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EFFECT OF RIB GEOMETRY ON BOND BEHAVIOR AND FAILURE MODES

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Key words: Bond(concrete to reinforcement), Deformed bars, Decreasing bearing angle theory, Shearing failure, Splitting failure, Wedging action.

Abstract: The ribs of deformed bars can cause a bond failure by splitting the concrete cover, by wedging action, or shearing the concrete in front of the ribs. As deformed bars are pulled out, the rib face angle is flattened by shearing of the concrete key, which decreases the rib face angle to a smaller bearing angle. Analytical expressions to predict bond resistances for splitting and shearing are derived, in which the bearing angle is found to be a key variable. The bearing angle produced between rigid steel ribs and quasi-brittle concrete tends to decrease, in nature, by the wedging action. As the bearing angle decreases, the splitting bond resistance decreases while the shearing bond resistance increases. The bearing angle is decreased to decrease the splitting bond resistance and to increase the shearing bond resistance. The decreasing bearing angle theory is proposed to better understand bond mechanisms between ribbed reinforcing bars and concrete. Experimental works with beam splice test specimens are also reported. The bond behaviors and failure mode discussed from the proposed theory agree well with this experimental observation.

1 INTRODUCTION

To be effective, a reinforced concrete member must have a positive interaction between the bars and the surrounding concrete. The bond between reinforcing steel and concrete, fundamental to the mechanics of reinforced concrete, is a many-faceted phenomenon, but follows a natural rule of the wedging action provided by the rib geometry.

There have been some conflicts on the effect of rib geometry on bond strength between reinforcing bars and concrete. Some researchers show that deformation has strong influence on bond strength. Other studies indicate that deformation pattern has little influence and it is common for bars with different patterns to produce nearly identical development and splice lengths (Darwin and Graham 1993).

Modern deformed bar rib geometries date from the work of Clark in 1949. Since then, knowledge concerning the bond between concrete and ribbed deformed steel has considerably increased based on both experimental work and analytical studies. At the time Clark recommended that the ratio of the shearing area (bar perimeter times distance between ribs) to the rib bearing angle (projected rib area normal to bar axis) be limited to a maximum of 10, and if possible 5 or 6. Today this criterion is usually expressed the ratio of the bearing area to the shearing area, which is known as the 'rib area,' 'relative rib area'. Relative rib area, R_r, will be used as the descriptive term in this paper.

During the late 1950s and the 1960s, researchers observed two phenomena accompanied by the slip of ribbed bars: (1) concrete is split by the wedging action of the ribs and (2) concrete between the ribs is crushed (Rehm 1957, Lutz and Gergely 1967). Rehm (1957, 1961) found that if the ratio of rib spacing to rib height is less than 7 and if the rib face angle is greater than 40 deg, the concrete in front of the ribs undergoes gradual crushing, followed by a pull-out failure (Fig. 1). If the ribs have a spacing to height ratio greater than 10, for a rib face angle greater than 40 deg, the concrete in front of the ribs first crushes and then forms wedges that induce tensile stresses that cause longitudinal splitting of the concrete.

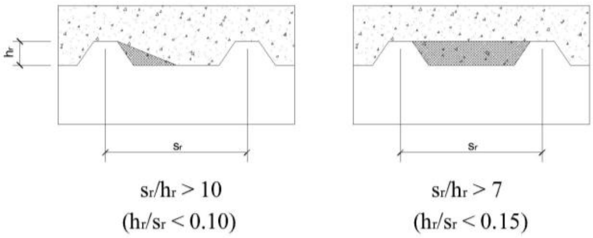


Figure 1: Failure mode depending on the ratio of rib spacing to rib height

Researchers observed that the ribs act as wedges and the concrete in front of the ribs undergoes gradual crushing, followed by a pull-out failure. When bars are moderately confined by concrete cover or transverse reinforcement, a high rib face angle on the ribs is flattened by shearing of the concrete key, which decreases the effective rib face angle to a smaller value. In experimental work, concrete powder was found against the loaded face of some of the ribs (Darwin and Graham 1993). The angle between the surface of the concrete powder and the bar shaft ranged 17 and 40 deg (Fig.2). In general, the higher the confinement, the more likely a pull-out failure. However, in most structural application with reinforcing bars used today, a splitting failure is more common.

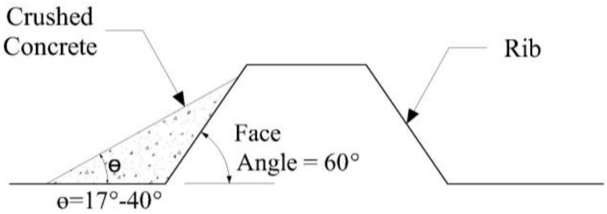


Figure 2: Schematic of rib and crushed concrete after failure (Darwin and Graham 1993)

A number of researchers have derived analytical expressions for bond mechanisms in splitting failures (Tepfers 1979, Cairns 1979). Analytical studies have found that bond generates a radial bursting pressure equal in magnitude to the bond stress, the so-called hydraulic pressure analogy. A number of experimental and analytical studies have been presented, examining a more fundamental definition of the local bond-slip law (Giuriani et al. 1991, Gambaroba and Rosati 1997, Plizzari et al. 1998). For the case of splitting failure, analytical studies of interfacial bond have been performed to predict the bond strength of ribbed reinforcing bars (Choi and Lee 2002), and in this paper, the role of the bearing angle on bond behavior is addressed. Bearing angle model for bond analysis of reinforcing bars to concrete was proposed where the bearing angle is defined as a wedging angle of the failure interface (Choi et al. 2010, Choi and Lee 2012). Analytical expressions to predict bond resistances for splitting and shearing failures were derived, in which the bearing angle is a key variable.

The rib geometry of deformed bars (ASTM A944-95 1997) governs bond behavior and is instrumental in guaranteeing adequate bond resistance. The influence of deformation pattern on bond performance has been studied and bond resistances have been observed to vary with the rib characteristics (Tefers 1979, Skorobogatov and Edwards 1979, Choi et al. 1991, Hamad 1995). Studies by Darwin et al (1996a, 1996b) have demonstrated that bond strength increases with an increase in the relative rib area, R_r, of bars under high confinement, but under low confinement, bond strength is independent of deformation pattern. Still the effect of rib geometry on bond is, however, poorly understood and studies to simulate experimental observation that the rib face angle is flattened have not been attempted. Failure mechanisms of bond, such as, splitting, shearing off, and pull-out failure are not clear yet. Bond strength is regarded as the sum of splitting and non-splitting components (Cairns and Jones 1996), but knowledge on the interaction between two components is very limited.

This study is intended to explain the nature of the wedging action of ribbed bars as they interact with concrete as a general aspect of bond. Further studies to examine the analytical expressions to predict bond resistances for splitting and shearing failures are reported in this paper. As the bearing angle is decreased, the splitting bond resistance decreases while the shearing resistance increases. In the cases of bars at a moderate level of confinement, the bearing angle is decreased to decrease the splitting resistance. The interaction between bond resistances from splitting concrete cover and shearing the concrete key is studied. The decreasing bearing angle theory is applied for analyzing the effects of rib geometry on bond behaviors of ribbed reinforcing bars to concrete and improving the understanding of bond failures of ribbed reinforcing steel in concrete structures.

2 ANALYTICAL EXPRESSIONS TO DETERMINE BOND RESISTANCES IN SPLITTING AND SHEARING FAILURE

2.1 Expression to predict bond resistance in splitting

Modeling of bond action of reinforcing bars to concrete can be simply extended using the hydraulic pressure analogy introduced by Tepfers. The capacity of a short anchorage was predicted by modeling the splitting cracks and hoop tension in the concrete. Wedging action by the rigid steel rib of deformed bars makes it possible to resolve bond forces into normal stress σ_n and tangential shear stress τ, as shown in Fig. 3. The resultant of normal components along the bar is what places the surrounding concrete in tension. When a reinforcing bar in tension P, concrete under the bearing side of a rib is placed in a state of tri-axial compression, with the major principal stress, the bearing stress, σ_q, on the rib acting parallel to the bar axis. Normal to the bearing stress, the minor principal stress σ_r acts radially around the bar. The extended method of analysis has been used previously by Choi and Lee (2002) to evaluate the bond strength in splitting failure (Choi and Lee 2012). The final equation to predict the splitting bond resistance is expressed as follows.

$$T_{split} = F_x \pi \tan\alpha \frac{(1+\mu\cot\alpha)}{(1-\mu\tan\alpha)} + A_r \frac{c}{\sin\alpha(\cos\alpha - \mu\sin\alpha)} \tag{1}$$

where F_x is confining force by the fracture of concrete cover or yielding of transverse reinforcement. α is the bearing angle defined as wedging angle of failure interface to bar axis, μ is the friction coefficient, c is cohesion and A_r is projected area of rib parallel to bar axis. The friction coefficient μ is one of the variables to determine the bond resistance. Bond resistance increases as the friction coefficient increases. The contribution from cohesion to bond resistance is small and diminishes as bars slip. When the interfacial material and confinement are determined by the structure itself, the bearing angle is the only variable in the first term of the left side in Eq. (1).

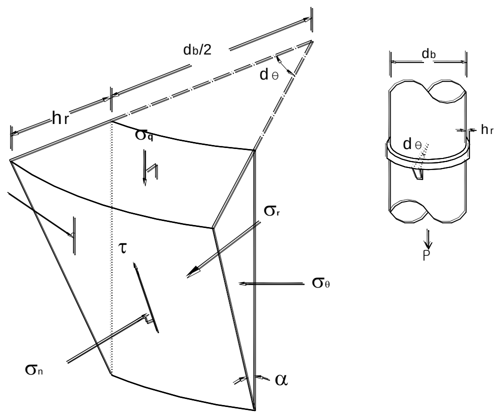


Figure 3: Stresses acting on rib of bar (Cairns 1979)

2.2 Expression to predict bond resistance in splitting

When loading an anchored bar, relative movement, slip, between steel and concrete will occur. The slip is caused mainly by crushing of the concrete in front of the ribs. The high pressure on the concrete in front of the ribs causes tensile stresses in the concrete around bar, which, in turn, create internal inclined cracks. Inclined cracks initiate at relatively low bond stresses at the point of contact between steel and concrete. With increasing induced slip, the concrete in front of the ribs will be crushed. Increasing the stress in the bar further more slip occurs because more local crushing takes place and later shear cracks in the concrete keys between ribs are initiated. At maximum bond resistance a part of, or the total, concrete key between the ribs has been sheared off, depending on the ratio of clear rib distance to average rib height.

As shown in Fig. 4, from the force boundary conditions, an angle α is made along the shear failure surface, where the tangential stresses and the radial stresses are in equilibrium. Based on a study by Birkeland and Birkeland (1966), for cracks in monolithic concrete, shear strength should not be assumed greater than $0.2f'_c A_c$ shown in Eq. (2).

$$V_n=0.2 f'_c A_c \tag{2}$$

where A_c is the area of cracked surface. The area of cracked surface A_c defined by the area of a cone with the angle of, α

$$A_c = \frac{\pi d_b h_r}{\sin\alpha} \tag{3}$$

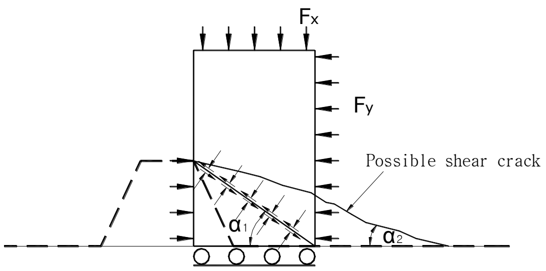


Figure 4: Shear cracks by the concrete key between bar ribs

Further, the concrete in contact with the bearing side of a rib is in a state of tri-axial stress which increases the shear strength of the concrete. The concrete is subjected to very high compression from the confining force F_x and the high compressive stress modifies the magnitude and direction of principal stress and increases the cracking load. Two parameters accounting for the beneficial effects on the shear strength owing to the tri-axial state and the high compression, κ_1 and κ_2 are proposed. Using Eqs. (2), (3) and the two parameters, the bond force resisting from shearing of the concrete key, the shearing bond resistance, is proposed by

$$T_{shear} = \kappa_1 \kappa_2 \frac{0.2 f'_c \pi d_b h_r}{\tan\alpha} \tag{4}$$

where κ_1 = tri-axial state parameter and κ_2 = high compression parameter. The value of 2.0 is proposed for κ_1 as the constant for two way shear action and the parameter κ_2 is proposed to range between 1.5 to 2.5 depending on the level of confinement provided by cover or transverse steel. Information on these two quantities shall be obtained from the results of future analytical or experimental studies.

3 DECREASING BEARING ANGLE THEORY

Let us examine again Eq. (1) and the key variables for the splitting bond resistance. The friction coefficient μ is one of the key variables to determine the bond resistance. Splitting bond resistance increases as the friction coefficient increases. The crack surface is naturally rough and irregular, but the coefficients of friction along the interface at the crack should be constant in nature. The contribution from cohesion to bond resistance is small and diminishes as bars slip. The confinement force F_x, provided by concrete cover or transverse reinforcement, is proportional to the bond force. The capacity of the confinement force is made up of the splitting resistance by concrete cover or by transverse reinforcement, thus the confinement force has a limitation. When the confinement is determined by the structure itself, the bearing angle is the only variable in Eq. (1) corresponding to the change of splitting bond resistance.

As discussed before, the bearing angle tends to decrease, which is the nature of the wedging action. The bearing angle of the failure surface of the concrete in front of the ribs also tends to decrease. While the concrete between the ribs sheared, the high rib face angle is flattened, which decreases the effective rib face angle. The predicted splitting bond resistances can be plotted versus bearing angle of α, as shown in Fig. 5. The bearing angle definitely changes the bond resistance in splitting. As the bearing angle is decreased, the splitting bond resistance decreases. Bond strength may not be linear to the confinement force as in Eq. (2). A hypothesis is stipulated that the bearing angle may vary depending on the degree of confinement and bond strength may not increase as much as the increase of confinement. The rate of increase in bond strength with the increase of confinement force may be reduced by the decreasing bearing angle, as illustrated in Fig. 5.

As in Fig. 6, the shearing resistance T_{shear} increases as h_r increases. There might be a lower limit on the bearing angle and the minimum value of the bearing angle can be obtained by the ratio of the rib spacing to the rib height.

Bond strength, the capacity of an anchorage, is defined as the maximum bond force at any state of failure. Normally, the weaker mode of the two failures considered to govern the strength. The two bond forces, the splitting resistance and the shearing resistance interact by a single variable, the bearing angle. The bearing angle is decreased to decrease in the splitting resistance and increase in the shearing resistance, until reaching the same resistance. Therefore, both bond resistances appear to control the bond strength. At reaching the bond resistance, the maximum bond capacity, th*e* concrete key can be sheared and the concrete cover may be split depending on the degree of confinement. The bearing angle is determined so that the splitting resistance can be equal to the shearing resistance, and finally the resistance itself becomes bond strength T_{bond}. Thus,

$$T_{split} = T_{shear} = T_{bond} \tag{5}$$

Equation (5) can be solved for the bearing angle α. The solution for the bearing angle to determine bond strength by the decreasing bearing angle theory is schematically illustrated in Figs. 5 and 6. Bond strength is determined by the bearing angle, so that the resistances of splitting and shearing failures should coincide at the same angle.

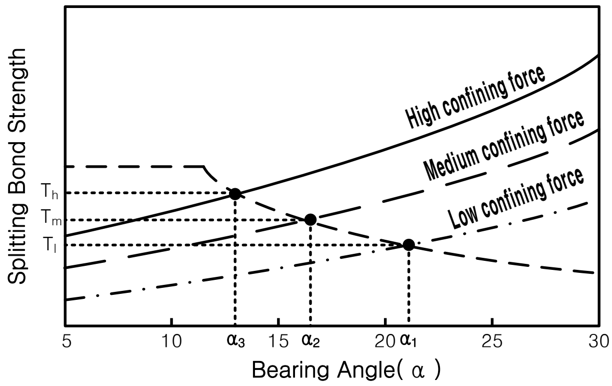


Figure 5: Schematic analysis of bond strength by bearing angle theory (different confinement)

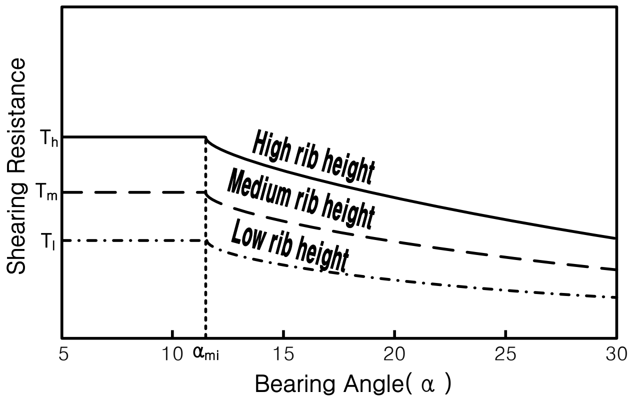


Figure 6: Schematic analysis of bond strength by bearing angle theory (different rib height)

4 PARAMETRIC STUDY AND DISCUSSION

4.1 Experimental program

The experimental program described in paper consisted of beam–splice specimens with test bar size of 25mm (No. 8). The principal parameters are the rib geometries, such as, the rib height, rib spacing, the relative rib area. To compare the theoretical work, test results of 4 specimens are reported in this paper. Three different deformation patterns were machined and tested in addition to the commercial pattern. As illustrated in Table 1, three rib heights, 1.6 mm, 2.4 mm and 3.2 mm were used with spacing ranged 18 mm and 36 mm to produce relative rib areas R_r, of 0.075 to 0.133. Specially designed bars with alternating high and low rib height were tested to explore the additional bond strength increase in this study.

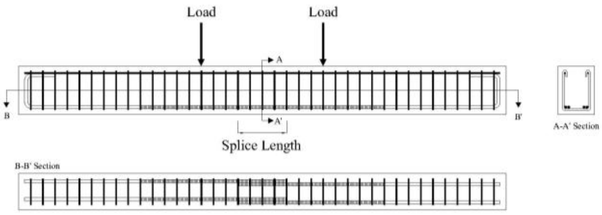


Figure 7: Beam splice test specimen

The splice specimens, 4.5m long, were tested as inverted simply supported beams to produce a 1.2 constant moment region, as shown in Fig. 7. The specimens contained two adjacent bottom-cast splices. 13 mm bars stirrups were spaced equally within splice region and outside the constant moment region to provide shear strength. The beam had nominal width of 300 mm and nominal depth of 395 mm. Total depth was constant 450 mm, thus bottom cover and side cover were 25mm. Splice specimens were inverted and tested, and loads were applied at end of cantilever reactions. Beams were loaded continuously to failure at the rate of about 10 kN per min at each end. Specimen properties and test results are listed in Table 1.

4.2 Analytical study and comparison

The bearing angle model is applied for the four specimens to understand the bond behaviors from the test results. It is generally agreed that as the combination of rib height and rib spacing affect bond strength. As the relative rib area, the rib height divided rib spacing, increases, bond strength increases, but not proportional. Further, there might be a limitation of the maximum relative rib area, such as the value should be less than 0.14 in ACI 408 report for the high relative rib area bars.

Fig. 8 shows the schematic determination of bond strength by the bearing angle model to understand the effects of rib height and rib spacing. For bars with conventional deformation with low rib height [Fig 8 (a)], the bearing angle, α_1, is low, thus the bond strength, T_l, is low. For bars with high rib height [Fig 8 (b)], the bearing angle, α_2, is high, thus the bond strength, T_2, is high. However, since the rib spacing is small, the rib height may not be fully effective and some part of rib does not contribute for the wedging action. The bond strength may not increase much due to this phenomenon. In addition, this explains that there might be limitation of the increase for bond strength for the high relative rib area bars.

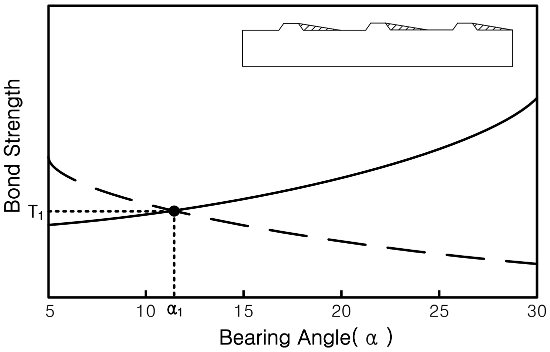
Table 1: Specimen properties and test results

Bar Designations*	Rib height	Rib spacing	R_r	Modified bond strength (kN)	Average (kN)
Bars with 50 mm concrete cover, with transverse stirrups D10@100 mm					
SP25CV	1.6	18	0.075	219 234 222	225
SP25HR	2.4	18	0.133	261 237	249
SP25HRWS	3.2	36	0.088	262	262
SP25AHLR	3.2 and 1.6	36	0.133	282 261 289	277

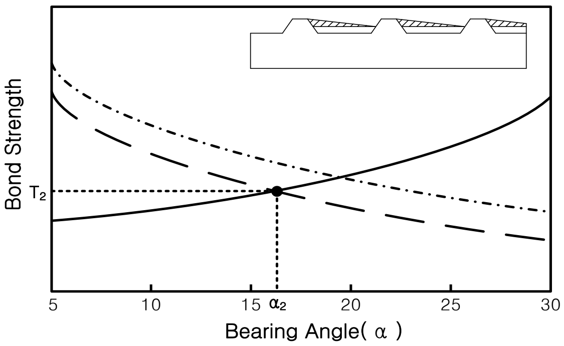
*Bar designation: CV = conventional bars, HR = high rib height bars, HRWS = high rib height and wide spacing bars, AHLR = alternating high and low rib height bars

- R_r of CV bars is reduced by 85 percent because of the longitudinal rib.

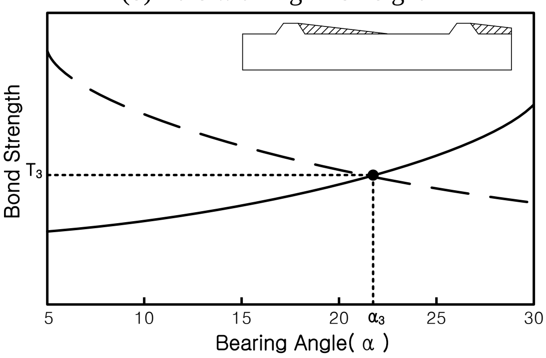
- Modified bond strength $(27/f_c')^{1/2}$



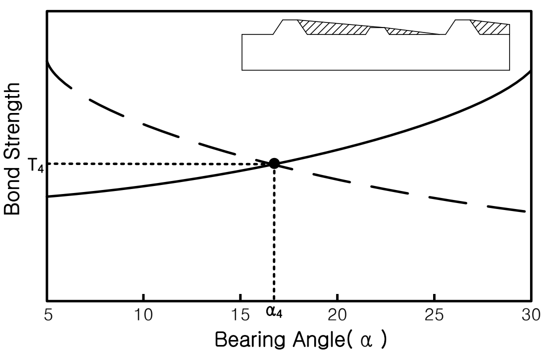
(a) Bars with low rib height (conventional bars)



(b) Bars with high rib height



(c) Bars with high rib height and large spacing



(d) Bars with alternating high and low ribs

Figure 8: Schematic analysis of bond strength by bearing angle theory (different rib height and spacing)

For bars with high rib height and large rib spacing [Fig. 8 (c)], the bearing angle, α_3, is high, thus the bond strength, T_3, becomes very high. Furthermore bond strength for alternating high and low rib height bars are much higher than those of conventional bars and bars with high relative area because the shear failure surface from the high rib surpass the low rib and the rib height is fully effective, resulting in the very high bond strength

From comparison of the analytical work to test results, there are several indications that rib geometry affects the bond performance similarly. In the Table 1, bond strength increases as R_r increases. However, bars with high rib height and wide spacing (HRWS) show much higher bond strength than that of bars with high relative area (HR). Furthermore bond strength for alternating high and low rib height bars (AHLR) are much higher than those of conventional bars (CV) and bars with high relative area (HR).

Fig. 8 shows the lowest value of bond strength, T_1, with conventional bars (CV). Bond strength of bars with high relative area (HR), T_2, is high but not significant, since the rib spacing is small and the rib height may not be fully effective Some part of rib does not contribute for the wedging action. For bars with high rib height and wide rib spacing (HRWS), the bearing angle, α_3, is high, thus the bond strength, T_3, becomes very high. Furthermore bond strength for alternating high and low rib height bars (AHLR), T_4, are much higher than those of conventional bars (CV) and bars with high relative area (HR).

The behaviors discussed from the proposed theory of decreasing bearing angle agree well with this experimental observation. In this study, the bearing angle becomes more flatten, decreasing the splitting resistance as the level of confinement increases. However, in turn, the shearing resistance increases decreasing the bearing angle. This observation can be explained by the indication that two failures to keep a balance maximizing the bond resistance by decreasing the bearing angle even all the cases of rib height and spacing.

The proposed theory helps to understand

the nature of the wedging action by the ribs of deformed bars to concrete. The failure modes of ribbed bar-concrete bond interaction and the mechanism of the flattened rib face angle become clear in this study. Bond strength is attained by the close interaction between the splitting resistance or failure and shearing resistance or failure. The theory will help to show the beneficial effects of high rib area bars on bond performance. The understanding for bond force considering the splitting, shearing and pullout-type failure is critical for design of the anchorage capacity. Further research related to the proposed theory will help to understand several behaviors of bar-concrete interaction and enhance the bond performance of deformed reinforcing steel for the safety of reinforced concrete structures.

5 CONCLUSIONS

From the analytical and experimental studies in this paper the following can be concluded:

1. Analytical expressions to determine the bond resistances for splitting and shearing failures are derived. The major variable is the bearing angle which plays the key role of bond behavior by rib geometry;

2. The bearing angle tends to be decreased, which is the nature of wedging action. Bearing angle decreases as rib face angle flattened observed in tests.

3. In the case of bars with different combination of rib height and spacing, the test results match well with the theoretical observations;

4. Bond strength for alternating high and low rib height bars is much higher than that of conventional bars and bars with high relative area;

5. Decreasing bearing angle theory is effective to predict bond strength and to simulate bond mechanisms between ribbed reinforcing bars and the surrounding concrete.

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