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Bond and Development of Steel Reinforcement in High-Strength Concrete—An Overview

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Synopsis: Considerations regarding bond and development of reinforcement in high strength concrete (HSC) are presented from a North American perspective. The information contained in this paper is a compilation of information from various sources and represents a survey of the basis for North American approaches to bond of normal and high strength concrete under monotonic and cyclic loading. The paper was presented in part at the Second US-Japan-New Zealand-Canada Multilateral Meeting on the performance of HSC held in Honolulu November 29-December 1, 1994.

Keywords: Bonding; cyclic loading; high-strength concrete; reinforced concrete

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INTRODUCTION

The development of increased concrete compressive strengths has been a major factor in the recent success of reinforced concrete in the marketplace. The increase in strengths has played a major role in decreasing member sizes, and hence dead weight, while providing economic advantages for designers, contractors and owners.

As with any technological step forward, the effects of increasing strength on structural concretes continues to require study. The overall system performance is not just a question of increased compressive strength, but rather a complex interaction of the properties of these new concretes and the demands placed on them. These high strength concretes (HSC) can be roughly defined as those with strengths of 56 MPa (8000 psi) and above. At the present time, mix designs with strengths of 105 MPa and above are routinely available in the marketplace, while in the not too distant past, a 56 MPa concrete mix was considered on the edge of technology. The important thing to observe is that much of the available data and knowledge, as well as present building code provisions, are based on testing and experience gained with lower strength concretes. Therefore, these higher strength concretes represent essentially a new class of material and should be treated as such in design. Thus, it may not be a simple case of scaling the existing design provisions by the cylinder (or cube) strength as the compressive strength increases. There may be significant changes in the behavior itself requiring that the existing rules may need to be modified or replaced.

The bond and development of reinforcing steel in high strength concrete represents a case in point. The existing bond data has been largely based on tests and observation of behavior for concretes at or around 30 MPa in strength. As the cylinder strength increases, there are significant changes in the behavior of the concrete and its failure mechanisms. Higher strength concretes tend to exhibit smoother failure surfaces and more brittle behavior than lower “normal” strength concretes (NSC). The failure surfaces are smoother since the cement paste can be stronger than the aggregates and so failure surfaces propagate through the aggregates instead of around them as is the case in NSC. These changes are significant and make existing bond relationships, based in many cases on moderate concrete strength, less accurate than intended.

The questions become even more profound when one considers that high strength concrete is typically used on columns and other primary load carrying elements whose reliability must be assured for structural survival. Moreover, many newer applications involve the use of HSC in structural systems that will be subjected to significant cyclic loading demands from earthquake. The effects of load reversals on bond and development of reinforcement of normal concretes are reasonably well understood from a qualitative standpoint but design rules are still the subject of research. The effects of load reversals on HSC are even more of a question since the response to loading is different, as are the failure mechanisms.

This paper will summarize what is known on the subject of bond and development of reinforcing steels placed in HSC. The view provided here is that of the author as a member of the North American research and design community, and will approach this problem from that perspective. What will follow is a summary of the load carrying mechanisms and the relationships and behaviors that have been established to date for HSC under monotonic and cyclic loading environments. The paper is organized so that information on behavior of NSC and HSC concretes under monotonic loading is presented, followed by specific information on cyclic loading effects of the type encountered in earthquake.

Behavior under Monotonic Loading

Background--There are three primary mechanisms in bond: adhesion of the bar to the concrete as well as resistance of the bar to movement through the surrounding concrete by friction and by interlock of the bar deformations in the concrete. Once some level of load has induced loss of adhesion and the frictional resistance has been overcome, the primary mechanism left is interlock. The concrete wedges against the reinforcing bar deformations and thus restrains the bar from moving relative to the surrounding concrete. The properties of the bar and its surface as well as the concrete environment all impact the effectiveness of this restraint.

Earlier versions of the ACI Building Code utilized a bond stress approach whereby the force developed in the bar divided by the surface area of the bar, a bond stress, was limited to a specific value. The assumption here was that the bond stress was *uniform* over the entire length of the development region.

$$u = A_b f_s/\pi d L_s = \pi d_b^2 f_s/4\pi d_b L_s = d_b\, f_s/4L_s$$

Given this simple starting point, the role of research has been to identify the parameters that control the bond effectiveness so as to determine design relationships. To that end there have been many studies conducted over the years to establish the behavior of reinforcing steels under monotonic loading. These studies have largely concerned normal to moderately high strength concretes. The design rules that have been developed are largely empirical in nature and are “skewed” towards the moderate end of the strength scale.

These earlier studies include those by Abrams (1) and a large study by Clark (15) that was a fundamental study of all bars produced in the United States. The result of this work was the elimination of deformation patterns that were not able to produce acceptable bond performance. Other important studies are those by Tepfers (45, 46), Orangun, Jirsa and Breen (40, 41), Ferguson and Breen (27), Ferguson and Thompson (29), Mathey and Watstein (38), Chamberlin (11, 12), and Chinn et al. (13).

Design Rules--The end result of these studies was the development of the design rules to predict bond performance. In particular, the Orangun, Jirsa and Breen study (40, 41) resulted in an important relationship to empirically depict the bond stress of bars under development:

$$u = [1.2 + 3C/d_b + 50\, d_b / L_s \; + A_{tr} f_{yt} / 500 s d_b] \sqrt{f'_c} \quad \text{[Customary Units]}$$

$$u = [0.10 + 0.25C/d_b + 4.15\, d_b / L_s \; + A_{tr} f_y t / 41.52 s d_b] \sqrt{f'_c} \quad \text{[SI Units]}$$

In this equation it can be seen that there are several discrete elements. The term $3C/d_b$ relates the effects of the confining effects of cover and the negative effects of close bar spacing on bond. The f'_c term is taken as the square root to indicate that the bond strength is a function of tensile strength and is not linear with cylinder strength. The bond strength is linear with bar diameter. The effects of confinement are contained in a separate term that is linear with confining steel area. There also is a term that is a constant and reflects the fact that a bar with no cover would still exhibit limited bond strength.

ACI Committee 408 (4) also has taken data from a variety of sources and has developed their own database of bond test data and produced an expression for bond and development of Grade 420 [60] reinforcement:

$$L_{db} = 5500\, A_b / [\phi K^* \sqrt{f'_c}] \qquad \text{[US Customary Units]}$$

where K* is the smaller of

(a) $0.5d_b + C_c + K_{tr}$ or
(b) $0.5d_b + C_c + [\Sigma K_{tr}/n]$; but no larger than $3d_{db}$

From these equations the familiar design expression found in the ACI Building Code (2) through the 1992 edition can be developed:

$$L_d = 0.04\, A_b\, f_y / \sqrt{f'_c} \qquad \text{[US Customary Units]}$$

$$L_d = 0.02\, A_b\, f_y / \sqrt{f'_c} \qquad \text{[SI Units]}$$

Here it can be seen that these design expressions represent a portion of the Orangun equation that has been simplified for design use. Also it can be seen that the K factor of the 408 equation has been replaced by a constant. For comparison purposes, given a standard bar configuration of a No. 8 Grade 420 [60] bar in 28 MPa [4000 psi] concrete, this equation predicts about a 30 bar diameter development length.

The important concept to note here is that the equations are empirical in nature and so are based on “best fit” equations to the data. As can be seen in Fig. 1, the data employed by Orangun et al. exhibited scatter that is customary in studies of bond because of the many variables that can affect the bond capacity of these systems. Thus, the equations predicted from the data are not exact for any particular data point, or design case, but rather encompass the range of behavior observed. This equation, and the derivative expressions from it, represented a significant step forward in the study of bond in the United States. The work established a philosophy of viewing bond as being comprised of three separate mechanisms that is still in use today. Lastly, it is important to note in the present context that this study contained only 62 specimens and that the cylinder strengths of the data all were below 42 MPa.

Studies by Darwin et al. (21) confirm that the bond capacities can be described in terms of parameters that include concrete strength, bar size, side and edge cover, bar spacing and the degree of confining steel. As shown in Fig. 2, the differences in the bond behavior when cover or confinement steel is present can be significant. A study of test data with no confining steel revealed an equation of

$$L_{db} = 0.06\, A_b\, f_y / \sqrt{f'_c} \qquad \text{[US Customary Units]}$$

which provides longer anchorage lengths than the ACI 318 expression above, providing a 45 bar diameter development length for the “standard bar configuration;" a value comparable with those obtained under present ACI 318-95 rules for development length.

De Vries et al. (39) reviewed the existing data and the ACI 318 and 408 expressions to simplify the procedures for designers. A rewritten form of the 408 equation was obtained as

$$L_{db} = 0.12\, A_b\, f_y / \sqrt{f'_c} \qquad \text{[US Customary Units]}$$

$$L_{db} = 1.4\, A_b\, f_y / \sqrt{f'_c} \qquad \text{[SI Units]}$$

In another important study Sozen and Moehle (44) evaluated data from some 223 tests from around the world. Some data was at 70 MPa and above, although most was from NSC tests. Typical results are presented in Figs. 3a and 3b show the scatter of the data and the role of compressive strength and bar size in the bond strength. Another significant aspect of these studies by Moehle et al. was the reorientation of development length to calculations to be in terms of numbers of bar diameters. This change allowed designers to return to the form used in earlier versions of the ACI Building Code and to make the design process more efficient.

Moreover, the work by Moehle and Sozen, together with research by others during the 1980's and early 1990's, provided a basis for a discussion of bond and development and the general question of how to depict bond in an accurate and straightforward manner. This work was a basis for the revised bond equation found in the 1995 ACI 318 Building Code (3):

$$L_d/d_b = [3 d_b f_y / 40] \frac{\alpha\beta\gamma\lambda}{[(c + K_{tr})/d_b]} \qquad \text{[US Customary Units]}$$

where α is the top bar factor and is either 1.0 or 1.3; β is the epoxy coating factor; γ is the size factor and λ is the lightweight aggregate factor; c is the cover or spacing and K_{tr} is the sum of transverse steel acting to confine the bars being developed.

The resulting equation also has the characteristic square root function of concrete strength. Significantly, the development length equation now is linear with bar diameter providing a simpler format for application by designers in practice while giving longer anchorage lengths than predicted in earlier Building Code editions. For the "standard bar configuration", the new equation provides a 40 bar diameter prediction for development length.

Another important observation is that the ACI 318 limits the concrete strength to 10 000 psi [69 MPa] in calculating the development lengths. This limit is recognition that the data is limited in the HSC regime and that the behavior is different.

Factors Affecting Bond Capacity of NSC-- It has been shown over the years that there are a number of factors that affect bond capacity of normal strength concrete and steel. These factors, represented in the expressions discussed earlier include:

<u>Concrete Material Properties</u>. Concrete slump, consolidation, aggregate strength and size, and the compressive strength of the concrete all play a role in bond. As a general rule the compressive strength decreases the required bond length by approximately the square root of the compressive strength for NSC.

The square root function implies that the tensile strength of the concrete may control the bond strength by delaying or preventing splitting cracks in the cover of the member that would - if large enough - allow the bar to slip relative to the member. There is evidence that the fracture energy, G_f, defined as area under the stress--crack opening curve, may in fact be a better parameter to use to determine the effectiveness of the concrete in anchoring a bar and preventing slippage (4). The larger the fracture energy, the more ductile the concrete is and the more the concrete is able to redistribute load from one region of the bar to another as cracking occurs. Bond data where fracture energy of the concrete is specifically measured is not a normally reported parameter that makes evaluation of the historical data difficult to do.

Reinforcing Bar Characteristics. The reinforcing bar itself can affect bond. In particular the rib height, rib spacing and face angle and the surface condition all can play a part in altering the bond capacity. The general nature of these effects has been known for quite some time. Studies by Darwin et al. (14, 22, 23) concerning development of a high relative rib area bar with increased bond performance have shown that the geometry of the rib in terms of height, profile, spacing and inclination all play a role in anchoring the bar. Increased deformation heights and reduced spacings act to increase the relative rib area, the ratio of deformation bearing area to shearing area, from current values of 0.07 to values above 0.10. The effects appear, however, only when the bar is confined either by two or more bar diameters of amounts of cover or by transverse steel or both. Tests conducted where low covers are present with no transverse steel result in splitting failures at loads similar to those obtained with standard bars. The effects of increased rib height on the bar can lead to increased peak load values indicating that the bond process is more efficient and higher loads can be achieved, thus decreasing the required amount of development length for design, as shown in Fig. 4. Without confinement the increased bond performance of these high relative rib area bars is less pronounced (22, 36).

The presence of epoxy or other surface coatings on the steel can have a profound effect on the bond capacity. Studies by Treece and Jirsa (48) and Johnston and Zia (35) resulted in the present set of adjustment factors found in the ACI Building Code where the development length is increased by a factor of as much as 1.5. The epoxy coating essentially makes the bar surface "slick" reducing friction and adhesion leaving only mechanical interlock to anchor the bar. Work by Cairns (10), Choi et al. (14), Darwin et al. (20), and Cleary and Ramirez (16) confirmed that the bond capacity of epoxy coated steel is reduced by as much as 50% of that of an uncoated bar of the same size for bars that have low levels of confinement.

Confinement. A significant enhancement of bond performance is produced by confinement. Whether by additional cover and wide bar spacing, or by additional confining steel, the confinement can play a large role in placing the concrete around the bar(s) in more of a three dimensional state of compression. This action increases the capacity over that found in a unconfined configuration (22, 23). As was shown in Figs. 2 and 4, the bond capacity of beam end test specimens is significantly increased with the presence of confinement. While the test result may be a higher load at failure, perhaps a more significant effect is that the failure is ductile and that the load can be maintained even after the peak value is attained, as shown in Fig. 5.

Studies of Bond Capacity of HSC

The bond capacity of steel embedded in HSC has been studied recently in a number of investigations. It is important to note that up until recently there have been relatively few data on HSC bond with most data at strength levels of

56 MPa and below. The studies that can be summarized as follows:

Hadje-Ghaffari et al. (30) conducted a limited study of bond with concrete strengths of 84 MPa and compared their results with beam end specimens with results from 48 MPa tests. The data showed virtually no increase in capacity with increasing concrete strength. These tests, however, were with unconfined specimens where splitting failures would be expected to occur and not be delayed by confinement. In a follow-up study, Darwin and Graham (22) studied the effects of bar and concrete parameters on bond strength and found that aggregate properties significantly affected bond performance.

Hwang, Leu and Hwang (34) studied the bond performance of specimens with 70 MPa concrete, thus placing their results in between the NSC and HSC regimes used in this presentation. Their paper compared results with Orangun, 318-89 and present 318-95 predictions and found that the equations generally did quite well in predicting anchorage strength with the ACI equations being more conservative in their predictions. In addition, the effects of confinement was more pronounced in HSC than in NSC.

Kimura and Jirsa (36) used a small "modified" pull out specimen and found that bond stress and splitting were indeed a function of the square root of the cylinder strength. The results show that the bond stresses, stiffness and first cracking strengths approximately follow the square root law with compressive strength.

French et al. (29) tested HSC using beam end specimens. The investigators found that the concrete compressive strength was a strong factor in strengths up to 70 MPa and that the effects were less pronounced as the concrete entered the high strength regime at strengths above 70 MPa. The data for No. 6 and 11 bars all exhibited a nonlinear curve that tended to "fall off" as the strength went above 70 MPa, as shown in Figs. 6 and 7.

The overall conclusion that can be reached is that while there is limited data available there is strong evidence that bond in HSC may not follow a square root law when the compressive strengths exceed a threshold value, perhaps 70 MPa. Moreover the results are dependent on the test parameters, perhaps significantly more so than with NSC.

One of the most definitive studies has been performed by Azizanamini et al. (7, 8) in which the bond capacity of HSC splice specimens with compressive strengths of 35-42 MPa for NSC and up to 100 MPa for comparison HSC specimens. The splice capacities were evaluated using the "Texas specimen" first used by Treece and Jirsa (48). The results indicate that the compression capacities are linear with cylinder strength while the tensile capacity follows approximately the square root law. Load is carried by the first few deformations of the reinforcing bar and that little load redistribution occurs. Crushing occurs at a higher mean stress in HSC than is normally found with concrete, thus the failure occurs rapidly. Load cannot be redistributed and carried by other deformations farther down the bar since the tensile capacity has not increased by the same amount and splitting occurs at a lower mean stress. Therefore, the bar attempts to carry the load through compression of the HSC around the deformations and once this is lost there is no alternative load path. Finally, the assumption that development length is linear with the number of deformations involved may hold for NSC but may be unconservative for HSC. There may be little additional capacity gained by simply adding more bar length alone. Failure of the concrete around the first highly loaded deformations may produce a failure that can progress quickly along the bar leading to a splitting failure of the cover. Thus, the requirement for a minimum amount of confining steel is more important with HSC than with NSC, particularly with small covers, to prevent these nonductile splitting failures from occurring.

Observations on Monotonic Bond Performance of HSC – Based on the studies done to date, a number of observations on the bond and development of steel in HSC:

1. The failure surfaces exhibited by HSC are smoother than those found in NSC, typically including a failure plain through, rather than around, aggregates. This smooth surface is indicative of a lack of interlock between failure surfaces. Moreover, the failure behavior tends to be sudden and brittle, more so than with NSC.

2. When reinforcing steel is being developed, there appears to be less "load sharing and redistribution" than in NSC. The bar deformations at the bar ends tend to take more load in any connection, what is significant is that in HSC there is a limited amount of redistribution of load into the connection or development length. Rather the load tends to be concentrated at the ends making the standard assumption of a uniform "bond stress" over the development length less correct than in NSC. When the critical failure stress is reached, there is limited load redistributed and carried in the interior of the bar length and, thus, less redundancy in the system. It has been noted that increased splice lengths do not correlate with an increase in connection capacity because of this phenomenon. Thus the normal approach of providing more deformations, bar development or splice length, to engage the surrounding concrete to achieve more capacity may not be applicable with HSC.

3. The failure stress locally in the concrete at splitting appears to be not linear with compressive strength across the strength range and is most probably a square root function of concrete strength. However, the square root relationship does not appear to hold at higher strength levels, perhaps 70 MPa. The failure process involves the formation of microcracks around the bar as the concrete is dilated due to bar loading and slippage relative to the surrounding concrete. The critical level of failure stress that can be reached prior to splitting failure of the concrete is related to the fracture properties of the concrete and not solely to strength. The fracture energy does not increase with compressive strength as would be expected. Thus, failure caused by fracture may occur at levels similar to those for NSC. Once the failure is initiated, it tends to be more sudden than with NSC with less redistribution of the load from the points of failure into the other portions of the bar development length or connection.

4. Confinement is important in HSC to an even larger degree than with NSC. Because the behavior is nonductile, designers need to properly detail structures so that the adverse effects of HSC can be controlled. However, there is limited data in this area upon which to build design rules, thus the need for more study. It is clear that high strength stirrups, that is, those with higher yield strength than "normal" stirrups are not of significant assistance because the HSC does not permit the stirrups to yield. Thus, adjusting the yield strength does not increase the capacity of the system. The data suggest that the effect of stirrups may be more related to area and stiffness provided than to yield force as is conventionally assumed in NSC.

5. The top bar effect is reduced in HSC probably because of the limited amount of bleeding around bars and the lower water cement ratio to start with. For compressive strengths above 100 MPa, top bar factors approaching one have been reported (14).

Bond Capacity of HSC under Cyclic Loading – The bond of reinforcing steel in NSC and HSC is made more complex by cyclic loading, such as that induced by earthquake. Here the loading direction changes and the area in front of the deformations is alternately loaded in compression and then in tension. Thus the concrete is subjected to load reversals that accelerate the damage process and can lead to bond failures at lower loads than found in monotonic loading. The proper detailing of reinforcing steel in a cyclic load environment is, thus, even more important than under unidirectional loading.

There is very limited information available on the performance of bond specimens under cyclic loadings. The best single source of information on the bond considerations of reinforced concrete systems for cyclic loadings is the ACI 408 State-of-the Art Report on Bond under Cyclic Loads (5). It is stated in this report that data on this subject is limited. However, seismic excitations are such that the observations applicable to monotonic situations generally hold under seismic demands as well. The key appears to be to hold the bond stresses to approximately 80 percent of ultimate bond stress or less. Limiting the bond stress helps control the demand in the bond areas. Once the area surrounding the bar is damaged by large stress demands, bond slippage will occur and the damage process will begin.

Another observation made in the ACI 408 report is that confinement is essential to keep the bond regions intact and to control damage and bar slippage. Once slippage occurs, stiffness of the member will be compromised. Thus, as in monotonic loading, the role of confinement appears to be the primary factor in determining how a member will behave under cyclic loading. The role of proper detailing, therefore, becomes a paramount consideration when one moves from NSC to HSC.

In the area of cyclic loading and its effects on bond, a significant consideration is bond slip as mentioned above. The larger the slippage of bond, the lower the stiffness of the member resulting in increased deformations. The prediction of the actual performance of bond during loading – the slip -- is difficult to measure experimentally and generally is reported as a loaded end slip or unloaded end slip (21). While this type of information is helpful, it is important to be able to predict how the slippage occurs as a function of demand on the bar. This information can help in defining better concepts for determining bond performance, as well as providing computational model for use in analysis. The area of bond models has not been a primary focus of North American research but rather has been employed by engineers in Europe and Japan as a means to study bond. Work by Eligehausen et al. (26) was fundamental in defining the bond model employed by the CEB Model Code (17). Recent work by Bijag (9) shows how useful this type of approach can be in understanding what the actual behavior is, and how to improve performance. Moreover, these types of models are useful in computer-based analysis of dynamic response to excitations to predict actual behavior more closely.

CONCLUSIONS

The bond of reinforcing bars is a complex phenomenon that has occupied researchers for most of this century. Experimental work has shown that the bond process can be greatly affected by many factors that only now are beginning to be understood. Most of this work, however, has concerned normal strength concrete. What research that has been done on HSC has shown that prediction of bond capacities should be approached with caution.

In particular, the challenge of designing HSC structures to resist seismic or other cyclic loading regimes is an area that needs significant work. Existing research indicates that longer development or splice lengths do not translate into increased safety and in fact can be dangerous. Instead designers need to carefully detail members to confine the concrete around the development or splice areas to overcome the tendency of HSC to nonductile failures. Adequate levels of confinement are more important with HSC than with HSC and are necessary to permit load redistribution along a splice or developing bar as highly loaded regions of the bar-concrete interface fail and release their load. This behavior is significantly different than in NSC where redistribution is more likely to occur even with nominal levels of confinement.

In summary, the role of HSC in modern reinforced concrete structures will continue to grow. To support this increased usage, research is needed to answer the fundamental issues relating the behavior of this new material to design. One of the primary needs is to build on the work already completed in NSC and HSC to increase our knowledge of bond and development of steel reinforcement in HSC.

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