

**Seismic Rehabilitation of RC Bridges by Using FRP and SRP:
Case Study of a Bridge in the South of Italy**

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Synopsis: The paper deals with a rehabilitation case study on a pre-stressed concrete (PC) bridge (named “Torrente Casale”), located in the south of Italy (on the Salerno-Reggio Calabria highway). The bridge, built in the '70s, was enlarged in 2001 in order to satisfy the new traffic demand. A seismic assessment of the bridge resulted necessary in order to verify its capacity to sustain both gravity and seismic loads. Both destructive and non-destructive tests have been performed in order to evaluate concrete and steel reinforcement mechanical properties. A theoretical analysis was performed, showing that the bridge piers existing cross section and internal reinforcement were not adequate to satisfy the seismic actions. Thus, two rehabilitation systems were investigated: a) an innovative technique based on the combined use of Fibre Reinforced Polymer laminates (FRP) and Steel Reinforced Polymer spikes (SRP), b) a traditional rehabilitation technique (i.e. RC jacketing). The design assumptions and calculations for the rehabilitation as well as the comparison between the effectiveness of the two investigated strategies are presented and discussed in the paper. The main construction phases of the strengthening technique, executed by following the first outlined strategy are also presented and illustrated.

Keywords: bridge, bridge belt, FRP, PC, RC, seismic rehabilitation, SRP

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INTRODUCTION

Most of western industrialized countries are characterized by highways that are more than 25-30 years old; in particular, a large part of the existing bridges were built after the second world war period or during the construction of the highway network (i.e. 1940 – 1970). They are mostly made by reinforced concrete (RC) or pre-stressed concrete (PC) members; durability problems as well as the poor maintenance, lead to a high demand of restoration and strengthening in order to ensure acceptable safety levels. In some cases rehabilitation systems are necessary as a result of increasing knowledge about the regions seismicity and, consequently, the design seismic actions. Furthermore, in some other cases it results necessary to enlarge the bridge dimensions due to the increased traffic demand. The present paper focuses on an existing bridge built in the '70s in the south of Italy (Salerno-Reggio Calabria highway); a bridge view is reported in Figure 1. In 2001 a new geometrical configuration was designed to supply the increased traffic demand; consequently an enlargement of the carriageway was planned. However, only some parts of the design project were effectively realized on the bridge. In particular, it was foreseen to enlarge the bridge belt by building two new external piers and strengthening the existing piers by RC jacketing. The existing six PC girders were removed and replaced with fourteen new ones; moreover two new external piers, an added part of footing connected to the existing one and two new foundation piles were also added. On the contrary nothing has been done to increase the structural capacity of the existing piers. Thus a seismic assessment of the bridge resulted necessary in order to verify its capacity to sustain both gravity and seismic loads. Since the theoretical analysis showed that the bridge capacity was insufficient with respect to the actual demand, a rehabilitation strategy was investigated. Traditional retrofit techniques (i.e. RC jacketing) were largely used in the past to strengthen bridge structural members, with particular attention to piers bending capacity^{1, 2, 3}. Nowadays the use of innovative strengthening techniques based on the use of Fibre Reinforced Polymer (FRP) are strongly grown^{4, 5, 6, 7} due to their easy and fast installation procedure and durability performances. The present paper deals with the rehabilitation intervention design and execution on the "Torrente Casale" bridge; both a traditional and innovative strengthening strategy was investigated. The assessment of the bridge capacity and the rehabilitation systems were performed by following different steps: 1) analysis of existing documentation related to the bridge (provided by "Autonomous National Company of Roads", ANAS); 2) materials characterization through destructive and non-destructive in situ tests; 3) bridge capacity assessment by using linear static analysis and structural deficiency evaluation; 4) design of two different rehabilitation techniques; based on a traditional RC jacketing or on an innovative technique based on the use of Carbon Fibre Reinforced Polymer laminates (CFRP) and Steel Reinforced Polymer spikes (SRP); 5) selection and execution of the structural rehabilitation.

DESCRIPTION OF THE STRUCTURE

The “Torrente Casale” bridge was built in the early 70’s; it is located at km 318 +175 of the Salerno-Reggio Calabria highway in the south of Italy. It consists of a double lane for each traffic direction. The “north direction” is the carriageway connecting Reggio Calabria to Salerno and the “south direction” is the opposite (see Figure 2). The original bridge belt configuration is reported in Figure 3. The “north direction” and the “south direction” carriageways have been realized by two unconnected decks. Each deck was sustained by three girders (height $H=1400$ mm [55.12 in.] 2500 mm [98.43 in.] spaced). The deck was about 17100 mm [673.23 in.] large. The girders length varied according to the bridge spans dimension; the external spans length were almost 15900 mm [625.98 in.] while the internal span length was about 13400 mm [527.56 in.] (see Figure 2). Bridge belt was characterized by the top beam (17100 mm [673.23 in.] wide), three circular piers (diameter $D=1000$ mm [39.37 in.] and height $H=5550$ mm [218.50 in.]) and a deep foundation system (foundation beam characterized by height $H=1200$ mm [47.24 in.] and width $B=1000$ mm [39.37 in.]; piles characterized by a circular cross section, diameter $D=1000$ mm [39.37 in.]). The piers and foundation piles were spaced at 6000 mm [236.22 in.] each other. The piles total length was equal to 12000 mm [472.44 in.] in order to reach the rock belt. An enlargement intervention was realized in the 2001 to satisfy the increased traffic demand; as a consequence, the geometrical and structural configuration of the bridge changed significantly. In Figure 4 the actual bridge configuration is reported; the original bridge belt configuration is pitted by hatching. In particular, seven PC girders have been placed for each deck on renovated setoffs realized on the top beam. The girder height was $H=1700$ mm [66.93 in.] and they were spaced at 1850 mm [72.83 in.]. The cap beam length changed from 17100 mm [673.23 in.] to 26500 mm [1043.31 in.] of the actual configuration. Furthermore, two external piers (diameter $D=1000$ mm [39.37 in.]) were added in order to ensure the geometrical dimension increase. The new piers were placed on the extension of the foundation beam; two piles (diameter $D=1000$ mm [39.37 in.]) were also added.

PRELIMINARY INVESTIGATION: MATERIALS CHARACTERIZATION

Destructive and non-destructive tests were performed on existing structures to increase the level of knowledge of the structural material mechanical characteristics and to properly model the structure. Non-destructive tests were performed on the bridge belt in order to assess the longitudinal and transversal steel reinforcement layout as well as the bars diameter. By using a pacometer, it resulted that transverse reinforcement on existing piers was made by 8 mm [0.32 in.] spiral bars spaced at 200 mm [7.87 in.], while 16 mm [0.63 in.] diameter bars were used as longitudinal reinforcement. The external added piers are characterized by 16 mm [0.63 in.] longitudinal bars diameter and transversal 12 mm [0.47 in.] spiral bars spaced about 150 mm [5.91 in.]. Bars of 28 mm [1.10 in.] diameter were found as longitudinal reinforcement of existing cap beam, while bars 26 mm [1.02 in.] diameter were used as longitudinal reinforcement during the construction of the cap beam enlargement. Two separate non-destructive tests were planned to evaluate the concrete mechanical properties on both existing and added piers by using sclerometric test (type Schmidt N/NR) and sonic test. The compressive strength, f_c , obtained by such tests are summarized in Table 1 and Table 2. The SONREB method was also adopted to take into account both rebound index (RI) for sclerometric tests and sonic waves propagation (V_{dir}) for sonic tests. In particular, three expressions were adopted:

$$- f_c = 9,27 \cdot 10^{-11} \cdot V_{dir}^{2,6} \cdot RI^{1,4} \quad (\text{RILEM formula}) \quad (1)$$

$$- f_c = 8,06 \cdot 10^{-8} \cdot V_{dir}^{1,85} \cdot RI^{1,246} \quad (\text{GASPARIK formula}) \quad (2)$$

$$- f_c = 1,2 \cdot 10^{-9} \cdot V_{dir}^{2,446} \cdot RI^{1,058} \quad (\text{DI LEO – PASCALE formula}) \quad (3)$$

The compressive strength obtained by using such expressions are summarized in Table 3. The compressive strength provided by non destructive tests were, especially for existing piers, very different from those expected by available design information (i.e. compressive strength equal to 20.75 MPa [296.4 psi] and 29.05 MPa [415 psi] for existing and added piers, respectively). This is probably due to the carbonation phenomenon, the absence of binder and higher maximum diameter of the rubble as showed in Figure 5. Thus the high values of compressive strength computed by using the non destructive tests lead destructive tests to became of primary importance. Hence, a program of destructive tests was planned on the bridge structural elements: existing and added piers, and on existing and added cap beam. Cylindrical samples were obtained from coring with a diameter – height ratio (D/H) equal to 1.00. The cores were extracted from the added and existing piers at different position and from the added portion of cap beam. Compression tests on cylindrical concrete specimens were performed according to the procedures outlined in UNI 6131:2002⁸ and UNI EN 12504-1:2002⁹. The average concrete compressive strength, f_{cm} , was computed starting from experimental results on each core, by using the following expressions:

$$f_c = \frac{f_{test} \cdot D}{1.5 + \frac{1}{\lambda}} \quad \text{and} \quad f_{cm} = \frac{\sum_{i=1}^n R_c}{n} \quad (4)$$

where f_{test} is the coring compression strength, D (about 94 mm [3.70 in.]) is the coring diameter, λ is the ratio between the coring geometrical properties (D/H) and n is the number of specimens. The results are summarized in Table 4 and Table 5. The average values of cylindrical compressive strength related to added and existing piers and to the new part of the cap beam are reported in Table 6. In the theoretical assessment of bridge capacity, the concrete compressive strength and steel yielding strength introduced into the structural model must be modified taking into account the construction knowledge level (KL). The level of knowledge (KL) is related to the Confidential Factor (CF). According to Eurocode 8 provisions and available data the KL2 was considered for the analysis of the bridge and consequently $CF=1.2$ was assumed in the calculations. Based on such assumptions, design concrete and steel strength, f_{cd} and f_{yd} , were computed as follows:

$$f_{cd} = f_{cm} \cdot \frac{1}{CF} \quad (\text{concrete}) \quad \text{and} \quad f_{yd} = f_y \cdot \frac{1}{CF} \quad (\text{steel}) \quad (5\text{-A: ductile structural element behavior})$$

$$f_{cd} = \frac{f_{cm}}{\gamma_c} \cdot \frac{1}{CF} \quad (\text{concrete}) \quad \text{and} \quad f_{yd} = \frac{f_y}{\gamma_s} \cdot \frac{1}{CF} \quad (\text{steel}) \quad (5\text{-B: brittle structural element behavior})$$

where f_{cm} is the concrete strength according to destructive test results, γ_i are the material factors derived from Eurocode 2, CF is the confidential factor and f_y is the yielding stress based on the steel type. Design values adopted in the structural modelling are reported in the last columns of the Table 6 and Table 7.

BRIDGE CAPACITY ASSESSMENT

According to the reached knowledge level a numerical analysis was implemented to verify the bridge capacity; in this particular case the linear static analysis is adequate for the aim. The theoretical bridge belt capacity was assessed by using the finite element analysis program SAP2000¹⁰, very commonly used by structural engineering practitioners. Thus, after defining the material mechanical properties, the bridge structural model and the acting loads were defined. The horizontal structural elements (i.e. PC girders) were replaced during the bridge enlargement and they were designed in order to stand the new traffic loads and structural demand increasing. They were made according to the design and thus their bending capacity assessment is out of the scope of the present paper. On the other hand, the rehabilitation technique of existing piers was not conforming to design provisions; therefore, the analysis has been focused on the bridge belt structural performances. Linear static analysis was implemented to analyze the bridge belt because of its structural scheme (cantilever scheme when longitudinal seismic actions are considered and frame scheme when the transverse seismic actions are analyzed); indeed the girder are not embedded on the cap beam and thus it is possible to model the vertical structural element only by applying due to horizontal structural members and no-bearing elements. The bridge belt geometrical model is reported in Figure 6; the structure-soil interaction has been modelled applying elastic spring one meter spaced along the foundation piles. The elastic constant of the spring into the model is placed equal to the soil Winkler constant k , generally accepted as a main soil characteristics, equal to 20000 t/m³ [13449685.73 tons/in.³]. The loads acting on the structure have been assessed according to European design rules by means of Eurocode 2¹¹ and Eurocode 8¹². Sixteen load combinations were computed in order to have the worst condition in terms of stresses and displacements of the bridge structural model. In order to evaluate the belt capacity under longitudinal and transversal seismic action (direction of vehicular flow and orthogonal to vehicular flow) two different models were adopted as reported in Figure 7 and Figure 8. For longitudinal seismic actions the bridge belt was modeled as a one degree of freedom system for which a mass m_1 was assigned; the mass m_1 (equal to 50.30 t/g [55.10 tons/g]) has been evaluated considering the masses of PC girders and decks connected to the bridge belt by fixed constrains; the structural elements linked to the bridge belt by sliding constrains were not taken into account. In the case of transversal earthquake actions, the mass m_2 (equal to 47.30 t/g [51.82 tons/g]) is computed by applying on the bridge belt both half masses of PC girders and decks on the left and on the right of the vertical structure, without considering any kinematics changing between sliding and the fixed constrain (Figure 8). Indeed, for transversal seismic direction the sliding constrains have been hypothesized equal to fixed constrains in terms of cinematic characteristics. In order to compute the seismic loads acting on the structure, an associated spectral acceleration must be taken into account by evaluating the design response spectra, using the structural factor, q . The structural factor, q , was computed by

evaluating local and global ductility. The structural factor q of the existing structure was evaluated through the structural ductility calculated in terms of displacement. To compute the q factor a structural model was adopted considering the bridge belt loaded by the structural elements self-weight only. In particular, the local ductility is computed with reference to the base piers cross section, where the plastic hinge formation is expected.

The local ductility, μ_χ , is given by the ratio between the ultimate curvature, $\chi_u = \varepsilon_{cu}/x_{cu}$ and the yield curvature, $\chi_y = \varepsilon_{sy}/d - x_c$ of the pier cross section (ε_{sy} is steel yield strain, d is the effective cross section height and x_c and x_{cu} are the neutral axis at yielding and ultimate respectively and ε_{cu} is ultimate concrete strain equal to 0.0035). According to Eurocode 2¹¹, the global ductility can be applied equal to q factor, if the vibration period of the main structure is higher than T_c (that is the period of the elastic response spectra at the end of the constant branch), as occurred on the present case. The global ductility is a function of local ductility, μ_χ , by using the following expression:

$$\mu_\delta = 1 + 3(\mu_\chi - 1) \cdot \frac{l_{pl}}{L} \cdot \left(1 - 0,5 \cdot \frac{l_{pl}}{L} \right) = 1 + 3(4,63 - 1) \cdot \frac{0,96}{6,00} \cdot \left(1 - 0,5 \cdot \frac{0,96}{6,00} \right) = 2,60 \quad (6)$$

where μ_δ is the global ductility in terms of displacements, μ_χ is the ductility in terms of curvature, l_{pl} is the plastic hinge length taken equal to d and L is the column length. Thus, in order to consider the worst condition in terms of stresses and displacements on the structure for seismic load combinations, in a conservative manner the q factor value was assumed equal to 2.50. The design response spectra computed by using the q factor allowed the associated seismic accelerations for both longitudinal ($a_l = 0.98 \text{ m/sec}^2$) and transversal ($a_t = 2.19 \text{ m/sec}^2$) directions to be computed. Then multiplying the masses and the accelerations, the seismic actions were computed ($F_i = a_i \times m_i$). Thus, at the end of the computations, two different seismic actions for longitudinal and transversal directions were assumed equal to $F_l = 49.20 \text{ t}$ [54.22 tons] and $F_t = 110.10 \text{ t}$ [121.33 tons], respectively.

EXISTING STRUCTURE

The structural analyses of the existing bridge belt indicate that a rehabilitation technique was necessary in order to make the structure safe. Several deficiencies both in terms of bending and shear capacity of piers and cap beam were detected respectively. In particular the results showed that for gravity load combinations the bridge reached an acceptable safety level according to Eurocode 8¹². On the contrary, the main structural problems are related to seismic load combinations in both longitudinal and transversal directions. The interaction domains point out the flexural capacity deficiency at the ends of the existing piers on both bottom and top cross sections. For instance, the modelling results have shown the need to increase the strength capacity by a factor of 25% and 18% at the bottom section and at the top section of the existing piers, respectively. The shear demand was higher than capacity on the added part of the cap beam (i.e. 2001) due to gravity loads combined to vehicular flow as showed in Figure 11 (the strength capacity need to be increased by a factor of about 11%).

REHABILITATION STRATEGIES

Once the structural deficiencies were assessed, two different rehabilitation strategies were investigated in order to reach a good safety level for the whole structure: a) an innovative technique based on the combined use of Carbon FRP (CFRP) laminates and SRP spikes, b) a traditional rehabilitation system (RC jacketing).

Innovative rehabilitation system

The innovative rehabilitation technique is based on the combined use of CFRP laminates and SRP spikes to increase the structural performances under both gravity and seismic loads; this rehabilitation technique does not require the changing of the structural model in terms of geometry, elastic stiffness and seismic actions. The design was performed according to CNR DT 200/2004¹³. In particular, SRP spikes are used to increase the bending capacity of RC piers at the ends, without enlarging their cross section, while CFRP wrapping is used to avoid existing steel vertical reinforcement and SRP spikes buckling as well as to improve the concrete compressive strength owing to the passive confinement generated by the FRP jacket as the concrete dilates under axial, shear and moment load combination. Z-3x2-Zinc SRP spikes were used; the 3X2 symbol indicates the number of wires making up the single "cord". The cord is composed by three wires wrapped by other two wires of the same diameter equal to 0.35 mm [0.014 in.]. The asymmetrical shape of the Z-3x2-Zinc SRP allows to reach excellent fatigue strength due to tension and high bending stresses. According to the manufacturer data¹⁴, the mechanical properties of the individual cord are characterized by modulus of elasticity of 184000 MPa [26686936.8 psi], tensile strength of 3070 MPa [445265.8 psi] and ultimate strain equal to 1.70% (see Table 8).

To assess the SRP spikes contribution on the cross section strength, the following expressions have been used. An equivalent steel area has been computed starting from the cords area; the latter was assumed equal to 0.48mm^2 [0.019 in.^2], as listed in the manufacturer data. 8.7 cords are present in one centimetre of strip:

$$\begin{aligned} Area_{(cords)} \cdot n^{\circ}_{cord} &= \frac{Area_{(SRP)}}{cm} & [\quad Area_{(cords)} \cdot n^{\circ}_{cord} &= \frac{Area_{(SRP)}}{in.} \quad] \\ 0.48\text{mm}^2 \times 8.7 &= 4.176 \frac{\text{mm}^2}{cm} & [\quad 0.000744\text{in}^2 \times 8.7 &= 0.00647 \frac{\text{in}^2}{in} \quad] \end{aligned} \quad (7)$$

As design hypothesis, an SRP wide strip around 70 mm [2.76 in.] was chosen corresponding to an equivalent steel area of:

$$\begin{aligned} L_{strip} \cdot \frac{Area_{(SRP)}}{cm} &= Area_{equivalent} & [\quad L_{strip} \cdot \frac{Area_{(SRP)}}{in.} &= Area_{equivalent} \quad] \\ 4.176 \times 7 &= 29.23\text{mm}^2 \cong 1\phi6 = 0.28\text{cm}^2 & [\quad 0.00647 \times 2.76 &= 0.0179\text{in.}^2 \cong 1\phi6 = 0.0434\text{in.}^2 \quad] \end{aligned} \quad (8)$$

The SRP spikes were located in appropriate holes realized drilled the beam foundation (pier bottom cross-section) and the cap beam (pier top cross-section) ensuring a bond length equal to 300 mm [11.81 in.]. The bond length was computed according to the following equation (extracted from EC2¹¹):

$$\begin{aligned} L_{bd} &= \frac{\phi}{4} \cdot \frac{f_{fd}}{f_{u,res}} \\ L_{bd} &= \frac{\phi}{4} \cdot \frac{550,0\text{MPa}}{3,0\text{MPa}} = 45.83 \cdot \phi = 275\text{mm} & L_{bd} &= \frac{\phi}{4} \cdot \frac{79770.74\text{psi}}{435.12\text{psi}} = 45.83 \cdot \phi = 275\text{mm} [10.83\text{in.}] \end{aligned} \quad (9)$$

where ϕ is the SRP diameter, f_{fd} is the SRP delamination strength and $f_{u,res}$ is the epoxy resin tensile strength. Sixteen spikes were used, placed around the piers cross section inside the holes before mentioned (see Figure 12). SRP spikes were supposed to be applied on the pier bottom cross-sections starting from the foundation beam to 770 mm [30.32 in.]. Such a length was computed in order to fully cover the critical zone, where demand is higher than capacity, equal to a height of 450mm [17.72 in.] from the foundations. Adopting a bond length equal to 300 mm [11.81 in.], a total length was assumed equal to 1070 mm [42.12 in.]. The same configuration was adopted for pier top cross-sections, starting from the cap beam. CFRP sheets were applied in order to prevent SRP spikes buckling due to cyclical loads. Considering that the bridge is exposed to aggressive environment, durability issue is the driving design criterion and thus, carbon fibres have been selected and preferred to glass fibres. In particular, uniaxial CFRP laminates characterized by 900 gr/m² density, 0.13 mm/ply [0.005 in./ply] dry fibres thickness, modulus of elasticity about 230000 MPa [3285714 psi] and tensile strength equal to 4830 MPa [69000 psi] were used. Two CFRP layers were adopted to wrap the piers ends; each layer was applied by using two sheets (height of 400 mm [15.74 in.] each) with an overlap length of 30 mm [1.18 in.], see Figure 13. They were designed in order to avoid the existing steel vertical reinforcement and SRP buckling, improve the concrete behavior and to avoid the slip of existing steel reinforcement in RC piers at the locations of lap splice (note that the overlap between the foundation beam reinforced bars and the pier reinforced bars was equal to 650mm [25.59 in.]). The number of CFRP layers used to confine the member cross sections was computed taking into account the buckling of longitudinal bars, by applying the CNR¹³ formula, reported in the following:

$$t_f \cong \frac{10 \cdot n \cdot d}{E_f} = \frac{10 \cdot 5 \cdot 1000}{230000} = 0,22\text{mm} [0.009\text{inc.}] \quad (8)$$

where n represents the total number of steel vertical reinforcement subjected to buckling (highlighted in the fig.13), d represent the size of cross section parallel to the bending plane and E_f is the CFRP Young's modulus. The use of SRP spikes wrapped by CFRP laminate lead to an increase of piers bending capacity at the ends equal to 40%, almost 10% higher than the existing cross-section capacity gap (see Figure 14). CFRP sheets were also used to increase the cap beam shear capacity; they were applied by using four U-shaped sheets characterized by 300 mm [12.00 in.] width and a total length equal to 3700 mm [145.67 in.] (see Figure 15). Three layers of CFRP U-wrap were installed on the top beam (Figure 15). This strengthening configuration provides 12% shear strength increase, reaching an acceptable safety level if compared to the shear capacity increase target equal to 11%.

Traditional strengthening system

The aim of the traditional rehabilitation strategy was to increase the bending strength of both existing and added piers of the existing structure. Such a goal can be achieved by means of RC jacketing system preserving the original piers cross section (circular cross section); the original cross-section piers pass from diameter of 1000 mm [39.37 in.] to 1120 mm [44.09 in.]. A concrete design strength equal to $f_c = 29.05$ MPa [415 psi] was assumed in the calculations. The longitudinal reinforcement of the jacketed piers was designed as 16 bars 20 mm [0.79 in.] diameter and 10 mm [0.39 in.] stirrups spaced at 250 mm [9.85 in.]. Steel bars and stirrups design strength were $f_y = 430$ MPa [6143 psi]. The added bars were anchored in the cap beam and foundation beam in order to get the existing part and the added safe collaborating one. The thickness of the cross section added part was computed in order to insert the longitudinal steel bars and spiral bars in the correct position thus preserving their integrity from the outside environment (Figure 16). The added structural elements (piers with RC jacketing) assessment in terms of strength and ductility was performed considering three main hypotheses: a) the jacketed elements should behave as a monolithic material, providing full adhesion between the existing concrete and added one (Figure 16), b) the axial load is applied to overall jacketed section and c) the concrete mechanical properties for the whole structure is assumed equal to the lowest between the added and existing one. To model the RC jacketed structure it was necessary to consider that, as a consequence of the piers enlargement, the bridge masses change (masses increase of about 25%); thus higher values of lateral forces have to be taken into account. The design rehabilitation based on the jacketed technique led to a bending strength increase of about 37%, which is higher than the target increase equal to 25% (see Figure 17). The shear strengthening of the top beam was achieved by increasing the cross-section, assuring the structural collaboration between the added element and existing one by using 10 mm [0.39 in.] diameter steel bars mechanical connectors (see Figure 18). The cross-section enlargement leads to shear strength increase of about 21%.

REHABILITATION SYSTEMS COMPARISON

The structural analysis results confirmed the effectiveness of the two rehabilitation methods investigated. In particular, the rehabilitation strategy based on the use of composite material increases the global bridge strength without affecting the structural stiffness and the masses involved. Thus, the seismic actions did not change compared to those of unreinforced structure. For that reason, this strengthening does not involve the foundation of the bridge, being subjected to the actions computed for the “as built” structure. In addition, the installation procedure is fast and easy and can be adopted also when time or space restrictions are the design driving criteria.

The traditional rehabilitation system provides a significant structural mass increase and, consequently, the seismic actions to be computed are higher. As a drawback, such technique may results much more invasive and difficult from constructability standpoint with a lengthy disruption of the structure function. As summary, the innovative rehabilitation system performs advantages as reported in the following: a) it provides a faster application and less invasive intervention on the structure, leading to costs reduction; b) closure of the vehicular traffic is not required; c) CFRP choice ensures a good strengthening durability also taking into account the aggressive environment conditions, typical of bridge structures. Based on these reasons it was decided to strengthen the bridge by using innovative rehabilitation strategy.

REHABILITATION INTERVENTION BY USING INNOVATIVE MATERIALS

The innovative solution was selected for this particular case study; the main steps installation procedures are reported in the following. Prior to laminates installation, unsound concrete was removed in all zones of the elements where crushing was detected; then the original cross-sections were restored using a non-shrinking mortar. In addition, all the holes caused by destructive tests realized on the bridge were epoxy-injected. Then, according to the design provisions, the holes were made at the ends of the piers by using a suitable hammer, reaching an appropriate hole deep (see Figure 19-a). Once primer was applied on the pier surfaces (see Figure 19-b) and the pier holes were filled by using epoxy mortar (see Figure 19-c). The SRP strips total length is equal to 1150 mm [45.28 in.]; 380 mm [14.96 in.] of the SRP strips total length were rolled at the end in order to realize the spikes that were put into the holes. The free parts of the SRP strips were rested on the ground (see Figure 19-c). An epoxy mortar layer was applied on the piers surface and the free parts of the SRP were placed on them, ensuring the adhesion between the mortar and the SRP strips; moreover the direct interaction between the SRP and CFRP was avoided in order to prevent corrosion phenomena (see Figure 19-d). Then, an epoxy resin layer was applied on the piers (Figure 19-e) and the first CFRP ply was wrapped on the piers surface (Figure 19-f). This procedure was repeated for the second layer. Another layer of mortar was applied on the last laminate coat in order to protect the rehabilitation system from environmental effects. Typical FRP installation procedure was made to realize the shear strengthening system, by rounding the corner to avoid stresses concentration. The procedure was repeated to install three FRP sheet layers.

CONCLUSIONS

Most of the bridges built in Italy between the '60s and '70s were designed considering gravity loads only. Current design codes have introduced new rules to evaluate the seismic actions combined with other loads. As a consequence of the new design rule requirements, several existing bridges need to be strengthened in order to reach an acceptable safety level. The paper deals with an RC bridge seismic rehabilitation for which two different strategies based on the use of innovative materials or on a traditional techniques have been evaluated. A structural model of the existing bridge was developed and two rehabilitation solutions were analysed.

Both investigated strengthening systems resulted effective to ensure a global acceptable safety level on the bridge. In particular, in the case study presented:

- RC jacketing could increase the piers bending capacity of about 37% (25% requested by the structural analysis) and the cap beam shear strength of about 21% ;
- The combined use of SRP spikes and FRP laminates could increase the piers bending capacity of about 40% and the cap beam shear strength of about 12%.

The first rehabilitation technique is realized by increasing the cross-section of the structural element, leading to a stiffness increase and changing the masses involved into the calculation of the bridge. Hence, the seismic actions acting on the structure are higher than those computed for the existing structure and the foundation system could be subjected to higher actions. The execution of the traditional rehabilitation technique appears difficult because it demands invasive methods to strengthen the foundation system and the structural element, supposing to close the vehicular traffic on the bridge and using particular machinery. All these conditions lead to increase the time to realise it and thus, the costs. On the other hand, the latter technique, based on innovative materials, does not change the stiffness of the structure and structural element masses. The innovative strengthening technique could be done without closing the traffic on the bridge. The rehabilitation case study presented herein was completed within two weeks. About calculations, the same finite element model was used to analyse the existing and the strengthening structures. Thus, the same actions by means of gravity and seismic combination loads were applied on the structural model, leading to perform the strengthening system, reducing the time. The costs-benefits analysis highlighted that the strengthening system based on SRP spikes and CFRP laminates ensure a performances level higher than those offered by the traditional one.

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Table 1—Rebound Index and sclerometric test results

Pier	Rebound Index - RI	f_{ck} - Mpa [psi] test result
# 1	37,8	27.40 [391.44]
# 2	58,5	>49.00 [700.00]
# 3	57,8	>49.00 [700.00]
# 4	56,8	>49.00 [700.00]
# 5	38	27.40 [391.44]

Table 2—Sonic test results

Pier	d - cm [in.]	V - km/sec [miles/sec]
# 1	40.0 [15.5]	4.18 [2.60]
# 2	40.0 [15.5]	3.80 [3.36]
# 3	40.0 [15.5]	3.88 [2.41]
# 4	40.0 [15.5]	3.85 [2.39]
# 5	40.0 [15.5]	4.13 [2.57]

V - km/sec [miles/sec]	Concrete Quality
>4,5 [2.80]	Very Good
3,5 - 4,5 [2.18 - 2.80]	Good
3,0 - 3,5 [1.86 - 2.18]	Uncertain
2,0 - 3,0 [1.24 - 1.86]	Bad
< 2,0 [<1.24]	Very Bad

Table 3—Non destructive test results

Pier	Rilem Formula f_{ck} – Mpa [psi] (1)	Gasparik Formula f_{ck} – Mpa [psi] (2)	Di Leo – Pascale Formula f_{ck} – Mpa [psi] (3)
# 1	25.97 [16.14]	25.48 [15.83]	28.80 [17.90]
# 2	46.48 [28.88]	44.65 [27.75]	42.08 [26.15]
# 3	48.39 [30.07]	45.82 [28.47]	43.82 [27.23]
# 4	46.23 [28.73]	44.16 [27.44]	42.16 [26.20]
# 5	31.54 [19.60]	30.46 [18.93]	32.70 [20.32]

Table 4—Concrete compressive strength of existing piers

Pier	Height of coring	Cores dimension (mm) [in.]		f_c MPa [psi]	f_{cm} N/mm ² [psi]
		D	H		
# 2	h=5000 mm [196.85 in.]	93.45 [3.68]	93.08 [2.93]	22.10 [315.70]	29.52 [421.70]
# 3	h=1000 mm [39.37 in.]	93.23 [3.67]	93.04 [3.66]	38.00 [542.90]	
# 4	h=3000 mm [118.11 in.]	93.54 [3.68]	93.50 [3.68]	34.00 [485.70]	
# 7	h=5000 mm [196.85 in.]	93.54 [3.68]	93.55 [3.68]	24.50 [350.00]	
# 8	h=1000 mm [39.37 in.]	93.15 [3.67]	93.08 [3.66]	29.70 [424.30]	
# 9	h=3000 mm [118.11 in.]	93.69 [3.68]	93.03 [3.66]	28.80 [411.50]	

Table 5—Concrete compressive strength of added piers

Pier	Height of coring	Cores dimension (mm) [in.]		f_c MPa [psi]	f_{cm} N/mm ² [psi]
		D	H		
# 1	h=5000 mm [196.85 in.]	94.10 [3.70]	98.80 [3.88]	20.00 [285.70]	36.08 [525.70]
# 1	h=1000 mm [39.37 in.]	94.20 [3.70]	94.20 [3.70]	41.00 [585.70]	
# 1	h=3000 mm [118.11 in.]	94.20 [3.70]	93.10 [3.66]	38.50 [550.00]	
# 6	h=5000 mm [196.85 in.]	94.20 [3.70]	91.50 [3.60]	40.50 [578.60]	
# 6	h=1000 mm [39.37 in.]	94.20 [3.70]	93.70 [3.68]	36.50 [521.50]	
# 6	h=3000 mm [118.11 in.]	94.20 [3.70]	93.60 [3.68]	40.00 [571.40]	

Table 6—Mechanical properties of Concrete

Concrete	f_{cm} -Mpa [psi]	CF	γ_c	f_{cd} – brittle- Mpa [psi]	f_{cd} – ductile- Mpa [psi]
<i>Added piers</i>	36.08 [525.70]	1.20	1.50	20.04 [286.28]	24.05 [343.56]
<i>Existing piers</i>	29.52 [421.70]	1.20	1.50	16.40 [234.28]	19.68 [281.14]

Table 7—Mechanical properties of Steel

Bars	f_{ym} -Mpa – [psi]	CF	γ_s	f_{yd} – brittle - Mpa [psi]	f_{yd} – ductile - Mpa [psi]
<i>Added piers</i>	440.00 [6285.70]	1.20	1.15	318.84 [4554.85]	382.61 [5465.84]
<i>Existing piers</i>	380.00 [5428.60]	1.20	1.15	275.37 [3933.85]	330.44 [4720.56]

Table 8—Mechanical properties of Innovative materials (CFRP and SRP)

Material	E_f - Mpa – [psi]	σ_{fu} – Mpa – [psi]	ϵ_{fu} - % -
<i>CFRP</i>	230000.00 [3285715.00]	4830.00 [69000.00]	2,10%
<i>Z – 3x2</i>	184000.00 [2628571.00]	3070.00 [43857.10]	1,70%



Fig. 1—“Torrente Casale” bridge, on the Salerno-Reggio Calabria Highway

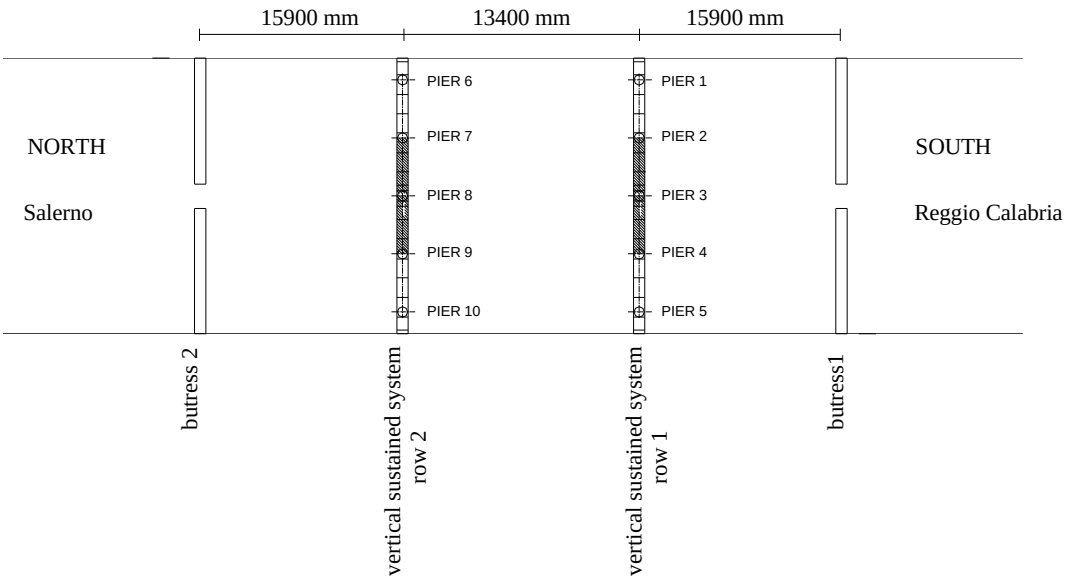


Fig. 2—Bridge plan view

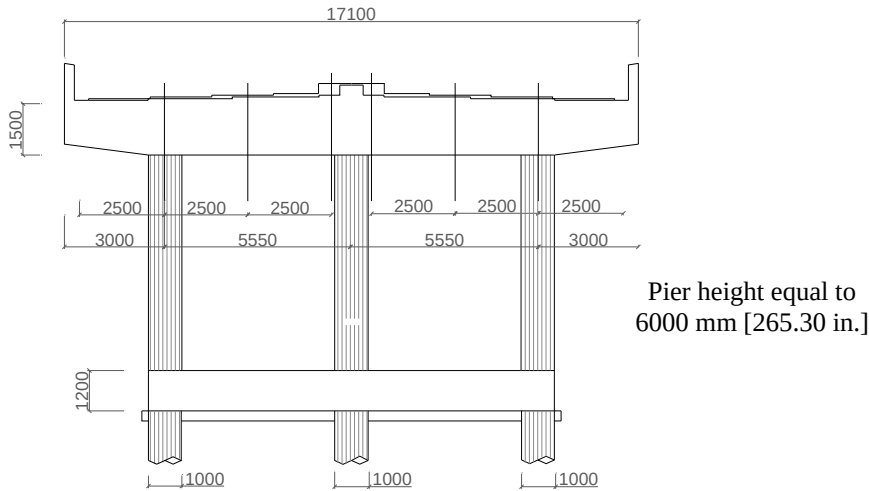


Fig. 3—Original bridge belt configuration

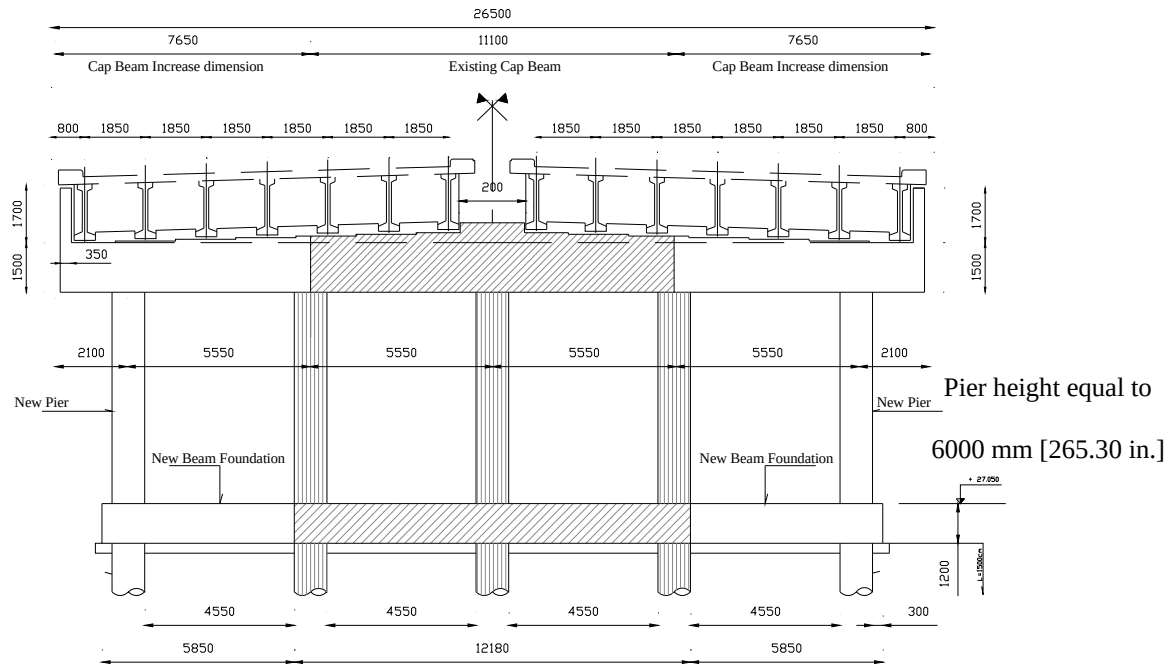


Fig. 4—Actual bridge belt configuration after enlargement interventions, (i.e. 2001)



Fig. 5—Carbonation phenomena, absence of binder and higher maximum diameter of the rubble

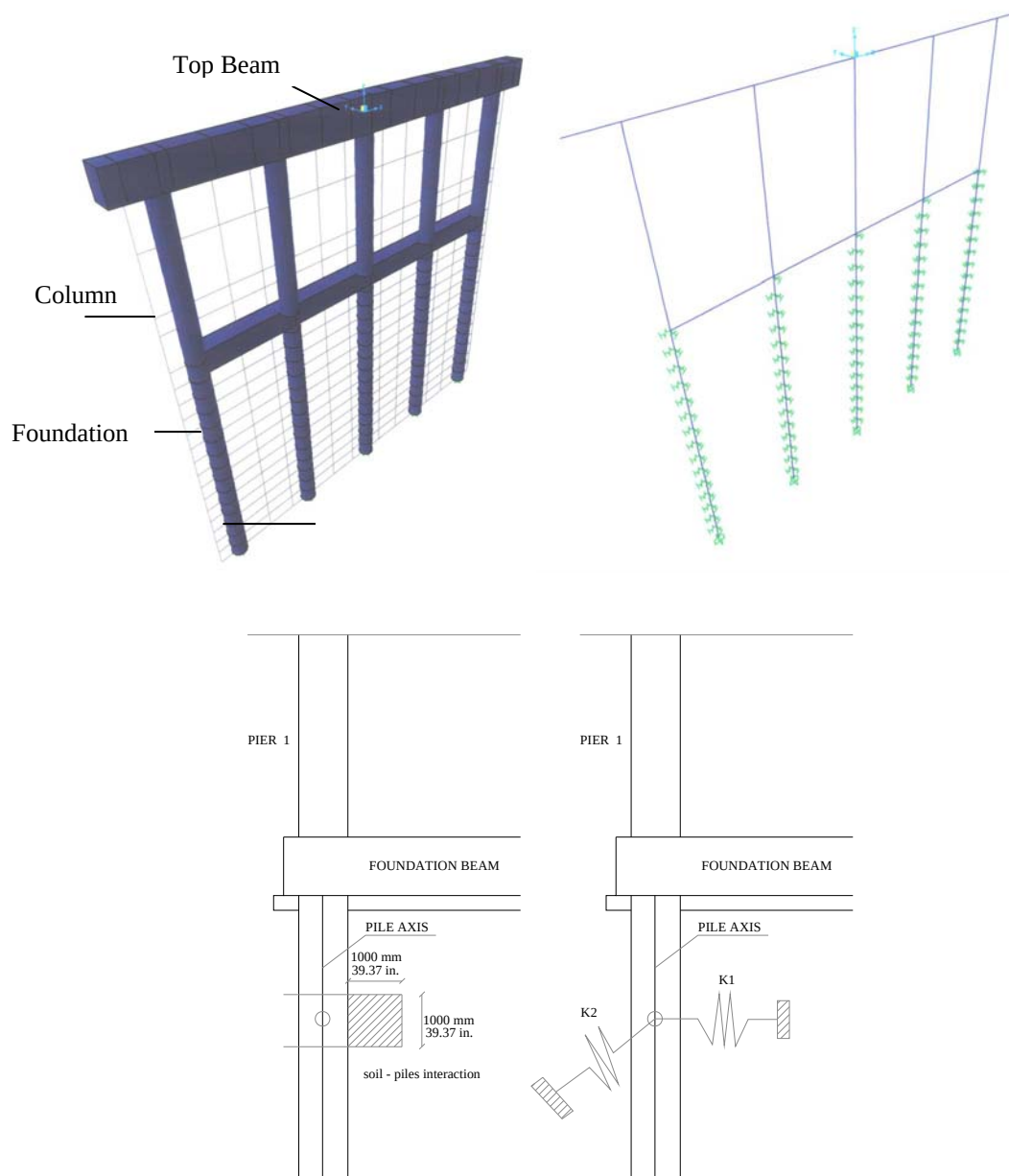


Fig. 6—Bridge belt geometrical modelling

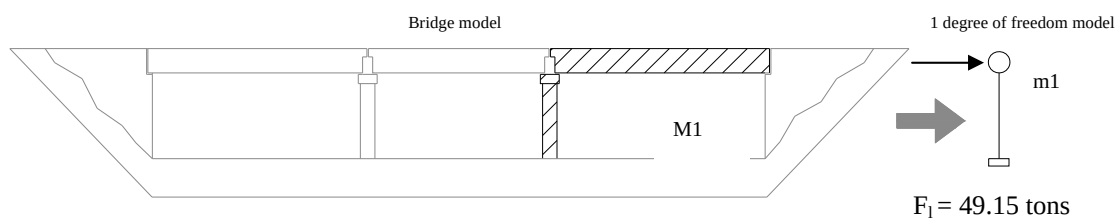


Fig. 7—Longitudinal earthquake bridge belt modeling

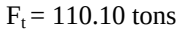


Fig. 8—Transversal earthquake bridge belt modelling



Fig. 9—Different constraints between Girders and Cap Beam

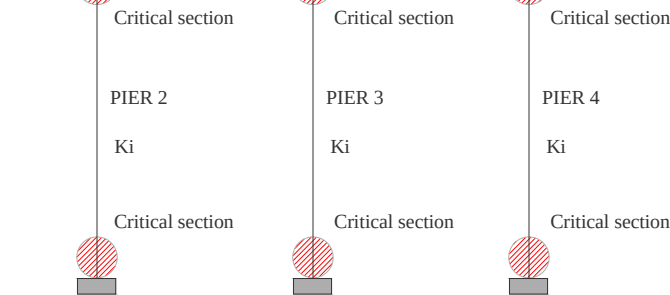


Fig. 10—Critical cross sections for bending loads

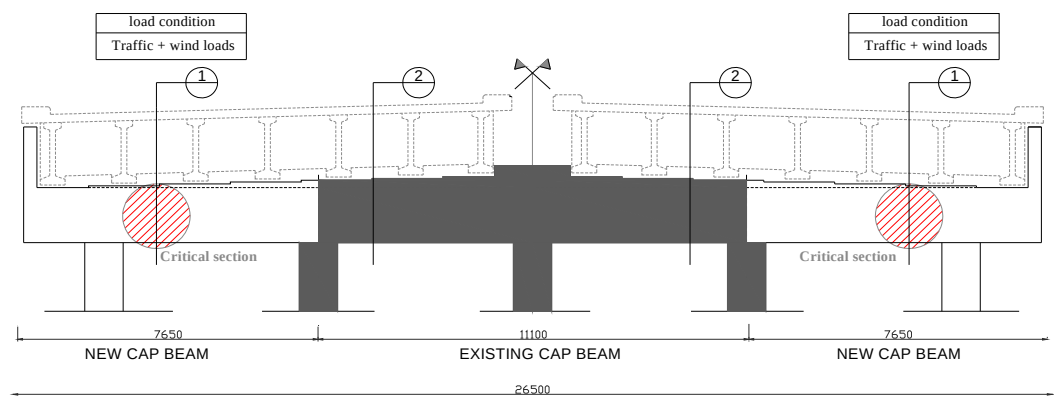


Fig. 11—Critical cross sections for shear loads

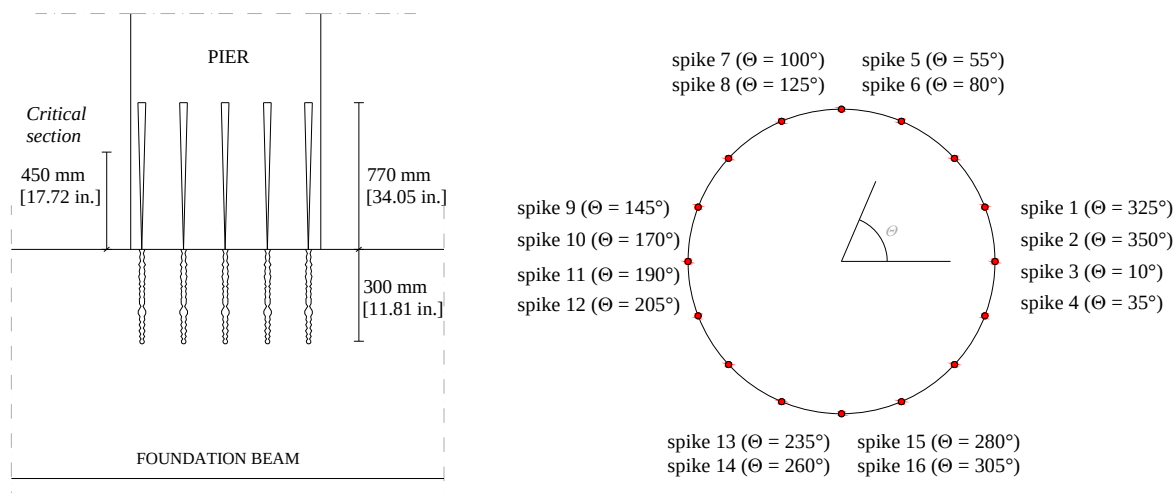


Fig.12—SRP spikes application

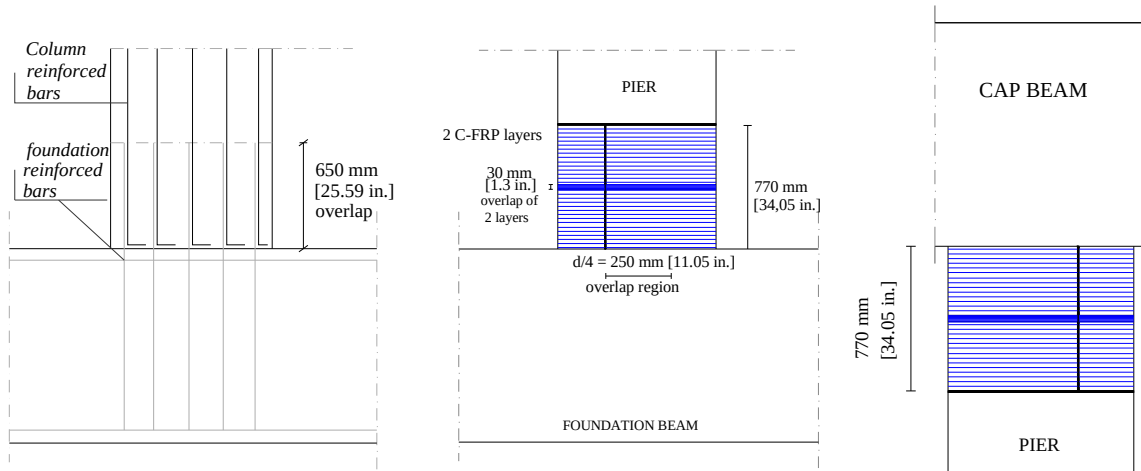


Fig.13—C-FRP wrapping application

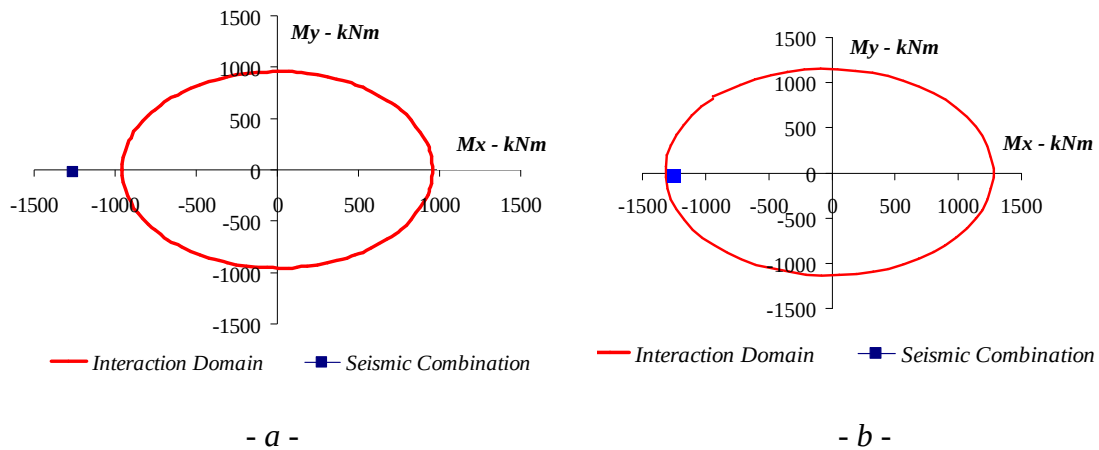


Fig.14—Interaction domain of central piers:

a) Initial cross-section condition; b) SRP reinforced cross section

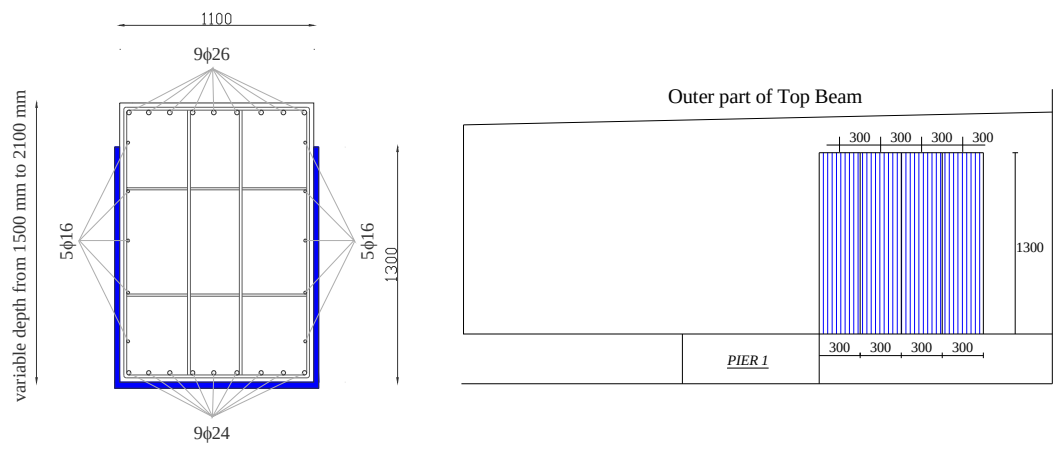


Fig.15—C-FRP shear strengthening configuration

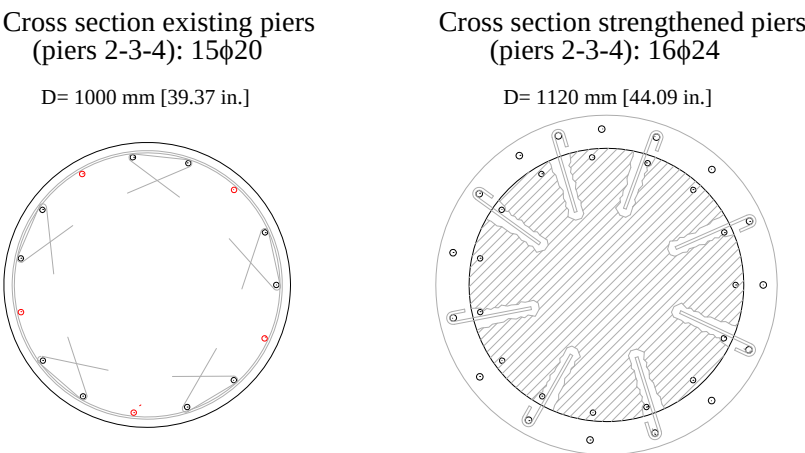


Fig. 16—Bending strengthening for existing piers

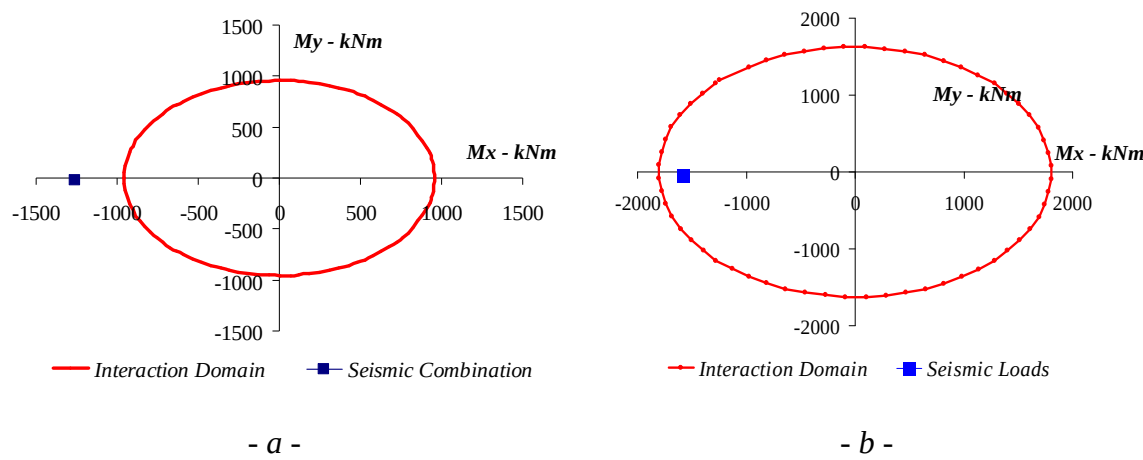


Fig.17—Domain of resistance of central piers:

a) Initial cross-sections; b) SRP reinforced cross-section

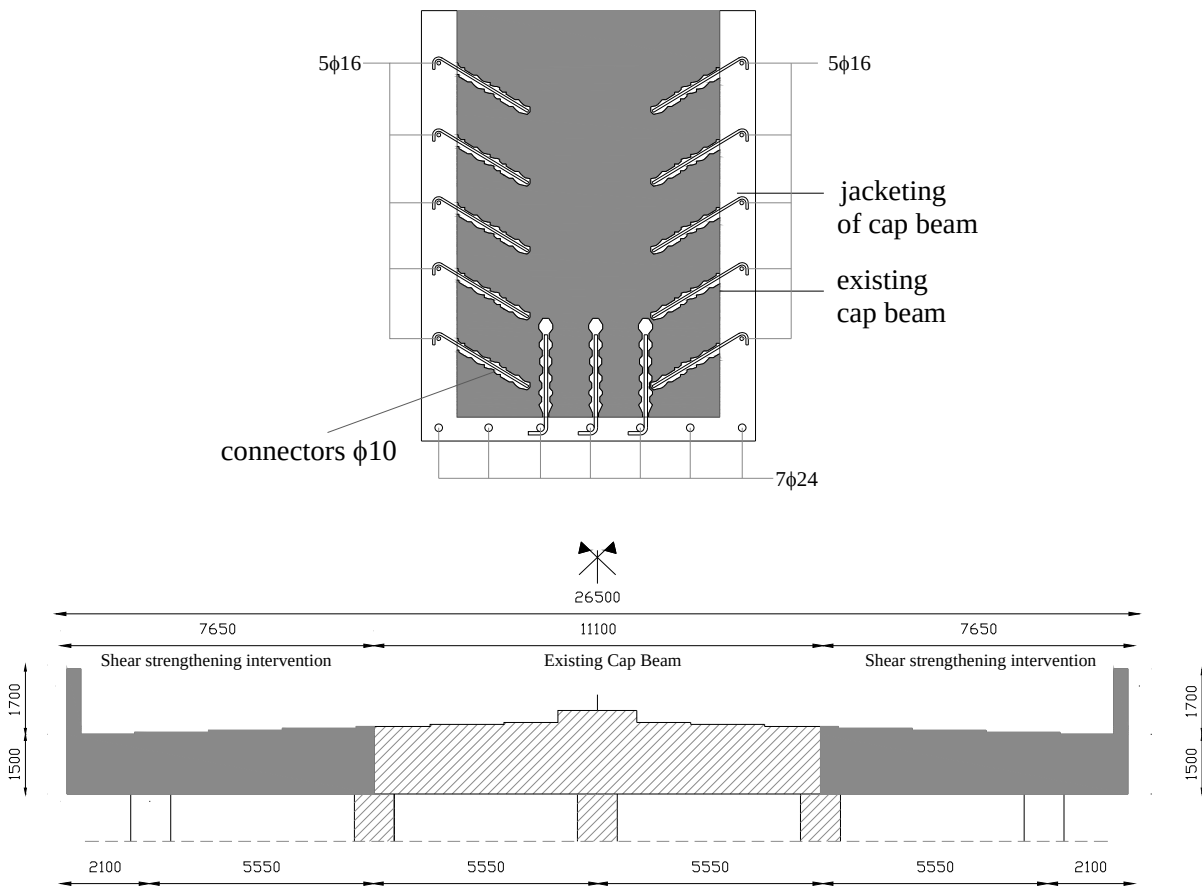


Fig.18—Top beam shear strengthening intervention



Fig.19—Innovative materials intervention: application phase

**Performance Evaluation and Maintenance of
Concrete Bridges in Cold Regions**

A. Hmidan, Y.J. Kim, D.K. Yoon, and S. Yazdani

Synopsis: This paper presents a unique approach to examine the performance of constructed concrete bridges in cold regions, based on a combined statistical analysis and geographic information system (GIS) method. A total of 3,013 bridges and 1,126 bridge decks selected from the State of North Dakota (one of the coldest regions in the United States) are analyzed. Detailed technical information of the examined bridges is obtained from the National Bridge Inventory (NBI) database constructed between 2006 and 2007. A statistical analysis is conducted to identify the critical sources of bridge deterioration in cold regions, in particular concrete bridges, using the ordinary least-square multiple regression method. The performance of concrete bridges under cold weather is in general satisfactory, while the deck slabs are the critical structural members and may require regular maintenance and repair. The contribution of the year-built and the presence of water are the most critical factors to the bridge deterioration. A case study is presented based on a 29-span bridge consisting of cast-in-place deck slabs supported by prestressed concrete and steel plate girders. Detailed inspection results are reported and adequate maintenance methods are discussed.

Keywords: cold region, concrete bridge, decks, deterioration, evaluation, geographical information system (GIS), maintenance, national bridge inventory (NBI)

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INTRODUCTION

The deterioration of constructed bridges is one of the most significant problems facing the infrastructure system over the world. An estimated budget of \$607 billion may be required to replace the deteriorated bridges in the United States (FHWA 2008). A number of factors influence the deterioration of existing bridges, for example, the number of daily traffic, age, and environmental parameters (Papavizas and Kostem 1980, Rizkalla and Labossiere 1999, Kim et al. 2008). Existing bridges are typically categorized into three groups, namely, *non-deficient*, *structurally deficient*, and *functionally obsolete* bridges. Structurally deficient bridges mean that certain components of a bridge need to be repaired or replaced because the load-carrying elements of such a bridge are not adequate (FHWA 2008), whereas functionally obsolete bridges indicate that functional characteristics of a bridge may not accommodate the needs of the present traffic (e.g., lane width and vertical clearance).

The most vulnerable member in a bridge structure system may be deck slabs, given that the bridge decks are directly exposed to traffic loads and other environmental effects. The use of deicing salts may create chloride in reinforced concrete members (Enright and Frangopol 2000). The chloride migrates through the concrete and causes corrosion of reinforcing steel. The volumetric expansion of corroded reinforcing bars can cause the spalling of concrete that may accelerate the deterioration of damaged concrete members. Correspondingly, significant amounts of maintenance and repair budgets are spent for deck slabs (Yunivich and Thompson 2003). The research community has made significant efforts to examine the contribution of various parameters (i.e., service load, temperature, freezing-and-thawing, and wet/dry) to the deterioration of constructed bridges; however, their overall synergetic effects on existing structures have rarely been reported. This paper presents an investigation into the performance of concrete bridges in cold regions through the existing bridges in North Dakota that is one of the coldest states in the United States (NWS 2009). The examination includes 3,013 concrete bridges (including prestressed concrete bridges) and 1,126 bridge decks associated with the concrete bridges (1,887 decks were not rated within the inspection period), using a combined approach of the statistical analysis and geographic information system (GIS). Extended discussions on other types of bridges such as steel and timber with more data samples are available in Kim and Yoon (2010). A case study of a 28-span bridge ($L = 820$ m [2,682 ft.]) is presented to better understand the deterioration of concrete bridges in cold regions and corresponding evaluation and maintenance/repair.

NATIONAL BRIDGE INVENTORY

The National Bridge Inventory (NBI) program was initiated in 1971 to manage the bridges in the United States. Typical items in the NBI database include the geometry of bridges, year-built, average daily traffic, location of bridges with latitude and longitude, design load, materials used, and deck rating (FHWA 1995). Bridge owners and state governments regularly update the NBI and the federal government uses the reported database for allocating necessary maintenance/repair budgets. The NBI data used for this study was created based on the inspection period between 2006 and 2007 (NDDOT 2007).

RESEARCH APPROACH

A combined statistical analysis and GIS method was employed to evaluate the present condition of concrete bridges in North Dakota. Brief descriptions of the research approach are as follows, while further details are available in Kim and Yoon (2010).

Geographical information system

ESRI's (Environmental Systems Research Institute) GIS software package ArcGIS 9.3 was used to analyze and visualize the relationship between bridge status and other attributes such as structural systems, environmental factors, and design information. Each bridge in the NBI database was visually represented in a spatial layer of the GIS system, based on the longitude and latitude coordinates. The constructed bridges were then correlated with the physical conditions and structural-evaluation ratings. A typical GIS map created for this study is shown in Fig. 1. The GIS model provided detailed geographical distributions of the concrete bridges in North Dakota.

Statistical Analysis

A multiple linear regression analysis was conducted to examine the relationship between bridge status and critical factors influencing the deterioration of concrete bridges. A total of 3,013 concrete bridges and 1,126 bridge decks were examined to identify the critical sources of bridge deterioration in cold regions. Three different regression models (Table 1) were constructed to evaluate the vulnerable factors for the constructed bridges within a given time period (t) of 2006 to 2007, as shown in Eq. 1.

$$C = f(P, E, S)t \quad (1)$$

where C is the evaluation category (i.e., deck status, structurally deficient, and functionally obsolete), P is the physical factors of the bridge (e.g., design load, number of spans, and year-built), E is the environmental factors (e.g., precipitation, snowfall, and temperature), and S is the service factors (e.g., highway, rail load, and urban/rural). To expand the investigation range of the NBI, environmental factors (i.e., temperature, precipitation, and snowfall) were additionally considered for this study using the database obtained from the North Dakota State Climate Office (2008) and USGS Northern Prairie Wildfire Research Center (2008).

PRESENT STATUS OF BRIDGE DECKS

The present status of the 1,126 bridge decks is shown in Fig. 1, including state routes and interstate highways. Although the NBI demonstrated rating factors from 0 (*Failed*) to 9 (*Excellent*), the deck rating for this study was simplified into four categories, namely, *failed* (rating 0), *poor* (rating 1 to 4), *fair* (rating 5 to 6), and *good* (rating 7 to 9). The deterioration level of the bridge decks located in the interstate highways (i.e., I-29 and I-94 shown in the key map of Fig 2) was significantly higher than that of other roads, as shown in Fig. 1. This is primarily attributed to the volume of traffic (Fig. 2), including the operation of heavy trucks transporting commodities produced in the Midwest of the United States and central Canada. Figure 3 shows a typical example of deteriorated decks in the interstate highway of I-29. The relationship between the 104 decks failed (rating of 0) and the constructed year is shown in Fig. 4. Over 73% of the failed decks were constructed in the 1960's and 1970's. This may indicate that the critical year for deck failure would be around 1975, based on the inspection of 2006-2007.

DETERIORATION OF CONCRETE BRIDGES

Figure 5 shows a geographical distribution of the bridges sorted by year-built. The interstate highways (I-29 and I-94) involved relatively old bridges that had been constructed before the 1960's, in particular near the north end of I-29. The present status of concrete bridges is shown in Fig. 6. Approximately 93% of the bridges were non-deficient (Fig. 6a) and the level of non-deficiency of the concrete bridges was well lower than that of other types of bridges (Fig. 6b). This observation supports that concrete bridges (including prestressed concrete members) may be an ideal structural type in cold regions. Most of the deteriorated bridges were randomly distributed across the state, whereas some concentrations of functionally obsolete bridges were found at north end of the interstate highway of I-29 (Fig. 6). This may be attributed to the fact that the bridges near the Canadian border were older than other bridges (Fig. 5), implying that the bridges constructed based on the out-of-date design specifications may not adequately accommodate the current traffic needs such as truck size and weight.

TYPICAL SOURCES OF CONCRETE BRIDGE DETERIORATION

The multiple regression results of the three categories (i.e., deck status, structurally deficient, and functionally obsolete) are listed in Table 1. The critical factors influencing bridge deterioration are represented by the standardized coefficient, β ; in other words, a high β value of a certain variable implies that the contribution of the variable becomes more associated with the deterioration of bridge components within an investigation range. It should, however, be noted that the β value has a statistical meaning only if the significance is lower than a certain level (e.g., 0.05 for the confidence interval of 95% and 0.01 for 99%). The negative β in Table 1 indicates that the level of deterioration increases when the β value decreases. For example, the β value of -0.125 in *Year-built* (Deck Status in Table 1) indicates that the deterioration level increased when the age of a bridge tended to be far from the inspection year of 2006-2007 (i.e., old bridge). Selected variables affecting the deterioration of concrete bridges are summarized in Fig. 7 with absolute values of β to simply compare the contribution of the selected variables to the deterioration, rather than their ascending or descending trends. The presence of water, including precipitation, was the most significant factor influencing the deterioration of bridge decks (Fig. 7a), followed by the year-built. Taking into account the cold weather in North Dakota, the presence of water might have caused freezing-and-thawing damage to the concrete and corrosion damage to the reinforcing steel. The strong correlation of the replacement length with the structurally deficient bridges (Fig. 7b) indicates that more structural components were replaced when the bridge became more structurally deficient. The year-built was the most significant contribution to the structurally deficient bridges. Other structural characteristics such as the number of spans also influenced the deterioration of concrete bridges. The effect of heavy traffics (e.g., average daily truck traffic) could be a critical source, whereas the significance level for such a factor was not enough to statistically claim the contribution, as shown in Table 1.

CASE STUDY

Description of Bridge Deterioration

The bridge, 12th Avenue North Bridge (ID: 294-000.965) in Fargo, ND, consists of prestressed concrete girders and cast-in-place concrete decks, including steel plate girders near the approach, as shown in Fig. 8. The 29-span bridge was constructed in 1978 and includes two traffic lanes. The length of the bridge is 820 m [2,682 ft.] and the width of the deck is 20.5 m [67.2 ft.]. The estimated average daily traffic is 17,000. The condition rating of the bridge was conducted based on the bridge rating scale of the Federal Highway Administration, as shown in Table 2. Table 3 shows the condition ratings of the bridge over the last 20 years. The superstructure of the bridge showed a significant drop in the rating between 1989 and 1999, whereas the rating was stabilized after 1999. The deck rating gradually decreased from 7 to 5 for 20 years and no significant decrease in the rating was observed for the substructure, as shown in Table 3. It should be noted that regular maintenance and repair have been done to maintain the gradual decrease in deck ratings. The bridge exhibited a failure of the expansion joint and deck drain, and severe chloride penetration resulted from the excessive use of deicing salts. The chloride concentrations drained through the deck joints, over the girders, and down to the pier caps and piers, as shown in Fig. 9. To examine the chloride contents of the bridge, a number of samples were extracted from the deck, piers, and girders, as shown in Table 4. The chloride levels at the girders and piers were higher than those of the deck. This observation explains the rapid deterioration of the superstructure (Table 3) when compared to that of the deck slabs that have been regularly maintained and repaired. Figure 10 shows the deteriorated locations of the deck slab. The *Chain-drag* method was used to identify and map the deteriorated concrete. The primary source of the deterioration was corrosion of the reinforcing steel induced by the intrusion of chlorides into the cracked concrete.

Maintenance

To provide adequate maintenance of the deteriorated bridge deck, the following procedures are recommended by North Dakota DOT-Fargo District to repair deteriorated concrete deck slabs:

1. Using the *Chain-drag* method to identify the locations of deteriorated areas
2. Grinding the entire deck as the first step with a typical depth of 40 mm [1.5 in.]
3. Removing the deteriorated concrete to a depth where a good solid piece of concrete is found.
4. Cleaning the rebar, if needed, with an adequate method to remove the corrosion and repair it to ensure the integrity of the completed repair
5. Coating the exposed rebar with high-performance anti-corrosion agents
6. Replacing the cracked concrete with fresh concrete up to the level of the ground deck
7. Applying an overlay over the entire deck.

Electrochemical Chloride Extraction (ECE) was used to remove the chloride from the concrete. The principle of the ECE method is that hydroxyl ions are generated to separate chlorides from reinforcing bars by applying an electric field between the concrete and the bars. ECE applied to a vertical or overhead surface typically employs a steel anode fastened to the concrete. Titanium anodes are generally specified for architectural applications where rusting of the anode is unacceptable. A spacer is used to keep the anode from being in direct contact with the concrete, allowing room for the electrolyte media between the anode and the concrete. The electrolyte media usually consists of a sprayed cellulose fiber mixture, which is easily applied to vertical and overhead surfaces. The electrolyte media is designed to retain moisture. The system is then wrapped with plastic to help retain moisture, and is continuously wetted. The treatment duration is typically 6 to 8 weeks, after which all materials are removed from the concrete surface. When steel anode is used there will be some rust staining. Abrasive blast cleaning is generally specified to remove the rust staining and prepare the surface for the application of an appropriate sealer or coating. Figure 11 shows a maintenance operation using the ECE method for the piers of the 12th Ave. N. Bridge. Wire mesh was wrapped around the area to be treated (after preparatory measures and repairs have been completed). The surface was then sprayed with a fiber mixture (similar to wet newspaper), after which the ECE process was started.

On-going renovation of the 12th Ave. N. Bridge

A two-year bridge renovation project started in 2008. The bridge was widened to accommodate four traffic lanes and a pedestrian walkway. The entire deck slab of the bridge was to be replaced. Approximately one half of the existing concrete beams were replaced and the steel beams were repainted. The first year included widening the structure and the work was completed about a half of the deck replacement. The remainder of the deck slab is currently being replaced.

SUMMARY AND CONCLUSIONS

This paper has presented a statistical approach combined with GIS technology to examine the sources of bridge deterioration in cold regions. The existing concrete bridges in North Dakota were sampled for this purpose. The performance of the concrete bridges was satisfactory, while the decks were the critical elements for deterioration. According to the statistical analysis, the age of bridges and the presence of water were the most critical factors that influenced the deterioration of the bridge decks. A case study was discussed to show a typical bridge deterioration in cold regions and corresponding maintenance and repair works.

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Table 1—Regression analysis for concrete bridges

Variables		Deck status		Bridge status (Structurally Deficient)		Bridge status (Functionally Obsolete)	
		Standardized coefficient (β)	Significance level	Standardized coefficient (β)	Significance level	Standardized coefficient (β)	Significance level
Physical factor	Design load	0.013	0.538	-0.047*	0.049	-0.052*	0.032
	Number of spans	-0.095**	0.000	-0.066**	0.002	0.015	0.486
	Width	0.046	0.056	-0.069**	0.009	-0.079**	0.003
	Year built	-0.125**	0.000	-0.117**	0.000	-0.018	0.444
	ADT	0.054	0.276	-0.011	0.840	-0.101	0.067
	Truck ADT	-0.010	0.634	-0.038	0.090	-0.025	0.269
	Future ADT	-0.025	0.606	0.012	0.830	0.080	0.145
Environmental factor	Replaced length	0.010	0.579	0.133**	0.000	0.035	0.079
	Precipitation	-0.137**	0.000	-0.057*	0.028	0.021	0.421
	Snow-fall	0.067**	0.000	0.000	0.970	-0.010	0.638
	Below 0	0.036	0.289	0.058	0.124	-0.011	0.775
	Below 32	-0.028	0.262	-0.011	0.679	-0.024	0.338
	Above 90	-0.089**	0.003	-0.033	0.325	0.075*	0.025
	Extreme low tem.	-0.022	0.381	0.032	0.246	0.005	0.869
Service factor	Average tem.	0.022	0.576	-0.015	0.716	-0.090*	0.036
	Over water	0.413**	0.000	-0.050*	0.022	-0.131**	0.000
	Urban/rural	-0.021	0.399	-0.020	0.467	0.026	0.352
	Highway	0.009	0.667	-0.003	0.885	-0.025	0.255
	Railroad	-0.001	0.942	-0.016	0.386	-0.022	0.224
	Toll	-0.028	0.090	-0.007	0.694	-0.009	0.611
Population		0.012	0.474	-0.011	0.561	-0.018	0.330
<i>N</i>		3013		3013		3013	
<i>R</i> ²		0.220		0.056		0.028	
<i>Adjusted R</i> ²		0.215		0.049		0.021	
<i>F</i>		40.240		8.419		4.093	

*: 95% confidence interval, **: 99% confidence interval

Note: *ADT* = average daily traffic, *Below 0* = number of days with temperature below 0°, *Below 32* = number of days with temperature below 32°, *Above 90* = number of days with temperature above 90°, *Precipitation* = annual mean precipitation, *Extreme low tem.* = lowest temperatures on record, *Average tem.* = annual mean temperature, *R*² = coefficient of determination, *N* = number of samples, and *F* = *F* ratio (the probability information produced by an ANOVA)

Table 2—Bridge condition codes (Federal Highway Administration)

Code	Description
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION - advanced section loss, deterioration.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION – out of service - beyond corrective action.
N	NOT APPLICABLE

Table 3—Condition rating of the 12th Ave. N. Bridge

Inspection year	Condition Rating		
	Deck	Superstructure	Substructure
1989	7	8	7
1999	6	4	6
2007	5	4	6

Table 4—Chloride content found at different locations on the bridge

Bridge component	Lab No.	Field sample No.	Station location	Chloride content
Deck	31	1	0+82-5' Rt of CL 2"	1.80
	32	2	0+72-18' Rt of CL 2"	0.83
	33	3	2+20-11' Rt of CL 2 ¼"	0.56
	34	4	3+40-7' Rt of CL 2 ¼"	1.25
	35	5	3+85-18' Rt of CL 2 ½ "	1.14
	36	6	7+30-7' Rt of CL 2 ¼"	0.83
	37	7	6+19-14' Rt of CL 2 ¼"	0.42
	38	8	8+85-10' Rt of CL 2'	1.56
	39	9	10+20-13' Rt of CL 2 ¾"	0.98
	40	10	11+50-3' Rt of CL 2 ¾"	3.83
	41	11	12+75-13' Rt of CL 2 ¾"	2.08
	42	12	13+46-10 Rt of CL 2 ¼"	0.48
	43	13	15+10-3 Lt of CL 2 ¾"	0.58
	44	14	16+31-8' Lt of CL 2 ½ "	0.90
	45	15	17+96-2' Lt of CL 2 ¾"	0.56
	46	16	18+40-1' Lt of CL 2 ⁵ / ₈ "	0.69
	47	17	20+00-15' Rt of CL 2 ³ / ₈ ,"	2.62
	48	18	21+15-15' Rt of CL 2 ³ / ₈ ,"	0.97
	49	19	22+47-5' Rt of CL 1 ⁷ / ₈ ,"	1.04
	50	20	24+54-5' Rt of CL 2 ¾"	1.11
	51	21	25+25-5' Rt of CL 2 ⁵ / ₈ "	1.01
	52	22	26+65-10' Rt of CL 2 ¼"	2.81
	53	23	26+75-16' Lt of CL 2 ¼"	1.11
	54	24	23+33-10' Lt of CL 2 ½ "	0.90
	55	25	21+57-4 Lt of CL 1 ⁷ / ₈ ,"	2.64
	56	26	20+04-16' Lt of CL 1 ⁷ / ₈ ,"	0.97
	57	27	18+88-3' Lt of CL 2 ³ / ₈ ,"	3.83
	58	28	17+45-5' Lt of CL 2 ¾"	0.42
	59	29	15+96-7' Lt of CL 2 ⁷ / ₈ ,"	2.64
	60	30	14+97-17' Lt of CL 2'	0.69
	61	31	13+71-6' Lt of CL 2 ¾"	1.28
	62	32	12+98-9' Lt of CL 2 ½ "	0.76
	63	33	11+70-1' Lt of CL 3 ¼"	3.21
	64	34	10+33-12' Lt of CL 2'	0.48
	65	35	8+25-4' Lt of CL 2 ³ / ₈ ,"	0.62
	66	36	7+07-15' Lt of CL 2 ¾"	1.14
	67	37	6+05-4' Lt of CL 2'	0.62
	68	38	4+33-4' Lt of CL 2 ⁵ / ₈ "	2.48
	69	39	2+57-10' Lt of CL 2 ½ "	0.48
	70	40	0+50-16' Lt of CL 2 ¾"	1.04
Pier	71	42	Pier #4 cap	2.71
	73	44	Pier #4 cap girder#6-7	9.30
	75	46	Pier #8 -9 N side of deck	7.29
Girder	72	43	Span #3 Girder #3	2.78
	74	45	Span 4 Girder #7	13.12

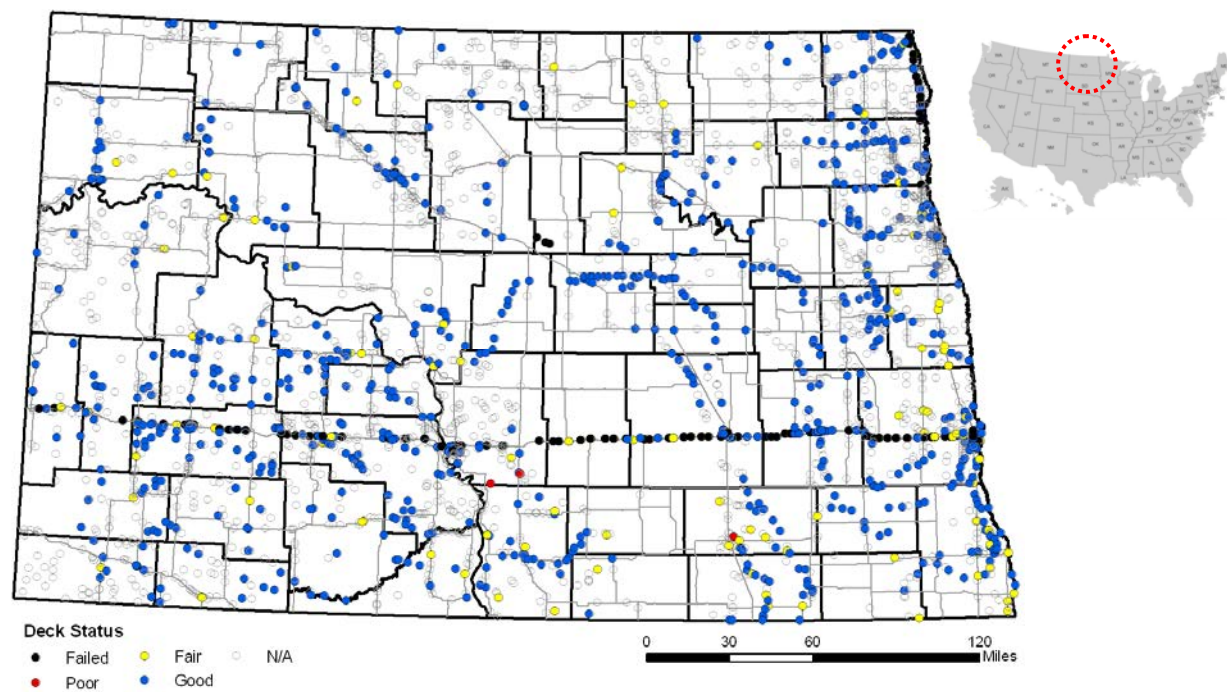


Fig. 1—Status of bridge decks

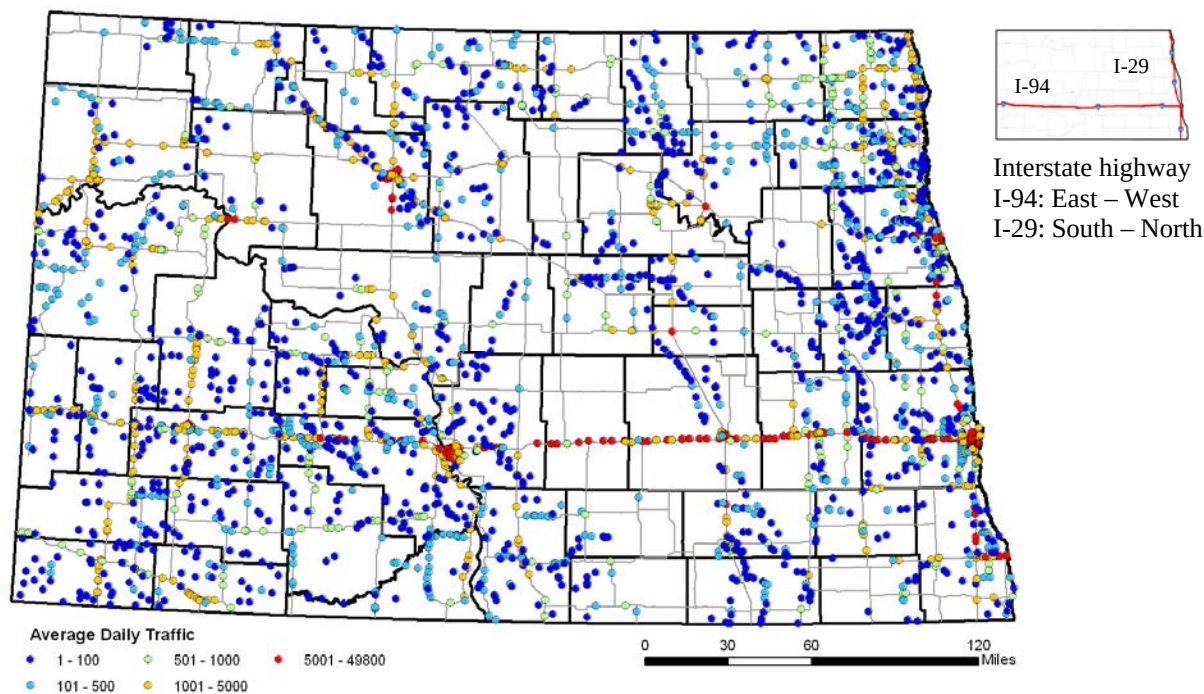


Fig. 2—Average daily traffic

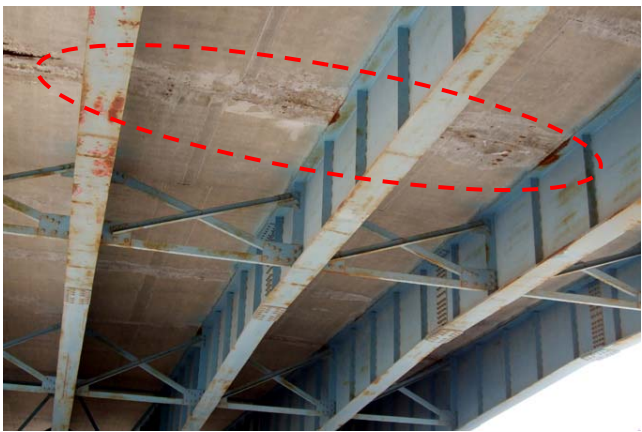


Fig. 3—Deteriorated bridge deck at I-29 and 52nd S., Fargo, ND

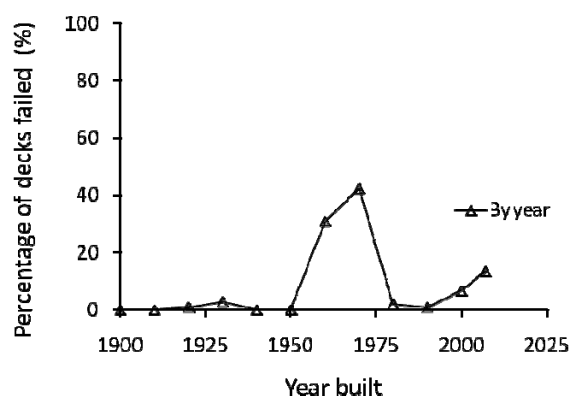


Fig. 4—Bridge decks failed

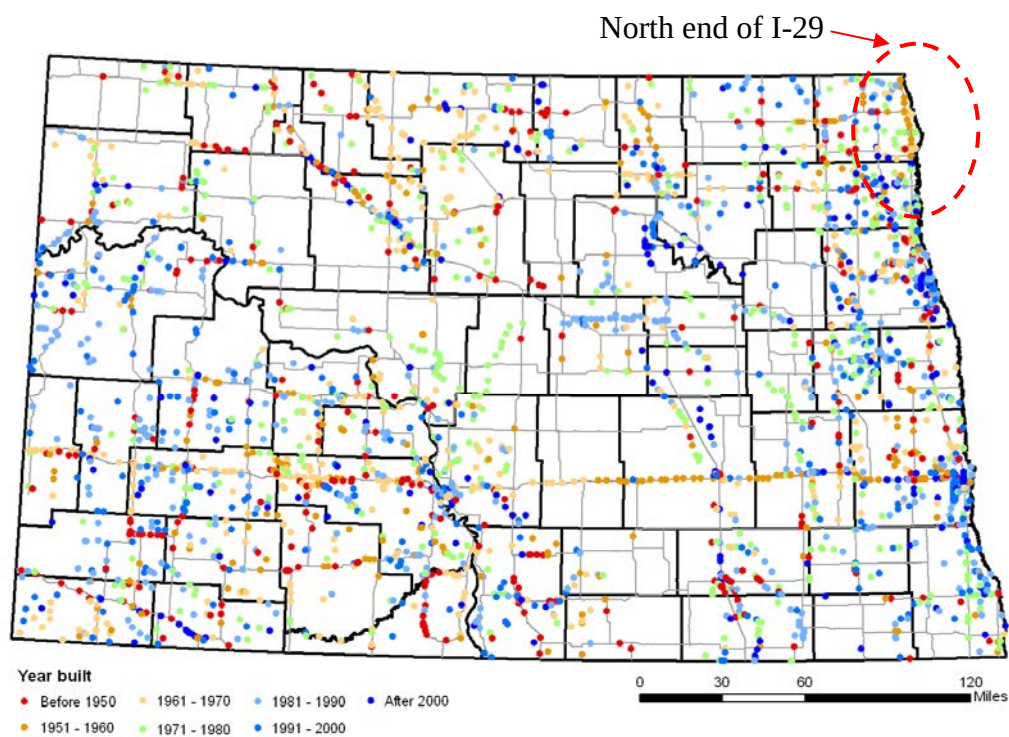
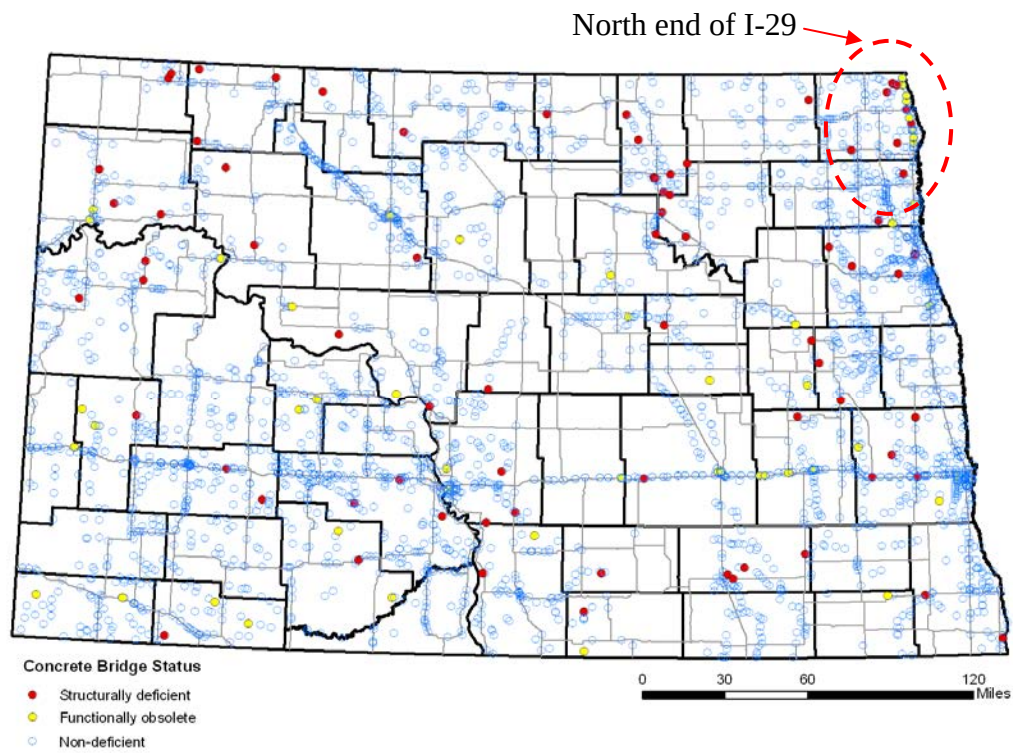
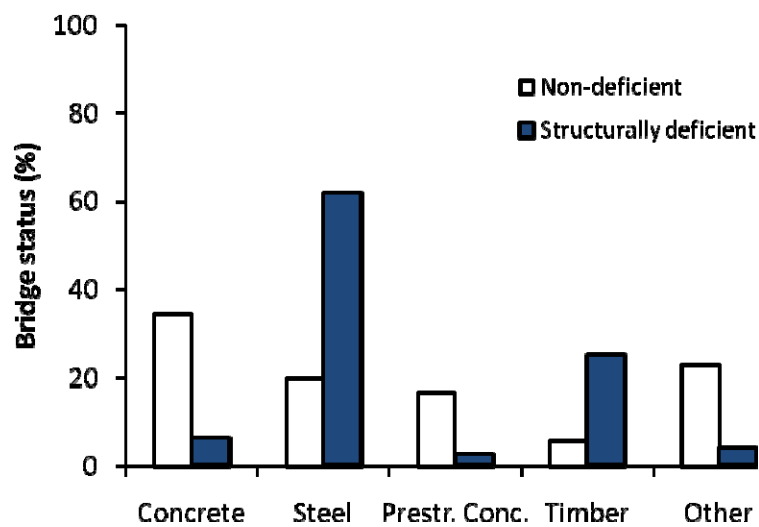


Fig. 5—Concrete bridges sorted by year-built

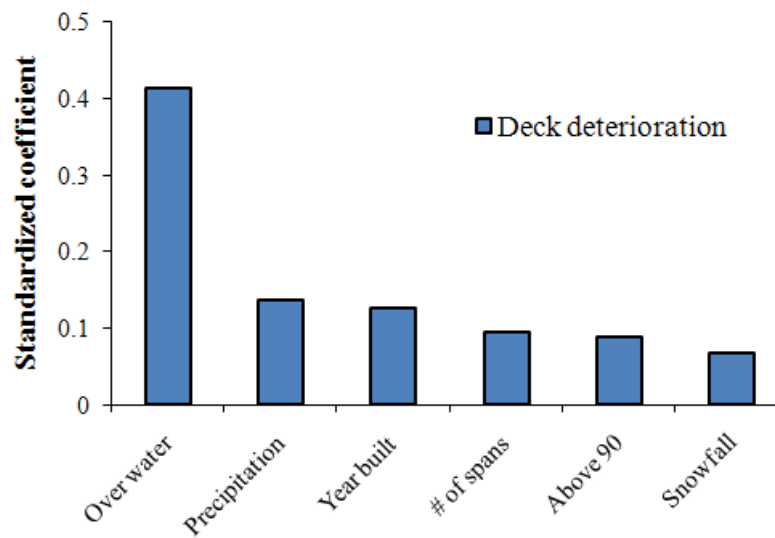


(a)

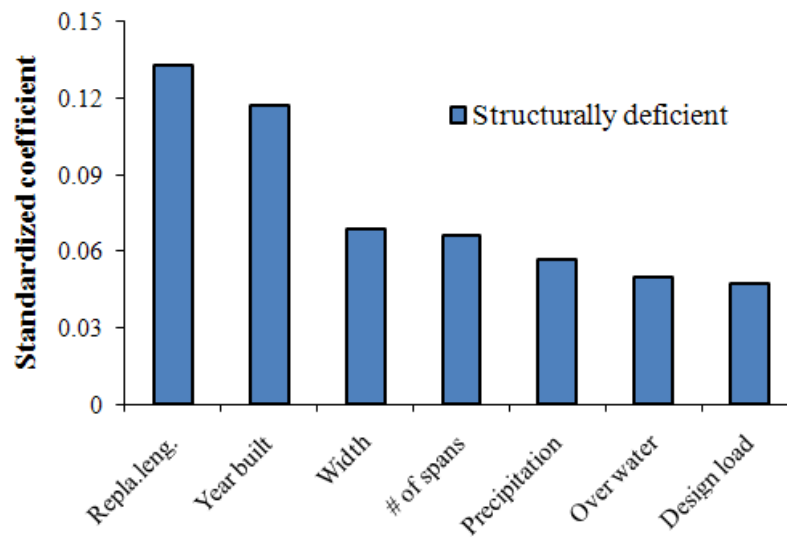


(b)

Fig. 6—Status of concrete bridges: (a) concrete bridges; (b) comparison of concrete bridges with other types of bridges (source from Kim and Yoon 2010)



(a)



(b)

Fig. 7—Selected variables influencing the deterioration of concrete bridges in cold regions: (a) deck deterioration; (b) structurally deficient [Note: *Above 90* = number of days with temperature above 90°, *Repla. leng.* = replacement length]

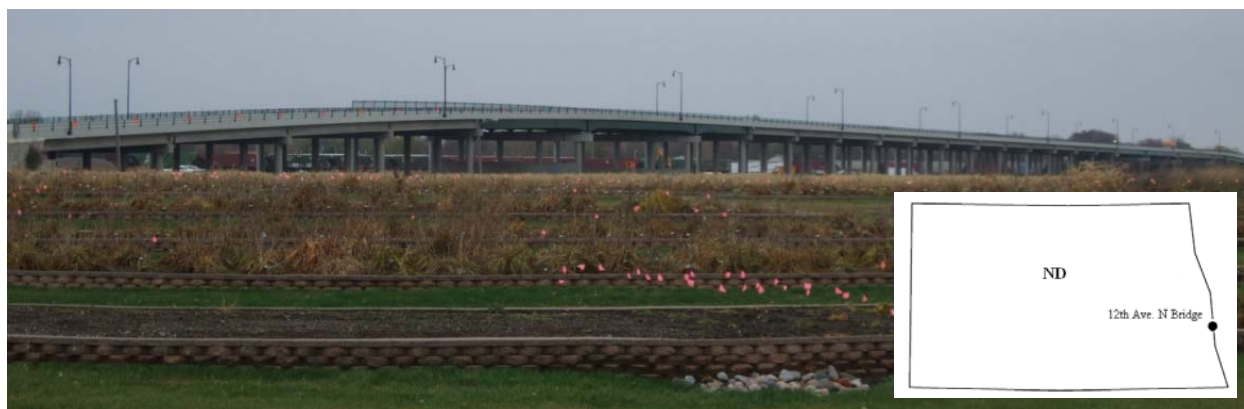


Fig. 8—12th Ave. N. Bridge, Fargo, ND



(a)



(b)



(c)

Fig. 9—Deterioration of the 12th Ave. N. bridge; (a) deterioration of girder; (b) corroded bearing; (c) deteriorated pier-cap concrete (courtesy of NDDOT)

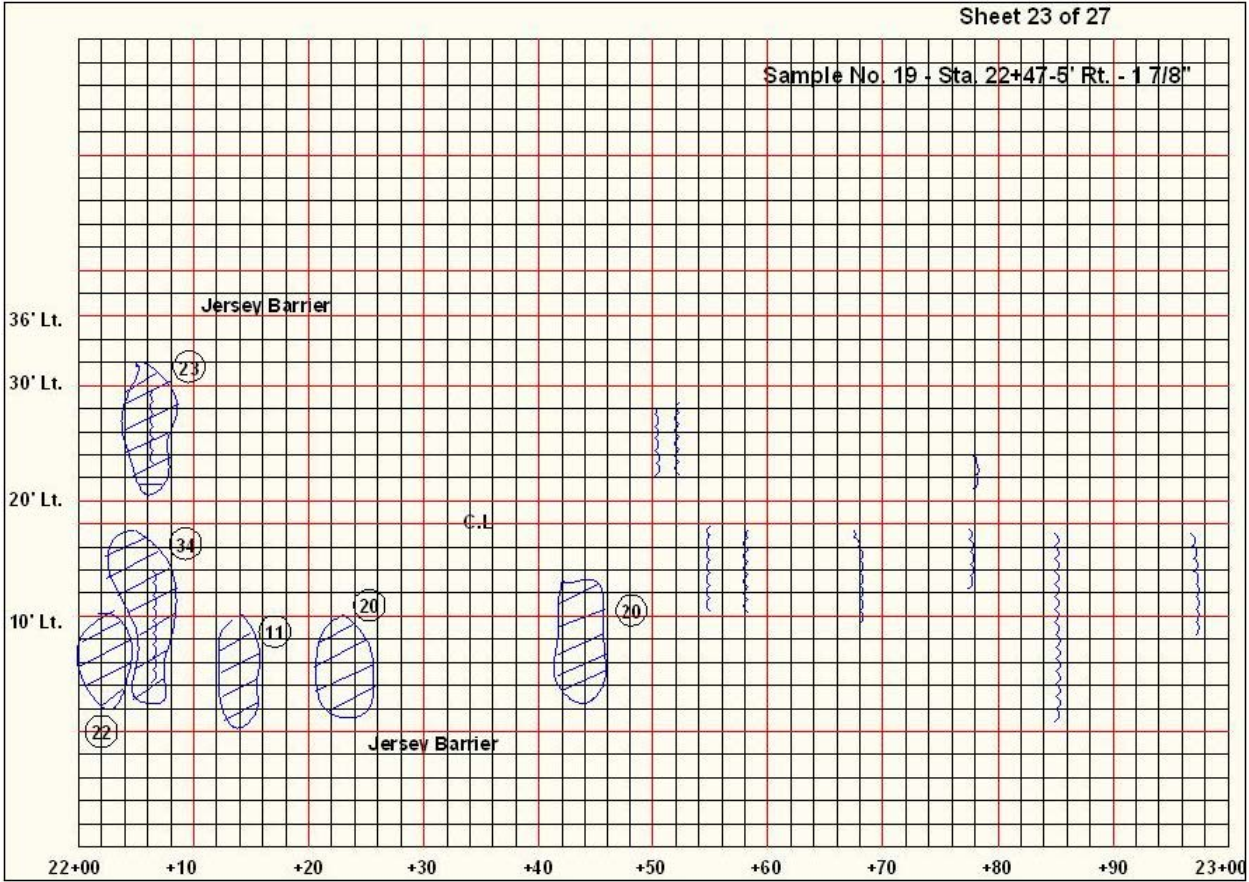


Fig. 10—The location of deterioration and cracks of the deck slab of the 12th Ave. N. Bridge.



(a)



(b)

Fig. 11—Electrochemical Chloride Extraction; (a) Steel woven wire mesh connected with low voltage current; (b) the wired mesh covered with plastic sheets (courtesy of NDDOT).

Texas' Use of CFRP to Repair Concrete Bridges

D. Yang, B.D. Merrill, and T.E. Bradberry

Synopsis: The Texas Department of Transportation (TxDOT) maintains over 33,000 on-system bridges. A considerable number of these bridges are damaged each year by extreme events or structural deterioration and must be repaired rapidly. Externally bonded carbon fiber reinforced polymer (CFRP) composites provide TxDOT with a viable technique for repairing many damaged concrete bridges. CFRP has been used extensively as structural reinforcement for its exceptional engineering properties, simplicity, flexibility, and rapid placement. TxDOT began using CFRP in 1999 and has repaired more than 30 impact-damaged concrete bridges, resulting in considerable time and money savings. This paper summarizes TxDOT's experience repairing concrete bridges damaged by impact, fire, corrosion, and alkali-silica reaction (ASR), focusing on damage assessment, determination of reparability, and procedures essential for effectiveness. TxDOT engineers have made a conscious effort to utilize CFRP materials to repair impact-damaged beams. CFRP has been used to supplement prestressed strands to restore flexural capacity, laterally 'harden' bottom flanges against damage from re-impacts, and enhance the ductility, shear strength, and integrity of concrete bridge beams. For repeatedly impact damaged beams, CFRP has been used as 'sacrificial' reinforcement to protect the primary reinforcement, the prestressed strands, and to increase survivability, thus preserving the structure. Recommendations regarding the effectiveness of such CFRP repairs are presented.

Keywords: TxDOT, Bridge, FRP, CFRP, continuous fiber, carbon fiber, epoxy resin, polymer matrix, compatibility, tensile reinforcement, wet layup, flexibility, structural repair, sacrificial reinforcement, strengthening, confinement, impact damaged, fire damaged, corrosion, ASR.

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INTRODUCTION

Use of fiber reinforced polymer composites (FRP) on civil structures has expanded engineers' options for repair and strengthening of concrete structures. FRP uses natural or synthetic continuous fibers of carbon, glass fibers, or aramid (Kevlar) encapsulated by thermoset polymers to form composites referred to as CFRP, GFRP, or AFRP. The thermoset polymers (epoxy, vinylester, polyester, or a blend thereof) form the matrix of the composite and bind the fibers, transfer stresses between fibers, and protect the fibers from potential environmental degradation. FRP has comparable mechanical properties to conventional reinforcement but is non-corrosive and much lighter. From a design standpoint, it is important to recognize that FRP composites are linearly elastic to failure and are thus non-ductile. FRP fabrics or laminates are externally bonded to the surface of concrete members to reinforce them providing a methodology for structural reinforcing and retrofitting. Furthermore, the installation process is flexible, simple and rapid. FRP reinforcements were first introduced to civil structures in the United States some 20 years ago and in the current decade have become increasingly popular as consensus guidelines and standards for design, construction, and quality control have been published^{1,2,3}. Among externally bonded FRP applications, carbon fiber reinforced polymer (CFRP), using carbon fiber fabrics, epoxy resins, and wet lay-up installation techniques, is perhaps the most adaptable FRP system available for repair and strengthening of reinforced concrete members. This application is particularly suitable for repairing concrete bridge members due to high compatibility between carbon fibers and steel reinforcement, excellent adhesion and mechanical properties of epoxy resin for both concrete and carbon fibers, and ease of installation.

Material compatibility is an important criterion for selecting materials to achieve effective structural repairs. CFRP uses high strength carbon fibers with a modulus of elasticity from 32,000 ksi (220,000 MPa) to 34,000 ksi (240,000 MPa), which is similar to steel's modulus of elasticity of 29,000 ksi (200,000 MPa). This similarity means that carbon fibers will resist a strain with a similar stress change as in the steel reinforcement; an important consideration for structural strengthening. It also has implications for structural confinement because the high modulus carbon fibers can create a high confining pressure indirectly strengthening the core of a round column⁴. Carbon fibers provide a tensile strength of approximately 550 ksi (3,800 MPa); almost ten times the yield strength of mild steel. In addition, carbon fibers have other engineering properties that are superior to those for glass and aramid fibers, such as a high resistance to moisture and chemical degradation and less susceptibility to creep-rupture^{1, 4, 5, 6}. It is well known that epoxies are the high-performance "structural adhesives"⁷ providing the highest strength and the least shrinkage among commonly used thermoset polymers⁵. Serving as bonding agent and saturating resin, epoxy provides a reliable matrix for the CFRP application. Wet lay-up installed CFRP can be tailored to fit almost any shape of concrete member. The installation is simple, requiring only hand held tools and can be done in a small working space (Figure 1). These advantages make this CFRP application particularly useful for repair of in-service bridge members, where delays to highway users are costly. The Texas Department of Transportation (TxDOT) started using a wet lay-up installed CFRP system for strengthening concrete bridge members and improving load capacity of bridges in 1999 based on findings of TxDOT-funded research^{8, 9, 10}. Over the years, TxDOT has developed specifications and standard repair details for CFRP use. TxDOT specifies the use of unidirectional carbon fiber fabrics, wet-lay-up installation techniques, and pre-approved proprietary CFRP systems¹¹ to ensure quality and cost-effectiveness.

TxDOT maintains more than 33,000 on-system bridges; the largest inventory of any state in the Nation. Of these bridges, approximately 85% have concrete spans and there are concrete elements in all types of bridges. A considerable number of bridges are damaged each year due to extreme events or structural deterioration and must be repaired. TxDOT has a well developed program for repairing impact damaged concrete bridges utilizing CFRP and other viable techniques to provide cost-effective repairs where replacement had previously been the only option, resulting in considerable time and money savings. TxDOT also uses CFRP in repairing bridges damaged by fire, corrosion, and alkali-silica reaction (ASR). It is expected that more CFRP will be used in bridge rehabilitation to extend service life. TxDOT is continually pursuing improvements of CFRP applications, such as a new research

project investigating the use of CFRP anchors, to expand strengthening capabilities for larger reinforced concrete elements.

To illustrate TxDOT's use of CFRP in repair of damaged concrete bridge members, case-studies are presented in the following four causes of damage: 1) vehicle impact, 2) fire, 3) reinforcing steel corrosion, and 4) alkali-silica reaction (ASR). The case studies focus on damage assessment, determination of the reparability and repair techniques to be used, repair procedures deemed essential for effective use of CFRP, and other viable repair techniques. A summary with recommendations is presented in the conclusions. TxDOT's use of CFRP systems predated the 2002 release of ACI 440.2R-02, "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures". This document did not include specific design methodologies for applications on prestressed concrete girders. Early designs largely relied on the technical information and design procedures supplied by the CFRP manufacturers. The 2008 version, ACI 440.2R-08, does include design procedures for CFRP applications on prestressed concrete girders. Other applications, such as confinement of ASR affected concrete columns are not included in the ACI document. In those cases, we applied conservative engineering judgment and attempted to utilize the design guide as much as possible.

REPAIR OF IMPACT DAMAGED CONCRETE MEMBERS

To date, more than 30 impact-damaged concrete bridges have been repaired using CFRP, including severely damaged bridges where replacement had previously been the only option. Most of these bridges are over Interstate and US highways where high traffic volumes require that repairs be done as quickly as possible to return the bridges to pre-damage levels of service. Use of CFRP systems often makes meeting such a demand possible. In most cases impact damage is to exterior beams directly facing the moving traffic. Occasionally the impact damage is to interior girders or to substructure elements such as columns. Each incident requires expeditious handling by experienced bridge engineers to ensure the damaged structure is stabilized to protect public safety. The extent and severity of the impact damage must be thoroughly investigated to assess stability and deficiency of the structure and to determine its reparability.

Prestressed Concrete Beam

Case 1 – Figure 2a shows the first impact damaged beam in Texas repaired using CFRP. The exterior beam was hit by an overheight vehicle, severely fracturing the entire web and bottom flange between two interior diaphragms. Only one of the 44 prestressing strands (3/8 in. dia. (10 mm)) was severed by the impact. The remaining strands all retained their original tension. A girder with this level of damage would normally be replaced. TxDOT engineers felt that wrapping the repair area with CFRP could add tensile strength, shear strength, and robustness to the repair. A cost and time comparison was made between repair and replacement. The repair option was selected after the comparison indicated that repairs could be made in one forth the time and for half the cost of replacing the beam.

After removing all loose and unsound concrete, a 40 kip (180 kN) loaded truck (H20 truck) was positioned to pre-load the beam prior to patching the concrete. This practice was established under a TxDOT-funded research project to develop and evaluate repair effectiveness for impact-damaged prestressed concrete girders¹². The loaded truck was moved off the bridge after completion of concrete repairs, epoxy injection of cracks, and repair material curing to attain a 3000 psi (21 MPa) compressive strength. One ply of a continuous CFRP "U" wrap, covering the entire bottom flange and web between the two diaphragms, was then installed using wet lay-up techniques (Figure 2b). The bridge was repaired in 5 days with a total direct cost of \$47,000. The installed CFRP was inspected 5 years later with no sign of delamination or other deterioration of the CFRP composite (Figure 2c).

Case 2 – TxDOT engineers have also used CFRP as tensile reinforcement to offset lost prestressing strands severed by impacts. One such case occurred in March, 2004 and is shown in Figures 3 and 4. Eight of the 32-½ in. (13 mm) diameter prestressing strands in the impacted beam were severed. The damaged area was limited to 8.5 ft (2.6 m) or 1/10 of the span length. The damaged beam had no additional deflection due to the loss of strands – an important indication that lateral support from the diaphragms and deck allowed the superstructure to redistribute the stresses to undamaged beams. The spalling and eight severed strands are clearly seen in Figure 3. TxDOT personnel determined that the beam was repairable using CFRP and a strand splicing system previously evaluated by a TxDOT research project¹². The design called for splicing five of the eight severed strands and using CFRP as supplemental tensile reinforcement to offset the three un-spliced strands. All eight severed strands were not spliced because the splicing devices had to be staggered to facilitate concrete placement, which requires removal of more sound

concrete. The use of CFRP reinforcement to offset lost strands can only restore the required ultimate flexural capacity of the beam; it does not affect the cracking moment capacity. This is because the CFRP system is passive reinforcement compared with the active prestressing strands whose strength the CFRP system replaces. CFRP has virtually no effect on the section modulus but it can help control the development of cracking resulting in smaller, more evenly distributed cracks⁴. A proprietary CFRP wet layup system was included in the design but the plans also provided design criteria so that substitution of another competing system could be allowed. The system used high-strength unidirectional carbon fiber fabrics with 0.0065 in. (0.17 mm) net thickness, an ultimate tensile strength of 550 ksi (3,800 MPa), and a tensile modulus of 33,000 ksi (230,000 MPa). The CFRP design was based on the design guidelines provided by the primary system's manufacturer. The basic assumptions for sectional strain and stress distribution and constitutive laws of materials at failure in the design guidelines were consistent with provisions in the ACI 318 Code and 440.2R Guide. Three plies of carbon fiber fabric were applied to offset the lost strands and extended beyond the repair zone a sufficient length to secure the bond. Figure 4 show the major repair procedures. Phased installation of the CFRP system was employed to keep one lane of traffic open under the bridge. This was accomplished by providing an overlap splice of the CFRP at the middle of the span. Additional "U" wraps were added at the splice location to enhance bond (Figure 5). Two direct pull-off tests to verify the bond strength were performed after the CFRP had cured for 24 hours but before applying topcoats on this project (Figure 6a). Both tests exceeded a tensile strength of 200 psi (1.4 MPa) at the bond-line between the CFRP and concrete. The bridge was repaired in 8 days at a cost of \$25,800 (Figure 6).

Case 3 – Figure 7 is a photo log of another emergency repair project. Two beams were damaged over nearly a third of their length, with the bottom flanges and most of the webs fractured. The bridge and lower roadway were closed and their respective traffic detoured. The initial inspection found that one beam had four broken strands and another had three damaged strands, while 85% to 90% of the prestressed strands in the two beams were undamaged and retained proper tension. The compression zone of the top flanges and deck were undamaged. The two end sections of the damaged beams were intact and they exhibited the same camber and alignment as the undamaged beams, indicating that the integrity of the superstructure had apparently not been affected. TxDOT engineers determined that the bridge could be repaired with strand splices and CFRP strengthening. An emergency contract was let with a local contractor and the bridge was repaired within one week after the impact at a cost of \$70,470. The bridge was opened to traffic in time to accommodate a popular sporting event in the city. Longitudinal CFRP sheets with the carbon fibers parallel to the beam were wrapped around the bottom flange and up the side of the beams. Wrapping the CFRP up the sides of the flanges reinforces the bottom flange laterally so as to armor the flange to withstand future impacts. Such impact was probable because of the low vertical clearance of the bridge. Almost exactly one year after the first impact event that resulted in the CFRP repair, the bridge was impact-damaged again (Figure 8). The CFRP did appear to laterally strengthen the bottom flange to resist re-impact and reduce the extent of damage. The concrete at the impact point was pulverized. One strand facing the impact was cut. Engineers determined that the beams could be repaired again with the same CFRP system. The damaged portion of CFRP was removed by saw cutting. The damaged concrete was removed and the severed strand spliced. The concrete was repaired and new CFRP plies were lap-spliced over the existing CFRP layers. The beams were repaired again as shown in Figure 8. The repair was completed in 7 days at a cost of \$35,000. In the past several years, a number of prestressed concrete beam bridges previously repaired with CFRP were impacted and repaired again with CFRP. As the vertical clearance of these bridges is low, the chances of being re-impacted are high. This situation has led TxDOT engineers to consider using CFRP as "sacrificial" reinforcement to protect the primary reinforcement, the prestressed strands, so as to increase the survivability of the existing bridges. The flexibility of the externally bonded application makes use of CFRP more viable as "sacrificial" reinforcement for multiple damage/repair cases. Figure 9 shows a typical repair detail for prestressed concrete beams where a layer of unidirectional carbon fiber fabric with the fiber direction parallel to the beam is placed along each side face of the bottom flange. In a vehicle impact situation, the back side of the beam bottom flange is in tension. The carbon fiber has a modulus of elasticity slightly higher than that of steel. Upon impact the carbon fibers on the back side of the beam will be engaged and thereby reduce the possibility of yielding or rupturing the strands. This could lead to a rupture of CFRP, but by sacrificing the CFRP, the beam section and strands may be preserved. CFRP also serves to "armor" the impact side of the bottom flange, reducing the overall damage to the concrete section at the impact point. In addition, since 2004, TxDOT engineers have used transverse CFRP "U-wrap" strip reinforcement to provide clamping restraint along the length of CFRP tensile reinforcement to mitigate debonding. These "U-wrap" strips are considered especially important at bottom CFRP termination points, lap-splice locations, and on either side of diaphragms.

Case 4 – Figure 10 shows a repeatedly impact-damaged bridge where CFRP has been used as “sacrificial” reinforcement in multiple damage/repairs. The bridge had major impact damage in May 2006. Large epoxy-sealed cracks from repairs of previous impacts could still be seen. The beam was repaired with CFRP in Sept. 2006. The bridge was impacted again and repaired with CFRP again in 2007. In 2008 the bridge was impacted yet a third time (not counting the original evidence of a prior impact repair) and repaired with CFRP a third time. Due to tightened budget and other restrictions, these bridges may not be raised anytime soon to increase clearance. Using CFRP as “sacrificial” reinforcement to preserve existing structures will continue to be a viable option for multiple damage/repairs.

Considerations for Impact-Damaged Prestressed Beams – The initial field inspection should obviously focus on public safety and in most cases a portion of the bridge is initially closed above the damaged area. Lower roadways may also need to be closed, or have traffic shifted from beneath the damage area, if falling debris is a concern. Once safety has been addressed, the field inspection and structural analysis needs to focus on the following areas:

1. The bottom flange and lower portion of the web, where the tension component of beam's moment capacity is provided, should be investigated to determine the number and location of obviously damaged strands and the extent of damaged concrete. Each seemingly undamaged exposed strand should also be checked by acoustic tapping with a small hammer to determine if it has retained its prestress force. No simple test is available to determine the actual tension of a prestressed strand in the field. A strand remaining in tension, indicated by a higher pitched sound, is likely in good condition. A loose strand, usually indicated by a dull sound, indicates fracture, yielding, or loss of bond. Loose strands should not be relied upon without taking measures to restore its tension force.
2. The top flange and deck, where the beam's compression component of moment capacity is provided should be investigated. Usually this area is undamaged for prestressed concrete girders because of the very strong composite connection with the deck; nevertheless, the condition and integrity of the concrete should be assessed for any signs of damage. Severe damage to the compression zone could be an indication that replacement is necessary.
3. The ends of the beam and the deviation points for harped or draped strands need to be investigated. The beam ends are the vital sections where the prestressed strands are anchored with high tensile stresses, and where the shear demand of the beam is highest. Without an adequate length of intact end section for anchorage and stress development the strands cannot act as prestressed reinforcement and repairs may not be feasible. If the damaged girder contained harped or draped strands, the deviation points need to be investigated to make sure the strands remain in the correct vertical location. Crushing of concrete at a deviation point can allow the strands to rise up and lose some of their original tension.
4. The camber and horizontal alignment of the damaged beam compared with other undamaged beams in the span needs to be assessed. Significant horizontal misalignment of the bottom flange is a good indicator of loss of strand tension and cause for replacement.
5. Practical evaluation of the loss of capacity due to strand failures is accomplished using simple analysis tools. TxDOT's Bridge Division developed a Load-Rating Spreadsheet which is very useful for quickly estimating the loss of capacity to determine whether supplemental CFRP reinforcement is needed after considering the lost or spliced strands. A working stress or service load analysis of a damaged beam is not practical as the dead load stresses are redistributed to adjacent beams. As mentioned earlier, the analysis performed only checks the ultimate capacity of the damaged girder to determine if supplemental CFRP reinforcing, strand splicing, or both are necessary.

Keys to Successful Repairs for Beams – Proper procedures are essential for effective repair of bridge members. For example, closing the shoulder and lane(s) above the beam during CFRP system application minimizes stresses in the beam due to superimposed live loads so the CFRP can engage to resist such loads when the beams are subjected to them under full service conditions. If the beam still carries live load at the time of installation, the CFRP may not engage as well under service conditions and the strengthening/repair will be less effective. For a bonded structural repair, the load-induced stresses in the base material to which the repair material is to be bonded should be minimized so that the new material can be integrated with the existing member to share superimposed load. TxDOT uses a 40 kip (180 kN) truck placed above the damaged region to pre-load the span prior to patching the concrete. The pre-load serves to pre-tension the bottom of the beam where concrete is to be patched. As a result, the newly placed patch concrete will undergo a small amount of pre-compression produced by subsequent removal of the pre-load after the patched concrete cures. It should be emphasized that the pre-load must be removed, and the beam

isolated from superimposed loads, before installing the CFRP system. The ability of the structure to support the pre-load during the concrete repairs should be assessed.

Concrete Columns

Concrete columns at bridge bents have been damaged by stray vehicles that leave the roadway and impact the columns. Depending on the magnitude of the impact force, the impact could collapse the bridge or simply damage the columns. In the past five years, there have been eleven on-system bridges in Texas where the columns have been impacted by large trucks. In six such incidents, the impacted columns were so severely damaged that replacement was the only viable option; however, none of the incidents resulted in collapse of the superstructure. In the remaining five incidents, the columns were severely cracked but were deemed repairable. These columns were repaired using CFRP for flexural strengthening and confinement. In one of these cases, the superstructure over the damaged column was shored using shoring towers to release both the dead and live loads from the column to engage more of the CFRP. Figure 11 shows one of the five cases, which was repaired with typical procedures. The fractured column exhibited three large horizontal flexural cracks with widths from 0.013 in. (0.33 mm) to 0.06 in. (1.5 mm) and spaced at an average of 14.5 in. (370 mm). The cracks were on the tension side of the impact point approximately five ft (1.5 m) above the ground. The cracks extended through 60% of the column's section. Shear cracks and a small area of crushed concrete in the compression side were observed at the top of the column. In this case TxDOT engineers considered the severity of the damage as follows. First, it is apparent that the vertical reinforcing bars on the extreme tension side of the impact in the cracked section have yielded. The strain in the reinforcing bars at that location can be estimated by dividing the crack width by the average space between adjacent cracks. In this case, the calculated strain (0.41%) in the vertical reinforcing bars is much greater than their yield strain (0.14%). The column is also slightly bent. Therefore, the column's capacity may have been compromised. Second, the locally crushed concrete at the top of column indicates that some flexural strength may have been lost at the section beneath the bent cap. Finally, the diagonal shear crack at the top of the column indicates that the column's lateral shear capacity may have been affected. TxDOT engineers determined that a CFRP strengthening and confinement repair was viable after considering the following: (1) The column is basically a compression member with very low flexural loading; (2) the sections separated by large cracks can be assumed to act similarly to concrete blocks in a masonry structure that are capable of carrying compression force after the cracks are sealed by injecting epoxy to integrate the blocks as a whole; and (3) the vertical reinforcing bars, though yielded, can still carry compression force. In this case, simply patching the spalled concrete and injecting the cracks with epoxy will not adequately restore the integrity and the designed flexural/shear strength of the column. CFRP strengthening and confinement can provide this additional strengthening.

Figure 12 shows the design details for the CFRP repair of the damaged column. It was assumed that two of the 8 vertical bars were yielded. To restore the flexural strength three plies of vertical CFRP strips were used on the tension side of the impacted column to supplementing the yielded reinforcing bars. The amount of CFRP plies was calculated using an effective strain of 0.4% in carbon fibers (a tensile strain limit used in ACI 440.2R), the net-thickness of carbon fiber fabrics, and the tensile modulus of the fibers given by the system's manufacturer. Continuous horizontal CFRP wraps are placed at the "hinge areas" at the top and bottom of the column to improve the shear strength (by stirrup effect) and also serve to anchor the vertical strips. In addition, 12 in. (300 mm) horizontal CFRP strips spaced at 12 in. (300 mm) are provided to supplement the confinement provided by the spiral reinforcement in the column to enhance the compressive strength and the integrity of the column. These measures are based on TxDOT engineers' judgment and experience. Figure 13 shows the repair of an impacted column in the case described and shown in Figure 11.

REPAIR OF FIRE DAMAGED CONCRETE BRIDGES

TxDOT has used CFRP in combination with conventional materials and techniques to repair fire damaged concrete bridge members. Fire damage of bridges is rare but occurs occasionally when vehicles ignite or explode beneath or near bridge spans and the resulting elevated temperature is high enough to damage the concrete cover. The heat dehydrates the concrete, depriving it of its stored pore water. This process can cause the cover concrete to weaken and pop off and reduces the compressive strength in the heat-affected areas. The reduction of strength is significant when the temperature in concrete exceeds 570 °F (300°C). The concrete could lose more than 70% of its original strength when the temperature exceeds 930 °F (500°C)^{13, 14}. Concrete exposed to damaging high temperatures may change to a pink color¹⁴. This pink discoloration can be an important indicator when assessing the depth of the damage and remaining structural integrity because of its association with strength loss. The heat may also result in cracking, delamination, and spalling from the expansion of aggregates and steel reinforcement. Figure 14 illustrates

the damage and repair of a fire damaged cast-in-place concrete bridge. The May 2005 incident occurred when a tanker truck went over the bridge rail, slid under the bridge, and burst into flames. Five beams, three columns, and the south-east interior bent cap were severely damaged. Structural assessment for fire damage should not be based solely on appearance. All critical sections and boundaries of damaged members should be checked by carefully chipping and removing damaged concrete to expose layers of discoloration and get to sound concrete (Figure 14b).

A thorough inspection revealed that the depth of damaged concrete was limited to the concrete cover in most areas. In some areas the depth was slightly deeper than 2 or 3 in. (50 mm or 75 mm) from the original surface. The core concrete within the reinforcing steel cage remained sound. However, the loss of 2 or 3 in. (50 mm or 75 mm) of concrete in the compression zone of bent caps and girders can be significant. Based on coloration, the steel reinforcement appeared to be undamaged; indicating that the concrete cover provided adequate thermal protection. Field engineers determined that the main body of the structure remained in fair condition and the damaged portions of the structure could be repaired by using CFRP in combination with enhanced concrete repair practices. Figures 14d thru Figure 14g show the major repair processes. The damaged portions of girders and columns were restored by removing all unsound concrete using small chipping hammers followed by abrasive blasting to prepare the surface. Steel pins were used to attach welded wire mesh to the concrete. Shotcrete was then applied to a saturated surface dry substrate. The bent cap with significant spalling was encapsulated by new reinforced concrete. After all girder sections were repaired and cured, one ply of longitudinal CFRP was placed on the underside of the girders with "U" wraps around the sides and bottom to enhance the integrity of the repaired concrete section as well as to reinforce the shotcrete cover concrete. Two plies of CFRP wraps were applied horizontally around each column to provide confinement and to restore the structural integrity and indirectly strengthen the columns. The bridge was repaired with an emergency maintenance contract in 24 days at a cost of \$640,000 (Figure 14h). In this project a saturator for pre-impregnation (pre-preg) of the fabric sheets was employed (Figure 14e). This device was used to fully saturate the carbon fiber fabrics with the resin prior to layup, known as "wet layup". However, to ensure sufficient saturant resin is supplied for the bonding layers and the polymer matrix, both the under-coat and the over-coat saturant resins should be applied.

REPAIR OF CORROSION DAMAGED CONCRETE MEMBERS

TxDOT has used CFRP to strengthen concrete members damaged by corrosion of steel reinforcement. Figure 15 shows typical corrosion damage. A bent cap was severely deteriorated by the corrosion of steel due to water penetrating through an unsealed deck joint. Several stirrups had 100% section loss in bottom legs due to corrosion. The damaged section was near the face of the column – a critical section for shear capacity of the bent cap. The reduction in shear capacity was a serious concern. In this case, the corrosion problem was addressed first. All unsound concrete was removed. Once concrete removal was performed, a chipping hammer was used to completely expose the entire perimeter of the rusted steel bars to provide a minimum clearance of 0.75 in. (20 mm) between the steel bar and surrounding concrete. The entire area, including exposed steel, was cleaned by sandblasting followed by a high-pressure air blast for final cleaning. To prevent future corrosion, embedded galvanic anode units were connected to each reinforcing bar in a manner recommended by the anode manufacturer (Figure 16a). After that, the area was repaired using a non-shrink cementitious concrete mix that could achieve a compressive strength of 4000 psi (28 MPa) in 48 hours, at which time CFRP installation was to begin. The damage was assessed by comparing the shear capacity calculated at the damaged section, ΦV_n , with the factored design shear force. The corroded shear stirrups resulted in a 35% reduction in shear capacity. CFRP shear strengthening was deemed to be a viable application for this situation. Figure 16b shows the CFRP shear strengthening details. The shear reinforcing design was based on guidelines provided by ACI 440.2R-02 (2002)¹⁵. The CFRP reinforcement was designed to supplement the lost primary shear reinforcement and restore the bent to its original shear capacity. Figure 17a shows the ports set for epoxy injection to seal internal cracks. Crack sealing is an important procedure for concrete repairs and is normally done after concrete repairs are completed because the patch material back-seals and stops the leakage of epoxy from internal cracks. Figure 17b shows the installed CFRP sheets. Workers drew lines on the prepared surface to accurately orient the CFRP sheets per the details. The total cost of the repair was \$15,645.72 and took 4.5 working days. The repaired cap was inspected in Oct. 2009. The repair and CFRP strengthening appeared to be in good condition (Figure 17c). ACI 440.2R-08¹ recommends an FRP strengthening limit based on an assessment of the existing strength of the member before and after strengthening as follows: $(\Phi R_n)_{existing} \geq (1.1S_{DL} + 0.75S_{LL})_{new}$.

REPAIR OF ASR DAMAGED CONCRETE COLUMNS

TxDOT has also used CFRP to provide structural confinement to restrain expansion caused by alkali-silica reaction (ASR) in concrete columns. ASR, the most common form of alkali-aggregate reaction in concrete, can cause serious expansion and cracking in concrete, resulting in major structural problems and sometimes necessitating demolition. The product of ASR is an alkali-silica gel that increases in volume by taking up water and in the process exerting an expansive internal pressure that can result in cracking and failure of the concrete. Cracking results when this pressure exceeds the concrete's tensile strength. Figure 18 shows ASR damage in the substructure of a bridge. Most of the columns and several bent caps were substantially cracked due to ASR. Unlike map or random cracking caused by ASR in unrestrained members, in these columns the major cracks are vertically oriented. This is because the columns are vertically restrained by the compression and the primary reinforcement. The relatively light transverse reinforcement provided by spirals is not sufficient to restrain the expanding concrete resulting in the vertical cracks. This is the first case where TxDOT used CFRP to confine ASR expansion to mitigate damage and extend the structure's service life. Figures 19a to 19f show the surface preparation process and the installation of CFRP confinement. The structure was approximately ten years old when the repairs were made. TxDOT contacted several ASR experts to determine the swelling pressure resulting from ASR. The consensus response at the time was that ASR could result in a swelling pressure of up to 600 psi (4.2 MPa). The petrography report on samples of concrete removed from the structure indicated that some of the ASR potential reaction had been consumed so the resulting CFRP design was based on providing confinement to restrain a swelling pressure of 500 psi (3.5 MPa). Circumferential CFRP wraps provided the primary hoop confinement with vertical strips providing secondary reinforcing. Windows or gaps were left in the CFRP so any additional cracking could be monitored. The repair was done under an emergency contract. The bridge crosses a normally full reservoir. Drought conditions had significantly lowered the lake level providing dry-land access to almost all of the columns. The soil around the bottom of each column was excavated approximately 1 ft (300 mm) to expose the construction joint between drill shaft foundation and the column. All surface areas from the excavated areas up to the top of the cap of bents or abutments were sand-blasting cleaned including cleaning debris out of large cracks. Two coats of alcohol-based Silane were applied to all exposed surfaces. This solution accelerates the drying process to about 24 hours instead of the normal four to five days. All cracks greater than 0.005 in. (0.13 mm) were sealed with paintable Acrylic Latex Caulk. The intent was to waterproof the concrete to the maximum extent possible while still CFRP was applied to the lower portion of all columns to provide structural confinement and was painted to provide UV protection for the CFRP wraps. The project, completed in 2003, took 45 days and cost \$376,384, including CFRP Confinement of 6,225 ft² (580 m²) at a cost of \$250,868, plus surface treatment and paint of 22,566 ft² (2,100 m²) at a cost of \$100,644. The rehabilitated bridge is shown in Figures 20a and 20b. The bridge has been closely monitored and inspected since 2003. No distress to the CFRP system was noted in both the 2005 and 2007 inspections. In May 2009, TxDOT again inspected the columns. The CFRP confinement system appeared to be holding up and performing very well as shown in Figure 21. In a few areas near the splash zones, caulk and some paint over old cracks in the window areas seemed to be beginning to peel off. However, no new cracks were observed, and overall the confinement repairs seem to be performing well. The performance to date demonstrates that CFRP confinement, in combination with viable surface treatment, can mitigate ASR damage and shows promise for future applications. A recently completed TxDOT research project has shown that the use of CFRP confinement can effectively confine column elements that are affected by expansive mechanisms such as ASR and Delayed Ettringite Formation (DEF)¹⁶.

RECOMMENDATIONS AND LESSONS LEARNED

Externally bonded CFRP composites are an excellent tensile reinforcement for structural repairs, strengthening, and rehabilitation of concrete bridge members. They offer exceptional mechanical properties, simplicity and flexibility of installation. CFRP composites can reduce user costs, measured in terms of road closures and delays, which traditional reinforcement and repair methods cannot offer. The combination of continuous carbon fibers with epoxy resin has been found to be particularly compatible with steel reinforced concrete and as a result the use of CFRP in repair and strengthening concrete structures has accelerated. Over the years TxDOT has broadened its use of CFRP to "sacrificial" reinforcement for repair of repeated impact-damaged prestressed concrete beams and to structural confinement for repair and mitigation of ASR-damaged concrete columns. These applications have shown great promise for addressing structural repair needs for damaged concrete bridge members. To make CFRP use more effective for bridge repair and strengthening applications the authors recommend the following:

- The designer should select proprietary CFRP systems with a complete package of wet layup techniques and materials for the application, including the system components, i.e. the primer, putty, and epoxy resin of

saturant, carbon fiber fabrics, and protection coatings, as well as construction specifications. The components should be well tested and compatible for the intended application. Manufacturers should provide technical support and training, and have a history of successful applications on bridge projects. Alternative designs with equivalent performance should be allowed in addition to the approved systems.

- Designs for wet layup installation should be based on the net-area properties of the carbon fiber fabrics of the CFRP for strengthening and confinement design following the procedures outlined in ACI 440.2R-08 *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*¹. This is a comprehensive reference for design and construction of externally bonded CFRP systems. The design concepts, assumptions, and methodology of this document are consistent with the AASHTO provisions for reinforced concrete bridge members. Design procedures based on net-fiber area properties are based on the assumption that the carbon fiber is the basic load carrying element in the CFRP composite. The area of polymer matrix is omitted from the design due to the variation of the thickness of field applied saturant resin. The tensile strength of cured epoxy resin is only 1/50 of the tensile strength of carbon fibers; therefore its omission from strength calculation is reasonable. Furthermore, the elongation (or the rupture strain) of cured epoxy at the peak ranges from 3.5% to 5.0% which is 2 to 3 times the rupture strain of the carbon fiber (1.5% to 1.67%). This means that the carbon fibers will rupture much earlier than the epoxy resin and thus failure is controlled by the carbon fibers. This difference in the rupture strain between epoxy polymer matrix and carbon fibers allows the full strength of the fiber component to be achieved.
- Structural repairs should ensure that the repair materials (concrete and CFRP) carry loads and thus are integrated with the existing member as part of its load carrying function. To achieve these purposes thoroughly engineered procedures, such as reducing or diverting loads (closing traffic lanes), structural shoring (changing loading and boundary condition), and pre-loading (pre-tension the patched area) should be used in the repair process.
- The bond between CFRP and concrete substrate is critical for flexural and shear strengthening – a “bond-critical application”. Efforts should be made to ensure that the damaged concrete has been thoroughly repaired, and the substrate surface has been carefully prepared before the CFRP installation. For wet layup installation, the saturant resin that serves as both bonding layer and the polymer matrix for CFRP should be sufficiently applied. TxDOT treats CFRP confinement – a “contact-critical application” – the same as the “bond-critical application” in terms of the installation. Because CFRP confinement is contact-critical, direct pull-off tests for the bond may not be necessary. See NCHRP Report 609³ for recommendations regarding FRP construction specifications for repair and retrofitting of concrete structures and quality control.
- Surface preparation on substrate concrete is critical for the performance of externally bonded FRP systems. NCHRP Project 10-59B³ identified concrete substrate surface preparation as one of the threshold issues that affects long-term performance of an FRP system. The concrete substrate should be thoroughly prepared using methods such as grinding, abrasive-blasting, and high-pressure air-blasting to provide a clean, smooth, and flat or convex surface with a surface profile of CSP 2 or CSP3, as defined by ICRI¹⁷, providing continuous and intimate contact between the CFRP and the concrete substrate.

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Fig. 1—Wet layup installations



a) The impact damaged beam.



b) A continuous CFRP "U" wrap was used.



c) The repaired beam after 5 years later.

Fig. 2—First impact damaged P/S beam repaired with CFRP



Fig. 3—impacted in 2004



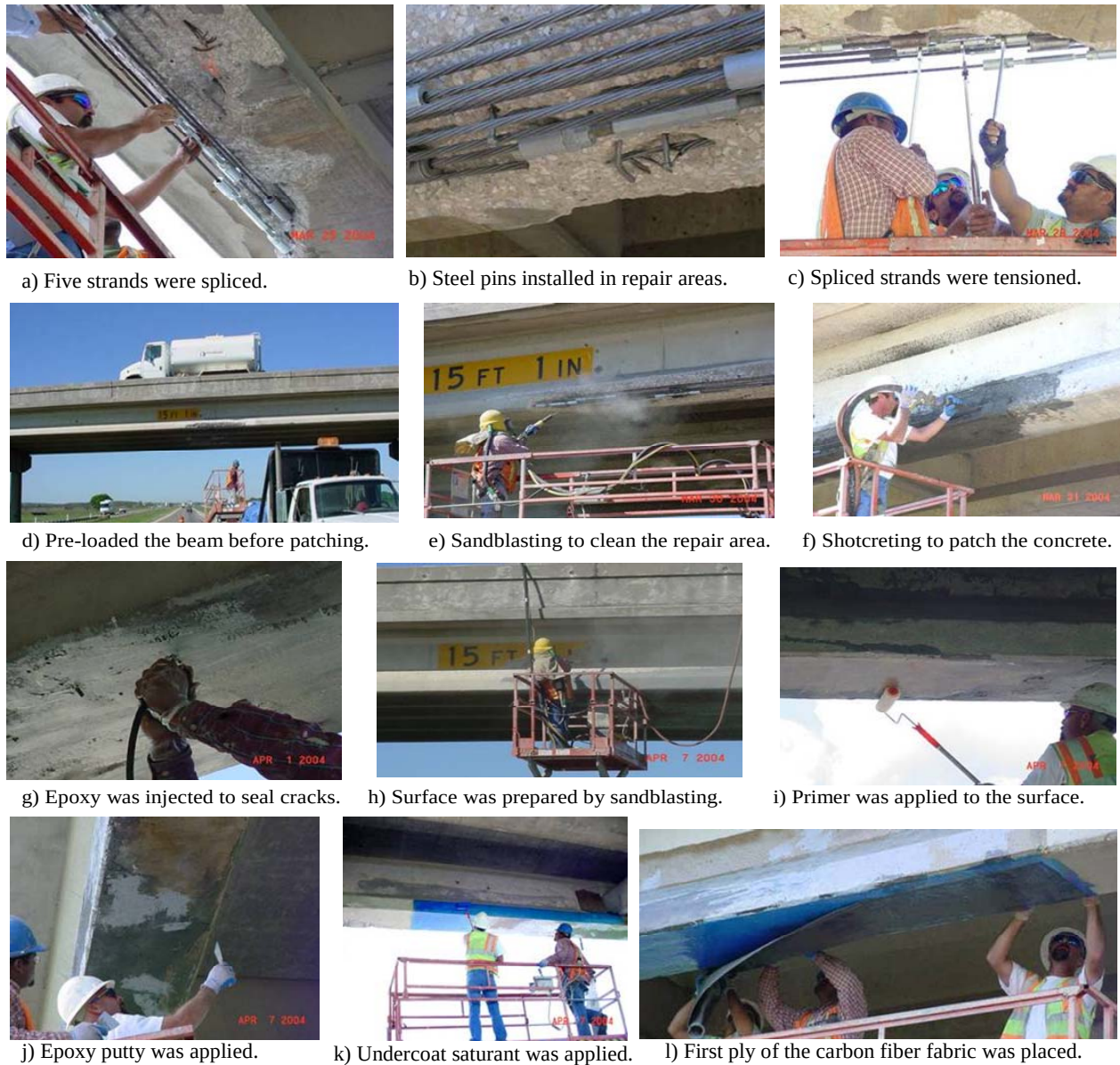


Fig. 4—Major procedures for repair of severed strands, patching and injecting concrete, and installing CFRP system.

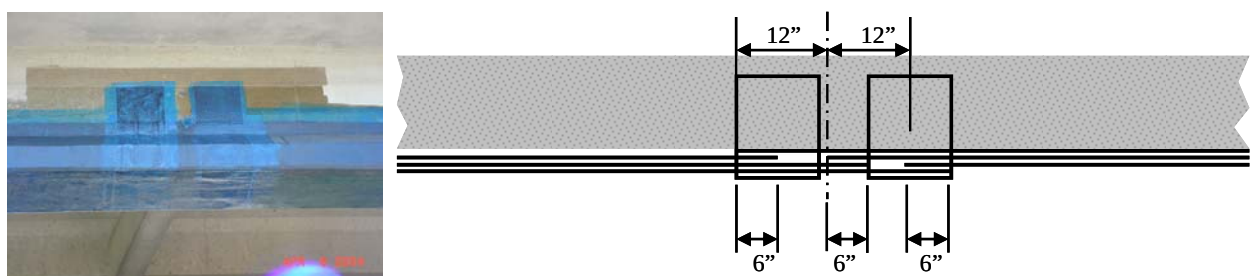


Fig. 5—Overlap splice with additional “U” wraps at mid-span to phase installation for traffic flow