Condition Assessment of Corrosion-damaged Bridge Girders Strengthened with Post-tensioned Composite Strips

Yail J. Kim¹, Jae-Yoon Kang², Jong-Sup Park², and Woo-Tai Jung²

¹ Department of Civil Engineering, University of Colorado Denver, Denver, CO 80217, USA
² Structural Engineering Research Division, SOC Research Institute, Korea Institute of Civil Engineering and Building Technology, Gyeonggi, Korea

woody@kict.re.kr

Abstract. This paper presents the condition evaluation of bridge girders upgraded using post-tensioned near-surface-mounted (NSM) carbon fiber reinforced polymer (CFRP) strips subjected to corrosion damage. Computational models are developed to predict the behavior of the girders over a 100-year service period. Dynamic analysis exhibits that damage localization takes place, in conjunction with various mode shapes. As the extent of damage rises, the effectiveness of the NSM CFRP increases and the flexural stiffness of the girders decreases.

1. Introduction

Among many detrimental factors affecting the performance of existing bridge structures, corrosion may be a critical attribute because of a reduction in steel area, concrete cover failure, and deteriorated concrete-steel interface [1,2]. On account of the pores in concrete, steel bars in a reinforced concrete member can be exposed to detrimental chemicals such as chlorides [3], which generate multiple stages in steel corrosion: initiation and progression. Recent research on corrosion damage for concrete structures involves deterministic and probabilistic investigations [4-6]. Structural rehabilitation with carbon fiber reinforced polymer (CFRP) composites is promising to upgrade the functionality of existing concrete bridges at affordable costs. Stallings et al. [7] repaired a decrepit bridge using CFRP sheets, which had been in service for over 40 years. The deflection of girders was measured under the operation of known-weight trucks, and was compared against the case without CFRP-strengthening. Badawi and Soudki [8] conducted a laboratory test to examine the behavior of corrosion-damaged concrete beams repaired with CFRP composites. The repair resulted in an increase in the beams' load-carrying capacity. Kasan et al. [9] proposed a strand-splicing repair method in tandem with CFRP sheets. An analytical model was constructed to evaluate the load rating of bridge girders.

A strengthening technique called near-surface-mounted (NSM) CFRP is emerging to enhance the performance of bridges. Along the tensile soffit of a concrete member, grooves are saw-cut to insert CFRP composites that serve as an additional reinforcement to the member. The CFRP-positioned grooves are filled with a structural epoxy. De Lorenzis and Teng [10] states that the NSM method is better than conventional externally bonded CFRP applications in terms of interfacial behavior between the concrete and CFRP. Alkhrdaji et al. [11] carried out a strengthening project for an in-situ bridge deck. The focus was on flexural capacity and failure characteristics. A predictive model was developed to verify the adequacy of test results. Dalfre and Barros [12] performed slab-strengthening
with several NSM CFRP types, and studied crack progression and moment redistribution. NSM CFRP may be post-tensioned to upgrade the efficacy for repair or strengthening.

Although extensive endeavors have been made to investigate a number of technical aspects related to NSM CFRP, scarce information is available on the behavior of bridge members upgraded with post-tensioned NSM CFRP when subjected to corrosion damage. In this research, computational investigations are conducted to examine the time-dependent response of bridge girders strengthened with post-tensioned NSM CFRP strips covering a service period of 100 years. The study approach includes three-dimensional finite element modeling from static and dynamic perspectives, in accordance with the degree of chloride-based corrosion.

2. Model Development

2.1. Bridge girders and CFRP-strengthening

Three benchmark prestressed concrete bridge girders were designed and employed for a numerical study. The girders heights and length varied from 1,750 mm to 2,200 mm and from 25 m to 35 m, respectively, as shown in Fig. 1(a). The specified compressive strength of the girder concrete was $f'_{c} = 40$ MPa. The low-relaxation prestressing tendons (12.7 mm in diameter) were draped and pretensioned at about 65% of $f_{pu}$, where $f_{pu}$ is the tendon’s ultimate capacity of 1,860 MPa with an elastic modulus of $E_p = 190$ GPa. Figure 1(b) illustrates a strengthening method for these girders, which needs the following steps:

a) a narrow groove is cut along the concrete substrate and CFRP is inserted
b) jacking apparatus is mounted to post-tension the CFRP
c) the post-tensioning force is permanently transferred to the girder
d) the groove is grouted to improve aesthetics

The CFRP tendons used had a cross sectional area of $A_f = 700$ mm$^2$ with an ultimate capacity of $f_{fu} = 2,500$ MPa and a modulus of $E_f = 165$ GPa.

The benchmark girders and strengthening method: (a) prestressed concrete girders; (b) proposed strengthening method (patent numbers: 10-0653632, 10-1005347, and 10-1083626)
2.1.1. Static model
A commercial finite element program was employed to predict the behavior of the strengthened and unstrengthened girders (Fig. 2). Shell and link elements represented the concrete girders and the prestressing tendons, respectively. The NSM CFRP was also modeled using the link elements. Perfect bond was assumed between the concrete and reinforcements, so that displacement compatibility was accomplished. The center of gravity of the prestressing tendons was determined and a single representative tendon was included in the model. Initial strains were applied to the CFRP and steel to simulate prestressing effects. Since the live load was in a service condition, material nonlinearity was not necessary in the girder models. Boundary conditions were involved by constraining nodes.

2.1.2. Dynamic model
The static model was expanded to investigate the dynamic characteristics of the bridge girders. Of interest was the analysis for mode shapes and damage localization. The mass of the individual materials was obtained from their densities: concrete = 2,400 kg/m$^3$, steel = 7,810 kg/m$^3$, and CFRP = 1,500 kg/m$^3$. The girders’ modes were acquired by the Block Lanczos method. A sparse matrix solver was exploited to obtain dynamic results. Figure 3 reveals model validation against experimental frequencies reported by Saiidi et al. [13] and Xia and Brownjohn [14]. Figure 3(a) shows the first frequency of laboratory-scale concrete beams (100 mm in width by 125 mm in depth by 3,600 mm in length) consisting of four reinforcing steel bars (284 mm$^2$) and a prestressing strand (99 mm$^2$). The average error of margin between the experiment and model was 6%. The other validation shown in Fig. 3(b) was concerned with a beam-slab assembly having dimensions of 250 mm deep by 1,000 mm wide. Good agreement with an average discrepancy of 19% was observed. It is worth noting that structural members with post-tensioned NSM CFRP were not able to be modeled and assessed, because no experimental work had been conducted previously.
2.1.3. Damage model

Corrosion initiation was estimated by [15]:

\[
t_i = \frac{C^2}{4D_c} \left[ \text{erf}^{-1}\left( \frac{C_{cr} - C_i}{C_i - C_0} \right) \right]^2
\]

(1)

where \( t_i \) is the corrosion initiation time in years; \( C \) is the concrete cover in cm; \( D_c \) is the diffusion coefficient in cm\(^2\)/sec; \( \text{erf} \) is the Gauss error function; \( C_{cr} \) is the critical chloride concentration; \( C_0 \) is the equilibrium chloride concentration on the concrete surface; and \( C_i \) is the initial chloride concentration (\( C_i = 0 \)). In accordance with others’ research [15,16], the following can be taken: \( D_c = 2.0 \times 10^{-8} \) cm\(^2\)/s; \( C_{cr} = 0.4\% \), and \( C_0 = 1.6\% \). The loss of steel area was predicted using Faraday’s law [17,18]:

\[
\phi(t_p) = \phi_0 - r_{corr}t_p \quad \text{with} \quad r_{corr} = C_c \frac{W_{d,corr}}{n}\rho
\]

(2)

\[
i_{corr}(t_p) = \frac{37.8(1 - w/c)^{1.64}}{C} \alpha \beta
\]

(3)

where \( \phi(t_p) \) is the diameter of the prestressing steel strands in mm; \( t_p \) is the time of corrosion in year; \( \phi_0 \) is the initial diameter of the strands in mm; \( r_{corr} \) is the corrosion rate in mm/year; \( C_c \) is a conversion factor (0.00327 for mm/year); \( W_a \) is the atomic weight (55.9 g/mol for steel); \( i_{corr} \) is the corrosion current density in \( \mu\text{A/cm}^2 \); \( n \) is the number of electrons involved (\( n = 2 \) for steel since \( \text{Fe} \rightarrow \text{Fe}^{2+} + 2e \)); \( \rho \) is the density (7 g/cm\(^3\)); \( i_{corr}(t_p) \) is the corrosion density in \( \mu\text{A/cm}^2 \); \( w/c \) is the water cement ratio (\( w/c = 0.4 \) was used); and \( \alpha \) and \( \beta \) are constants (\( \alpha = 1 \) and \( \beta = 0 \) up to one year after corrosion initiation and \( \alpha = 0.85 \) and \( \beta = -0.3 \) beyond the one year). The corrosion of the girders was assumed to be uniform without considering the contribution of the NSM CFRP. This assumption is justified by the fact that the narrow NSM CFRP does not contribute to inhibiting chloride ingress into the girder concrete. The long-term prestress loss of the CFRP was represented by [19]:

\[
f_{\rho} \approx 102.0 - 3.2 \log(t)
\]

(4)

where \( f_{\rho} \) is the normalized CFRP stress at year \( t \).

3. Results

3.1. Long-term behavior of reinforcement

The long-term behavior of the CFRP and steel is provided in Fig. 4. It was predicted that the steel diameter gradually decreased after the initiation of corrosion at 12.8 years. The relatively rapid reduction in steel diameter between 12.8 years and 20 years may be attributed to a rapid reaction in chloride diffusion when the corrosion process began. The variation of the post-tensioned CFRP was not significant for a 100-year service period, including a decrease of approximately 4%.

3.2. Static deflection

Figure 5(a) exhibits the development of girder deflections with service year. For conciseness, the deflections of the control and 100-year cases (\( L = 30 \) m) are only provided for the 30 m girders.
Upon strengthening, the midspan deflection of the control increased by 26% owing to post-tensioning. As the girders were corrosion-damaged, the upward deflection decreased. Irrespective of CFRP-strengthening, the girder deflection induced by live load did not change (Fig. 5(b)).

3.3. Dynamic mode
The five modes of the strengthened girder at 100 years ($L = 35$ m) are shown in Fig. 6. The first mode was in-plane single curvature bending (Fig. 6(a)), whereas the second mode was double curvature bending (Fig. 6(b)). The third mode was also single curvature bending (Fig. 6(c)), followed by lateral sway (Fig. 6(d)) and twisting (Fig. 6(e)).
3.4. Frequency
Figure 7 shows the frequency variation of the strengthened girders. The fundamental frequencies of the strengthened girders increased in comparison with those of the unstrengthened ones owing to the CFRP’s contribution (Fig. 7(a)). This tendency, however, was not apparent as the mode rose (Fig. 7(b)). The individual frequencies of the strengthened girders \((L = 35 \text{ m})\) were normalized by the fundamental frequency and their time-dependent responses were assessed (Fig. 7(c)). At an intact state (service year = 0), the second and third modes were almost the same; however, the fourth and fifth modes were markedly higher. Frequency differences with and without corrosion damage \((L = 25 \text{ m})\) are plotted in Fig. 7(d). The fundamental frequency of the girder showed an increase of 7% at 100 years, while the difference was reduced in higher modes.

4. Conclusions
The long-term response of prestressed concrete bridge girders strengthened with post-tensioned NSM CFRP composites was discussed when subjected to chloride-induced corrosion damage. The length of the girders varied from 25 m to 35 m. To predict the behavior of the girders over a 100-year service period, computational models were developed. The following is concluded:

- When corrosion damage initiated, the diameter of the steel tendons rapidly decreased. The time-dependent CFRP properties were relatively stable, including a 5% reduction in post-tensioning force for 100 years.
The efficacy of the NSM CFRP became more pronounced with an increase in service year. The girders’ fundamental frequency was influenced by corrosion damage within a variation range of 7%.

References
[1] Clifron J R 1993 Predicting the service life of concrete *ACI Mat. J.* 90 611-617.
[2] O’Connor A J, Sheils E, Breysse D, and Schoefs F 2013 Markovian bridge maintenance planning incorporating corrosion initiation and nonlinear deterioration, *J. Bridge Eng.*, 18(3) 189-199.
[3] Dyer T 2014 *Concrete durability*, CRC Press, Boca Raton, FL.
[4] Vu K, Stewart M G, and Mullard J 2005 Corrosion-induced cracking: experimental data and predictive models *ACI Struct. J.* 102(5) 719-726.
[5] Du Y G, Chan A H C, Clark L A, Wang X T, Gurkalo F, and Bartos S 2013 Finite element analysis of cracking and delamination of concrete beam due to steel corrosion, *Eng. Struct.* 56 8-21.
[6] Li S, Xu Y, Li H, and Guan X 2014 Uniform and pitting corrosion modeling for high-strength bridge wires *J. Bridge Eng.* 19(7) 04014025
[7] Stallings J M, Tedesco J W, El-Mihilmy M, and McCauley M 2000. Field performance of FRP bridge repairs *J. Bridge Eng* 5(2) 107-113.
[8] Badawi M and Soudki K 2010 CFRP repair of RC beams with shear-span and full-span corosions *J. Compos. Constr.* 14(3) 323-335.
[9] Kasan J L, Harries K A, Miller R, and Brinkman R J 2014. Repair of prestressed-concrete girders combining internal strand splicing and externally bonded CFRP techniques *J. Bridge Eng.* 19(2) 200-209.
[10] De Lorenzis L and Teng J G 2007 Near-surface mounted FRP reinforcement: an emerging technique for strengthening structures *Compos. Part B* 38 119-143.
[11] Alkhrdaji T, Nanni A, and Chen G 1999. Destructive and non-destructive testing of bridge J857 Phelps County Missouri Technical Report CIES 99-08A.
[12] Dalfre G M and Barros J A O 2013. NSM technique to increase the load carrying capacity of continuous RS slabs *Eng. Struct.* 56 137-153.
[13] Saiidi M, Douglas B, and Feng S 1994 Prestress force effect on vibration frequency of concrete bridges *J. Struct. Eng.* 120(7) 2233-2241.
[14] Xia P -Q and Brownjohn M W 2004. Bridge structural condition assessment using systematically validated finite element model *J. Bridge Eng.* 9(5) 418-423.
[15] Thoft-Christensen P, Jensen F M, Middleton C R, and Blackmore A 1996. Assessment of the reliability of concrete slab bridges 7th IFIP WG 7.5 Working Conf. Boulder CO 1-8.
[16] Stewart M G and Rosowsky D V 1998 Time-dependent reliability of deteriorating reinforced concrete bridge decks *Struc. Safe.* 20 91-109.
[17] Vu K A T and Stewart M G 2000 Structural reliability of concrete bridges including improved chloride-induced corrosion models *Struc. Safe.* 22 313-333.
[18] Ahmad Z 2006 *Principles of corrosion engineering and corrosion control* Butterworth-Heinemann Oxford UK.
[19] El-Hacha R, Wight R G, and Green M F 2003 Innovative system for prestressing fiber-reinforced polymer sheets *ACI Struc. J.* 100(3) 305-313.