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Bond of ribbed galvanized reinforcing steel in concrete

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Abstract

The ASTM beam end test (ASTM A944) has been used to compare the bond and slip behaviour of deformed (i.e. ribbed) galvanized, epoxy-coated and black steel bars in concrete. The objective was to determine whether galvanizing adversely affects bond strength. From a series of thirty specimens, the average bond strength of black steel and galvanized steel reinforcement used in these tests has been determined and bond stress has been shown to act uniformly over the embedded bar area. A slip value of approximately 0.4 mm has been confirmed to be associated with bond failure by concrete splitting. The results indicated that while epoxy coating resulted in a significant loss in bond strength of the order of 20% compared to black steel, there is no adverse effect on bond with the use of galvanized steel. Chromate treatment of galvanized bars is deemed unnecessary since there was no evidence of long term reduction in bond due to the possible effects of hydrogen gas evolution resulting from the reaction between zinc and wet concrete.

Keywords: Bond; Slip; Bond stress; Galvanized; Epoxy-coated; Reinforcement; Concrete; Chromate

1. Introduction

The coating of bars has become an efficient and popular method to protect steel reinforcement from corrosion in concrete. The most widely used commercial coating methods are fusion bonded epoxy coating and galvanizing. While both coating systems successfully reduce the corrosion risk in most exposure situations, and this has been widely reported, there has been some concerns expressed regarding the nature of the bond between coated reinforcement and the concrete. It has been established that epoxy coating reduces bond strength and, to compensate for this, design codes stipulate an increase in the development (or embedment) length of bars [1]. The bond of galvanized reinforcement has not been as thoroughly investigated in this respect, but prevailing information has been that bond strength between galvanized steel and concrete may be reduced because of the formation of bubbles of hydrogen gas when zinc is immersed in wet cement [2,3]. To prevent the hydrogen gas evolution, the zinc surfaces need to be passivated and this is achieved by either adding a dilute chromate solution to the concrete mix water, or by dipping the freshly galvanized bar in a 0.2% sodium dichromate solution [4].

Growing concerns over the past decade in particular regarding both the occupational health risk and environmental hazards associated with the use of chromates has led to imposition of extremely stringent regulations on the use of chromates. This has had an effect on many industries including general galvanizing where the use of chromates to prevent so-called white rusting of freshly galvanized steel is now severely restricted, and also in the concrete pre-casting industry where chromate additions to mix water is virtually banned. The concern over occupational health has extended to cement manufacture in some countries where an upper limit of 2 ppm of naturally occurring chromium ions in concrete should not be exceeded [5]. Obviously, such regulations generally rule out the passivation of galvanized reinforcement. This has led to a reconsideration of the need/requirement for chromate treatment in internationally recognized standards such as ASTM A767 [6].

Nevertheless, two questions remain: the first is whether the hydrogen evolution actually occurs in real situations of concrete reinforced with galvanized bars;

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and the second is whether the bond between steel and concrete is reduced because of galvanizing and, if so, are the ribs on deformed reinforcement sufficient to compensate for the adverse effect of bond reduction?

Early work by Bird [2], indicated that the hydrogen evolution at the surface of galvanized steel immersed in Portland cement paste occurs where iron and zinc are in contact, and not on the surface of pure zinc immersed in cement paste. Hot dip galvanizing results in the formation of a series of metallurgically bonded zinc-iron alloys layers, each successive layer from the steel substrate outwards containing a higher proportion of zinc with a relatively pure layer of zinc at the outer surface [7–10]. It would thus appear that the evolution of hydrogen is minimized where the galvanized coating has a pure zinc outer layer, even if it were quite thin [2]. Recent work by Andrade and Macais [11] has shown the importance of the pure zinc layer in the passivation of galvanized steel in concrete to the extent that some 10 microns of zinc is consumed during the initial passivation reaction, and some zinc must remain on the surface for the passive state to be maintained over long periods of exposure. Reactions such as these between zinc and concrete result in a high degree of adhesion to the galvanized surface which has beneficial effects on bond capacity, even for plain (i.e., smooth) galvanized bars embedded in concrete [12]. It is worth noting here that previous studies of bond effects for galvanized smooth and ribbed bars produced evidence of a significant loss of bond compared to non-galvanized samples [13,14]. In these studies however, the age of concrete at the time at which the pull-out testing was done was seven days. Hofsoy and Gukild [14] have reported that pronounced retardation occurred on the concrete when zinc powder was added to cement of low chromate content. More recently, it was shown that the formation of zinc oxide and zincates on the surface of galvanized bars delays the setting and reduces the early strength and adhesion bond [15]. Nevertheless, it has been concluded that given more time, the strength of concrete close to the coating surface and the bond strength both increase and may result in higher bond than in the case of ungalvanized bars [15]. Recent work [16] has verified this retardation and consequential delay in the development of bond when zinc is in contact with wet concrete. The results show that while the bond strength of plain galvanized bars at 7 and 14 days curing was significantly less than that of equivalent black steel bars, by 28 days the bond performance of both galvanized and black steel bars was similar.

As to the effect of the deformation ribs, the bond capacity of ribbed steel bars has been found to reduce significantly because of epoxy coating [17,18]. The magnitude of the reduction, however, is influenced by the deformation pattern, relative rib-bearing area, size of bar and coating thickness [19]. Cairns and Abdullah [20] concluded that it is likely that the bond failure in the case of epoxy-coated bars is characterized by the failure path running along the bar/concrete interface and not through the concrete. This supports the universally accepted observation that there is virtually no adhesion between concrete and the epoxy coating [21,22]. It has also been shown that the relative rib area and the rib face angle significantly affect the bond strength of epoxy-coated bars [19,21]. Nevertheless, it is not feasible to impose specific rib geometry restrictions on the bars selected for coating in order to increase their bond strength [23]. Thus the presence of the ribs is not sufficient to compensate for the loss in bond with epoxy coating.

On the other hand, pullout tests conducted on ribbed galvanized bars gave results for pullout strength that were not significantly different from that of similarly ribbed black steel bars [12]. Such tests, however, are not conclusive mainly because of the dissimilarity between the pullout test conditions and actual loading situations encountered in service, especially in flexural members. In traditional "lollipop" type pullout tests, the bar is loaded in tension with the concrete being restrained in compression, generally against the loading frame. This is quite unrealistic compared to real engineering situations and, while such pullout tests do provide useful comparative data, it is difficult to extrapolate this test data to real-life design. Another serious limitation of pullout testing is that failure by splitting of the concrete mass (cylinder or small beam) may occur at a load below the maximum bond capacity when ribbed bars are used. This also makes reliable interpretation of the data difficult.

The ASTM has recently introduced a standard beam end test that simulates the behaviour of reinforcement in flexural members [24]. This test is based on extensive research by Darwin and Graham [25], and has proven satisfactory in providing meaningful results on various aspects of the bond phenomenon of deformed reinforcement in structural concrete.

2. Experimental program

Two series of tests were conducted. In the first series, the embedment length of the bar was set at 150 mm, while in the second series the embedment length was reduced to 120 mm. The requirements of ASTM Standard Test 944-95 [24] were followed through testing. A schematic of the test setup is shown in Fig. 1, and specific details of the samples are shown in Fig. 2.

The first test series consisted of 18 beam end tests; six with black ribbed bars, six with ribbed galvanized bars and six with ribbed fusion bonded epoxy-coated bars.

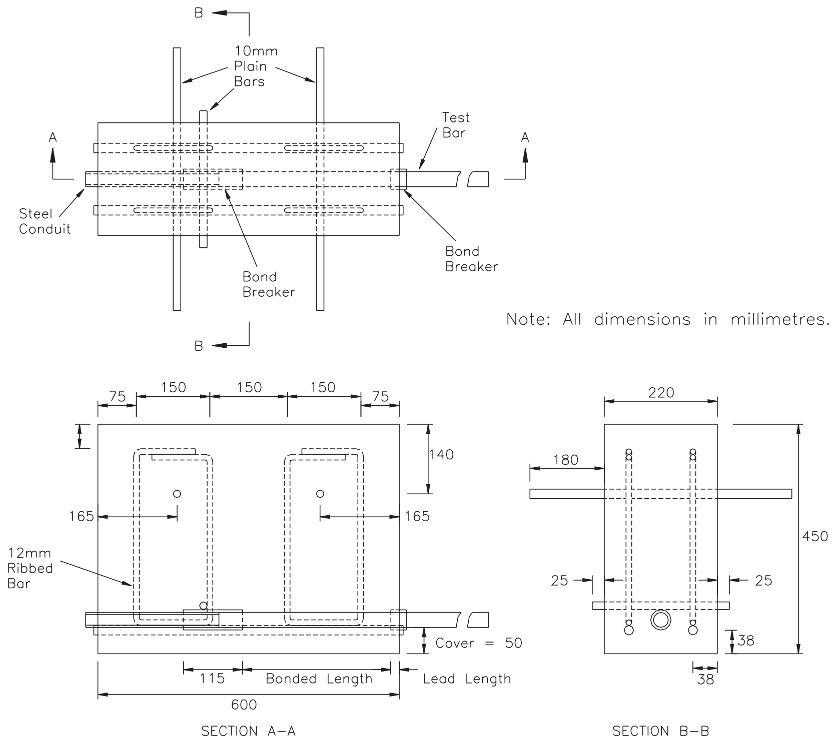

Fig. 1. Schematic of the beam end test.

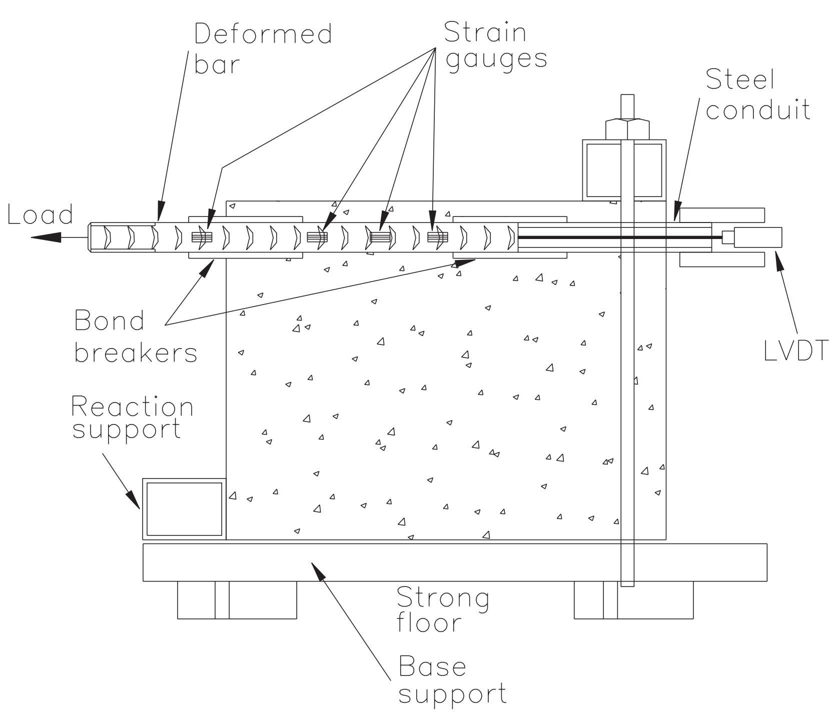

Fig. 2. Details of test sample.

All bars were 16 mm nominal diameter and of identical rib geometry as shown in Fig. 3. Ready mix concrete designed for a characteristic strength of 32 MPa was used. The 28 day compressive strength was 35.5 MPa and the fresh concrete had a slump of 100 mm. Mixture quantities of compacted concrete, based on saturated

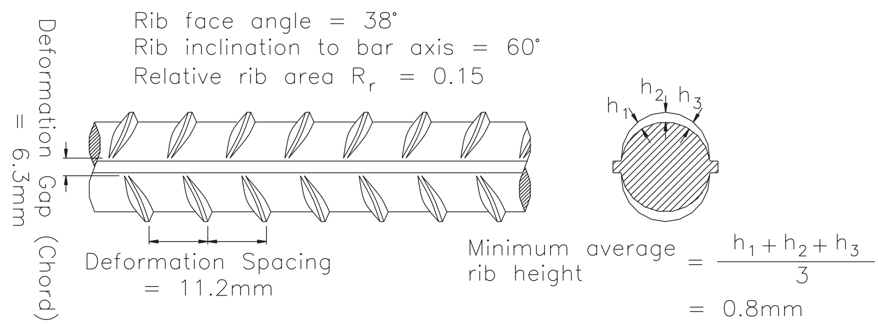

Fig. 3. Bar geometry.

Table 1
Concrete mix details

	20 mm Coarse aggregate	10 mm Coarse aggregate	Blended sand	Slagment	Ordinary portland cement	Water reducing agent	Air entraining agent	Water
Quantity per cubic metre	713 kg	274 kg	800 kg	175 kg	196 kg	888 ml	80 ml	96 l

surface dry condition of the aggregates, are shown in Table 1. A 150 mm embedment length was used and the slip at the free end of the bar was continuously recorded using a linear variable differential transformer (LVDT). The load and slip were continuously recorded and read into a computer. The rate of loading was 27 kN per min. The data were analyzed and the average load–slip curve for all bars of each group was determined. Due to the large amount of data collected, a 5 kN load interval was adopted and the value of slip was taken at each load interval. The average slip value at a particular load for all the bars of the one type was then calculated.

The second series consisted of a further 12 beam end tests, six with black ribbed bars and six with similar galvanized bars. In this series the embedded length was reduced to 120 mm. Moreover, four strain gauges were fixed onto each bar at distances of 8.5, 60, 111.5 and 230.5 mm from the free end of the bar. The first three strain gauges were within the embedded length, while the last strain gauge was located on the load receiving unbonded portion as indicated schematically in Fig. 2.

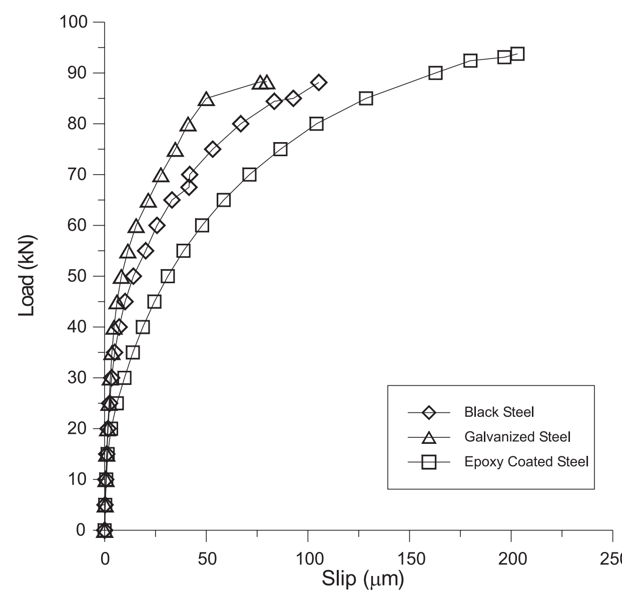

Fig. 4. Load–slip relationship for black, epoxy-coated and galvanized ribbed bars with 150 mm embedment.

3. Results and discussion

3.1. Slip and bond of black, galvanized and epoxy-coated bars

3.1.1. 150 mm embedment length

Average load–slip curves for the first series of tests with an embedment length of 150 mm are shown in Fig. 4 where each line represents the average result for a group of six identical samples. The 150 mm embedment proved to be sufficient for the three types of reinforcement to develop the full tensile strength of the steel. The differences reported therefore are in the slip values at the various load intervals. In this testing regime, the load–slip data was self-limiting by the bar reaching its (ultimate) tensile strength (f_{su}), then failing by ductile tearing. The tensile strength of the steel used was

Table 2
Load at specific slip values for the three types of ribbed bars

Type of reinforcement	Slip value (mm)	Mean load (kN)	Standard deviation (kN)	Difference from reference (kN)	95% confidence limits	Is the mean significantly different from the reference?
Black steel	0.025	62[a]	7.8	Reference	55–68	Reference
	0.05	75[a]	7.9	Reference	68–82	Reference
	0.1	48.3[b]	9.4	Reference	40.8–55.8	Reference
	0.2	64.4[b]	9.3	Reference	57–72	Reference
	0.3	75.5[b]	6.6	Reference	69–82	Reference
Galvanized steel	0.025	73[a]	8.9	+18	67–78	No
	0.05	77[a]	8.2	+3	70–83	No
	0.1	54.8[b]	8.9	+13.5	47.7–61.9	No
	0.2	69.3[b]	7.3	+7.6	63.4–75.2	No
	0.3	75.2[b]	5.0	−0.4	70–80	No
Epoxy-coated steel	0.025	46[a]	5.8	−26	41–51	Yes
	0.05	61[a]	6.1	−19	56–66	Yes

[a] 150 mm embedment.
[b] 120 mm embedment.

approximately 460 MPa which, for 16 mm diameter bars, equates to a failure load of 93 kN. This corresponds to the maximum load shown in Fig. 4, irrespective of the type of steel, either black or coated, and the type of coating, either galvanized or epoxy-coated.

It can be seen that the epoxy-coated steel exhibits larger slip values than both black and galvanized steel at all intervals. This is expected because of the low (or nil) adhesion along the smoother epoxy surface, and thus the loss of the frictional component of the bond occurs sooner. For the same load values, galvanized steel exhibited noticeably smaller slip. Statistical analysis of the load data at 25 μm and 50 μm slip showed no significant difference in the load at slip of the galvanized and black steel bars. However, the difference between the galvanized and epoxy-coated steels was statistically significant at both slip values, as was the difference between the black and the epoxy-coated steel at both slip values. Table 2 summarizes these results.

If slip at a particular load is taken as indicative of bond strength, then it may be concluded that the bond strength of black and galvanized steel in concrete are, respectively, 19% and 26% greater than that of epoxy-coated steel. Previous bond–slip tests conducted on simply supported concrete beams reinforced with either ribbed black, galvanized or epoxy-coated bars, have also shown that at 0.02 mm (20 μm) slip there is no significant difference in behaviour between galvanized and black steel bars. However, a significant reduction in bond of 23% was recorded for epoxy-coated steel [26]. The results obtained here following the standard ASTM test procedure are thus in general agreement with the previously conducted tests on beams in flexure.

While providing the ribs contributes an increase of more than 40% in bond strength compared to plain bars [22,26,27], it is clear that the ribs alone are not sufficient to overcome the difference in adhesion and friction that results from using a coating system such as fusion bonded epoxy. This result may also indicate that there is very little difference in the adhesion and friction components of bond strength between galvanized and black steel bars.

3.1.2. 120 mm embedment length

Fig. 5 shows the results of tests performed on ribbed black and galvanized bars with an embedment length of 120 mm. Again, each line in this figure represents the average result for six identical samples. It was observed

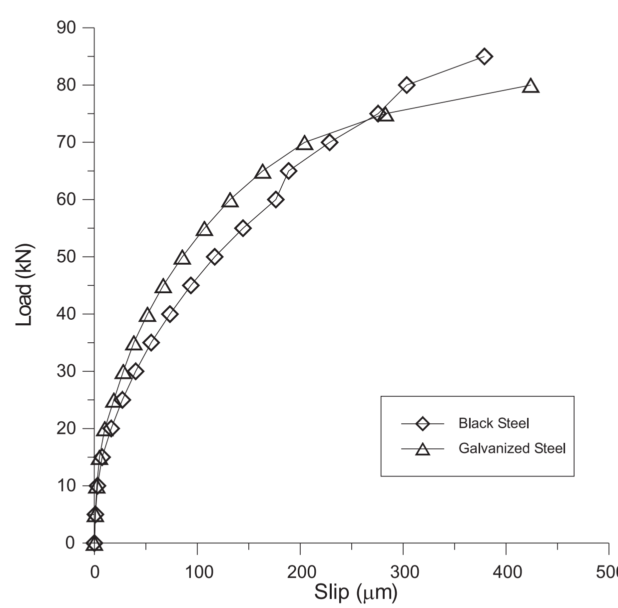

Fig. 5. Load–slip relationship for black and galvanized ribbed bars with 120 mm embedment.

that reducing the embedded length from 150 to 120 mm resulted in a large increase in slip, at the same load for both types of steel. Failure in this case was manifest by splitting of the concrete since tensile failure of the bars did not occur at the reduced embedment length of 120 mm. This can be seen in Fig. 5 where the maximum load for both the black and galvanized steel is lower than the previously identified 93 kN failure load of the bars, even at large slip values. The average slip value for both the galvanized and black steel bars at failure was about 0.4 mm (400 μm). This is in contrast to the average value of 0.09 mm (90 μm) when the embedded length was 150 mm. It should be noted however, that the value of 0.09 mm slip for the 150 mm embedment is a coincidental value when the galvanized and black steel bars both reached their failure loads.

It is interesting to note here that Abrishami and Mitchell [28] have reported that a slip value of about 0.36 mm is associated with bond failure, and that this is approximately the same for both small and large bars. Their results are in close agreement with the results obtained here. This serves to confirm the conclusion that the amount of slip associated with ultimate bond capacity for the same type of bond failure is similar [28]. Nevertheless, specimen size [29], test set-up [30] and the confining action [31,32] have been previously found to influence bond capacity and mode of failure.

The load–slip curves representing black and galvanized steels revealed slight differences in their behaviour. Statistical analysis for the two groups at slip values of 0.1, 0.2 mm and 0.3 mm is shown in Table 2.

3.2. Estimation of bond strength

The authors believe that the results discussed above provide strong evidence for the superiority of galvanized or black steel when compared to epoxy-coated reinforcement. Subsequently, the authors were interested in drawing a similarly clear conclusion regarding the effect of galvanizing on bond. For this purpose, two independent approaches were used to interpret the results obtained in this study.

3.2.1. Uniform simple average stress

The first approach used here depends on the assumption that bond stress (σ_b) is uniform along the embedded length [28]. Therefore bond stress at any stage of loading may be determined by simply dividing the load by the embedded area of the bar. Failure in the tests conducted in this series was initiated by concrete splitting, and therefore it is considered that bond failure has occurred at the splitting load.

The average ultimate bond stress could also be calculated by dividing the ultimate load by the embedded surface area of the bar. By this approach, the average bond stress has been found equal to 13.2 MPa for black and 13.5 MPa for galvanized steel. These values therefore represent the bond strength of the samples as tested. Although galvanized steel showed a slight improvement over black steel, statistical testing of the results indicated no significant difference between the two groups.

3.2.2. Evaluation of bond stress from strain measurement

Monitoring stress development at the four different strain gauge positions on the bar made it possible to also determine the bond stress by an alternative approach as follows.

The average bond stress in a certain finite length $\boldsymbol{\delta L}$ of the embedded bar is expressed as

$$\sigma_b = (d_b/4) \cdot ([f_{s2} - f_{s1}]/\delta L), \tag{1}$$

where d_b is the diameter of the bar, f_{s1} the stress in the bar at a particular location and f_{s2} is the stress in the bar at a second location a distance δL from the first.

The stress in the steel at two locations can be determined from the strain measurements at the two middle positioned strain gauges placed 51.5 mm apart. The value of bond stress at any stage of loading could then be calculated using Eq. (1) above.

Consider an embedded length L of a bar between the point $x = x_1 = 0$ at the free end, to the point where $x = x_2 = L$ towards the loaded end as illustrated in Fig. 6(a). At the free end, the bond stress is assumed to be equal to zero. At the loaded end, the stress in the bar is f_{s2} and the load is P. Hence $P = f_{s2} \cdot \pi r^2$, where r is the radius of the bar. The load P is equal to the bond action over the entire embedded surface area of the bar.

It is now assumed that the bond stress is distributed along the length L according to a function $F(x)$ as shown in Fig. 6(b). The total force exerted by the bond stresses is equal to the area under the curve of σ_b versus x, represented by $F(x)$, multiplied by the circumference of the bar. That is,

(Area under the σ_b versus x curve)$\cdot(2\pi r) = P = f_{s2}.\pi r^2$ and so at any distance x from the free end.

Area under the σ_b versus x curve $= f_{s(x)}.(r/2)$.

By denoting the area under the σ_b versus distance curve by β, one gets

$$\beta_x = f_{s(x)} \cdot (r/2).$$

β_x can now be plotted as a function of x and, since $\beta_x = \int \sigma_b.dx$, it follows that

$$d(\beta_x)/dx = \sigma_b. \tag{2}$$

What this indicates is that the slope at any point x of the curve of β_x versus x, is equal to the bond stress at the point x. If the relationship of β_x versus x approximates a straight line, then the slope of the curve may be considered constant and therefore the value of σ_b may be considered constant.

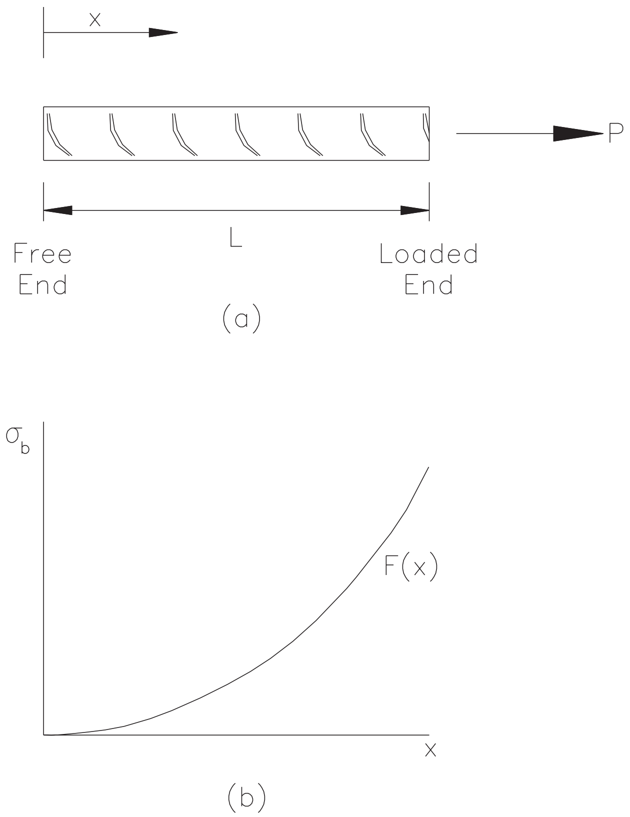

Fig. 6. Expected development of bond stress along the embedded length.

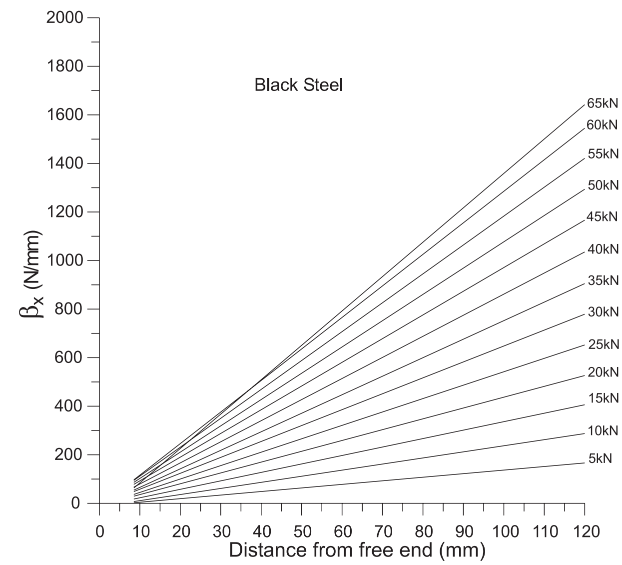

Fig. 7. Relationship between β_x and distance from the free end for the stages of loading of black steel samples.

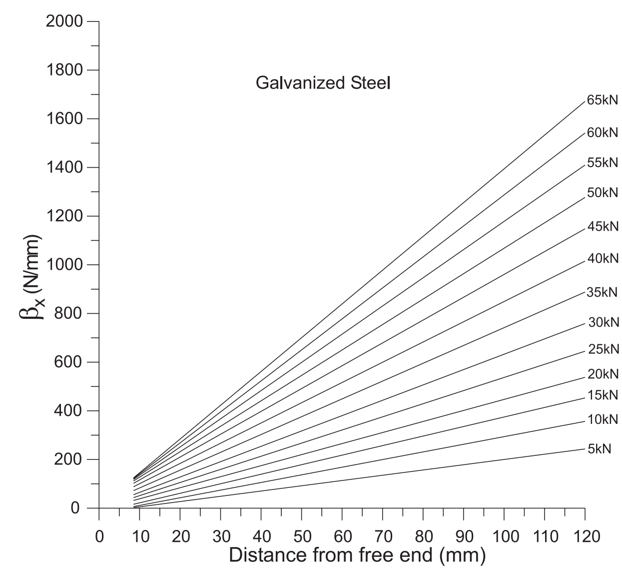

Fig. 8. Relationship between β_x and distance from the free end for the stages of loading of galvanized steel samples.

In separate tests, the stress–strain relationship for the steels used in this work has been established. The proportional limit is typically about 418 MPa while the modulus of elasticity (E) is 200 GPa. Stresses in the steel bars were therefore calculated as long as they were below the proportional limit, and thus they were purely elastic stresses. It was therefore possible to determine the value of β_x at any position x for a load P as long as the stress is below the yield stress. It was thus possible to establish, at any load, the relationship between β_x and x and this is shown for different stages of loading for both black steel and galvanized steel in Figs. 7 and 8.

It has been consistently found that the curve of β_x versus x within the length of the steel that was strain-gauged, can best be represented by a straight line. Fig. 9 shows the relationship between β_x and x for both types of steel for the case of 65 kN maximum load and where failure was manifest by splitting of the concrete. Although the results that correspond to galvanized steel are slightly superior to those that correspond to black steel, there was no statistical difference between the two groups. Therefore, the data from both groups were considered to belong to the same population. Using a curve fitting facility within the Grapher™ software, it is observed, with the exception of the locations in the immediate vicinity of the free end, that the relationship is mainly a straight line. The slope of this line is approximately 13.0 MPa. As has been discussed above, this value is the value of bond stress at this load. Since this value of the load is the value at which splitting was observed to have commenced, it is considered that the bond stress determined here is the bond strength developed between the bars and concrete. In the same manner, the bond stresses for several load intervals have been determined and are summarized in Fig. 10. It is

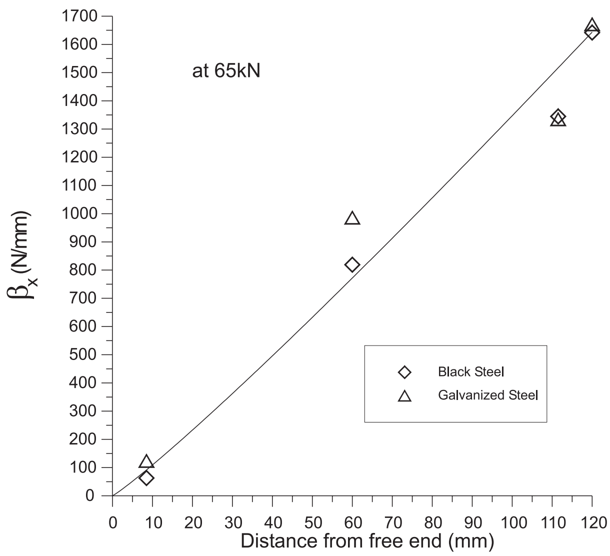

Fig. 9. Relationship between β_x and distance from the free end for the case of 65 kN maximum load.

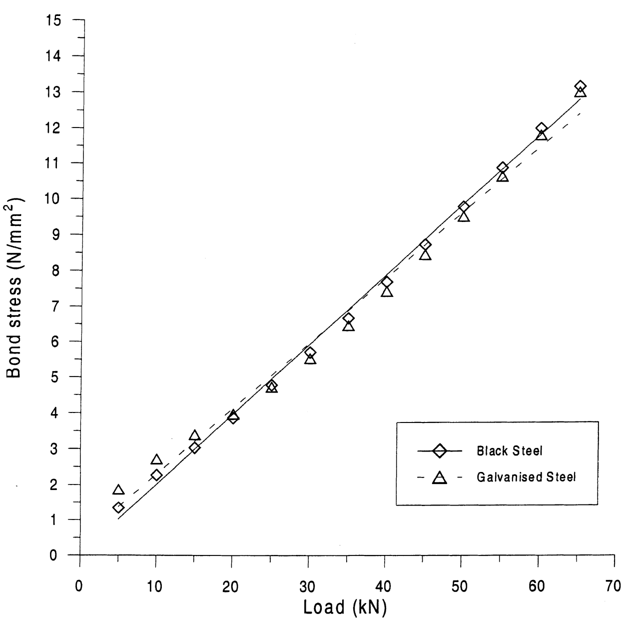

Fig. 10. Relationship between the load value and bond stress.

seen here that the relationship between bond stress and the load approximates a straight line for both black and galvanized bars.

These results again show that the galvanized steel did not behave differently to the black steel as far as bond is concerned. The maximum bond stress developed before bond failure occurred was nearly the same for both types of steel. Moreover, these results show that the bond stress along the embedded length is mostly uniform. They further confirm that the value of bond stress can be determined quite accurately, as would be anticipated, by simply dividing the load applied to the bar by the embedded surface area of the bar.

In the context of the comparison between black and galvanized steel, there is no significant difference between the two types of reinforcement at any stage of loading. Since the galvanized steel in both test series was not chromate treated, that is the black steel and the galvanized steel were tested in their natural condition, it is therefore reasonable to conclude that galvanizing of steel reinforcement does not adversely affect bond strength. Further, it can be concluded that galvanized steel does not require chromate treatment as far as its bond characteristics and load bearing capacity in concrete are concerned.

It must, however, be emphasized that the results obtained here are after 28 days of curing. It is thus apparent that the loss of bond that had been observed in 7 day old concrete is more than compensated for after curing for 28 days has occurred. This aspect of time dependancy is being studied by the authors in order to establish precisely the age at which the observed retardation in strength and bond is eliminated [27]. This aspect may be important in circumstances where early loading of some structures is required.

4. Conclusions

1. The results of this work confirm that there is no statistically significant difference in bond strength of ribbed black and galvanized bar reinforcement in 28 day cured concrete.

2. A significant reduction in bond ranging from approximately 25% to 42% was recorded for epoxy-coated ribbed steel. Thus the presence of the deformation ribs alone is not sufficient to compensate for the loss of adhesion and friction in epoxy-coated bars.

3. There is no evidence to suggest that the bond capacity of galvanized reinforcement in concrete has been reduced due to the possible effects of hydrogen evolution. Therefore, galvanizing of steel reinforcement did not adversely affect bond strength. It is further concluded that galvanized steel may not require chromate treatment as far as its bond characteristics and load bearing capacity in concrete are concerned.

4. By the age of 28 days, any adverse effect on bond for galvanized bars that may have been caused by early retardation has completely disappeared. The precise age at which this adverse effect disappears, is yet to be determined.

5. Bond stress has been demonstrated to be acting uniformly along the bar. Moreover, failure in bond between the reinforcement and the concrete has been associated with slip values of about 0.40 mm. This value

agrees with previous published data that also found that splitting bond failure occurred at nearly the same slip value, regardless of bar size.

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