Experimental Study in Direct Shear Strength of Fiber Reinforced Concrete

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Abstract

Concrete members like brackets, corbels and ledger beams may fail in direct shear. Such failure can be brittle and sudden (without warning). Steel fibers restrain cracking, increase tensile strength and enhance the ductility and energy absorption characteristics.

This study presents an experimental investigation into the behavior of connections subjected to direct shear by testing push-off specimens. The main variables to be studied were fiber percentage, and the shear reinforcement ratio crossing the shear plane at right angles. Measurements such as slip, lateral separation, strains in both the parallel and the shear reinforcements and strain in the concrete were recorded throughout the test.The experimental results show that the fiber reinforcement increases shear strength and this fiber and stirrup reinforcements have an effective role within limited ratios.

Introduction

Transfer of shearing forces in reinforced concrete structural connections is significant in many types of reinforced concrete structures including ordinary and deep beams, corbels, brackets, shear walls , shear diaphragms and deep beams cast in stages (containing construction joints).

The shear transfer problem was discussed in details previously by many researchers . The push-off test specimen, shown in Fig.1, is most widely used to study the influence of direct shear stresses (Esam,2002).

Mattock et al.(1975) modified his previous proposed equation for shear transfer in initially uncracked concrete having reinforcement at right angle to the shear plane. His tests indicated that the hypotheses for shear transfer behavior are applicable to the cases of concrete having reinforcement at any angle to the shear plane, Fig.2. They derived an equation for shear and other for direct stress acting across the shear plane, and compared them with the test results, so the two equations become (Mattock, 1974).

$$v_u = 2.76 + 0.8\left(\rho . f_y + \sigma_{nx}\right) \qquad \ldots(1)$$

and

$$v_u = 2.78\sqrt{\rho . f_y + \sigma_{nx}} \quad \ldots(2)$$

where ρ is the longitudinal steel ratio, f_y the steel yield strength and σ_{nx} is the normal compressive stress applied on the shear plane. The addition of randomly dispersed discrete steel fibers to concrete improves many engineering properties of the material such as tensile strength, flexural strength, fracture toughness, fatigue resistance, impact resistance and crack control mechanism (Bayasi & Soroushian,1992). Swamy and Bahia (1979) studied the shear transfer through the dowel action in fiber reinforced normal weight concrete T-beams. They concluded that the presence of fiber reinforcement delays and controls dowel cracking and thereby improves the stiffness and deformation characteristics of the dowel crack zone. Hsu, *et al* (1987) proposed a theory for the prediction of shear transfer in initially uncracked concrete. Their theory is based on the truss model and incorporates a softened compression stress-strain relation along the concrete struts.The theory predicts that steel reinforcement parallel to the shear plane also contributed to the shear transfer strength, while the shear friction concept in the current design codes recognizes only the contribution of steel reinforcement crossing the shear plane. Workability and placement impose limitations on fiber content to not exceed 2% and aspect ratios not less than 100 in fresh concrete (Hughes & Fattuhi,1976). ACI Code (1999) computes shear strength V_u when shear friction reinforcement is perpendicular to the shear plane, as follows:

$$V_u = (\phi . A_{sh} . f_y + \sigma_{nx})\, \mu \quad \ldots(3)$$

where,

V_u: factored shear strength in the shear transfer plane, which shall not be greater than $0.2 f'_c \, \phi \, A_g$ nor 5.5 $\phi \, Ag$. Here, μ: coefficient of friction (0.6 to 1.4), A_{sh} :area of the shear reinforcement, A_g : gross sectional area of the interface (shear plane) and σ_{nx} : externally applied direct normal compressive stress . Birkeland (1968) proposed a simple expression to evaluate the shear strength

$$v_u = 2.78\sqrt{\rho \, f_y} \quad \text{(in MPa)} \quad \ldots(4)$$

This equation eliminates the conservative disadvantage of the ACI equation. An improved equation is given by Hermansen and Cowmen (1974)

$$v_u = 4.0 + 0.8(\rho . f_y) \quad \ldots(5)$$

where 4 MPa is the value taken for the cohesion stress of concrete.

It is noted that the use of a strength reduction factor φ equal to 0.85 would lead to more conservative shear transfer design.The above equations are merely related to concrete sections without fiber reinforcement. As stated, fiber reinforcement increases the shear strength of concrete sections.

The ultimate shear capacity (or strength) of a fiber reinforced concrete section is consequently of two parts:

$$v_u = v_c + v_f \qquad \ldots(6)$$

where v_c is the shear strength given by concrete and can be estimated by ACI code formula or other formula (1 or 3) while v_{fi} is the increase (gain) in shear strength by the presence of steel fibers. This is given by:

$$v_{fi} = k_f\, f_t\, bd \qquad \ldots(7)$$

where

$$k_f = 0.155\ (\ F\ f_t\ )^{0.36}$$

$$F = l_f/d_f\ v_f\ b_{fi}$$

b_{fi} = fiber factor from 0.9 to 1.2 for deform fibers and 1.0 for some studies (Narayanan & Darwish,1987).

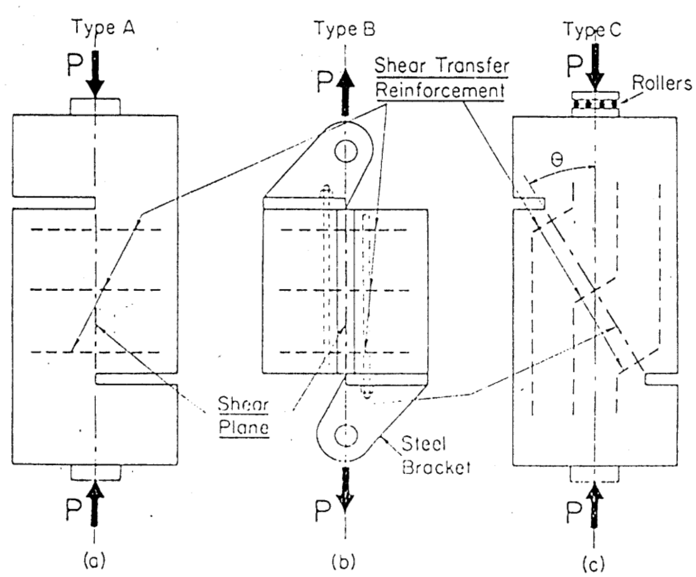

Fig.1: Shear transfer test specimens(a) Push-off,(b) Pull-off, (c) Modified push-off .

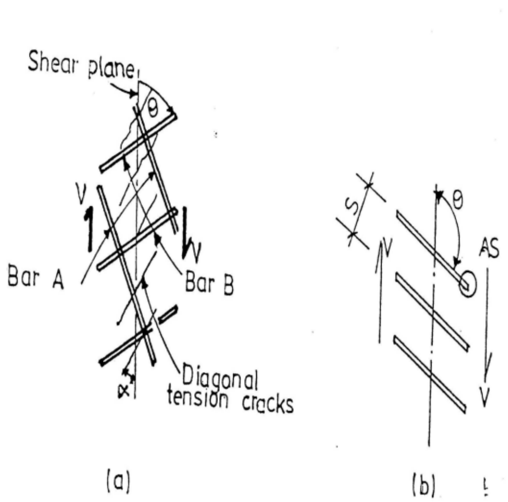

Fig.2: Shear Transfer in Initially Uncracked Concrete with Orthogonal and oblique Parallel Reinforcement (Mattock,1974).

<u>experimental work and test program</u>

General Description and Test Variables:

The main objective of the experimental work is to examine the behavior and failure mode of the push-off specimens under axial concentric loading.

Twelve groups of specimens were cast with and without fibers. Each group consists of three specimens to study the behavior of fiber reinforced concrete in shear transfer. The dimensions of a typical push-off test specimen are suggested according to previous researches.The shear reinforcement and fiber volume fraction for each group of specimens are given in Table 1.

The reinforcement across the shear plane is in the form of closed stirrups, anchored by wrapping round the longitudinal reinforcement.

Table 1: Details of test specimens

V_f%	No. of two leg stirrups	Shear Reinf.	Parallel Reinf.	Actual shear plane dimen. d×b (mm×mm)	Spec. No.
0	0	None	None	132××249	PC1
0.25	0	None	None	129×250	FC2
1	0	None	None	132×249	FC3
0	3	6ϕ8mm	4ϕ12mm	131×250	RC1
0.25	3	6ϕ8mm	4ϕ12mm	133×250	FRC1
1	3	6ϕ8mm	4ϕ12mm	131×250	FRC4
0	3	6ϕ10mm	4ϕ12mm	130×249	RC2
0.25	3	6ϕ10mm	4ϕ12mm	132×250	FRC2
1	3	6ϕ10mm	4ϕ12mm	130×250	FRC5
0	4	8ϕ10mm	4ϕ12mm	128×248	RC3
0.25	4	8ϕ10mm	4ϕ12mm	133×250	FRC3
1	4	8ϕ10mm	4ϕ12mm	130×250	FRC6

Material and Fabrication:

cement: Ordinary Portland cement manufactured by Al-Tameem cement factory conforming to Iraqi Cement Standard (no.5) was used.

Fine aggregate: Natural river sand was used. Fig.3 shows that the fine aggregate conforms with the limits of ASTM C33. The fineness modulus and the bulk specific gravity of the sand were 2.3 and 1.8 respectively.

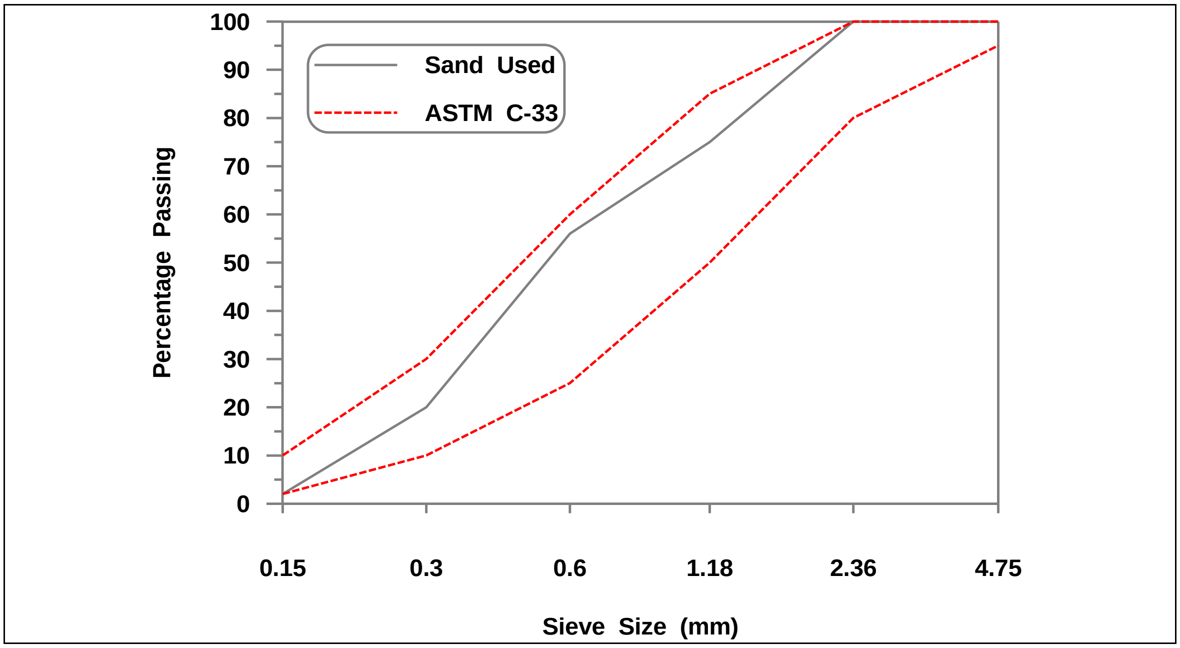

Fig.3: Grading of Fine Aggregate.

Coarse aggregate: Concrete with fiber reinforcement differs from the conventional concrete in having a higher cement content, lower content and smaller size of coarse aggregate (10 mm maximum size). Therefore, natural gravel was used in this work with a maximum coarse aggregate size of 9.5 mm. Fig.4 shows that the coarse aggregate conforms with the limits of ASTM C33. The bulk specific gravity of this aggregate was 1.7.

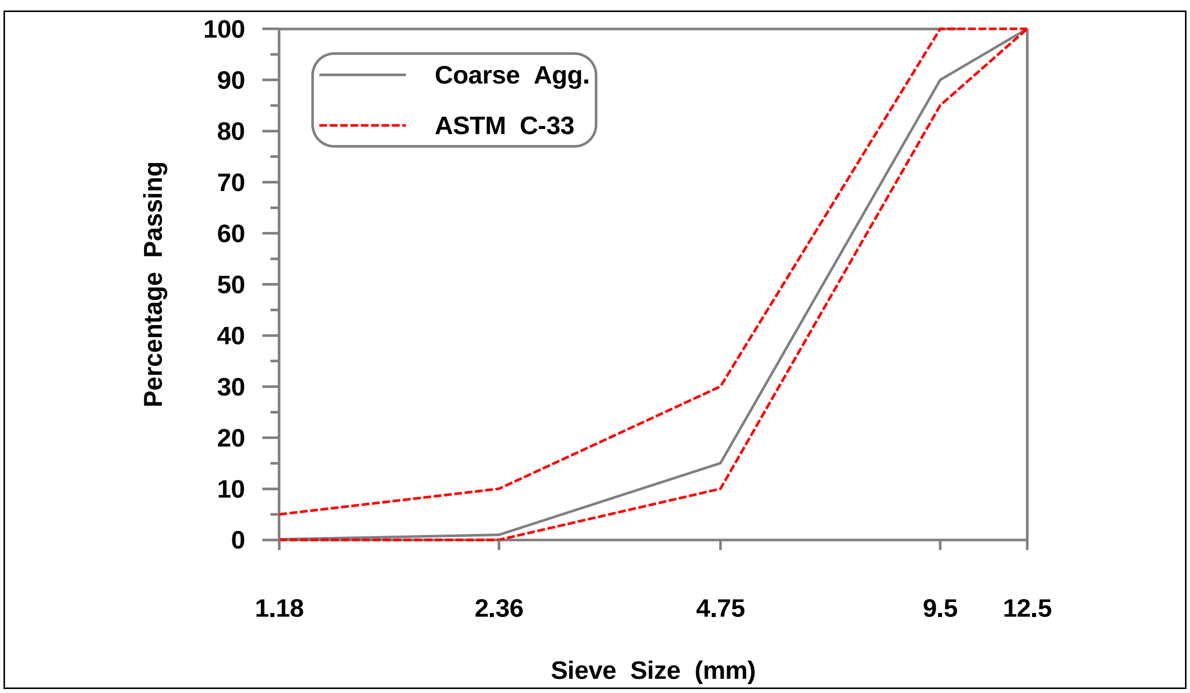

Fig.4: Grading of Course Aggregate.

Steel Fibers: One type of deformed crimped steel fibers of size (0.3×40 mm) with the corresponding aspect ratio (l_f/d_f) equal to 133.3 was used.

Deformed Rebars: Deformed bars were used with nominal diameters of 8, 10 and 12 mm and with yield and ultimate strength f_y equal to 414 MPa and f_u equal to 615 MPa were used.

Each specimen was cast horizontally in one layer. For each specimen, nine cylinders (150×300mm) were cast, three of which were tested to determine the compressive strength f_c', the other three for indirect tensile strength (splitting strength) f_t, and the last three for the modulus of elasticity E_c. Three (150×150×650mm) prisms were cast to be tested for the modulus of rupture f_r. The specimens and control cylinders and prisms were cured in a water bath after 24 hours being remained under polyethylene sheet. They were stored in a water bath until the time of test. The test of the control cylinders and prisms were carried out according to ASTM. The results of the tests are shown in Table 2.

Table 2: Concrete properties

V_f%	**Modulus of elasticity Ec MPa**	**Modulus of rupture (f_r) MPa**	**Splitting tensile strength (f_t) MPa**	**Cylinder strength (f'_c) MPa**	**Spec. No.**
0	20139.5	2.74	2.4	23.5	PC1,RC1,RC2, RC3
0.25	24185.5	3.4	3.0	24.3	FC2, FRC1, FRC2, FRC3
1	25238.5	4.53	3.5	26.25	FC3, FRC4, FRC5, FRC6

Notes: 1- For all specimens, concrete mix was 1: 2: 3 with water cement ratio w/c = 0.55 and slump = 67 mm.
2- Each value in the table represents the average test results of three specimens.

Adequate end reinforcement is provided away from the shear plane to prevent the occurrence of failure in undesired planes. The reinforcing steel was oriented as indicated in Fig.5. A concrete cover of 20mm was used.

In mixing of steel fiber reinforced concrete (SFRC), initially the fine and the coarse aggregates were poured in the mixer, followed by 25% of the mixing water to wet them. Then half of the amount of fiber was incorporated into the mix by hand sprinkling into the rotating drum. The cement was added then, followed by the remaining fibers, followed by 50% of the mixing water. The remaining 25% of the mixing water was added gradually to the mix. The mixer was stopped when a good homogeneous mix was produced.

2ϕ12mm

4ϕ8mm Stirrups

2ϕ12mm

Number and size of stirrups varying, see table (3.1)

2ϕ12mm

4ϕ8mm Stirrups

2ϕ12mm

A

A

8ϕ12mm

Number and size of stirrups varying

250mm

150mm

25mm

25mm

150mm

300 mm

Fig.5: Push-off specimen with shear & parallel reinforcement. Groups RC&FRC.

Test procedure and measurements:

All specimens have been tested at ages 37-40 days. The dial gages were fixed as shown in Fig.6.

Plate 1: Arrangement of test of push-off Specimen

dial gage

zone (1)

zone (3)

zone (3)

zone (4)

shear plane

diagonal tension crack

open slots

Fig.6: Location of demec points used for measuring strain in concrete and dial gages for measuring vertical and horizontal displacements and slip.

The specimens were placed vertically on the lower plate of a compression testing machine and the load was applied from the top through a set of parallel plates. The load was increased by equal increments of 20 kN till failure, with small pauses at each stage to mark any cracks which may have occurred, and to record the required information such as, strains, slip, lateral separation, crack growth pattern ,crack load, and failure load. To measure the horizontal and vertical displacements (or movements) at the shear plane and the slip along the shear plane a small frame with dial gages were used.The strains in the concrete and in both parallel and shear reinforcements were measured by using 0.002 mm mechanical strain gage.

Behavior of shear transfer tests specimens

The general behavior and modes of failure of the shear transfer specimens play an important role in developing any theoretical study of such models. Specimens of group (PC) exhibited the formation of the first diagonal tension crack at about (96.9%) while the specimens of groups (FC) at about (74.8%-79%) of the ultimate shear strength. These cracks were discrete, very short in length and closely spaced.

The observed failure was very brittle and sudden with no warning by breaking into two pieces for group PC (of no fibers) while failure of the group FC with specimens having V_f values of 0.25% and 1.0% occurred when multiple diagonal cracks became crowded forming a crack band along the shear plane, and the fibers which were bridging the cracks were in pullout situation. Failure of such group of fiber reinforced specimens were gradual and visual without sudden failure because of the presence of fibers. For specimens with stirrup reinforcement crossing the shear plane with or without fibers, i.e. groups (RC) and (FRC), the first crack occurred at about (69.3%-69.6%) and (69.5%-70.7%) of the ultimate shear strength respectively. The failure started with the formation of discrete diagonal cracks. These cracks were formed at an angle of about (20-25) degree. to the shear plane, they were about (40-60)mm long, between (30-60)mm apart and the width of cracked region was (20-35)mm. Then these cracks joined together, forming a dominant crack along the shear plane. Failure of the shear reinforced specimens with and without fibers was gradual and visual. Deformations and slippage were increased rapidly prior to failure.

Generally all the specimens failed by direct shear except the specimens with high shear reinforcement ratio (ρf_y=8.12 MPa) with or without steel fibers. This behavior is attributed to the high shear reinforcement across the shear plane and failure of the specimens started by crushing of the concrete under the bearing plate. Results of the experimental work are listed in Table 3 .Also this table shows the ultimate shear strength calculated by equation (6), the results shown a conservative shear transfer design.

Figs.7 & 8 indicate that the ultimate shear stress and the first crack load were increased with increasing (V_f), but the fiber was more effective with group (PC) than with groups (RC) and (FRC).

Fig.9 shows that the strength gain due to increasing the value of (V_f) from

(0%-1.0%) and ρf_y from (0-6.05 MPa) reached a maximum value of 70.7% and 118.13% respectively. Also, it can be seen from these figures that the steel fiber is more effective for plain concrete than for reinforced concrete. Fig.10 shows that the gain in concrete strength properties (f'_c, E, f_t and f_{cr}) as (V_f) was increased from (0%-1.0%) had reached maximum values of 11.7%, 25%, 45.8% and 65% respectively.

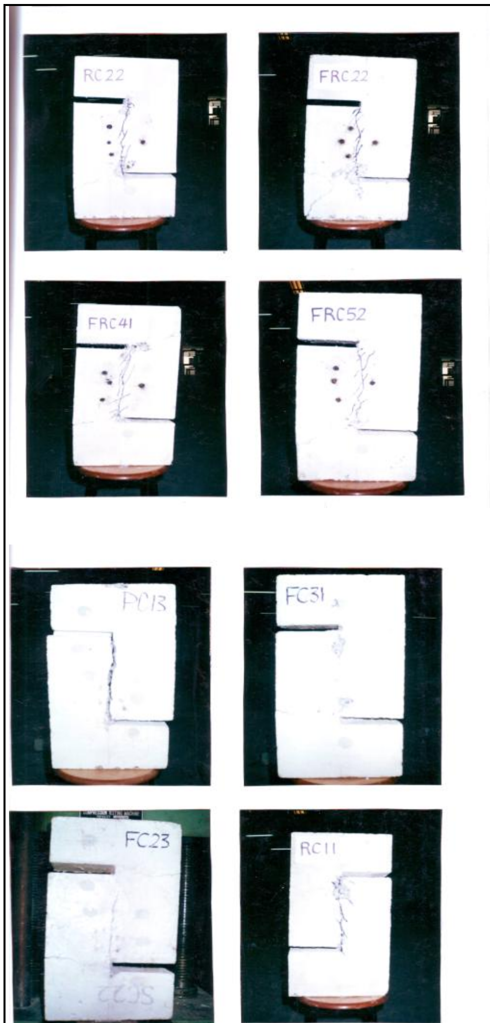

Plate 2: Mode of failure of the specimen

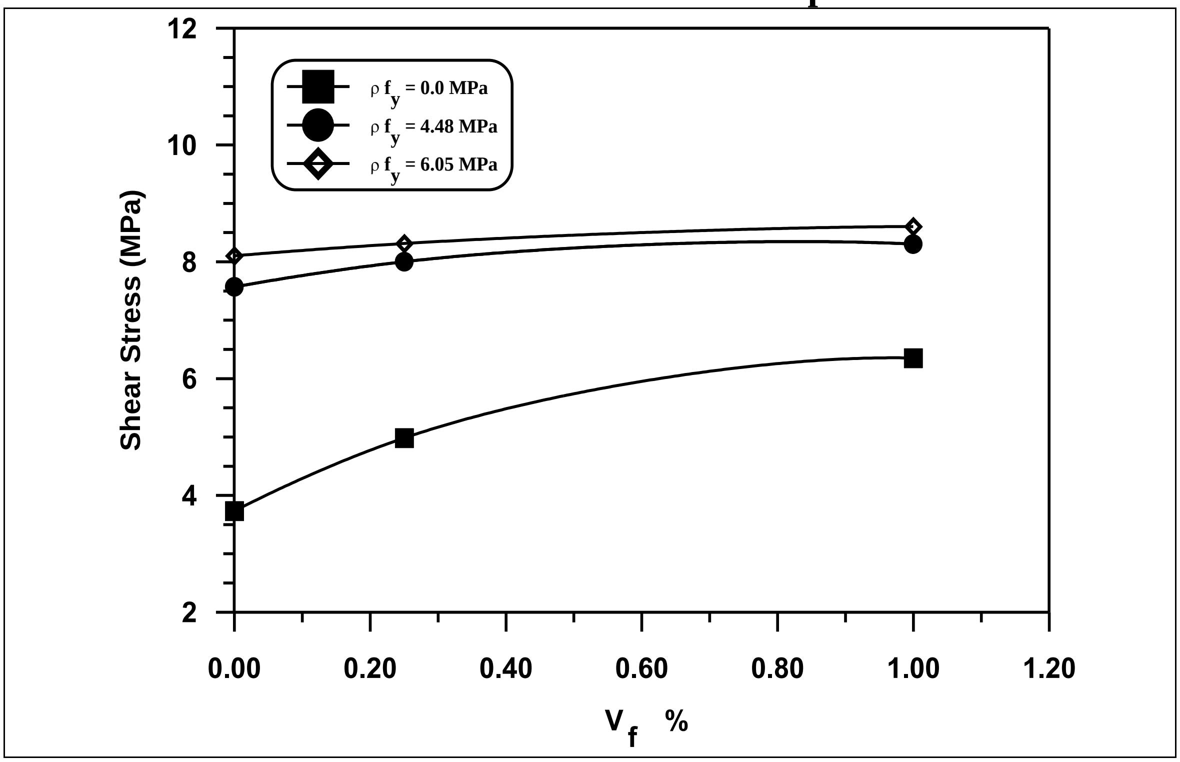

Fig.7: Ultimate Shear Stress Versus V_f for Specimens with Various ρf_y.

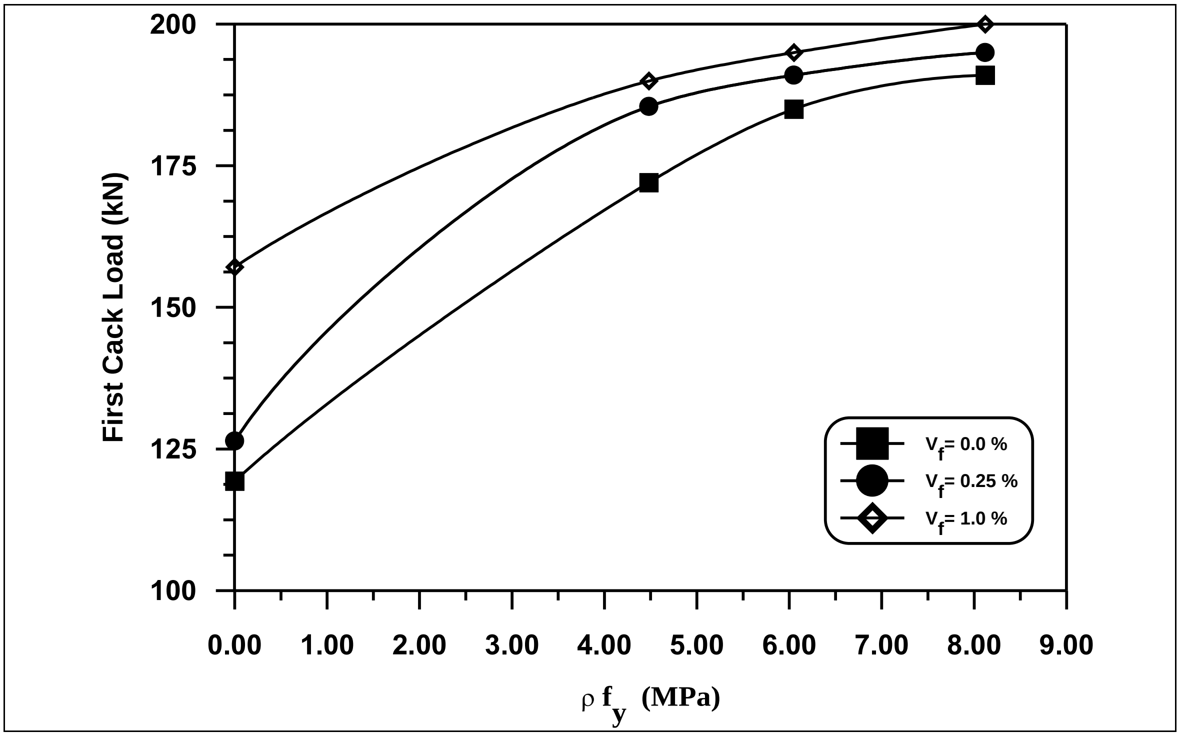

Fig.8: First Crack Load Versus ρf_y for Specimens with Various V_f.

V_f %

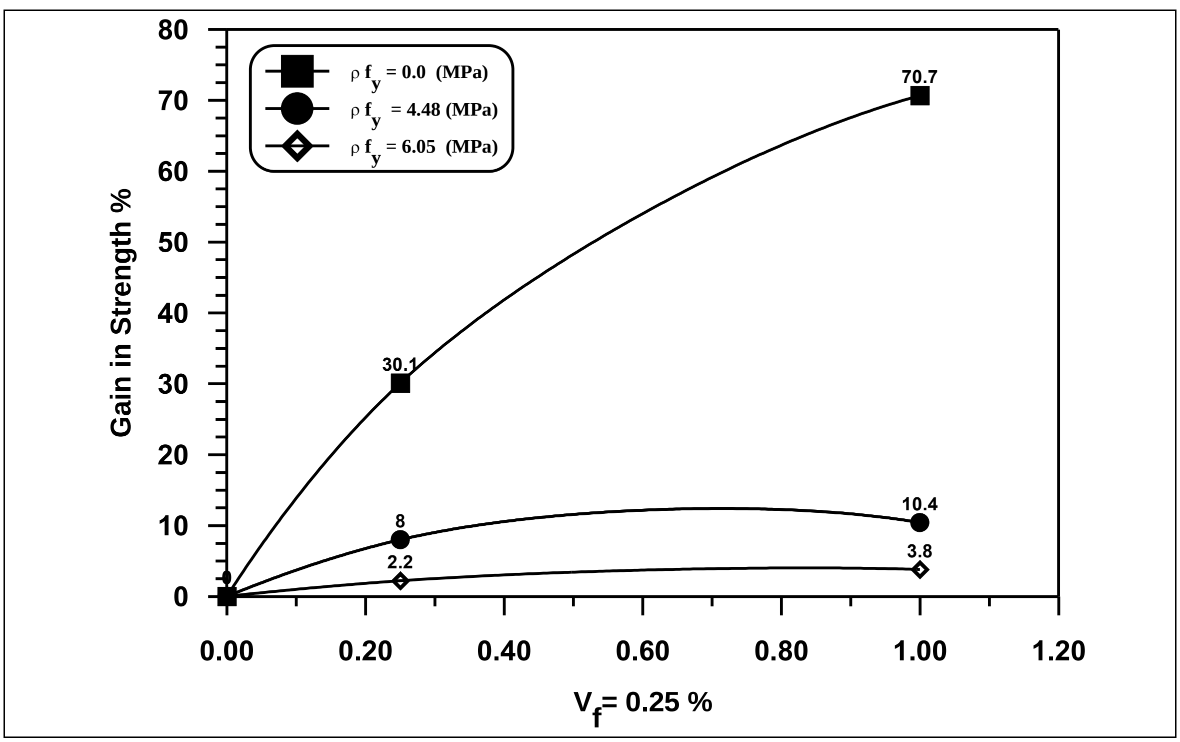

Fig.9: Percentage Gain in Strength Versus V_f for Specimens with Various ρf_y.

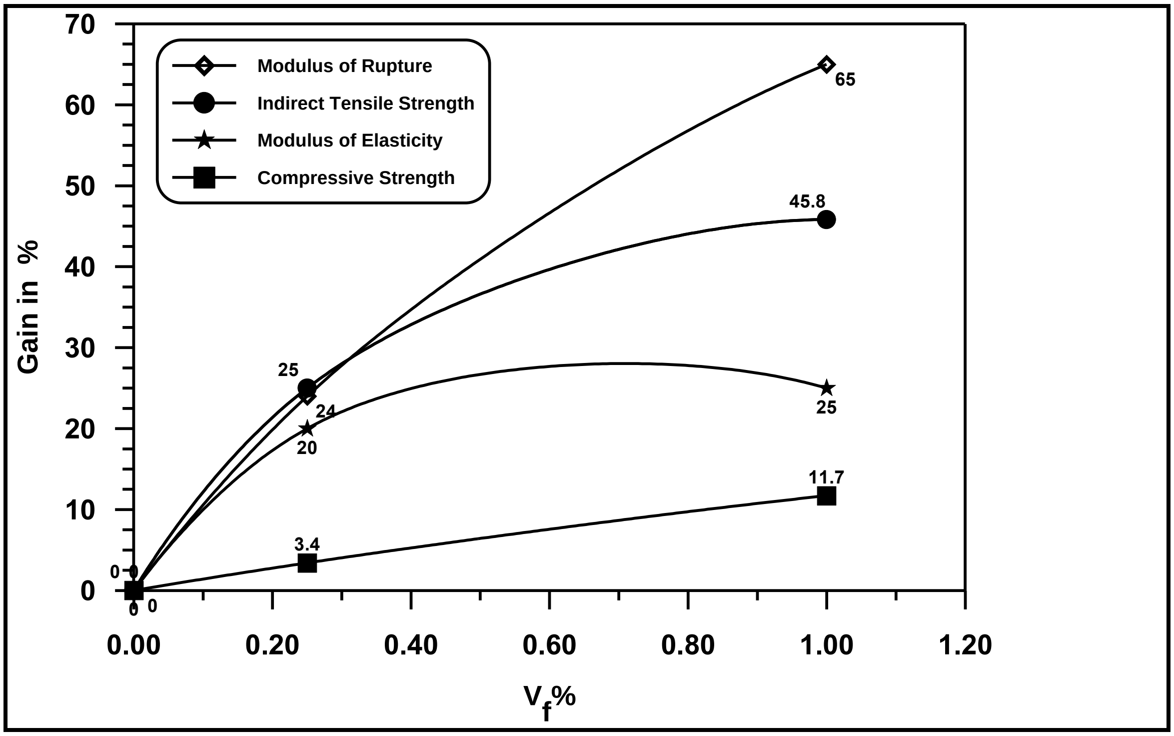

Fig.10: Percentage Gain in Strength Versus V_f for Concrete Properties.

Conclusions

Based on the limited number of test specimens with fiber reinforcement in direct shear, the following conclusions can be stated:

1- Specimens with plain concrete (i.e. group PC) showed brittle failure with no warning before collapse, but specimens with fibers developed first several separated small diagonal cracks that later were joined together and consequently the specimens failed in less brittle manner. On the other hand the failure of specimens with stirrup reinforcement was rather ductile in comparison with groups PC and FC.

2- Increasing the fiber ratio up to 1% and shear reinforcement up to 2% will be effective to gain maximum shear strength.

3- For cylindrical control specimens with fibers, it was noticed that with rising V_f up to 1%, the indirect tensile strength and modulus of elasticity were increased noticeably, while the compressive strength increase was less noticeable.

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Notations

A_g Gross section area in direct shear (the shear plane)
A_{sh} Shear reinforcement area (crossing the shear plane)
$f_c^{'}$ Cylinder compressive strength (of plain concrete)
E_c Modulus of elasticity of concrete
f_r Modulus of rupture (flexural strength) of concrete
f_t Tensile strength of concrete (indirect splitting test)
f_y Yield strength of reinforcement steel bars
V_f Volume fraction of fibers (ratio of volume of fibers to volume of concrete)
l_f/d_f Aspect ratio of fibers
V_u Ultimate (factored) shear force in the shear plane
v_u Ultimate (factored) shear stress in the shear plane
μ Coefficient of friction in concrete shear plane ($\mu = 0.6$ for just cracked plane to 1.4 for uncracked concrete plane)
v_{nx} Direct compressive stress on shear plane
Ø Shear reduction factor (for conservative design Ø = 0.85)

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الخلاصة

تتعرض المقاطع الانشائية في المساند (brackets, corbels) والعتبات الساندة (ledger beams) إلى القص المباشر. قد يكون هذا النوع من الفشل قصيفا ومفاجئا(بدون إنذار). إن وجود الألياف الحديدية في الكونكريت يحد من ظهور التشققات ويزيد من مقاومة الشد ويزيد من خاصيته المطيلية وطاقة امتصاصه.

البحث الحالي يقدم دراسة عملية في تصرف مناطق الاتصال (connections) للأعضاء الكونكريتية المعرضة إلى إجهاد القص المباشر ويتضمن البحث دراسة تأثير التسليح بالالياف الحديدية على مقاومة وتصرف تلك المناطق المتصلة.

المتغيرات التي اعتمدت هي نسبة الألياف الحديدية ونسبة حديد التسليح التي تعبر منطقة القص بشكل عمودي. تم قياس كل من (الانزلاق ، الانفصال الجانبي والانفعال في التسليح الموازي والعمودي لمستوي القص إضافة إلى الانفعال في الكونكريت) اثناء الاختبار.

النتائج العملية تظهر بان التسليح بالألياف تزيد من مقاومة القص، وأن هذه الألياف مع حديد التسليح لهما دور في التأثير ضمن نسب محددة.