of concrete:

As discussed in the preceding paragraph, the shear failure is more prominent when the longitudinal steel ρ>1%. The shear stress induced in the beams is worked out by dividing the failure shear by the effective shear area of bd. The shear strength are shown in Figs. 6-7.

The actual values and ACI code values are falling in very close proximity to each other and the coefficient of variation is 1.04 for the observed values. Hence the general impression that the ACI-318 code (Table 4) is conservative for the shear strength of beams without transverse steel is not exhibited by the results.

6. CONCLUSIONS

(i) The failure of the beams with longitudinal steel ratio ρ<1% is mainly caused by the flexure.

(ii) The shear failure is more anticipated in the HSC beams with ρ>1%.

(iii) The shear failure of HSC beams is sudden and brittle as compared to NSC beams. Hence great care is required for shear design of HSC beams.

TABLE 2. COMPARISON OF THEORETICAL FLEXURAL CAPACITY OF BEAMS AND ACTUALLY OBSERVED VALUES OF FAILURE LOAD FOR P<1% FOR 14 BEAMS WITHOUT TRANSVERSE STEEL

Beam Title	Steel Ratio (ρ)(%)	Span (cm)	a/d	Flexural Failure Loads (KN)		Remar
				Actual	Theory	
B1-1	0.33	152	3.0	63.43	057.20	The theoretical and values are falling very close and the failure is mainly caused by flexure for ρ_{min}=200/fy
B1-2	0.33	178	3.5	54.49	049.02	
B1-3	0.33	203	4.0	45.19	042.88	
B1-4	0.33	229	4.5	43.06	038.12	
B1-5	0.33	254	5.0	37.90	034.29	
B1-6	0.33	279	5.5	33.98	031.18	
B1-7	0.33	305	6.0	28.87	028.60	
B2-1	0.73	152	3.0	110.6	124.10	The failure is mainly caused by flexure as the values are closer
B2-2	0.73	178	3.5	102.1	106.40	
B2-3	0.73	203	4.0	93.14	093.10	
B2-4	0.73	229	4.5	84.20	082.73	
B2-5	0.73	254	5.0	75.62	074.46	
B2-6	0.73	279	5.5	66.54	067.70	
B2-7	0.73	305	6.0	48.13	062.05	

(iv) The shear strength of HSC beams increases with the increase of longitudinal steel as NSC beams.

(v) The Shear capacity of the beams decreases with the increase of shear span to depth ratio.

(vi) The provisions of ACI-318 building code for shear design are less conservative for ρ<1%.

(vii) The ACI-318 provision for shear strength of beams without transverse reinforcement are reasonably correct for ρ<1% having very little coefficient of variation.

TABLE 3. COMPARISON OF THEORETICAL FLEXURAL CAPACITY OF BEAMS AND ACTUALLY OBSERVED VALUES OF FAILURE LOADS FOR P>1% FOR BEAMS WITHOUT TRANSVERSE STEEL (21 BEAMS)

Beam Title	Steel Ratio (ρ)(%)	Span (cm)	a/d	Flexural Failure Loads (KN)		Remarks
				Actual	Theory	
B3-1	1.0	152	3.0	115.10	167.80	The failure loads are less than the theoretical values of flexure loads. Hence the failure has been caused by shear failure
B3-2	1.0	178	3.5	111.80	143.80	
B3-3	1.0	203	4.0	102.60	125.80	
B3-4	1.0	229	4.5	093.94	111.80	
B3-5	1.0	254	5.0	085.00	100.70	
B3-6	1.0	279	5.5	076.42	091.54	
B3-7	1.0	305	6.0	067.34	083.89	
B4-1	1.5	152	3.0	195.20	245.50	The difference between the observed and theoretical values is increasing and as steel, with the increase of longitudinal steel and shear failure is more obvious
B4-2	1.5	178	3.5	185.00	210.50	
B4-3	1.5	203	4.0	179.70	184.10	
B4-4	1.5	229	4.5	159.30	163.70	
B4-5	1.5	254	5.0	142.69	147.30	
B4-6	1.5	279	5.5	134.60	133.90	
B4-7	1.5	305	6.0	125.00	122.80	
B5-1	2.0	152	3.0	216.30	319.10	The difference between the two values is enormous for all seven observations, when the value of ρ=2%, which mean that the shear failure is more prominent when the longitudinal steel ratio is 2% or more
B5-2	2.0	178	3.5	195.5	273.50	
B5-3	2.0	203	4.0	188.4	239.30	
B5-4	2.0	229	4.5	161.1	212.70	
B5-5	2.0	254	5.0	152.5	191.40	
B5-6	2.0	279	5.5	135.5	174.10	
B5-7	2.0	305	6.0	125.0	159.50	

FIG 4. TYPICAL FAILURE MODE OF THE BEAM B1-1, AT MINIMUM LONGITUDINAL STEEL AND NO WEB REINFORCEMENT. TYPICAL FLEXURE FAILURE DUE CRACKS IN THE MIDDLE THIRD REGION (ρ=0.33%, A/D=3.00, SPAN=5-0")

FIG 5. TYPICAL FLEXURAL FAILURE OF BEAM B2-1 WITH OUT WEB REINFORCEMENT AT ρ<1% (ρ=0.73%, A/D=3 SPAN=5FT)

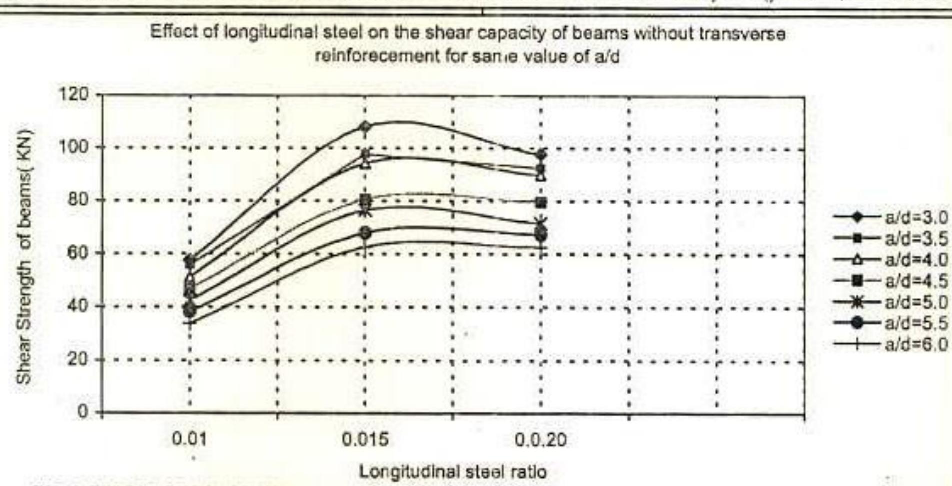

FIG 6. EFFECT OF LONGITUDINAL STEEL RATIO ON THE SHEAR STRENGTH OF BEAMS WITHOUT STIRRUPS

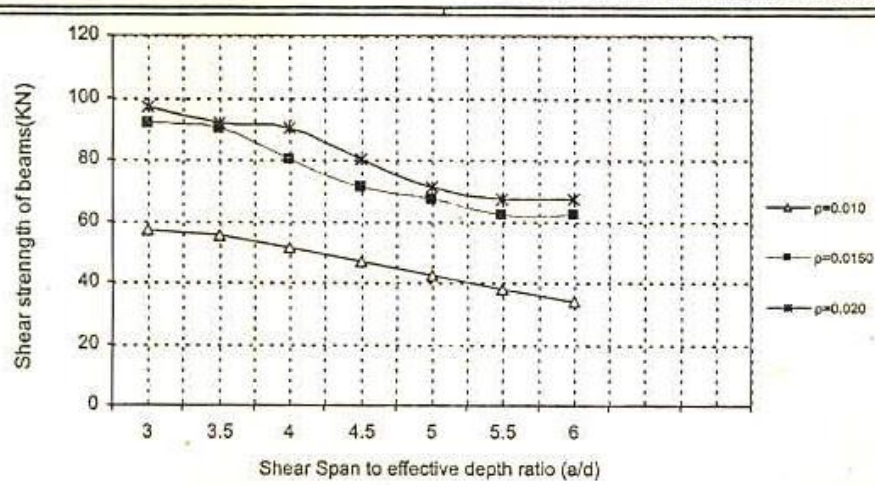

FIG 7. EFFECT OF SHEAR SPAN TO DEPTH (A/D) RATIO ON THE SHEAR CAPACITY OF BEAMS WITHOUT TRANSVERSE REINFORCEMENT FOR DIFFERENT VALUES ρ

ACKNOWLEDGEMENTS

The authors are highly grateful to the staff of Department Civil Engineering, Structural and Concrete Laboratories, University of Engineering & Technology, Taxila, Pakistan, and Higher Education Commission Pakistan, for their assistance and funding of the work under faculty research project.

TABLE 4. COMPARISON OF THE ACTUAL SHEAR STRENGTH OF THE BEAMS WITH ACI-318 PROVISIONS FOR P≥1%

Beam Title	Steel Ratio (ρ)	Span (cm)	a/d	Shear Strength(KN)		Ver/Vaci
				Actual	ACI	
B3-1	1.0	152	3.0	57.55	67.08	0.86
B3-2	1.0	178	3.5	55.9	69.21	0.81
B3-3	1.0	203	4.0	51.3	68.73	0.75
B3-4	1.0	229	4.5	46.97	68.37	0.69
B3-5	1.0	254	5.0	42.5	68.09	0.62
B3-6	1.0	279	5.5	38.21	67.87	0.56
B3-7	1.0	305	6.0	33.67	67.69	0.50
B4-1	1.5	152	3.0	97.5	70.72	1.37
B4-2	1.5	178	3.5	97.77	70.06	1.40
B4-3	1.5	203	4.0	94.21	69.52	1.36
B4-4	1.5	229	4.5	80.55	69.12	1.17
B4-5	1.5	254	5.0	76.28	68.81	1.11
B4-6	1.5	279	5.5	67.79	68.54	0.99
B4-7	1.5	305	6.0	62.54	68.32	0.92
B5-1	2.0	152	3.0	97.59	72.50	1.35
B5-2	2.0	178	3.5	92.52	71.57	1.29
B5-3	2.0	203	4.0	89.85	70.86	1.27
B5-4	2.0	229	4.5	79.66	70.28	1.13
B5-5	2.0	254	5.0	71.35	69.83	1.02
B5-6	2.0	279	5.5	67.30	69.48	0.97
B5-7	2.0	305	6.0	62.54	69.17	0.90
					Mean	0.96
					Standard Deviation	1.00
					CoV	1.04

REFERENCES

[1] Ritter, W., "Die Bauweise Hennebique", Shweizerische Bauzeitung, Volume 33, No. 7, pp. 59-61, Germany, 1899.

[2] Mörsch, E., "Reinforced Concrete Construction- Theory and Application", 5th Edition, Wittwer, Stutgart, Volume 1, Part-1, France, 1920.

[3] ACI Committee 446, "Fracture Mechanics of Concrete: Concepts, Models and Determination of Properties", ACI 446 R-91, USA, 1989.

[4] ASCE-ACI Committee 326, "Shear and Diagonal Tension", ACI Journal, Volume 59, No. 1, pp. 227-344, USA, 1962.

[5] Talbot, A.N., "Test of Reinforced Concrete Beams: Resistance of Web Stresses Series of 1907 and 1908", Bull. 29, University of Illinois, Engineering Experiment Station, Urbana, USA, 1909.

[6] Kani, G.N.J., "The Riddle of Shear Failure and its Solution", ACI Journal, Volume 61, No. 4, pp. 441-467, USA, 1964.

[7] Shioya, T., Inure, M., Nojiri, Y., Akiayama, H., and Okada, T., "Shear Strength of Large Reinforced Concrete Beams, Fracture Mechanics. Application to Concrete", SP-118, ACI, Detroit, pp. 259-279, USA, 1989.

[8] Brown, M.D., Bayrak, O., and Jisra, J.O., "Design of Shear Based on Loading Conditions", ACI Structural Journal Volume 103, No. 4, pp. 541-550, USA, July-August, 2006.

[9] ASCE-ACI Committee 445, "Recent Approaches to Shear Design of Structural Concrete", Journal of Structural Engineering, Volume 124, No. 12, pp. 1375-1417 USA, December, 1998.

10] Bazant, Z.P., and Oh, B.H., "Crack Band Theory for Fracture of Concrete", RILEM, Volume 16, No. 93, pp. 155-177, Germany, 1983.

[11] Frosch, R.J., "Behavior of Large Scale Reinforced Concrete Beams with Minimum Shear Reinforcement", ACI Structural Journal, USA, November-December, 2000.

[12] Zararis, P.D., "Shear Strength and Minimum Shear Reinforcement of Reinforced Concrete Slender Beams" ACI Structural Journal, Volume 100, No. 2, pp. 203-215, USA, March- April, 2003.

[13] Eric, J.T., and Frisch, R.J., "Influence of Beam Size, Longitudinal Reinforcement and Stirrups on the Concrete Shear Strength", ACI S Journal, Volume 99, No. 5, USA, September-Octotober, 2002.

[14] Bentz, E.C., Vecchio, F.J., and Collins, M.P., "Simplified Modified Compression Field Theory for Calculating Shear Strength of Reinforced Concrete Elements", ACI Structural Journal, Volume 103, No. 4, USA, July-August, 2006.

[15] Stephen, J.F., and Adnan, J.M., "Evaluation of Efficiency Factor Models Used in Strut and Tie Modeling of Non-Flexural Members", ASCE Journal of Structural Engineering, Volume 128, No. 5, pp. 569-577, USA, May, 2002.

[16] Hamed, M.S., "The Micro Truss Model: An Innovative Rational Approach for Reinforced Concrete", Journal of Advanced Concrete Technology, Concrete Institute, Volume 2, No. 1, pp. 77-87, Japan, 2004.

[17] Hwang, S.J., Lee, H.J., "Strength Prediction for Discontinuity Regions by Softened Strut-and-Tie Model", Journal of Structural Engineering, Volume 128, No. 12, pp. 1519-1526, USA, December, 2002.

[18] Ramirez, J.A., and Breen, J.E., "Evaluation of Modified Truss Model Approach in Shear", ACI 88 Structural Journal, Volume 5, pp. 562-571, USA, 1999.

[19] Duthinh, D., and Carino, N.J., "Shear Design of High-Strength Concrete Beams: A Review of the State-of-the-Art", Building and Fire Research Laboratory, National Institute of Standards and Technology, USA, 1996.

[20] Sarkar, S., Adwan, O., and Bose, "Shear Stress Contribution and Failure Mechanisms of High Strength Concrete Beams", Material and Structures, RILEM, Volume 32, Germany, 1999.

[21] Wang, P.T., Shah, S.P., and Naaman, A.E., "Stress Strain Curves of Normal and Light Weight Concrete in Compression", Proceedings of ACI Journal, Volume 75, No. 10, pp. 603-61, USA,1978.