Effect of Embedded Length on Bond Behaviour of Steel Reinforcing Bar in Fiber Reinforced Concrete.

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ABSTRACT

Adequate embedded length of steel reinforcing bar in concrete, ensures the stress transfer among steel and concrete. The inclusion of steel fibers in high strength concrete controls the sudden crack propagation and changes the mode of failure from ductile to brittle. The confinement provided by these fibers improves the bond strength. There was a need to study the effect of change in embedded length for fiber reinforced high strength concrete. Experimental study was done to determine the bond strength of fiber reinforced high strength concrete for different embedded lengths. The whole post peak bond behaviour was studied. The results of this experimentation confirmed that by increasing the development length from 3.5 db to 4.5 db, the bond strength of deformed steel bar in fiber reinforced high strength concrete increased from 15% to 90%, the corresponding slip reduced from 35% to 40%. This increase in bond strength and decrease in slip as a function of increase in development length is for short embedded lengths. This is due to increased mechanical bond strength of increased number of concrete keys that resisted the 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: Bond stress, Fiber reinforced concrete, Slip of reinforcement, Embedded length, Concrete key

1 INTRODUCTION

High strength concrete is used in compressive stress carrying structural members due to its high strength and durability. Structural application of this material requires that adequate bond should exist between steel reinforcement and concrete [1, 2, 4, 5, 6]. In high strength concrete the boundary between steel and concrete is highly improved due to very dense packing and pozzolanic effect of silica fumes. This results in strong adhesion and friction bond. Moreover the increased compressive and tensile strength of concrete key offer resistance to crushing of concrete infront of the steel bar ribs and splitting of concrete due to tangential bond stress [7, 8, 9]. The bond tangential and radial stresses are shown in Fig.1. Therefore bond strength is increased and the development length can be decreased as compared to normal strength concrete. However the brittle behaviour of high strength concrete may lead to sudden failure [14].

The inclusion of randomly distributed steel fibers, offer a solution to this problem. These steel fibers arrest the splitting cracks. These cracks are initiated as the tangential bond stress exceeds the tensile strength of the concrete. However, the propagation of these cracks is stopped by the bridging action of these steel fibers [2]. As an advancing crack is encountered by a steel fiber, there are two possibilities that either, the crack propagation will stop and stress will be transferred through the steel fiber. Other possibility is that the fiber would be pulled out of the concrete. This happens in case of insufficient embedded lengths of the fibers. In first case the splitting stress is transferred from the concrete to the steel fibers. The pullout or failure of these fibers depends upon the length of the steel fibers used and their tensile strength respectively. The indirect confinement provided by the crack bridging, improves the bond strength [2, 15]. Davies concluded that 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. Ezeldin and Balaguru concluded that 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. Soroushian concluded that there is 30% increase in bond strength for 0.5% by volume addition of steel fibers.

This necessitates the study of change in embedded length for fiber reinforced high strength concrete (FRC). In order to study the effect embedded length, it was varied from 3.5db to 4.0 db and then further to 4.5 db, for 19 mm diameter steel reinforcing bars in 40.0 MPa, 50.0 MPa and 60.0 MPa steel fiber reinforced concrete.

The results of the experimentation showed that by increasing the embedded length, bond strength increased and corresponding slip decreased. Another fact observed was, as the first concrete key failed there was no sudden drop in bond strength as it happened in plain high strength concrete. However there was a gradual bond failure due to the steel fibers. Confinement by inclusion of steel fibers, increase the post cracking resistance of the concrete. Harajli concluded that fibers have little effect on bond strength when specimen failed by pullout [11]. Increased confinement reduced the nonuni-

form 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 [12].

Radial stress  Tangential stresses

Fig.1 Stresses in concrete key

[Image: Stresses in concrete key]

2  FRACTURE MECHANICS APPROACH

The fracture process zone in front of longitudinal splitting bond cracks in fiber reinforced concrete is small and zone of perfect plasticity even further small as shown in Fig.2 [1,2,14,17]. 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 [2,14].

Fig.2: Fracture process zone in high strength concrete pullout samples [13]

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 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 [2, 14, 18].

3  EXPERIMENTATION

High strength concrete having 40.0 MPa, 50.0 MPa and 60.0 MPa as compressive strength was used in the study. Silica fumes having particles size 0.1 to 1.0 micron and silica content 92% was used. 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 197 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 polycarboxylate ether 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°C and relative humidity at 81%. PVC pipes were used to debound the steel from concrete in order to achieve the desired development lengths as shown in Fig.3.

Fig.3 Steel bars for pullout tests

During concreting steel fibers were added after the addition of coarse aggregates and sand. These fibers were 30.0 mm long, 1.05 mm in diameter and they were added 1.5 % by volume of the concrete. They were distributed slowly and gently 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 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 pull-out samples. The pullout samples 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 (LVDTs) measured the slip between steel reinforcement and concrete. The pull-out test is shown in Fig.6.

The load was applied and both steel and concrete experienced 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° along the circumference of the cylinder as shown in Fig.7.

![Image](image1.png)

**Fig.4 Mixing of steel fibers in plastic concrete**

### Table 1

**Geometric properties of reinforcing bar**

<table>
<thead>
<tr>
<th>Bar Dia meter (mm)</th>
<th>Rib Height 'a' (mm)</th>
<th>Rib Width 'b' (mm)</th>
<th>Avg. Rib c/c Rib spacing 'c' (mm)</th>
<th>Clear distance between ribs (mm)</th>
<th>a/c</th>
</tr>
</thead>
<tbody>
<tr>
<td>19</td>
<td>1.48</td>
<td>1.79</td>
<td>7.97</td>
<td>4.944</td>
<td>0.18</td>
</tr>
<tr>
<td>19</td>
<td>1.51</td>
<td>1.83</td>
<td>8.02</td>
<td>5.573</td>
<td>1.08</td>
</tr>
</tbody>
</table>

**Table 2**

**Properties of pullout samples**

<table>
<thead>
<tr>
<th>S.N o</th>
<th>Diameter mm</th>
<th>100mmØ 200mm High Cylinder</th>
<th>High strength concrete</th>
<th>High strength concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>d_b (mm)</td>
<td>Cover “c” in mm</td>
</tr>
<tr>
<td>1</td>
<td>19</td>
<td>40.5</td>
<td>2.13</td>
<td>3.5d_b = 66.5</td>
</tr>
<tr>
<td>2</td>
<td>19</td>
<td>40.5</td>
<td>2.13</td>
<td>4.0d_b = 78.0</td>
</tr>
<tr>
<td>3</td>
<td>19</td>
<td>40.5</td>
<td>2.13</td>
<td>4.5d_b = 85.5</td>
</tr>
</tbody>
</table>

![Image](image2.png)

**Fig.5 Line diagram of pullout assembly**

**Fig.6. Pullout test in UTM**

**Fig.7. Sample after test**

### 6 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.

$$\frac{A_b f_s}{\pi d_b l_d} = U_b$$ (1)

Where:
- $f_s =$ Steel Stress
- $A_b =$ Area of steel bar
- $d_b =$ Bar Diameter
- $l_d =$ Development length

Embedded length is a significant factor influencing bond stress and slip relationship. Bond stress distribution depends upon the development length. 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 [13]. However for short development lengths like 3.5$d_b$ to 4.5$d_b$ as used in this experimentation, the bond stress distribution is close to uniform[4, 5, 6, 7].

Therefore any increase in embedded length, results in more number of concrete keys resisting the bond failure and bond strength improves. Moreover, the provision of steel...
fibers, arrested the advancing longitudinal bond splitting cracks. This resulted in less brittle bond failure. In this experimentation, the embedded length was changed from 3.5\(d_b\) to 4.0\(d_b\) and from 4.0\(d_b\) to 4.5\(d_b\) for different strengths of fiber reinforced concrete (FRC). The results of the experimentation showed that by increasing the development length from 3.5\(d_b\) to 4.0\(d_b\), bond strength increased by 85\% in 40.0 MPa FRC, 90\% in 50.0 MPa FRC, 15\% in 60.0 MPa FRC. When the embedded length was increased from 4.0\(d_b\) to 4.5\(d_b\), it increased by about 15\% 40.0 MPa FRC, 35\% in 50.0 MPa FRC, 35\% in 60.0 MPa FRC as shown in Fig.8 to Fig.16. This may be due to more number of concrete keys resisting the slip and bond strength increased.

The results also showed that peak load slip of steel reduced by 40\% 40.0 MPa FRC, 35\% in 50.0 MPa and 60.0 MPa FRC when embedded length increased from 3.5\(d_b\) to 4.0\(d_b\). Similarly the peak load slip of steel reduced by 35\% in 40.0 MPa FRC, 40\% in 50.0 MPa and 60.0 MPa FRC, when embedded length increased from 4.0\(d_b\) to 4.5\(d_b\). This may be due to the reason that for long embedded lengths, more number of concrete keys offered more resistance to bond failure, also due to improved mechanical bond, bond strength increased and corresponding slip decreased. Increase in bond strength is 85\% when embedded length increased from 3.5\(d_b\) to 4.0\(d_b\), however it increased by about 15\% to 35\% when embedded length increased from 4.0 \(d_b\) to 4.5\(d_b\). The comparison of bond strengths is shown in Fig.17 and Table 3; comparison of slips is shown in Fig.18 and Table 4.

Once one key failed, failure was to propagate immediately however increased confinement and bridging action by fibers increased the bond strength and failure was gradual.
The bond strength for different compressive strengths of FRC

<table>
<thead>
<tr>
<th>Concrete Strength</th>
<th>Bond Strength 3.5db</th>
<th>Bond Strength 4.0db</th>
<th>Bond Strength 4.5db</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 MPA</td>
<td>3.31</td>
<td>6.1</td>
<td>7.02</td>
</tr>
<tr>
<td>50 MPA</td>
<td>4.3</td>
<td>8.2</td>
<td>10.96</td>
</tr>
<tr>
<td>60 MPA</td>
<td>7.57</td>
<td>9.45</td>
<td>12.6</td>
</tr>
</tbody>
</table>

The slip for different compressive strengths of FRC

<table>
<thead>
<tr>
<th>Concrete Strength</th>
<th>Slip 3.5db</th>
<th>Slip 4.0db</th>
<th>Slip 4.5db</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 MPa</td>
<td>6.69</td>
<td>4.04</td>
<td>2.64</td>
</tr>
<tr>
<td>50 MPa</td>
<td>7.37</td>
<td>4.69</td>
<td>2.72</td>
</tr>
<tr>
<td>60 MPa</td>
<td>14.44</td>
<td>9.32</td>
<td>5.4</td>
</tr>
</tbody>
</table>

The mathematical equation to describe the initial 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)

\[ u_c = \text{Bond Stress} \]
\[ s = \text{Slip of the steel} \]
\[ \alpha, \beta = \text{Coefficients} \]
Table 5
The values of the coefficient for the pullout bond behaviour

<table>
<thead>
<tr>
<th>S.No</th>
<th>Concrete</th>
<th>Co-efficient</th>
<th>Co-efficient of correlation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength</td>
<td>Min α</td>
<td>Max α</td>
</tr>
<tr>
<td>1</td>
<td>40</td>
<td>0.020</td>
<td>0.35</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>0.030</td>
<td>0.42</td>
</tr>
<tr>
<td>3</td>
<td>60</td>
<td>0.025</td>
<td>0.22</td>
</tr>
</tbody>
</table>

6 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). 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 [10,12,14].

\[
\frac{u}{u_1} = \left( \frac{S}{S_1} \right) \alpha \quad \text{For Ascending part of the curve (3)}
\]

\[
\frac{u}{\beta u_{\text{max}}} = \left( \frac{S}{S_{\text{max}}} \right) \quad \text{For descending part of the curve (4)}
\]

\[u_{\text{max}} = \text{maximum bond stress in splitting failure}\]
\[S_{\text{max}} = \text{maximum slip in splitting failure}\]
\[\alpha = 0.7 \quad \text{For High strength concrete [11]}\]

\[\beta = 0.3 \text{ to } 0.5 \quad \text{For High strength concrete [11]}\]

Inclusion of the steel fibers increases the post cracking resistance of concrete in splitting bond failure. These fibers arrest the longitudinal splitting bond cracks and therefore increase the bond strength. 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 [2,14].

7 CONCLUSIONS

1. It is concluded from this research work that by increasing the development length from 3.5 \(d_b\) to 4.0 \(d_b\) for 19.0 mm bar, bond strength increased by 25% to 90% for different strengths of fiber reinforced concrete.

2. Similarly by increasing the development length from 4.0 \(d_b\) to 4.5 \(d_b\) for 19.0 mm bar, bond strength increased by 15% to 35% for different strengths of fiber reinforced concrete.

3. This is attributed to increased number of high strength fiber reinforced concrete keys that took part in resisting the slip and bond strength increased.

4. Fiber reinforced concrete keys have strong mechanical bond strength and improved resistance to splitting tangential bond stress.

5. The fibers bridge the splitting cracks and carry the tangential bond stress thereby increase the bond strength.

6. The failure of samples was gradual as the splitting cracks were bridged by the steel fibers and more bond energy was required to propagate the cracks leading to bond failure.

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REFERENCES JOURNALS


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