

Corrosion-Induced Bond-Transfer Deterioration in Bridge Deck Concrete: Tension Stiffening, Crack Spacing, and Member-Level Response in Fly Ash and Normal Portland Cement Systems

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Abstract: Chloride-induced reinforcement corrosion degrades steel–concrete interaction, affecting tension stiffening and cracking in reinforced concrete. This study re-analyzes a legacy direct-tension dataset comprising 30 specimens from three concrete systems: normal Portland cement (NPC) concrete with ($w/c = 0.52$), higher-quality NPC concrete with ($w/c = 0.32$), and high-volume fly ash (HVFA) concrete ($w/cm = 0.32$; 58% fly ash). Twenty-four specimens were subjected to accelerated corrosion in 5% NaCl under 5 V DC, while six served as controls. Archived specimen-level data were re-audited for measured mass loss, cracking, crack spacing, loads, chloride content, and failure response. Increasing corrosion reduced tension stiffening and increased the transfer length required for formation of new transverse cracks. The crack-spacing ratio (S_0/S) is interpreted as a crack-spacing-derived relative bond stress (u/u_0), rather than a direct measure of local bond strength. For Series C7, u/u_0 decreased from 0.83 at 8% mass loss to 0.17 at 24% mass loss; for Series C3, $u/u_0 \approx 0.23$ at 14.5% mass loss. At 27% mass loss, specimen C7-9C developed no new transverse crack before steel yielding, precluding determination of u/u_0 . HVFA specimens exhibited no corrosion-induced longitudinal splitting, although localized corrosion and spalling occurred. Comparison with a published bond-reduction relationship showed reasonable agreement for Series C7 but generally lower relative bond stress for the lower- w/cm C3 and C4 series. The results demonstrate that corrosion level, localized deterioration, concrete system, and cracking response should be considered collectively and highlight limitations in applying a single mass-loss-based bond-reduction relationship across different concrete systems and corrosion ranges.

Author keywords: Bond deterioration; corrosion; tension stiffening; crack spacing; fly ash; high-volume fly ash (HVFA); supplementary cementitious materials (SCMs)

Introduction

The bond between reinforcing steel and concrete governs not only local anchorage capacity but also the distribution of deformations and cracks across a member under load. In the presence of corrosion, this bond deteriorates progressively, and the structural consequences extend well beyond the conventional concern with rebar cross-section loss. As bond weakens, the tension-stiffening contribution of concrete between cracks diminishes, crack spacings widen, crack widths increase at a given load level, and ultimately the load-carrying capacity of the member is compromised.^{1,2}

The majority of experimental studies on corrosion-induced bond deterioration have employed pullout-type specimens, which provide valuable local bond-slip information but do not reproduce the distributed cracking and tension-stiffening response of a reinforced-concrete tension zone. Systematic reviews report substantial dispersion among corrosion-bond models because of differences in specimen configuration, corrosion protocol, confinement, and damage metric.³ Recent direct-tension testing of 28 reinforced-concrete specimens further showed that advanced corrosion markedly reduces tension stiffening and tensile stress transfer between cracks and that current crack-spacing and crack-width models remain inadequate for corroded members.⁴ The present dataset is therefore revisited not as a new experimental campaign, but because it contains member-level direct-tension observations extending from the uncorroded state to severe measured corrosion, together with crack mapping, chloride profiles, localized corrosion observations, and a field-referenced bridge concrete.

The pullout program that forms Part I of the broader research program was originally reported in Amleh⁵: 192

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specimens across six cementitious systems and four cover thicknesses, including empirical relationships for bond deterioration with corrosion. A separate Part I journal manuscript is being developed from those thesis data; because it is not yet a published source, the present paper cites Amleh⁵ for the underlying pullout data. The present article addresses the companion direct-tension program: 30 specimens across three concrete types representing member-level response to the same corrosion process. Together, the archived datasets span local interface behavior (pullout) and distributed member response (direct tension), with chloride profiling and field context from the Dickson Bridge in Montreal.

The three concrete types selected reflect distinct practical scenarios. The Dickson Bridge concrete (NPC, $w/c = 0.52$) represents a field-referenced conventional bridge-deck concrete with documented chloride-induced deterioration. The bridge was constructed in 1959 in the east end of Montreal and had been exposed to a corrosive service environment for approximately 35 years when the detailed field investigation was undertaken. Its superstructure was approximately 366 m long and 27 m wide and comprised a 150 mm heavily reinforced concrete deck supported primarily on continuous reinforced-concrete beams, with the central spans over railway tracks supported on steel plate girders. Series C7 reproduced the original deck concrete using information from contract drawings and specifications, corroborated by petrographic examination. The NPC 0.32 concrete provides a higher-quality conventional reference, while the Sundance HVFA concrete represents a low-permeability SCM system used to examine how delayed corrosion initiation may coexist with different post-initiation member-level deterioration.

Legacy dataset and contribution of the present study. The experimental program was originally conducted as part of Amleh,⁵ and the underlying tests are not claimed here as newly generated data. The new contribution of this manuscript is the consolidated and critically re-audited analysis of the direct-tension dataset in the context of current corrosion research. Specifically, the paper (i) reconstructs and verifies specimen-level geometry, cracking, mass-loss, chloride, and load-response data from the archived records; (ii) examines the transition from local corrosion damage to member-level loss of tension stiffening and composite force transfer over corrosion levels extending to approximately 27% measured mass loss; (iii) retains the original relative-bond formulation, $u/u_0 = S_0/S$, while explicitly defining u/u_0 as a crack-spacing-derived relative bond stress rather than a direct local bond measurement; (iv) distinguishes corrosion-initiation resistance from post-initiation structural interaction through comparison of Portland-cement and high-volume fly-ash concretes; and (v) places the laboratory observations alongside detailed Dickson Bridge field measurements; and (vi) tests the transferability of a contemporary direct-tension bond-reduction relationship against the re-audited specimen-level predeterminations. The direct-tension data complement the predominantly local

bond-test literature and the recent renewed interest in tension stiffening of corroded members.^{3,4}

The objectives are (i) to characterize load-elongation response across the three concrete types and corrosion stages; (ii) to quantify the progressive loss of tension stiffening; (iii) to document transverse-crack spacing and crack-width evolution; (iv) to use crack spacing as a normalized member-level indicator of bond-mediated force transfer; (v) to compare the laboratory Series C7 results with the Dickson Bridge field observations while explicitly recognizing differences between accelerated and natural corrosion; and (vi) to assess the transferability of a contemporary direct-tension bond-reduction relationship across the three concrete systems.

Background

Tension stiffening is the contribution of the concrete between cracks to the overall stiffness of a reinforced concrete tension member, which arises from the bond between steel and concrete that allows partial force transfer even at cracked sections. The fundamental bond-corrosion relationship in pullout specimens has been characterised experimentally across a range of cementitious systems and corrosion levels, establishing that bond strength initially increases at low mass loss and then declines sharply as cover cracking develops.^{6–8} In an uncorroded member, this contribution is well characterized by models based on the fundamental bond-slip relationship.^{9,10} As corrosion develops, several interacting mechanisms degrade this contribution.

First, the expansive corrosion products generate bursting pressure that propagates longitudinal splitting cracks along the bar axis.^{11,12} These cracks reduce the concrete area surrounding the bar that can develop hoop tensile stresses, directly reducing bond resistance. Second, the corrosion products form a weak, friable layer at the steel–concrete interface that disrupts adhesion and reduces the friction bond.⁷ Third, the loss of bar rib profile, through corrosion pit development and rib degradation, reduces the primary mechanical interlocking mechanism for deformed bars. The net effect is that as corrosion progresses, the stabilized transverse-crack spacing S increases because the bond over a given length of bar is insufficient to introduce enough force into the concrete to form a new crack. This increase in crack spacing is both a symptom and a measure of bond deterioration.^{13,14}

Corrosion-induced deterioration of tension stiffening has received growing attention in recent literature. The steel–concrete bond mechanism under corrosion has been modeled analytically and numerically, with finite-element approaches demonstrating that the expansive pressure distribution is the primary driver of bond loss.^{15–17} Kunawisarut et al.¹ tested directly loaded corroded RC members and proposed an exponential bond-reduction relationship derived from crack-spacing-based residual bond capacity and corrosion mass loss. Syll and Kanakubo³ reviewed 84 published

bond-corrosion studies and noted that the tension specimen approach, while providing the most structurally relevant data, has been used in only a small fraction of studies. A further overview of the corrosion-bond loss relationship confirms that bond failure is a primary structural consequence of corrosion, with the transition from local adhesion loss to global member degradation governed by cover cracking, bar lug profile loss, and confining reinforcement,¹⁸ with most relying on pullout geometry. Recent work has demonstrated that current tension-stiffening models are insufficient for corroded members, underestimating stiffness loss and overpredicting crack spacing at intermediate corrosion levels.^{1,4,19} Even for uncorroded RC ties, tension-stiffening relationships exhibit strong dependence on concrete strength class, reinforcement ratio, and shrinkage history²⁰; these sensitivities are substantially compounded by corrosion. Experimental programs across multiple concrete grades have confirmed that the bond stress-slip response is nonlinear and that higher-grade concretes show better corrosion tolerance, with the initial bond-strength increase at low corrosion levels followed by a non-linear decline.^{21,22}

For SCM concretes specifically, the tension-stiffening literature is sparse.²³ In the broader bond-corrosion literature, fly ash concretes and alkali-activated slag concretes have been shown to delay corrosion cracking and maintain higher residual bond strength than OPC concretes at equivalent low-to-moderate corrosion levels, while showing comparable or lower bond strength once cracking occurs.²⁴ Bilodeau and Malhotra²⁵ noted that high-volume fly ash concretes exhibit slightly lower direct tensile strength than NPC concretes of equivalent compressive strength, a fact with direct implications for cracking load and initial crack spacing in tension members. The present study contributes a rare member-level direct-tension dataset on tension-stiffening deterioration in high-volume fly ash concrete under accelerated corrosion conditions.

Experimental Program

Materials and mixture proportions

Three concrete mixtures were used in this study, selected to span the range of cementitious systems relevant to Canadian infrastructure:

(1) NPC ($w/c = 0.52$) “Dickson Bridge Concrete” (Series C7): Normal Portland cement (CSA Type 10), limestone coarse aggregate ($1,020 \text{ kg/m}^3$), St. Gabriel de Brandon sand (720 kg/m^3), water (210 kg/m^3). This mixture replicates the concrete used in the Dickson Bridge deck as built in 1959, verified through contract drawings and petrographic studies. Compressive strength: 42.0 MPa (365-day); effective tensile strength inferred from the tension-member cracking response: 1.92 MPa; splitting tensile strength: 2.48 MPa.

(2) NPC ($w/c = 0.32$) “Normal Cement Concrete” (Series C3): As used for the companion pullout specimens.

Compressive strength: 60.2 MPa (365-day); effective tensile strength inferred from the tension-member cracking response: 2.84 MPa; splitting tensile strength: 3.23 MPa.

(3) Sundance fly ash concrete ($w/cm = 0.32$) “Sundance Concrete” (Series C4): Sundance high-calcium Class F fly ash ($\text{CaO} = 12.5\%$, Blaine = $408 \text{ m}^2/\text{kg}$) at 58% replacement of total cementitious material mass. Compressive strength: 66.6 MPa (365-day); effective tensile strength inferred from the tension-member cracking response: 2.59 MPa; splitting tensile strength: 3.32 MPa.

Reinforcing steel: 20M deformed bars ($d_b = 19.5 \text{ mm}$, yield strength 432 MPa) conforming to CSA G30.14-M83/ASTM A615-72. Bare bar load-elongation curve was recorded for each series.

Here, effective tensile strength denotes the value inferred in the archived analysis from the tension-member first-cracking response; it was not measured in a separate concrete tensile-strength test. These values are used only as comparative descriptors of the cracking response among the three series.

Specimen geometry and preparation

The direct-tension specimens were cylindrical, 1000 mm long and 120 mm in diameter, with a single centrally placed 20M deformed bar ($d_b = 19.5 \text{ mm}$). The bar extended approximately 150 mm beyond each end of the concrete cylinder for gripping. The nominal radial concrete cover was $(120 - 19.5)/2 = 50.25 \text{ mm}$, reported herein as approximately 50 mm ($c/d_b \approx 2.58$). The bar extensions and approximately 20 mm of bar within each specimen end were protected from accelerated corrosion using epoxy coating and tape. A $100 \text{ mm} \times 100 \text{ mm}$ surface grid was marked for crack observations. Six specimens (two per concrete mixture) were uncorroded controls, and 24 specimens were subjected to accelerated corrosion. The specimen concrete length was 1000 mm, the cylindrical diameter was 120 mm, the centrally placed 20M bar ($d_b = 19.5 \text{ mm}$) extended approximately 150 mm beyond each end, the nominal radial cover was approximately 50 mm, and elongation was measured over a 1010 mm extensometer gauge length as shown in Fig. 1. Specimens were moist-cured at $23 \pm 2^\circ\text{C}$ for 91 days, matching the pullout specimen curing protocol, to ensure completion of secondary pozzolanic reactions in the Sundance fly ash mixture.²⁵

Accelerated corrosion

The accelerated-corrosion program used an electrolytic solution of 5% NaCl by weight of water. The tension specimens were immersed in the solution maintained approximately 50 mm above the top of the concrete specimens, and the solution was replaced weekly to limit changes in NaCl concentration and pH. The embedded reinforcing bar served as the anode, and a steel bar/mesh assembly served as the cathode under a constant applied potential of 5 V DC. Current was recorded for each specimen approximately every 48 h using a digital multimeter after the reading

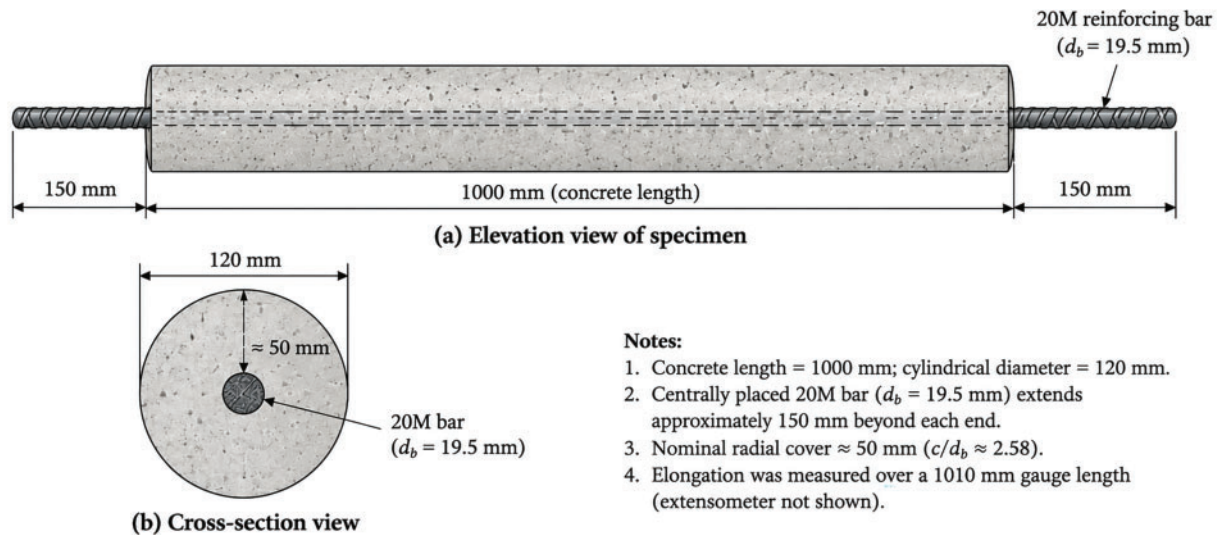


Figure 1. Schematic of the tension-specimen geometry

stabilized for approximately 5–10 min. Half-cell potentials were also monitored periodically. For the tension specimens, corrosion-rate measurements were made at three locations after the target corrosion stage was reached. The original accelerated program was designed to reach severe corrosion over approximately 15–20 weeks. Cumulative charge histories and specimen-specific Faraday-law estimates were not retained in a form that can be reconstructed reliably and are therefore not reported retrospectively. Corrosion stages were defined using multiple observations, including current response, longitudinal crack development, measured mass loss, cross-sectional loss, and chloride content.

A further limitation concerns corrosion-product chemistry. Accelerated impressed-current corrosion can produce corrosion-product assemblages and spatial distributions that differ from those formed during long-term natural chloride exposure. Because different iron oxides, oxyhydroxides, and hydroxides have substantially different volumetric expansion relative to the consumed steel, the relationship among gravimetric mass loss, interfacial pressure, cracking, and bond deterioration is not unique. Molina et al.²⁶ demonstrated the strong sensitivity of corrosion-induced expansion to assumed corrosion-product composition. Corrosion products were not chemically characterized in the present program; consequently, differences in product chemistry constitute an additional uncertainty when extrapolating the accelerated-corrosion observations to naturally corroded bridge members.

Direct tension testing

Direct-tension tests were conducted in the Structures Laboratory at McGill University using a servo-hydraulic MTS machine with a 1000 kN tension capacity. The specimens were centered to achieve axial tension and loaded incrementally under displacement control through grips attached to the reinforcing bar at both ends. Load, displacement, and strain data were acquired every 5 s during loading. An

external MTS extensometer was clamped to the reinforcing bar immediately outside the two concrete end faces; thus, the 1010 mm gauge length measured the elongation across the 1000 mm concrete member plus the short exposed bar segments between the concrete faces and extensometer clamps, not the full approximately 1300 mm bar length including the gripping extensions. Loading was paused at key stages for crack mapping and photography. Cracks were traced as soon as visible, the corresponding load was marked, and crack widths were measured with a crack-width comparator; a 10x hand microscope was used where necessary to follow crack propagation.

A concrete shrinkage pre-strain correction of approximately (0.3×10^{-3}) , based on Bischoff,²⁷ was applied to the control-specimen elongation measurements. No corresponding correction was applied to the corroded specimens because they had remained continuously immersed in NaCl solution and showed no shrinkage pre-strain.

Chloride profiles

After tension testing, chloride ion content used for the specimen profiles was determined at nominal depths of approximately 7, 25, and 50 mm from the concrete surface using the specific ion probe method (ASTM C1152-90).²⁸ The deepest measurement level corresponds approximately to the reinforcing-steel level in the 120 mm diameter specimens.

Results

Corrosion stages and bar profile

The experimental program is summarized in Table 1. Table 2 summarizes selected representative corrosion and cracking observations. The complete 30-specimen dataset is in Supplementary Table S1. Table 2 displays selected stages;

Table 1. Experimental program overview, three tension specimen series, 30 specimens total

Series	Concrete type	w/cm	f'_c (MPa)	f_t (MPa)	No. of specimens	Corrosion stages
C7	NPC–Dickson Bridge deck ($w/c = 0.52$)	0.52	42.0	1.92	10	1–8 + 2 control
C3	NPC ($w/c = 0.32$)	0.32	60.2	2.84	10	1–7 + 2 control
C4	Sundance fly ash, 58% FA ($w/cm = 0.32$)	0.32	66.6	2.59	10	1–8 + 2 control

Note: f'_c = 365-day compressive strength (MPa); f_t = effective tensile strength inferred from the tension-member cracking response (MPa). All specimens: 20M deformed bar ($d_b = 19.5$ mm), 120 mm specimen diameter, approximately 50 mm radial concrete cover ($c/d_b \approx 2.58$), accelerated corrosion in 5% NaCl at 5 V DC, 91-day moist cure. FA = fly ash.

Table 2. Selected representative corrosion and cracking observations from the tension program

Specimen	Stage	Long. cracks	Avg. long. width (mm)	ML (%)	No. trans. cracks	Avg. spacing (mm)	1st crack load (kN)	Yield load (kN)
C7-1C (ctrl)	0	0	0	0	11	83	18	127
C7-4C	1	1	1.2	8	9	100	23	116
C7-5C	2	1	1.5	11	7	125	24	102.5
C7-9C	8	2	4.0	27	0	N/A	N/A	45
C3-1C (ctrl)	0	0	0	0	12	77	19	127
C3-5C	1	0	0	1.5	12	77	34	125
C3-6C	5	3	2.0	7.7	4	200	31	115
C3-4C	7	3	3.0	14.5	2	333	37	83
C4-2C (ctrl)	0	0	0	0	10	91	21	128
C4-10C	1	0	0	0.9	11	83	31	127
C4-4C	6	0	0	2.5	7	125	36	124.5
C4-8C	8	0	0	4.5	6	143	30	90

Note: Control-specimen first-cracking values were adjusted for concrete shrinkage pre-strain. For C7-9C, no transverse cracks formed before steel yielding. ML = rebar mass loss (%). Control specimens: 2 per series (one shown as representative). Long. = longitudinal.

the complete specimen-by-specimen archival dataset is provided in Supplementary Table S1. Stage numbers denote the archived corrosion progression within each mixture and are not assumed to represent identical measured mass loss across mixtures. Cross-series comparisons are therefore made using measured mass loss and observed response rather than nominal stage number. A representative bar profile examination (Fig. 2) revealed localized corrosion concentrated at discrete regions along the bar length, with deep corrosion pits developing preferentially at rib roots. This localized character meant that some bar ribs remained in good condition adjacent to heavily corroded zones, accounting for the residual concrete contribution observed even in severely corroded specimens. During the early corrosion stages, visual examination indicated localized attack rather than a regularly spaced sequence of pits at every rib root. Some rib roots showed preferential attack while adjacent ribs remained comparatively intact, and rust accumulation was correspondingly non-uniform along the bar. The original program documented pit depth, bar-diameter change, rib degradation, and general corrosion morphology after testing but did not record a statistically complete pit-to-pit spacing distribution or rust thickness at each rib root. The

revised manuscript therefore reports the observed localization without assigning an unsupported characteristic pitting frequency or spacing.

The Sundance fly ash specimens (Series C4) were distinctive in that none developed visible longitudinal splitting cracks due to corrosion (Table 2). Nevertheless, localized corrosion and local spalling were observed. Restricted migration of corrosion products within the denser fly-ash matrix is one possible explanation for this localization, but corrosion-product transport and interfacial pressure were not measured directly. The permeability-related localization mechanism is therefore treated only as a working hypothesis.

Load-elongation responses

Figs. 3 through 5 present the load-elongation responses for representative specimens from each series alongside the corresponding control specimen and the bare bar response. The following features are common to all three concrete types and consistent with the standard load-elongation model for reinforced tension members:

(i) An initial uncracked elastic phase, in which the load-elongation response is stiffer than the bare bar due to the concrete contribution.

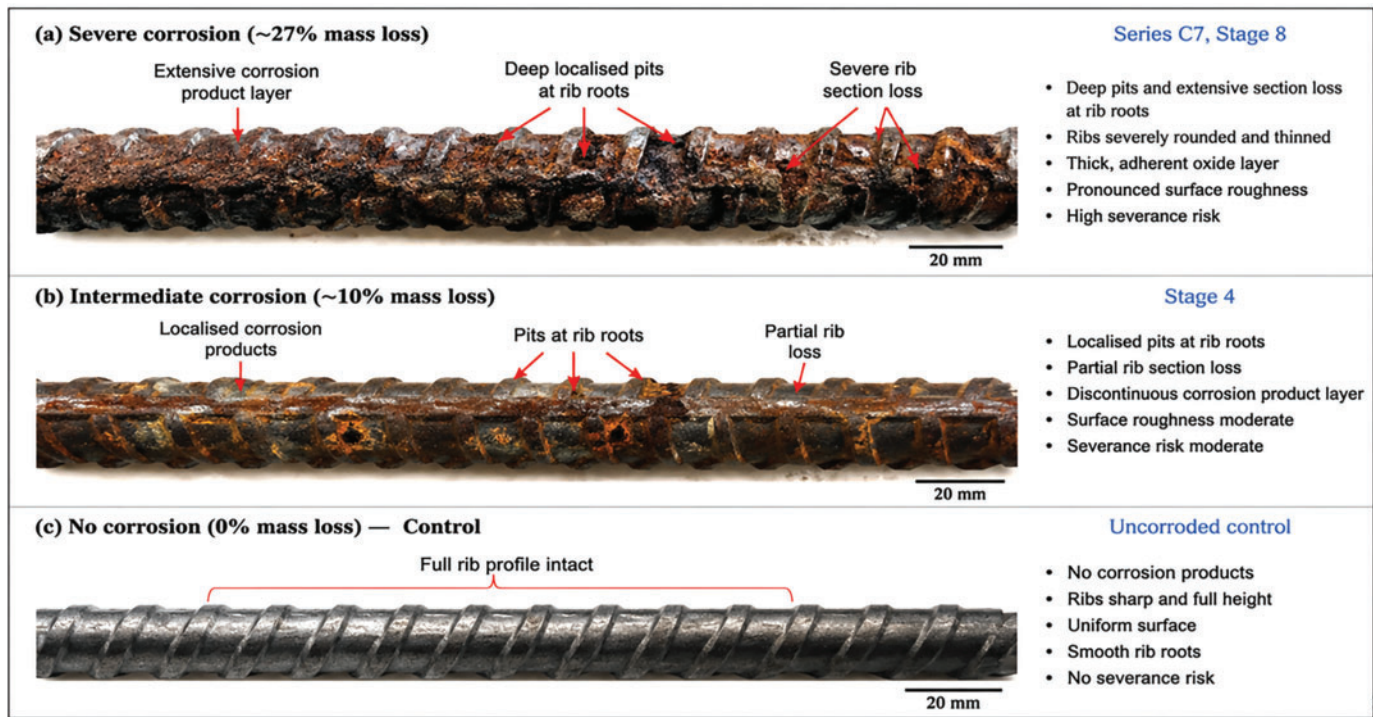


Figure 2. Representative post-test bar profiles: (a) heavily corroded Series C7 bar at Stage 8 showing localized pits and severe rib degradation; (b) intermediate-corrosion bar showing selective rib loss; and (c) uncorroded control bar

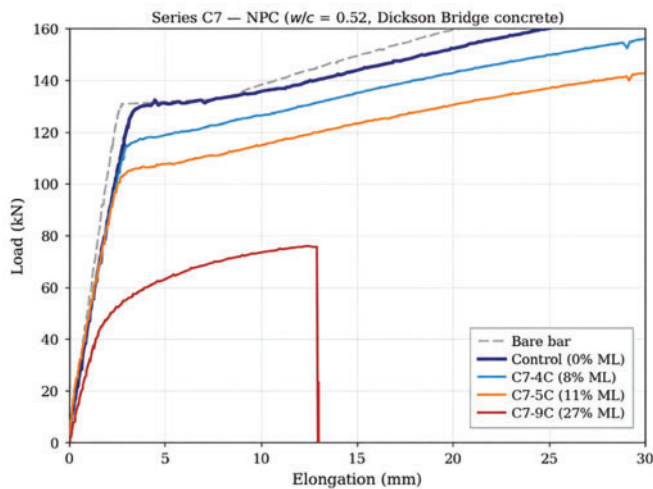


Figure 3. Load-elongation responses: Series C7 (NPC, $w/c = 0.52$, Dickson Bridge concrete). Control specimen C7-2C, corroded specimens C7-4C (8% ML), C7-5C (11% ML), and C7-9C (27% ML), compared with the bare-bar response

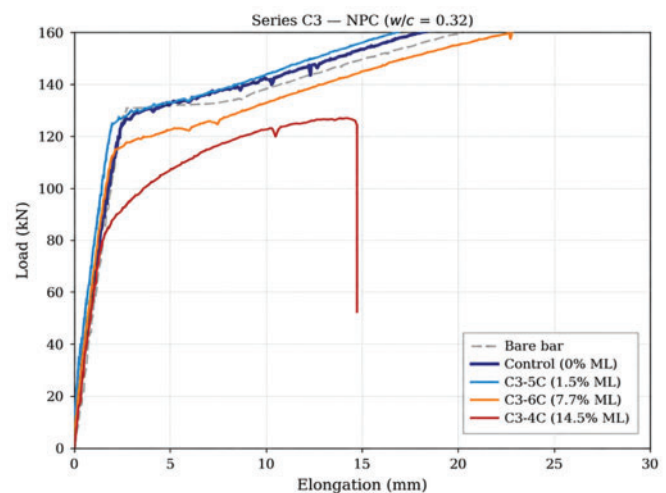


Figure 4. Load-elongation responses: Series C3 (NPC, $w/c = 0.32$). Control specimen C3-1C, C3-5C (1.5% ML), C3-6C (7.7% ML), C3-4C (14.5% ML), and bare-bar response

(ii) A cracking transition, in which the tensile strength of the concrete is exceeded and transverse cracks form. At each cracking event, the load drops slightly as the concrete contribution at that section is lost.

(iii) A stabilized cracking phase, in which the load-elongation curve becomes parallel to the bare bar response. Following cracking, the specimen response remains above

and to the left of the bare-bar response because of the concrete contribution between cracks. This tension-stiffening contribution may be expressed either as an additional load at a given elongation or, as adopted in Section 4.3, as a reduction in elongation at a given load level.

(iv) Onset of yielding of the steel bar. The yield loads reported in Tables 2 and S1 denote the identified onset of steel yielding; the recorded load-elongation response may continue beyond this point because of post-yield steel strain

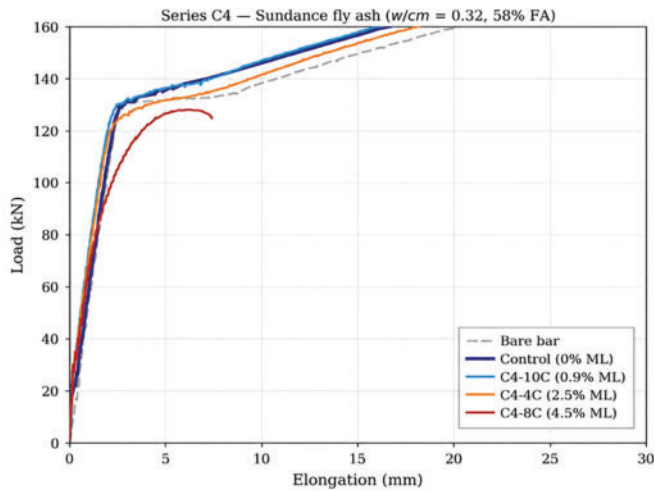


Figure 5. Load-elongation responses: Series C4 (Sundance fly ash, $w/cm = 0.32$). Control specimen C4-2C, C4-10C (0.9% ML), C4-4C (2.5% ML), C4-8C (4.5% ML), and bare-bar response

hardening. The terminal vertical drop in a plotted curve represents unloading or rupture at the end of the recorded test and should not be interpreted as the yield load.

As corrosion level increases, the load-elongation curve shifts progressively towards the bare bar response, indicating progressive loss of tension stiffening. This pattern is consistent with recent direct tension tests on corroded RC members using Digital Image Correlation, which similarly found that corrosion significantly reduces tension stiffening, with larger bar sizes exacerbating the effect.⁴ Syll et al.²⁷ similarly demonstrated that bond degradation due to corrosion-induced longitudinal cracking reduces the number of transverse cracks and diminishes tension stiffening, with the effect most pronounced at deformations below 1.5 mm. The most severely corroded specimens (C7-9C at 27% ML, C3-4C at 14.5% ML, C4-8C at 4.5% ML) show virtually no concrete contribution in the load-elongation plot; their response is essentially that of the bare bar with reduced yield load due to cross-section loss.

For severely pitted specimens, a yield load calculated from the average residual area and the uncorroded-bar yield stress is not expected to reproduce the measured onset of yielding. Average mass loss does not represent the minimum load-carrying cross-section, and localized pitting may govern yielding before the average residual section reaches the nominal uncorroded yield stress.

At very low corrosion levels (Stage 1), a slight increase in the cracking load relative to control specimens was observed in all three series. This is attributable to the absence of concrete shrinkage pre-strain in the continuously immersed corroded specimens,²⁷ rather than a genuine improvement in tensile strength or bond.

Tension stiffening quantification

The tension-stiffening response was first evaluated from the horizontal separation between the specimen and bare-bar load-elongation curves at equivalent load levels. This separation represents the reduction in average elongation associated with the contribution of the surrounding concrete.

To account for corrosion-induced loss of reinforcement cross-sectional area, the measured responses were also expressed as derived nominal stress-strain relationships. For each specimen, the equivalent residual steel area was calculated as $A_{ms} = A_{so} (1 - ML/100)$, where $A_{so} = 300 \text{ mm}^2$ is the nominal cross-sectional area of the 20M reinforcing bar, and ML is the verified gravimetric mass loss in percent.

The derived nominal stress was calculated as $\sigma = P/A_{ms}$, where P is the measured tensile load. Average strain was calculated as $\varepsilon = \Delta L/1010$, using the 1010 mm extensometer gauge length.

The resulting relationships for representative specimens from Series C7, C3, and C4 are presented in Figs. 6–8, respectively. Normalizing the measured load by the reduced residual steel area increases the derived nominal stress for a given load, particularly at higher mass-loss levels. Consequently, increased separation from the bare-bar response in this representation should not be interpreted as a physical increase in the concrete contribution with corrosion. It reflects the combined influence of tension stiffening and normalization by the reduced steel area.

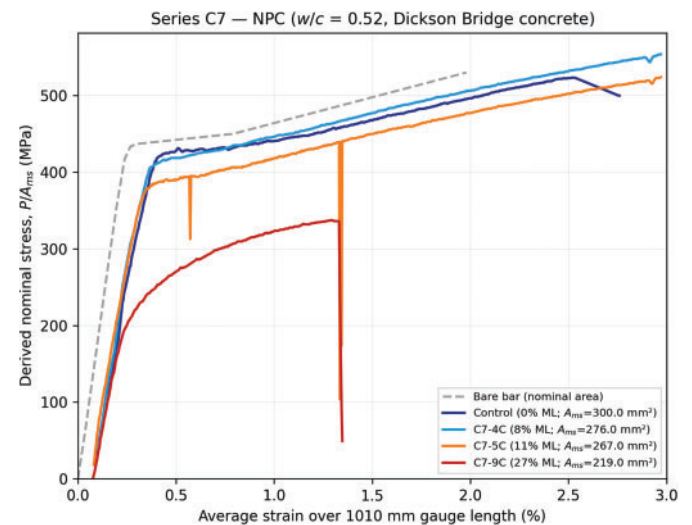


Figure 6. Derived nominal stress-strain relationships for representative Series C7 specimens. Derived nominal stress was calculated using the specimen-specific equivalent residual steel area, A_{ms} , determined from the verified mass loss. Average strain was calculated over the 1010 mm extensometer gauge length

The two representations therefore provide complementary information. The load-elongation curves in Figs. 3–5 describe the measured member response, including changes in load capacity and deformation capacity. The derived nominal stress-strain relationships in Figs. 6–8 facilitate

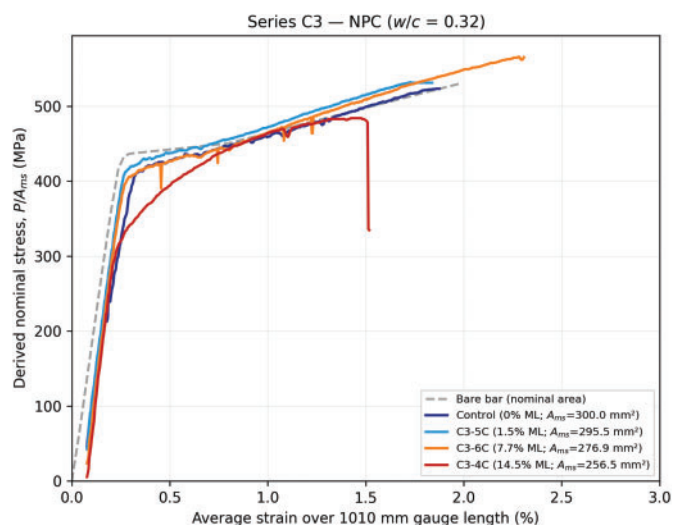


Figure 7. Derived nominal stress–strain relationships for representative Series C3 specimens. Derived nominal stress was calculated using the specimen-specific equivalent residual steel area, A_{ms} , determined from the verified mass loss. Average strain was calculated over the 1010 mm extensometer gauge length

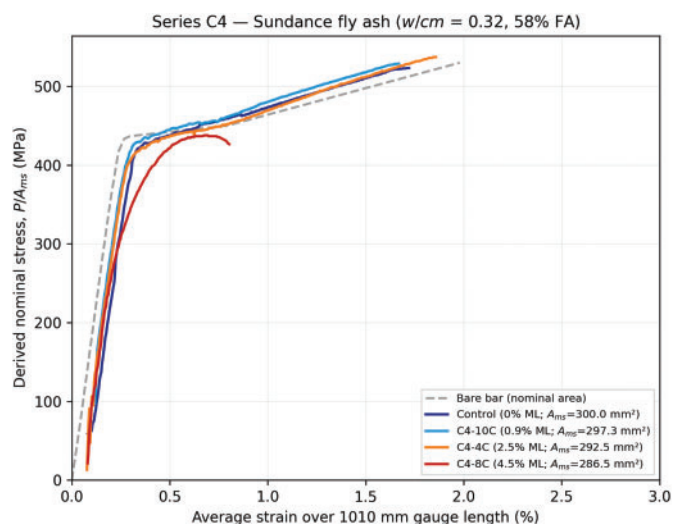


Figure 8. Derived nominal stress–strain relationships for representative Series C4 specimens. Derived nominal stress was calculated using the specimen-specific equivalent residual steel area, A_{ms} , determined from the verified mass loss. Average strain was calculated over the 1010 mm extensometer gauge length

comparison after accounting for the estimated corrosion-induced reduction in steel area; they do not independently separate the force carried by the concrete.

Cracking behavior

NPC Concrete ($w/c = 0.52$) Dickson Bridge Series: Control specimens C7-1C and C7-2C developed 11 transverse cracks

with an average stabilized spacing of 83 mm. With an approximately 50 mm radial cover, twice the cover is approximately 100 mm. The observed 83 mm average spacing is therefore of the same order as, and somewhat below, this simple cover-based characteristic length; it should not be interpreted as a strict theoretical minimum because stabilized crack spacing also depends on bond-transfer length, concrete tensile strength, bar geometry, and the distribution of cracks along the member. The first cracking load (adjusted for shrinkage pre-strain) was approximately 18–19 kN.

With increasing corrosion, the number of transverse cracks decreased, and their spacing increased. At the intermediate C7 condition represented by C7-8C at 16% measured mass loss, six transverse cracks formed, with an average stabilized spacing of 143 mm. At C7-9C (27% measured mass loss), no new transverse crack appeared before yielding of the steel bar. Accordingly, a finite stabilized crack spacing and the crack-spacing-derived relative bond stress u/u_0 cannot be calculated for this terminal specimen. Its response is discussed qualitatively as severe loss of bond-mediated force transfer and tension stiffening. The maximum crack width increased substantially with corrosion level at a given steel stress (Table 3).

NPC Concrete ($w/c = 0.32$): Control specimens C3-1C and C3-2C developed 12 transverse cracks at an average spacing of 77 mm, reflecting the higher concrete quality ($f'_c = 60.2$ MPa vs. 42.0 MPa for Series C7) and more effective initial bond-mediated force transfer. The first cracking load was approximately 34 kN, substantially higher than for Series C7, consistent with the higher splitting tensile strength.

The Series C3 specimens showed the same systematic increase in crack spacing with corrosion level. At Stage 7 (C3-4C, 14.5% mass loss), only 2 transverse cracks formed. At 1.5% measured mass loss, C3-5C retained a yield load essentially equal to the control, an average transverse-crack spacing of 77 mm, and $u/u_0 = 1.00$, with no major longitudinal splitting. This is consistent with the substantial approximately 50 mm radial cover and relatively dense NPC $w/c = 0.32$ concrete, for which this early measured corrosion state had apparently not produced visible cover splitting or a measurable increase in stabilized transverse-crack spacing. Because corrosion-product pressure was not measured directly, this observation does not establish a quantitative cover-cracking threshold. The appearance of two longitudinal cracks in the separate C3-8C specimen at 2.2% measured mass loss is qualitatively consistent with the threshold-like nature of corrosion-induced splitting once expansive stresses become sufficient relative to the tensile resistance and confinement of the surrounding concrete. Because C3-5C and C3-8C are different specimens, the interval between 1.5% and 2.2% ML should not be interpreted as a measured transition threshold for an individual member.

Sundance Fly Ash Concrete ($w/cm = 0.32$): Control specimens C4-1C and C4-2C developed 10 transverse cracks at an average spacing of 91 mm, yielding at approximately 127 kN (steel stress 423 MPa). The first cracking load was 22 kN, lower than for C3 (34 kN) despite the higher compressive strength, consistent with the lower effective tensile strength

Table 3. Maximum transverse crack width (mm) at selected steel-stress levels for representative specimens

Specimen	100 MPa	150 MPa	200 MPa	250 MPa	300 MPa	350 MPa	At yield
C7-1C (control)	0.10	0.12	0.18	0.22	0.28	0.35	0.40
C7-4C (8% ML)	0.20	0.35	0.55	0.70	1.00	—	—
C7-5C (11% ML)	0.20	0.40	0.70	0.80	—	—	—
C3-1C (control)	0.08	0.10	0.15	0.17	0.20	0.28	0.35
C3-5C (1.5% ML)	0.08	0.10	0.15	0.18	0.22	0.30	0.40
C3-6C (7.7% ML)	0.20	0.25	0.50	1.00	—	—	—
C4-2C (control)	0.08	0.15	0.20	0.50	0.55	—	—
C4-8C (4.5% ML)	0.15	0.20	0.23	0.33	0.43	0.50	0.55

Table 4. Crack-spacing-derived relative bond stress as a function of measured reinforcement mass loss for selected representative specimens

Series/specimen	Measured mass loss (%)	S_0 (mm)	S (mm)	$u/u_0 = S_0/S$
C7-4C	8	83	100	0.83
C7-5C	11	83	125	0.66
C7-6C	24	83	500	0.17
C7-9C	27	83	Undefined	N/A: no new transverse crack before yield
C3-5C	1.5	77	77	1.00
C3-4C	14.5	77	333	0.23
C4-10C	0.9	91	83	1.10
C4-8C	4.5	91	143	0.64

inferred from the tension-member response of the fly ash concrete (2.59 MPa vs. 2.84 MPa for C3), as reported by Bilodeau and Malhotra²⁵ for HVFA concretes.

Critically, the Series C4 specimens showed no visible longitudinal splitting cracks due to corrosion. Despite this apparently favorable surface-cracking behavior, increasing corrosion was accompanied by wider transverse-crack spacing and reduced bond-mediated force transfer. At approximately matched low corrosion levels, the crack-spacing-derived relative bond stress for C4 was slightly lower than for C3, but the available observations do not isolate fly ash as the controlling cause. The results instead indicate that visible longitudinal splitting alone is not sufficient to characterize post-initiation structural deterioration in HVFA concrete. Localized rib damage, local spalling, concrete tensile properties, and corrosion-product localization are plausible contributors; the present program did not measure these mechanisms independently.

Crack-spacing-derived relative bond stress

Following the original mechanics-based formulation, a normalized relative bond stress was inferred from the measured stabilized crack spacing:

$$u/u_0 = S_0/S \quad (1)$$

where S_0 is the average stabilized crack spacing of the uncorroded control specimens, and S is the corresponding spacing of a corroded specimen. The normalized quantity

u/u_0 is interpreted as a crack-spacing-derived indicator of relative bond-mediated force transfer and not as a direct measurement of local interfacial bond strength.

The maximum transverse crack widths recorded at selected steel-stress levels are summarized in Table 3. These measurements complement the crack-spacing analysis by illustrating the progression of localized cracking with corrosion.

The dash indicates that a crack-width value was unavailable in the archived record at the stated steel-stress level. For several corroded specimens, this reflects yielding or termination of the test before that stress level was reached. For the uncorroded C4-2C control at 350 MPa, the dash is treated as an archival data gap because the specimen yielded at approximately 423 MPa. Where an exact stress level was not recorded, the crack width was obtained by interpolation between adjacent archived observations.

The crack-spacing-derived relative bond-stress values calculated using Eq. (1) are summarized for selected representative specimens in Table 4. The specimen selections differ from those in Table 2 because the tables serve different purposes. The complete specimen-level dataset is provided in Supplementary Table S1 in the online supplementary material.

ML denotes measured reinforcement mass loss. For C7-9C, no new transverse crack formed before steel yielding; therefore, a finite stabilized crack spacing could not be established, and S and u/u_0 are undefined.

The progression of u/u_0 with measured reinforcement mass loss is presented separately for Series C7, C3, and C4 in Figs. 9–11, respectively. The symbols represent individual specimen observations. Lines connect the observations within each series to illustrate the measured response progression and do not represent fitted regressions or predictive relationships.

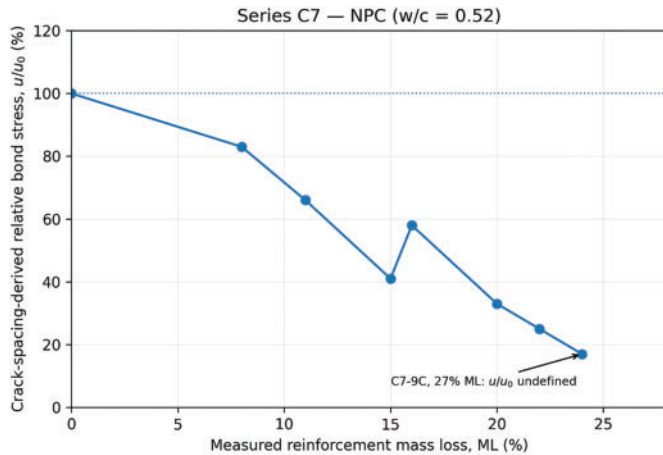


Figure 9. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for Series C7 (NPC, $w/c = 0.52$). Symbols represent verified specimen observations, and lines connect observations only to illustrate response progression. C7-9C at 27% ML is not assigned a numerical u/u_0 because no new transverse crack formed before steel yielding

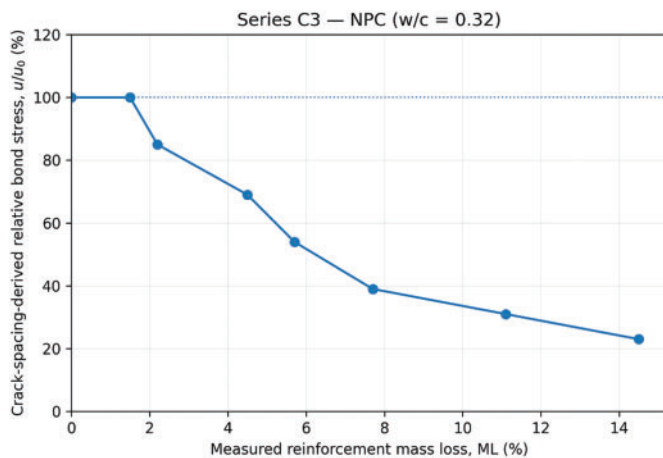


Figure 10. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for Series C3 (NPC, $w/c = 0.32$). Symbols represent verified specimen observations, and lines connect observations only to illustrate response progression

NPC ($w/c = 0.52$). As shown in Fig. 9, the verified spacing was 100 mm at 8% measured mass loss, compared with 83 mm for the control, giving $u/u_0 = 83/100 = 0.83$. The crack-spacing-derived relative bond stress decreased

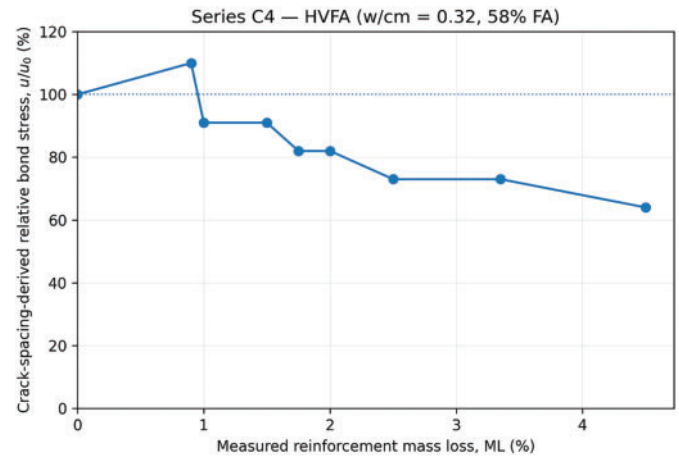


Figure 11. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for Series C4 (Sundance HVFA, $w/cm = 0.32$). The lowest-corrosion specimen exhibits an apparent value above unity; this response is interpreted cautiously because crack spacing is influenced by concrete tensile response and crack stabilization as well as bond-mediated force transfer

further with advanced corrosion, reaching approximately 0.17 at 24% mass loss. For C7-9C at 27% mass loss, no new transverse crack formed before steel yielding. A finite stabilized crack spacing could not therefore be established, and Eq. (1) could not be applied. This specimen is reported qualitatively as a terminal condition with severely impaired bond-mediated force transfer rather than being assigned a numerical residual value.

NPC ($w/c = 0.32$). Fig. 10 shows that, at 1.5% mass loss, the average crack spacing remained 77 mm, equal to that of the control, giving $u/u_0 = 1.00$. With increasing corrosion, spacing increased progressively. For C3-4C at 14.5% mass loss, $S = 333$ mm and $u/u_0 = 77/333 \approx 0.23$.

Sundance fly ash ($w/cm = 0.32$). As shown in Fig. 11, the lowest-corrosion specimen exhibited a small apparent increase in u/u_0 because its measured crack spacing was slightly smaller than that of the control. At higher corrosion levels, transverse-crack spacing increased and u/u_0 decreased, reaching approximately 0.64 at 4.5% mass loss. This response is interpreted cautiously because concrete tensile strength and crack stabilization influence crack spacing in addition to bond-mediated force transfer, and the achieved corrosion states were not exact replicates.

Chloride profiles

Fig. 12 shows the chloride ion content profiles across the cover depth for all corroded specimens with archived profile data. Nominal profile depths are approximately 7, 25, and 50 mm from the surface, with the deepest level at approximately the reinforcing-steel level (120 mm diameter specimen, 50 mm radial cover). The three panels use different horizontal scales to accommodate the markedly different

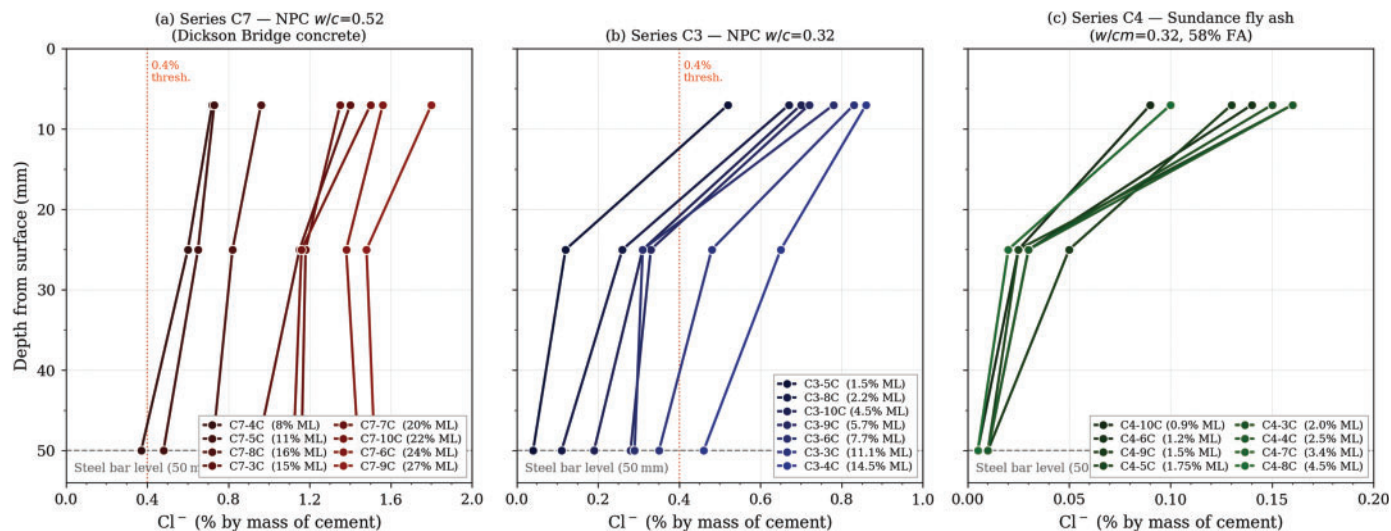


Figure 12. Chloride ion content profiles across cover depth for all corroded specimens with archived profile data: (a) Series C7; (b) Series C3; and (c) Series C4

chloride-concentration ranges; comparisons should therefore be made using the axis values rather than visual slope alone. Key observations:

(i) The Dickson Bridge concrete ($w/c = 0.52$) shows substantially higher chloride ingress at all depths, consistent with its higher permeability, with chloride contents at the rebar level reaching 0.7–1.4% by mass of cement at the higher corrosion stages. These values are broadly consistent with the range observed in the Dickson Bridge field survey (Section 4.7).

(ii) The NPC 0.32 and Sundance fly ash concretes show markedly lower chloride penetration than Series C7, but their rebar-level ranges differ materially. Series C3 shows approximately 0.04–0.46% chloride by mass of cement at the steel level, whereas Series C4 remains substantially lower, generally approximately 0–0.02%. The markedly lower C4 values are consistent with reduced chloride ingress,²⁹ although the present data do not isolate chloride ingress, corrosion morphology, and structural response as independent effects. The difference between the two low- w/cm systems was less pronounced in the companion pullout specimens, which included a wider 25–100 mm cover range than the fixed approximately 50 mm radial cover of the tension specimens.

(iii) Preferential chloride transport through cracks relative to intact concrete has been documented in the literature.¹⁹ However, the present chloride profiles (Fig. 12) were measured at single representative depths per specimen rather than as paired crack-location and inter-crack-location profiles; they therefore describe the overall chloride concentration gradient through the cover and do not directly demonstrate crack-enhanced transport in these specimens.

Correlation with Dickson bridge field data

Fig. 13 compares Series C7 laboratory observations with Dickson Bridge field measurements. The field investigation included detailed testing of four randomly selected approximately 5 m × 6 m deck areas, with many measurements collected on a 0.25 m × 0.25 m grid.^{30,31} Field points showed considerable scatter, particularly around chloride contents of approximately 0.3–0.5% and mass losses of roughly 5–20%. This is expected because measured chloride content is a local state variable at the time of sampling, whereas steel mass

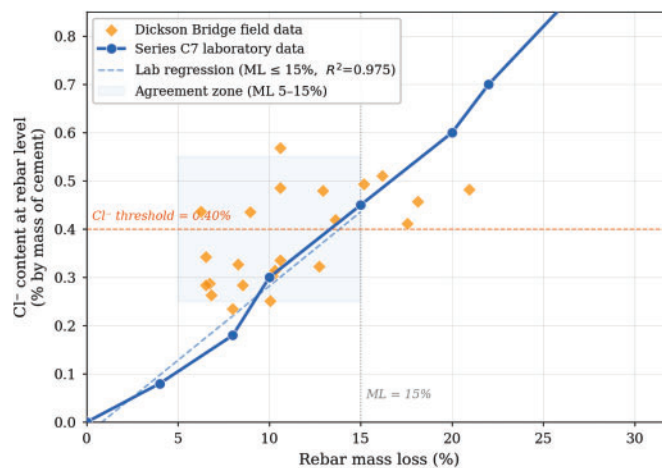


Figure 13. Correlation between Series C7 laboratory data and Dickson Bridge field survey: rebar mass loss vs. chloride content at steel level. Field data cluster between 0.3–0.5% Cl^- and 5–20% mass loss, showing qualitative consistency with laboratory C7 data for $ML < 15\%$

loss integrates the preceding exposure history. Deicing-salt use ceased after the bridge was abandoned in 1993, and subsequent snowmelt could redistribute and leach chlorides within the deck. Spatial variability in cover, cracking, permeability, moisture availability, and localized pitting also contributes to the scatter. For mass losses below approximately 15%, the average trends in the field and Series C7 datasets are qualitatively consistent; this comparison is treated as field consistency rather than formal validation of the accelerated-corrosion method.

The relatively large spread in field mass loss around approximately 0.4–0.5% chloride content is physically plausible and should not be interpreted as a unique chloride-to-mass-loss relationship. The Dickson Bridge field investigation documented substantial spatial variability in concrete quality, compaction, cover, electrical resistivity, moisture/exposure history, and localized surface condition. In addition, de-icing salt application ceased after bridge decommissioning, and subsequent seasonal melt could redistribute or leach chlorides, so the chloride concentration measured at the time of sampling need not equal the concentration that initiated or sustained the accumulated corrosion. Local pitting further decouples gravimetric average mass loss from a single chloride measurement. Future programs should therefore use replicate specimens, spatially resolved chloride and moisture/resistivity measurements, repeated electrochemical monitoring, and quantitative pit-depth/section-loss mapping, and should analyze uncertainty explicitly rather than fitting a deterministic relationship to single specimens.

Discussion

Mechanistic interpretation: six-stage cracking model

The experimental observations are consistent with a six-stage deterioration model for bond and cracking in tension members under progressive corrosion:

Stage 1: Initial corrosion products: A thin layer of iron oxide corrosion products forms at the bar surface, occupying a substantially larger volume than the parent steel, typically two to seven times,²⁵ and increasing hoop stress and frictional resistance. The crack-spacing-derived relative bond stress may show a small apparent increase at very low corrosion; C4-10C gave $u/u_0 = 1.10$ at 0.9% measured mass loss. This is treated as an apparent low-corrosion enhancement rather than a direct measurement of increased local bond strength because crack spacing is also influenced by concrete tensile response and crack stabilization.

Stage 2: Expansive pressure buildup: Further corrosion product accumulation generates sufficient bursting pressure to create tensile stresses exceeding the concrete tensile strength along the bar axis. Finite-element modelling has confirmed that this pressure distribution is highly non-uniform, with stress concentrations at rib roots.¹⁵ Longitudinal splitting cracks initiate.³² Experimental pullout

data confirm that bond strength peaks in the range of 0.5–1.5% mass loss before declining.^{6,7}

Stage 3: Cover cracking and bond-loss onset: Longitudinal crack propagation disrupts the surrounding concrete ring and reduces effective confinement of the bar. Adhesion at the steel-concrete interface is progressively lost, and bond-mediated force transfer begins to decline measurably, consistent with analytical interface modelling.¹⁶

Stage 4: Force transfer to ribs: Load transfer shifts from adhesion to mechanical interlocking at the ribs. Internal cracks form adjacent to main transverse cracks,¹³ redistributing force transfer.

Stage 5: Internal cracking proliferation: Progressive internal cracking around the bar reduces the rate of force transfer, increasing the stabilized transverse-crack spacing S . Fewer transverse cracks form during loading, and those that form are more widely spaced. This bond-spacing relationship has been confirmed experimentally in corroded RC members under direct loading.^{1,33}

Stage 6: Near-complete bond loss: Rib profile degradation, heavy corrosion product layer, and extensive longitudinal cracking combine to reduce bond transfer capacity to negligible levels. At the limiting condition, no new transverse cracks may form before steel yielding, as observed for C7-9C at 27% measured mass loss. Other severely corroded specimens, such as C3-4C at Stage 7, still formed a small number of widely spaced transverse cracks (two at 333 mm spacing) but with strongly reduced tension stiffening, approaching the bare-bar response. The state-of-the-art review by Zhao and Lin²² confirms this as the terminal stage of the corrosion-bond deterioration process, and Apostolopoulos and Koulouris¹⁸ document that rib profile loss is a primary driver of near-complete bond loss at high corrosion levels.

The mass-loss level at which these response stages become apparent is not universal. It depends on cover, concrete tensile properties, confinement, corrosion localization, corrosion-product transport, and exposure history. Thinner cover and lower tensile resistance generally reduce resistance to splitting, while differences in permeability and corrosion-product migration may alter the localization and surface expression of damage. The present dataset does not isolate these influences sufficiently to assign universal stage-transition thresholds.

Comparison of three concrete types

Practical implications for bridge engineering. The results demonstrate that corrosion-related changes in crack spacing and tension stiffening can become pronounced well before complete section loss. Because the crack-spacing index is indirect, and because the original paired target-condition design produced generally unique achieved mass-loss states rather than replicated observations at identical measured corrosion levels, the data should be used to inform mechanistic understanding and model assessment rather than as stand-alone rehabilitation thresholds.

NPC ($w/c = 0.52$): Dickson Bridge: This series had the highest permeability and most extensive longitudinal cracking and reached advanced corrosion states over the

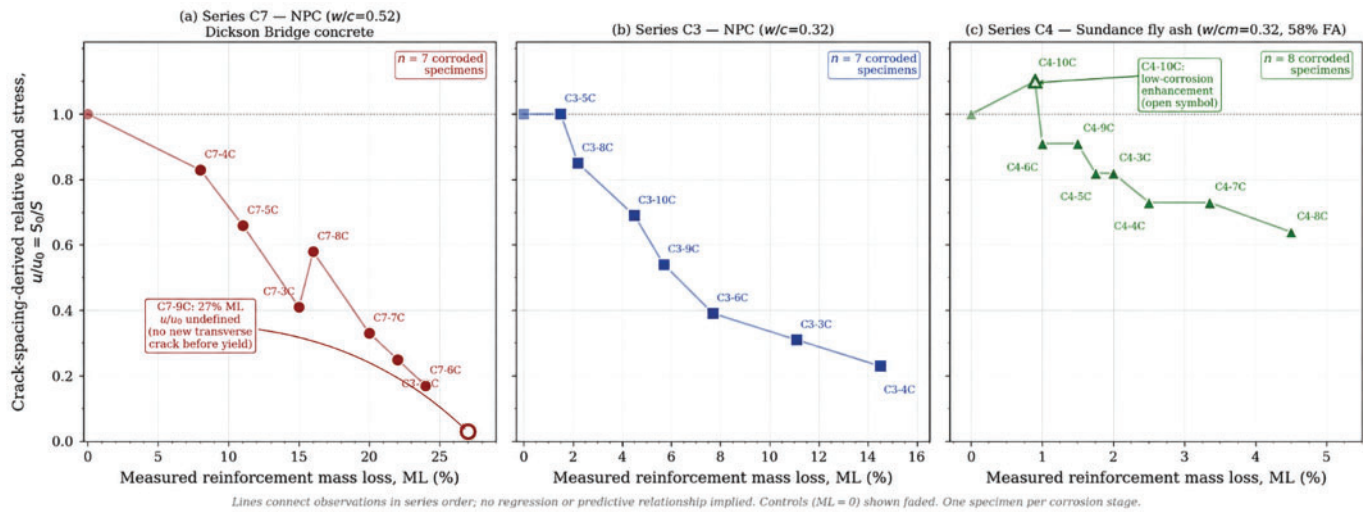


Figure 14. Crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$, versus measured reinforcement mass loss for the three concrete series

accelerated exposure program. Bond-mediated force transfer decreased progressively over the measured 0–24% mass-loss range, while C7-9C at 27% ML reached a terminal condition in which no new transverse crack formed before steel yielding. The laboratory observations show qualitative consistency with the Dickson Bridge field data over the moderate mass-loss range, but differences in corrosion history, spatial variability, and accelerated versus natural exposure preclude treating the comparison as formal validation.

NPC ($w/c = 0.32$): Highest effective tensile strength inferred from the tension-member cracking response and the smallest uncorroded stabilized crack spacing among the three series (77 mm vs. 83 mm for C7 and 91 mm for C4), consistent with effective initial bond-mediated force transfer. Most resistant to longitudinal splitting. The crack-spacing-derived relative bond stress follows the same qualitative deterioration pattern but is delayed in mass loss terms relative to Series C7.

Implications for structural assessment and model development

The verified crack-spacing data provide quantitative member-level indicators that can support evaluation and future calibration of tension-stiffening and corrosion-damage models. Their particular value is that they arise from direct-tension members rather than pullout specimens, thereby preserving distributed transverse cracking and the progressive loss of concrete contribution between cracks. The quantity $u/u_0 = S_0/S$ is therefore reported as a descriptive, crack-spacing-derived relative bond stress and not as a direct local bond measurement, design resistance factor, or universally transferable predictive relationship.

For Series C7, the re-audited data give $u/u_0 = 0.83$ at 8% mass loss, decreasing to 0.17 at 24% mass loss. At 27% mass loss, no new transverse crack formed before yielding and u/u_0 is therefore not calculable. For Series C3, the index decreases to approximately 0.23 at 14.5% mass loss.

These trends demonstrate substantial degradation of bond-mediated force transfer but should not be converted directly into rehabilitation trigger criteria without independent calibration. The measured progression for all three concrete series is summarized in Fig. 14.

Symbols represent individual specimens, and lines connect observations within each series to illustrate the measured response progression; no regression or predictive relationship is implied. C7-9C is identified as a terminal response but is not assigned a numerical u/u_0 because no new transverse crack formed before steel yielding.

The data in Fig. 14 do not define universal rehabilitation thresholds. Although the original program was designed with two specimens per nominal target corrosion condition, the achieved corrosion states were generally unique rather than replicated at identical measured mass-loss levels. The response also depends on concrete tensile strength, corrosion localization, crack stabilization, confinement, and exposure history. The results are therefore best used as mechanistic evidence and for external model assessment rather than as statistically calibrated rehabilitation criteria.

For the Sundance HVFA concrete, the absence of longitudinal splitting should not be interpreted as evidence that member-level steel-concrete interaction was preserved. Localized corrosion and spalling were observed, while the crack-spacing and load-elongation responses still changed with corrosion. These observations support distinguishing resistance to corrosion initiation from post-initiation structural response. However, the proposed mechanism requires confirmation through replicated tests and spatially resolved corrosion measurements.

No design resistance factor or regression equation is recommended from the present dataset. Future calibration should include replicate specimens at matched measured corrosion levels, independent bond measurements, uncertainty quantification, and validation against naturally corroded members. These limitations are also relevant when applying

recent bond-strength and development-length models for corroded reinforcement.³⁴

Contemporary interpretation: from corrosion metrics to loss of composite action

The re-audited results suggest that the most useful contemporary interpretation is not a single mass-loss-to-bond equation but a progression in member response. At low corrosion, transverse cracking remains distributed, and tension stiffening is largely retained. The slight Stage-1 increase in first-cracking load is attributed primarily to the absence of shrinkage pre-strain in the continuously immersed corroded specimens rather than to a demonstrated increase in bond. Separately, the C4 crack-spacing index exceeds unity at 0.9% ML; this may reflect low-corrosion confinement or interfacial effects, but because crack spacing is also sensitive to concrete tensile response and crack stabilization, it is treated as an apparent rather than independently measured bond enhancement.

With increasing damage, fewer new transverse cracks form, average crack spacing increases, and the load-elongation response approaches that of the bare or equivalently corroded bar. At advanced deterioration, severe rib degradation, localized pitting, longitudinal splitting or local spalling, and reduced steel area act together; in the limiting C7-9C specimen, no new transverse crack formed before steel yielding. This sequence is consistent with recent direct-tension observations of corrosion-induced loss of tension stiffening.⁴

Average gravimetric mass loss should not be treated as a unique structural-damage variable. Mass loss is a useful global index, but specimens with similar average loss can differ in pit severity, rib degradation, corrosion cracking, confinement, and spatial distribution of corrosion products. The archived observations of deep localized pits and partially intact adjacent ribs indicate that global mass loss and local force-transfer capacity are complementary rather than equivalent descriptors. This heterogeneity also explains why specimens exposed under nominally similar accelerated-corrosion conditions need not achieve identical measured corrosion states.

For bridge assessment, the practical implication is therefore multi-indicator interpretation. Chloride content and half-cell or corrosion-rate measurements describe exposure and electrochemical state, while crack morphology, crack spacing, tension-stiffening response, and localized section loss provide information on structural consequence. The Dickson Bridge comparison illustrates both the value and the limitation of linking these domains: field chloride levels and mass loss show qualitative consistency with the laboratory trend, but spatial variability and post-decommissioning chloride redistribution preclude a one-to-one conversion. This interpretation aligns with current recommendations to integrate laboratory and in-situ observations when assessing chloride-induced corrosion.³⁵

Fig. 15 synthesizes the verified specimen-level data as a descriptive deterioration-response map. It does not define calibrated thresholds. Instead, it places the measured

$u/u_0 = S_0/S$ observations in the context of the progression from effective force transfer through increasing transfer length and reduced tension stiffening to the terminal C7-9C response, for which u/u_0 is not calculable because no new transverse crack formed before yielding. The regions are descriptive and do not represent calibrated deterioration or rehabilitation thresholds.

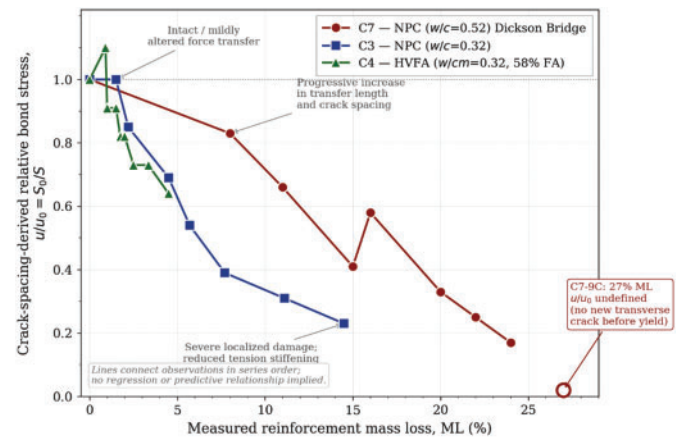


Figure 15. Descriptive deterioration-response map based on the verified direct-tension observations

The ordinate for Fig. 15 is the crack-spacing-derived relative bond stress $u/u_0 = S_0/S$. Region labels summarize the observed progression in force-transfer response; they are interpretive descriptors rather than statistically calibrated thresholds. C7-9C is shown as a terminal response but is not assigned a numerical u/u_0 value because no new transverse crack formed before steel yielding. The verified u/u_0 dataset can therefore serve as an external assessment benchmark rather than a calibrated design relationship. Section 5.5 applies this use by evaluating whether a contemporary direct-tension bond-reduction model reproduces the observed crack-spacing-derived relative bond-stress trends across the three concrete systems.

Comparison with a contemporary direct-tension bond-reduction model

Kunawisarut et al.¹ provide a particularly relevant external comparison because their experimental program also used single-bar reinforced-concrete members subjected to direct tension. Their bond formulation was derived through force equilibrium using average tensile-crack spacing, and the residual bond capacity was normalized to the corresponding uncorroded condition:

$$R_K = \tau/\tau_{\text{exp}} \exp(-0.054 ML) \quad (2)$$

where R_K is the residual bond ratio predicted by Kunawisarut et al. and ML is the average reinforcement mass loss expressed as a percentage. The published relationship was fitted after excluding the low-corrosion bond-enhancement regime and was proposed for corroded reinforced-concrete members subjected to direct tension. Kunawisarut et al. also observed that direct-tension members generally exhibit less bond degradation than pullout

specimens, emphasizing the importance of test configuration when transferring corrosion–bond models.

The normalized Kunawisarut relationship is mechanically comparable to the present within-series quantity $u/u_0 = S_0/S$ although neither quantity represents a direct local bond-stress measurement.

Geometric comparability. One geometric difference between the two experimental programs warrants consideration. The Kunawisarut et al. specimens had a 150×150 mm square cross-section with a reinforcing bar approximately 19 mm in diameter, whereas the present specimens were 120 mm diameter cylinders with a centrally positioned 20M bar ($d_b = 19.5$ mm). The corresponding geometric factor $A_c/(\pi d_b)$, which enters the force-equilibrium expression for average bond stress, is approximately 377 mm for the Kunawisarut specimens and 185 mm for the present specimens.

This difference can influence absolute crack spacing and inferred bond stress. However, $u/u_0 = S_0/S$ is normalized within each concrete series, so the fixed geometric terms cancel to first order. Geometry therefore does not preclude comparison of normalized deterioration trends, although differences in section geometry, concrete tensile response, reinforcement characteristics, and crack development remain relevant. The comparison is consequently treated as an external trend assessment rather than a demonstration of quantitative equivalence.

Model comparison. Eq. (2) was evaluated against the 22 corroded specimens for which a finite stabilized transverse-crack spacing, and hence u/u_0 , could be calculated. The six uncorroded controls define the within-series reference condition $u/u_0 = 1.0$ but were not included in the error calculations. C7-9C was excluded because no new transverse crack formed before steel yielding, and C3-7C was excluded because its test was not completed. For each concrete series, the mean absolute error was calculated as

$$MAE = \frac{1}{n} \sum_{i=1}^n \left| \left(\frac{u}{u_0} \right)_{obs,i} - \left(\frac{u}{u_0} \right)_{pred,i} \right|,$$

and the mean signed error was calculated using the observed-minus-predicted residuals. Each calculable corroded specimen received equal weight; the errors were not weighted by mass loss.

For Series C7 over the 8–24% ML range, $MAE = 0.093$ and the mean signed error was $+0.035$. For Series C3 over the 1.5–14.5% ML range, $MAE = 0.163$ and the mean signed error was -0.141 . For Series C4 over the 0.9–4.5% ML range, $MAE = 0.095$ and the mean signed error was -0.058 . All eight calculable corroded C4 observations were included in the external-model error assessment. Except for the 0.9% ML observation, which exhibits apparent low-corrosion enhancement, all C4 observations lie below the published relationship. The specimen observations and model comparisons are presented in Fig. 16.

Symbols represent individual corroded specimens, the dashed curve represents the published Kunawisarut relationship, and the solid curve represents the descriptive exponential fit to the present observations. C7-9C is excluded because u/u_0 is undefined.

Descriptive exponential comparison. In addition to the published model, Fig. 16 presents a one-parameter descriptive exponential fit for each series:

$$\frac{u}{u_0} = \exp(-a ML). \quad (3)$$

The coefficient a was obtained by unweighted nonlinear least-squares minimization of residuals in the original u/u_0 domain, with the uncorroded intercept fixed at $u/u_0 = 1.0$. The C4-10C observation at 0.9% ML and $u/u_0 = 1.10$ is displayed in Fig. 16 but was excluded from the C4 descriptive fit because it represents the low-corrosion enhancement regime. It was retained in the external-model error calculation because that calculation evaluates model performance over the complete observed range.

The fitted coefficients are $a = 0.050, 0.100$, and 0.100% ML^{-1} for Series C7, C3, and C4, respectively, compared with $a_K = 0.054\%$ ML^{-1} for the Kunawisarut et al. relationship. The coefficients and their ratios to a_K are summarized in Fig. 17.

The dashed horizontal line represents the Kunawisarut et al.¹ reference coefficient, $a_K = 0.054\%$ ML^{-1} . The C4 low-corrosion enhancement observation at 0.9% ML was excluded from the descriptive coefficient fit.

The ratios a/a_K are approximately 0.93, 1.85, and 1.86 for Series C7, C3, and C4, respectively. These ratios describe differences in fitted curve shape and should not be interpreted as indicating that one concrete deteriorates a corresponding number of times faster. The three series cover substantially different measured mass-loss ranges; therefore, the fitted coefficients are descriptive comparison metrics rather than calibrated material constants or statistically validated deterioration rates.

The nearly identical descriptive coefficients for Series C3 and C4 indicate that the deviation from the published relationship cannot be attributed to fly ash alone. Both series have a lower w/cm than Series C7, and their shared concrete-quality characteristics may contribute to the observed similarity. However, the present dataset does not isolate the governing material parameter.

The Series C4 observations remain scientifically notable. At approximately matched low corrosion levels, the crack-spacing-derived relative bond stress for C4 tends to be slightly lower than that of C3. For example, $u/u_0 = 0.91$ for C4 compared with 1.00 for C3 at 1.5% ML, and 0.64 compared with 0.69 at 4.5% ML. This difference occurs even though the C4 specimens developed no visible corrosion-induced longitudinal splitting cracks.

The C4 response indicates that the absence of longitudinal splitting is not sufficient evidence that member-level bond-mediated force transfer has been preserved. Localized rib damage, local spalling, concrete tensile properties, corrosion-product localization, and the substantially lower chloride contents measured at the reinforcement level in Series C4 may all have influenced the observed response. The present program cannot separate their individual effects.

Uncertainty and scope. No formal confidence bands or coefficient confidence intervals are reported. Although the

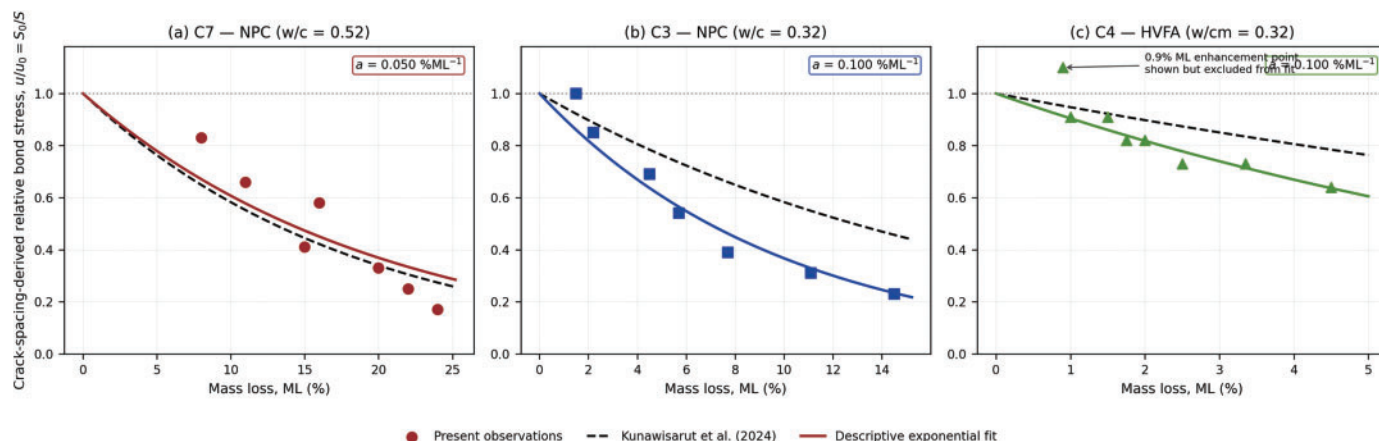


Figure 16. Comparison of the Kunawisarut et al.¹ direct-tension bond-reduction relationship, $R_K = \exp(-0.054ML)$, with the present crack-spacing-derived relative bond stress, $u/u_0 = S_0/S$: (a) Series C7, NPC ($w/c = 0.52$); (b) Series C3, NPC ($w/c = 0.32$); and (c) Series C4, HVFA ($w/cm = 0.32$)

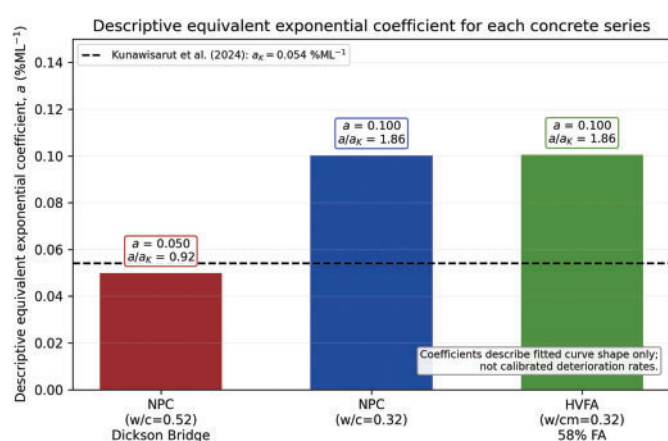


Figure 17. Descriptive equivalent exponential coefficient a obtained from $u/u_0 = \exp(-a ML)$ for the three present concrete series

program was designed around nominal target corrosion conditions, the specimens generally reached different measured mass-loss levels. Treating observations from adjacent corrosion stages as statistical replicates would combine actual corrosion progression with specimen-to-specimen variability and could produce a misleading uncertainty estimate.

The control pairs exhibited essentially identical stabilized crack spacings within each series, but they do not quantify uncertainty under corroded conditions. The present analysis is therefore an external model assessment using individual observations and descriptive error metrics rather than a statistical validation. Replicated tests at matched measured corrosion levels are required to quantify predictive uncertainty.

Conclusions

1. Thirty direct-tension specimens from three concrete systems were examined using the archived experimental program. The re-audited specimen geometry is

1000 mm concrete length and 120 mm cylindrical diameter, with a centrally placed 20M reinforcing bar ($d_b = 19.5$ mm), approximately 50 mm radial concrete cover ($c/d_b \approx 2.58$), and a 1010 mm extensometer gauge length.

2. Tension stiffening decreased progressively with corrosion in all three concrete systems. At advanced corrosion levels, the load-elongation response approached that of the corroded steel bar, indicating limited concrete contribution.
3. Transverse-crack spacing increased with corrosion. The revised analysis retains the original relation $u/u_0 = S_0/S$ and explicitly defines u/u_0 as a crack-spacing-derived relative bond stress rather than a direct measurement of local bond capacity. For C7-9C, u/u_0 is not calculable because no new transverse crack formed before steel yielding.
4. The Sundance HVFA specimens did not develop corrosion-induced longitudinal splitting cracks, although two specimens experienced highly localized corrosion and local spalling. A permeability-related localization mechanism is presented only as a working hypothesis.
5. The verified archived data show $u/u_0 = 0.83$ for Series C7 at 8% mass loss and 0.17 at 24% mass loss; approximately 0.23 for Series C3 at 14.5% mass loss; and approximately 0.64 for Series C4 at 4.5% mass loss. These values are descriptive trends from a dataset whose original paired target-condition design yielded generally unique achieved mass-loss states rather than replicated observations at identical measured corrosion levels.
6. Series C7 laboratory results show qualitative consistency with Dickson Bridge field observations for moderate mass loss, but the comparison does not constitute full validation of accelerated corrosion. Field chloride and mass-loss data display substantial spatial and exposure-history variability.

7. The study is a contemporary re-analysis of a legacy experimental dataset rather than a new experimental campaign. Its contribution is the verified specimen-level consolidation of the direct-tension records, reinterpretation of crack spacing as a member-level bond-transfer indicator, external assessment against a contemporary direct-tension bond-reduction relation, explicit comparison with current high-corrosion tension-stiffening evidence, and integration of laboratory response with Dickson Bridge field observations.
8. Comparison with Kunawisarut et al.¹ shows that the published exponential direct-tension bond-reduction relation captures the average C7 trend reasonably well but generally overpredicts retained crack-spacing-derived bond response for the lower-w/cm C3 and C4 series. Using descriptive one-parameter exponential fits and excluding the C4 low-corrosion enhancement point from the C4 coefficient fit, the equivalent coefficients are approximately 0.050, 0.100, and 0.100 %ML⁻¹ for C7, C3, and C4, respectively, compared with 0.054 %ML⁻¹ in the published relation (the slight difference between the C3 (1.85) and C4 (1.86) ratios reflects fitting precision before rounding to 0.100.). Because C3 and C4 show nearly identical descriptive coefficients and the three series span different corrosion ranges, the deviation cannot be attributed to fly ash alone, and the coefficients should not be treated as calibrated material deterioration rates. The C4 observations are nevertheless notable because reduced bond-mediated force transfer occurred despite the absence of visible corrosion-induced longitudinal splitting cracks.
9. The absence of reconstructable specimen-specific cumulative-charge histories prevents retrospective comparison with Faraday-law estimates. Future studies should retain time-resolved electrochemical records and include true replicates at matched measured corrosion levels. For bridge assessment, the findings support a multi-indicator approach combining chloride exposure, electrochemical condition, crack morphology and spacing, tension-stiffening response, and localized section loss rather than relying on average mass loss alone. The six-stage framework is therefore retained as a qualitative interpretation of the progression from distributed cracking and tension stiffening to severely impaired composite action, not as a calibrated deterioration law.

Overall, the six-stage framework is retained as a qualitative mechanistic interpretation of the observed progression from distributed cracking and tension stiffening toward severely impaired composite action. It is not proposed as a calibrated deterioration law. The legacy dataset is most valuable when used transparently alongside contemporary replicated experiments and field observations.

Declaration of Competing Interest

The author declares no competing financial interests or personal relationships that could have influenced the work reported in this paper.

Data Availability

The complete experimental dataset (load-elongation curves, crack width measurements, crack spacing data, chloride profiles, and mass loss values for all 30 specimens) is available from the corresponding author upon reasonable request.

References

- [1] Kunawisarut A, Iwanami M, Chijiwa N, Nakayama K. Assessment of bond deterioration in corroded RC members incorporating cracking response and tension stiffening. *Struct Infrastruct Eng.* 2024;20(4):581–593. doi:10.1080/15732479.2022.2131843.
- [2] Imperatore S. Modelling strategies for reinforced concrete elements under corrosion degradation. *Materials.* 2022;15(13):4601. doi:10.3390/ma15134601.
- [3] Syll AS, Kanakubo T. Impact of corrosion on the bond strength between concrete and rebar: a systematic review. *Materials.* 2022;15(19):7016. doi:10.3390/ma15197016.
- [4] Ahmadi E, Aminulai H, Kashani MM. Influence of corrosion on nonlinear tension stiffening of concrete under monotonic and cyclic loading. *Eng Struct.* 2025;337:120530. doi:10.1016/j.engstruct.2025.120530.
- [5] Amleh L. *Bond Deterioration of Reinforcing Steel in Concrete due to Corrosion*. Ph.D. Thesis, McGill University, Montreal, Canada. [Pullout test programme and design equations: Chapter 8 and Chapter 10]; 2000.
- [6] Amleh L, Mirza MS. Corrosion influence on bond between steel and concrete. *ACI Struct J.* 1999;96(3):415–423. doi:10.14359/676.
- [7] Al-Sulaimani GJ, Kaleemullah M, Basunbul A, Rasheeduz-zafar. Influence of corrosion and cracking on bond behavior and strength of reinforced concrete members. *ACI Struct J.* 1990;87(2):220–231. doi:10.14359/2732.
- [8] Fang C, Lundgren K, Chen L, Zhu C. Corrosion influence on bond in reinforced concrete. *Cement Concr Res.* 2004;34(11):2159–2167. doi:10.1016/j.cemconres.2004.04.006.
- [9] Tepfers R. Cracking of concrete cover along anchored deformed reinforcing bars. *Mag Concr Res.* 1979;31(106):3–12. doi:10.1680/mac.1979.31.106.3.
- [10] fib. *fib Model Code for Concrete Structures 2010*. Berlin: Ernst & Sohn; 2013.
- [11] Bertolini L, Elsener B, Pedferri P, Redaelli E, Polder RB. *Corrosion of Steel in Concrete: Prevention, Diagnosis, Repair*. 2nd ed. Weinheim: Wiley-VCH; 2013.
- [12] Wong HS, Zhao YX, Karimi AR, Buenfeld NR, Jin WL. On the penetration of corrosion products from reinforcing steel into concrete due to chloride-induced corrosion. *Corros Sci.* 2010;52(7):2469–2480. doi:10.1016/j.corsci.2010.03.025.
- [13] Goto Y. Cracks formed in concrete around deformed tension bars. *ACI J.* 1971;68(4):244–251. doi:10.14359/11325.
- [14] Houde J, Mirza MS. A finite element analysis of shear strength of reinforced concrete beams. In: *Shear in Reinforced*

- Concrete, *ACI SP-42*. Detroit, MI, USA: American Concrete Institute; 1974;1:103–128.
- [15] Amleh L, Ghosh A. Modeling the effect of corrosion on bond strength at the steel–concrete interface with finite-element analysis. *Can J Civil Eng*. 2006;33(6):673–682. doi:10.1139/106-052.
- [16] Hussein L, Amleh L. Analytical modelling of bond stress at steel concrete interface due to corrosion. *Struct Concr*. 2016;17(4):541–552. doi:10.1002/suco.201500109.
- [17] Ying Z, Wu S, Du C, Li J, Zhao Z, Zhao C. Bond slip of the corroded rebar and its effect on the load-bearing capacity of a flexural beam. *Structures*. 2025;74:108597. doi:10.1016/j.istruc.2025.108597.
- [18] Apostolopoulos C, Koulouris K. Corrosion effect on bond loss between steel and concrete. In: *Structural Integrity and Failure*. London, UK: IntechOpen; 2020. doi:10.5772/intechopen.94166.
- [19] Angst UM, Isgor OB, Hansson CM, Sagüés A, Geiker MR. Beyond the chloride threshold concept for predicting corrosion of steel in concrete. *Appl Phys Rev*. 2022;9(1):011321. doi:10.1063/5.0076320.
- [20] Kaklauskas G, Tamulenas V, Gribniak V, Ng PL, Kupliauskas R. Tension-stiffening behaviour of reinforced concrete ties of various strength classes. *Proceedings of the Second International Conference on Performance-based and Life-cycle Structural Engineering*; Dec 9–11, 2015; Brisbane, Australia. p. 582–590.
- [21] Bidari O, Singh BK, Maheshwari R. Effect of corrosion on bond between reinforcement and concrete—an experimental study. *Discov Civil Eng*. 2024;1(1):67. doi:10.1007/s44290-024-00061-3.
- [22] Zhao Y, Lin H. The bond behaviour between concrete and corroded reinforcement: state of the art. *Proceedings of the 6th International Conference on Durability of Concrete Structures (ICDCS 2018)*; Jul 18–20, 2018; Leeds, UK (University of Leeds). p. 63–73.
- [23] Cui Y, Qu S, Gao K, Tekle BH, Bao J, Zhang P. Effect of corrosion on the bond behavior of steel-reinforced, alkali-activated slag concrete. *Materials*. 2023;16(6):2262. doi:10.3390/ma16062262.
- [24] Bilodeau A, Malhotra VM. Concrete incorporating high volumes of ASTM Class F fly ash: mechanical properties and resistance to deicing salt scaling and to chloride-ion penetration. In: Malhotra VM. (Ed.), *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete, ACI SP-132*. Farmington Hills, MI: American Concrete Institute; 1992:319–349.
- [25] Molina FJ, Alonso C, Andrade C. Cover cracking as a function of rebar corrosion: Part II—numerical model. *Materials Struct*. 1993;26(9):532–548. doi:10.1007/bf02472864.
- [26] Bischoff PH. Influence of shrinkage on tension stiffening of concrete. *ACI Struct J*. 1995;92(2):249–257.
- [27] Syll AS, Shimokobe H, Kanakubo T. Effect of local bond behavior degradation on tension stiffness in reinforced concrete with pre-existing longitudinal cracks. *Case Stud Construct Mater*. 2024;20:e03017. doi:10.1016/j.cscm.2024.e03017.
- [28] Han S-J, Lee D. Bond strength and development length model for corroded reinforcing bars. *J Earthq Eng Soc Korea*. 2024;28(5):267–274.
- [29] Ali M, Shams MA, Bheel N, et al. A review on chloride induced corrosion in reinforced concrete structures: lab and in situ investigation. *RSC Adv*. 2024;14(50):37252–37271. doi:10.1039/D4RA05506C.
- [30] Abd El-Fattah H, Abd El-Zaher Y, Kohail M. A study of chloride binding capacity of concrete containing supplementary cementitious materials. *Sci Rep*. 2024;14(1):12970. doi:10.1038/s41598-024-62778-6.
- [31] Amleh L, Mirza MS. Corrosion Deterioration of Dickson Bridge. Departmental Report, McGill University, Montreal; 2000.
- [32] Amleh L, Mirza MS. Corrosion response of a decommissioned deteriorated bridge deck. *J Perform Constr Facil*. 2004;18(4):185–191. doi:10.1061/(ASCE)0887-3828(2004)18:4(185).
- [33] Andrade C, Alonso C, Molina FJ. Cover cracking as a function of bar corrosion: Part I—experimental test. *Mater Struct*. 1993;26(8):453–464. doi:10.1007/BF02472805.
- [34] ASTM International. *ASTM C1152/C1152M-90: Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*. Conshohocken, PA: ASTM International; 1990.
- [35] Lothenbach B, Scrivener K, Hooton RD. Supplementary cementitious materials. *Cement Concr Res*. 2011;41(12):1244–1256.

Supplementary Table S1. Complete specimen-by-specimen archival dataset used in the revised analysis, provided in the online supplementary material. C3-7C was not completed because of an MTS malfunction; unavailable quantities are reported as N/A

Series	Specimen	Stage	Mass loss (%)	Major long. cracks	Avg long. width (mm)	1st crack load (kN)	Yield load (kN)	No. trans. cracks	Avg spacing (mm)	Cross-sec. loss (%)	Cl at steel (%)	u/u0
C7	C7-1C	0	0	0	0	18	127	11	83	0		1.00
C7	C7-2C	0	0	0	0	19	127	11	83	0		1.00
C7	C7-4C	1	8	1	1.2	23	116	9	100	8.7	0.37	0.83
C7	C7-5C	2	11	1	1.5	24	102.5	7	125	19.3	0.48	0.66
C7	C7-8C	3	16	1	1.2	23	96	6	143	24.4	0.72	0.58
C7	C7-3C	4	15	2	1.3	17	87.5	4	200	31	0.94	0.41
C7	C7-7C	5	20	1	3	28	83	3	250	34.6	1.16	0.33
C7	C7-10C	6	22	2	4	59	81.5	2	333	35.8	1.12	0.25
C7	C7-6C	7	24	3	2	63	70	1	500	44.9	1.44	0.17

Supplementary Table S1. (Continued)

Series	Specimen	Stage	Mass loss (%)	Major long. cracks	Avg long. width (mm)	1st crack load (kN)	Yield load (kN)	No. trans. cracks	Avg spacing (mm)	Cross-sec. loss (%)	Cl at steel (%)	u/u0
C7	C7-9C	8	27	2	4	N/A	45	0	N/A	64.6	1.52	N/A
C3	C3-1C	0	0	0	0	19	127	12	77	0		1.00
C3	C3-2C	0	0	0	0	19	127	12	77	0		1.00
C3	C3-5C	1	1.5	0	0	34	125	12	77	1.6	0.04	1.00
C3	C3-8C	2	2.2	2	0.08	48	124	10	91	2.4	0.11	0.85
C3	C3-10C	3	4.5	2	0.35	35	121	8	111	4.7	0.19	0.69
C3	C3-9C	4	5.7	3	1.5	30	122	6	143	3.9	0.28	0.54
C3	C3-6C	5	7.7	3	2	31	115	4	200	9.4	0.29	0.39
C3	C3-3C	6	11.1	2	3	48	93	3	250	26.8	0.35	0.31
C3	C3-4C	7	14.5	3	3	37	83	2	333	34.6	0.46	0.23
C3	C3-7C	N/A*	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
C4	C4-1C	0	0	0	0	22	126	10	91	0		1.00
C4	C4-2C	0	0	0	0	21	128	10	91	0		1.00
C4	C4-10C	1	0.9	0	0	31	127	11	83	0	0.002	1.10
C4	C4-6C	2	1.0	0	0	29	124	9	100	2.4	0.003	0.91
C4	C4-9C	3	1.5	0	0	27	126	9	100	0.8	0.013	0.91
C4	C4-5C	4	1.75	0	0	27	125	8	111	1.6	0.004	0.82
C4	C4-3C	5	2.0	0	0	32	124	8	111	2.4	0.02	0.82
C4	C4-4C	6	2.5	0	0	36	124.5	7	125	2.0	0.01	0.73
C4	C4-7C	7	3.35	0	0	22	110	7	125	13.4	0	0.73
C4	C4-8C	8	4.5	0	0	30	90	6	143	29.1	0.006	0.64