Effect of Longitudinal Tensile Reinforcement Ratio on Minimum Shear Reinforcement in Reinforced Concrete Beams - I

By

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DEDICATED

TO

MY PARENTS, WIFE, KIDS, TEACHERS

AND

WELL WISHERS

ACKNOWLEDGEMENT

All glory to Allah, the creator of the heavens, the earth, and everything between them. The creator of the light, the time, the man and the woman. The ultimate source of knowledge and power. The one, who gave me ability to pursue my education, complete the project and write this dissertation.

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ABSTRACT

To avoid abrupt shear failure, ACI 318 – 08 specifies the requirements for the minimum amount of shear reinforcement for Reinforced Concrete Beams. The equation given for minimum shear reinforcement is based on previous experimental data from beams of normal and high strength concrete without any consideration for the effects of longitudinal reinforcement.

Minimum amount of shear reinforcement corresponds to a value that restrains the growth of inclined cracking and prevents a sudden shear failure. The growth and width of inclined cracks, beside other factors, is also influenced by longitudinal reinforcement. Therefore, with the change in longitudinal reinforcement ratio, requirement of minimum shear reinforcement should also change.

Aforementioned in view, the influence of longitudinal tensile reinforcement ratio on minimum shear reinforcement in Reinforced Concrete Beams has been investigated through literature review and laboratory tests. In the study, 6 x beam specimens (divided into two groups) having cross section 10” x 16” were cast and tested.

The test results indicated that reserve strength (shear strength of beam with minimum shear reinforcement / shear strength of beam without shear reinforcement) of tested beams increased as the longitudinal tensile reinforcement ratio increased. Therefore, inclusion of factor ρ_ℓ in the ACI code for determining minimum shear reinforcement should be considered.

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LIST OF ABBREVIATIONS / NOTATIONS

f'_c	Specified compressive strength of concrete, psi
V_{cz}	The shear in the compression zone
V_{ay}	The vertical component of the shear transferred across the crack by interlock of aggregate particles on the two faces of the crack
V_d	The dowel action of the longitudinal reinforcement
V_c	Shear carried by concrete, lb
b_w	Web width, in.
d	Distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in.
A_v	Area of shear reinforcement, in^2.
$A_{v, min}$	Minimum area of shear reinforcement, in^2.
f_y	Specified yield strength of reinforcement, psi.
f_{yt}	Specified yield strength f_y of transverse reinforcement, psi.
ρ_ℓ	Longitudinal Tensile Reinforcement Ratio
ρ_t	Transverse Tensile Reinforcement Ratio
s	Center to center spacing of transverse reinforcement, in.
MCFT	Modified Compression Field Theory

Chapter - 1

INTRODUCTION

1.1 GENERAL

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

The prime objective of shear design is to prevent an abrupt shear failure by identifying where shear reinforcement is required and how much is it required. Shear reinforcement is provided in the form of stirrups. These stirrups link flexural tension and flexural compression sides of a member and ensures that the two sides act as a unit. Shear failures involve breakdown of this linkage and the opening of major diagonal crack in members without shear reinforcement.

1.2 BRIEF OVERVIEW OF DEVELOPMENT OF SHEAR DESIGN PROVISIONS

ACI 318 – 51, based on allowable stresses, specified that web reinforcement must be provided for the excess shear if the shear stress at service loads exceeded 0.03 f'_c. Calculation of the area of shear reinforcement was based on 45 degree truss analogy. The 1955 shear failure of beams in two Air Force warehouses initiated the major review of traditional ACI design procedures. Followed by a thorough research, the study of these failures clearly indicated that shear and diagonal tension was a complex problem (ACI 445 R – 99).

In the 25 years after 1960, considerable research was conducted to understand the beam action and arch action - mechanisms by which cracked reinforced concrete beams transmitted shear. Shear force in a region indicates that the moment is changing along the length of the member. "Moment can change either by the tension in the reinforcement changing, which is called beam action, or by the internal lever arm changing along the length, which is called arch action. If arch action is to carry the entire shear in a region that has constant shear, then the concrete compression zone must form an inclined strut going from the load to the support. It was shown that, because of the geometric incompatibility of the two mechanisms, with beam action typically being much stiffer than arch action, nearly all of the shear would be carried by beam action until this mechanism failed. After failure of the beam mechanism, an internal redistribution of stresses could occur and the remaining arch mechanism could then carry even higher shears if the distance between the applied load and the support was sufficiently short" (Fenwick / Paulay 1968).

Using detailed measurements of cracked beams and direct measurements on subassemblies, Fenwick / Paulay (1968) concluded that approximately 70% of the vertical shear at a flexurally cracked section is carried across the flexural cracks. For slender beams without stirrups, the breakdown of aggregate interlock action triggers the shear failure. The key role played by aggregate interlock in transferring shear stresses across inclined flexural cracks was confirmed by Taylor (1970), Kani / Huggins / Wittkopp (1979) and Sherwood / Bentz / Collins (2007). Walraven (1981) showed that the shear stress that can be transmitted across a crack by aggregate interlock decreases as crack width increases and as aggregate size decreases. Fig 1.1 – Appendix I shows how experimental shear research has varied intensely over the period (Collins / Bentz / Sherwood 2008).

1.3 MINIMUM SHEAR REINFORCEMENT

Minimum amount of shear reinforcement corresponds to a value that restrains the growth of inclined cracking (substantially the growth of critical diagonal crack), providing required ductility and preventing a sudden shear failure. A number of factors that influence the determination of the required minimum amount of shear reinforcement have not yet been fully explored and understood. As a consequence, the current ACI code provisions are still based on semi empirical considerations (Zarais 2003).

To avoid abrupt shear failure, ACI 318 – 08 specifies that minimum amount of shear reinforcement must be provided in Reinforced Concrete Beams. The equation given by ACI for minimum shear reinforcement is based on previous experimental data from beams of normal and high strength concrete with little consideration for the effects of longitudinal reinforcement (Lee / Kim 2008).

The ultimate goal of specifying minimum shear reinforcement is to ensure that total shear is transferred across cracks by aggregate interlock and prevent excessive crack opening that can cause reduction in interface shear transfer. The growth and width of inclined cracks, beside other factors is also influenced by longitudinal reinforcement. Therefore, with the change in longitudinal reinforcement ratio, requirement of minimum shear reinforcement should also change.

1.4 OBJECTIVE

The objective of this research is to study the effect of longitudinal tensile reinforcement ratio on minimum shear reinforcement in Reinforced Concrete Beams.

Chapter - 2

MINIMUM SHEAR REINFORCEMENT

2.1 SHEAR STRENGTH (Vc) PROVIDED BY CONCRETE

In beam without stirrups, shear is transferred across inclined crack by V_{cz}, the shear in the compression zone, by V_{ay}, the vertical component of the shear transferred across the crack by interlock of aggregate particles, and by V_d, the dowel action of the longitudinal reinforcement. Diagrammatically the detail of these forces is given in Fig 2.1 – Appendix II. In the ACI Code V_{cz}, V_{ay} and V_d are lumped together as V_c, which is referred to as "shear carried by concrete" and for normal weight concrete is given by following equation (ACI Equation 11-3).

$$V_c = 2\sqrt{f'c}\, b_w d \qquad (2.1)$$

The above mentioned ACI equation for V_c becomes less conservative as member depth d increases; as concrete strength f'_c increases; as maximum aggregate size decreases; and as stress in the longitudinal reinforcement f_s increases. The ACI requirement of minimum shear reinforcement where V_u exceeds $0.5\phi V_c$ mitigates the consequences as a result of these trends (Collins / Bentz / Sherwood 2008).

2.2 MINIMUM SHEAR REINFORCEMENT

In beams with at least minimum shear reinforcement, the stirrups holds the crack faces together so that the shear transfer across the cracks by aggregate interlock is not lost. Where required, the minimum shear reinforcement shall be computed by the equations (ACI Section 11.4.6.3) reproduced below. Equation 2.2 is new in the code and was introduced in ACI 318-05 to account for the influence of compressive strength of concrete.

$$A_{v,\,min} = 0.75\sqrt{f'c}\,\frac{b_w\,s}{f_{yt}} \tag{2.2}$$

But not less than

$$A_{v,\,min} = \frac{50\,b_w s}{f_{yt}} \tag{2.3}$$

Spacing of shear reinforcement placed perpendicular to axis of member has been restricted to d/2 in non prestressed members, or 24 in. This requirement ensures that any potential diagonal crack that may develop will be intercepted by a vertical stirrup.

2.3 RELATION OF DIAGONAL CRACK WIDTH WITH MINIMUM SHEAR REINFORCEMENT

2.3.1 Diagonal Cracking

A crack will form in concrete when the principal tensile stress at some location reaches the cracking strength of concrete. The crack will form normal to the direction of principal tensile stress. For members subjected to pure axial tension or to pure flexure, the principal tensile stresses are parallel to the longitudinal axis of the member and cracks form perpendicular to the member axis. If a cross section of a member is subjected to shear stresses, the principal tensile stress directions are inclined to the longitudinal axis of the member. A crack forms at a location where significant shear stresses exist, and is inclined to the member axis. Such cracks are called diagonal cracks. Diagrammatically such a crack is shown in Fig 2.2 – Appendix II (Collins / Mitchell 1997).

2.3.2 Diagonal Crack Width

"The inclined crack width cannot be predicted by calculating principal stresses in an uncracked beam. Their slope, spacing and width depends on many factors like flexural and shear reinforcing steel areas, shape and dimension of cross section, shear stresses and mechanical properties of concrete and steel. Hence the control of inclined cracking width can be exercised using empirical equations, based on empirical works only" (Jensen / Lapko 2009).Over the period, various research studies as mentioned below have been carried out to find the empirical expression for determining crack widths.

- Borishansky (1964) assumed that mean inclined crack width is directly proportional to strain in stirrups and crack spacing.
- Placas and Regan (1971) concluded that maximum crack width is directly proportional to spacing of stirrups and inversely proportional to A_v,$(f'_c)^{1/3}$ and d.
- Bentz, Vecchio and Collins (2006), reasoned out in MCFT that crack width is equal to the product of crack spacing and principal tensile strain.
- More recently,Muttoni and Ruiz (2008) stated that critical crack width is proportional to the product of longitudinal strain in the control depth (0.6 d) times effective depth of element.

2.3.3 Minimum Shear Reinforcement and Interface Shear Transfer

In the light of detailed discussions on the subject by joint ASCE-ACI Task Committee 426, it has been shown that in beams without web reinforcement, 20 to 30 percent of the total shear is resisted by the compression zone, 15 to 25 percent of the shear resistance is provided by the dowel action, and 30 to 60 percent is contributed by interface shear resistance across a crack. These resistance ranges demonstrate that interface shear contributes a significant share of shear resistance; therefore, minimum shear reinforcement must ensure full shear transfer across cracks by aggregate interlock. Also, such requirements should prevent excessive crack opening that can cause a reduction in interface shear transfer (Karuthammer 1992).

2.4 CONCLUSIONS FROM PREVIOUS EXPERIMENTAL PROGRAM

Lee and Kim (2008) have presented test results of 26 reinforced concrete beams having minimum shear reinforcement. Three parameters were considered in the investigation: longitudinal tensile reinforcement ratio, shear span-depth ratio (a/d) and compressive strength of concrete. The influence of three parameters was investigated by considering the reserve strength and the reserve deflection beyond diagonal cracking. In the study, **reserve strength** is defined as the ratio between the ultimate shear capacity of the beams with the minimum shear reinforcement and that of the beams without shear reinforcement. Likewise, **reserve deflection** is defined as the ratio between the deflection corresponding to the ultimate load of beams with minimum shear reinforcement and the deflection of beams without shear reinforcement. Salient of the conclusions drawn by Lee and Kim are appended below

- Reserve strength and reserve deflection are related with the longitudinal ratio ρ_ℓ.
- The existing design codes contain an expression for the minimum shear reinforcement that depends on neither the longitudinal tensile reinforcement ratio nor the a/d. This may lead to the reserve strength and the reserve deflection of a reinforced concrete beam with a small

$\rho_\ell f_y$ being lower than that of a reinforced concrete beam with a large $\rho_\ell f_y$, resulting in an inadequate shear behavior after cracking.

- The test result of reinforced concrete beams having the minimum shear reinforcements showed a 16% difference in the reserve strengths between beams having ρ_ℓ equal to 0.0093 and 0.0279. Thus, inclusion of factor ρ_ℓ in the design codes for determining minimum shear reinforcement should be considered.
- The amount of minimum shear reinforcement needs to increase (decrease) as ρ_ℓ decreases (increases) to achieve uniform reserve strength and deflection.

Chapter - 3

EXPERIMENTAL PROGRAM

3.1 GENERAL

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

3.2 MIX DESIGN

Concrete strength in conjunction with member size, were so selected that minimum shear reinforcement requirement is governed by the empirical equation 2.2 and not by the spacing requirements (equation 2.4). In the study, f'_c was selected as 6000 psi. The mix design (Table 3.1 – Appendix III), was used from another study carried out in NICE, NUST.

3.3 MATERIALS

3.3.1 Cement

The Type I cement conforming to ASTM C 150 – 04 was used. Results of the tests carried out to ascertain the properties of cement are presented in Table 3.2 – Appendix III. 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

Sand from Qibla Bandi was used. Results of the tests conducted for verification of properties of sand are tabulated in Table 3.3-Appendix III. The gradation of the fine aggregate is tabulated in Table 3.4-Appendix III, and graphically shown in Fig 3.1-Appendix III.

3.3.3 Coarse Aggregate

Aggregate from Margalla site was used in this research. 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, 4.75 mm. The laboratory test results are tabulated in Table 3.5 -Appendix III. The gradation and sieve analysis was determined in accordance with ASTM C 136 – 01 and tabulated in Table 3.6 -Appendix III, and graphically illustrated in Fig 3.2-Appendix III.

3.3.4 Reinforcing Steel

8 and # 6 bars were used as longitudinal tensile and compressive reinforcement respectively. # 2 bar was used as shear reinforcement. The grade 60 and grade 40 steel was used for longitudinal and shear reinforcement respectively. Stress-strain diagram (#8 bar) is shown in Fig 3.4-Appendix III.

3.3.5 High Range Water Reducing Agent

Glenium 51 (product of BASF Chemical Company), is a high performance concrete superplasticizer based on modified polycarboxylic ether, was used in the research. The dosage was kept constant throughout the research work as 1.2 % by weight of cement. The technical data of Glenium 51 is tabulated in Table 3.7-Appendix III.

3.3.6 Water

Fresh water available in the university was used as mixing water and for curing of concrete.

3.4 CASTING OF SPECIMEN

Casting of specimens was carried out as per ASTM C 192 – 02. Six beams were prepared along with 5 cylinders (two 6x12 and three 4x8) for each beam.

3.4.1 Description of Specimens

Six reinforced concrete beams were cast to investigate the influence of longitudinal tensile steel reinforcement ratio ρ_{ℓ}. These beams having different longitudinal tensile steel reinforcement ratios ρ_{ℓ} = 1.053 %, 2.179 % and 2.705 %, were divided into two groups depending upon the amount of shear reinforcement ratio. Group-A did not have any shear reinforcement, whereas Group-B had minimum amount of shear reinforcement ratio as required by ACI 318 – 08. The cross-sectional dimension of all the six beams was 10” x 16”. The specification of specimens and material properties are shown in Table3.8-Appendix III, and diagrammatically illustrated in Fig 3.3 - Appendix III. Test results of concrete cylinder strength are shown in Table3.9 - Appendix III.

3.4.2 Fabrication of Specimens

The specimens were cast in 3/4" thick plywood shuttering. The shuttering was prepared in such a manner that it could be dismantled easily. The steel reinforcement cage was placed in the shuttering over the 0.5” spacers and tied up with the bars. The concrete for the beams was mixed in a rotary mixer. In one batch, 1 bag of cement was used. Three standard cylinders (6” x 12”) and two standard cylinders (4” x 8”) were cast for each beam. The shuttering was removed from beams after 48 hours. Beams (B1A to B3A) were cast on 12 January and Beams (B1B to B3B) were cast on 19 January 2011. Hessian cloth was placed on the beams and cured under shade along with the test cylinders.

3.5 TESTING OF SPECIMEN

3.5.1 Test Setup

The specimens were transported to Structural Laboratory of University of Engineering and Technology Lahore, where a Reaction Frame made from I-Girders having 100 ton load capacity is installed. The load was applied through a hydraulic jack and pump having 100 ton capacity. A pressure gauge having maximum reading of 400 kg/cm^2 and least count of 10 kg/cm^2, was attached to hydraulic pump to control the loading. The load was applied at centre point in 10 kg/cm^2interval. Three deflection gauges were placed under the beam at mid span and at quarter points. The beams were placed on the supports with the help of a gantry crane. Diagrammatically the test setup is shown in Fig 3.5 to Fig 3.8 Appendix III.

3.5.2 Testing Procedure

The beams were planned to be tested under one point load. The load was applied after centering and aligning the specimens on the test setup and making all necessary arrangements for recording the load and deflection. The load was applied in increments of 10 kg/cm^2, and deflections recorded at each load increment. During the application of load, the cracks were observed and marked on the beams.

Chapter - 4

EXPERIMENTAL RESULTS

4.1 GENERAL

The load was applied at midspan of beams in increments of 10 kg/cm^2.Each test was carried out in approximately one hour. At each load increment, deflections were recorded and cracks observed / marked before applying next load increment. Five cylinders were cast for each beam during casting of the specimens. Three cylinders (6” x 12”) were tested after 28 x days and two cylinders (4” x 8”) were tested on the day of testing of beams (Table 4.1-Appendix IV).

4.2 TEST BEHAVIOR OF SPECIMENS

4.2.1 Specimen B1A

The beam was loaded on 27 February 2011. The initial flexural cracks appeared at 9.52 Ton load. Increased load widened the cracks and additional cracks appeared between 16.2 Ton and 19.1 Ton loads. Inclined cracks appeared at 22.86 Ton load and the beam failed abruptly at 23.81 Ton load. Load deflection data plot are shown in Table 4.2 and Fig 4.1 - Appendix IV, respectively. The crack pattern of beam is shown in the Fig 4.4 - Appendix IV.

4.2.2 Specimen B1B

The beam was loaded on 1 March 2011. The initial flexural cracks appeared at 13.33 Ton load. Increased load widened the cracks and additional cracks appeared at17.14 Ton load. First inclined crack appeared at 22.86 Ton load after which more flexural cracks were observed and old cracks also propagated. Inclined cracks kept on appearing along with propagation of few flexural cracks. The beam developed several cracks before failing at 34.29 Ton load. Load deflection data plot are shown in Table 4.3 and Fig 4.1 - Appendix IV, respectively. The crack pattern of beam is shown in the Fig 4.5 - Appendix IV.

4.2.3 Specimen B2A

The beam was loaded on 27 February 2011. The initial flexural cracks appeared at 20.95 Ton load. Increased load widened the cracks and additional cracks appeared between 28.57 Ton and 30.48 Ton loads. Inclined cracks appeared at 30.48 Ton load and the beam failed abruptly at 32.38 Ton load. Load deflection data plot are shown in Table 4.4 and Fig 4.2 - Appendix IV, respectively. The crack pattern of beam is shown in the Fig 4.6 - Appendix IV.

4.2.4 Specimen B2B

The beam was loaded on 1 March 2011. The initial flexural cracks appeared at 24.76 Ton load. Increased load widened the cracks and additional cracks appeared at30.48 Ton load. First inclined crack appeared at 26.67 Ton load after which more flexural cracks were observed and old cracks also propagated. Inclined cracks kept on appearing along with propagation of few flexural cracks. The beam developed several cracks before failing at 55.24 Ton load. Load deflection data plot are shown in Table 4.5 and Fig 4.2 - Appendix IV, respectively. The crack pattern of beam is shown in the Fig 4.7 - Appendix IV.

4.2.5 Specimen B3A

The beam was loaded on 28 February 2011. The initial flexural cracks appeared at 20.95 Ton load. Increased load widened the cracks and additional cracks appeared between 22.86 Ton and 26.67 Ton loads. Inclined cracks appeared at 28.57 Ton load and the beam failed abruptly at 34.29 Ton load. Load deflection data plot are shown in Table 4.6 and Fig 4.3 - Appendix IV, respectively. The crack pattern of beam is shown in the Fig 4.8 - Appendix IV.

4.2.6 Specimen B3B

The beam was loaded on 28 February 2011. The initial flexural cracks appeared at 19.05 Ton load. Increased load widened the cracks and additional cracks appeared at 20.95 Ton load. First inclined crack appeared at 28.57 Ton load after which more flexural cracks were observed and old cracks also propagated. Inclined cracks kept on appearing along with propagation of few flexural cracks. The beam developed several cracks before failing at 60.95 Ton load. Load deflection data plot are shown in Table 4.7 and Fig 4.3 - Appendix IV, respectively. The crack pattern of beam is shown in the Fig 4.9 - Appendix IV.

4.2.7 Summary

Behavior of beams can be briefly described as under:

- Initial flexural cracks appeared between 9.52 Ton and 24.76 Ton load.
- As the load increased, some of these cracks were gradually inclined towards the loading point.
- The beams without shear reinforcement (the beams in Group-A) exhibited a sudden failure induced by diagonal cracking.
- The beams having minimum shear reinforcement (the beams in Group-B) showed several flexural diagonal cracks.

- The location of significant diagonal crack was between load and support points.
- The crack angle was observed to be between 35° and 45°.
- A drop in the applied load was observed in Group-B beams, when cracks approached the loading point.

Chapter - 5

ANALYSIS AND INTERPRETATION OF TEST RESULTS

5.1 GENERAL

The objective of the tests was to evaluate the influence of the longitudinal tensile reinforcement ratio on the response of beams having minimum shear reinforcement. The influence of this parameter was investigated by considering the reserve strength and the reserve deflection of the beams.

Beams in Group – A (beams without shear reinforcement) exhibited sudden failure induced by diagonal cracking. The location of significant diagonal crack was between load and support points. A sudden drop in the applied load was observed after reaching the maximum load and beams failed due to extended diagonal crack widths.

Beams in Group – B (beams having minimum shear reinforcement) developed several diagonal cracks before failure. Small flexural cracks developed at or near the mid span during the early stages of the loading. These cracks appeared at the bottom of the beam where the flexural stresses were maximum. The flexural cracks developed over the entire length of the beam with increase in load and changed their orientation as they propagated above the longitudinal reinforcement. After reaching the maximum load, there was significant increase in deflection accompanied by a small increase / decrease in the load.

5.2 INTERPRETATION OF TEST RESULTS

5.2.1 Load – Deflection Relationship

The load – deflection curves at mid span are shown in Fig 4.1 to Fig 4.3 - Appendix IV. After reaching the maximum load, the beams showed sudden drop in the load followed by increase in deflection for Group - B Beams and sudden failure in Group - A Beams. Since, the readings were recorded manually, only two to three readings were taken just prior to failure. This trend is reflected in the aforementioned graphs in the descending portion of the curve shown as dotted lines. The beams without shear reinforcement (Group – A) exhibited sudden failure induced by diagonal cracking. The ultimate load and the midspan deflection of Group – B beams was much greater than those of beams without shear reinforcement (Group – A) and increased as the longitudinal tensile reinforcement ratio increased.

5.2.2 Moment – Curvature Relationship

Moment – curvature (M – Ø) curve is shown from Fig 5.1 to Fig 5.3 - Appendix V and details given in Table 5.1 - Appendix V. These curves illustrate theoretical and experimental M – Ø relationships of the six beams. The experimental M – Ø curve has been developed from load – deflection data as under.

- For each load point, moment is determined from the load.
- From the three deflection gauges placed under the beams at third points, the deflected shape has been developed for each load increment and shown from Fig 5.4 to Fig 5.9 - Appendix V.
- From the deflected shape, the equation of curve is determined.

- For curvature values at a given load point, second order derivative of deflection equations are calculated.

5.2.3 Reserve Strength and Reserve Deflection

In the study, reserve strength is defined as the ratio between the ultimate shear capacity of the beams with the minimum shear reinforcement and that of the beams without shear reinforcement i.e. V_b / V_a. Likewise, reserve deflection is defined as the ratio between the deflection corresponding to the ultimate load of beams with shear reinforcement and the deflection of beams without shear reinforcement i.e. Δ_b / Δ_a.

5.2.4 Behavior of Beams

The reserve strength and deflections are given in detail in Table 5.2 – Appendix V. Fig 5.10 – Appendix V illustrates the relationship between longitudinal tensile reinforcement ratio and reserve strength. Clearly, the reserve strength increases as the amount of longitudinal tensile reinforcement ratio increases. Beams (B1B to B3B) designed according to ACI 318-08 shared the same amount of minimum shear reinforcement ratio ρ_t= 0.00154. However, the reserve strength of beams B1B, B2B and B3B increased with the increase in ρ_ℓ. The reserve strength ratios of beams B1B, B2B and B3B were 1.24, 1.52 and 1.81 respectively.

The increase in the amount of longitudinal tensile reinforcement ratio also increased the reserve deflection of the beams. The reserve deflections of beams B1B, B2B and B3B are 1.50, 2.28 and 3.71 respectively. Fig 5.11 - Appendix V illustrates the relationship between longitudinal tensile reinforcement ratio and reserve deflection.

5.2.5 Analysis of Test Results

Based on the test results of the six reinforced concrete beams, following inferences can be clearly drawn:-

- The load – deflection plot shows that the beams having minimum amount of shear reinforcement (Group – B beams) resist more loads and hence exhibit more deflections before failure as compared to the beams without shear reinforcement
- The moment – curvature plot indicates that the resisting moment capacity and curvature of beam is more in beams having minimum amount of shear reinforcement than beams without shear reinforcement.
- With the increase in longitudinal tensile reinforcement ratio, there is less propagation as well as widening of cracks which results in increase in shear strength of the member.
- With the increase in longitudinal tensile reinforcement ratio, reserve strength and reserve deflections also increase. Thus the reserve strength and the reserve deflection of a reinforced concrete beam with a small ρ_ℓ tends to be lower than that of a reinforced concrete beam with a large ρ_ℓ.
- In ACI 318 – 05, the equation for calculating minimum shear reinforcement was modified to include the effect of concrete compressive strength; however the influence of longitudinal tensile reinforcement ratio still needs to be incorporated in the provisions for minimum shear reinforcement.

5.3 MINIMUM SHEAR REINFORCEMENT PARAMETERS

5.3.1 Other Test Programs

Lee and Kim (2008) presented test results of 26 reinforced concrete beams having minimum shear reinforcement. In the study, 12 beams with different longitudinal tensile reinforcement ratios were tested to study the effect of longitudinal tensile reinforcement ratio on minimum shear reinforcement as described in Chapter 2. Similarly, Usman (2011) has presented test results of 6 reinforced concrete beams having minimum shear reinforcement and different longitudinal tensile reinforcement.

These available test results have been recorded as in Table 5.3 – Appendix V and for analysis the rudimentary evaluation of these data leads to the influencing factors for establishing requirements of minimum shear reinforcement.

These factors i.e. f'_c, a/d, $f_{yt}, f_{y\ell}$ and ρ_ℓ must be investigated for their individual as well as collective relationships with Reserve Strength (V_b/V_a).

5.3.2 Concrete Compressive Strength ($\mathbf{f'_c}$)

The Reserve Strength and Reserve Deflection seem to be proportional to $\sqrt{f'_c}$ as shown in Fig 5.12 – Appendix V and Fig 5.13 – Appendix V respectively. Though, the scatter in both the graphs does not follow any regular trend or pattern.

5.3.3 Shear Span to Depth Ratio ($\mathbf{a/d}$)

The Reserve Strength and reserve deflection seem to be inversely proportional to Shear Span to Depth Ratio (a/d) as shown in Fig 5.14 – Appendix V and Fig 5.15 – Appendix V respectively. The trend in the graphs show a pattern of decreasing trend of reserve strength and reserve deflection with increase in shear span to depth ratio.

5.3.4 Longitudinal and Transverse Reinforcement Tensile Strength Ratio $(\mathbf{f_{y\ell}/f_{yt}})$

The Reserve Strength and reserve deflection seem to have not any significant effect on Longitudinal and Transverse Reinforcement tensile strength ratio $(f_{y\ell}/f_{yt})$ as shown in Fig 5.16 – Appendix V and Fig 5.17 – Appendix V respectively.

5.3.5 Longitudinal Tensile Reinforcement Ratio $(\boldsymbol{\rho_\ell})$

The Reserve Strength and reserve deflection seem to be proportional to Longitudinal Tensile Reinforcement Ratio (ρ_ℓ) as shown in Fig 5.18 – Appendix V and Fig 5.19 – Appendix V respectively.

5.3.6 Minimum Shear Reinforcement Parameters in Combinations

As the scattered pattern in the aforementioned paragraphs do not give any clear picture of correlation between different tests so these parameters were incorporated together in combinations to find a correlation. Different combinations were tried to reach at some correlation between the reserve strength and the related parameters (as illustrated in Fig 5.20 - Appendix V and Fig 5.21 - Appendix V). Resultantly the plot of ρ_ℓ vs. $\frac{V_b}{V_a} \cdot \frac{a}{d} \cdot \frac{f_{y\ell}}{f_{yt}} \cdot \sqrt{f'_c}$ and $\frac{\Delta_b}{\Delta_a} \cdot \frac{a}{d} \cdot \frac{f_{y\ell}}{f_{yt}} \cdot \sqrt{f'_c}$ presented in Fig 5.22 – Appendix V and Fig 5.23 – Appendix V respectively show a more closer pattern towards finding a correlation between these variables.

5.4 COMPARISON WITH ACI CODE 318 - 08

ACI 318 – 08provide the expression for minimum shear reinforcement ratio that takes into account the effect of concrete compressive strength and yield stress of shear reinforcement. However, the test results indicate that the amount of minimum shear reinforcement needs to increase (decrease) as ρ_ℓ decreases (increases).

5.5 CONCLUSIONS

Six reinforced concrete beams were tested to verify the influence of longitudinal tensile reinforcement ratio on minimum shear reinforcement. Based on the test results, following conclusions are drawn:-

- Reserve strength and reserve deflection are positively related with the longitudinal ratioρ_ℓ.
- The ACI 318 - 08 contains an expression for the minimum shear reinforcement that does not take into account the effect of longitudinal tensile reinforcement ratio. This may lead to the reserve strength and the reserve deflection of a reinforced concrete beam with a small $\rho_\ell f_y$ being lower than that of a reinforced concrete beam with a large $\rho_\ell f_y$.
- In order to have uniform reserve strength, inclusion of factor$\rho_\ell$in the ACI code for determining minimum shear reinforcement should be considered**.**

5.6 RECOMMENDATIONS FOR FUTURE WORK

Mechanism of shear resistance is yet to be understood. Thus, it is imperative to undertake extensive research on the subject. Following is recommended for future studies:-

- Further experimental work is needed to confirm the test results of this study.
- Relation of inclined cracks and crack widths with the longitudinal steel.
- Effect of a/d on the minimum shear reinforcement.

APPENDIX – I

APPENDIX I

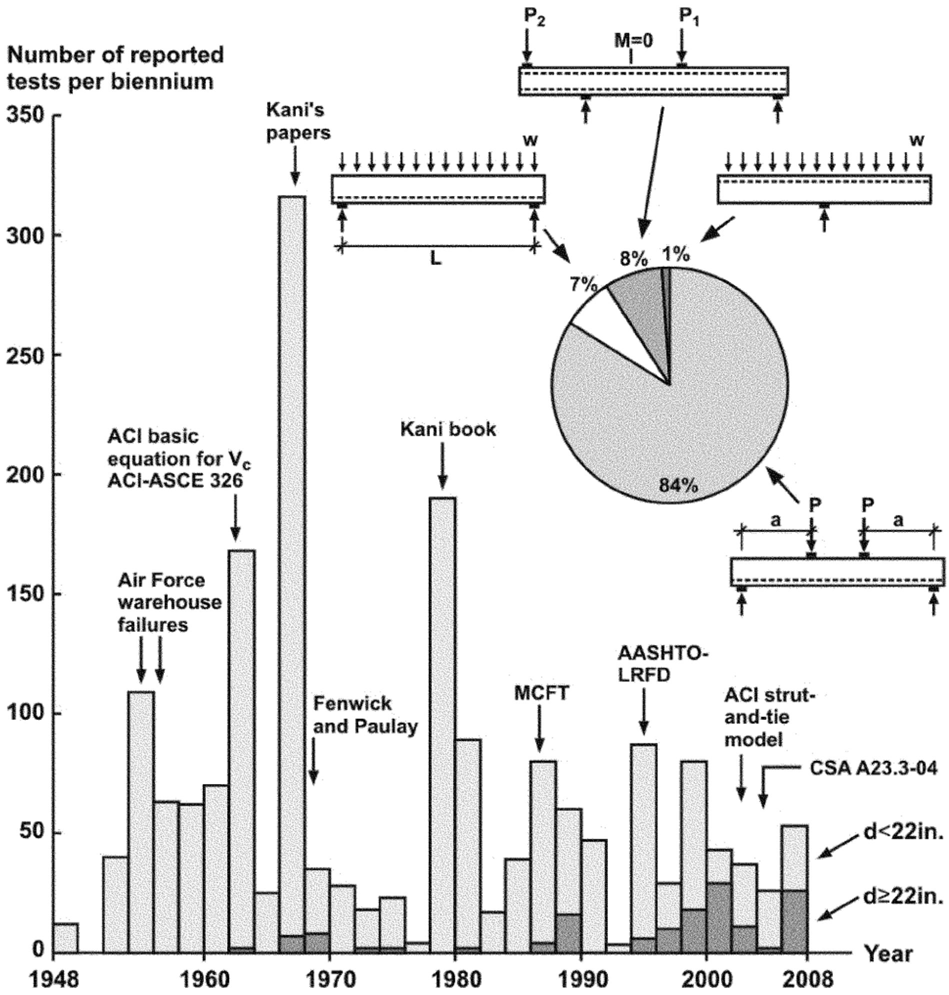


Fig 1.1 Sixty Years of Shear Research

APPENDIX – II

APPENDIX II

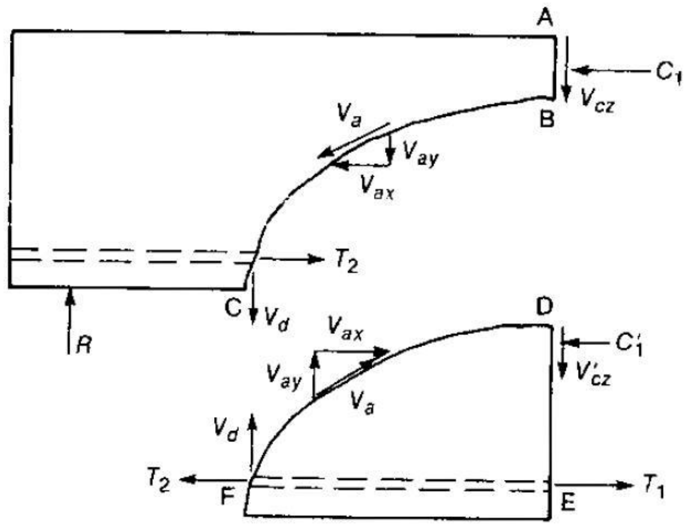


Fig 2.1 Internal Forces in a Cracked Beam without Stirrups

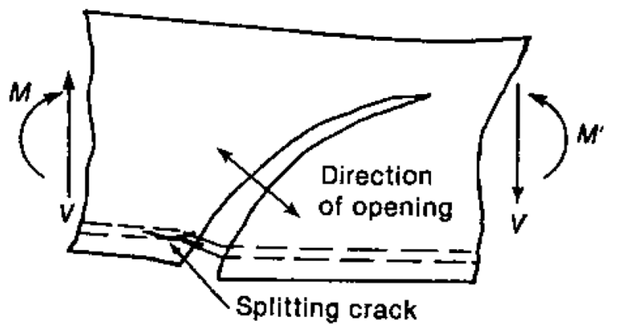


Fig 2.2 Opening of Diagonal Crack

APPENDIX – III

APPENDIX III

Table 3.1 Mix Design

Description	Details
Cement	455 Kg / m^3
Fine Aggregate	724 Kg / m^3
Coarse Aggregate	1063 Kg / m^3
W/C ratio	0.37
Superplasticizer (Glenium 51)	1.2 % by weight of cement
Mix ratio	1:1.59:2.34

Table 3.2 Properties of Cement

Tests	Test results	Specifications
Specific gravity	3.10	ASTM C 188 – 95
Initial setting time	150 minutes at 17^0C	ASTM C 191 – 01
Final setting time	390 minutes at 17^0C	ASTM C 191 – 01

Table 3.3 Properties of Fine Aggregate

Tests	Test results	Specifications
Specific gravity	2.65	ASTM C 128 – 01
Absorption	1.1%	ASTM C 128 – 01
FM	2.45	ASTM C 33 – 02

Table 3.4 Gradation of Fine Aggregate

Sieve No	Weight Retained (gm)	Percent Retained	Cumulative Percent Retained	Percent Passing	
				Actual	ASTM C 33 - 02
#4	2	0.2	0.2	99.8	95 - 100
#8	18	1.8	2	98	80 - 100
#16	130	13	15	85	50 - 85
#30	300	30	45	55	25 - 60
#50	413	41.3	86.3	13.7	5 - 30
#100	100	10	96.3	3.7	0 - 10
#200	34	3.4	99.7	0.3	
Pan	3	0.3	100	0	

Table 3.5Properties of Coarse Aggregate

Detail of tests	Test results
Impact value (percent)	13.9
Crushing value(percent)	20.2
Abrasion value(percent)	15.2
Specific gravity	2.67

Table 3.6 Gradation of Coarse Aggregate

Sieve Size (mm)	Weight Retained (gm)	Percent Retained	Cumulative Percent Retained	Percent Passing	
				Actual	ASTM C 33 - 02
19	0	0	0	100	100
12.5	80	8	8	92	90 - 100
9.5	401	40.1	48.1	51.9	40 - 70
4.75	494	49.4	97.5	2.5	0 - 15
2.36	25	2.5	100	0	0 - 5

Table 3.7Technical Data – High Range Water Reducing Agent

Description	Details
Name	Glenium 51
Form	Viscous liquid
Color	Light brown
Specific gravity	1.08 ± 0.02 g/cm^3
pH-value	7.0 ± 1
Alkali content (%)	Less than or equal to 5.0
Chloride content (%)	Less than or equal to 0.10

Table 3.8 Specification of Specimens and Material Properties

Beams	f'_c (psi)	Longitudinal tensile bars			Shear steel bars		a/d	d (in)
		No	ρ_L(%)	f_{ly} (ksi)	No	f_{ty} (ksi)		
Group-A								
B1A	5554	2 # 8	1.053	64	-	-	3.2	15
B2A	5554	4 # 8	2.179	64	-	-	3.3	14.5
B3A	5554	5 # 8	2.705	64	-	-	3.3	14.5
Group-B								
B1B	5554	2 # 8	1.053	64	# 2 @ 6.5” c/c	40	3.2	15
B2B	5554	4 # 8	2.179	64	# 2 @ 6.5” c/c	40	3.3	14.5
B3B	5554	5 # 8	2.705	64	# 2 @ 6.5” c/c	40	3.3	14.5

Table 3.928 Days Compressive Strength of Cylinders

Specimen	Size of cylinders (in)	28 days compressive strength (psi)
B1A – 1	6 x 12	4050
B1A – 2	6 x 12	5015
B1A – 3	6 x 12	5556
B1B – 1	6 x 12	4410
B1B – 2	6 x 12	3996
B1B – 3	6 x 12	4611
B2A – 1	6 x 12	5760
B2A – 2	6 x 12	5540
B2A – 3	6 x 12	5680
B2B – 1	6 x 12	5445
B2B – 2	6 x 12	5905
B2B – 3	6 x 12	5775
B3A – 1	6 x 12	4120
B3A – 2	6 x 12	4751
B3A – 3	6 x 12	5960
B3B – 1	6 x 12	4875
B3B – 2	6 x 12	4860
B3B – 3	6 x 12	4699
Average	**-**	**5056**

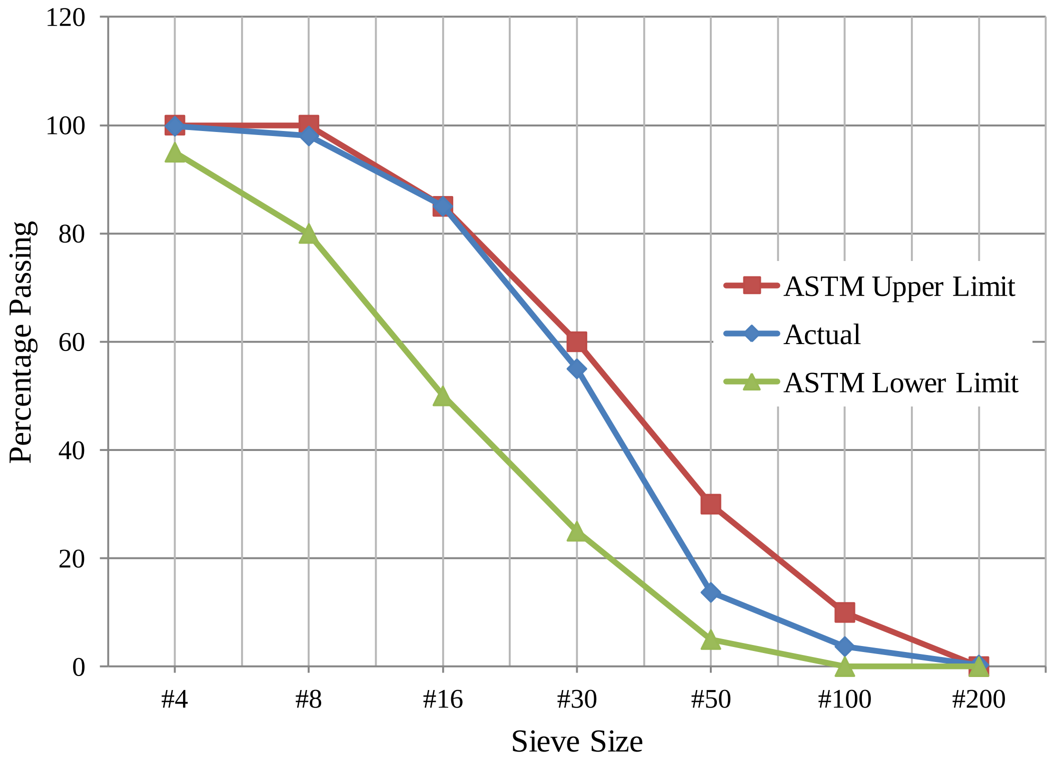


Fig 3.1 Particle Size Distribution of Fine Aggregate

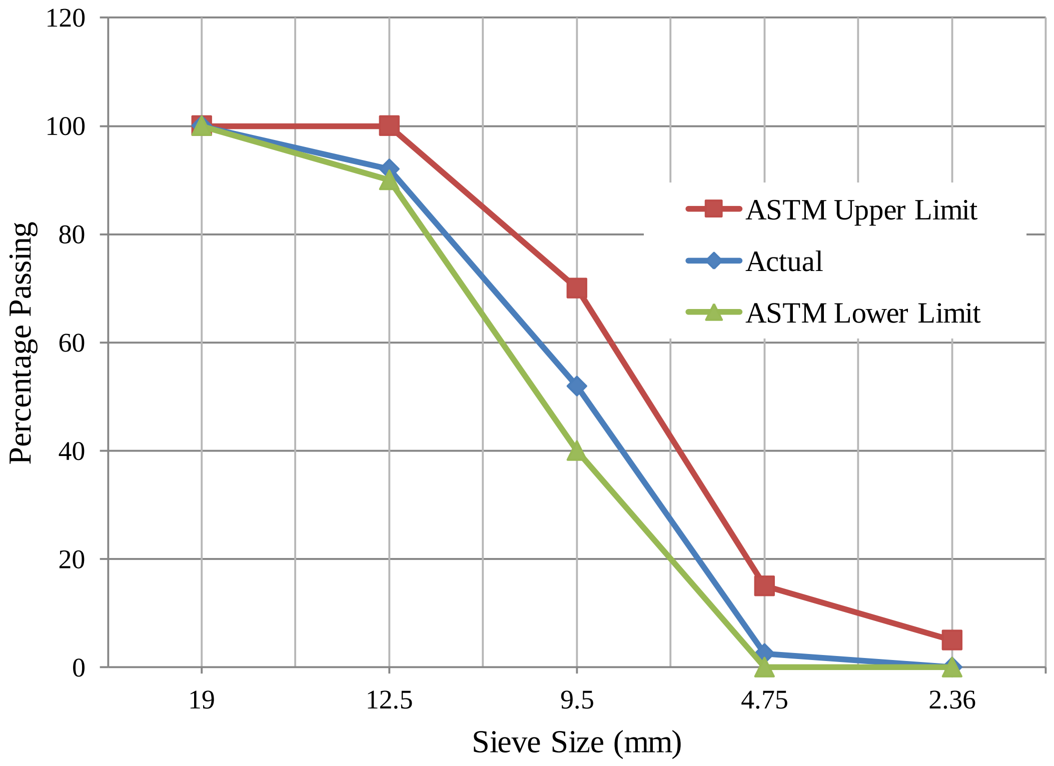
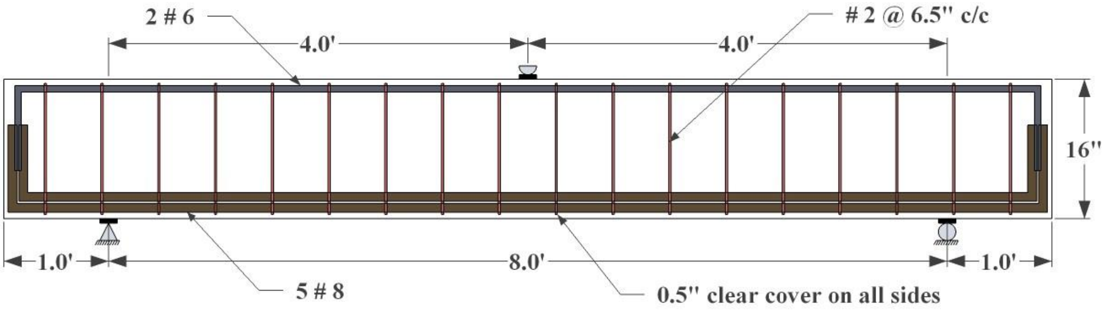
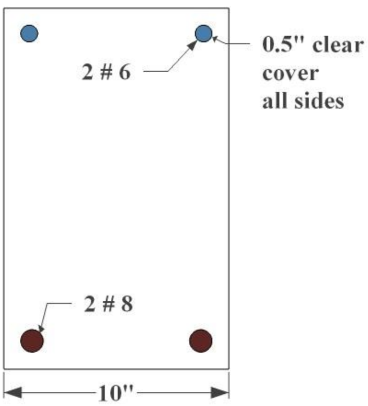


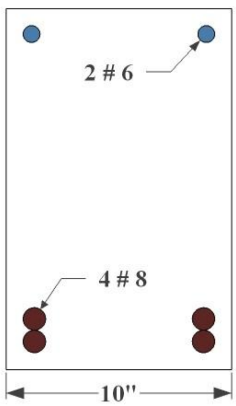
Fig 3.2 Particle Size Distribution of Coarse Aggregate



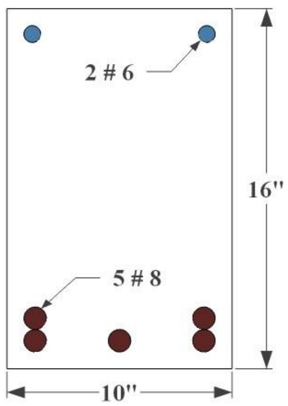
Longitudinal Section of Test Beam B3B



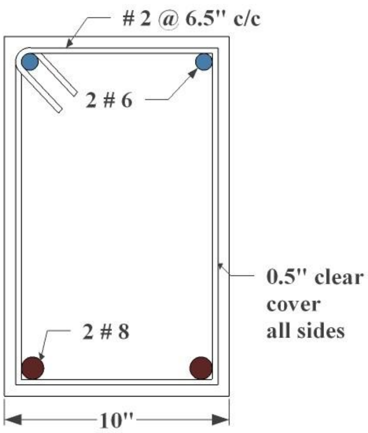
X-Section B1A



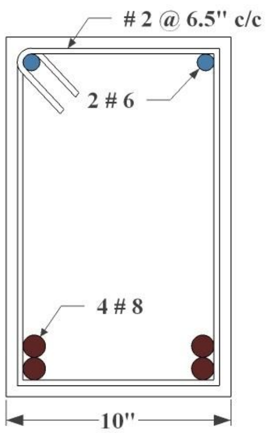
X-Section B2A



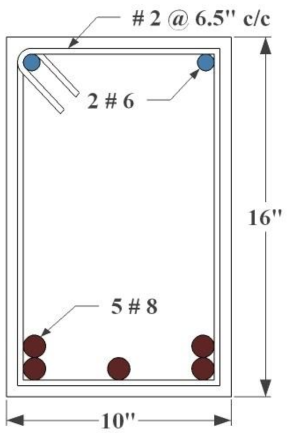
X-Section B3A



X-Section B1B



X-Section B2B



X-Section B3B

Fig 3.3 Dimensions and Reinforcement Detail of Test Beams

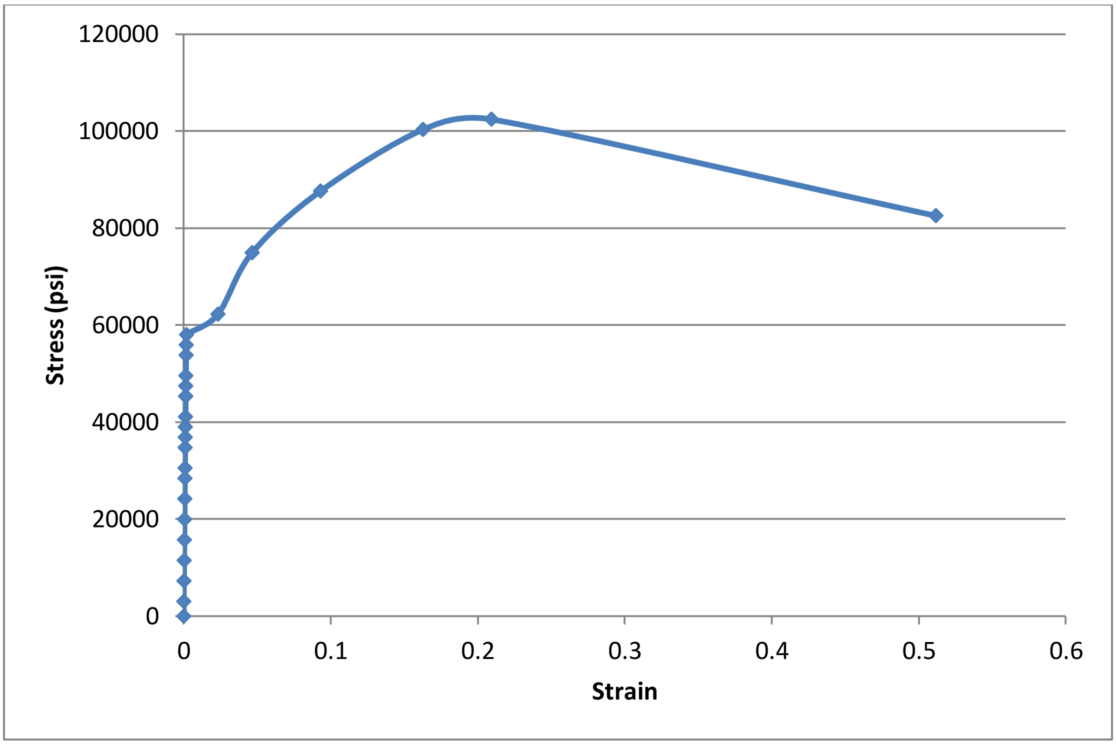


Fig 3.4 Stress-Strain Curve of Steel (#8 bar)

Fig 3.5 Test Setup

Fig 3.6 Test Setup

Fig 3.7 Test Setup (Deflection Gauges Placed at Third Point)

Fig 3.8 Test Setup (Supports)

APPENDIX – IV

APPENDIX IV

Table 4.1Compressive Strength on Day of Test

Specimen	Size of cylinders (in)	Compressive Strength (psi)
B1A – 4	4 x 8	5379
B1A – 5	4 x 8	5804
B1B – 4	4 x 8	5704
B1B – 5	4 x 8	4866
B2A – 4	4 x 8	6003
B2A – 5	4 x 8	4983
B2B – 4	4 x 8	5940
B2B – 5	4 x 8	5833
B3A – 4	4 x 8	5330
B3A – 5	4 x 8	5970
B3B – 4	4 x 8	5432
B3B – 5	4 x 8	5405
Average	**-**	**5554**

Table 4.2 Load Deflection Data - Beam B1A

Pressure Gauge Reading (kg/cm^2)	Load (Tons)	Deflection (mm)		
		Quarter Point	Mid Span	Quarter Point
0	0	0	0	0
10	1.90	0.127	0.178	0.165
20	3.81	0.203	0.305	0.254
30	5.71	0.356	0.508	0.432
40	7.62	0.508	0.737	0.622
50	9.52	0.762	1.194	0.889
60	11.43	1.118	1.753	1.270
70	13.33	1.372	2.159	1.702
80	15.24	1.626	2.565	1.842
90	17.14	1.930	3.124	2.235
100	19.05	2.235	3.556	2.540
110	20.95	2.565	4.140	2.870
120	22.86	2.845	4.750	3.302
125	23.81	11.303	8.763	7.493
0	0		2.388	

Table 4.3 Load Deflection Data - Beam B1B

Pressure Gauge Reading (kg/cm^2)	Load (Tons)	Deflection (mm)		
		Quarter Point	Mid Span	Quarter Point
0	0.00	0.000	0.000	0.000
10	1.90	0.178	0.165	0.114
20	3.81	0.368	0.406	0.318
30	5.71	0.546	0.635	0.508
40	7.62	0.737	0.889	0.737
50	9.52	0.991	1.295	1.016
60	11.43	1.321	1.803	1.397
70	13.33	1.753	2.413	1.791
80	15.24	2.108	2.896	2.159
90	17.14	2.489	3.505	2.591
100	19.05	2.845	4.013	2.997
110	20.95	3.124	4.445	3.277
120	22.86	3.480	4.953	3.632
130	24.76	3.912	5.563	4.064
140	26.67	4.369	6.198	4.750
150	28.57	4.953	7.112	5.118
160	30.48	8.179	13.106	8.382
166	31.62	11.405	19.101	11.608
170	32.38	12.040	20.320	12.243
176	33.52	15.367	26.492	15.875
180	34.28	16.510	26.518	17.145
100	19.05	15.697	24.790	16.307
50	9.52	14.072	21.869	14.681
0	0.00	11.786	17.907	12.319

Table 4.4 Load Deflection Data - Beam B2A

Pressure Gauge Reading (kg/cm^2)	Load (Tons)	Deflection (mm)		
		Quarter Point	Mid Span	Quarter Point
0	0	0	0	0
10	1.90	0.051	0.076	0.064
20	3.81	0.127	0.191	0.152
30	5.71	0.254	0.330	0.279
40	7.62	0.330	0.445	0.356
50	9.52	0.521	0.660	0.533
60	11.43	0.597	0.851	0.660
70	13.33	0.762	1.118	0.838
80	15.24	0.927	1.372	0.991
90	17.14	1.118	1.651	1.194
100	19.05	1.321	1.956	1.372
110	20.95	1.499	2.210	1.549
120	22.86	1.727	2.515	1.765
130	24.76	1.930	2.858	1.981
140	26.67	2.159	3.200	2.210
150	28.57	2.413	3.556	2.438
160	30.48	2.553	3.785	2.565
170	32.38	2.870	4.242	2.845
170	32.38	4.216	7.785	12.954
0	0		2.286	

Table 4.5 Load Deflection Data - Beam B2B

Pressure Gauge Reading (kg/cm^2)	Load (Tons)	Deflection (mm)		
		Quarter Point	Mid Span	Quarter Point
0	0	0	0	0
10	1.90	0.076	0.076	0.051
20	3.81	0.178	0.216	0.178
30	5.71	0.305	0.394	0.292
40	7.62	0.432	0.559	0.432
50	9.52	0.584	0.787	0.584
60	11.43	0.711	0.991	0.724
70	13.33	0.864	1.245	0.889
80	15.24	1.118	1.600	1.118
90	17.14	1.245	1.803	1.245
100	19.05	1.410	2.057	1.422
110	20.95	1.600	2.337	1.626
120	22.86	1.778	2.616	1.803
130	24.76	1.981	2.896	2.007
140	26.67	2.184	3.200	2.210
150	28.57	2.362	3.505	2.413
160	30.48	2.616	3.886	2.667
170	32.38	2.896	4.267	2.896
180	34.28	3.188	4.674	3.188
190	36.19	3.505	5.131	3.493
200	38.09	4.140	6.020	3.937
210	40.00	4.572	6.375	4.318
220	41.90	4.953	6.909	4.775
230	43.81	5.334	7.493	5.207
240	45.71	5.588	8.179	5.639
250	47.62	6.198	8.915	6.172
260	49.52	6.706	9.652	6.731
270	51.43	8.382	12.802	8.433
280	53.33	11.303	17.780	11.430
290	55.24	13.843	22.987	13.843
290	55.24	15.367	26.543	16.256
100	19.05	13.335	15.596	14.478
50	9.52	11.811	8.077	12.700
0	0	9.830	5.080	11.024

Table 4.6 Load Deflection Data - Beam B3A

Pressure Gauge Reading (kg/cm^2)	Load (Tons)	Deflection (mm)		
		Quarter Point	Mid Span	Quarter Point
0	0	0	0	0
20	3.81	0.330	0.432	0.330
30	5.71	0.457	0.584	0.483
40	7.62	0.508	0.660	0.533
50	9.52	0.584	0.762	0.635
60	11.43	0.711	0.940	0.711
70	13.33	0.813	1.067	0.813
80	15.24	0.965	1.295	0.965
90	17.14	1.143	1.524	1.143
100	19.05	1.295	1.803	1.308
110	20.95	1.473	2.057	1.486
120	22.86	1.651	2.286	1.651
130	24.76	1.829	2.565	1.829
140	26.67	1.981	2.794	2.007
150	28.57	2.159	3.023	2.159
160	30.48	2.362	3.327	2.400
170	32.38	2.565	3.658	2.604
180	34.28	3.353	5.359	5.461
0	0	2.210	3.708	2.997

Table 4.7 Load Deflection Data - Beam B3B

Pressure Gauge Reading (kg/cm^2)	Load (Tons)	Deflection (mm)		
		Quarter Point	Mid Span	Quarter Point
0	0	0	0	0
10	1.90	0	0	0
20	3.81	0.229	0.229	0.102
30	5.71	0.406	0.406	0.254
40	7.62	0.533	0.584	0.368
50	9.52	0.737	0.813	0.559
60	11.43	0.864	0.978	0.711
70	13.33	1.041	1.219	0.876
80	15.24	1.245	1.499	1.067
90	17.14	1.448	1.803	1.295
100	19.05	1.600	2.032	1.473
110	20.95	1.803	2.311	1.664
120	22.86	2.045	2.667	1.905
130	24.76	2.184	2.870	2.070
140	26.67	2.311	3.048	2.210
150	28.57	2.515	3.353	2.451
160	30.48	2.756	3.708	2.718
170	32.38	2.972	4.039	2.946
180	34.28	3.251	4.445	3.251
190	36.19	3.556	4.826	3.556
200	38.09	4.077	5.385	3.937
210	40.00	4.394	5.817	4.293
220	41.90	4.724	6.350	4.775
230	43.81	5.080	6.858	5.232
240	45.71	5.461	7.468	5.639
250	47.62	5.969	8.128	6.147
260	49.52	6.375	8.738	6.604
270	51.43	6.833	9.296	7.061
280	53.33	7.366	9.982	7.696
290	55.24	8.331	11.049	8.585
300	57.14	9.347	12.192	9.601
310	59.05	10.414	13.564	10.922
320	60.95	14.402	16.002	15.113
260	49.52	19.812	21.082	20.447
250	47.62	20.320	27.203	21.082
150	28.57	23.368	25.578	25.654

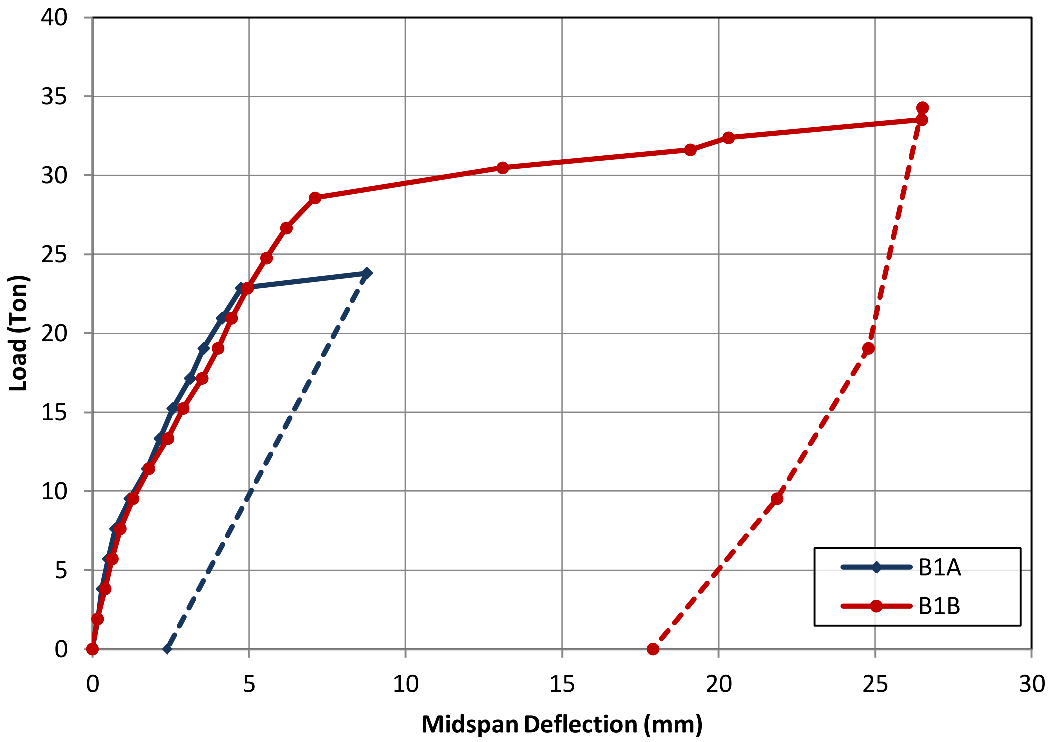


Fig 4.1 Load – Deflection Curves of Beams B1A and B1B

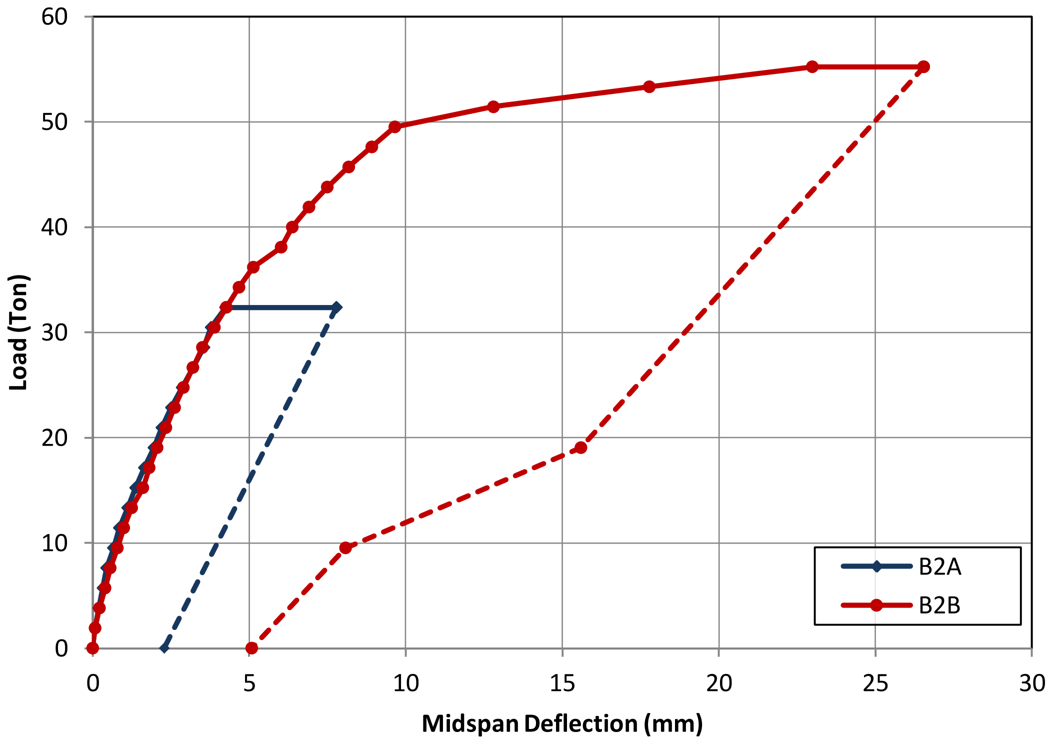


Fig 4.2 Load – Deflection Curves of Beams B2A and B2B

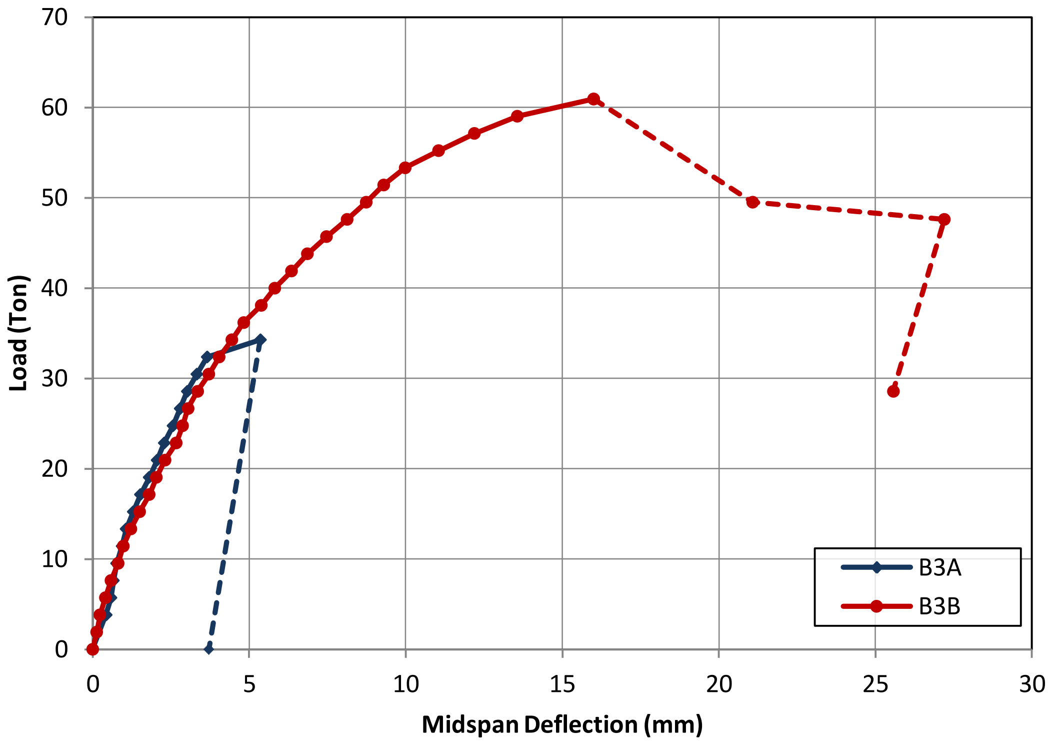


Fig 4.3 Load – Deflection Curves of Beams B3A and B3B

Fig 4.4 Crack Pattern of Beam B1A

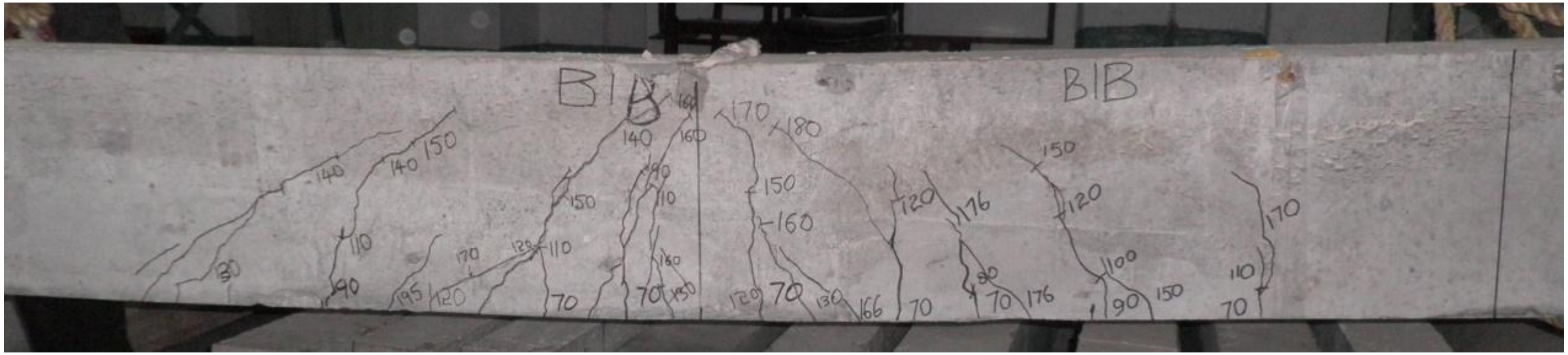


Fig 4.5 Crack Pattern of Beam B1B

Fig 4.6 Crack Pattern of Beam B2A

Fig 4.7 Crack Pattern of Beam B2B

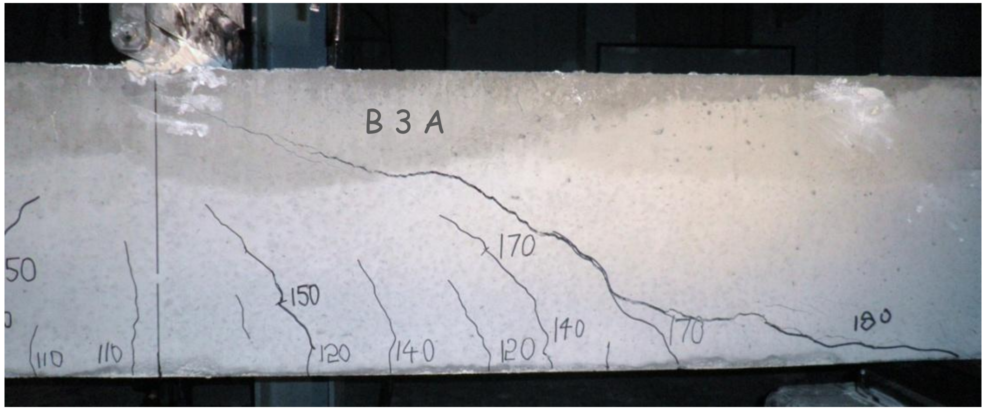


Fig 4.8 Crack Pattern of Beam B3A

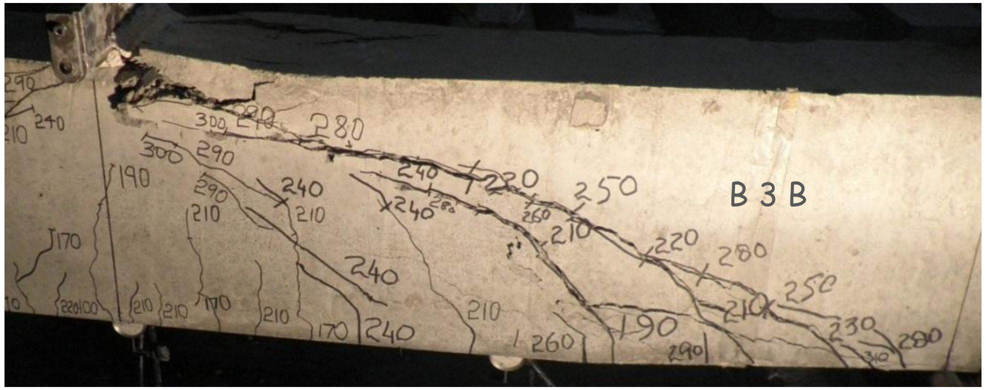


Fig 4.9 Crack Pattern of Beam B3B

APPENDIX – V

APPENDIX V

Table 5.1 Moment – Curvature Relationship

Theoretical		B1A		B1B		Theoretical		B2A		B2B		Theoretical		B3A		B3B	
M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$
(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
288.10	17.20	116.78	2.78	116.78	6.00	317.48	17.83	116.78	2.53	116.78	3.12	337.03	18.20	217.53	13.94	116.78	2.72
1290.20	212.59	217.53	9.76	217.53	14.00	1533.88	166.09	217.53	4.88	217.53	8.00	1658.84	152.62	318.29	20.88	217.53	2.72
1327.49	219.08	318.29	19.52	318.29	20.00	2428.21	275.87	318.29	10.88	318.29	14.27	2985.47	297.85	519.79	28.90	318.29	16.00
1464.07	806.78	419.04	24.40	419.04	40.00	2486.34	425.82	419.04	14.88	419.04	20.00	3009.40	340.56	721.29	39.91	419.04	16.96
1496.55	927.46	519.79	37.28	519.79	42.88	2501.05	499.72	519.79	21.44	519.79	26.11	3032.60	399.79	822.04	46.11	519.79	34.08
1524.62	1042.03	620.54	60.16	620.54	71.52	2508.28	571.93	620.54	28.64	620.54	37.95	3043.80	457.49	922.79	60.00	620.54	39.84
1548.90	1149.15	721.29	77.60	721.29	85.76	2514.49	640.00	822.04	48.64	721.29	45.90	3046.63	512.89	1023.54	73.90	721.29	40.10
1590.68	1342.37	822.04	88.80	822.04	108.64	2547.84	756.49	1023.54	65.76	822.04	60.58	3038.85	617.35	1225.04	81.06	822.04	54.08
		922.79	100.00	922.79	114.40	2565.29	861.75	1225.04	85.76	922.79	60.86			1426.55	102.88	922.79	59.84
		1124.29	137.60	1023.54	131.01			1426.55	105.76	1023.54	62.88			1728.80	125.76	1023.54	62.72
		1225.04	157.60	1124.29	159.76			1628.05	122.88	1124.29	82.88					1124.29	79.84
				1225.04	163.04			1728.80	139.14	1225.04	85.76					1225.04	100.00
				1325.80	183.04					1325.80	102.88					1325.80	101.15
				1426.55	228.61					1426.55	105.76					1426.55	105.60
				1527.30	239.52					1527.30	108.64					1527.30	111.36
				1628.05	428.80					1628.05	125.76					1628.05	131.36
				1688.50	657.60					1728.80	142.30					1728.80	141.06
				1728.80	795.20					1829.55	160.00					1829.55	160.00

Theoretical		B1A		B1B		Theoretical		B2A		B2B		Theoretical		B3A		B3B	
M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$	M	$\phi(x10^{-6})$
(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)	(Kip-in)	(rad/in)
				1789.25	915.20					1930.30	161.15					1930.30	161.06
				1829.55	995.20					2031.05	176.96					2031.05	179.84
										2131.80	239.52					2131.80	206.68
										2232.55	279.52					2232.55	214.98
										2434.06	399.04					2333.31	220.16
										2534.81	459.04					2434.06	225.92
										2736.31	609.60					2534.81	228.80
										2837.06	617.28					2635.56	289.86
										2937.81	625.92					2837.06	397.70
																2937.81	428.80
																3139.31	457.60

Table 5.2Test Results of Reinforced Concrete Beams

Beam	**V_a (Kip)**	**V_b (Kip)**	**Δ_a (in)**	**Δ_b (in)**	**V_b / Va**	**Δ_b / Δa**
B1A	25.85		0.19		1.24	1.50
B1B		32.15		0.28		
B2A	36.35		0.17		1.52	2.28
B2B		55.24		0.38		
B3A	36.35		0.14		1.81	3.71
B3B		65.74		0.53		

Table 5.3 Comparison of Results with Other Test Programs

Details	f'c	bw	h	d	w (dead)	As	ρ_L	fy_L	s	Av	ρ_T	L	a	a/d	fy_T	A Series			B Series			V_b/V_a	Δ_b/Δ_a
	psi	in	in	in	k/ft	in^2		ksi	in	in^2		ft	in		ksi	P (Kip)	V (Kip)	Δ (in)	P (Kip)	V (Kip)	Δ (in)		
Javaid																							
B1	5554	10.00	16.00	14.75	0.167	1.58	0.0107	64.00	6.50	0.10	0.0015	8.00	48.00	3.25	40.00	51.71	25.85	0.19	64.31	32.15	0.28	1.24	1.50
B2	5554	10.00	16.00	14.25	0.167	3.16	0.0222	64.00	6.50	0.10	0.0015	8.00	48.00	3.37	40.00	72.70	36.35	0.17	110.48	55.24	0.38	1.52	2.28
B3	5554	10.00	16.00	14.35	0.167	3.95	0.0275	64.00	6.50	0.10	0.0015	8.00	48.00	3.34	40.00	72.70	36.35	0.14	131.47	65.74	0.53	1.81	3.71
Usman																							
B4	5409	8.00	18.00	16.88	0.150	1.32	0.0098	64.00	8.50	0.10	0.0015	8.00	48.00	2.84	40.00	45.29	22.65	0.17	68.39	34.19	0.32	1.51	1.82
B5	5409	8.00	18.00	16.75	0.150	2.37	0.0177	64.00	8.50	0.10	0.0015	8.00	48.00	2.87	40.00	59.99	29.99	0.21	106.18	53.09	0.37	1.77	1.80
B6	5409	8.00	18.00	16.25	0.150	3.16	0.0243	64.00	8.50	0.10	0.0015	8.00	48.00	2.95	40.00	61.25	30.62	0.15	110.38	55.19	0.40	1.80	2.62
Lee and Kim (L1 - L3) 2008																							
L1	5916	13.80	17.70	16.14	0.254	3.99	0.0179	76.13	3.15	0.08	0.0018	7.58	48.43	3.00	31.18	88.55	44.28	0.16	98.23	49.11	0.30	1.11	1.82
L2	5916	13.80	17.70	15.75	0.254	6.98	0.0321	76.13	3.15	0.08	0.0018	7.58	47.24	3.00	31.18	101.15	50.58	0.14	116.45	58.23	0.25	1.15	1.74
L3	5916	13.80	17.70	15.16	0.254	9.96	0.0476	76.13	3.15	0.08	0.0018	7.58	45.47	3.00	31.18	101.38	50.69	0.12	144.35	72.18	0.47	1.42	3.98
Lee and Kim (L4 - L6) 2008																							
L4	4423	8.70	12.60	11.02	0.114	0.89	0.0093	79.75	5.51	0.08	0.0016	7.88	33.07	3.00	31.18	32.62	16.31	0.14	35.88	17.94	0.21	1.10	1.56
L5	4423	8.70	12.60	11.02	0.114	1.78	0.0186	79.75	5.51	0.08	0.0016	7.88	33.07	3.00	31.18	42.07	21.04	0.11	47.25	23.62	0.26	1.12	2.42
L6	4423	8.70	12.60	10.24	0.114	2.48	0.0279	79.75	5.51	0.08	0.0016	7.88	30.71	3.00	31.18	46.35	23.17	0.10	58.05	29.02	0.24	1.25	2.55

Details	f'c	bw	h	d	w (dead)	As	ρ L	fyL	s	Av	ρ T	L	a	a/d	fyT	A Series			B Series			V_b/V_a	Δ_b/Δ_a
	psi	in	in	in	k/ft	in^2		ksi	in	in^2		ft	in		ksi	P (Kip)	V (Kip)	Δ (in)	P (Kip)	V (Kip)	Δ (in)		
Lee and Kim (S1 – S3) 2008																							
S1	5916	13.8	17.7	16.14	0.254	4.99	0.0224	76.13	3.03	0.08	0.0018	5.38	32.28	2.00	31.18	88.44	44.22	0.06	158.19	79.10	0.17	1.79	2.95
S2	5916	13.8	17.7	16.14	0.254	4.99	0.0224	76.13	3.03	0.08	0.0018	8.07	48.42	3.00	31.18	81.70	40.85	0.14	101.73	50.86	0.33	1.25	2.29
S3	5916	13.8	17.7	16.14	0.254	4.99	0.0224	76.13	3.03	0.08	0.0018	10.7	64.56	4.00	31.18	77.89	38.94	0.28	106.69	53.34	0.62	1.37	2.20
Lee and Kim (S4 – S6) 2008																							
S4	4422.5	8.7	12.6	11.02	0.114	1.34	0.0140	79.75	5.51	0.08	0.0016	5.51	33.07	3.00	31.18	39.10	19.55	0.12	46.30	23.15	0.25	1.18	2.07
S5	4422.5	8.7	12.6	11.02	0.114	1.34	0.0140	79.75	5.51	0.08	0.0016	7.35	44.09	4.00	31.18	37.74	18.87	0.28	42.46	21.23	0.51	1.13	1.84
S6	4422.5	8.7	12.6	11.02	0.114	1.34	0.0140	79.75	5.51	0.08	0.0016	9.18	55.10	5.00	31.18	34.57	17.29	0.50	38.17	19.09	0.86	1.10	1.73

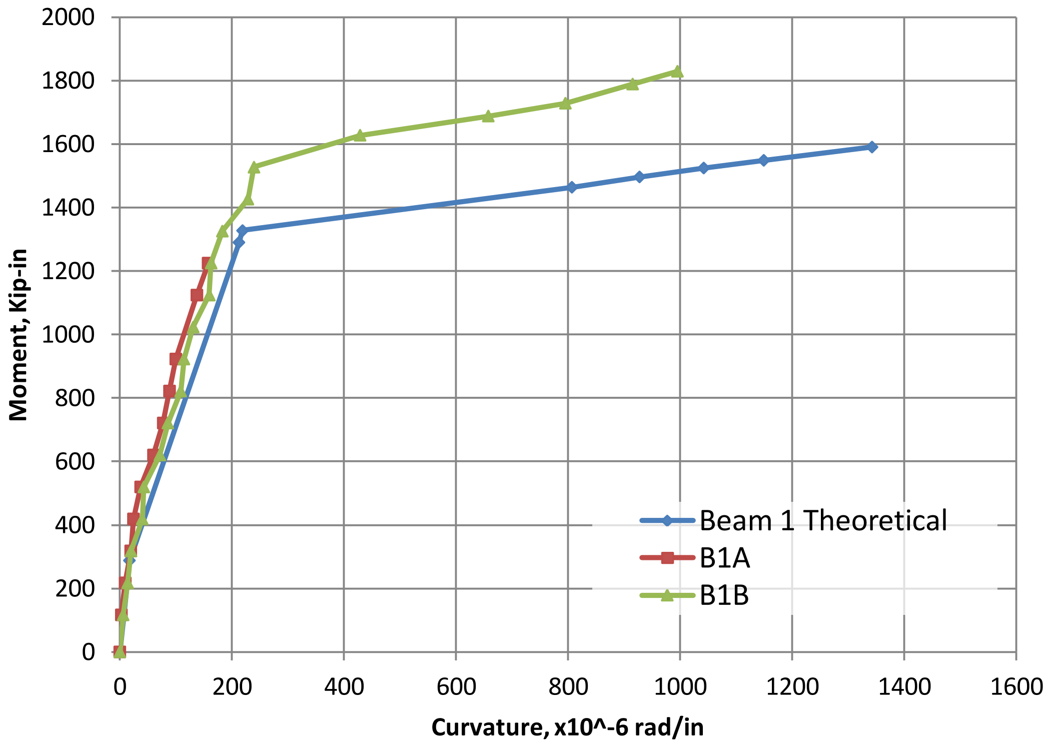


Fig 5.1 Moment – Curvature Relationship (B1A and B1B)

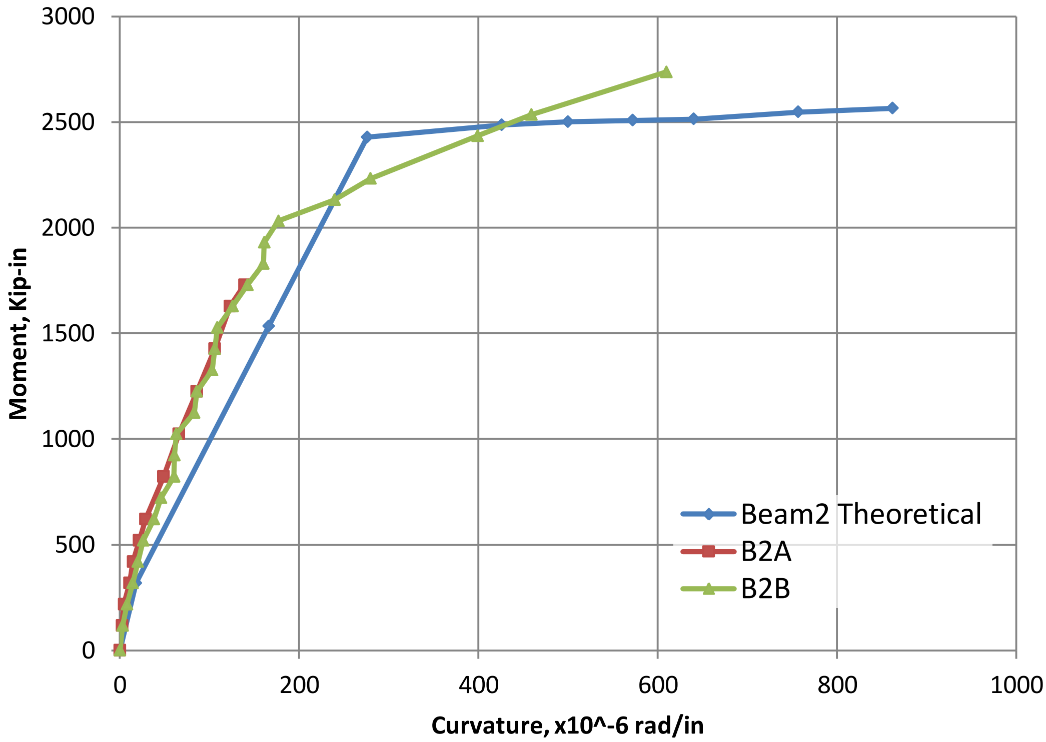


Fig 5.2Moment – Curvature Relationship (B2A and B2B)

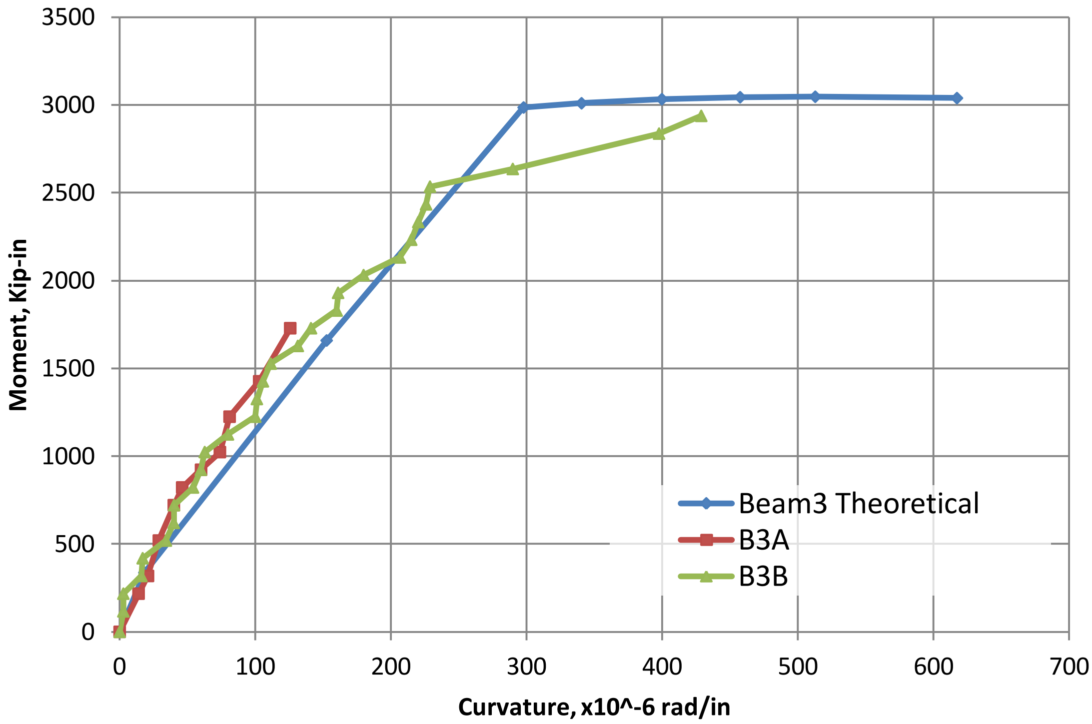


Fig 5.3 Moment – Curvature Relationship (B3A and B3B)

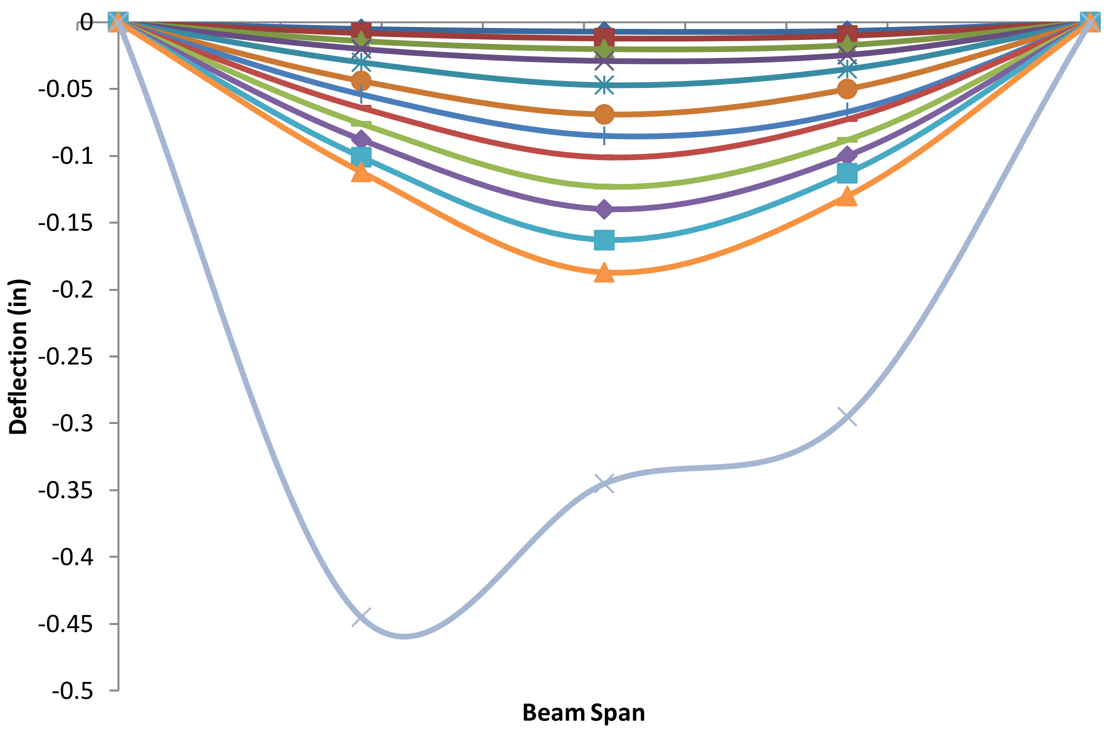


Fig 5.4 Deflected Shape of Beam at Different Load Levels (B1A)

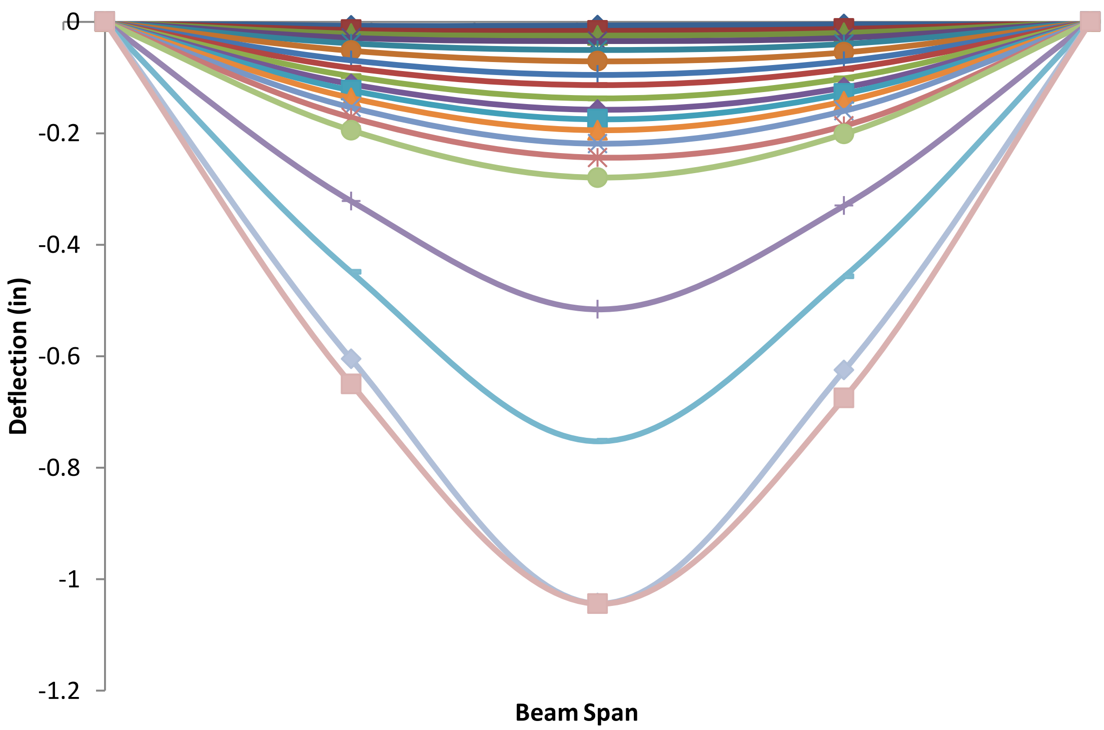


Fig 5.5 Deflected Shape of Beam at Different Load Levels (B1B)

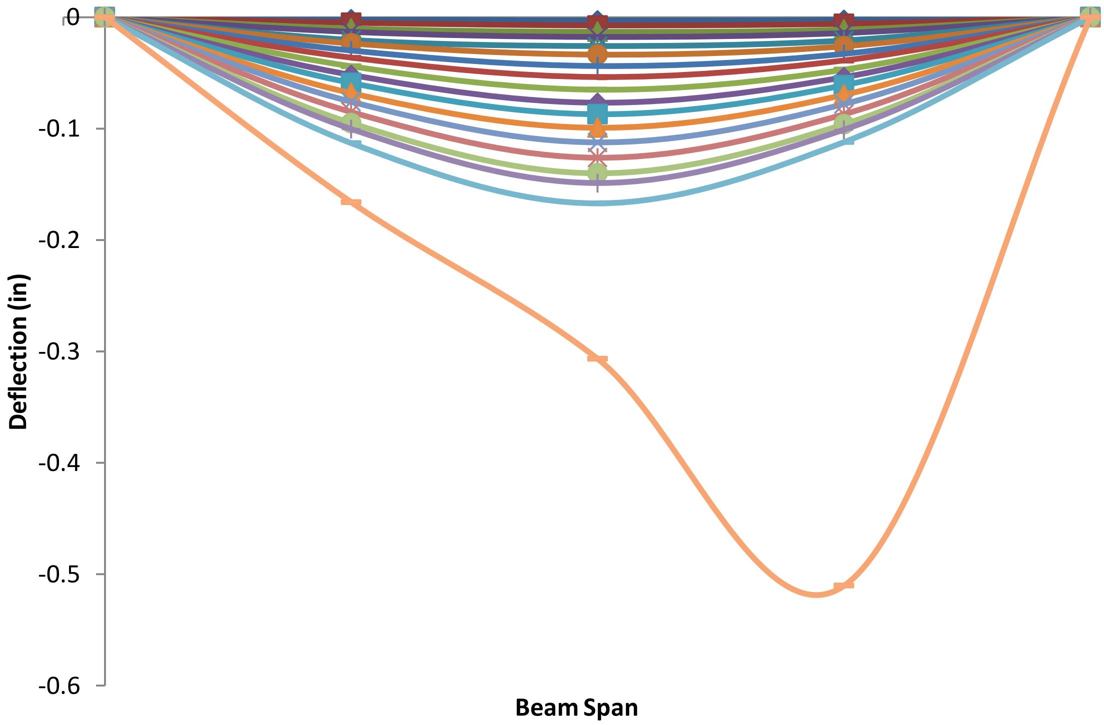


Fig 5.6 Deflected Shape of Beam at Different Load Levels (B2A)

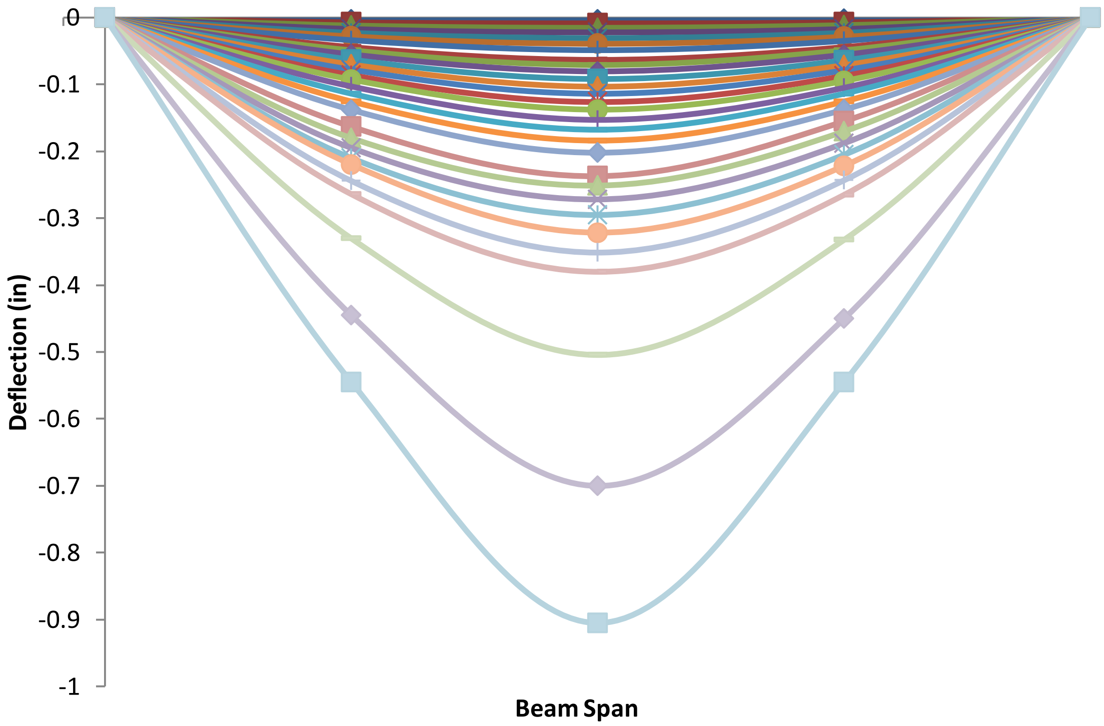


Fig 5.7 Deflected Shape of Beam at Different Load Levels (B2B)

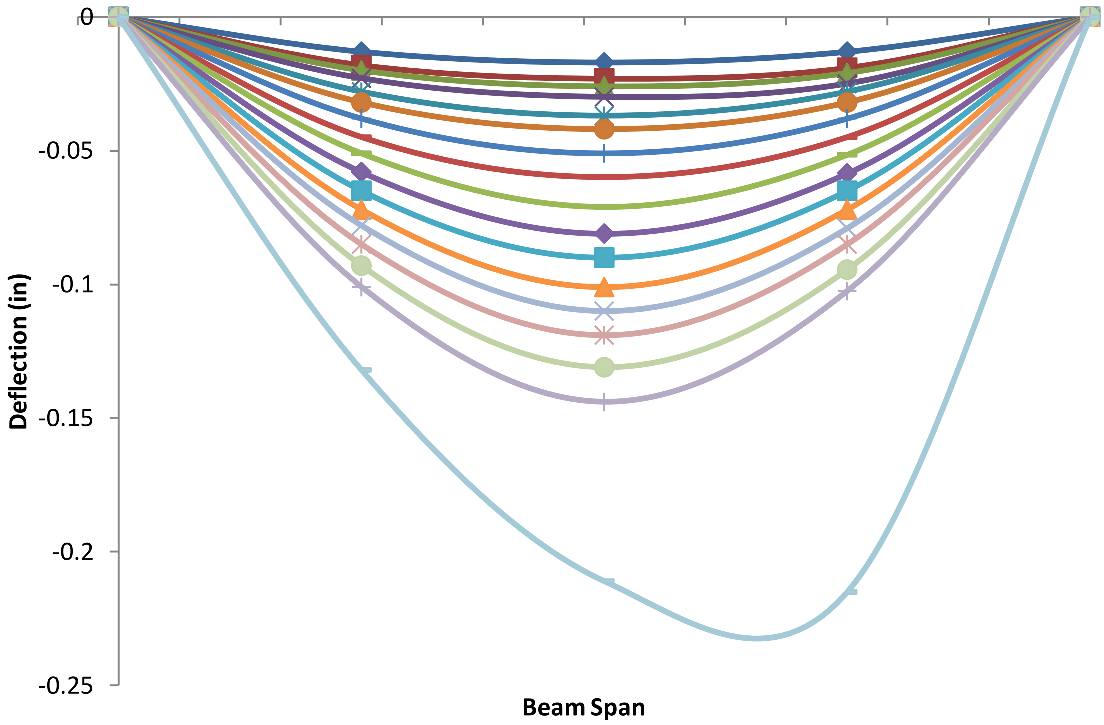


Fig 5.8 Deflected Shape of Beam at Different Load Levels (B3A)

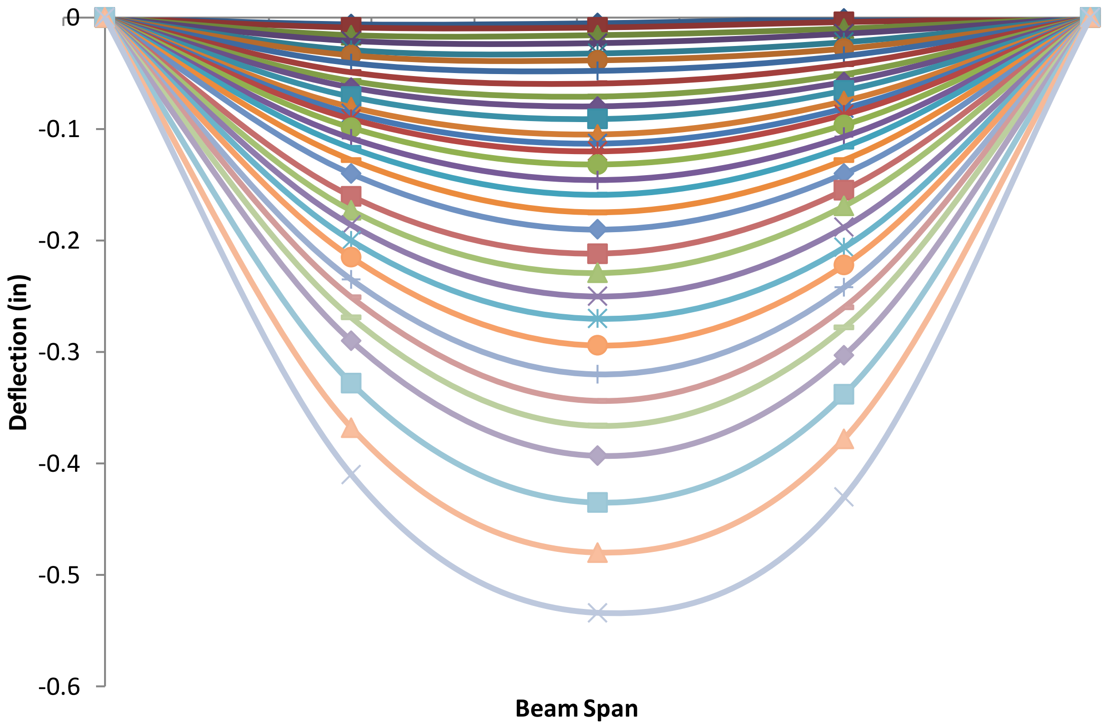


Fig 5.9 Deflected Shape of Beam at Different Load Levels (B3B)

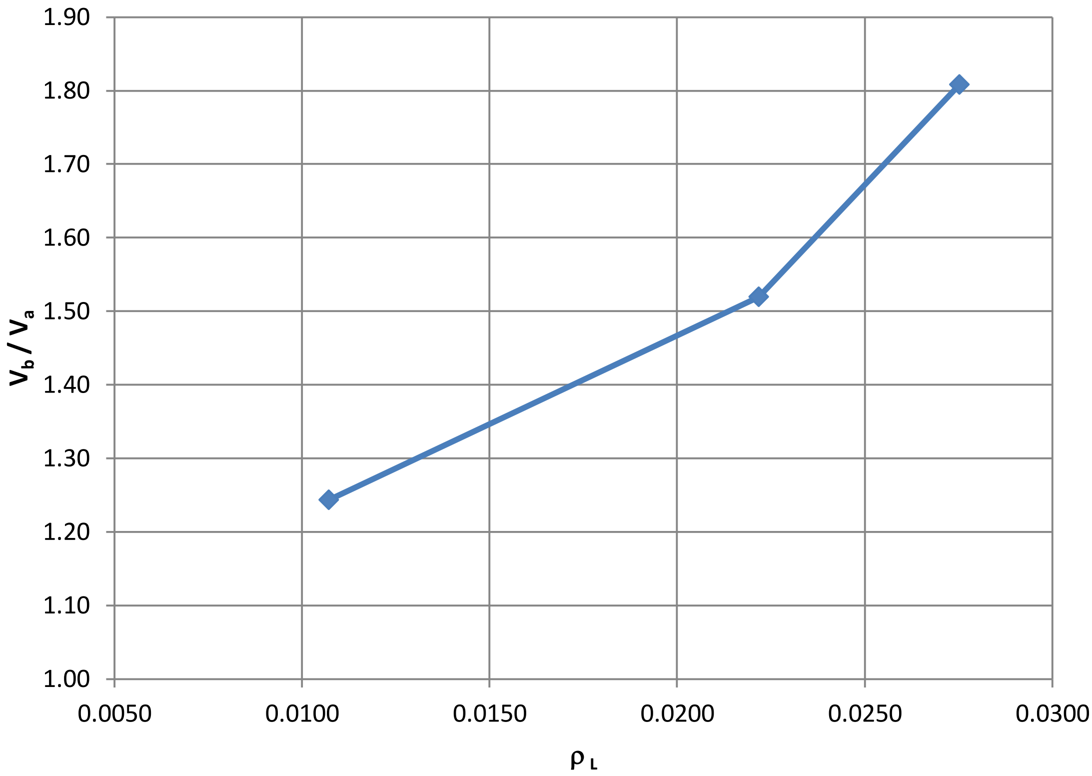


Fig 5.10Reserve Strength vs. Longitudinal Tensile Reinforcement Ratio

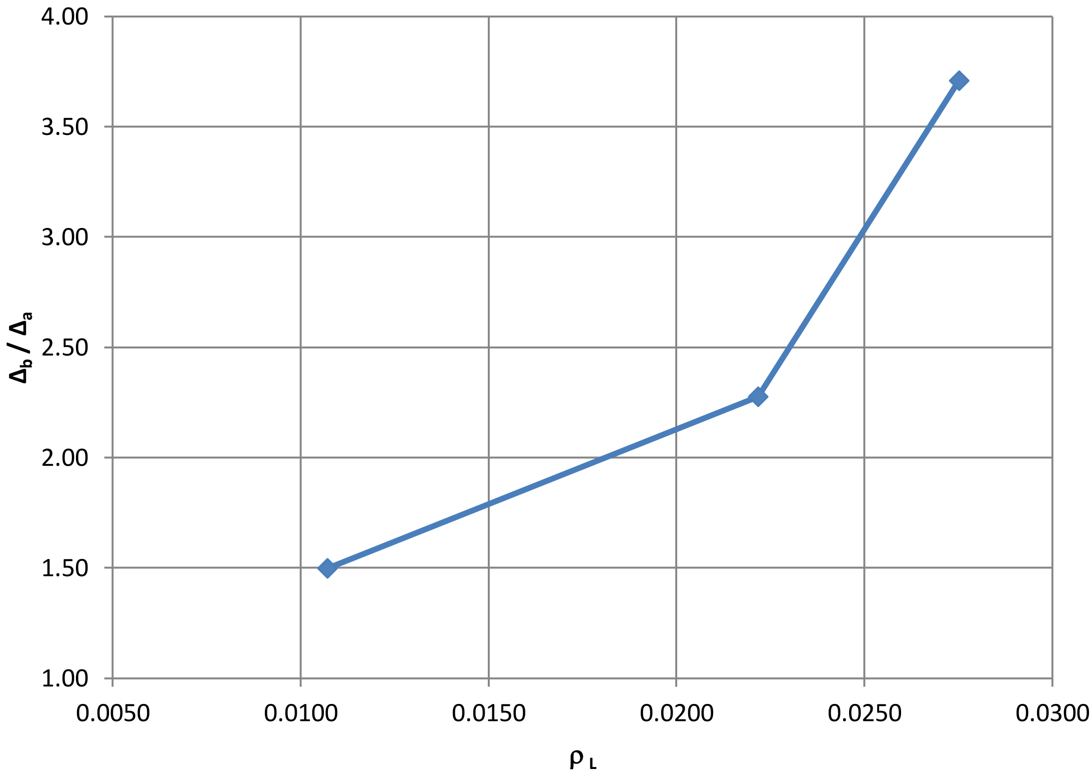


Fig 5.11 Reserve Deflection vs Longitudinal Tensile Reinforcement Ratio

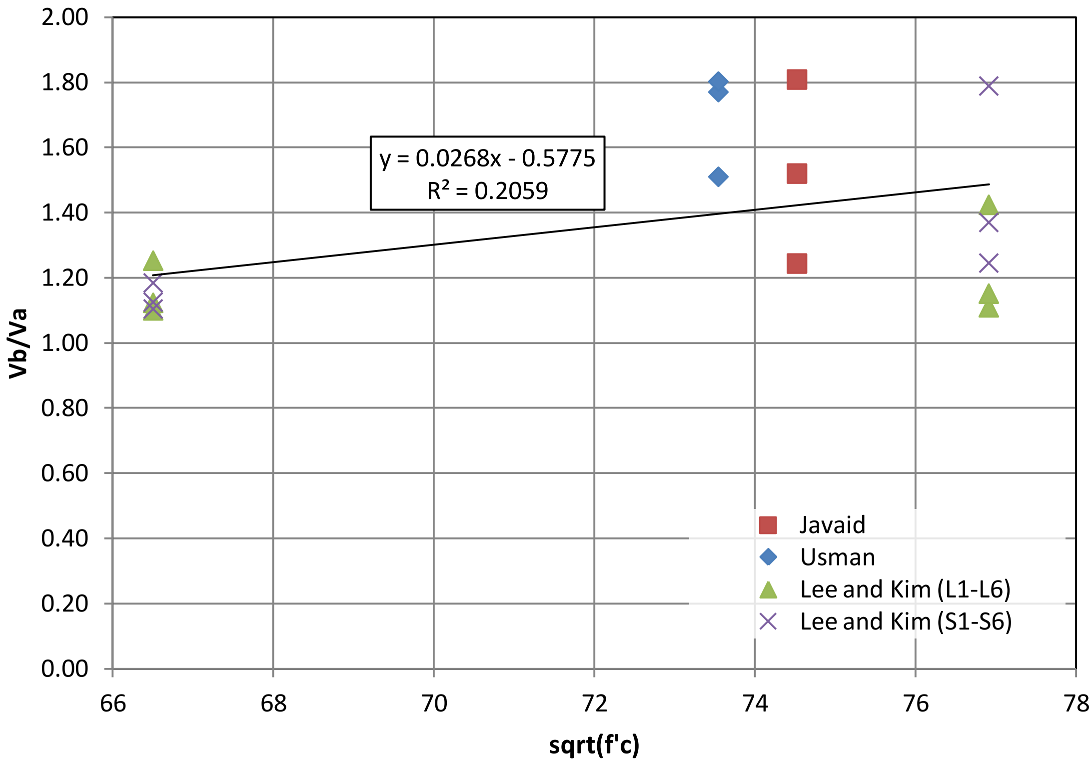


Fig 5.12 Reserve Strength vs Concrete Compressive Strength

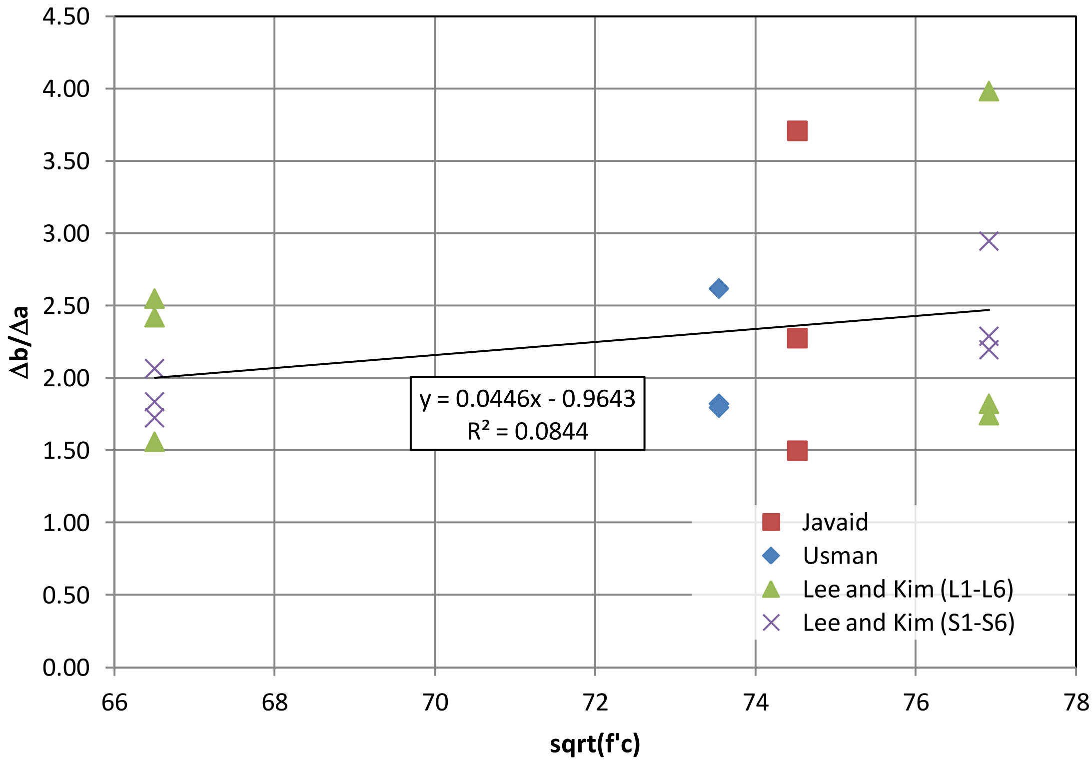


Fig 5.13 Reserve Deflection vs Concrete Compressive Strength

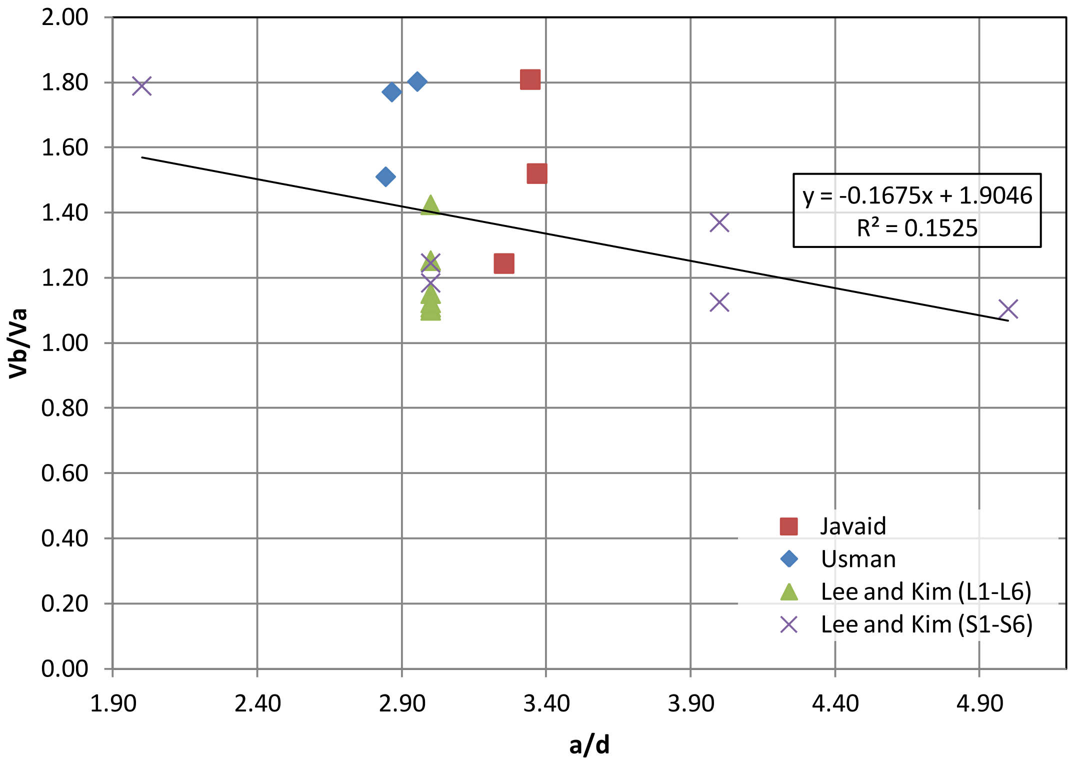


Fig 5.14 Reserve Strength vs Shear Span to Depth Ratio

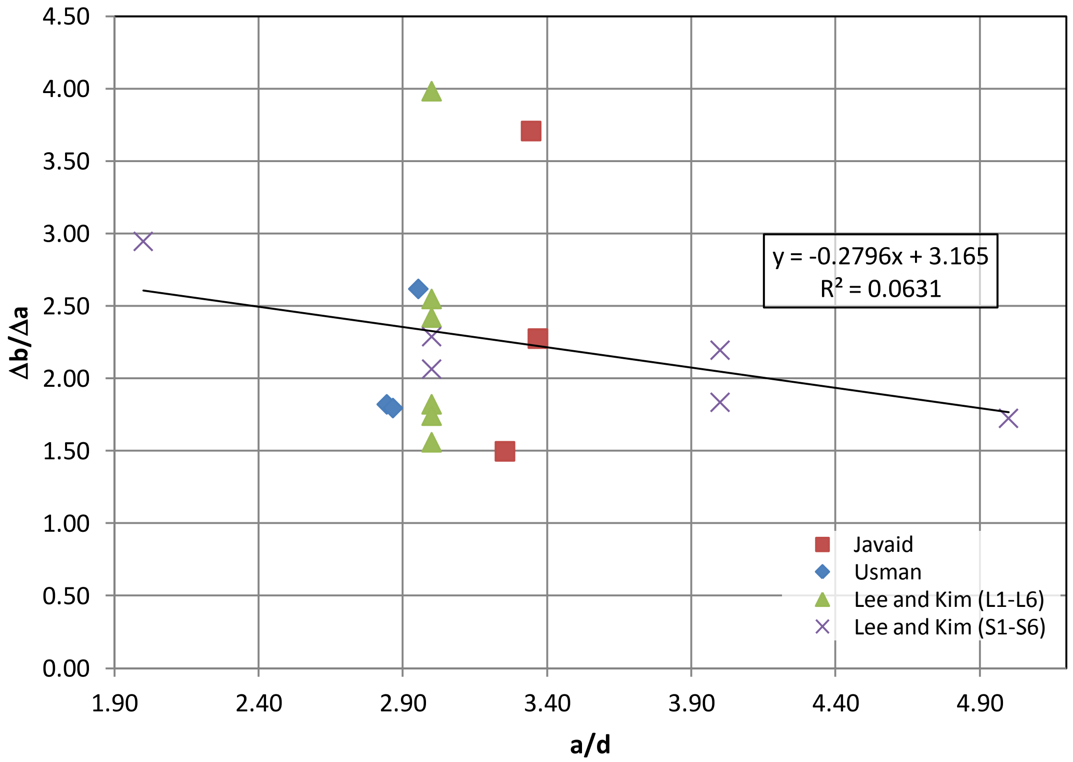


Fig 5.15 Reserve Deflection vs Shear Span to Depth Ratio

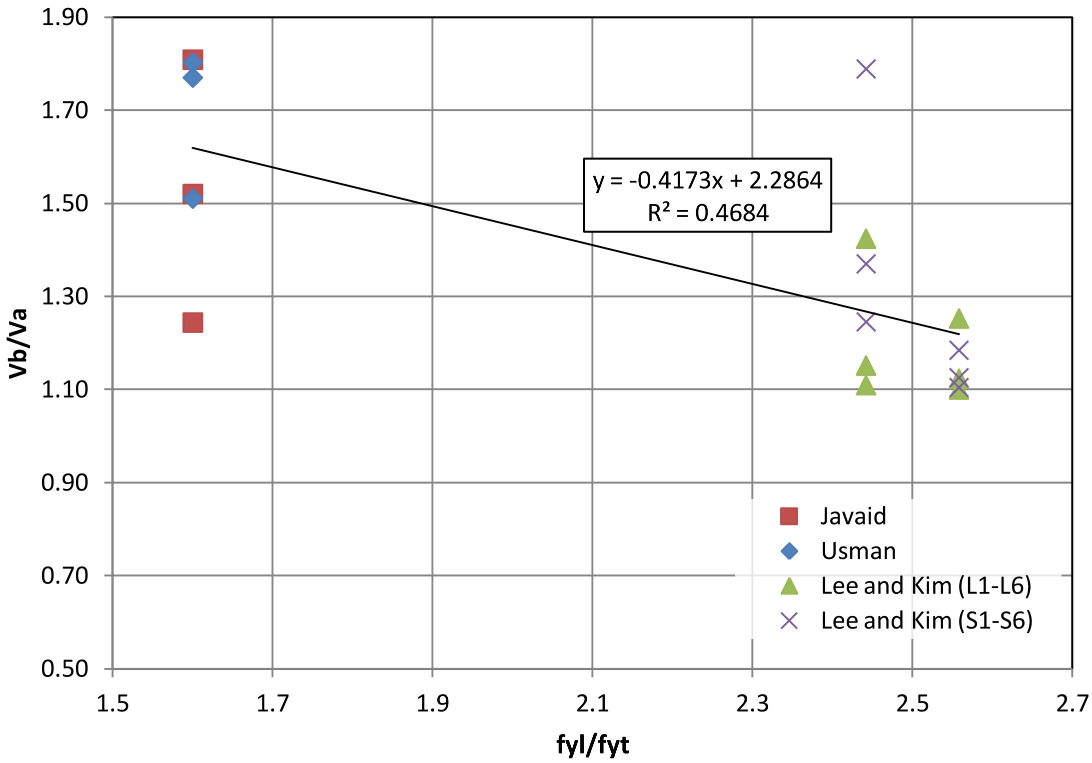


Fig 5.16 Reserve Strength vs Longitudinal and Transverse Reinforcement Tensile Strength Ratio ($f_{y\ell}/f_{yt}$)

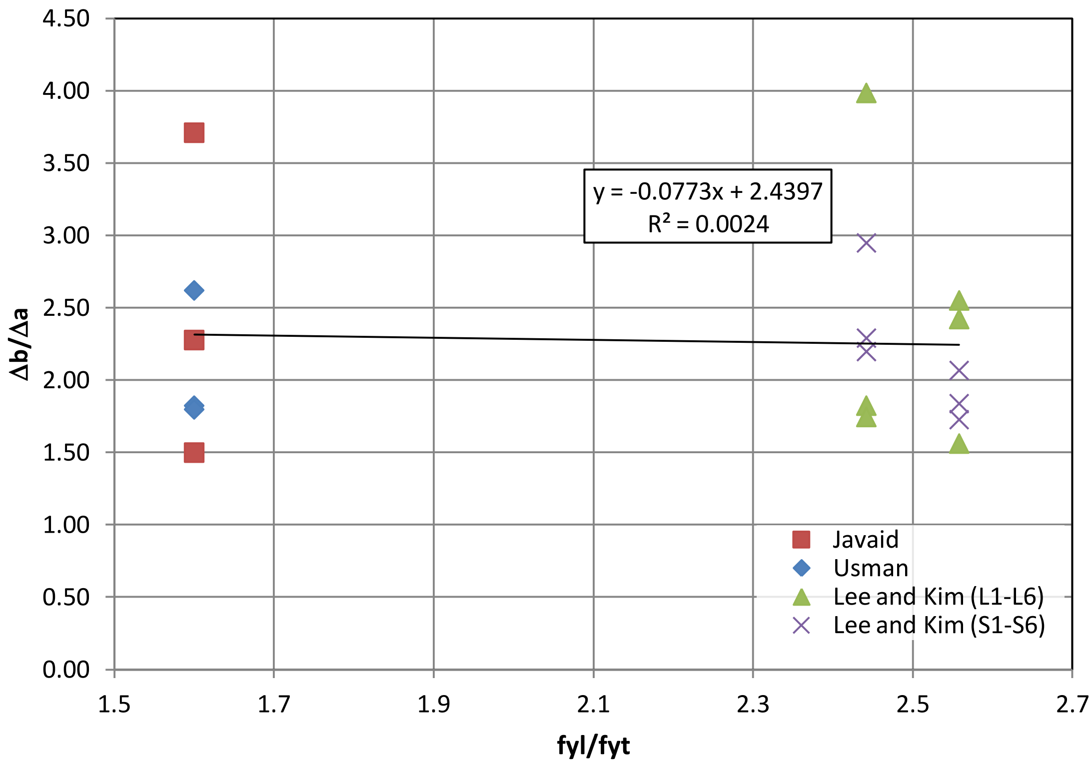


Fig 5.17 Reserve Deflection vs Longitudinal and Transverse Reinforcement Tensile Strength Ratio ($f_{y\ell}/f_{yt}$)

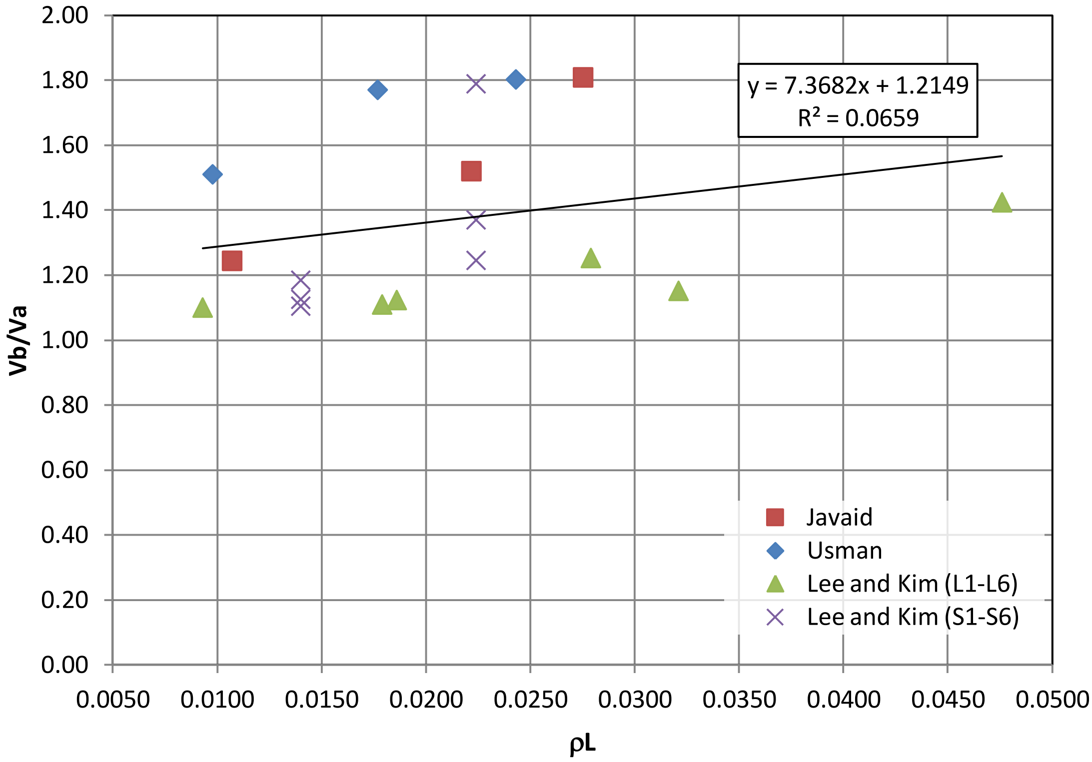


Fig 5.18 Reserve Strength vs Longitudinal Tensile Reinforcement Ratio

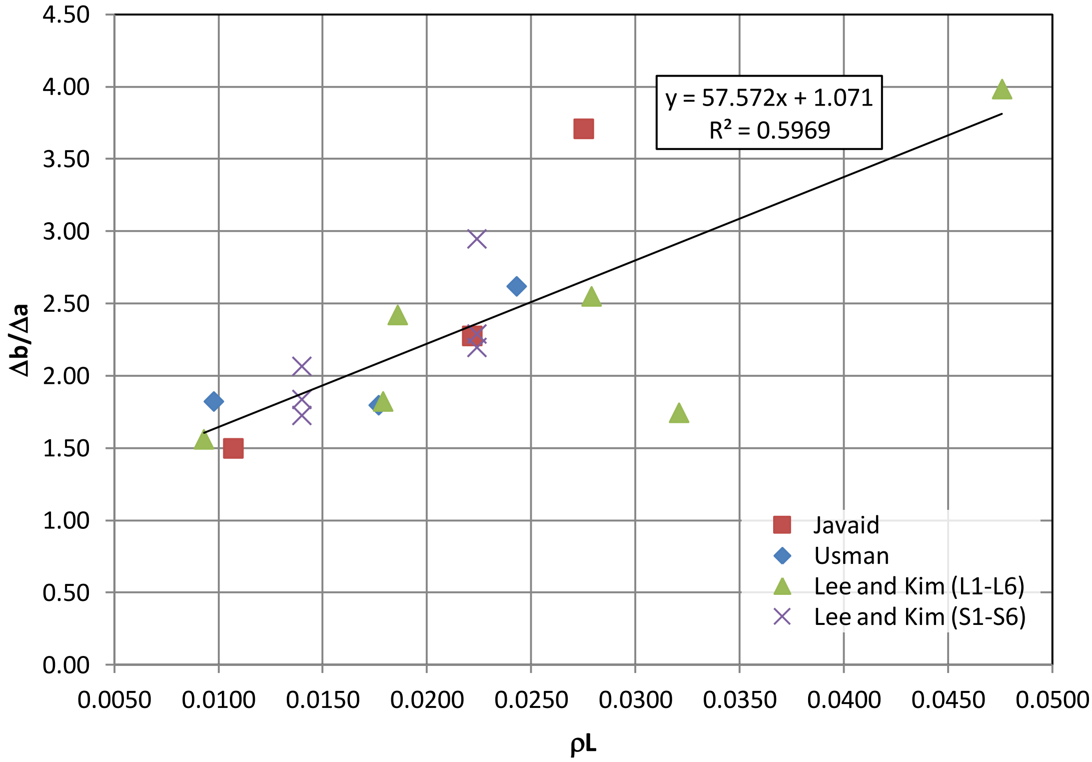


Fig 5.19 Reserve Deflection vs Longitudinal Tensile Reinforcement Ratio

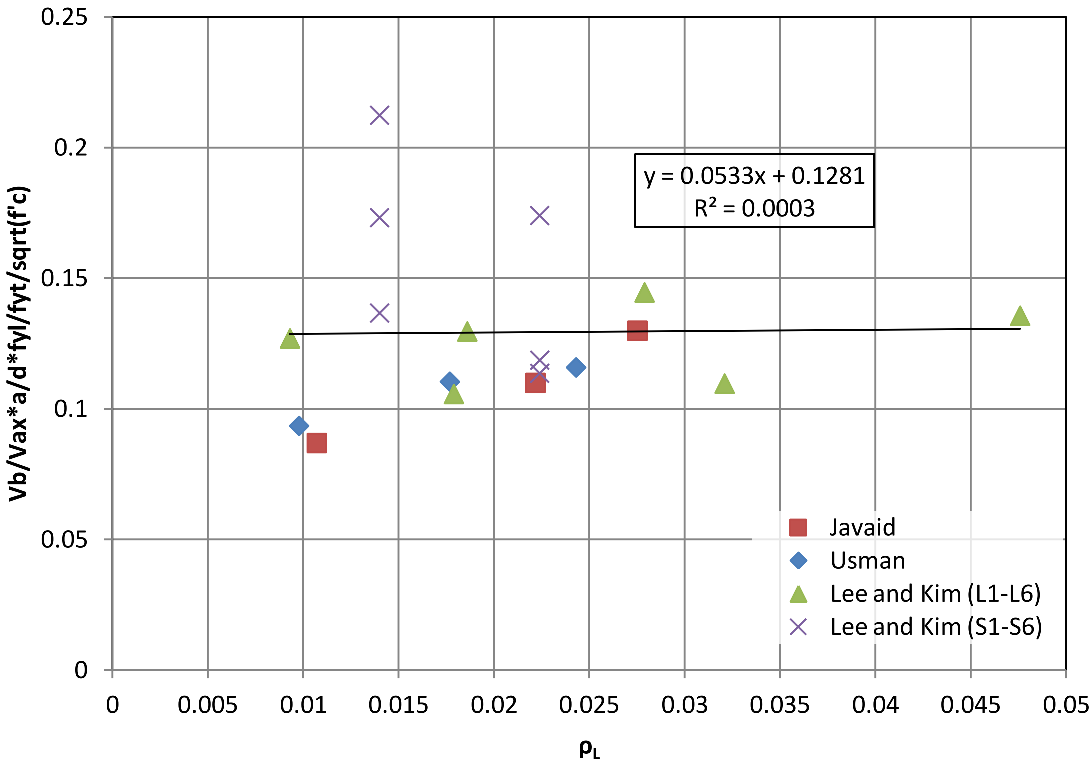


Fig 5.20 Reserve Strength vs Longitudinal Tensile Reinforcement Ratio (Effect of Parameters in Combination)

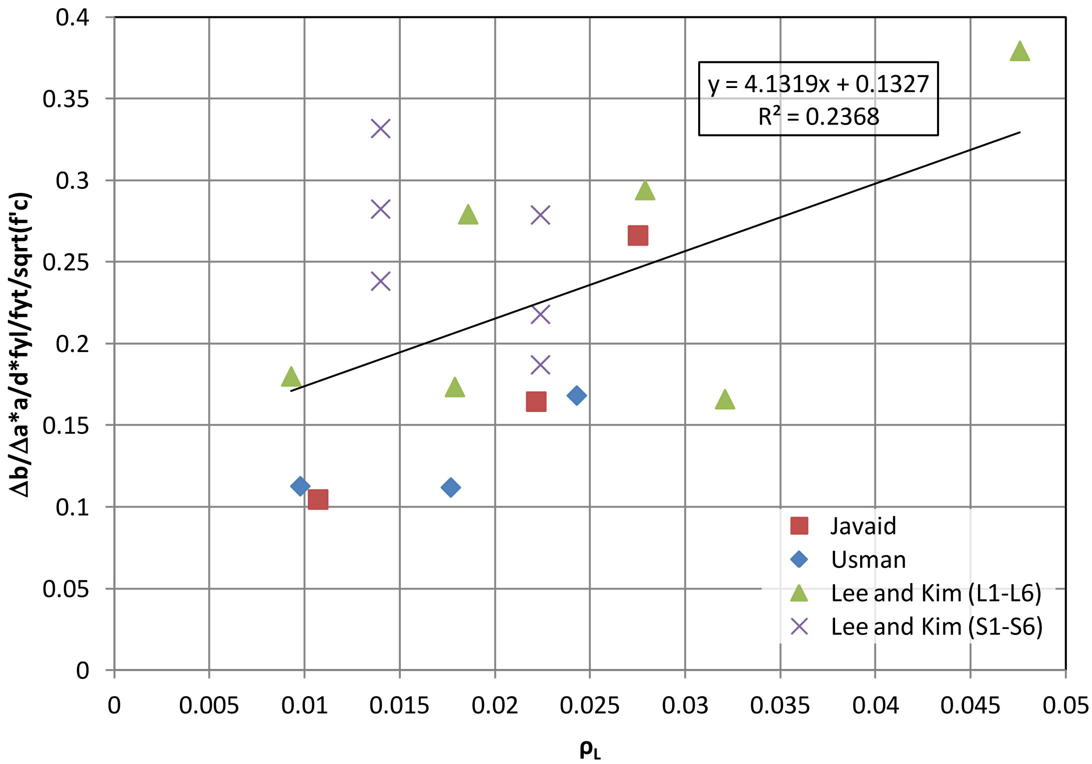


Fig 5.21 Reserve Deflection vs Longitudinal Tensile Reinforcement Ratio (Effect of Parameters in Combination)

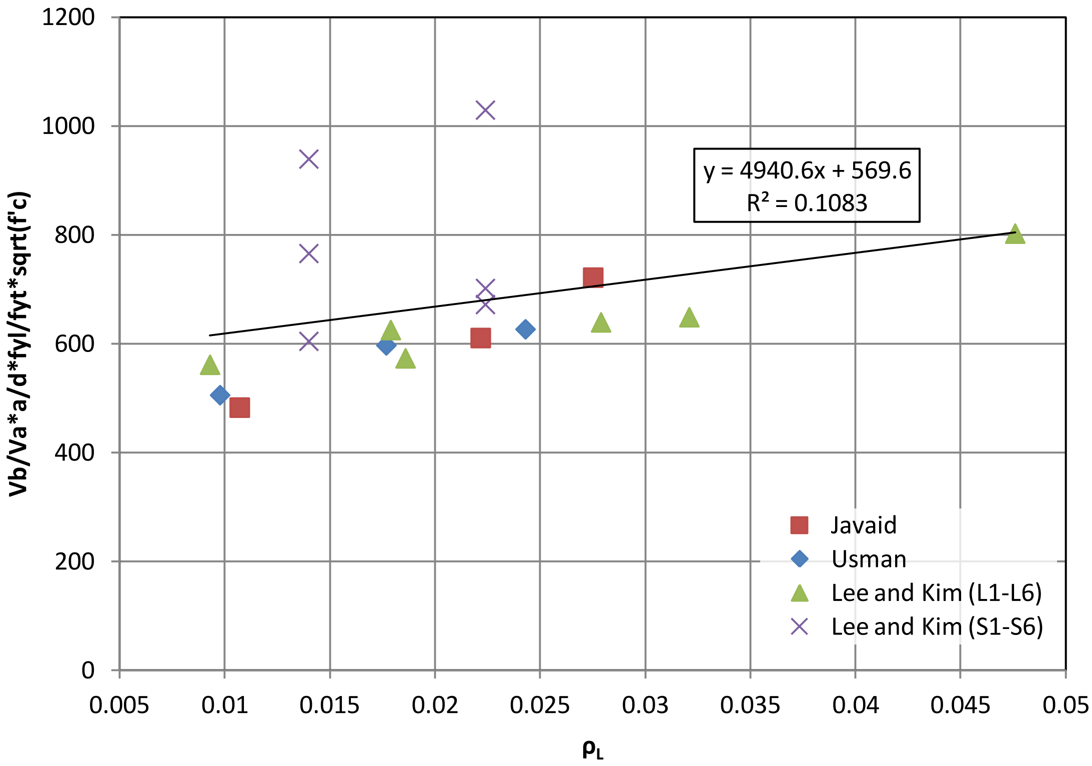


Fig 5.22 Reserve Strength vs Longitudinal Tensile Reinforcement Ratio (Effect of Parameters in Combination)

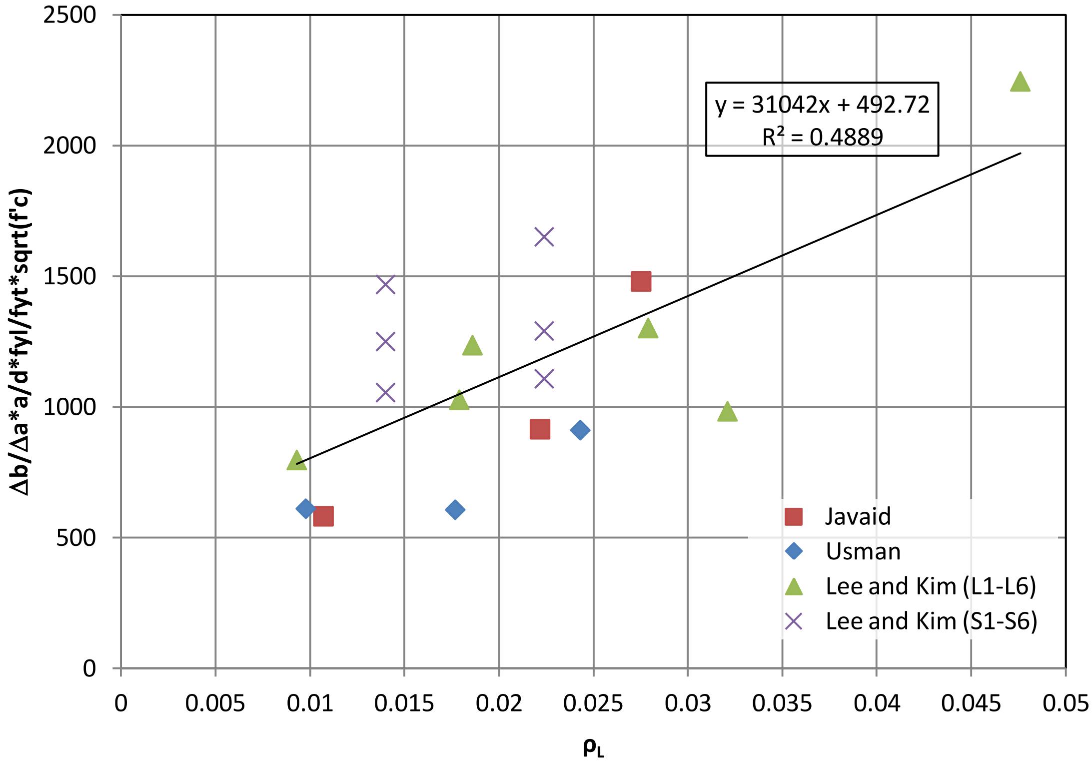


Fig 5.23 Reserve Deflection vs Longitudinal Tensile Reinforcement Ratio (Effect of Parameters in Combination)

REFERENCES

James G Macgregor and James K Wight,"Reinforced Concrete: Mechanics and Design,"4^{th} Edition, Prentice Hall, New Jersey, 2005, 1132 pp.

Joint ACI–ASCE Committee 445, "Recent Approaches to Shear Design of Structural Concrete, ACI 445R-99 (Reapproved 2009)," 55 pp.

Fenwick R.C., and Paulay T., "Mechanisms of Shear Resistance of Concrete Beams," Journal of the Structural Division, ASCE, V.94, No. ST10, October 1968, pp. 2235 – 2350.

Taylor, H. P. J., "Investigation of Forces Carried across Cracks in Reinforced Concrete Beams in Shear by Interlock of Aggregate," TRA 42.447, Cement and Concrete Association, Wexham Springs, UK, 1970, 22 pp.

Kani M. W., Huggins M. W., and Wittkopp, R. R., "Kani on Shear in Reinforced Concrete," Department of Civil Engineering, Toronto, Canada, 1979, 225 pp.

Sherwood E. G., Bentz E. C. and Collins M. P., "Effect of Aggregate Size on Beam-Shear Strength of Thick Slabs," ACI Structural Journal, V. 104, No. 2, March – April 2007, pp. 180 – 191.

Walraven J. C., "Fundamental Analysis of Aggregate Interlock," Journal of the Structural Division, ASCE, V. 107, No. ST11, November 1981, pp. 2245 – 2270.

Michael P. Collins, Evan C. Bentz and Edward G. Sherwood, "Where is Shear Reinforcement Required? Review of Research Results and Design Procedures," ACI Structural Journal, V. 105, No. 5, September – October 2008, pp. 590 – 600.

Prodromos D. Zarais, "Shear Strength and Minimum Shear Reinforcement of Reinforced Concrete Slender Beams," ACI Structural Journal, V. 100, No. 2, March – April 2003, pp. 203 – 214.

Jung-Yoon Lee and Uk-Yeon Kim, "Effect of Longitudinal Tensile Reinforcement Ratio and Shear Span-Depth Ratio on Minimum Shear Reinforcement in Beams," ACI Structural Journal, V. 105, No. 2, March – April 2008, pp. 134 – 144.

ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary," American Concrete Institute, Farmington Hills, MI, 465 pp.

Michael P. Collins and Denis Mitchell, "Prestressed Concrete Structures," Response Publications, Canada, 1997, 766pp.

B. Ch. Jansen and A. Lapko, "On Shear Reinforcement Design of Structural Concrete Beams on the Basis of Theory of Plasticity," Journal of Civil Engineering and Management, V. 15, No. 4, 2009, pp. 395 – 403.

Alexander Placas and Paul E. Regan, "Shear Failure of Reinforced Concrete Beams," ACI Journal, 1971, pp. 763 – 773.

Evan C. Bentz, Frank J. Vecchio and Michael P. Collins, "Simplified Modified Compression Field Theory for Calculating Shear Strength of Reinforced Concrete Elements," ACI Structural Journal, V. 103, No. 4, July –August 2006, pp. 614 – 624.

Aurelio Muttoni and Miguel F. Ruiz, “Shear Strength of Members without Transverse Reinforcement as Function of Critical Shear Crack Width,” ACI Structural Journal, V. 105, No. 2, March – April 2008, pp. 163 – 172.

Theodor Karuthammer, “Minimum Shear Reinforcement Based on Interface Shear Transfer,” ACI Structural Journal, V. 89, No. 1, January – February 1992, pp. 99 – 105.