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Original Research Article

Effect of Corrosion on Bond Strength of Tension Lap-Splices

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Abstract

This study aimed to investigate the effect of corrosion on the bond strength of tension lap splices in reinforced concrete beams. The test variables were the corrosion level (0%, 2.5% and 5.0%) and the concrete cover to bar diameter (c/d) ratio (c/d=1.5, 2.0, and 2.67). The observed failure mode was splitting of the concrete cover on the tension side. An accelerated corrosion technique was used to corrode the reinforcing lap-spliced bars in a reasonable time frame. Corrosion did not affect failure mode but caused a reduction in the ultimate strength and deflection at failure. The bond strength was reduced by 15% to 26% at 2.5% corrosion level and by 21% to 38% at 5% corrosion level depending on the c/d ratio. A model was proposed to predict the bond strength of corroded tension lap-splices in concrete. The model results correlated well with the measured bond strength.

Keywords: Lap-splice; bond; Corrosion; Concrete.

INTRODUCTION

Bond is the interaction of two materials, concrete and steel reinforcement (Bond is an adhesion of concrete or mortar to reinforcement or other surfaces against which it is placed, including friction and mechanical interlock). Bond is the most important factor that maintains the integrity in the composite material known as reinforced concrete (RC). In (RC) structures, the external applied load is carried by the concrete and the internal steel reinforcement. Forces are transferred to the steel reinforcement from the surrounding concrete by shear stresses along the steel to concrete interface. Inherent in the analysis of a reinforced concrete section is the assumption of strain compatibility which means that the strains in the concrete and steel are equal at the location of the steel reinforcement. This implies perfect bond between the concrete and the steel reinforcement. The bond transfers from the concrete to the steel reinforcement by: 1) Chemical adhesion between the concrete and the steel bar, 2) Friction due to small indentations on the surface of the steel bar and 3) mechanical interaction between the ribs of the bar and the surrounding concrete (ACI Committee 408 2003). It has been well established that the bond strength of deformed bars is affected by: concrete cover, bar spacing, amount of confining transverse reinforcement, splice and development length, concrete compressive strength, bar size, and relative rib area (fib 2000). The effects of splice length, concrete cover and the bar size are the principal factors studied in this paper. A variety of test specimen configurations have been used to study the bond between the reinforcing bars and concrete. The most common configurations are pullout test specimen, beam end specimen, beam anchorage specimen, and splice specimen (ACI Committee 408 2003). The splice specimen represents a larger-scale specimen designed to measure the bond in lap-spliced bars that are located in the constant moment region. The splice specimens simulate a member with flexural cracks and known bonded length. The splice specimen provides a realistic measure of the bond strength in the actual RC structure.

Corrosion is the number one deterioration mechanism that rapidly decreases the service life of reinforced concrete structures. Many structures in severe environments have experienced an unacceptable loss in serviceability earlier than anticipated due to corrosion of their steel reinforcement. Corrosion causes deterioration in reinforced concrete structure by reducing the cross sectional area of the reinforcing steel bars and by causing cracking and spalling of the concrete due to expansive forces from the corrosion products, and hence leading to a loss of structural bond between the reinforcement and concrete (ACI

Committee 222 2010). In previous work, the effect of corrosion on bond strength was found to slightly increase the ultimate bond strength at low levels of corrosion and then bond strength decreases significantly as corrosion increases (Amleh and Mirza 1999 and Craig 2005).

The effect of corrosion on the bond strength of steel reinforcement in concrete was studied by Al-musallam *et al*. (1996). The experimental program was designed to evaluate the bond strength, free-end slip, and modes of failure of the test specimen, in pre-cracking, cracking, and post cracking stages, as well as the effect of different crack widths and degradation of rib profile for different corrosion level. Based on the analysis of test result, the following conclusions were made: Up to 4% corrosion level, the ultimate bond strength increased by about 17 % and rebar slip decreased. Bond strength increased because corrosion products increased the rebar roughness and the confinement of the concrete. Above 5% corrosion level, the bond strength decreased gradually for additional 1% corrosion level and thereafter decreased rapidly.

Amleh and Mirza (1999) investigated the corrosion influence on bond between steel and concrete. 14 tension specimens (100 mm diameter and 1-m long) were tested. Based on the test results the authors found that corrosion causes a significant reduction of the interlocking forces between the ribs and the concrete keys due to the deterioration of the reinforcing bar ribs. As the corrosion level increases, the width of transverse cracks due to corrosion increases and causes a reduction of bond between the reinforcing steel bar and the concrete.

Stanish *et al.* (1999) assessed the effect of corrosion products on the bond strength. A total of ten one-way slab specimens were cast having cross section of 350 mm (width) by 150 mm (height) and 1300 mm span. The degree of corrosion in the tensile reinforcement was varied: 0%, 2.0%, 5.0%, 8.0%, and 10% mass loss. The test result illustrated that bond resistance was adversely affected by corrosion. The corrosion products led to cracking that relieved the internal pressure and weakened the anchorage of the reinforcing steel; creating a weak layer of corrosion product that will break off under low stress levels.

Wei-liang and Yu-xi (2001) examined the effect of reinforcement corrosion on the bond and flexural behavior of reinforced concrete beam. The test variables included: plain vs. deformed bars, anchorage length, and type of loading: pullout vs. beam tests. Based on the test results, the following conclusions were made: initially, the bond strength between the deformed bars and concrete increased up to a 5% corrosion level. After 5% corrosion level, the bond strength declined consistently because of the reduction in the profile of the bar ribs. With higher corrosion level, the failure mode of corroded reinforced concrete beams changed from a ductile to a brittle failure. The distribution of cracks in the corroded beams became more concentrated.

Fang (2004 and 2006) evaluated the effects of corrosion on the bond and bond–slip behaviour using pull-out specimens. Test variables included: corrosion level (0% to 9%) and transverse reinforcement. Based on the test results, the author found that the bond strength of the deformed bar without confinement was very sensitive to the corrosion level. Bond strength decreased rapidly as the corrosion level increased; bond strength at 9% corrosion was only one third of that of the non-corroded specimens. The exception was that when the corrosion level was very low when bond strength increased as the corrosion level increased. For confined deformed bars, a corrosion level of 4% had no substantial influence on the bond strength. But substantial reduction in bond took place when corrosion increased thereafter to a higher level of around 6%.

To the author's knowledge, no study was reported in the literature on the effects of corrosion on tension lap-spliced RC beams. This study aims to fill this gap in the literature.

Research significance

Various structures are subjected to corrosion of steel reinforcement at critical region, which can cause pre mature failure before reaching its strength capacity. This fact is often over looked in the analysis and design of concrete structures. Also, the changes in the corroded structural behavior due to wrapping of the critical region, lap-splices, with FRP sheets have not been investigated. This study investigates the effect of different corrosion levels and cover-to-diameter ratio on the bond strength of tension lap splices in concrete. This study proposed a corrosion reduction parameter for the ACI bond stress equation to predict the effect of corrosion at critical region. This reduction parameter, however, should be validated in future research where more variables such as steel bar size, and cover-to-diameter ratio are investigated. In this study, lap-splices specimens were used to provide realistic measures of bond strength in actual structures according to ACI 408-03.

Experimental investigation

The test program comprised of nine lap-spliced reinforced concrete (RC) beams. The beams were divided into three series based on their concrete cover to bar diameter (c /d) ratio of 1.5, 2.0, and 2.67. Each group consisted of three beams: one beam in each group was not corroded (control beam) and two beams were corroded to a 2.5% and 5.0% (theoretical) mass loss (low and high corrosion levels). At the end of the corrosion phase, all beams were tested in flexure.

Test specimen

Figure 1 shows the specimen geometry and reinforcement details. Table 1 gives the details of the test specimens. The specimens were designed according to ACI 318 (2008). The overall geometry and reinforcement configurations for all the beams were the same. The beam cross-section was 200 mm wide x 300 mm [8 in wide x 12 in deep]. The beam length was 2000 mm [79 in] with a span of 1800 mm [71 in] between the supports. The length of the constant moment region or the distance between the two applied loads was 600mm [24 in]. The tensile reinforcement consisted of two deformed bars (15M or 20M) spliced at mid span. The splice length for the bars was 300mm [12 in] to develop a steel stress in the tensile reinforcement less than its yield strength to ensure a bond splitting mode of failure. The transverse reinforcement in the shear spans of the beams consisted of 10M stirrups at a spacing of 100 mm [4 in]. There were no transverse reinforcements in the splice region to examine the bond behaviour of the corroded steel reinforcement in concrete without the contribution of transverse reinforcement. A 15 mm [0.6 in] diameter hollow stainless steel bar was placed at 120 mm [5 in] from the soffit of the specimens in all the corroded beams. The hollow stainless steel bar was used as a cathode terminal for the corrosion process.

Material properties

Two concrete mixes, unsalted and salted, were used with same w/c ratio of 0.55. The 28-day compressive strength of both was determined the compressive strength of the concrete at 28 days was 43 MPa [6237 psi]. The yield strength of the tension reinforcement was 510 MPa [73969 psi] and the corresponding yield strain was 2550 $\mu\varepsilon$.

Accelerated corrosion technique

An accelerated corrosion technique was used to corrode the reinforcing lap-spliced bars in a reasonable time frame. The estimated time for theoretical corrosion level was calculated based on Faraday's law with a specified current density impressed through the steel reinforcement (Jones 1996). The current density was 150μA/cm^2 as recommended by Maddawy and Soudki (2003) to avoid the damaging impudence of high current on the steel and concrete interfacial bond. A galvanostatic technique was used by impressing an electric current through the main reinforcement. The beams were connected in series with power supplies to maintain the same constant current density in the beams. The current flows through the circuit with the main reinforcement acting as an anode while a hollow stainless steel bar was placed close to the neutral axis of the beam to act as a cathode terminal. The concrete in the desired corrosion location was pre-mixed with a 2.25% chloride by weight of cement. The specimens were placed in a corrosion chamber with a high relative humidity (mist condition close to 100% relative humidity) at room temperature. Figure 2 shows the location of salted concrete and the electrical connection of the corroded beam. To determine the actual mass loss in the reinforcing steel bars, the corroded tension steel bars in the lap-splice were extracted to perform gravimetric mass loss measurements according to ASTM Standards G1-90 (ASTM 1990). The determination of the actual mass loss was done by taking the difference in weight between the corroded and un-corroded steel bars cut from the same batch to a representative length and subjected to the same procedure as per ASTM Standards G1-90.

Test setup and instrumentation

All beams were loaded in four-point bending using a servo- hydraulic actuator with a capacity of 332 kN [732 kips]. The loading configuration had a clear span of 1800 mm [71 in] and a constant moment region of 600 mm [24 in] (see Figure 3). The lap-spliced bars within the constant moment region were subjected only to tension forces using this loading configuration. Strain gauges were mounted on the steel reinforcement and were used to determine the strain distribution along the lap-splice zone. Two slots (measuring 2 x 6 mm) [0.08 x 0.24 in] were made using a milling machine through the reinforcing bars at the middle and the end of the lap-splice length. The strain gauges were installed inside the slots that were made through the reinforcing bars. Wax was used to cover the slot to protect the strain gauge from rust during the corrosion process. Figure 4 shows the location of the strain gauges on the lap spliced bar. Figure 4 shows the slot configuration and the strain gauge installation inside the slot of the reinforcing bars. One strain gauge (60 mm) [2.4 in] was mounted on the concrete compression surface at mid span. One linear variable differential transformers (LVDT) was used to measure the midspan deflection (Figure 3).

EXPERIMENTAL RESULTS AND DISCUSSION

Corrosion cracking

Concrete cracking due to corrosion of the lap splices occurred because the tensile stresses caused by the expansion of the corrosion products exceeded the tensile strength of the concrete. These cracks formed longitudinally along the corroded zone and were parallel to the corroded main reinforcement. Two crack patterns were observed. The first crack pattern consisted of two longitudinal cracks at the soffit of the beam along the corroded region parallel to the main corroded reinforcement. The second pattern consisted of one crack at the bottom of the beam and one crack on the side of the beam. The possible reason for the different cracking patterns is that the corrosion products are not uniformly distributed around the cross section of the bar.

The crack length and width depends on many factors including the concrete strength, the concrete cover, and the corrosion level. The maximum crack width varied among the different beam series because of the differences in the concrete cover to bar diameter (c/d) ratio and the level of corrosion Figure 5 shows the effect of corrosion level on the crack patterns for a beam with (c/d) ratios of 1.5. The figure is divided into two parts: (a) shows the crack pattern for 2.5% theoretical corrosion level and (b) shows the crack patterns for 5% theoretical corrosion level. The corrosion level had a significant effect on the crack widths. Beams with (c/d) =1.5 had maximum crack widths of 0.5 mm [0.02 in] and 0.8 mm [0.02 in] for the 2.5% and 5.0% corrosion level, respectively. Beams with c/d = 2.0 and 2.67 had maximum crack widths of 0.4 mm [0.02 in] and 0.6 mm [0.025 in] for the 2.5% and 5.0% corrosion level, respectively.

The concrete cover to the bar diameter (c/d) ratio also affected the crack width. There were differences in crack widths for (c/d) ratios of 1.5, 2.0, and 2.67, respectively. Based on the test results, it is clear that increasing the bar diameter for a given concrete cover resulted in higher crack widths. For beams with c/d =1.5, the maximum crack width was 0.8 mm [0.02 in] and for beams with c/d =2.0, it was 0.6 mm [0.025 in]. For a given concrete cover (c = 30 mm) [c = 1.2 in] , the maximum crack width for beams reinforced with 20M steel bars was greater than that for beams reinforced with 15M bars. Increasing the concrete cover for the same bar diameter decreased the crack width. For beams with 15M [0.6 in] bars, as the concrete cover increased, the crack width due to corrosion decrease at the same corrosion level. The crack width of 30 mm and 40 mm was 0.6 mm [0.025 in] and 0.5 mm [0.02 in], respectively.

Mass loss analysis

After load testing the beams to failure, the corroded tension steel bars in the lap-splice were extracted to determine the actual mass loss due to corrosion. The mass loss analysis was carried out on 150 mm [6 in] long coupon according to the procedure given in ASTM standard G1-03 designation C.3.5 (2011). The theoretical and experimental mass losses along with the attack penetration depth are given in Table 2. The maximum measured mass losses for all the beams were 2.61% and 3.82% for 2.5% and 5% theoretical mass loss, respectively. The difference between the actual and the theoretical mass loss is possibly due to the way the current was induced in the lap-spliced bars. The electrical current was connected to a steel bar at one end of the beam and the current travelled to the second bar through the lap splice. It seems that the efficiency of electrical connectivity in the lap-splice decreased with higher corrosion level because of the formation of corrosion products on the spliced bars.

Failure mode

Figure 6 shows the typical failure modes of the beams. All beams failed by bond splitting below their flexural capacity. The bond splitting occurred in the concrete cover surrounding the tension lap-splices. Upon failure by bond splitting, an abrupt loss of the load-carrying capacity occurred and was accompanied with spalling of the concrete cover.

Load deflection behaviour

The load-deflection curves for beams with (c/d) ratio equal to 1.5, 2.0 and 2.67 are compared in Figures 7, 8 and 9, respectively. For each c/d ratio, the flexural stiffness of the beams was almost identical regardless of the corrosion level. There was a consistent decrease in the cracking load and maximum load as the corrosion level increased. Beams with c/d ratios of 1.5 and 2.0 exhibited similar behavior. As corrosion increased from 2.5% to 5.0%, the cracking load decreased by 10% to 20%; the ultimate load decreased by 21% to 39% of the control. For beams with c/d ratio of 2.67, as corrosion increased from 2.5% to 5.0%, the cracking load decreased by 30% and 10% of the control while the ultimate load decreased by 12% and 22% of the control beam. The load dropped rapidly after reaching the peak and the deflection slightly increased. Corrosion caused a reduction in midspan deflection at ultimate load thus affecting the ductility of failure of a corroded beam. The maximum reduction in deflection was 19% at 5.0% corrosion. Table 3 gives a summary of the test results.

Measured strain

All the beams had a similar strain response. Figure 10 shows a typical load- strain response for one bar in Beam N-1.5. The strain response was characteristic by a tri-linear response with pre-cracking, post-cracking and post-slip phase. Initially, the concrete was un-cracked and resisted all the tensile forces. At 42 kN [93 kips], the concrete cracked and the tensile forces carried by the concrete were transmitted to the steel reinforcement. The steel strain at onset and cracking was 267 με. As the load increased from 42 kN [93 kips] to 58 kN [128kips], the steel strain increased from 267 με to 985 με. The main reinforcement was slipping due to the splitting cracks that occurred at the end of the lap-spliced bars at load levels between 42 kN [93 kips] and 58 kN [128 kips]. As loading continued further, the steel strain increased almost linearly until reaching a peak load of 120 kN [265 kips] with a corresponding steel strain of 2400 με which is lower than yield strain (2550 με).

The strain profile along the lap splice was almost identical for all the beams. The strain gauges on a steel bar were located at the middle and the ends of the lap splices. Figures 11a and 11b show a typical strain distribution with distance along the lap splice for two bars in beam N-1.5. The strain profile was almost linear along the lap-splice region. The maximum strain which ranged between 1980 and 2450 με was at the loaded end of the spliced steel bars and was lower than the yield strain of the steel bars of 2550 με. It is evident that below a load of 60 kN, the strain in the lap-splice were small. Above 60 kN, there was a sudden jump in strain values which reflect the transfer of tensile forces from the concrete to the steel within the lap-splice

Bond strength

The average bond stress between the bar and concrete was calculated based on the assumption that the tensile force in the bar is resisted by a uniform bond stress along the full length of the splice. The tensile force developed in the longitudinal steel reinforcing bars in the beam specimen was calculated according to elastic cracked section analysis. The analysis assumes linear concrete stress-strain behaviour and ignores the tensile strength of the concrete below the neutral axis. Using this approach, the bond strengths of the tension lap-spliced bars were calculated for the different beams as given in table 4. Corrosion reduced the bond strength of the tension lap-splice in concrete. The decrease in bond strength was non-linear with corrosion level and highest for beams with the lowest c/d ratio of 1.5. At low corrosion level (2.5%), the bond strength decreased by 25%, 20% and 14% for tension lap-spliced bars in beams with c/d = 1.5, 2.0 and 2.67, respectively. While at medium corrosion level (5.0%), the decrease in bond strength was 41%, 33% and 23% for lap-spliced bars in beams with c/d = 1.5, 2.0 and 2.67, respectively.

Empirical model for bond strength

The bond strength of a reinforcing bar in concrete is influenced by the diameter of the steel reinforcing bar, the thickness of the concrete cover, the concrete strength and the amount of transverse reinforcement (stirrups). In the current work, there was no transverse reinforcement (stirrups) confining the lap spliced bars in the constant moment region; hence the contribution of transverse reinforcement to the bond strength is nil.

ACI Committee 408 (2003) proposed Equation (1) based on extensive experimental data to estimate the maximum bond strength of a deformed steel bar in concrete.

$$U_c = \frac{T_c}{\pi d_b l_s} \quad (1)$$

$$\frac{T_c}{f'_c{}^{1/4}} = [59.9 l_s (c_{min} + 0.5 d_b) + 2400 A_b] \quad (2)$$

Where

T_c : The concrete contribution to the bond force (K)

f'_c: The concrete compressive strength (psi)

l_s: Development length or Splice length (in)

c_{min} : Minimum concrete cover (in)

d_b : Bar diameter (in)

A_b : Bar area (in^2)

The effect of corrosion on bond strength may be included as a reduction factor applied to equation (1). The reduced bond strength (U_t) considering effects of corrosion can be expressed as:

$$U_t = R_2(U_c) \quad (3)$$

To determine the reduction factor due to corrosion, the bond stresses versus (c/d) ratios are plotted for the different corrosion levels as shown in Figure 12. Using the data plotted in Figure 12, the reduction factor due to corrosion can be expressed as follows:

$$R_2 = (1 - 10 * m\%) \quad (4)$$

Where:

R_2: The reduction factor due to corrosion

m% : Percentage of theoretical mass loss

The predicted bond stresses using Eq.4 were compared to the measured values for all specimens as given in Table 5. It is evident that the predicted bond strength values correlated well with measured values. The model results were conservative with the ratio of predicted /experimental values ranging from 1.07 to 1.33 for the control (un-corroded) lap-splices while the predicted /experimental ratios for the corroded specimens ranged from 1.07 to 1.17.

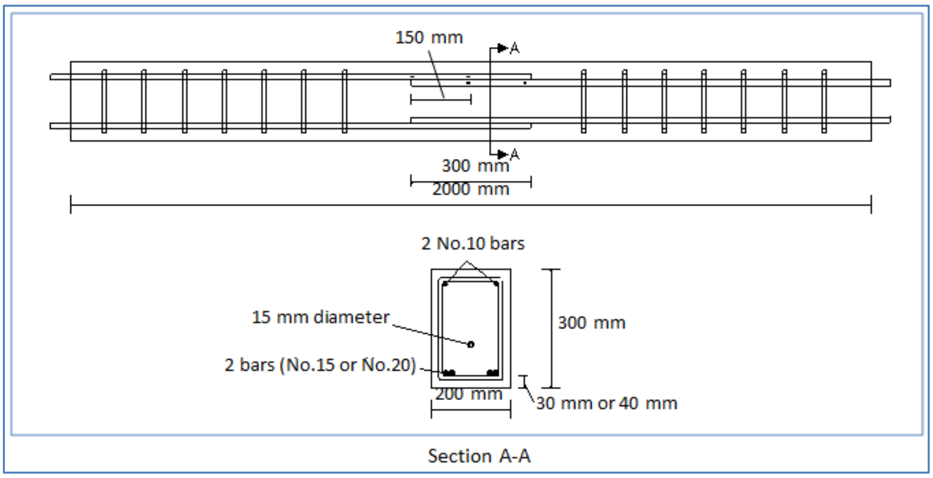


Fig-1: Specimen geometry and reinforcement details. (1 mm = 0.03937 in)

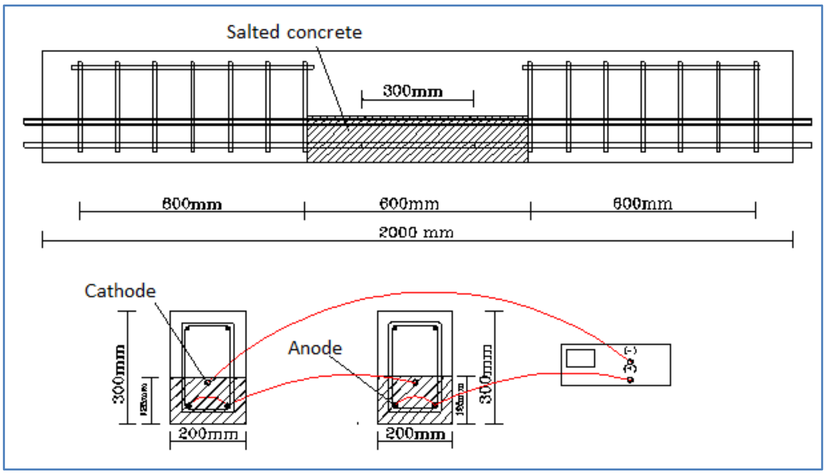


Fig-2: Electrical connection of the system (1 mm = 0.03937 in)

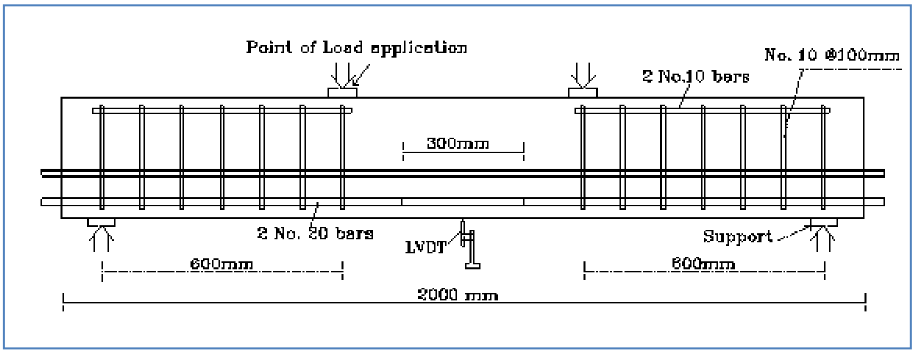


Fig-3: Test setup. (1 mm = 0.03937 in)

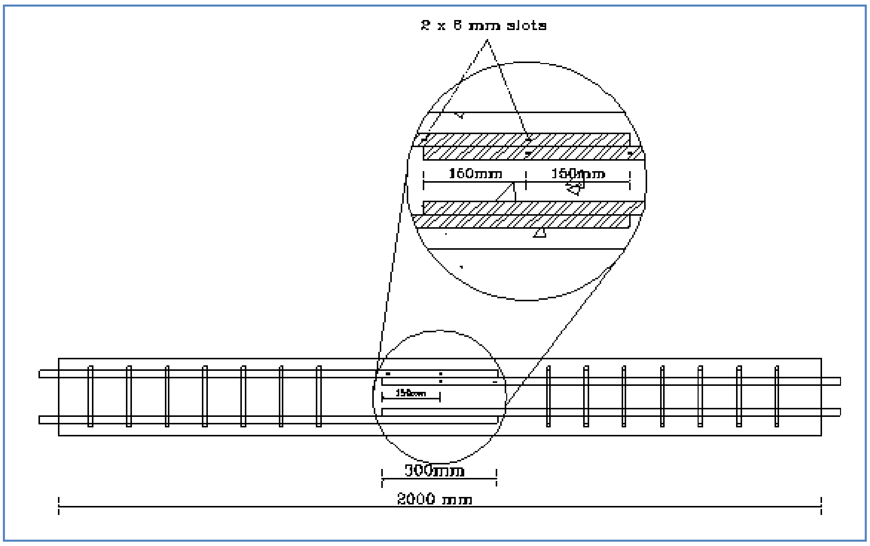
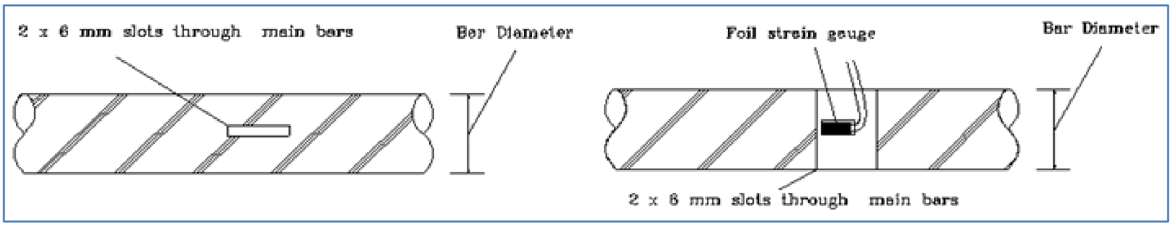
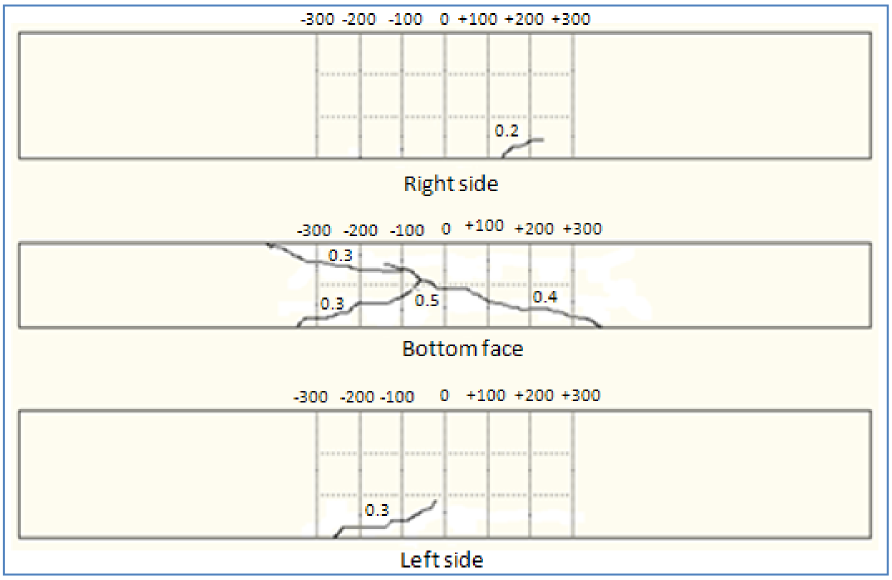


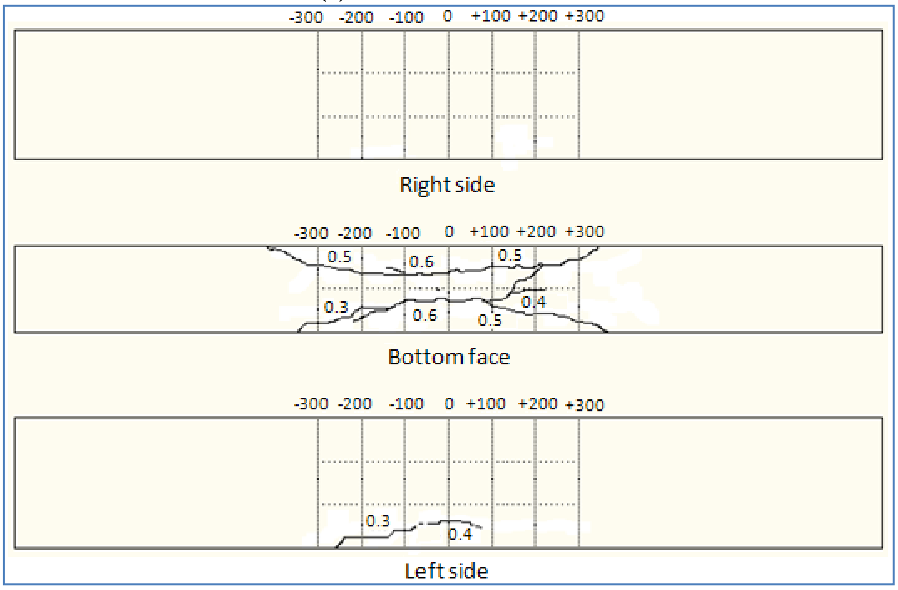
Fig-4a): Location of the strain gauges on the lap splice



b) Close up of strain gauge installation inside the slot of the reinforcing bar.
(1 mm = 0.03937 in)

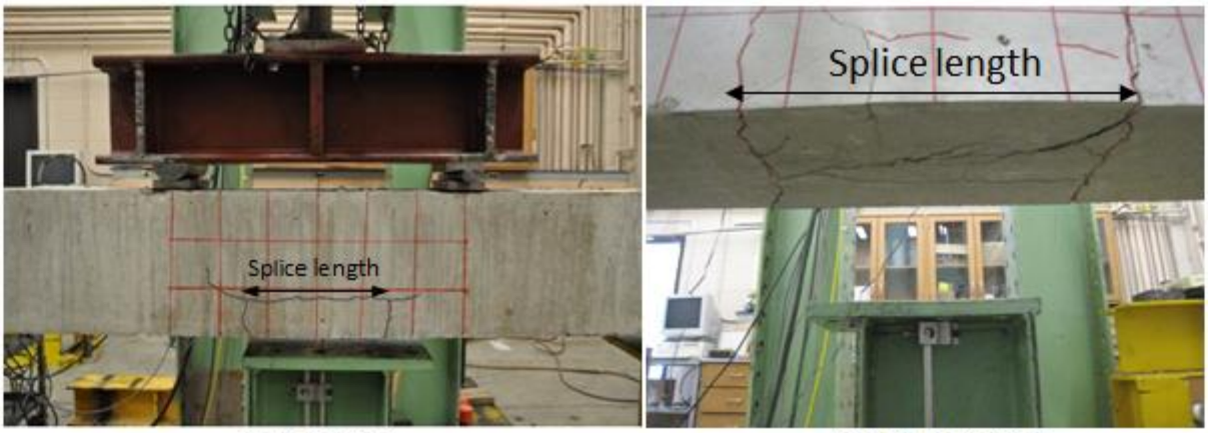


(a) Low corrosion level



(b) High corrosion level

Fig-5: Corrosion cracking pattern for beam c/d =1.5 (1 mm = 0.03937 in)



(a) side view **(b) Bottom view**

Fig-6: Typical failure mode of a lap-spliced beam

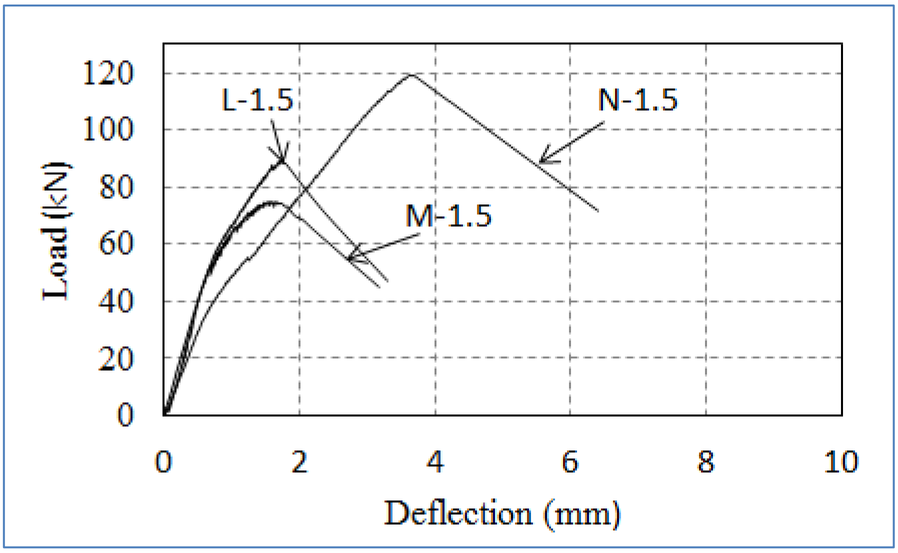


Fig-7: Load deflection curves for beams with c/d ratio=1.5 (1 mm = 0.03937 in and 1 kN = 0.224809 kips)

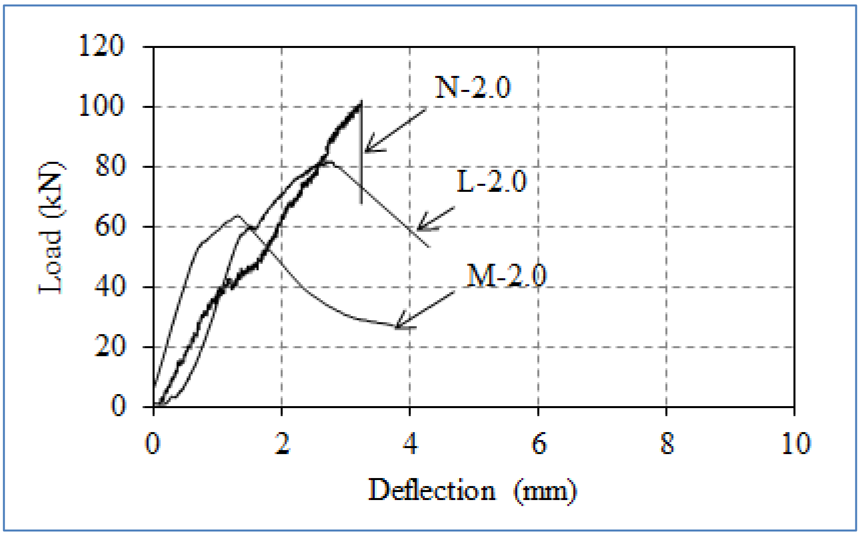


Fig-8: Load deflection curves for beams with c/d ratio=2.0 (1 mm = 0.03937 in and 1 kN = 0.224809 kips)

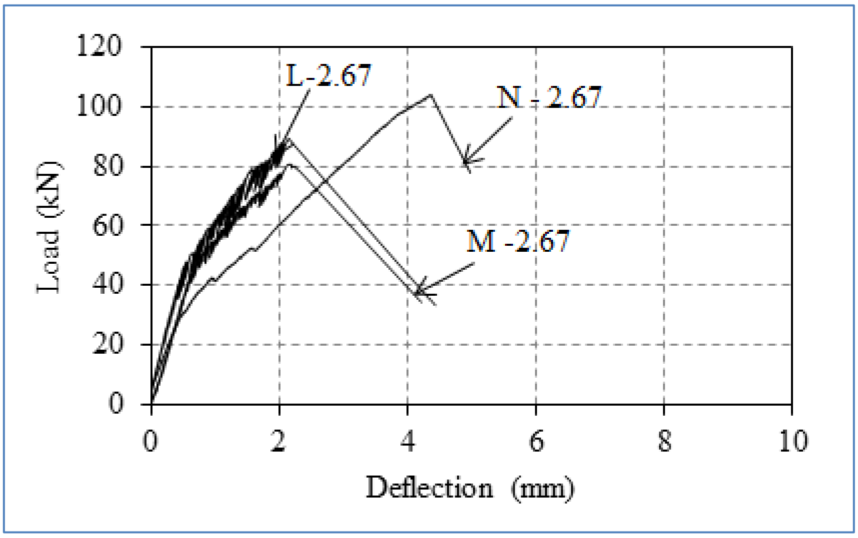


Fig-9: Load deflection curves for beams with c/d ratio= 2.67 (1 mm = 0.03937 in and 1 kN = 0.224809 kips)

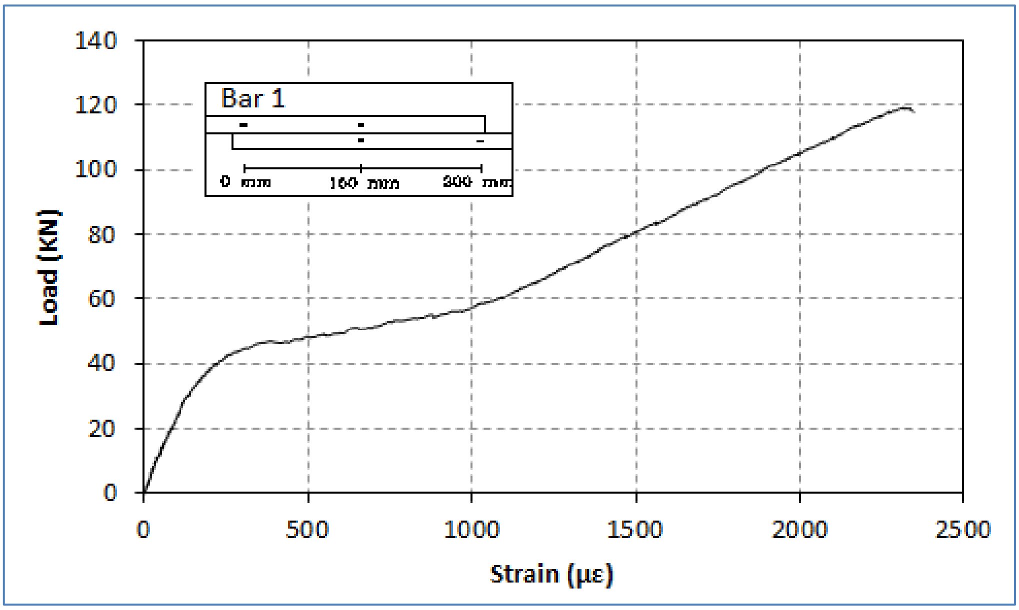


Fig-10: Strain response in bar (1) of gauge at (0 mm) distance along the lap splice (1 kN = 0.224809 kips)

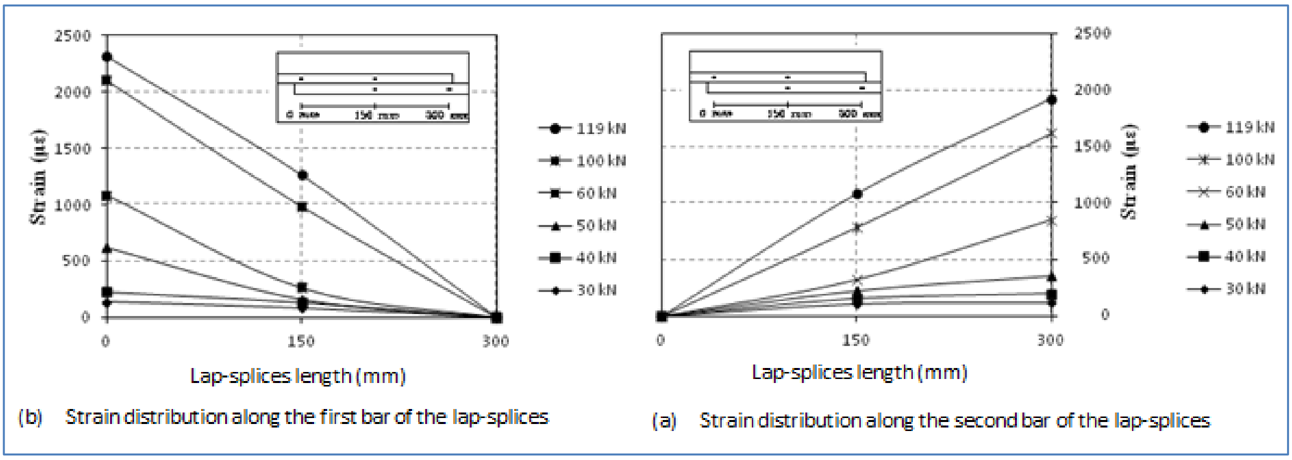


Fig-11: Strain distributions along the lap splice in beam N-1.5 (1 mm = 0.03937 in)

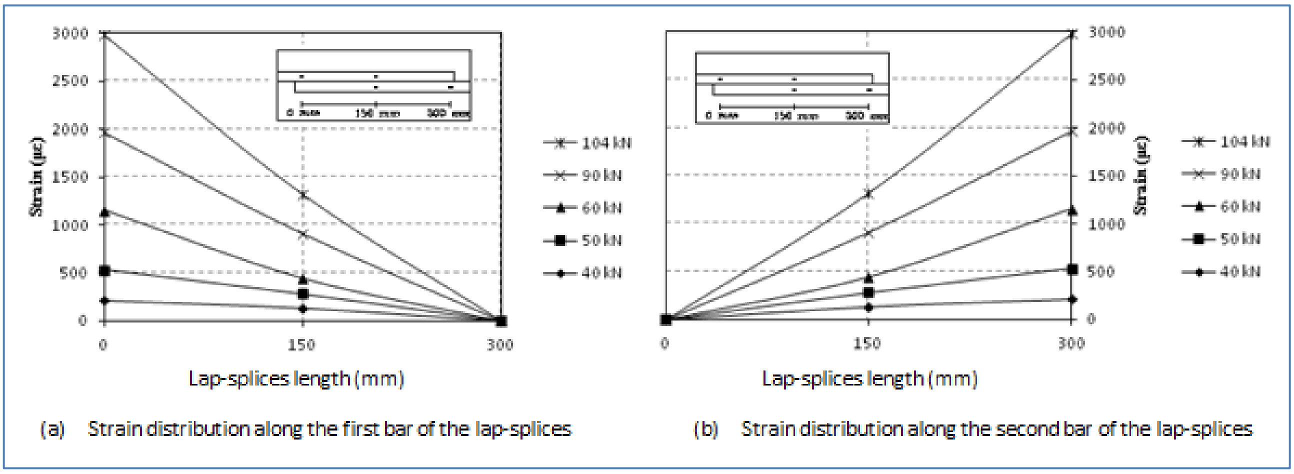


Fig-12: Strain distributions along the lap splice in beam N-2.67 (1 mm = 0.03937 in)

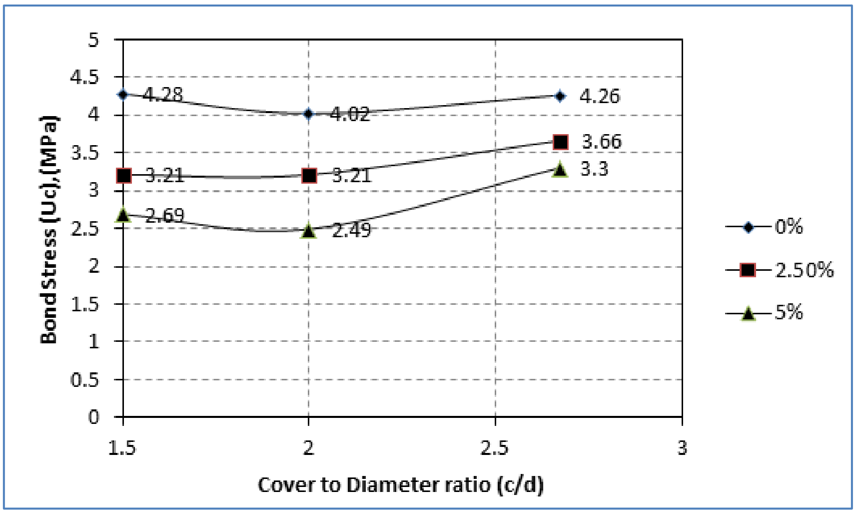


Fig-13: Effect of corrosion on the bond stress (1 MPa = 0.145038 ksi)

Table-1: Details of the test specimen configuration

Beam notations	Corrosion Level	c/d	Main reinforcement	Clear Concrete Cover	Transverse reinforcement
N-1.5	0.0%	1.5	2-20M	30 mm (1.18 in)	10M@100
L-1.5	2.5%				
M-1.5	5.0%				
N-2.0	0.0%	2.0	2-15M	30 mm (1.18 in)	10M@100
L-2.0	2.5%				
M-2.0	5.0%				
N-2.67	0.0%	2.67	2-15M	40 mm (1.57 in)	10M@100
L-2.67	2.5%				
M-2.67	5.0%				

Table-2: Measured corrosion mass loss

Beam designation	Theoretical mass loss (%)	Experimental mass loss (%)	Corrosion attack Penetration depth	
L-1.5	2.5	1.92 ± 0.27	0.39 (mm)	0.015 (in)
M-1.5	5.0	3.31 ± 0.51	0.38(mm)	0.014 (in)
L-2.0	2.5	2.12 ± 0.49	0.62(mm)	0.024(in)
M-2.0	5.0	3.21 ± 0.50	0.76(mm)	0.030(in)
L-2.67	2.5	2.07 ± 0.35	0.59(mm)	0.023(in)
M-2.67	5.0	3.26 ± 0.49	0.72(mm)	0.028(in)

Table-3: Summary of test results

Beam	c/d	Corrosion Level (%)	Cracking				Ultimate			
			Load (kN)	Load (kips)	Deflection (mm)	Deflection (in)	Load (kN)	Load (kips)	Deflection (mm)	Deflection (in)
N-1.5	1.5	0	42	9.44	1.1	0.043	120	27	6.2	0.24
L-1.5		2.5	38	8.54	0.8	0.031	89	20	3.4	0.13
M-1.5		5.0	37	8.80	0.8	0.031	74	16.6	3.2	0.12
N-2.0	2.0	0	56	12.6	1.8	0.071	103	23	3.2	0.12
L-2.0		2.5	50	11.24	1.2	0.047	82	18	4.4	0.17
M-2.0		5.0	45	10.11	0.5	0.019	63	14	3.8	0.14
N-2.67	2.67	0	42	9.44	1.0	0.042	104	23	5.0	0.19
L-2.67		2.5	41	9.21	0.4	0.016	92	20	4.4	0.17
M-2.67		5.0	38	8.54	0.6	0.024	81	18	4.2	0.16

Table-4: Bond strength

Beam Notations	corrosion (%)	Main reinforcement	Measured Ultimate load P_{max}		Steel stress f_s		Bond strength	
			(kN)	kips	(MPa)	(psi)	(MPa)	(Psi)
N-1.5	0	2-20M	120	27	257	37275	4.28	620
L-1.5	2.19		89	20	193	28000	3.21	465
M-1.5	3.82		74	16.6	162	23450	2.49	361
N-2.0	0	2-15M	103	23	322	46700	4.02	583
L-2.0	2.61		82	18	257	37275	3.22	465
M-2.0	3.71		63	14	200	29000	2.69	390
N-2.67	0	2-15M	104	23	341	49450	4.26	617
L-2.67	2.42		92	20	293	42495	3.66	530
M-2.67	3.75		81	18	264	38285	3.3	478

Table-5: Measured and predicted maximum bond stress

Notations	Maximum Measured Mass loss(%)	U_t (model)		U_t (experimental)		Ratio (model/experimental)
		(psi)	(MPa)	(psi)	(MPa)	
N-1.5	0	667	4.6	620	4.28	1.07
L-1.5	2.19	500	3.59	465	3.21	1.11
M-1.5	3.82	333	2.84	361	2.49	1.14
N-2.0	0	713	4.91	583	4.02	1.22
L-2.0	2.61	534	3.63	465	3.21	1.13
M-2.0	3.71	355	3.09	390	2.69	1.15
N-2.67	0	824	5.68	617	4.26	1.33
L-2.67	2.42	618	4.30	530	3.66	1.17
M-2.67	3.75	410	3.55	478	3.30	1.07

CONCLUSIONS

This research study assessed the effect of corrosion on tension lap-spliced bars in reinforced concrete beams. The findings are only valid for the conditions and variables considered in this study and should not be applied to a general case without further investigation. Based on the test results the following conclusions are made:

- Beams with corroded lap-splices, failed just after longitudinal splitting cracks formed in the bottom and side covers to the bars. The final mode of failure was a face- and side splitting of the concrete cover. The failure was sudden, brittle, and noisy.
- The measured strain profile was linear along the lap end splice. The maximum strain was at the loaded end of the lap-spliced bar and was lower than the yield strain of the steel bars.
- Corrosion level had a significant effect on corrosion cracking.
- Increasing corrosion level results in higher crack width.
- Increasing the bar diameter for a given concrete cover results in higher crack width.
- Increasing the concrete cover for the same bar diameter decreased the crack width.
- Corrosion of the lap-spliced bar led to a significant reduction in ultimate bond strength.
- For a 2.5% corrosion level, the reduction ranged between 16% and 25% depending on the c/d ratio. The highest reduction was for c/d = 1.5.
- For a 5.0% corrosion level, the reduction ranged between 20% and 45% depending on the c/d ratio. The highest reduction was for c/d = 1.5.
- A model was proposed to predict the bond stresses of the corroded tension lap-splices in concrete. The predicted bond stresses correlated well with the experimentally measured bond stresses.

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