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Residual flexural strength of corroded reinforced concrete beams

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ABSTRACT

In order to develop a cost-effective retrofit system for deteriorated concrete members, there is a need for sound evaluation of their remaining strength. Unfortunately, there is a lack of reliable models for evaluating the residual strength of steel reinforced concrete beams, and further, there is no simplified analytical model that can reasonably estimate the ultimate capacity of corroded reinforced concrete (CRC) beams. To address these issues, this research presents a new methodology for the evaluation of the residual flexural strength of CRC beams. Finite Element (FE), detailed, and simplified analytical models that account for damaged material properties and geometry, were developed and verified against the experimental data obtained by others. The detailed analytical model was employed to investigate the effects of corrosion degree, length of corroded zone, and span to depth ratio on flexural strength. This analysis showed that for beams with high corrosion degrees, the decrease in capacity is mainly because of the decrease in both the steel reinforcement cross-section and strength due to corrosion, whereas the major cause of strength reduction for beams subjected to low corrosion degrees is the loss of bond between steel and surrounding concrete.

1. Introduction

The Bureau of Transportation Statistics suggests that about 25% of the bridges in the United States of America are structurally deficient or functionally obsolete [1].

The Federal Highway Administration (FHWA) also estimates that over 30% of the existing bridges have already exceeded their 50 year service life, which will necessitate more costly maintenance in the near future [2]. Further, the vast majority of the concrete girder bridges built in the United States between 1930 and 1955 are heavily deteriorated. These bridges are reinforced with hooked and bent-up reinforcing rebars, and will not experience bond mode of failure even under severe deterioration stages despite bond deterioration over most of the length of steel bar. Fig. 1 shows corrosion and loss of bond in a deteriorated concrete bridge girder and bent cap beam. However, a limited number of investigations were carried out to estimate the ultimate flexural strength of deteriorated reinforced concrete structures. The development of a simplified practical model for the estimation of the residual strength of bridge members will help engineers to better assess the structural capacity of existing bridges, resulting in a cost-effective retrofit system, improved bridge conditions, and higher safety level.

The effects of bond failure between tensile steel reinforcement and the surrounding concrete has been experimentally investigated [3–6]. Minkarah and Ringo [3] found that while a 20% exposed reinforcement length did not affect the ultimate flexural strength of the beam samples, a 60% exposed length resulted in a 20% decrease in the ultimate flexural capacity. Cairns and Zhao [4] observed a 50% decrease in the ultimate flexural strength for a beam with a 1.5% tensile reinforcement ratio and an exposed length over 90% of the span, whereas there was no reported loss of strength for a beam with 0.5% tensile reinforcement ratio and an exposed length over 90% of the span. Sharaf and Soudki [6] reported 35% reduction in ultimate flexural strength for a beam sample with unbonded reinforcement over 90% of the span. Other researchers performed experimental studies on CRC beams with partial corrosion [7–10]. While El Maaddawy et al. [8] concluded that corrosion causes a reduction in ultimate strength and this reduction is proportional to the reduction in steel cross-sectional area, Du et al. [9] stated that not only does corrosion decrease the beam's flexural strength, it also changes its mode of failure.

Several researchers employed Finite Element Analysis (FEA) to study the behavior of fully corroded RC beams [11–17], of particular note, Wang and Chen [10] performed a study on partially corroded RC beams and noted that the length of exposed steel reinforcement does not affect the strength of concrete beams but

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Nomenclature

A_s	area of tensile steel reinforcement
$A_{s(corr)}$	area of corroded tensile steel reinforcement
A'_s	area of compressive steel reinforcement
b	width of the cross section
β_1	ratio of depth of rectangular stress block to depth of neutral axis
c	distance from extreme compression fiber to the neutral axis
$c_{cor.}$	distance from extreme compression fiber to the neutral axis of a cross-section with corroded reinforcement
$c_{cor.(i)}$	distance from extreme compression fiber to the neutral axis of a cross-section with corroded reinforcement at step i
c_{ub}	distance from extreme compression fiber to the neutral axis of a cross-section with unbonded reinforcement
C_c	total compressive force of concrete at critical section
C_s	total compressive force of compressive steel at critical section
d	distance from extreme compression fiber to centroid of tensile reinforcement
d'	distance from extreme compression fiber to centroid of compressive reinforcement
d_0	diameter of reinforcing bar
$d_{0(corr)}$	diameter of corroded reinforcing bar
E_c	modulus of elasticity of concrete
E_s	modulus of elasticity of steel
f_c	concrete compressive strength at strain ε
f'_c	specified compressive strength of concrete
f_y	yield strength of tensile steel
$f_{y(corr)}$	yield strength of corroded tensile steel
f_{yc}	yield strength of compressive steel
h	overall concrete depth
i	variable accounts for number of iterations
L	span of beam
L_{corr}	length of corroded reinforcement
L_{ub}	length of beam over which reinforcement is unbonded
L_o	length of equivalent plastic region of unbonded reinforced concrete beam
M_{corr}	calculated ultimate bending strength of RC beams with corroded steel
M_n	calculated ultimate bending strength of RC beams with perfectly bonded steel
M_n^c	calculated ultimate bending strength of RC beams with corroded bonded steel
Q_{corr}	corrosion degree, which is calculated as the mass loss (%)
T	total tensile force of tensile steel at critical section
y	distance from extreme compression fiber to centroid of concrete in compression
B_1	coefficient for determining stress block height based on concrete strength
ε	concrete strain corresponding to concrete compressive strength f_c
ε_{cx}	strain of concrete adjacent to steel rebars in unbonded region at a distance x from the end of the beam
ε_{cu}	ultimate compressive strain of concrete
ε_s	strain of tensile perfectly bonded steel reinforcement
ε_{sc}	strain of compressive steel reinforcement
$\varepsilon_{scor.}$	strain of tensile corroded steel reinforcement
$\varepsilon_{scor.(i)}$	strain of tensile corroded steel reinforcement at step i
ε_{sub}	strain of tensile unbonded steel reinforcement
ε_y	yield strain of tensile steel reinforcement
ε_{yc}	yield strain of compressive steel reinforcement
ε_0	concrete strain corresponding to concrete compressive strength, taken as $\varepsilon_0 = 1.8f'_c/E_c$
ρ	tensile steel reinforcement ratio
ρ_b	tensile steel reinforcement ratio at the balanced condition (ACI)
ρ_{max}	maximum tensile steel reinforcement ratio (ACI)
ρ_{min}	minimum tensile steel reinforcement ratio (ACI)
Ψ	ratio of equivalent length over depth of the neutral axis L_o/c

that it does affect rigidity. In addition, it was determined that the decrease in the rebar cross-sectional area is proportional to the decrease in the ultimate flexural strength of the reinforced concrete member. This was attributed to the fact that the flexural strength is directly proportional to the amount of steel reinforcement, and the steel reinforcement in all of the beams investigated had reached its yield strength. It was found that there was no decrease in the ultimate flexural strength as a result of the loss of localized bond between the rebars and the surrounding concrete.

While corrosion decreases the cross-sectional area and the strength of the steel reinforcement, it also deteriorates the bond between the steel bars and the surrounding concrete. In the absence of bond, the strain in the steel bars is no longer compatible with the strain in the surrounding concrete, and the code equations for analyzing an RC cross-section are no longer applicable.

1.1. Reduction in ultimate flexural strength due to reduction in steel reinforcement cross-sectional area and strength

Several researchers experimentally investigated the effect of corrosion on the mechanical properties and strength of steel reinforcement [18–20,21]. They reported a decrease in the cross-sectional area and strength of steel reinforcement embedded in concrete due to corrosion. Based on both the accelerated and simulated corrosion tests on bare bars and on bars embedded in concrete, Du et al. [21] suggested Eqs. (1)–(3) to estimate the residual cross-sectional area and strength of corroded reinforcing bars. These equations are based on a regression analyses performed on both sets of test data.

$$A_{s(corr)} = (1 - 0.01 \cdot Q_{corr}) \cdot A_s \tag{1}$$

$$f_{y(corr)} = (1 - 0.005 \cdot Q_{corr}) \cdot f_y \tag{2}$$

$$Q_{corr} = \left[1 - \left(\frac{d_{0(corr)}}{d_0}\right)^2\right] \times 100 \tag{3}$$

where $A_{s(corr)}$ is the average cross-sectional area of corroded reinforcement, A_s is the initial cross-sectional area of non-corroded reinforcement, $f_{y(corr)}$ and f_y are the yield strengths of corroded and non-corroded reinforcement, respectively, Q_{corr} is the amount of corrosion of reinforcement (%) (also known as corrosion degree), and $d_{0(corr)}$ and d_0 are the diameters of corroded and non-corroded reinforcing bars respectively. Based on the above equations, and while ignoring the effects of loss of bond between the steel reinforcement and the surrounding concrete, it can be concluded that the reduction in ultimate flexural strength due to corrosion is almost proportional to the reduction in the steel cross-sectional area.