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Impact of Corroded Bars and Spalling on the Bond Strength of Reinforced Concrete Structures

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Abstract. The mechanism of the composite materials in Reinforced Concrete Structure is such that they both have to undergo the same stress and deformation as the surrounding, so as to prevent discontinuity, slip, or separation of both materials. Bond strength results from a combination of several parameters, such as the mutual adhesion between the concrete and steel interfaces and the pressure of the hardened concrete against the steel bar or wire due to the drying shrinkage of the concrete. A total of 24 singly reinforced beams with plain 10mm and 12mm steel rods were cast for this study. The beams were designed to fail by flexure and to allow bond slips occur before ultimate failure. The spalling effect was simulated by debonding the bars along its entire length with PVC pipes thus restricting concrete and steel contact. The beams are then subjected to flexural test by inducing a point load on its mid span. The results of the investigation reveals a slight increase in the bond strength of about 2.68% with the use of already corroded bars and the bond strength decreased by 39.2% and 81.8% for bars that had 25% and 50% of their perimeters debonded. The increase in bond strength with the use of corroded bars can be ascribed to the increased frictional resistance owing to the corroded bars surface roughness. The loss of bond strength with decrease in contact area was as a result of the bar being unable to resist perpendicular stresses and was easily pushed out of the beam.

1. Introduction

Reinforced concrete structures are composite materials comprising of steel bars and concrete, the steel bars are embedded in the concrete zones prone to tensile stresses so as complement the low tensile capacity of plain concrete. The composite action of reinforced concrete structures is anchored on the fact that both constituent materials undergo similar stress-strain distribution and deformation when loaded [1]. During loading, the concrete receives the direct load and transmits it to steel bars via surface interaction between the two materials, the interface of both materials thus creates a medium for the stress transfer. This transfer is provided by the resistance to relative motion or slippage between the concrete and the rib face of the embedded steel bar. The resistance to slippage is defined as bond or bond stress [2]. It is the force per unit of nominal surface area of reinforcing bar acting on the interface between the reinforcing bar and the surrounding concrete [3].

The theory of bending in reinforced concrete assumes that concrete cracks in the region that bears the tensile stresses and after the cracking all tensile stresses are transferred to the steel bars. If the steel bar embedded is above a certain length which is the development length, it becomes



impossible for the concrete section to fail by bond slips as the steel bar yields before a slip occurs. The interface of both materials thus creates a medium for the stress transfer. The development length has been defined to be a function of the characteristic strength of the steel bar and concrete, diameter of the steel bar and the type of steel bar. The bond strength and the corresponding development length are related [4, 5].

When external forces are progressively applied to a reinforced concrete beam interfacial forces are generated and the capacity of efficient load transfer weakens. Irreparable damages spread along the surrounding concrete in the form of splitting and radial cracks. The concrete section fails owing to its inability to resist tensile stresses and as such cracks occur at the tension zones and the steel bar is subjected to tension. Complete bond failure occurs by the sliding of the steel relative to the concrete, this leads to immediate collapse of the beam as total slip cracks propagate all along the bar [6].

The discontinuity or slip in the steel –concrete bond can be as a result of various factors such as adhesion between the concrete and the steel, gripping effect from the dry shrinkage of concrete, frictional resistance in the interface, concrete quality, reinforcement properties such as diameter and spacing, and the load history of the structure [7]. Once the slip occurs, bond is mainly provided by mechanical interlock between the ribs and the surrounding concrete, which is dependent on the gripping force between the concrete and the contact surface area [8, 9].

Steel is thermodynamically unstable; it tends to return to its original ore state on exposure to certain elements in the environment. Concrete that embeds the steel bars in reinforced structural elements generally provides an alkaline environment for the steel, which normally creates a passive layer on the reinforcement. However under certain conditions such as the presence of chloride ions in the concrete mix, carbonisation of the concrete, cracks resulting from mechanical loading, error in water cement ratio; the protective cover is breached [10, 11].

The internal corrosion of steel will result in the loss of steel cross section, reduction of mechanical strength of the steel and impair load distribution between the materials [12-14]. This is also followed by the formation of corrosion materials that would induce radial stresses on the surrounding cover, leading to the loss of adhesion at the concrete-steel interface and subsequent falling off of the concrete in the tension zone thus exposing the surface of the steel bar [15, 16]. Eventually, additional cracks may develop leading to spalling of the cover, i.e. large fragments of concrete falling off, thus leaving the reinforcement bars exposed, which in turn accelerate the corrosion rate [17, 18].

This research investigates the effect of corrosion of reinforcing steel bars on the bond strength of reinforced concrete structures by comparing the bond strength of beams reinforced with pure and corroded steel through flexural test. The study further simulate the action of spalling by debonding the bottom areas of steel bars and investigate its effect on the bond strength using flexural test on reinforced concrete beams.

2. Methodology

2.1 Materials and concrete mix design

The concrete was designed to have a characteristic strength of 35N/mm², this is to ensure that the beam fails by flexure before its shear capacity is reached. Sharp sand passing through the 5mm sieve was used as the fine aggregates while crushed granites of 12 mm diameter was used as coarse aggregates. Ordinary Portland Cement (OPC) was used with a mix ratio of 1:1.5:3. The mix consisted of 454.54Kg/m of cement, 681.82Kg/m of fine aggregates and 1363.64 Kg/m of coarse aggregates with 0.6 water cement ratio. The mixing of materials was done by volume to eliminate errors due to variations in proportion of voids contained in a specific volume. The steel that was used for the study is mild yield steel of diameter 12mm and 10mm with characteristic strength of 250N/mm². Both corroded and un-corroded specimen of the selected diameters were used.

2.2 Specimen selection

The beams dimensions were selected from the results of an optimization of the various possible configurations. The cross section of the beam should provide sufficient shear resistance, the length should be sufficient enough able to generate enough moment to cause failure by flexure and yet considerably lesser than the development length of the steel so as to ensure bond slip rather than yielding of the embedded steel bars at failure. The specimen selected thus was 100 x150 x500 mm beam. Other specifications of the reinforced beam selected are:

The shear capacity of the beams = 1.1 N/mm²

Expected shear stress = 0.73 N/mm²

Development length = 363.1mm

Embedded length = 250mm.

2.3 Simulation of the spalling effect

The simulation of the falling off of the concrete cover was done by restriction of the formation of the bond between the bottom steel section and the surrounding concrete. The provision of Polyvinyl chloride (PVC) pipes along the entire length of the bar was sufficient in preventing the interaction between the steel and the concrete and possible stress transfer. Adjusting the perimeter of the PVC pipes was done to vary the degree of spalling. The PVC pipes used were sawn in such a way they encased 25% and 50% of the bars perimeter to simulate the pop off and de-lamination effect of spalling respectively.



Figure 1. 50% and 25% debonded steel bars.

2.4 Casting of the beams and cube

A total of 24 beams of 100mm x 150mm x 500mm of concrete were cast, each having a single bar embedded in it and cured for 28 days in the laboratory according to BS EN 12390-2 [19]. Six beams were cast with surface corroded reinforcements; three of the beams were reinforced with 12mm bars and the remaining beams with 10mm bars. Another six beams were cast with the reinforcements debonded at the bottom section in such a way that only three quarters of the bar section to bond with the concrete. This was used to simulate the pop out spalling effect. Three beams had embedded in them 12mm bars and three beams had 10mm bars embedded in them.

Further, six beams with reinforcements de-bonded at the bottom section in a manner that allowed about half of the bar cross section to form bonds with the concrete, were cast. These were used to simulate the delaminating spalling effect. Three beams had embedded in them 12mm bars and three beams had 10mm bars embedded in them. Six control beams were reinforced with bars that were neither corroded nor debonded. Three of the beams were reinforced with 12mm bars and the remaining beams were reinforced with 10mm bars. The beams were labelled in alphabets A-X, and grouped in threes.

2.5 Corrosion Level Test

The corrosion level test was done so as to estimate roughly the state of the steel bars as regards corrosion. This estimate was determined as a factor of the mass lost by the corroded steel bars. For the test, the mass of a meter length of the corroded steel bar was compared against the mass of standard steel bar.

2.6 Bending Test

The beams were subjected to a one-point bending test. A point load was applied at mid span until failure due to flexure happened. At the point of failure the bond between the steel and concrete is broken and as such there is no transfer of stresses. The beam failed as a result of the slip which occurs at the steel-concrete interface, thus impeding the effective transfer of stresses between these two materials. Concrete being weak in resisting tensile loads thus failed. Figure 2(a) shows crack pattern of a failed beam and 2(b) shows bond slips in a loaded reinforced beam. Figure 3 shows the flexural test on beam in progress.

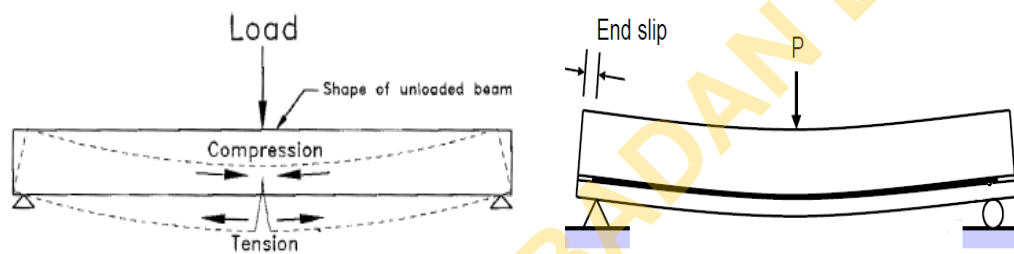


Figure 2 (a). Cracked pattern of a failed beam and (b) bond slips in a loaded reinforced beam.



Figure 3. Flexural test on beam in progress.

2.7 Computation of Bond Stress

From the experiments, the force causing failure was determined. For the computation of the bond stress the following parameters have to be generated; the shearing force, the moment induced by the force, and the lever arm. To do this a programme was developed with MathLab that performed the necessary calculations assuming an equivalent rectangular stress block distribution in the concrete section.

3. Results and discussions

3.1 Dimensional analysis of the beam

The compressive strength on the 28th day of curing carried out on concrete cubes simultaneously cast with the beams was 41.2N/mm². The anchorage length required so as to allow failure by bond should be less than the development length of the bar.

$$l = \frac{0.217f_y}{f_{bu}} * \phi$$

$$\beta = 0.28 \text{ for plain bars, } f_{bu} = \beta\sqrt{f_{cu}}$$

$$l = \frac{0.217 * 250}{0.28\sqrt{41.2}} * 12 = 363.1mm$$

The beams composition and geometric properties allows it to fail in flexure before the shear capacity of the concrete is exceeded. The beam specimen with dimensions 100 x 150 x 500 all in mm was selected as it satisfies all requirements. The selected reinforced beam will fail at a force less than the maximum deliverable by the machine. The anchorage length should thus be < 363.1mm. The anchorage length provided is less than the development length as such bond failure thus occurs before the yielding of the bar.

From the results the moment capacity of the specimen selected = 2.0 KN-m and the force that would generate this moment is <100 KN; The shear stress on the beam is 0.727354 N/mm² and it has a shear capacity of 1.072N/mm². Thus implying that the failure of the beam section would be by flexure and not shear. A beam length of 500mm was selected, having approximately 250mm from load application to the beams support on both ends. 250mm < 363.1mm implying that the development length has not been reached. Thus. failure would be as a result of bond failure after the flexural cracks appear.

3.2 Corrosion level test

The corrosion level of reinforcing bars can be measured as a function of the mass lost owing to the interaction between the steel and environment. The extent of corrosion of the locally available bars used was minimal as there was no considerable difference in the mass per meter run of the corroded and un-corroded specimen. The bars were just surface corroded and the loss in mass negligible. Table 1 shows the mass of the 12mm and 10mm mild steel bars used for the investigation and the weight of standard 12mm and 10mm round mild steel bars.

Table 1. Mass of steel bars

Bar diameter (mm)	Measured mass(Kg/m)	Standard mass(Kg/m)
10	0.63	0.62
12	0.67	0.882

3.3 Flexural test on beams

When loaded the control beams maintained flexural stiffness for a while, cracks however began to appear mid span of the concrete indicating flexural failure. The beams continued to bear load even after the formation of the initial cracks thus it can be deduced that the steel bars resisted most of the tensile stresses induced by the load till final failure of the beam occurred. The beams failed when the flexural cracks propagated along the whole depth of the beam. This is as a result of the bond slip as the beams lack the required embedded length needed for the bar to develop its full yield strength when loaded .Thus the flexural failure occurs is as a result of the bonds inability to transfer stresses.

The beams reinforced with bars that had a quarter and half of the bars perimeter debonded behaved as the control at the initial stage of the loading process, this can be explained from the fact that the concrete section is the member in direct contact with the load and thus bears most of the load before transfer and cracking of the concrete section. However the difference in the behaviour of these two groups was noticed in the abrupt manner of failure of the beams reinforced with debonded bars. The steel bars continued to resist the applied load after the formation of the initial cracks but the rate of progression of the cracks to the top of the beams was abrupt and failure was instantaneous as opposed to the gradual manner of the normal steel yield response. It can thus be deduced from this behaviour that the bars have reached their ultimate slip and no stress transfer occurs between the steel and concrete. The beam has no residual strength, and also the force causing the failure is not sufficient to cause steel to yield. Table 2 below shows the results of the flexural tests carried out on the beams.

Table 2. Flexural strength of beams

Beam Label	Reinforcement Use	Average Force(kN)
A, B, C	12 mm uncorroded bars	23.67
D, E, F	10 mm uncorroded bars	22.20
G, H, I	12 mm corroded bars	33.20
J, K, L	10 mm corroded bars	22.36
M, N, O	12 mm $\frac{1}{4}$ debonded bars	18.86
P, Q, R	10 mm $\frac{1}{4}$ debonded bars	15.40
S, T, U	10 mm $\frac{1}{2}$ debonded bars	12.46
V, W, X	12 mm $\frac{1}{2}$ debonded bars	12.87

3.4 Bond stress

From the analysis of the failure force obtained from the experiments with the MathLab program developed; the following data were derived; the moment generated in the beam, the k factor, the lever arm of each beam, the shear force in the reinforcing bars and the value of the bond strength. Table 3 shows the average values of each parameter for each of the beam groups. The discussion of results based on the effect of use of corroded bars for construction, spalling on bond strength

Table 3. Results from MathLab analysis of beams

Beam Label	Force (KN)	Moment (KN-m)	K	Z (Mm)	V (KN)	Bond Stress (N/mm ²)
A, B, C	23.7	2.96	0.0874	65.80	12.00	4.84
D, E, F	22.2	2.78	0.0316	67.62	11.10	5.22
G, H, I	33.2	4.15	0.1217	64.48	13.65	5.70
J, K, L	22.4	2.79	0.0804	67.56	11.35	5.36
M, N, O	18.87	2.35	0.0697	68.41	9.42	3.69
P, Q, R	15.4	1.93	0.0554	70.12	8.20	3.75
S, T, U	12.47	1.60	0.0458	70.70	6.23	2.87
V, W, X	12.87	1.60	0.0475	69.86	6.43	2.44

3.5 Effect of the use of corroded bars for construction

The average bond strength for the beams reinforced with 12mm uncorroded bars was 4.84N/mm² while the average bond strength for the beams reinforced with corroded 12mm bars was 5.70N/mm². The loss in bond strength in percentage thus becomes;

$$\% \text{ difference in bond strength} = \frac{5.70 - 4.84}{4.84} * 100\% = 17.77\%$$

For the 10mm bars; the average bond strength of the beams with the uncorroded bars was 5.22N/mm² and an average bond strength of 5.36N/mm² for the beams reinforced with surface corroded bars. The loss in bond strength that resulted from the increase in bar diameter can be represented thus;

$$\% \text{ difference in bond strength} = \frac{5.36 - 5.22}{5.22} * 100\% = 2.68\%$$

It was observed that the use of surface corroded bars brought about an increase in the bond strength formed between the steel and the concrete. This holds for both 12mm and 10mm bars. The reason for this observation can be ascribed to the surface roughness of the corroded bars. The corroded bars has iron oxides deposits on its surface giving it a bit of deformity. Thus when embedded in concrete a better grip is formed which results in an increase of the sliding friction resistance. This makes the composite structure less susceptible to bond slips. The variations in the loss in bond for the 10mm and 12mm bars is owing to the fact that the corrosion process in the bars were not monitored and as such had various degrees of surface corrosion thus leading to the non-uniformity of the surface roughness.

3.6 Effect of spalling on bond strength

The control beams reinforced with 12mm bars and no loss in the concrete cover had an average bond strength of 4.84N/mm² and at the loss of 25% of the concrete cover an average bond strength of 3.69N/mm² was recorded. The percentage loss in bond strength thus becomes;

$$\% \text{ difference in bond strength} = \frac{4.84 - 3.69}{3.69} * 100\% = 31.17\%$$

At the loss of 50% of the concrete cover, the bond strength is 2.44N/mm² ;

$$\% \text{ difference in bond strength} = \frac{4.84 - 2.44}{2.44} * 100\% = 98.36\%$$

% For the 10mm bars, the average bond strength of the control beams was 5.22N/mm², at the loss of 25% of the surrounding concrete cover, the bond strength = 3.75N/mm²;

$$\% \text{ difference in bond strength} = \frac{5.22 - 3.75}{3.75} * 100\% = 39.2\%$$

At the loss of 50% of the concrete cover of the 10mm bars, the average bond strength = 2.87N/mm²,

$$\% \text{ difference in bond strength} = \frac{5.22 - 2.87}{2.87} * 100\% = 81.80\%$$

It can be concluded that there is a linear decrease in the bond strength with the loss of the concrete cover for both 10mm and 12mm bars. The loss of the bottom concrete section renders the beam unable to counteract the perpendicular stresses that are developed when the beam is loaded. If only a small portion of the perimeter is not confined, then the perpendicular stress components can be equilibrated by the other sections of the perimeter. In the case where half of the perimeter of the bar is unconfined, then there will be nothing to resist the perpendicular stress components of the bonded perimeter. This will result in the pushing out and sliding of the bar when loaded

4. Conclusion

From the study carried out, the following conclusions were reached;

- The use of locally available surface corroded steel bars as reinforcements in casting marginally increases the strength of the bond formed as the surface roughness increases the frictional resistance thus allowing for better grip and anchorage.
- Spalling off of the concrete cover deteriorates the bond strength. First exposing the steel bars to adverse environmental condition thus creating room for more corrosion.
- As the beams cast with 50% of the steel bars perimeter de bonded failed as though they were un reinforced. The beams lacked flexural rigidity and failed immediately the concrete began to crack. A clear indication that the steel bars played little or no role in the load bearing process.
- The load bearing capacity of the beams decreased exponentially with the loss in 25% and 50% of reinforcing bars concrete cover and a slight reduction is also observed in the load bearing capacity with the reduction of the reinforcing bar diameter.

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