# PREDICTION OF SHEAR STRENGTH IN HIGHER STRENGTH CONCRETE BEAMS WITH OUT WEB REINFORCEMENT


**By**

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**(**2006-NUST-MS Ph.D Str-05)


A Thesis submitted in partial fulfillment of

the requirements for the degree of

**Master of Science**

In


**Department of Civil Engineering**

**National Institute of Transportation**

**National University of Sciences and Technology**

**Rawalpindi, Pakistan**


**(2008)**

This is to certify that

thesis entitled

# PREDICTION OF SHEAR STRENGTH IN HIGHER STRENGTH CONCRETE BEAMS WITH OUT WEB REINFORCEMENT

**Submitted by**

**Ahmad Khursheed**

Has been accepted towards the partial fulfillment
of
the requirements
for
Master of Science in Civil Engineering

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**DEDICATED**

**TO**

**MY PARENTS, WIFE, KIDS, TEACHERS**

**AND**

**WELL WISHERS**

# ACKNOWLEDGEMENT

I am grateful to Almighty Allah whose blessing gave me the strength to complete this task. I express my gratitude and sincere thanks to my advisor Brigadier (Retired) Dr Khaiq-ur- Rashid Kayani for his advice and untiring guidance and supervision throughout the period of research.

I extend my gratitude to MCE Concrete Laboratory Staff, who provided technical assistance and corporation during the execution of work.

I am also thankful to the administration of the National Institute of Transportation for their help extended in the research work.

I must not forget the supporting role of  Lt col Fahim and Maj Shahid of TTT battalion for providing assistance in preparation of steel shuttering, and all out support in transportation and provision of man power.

I appreciate full support provided to me by my wife, mother and children, this research work would not have been completed without their prayers and whole hearted encouragement.

# ABSTRACT

An experimental investigation of the shear strength of Higher – Strength concrete beams was conducted. Six beams without web reinforcement were designed in accordance with provision of ACI 318-05. The beams were designed in such a manner that they should fail in shear. Primary design variables were concrete compressive strength and shear span to depth ratio. Concrete with compressive strength varying from 6,000 to 10,000 psi was used in these specimens. Shear reinforcement was not provided. Results of the investigations were tabulated for the comparisons and analysis with other studies.

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| | |
|---|---|
| HSC | Higher strength concrete |
| NSC | Normal strength concrete |
| $f'c$ | Compressive concrete strength |
| $f_y$ | Yield strength of tension reinforcement |
| M | Moment |
| $M_n$ | Nominal moment strength according to ACI Code |
| V | Shear force |
| s | stirrups Spacing |
| $\rho$ | Tension reinforcement ratio (As/bd) |
| b | Beam width |
| d | Effective beam depth |
| h | Overall height of beam |
| $V_c$ | Shear strength provided by concrete |
| $V_s$ | Shear strength provided by concrete |
| a | Shear span |
| $V_n$ | National shear strength |
| $A_s$ | Longitudinal steel area |
| w/c | Water cement ratio |
| Dia | Diameter |
| mm | Millimeter |
| in | Inch |
| Psi | Pounds per square inch |

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*Chapter – 1*

# INTRODUCTION

## 1.1 GENERAL

Use of concrete as a construction material has increased tremendously during the last few decades. The production of a high strength structural concrete in the range of 138 MPa is common in the concrete industry. The high compressive strength concrete mixtures are generally characterized by low water to cement ratio, high cement content, and presence of several admixture types, such as water reducing, set retarding, and other mineral admixtures. The introduction of new admixtures (i.e super- plasticizers) and cementitious materials has allowed the production of highly workable concrete with superior mechanical properties and durability. These developments have occurred because of the economical gains in producing stronger structures that are smaller in component dimensions while larger in space availability. This new class of concrete is named as high strength concrete (Sarkar 1991).

With the commercial availability of concrete of compressive strength higher than 150 MPa , many questions may be raised regarding the design provisions stipulated in ACI 2005 “Building Code Requirements for Structural Concrete”. For Shear Design, ACI code contains a number of empirical equations for different types of loadings and different types of members. Most of the design parameters and equations are derived from results of experimental research programs using concrete with compressive strength less than 40 MPa . Therefore it is reasonable to question if all the design provisions in 2005 ACI Code are applicable for higher strength concrete members.

## 1.2 SHEAR IN BEAMS

A beam resists loads primarily by means of internal moments, M and shear, V. In design of reinforced concrete members, flexure is considered first.It leads to the size of the section and the arrangements of the reinforcement to provide necessary moment resistance. Limits are placed on the amounts of flexure reinforcement which can be used to ensure that if failure were ever to occur; it would develop gradually giving a warning of the impending failure. The beam is then proportioned for shear. Since the shear failure is frequently sudden and brittle, the design must ensure that the shear strength equals or exceeds the flexural strength at all points in the beam. The manner in which shear failure can occur varies widely depending on the dimensions, geometry, loading and properties of the members. For this reason there is no unique way to design for shear (McGregor 2005).

## 1.3 SCOPE

The scope of this research is to produce concrete of appropriate strength initially and study the existing empirical expressions of ACI code predicting the shear capacity of these beams without web reinforcement.

- Use of indigenous construction material with silica fume for higher concrete strength.
- Production of higher strength concrete beams without web reinforcement.

## 1.4 OBJECTIVES

The objectives of the research are:-

- Evaluation and testing of mix design for concrete with compressive strength up to 68 MPa.
- Study the existing empirical expressions of ACI 2005 predicting the shear capacity of beams.
- Provide experimental data to examine existing empirical expressions predicting shear capacity of beams without web reinforcement and to purpose improved expressions.

*Chapter – 2*

# SHEAR CHARACTERISTICS OF CONCRETE BEAM

## 2.1 BEHAVIOR OF BEAM WITHOUT STIRRUPS UNDER SHEAR LOADING

The manner in which shear failures can occur varies widely with dimensions, geometry, loading, and the properties of members.  Shear span “a” is the distance between load application point and support as shown in (Figure.2.1-Appendix I). Shear span can be divided in to three types; short, slender and very slender shear spans. Beams with a/d from 0-1 are termed as very short shear spans, develop inclined cracks joining the load and the support. Common mode of failure is the anchorage failure at the ends of the tension ties. In beams with a/d 1-2.5, redistribution of internal forces after initiation of inclined cracking enables the beams to carry additional loads by arch action. The final failure in such beams will be caused by bond failure, splitting failure or a dowel failure along the tension reinforcement and is termed as shear tension failure. If failure occurs by crushing of compression zone at the crack end, it is referred as shear compression failure (Figure 2.2-Appendix I). In beams with shear span from 2.5 to 6, the inclined cracks disrupt equilibrium to such an extent that failure occurs at the inclined cracking load. Beams with a/d greater than 6 fail in flexure prior to formation of substantial inclined cracking (McGregor 2005).

## 2.2 DESIGN CONSIDERATIONS RELATING TO SHEAR SPAN.

As a result of the shear span effect, the critical position (on the basis of diagonal tension failure) for a concentrated load on the usual simple span test is not adjacent to the reaction but at some distance from it. The ACI 2005 uses the distance'd' from the face of support. In test the extra resistance beyond initial diagonal cracking is created in beams by the vertical compressive stress under the load and over the reaction. Load or reactions applied as side face shears, create very little vertical compression and add very little shear resistance. If loads are applied below mid depth of girder, say, for precast beams bearing on a lower flange or bracket attached near the bottom of the girder, diagonal cracking occurs at lower loads and ultimate strength is reduced. Extra stirrups as hangers to pick up such loads are essential in such cases.

## 2.3 CONCLUSIONS FROM DIFFERENT EXPERIMENTAL PROGRAMS.

A number of experimental programs to predict the shear strength in higher strength concrete beams have been carried out. Important conclusions drawn by them are:-

- Beams without web reinforcement can take shear loads in excess of that allowed in different design codes. For example in a test, the ratio of observed shear strength of beams with no web reinforcement to ACI 2005 was 3.89 for small a/d ratios and was 2.84 for large ratios (Mathey / Wastein 1963).

- The shear strength decreased roughly in a linear manner as the corresponding longitudinal steel stress increases in beams having the same shear span to depth ratio (Mathey / Wastein 1963).
- The shear strength of beams corresponding to the diagonal tension cracking load is independent of the tensile properties of the longitudinal reinforcement (Mathey / Wastein 1963).
- In ordinary rectangular reinforced concrete beams (with no web reinforcement) failing in shear, inclined shear cracking starts after the development of the nearby flexural cracks, near the middle of the shear span and just above the longitudinal reinforcement (Kim / White 1991).
- Both shear and diagonal tension failure occur after yielding of the longitudinal reinforcement (Morrow /Viest 1957).
- Reinforced concrete members without web reinforcement and with moderately long shear spans fail upon formation of first diagonal tension cracks and such failures are sudden and complete. On the other hand, in members with short shear spans, one or more diagonal tension cracks form first and a shear compression failure follows at higher loads (Morrow /Viest 1957).
- Even in short beams without web reinforcement, the presence of a diagonal tension crack is dangerous, because they are considerably wider than ordinary tension cracks. Furthermore, a few repetitions of load may cause the diagonal tension cracks to spread and possibly

result in splitting along the reinforcement or in a premature shear failure (Morrow /Viest 1957).

- High strength lightweight aggregate concrete beams have relatively high ultimate shear strength due to large margin between the diagonal cracking and the ultimate strength (Thornfeldt /Drangshool 1990).
- The ultimate shear strength of the beams without web reinforcement cast by higher strength concrete, increase with decreasing shear span to depth ratio and with increasing longitudinal reinforcement ratio. Also diagonal-cracking strength increases significantly with longitudinal reinforcement ratio (Thornfeldt /Drangshool 1990).
- The ductility of concrete in tension may be quantified by the ratio of the fracture energy per unit area of crack to the maximum elastic energy per unit concrete volume. This ratio is often called as characteristic length. The shorter the characteristic length, more brittle behavior is expected. Tests have proved that characteristic length continuously decreases with increase in compressive strength for normal density concrete. It is generally accepted that cracks will tend to propagate more easily in concrete with short characteristic length (Hesten 1990).
- High strength concrete beams without web reinforcement present a fragile behavior. The higher the concrete compressive strength, the brisker the failure (Cladera 2000).

- In beams without web reinforcement, the failure shear strength generally increased as concrete compressive strength increased (Cladera 2000).
- In shear span, the tensile force in the longitudinal reinforcement is linearly increasing with the distance from the support (Russo 2005).
- Internal shear force must have the same value whatever the distance from the support (Russo 2005).
- Shear is initially resisted by three shear carrying mechanisms: cantilever action, aggregate interlock and dowel action. These mechanisms create a state of tensile stress in concrete that leads to the development of critical shear crack. The development of critical shear crack cancels the three previous shear carrying mechanisms. A new one, the arching action is activated (Muttoni 2008).
- The parameter governing the arching action and thus shear strength are the location of critical shear crack, its width and aggregate (Muttoni 2008).

## 2.4 SHEAR THEORIES

### 2.4.1 Truss Model

The behavior of beams failing in shear must be expressed in terms of a mechanical mathematical model. The best model for beams with web reinforcement is the truss model. The Swiss engineer Ritter and the German engineer Morsch, independently published papers proposing the truss analogy for the design of reinforced concrete beams for shear (1899 to 1902). These procedures provide an excellent conceptual model to show the forces that exist in a cracked concrete beam.

Beam with inclined cracks develop compressive and tensile forces, C and T, in its top and bottom flange portions; Vertical tension in the stirrups and inclined compressive forces in the concrete "diagonals" between the inclined cracks as shown in (Figure 2.3-Appendix I). An analogous truss replaces this highly indeterminate system of forces.

Several assumptions and simplifications are needed to derive the analogous truss. The truss has been formed by lumping all the stirrups cut by section A-A into one vertical member, b-c and all the diagonal concrete members cut by the section B-B into one diagonal member e-f. This diagonal member is stressed in compression to resist the shear on section B-B. The compression chord along the top of the truss is actually a force in the concrete but is shown as a truss member. The compressive members in the truss are shown with dashed lines to imply that they are really forces in the concrete, not separate truss members. The tensile members are shown with solid lines (Figure. 2.4-Appendix I) (McGregor 2005).

### 2.4.2 Beam Theory

By the traditional theory for homogenous, elastic, uncracked beams, we can calculate the shear stresses, υ, on elements of a beam section as:-

$$v = \frac{VQ}{Ib} \tag{2.1}$$

where

V = Shear force on the cross section

Q = First moment about the neutral axis of the cross section

I = Moment of inertia of the cross section

b = Width of the member where the stresses are being calculated.

It should be noted that equal shearing stresses exist on both the horizontal and vertical planes through an element (Figure 2.5-Appendix I). The horizontal shear stresses are important in the design of construction joints, web-to-flange joints, or regions adjacent to the holes in beams. For an un-cracked rectangular beam, Eq 2.1 gives the distribution of shear stresses (Figure 2.5-Appendix I).

Generally, Eq 2.1 can not be applied to reinforced concrete beams for the following reasons:

- Reinforced concrete is a combination of two distinct materials whose strength and stiffness differ significantly and is not a homogenous material.
- Concrete is subjected to creep and therefore is not elastic.
- Cross sections may be cracked or uncracked. Since the extent of cracking at a specified location along the length of the beam is unpredictable, the actual cross sectional properties on which to base computations of moment of inertia, moment of area, and so forth, can not be determined.
- Because of cracking, which varies unpredictably, the effective cross section of reinforced concrete members varies along their length. In continuous structures, variations in cross section influence the magnitude of internal forces.

Due to above-mentioned reasons, a precise evaluation of shear stress intensity is not possible for a reinforced beam. The ACI 2005 has therefore adopted a simple procedure for establishing the order of magnitude of the average shear stress on a cross section. The shear stress is computed by dividing the shear force by $b_w$ d, the effective area of concrete.

$$v = \frac{V}{b_w d} \tag{2.2}$$

Where v = Shear stress at a section

V = Shear force at section

$b_w$ = Width of beam web

d = Distance between compression surface and centroid of tension steel

### 2.4.3 Modified Compression Field Theory

The research has shown that, the angle of inclination of the concrete compression is not 45 degree, and that equations based on variable angle truss provide a more realistic basis for shear design. In addition, tests of reinforced concrete panels subjected to pure shear improved the understanding of the stress-stain characteristics of diagonally cracked concrete. Concrete stress-stain relationship made it possible to develop an analytical model called the modified compression field theory, which proved capable of predicting accurately the response of reinforced concrete subjected to shear. Cracked reinforced concrete transmits load in a relatively complex manner involving opening or closing of pre-existing cracks, formation of new cracks, interface shear transfer at rough crack surfaces, and significant variation of stresses in reinforcing bars due to bond, with the highest steel stresses occurring at crack locations. The modified compression field model attempts to capture the essential features of this behavior without considering all of the details. The crack pattern is idealized as a series of parallel cracks all occurring at angle θ to the longitudinal direction. The shear stress that can be transmitted across the crack is a function of the crack width w, aggregate size a, and is given as (Mitchell and Collins)

$$V_{ci} = \frac{2.16\sqrt{f_c^{/}}}{0.3 + \dfrac{24w}{a + 0.63}} \tag{2.3}$$

*Chapter 3*

# EXPERIMENTAL PROGRAM

## 3.1 GENERAL

A brief on the materials used and experimental/testing procedures followed for the research program are summarized in the succeeding paragraphs.

## 3.2 ESTABLISHMENT OF VARIABLES AND CONSTANTS

The concrete constituents used in the course of this research, based on availability of time and literature review were divided in two categories; the variable and constant constituents. W/C ratio and percentage of silica fumes were selected. The following constituents were kept constant (Arshad 2006).

- Use of indigenous construction materials with silica fume.
- Dosages and type of High range water reducing agent.
- Size and grading of coarse aggregate.
- Grading of fine aggregate.
- Size of reinforcement.

## 3.3 MATERIALS

### 3.3.1 Cement

The Type I cement conforming to ASTM C 150 and C 595 was used. Results of the tests carried out to ascertain the properties of cement are presented in (Table

3.1-Appendix II). Variation in the chemical composition and physical properties of the cement affect concrete compressive strength more than variations in any other single material.

### 3.3.2 Fine Aggregate

The required range of fineness modulus was 2.70 to 3.20 (Arshad 2006). Sand from Lawrancepur and Margala pan crush was mixed together to bring the value of fineness modulus to 2.88. Results of the tests conducted for verification of properties of sand are tabulated in (Table 3.2-Appendix II). The gradation of the fine aggregate is tabulated in (Table 3.3-Appendix II) and graphically shown in (Figure 3.1-Appendix II).

### 3.3.3 Coarse Aggregate

Samples from Margala, Kiriana, and Kala Chita range being the well known sites for better quality of aggregates were collected. The laboratory test results for the three aggregate sources are tabulated in (Table 3.4-Appendix II). Comparison of the test results indicate that crushed aggregate from Kiriana hills had best physical properties.

For this research, the quantity of coarse aggregate in all mix designs was kept constant. Maximum size for the aggregate was kept as 12.7 mm. For gradation purpose only three sizes were considered i.e.12.7 mm, 9.5 mm, 4.7 mm. The gradation and sieve analysis was determined in accordance with ASTM C 136-93 and tabulated in (Table 3.5 -Appendix II) and graphically illustrated in (Figure 3.2-Appendix II).

### 3.3.4 Silica Fume (SF)

For the purpose of this research, SF was selected as a Pozzolanic cementitious material. SF produces best high early strength and durable concrete as compared to other pozzolanic materials (ACI 2005). The SF inclusion in the concrete mix increases the water demand and there by reduces the workability. More recently, the availability of high range water reducing agent has opened up new possibilities for the use of SF as part of the cementing material in concrete to produce very high strength or very high level of durability or both (ACI 2005). The chemical composition of the SF is tabulated in (Table 3.6-Appendix II).

### 3.3.5 High Range Water Reducing Agent

The High Range Water Reducing Agent used in the research, is a modified "polycarboxlate" type agent. The dosage was kept constant throughout the research work as 4 per cent by weight of cementitious materials. The technical data of polycarboxlate is tabulated in (Table 3.7-Appendix II).

### 3.3.6 Mixing Water

Potable water from Nowshera was used for entire experimental work while curing compound was used for curing.

## 3.4 WORKABILITY OF FRESH CONCRETE

The concept of low w/c ratio retards the characteristics of fresh concrete workability to its minimum. By use of High Range Water Reducing Agent the reduction in workability due to low w/c ratios is improved. Slump of 48 mm for 15-25% of SF was selected as constant in mix design (Arshad 2006) as tabulated in (Table 3.8-Appendix II).

## 3.5 WATER TO CEMENTITIOUS MATERIAL ( W/C ) RATIO

An important variable in achieving high strength concrete is the water to cement ratio. The relationship between w/c ratio and compressive strength which has been identified in normal strength concrete has been found to be valid for HSC as well. The use of chemical admixtures and other cementations materials have been proven generally essential for producing workable concrete with low w/c ratio. W/C ratio for HSC typically ranges from 0.20 to 0.5 (ACI 2005).

Mix was planned with low w/c ratio ranging from 0.22 to 0.23. Due to very low w/c ratio, workability problems were anticipated therefore polycarboxlate was used accordingly.

## 3.6 MIXING

The mixing of HSC ingredients is little different from normal strength concrete mixing. The concrete containing SF requires very careful and calculated mixing of ingredients. Over mixing of such concrete may produce adverse effect on strength development of the concrete. While preparing concrete in the laboratory, the key is batching the SF at the appropriate time and mixing the concrete adequately. ASTM C 192, Standard Practice for Making and Curing concrete Test Specimens in the laboratory, recommends: "Mix the concrete, after all the ingredients are in the mixer, for 3 minutes, followed by a 3 minutes rest, followed by a 2 minutes final mixing". These recommended mixing times were found not enough to break down the agglomerations and to disperse the SF.

Therefore, the following procedure was adopted to mix the ingredients to attain the full dispersion of admixtures in the mix (Holland 2005):

- SF must always be added with the coarse aggregate and some of the water. Batching SF alone or first can result in head packing or balling in the mixer. Mix SF, coarse aggregates, and water for 0.5 minutes.
- Add the Portland cement and any other cementitious material if any. Mix for an additional 1.5 minutes.

- Add the fine aggregate and use the remaining water to wash in chemical admixtures added at the end of the batching sequence. Mix for 5 minutes, rest for 3 minutes, and mix for 5 minutes. If there are doubts that full dispersion and efficient mixing has not been accomplished, mix longer. However, SF concrete cannot be over mixed.

## 3.7 CASTING OF SPECIMEN

Casting of specimens was carried out as per ASTM C 192M – 02. Six beams were prepared along with 6 cylinders for each beam with detail in (Table3.9 to Table3.14-Appendix II).

### 3.7.1 Description of Specimens

The size and details of the specimens were decided in such a way that they should not fail in flexure. The section selected was 190.5 x 254 mm with two # 8 bars and two # 4 bars bundled together as shown in (Figure 3.3-Appendix II).The overall depth was kept as 254 mm and effective depth‘d’ was kept as 209.55 mm. A total of six beams were cast and tested. The beams were designated as BO-1, BO-2, and BO-3. The alphabet B indicates “Beam” and O means without stirrups. The last digit means the serial number in the category.

### 3.7.2 Reinforcing Steel

All the longitudinal bars were # 8 & # 4 deformed bars. The stress-strain diagram of 1 inch bar is shown in (Figure 3.4-Appendix II). The grade 60 steel was used for longitudinal bars. The tension test data is given in (Table 3.15-Appendix II).

### 3.7.3 Fabrication of Specimens

The specimens were cast in steel shuttering designed for the purpose. The shuttering was prepared in such a manner that it could be dismantled easily. The steel reinforcement cage was bound with 18 gauge steel wire .The cage was placed in the shuttering over the 25.4 mm spacers and tied up with the bars. The concrete for the beams was mixed in a rotary mixer hired from local market. The capacity of the mixer was 7 cubic feet. One batch was prepared for one beam and its associated test cylinders. In one batch, 2 bags of cement were used. The standard cylinders cast with beam were 5 to 6 in numbers for each batch. The beams and associated cylinders were covered with Anti sole E-10 and kept under similar environments. The shuttering was removed from beams after 24 hours. During the process of removing shuttering few hairline cracks were observed. The cracks were observed for their depth and extent. It was established that the cracks had not penetrated into the core of the beam and were only surface cracks occurred by shrinkage due to excessive silica fume.

### 3.7.4 Specimen BO-1 to BO-3

Beams were cast on 3 Feb 2008. Other details of the subject beams are tabulated in (Table 3.16-Appendix II). The detail of the materials used in each of these beams is tabulated in (Table 3.17-Appendix II).

### 3.7.5 Specimen BO-4 to BO-6

Beams were cast on 5 April 2008. Other details of the subject beams are tabulated in (Table 3.18-Appendix II). The detail of the materials used in each of these beams is tabulated in (Table 3.19-Appendix II).

## 3.8 INSTRUMENTATION

Electrical strain gauges were fixed on web and longitudinal reinforcement for recording the strain at different points in shear span as well as in mid span region. All the beams (BO-1, BO-2, BO-3, BO-4, BO-5, BO-6), had one strain gauge fixed at the center and the other in shear span on longitudinal reinforcement. The electrical strain gauges of EA-06-240LZ-120/E nomenclature were used to record the strain in the beams. These gauges had 120.0 + 0.3% grid resistance in ohms with gauge factor 2.080 ± 0.5 at 24 $^{0}$C. These gauges are manufactured with self-temperature compensation characteristics to minimize thermal output. The EA series gauges are a general purpose family of constant alloy strain gauges widely used in experimental stress analysis. EA gauges are constructed with a 0.03-mm tough, flexible polyamide film backing. Strain gauges were soldered and checked with the help of digital multimeter.

### 3.8.1 Test Set Up

The specimens were transported to Structure Laboratory of University of Engineering and Technology Lahore, where a 50 ton universal testing machine made by Schmadu Tokyo, Japan, is installed. The machine is hydraulically operated and is connected to the computerized data acquisition system, as shown in (Figure 3.5 Appendix II). Fifty (50) tons jack was used for the loading of specimens through a steel girder and base plates to create two point load system. The beams were placed on the supports with the help of a gantry crane.

### 3.8.2 Testing Procedure

Beams were divided into two groups, each group consisting of three beams. The beams were planned to be tested with a/d 2.5. The load was applied after centering and aligning the specimens on the testing machine and making all necessary corrections for recording the load, strain and deflection. The computer automatically acquired the strain, load and deflection data. During the application of load, the cracks were observed and marked on the beams.

*Chapter – 4*

# EXPERIMENTAL RESULTS

## 4.1 CONCRETE STRENGTH

Six cylinders were cast from each batch of concrete during the casting of the specimens. Six cylinders for BO-4, BO-5 and BO-6 were tested after 14 and 28 days, and the remaining three cylinders were tested on the day of the testing of beams (Table 4.1 to 4.3-Appendix III). The average compressive strengths of the concrete are tabulated in (Table 4.4-Appendix III).

## 4.2 STRAIN MEASUREMENTS

The strain was measured by using two electrical strain gages on the two outer longitudinal reinforcement.

## 4.3 TEST BEHAVIOR OF SPECIMENS

### 4.3.1 Specimen BO-1

The beam was loaded on 25 April 2008 with shear span of 533.4 mm (a/d = 2.54). The reading from strain gages, load cell and deflections were recorded using data acquisition system. The initial flexural cracks developed at 86 KN load. Increased load widened the flexural cracks and additional cracks appeared between 117 KN to 146 KN loads. Inclined cracks appeared at a load of 155 KN. Beam failed at 183 KN. Load deflection data and plot are given in (Table 4.5 and Figure 4.1) respectively (Appendix III). Load vs. strains and plot are shown in (Table 4.11 and

Figure 4.7) respectively (Appendix III). The crack pattern of the beam is shown in the (Figure 4.13-Appendix III).

### 4.3.2 Specimen BO-2

This beam was loaded on 25 April 2008 with a shear span of 533.4 mm (a/d = 2.54). Initial flexural cracks appeared at a load of 45 KN. Increased load widened the flexural cracks and additional cracks appeared at 137 KN load. After which no cracks appeared. Beam failed at a load of 203 KN. Load deflection data and plots are shown in (Table 4.6 and Figure 4.2-Appendix III). Load strain data and plots are shown in (Table 4.12 and Figure 4.8-Appendix III). Cracking pattern is shown in (Figure 4.14-Appendix III).

### 4.3.3 Specimen BO-3

This beam was loaded on 26 April 2008 with a shear span of 533.4 mm (a/d = 2.54). Initial 2 to 3 flexural cracks appeared at a load of 89 KN. Inclined cracks appeared at load of 181.5 KN after which the beam abruptly failed at 188 KN. The same behavior can be seen in (Figure 4.15-Appendix III). The load deflection and load strain data are shown in (Table 4.7 and 4.13-Appendix III). The load-deflection and load-strain plots are shown in (Figures 4.3 and 4.9 -Appendix III).

### 4.3.4 Specimen BO-4

The beam was loaded on 26 April 2008 with a shear span of 533.4 mm (a/d = 2.54). Flexural cracks appeared at 96 KN. Increased load widened the flexural cracks and additional cracks appeared at 130.5 KN load. First inclind crack appeared at 145 KN after which more flexural were observed and old cracks also propogated. Inclined cracks kept on appearing alongwith propogation of few flexural cracks. Beam failed at a load of 254 KN. The cracking pattern is shown in the (Figure 4.16-Appendix III).

The load deflection data and load strain data are shown in (Table 4.8 and 4.14-Appendix III). The load deflection and load strain plots are shown in (Figures 4.4 and 4.10-Appendix III).

### 4.3.5 Specimen BO-5

This beam was loaded on 26 April 2008 with a shear span of 533.4 mm (a/d = 2.54). Flexural cracks appeared at 91 KN load and continued till 155.5 KN. Inclined cracks occurred at 158.5 KN load. Flexural cracks started appearing at 184 KN . Inclind cracks widened with increase in load and beam failed at 340 KN. The load-deflection and load-strain data are shown in (Table 4.9 and 4.15-Appendix III). The load-deflection and load-strain plots are shown in (Figures 4.5. and 4.11-Appendix III). The cracking pattern is shown in (Figure 4.17-Appendix III).

### 4.3.6 Specimen BO-6

This beam was loaded on 27 April 2008 with a shear span of 533.4 mm (a/d= 2.54). The initial flexural cracks appeared at 110 KN load. Inclined cracks occurred at 209 KN load. By further increasing the load beam failed at a load of 340 KN. The load-deflection and load-strain data are shown in (Table 4.10 and 4.16-Appendix III). The load-deflection and load-strain plots are shown in (Figures 4.6. and 4.12-Appendix III). The cracking pattern is shown in the (Figure 4.18-Appendix III).

**4.4.7** Behavior of the beams with load can be described briefly as given below:

- Initial flexural cracks occurred between load level 45 KN and 155 KN for shear span to depth ratio of 2.5.
- Existing cracks extended and new flexural cracks appeared in the shear span with increased load. The flexural cracks in the shear spans tend to

become more inclined as load is increased further. However BO-3 failed by applying very less load after initial cracking.

- Sudden occurrence of a wide diagonal shear crack in the shear span with further loading. In some cases, this crack coincided partially with the inclined part of the flexural cracks. The occurrence of the shear crack was accompanied by drop in the load, which was easy to detect on automatic printed data.
- The crack angle was observed to be between 45 and 35.
- All beams failed suddenly with large widening of and sliding in one of the inclined cracks.

*Chapter 5*

# ANALYSIS AND INTERPRETATION OF TEST RESULTS

## 5.1 General

Test data was analyzed to develop understanding of shear behavior of higher strength concrete beams without web reinforcement. Shear strength of a beam varies with $f_c^{/}$, a/d ratio, and amount of longitudinal reinforcement. Available data from literature was used to evalute test results and to draw logical conclusions.

## 5.2 General behavior of beams

All six beams (B0-1, B0-2, B0-3, B0-4, B0-5 & BO-6) failed in shear. Small flexural cracks developed at mid span during the early stages of loading. These cracks started at the bottom of the beams where the flexural stresses are maximum. Initial cracks surfaced at shear loads of 86 KN, 45 KN, 89 KN, 96 KN, 91 KN, and 110 KN in beams B0-1, B0-2, B0-3, B0-4, B0-5 and BO-6 respectively. As the load increased further, the existing cracks widened, and new cracks developed. Flexural cracks developed along the entire length (in shear spans also) of the beams as the load increased. The flexural cracks in the shear span inclined as they propagated above the longitudinal reinforcement. The inclined cracks at the ends of the beam are due to shear and are termed as diagonal tension cracks. Such cracks must exist before a beam can fail in shear. The average crack angle which depends on the shear span was observed to be between 43 and 37 degrees. The inclination of failure cracks is normally related to shear span and longitudinal reinforcement. As the shear span and longitudinal reinforcement were kept constant in this experimental program the inclination may be related to the variation in the compressive strength of concrete. Failure of all six beams (B0-1, B0-2, B0-3, B0-4, B0-5 & BO-6) was brittle and sudden without any large deflections.

## 5.3 MECHANICS OF RESISTANCE IN CONCRETE

**5.3.1.** It is established that shear strength of reinforced concrete beams depends upon following factors:

- Shear Strength of concrete section which is a function of its compressive strength and certain other factors.
- Longitudinal reinforcement.
- Shear reinforcement. This study excludes shear reinforcement to determine the contribution of first two components of shear resistance.

### 5.3.2 Concrete Contribution $(V_c)$

**5.3.2.1.** For members subjected to shear and flexure, ACI 2005 uses following relation to calculate the shear capacity

$$V_c = 2\sqrt{f_c^{'}}bd \tag{5.1}$$

Where

$V_c$ = Shear strength provided by concrete

$b$ = Web width

$d$ = Distance from longitudinal tension reinforcement

$f_c^{/}$ = Specified compressive strength of concrete.

**5.3.2.2.** The use of $\sqrt{f_c^{'}}$ as a sole predictor of the shear strength for concrete is perhaps not justified. (Figure 5.1-Appendix IV) shows plot of observed shear strength of beams with $\sqrt{f_c^{'}}$. Equation 5.1 is based on standard test on 6 inch square x 18 inch long prisms loaded at third points with a fixed $a/d$ ratio of 1. It is believed that $a/d$ ratio shall affect the shear behavior of concrete beams. The relationship is obviously inverse. The concrete shear strength may be expressed as under

$$V_c = \frac{2\sqrt{f_c^{'}}bd}{a/d} \tag{5.2}$$

### 5.3.3. Longitudinal Reinforcement

**5.3.3.1**. Traditional beliefs and studies state that shear strength of a concrete section is influenced by the amount of longitudinal steel in term of their dowel action. It also reduces the width of cracks which may add to the aggregate interlock across the diagonal plane formed by the shear cracks.

**5.3.3.2**. Dowel action provides resistance to applied shear depending upon the stress in longitudinal steel. The shear capacity of beams without web reinforcement is the sum of contributions form concrete section and longitudinal reinforcement. As discussed earlier, it is suggested that the beams are influenced by $a/d$ ratio for shear behavior and should be incorporated in both the components of shear resistance. Figure 5.2-Appendix IV shows plot of observed shear strength $V_t$ of beam without shear reinforcement against $a/d$.

## 5.4 SHEAR STRENGTH

### 5.4.1 General

Shear failures in reinforced concrete members are sudden and catastrophic in nature and should be catered for in the design. Reinforced concrete members are proportioned first for flexure and then checked for shear. The effect of shear is to induce tensile stresses on inclined planes oriented at approximately $45^\circ$ to the plane on which the shear stresses act. Failure occurs when these stresses, combined with horizontal stresses due to bending exceed the tensile strength of concrete. The failures occur in an inclined plane. However, it is difficult to determine the value of the diagonal tensile stress in a reinforced concrete beam because the distribution of shear and flexural stresses over a cross section is not certain. Accordingly, shear strength prediction in reinforced concrete members is an empirical solution based on the assumption that a shear failure at the critical section occurs on a vertical plane when the averaged shear stress at that section, $V/bd$, exceeds the concrete shear strength.

### 5.4.2 Cracking Strength

The cracking shear strength $V_{cr}/bd$ is defined as the shear stress at the occurrence of the initial diagonal crack. Cracking starts with the development of very fine vertical flexural cracks, followed by reduction of bond between reinforcing steel and surrounding concrete in the support region. The diagonal cracks develop at 1 d to 2 d from the face of support. The cracking shear strength includes the resistance from the longitudinal reinforcement in form of dowel action. The cracking shear strength can be assessed as under;

$$V_{cr} = V_c + V_d \tag{5.3}$$

where

Vcr = Cracking shear strength

Vd = Shear due to dowel action of longitudinal reinforcement at cracking

**5.4.2.1.** $V_c$ has already been discussed in para 5.3.2 and is considered safe when calculated as per Equation 5.2. Dowel action contribution can be evaluated by plotting $V_{cr} - V_c$ against the amount of longitudinal reinforcement. The plot is given in Figure 5.3(a). Basing on the regression analysis, the best fit line $V_d = 0.1453\left(\frac{A_s f_y}{a/d}\right) + 37.081$. A workable and safe design value can be ascertained from Figure 5.3(b) which is;

$$V_d = f_v\left(\frac{A_s}{a/d}\right)$$

where

fv = Vertical shear stress in longitudinal bars.

$$V_d = 0.125\left(\frac{A_s f_y}{a/d}\right) \tag{5.4}$$

### 5.4.3 Ultimate shear strength

The ultimate shear strength, $\frac{v_u}{bd}$ is defined as the strength when shear failure occurs as a beam. The ultimate/failure shear strength is determined by the ability of the concrete to stabilize the first diagonal crack and to postpone further propagation of the crack into the compression zone. The test shear strength can be expressed as;

$$V_t = V_c + V_{d'} \tag{5.5}$$

$$V_{d'} = V_t - V_c \tag{5.6}$$

where

Vd' = Shear due to dowel action at ultimate

**5.4.3.1** Dowel action shear force can be determined by plotting $V_t - V_c$ with $\left(\frac{A_s f_y}{a/d}\right)$ as in Figure 5.4(a)-(Appendix IV). The regression analysis of the results shows the best fit line as $V_{d'} = 0.5077\left(\frac{A_s f_y}{a/d}\right) + 22.096$ for the data. A safe estimate of dowel action shear force at ultimate load can be given as Figure 5.4(b)-(Appendix IV).

$$V_{d'} = 0.4\left(\frac{A_s f_y}{a/d}\right) \tag{5.7}$$

**5.4.3.2** The expression corresponds to generally accepted maximum shear stress at about 40% of tensile strength of steel bars.

## 5.5 SHEAR STRENGTH RELATIONSHIPS

### 5.5.1 Cracking Shear Strength

From the previous section, an expression for determination of cracking shear force can be proposed as;

$$V_{cr} = \frac{2\sqrt{f_c^{'}}bd + 0.125A_s f_y}{a/d} \tag{5.8}$$

### 5.5.2 Ultimate Shear Strength

Similarly, the ultimate shear force of a RC beam can be computed as;

$$V_t = \frac{2\sqrt{f_c^{'}}bd + 0.4A_s f_y}{a/d} \tag{5.9}$$

## 5.6 CONCLUSIONS

- ACI 2005 is too conservative in prediction of shear capacity of high strength concrete beams.
- Shear span to depth ratio affects the shear strength of the concrete. More is the ratio, lesser is the strength.
- Shear strength of concrete is directly proportional to the amount of longitudinal reinforcement present in the beam.

## 5.7 RECOMMENDATIONS FOR FUTURE WORK

Mechanism of shear resistance is yet to be understood. The performance of concrete member under laboratory test loads and actual loads has to be brought with in the jurisdiction of structural mechanics. There is a requirement to carryout extensive studies on the subject before shear in concrete can be understood. Following are the suggested studies.

- Development of shear resistance relationship with cross section development.
- Effect of $a/d$ ratio on shear strength.
- Effect of compressive strength, $f_c^/$ on shear strength to include very high concrete strength.

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