Bond Strength-Splice Length in Concrete Beams Confined by Transverse Reinforcement

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Abstract

This work aim to study the effect of transverse reinforcement , area of splice bar, concrete cover thickness , rib area and the increasing in concrete strength (high-strength concrete) on bond strength between concrete and reinforcing spliced bars . Therefore, a new simple equation is derived for beams with spliced bars and confined by transverse reinforcement to calculate bond strength and reflects the effects of these factors .Where many of existing codes and provisions used to calculate the spliced strength do not include or reflect the influencing of these factors in bond strength estimation . Based on experimental results from previous works , (116) confined beams with spliced bars are investigated in this study , where concrete compressive strength (f_c') ranging from 25 MPa to 113.793 MPa ,amount of transverse reinforcement vary in a wide range and , conventional and high relative rib area of deformed bars are present in these beams . The proposed method exceed the limitation of ($f_c' \leq 69 MPa$) that given by ACI code .Where the proposed method is examined and applicable for concrete compressive strength up to 113 MPa . Also, in this work the second root of f_c' is examined , as concrete strength increased with high-strength concrete , to reach a suitable value for both normal and high- strength concrete and to be more appropriate with the heavy present of transverse reinforcement . Power of (0.35) is adopted and used in this work instead of the second root of f_c' .

Keywords: Beams , Concrete , Bond Strength , splice length , transverse reinforcement ,Confinement , Rib Area , high-strength concrete, deformed bar.

قوة الترابط – الطول المتراكب للعتبات الخرسانية المقيدة بحديد التسليح العرضي

الخلاصة

هذا العمل يهدف الى دراسة تاثير كل من حديد التسليح العرضي ، سمك الغطاء الخرساني ، مساحة التشوه في حديد التسليح والزيادة في قوة تحمل الخرسانة (الخرسانة العالية التحمل) على قوة الترابط بين الخرسانة وحديد التسليح المتراكب . ولذلك تم اشتقاق معادلة جديـــدة وبســـيطة للعتبات ذات حديد التسليح المتراكب والمقيدة بحديد التسليح العرضي لحساب قوة الترابط وعكس تاثيرات هذه العوامل على قوة الترابط . حيث ان العديد من المواصفات والطرق المتبعة في حساب قوى الترابط لاتتضمن او تعكس تاثيرات هذه العوامل في حساب قوة الترابط . بالاعتماد علـــى النتائج العملية لدراسات سابقة فان (116) عتبة ذات تسليح متراكب ومقيدة عرضيا تمت دراستها

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في هذا العمل ، حيث ان مقاومة انضغاط الخرسانة (f'_c) لهذه العتبات تتراوح بين 25.0 MPa و 113.8 MPa ، كمية حديد التسليح العرضي متغيره ضمن مدى واسع ، مساحة الشتوه لحديد التسليح المتراكب المتوفرة في هذه العتبات تتراوح بين التقليدية والمرتفعة . ان الطريقة المقترحة في هذا العمل تتجاوز حد ($f'_c \geq$ 69 MPa) المحدد في المواصفات الامريكية ، حيث ان الطريقة المقترحة تم اختبارها وطبقت لمقاومة انضغاط تصل الى 113 MPa . وعلى النقيض من الطرق الاخرى التي تستعمل اس مختلف ل f'_c فان الطريقة المقترحة تستخدم $f'^{o.35}_c$, والذي يودي الى الحصول على معامل تباين (COV) مقداره 26.2 بالمئة مقابل 31.6%-44.2% للطرق الاخرى .

Introduction

Over the years , the estimation of bond strength between concrete and reinforcing bars has been improved .This improvement added a significant knowledge on bond behavior. The accuracy of the bond strength prediction increased and improved due to the increase in available test results that recently studied the effect of many factors on bond strength . The effect of admixtures of silica fume and other admixtures , the effect of lightweight aggregate and the effect of epoxy-coated reinforcing bars has been studied for their influence on bond strength [1-5] . In the last few years , high-strength concrete (HSC) has gained popularity for diverse applications such as bridges ,tall buildings , pavements ,etc .As the effect of concrete strength has been investigated [2,6-8] , a comparison of the effects on bond strength of normal-strength concrete (NSC) and HSC has been made . Research indicates that bond strength increased with the increase of compressive strength of concrete and cover thickness [4,6] . The extent of damage at the steel-concrete interface depends on concrete strength and bar deformation pattern[2].This damage was more extensive near the discontinuous ends of splices[6].

Bond failure of ribbed reinforcing bars generally involves splitting of the surrounding concrete cover unless heavy confinement reinforcement are present .Splitting failure results from a fracture of the concrete along the bar due to the lateral tension caused by wedging action provided by the bar deformations [6,9-13] .

The use of transverse reinforcement is useful for ductility problems, where the unconfined beams tend to fail in more brittle mode than the confined ones-especially , when HSC is used , in which the drop in ductility is obvious [6,12] .

Azizinamini et al.[13] studied the effect of HSC on bond in spliced beam tests . When results indicate that the average bond stress values along the spliced reinforcing bars normalized with respect to the square root of concrete compressive strength ($\sqrt{f'_c}$),the concrete strength influence decreased with rising f'_c . The rate of decrease becomes more pronounced as the splice length increases . They also noticed that the bearing capacity of concrete is related to f'_c while tensile capacity is related to ($\sqrt{f'_c}$) . HSC development failure in beams proved to be more brittle than with NSC .Therefore a suitable amount of transverse reinforcement must be used especially in the case of HSC . Zuo et al.6 have applied the influence of f'_c in their proposed design .Instead of using $\sqrt{f'_c}$,they proposed using $f_c'^{1/4}$.

Research Significance

This work intends to give a better understanding of the influence on bond strength (bond between concrete and reinforcing bars being spliced or developed) of the different factors, by providing a new simple and more accurate approach . Concrete strength properties , availability of transverse reinforcement within the splice region and the relative rib area are studied and examined . The proposed equations effect of these factors on the estimation of the splice strength.

Also, the use of the square root of concrete compressive strength ($\sqrt{f'_c}$)examined and the effect of concrete strength on this term is studied .

Bond Strength Mechanism For Deformed Bars

When deformed reinforcing bars are embedded in concrete , their development is provided by chemical adhesion , friction between the reinforcing bars and the surrounding matrix , and bearing against the face of the ribs . The total bond force is the sum of the components of the bearing and friction forces on the rib acting parallel to the reinforcing bar axis.

Although, adhesion and friction are present when a deformed bar is loaded for the first time , these bond transfer mechanisms are quickly lost leaving the bond to be transformed by bearing on the deformations of the bars ,Fig.(1a). Equal and opposite bearing stresses act on the concrete ,Fig.(1b) .The forces on the concrete have both a longitudinal and a radial component , Fig.(1 c and d) .The concrete will split parallel to the bar and the resulting crack will propagate out to the side or bottom surface of the beam[14] .

Nominal Bond Strength Estimation

Generally , for any reinforced concrete beam , flexural tensile forces are provided by reinforcing bars . Therefore there must be a force transfer (bond) between the two materials . Internal forces acting in the beam and forces acting in the reinforcing bars are illustrated in Fig.(2a and 2b)[14] . When this bond strength is lost the reinforcing bar will pull out of the concrete and the tensile force (T) will drop to zero causing the beam to fail .Stresses or forces in the reinforcing bars vary from point to pint along the length of bar, Fig.(3) [14] . If f_{s2} is greater than f_{s1} ,bond strength (u) must act on the surface of reinforcing bar to ensure equilibrium . By summing all the forces parallel to the reinforcing bar the average bond strength will be[14]

$$(f_{s2} - f_{s1})\frac{pd_b^2}{4} = u_{avg.}(pd_b)L \qquad ....(1)$$

And

$$(f_{s2} - f_{s1}) = \Delta f_s \qquad ....(2)$$

By substituting Eq.(2) in Eq.(1) then

$$u_{avg.} = \frac{\Delta f_s d_b}{4L} \qquad ....(3)$$

Where the use of term ($u_{avg.}$) is due to the non-uniform distribution of bond strength along the reinforcing bar[14,15] .

When a reinforcement bar reaches the yield stress , the term Δf_s will be replaced by steel yield stress (f_y), Eq.(4)

$$u_{avg.} = \frac{f_y d_b}{4L} \qquad ....(4)$$

Bond Strength-Splice Length, In ACI 08 [16] and Previous Provisions

Many studies tried to obtain a reliable formula that gives a suitable estimation for the bond strength between the concrete and the reinforcing bars that are being spliced or developed . Also, several equations have been derived to determine the splice or development length . As for the ACI code 08 [16] , the estimation of splice or development length has been expressed in a reliable formula Eq.(5)

$$l_d = \left(\frac{f_y}{1.1\sqrt{f_c'}} \frac{y_t \quad y_e \quad y_s \quad l}{\left(\frac{c_b + k_{tr}}{d_b} \right)} \right) d_b \qquad ....(5)$$

In which , the term $\left(c_b + k_{tr}/d_b\right)$ shall not be taken greater than 2.5 , the product of $y_t \quad y_e$ do not exceed 1.7 , and

$$k_{tr} = \frac{A_{tr} \quad f_{yt}}{10 \quad s \quad n} \qquad ....(6)$$

Other studies proposed different approaches to predict the splice/development length and the bond strength .

Esfahani et al.[8] provided a set of equations to calculate the bond strength and the splice length

$$u = u_c \frac{1 + 1/M}{1.85 + .0024\sqrt{M}} \left(.88 + .12 \frac{C_{med}}{C} \right) \left(1 + .015 \frac{A_t \quad A_b}{C \quad S} \right) \qquad ....(7)$$

Where A_t in this provision represents the area of one transverse reinforcing bar .

In which

$$u_c = 2.7 \frac{C/d_b + .5}{C/d_b + 3.6} \sqrt{f_c'} \quad \text{for NSC} \qquad ....(8)$$

$$u_c = 4.7 \frac{C/d_b + .5}{C/d_b + 5.5} \sqrt{f_c'} \quad \text{for HSC} \qquad ....(9)$$

$$M = \cosh\left(.0022L \sqrt[3]{\frac{f_c'}{d_b}} \right) \qquad ....(10)$$

Also, Esfahani et al.[8] derived another equation to obtain the development / splice length as in Eq.(11)

$$L = \frac{T}{a\sqrt{f_c'}} = \frac{A_b \quad f_s}{a\sqrt{f_c'}} \qquad ....(11)$$

Where :

$$a = 7.2 d_b \frac{C/d_b + .5}{C/d_b + 3.6} \qquad ....(12)$$

And f_s is the tensile stress in the reinforcing bar at failure .Conservatively f_s can be replaced by f_y which also provides a margin of safety [8] .

Zuo et al.[6] obtained a new design expression for the splice length ,in which the effects of concrete strength , coarse aggregate quantity , type and reinforcing bar geometry and amount of transverse reinforcement are evaluated . The power ¼ of compressive strength f_c' best characterizes the effect of concrete strength on splice strength without transverse reinforcement . While the power ¾ characterizes the effect of concrete strength on the additional splice strength provided by transverse reinforcement .Reference 6 gives :

$$\frac{l_d}{d_b} = \frac{\dfrac{f_y}{f_c'^{\,1/4}} - 241.7}{-0.011\left(\dfrac{c + K_{tr}}{d_b}\right)}$$

….(13)

Canbay et al.[17] tried to derived a simple equation to estimate the splice

length based on physical model of tension cracking of concrete in the lap spliced region .

$$\frac{l_d}{d_b} = 1.23\times10^{-5}\,\frac{f_y^2\,\sqrt{d_b}}{\sqrt{f_c'}}$$

(changed to SI units)….(14)

New Approach for Bond Strength Calculation

In order to obtain a better understanding for bond between concrete and reinforcing bars in spliced beams, a new equation is derived to calculate the bond strength and try to reflect the effect of different factors on bond behavior . Transverse reinforcement , concrete strength , cover thickness and the rib area of bars have been considered and represented in the proposed equation . The new formula is more simple than others6,8,17except for ACI 0816 . The effect of transverse reinforcement is represented as total area along the spliced region(At) .And the effect of transverse reinforcement is related with the spacing between stirrups within the spliced length (S) . Cover thickness, also has an effect on bond strength of concrete surrounding the spliced bars.The proposed equations examine this effect . A term of $\left(\left(\dfrac{A_b}{A_t}\right) \cdot \left(\dfrac{C}{S}\right)\right)$ is given by the factor (K),Eq.(15) . Where (Ab) is the individual splice bar diameter , and (C) factor represent the minimum concrete thickness concrete surrounding the spliced bars .

$$K = \left(\frac{A_b}{A_t}\right) \cdot \left(\frac{C}{S}\right) \qquad \text{…(15)}$$

Where , factor (K) reflects the influence of the surrounding matrix of spliced bar . Factor (K) can be used only for spliced beams confined by transverse reinforcement. This factor will be added to the nominal bond strength (u_o) to get the final bond strength expression , Eqs.16-19

$$u_o = \frac{f_y \cdot d_b}{4\,L}$$

…(16)

$$u = u_o + K$$

…(17)

$$u = \frac{f_y \cdot d_b}{4\,L} + \left[\left(\frac{A_b}{A_t}\right) \cdot \left(\frac{C}{S}\right)\right]$$

…(18)

Where $(K = 0)$ for beams not confined by transverse reinforcement . By using regression analysis a factor (H) is found to reflect the increase in concrete strength(in case of HSC) . This factor will vary as concrete strength increased .In order to simplify the proposed equation , this factor can be taken as a constant value equal to (1.176) . Therefore Eq.(18) will be written as follows :

$$u = \frac{f_y \cdot d_b}{4\,L} + \left[\left(\frac{A_b}{A_t}\right) \cdot \left(\frac{C}{S}\right)\right] H$$

…(19)

Also, in this work the power of f_c' is examined , as concrete strength increased with HSC , to reach a suitable value for both (NSC & HSC) and to be more appropriate with the heavy presence of transverse reinforcement . Power of (0.35) is used instead of the second root of f_c' , Fig.(4). Fig.(4) $f_c'^{\,o.35}$ gives a more reasonable fit linear line than ($\sqrt{f_c'}$) ,where the power of 0.5 tends to behave unconservatively with the increase of f_c' . Therefore , in this work $f_c'^{o.35}$ will be adopted instead of ($\sqrt{f_c'}$) for splice length calculation Eq.(20) .

$$L = \left(\frac{f_y}{f_c'^{0.35}} \frac{y_t \; y_e \; y_s \; l}{\left(\dfrac{C \; + \; k_{tr}}{d_b} \right)} \right) d_b \qquad \ldots(20)$$

Bond Strength and the Effect of Transverse Reinforcement Within Splice Region

Generally , transverse reinforcement has a significant effect on bond strength due to the ductility problems . For spliced not confined beams the failure occurred suddenly , with a quick drop in load after the peak . In contrast, beams confined by transverse reinforcement gave a more ductile behavior , with a slow drop in load after the peak . And failure occurred in more ductile manner .When HSC is used in spliced beams the failure occurred in a more brittle manner than beams made with NSC[6].

From the above , the important effects of transverse reinforcement are obvious , as a solution for ductility problems . Confinement is more essential for HSC than NSC .

This work studies the effect of transverse reinforcement on bond strength , in which NSC and HSC are used , by using experimental data from existing research .

Fig.(5) illustrates the relation between experimental bond strength and calculated bond strength (u_{test} / u_{calc}) , and the total area of transverse reinforcement (A_t) along the spliced bars . Where an obvious raise in bond strength ratio (u_{test} / u_{calc}) is noticed with the increase of transverse reinforcement . As well as , for f_c' , Fig.(6).

The effect of transverse reinforcement present shall not be taken without the use of HSC considerations , where both of transverse reinforcement and HSC have a simultaneous effects on bond between concrete and reinforcing bars .

Effect of Concrete Strength , Rib Area and Cover Thickness on Bond Strength

Bond failure at the concrete-steel interface occurred due to the concrete crushing at the face of rib . Concrete damage depended on the concrete strength and bar deformation pattern. Usually damage is more extensive near the discontinuous ends of splices[6] .For conventional bars , concrete crushed between the bars ribs .In contrast, high rib area concrete both crushed and sheared[10] . In confined beams with HSC, concrete damage at the interface surface is similar in NSC beams , but the damage occurred over a longer region in the former[6] . In general, the use of high rib deformed bars reduction the splice length for both NSC and HSC .

Fig.(7) provides a relationship between area of individual splice bar (A_b) and (u_{test} / u_{calc}) , where the effect of (A_b) on bond strength for beams confined by transverse reinforcement is illustrated .

Similarly , concrete thickness factor (C) have a great effect on the splice bond strength , in which the increase in concrete cover thickness has provides an increase in bond strength . This increased in bond strength is limited because a pullout failure is expected and the increase in cover is unlikely to increase the anchorage capacity[16].

Discussion and Conclusions

This work studies the effect of different factors on bond strength between concrete and reinforcing bars . The influence of transverse reinforcement , area of individual splice bar (A_b) , concrete cover , rib area and concrete strength (NSC & HSC) are investigated by using experimental results from previous works [2,6,7,11,18,19] .This study tries to cover a very wide range of these factors values . Beams with different amounts of transverse reinforcement along splice region are studied , where transverse reinforcement varies in a very wide range .Concrete compressive strength (f_c') varies in a range of (25.0 MPa to 113.8 MPa) to give a realistic indication to the increase in concrete strength behavior , deformed bars with conventional and high relative rib area has been covered and beam geometrical dimensions are also varied in a wide range .Table (1) summarizes the details and tested bond strength for the beams studied in this work .

Several provisions are used to estimate the bond between concrete and reinforcing spliced bars , including the proposed equations in this work . A statistical program is used to calculate the coefficient of variation (COV) , in order to describe the results of each method . The proposed equations give the lowest COV (26.145%) with mean value of (1.846) of bond strength ratio (u_{test}/u_{calc}), and with no beams failing ($u_{test}/u_{calc} < 1$) . ACI 08 [16] have COV of (31.573%) with a very low mean value of (1.059) where several beams failed ($u_{test}/u_{calc} < 1$) . Esfahani [8] provision have the highest COV (44.189%) and a mean value of (1.263) , also this provision is very complicated and difficult especially for design purposes. Darwin[6] and Canbay[17] have COV values of (34.733%) and (35.287%) (which are also high values) with mean of (1.709) and (2.866) respectively , where Canbay[17] has the highest mean value and tends to be extremely unconservative with the increased of (f_c') and (A_t) respectively .Table (2) summarizes the results of COV and mean values for all provisions studied in this work.

From all above, the following facts can be concluded :

1. Bond strength between concrete and reinforcing bars is highly affected by the amount of transverse reinforcement , where the bond strength increases with the increase in transverse reinforcement amount, Fig.(5).And the failure occurred in more ductile manner.
2. Bond strength as well as splice length for beams confined by transverse reinforcement increase with the increase in splice bar diameter .
3. The proposed equations have a good predicted bond strength with the increase of concrete strength ,and especially for HSC Fig.(6) .The proposed method is examined and is applicable for concrete compressive strength up to 113 MPa , despite the fact that the

proposed method exceed the limitation of ($f_c' \leq 69MPa$) that given by ACI 08 [16] .

4. For beams with HSC, heavy transverse reinforcement is highly recommended in order to avoid the brittleness problem and to get the maximum benefit from the use of HSC for bond strength.
5. The term $f_c'^{o.35}$ is used instead of the second root of f_c' (or other powers)in order to reflect the effect of HSC and the heavy present of transverse reinforcement.

Notations

A_b = area of an individual bar being spliced or developed .

A_{tr}, A_t = total cross-sectional area of all transverse reinforcement within the reinforcement length being spliced or developed .

C, c_b,c = smaller of (a) the distance from center of a bar to nearest concrete surface , and (b) one-half the center-to-center spacing of bars being spliced or developed .

C_{med} = median of side and bottom concrete cover and one-half the center-to-center spacing of bars being spliced or developed .

COV = coefficient of variation .

d_b= longitudinal reinforcing bar diameter .

d_t = transverse reinforcing bar diameter .

f_c' = concrete compressive strength .

f_{s1} and f_{s2} =stresses acting in the reinforcement bar .

f_y = specified steel yield strength .

f_{yt} = specified yield strength of transverse reinforcement .

H = factor used to reflect the effect of high-strength concrete on bond strength .

HSC = high-strength concrete .

L,l_d= splice or development length .

n = number of bars being spliced or developed .

N = number of transverse reinforcement within the splice region .

NSC = normal-strength concrete .

S,s= center-to-center spacing of transverse reinforcement .

u = calculated bond strength along the reinforcement bar being spliced .

$u_{avg.}$=average bond strength along the reinforcement bar .

u_o = nominal bond strength between concrete and reinforcing bars .

u_{test} = experimental (measured)bond strength .

λ = modification factor related to density of concrete .

ψ_e = factor used to modify spliced or development length based on reinforcement coating .

ψ_s = factor used to modify spliced or development length based on reinforcement size .

ψ_t = factor used to modify spliced or development length based on reinforcement location .

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Table (1) Details and test result for splice beams confined by transverse reinforcement

beam No.	beam name or no.	f'_c (Mpa)	d_b (mm)	n	d_t (mm)	N	u_{test} (Mpa)	Ref.
1	19.3 NNL	29.310	25.4	3	9.525	3	4.101	6
2	20.1 NNL	35.035	35.814	3	12.7	8	4.316	6
3	20.3 NNL	35.035	35.814	3	12.7	5	4.16	6
4	21.1 NNL	29.862	25.4	3	15.875	6	5.302	6
5	21.5 NNL	29.862	25.4	2	12.7	5	5.329	6
6	23a.1HHL	62.621	25.4	3	9.525	4	6.469	6
7	21.3 NNL	29.862	25.4	3	15.875	5	5.254	6
8	23a.4HHL	62.621	25.4	3	9.525	4	6.492	6
9	23b.1HHL	57.724	25.4	3	12.7	5	7.564	6
10	23b.5HHL	31.035	35.814	2	12.7	5	5.324	6
11	27.2 HHL	74.552	25.4	3	9.525	6	5.88	6
12	27.4 HHL	74.552	25.4	3	12.7	5	7.389	6
13	27.6 HHL	74.552	25.4	3	12.7	4	7.504	6
14	28.1 HHL	86.966	35.814	2	9.525	5	6.92	6
15	28.3 HHL	86.966	35.814	3	9.525	4	5.814	6
16	29.2 HHB	73.241	25.4	3	9.525	3	7.204	6
17	29.4 HHB	73.241	25.4	3	9.525	6	7.46	6
18	29.6 HHB	73.241	25.4	3	9.525	4	8.367	6
19	30.1 HHB	91.172	35.814	2	9.525	3	6.419	6
20	30.3 HHB	91.172	35.814	3	9.525	2	5.801	6
21	31.3 HHB	88.897	25.4	2	9.525	2	7.02	6
22	33.2 NHL	36.966	25.4	3	12.7	6	5.878	6
23	33.3 NHL	36.966	25.4	3	9.525	4	5.512	6
24	33.4 NHL	36.966	25.4	3	9.525	4	5.581	6
25	33.6 NHL	36.069	25.4	2	9.525	2	4.536	6
26	35.1 NNL	36.759	25.4	2	9.525	5	5.894	6
27	35.3 NNL	36.759	25.4	2	9.525	5	5.32	6
28	37.4 NNL	33.103	25.4	3	12.7	7	6.052	6
29	39.2 HHB	99.655	25.4	3	9.525	4	7.508	6
30	39.3 HHB	99.655	25.4	3	9.525	4	8.393	6
31	40.1 HHB	107.931	35.814	2	9.525	4	7.033	6
32	40.4 HHB	107.931	35.814	2	9.525	4	6.212	6
33	41.1 HHL	70.207	25.4	2	9.525	2	7.123	6
34	41.2 HHL	70.207	25.4	3	15.875	4	8.938	6
35	41.3 HHL	70.207	25.4	3	12.7	4	8.543	6
36	41.4 HHL	70.207	25.4	3	15.875	4	8.319	6
37	41.6 HHL	72.414	25.4	3	9.525	2	7.039	6
38	42.1 HNL	82.276	25.4	2	9.525	2	6.768	6
39	42.4 HNL	82.276	25.4	3	12.7	4	7.611	6
40	42.5 HNL	82.276	25.4	3	15.875	4	8.389	6

Table (1) Continued

beam No.	beam name or no.	f'_c (Mpa)	d_b (mm)	n	d_t (mm)	N	u_{test} (Mpa)	Ref.
41	43.2 HNL	79.517	25.4	2	9.525	2	6.992	6
42	43.3 HNL	79.517	25.4	3	12.7	4	8.484	6
43	43.6 HNL	79.517	25.4	3	15.875	4	8.906	6
44	8N3-16-2-U	41.310	25	3	9.5	2	5.987	18
45	8N3-16-1-C	41.310	25	3	9.5	1	4.048	18
46	8N3-16-2-C	41.310	25	3	9.5	2	4.525	18
47	8C3-16-2-U	42.759	25	3	9.5	2	4.683	18
48	8C3-16-2-C	42.759	25	3	9.5	2	4.027	18
49	8S3-16-2-U	41.517	25	3	9.5	2	4.938	18
50	8S3-16-2-C	41.517	25	3	9.5	2	3.391	18
51	8S3-16-2-C	41.517	25	3	9.5	2	3.211	18
52	8S3-16-2-U	44.483	25	3	9.5	2	5.023	18
53	8S3-16-3-U	44.483	25	3	9.5	3	5.34	18
54	8S3-16-2-C	44.483	25	3	9.5	2	3.401	18
55	8S3-16-3-C	44.483	25	3	9.5	3	3.264	18
56	8C3-16-2-U	37.862	25	3	9.5	2	4.938	18
57	8C3-16-3-U	37.862	25	3	9.5	3	4.578	18
58	8C3-16-3-C	37.862	25	3	9.5	3	3.677	18
59	8C3-22¾-3-U	40.345	25	3	9.5	3	4.188	18
60	8C3-22¾-4-U	40.345	25	3	9.5	4	4.143	18
61	8C3-22¾-3-C	40.345	25	3	9.5	3	2.653	18
62	8C3-22¾-4-C	40.345	25	3	9.5	4	2.87	18
63	8C3-16-3-U	36.138	25	2	9.5	3	5.457	18
64	8C3-16-3-C	36.138	25	2	9.5	3	5.4.102	18
65	1a	27.500	25.2	2	6.35	6	4.099	19
66	3a	27.500	25.2	3	6.35	6	3.814	19
67	4a	27.800	29.9	3	6.35	4	3.397	19
68	1b	26.200	25.2	2	6.35	6	3.881	19
69	3b	26.200	25.2	3	6.35	6	3.452	19
70	4b	25.700	29.9	3	6.35	5	3.003	19
71	6	25.000	25.2	3	7.94	8	3.971	19
72	7	25.000	25.2	3	16	4	5.443	19
73	8	25.000	25.2	3	16	3	5.019	19
74	9	26.800	29.9	3	11.3	10	4.292	19
75	10	28.200	29.9	3	16	7	6.194	19
76	C1S0	51.400	25	2	10	1	7.87	2
77	C2S0	65.000	25	2	10	2	8.98	2
78	C3S0	65.100	25	2	10	3	10.11	2
79	C1S8	75.900	25	2	10	1	7.48	2
80	C2S8	80.200	25	2	10	2	8.16	2

Table (1) Continued

beam No.	beam name or no.	f_c' (Mpa)	d_b (mm)	n	d_t (mm)	N	u_{test} (Mpa)	Ref.
81	C3S8	75.600	25	2	10	3	8.98	2
82	C1S16	98.400	25	2	10	1	5.84	2
83	C2S16	96.200	25	2	10	2	8	2
84	C3S16	81.300	25	2	10	3	8.99	2
85	70-L300-9S1	71.400	28.7	2	9.5	5	11.77	7
86	70-L200-9S1	61.700	28.7	2	9.5	3	15.4	7
87	70-L200-9S2	72.000	28.7	2	9.5	3	11.67	7
88	55-L300-9S1	59.400	28.7	2	9.5	5	9.02	7
89	55-L300-9S2	54.400	28.7	2	9.5	5	10.49	7
90	55-L200-9S1	58.200	28.7	2	9.5	3	12.45	7
91	55-L150-9S1	56.700	28.7	2	9.5	3	13.53	7
92	40-L300-9S1	48.000	28.7	2	9.5	5	8.63	7
93	9	110.366	25.4	2	10	7	7.641	11
94	11	108.372	25.4	2	10	4	7.641	11
95	13	109.545	25.4	2	10	5	5.814	11
96	22	108.372	25.4	2	10	6	12.736	11
97	24	108.372	25.4	2	10	3	12.736	11
98	26	109.545	25.4	2	10	4	9.791	11
99	27	108.255	25.4	2	10	3	10.054	11
100	14	108.255	25.4	2	10	4	5.97	11
101	10	110.366	25.4	2	10	5	7.641	11
102	49	108.69	36	3	10	9	6.902	11
103	51	108.621	36	3	10	8	5.47	11
104	52	102.414	36	3	10	7	6.135	11
105	54	104.134	36	3	10	13	4.801	11
106	55	104.134	36	3	10	9	4.801	11
107	57	113.793	36	3	10	6	4.801	11
108	61	104.766	36	2	10	9	13.913	11
109	62	110.366	36	2	10	5	13.529	11
110	63	110.366	36	2	10	4	13.529	11
111	65	100.538	36	2	10	6	11.274	11
112	67	106.324	36	2	10	8	9.938	11
113	68	106.324	36	2	10	6	9.938	11
114	69	108.255	36	2	10	4	9.663	11
115	70	108.255	36	2	10	3	9.663	11
116	53	102.690	36	3	10	5	4.33	11

Table (2) Statistical results for provisions studied in this work

Statistical measurements	Provisions studied in this work				
	Proposed method (Eqs. 19 and 20)	ACI 08	Darwin 2000	Esfahani 05	Canbay 06
COV. %	26.145	31.573	34.733	44.189	35.287
Mean (average)	1.846	1.059	1.709	1.263	2.866
Max.	3.223	2.281	3.935	2.251	6.793
Min.	1.014	0.497	0.760	0.513	1.240
$\frac{Max.}{Min.}$	3.179	4.590	5.178	4.388	5.478

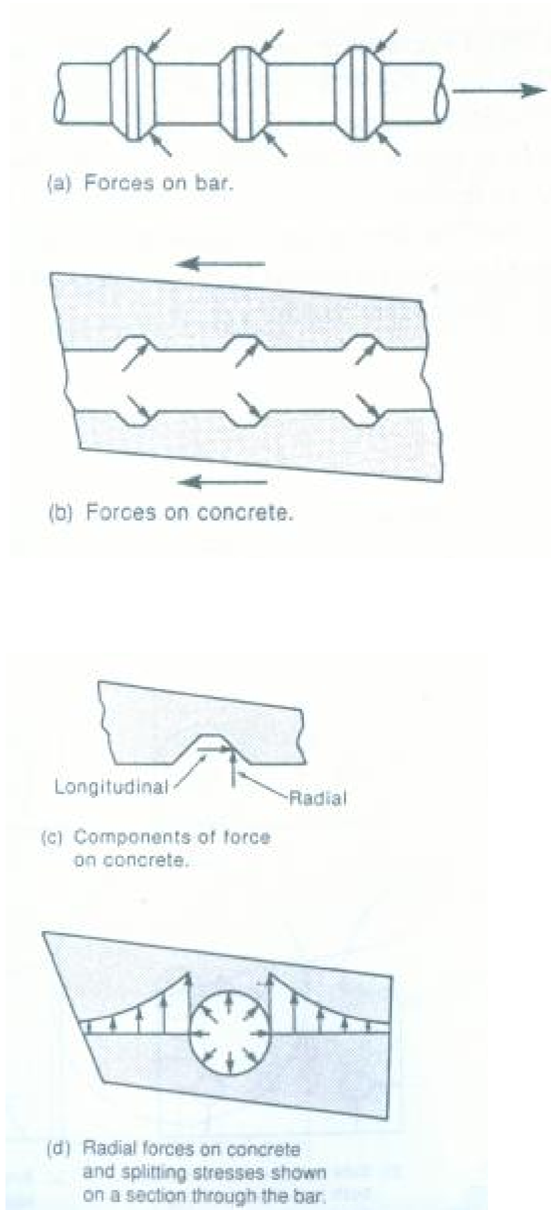

Figure (1) [14] Bond transfer mechanism .

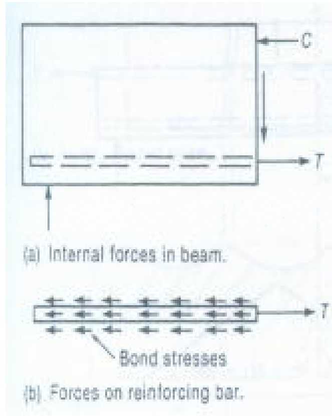

Figure (2) [14] Need for bond stresses .

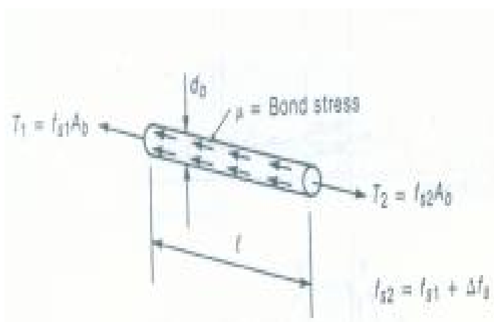

Figure (3) [14] Relationship between change in bar stress and average bond stress .

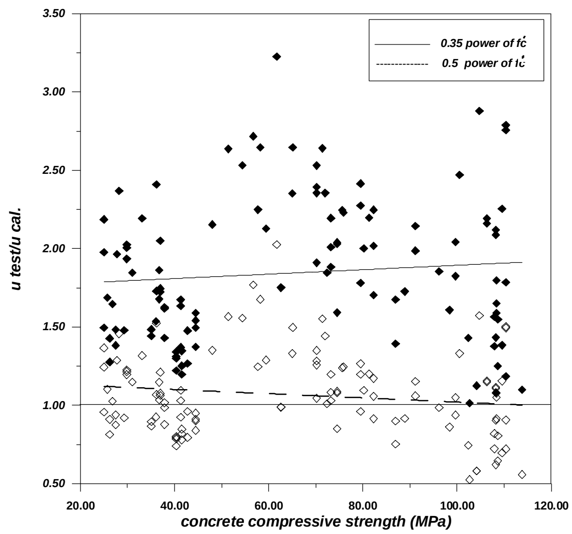

Figure (4) Concrete compressive strength powers .

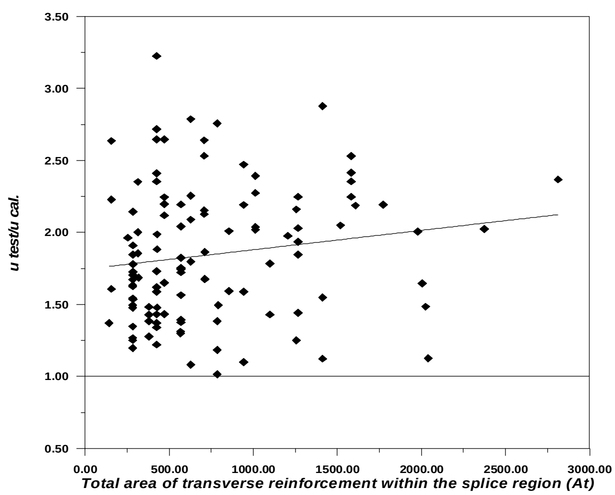

Figure (5) Relationship between bond strength ratio and the total area of transverse reinforcement (A_t) mm^2 along the spliced bars.

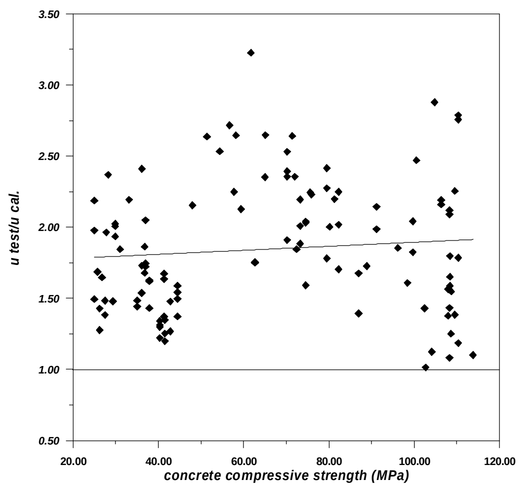

Figure (6) Effect of concrete compressive strength (f_c')MPa on the bond strength ratio .

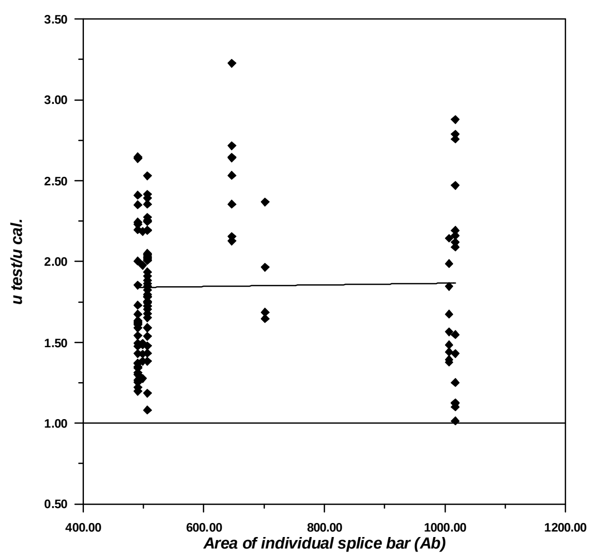

Figure (7) Relationship between bond strength ratio and the area of individual splice bar (A_b) mm^2.