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Durability, Service Life, and Long-Term
Integrity of Concrete Materials, Bridges,
and Structures

SP-351

Editors:

Yail J. Kim, Chris P. Pantelides, and Xianming Shi



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Durability, Service Life, and Long-Term Integrity of Concrete Materials, Bridges, and Structures

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Durability, Service Life, and Long-Term Integrity of Concrete Materials, Bridges, and Structures

Durability is one of the most important requirements for built-environments. Federal, state, and local agencies expend significant effort to maintain the quality and condition of aging civil infrastructure, especially in aggressive service environments. Among many factors, durability influences the service life, integrity, and reliability of concrete materials and structures. Extensive research has been conducted to understand the deterioration mechanisms of concrete in an effort to extend the longevity of concrete members. This Special Publication (SP) contains nine papers selected from three technical sessions held during the virtual ACI Fall Convention in October 2021. Emphasis is placed on durable reinforcing schemes, service life prediction, structural integrity, repair and retrofit, corrosion mitigation, inspection techniques, and the application of state-of-the-art construction materials. All manuscripts were reviewed by at least two experts in accordance with the ACI publication policy. The Editors wish to thank all contributing authors and anonymous reviewers for their rigorous efforts. The Editors also gratefully acknowledge Ms. Barbara Coleman at ACI for her knowledgeable guidance.

Yail J. Kim, Chris P. Pantelides, and Xianming Shi

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Axial Compression Capacity of Concrete Columns Reinforced with GFRP and Stainless Reinforcement

J.W. Wright and C.P. Pantelides

Synopsis: Axial compression performance of concrete columns reinforced with GFRP bars and spiral, 2304 duplex stainless bars and spiral, and 316L stainless clad bars, in varying combinations is examined after exposure to accelerated corrosion. The hybrid columns were reinforced with a combination of metallic and GFRP reinforcement. After corrosion exposure the columns were tested under axial compression to failure. Columns with GFRP vertical bars and stainless steel spiral were less corrosion resistant and had smaller axial load capacity than hybrid columns with stainless clad or stainless steel vertical bars and GFRP spiral. Columns reinforced with stainless steel spiral achieving two to three times the maximum axial displacement of columns with GFRP spiral. Axial compression capacity of hybrid columns in both corroded and uncorroded conditions was modeled using concrete confinement models for metallic and GFRP reinforcement with good agreement.

Keywords: carbon steel, columns, concrete, corrosion, glass fiber reinforced polymer, stainless clad, stainless steel.

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INTRODUCTION

Reinforced concrete structures are susceptible to corrosion-induced damage; exposure to de-icing salts or saltwater causes chlorides to penetrate concrete and accelerate corrosion of the reinforcing steel. Corrosion-resistant reinforcement includes stainless steel, stainless clad reinforcement with a carbon steel core and stainless cladding, and glass fiber reinforced polymer composite (GFRP) bars. The upfront material cost of corrosion-resistant reinforcement is greater than carbon steel, but increase in service life and decrease in maintenance and repair costs could offset the initial investment. Duplex stainless steel with a dual-phase austenitic and ferritic microstructure, and reinforcement with a carbon steel core and outer austenitic stainless steel cladding have the potential to resist corrosion much longer than carbon steel or epoxy coated bars [1]. An economical option for reinforcement is 2304 stainless steel (alloy including 23% chromium and 4% nickel); this material has higher corrosion resistance than carbon steel and lower cost than alternative stainless and stainless clad options [2]. Stainless clad reinforcement performs similarly to solid stainless, with 0.25 mm to 0.80 mm thick 316L austenitic stainless cladding providing the best performance [3].

GFRP composites do not experience electrochemical corrosion although other environmental factors such as moisture, temperature, pressure and acidity can affect bar tensile strength or degrade the resin matrix. GFRP bars encased in concrete are projected to retain over 70% of their tensile capacity, even after continual submersion in tap and salt water for a 100-year service life [4]. Chloride-driven degradation in GFRP bars has been explored and found to be insignificant [5]. GFRP reinforced concrete columns immersed in distilled water and chloride saturated water had superior retention of long term capacity compared to carbon steel [6]. Hybrid columns with carbon steel vertical bars and GFRP spiral had favorable corrosion behavior compared to all-carbon steel columns [7]. GFRP spiral in confinement exhibits different behavior than steel due to its comparatively low modulus of elasticity; analytical models can predict its performance when used for concrete confinement [8].

This research compares the performance of corrosion-resistant materials in an accelerated corrosion test and mechanically, using medium scale column specimens containing various combinations of GFRP and 2304 stainless-steel bars and spiral, and 316L stainless clad vertical bars. A set of columns using GFRP vertical bars and GFRP spiral was also tested for comparison. The columns were subjected to 60 days of accelerated corrosion. Qualitative effects of corrosion such as spalling and cracking were compared. Theoretical mass loss was calculated using Faraday's law based on recorded current. The columns were tested under axial compression to failure, and their axial compressive strength and displacement capacity were compared; in addition, these tests allowed examination of the degree of corrosion of the reinforcement. The axial compressive capacity of corroded and uncorroded columns was determined using theoretical models developed by Mander et al. [9] and Hales et al. [8]; the axial capacity obtained in the experiments was compared to the theoretical axial capacity.

The primary objective of this study was to investigate how 2304 stainless steel, 316L stainless clad bars and GFRP bars and spiral are affected by corrosion. The effect of corrosion on the axial compression capacity of medium-scale columns in axial compression when reinforced with 2304 stainless steel and GFRP bars and spiral, and 316L stainless clad bars in varying combinations was a second objective. Axial compression capacity of the columns in both the corroded and uncorroded conditions was also modeled using concrete confinement models for metallic and GFRP reinforcement.

RESEARCH SIGNIFICANCE

Research on concrete columns reinforced with a combination of metallic and non-metallic corrosion-resistant reinforcement is rare. In this research, corrosion performance and axial compression capacity of medium-scale concrete columns reinforced with combinations of corrosion-resistant reinforcement was investigated. The materials studied include 316L stainless clad bars, 2304 duplex stainless steel bars and GFRP bars and spirals. The effect of corrosion on the axial compression capacity of medium-scale columns in axial compression when reinforced with 2304 stainless steel and GFRP bars and spiral, and 316L stainless clad bars in varying combinations is examined. Axial compression capacity of the columns in both the corroded and uncorroded conditions is predicted using concrete confinement models for metallic and GFRP reinforcement.

MATERIAL SELECTION AND SPECIMEN DESIGN

The medium-scale columns had a 305 mm (12 in.) diameter and were 813 mm (32 in.) tall. All columns were cast with pea-gravel concrete and cured for 28 days. Concrete test cylinders 102 mm (4 in.) by 203 mm (8 in.) were tested to determine the compressive strength of the concrete. The 28-day concrete compressive strength was 29 MPa (4.2 ksi) which increased to 45 MPa (6.5 ksi) at the time of column testing. The properties of all materials except GFRP were measured using experimental methods in this research. GFRP bars and spiral had an ultimate tensile strength of 725 MPa (105 ksi), ultimate tensile strain of 1.57% elastic modulus of 46 GPa (6.7 Msi) as specified by the manufacturer; the compressive strength and strain of the GFRP bars was measured indirectly by averaging strain gauge readings from the vertical GFRP bars in the columns as 356 MPa (52 ksi) and 0.77%, respectively. Regarding 2304 stainless steel bars and spiral the yield strength was 660 MPa (96 ksi) with a yield strain of 0.33%, and an ultimate tensile strength of 790 MPa (115 ksi) with an ultimate strain of 20.0% and elastic modulus of 200 GPa (29 Msi), as shown in Fig. 1. Finally, 316L stainless clad bars had a yield strength of 500 MPa (73 ksi) with a yield strain of 0.25%, and an ultimate tensile strength of 715 MPa (104 ksi) with an ultimate strain of 23.0% and elastic modulus of 200 GPa (29 Msi), as shown in Fig. 2.

The three materials were used in four different combinations: GFRP vertical bars with solid stainless spiral (GN series), solid stainless vertical bars with GFRP spiral (NG series), stainless clad vertical bars with GFRP spiral (CG series), and all-GFRP reinforcement (GG series). These combinations are given in Table 1 with the reinforcement dimensions. Figure 3 shows the dimensions of a typical column; these varied between columns using metallic and GFRP spirals. The lower elastic modulus of the GFRP spiral reduces confinement efficiency, so a 16 mm (5/8 in.) GFRP spiral was chosen to match the capacity of columns with a 13 mm (1/2 in.) metallic spiral. The four vertical bars had a 16 mm (5/8 in.) diameter; typical concrete cover ranged from 25 to 38 mm (1.0 to 1.5 in.) depending on the spiral diameter. A total of 15 columns were cast, 8 of which were subjected to current-accelerated corrosion (referred to here as *corrosion* or “R” specimens) whose reinforcement is shown in Fig. 4. The remaining 7 columns were subjected to room temperature conditions (referred to as *control* or “C” specimens). Each corroded specimen with metallic reinforcement had copper wire attached, as shown in Fig. 4, and was connected to a power supply during the corrosion period.

CURRENT-ACCELERATED CORROSION EXPERIMENTS

The technique used to accelerate corrosion was similar to the Florida Department of Transportation method [10,11] concrete samples were submerged in a saltwater solution and an electrical potential was applied to the metallic reinforcement. A 5% NaCl by weight saltwater solution was used, with the samples submerged up to approximately half the column specimen height. The rate of corrosion was standardized using current density, defined as the amount of induced current divided by the surface area of steel exposed to the chloride solution. A current density of 2.0 $\mu\text{A}/\text{mm}^2$ (1.3 $\text{mA}/\text{in.}^2$) produces significant acceleration in the rate of concrete strain-crack formation [11]. A moderate rate of 2.2 $\mu\text{A}/\text{mm}^2$ (1.4 $\text{mA}/\text{in.}^2$) was used, which was the total current in the system, for hybrid specimens combining metallic reinforcement with GFRP bars and spiral. The columns were submerged in saltwater solution for a period of two weeks prior to application of current to allow infiltration of the electrolyte to the rebar before current was applied. A galvanized steel grate was placed at the tank bottom to complete the circuit through the electrolyte. The target current density was maintained throughout the 60-day corrosion period by measuring the current three times a week and adjusting the power. The average applied current for each specimen series was maintained within 15% of the target. Voltage was also read from the power supply at this time, and a constant rate of corrosion was maintained in each specimen, with the measured current accounting for the potential applied by the power supply and the potential created by the internal corrosion cell of the corroding reinforcement. Using the average measured current, the theoretical mass loss was calculated using Faraday’s law as:

$$M_{th} = \frac{M_A I_{app} T}{vF} \quad (1)$$

where M_{th} = theoretical mass loss in grams, F = 96487 ampere-second or Faraday’s constant, v = 2.0, the valency of the corrosion product, T = total time of current application in seconds, and I_{app} = average applied current in amps. The atomic mass for the alloyed steel was a weighted average of individual components [12]. Using this method, M_A = 56.352 grams (0.125 lbs) for the 2304 duplex stainless steel, and M_A = 57.936 grams (0.128 lbs) for the 316L stainless clad bars. This method of calculation overestimates actual mass loss by a factor of 1.2; this was taken into consideration in finding the mass loss expressed in Table 2 as the adjusted theoretical mass loss [13].

Hybrid Column Corrosion Results

Throughout the corrosion period, the voltage fluctuated as the physical specimen conditions changed. Figure 5 illustrates the change in voltage over time. NG and CG series specimens with stainless and stainless clad vertical reinforcement respectively required 85% of the voltage as the GN series (with GFRP vertical reinforcement and stainless steel spiral) to reach the same rate of corrosion, even though the target current for the NG and CG series specimens was 40% that of the GN series. This means that hybrid specimens with GFRP spiral required greater voltage due to greater resistance in the system and greater corrosion resistance overall in terms of corrosion energy, due to the non-corrosive properties of the GFRP spiral. Pantelides et al. [7] found that when GFRP spiral is used in conjunction with carbon steel vertical bars, the vertical bars are protected from corrosion due to the fact that GFRP spiral does not corrode; this minimizes cracking of the concrete cover such that chlorides cannot penetrate the column as easily; thus, GFRP spiral increases the protective barrier to vertical steel bars which is sufficient to eliminate or significantly reduce corrosion of carbon steel bars.

Cracking of concrete was first observed in specimens GN-R-1 and GN-R-2 after 32 days with small amounts of iron oxide effluent deposited around the cracked concrete. The final extent of cracking for the two specimens is shown in Fig. 6. The remaining specimens with GFRP spiral and either stainless steel or stainless clad vertical reinforcement did not show obvious exterior cracking, although the voltage applied began to gradually drop starting at 37 days, after which minor cracking was observed although not to the same extent as the GN specimens. After the 60-day corrosion period, the majority of cracking and discoloration in hybrid specimens with metallic vertical bars was observed on the underside of the column; some specimens had cracks on the side, such as columns CG-R-2 and NG-R-2, shown in Fig. 7. No major spalling occurred in any of the specimens.

Faraday's law was used to determine the amount of mass loss. Referring to Table 2, the columns had a range of mass loss from 2.1% to 2.9%, which was expected since the sustained current density was low. After destructive testing in axial compression, examination of the GN series specimens confirmed that the majority of corrosion occurred only where cracking occurred, and was concentrated in areas of pitting. Figure 8(a) shows discoloration of stainless steel vertical bar and GFRP spiral of NG-R-2; corrosion occurred on both types of metallic vertical reinforcement, but pitting was not seen to the same extent as on the stainless steel spiral of the GN series. The GN series had corrosion damage on the outward-facing stainless steel spiral, as shown in Fig. 8(b), with localized pitting in the spiral of specimen GN-R-1. The effect of this was minimal on the performance of corroded columns since none of the spirals fractured; instead compressive failure of GFRP vertical bars occurred prior to overall column compressive failure. Within the CG series, the end caps used for stainless clad vertical bars did not corrode.

All-GFRP Column Corrosion Results

The GG series specimens reinforced with GFRP vertical bars and GFRP spiral represented a unique case. The GG series suffered little external discoloration and no cracking despite exposure to corrosion and dissolved corrosion product from metallic reinforcement from the remaining specimens. This was confirmed after destructive testing as the GFRP reinforcement in Fig. 9(b) showed no discoloration, in contrast to the GFRP spiral in the hybrid specimens which had stained as a result of cracking.

EXPERIMENTAL RESULTS OF AXIAL COMPRESSION EXPERIMENTS

The axial compressive strength of each column was compared for the corroded and control specimens. The critical displacement was measured at the point of failure, defined here as the point after peak load where the stress drops to 85% of the maximum. Displacement measurements were taken from a pair of external LVDT's between two L-brackets, attached to the column surface, as shown in Fig. 9(a). Tables 3 and 4 summarize the quantitative results and relative capacity of the control and corroded specimens, respectively. To ensure that specimens failed in the middle third of the column height during axial testing, a steel collar and one carbon FRP wrap was applied at the top and bottom 200 mm of the specimens.

Hybrid Column axial Compression Results

Figures 10 and 11 show the axial compression force versus displacement for the three hybrid series and the all-GFRP series. Series GN with stainless steel spiral and GFRP vertical bars had the highest axial displacement for the control and corroded specimens; the maximum displacement ranged from two to three times that of the remaining three series. Fig. 12 shows a comparison of NG series corroded and uncorroded specimens. Column NG-C-1 reached a maximum axial compression of 3,320 kN (746 kip) which was higher than the 3,007 kN (676 kip) peak reached by NG-C-2, indicating some variation in the performance of uncorroded columns. Although there was little overall mass loss,

corroded specimens from the NG series experienced a 4% axial compression capacity reduction compared to the control, with NG-R-1 reaching a maximum of 3,099 kN (697 kip) and NG-R-2 reaching 2,945 kN (662 kip). The amount of mass loss due to corrosion did not correlate with reduction in capacity; NG-R-1 experienced a mass loss of 4.4% compared to a mass loss of 4.3% for NG-R-2, which experienced a greater drop in axial load capacity and displacement. Moreover, only the metallic spiral hybrid GN specimens and uncorroded GG-C-1 showed progressive failure modes compared to the sudden failure of the remaining specimens.

The CG series specimens retained a higher percentage of axial compression capacity than the NG series, which suggests that stainless clad bars exhibited greater corrosion resistance than stainless steel; this combined with the lack of degradation in the GFRP spiral enabled corroded specimens in the CG series to show the highest overall compressive capacity of any of the corroded columns of 3,178 kN (714 kip) and 3,384 kN (761 kip), as shown in Table 4. Fig. 13 shows the force versus displacement for the CG series, indicating similar behavior to the NG series. For all hybrid columns with metallic vertical reinforcement and GFRP spiral, control and corroded specimens failed suddenly with rupture of the GFRP spiral, as was shown in Fig. 8(a) and Fig. 9(b), often accompanied with buckling of vertical bars, as shown in Fig. 9(c), where the GFRP spiral became ineffective in restraining bar buckling. The GFRP spiral fractured in at least one location for each of the NG and CG hybrid specimens and often in multiple locations, which indicates that the GFRP spiral is vulnerable to transverse stress concentrations. The GFRP spiral also experienced exterior staining, indicating that cover cracking allowed for infiltration of the electrolyte. Pantelides et al. [7] found that when GFRP spiral is used in conjunction with carbon steel vertical bars, the vertical bars are protected from corrosion due to the fact that GFRP spiral does not corrode, and this was also observed in this study. Conversely, when carbon steel spiral is used, rust deposition and bond stresses generate lateral stresses over the spiral bar surface which accelerate splitting of the cover as was observed by Tastani and Pantazopoulou [14].

The GN control columns had the smallest compressive capacity of any of the uncorroded specimens of 2,758 kN (620 kip) and 2,874 kN (646 kip) shown in Table 3, although they did exhibit much greater deformability than any of the specimens with GFRP spiral. Both uncorroded columns continued to maintain relatively high compressive load at displacements of 32 mm and 31 mm for GN-C-1 and GN-C-2, respectively, and did not experience sudden failure. The corroded GN specimens had more well-defined points at failure, an average axial compression capacity 2.6% lower than the uncorroded specimens and a 10.0% lower displacement capacity. Both specimens showed lower displacement, while GN-R-1 showed a 4.9% drop in axial capacity while GN-R-2 experienced a 0.4% drop. Fig. 14 shows the force versus displacement for the GN series, and indicates that greater displacements were reached by the corroded GN specimens compared to other series. This can be attributed to the higher strain capacity of stainless steel spiral compared to GFRP spiral. Table 3 indicates that all but two of the uncorroded specimens (GN-C-1 and GN-C-2) achieved a maximum compression greater than 3,000 kN (674 kip). Table 4 shows that all but two of the corroded specimens (GN-R-1 and GN-R-2) reached a maximum compression greater than 2,900 kN (652 kip). The exception in both cases is the GN series with GFRP vertical bars and stainless steel spiral. The GFRP spiral was less affected by corrosion and enabled columns to reach a satisfactory compression capacity; however, GFRP spiral exhibited a brittle failure mode and caused the specimens to reach much lower displacements prior to failure. For comparison, the compression capacity reduction of similar all-carbon steel column specimens ranged from 13.0% to 24.0% with a corresponding mass loss ranging from 16.0% to 18.0%, as reported in a previous study by Pantelides et al. [7]. The compression capacity reduction of hybrid specimens with all-carbon steel vertical bars and GFRP spiral was 5.0% with a corresponding mass loss of 7.0%, as reported in [7].

All-GFRP Specimen Axial Compression Results

The specimens reinforced with GFRP spiral and GFRP vertical bars exhibited a high maximum compression capacity compared to hybrid series, ranging from 3,126 kN (703 kip) for GG-R-1 for corroded specimens (Table 4) to 3,252 kN (731 kip) for GG-C-2 for control specimens (Table 3). All-GFRP specimens showed a lower overall axial displacement capacity, averaging 18 mm (0.71 in.) for the control specimens and 16 mm (0.63 in.) for the corroded specimens. Specimen GG-C-1 maintained some stiffness after the point of failure, but the other columns in the series failed suddenly, as illustrated in Fig. 15. The GFRP spiral fractured in multiple places while the GFRP vertical bars fractured in compression, as shown for specimen GG-R-2 in Fig. 9(b). Axial compression capacity of corroded specimens was 7.0% less and axial displacement capacity was 10.0% less compared to control specimens; GG series was the only set of columns that did not experience external cracking or visible staining on the reinforcement. These variations between corrosion and control specimens are consistent with D'Antino et al. [15] who suggest strength reductions of less than 8% for GFRP reinforcing bars exposed to saltwater for 60-days or less. Even excluding the potential degradation seen by the GG series specimens, Figs. 10, 11, and 15 indicate that on average, hybrid specimens

with either metallic vertical bars or metallic spiral exhibit greater ductility than specimens reinforced with all-GFRP reinforcement.

THEORETICAL AXIAL COMPRESSION CAPACITY OF COLUMN SPECIMENS AND COMPRESSION TO EXPERIMENTAL RESULTS

The axial compression capacity of concrete columns can be evaluated as the sum of the capacity of the longitudinal steel and confined concrete. The key factor in the analysis is the confined concrete compressive strength contribution from spiral reinforcement, since the spiral material and its state of corrosion have the greatest impact on overall column compression capacity.

Theoretical Models for Confinement of Concrete with Metallic and GFRP Spirals

For all-carbon steel reinforcement, the Mander et al. [9] model provides accurate estimates of confined concrete strength. It is assumed that the stainless steel spiral performs similar to carbon steel and the Mander et al. [9] model applies. The model is given as:

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad (2)$$

where f'_{co} is the unconfined concrete compressive strength; the confining pressure f'_l is defined as:

$$f'_l = \frac{2 f_{yh} A_{sp}}{D' s} k_e \quad (3)$$

where f_{yh} is the yield strength of the transverse reinforcement, A_{sp} is the area of the steel spiral, D' is the interior diameter of the spiral, s is the spiral pitch, and k_e , is the confinement effectiveness given as:

$$k_e = \frac{1 - \frac{s}{2d_s}}{1 - \rho_{cc}} \quad (4)$$

where ρ_{cc} is the longitudinal reinforcement ratio with respect to the concrete core enclosed by the centerline of the spiral. For all-metallic reinforced columns k_e is equal to 0.907.

Hales et al. [8] found the Mander et al. [9] confinement model did not predict well the confining pressure from GFRP spiral. As such, the GG, CG, and NG series columns use the Hales et al. [8] model, in which the confined concrete strength is given as:

$$f'_{cc} = f'_{co} \left(-3.373 + 4.373 \sqrt{1 + \frac{2.134 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad (5)$$

where f'_l is still determined by Eq. (5) after substituting f_{yh} with f_{fu} , the ultimate tensile strength of the GFRP spiral, expressed as:

$$f_{fu} = E_f * \varepsilon_{fu} * k_{ef} \quad (6)$$

where k_{ef} is an efficiency factor ranging from 0.42 to 0.77 as determined by Sankholkar et al. [16]; the higher value of 0.77 was used for analysis. In addition, E_f is the modulus of elasticity and ε_{fu} the ultimate strain of GFRP reinforcement. A value for ε_{fu} of 0.77% for the vertical GFRP bars was adopted from the average of two maximum compressive strains measured by strain gages on the vertical GFRP bars during the axial compression tests. Using this method f_{fu} was assigned approximately 52% of the bar's tensile capacity.

Once the confined concrete strength was found using the Mander et al. [9] or Hales et al. [8] models, the overall axial compression capacity of the columns P_n was determined as:

$$P_n = 0.85[0.85f'_{cc}(A_g - A_s) + f_y A_s] \quad (7)$$

with variations being used for different material combinations of each series. The theoretical capacity of the uncorroded specimens is presented in Table 3, as well as the difference from the experimental average.

For the corroded columns, the theoretical mass loss was used to determine the percentage of steel mass loss (or area), and confinement was calculated using the adjusted value. For specimens with metallic spiral all of the corrosion-induced reduction in cross-sectional area occurred in the spiral. The new equation for the confining pressure of corroded metallic spiral is:

$$f'_{lc} = \frac{2f_{yh}[A_{sp} * (1 - M_l)]}{D's} k_e \quad (8)$$

where M_l is the theoretical percent of mass loss as a result of corrosion. The term f'_{lc} replaces f'_l in Eq. (4), with the modified confined concrete strength substituted into Eq. (9) to account for the reduced spiral section. Group 2 specimens with metallic vertical bars and GFRP spiral experienced corrosion-induced mass loss only in the vertical reinforcement; confined concrete strength was assumed to be the uncorroded value. Reducing the theoretical axial compressive capacity proportionally to the vertical reinforcement area loss results in:

$$P_n = 0.85[0.85f'_{cc}(A_g - A_{frp}) + f_y A_s(1 - M_l)] \quad (9)$$

There is no theoretical model available that could be used to predict degradation of the GFRP reinforcement from immersion in salt solution, and as such no theoretical estimates were made for the all-GFRP reinforced specimens. The theoretical column capacity for the corroded columns is shown in Table 4, along with the difference from the experiments.

Comparison of Theoretical to Experimental Axial Compression Capacity

The Mander et al. (1988) model predicted a similar capacity for uncorroded specimens and corroded specimens for the GN series with stainless steel spiral reinforcement. Axial compression capacity reduction was determined by comparing the theoretical values for corroded and uncorroded specimens and is summarized in Tables 3 and 4. The experimental capacity was less than the analytical estimate for the two specimens with GFRP vertical bars and stainless steels spiral (GN), which overpredicted the experimental performance by 4.9% and 0.6%, respectively. This indicates that the stainless steel spiral likely cannot develop its full confinement capacity when paired with the weaker GFRP vertical bars. The GN series with stainless spiral and GFRP vertical bars experienced mass losses below 6.0% and on average performed within 0.6% of the theoretical compression capacity reduction. Hybrid columns with GFRP spiral and metallic vertical bars were less predictable. The NG series with stainless steel vertical bars experienced an experimental compression capacity reduction greater than the theoretical prediction; uncorroded columns in this series experience a 10.0% range of variation in axial compression capacity; the compression capacity reduction of 4.5% is within that range. The possibility that the axial compression capacity of the solid stainless steel bars could be sensitive even to low amounts of mass loss cannot be discounted. This contrasts with the stainless clad vertical bars, which showed no such sensitivity, and had no compression capacity reduction compared to uncorroded specimens.

CONCLUSIONS

Corrosion performance and axial compression capacity of medium-scale concrete columns reinforced with combinations of corrosion-resistant reinforcement was investigated. The columns were subjected to accelerated corrosion for 60 days. For specimens undergoing moderate accelerated corrosion, the following conclusions are reached: (1) the highest axial compressive capacity of corroded columns was achieved by specimens with 316L stainless clad vertical bars and GFRP spiral; this series reached a compressive capacity 11.0% higher than the average, due to the higher corrosion resistance of the stainless clad vertical bars combined with the confining pressure of the GFRP spiral; (2) twice the electric potential per unit current density was required to reach the target current density for the specimens with metallic vertical bars and GFRP spiral, indicating greater corrosion resistance than hybrid columns with GFRP vertical bars and stainless steel spiral; (3) hybrid columns with 316L stainless clad vertical bars and GFRP spiral were the most corrosion resistant, experiencing a drop in axial capacity of only 2.3%; (4) specimens with stainless steel reinforcement are potentially sensitive to uneven concentration of pitting corrosion in the 2304 stainless steel spiral, which lead to an average of 4.7% axial compression capacity reduction due to localized section loss, in hybrid specimens reinforced with GFRP spiral with stainless vertical bars and specimens reinforced with

stainless spiral and GFRP vertical bars; (5) columns reinforced with GFRP spiral and GFRP vertical bars showed a 5.9% reduction in axial compression capacity after 60 days of continuous saturation in saltwater solution due to absorption of water in the resin; (6) specimens reinforced with GFRP spiral reached lower maximum displacements and exhibited sudden brittle failure modes compared to specimens reinforced with 2304 stainless steel spiral; (7) all hybrid specimens were more ductile than the all-GFRP specimens, with specimens reinforced with GFRP vertical bars and stainless steel spiral achieving two to three times the maximum axial displacement of other series; (8) only the metallic spiral hybrid GN specimens and uncorroded GG-C-1 specimen showed progressive failure modes compared to the sudden failure of the remaining specimens; (9) for both control and corroded specimens, a theoretical model underpredicted the axial compression capacity of columns with GFRP spiral by 5.0% to 12.0%; (10) for both control and corroded specimens, a theoretical model slightly overpredicted the axial compression capacity of columns with stainless steel spiral by up to 5.0%.

The experimental results obtained in this research, are comparable to results obtained in a previous study [7], in which the compression capacity reduction of all-carbon steel columns ranged from 13.0% to 24.0% with a mass loss ranging from 16.0% to 18.0%; the compression capacity reduction of hybrid specimens with all-carbon steel vertical bars and GFRP spiral was 5.0% with a corresponding mass loss of 7.0%. More extensive research is warranted for larger column specimens reinforced with stainless clad vertical bars confined with GFRP spiral in aggressive environments; additional research is required regarding the performance of such columns under cyclic loads simulating earthquakes.

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NOMENCLATURE

A_g – gross area of column
 A_{frp} – area of vertical GFRP reinforcement
 A_s – area of vertical reinforcement
 A_{sp} – cross-sectional area of spiral reinforcement
 CR_{cc} – percentage of drop in axial compression capacity per unit corrosion energy
 CR_{mass} – corrosion resistance in terms of mass loss per unit energy
 D' – Interior diameter of spiral
 d_s – diameter of reinforcing spiral
 E_f – modulus of elasticity of GFRP composite
 E_{total} – total corrosion energy
 F – Faraday's constant
 f_{fu} – tensile strength of GFRP reinforcement
 f'_{cc} – confined concrete strength
 f'_{co} – unconfined concrete strength
 f'_l – confining pressure
 f'_{lc} – confining pressure from corroded transverse reinforcement
 f_y – yield stress of metallic reinforcement
 f_{yh} – yield strength of transverse reinforcement
 I_{app} – average applied current
 I_{avg} – average current
 k_e – confinement effectiveness ratio
 k_{ef} – GFRP strain efficiency factor
 M_A – atomic mass
 M_{th} – theoretical mass loss
 M_l – theoretical mass loss percentage
 P_n – axial compressive capacity
 s – spiral pitch
 T – time
 v – valency

V_{avg} – average applied voltage

ε_{fu} – ultimate strain of GFRP reinforcement

ρ_{cc} – longitudinal reinforcement ratio

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Table 1—Summary of specimen materials and dimensions; (R=corroded; C=control)

Specimen Type	Number Specimens	Vertical Bar	Spiral Material	Bar Dia. (mm)	Spiral Avg. Dia. (mm)
GN-R	2	GFRP	2304 Stainless	13	197
GN-C	2	GFRP	2304 Stainless	13	197
CG-R	1	316L Stainless Clad	GFRP	16	222
CG-C	2	316L Stainless Clad	GFRP	16	222
NG-R	2	2304 Stainless	GFRP	16	222
NG-C	2	2304 Stainless	GFRP	16	222
GG-R	2	GFRP	GFRP	16	222
GG-C	2	GFRP	GFRP	16	222

1 mm = 0.04 in.

Table 2—Summary of corrosion analysis

Specimen	Target Current Density	Average Current Density	Adjusted Theoretical Mass Loss	Adjusted Theoretical % Mass Loss
	($\mu\text{A}/\text{mm}^2$)	($\mu\text{A}/\text{mm}^2$)	(kg)	(%)
GN-R-1	2.20	2.32	0.29	5.9
GN-R-2	2.20	2.06	0.26	5.2
CG-R-1	2.20	2.50	0.12	5.1
CG-R-2	2.20	2.20	0.11	4.5
NG-R-1	2.20	2.15	0.10	4.4
NG-R-2	2.20	2.11	0.10	4.3

1 $\mu\text{A}/\text{mm}^2$ = 0.64 mA/in.²; 1 kg = 2.20 lbs**Table 3**—Comparison of axial compressive capacity of control specimens using experimental results and theoretical capacity

Specimen	Theo. Model	Theo. Capacity	Exp. Capacity	Percent Diff.	Exp. Critical Displacement
		(kN)	(kN)	(%)	(mm)
GN-C-1	M, [9]	2893	2758	4.9	32
GN-C-2	M, [9]	2893	2874	0.6	31
NG-C-1	H, [8]	2992	3320	-9.9	19
NG-C-2	H, [8]	2992	3007	-0.5	16
CG-C-1	H, [8]	2908	3251	-10.6	22
GG-C-1	H, [8]	2812	3168	-11.2	19
GG-C-2	H, [8]	2812	3252	-13.5	17

1 kN = 0.225 kip; 1 mm = 0.04 in.

Table 4—Comparison of axial compressive capacity of corrosion specimens using experimental results and theoretical capacity from Eqs. (9) and (11)

Specimen	Theo. Model	Theo. Cap. (kN)	Exp. Cap. (kN)	Percent Diff. (%)	Mass Loss (%)	Theo. Cap. Drop (%)	Exp. Cap. Drop (%)	Exp. Crit. Disp. (mm)	Crit. Disp. Drop (%)
GN-R-1	M, [9]	2832	2677	5.8	5.9	2.1	4.9	29	-8.9
GN-R-2	M, [9]	2839	2805	1.2	5.2	1.8	0.4	28	-11.3
NG-R-1	H, [8]	2955	3099	-4.7	4.4	1.2	2.0	19	0.0
NG-R-2	H, [8]	2956	2945	0.4	4.3	1.2	6.9	17	-2.9
CG-R-1	H, [8]	2864	3178	-9.9	5.1	1.5	2.3	18	-18.4
CG-R-2	H, [8]	2870	3384	-15.2	4.5	1.3	0.0	18	-22.2
GG-R-1	H, [8]	2812	3126	-10.0	0.0	N/A	N/A	16	-10.5
GG-R-2	H, [8]	2812	2917	-3.6	0.0	N/A	N/A	17	-9.1

1 kN = 0.225 kip; 1 mm = 0.04 in.

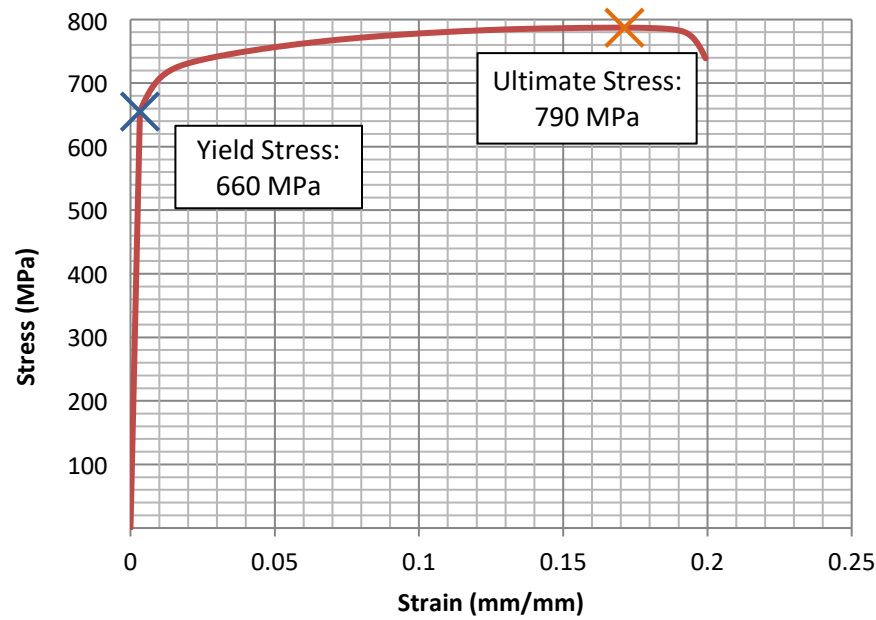


Fig. 1—Stress-strain curve of 2304 stainless steel material. (1 MPa = 0.145 ksi; 1 mm = 0.04 in.)

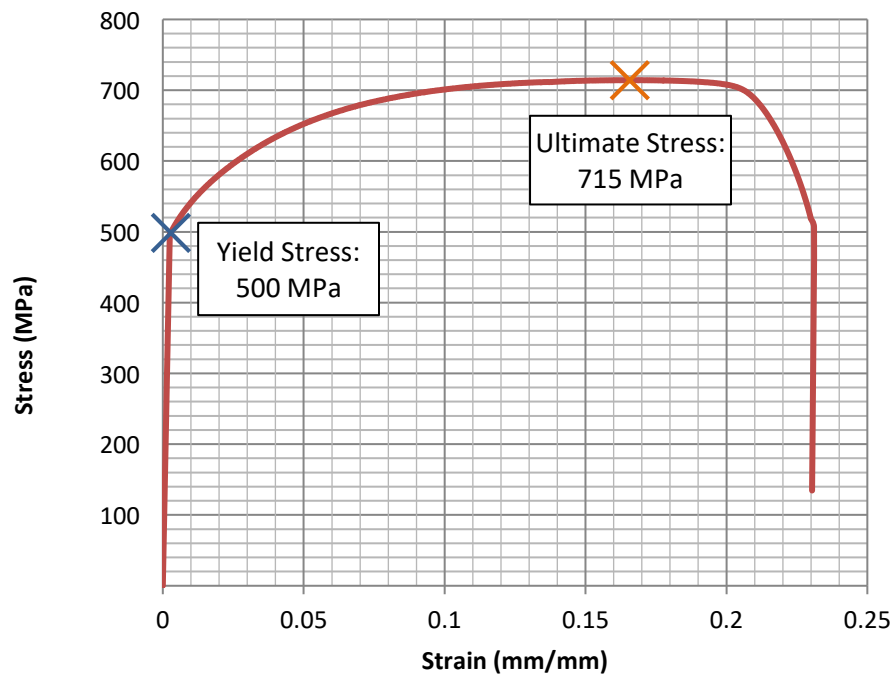


Fig. 2—Stress-strain curve of 316L stainless clad material. (1 MPa = 0.145 ksi; 1 mm = 0.04 in.)

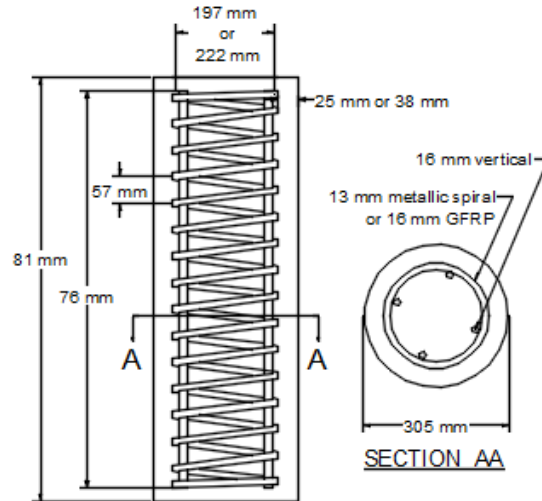


Fig. 3—Typical column specimen dimensions. (1 mm = 0.04 in.)

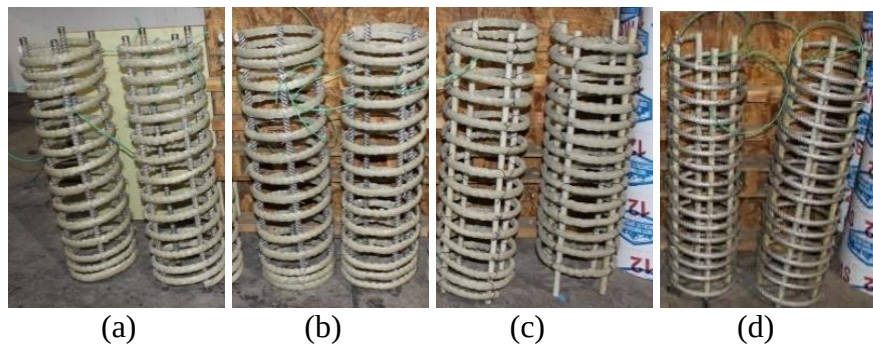


Fig. 4—Reinforcement of corroded specimen pairs: (a) CG, (b) NG, (c) GG and (d) GN series.

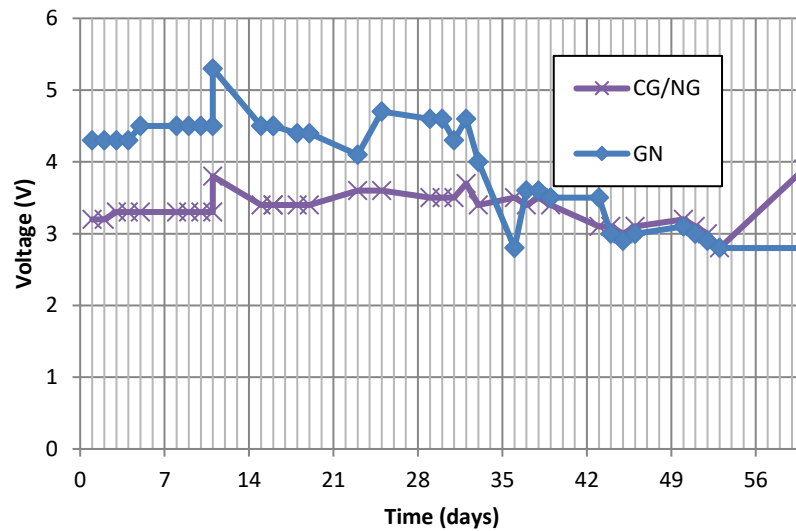


Fig. 5—Change in voltage over time.

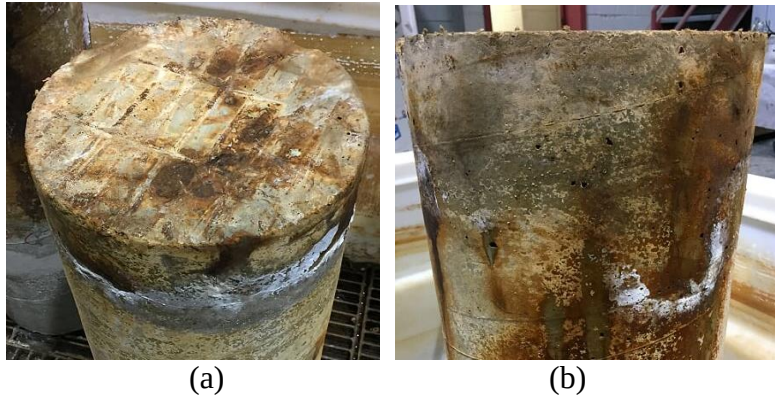


Fig. 6—External signs of corrosion on day 60: (a) GN-R-1; (b) GN-R-2.

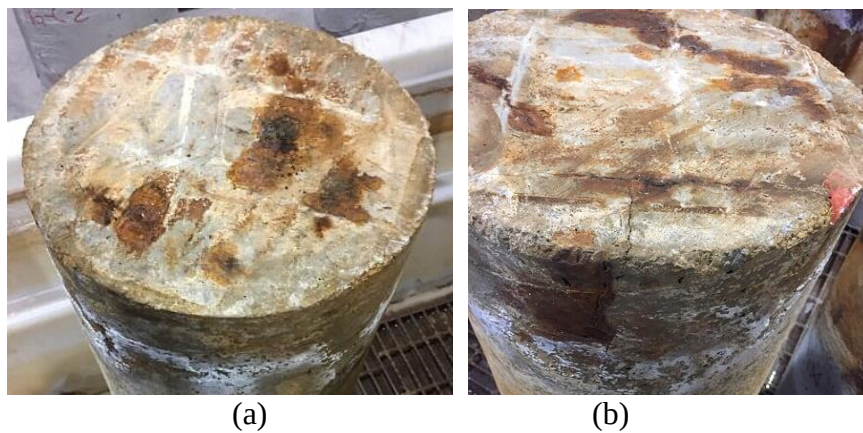


Fig. 7—External signs of corrosion and cracking on day 60: (a) CG-R-2; (b) NG-R-2.

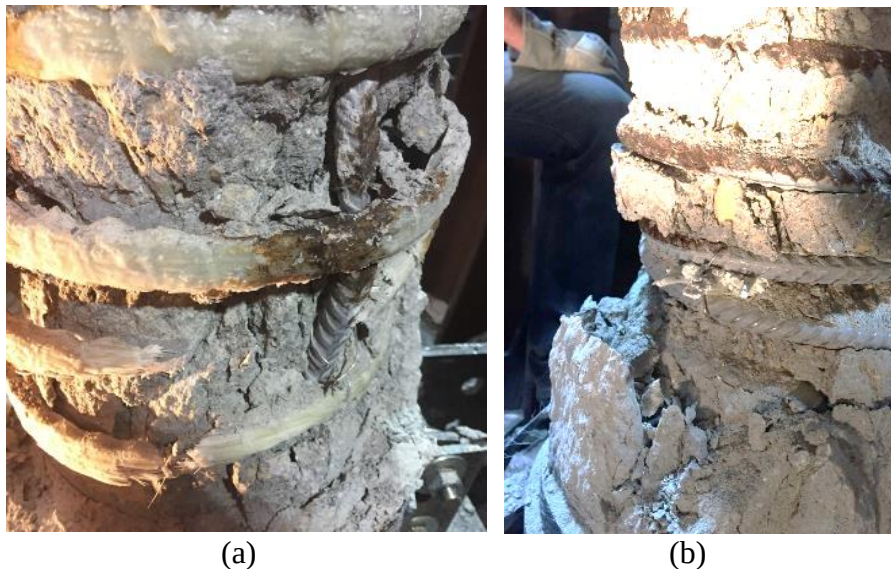


Fig. 8—Axial compression tests: (a) staining GFRP spiral and corrosion on the stainless vertical of NG-R-2; (b) corrosion damaged spiral and fractured GFRP vertical of GN-R-1.

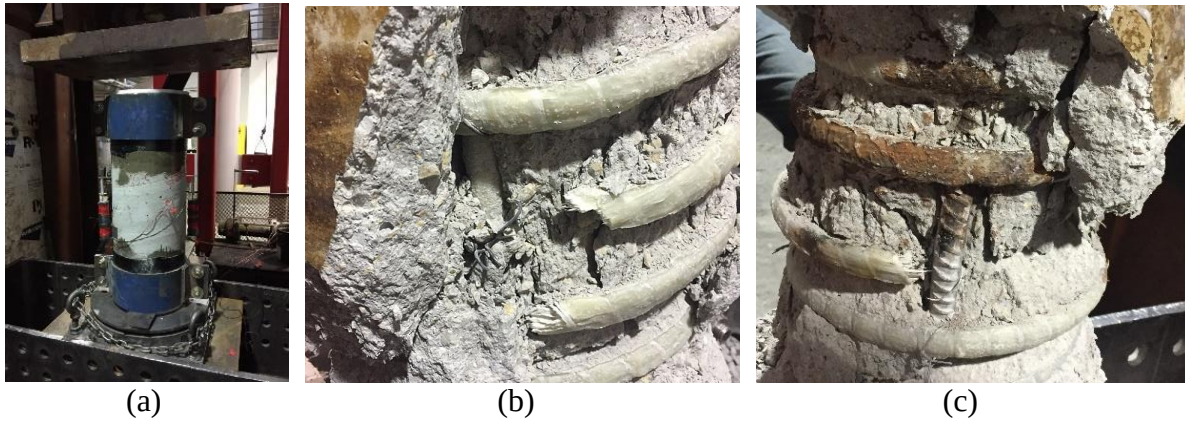


Fig. 9—Axial compressive tests: (a) test setup; (b) GG-R-2 tensile failure of GFRP spiral without visible staining of GG-R-2; (c) buckling failure of stainless clad vertical reinforcement and tensile rupture of GFRP spiral of CG-R-2.

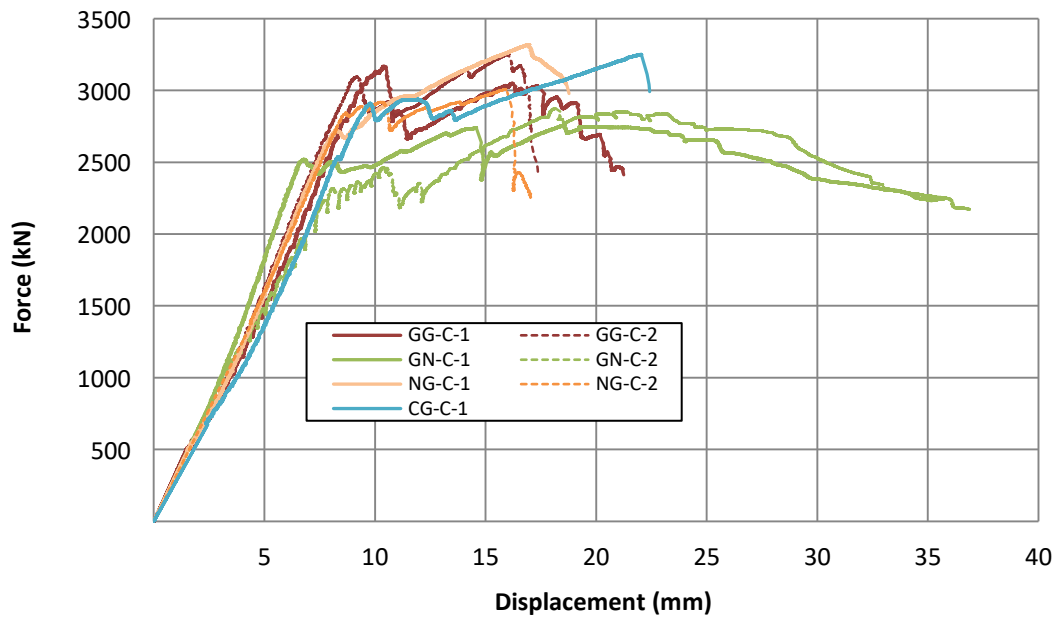


Fig. 10—Force versus displacement for control specimens. (1 kN = 0.225 kip; 1 mm = 0.04 in.)

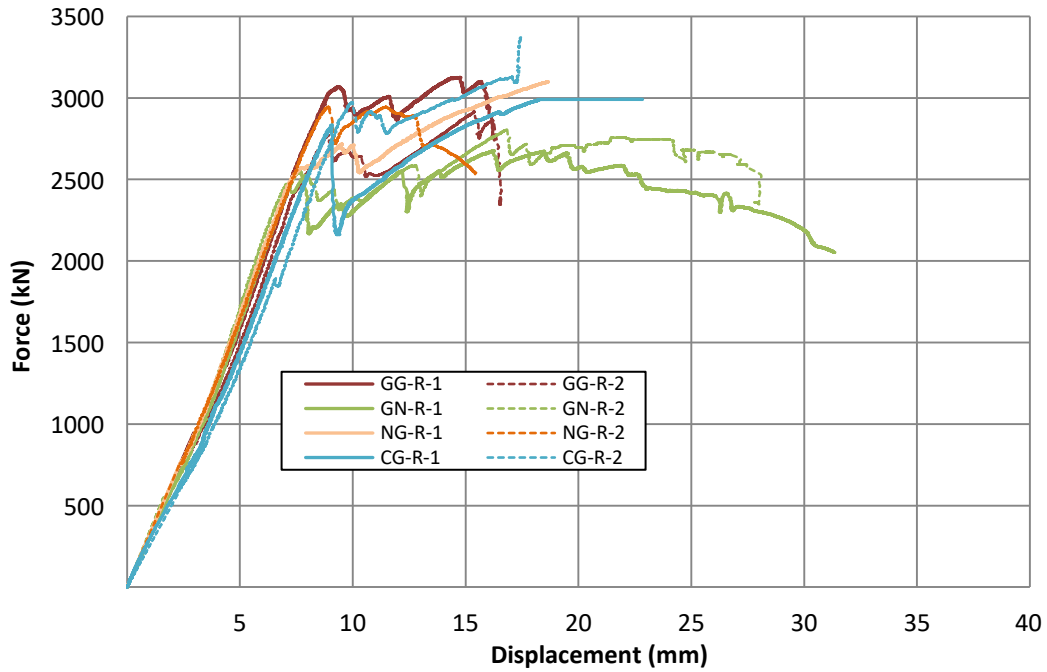


Fig. 11—Force versus displacement for corroded specimens. (1 kN = 0.225 kip; 1 mm = 0.04 in.)

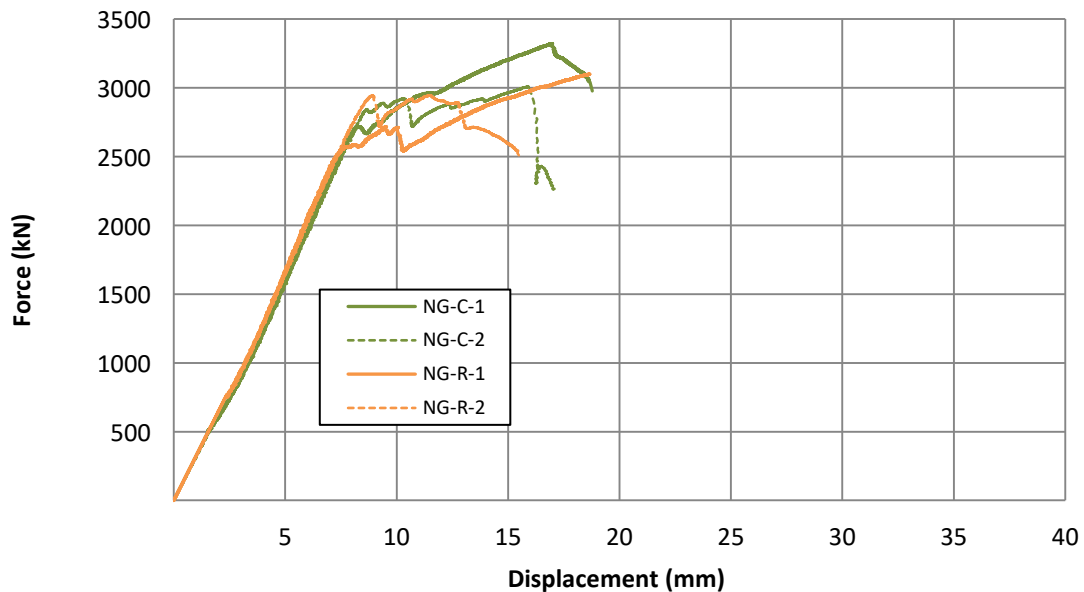


Fig. 12—Force versus displacement for NG series control (C) and corroded (R) specimens. (1 kN = 0.225 kip; 1 mm = 0.04 in.)

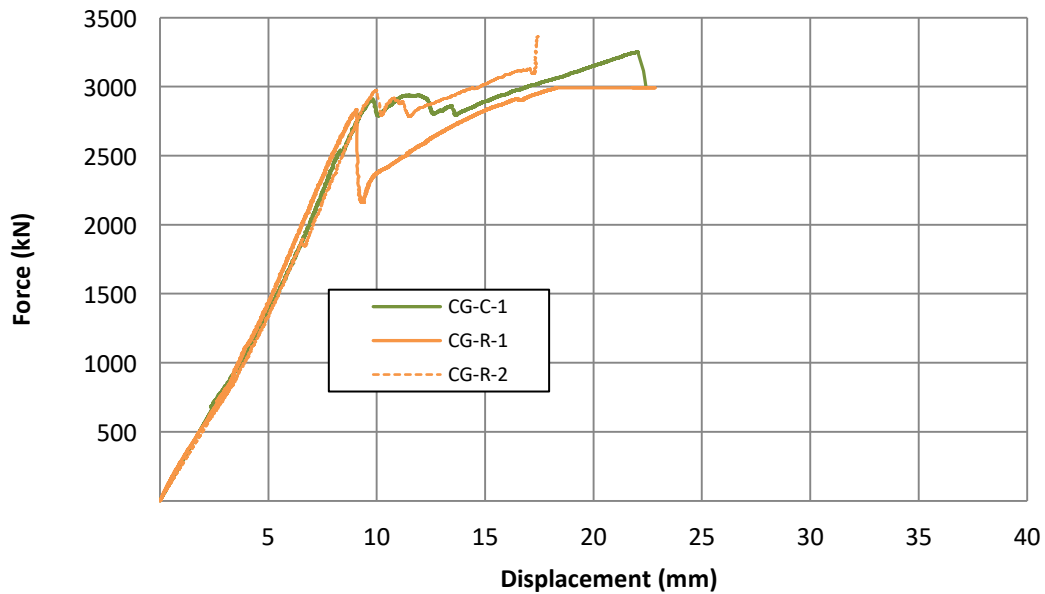


Fig. 13—Force versus displacement for CG series control (C) and corroded (R) specimens. (1 kN = 0.225 kip; 1 mm = 0.04 in.)

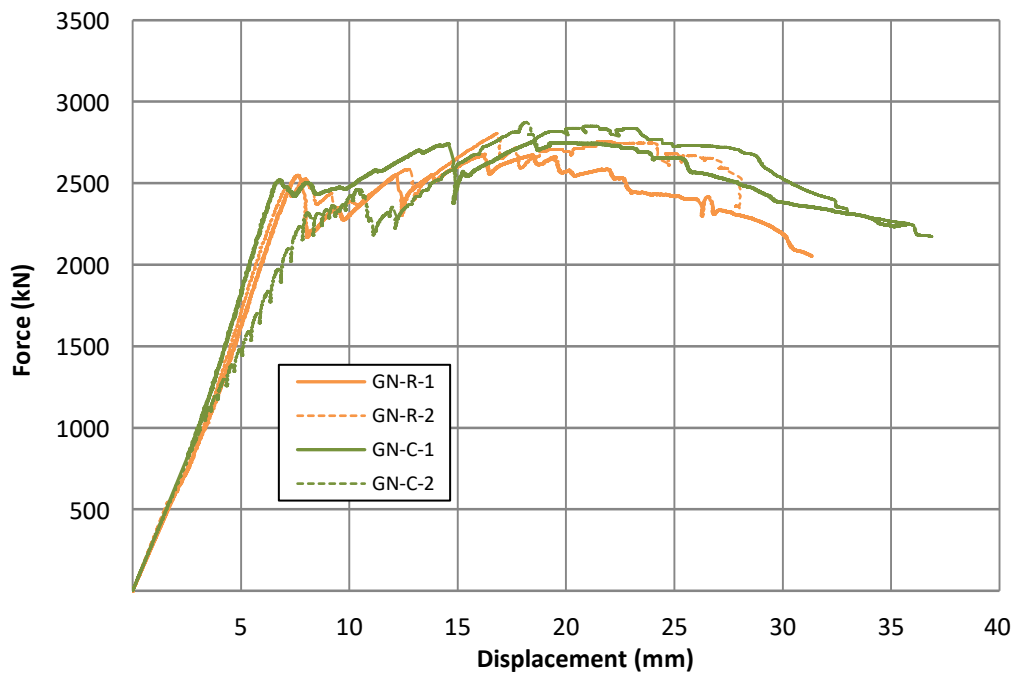


Fig. 14—Force versus displacement for GN series control (C) and corroded (R) specimens. (1 kN = 0.225 kip; 1 mm = 0.04 in.)

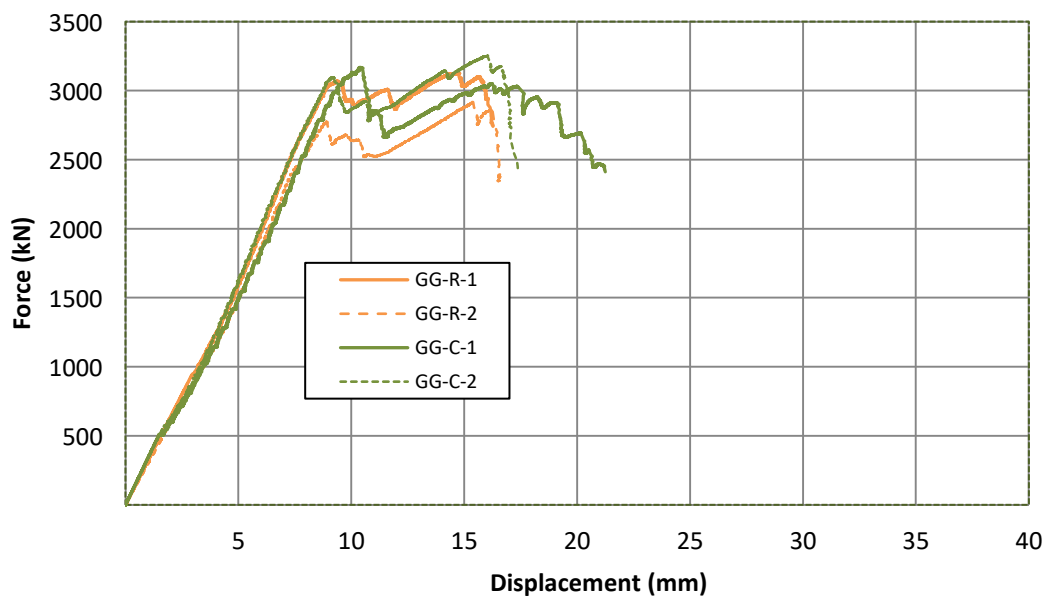


Fig. 15—Force versus displacement for GG series control (C) and corroded (R) specimens. (1 kN = 0.225 kip; 1 mm = 0.04 in.)

Remaining Service Life Assessment of Bridge Abutments Using Different Models: Comparative Study

Abeer Al-Shammari and Mojtaba Afzali

Synopsis: The United States has over 600,000 bridges, nearly 40% of which are 50 years old or older. Additionally, over 46,000 bridge structures in the country were evaluated to be structurally deficient according to the National Bridge Inventory (NBI) 2019. The average age of America's bridges keeps increasing and many of these bridges are approaching the end of their design life. Therefore, evaluating the existing condition and predicting the remaining service life of these bridges is of vital importance to agencies making the decisions to maintain, repair, or replace bridge structures based on their present and future expected conditions. Several models are available to predict the service life of new concrete structures. However, very few models are available to estimate the remaining service life of existing concrete structures. This paper presents a comparative study between two service life prediction models (Life-365 and NCHRP report 558). The service life prediction analysis was performed on bridge substructures (two abutments, namely North Abutment and South Abutment) subjected to corrosion-induced deterioration. Constructed in 1958, this bridge is located in Northern New Jersey and was 60 years old at the time this study was conducted. Review of the existing bridge documents, visual inspection, and concrete damage survey, field sampling and testing, and lab tests were performed to evaluate the existing condition of the structure and to obtain the parameters required for analysis. A good agreement between the analysis results of Life-365 and NCHRP report 558 models was obtained.

Keywords: corrosion; reinforced concrete structure; remaining service life

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INTRODUCTION

Billions of dollars are spent each year in the US repairing and even replacing bridges prematurely damaged due to the corrosion of embedded steel reinforcement. This corrosion often occurs due to the penetration of chloride from deicing salts, seawater, or ground water into the reinforcement [1]. Considering the magnitude of the problem, a significant amount of research has been carried out in the area concerning corrosion of reinforced concrete structures [2][3]. One of the challenges for bridge owners is the ability to predict the time to maintenance or repair [4]. To address this, a number of predictive models estimating the service life of concrete structures subjected to chloride ingress have been developed in recent years and some of these models are now available on a commercial basis [5]. However, most of these models are used to predict the service life of new concrete structures with very few models available to estimate the remaining service life of existing structures. Some of these predictive models include:

1. Life-365™ software was first released to the industry in August 2001 based on a need realized through the American Concrete Institute (ACI) to devise a service life prediction model for concrete. Since its initial inception, it has gone through a series of committees and software versions and improvements with the most recent version of the software (v.2.2.3) having been released in June, 2018 [6]. The primary developers of Life-365 are Evan C. Bentz, Michael Thomas, and Mark A. Ehlen. In the Life-365 software, analyses are divided into four steps: predicting the corrosion initiation period, predicting the corrosion propagation period, determining the repair schedule after the first repair, and estimating life cycle costs based on initial construction and future repair costs [5].
2. The NCHRP report 558 service life computational software is a mathematical model developed by Ali Akbar Sohahnpurwala, the author of NCHRP report 558. It was made available to users in 2006 as a macro in a spreadsheet. This model allows the user to estimate the time to initiate corrosion on both black and epoxy-coated steel rebars and to estimate the past progression of concrete damage of a bridge structure as a percentage of surface area and project the development of future damage as a function of bridge age. It should be noted that the macro may not be as user friendly as desired and familiarity with the spreadsheet program is required to effectively use this macro [2].
3. STADIUM software was developed by Samson and Marchand in 2007. STADIUM is a finite-element analysis model developed to predict the ingress of chloride and other contaminant ingresses in cementitious materials. This model accounts for the complex interaction between the contaminants and the nature of pore structure and transport processes [4][7].
4. The DuraCrete project, a probabilistic performance-based durability design of concrete structures, was developed by Siemes and Edvardsen in 1999 with financial contribution from the European Commission [8][9]. DuraCrete is not just a service life prediction model; it is a complete alternative approach for the design of concrete structures, which can overcome the limitations of conventional design methods based on deem-to-satisfy criteria/rules by explicitly using performance requirements, reliability index, and desired service life. However, its challenges lie in the reliability of modeling the input parameters [10].
5. The ClinConc model takes into consideration diffusion phenomenon alongside convection and multi-ionic characteristics. The model inputs represent the independently measured data (i.e., it does not rely on curve fitting procedures). However, ClinConc was primarily developed for submerged structures and requires certain modifications when applied to other structures [10]. In 2008, the physical model ClinConc was expressed in a more engineer-friendly way [11].

This paper presents a comparative study of two of the available service life prediction models: Life-365 v.2.2.3 and the NCHRP report 558 service life models. The study was performed using data collected from bridge abutment breast walls. Both models were intended to be applied to reinforced concrete structures exposed to chloride-induced corrosion. The methodology of the NCHRP report 558 model was developed to assess the remaining service life of existing bridge structures as a function of concrete deterioration with time. On the other hand, life-365 model is designed to predict the service life of a new reinforced concrete structure using construction details, environmental conditions, and accelerated testing. However, it is used in this study to predict the remaining service life of an existing structure using real field data. In fact, doing so can enhance the accuracy of the outputs from the model by using long-term data drawn from the in-service performance of the concrete structure under study.

RESEARCH SIGNIFICANCE

Recognizing the magnitude of the problem with regard to the impact of corrosion on US bridge infrastructure as they age, and the rising cost of dealing with this problem, there is an essential need to evaluate the remaining service life of bridge structures and predict their future corrosion activity. In other words, it is crucial to perform service life evaluations on concrete bridge structures and implement an optimal course of action (maintenance or repairs) as necessary to address any potential corrosion concerns before it is too late, such as when addressing such concerns ends up costing much more (i.e., when major rehabilitation or replacement become a requirement). In fact, performing such evaluation can help owners to assess the tendency for future corrosion-induced damage, identify cost-effective maintenance options to control corrosion and avoid regular repair cycles. This paper compares remaining service life evaluation using two of the available open-access service life prediction models: the life-365 and NCHRP report 558 models. Such comparison facilitates a better understanding of the similarities and differences of these models and a better interpretation of the output of each.

REINFORCING STEEL CORROSION MODEL, SERVICE LIFE, AND LIMIT STATES

Many of the corrosion models for reinforced concrete adopt the same approach and are based on the general deterioration model first proposed by Tuutti (1982) [12]. This model is based on corrosion deterioration which consists of two distinct periods: initiation (no corrosion period) and propagation (active corrosion), as shown in Fig. 1. The corrosion process is initiated by diffusion of chloride ions into the depth of the reinforcing steel, by carbonation therein, or by a combination of both [9]. As chloride-induced corrosion is identified as one of the most common deterioration processes affecting concrete bridge structures [13], the models presented in this paper are limited to this particular degradation process.

During the initiation period (t_i), chloride ions penetrate from the environment through the concrete cover and accumulate on the surface of the embedded steel in sufficient quantity (defined as the chloride threshold level, C_t) to initiate an active state of corrosion. The propagation period (t_p) is the time from corrosion initiation until an unacceptable level of damage or a limit state is reached. These limit states include the serviceability (excessive cracking, spalling, delamination, and excessive deformation) and ultimate limit states (flexural failure, shear failure, and punching shear failure) [14]. Thus, the length of this period depends not only on the corrosion rate but also on the level of damage that is considered "unacceptable damage," which varies based on the design and properties of the structure [5]. Therefore, the service life is defined as the period between construction and the time to the first repair (t_d) after reaching a defined unacceptable level of damage, or $t_d = t_i + t_p$ as shown in Fig. 1. The remaining service life is an estimate of the useful life of a structure based on its current rate of deterioration assuming continued exposure to given service conditions without repairs.

DIFFUSION OF CHLORIDE IONS AND A BREIF COMARISON OF SERVICE LIFE MODELS

Both models presented in this paper, Life-365 and NCHRP report 558, predict the initiation period assuming ionic diffusion to be the dominant mechanism. For both, the diffusion process of chloride ions is modeled by solving a one-dimensional equation for Fick's second law of diffusion (given the assumption that there are no cracks in the concrete which can overestimate the predicted service life):

$$\frac{\partial C}{\partial t} = D \left(\frac{\partial^2 C}{\partial x^2} \right) \quad (1)$$

where

C = chloride ion concentration

t = time

D = diffusion coefficient

This is usually solved by using the error function solution:

$$C_{(x,t)} = C_0 \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{Dt}} \right) \right] \quad (2)$$

where

$C_{(x,t)}$ = chloride concentration at depth x at time t

C_0 = surface concentration

erf = error function

The main difference between the two models lies in the approach used to predict the corrosion propagation period and the way they define the end of service life. The Life 365 model assumes a fixed period of time for propagation and considers the completion of this period as the end of the structure's service life. This approach is based on studies by Weyers, 1998 [15] which determined that the length of the period between corrosion initiation and cracking varies in range from 3 to 7 years for bridge decks in the US. A value of 6 years for the propagation period is used in the software for black steel, as indicated in Table 2. NCHRP 558 model, on the other hand, projects the propagation of corrosion deterioration with time, based on a chloride ingress predication and existing damage of the structure at a given time. The end of service life is when the damage reaches an unacceptable level. Such level is determined based on the structural design, service requirement, and other factors.

Although the two models follow Eq. (1) for modeling chloride penetration, they use different input variables, as shown in Table 1. Thus, some variation between the outputs of the two models is expectable.

CONDITION ASSESSMENT AND INPUT PARAMETERS FOR REMAINING SERVICE LIFE MODELING

Predicting the remaining service life of a concrete structure requires evaluation of its existing characteristics and obtaining the required parameters to perform remaining service life modeling. In this study, the following four tasks have been performed to collect the required information:

1. Review of the structure's as-built plans to determine the geometries required for the analysis. For example, obtaining the thickness of the abutment breast walls, which was used in the Life-365 model.
2. Visual inspection and concrete damage surveying for presence of degradation and calculating the present percentage of concrete damage which was used in the NCHRP model.
3. Field sampling and testing, which included:
 - a. Ground Penetrating Radar (GPR) scanning to mark out steel rebars in the areas to be sampled and to obtain clear concrete cover measurements which were used in both the Life-365 and NCHRP models.
 - b. Concrete coring in accordance with ASTM C 42 [16]: "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" to obtain the concrete samples required for lab testing. A total of eight cores were extracted, four from each abutment breast wall. Locations of cores on the abutment breast walls are shown in Fig. 2 and Fig. 3 for North abutment and South abutment, respectively.
 - c. Half-cell corrosion potential testing in accordance with ASTM C876 [17]: "Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete" to identify areas with likely active corrosion. Two locations were surveyed, one at each abutment breast wall.
4. Lab testing, which included:
 - a. Acid-soluble chloride content testing on the retrieved cores in accordance with ASTM C1152 [18]: "Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete" to obtain the chloride ingress

- profiles. Chloride content test setup is presented in Fig. 4. Chloride content test results, including background chloride content, were used in the both Life-365 and NCHRP models.
- b. Carbonation testing on a freshly cut plane of concrete (oriented perpendicular to the exposed concrete surface) for carbonation depth using a phenolphthalein indicator solution. Carbonation test is presented in Fig. 5.
 - c. Petrographic examination in accordance with ASTM C856 [19]: *"Standard Practice for Petrographic Examination of Hardened Concrete"* on halves of two cores which were used for chloride content testing. This test assists in determination of concrete constituents, concrete quality, cause(s) of distress and degree of damage. It should be noted that petrographic examination results are not presented in this paper except water-cement ratio (w/cm) which was used in Life-365 model.

The input parameters obtained from these four tasks, including the thickness of the abutment breast walls, the clear concrete cover, the present percent concrete damage, background chloride content, and w/cm are presented in Table 2. The chloride content profiles are presented in Table 5. The field and lab testing results are presented in the following sections:

Visual inspection and concrete damage survey results

Extensive cracking, heavy efflorescence, and spalling with exposed reinforcement were noted during visual inspection. Rust staining from corroding rebar in the breast wall was also observable at one location. The damaged areas within the concrete were measured and presented in Table 3. The present percentage of damaged concrete is used in projecting future damage following the NCHRP report 558 model. It should be noted that only areas with spalling, delamination, and patching (from previous repairs) are considered as damaged areas based on NCHRP report 558 guidelines.

Clear concrete cover results

The clear concrete cover was estimated by GPR testing. The data was analyzed using the GPR manufacturer's software, RADAN 7. An example of RADAN 7 output is presented in Fig. 6. Table 4 presents a summary and statistical analysis of the clear concrete cover measurements. Only one value of clear concrete cover per element is required (and thus the average clear concrete cover was used) in the Life-365 model, whereas clear concrete cover measurements (cover distribution) were used in the NCHRP model. It should be noted that a minimum of 30 measurements per element is generally required based on NCHRP report 558 guidelines.

Half-Cell potential testing results

The half-cell test result graphs and the interpretation of test results according to the ASTM C876 (with the corresponding color configuration) are presented in Fig. 7. Voltage readings more negative than -350 mV are considered to indicate high probability of active rebar corrosion. The results of the survey indicate that all areas surveyed had actively corroding reinforcement. It should be noted that half-cell potential testing results are not used in either model. However, they are indicative of active corrosion in the tested abutment breast walls which can supplement other findings from this study, such as the high chloride content results and the high level of existing concrete damage most likely resulting from corrosion activities.

Carbonation testing results

Minimal carbonation was detected. The depth of carbonation ranged from 0.20 to 0.31 inches (5.08 to 7.87 mm). Some amount of carbonation is expected in a structure this old. However, carbonation by itself is not expected to impact the durability of this structure. It should be noted that carbonation depth results are not used in either model. However, it gives information about the source of the noticed active corrosion in the walls under study, which is clearly not a result of carbonation, but rather chloride ingress.

Chloride content test results

Chloride content was measured for eight cores, including four from each abutment breast wall. For each core, seven subsections of the concrete were extracted and analyzed to provide a 7-point chloride profile for the concrete. The depths at which the chloride test was performed are as follows:

- 0.0 to 0.5 inches (0.0 to 12.7 mm)
- 0.5 to 1.0 inches (12.7 to 25.4 mm)

- 1.0 to 1.5 inches (25.4 to 38.1 mm)
- 1.5 to 2.0 inches (38.1 to 50.8 mm)
- 2.0 to 2.5 inches (50.8 to 63.5 mm)
- 3.0 to 3.5 inches (63.5 to 88.9 mm)
- 5.0 to 5.5 inches (8.9 to 139.7 mm)

The samples were prepared by slicing the concrete core using a dry saw and pulverizing the sample to the required size distribution using a vibrating cup pulverizer. The chloride content test results are provided in Table 5. Additionally, the average chloride content of the concrete sample at depths from 5.0 to 5.5 inches (8.9 to 139.7 mm) of values less than 0.01 % of weight of concrete (wt. conc.) is used as background chloride or initial chloride, C_i . This value was found to be approximately 0.004 % wt. conc. (provided in Table 2) which was subtracted from the chloride contents for all cores at each depth to obtain the adjusted chloride contents used in the analysis. Chloride content profiles and background chloride content were used in both the Life-365 and NCHRP models.

CHLORIDE PROFILE ANALYSIS RESULTS

The maximum surface chloride ion concentration is an apparent parameter used primarily to quantify the aggressiveness of the environment and the diffusion coefficient is an estimate of the rate of diffusion of the chloride ions into the concrete. These parameters can be obtained by "fitting" the chloride profile data of each core to Fick's second law of diffusion. The results of which for both models (NCHRP report 558 and Life-365) are presented in Table 5. A good agreement between the results from the two models was found, with the exception of cores 9 and 11 (which are highlighted in yellow in Table 5). The difference between results from the two models for cores 9 and 11 might belong to the atypical chloride profile for these cores resulting in two different trends for the predicted diffusion fits from the two models.

The abutments are exposed to chloride ions leaking through failed expansion joints above them and/or they have admixed chlorides. Cores 9 and 11 exhibit low to medium diffusion coefficients while the remaining cores exhibit high diffusion coefficients. The high diffusion rates of the remaining cores would likely be a result of high permeability, high chloride exposure, or admixed chlorides.

Initiation of corrosion on reinforcement is dependent on the chloride-to-hydroxide ratio; however, many authors use a threshold value of chloride ion content to signify initiation. It is generally given that the chloride threshold value, C_t , is approximately 0.025 % to 0.033 % wt. conc. [2]. Several probability distributions of corrosion initiation as a function of chloride ion content have been proposed. Based on these proposed distributions, 0.026 % wt. conc. of chloride ion concentration at the steel-concrete interface is generally associated with a probability of 20%. The New Jersey Department of Transportation Design Manual for Bridges and Structures [20] provides a threshold limit of 0.05 % wt. conc. For this analysis, 0.05 % wt. conc. is used as the threshold for corrosion initiation which is provided in Table 2. The orange color in Table 5 identifies depths at which the chloride ion concentration exceeds 0.05 % wt. conc.

In each model (NCHRP report 558 and Life-365), the resulting chloride profiles from each abutment were averaged and average chloride profile was developed. A diffusion curve was then fit to the average chloride profile and the surface chloride ion concentration and diffusion coefficient for this curve were back calculated. These values were reported as "Average" in Table 5 and a good agreement between the results from the two models was noticed. For example, a surface chloride content value of 0.272 % wt. conc. was obtained from the NCHRP report 558 model vs 0.273 % wt. conc. from the Life-365 model for the North abutment. The results of this analysis for the North abutment are presented in Figs. 8 and 9 from the NCHRP model and the Life-365 model, respectively, and those for the South abutment are presented in Figs. 10 and 11 from the NCHRP model and the Life-365 model, respectively. These figures show very good agreement between the models' outputs. The Chloride Front, which is defined as the depth at which the diffusion curve estimates a chloride ion concentration of 0.05 % wt. conc., indicates that chloride ions exceeded the depth of 8.0 inches (200 mm) and 3.5 inches (87.5 mm) on the North and South Abutments, respectively, meaning that all reinforcing steel is exposed to the threshold level of chloride ions.

In the NCHRP model, at least 30 cover measurements of the top mat per element were collected and used in the analysis as follows: at the Chloride Front, the percentile of clear concrete cover is identified from the cumulative clear concrete cover distribution to provide the percentage of cover measurements, exposed to sufficient chloride ions for corrosion to initiate. The cumulative clear cover distribution curves, shown in Figs. 8 and 10, indicate that 100% and

99% of the Chloride Front is exposed to the chloride threshold for the North and South abutments, respectively. Studying the clear cover distribution curves is not part of Life-365 model.

SERVICE LIFE MODELING

Utilizing the mathematical model proposed in NCHRP report 558, chloride profiles and damage information for each reinforced concrete element were used to project the damage as a function of age. The results of the model are presented in Fig. 12. The North and the South Abutments were modeled separately. The results indicate rapid damage progress for the two abutments. The South abutment currently suffers more damage and is expected to deteriorate faster than the North abutment, as shown in Fig. 12. This might not correlate well with the chloride profile analysis results for the abutments as the Chloride Front was found to be deeper for the North abutment than the South abutment. In addition, higher chloride profiles, surface chloride ion concentrations, and diffusion coefficients were found for the North abutment compared to the South abutment, which is expected to result in a higher rate of damage for the North abutment. A possible explanation for this contradiction in the outputs is that the existing concrete damage for either or both abutments was not precisely estimated.

The outcome from the Life-365 service life prediction model is represented by estimating the service life of the existing reinforced concrete structure. At a given age, the remaining service life of a structure can be calculated by subtracting the bridge age from the predicted service life, as presented in Table 6. This table shows that the North abutment already exceeded its predicted service life by 7 years while the South abutment still has 20 years of remaining service life. These results correlate well with the chloride profile analysis results demonstrating that the South abutment is in better condition than the North abutment in terms of surface chloride ion concentration, diffusion coefficient, chloride profiles, and the depth at which the chloride content reached the chloride threshold.

The modeling steps and analysis progress, along with sample inputs and outputs, are presented in Fig. 13 and Fig. 14 for the NCHRP model and Life-36 model, respectively.

The deck and superstructure of this bridge are planned to be replaced, and the new ones will have of at least 50 years. Based on the output of this study and the projected damage, it was decided that the substructure cannot last 50 years safely and within a reasonable cost. This is partly because no corrosion mitigation was performed earlier, before the substructure deteriorates to a hardly reversible level. Based on this, the authors highly recommend performing remaining service life assessment to bridge substructures at an earlier age (preferably by the middle period of their design life) and apply any course of the required action (maintenance or repairs) to address corrosion concerns before it becomes too late when major rehabilitation or replacement becomes inevitable.

CONCLUSION

Based on the study presented herein, the following conclusions can be drawn:

1. Life 365 and NCHRP 558 models generally outputted close or similar Chloride profiles, surface chloride ion concentrations, diffusion coefficients, and Chloride fronts. This is despite the difference in the input parameters of the two models. For example, the surface chloride content of the North abutment was determined as 0.272 % wt. conc. by the NCHRP model and 0.273 % wt. conc. by the Life-365 model. Chloride Front of the same abutment was found to be 8.5 inches (212 mm) in the NCHRP model vs 8.2 inches (208 mm) in the Life-365 model. For the South abutment, it was found to be 3.6 inches (91 mm) in the NCHRP model vs 3.9 inches (98 mm) in the Life-365 model.
2. The cumulative clear cover distribution curves, using the NCHRP model, indicate that 100% and 99% of the Chloride Front is exposed to the chloride threshold for the North and South abutments, respectively. The clear concrete cover distribution is not part of the Life-365 model.
3. The rate of increase in concrete damage with age from NCHRP service life modeling did not correlate well with the chloride profile analysis results. Chloride profile analysis results indicated that the North abutment is more contaminated with chloride than the South abutment. However, the service life modeling results showed that the South abutment will deteriorate faster than the North abutment. The reason for this contradiction is believed to be due to the imprecise estimation of the existing percentage of concrete damage for either or possibly both abutments.
4. The estimated remaining service life from the Life-365 model correlated well with chloride profile analysis results. The North abutment has passed its predicted service life, which correlated with high chloride profile

values and a high depth of the Chloride Front (exceeding 8.0 inches (200 mm)). The South abutment still has 20 years of remaining service life, which is with lesser chloride contamination for this abutment as compared with the North abutment.

5. Based on the remaining service life assessment results obtained in this study for a 60-year-old bridge substructure, the authors highly recommend performing such assessments at an earlier age (preferably by mid of the structure's design life) and take the appropriate action to address corrosion concerns before addressing such concerns ends up costing significantly more.

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Table 1—Different variables considered by the service life prediction models

Variable	Life-365	NCHRP 558
Percent of existing concrete damage	×	✓
Concrete temperature monthly history	✓	×
w/c	✓	×
Cover distribution (variation)	×	✓
Geographic location and nature of exposure	✓	×
Type of structure	✓	×
Type of steel	Black, epoxy coated or 316 stainless steel	Black or epoxy coated
Barrier (non, membrane or sealer)	✓	×

Table 2—Values of input parameters required for modeling

Input Parameter	Symbol	Value	Model
Age of Bridge	---	60 years	Life-365 and NCHRP 558
Thickness of the Abutment Breast Wall	---	54 inch (1.37 m)	Life-365
Concrete Cover Depth	X_d	2.50 inch (63.5 mm) North 2.66 inch (67.5 mm) South	Life-365 and NCHRP 558
% Damaged of Concrete	---	3.37% North 9.16% South	NCHRP 558
Background/Initial Chloride Content	C_i	0.004 wt. conc.	Life-365 and NCHRP 558
Water-Cement Ratio	w/cm	0.42	Life-365
Critical Chloride Threshold	C_t	0.05 % wt. conc.	Life-365 and NCHRP 558
Time to Propagate	t_p	6 years	Life-365 and NCHRP 558
Type of Steel	---	Black Steel	Life-365 and NCHRP 558

Table 3—Summary of concrete damage

Abutment	Spalled Area sq. ft. (m ²)	Delaminated Area, sq. ft. (m ²)	Surface Area, sq. ft. (m ²)	Concrete Damage, %	Damage Scaled Area, sq. ft. (m ²)
North Abutment	3 (0.3)	4 (0.4)	207.5 (19.3)	3.37%	13 (1.2)
South Abutment	17 (1.6)	2 (0.2)	207.5 (19.3)	9.16%	13 (1.2)

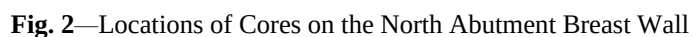
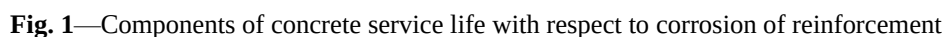
Table 4—Clear concrete cover summary and statistics

Statistics	North Abutment			South Abutment		
	Horizontal	Vertical	Combined	Horizontal	Vertical	Combined
Average of Cover, inch (mm)	2.54 (64.52)	2.43 (61.72)	2.49 (63.25)	2.70 (68.58)	2.62 (66.55)	2.66 (67.56)
Std Dev of Cover, inch (mm)	0.40 (10.16)	0.37 (9.40)	0.39 (9.91)	0.36 (9.14)	0.44 (11.18)	0.39 (9.91)
Min of Cover, inch (mm)	1.73 (43.94)	1.79 (45.47)	1.73 (43.94)	2.13 (45.10)	1.90 (48.26)	1.90 (48.26)
Max of Cover, inch (mm)	3.15 (80.01)	3.26 (82.80)	3.26 (82.80)	3.49 (88.65)	3.26 (82.80)	3.49 (88.65)
Count of Cover	19	17	36	17	15	32

Table 5—Chloride Profile Results

Abutment	Core Field ID	Chloride Ion Concentration, %wt. Conc. at Depth, inches (mm)							Surface Chloride Ion Concentration, %wt. conc.		Diffusion Coefficient, in ² /yr (m ² /sec)	
		0.5 (12.7)	1.0 (25.4)	1.5 (38.1)	2.0 (50.8)	2.5 (63.5)	3.5 (88.9)	5.5 (139.7)	NCHRP Model	Life-365 Model	NCHRP Model	Life-365 Model
North Abutment	Core 3	0.1232	0.2413	0.3076	0.2855	0.2526	0.2056	0.1120	0.412	0.404	0.215 (4.40E-12)	0.216 (4.41E-12)
	Core 4	0.1035	0.1525	0.2833	0.3304	0.2281	0.1857	0.1104	0.513	0.473	0.116 (2.37E-12)	0.147 (3.00E-12)
	Core 5	0.0573	0.1149	0.1053	0.0812	0.0708	0.0646	0.0526	0.126	0.124	0.238 (4.87E-12)	0.258 (5.28E-12)
	Core 6	0.1347	0.1738	0.1983	0.1699	0.1668	0.1798	0.1735	0.198	0.186	High Value	23.65 (4.84E-10)
	Average	0.1047	0.1706	0.2236	0.2168	0.1796	0.1589	0.1121	0.272	0.273	0.326 (6.67E-12)	0.340 (6.95E-12)
South Abutment	Core 9	0.1461	0.0497	0.0238	0.0142	0.0273	0.0446		0.276	0.030	0.005 (0.10E-12)	9.58E+38 (1.96E+28)
	Core 11	0.0832	0.0053	0.0038	0.0318	0.0042	0.0046	0.0036	0.360	0.013	0.001 (0.02E-12)	0.169 (3.46E-12)
	Core 12	0.1807	0.232	0.278	0.2203	0.1502	0.0725	0.0276	0.523	0.530	0.049 (1.00E-12)	0.046 (0.95E-12)
	Core 14	0.1246	0.1029	0.1252	0.098	0.0843	0.0822	0.0828	0.124	0.120	0.529 (0.11E-12)	0.914 (0.18E-12)
	Average	0.1337	0.0975	0.1077	0.0911	0.0665	0.051	0.038	0.145	0.141	0.119 (2.43E-12)	0.136 (2.78E-12)

**Fixed valued, entered in the software



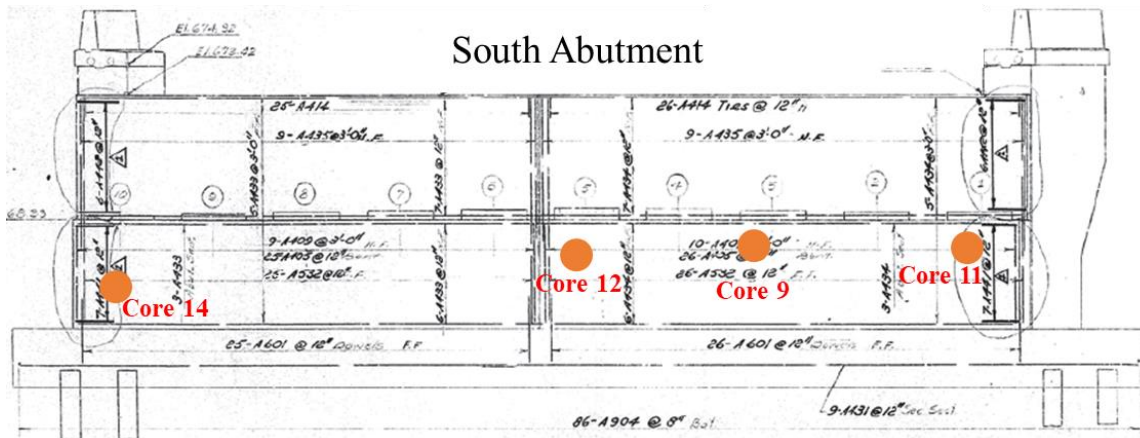


Fig. 3—Locations of Cores on the South Abutment Breast Wall

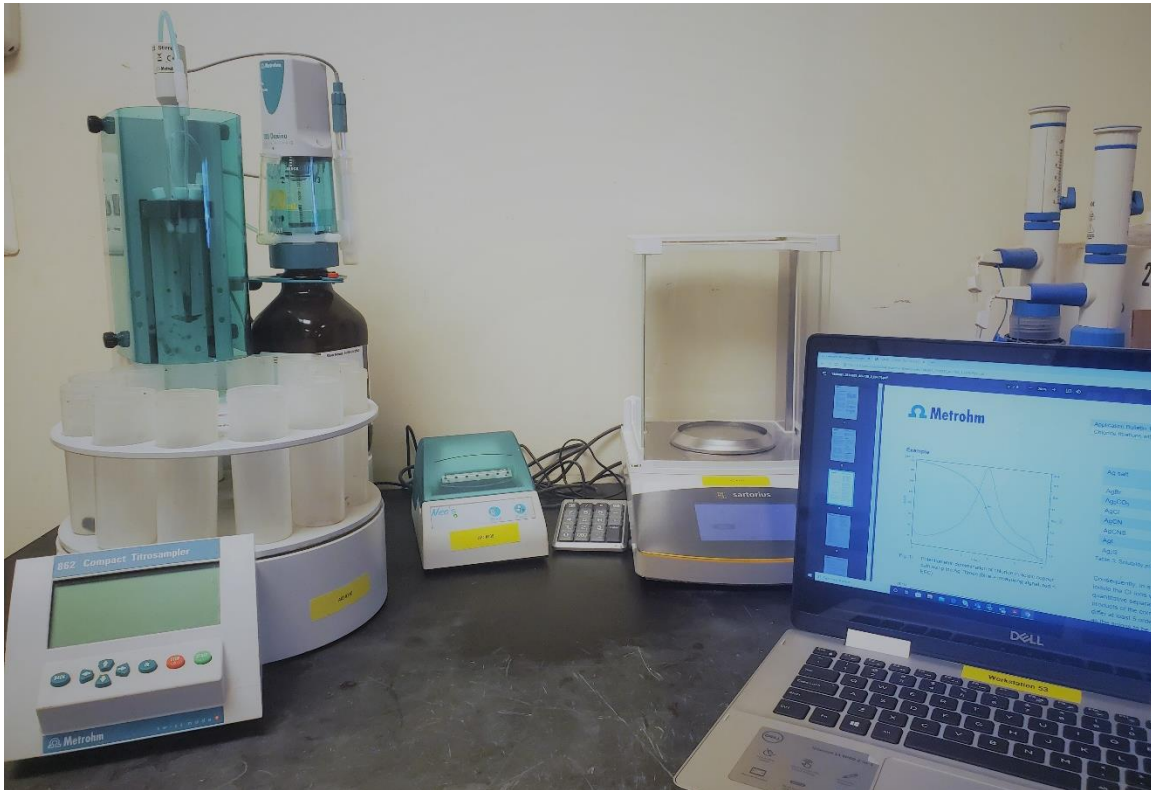


Fig. 4—Chloride test set up

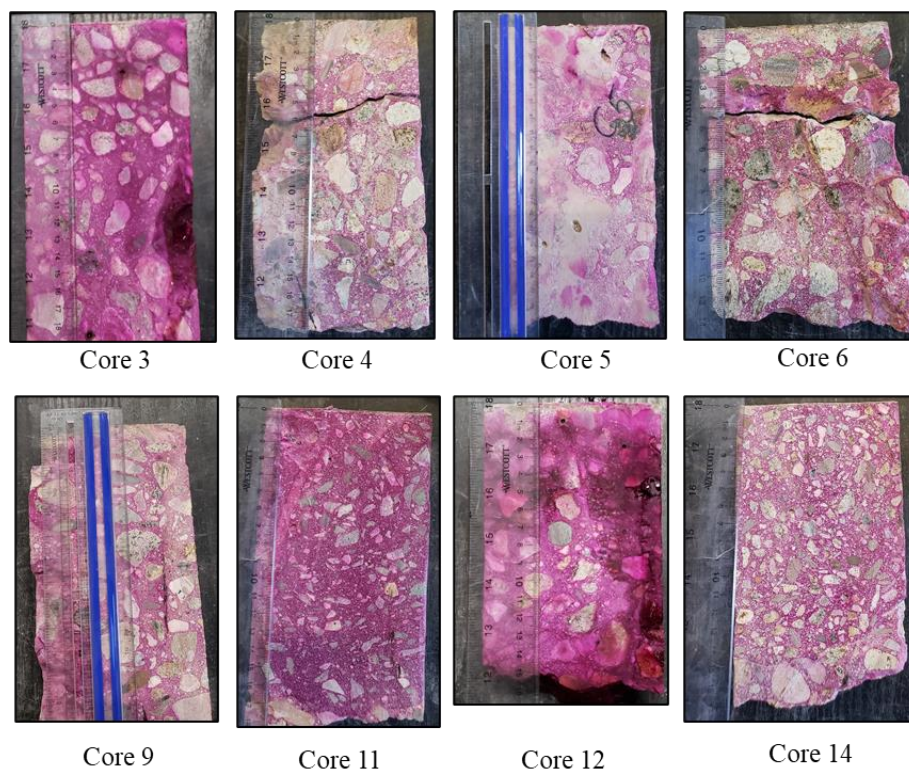


Fig. 5—Carbonation Test

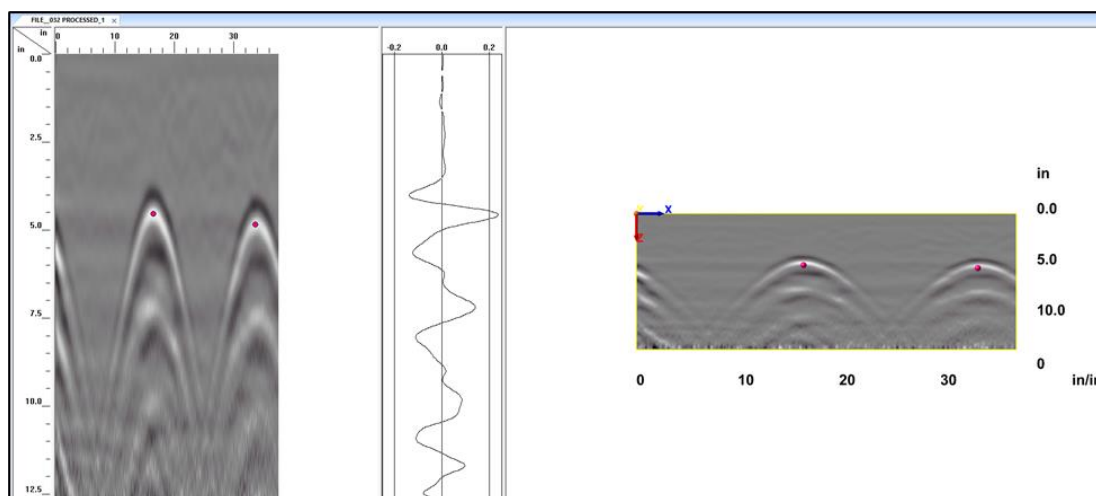


Fig. 6—GPR Software, RADAN 7, Sample Output for Concrete Cover Measurement

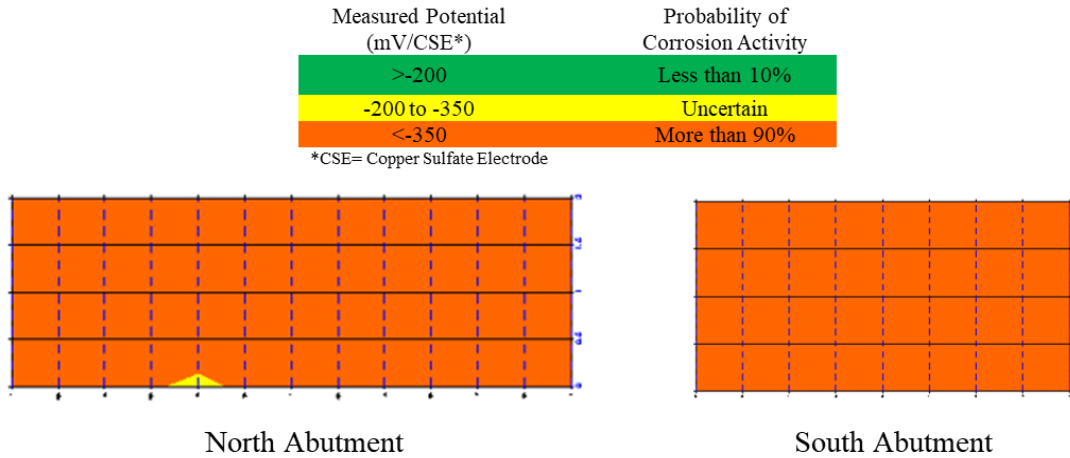


Fig. 7—Half-cell potential survey results

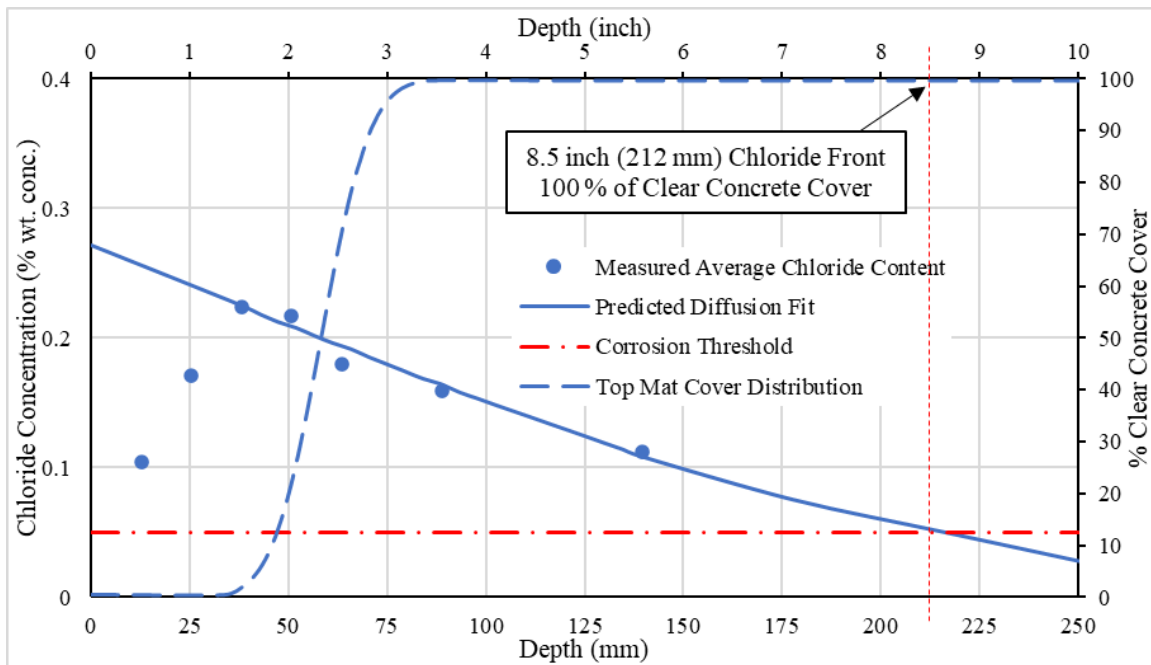


Fig. 8—Average Distribution of Chloride Ions and Cumulative Clear Concrete Cover for North Abutment (using NCHRP model)

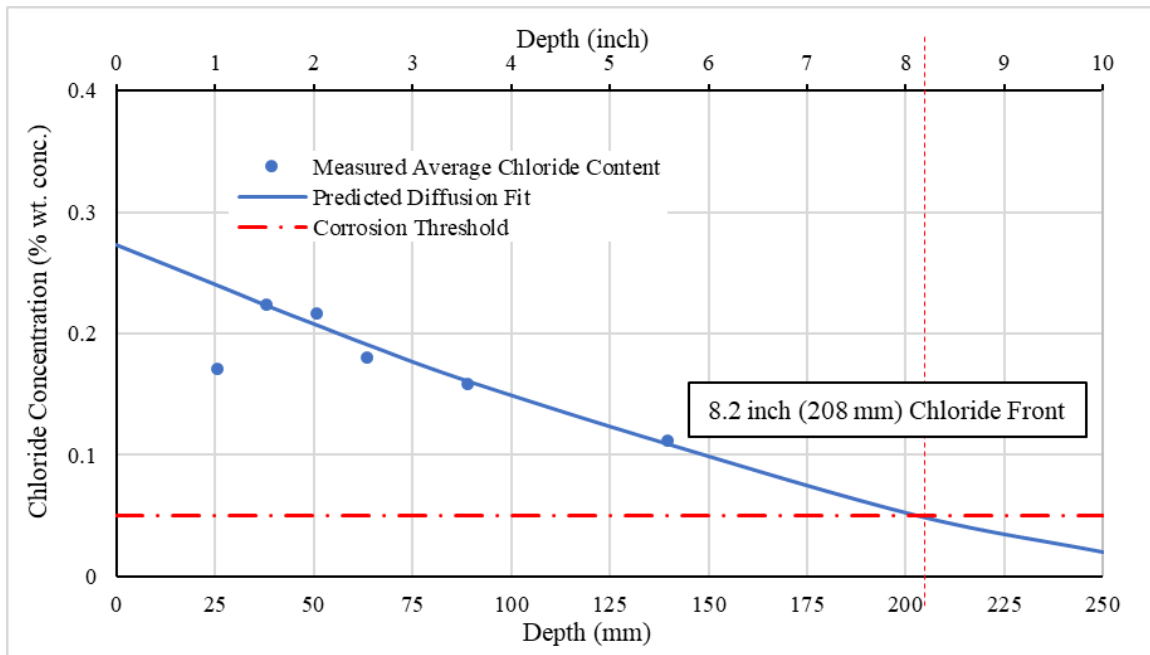


Fig. 9—Average Distribution of Chloride Ions for North Abutment (using Life-365 model)

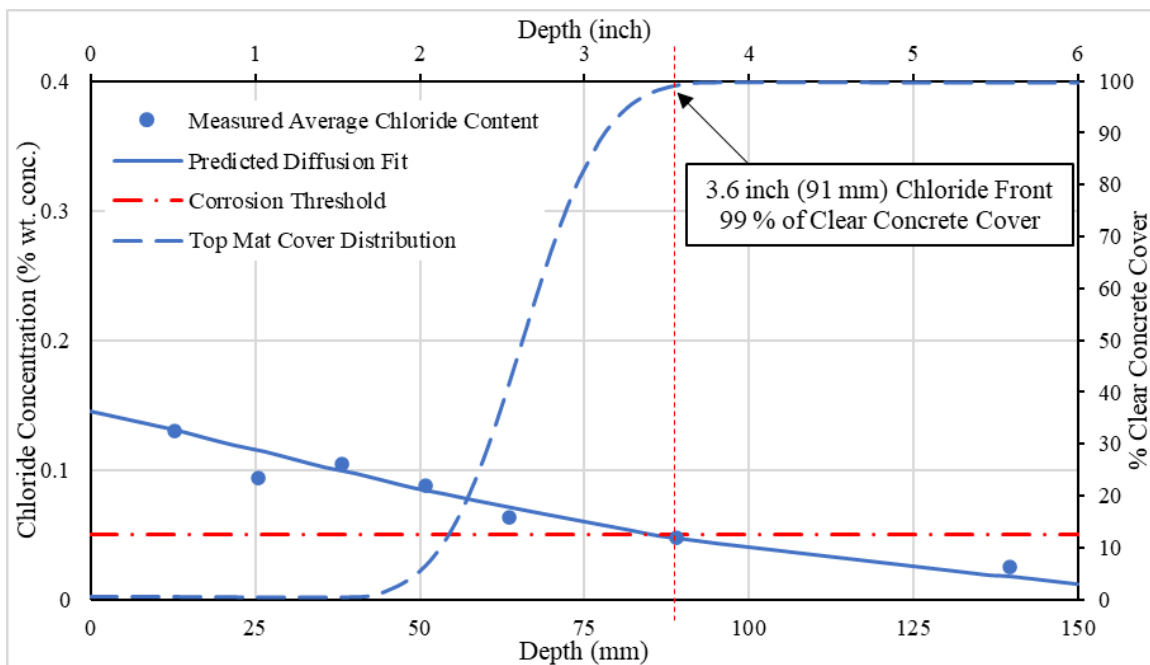


Fig. 10—Average Distribution of Chloride Ions and Cumulative Clear Concrete Cover for South Abutment (using NCHRP model)

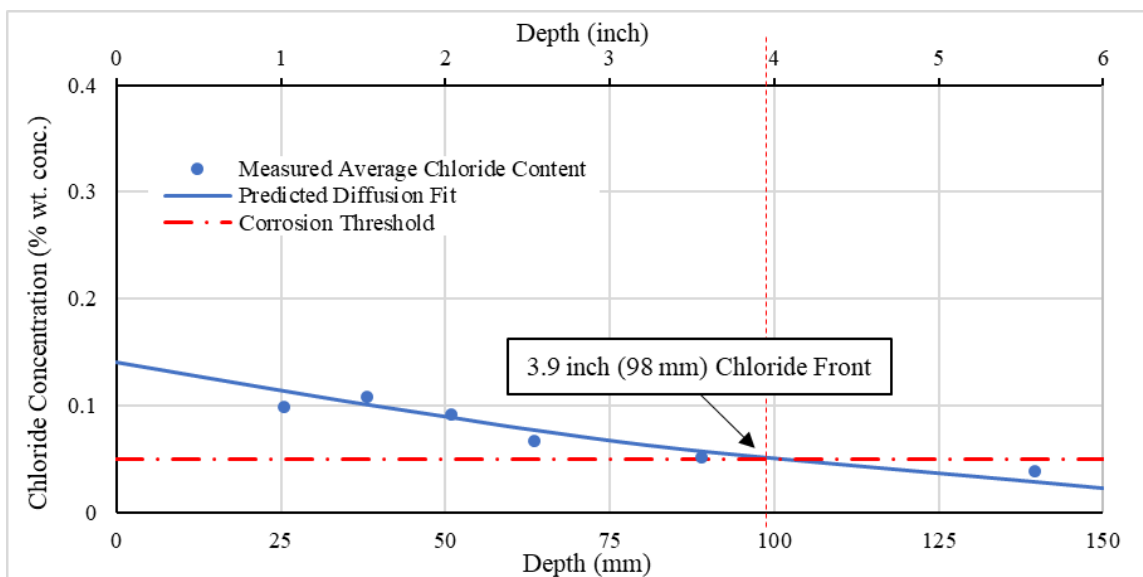


Fig. 11—Average Distribution of Chloride Ions for South Abutment (*using Life-365 model*)

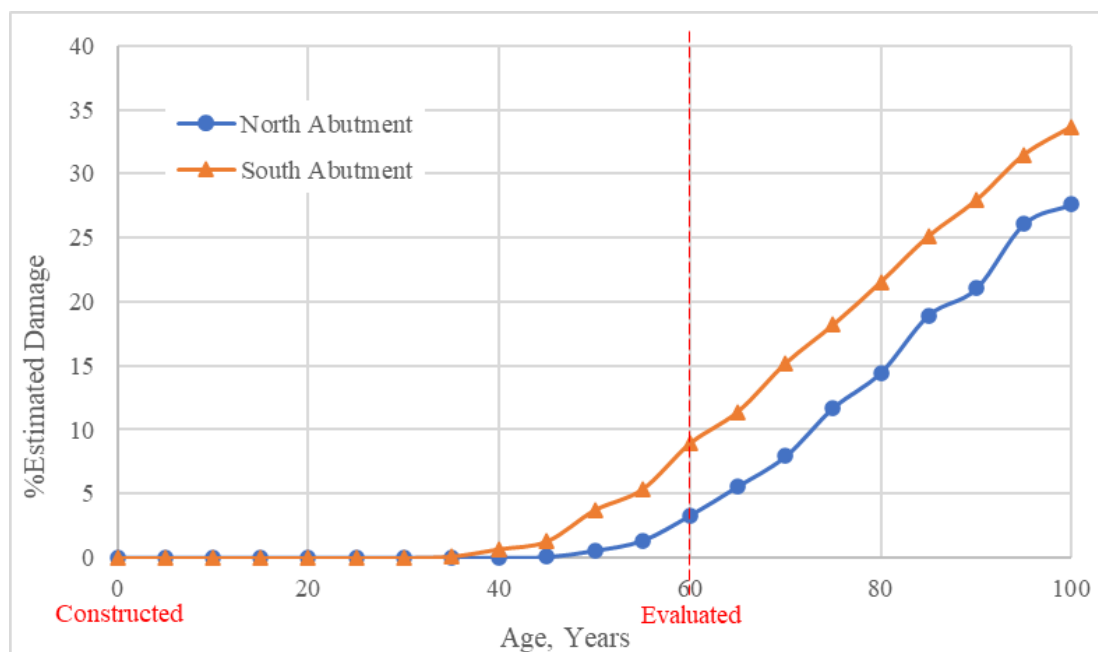


Fig. 12—Projection of damage vs age of the abutments (*using NCHRP model*)

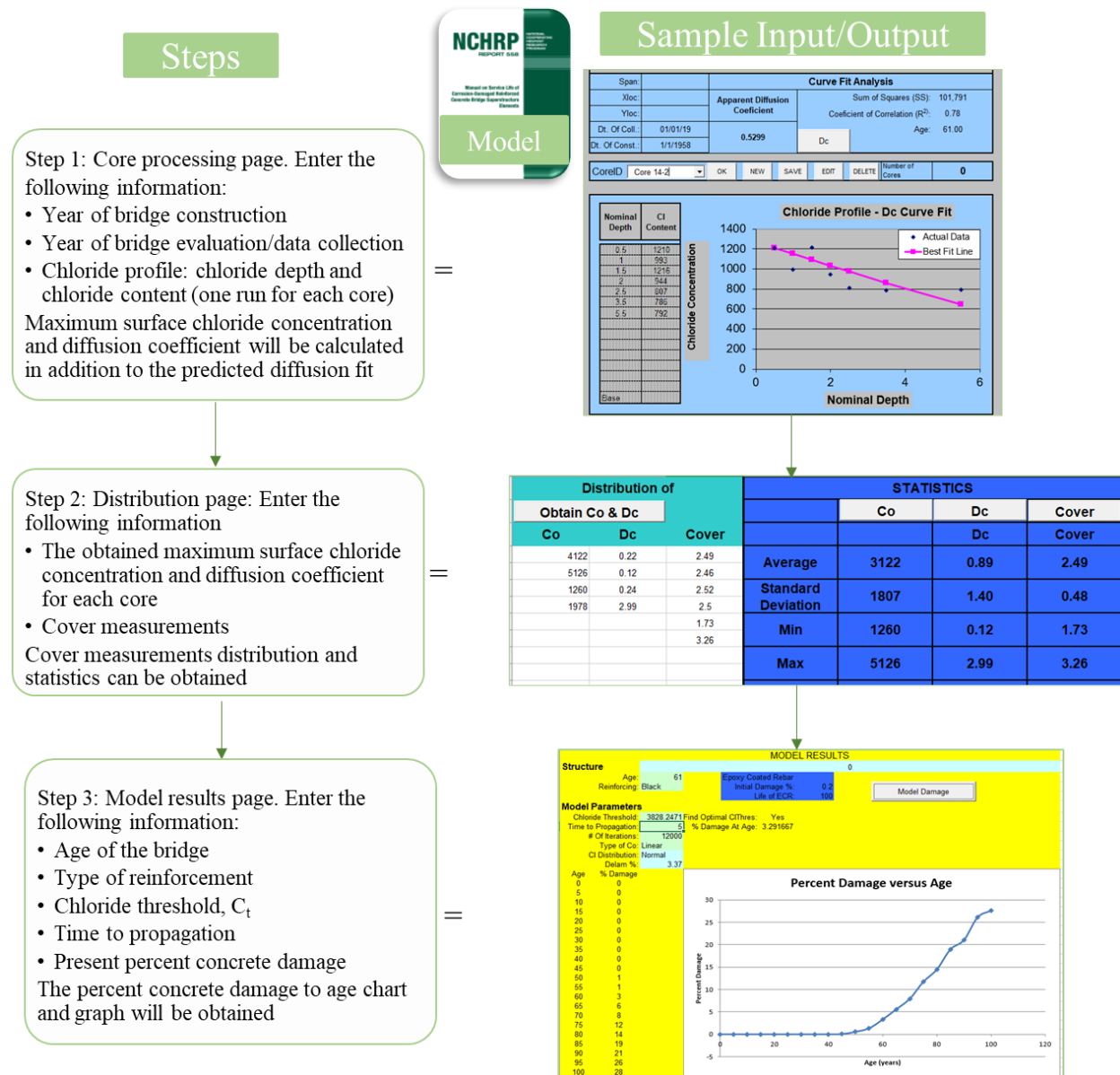


Fig. 13—Analysis progress and sample inputs/outputs using NCHRP model

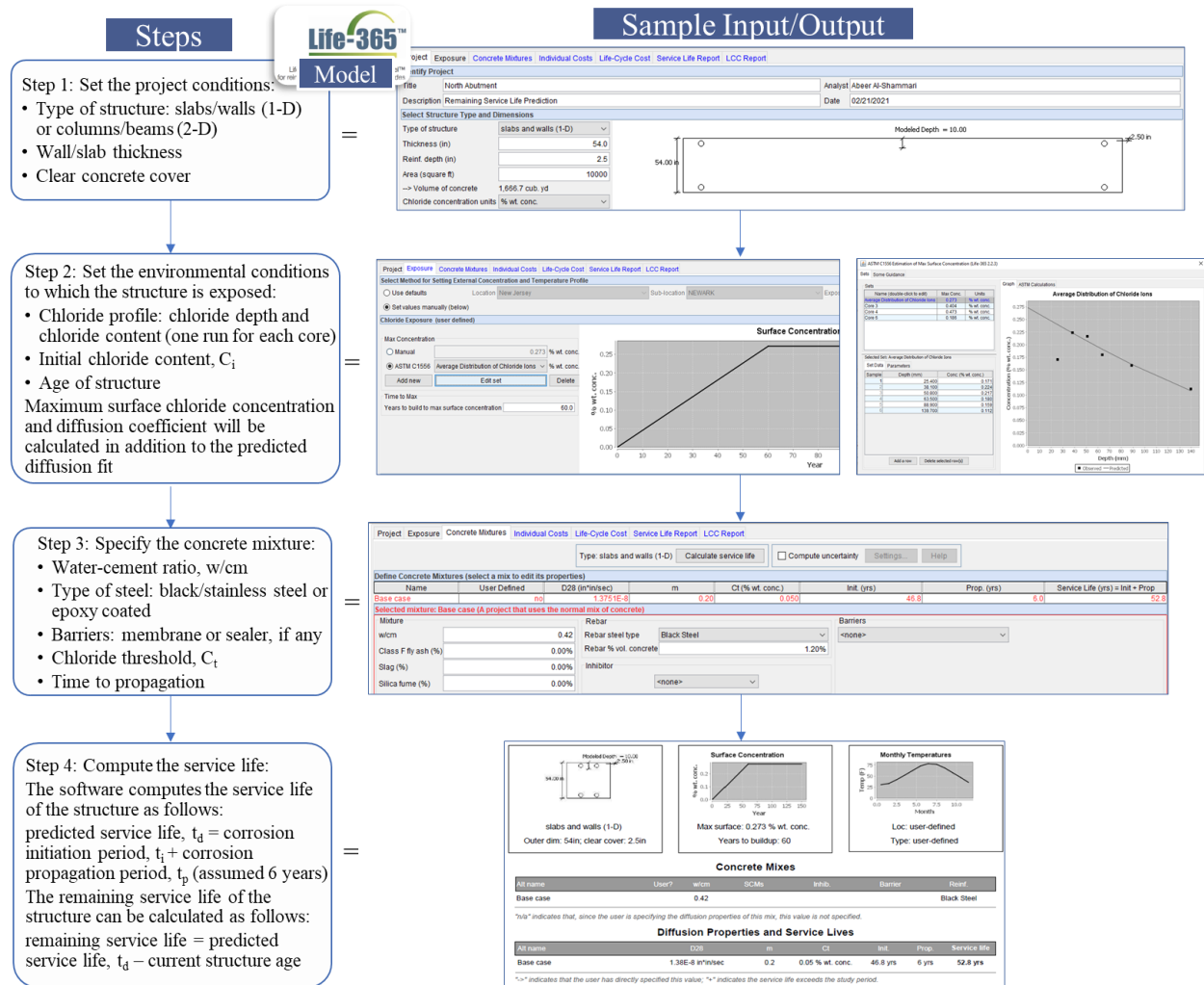


Fig. 14—Analysis progress and sample inputs/outputs using Life-365 model

A Consideration of the Structural Integrity of Time-Varying Mass Systems

Vanissorn Vimonsatit, Phung Tu, and Jack Fletcher

Synopsis: Traditionally, a time-varying mass system is viewed as the motion of moving bodies exiting or colliding with the system, such as rockets. A standing structure is not typically considered a time-varying mass system, but a silo during discharge of the infill is a subtle time-varying mass structure. Slender silos and silos with insufficiently stiffened supports are vulnerable to excessive vibration (silo quaking) and loud disruptive noises (silo honking) caused by the flow of the exiting masses. Using principles of mechanics and conservation of momentum, the equation of motion of such systems can be formulated to incorporate the discharge rate, material properties and the time-dependent characteristics of the system (mass, damping and stiffness). In this paper, the acceleration and mass flow of granular fill in a perspex tubing during discharge have been reproduced to simulate silo honking. By controlling the majority of influential factors, the replication of a small-scale silo design was possible with the repeatability of silo honking achieved in a controlled environment. A comparative study between discharge testing results of the sand fill with 0% (control), 5% and 10% moisture content shows that increasing the moisture content of the fill reduces the vibrational effect on the silo walls, and in turn reduces the magnitude of silo honking. Further, the effect of the sudden mass loss on a system of reinforced concrete columns depicting that of silo supports is investigated. The results show the exponential changes in the acceleration and velocity responses of the structure when subjected to a sudden mass loss. Finally, notes on how to consider the system of the forces in the silo structure based on the existing silo theory are provided.

Keywords: structural dynamics; silo integrity; silo quaking; time-varying mass structures

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INTRODUCTION

The integrity of time-varying mass systems depends on an accurate prediction of the forces interacting between the remaining and exiting bodies. As the bodies are in motion, the net and relative velocities generate forces that will impact the structural dynamic characteristics, and vice versa, to remain in its time-dependent state. Systems that have mass varies with respect to time are found in mass flow problems in various fields of applications such as astronomy, machine dynamics, biomechanics, rocketry, or fluid dynamic [1]. In the field of structural engineering, time-varying mass problems are not commonly deliberated as building structures are typically considered to have constant masses, and the rate of mass changes due to the movement of live loads is small relative to the mass of the structure. Obvious cases when the mass changes are not relatively small can be found in robots, conveyor belts and rockets [2]. In such cases, the motion during the separation (or collision) of the moving bodies can be explained using the conservation of momentum and Newton's laws of motion. The physics of a time-varying mass system are to account for changes in the mass and momentum, and their effect on the external and internal forces of the system [3].

Silo structures such as train load out bins commonly used in mining are another type of time-varying mass problems [4]. Existing methods for silo structural designs are based on Janssen's theory [5] which requires cross-disciplinary fields of mechanical engineering and structural engineering. The flow of the granular infill during discharge is generally modelled to predict the acceleration of the flowing materials by assuming the structure is infinitely rigid [5]. As a result, the effect of the dynamic excitation of the structure on the flow model is not incorporated [6]. To account for the interaction between the infill and the silo structure, the silo, the supporting structure, and the material infill must be considered as an integral system during discharge whereby the infill flowing out of the silo is the mass loss of the system. The physics of the flowing material exiting the supporting structure such as in a silo system, or simply a salt shaker, tell us that the motion generates forces, which when interacting with the supporting structure, the structure will vibrate to maintain its equilibrium [6]. The integrity of the structure then depends on its natural characteristics (mass, stiffness and damping), any changes in these characteristics may cause excessive vibration. In a silo system, the excessive vibration which leads to a structural failure is known as silo quaking [7], which is caused by self-excited dynamic effects of granular material during discharge. These dynamic effects are generally accompanied by earthquake like shocks and a booming sound, referred to as silo music or silo honking [8]. The honking noise is produced by the interaction between the silo wall and sliding granular material during mass flow.

This paper presents a measurement of the acceleration and mass flow of granular fill based on recent experimental studies by Griffiths [9] and Fletcher [10]. A perspex tubing during discharge has been reproduced in a laboratory setting. By controlling the majority of influential factors, the replication of a small-scale silo design was possible with the repeatability of silo honking achieved in a controlled environment. A comparative study between the 0% (control), 5% and 10% moisture content discharge testing results shows that by increasing the moisture content of the fill it reduces the vibrational effect on the silo walls and in turn reduces the magnitude of silo honking. The degree at which changing the moisture content effect silo honking is still unknown and requires further testing. The next part of the paper is to study the effect of a sudden mass loss on a reinforced concrete structure typically depicted that of a silo supporting system. A numerical analysis on a system of four reinforced concrete columns is performed to investigate the effect of the mass loss on the dynamic characteristics of the structure.

RESEARCH SIGNIFICANCE

Understanding factors affecting the structural integrity of time-varying mass systems is critical in achieving accurate design and operation safety. The first challenge in dealing with such structures is to recognize how critical it is to consider the systems as having variable masses. It is obvious when dealing with a rocket problem, but may not be so with other standing structures such as a silo system. Literature shows discharging silos could suffer excessive vibrations (silo quaking) and loud noises (silo honking), leading to possible failure, endangering the health and safety of the operators, and disrupting the businesses, which could cost millions per day in a mine site. This paper highlights what need to be considered in a time-varying mass problem and how the structural responses to dynamic excitations are affected by the changes in mass, which can be due to material deterioration during the service life of the structure.

SCOPE OF STUDY

The scope of this study is firstly to demonstrate the phenomenon of silo honking through acceleration measurements based on a physical testing of silo models in a laboratory setting. The study is to explore the effect that the moisture content of fill has on exciting the vibrations that drive silo-quake in an experimental model test environment. This will be achieved through a comparative study of the accelerations experienced by the model silo wall at multiple heights, with fill of increased moisture contents. Secondly, this study is to highlight the effect of a sudden change in the total mass of a reinforced concrete system, which is typically that of silo supports, that can cause the concrete structure to vibrate to maintain its equilibrium. The deterioration of concrete material such as spalling can cause the concrete structure to lose its stiffness, which can cause the concrete structure to vibrate stronger when subjected to the same mass loss. The strong motion can negatively affect the integrity of the concrete structure.

TIME-VARYING MASS STRUCTURE – SILO STRUCTURE

A typical time varying mass structure is a silo system where the total mass of the silo structure rapidly reduces with respect to time during discharge. According to the fundamentals of applied physics and engineering mechanics, initial conditions (accelerations, velocities, and displacements) of the structure, properties of the exiting infill and the rate of change of the total mass of the structure are required to determine the response of the structure when the mass that causes initial conditions are suddenly lost. Tu [7] recently proposed a silo quaking equation to predict the granular-structure interaction of the silo system. The equation is in the form of the dynamic equilibrium of motion in which the properties of the structure (mass, damping and stiffness), properties of the granular material inside the container, the displacements of the granular particles and the dynamic effects due to the change in momentum for the entire duration of the discharge cycle are accounted for. Based on this equation, it can be explained that silo loads during discharge are governed by the structural properties (mass, damping, and stiffness), material properties and discharge rate. As such, the silo dynamic loads during discharge can be controlled and minimized by altering the structural properties (mass, damping, and stiffness), material properties and discharge rate.

Fig. 1 illustrates the equilibrium of forces inside a silo before discharge such that the frictional force (τ_w) on the wall is balanced by the vertical stiffness of the supporting structure of the silo (k_{st}), the lateral force (F_w) is balanced by the lateral stiffness of the silo wall (k_w), and the vertical force ($F_v + \Delta h \rho_b g$) acting through a material layer Δh is balanced by the stiffness of the material under (k_{eff}). Fig. 2 illustrates the equilibrium of forces inside a silo during discharge such that the time varying frictional force ($\tau_w(t)$) on the wall is balanced by the dynamic structural properties ($k_{st}(t)$, $c_{st}(t)$, $m_{st}(t)$) of the supporting structure of the silo, the time varying lateral force ($F_w(t)$) is balanced by the dynamic structural properties ($k_w(t)$, $c_w(t)$, $m_w(t)$) of wall of the silo, and the vertical force ($F_v(t) + \Delta h \rho_b (g \pm \ddot{y}(t)) + v_e(t) \dot{m}(t)$) is balanced by the dynamic structural properties ($k_{eff}(t)$, $c_{eff}(t)$, $m_{eff}(t)$) of the material layer under. Fig. 2 also highlights that sound can be heard due to the vibrating silo wall. Recent experimental studies where the flexible silo wall was simulated with polycarbonate [9] and perspex [10] tubes filled with sand show that the wall vibrated, caused the loud sound, and stronger wall material dampened the sound.

The mechanics of the forces presented in Fig. 1 and Fig. 2 are mathematically summarized in Eq. (1), which was formulated by Tu [7] by uniting the traditional equation of motion with the rocket equation. As such, it has applications across many areas of engineering and science and may be considered as the equation of time varying mass dynamic system.

$$(m(t) - dm)\ddot{y}(t) + (2\delta(t)\sqrt{k(t)m(t) - dm})\dot{y}(t) + k(t)y(t) = \sum f(t) \quad (1)$$

SILO HONKING STUDY

The origin of silo honking has been discussed loosely amongst researchers. The main topics of discussion are centred around investigations into interactions between the fill, silo walls and the hopper, the height at which the honking occurs, and the magnitude of honking at varying depths. Determining these three effects is important because it dictates the silo design selected for experimentation and the location of testing equipment that are used. Several hypotheses have been proposed as the cause of the dynamic effect stimulating silo honking, these include; grain collision [11], insufficient flowability of granular material in silos [12], energy release by falling bulk solids in silos [13], changing flow patterns [14] and change in acceleration of granular material between hopper and bin [15]. The most agreed upon proposal is the slip-stick motion of bulk solids sliding along the smooth face of a silo wall [16]. This interaction between granular material and the silo walls, influenced by interparticle friction, causes them to slip and stick along the silo's inner surface. The first phase, "stick", is due to the lateral pressure pushing the silo fill against the silo walls in relation to the frictional coefficient of the walls, and the stationary material below. Since the outlet diameter is always less than the internal diameter of the silo, it forces the material to converge and as the lower granules fall away during the discharge, the static friction is overcome and the granules "slip" along the walls of the silo. The slip-stick behaviour can propagate through the fill material onto the silo structure resulting in vibrations and in turn causing silo honking. Silo honking always occurs where this