AN ANALYTICAL INVESTIGATION ON SHEAR STRENGTH OF HIGH PERFORMANCE CONCRETE BEAMS

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AN ANALYTICAL INVESTIGATION ON SHEAR STRENGTH OF HIGH PERFORMANCE CONCRETE BEAMS

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Patel Jignesh I.
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Guide
Prof. Himat T. Solanki

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CERTIFICATE

This is to certify that the Major Project entitled "**An Analytical Investigation on Shear Strength of High Performance Concrete Beams**" submitted by **Mr. Patel Jignesh I. (07MCL010)**, towards the partial fulfillment of the requirements for the degree of **Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design)** of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Although High-Strength Concrete has been increasingly used in the construction industry during the last few years, An increase in the strength of concrete is directly associated with an improvement in most of its properties, in special the durability, but this also produces an increase in its brittleness and smoother crack surfaces which affects significantly the shear strength.

Performed experimental work of four beams using two point loading condition to conclude the effect on variation of longitudinal reinforcement ratio.

An analytical investigation on Shear Strength of High Performance Concrete (HPC) beams with vertical reinforcement of stirrups was carried out. The analytical investigation was involved in a development of theory based on the truss analogy, as well as capable of predicting the response and shear strength of beams.

The analytical study covered a number of parameters including concrete cover to shear reinforcement cage, shear reinforcement ratio, longitudinal tensile steel ratio, overall beam depth, shear span to depth ratio and concrete compressive strength.

The theory predicted the shear strength of the beams in present study well. The analytical study verified with previously available experimental result. Apart from this, comparisons of shear strength were also made with the predictions by the shear design provisions considering the Indian Standard IS 456:2000, American Concrete Institute Building Code ACI 318-05, and Euro code EC2 Part I.

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ABBREVIATION NOTATION AND NOMENCLATURE

a	Shear span of a beam (Theory)
A_g	The area of longitudinal tension reinforcement, mm^2
A_{sl}	Total cross-sectional area of longitudinal steel (Theory)
A_{slM}	Cross-sectional area of longitudinal tensile steel attributed to flexure (Theory)
A_{slV}	Cross-sectional area of longitudinal tensile steel attributed to shear (Theory)
A_{st}	Cross-sectional area of transverse steel (Theory)
A_{sv}	Area of shear reinforcement within spacing s
A_{sw}	Is the cross-sectional area of shear reinforcement (EC2 part 1)
b_v	Effective width of a beam for shear (Theory)
b_w	Web width, mm
b_w	Effective width of a beam for shear (ACI 318-05, EC2 part 1)
D	Over all depth of a beam
d	Nominal effective depth of a beam taken from the extreme compression fibre to the centroid of the tensile force of the longitudinal tensile reinforcement
d_o	Distance from the extreme compression fibre to the centroid of the outer most layer of tensile reinforcement
d_v	Effective depth of a beam for shear taken as 0.9 d_o
d_v	Lever arm resisting flexural moment (usually equal to 0.9d)
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
f'_c	Characteristic concrete compressive cylinder strength
f_{cd}	Characteristic concrete compressive cylinder strength (EC2 part 1)
f_{ck}	Characteristic compressive strength of concrete, N/mm^2
f_{sl}	Longitudinal steel stress (Theory)
f_{sly}	Longitudinal steel yield stress (Theory)
f_{st}	Transverse steel stress (Theory)
f_{sty}	Transverse steel yield stress (Theory)
f_{sv}	Yield strength of shear reinforcement
f_y	Yield stress of shear reinforcement
f_{ywd}	Yield stress of shear reinforcement (EC2 part 1)
L	Total length of a beam
M	Moment at the critical section of a beam (Theory)

M_u	Moment at a section of a beam (ACI 318-05)
N	Axial force in beam (Theory)
s	Spacing of stirrups
V	Shear force at the critical section of a beam (Theory)
v_1	Strength reduction factor for concrete cracked in shear (EC2 part 1)
V_{Base}	Total base shear
V_c	Concrete contribution to shear (ACI 318-05)
V_e	Test shear strength of a beam
V_n	Predicted shear strength of a beam (ACI 318-05)
$V_{RD,max}$	Design value of the maximum shear force
V_s	Steel contribution to shear (ACI 318-05)
V_u	Shear force at a section of a beam (ACI 318-05)
z	Lever arm of internal forces
α	Angle of inclination (which varies between 30 and 60 degrees) of the Diagonal compressive struts to the longitudinal axis of the beam.
β	Angle of inclined stirrups to the longitudinal axis of the beam
ς	Stress and strain softening factor (Theory)
θ	Angle of inclination of the principle compressive stress direction with respect to the longitudinal axis of a beam (Theory)
ρ_l	longitudinal steel reinforcement ratio $A_{slV}/(0.9\ b_v\ d_o)$
ρ_l	nominal longitudinal tensile steel reinforcement ratio $A_{sl}/b_w d$ (EC2 Part 1)
ρ_l	nominal longitudinal tensile steel reinforcement ratio $A_{sl}/b_v d_o$
ρ_t	transverse steel reinforcement ratio (Theory)
ρ_w	nominal longitudinal tensile steel reinforcement ratio (ACI 318-05)
σ_d	principal concrete compressive stress (Theory)
σ_l	average concrete stress in the direction of the longitudinal axis of a beam
σ_r	principal concrete tensile stress (Theory)
σ_t	average concrete stress in the direction transverse to the longitudinal axis of a beam (Theory)

1. INTRODUCTION

1.1 GENERAL

Normal concrete contain the basic element of coarse aggregate, fine aggregate, cement and water. With addition such as silica fume, fly ash, GGBFS (Ground Granulated Blast Furnace Slag) and superplasticiser, the strength and performance of concrete can be improved. This has brought about the special term “high strength concrete (HSC)” and “high performance concrete (HPC)”.

High performance concrete has properties which satisfy certain performance criteria. In 1990, these properties have been defined by the Strategic Highway Research Program (SHRP) as follows:

1) It shall have one of the following strength characteristics:
    - 4-hour compressive strength ≥ 21 MPa (3,000 psi) termed as very early strength concrete (VES), or
    - 24-hour compressive strength ≥ 34 MPa (5,000 psi) termed as high early strength concrete (HES), or
    - 28-day compressive strength ≥ 69 MPa (10,000 psi) termed as very high strength concrete (VHS).
2) It shall have a durability factor > 80% after cycles of freezing and thawing.
3) It shall have water-to-cementitious materials ratio ≤ 0.35.

The American Concrete Institute (ACI) defines HPC as “concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing and curing practices”. These requirements may involve qualities of the following:

- Ease of placement and completion without segregation
- Long-term mechanical properties
- Early-age strength
- Toughness
- Volume stability
- Long life in severe environments

HSC can be considered as HPC if it satisfies the above requirements for its application. In most practical case, HPC actually leads to HSC. In this study, there is no clear difference between these two terms and HPC is used to adequately represent HSC as well.

Concrete of higher strength have been produced with the progression of time since its early history. Commercial concrete with compressive strength of 30 MPa was available in the 1950s and during recent times, Nowadays, concrete with a 28 days curing and has characteristic cube strength of 60 MPa and above will be considered as a high strength concrete.

HPC with 140 MPa is currently being use in High rise structure in USA and Europe. An example of the use of HSC is in construction of the Petronas Twin Towers (1998) at Kuala Lumpur City Center which high early strength about 15 MPa were achieved within 12 hours after casting (Fig. 1.1).

In India, HPC of the strength 60 MPa was used for the first time for the construction of containment dome at Kaiga (Fig. 1.2) and Rajasthan Atomic Power Projects.

Fig. 1.1 Petronas Twin Towers, Kuala Lumpur

Fig. 1.2 Containment Dome, Kaiga

1.2 OBJECTIVE OF STUDY

Much study has been carried out with respect to shear in concrete beams recent tests have focused on HPC. HPC has been accepted as a new material and is especially different from conventional concrete which has been used extensively over the past few decades.

The shear design provision contained in current code IS 456:2000[1] is mainly based on test data from concrete with compressive strength less then 40 MPa. The significant difference in behavior (shear strength) between high performance concrete (HPC) and normal strength concrete (NSC). In HPC increase the strength of concrete is directly associated with an improvement in most of its properties, in special the durability, but this also produces an increase in its brittleness and smoother crack surfaces compare to those in NSC which affects contributions to shear due to aggregate interlock action.

Furthermore, bond action between reinforcing bar in NSC and HPC may be different. Therefore, there is a need to examine the shear of HPC beams and the design formula in the current codes should be updated accordingly.

In order to achieve these general aims, the following specific objectives are proposed:

i. To study the behavior of reinforced HPC beams with vertical stirrups subjected to combined bending moment and shear force.

ii. To evaluate the performance of the shear provisions in the current Indian Standard IS 456:2000 [1] and in other codes and to study the correlation with previously available experimental result on HPC beams.

iii. To propose a simplified shear design method (Strut-and Tie Model) for predicting the failure shear strength for high-strength concrete beams, including a proposal for the minimum amount of transverse reinforcement.

iv. To validate the theory, four beams will be tested to evaluate the effect on variation of longitudinal reinforcement ratio by using two point loading condition.

1.3 SCOPE OF THE WORK

The experimental work involved two point loading condition with variation of longitudinal reinforcement ratio.

The investigation involved comprising 133 beams covered a number of parameters including concrete cover to shear reinforcement cage, shear reinforcement ratio, longitudinal tensile steel ratio, overall beam depth, shear span to depth ratio and concrete compressive strength. As far as the analytical work concerned, a theory based on Stress theory with Strut-and Tie Model was developed to predict the response and shear strength of reinforced concrete beams. In addition, the shear design provisions in several codes of practice (IS 456:2000 [1], ACI 318-05 [2], and EC2 Part I [3]) were examined in the light of the test results.

1.4 ORGANIZATION OF MAJOR PROJECT

The thesis is divided into six chapters. In **Chapter 1** general aspects of HPC discussed. It also includes objectives of study and scope of work.

Chapter 2 describes the literature review base on previous research work related to the topics. Both analytical and experimental components of past research are described.

Detailed of proposed theory of the present work along with example is discussed in **Chapter 3**.

Chapter 4 describes the experimental work. Material and equipment used in the test programme, the specimen details along with photographs and the test procedure used are reported hear.

Chapter 5 deals with the available result in the literature are compared with analytical predictions, and the shear strengths are also compared with code predictions.

Finally, **Chapter 6** describes the summary of this investigation and presents general and specific conclusions, together with Recommendations for future research.

Complete test data and crack pattern of test beam are given in **Appendix - A** and **Appendix - B** respectively.

Also List of useful websites and list of paper communicated are given in **Appendix - C** and **Appendix - D** respectively.

2. LITERATURE SURVEY

2.1 GENERAL

In the past, many researches have been going on shear strength of High-Performance Concrete (HPC) beams their work presented in many ways. This chapter focuses on recent theoretical concepts for shear in reinforced concrete beams. The review of various papers related to the shear strength of HPC beams is also described which is very helpful to understand the way of study.

A brief review of the shear provisions in the Indian Standard IS 456:2000 [1], American Concrete Institute Building Code ACI 318-05 [2], and Eurocode EC2 Part I [3] are also discussed.

2.2 EXPERIMENTAL STUDY ON SHEAR STRENGTH OF HPC BEAMS

Beams with shear reinforcement tested recently by other researchers are considered hear. The details and shear capacities of these beams are given in the following subsection.

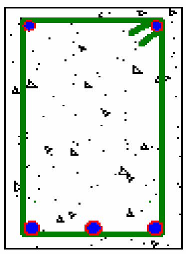
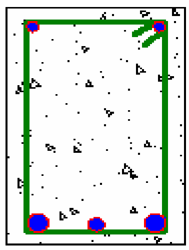
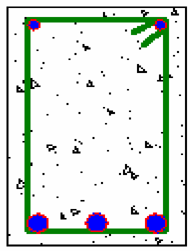
Shear failure in a beam without web reinforcement is sudden and brittle. Therefore, it is necessary to provide a minimum amount of shear reinforcement, which must prevent sudden shear failure on the formation of first diagonal tension cracking and, in addition, must adequately control the diagonal tension cracks at service load levels.

N. Subramanian [30] studied the formula available in various codes of practices in different countries for predicting the shear strength of concrete beams are empirical and are based on tests conducted on normal strength concrete (NSC) beams. He compared the formulae given in various codes with available data on shear strength of HSC beams. From comparison, it is found that the Indian Standard code formula gives very conservative results in estimating the shear strength contribution of NSC and HSC while the provision for shear strength contained in the American, the British and the Norwegian codes are conservative but they may be adopted for the safe design of HPC beams.

2.2.1 Sarsam and Al-Musawi [5]

Sarsam et al. tested three beam series of ten beams reinforced with 4 mm diameter high yield cold-drawn smooth wire stirrups. Overall dimension of the beams were 180 mm x 270 mm as shown in Table 2.1. The beam series of A, B and C corresponded to members with longitudinal steel reinforcement of 3 – 20 mm, 2 – 25 mm and 1 – 16 mm, and 3 – 25 mm diameter deformed bars respectively. The shear spans varied from 580 to 940 mm with M/Vd_o ratios of 1.50 and 3.00, and the specimens were loaded with two symmetrically placed point loads 400 mm apart. The complete details of these beams are given in Table 2.2.

Table 2.1 Cross-Sectional of Reinforced Concrete Beams Tested by Sarsam and Al-Musawi

	Series A 	Series B 	Series C 
Dimensions (mm)	180 x 270	180 x 270	180 x 270
Stirrups Yield Stress (MPa)	4 mmØ 820	4 mmØ 820	4 mmØ 820
Conc. Cover (mm) to Long. Steel	25	25	25
Long. Steel Yield Stress (MPa)	2-10mmØ(T); 2-25mmØ+ 1-16mmØ(B) 450(T); 543,525(B)	2-10mmØ(T); 3-25mmØ(B) 450(T); 543(B)	2-10mmØ(T); 3-25mmØ(B) 450(T); 543(B)

The primary objectives of this study were as follows:

- To study the influences of variables such as the concrete compressive strength, shear reinforcement ratio, longitudinal steel reinforcement ratio and *a/d* ratio on the shear strength.

- To examine the shear strength of concrete beams made from HSC and conventional concrete.
- To compare test shear strengths with predictions from the ACI, Canadian, New Zealand and British codes of practice.

Table 2.2 Details of Reinforced Concrete Beams Tested by Sarsam and Al-Musawi

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
AL2-H	75.3	180	270	235	235	940	4.00	943	495	0.00093	150	820	122.6
AS2-H	75.5	180	270	232	232	580	2.50	943	495	0.00093	150	820	201.0
AS3-H	71.8	180	270	235	235	588	2.50	943	495	0.00140	100	820	199.1
BL2-H	75.7	180	270	233	233	932	4.00	1181	540*	0.00093	150	820	138.3
BS2-H	73.9	180	270	233	233	583	2.50	1181	540*	0.00093	150	820	223.5
BS3-H	73.4	180	270	233	233	583	2.50	1181	540*	0.00140	100	820	228.1
CL2-H	70.1	180	270	233	233	932	4.00	1470	543	0.00093	150	820	147.2
CS2-H	70.2	180	270	233	233	583	2.50	1470	543	0.00093	150	820	247.2
CS3-H	74.2	180	270	233	233	583	2.50	1470	543	0.00140	100	820	247.2
CS4-H	75.7	180	270	233	233	583	2.50	1470	543	0.00186	75	820	220.7

Note: * refers to an average yield stress representative of 2-25 mmØ bars (543 MPa) and 1-16 mmØ bar (525 MPa).

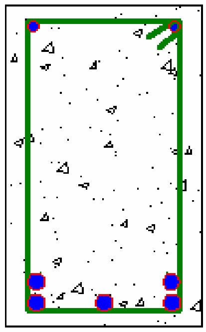
From this research, Sarsam and Al-Musawi were able to conclude that:

- Both the ACI and Canadian codes were conservative.
- The results suggested that size or depth factor did not have a significant effect on the shear strength of beams with shear reinforcement.
- Increasing the concrete compressive strength up to about 80 MPa did not reduce the safety factor (i.e., ratio of test shear strength to predicted shear strength) for the ACI code predictions.

2.2.2 Johnson and Ramirez [7]

Out of three beams, two beams with shear reinforcement of 6.4 mm diameter deformed bar were tested by Johnson and Ramirez are considered in this report. The beam had a cross-section of 305 mm x 610 mm (Table 2.3). Two symmetrically placed concentrated loads were applied on each beam which had a clear span of 4254 mm. The longitudinal steel reinforcement was also maintained the same for all the beams. Other details of these beams are given in Table 2.4.

Table 2.3 Cross-Sectional of Reinforced Concrete Beams Tested by Johnson and Ramirez



Dimensions (mm)	305 x 610
Stirrups Yield Stress (MPa)	6.4 mmØ 479
Conc. Cover (mm) to Long. Steel	38(sides); 25(top and bottom)
Long. Steel Yield Stress (MPa)	2-#9(T); 5-#10(B) 540(T); 525(B)

Note: #9 : 29 mm (1.125 inch) diameter bar.
#10: 32 mm (1.250 inch) diameter bar.

Table 2.4 Details of Reinforced Concrete Beams Tested by Johnson and Ramirez

Beam Mark	**f'_c (MPa)**	**b_v (mm)**	**D (mm)**	**d (mm)**	**d_o (mm)**	**a (mm)**	**a/d_o**	**A_{sl} (mm^2)**	**f_{sly} (MPa)**	**ρ_t**	**s (mm)**	**f_{sty} (MPa)**	**V_e (Exp) (kN)**
3	72.3	305	610	539	562	1670	2.97	3960	525	0.00078	267	479	262.7
4	72.3	305	610	539	562	1670	2.97	3960	525	0.00078	267	479	315.9

The primary objective of this study was to evaluate the adequacy of the minimum amount of shear reinforcement in beams with relatively high concrete compressive strength according to the ACI 318-83 code provisions.

They derived the main conclusions from their study were:

- The number of inclined cracks increased with increase in amount of shear reinforcement which indicated greater redistribution of internal forces.

- The reserve capacity provided by shear reinforcement increased significantly as the amount of shear reinforcement was increased from the minimum amount required to twice this amount.
- The overall reserve shear strength after diagonal tension cracking diminished with the increase in greater compressive strength for beams with the same minimum amount of shear reinforcement.

2.2.3 Elzanaty, Nilson and Slate [20]

Elzanaty et al. tested three beams with web reinforcement. One of them was made of high-strength concrete. According to their tests, beam with shear reinforcement of 6.4 mm diameter smooth round bar was tested by these investigators. The beam was 178 mm x 305 mm in cross-section (Table 2.5). Two symmetrically placed point loads were applied on the beams. The M/Vd_o ratio and shear reinforcement ratio 3.00 and 0.00171 respectively. Other details for these beams are summarised in Table 2.6.

Table 2.5 Cross-Sectional of Reinforced Concrete Beams Tested by Elzanty et al.

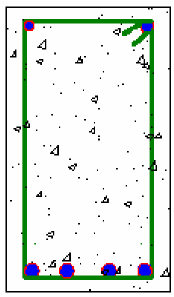
	Beam G4 
Dimensions (mm)	178 x 305
Stirrups Yield Stress (MPa)	6.4 mmØ 379
Conc. Cover (mm) to Long. Steel	25
Long. Steel Yield Stress (MPa)	2-6.4mmØ(T); 4-22mmØ(B) 379(T); 434(B)

Table 2.6 Details of Reinforced Concrete Beams Tested by Elzanty et al.

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	ρ_l	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
G4	62.8	178	305	268	268	1072	4.00	0.033	434	0.00171	210	379	150

Beam G4 is part of eighteen beams tested by Elzanty et al. (the other fifteen beams were without shear reinforcement). The main objective of testing this beam was to determine the influence of the concrete compressive strength on the shear strength.

From this study, the following conclusions were drawn:

- The ACI code was more conservative at greater concrete compressive strength.
- Shear strength of beam increased with greater concrete compressive strength.
- Shear failure was more sudden and the cracked surfaces were smoother for higher concrete compressive strength.

2.2.4 Watanabe [10]

Seven beams with HSC in the range of 73.5 to 111.0 MPa were used in this investigation. Each specimen had a central test region which was 1100 mm long and two anchorage regions of 550 mm long effective for the bending moment. All the test regions of the beams were 150 mm x 300 mm. Out of seven beams, cross-section of beam B-6 and PB-4 are shown in Table 2.7 are considered in this report. The set-up can be visualised as being made up of two identical and symmetrical simply supported bending moments, each due to a concentrated load at mid span; connected to each other but on opposite sides of the beams. Full details of the beams are provided in Table 2.8. Equal top and bottom longitudinal steel reinforcement were used in all beams.

The main objectives of this research were:

- To determine the effects of concrete compressive strength, amount of shear reinforcement and amount of longitudinal steel on the shear strength of beams.

- To investigate the shear design methods for beams with HSC.

Table 2.7 Cross-Sectional of Reinforced Concrete Beams Tested by Watanabe

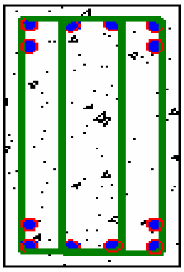
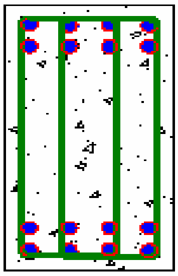
	Series B 	Series PB 
Dimensions (mm)	150 x 300	150 x 300
Stirrups Yield Stress (MPa)	6 or 8 mmØ 290; 784	6 or 8 mmØ 290; 784
Conc. Cover (mm) to Long. Steel	22	22
Long. Steel Yield Stress (MPa)	8-16mmØ(T); 8-16mmØ(B) 996(T); 996(B)	8-16mmØ(T); 8-16mmØ(B) 996(T); 996(B)

Table 2.8 Details of Reinforced Concrete Beams Tested by Watanabe

Beam Mark	**f'_c (MPa)**	**b_v (mm)**	**D (mm)**	**d (mm)**	**d_o (mm)**	**A_{sl} (mm^2)**	**f_{sly} (MPa)**	ρ_t	**s (mm)**	**f_{sty} (MPa)**	**V_e (Exp) (kN)**
PB-4	111.0	150	300	240	240	1600	996	0.02640	50	727	730.0
B-6	73.5	150	300	240	240	1200	953	0.00570	75	411	291.0

Note: A_{sl} is one-half the total longitudinal steel area.

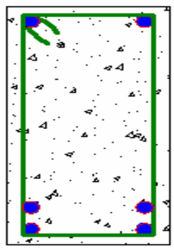
The following were the conclusions drawn from the tests:

- The ACI code gave over-conservative predictions of shear strength for beams with large amount of shear reinforcement.

2.2.5 Xie, Ahmad, Yu, Hino and Chung [4]

Xie et al. tested six beams reinforced with 6.4 mm diameter smooth bars were part of testing programmed. Three of them were made of high-strength concrete. These beams were loaded by a concentrated load at mid span. The concrete compressive strength (f'_c) ranged from 94 to 109 MPa. All the beams were 127 mm x 254 mm in cross section as shown in Table 2.9. The experiment investigation was conducted to study the ductility of normal and HSC beams. Variables such as concrete compressive strength, shear span-to-depth ratio and amount of reinforcement were considered in these tests. Other details of the beam are given in Table 2.10.

Table 2.9 Cross-Sectional of Reinforced Concrete Beams Tested by Xie et al.

	Series NHW 
Dimensions (mm)	127 mm x 254 mm
Stirrups Yield Stress (MPa)	6.4 mmØ 324
Long. Steel Yield Stress (MPa)	2-#13(T); 4-#19(B) 421

A summary of the findings attained by Xie et al. from their tests is as follows:

- Beam with shear reinforcement gave stable and reproducible post-deck load versus midspan deflection characteristics when tested in an energy-absorbing stiff testing machine.
- For beams with *a/d* ratio of 3 (i.e., NHW-3), the shear ductility index (area under the load-deflection curve) was not significantly influenced by an increase in concrete compressive strength.

- High strength concrete beams with *a/d* ratio of 3 (i.e., NHW-3), demonstrated near plastic post-peck response when twice the minimum amount of shear reinforcement according to ACI 318-89 was used.

Table 2.10 Details of Reinforced Concrete Beams Tested by Xie et al.

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
NHW-2	99.7	127	254	195	195	400	1.9	1135	421	0.0051	99	324	178.6
NHW-3	103.4	127	254	195	195	600	2.8	1135	421	0.0051	99	324	102.6
NHW-4	104.0	127	254	195	195	800	3.8	1135	421	0.0051	99	324	94.0

Note: Clear concrete cover of 25 mm (1 inch) was assumed at the bottom of the beams.

2.2.6 Roller and Russell [9]

Ten beams were tested with concrete compressive strength ranging from 72.4 to 125.3 MPa. Out of ten, six beams considered in this report. These beams had rectangular cross-section of 356 mm x 635 mm to 356 mm x 743 mm (beam 2) in the first series, and 457 mm x 870 mm (beam 6 to 10) in the second series as shown in Table 2.11. Except for the shear reinforcement in beam 1, all the steel bars conformed to ASTM A615 (Grade 60). Swedish 6 mm stirrups were used in beam 1 only. No top steel was used in any of the beams. All the beams were loaded with a central point load in a simply supported span. The shear spans were 1397 mm in the first series and 2286 mm in the second series. Full details of these beams are provided in Table 2.12. The beams were tested open stirrups which may have resulted in the lower than predicted shear strengths in beams 8 and 9 due to poor anchorage of the stirrups.

The main objectives of these tests were as follows:

- To study the effect of the shear reinforcement ratio on the shear strength of the beams.
- To study the effect of the concrete compressive strength on the shear strength of the beams.
- To consider the adequacy of the ACI 318-83 code requirement for the minimum amount of shear reinforcement extended to HSC beams.

Table 2.11 Cross-Sectional of Reinforced Concrete Beams Tested by Roller and Russell

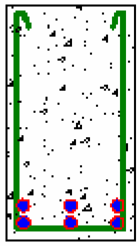
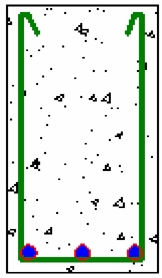
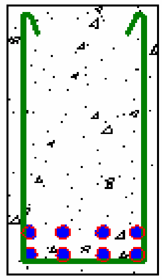
	Beam 2 	Beam 6 	Beam 7 
Dimensions (mm)	457 x 870	356 x 679	356 x 679
Stirrups Spacing (mm) Yield Stress (MPa)	9.5 mmØ 197 445	12.7 mmØ 165 448	12.7 mmØ 165 448
Long. Steel Yield Stress (MPa)	4-31.8mmØ; 4-31.8mmØ 483	3-34.9mmØ; 3-34.9mmØ 431	3-34.9mmØ; 3-34.9mmØ 431

Table 2.11 Continue.

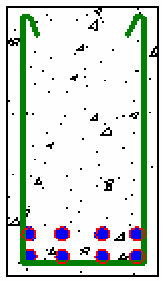
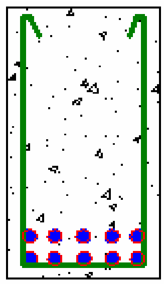
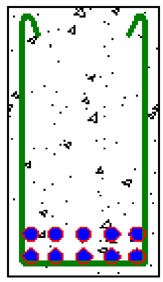
	Beam 8 	Beam 9 	Beam 10 
Dimensions (mm)	457 x 870	457 x 870	457 x 870
Stirrups Spacing (mm) Yield Stress (MPa)	9.5 mmØ 197 445	9.5 mmØ 133 445	9.5 mmØ 133 445
Long. Steel Yield Stress (MPa)	5-31.8mmØ; 5-31.8mmØ 483	5-34.9mmØ; 5-34.9mmØ 472;431	5-34.9mmØ; 5-34.9mmØ 472;431

Table 2.12 Details of Reinforced Concrete Beams Tested by Roller and Russell

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
2	120.1	356	679	559	559	1397	2.33	5740	431	0.00431	165	448	1099.1
6	72.4	457	870	762	793	2286	2.88	5740	464	0.00081	381	445	665.1
7	72.4	457	870	762	795	2286	2.88	6360	483	0.00157	197	445	787.6
8	125.3	457	870	762	795	2286	2.88	6360	483	0.00081	381	445	482.6
9	125.3	457	870	762	795	2286	2.88	7940	483	0.00157	197	445	749.1
10	125.3	457	870	762	793	2286	2.88	9560	464	0.00233	133	445	1171.7

Note: * refers to average yield stress representative of 12-31.8 mmØ bars (472 MPa) and 4-34.9 mmØ bars (431 MPa)

From their study, Roller and Russell concluded that:

- The ACI 318-83 code provisions mainly over-predicted the shear strengths of the beams.
- The minimum amount of the shear reinforcement in the ACI code of $0.35b_ws/f_y$ should be increased for higher strength concrete.

2.2.7 Ganwei and Nielsen [8]

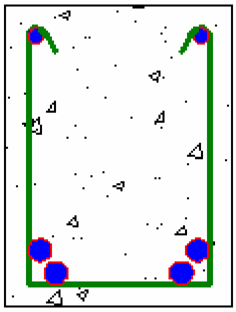
Out of five beams, two beams with a concrete compressive strength of 83.2 MPa tested by Bernhardt and Fynboe were reported in Ganwei and Nielsen. The beams had a cross-section of 150 mm wide x 200 mm deep as given in Table 2.13. The tests were carried out to study the shear behaviour of high strength concrete beams reinforced with open stirrups. Further details of these two beams are provided in Table 2.14.

The main conclusions from these tests were as follows:

- The experimental shear capacities of the beams were only 60% - 70% of the predictions from the Plasticity Theory.

- The low test shear capacities were attributed to the open stirrups used. However, there were no independent tests to confirm if better stirrup cage constructions would have produced beams with greater shear strengths.

Table 2.13 Cross-Sectional of Reinforced Concrete Beams in Ganwei and Nielsen

	Beam S-8-A & B 
Dimensions (mm)	150 x 200
Stirrups Spacing (mm) Yield Stress (MPa)	8 mmØ 150 427
Conc. Cover (mm)	15
Long. Steel Yield Stress (MPa)	2-8mmØ(T); 4-20mmØ(B) 510(B)

Note: Yield stress for the top longitudinal steel bars is not known.

Table 2.14 Details of Reinforced Concrete Beams in Ganwei and Nielsen

Beam Mark	**f'_c (MPa)**	**b_v (mm)**	**D (mm)**	**d (mm)**	**d_o (mm)**	**a (mm)**	**a/d_o**	**A_{sl} (mm^2)**	**f_{sly} (MPa)**	**ρ_t**	**s (mm)**	**f_{sty} (MPa)**	**V_e (Exp) (kN)**
S-8-A	83.2	150	200	160	160	550	3.29	1256	510	0.00447	150	427	125
S-8-B	83.2	150	200	160	160	550	3.29	1256	510	0.00447	150	427	135

2.2.8 Kong and Rangan [11]

Kong and Rangan studied test on 48 reinforced high-performance concrete (HPC) beams with included various parameters like, concrete cover-to-shear reinforcement cage, shear reinforcement ratio, longitudinal tensile reinforcement ratio, overall beam depth, shear span-to-depth ratio, and concrete compressive strength.

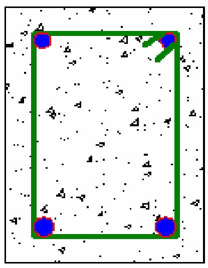
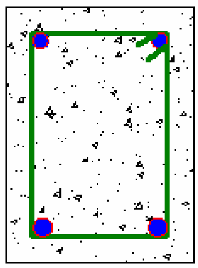
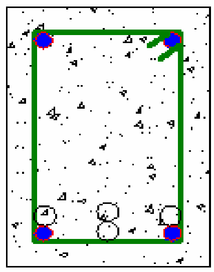
They also reported the results of a statistical analysis performed on 147 earlier test results. The analytical research comprised the development of a theory based on stress analysis of a strut-and-tie model.

The test shear strengths were also compared with the prediction by the shear provisions given in various codes of Australian Standard AS 3600, ACI 318-95, Canadian Standard and Eurocode EC2 part I. Six beams were manufactured and tested in each series.

Beams were simply supported and loaded by one or two point loads placed symmetrically on the span. All the beams were rectangular in cross section with a constant width of 250 mm.

The cross-section of these specimens and complete details of the beams are given in Table 2.15 and 2.16 respectively.

Table 2.15 Cross-Sectional of Reinforced Concrete Beams Tested by Kong and Rangan

	Series 1 	Series 2 and 8 	Series 3 and 6 
Dimensions (mm)	250 x 350	250 x 350	250 x 350
Stirrups	5 mmØ	4 mmØ	4 mmØ
Conc. Cover (mm) to Long. Steel	35	35	35
Long. Steel (mm)	2-12Ø(T); 2-36Ø(B)	2-12Ø(T); 2-28Ø, or 2-36Ø, or 6-24Ø(B)	2-12Ø(T); 2-28Ø, or 2-36Ø, or 6-24Ø(B)

Continued

Table 2.15 Cross-Sectional of Reinforced Concrete Beams Tested by Kong and Rangan (*Continued*)

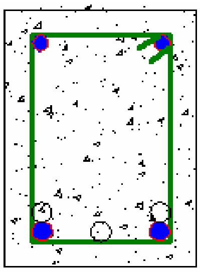
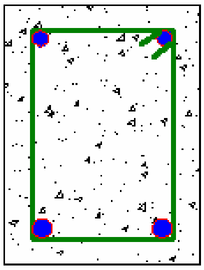
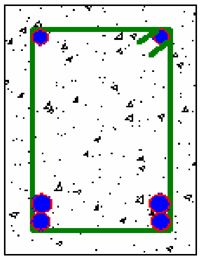
	Series 4 	Series 5 	Series 7 
Dimensions (mm)	-	250 x 350	250 x 350
Stirrups	5 mmØ	5 mmØ	5 mmØ
Conc. Cover (mm) to Long. Steel	35	35	35
Long. Steel (mm)	2-12Ø(T); 4-36Ø, or 4-32Ø, or 4-28Ø, or 2-36Ø, or 4-24Ø, or 4-24Ø(B)	2-12Ø(T); 2-36Ø(B)	2-12Ø(T); 4-32Ø(B)

Table 2.16 Details of Reinforced Concrete Beams Tested by Kong and Rangan

Beam Mark	**f'_c (MPa)**	**b_v (mm)**	**D (mm)**	**d_o (mm)**	**a (mm)**	**Side Cover (mm)**	**a/d_o**	**A_{sl} (mm^2)**	**f_{sly} (MPa)**	**ρ_t**	**s (mm)**	**f_{sty} (MPa)**	**V_e (Exp) (kN)**
S1-1	63.6	250	350	292	730	25	2.50	2046	452	0.00157	100	569	228.3
S1-2	63.6	250	350	292	730	25	2.50	2046	452	0.00157	100	569	208.3
S1-3	63.6	250	350	292	730	35	2.50	2046	452	0.00157	100	569	206.1
S1-4	63.6	250	350	292	730	35	2.50	2046	452	0.00157	100	569	277.9
S1-5	63.6	250	350	292	730	50	2.50	2046	452	0.00157	100	569	253.3
S1-6	63.6	250	350	292	730	50	2.50	2046	452	0.00157	100	569	224.1
S2-1	72.5	250	350	292	730	35	2.50	2046	452	0.00105	150	569	260.3
S2-2	72.5	250	350	292	730	35	2.50	2046	452	0.00126	125	569	232.5
S2-3	72.5	250	350	292	730	35	2.50	2046	452	0.00157	100	569	253.3
S2-4	72.5	250	350	292	730	35	2.50	2046	452	0.00157	100	569	219.4
S2-5	72.5	250	350	292	730	35	2.50	2046	452	0.00209	75	569	282.1
S2-6	72.5	250	350	292	730	35	2.50	2046	452	0.00262	60	569	359.0*
S3-1	67.4	250	350	297	740	35	2.49	1232	450	0.00101	100	632	209.2

Table 2.16 Details of Reinforced Concrete Beams Tested by Kong and Rangan (*Continued*)

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d_o (mm)	a (mm)	Side Cover (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
S3-2	67.4	250	350	297	740	35	2.49	1232	450	0.00101	100	632	178.0
S3-3	67.4	250	350	293	730	35	2.49	2046	452	0.00101	100	632	228.6
S3-4	67.4	250	350	293	730	35	2.49	2046	452	0.00101	100	632	174.9
S3-5	67.4	250	350	299	720	35	2.41	2760	442	0.00101	100	632	296.6
S3-6	67.4	250	350	299	720	35	2.41	2760	442	0.00101	100	632	282.9
S4-1	87.3	250	600	542	1300	35	2.40	4092	452	0.00157	100	569	354.0
S4-2	87.3	250	500	444	1070	35	2.41	3284	433	0.00157	100	569	572.8
S4-3	87.3	250	400	346	830	35	2.40	2464	450	0.00157	100	569	243.4
S4-4	87.3	250	350	292	730	35	2.50	2046	452	0.00157	100	569	258.1
S4-5	87.3	250	300	248	590	35	2.38	1840	442	0.00157	100	569	321.1*
S4-6	87.3	250	250	198	500	35	2.53	1380	442	0.00157	100	569	202.9
S5-1	89.4	250	350	292	880	35	3.01	2046	452	0.00157	100	569	241.7
S5-2	89.4	250	350	292	800	35	2.74	2046	452	0.00157	100	569	259.9
S5-3	89.4	250	350	292	730	35	2.50	2046	452	0.00157	100	569	243.8
S6-1	68.9	250	350	297	810	35	2.73	1232	450	0.00101	100	632	155.4*
S6-2	68.9	250	350	297	810	35	2.73	1232	450	0.00101	100	632	155.1*
S6-3	68.9	250	350	293	800	35	2.73	2046	452	0.00101	100	632	178.4
S6-4	68.9	250	350	293	800	35	2.73	2046	452	0.00101	100	632	214.4
S6-5	68.9	250	350	299	790	35	2.64	2760	442	0.00101	100	632	297.0
S6-6	68.9	250	350	299	790	35	2.64	2760	442	0.00101	100	632	287.2
S7-1	74.8	250	350	294	970	35	3.30	3284	433	0.00105	150	569	217.2
S7-2	74.8	250	350	294	970	35	3.30	3284	433	0.00126	125	569	205.4
S7-3	74.8	250	350	294	970	35	3.30	3284	433	0.00157	100	569	246.5
S7-4	74.8	250	350	294	970	35	3.30	3284	433	0.00196	80	569	273.6
S7-5	74.8	250	350	294	970	35	3.30	3284	433	0.00224	70	569	304.4
S7-6	74.8	250	350	294	970	35	3.30	3284	433	0.00262	60	569	310.6
S8-1	74.6	250	350	292	730	35	2.50	2046	452	0.00105	150	569	272.1
S8-2	74.6	250	350	292	730	35	2.50	2046	452	0.00126	125	569	250.9
S8-3	74.6	250	350	292	730	35	2.50	2046	452	0.00157	100	569	309.6
S8-4	74.6	250	350	292	730	35	2.50	2046	452	0.00157	100	569	265.8
S8-5	74.6	250	350	292	730	35	2.50	2046	452	0.00196	80	569	289.2
S8-6	74.6	250	350	292	730	35	2.50	2046	452	0.00224	70	569	283.9

Base on the research, the following conclusion made:

- The concrete cover-to-shear reinforcement cage not affected the shear strength of beams.

- The nominal stress at failure V_e/b_vd_o decrease with increasing overall beam depth.
- The shear strength of beams increase with an increase in the shear reinforcement ratio, also increase with an increase in the longitudinal tensile reinforcement ratio.
- The shear span-to-depth ratio a/d_o did not effect the shear strength when $a/d_o \geq 2.50$. However, when $a/d_o < 2.50$, the shear strength increase because of arch action.
- The overall mean test/predicted shear strength ratio V_e/V_p is 1.23 with a coefficient of variation of 32.8 percent for the 44 test result.

2.2.9 Test at Curtin University of Technology [14]

Twelve beams were tested with varying concrete compressive strength and amount of shear reinforcement. Nine of them were made of high-strength concrete. These tests were part of a research project carried out by Ilyas (1993) to study the shear capacities of reinforced HPC beams. Details of these specimens are given in Table 2.17.

These tests were performed to study the effects of the longitudinal steel ratio, transverse steel ratio and shear span-to-depth ratio on the shear strength. The scatter in the results for the shorter beams may have been due to arch action effect in those beams.

Table 2.17 Details of Reinforced Concrete Beams Tested at Curtin University of Technology

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
B22	63.2	200	350	317	317	1160	3.66	1860	450	0.001255	100	516	237
B24	63.2	200	350	316	316	1160	3.67	1860	450	0.001227	160	510	255
C22	63.2	200	350	292	317	1160	3.66	2480	450	0.001255	100	516	311
C23	61.4	200	350	292	317	1160	3.66	2480	450	0.001673	75	516	379
C24	63.2	200	350	291	316	1160	3.67	2480	450	0.001227	160	510	301
D22	61.4	200	350	297	317	1160	3.66	3100	450	0.001255	100	516	290
D23	61.4	200	350	297	317	1160	3.66	3100	450	0.001673	75	516	344
D24	61.4	200	350	296	316	1160	3.67	3100	450	0.001227	160	510	295
D25	61.4	200	350	295	315	1160	3.68	3100	450	0.001948	160	496	404

2.2.10 Mphonde [6]

Six high strength concrete beams were tested with constant clear span of 2134 mm in all the beams and loaded by a concentrated load at midspan. Out of six, five beams are considered in this report. The cross-section was the same for all the beams but the shear reinforcement varied as shown in the Table 2.18. Other details of these beams are provided in Table 2.19.

The following is a summary of the objectives of this study:

- To determine the cracking strengths and the ultimate shear capacities of reinforced concrete beams.
- To examine the adequacy of ACI shear design method for beams with concrete compressive strength grater then 60 MPa.
- To determine the effect of the amount of shear reinforcement on the shear strength.
- To determine the effect of concrete compressive strength (which varied from 60 to 83 MPa) on shear strength.

Table 2.18 Cross-Sectional of Reinforced Concrete Beams Tested by Mphonde

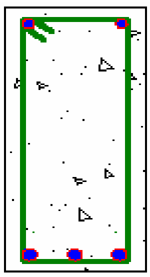
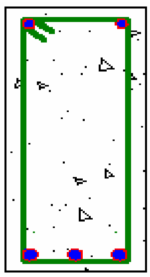
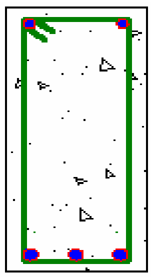
	B50 Series 	B100 Series 	B150 Series 
Dimensions (mm)	152 x 337	152 x 337	152 x 337
Stirrups	3.2 mmØ	4.8 mmØ	3.2 & 4.8 mmØ
Conc. Cover (mm) to Long. Steel	25	25	25
Long. Steel (mm)	2-10Ø(T); 3-25Ø(B)	2-10Ø(T); 3-25Ø(B)	2-10Ø(T); 3-25Ø(B)

Table 2.19 Details of Reinforced Concrete Beams Tested by Mphonde

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
B50-11-3	59.7	152	337	298	298	1067	3.58	1470	448	0.001176	90	303	98.1
B100-11-3	68.6	152	337	298	298	1067	3.58	1470	448	0.002646	90	269	152.1
B100-15-3	81.9	152	337	298	298	1067	3.58	1470	448	0.002646	90	269	115.9
B150-11-3	69.5	152	337	298	298	1067	3.58	1470	448	0.003821	90	280	161.9
B150-15-3	82.7	152	337	298	298	1067	3.58	1470	448	0.003821	90	280	150.3

Base on the research, the following conclusion made:

- Scatter in the shear strength tended to increase with greater amount of shear reinforcement.
- As the amount of shear reinforcement increased, the beams failed in a more ductile manner.
- Beams with shear reinforcement had grater ductility in diagonal tension failures compared to beams without shear reinforcement. The failures were not as sudden or explosive.
- Beams provided with a reasonable amount of shear reinforcement generally failed in shear compression. Sudden diagonal tension failures occurred if the amount of shear reinforcement was very small.

2.2.11 Gabrielsson [17]

Six reinforced HPC beams with shear reinforcement and overall depth in the range of 200 to 300 mm. The concrete compressive strength for these specimens was derived from 100 mm and 150 mm cubes. A conversion factor of 0.8 was used to establish the equivalent cylinder strengths given in Table 2.20.

The aims of this research were as follows:

- To check the applicability of the Swedish design rules for HPC beams where a concrete contribution is added to a steel contribution for a 45° Truss Model.
- To compare Swedish shear design predictions to the prediction from the Modified Compression Field Theory.

Table 2.20 Details of Reinforced Concrete Beams Tested by Gabrielsson

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
S2	72.8	200	300	152	152	500	3.29	1000	664	0.00242	208	521	172.5
S3	90.4	200	300	152	152	500	3.29	1000	664	0.00322	156	521	210.0
HS1	81.6	200	300	260	260	800	3.08	1600	664	0.00296	170	521	250.5
HPS1	98.4	200	300	225	225	550	2.44	1600	664	0.00335	150	521	324.0
HPS2	103.2	200	300	225	225	550	2.44	1600	664	0.00335	150	521	305.0
HB1	86.4	200	300	223	223	500	2.16	2000	475	0.00405	124	521	322.0

Based on Six beams tested, Gabrielsson made the following conclusions:

- The Swedish design rules based on traditional truss theory overestimated the shear strengths of beams.
- The MCFT described the failure reasonably well but it underestimated the shear strengths of all the test beams.
- For HPC beams, cracking began at a higher percentage of the ultimate strength compared to conventional concrete beams. The compressive stress-strain curve for HPC was quite different from that for NSC.
- It was suggested that a shear analysis be performed on a section within the shear span and not where the maximum bending moment occurred since shear failure occurred inside the shear span where the bending moment was considerably smaller.

2.2.12 Thirugnanasundralingam, Sanjayan and Hollins [15]

These investigators tested nine beams altogether; two of which had shear reinforcement having 150 mm wide, 350 mm deep and 2000 mm long. The clear span in each beam was 1800 mm with a constant shear span of 750 mm. Smooth 8 mm diameter mild steel round bar was used for shear reinforcement. The cross-sections of these beams are shown in Table 2.21.

Further details of the beams are provided in Table 2.22. Beams 7 and 9 were tested to study the effect of the amount of shear reinforcement on the shear strength of reinforced high strength concrete beams.

Table 2.21 Cross-Sectional of Reinforced Concrete Beams Tested by Thirugnansundralingam et al.

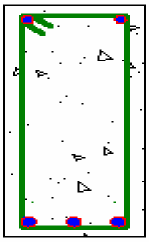
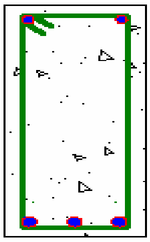
	Beam 7 	Beam 9 
Dimensions (mm)	150 x 350	150 x 350
Stirrups Spacing (mm)	8 mmØ 200	8 mmØ 300
Conc. Cover (mm) to Stirrup	10	10
Long. Steel (mm)	2-12Ø(T); 3-24Ø(B)	2-12Ø(T); 3-24Ø(B)

Table 2.22 Details of Reinforced Concrete Beams Tested at by Thirugnansundralingam et al.

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
7	84	150	350	320	320	750	2.34	1357	400	0.00335	200	250	111
9	84	150	350	320	320	750	2.34	1357	400	0.00223	300	250	113

The conclusions from these tests with regard to beams with shear reinforcement were:

- The diagonal cracking shear force was not influenced by the stirrup spacing. All the three beams had a cracking shear force of 70 kN.
- The ACI 318-89 code predictions were conservative for these beams.
- Crack widths were smaller in beams with shear reinforcement compared to those in beams without shear reinforcement. In addition, the crack widths in the beams with shear reinforcement did not grow until close to failure.

- Shear reinforcement contributed to enhanced dowel action and aggregate interlock witch increase the post-cracking shear strength of the beams.

2.2.13 Kriski and Loov [16]

Kriski and Loov tested six HSC beams in their investigation, four of which had considered in this report. The cross-section of these specimens is given in Table 2.23. All of the beams were 360 mm wide and 400 mm deep with an effective depth of 345 mm. The simply supported beams were loaded at midspan by a concentrated load. Complete details of the beams are given in Table 2.24.

Kriski and Loov concluded that the inclined cracks from the tests were steeper then those predicted by the Shear-Friction Theory. It was argued that a steeper cracks would give a higher strength because the longitudinal steel witch had much larger cross-section area then the shear reinforcement, was more perpendicular to the crack and provided a larger clamping force.

Table 2.23 Cross-Sectional of Reinforced Concrete Beams Tested by Kriski and Loov

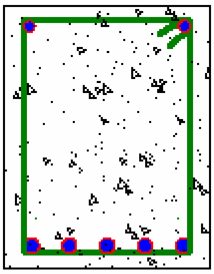
	
Dimensions (mm)	360 x 400
Stirrups Spacing (mm)	5.68 mmØ 150
Conc. Cover (mm) to Long. Steel	43
Long. Steel (mm)	2-10Ø(T); 5-25Ø(B)

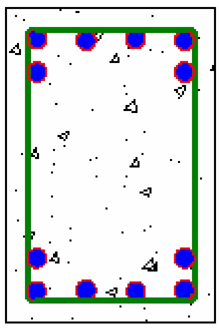
Table 2.24 Details of Reinforced Concrete Beams Tested by Kriski and Loov

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
7	74.3	360	400	345	345	1052	3.05	2500	433	0.00094	150	600	304.5
8	77.8	360	400	345	345	900	2.61	2500	433	0.00094	150	600	391.0
9	77.0	360	400	345	345	1052	3.05	2500	433	0.00094	150	600	242.0
10	76.3	360	400	345	345	900	2.61	2500	433	0.00094	150	600	390.5

2.2.14 Agussalim, Kaku and Matsuno [12]

Twelve reinforced concrete beams were tested in order to investigate the shear resistance behaviour of high strength concrete. Two of them were considered in this report. All specimens have 200 mm width, 300 mm depth and 1300 mm clear span. The cross-section of these specimens is given in Table 2.25. Complete details of the beams are given in Table 2.26.

Table 2.25 Cross-Sectional of Reinforced Concrete Beams Tested by Agussalim et al.



Dimensions (mm)	200 x 300
Stirrups	10 mmØ
Conc. Cover (mm) to Long. Steel	30
Long. Steel (mm)	6-19Ø(T); 6-19Ø(B)

The aims of this research were as follows:

- Whether the stress shift of longitudinal bar due to truss and or arch action occurs really or not?
- Whether the observed strut inclination (concrete strut inclination) is the same as theoretical one or not?
- How much is the effect of supplemental ties on shear strength?

Table 2.26 Details of Reinforced Concrete Beams Tested by Agussalim et al.

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d_o (mm)	A_{sl} (mm^2)	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
No. 7	97.3	200	300	214	3402	727	0.0030	240	340	169.5
No. 12	61.3	200	300	214	3402	727	0.0030	240	340	182.9

The test result showed the following remarks:

- The increase of concrete strength from 61.3 to 97.3 MPa did not bring any increase of shear strength of RC beams.
- The number of stirrups legs does not influence the shear strength significantly.
- The measured stress distribution of longitudinal bar clearly showed the shift to tension side comparable with truss and arch action. This shift did not occur all over the clear span but occur only in the flexure tension region.

2.2.15 Ozcebe , Ersoy and Tankut [13]

Twelve beams having the minimum shear reinforcement required by ACI 318-83, the Turkish Code, and the equation proposed in the research were tested.

Ten of them were made of high-strength Concrete which is varied between 60 and 80 MPa. All specimens having 150 mm x 360 mm 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. The cross-section of these specimens and complete details of the beams are given in Table 2.27 and 2.28 respectively.

Table 2.27 Cross-Sectional of Reinforced Concrete Beams Tested by Ozcebe et al.

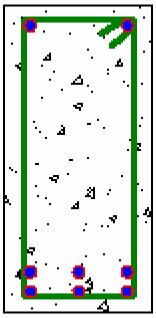
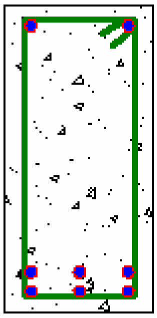
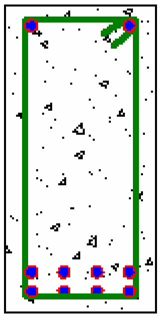
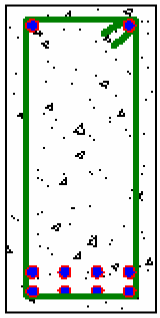
	Series 36	Series 39	Series 56	Series 59
				
Dimensions (mm)	150 x 360	150 x 360	150 x 360	150 x 360
Stirrups	4 mmØ	4 mmØ	4 mmØ	4 mmØ
Conc. Cover (mm) to Long. Steel	30	30	30	30
Long. Steel (mm)	2-10Ø(T); 6-16Ø(B)	2-10Ø(T); 4-16Ø + 2-20Ø(B)	2-10Ø(T); 8-16Ø(B)	2-10Ø(T); 4-16Ø + 4-20Ø(B)

Table 2.28 Details of Reinforced Concrete Beams Tested by Ozcebe et al.

Beam Mark	**f'_c (MPa)**	**b_v (mm)**	**D (mm)**	**d (mm)**	**d_o (mm)**	**a (mm)**	**a/d_o**	**A_{sl} (mm^2)**	**f_{sly} (MPa)**	**ρ_t**	**s (mm)**	**f_{sty} (MPa)**	**V_e (Exp) (kN)**
TH 56	63	150	360	310	327	1635	5	1766	450	0.00167	100	360	103.5
TS 56	61	150	360	310	327	1635	5	1766	450	0.00239	70	360	129.2
ACI 59	82	150	360	310	325	1625	5	1257	438	0.00139	120	360	96.5
TH 59	75	150	360	310	325	1625	5	1257	438	0.00187	90	360	119.3
ACI 36	75	150	360	310	327	981	3	1263	450	0.00139	120	360	105.3
TH 36	75	150	360	310	327	981	3	1263	450	0.00167	100	360	140.9
TS 36	75	150	360	310	327	981	3	1263	450	0.00239	70	360	155.9
ACI 39	73	150	360	310	325	975	3	1590	438	0.00139	120	360	111.8
TH 39	73	150	360	310	325	975	3	1590	438	0.00170	80	360	142.9
TS39	73	150	360	310	325	975	3	1590	438	0.00279	60	360	179.2

The main conclusions from these tests were as follows:

- The reserve strength V_u/V_c increase with increasing shear reinforcement index.

- The ACI 318-83 requirements for minimum shear reinforcement are not satisfactory when high-strength concrete is used.

- The results indicate that the ACI Building Code underestimate the concrete contribution V_c in beams having high shear reinforcement index.

2.2.16 Cladera and Mari [21]

To batter understand the response of high-strength concrete beams failing in shear with web reinforcement, six reinforced concrete beams were tested as a part of extensive research on shear design of reinforced high-strength concrete beams.

The concrete compressive strength of the beams at the age of test ranged from 60 to 87 MPa. Table 2.29 and 2.30 show the detail beam specimens that were tested a shear span of 1080 mm.

Table 2.29 Cross-Sectional of Reinforced Concrete Beams Tested by Cladera and Mari

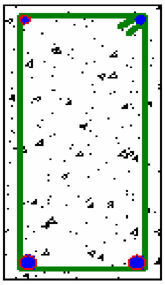
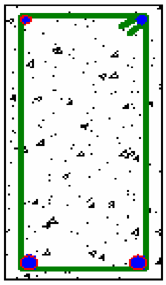
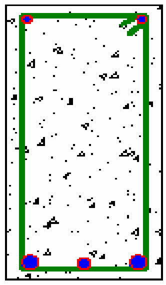
	H60/2, H75/2, 100/2 	H100/3 	H60/4, H100/4 
Dimensions (mm)	200 x 400	200 x 400	200 x 400
Stirrups	6 mmØ	8 mmØ	8 mmØ
Conc. Cover (mm) to Long. Steel	35	35	35
Long. Steel (mm)	2-8Ø(T); 2-32Ø(B)	2-8Ø(T); 2-32Ø(B)	2-8Ø(T); 2-32Ø + 1-25Ø(B)

Table 2.30 Details of Reinforced Concrete Beams Tested by Cladera and Mari

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d_o (mm)	a (mm)	a/d_o	ρ_l	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
H60 / 2	60.8	200	400	353	1080	3.06	0.0228	500	0.00141	200	530	179.74
H60 / 4	60.8	200	400	351	1080	3.08	0.0299	500	0.00239	210	530	308.71
H75 / 2	69.9	200	400	353	1080	3.06	0.0228	500	0.00148	200	530	203.94
H100 / 2	87.0	200	400	353	1080	3.06	0.0228	500	0.00171	165	530	225.55
H100 / 3	87.0	200	400	351	1080	3.08	0.0229	500	0.00239	210	540	253.64
H100 / 4	87.0	200	400	351	1080	3.08	0.0299	500	0.00239	210	540	266.53

Based on the test results of nine beam specimens, the following conclusion can be draw:

- High strength concrete beams with stirrups presented a less fragile response then similar beam without web reinforcement.

- For high-strength concrete beams with stirrups, the limitation of the amount of longitudinal reinforcement to 2% is not experimentally justified.

- Beam specimens with longitudinally distributed web reinforcement along the web showed better behaviour then similar beams without any kind of shear reinforcement.

2.2.17 Yoon, Cook and Mitchell [25]

Six shear tests were conducted on full-scale beam specimens having concrete compressive strength of 67and 87 MPa. Different amounts of minimum shear reinforcement were investigated, including the traditional amounts required by order codes. Table 2.31 shows the details of the 375 mm wide x 750 mm deep specimens that were tested with clear shear span of 2150 mm. complete details of the beams are given in Table 2.32.

The tests on the full scale beams resulted in the following conclusions:

- The 1983 ACI Code and 1984 CSA Standard both contained an expression for the minimum amount of shear reinforcement that did not depend on the concrete strength.

- For high-strength concrete members, amount of minimum shear reinforcement may not provide adequate reserve of strength after shear cracking unless the shear reinforcement can develop significant strain hardening.
- The provision of an appropriate amount of minimum shear reinforcement also helps to control bond splitting cracks that otherwise could lead to brittle shear-bond failures.

Table 2.31 Cross-Sectional of Reinforced Concrete Beams Tested by Yoon et al.

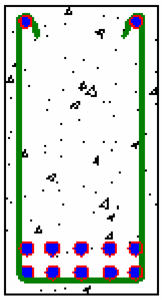
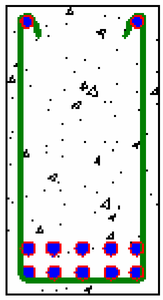
	M1-N, H1-N 	M2-S, M2-N, H2-S, H2-N 
Dimensions (mm)	375 x 750	375 x 750
Stirrups	8 mmØ	9.5 mmØ
Conc. Cover (mm) to Long. Steel	40	40
Long. Steel (mm)	2-10Ø(T); 10-30Ø(B)	2-10Ø(T); 10-30Ø(B)

Table 2.32 Details of Reinforced Concrete Beams Tested by Yoon at al.

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d (mm)	d_o (mm)	a (mm)	a/d_o	ρ_l	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
M1-N	67	375	750	655	695	2150	3.1	0.028	400	0.00082	325	430	405
M2-S	67	375	750	655	695	2150	3.1	0.028	400	0.00116	325	430	552
M2-N	67	375	750	655	695	2150	3.1	0.028	400	0.00163	230	430	689
H1-N	87	375	750	655	695	2150	3.1	0.028	400	0.00082	325	430	483
H2-S	87	375	750	655	695	2150	3.1	0.028	400	0.00140	270	430	598
H2-N	87	375	750	655	695	2150	3.1	0.028	400	0.00233	160	430	721

2.2.18 Tanimura and Sato [24]

To evaluation of shear strength of deep beams with stirrups, four reinforced high-strength concrete beams are cast, three of them are considered in this report. The beams were placed over roller supports and subjected to a symmetric two-point concentrated load. Table 2.33 shows the complete details of the 300 mm wide x 450 mm deep specimens.

Table 2.33 Details of Reinforced Concrete Beams Tested by Tanimura and Sato

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d_o (mm)	a (mm)	a/d_o	ρ_l	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
46 (F)	97.5	300	450	350	350	1.0	0.0214	750	0.0021	100	957	1243
47 (F)	96.3	300	450	350	350	1.0	0.0214	750	0.0048	100	953	1300
48 (F)	94.5	300	450	350	525	1.5	0.0214	750	0.0021	100	957	932

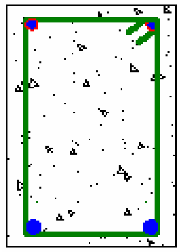
The conclusions obtained by this research are as followed:

- As a results of experiments on deep beams whose shear span ratio *a/d* is 0.5, 1.0, or 1.5 the shear strength increased when *a/d* = 1.0 and 1.5, though the shear reinforcement effect of the stirrups was hardly noticeable when *a/d* = 0.5.
- The experiment verified that the shear span ratio and the transverse reinforcement ratio are important parameters influencing the effect of shear reinforcement of stirrups.

2.2.19 Pandyala and Mendis [22]

The experimental program consisted of testing four beams. Two of them were made of high-strength concrete with web reinforcement. The main longitudinal bars were cut from 12 mm diameter high-strength deformed bars, and the stirrups were fabricated from 6 mm diameter mild steel bars. Thus, 2_210(100)(2) represents the second specimen with a constant stirrups spacing 210 mm throughout, nominal concrete strength of 100 MPa, and *a/d* of 2. Table 2.34 and 2.35 show the detail of the 80 mm (wide) x 160 mm (deep) beam specimens that were tested a shear span of 280 mm.

Table 2.34 Cross-Sectional of Reinforced Concrete Beams Tested by Pendyala and Mendis



Dimensions (mm)	80 x 160
Stirrups	6 mmØ
Conc. Cover (mm) to Long. Steel	15
Long. Steel (mm)	2-6Ø(T); 2-12Ø(B)

Table 2.35 Details of Reinforced Concrete Beams Tested by Pendyala and Mendis

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d_o (mm)	a (mm)	a/d_o	ρ_l	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
210(100)(2)	91	80	160	140	280	2	0.02	410	0.00163	210	370	18.7
2_210(100)(2)	83	80	160	140	280	2	0.02	410	0.00163	210	370	21.5

The following conclusions were drawn based on this study:

- The shear strength of concrete does not increase in the range of 50 to 70 MPa. The shear strength of concrete appears to level off above concrete strengths of 90 MPa.
- The provisions for shear design contained in ACI 318-95 and NS 3473E are conservative and are applicable for HSC beam design.

2.2.20 Ahmad, Park and El-Dash [23]

Eight beams made of high-strength concrete were tested to study the effects of web reinforcement placement on load-carrying capacity. The shear span-to-depth ratio *a/d* was 0.6.

Out of eight, two high-strength concrete beams considered in this report. The typical beam was of 102 x 204 mm cross-section with 25 mm concrete cover from all four sides measured from the center line of the web reinforcement. The cross-section of these specimens is given in Table 2.36. Complete details of the beams are given in Table 2.37.

Table 2.36 Cross-Sectional of Reinforced Concrete Beams Tested by Ahmad et al.

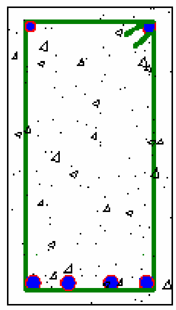
	
Dimensions (mm)	102 x 204
Stirrups	6.35 mmØ
Conc. Cover (mm) to Long. Steel	25
Long. Steel (mm)	2-6.35Ø(T); 4-12.7Ø(B)

Table 2.37 Details of Reinforced Concrete Beams Tested by Ahmad et al.

Beam Mark	f'_c (MPa)	b_v (mm)	D (mm)	d_o (mm)	a (mm)	a/d_o	ρ_l	f_{sly} (MPa)	ρ_t	s (mm)	f_{sty} (MPa)	V_e (Exp) (kN)
A1H	82.8	102	204	178	107	0.6	0.0207	413	0.00349	89	413	71.54
A2H	93.1	102	204	178	107	0.6	0.0207	413	0.00349	89	413	76.08

Base on the results of this study, the following conclusion can be drawn.

- Based on experimental results discussed in this study, ACI 318-89 shown a good capability for predicating the shear strength V_c of the beams. But provisions may overestimate V_u for beam with web reinforcement and a/d > 2.0.

- The BS 8110 for shear strength predicts the beam capacity well for *a/d* ~ 2.0. For shear span much lower then 2.0, the equation may not fit the actual concrete shear strength V_c. Also, it may overestimate the ultimate shear capacity of beam V_u for *a/d* ≥ 2.0.

2.3 SHEAR DESIGN PROVISIONS IN CODES

Three national codes of practice are considered for comparisons of test to predicted shear strengths of reinforced concrete beams: Indian Standard IS 456: 2000 [1], American Concrete Institute Building Code ACI 318 -05 [2], and Eurocode EC2 Part I [3].

2.3.1 Review of Codal Provisions

In most concrete codes, the shear capacity of a Reinforced Concrete (RC) beam is taken to be the sum of the shear capacity of the concrete component and steel component. This simplification is purely for convenience, even through the resistance offered by the concrete and steel form parts of a complex interaction. The equation for the steel component V_s in most codes, including the ACI 318, and the IS code is assumed to be independent of the concrete strength, and is generally of the form:

$$v_s = \frac{A_{sv} f_{sv} d_v (\cot\beta + \cot\alpha)\sin\beta}{s} \quad \text{.... (2.2)}$$

Where,

A_{sv} = area of shear reinforcement within spacing *s*

f_{sv} = yield strength of shear reinforcement

d_v = lever arm resisting flexural moment (usually equal to 0.9d)

β = angle of inclined stirrups to the longitudinal axis of the beam, and

α = angle of inclination (which varies between 30 to 60 degrees) of the diagonal compressive struts to the longitudinal axis of the beam.

The steel component in equation (2.2) is independent of the concrete strength. Most codes specify more then one method to assess the shear strength of concrete such as the variable angle truss method, modified compression field theory, and strut-and-tie methods. Only the simplified methods specified in each of the codes have been used in this comparative study.

All the codal formulae are empirical in nature and most of them are based on experimental results on concrete beams having strength upto 41 MPa only.

2.3.2 Indian Standard IS 456 :2000 [1]

When τ_v is less than τ_c given in Table 2.38, minimum shear reinforcement shall -be provided in accordance with Cl: 26.5.1.6. (Cl: 40.3, IS 456 :2000)

Table 2.38 Design Shear Strength of Concrete, τ_c, N/mm^2 (Table 19, IS 456 :2000)

$100\frac{A_s}{bd}$	Concrete Grade					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.40	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE – The term A_g is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to 26.2.2 and 26.2.3.

$$\frac{A_{sv}}{b \times s_v} = \frac{0.4}{0.87 f_y} \qquad \text{.... (2.3)}$$

Where;

A_{sv} = total cross-section area of stirrup legs effective in shear,

s_v = stirrup spacing along the length of the member,

b = breadth of the beam or breadth of the web of flange beam, and

f_y = characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.

When τ_v exceeds τ_c given in Table 2.17, shear reinforcement shell be provided in any of the following forms: (Cl: 40.4, IS 456 :2000)

a) Vertical stirrups,
b) Bent-up bars along with stirrups, and
c) Inclined stirrups.

Where bent-up bars are provided, their contribution to wards shear resistance shall not be more then the half that of the total shear reinforcement.

Shear reinforcement shall be provided to carry a shear equal $V_u - \tau_c bd$. The strength of shear reinforcement V_{us} shall be calculated as below:

a) For vertical stirrups:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v} \qquad \text{.... (2.4)}$$

b) For inclined stirrup or a series of bars bent-up at different cross-sections:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}(\sin\alpha + \cos\alpha) \qquad \text{.... (2.5)}$$

c) For single bar or single group of parallel bars, all bent-up at the same cross-section:

$$V_{us} = 0.87 f_y A_{sv} \sin\alpha \qquad \text{.... (2.6)}$$

where,

A_{sv} = total cross-sectional area of stirrup legs or bent-u bars within a distance s_v.

s_v = spacing of the stirrups or bent-up bars along the length of the member,

τ_v = nominal shear stress,

τ_c = design shear strength of the concrete,

b = breadth of the member which for flanged beams, shall be taken as the breadth of the web b_w,

d = effective depth,

f_y = characteristic strength of the stirrup or bent-up reinforcement which shall not be taken greater then 415 N/mm^2, and

α = angle between the included stirrup or bent-up bar and the axis of the member, not less then 45°.

N. Subramanian [30] suggest the following expression of (based on Table: 19, IS 456: 2000 [1]) ultimate strength for concrete shear V_{uc}, is based on the Australian research:

$$V_{uc} = \frac{b_w d \times 0.85\sqrt{0.8 f_{ck}}\left(\sqrt{(1+5\beta)}-1\right)}{6\beta} \quad \text{.... (2.7)}$$

Where, $$\beta = \frac{0.8 f_{ck}}{689 A_g / b_w d} \geq 1.0 \quad \text{.... (2.8)}$$

The factor 0.8 in the formula is for converting cylinder strength to cube strength and 0.85 is a reduction factor similar to partial safety factor for materials. Moreover, this formula is valid for concrete grades upto M 40 only.

2.3.3 American Code ACI 318-05 [2]

Modified truss models are used in more recent design codes. For example, ACI Building Code 318-05 [2] still adds a concrete contribution term to the shear reinforcement capacity obtained, assuming a 45° truss. Another procedure involves the use of a truss with a variable angle of inclination for the diagonals. The inclination if the truss diagonals are allowed to deviate from 45° within certain limits based on the theory on of plasticity.

The ACI code adopts the 45° truss model with an additional term for concrete contribution. The ACI code of practice presents two different procedures for calculating the failure shear strength for concrete beams without shear reinforcement.

$$V_n = V_c + V_s \quad \text{.... (2.9)}$$

where;

$$V_c = \left(0.16\sqrt{f'_c} + 17.2\rho_w \frac{V_u d}{M_u}\right) b_v d_o \leq 0.29\sqrt{f'_c} b_w d$$

[ACI 318-05 Eq. (11-5)] .... (2.10)

Where,

V_n = total nominal shear strength V_c + V_s;

V_c = nominal shear strength provided by concrete;

V_s = nominal shear strength from shear reinforcement;

V_u = factored shear force at a given section;

ρ_w = ratio of nonprestressed tension reinforcement;

M_u = factored moment at section;

The $V_u d/M_u$ is generally small. Therefore, ACI 318-05 allows the simplified equation is as follows.

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

[ACI 318-05 Eq. (11-3)] .... (2.11)

For the stirrup contribution to shear, the conservative 45° truss solution is used:

$$V_s = \frac{A_v f_y d}{s}$$

[ACI 318-05 Eq. (11.17)] .... (2.12)

Where f'_c < 70 MPa, and all the other variables are as defined previously.

2.3.4 Eurocode EC2 Part I [3]

For members requiring design shear reinforcement, their design is based on a truss model. For members with vertical shear reinforcement, the shear resistance, $V_{Rd,s}$, should be taken to the lesser, either:

$$V_{Rd,S} = \frac{A_{SW}}{s} z f_{ywd} \cot\theta$$

[EC2 Part I Eq. (6.8)] .... (2.13)

or,

$$V_{Rd,\max} = \frac{\alpha_{cw} b_w z v_1 f_{cd}}{\cot\theta + \tan\theta}$$

[EC2 Part I Eq. (6.9)] .... (2.14)

The recommended limiting values for $\cot\theta$ are given by the expression

$$1 \leq \cot\theta \leq 2.5$$

Where,

$V_{Rd,s}$ = Design value of shear force,

A_{sw} = Cross-sectional area of shear reinforcement,

s = Spacing of stirrups,

f_{ywd} = Yield strength of the shear reinforcement,

f_{cd} = Is the design value of the concrete compression force in the direction of the longitudinal member axis.

v_1 = Strength reduction factor for concrete cracked in shear (may be taken to be 0.6 for $f_{ck} \leq 60$ MPa, and $0.9\text{-}f_{ck}/200$ for high-strength concrete beams)

θ = Is the angle between the concrete compression strut and the beam axis perpendicular to the shear force,

α_{cw} = Coefficient tacking account of the state of the stress in the compression chord, (= 1 for non-priestesses structures)

z = Is the inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration. In the shear analysis of reinforced concrete without axial force, the approximate value $z = 0.9\,d$ may normally be used.

2.4 COMPARISONS OF CODE METHODS

The formulae given in most of the codes of different countries for calculation the shear strength contribution of concrete are empirical and are based on the tests conducted on normal strength concrete having strength less then 40 MPa. Currently the formulae are being interpolated to concretes having strength upto 100 MPa.

But from the comparison of test results on HPC/HSC beams available in literature with the formulae of different codes display the following.

i. The Indian code is valid for normal strength concrete. Hence there is scope for improving the formula to realistically assess the concrete strength and to save on shear reinforcement.

ii. The provisions for shear of reinforced concrete members (beams) contained in the ACI 315-05 [2] and Eurocode EC2 part-I [3] are based on normal strength concrete beams.

2.5 CONCLUSIONS OF THE LITERATURE SURVEY

During the past 30 years, considerable improvements have been made in understanding the behaviour and improving the design of structural concrete member subjected to shear.

To enhance the shear strength of reinforced concrete member, experimental verification was considered by studying various parameters such as concrete cover to shear reinforcement cage and/or, shear reinforcement ratio and/or, longitudinal tensile steel ratio and/or, overall beam depth and/or, shear span to depth ratio and/or, concrete compressive strength.

Based on above studies individual researcher has concluded and suggested in this paper for the further experimental work for improvement of the reinforced concrete subjected to shear.

3. THEORY

3.1 GENERAL

This chapter present a theory developed to predict the shear response and the shear strength of reinforced concrete beam with vertical stirrups. The theory is simplified theory based on the stress analysis of Strut- and Tie model.

3.2 ANALYTICAL MODEL

Fig. 3.1 shows a section of reinforced concrete beam subjected to bending moment M, shear force V and axial force N. The section is sufficiently away from the disturbance caused by concentrated loads, supports and openings.

Fig. 3.1(a) is the beam cross-section where A_{sl} is the total area of longitudinal steel in tension zone, b_v is the width of the web and d_o is the distance from the extreme compression fiber to the centroid of the outermost layer of tensile steel. The actions on the beam [27] are shown in Fig. 3.1(b) and the internal forces are illustrated in Fig. 3.1(c), (d) and (e). The bending moment is resisted by the compressive force C and the tensile force T. The force C is provided by the concrete and the longitudinal steel in the compression zone of the beam and T is given by that part of A_{sl} designated as A_{slM}.

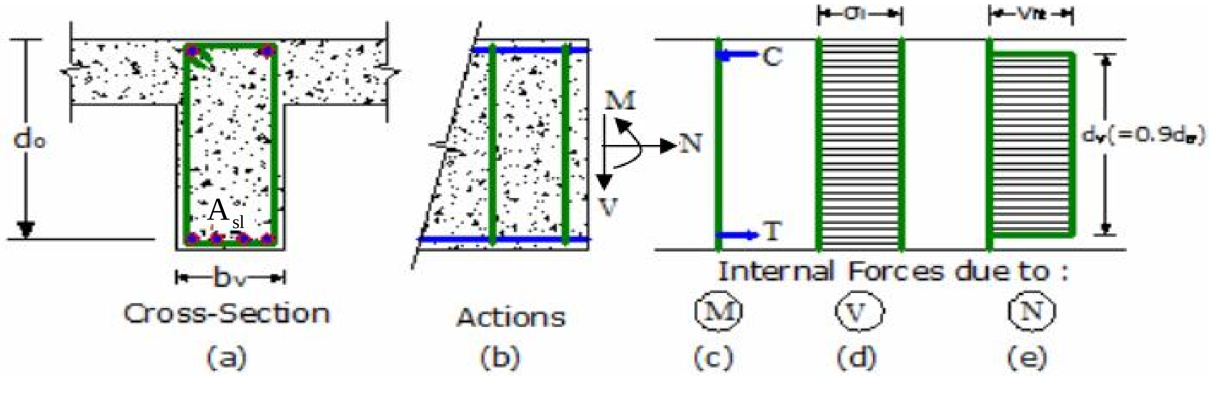


Fig. 3.1 Segment of a Reinforced Concrete Beam

The resultant of the axial force is represented by a uniform stress σ_1 (Fig. 3.1(d)). The shear force V is assumed to be uniformly distributed within a shear effective depth d_v taken equal to $0.9d_o$ (Fig. 3.1(e)). This assumption implies that V is primarily resisted by the web of the cross-section.

It satisfies the boundary condition of zero shear stress at the top and bottom of the beam and is considered to be reasonable.

The shear response and the shear strength of a region of beam can be evaluated by performing a stress analysis of a cracked concrete element the depth d_v.

The cracked concrete element [27] may be represented in the form of a truss comprising a concrete strut, tied together by reinforcing bar in the longitudinal and transverse directions as shown in Fig. 3.2.

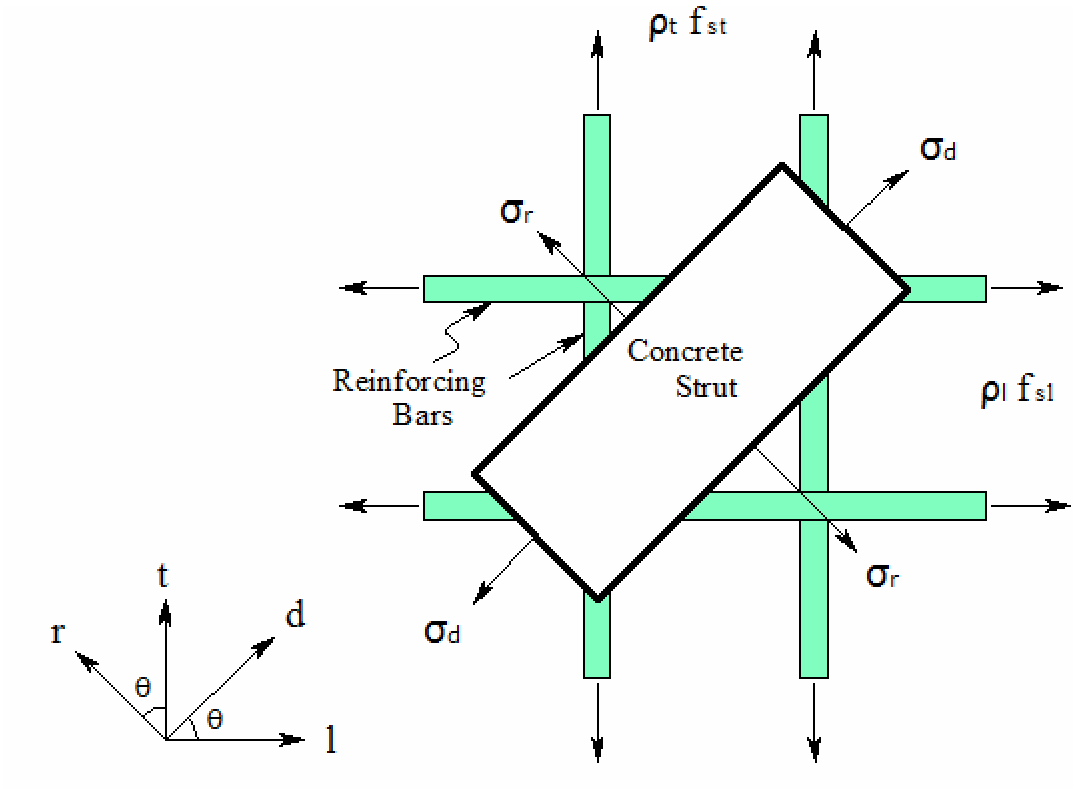


Fig. 3.2 Reinforced Concrete Element for Stress Analysis

Unlike an element in a structural wall which usually contains uniform reinforcement (i.e., bars of the same diameter at equal spacing) in both longitudinal and transverse directions, reinforcement in the web portion of a beam is discrete. Some intuition, therefore, is needed to visualize the truss model illustrated in Fig. 3.2. It is assumed that the part of the longitudinal tensile steel not utilized to resist the bending moment, chosen as A_{slV}, is available to resist the shear force, i.e., $A_{slV} = A_{sl} - A_{slM}$.

The vertical stirrups in the region of the beam constitute the transverse steel. Both these reinforcement are considered to be smeared in the web of the beam in order to perform the analysis of the truss. The reinforcement carries only axial stresses.

The "stress analysis" of the truss can proceed by considering equilibrium, strain compatibility and stress-strain relationships of concrete and steel.

3.3 STRESS ANALYSIS

3.3.1 Equilibrium

In the truss model (Fig. 3.3), the concrete strut which is inclined at an angle θ to the longitudinal direction (i.e., l–direction) develops a compressive stress σ_d along its axis (i.e., d-direction) and a tensile stress σ_r in the orthogonal direction (i.e., r-direction).

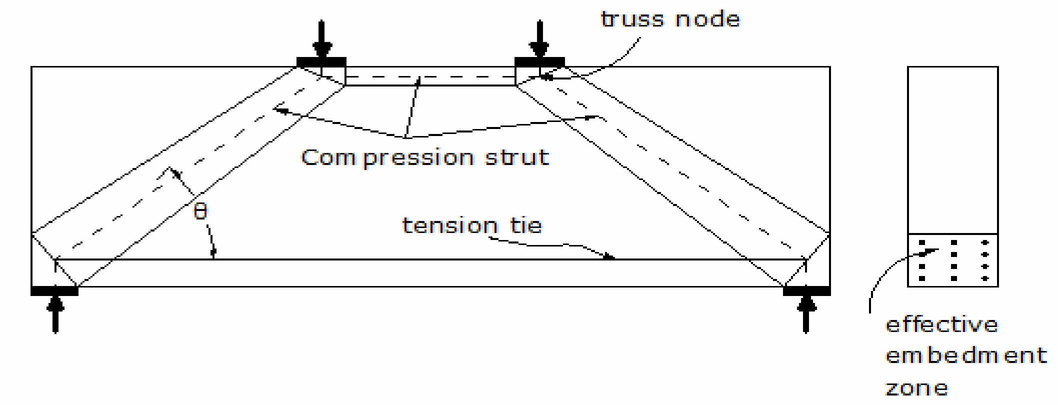


Fig. 3.3 Idealized Truss Model for Beam

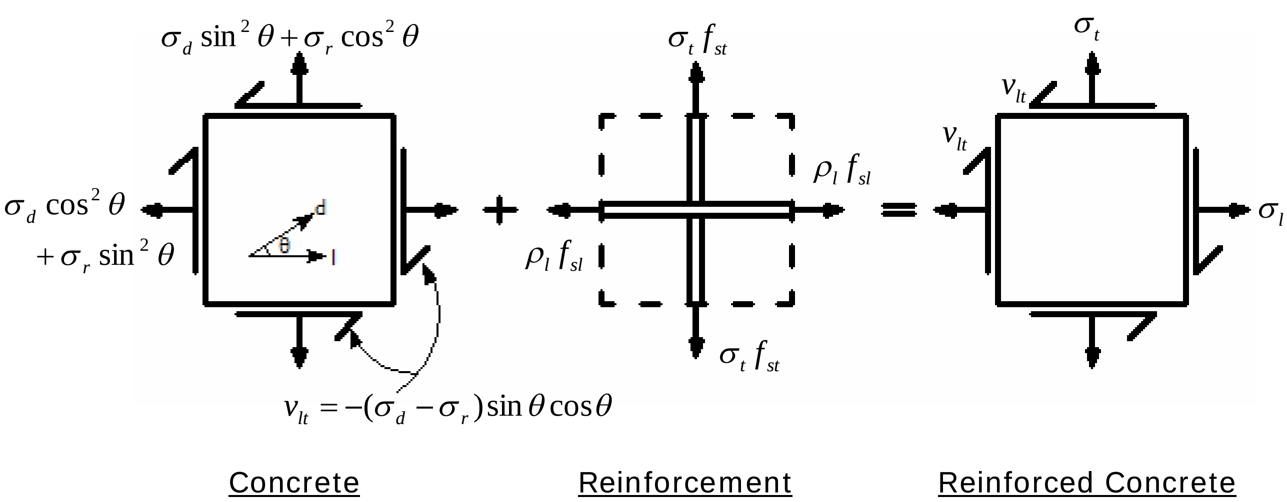


Fig. 3.4 Superposition of stresses

Both σ_d and σ_r are taken as principal stresses. A convenient way to deal with these principle stresses is to transform them in the l- and t-directions using a Mohr's stress circle. This superposed on the stresses in the reinforcement as shown in Fig. 3.4 [27]

For equilibrium, the equations are:

$$\sigma_l = \sigma_d \cos^2\theta + \sigma_r \sin^2\theta + \rho_l f_{sl} \quad \text{.... (3.1)}$$

$$\sigma_t = \sigma_d \sin^2\theta + \sigma_r \cos^2\theta + \rho_t f_{st} \quad \text{.... (3.2)}$$

$$v_{lt} = -(\sigma_d - \sigma_r)\sin\theta\cos\theta \quad \text{.... (3.3)}$$

Where;

σ_l, σ_t = Normal stresses in l- and t-directions respectively and are positive for tension

v_{lt} = Average shear stress in the l- and t-coordinate system, is positive as shown in Fig. 3.4 and is taken as $\left(\dfrac{v}{b_v(0.9d_o)}\right)$

σ_l = $\left(\dfrac{A_{slv}}{b_v(0.9d_o)}\right)$

σ_t = $\left(\dfrac{A_{sv}}{b_v s}\right)$

A_{sv} = Total area of all legs of vertical stirrups across the width of a beam

s = Spacing of stirrups along the longitudinal axis of a beam, and

f_{sl}, f_{st} = Stresses in longitudinal and transverse reinforcement respectively.

3.3.2 Strain Compatibility

The principle strain directions are assumed to coincide with the corresponding principal stress direction. The average strain in the l- and t-directions may be related to principal strains by means of Mohr's strain circle as follows:

$$\varepsilon_l = \varepsilon_d \cos^2\theta + \varepsilon_r \sin^2\theta \quad \text{.... (3.4)}$$

$$\varepsilon_t = \varepsilon_d \sin^2\theta + \varepsilon_r \cos^2\theta \quad \text{.... (3.5)}$$

$$\gamma_{lt} = -2(\varepsilon_d - \varepsilon_r)\sin\theta\cos\theta \quad \text{.... (3.6)}$$

Where,

γ_{lt} = Average shear strain in the element in l- and t-coordinate system.

$\varepsilon_l, \varepsilon_t$ = Average strain in the element in l- and t-directions respectively and are positive for tension

$\varepsilon_d, \varepsilon_r$ = Average principal strain in the element in l- and t-directions respectively, and

3.3.3 Stress-Strain Relationships of Concrete and Steel

- **Softened Concrete in Compression**

The ascending and descending branches of the stress strain curve for high strength concrete are steeper then those for normal strength concrete.

A well-known stress-strain relationship with a pronounced post peak decay which satisfactorily modeled HSC was introduced by Thorenfeldt et al. [26] However, Vecchio et al. [27] recognized that the effective compressive strength of a strut in a reinforced concrete element was less then the uniaxial concrete compressive strength due to the present of tensile strains in the perpendicular direction. This effect may be taken into account by means of a softening factor. Fig. 3.5 shows a softened concrete compressive stress-strain curve, where ζ is the softening factor.

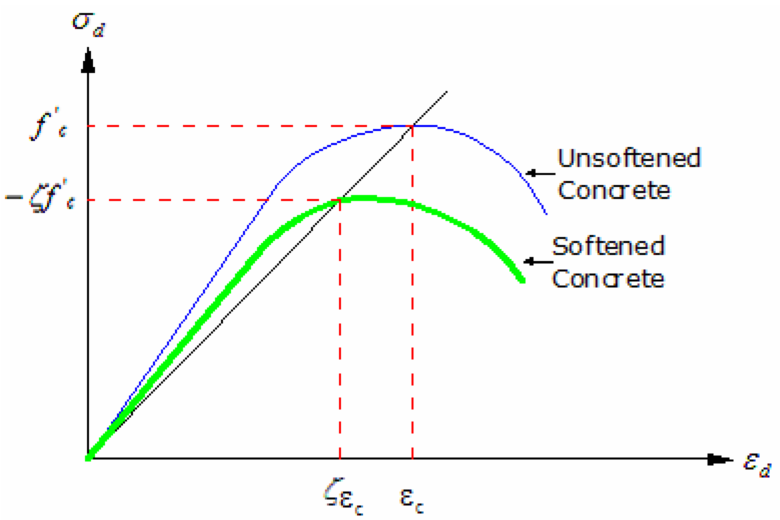


Fig. 3.5 Softened Concrete in Compression

Vecchio et al. [27], proposed a softening factor applicable to high strength concrete as well as normal strength concrete. Based on the results of 116 test specimens, the following softening factor was established:

$$\xi = \frac{1}{1.0 + K_f K_c} \qquad \text{.... (3.7)}$$

where;

$$K_f = 0.1825\sqrt{f_c'} \geq 1.0, \text{ and} \qquad \text{.... (3.8)}$$

$$K_c = 0.35\left[\left(\frac{\varepsilon_1}{\varepsilon_d}\right) - 0.28\right]^{0.8} \geq 1.0 \qquad \text{.... (3.9)}$$

In which, f'_c = characteristic concrete compressive cylinder strength,

ε_d = maximum strain in concrete,

= 0.0035 (varies from 0.003 to 0.005).

Based on Compression Field Theory (CFT), the ability of the concrete to transmit shear across cracks depends on the width of the cracks, which, in turn, is related to the tensile straining of the concrete. The principal tensile strain, ε_1 (ASCE-ACI Committee 445 [28])

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\theta \qquad \text{.... (3.10)}$$

Where, ε_s is the required strain in the tension tie (usually taken as ε_y), the value 0.002 can be assumed strain in the compressive strut at crushing, θ is the angle between the strut and the tension tie that crosses the strut is shown in Fig. 3.5.

The angle θ can be computed when the shear stresses are less then those causing first yield of reinforcement. Hsu [29] proposed the following equation by assuming yielding of steel:

$$\tan\theta = \sqrt{\frac{\rho_t f_{sty}}{\rho_l f_{sly}}} \qquad \text{.... (3.11)}$$

where;

ρ_t = Transverse steel reinforcement ratio,

ρ_l = Longitudinal steel reinforcement ratio.

Based on the experimental studies indicate that the ability of diagonally cracked concrete to resist compression decrease as the amount of tensile straining increase. The principle compressive stress:

$$\sigma_d = -\zeta \times f_c^{'} \leq f_c^{'} \qquad \text{.... (3.12)}$$

Based on Compression Field Theory (CFT), the average principle tensile stress in concrete is deceases as the principle tensile stress increases and it can be taken as:

$$\sigma_r = \frac{0.31\sqrt{f_c^{'}}}{1+\sqrt{500 \times \varepsilon_1}} \qquad \text{.... (3.13)}$$

- **Reinforcing Steel**

In this study, assuming the steel to be elastic-perfectly plastic, the stress-strain relationships of the longitudinal and transverse steel reinforcement can be expressed as follows:

f_{sl}	=	$E_s \varepsilon_l$	when $\varepsilon_l \leq f_{sly}/E_s$
	=	f_{sly}	when $\varepsilon_l > f_{sly}/E_s$
f_{st}	=	$E_s \varepsilon_t$	when $\varepsilon_t \leq f_{sty}/E_s$
	=	f_{sty}	when $\varepsilon_t > f_{sty}/E_s$

where;

f_{sly} = Yield stress of longitudinal steel

f_{sty} = Yield stress of transverse steel, and

E_s = Modulus of elasticity of steel taken as 200 x 10^3 MPa.

3.4 CALCULATION STEPS

For the given beam, the calculation steps are as follows:

Step 1: Input Beam Data, including geometrical, sectional and material properties and material properties.

Step 2: Determine effective longitudinal tensile steel reinforcement ratio based on ACI 318-05 [2],

$$\rho_{l(\max)} = \frac{0.6375 \times \beta \times \left(\dfrac{f_c^{'}}{f_y}\right) \times 600}{600 + f_y} \qquad \text{.... (3.14)}$$

Step 3: Calculate θ by using Equation 3.11.

Step 4: Calculate ζ (softening factor) by Equation 3.7. Hence, calculate K_f and K_c by Equations 3.8 and 3.9 respectively.

Step 5: Calculate σ_d by Equation 3.12.

Step 6: Calculate σ_r by Equation 3.13.

Step 7: Determine V_{lt} (Shear Stress in Concrete) by using Equation 3.3.

Step 8: Determine V_u (Predicted Shear Strength of a Beam).

Step 9: Compare the calculated V_u value with the $V_{e(Exp)}$.

3.5 EXAMPLE

The solution is illustrated below for the beam S8-2, tested by Kong and Rangan [20].

Step 1: Input Beam Data.

The beam cross-section is shown in Fig. 3.6.

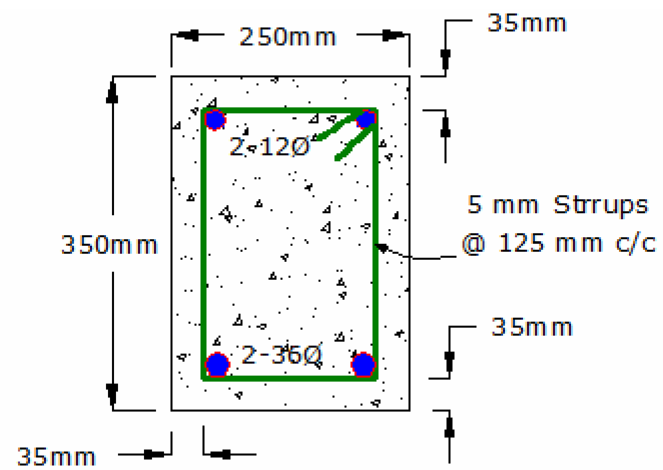


Fig. 3.6 Cross-Section of Beam S8-2 tested by Kong and Rangan [20]

The following data apply:

f'_c =	74.60 MPa	A_s =	2046 mm^2
b_v =	250.0 mm	f_{sly} =	452 MPa
D =	350.0 mm	ρ_t =	0.00126 mm
d_o =	292.0 mm	f_{st} =	569 MPa
d_v =	262.8 mm	$V_{e(Exp)}$ =	250.9 kN

The beam was subjected to two equal concentrated load placed symmetrically on the span. The shear span was 730 mm.

Step 2: Determine effective longitudinal tensile steel reinforcement ratio.

β = 0.34, have suggested by Schlaich et al. (1987) for skew cracks with extraordinary crack width (ASCE-ACI Committee 445 [28] (Table: 1)).

Eq. 3.14:
$$\rho_{l(\max)} = \frac{0.6375 \times 0.34 \times \left(\frac{74.6}{452}\right) \times 600}{(600 + 452)}$$

$$\rho_{l(\max)} = 0.0204$$

Step 3: Calculate angle, θ

Eq. 3.11:
$$\tan\theta = \sqrt{\frac{0.00126 \times 469}{0.0204 \times 452}}$$

= 0.2788

θ = $\tan^{-1}$ (0.2788) = 15.58°

Step 4: Calculate softening factor, ζ

Eq. 3.8:
$$K_f = 0.1825\sqrt{f_c^{'}}$$

$$= 0.1825 \times \sqrt{74.6}$$

$$= 1.576 \geq 1.0$$

Eq. 3.9: $$K_c = 0.35\left[\left(\frac{\varepsilon_1}{\varepsilon_d}\right) - 0.28\right]^{0.8}$$

Where; ε_d = 0.0035

Eq. 3.10: $$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2\theta$$

$$= 0.002 + (0.002 + 0.002) \times \cot^2\theta$$

$$= 0.0535$$

$$K_c = 0.35\left[\left(\frac{0.0535}{0.0035}\right) - 0.28\right]^{0.8}$$

$$K_c = 3.053 \geq 1.0$$

Eq. 3.7: Softening factor, $$\zeta = \frac{1}{1 + K_f K_c}$$

$$= \frac{1}{1 + 1.56 \times 3.053}$$

$$= 0.172$$

Step 5: Calculate, σ_d

Eq. 3.12: $$\sigma_d = -\zeta \times f_c^{'}$$

= - 0.172 x 74.6

= -12.840 MPa

Step 6: Calculate, σ_r

Eq. 3.13: $$\sigma_r = \frac{0.31\sqrt{f_c^{'}}}{1 + \sqrt{500 \times \varepsilon_1}}$$

$$= \frac{0.31\sqrt{74.6}}{1 + \sqrt{500 \times 0.0535}}$$

= 0.434 MPa

Step 7: Determine Shear Stress in Concrete, V_{lt}

Eq. 3.3: $V_{lt} = -(\sigma_d - \sigma_r) \times \sin\theta \times \cos\theta$

= - (-12.84 – 0.434) x sinθ x cosθ

= 3.430 MPa

Step 8: Determine Predicted Shear Strength of a Beam, V_u

$\mathrm{V_u = V_{lt} \times b_v \times d_o}$

= 3.430 x 250 x 292

= 250.57 kN

Step 9: Check V_u

$\mathrm{V_u}$ (= 250.57 kN) which is calculated in step 8. This value agrees with the $\mathrm{V_{e(Exp)}}$ (= 250.9 kN) value. Therefore, the solution is acceptable.

4. MANUFACTURE AND TESTING OF HPC BEAMS

4.1 GENERAL

This chapter describes the experimental work. To better understand the response of high-strength concrete beams failing in shear with shear reinforcement, four reinforced concrete beams were tested at the Structural Laboratory of the Department of Civil Engineering at the College of Nirma Institute of Technology, Ahmedabad. Experimental work related to shear strength and compressive strength has been carried out on the beam specimens tested under two-point loading conditions.

This chapter describes the objectives of the experimental campaign, details of the beam specimens, their construction, material properties, the instrumentation utilized, and the testing procedure that was used. The results of the tests and a discussion are presented in Chapter 5.

4.2 OBJECTIVE OF THE EXPERIMENTAL CAMPAIGN

The main objectives of the experimental campaign carried out were:

- To study the influence of the concrete compressive strength in beams with shear reinforcement.
- Strain Measurement & Deflection measurement
- Ultimate Failure Load
- Crack & failure patterns.

4.3 DESIGN OF TEST SPECIMEN

The following data apply:

Span of beam	=	1500 mm
Overall depth (D)	=	300 mm
Width of beam (b)	=	200 mm
Effective depth (d)	=	265 mm
Clear cover	=	35 mm

The beam is subjected to two point load place on the span. Considered all loads, is to be act on simply supported beam and calculate ultimate load.

The following loads are to be considering in design based on IS 456 :2000 [1]

Assume, thickness of slab and wall is considered 120 mm and 300 mm respectively,

			D.L. (kN/m)	L.L (kN/m)
Weight of slab	=	0.12 x 25	= 3.00	= 0.00
Self weight of Beam	=	0.20 x 0.30 x 25	= 1.50	= 0.00
Wall weight	=	3.0 x 0.30 x 20	= 18.00	= 0.00
Live load (Assume)			= 0.00	= 3.00
Total working load (w)	= D.L. + L.L.		= 22.50 +	3.00
			= 25.50	
Total Ultimate load (w_u)	= 1.5 x w	= 1.5 x 25.50		
		= 38.25 kN/m		

Design moment, $$M_u = \frac{w_u \times l^2}{8} = \frac{38.25 \times 1.5^2}{8} = 10.76 \text{ kN.m}$$

$$V_u = \frac{w \times l}{2} = \frac{38.25 \times 1.5}{2} = 28.68 \text{kN}$$

Maximum ultimate moment of resistance of rectangle section: (Annex G, G-1.1.c, pg. 96, IS 456: 2000 [1])

$$M_{u,lim} = 0.36 \frac{x_{u,max}}{d}\left(1 - 0.42 \frac{x_{u,max}}{d}\right) \times b \times d^2 \times f_{ck}$$

$$\frac{x_{u,max}}{d} = 0.48$$ (Cl. 38.1, pg. 70, IS 456 :2000[1])

Where;

f_{ck} = 60 MPa

$M_{u,lim}$ = 0.36 x 0.48 x (1 - 0.42 x 0.48) x 200 x 265^2 x 60

= 116.26 kN.m > M_u

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \left(\frac{4.6 \times M_u}{f_{ck} \times b \times d^2}\right)}\right] \times b \times d$$

= 114.2 mm^2

Provide; 2-#10 mm diameter HYSD bars having f_y = 415 N/mm^2 as main reinforcement. ($A_{st,pro}$ = 157 mm^2)

To find τ_v: (Cl. 40.1, pg. 72, IS 456 :2000 [1])

$$\tau_v = \frac{V_u}{b \times d} = \frac{28.68 \times 10^3}{200 \times 265} = 0.541 \text{ N/mm}^2$$

To find τ_c: (Table-19, pg. 73, IS 456 :2000 [1])

$$P_t = \frac{100A_{st}}{b \times d} = \frac{100 \times 157}{200 \times 265} = 0.296\%$$, For M-60

τ_c =0.394

Here, τ_v > τ_c so, required to design the shear reinforcement

To find V_{us} : (Cl. 40.4, pg. 73, IS 456 :2000 [1])

V_{us} = V_u - τ_c x b d

= 28.68 x 10^3 – 0.394 x 200 x 265

= 7798 N

To find spacing of stirrups :

Consider, 2-legged, 6 mm-Ø, mild steel having f_y = 250 N/mm^2 vertical stirrups

$$S_v = \frac{0.87 \times f_y \times A_{sv} \times d}{V_{us}}$$

$$= \frac{0.87 \times 250 \times 56.55 \times 265}{1798}$$

= 1812 mm

Check for maximum spacing : (Cl. 26.5.1.6, pg. 48, IS 456 :2000 [1])

(i) 0.75 x d = 0.75 x 265 = 198.75 mm

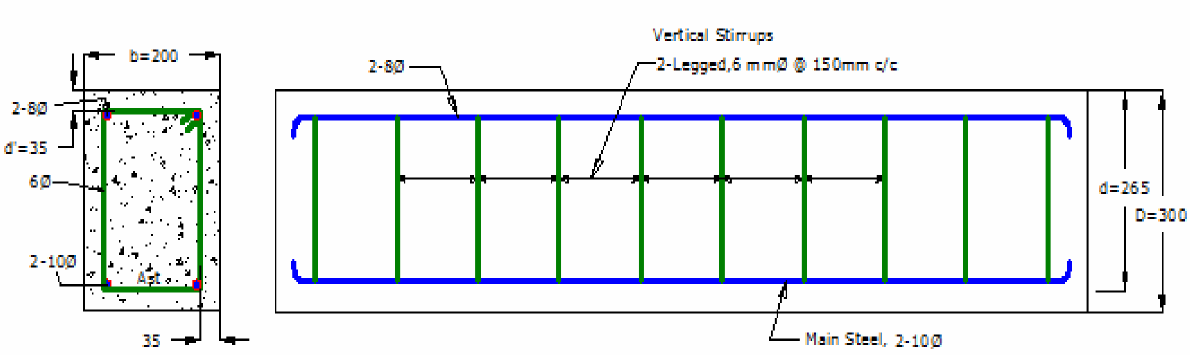
(ii) 300 mm

(iii) $S_v = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$

$$= \frac{0.87 \times 250 \times 56.55}{0.4 \times 200}$$

= 153.74 mm

So provided, 2-legged, 6 mm-Ø, vertical stirrups used at 150 mm c/c. Detail of test specimen are summarised in Fig. 4.1.



(a) Cross-Section (b) Longitudinal Section

Fig. 4.1 Detail of High Performance Concrete Beam Specimen-I

4.4 SPECIMEN DETAILS

Geometry of beams is decided based on extensive literature survey. A total of four beam specimens of 200 x 300 mm cross-section with an effective depth of 265 mm were subjected to two point loads spaced 400 mm apart. Shear span to depth ratio (*a/d*) was 1.51. Cover of 35 mm has been provided on all four sides of reinforcement in beams. The reinforcement cages are also required to be prepared their details are shown in Fig. 4.2 to 4.5. Other details of beams are summarised in Table 4.1.

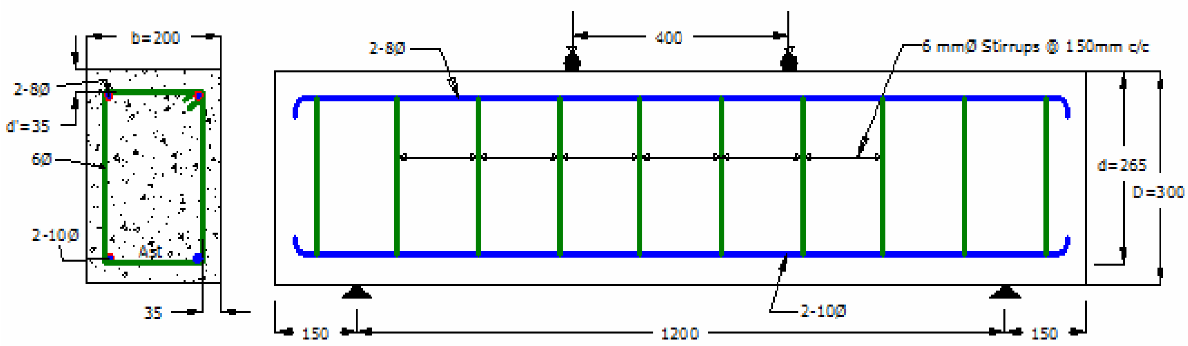


Fig. 4.2 Schematic Drawing of High Performance Concrete Beam Specimen-I

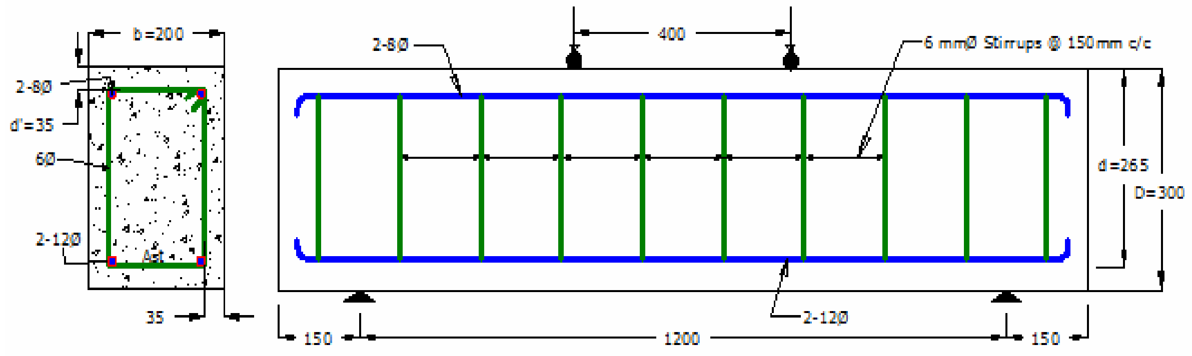


Fig. 4.3 Schematic Drawing of High Performance Concrete Beam Specimen-II

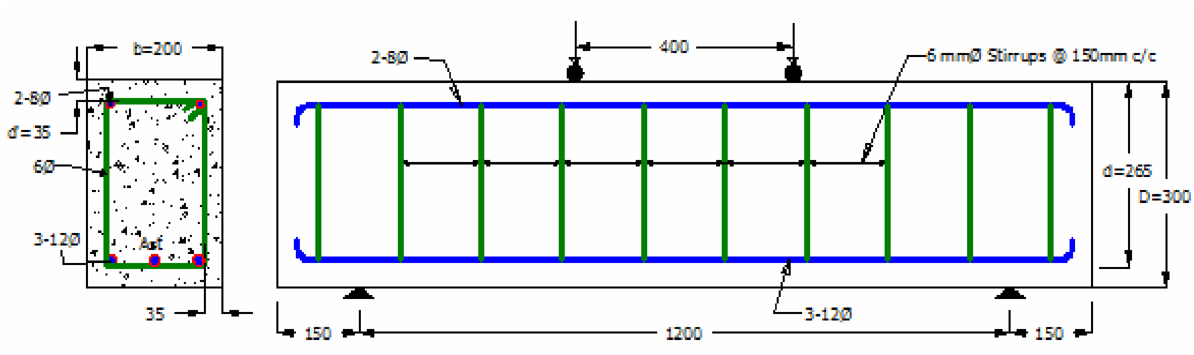


Fig. 4.4 Schematic Drawing of High Performance Concrete Beam Specimen-III

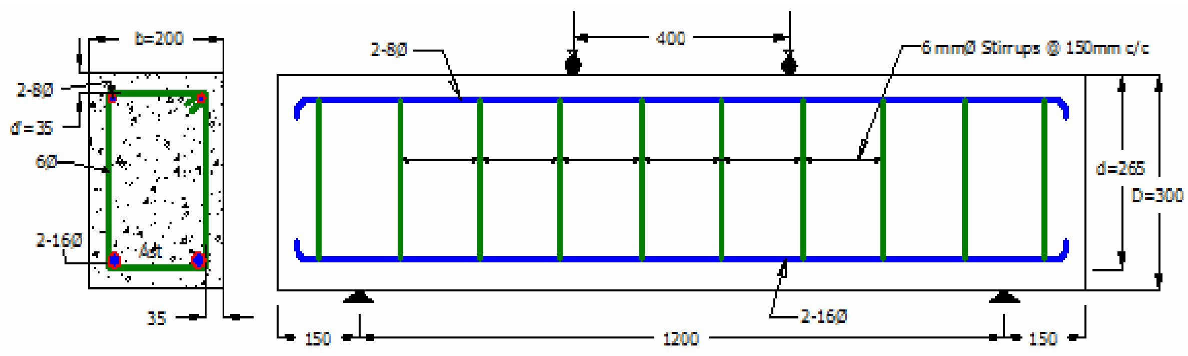


Fig. 4.5 Schematic Drawing of High Performance Concrete Beam Specimen-IV

Table 4.1 Detail of Test Specimen

Beam Mark	b_v (mm)	D (mm)	d (mm)	Total Length (mm)	Span L_e (mm)	Over hang (mm)	a (mm)	a/d	Concrete Cover (mm)			Cast Date	Test Date
									Side	Top	Bottom		
1	200	300	265	1500	1200	150	400	1.51	35	35	35	03-3-'09	4-4-'09
2	200	300	265	1500	1200	150	400	1.51	35	35	35	03-3-'09	3-4-'09
3	200	300	265	1500	1200	150	400	1.51	35	35	35	26-2-'09	5-4-'09
4	200	300	265	1500	1200	150	400	1.51	35	35	35	26-2-'09	5-4-'09

4.5 MATERIAL PROPERTIES

4.5.1 Concrete properties

Concrete mix proportion were prepared base on to study the various available research paper. A maximum aggregate size of 20 mm was used in all four beams. Standard 150 mm x 150 mm cubes were cast with the specimens to obtain the compressive strength of each concrete mix. These cubes were kept under the same environment conditions as the beam specimens until the time of testing.

The mix proportions for high-strength concrete are presented in Table 4.2. This proportion of mixture would produce approximately 50 to 65 MPa cube strength.

Table 4.2 Mix proportions of Concrete

W/C ratio	Water (Kg/m^3)	Cement (Kg/m^3)	Silica Fume (Kg/m^3)	Sand (Kg/m^3)	Aggregate (Kg/m^3)		Admixture* (%)
0.239	132	468	83	612	1136		1.8
					< 20 mm 455	< 12mm 681	

Note: * Super Plasticizer percent of cement weight

Type I Portland cement was used with locally available natural sand having a fineness modulus of 2.62 and a specific gravity of 2.59 was used as a fine aggregate. Crushed granite with maximum size of 20 mm and specific gravity 2.89 was used as a coarse aggregate. The mixes contained about 15% silica fume of total weight of cement. The workability of the mix was improved by using high-range water-reducing admixture (super plasticizer). Although the same mix was used throughout the programmed, different average compressive strengths were obtain by testing the beams at different ages.

4.5.2 Superplasticizer

Commercially available water soluble polycondensates type water reducing superplasticizer (admixture) was used as modifier in the experimental program. Product specifications: super plasticizer meets/exceeds the requirements as per

IS-9103 and AMTS C 494 Type A, B and D. Characteristic of superplasticizer is shown in Table 4.3.

Table 4.3 Characteristic of superplasticizer

Colour :	Dark Brown
Specific gravity :	1.20 $\pm$ 0.02
PH value :	7-8
Chloride contact:	less then 0.05%
Dosage :	0.3% to 1.8% by weight of cement or cementitious materials

4.5.3 Reinforcing Steel Properties

All beams were provided with both top and bottom longitudinal bars (Fig. 4.6). 10 mm diameter bars used as top steel in all beams. Other details of shear reinforcement and longitudinal reinforcement are summarised in Table 4.4. Permissible stresses for longitudinal reinforcement 10, 12 and 16 mm diameter Grade Fe -415 (415 MPa) yield strength deformed bars were used and 6 mm diameter smooth steel bar Grade Fe -250 (250 MPa) were used for transverse reinforcement.

Table 4.4 Reinforcement Detail

Beam Mark	Shear Reinforcement				Longitudinal. Reinforcement			
	Diameter	Spacing (mm)	ρ_t	f_{sty} (MPa)	Top Steel	Bottom Steel	ρ_l	f_{sly} (MPa)
1	6 mm Ø	150	0.002	250	2-8 mm Ø	2-10 mm Ø	0.0030	415
2	6 mm Ø	150	0.002	250	2-8 mm Ø	2-12 mm Ø	0.0043	415
3	6 mm Ø	150	0.002	250	2-8 mm Ø	3-12 mm Ø	0.0064	415
4	6 mm Ø	150	0.002	250	2-8 mm Ø	2-16 mm Ø	0.0076	415

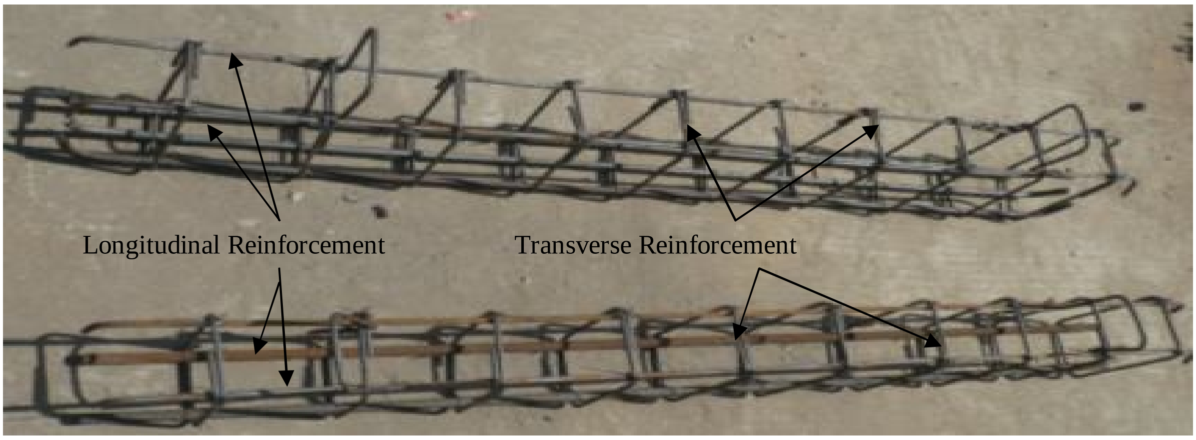
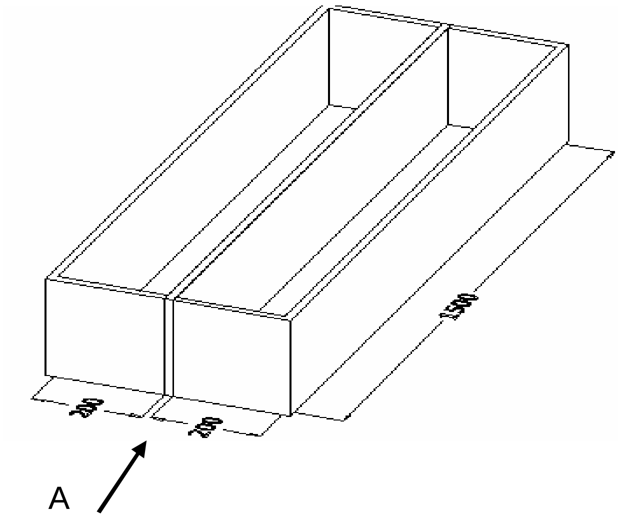
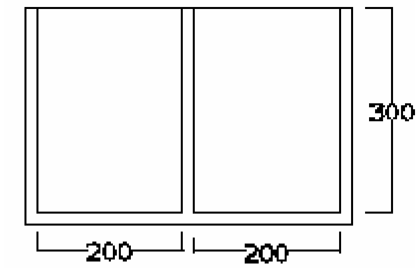


Fig. 4.6 Steel Cage with Web Reinforcement

4.5.4 Details of Form work

For casting specimen form-work are required to be prepared. The dimensions and details of form-work are as shown in Fig. 4.7.





A. 3-D Model of Formwork

B. A-Side view of Formwork

Fig. 4.7 Detail of Formwork

4.5.5 Manufacture of the Test Specimens

Four beams were manufactured at the Structural Laboratory of the Department of Civil Engineering, Nirma Institute of Technology, Ahmedabad. Concrete was placed in layers into the timber moulds (Fig. 4.8).

Fig. 4.8 Reinforcement Cage in Timber Moulds

Hand-held mechanical vibrators were used to compact the fresh concrete (Fig. 4.9(D)). Control cubes were compacted in layers on a vibrating table (Fig. 4.9(C)). The concrete components, reinforcement bars, moulds, and procedures were those actually used at that plant. Fig. 4.9 shows some picture of the fabrication of the beam specimens.

A. Mixing of super plasticizer and water

B. Concrete mix

C. Vibrating Table

D. Surface vibrator

Fig. 4.9 Specimen fabrication at the structural laboratory
(Nirma Institute of Technology, Ahmedabad)

3rd and 4th beam were cast on February 26, 2009. After five day, 1st and 2nd beam cast on March 3, 2009. All beams and 150 x 150 mm cubes were stored at the plant for approximately 28 days. Note that for each batch, only 0.055 m^3 was used.

4.5.6 Concrete Compressive Strength

In each beam, 150 mm x 150 mm cubes were tested in compression at various ages at the time of beam test. The results of the concrete compressive strength tests are given in Table 4.5. The compressive strength development of concrete beam is shown in Fig. 4.10.

Table 4.5 Concrete Compressive Strength

Beam Mark	Cast Date	Compressive Strength (kN)		
		7 (Day)	14 (Day)	28 (Day)
1	3/3/2009	42.66	54.22	61.11
2	3/3/2009	42.66	54.22	61.11
3	26/2/2009	36.88	43.11	58.66
4	26/2/2009	36.88	43.11	58.66

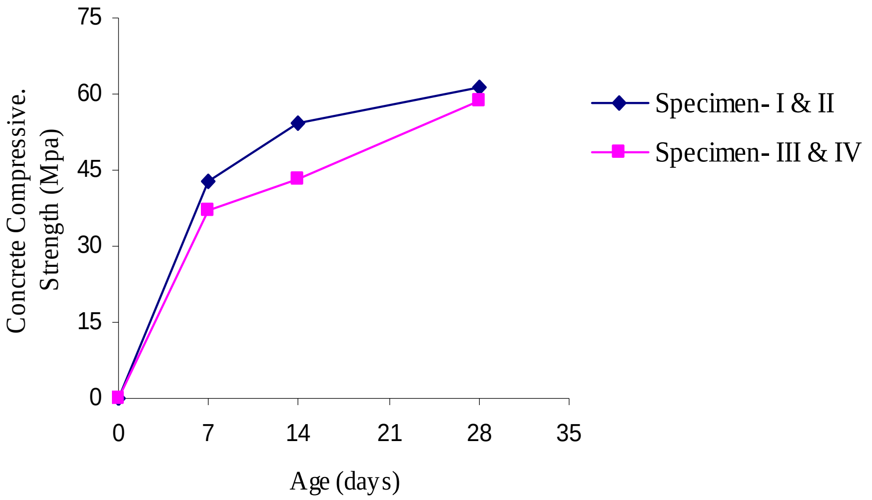


Fig. 4.10 Concrete Compressive Strength Development

4.6 TEST SETUP

Testing of column has been carried out on loading frame in concrete technology laboratory of 1000 kN capacity. As discussed the specimen are required to be tested under two-point loading applied by the help of hydraulic jack at concrete technology laboratory which is of 500 kN capacity. Detail of test set-up is as shown in Fig. 4.11, where the ratio of shear span (a) to depth (d) ratio was 1.51. The load was applied at midspan of the beam specimen and transferred from jack to steel I beam to supporting plate to knife plate and finally on fix support. As shown in Fig. 4.11 the beam is to be placed simply supported on either side by some sort of support (steel column).

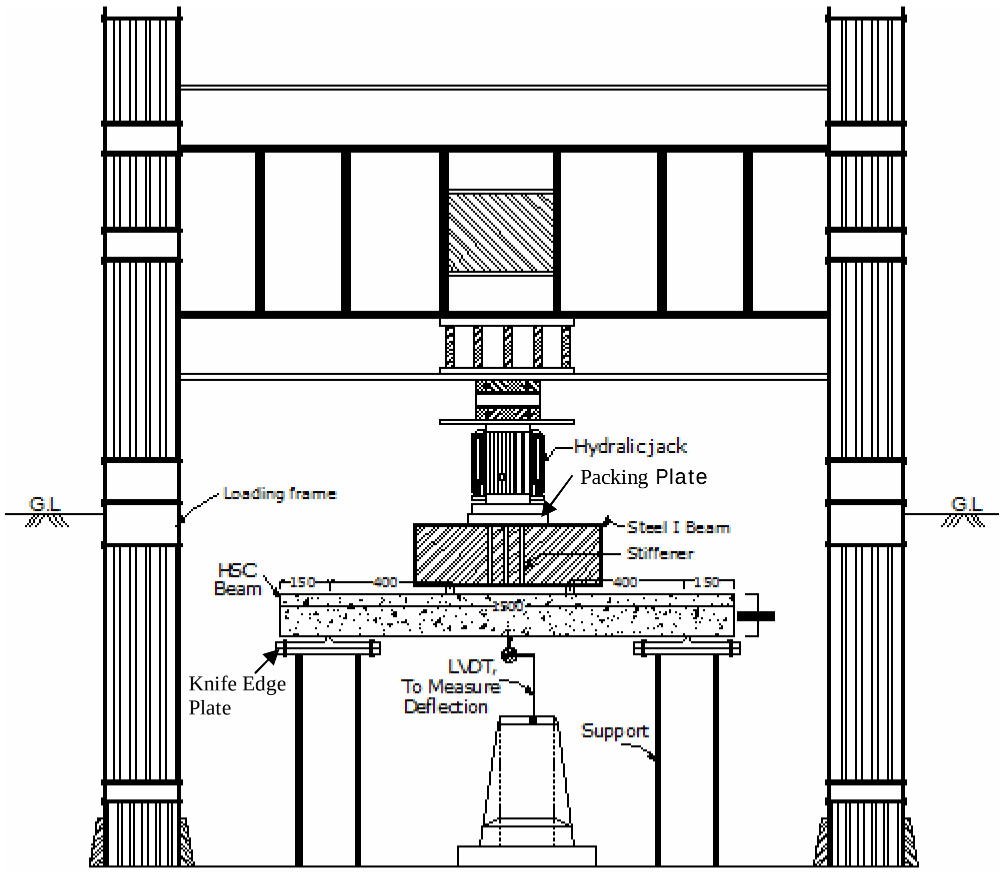


Fig. 4.11 `Details of Test set-up

To monitor the behaviour of the tested beams, the applied loads, strains at the external surface of concrete, and displacement were measured using different instruments such as Linear Variable Differential Transformers (LVDT), dial gauge and mechanical strain gauges. Photography and video equipment were also utilized. Photography and video equipment were also utilized.

LVDT is used to measure deflection of the beam from the bottom. LVDT is kept in such a way that it remains in contact with bottom edge of HPC beam as shown in

Fig. 4.11. Also one dial gauge remains in contact at the bottom of HPC beam to measure the deflection of beam as load is applied through hydraulic jack. For strain measurement at external of surface HPC beam by using mechanical strain gauges. Mechanical strains gauges are attached at different height of the beam having positioning are shown in Fig. 4.12 to 4.15.

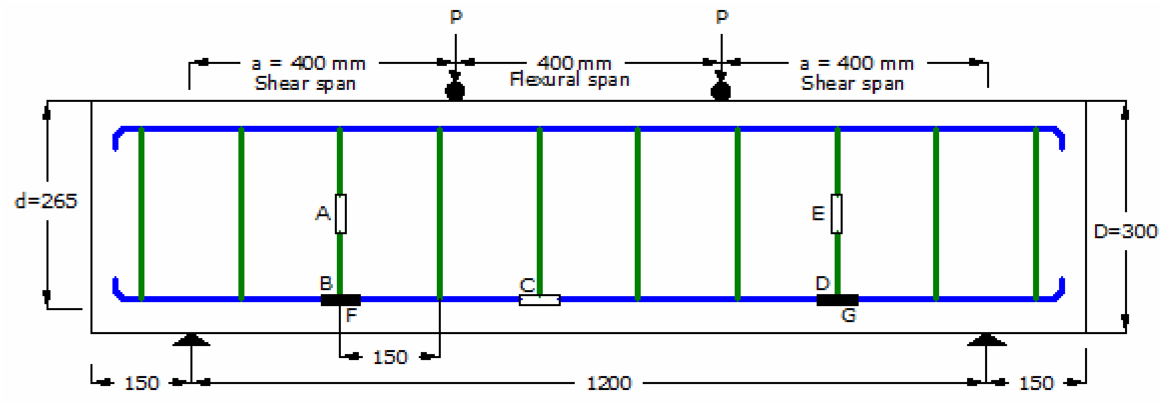


Fig. 4.12 Strain Gauges Positions in Beam-I

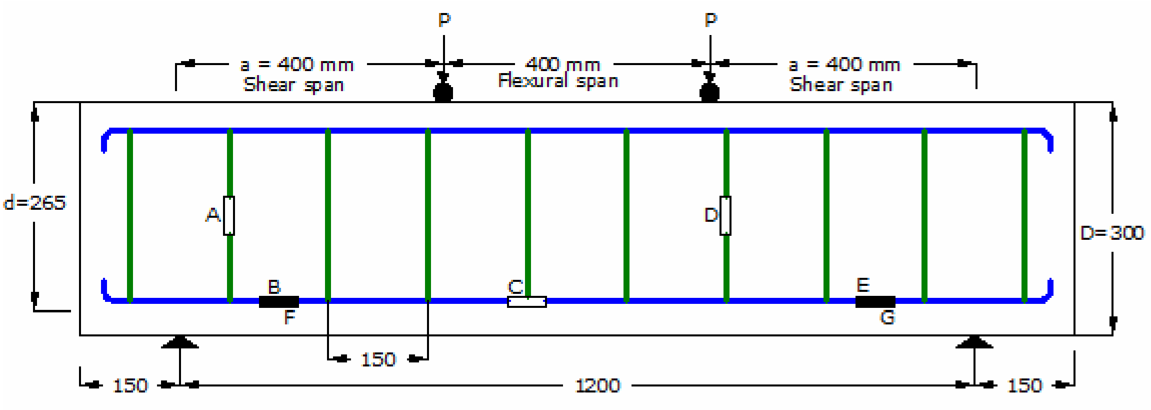


Fig. 4.13 Strain Gauges Positions in Beam-II

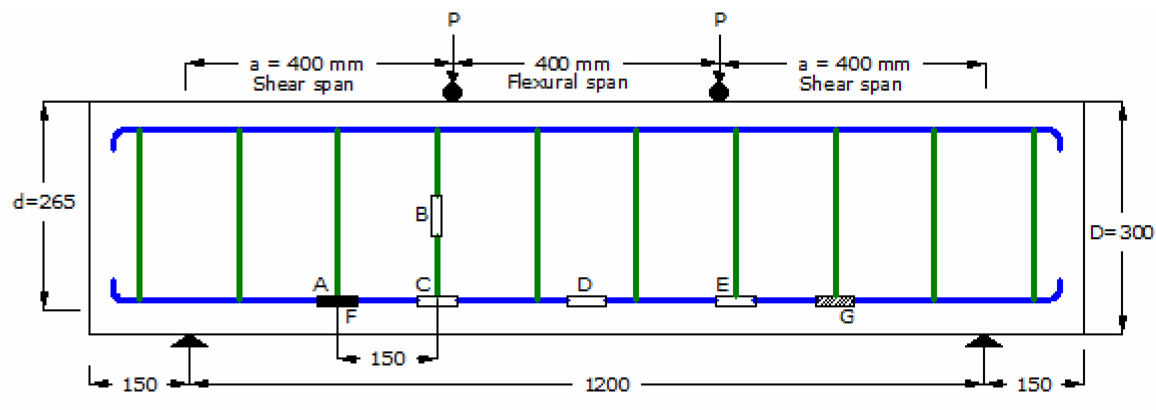


Fig. 4.14 Strain Gauges Positions in Beam-III

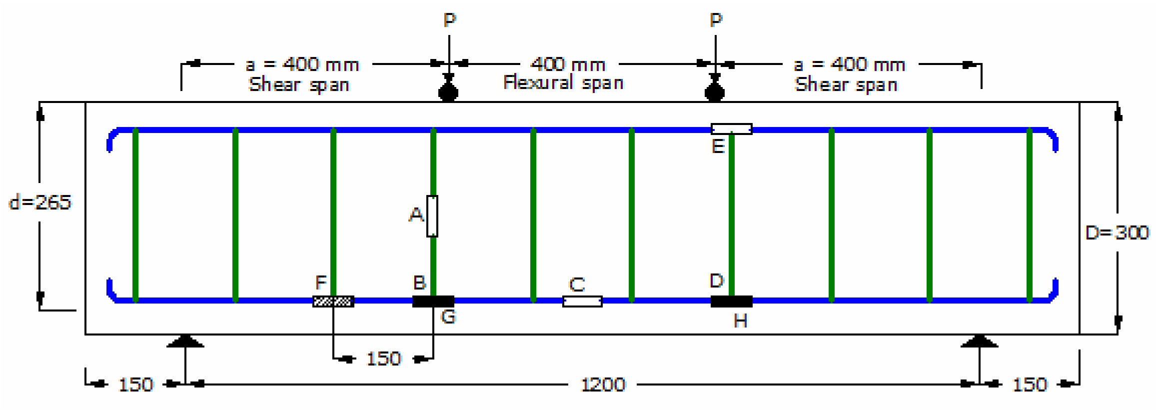


Fig. 4.15 Strain Gauges Positions in Beam-IV

Note: All dimensions are in millimeter.

- Indicate strain gauge on front side.
- Indicate strain gauge on both sides.
- Indicate strain gauge on back sides.

The photographs in Fig. 4.16 give a detailed view of the supporting beam and the hydraulic jack.

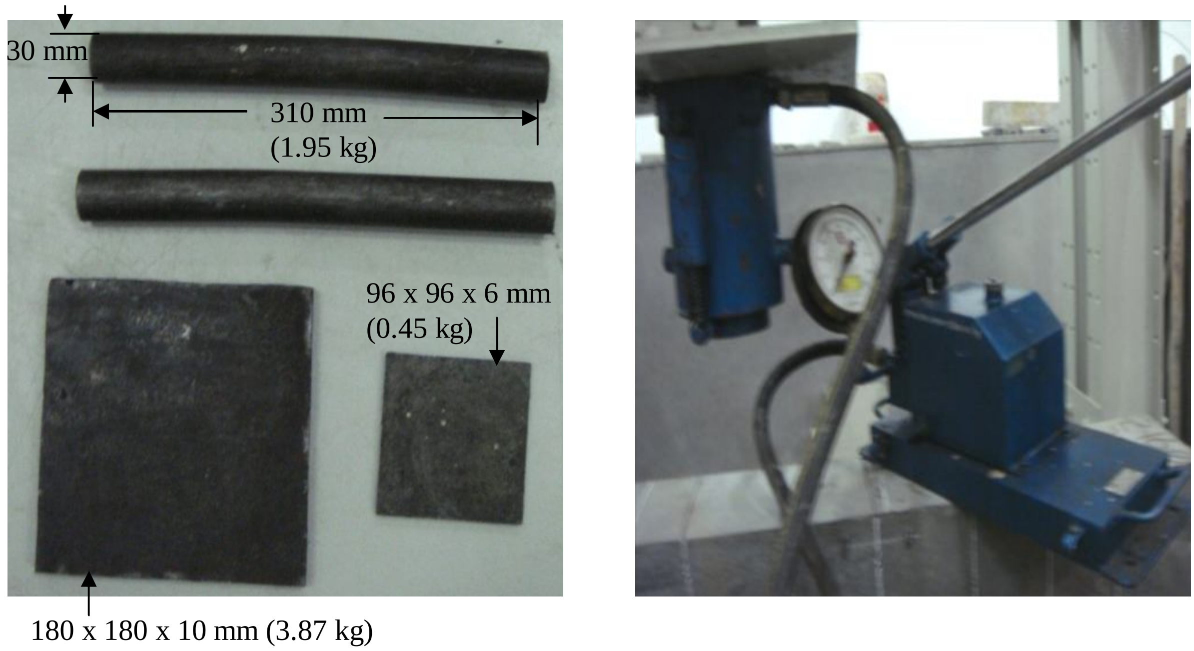


Fig. 4.16 Details of the supporting beam and Hydraulic jack

4.7 BEAM TEST

One day before testing, the test specimens and their respective 150 x 150 mm control cubes were taken out of the moist room and allowed to dry. The next day, the control cubes were capped and tested in compression to determine the strength of the concrete at that time. The beams were loaded to failure in a 500 kN capacity, under two symmetrical point loads at 400 mm spacing with constant shear span to depth ratio 1.51.

The specimens were tested to their maximum load-carrying capacity by monotonically increasing the load with the help of hydraulic jack. At each load increment three different measurements were taken: the vertical deflection at midspan, the strains in the longitudinal bars, and the strains in the stirrups. At each loading stage, the crack pattern in the clear span was also observed and recorded. The critical inclined crack is defined as the inclined crack that occurs in the shear span and crosses the beam at mid-depth eventually causing the failure of the specimen. The collapse load is defined as the load that caused failure of the test beam.

4.8 TEST PROCEDURE

General arrangement of testing setup is shown in Fig. 4.17.

Fig. 4.17 Typical Test Setup

All beams were loaded to failure. Each beam was initially “exercised” by applying a small load to ensure that the test set-up and the instrumentation worked properly. The beam was then unloaded and datum readings were taken. Initially, the beam was loaded in increments of 10 kN until the load reached at failure of beam. After failure, each beam was photographed to show the crack pattern and the mode of failure. Appendix B contains photographs of all the beams after failure. The test results are presented in the next chapter.

4.9 INSTUMENTATION

Strain at different heights of the column and loads are measured during the experiments by making use of various instruments. Different instruments used in experimental work are as follows:-

1. LVDT (Linear Variable Differential Transducer)
2. Hydraulic Jack
3. Mechanical Strain Gauges

4.9.1 LVDT (Linear Variable Differential Transducer)

LVDT is used to measure displacement of the HPC beam when the load is being applied on it. LVDT is attached at the position where deflection is to be measured. Attachment of LVDT and digital displacement indicator is shown in Fig. 4.18 and 4.19 respectively. Strength of the LVDT sensor's principle is that there is no electrical contact across the transducer position sensing element for which the user of the sensor means clean data, infinite resolution and a very long life.

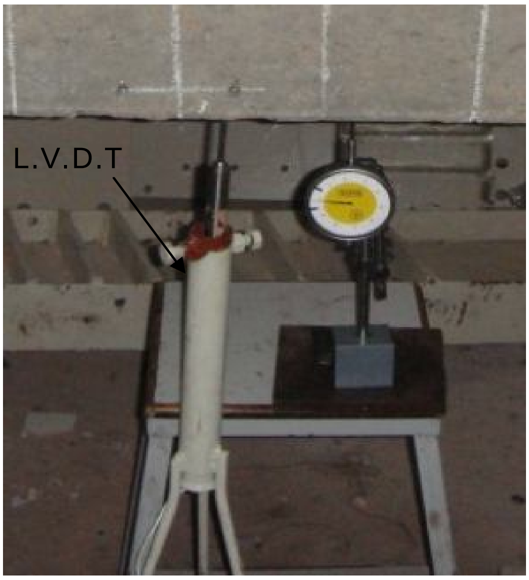


Fig. 4.18 Attachment of LVDT at bottom edge of beam

Fig. 4.19 Digital Displacement Indicator

4.9.2 Hydraulic Jack

Hydraulic jack of capacity of 500 kN is used and is working based on Pascal's principle. Basically, the principle states that the pressure in a closed container is the same at all points. Pressure is described mathematically by a Force divided by Area. Therefore if there are two cylinders connected together, a small one & a large one, and apply a small Force to the small cylinder, this would result in a given pressure.

Fig. 4.20 shows the hydraulic jack which has been used for the application of two point loading condition.

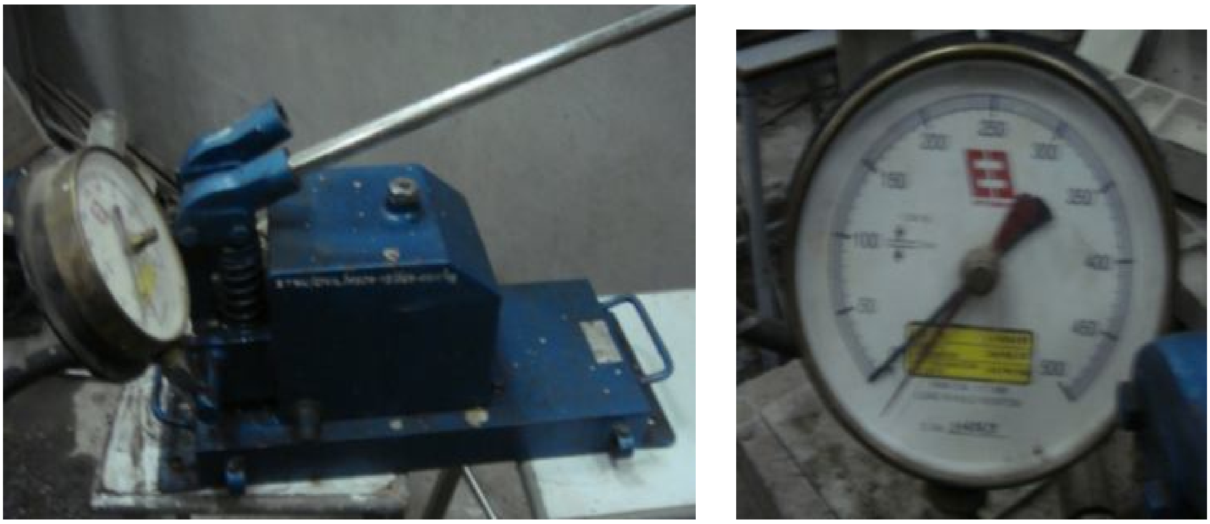


Fig. 4.20 Hydraulic jack (500 kN Capacity)

4.9.3 Mechanical Strain Gauges (DEMEC)

Mechanical strain gauges which are known as DEMEC (Demountable Mechanical) strain gauges. The DEMEC gauges consist of a digital dial gauge attached to an Invar bar. A fixed conical point is mounted at one end of the bar, and a moving conical point is mounted on a knife edge pivot at the opposite end. The pivoting movement of this second conical point is measured by the dial gauge.

A setting out bar is used to position pre-drilled stainless steel discs which are attached to the structure using a suitable adhesive. In this way, strain changes in the structure are converted into a change in the reading on the dial gauge Fig. 4.21 shows is indicating mechanical strain gauge.

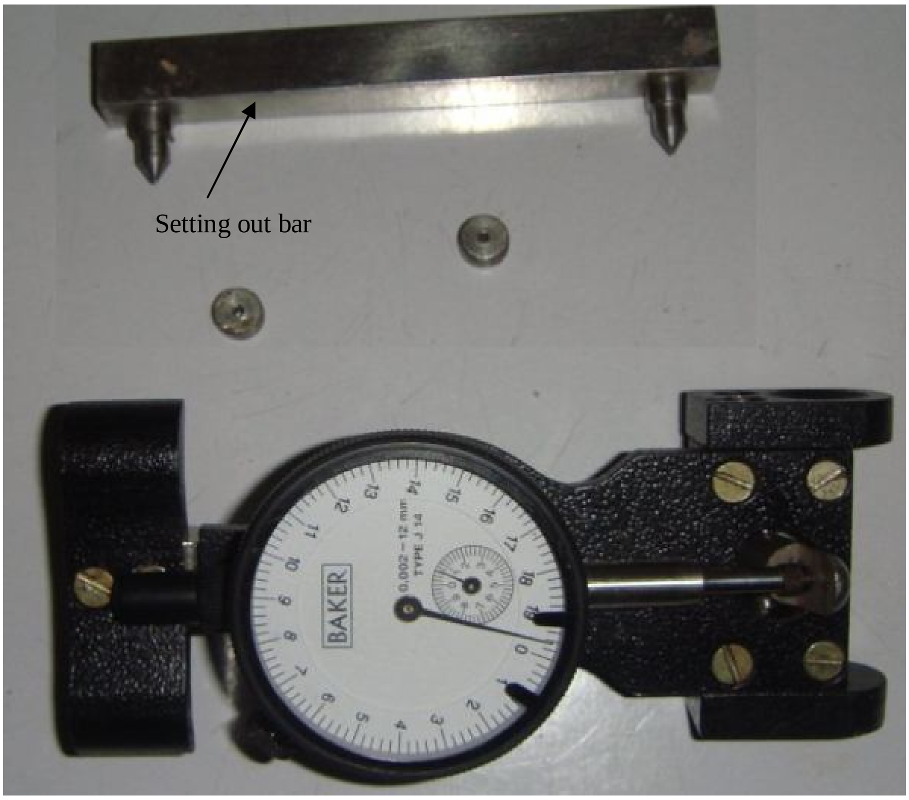


Fig. 4.21 Mechanical Strain Gauge (DEMEC)

5. PRESENTATION AND DISCUSSION OF RESULTS

5.1 INTERODUCTION

The test results and the effects of various parameters on the shear strength based on available research are elaborated in this chapter. The behaviour of the test beams is discussed and the shear strength is tabulated.

The available test results were compared with proposed theory outlined in this report. Only shear failure mode is considered in the analysis. Comparisons of available test results from previous investigations with predictions from the theory are also given.

All the test data of previous investigations are compared with code predictions. The codes which are considered here are IS 456:2000 [1], ACI 318-05 [2], and Eurocode EC2 Part I [3].

5.2 TEST RESULTS

5.2.1 Behaviour of Test Beams

All the beams failed in shear. A summary of experimental results are given in Table 5.1. Complete details are given in Appendix A.

Table 5.1 Summary of Test Results

Beam Mark	f_{ck} (MPa)	b_v (mm)	D (mm)	d (mm)	Side Clear Cover (mm)	a	a/d	ρ_l	f_{sly} (MPa)	ρ_t	f_{sty} (MPa)	V_e (Exp.) (kN)
1	61.11	200	300	265	35	400	1.51	0.0030	415	0.002	250	130.0
2	61.11	200	300	265	35	400	1.51	0.0043	415	0.002	250	145.0
3	58.66	200	300	265	35	400	1.51	0.0064	415	0.002	250	180.0
4	58.66	200	300	265	35	400	1.51	0.0076	415	0.002	250	195.0

The behaviour of all test beams was similar. First small crack occurred in surrounding area of the center span under a load. Subsequently, diagonal cracks were caused between the supports and the loading point after wards. Flexure cracks extended as flexure-shear cracks. However, the diagonal cracks did not progress to the upper part, and the load increased as loading continued. Finally, the concrete between the support and the loading point crashed.

A main shear crack developed suddenly and persisted in opening up with increasing load until the failure of the beam. It was not possible to maintain the load. At this time, the tensile reinforcing steel bar did not yielded. Thus, all specimens failed in shear compression.

Crack patterns and failure modes of all specimens photographs are given in Appendix B.

5.2.2 Effects of Test Parameters

The effect of test parameter on the shear strength is discussed below.

- ***Longitudinal Tensile Reinforcement Ratio***

Specimens 1 to 4 were used to study the effect of the longitudinal tensile reinforcement ratio on the shear strength of beams. Fig. 5.1 shows the shear strength plotted against the longitudinal tensile reinforcement ratio for specimen 1 to 4. It shows, trend of increasing in shear strength with increase in the longitudinal tensile reinforcement ratio.

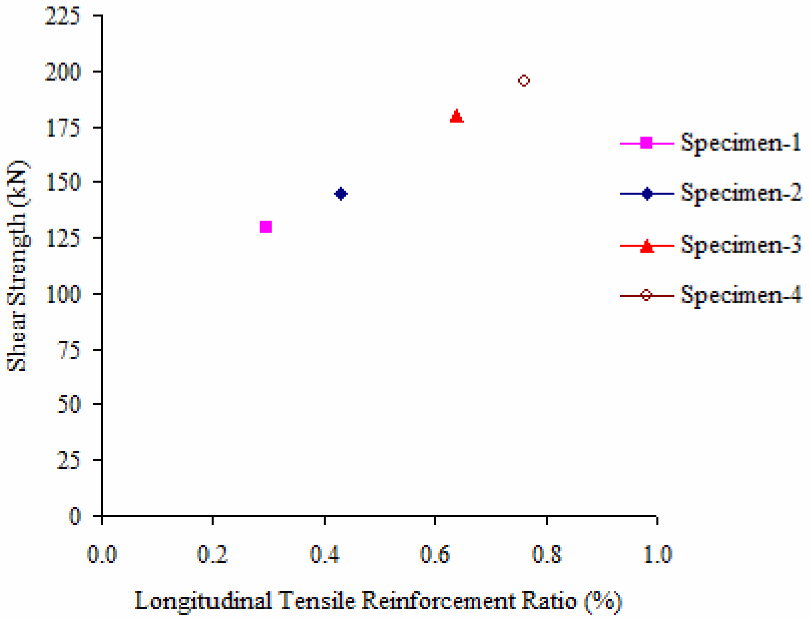


Fig. 5.1 Shear Strength versus Longitudinal Tensile Reinforcement Ratio

5.2.3 Vertical Midspan Deflection

Fig. 5.2 shows the shear forces versus midspan deflection curve for specimen 1 to 4 which are typical for the test beams. Complete test data of midspan deflection are given in Appendix A.

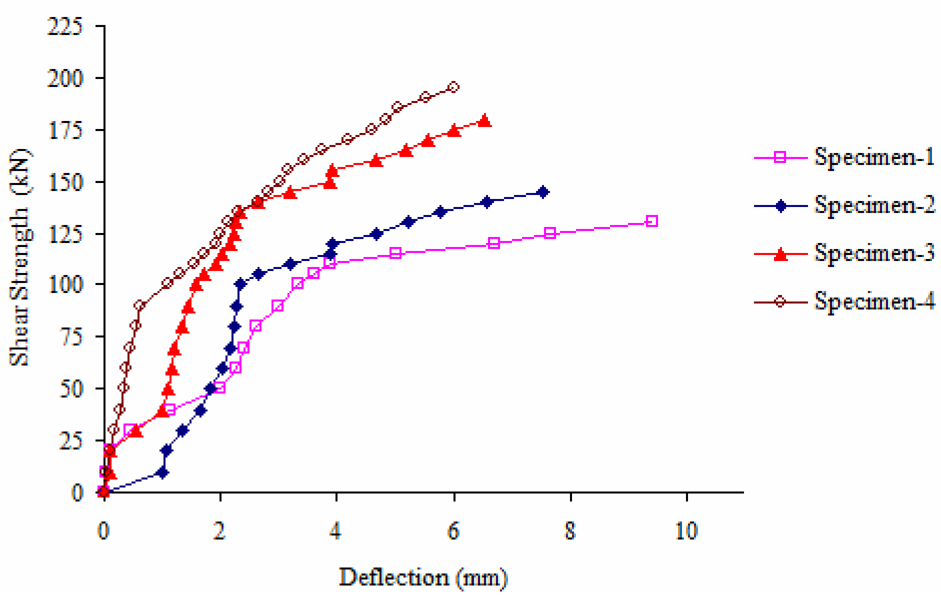


Fig. 5.2 Shear Strength versus Midspan Deflection

All the beams indicated a loss of stiffness as the shear force increased. This loss of stiffness was caused by the flexure and shear cracks propagating in the beams.

The total midspan deflection was equal to the sum of the deflection contributions from shear and flexure.

5.2.4 Strain in Transverse and Longitudinal Tensile Bars

Fig. 5.3 to 5.6 shows some typical curve of shear forces versus strain in transverses and longitudinal tensile bars. For figure note that Strains gauges are attached at different height of the beam having positioning are shown Chapter 4 (Fig. 4.12 to 4.15).

Details of test data are illustrate in Appendix A.

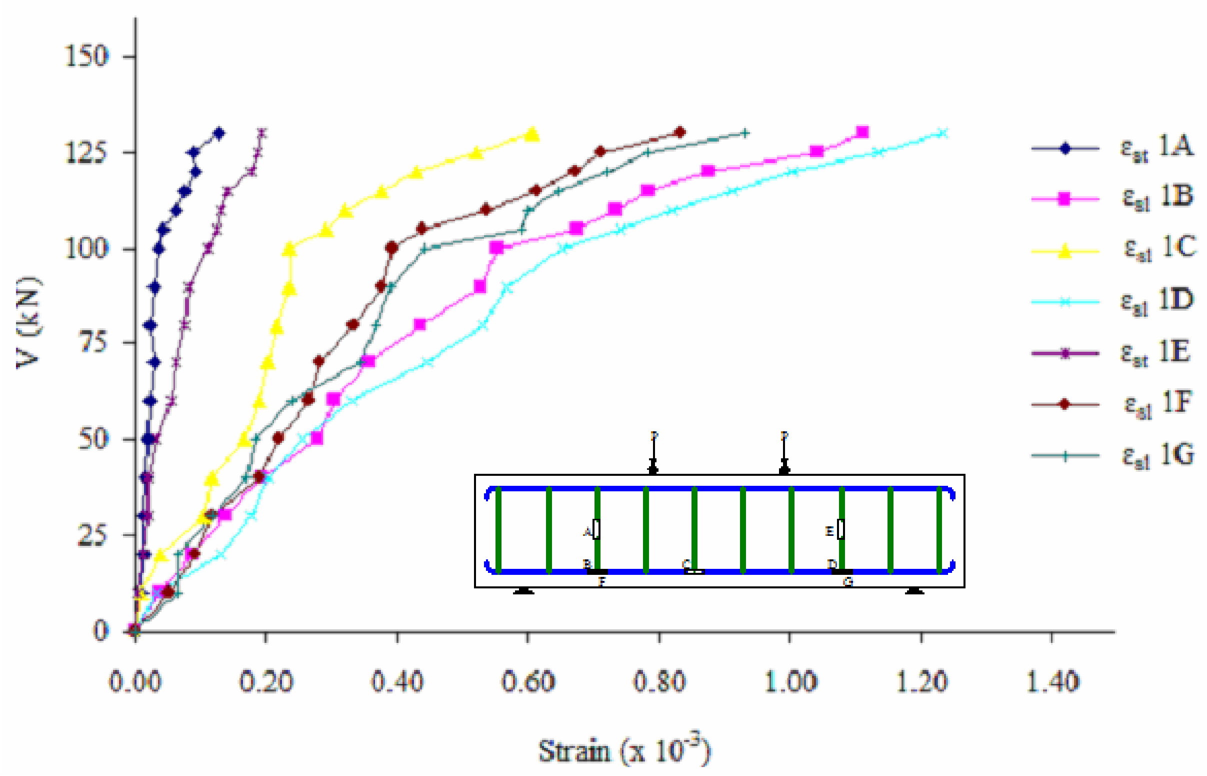


Fig. 5.3 Shear Force versus Steel Strains for Specimen-I

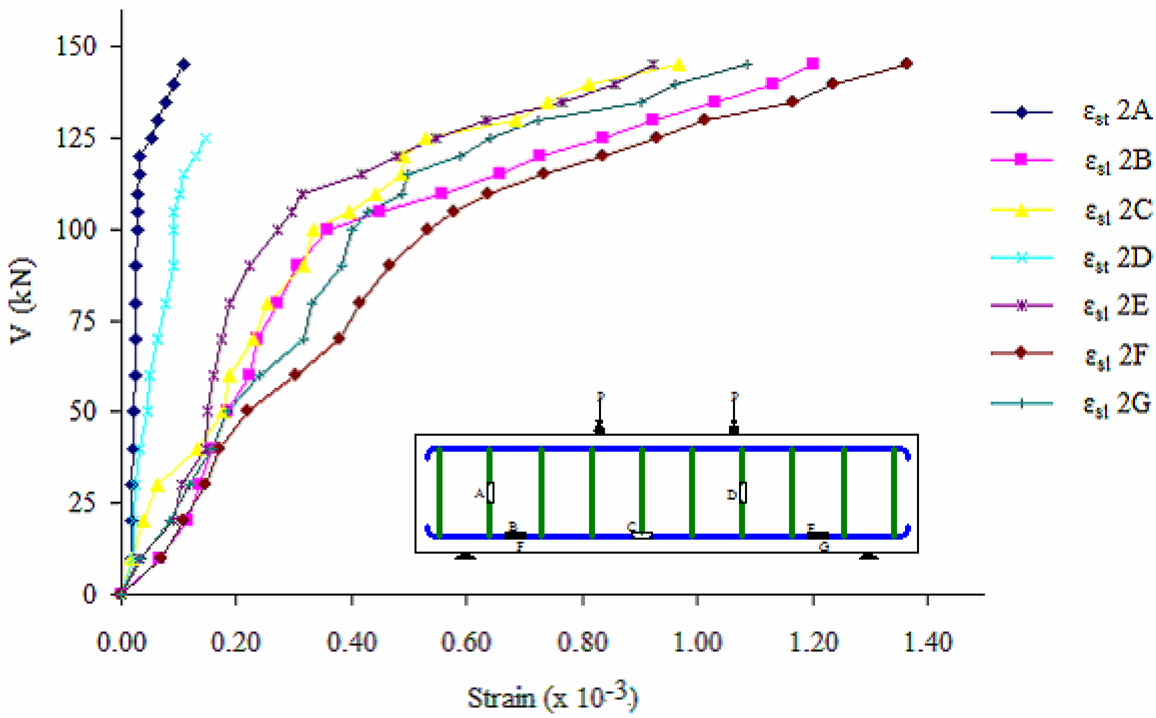


Fig. 5.4 Shear Force versus Steel Strains for Specimen-II

Note: ε_{sl} Strain in longitudinal tensile steel

ε_{st} Strain in transverse steel

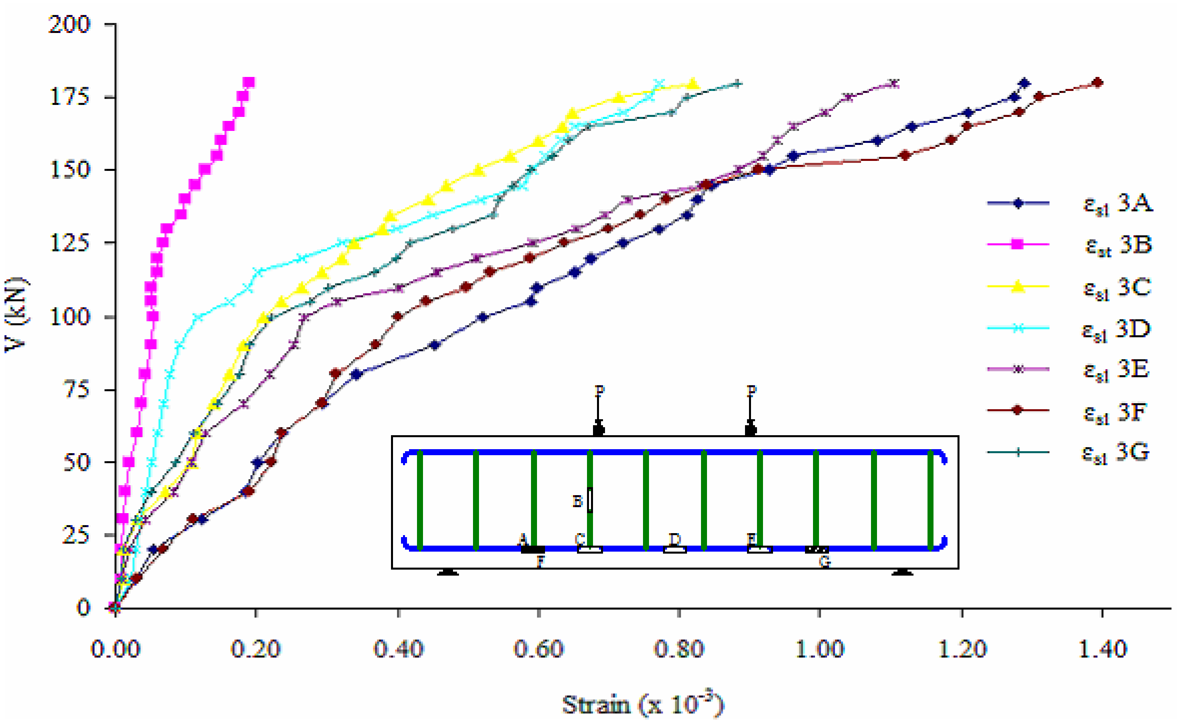


Fig. 5.5 Shear Force versus Steel Strains for Specimen-III

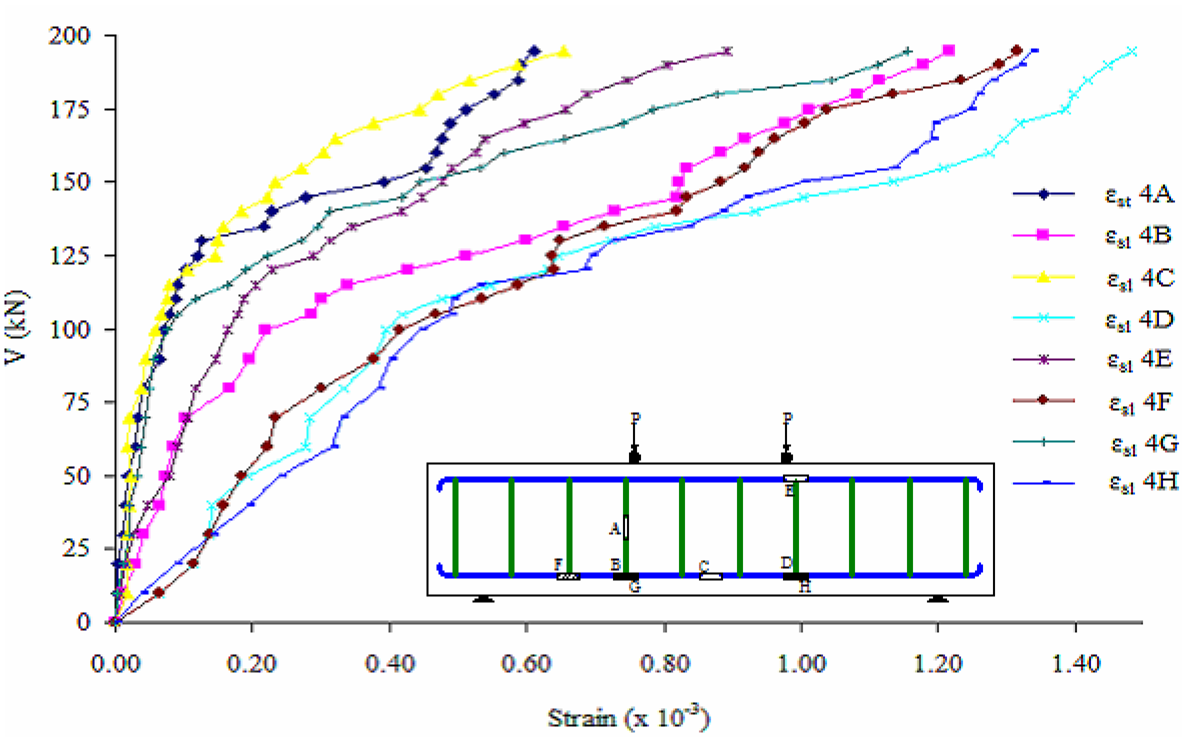


Fig. 5.6 Shear Force versus Steel Strains for Specimen-IV

Note: ε_{sl} Strain in longitudinal tensile steel

ε_{st} Strain in transverse steel

5.3 EFFECTS OF PARAMETERS BASED ON AVAILABLE RESEARCH

Effect of parameters based on available research on shear strength of reinforced concrete beams is discussed below. The ultimate shear strength of beams (V_u) was divided by bd_o to give the nominal shear stress V_u/bd_o at failure.

Following are the effects of parameters considered:

1) Concrete cover to shear reinforcement cage: The concrete cover to the shear reinforcement spells at the time of failure and that the effective width of a beam in shear is equal to the total beam width less the clear cover in both side of a beam.

2) Shear reinforcement ratio: The trades of increasing shear strength with increase in the shear reinforcement ratio.

3) Longitudinal tensile steel ratio: The shear strength against the longitudinal tensile reinforcement ratio indicates very small increases in the shear stress V_e/b_vd_o.

4) Overall beam depth: The shear strength increase of shear stress at failure with expanding depth.

5) Shear span to depth ratio: The shear strength increase sharply when a/d_o decreased due to arch action.

6) Concrete compressive strength: The concrete compressive strength had little influence on the shear strength of beams. This is contrary to the conventionally accepted understanding of the effect of concrete compressive strength on shear strength.

5.4 CORRELATION OF TEST RESULTS WITH PREDICTIONS BY THEORY

5.4.1 Shear Strength of Beams

The shear strength of test beams was calculated using the theory presented in Chapter 3. The specimens from previous investigations were included in the strength comparisons. The test results of the previous studies were given in Chapter 2.

Only beams that failed in shear were considered.

Before studying the correlation between measured and predicted shear strength, the following points need attention:

- The concrete compressive strength of the test beams from various investigations ranged from 60 MPa (beam B50-11-3 tested by Mphonde [6]) to 125.3 MPa (beam 10 tested by Roller and Russell [9]).
- Nearly all the test beams were simply supported and loaded by one or two concentrated load. The critical section for shear failure was taken to be at a distance d_o from the concentrated load.

Comparisons of test shear strength to prediction by the theory are presented in Table 5.2. There were 133 test results altogether. The mean $V_{e(Exp.)}/V_{u(theory)}$ value of the ultimate shear strengths is 0.89 with a coefficient of variation of 0.21.

Also, Correlation of test and predicted shear strengths in present study are presented in Table 5.3 (Present Study). The mean $V_{e(Exp.)}/V_{u(theory)}$ value of the ultimate shear strengths is 1.17 with a coefficient of variation of 0.17. The correlation of test versus predicted shear strengths of the beams is shown in Fig. 5.7. The majority of the test data fall either within a ±20% band of the ideal 1:1 test shear capacity versus predicted shear capacity line, or above this bend.

A summary of correlation is shown in Table 5.4.

Only the shear strength of beams tested by Roller and Russell [9], Yoon et al. [25], and Yuichi et al. [19] are greater then 800 kN. Most of the other results are lumped into lower region of the graph in Fig. 5.8. These results can be seen more clearly if the data points for Roller and Russell [9], Yoon et al. [25], and Yuichi et al. [19] are excluded as shown in Fig. 5.8.

Table 5.2 Correlation of test and predicted shear strengths

Source	Beam Mark	V_e Experiment (kN)	V_u theory, (kN)	$\frac{V_{e(Exp)}}{V_{u,(theory)}}$
Xie et al. [4]	NHW-2	178.6	171.7	1.04
	NHW-3	102.6	169.3	0.61
	NHW-4	94	157.4	0.60
Sarsam and Al-Musawi [5]	AL2-H	122.6	161.1	0.76
	AS2-H	201.0	158.8	1.27
	AS3-H	199.1	248.1	0.80
	BL2-H	138.3	166.2	0.83
	BS2-H	223.5	168.0	1.33
	BS3-H	228.1	253.7	0.90
	CL2-H	147.2	172.4	0.85
	CS2-H	247.2	172.3	1.43
	CS3-H	247.2	253.4	0.98
	CS4-H	220.7	327.9	0.67
Johnson and Ramirez [7]	3	262.7	313.0	0.84
	4	315.9	313.0	1.01
Ganwei and Nielsen [8]	S-8-A	125	209.4	0.60
	S-8-B	135	217.8	0.62
Roller and Russell [9]	2	1099.1	1510.9	0.73
	6	665.1	595.1	1.12
	7	787.6	1270.0	0.62
	8	482.6	427.7	1.13
	9	749.1	938.7	0.80
	10	1171.7	1438.4	0.81
Watanabe [10]	PB-4	730.0	1203.2	0.61
	B-6	291.0	488.2	0.60
Agussalim et al. [12]	No. 7	169.5	240.0	0.71
	No. 12	182.9	281.6	0.65
Ozcebe, Ersoy and Tankut [13]	TH 56	103.5	151.0	0.69
	TS 56	129.2	216.2	0.60
	ACI 59	96.5	105.3	0.92
	TH 59	119.3	153.7	0.78
	ACI 36	105.3	112.9	0.93
	TH 36	140.9	138.5	1.02
	TS 36	155.9	203.2	0.77
	ACI 39	111.8	112.5	0.99
	TH 39	142.9	140.5	1.02
	TS39	179.2	236.5	0.76
Curtin University [14]	B22	237.0	210.9	1.12
	B24	255.0	202.7	1.26
	C22	311.0	219.2	1.42
	C23	379.0	286.0	1.33
	C24	301.0	223.6	1.35
	D22	290.0	213.6	1.36
	D23	344.0	286.0	1.20
	D24	295.0	205.5	1.44
	D25	404.0	317.2	1.27

Continued

Table 5.2 Correlation of test and predicted shear strengths (*Continued*)

Source	**Beam Mark**	**V_e Experiment (kN)**	**V_u theory, (kN)**	**$\frac{V_{e(Exp)}}{V_{u,(theory)}}$**
Kong and Rangan [11]	S1-1	228.3	336.8	0.68
	S1-2	208.3	336.8	0.62
	S1-3	206.1	336.8	0.61
	S1-4	277.9	336.8	0.83
	S1-5	253.3	336.8	0.75
	S1-6	224.1	336.8	0.67
	S2-1	260.3	208.7	1.25
	S2-2	232.5	254.1	0.91
	S2-3	253.3	319.6	0.79
	S2-4	219.4	319.6	0.69
	S2-5	282.1	424.7	0.66
	S2-6	359.0	524.8	0.68
	S3-1	209.2	236.0	0.89
	S3-2	178.0	236.0	0.75
	S3-3	228.6	233.3	0.98
	S3-4	174.9	233.3	0.75
	S3-5	296.6	235.6	1.26
	S3-6	282.9	235.6	1.20
	S4-1	354.0	545.0	0.65
	S4-2	572.8	437.9	1.31
	S4-3	243.4	347.2	0.70
	S4-4	258.1	293.6	0.88
	S4-5	321.1	246.9	1.30
	S4-6	202.9	197.1	1.03
	S5-1	241.7	290.2	0.83
	S5-2	259.9	290.2	0.90
	S5-3	243.8	290.2	0.84
	S6-1	155.4	133.5	1.16
	S6-2	155.1	133.5	1.16
	S6-3	178.4	230.8	0.77
	S6-4	214.4	230.8	0.93
	S6-5	297.0	233.1	1.27
	S6-6	287.2	233.1	1.23
	S7-1	217.2	202.7	1.07
	S7-2	205.4	247.1	0.83
	S7-3	246.5	311.6	0.79
	S7-4	273.6	390.1	0.70
	S7-5	304.4	444.4	0.68
	S7-6	310.6	515.1	0.60
	S8-1	272.1	205.7	1.32
	S8-2	250.9	250.6	1.00
	S8-3	309.6	315.7	0.98
	S8-4	265.8	315.7	0.84
	S8-5	289.2	394.9	0.73
	S8-6	283.9	449.5	0.63
Elzanty et al. [20]	G4	150.0	156.7	0.96

Continued

Table 5.2 Correlation of test and predicted shear strengths (*Continued*)

Source	**Beam Mark**	**V_e Experiment (kN)**	**V_u theory, (kN)**	**$\frac{V_{e(Exp)}}{V_{u,(theory)}}$**
Gabrielsson [17]	S2	172.5	221.5	0.78
	S3	210.0	270.8	0.78
	HS1	250.5	420.7	0.60
	HPS1	324.0	405.0	0.80
	HPS2	305.0	399.1	0.76
	HB1	322.0	405.9	0.79
Cladera and Mari [21]	H60 / 2	179.74	290.1	0.62
	H60 / 4	308.71	474.7	0.65
	H75 / 2	203.94	287.6	0.71
	H100 / 2	225.55	302.9	0.74
	H100 / 3	253.64	420.2	0.60
	H100 / 4	266.53	432.4	0.62
Pendyala and Mendis [22]	210(100)(2)	18.7	27.3	0.68
	2_210(100)(2)	21.5	28.8	0.75
Ahmad, Xie and Yu [18]	LHW-1	277.37	206.1	1.35
	LHW-2	135.43	203.4	0.67
Tanimura and Sato [24]	46 (F)	1243.0	1139.1	1.09
	47 (F)	1300.0	2069.8	0.63
	48 (F)	932.0	1146.8	0.81
Yoon, Cook and Mitchell [25]	M1-N	405.0	406.9	1.00
	M2-S	552.0	602.3	0.92
	M2-N	689.0	871.1	0.79
	H1-N	483.0	346.1	1.40
	H2-S	598.0	643.3	0.93
	H2-N	721.0	1123.9	0.64
Ahmad et al. [23]	A1H	71.5	117.1	0.61
	A2H	76.1	112.0	0.68
Mphonde [6]	B50-11-3	98.1	80.7	1.22
	B100-11-3	152.1	159.9	0.95
	B100-15-3	115.9	146.6	0.79
	B150-11-3	161.9	240.6	0.67
	B150-15-3	150.3	224.6	0.67
Kriski and Loov [16]	7	304.5	322.5	0.94
	8	391.0	314.6	1.24
	9	242.0	316.3	0.76
	10	390.5	317.9	1.23
Thirugnansundralingam et al. [15]	7	111.0	173.9	0.64
	9	113.0	110.8	1.02
Yuichi et al. [19]	B-6	297	458.9	0.65
	B-7	443.4	743.9	0.60
	B-8	480.2	779.5	0.62

Mean :	0.88
Coefficient of Variation :	0.20

Table 5.3 Correlation of test and predicted shear strengths (*Present Study*)

Source	Beam Mark	V_e Experiment (kN)	V_u theory (kN)	$\frac{V_{e(Exp)}}{V_{u,(theory)}}$
Present Study	1	130.0	136.6	0.95
	2	145.0	136.6	1.06
	3	180.0	139.1	1.29
	4	195.0	139.1	1.40
			Mean :	1.17
			Coefficient of Variation :	0.17

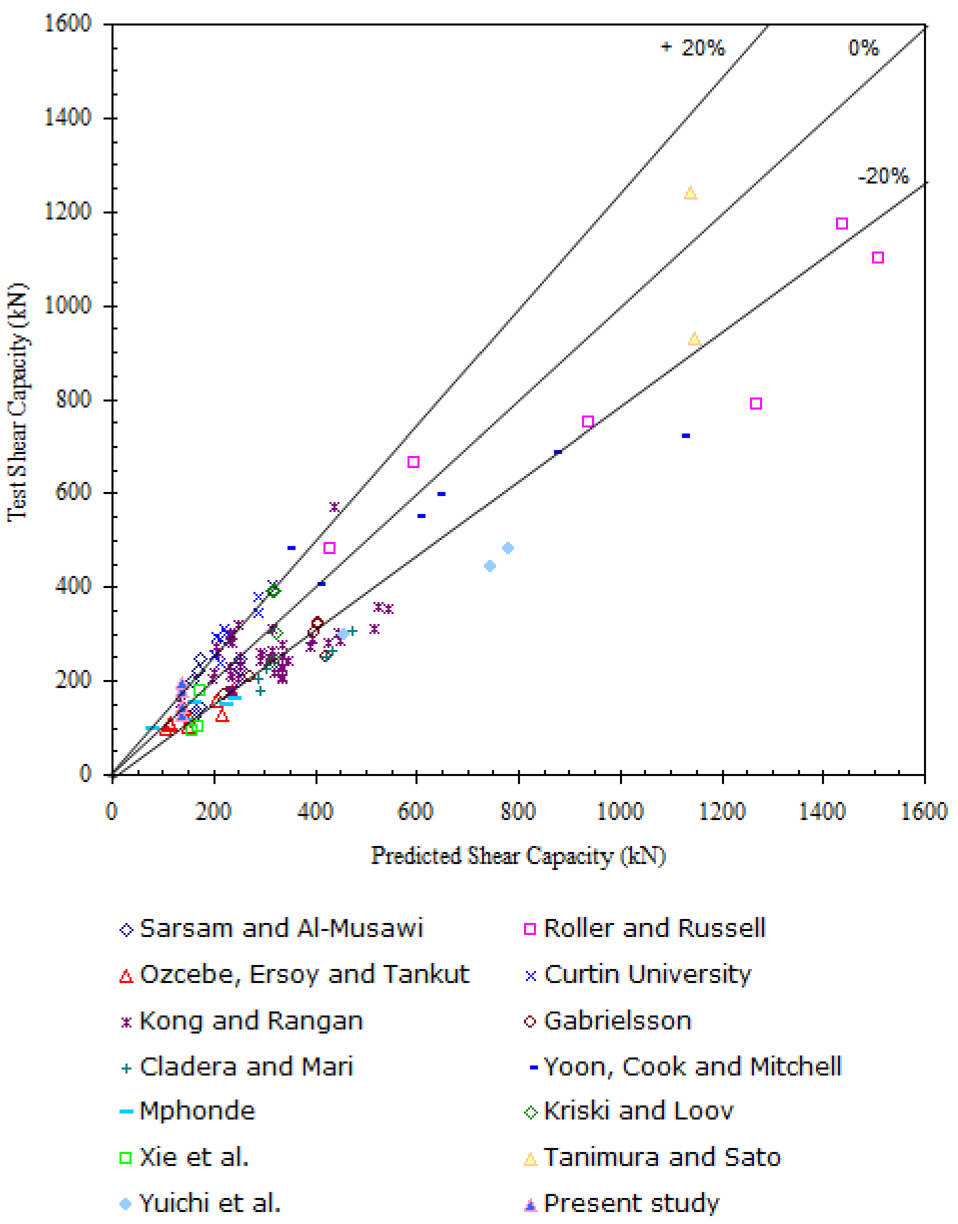


Fig. 5.7 Correlation of Test and Predicted Shear Strengths

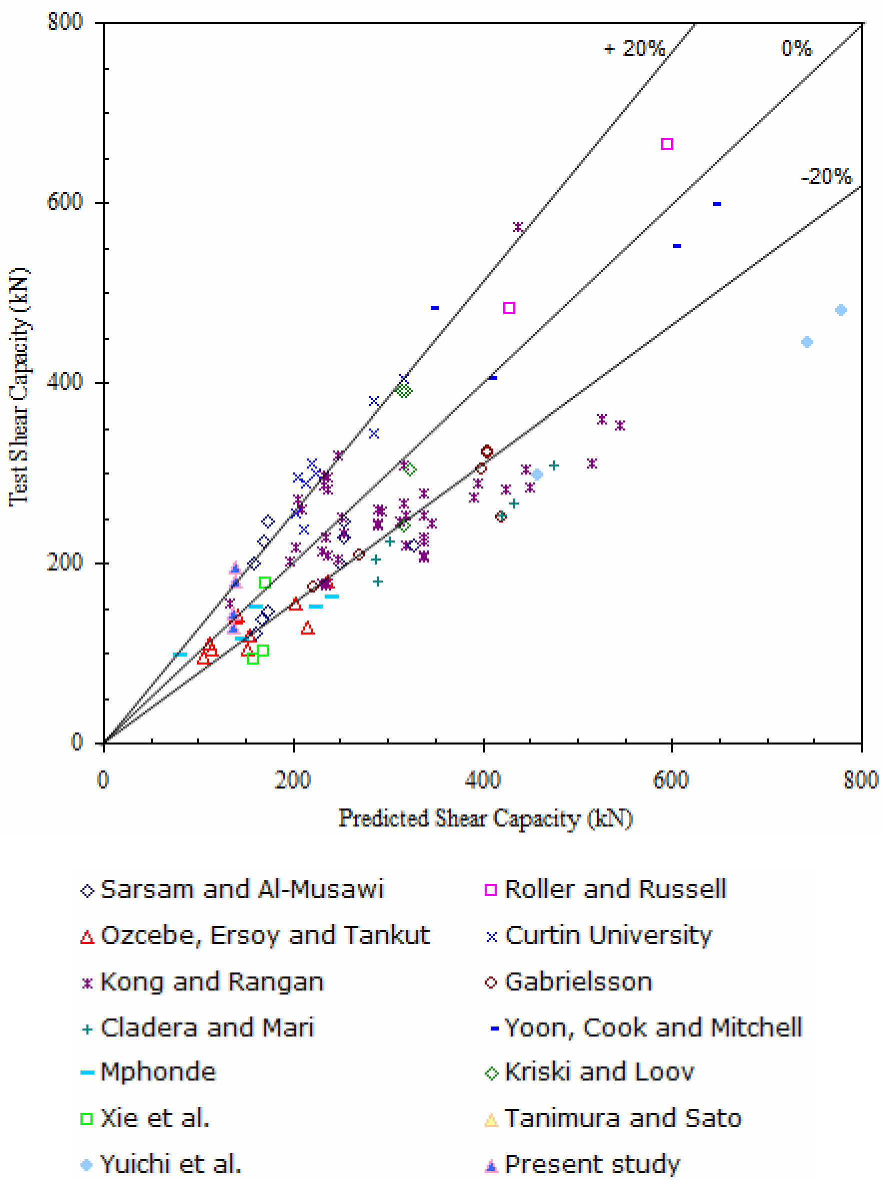


Fig. 5.8 Correlation of Test and Predicted Shear Strengths

The mean $V_{e(Exp.)}/V_{u(theory)}$ value for the nine beams test at Curtin University [14] is 1.31 with a coefficient of variation of 0.08. The theory predicted the shear strength well.

For the beams reported by Ganwei and Nielsen [8], the theory predicted the shear strength well, with a mean $V_{e(Exp.)}/V_{u(theory)}$ value for two beams is 0.608 with a coefficient of variation of 0.01.

The mean $V_{e(Exp.)}/V_{u(theory)}$ value for the forty-five beams reported by Kong and Rangan [11] is 0.89 with a coefficient of variation of 0.19. The theory predicted the shear strength well.

The beams tested by Roller and Russell [9], had fairly deep sections witch gave some of the largest shear capacities compared to all other beams reported hear. The mean $V_{e(Exp.)}/V_{u(theory)}$ value is 0.87 for these beams, with a coefficient of variation of 0.17.

For the beams tested by Ahmad et al. [18], the mean $V_{e(Exp.)}/V_{u(theory)}$ value for two beams is 1.01 with a coefficient of variation of 0.34. The test results were conservative when compared with predictions from the theory.

Table 5.4 Summary of correlation

Source	**No. of test beams**	**$V_{e(Exp)}/V_{u(Theory)}$ ratio**	
		Mean	**Coefficient of Variation,**
Xie et al. [4]	3	0.748	0.19
Sarsam and Al-Musawi [5]	10	0.983	0.22
Mphonde [6]	5	0.860	0.18
Johnson and Ramirez [7]	2	0.924	0.09
Ganwei and Nielsen [8]	2	0.608	0.01
Roller and Russell [9]	6	0.868	0.17
Watanabe [10]	2	0.601	0.01
Kong and Rangan [11]	45	0.892	0.19
Agussalim et al. [12]	2	0.678	0.03
Ozcebe et al. [13]	10	0.846	0.13
Curtin University [14]	9	1.305	0.08
Thirugnansundralingam et al. [15]	2	0.829	0.19
Kriski and Loov [16]	4	1.045	0.19
Gabrielsson [17]	6	0.751	0.05
Ahmad, Xie, and Yu [18]	2	1.006	0.34
Iwai Yuichi et al. [19]	3	0.620	0.02
Elzanty et al. [20]	1	0.957	0.00
Cladera and Mari [21]	6	0.657	0.05
Pendyala and Mendis [22]	2	0.716	0.03
Ahmad et al. [23]	2	0.645	0.03
Tanimura, and Sato [24]	3	0.844	0.16
Yoon et al. [25]	6	0.945	0.17
Present Study	4	1.171	0.17
Total Test Data	**137**	**0.893**	**0.21**

From Table 5.5, it can be seen that the theory predicted the shear strength well, for f'_c > 90 MPa with a mean $V_{e(Exp.)}/V_{u(theory)}$ value 0.78 with a coefficient of variation of 0.12. The test results were slight conservative when $f'_c \leq 70$ MPa.

Table 5.5 Test Shear Strength/Predicted Shear Strength Values

Parameter	Category	n	All the test Results $V_{e(Exp)}/V_{u(Theory)}$ Mean	C.O.V
Concrete Compressive Strength (MPa)	$f'_c \leq 70$ MPa	40	0.97	0.24
	70 Mpa < f'_c < 80 Mpa	48	0.88	0.18
	80 MPa < f'_c < 90 Mpa	28	0.83	0.19
	f'_c > 90 MPa	17	0.78	0.12
Amount of Shear Reinforcement Ratio (%)	ρ_t (%) ≤ 0.15	51	1.05	0.19
	0.15 < ρ_t (%) < 0.20	37	0.85	0.16
	0.20 < ρ_t (%) < 0.30	22	0.73	0.11
	ρ_t (%) > 0.30	23	0.71	0.11

Note: n is the number of beam specimens.
C.O.V. is the coefficient of variation.

When the amount of shear reinforcement ratio used was less then the minimum amount of required by the ACI 318-05 [2] method, the predicted shear strength of a beam could be unconservative. For grater amount of shear reinforcement (0.20 < ρ_t (%) < 0.30 and ρ_t (%) > 0.30), the theory predicted the shear strength well.

5.5 CORRELATION OF TEST SHEAR STRENGTH WITH PREDICTIONS BY CODES

Various code provisions for shear strength of concrete beams were described in Chapter 2. The experimental shear strength of the 137 beams tested in the present study and previous investigations are compared to the predictions by the following:

1. Indian Standard IS 456: 2000 [1]
2. American Concrete Institute Building Code ACI 318-05 [2], and
3. Euro code EC2 Part I [3].

The comparisons of test shear strengths to predictions by the IS 456: 2000 [1], ACI 318-05 [2] and EC2 Part-I [3] codes are given in Table 5.6. A summary of the correlation is given in Table 5.7.

Table 5.6 Correlation of test and predicted shear strengths by Code

Source	Beam Mark	$V_{e(Exp)}$, (kN)	$V_{u,IS:456}$ (kN)	$V_{u,ACI\ 318\text{-}05}$ (kN)	$V_{u,EC2\ Part\text{-}I}$ (kN)	$\frac{V_{e(Exp)}}{V_{u,IS:456}}$	$\frac{V_{e(Exp)}}{V_{u,ACI\ 318\text{-}05}}$	$\frac{V_{e(Exp)}}{V_{u,EC2\ Part\text{-}I}}$
Xie et al. [4]	NHW-2	178.6	34.36	95.21	119.64	5.20	1.88	1.49
	NHW-3	102.6	34.52	95.98	119.64	2.97	1.07	0.86
	NHW-4	94	34.54	96.11	119.64	2.72	0.98	0.79
Sarsam and Al-Musawi [5]	AL2-H	122.6	42.97	81.87	43.81	2.85	1.50	2.80
	AS2-H	201.0	42.65	80.91	43.25	4.71	2.48	4.65
	AS3-H	199.1	42.76	90.25	65.96	4.66	2.21	3.02
	BL2-H	138.3	46.75	83.10	47.39	2.96	1.66	2.92
	BS2-H	223.5	46.63	82.35	47.39	4.79	2.71	4.72
	BS3-H	228.1	46.59	92.79	71.34	4.90	2.46	3.20
	CL2-H	147.2	50.36	80.87	47.65	2.92	1.82	3.09
	CS2-H	247.2	50.37	80.92	47.65	4.91	3.05	5.19
	CS3-H	247.2	50.72	93.30	71.74	4.87	2.65	3.45
	CS4-H	220.7	50.85	104.39	95.31	4.34	2.11	2.32
Johnson and Ramirez [7]	3	262.7	171.51	304.95	151.47	1.53	0.86	1.73
	4	315.9	171.51	304.95	151.47	1.84	1.04	2.09
Ganwei and Nielsen [8]	S-8-A	125	34.15	91.93	123.10	3.66	1.36	1.02
	S-8-B	135	34.15	91.93	123.10	3.95	1.47	1.10
Roller and Russell [9]	2	1099.1	234.52	740.42	765.98	4.69	1.48	1.43
	6	665.1	311.69	634.60	294.48	2.13	1.05	2.26
	7	787.6	325.10	767.79	594.16	2.42	1.03	1.33
	8	482.6	339.77	798.91	306.54	1.42	0.60	1.57
	9	749.1	373.44	926.74	594.16	2.01	0.81	1.26
	10	1171.7	403.59	1039.15	847.09	2.90	1.13	1.38
Watanabe [10]	PB-4	730.0	50.00	1011.08	1157.24	14.60	0.72	0.63
	B-6	291.0	42.67	248.02	440.00	6.82	1.17	0.66
Agussalim et al. [12]	No. 7	169.5	72.17	165.12	210.03	2.35	1.03	0.81
	No. 12	182.9	66.52	150.31	210.03	2.75	1.22	0.87
Ozcebe , Ersoy and Tankut [13]	TH 56	103.5	56.70	97.69	78.63	1.83	1.06	1.32
	TS 56	129.2	56.45	111.75	112.52	2.29	1.16	1.15
	ACI 59	96.5	51.45	99.89	63.70	1.88	0.97	1.51
	TH 59	119.3	50.96	106.55	85.69	2.34	1.12	1.39
	ACI 36	105.3	51.06	97.55	65.44	2.06	1.08	1.61
	TH 36	140.9	51.06	103.40	78.63	2.76	1.36	1.79
	TS 36	155.9	51.06	118.47	112.52	3.05	1.32	1.39
	ACI 39	111.8	55.60	95.85	63.70	2.01	1.17	1.76
	TH 39	142.9	55.60	102.16	77.90	2.57	1.40	1.83
	TS39	179.2	55.60	124.36	127.85	3.22	1.44	1.40
Curtin University [14]	B22	237.0	70.30	121.49	80.56	3.37	1.95	2.94
	B24	255.0	70.16	120.31	78.52	3.63	2.12	3.25
	C22	311.0	74.15	111.91	74.21	4.19	2.78	4.19
	C23	379.0	73.84	121.76	98.92	5.13	3.11	3.83
	C24	301.0	73.98	110.79	72.30	4.07	2.72	4.16
	D22	290.0	80.64	112.67	75.48	3.60	2.57	3.84
	D23	344.0	80.64	123.85	100.62	4.27	2.78	3.42
	D24	295.0	80.46	111.55	73.55	3.67	2.64	4.01
	D25	404.0	80.28	130.31	116.37	5.03	3.10	3.47

Continued

Table 5.6 Correlation of test and predicted shear strengths by Code (*Continued*)

Source	Beam Mark	$V_{e(Exp)}$, (kN)	$V_{u,IS:456}$ (kN)	$V_{u,ACI\ 318\text{-}05}$ (kN)	$V_{u,EC2\ Part\text{-}I}$ (kN)	$\frac{V_{e(Exp)}}{V_{u,IS:456}}$	$\frac{V_{e(Exp)}}{V_{u,ACI\ 318\text{-}05}}$	$\frac{V_{e(Exp)}}{V_{u,EC2\ Part\text{-}I}}$
Kong and Rangan [11]	S1-1	228.3	79.62	150.77	116.56	2.87	1.51	1.96
	S1-2	208.3	79.62	150.77	116.56	2.62	1.38	1.79
	S1-3	206.1	79.62	150.77	116.56	2.59	1.37	1.77
	S1-4	277.9	79.62	150.77	116.56	3.49	1.84	2.38
	S1-5	253.3	79.62	150.77	116.56	3.18	1.68	2.17
	S1-6	224.1	79.62	150.77	116.56	2.81	1.49	1.92
	S2-1	260.3	80.84	140.31	77.95	3.22	1.86	3.34
	S2-2	232.5	80.84	147.24	93.54	2.88	1.58	2.49
	S2-3	253.3	80.84	157.47	116.56	3.13	1.61	2.17
	S2-4	219.4	80.84	157.47	116.56	2.71	1.39	1.88
	S2-5	282.1	80.84	174.63	155.16	3.49	1.62	1.82
	S2-6	359.0	80.84	192.12	194.51	4.44	1.87	1.85
	S3-1	209.2	66.22	137.37	75.93	3.16	1.52	2.76
	S3-2	178.0	66.22	137.37	75.93	2.69	1.30	2.34
	S3-3	228.6	80.33	135.67	75.24	2.85	1.68	3.04
	S3-4	174.9	80.33	135.67	75.24	2.18	1.29	2.32
	S3-5	296.6	91.04	137.70	75.08	3.26	2.15	3.95
	S3-6	282.9	91.04	137.70	75.08	3.11	2.05	3.77
	S4-1	354.0	157.66	311.38	216.35	2.25	1.14	1.64
	S4-2	572.8	128.11	251.77	169.78	4.47	2.28	3.37
	S4-3	243.4	98.35	198.51	137.50	2.47	1.23	1.77
	S4-4	258.1	82.47	167.76	116.56	3.13	1.54	2.21
	S4-5	321.1	71.65	141.50	96.80	4.48	2.27	3.32
	S4-6	202.9	55.80	112.98	77.29	3.64	1.80	2.63
	S5-1	241.7	82.67	169.14	116.56	2.92	1.43	2.07
	S5-2	259.9	82.67	169.14	116.56	3.14	1.54	2.23
	S5-3	243.8	82.67	169.14	116.56	2.95	1.44	2.09
	S6-1	155.4	66.35	138.52	75.93	2.34	1.12	2.05
	S6-2	155.1	66.35	138.52	75.93	2.34	1.12	2.04
	S6-3	178.4	80.54	136.80	75.24	2.22	1.30	2.37
	S6-4	214.4	80.54	136.80	75.24	2.66	1.57	2.85
	S6-5	297.0	91.31	138.85	75.08	3.25	2.14	3.96
	S6-6	287.2	91.31	138.85	75.08	3.15	2.07	3.83
	S7-1	217.2	97.33	141.48	75.19	2.23	1.54	2.89
	S7-2	205.4	97.33	148.17	90.23	2.11	1.39	2.28
	S7-3	246.5	97.33	158.03	112.42	2.53	1.56	2.19
	S7-4	273.6	97.33	170.44	140.35	2.81	1.61	1.95
	S7-5	304.4	97.33	179.35	160.40	3.13	1.70	1.90
	S7-6	310.6	97.33	191.45	187.61	3.19	1.62	1.66
	S8-1	272.1	81.09	141.83	77.95	3.36	1.92	3.49
	S8-2	250.9	81.09	148.76	93.54	3.09	1.69	2.68
	S8-3	309.6	81.09	158.99	116.56	3.82	1.95	2.66
	S8-4	265.8	81.09	158.99	116.56	3.28	1.67	2.28
	S8-5	289.2	81.09	171.86	145.51	3.57	1.68	1.99
	S8-6	283.9	81.09	181.10	166.30	3.50	1.57	1.71
Elzanty et al. [20]	G4	150.0	57.91	99.67	79.66	2.59	1.50	1.88

Continued

Table 5.6 Correlation of test and predicted shear strengths by Code (*Continued*)

Source	Beam Mark	$V_{e(Exp)}$, (kN)	$V_{u,IS:456}$ (kN)	$V_{u,ACI\ 318\text{-}05}$ (kN)	$V_{u,EC2\ Part\text{-}I}$ (kN)	$\frac{V_{e(Exp)}}{V_{u,IS:456}}$	$\frac{V_{e(Exp)}}{V_{u,ACI\ 318\text{-}05}}$	$\frac{V_{e(Exp)}}{V_{u,EC2\ Part\text{-}I}}$
Gabrielsson [17]	S2	172.5	35.81	92.94	109.91	4.82	1.86	1.57
	S3	210.0	36.72	114.13	146.24	5.72	1.84	1.44
	HS1	250.5	60.50	182.06	229.96	4.14	1.38	1.09
	HPS1	324.0	56.55	175.98	225.22	5.73	1.84	1.44
	HPS2	305.0	56.84	177.81	225.22	5.37	1.72	1.35
	HB1	322.0	60.29	156.28	193.05	5.34	2.06	1.67
Cladera and Mari [21]	H60 / 2	179.74	74.25	143.36	111.99	2.42	1.25	1.60
	H60 / 4	308.71	81.89	176.94	188.75	3.77	1.74	1.64
	H75 / 2	203.94	75.44	152.59	117.55	2.70	1.34	1.73
	H100 / 2	225.55	77.18	172.31	135.82	2.92	1.31	1.66
	H100 / 3	253.64	77.05	195.20	188.75	3.29	1.30	1.34
	H100 / 4	266.53	85.59	195.20	188.75	3.11	1.37	1.41
Pendyala and Mendis [22]	210(100)(2)	18.7	11.70	25.65	16.84	1.60	0.73	1.11
	2_210(100)(2)	21.5	11.60	24.83	16.84	1.85	0.87	1.28
Ahmad, Xie, and Yu [18]	LHW-1	277.37	38.95	102.62	134.36	7.12	2.70	2.06
	LHW-2	135.43	39.17	103.51	134.36	3.46	1.31	1.01
Tanimura and Sato [24]	46 (F)	1243.0	119.05	341.63	372.09	10.44	3.64	3.34
	47 (F)	1300.0	118.91	553.17	637.24	10.93	2.35	2.04
	48 (F)	932.0	118.69	338.90	372.09	7.85	2.75	2.50
Yoon, Cook and Mitchell [25]	M1-N	405.0	283.66	422.35	181.27	1.43	0.96	2.23
	M2-S	552.0	283.66	455.76	256.43	1.95	1.21	2.15
	M2-N	689.0	283.66	501.94	360.33	2.43	1.37	1.91
	H1-N	483.0	292.46	470.04	181.27	1.65	1.03	2.66
	H2-S	598.0	292.46	527.03	309.49	2.04	1.13	1.93
	H2-N	721.0	292.46	618.40	515.08	2.47	1.17	1.40
Ahmad et al. [23]	A1H	71.5	19.09	54.26	57.36	3.75	1.32	1.25
	A2H	76.1	19.30	55.95	57.36	3.94	1.36	1.33
Mphonde [6]	B50-11-3	98.1	51.78	83.36	53.69	1.89	1.18	1.83
	B100-11-3	152.1	52.70	117.47	120.81	2.89	1.29	1.26
	B100-15-3	115.9	53.82	123.38	120.81	2.15	0.94	0.96
	B150-11-3	161.9	52.79	141.73	174.46	3.07	1.14	0.93
	B150-15-3	150.3	53.88	147.56	174.46	2.79	1.02	0.86
Kriski and Loov [16]	7	304.5	120.92	232.55	113.74	2.52	1.31	2.68
	8	391.0	121.44	236.79	113.74	3.22	1.65	3.44
	9	242.0	121.33	235.83	113.74	1.99	1.03	2.13
	10	390.5	121.22	234.98	113.74	3.22	1.66	3.43
Thirugnansundral-ingam et al. [15]	7	111.0	54.19	139.11	144.72	2.05	0.80	0.77
	9	113.0	54.19	117.60	96.34	2.09	0.96	1.17
Yuichi et al. [19]	B-6	297	44.76	268.71	476.67	6.64	1.11	0.62
	B-7	443.4	44.76	372.78	533.17	9.91	1.19	0.83
	B-8	480.2	44.76	711.00	743.01	10.73	0.68	0.65
Present study	1	130.0	21.89	104.34	60.17	5.94	1.25	2.16
	2	145.0	25.76	104.34	72.18	5.63	1.39	2.01
	3	180.0	30.75	103.08	88.47	5.85	1.75	2.03
	4	195.0	33.07	103.08	93.04	5.90	1.89	2.10

Table 5.7 Summary of correlation of code predictions

Code Method	**$V_{e(Exp)}/V_u$, ratio**	
	Mean	**Coefficient of Variation,**
IS 456: 2000 [1]	3.63	1.30
ACI 318-05 [2]	1.58	0.44
EC2 Part I [3]	2.14	0.75
Proposed Method	0.89	0.21

The summary of correlation in Table 5.7 indicates significant scatter in the predictions by the codes. For the three methods of prediction, the coefficient of variation ranged from 0.44 (ACI 318-05 [2]) to 1.30 (IS 456: 2000 [1]).

Proposed method gave the best prediction with the smallest scatter. The mean $V_{e(Exp.)}/V_u$ value is 0.89 with a coefficient of variation of 0.21.

All other code methods apart from ACI 318-05 [2] gave overall conservative predictions. The most conservative in estimating the shear strength design of HPC beams were given by the IS 456:2000 [1].

The comparisons in section 5.4.1 and above identified the present theory and the ACI 318-05 [2] method as the most promising methods for determining the shear strength of NSC and HPC beams with vertical shear reinforcement.

Table 5.8 Test Shear Strength/Predicted Shear Strength Values (Code)

Parameter	**Category**	**n**	**All the test Results**					
			$V_{e(Exp)}/V_{u,IS\ 456:2000}$		**$V_{e(Exp)}/V_{u,ACI\ 318-05}$**		**$V_{e(Exp)}/V_{u,EC2\ Part-I}$**	
			Mean	**C.O.V**	**Mean**	**C.O.V**	**Mean**	**C.O.V**
Concrete Compressive Strength (MPa)	60 Mpa < f'_c < 70 Mpa	40	2.99	0.63	1.72	0.50	2.52	0.85
	70 Mpa < f'_c < 80 Mpa	48	3.63	1.23	1.60	0.36	2.32	0.77
	80 MPa < f'_c < 90 Mpa	28	3.18	0.88	1.40	0.33	1.70	0.55
	f'_c > 90 MPa	17	5.32	2.75	1.52	0.63	1.46	0.43
Amount of Shear Reinforcement Ratio (%)	ρ_t (%) $\leq$ 0.15	51	2.90	0.77	1.67	0.50	2.87	0.74
	0.15 < ρ_t (%) < 0.20	37	3.05	0.67	1.60	0.39	2.11	0.48
	0.20 < ρ_t (%) < 0.30	22	3.70	1.21	1.55	0.40	1.53	0.38
	ρ_t (%) > 0.30	23	5.70	2.34	1.42	0.40	1.14	0.36

Note: n is the number of beam specimens.
C.O.V. is the coefficient of variation.

From Table 5.8, it can be seen that the IS 456 :2000 [1] gives most conservative results, for HPC beams (f'_c > 60 MPa) and for the ACI 318 -05 [2], test results were give slight conservative results (when 80 MPa < f'_c < 90 Mpa), with a mean $V_{e(Exp.)}/V_{u,\ ACI\ 318\ -05}$ value 1.40 with a coefficient of variation of 0.33.

For grater amount of shear reinforcement, the IS 456 :2000 [1] code method gave generally more conservative for high strength concrete beams. When the amount of shear reinforcement ratio used was less then the minimum amount of required by the ACI 318-05 [2] method and EC2 Part-I [3], the shear strength of a beam could be unconservative.

5.6 TRANDS OF TEST PARAMETERS

This section compares the trends of test parameters with those predicted by the theory and the code methods. The parameter considered is effect on variation of longitudinal reinforcement ratio.

5.6.1 Shear Strength versus Longitudinal Tensile Reinforcement Ratio

In specimen 1 to 4, the longitudinal tensile reinforcement ration was the test parameter. Fig. 5.9 shows the test shear strengths and the predicated trends. The test results, IS 456:2000 [1] and EC2 Part-I [3] method indicate that the shear strength increased with longitudinal tensile reinforcement ratio.

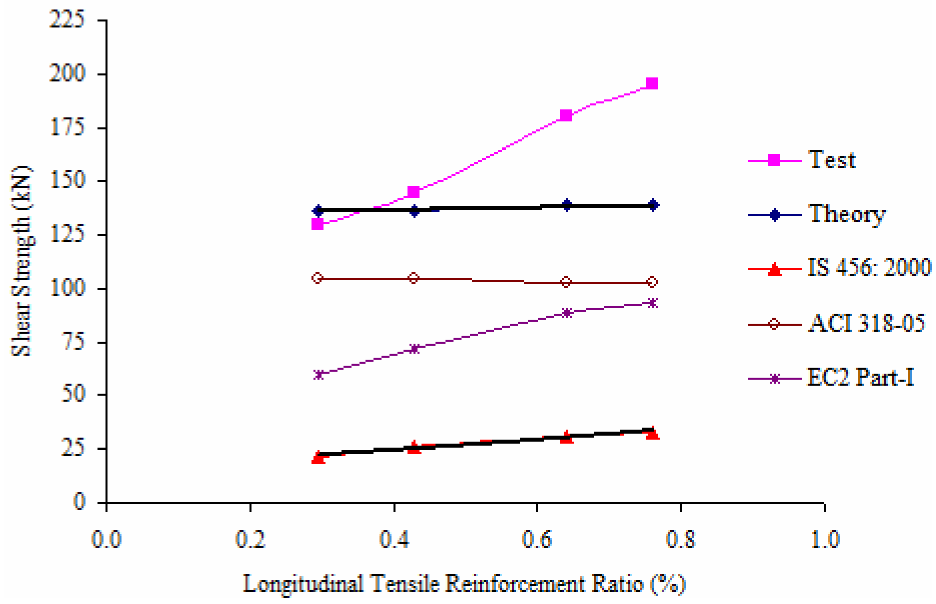


Fig. 5.9 Shear Strength versus Longitudinal Tensile Reinforcement Ratio

Fig. 5.9 shows that, the theory and ACI 318-05 methods indicate significant effect on the shear strength with increases in longitudinal tensile reinforcement ratio.

5.7 SUMMARY

The report presented the analytical investigation on shear strength of reinforced high-performance concrete (HPC) beams with vertical shear reinforcement subjected to shear force. In all, 137 beams were analyzed. The analytical study comprised the development of a theory based on propose theory.

The following conclusions are drawn:

1) The shear strength of beams increased with an increase in the shear reinforcement ratio.
2) The shear strength also increased with an increase in the longitudinal tensile reinforcement ratio. Bundling of bars may have contributed to increased shear resistance.
3) The nominal stress at failure $V_e/b_v d_o$ decreased with increasing overall beam depth.
4) The shear span-to-depth ratio a/d_o did not have a significant effect on the shear strength when $a/d_o \geq 2.50$. However, when $a/d_o < 2.50$, the shear strength increased because of arch action.
5) The ultimate shear strengths predicted by the theory correlated with the test results of other specimens available in the literature.
6) As the amount of shear reinforcement increased, the beams failed in a more ductile manner.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 INTRODUCTION

This chapter presents the conclusions of the present study. The main purpose of this report was to improve the understanding of the behaviour of high-strength reinforced concrete beams failing in shear. In general, both the principal and specific objectives as indicated in Chapter 1 have been met. The findings from the experimental and analytical work with regard to the response and shear strength of High Performance Concrete (HPC) beams subjected to shear force are highlighted hear.

The analytical work involved the development of a theory capable of predicting the deformation and shear strength of a reinforced concrete beam. The theory was based on a truss analogy approach, with due to consideration given to equilibrium, compatibility and material relationships.

The experimental part of the study involved testing of four beams using two symmetrically placed concentrated load condition to conclude the effect on variation of longitudinal reinforcement ratio.

All four beams failed in shear. The deformation in terms of midspan deflection, strain in the longitudinal steel bars were monitored during the tests. All the beams were 200 mm wide and 300 mm deep, with an effective span of about 1.2 meters. The concrete compressive strength ranged from 58 to 62 MPa and the shear span-to-depth ratio was approximately 1.51.

The results from previous investigations were also studied. The shear design provisions given by the Indian Standard IS 456:2000 [1], American Concrete Institute Building Code ACI 318-05 [2], and Euro code EC2 Part I [3] were also examined. Comparisons made between the test shear strength and prediction by the various codes of practice.

Recommendations for future work are proposed at the end of the chapter.

6.2 CONCLUSIONS

The report presented the theoretical investigation of 133 beams on shear strength of reinforced HPC beams with vertical shear reinforcement subjected to shear force. The theoretical study comprised the development of a theory based on the test result of 133 HPC beams in the proposed theory.

The following conclusions are drawn:

- The shear strength of beams increased with an increase in the concrete compressive strength (Fig. 6.1).

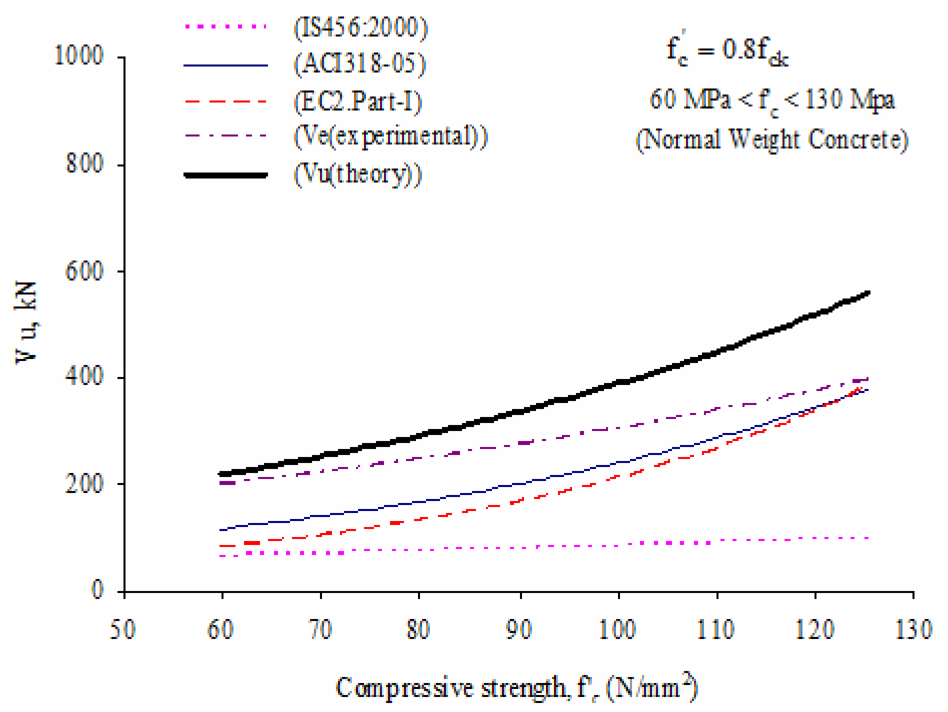


Fig. 6.1 Influence of Concrete Compressive Strength

- The shear strength also increased with an increase in the shear reinforcement ratio (Fig. 6.2) and longitudinal tensile reinforcement ratio (Fig. 6.3). Bundling of bars may have contributed to increased shear resistance.

- The shear span-to-depth ratio a/d_o did not have a significant effect on the shear strength when $a/d_o \geq 2.50$ (Fig. 6.4). However, when $a/d_o < 2.50$, the shear strength increased because of arch action.

- The nominal stress at failure $V_e/b_v d_o$ decreased with increasing overall beam depth.

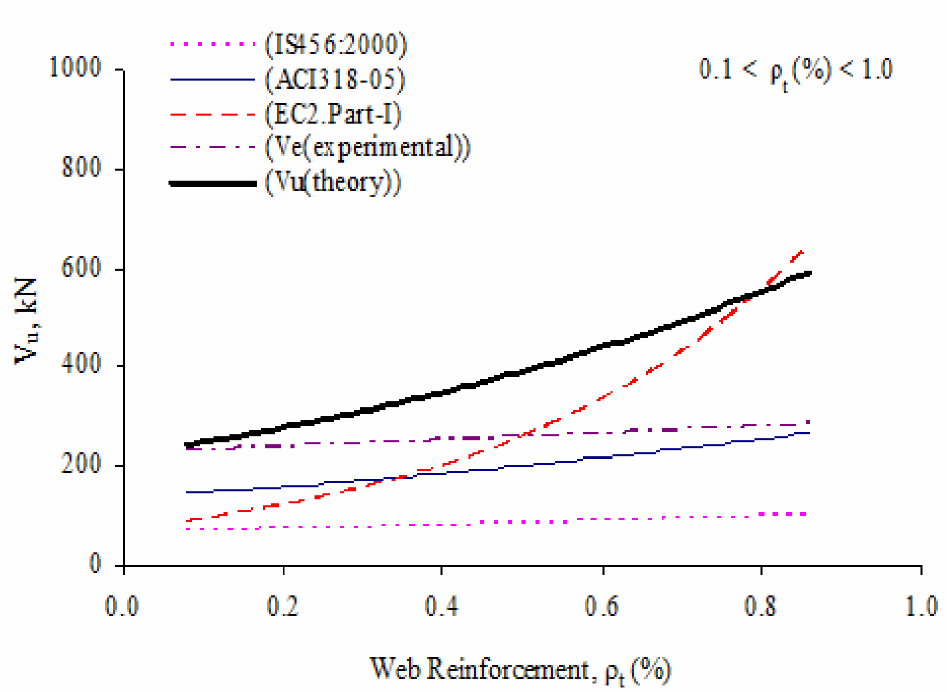


Fig. 6.2 Influence of Web Reinforcement Ratio

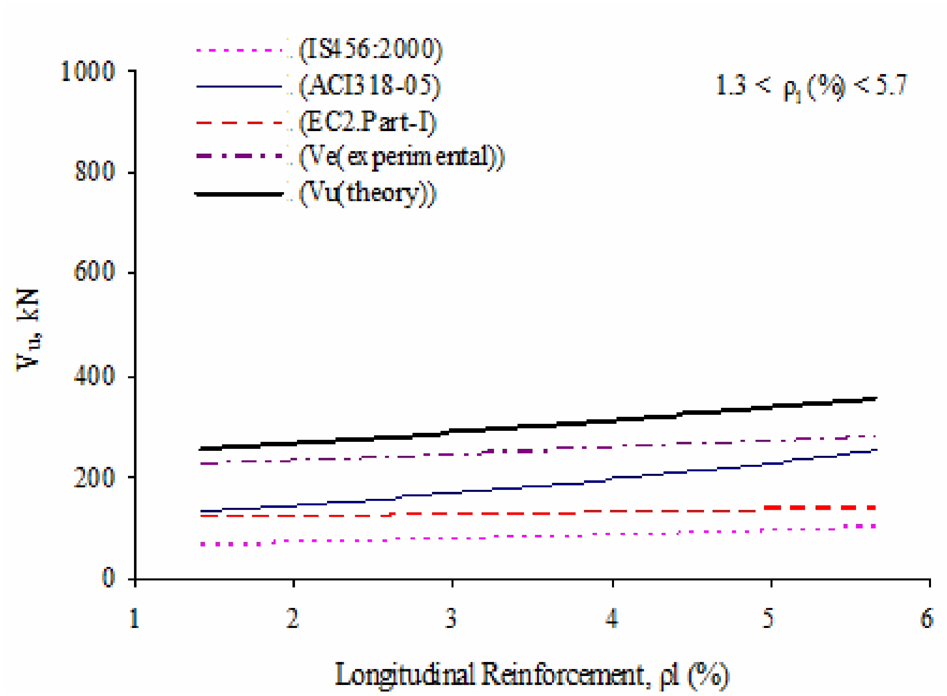


Fig. 6.3 Influence of Longitudinal Reinforcement Ratio

- The theory also predicted the shear strength of reinforced HPC beams from other investigation well. The ultimate shear strengths predicted by the theory correlated with the test results of other specimens available in the literature and it was found that the overall mean Experiment/predicted shear strength ratio of 0.89 with a coefficient of variation of 0.21 for the 133 test results.

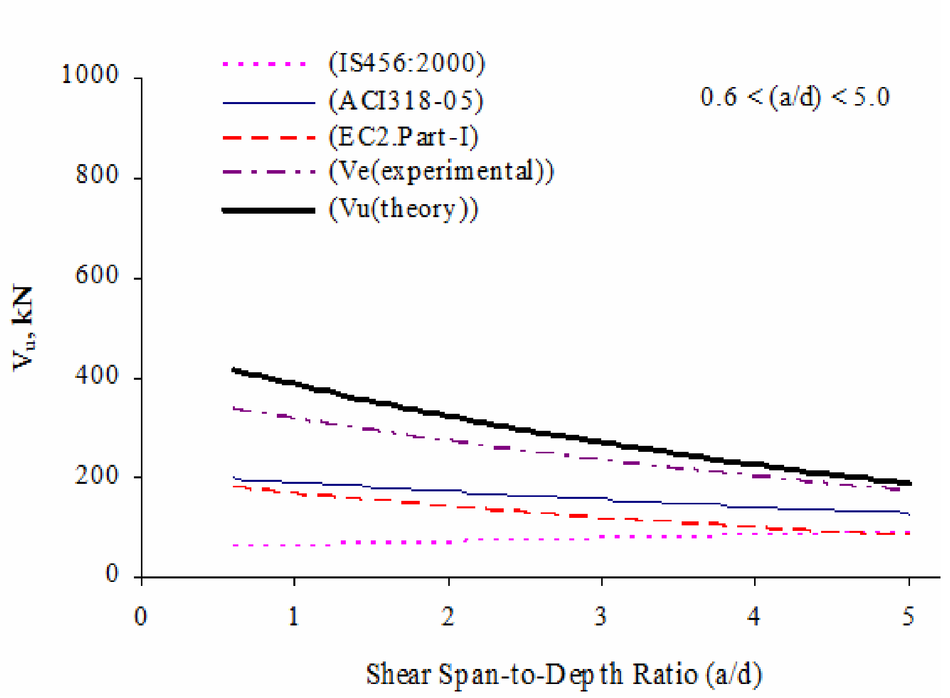


Fig. 6.4 Influence of Shear Span-to-Depth Ratio

- From the investigation, it can be seen that the predictions by the shear design provisions outlined in IS 456:2000 [1], ACI 318-05 [2], and EC2 Part I [3] and from comparison, it was found that the Indian Standard code formula gives most conservative results in estimating the shear strength contribution of HSC. The provision for shear strength contained in ACI 318-05 [2], and EC2 Part I [3] are also conservative and may be adopted for the safe design of HPC beams.

- The proposed theory for concrete shear strength has good predictive capability as shown by the comparison with previously available experimental results.

6.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The following is a list of areas where future research may be directed:

- Axially loaded beams and prestressed concrete beams should be tested to determine whether the proposed theory is applicable to such beams.

- Performed experimental work to conclude the effect on variation of span-to-depth ratio, transverse reinforcement ratio, and concrete compressive strength using with multiple loads and uniformly distributed load should be done.

- Beams with thin webs, for example I, T and rectangular/circular hollow section beams should be tested to extend the theory to such specimens.

- The minimum shear reinforcement requirement stipulated in the Indian code (IS 456: 2000) should be examined in the light of shear strength calculation methods.

- The bundling of longitudinal reinforcements may contribute to an increase in the shear strength of a beam. More tests should be done to confirm this behaviour.

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APPENDIX - A

TEST DATA

Test data of each specimen of the present study are illustrate in following pages. Table A -1 to A -4 showing the midspan deflection and stain in transverse and longitudinal tensile steel bars at each 10 kN load interval up to 100 kN and after that 5 kN interval up to ultimate load.

Table A -1 Test Data for Specimen -I

Load (kN)	Deflection (mm)	Strain (x 10^{-3})						
		ε_{st} **1A**	ε_{sl} **1B**	ε_{sl} **1C**	ε_{sl} **1D**	ε_{st} **1E**	ε_{sl} **1F**	ε_{sl} **1G**
0	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.03	0.011	0.037	0.009	0.033	0.002	0.053	0.065
20	0.11	0.012	0.088	0.039	0.132	0.012	0.092	0.065
30	0.45	0.012	0.139	0.106	0.176	0.021	0.117	0.117
40	1.12	0.018	0.193	0.119	0.203	0.021	0.192	0.172
50	1.99	0.021	0.278	0.167	0.256	0.032	0.219	0.183
60	2.26	0.022	0.306	0.189	0.332	0.056	0.266	0.239
70	2.42	0.028	0.359	0.202	0.445	0.062	0.283	0.345
80	2.61	0.022	0.438	0.217	0.532	0.077	0.334	0.367
90	2.98	0.031	0.527	0.236	0.567	0.083	0.378	0.392
100	3.35	0.036	0.555	0.236	0.654	0.112	0.395	0.442
105	3.62	0.043	0.676	0.291	0.741	0.126	0.441	0.592
110	3.90	0.062	0.736	0.321	0.822	0.132	0.537	0.601
115	5.02	0.074	0.785	0.376	0.912	0.141	0.614	0.645
120	6.70	0.091	0.878	0.431	1.004	0.178	0.674	0.723
125	7.68	0.089	1.045	0.522	1.135	0.188	0.712	0.786
130	9.42	0.129	1.112	0.606	1.234	0.193	0.834	0.932

Table A -2 Test Data for Specimen -II

Load (kN)	Deflection (mm)	Strain (x 10^{-3})						
		ε_{st} **2A**	ε_{sl} **2B**	ε_{sl} **2C**	ε_{st} **2D**	ε_{sl} **2E**	ε_{sl} **2F**	ε_{sl} **2G**
0	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	1.00	0.018	0.065	0.017	0.022	0.033	0.068	0.036
20	1.07	0.018	0.115	0.038	0.022	0.092	0.108	0.085
30	1.35	0.018	0.136	0.061	0.023	0.106	0.147	0.117
40	1.64	0.021	0.157	0.131	0.031	0.145	0.169	0.157
50	1.83	0.022	0.184	0.176	0.044	0.148	0.219	0.183
60	2.04	0.023	0.222	0.187	0.049	0.159	0.302	0.239
70	2.15	0.023	0.235	0.228	0.063	0.173	0.378	0.316
80	2.22	0.023	0.272	0.255	0.076	0.188	0.414	0.33
90	2.27	0.025	0.305	0.317	0.089	0.222	0.467	0.384
100	2.34	0.027	0.358	0.334	0.089	0.273	0.534	0.401
105	2.65	0.028	0.448	0.397	0.091	0.296	0.577	0.428
110	3.20	0.029	0.558	0.443	0.101	0.312	0.638	0.488
115	3.89	0.031	0.657	0.487	0.108	0.419	0.734	0.496
120	3.91	0.031	0.728	0.492	0.128	0.478	0.837	0.587
125	4.68	0.052	0.837	0.528	0.145	0.547	0.929	0.639
130	5.21	0.063	0.921	0.684	-	0.632	1.012	0.724
135	5.79	0.075	1.029	0.742	-	0.765	1.165	0.901
140	6.58	0.089	1.132	0.811	-	0.855	1.236	0.959
145	7.53	0.107	1.202	0.967	-	0.923	1.365	1.086

Table A -3   Test Data for Specimen -III

Load (kN)	Deflection (mm)	Strain (x 10^{-3})						
		ε_{sl} 3A	ε_{st} 3B	ε_{sl} 3C	ε_{sl} 3D	ε_{sl} 3E	ε_{sl} 3F	ε_{sl} 3G
0	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.09	0.029	0.008	0.012	0.022	0.008	0.031	0.008
20	0.11	0.053	0.009	0.012	0.028	0.021	0.067	0.012
30	0.56	0.121	0.012	0.032	0.036	0.044	0.112	0.028
40	1.01	0.184	0.013	0.072	0.043	0.082	0.192	0.051
50	1.09	0.202	0.019	0.107	0.052	0.107	0.221	0.086
60	1.17	0.235	0.031	0.118	0.059	0.127	0.237	0.112
70	1.21	0.293	0.038	0.139	0.067	0.182	0.294	0.145
80	1.33	0.342	0.043	0.162	0.076	0.218	0.312	0.176
90	1.45	0.453	0.051	0.183	0.092	0.254	0.371	0.191
100	1.57	0.522	0.055	0.211	0.116	0.267	0.402	0.221
105	1.73	0.589	0.051	0.236	0.163	0.312	0.442	0.275
110	1.92	0.599	0.052	0.265	0.189	0.402	0.497	0.302
115	2.04	0.652	0.059	0.293	0.203	0.456	0.533	0.367
120	2.15	0.675	0.061	0.321	0.264	0.513	0.589	0.398
125	2.22	0.719	0.068	0.339	0.321	0.592	0.637	0.417
130	2.27	0.772	0.074	0.378	0.398	0.656	0.701	0.478
135	2.34	0.811	0.093	0.391	0.451	0.694	0.746	0.535
140	2.65	0.826	0.101	0.444	0.517	0.726	0.784	0.545
145	3.20	0.845	0.114	0.471	0.578	0.832	0.841	0.564
150	3.89	0.929	0.128	0.515	0.592	0.884	0.913	0.589
155	3.91	0.962	0.145	0.562	0.609	0.919	1.121	0.621
160	4.68	1.081	0.152	0.601	0.631	0.939	1.187	0.642
165	5.18	1.129	0.161	0.636	0.652	0.962	1.209	0.673
170	5.56	1.209	0.176	0.649	0.719	1.007	1.284	0.789
175	6.02	1.276	0.183	0.713	0.757	1.038	1.313	0.812
180	6.52	1.288	0.192	0.819	0.772	1.103	1.395	0.883

Table A -4 Test Data for Specimen -IV

Load (kN)	Deflection (mm)	Strain (x 10^{-3})							
		ε_{st} **4A**	ε_{sl} **4B**	ε_{sl} **4C**	ε_{sl} **4D**	ε_{sl} **4E**	ε_{sl} **4F**	ε_{sl} **4G**	ε_{sl} **4H**
0	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.02	0.002	0.012	0.018	0.065	0.006	0.065	0.002	0.037
20	0.11	0.004	0.029	0.018	0.115	0.016	0.115	0.012	0.088
30	0.18	0.012	0.042	0.018	0.136	0.027	0.136	0.021	0.139
40	0.29	0.015	0.065	0.021	0.139	0.046	0.157	0.021	0.193
50	0.35	0.019	0.073	0.022	0.193	0.078	0.184	0.031	0.239
60	0.38	0.028	0.085	0.019	0.278	0.091	0.222	0.038	0.316
70	0.44	0.032	0.102	0.021	0.283	0.106	0.235	0.043	0.33
80	0.56	0.041	0.167	0.037	0.334	0.118	0.302	0.051	0.384
90	0.61	0.063	0.195	0.043	0.378	0.145	0.378	0.055	0.401
100	1.10	0.071	0.218	0.059	0.395	0.164	0.414	0.076	0.443
105	1.32	0.079	0.286	0.067	0.419	0.178	0.467	0.092	0.487
110	1.54	0.089	0.301	0.076	0.478	0.188	0.534	0.116	0.492
115	1.72	0.092	0.339	0.079	0.547	0.205	0.587	0.163	0.528
120	1.93	0.102	0.428	0.106	0.632	0.228	0.639	0.189	0.684
125	2.01	0.119	0.511	0.145	0.645	0.289	0.636	0.222	0.694
130	2.14	0.127	0.598	0.148	0.723	0.313	0.649	0.273	0.726
135	2.30	0.217	0.656	0.159	0.786	0.345	0.713	0.296	0.832
140	2.64	0.229	0.727	0.184	0.932	0.419	0.819	0.312	0.884
145	2.81	0.278	0.818	0.222	1.004	0.446	0.832	0.419	0.919
150	3.02	0.392	0.823	0.235	1.135	0.478	0.884	0.445	1.004
155	3.17	0.452	0.834	0.272	1.209	0.491	0.919	0.532	1.135
160	3.43	0.467	0.883	0.305	1.276	0.527	0.939	0.567	1.162
165	3.75	0.478	0.917	0.321	1.296	0.539	0.962	0.654	1.189
170	4.19	0.487	0.977	0.378	1.319	0.596	1.007	0.741	1.193
175	4.62	0.513	1.012	0.444	1.385	0.659	1.038	0.785	1.245
180	4.86	0.553	1.081	0.471	1.399	0.688	1.135	0.878	1.256
185	5.07	0.589	1.114	0.518	1.418	0.745	1.234	1.045	1.279
190	5.52	0.592	1.178	0.587	1.447	0.804	1.289	1.112	1.319
195	6.02	0.612	1.216	0.656	1.482	0.892	1.315	1.156	1.337

APPENDIX - B

CRACK PATTERN OF TEST BEAMS

Photographs showing the crack patterns of the front and back faces of each test beam of the present study are given in following pages.

Fig. B -1   Crack Pattern of the Front Face of Specimen-I at 130 kN.

Fig. B -2   Crack Pattern of the Back Face of Specimen-I at 130 kN.

Fig. B -3   Crack Pattern of the Front Face of Specimen-II at 145 kN.

Fig. B -4   Crack Pattern of the Back Face of Specimen-II at 145 kN.

Fig. B -5   Crack Pattern of the Front Face of Specimen-III at 180 kN.

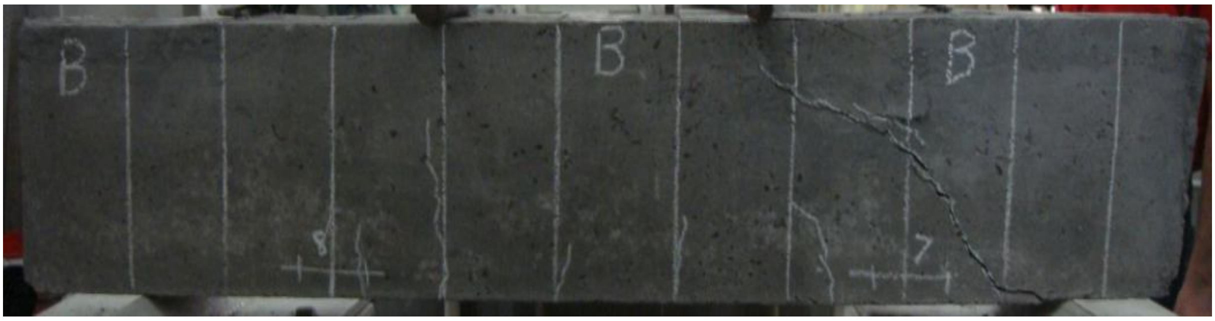

Fig. B -6   Crack Pattern of the Back Face of Specimen-III at 180 kN.

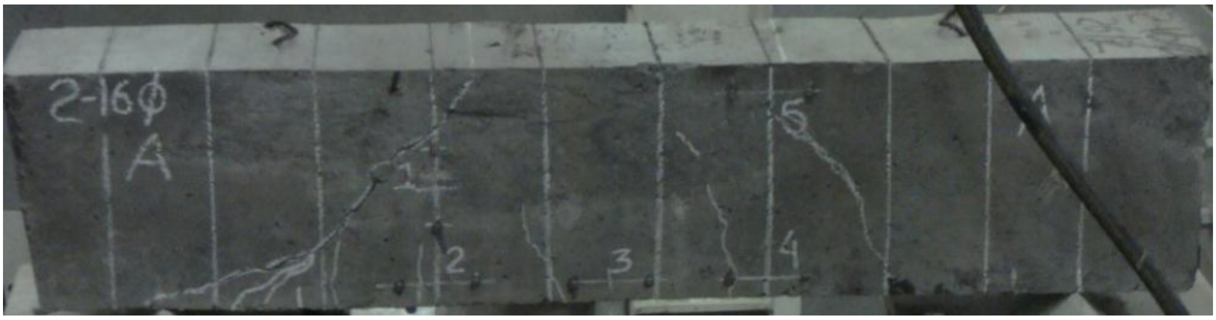

Fig. B -7   Crack Pattern of the Front Face of Specimen-IV at 195 kN.

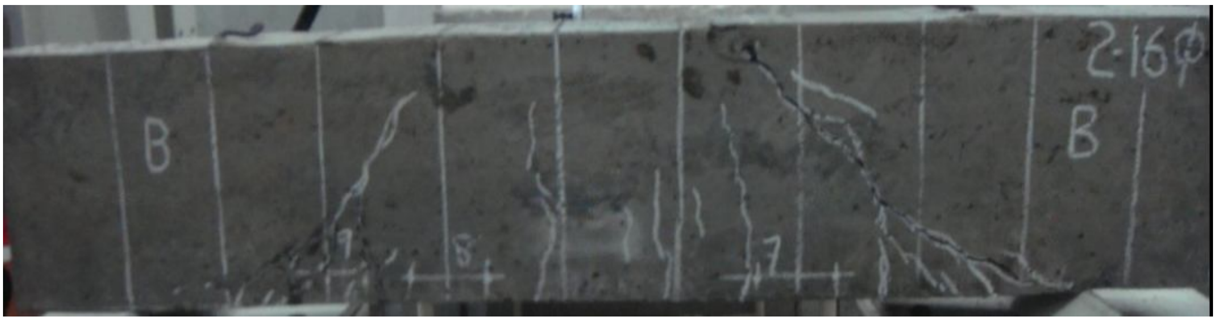

Fig. B -8   Crack Pattern of the Back Face of Specimen-IV at 195 kN.

APPENDIX - C

LIST OF USEFUL WEBSITES

- www.google.com
- www.ACI.org
- www.springerlink.com
- www.ASCE.org
- www.elsevier.com
- www.sciencedirect.com
- www.concrete.org
- www.icjonline.com
- www.concreteresearchonline.com

APPENDIX - D

LIST OF PAPER COMMUNICATED

1. Jignesh I. Patel and Himat T. Solanki, “Evaluation of Shear Strength of High Performance Concrete Beams with Web Reinforcement”, *National Conference on Advances in Steel, Concrete and Composite Structures (ASCCS09),* Government College of Technology, Coimbatore, April 24-25, 2009. (Abstract accepted)

2. Jignesh I. Patel and Himat T. Solanki, “An Analytical Investigation on Shear Strength of High Performance Concrete Beams with Web Reinforcement”, *International Conference on Advances in Concrete, Structural and Geotechnical Engineering* (ACSGE 2009), Rajasthan, October 25-27, 2009. (Abstract accepted)

3. Jignesh I. Patel and Himat T. Solanki, “Theoretical Predictions of Shear Strength of High Performance Concrete Beams with Web Reinforcement”, *Engineering Structures.* (Submission being processed)