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Strut and tie modeling for RC short beams with corroded stirrups

Abstract

Corrosion of steel reinforcement is one of the major problems that shorten the service life of reinforced concrete (RC) structures. Steel stirrups, due to their location as an outer reinforcement, are more susceptible to corrosion problems and damage. However, there is limited research work in the literature on the effects of stirrup corrosion on the shear strength of RC beams. This paper attempts to evaluate analytically the residual shear strength of RC short beams with corrosion-damaged stirrups. The shear strength of short or deep beams are generally determined using the strut and tie model. The corrosion effects were implemented in the model to make it capable of predicting the residual shear capacity of RC beams with corroded stirrups. The effect of corrosion is implemented considering the reduction in geometry of the concrete cross section due to spalling and reduction in effective compressive strength of concrete due to corrosion cracks. The proposed strut and tie model which accounts for the corrosion effects was verified using the experimental data available in the literature, and good agreement was found.

Keywords

Corrosion; Concrete beams; Shear strength; Stirrups; Strut and tie model.

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1 INTRODUCTION

Corrosion of steel reinforcement is one of the major causes inducing deterioration of reinforced concrete (RC) structures. Chloride contamination is the most significant contributor to corrosion of reinforcing steel. Generally, concrete protects reinforcing steel from corrosion due to its high alkalinity. The concrete's hydration reaction produces hydroxyl ions which contribute to the alkalinity of the concrete. The hydroxyl ions create a passive layer on the steel reinforcement. This passive oxide layer prevents the corrosion process from occurring (ACI 222R-96, 1996). The corrosion process can commence when the environmental conditions disrupt the formation of the passive layer. Taking into consideration that concrete is a brittle, permeable material that is susceptible to cracking, ingress of chlorides or carbon dioxides, either through cracks or by diffusion through the concrete cover, can destroy the passive layer that normally protects reinforcing or prestressing steel from corrosion. Once the passive layer is disrupted, the corrosion of the reinforcing steel can be initiated. At the time of corrosion cracking, however, the reduction in the steel cross-sectional area is insignificant (Andrade *et al.* 1993). Nevertheless, if repairs are not taken at this early stage, the corrosion will continue, leading to concrete delamination and spalling rendering exposed steel reinforcement. This would accelerate the corrosion rate; further reduce the steel cross-sectional area to a level that might cause sudden rupture of the reinforcing steel.

A number of experimental studies have investigated the effect of corrosion on the mechanical characteristics of reinforcing steel bars (Lee *et al.*, 1996; Almusallam, 2001). The effects of corrosion on the bond strength between reinforcing steel and the surrounding concrete have been also examined (Lee *et al.,* 2002; Auyeung *et al.*, 2000; Amleh and Mirza, 1999; Stanish *et al.*, 1999; Al-Sulaimani *et al.*, 1990). Another group of experimental studies have focused on the corrosion effects on the flexural strength of RC beams (Oyado *et al.*, 2011; Chung *et al.*, 2008; Azad *et a*l., 2007; Du *et al.*, 2007; Ballim and Rie, 2003; Yoon *et al.*, 2000; Mangat and Elgarf, 1999; Almusallam *et al.*, 1996; Cabrera, 1996). Analytical studies have been also conducted to model the corrosion effects on the flexural and bond behavior of RC beams with longitudinal corroded steel bars (Ting and Nowak, 1991; Coronelli, 2002; Azad *et al.*, 2007). On the other hand, there has been limited research on the corrosion effects on the shear behavior of RC beams (Higgins and Farrow, 2006; Suffern *et al.*, 2010; Xia *et al.*, 2011). Further and to the best knowledge of the author, no corrosion models are yet available to predict the residual shear strength of RC beams with corroded stirrups.

It is well established that the shear strength of reinforced concrete beams is influenced significantly by the shear span-to-depth ratio, a/d. Based on this ratio, the reinforced concrete beams are divided into two categories with different shear behavior and strength (Zsutty 1971, 1968). The first category is "slender beams" having a shear span-to-depth ratio, a/d, greater than 2.5, while the second category is "short or deep beams" having a shear span-to-depth ratio, a/d, less than 2.5. In slender concrete beams the behavior of the beams is governed by the beam action and the contribution of the arch action to the strength and behavior of such beams is insignificant. On the other hand, the arch action significantly contributes to the shear strength and behavior of deep concrete beams.

Reinforced concrete deep beams have a wide range of applications in structural engineering, such as pile caps, foundations, bridge girders, offshore structures, and transfer girders in tall buildings (Teng *et al.* 2000). According to span-to-depth ratio, the strength of deep beams is usually controlled by shear rather than flexure if normal amounts of longitudinal reinforcement are used (Oh and Shine 2001). This investigation focuses on developing a model capable of predicting the residual shear strength of RC short beams suffering the corrosion of shear reinforcement.

2 RESEARCH SIGNIFICANCE

Shear failures in RC members are sudden and catastrophic in nature and should be avoided. Corrosion of steel stirrups may affect the shear capacity of concrete members and results in undesirable premature shear failures. Given the location of stirrups as an outer reinforcement, they are more susceptible to corrosion and may be subjected to related deterioration, which reduces the service life of the structure. In addition, the diameter of steel stirrups is generally small compared to that of longitudinal reinforcing bars so that relative loss of the cross-sectional area due to corrosion in the stirrups is expected to be much more significant than that in the longitudinal bars. The extent of the stirrups damage due to corrosion and its consequences on the behavior of the structures must be established as part of the preliminary analysis to determine the reliability of the existing structure prior to repair. This paper presents analytical procedure accounting for the effects of stirrup corrosion on the shear resistance of RC short beams. This will be of interest of practicing engineers, designers, and owners of RC structures as the first step of planning and designing efficient repair systems.

3 REVIEW OF PREVIOUS STUDIES

There are a limited number of studies that have investigated corrosion of shear reinforcement in concrete beams. Higgins and Farrow (2006) conducted a study designed to investigate the shear capacity of beams where the stirrups were damaged due to the effects of corrosion. In their study, a total of 14 beams were constructed: 8 of these beams had a rectangular cross section, 3 beams had a T section, and 3 beams had an inverted T configuration. The beams were 3050 mm in length with a 2440 mm clear span. The rectangular section dimensions were 254 mm by 610 mm. The T section was 610 mm deep with a flange width of 610 mm and a web width of 254 mm. The beams were tested in four-point bending with a shear span to effective depth ratio of 2.04. The main variables studied in this case were the stirrup spacing (203 mm, 254 mm, and 305 mm) and the degree of corrosion (3 levels). The specimens were subjected to accelerated corrosion by impressed current, and then they were visually inspected and assigned grades based on the severity of corrosion damage.

The results were categorized based on the expected mass loss level (none, light, moderate, or severe). The results indicated that at all corrosion levels there was a reduction in the shear capacity of the beam as well as a loss in ductility. For rectangular beams, the corrosion mass loss results varied considerably between different stirrups; the maximum mass loss for the beam with stirrups spaced at 254 mm were 12.7%, 28.9%, and 43.9% for light, moderate, and severe corrosion levels, respectively. The corresponding strength losses were 12%, 19%, and 30% relative to the control (uncorroded) beam. Shear compression failures for the control and lowest corrosion level beams were observed. In the higher corrosion level beams, failure by stirrup fracture was observed. The stirrup fracture was due to significant localized corrosion and the associated section loss. The maximum strength reductions for the T and inverted T sections were 26% and 42%, respectively. The maximum strength loss occurred when the locations of pitting corrosion match the location of the diagonal crack. It was concluded that structural performance in shear can be decreased significantly when sequential stirrups have a reduction in cross sectional area.

Val (2007) conducted an analytical study on the reliability of beams where the shear reinforcement was subjected to corrosion. The analysis was carried out for beams subjected to corrosion at different levels of intensity. The results showed that corrosion (both general and pitting) with low and moderate rates had insignificant influence on the reliability of the considered beams.

For higher rates the effect of corrosion became significant. At higher rates pitting corrosion is more dangerous than general corrosion. Moreover, at these rates the reduction of shear resistance due to corrosion of stirrups, especially pitting corrosion, has a major effect on the beam reliability. The results demonstrate that assessment of the performance of reinforced concrete beams in corrosive environment should include consideration of the effect of corrosion of stirrups on the shear resistance of a beam, otherwise reliability of the beam may be significantly overestimated.

Suffern *et al.* (2010) examined the structural performance of disturbed regions in reinforced concrete beams with corrosion damage to the embedded steel stirrups. A total of 15 reinforced concrete beams were included in this study. The test beams were 350 mm deep, 125 mm wide, and 1850 mm long. The beams were tested in three-point bending under a simply-supported span of 1500 mm. Nine beams among the test beams had the embedded stirrups subjected to accelerated corrosion. Three different levels of corrosion damage of the stirrups were included in this study. The shear span-to-depth (a/d) ratio ranged between 1 and 2. These a/d ratios were considered to reflect the deep beam action as a typical example of disturbed regions. The test results indicated that the corroded beams exhibited reduced shear strength in comparison to the uncorroded control specimens. The shear strength reduction was up to 53%. Furthermore, the reduction in shear strength due to stirrups corrosion was found to be increased with the decrease of the shear span-to-depth ratio of the tested beams.

Xia *et al.* (2011) have investigated the shear performance of RC beams with corroded stirrups. A total of 18 RC beams were constructed and tested, which included 15 corroded beams and three uncorroded beams. All the beams had the same dimensions, which were 230 mm in depth, 120 mm in width and 1200 mm in length. The outcomes of the study showed that the crack width of the concrete cover induced by the reinforcing steel corrosion can be used as an indicator of the corrosion level of the corroded reinforcing steel bars. It was also concluded that the shear capacity of the beams decreased with the increase of the corrosion level of the corroded reinforcing steel stirrups. In addition, the shear failure mode of the beams changed from concrete crushing to stirrup rupture as the corrosion level became severer.

4 STRUT AND TIE MODELING

The strut and tie model (STM) was presented by Schlaich *et al.* (1987) as alternative design methodology. Schlaich *et al.* (1987) differentiated between two types of regions in any reinforced or prestressed concrete structure: B regions, where B stands for beam or Bernoulli; and D regions, where D stands for discontinuity or disturbed. In D regions the distribution of strain is nonlinear, whereas the distribution is linear in B regions. A structural concrete member can consist entirely of a D region; however, more often D and B regions will exist within the same member or structure. Based on Saint Venant's principle, D regions extend a distance equal to the member depth away from any discontinuity, such as change in cross section (geometric discontinuity) or the presence of concentrated loads (static discontinuity). Short or deep beams are a typical example of D regions.

Generally, the ultimate shear strength, V_u, of reinforced concrete deep beams can be determined by two different approaches: sectional approach and member approach. The sectional approach is based on empirical equations derived by fitting the equations to the experimental results, whereas the member approach is based on the STM. Most of the design codes have replaced the sectional design procedures of deep members by STM. The ACI 318 building code (ACI 318 2002), for example, has replaced the sectional method, described in earlier codes, by STM for designing reinforced concrete deep beams. The Canadian Standard CSA-A23.3-04 (2004) as well as the Eurocode (2004) recommends the STM for designing deep members. In this model, the internal flow of forces can be modeled using concrete compressive struts to represent the concrete in uniaxial compression, tension ties to model the principal reinforcement, and nodal zones which represent the regions of concrete subjected to multidirectional stresses where the struts and the ties meet. The design codes do not provide specific guidance on suitable STMs for different cases and designers usually have more than one choice of STMs for the same problem in order to carry the imposed loads through the D regions to the supports.

4.1 Direct strut and tie model

According to the ACI 318 code (ACI 318, 2011), short or deep beams are defined as members loaded on one face and supported on the opposite face, so that compression struts can develop between loads and supports. Moreover, deep beams have either: a) $l_n/h \leq 4.0$; or b) $a/h \leq 2.0$, where l_n is the clear span of the deep beams, h is the overall depth, and a is the shear span length.

In the direct STM, the shear is carried from the load points to the supports directly by major compression struts. A typical strut and tie model for a deep beam subjected to two point loads is shown in Figure 1. It can be noticed that the model is a statically determinate truss including three components: the concrete struts, the steel tie from the main reinforcing bars, and the nodal zones. It can be noticed also that the existence of the stirrups is not explicitly accounted for in this direct STM. However, the effect of stirrups is considered in determining the capacity of inclined struts as it will be presented in the following. Figure 2 shows the geometrical notations of the STM.

4.1.1 Top horizontal strut

The depth of the top horizontal strut, a_1, is determined as the depth of the compression block from flexural analysis:

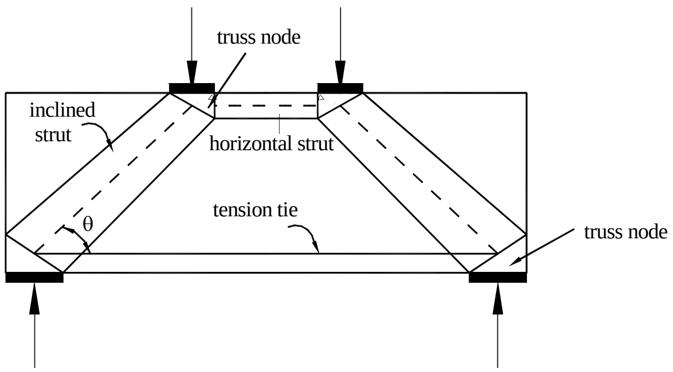

Figure 1: Typical direct strut and tie model for a deep beam.

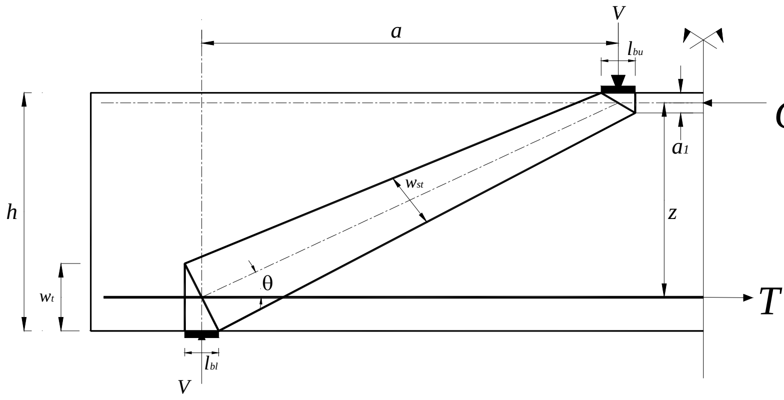

Figure 2: Notations of STM.

$$a_1 = \frac{A_s f_y}{0.85 f_c' b} \tag{1}$$

where A_s and f_y are the area and yield strength of main steel reinforcement; f_c' is the specified concrete strength; and b is the beam width.

4.1.2 Inclined strut

The dimensions of the inclined strut are determined based on the dimensions of the nodal zones. The inclined strut is generally non-prismatic in shape. The lower and upper widths of the inclined strut are calculated according to the following equations with reference to Figure 3.

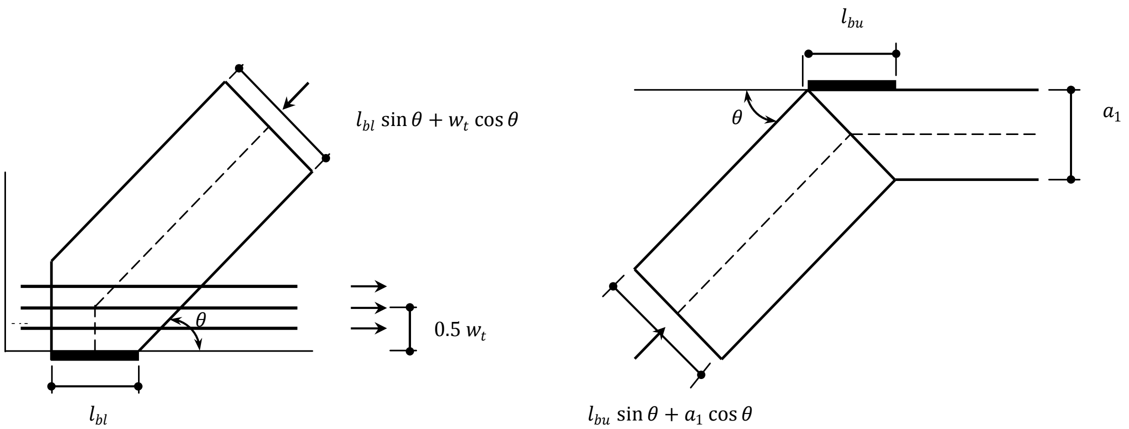

Figure 3: Strut dimensions.

$$\text{lower width of strut} \qquad w_{st} = l_{bl} \sin\theta + w_t \cos\theta \tag{2}$$

$$\text{upper width of strut} \qquad w_{st} = l_{bu} \sin\theta + a_1 \cos\theta \tag{3}$$

where l_{bl} and l_{bu} are the plate widths at support and loading point, respectively; w_t is the tie width; and θ is the inclination angle of the strut to the longitudinal axis of the beam and can be determined such that:

$$\theta = arctan\ (z\ /a) \tag{4}$$

in which, $z = h\text{-}a_1/2\text{-}w_t/2$.

The design codes provide recommendations for evaluating the strength of concrete struts. In Appendix A of ACI 318-11 code (2011), the strength of concrete struts F_{ns} is calculated as follows:

$$F_{ns} = f_{ce} A_{cs} \tag{5a}$$
$$f_{ce} = 0.85 \beta_s f_c' \tag{5b}$$

where A_{cs} is the cross-sectional area at one end of the strut, f_{ce} is effective compressive strength of the concrete in the strut; and β_s is the efficiency factor. The values of β_s as recommended by the ACI 318 code (2011) are given in Table 1.

Strut	Efficiency factor β_s
- Uncracked strut with uniform cross-sectional area (prismatic)	1.0
- Inclined strut (non-prismatic) with web reinforcement satisfying Clause A.3.3 in Appendix A of the ACI 318 code	0.75
- Inclined strut (non-prismatic) without web reinforcement	0.6

Table 1: Recommended values for the efficiency factor β_s for the concrete struts (normal-weight concrete).

It can be noted from Table 1 that the effect of stirrups is accounted for by increasing the efficiency factor β_s from 0.6 for beams without web reinforcement to 0.75 for beams with web reinforcement satisfying the minimum required by the code. This indicates that the main role of the web reinforcement in deep members is to control diagonal cracking to avoid premature failure before reaching the crushing capacity of the struts. For the typical strut and tie model in Figure 1, a value of $\beta_s = 1$ can be taken for the top horizontal strut while a value of $\beta_s = 0.75$ or 0.6 can be taken for the inclined strut depending on the availability of web reinforcement.

4.1.3 Reinforcing tie

The effective tie width w_t is generally taken as twice the distance from the centroid of main steel reinforcement to extreme tension fiber of the beam. Appendix A of ACI 318-11 code (2011) recommends a practical upper limit of the tie width corresponding to the width in a hydrostatic nodal zone, calculated as

$$w_{t,max} = F_{nt}\ /(f_{ce} b) \tag{6}$$

where F_{nt} is the strength of the tie and f_{ce} is the effective compressive strength of the concrete in a hydrostatic nodal zone.
The strength of the tie, F_{nt}, is calculated as:

$$F_{nt} = A_s f_y \tag{7}$$

where A_s and f_y are the area and yield strength of main steel reinforcement.

4.1.4 Nodal zones

The compression strength of a nodal zone, F_{nn}, is determined such that:

$$F_{nn} = f_{ce} A_{nz} \tag{8a}$$

$$f_{ce} = 0.85 \beta_n f_c' \tag{8b}$$

where A_{nz} is the smaller of :
a) The area of the face of the nodal zone on which the force acts, taken perpendicular to the line of action of the force,
b) The area of a section through the nodal zone, taken perpendicular to the line of the resultant force on the section,
f_{ce} is the effective compressive strength of the concrete in the nodal zone and β_n is the efficiency factor. The values of β_n as recommended by the ACI 318 code (2011) are given in Table 2.

The calculation procedure starts by determining the depth of the compression block, a_1, then the inclination angle θ of the direct strut can be calculated. The area of the direct strut is determined based on the smaller of top and bottom nodes and used to determine the strut capacity. The strut capacity is used to calculate the shear load. The stress in the top and bottom nodes is checked against appropriate stress limits. In addition, the tension tie is checked against the force resulted from the calculated shear load.

Nodal zone*	Efficiency factor β_n
- CCC	1.0
- CCT	0.8
- CTT	0.6

* CCC = nodal zones bounded by struts or bearing areas or both;
CCT = nodal zones anchoring one tie; CTT = nodal zones anchoring two or more ties.

Table 2: Recommended values for the efficiency factor β_n for the nodal zones.

5 DIRECT STRUT AND TIE MODELING FOR RC BEAMS WITH CORRODED STIRRUPS

The damage that directly results from corrosion includes the sectional loss of the reinforcing steel bars of stirrups and the corrosion cracks. The average cross section loss ratio represents the deterioration of the reinforcing steel bars in stirrups and the crack width reflects the damage of concrete cover as the consequence of the stirrup corrosion. Xia et al. 2011 indicated that both sectional loss and corrosion crack width were proportional to the reduction in shear capacity due to stirrup corrosion and consequently may be linked to the residual shear capacity of corroded beams. Taking into account that the average sectional loss of reinforcing steel bars in real structures is difficult to be evaluated and there is no nondestructive method available at present for determining this parameter, the corrosion crack width measurements can be used instead, to evaluate the residual shear capacity of corroded beams. In fact and from a practical perspective, this is what would be available to practicing engineers during inspecting real structures to assess the corrosion damage. In this modeling, the corrosion damage is incorporated into the direct strut model in two ways. First, the effective concrete strength of the compression strut is modified based on a reduction model. Second, a reduction in the cross section width accounting for the spalling and delamination of the concrete cover is included.

5.1 Effective concrete strength

It is well established that cracked concrete under compression exhibits lower strength than uncracked concrete (Vecchio and Collins 1986). This compression softening effect depends on the degree of transverse cracking and straining. It can be noted that this effect is considered in the efficiency factors β_s and β_n in Tables 1 and 2, respectively. Moreover, Tjhin and Kuchma (2002) found that the capacity of struts is significantly affected by any disturbance in the strut such as initial cracks parallel or inclined to the strut axis and tensile transverse stress or strain induced by a crossing tie or another effect. Previous experimental results indicated that cracks developed due to corrosion of steel stirrups significantly affected the capacity of the diagonal struts (Higgins and Farrow, 2006; Suffern et al., 2010; Xia et al., 2011). The effective concrete strength reduced as a consequence of corrosion cracks can be calculated by using the following equation (Coronelli and Gambarova 2004):

$$f_{ce} = \frac{f_c'}{1 + K\frac{\varepsilon_1}{\varepsilon_{co}}} \tag{9}$$

where K is a coefficient related to bar roughness and diameter [K =0.1 for medium diameter ribbed bars as proposed by Capé (1999)], f_c' compressive strength of uncracked concrete, ε_{co} is the strain at the peak compressive stress f_c', and ε_1 is the average tensile strain in the cracked concrete perpendicular to the applied compression which can be evaluated by the following equation:

$$\varepsilon_1 = \frac{n_{bars} w_{cr}}{b} \tag{10}$$

where n_{bars} is the number of reinforcing bars in the compression zone, b is the undamaged beam section width, and w_{cr} is the crack width for a given corrosion level. Thus, by knowing the corrosion crack width w_{cr}, the average tensile stain ε_1 can be calculated according to Eq. (10) which can be substituted in Eq. (9) to calculate the effective concrete strength f_{ce}.

By equating Eqs. (5b) and (9), the efficiency factor $\beta_{s\text{-}corr}$ for the inclined strut in beams with corroded stirrups is given as follows:

$$\beta_{s-corr} = \frac{1.17}{1 + K\frac{\varepsilon_1}{\varepsilon_{co}}} \le 0.75 \tag{11}$$

By determining $\beta_{s\text{-}corr}$, the effective concrete strength f_{ce} can be calculated for beams with corroded stirrups and then calculating the capacity of the inclined strut according to Eq. (5a).

Similarly and by equating Eqs. (8b) and (9), the efficiency factor $\beta_{n\text{-}corr}$ for the nodal zones for beams with corroded stirrups is given as follows:

$$\beta_{n-corr} = \frac{1.17}{1 + K\frac{\varepsilon_1}{\varepsilon_{co}}} \le k_1 \tag{12}$$

where k_1 = 1, 0.8, or 0.6 for CCC, CCT, or CTT nodes, respectively. By determining $\beta_{n\text{-}corr}$, the effective concrete strength f_{ce} can be calculated for the nodal zones in beams with corroded stirrups and then calculating the capacity of the nodes according to Eq. (8a).

5.2 Effective beam width

The concrete geometry is influenced by corrosion of steel stirrups. The corrosion of the shear reinforcement causes cracking, delamination, and spallling. This affects the concrete cover and makes the beam cross section less effective in resisting imposed loads. Consequently, an effective concrete width is proposed to account for these effects. Higgins *et al.* (2003) used an effective section width model based on the concrete cover thickness, stirrup diameter, and stirrup spacing. They suggested that when the stirrups were spaced closer together, more interaction between corrosion cracks occurred and this interaction can cause an increase in the severity of the spalling.

In this study, a simple and conservative way to consider the effects of corrosion on the concrete section at the ultimate stage would be to reduce the section width by completely ignoring the concrete cover. Equation 13 provides the proposed effective width formulation.

$$b_{eff} = b - 2c \tag{13}$$

where b_{eff} is the effective width and c is the side concrete cover. The effective width is used in calculating the effective strut area $A_{cs\text{-}eff}$ and effective nodal area $A_{nz\text{-}eff}$ which in turn used in evaluating the strut capacity and the nodal capacity as per Eqs. (5a) and (8a), respectively.

6 EXPERIMENTAL VERIFICATION

To verify the proposed corrosion models implemented in the direct STM described in the previous section, experimental data from the literature have been collected. As presented earlier in the paper, there are only three experimental studies on this topic available in the literature: Higgins and Farrow, 2006; Suffern *et al.*, 2010; and Xia *et al.,* 2011. By examining the data available in these studies, it was found that there are missing data that have not been presented in the work of both Higgins and Farrow, 2006 and Xia *et al.*, 2011. The corrosion crack width, which is required for calculating the effective concrete strength as per Eqs. (9) and (10), was not given by Higgins and Farrow, 2006. While the corrosion crack width data were given by Xia *et al.*, 2011, the data about the length of the shear span of the beams and the dimensions of the loading and support plates, which are required for generating the geometry of the STM, were not given. Therefore, the experimental data only from Suffern *et al.*, 2010 have been considered in this verification as the information required for generating STM and calculating the effective concrete strength was given. The experimental results of six uncorroded beams and nine corroded beams tested by Suffern *et al.*, 2010 are given in Table 3. The uncorroded beams included three beams without stirrups and three beams with stirrups.

The ultimate shear capacity of the tested beams was predicted using the direct strut and tie model described earlier. For the uncorroded control beams, the capacity was calculated based on $\beta_s = 0.6$ for the inclined struts of beams without stirrups and 0.75 for beams with stirrups and considering the undamaged beam width b. For the corroded beams, the capacity was calculated considering the efficiency factor $\beta_{s\text{-}corr}$ for inclined strut according to Eq. (11) along with using the effective beam width as per Eq. (13). The nodal stresses were checked against the stress limits considering the efficiency factor $\beta_{n\text{-}corr}$ according to Eq. (12) and also using the effective beam width as per Eq. (13).

A comparison between the experimental shear strength, $V_{u,\ exp}$, and predicted shear strength, $V_{u,\ pred}$ of the tested beams is given in Table 3 and presented in Figure 4 for both uncorroded and corroded beams. It can be seen that the STM provided conservative predictions for the uncorroded beams except for beam 0-2.0-UR without stirrups. The experimental to predicted shear strength, $V_{u,\ exp} / V_{u,\ pred}$ for this beam was 0.68 indicating that something may have happened during testing this beam causing premature failure. However, the average ratio of $V_{u,\ exp} / V_{u,\ pred}$ for the six uncorroded beams was 1.19 with a coefficient of variation of 22%. By excluding beam 0-2.0-UR, the predictions for the remaining uncorroded five beams become more consistent with a coefficient of variation of 7%, whereas the level of conservatism will be increased resulting an average ratio of $V_{u,\ exp} / V_{u,\ pred}$ of 1.29. It can be also seen that the modified direct STM which incorporates the corrosion effects provided conservative predictions for the shear strength of the corroded beams except for beam M-1.0-R as the ratio of $V_{u,\ exp} / V_{u,\ pred}$ for this beam was 0.9. The average ratio of $V_{u,\ exp} / V_{u,\ pred}$ for all nine corroded beams, however, was 1.31 with a coefficient of variation of 18%. Thus, the level of conservatism of STM predictions appeared to be approximately similar for both corroded and uncorroded beams indicating that the modifications implemented in STM made it able to capture the effects of the corrosion damages on the shear resistance of the beams.

7 CONCLUSIONS

The research described in this paper presented a procedure for evaluating the residual shear strength of RC short beams with corrosion-damaged stirrups. The corrosion effects were implemented in the strut and tie model as the change in the geometry of the cross section of the beams and as the change in the concrete strength. The width of the cross section was reduced to account for the spalling of the concrete while the effective concrete strength was reduced to account for the corrosion cracks. One important feature of this modeling is that the corrosion crack width, which can be measured directly in real practice, serves as an indicator of the corrosion damage and can be used to determine the effective concrete strength reduced due to corrosion. The proposed modifications of strut and tie model were checked against experimental data available in the literature for beams with corroded stirrups. The results of this comparison indicated that the modified strut and tie model reasonably predicted the residual shear strength of the corroded beams. More experimental data are required for further verifying the model.

Beam		**Concrete strength (MPa)**	**Corrosion crack width (mm)**	**Experimental shear strength $V_{u,\ exp}$, (kN)**	**Predicted shear strength $V_{u,\ pred}$, (kN)**	$V_{u,\ exp} / V_{u,\ pred}$
Uncorroded beams	0-1.0-UR	35.7	--	401	284	1.41
	0-1.0-R	41.3	--	473	392	1.21
	0-1.5-UR	41.3	--	352	255	1.38
	0-1.5-R	41.3	--	396	318	1.24
	0-2-UR	41.3	--	150	221	0.68
	0-2.0-R	41.3	--	337	276	1.22
					Mean	**1.19**
					Standard deviation	**0.27**
					Coefficient of variation (%)	**22**
Corroded beams	L-1.0-R	45.4	0.30	356	284	1.25
	M-1.0-R	40.5	0.50	221	246	0.90
	H-1.0-R	43	0.90	283	224	1.26
	L-1.5-R	45.4	0.30	308	233	1.32
	M-1.5-R	40.5	0.30	307	220	1.39
	H-1.5-R	43	1.00	201	180	1.12
	L-2.0-R	45.4	0.30	273	209	1.31
	M-2.0-R	40.5	0.35	330	192	1.72
	H-2.0-R	43	0.6	282	183	1.54
					Mean	**1.31**
					Standard deviation	**0.23**
					Coefficient of variation (%)	**18**

Table 3: Experimental versus predicted shear capacity of the beams tested by Suffern *et al.* 2010.

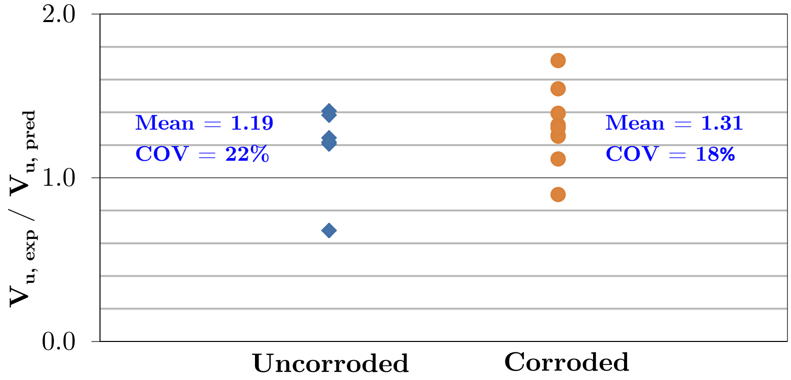

Figure 4: Comparison of the experimental and predicted shear capacity of the beams tested by Suffern *et al.* 2010.

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Nomenclature

The following symbols are used in this paper:

A_{cs}	= cross-sectional area at one end of strut;
$A_{cs\text{-}eff}$	= effective cross-sectional area at one end of strut;
A_{nz}	= area of nodal zone;
$A_{nz\text{-}eff}$	= effective area of nodal zone;
A_s	= cross-sectional area of main steel reinforcement;
a	= shear span;
a_1	= depth of top horizontal strut;
b	= width of beam;
b_{eff}	= effective width of beam;
C	= compression internal force;
c	= side concrete cover;
d	= effective depth of tension steel;
F_{nn}	= strength of nodal zone;
F_{ns}	= strength of concrete strut;
F_{nt}	= strength of tie;
$f_c^{'}$	= specified compressive strength of concrete;
f_{ce}	= effective concrete strength;
f_y	= yield strength of main steel reinforcement;
h	= total depth of beam;
K	= coefficient related to bar roughness and diameter;

l_{bl} = length of support plate;

l_{bu} = length of loading plate;

l_n = clear span of beam;

n_{bars} = number of reinforcing bars in compression zone;

T = tensile force in main steel reinforcement;

V_u = ultimate shear strength;

$V_{u,\ exp}$ = experimental ultimate shear strength;

$V_{u,\ pred}$ = predicted ultimate shear strength;

w_{cr} = corrosion crack width;

w_{st} = width of strut;

w_t = width of tie;

$w_{t,\ max}$ = maximum width of tie;

z = distance between internal tensile and compressive forces in a section;

β_n = efficiency factor for nodal zone;

$\beta_{n\text{-}corr}$ = efficiency factor for nodal zone of corroded beam;

β_s = efficiency factor for strut;

$\beta_{s\text{-}corr}$ = efficiency factor for strut of corroded beam;

ε_{co} = concrete strain at peak compressive stress;

ε_1 = average tensile strain in cracked concrete perpendicular to applied compression;

θ = inclination angle of strut to longitudinal axis of beam.