Synthesis of Repair Practices of Damaged Precast/Prestressed Concrete Girders

SPR-P1 (19) M090

March 2020

Synthesis of Repair Practices of Damaged Precast/Prestressed Concrete Girders

A Report on Research Sponsored by

Nebraska Department of Transportation (NDOT)

Principle Investigator

George Morcous, Ph.D., P.E.
Professor
Durham School of Architectural Engineering and Construction
University of Nebraska-Lincoln

Co-Principle Investigator

Richard L. Wood, Ph. D.
Assistant Professor
Department of Civil Engineering
University of Nebraska-Lincoln

Graduate Research Assistant

Antony Kodsy, M.Sc.
Durham School of Architectural Engineering and Construction
University of Nebraska-Lincoln

March, 2020

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
SPR-P1 (19) M090		

4. Title and Subtitle Synthesis of Repair Practices of Damaged Precast/Prestressed Concrete Girders	**5. Report Date** March, 2020 **6. Performing Organization Code**
7. Author(s) Antony Kodsy, George Morcous, and Richard L. Wood	**8. Performing Organization Report No.**
9. Performing Organization Name and Address University of Nebraska – Lincoln 1400 R Street Lincoln, NE 68588	**10. Work Unit No.** **11. Contract** SPR-1(19) (M090)
12. Sponsoring Agency Name and Address Nebraska Department of Transportation 1400 Highway 2 PO BOX 94759 Lincoln, NE 68509	**13. Type of Report and Period Covered** Final Report July 2018 – March 2020 **14. Sponsoring Agency Code**

15. Supplementary Notes

16. Abstract

Bridge girders are constantly subjected to various types of damage during their service life. There is currently limited knowledge or guidelines provided by the NDOT Bridge Office Policies and Procedures (BOPP) regarding the assessment and repair procedures of damaged precast/prestressed concrete girders. This report aims to develop a comprehensive repair manual for precast/prestressed concrete girders subjected to damage caused by over-height vehicular collision and damage located at the girder ends. Over-height vehicular collision damage typically occurs at the middle portion of the exterior girders and the primary concerns are focused on flexural deficiencies. Girder end damage can occur due to corrosion of prestressing strands or reinforcement, malfunctioning joints, or during deck/abutment replacement; where the primary concerns are focused on shear deficiencies. When damage occurs, the decision-making process regarding whether to repair, rehabilitate, or replace the girder is typically challenging. A literature review on the classification of damage and a proposed damage classification are presented for each damage type. Repair methods and procedures for each damage class are then presented for each damage type. Previous repair cases done by NDOT are documented and their performance is evaluated by visual inspection records. Ultimate limit state structural calculations are presented in the form of design examples to calculate the flexure or shear strength of the undamaged and damaged girder according to AASHTO LRFD. The ultimate flexure or shear strength of a strengthened girder using FRP wrapping is also presented according to ACI 440.2R-17 as a design example. Suggested material properties are presented for each repair method according to previous research work and previous NDOT repair cases.

17. Key Words	18. Distribution Statement
Repair, Damage Assessment, Bridge Structure, Vehicle Collision, Girder End	No restrictions. This document is available through the National Technical Information Service. 5285 Port Royal Road Springfield, VA 22161

19. Security Classification (of this report)	20. Security Classification (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	151	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Contents

Table of Figures

List of Tables

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. The contents do not necessarily reflect the official views or policies neither of the Nebraska Department of Transportations nor the University of Nebraska-Lincoln. This report does not constitute a standard, specification, or regulation. Trade or manufacturers' names, which may appear in this report, are cited only because they are considered essential to the objectives of the report. Recommendations made in this report are based on work done by other researchers.

The United States (U.S.) government and the State of Nebraska do not endorse products or manufacturers. This material is based upon work supported by the Federal Highway Administration under SPR-1(20) (M090). Any opinions, findings and conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the Federal Highway Administration."

Abstract

Bridge girders are constantly subjected to various types of damage during their service life. There is currently limited knowledge or guidelines provided by the NDOT Bridge Office Policies and Procedures (BOPP) regarding the assessment and repair procedures of damaged precast/prestressed concrete girders. This report aims to develop a comprehensive repair manual for precast/prestressed concrete girders subjected to damage caused by over-height vehicular collision and damage located at the girder ends. Over-height vehicular collision damage typically occurs at the middle portion of the exterior girders and the primary concerns are focused on flexural deficiencies. Girder end damage can occur due to corrosion of prestressing strands or reinforcement, malfunctioning joints, or during deck/abutment replacement; where the primary concerns are focused on shear deficiencies. When damage occurs, the decision-making process regarding whether to repair, rehabilitate, or replace the girder is typically challenging. A literature review on the classification of damage and a proposed damage classification are presented for each damage type. Repair methods and procedures for each damage class are then presented for each damage type. Previous repair cases done by NDOT are documented and their performance is evaluated by visual inspection records. Ultimate limit state structural calculations are presented in the form of design examples to calculate the flexure or shear strength of the undamaged and damaged girder according to AASHTO LRFD. The ultimate flexure or shear strength of a strengthened girder using FRP wrapping is also presented according to ACI 440.2R-17 as a design example. Suggested material properties are presented for each repair method according to previous research work and previous NDOT repair cases.

Chapter 1 – Introduction

1.1 Project Overview

This report aims to document the current repair practices used for damaged precast/prestressed concrete girders. During their service life, bridge girders are subjected to multiple causes of damage such as the accidental collision of over-height vehicle/equipment, incidental damage during abutment/deck replacement, malfunctioning of supports/joints, and/or corrosion of reinforcing steel/strands. Over-height vehicular collision impact damage is typically evident at the girder's middle sections, while most other damage sources are evident at girder ends. Girder damage will be divided into damage caused by the collision of over-height vehicles, and damage at the girder ends. For each damage type, the classification of damage and suggested repair methods and procedures will be presented. The report also will document previous experiences done by NDOT on these damage cases. The suggested damage classification, repair methods, and procedures are based on the recommendations of other project reports, state or provincial department of transportation offices, and NDOT previous repair cases. The report also will present some design examples to help in the structural calculations of damage assessment and repair selection.

1.2 Problem Statement

There is currently limited knowledge or guidelines provided by the NDOT Bridge Office Policies and Procedures (BOPP) regarding the assessment and repair procedures of damaged precast/prestressed concrete girders. Also, there is no manual or guidelines to help contractors make time and cost-effective repair decisions at these incidents.

1.3 Project Objective and Scope

The objective of the project is to develop a comprehensive repair manual subjected to damage caused by over-height vehicular collision and damage located at the girder ends. The manual shall describe damage classification, and repair methods and procedures. The scope of the project is limited to I-shaped precast/prestressed concrete girders (e.g. AASHTO, and NU bridge girders).

1.4 Report Outline

Chapter 2 presents an overview of the possible damage assessment techniques that could be used to investigate the extent of damage to the girder.

Chapter 3 focuses on vehicular collision damage. First, a literature review is presented for the classification of damage levels. Based on the review, a proposed method is summarized and presented to classify the damage with the recommended repair methods for each class. Then, the recommended repair methods are discussed in more detail. The end of this chapter presents the proposed methods and procedures for each damage class.

Chapter 4 focuses on damage at the girder ends. First, a literature review is presented for the condition rating assessment for prestressed concrete bridge girders. Based on the review, a proposed method is summarized and presented to classify the damage with the recommended repair methods for each class. Then the recommended repair methods are discussed in more detail. The end of this chapter presents the proposed methods and procedures for each damage class.

Chapter 5 presents a summary of the report findings.

Appendix A presents a review of six previous repair/replacement cases done by NDOT after vehicular collision damage. These cases are documented to support the proposed damage classification and repair methods presented in chapter 3.

Appendix B presents a review of four previous repair/rehabilitation cases done by NDOT after girder end damage. These cases are documented to support the proposed damage classification and repair methods presented in chapter 4.

Appendix C presents a summary of damage classification and repair methods and procedures for over-height vehicular collision damage that can be used as an inspection/reference manual.

Appendix D presents a summary of the damage classification and repair methods and procedures for girder end damage that can be used as an inspection/reference manual.

Appendix E presents structural calculations design examples. Several Mathcad design examples are presented to provide some guidelines to the structural engineer when selecting some specific repair methods.

Chapter 2 - Damage Assessment Techniques

Harries et al., 2012 provides a review of the non-destructive testing and evaluation techniques for assessing prestressed concrete elements. Several damage assessment techniques are discussed but not all techniques are applicable to bridge girders with corrosion damage. Viable techniques to assess damage to concrete bridge girders are highlighted below.

2.1 Visual Inspection

The initial damage inspection should be conducted visually. Visual inspection of the concrete surface for cracks extending over the reinforcement length may provide preliminary information about the structure. However, it is dependent on the skill level of the operator and while it provides qualitative information, it does not provide quantitative information about the extent of the damage. Except in the presence of significant damage, only information about the cover concrete and outermost layers of steel may be assessed.

2.2 Manual Inspection

Visual inspection can be conducted with the help of some tools such as a chipping hammer and magnifying glass. A simple method of assessing the condition of concrete is to use a sounding technique. This invokes tapping the surface with a hammer and listen to the resulting tone. A high-frequency pitch indicates a sound concrete whereas a lower frequency pitch indicates the presence of flaws. Sections that are chipped during the inspection must be repaired afterward. For multi-strand tendons, the 'screw-driver-test' tests the state of the tendon by trying to wedge a flat-head screwdriver between strands. Nonetheless, only a limited number of wires can be visually inspected. With great care, the screw-driver test may be conducted on individual strands.

2.3 Half-cell Potential Survey

Where corrosion of non-visible reinforcement or strand is suspected, the surface potential survey/half-cell potential survey is a well-established standardized inspection technique (ASTM C876). It is presently the most viable and widely used in situ approach alongside visual and other manual forms of inspection. The entire surface is mapped by recording the surface potentials with respect to a reference electrode. Locations with higher negative potentials indicate areas of corrosion. This is a quick and inexpensive method that may be used during the planning of areas that need repair. However, results may be affected by the degree of humidity of concrete, oxygen content near the reinforcement, existence, and extent of micro-cracks, or stray electrical currents. Due to these reasons, ASTM has specified certain conditions where the technique should not be applied. Among these conditions is that concrete surfaces that are coated or treated with sealers may not provide an acceptable electrical circuit. In addition, Concrete surfaces in building interiors and desert environments lose sufficient moisture so that the concrete resistivity becomes significantly high which will require special techniques not covered by the ASTM test method.

2.4 Remnant Magnetism

This technique is useful to get information about the location of prestressing steel fractures and the degree of damage to a strand. The process is performed by an electromagnet along the direction of the tendon. Fractures and breaks in the prestressing tendons are detectable but the size of the defect or loss of section is not. Limitations of the method are related to the density of the reinforcement present and the minimum degree of damage that is sought. The method can be applied on the vertical face or the bottom face of a member. The magnetic properties of the steel change with different levels of prestressing. The fracture can be detected even if it is screened by other wires or if the resulting gap in the steel is relatively small. Commercially available systems are primarily aimed at the detection of flaws/damage in prestressed slabs. Available systems could be readily adapted to high-speed applications on bridge soffits.

2.5 Acoustic Emission

This technique can be used to identify new cracks but will not provide information on previously existing damage. The method is applicable for real-time health monitoring of a bridge or a girder. AE was very successfully used to quantify and precisely locate damage to two prestressed box girders tested to failure. AE monitoring was performed on an elevated portion of the I-565 interstate highway in Huntsville, Alabama, to investigate the feasibility of using AE testing to assess the performance of prestressed concrete bridge girders.

2.6 Nebraska Method

The Nebraska method was developed to measure the effective prestress in prestressed concrete bridge girders. A cylindrical hole having a diameter of 1 in. is drilled into the concrete and then a crack is induced in the hole. This crack extends in the direction parallel to the main axis of stress. Then, an external force is applied perpendicular to the direction of the crack and the stress necessary to close the crack is determined. This value is then related to the effective prestress. Although special hardware has been developed to clamp to the underside of the bridge girders, it may still be difficult to apply this method in situations where the geometry does not allow it. So, its applicability is limited in this sense. The method has limited application and calibration is not certain, hence this method is not anticipated to be practical for in-situ assessment.

2.7 Rebound Hammer

According to Feldman et al., 1996, among the non-destructive testing methods, a Schmidt hammer, also known as a rebound hammer, works well to determine areas of delaminated and cracked concrete. Although the Schmidt hammer is usually used to get an indication of in-situ concrete strength, it can also be used to indicate internally damaged areas within a concrete member. To determine areas of unsound concrete, a Schmidt hammer is used in the same manner as for determining concrete strength at different locations within the girder. Areas of extensive internal cracking and delamination will yield lower Schmidt hammer readings than areas of sound concrete.

Chapter 3 - Vehicular Collision Damage

3.1 Classification of Damage

This section presents a background about the previous literature used to develop the NDOT classification of vehicular collision damage. Different project reports and state manuals are presented with the focus on the damage classification and suggested repair or replacement decisions.

3.1.1 Feldman et al., 1996

This report was a Federal Highway Administration (FHWA) research project conducted for the Texas Department of Transportation and done at the University of Texas at Austin. Another report number FHWA/TX-97 /1370-3F was published for the same project in 1997. Both reports mention that the occurrence of impact damage within the state of Texas alone was estimated as 241 incidences over a five-year period between the years 1987 and 1992, and 1008 incidents across other states. Impact damage to bridge members was divided into three main classes.

3.1.1.1 Minor Damage

In this work, minor damage includes concrete cracks, nicks, shallow spalls, and scrapes. Figure 1 shows a sample example for minor damage. Figure 2 shows a step-by-step repair procedure for minor damage. Two-thirds (66%) of incidents of minor damage that were observed in Texas during the mentioned five-year period were not repaired.

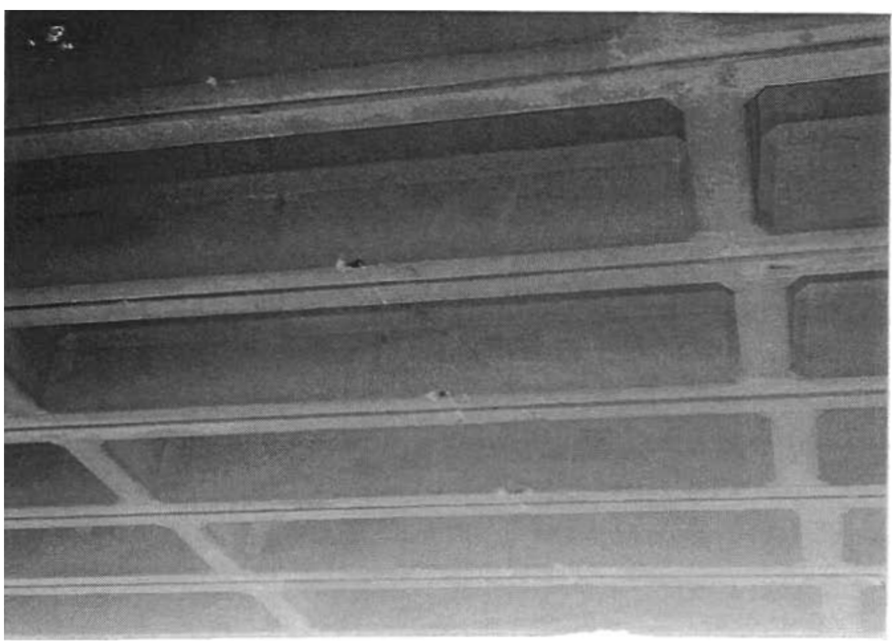


Figure 1: Minor damage, courtesy of Feldman et al., 1996

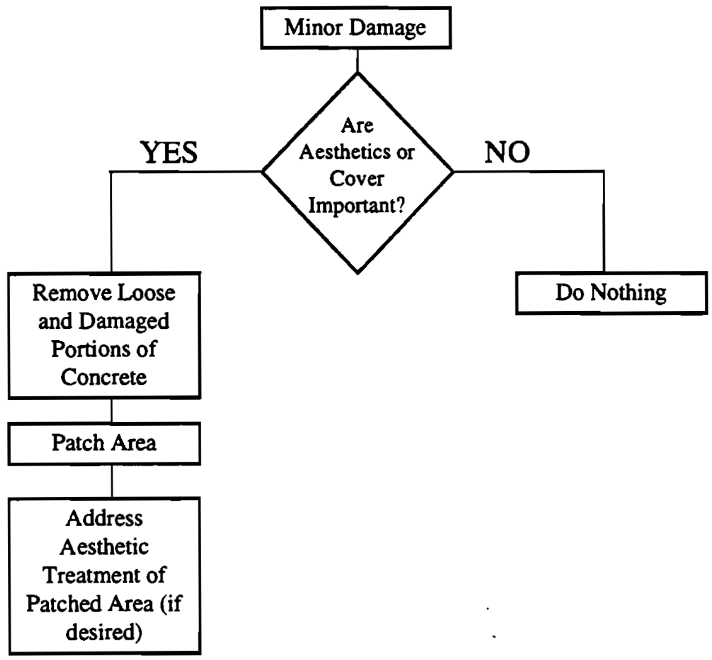


Figure 2: Minor damage step-by-step repair procedure, courtesy of Feldman et al., 1996

3.1.1.2 Moderate Damage

In this work, moderate damage includes large concrete cracks and spalls, exposed undamaged tendons. Figure 3 shows a sample example for moderate damage. Figure 4 shows a step-by-step repair procedure for moderate damage. Only 14% of incidents of moderate damage that were observed in Texas during the mentioned five-year period were not repaired.

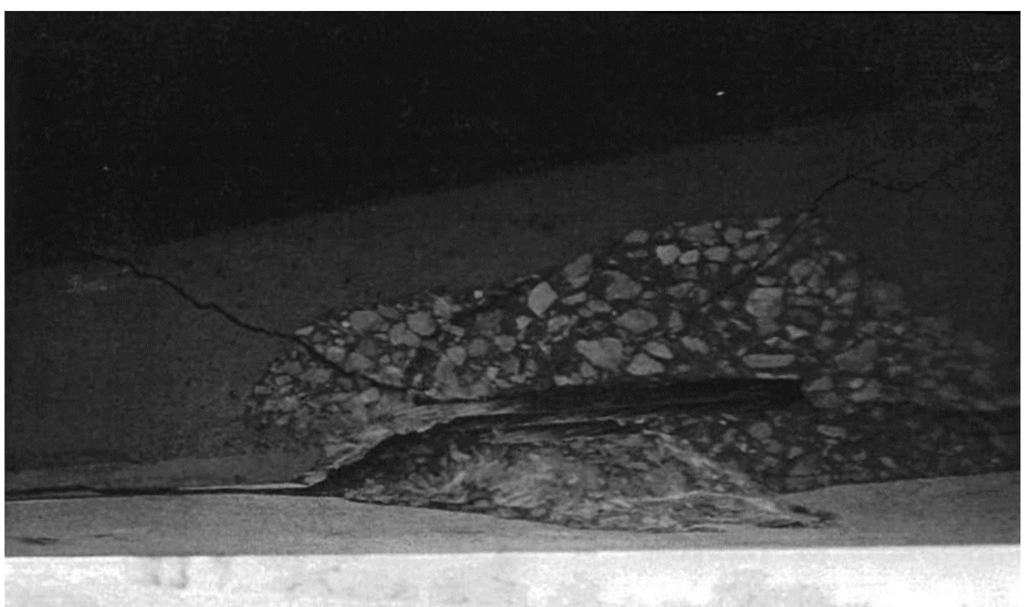


Figure 3: Moderate damage, courtesy of Feldman et al., 1996

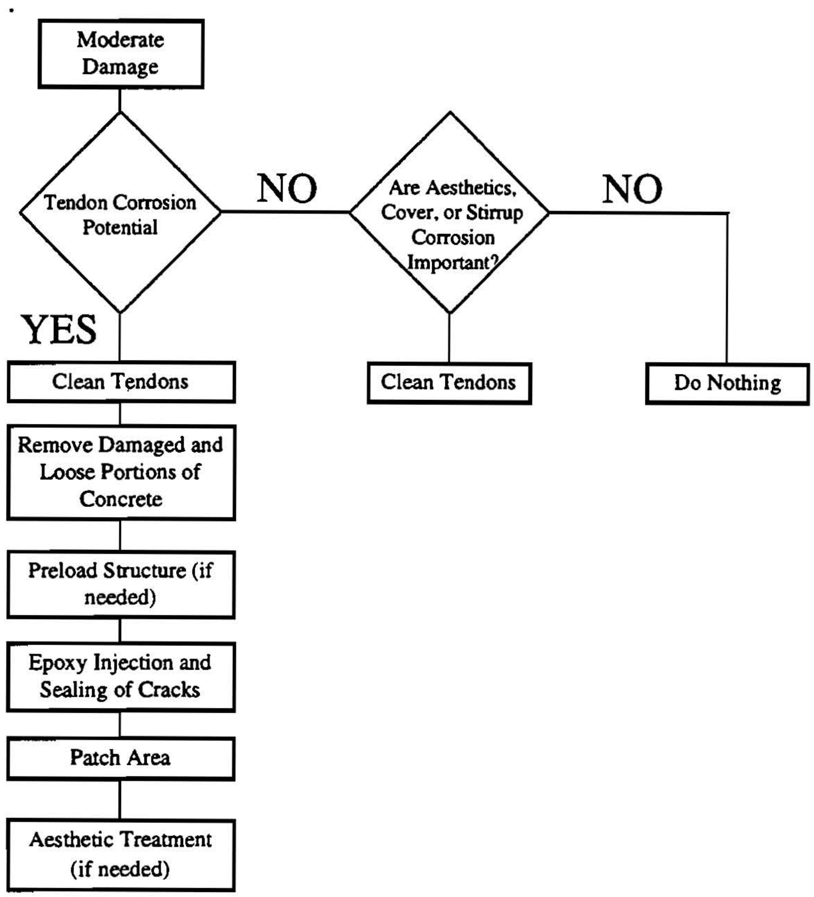


Figure 4: Moderate damage step-by-step repair procedure, courtesy of Feldman et al., 1996

3.1.1.3 Severe Damage

In this work, severe damage includes exposed and damaged tendons, loss of a significant portion of the concrete section, distortion or misalignment of the girder. Figure 5 shows a sample example of severe damage. Figure 6 shows a step-by-step repair procedure for severe damage. In only one case of severe damage recorded between 1987 and 1992 were the severed strands in a girder repaired. The repair consisted of using internal splices to repair the severed strands.

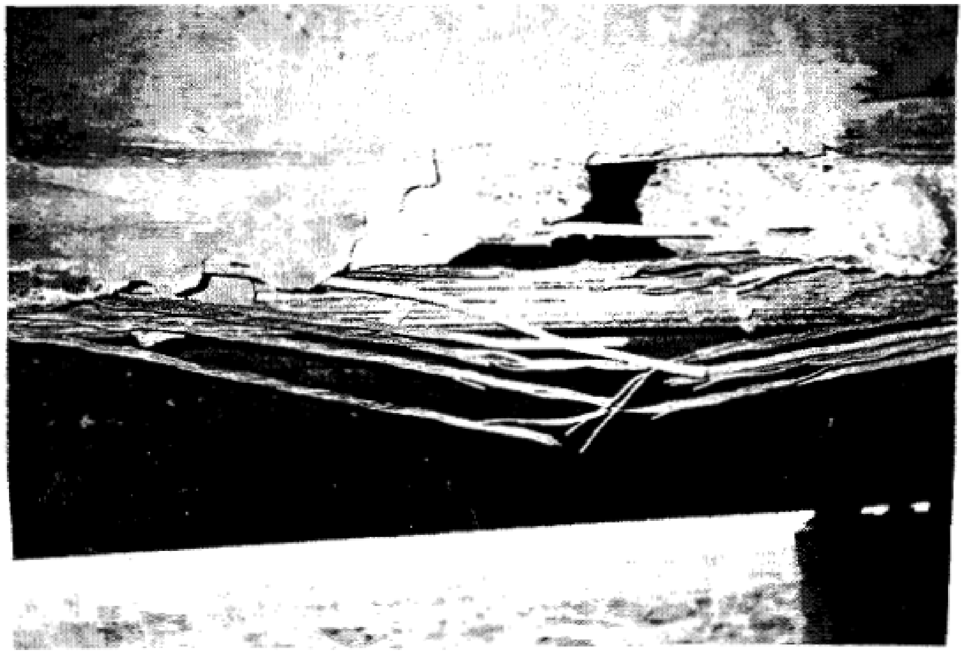


Figure 5: Severe damage, courtesy of Feldman et al., 1996

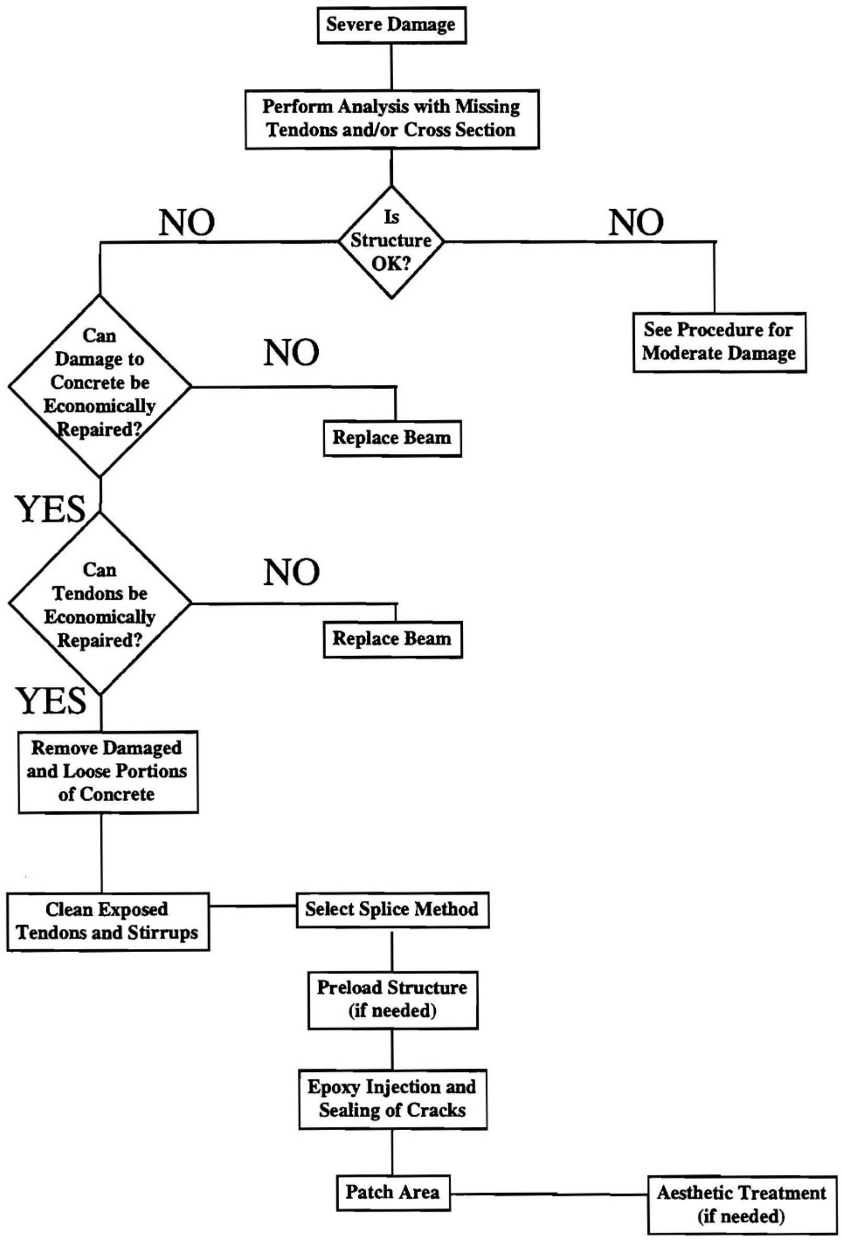


Figure 6: Severe damage step-by-step repair procedure, courtesy of Feldman et al., 1996

3.1.2 Alberta Infrastructure and Transportation, 2005

In this work, the damage classification is very close to Feldman et al., 1996 with further division of the minor class. Impact damage was classified as surface, minor, moderate, or severe. The assessment of the extent of damage to a girder is described as mentioned below.

3.1.2.1 Surface Damage

Surface scrapes and small nicks less than 0.25 in. deep. This type of damage does not warrant repairs unless it is associated with other bridge maintenance repairs.

3.1.2.2 Minor Damage

Isolated concrete cracks, nicks, and spalls up to 1.2 in. deep with no reinforcing or prestressing strands exposed. Minor damage adversely affects the aesthetics; however, the structural capacity is not typically reduced in its initial state. It is important to restore concrete cover to prevent reinforcing steel from eventually becoming exposed and corroded that may occur over time.

3.1.2.3 Moderate Damage

Concrete cracks and wide spalls exposing reinforcing steel and prestressing strand. There is no immediate effect on structural capacity. Although cracks and exposed reinforcement can reduce structure life due to corrosion and freeze-thaw action.

3.1.2.4 Severe Damage

Exposed and damaged prestressing strands and reinforcing steel along with loss of significant cross-section and possible lateral misalignment due to girder distortion.

3.1.3 Harries et al., 2009

This work was based on the NCHRP Report 226 (Shanafelt and Horn, 1980), which established three damage classifications minor, moderate, severe. Since minor and moderate damage does not require structural repairs. The emphasis was placed on severe class with further division into three sub-categories.

3.1.3.1 Minor Damage

Concrete with shallow spalls, nicks and cracks, scrapes and some efflorescence, corrosion or water stains. Damage at this level does not affect member capacity. Repairs are for aesthetic or preventative purposes.

3.1.3.2 Moderate Damage

Moderate damage does not affect member capacity. This classification includes larger cracks and sufficient spalling or loss of concrete to expose strands. Repairs are intended to prevent further deterioration.

3.1.3.3 Severe I Damage

Sever damage class, in general, is any damage requiring structural repairs. Typical damage at this level includes significant cracking and spalling, corrosion and damaged strands. Severe I damage requires a structural repair that can be done using a non-prestressed or a non-post-tensioned method. This may be considered as repair to restore Strength (or ultimate) limit state (ULS) requirements. Table 1 shows the classification limits of the three severe sub-categories. Figure 7 shows an example of a severe I damage class.

Table 1: Damage classification, courtesy of Harries et al., 2009

Damage Class	Severe I	Severe II	Severe III
Repair philosophy	ULS only	ULS and SLS	-
Action	the repair	PT repair	replace
Live load capacity replacement	up to 5%	up to 30%	100%
Ultimate load capacity replacement	up to 8%	up to 15%	100%
Replace lost strands	2-3 strands	up to 8 strands	>8 strands
Vertical deflection	loss of camber	up to 0.5%	>0.5%
Lateral deflection	within construction tolerance		permanent lateral deflection exceeding construction tolerance

Figure 7: AASHTO I-girder; reportedly repaired, courtesy of Harries et al., 2012

3.1.3.4 Severe II Damage

Damage requires structural repair involving the replacement of prestressing force through new prestress or post-tensioning. This may be considered as repair to affect the Service limit state (SLS) in addition to the ultimate limit state (ULS). Figure 8 shows an example of a severe II damage class.

Figure 8: AASHTO I-girder; reportedly repaired, courtesy of Harries et al., 2012

3.1.3.5 Severe III Damage

When damage is too extensive and exceeds the severe II class limit shown in Table 1. Repair becomes not practical and the element must be replaced. Figure 9 shows an example of a severe III damage class.

Figure 9: Exterior girder FM479 over Kerr Road, San Antonio TX. AASHTO I-girders; reported that the entire bridge was demolished and replaced, courtesy of Harries et al., 2012

3.1.4 Harries et al., 2012

In this work, the same classification of damage as Harries et al., 2009 is presented with further division of severe damage class. Also, some slight changes can be noticed in the description of severe damage classes to include the strand loss as a percentage of the total number of strands as shown in Table 2.

Table 2: Damage classification, courtesy of Harries et al., 2012

Damage Class	Description	strand loss	camber
Minor	Concrete with shallow spalls, nicks and cracks, scrapes and some efflorescence, rust or water stains. Damage does not affect member capacity. Repairs are for aesthetic and preventative purposes only	no exposed strands	no effect of girder camber
Moderate	Larger cracks and sufficient spalling or loss of concrete to expose strands. Damage does not affect member capacity. Repairs are intended to prevent further deterioration	exposed strands no severed strands	no effect of girder camber
Severe I	Damage affects member capacity but may not be critical – being sufficiently minor or not located at a critical section along the span. Repairs to prevent further deterioration are warranted although structural repair is typically not required	less than 5% strand loss	partial loss of camber
Severe II	Damage requires structural repair that can be effected using a non-prestressed/post-tensioned method. This may be considered as repair to affect the STRENGTH (or ultimate) limit state	strand loss greater than 5%	complete loss of camber
Severe III	Decompression of the tensile soffit has resulted. Damage requires structural repair involving the replacement of prestressing force through new prestress or post-tensioning. This may be considered as repair to affect the SERVICE limit state in addition to the STRENGTH limit state	strand loss exceeding 20%. In longer and heavily loaded sections, decompression may not occur until close to 30% strand loss	vertical deflection less than 0.5%
SEVERE IV	Damage is too extensive. Repair is not practical, and the element must be replaced	strand loss greater than 35%	vertical deflection greater than 0.5%

3.1.5 Iowa DOT, 2014

Iowa state department of transportation published an emergency response manual for over height collisions to bridges. The manual provides general guidance on the levels of damage as minor, moderate, and severe levels.

3.1.5.1 Minor Damage

When no repair or minimal repair is required and includes the following cases:

- Minor concrete spalling of the bottom flange as shown in Figure 10
- Mild reinforcing steel or prestressing strand may be partially exposed due to loss of cover concrete only; mild reinforcing steel or prestressing strands are not damaged and remain embedded in concrete
- Concrete cracks are difficult to see from the ground and do not reflect from one side of the girder to the other

Figure 10: Minor bottom flange spalling, courtesy of Iowa DOT, 2014

3.1.5.2 Moderate Damage

When repair works are required and include the following cases:

- Moderate concrete spalling is typically limited to the bottom flange and includes exposed stirrups and strands as shown in Figure 11
- Through cracking of bottom flange and/or lower half of web

- A horizontal crack at the junction of the web and the top flange of a prestressed concrete girder narrower than 1/16 inch
- Up to 2 of the bottom flange strands are severed or partially severed

Figure 11: Bottom flange damage with mild reinforcing and prestressing strands intact, courtesy of Iowa DOT, 2014

3.1.5.3 Severe Damage

When girder replacement is the optimum decision to make and includes the following cases:

- Severe concrete spalling including exposed stirrups and strands as shown in Figure 12
- Through cracking of bottom flange extending into the upper half of the web
- Horizontal cracks at the junction of the web and the top flange of a prestressed concrete girder wider than 1/16 inch
- Excessive loss of concrete section
- More than 2 of the bottom flange strands are severed or partially severed

Figure 12: Heavy damage with web cracking and severed mild reinforcing and prestressing strands, courtesy of Iowa DOT, 2014

3.1.6 Tabatabai, 2019

A research project report sponsored by the Wisconsin Department of Transportation was published is 2019 discussing the assessment and repair of damaged prestressed bridge girders. Covered damage causes were damage due to accidental impact by over-height vehicles on the bottom of the girder, or damage to the top flange of the girder during deck removal operations. Table 3 shows the proposed classification for bottom flange damage related to a vehicular collision.

Table 3: Bottom flange damage classification, courtesy of Tabatabai, 2019

Damage Category	Description
Minor	Concrete nicks, gouges, scrapes, and cracks that are less than 0.006 in wide, without any exposed or partially exposed strands.
Moderate	Cracking and spalling of concrete that exposes at least one strand, but no severed strands.
Significant	Cracking and spalling of concrete and less than 15% of all strands severed in the area of maximum damage.
Serious	Cracking and spalling of concrete; severed strands are more than 15% and less than 25% of all strands.
Severe	Cracking and spalling of concrete; severed strands are more than 25% of all strands.

3.1.7 Proposed NDOT Classification of Vehicular Collision Damage

This section will present the proposed classification of vehicular collision damage to be followed by NDOT. Table 4 presents the proposed damage classes with descriptions, examples, and proposed repair/replacement methods for each class.

Table 4: Proposed NDOT vehicular collision damage classification

Damage Class	Description	Reference	Examples and Figures	Effect on Structural Capacity	Proposed Repair/Replacement Method
Minor	Concrete cracks, chips, and spalls up to 1.2 in. deep with no exposed reinforcing steel or prestressing strands. Concrete cracks are not observed from both sides of the girder.	Feldman et al., 1996. Alberta Infrastructure and Transportation, 2005. Iowa DOT, 2014.	Figure 1 Figure 10 Figure 13	No immediate effect on the structural capacity	Removal of loose materials, patching, and/or epoxy injection based on aesthetic needs
Moderate	Concrete cracks and wide spalls exposing reinforcing steel or prestressing strands but bars and strands remain undamaged.	Feldman et al., 1996. Alberta Infrastructure and Transportation, 2005. Iowa DOT, 2014.	Figure 3 Figure 11 Figure 14	No immediate effect on the structural capacity	Removal of loose materials, strand cleaning, patching and/or epoxy injection based on corrosion potential and aesthetic needs
Severe I	Any of the following: 1 or 2 strands damaged, or less than 5% of the total number of strands Loss of vertical camber but no downward deflection	Harries et al., 2009 Harries et al., 2012	Figure 15 Figure 16 Figure 17	Loss in live load capacity up to 5%. Loss in ultimate load capacity of up to 8%.	FRP wrapping, steel jacket, or strand splicing to satisfy strength limit state, combined with patching and/or epoxy injection.
Severe II	Any of the following: 3 to 8 strands damaged, or greater than 5% and less than 20% of the total number of strands Vertical downward deflection but less than 0.3% of girder length	Harries et al., 2009 Harries et al., 2012	Figure 18 Figure 19 Figure 20	Loss in live load capacity of up to 30%. Loss in ultimate load capacity of up to 15%.	Strand splicing or external post-tensioning to satisfy service limit state in addition to strength limit state, combined with patching and/or epoxy injection.
Severe III	Any of the following: More than 8 strands damaged, or more than 20% of the total number of strands Vertical downward deflection exceeding 0.3% of girder length Lateral deformation exceeding construction tolerance Damage extending beyond bottom flange and lower half of web	Harries et al., 2009 Harries et al., 2012 Iowa DOT, 2014	Figure 21 Figure 22 Figure 23	Loss in live load capacity up to 100% Loss in ultimate load capacity up to 100%	Girder replacement

Figure 13: Minor damage, NDOT York Bridge East, courtesy of NDOT

Figure 14: Moderate damage, exposed intact strands, courtesy of Pantelides et al., 2010

Figure 15: Severe I damage, one severed strand, NDOT Wood River Interchange Bridge Girder (A), courtesy of NDOT

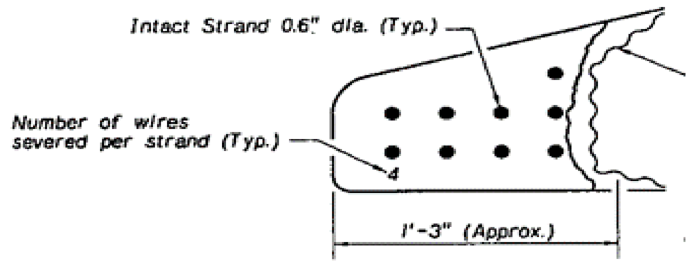


Figure 16: Severe I damage, one severed strand and loss of vertical camber, camber of +1 in. at undamaged girders and +0.06 in. at Girder (A), NDOT Wood River Interchange Bridge Girder (A), courtesy of NDOT

Figure 17: Severe I damage, one severed strand, courtesy of Harries et al., 2012

Figure 18: Severe II damage, five severed strands, NDOT Wood River Interchange Bridge Girder (E), courtesy of NDOT

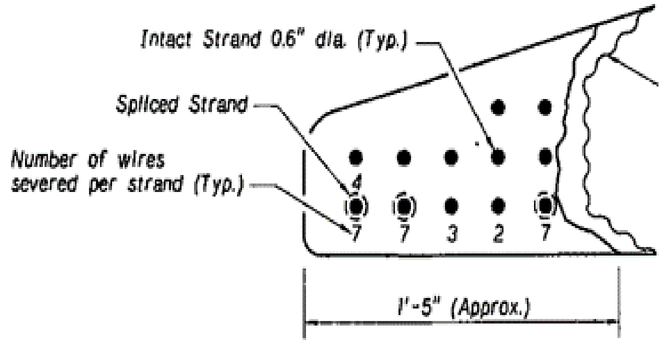


Figure 19: Severe II damage, five severed strands and vertical downward deflection, camber of +1 in. at undamaged girders and -0.48 in. at Girder (E), NDOT Wood River Interchange Bridge Girder (E), courtesy of NDOT

Figure 20: Severe II damage, three severed strands, NDOT Scottsbluff Gering Bypass Bridge, courtesy of NDOT

Figure 21: Severe III damage, several damaged strands, NDOT Schuyler Bridge, courtesy of NDOT

Figure 22: Severe III damage, several damaged strands, courtesy of Iowa DOT, 2014

Figure 23: Severe III damage, several damaged strands, courtesy of Harries et al., 2012

3.2 Repair Methods

This section will present viable repair methods for the previously defined damage classes. A suggested procedure is presented for each individual repair method. At the end of this section, the overall proposed repair procedure for each damage class is presented.

3.2.1 Epoxy Injection

This method can be used for the sealing of cracks 0.007 in. wide and narrower. Cracks can be so small and narrow as to be noticeable only after soaking with water. This method is discussed in more detail in the PCI manual for the evaluation and repair of precast/prestressed concrete bridge products, and FHWA-NHI-14-050 bridge maintenance reference manual. Before starting to use this repair method the feasibility of epoxy injection of cracks should be investigated. The investigation should include an estimation if there are so many cracks that the structural integrity of the element is too compromised for this type of repair. Also, the size and depth of the cracks should be determined, and if the cracks are active.

In I-680 over US-75 Bridge (discussed in Appendix A - Previous NDOT Repair Cases for Over-height Vehicular Collision) patching and epoxy injection were used for moderately damaged girders with no exposed strands. Figure 24 shows the epoxy injection process done in I-680 over the US-75 Bridge. For 168th St. over West Dodge Road Bridge, Scottsbluff Gering Bypass Bridge, and Wood River Bridge (discussed in Appendix A - Previous NDOT Repair Cases for Over-height Vehicular Collision) patching and epoxy injection were combined and used with strand splicing to seal cracks greater than 0.01 in. Figure 25 shows an example of epoxy injection ports placed at a maximum of 6-inch spacing according to Alberta Infrastructure and Transportation.

Figure 24: Epoxy injection process done in I-680 over US-75 Bridge, courtesy of NDOT

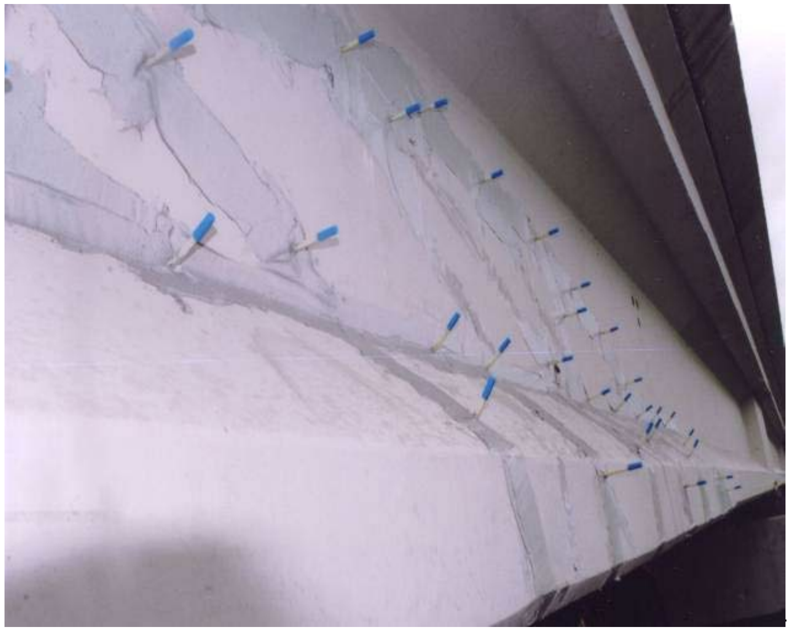


Figure 25: Epoxy injection ports, courtesy of Alberta Infrastructure and Transportation, 2005

Suggested Procedure

FHWA-NHI-14-050 bridge maintenance reference manual provides a detailed procedure for epoxy injection as follows:

1. Surface Preparation: Before injection, the interior of the crack should be cleared of all dust, dirt, oil, grease, or fine particles of concrete that could prevent epoxy penetration and bonding. Harsh chemicals or detergents should not be used to clean the cracks because they may compromise the ability of the epoxy to bond to the concrete.

2. Sealing the crack surfaces: The exterior of the cracks should be sealed and allowed to harden to prevent the injected epoxy from leaking out of the crack. Cracks can be sealed by applying epoxy, polyester, or other appropriate sealing material to the surface of the crack. For cracks that extend through the entire member section, the opposite side of the injection should be sealed as well. If the cracks on each side do not connect, the epoxy injection should be performed on each side individually. If extremely high injection pressures are needed, the crack can be cut out to a depth of 1/2 inch and width of 3/4 inch in a V-shape, filled with epoxy, and struck off flush with the surface.

3. Installing the entry and venting ports: Two general methods can be used to install the entry and venting ports; surface mounted and socket mounted. Entry ports are typically tube devices that allow the pressurized epoxy resin to be pumped into the crack. The entry port spacing is typically at 8 inches on center but can be increased for wider cracks. Port spacing depends on the crack width and the amount of pressure applied, however, the spacing should be limited to the thickness of the repaired member if the cracks pass all the way through. Surface-mounted entry ports are normally adequate for most cracks, but socket-mounted ports are used when cracks are blocked.

In some cases, it may be necessary to drill holes approximately 3/4 inch in diameter and 1/2 to 1 inch below the surface of the crack to place the entry or exhaust port.

4. Injecting the material: The injection progresses from port to port, normally starting at the lowest point, and continuing until the epoxy is extruded from the next port. The distance between the ports should not exceed the expected penetration depth. A handgun or pressure pot can be used, but various types of machines are available that assure the proper proportioning, mixing, and temperature of the two-part epoxy and the proper injection pressure. This technique may also be used to fill isolated voids or delamination in concrete. In this case, injection pressure must not be too high.

Selection of Materials

Crack sealing repair works were done on AASHTO Type II prestressed girders with severe cracks at girder ends with high strength epoxy resin, Choo et al., 2013. The resin had a 28-day compressive strength of 13 ksi at 73 ^{0}F, a tensile strength of 8.9 ksi, modulus of elasticity in compression of 320 ksi at seven days, and modulus of elasticity in tension of 420 ksi at 14 days.

Epoxy injection repairs were also done by NDOT in I-680 over US-75 Bridge (discussed in Appendix A - Previous NDOT Repair Cases for Over-height Vehicular Collision). The epoxy resin used had a compressive strength of 12 ksi, tensile strength of 7.12 ksi, and modulus of elasticity in compression of 265 ksi.

3.2.2 Patching

This method involves patching any spalls or concrete section loss to return the girder to its original cross-section as shown in Figure 27 and Figure 28. This method is also discussed in more detail in the PCI manual, 2006 for the evaluation and repair of precast/prestressed concrete bridge products, and FHWA-NHI-14-050, 2015 bridge maintenance reference manual. Often, following spall repair, an FRP wrap would be placed around the bottom flange of the girder in the damaged area and up the sides of the girder web. The FRP wrap can provide added shear strength to the girder and serves to confine and contain the spall repairs to prevent them from separating from the girder and falling onto traffic below. Concrete patching repair methods, in general, include a variety of materials and application methods including drypack, mortar patch, concrete replacement, synthetic patching, and the use of prepackaged patching compounds. Surface preparation according to the PCI manual should be sound, clean, dry, free of curing compounds, laitance, oil, dust, and moisture.

Concrete spalls between ½ and 2 inches deep can be repaired by patching without the use of formwork. Patching or epoxy injection can be used as a stand-alone repair method for minor and moderate damage classes discussed previously. Practice usually combine patching and epoxy injection to seal any narrow cracks before patching material is applied.

Patching can be used prior to most other repair methods to restore section loss. In Scottsbluff Gering Bypass bridge, and Wood River Bridge (Appendix A - Previous NDOT Repair Cases for

Over-height Vehicular Collision) patching and epoxy injection were used after strand splicing to restore concrete section capacity as shown in Figure 26.

Pre-loading is usually done by loading the bridge over the damaged girder prior to placing the patching material to apply compressive forces on the patch after it is cured and live load is removed. For Scottsbluff Gering Bypass bridge, and Wood River Bridge (Appendix A - Previous NDOT Repair Cases for Over-height Vehicular Collision) a 40 kip truck was used for pre-loading.

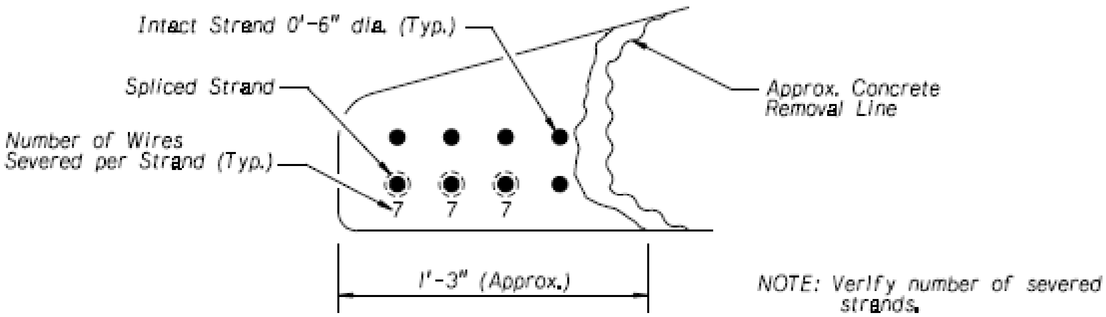


Figure 26: Patching performed after strand splicing to restore the concrete section, Scottsbluff Gering Bypass Bridge, courtesy of NDOT

Figure 27: Patching formwork around girders, courtesy of Alberta Infrastructure and Transportation, 2005

Figure 28: Pumping concrete into plywood formed section, courtesy of Alberta Infrastructure and Transportation, 2005

Structural Calculations

A general procedure for preloading is to estimate the required applied external moment (M_{ext}) on the girder. The external moment is applied so that when it is removed the patch is subjected to compression stress equal to ($M_{ext.}/S$). The external moment must not cause any tensile stresses greater than the allowable design tensile stresses (f_t). The maximum external moment that could be applied is given by the following equation:

$$f_t = -\frac{P}{A} - \frac{P \cdot e}{S_{b.nc}} + \frac{M_d}{S_{d.nc}} + \frac{M_{ext.max}}{S_{d.c}}$$

Where (P) is the effective force in prestressing reinforcement (after all losses); (e) is the eccentricity of prestressing steel with respect to the centroidal axis of the member; ($S_{b.nc}$) is the bottom section modulus of the undamaged non-composite girder section; ($S_{d.nc}$) is the bottom section modulus of the damaged non-composite girder section; ($S_{d.nc}$) is the bottom section modulus of the damaged composite girder section.

For simple span girders, the equivalent external concentrated load is calculated according to the following equation:

$$P_{ext} = \frac{M_{ext.max} \cdot L}{(x) \cdot (L - x)}$$

Where (P_{ext}) is the maximum equivalent external concentrated load that can be applied on the damaged girder before patching; (L) is the simple span length of the girder; (x) is the length between the concentrated load and the closest girder support. For continuously spanned girders, the external load is obtained by structural analysis of the statically indeterminate system.

Corrosion protection

There are various techniques to address corrosion protection of exposed strands or reinforcing steel depending on the existing degree of corrosion and environmental exposure. According to previous NDOT practices existing corrosion products were removed before patching by lightly sandblasting the exposed reinforcement. Also, catholic protection can be achieved by installing anode devices to prevent future corrosion of the exposed reinforcement. In addition, corrosion inhibitors can be applied to the exposed reinforcement surface to prevent future corrosion.

Selection of Materials

FHWA/TX-96/1370-1, 1996, provides the material selection factors to be considered for impact damaged bridge girders. The patching materials must be compatible with the base concrete in a given member; otherwise, premature failure of the patch or the surrounding concrete could occur. Factors to be considered include freeze-thaw cycles, exposure to deicing salt, extreme temperatures, rapid temperature changes, and dynamic and static loading. All cementitious patching materials shrink as they dry. Preferably, most of this shrinkage occurs soon after casting, while the patch is still plastic and has not fully bonded to the concrete. If the selected patch material shrinks excessively after it has hardened and bonded to the base concrete, significant stresses will occur at the interface due to this shrinkage, and the repair material may crack or de-bond from the base concrete. Shrinkage can be minimized by using a low water-to-cement ratio, and by extending the patch material with coarse aggregate. Low-shrinkage or expansive grouts may be useful in some cases. Also, permeability is an important factor to be considered, repair durability will be improved if the patch material has a lower permeability than the base concrete.

The compressive strength of the patch material should be equal or greater than the base concrete, since the repaired member is likely to be subjected to the same loading conditions that existed prior to being damaged. The elastic modulus of the patch material should be as close as possible to that of the original concrete. A patching material with a modulus higher than that of the base concrete will tend to carry a greater portion of the load, while a patch material with a lower modulus will not carry as much of the load as the base concrete.

Rapid strength gain allows for the return of the structure to service as quickly as possible. The most common way to achieve rapid strength gain is through the use of high early strength concrete. However, the faster a patch material sets, the more linear shrinkage will occur once the patch has hardened. In addition, it should be considered that high early strength concretes have a lower later age strength than do normal concretes.

According to Tabatabai et al., 2004 (Wisconsin Highway Research Project Report No. 0092-01-06) commonly used classes of patch materials include portland-cement concrete (e.g. Type III cement concrete), hydraulic cement concrete (e.g. Pyrament 505), and polymer-based (e.g. epoxy) patches. Portland-cement patches are the most commonly used, and construction workers are typically familiar with the installation techniques. Hydraulic (fast-setting) cement concrete materials are similar to regular concrete. They are generally self-leveling, do not require mechanical vibration, and are more stable at higher temperatures than cementitious materials. Polyurethanes and epoxies are relatively new patch materials. Proportioning and mixing are critical to material performance. Also, because of their relatively low viscosity, they are more difficult to place on vertical surfaces.

Patch repairs were done by NDOT in 168th St. over West Dodge Road Bridge (discussed in Appendix A - Previous NDOT Repair Cases for Over-height Vehicular Collision). The damaged area was filled with concrete having a minimum 24-hours compressive strength of 1,500 psi, 3,000 psi at 3 days, and 5,000 psi at 28-days. Concrete also had a minimum bond strength of 2,000 psi according to ASTM C882.

Suggested Procedure

According to FHWA-NHI-14-050 bridge maintenance reference manual, 2015, there are four different procedures for patching:

<u>1- Patching with trowel-applied or poured mortar</u>

Used for shallow spalls between ½ and 2 inches deep spalls that do not require forming as follows:

- Remove any loose concrete in the area to be patched.
- Sawcut the perimeter of the removed concrete region in straight lines to a depth of ¾ inch to make a clean patch line. If possible, bevel the sawcut at 45 degrees inward to lock the patch in place.
- Clean the surface of the existing concrete using hand tools, sandblasting and compressed air.
- Thoroughly wet the concrete surface and allow it to dry on the surface.
- Mix the patching mortar in accordance with the manufacturer's recommended specification for vertical or inverted patches.
- Apply a bonding agent to the existing surface if required. Do not let the bonding agent dry before the patch material is placed. (Follow the manufacturer instructions on bonding agent).
- Use a trowel to firmly apply the patch material into the void created by the spall.
- Use trowels to sculpt the member shape and texture the finish.
- Spray apply a curing compound or wet cure for 7 days over the patched area, unless latex-modified concrete patch material is used.

<u>2- Recasting with new concrete</u>

Used for larger spalls that require forming, the suggested procedure is as follows:

- Before starting to sawcut concrete, determine if there are any structural capacity concerns from removing unsound concrete to the depths and limits necessary for the repair. Place any required temporary shoring or bracing necessary to support the structure during the repair.
- Sawcut the perimeter edges straight to a depth of ¾ inch. Remove any loose concrete in the area to be patched. Concrete should be removed 1 inch all around exposed rebar whenever possible.
- The existing surface should be cleaned by light sandblasting. The concrete surface should be saturated with water spray, if dry, and then allowed to return to a surface dry condition. This will prevent the old concrete surface from absorbing the new concrete mixing water.
- Install formwork. The formwork should be rigid enough to prevent new concrete from sagging away from the existing concrete under the weight of new concrete. The formwork should withstand forces from concrete pumping and the vibrating used to consolidate the concrete. Plywood is often used for concrete formwork. Steel forms can be used but they are heavy and not easily handled. Forms are typically attached to the member being repaired or hung from the deck and should be well constructed to prevent leakage of the patch material.
- Prior to placing the concrete, the forms should be cleaned, sprayed with a form release agent and wetted to prevent absorption of the water used in the concrete.
- Apply a bonding agent (usually a cement grout) onto the concrete surface just before the installation of formwork. It is very important that the bonding grout does not dry out before the repair concrete can be placed. A dry bonding grout can destroy the bond of the new concrete to the existing concrete. For this reason, many owners do not allow bonding

agents. The use of specially formulated polymer bonding agents may be required if the formwork cannot be placed before the grout will dry.

- Place the new concrete through holes in the top of the formwork for vertical patches if the top is not accessible. Inverted patches should be cast from above if possible through fill holes in the member. If inverted patches cannot be recast from above, consider using the shotcrete repair method. Concrete for recasting should easily flow and fill all the voids in the form. Typically, 3/8 inch coarse aggregates are used in the mix to improve flow and consolidation. Limit the water to cement ratio to avoid shrinkage cracking of the repair. Concrete additives may be used to provide workability without resorting to adding additional water.
- Internally vibrate the newly placed concrete through the fill holes in the forms or by vibrating the forms from the outside. Vibration should be done along the length of the repair after shallow lifts of concrete have been placed. Good compaction is achieved by placing the concrete in small amounts and vibrating effectively as the work proceeds. An option to vibration is to use self-consolidating concrete which does not need any vibration.
- Allow the concrete to cure.
- Remove the formwork and grind off any excess concrete or fill any voids that formed.

<u>3- Prepacking dry aggregates and grouting</u>

Similar to recasting in surface preparation and formwork installation. The only difference is that a uniform size dry aggregate is packed in the space behind the form so that it fills the space completely. Grout is then pumped from the lowest to the highest point to fill the space between the aggregate. The advantage of prepacking dry aggregate and grouting is that the overall shrinkage of the repair is greatly reduced.

<u>4- Shotcrete</u>

Generally, achieving shotcrete compressive strength greater than or equal to the girder compressive strength (typically 8 ksi) is challenging. Shotcrete is desirable on vertical and overhead patches because no forming is required, and the pneumatically applied mortar can repair large surface areas in relatively short periods of time. It contains the same cement, aggregate, and water as concrete except that there are no coarse aggregates in the mix. Compaction is achieved by the velocity of the mixture when applied. Shotcrete repairs require a highly trained operator to obtain long-lasting results. The mix has high cement content and a low water/cement ratio. The addition of silica fume, fly ash and/or slag can enhance the performance of shotcrete. Steel or synthetic fibers have also been used to increase tensile strength and decrease the potential for cracking. When properly applied, the mortar is dense, durable, and has superior bonding characteristics.

- Prepare the existing surface. The edges of the repair area should be sawcut at least ¾ inch deep at a 45-degree angle into the repair area to prevent the rebound of the shotcrete material. All deteriorated concrete should be removed to a minimum of 1 inch behind exposed reinforcement. All surfaces should be cleaned with high-pressure water or by sandblasting.

- For repairs 3 inches or deeper, welded wire fabric or wire mesh should be mechanically affixed to the existing concrete surface prior to the placement of the shotcrete. The wire mesh will help ensure the integrity of the repaired area and limit cracking.
- Wet the existing surface so it does not absorb water from the pneumatic mortar.
- Apply shotcrete. A thin bond coat should be applied first with subsequent layers building up the desired thickness. When applying, place the shotcrete nozzle at 90-degree angles to the repair surface whenever possible. Corners should be applied at a 45-degree angle to prevent rebound. Maintain a uniform flow of material and limit the layer thickness to prevent sags or sloughing to occur. The natural handgun finish is preferred from bond and durability standpoints. Scraping or cutting may be used to remove high points and material that has exceeded the limits of the repair after the mortar has become still enough to withstand the pull of the cutting device. Troweling or another surface finishing is discouraged as it has a tendency to disturb the bond.
- Curing is very important for the rich mixes and thin sections used with pneumatic mortar. Seven days of water curing is generally advisable to promote good hydration of the cement, keep the mortar cool in hot weather, and prevent early shrinkage that may disturb the bond.

3.2.3 Strand Splicing

Generally, the most severe damage to prestressed concrete girder bridges involves damaging mild shear reinforcing steel and cutting any prestressing strands as discussed earlier. If girder damage involves the severing of two or more prestressing strands, a possible temporary repair alternative may involve patching of the girder spalls, epoxy grouting girder cracks, and either splicing the severed prestressing strands or supplementing the damaged strands with external prestressing. However, according to Iowa Dot, 2014, these types of repairs should only be considered temporary until the girder or the entire bridge can be replaced. Figure 29 shows an example of strand splicing.

Strand splicing was done in Nebraska in Scottsbluff Gering Bypass bridge where three strands were spliced as shown in Figure 30, and Wood River Bridge (Appendix A - Previous NDOT Repair Cases for Over-height Vehicular Collision) where three strands were also spliced as shown in Figure 31. For these two cases, the strand splice system was GRABB- IT Cable Splice, a product of Prestress Supply Inc, Florida. Strands were spliced and tightened with an approved and calibrated torque wrench to a tension force of 31,000 lbs in each ½" diameter 270LL strand. That force represents 75% of the strand capacity in tension f_{pu}. Prior to the actual installation of the splice system, a mock-up installation was performed with a calibrated torque wrench by a 3-person work crew to test and demonstrate that the system can be installed to the satisfactory of the Engineer. Splices were staggered to provide adequate bonding space for patch material around and between the splice components Anode devices were installed to prevent future corrosion of the exposed reinforcement.

Figure 29: Strand splicing, courtesy of Jones, 2017

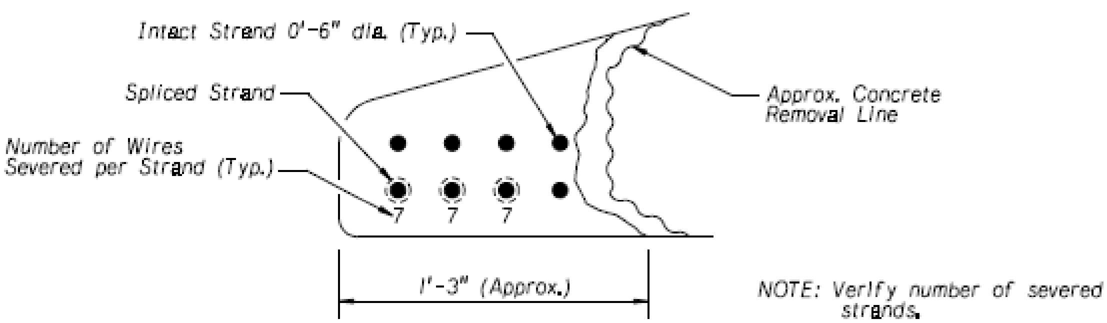


Figure 30: Spliced strands, Scottsbluff Gering Bypass Bridge, courtesy of NDOT

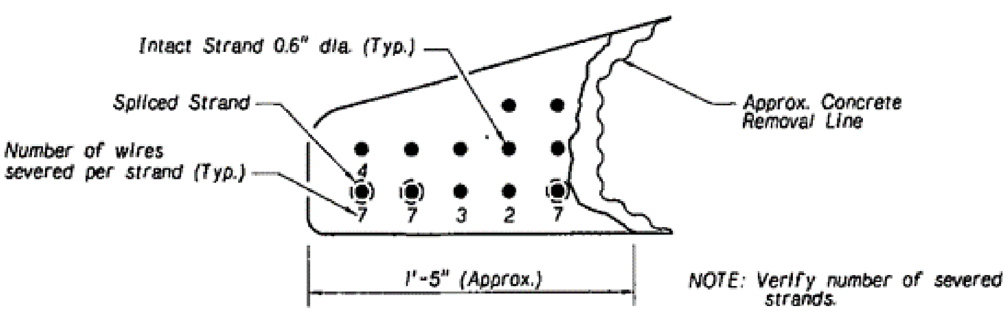


Figure 31: Spliced strands, Wood River Interchange Bridge, courtesy of NDOT

Some concerns were raised regarding fatigue problems related to strand splicing methods due to the presence of a sudden change in the strand section causing a concentration of stresses. Olson et al. (1992) reported a strand splice-repaired test girder that was tested in fatigue failed in tension at less than 82% of the original girder capacity. Possible reasons cited for that failure were attributed to fatigue.

Suggested Procedure

FHWA-NHI-14-050 bridge maintenance reference manual provides a general procedure for strand splicing as follows:

1. Before starting to sawcut concrete, determine if there are any structural capacity concerns from removing unsound concrete to the depths and limits necessary for the repair. Place any required temporary shoring or bracing necessary to support the structure during the repair.

2. Sawcut the perimeter edges straight to a depth of 3/4 inch. Remove any loose concrete in the area to be repaired. Concrete should be removed 1 inch all around exposed rebar whenever possible. The minimum length of concrete removal necessary in order to install all the strand splicing and tensioning devices is approximately six feet.

3. Saw cut the broken strand to remove any frayed or damaged length. Leave at least 4 inches of strand exposed to install the splice devices.

4. Install splice hardware consisting of a coupler, stressing gauge and tensioning device. The arrangement for these devices may be changed if it is more convenient.

5. Torque the splice hardware to tension the strand.

6. Preload the member according to structural calculations discussed earlier. Preloading adjacent girders in consciously spanned bridges could be beneficial to recover the camber.

7. Replace the concrete using any of the different patching procedures discussed earlier.

3.2.4 External Post-tensioning

External post-tensioning end blocks (typically referred to as ‘bolsters’) are added to the girder to allow for anchoring additional prestressing strands to restore the loss in compression force in the girder. The strands are then tensioned by jacking against the bolster. External post-tensioning could be done using prestressing steel anchors (example: Dywidag bars) as shown in Figure 32, or carbon fiber reinforced polymers CFRP strips, as shown in Figure 33. Harries et al. (2009) provide a general procedure for the structural calculations required for external post-tensioning.

Figure 32: External post-tensioning end block, courtesy of Alberta Infrastructure and Transportation, 2005

Figure 33: Sika carbostress system external CFRP post-tensioning, courtesy of Kasan, 2009

Among the disadvantages of this method is that it required environmental protection against corrosion. The external system could also be subjected to successive impacts or vandalism. Also, the applicability of this method is limited by the residual capacity of the un-strengthened girder which must safely resist any expected nominal load. According to Iowa DOT., 2014, splicing of damaged prestressing strands or the addition of external post-tensioning should be considered a temporary repair. Iowa DOT expects that a damaged girder shall be restored to its original condition, even if it requires partial deck removal and replacement of the damaged girder.

Structural calculations

A design example is presented in (Appendix E – Structural Calculations Design examples) according to Harries et al., 2009. The design example calculated the required external post-tensioning force at a given eccentricity. The goal of external post-tensioning is to restore the compressive stress at the bottom of the girder as intended by the original prestressed strands as well as increase the flexural capacity. Analysis of the section after strand loss should be done by sectional analysis.

3.2.5 FRP Wrapping

Carbon fiber and FRP wraps are commonly used to help contain damage to prestressed concrete girders and to restore structural integrity to the damaged girder. An example of an in-place FRP-wrapped repair for a prestressed concrete girder is shown in Figure 34. As mentioned in the patching section, FRP wrapping is recommended to provide confinement and add shear capacity to the patched section.

The ACI 440.2R-17 provides structural calculations for externally bonded FRP systems for the strengthening of concrete structures. A design example is presented in (Appendix E – Structural Calculations Design examples) for restoring the ultimate limit strength of a girder suffering from Severe I damage. When the FRP wrap is required to increase flexural resistance only of a section, FRP plies can be glued to the soffit of the girder. Strain in FRP reinforcement will be limited to the de-bonding limit, which can reduce the utilization of the FRP material. Different anchorage systems are provided in the ACI 440.2R-17. Anchoring the FRP layers can increase the effective strain up to its tensile rupture, which can significantly increase the strengthening effect of the FRP system.

Figure 34: FRP Wrapped Repair, courtesy of Iowa Dot, 2014

Selection of Materials

The ACI 440.2R-17 provides general guidelines for the selection of FRP systems and materials. One important criterion to consider is the environmental exposure. The mechanical properties (for example, tensile strength, ultimate tensile strain, and elastic modulus) of some FRP systems degrade under exposure to certain environments, such as alkalinity, saltwater, chemicals, ultraviolet light, high temperatures, high humidity, and freezing-and-thawing cycles.

The performance of the FRP system overtime in an alkaline or acidic environment depends on the matrix material and the reinforcing fiber. Dry, unsaturated bare or unprotected carbon fiber is resistant to both alkaline and acidic environments, while bare glass fiber can degrade over time in these environments. A properly applied resin matrix should isolate and protect the fiber from the alkaline/acidic environment and retard deterioration. The FRP system selected should include a resin matrix resistant to alkaline and acidic environments. Sites with high alkalinity and high moisture or relative humidity favor the selection of carbon-fiber systems over glass-fiber systems.

Preformed CFRP strips have been used successfully in several bridge repair applications. Kasan, 2009 used a commercial product (Sika CarboDur strips) as a system for the repair of a damaged girder. The system has a design tensile strength of 406 ksi, modulus of elasticity of 23,200 ksi, rupture strain ε_{fu} of 0.017, and thickness of about 0.05 in.

Elsafty, 2012 utilized CFRP laminates to restore the flexural capacity of damaged AASHTO Type II girders having ruptured strands and suffered concrete section loss. The system had a tensile strength of 121 ksi, modulus of elasticity of 11,900 ksi, rupture strain ε_{fu} of 0.0085, and thickness of about 0.04 in.

Harries et al., 2012 provides a guide to the available preformed CFRP strips from a variety of manufacturers in discrete sizes and a number of ‘grades’ of CFRP: high strength (HS), high modulus (HM) and ultra-high modulus (UHM). Properties of each of these are listed in Table 3.

Table 5: Representative properties of available preformed FRP materials

	HS-CFRP	HM-CFRP	UHM-CFRP	UHM-GFRP
Tensile modulus, E_f (ksi)	23200	30000	44000	6100
Tensile strength, f_{fu} (ksi)	406	420	210	130
Rupture strain, ε_{fu}	0.017	0.014	0.005	0.021
Typically available strip thickness, t_f (in.)	0.047	≈0.05	≈0.05	0.075
Typically available strip widths, b_{f1} (in.)	2, 3 and 4	4	4	2 and 4

Suggested Procedure

FHWA-NHI-14-050 bridge maintenance reference manual provides a general procedure for FRP application as follows:

- Remove any unsound concrete that is within the area the FRP is to be applied.
- Cracks wider than 0.010 inches wide should be filled with epoxy resin following the epoxy injection procedure in this section.
- Where fibers wrap around corners, the corners should be rounded to a minimum of 0.5 in. radius to reduce stress concentrations in the FRP system.
- The concrete surface should be thoroughly cleaned using abrasive hand tools or blast equipment. The surface should be blown clean with compressed air.
- The FRP should be cut to the size with heavy shears as specified by the engineer and in accordance with the manufacturer's recommendations.
- Prime the clean and dry concrete surface with epoxy resin using a trowel. The primed area should exceed the FRP size by approximately one-half inch on all sides. Allow the epoxy to become tacky.
- Prime the surface of the FRP to be placed on the concrete.
- Place the FRP strips on the primed concrete such that the epoxy primed sides stick together epoxy to epoxy.
- Use a rubber roller to press the FRP flat and smooth on the concrete.
- Allow the epoxy resin to fully cure.

3.2.6 Steel Sleeve or Jackets

According to the PCI manual, this method is often employed for exterior girders that may be subjected to repeated impacts by over height vehicles. This method involves repairing the spalled areas, then encase the bottom of the girder with a two-piece steel sleeve. The steel sleeve is anchored with concrete anchors into the girder, and then the space between the concrete girder and the inside of the steel sleeve would be injected with epoxy to bond the sleeve to the concrete girder. The steel sleeve serves to armor the bottom of the girder and thus provide a greater degree of protection from future vehicle strikes.

Figure 35: Grouted steel sleeve repair, courtesy of Iowa Dot, 2014

3.2.7 Sprayed GFRP

This method has only been done experimentally and was not performed on actual bridge girders. The technique consists of spraying discontinuous glass fibers onto the concrete surface concurrently with a vinyl ester resin as shown in Figure 36. Boyd et al, 2006 evaluated the potential for this repair method on impact damaged girders with a series of three AASHTO Type II girders, 43.6 feet in length. Specimens were tested in flexure with one control undamaged girder, one damaged unrepaired girder, and one damaged and repaired girder using that method as shown in Figure 37. Results showed the success of this method in reaching the target rehabilitation goal of 95% of the original undamaged girder strength specimen.

Figure 36: Sprayed GFRP technique, courtesy of Boyd et al, 2006

Figure 37: Repaired specimen, courtesy of Boyd et al, 2006

3.2.8 Near Surface Mounted C-FRP Bars

This method resembles using external C-FRP bars with some extra advantages in protecting the bars against any future impact and environmental exposure. Groves are cut into cover then partly filled with epoxy adhesive before inserting the bar and final injection is performed to fill the slot as shown in Figure 38. Same as external C-FRP bars, this technique allows the bars to be prestressed if serviceability is a concern. However, Prestressing applications are very difficult and has only been demonstrated in laboratory applications, Kasan, 2009. This method has been reported to be used in the strengthening of a concrete bridge deck as shown in Figure 39 but was not performed on bridge girders to date.

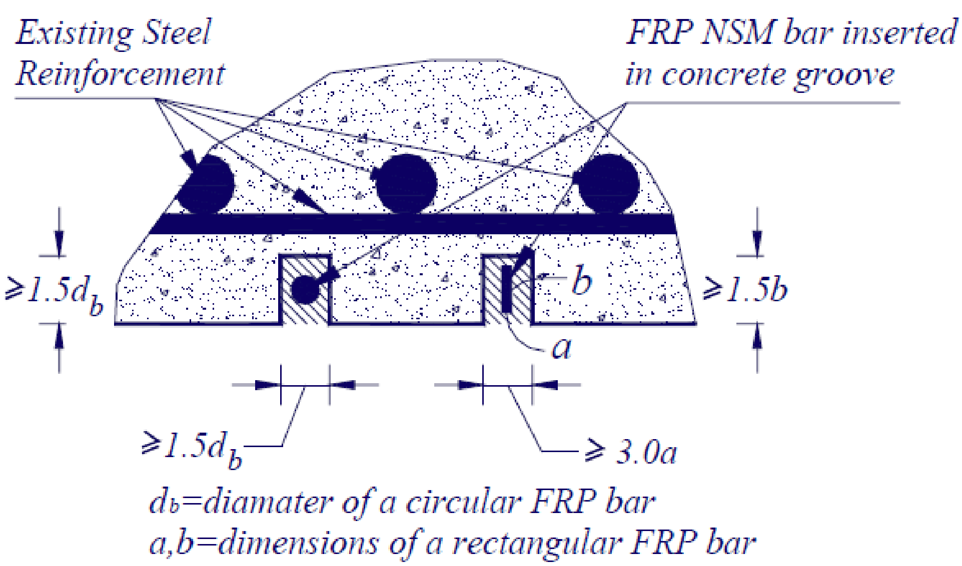


Figure 38: Near-surface mounting detailing, courtesy of Casadei, 2006

Figure 39: Use of near-surface mounted C-FRP in bridge decks, courtesy of Parretti et al 2004

3.2.9 Proposed NDOT Repair Methods for Over-height Vehicular Collision

The general procedure was adopted from FHWA/TX-96/1370-1 report, 1996 step-by-step flow charts. In addition, viable repair methods were included in the flow charts to provide some extra details. The detailed procedure for each repair method is presented in the discussion of each individual repair method.

A. For Minor damage class

1. Remove unsound concrete
2. Epoxy injection if required (according to suggested procedure discussed in section 3.2.1)
3. Prepare base concrete surface (surface should be cleaned, then pre-wetted, then apply bonding agent)
4. Place new concrete (following patching with trowel-applied or poured mortar suggested procedure discussed in section 3.2.2)
5. Address aesthetic treatment if required

B. For Moderate damage class

1. Remove unsound concrete
2. Epoxy injection if required (according to suggested procedure)
3. Clean exposed strands or bars (by sandblasting)
4. Preload girder, if required
5. Place new reinforcement as needed (making sure it is properly lapped, anchored, or mechanically attached to the existing steel)
6. Address future corrosion protection (if required)
7. Prepare base concrete surface (as in Minor damage)

8. Assemble the forms
9. Place new concrete (following patching with recasting with new concrete suggested procedure discussed in section 3.2.2)
10. Apply FRP wrapping (if required, according to suggested procedure discussed in section 3.2.5)
11. Address aesthetic treatment if required

C. For Severe I damage class

1. Restrict vehicle loads on the affected girder by directing traffic to the far side of the bridge until a structural review is performed
2. Follow same procedure as moderate damage
3. Apply FRP wrapping (if required, according to structural calculations and suggested procedure discussed in section 3.2.5)

D. For Severe II damage class

1. Restrict vehicle loads on the affected girder by directing traffic to the far side of the bridge until structural review is performed
2. Strand splicing or external post tensioning (according to suggested procedure discussed in section 3.2.3, or section 3.2.4)
3. Follow same procedure as moderate damage
4. Apply FRP wrapping (if required, according to structural calculations and suggested procedure discussed in section 3.2.5)

Chapter 4 - Damage at Girder Ends

Prestressing strands are more susceptible to corrosion than lower grades of steel (due both to the composition of prestressing steel and the increased surface area-to-cross section area ratio of a seven-wire strand), therefore prestressed concrete beams are susceptible to corrosion, especially at beam ends. Since prestressed strands are anchored in the beam ends, strand corrosion in this area can be detrimental to girder performance, (Harries et al., 2012).

4.1 Classification of Damage

This section presents a background about the previous literature used to develop the NDOT classification of girder end damage. Different state manuals and reports are presented with the focus on the damage classification.

4.1.1 Shanafelt, and Horn, 1985

This report was originally prepared for vehicular collision damage. However, it mentions that the presence of any exposed strands having corrosion damage leads to severe damage and girder replacement is recommended. In corrosive environments, minor nicks, spalls, and scrapes may deserve more attention than they usually get. Because of the effectiveness of concentrating strands near the bottom of prestressed girders, the concrete cover is usually the minimum permitted by specifications. Reducing this cover and scraping away the concrete surface finish may permit the intrusion of corrosive elements to the strands. Strong consideration should be given to cleaning these surfaces and sealing with the two coats of a penetrating sealer.

MINOR damage: is defined as concrete with shallow spalls, nicks and cracks, scrapes and some efflorescence, rust or water stains. Damage at this level does not affect member capacity. Repairs are for aesthetic or preventative purposes.

MODERATE damage: includes larger cracks and sufficient spalling or loss of concrete to expose strands. Moderate damage does not affect member capacity. Repairs are intended to prevent further deterioration.

SEVERE damage: is any damage requiring structural repairs. Typical damage at this level includes significant cracking and spalling, corrosion, and exposed and broken strands.

4.1.2 Naito et al., 2006

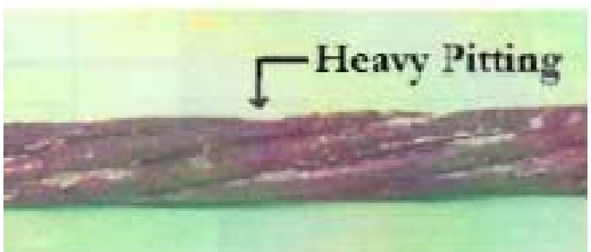
The continuum of corrosion damage of seven-wire prestressing strands is illustrated in Figure 40. In general, the progression of corrosion-related damage tends to be exponential in time. Repairing such types of damage must be accompanied my mitigating the source of the damage where possible.

A: Concrete spalling

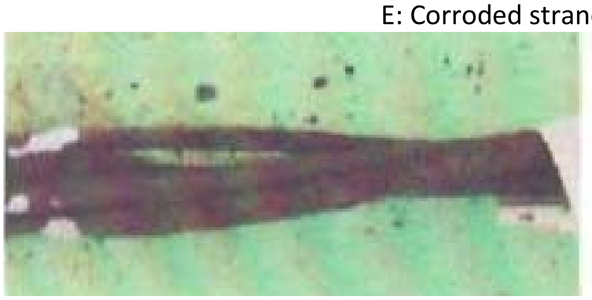
B: Exposed strands without corrosion

C: Corrosion without pitting

D: Corroded strand with light pitting



E: Corroded strand with heavy pitting



F: Partial loss of strand area

G: Complete loss of strand area

Figure 40: Continuum of corrosion damage, courtesy of Naito et al., 2006

4.1.3 AASHTO Bridge Element Inspection Manual, 2010

The two major rating guideline systems currently in use are the FHWA's Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO Guide Manual for Bridge Element Inspection for element level condition state assessment, FHWA NHI 12-049, 2012.

Table 6: Condition state definitions, AASHTO Bridge Element Inspection Manual, 2010

<table>
<tr><th></th><th>Condition State 1</th><th>Condition State 2</th><th>Condition State 3</th><th>Condition State 4</th></tr>
<tr><td>Spalls/ Delamination/ Patch Areas</td><td>None</td><td>Moderate spall or patch areas that are sound</td><td>Severe spall or patched area showing distress</td><td rowspan="6">The condition is beyond the limits established in condition state three (3) and/or warrants a structural review to determine the strength or serviceability of the element or bridge.</td></tr>
<tr><td>Exposed Rebar</td><td>None</td><td>None</td><td>Corrosion without section loss</td></tr>
<tr><td>Exposed Prestressing</td><td>None</td><td>None</td><td>Present with no section loss</td></tr>
<tr><td>Cracks</td><td>Hairline Cracks Only</td><td>Narrow size or density</td><td>Medium-size or density</td></tr>
<tr><td>Efflorescence</td><td>None</td><td>Moderate but without rust</td><td>Severe with rust staining</td></tr>
<tr><td>Load Capacity</td><td>No Reduction</td><td>No Reduction</td><td>No Reduction</td></tr>
<tr><td>Feasible Actions</td><td>Do Nothing
Protect</td><td>Do Nothing
Protect</td><td>Do Nothing
Protect
Repair
Rehab</td><td>Do Nothing
Rehab
Replace</td></tr>
</table>

4.1.4 WisDOT, 2018

Condition assessment for prestressed concrete bridge superstructures is provided in the Structure Inspection Field Manual document. Table 7 shows crack width limits defining hairline to wide cracks and adopted from AASHTO Bridge Element Inspection Manual, 2010. Table 8 shows the condition assessment for prestressed concrete elements on a good, fair, poor, severe scale. Figure 41 through Figure 45 show examples of corrosion damage cases in concrete bridge girders.

Table 7: Crack width limits of prestressed concrete bridge elements, courtesy of WisDOT, 2018

Crack Definition	Crack Width
Hairline	< 0.004”
Narrow	0.004” to 0.009”
Medium	0.01” to 0.03”
Wide	> 0.03”

Figure 41: Condition State Poor, spalling on concrete bridge girder, courtesy of WisDOT, 2018

Figure 42: Condition State Fair, cracking on concrete bridge girder, courtesy of WisDOT, 2018

Figure 43: Condition State Fair, exposed strand on concrete bridge girder, courtesy of WisDOT, 2018

Figure 44: Condition State Poor, exposed strands on concrete bridge girder, courtesy of WisDOT, 2018

Table 8: Condition assessment for prestressed concrete elements, courtesy of WisDOT, 2018

	1	2	3	4
	Good	**Fair**	**Poor**	**Severe**
Delamination/ Spalls/Patch Areas/Exposed Prestressing	Patched area that is sound.	Delamination/Spalls 1 in. or less deep or less than 6 in. diameter. Reinforcing steel exposed. Corrosion may be present, but without section loss. Prestressing strands may be exposed without corrosion	Delamination/spalls greater than 1 in. deep or greater than 6 in. diameter. Patched area that is unsound or showing distress. Reinforcing steel present with measurable section loss. Prestressing strands exposed with corrosion. Does not warrant structural review.	Condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects impact strength or serviceability of the element or bridge
Cracking	Width less than 0.004 in. or sealed cracks	Width 0.004 – 0.009 in. Where efflorescence is present, it's minor and no evidence of rust staining	Width greater than 0.009 in. Where efflorescence is present, there is heavy build-up and/or rust staining	
Chloride Concentration	Chloride concentration at level of rebar tested below the threshold for potential active corrosion	Chloride concentration at level of rebar tested equal to or greater than the threshold for potential active steel corrosion. No visual signs of active corrosion exist	Chloride concentration at level of rebar tested greater than the threshold for potential active steel corrosion. Testing methods (such as half-cell potential) have been used and have verified active steel corrosion	Not used for this defect. Other reinforced or prestressed concrete defects control the Condition State over chloride concentrations (elevated levels of chloride concentrations may be a cause of controlling defects)

4.1.5 PennDOT BMS2, 2018

This report provides superstructure condition rating for deck and superstructure inspection as shown in Table 9. For Excellent rating, the element should have no deficiencies, while a Very Good rating is when there are no noticeable or noteworthy deficiencies affecting the condition. The structural members should be inspected for signs of distress which may include cracking, deterioration, section loss, and malfunction and misalignment of bearings. The condition of bearings, joints, paint system, etc., shall not be included in this rating, except in extreme situations, but should be noted on the inspection form. A condition rating for joints is given in Table 10.

Table 9: Condition rating for the superstructure, courtesy of PennDOT BMS2, 2018

Condition Rating	Percent of Strands Exposed	Deterioration of P/S Concrete Beams	
9 – Excellent	0%		No cracks, stains or spalls
8 – Very Good	0%		No cracks, stains or spalls
7 – Good	0%		Map cracks and miscellaneous hairline cracks
6 – Satisfactory	0%	Spalls	Minor Spalls/Delamination, < 5%
		Cracks	Map cracks and misc. hairline cracks
5 – Fair	1-5%	Spalls	Spalls/Delamination, < 15%
		Longitudinal Cracks	Hairline longitudinal cracks in the bottom flange
		Longitudinal Joints	Leakage at joints with light efflorescence
4 - Poor	6-15%	Spalls	Spalls/Delamination, 15 – 25%
		Transverse Cracks	Hairline flexure cracks across the bottom flange
		Longitudinal Cracks	Minor efflorescence and/or minor rust stains
		Longitudinal Joints	Heavy efflorescence and/or minor rust stains
		Transverse Tendons	Loose or heavily rusted
		Web Cracks	Initiation of vertical or diagonal cracks in P/S beam near open joints in barrier (< 3" length)
3 - Serious	15-20%	Spalls	Spalls/Delamination, > 25%
		Transverse Cracks	Open flexure cracks in the bottom flange
		Web Cracks	Vertical or diagonal cracks in P/S beam near open joints in barrier
		Camber	Sagging/Loss of camber
		Transverse Tendons	Broken or missing
2 – Critical	> 20%	All	Any condition worse than detailed above

Table 10: Condition rating for joints, courtesy of PennDOT BMS2, 2018

	1	2	3	4
	Good	**Fair**	**Poor**	**Severe**
Leakage	None	Minimal. Minor dripping through the joint	Moderate. More than a drip and less than free flow of water.	Free flow of water through the joint.
Seal Adhesion	Fully Adhered	Adhered for more than 50% of the joint height	Adhered 50% or less of joint height but still some adhesion	Complete loss of adhesion.
Seal Damage	None	Seal abrasion without punctures	Punctured or ripped or partially pulled out.	Punctured completely through, pulled out, or missing.
Seal Cracking	None	Surface crack	Crack that partially penetrates the seal.	Crack that fully penetrates the seal.
Debris Impaction	None	Partially filled with hard-packed material, but still allowing free movement	Completely filled and impacts joint movement	Completely filled and prevents joint movement.

4.1.6 MnDOT, 2019

The State of Minnesota Bridge Inspection Field Manual presents guidelines to superstructure condition rating as shown in Table 11 according to National Bridge Inspection Standards (NBIS). Corrosion related deterioration can be assessed by using these guidelines.

Table 11: Superstructure condition rating, courtesy of MnDOT, 2019

Code	Condition Rating
9	**Excellent Condition:** Superstructure is in new condition (recently constructed).
8	**Very Good Condition:** Superstructure has very minor (and isolated) deterioration.
7	**Good Condition:** Superstructure has minor (or isolated) deterioration. • Concrete: minor scale or non-structural cracking (isolated spalling/delamination)
6	**Satisfactory Condition:** Superstructure has minor to moderate deterioration. Members may be slightly bent or misaligned – connections may have minor distress. • Concrete: moderate scale or cracking (minor spalling/delamination)
5	**Fair Condition:** Superstructure has moderate deterioration. Members may be bent, bowed, or misaligned. Bolts/rivets may be loose/missing, but connections remain intact. • Concrete: extensive scaling or cracking (structural cracks may be present), moderate spalling or delamination (reinforcement may have some section loss)
4	**Poor Condition:** Superstructure has advanced deterioration. Members significantly bent or misaligned. Connection failure may be imminent. Bearings severely restricted. • Concrete: advanced scaling, cracking, or spalling (significant structural cracks may be present – exposed reinforcement may have significant section loss)
3	**Serious Condition:** Superstructure has severe deterioration – immediate repairs or structural evaluation may be required. Members may be severely bent or misaligned - connections or bearings may have failed. • Concrete: severe structural cracking or spalling
2	**Critical Condition:** Superstructure has critical damage or deterioration. Primary structural elements may have failed (severed, detached or critically misaligned). Immediate repairs may be required to prevent collapse or closure.
1	**Imminent Failure Condition:** Bridge is closed. Superstructure is no longer stable (corrective action might return the structure to restricted service).
0	**Failed Condition:** Bridge is closed due to superstructure failure and is beyond corrective action (replacement required).

Another MnDOT agency developed a condition rating system is provided for different structural elements on a scale of 1-4 as shown in Table 12. Condition state 1 is the best condition, with condition state 4 being the worst condition (this is the reverse of the NBI condition ratings).

Table 12: Superstructure condition rating, courtesy of MnDOT, 2019

	1	**2**	**3**	**4**
	Good	**Fair**	**Poor**	**Severe**
Structural Review	Structural review is not required.	Structural review is not required.	Structural review is not required **or** structural review has determined that the strength of the element has not been impacted.	Condition warrants structural review **or** structural review has determined that the strength of the element has been reduced.
Repairs	No repairs are present.	Existing repair in sound condition.	Repairs are recommended **or** existing repair is unsound.	Immediate repairs are required.
Delamination, Spall, or Exposed Rebar	None	Delamination. Spall 1" or less deep **and** 6" or less in diameter.	Spall greater than 1" deep **or** greater than 6" diameter. Exposed rebar with corrosion or section loss	Spalling deeper than 4" or exposed rebar with severe section loss.
Efflorescence, Rust Staining	None	Leaching without build-up (stalactites). Minor rust stains (rebar chairs).	Leaching with heavy build-up (stalactites).Rust stains indicating rebar corrosion.	Severe leaching (concrete unsound).
Scale, Abrasion, or Wear	Superficial	Coarse aggregate is exposed but remains secure	Coarse aggregate is loose or has popped out.	Severe voiding (concrete unsound).
Misalignment	None	Slightly out of position or alignment.	Significantly out of position or alignment.	Severely misaligned.
Cracking	Minor cracks	Moderate cracks or moderate map cracking. Sealed cracks.	Wide cracks or heavy map cracking. Minor or moderate shear/flexure cracks	Severe cracks or fractures. Wide shear or flexure cracks.

Figure 45: Condition State 2 cracking on precast concrete channel girders, courtesy of MnDOT, 2019

Figure 46: Condition State 3 water saturation, rust staining, and spalling on a cast-in-place concrete T-girder, courtesy of MnDOT, 2019

Figure 47: Condition State 3 cracking, delamination, and rust staining a precast concrete channel girder, courtesy of MnDOT, 2019

4.1.7 Proposed NDOT Classification of Damage at Girder Ends

This section will present the proposed classification of corrosion damage to be followed by NDOT. Table 13 presents the proposed damage classes with percent of exposed strands, examples, and proposed repair decision for each class.

Table 13: Proposed NDOT girder end damage classification

	Minor	Moderate	Extensive	Severe
Percent of Exposed Strands	0%	1-5%	6-15%	> 15%
Exposed Strands Surface Condition	None	Surface corrosion without pitting	Surface corrosion with light pitting	Heavy pitting, loss of strand area
Spalling	Isolated spalls less than 1” deep **and** less than 6” in diameter	Spall greater than 1” and less than 2” deep **and** less than 6” in diameter	Spall greater than 2” and less than 4” deep **or** greater than 6” diameter	Spalling deeper than 4”
Bearings Condition	Light corrosion not causing restriction	Light corrosion not causing restriction	Corrosion causing restriction	Failed Bearings
Joints Condition	No leakage	Minor dripping through the joint. Free movement still allowed	More than a drip and less than free flow of water. Partial movement allowed	Free flow of water through the joint. Movement restrained
Scale, Abrasion, or Wear	Coarse aggregate is exposed but remains secure	Coarse aggregate is loose or has popped out	Coarse aggregate is loose or has popped out	Severe voiding (concrete unsound).
Cracking	Isolated hairline cracks narrower than 0.004”	Hairline or map cracks 0.004” to 0.03” wide	Cracks wider than 0.03” or heavy map cracking. Initiation of shear/flexure cracks	Severe cracks or fractures. Wide shear or flexure cracks.
Proposed Repair/Replacement Method	Epoxy injection, and patching if required	Epoxy injection, patching, and FRP wrapping if required	Epoxy injection, patching, and FRP wrapping	Rehab or Replace girder
Example Figures	Figure 40 (A)	Figure 40 (C) Figure 42 Figure 43 Figure 45	Figure 40 (D) Figure 41 Figure 46 Figure 47	Figure 40 (E) Figure 48 Figure 50
References	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006

Figure 48: Severe class corrosion damage at girder end causing severe cracks, courtesy of Pantelides et al., 2010

Figure 49: Severe class corrosion damage at girder end causing severe cracks, courtesy of Choo et al., 2013

Figure 50: Severe class corrosion damage at girder end, MN/RC 2018-07 Report

4.2 Repair Methods

This section will present viable repair methods for the previously defined damage classes. A suggested procedure is presented for each individual repair method. At the end of this section, the overall proposed repair procedure for each damage class is presented.

4.2.1 Epoxy Injection

As discussed earlier in section 3.2.1, the epoxy injection can be used for sealing of cracks 0.007 in. wide and narrower. According to FHWA-NHI-14-050 bridge maintenance reference manual, cracks of 0.025 inch or wider should be epoxy injected. 0.025-inch-wide cracks fall in the range of medium cracks according to AASHTO Bridge Element Inspection Manual, 2010.

According to FHWA/TX-96 /1370-1, it has been recommended that cracks wider than 0.008 in. should be epoxy injected to restore girder durability, while finer cracks should be sprayed or brushed with saline seal to prevent the entry of moisture and deicing salts. 0.008-inch-wide cracks fall in the range of narrow cracks according to AASHTO Bridge Element Inspection Manual, 2010. The suggested procedure shall be similar to what is described in section 3.2.1.

4.2.2 Casting an End Block

Patching (discussed earlier in section 3.2.2) is used to restore the original girder cross-section, and it can be expanded to cast and end block. A common patching method for girder ends to eliminate the need for forming is shotcreting as shown in Figure 51 and Figure 52. According to Tabatabai et al., 2004 (Wisconsin Highway Research Project Report No. 0092-01-06), patch treatments can mend spalls, but typically do not retard chloride-induced corrosion. In such cases, this type of repair will typically fail prematurely since no measures are taken to mitigate the primary source of deterioration. In addition, since the newly placed concrete consists of minimal to no concentration of chlorides, a reverse chloride gradient is created between the patch repair and the existing concrete. The suggested procedure shall be similar to what is described in section 3.2.2. Casting an end block did not perform satisfactorily and the repair suffered significant cracks and spalls in two cases reported in MN/RC 2018-07 Report, and NDOT Kearney South Platte River Bridge (Appendix B - Previous NDOT Repair Cases for Girder End Damage).

Figure 51: Shotcreting repair at girder end, MN/RC 2018-07 Report

Figure 52: Shotcreting the bottom surface of the repair at girder end, MN/RC 2018-07 Report

Selection of Materials

Patch material used in Platte River Bridge East of Grand Island, Kearney South Platte River Bridge, and Platte River South Bridge (discussed in Appendix B - Previous NDOT Repair Cases for Girder End Damage) had a 28-day compressive strength of 6 ksi, modulus of elasticity in compression of 2940 ksi, 28-day drying shrinkage of 0.038 % (done on 3x3x11-1/4'' prism), and splitting tensile strength of 0.9 ksi.

4.2.3 FRP Wrapping

As discussed earlier in section 3.2.5, FRP wrapping is an effective method in both repairing corrosion damage and protection against future corrosion damage by excluding chloride-bearing water from the concrete (Tabatabi et al. 2004). AASHTO type II prestressed girders were repaired from severe cracking discussed by Choo et al., 2013, and shown in Figure 49 by using CFRP wrapped repair. A research project was done by the University of Kentucky and in cooperation with Kentucky Transportation Center and Federal Highway Administration to document and monitor the repairs. The repairs were done on two phases, first concrete cracks were sealed by means of high strength epoxy resin, and then the girders were strengthened with CFRP fabrics as shown in Figure 53. Crack monitoring gauges were installed to ensure the effectiveness of the repair. In addition, linear variable displacement transducers LVDTs were instrumented on two of the bridge girders to monitor vertical and horizontal translations. The CFRP repairs were effective in curtailing relative movement in horizontal directions as evidenced by the lack of movement as opposed to the volatile movement prior to the retrofit. However, vertical wrapping did not show a significant positive effect in reducing vertical relative displacements. The suggested procedure shall be similar to what is described in section 3.2.5.

Figure 53: CFRP fabrics installation on I-65 expressway bridge girders, courtesy of Choo et al., 2013

A research project was recently conducted by the University of Illinois and sponsored by the Illinois Department of Transportation (IDOT) to study the use FRP materials to repair and retrofit damaged ends of prestressed concrete beams as shown in Figure 54, Andrawes et al., 2018. Three-point bending tests were performed on small and full-scale prestressed concrete girders. Full-scale girders were retrieved from actual field bridges. The purpose of the full-scale tests was to evaluate the repair technique in repairing and retrofitting damaged girders and also the improvement of shear behavior. Five AASHTO Type II prestressed girders that were extracted after more than 40 years in service. Severe girder end region damage was simulated by removing the concrete cover to the centerline of the stirrups from the web all the way through the bottom flange. Afterward, a quick set mortar was applied to the damaged region to restore the shape of the girder, followed by CFRP laminate application. It was concluded that a mortar repair alone is not sufficient enough to recover the shear strength and ductility of the girder with the damaged end. A weak bond surface between mortar and base concrete and cracks developed above bearing plates diminished the repair effect from the mortar. And that externally bonded CFRP shear reinforcement repair was effective in recovering and even exceeding the shear capacity and ductility of the undamaged girders.

Figure 54: Applying shear CFRP to full-scale prestressed girder end, courtesy of Andrawes et al., 2018

Selection of Materials

Preformed CFRP strips have been used by NDOT in the girder end repair of Platte River Bridge East of Grand Island (discussed in Appendix B - Previous NDOT Repair Cases for Girder End Damage). The system had a design tensile strength of 550 ksi, modulus of elasticity of 33,000 ksi, rupture strain ε_{fu} of 0.017, and thickness of about 0.0065 in./ply.

The used CFRP fabrics used by Choo et al., 2013 (discussed above in this section) had a tensile strength of 120.5 ksi, tensile modulus of 12,320 ksi, ultimate tensile strain of 0.01, and a nominal ply thickness of 0.035 in.

Structural Calculations for Shear

Generally, the most critical concern related to corrosion damage at girder ends is the shear deficiency caused by section loss. The ACI 440.2R-17 provides guidelines to shear strengthening using externally bonded FRP systems. The shear contribution (V_f) of the FRP shear reinforcement is given by the below equation:

$$V_f = \frac{A_{fv}\, f_{fe}\, (\sin\alpha + \cos\alpha)\, d_{fv}}{S_f}$$

Where, A_{fv} is the area of FRP shear reinforcement with center-to-center spacing S_f, d_{fv} is the effective depth of FRP shear reinforcement, α is the angle of application of FRP reinforcement direction relative to longitudinal axis of the member, f_{fe}, and ε_{fe} is the effective stress and strain respectively in the FRP at failure and calculated according to the below equation

$$f_{fe} = E_f\, \varepsilon_{fe}$$

$$\varepsilon_{fe} = k_v \, \varepsilon_{fu} \leq 0.004$$

k_v is a bond dependent coefficient for shear of bonded U-wraps or bonded face plies and is empirically calculated depending on concrete compressive strength, bonded area, number, thickness, and tensile modulus of elasticity of FRP plies.

An additional reduction factor Ψ_f is applied to the contribution of the FRP system as follows:

$$\Phi \, V_n = \Phi\left(V_c + V_s + \Psi_f V_f\right)$$

Ψ_f is recommended to be taken by 0.95 for completely wrapped members, and 0.85 Three-side or two-opposite-sides schemes strengthening.

Andrawes et al., 2018, proposed a design methodology to estimate the required thickness of CFRP. The total loss of shear capacity (ΔF) is first estimated. Then the below equation is used as follows:

$$\Delta F = f_r \times l \times c$$

Where fr is the concrete modulus of rupture, c is the estimated thickness of the damaged concrete section, and l is the longitudinal projection of initial shear crack as shown in Figure 55. The initial shear crack appears mainly in the web, and for simplicity, it can be assumed to be at an angle of approximately 45° with respect to the longitudinal axis of the girder. Then the average longitudinal tensile strain in the web ($\boldsymbol{\varepsilon}_x$) is estimated depending on the applied loads and prestressing and shown in the below equation.

$$\varepsilon_x = \frac{\frac{|M_u|}{d_v} + 0.5\, N_u + 0.5\left|V_u - V_p\right| \cot\theta - A_{ps}\, f_{po}}{2\left(E_s A_s + E_p A_{ps}\right)}$$

Where $\boldsymbol{\theta}$ is the angle between diagonal compressive stress and the longitudinal axis of the beam as shown in Figure 7.4; $\boldsymbol{M_u}$ is factored moment and is not taken less than $\boldsymbol{V_u d_v}$; $\boldsymbol{V_u}$ is factored shear force; $\boldsymbol{N_u}$ is factored axial force; $\boldsymbol{V_p}$ is component in the direction of the applied shear of the effective prestressing force; $\boldsymbol{d_v}$ is effective shear depth. $\boldsymbol{A_s}$ and $\boldsymbol{A_{ps}}$ are the area of non-prestressing tensile reinforcement and area of prestressing steel, respectively; $\boldsymbol{E_s}$ and $\boldsymbol{E_p}$ are the Young's modulus of non-prestressing tensile reinforcement and prestressing steel, respectively. $\boldsymbol{f_{p0}}$ is a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and surrounding concrete, for the usual levels of prestressing, a value of $0.7\boldsymbol{f_{pu}}$ is appropriate (AASHTO 2017).

Next principle tensile strain (ε_1) is calculated and as follows:

$$\varepsilon_1 = \varepsilon_x + \left[\varepsilon_x + 0.002\left(1 - \sqrt{1 - \frac{v_u}{f'_c}\,\frac{0.8 + 170\, \varepsilon_1}{\sin\theta \cos\theta}}\right)\right] \cot^2\theta$$

The directions of $\boldsymbol{\varepsilon_1}$ and $\boldsymbol{\varepsilon}_y$ are described in Figure 56. The vertical strain component ($\boldsymbol{\varepsilon_y}$) is obtained by multiplying $\boldsymbol{\varepsilon_1}$ by cos θ. The FRP design strain ($\boldsymbol{\varepsilon}_{FRP}$) is computed by dividing $\boldsymbol{\varepsilon_y}$ by a factor $\boldsymbol{\mu}$. The factor $\boldsymbol{\mu}$ is defined as the ratio between the vertical component of the ultimate strain and the strain of FRP laminate at peak force. Based on the small-scale and full-scale girder tests done by Andrawes et al., 2018, the value of $\boldsymbol{\mu}$ was found to be on average equal to 6.0. If the computed $\boldsymbol{\varepsilon}_{FRP}$ is found to be greater than 0.004, a value of 0.004 should be used to design FRP laminate as recommended by the ACI 440.2R-17.

$$\varepsilon_{FRP} = \frac{\varepsilon_y}{\mu} = \frac{\varepsilon_1 \, cos\theta}{\mu}$$

Finally, the required material thickness is estimated as follows:

$$t = \frac{\Delta F}{E_{FRP} \; x \; L \; x \; \varepsilon_{FRP}}$$

Where $\boldsymbol{E}_{FRP}$ is the Young's modulus of the selected FRP material, $\boldsymbol{L}$ is the length of shear span required to be strengthened.

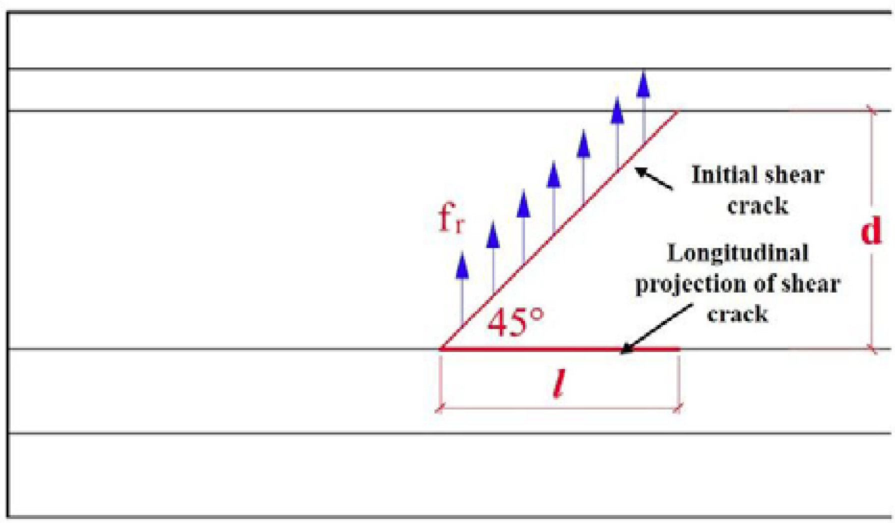


Figure 55: Tensile force loss, courtesy of Andrawes et al., 2018

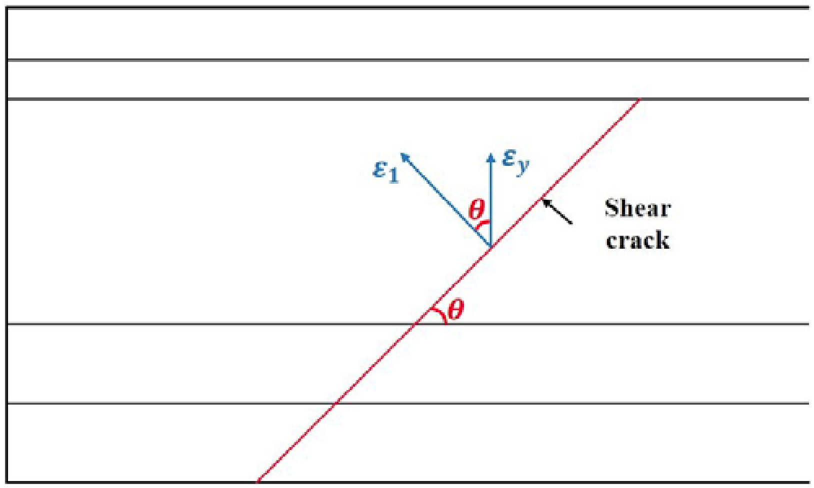


Figure 56: Principle strain direction, courtesy of Andrawes et al., 2018

4.2.4 Bearing and Joints Replacement

Malfunctioning of bearings or joints usually causes girder end deterioration if the problem was not addressed as soon as it occurs. When bearings start to show signs of heavy corrosion, a partial restriction of movement is introduced to the bridge girder leading to the initiation of structural cracks. When joints start to show signs of leakage, the degree of exposure to de-icing salts and harmful substances increases significantly under their location which is typically at the girder ends. Replacement of malfunctioning bearing or joints is considered an essential part of repairing the girder end and its protection against future deterioration.

4.2.5 Ultra-high Performance Concrete

The Connecticut Department of Transportation has been leading efforts to develop a UHPC repair solution for deteriorated steel bridge girder end through work at the University of Connecticut, Zmetra, 2015, and Graybeal, 2017. Large-scale experimental testing and detailed analytical modeling are done on corrosion damage in steel bridge girders' end. A repair was done by casting two thin UHPC panels on each side of the girder web and connecting them by shear studs welded at the undamaged part of the web and bottom flange as shown in Figure 57. The use of the UHPC panels is to provide an alternate load path at the end zone of the bridge girder. The main conclusions were that UHPC allows for both increased shear resistance of the girder as well as increased bearing resistance at the support. Experimental studies showed that girder ends repaired using this concept could meet or exceed their intended capacity at the ultimate limit state.

UHPC has been used lately in three concrete girder repair cases according to FHWA UHPC bridges interactive map: beam-end repair in Providence, Rhode Island in 2018, a prestressed U-beam repair in Jacksonville, Florida in 2017, and the repair of concrete girder bearing seat in Lakehead, California in 2016. Also, UHPC has been reported to be used as structural patching for prestressed concrete bridge girders, Haber and Graybeal, 2019. A shear strength design example of the undamaged section according to AASHTO LRFD, 2017, and shear strength of the repaired section according to AFGC, 2013 is presented in (Appendix E – Structural Calculations Design examples).

Figure 57: UHPC encasement at the repaired girder end zone, courtesy of Zmetra, 2015

Suggested Procedure for Recasting with New UHPC

According to patching with recasting new concrete discussed in section 3.2.2, and a previous case study published on Ductal website (recasting a thin UHPC jacket around a deteriorated bridge pier), the following procedure is recommended:

- Before starting to prepare the surface, determine if there are any structural capacity concerns from removing unsound concrete to the depths and limits necessary for the repair. Place any required temporary shoring or bracing necessary to support the structure during the repair.
- Sawcut the perimeter edges straight to a depth of ¾ inch. Remove any loose concrete in the area to be patched. Concrete should be removed 1 inch all around exposed rebar whenever possible.
- The existing surface should be cleaned by light sandblasting. The concrete surface should be saturated with water spray, if dry, and then allowed to return to a surface dry condition. This will prevent the old concrete surface from absorbing the new concrete mixing water.
- Install formwork. The formwork should be rigid enough to prevent new concrete from sagging away from the existing concrete under the weight of new concrete. The forms should be watertight as UHPC is a flowable, self-leveling material.
- Prior to placing the concrete, the forms should be cleaned, sprayed with a form release agent and wetted to prevent absorption of the water used in the concrete.

- Apply a bonding agent (usually a cement grout) onto the concrete surface just before the installation of formwork. Interface shear resistance of UHPC to roughened concrete surfaces could be sufficient without the need of a bonding agent. The manufacturer's recommendations should specify whether a bonding agent is required or not.
- Place UHPC through holes in the top of the formwork for vertical patches as shown in Figure 58 and Figure 59. If the top of formwork is not accessible, inverted patches could be cast from below through fill holes in the member as shown in Figure 60. If inverted patches cannot be recast from above consider using the shotcrete repair method.
- Allow the concrete to cure.
- Remove the formwork and grind off any excess concrete or fill any voids that were formed.

Figure 58: Casting UHPC at a steel girder end from the top of formwork, courtesy of Zmetra, 2015

Figure 59: Casting UHPC for a thin jacket for a bridge pier from the top of formwork, CN Rail bridge, Quebec, Canada, obtained from Ductal website

Figure 60: Casting UHPC for a thin jacket for a bridge pier from middle openings in the formwork, CN Rail bridge, Quebec, Canada, obtained from Ductal website

Suggested Procedure for Shotcreteing

Sprayable UHPC has been reported to be used in several applications. Bernardi et al., 2016, reported the production of sprayable UHPC having 17 ksi 28-day compressive strength and a significantly lower diffusion coefficient compared to other cementitious materials. Sprayable UHPC can be used as a shotcrete patching material alternative as reported earlier in section 4.2.2.

According to FHWA-NHI-14-050 bridge maintenance reference manual suggested procedure for shotcrete patching discussed in section 3.2.2, the following procedure is recommended

- Prepare the existing surface. The edges of the repair area should be sawcut at least ¾ inch deep at a 45-degree angle into the repair area to prevent the rebound of the shotcrete material. All deteriorated concrete should be removed to a minimum of 1 inch behind exposed reinforcement. All surfaces should be cleaned with high-pressure water or by sandblasting.
- For repairs 3 inches or deeper, welded wire fabric or wire mesh should be mechanically affixed to the existing concrete surface prior to the placement of the shotcrete. The wire mesh will help ensure the integrity of the repaired area and limit cracking.
- Wet the existing surface so it does not absorb water from the pneumatic mortar.
- Apply shotcrete according to the manufacturer’s recommendations. Figure 61 shows an example of using UHPC shotcrete in the renovation of a metal culvert.
- Allow UHPC to cure until reaching the desired strength.

Figure 61: UHPC shotcrete to renovate a deteriorated metal culvert suffering from corrosion damage, obtained from Ductal website

4.2.6 Proposed NDOT Repair Methods and Procedures for Girder End Damage

Based on the existing literature, the suggested procedure for individual repair methods like epoxy injection, patching, and FRP wrapping is presented in sections 3.2.1, 3.2.2, and 3.2.5 respectively for vehicular collision damage, and sections 4.2.1, 4.2.2, and 4.2.3 respectively for girder end damage.

The proposed methods and procedures in this section are according to FHWA-NHI-14-050 bridge maintenance reference manual and previous NDOT girder end repair cases.

A. For Minor damage class

Patching can be performed with trowel-applied or poured mortar method (discussed in section 3.2.2). The suggested procedure shall be as follows:

1. Remove the deteriorated and unsound concrete in steps.
2. Epoxy inject any visible cracks.
3. Address future corrosion protection (if required).
4. Apply epoxy bonding agent to prepare the surfaces of the girder end.
5. Place the new concrete. A non-shrink additive should be used in the new concrete.
6. Check for possible distress in the repaired area.

B. For Moderate damage class

1. Remove the deteriorated and unsound concrete in steps.
2. Epoxy inject any visible cracks not within the removal limits.
3. Clean exposed reinforcement and strands by sandblasting.
4. Place new reinforcement as needed, making sure it is properly lapped, anchored, or mechanically attached to the existing steel. Bars can be welded to the existing longitudinal bars as well.
5. Address future corrosion protection (if required).
6. Apply an epoxy bonding agent to prepare the surfaces of the girder end.

7. Assemble the forms for the new concrete in cast of patching with recasting new concrete.
8. Place the new concrete. A non-shrink additive should be used in the new concrete.
9. Apply FRP wrapping (if required) according to structural calculations.
10. Check for possible distress in the repaired area.

C. For Extensive or Severe damage classes

1. Restrict vehicle loads on the affected girder by directing traffic to the far side of the bridge until repairs on the girder end are complete.
2. Determine if the existing substructure can be used to jack the bridge up or if a jacking bent will need to be constructed. The jacking supports and jacking procedures should be reviewed by an engineer before any lifting begins.
3. Place jacks and raise the entire end of the bridge. The lift should only be enough to take the load off and to allow a piece of sheet metal to be inserted on the girder seat as a bond breaker for the new concrete. Check with an engineer if this step is necessary
4. Sawcut the concrete edges in a stepped fashion to avoid feathered edges and to provide bearing surfaces for the new concrete.
5. Follow steps 1 through 8 in moderate damage (consider casting an end block or using UHPC).
6. Replace bearing assemble.
7. Apply FRP wrapping according to structural calculations.
8. Uniformly lower the end of the bridge. After the concrete has reached sufficient strength, and enough curing time is provided for the FRP adhesive.
9. Check for possible distress in the repaired area.
10. Remove the jacking system.
11. Inspect leaking joints. Remove any debris inside the joint (manually by brushing, chipping and scraping, or by high-pressure jet washing). Replace any loose or damaged joint seals. Replace or relocate the entire joint if severely deteriorated.

Chapter 5 – Summary

For the over-height vehicular collision damage:

Damage levels were divided into minor, moderate, severe I, severe II, severe III. Classification is done according to cracks, spalls, exposed or damaged strands, and loss of camber/deflection. Generally minor class is when no strands or reinforcement are exposed; moderate class is when strands are exposed but not damaged; severe I is when live load capacity is not significantly affected (only loss of camber); severe II is when less than 20% of the strands are damaged; severe III is when damage exceeds severe II limits and girder replacement is recommended.

Repair methods and procedures vary by each damage class. For minor damage, minimal repair works are required, patching, and/or epoxy injection could be done based on aesthetic needs; for moderate damage, patching and/or epoxy injection should be done to prevent future corrosion of exposed bars; for severe I, FRP wrapping, steel jacket, or strand splicing should be sufficient to satisfy strength limit state, followed by patching and/or epoxy injection; for severe II, strand splicing or external post-tensioning should be sufficient without the need to replace the girder.

Six previous over-height vehicular collision impact cases were documented to support the proposed classification and repair/replacement decisions taken. The proposed damage classification and repair methods/procedures are summarized in Appendix C – Summary of Damage Classification and Repair Methods and Procedures for Over-height Vehicular Collision Damage.

For damage at girder ends:

Damage levels were divided into minor, moderate, extensive, severe. Classification is mainly done according to cracks, spalls, fractures, and exposed strands and their surface condition. Generally minor class is when spalls are too small and shallow so that no strands or reinforcement are exposed; moderate class is when strands are exposed but their surface condition shows no pitting; extensive class is when exposed strands have surface pitting, and shear or flexure cracks are initiating; severe class is when exposed strands have heavy pitting indicating section loss, and severe cracks or fractures are evident.

Repair methods commonly used are epoxy injection, patching, and FRP wrapping. For minor damage, minimal repair works are required, patching, and/or epoxy injection could be done based on aesthetic needs; for moderate damage, patching and/or epoxy injection must be done to prevent future corrosion of exposed bars; for extensive damage, patching and epoxy injection followed by FRP wrapping is recommended; for severe damage, girder rehabilitation to change bearing assemblies is recommended, girder replacement could be an option if damage is too severe and at both ends.

Four previous girder end damage cases were documented to support the proposed classification and repair/rehabilitation/replacement decisions taken. The proposed damage classification and repair methods/procedures are summarized in Appendix D – Summary of Damage Classification and Repair Methods and Procedures for Girder Ends Damage.

References

AASHTO LRFD Bridge Design Specifications. Washington, D.C.: American Association of State Highway and Transportation Officials, 8th edition, 2017.

AASHTO Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements. Washington, D.C.: American Association of State Highway and Transportation Officials, 1st edition, 2012.

AASHTO Bridge Element Inspection Guide Manual. Washington, D.C.: American Association of State Highway and Transportation Officials, 1st edition, 2010.

Alberta Infrastructure and Transportation, Bridge Engineering Section Technical Standards Branch, Repair Manual for Concrete Bridge Elements, 2005.

Ali, M. G. and Maddocks, A. R. Evaluation of Corrosion of Prestressing Steel in Concrete Using Non-destructive Techniques, Corrosion and Materials, Vol. 28, No. 5-6, pp 42-48, 2003.

American Concrete Institute (ACI) Committee 440. ACI 440.2R-17 “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures”, 76pp, 2008.

Andrawes, A., Shaw, D., and Zhao, H., Repair & Strengthening of Distressed/Damaged Ends of Prestressed Beams with FRP Composites, University of Illinois at Urbana-Champaign, Report No. FWHA-ICT-18-001, 2018.

Association Francaise de Génie Civil. Ultra high performance fibre-reinforced concretes. Interim Recommendations. AFGC publication, France, 2013.

Belarbi, Abdeldjelil. Design of FRP systems for strengthening concrete girders in shear. Vol. 678. Transportation Research Board, 2011.

Bernardi, S., Perez, F., and Fenzy, E., New UHPFRC for the realization of complex elements, First International Symposium on UHPC, 2016.

Boyd, A., Liang, N., Green, P., and Lammert, K., Sprayed FRP repair of simulated impact in prestressed concrete girders, Construction and Building Materials 22, 411–416, 2008.

Bridge Office Policies and Procedures (BOPP) Manual, Nebraska Department of Roads, 2016.

Casadei, P., Galati, N., Boschetto, G., Tan, K.Y., Nanni, A. and Galeki, G. Strengthening of Impacted Prestressed Concrete Bridge I-Girder Using Prestressed Near Surface Mounted CFRP Bars, Proceedings of the 2nd International Congress, Federation Internationale du Beton, Naples, Italy, 2006.

Chauvin, M., Shield C.K., French, C.E., Smyrl, W. Evaluation of Electrochemical Chloride Extraction (ECE) and Fiber Reinforced Polymer (FRP) Wrap Technology. Minnesota Department of Transportation, St. Paul, MN, 2000.

Choo, C., Peiris, A., and Harik, I., Repair of I-65 Expressway Bridges using Carbon Fiber Reinforced Polymer (CFRP) Composites, Kentucky Transportation Center, College of Engineering, University of Kentucky, 2013.

Commonwealth of Pennsylvania Department of Transportation Bureau of Maintenance and Operations, Bridge Management System 2 - Coding Manual, PennDOT BMS2 Publication 100A, Harrisburg, PA, 2018.

Ductal Website, https://www.ductal.com/en/engineering/cn-rail-bridge-quebec-canada, N Rail bridge, Quebec, Canada - Pier Jacketing, information obtained in May 2019.

Ductal Website, https://www.ductal.com/en/engineering/regeneration-and-reinforcement-of-metal-culvert, Regeneration and reinforcement of metal culvert, information obtained in May 2019.

ElSafty, A., Graeff, M., and Fallaha, S., Behavior of Laterally Damaged Prestressed Concrete Bridge Girders Repaired with CFRP Laminates Under Static and Fatigue Loading, International Journal of Concrete Structures and Materials, 2013.

Federal Highway Administration, Bridge Maintenance Reference Manual, Publication number FHWA-NHI-14-050, 2015.

FHWA. 2018. Ultra-High Performance Concrete Interactive Web Map. http://usdot.maps. arcgis.com/apps/webappviewer/index.html?id=41929767ce164eba934d70883d775582, information obtained in February 2018.

Feldman, L., Jirsa, O., Fowler, D., and Carrasquillo, R., Current Practice in the Repair of Prestressed Bridge Girders. Project Report No. FHWA/TX-96 /1370-1, 1996.

Graybeal, B. A. Structural Behavior of Ultra-High Performance Concrete Prestressed I-Girders. Publication No. FHWA-HRT-06-115. United States. Federal Highway Administration. Office of Infrastructure Research and Development, 2006.

Graybeal, Benjamin A. "Emerging UHPC-based bridge construction and preservation solutions.", 2017.

Haber, Zachary B., and Benjamin A. Graybeal. Advancing Bridge Repair and Preservation Using Ultra-High-Performance Concrete. ASPIRE magazine, 2019.

Hadzor, Thomas J., et al. Development of acoustic emission evaluation method for repaired prestressed concrete bridge girders. No. FHWA/ALDOT 930-601-1. Auburn University. Highway Research Center, 2011.

Harries, K., Kasan, J., and Miller, R., Guide to Recommended Practice for the Repair of Impact-Damaged Prestressed Concrete Bridge Girders, NCHRP Project 20-07 Report, 2012

Harries, K., Kasan, J., and Aktas, C, Repair Method for Prestressed Girder Bridges, Commonwealth of Pennsylvania Department of Transportation, 2009

Iowa Department of Transportation, Highway Division, Office of Bridges and Structures, Iowa DOT, Emergency Response Manual for Over Height Collisions to Bridges, 2014.

Jones, Mark Stevens. Repair of Impact-Damaged Prestressed Bridge Girders Using Strand Splices and Fabric Reinforced Cementitious Matrix. Diss. Virginia Tech, 2017.

Kasan J., Structural Repair of Prestressed Concrete Bridge Girders, Master of Science Thesis, University of Pittsburgh, Pennsylvania, 2009.

Kay, T., Assessment and Renovation of Concrete Structures. Long man Scientific & Technical, Essex, England, 1992.

Lim, W. Y., & Hong, S. G. Shear Tests for Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) Beams with Shear Reinforcement. International Journal of Concrete Structures and Materials, 10(2), 177–188, 2016.

Mehta, P. K. and Monteiro, P. J. M. Concrete, Microstructure, Properties, and Materials, McGraw-Hill, 2006.

Minnesota Department of Transportation, Project Report No. MN/RC 2018-07, BR27568 – Experimental Shear Capacity Comparison between Repaired and Unrepaired Girder Ends, 2018.

Minnesota Department of Transportation, Bridge and Structure Program Inspection Manual, 2019.

Naito, C., Sause, R., Hodgson, I., Pessiki, S. & Desai, C. "Forensic Evaluation of Prestressed Box Beams from the Lake View Drive over I-70 Bridge", ATLSS Report No. 06-13, Lehigh University, Bethlehem, PA. 62 pp., 2006.

Pantazopoulou, S.J., Bonacci, J.F., Thomas M.D.A., Hearn, N. Repair of Corrosion Damaged Columns with FRP Wraps. The Journal of Composites of Construction, v. 5, n. 1, p. 3-11, February 2001.

Pantelides, P., Reaveley, D., and Burningham. A., Repair of prestressed concrete girder ends and girder collision repair. Report No. UT-10.04. 2010.

Parretti R., and Nanni A., Strengthening of RC Members Using Near-Surface Mounted FRP Composites: Design Overview, Advances in Structural Engineering Vol. 7 No. 5, 2004

Precast/Prestressed Concrete Institute, PCI Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products, Chicago, IL, 2006.

Ramadan, S., Gaillet, L., Tessier, C., Idrissi, H. Detection of Stress Corrosion Cracking of High-Strength Steel Used in Prestressed Concrete Structures by Acoustic Emission Technique, Applied Surface Science, Vol. 254, No. 8, pp 2255-2261., 2007.

Scheel, H. and Hillemeier, B. Location of Prestressing Steel Fractures in Concrete, ASCE Journal of Materials in Civil Engineering, Vol. 15, No. 3, pp 228-234., 2003.

Shanafelt, O. and Horn, B. “Damage Evaluation and Repair Methods for Prestressed Concrete Bridge Members”, NCHRP Report 226, Project No. 12-21, Transportation Research Board, Washington, D.C., 1980.

Shanafelt, O., and Horn, B. "Guidelines for evaluation and repair of prestressed concrete bridge members, NCHRP Report 280, Project No. 12-21 (1)." Transportation Research Board, Washington, DC 1985.

Tabatabai, H., Ghorbanpoor, A., Nass. A., Rehabilitation Techniques for Concrete Bridges, Wisconsin Highway Research Program Project Report No. 0092-01-06, 2004.

Tabatabai, H., Nabizadeh, A., Strength and Serviceability of Damaged Prestressed Girders, Wisconsin Highway Research Program Project Report No. 0092-17-02, 2019.

Turker, H. T. Theoretical Development of the Core-Drilling Method for Nondestructive Evaluation of Stresses in Concrete Structures, Doctoral. Dissertation, Lehigh University, 2003.

Walther, A. and Hillemeier, B. Fast NDT Localization of Prestressing Steel Fractures in P-T Concrete Bridges, 12th International Conference and Exhibition on Structural Faults and Repair, Edinburgh, Scotland, 2008.

Whiting, D., Tabatabai, H., Stejskal, B., Nagi, M. Rehabilitation of Prestressed Concrete Bridge Components by Non-Electrical (Conventional) Methods. Federal Highway Administration Publication FHWA-RD-98-189, McLean, VA, February 1999.

Wipf, T., Phares, B., Sritharan, S., Degen, B., & Giesmann, M. T. Design and Evaluation of a Single-Span Bridge Using Ultra-High Performance Concrete, 2009.

Wisconsin Department of Transportation, Structure Inspection Field Manual, 2018.

Zmetra, Kevin M. "Repair of Corrosion Damaged Steel Bridge Girder Ends by Encasement in Ultra-High Strength Concrete.", 2015.

Zobel, R., Jirsa, O., Fowler, D., and Carrasquillo, R., Evaluation and Repair of Impact-damaged Prestressed Concrete Bridge Girders, Project Report No. FHWA/TX-97 /1370-3F, 1997.

Appendix A - Previous NDOT Repair Cases for Over-height Vehicular Collision

168th St. over West Dodge Road Bridge

The bridge was located in Douglas County. The bridge construction took place around the year 1998. The bridge structural system consisted of 21 prestressed NU1100 girders, with a simple span length of 65 feet as shown in Figure 62. Each girder had a total of 36 strands distributed on two bottom rows as shown in Figure 63.

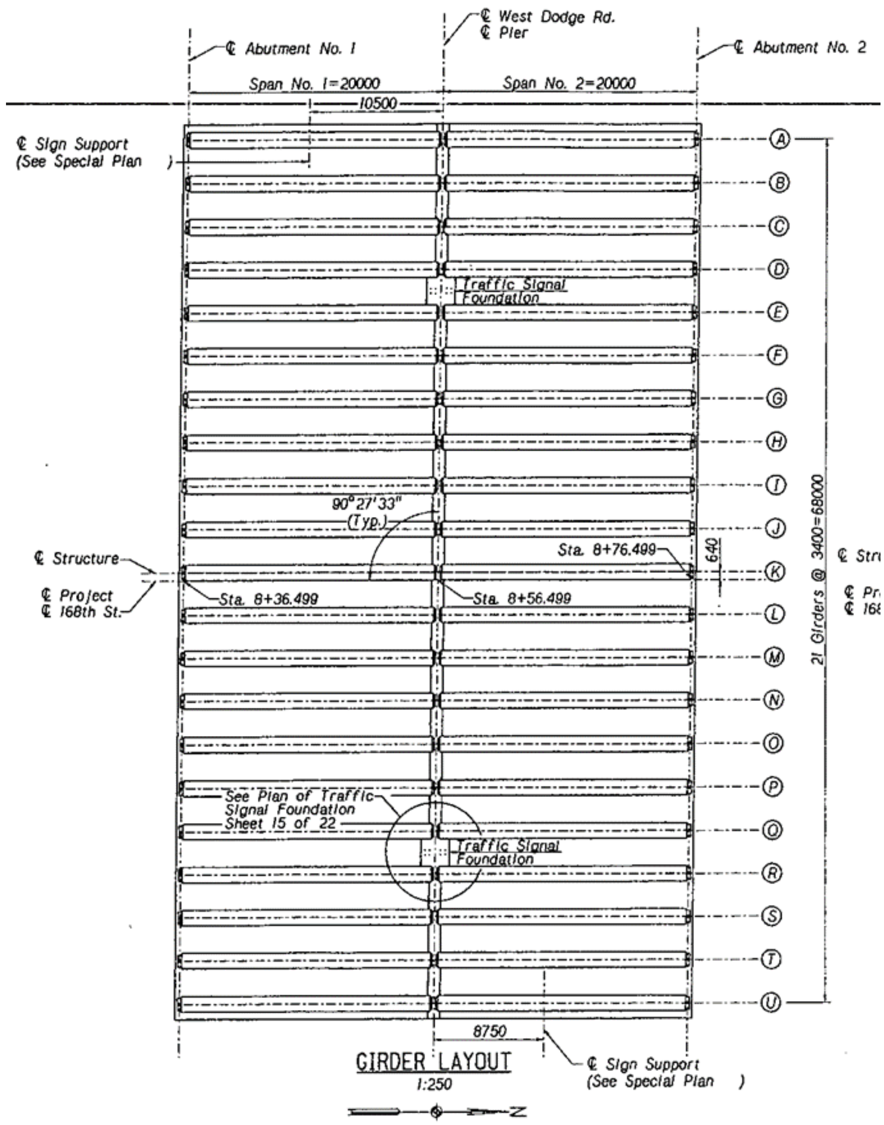


Figure 62: Plan view from the construction documents of 168th St. over West Dodge Road Bridge system, courtesy of NDOT

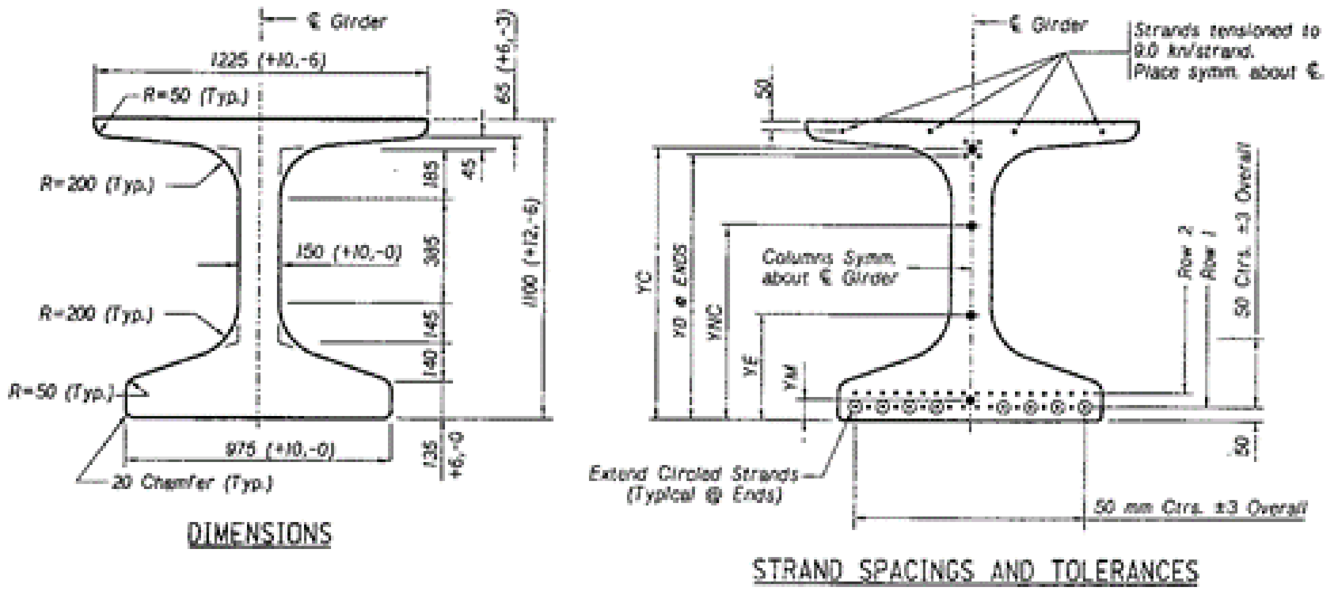


Figure 63: Girder dimensions and strand distribution, from the construction documents of 168th St. over West Dodge Road Bridge, courtesy of NDOT

Damage occurred to the bottom flange of an exterior girder as shown in Figure 64 and Figure 65. The spalled area was about 16.91 in. deep over 29 in. length of the bottom flange as shown in Figure 66. At least four strands severed as shown in Figure 66. According to the proposed NDOT damage classification, this damage was classified as Severe II class.

Figure 64: Level of damage, 168th St. over West Dodge Road Bridge, courtesy of NDOT

Figure 65: Level of damage, 168th St. over West Dodge Road Bridge, courtesy of NDOT

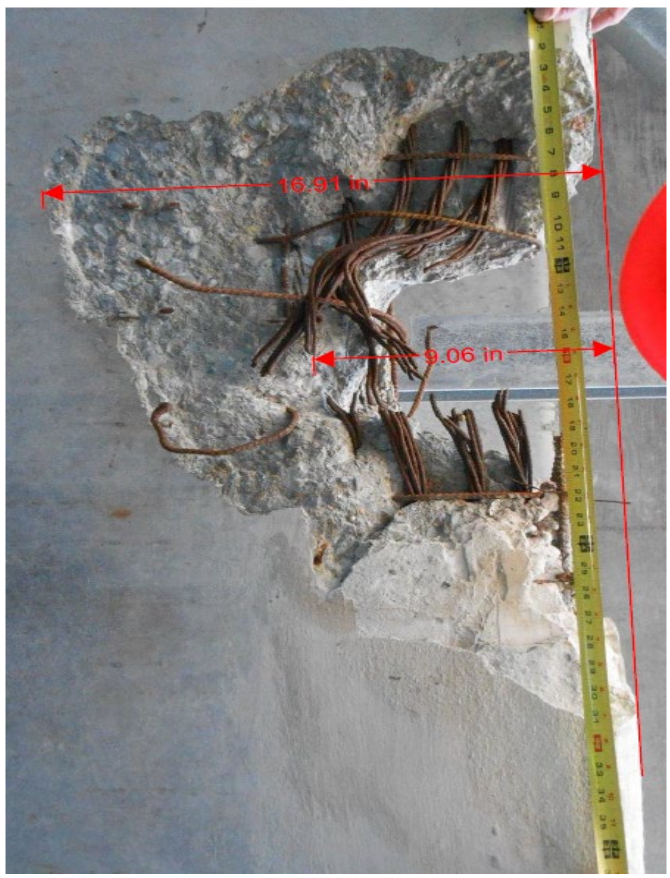


Figure 66: Level of damage, 168th St. over West Dodge Road Bridge, courtesy of NDOT

Repair works took place on June 2015. The latest inspection done on February 2016 reported the superstructure NBI condition rate to be 8 with no notable deficiencies on the repaired girder. A visual inspection visit was done on August 2019 with no notable deficiencies on the repaired girder as shown in Figure 67.

Figure 67: Repaired girder with no notable deficiencies, 168th St. over West Dodge Road Bridge, courtesy of NDOT

Schuyler Bridge

The bridge was located in Colfax County. The bridge construction took place around the year 2001. The bridge structural system consisted of 5 prestressed NU1100 girders, with a simple span length of 78 feet as shown in Figure 68. Each girder had a total of 56 strands distributed on six rows as shown in Figure 69.

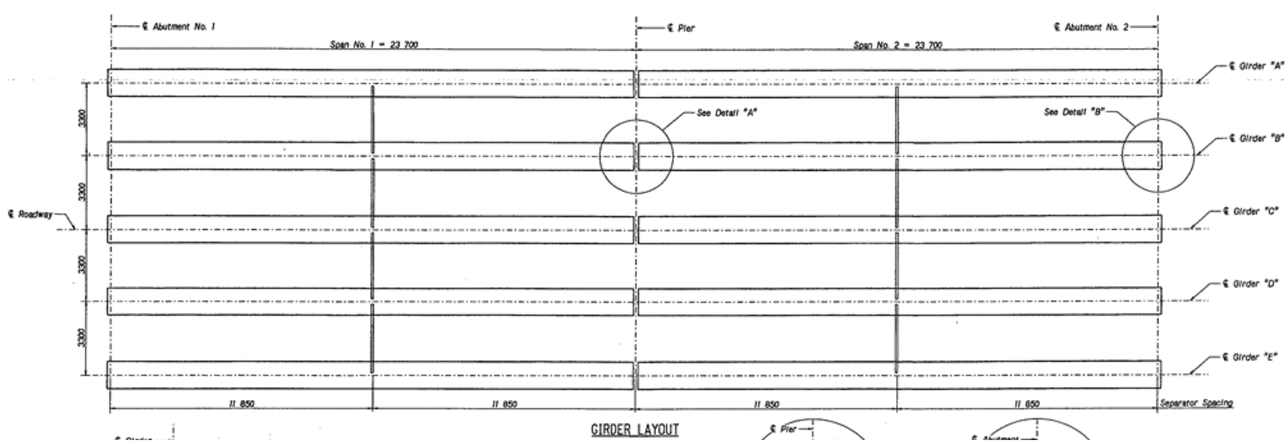


Figure 68: Plan view from the construction documents of Schuyler Bridge system, courtesy of NDOT

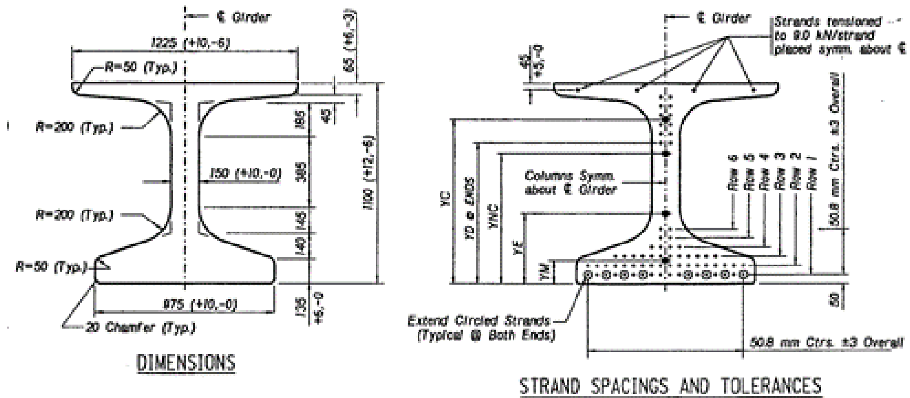


Figure 69: Girder dimensions and strand distribution, from the construction documents of Schuyler Bridge, courtesy of NDOT

With reference to the accident report, on December 2017 a Union Pacific truck with a boom on the back end hit the bridge at an exterior girder in span 2 as shown in Figure 70. Damage occurred at the bottom flange and the web and severed at least 8 strands as shown in Figure 71. No other damage appeared to the bridge girders or components. The girder replacement was done as shown in Figure 72. According to the proposed NDOT damage classification, this damage was classified as Severe III class.

Figure 70: Level of damage, Schuyler Bridge, courtesy of NDOT

Figure 71: Level of damage, Schuyler Bridge, courtesy of NDOT

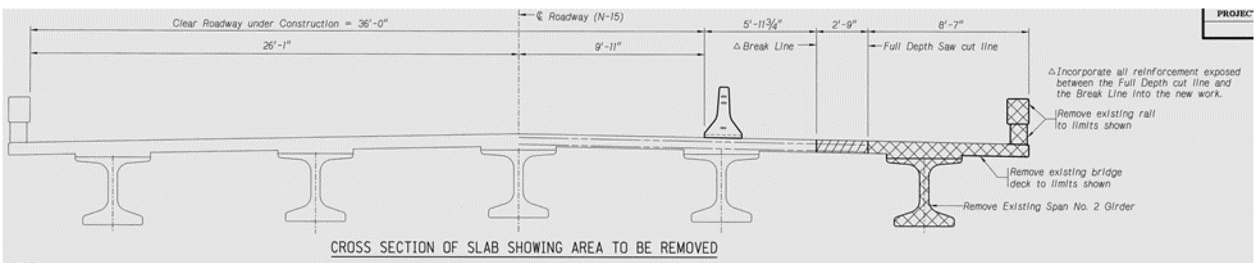


Figure 72: Girder replacement cross-section, from the replacement documents after the damage of Schuyler Bridge, courtesy of NDOT

Replacement works took place on January 2018. The latest inspection done on June 2019 reported the superstructure NBI condition rate to be 8 with no notable deficiencies on the replaced girder. A visual inspection visit was done on August 2019 with no notable deficiencies on the replaced girder as shown in Figure 73.

Figure 73: Replaced girder with no notable deficiencies, Schuyler Bridge, courtesy of NDOT

Scottsbluff Gering Bypass Bridge

The bridge was located in Scottsbluff County. The bridge was constructed in 2003. The bridge structural system consisted of 4 prestressed NU1100 girders, with a simple span length of 60, 105, and 60 feet as shown in Figure 74. The system consisted of two twin bridges identical in everything. The bridge had a skew angle of 45^0. The intermediate span girders at which damage occurred had a total of 38 strands distributed on three rows as shown in Figure 75.

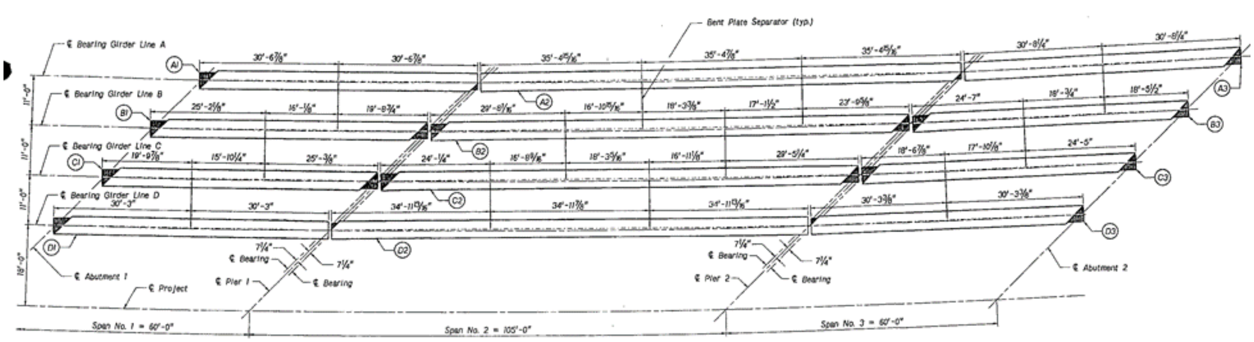


Figure 74: Plan view from the construction documents of Scottsbluff Gering Bypass bridge system, courtesy of NDOT

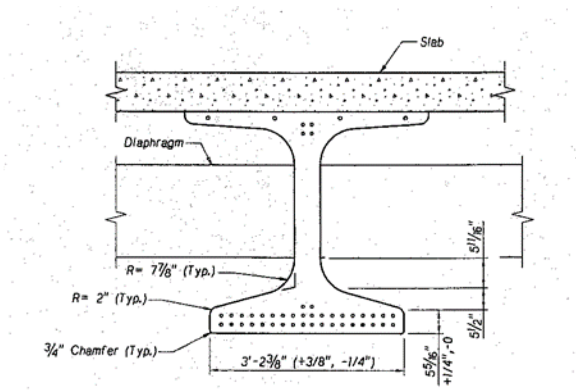


Figure 75: Girder reinforcement and strand distribution, from the construction documents of Scottsbluff Gering Bypass Bridge, courtesy of NDOT

With Reference to the accident report, six strands were completely exposed and two were partially exposed. Three strands from the bottom row were completely severed as shown in Figure 76 and Figure 77. No damage was found on the other strands. Nine D4 wires from WWF5 were exposed out of which five were severed at the bottom. According to the proposed NDOT damage classification, this damage was classified as Severe II class.

Figure 76: Level of damage, Scottsbluff Gering Bypass Bridge, courtesy of NDOT

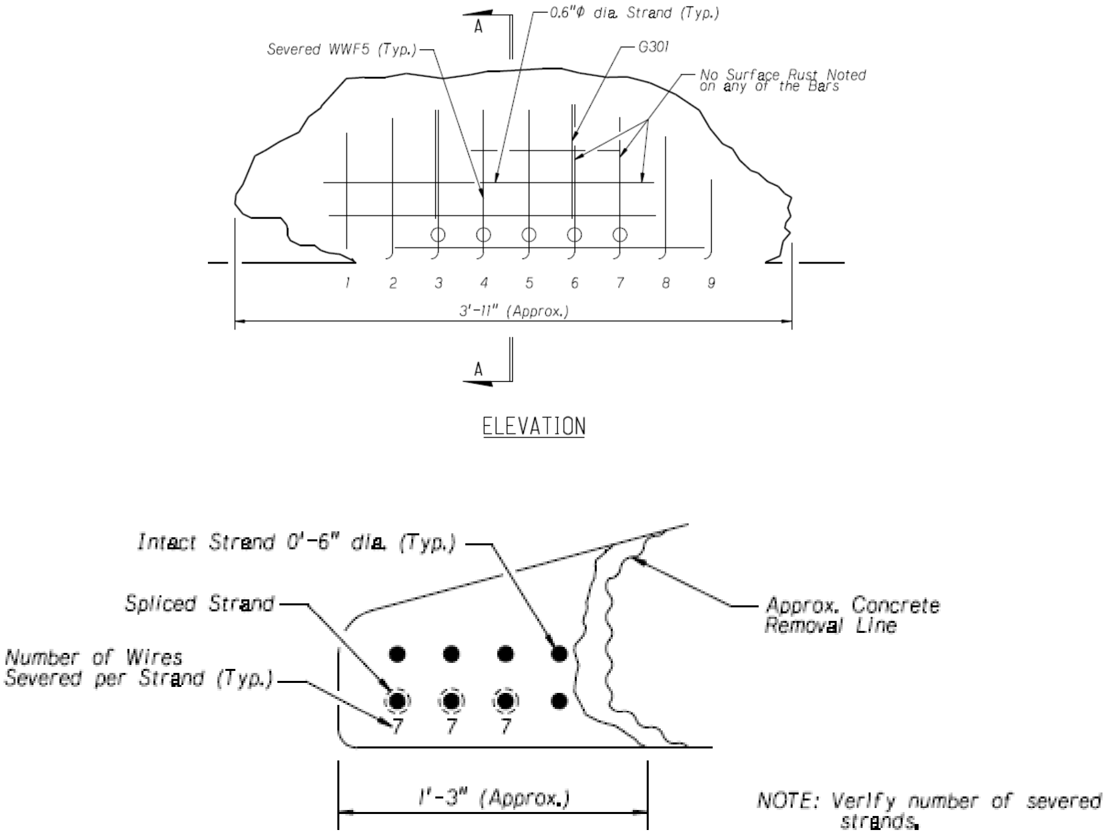


Figure 77: Severed strands and transverse reinforcement, from the repair documents of Scottsbluff Gering Bypass Bridge, courtesy of NDOT

Strand Splicing

The repair procedure included splicing the three severed strands. The strand splice system was GRABB- IT Cable Splice, a product of Prestress Supply Inc, Florida. Strands were spliced and tightened with an approved and calibrated torque wrench to a tension force of 31,000 lbs in each ½" diameter 270LL strand. That force present 75% of the strand capacity in tension $f_{pu.}$

Prior to the actual installation of the splice system, a mock-up installation was performed with a calibrated torque wrench by a 3-person work crew to test and demonstrate that the system can be installed to the satisfactory of the Engineer. Splices were staggered to provide adequate bonding space for patch material around and between the splice components Anode devices were installed to prevent future corrosion of the exposed reinforcement.

Patching and Epoxy Injection

Pre-loading was done by loading the bridge over the damaged girder with a 40 kip truck prior to placing patching material. Epoxy injection was used to seal all cracks (greater than 0.01 in.) in damaged girders as part of concrete girder repair. NDOT approved epoxy injection material was also used.

For the welded wire reinforcement all broken WWF5 fabric is to be straightened and lapped with D4 wires, at least 6 inches of overlap shall be provided between old and new wires. All exposed steel surfaces were sandblasted and cleaned to remove any loose material.

Surface preparation for patching was done by achieving at least 1/8 inch roughness to bond the patching material to the existing concrete surface. Areas contaminated with any oil leaks were thoroughly cleaned with an approved detergent or shall be removed to the necessary depth. Patching was done using cementitious repair material compatible with galvanic corrosion protection. Patching material was placed 1/8 inch thicker than the existing surface level and cured then ground to level the surface.

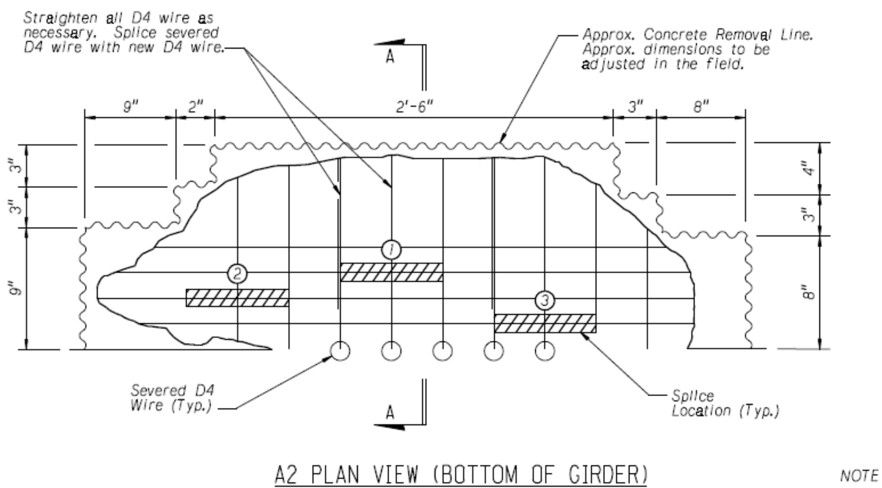


Figure 78: Staggering strand splices, from the repair documents of Scottsbluff Gering Bypass Bridge, courtesy of NDOT

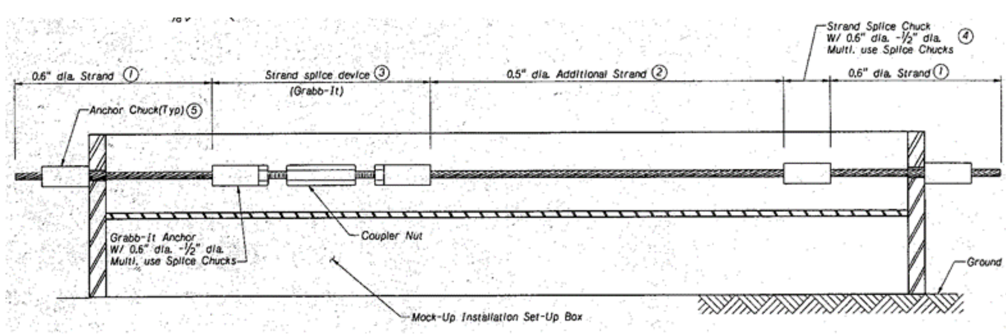


Figure 79: Mock-up installation before strand splicing, from the repair documents of Scottsbluff Gering Bypass Bridge, courtesy of NDOT

Repair works took place on April 2009. The latest inspection done on May 2018 reported the superstructure NBI condition rate to be 9 with no notable deficiencies on the repaired girder.

Wood River Interchange Bridge

The bridge was located in Hall County. The bridge was constructed in 2003. The bridge structural system consisted of 6 prestressed NU1100 girders, with two continuous span lengths of 145 feet as shown in Figure 80. The bridge had a skew angle of 20^0. The girders at which damage occurred had a total of 58 strands distributed on six rows as shown in Figure 81.

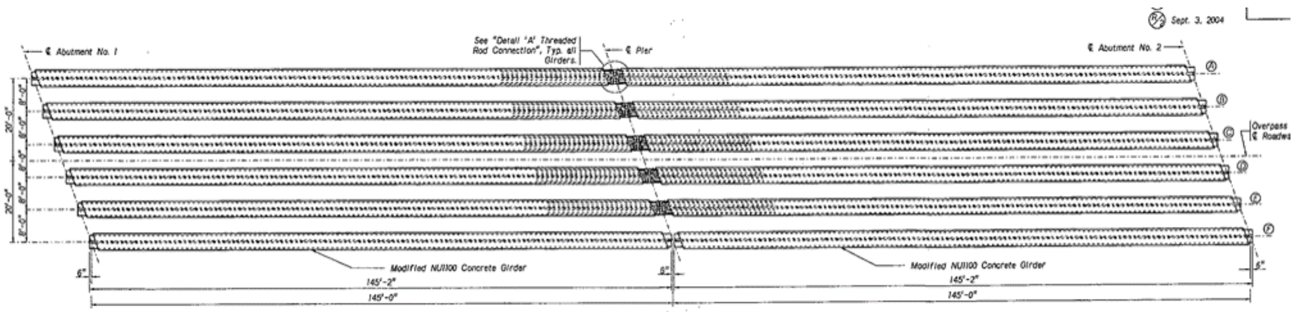


Figure 80: Plan view from the construction documents of Wood River Interchange bridge system, courtesy of NDOT

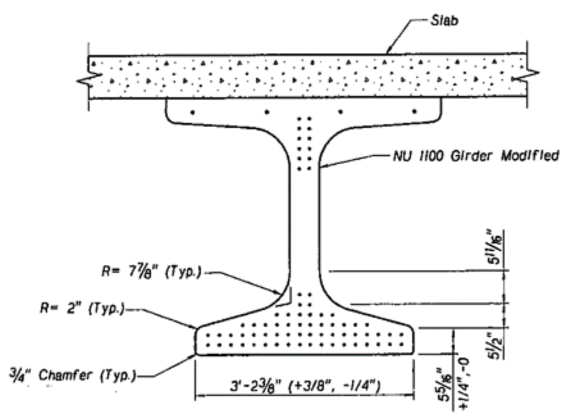


Figure 81: Girder reinforcement and strand distribution, from the construction documents of Wood River Interchange Bridge, courtesy of NDOT

With reference to the accident report two girders were damaged:

Girder A

Six strands were completely exposed, and three strands were partially exposed as shown in Figure 82. Four wires from one strand were severed. No damage was found on the other strands.

In order to determine if there is any loss of camber, girder soffit elevations were measured near the ends and middle of the span. An approximate camber of +1 inch was measured at the undamaged girders. Camber measured at Girder A was +0.06 inch. According to the proposed NDOT damage classification, this damage was classified as Severe I class.

Girder E

Eleven strands were completely exposed, and one strand was partially exposed as shown in Figure 83. Three strands were completely severed. Three strands had 2 to 4 wires cut.

In order to determine if there is any loss of camber, girder soffit elevations were measured near the ends and middle of the span. An approximate camber of +1 inch was measured at the undamaged girders. Camber measured at Girder E was -0.48 inch. According to the proposed NDOT damage classification, this damage was classified as Severe II class.

Figure 82: Girder (A) level of damage, Wood River Interchange Bridge, courtesy of NDOT

Figure 83: Girder (E) level of damage, Wood River Interchange Bridge, courtesy of NDOT

No strand splicing was done at girder A as shown in Figure 84. Three strands in Girder E were spliced as shown in Figure 85. Same notes, equipment, and repair procedure as Scottsbluff bridge were followed.

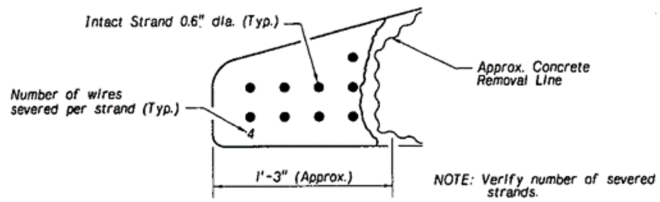


Figure 84: Severed strand in Girder A, from the repair documents of Wood River Interchange Bridge, courtesy of NDOT

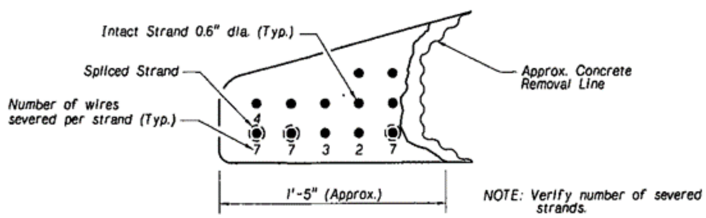


Figure 85: Severed strands in Girder E, from the repair documents of Wood River Interchange Bridge, courtesy of NDOT

Repair works took place on April 2009. The latest inspection done on August 2017 reported the superstructure NBI condition rate to be 5 with noting that the previous patch was damaged again. A visual inspection visit was done on July 2019. For Girder E, it was noticed that the patched area was damaged with a significant volume of the patching material removed leaving the bottom flange transverse reinforcement exposed as shown in Figure 86. In addition, it was noticed that Girder E had less camber than the neighboring girders. For Girder A there were no notable deficiencies on the repaired girder as shown in Figure 87.

Figure 86: Damage at the patched area on Girder E, Wood River Interchange Bridge, courtesy of NDOT

Figure 87: Patched area on Girder A, Wood River Interchange Bridge, courtesy of NDOT

York East Bridge

The bridge was located in York County. The bridge was constructed around the year 1965. The bridge structural system consisted of 3 prestressed AASHTO Type III girders, with four simple span lengths of 59.5, 69.5, 69.5, and 59.5 feet as shown in Figure 88. The girders at which damage occurred had a total of 26 strands distributed on three rows.

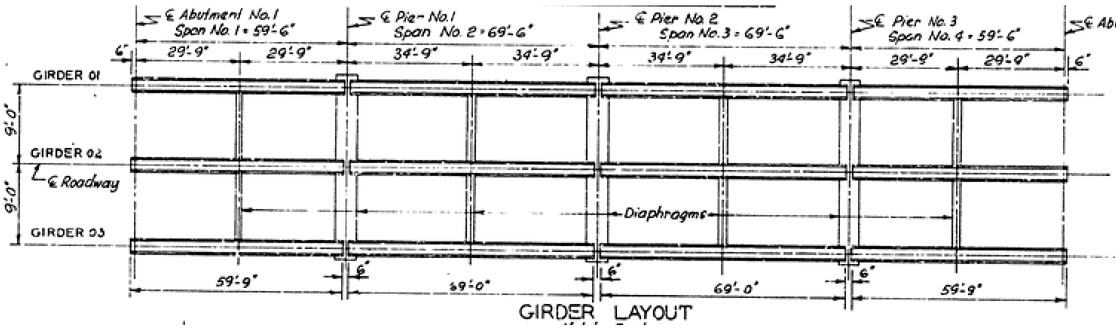


Figure 88: Plan view from the construction documents of York East bridge system, courtesy of NDOT

With reference to the accident report, over 40 feet of the exterior girder has been damaged with numerous cracks and spalls as shown in Figure 89. No evidence of any severed strands. No damage was observed on the deck. All loose materials were immediately removed for the safety of I-80 traffic as shown in Figure 90. According to the proposed NDOT damage classification, this damage was classified as Severe II class. However, the bridge girder was replaced since the length of damage was over half of the span length.

Figure 89: Level of damage, York East Bridge, courtesy of NDOT

Figure 90: Removing all loose concrete immediately from the girder, York East Bridge, courtesy of NDOT

Girder replacement was done by first cutting the deck at two sections spaced at 2.75 feet to have splicing rebar as shown in Figure 91. The same cutting procedure was done at the intermediate diaphragms as shown in Figure 92. The new girder and deck reinforcement were placed and spliced as shown in Figure 93. New intermediate diaphragms reinforcement was placed and spliced as shown in Figure 94.

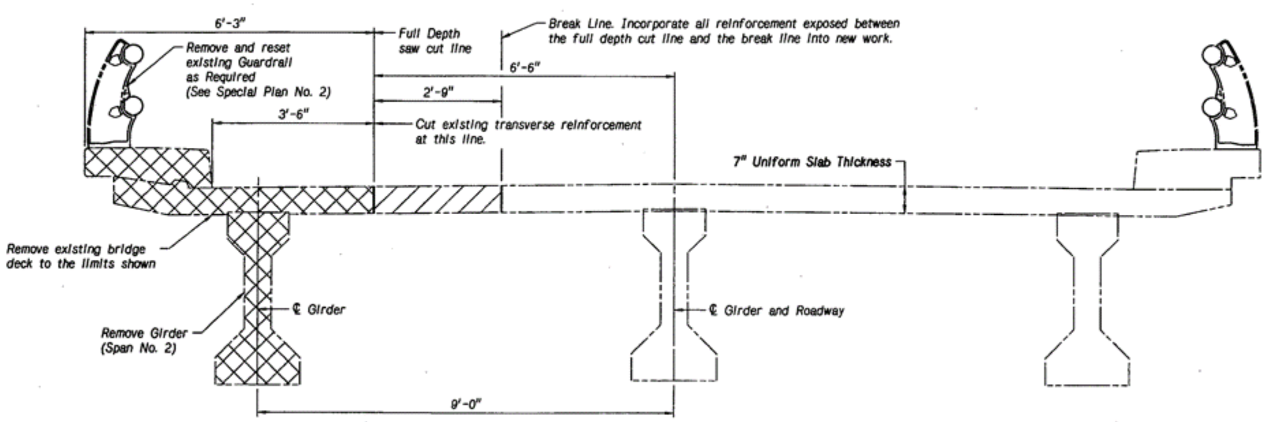


Figure 91: Girder replacement procedure, cross-section to be removed, from the replacement documents of York East Bridge, courtesy of NDOT

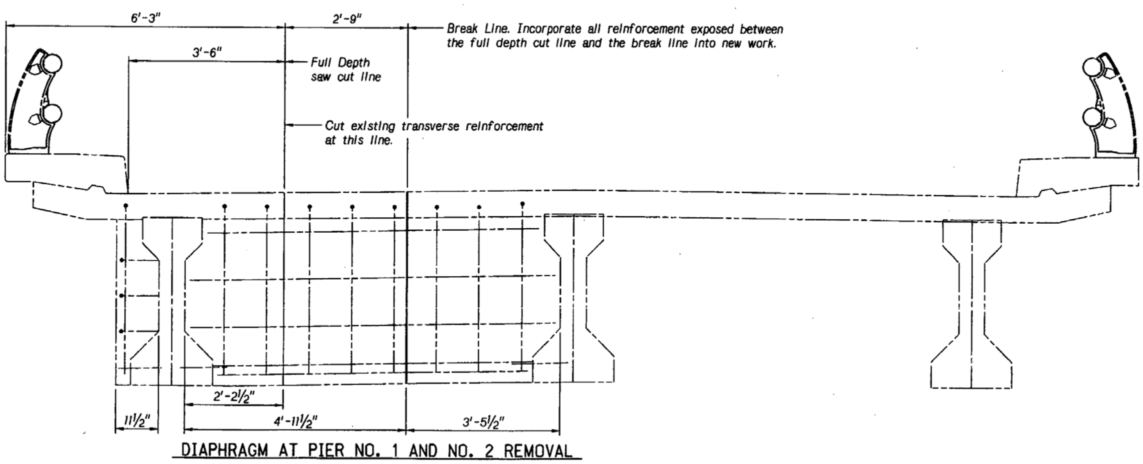


Figure 92: Girder replacement procedure, diaphragm section to be removed, from the replacement documents of York East Bridge, courtesy of NDOT

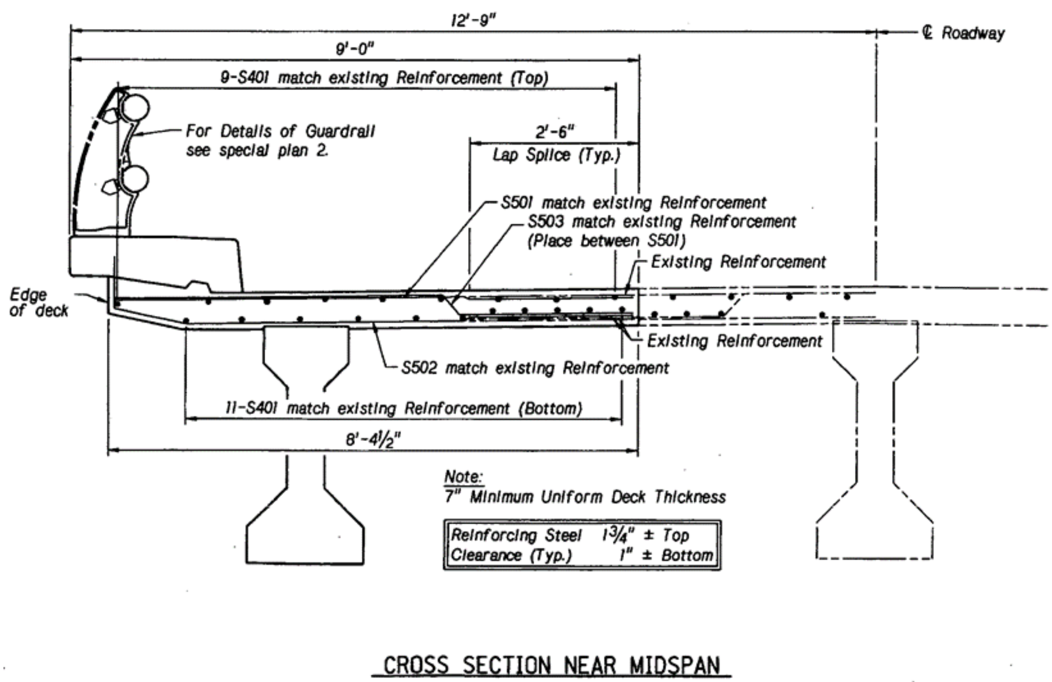


Figure 93: Girder replacement procedure, splicing deck reinforcement, from the replacement documents of York East Bridge, courtesy of NDOT

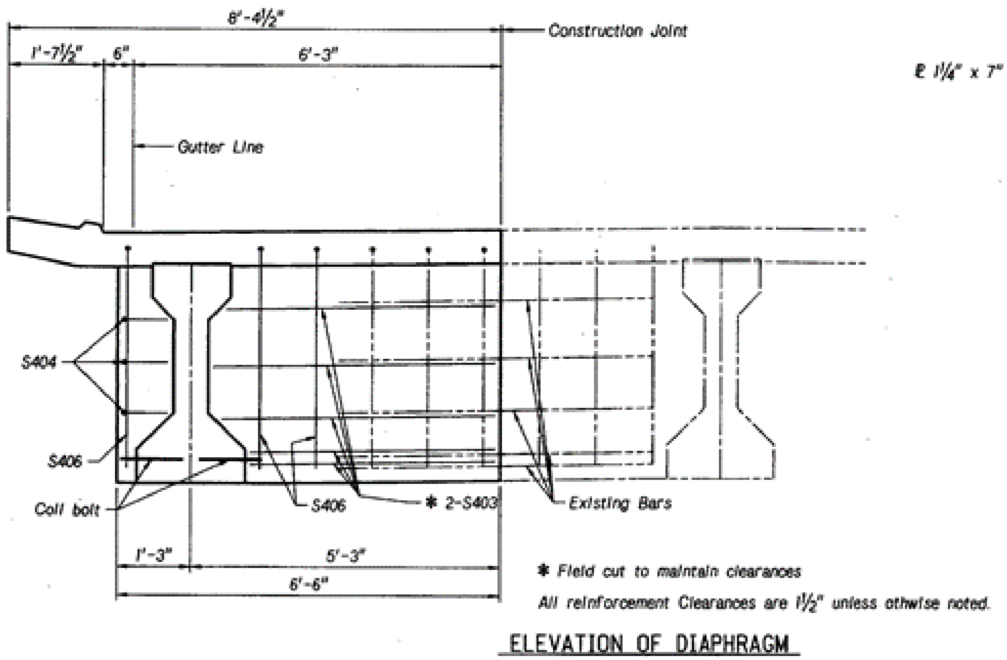


Figure 94: Girder replacement procedure, splicing diaphragm reinforcement, from the replacement documents of York East Bridge, courtesy of NDOT

Replacement works took place on July 2012. The latest inspection done on December 2018 reported the superstructure NBI condition rate to be 7 noting minor impact damage to the replaced girder and one other girder with no exposed reinforcement or strands. A visual inspection visit was done on July 2019 noting the minor impact damage as shown in Figure 95 with no notable deficiencies on the replaced girder as shown in Figure 96.

Figure 95: Impact damage at the replaced girder, York East Bridge, courtesy of NDOT

Figure 96: Replaced girder, York East Bridge, courtesy of NDOT

I-680 over US-75 Bridge

The bridge was located in Douglas County. The bridge was constructed around the year 1998. The bridge structural system consisted of twin systems of 5 prestressed AASHTO type II girders for each system. The bridge had four simple span lengths of 34, 47, 47, and 34 feet. The bridge had a skew angle of 4.50. The girders at which damage occurred had a total of 12 strands distributed on three rows.

Damage photos show one girder severely damaged with all strands exposed and a large portion of the web spalled as shown in Figure 97. Adjacent girders suffered from shallow spalls. According to the proposed NDOT damage classification, this damage was classified as Severe III class.

Figure 97: Level of damage, I-680 over US-75 Bridge, courtesy of NDOT

Repair/Replacement Procedure:

The severely damaged girder was replaced. The adjacent girders were repaired according to special provisions as shown in Figure 98. Girder replacement was done by first cutting the deck at two sections spaced at 2.75 feet to have splicing rebar as shown in Figure 99. The new girder and deck reinforcement were placed and spliced as shown in Figure 100. New intermediate diaphragms reinforcement was placed and spliced as shown in Figure 101.

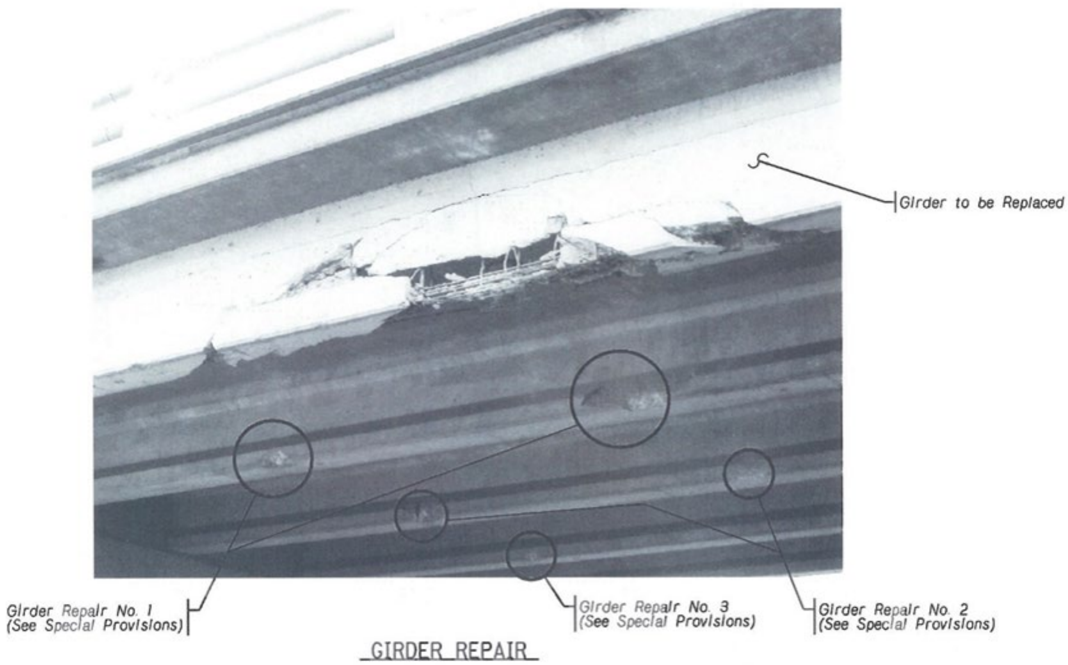


Figure 98: Girders to be repaired or replaced, from the repair documents of I-680 over US-75 Bridge, courtesy of NDOT

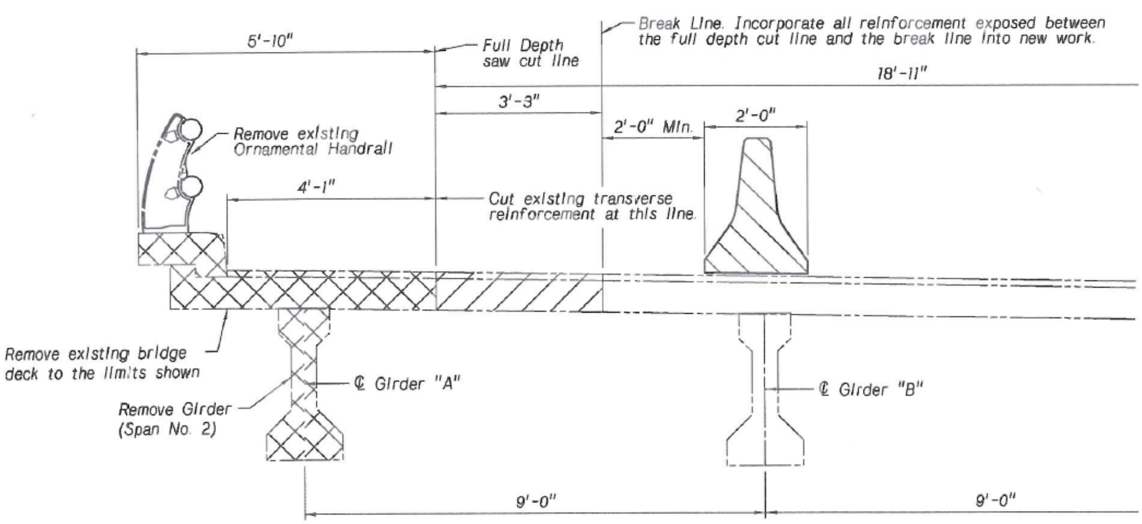


Figure 99: Girder replacement procedure, cross-section to be removed, from the repair documents of I-680 over US-75 Bridge, courtesy of NDOT

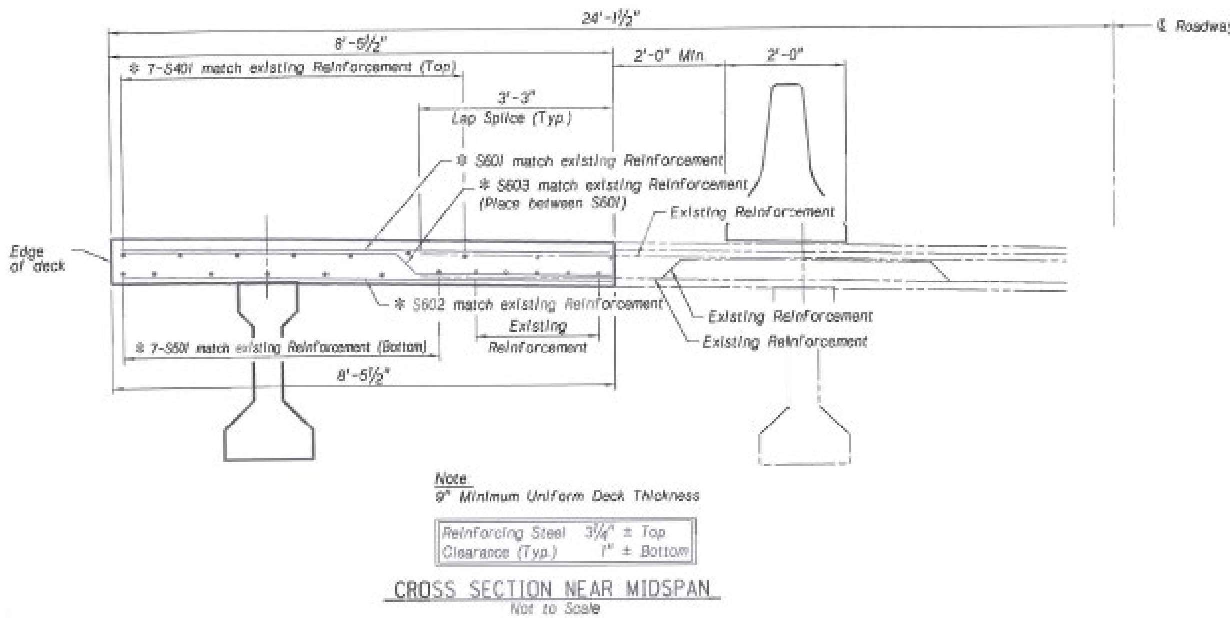


Figure 100: Girder replacement procedure, splicing deck reinforcement, from the repair documents of I-680 over US-75 Bridge, courtesy of NDOT

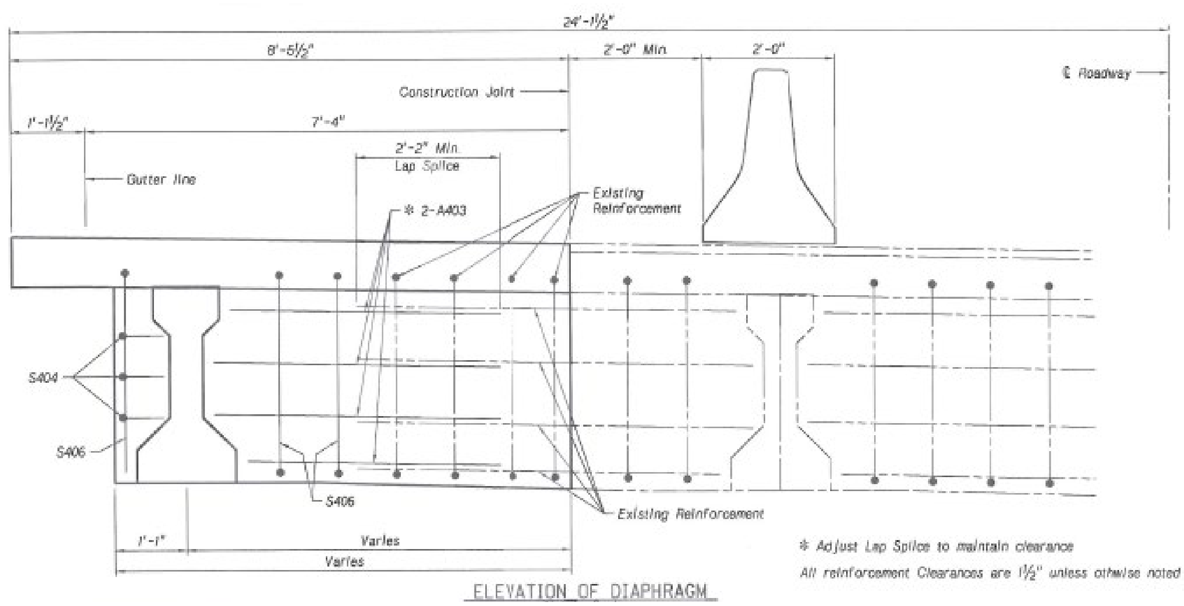


Figure 101: Girder replacement procedure, splicing diaphragm reinforcement, from the repair documents of I-680 over US-75 Bridge, courtesy of NDOT

Repair and replacement works took place on August 2012. The latest inspection done on March 2019 reported the superstructure NBI condition rate to be 6 with no notable deficiencies on the replaced girder or the other girder with patching and epoxy injection works. A visual inspection visit was done on August 2019 with no notable deficiencies on the replaced girder or the girders with patching and epoxy injection works as shown in Figure 102 and Figure 103.

Figure 102: Girders with patching and epoxy injection works, I-680 over US-75 Bridge, courtesy of NDOT

Figure 103: Replaced girder and girders with patching and epoxy injection works, I-680 over US-75 Bridge, courtesy of NDOT

Appendix B - Previous NDOT Repair Cases for Girder End Damage

Platte River Bridge East of Grand Island

The bridge was located in Hall County. The bridge construction took place around the year 1969. The bridge structural system consisted of 6 prestressed AASHTO type III girders. The bridge consists of four continuous structures of 3, 4, 4, and 3 spans with three expansion joints as shown in Figure 104. Span lengths were 65 feet ± 6 inches.

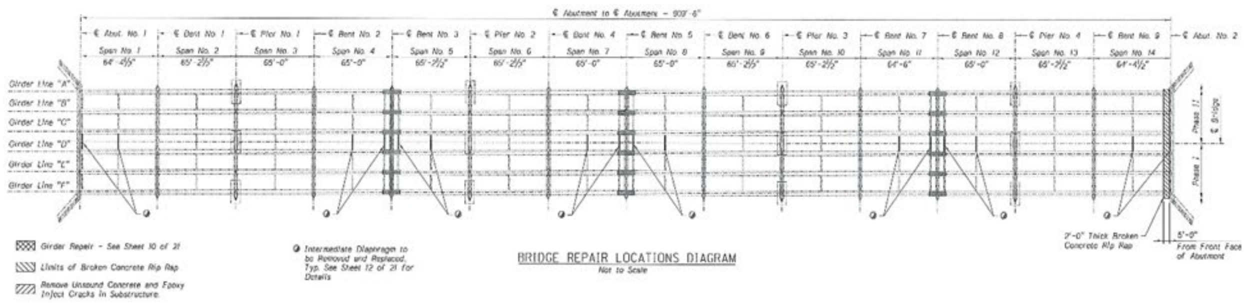


Figure 104: Girder spans and location of repair works, from the repair documents of Platte River Bridge East of Grand Island, courtesy of NDOT

The bridge suffered from corrosion damage at girder ends, abutments, bearing assemblies, and piers as shown in Figures 105 through 107. All loose materials were removed, and exposed reinforcement or prestressing was sandblasted as shown in Figure 108. Cracks were epoxy injected, then a bonding agent was applied and the defected areas were patched. Girder ends were confined with CFRP wraps as shown in Figure 109 and Figure 110. Girders were supported on temporary supports while repairs took place. Girders were not allowed to rest on bearings prior to 1-day cure of the CFRP system. Bearing assemblies were replaced as shown in Figure 111 and Figure 112.

Repair works took place on April 2015. Before the repair works the superstructure NBI condition rate was 6 noting girder ends spalling at piers, and bearings continuing to corrode (with most under exterior girders). The latest inspection done on December 2018 reported the superstructure NBI condition rate to be 7 noting that the structure has been rehabbed and with no notable deficiencies on the repaired girders.

Figure 105: Corrosion damage at girder end and abutment, Platte River Bridge East of Grand Island, courtesy of NDOT

Figure 106: Corrosion damage at bearing assemble, Platte River Bridge East of Grand Island, courtesy of NDOT

Figure 107: Corrosion damage at bearing assemble, Platte River Bridge East of Grand Island, courtesy of NDOT

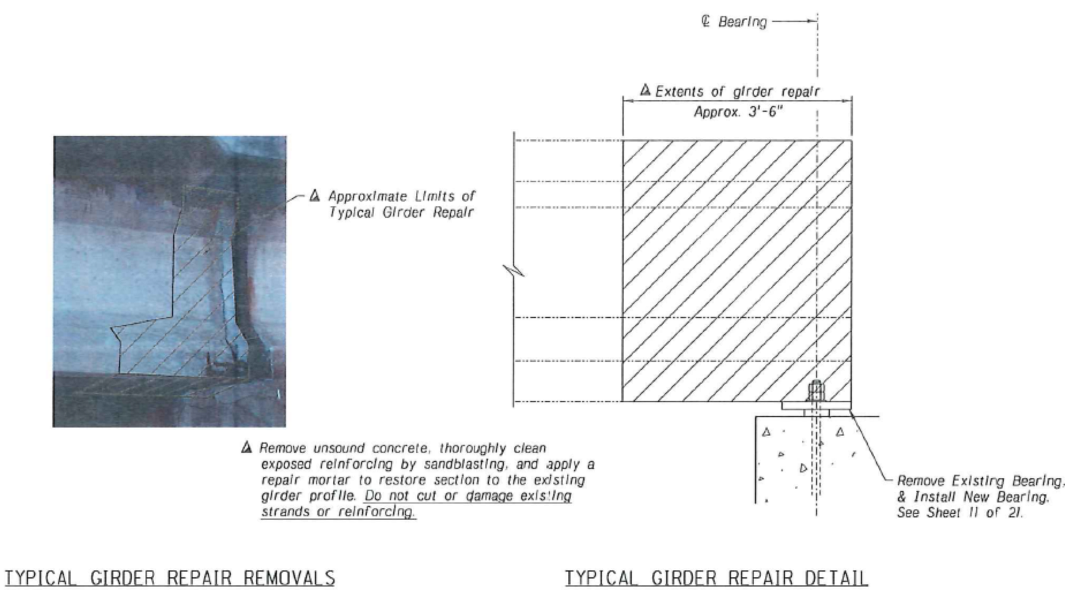


Figure 108: Removing loose materials and cleaning exposed reinforcement at girder end, from the repair documents of Platte River Bridge East of Grand Island, courtesy of NDOT

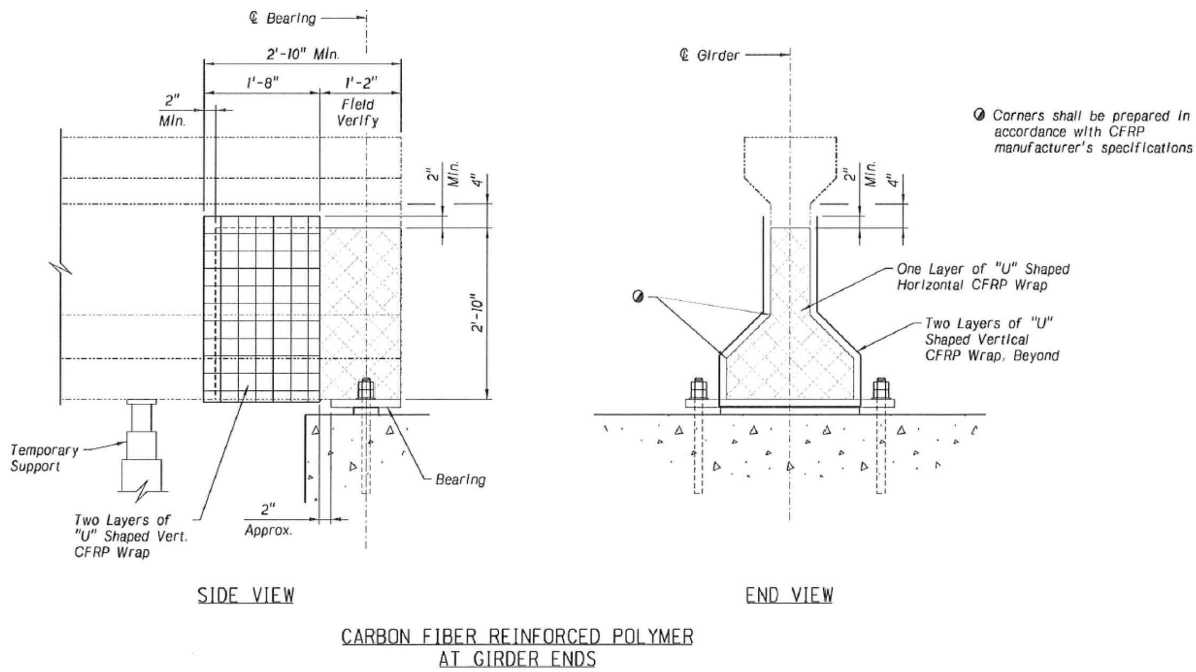


Figure 109: CFRP confinement for the patch at girder end, from the repair documents of Platte River Bridge East of Grand Island, courtesy of NDOT

Figure 110: Girder end after CFRP confinement and bearing assemble replacement, Platte River Bridge East of Grand Island, courtesy of NDOT

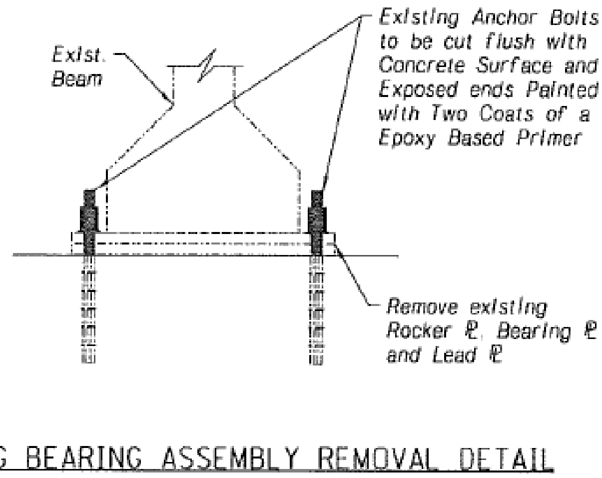


Figure 111: Existing bearing assembly removal detail, from the repair documents of Platte River Bridge East of Grand Island, courtesy of NDOT

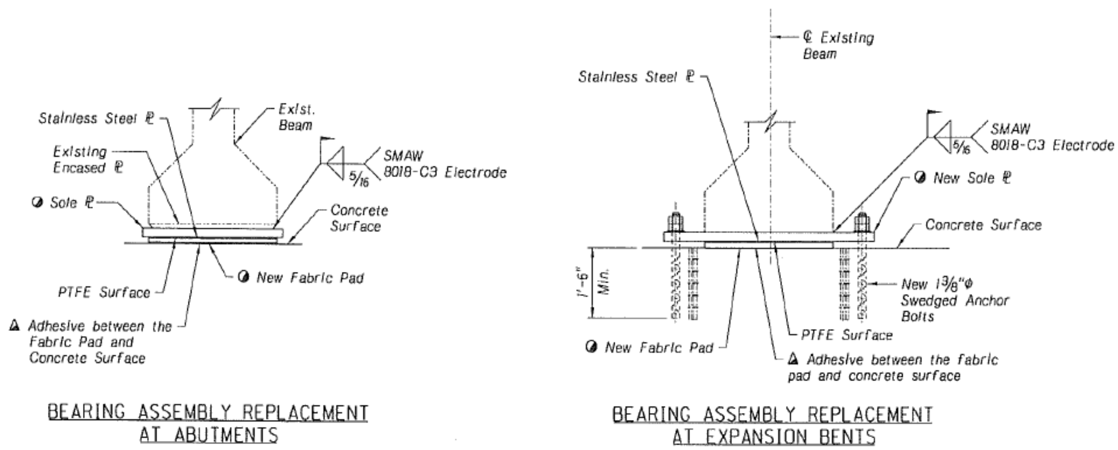


Figure 112: Bearing assembly replacement at girder ends, from the repair documents of Platte River Bridge East of Grand Island, courtesy of NDOT

Kearney South Platte River Bridge

The bridge was located in Buffalo County. The bridge construction took place around the year 1973. The bridge structural system consisted of 6 prestressed AASHTO type III girders. The bridge consists of four continuous structures of 3, 4, 4, and 3 spans with three expansion joints as shown in Figure 113. Span lengths were 65 feet ± 6 inches. Figures 115 through 121 show details and photos of the girder end repair works.

The bridge had repair works at girder ends on two occasions on January 2009 and May 2015. The superstructure NBI condition rate was 6 before and after repair works. Before the 2009 repair works, girder ends were cracking at several locations and the bearing plates were slipping out and need to be reset. The defected girder ends were repaired and encased in concrete and new roadway expansion devices were installed over the piers at the location of damaged girder ends.

In August 2013 inspection it was noted that the repaired concrete blocking around girder ends at expansion joints are spalling at ends in several locations, and the bearing plates are still slipping out and need to be reset, and one bearing was missing. In the 2015 repair, all the defected repair locations at girder ends were encapsulated in additional concrete, and the bearings were replaced and bearing plates were added.

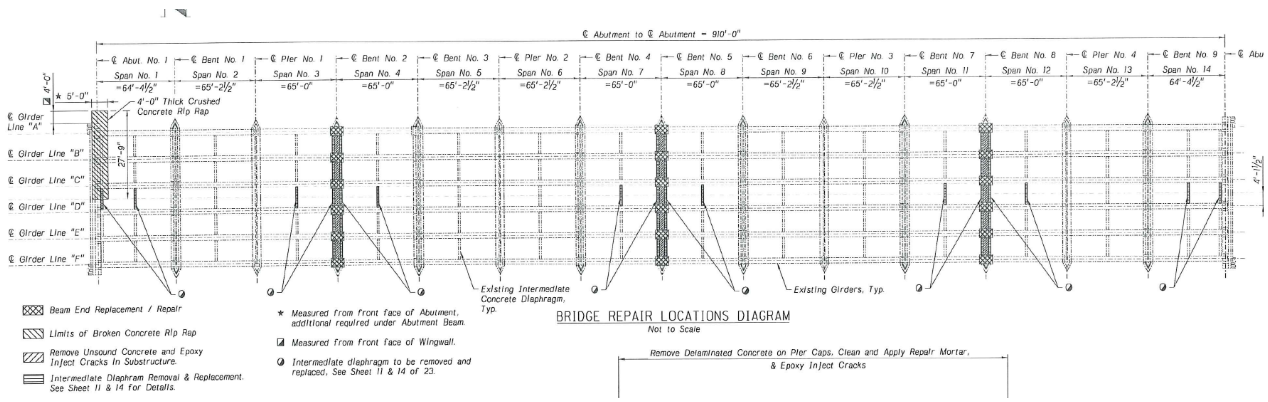


Figure 113: Girder spans and location of repair works, from the repair documents of Kearney South Platte River Bridge, courtesy of NDOT

Figure 114: Damage at girder end, Kearney South Platte River Bridge, courtesy of NDOT

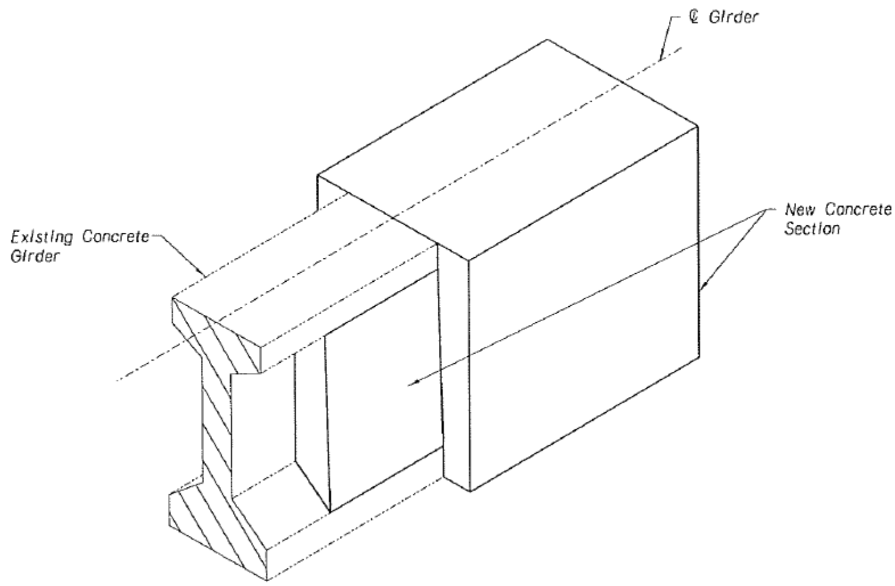


Figure 115: Isometric detail of girder end encasement on the 2015 repair works, from the repair documents of Kearney South Platte River Bridge, courtesy of NDOT

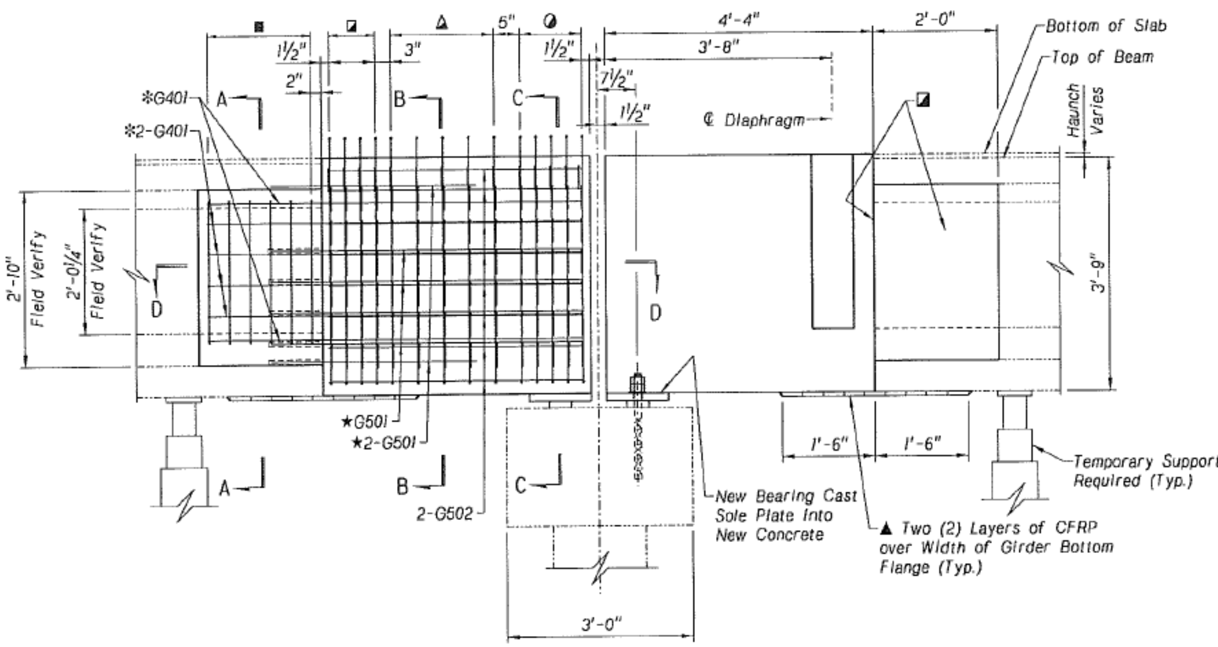


Figure 116: Detail of girder end encasement and CFRP layers on the 2015 repair works, from the repair documents of Kearney South Platte River Bridge, courtesy of NDOT

Figure 117: Girder end encasement and CFRP layers, Kearney South Platte River Bridge, courtesy of NDOT

Figure 118: Girder end encasement and CFRP layers, visible crack at the diaphragm, Kearney South Platte River Bridge, courtesy of NDOT

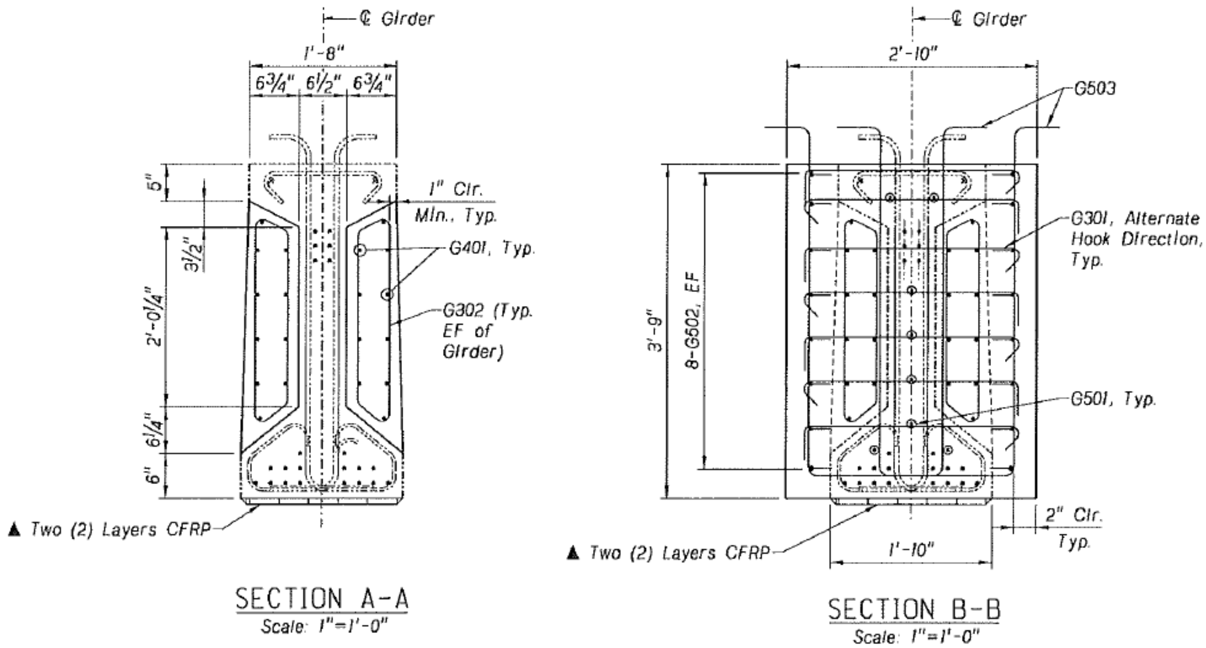


Figure 119: Detail of girder end encasement reinforcement and CFRP layers, from the repair documents of Kearney South Platte River Bridge, courtesy of NDOT

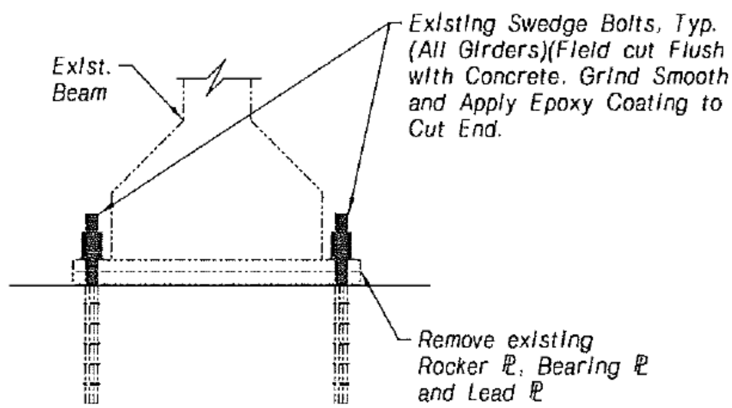


Figure 120: Detail of bearing assemble removal, from the repair documents of Kearney South Platte River Bridge, courtesy of NDOT

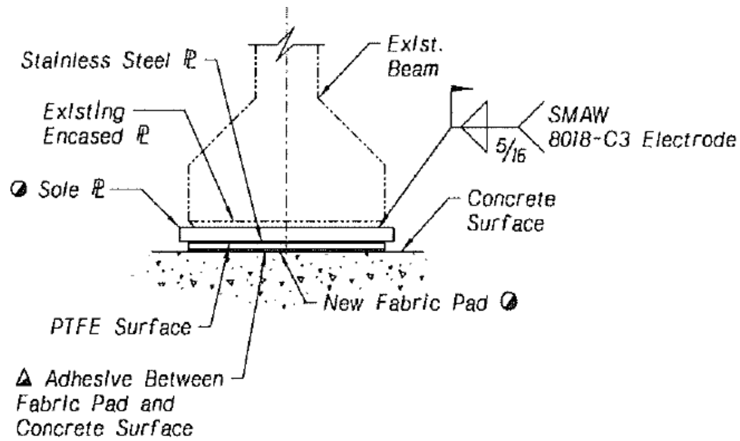


Figure 121: Detail of bearing assemble replacement, from the repair documents of Kearney South Platte River Bridge, courtesy of NDOT

Platte River South Bridge

The bridge was located in Dawson County. The bridge construction took place around the year 1963. The bridge structural system consisted of 4 prestressed AASHTO girders. The bridge consists of four continuous structures of 3, 4, 4, and 3 spans with three expansion joints and with span lengths of 68 feet ± 6 inches. Figures 122 through 126 show damage at girder ends and diaphragms.

The latest inspection done on October 2017 reported the superstructure NBI condition rate to be 4. Previous inspection records are noting cracking and spalling at girder ends at deck joints with a specific girder having an excessive amount of spalling in the bottom flange and web exposing reinforcement and the girder is losing the bearing.

Repair works are planned to be executed. Figure 127 and Figure 128 details are planned for the girder end repair and bearing assembly replacement.

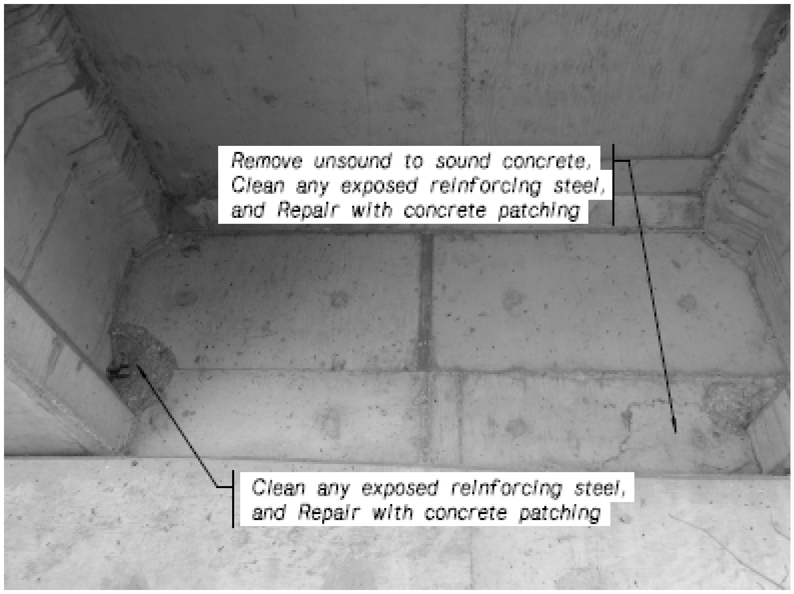


Figure 122: Damage at girder ends and diaphragm, Platte River South Bridge, courtesy of NDOT

Figure 123: Damage at girder ends, Platte River South Bridge, courtesy of NDOT

Figure 124: Damage at girder ends, Platte River South Bridge, courtesy of NDOT

Figure 125: Damage at girder ends, Platte River South Bridge, courtesy of NDOT

Figure 126: Damage at girder ends, Platte River South Bridge, courtesy of NDOT

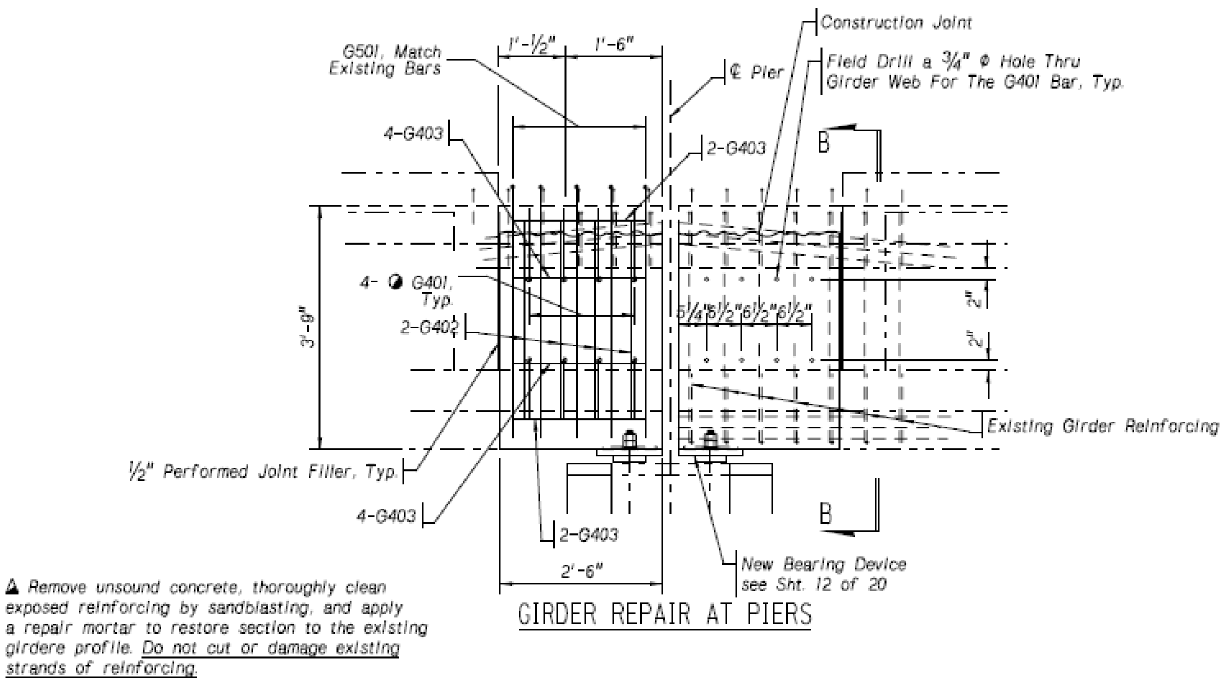


Figure 127: Girder end repair detail, from the repair documents of Platte River South Bridge, courtesy of NDOT

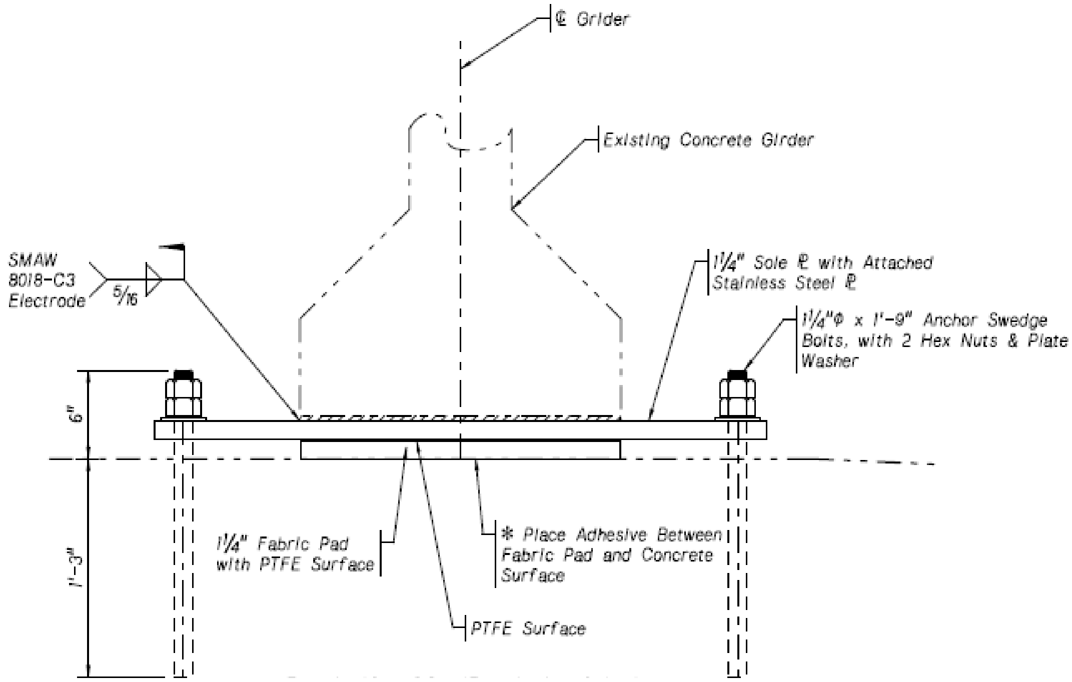


Figure 128: Bearing assemble replacement detail, from the repair documents of Platte River South Bridge, courtesy of NDOT

Alma South Bridge

The bridge was located in Harlan County. The bridge construction took place around the year 1950. The bridge structural system consisted of 7 prestressed AASHTO girders. The bridge consists of eight spans, span lengths were 79 feet, and 92 feet for the exterior and interior spans respectively.

Repair works took place on April 2017. The superstructure NBI condition rate was 6 before repair works and 7 after repair works. Inspection notes before repair works were that one girder had a 0.05" crack at girder end, another girder had a large crack and spall and exposed strands in the bottom flange at girder end. Also, two abutments had considerable spalling with areas of exposed reinforcement. Figures 129 through 132 show deteriorated abutment and girder ends, while Figure 133 shows the repaired girder end after having a concrete encasement.

Figure 129: Exposed reinforcement at the abutment, Alma South Bridge, courtesy of NDOT

Figure 130: Cracking at girder end, Alma South Bridge, courtesy of NDOT

Figure 131: Exposed strands at bottom flange, Alma South Bridge, courtesy of NDOT

Figure 132: Heavy cracking at girder end, Alma South Bridge, courtesy of NDOT

Figure 133: Repaired girder end with a concrete encasement, Alma South Bridge, courtesy of NDOT

Appendix C – Summary of Damage Classification and Repair Methods and Procedures for Over-height Vehicular Collision Damage

Proposed NDOT vehicular collision damage classification and repair methods

Damage Class	Description	Reference	Examples and Figures	Effect on Structural Capacity	Proposed Repair/Replacement Method
Minor	Concrete cracks, chips, and spalls up to 1.2 in. deep with no exposed reinforcing steel or prestressing strands. Concrete cracks are not observed from both sides of the girder.	Feldman et al., 1996. Alberta Infrastructure and Transportation, 2005. Iowa DOT, 2014.	Figure 1 Figure 2 Figure 3	No immediate effect on the structural capacity	Removal of loose materials, patching, and/or epoxy injection based on aesthetic needs
Moderate	Concrete cracks and wide spalls exposing reinforcing steel or prestressing strands but bars and strands remain undamaged.	Feldman et al., 1996. Alberta Infrastructure and Transportation, 2005. Iowa DOT, 2014.	Figure 4 Figure 5 Figure 6	No immediate effect on the structural capacity	Removal of loose materials, strand cleaning, patching and/or epoxy injection based on corrosion potential and aesthetic needs
Severe I	Any of the following: 1 or 2 strands damaged, or less than 5% of the total number of strands Loss of vertical camber but no downward deflection	Harries et al., 2009 Harries et al., 2012	Figure 7 Figure 8 Figure 9	Loss in live load capacity up to 5%. Loss in ultimate load capacity up to 8%.	FRP wrapping, steel jacket, or strand splicing to satisfy strength limit state, combined with patching and/or epoxy injection.
Severe II	Any of the following: 3 to 8 strands damaged, or greater than 5% and less that 20% of the total number of strands Vertical downward deflection but less than 0.3% of girder length	Harries et al., 2009 Harries et al., 2012	Figure 10 Figure 11 Figure 12	Loss in live load capacity up to 30%. Loss in ultimate load capacity up to 15%.	Strand splicing or external post-tensioning to satisfy service limit state in addition to strength limit state, combined with patching and/or epoxy injection.
Severe III	Any of the following: More than 8 strands damaged, or more than 20% of the total number of strands Vertical downward deflection exceeding 0.3% of girder length Lateral deformation exceeding construction tolerance Damage extending beyond bottom flange and lower half of web	Harries et al., 2009 Harries et al., 2012 Iowa DOT, 2014	Figure 13 Figure 14 Figure 15	Loss in live load capacity up to 100% Loss in ultimate load capacity up to 100%	Girder replacement

Figure 1: Minor damage, bottom flange spalling, Iowa DOT, 2014

Figure 2: Minor damage, NDOT York Bridge East

Figure 3: Minor damage, bottom flange cracks, Feldman et al., 1996

Figure 4: Moderate damage, exposed intact strands, Iowa DOT, 2014

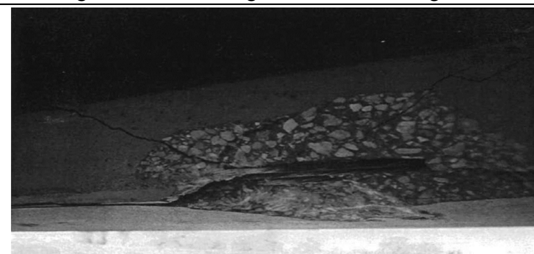


Figure 5: Moderate damage, exposed intact strands, Feldman et al., 1996

Figure 6: Moderate damage, exposed intact strands, Pantelides et al., 2010

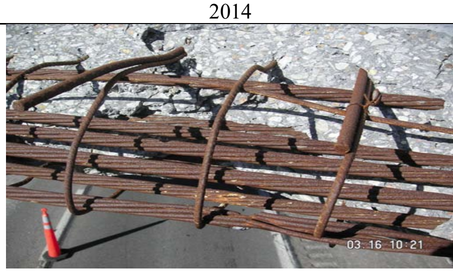


Figure 7: Severe I damage, one severed strand, NDOT Wood River Interchange Bridge Girder (A)

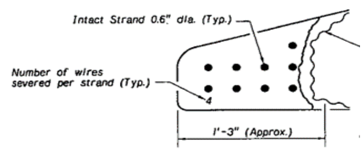


Figure 8: Severe I damage, one severed strand and loss of vertical camber, camber of +1 in. at undamaged girders and +0.06 in. at Girder (A), NDOT Wood River Interchange Bridge Girder (A)

Figure 9: Severe I damage, one severed strand, Harries et al., 2012

Figure 10: Severe II damage, five severed strands, NDOT Wood River Interchange Bridge Girder (E)

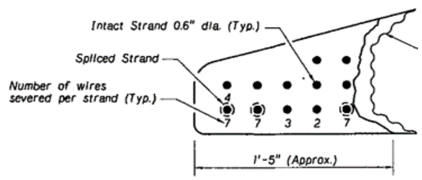


Figure 11: Severe II damage, five severed strands and vertical downward deflection, camber of +1 in. at undamaged girders and -0.48 in. at Girder (E), NDOT Wood River Interchange Bridge Girder (E)

Figure 12: Severe II damage, three severed strands, NDOT Scottsbluff Gering Bypass Bridge

Figure 13: Severe III damage, several damaged strands, NDOT Schuyler Bridge

Figure 14: Severe III damage, several damaged strands, Iowa DOT, 2014

Figure 15: Severe III damage, several damaged strands, Harries et al., 2012

Minor Damage	Moderate Damage	Severe I	Severe II	Severe III
1. Remove unsound concrete **2.** Epoxy injection if required (according to suggested procedure) **3.** Prepare base concrete surface (surface should be cleaned, then pre-wetted, then apply bonding agent) **4.** Place new concrete (following patching with trowel-applied or poured mortar suggested procedure) **5.** Address aesthetic treatment if required	**1.** Remove unsound concrete **2.** Epoxy injection if required (according to suggested procedure) **3.** Clean exposed strands or bars (by sandblasting) **4.** Preload girder, if required **5.** Place new reinforcement as needed (making sure it is properly lapped, anchored, or mechanically attached to the existing steel) **6.** Address future corrosion protection (if required) **7.** Prepare base concrete surface (as in Minor damage) **8.** Assemble the forms **9.** Place new concrete (following patching with recasting with new concrete suggested procedure) **10.** Apply FRP wrapping (if required, according to suggested procedure) **11.** Address aesthetic treatment if required	**1.** Restrict vehicle loads on the affected girder by directing traffic to the far side of the bridge until structural review is performed **2.** Follow same procedure as moderate damage **3.** Apply FRP wrapping (if required, according to structural calculations and suggested procedure)	**1.** Restrict vehicle loads on the affected girder by directing traffic to the far side of the bridge until structural review is performed **2.** Strand splicing or external post tensioning (according to suggested procedure) **3.** Follow same procedure as moderate damage **4.** Apply FRP wrapping (if required, according to structural calculations and suggested procedure)	Replace Girder

Figure 16: Epoxy injection ports, AIT, 2005

Figure 17: Patching steel formwork fabricated for girder restoration, AIT, 2005

Figure 18: Pumping concrete into plywood formed section, AIT, 2005

Figure 19: Shotcrete patch, Harries et al., 2012

Figure 20: Patching plywood form to restore cross-section, AIT, 2005

Figure 21: Epoxy injection process done in I-680 over US-75 Bridge

Figure 22: FRP wrapping, CFRP confinement of patch, Harries et al., 2012

Figure 23: FRP Wrapped Repair, Iowa Dot, 2014

Figure 24: CFRP fabrics installation, Choo et al., 2013

Figure 25: Strand splicing, Jones, 2017

Figure 26: Strand splicing, dial gauges and devices to monitor elongations and strand force, AIT, 2005

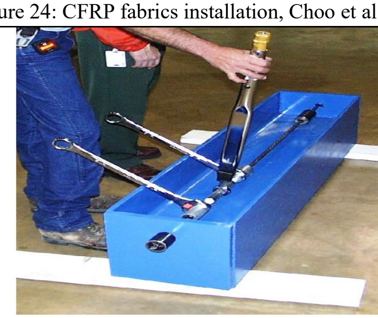


Figure 27: Strand splicing, mock-up to test the splices before installation, Baishya et al., 2010

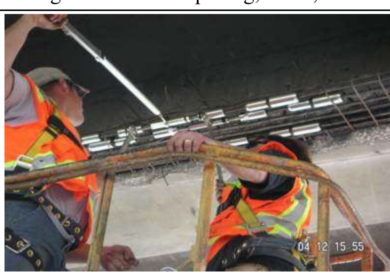


Figure 28: Strand splicing with a torque wrench, Enchayan, 2010

Figure 29: External post-tensioning end block, AIT, 2005

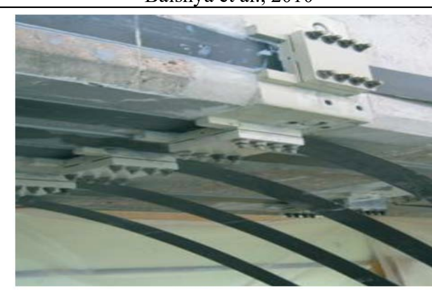


Figure 30: Sika carbon stress system external CFRP post-tensioning, Kasan, 2009

Appendix D – Summary of Damage Classification and Repair Methods and Procedures for Girder Ends Damage

Proposed NDOT girder end damage classification and Repair Methods

	Minor	Moderate	Extensive	Severe
Percent of Exposed Strands	0%	1-5%	6-15%	> 15%
Exposed Strands Surface Condition	None	Surface corrosion without pitting	Surface corrosion with light pitting	Heavy pitting, loss of strand area
Spalling	Isolated spalls less than 1" deep and less than 6" in diameter	Spall greater than 1" and less than 2" deep and less than 6" in diameter	Spall greater than 2" and less than 4" deep or greater than 6" diameter	Spalling deeper than 4"
Bearings Condition	Light corrosion not causing restriction	Light corrosion not causing restriction	Corrosion causing restriction	Failed Bearings
Joints Condition	No leakage	Minor dripping through the joint. Free movement still allowed	More than a drip and less than free flow of water. Partial movement allowed	Free flow of water through the joint. Movement restrained
Scale, Abrasion, or Wear	Coarse aggregate is exposed but remains secure	Coarse aggregate is loose or has popped out	Coarse aggregate is loose or has popped out	Severe voiding (concrete unsound).
Cracking	Hairline cracks narrower than 0.004"	Hairline or map cracks 0.004" to 0.03" wide	Cracks wider than 0.03" or heavy map cracking. Initiation of shear/flexure cracks	Severe cracks or fractures. Wide shear or flexure cracks.
Proposed Repair/Replacement Method	Epoxy injection, and patching if required	Epoxy injection, patching, and FRP wrapping if required	Epoxy injection, patching, and FRP wrapping	Rehab or Replace girder
Example Figures	Figure 1	Figure 2 Figure 3 Figure 4	Figure 5 Figure 6 Figure 7	Figure 8 Figure 9 Figure 10 Figure 11 Figure 12
References	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006	FHWA NHI 12-049, 2012 PennDOT BMS2, 2018 MnDOT, 2019 Naito et al., 2006

Figure 1: Minor damage, surface spalling with no exposed reinforcment, Naito et al., 2006

Figure 2: Moderate damage, exposed strands having corrosion without pitting, Naito et al., 2006

Figure 3: Moderate damage, hairline cracks, WisDOT, 2018

Figure 4: Moderate damage, one exposed strand on concrete bridge girder, WisDOT, 2018

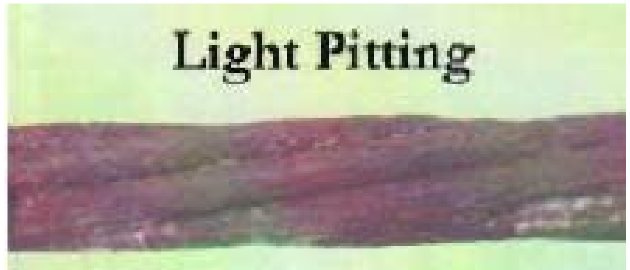


Figure 5: Extensive damage, exposed strands having corrosion with light pitting, Naito et al., 2006

Figure 6: Extensive damage, water saturation, rust staining, and spalling on a cast-in-place concrete T-girder, MnDOT, 2019

Figure 7: Extensive damage, delamination, and rust staining a precast concrete channel girder, MnDOT, 2019

Figure 8: Severe damage, exposed strands having corrosion with heavy pitting, Naito et al., 2006

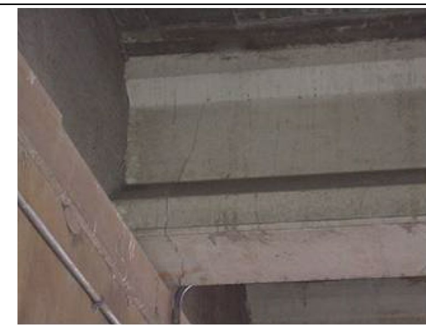


Figure 9: Severe damage, severe cracks, Choo et al., 2013

Figure 10: Severe damage, exposed strands > 15 %, MN/RC 2018-07 Report

Figure 11: Severe damage, exposed strands, NDOT Platte River South Bridge

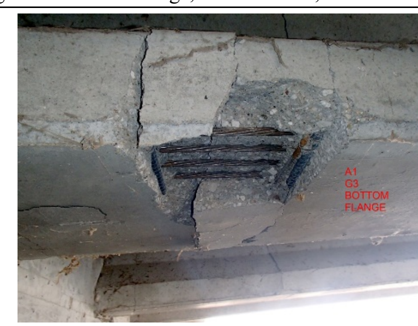


Figure 12: Severe damage, exposed strands, severe cracks or fractures, NDOT Alma South Bridge

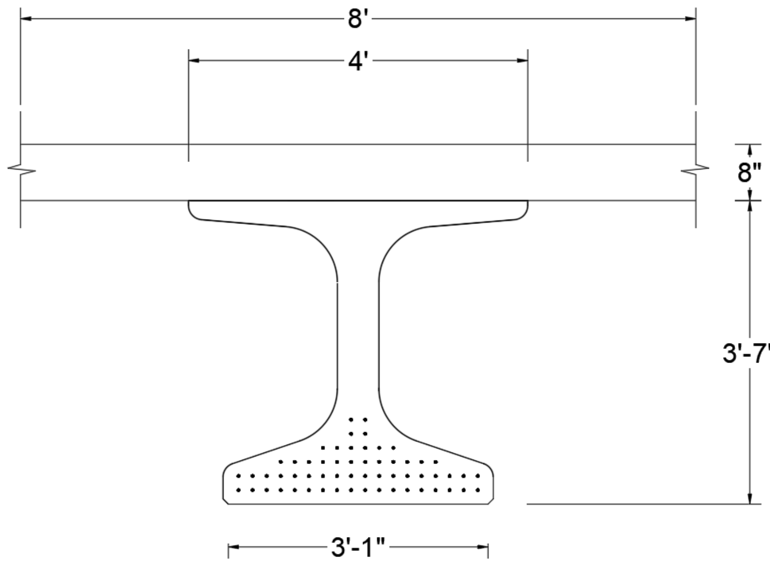
Minor Damage	Moderate Damage	Extensive Damage	Severe Damage
1. Remove the deteriorated and unsound concrete in steps. **2.** Epoxy inject any visible cracks. **3.** Address future corrosion protection (if required). **4.** Apply epoxy bonding agent to prepare the surfaces of the girder end. **5.** Place the new concrete. A non-shrink additive should be used in the new concrete. **6.** Check for possible distress in the repaired area.	**1.** Remove the deteriorated and unsound concrete in steps. **2.** Epoxy inject any visible cracks not within the removal limits. **3.** Clean exposed strands or bars by sandblasting. **4.** Place new reinforcement as needed, making sure it is properly lapped, anchored, or mechanically attached to the existing steel. **5.** Address future corrosion protection (recommended) **6.** Apply epoxy bonding agent to prepare the surfaces of the girder end. **7.** Assemble the forms for patching with recasting new concrete (not required for shotecreting) **8.** Place the new concrete. A non-shrink additive should be used in the new concrete. **9.** Apply FRP wrapping (if required) according to structural calculations. **10.** Check for possible distress in the repaired area.	**1.** Restrict vehicle loads on the affected girder by directing traffic to the far side of the bridge until repairs on the girder end are complete. **2.** Determine if the existing substructure can be used to jack the bridge up or if a jacking bent will need to be constructed. The jacking supports and jacking procedure should be reviewed by an engineer before any lifting begins. **3.** Place jacks and raise the entire end of the bridge. The lift should only be enough to take the load off and to allow a piece of sheet metal to be inserted on the girder seat as a bond breaker for the new concrete. **4.** Sawcut the concrete edges in a stepped fashion to avoid feathered edges and to provide bearing surfaces for the new concrete. **5.** Follow steps 1 through 8 in moderate damage (consider casting an end block or using UHPC) **6.** Replace bearing assemble **7.** Apply FRP wrapping according to structural calculations (recommended) **8.** Uniformly lower the end of the bridge. After the concrete has reached sufficient strength, and enough curing time is provided for the FRP adhesive. **9.** Check for possible distress in the repaired area. **10.** Remove the jacking system. **11.** Inspect leaking joints. Remove any debris inside the joint (manually by brushing, chipping and scraping, or by high pressure jet washing). Replace any loose or damaged joint seals. Replace or relocate entire joint if severely deteriorated.	Follow same procedure as Extensive Damage, or replace girder

Appendix E – Structural Calculations Design examples

Flexural Strength of a Prestressed Concrete Girder Section according to AASHTO LRFD, 2017

Note: The capacity of the girder should be calculated using the original design code at the time of construction, which can be different from the current AASHTO LRFD, 2017.

1- Material and Section Properties



Girder cross section

Girder Compressive Strength $f_c' := 8000\ psi$

Deck Compressive Strength $f_{cd}' := 4000\ psi$

Girder Gross Moment of Inertia $I_g := 182384\ in^4$

Girder Cross-sectional Area $A_g := 695\ in^2$

Girder Height $h_g := 43.31\ in$

Flange Width $w := 48.25\ in$

Effective Deck Width $w_d := 8\ ft$

Deck Thickness $t_d := 8\ in$

Total Section Area $A_c := A_g + w_d \cdot t_d = 1463\ in^2$

Neutral Axis Height from Bottom $y_b := 19.56\ in$ $y_t := h_g - y_b = 23.75\ in$

Prestressing MOE $E_p := 28500\ ksi$

Ultimate Strength $f_{pu} := 270\ ksi$

Low Relaxation Strands $k := 0.28$
(AASHTO LRFD, 2017 Table C5.6.3.1.1-1)

Number of Bottom Strands $n := 58$

Area of One Strand $A_{ps1} := 0.217\ in^2$

Area of Prestressing $A_{ps} := n \cdot A_{ps1} = 12.586\ in^2$

C.G. of Strands from Top of Girder $d_p := 38.69\ in$

Eccentricity of Presressing Force $e := d_p - y_t = 14.94\ in$

2- Calculations

Concrete Strain at Failure $\varepsilon_c := 0.003$
(AASHTO LRFD, 2017 Section 5.6.2)

Compression Stress Block Factors $\beta_1 := 0.65$
(AASHTO LRFD, 2017 Section 5.6.2.2)

$\alpha_1 := 0.85$

Neutral Axis Depth
(AASHTO LRFD, 2017 Section 5.6.3)

$$c := \frac{A_{ps} \cdot f_{pu}}{\alpha_1 \cdot f_{cd}' \cdot \beta_1 \cdot w_d + k \cdot A_{ps} \cdot \dfrac{f_{pu}}{d_p}} = 14.353\ in$$

Stress in Prestressing
(AASHTO LRFD, 2017 Section 5.6.3)

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) = 241.953\ ksi$$

Depth of Extreme Tension Steel $d_t := h_g - 2\ in = 41.31\ in$

Strain in Extreme Tension Steel $\varepsilon_t := \dfrac{d_t - c}{c} \cdot \varepsilon_c = 0.006$

Strength Reduction Factor (AASHTO LRFD, 2017, section 5.5.4.2)

$$\phi := min\left(1, \max\left(0.75, 0.75 + 0.25 \cdot \left(\frac{\varepsilon_t - 0.002}{0.005 - 0.002}\right)\right)\right) = 1$$

Nominal Flexural Resistance
(AASHTO LRFD, 2017 Section 5.6.3)

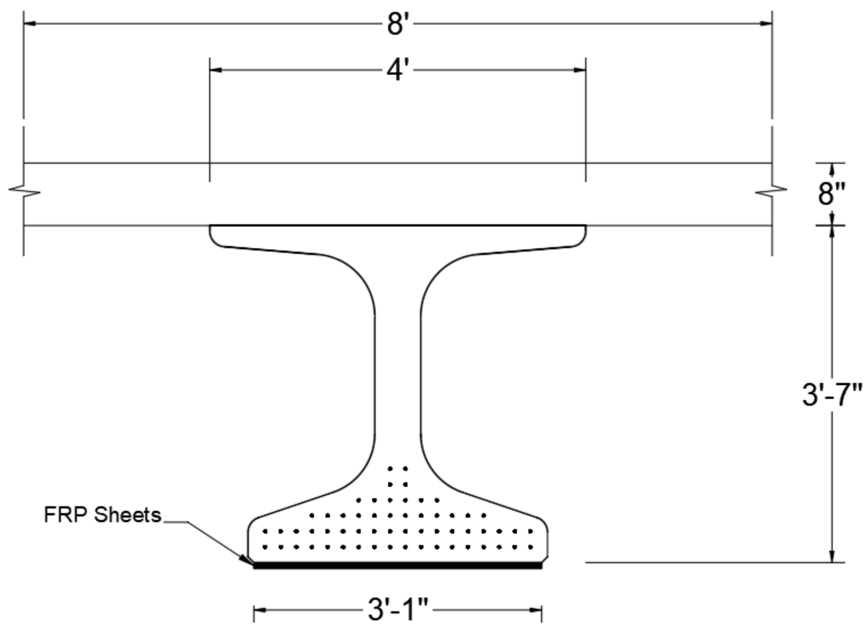
$$M_{np} := \begin{cases} A_{ps} \cdot f_{ps} \cdot \left(d_p - \dfrac{\beta_1 \cdot c}{2}\right) + \alpha_1 \cdot f_{cd}' \cdot w_d \cdot t_d \cdot \left(\dfrac{\beta_1 \cdot c}{2} - \dfrac{t_d}{2}\right) & \text{if } \beta_1 \cdot c \geq t_d \\ A_{ps} \cdot f_{ps} \cdot \left(d_p - \dfrac{\beta_1 \cdot c}{2}\right) & \text{if } \beta_1 \cdot c < t_d \end{cases} = 8779.2\ \boldsymbol{kip \cdot ft}$$

Flexural Strength of the Section $\phi M_n := \phi \cdot M_{np} = 8779.2\ \boldsymbol{kip \cdot ft}$

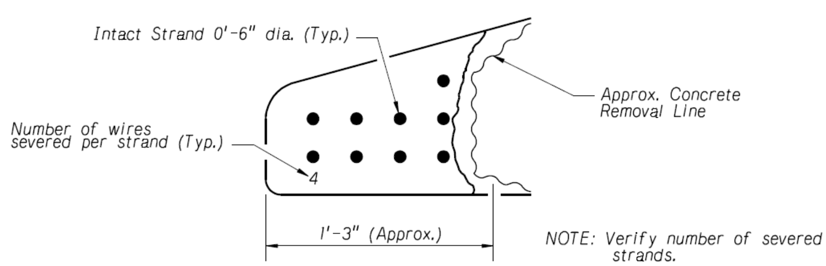
Flexural Strength of a Prestressed Concrete Girder Section Strengthed with FRP sheets according to AASHTO, 2012, ACI 440.2R-17, and Harries et al., 2009

Note: The capacity of the girder should be calculated using the original design code at the time of construction, which can be different from the current AASHTO LRFD, 2017.

1- Material and Section Properties



Girder and deck cross section view with FRP sheets location



Location of damaged strand

Environmental Reduction Factor (ACI 440.2R-17 Table 9.4) $C_e := 0.85$

Ultimate Tensile Strength of FRP $f_{fu}' := 90\ ksi$

Ultimate Rupture Strain of FRP $\varepsilon_{fu}' := 0.015\ \frac{in}{in}$

Design Ultimate Strength of FRP $f_{fu} := C_e \cdot f_{fu}' = 76.5\ \boldsymbol{ksi}$

Design Rupture Strain of FRP $\varepsilon_{fu} := C_e \cdot \varepsilon_{fu}' = 0.013$

Girder Compressive Strength $f_c' := 8000\ \boldsymbol{psi}$

Deck Compressive Strength $f_{cd}' := 4000\ \boldsymbol{psi}$

Concrete MOE
(AASHTO LRFD, 2017 Section 5.4.2.4)
$$E_c := 33000 \cdot 0.145^{1.5} \cdot \sqrt{\frac{f_c'}{\boldsymbol{ksi}}} \cdot \boldsymbol{ksi} = 5153.6\ \boldsymbol{ksi}$$

Girder Gross Moment of Inertia $I_g := 182384\ \boldsymbol{in}^4$

Girder Cross-sectional Area $A_g := 695\ \boldsymbol{in}^2$

Raduis of Gyration $r := \sqrt{\frac{I_g}{A_g}} = 16.199\ \boldsymbol{in}$

Girder Height $h_g := 43.31\ \boldsymbol{in}$

FRP width $w := 37\ \boldsymbol{in}$

Effective Deck Width $w_d := 8\ \boldsymbol{ft}$

Deck Thickness $t_d := 8\ \boldsymbol{in}$

Total Section Area $A_c := A_g + w_d \cdot t_d = 1463\ \boldsymbol{in}^2$

Neutral Axis Height from Bottom $y_b := 19.56\ \boldsymbol{in}$

$y_t := h_g - y_b = 23.75\ \boldsymbol{in}$

Number of FRP Plies $n_f := 2$

Nominal Thickness of 1 Ply FRP $t_f := 0.05\ \boldsymbol{in}$

FRP MOE $E_f := 5360\ \boldsymbol{ksi}$

FRP Area $A_f := n_f \cdot t_f \cdot w = 3.7\ \boldsymbol{in}^2$

2- Prestressing Properties

Prestressing MOE $E_p := 28500\ \boldsymbol{ksi}$

Ultimate Strength $f_{ys} := 270\ ksi$

Ultimate Strain $\varepsilon_{ys} := \dfrac{f_{ys}}{E_p} = 0.009$

Number of Bottom Strands $n := 57$

Area of One Strand $A_{ps1} := 0.217\ in^2$

Area of Prestressing $A_{ps} := n \cdot A_{ps1} = 12.369\ in^2$

Effective Prestressing Stress (Assuming 20% final losses) $f_{pe} := 162\ ksi$

Effective Prestressing Force $P_e := A_{ps} \cdot f_{pe} = 2003.8\ kip$

Effective Prestressing Strain $\varepsilon_{pe} := \dfrac{f_{pe}}{28500\ (ksi)} = 0.00568$

C.G. of Strands from Top of Girder $d_p := 38.69\ in$

Eccentricity of Presressing Force $e := d_p - y_t = 14.94\ in$

3- Strain Limits

Debonding Strain Limit (ACI 440.2R-17, equation 10.1.1) $\varepsilon_{fd} := 0.083 \cdot \sqrt{\dfrac{f_c'}{n_f \cdot E_f \cdot \dfrac{t_f}{in}}} = 0.0101$

$$\varepsilon_{fd} := min\left(\varepsilon_{fd}, 0.9\ \varepsilon_{fu}\right) = 0.0101$$

Girder Self Weight $w_g := A_g \cdot 0.15 \left(\dfrac{kip}{ft^3}\right) = 0.724\ \dfrac{kip}{ft}$

Deck Weight plus wearing surface $w_{SIDL} := t_d \cdot w_d \cdot 0.15 \left(\dfrac{kip}{ft^3}\right) + 0.3 \left(\dfrac{kip}{ft}\right) = 1.1\ \dfrac{kip}{ft}$

Span Length $l := 145\ ft$

AppliedDead Load Moment on Girder Section $M_d := \dfrac{\left(w_g + w_{SIDL}\right) \cdot l^2}{8} = 4793.6\ kip \cdot ft$

Initial Strain at Beam Soffit (Assuming no live load and no pre-load at FRP installation, (ACI 440.2R-17, section 10.3))

$$\varepsilon_{bi} := \frac{-P_e}{E_c \cdot A_g}\left(1+\frac{e \cdot y_b}{r^2}\right)+\frac{M_d \cdot y_b}{E_c \cdot I_g} = 1.464 \cdot 10^{-5}$$

4- Assumed Compression Block and Effective Strain at FRP

Assumed Compression Block Depth $c := 17.7\ in$

Effective Level of Strain in FRP (ACI 440.2R-17, section 10.3)

$$\varepsilon_{fe} := 0.003 \cdot \left(\frac{h_g + t_d - c}{c}\right) - \varepsilon_{bi} = 0.006$$

$$\textbf{check}_1 := \textbf{if}\left(\varepsilon_{fe} > \varepsilon_{fd}, \text{“Debonding Failure”}, \text{“Rupture Failure”}\right) = \text{“Rupture Failure”}$$

AASHTO Limit of Usable Strain at FRP/Concrete Interface (AASHTO, 2012 Section 3.2)

$$\varepsilon_{fe} := min\left(\varepsilon_{fe}, \varepsilon_{fd}, 0.005\right) = 0.005$$

5- Stress and Strain at Prestressing Steel

Net Strain in Prestressing Steel (ACI 440.2R-17, section 10.3)

$$\varepsilon_{pnet} := \left(\varepsilon_{fe} + \varepsilon_{bi}\right) \cdot \left(\frac{d_p - c}{h_g - c}\right) = 0.004$$

Strain in Prestressing Steel (ACI 440.2R-17, section 10.3)

$$\varepsilon_{ps} := \varepsilon_{pe} + \frac{P_e}{A_c \cdot E_c}\left(1 + \frac{e^2}{r^2}\right) + \varepsilon_{pnet} = 0.0103$$

$$\textbf{check}_2 := \textbf{if}\left(\varepsilon_{ps} < 0.035, \text{“OK”}, \text{“Reiterate”}\right) = \text{“OK”}$$

Stress in Prestressing Steel (ACI 440.2R-17, section 10.3)

$$f_{ps} := \begin{cases} E_p\ \varepsilon_{ps} & \text{if } \varepsilon_{ps} \le 0.0086 \\ \left(270 - \dfrac{0.04}{\varepsilon_{ps} - 0.007}\right) \cdot (ksi) & \text{if } \varepsilon_{ps} > 0.0086 \end{cases} = 257.8\ ksi$$

Effective Stress in FRP

$$f_{fe} := E_f \cdot \varepsilon_{fe} = 26.8\ ksi$$

6- Check Equilibruim of Forces

Concrete Strain at Failure

$$\varepsilon_c := \left(\varepsilon_{fe} + \varepsilon_{bi}\right) \cdot \left(\frac{c}{h_g + t_d - c}\right) = 0.0026$$

Strain Corresponding to fc'
(ACI 440.2R-17, table 16.5b)

$$\varepsilon_c' := 1.7 \frac{f_c'}{E_c} = 0.0026$$

Compression Stress Block Factors
(AASHTO LRFD, 2017 Section 5.6.2.2)

$$\beta_1 := 0.65$$

$$\alpha_1 := 0.85$$

$$c_{new} := \frac{A_{ps} \cdot f_{ps} + A_f \cdot f_{fe}}{\alpha_1 \cdot f_{cd}' \cdot \beta_1 \cdot w_d} = 15.499 \; \textit{in}$$

$$\textbf{check}_3 := \textbf{if} \left(0.97\ c < c_{new} < 1.03\ c, \text{“OK”}, \text{“Go to check5”}\right) = \text{“Go to check5”}$$

$$\textbf{check}_4 := \textbf{if} \left(c_{new} < t_d, \text{“Discard Ctf”}, \text{“Top Flange Compression”}\right) = \text{“Top Flange Compression”}$$

$$c_{tf} := \frac{A_{ps} \cdot f_{ps} + A_f \cdot f_{fe} - \alpha_1 \cdot f_{cd}' \cdot \beta_1 \cdot t_d \cdot w_d}{\alpha_1 \cdot f_c' \cdot \beta_1 \cdot w} = 9.728 \; \textit{in}$$

$$\textbf{check}_5 := \textbf{if} \left(0.97\ c < c_{tf} + t_d < 1.03\ c, \text{“OK”}, \text{“Reiterate using td+Ctf”}\right) = \text{“OK”}$$

7- Nominal Flexure Capacity

FRP Strength Reduction Factor
(ACI 440.2R-17, section 10.3)

$$\phi_{FRP} := min\left(0.9, \max\left(0.65, 0.65 + \frac{0.25 \cdot \left(\varepsilon_{ps} - 0.01\right)}{0.013 - 0.01}\right)\right) = 0.674$$

Recommended Additional Reduction Factor for FRP Contribution
(ACI 440.2R-17, section 11.3)

$$\psi := 0.85$$

Damaged Girder Strength Reduction Factor
(AASHTO LRFD, 2017, section 5.5.4.2)

$$\phi_{Damaged} := min\left(1, \max\left(0.75, 0.75 + 0.25 \cdot \left(\frac{\varepsilon_{ps} - 0.002}{0.005 - 0.002}\right)\right)\right) = 1$$

Prestressing Contribution to Capacity
(AASHTO LRFD, 2017, section 5.5.4.2)

$$M_{np} := \begin{cases} \text{if } \beta_1 \cdot c \ge t_d \\ \quad A_{ps} \cdot f_{ps} \cdot \left(d_p - \dfrac{\beta_1 \cdot c}{2}\right) + \alpha_1 \cdot f_{cd}{}' \cdot w_d \cdot t_d \cdot \left(\dfrac{\beta_1 \cdot c}{2} - \dfrac{t_d}{2}\right) \\ \text{if } \beta_1 \cdot c < t_d \\ \quad A_{ps} \cdot f_{ps} \cdot \left(d_p - \dfrac{\beta_1 \cdot c}{2}\right) \end{cases} = 9134.7 \; kip \cdot ft$$

FRP Contribution to Capacity
(ACI 440.2R-17, section 10.3)

$$M_{nf} := A_f \cdot f_{fe} \cdot \left(h_g - \frac{\beta_1 \cdot c}{2}\right) = 310.4 \; kip \cdot ft$$

Damaged Section Capacity $\phi M_n := \phi_{Damaged} \cdot M_{np} = 9134.7 \; kip \cdot ft$

Strengthed Section Capacity $\phi M_n := \left(\phi_{Damaged} \cdot M_{np} + \phi_{FRP} \cdot \psi \cdot M_{nf}\right) = 9312.4 \; kip \cdot ft$

8- Check Service Stress in FRP (ACI 440.2R-17, section 10.2)

Live Load Moment $M_{ll} := 5000 \; kip \cdot ft$

Service Moment $M_s := M_d + M_{ll} = 9793.6 \; kip \cdot ft$

$$f_{fs} := \left(\frac{E_f}{E_c}\right) \cdot \frac{M_s \cdot y_b}{I_g} - \varepsilon_{bi} \cdot E_f = 13.03 \; ksi$$

$$\mathbf{check_6} := \mathbf{if}\left(f_{fs} < 0.55 \cdot f_{fu}, \text{“OK”}, \text{“Increase FRP Area”}\right) = \text{“OK”}$$

9- Development Length of FRP System

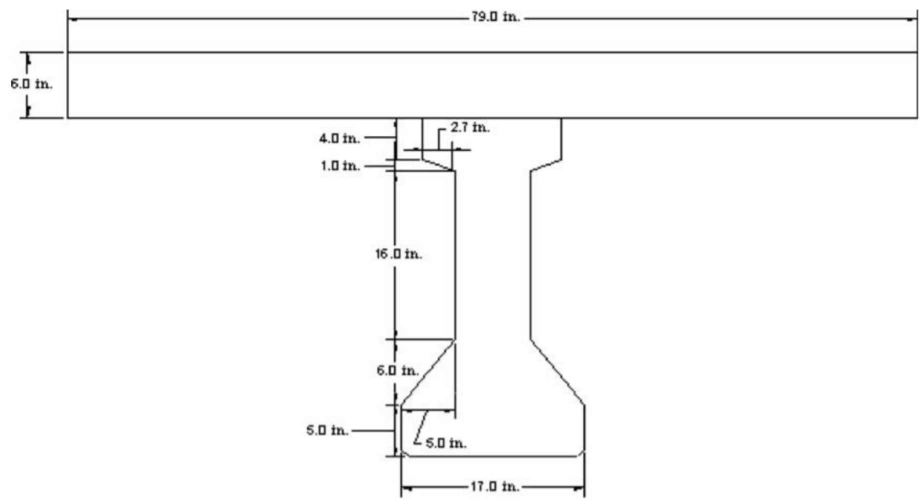
Development Length of FRP Sheets
(ACI 440.2R-17, section 14.1)

$$l_{df} := 0.057 \cdot \sqrt{\frac{n_f \cdot \left(\dfrac{E_f}{psi}\right) \cdot \left(\dfrac{t_f}{in}\right)}{\sqrt{\left(\dfrac{f_c{}'}{psi}\right)}}} \cdot (in) = 4.413 \; in$$

Shear Strength of a Prestressed Concrete Girder Section according to AASHTO LRFD, 2017, AASHTO, 2012, ACI 440.2R-17 and AFGC, 2013, and Belarbi et al, 2011

Note: The capacity of the girder should be calculated using the original design code at the time of construction, which can be different from the current AASHTO LRFD, 2017.

1- Material and Section Properties



Girder Compressive Strength $f_c' := 7000\ psi$

Girder Height $h_g := 32\ in$

Girder Cross-sectional Area $A_g := 334.7\ in^2$

Web Width $b_w := 7\ in$

Prestressing MOE $E_p := 28500\ ksi$

Prestressing Ultimate Strength $f_{pu} := 270\ ksi$

Assumed Value for Locked-in Stress in Prestressing Strands $f_{po} := 0.7 \cdot f_{pu} = 189\ ksi$

Number of Bottom Strands $n := 14$

Area of One Strand $A_{ps1} := 0.153\ in^2$

Area of Prestressing $A_{ps} := n \cdot A_{ps1} = 2.142\ in^2$

C.G. of Strands from Top of Girder $d_p := 28.8\ \textit{in}$

Shear Depth
(AASHTO LRFD, 2017 Section 5.7.2.8) $d_v := \max\left(0.9 \cdot d_p, 0.72\ h_g\right) = 25.92\ \textit{in}$

Transverse Reinforcement Area $A_v := 2 \cdot 0.11\ \textit{in}^2 = 0.22\ \textit{in}^2$

Transverse RFT Yield Strength $f_{yt} := 60\ \textit{ksi}$

Spacing of Transverse RFT
(at critical section) $S := 12\ \textit{in}$

Angle of Transverse RFT with
Longitudinal Axis $\alpha := 90\ \textit{deg}$

2- Factored Loads (Strength I)

$V_u := 100\ \textit{kip}$

$V_p := 0\ \textit{kip}$ Straight Strands

$M_u := 660\ \textit{ft} \cdot \textit{kip}$

$N_u := 0\ \textit{kip}$ -ve if Compression

3- Calculations

Longitudinal Tensile Strain (AASHTO LRFD, 2017 Section 5.7.3.4)

$$\varepsilon_s := \frac{\max\left(\frac{M_u}{d_v}, \left|V_u - V_p\right|\right) + 0.5\ N_u + \left|V_u - V_p\right| - A_{ps} \cdot f_{po}}{E_p \cdot A_{ps}} = 1.175 \cdot 10^{-5}$$

$$\varepsilon_s := \textbf{if}\left(\varepsilon_s < 0, 0, \varepsilon_s\right) = 1.175 \cdot 10^{-5}$$

Factor of Concrete Ability to Transmit Shear Forces
(AASHTO LRFD, 2017 Section 5.7.3.4) $\beta := \frac{4.8}{1 + 750 \cdot \varepsilon_s} = 4.758$

Angle of Diagonal Compression Strut
(AASHTO LRFD, 2017 Section 5.7.3.4) $\theta := 29 + 3500\ \varepsilon_s = 29.041$

Concrete Section Contribution to Shear
Resistance (AASHTO LRFD, 2017 Section
5.7.3.4) $V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c' \cdot \textit{ksi}} \cdot b_w \cdot d_v = 72.177\ \textit{kip}$

Transverse RFT Contribution to Shear Resistance (AASHTO LRFD, 2017 Section 5.7.3.4)

$$V_s := \frac{A_v \cdot f_{yt} \cdot d_v \cdot \left(\cot\left(\frac{\pi\ \theta}{180}\right) + \cot\left(\frac{\pi}{2}\right)\right) \cdot \sin\left(\frac{\pi}{2}\right)}{S} = 51.35\ \boldsymbol{kip}$$

Undamaged Girder Nominal Shear Resistance $\quad V_n := V_c + V_s + V_p = 123.5\ \boldsymbol{kip}$

Shear Strength Reduction Factor (AASHTO LRFD, 2017, section 5.5.4.2) $\quad \phi := 0.9$

AASHTO Ultimate Undamaged Girder Shear Resistance $\quad V_u := \phi \cdot V_n = 111.2\ \boldsymbol{kip}$

4- Using CFRP U-Wraps at Critical Section
(According to AASHTO, 2012, and ACI 440.2R-17)

Thickness of One Layer $\quad t_f := 0.0065\ \boldsymbol{in}$

CFRP Tensile Strength $\quad f_{fu} := 550\ \boldsymbol{ksi}$

Modulus of Elasticity $\quad E_f := 33000\ \boldsymbol{ksi}$

Ultimate Strain $\quad \varepsilon_{fu} := \frac{f_{fu}}{E_f} = 0.017$

Number of CFRP Plies $\quad n_f := 1$

Width of CFRP Sheets $\quad w_f := 4\ \boldsymbol{in}$

Center-to-center Spacing of CFRP Sheets $\quad S_f := 12\ \boldsymbol{in}$

Orientation of CFRP Sheets $\quad \alpha_f := 90\ \boldsymbol{deg}$

Effective Depth of CFRP Sheets $\quad d_f := 26\ \boldsymbol{in}$

Active Bonded Length (ACI 440.2R-17 Section 11.4.1)

$$L_e := \frac{2500\ \boldsymbol{in}}{\left(n \cdot \frac{t_f}{\boldsymbol{in}} \cdot \frac{E_f}{\boldsymbol{psi}}\right)^{0.58}} = 0.4\ \boldsymbol{in}$$

Bond Reduction Coefficients
(ACI 440.2R-17 Section 11.4.1)

$$k_1 := \left(\frac{f_c{}'}{4000.\ psi}\right)^{\frac{2}{3}} = 1.452$$

$$k_2 := \frac{d_f - L_e}{d_f} = 0.983$$

Bond Dependent Coefficient
(ACI 440.2R-17 Section 11.4.1)

$$k_v := min\left(\frac{k_1 \cdot k_2 \cdot L_e}{468 \cdot \varepsilon_{fu} \cdot in}, 0.75\right) = 0.08$$

Effective Strain in CFRP Layers
(ACI 440.2R-17 Section 11.4.1)

$$\varepsilon_{fe.ACI} := min\left(k_v \cdot \varepsilon_{fu}, 0.004, 0.75 \cdot \varepsilon_{fu}\right) = 0.001$$

FRP Shear RFT Area
(AASHTO, 2012 Section 4.3.2)

$$\rho_f := \frac{2 \cdot n_f \cdot t_f}{b_w} = 0.002$$

Reduction Factor
(AASHTO, 2012 Section 4.3.2)

$$R_f := min\left(\max\left(4 \cdot \left(\rho_f \cdot \frac{E_f}{ksi}\right)^{-0.67}, 0.088\right), 1\right) = 0.254$$

Effective Strain in CFRP Layers
(AASHTO, 2012 Section 4.3.2)

$$\varepsilon_{fe.AASHTO} := R_f \cdot \varepsilon_{fu} = 0.004$$

Design Effective Strain in CFRP Layers

$$\varepsilon_{fe} := min\left(\varepsilon_{fe.ACI}, \varepsilon_{fe.AASHTO}\right) = 0.001$$

Area of CFRP Wraps

$$A_{fv} := 2 \cdot n \cdot t_f \cdot w_f = 0.728\ in^2$$

Shear Resistance of CFRP Wraps
(AASHTO, 2012 Section 4.3.2, and
ACI 440.2R-17 Section 11.4.1)

$$V_f := \frac{A_{fv} \cdot \left(E_f \cdot \varepsilon_{fe}\right) \cdot \left(\sin(\alpha) + \cos(\alpha)\right) \cdot d_f}{S_f} = 69.471\ kip$$

Additional Reduction Factor for CFRP U-Wraps
(AASHTO, 2012 Section 4.3.1, and ACI 440.2R-17 Table 11.3)

$$\psi_f := 0.85$$

CFRP Strengthened Girder Shear Resistance
(Assuming Transverse Steel Reinforcement is
Completely Severed)

$$V_{uf} := \phi \cdot \left(V_c + \psi_f \cdot V_f\right) = 118.105\ kip$$

5- Using UHPC for Girder Web Area Repair
(According to AFGC, 2013)

UHPC Compressive Strength $f_{c.uhpc}' := 18\ \mathbf{ksi}$

AFGC Recommended Partial Safety Factors (AFGC, 2013 Section 6.2) $\gamma_{cf} := 1.3$ $\gamma_E := 1.15$

Level of Prestressing in the Section $\sigma_{cp} := \dfrac{A_{ps} \cdot f_{po}}{A_g} = 1.21\ \mathbf{ksi}$

Prestressing Factor
(AFGC, 2013 Section 6.2) $k := 3 \cdot \dfrac{\sigma_{cp}}{f_c'} = 0.518$

Concrete Section Contribution to Nominal Shear Resistance (AFGC, 2013 Section 6.2)

$$V_{Rd.c} := \frac{0.24}{\gamma_{cf} \cdot \gamma_E} \cdot k \cdot \sqrt{f_{c.uhpc}' \cdot \mathbf{MPa}} \cdot b_w \cdot d_v = 24.4\ \mathbf{kip}$$

Assumed post-cracking Residual Tensile Strength (according to lower bound suggested by Graybeal, 2006) $\sigma_{Rd.f} := 1.0\ \mathbf{ksi}$

Steel Fibers Contribution to Nominal Shear Resistance (AFGC, 2013 Section 6.2)

$$V_{Rd.f} := \frac{b_w \cdot d_v \cdot \sigma_{Rd.f}}{\tan\left(\frac{\pi \cdot \theta}{180}\right)} = 326.773\ \mathbf{kip}$$

Repaired Girder Nominal Shear Capacity $V_{n.R} := V_{Rd.c} + V_{Rd.f} = 351.169\ \mathbf{kip}$

Ultimate Repaired Girder Shear Resistance
(Assuming Transverse Steel Reinforcement is Completely Severed) $V_{u.R} := \phi \cdot V_{n.R} = 316.1\ \mathbf{kip}$

External Post-tensioning Design Example, according to Harries et al., 2009

1- Flexural Strength of Girder Before and After Damage

Undamaged Girder Flexural Strength $M_n := 4590\ kip \cdot ft$

Damaged Girder Flexural Strength $M_{n.d} := 3731\ kip \cdot ft$

2- Girder and Prestressing Properties Before and After Damage

Girder Compressive Strength $f_c' := 7\ ksi$

Girder Cross-sectional Area $A_g := 1272\ in^2$

Girder Bottom Section Modulus (Non-composite) $S_{nc} := 12212\ in^3$

Girder Bottom Section Modulus (Composite) $S_c := 64320\ in^3$

Deck Thickness $t_d := 6\ in$

Effective Deck Width $w_d := 30\ in$

Undamaged Effective Prestressing Force $P_{UD} := 721.4\ kip$

Damaged Effective Prestressing Force $P_D := 591.6\ kip$

Prestressing Eccentricity of Undamaged Section $e_{UD} := 26.8\ in$

Prestressing Eccentricity of Damaged Section $e_D := 26.1\ in$

Eccentricity of Post-tensioning System $e_{PT} := 11\ in$

Moment due to Deal Loads $M_{DL} := 1372\ kip \cdot ft$

3- Verify that The Damaged Girder Remains Uncracked under Dead Loads

$$Check := \text{if}\left(\frac{M_{DL}}{S_{nc}} < \left(\frac{P_D}{A_g} + \frac{P_D \cdot e_D}{S_{nc}}\right), \text{“OK”}, \text{“Post Tensioning is Not Recommended”}\right) = \text{“OK”}$$

4-Lost Stress at Girder Soffit

$$f_{loss} := \left(\frac{-P_{UD}}{A_g} - \frac{P_{UD} \cdot e_{UD}}{S_{nc}} + \frac{M_{DL}}{S_{nc}}\right) - \left(\frac{-P_D}{A_g} - \frac{P_D \cdot e_D}{S_{nc}} + \frac{M_{DL}}{S_{nc}}\right) = -0.421 \ \boldsymbol{ksi}$$

5- Required Post-tensioning Force

$$P_{PT} := \frac{f_{loss}}{\left(\frac{-1}{\left(A_g + t_d \cdot w_d\right)} - \frac{e_{PT}}{S_c}\right)} = 489.5 \ \boldsymbol{kip}$$

6- Check Allowable Compressive Stresses

Allowable Compressive Stress $\quad f_{c.all} := -0.6 \cdot f_c{}' = -4.2 \ \boldsymbol{ksi}$

Compressive Stresses at Girder Soffit (after PT) $\quad f_b := \left(\frac{-P_D}{A_g} - \frac{P_D \cdot e_D}{S_{nc}} + \frac{M_{DL}}{S_{nc}}\right) + \left(\frac{-P_{PT}}{\left(A_g + t_d \cdot w_d\right)} - \frac{P_{PT} \cdot e_{PT}}{S_c}\right) = -0.802 \ \boldsymbol{ksi}$

$Check := \textbf{if}\left(f_b > f_{c.all}, \text{“OK”}, \text{“Post Tensioning is Not Recommended”}\right) = \text{“OK”}$