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Research Article

Effect of Compressive Strength on Bond Behaviour of Steel Reinforcing Bar in Fiber Reinforced Concrete

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Abstract

The addition of steel fibers in high strength concrete bridges the cracks and stops their propagation. The mode of failure becomes less brittle. The confinement provided by these fibers improves the bond strength. The adequate bond is necessary for composite action of steel reinforcing bars and adjoining concrete. There was a need to study the affect of increase in compressive strength of fiber reinforced high strength concrete, on bond performance. Experimental study was done to determine this effect for short embedded lengths in steel fiber reinforced high strength concrete. In this study the complete experimentation was done first for $3.5d_b$ embedded length, then for $4.0d_b$ embedded length and finally for $4.5d_b$ embedded length. The c/d_b ratio was kept constant. The whole post peak bond behaviour was studied. The results of this experimentation confirmed that by increasing the Compressive strength from 40.0 MPa to 50.0 MPa and 60.0 MPa, the bond strength of deformed steel bar in fiber reinforced high strength concrete increased from 15% to 75%, the corresponding peak load slip increased from 05% to 95%. This increase in bond strength and corresponding peak load slip as a function of increase in compressive strength is for short embedded lengths. This is due to increased mechanical bond strength of concrete keys that resisted the bond stress. This mechanical bond is a function of compressive strength and confinement provided by the steel fibers. Moreover, the crushing of concrete in front of steel bar ribs, takes place at high bond stress. During this the distribution of bond strength over the embedded length remains uniform. The results of this study may have a direct impact on development and splice length provisions of high strength fiber reinforced concrete.

Keywords: *Concrete keys, Tangential Stress, Fracture Process Zone, Embedded Length.*

1. Introduction

High strength concrete is extensively used in construction industry for columns of multistory buildings and girders of long span bridges. The monolithic behaviour of high strength reinforced concrete requires adequate bond between reinforcing steel bars and adjoining concrete to transfer the stresses between them (Kafeel Ahmed *et al*, 2014, 2013, H. Abrishami *et al*, 1996). In high strength concrete, optimized packing of the constituent materials and use of micro silica, increase the adhesion and friction bond. The concrete present between the ribs of the steel bar is called concrete key (Kafeel Ahmed *et al*, 2007, J. Ma *et al*, 2002). These concrete keys are strong due to high compressive and tensile strength of the concrete. The crushing of the concrete in front of the ribs require increased bond loading. This results in increased strength of the mechanical bond (Y. L. Mo *et al*, 1996, D. Weisse *et al*, 2003). However, when concrete keys slip over the ribs, tangential stresses develop in the surrounding concrete. The bond tangential and radial stresses are show in Fig.1. Hence the bond strength of high strength concrete is increased (Kafeel Ahmed *et al*, 2008, M. H. Harajli *et al*, 2004). When these stresses exceed the tensile strength of high strength concrete, longitudinal bond splitting cracks are initiated. The addition of steel fibers increases the bond strength in various manners (M. H. Harajli *et al*, 2004, X. Wang *et al*, 2003, Kafeel Ahmed *et al*, 2009). These cracks are bridged by the steel fibers and stress is shifted from concrete to these fibers. When adequate embedded length of the fibers is provided, increased bond energy is required for crack propagation. This is ensured by providing adequate length and volume of the steel fibers. All that result in increase in bond strength of the concrete. When the compressive strength of the fiber reinforced concrete is increased, the brittleness increases, however, steel fibers reduce the brittleness.

The use of fibers (90.0 mm) results in higher average bond strength. The increase was less than the increase in $(fc')^{1/2}$ due to increase in fiber volume. The bond strength increases when steel fiber volume is greater than 0.50% to 0.75%. Moreover improvement was more for large diameters bars than smaller diameters bars. There is 30% increase in bond strength for 0.5% by volume addition of

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steel fibers (M. H. Harajli *et al*, 2004). This necessitates the study of change in compressive strength of fiber reinforced high strength concrete (FRC). In order to study the effect of change in compressive strength, it was varied from 40.0 MPa, to 50.0 MPa and 60.0 MPa. The results of the experimentation showed that by increasing the compressive strength, bond strength increased and corresponding slip decreased. Another observation was that, when the first concrete key failed there was no sudden drop in bond strength as it occurred in plain high strength concrete. However, a gradual bond failure due to the steel fibers, took place. The inclusion of steel fibers, provide the necessary confinement through crack bridging action. This increases the post cracking resistance of the concrete. The fibers have little effect on bond strength when specimen failed by pullout (M. H. Harajli *et al*, 2004). Increased confinement reduced the non uniform bond stress distribution along the short embedment length. Stress concentration on the front keys is reduced. Hence bond strength improves even before the fibers perform their function (X. Wang *et al*, 2003).

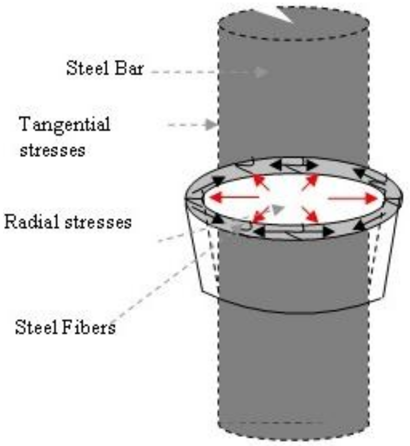

Fig.1 Tangential stress in concrete key

2. Fracture Mechanics Approach

Mechanics of bond crack is considered to study the crack behaviour. In fiber reinforced concrete, in front of longitudinal splitting bond cracks, there is a fracture process zone. In this zone bond energy is consumed before crack propagation starts. The other type of energy required is surface energy. This energy is required for de-cohesion of the grains of the material. In high strength materials, this zone is small. Therefore for high strength materials little energy is required for fracture process zone. However, surface energy must be sufficient for crack propagation. This fracture process zone is shown in Fig.2 (Kafeel Ahmed *et al*, 2009, D. Broek *et al*, 1974). This behavior is close to linear elastic fracture mechanics due to absence of strain softening and stress redistribution in high strength concrete. In pullout samples longitudinal splitting cracks initiate at much higher bond stress (Kafeel Ahmed *et al*, 2009, 2013). However, as these splitting crack are bridged by the steel fibers, the energy is required to pullout these fibers or fail them under tensile stress. Hence sufficient energy is consumed in crack propagation, this results in less brittle response of fiber reinforced pullout samples. When this energy is supplied by the loading, the crack again starts propagating leading to the failure of the bond. This crack propagation becomes gradual due to absorption of the bond energy. This energy is a function of embedded length of the fibers and mineral composition of the concrete. Therefore more energy is absorbed by the fiber reinforced concrete (Kafeel Ahmed *et al*, 2009, 2013, ACI 446.1, 1999).

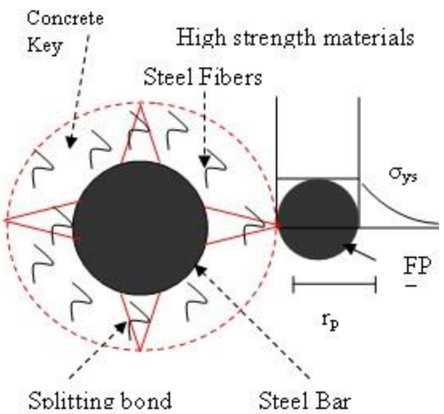

Fig.2 Fracture process zone I FRC

3. Experimentation

Concrete of 40.0 MPa, 50.0 MPa and 60.0 MPa compressive strength, was used in the study. Silica fumes having particles size 0.1 to 1.0 micron and silica content 92% were used for concrete. Hot rolled deformed steel bars of 19.0 mm diameters and yield strength of 415.0 MPa as per ASTM A_36, were used for pullout samples, consisting of 100.0 mm Ø 200.0 mm high concrete cylinders. Ordinary Portland cement conforming to EN 196 and ASTM C 150, Lawrancepur sand of 4.0 mm maximum size, Sargodha crush in two fractions 9.5 mm to 8.0 mm and 6.7 mm to 5.6 mm according to ASTM C 33 and third generation superplasticizer according to ASTM C 494, were used for high strength concrete. Aggregates were used in saturated surface dry conditions.

Laboratory temperature was kept at 25^{o}C and relative humidity at 81%. PVC pipes were used to break the bound between steel and concrete in order to achieve the desired embedded lengths as shown in Fig.3. Steel fibers of 30.0 mm length, 1.05 mm in diameter were used in the experimentation. These were added to the concrete after the addition of coarse aggregates and sand. The dose was 1.5 % by volume of the concrete. The distribution was slow and gentle to avoid any ball formation of fibers. The distribution of fibers during concrete mixing is shown in Fig.4. Immediately after pouring samples were covered with polyethylene sheets to avoid the loss of moisture. After 24 hours, demoulding was carried out and all the specimens were placed in curing water tank, making sure that the projecting bars should not be submerged. The samples for compressive strength were tested at 3, 7, 14 and 28 days and pullout tests were performed at the age of 28 days. Table 1 shows the geometric properties of steel bars used. Table.2 shows the diameter, cover and development length used for various pullout samples. These were gripped in an assembly having a hinge at the bottom to eliminate any eccentricity of the pullout sample as shown in Fig.5. Strain controlled pullout testing was done in universal testing machine (UTM). Data acquisition system with high precision linear displacement transducers (LDTs) measured the slip between steel reinforcement and concrete. The pullout test is shown in Fig.6. The load was applied and both steel and concrete experienced the strain. As the adhesion and friction bond failed slip was recorded by the data acquisition system. Concrete keys experienced the tangential stress and splitting cracks were initiated as the tangential stress exceeded the tensile strength of concrete. These splitting cracks were visible from the surface of the cylinders. When the mechanical bond failed, the sample also failed by experiencing splitting cracks oriented at about 120^{o} along the circumference of the cylinder as shown in Fig.7.

Fig.3 Steel bars for pullout tests

Fig.4 Mixing of steel fibers in plastic concrete

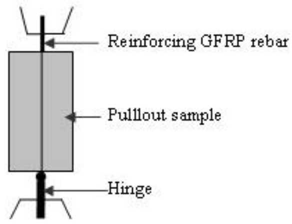

Fig.5 Line diagram of pullout assembly

Table 1 Geometric properties of reinforcing bar

Bar Dia meter	Rib Height 'a'	Avg. Rib Width 'b'	Avg. c/c Rib spacing 'c'	Clear distance between ribs	a/c
mm	mm	mm	mm	mm	
19	1.48	1.79	7.97	4.944	0.18
19	1.51	1.83	8.02	5.573	1.08

Table 2 Properties of pullout samples

S.No	Diameter mm	100mmØ 200mm High Cylinder		
		High strength concrete		
	d_b	Cover "c" in mm	c/d_b	Development length
1	19	40.5	2.13	$3.5d_b = 66.5$
2	19	40.5	2.13	$4.0d_b = 78.0$
3	19	40.5	2.13	$4.5d_b = 85.5$

Fig.6 Pullout test in UTM **Fig.7** Sample after test

4. Test Results and Discussion

The bond stress was calculated by using the formula as shown below. The force required for the given slip in the strain controlled testing machine was measured from the data acquisition system of the machine. Then this force was divided by the bonded area of the steel bar present over the development length or embedded length.

$$\frac{A_b f_s}{\pi d_b \, l_d} = U_b \tag{1}$$

f_s = Steel Stress A_b = Area of steel bar
d_b = Bar Diameter l_d = Development length

The compressive strength is an important factor influencing bond stress and slip relationship. Bond strength depends upon the mechanical bond. The mechanical bond is a function of compressive strength of concrete key that further depends upon slip over the crushed concrete in front of ribs. As the strength of the concrete is increased, concrete crush at higher bond stress and splitting cracks initiate at high tangential stress level. This results in increase the bond strength. Long embedded length, results in non uniform bond stress distribution as initial keys are always subjected to higher bond stress as compared to free end keys (S. P. Tastani *et al*, 2002). However for short development lengths like $3.5d_b$ to $4.5d_b$ as used in this experimentation, the bond stress distribution is close to uniform (Kafeel Ahmed *et al*, 2007, 2008, 2009). Moreover, the provision of steel fibers, arrested the advancing longitudinal bond splitting cracks. This resulted in less brittle bond failure. In this experimentation, the compressive strength of FRC was changed from 40.0 MPa to 50.0 MPa and then 60.0 MPa. The results of the experimentation showed that by increasing the compressive strength of FRC from 40.0 MPa to 50.0 MPa and 50.0 MPa to 60.0 MPa the bond strength increased by 30.0% and 75.0% respectively, the peak load slip increased by10.0% and 95.0% respectively for 3.5 d_b embedded lengths as in in Fig.8 to Fig.10, the bond strength increased by 35.0% and 15.0% respectively, the peak load slip increased by 15.0% and 95.0% respectively for 4.0 d_b embedded lengths as shown in Fig.11 to Fig.13, finally the bond strength increased by 55.0%, 15.0% respectively and peak load slip increased by 5.0% and 95.0% respectively for 4.5 d_b embedded length as shown in Fig.14 to Fig.16. The comparison of bond strengths is shown in Fig.17 and Table 3, comparison of slips is shown in Fig.18 and Table 4. The mode of failure was gradual due to crack bridging of the steel fibers. The increase in brittleness due to increase in compressive strength was reduced by steel fibers. Therefore brittle bond failure in high strength concrete can be controlled by steel fibers effectively.

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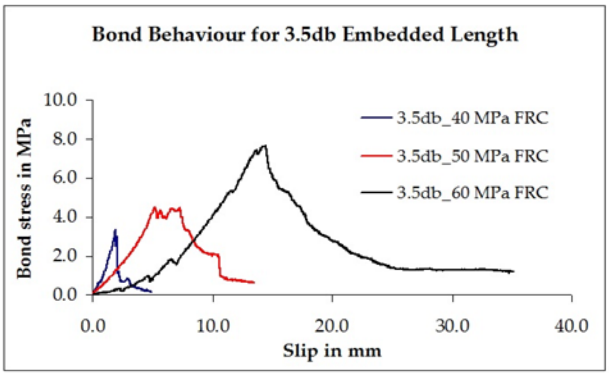

Fig.8 Bond behavior for 3.5 db embedded length in FRC

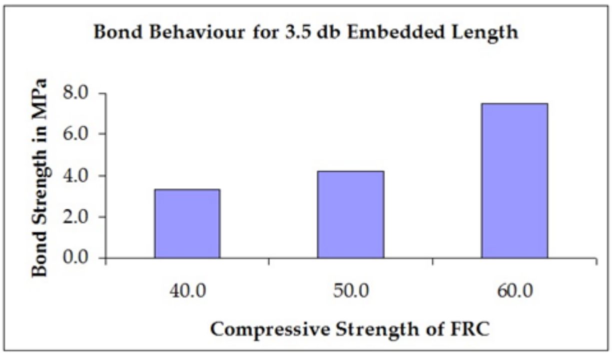

Fig.9 Bond behavior for 3.5 db embedded length in FRC

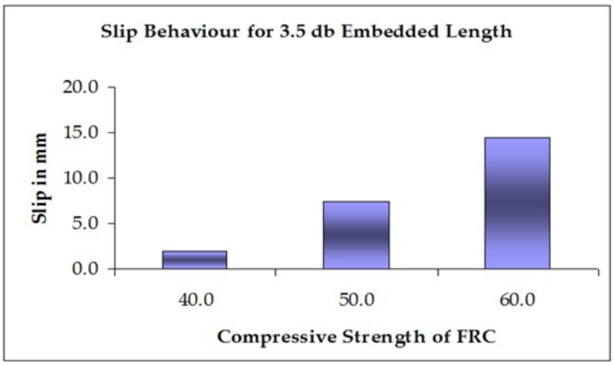

Fig.10 Slip behavior for 3.5 db embedded length

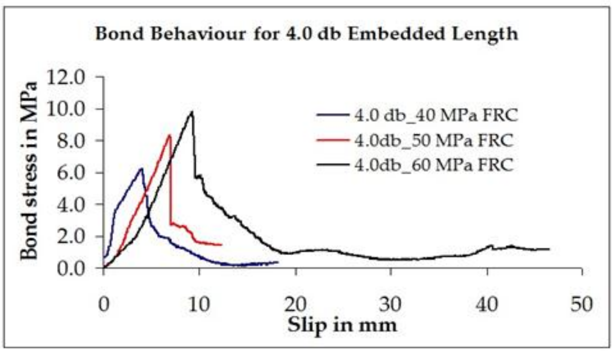

Fig.11 Bond behavior for 4.0 db embedded length FRC

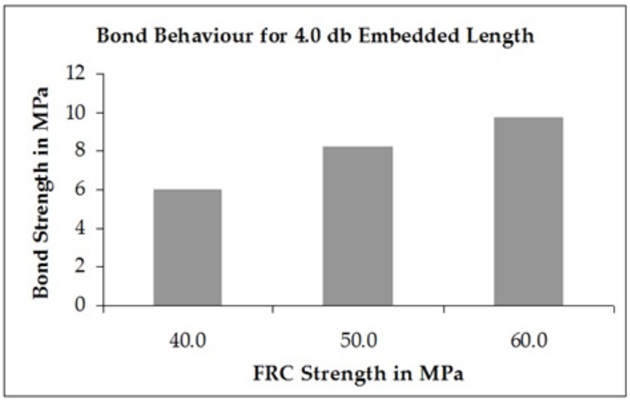

Fig.12 Bond behavior for 4.0 db embedded length FRC

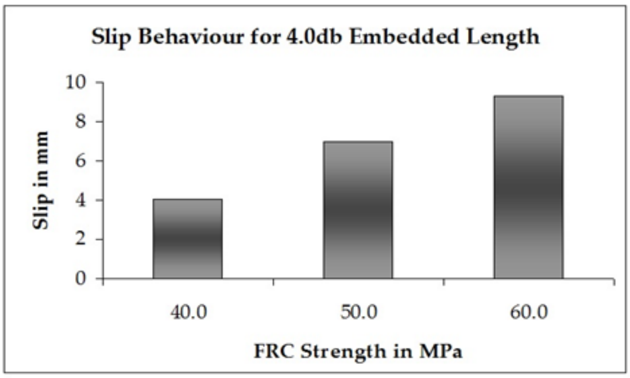

Fig.13 Bond behavior for 4.0 db embedded length FRC

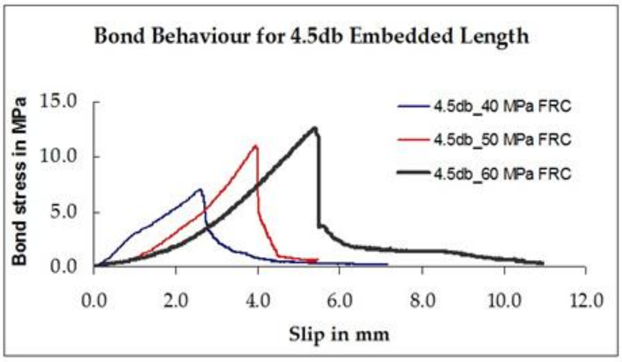

Fig.14 Bond behavior for 4.5 db embedded length for FRC

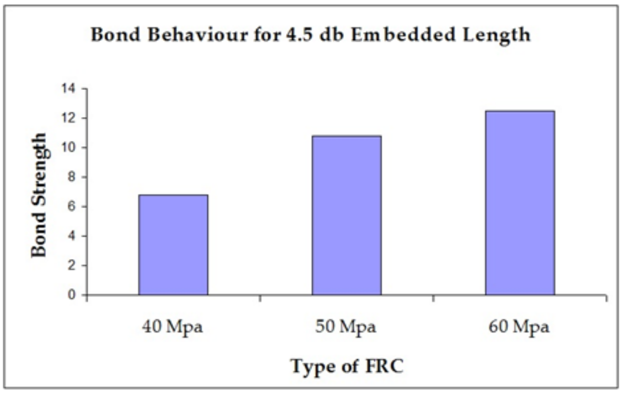

Fig.15 Bond behavior for 4.5 db embedded length for FRC

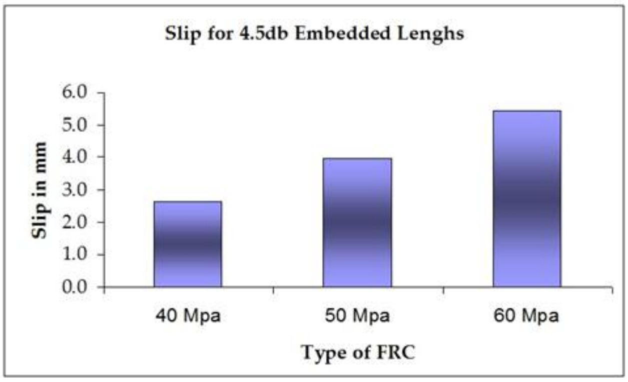

Fig.16 Bond behavior for 4.5 db embedded length in FRC

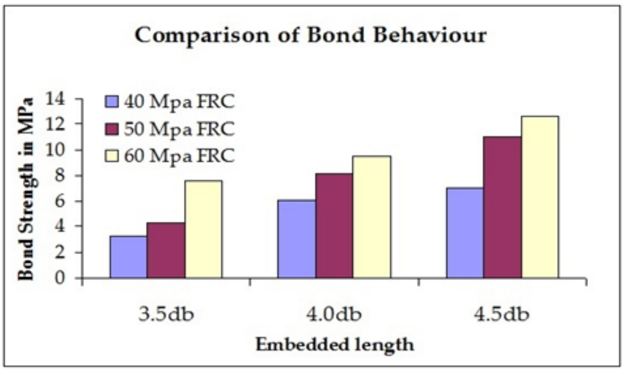

Fig.17 Bond behavior for 4.5db embedded length in FRC

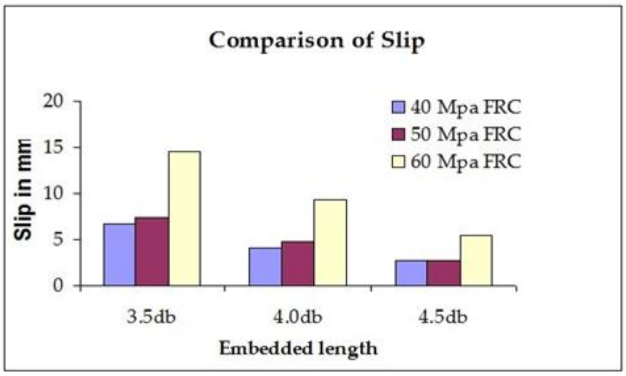

Fig.18 Bond behavior for 4.5db embedded length in FRC

Table 3 Bond strength for various FRCs

Concrete Strength	Bond Strength		
	3.5d_b	4.0d_b	4.5d_b
	MPa	MPa	MPa
40 Mpa	3.31	6.1	7.02
50 Mpa	4.3	8.2	10.96
60 Mpa	7.57	9.45	12.6

Table 4 Slip for various FRCs

Concrete Strength	Slip		
	3.5d_b	4.0d_b	4.5d_b
	mm	mm	mm
40 MPa	6.69	4.04	2.64
50 MPa	7.37	4.69	2.72
60 MPa	14.44	9.32	5.4

The mathematical equation to describe the pre-peak bond response of the bond stress and slip relationships is given below. The values of the coefficients and co-efficient of co relation are given in the Table 5.

$$u_c = \alpha s^2 + \beta s \quad (2)$$

u_c = Bond Stress
s = Slip of the steel
α , β = Coefficients

Table 5 The co-efficient for the pullout bond behaviour

S.No	f_c'	Co-efficient				co-relation Co-efficient
	MPa	"α"		β		*R*
		Min	Max	Min	Max	
1	40	0.022	0.34	0.043	1.08	0.9994
2	50	0.031	0.41	0.041	0.64	0.9953
3	60	0.026	0.23	0.006	1.50	0.9919

5. Comparison with Local Bond Constitutive Model

The post peak response of the bond failure is given by the bond constitutive model given by Eligehausen *et al* (1983), (ascending part adopted by Comite- International du Beton- Federation International de la Precontrainte Model Code 1990) (Harajli *et al* , 2004). It shows that the response of the pullout samples is close to splitting bond failure and not to pullout bond failure. The reason for this behavior is that crushing of concrete in front of the ribs of the steel bar is insignificant and mainly the splitting of the concrete due to circumferential tensile bond stress took place. This local bond model is shown in Fig.19. This ascending part is mathematically given by Eligehausen *et al* (1983) and shown below. The descending part could not be determined in this set of experimentation (Harajli *et al* , 2004).

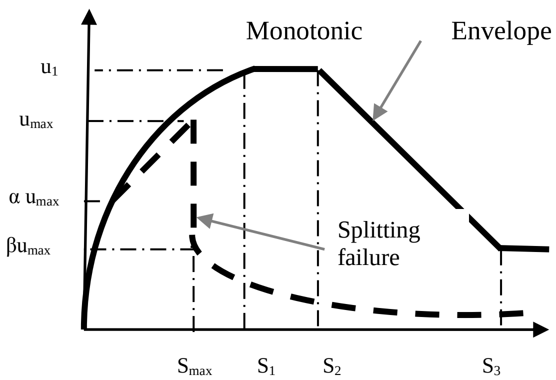

Fig.19 Bond constitutive model (Harajli *et al* , 2004)

$$\frac{U}{u_l} = \left(\frac{s}{s_l}\right)^{v} \quad \text{For Ascending part of the curve} \quad (3)$$

$$\frac{U}{\beta u_{max}} = \left(\frac{s}{s_{max}}\right)^{v} \quad \text{For descending part of the curve} \quad (4)$$

u_{max} = maximum bond stress in splitting failure

S_{max} = maximum slip in splitting failure

α=0.7 For High strength concrete (Harajli *et al* , 2004)
β=0.3 to 0.5 For High strength concrete (Harajli *et al* , 2004)

In case of pullout bond failure, inclusion of fibers does not affect the bond performance because longitudinal splitting cracks are not formed in this type of failure. In all the above mentioned tests, confinement by the fibers increased the bond strength and changed the mode of failure of FRC from brittle to ductile (Kafeel Ahmed at el, 2009, 2013).

Conclusions

1. It is concluded from this research work that by increasing the compressive strength of FRC from 40.0 MPa to 50.0 MPa and then to 60.0 MPa, the bond strength increased by 15% to 75% for different embedded lengths of steel fiber reinforced concrete.
2. Similarly by increasing the compressive strength of FRC from 40.0 MPa to 50.0 MPa and then to 60.0 MPa, the peak load slip also increased from 5.0% to 95.0%.
3. This is attributed to crack bridging action of steel fibers present in FRC. These fibers carry the tangential stresses after cracking of concrete, this results in increase in bond strength, however the slip takes place without failure.
4. The mechanical bond strength of FRC keys increases by increasing the concrete strength. This results in improved resistance to splitting tangential bond stress.
5. The increase in brittleness due to increase in compressive strength was reduced by steel fibers.
6. The brittle bond failure in high strength concrete can be controlled by steel fibers effectively.
7. The fibers bridge the splitting cracks and carry the tangential bond stress thereby increase the bond strength.
8. The inclusion of steel fibers does not affect the pullout bond failures as there are no splitting cracks present in pullout failures.

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