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Top-bar effect of steel bars in self-consolidating concrete (SCC)

M. Reza Esfahani [a], M. Lachemi [b], M. Reza Kianoush [b,*]

[a] Ferdowsi University, Mashhad, Iran
[b] Department of Civil Engineering, Ryerson University, 350 Victoria Street, Toronto, Ont., Canada M5B 2K3

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Abstract

Self-consolidating concrete (SCC) is known for its excellent deformability, high resistance to segregation, and use in congested reinforced concrete structures characterized by difficult casting conditions without applying vibration. The bond characteristics of such SCCs are very important for their application in practical construction. This paper presents the results of an investigation dealing with the local bond strength between self-consolidating concrete and steel reinforcing bars. Twelve concrete specimens grouped in two series made of normal concrete (NC) and SCC were tested. In each specimen, three steel bars were embedded in concrete at different locations: bottom, middle and the top of the specimen. Cover thickness varied for different embedded bars. Bond strength and slip of each steel bar were measured in a pullout test. The type of bond failure was by splitting for all specimens. Test results were compared with the values calculated by two proposed equations and ACI 318 Code. Based on the results of this study, the local bond strength of top bars for SCC is about 20% less than that for NC. For the bottom bars, however, the results were almost the same. Comparison of the local bond strength between test results with the values calculated by ACI 318 Code shows that in the case of SCC, the location factor of ACI Code should be increased.

Keywords: Bond strength; Flowable concrete; Pull-out test; Reinforcing bars; Self consolidating concrete; Top-bar effect

1. Introduction

The consolidation of fresh concrete in congested reinforced concrete members is an important consideration in concrete placement and durability of structures. Achieving proper consolidation can require internal and external vibration. Inadequate consolidation can lead to surface and structural defects and improper bond development with the reinforcement. With the increase in use of high concentration of rebars in reinforced concrete (RC) structures, there is a growing interest in specifying high workability concrete. The required workability for casting concrete depends on several factors, such as the type of construction, the selected methodology of placement and consolidation, the shape of formwork, and the amount of reinforcement.

Self-consolidating concrete (SCC) is a specially proportioned concrete that can flow under gravity and fill in the formworks without the need of any internal or external vibration. While being highly fluid, SCC needs to be sufficiently cohesive as well to prevent segregation, bleeding and blockage of aggregates during flowing. The enhanced cohesiveness can ensure better suspension of solid particles in the fresh concrete and, therefore, good deformability and filling capability during the spread of fresh concrete around various obstacles [1,2]. In general, the mixture proportion of SCC includes mineral additives, such as fly ash and slag, as well as chemical admixtures, such as high-range water reducing (HRWR) admixtures and/or viscosity modifying agents (VMA), to adjust its deformability and cohesiveness [3].

* Corresponding author. Tel.: +1 416 979 5000x6455; fax: +1 416 979 5122.
E-mail address: kianoush@ryerson.ca (M. Reza Kianoush).

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The use of SCC can remarkably lower the complexity of construction by reducing the demand for a significant amount of consolidation practice and skillful workmanship. Therefore, the self-consolidation characteristics of SCC allow a much easier construction schedule and result in a more reliable quality in concrete placement and a more homogeneous material structure. Without consolidation, the influence of intrinsic deficiencies and material defects due to bleeding or segregation induced by improper vibration practice can be avoided [4]. As a result, the homogeneity of SCC can be ensured and may substantially enhance the mechanical properties of RC members. In other words, the uncertainties in RC structures caused by construction factors can be effectively eliminated. Therefore, the designed structural performance and the expected durability can be enhanced.

In a recent study dealing with pull-out tests, Chan et al. [5] have reported that, as compared to normal concrete (NC), SCC exhibits higher bond to reinforcing bars and lower reduction in bond strength due to top-bar effect.

Khayat [6] studied the bond strength of SCC with special focus on the effect of VMA (welan gum) to reduce the top-bar effect of anchored bars. Accumulation of bleed water under the reinforcement and separation of fresh paste from the reinforcement due to segregation and settlement can significantly reduce the bond. A total of 25 specimens were prepared by Khayat [6] to evaluate the effect of specimen height (500, 700 and 1100 mm) and bar anchored length (2.5 and 5 times bar diameter) on external bleeding, surface settlement, segregation and relative bond strength (from pullout tests) of horizontally embedded bars. The findings indicated that the use of VMA reduced surface settlement (that is related to bleeding and segregation) and significantly reduced the top-bar factor. Sonebi et al. [7] performed bond tests (pull out tests) with 12- and 20-mm deformed bars placed in concrete specimens of $100 \times 100 \times 150$ mm to study the performance of SCC compared to NC. The test results showed 10–40% higher normalized bond strength in SCC compared to NC.

This paper describes pullout tests and compares the local bond strength of SCC and NC. Twelve concrete specimens were manufactured from each concrete type. SCC specimens were non-vibrated, while the control specimens with NC were cast by conventional procedures with substantial consolidation energy. Each specimen included three reinforcing bars embedded in concrete close to sample edges. To insure a uniform bond stress distribution over the reinforcing bars, short embedded lengths were used. Because the concrete cover for each reinforcing bar was small, bond failure was due to splitting of concrete in all tests. This type of bond failure is common in RC beams and slabs. The variation of local bond strength along the sample height (vertical casting) of pullout specimens is compared for both SCC and NC taking into account the concrete compressive strength and top-bar effect. The measured local bond strength of reinforcing bars is used to evaluate the feasibility of using SCC in RC construction.

2. Bond strength in reinforced concrete members

In many of the studies reported in literature, the comparison between SCC and NC is performed in terms of normalized bond strength $u/\sqrt{f'_c}$ in different tests and based on the assumption that the normalized bond strength $u/\sqrt{f'_c}$ does not depend on concrete compressive strength f'_c. For instance in the study done by Chan et al. [5], the difference between the compressive strength of SCC and NC was significant. As previously shown [8–10], the normalized bond strength $u/\sqrt{f'_c}$ may be dependent on the concrete compressive strength f'_c. Therefore, in cases where the compressive strengths of concretes are significantly different, the use of the normalized bond strengths $u/\sqrt{f'_c}$ may not be appropriate. The normalized bond strength $u/\sqrt{f'_c}$ also varies with the development/splice length. It should be noted that the type of failure in pull-out tests conducted by Chan et al. [5] was shearing bond type due to the large cover thicknesses and confining effect of transverse reinforcement.

Bond between a reinforcing bar and the surrounding concrete is described as a shear force. Assuming that the bond stress u is uniformly distributed over the length of a reinforcing bar, u is given by $A_b f_s/\pi d_b l_d$, where d_b is the bar diameter, A_b is the area of bar, f_s is the tensile stress in the bar, and l_d is the embedded length. The assumption of uniform bond stress distribution over the length of a reinforcing bar can only be acceptable for short embedded/development/splice lengths l_d. In this case, the value of $u/\sqrt{f'_c}$ does not depend on l_d. For calculating $u/\sqrt{f'_c}$ in practical development/splice lengths, Orangun et al. [11] proposed the following equation:

$$\frac{u}{\sqrt{f'_c}} = 1.2 + \frac{3C}{d_b} + \frac{50 d_b}{l_d} \text{ (Imperial Units)} \tag{1}$$

where C is concrete cover, d_b is reinforcing bar diameter, and l_d is development/splice length. The above equation will be used as the basis for calculating the bond strength according to the ACI 318 Code [12].

A new attempt has been made by Zuo and Darwin [13] to obtain an equation to calculate the bond strength and the development/splice length. They have proposed a statistically-based expression to predict the bond strength and the required development/splice length. Their proposed equation for bond strength calculation which correlated well with available test results is given by:

$$u = \frac{(f'_c)^{1/4}}{\pi d_b l_d}[59.8 l_d (c_{min} + 0.5 d_b) + 2350 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) \tag{2}$$

In Imperial units, where A_b is the single spliced bar area, in.^{2}; l_d the splice or development length, in.; c_{min}, c_{max} the minimum or maximum value of c_s or $c_b (c_{max}/c_{min} = 3.5)$, in.; c_s the $\min(c_{si} + 0.25 \text{ in.}; c_{s0})$ in.; c_{si} the 1/2 of clear spacing between bars, in.; c_{s0}, c_b the side or bottom cover of reinforcing bars, in.; f'_c the concrete compressive strength, psi; $f_c'^{1/4}$, psi; and d_b is the reinforcing bar diameter.

In different semi-analytical studies conducted by Esfahani and Rangan [8,10], it was shown that depending on the development/splice length of the reinforcing bar, the value of $u/(f_c')^{1/2}$ for high strength concrete (HSC) may be either larger or smaller than that for normal strength concrete. For short lengths, the value of $u/(f_c')^{1/2}$ for HSC is usually larger than that for normal strength concrete (NSC) [8]. With increasing the development/splice length, the value of $u/(f_c')^{1/2}$ decreases for HSC due to the decrease of uniformity of bond stress distribution over the reinforcing bar embedded in HSC [10]. Esfahani and Rangan [10] introduced a new parameter, M, to account for the effect of bond stress distribution over the reinforcing bars on bond strength in different lengths. For large lengths, the value of $u/(f_c')^{1/2}$ may be smaller for the case of HSC as compared to NSC. According to Esfahani and Rangan [10], the bond strength for all ranges of development/splice lengths (i.e. short or practical lengths) can be calculated by:

$$u = u_c \frac{1 + 1/M}{0.85 + 0.024\sqrt{M}} \left(0.88 + 0.12 \frac{C_{med}}{C}\right) \tag{3}$$

where the local bond strengths for normal and high strength concrete are

$$u_c = 2.7 \frac{C/d_b + 0.5}{C/d_b + 3.6} \sqrt{f_c'} \tag{4}$$

and

$$u_c = 4.7 \frac{C/d_b + 0.5}{C/d_b + 5.5} \sqrt{f_c'} \tag{5}$$

respectively. Also

$$M = \cosh\left(0.0022L\sqrt{3\frac{f_c'}{d_b}}\right) < 1000 \tag{6}$$

which accounts for the bond stress distribution along the development/splice length. C_{med} is the median of C_x, C_y, and $(C_s + d_b)/2$; C is the minimum of C_x, C_y, and $(C_s + d_b)/2$; C_x and C_y are the side cover and bottom cover of reinforcing bars in mm, respectively. C_s is the spacing between spliced bars in mm; d_b is the bar diameter in mm; L is the length of the splice in mm; and f_c' is the compressive strength of concrete in MPa.

For the case of short pull-out specimens with splitting failure, all expressions, except u_c, on the right hand side of Eq. (3) are close to 1. Therefore, Eq. (3) changes to $u = u_c$ which calculates the bond strength of short lengths (i.e. local bond strength). Recently, Eq. (3) has been modified to account for the parameters of rib properties of reinforcing bars and transverse reinforcement on bond strength [14].

In a recent study conducted by Harajli [15], the values of the normalized bond strength with respect to the square root of concrete compressive strength $u/(f_c')^{1/2}$ in cases of short lengths, the practical development/splice lengths for NSC and HSC were compared. The results are in agreement with those obtained by Esfahani and Rangan [10]. Harajli [15] showed that below a certain limit of the development/splice length (about 15–20d_b), the average bond strength at failure normalized to $(f_c')^{1/2}$, was larger for HSC as compared to NSC. However, as the development length increased beyond that limit, the normalized bond strength of HSC became progressively smaller than that of NSC. The analysis presented by Harajli [15] led to almost identical normalized bond strength results for NSC and HSC within the practical range of development/splice lengths.

According to ACI 318 [12], for deformed bars, the development length shall be as follows:

$$l_d = d_b \left(\frac{3f_y\alpha\beta\gamma}{5\sqrt{f_c'}}\right) \tag{7}$$

where α is the reinforcement location factor ($\alpha = 1.3$ for top bars and $\alpha = 1$ for bottom bars), β is coating factor ($\beta = 1$ for uncoated reinforcement), and γ is reinforcement size factor ($\gamma = 1$ for no. 22 and larger bars). By inserting Eq. (7) in the bond strength equation $u = A_b f_s/\pi d_b l_d$, the bond strength equation based on ACI 318–2002 [12] and values of $\beta = 1$, $\gamma = 1$ can be given as follows:

$$u = \frac{5\sqrt{f_c'}}{12} \quad \text{(for bottom bars)} \tag{8}$$

$$u = \frac{5\sqrt{f_c'}}{12 \times 1.3} \quad \text{(for top bars)} \tag{9}$$

In this study, local bond strength of SCC is compared with that of NC. Local bond strength depends on various parameters such as concrete cover, concrete strength, and bar diameter. To study the influence of different parameters on bond strength, it is more appropriate to keep the concrete strength constant in different test specimens. Otherwise, the concrete strength becomes a variable. In this study, an attempt has been made to have almost the same compressive strength for both SCC and NC concretes. For the same concrete strength, local bond strengths of the two concrete series are comparable.

3. Experimental program

3.1. Tested specimens

The bond properties of reinforcing bar in SCC and NC were studied by conducting direct pullout test of reinforcing bars embedded in SCC and NC specimens. Twelve concrete specimens grouped into two series, namely Series 1 and Series 2, were manufactured and tested. In Series 1, six specimens were cast using SCC while NC was used for Series 2 specimens. The positions of reinforcing bars at the time of casting and the direction of concrete casting are shown in Fig. 1. Each test series comprised two groups with different concrete cover-to-bar diameter ratios (C/d_b). Each group included three similar specimens to provide statistically reasonable results. Each specimen contained

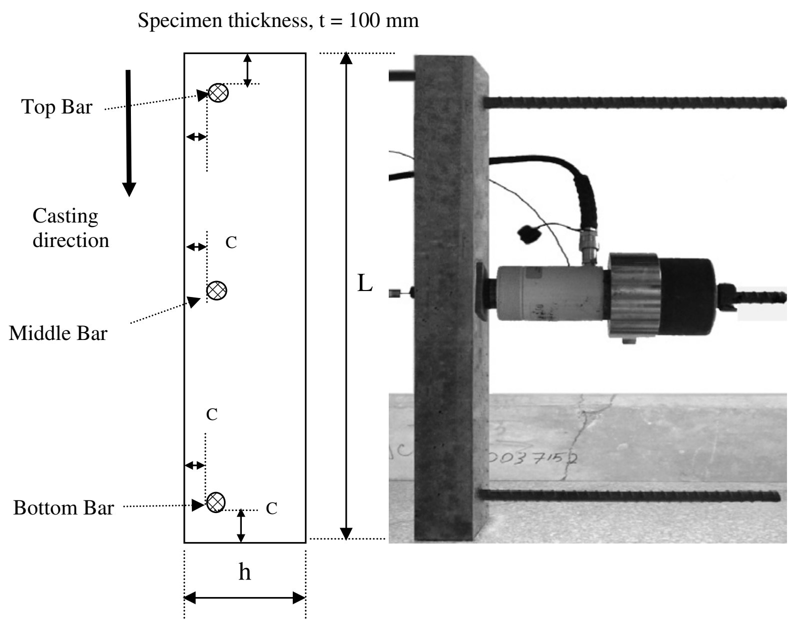

Fig. 1. Specimens and test set up.

three embedded deformed reinforcing bars at different levels – designated as bottom (B), as middle (M) and as top (T) – in order to study the effect of casting position on bond strength. Reinforcing bars of 25 mm nominal diameter were embedded in concrete (Fig. 1). The reinforcing bars were protruded from the sides of the specimens by about 30 and 300 mm. The values of cover thicknesses for different groups of each series, and the dimensions of different specimens are given in Fig. 1 and Table 1. In all specimens, the embedded length of bars was 100 mm with concrete cover varying between 30 and 60 mm.

Table 2
Mixture proportions used for different test series

Mix series	NC	SCC
W/C	0.42	0.40
Water (W), kg/m^3	202	180
Cement (C), kg/m^3	481	450
Coarse aggregate, kg/m^3	1015	715
Fine aggregate, kg/m^3	641	1050
HRWR, L/m^3	–	6.92
VMA, percent of C	–	0.062
Slump, mm	70	–
Slump flow, mm	–	600

3.2. Materials and preparation procedure

A Type 10 Canadian Portland cement (similar to ASTM Type I) was used for all concrete mixtures. The maximum sizes of coarse and fine aggregates (natural river gravel and local sand) were 12.5 and 2.5 mm, respectively. The proportions of concrete mixtures are summarized in Table 2. The stability of SCC was enhanced by incorporating a commercially available Viscosity-Modifying Admixture (VMA). All mixtures were designed to develop a 28-day compressive strength of about 60 MPa.

All concrete mixtures were mixed for 5 min in a laboratory counter–current mixer. NC was vibrated thoroughly during casting. Tests were conducted on fresh SCC mixtures to determine slump flow using slump flow test [16], flow time using V-funnel test [2], segregation resistance in terms of segregation index (SI) as per Fujiwara [17] and

Table 1
Summary of testing program

Series 1 (SCC)			Series 2 (NC)		
f'_c (MPa)	Dimensions L, h, t (mm)	Test[a]	f'_c (MPa)	Dimensions L, h, t (mm)	Test[a]
62	900, 200, 100	SC_{b40} SC_{m30} SC_{t40}	58	900, 200, 100	NC_{b40} NC_{m30} NC_{t40}
68	900, 300, 100	SC_{b60} SC_{m50} SC_{t60}	61	900, 300, 100	NC_{b60} NC_{m50} NC_{t60}

The last digits (j) indicates the cover thickness in mm.
[a] SC_{ij} and NC_{ij}: S for Self consolidating concrete and N for normal concrete, "C_i" = C_b bottom bar, C_m middle bar and C_t top bar.

air content. Fresh properties of all SCC mixtures satisfied the criteria for SCC. From each concrete mixture, six 100×200-mm cylinders were cast for the determination of compressive strength. After casting, all the molded specimens were covered with plastic sheets and water-saturated burlap, and left in the casting room for 24 h. They were then demoulded and transferred to the moist-curing room at 23 ± 2 °C and 100% relative humidity until tested.

Compressive strength of the concrete mixtures was also determined from the cored cylinders taken from the specimens after the pullout tests. In each concrete specimen, the cores were taken from different locations as close to the bar locations as possible. According to the results of core tests, the strength of concrete at the top of the specimens was, on average, 5% less than that at the bottom of the specimens. However, no significant difference was observed between the strength of concrete at the bottom and middle height of the specimens.

The nominal yield strength of the reinforcing bars was 420 MPa. The pattern of the ribs of the reinforcing bars is shown in Fig. 2. The pattern consisted of ribs perpendicular to the longitudinal axis of the bar. The relative rib area R for the bar was approximately 0.07. According to Zuo and Darwin [13], The relative rib area is defined as:

$$R = \frac{\text{projected rib area normal to the bar axis}}{\text{nominal bar perimeter} \times (\text{center-to-center rib spacing})} \tag{10}$$

3.3. Test set-up and testing procedure

Fig. 1 shows the pull-out test setup and one specimen under testing, respectively. The protruding reinforcing bar was connected to an extension rod by a coupler. The load cell, the hydraulic jack, the mounting system, and the LVDT fixtures are shown in Fig. 1. A steel plate located between the specimen and the hydraulic jack was used as a bearing plate. The dimensions of the plate were $100 \times 100 \times 10$ mm, with a hole of 30 mm diameter at its center. A manual hydraulic jacking system was adopted to apply a concentric pullout force to the reinforcing bar. When bond slip occurred, the displacement of the reinforcing bar was recorded by LVDT placed near the unloaded end of the bar. During testing, pullout load and reinforcing bar slip displacement were measured and recorded by a computer aided data acquisition system. Each test lasted about 6 min.

Fig. 2. Patterns of ribs of the reinforcing bars.

4. Results and discussion

All the specimens had failure due to splitting of concrete and no pullout failure of bars was observed. Typical splitting failure of the specimens is shown in Fig. 3. The test results are summarized in Tables 3 and 4, for SCC and NC, respectively.

By conducting the pullout tests, the local bond strength between concrete and reinforcements can be obtained from the pullout load-versus-slip relation. In this research, the bond strengths obtained by pullout test are used for comparison of variables. If the measured bond strengths are to be applied for design purpose, the characteristics of pullout test need to be taken into consideration. In general, the bond stress corresponding to the maximum pullout load – that is, the peak of a pullout load-versus-displacement curve – can be regarded as the bond strength, or, to be more specific, the ultimate bond strength. The criterion of ultimate bond strength has been widely adopted by most researchers because of its clear definition and the simplicity in bond strength interpretation and was used in this study [8,18,19]. Nevertheless, there are researchers who proposed an alternative interpretation criterion called critical bond strength [20]. The critical bond strength is defined as the bond stress of a reinforcing bar corresponding to a slip value of 0.25 mm.

During the pullout test, the pullout load and reinforcing bar slip are recorded. The pullout load is then converted into average bond stress (u) based on the embedment length and reinforcing bar perimeter using equation $F/\pi d_b l_d$, where F, d_b, and l_d refer to the peak load, bar diameter (=25 mm), and embedment length (=100 mm), respectively.

A comparison of local bond strength between test results and the proposed Eqs. (2) and (3) is presented in

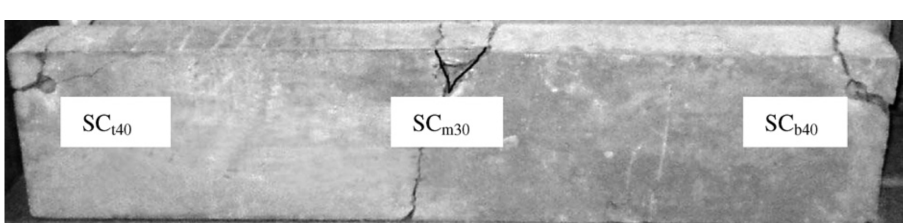

Fig. 3. Typical splitting failure of pullout specimen.

Table 3
Test results for series 1 (SCC)

f'_c (MPa)	Test	Peak load F (kN)	u (MPa)	u_{mean} (MPa)
62	SC_{b40}	47.56	6.06	8.29
		72.33	9.21	
		75.25	9.59	
	SC_{m30}	68.04	8.67	8.24
		43.93	5.60	
		82.18	10.47	
	SC_{t40}	29.78	3.79	3.96
		24.09	3.07	
		39.37	5.01	
68	SC_{b60}	59.77	7.61	9.85
		79.99	10.19	
		92.16	11.74	
	SC_{m50}	63.55	8.10	11.60
		105.44	13.43	
		104.23	13.28	
	SC_{t60}	48.01	6.12	7.00
		66.92	8.52	
		49.87	6.35	

Table 4
Test results for series 2 (NC)

f'_c (MPa)	Test	Peak load F (kN)	u (MPa)	u_{mean} (MPa)
58	NC_{b40}	65.96	8.40	7.70
		55.07	7.01	
		60.38	7.69	
	NC_{m30}	54.18	6.90	7.88
		59.83	7.62	
		71.62	9.12	
	NC_{t40}	30.26	3.85	4.34
		23.66	3.01	
		48.37	6.16	
61	NC_{b60}	85.84	10.94	9.88
		47.31	6.03	
		99.53	12.68	
	NC_{m50}	77.51	9.87	11.26
		83.20	10.60	
		104.40	13.30	
	NC_{t60}	64.05	8.16	8.53
		66.18	8.43	
		70.60	8.99	

Table 5. Since the proposed equations have been given for bottom bars in beams, only the results of tests NC_{b40}, NC_{b60}, SC_{b40} and SC_{b60} are presented in Table 5. Comparison of the measured local bond strength over predicted values, $u_{test}/u_{Eq.\ (2)}$ and $u_{test}/u_{Eq.\ (3)}$ shows that Eq. (3) predicts the local bond strength much better than Eq. (2). Therefore, Eq. (3) which was obtained based on a semi-analytical method can be used for the prediction of bond strength not only in the case of practical development lengths and spliced bars but also in the case of short lengths.

To compare the bond strengths of specimens in the two series (SCC and NC), the variation of compressive strength (f'_c) has to be taken into account. According to the provisions of ACI 318 [12] the development length of reinforcing bar for sufficient anchorage is inversely proportioned to the square root of the compressive strength, implying that the bond strength should be linearly proportional to the square root of the compressive strength. The bond strength is normalized by dividing it by $\sqrt{f'_c}$.

Comparison between the local bond strength of SCC and NC is illustrated in Figs. 4 and 5. As shown in Fig. 4, the local bond strength of bottom bars is almost the same for both NC and SCC. For the top bars, however, the local bond strength in the case of SCC is on average about 20% smaller than that in the case of NC (Fig. 5). The horizontal dashed lines in Figs. 4 and 5 show the calculated values of the normalized local bond strength using Eqs. (8) and (9) for bottom and top bars, respectively. Fig. 6 shows u_{test}/u_{ACI} versus C/d_b relationship for bottom bars in cases of SCC and NC. For both SCC and NC, the average ratio of u_{test}/u_{ACI} of all tests is about 2.7. For top bars, the average ratio of u_{test}/u_{ACI} is about 2.1 for SCC and 2.6 for NC (Fig. 7). The values of u_{ACI} for bottom and top bars were calculated using Eqs. (8) and (9), respectively.

The comparison of u_{test}/u_{ACI} versus C/d_b relationship between bottom and top bars in the case of SCC is shown in Fig. 8. From the results in Fig. 8, it can be concluded that the location factor of $\alpha = 1.3$ for top bars (ACI 318 Code [12]) should be multiplied by another factor of 1.3 in the case of SCC. For normal concrete (NC), the location factor of 1.3 seems to be appropriate (Fig. 9).

The reduction of the local bond strength of top bars is partly due to the smaller concrete strength at that location. According to the results of the core tests, the cores at the top of the specimens had compressive strengths approximately 5% less than that at the bottom of the specimens. Therefore, the smaller local bond strength of the top bars can be mainly due to the settlement of fresh concrete in specimens after casting. The settlement of concrete causes a void to occur in the concrete under the reinforcing bar and for a longitudinal crack in concrete at the top of the bar. By comparing the local bond strength of top bars in cases of SCC and NC, it may be concluded that the settlement

Table 5
Comparison of the measured local bond strength with calculated values

Test (–)	f'_c (MPa)	L (mm)	d_b (mm)	C_x (mm)	C_y (mm)	u_{test} (MPa)	$u_{Eq.\ (3)}$ (MPa)	$u_{test}/u_{Eq.\ (3)}$ (–)	$u_{Eq.\ (2)}$ (MPa)	$u_{test}/u_{Eq.\ (2)}$ (–)
NC_{b40}	58	100	25	40	40	8.4	8.18	1.03	11.7	0.72
NC_{b60}	61	100	25	60	60	10.94	10.01	1.09	12.94	0.85
SC_{b40}	62	100	25	40	40	8.29	8.42	0.98	11.9	0.7
SC_{b60}	68	100	25	60	60	9.85	10.48	0.94	13.29	0.74

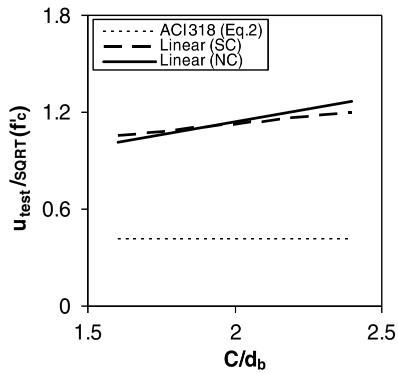

Fig. 4. $u/\sqrt{f'_c}$–C/d_b relationship for bottom bars in cases of SCC and NC.

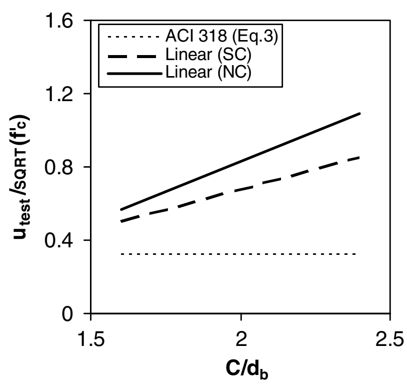

Fig. 5. $u/\sqrt{f'_c}$–C/d_b relationship for top bars in cases of SCC and NC.

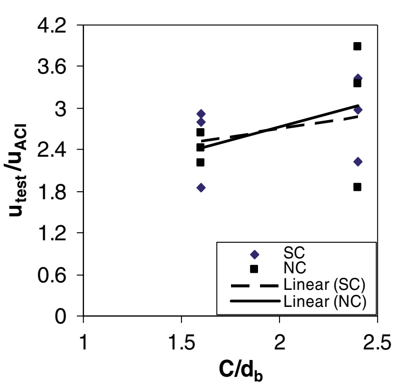

Fig. 6. u_{test}/u_{ACI}–C/d_b relationship for bottom bars in cases of SCC and NC.

of SCC is the main reason for the lower local bond strength of top bars in the case of SCC, as compared with the case of NC.

Tables 3 and 4 also present the local bond strength of middle bars. The concrete clear cover of the middle bars in all specimens was 10 mm less than that of top and bottom bars. However, the local bond strength of middle bars is generally larger than that of bottom bars. This is because of the V-notch splitting failure type of concrete around these bars Fig. 3. The V-notch failure type usually occurs in slabs. As reported previously [21], for this type of failure,

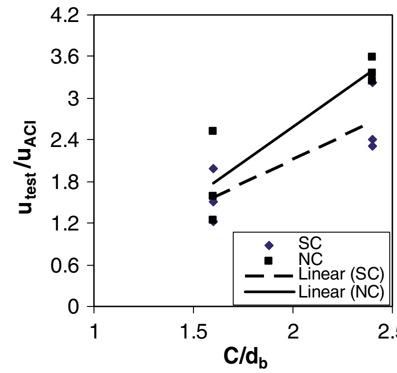

Fig. 7. u_{test}/u_{ACI}–C/d_b relationship for top bars in cases of SCC and NC.

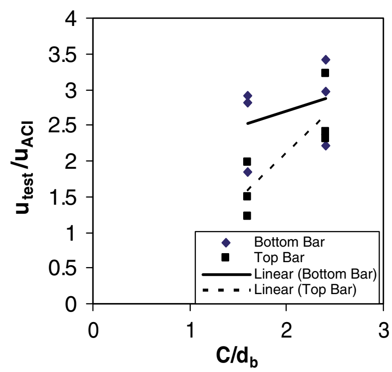

Fig. 8. u_{test}/u_{ACI}–C/d_b relationship for bottom and top bars in the case of SCC.

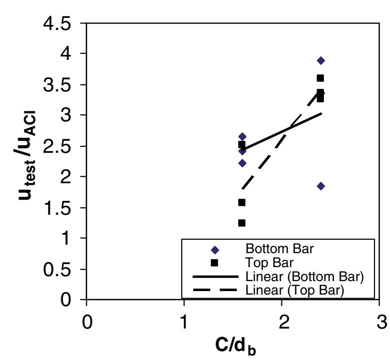

Fig. 9. u_{test}/u_{ACI}–C/d_b relationship for bottom and top bars in the case of NC.

the local bond strength is larger than that for other type of failures which are common in beams.

In order to determine the behavior of bond in different tests, the slips of the exposed end of the reinforcing bars were measured. Figs. 10–12 show the relationship between load and slip of top, bottom and middle bars in a specimen. Comparison between Figs. 10 and 11 shows that there is no a significant difference between the behavior of the top and bottom bars except that the bottom bar has larger local bond strength with concrete. After the peak load is

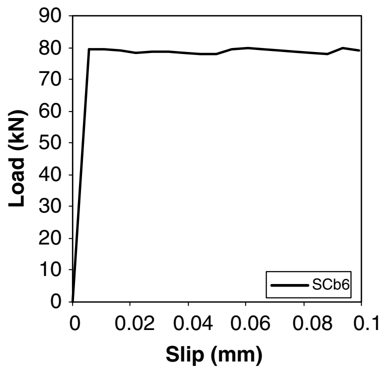

Fig. 10. Load–slip relationship for SC_{b60}.

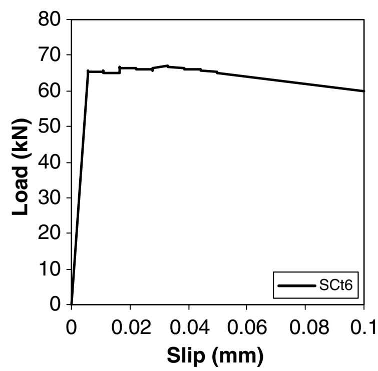

Fig. 11. Load–slip relationship for SC_{t60}.

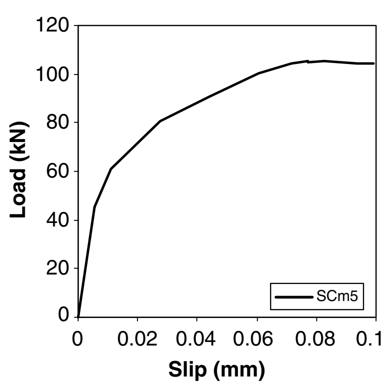

Fig. 12. Load–slip relationship for SC_{m50}.

reached, the top bar shows a reduction in load carrying capacity as slip is gradually increased. The bond behavior of the middle bar is different from that of top and bottom bars (Fig. 12). The slip of the middle bar gradually increases with the increased load until failure. The values of slip of bars in this study are similar to the values obtained in a previous study on local bond [22].

5. Conclusions

This paper presents an experimental investigation on local bond characteristics of deformed steel bars embedded in a conventionally vibrated normal concrete (NC) and a self consolidating concrete (SCC). The bond strengths of reinforcing bars were measured using pullout tests. The effect of bar position in concrete has been studied by comparing the local bond strength of top, middle and bottom bars. The results have been compared with the predictions of two recent proposed equations as well as ACI 318 Code [12]. Based on the results obtained from this investigation for bond strength of short embedded lengths, the following conclusions are drawn:

1. The comparison between the results shows that the local bond strength of bottom cast bars is almost the same in both cases of NC and SCC. However, for the top cast bars, the local bond strength for SCC is about 20% less than that for NC.
2. The comparison of the local bond strength between test results with the values calculated by ACI 318 Code [12] shows that in the case of SCC, the location factor of $\alpha = 1.3$ of ACI Code should be multiplied by another factor of 1.3.
3. The equation for bond strength (Eq. (3)) which has been obtained based on a semi-analytical method can be used for the prediction of bond strength of not only in the case of practical development lengths and spliced bars but also in the case of short lengths.
4. For the same concrete cover, the local bond strength of bars in slabs, with V-notch Type of failure, is larger than that in beams.

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References

[1] Okamura H, Ozawa K. Mix design for self-compacting concrete. Concrete Libr JSCE 1995;25(June):107–20.
[2] Ozawa K, Sakata N, Okamura H. Evaluation of self-compactability of fresh concrete using the funnel test. Concrete Libr JSCE 1995;25(June):59–75.
[3] Lachemi M, Hossain KM, Lambros V, Bouzoubaâ N. Development of cost-effective self-consolidating concrete incorporation fly ash, cement slag or viscosity modifying admixtures. ACI Mater J 2003;100(5):419–25.
[4] Hoshino M. Relationships between bleeding, coarse aggregate, and specimen height of concrete. ACI Mater J 1989;86(2):185–90.
[5] Chan YW, Chen Y-S, Liu Y-S. Development of bond strength of reinforcing steel in self-consolidation concrete. ACI Struct J 2003;100(4):490–8.

[6] Khayat KH. Use of viscosity-modifying admixtures to reduce top-bar effect of anchored bars cast with fluid concrete. ACI Mater J 1998;95(2):158–67.
[7] Sonebi M, Bartos PJM, Zhu W, Gibbs J, Tamimi A. Properties of hardened concrete. Final report. Advanced Concrete Masonry Centre, University of Paisley, Scotland, UK; 2000. http:/scc.ce.luth.se.
[8] Esfahani MR, Rangan BV. Local bond strength of reinforcing bars in normal strength and high strength concrete (HSC). ACI Struct J 1998;95(2):96–106.
[9] Esfahani MR. Bond strength of reinforcement in high-performance concrete: the role of silica fume, casting position, and superplasticizer dosage. ACI Mater J 1999;96(4):510–1 [paper by Ahmad and Itani, Discussion].
[10] Esfahani MR, Rangan BV. Bond between normal strength and high strength concrete (HSC) and reinforcing bars in splices in beams. ACI Struct J 1998;95(3):272–80.
[11] Orangun CO, Jirsa JO, Breen JE. A reevaluation of test data on development length and splices. ACI J Proc 1977;74(3):114–22.
[12] ACI Committee 318. Building code requirements for structural concrete (ACI 318-02) and commentary (318R-02). American Concrete Institute, Farmington Hills, MI; 2002. 443 pp.
[13] Zuo J, Darwin D. Splice strength of conventional and high relative rib area bars in normal and high strength concrete. ACI Struct J 2000;97(4):630–41.
[14] Esfahani MR, Kianoush MR. Development/splice length of reinforcing bars. ACI Struct J 2005;102(1):22–30.
[15] Harajli MH. Comparison of bond strength of steel bars in normal- and high-strength concrete. J Mater Civil Eng 2004;16(4):365–74.
[16] Sakata N, Maruyama K, Minami M. Basic properties and effects of Welan gum on self consolidating concrete. In: Bartos PJM et al., editors. Production methods and workability of concrete. UK: E&FN Spon; 1996. p. 237–52.
[17] Fujiwara H. Fundamental study of self-compacting property of high-fluidity concrete. Proc Jpn Concrete Inst 1992;14(1):27–32.
[18] Hwang SJ, Leu YR, Hwang HL. Tensile bond strengths of deformed bars in high-performance concrete. ACI Struct J 1996;93(1):1–20.
[19] Hossain, K.M.A. Bond in lightweight volcanic pumice concrete. In: Proceedings of the 7th East Asia-Pacific conference on structural engineering and construction (EASEC7), vol. 2, Kochi, Japan, 27–29 August 1999. p. 1460–5.
[20] Mathey RG, Watstein D. Investigation of bond in beam and pull-out specimens with high-yield-strength deformed bars. ACI J Proc 1961;62(March):1071–90.
[21] Morita S, Fujii S. Bond capacity of deformed bars due to splitting of surrounding concrete. In: Proceedings of the international conference on bond in concrete, Paisley, Scotland, June 1982. p. 331–41.
[22] Esfahani MR, Rangan BV. Studies on bond between concrete and reinforcing bars. Research report no. 1/96. Curtin University of Technology, Perth, Western Australia, March 1996. 315 p.