

# Effect of Intersecting Beam on Bond Behaviour at the Joint

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**Abstract**—Adequate and effective bond is necessary for composite action of reinforced concrete structural elements. At the joints of flexural members, bending stress of one beam magnifies the tangential bond stress around the main steel of the intersecting beam. This reduces the bond strength of intersecting beams and longitudinal splitting bond cracks initiate at lower loads, reducing the load carrying capacity of the structural member. Experimental study was carried out to determine this decrease in bond strength at the joint of intersecting beams. Beams, designed to fail in bond, were casted for this purpose. Each set consisted of three beams, two intersecting and one control. In all the beams steel and concrete strain gauges were used to measure the strain developed in steel and concrete. The results of the experimentation showed that the bond strength of primary beam of the joint of intersecting beams, reduced 15 to 30% as compared to bond strength of control beam. These test results may have an implication of on development length and splice length provisions at the joints of intersecting beams, in the building codes. This reduction in bond strength, necessitate the provision of bond improving measures like extra confinement through the stirrups or increased development length, may offer a solution to this problem.

**Key words.** Steel slip, Bond splitting cracks, Fracture process zone, tangential bond stress, Cover to bar diameter ratio.

**1 Introduction** The bond behaviour of concrete and embedded reinforcing steel is essential for composite action in reinforced concrete construction [1,2,3,6,7]. Though the pull out test to determine the bond strength, is easy to perform, however, the results do not directly represent the actual stress state. This is due to the fact that stress distribution that results in pull out test is different from that present in flexural members. Moreover the transverse confinement provided is also different. This transverse confinement affects the normal pressure on the pull out samples. The initiation and propagation of the bond splitting cracks are a function of confinement [1,8,9].

In reinforced concrete flexural members, the joints are critical as there are more chances of bond failure and subsequent slip of steel relative the concrete. Therefore it is necessary to study their bond performance. Heavy reinforcement at the joints, leaves very little space for concrete to be placed. Poorly compacted honey combed concrete results in low strength concrete keys. These are present between the ribs of the steel bar. The tensile strength of this concrete is reduced as compared to the concrete, in other parts of the beam. Due to the slip of the steel against the steel bar ribs, radial and tangential stresses develop around the steel bar. These tangential bond stresses are a function of tensile strength of the concrete. When the beams are loaded under service conditions, these concrete keys fail due to tangential bond stresses. The longitudinal splitting cracks initiate and propagate[15].

Hence reduction in bond stiffness at the joint may result in excessive joint rotation and mid span deflection. Keeping in view the importance of bond behaviour at the joints, bond beam tests with models of intersecting beams were planned to study the bond stress and slip relations of steel and concrete. In bond beam and models of intersecting beams, development length was kept constant. Bond beam acted as control beams and beams of intersecting model were named as primary and secondary beams. All these beams and models were tested and data was recorded. This data was processed and relationships were developed. This study was done for different strengths of concrete. In all the samples, bond strength of primary beam reduced

as compared to control beams. The flexural stress, present at the mid span of the secondary beam, was acting in the same direction as tangential stress (circumferential tensile bond stress) developed around the steel reinforcing bars. This circumferential tensile bond stress developed due to the slip of the concrete key present between the two ribs of the reinforcing steel bars. Therefore this stress is magnified due to the flexural action of secondary beam. When this stress exceeded the tensile strength of the concrete, bond cracks initiated along the circumference of the steel reinforcing bar along the length. These longitudinal splitting cracks propagated rapidly, diminishing the bond strength of primary beam [15].

## 2. Bond Fracture Mechanics

In normal strength concrete, bond strain softening and bond stress redistribution adjoining the reinforcing steel bar take place. The fracture process zone in front of primary and longitudinal splitting bond cracks is large as shown in Fig.1. and zone of perfect plasticity is well defined, the bond fracture energy consists of energy consumed in zone of perfect plasticity and surface energy[15,16,18]. This results in gradual crack propagation. The bond stress and slip relationship exhibited by normal strength concrete samples showed a non linear response [18]. Cracks in normal strength concrete initiate at lower load level of the ultimate load [10,17]. Therefore in bond beam tests and model tests, interface de bonding cracks and longitudinal splitting cracks initiate at lower bond stress.

The bond stress and slip relationship exhibited by high strength concrete samples showed an initial stiff linear response [15]. Cracks in high strength concrete initiate at much higher load level, typically 70 to 80% of the ultimate load [15,17]. Therefore in bond beam tests and model tests, interface de bonding cracks and longitudinal splitting cracks initiate at much higher bond stress. This results in accumulation of bond strain energy in high strength materials. The bond fracture energy consists of surface energy. Once a crack forms at the interface due to slip between steel and concrete, all the accumulated bond strain energy is poured in to this crack where it is dissipated in

the form of surface energy. In high strength concretes, the fracture process zone is small and zone of perfect plasticity is even smaller as shown in Fig.1. Therefore little cracking occurs and whole energy is used in immediate crack propagation in abrupt manner. Failure was sudden, showing a brittle response [18].

This bond fracture behaviour of high strength concrete can be explained by linear elastic fracture mechanics. During this, all the grains present on the fracture path rupture due to de-cohesion of bonds and remaining material shows elastic response. Stress redistribution and strain softening do not occur in high strength concrete. Mathematical relationships mentioned below is for high strength materials and developed by David and Broeks that support the fracture behavior of high strength concrete adequately [16].

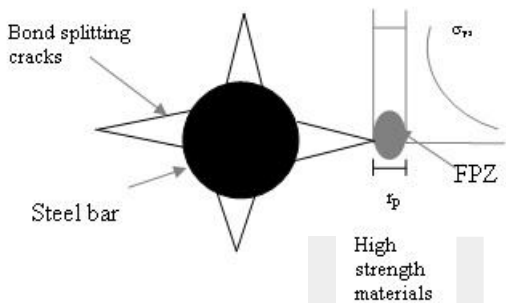


Fig.1: Fracture process zone in high strength concrete pullout samples [13]

$$\sigma_{ys} = \frac{ki}{\sqrt{2 \pi r_p}} \quad (1)$$

$$r_p = \frac{Ki^2}{2\pi\sigma_{ys}^2} \quad (2)$$

$$r_p \propto \frac{Ki^2}{\sigma_{ys}^2} \quad (3)$$

$\sigma_{ys}$  = Yield strength of material  
 $r_p$  = Size of fracture process zone  
 $K_i$  = Stress intensity factor

3. Experimentation

Hot rolled deformed steel bar conforming to ASTM C 36 was used in the experimentation. Its geometrical properties are shown in Table 3 and Table 4. Four control beams of normal and high strength concrete were casted. In each beam 5.0  $d_b$  embedded length was provided as development length to determine the bond strength at the required embedded lengths. To break the bond between steel and concrete in the remaining part of the beam, PVC pipes were used as shown in Fig.2. The cross section of the beam is shown in Fig.3. The surface of the steel bar was prepared for the installation of strain gauge according to the guidelines of the strain gauge manufacturer. In this experimentation, 7.0 mm gauge length uni-axial strain gauges with pre attached wires were fix to the reinforcing steel bars as shown in Fig.4 and Fig.5. Concrete stain was determined by using polymer concreted covered strain

gauges as shown in Fig.6.

Table 1  
Properties of steel bar used

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	
13	1.20	1.91	7.39	4.944	0.16
13	1.36	1.86	7.97	5.029	0.17
19	1.48	1.79	7.97	4.944	0.18
19	1.51	1.83	8.02	5.573	1.08

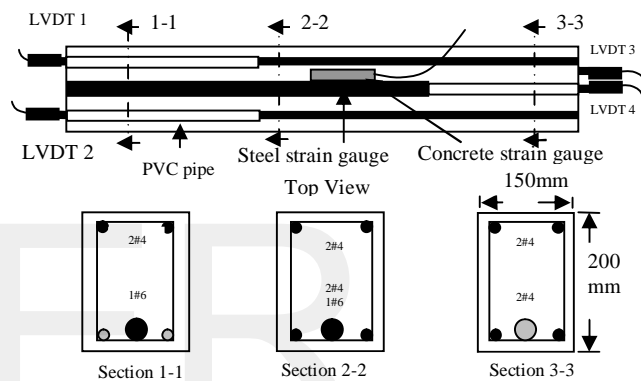


Fig.2 Plan for Beam Test No-1

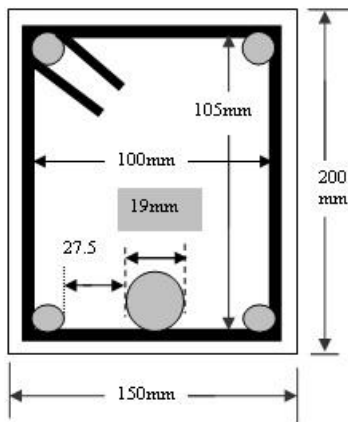


Fig.3 X-section of the beam



Fig.4 Tied steel with PVC pipe

During the concreting of the beams, care was done to avoid the damage to concrete and steel strain gauges and their wires. Immediately after pouring beams were covered with plastic sheet to stop the loss of water due to evaporation. The concreting of the beam is shown in Fig.7(a) and Fig.7(b). Curing of the beams was started before the initial setting time of the cement as shown in Fig.8. To determine the compressive strength of the concrete, cylinders as per ASTM standards, were casted. The compressive strength of the concrete was determined by testing these cylinders in strain controlled universal testing machine (UTM) at 7, 14 and 28 days. The results of compressive strength tests of all the set of beams and models of intersecting beams are shown in Table.5. The mould of the beam was removed 72 hours after pouring. Beam samples were covered with wet jute bags and then plastic sheet was wrapped on all sides to stop the loss of water from even jute bags. After 28 days, these beam samples were taken out, dried and painted. The marking was done according to the testing scheme of data logger. All the channels of the data logger were reserved for the selected outputs.

Bond beam testing was done in UTM as shown in Fig.9. Linear variable displacement transducers (LVDTs) were used to measure the slip of steel and concrete as shown in Fig10. Load cells, attached with the data logger, were used to confirm the load values obtained from the UTM. Similarly strain gauges were also attached with the data acquisition system. Two point loading was applied through the load cells. Deflection was measured through the data logger of the universal testing machine.

In the first set of the beams, the strength of the concrete used, was 30.0 MPa. The size of the beam was 150.0 x 200.0 x 1080.0 mm. As the beam was loaded, steel and concrete deformed monolithically and both showed expansion as recorded by strain gauges and LVDTs. However, when the friction bond was failed, slip occurred. When the slip increased and steel strain reduced, concrete strain experienced increase. The steel bar relaxed and returned back after the failure of mechanical bond. Failure initiated by the formation of flexural cracks present at locations of PVC pipes. Further loading multiplied the cracks and increased the crack propagation. These cracks propagated and converted into horizontal bond cracks as shown in Fig.11. Finally a "v" notch bond failure was observed.



Fig.6 Concrete strain gauge



Fig.7(a) Process of concreting



Fig.7(b) Process of concreting



Fig.8 Beam after pouring



Fig.5 Tied steel with stain gauges



Fig.9 Testing of beam in progress

Table. 5  
Results of compressive strength tests with age

Age	Sample	Compressive Strength			
		1st Set	2nd Set	3rd Set	4th Set
Days	Cylinder	MPa	MPa	MPa	MPa
7	100.0 mmX 200.0 mm	20.0	32.0	43.0	60.0
14	100.0 mmX 200.0 mm	23.0	36.0	56.0	72.0
28	100.0 mmX 200.0 mm	30.0	50.0	75.0	100.0



Fig.10 Testing of beam in progress

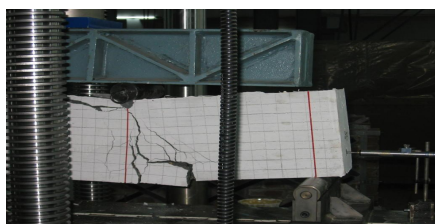


Fig.11 failure in progress

In the 1<sup>st</sup> model of intersecting beams, the beam with greater effective depth of steel, was named as primary beam and other intersecting beam was named as secondary beam. The compressive strength of the concrete used, was 30.0 MPa. The x-section, length and arrangement of steel in both primary and secondary beam were same as that of 1<sup>st</sup> beam of bond beam test. The concreting of the model of the intersecting beams is shown in Fig.12. The model of intersecting beams before testing is shown in Fig.13. To study the crack initiation, propagation and failure, the grid was marked and instrumentation was done in the same way as it was done on bond beam.

The loading of the model of intersecting beam was done through a special testing assembly having same shape as that of the model to be tested as shown in Fig.14. In this way, two point load could be applied simultaneously on both the beams. Load cell was attached with the data logger. The whole testing arrangement is shown in Fig. 15. In the next 2<sup>nd</sup>, 3<sup>rd</sup> and 4<sup>th</sup> set of beams and intersecting models the strength of the concrete was increased to 50.0, 75.0 and 100.0 MPa respectively. This was done on the basis of research findings of pull out tests and that of other researchers [11,12,13]. Testing was performed in the same way as that of 1<sup>st</sup> set. The results of the experimentation, were obtained and processed. Then relations were developed and concluded. They are shown in Fig.16 to Fig.19 and all compared in Fig.20 and Table.6



Fig. 12 Tied steel and pouring in progress



Fig.13 Model of intersecting beams before test

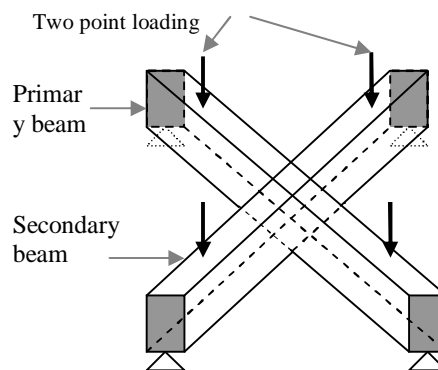


Fig. 14 Loading of intersection beam



Fig.15 Testing of the model is in progress

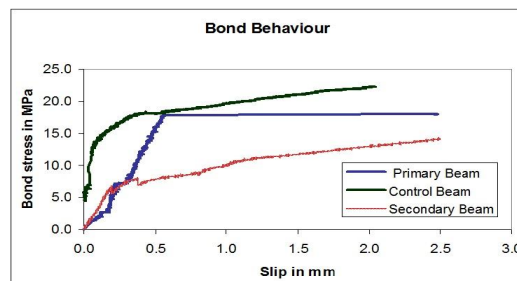


Fig.16 The Bond behaviour of Set\_1

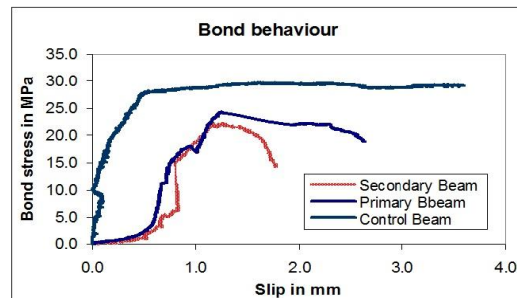


Fig.17 The Bond behaviour of Set\_2



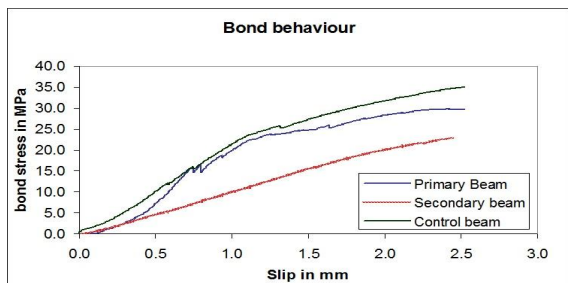


Fig.18 The Bond behaviour of Set\_3

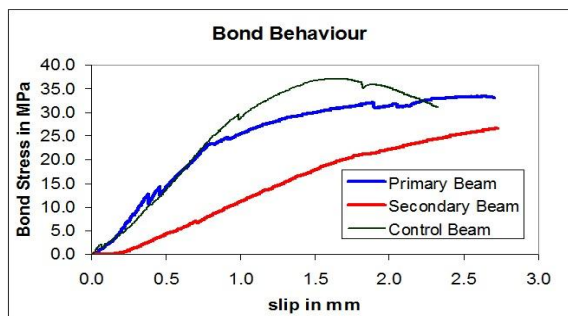


Fig.19 The Bond behaviour of Set\_4

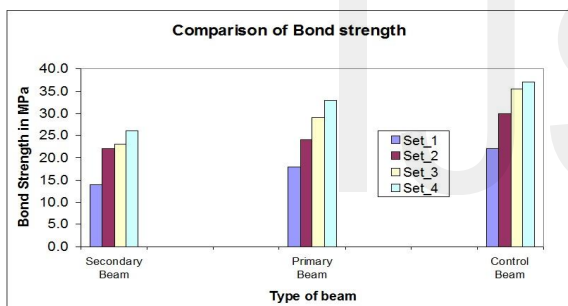


Fig.20 Comparison of Bond behaviour of all the beams

5. Analysis of The Results And Discussion:

The bond strength of beams was calculated using the formula as shown below. The strain used, was measured from the steel strain gauge. Stress present in steel was calculated using the modulus of elasticity of steel and strain measured. The force in steel was calculated by using area of steel and stress in steel. Then this force was divided by the bonded area of the steel bar present over the embedded length.

(4)

$$\frac{A_b f_s}{\pi d_b l_d} = U_b \quad (5)$$

$f_s$  = Steel Stress      $A_b$  = Area of steel bar

$\epsilon_s$  = Steel Strain      $d_b$  = Bar Diameter

$E_s$  = Modulus of elasticity of steel      $l_d$  = Embedded length

The results of the experimentation show that in 1<sup>st</sup> set of experimentation the bond strength of primary beam reduced by about 9.0%, in 2<sup>nd</sup> set by about 10.0%, in 3<sup>rd</sup> set by about 20.0% and in 4<sup>th</sup> set by about 21.0% as compared

to control beam as shown in Table.6. Following equation was developed for the control beam and primary beam of the model of intersecting beam. The co-efficient of correlations of these result varied from 0.97 to 0.98. The value of “ $\alpha$ ” for control beam ranged from 1.5 to 2.7 and for primary beam 4.5 to 5.4. Similarly the value of “ $\beta$ ” for control beam varies from 10 to 12 and for primary beam it varies from 14 to 18.

$$u_c = \alpha s^2 + \beta s \quad (6) \text{ For Control Beam}$$

$$u_p = \alpha s^2 + \beta s \quad (7) \text{ For Primary Beam}$$

$u_p$  = Bond stress of primary beam

$u_c$  = Bond Stress of Control Beam

$s$  = Slip of the steel

$\alpha, \beta$  = Coefficients

As the load was applied on the two intersecting beams, steel has the tendency to slip relative to the concrete due to difference in stiffness. Concrete present between two ribs is called as concrete key. Concrete, in front of the ribs of steel bar, was crushed forming a wedge. Adjoining concrete i.e. concrete keys, slipped on this wedge and expanded like a thick walled pressure vessel, radial tensile stress and tangential bond stress (circumferential tensile bond/ hoop stress) developed in the surrounding concrete keys [12,15,19]. These tangential bond stresses when exceeded the tensile strength of concrete, longitudinal splitting cracks around the steel bar, were initiated as shown in Fig.21. The tangential stress magnified by the tensile component of the flexural stress of intersecting beam. Hence stress magnification reduced the bond strength of the beam. Brittle behaviour was observed in high strength concrete. The mechanics of intersecting model is shown in Fig.22 [19]. The mechanism for stress magnification is shown in Fig.23 [15].

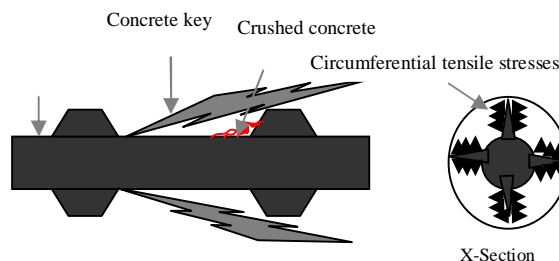


Fig.21 Longitudinal splitting cracks

Table.6  
Comparison of the bond strengths

Type of beam	Bond Strength in MPa			
	Set-1	Set-2	Set-3	Set-4
Secondary Beam	14.0	22.0	23.0	26.0
Primary Beam	18.0	24.0	29.0	33.0
Control Beam	22.0	30.0	35.4	37.0

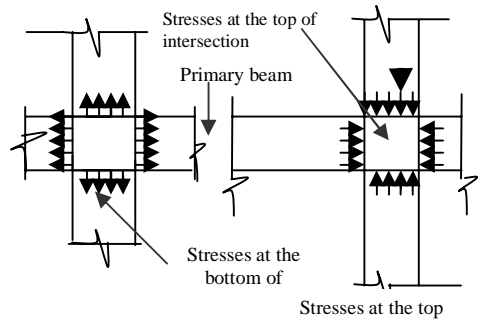


Fig. 22 Mechanics of model [16]

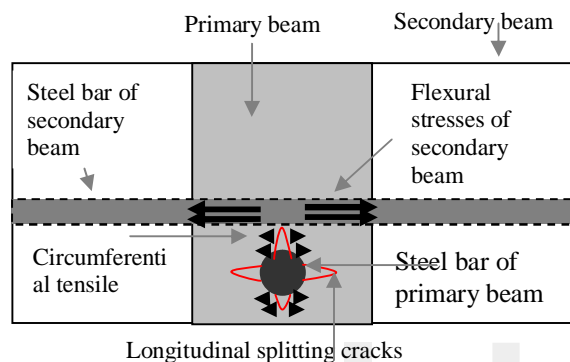


Fig. 23 Stress magnification at intersecting beam [10]

Proposed mathematical model for stress magnification  
In order to calculate the magnitude of the bond splitting stress, Tepfer et al, used the thick walled cylinder analogue to the expanding concrete key over the ribs in the anchorage zone. The inner radius of the cylinder is taken as bar size and out radius as the concrete cover. The radial pressure "p<sub>i</sub>" is equal to tangential tensile stress as shown in Fig.24. In the plane of the steel bar, the radial pressure is a function of concrete bond stress by the assumption of Mohr-Columb failure criterion, these two parameters are defined as "τ" and "p<sub>i</sub>" [14,21]

In the model, steel of radius "R<sub>0</sub>" is embedded in concrete of radius "R<sub>i</sub>", Where as "R<sub>i</sub>" defines the crack front as shown in Fig. 25. At the crack front tangential bond stress is equal to the tensile strength of the concrete f<sub>ct</sub>. Boundary condition is used to calculate "p<sub>i</sub>" in the elastic outer part. The radial and tangential stresses are given by the classic theory of elasticity[13,14,19,20]

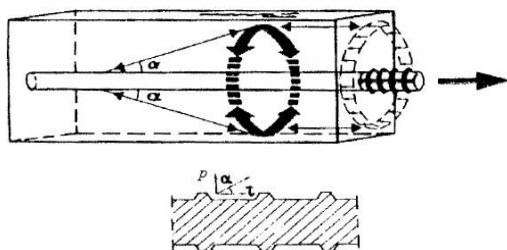


Fig.24 Radial component of the bond stress

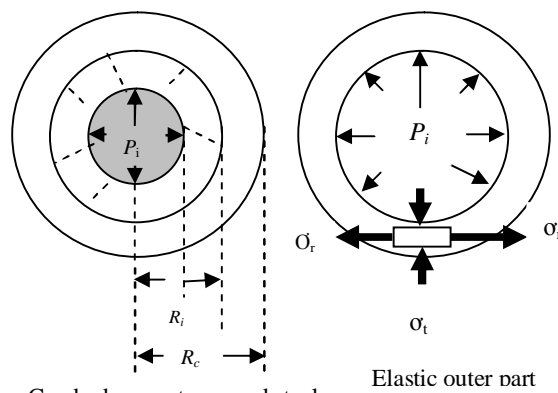


Fig.25 Model for partly cracked concrete [14]

$$p_i = \tau \tan \alpha \tag{8}$$

Assume  $\alpha = 45^\circ$

$$p_i = \tau \tag{9}$$

$$\sigma_t = p_i \frac{R_c^2 + r^2}{R_i^2 - 1} \quad \sigma_r = p_i \frac{R_c^2 - r^2}{R_i^2 - 1} \tag{10}$$

At the crack front  $\sigma_t = f_{ct} \quad r = R_i$

$$\frac{R_c^2 - R_i^2}{R_c^2 + R_i^2} f_{ct} = p_i \tag{11}$$

The flexural stress of the primary beam of the intersecting beam model is given by the bending stress theory and shown in Fig.26. At the joint of the intersecting beams these two stresses add up and magnify the total stress that is responsible for the splitting of the concrete keys in the anchorage zone of the ribs of the steel bar

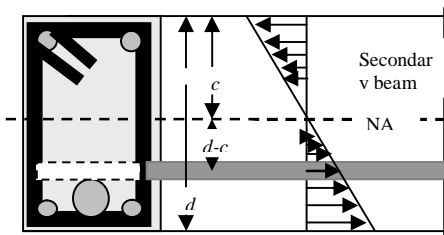


Fig.26 Stress magnification at the joint

$$\sigma_t = \frac{M(d-c)}{I} \tag{12}$$

$$\sigma_t = p_i \frac{R_c^2 + 1}{R_i^2 - 1} + \frac{M(d-c)}{I} \tag{13}$$

6. Comparison with Local Bond Constitutive Model

The bond constitutive model given by Eligehausen et al (1983) ( ascending part adopted by Comitè- International du Beton- Federation International de la Precontrainte Model Code 1990), is compared with the bond behavior of the control beam and primary beam of the intersecting model. The response of the beams is close to splitting bond failure and not to pull out bond failure. The reason for this

behavior is the splitting of the concrete due to tangential bond stress. This local bond model is shown in Fig.27. 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.

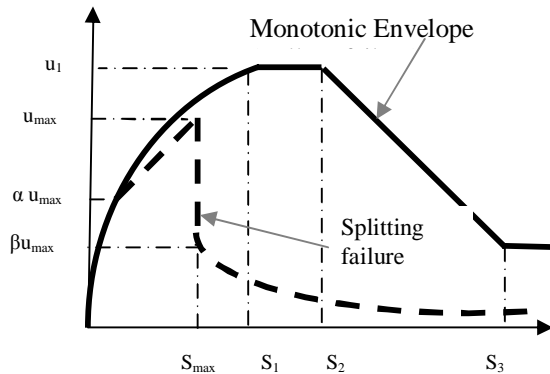


Fig. 27 Bond constitutive model given by Eligehausen et al [8,13]

$$\frac{U}{u_l} = \left(\frac{s}{s_l}\right)^v \quad \text{For Ascending part of the curve (14)}$$

$$\frac{U}{\beta u_{max}} = \left(\frac{s}{s_{max}}\right)^v \quad \text{For descending part of the curve (15)}$$

## 7. Conclusions

1- When bond behaviour of primary beam is compared with control beam of the modal of intersecting beams, then it is clear that bond strength of primary beam reduced as compared to control beam. This result is present incase of all the sets of the beams.

2- The bond strength of primary beam of intersecting model reduced for all strengths of concrete. The magnitude of the reduction is from 9.0% to 21.0 %.

3- This may be attributed to the flexural action of secondary beam that magnified the circumferential tensile bond stresses of primary beam, enhanced the longitudinal splitting cracks and reduced the bond strength of primary beam as compared to control beam.

4-Similarly the bond strength of secondary beam reduced as compared to primary beam. The same circumferential tensile bond stress magnification due to flexural action of secondary beam is responsible for this.

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