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# Shear Strength and Behaviour of High Strength Fibrous Reinforced Concrete Deep Beams (HSFRCDB) With Shear Reinforcement

Omar Qarani Aziz*

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## Abstract

*Twelve high strength fibrous reinforced concrete deep beams (HSFRCDB) were tested under two point loading. All the beams were reinforced with main steel reinforcement ratio ($\rho_w$) of about 2.4 percent to avoid flexural failure. The shear span-to-depth ratio (a/d) of the beams was about 2 and the volume fraction of the steel fibre was 0.5 percent. The beams were tested for different vertical stirrup nominal strength ($\rho_v fy$), ranging from 0.15 to 4.44MPa, horizontal stirrup nominal strength ($\rho_h fy$), ranging from 0.4 to 3.63MPa and vertical and horizontal(combination) stirrup nominal strength ($\rho_{vh} fy$), ranging from 2.57 to 8.07MPa in addition to one beam without stirrups.*

*Test results indicate that horizontal stirrups has little influence on the magnitude of the failure load. The test results of this investigation as well as those available in the literature (244 high and normal strength concrete deep beams with stirrups) were compared with predictions based on the (ACI, BS, Canadian and New Zealand Codes) and works of other investigators (Zustty, Siao and Sarsam) and a Proposed Equation. The ACI, BS and Saio Equations are simultaneously conservative with relatively low COV values. The Proposed Equation has the lowest standard deviation, COV and conservative predictions.*

## سلوك و مقاومة القص للعتبات الخرسانية ذات مقاومة عالية و المعززة بالالياف الفولاذية و تسليح القص

## الخلاصة

فى هذا البحث تم صب و فحص 12 عتبة خرسانية عميقة و المعززة بالالياف الفولاذية وتسليح القص جميع العتبات لها نفس نسبة التسليح الطولى تقريبا " و هذه النسبة كانت % 2,4 و ذلك لتفادى فشل الانحناء نسبة فضاء القص الى

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* Assistant Prof., BSc. Civil Engineering, M. Sc. and Ph.D. Structural Eng.
**Civil Eng. Dept., College of Eng., University of Salahaddin - Arbil, Iraq.

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University of Technology-Iraq, Baghdad, Iraq/2412-0758

العمق للنماذج كانت بحدود 2 و كمية الالياف الفولاذية 0,5%. تم فحص النماذج لاربعة قيم لمقاومة القص العمودى وكانت تتراوح من ( 4.44 - 0.15) ميكا باسكال و ثلاثة قيم لمقاومة القص الأفقى و كانت تتراوح من ( 3,63 - 0.40)ميكا باسكال وأربعة قيم لمقاومة القص العمودية و ألافقى معا "حيث كانت المقاومة تتراوح من 2.57الى 8.07ميكا باسكال بالاضافة الى نموذج بدون حديد تسليح القص .

أظهرت النتائج بان حديد تسليح القص ألافقى لها تأثير أقل من تأثير حديد تسليح القص العمودى تم مقارنة النتائج العملية لهذا البحث و نتائج البحوث السابقة (244 نموذج ذات مقاومة خرسانية عالية و أعتيادية مع تسليح القص) مع نتائج المعادلات الموجودة فى المواصفات الامريكية ,البريطانية و الكندية والنيوزلاندية وأيضا مع نتائج معادلات المقترحة من قبل باحثين Siao, Zustty و Sarsam و المعادلة المقترحة فى هذا البحث. كانت نتائج معادلات المواصفات ألامريكية البريطانية و Siao أكثر أمانا "مع قيمة واطئة Cov بينما المعادلة المقترحة فى هذا البحث اعطت أقل القيم ل Cov و الانحراف المعيارى مع نسبة امان مقبولة.

**Key words**: concrete, deep beams, high strength, reinforced concrete, shear stress, shear span, stirrups , steel fibres.

## INTRODUCTION

Reinforced concrete deep beams appear as common structural elements in many structures from tall buildings to offshore gravity structures. They are used as panel beams and, more recently, as deep grid walls in offshore gravity-type concrete structures [1] . The term **deep beams** is applied to any beam which has a depth to span ratio great enough to cause non-linearity in the elastic flexural stresses over the beam depth and the distribution of shear stress to be non-parabolic [2] . The combination of stresses (bending and shear) in the shear span results inclined cracks which transform the beam into a tied-arch [3] . Final failure occurs in different modes, depending on the beam properties, such as:

1- Flexure failure, either of steel reinforcement due to yielding or fracture or compression failure of concrete.

2- Shear failure by crushing of concrete in the region between the support and the load.

3- Anchorage bond failure of the tension reinforcement in the support zones.

4- Bearing failure near the reactions or under the loads.

The strength of deep beams is usually controlled by shear, rather than flexure, provided that normal amounts of longitudinal reinforcement are used. On the other hand, shear strength of deep beams is significantly greater than that predicted using expressions developed for slender beams, because of their capacity to redistribute internal forces before failure and develop mechanisms of force transfer quite different from beams of normal proportions [4]. Beam clear span to depth ratio is an important variable in reinforced concrete deep beams. Different limits for span to depth ratio were given in codes of practice[5,6,7]. Number of researchers[8] presented relations between ultimate shear stress and shear span to depth ratio( a/d ) , which shows the different types of failure that occur in different regions of the beam corresponding to the values of the shear span to depth ratio ( a/d ) . When a/d is less than 2.5 the ultimate shear strength significantly exceeds the inclined cracking strength, while is approximately equal to the cracking strength for a/d greater than 2.5[9,10] for beams without stirrups. Some codes of practice assumed that the nominal shear strength provided by the concrete section is equal to the shear causing inclined cracking. It was assumed that the shear strength is proportional to the concrete compressive strength raised to a certain power. This assumption was based on the relation between the diagonal cracking load and the tensile strength of concrete. Recent studies[11,12] have shown that small amounts of web reinforcement have a significant effect on the shear strength more than that predicted by the modified truss analogy. The horizontal projection of the inclined cracks tend to decrease as the web reinforcement ratio increases, so that the increase in the number of stirrups crossed cracks is less than the increase in shear reinforcement. Web reinforcement near the bottom of diagonal cracks is very effective in preventing dowel splitting cracks and increase the bond strength by providing confinement. Ahmed, Khaloo and Poveda [13] concluded that the ACI Eqn. (11-3) is conservative for beams with low a/d ratios.

Narayanan and Darwish[14] carried out an experimental study on reinforced concrete deep beams with steel fibres and with different values of a/d, they concluded that the inclusion of steel fibres in concrete deep beams lead to stiffness enhancement and increased spall resistance at all stages of loading up to failure, while reduced crack widths. Mansur and Ong[15] found that the addition of discrete steel fibre in the concrete mix lead to better crack control and enhances the strength and deformation characteristics of deep beams containing conventional reinforcement.

## RESEARCH SIGNIFICANCE

The main objectives of this investigation are:

1. To study the effect of vertical and horizontal shear reinforcement on the shear strength behaviour of high strength fibrous reinforced concrete deep beams (HSFRCDB).
2. To obtain more representative and economic shear design equation, for high and normal strength reinforced concrete deep beams(with and without steel fibres) from the tests carried out in this investigation and those available in the literature.
3. To detect the applicability of previous methods (given by codes of practice and other researchers) for predicting ultimate shear strength of normal and high strength reinforced fibrous concrete deep beams with shear reinforcement.

## EXPERIMENTAL PROGRAM

### Specimen details and materials

Twelve reinforced concrete deep beams were tested under two symmetrically placed concentrated loads. Each beam was 1250 mm long with an overall cross-section of 100x200 mm. All test specimens were simply supported over a span of 1000mm. The tested beams were divided into three series. Table 1 and Fig.1 give the properties and the details of the tested specimens. All the specimens were designed to fail in shear and had a/d of about 2, main steel reinforcement ratio ($\rho_w$ )of about 0.024 and volume fraction of steel fibres ($V_f$ ) of 0.5%.

The five beams in series one (namely, $B\rho_{vh0}$ to $B\rho_{v4}$ ) were provided with varying amount of vertical shear reinforcement ($\rho_v fy$ = 0.00 to 4.44MPa). The three beams tested in series two (namely $B\rho_{h1}$ to $B\rho_{h3}$) had a varying horizontal shear reinforcement ($\rho_h fy$ = 0.40 to 3.63MPa). Series three (namely, $B\rho_{vh1}$ to $B\rho_{vh4}$ ) had a varying vertical and horizontal shear reinforcement (combination between them).

Ordinary Portland cement, 12.5 mm maximum size of coarse aggregate, sand of 2.64 fineness modulus and mix proportion of about ( 1: 1.24 : 1.88 ), ( cement: sand : gravel), with w/c ratio of 0.3 were used throughout tests to obtain concrete with compressive strength greater than 41.4 MPa(HSC). Locally available melamine Plastisizer (Type F) was used conforming ASTM $C_{494-86}$ specifications. Deformed steel fibres, 0.4mm in diameter, having an aspect ratio of 100 and ultimate strength of about 1150 MPa were used. Deformed steel bars with 16 mm diameter and yield strength of about 416 MPa were used to provide the main tensile reinforcement. Each beam was reinforced with two bars and hooked at ends as shown in Fig.1 to insure end anchorage.The amount of reinforcement in each beam corresponded a value of $\rho_w$ =0.024. Plain steel bars with diameters of 3, 4 and 6mm with yield strength (fy) of about 350, 562 and 400MPa respectively and deformed steel bar with diameter of 10mm and yield strength of 480MPa were used as stirrups.

## Fabrication

A rotary mixer of 0.80m$^3$ capacity was used. Initially the fine and coarse aggregate were poured in the mixer, followed by 25% of the mixing water(water and admixture) to wet them; afterwords the cement was added and the material were mixed until a uniform colour was obtained. Finally the remaining water was added gradually to the mix, The steel fibres were introduced last and the mixing operation was continued until homogenous concrete was obtained.

## Testing

The specimens were simply supported and tested under two symmetrical point loads (using universal testing machine, type Marue Mie, maximum capacity of 100 ton, No. 19258-Japan). Loads and reactions were applied through rollers and bearing blocks to allow free rotation and horizontal movement of the end supports. Deflections were measured at centre of span using dial gauge of 0.01mm accuracy with a maximum travel of 30mm.

An incremental stage loading was applied in order to obtain a continuous view of the performance of each beam. The deflection was recorded at each load stage and a search was made for cracks and their extensions. First cracking load was recorded and the loading was continued until failure. The failure load was recorded and finally some photographs were taken to show the crack patterns.

## RESULTS OF THE TESTED SPECIMENS AND DISCUSSION

Test results of twelve high strength fibrous concrete deep beams and their crack patterns are included to study the effect of vertical and horizontal shear reinforcement on the ultimate shear stress and behaviour of such beams.

### Crack Patterns and Modes of Failure

Cracks in the concrete beams are formed generally in regions where tensile stresses exist and exceed the specified tensile strength of concrete. Two types of cracks were observed in the tested beams; the flexural cracks which resulted due to flexural tensile stresses in the region of the beam cross-section below the neutral axis for positive bending and shear cracks which are formed as a result of the inclined or "principal" tensile stresses acting on the web of the beam in the region of combined bending and shear. Typical crack patterns of the tested beams are shown in Fig. 2. The beams were failed in shear – compression or shear tension according to the following sequence:

1- Vertical shear-flexural cracks formed at the shear span.

2- The crack propagation continued but in a curved path towards the point load, approaching the compression zone.

3- As the load increased, the cracks extended in two directions; the first towards the compression zone and the second followed a horizontal path at the reinforcement level towards the supports.

4- Crack propagation continued until it reached the point load region, after which the beam carried further loads without much cracking. Finally the crack extended in the compression zone towards the pure moment region and beyond the point load or extended in the tension zone towards the supports causing failure.

### Load Deflection Relationship

At the early stages of loading, the beams behaved in an elastic manner up to about (60 – 80) percent of the ultimate load depending on the amount of shear reinforcement as shown in the Figs.3, 4 and 5. Inelastic stages then followed by increasing deformation until the ultimate load was reached. The curves indicate improvement in beam ductility with an increase in the amount of shear reinforcement.

In all specimens diagonal shear cracks were observed first at or near the support. They were initiated along a line joining the loading and reaction points. All the beams developed such cracks and behaved essentially as tied arches until collapse. Effect of vertical and horizontal shear reinforcement on the shear stress as shown in Figs.6, 7 and 8. As shown by increasing $\rho_v fy$ from 0.15 to 4.44Mpa, the shear stress increased by 29%, by increasing $\rho_h fy$ from 0.40 to 3.63Mpa, the shear stress increased by 13.9%, by increasing $\rho_v fy+\rho_h fy$ from 2.57 to 8.1MPa (combined effect), the shear stress increased by 18.8%. It is also clear that the vertical stirrups are more effective than the horizontal stirrups.

### Cracking and Ultimate Shear Strsses

Cracking shear strength ($v_{cr}$) or diagonal cracking strength is defined here as the shear strength at which an inclined crack was formed within the shear span traversing the centroidal axis of the beam. As shown in Table 1 and Figs. 6, 7 and 8, the shear stresses were observed to increase almost linearly with increase in the amount of shear reinforcement. The increase in the ultimate shear stress is attributed to the types of shear transfer mechanisms.

## EVALUATION OF THE EXPERIMENTAL RESULTS

### Shear design equations

Many design equations were proposed[5,6,7,8,10,21,22,23,32,36,37,38,3] to predict shear strength. Predictions by ACI Code[5] method (Equation 1) and British Standard Code[7] take into account different parameters affecting the shear strength (Equation 2). The Canadian Code method[37], material reduction factor is significantly different for steel and concrete ( Equation 3). Newzealand Code method[38] has Equation 4 to apply test results to this approach.

Zsutty's method[21] (Equation 5) is also well known and was recommended for further study by the ACI-ASCE committee 426. Siao[22] and Sarsam[23] Equations 6 and 7 are simple and show results close to the Zsutty Equation.

Finally the recently published design equation[39] will be modified and applied to the 255 deep beams with stirrups. To compare between the design methods the ultimate shear stress ($v_u$ ) will be used instead of nominal shear stress ($v_n$ )..

### a- ACI Code Method[5]

$$vu = 0.85[vc + vs] \qquad (1a)$$

$$v_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left[\left(\sqrt{fc'} + 120\rho_w\frac{V_u d}{M_c}\right)/7\right] \qquad (1b)$$

$$v_s = \left[\frac{A_v}{s}\left(\frac{1+1/d}{12}\right) + \frac{A_h}{s_2}\left(\frac{11-1/d}{12}\right)\right]f_y d/bs \qquad (1c)$$

### b- British Standard Method[7]

$$v_u = v_c + v_s \qquad (2a)$$

$$v_c = \left[0.79\left(\frac{100A_s}{bd}\right)^{0.333} * \left(\frac{400}{d}\right)^{0.25} * \left(\frac{f_{cu}}{25}\right)^{0.333} * \frac{1}{1.25} * \frac{2d}{a}\right] \qquad (2b)$$

$$v_s = \frac{0.87 f_{yv} A_v}{bs}.\frac{d}{s} \qquad (2c)$$

### c- Canadian Code Method[37]

$$v_u = v_c + v_s \qquad (3a)$$

$$v_c = 0.6\left[0.2\sqrt{fc'}\right] \qquad (3b)$$

$$v_s = 0.85\frac{A_v}{bs} f_{yv} d/s \qquad (3c)$$

### d- New Zealand Code Method[38]

$$v_u = v_c + v_s \qquad (4a)$$

$$v_c = (0.07 + 10\rho_w)\sqrt{fc'}\frac{2d}{a} \qquad (4b)$$

$$v_s = \frac{A_v}{bs} f_y\left(\frac{a}{d} - \frac{1}{2}\right) + \frac{A_h}{bs_2} f_y\left(\frac{3}{2} - \frac{a}{d}\right), a/d < 1.5 \qquad (4c)$$

### e- Zsutty Method[21]

$$v_u = v_c + v_s \qquad (5a)$$

$$v_c = 2.51\left(fc'\rho_w/(a/d)\right)^{0.33} \quad (5b)$$

$$v_s = \rho_v f_y d/s \quad (5c)$$

### f- Siao Method[22]

$$v_u = 1.05\sqrt{\phantom{x}}fc'\left\{1+\frac{Es}{Ec}\begin{pmatrix}\rho_h \sin^2\theta \\ +\rho_v \cos^2\theta\end{pmatrix}\right\} \quad (6)$$

### g- Sarsam Method[23]

$$v_u = 1.85\left(fc'\rho_w d/a\right)^{0.38} * 2.5\frac{d}{a} + \rho_v f_y d/s \quad (7)$$

### h- Proposed Method

$$v_u = 0.85\left[v_c + v_s\right] \quad (8a)$$

$$v_c = 1.51\left[\left(fc'\rho_w(1+F)bd\right)/(1a)\right]^{0.46} \quad (8b)$$

$$v_s = \rho_v f_y + \rho_h f_y \quad (8c)$$

## Comparison between the design methods

Tables 2 and 3 compare the eight design methods for 61 high strength concrete deep beams (HSC) and all 255 deep beams (high and normal strength, with and without steel fibres), respectively. Considering a minimum ratio of ($v_{u\ exp.}/v_{u\ pred.}$) equal or greater than one as a measure of conservatism: there is no design equation qualify all tests in both tables. Only the ACI Code, British Standard, Siao and Sarsam methods are simultaneously conservative with relatively low COV.

As a measure of shear capacity representation, the lowest COV value and mean are with Equation 8, it is, the proposed method. For HSC, the respective value of COV value is 39.6 percent and for all the beams is 38.4 percent. It is clear that the proposed Equation has the lowest standard deviation among the eight Equations and the predictions are conservative.

## Influence of major parameters

The same previous 255 beam test results were used to investigate the reasons behind the weak representation of design equations for the shear stress prediction of reinforced concrete deep beams with stirrups. To do this, a series of graphs (Figs. 9 - 13 ) were plotted using the main factors affecting the shear stress ($fc$`,$\rho_w$, l/d, a/d, $\rho_v$ fy+$\rho_h$fy ). as x-axis and the values of $v_{u\ exp.}/v_{u\ pred.}$ as y-axis using the estimation of Eqs.(1, 2, 6 and 8).

## CONCLUSIONS

Based on tests of high strength fibrous concrete deep beams with web reinforcement, the following conclusions are made:

1. For the same cross-section, l/d, a/d and main reinforcement, the ultimate shear stress improved by about 29% by increasing the vertical stirrup nominal shear strength from 0.15 to 4.44MPa.
2. For the same cross-section, l/d, a/d and main reinforcement, the ultimate shear stress improved by about 13% by increasing the horizontal stirrup nominal shear strength from 0.40 to 3.63MPa.
3. For the same cross-section, l/d, a/d and main reinforcement, the ultimate shear stress improved by about 18.8% by increasing the vertical and horizontal stirrup nominal shear strength from 2.57 to 8.07MPa.
4. The proposed equation was compared with those of codes of practice and those proposed by other researchers and showed a lower value of mean, COV and standard deviation.
5. ACI Code method, British Standard and Siao methods are simultaneously conservative with relatively low COV.

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## *NOTATION*

a : Shear span, distance between concentrated load and face of support, mm.

a/d : Shear span to depth ratio.

$A_s$ : Area of tension reinforcement, $mm^2$.

b : Width of the beam , mm.

d : Effective depth of the beam, mm.

$f_{cu}$ : Compressive strength of concrete based on BS 8110 specifications, MPa.

$f_{c'}$ : Compressive strength of concrete based on ASTM specifications, MPa.

$f_y$ : Yeild strength of steel reinforcement, MPa.

F : Fiber factor equal to $l_f / d_f\ V_f\ \beta$

h : Overall depth of the beam , mm

l : Clear span of the beam ,mm

$l/d$ : Clear span to effective depth ratio.

$l_f/d_f$ : Aspect ratio of steel fibers

$M_u$ : Ultimate moment of the section, kN. m.

$v_u$ : Ultimate shear stress of reinforced concrete beams, MPa.

$\beta$ : Bond factor equal to 0.75 for deformed steel fiber.

$\rho_w$ : Reinforcement ratio of the main steel.

$\rho_v fy$ , $\rho_h fy$ : Shear stress of vertical and horizontal stirrups respectively.

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**Table 1- Details of the Tested Specimens**

| Beam designation | B mm | d mm | l/d | a/d | $\rho_w$ % | $f_c'$ MPa | $\rho_v$ fy MPa | $\rho_h$ fy MPa | $v_{cr}$ Mpa | $v_u$ MPa |
|---|---|---|---|---|---|---|---|---|---|---|
| Series1<br>1.Bρvho | 111 | 177 | 5.65 | 1.92 | 2.04 | 49.35 | 0.00 | 0.00 | 3.00 | 5.37 |
| 2.Bρv1 | 100 | 177 | 5.65 | 1.92 | 2.27 | 58.81 | 0.15 | 0.00 | 3.06 | 5.97 |
| 3.Bρv2 | 106 | 167 | 5.99 | 2.04 | 2.27 | 53.90 | 0.28 | 0.00 | 3.32 | 6.65 |
| 4.Bρv3 | 102 | 167 | 5.99 | 2.04 | 2.36 | 60.92 | 0.88 | 0.00 | 3.45 | 7.34 |
| 5. Bρv4 | 103 | 172 | 5.81 | 1.98 | 2.27 | 51.70 | 4.44 | 0.00 | 3.75 | 7.72 |
| Series2<br>6. Bρh1 | 103 | 172 | 5.81 | 1.98 | 2.27 | 44.82 | 0.00 | 0.40 | 3.00 | 5.60 |
| 7. Bρh2 | 103 | 172 | 5.81 | 1.98 | 2.27 | 49.01 | 0.00 | 2.29 | 3.07 | 6.15 |
| 8. Bρh3 | 104 | 181 | 5.52 | 1.88 | 2.14 | 53.00 | 0.00 | 3.63 | 3.13 | 6.38 |
| Series3<br>9.Bρvh1 | 100 | 180 | 5.56 | 1.89 | 2.23 | 49.02 | 0.28 | 2.29 | 3.10 | 6.40 |
| 10.Bρvh2 | 105 | 172 | 5.81 | 1.98 | 2.23 | 49.40 | 0.28 | 3.63 | 3.26 | 6.81 |
| 11.Bρvh3 | 110 | 180 | 5.56 | 1.89 | 2.03 | 56.01 | 4.44 | 2.29 | 3.40 | 7.35 |
| 12.Bρvh4 | 107 | 181 | 5.52 | 1.88 | 2.08 | 53.25 | 4.44 | 3.63 | 3.50 | 7.60 |

### Table 2- Comparison between $v_{experimental}$ and $v_{predicted}$ for 61 HSC deep beams

| Ratio | $v_{exp.}$<br>-------<br>$v_{ACI}$ | $v_{exp.}$<br>-------<br>$v_{BS}$ | $v_{exp.}$<br>-------<br>$v_{Canadian}$ | $v_{exp.}$<br>-------<br>$v_{Newzel.}$ | $v_{exp.}$<br>-------<br>$v_{Zusty}$ | $v_{exp.}$<br>-------<br>$v_{Siao}$ | $v_{exp.}$<br>-------<br>$v_{Sarsam}$ | $v_{exp.}$<br>-------<br>$v_{Proposed}$ |
|---|---|---|---|---|---|---|---|---|
| Equation used | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Mean | 2.18 | 1.99 | 2.47 | 1.61 | 2.11 | 2.13 | 1.93 | 1.15 |
| Standard deviation | 3.60 | 3.53 | 3.75 | 3.60 | 3.77 | 3.34 | 3.46 | 2.49 |
| COV. % | 57.23 | 56.15 | 59.60 | 57.25 | 60.00 | 53.22 | 55.00 | 39.60 |
| Range Low | 1.08 | 1.08 | 1.00 | 1.02 | 1.02 | 1.10 | 1.09 | 1.03 |
| High | 8.05 | 9.11 | 10.13 | 7.94 | 8.68 | 4.12 | 4.75 | 2.43 |
| High / Low | 7.45 | 8.44 | 10.13 | 7.78 | 8.50 | 3.75 | 4.36 | 2.36 |
| Number < 1* | 13 | 15 | 10 | 18 | 34 | 2 | 14 | 8 |

* Number < 1 indicates the number of specimens for which $\mathbf{v}_{\mathbf{experimental}}$ $< \mathbf{v}_{\mathbf{predicted}}$

### Table 3- Comparison between $v_{experimental}$ and $v_{predicted}$ for all the 255 deep beams

| Ratio | $v_{exp.}$<br>-------<br>$v_{ACI}$ | $v_{exp.}$<br>-------<br>$v_{BS}$ | $v_{exp.}$<br>-------<br>$v_{Canadian}$ | $v_{exp.}$<br>-------<br>$v_{Newzel.}$ | $v_{exp.}$<br>-------<br>$v_{Zusty}$ | $v_{exp}$<br>-------<br>$v_{Siao}$ | $v_{exp.}$<br>-------<br>$v_{Sarsam}$ | $v_{exp.}$<br>-------<br>$v_{Proposed}$ |
|---|---|---|---|---|---|---|---|---|
| Equation used | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Mean | 2.57 | 2.95 | 3.32 | 2.54 | ٢,٥٤ | 2.37 | 2.18 | 1.15 |
| Standard deviation | 3.17 | 3.18 | 3.29 | 3.34 | 3.41 | 3.10 | 3.10 | 2.03 |
| COV. % | 59.7 | 59.9 | 62.15 | 63.10 | 64.00 | 57.99 | 57.76 | 38.4 |
| Range Low | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.08 | 1.00 | 1.00 |
| High | 9.54 | 13.00 | 16.65 | 12.01 | 8.63 | 3.81 | 7.64 | 4.99 |
| High / Low | 9.54 | 13.00 | 16.65 | 12.01 | 8.63 | 3.53 | 7.64 | 4.99 |
| Number < 1* | 64 | 72 | 54 | 106 | 160 | 3 | 74 | 20 |

* Number < 1 indicates the number of specimens for which $\mathbf{v}_{\mathbf{experimental}}$ $< \mathbf{v}_{\mathbf{predicted}}$

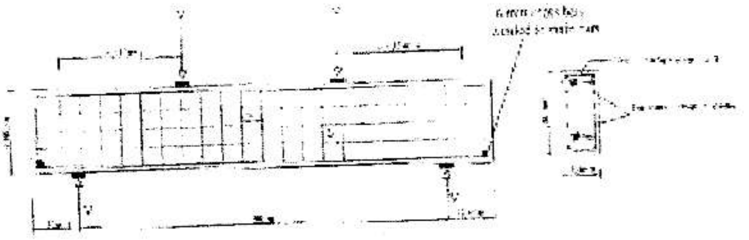

Fig. 1 Specimen Details

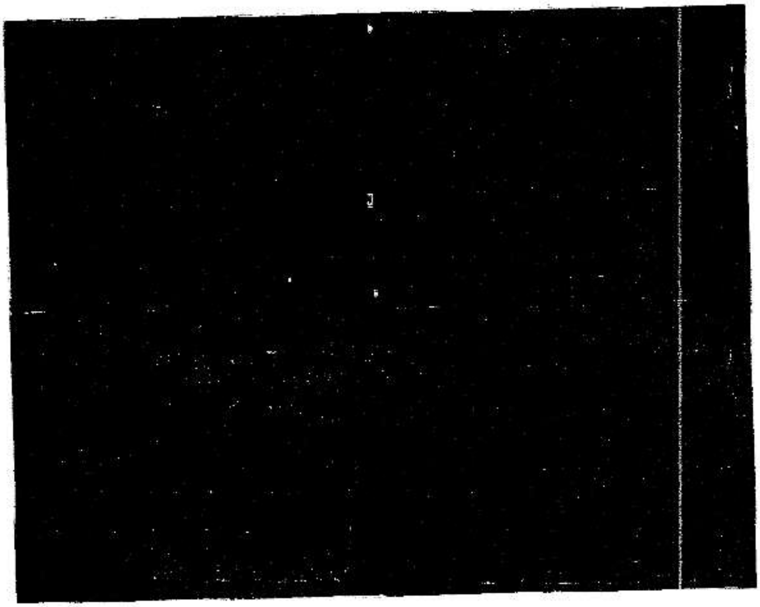

Fig. 2 Crack Patterens of the Tested Beams

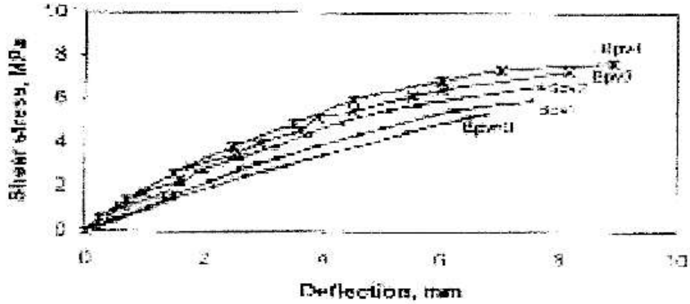

Fig.3 Shear stress - deflection relationship of beams in series one

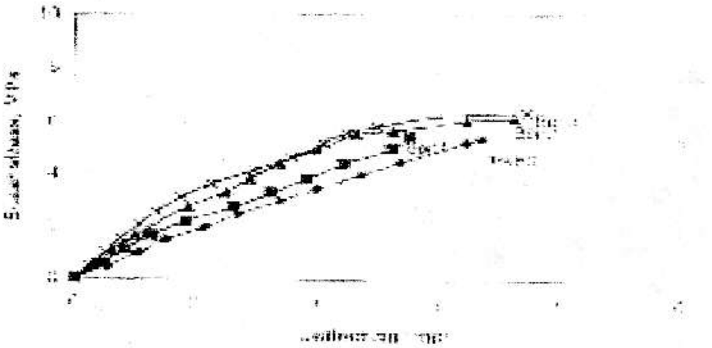

Fig.4 Shear stress - deflection relationship of beams in series two

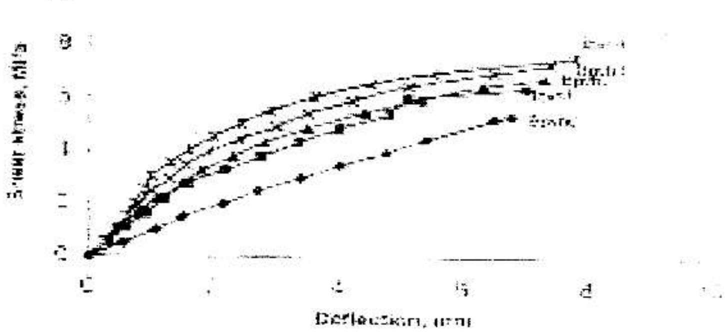

Fig.5 Shear stress - deflection relationship of beams in series three

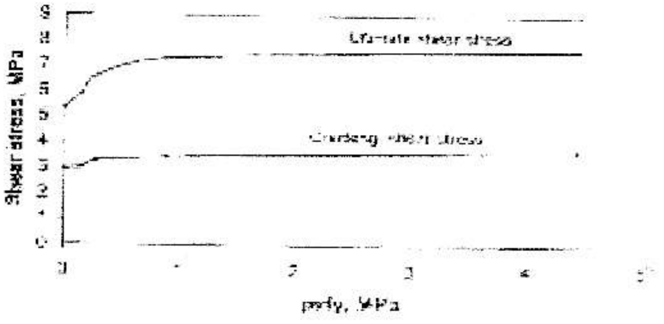

Fig. 6 Shear stress versus normal vertical stirrup shear strength

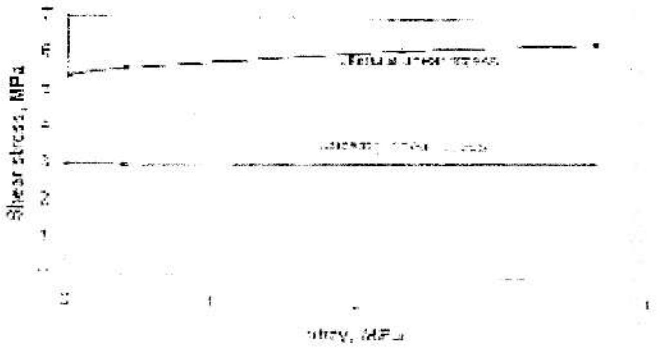

Fig. 7 Shear stress versus normal horizontal stirrup shear strength

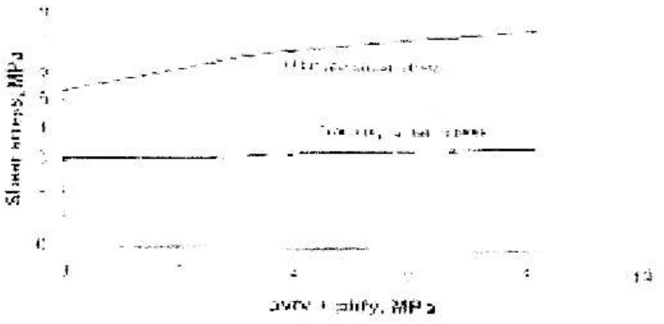

Fig. 8 Shear stress versus combine vertical and horizontal stirrup shear strength

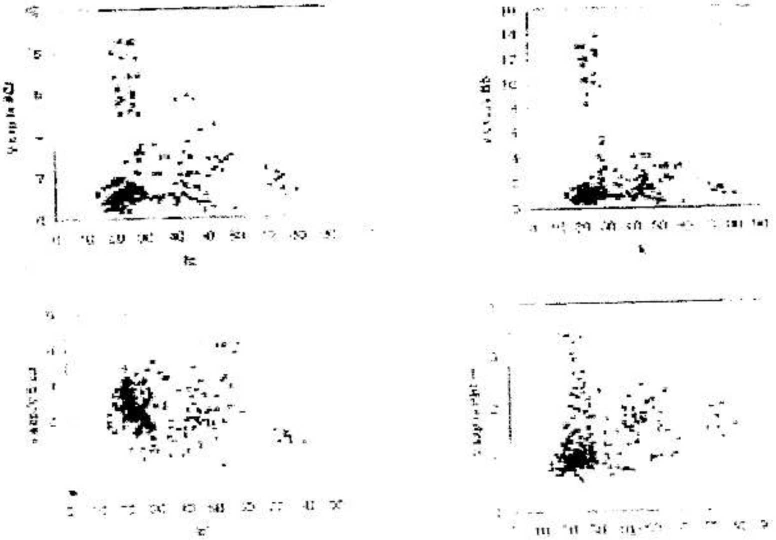

Fig. [illegible] Influence of the [illegible]

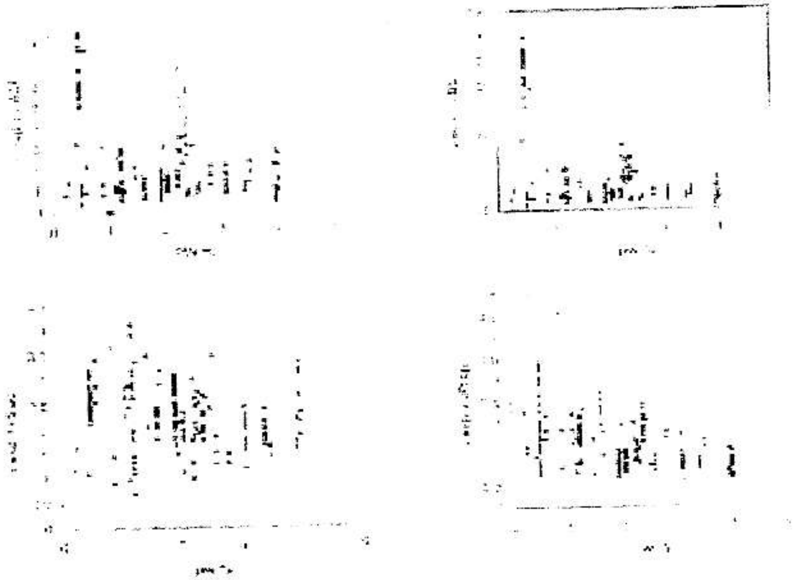

Fig. 10 Influence of longitudinal steel [illegible]

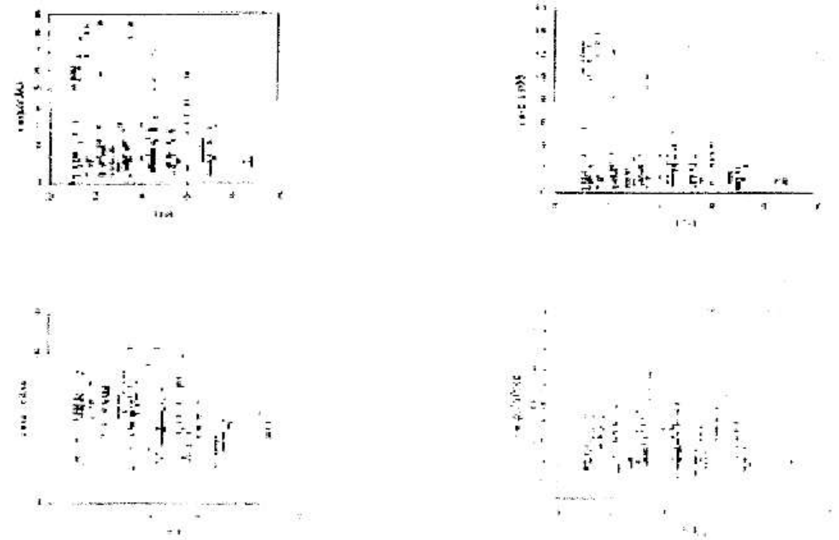

Fig. 11 [illegible]

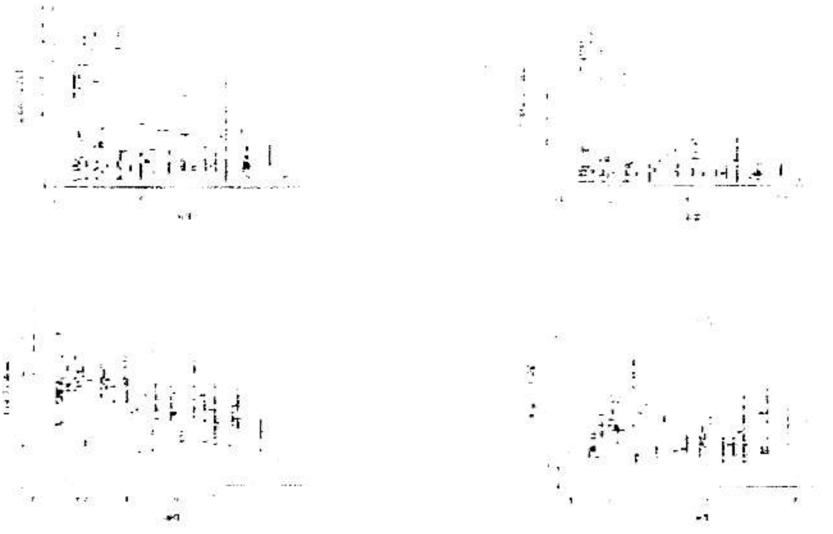

Fig. 12 [illegible]

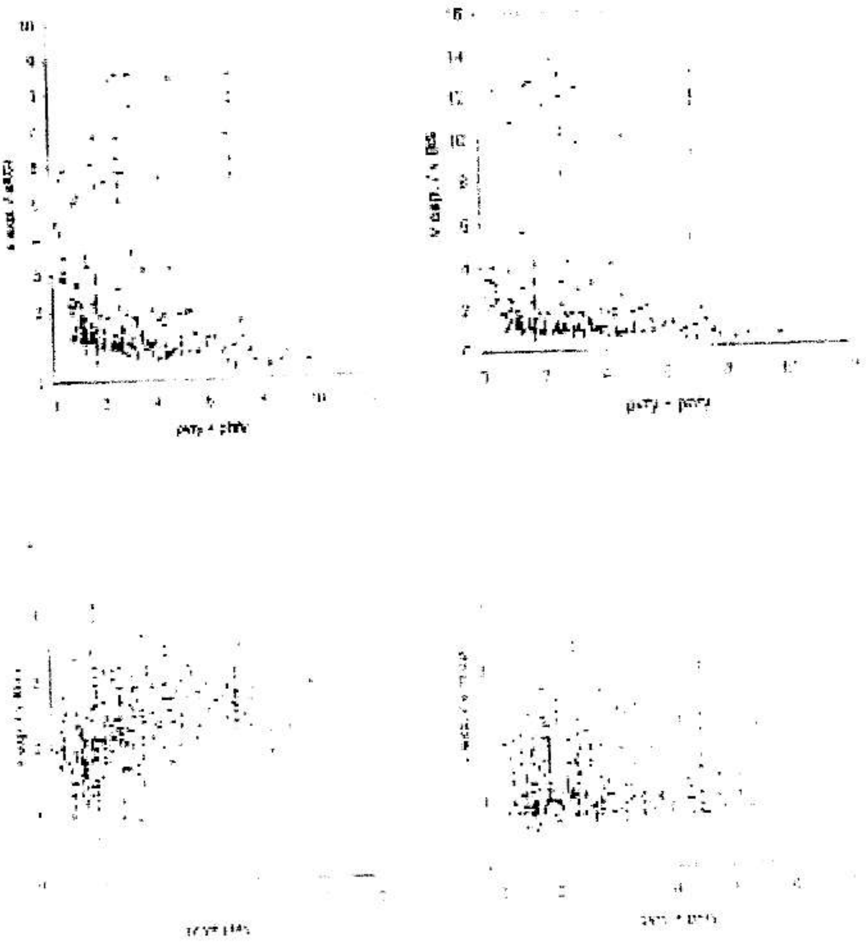

Fig. 11 Influence [illegible]