Strength and Serviceability of Damaged Prestressed Girders

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This study addresses the assessment and repair of damaged prestressed bridge girders due to either 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. Both strength and serviceability limit states are considered. Procedures are developed to calculate changes in service stress due to damage based on a differential approach. The strength calculations are based on the strain compatibility approach. A software program, Prestressed Bridge Assessment, Repair, and Strengthening (PreBARS), has been developed in this study. The program calculates live and dead load forces, distribution factors, prestress losses (refined and approximate), serviceability and strength limit state load combinations, etc. for any point on a bridge with up to three continuous spans. The program also calculates sectional service stresses and strengths for undamaged, damaged and repaired conditions. Repairs may include patches, strand splices, and external Carbon Fiber Reinforced Polymer (CFRP) reinforcement. Application of preload during repair is included in the software. Using the PreBARS program, several case studies were conducted on two prestressed I-girder bridges. Different levels of loss of strands in the bottom flange (up to 25% loss) were simulated on both structures, and various repairs were applied in each case. Similarly, different levels of top flange damage were introduced in both structures up to 50% loss of the top flange. Various damage categories have been defined for both top and bottom flange damage cases. Detailed recommendations have been made regarding inspections, assessment, and repair of damaged prestressed girders for bottom and top damage scenarios.</td></tr>
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Executive Summary

This study addressed the assessment and repair of damaged prestressed bridge girders due to either accidental impact by over-height vehicles on the bottom flange of the girder, or damage to the top flange of the girder during deck removal operations. To properly assess the structural condition of damaged bridge girders, it is important that the damage conditions be carefully assessed in the field and calculations be performed for both serviceability and strength limit states. Based on the inspection results and the structural calculations, the damage conditions could be categorized, and appropriate repair/replacement decisions could be made.

There is substantial prior work on damage to bottom flanges of prestressed bridge girders due to impact. However, most such prior works focus on sectional strength issues alone, and do not address the serviceability stress checks that are part of the design requirements in the AASHTO LRFD bridge design specifications. To address the need to calculate changes in service stresses due to damage, undamaged and damaged transformed section properties must first be calculated. Then, procedures must be developed to calculate changes in stress under the service conditions. The damaged section would consist of the undamaged section minus all spalled concrete, severed strands, and cracks that effectively reduce the sectional areas. This would likely result in an unsymmetrical cross section, thus further complicating the calculation process. In this study, a set of procedures are developed to calculate changes in sectional stress due to loss of section that results in a generalized unsymmetrical cross section. These procedures apply equally to cases of bottom or top damage in prestressed girders. Verification of the procedures are provided through a 3-dimensional finite element model.

Considering the irregular pattern of damage in typical field damage cases, it is necessary to develop tools to accurately calculate the irregular section properties for stress calculations. An approach involving the use of spreadsheets was adopted in this study. Each cell in the spreadsheet represented a 0.5-in x 0.5-in square in the cross section. For all standard prestressed girder sections used in Wisconsin, the undamaged cross sections were generated within the spreadsheet. Deck slab is similarly added to make a composite section. The appropriate strand patterns would then be incorporated into the section. The numerical content of a cell would determine whether it is girder concrete, deck concrete, strand, CFRP, strand splice or patch repair material. The section

properties utilize the cell contents and positions for calculations. For damaged properties, the user would “zero-out” all the cells that have been damaged/spalled/severed by deleting the corresponding cell contents. The damaged section properties would then be automatically calculated within the spreadsheet. The strength calculations, although rigorous, are well established and involve using the strain compatibility method allowed in the AASHTO LRFD specifications. This would also allow, within the same calculation process, consideration of loss of strand in the tensile zone, loss of concrete in the compression flange, external CFRP reinforcement, and strand splices.

A software program, Prestressed Bridge Assessment, Repair, and Strengthening (PreBARS), was developed in this study. The program runs within the Excel spreadsheet and involves Visual Basic and spreadsheet calculations. Based on the user input, the program calculates AASHTO HL-93 moments, distribution factors, prestress losses (refined and approximate), serviceability and strength limit state loads, etc. for any point on a bridge with up to three continuous spans. The program calculates sectional service stresses and flexural strengths for undamaged, damaged and repaired conditions. Repairs may involve patches, strand splices, and external CFRP reinforcement. The CFRP reinforcement may be installed at the soffit, the webs, or on the sloped and vertical faces of the bottom flange. The preloading option (involving a loaded dump truck) is also incorporated into the program. The features of PreBARS are described in this report and a user guide is provided in the Appendix.

Using the PreBARS program, several case studies were conducted on two example prestressed I-girder bridges, one with a long span (146 ft) and the other with a short span (50 ft). Different levels of loss of strands in the bottom flange (up to 25% loss) were simulated on both structures, and various repairs were applied in each case. Similarly, different levels of top flange damage were introduced in both structures up to 50% loss of the top flange. CFRP repairs applied on the soffit and the webs were examined in conjunction with the partial splicing of the severed strands for bottom damage cases.

Various damage categories have been defined for both top and bottom flange damage scenarios. For bottom damage, the damage categories include minor, moderate, significant, serious, and severe. For top damage, the categories are minor, moderate, and significant. The following table provides a description for each damage category. Several detailed recommendations have been made regarding inspections, assessment, and repair of damaged prestressed girders for bottom and top damage scenarios.

Bottom Flange Damage	
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 at 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.
Top Flange Damage	
Damage Category	Description
Minor	Concrete nicks, gouges, scrapes, and cracks that are less than 0.006 in wide. Spalled area less than 25% of top flange area; no cracking near supports due to girder uplift; girder sweep <1/8-in over 10 ft length; no damage to interface shear reinforcement.
Moderate	Spalled area more than 25% and less than 50% of top flange area; no cracking near supports due to girder uplift; girder sweep <1/8-in over 10 ft length; little to no damage to interface shear reinforcement.
Significant	Spalled area more than 50% of top flange area; or cracking near supports due to girder uplift; or girder sweep >1/8-in over 10 ft length; or extensive damage to interface shear reinforcement.

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Chapter 1. Introduction

1.1 Background

The performance record of over 92,000 prestressed concrete bridge structures in the US is generally satisfactory (Novokshchenov, 1988 and Whiting et al., 1993). However, accidental damage and deterioration does occur, which can cause serious damage to prestressed concrete bridges in all states (Shanafelt and Horn, 1985).

There are three primary modes of damage to prestressed concrete bridge girders, two of which are accidental or unintended:

1) Accidental impact damage due to over-height vehicles, primarily at the bottom flange and web areas. This mode of damage is not unique to prestressed girders, but the emergency decisions to be made and the repair processes may be special and unique for prestressed concrete (Figure 1.1.a).
2) Unintended construction-related damage during deck removal processes, primarily at the top flange area. This mode of damage has frequently occurred in prestressed bridge girders (Figure 1.1.b).
3) Long-term corrosion and deterioration of prestressed girders, primarily at the beam end areas adjacent to expansion joints, as well as overall deterioration. This long-term mode of damage typically results from exposure to deicing salts leaking through expansion joints (Figure 1.1.c).

This study addresses the first two modes of damage. Item No. 3 has been previously studied extensively by Tabatabai et al. (2005 and 2009) for the Wisconsin-Highway Research Program (WHRP). There are specific recommendations given for corrosion-induced damage prevention and repair of prestressed beam ends (Tabatabai et al., 2005). Tabatabai et al. (2009) provided detailed information and recommendation on the use of patch repairs, zinc anodes, thermal-spray zinc as well as the use of sealers and coating for corrosion repair of reinforced and prestressed concrete bridges.

When faced with an accidental damage, an engineer may need to first make emergency decisions regarding public safety, stabilization of the situation, and possible restriction of access to the bridge (top or underside) based on the initial (limited) observations of the damage. The next step would be to obtain relevant information about the damaged girder(s) including material properties, geometry, reinforcement, and location of the girder (interior or exterior). The extent and severity of damage must then be assessed. This assessment is generally made using visual inspections. However, non-destructive testing may also be used. Analyses (hand calculations or computer models) may also be performed to further clarify the nature and extent of damage, and to help with proper design of repair. Such analyses must consider both serviceability and strength limit states. The repairs should not only restore acceptable serviceability and strength condition but must also be durable and be able to serve their purposes during the remaining service life of the bridge.

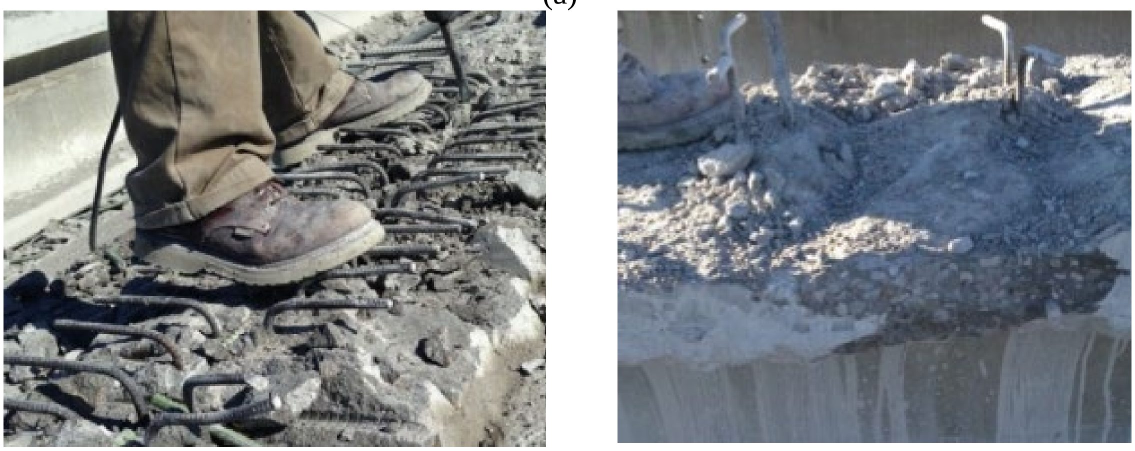
1.2 Impact Damage to Bottom Flange

Impact damage can range from minor scrapes to severe structural damage, mostly occurring in the exterior girder. However, damage to interior girders could also occur as the vehicle may bounce back after initial impact. Assessment of damage and choosing the right repair techniques for prestressed bridge girders have been a challenge for bridge engineers. Impact damage can affect concrete and/or pre-stressing strands. Therefore, repair techniques require awareness of concrete repair methodologies, prestressing strand replacement, and possible restoration of prestressing in concrete (Shanafelt and Horn, 1985).

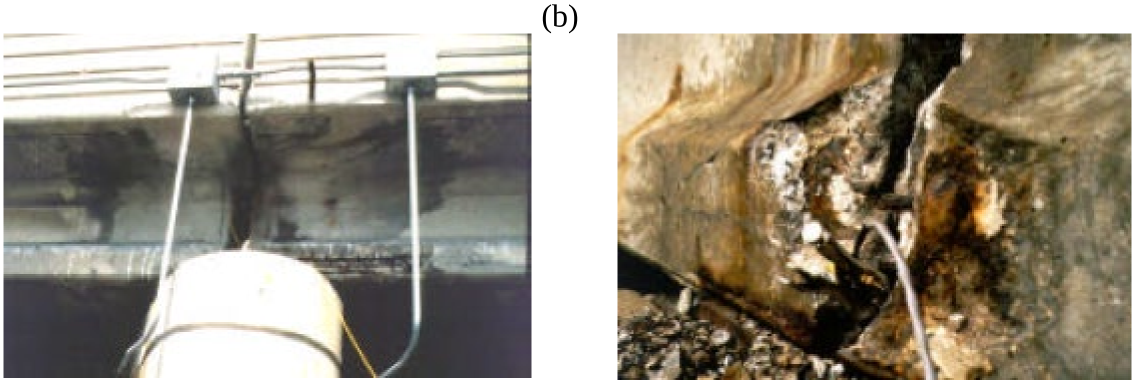
Impact damage caused by over-height vehicles is usually located in the middle one-third region of the girders. Corrosion damage can be one of the long-term consequences of impact damage, due to exposure of strands to the environment.

There is significant detailed information available in the literature regarding accidental damage to prestressed girders due to over-height vehicles. Detailed guidelines have been proposed that address assessments and classification of damage as well as specific repair techniques and procedures for a variety of damage conditions. Notable examples include NCHRP Report 280 (Shanafelt and Horn, 1985), NCHRP Report 226 (Shanafelt and Horn, 1980), and a PhD dissertation by Kassan (2012). However, these and other studies do not address the problem of estimating service stresses in damaged prestressed girders, and do not provide detailed procedures or software regarding the various aspects of assessment and repair of damage.

(a)



(b)



(c)

Figure 1.1. Modes of damage to prestressed girders: a) Accidental over-height vehicle impact (Harries 2006); b) Unintended damage during deck slab removal (Assad, 2014); and c) Corrosion damage to prestressed beam ends (Tabatabai et al., 2005)

NCHRP Report 226 provides general guidelines for assessment, inspection, and repair. It classifies damage as minor, moderate, and severe. In NCHRP Report 280, some of the repair methods discussed in the NCHRP Report 226 were load-tested and detailed recommendations for their applications were given. Guidelines were proposed based on serviceability conditions, strength, fatigue life, durability, cost, user inconvenience, speed of repairs, aesthetics and range of applicability (Kasan 2012).

Feldman et al. (1996) assessed impact damage reports for the Texas DOT. Harries et al. (2009) investigated repair methods for prestressed bridges in a major report to the Pennsylvania DOT. Harries et al. (2009) discussed the NCHRP 280 report and reviewed their assessment processes. They also conducted a survey of practice in North America.

The repair methods for severe damage discussed in NCHRP 226 included external splicing, internal splicing, and jacketing methods. External splicing is a method for restoring the strength and serviceability of damaged members and increasing the strength of deficient members. High-strength threadbars or seven wire strands are generally used as post-tensioning elements along with corbels for transfer of post-tensioning forces (Shanafelt and Horn 1980). Internal splicing (strand splice) is a technique for repairing one or more severed strands. Strand splices are designed to relink severed strands and compensate for lost tension. The general procedures involve preloading the beam, splicing the strand, injecting epoxy resin into cracks and patching the area. Preload can then be removed after the patch concrete has gained the necessary strength (Shanafelt and Horn 1985). Steel jacketing involves the use of steel plates covering the bottom flange of the girder to restore flexural strength of the girder. However, because of issues with practicality and field welding requirements, it has not seen widespread use in recent years.

Externally-bonded Carbon Fiber Reinforced Polymers (CFRP) may be an effective technique in restoring the original flexural capacity and stiffness of damaged bridge girders (Klaiber, 2003; Miller, 2006; Rosenboom and Rizkalla, 2006). Several field investigations (Stallings et al. 2000, Schiebel et al. 2001, Tumialan et al. 2001, and Di Ludovico 2003) and a few laboratory tests (Klaiber et al. 1999, Green et al. 2004, Di Ludovico et al. 2005) have shown the efficiency of CFRP application for repair of damage to prestressed concrete. Rosenboom (2008) analyzed results of tests on five full-scale AASHTO Type II prestressed girders that were repaired with CFRP under static and fatigue loading. The results were considered promising for restoration of original strength. However, CFRP repair technique should involve careful steps to preclude debonding failures. In some cases, use of transverse U-wrapped CFRP strips were recommended to hold the CFRP and underlying concrete patch and mitigate debonding at the CFRP location (Reed et al., 2005).

Feldman et al. (1995) conducted a research project on repair of prestressed girders for the Texas Department of Transportation. The authors conducted a survey of state districts to determine repair practices. Survey results indicated that 1/5th of impact cases involved moderate damage, while 1/8th of impacts resulted in severe damage. Minor damage was repaired by patching concrete. Moderate damage was repaired through patching and, in some cases, epoxy was injected into the cracks. The girders were replaced in two cases. This report also includes a survey of common repair practice in the Unites States and Canada.

In a study sponsored by the Pennsylvania Department of Transportation (PennDOT), a survey of all US DOTs and PennDOT Districts was conducted (Harries et al. 2009). The authors investigated different repair methods including CFRP repairs, strand splicing and steel post tensioning repairs. They considered adjacent box beams (AB), multi-box (spread box) beams (SB), and I-beams (AASHTO-type beams) (IB) in their analytical models.

Detailed results of these and other studies related to damage to bottom flange of prestressed girder bridges are presented in Chapter 2 of this report.

1.3 Deck Removal Related Damage

Removal of an old bridge deck can be accomplished using equipment such as pneumatic and hydraulic hammers and breakers, saws, drills, etc. Pneumatic hammers and rig-mounted breakers can cause micro-cracking or spalling in the top flange (Tadros and Baishya 1998).

Prestressed AASHTO and Bulb-T girders provide narrow and wide top flanges, respectively. Bulb-T girders offer sectional efficiencies, provide a larger platform for workers, and increase girder stability. Both the older AASHTO girders and the more modern Bulb-T girders could sustain damage during deck replacement procedures. The bulb-T girders have generally thinner and wider flanges than the AASHTO girders, but unlike AASHTO girders, are reinforced with steel bars.

According to a survey by Assad (2014), most state DOTs use saw-cutting between girders. After removing the concrete segments, jackhammering is used to remove the remaining parts of the deck. Four out of 10 states apply hydro-demolition. Some states use pneumatic hammers attached to mini-excavators or backhoes for removal of top half of the deck. However, the risk of major damage to the top flange cannot be ignored. The remaining part of the deck would be removed through hand-held chipping hammers or small jackhammers. Assad reports that having a debonded strip at the edge of the top flange could facilitate deck removal.

Assad (2014) investigated different deck removal methods and their effect on the performance of the pre-stressed girder with wide and narrow top flanges. The author studied saw cutting and jackhammering techniques implemented on the Camp Creek Bridge over I-80 in Lancaster County, Nebraska, and investigated the extent of damage to the girder as well as cost, duration, and environmental impact of each method. The author took two girders to the lab to establish the structural properties of the girder after implementation of the deck removal. He also analyzed flexural capacity, horizontal shear capacity, and deflection of the girder at the time of construction and in service. He studied the effect of reducing 50% of the width of top flange for cases with different span-to-depth ratios. He suggested checking the flexural capacity, horizontal shear capacity and deflections during construction and at the strength limit state.

Detailed results of these and other research related to damage to top flanges of prestressed bridge girders are provided in Chapter 2 including recommendations previously made to minimize damage to girders and provisions by a few states on ways to address this issue.

1.4 Research Objectives

The overall objective of this project was to develop recommendations and guidelines for the inspection, evaluation, repair, or other needed safety and operational response related to damaged prestressed concrete girders. These guidelines would aid the Wisconsin Department of Transportation (WisDOT) in making prompt decisions on the appropriate course of action when a girder is damaged. These guidelines would include actions and repair techniques that are based on the common types of damage encountered in Wisconsin.

Specific objectives were to address inspection requirements to determine and categorize the extent of damage, assess the effect of damage on strength and serviceability conditions (using computer models), provide recommendations regarding the need for repair(s), and recommend applicable repair procedures.

1.5 Research Scope

The specific tasks performed during this study were as follows:

- Task A –Review of WisDOT Data

This task involved contacting WisDOT personnel to obtain information regarding damage cases, assessments and repair procedures. The information gathered were incorporated in various phases of this study.

- Task B: Literature and Information Search

A comprehensive review of literature was conducted related to the topics of this study. Online databases as well as personal contacts were used to obtain relevant information.

- Task C: Analytical Modeling

A comprehensive software program (Prestressed Bridge Assessment, Repair, and Strengthening - PreBARS) for assessing serviceability and strength characteristics of girders under undamaged, damaged, and repaired conditions was developed. This program contains parameters and details used specifically by WisDOT. The program calculates AASHTO LRFD bridge loads (permanent and transient), distribution factors, prestress losses (refined and approximate) and various strength and serviceability load combinations for standard Wisconsin prestressed I girders and box girders. A specific set of procedures were developed to calculate changes in stresses under service conditions following damage to a girder. The software can calculate stress and strength under repair conditions that may include patching, internal strand splices, external CFRP reinforcement, or a combination of these methods. The Abaqus finite element program was used to generate 3-dimensional models and to calculate the stress redistribution in damaged girders through removal of elements. Several case studies were simulated using PreBARS to study the effects of various levels of damage and repair.

- Task D: Field Assessments

A total of five Wisconsin bridges were inspected that exhibited various levels of damage to either bottom or top flanges, including some that had been repaired. Information obtained during these inspections were used in the development of the software program and in developing the study recommendations.

- Task E: Recommendations, Guidelines, and Reporting

  The research team has developed a set of recommendations and guidelines for assessment and repair of accidental damage to top and bottom flanges of prestressed bridge girders. Guidelines have been developed to categorize the extent of damage. These include Minor, Moderate, Significant, Serious, and Severe categories for bottom damage cases, and Minor, Moderate and Significant categories for top damage cases. Inspection, calculations, and repairs are discussed for each damage category.

Chapter 2. Literature Review

In this chapter, results of a comprehensive review of literature regarding damage to prestressed concrete bridge girders and the repair of such damage is presented with detailed discussions of works that address the subject issues in a substantive manner. The prior works are organized into three sections: bottom damage, top damage, and repair methods and materials. Where applicable various state practices are also discussed. Tables 2.1 and 2.2 show a summary of major studies that are discussed in detail here.

Table 2.1. Summary of major studies on bottom damaged prestressed girders

Research name	Discussed Methods	Damage classifications proposed?
Shanafelt and Horn (1980) (NCHRP 226)	This is the first comprehensive study on evaluation and assessment of the damaged girder(s), external post-tensioning, internal strand splices	Yes
Shanafelt and Horn (1985) (NCHRP 280)	Epoxy injection, patching, application of preload, internal strand splices, external post-tensioning, metal sleeve splices, and girder replacement.	Yes
Olson et al. (1992)	internal strand splices, external post-tensioning	No
Feldman et al. (1996)	Patching, protection coating, internal splicing, girder replacement.	Yes
Zobel and Jirsa (1998)	internal strand splices	No
Di Ludovico (2003)	CFRP	No
Wipt et al. (2004)	CFRP, concrete patch	No
Kim et al. (2008)	CFRP sheets	No
Harries et al. (2009)	CFRP fabrics, CFRP strips, near-surface mounted (NSM) CFRP, prestressed CFRP, post-tensioned CFRP, strand splicing and external steel post-tensioning	No
Brinkman (2012)	CFRP	No
Harries et al. (2012)	externally bonded CFRP (EB-CFRP), externally-bonded post-tensioned CFRP (bPT-CFRP), post-tensioned steel (PT-steel), internal strand splicing, and combination of these methods with internal splicing	Yes
El Meski and Harajli (2013)	CFRP	No
Montero (2015) and Yazdani and Montero (2016)	GFRP	No
Liesen (2015)	FRP, internal splicing, application of preload	No
Gangi (2015) and Gangi et al. (2018)	internal strand splices, externally applied FRP, and externally applied fabric reinforced cementitious matric (FRCM)	No
PCI (2006)	Repair procedures, patching and epoxy injection procedures, patching materials	No
Wipf et al. (2004) (Vol 3 of 3)	Patch materials, CFRP	No

Table 2.2. Summary of major studies on top damaged prestressed girders

Research name	Discussed Methods	Damage classifications proposed?
Tadros and Baishya (1998) (NCHRP 407)	Rapid bridge deck replacement, deck removal methods	No
Badie and Tadros (2000)	proposed new deck system, bridge deck removal methods, shear connectors	No
Consolazio and Hamilton (2007)	lateral stability of prestressed concrete girders	No
Phares et al. (2014)	Bridge deck removal methods	No
Potisuk et al. (2016)	Deck replacement methods	No
PCI (2006)	Repair procedures, patching and epoxy injection procedures, patching materials	No
Wipf et al. (2004) (Vol 3 of 3)	Patch materials, CFRP	No

2.1 Literature related to damage to bottom flange of prestressed girders due to impact

2.1.1 Shanafelt and Horn (1980) (NCHRP 226)

This 1980 report provides guidance on assessment and repair of prestressed bridge members that have sustained accidental damage from collision, fire, or manufacturing problems. A survey of transportation agencies in all 50 US states as well as Canadian provinces was conducted. The authors reported that 0.86% of prestressed girder bridges were involved in accidents over a year of reports by DOTs. Approximately 81% of all damage is due to over-height vehicle impact, while only 2.5% of all cases are related to fire damage. The authors report that roughly two-thirds of all bridge girders are I-shaped (AASHTO or Bulb-T), about one quarter are box girders, and the rest are slab bridges. The proportions associated with minor, moderate, severe, and critical damage categories were reported to be 72, 8, 15, and 5%, respectively. The authors listed load capacity and durability as important factors in selection of repair methods and materials. They further highlighted the importance of engineering analysis/calculation to be conducted after an inspection assessment of damage.

The authors made recommendations regarding inspection procedures, including a recommendation to develop and use standard forms for inspection of damage. They suggest that structural engineers should perform the inspections but avoid recommendations on repair/replacement in the inspection report, as that may result in premature decisions that are not based on careful analyses. The inspectors should also assess any traffic restrictions that may be required for safety purposes. They recommend that police reports be included in the damage inspection report, and a data recording system be implemented.

The authors suggest that load capacity should be the most important factor in selecting repair methods. They suggest use of high quality materials, effective surface preparation and procedures, preloading (before epoxy injection), epoxy injection of cracks wider than 3 mils, use of nominal reinforcement to tie old and new concrete, and post-tensioning to achieve durable repairs. Compatibility and aesthetics should be a factor in the design of the repair.

The authors conclude that although the replacement option fully restores structural capacity, other considerations favor repair-in-place techniques. They suggest that the need for preloading should be evaluated through calculations for all girders being repaired that have significant loss of concrete. Excessive preloading should be avoided to prevent cracking of concrete at other sections. Preloading can be implemented through placement of a loaded vehicle or jacking.

The authors provided detailed guidelines for assessment of damage. They recommend developing procedures to handle accidental emergencies in advance. They define damage as: minor, moderate, severe, and critical. Definitions of various types of damage are as follows:

<u>Minor</u>: Regardless of the extent of spalling, damage is restricted to concrete only without exposed strands or reinforcing bars, and with only small cracks (less than 3 mils wide) emanating from spalled areas. In such cases, the inspection report should include the location and size of all spalled areas as well as the location, width, and lengths of all cracks. Structural calculations would be needed in case of presence of any spalled areas.

<u>Moderate</u>: Damage is in the form of concrete spalling and cracks that may be wider than 3 mils, with exposed bars and/or strands, but without any severed strands. Cracks should be closed below the surface. In addition to the reporting requirements for minor damage, the condition of any exposed strands and bars should be reported including presence of all visible marks and distortions. Structural calculations should be performed, and possible traffic restrictions should be assessed.

<u>Severe</u>: Severe damage to concrete and steel including one or more of the following: transverse cracks extending across the width of bottom flange, but closed below the surface.; major

or total loss of the concrete in the bottom flange; major loss of concrete in the web area (but not at the same section as bottom flange damage); severed strand(s) that are visibly deformed; and excessive horizontal misalignment of the bottom flange (or excessive vertical misalignment of the web). In addition to the reporting requirements for the moderate damage, the location of any hold-down devices should be reported relative to the damaged area, and horizontal and vertical misalignments shall be reported. Cracks should also be assessed whether they are closed below the surface. Structural calculations should be performed and traffic restrictions should be evaluated. Use of longitudinal steel girders above the affected girder or use of temporary falseworks should be assessed. Severe and critical damage should be inspected by a structural engineer(s) who will also perform the assessment.

Critical: This involves damage to concrete and steel elements in the form of one or more of the following conditions: Cracking across the width of the bottom flange and/or in the web directly above the damaged flange with cracks not closed below the surface (indicating yielding of strand); an abrupt lateral offset of the bottom flange, or lateral distortion of the exposed strands; extensive loss of prestress such that calculations show ineffectiveness of any repairs; excessive vertical misalignment; and longitudinal cracks at the interface between the top flange and the web that are not closed below the surface. Reporting requirements are the same as the "severe" case. Temporary support or falsework may be required at the time of inspection. Critical damage requires replacement of the girder.

The authors recommend using a standard form for reporting of damage. They suggest not including repair recommendations, but to discuss factors that may influence various solutions. The dates of damage and inspection as well as the law enforcement accident reports should be included. A system for easy access to damage data and photographs should be provided. Except for minor damage, structural calculations should be performed. Calculated and design stresses, shoring requirements and travel restrictions should be evaluated.

The authors list five factors to be considered for the repair: service load stresses, ultimate strength, overload capacity, fatigue, and durability. Aside from minor differences, the service stresses and strength capacities should be the same as the original girder.

2.1.2 Shanafelt and Horn (1985) (NCHRP 280)

This publication followed the NCHRP 226 publication by the same authors. Guidelines are provided for the evaluation and repair of prestressed concrete bridge members. Several repair methods were experimentally evaluated including internal splicing of strands, external post-tensioning, and a metal sleeve splice repair. Repair methods considered included epoxy injection, patching, application of preload, internal strand splices, external post-tensioning, metal sleeve splices, and girder replacement. They recommend that internal splicing of strands be limited to a maximum of 25% of all strands, unless external strengthening is also utilized. They recommend application of preload before patching, with the removal of preload occurring after the patch has gained sufficient strength. The use of external post-tensioning was successful in restoring and enhancing the load-displacement response. The use of metal sleeve splice showed one of the best load-displacement responses, even when 6 out of the 16 strands were severed (and not internally spliced). A typical metal sleeve splice includes five steel plates that are welded together in the field to encase the bottom flange in the damaged area, and the space between the plates and the girder would be grouted. Internal strand splices were also effective. The authors also discuss the importance of preloading before epoxy injection. They suggest that repairing box girders would be generally the same as the corresponding repairs for the I-girders, but repair configurations may be different. The bottom exterior corner of box girders is generally impacted due to over-height vehicles. However, the interior areas of box girders would not be accessible in most cases.

The authors recommend that minor nicks, spalls and scrapes be cleaned and sealed with penetrating sealers. Minor gouges may be cleaned and coated with penetrating sealer coats. If the gouge acts as a stress concentration point at the bottom of the girder, then the girder should be preloaded, and the gouge should be cleaned and filled with epoxy grout. Cracks should also be epoxy injected. Epoxy resin can penetrate cracks with width of at least 0.002 in (0.05 mm). The authors suggest that there is no specific limit for the quantity of loose concrete that can be removed.

The authors conclude that metal sleeve splices can be used without restoring compression through preload. However, concrete tensile stresses at the top and ends of the sleeve should not be excessive. A careful examination of exposed strands should be performed to assess damage. They suggest that a “nick” in one wire is not considered serious. However, severed, flattened or bent wires in strands are considered more serious, and such strands should be considered severed. The authors report that during load tests to 75% of moment strength, the maximum crack widths ranging from 0.016 in (0.4 mm) to 0.024 in (0.6 mm) were noted. However, these cracks subsequently closed to hairline width after removal of load, indicating that the strands were not yielded. Therefore, they suggest that closed vertical cracks in pre-compressed zones indicate that the steel had not yielded. They suggest that most states allow one to three severed strands, and do not see the need for setting an upper limit on the number of spliced strands.

The authors further suggest that web reinforcement can be repaired in place by straightening bent bars, lap splicing with another bar, or welding bars together. The authors consider abrupt lateral offsets (displacements) as an important concern. In such cases, the number of strands with abrupt deformations should also be determined. Those strands would be considered yielded and ineffective. The authors believe that significant vertical deflection is not common in impact damage, unless a large fraction of strands is severed. Upward deflection may occur when large areas of concrete have been damaged without damage to strands. Permanent twisting can occur because of impact, with tension introduced by torqueing the splice device. They suggest tensioning the splices, followed by applying preload, epoxy injecting cracks, application of patching (until it gains sufficient strength), and finally removal of preload. They suggest limiting splicing to 25% of total number of strands.

They recommend using metal steel splices when many strands are severed, or when large volumes of concrete are lost. Epoxy resin is used to bond galvanized steel plates to concrete. They recommended using a minimum "lap" length of 160 times the diameter of strands on either side of the strand fracture when more than 6 strands are severed. A minimum "lap" length of 63 in (1.6 m) is recommended when six or fewer strands are severed. They further suggest that preloading would not be required, unless stresses above or beyond the sleeves exceed allowable values. Combining different splice methods may also offer advantages. Complete replacement is the most expensive option, especially in continuous girders, as it requires partial removal of slab. The authors express the opinion that nearly all accidental damage can be repaired in place. The authors report that internal single-strand splices are inexpensive and easy to install

Regarding inspection reports, the authors provide the following list of items to be included in an inspection report:

1) Bridge name
2) Bridge location description including location map
3) Date of damage
4) Date of inspection
5) Law enforcement accident report
6) Cause of damage
7) Site conditions
    a. Damage over traffic
    b. Damage over water
    c. Other
8) Information on user inconvenience
9) Bridge plans if available
10) Supplementary sketches
11) Type of member
12) Member identification
13) Member category
    a. Fracture critical
    b. Primary
    c. Secondary
14) Damage to prestressing elements
    a. Exposed
    b. Nicked
    c. Gouged
    d. Deformed
    e. Severed
    f. Damage locations
    g. Narrative description
15) Damage to concrete
    a. Spalls, nicks, scrapes, and gouges (give sizes)

b. Cracks including width, length, and configuration
c. Loose concrete
d. Shattered concrete
e. Damage locations.
f. Narrative description

16) Member displacement
    a. Lateral
    b. Vertical
    c. Rotation or twist
    d. Narrative description
17) Photographs
18) Factors that may affect repair solutions
19) Description of initial action
    a. Traffic restriction
    b. Member strengthening
    c. Other

2.1.3 Olson et al. (1992)

The authors report that 200 prestressed bridge girders are damaged annually. They performed tests on four girders that were produced in 1967 and removed from service in 1984. The objectives were to measure effective prestress, assess performance of damaged girders, and evaluate two repair schemes. One repair scheme involved internal splicing of strands and the other involved external post-tensioning using threadbars. The test girders were Type III and had a span of approximately 65 ft. Thirty ½-in prestressing strands were used in each girder. The strand splices used for repairing strands (consisting of two strand chuck mating pieces, a turnbuckle, and two threaded rods) was made by researchers using 1045 steel. The turnbuckle was instrumented to measure load. Unsymmetrical damage was introduced and static (service load), fatigue, and strength tests were performed.

The authors concluded that the internal splices were better and more efficient than the external post-tensioning in restoring capacity. However, both were more susceptible to fatigue than other elements. The failure of spliced strands and PT anchorages also occurred earlier than other strands during the ultimate test. Stiffness was improved and stress ranges in undamaged strands were reduced, but the ultimate strength was not completely restored.

2.1.4 Feldman et al. (1996)

This 1996 report presents a literature search and a survey of practice in the US and Canada regarding repair of prestressed concrete bridge girders. The authors also reviewed repair materials and procedures and discussed repair procedures.

Survey results from Texas Department of Transportation (TxDOT) districts indicated that at least two-third of all damages were classified as minor, 20% as moderate, and 14% as severe. Minor damage was defined as cracks, shallow spalls and scrapes while spalls causing exposure of strands was classified as moderate, and severe damage included damaged tendons or loss of major portion of concrete section. From 1987 to 1992, an average of one girder per week was damaged in Texas due to impact. A great majority of damage inspections (88%) were performed visually without any non-destructive tests. In only 4% of the cases, any analysis calculations were performed. The respondents considered interruption of service and strength of repaired girder as the most important factors in deciding repair procedures. Cost of repair and aesthetics were reported to be of lowest priority.

Regarding repair, about two-thirds of minor damages were not repaired. Patching was the most common repair for minor damage, with an epoxy coating applied over the surface in only a few cases. A great majority (86%) of girders classified as moderate damage were repaired. Removal of loose concrete, surface preparation, and patching with patch material or concrete constituted the repair procedure. Beams were typically preloaded during repair. Severe damage cases involved patching with epoxy-injections in some cases. Only one case of repair of severed strands was reported, in which the strands were repaired using internal splices. Approximately 70% of severe damage girders were replaced in Texas.

A subsequent survey of the US states and Canadian provinces and territories indicated similar proportions of minor, moderate and severe damage as the Texas survey. About one-fourth of damage cases were classified as moderate and one-eight as severe. Only 10% of inspections involved any non-destructive testing, and only 28% of cases included some form of analysis. US and Canadian agencies considered strength of repaired girder and interruption of service as the two most important considerations in determining repair procedures. Cost and aesthetics had the lowest priority.

Approximately two-thirds of minor damage cases were repaired, typically patching. Repair in moderate damage cases involved removing loose and damaged concrete, surface preparation and patching. Three-quarters of such US/Canada cases were also epoxy injected. About 55% of girders were replaced in the severe US/Canada cases. Internal splicing was the most common repair method for severed strands.

Other repair approaches by various states were also noted. California has used a supplemental girder behind the damaged girder and has reportedly used steel beams above the girder to reduce dead load stresses. Hawaii sometimes added steel channels to the girder to increase strength. Alaska and Saskatchewan devised their own internal splicing devices and procedures.

The authors suggest that repairs are generally more economical than replacement. They suggest simple non-destructive methods, such as hammer sounding and Schmidt hammer, to identify hidden delaminated and cracked concrete. They recommend classification of damage as minor, moderate and severe, using the following definitions:

Minor: “Shallow concrete cracks and nicks, shallow spalls and/or scrapes”.

Moderate: “Large concrete cracks, and spalls large enough to expose undamaged prestressing strands”.

Severe: “Exposed damaged tendons or loss of significant portions of concrete cross section as well as possible girder distortion resulting in lateral misalignment.”

The authors describe typical cracking pattern (Figure 2.1) associated by side impact by a truck (Feldman et al., 1996).

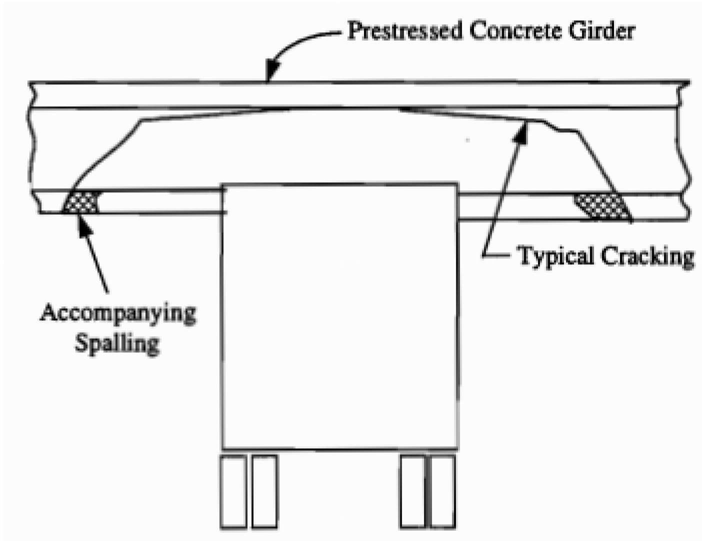
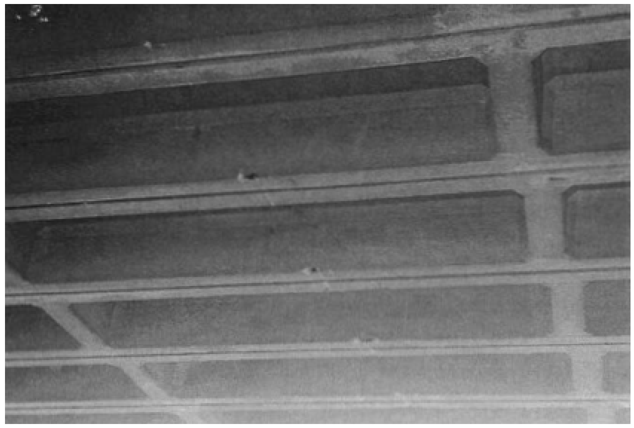


Figure 2.1. Typical crack pattern due to side truck impact (from Feldman et al., 1996)

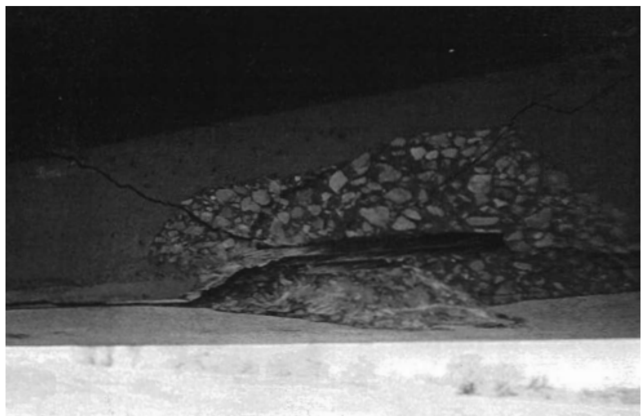
Examples of minor, moderate and severe damage (from Manitoba, Texas, and Washington DOTs, respectively) are shown in Figure 2.2 (Feldman et al., 1996).

The authors recommend that loose and delaminated concrete should be removed, the surface should be roughened and free from dust or other foreign materials, and the patch boundaries should be "sharply defined" during patching operations. Chipping hammers (maximum 15 to 30 lbs.) should be used with extreme caution near strands. They consider hydrodemolition to be a possible safer option. The patch material must be evaluated with respect to the properties of base concrete and must be suitable for overhead and vertical patching. Extending the patch with aggregates would reduce shrinkage can control thermal properties. Hand application is typically sufficient for minor repair, and shotcreting has sometimes been used (in Texas) in heavy traffic conditions. Forming may be required in other cases to restore girder dimensions.

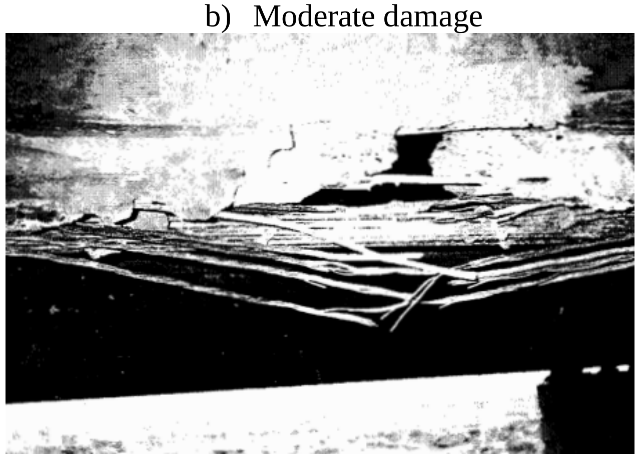
In moderate damage, it is recommended that concrete be removed ¾ in (20 mm) around the steel. Feathered edges should be removed by shallow saw cutting that is carefully done to avoid damaging steel. Rust on the steel should be cleaned. Preloading would restore some prestress in the patch and thus provide some allowance for shrinkage of the patch material. The authors report on the work on others suggesting that cracks wider than 0.008 in (0.2 mm) should be injected, but do not indicate whether it should be done under preload. They report that narrower cracks should receive a sealer application. Painting of girder may be done for aesthetic purposes.



a) Minor damage



b) Moderate damage



c) Severe damage

Figure 2.2. Examples of damage classifications: a) minor damage (Manitoba Highways and Transportation); b) moderate damage (Texas Department of Transportation); and c) severe damage (Washington Department of Transportation) (all from Feldman et al., 1996).

The authors proposed a course of actions in the flowchart for the assessment and repair of severely damaged girders (Figure 2.3). They suggested performing structural analysis considering the loss of concrete section and any severed strands. However, they did not provide specific procedures or guidelines for performing the structural calculations. The concrete removal, cleaning procedures are as described earlier for moderate damage. In case of internal splicing additional concrete may have to be removed to accommodate splicing hardware. They reported that internal splicing was inexpensive and relatively easy to install (Figure 2.4), and unlike external post-tensioning, does not negatively impact the appearance of the girder. Application of torque on the splice hardware allows stressing of the strands (Figure 2.5). Because of the size of the splicing hardware, it may not be possible to splice many strands. Epoxy injection and patching procedures are as described earlier for moderate damage.

2.1.5 Civjan et al. (1996)

The authors report on the development of a device to measure stress remaining in strands following impact damage to prestressed girders. The authors considered a method involving application of a transverse force to the strand and measuring its deflection ("lateral force – deflection approach") as the most practical method for assessing existing force in strands. This involved incremental increases in the transverse force and recording the resulting displacements. Prototypes were designed, and experiments were performed using this approach. The authors concluded that the device had an accuracy of ±10%, and recommendations were made to improve the accuracy.

2.1.6 Zobel and Jirsa (1998)

A bridge girder that was damaged due to impact was removed from a railroad bridge and tested to assess the effectiveness of strand splices. Four different internal strand splices were used. Structural performance, load distribution between strands, and ease of use was considered. The authors suggested that all assemblies had losses on the order of 5 to 10% of initial stress. Load-deflection results indicated that the stiffness of the girder was essentially unchanged when two out of 28 strands were not repaired (2 out of 4). However, a slight difference was observed when all four strands were not repaired. The authors concluded that internal splices can be used for restoration of strength if fatigue is not a major concern. However, the authors cautioned against repairing more than 10 to 15% of the total number of strands in the cross section. They suggest that the repaired strand could achieve at least 85% of the nominal strength of the strand.

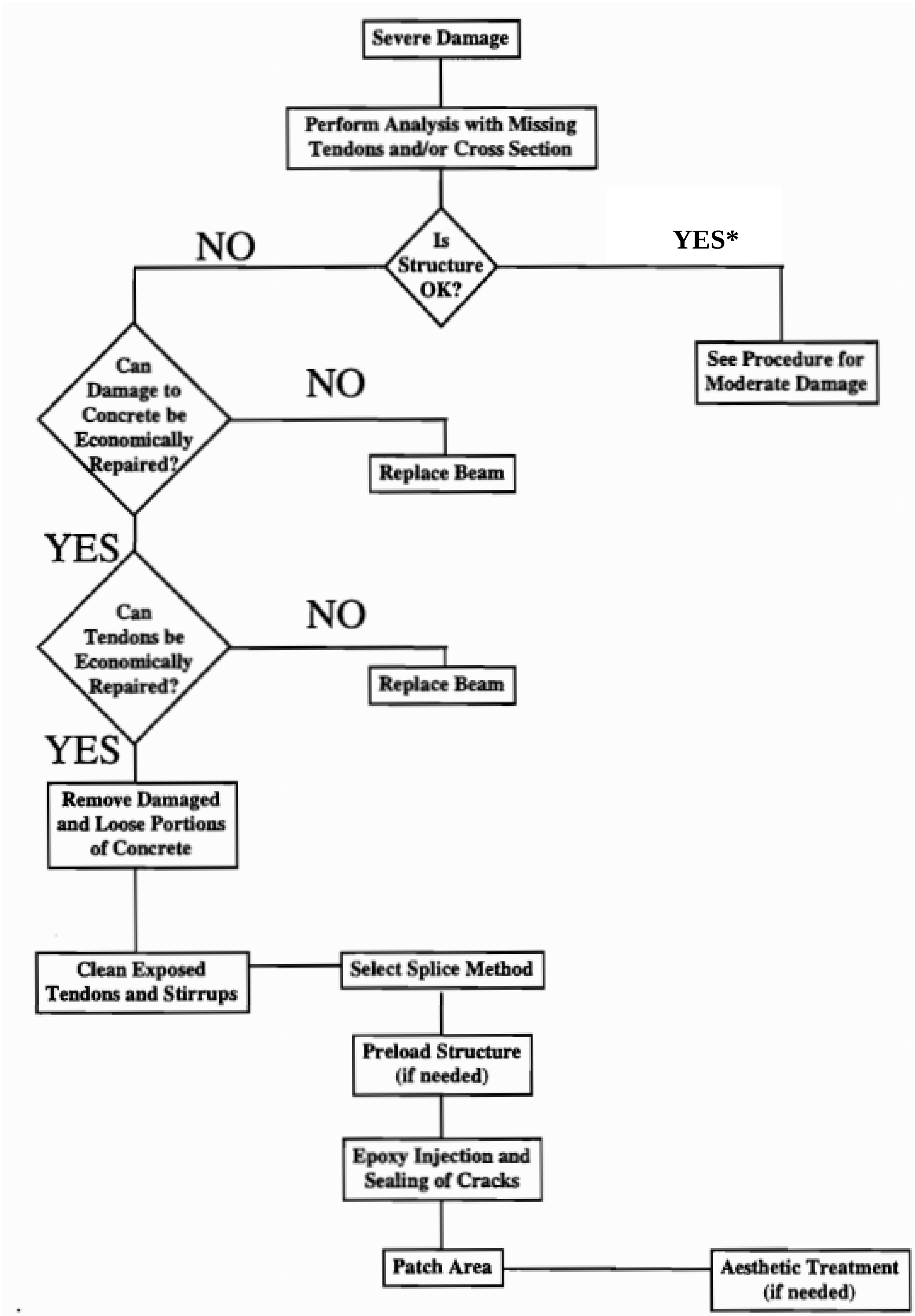


Figure 2.3. Flowchart for repair of severely damaged girders (from Feldman et al., 1996) (*apparent typographical error has been corrected).

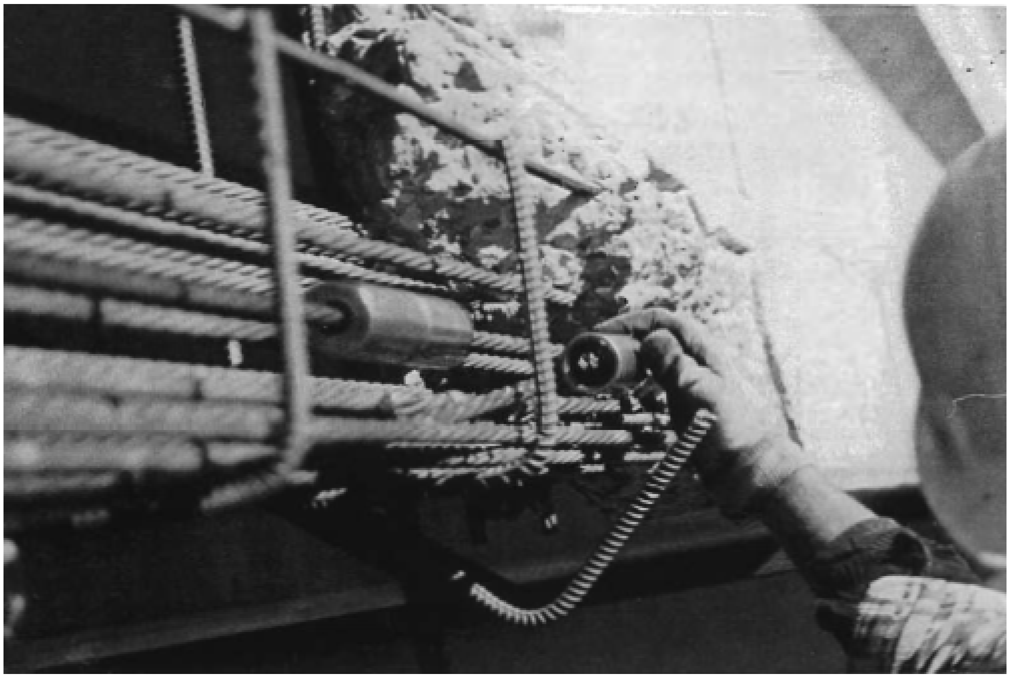


Figure 2.4. Installation of internal splice device (from Feldman et al., 1996)

Figure 2.5. Tensioning of internal splices (from Feldman et al., 1996)

2.1.7 Di Ludovico (2003)

In this study, two prestressed concrete girders, one undamaged and another intentionally damaged and repaired using CFRP were tested. The girders were composite with a deck slab, but the width of the deck slab was smaller than in usual bridges. A discussion of nominal moment strength calculations is given. The failure modes considered include crushing of concrete (strain = 0.003), yielding of steel followed by rupture of FRP, cover delamination, and debonding of FRP from concrete. Tests indicated a FRP debonding failure at a strain of approximately 0.007, about half the strain value that was originally estimated. The author considered CFRP laminate as a viable option for flexural upgrade.

2.1.8 Wipt et al. (2004)

This report constitutes the first volume of a 3-volume study performed for the Iowa Department of Transportation. This volume addressed the repair of impact-damaged prestressed concrete beams. The authors report that five or six significant over-height truck bridge impacts occur in Iowa annually, and the average cost for repair of each damaged bridge is on the order of $38,900 (in 2004).

Carbon Fiber Reinforced Polymer (CFRP) layers were examined for repair and strengthening of damaged beams. They were considered suitable because of their high strength to weight ratio, enhanced corrosion and fatigue properties, and relative ease of installation. Laboratory tests as well as three field installations were performed.

A national survey of state transportation agencies was conducted, and laboratory tests on four full-size and repaired prestressed concrete beams were performed. Portions of bottom flange were removed, and some strands were severed. A concrete patch and CFRP sheets were bonded to the bottom flange. Three beams were subjected to service load followed by loading to failure. The 4th beam was subjected to cyclic loading prior to failure test.

Three prestressed concrete bridges that had been damaged due to impact from over-height trucks (and had significant loss of concrete as well as severed strands) were repaired, after being load tested in damaged condition. Service loads were reapplied on one bridge after repair. A guide was developed for the design and installation of CFRP repair systems.

The authors discuss the issue of debonding (or peeling) of the FRP from concrete, which is attributed to crack propagation along the interface between FRP and concrete and discuss prior literature on the subject. They report on the work of Arduini and Nanni (1997) in which better bond was achieved when sandblasting was used instead of grinding. Tests by Arduini et al. (1997) were reported to indicate the contribution of transverse CFRP wraps in increasing the strength and ductility of the beams.

The authors further discuss that premature debonding stems from “shear and normal stress concentrations in the adhesive” and discuss equations to calculate those stresses. Stress concentrations appear primarily in the end areas of FRP sheets. The durability of CFRP applications when exposed to freeze-thaw cycles, moisture, chlorides, fatigue, and UV radiation is another area of concern.

The laboratory beam tests were conducted on four composite prestressed concrete beams. The authors used strain compatibility to also calculate the flexural strength of the repaired section. The authors further used an equation provided by the CFRP manufacturer for calculating the development length of CFRP. While Beams 1, 2, and 4 had only two severed strands (17% loss of area) at the bottom of the bottom flange, Beam 3 had 4 severed strands (33% loss of area). Beam 1 did not have transverse CFRP wraps (jacket), while other beams also included transverse jackets. These were added to prevent debonding and enhance confinement of the patch. In beam 2, the jackets did not extend into the web, but Beams 3 and 4 had jackets that were extended into the web up to a point just below the point where the web and the top flange meet.

Debonding was observed in all beam tests. Although not noted by the authors, significant inelastic behavior (yielding of steel was noted) prior to debonding. The authors note that CFRP sheets debonded at 27% of their own tensile strength. However, the strain at debonding was nearly compatible with the maximum strain limits noted in standards that were written after this study was concluded. The authors suggest that flexural strengthening of impact damaged prestressed girders would be feasible when at least 85% of the prestressing strands are intact.

2.1.9 Kim et al. (2008)

The authors discuss the use of prestressed CFRP applications for the repair of impact-damaged prestressed bridge girders. They suggest that prestressing the CFRP would allow a more effective use of CFRP strength, enhance durability and serviceability, and improve shear and flexural strength, among other benefits. The authors recommend direct tensioning of the CFRP against the structure. Prestressed CFRP repairs were applied on a 40-year-old prestressed girder bridge in addition to laboratory and analytical investigations. The prestressed girders were 660-mm-deep "C-shaped" members reinforced with "No. 13 steel strands". Three layers of CFRP sheets were stressed to 21% of the ultimate fiber strain and bonded to the soffit of the damaged girder. An anchorage system consisting of two steel plates with welded angle brackets was designed, which was then bolted to the vertical sides of the beam's web stem. A load test (using trucks placed on the bridge) and finite element analyses were performed. The authors report that the flexural capacity of the bridge was restored fully.

2.1.10 Harries et al. (2009)

Repair methods for three types of prestressed girder bridges, adjacent boxes (AB), spread boxes (SB), and I-beams (IB), are discussed for four different levels of damage. Repairs included CFRP fabrics, CFRP strips, near-surface mounted (NSM) CFRP, prestressed CFRP, post-tensioned CFRP, strand splicing and external steel post-tensioning. Various methods and design examples are discussed. The authors suggest that repair selections should be made on a case-by-case basis, and they consider girder replacement to be appropriate when 25% of strands in the girder are not participating in resisting loads.

2.1.11 Kasan (2012)

The author considers the effect of redevelopment of strand beyond the point of damage. If girder damage occurs away from the point of maximum moment, then the rating of the bridge should consider the redevelopment of strand at the point of maximum moment. A girder was from a decommissioned side-by-side box girder bridge and tests were conducted involving cutting strands and measuring strains of exterior wires on strands at various distances from the cut. The authors conclude that severed strand constitutes a “local effect”, and the strand stress redevelops over the transfer length.

2.1.12 Brinkman (2012)

The objective of this study was to identify CFRP repair techniques for various types and extents of damage to prestressed I-girders. Calculation procedures for the effectiveness of CFRP repairs are also presented. The author provides a comprehensive review of prior research. A bridge girder was modelled using the “Xtract” program. The author discusses an equation to estimate the maximum number of severed strands that can be repaired with CFRP. A repair selection criteria table is provided in which various parameters such as the maximum fraction of prestressing steel that can be replaced with CFRP (10% in case of I-beams and 20% in case of box beams), application of U-wraps, durability, cost, etc. The author suggests that use of U-wraps would make application of preloading unnecessary. The author points out an important paradox in CFRP repair of prestressed beam. An undamaged beam with CFRP applied could, under some circumstances, have a lower strength then the undamaged girder without the CFRP. This is because the debonding criteria would control the problem. The author also discusses the possibility of adding CFRP to the sides of the beam, but argue that this would diminish the effectiveness of the CFRP.

2.1.13 Harries et al. (2012)

The objectives of this comprehensive study were to develop criteria for repair/replace decisions, identify knowledge gaps, and to propose a set of repair guidelines for repair of collision damage to prestressed bridge girders. A focus of the study was on the use of FRP composites for repair of such structures. Applications of externally bonded CFRP (EB-CFRP), externally-bonded post-tensioned CFRP (bPT-CFRP), post-tensioned steel (PT-steel), internal strand splicing, and combination of these methods with internal splicing were considered.

The authors discuss various inspection methods including non-destructive tests and identify visual inspection as the sole practical tool for immediate post-impact assessment. Regarding preloading, the authors suggest that it was recommended for most repairs and it was more effective in cases with lower dead-to-live ratio, but caution must be exercised to avoid overloading. External post-tensioning involves handling bolster regions with concentrated compressive forces, while steel jacketing involves field welding, and needs to be grouted, making it a cumbersome repair technique.

The authors note that the CFRP-concrete bond is the dominant limit state in bonded CFRP applications. The shear capacity of the substrate (concrete) can limit force transfer regardless of the type of anchorage or adhesive used. U-warp anchorage is an alternative to mechanical anchorages. The authors report that most studies indicate deterioration of performance of bonded CFRP systems when subjected to fatigue loading. Regarding durability, the moisture/heat effects are primary deterioration factors for CFRP materials. Water can also damage the FRP matrix. The authors state that both soffit and web can be used to attach CFRP sheets. The debonding limit state for CFRP limits the strain that can be sustained by the CFRP. The authors note that web-installed CFRP would have reduced efficiency. However, the authors do not address the fact that although the efficiency of CFRP may be maximized when installed at the soffit, the efficiency of the prestressing steel may be diminished because of the limiting CFRP strain.

The authors point out that repairs should not diminish the vertical clearance under the bridge. The authors also discuss the need to address the residual strength of girder should the repair fail. ACI 440 suggests that the capacity of the damaged girder (without FRP) should be at least 1.1DL and 0.75LL.

The authors used a parametric approach to determine the “design space” for various levels of damage and different repair techniques. The XTRACT fiber sectional analysis program was used to determine sectional strength based on AASHTO LRFD Bridge Design Specifications (2010). However, service stresses under damage states were not considered. Rating factors were calculated. However, to avoid a full design of the bridge including calculation of actual AASHTO live load forces, the authors used the calculated undamaged capacity to estimate the live load moments at the damaged section. Strands were removed (beginning from the outboard web-soffit corner) to simulate severed strands.

The authors report that using preformed CFRP strips is preferred over the wet lay-up process. They further note that the ACI 440.2R (ACI 440, 2017) equation for calculating the debonding strain limit in CFRP shows the diminishing contribution of additional CFRP layers. The authors discourage the use of multiple CFRP layers. The authors also point out the possibility that the calculated strength of CFRP repaired beams may be lower than the strength of the unrepaired girder, due to the bending failure mode, which they refer to as “negative strengthening effect”.

The authors also discuss near-surface mounted (NSM-CFRP) applications in which the CFRP material is installed in groves cut into the surface. Although bond strength is improved, but the amount of reinforcement that can be applied is limited. For prestressed girder application, the NSM method could provide up to only 50% of the CFRP material that could be provided using externally bonded CFRP strips, but the cost of NSM would be greater. Therefore, the authors do not recommend NSM applications for positive moment repair of prestressed bridge girders. The advantage of NSM may be in negative moment regions.

The authors also discuss the concept of prestressing the CFRP repair (P-CFRP or PT-CFRP). This would allow an increase in the debonding strain equivalent to the amount of prestressing strain. However, there are significant prestress losses. The authors do not recommend pretensioned (P-CFRP) or unbonded post-tensioned CFRP applications (uPT-CFRP) due to stressing equipment required and potential abrasion or fretting damage due to differential displacement between concrete and CFRP. Proprietary anchorage systems (to concrete substrate) are available. However, the authors suggest that spacing requirements between CFRP strips reduce the effectiveness of PT-CFRP.

The authors discuss internal strand splices. They consider these devices to be effective and well-established. They report ability to develop strengths of up to 96% of the nominal strand strength. Tensioning mechanisms are typically designed to redevelop 60% of the nominal strength. The physical dimensions of the splices pose some limitations with respect to typical spacing of strands. Splices for adjacent strands may have to be staggered. Cover over the splices may be less than allowable for exterior strands. There are also “chuck splices” (diameter of 2 in) that are used to replace damaged strand and allow staggering of splices (Figure 2.6). Strand splices may interfere with stirrups, which may have to be removed and replaced with hairpins. Alternatively, CFRP U-Wraps may be used to restore confinement and shear capacity. Splicing too many strands would be cumbersome because of spacing considerations and interferences (Figure 2.7). The authors recommend combining strand splicing with external repairs when large number of strands are damaged, especially when it is concentrated along a short length of girder. They also note that the splice should be installed on uncorroded strand.

Limits on useable strength reported by various researchers are discussed. These range from 80% of capacity (by Olsen et al., 1992) to 100% by Labia et al. (1996). The authors recommend limiting the capacity to 85% to spliced strands. The authors further recommend that the number of strands that are splices within a girder be limited to 15% of strands even when staggering is used.

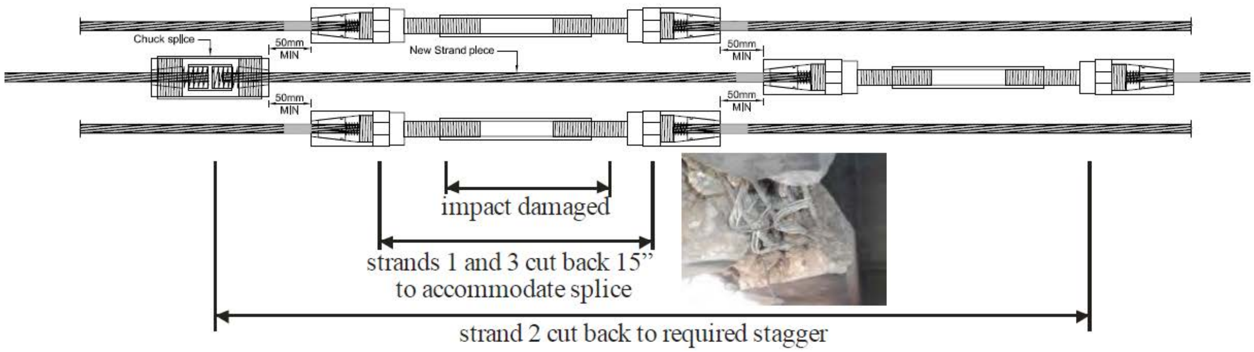


Figure 2.6. Internal strand splices and chucks (Harries et al. 2012 and Prestress Supply 2018)

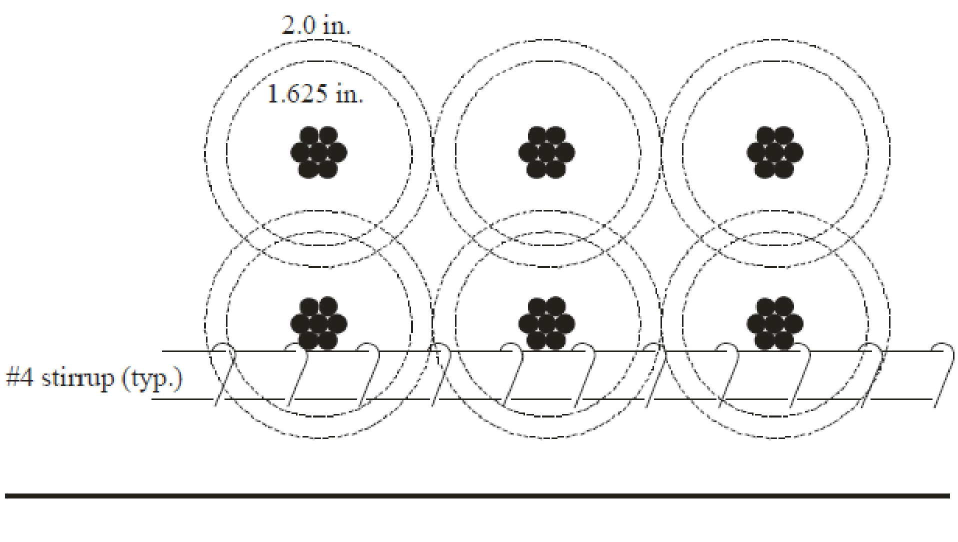


Figure 2.7. Physical interference of internal strand splices (Harries et al. 2012)

In general, the authors do not recommend using splices in adjacent box girder bridges unless used in a few isolated strands. This is due to concerns for limited cover and inconsistent strand spacing in girders built in the 1960s and 1970s. Also, according to the authors, hairpins or U-wraps cannot be used effectively in box girders.

Regarding repair using post-tensioning steel (PT-Steel), the authors note that it could apply to any level of damage and the procedures are well established. The design is concerned with the corbels (bolsters) that are meant to transfer the PT force to the girder through shear friction or direct bearing. They suggest that external PT may not be as practical in box girders.

The authors used the goal of achieving the undamaged strength after repair. However, they note that lower strengths are acceptable, especially in cases of over-strength of the undamaged girder. A list of repair techniques and applicability to various types of bridges is shown in Table 2.1.

Table 2.3. Application of repair techniques to various girder types (Harries et al. 2012)

Repair technique	dominant limit state and/or primary design consideration (Section 2.3.2)	Girder type (Section 2.3.1)		
		Adjacent box (**AB**) and slab bridges	Spread box (**SB**) bridges	I-girder (**IB**) and similar bridges
all external techniques	failure/loss of repair material	$C_D \geq 1.1DC + 1.1DW + 0.75(LL+IM)$		
EB-CFRP	debonding	soffit only: $b_f \leq b$	soffit and web although reduced efficiency on web	soffit and bulb although reduced efficiency on bulb
uPT-CFRP	anchorage of PT forces	soffit only: $b_f \leq \approx 0.5b$		
bPT-CFRP	anchorage of PT forces and debonding			
NSM-CFRP	geometry of slots and debonding	soffit only: $b_f \leq \approx 0.5b$		
PT-Steel	anchorage of PT forces	not appropriate due to vertical clearance encroachment	web or between girders	web above bulb or between girders
strand splicing	geometry/spacing of repaired strands	limited applicability – isolated strands		appropriate

In their parametric studies, the authors reported obtaining better results from applications of CFRP on the beam soffit for adjacent box girders (AB) and separated box girders (SB) compared to I-beams (IB). In AB and SB cases, the maximum use of CFRP on the soffit (without splicing) was able to restore all the pre-damage capacity in damage cases up to 25% loss of strand. However, the efficiency of this approach was far less effective in the IB case, even when the CFRP was partially extended to the two sides of the bottom flange (without strand splices). The strength was recovered only up to a 9% loss of strand. The soffit area is typically much smaller than in AB or

SB bridge girders, and the centroid of steel is farther from the soffit in IB bridges compared to AB or SB. A combination of strand splice and CFRP repair case for an IB bridge was also considered. Five out of 8 severed strands were assumed to have splices applied at 85% efficiency. The allowed restoration of up to 95% of the original capacity.

The authors note that a strand redevelops the prestress at a distance of 60 times its diameter. Therefore, loss of a strand does not affect capacity calculations at distances away from the impact. The authors discuss damage to strands near the supports and consider these as additional unbonded strands. Since AASHTO limits the number of unbonded strands at the end of a beam to 25%, they apply the same limit to the number of damaged strands.

The authors suggest that repair decisions and designs must be made on a case-by-case basis. However, they present a table of maximum strand loss that could be restored (to the undamaged strength – not required strength) for the structures considered in their parametric study and present the number of lost strands that would cause decompression (Table 2.2).

Table 2.4. Maximum number of lost strands that can be restored for considered structures (Harries et al. 2012)

	AB Prototype	SB Prototype	IB Prototype
girder detail	42 x 48 in. box girder (BIV-48) with 3 in. composite deck	42 x 48 in. box girder (BIV-48) spaced at 8'-9" with 7.5 in. composite deck	40 in. C-girder spaced at 7'-3" with 7 in. composite deck
prestressing strands	57-3/8" 250 ksi strands	68-3/8" 250 ksi strands	32-0.5" 270 ksi strands
strand loss to cause decompression	≈12 strands (21%)	≈20 strands (29%)	≈8 strands (25%)
EB-CFRP	$RF_R = 0.69$ 13 strands (23%)	$RF_R = 0.61$ 18 strands (26%)	$RF_R = 0.84$ 3 strands (9%)
bPT-CFRP	$< RF_R = 0.40$ 23 strands (40%)	$< RF_R = 0.44$ 26 strands (38%)	$< RF_R = 0.84$ 3 strands (9%)
NSM-CFRP	NSM not considered practical		$RF_R = 0.95$ 1 strands (3%)

The authors used an approximate approach using assumed position of the centroid of compression force and the centroid of prestressing steel to estimate the maximum number of strands that could be replaced with CFRP for different types of cross sections (Table 2.3).

Appendix A of this report included a "Guide to Recommended Practice for the Repair of Impact-Damaged Prestressed Concrete Bridge Girders." The extent of damage was classified into six categories: Minor, Moderate, Severe I, Severe II, Severe III, and Severe IV. The description of each proposed damage category is given by the authors in Table 2.4.

The authors consider the transition from Severe III to Severe IV as the threshold between repair and replace decisions. In addition, they recommend that the following conditions be additional bases for replacement of girder in lieu of repair (Shanafelt and Horn, 1985):

- Permanent lateral deflection (exceeding standard girder tolerance specified in NCHRP 280)
- Permanent vertical deflection (more than 0.5% of span)
- Open cracks at the web/flange interface (yield of transverse steel)
- Loss of prestress at harping point
- Loss of prestress at girder ends exceeding 25% of the total number of strands
- Damaged girder strength that is below the ACI 440.2R (ACI 440.2R, 2017) lower limit for applicability of FRP repairs, unless strand splices are used such that the girder strength is raised above the ACI 440.2R limit.

The authors' proposed repair flow charts for I-beams and box girders are shown in Figures 2.8 and 2.9, respectively. It is pointed out that application of U-wrap for confinement is not practical in the case of box girders. In all cases, however, it is noted that assessment must be on a case-by-case basis.

The authors express concern regarding application of repairs under traffic loading. They suggest that traffic restrictions be implemented during the initial curing of the epoxy (for CFRP), such as closing the outside lane to protect the repair for the damaged exterior girder. They further recommend reduction factors for strength, modulus, and bond capacity (debonding strain) (0.9 for all three in case of CFRP strips). However, ACI 440.2R, while including a factor of 0.85 for CFRP strength and ultimate strain, it does not apply it to the modulus or debonding strain. The authors exclusively considered the CFRP strips, which have much more fiber content per laminate thickness compared the fabric alternative.

The authors report that the externally bonded CFRP systems can deteriorated when subjected to fatigue loading. Although ductility could be reduced, the authors state that it would not be a significant concern for prestressed concrete girders. The choice of correct adhesives can mitigate some of the fatigue effects. Soft and stiff adhesive would be suitable for high and low fatigue stress ranges, respectively. Stiffer adhesives would be preferable for stress transfer considerations. The authors conclude that, except in cases where fatigue effects are dominant, EB-CFRP systems that are designed and detailed well would perform well.

It is noted that the dominant failure mode is debonding of the CFRP. Therefore, applications of small amounts of CFRP may in fact reduce the nominal strength of the cross section. The authors discuss some of the methods presented in NCHRP 280 (Shanafelt and Horn, 1985) and conclude that steel jackets are not recommended as they are cumbersome and would require field welding and grouting. Post-tensioned repairs using steel bars could be applicable to all levels of damage but may be more suitable in I-girders than in box girders.

Table 2.5. The estimated highest number of strands that could be replaced with CFRP in typical sections (Harries et al. 2012)

Shape (PCI 2003)		H (in.)	b (in.)	typical strands[3] (PCI 2003)	equivalent 3/8" 250 ksi strand[4]					
					EB-CFRP	**bPT-CFRP**		**uPT-CFRP**		**NSM-CFRP**
					n_{max}	n_{max}	$n_{max\text{-}PT}$	n_{max}	$n_{max\text{-}PT}$	n_{max}
AASHTO I-girders	I	28	16	20	6	13	10	10	6	7
	II	36	18	30	7	15	11	12	7	8
	III	45	22	50	8	19	14	14	8	9
	IV	54	26	66	10	22	16	17	10	11
	V	63	28	80	11	24	18	18	10	12
	VI	72	28	80	11	24	18	18	10	12
	C[1]	40	22	50	8	19	14	14	8	9
Boxes	B-36	27, 33, 39 & 42	36	34	13	28	26	21	15	13
	B-48[2]		48	46	17	37	34	29	20	18
Bulb Tees	BT	54, 63 & 72	26	40	10	22	16	17	10	11
Deck Tees	-	35, 53 & 65	25	32	9	20	17	16	10	10
Slabs	S-36	12, 15, 18 & 21	36	17	13	27	26	21	15	13
	S-48		48	23	17	37	34	29	20	18

[1] IB Prototype
[2] AB and SB prototype
[3] Typical number of strands in a section; taken as:
maximum number of strands that may be located in confines of the bottom flange or bulb;
maximum number of strands in one layer for slabs; or
[4] to convert tabulated values to…
equivalent 3/8" 270 ksi strand, multiply tabulated value by (0.080/0.085)(250/270) = 0.87
equivalent ½" 250 ksi strand, multiply tabulated value by (0.080/0.144) = 0.56
equivalent ½" 270 ksi strand, multiply tabulated value by (0.080/0.153)(250/270) = 0.48
equivalent 0.6" 250 ksi strand, multiply tabulated value by (0.080/0.216) = 0.37
equivalent 0.6" 270 ksi strand, multiply tabulated value by (0.080/0.215)(250/270) = 0.34

Table 2.6 Classification of damage proposed by Harries et al. (2012).

		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 (*NCHRP 280*).	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 (*NCHRP 280*).	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 [2.5]. 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 affected 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 [2.6.1.2]. Damage requires structural repair involving 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%

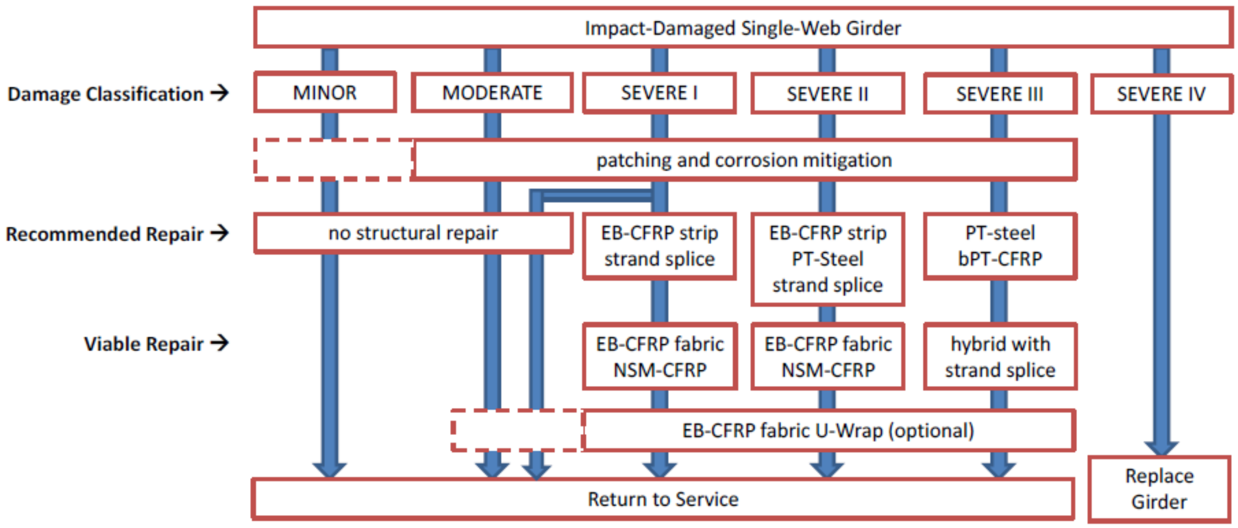


Figure 2.8. Selection of repairs for damaged prestressed I girders (Harries et al. 2012)

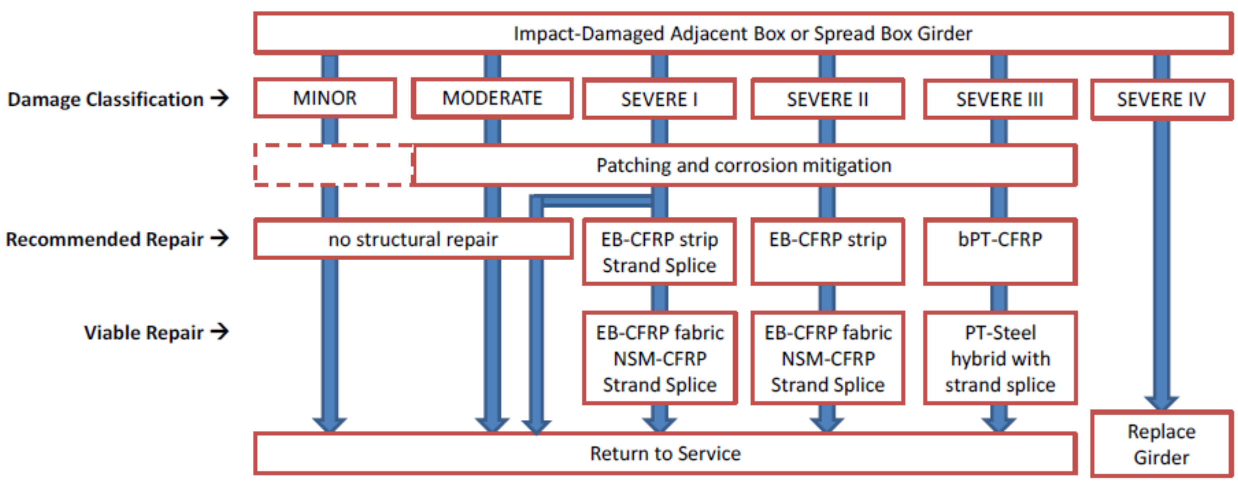


Figure 2.9. Selection of repairs for damaged prestressed box girders (Harries et al. 2012)

Strand splices are considered effective by the authors. Splicing of adjacent strands requires staggered splices. Chuck splices can be used to shift the splice location. Installation on exterior strands would reduce the cover below minimum required and may require removal of transverse reinforcement. Hairpins and U-wraps may be used to recover confinement and lost sections. They suggest that splicing is not practical for large number of severed strands, especially in adjacent strands. Corroded strands should be removed (possibly corroded while left exposed after damage) before installation of splice. The authors recommend the following regarding splices:

- Use for strand diameters ≤ 0.5 in.

- Limit strength to 0.85fpu
- Stagger splices for adjacent strands.
- Limit number of spliced strands to 15% of total strands in a girder

The authors do not recommend use of splices in box girders, especially older box girders inconsistent spacing of strands and cover depths may be encountered.

Regarding preloading, the authors preloading to restore prestress force in strands is impractical but preloading to induce compression in patch materials is recommended. Overloading of the structure with the preload should be avoided.

Regarding inspections, the authors believe that visual inspection is the only practical method for initial assessment of damage in impacted prestressed bridge girders. Photographs are highly recommended and crack maps including crack widths are required. Sounding methods may be used to identify spalled areas. Non-destructive testing methods may be used to further examine areas identified through visual inspection. The following visual indications are sought:

- exposed or corroded or severed strands
- cracks - longitudinal and transverse
- spalling of concrete
- efflorescence
- corrosion stains
- evidence of water/leakage
- longitudinal cracks on deck
- evidence of displacement between beams
- out-of-plumbness of web
- relative dislocation of girder from bearing
- shear or flexure cracking

Regarding patching, the authors refer to the PCI Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products (PCI MNL, 2006) for guidance on patching. All unsound concrete must be removed. Chipped areas must be at least 1 in deep with straight edges that are perpendicular to the surface. Pneumatic chipping guns and power saws may be used, but care should be exercised to not damage strands or other reinforcement. The size of the patch area, rheology of patch material, bond strength to existing concrete and steel, and strength and durability of patch are important factors in selecting the patch material. CFRP U-wraps may be used to confine the patch material and avoid “pop-out” failure.

Finally, the authors provide a list of repair selection criteria, a part of which is reproduced here in Table 2.5.

2.1.14 El Meski and Harajli (2013)

The authors performed an experimental study on the effect of FRP external reinforcement on the strength and deformation capacity of internally unbonded and bonded prestressed concrete beams and slabs as well as companion reinforced concrete members. Beam cross sections were 150 mm wide by 250 mm deep (18 specimens), and slab sections were 360 mm wide and 120 mm deep (18 specimens). All specimens had a span length of 3 m. In each group 12 specimen were unbonded, 3 bonded and 3 reinforced concrete members. Control and FRP-strengthened specimens (up to two layers) were used. CFRP was attached to the soffit of beams and slabs. Cyclic loads (30% to 70% of nominal strength) were applied before and after CFRP installation.

Failure of bonded and unbonded specimens included concrete crushing as well as rupture and debonding of FRP. All FRP-strengthened specimens experienced brittle failures. Measured strain in FRP at failure ranged from approximately 0.0046 to 0.0078. Two unbonded control specimens developed strains near yield at failure, but none of the FRP strengthened unbonded specimens reached yield. Bonded control and FRP-strengthened beams (except one) reached yield at flexural failure. All reinforced concrete specimens exceeded yield at far higher strains than the corresponding unbonded or bonded specimens.

The addition of FRP was able to increase the strength substantially. It also increased the post-cracking stiffness of specimens. There was a large reduction in deflection at peak load for FRP-strengthened beams compared to those without FRP.

Table 2.7. Repair selection criteria for damaged prestressed girders (Harries et al., 2012)

Selection Criteria	EB-CFRP strips		EB-CFRP fabric[1]		NSM-CFRP[2]	
primary reference	this report		this report		this report	
commercially available?[7]	yes		yes		yes	
girder type	box girder	I-girder	box girder	I-girder	box girder	I-girder
generally recommended?	yes	yes	no	yes, if bulb to be wrapped	no	no
dominant repair limit state	CFRP bond		CFRP bond		NSM slot geometry and CFRP debonding	
damage that may be repaired	Severe I and II		Severe I		Severe I and II	
prestress steel that may be replaced	≤20% of strands	≤10% of strands	≤20% of strands	≤10% of strands	≤20% of strands	≤10% of strands
active or passive?[8]	passive		passive		passive	
behavior at ultimate load	good		fair		good	
resistance to overload	limited by bond		limited by bond		good	
fatigue performance	limited by bond[9]		limited by bond[9]		uncertain	
strengthening beyond undamaged capacity?	yes		yes		yes	
combining splice methods	possible with strand splicing		possible with strand splicing		unlikely	
preload for repair[10]	no		no		no	
FRP U-wrap[10]	not feasible	recommended[11]	not feasible	recommended[11]	not required	
restore loss of concrete	patch prior to repair		patch prior to repair		patch prior to repair	
preload for patch[10]	possibly		possibly		yes	
speed of mobilization	fast		fast		moderate	
constructability	easy		easy		difficult	moderate
specialized labor required[13]	no		yes		no	
proprietary tools required	no		yes, saturation bath		no	
lift equipment required[14]	no		no		perhaps	
closure below bridge	single lane possible		single lane possible		full carriageway closure	
time for typical repair	1-2 days		2-4 days		2-4 days	
environmental impact of repair process	dust from surface preparation		VOCs from saturant and dust from surface preparation		dust from concrete sawing	
durability	requires environmental protection		requires environmental protection		excellent	
cost	low		low		moderate	
aesthetics	good		good		excellent	
retain capacity in event of subsequent impact	good		very good		very good	

2.1.15 Dominiguez (2014)

A case study of the evaluation and repair of a 46-in-deep impact-damaged bridge girder in Dallas, Texas is presented. Various repair methods are discussed. There were no severed strands involved. Removal of loose concrete, cleaning and patching was performed in addition to epoxy injection of cracks. A load test was performed using a 90-kip truck and strains were measured at several locations on the damaged and undamaged areas of the beam. A computer model was also generated. It was concluded that the girder stiffness was not recovered based on deflection readings before and after repairs, which was attributed to internal cracking. Nonetheless, the author considered the repair to be effective.

2.1.16 Montero (2015) and Yazdani and Montero (2016)

The objective of this study was to understand the behavior of prestressed beams repaired using GFRP bars. Analytical models of the girder before and after repairs were developed and compared field test results. The repair procedures involved removing the damaged concrete, chipping the concrete surface to achieve a rough surface, drilling concrete to place four transverse and longitudinal GFRP bars in them, applying a bonding agent and then patching the area. The damage was characterized as moderate without any damaged strands (i.e. no strength degradation). Short #4 GFRP dowel bars were used at 6 in. spacing across the boundary between old and new concrete. The author concluded that there was good agreement between analytical model and experimental results (field load test).

2.1.17 Liesen (2015)

Six flexural tests were performed on four Type III prestressed girders that were intentionally damaged and repaired. The first girder was undamaged, the second girder had eight severed strands that were internally spliced, the third girder had four severed strands and repaired with FRP, and the last specimen had four severed strands that were repaired using internal splices. Only 88% of the nominal strength of the girder with eight internally spliced strands was restored. The author concludes that splices alone would not fully restore the original strength but can be a valid repair method when the percentage of lost strands is low. FRP would be a better repair approach when additional strength is required. The author further makes repair recommendations including application of preload and proposes extending FRP for the entire length of beam to reduce risk of debonding. Future consideration of the effects of cross sectional parameters is proposed.

2.1.18 Jing et al. (2016)

The dynamic impact against a prestressed bridge girder was simulated in a test facility using an impact car on an elevated track that would hit the bottom of the AASHTO Type I girder (56 ft long) after gaining speed along a rail track with a maximum height of 11 ft. The impact car was a 50 ft^3 concrete block on a steel frame with castors and a 10-in cube as impactor. Steel plates covered the four sides of the impactor. The weight of the impact car was 9000 lbs. Accelerometers, potentiometers, and strain gages were monitored at high sampling rate. The failure mode is a punching shear around the impact zone.

An explicit 3-D finite element model (ABAQUS Explicit) is generated. The maximum impact force was determined to be 1039 kips. The peak force is reached in 0.5 ms. The concrete spalling could not be modelled in the FE model because of lack of element removal after failure.

2.1.19 Gangi (2015) and Gangi et al. (2018)

The author describes a research study to evaluate three different repair techniques: internal strand splices, externally applied FRP, and externally applied fabric reinforced cementitious matric (FRCM). Four AASHTO Type III girders were tested. One served as control while the others were intentionally damaged and repaired. Three-dimensional finite element models were generated to predict beam behavior during tests.

FRCM is a relatively new technique (compared to FRP) that includes a fiber grid and a cementitious matrix. The biaxial nature of the grid (primary and secondary directions) provides strength in both directions. ACI committee 549 (ACI 549, 2013) makes recommendations regarding externally bonded FRCM repairs.

Four girders were removed from a bridge that was being replaced due to previous impacts. The girders obtained included a deck with a narrow width equivalent to the width of the girder. Either 4 or 8 strands were severed out of the 50 Grade 250 strands (3/8” diameter).

The author report that the failure stress in splices depends on the number of strands that are spliced. They report a maximum stress of 225 ksi (90%) for an 8-stand splice, and 250 ksi (100%) for a 4-strand splice. The author further recommends not using strand splice repairs on the higher level of damage (8 strands). FRP repairs restored the least ductility. FRP repair on 4-severed-strand beam restored 100.6% of strength but only 11.6% of ductility because of debonding failure of FRP. The least ductile repair was reported for the FRCM repair. concluded that FRP overlays restored the most strength, while strand splices restored the most ductility.

2.1.20 State Practices

A review of various bridge design and maintenance manuals was performed. Only a few address repairs that would inevitably have to be completed during the lifespan of a bridge. Approximately ten states address these concerns, but the details are often vague. Most of these states discuss the need to repair cracks and spalling by epoxy injection and patching, respectively. These types of repairs are mentioned by states such as Indiana, Nebraska and West Virginia without further detail. Delaware provides specifications about the minimum strengths that epoxy, or concrete patch materials, must attain.

Pennsylvania (PennDOT, 2015), Washington (WSDOT, 2016) and Utah (UDOT, 2015) are among the few states that provide an outline of the procedures they use to assess damage and proceed with repairs. This generally consists of visual inspection, structural evaluation based on judgement and calculations and then determining the best repair method. Pennsylvania details a comprehensive program for analyzing and rating the conditions of pre-stressed concrete bridges that involves a PennDOT computer program.

Some states indicate methods of repair such as strand splicing, external post-tensioning and fiber reinforced polymer (FRP) wraps. Nevada (NDOT, 2008) suggests the use of FRP wraps when shear strength is inadequate but does not define what is considered inadequate nor gives specifications for FRP installation. Indiana (INDOT, 2013) advises the use of wrapping, but only to protect against deterioration, not to increase bridge capacity. Utah (UDOT, 2015) suggests using FRP wraps to cover repairs and to strengthen. Washington (WSDOT, 2016) provides details about strand splicing, but also notes that they may not always be practical due to space limitations. It advises girder replacement if more than 25% of the strands need splicing.

Washington DOT had earlier (2007) issued a memo providing specific criteria regarding when a prestressed girder must be replaced and when it would require further evaluation (Khaleghi, 2007). They determined that girders must be replaced when one of the following conditions are met: 1) more than 25% of the prestressing tendons have been damaged or severed; 2) when the girder has significant displacement; or 3) when there is significant concrete damage at the harping point or girder ends. It also explained that the repair limits for strand splicing was related to the fact that only a certain number of couplers could fit into the damaged area. When replacement is not automatically warranted there are other considerations for further analysis. These include the load capacity of the adjacent girders, the damage history of the newly damaged and the adjacent girders and the cost of repair vs replacement. Replacement may be recommended if repair exceeds 70% of the replacement cost.

Iowa's Bridge Maintenance Manual details step by step directions for various repair procedures of damaged prestressed beams (IDOT, 2014). For cracks they outline the procedure for using epoxy injection. For more significant damage due to vehicle impact they outline procedures for repairs with or without FRP.

2.2 Literature related to damage to top flange of prestressed girders

2.2.1 Tadros and Baishya (1998) (NCHRP 407)

This report summarizes a study performed to evaluate existing and new methods to rapidly replace bridge decks. This included consideration of demolition equipment and procedures as well as the design of the deck and the deck-girder connections. A set of special provisions were recommended for deck removal. The report also discussed the beneficial effect of reducing deck reinforcement and use of welded wire fabric in facilitating future deck removal. Finally, a new continuous precast prestressed stay-in-place system was proposed. Evaluation of deck removal and replacement procedures was an important objective of the study.

They considered the need to improve the connection system between the girder and deck to allow more raid demolition. Two connection systems were proposed, including one for concrete girders. For the concrete girder, a "debonded shear key system" and a set of deformed steel connectors were proposed.

The authors report that most common equipment used for removal of bridge decks on concrete girders are boom-mounted breakers, saws and hand-held hammers. Other common methods include “water jets, pressure bursting, crane and ball, and blasting.” Less common methods include “mechanical hammers, planning machines (grinders), shot blasting, roto-mills, and whiphammers.” All these methods have their associated problems. The authors report that boom-mounted breakers generate substantial noise and vibration, and both the boom-mounted breakers and hydraulic jaws can damage the top flange. Sawing can damage the top flange and requires a continuous flow of coolant. The hand-held hammers are labor intensive and thus expensive. Deck removal by water jet is reportedly expensive and environmental issues with run-off water must be addressed.

Appendix P of the report provides suggested special provisions for removal of existing bridge decks. These provisions offer the contractors the freedom to choose the equipment and methods without “compromising structural and environmental concerns”. The authors note that the concrete flange offers very little in terms of positive moment strength in a composite arrangement with the slab, thus localized loss of the top flange during removal would not have any practical effect on the composite slab-girder flexural strength if “full composite connection is preserved”. The authors state that “experimental studies have demonstrated that concrete girders with up to 50 percent of the top flange width damaged at the maximum positive moment location caused no noticeable structural deficiency… Thus, accidental damage to the top flange should be permitted if it would expedite deck removal. The extent of damage should be limited to that corresponding to the maximum horizontal shear stress on the concrete interface, limited according to AASHTO LRFD specifications by the lesser of 0.2 f’c and 800 psi (5.5 MPa)”. However, relevant reference or references for the subject experimental studies were not identified. The authors point out that the girder top flange also perform other functions, such as providing stability during erection. However, issues related to service stresses in a prestressed girder damaged at the top flange, or lateral stability of the damaged girder after full removal of the old deck was not discussed in the report.

2.2.2 Badie and Tadros (2000)

The authors implemented two new connection systems that were developed in an earlier NCHRP 407 (1998) study, one of which was for prestressed girder connections and the other for steel girders. This involved a debonded shear key system in lieu of the conventional roughened top surface of the flange (Figure 2.10). The interface shear reinforcement was epoxy coated.

The authors concluded that the debonded shear key system was a "competitive replacement" for the traditional roughened surface and would minimize damage to top flange of NU girders during deck replacement operations.

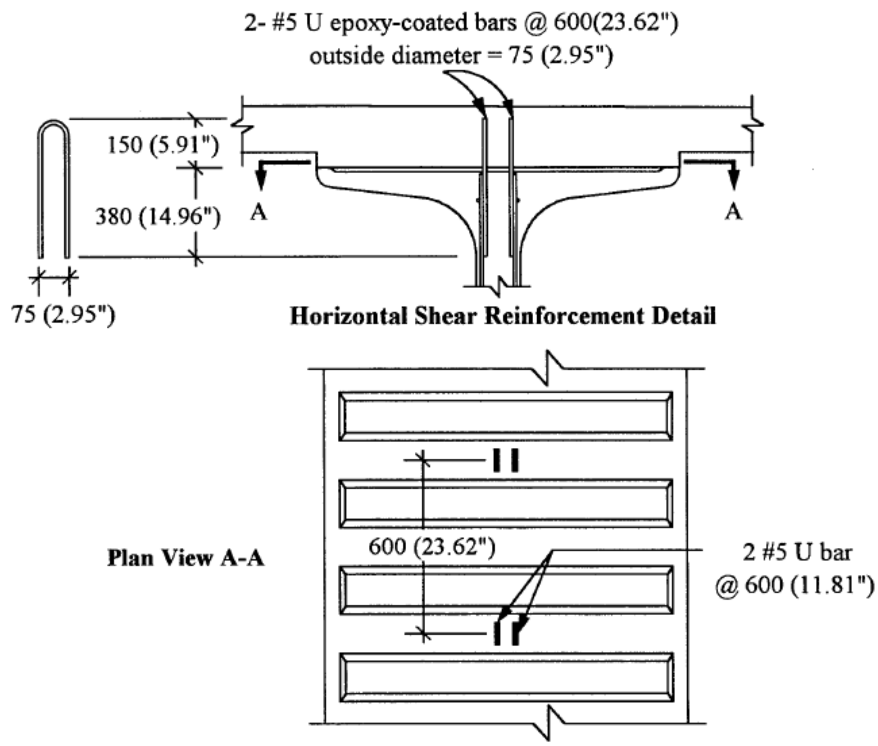


Figure 2.10. Debonded shear key system (Badie and Tadros, 2000)

The authors performed a comprehensive study on damage to top flange of prestressed I-girders during deck removal. Although the wider top flange in modern I-girders provides for a more efficient section, reduces the deck slab span, and provides a wider work platform for construction workers, it is also more susceptible to damage during removal of bridge decks. The author reports on a lack of available guidelines for the problem of damage to top flange during deck removal. Factors related to two methods of deck removal are discussed in detail: saw cutting and jack hammering. These factors include extent of damage to girder, cost, and environmental impact. Different removal techniques were implemented on a deck removal project in the field. Subsequently, two girders were removed from the bridge and brought into a laboratory for further testing. Deck removal and re-decking was followed with load testing. Different levels of deck removal around shear connectors were tested. When deck was not fully removed from around the shear connectors, the strength was still found to be adequate. The effect of reducing the width of top flange by 50% (through saw cutting) was studied experimentally and analytically. The author concluded that "in some cases top flange width does not have significant impact on the structural performance of I-girders."

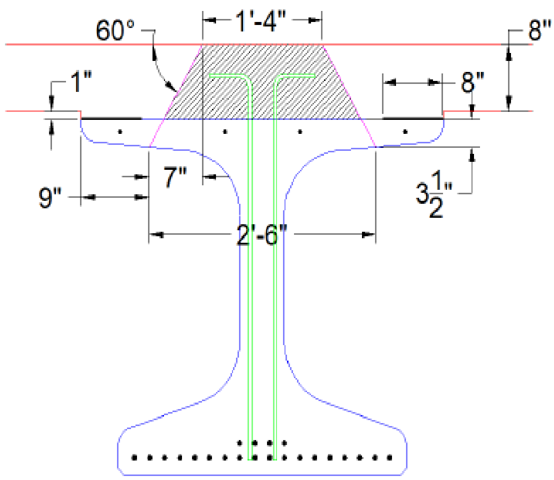
The author reports that the Alberta Ministry of Transportation limits the size of jack hammers to less than 14 kg, and chipping hammers to less than 7 kg when full deck removal projects are involved. According to a survey conducted by the author, most respondents use saw-cutting between girders. After removing the concrete segments, jackhammering is used to remove the remaining parts of the deck over the girder. Four out of the 10 respondents use hydro-demolition. They report a low risk of damage, but noise, cost and control of water are issues that must be considered. Some states use pneumatic hammers attached to mini-excavators or backhoes for removal of the top half of the deck. However, the risk of major damage to the top flange cannot be ignored. The remaining bottom part of the deck would then be removed using hand-held chipping hammers or small jackhammers. Pennsylvania reports an estimated cost of $600-$700 per cubic yard when a combination of pneumatic hammers and hand chipping is used. This cost would increase to $900-$1000 per cubic yard with hand chipping alone is used. Two states (Minnesota and Oregon) recommend using a debonded strip (6 in or 8 in) at the edges of the top flange to facilitate future deck removal. Debonding is also mentioned in Missouri's response to facilitate deck removal. Minnesota suggested using steel troweled finish on the top of the flange in addition to the debonding strip. Texas suggests using 15-20 lbs. pneumatic hammers below the top deck reinforcement over the girder. Florida suggests using small jack hammers or hydro-blasting depending on the cost involved. Florida suggests sloped saw cutting over the tip of the flange to allow support of the deck until it is removed.

The author also reports on the survey performed by the Iowa State University (ISU) on deck removal on concrete and steel bridges. Twenty-eight states responded to the ISU survey. Results indicate that hydro-demolition has the lowest risk of damage to top flange, but it is considered to be more costly than other methods.

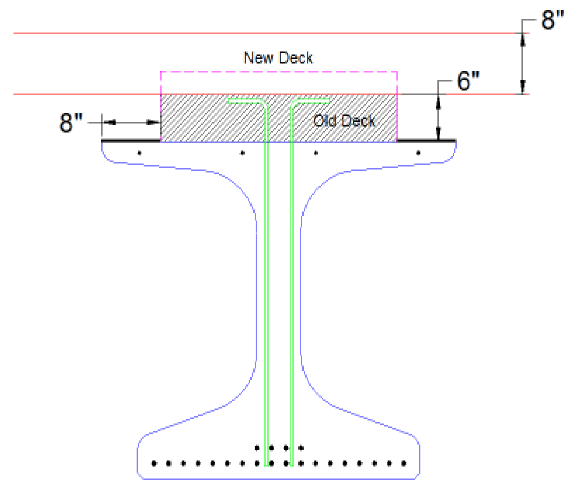
The author further reports on ISU tests of shear capacity of shear connectors with varying levels of concrete removal from around the connector (50%, 75%, and 100%). Headed shear studs, C-channel connector and steel angles were considered. There was no correlation between the level of concrete removal and the connector response under load. It was concluded that the extent of concrete removal around the connector would not affect the behavior of the connection under load.

The author discusses four possible deck removal methods as illustrated in Figure 2.11. Method 1 involved saw cutting at an angle using a blade that can pivot and cutting through the top slab and the flange as shown. The shaded area is then jack hammered and removed followed by placement of the new deck. In the second method the top of the deck is milled 2-3 inches, saw cuts are made, and the deck panels are removed. The remaining old deck is kept, and a new deck is placed above it, thus raising the elevation of the top of the deck by 5-6 inches. In the third method, saw cuts are made at the edge of the debonded zones, and a mini-excavator is used to remove the concrete over the girder, and a new deck is placed. In the last method, transverse and longitudinal cuts are made around all shear connectors using small jack hammers or manual hydro-blasting. Then the remaining deck is removed (it assumes that the crane lifting the old panel should be able to break the remaining bond over the top flange), and a new deck is cast.

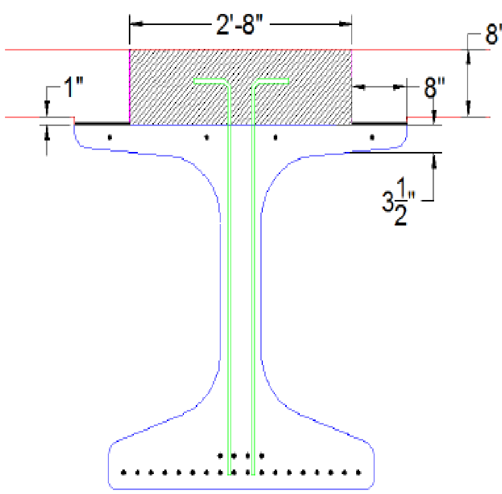
The author notes that the cost (2016) cost of “break and fall” approach (if there are no environmental restrictions) would be on the order of \$0.99 per square ft. On the other hand, the saw-cut and lift method would cost approximately \$3.16 per square ft. The jack hammering method would cost on the order of \$15.73 per square ft.



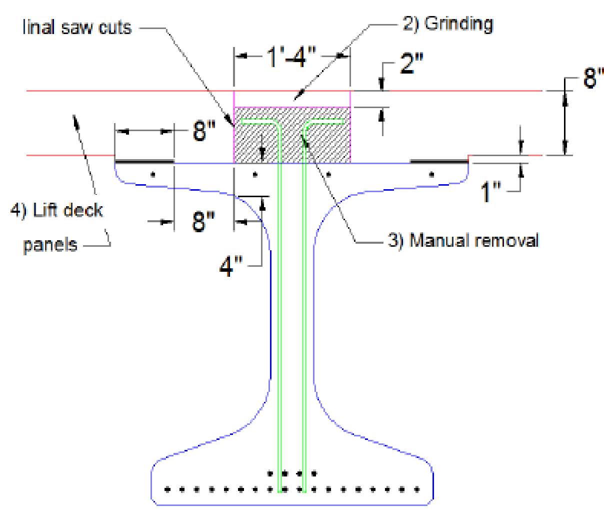
Method 1: Sloped saw cut



Method 2: New elevated new deck



Method 3: Saw cut at edge of debonded zone



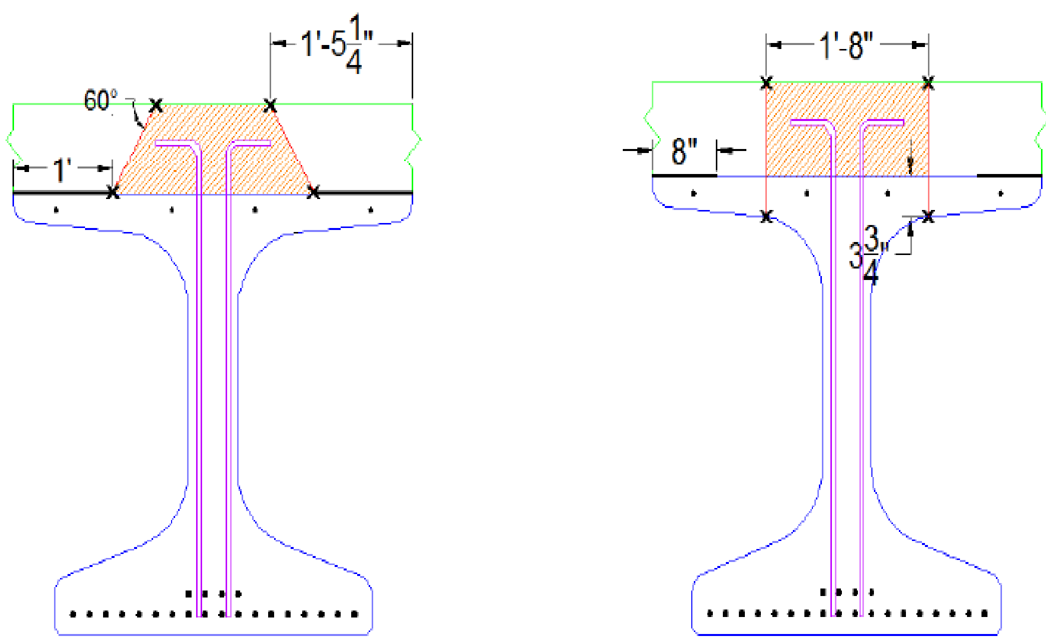
Method 4: Saw cut outside shear connectors

Figure 2.11. Four methods of deck removal tested (Assad 2016)

The author considered remaining moment strength (non-composite and composite) on example bridges in which a total of two feet of the top flange (1 ft on each side) was removed. The analyses indicated strength reductions of 8 and 20% for the non-composite section, but no reductions in the composite flexural strengths. In some cases, service checks for compression exceeded the allowable stresses. The author further studied deck removal methods on the Camp Creek Bridge over I-80 in Lancaster County, Nebraska, and investigated the extent of damage to the girder as well as cost, duration, and environmental impact of each method. The author took two girders to the lab to establish the structural properties of the girder after deck removal.

The author concluded that debonding the edges of the top flange was an effective way for lifting deck panels that were saw cut. The author also concluded the effect of cutting 50% of the top flange on structural performance depended on the span to depth ratio (negligible in low ratio and significant in high ratio cases). He suggested to check the flexural capacity, horizontal shear capacity and deflections during construction and the strength limit state condition.

Finally, the author recommended a method for new construction involving increased debonding length and 60-degree sloped cuts (Method A in Figure 2.12), and a method for existing bridges with low span-to-depth ratios involving vertical cuts through the deck and flange, and removal of the remaining concrete using 60-lb jack hammers above the shear connectors, and 30-lb hammers below the shear connectors (Method B in Figure 2.12).



Recommend Method A: 1-ft-wide debonding width and sloped cuts

Recommended Method B: Vertical cuts through deck and flange and jack hammer removal

Figure 2.12. Recommended deck removal methods (Assad 2016)

2.2.3 Consolazio and Hamilton (2007)

The authors evaluated the lateral stability of long-span prestressed concrete bulb-tee girders while supported on neoprene bearing pads during construction. Analytical models were used to estimate buckling capacities for different girder sections, span lengths, and skew angles. The authors concluded that the buckling strength was greatly influenced a combination of skew angle and girder slope. The following equation is proposed for calculating the girder buckling capacity:

$$C = \frac{\sqrt{EI_yGJ}}{L^3} 24(1.0 - 0.0017L)(1.0 - 0.006\theta)$$

Where C is the maximum uniformly distributed load applied on the girder (kips/ft), E and G are the modulus of elasticity and shear modulus of concrete, respectively (ksf), L is the span length (ft), θ is the skew angle (degrees), J is the torsional constant ((ft^4), and Iy is the moment of inertia with respect to the weak (y) axis (ft^4).

2.2.4 Phares et al. (2014)

The authors conducted a literature review, surveys, interviews, workshops, and small-scale tests to explore cost-effective, reliable and sustainable bridge deck removal methods. Small-scale tests addressed hydro-demolition, chemical splitting and peeling methods. According to the authors, peeling is a relatively new method that involves application of vertical forces on the deck to separate it from the girder. This method uses "an excavator, a slab crab, and machine mounted bucket attachments." The authors state that despite the relative speed of this method, other issues such as vibration, noise, dust, and falling materials exist. Survey results indicated that ten of the 28 state DOTs that responded specify deck removal procedures as special provisions.

The authors note that current deck removal techniques often result in damage to the girders, and generate concern due to noise, vibration, dust, and falling debris.

The authors concluded that hydro-demolition was suitable for partial and full-depth deck removal applications, but the containment and treatment of the used water would be costly. Chemical splitting did not sufficiently break the concrete. Although peeling appeared to be effective, further tests are needed for verification. It is not necessary to remove all concrete from around the interface shear reinforcement to maintain their effectiveness in a new deck.

2.2.5 Potisuk et al. (2016)

This article discusses an unusual deck replacement project during which the super-elevation of the bridge was reversed in Oregon. The deck and flanges of bulb-tee girders were saw cut and the web was extended to achieve the new super-elevation (Figure 2.13.) A sweep of up to 1 in. was noted on the exterior girder after the deck was cut, which “was stabilized by the existing intermediate diaphragm in each span.” Cracks were also noted at the bottom of the girders near support, but these cracks closed after placement of the new deck slab. Cracks that developed on the webs of the extended girders near supports were epoxy injected. The authors considered the crack issues to be minor and suggested that restraining elements be provided to avoid girder sweeping when the deck and top flange are removed.

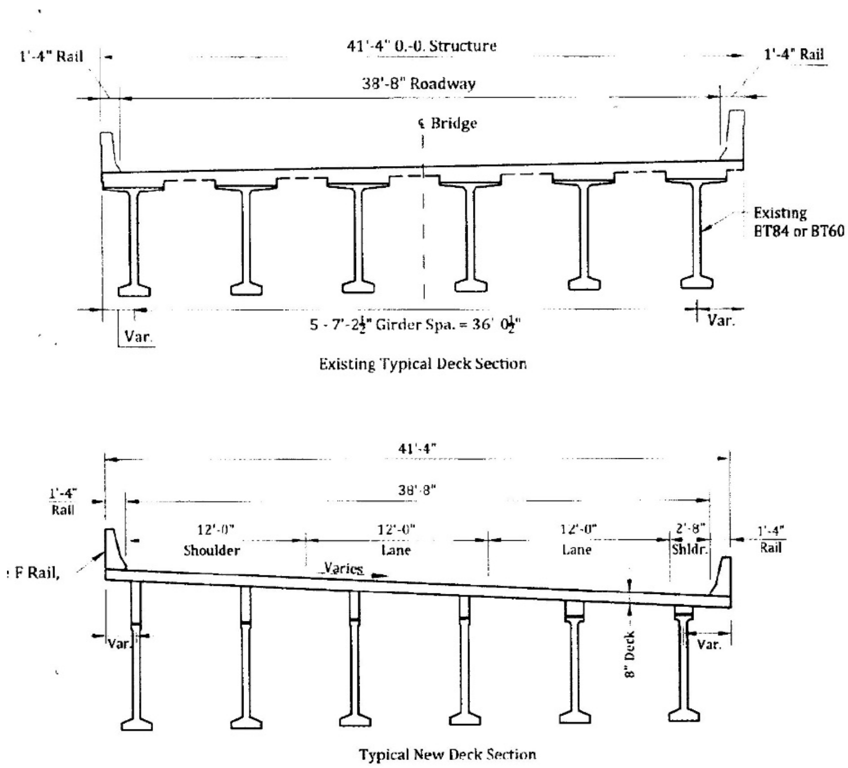


Figure 2.13. Deck removal and raising the height of prestressed girder (Potisuk et al., 2016)

This research addresses the problem of removal of decks on modern prestressed I-girders with wide top flange. Various top flange surface treatments, bond breakers, interface shear reinforcement type, strength and amount were tested in push-off tests to find a combination of parameters that would maintain the composite action but facilitate deck removal. In addition, tests on full size NU (Nebraska) prestressed girders performed. Deck was placed and then removed to assess extent of flange damage. The deck was recast followed by fatigue and static tests to failure.

The proposed connection includes a top flange surface that is partially debonded and partially roughened, which reported results in 2/3 reduction in deck removal efforts.

The authors conclude that the use of roofing felt along the two top edges of flange would reduce the deck removal effort and potential for damage to girder flange substantially. The authors further suggest setting the maximum cut depth of saws to ½ in less than the deck thickness (over the girder) and using reinforcing bar locators (such as GPR-based) to locate interface shear reinforcements, or eliminating transverse cuts through the deck over the girder where interface shear reinforcement is located. The authors further suggest that the contractors use adhesive to bond the roofing felt in place and avoid movement during construction.

The following two deck removal procedures are recommended by the authors:

<u>Procedure A: For Girders with a Roughened and or Troweled Top Flange (Li et al., 2017)</u>

1. *"Perform a series of saw-cuts between girders transverse to the girder axes to create a series of panels to facilitate lifting and disposal. Through thickness cuts are appropriate when clear of girders. When near to the girder top flanges, limit the saw-cut depth to 0.5 in. less than the deck thickness.*
2. *Using a crane or other piece of lifting equipment in tandem with saws, separate the ends of each panel from the deck concrete over the girders and lift clear for disposal.*
3. *Use 30-lb demolition hammers to remove the concrete over the bridge girders down to the level of the bottom layer of deck reinforcement. The 30-lb demolition hammers should be used at an angle not exceeding 45 degrees from horizontal.*
4. *Once the bottom layer of deck reinforcement is exposed, 15-lb demolition hammers should be used to remove the remaining concrete and install a roughness in compliance with project specifications. Where the deck concrete immediately over the flange is sound, it may not be necessary to fully remove all deck concrete. In this study, it was observed that the deck and girder concrete form a strong bond where the concrete was roughened prior to casting of the deck that makes complete removal of deck concrete difficult. The judgement of the engineer should govern the extent of removal necessary considering the condition of the system and risk of damage to the underlying girders."*

Procedure B: For Girders with Roofing Felt Placed over the Top Flange Except for Directly over the Web (Li et al., 2017)

1. *“Locate the centerline of each girder and the extent of horizontal shear reinforcement (the extent of shear reinforcement can be estimated from design documents or determined with GPR-based rebar locating equipment). Clearly mark two longitudinal lines on the deck demarcating the likely extent of horizontal shear reinforcement. Perform longitudinal saw-cuts (along the longitudinal lines) to a depth of 0.5 in. less than the deck thickness.*
2. *a) Perform a series of saw-cuts transverse to the girder axes to create a series of disposable panels. Care should be taken to avoid contact with horizontal shear reinforcement. Through-thickness cuts are appropriate when clear of girders. When near to the girder top flanges, limit the saw-cut depth to 0.5 in. less than the deck thickness.*

   b) Perform a series of saw-cuts transverse to the girder axes to create a series of disposable panels. The cuts should not penetrate the middle strip of deck located over the web of the girder. Through-thickness cuts are appropriate when clear of girders. When near to the girder top flanges, limit the sawcut depth to 0.5 in. less than the deck thickness.
3. *Using a crane or other piece of lifting equipment in tandem with demolition hammers and prybars, separate the panels from the deck concrete over the girder webs and lift clear for disposal.*
4. *Use 30-lb demolition hammers to remove the concrete over the bridge girders down to the level of the bottom layer of deck reinforcement. The 30-lb demolition hammers should be used at an angle not exceeding 45 degrees from horizontal.*
5. *Once the bottom layer of deck reinforcement is exposed, 15-lb demolition hammers should be used to remove the remaining concrete and install a roughness in compliance with project specifications. Where the deck concrete immediately over the web is sound, it may not be necessary to fully remove all deck concrete. In this study, it was observed that the deck and girder concrete form a strong bond where the concrete was roughened prior to casting of the deck that makes complete removal of deck concrete difficult. The judgement of the engineer should govern the extent of removal necessary considering the condition of the system and risk of damage to the underlying girders.”*

2.2.6 State Practices

Only a few states address deck removal or damage to top flange of girders during deck removal. In the following, the relevant provisions are reproduced.

2.2.6.1 KDOT 2015, Section 202.3, Standard Specifications for State Road and Bridge Construction 2015

> *"Clearly mark the location of the existing girder top flanges on top of the existing deck concrete. Mark the entire length of all girders before sawing or removing any concrete. Limit concrete sawing to a maximum depth of 3 inches directly above any girder and within 3 inches of either edge of a girder top flange. Do not use drop-type pavement breakers. Do not use a hoe ram directly above any girder or within 1.0 foot of either edge of a girder top flange. Use a jackhammer no heavier than 15 pounds to remove concrete above and within 1.0 foot of either side of a girder top flange."*

2.2.6.2 NDOR 2016, Nebraska Department of Roads, Section 2 – 354, Bridge Office Policies and Procedures, 2016, p. 2.53.

> *"When breaking existing concrete, the use of a 15 lb. maximum hammer applied at a 45° angle is required to chip along the edges of removal, and a 30 lb. maximum hammer applied at a 45° angle is required for all other concrete removal."*

2.2.6.3 South Dakota Department of Transportation (SDDOT, 2018), Structures Construction Manual, P. 16-324, 2018.

> *"On a girder bridge, it is critical that care be taken during the removal process to ensure that the existing girders are not damaged. Plans will require that the limits of the existing girder top flanges be marked (usually with spray paint) on the top of the bridge deck. The Contractor should not be allowed to use any impact type breakout equipment larger than hand tools for removal or saw cut of the slab within 6 inches of the actual limits of the flange. It is recommended that the Contractor be limited to a 15 lb. chipper hammer immediately above the girders.*

Watch to make sure the breakout method does not nick, gouge, or scratch the top of the girder or any other structural steel that is to be reused. Steel girders are of particular importance as they can develop fatigue cracks from even the seemingly smallest nicks or gouges. Absolutely no nicks or gouges should be allowed on existing steel girders. If any nicks, gouges, or scratches occur, notify the Office of Bridge Design immediately. The Office of Bridge Design will most likely need to review the extent of the damage and then will recommend a method of repair. Some existing girders have shear connector angles or studs that make the girder function integrally with the deck slab for flexural strength. If any of these existing connectors are failed or removed in demolition, they must be replaced. The plan notes will usually provide notification of the connectors, and direction for what is required to replace them if they are damaged. If not, contact the Office of Bridge Design for further instructions."

"The bridge deck should be cut into sections and lifted off the bridge. Do not allow the sections to fall into the canopy. Generally, the Contractor will saw cut the deck at a depth that cuts at least the bottom mat of reinforcing steel. If the structure is a girder bridge, watch to ensure that the depth is monitored such that the existing girder flanges are not damaged."

2.3 Repair Procedures and Materials

2.3.1 PCI (2006)

The PCI manual for Evaluation and Repair of Precast Prestressed Concrete Bridge Products addresses repair of prestressed bridge elements and provides specific repair procedures for various types of product defects. It also discusses patching and epoxy injection procedures. Most discussions are addressed toward problems that may arise in production facilities, but the recommended procedures can also be used later in the service life of the precast elements. The manual specifically addresses "spalls and voids in the bottom flange with exposed prestressing strand" and "damaged flange requiring removal and replacement of concrete."

Regarding damage to bottom flange, the following procedures are recommended: The recommended procedures are limited to situations when voids do not extend beyond a depth of 4-in and length of 4 ft.

"A. Remove all loose concrete.

B. Square interfaces with existing concrete to be in contact with the patch.

C. Clean the excavated area with a stiff wire brush, blowing away dust with high-pressure air, filtering out any oils from the compressor.

D. Repair Option 1: Prepare surfaces to a saturated surface dry condition. Fill the void area with an approved polymer modified cementitious, shrinkage-compensating patching material with a compressive strength equal to or greater than the specified design strength of the beam. Prepackaged patching material is preferred to control quality. Cure properly. See Chapter 4 for further discussion.

E. Repair Option 2: Coat contact surface with an approved bonding agent, following the manufacturer's instructions. Fill the void area with a high-strength, cement-based, shrinkage-compensating mortar, following the manufacturer's instructions. Cure properly. See Chapter 4 for further discussion.

> *NOTE 1: Use of the same concrete mix as used in the original pour can work well with proper curing.*
>
> *NOTE 2: If the repair is made prior to transfer of prestress, the beam should be cooled to ambient temperature before beginning repairs.*

F. Detensioning should not occur until the patch reaches the specified compressive release strength.

G. For larger spalls of a similar nature or involving more strands, the same repair techniques may be employed, but a repair plan must be submitted to the owner/engineer for evaluation and approval.

H. All patches located over traffic or sidewalk areas should either encapsulate existing reinforcement or be anchored by supplemental reinforcement or other anchoring devices."

Regarding damage to top flange, the following procedures are recommended. They apply to situations when no more than half the cantilever flange and not more than 3 ft long are involved.

"A. Remove all loose concrete.

B. Preserve and thoroughly clean all mild steel reinforcement.

C. Clean the remaining concrete contact surfaces with a stiff wire brush, and blow away dust with high-pressure air, filtering out any oils from the compressor.

D. Repair Option 1: Prepare surfaces to a saturated surface dry condition. Re-pour the missing section with an approved polymer-modified, cementitious, shrinkage-compensating patching material with a compressive and tensile strength equal to or greater than the design strength of the beam. Prepackaged material is preferred to control quality. Cure properly. See Chapter 4 for further discussion.

E. Repair Option 2: Coat the contact surfaces with an approved bonding agent, following the manufacturer's instructions. Re-pour with concrete meeting or exceeding the beam design strength specified. Moist cure in accordance with contract specifications. See Chapter 4 for further discussion."

2.3.2 Wipf et al. (2004) (Vol 3 of 3)

This report is the third volume of a 3-volume report prepared for the Iowa Department of Transportation addresses evaluation of various patch materials for damaged concrete. The first two volumes of this report addressed CFRP for repair of damaged prestressed and the use of FRP to prevent chloride penetration in bridge columns. The objective of the study reported in the third volume was to find factors that affect long-term performance of repair materials, and to provide guidance on material selection and application procedures. The authors provided detailed reviews of prior works performed in this subject area including those by Traub et al. (1996), Mangat and Limbachya (1995), and Wall and Shrive (1988). In their experimental work, the authors selected five different repair materials: a one-component, polymer-modified patch material with corrosion inhibitor (A), a rheoplastic shrinkage-compensated cement-based material with silica fume, fibers, and a corrosion inhibitor (B), a 2-component, polymer-modified, fast setting Portland cement mortar (C), a one-component cementitious mortar with low-density aggregates (D), and a dry hydraulic cement material (E). Test beams were built and repaired using the selected repair materials. Cylinder specimens were used for shear bond strength. The wedge cylinders were subjected to freeze-thaw tests and axial load. Beam specimens to assess load-deflection behavior, cracking load, patch bond, etc.

The authors emphasize the importance of modulus of elasticity as the most important material property. The authors identify the following key parameters for selection of repair materials: modulus of elasticity, bond strength, coefficient of thermal expansion, and compressive strength. They suggest comparing different materials by creating a table with these test parameters listed. Give numerical ratings to each parameter based on the reported test results. Considering the importance of modulus of elasticity and bond strength, a weighing factor of two is suggested for these two parameters, with the other parameters receiving a weighing factor of one. The weighted ratings are then added for each material, and a ranking is determined. The authors identified materials B and E as the best ranked materials in the group of materials that they tested.

The authors suggest using materials with modulus of elasticity that is similar to concrete and discourage selecting repair materials based solely on compressive strength. They make several recommendations for field application of repair materials.

Chapter 3. Field Inspection of Wisconsin Bridges

A number of bridges around the state of Wisconsin include girders that have incurred damage to their top or bottom flanges due to vehicle impact or during removal of the bridge deck. Many of these bridges have been repaired. Several bridges were identified by the Project Oversight Committee (POC) for this project. The research team selected a number of bridges for inspection. The following describes the conditions of the various bridges that were inspected. The original intent of the project was to use some of these bridges for analyses under the case studies section (chapter 8). However, it became clear that the extent of bottom damage for available bridges was not in the serious or severe categories. Furthermore, detailed information on the bridges was not available to allow case studies. Therefore, simulated cases were used in chapter 8.

3.1 Top flange repair

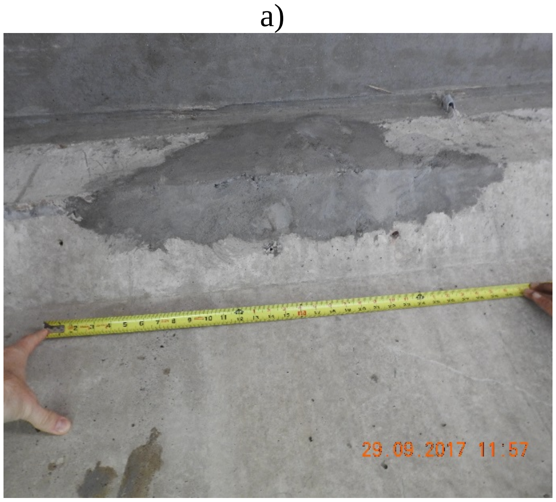
3.1.1 B-64-122 (I-43 NB over Elm Ridge Road, Delavan, WI), and B-64-123 (I-43 SB over Elm Ridge Road, Delavan, WI)

Structures B-64-122 and B-64-123 are a pair of parallel bridges in Walworth County that sustained a significant amount of damage to the top flanges of several girders during a bridge deck replacement project. Figure3.1 shows the damage incurred during deck removal to a section of the top flange of a girder on structure B-64-122 that was repaired using patching material. There are two girders that have approximately 15-20-foot-long repair patches to the top flange (Figure 3.1) and another girder that incurred damage to the top flange on both faces of the flange. Figures 3.2 and 3.3 show the repair done to the damaged girders of bridge B-64-122 and B-64-123, respectively. The available repair information for bridge B-64-123 is shown in Figure 3.4.

Figure 3.1. Damage to the top flange of prestressed concrete girder on B-64-122 resulting from deck removal.

a)

b)



c)

d)

Figure 3.2. Repair patch of damaged (top flange) prestressed girder on B-64-122

a)

b)

c)

d)

Figure 3.3. Repair patch of damaged (top flange) prestressed girder on B-64-123

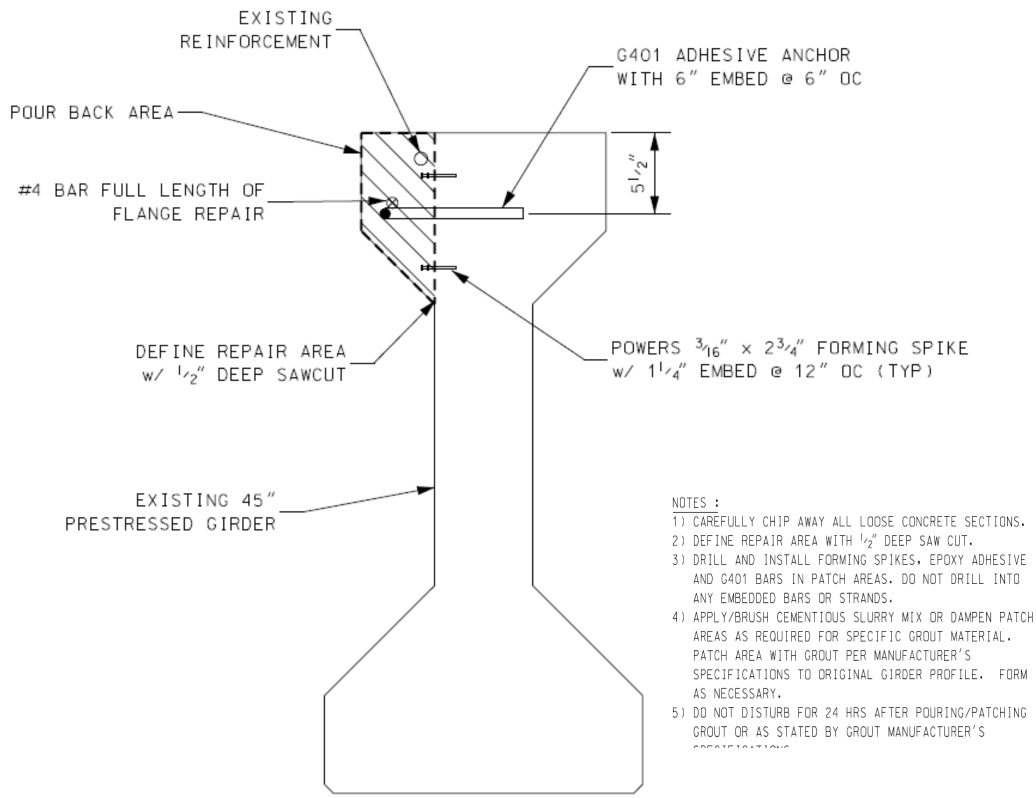


Figure 3.4. Girder repair section, (WISDOT, HSI report)

3.2 Bottom flange damage

3.2.1 B-40-485 (Airport Spur Over Howell Avenue, Milwaukee, Wisconsin)

This structure had numerous instances of minor damage to the bottom flanges of girders (Figure 3.5) as well as spalling on the bridge deck soffit. Deck spalling appears to be unrelated to the accidental flange damage. While the girders on this bridge do not appear to have any major damage, there are numerous nicks and gouges to the exterior girders. Figure 3.6(a) shows small nicks and gouges to an exterior girder and bottom of the barrier facing oncoming traffic. These are considered minor as they are small and do not expose any steel. Figure 3.6(b) shows a repair patch approximately 18-24 inches long that has subsequently experienced more minor damage.

Figure 3.5. Bridge B-40-485 with minor damage on the prestressed girder

a)

b)

Figure 3.6. Bridge B-40-485: a) Nicks and gouges to girder and barrier, and b) Girder patch with nicks.

3.2.2 B-20-157 (County Highway Y over US Highway 151, Fond-du-Lac)

Structure B-20-157 includes two girders that were struck by an over-height vehicle. As sometimes occurs with vehicle impact, the exterior girder facing oncoming traffic sustained the damage seen in Figure 3.7. As the vehicle rebounded, it left the interior girders untouched and impacted Girder 1, the exterior girder on the other side of the bridge, which sustained the damage shown in Figure 3.8. In this case, the damage sustained from the rebound impact was actually more extensive than the damage from the initial impact. In both instances there were numerous exposed prestressing strands that exhibited some degree of corrosion. The corrosion to the strands and reinforcing concrete can be seen clearly in Figure 3.9. Although there are a significant number of exposed strands in these girders, this is considered moderate damage because the strands are intact. Nevertheless, these girders were replaced.

The damage shown in Figure 3.7 and Figure 3.8 show the impact damage. Concrete damage was relatively narrow (approximately 1 to 1.5ft) at the bottom of the flange where impact occurred. The damage and spalling were wider (a length of 3-4 feet) at the top of the bottom flange. Both instances of damage exhibit a similar pattern.

Figure 3.7. Impact damage to bottom flange of Girder 4 in B-20-157 (initial strike)

Figure 3.8. Damage to the bottom flange of Girder 1 in B-20-157 (rebound impact)

3.2.3 B-20-147 (US Highway 45 over US Highway 151, Fond-du-Lac)

Structure B-20-147 had major damage to one girder. This damage occurred at the interior side of the exterior girder, Girder 1, and is somewhat unusual in that none of the preceding (when considering traffic direction) five girders experienced damage. It would be expected for damage to this area of the structure to come from a rebound of the vehicle after initially striking the first girder, as was the case previously documented with Structure B-20-157. The damage itself is typical of an over-height vehicle impact and is shown in Figure 3.9. The vehicle impact resulted in the spalling of concrete approximately four feet in length with 6 to 7 prestressing strands exposed and exhibiting corrosion for approximately three feet. It appears that at least one of the strands was nearly severed.

Figure 3.9. Corrosion of prestressing strands and reinforcing steel in Girder 4 on B-20-147

Figure 3.10. Damage with exposed strands to Girder 1 in B-20-147

Chapter 4. Stress Calculations in Damaged Prestressed Girders

4.1 Background

The design of prestressed concrete girders for buildings and bridges consists of two primary design considerations, both of which are based on sectional analyses at critical sections: 1) stress checks at the serviceability limit state, and 2) strength check at the strength limit state. Typically, the number and layout of prestressing steel strands are selected to meet the tension and compression stress limits under the applicable serviceability-based load combination(s), and then strength checks are made for the strength-based load combinations.

The calculation of stresses (on any desired section) and checking those stresses against the corresponding allowable limits specified by the building and bridge design codes are well-established and straightforward for undamaged prestressed girders. There is at least one axis of symmetry present, thus the basic bending stress equation would apply. The stresses at critical locations due to prestressing and girder self-weight are calculated at the "initial" state (in prestressing plant right after de-tensioning), and later when the girder is placed on the structure (following prestress losses). The non-composite girder section properties are used to calculate stresses based on the general linear stress relationship shown below:

$$\sigma = \frac{P}{A} \pm \frac{Pey}{I} \pm \frac{My}{I}$$

Where σ is the bending stress at any point y (vertical distance from the principle horizontal axis), A is the cross-sectional area, e is the eccentricity of prestressing force from the centroid (along the y axis), M is the applied moment, and I is the moment of inertia with respect to the principle horizontal axis. Following the installation of girder on site, the stress imposed by the weight of the fresh concrete slab is also applied on the non-composite girder properties, while the composite dead loads and live loads are applied on the composite section (with a composite moment of inertia), using the same basic flexural stress equation.

4.2 Problem of post-damage stress calculations

In a composite prestressed concrete girder/slab system (where girder is not shored during construction), the dead loads due to the weight of the girder and the slab are resisted by the non-composite section, and the slab would theoretically not sustain any dead load stresses expect for dead loads that are subsequently applied on the composite section. However, in a composite prestressed girder under dead load, any damage to the girder would redistribute the existing dead load stresses, and these stresses would shift into the slab as well. To estimate the change in stresses due to loss of section, a set of analytical procedures must be developed.

The process to calculate service level stresses when a part of the cross section is lost is not well established at this point. Most researcher studying damaged prestressed bridge girders have focused on calculating and restoring sectional (and member) strength (such as Harries et al., 2012), and not assessing whether service stresses would in fact remain within the allowable limits. This is due mainly to the complexity of assessing the effects of loss of section on the state of stress within the post-loss section. Shanafelt and Horn (1985) tried to bracket the problem of estimating stresses following damage by assuming that the girder and slab dead load could be resisted by the girder alone, or by the full composite section, and thus finding the range of possible stress values.

The problem of finding changes in stresses due to section loss in a composite (or non-composite) section while under existing dead load stresses is a complicated problem. In this research, a methodology is proposed to estimate such changes in stress because of damage.

The sequence of dead load stress calculations for bottom damage cases is as follows:

1) Calculate stresses (at various points in the cross section) due to prestress and girder dead load at the time of deck placement
2) Add (to Step 1) stresses due to deck slab weight (σ=Mc/I, using non-composite I)
3) Add (to Step 2) stresses due to composite dead loads (using composite moment of inertia)
4) Calculate the <u>changes</u> in stress due to loss of section (concrete and/or prestressing steel). The stresses following damage would be equal to the stresses in Step 3 plus changes calculated in Step 4.
5) Calculate stress changes following repair: preloading (if applicable), patching, and CFRP repair (if any). If preloading is not applied, there would not be any change in dead load stresses following repairs.

The sequence of dead load stress calculations for top damage cases is as follows:

1) Calculate stresses (at various point in the cross section) due to prestress and girder dead load at the time of deck placement
2) Add (to Step 1) stresses due to deck slab weight (σ=Mc/I, using non-composite I)
3) Add (to Step 2) stresses due to composite dead loads (using composite moment of inertia)
4) Remove slab and other dead loads except the girder dead load. Stress due to girder dead weight on the non-composite girder section would remain.
5) Calculate <u>changes</u> in stress due to loss of section (in the top flange of the girder) and add to stresses in Step 4 to arrive at post-damage stresses.
6) Calculate stress changes following repair: preloading (if applicable), patching, etc.

In the following sections, a set of procedures are developed for calculating the changes in stress due to damage. First, the concept of transformed section is discussed to implicitly account for prestress losses and gains when calculating stresses. Then the effect of section loss is considered with or without a deck slab (an additional area that is initially unstressed under dead load). Later, a differential form of the general bending equation is used to estimate changes in stress. Finally, the results are compared with Abaqus finite element results for a specific example.

4.3 Basic consideration of concentrically-prestressed sections

To set the stage for the proposed methodology, it is informative to consider a few simple examples involving sections with a concentric prestressing force applied at the geometric center of the section. Figure 4-1 shows a rectangular concrete cross section (b x h) with a prestressing tendon (with area A_s) located at the centroid of the section. In the following calculations, stresses due to the application of the prestressing force are calculated using non-transformed and transformed sections.

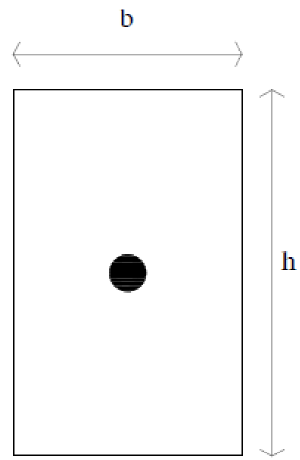


Figure 4.1. A concrete cross section with a prestressing force applied at its centroid

4.3.1 Transformed and non-transformed sections

Assume that P_i is the initial prestress force applied on a cross section using a prestressing tendon with an area A_s on a concrete area A_c. The prestressing steel and concrete are bonded together (i.e. compatibility of strains between the two apply).

$$A_c = bh - A_s$$

If non-transformed area is used, i.e. the steel area is <u>not</u> converted into an equivalent concrete area using the modular ratio ($n = \frac{E_s}{E_c}$), the initial stress in concrete (σ_1) is:

$$\sigma_1 = \frac{P_i}{A_c}$$

Define:

$$x = \frac{E_s}{E_c}\frac{A_s}{A_c} = n\rho$$

where ρ is the reinforcement ratio (A_s/A_c), and E_s and E_c are the moduli of elasticity for prestressing steel and concrete, respectively. Because of the application of σ_1, and the resulting compressive strain in concrete, the corresponding strain in the steel is reduced, and the prestressing force is reduced as well. This causes a change in the stress in concrete due to elastic shortening, with the stresses in steel and concrete changing subsequently in an iterative process:

$$\sigma_2 = \sigma_1 - \frac{\sigma_1}{E_c}\frac{E_s}{A_c}A_s = \sigma_1 - \sigma_1 x$$

Similarly, the reduction in concrete stress causes an increase in the steel stress:

$$\sigma_3 = \sigma_2 - \frac{\sigma_2 - \sigma_1}{E_c}\ \frac{E_s}{A_c}A_s = \sigma_2 + \sigma_1 x^2 = \sigma_1 - \sigma_1 x + \sigma_1 x^2$$

…

$$\sigma_m = \sigma_1 - \sigma_1 x + \sigma_1 x^2 - \sigma_1 x^3 + \sigma_1 x^4 + \cdots + \sigma_1 x^{m-1} - \sigma_1 x^m$$

This can be rewritten in the following form:

$$\sigma_m = \sigma_1(1 - x + x^2 - x^3 + x^4 + \ldots + x^{m-1} - x^m\ ) = \sigma_1 S_m$$

Where

$$S_m = 1 - x + x^2 - x^3 + x^4 + \cdots + x^{m-1} - x^m$$

$$xS_m = x -\ x^2 + x^3 -\ x^4 + x^5 + \cdots + x^m - x^{m+1}$$

Adding the above two expressions results in:

$$S_m + xS_m = S_m(1 + x) = 1 - x^{m+1}$$

Considering that the value of the modular ratio n is generally on the order of 8 to 9, but $A_c$is far larger than A_s, x is typically a number that is less than 1.

Therefore, since x<1 => $\lim_{m\to\infty}(1 - x^{m+1}) = 1.0$=>$S_m = \frac{1}{1+x}$

$$=>\sigma_{final} = \sigma_f = \sigma_1(\frac{1}{1+x}) = \sigma_1(\frac{1}{1+n\rho}) = \frac{A_c}{A_c+nA_s}\sigma_1 \qquad (Eq. 4.1)$$

Now consider a transformed cross section in which the prestressing steel is converted into an equivalent concrete. Therefore, the transformed cross-sectional area is (A_c+nA_s). The final stress can be determined without iteration:

$$\sigma_f = \frac{P_i}{A_c+nA_s} = \frac{P_i}{A_c}\frac{A_c}{A_c+nA_s} = \sigma_1 \frac{A_c}{A_c+nA_s} \qquad (Eq. 4.2)$$

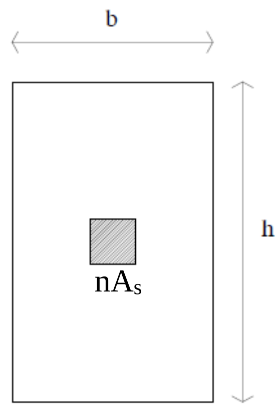


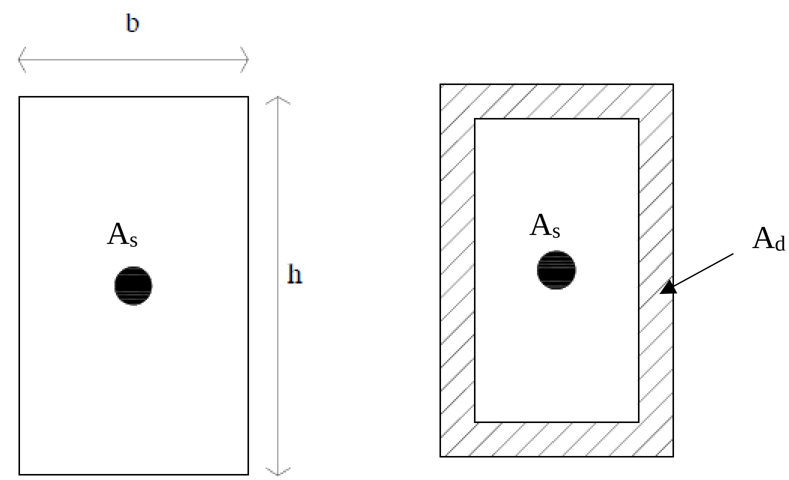
Figure 4.2. Transformed concrete cross section with the area of steel converted to concrete

Equations 4.1 and 4.2 are identical. This illustrates that, to avoid iteration and to avoid explicitly accounting for elastic shortening loss (or gain) when calculating stresses, a transformed section can be used (PCI 2011). A similar approach can be taken to address stress change due to loss of section as indicated below.

4.3.2 Symmetrical Loss of section

The following simple example illustrates the effect of loss of cross section. Assume that the cross-sectional area of the concrete section in the previous example suffers a symmetrical loss of cross-sectional area (A_d) (cross hatched) as shown in Figure 4.3.

A_d $= loss\ of\ section\ due\ to\ damage$



a) Undamaged section b) Damaged section (loss of A_d)

Figure 4.3. Undamaged (a) and damaged (b) cross sections

The transformed areas are used here to avoid calculating elastic losses and gains. The transformed cross-sectional area in undamaged (A_t) and damaged sections (A'_t) can be calculated as follows:

$$A_t = bh - A_s + \mathrm{n}A_s = A_c + nA_s$$

$$A'_t = bh - A_s - A_d + nA_c = A'_C + nA_s$$

where, $A'_C = A_c - A_d$

The stress that exists before occurrence of damage is:

$$\sigma_1 = \frac{P_u}{A_t}$$

where, P_u is the prestressing force before damage occurs.

Due to loss of section, a change in stress occurs:

$$\sigma'_1 = \frac{P_u}{A'_t} - \frac{P_u}{A_t} = P_u\left(\frac{1}{A'_t} - \frac{1}{A_t}\right) = \frac{A_t - A'_t}{A'_t A_t}$$

$$\sigma_1' = P_u \frac{(A_C+nA_S)-(A_C'+nA_S)}{(A_C+nA_S)(A_C'+nA_S)} = \frac{A_C-A_C'}{(A_C+nA_S)(A_C'+nA_S)} P_u = \frac{A_d}{A_t' A_t} P_u \quad \text{(Eq. 4.3)}$$

This change in stress due to loss of section causes prestress losses (elastic gain or loss) that are directly accounted for when using transformed sections (as discussed in the example above).

The change in stress ($Eq.$ 4.3) can also be calculated using a differential approach. Using transformed sections to avoid elastic gain/loss calculations, the basic stress relationship can be written as:

$$\sigma = \frac{P_u}{A_t} \quad \text{(Eq. 4.4)}$$

$$\Delta\sigma = \frac{(\Delta P_u)(A_t) - (\Delta A_t)P_u}{{A_t}^2}$$

Since transformed section properties are used, ΔP_u can be set equal to zero. Thus,

$$\Delta\sigma = \frac{-(\Delta A_t)P_u}{{A_t}^2}$$

The total stress $= \sigma_1 + \Delta\sigma$

However, the change in A_t or (ΔA_t) is equal to the loss in sectional area or A_d. Substituting A_d for ΔA_t,

$$\Delta\sigma = -\frac{A_d P_u}{(A_t)^2} \quad \text{(Eq. 4.5)}$$

Eq. 4.3 approaches Eq. 4.5 when A_d is small. The differential form may be used to calculate stress change when A_d is relatively small. However, as damage gets larger, Eq. 4.5 would be expected to lose accuracy (the differential solution may not apply). Eq. 4.3 may be used in such cases.

4.3.3 Section gain

In the previous example, the effect of loss of cross-sectional area on stresses was discussed. The same discussion can be made regarding a sectional gain (increase in cross-sectional area). In this example, assuming that the prestress force was applied on the original (non-augmented) section, the subsequent addition of an area (area gain or A_g) to the section would not redirect stresses to the new areas. This is analogous to adding a deck area on the girder, which would remain stress free without the action of creep, shrinkage, and composite dead loads. In a composite prestressed girder/slab system, the addition of slab augments the beam area without a redistribution of the stresses that already exist in the girder alone. If a subsequent loss of section occurs (A_d), it would not be immediately clear how existing forces would be redistributed in the composite section ($A_c + nA_s + A_g$). It is assumed here that both the gain and loss areas occur symmetrically with respect to the original section.

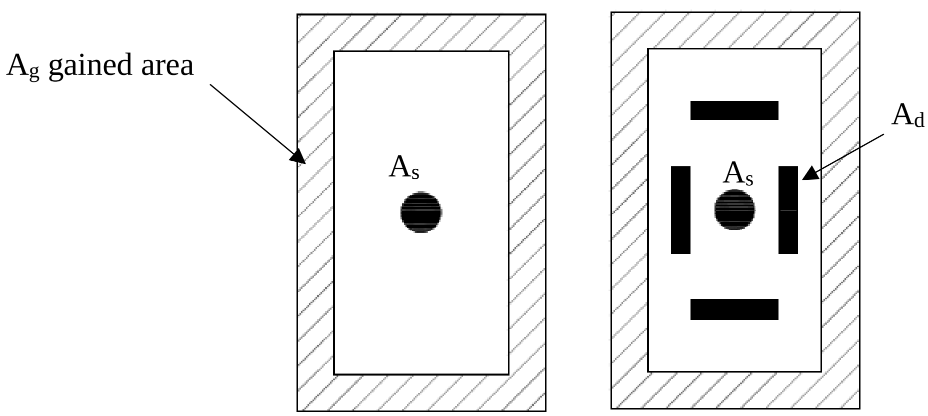


Figure 4.4. Addition of area (A_g) with a subsequent loss in section due to damage (A_d)

For this condition, Eq. 4.3 can be rewritten in the following form to calculate stress change due to damage A_d

$$\Delta\sigma = -\frac{A_d P_u}{(A_c+nA_s)(A_c+A_g-A_d+nA_s)} = \frac{A_d P_u}{(A_t)(A'_{tc})} \qquad (Eq. 4.6)$$

where A'_{tc} is the transformed composite (damaged) area. This use of this approach in analyzing damage to prestressed girders is evaluated using an ABAQUS finite element model in a subsequent section of this chapter. The differential approach proposed can be extended to unsymmetrical sections (including eccentric prestressing force) and moments.

4.4 Eccentric prestress force:

In the previous section, examples involving concentric prestressing force applied on a section was presented. In this section, it is illustrated that the use of the transformed section can similarly eliminate the need for iteration for prestress losses (and gains) when the prestress force is eccentric with respect to the centroid of the cross section (Figure 4.5).

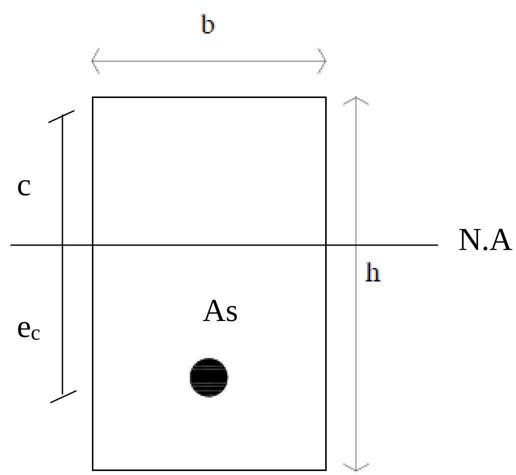


Figure 4.5. Eccentric prestressing force applied on a cross section

In addition to the parameters defined earlier,

I_c = moment of inertia of concrete section

c = distance from the top fiber to the centroid of the concrete section.

e_c = eccentricity of prestressing force relative to the centroid of concrete section.

Using the non-transformed section, the initial flexural stress at the centroid of prestressing force can be calculated as:

$$\sigma_1 = \frac{P_i}{A_c} + \frac{P_i {e_c}^2}{I_c}$$

This level of stress causes a strain in concrete $\frac{\sigma_1}{E_c}$, which in turn causes a loss in prestressing steel due to strain compatibility ($\frac{\sigma_1}{E_c}$ E_s). The stress in concrete at the centroid of steel would then become:

$$\sigma_2 = \sigma_1 - \frac{\sigma_1}{E_c} E_s A_s \left(\frac{1}{A_c} + \frac{{e_c}^2}{I_c}\right)$$

Define $x = \frac{E_{s A_s}}{E_c}\left(\frac{1}{A_c} + \frac{{e_c}^2}{I_c}\right)$

$$\sigma_2 = \sigma_1 - \sigma_1 x$$

This would further change both the steel and concrete stresses in an iterative fashion.

$$\sigma_3 = \sigma_1 - \sigma_1 x + \sigma_1 x^2$$

…

$$\sigma_m = \sigma_1 - \sigma_1 x + \sigma_1 x^2 - \sigma_1 x^3 + \cdots + \sigma_1 x^{m-1} - \sigma_1 x^m = \sigma_1 (1 - x + x^2 - x^3 + \cdots + x^{m-1} - x^m)$$

As discussed earlier, the limit for the above series $(1 - x + x^2 - x^3 + \cdots + x^{m-1})$ is equal to $\frac{1}{1+x}$, therefore, the final stress is:

$$\sigma_{final} = \sigma_1 \left(\frac{1}{1+x}\right) = \left[\frac{P_i}{A_c} + \frac{P_i {e_c}^2}{I_c}\right]\left[\frac{1}{1 + nA_s\left(\frac{1}{A_c} + \frac{{e_c}^2}{I_c}\right)}\right]$$

$$\sigma_{final} = \left[\frac{P_i}{A_c} + \frac{P_i {e_c}^2}{I_c}\right]\left[\frac{1}{\frac{A_c I_c + nA_s I_c + nA_s {e_c}^2 A_c}{A_c I_c}}\right]$$

$$\sigma_{final} = \left[\frac{P_i}{A_c} + \frac{P_i {e_c}^2}{I_c}\right]\left[\frac{A_c I_c}{I_c (A_c + nA_S) + nA_S {e_c}^2 A_c}\right]$$

$$= \left[\frac{P_i}{A_c} + \frac{P_i {e_c}^2}{I_c}\right]\left[\frac{1}{\frac{I_c + nA_s {e_c}^2}{I_c} + \frac{nA_S}{A_C}}\right]$$

$$\sigma_{final} = P_i(\frac{I_c + A_c e_c{}^2}{A_c I_c})(\frac{A_c I_c}{A_c(I_c + nA_S e_c{}^2) + nA_S I_c})$$

$$\sigma_{final} = P_i(\frac{I_c + A_c e_c{}^2}{A_c\left(I_c + nA_S E_c{}^2\right) + nA_S I_c})$$

$$\sigma_{final} = P_i\left(\frac{I_c + A_c e_c{}^2}{(I_c(A_c + nA_S) + nA_S A_c e_c{}^2)}\right) \qquad \text{(Eq. 4.7)}$$

Now, using the transformed section properties, the following can be written:

$$A_c = bh - A_s$$

$$c_t = \frac{cA_c + nA_s d}{A_c + nA_s}$$

where c_t is the distance from the top fiber to the neutral axis of the transformed section, and $e_{tc} = d - c_t$

Therefore, $e_{tc} = d - \frac{cA_c + nA_s d}{A_c + nA_s} = \frac{dA_c + nA_s d - cA_c - nA_s d}{A_c + nA_s}$

$$e_{tc} = \frac{A_c(d - c\ )}{A_c + nA_s}$$

Since $e_c = d - c \Rightarrow e_{tc} = \frac{A_c e_c}{A_c + nA_s}$

Using parallel axis theorem, the transformed moment of inertia of section can be calculated as follows:

$$I_{tc} = I_c + A_c(e_c - e_{tc})^2 + nA_S(e_{tc})^2$$

$$I_{tc} = I_c + A_c(e_c - \frac{A_c e_c}{A_c + nA_s})^2 + nA_S(\frac{A_c e_c}{A_c + nA_s})^2$$

$$I_{tc} = I_c + A_c(\frac{A_c e_c + nA_S e_c - A_c e_c}{A_c + nA_s})^2 + nA_S(\frac{A_c e_c}{A_c + nA_s})^2$$

$$I_{tc} = I_c + \frac{A_c(n^2 A_s^2 e_c^2)}{(A_c + nA_s)^2} + \frac{nA_S\left(A_c{}^2 e_c{}^2\right)}{(A_c + nA_s)^2}$$

$$I_{tc} = I_c + \frac{e_c^2 A_c}{A_c + nA_s} \frac{n^2 A_s^2 + nA_s A_c}{A_c + nA_s}$$

$$= I_c + \frac{e_c^2 A_c}{A_c + nA_s} \frac{nA_s(A_c + nA_s)}{A_c + nA_s}$$

$$= I_c + \frac{nA_s e_c^2 A_c}{(A_c + nA_s)} = I_c + nA_S \frac{A_c}{(A_c + nA_s)} \; e_c^2$$

$$A_{tc} = A_c + nA_s$$

$$\sigma_{final} = \frac{P_i}{A_{tc}} + \frac{P_i e_{tc}^2}{I_{tc}}$$

$$= \frac{P_i}{A_c + nA_s} + \frac{P_i(\frac{A_c}{A_c + nA_s}^2)e_c^2}{I_c + nA_s\,(\frac{A_c}{A_c + nA_s})e_c^2}$$

$$= P_i \frac{I_c + nA_s\left(\frac{A_c}{A_c + nA_s}\right) e_c^2 + \left(\frac{A_c}{A_c + nA_s}\right)^2 e_c^2(\,A_c + nA_s)}{(\,A_c + nA_s)\,(I_c + nA_s\,\left(\frac{A_c}{A_c + nA_s}\right) e_c^2)}$$

$$= P_i \frac{I_c + \left(\frac{nA_s\,A_c}{A_c + nA_s}\right) e_c^2 + A_c^2 e_c^2(\frac{1}{A_c + nA_s})}{I_c(\,A_c + nA_s) + nA_s\,A_c e_c^2}$$

$$= P_i \frac{I_c + \left(\frac{nA_s\,A_c e_c^2 + A_c^2 e_c^2}{A_c + nA_s}\right)}{I_c(\,A_c + nA_s) + nA_s\,A_c e_c^2}$$

$$= P_i \frac{I_c + A_c e_c^2}{I_c(A_c + nA_s) + nA_s A_c e_c^2} \qquad \text{(Eq. 4.8)}$$

Eq. 4.7 and Eq. 4.8 are identical indicating that the use of transformed section can eliminate the need for iterations to calculate stresses due to prestress gains and losses. Similar approach can be extended to section losses, section gains, and losses after section gain as discussed earlier under the case of concentric prestress force.

4.5 Differential approach for calculating changes in bending stress due to section loss

In a cross section with at least one axis of symmetry, a generic linearly elastic equation for bending stress (σ) can be written as a function of applied moment (M), distance to desired location for bending stress calculation (y), and moment of inertia (I):

$$\sigma = \frac{My}{I} \qquad \text{(Eq. 4.9)}$$

The change in stress due to damage can be written in a differential form:

$$\Delta\sigma = \frac{\big((\Delta M)y + M(\Delta y)\big)I - (\Delta I)(My)}{I^2} \qquad (Eq. 4.10)$$

Although the externally applied moment may remain essentially unchanged, and elastic loss/gain are implicitly accounted for, the change in the position of neutral axis can modify the moment due to prestressing. In the above equation, Δy is the change in position of the desired point with respect to the neutral axis, (which shifts because of damage) and**,** ΔI is the change in the moment of inertia as a result of damage.

To account for the elastic gain or loss effects implicitly, the transformed section properties should be used (steel converted to equivalent concrete). Furthermore, the state of the cross section at the time of damage should be the basis for the type of section properties used. For example, If the cross section is a composite of a prestressed girder and a slab, the I, ΔI, y and Δy parameters should be based on the composite transformed section properties.

Note that, in Eq. 4.10 above, it is assumed that any damage would occur such that at least one axis of symmetry would remain in the beam cross section. Otherwise, the basic bending equation (Eq. 4.9) would not be valid, and a different (more complicated equation) would be applicable.

When the section has no axis of symmetry, or when the damage causes the section to become non-symmetrical, the basic bending equation presented above would not be valid. The basic equation for calculating bending stress under biaxial bending in an unsymmetrical section at a point (x, y) is:

$$\sigma = \frac{M_y I_x - M_x I_{xy}}{I_x I_y - I_{xy}} x + \frac{M_x I_y - M_y I_{xy}}{I_x I_y - I_{xy}} y \qquad (Eq. 4.11)$$

where M_x and M_y are applied bending moments about the horizontal (x) and vertical (y) axes through the centroid of the cross-section, I_x and I_y are moments of inertia in the horizontal and vertical direction, and I_{xy} is the product of inertia. This equation would become the same as $Eq. 4.9$ when $M_y = 0$ and $I_{xy} = 0$.

The differential form of $Eq. 4.11$ can be written as follows:

$$\Delta\sigma = \left(\frac{\widehat{D} - \hat{E}\hat{A}}{\hat{B}^2}\right) x + \left(\frac{\hat{B}\hat{F} - \hat{E}\hat{C}}{\hat{B}^2}\right) y$$

Where;

$$\hat{A} = M_y I_x - M_x I_{xy}$$

$$\hat{B} = I_x I_y - {I_{xy}}^2$$

$$\hat{C} = M_y I_y - M_y I_{xy}$$

$$\widehat{D} = x[\Delta M_y I_x + M_y \Delta I_x - \Delta M_{Xx} I_{xy} - M_x \Delta I_{xy}] + \Delta x \hat{A}$$

$$\hat{E} = \Delta I_x (I_y) + I_x (\Delta I_y) - 2 I_{xy} (\Delta I_{xy})$$

$$\hat{F} = y[\Delta M_x I_y + M_x \Delta I_y - \Delta M_y I_{xy} - M_y \Delta I_{xy}] + \Delta y \hat{C}$$

In addition to the effects of biaxial external bending moment, the effect of prestressing must also be considered by adding the following stress components to the basic bending equation given in Eq. 4.11. The change in stress due to the prestressing force can then be

$$\sigma = \frac{P}{A} + \left(\frac{(Pe_y)y}{I_x}\right) + \left(\frac{(Pe_x)x}{I_y}\right)$$

$$\Delta\sigma = \frac{\Delta PA - \Delta AP}{A^2} + \frac{\left(\Delta Pe_y y + P\left(\Delta e_y y + \Delta y e_y\right)\right) I_x - \Delta I_x Pe_y y}{I_x^2} + \frac{\left(\Delta Pe_x x + P(\Delta e_x x + \Delta x e_x)\right)I_y - \Delta I_y Pe_x x}{I_y^2}$$

4.5.1 Generalized differential approach

The generalized equation for calculating stress under the biaxial effects of prestressing force and biaxial external bending moments on a section without any axis of symmetry (as in a damaged girder) can be calculated using the following equation:

$$\sigma = \frac{P}{A} + \left[\frac{(M_x+M_{Px})I_y-(M_y+M_{Py})I_{xy}}{I_xI_y-I_{xy}^2}\right]y + \left[\frac{(M_y+M_{Py})I_x-(M_x+M_{Px})I_{xy}}{I_xI_y-I_{xy}^2}\right]x \qquad \text{(Eq. 4.12)}$$

Where M_x *and* M_y are the total external moments about the x *and* y axes, due to imposed dead and/or live loads and M_{Px} and M_{Py} are moments due to eccentricity of axial force with respect to horizontal and vertical axes, respectively. The coordinates of the point for which the stress is being assessed is x and y. Furthermore,

$$M_{Px} = Pe_y \text{ and } M_{Py} = Pe_x$$

where e_x and e_y are the eccentricities of prestressing force (P) with respect to the y and x-axes, respectively, and I_{xy} is the product of inertia.

The change in stress due to changes in the cross-sectional properties (as in loss of section) can be estimated using the differential form (based on Eq. 4.12):

$$\Delta\sigma = \Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3 \qquad \text{(Eq. 4.13)}$$

$$\Delta\sigma_1 = \frac{(\Delta P)A_t - (\Delta A)P}{(A_t)(A'_{tc})}$$

$$\Delta\sigma_2 = \left(\frac{\bar{B}\bar{F} - \bar{E}\bar{C}}{\bar{B}^2}\right) y + (\Delta y)\left(\frac{\bar{C}}{\bar{B}}\right)$$

$$\Delta\sigma_3 = \left(\frac{\bar{B}\bar{D} - \bar{E}\bar{A}}{\bar{B}^2}\right) x + (\Delta x)\left(\frac{\bar{A}}{\bar{B}}\right)$$

$$\bar{A} = \left(M_y + M_{Py}\right) I_x - (M_x + M_{Px}) I_{xy}$$

$$\bar{B} = I_x I_y - I_{xy}^2$$

$$\bar{C} = (M_x + M_{Px}) I_y - \left(M_y + M_{Py}\right) I_{xy}$$

$$\bar{D} = \Big[\Big(\left(\Delta M_y\right) + \left(\Delta M_{Py}\right)\Big) I_x + \left(M_y + M_{Py}\right)(\Delta I_x) - \big((\Delta M_x) + (\Delta M_{Px})\big) I_{xy} - (M_x + M_{Px})\left(\Delta I_{xy}\right)\Big]$$

$$\bar{E} = (\Delta I_x) I_y + I_x\left(\Delta I_y\right) - 2 I_{xy}\left(\Delta I_{xy}\right)$$

$$\bar{F} = \left[(\Delta M_x + \Delta M_{Px}) I_y + (M_x + M_{Px})\left(\Delta I_y\right) - \left(\Delta M_y + \Delta M_{Py}\right) I_{xy} - \left(M_y + M_{Py}\right)\left(\Delta I_{xy}\right)\right]$$

$$\Delta M_{Px} = (\Delta P) e_y + P\left(\Delta e_y\right)$$

$$\Delta M_{Py} = (\Delta P) e_x + P(\Delta e_x)$$

4.6 Suggested procedures for calculating change in stress due to damage

The suggested procedures for calculating changes in stress due to top and bottom damage cases are described below. Both cases utilize Eq. 4.13 for calculating the change in stress. In both cases, all parameters related to section properties must be calculated for both undamaged and damaged states. When the structure undergoes change from an undamaged to damaged state, the non-differential parameters refer to the undamaged state (just prior to damage) and the differential terms relate to the difference between damaged and undamaged states.

$$\Delta(Parameter) = (Parameter)_{Damaged} - (Parameter)_{Undamaged}$$

For bottom damage cases, the non-differential terms in Eq. 4.13 refer to the properties of the undamaged properties of the composite girder/concrete slab, and the differential terms refer to the change between damaged and undamaged composite states. In top damage cases, the non-differential terms relate to the non-composite undamaged girder properties, and the differential terms address changes between damaged and undamaged non-composite conditions.

4.7 Finite Element Modeling

There are number of techniques that are used to model the bond between concrete and prestressing strand in finite element models. Several factors are typically considered including the Hoyer Effect, bond-slip behavior, and transfer length.

Kannel et al. (1997) varied the diameter of the steel tendon linearly from zero at the end of the girder to the full diameter of the steel at the end of the transfer length. Mercan et al. (2010) divided the tendon into sections that gradually increased in cross-sectional area over the transfer length. For these methods, the transfer length needs to be predetermined from either calculations and/or assumptions.

Bond behavior between steel and concrete must be properly considered to accurately model prestressing in FEM. Embedment is a popular technique as contact surfaces do not need to be defined and is considered to be a computationally efficient approach (Arab et al., 2011). This involves confining the steel tendon within the host concrete element, constraining the nodes of the concrete to be slave to the nodes of the steel master. However, embedment assumes that perfect bond exists between concrete and steel and therefore neglects the effect of tendon slippage (Abdelatif et al., 2015). Extrusion is another technique that uses interaction properties to define the bond behavior between steel and concrete. These properties include normal, tangential and cohesive behavior and can more accurately account for the bond behavior between the two materials. Since tangential behavior is governed by friction, it can more accurately represent the variable bond stress between tendon and concrete. The cohesive behavior settings account for the slippage of the prestressed tendon within the concrete (Yapar et al., 2015). In both techniques the prestressing force is applied in a predefined field in the Abaqus finite element program.

In this research, a 3D finite element model using Abaqus/Explicit was developed to investigate the stress distribution before and after damage in an example prestressed girder. A rectangular section with a depth of 36 in and a width of 10.25 in (roughly equivalent to an AASHTO Type II girder with respect to height, area and moment of inertia) was selected. A 16-strand prestressing tendon (0.6-in diameter) was modeled with the centroid of prestressing steel located 6.75 in above the beam soffit (Figure 4.6). Span length was selected to be 50 ft.

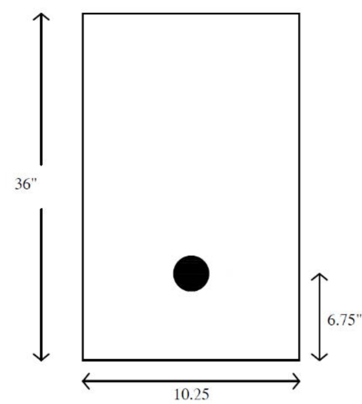


Figure 4.6. Cross section of undamaged prestressed girder modelled in Abaqus

Three-dimensional deformable solid elements were used to model the concrete and prestressing tendons (Figures 4.7 and 4.8). Taking advantage of symmetry (axis of symmetry in z direction at mid-span), only half of the span length was modeled for the simulation. End support was defined by constraints in x and y directions. Both concrete and steel were modeled as linear elastic materials. Materials' properties are shown in Table 4.1.

Table 4.1. Material properties used in Abaqus model.

Material	Mass density (lb/ft^3)	Modulus of elasticity (ksi)	Poisson's ratio
Steel	485	28500	0.3
Concrete	150	4230	0.15

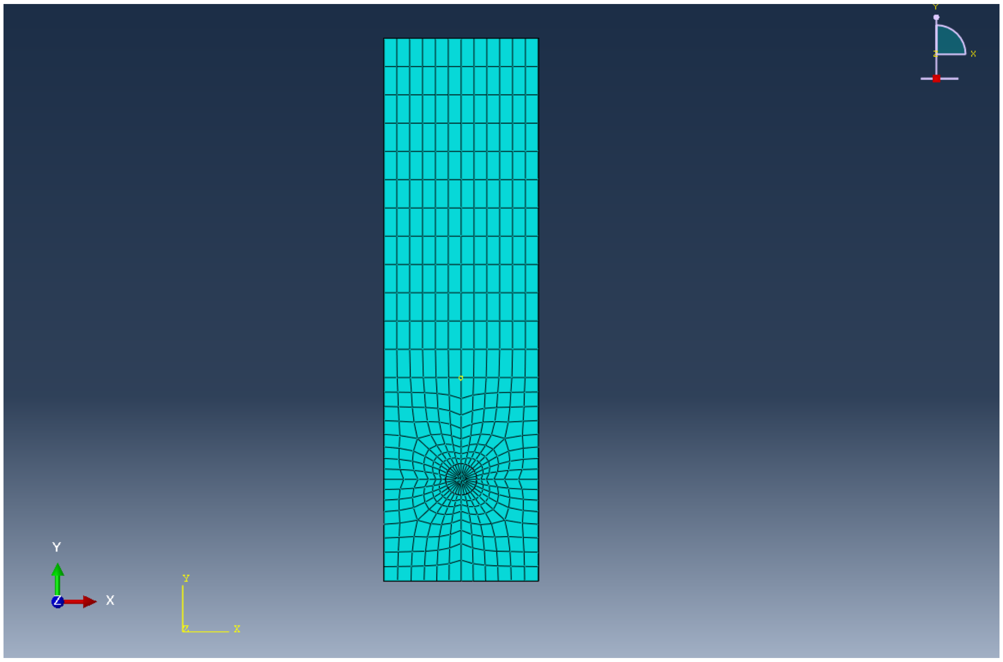


Figure 4.7. Cross sectional view of the undamaged rectangular prestressed beam

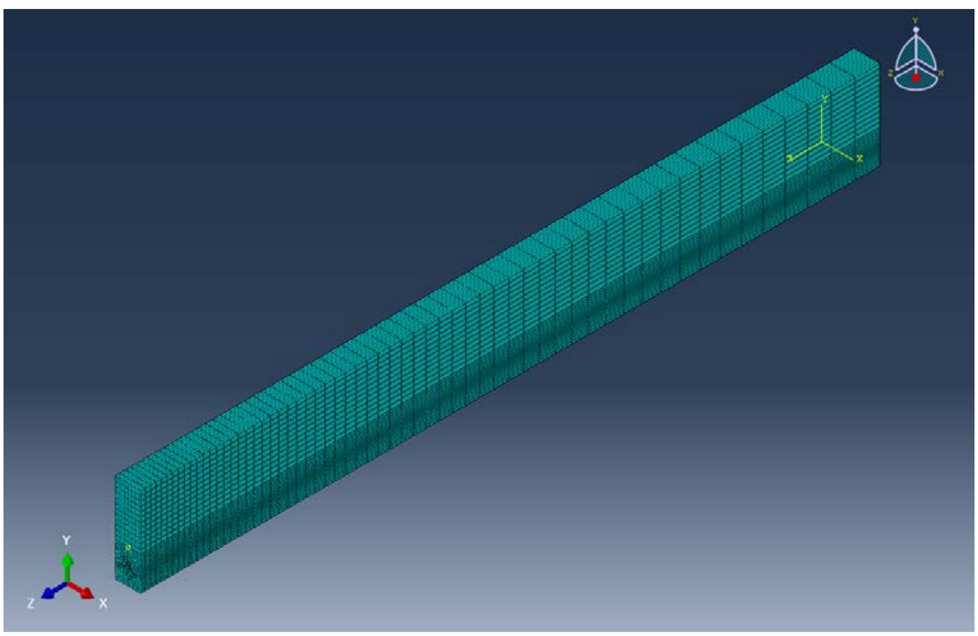


Figure 4.8. Three-dimensional view of the prestressed girder model

The interaction between steel and concrete including normal, tangential and cohesive behavior was modelled to simulate the transfer of forces at the steel-concrete interface. To simulate the bond at the interface of steel and concrete, steel was treated as the master surface and the concrete as the slave surface. The concrete also had a finer mesh than that of the steel, as suggested by the Abaqus user manual.

Prestressing forces was specified as a predefined field stress in the longitudinal direction of the strands, considering the effective stresses in each strand, and accounting for the calculated effects of long-term relaxation, shrinkage and creep. Elastic shortening was modelled through the finite element model itself. The model change technique was used to simulate a damage scenario in the beam. The damage was introduced at (and near) midspan as shown in Figure 4.9.

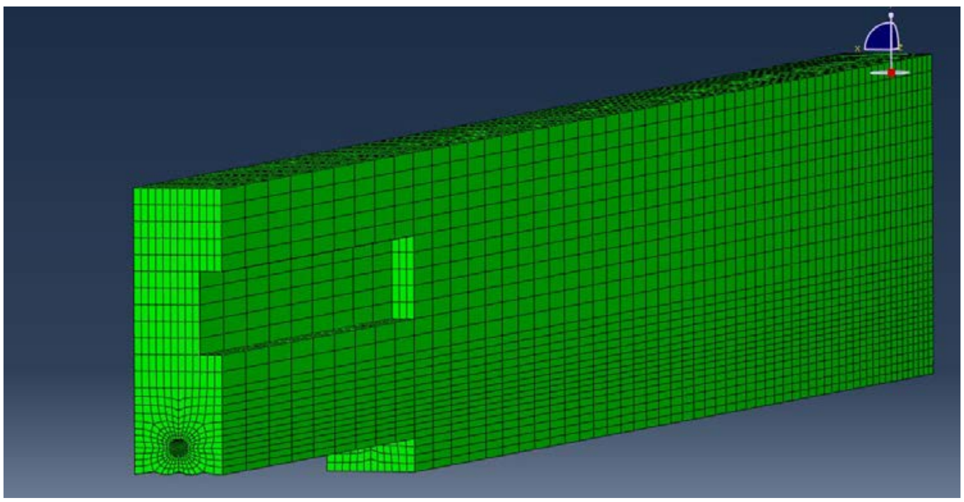


Figure 4.9. Damage simulation near the midspan of the beam (the cross section seen on the left is at midspan)

Prestressing was represented as a predefined field stress at the initial state in the strands. The analysis was performed in two steps following the initial state. Step 1 included application of girder self-weight as gravity load and transfer of prestressing force from steel to the concrete. Step 2 involved initiation of damage (removal of elements) in the model.

Figure 4.10 shows the longitudinal stress contour in the beam at the end of step 2. The stresses before and after damage (stress change due to damage) at four corners points of the section at midspan were calculated and compared with the calculations using the equations proposed in this chapter.

The transformed section properties calculated for the undamaged and damaged sections are shown in Table 4.2, without considering the secondary effects due to upward deflection of the beam. The secondary P-Δ effect due to deflection can be handled in the calculations by increasing the Δe_y value by the amount of vertical deflection as the beam undergoes change from undamaged to damaged condition. Moments due to dead load and effective prestressing force for the considered section are calculated as shown in Table 4.3.

Table 4.2. Section properties for non-composite rectangular section before and after simulated damage

	Undamaged section	Damaged section	Δ
Area (in^2)	387.05	322.05	-65
I_x (in^4)	41888	30476	-11412
I_y (in^4)	3231	2477	-754
I_{xy} (in^4)	0	120	120
e_x (in)	0	0.29	0.29
e_y (in)	-10.47	-12.11	-1.64

Table 4.3. Moments and prestressing forces

P (kips)	564	ΔP (kips)	0
M_x (k-in)	1441.2	ΔMx (k-in)	0
M_y (k-in)	0	ΔMy (k-in)	0
M_{Px} (k-in)	-6722.88	ΔM_{Px} (k-in)	-1169.17
M_{Py} (k-in)	0	ΔM_{Py} (k-in)	163.56

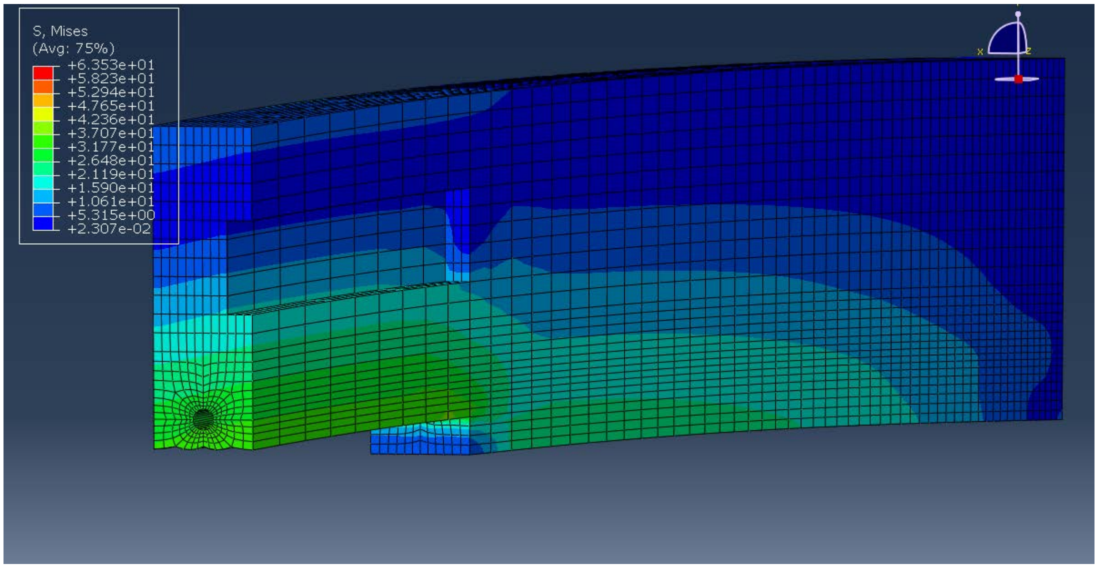


Figure 4.10. Stress contour after introduction of damage

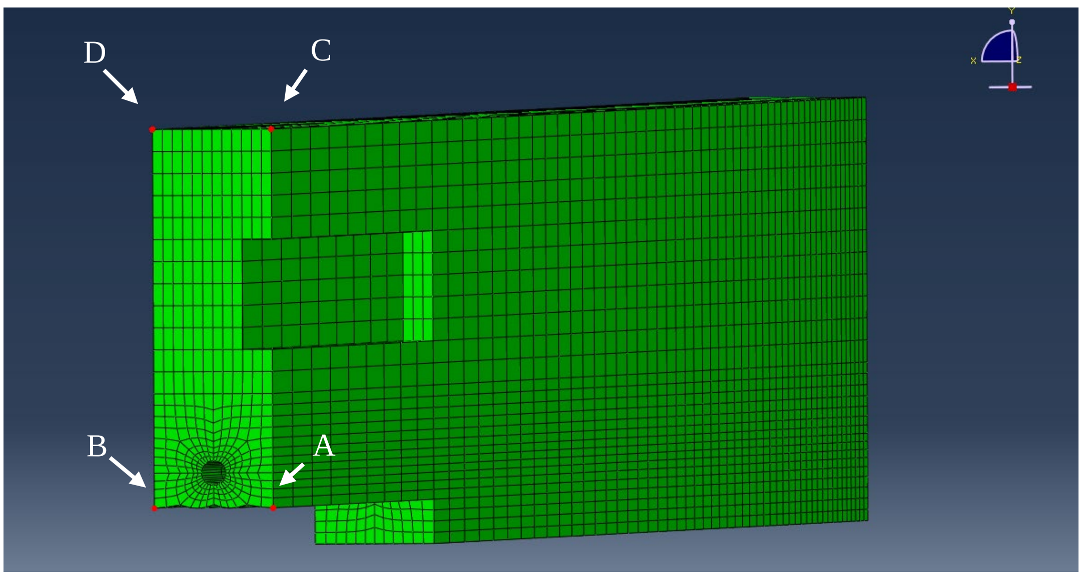


Figure 4.11. Measurement points (A, B, C, D) for calculations of change in stress due to damage

Using above properties and forces (Tables 4.2 and 4.3), and Eq. 4.13, stress change due to damage in the section can be calculated as follows:

$$\Delta\sigma = \Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3$$

$$\Delta\sigma_1 = \frac{(\Delta P)A_t - (\Delta A)P}{(A_t)(A'_{tc})}$$

$$\Delta\sigma_2 = \left(\frac{\bar{B}\bar{F} - \bar{B}\bar{C}}{\bar{B}^2}\right)y + (\Delta y)\left(\frac{\bar{C}}{\bar{B}}\right)$$

$$\Delta\sigma_3 = \left(\frac{\bar{B}\bar{D} - \bar{B}\bar{A}}{\bar{B}^2}\right)x + (\Delta y)\left(\frac{\bar{C}}{\bar{B}}\right)$$

$$\bar{A} = \left(M_y + M_{Py}\right)I_x - (M_x + M_{Px})I_{xy}$$

$$\bar{B} = I_x I_y - I_{xy}^2$$

$$\bar{C} = (M_x + M_{Px})I_y - \left(M_y + M_{Py}\right)I_{xy}$$

$$\bar{D} = \Big[\Big(\left(\Delta M_y\right) + \left(\Delta M_{Py}\right)\Big)I_x + \left(M_y + M_{Py}\right)(\Delta I_x) - \left((\Delta M_x) + (\Delta M_{Px})\right)I_{xy} - (M_x + M_{Px})\left(\Delta I_{xy}\right)\Big]$$

$$\bar{E} = (\Delta I_x)I_y + I_x\left(\Delta I_y\right) - 2I_{xy}\left(\Delta I_{xy}\right)$$

$$\bar{F} = \left[(\Delta M_x + \Delta M_{Px})I_y + (M_x + M_{Px})\left(\Delta I_y\right) - \left(\Delta M_y + \Delta M_{Py}\right)I_{xy} - \left(M_y + M_{Py}\right)\left(\Delta I_{xy}\right)\right]$$

Table 4.5 shows the stress changes using (Eq. 4.13) and results from the Abaqus model at points A, B, C, and D with or without consideration of secondary effects. The results show reasonable agreement between calculated and finite element results. The remaining differences are mainly due to other secondary effects that are not considered in the calculations. These secondary effects are normally not considered in the design process for prestressed bridge members.

Table 4.4. Comparison of stress change due to damage, with or without the secondary effect of deflection, with Abaqus results

Point	x	y	Δσ (ksi)	Δσ (ksi)	Δσ (ksi)
			Without secondary effect	Considering secondary effect	Abaqus
Point A	5.1	-13.5	1.458	1.896	1.890
Point B	-5.1	-13.5	0.897	1.335	1.451
Point C	5.1	18.5	-0.209	-0.418	-0.524
Point D	-5.1	18.5	-0.770	-0.978	-0.970

Chapter 5. Strength Calculations

The flexural strength calculations are performed in this study to determine flexural strength of girder sections under undamaged, damaged and repaired conditions. The effects of loss of section, internal strand splices, CFRP external reinforcement and patch repairs are considered. The strain compatibility approach specified in Section 5.6.3.2.5 of the AASHTO LRFD specifications (AASHTO 2017) is the basis for the calculations performed. The cross section is divided into 0.5 in x 0.5 in cells within a spreadsheet that can represent girder concrete, slab concrete, prestressing steel, strand splice, patch material, and CFRP. The numerical content of each cell defines the material properties associated with the cell.

In sections with at least one axis of symmetry, the principle axes are horizontal and vertical lines that run though the centroid of the cross section. Depending on the failure criterion, each cell is assigned a strain at the strength limit states. The strain is calculated based on the assumption of linearity of strain diagram. The stress associated with the cell strain level is calculated using the relevant stress-strain curve for the material. The stress for each cell is multiplied by the area of the cell that is adjusted by the applicable modular ratio to determine the force in each cell. Integration of those cell forces determines the sectional moment strength associated with the failure criterion.

In sections without any CFRP external reinforcement, the strain limit at the extreme concrete fiber under compression is set to 0.003, based on AASHTO LRFD specifications. When a CFRP external reinforcement is used anywhere in the section, a second strain limit specified in the ACI 440.2R standard (ACI 440.2R 2017) must also be considered.

To calculate the moment strength of sections, the neutral axis (c) of the section under ultimate strength state must be first determined. The PreBARS program uses an iterative process to find the neutral axis, measured with respect to the extreme fiber associated with the strain limit. Considering the compatibility approach, several neutral axis positions (c values) are examined and tension and compression forces in the section are calculated with respect to the corresponding “c” value. The position of neutral axis is determined when the calculated compression and tension forces balance each other.

The total compression force is the summation of forces generated in each cell above the neutral axis (typically concrete elements). The stress in concrete elements is derived based on the value of strain and the stress-strain functions (Eq. 5.1) proposed by Van Gysel and Taerwe (1996).

$$0 \leq \varepsilon \leq \varepsilon_{max}, f_c = f_c'[\left(\frac{E_{it}}{E_0}\right)\left(\frac{\varepsilon}{\varepsilon_0}\right) - \left(\frac{\varepsilon}{\varepsilon_0}\right)^2]/[1 + \frac{E_{it}}{E_0} - 2)(\frac{\varepsilon}{\varepsilon_0})]$$

$$\varepsilon > \varepsilon_{max}, f_c = f_c'/(1 + \left[\frac{\frac{\varepsilon}{\varepsilon_0}-1}{\frac{\varepsilon_{max}}{\varepsilon_0}-1}\right]^2)$$

$$\varepsilon_{max} = \varepsilon_0 \left\{\frac{\left[\frac{E_{it}}{2E_0}+1\right]}{2} + \frac{\left[\frac{E_{it}}{2E_0}+1\right]^2}{4} - \frac{1}{2}\right\}^2$$

$$E_{it} = 21500\alpha_E \left(\frac{f_c'}{10}\right)^{\frac{1}{3}};\ \varepsilon_0 = 700(f_c')^{0.31} * 10^{-6} \qquad \text{(Eq. 5.1)}$$

f_c' : unconfined concrete compressive strength

E_{it}: initial tangent modulus of elasticity

E_0: secant modulus at peak stress ($E_0 = f_c'/\varepsilon_0$)

α_E: coarse aggregate coefficient (1.2 for basalt dense limestone aggregates,

1.0 for quartzite aggregates, 0.9 for limestone aggregates, and 0.7 for sandstone aggregates)

ε_{max}: concrete strain when concrete stress is equal to $0.5f_c'$ on the descending part of the stress-strain curve

ε_0: peak strain of unconfined concrete strength f_c'

Tension force in undamaged sections is generated by strain in the strands. Therefore, summation of forces in each cell containing strands results in tension force in an undamaged section. Tension force in a damaged section is calculated the same as tension force in undamaged section, ignoring severed strands.

Tension force in a repaired section consists of forces in undamaged strands, spliced strands, and CFRP (if applicable).

$$F_{tension} = F_{strands} + F_{spliced-strands} + F_{CFRP}$$

The stress-strain behavior of strands is determined according to the PCI equation (PCI 2017):

$for\ f_{pu} = 250\ ksi$	$for\ f_{pu} = 270\ ksi$
$\varepsilon_s \leq 0.0076, \quad f_s = 28800\varepsilon_s$	$\varepsilon_s \leq 0.0085, \quad f_s = 28800\varepsilon_s$
$\varepsilon_s > 0.0076, \quad f_s = 250 - \left(\frac{0.04}{\varepsilon_s - 0.0064}\right)$	$\varepsilon_s > 0.0085, \quad f_s = 270 - (\frac{0.04}{\varepsilon_s - 0.007})$

In the above stress-strain equations, f_{pu} is the nominal tensile strength of prestressing strand, ε_s is the total strain in the prestressing steel, and f_s is the corresponding stress in the prestressing steel. The total strain in the strand to be used in the above functions consists of two parts: ε_{s1} from the linear strain diagram corresponding to the failure strain being reached at the extreme compression fiber (Figure 5.1), and ε_{s2} due to the effective prestrain in the steel as well as the decompression strain.

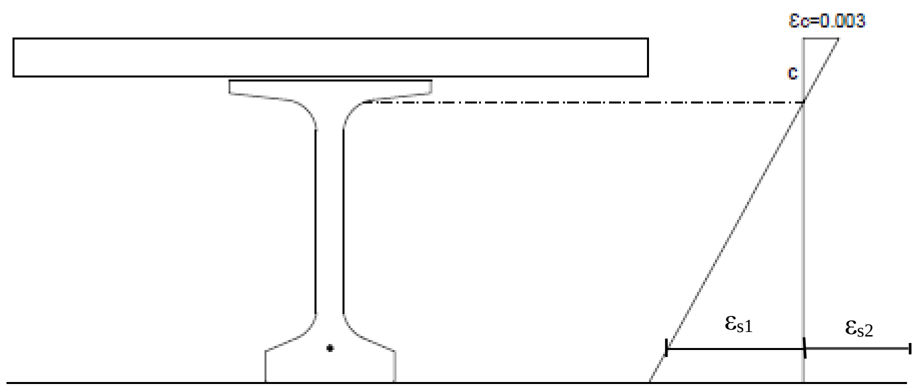


Figure 5.1. Strain diagram at strength limit state associated with crushing of concrete

$$\varepsilon_s = \varepsilon_{s1} + \varepsilon_{s2} \qquad \text{(Eq. 5.2)}$$

ε_{s1} is calculated based on the limit state in which the maximum strain in concrete equals 0.003 in undamaged, damaged, and repaired sections. In sections containing CFRP, a second strain limit associated with debonding of CFRP must be considered. ε_{s1} is calculated based on maximum strain in the CFRP (Figure 5.6 and Figure 5.7).

ε_{s2} is calculated according to the following equation:

$$\varepsilon_{s2} = \varepsilon_{initial} - [\varepsilon_{LT} + \varepsilon_{ES} + \frac{P_{pe}}{A_{tc}E_c}(1 + \frac{e_{tc}^2 A_{tc}}{I_{tc}})] \qquad \text{(Eq. 5.3)}$$

$\varepsilon_{initial}$: initial prestrain

ε_{LT}: strain loss due to long term prestress losses

ε_{ET}: strain loss due to elastic shortening

P_{pe}: effective prestress force

A_{tc}: area of transformed composite section

E_c: modulus of elasticity of concrete

I_{tc}: moment of inertia of transformed composite section

e_{tc}: distance between the centroid of strands and the elastic neutral axis of the transformed composite section

In a retrofitted section or repaired section with external CFRP reinforcement, the strain compatibility diagram is constructed considering the maximum strain in the CFRP not exceeding the debonding strain limit (ε_{d_CFRP}) calculated using Eq. 5.4, (ACI 440.2R-17).

$$\varepsilon_{d\_CFRP} = 0.083\sqrt{f_c'/(nt_{FRP}E)} \qquad \text{(Eq. 5.4)}$$

n: number of CFRP plies

t_{CFRP}: thickness of each CFRP layer

E: modulus of elasticity of CFRP

A linear stress-strain behavior is considered for CFRP (Eq. 5.5).

$$f_{CFRP} = E\varepsilon_{CFRP} \quad \text{(Eq. 5.5)}$$

Since the CFRP is typically installed when the girder is already subjected to dead load and prestress effects, the debonding strain limit given by Eq. 5.4 must be adjusted. The strain in the CFRP is zero while the adjoining concrete is already subjected to dead load and prestress effects. The maximum value of ε_{CFRP} used in the strain compatibility diagram and Eq. 5-5 is therefore calculated using the following equation:

$$\varepsilon_{CFRP} = \varepsilon_{bi} + \varepsilon_{d\_CFRP} \quad \text{(Eq. 5.6)}$$

$$\varepsilon_{bi} = \frac{M_{NCDL}c_{tf}}{I_{tf}E_c} + \frac{M_{CDL}c_{tc}}{I_{tc}E_c} - \frac{P_{pe}}{A_{tf}E_c}\left(1 + \frac{e_{tf}b_{tf}A_{tf}}{I_{tf}}\right) \quad \text{(Eq. 5.7)}$$

M_{CDL}: moment due to composite dead load

M_{NCDL}: moment due to non-composite dead load

c_{tf}: distance of the bottom fiber of FRP from the centroid of non-composite transformed section

c_{tc}: distance of the bottom fiber of FRP from the centroid of composite transformed section

After stresses in all cross-sectional elements are determined, the total tension and compression forces are determined, and the difference between them are calculated.

$$dF = F_{compression} - F_{tension}$$

As c increases, the difference between tension and compression forces decrease until it approaches zero. At a point that the difference changes the sign from negative to positive, dF passes the zero value and the iteration in the loops stops at that point reporting the last two c values, tension and compression forces for further strength analysis. The PreBARS program interpolates the last two c values corresponding to the last two steps in the loop. The moment strength of the section can be calculated using the net value of tension or compression forces and the distance between the centroids of the two forces (Figure. 5.2).

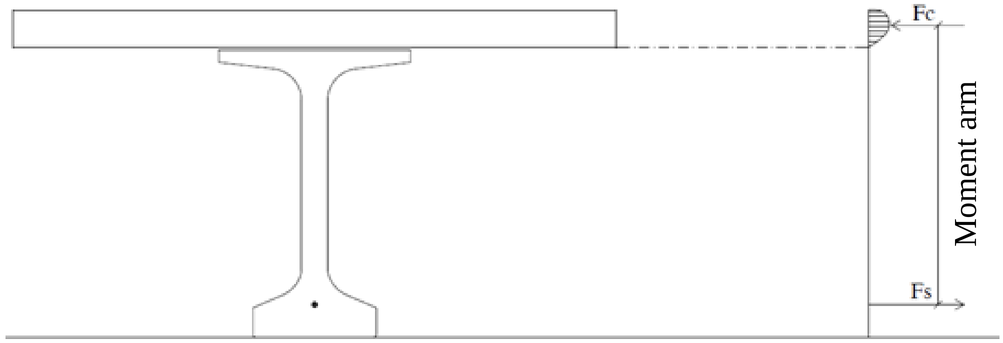


Figure 5.2. Strain and stress diagrams at the strength limit state

A table reporting location of the neutral axis, curvature, tension force, compression force, and corresponding strength for undamaged, damaged, and repaired sections is created in the PreBARS program for cases involving both bottom flange and top flange damage.

Inventory and operating rating factors for strength limit states are calculated and reported in strength tables for undamaged, damaged, and repaired sections according to Eq. 5.8 through Eq. 5.11.

For bottom damage cases:

$$RF_{Inv.} = \frac{C-1.25DC-1.5DW}{1.75(LL+IM)} \quad \text{(Eq. 5.8)}$$

$$RF_{Oper.} = \frac{C-1.25DC-1.5DW}{1.35(LL+IM)} \quad \text{(Eq. 5.9)}$$

where $RF_{Inv.}$ And $RF_{Oper.}$ are the inventory and operating rating factors, respectively. C is the calculated flexural strength capacity of section, DC is the moment due to dead load of components, DW is the moment due to the future wearing surface, LL is the moment due to AASHTO HL-93 live load, and IM is the dynamic allowance (1.33).

For top damage cases, as assessment is made regarding the ability of the girder alone to resist the weight of the fresh deck slab. In this case, the capacity of the section (C) is the strength of the non-composite girder and the slab is treated as a live load with a load factor of 1.75. Therefore,

$$RF\ Constr_{Inv.} = \frac{C-1.25DC_g}{1.75(DC_{slab+build-up})} \quad \text{(Eq. 5.10)}$$

$$RF\ Constr_{Oper.} = \frac{C - 1.25DC_g}{1.35(DC_{slab+build-up})} \quad \text{(Eq. 5.11)}$$

Where $DC_{(g)}$ is the moment due to girder self-weight and $DC_{(slab+build-up)}$ is the moment due to dead load of deck slab and build-up.

The PreBARS program calculates and reports strains at the strength limit state at the following locations: top of the section (top of the girder and slab), centroid of the strands, bottom of CFRP (if applicable), and bottom of the girder for both bottom and top damage situations.

Typical strain diagrams in undamaged, damaged, and repaired sections for top and bottom damage cases are shown in Figure. 5.3 through Figure. 5.9.

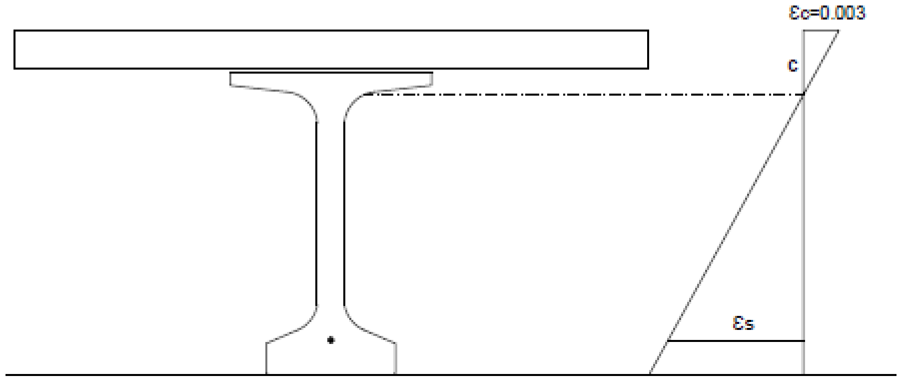


Figure 5.3. A typical strain compatibility diagram for undamaged composite section (before applying bottom damage)

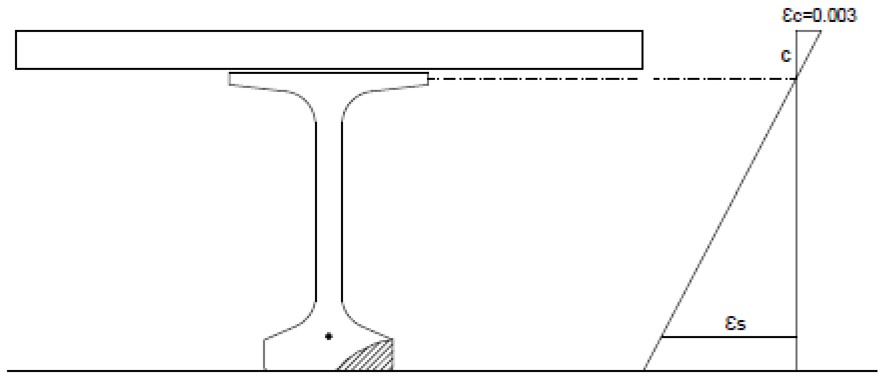


Figure 5.4. A typical strain compatibility diagram for bottom damage composite section

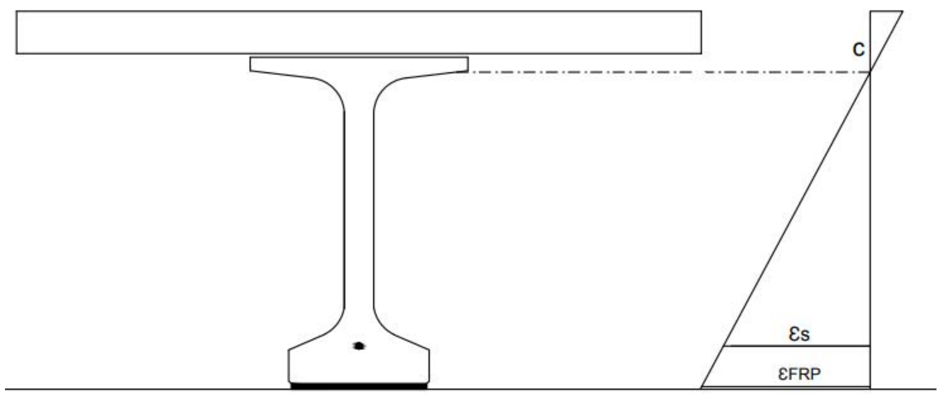


Figure 5.5. Typical strain compatibility diagram for composite section that is repaired after bottom damage (CFRP on soffit)

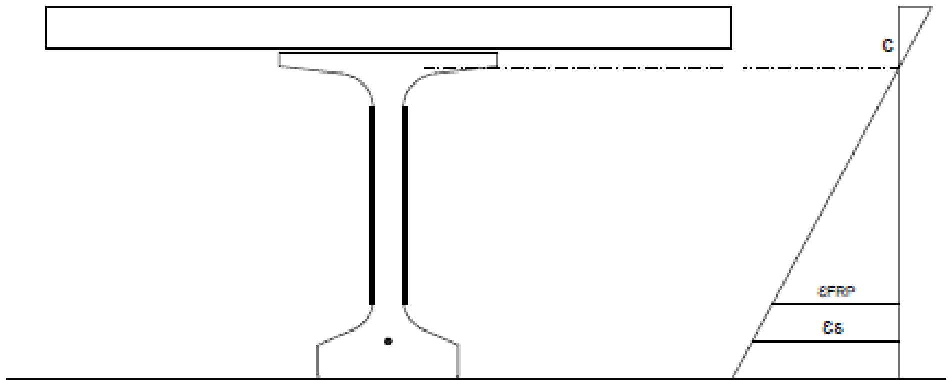


Figure 5.6. Typical strain compatibility diagram for repaired composite section after bottom damage (CFRP on web)

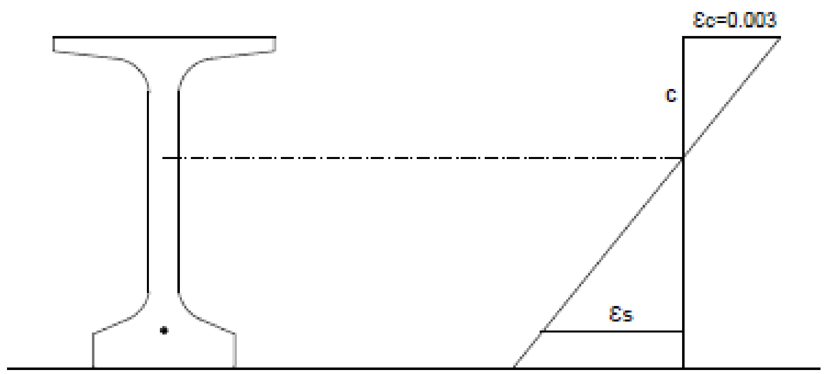


Figure 5.7. Typical strain compatibility diagram for undamaged non-composite section before top damage

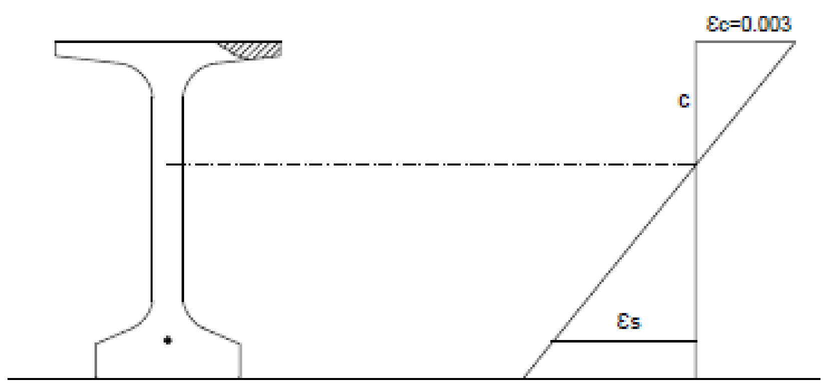


Figure 5.8. Typical strain compatibility diagram for top damage non-composite section

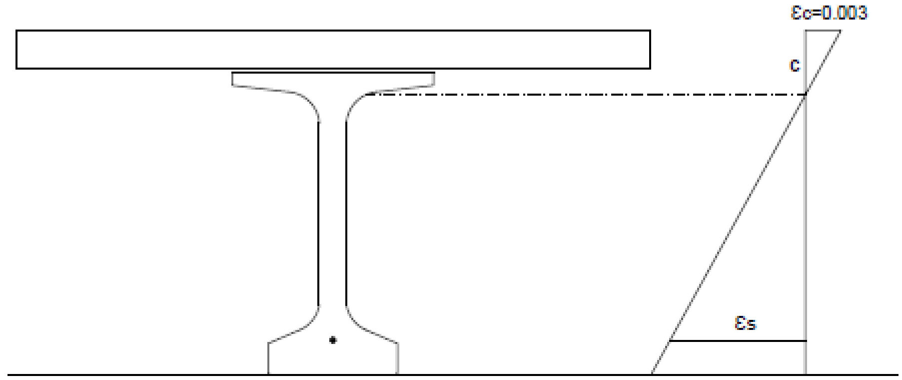


Figure 5.9. Typical strain compatibility diagram for repaired composite section after top damage

Chapter 6. PreBARS Software

6.1 Introduction

The Prestressed Bridge Assessment, Repair and Strengthening (PreBARS) software program is designed to analyze cross sections of prestressed concrete bridge girders at strength and serviceability limit states under undamaged, damaged and repaired conditions. Damage could be at the bottom flange of the girder due to an over-height truck impact, or to the top flange of the girder during removal of deck slab during re-decking operations. Repair options embedded in PreBARS include patching, internal strand splices, and CFRP external reinforcement, either individually or in combination. PreBARS is developed within Microsoft excel using VBA (visual basic application) capability.

The program simulates the pre- and post-damage conditions in prestressed concrete bridge girders and estimates section properties as well as moment strength and service level stresses for the desired cross section based on the provisions of the 8th edition of the AASHTO LRFD Bridge Design Specifications (AASHTO 2017). Simply-supported and continuous span bridges (up to three spans) can be modelled. The maximum programmable length of each span is 160 ft, with an overall maximum bridge length of 480 ft. Span length(s) and location of maximum damage on the bridge must be entered as a whole number in feet (no fractions). Various properties are calculated at 1-ft increments over the entire length of the bridge. In its current (2018) state of development, shear and negative moment effects as well as fatigue are not considered in the PreBARS program.

6.2 Calculation of loads

The program determines moment and shear influence lines at 1-ft increments over the entire length of the bridge using slope-deflection equations. These influence lines are then used to calculate AASHTO LRFD moments and shears due to truck, tandem, lane loads, and special 2-truck loading for negative moments (HL-93 loading). The applicable AASHTO design maximum positive and negative moment envelopes are calculated at 1-ft increments. The AASHTO distribution factor equations and lever rule factors are calculated based on the type of girder, exterior/interior location, skew angle, and other bridge parameters, and the governing distribution factor is determined. Forces associated with Service I, Service III, Strength I, and Extreme Event

II (collision) load combinations are calculated. Figure 6.1 shows an example Strength I moment envelope for a prestressed concrete girder bridge with three equal spans of 120 ft. The various loads for a point located 55 ft from the left support of the bridge is shown in Figure 6.2.

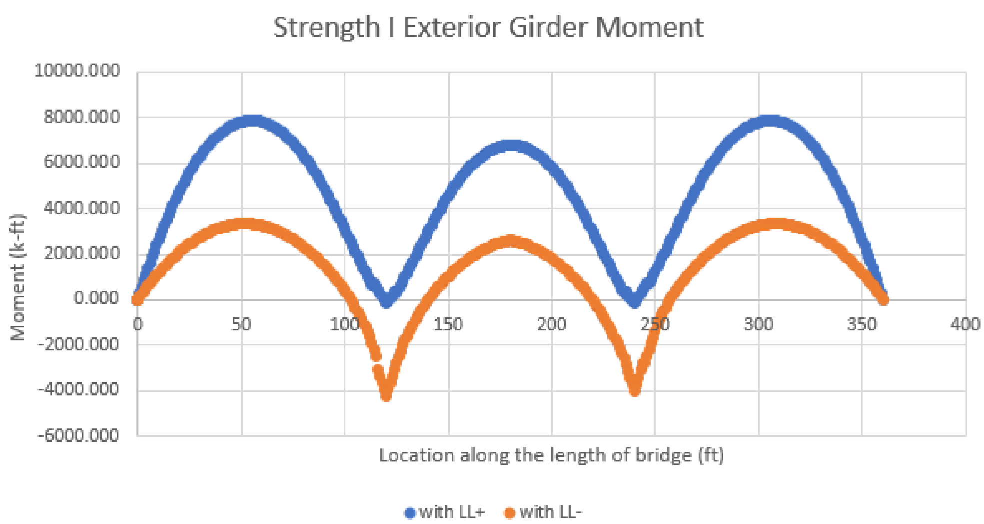


Figure 6.1. Example of Strength I moment envelope for a three-span bridge.

Position		55	ft
Position must be less than the total bridge length			
Exterior Girder	Max Strength I Moment	7881.79	K-ft
Exterior Girder	Min Strength I Moment	3323.08	K-ft
Exterior Girder	Girder DL Moment	1428.14	K-ft
Exterior Girder	NCDL Moment	2832.07	K-ft
Exterior Girder	CDL Moment	414.94	K-ft
Exterior Girder	Service III Live Load Mom	2152.38	K-ft
Exterior Girder	Service I Live Load Mome	2152.38	K-ft
Interior Girder	Max Strength I Moment	8173.14	K-ft
Interior Girder	Min Strength I Moment	3630.83	K-ft
Interior Girder	Girder DL Moment	1428.14	K-ft
Interior Girder	NCDL Moment	3075.99	K-ft
Interior Girder	CDL Moment	414.94	K-ft
Interior Girder	Service III Live Load Mom	2144.63	K-ft
Interior Girder	Service I Live Load Mome	2144.63	K-ft

Figure 6.2. Example of moment information at a location on a 3-span bridge

The 8th edition of the AASHTO LRFD specifications (AASHTO 2017) allows a Service III live load factor of 1.0, in lieu of the long-standing 0.8 factor, when both transformed section properties and refined prestress losses are used. PreBARS gives an option to the user to select either a 1.0 or 0.8 factor for the Service III load combination. This load combination is used when checking tension in prestressed concrete.

In addition to the service and strength load combinations, PreBARS also reports the minimum required strength of the member required for the application of CFRP external reinforcement. This limit is provided in Eq. 9.2 of the ACI 440.2R standard (ACI 400.2R 2017) and is shown blow:

Design strength of existing (unrepaired) member $\geq 1.1M_{DL} + 0.75M_{LL}$ (Eq. 6.1)

Where M_{DL} and M_{LL} are the dead and live load moments at the damaged section. This minimum level of strength is needed to ensure that the beam without CFRP would have sufficient strength following a debonding failure of CFRP.

6.3 Calculation of prestress losses

Refined and approximate prestress losses are calculated in accordance with the AASHTO LRFD specifications (AASHTO 2017). The user has the option to select the method of loss calculation. Both non-transformed and transformed section properties are determined and used in various loss and stress calculations. The composite section properties include conversion of the slab concrete into an equivalent girder concrete by adjusting the effective width of the slab by the ratio of moduli of elasticity of slab and girder concretes. The transformed section properties involve conversion of prestressing steel into equivalent girder concrete using modular ratio between prestressing steel and girder concrete. The transformed section properties are used exclusively for service stress calculation in PreBARS. This would allow an implicit consideration of prestress gains and losses when calculating stresses due to various loads without the need for iteration (PCI 2011).

6.4 User input data

The user is asked to input the basic bridge information including span length(s), girder type (I-girder or box girder), girder size, girder spacing, strand pattern, slab thickness (structural and initial wearing surface), thickness of build-up over the top flange), barrier type, sidewalk area (if any), compressive strength of girder and slab concretes, skew angle, etc. The standard Wisconsin Department of Transportation (WisDOT) girder sizes, strand patterns, barrier types, etc. are embedded into the software and can be selected by the user. Figure 6.3 shows the user input information as cells that are colored yellow. The contents of the pull-down menu items are shown in Figure 6.3 as well. The user input is discussed in detail Appendix A of this report (PreBARS user manual).

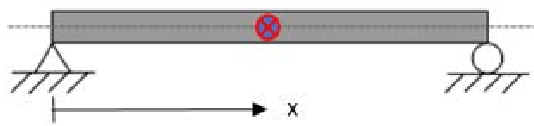
Bridge Information			
Girder Category	W_Girder	<< Pull-down menu <<	
Type of Girder	I-72W"	<< Pull-down menu <<	
Strand Configuration	46 STRANDS-WITH DRAPED	<< Pull-down menu <<	
Type of Railing	SFP LF	<< Pull-down menu <<	
Service 3 LL factor	0.8	<< Pull-down menu <<	
No. of Railing Lines	2		
Exterior / Interior Girder?	Interior	<< Pull-down menu <<	
Sidewalk Cross Sectional Area (ft^2)	0	Enter 0 if no sidewalk	
Span 1	146	ft	
Span 2	0	ft	
Span 3	0	ft	
Girder Spacing	7.5	ft	
thickness of slab	7.5	in	Structural thickness only
Initial wearing surface	0.5	in	or existing wearing surface
Girder - f'c	8	ksi	
Slab f'c	4	ksi	
Girder - f'ci	6.8	ksi	
No. of strands	46		
Size of strands	0.6	in	
Type of strand	Low Relaxation	<< Pull-down menu <<	
Grade of strand	270	ksi	<< Pull-down menu <<
f_y of Strand	243	ksi	
d_e	1.25	ft	Assumed the same in all spans
Skew angle	20	degrees	
N_b	6	No. of beams in cross section	
Build-up	2	in	
Curb-to-curb width	40	ft	
Average annual humidity	70	%	
Concrete age at transfer	1	days	
Concrete age at deck placement	90	days	
Concrete age - current	20000	days	
Prestress Loss Method	Approximate Method	<< Pull-down menu <<	
Girder concrete unit weight	0.15	kcf	
Deck concrete unit weight	0.15	kcf	
Damage Location (x)	73	ft	

Figure 6.3. PreBARS user input

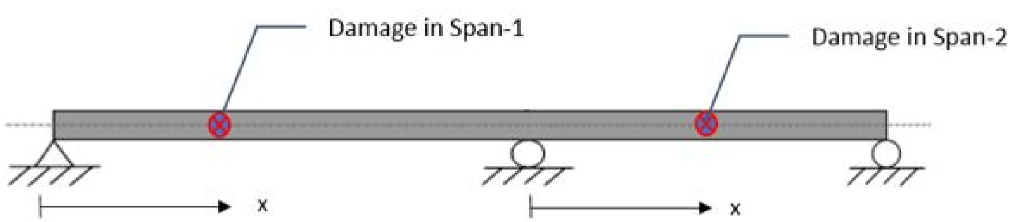
The user provides information regarding damage to the prestressed girder (at the point of maximum damage) by first specifying whether bottom or top damage has occurred (Figure 6.4). The user also specifies whether any strands have been severed. It is recommended that any strand with damaged, bent, or cracked wire(s) be considered severed for these analyses. The location of the damage (or maximum damage) is input as span number and distance from the beginning of the span. The beginning of the span is defined as the first point on the span along the direction of stationing (Figure 6.5).

Damage Information		
Damage Type	Bottom_Damage	<< Pull-down menu <<
Damaged Span	1	Enter 1, 2 or 3?
Damage Location (x)	73	ft
Strands severed?	Yes	<< Pull-down menu <<

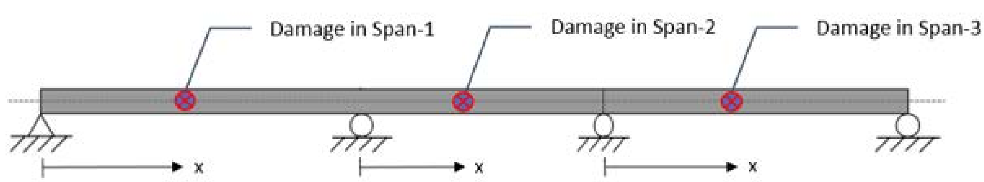
Figure 6.4. User input regarding damage



a) Single-span bridge



b) Two-span continuous bridge



c) Three-span continuous bridge

Figure 6.5. Location of damage.

User-defined information regarding repairs are provided as shown in Figure 6.6. The user is asked to provide information on CFRP repairs (if any) and patch information. The user can specify the location for the application of CFRP (on the beam soffit, on the two sides of the web, or on the side faces of the bottom flange), properties of CFRP per layer (thickness and modulus of elasticity), and the number of layers of CFRP. The provisions of the ACI 440.2R standard (ACI 440.2R 2017) is incorporated within PreBARS including the debonding strain and strength calculations.

The properties of patch material (modulus of elasticity and strength) are also provided by the user. As discussed later, the use of internal strand splices is specified by the user on a graphical display of the cross section. The strand splice restoration factor is calculated by PreBARS based on the size of strand. For strands that have a diameter of 0.5 in or less, the restoration factor is 0.85 based on recommendations from prior research. Currently (2018), there is no commercially-available strand splice for 0.6 in strands in the United States. Instead, a reduction coupler together with a 0.5 in splice is used. The restoration factor for 0.6 in strand is therefore reduced to 0.60 (ratio of 0.5 in to 0.6 in strand areas times the reduction factor of 0.85).

Repair Information		
Apply FRP?	Yes	<< Pull-down menu
Location of FRP Application	Only Bottom of Girder	<< Pull-down menu
How many Plies of FRP	1	<< Pull-down menu
FRP Thickness	0.047	in
E_FRP	2.32E+04	ksi
E_Patch	4.89E+03	ksi
f'c_Patch	6.5	ksi
Strand Splice Restoration Factor	60	%
Apply Preload?	No	<< Pull-down menu
Direction of preloading Truck		

Figure 6.6. User input regarding repair

6.5 Preload application

Preload is commonly applied prior to patch repairs when damage to the bottom flange has occurred. Since patch material is applied when the damaged girder is under the action of prestressing as well as girder and slab dead loads, any subsequent live loads would generate tension in the patch material, which could then exceed the allowable tension under Service III loading. An application of preload can reduce or eliminate this tension. This typically involves strategic placement of a loaded truck (or load(s) applied through hydraulic jacks) over the damaged area and maintaining the load while the patch hardens. Subsequent removal of preload would introduce compressive stresses in the patch based on the modulus of elasticity of the patch material. In PreBARS, the user can specify whether preload would be applied prior to patching. To simplify the preload application, it is assumed that a 4-axle dump truck (typically loaded with sand) would be used to apply the preload. In the current version of PreBARS, the total weight of the truck is assumed to be 80,000 lbs with the axle loads and spacings as shown in Figure 6.7. PreBARS would provide guidance as to the direction that the truck should face to maximize the moment introduced into the damaged girder. The axle loads and spacings can be changed (as an internal change to the program) if desired by the user.

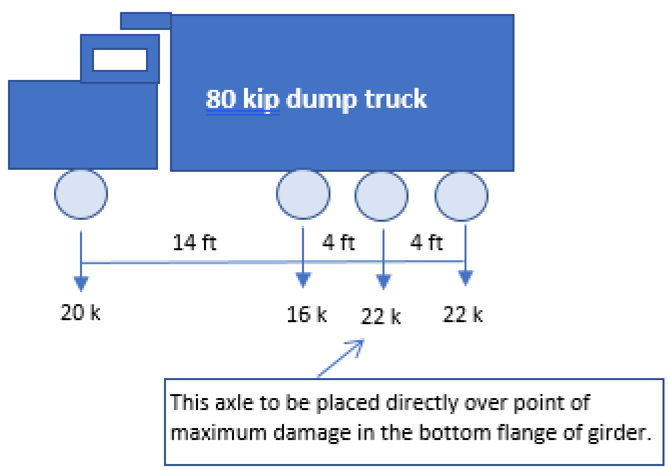


Figure 6.7. Preload 4-axle truck assumed in PreBARS

6.6 Generation of Undamaged Section

A cross section of an exterior or interior girder from a bridge can be modeled within the excel spreadsheet, in which cells representing 0.5 in x 0.5 in square blocks make up the cross section with identifying cell numbers and colors. Girder concrete, deck concrete, prestressing strand, patch material, CFRP, strand splices, etc. are given unique numbers in cells corresponding to their specific position(s). With the approach of allocating numbers and coordinates to each cell, the section properties, stress and strength can be automatically calculated for undamaged, damaged, and repaired sections. The program is designed to relate each cell's row and column with a specific position in the coordinate system, and to specified material properties associated with the numerical content of the cell.

When the girder type, size and strand pattern is selected by the user (from pull-down menus), the undamaged cross-section is automatically generated by the program. The girder cross-section is populated in a specific worksheet based on the predefined geometry of the girder. However, due to complexity of showing curves and haunches as a graphic in excel viewport, flange angles and haunches are modeled as step-wise straight lines over short distances. An example of the Wisconsin 72W girder is illustrated in Figure 6.8. The user can then calculate the non-transformed, transformed, non-composite and composite section properties by clicking a button.

PrePARS allocates numbers to each material with specific properties assigned to them for use in the calculation of section properties as well as strength and stress calculations. A value of "1" with a light gray color is assigned to all cells representing girder concrete. Slab concrete is identified with a number "2" in dark gray color, as shown in Figures 6.8 and 6.9a. Strands are given a value of "3" and a black color (Figure 6.8 and 6.9. b). The strand configuration is generated based on the strands' positions with respect to a reference point in the cross section. Common strand patterns for all Wisconsin standard I-girders and box girders are predefined within the PreBARS program. A matrix is defined for each strand pattern for various girders, which indicates the coordinates of each strand with respect to the reference point. Each strand is shown in a single cell regardless of the sectional area of the strands. Area of the cell corresponding to strands are assigned the actual sectional area of the strand in the calculations.

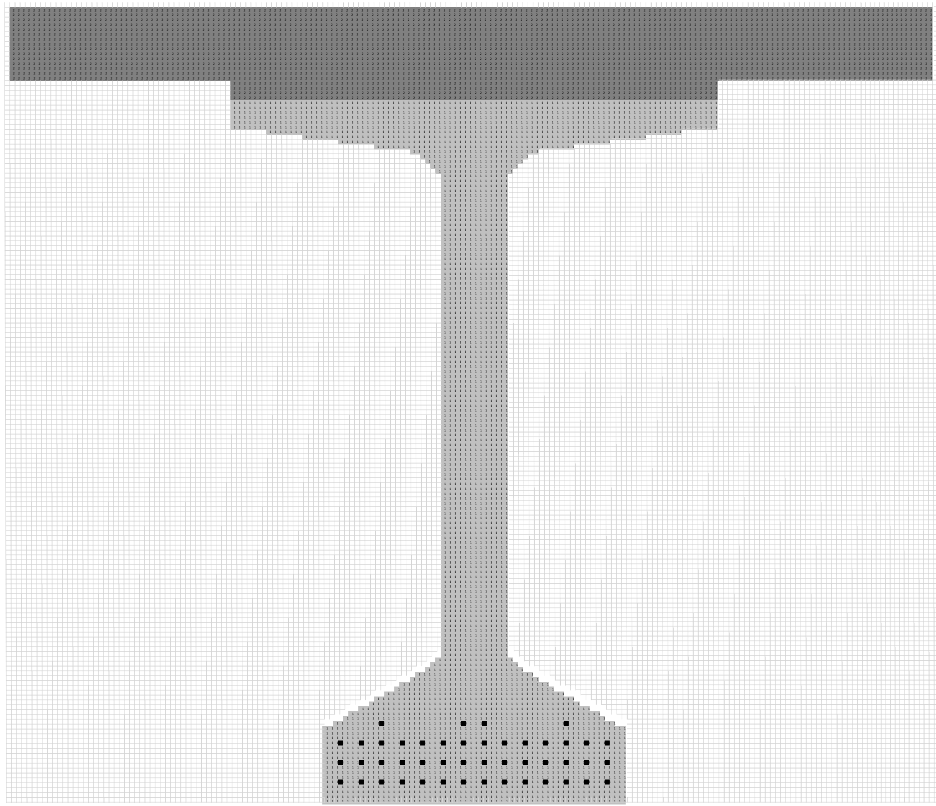
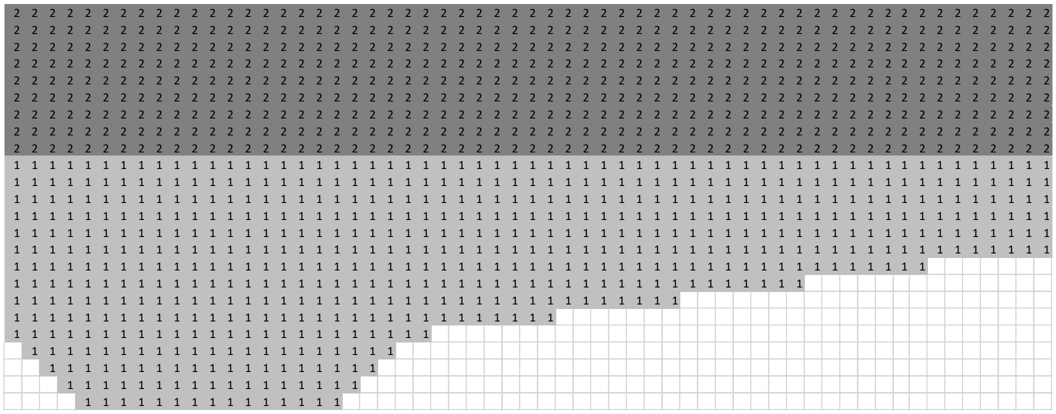


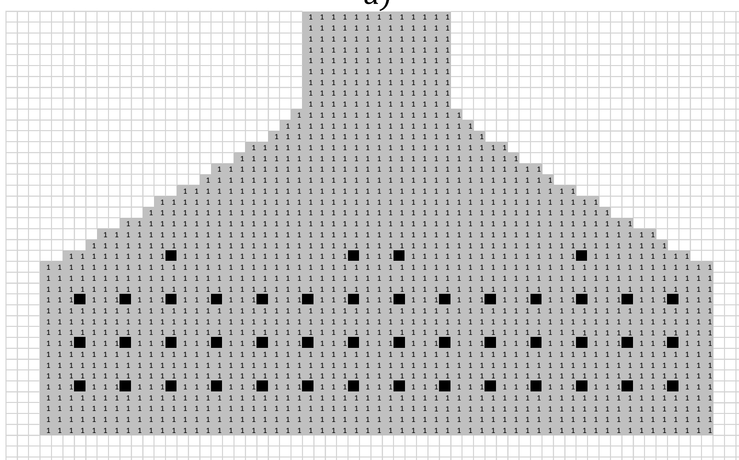
Figure 6.8. Undamaged Wisconsin 72W girder/slab cross-section with forty-six 0.6-in-diameter strands

6.7 Generation of Damaged Section

PreBARS can simulate damage (symmetrical or unsymmetrical damage) in prestressed sections using a simple and practical approach. Loss of prestressing strands and concrete (through spalling and cracking) is modelled through the elimination of cross sectional elements (zeroing out of affected spreadsheet cells) using mouse clicks on the computer screen. The damage is simulated through simple deletion of cell contents in the damaged areas. In addition to the position of centroid, cross sectional area, and moments of inertia about the horizontal and vertical axes through the centroid, PreBARS also calculates the product of inertia, which is a non-zero number when the section does not have at least one axis of symmetry. Figures 6.10 and 6.11 show example simulations of damage to bottom and top flanges of a prestressed girder, respectively.



a)



b)

Figure 6.9. Assignment of materials to spreadsheet cells; a) cells for girder and slab concrete have values of “1” (light gray) and “2” (dark gray), respectively; b) Cells containing strands have a value of “3” in black color

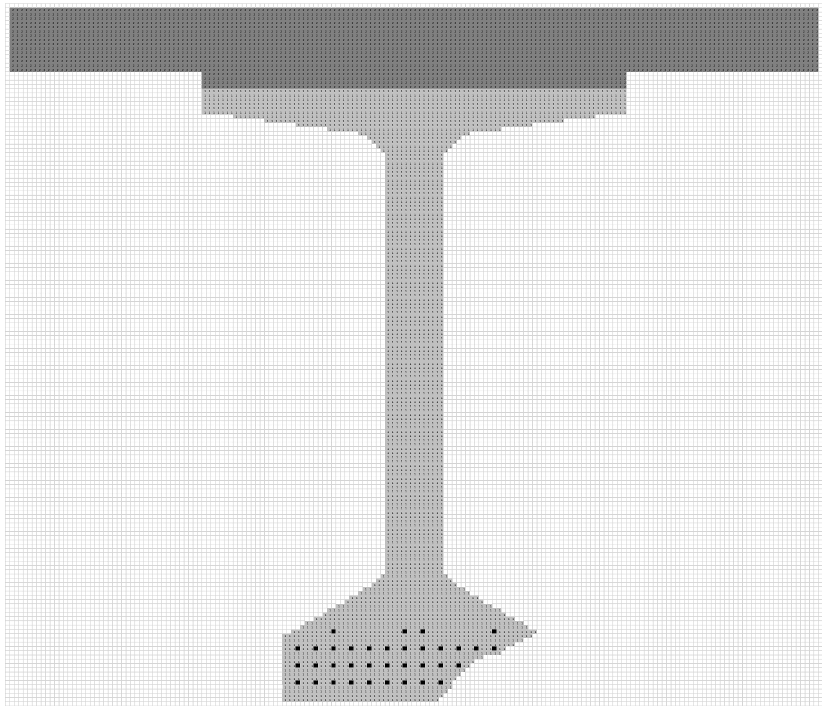


Figure 6.10. An example of modeling bottom damage with spalled concrete and severed strands

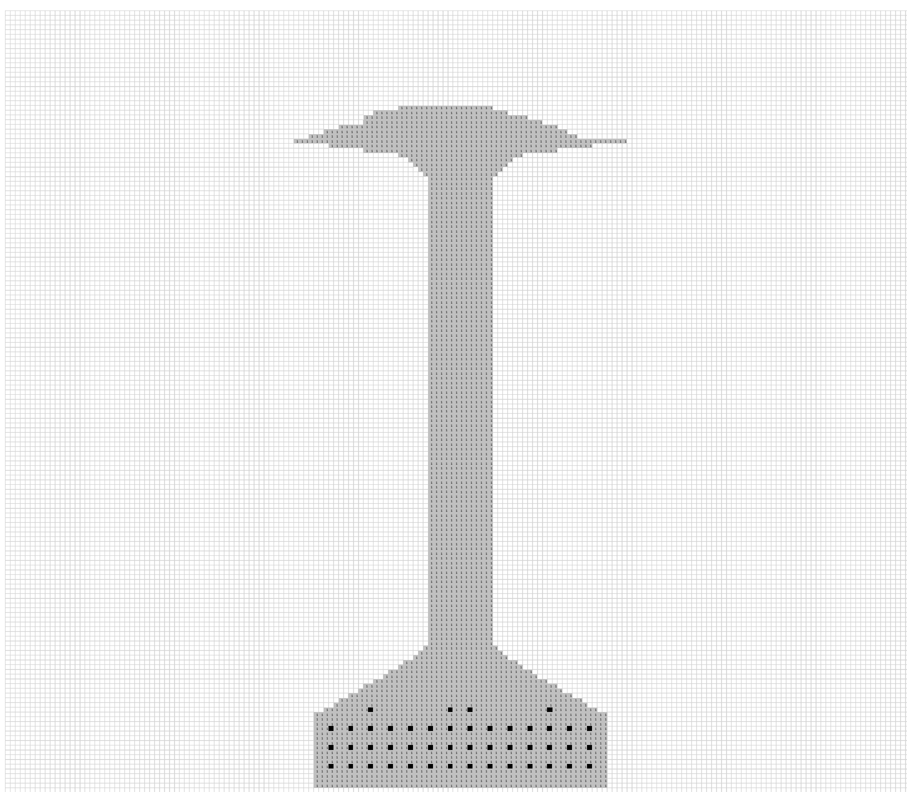


Figure 6.11. An example of modeling top damage

As discussed earlier, damaged area and severed strands would be deleted (value of zero) to allow analysis of the damaged section. However, severed strands would be identified by the number "33" in the repaired girder, for reasons to be discussed in the following section.

6.8 Generation of Repaired Section

Repair are simulated and analyzed in the PreBARS program by first "adding" patch repair materials to the damaged section. All cells representing damaged concrete areas (which were earlier zeroed out due to loss) would be replaced with content representing patch materials. This is done though automatic placement of a number "4" in each of the affected cells and shading the cell with a medium gray color (Figure 6.12). The number "4" in any cell would represent the properties of the patch material that was input by the user.

In the program, all cells containing severed strands are automatically populated with a numerical value of "33" and shown in white color during the repair analysis. If an internal strand splice were to be applied on any of the severed strand, the value "33" allocated to the corresponding cell should be manually replaced by the user with the number "34". The cell would then be shown in brown color within the repaired section. The user would select which of the severed strands should be spliced and would manually over-write the existing pre-existing cell value (34 instead of 33).

The user's input would be needed in selecting the spliced strands since not all strands may be spliced within the same damaged cross section. Although staggering the splices would allow more work space, in general, it is difficult to place and tension splices for multiple side-by-side strands. Specifically, the use of torque wrenches to stress the strands would become difficult in limited spaces. Depending on the type of strand splice used, and the proximity of severed strands to each other, the user would need to select the spliced strands.

It should be noted that the efficiency and strength of spliced strands would not be the same as the undamaged strands. Prior research indicates that a factor of 0.85 should be used to account for possible degradation in strength and fatigue resistance. Considering that a splice for 0.6-in strand is not be commercially available at the time of writing this report (2018), the option to use a strand reducer along with a 0.5-in strand splice may be considered. Thus, the PreBARS program automatically assigns a splice restoration factor of 0.6 for 0.6-in strands, and a splice restoration factor of 0.85 when smaller strands are spliced (0.5-in or smaller).

The user also has the option to specify the use of CFRP for repair and strengthening. The user would select the location of CFRP (soffit, the two sides of web, the side faces of bottom flange), and the program automatically assigns a numerical value of "5" (green color) at the selected location(s) for the CFRP. PreBARS assigns the defined material properties to each cell through the cell value. Addition of CFRP to the repair section is shown as a single cell layer in a graphical viewport, regardless of the number of layers (and the thickness of each layer) specified in the input data. However, the actual total CFRP thickness would be incorporated in the calculations. CFRP can be added either as a repair to a damaged girder or as a retrofit to enhance strength of undamaged girders.

6.9 Section Properties

Section properties are calculated for all generated sections (undamaged, damaged, and repaired), based on the coordinates of each cell and the cell values (representing material properties and modular ratios associated with each cell). Possible cell values are: Blank or "0" cell (spalled or cracked material), "1" (intact girder concrete), "2" (intact slab concrete), and "3" (intact prestressing strand), "33" (severed prestressing strands), "4" (patch material), "5" (CFRP material), and "34" (internally spliced strands). Top and bottom damage cases are considered. Non-composite (without deck slab) and composite (with deck slab) section properties are calculated for undamaged, damaged, and repaired states. The area of each cell within the cross section is adjusted according to the modular ratio (modulus of elasticity of the cell material divided by the modulus of elasticity of girder concrete) to convert the area into equivalent girder concrete. Both non-transformed and transformed section properties are calculated. In non-transformed section properties, the area of prestressing steel is not converted into equivalent girder concrete (only concrete area is considered). Non-transformed section properties are calculated for the undamaged sections only. For transformed section properties, the area of prestressing steel is converted into equivalent girder concrete using the modular ratio for prestressing steel. The transformed section properties are used for calculation of stresses exclusively. Use of transformed section properties eliminates the need for iterations for prestress losses, and implicitly considers effects of prestress gains and losses on concrete stresses (PCI 2012).

Undamaged section properties include those associated with the girder alone at transfer of prestress (initial) and final (after prestress losses) conditions (non-composite) as well as final composite section properties (Table 6.1). Explanations of various parameters used in Table 6.1 are given in the list of notations at the beginning of this report. Reported section properties include the horizontal (x) and vertical (y) positions of the centroid, cross sectional area, moment of inertia about the x and y axes (I_x and I_y), product of inertia (I_{xy}), radius of gyration (r_x and r_y), position of centroid of prestressing force, eccentricity of prestressing force, etc. After calculating section properties, the data is transferred to another spreadsheet for AASHTO LRFD load and moment calculations.

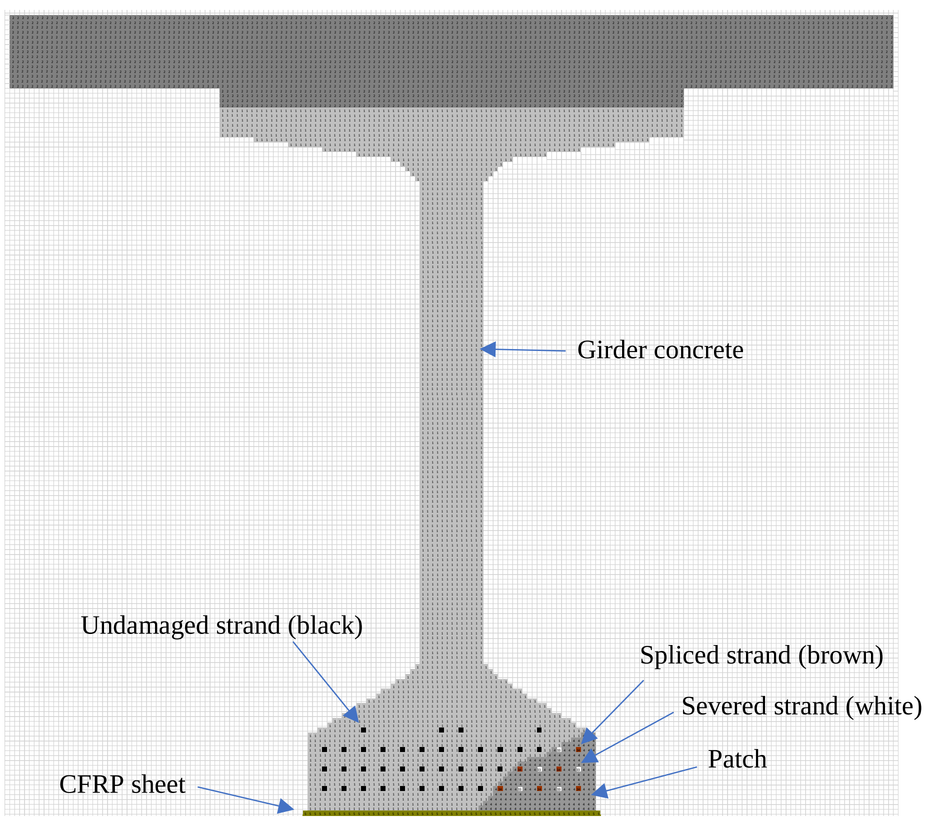


Figure 6.12. Repaired section for a bottom-damaged girder

Table 6.1. Section properties reported in PreBARS for initial and final states before damage

Designed Section									
Before Applying Deck						After Applying Deck			
The Girder		At Initial Strength		At Final Strength		At Final Strength		At Final Strength	
Girder Only		Noncomposite (girder + strands)		Noncomposite (girder + strands)		Composite non-transformed		Composite transformed	
A_g	in^2	A_{ti}	in^2	A_{tf}	in^2	A_c	in^2	A_{tc}	in^2
I_g	in^4	I_{ti}	in^4	I_{tf}	in^4	I_c	in^4	I_{tc}	in^4
Y_t	in	Y_{tti}	in	Y_{ttf}	in	Y_{tc}	in	Y_{ttc}	in
Y_b	in	Y_{bti}	in	Y_{btf}	in	Y_{bc}	in	Y_{btc}	in
S_t	in^3	S_{tti}	in^3	S_{ttf}	in^3	S_{tc}	in^3	S_{ttc}	in^3
S_b	in^3	S_{bti}	in^3	S_{btf}	in^3	S_{bc}	in^3	S_{btc}	in^3
I_y	in^4	I_y	in^4	I_y	in^4	I_y	in^4	S_{dtc}	in^3
I_{xy}	in^4	I_{xy}	in^4	I_{xy}	in^4	I_{xy}	in^4	I_y	in^4
r_x^2	in^2	r_x^2	in^2	r_x^2	in^2	r_x^2	in^2	I_{xy}	in^4
r_y^2	in^2	r_y^2	in^2	r_y^2	in^2	r_y^2	in^2	r_x^2	in^2
x_{b_left}	in	x_{b_left}	in	x_{b_left}	in	x_{b_left}	in	r_y^2	in^2
x_{t_left}	in	x_{t_left}	in	x_{t_left}	in	x_{t_left}	in	x_{b_left}	in
		$\bar{x}_{sl}$		$\bar{x}_{sl}$				x_{t_left}	in
		$\bar{y}_{sb}$		$\bar{y}_{sb}$				$\bar{x}_{sl}$	in
		e_ti		e_tf				$\bar{y}_{sb}$	in
								e_tc	in
Total Height	in	Total Height	in	Total Height	in	Total Height	in	Total Height	in

6.10 Bottom Flange Damage

Damage to the bottom flange is applied on a transformed composite section. Damage can be applied to the concrete section alone or may involve severed strands. The user would introduce damage to the section based on the inspector's report of the extent of damage at the cross section with maximum damage. Spalled concrete as well as any cracking that would extend vertically (or nearly vertically) at or near the section with the maximum damage should be considered as lost concrete section when modeling damage. Other types of damage and/or cracking should be examined by the engineer to decide the extent of concrete removal from the cross section.

Severed strands can be spliced which recovers partial strength of undamaged strands. Repair of bottom damage includes patch of damaged girder concrete, splicing strands, and installation of CFRP. The PreBARS program provides options for FRP to be installed on the soffit or web of the girder.

A table of section properties for undamaged, damaged, and repaired section is created to show the change in the section properties during the process of damage and repair in the section (Table.6.2). Fig. 6.10 shows an example of bottom damage created in PreBARS.

6.11 Top Flange Damage

Top flange damage is coded to be applied on a transformed non-composite section, assuming the entire slab is removed before the top flange damage is applied. It is considered that the damage would be applied to the non-composite section as well. Repair is applied by patching and placing slab at the same time. In the repair process for top damage, no FRP was considered in the examples in this report, however, the option of FRP installation for top damage is provided in the program.

For more detail on the software and steps on bottom and top damage application refer to the Appendix at the end of this report.

Chapter 7. Case Studies

To understand the effect of damage and repair on the strength and service stresses in prestressed bridges, two example structures were modelled in the PreBARS program. Each was subjected to a series of damage scenarios, first at the bottom flange and then at the top flange. Bottom damage cases were assessed up to approximately 25% loss of strands (i.e. 25% of strands severed). Top damage cases involved up to a maximum of 50% of the top flange removed. Undamaged, damaged and repaired conditions were examined. In the case of bottom damage, repair involved patching, internal splices of approximately half of the strands (congestion may not allow more than half of adjacent strands to be effectively tensioned).

7.1 Structures

Structure 1: A 146-ft span simply supported bridge utilizing an interior 72in-deep Wisconsin prestressed I girder (Wisconsin Type 72W") with a girder spacing of 7.5 ft., a structural slab thickness of 7.5 in, initial wearing surface of 0.5 in (i.e. total slab thickness of 8 in), and design concrete compressive strengths of 6.8 ksi for the girder and 4.0 ksi for the slab. Forty-six 0.5 in, Grade 270 low-relaxation strands were used in a standard Wisconsin 46-strand pattern. The girder was draped. The location of damage (study location) was at midspan (73 ft from the support). Two lines of railing were used, each conforming to the Wisconsin "SFP LF" standard. The skew angle was 20 degrees. The curb-to-curb width was 40 ft and six girders were used in the cross section.

Structure 2: A 50-ft span simply supported bridge utilizing an interior AASHTO 36-in prestressed I girder (Wisconsin I-36") with a girder spacing of 8.0 ft., a structural slab thickness of 7.5 in, initial wearing surface of 0.5 in (i.e. total slab thickness of 8 in), and design concrete compressive strengths of 5.8 ksi for the girder and 4.0 ksi for the slab. Sixteen 0.6 in, Grade 270 low-relaxation strands were used in a standard Wisconsin 16-strand pattern. The girder was not draped. The location of damage (study location) was at midspan (25 ft from the support). Two lines of railing were used, each conforming to the Wisconsin "SFP LF" standard. The skew angle was zero degrees. The curb-to-curb width was 40 ft and six girders were used in the cross section.

7.2 Damage Scenarios

Table 7.1 shows the various damage scenarios examined for the top and bottom damage cases for each structure (girder type). The undamaged and repaired conditions were also examined in each case. For bottom damage, repair involved patching the spalled area, placing internal strand splices at approximately half of the severed strands, and using external CFRP reinforcement either at the soffit of the beam or on the two faces of the web. Various strength and stress parameters were determined for each damage and repair scenario. The option to preload or not to preload is considered as well. When preload is not used, the patch material is expected to sustain large tensile stresses under the action of live load. Preloading can mitigate the development of tensile stresses in the patch and elsewhere. It is assumed that CFRP is applied after preload (if any) has been removed.

Table 7.1. Damage scenarios considered for the two structures (girder sizes)

72W" Girder		36" Girder	
Approximate Bottom Damage*	Approximate Top Damage**	Approximate Bottom Damage*	Approximate Top Damage**
0	0	0	0
5%	10%	5%	10%
10%	20%	10%	20%
15%	30%	---	30%
20%	40%	20%	40%
25%	50%	25%	50%

*Approximate % loss of strands (severed strands). Actual loss based on number of severed strands.

**Approximate loss of the area of top flange

For the top damage cases, repair involved patching of the top flange and placement of the deck. The flange may be patched and allowed to cure before casting of the slab, or it can be "patched" with the placement of the deck slab.

Figures 7.1 through 7.6 shows examples of damage and repair conditions in various damage cases for both structures.

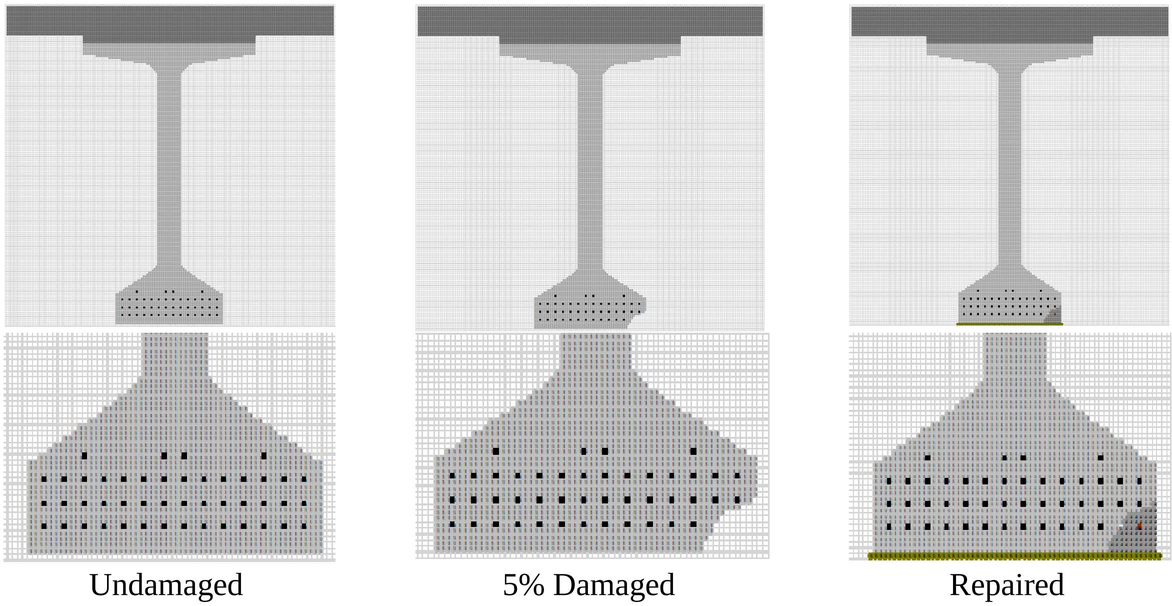


Figure 7.1. Structure 1 under the 5% bottom damage case

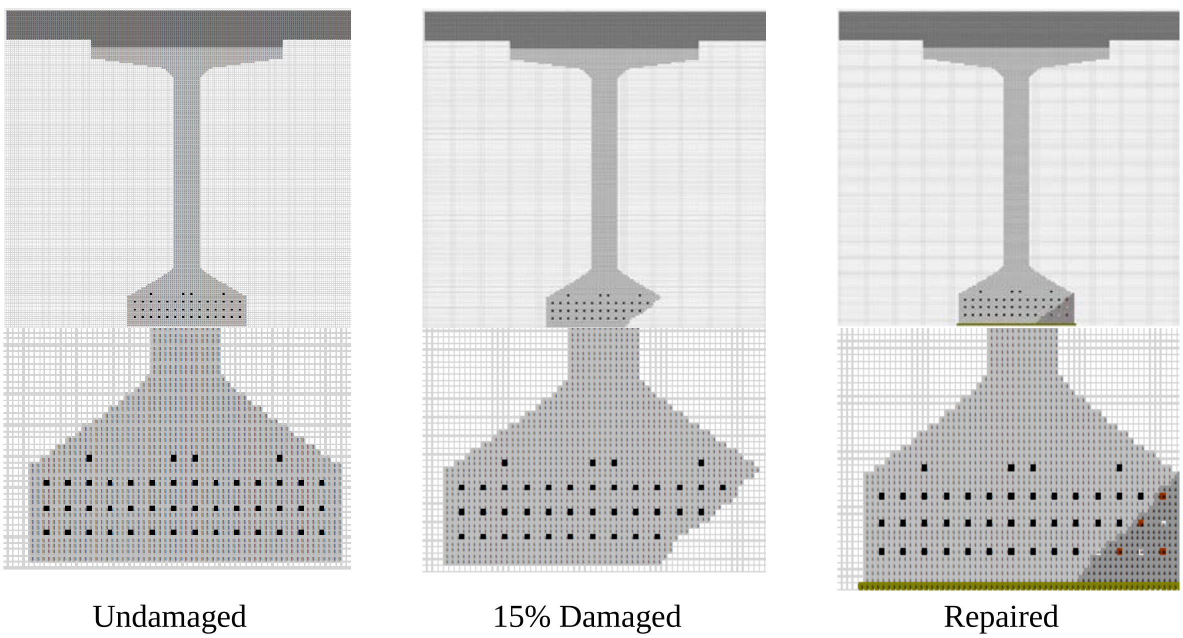


Figure 7.2. Structure 1 under the 15% bottom damage case

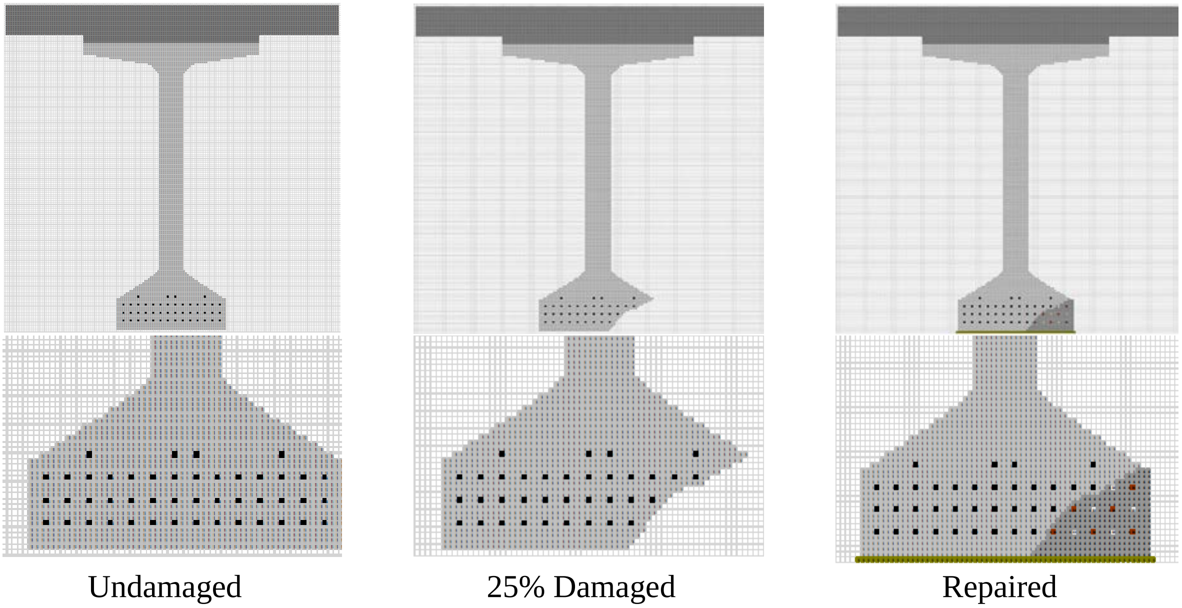


Figure 7.3. Structure 1 under the 25% bottom damage case

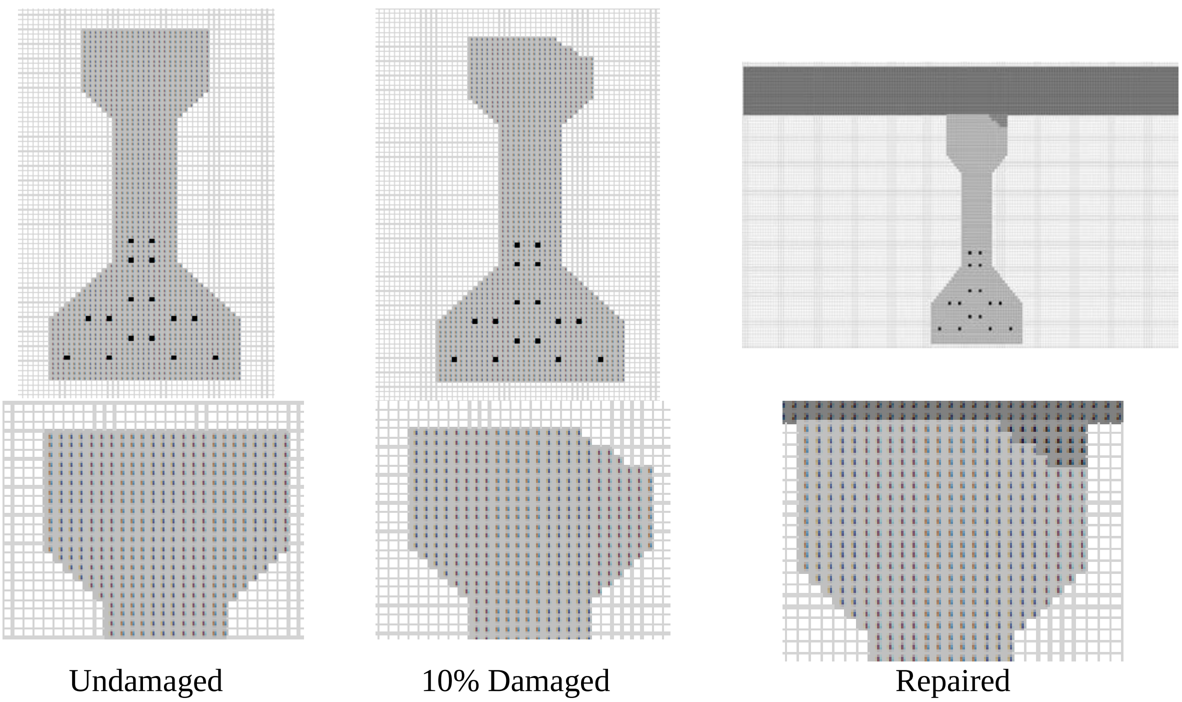
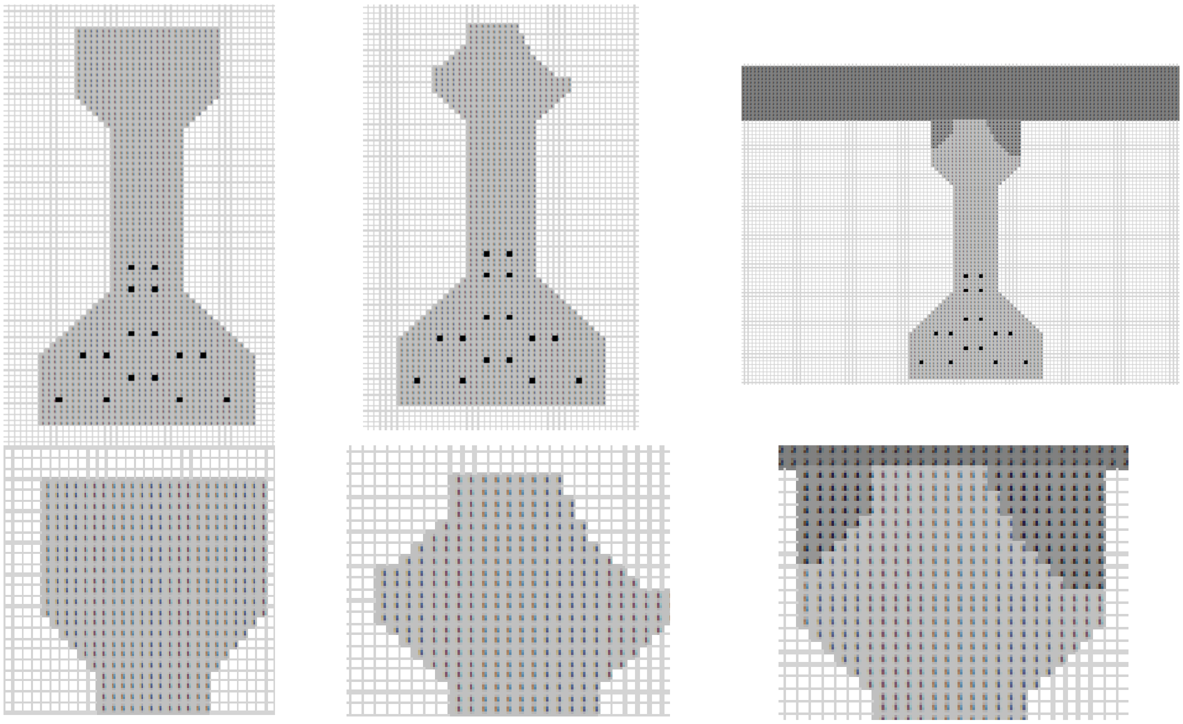
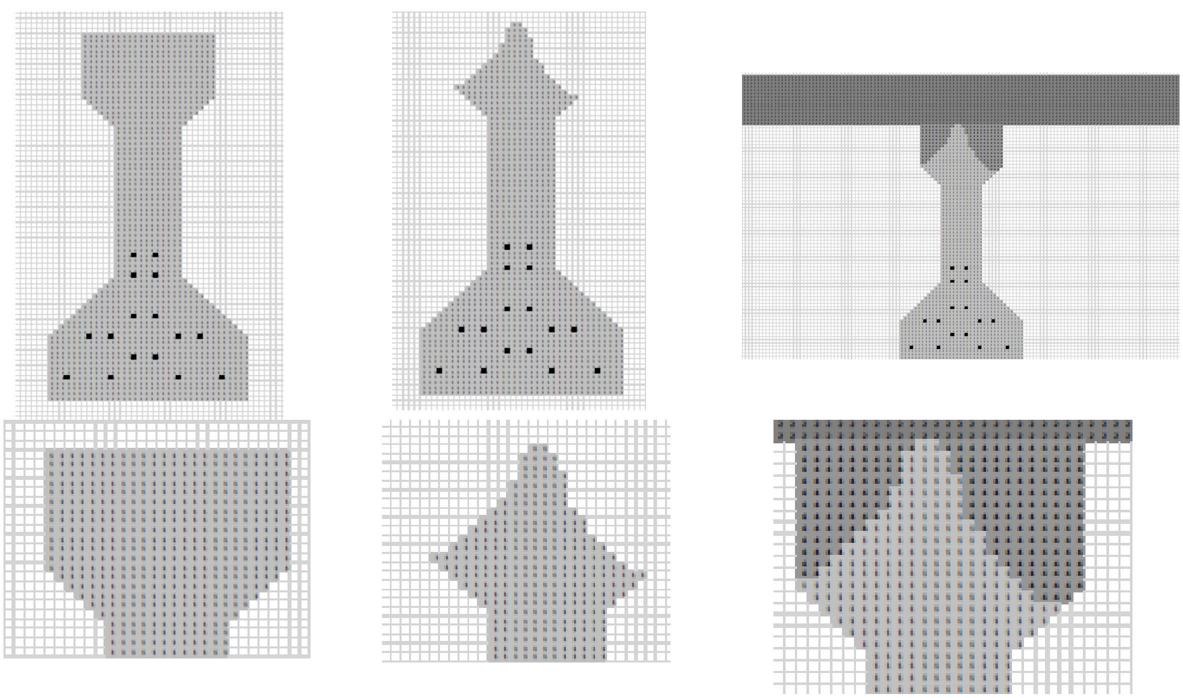


Figure 7.4. Structure 2 under the 10% top damage case



Undamaged 30% Damaged Repaired

Figure 7.5. Structure 2 under the 30% top damage case



Undamaged 50% Damaged Repaired

Figure 7.6. Structure 2 under the 50% top damage case

7.3 Case Study Results

7.3.1 Structure 1 Bottom Damage

Figure 7.7 shows the variations of various moment strengths with respect to the loss of prestressing steel. The damaged moment strength (Mn-D), repaired moment strength when some of the strands are spliced only (Mn-R-Spl), repaired moment strength when CFRP is applied on the soffit plus application of some strand splices (Mn-R-Soffit), and repaired moment strength when CFRP is applied on the web plus application of some strand splices (Mn-R-Web). The AASHTO LRFD Strength I moment (MStr-I) and Extreme Event II (MExEv-II) are also shown as horizontal lines on the graph. The undamaged strength is equivalent to damaged strength when % loss of strand is zero. The loss of prestressing strands up to approximately 25% substantially reduces the moment strength. However, because of the inherent over-strength in the prestressed beam, the available capacities are higher than the Strength I moment requirement. Despite this, the girder should be repaired. The repair using selected splices increases the moment strength, but not to the level of the original strength. The addition of one layer of preformed CFRP further increases the moment strength, but still to a level that is below the initial strength.

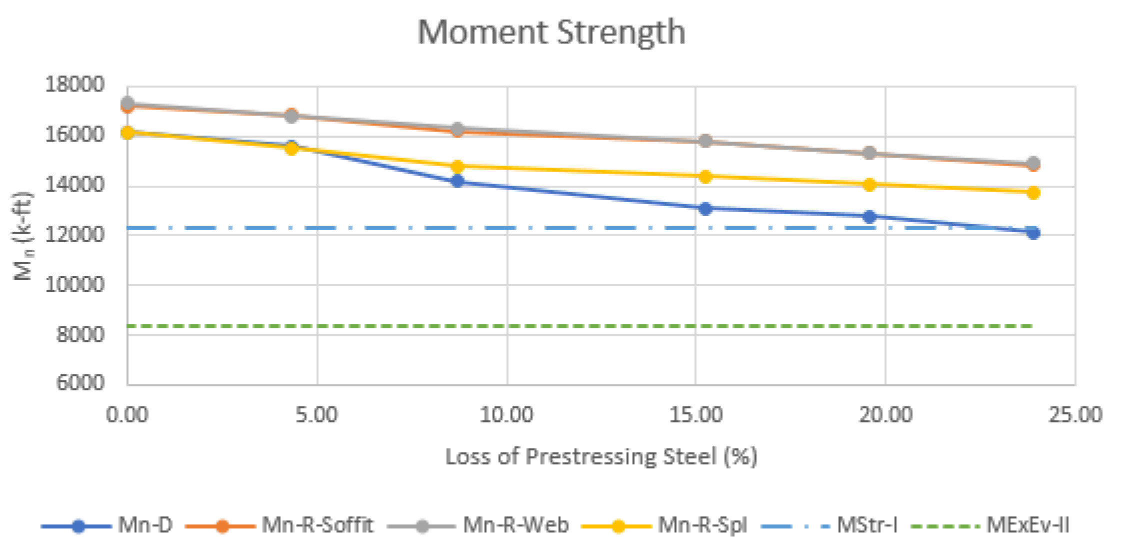


Figure 7.7. Structure 1 – variation of moment strength with loss in prestressing steel

It should be noted that the application of CFRP to the soffit or on the webs yield similar strengths in this case. The application to the soffit involves overhead work and is more difficult than the application on the vertical surface of the web. However, more CFRP material is used (both sides of the web). This indicates that, in some cases, applications on the web surface may be considered as conditions require.

Figure 7.8 shows variation of the inventory rating factor based on the moment strength. The trends are similar to those observed in Figure 7.7 with repaired girders retaining rating factors that are substantially above 1.0.

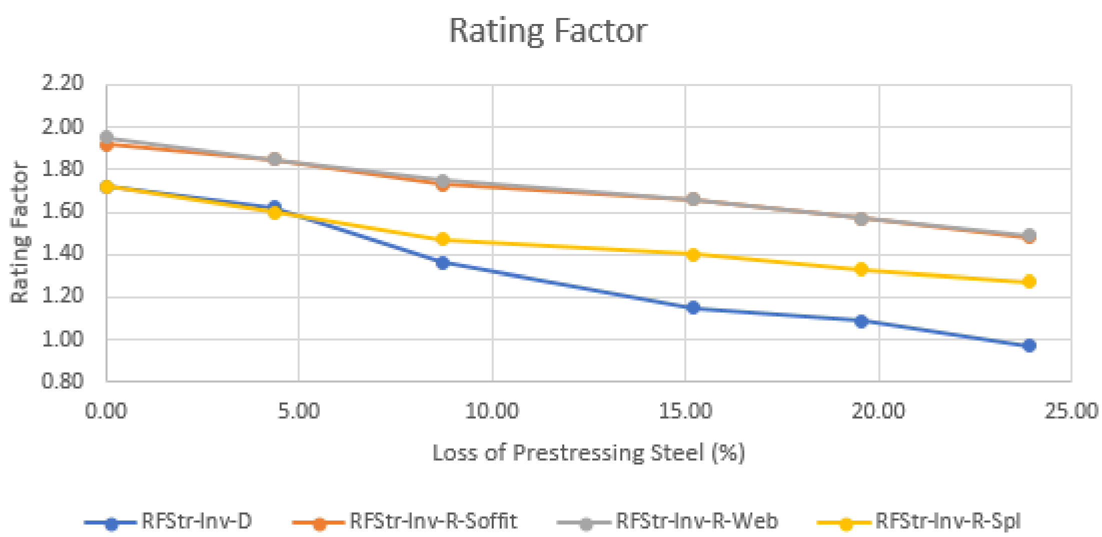


Figure 7.8. Structure 1 – variation of inventory rating factor with loss in prestressing steel

Figure 7.9 illustrates the effect of repair on reduction of girder ductility (here measured with the steel strain). While the undamaged, damaged (εs-D), and repaired girder with internal splices (εs-R-Spl) range between 2.5% TO 3.5% strain), the addition of CFRP, whether on the soffit (εs-R-Soffit) or the web (εs-R-Web), substantially reduces the steel strain at failure. It should be noted, however, that a repaired strain of under 1.5% is still considered to be an accepted level of ductility.

7.3.2 Structure 2 Bottom Damage

Similar graphs are shown in Figures 7.10 through 7.12 for bottom damage to Structure 2, where the span and girder depth is much smaller than Structure 1. In this shorter span, the addition of CFRP to the soffit improves strength, but other observations remain similar. In both structures, practical and effective repairs can be employed to restore a substantial part of the original girder strength following damage.

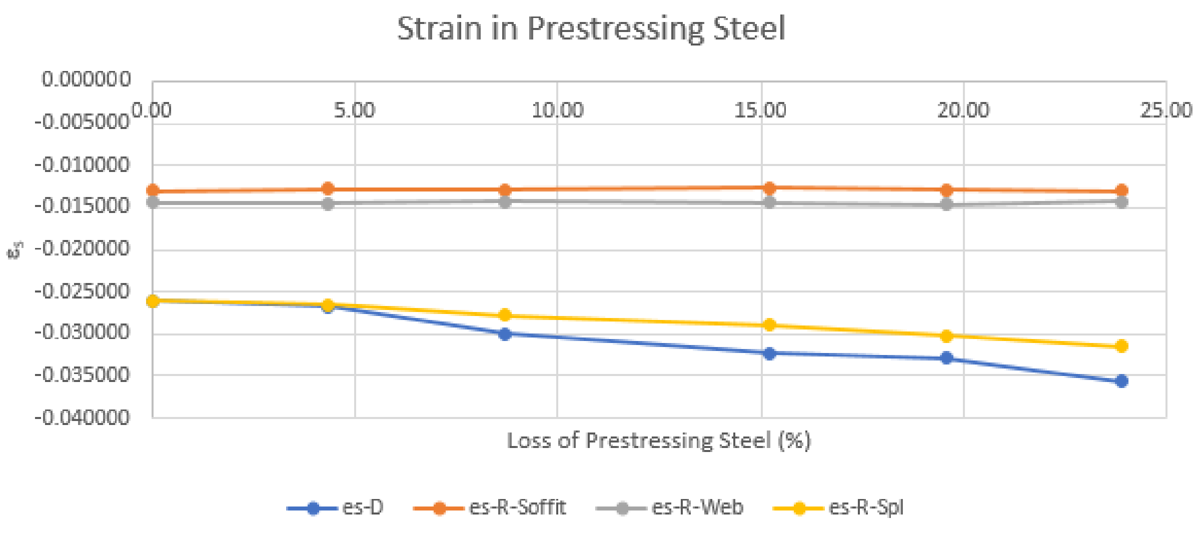


Figure 7.9. Structure 1 – variation of prestressing strain (at flexural failure) with loss in prestressing steel

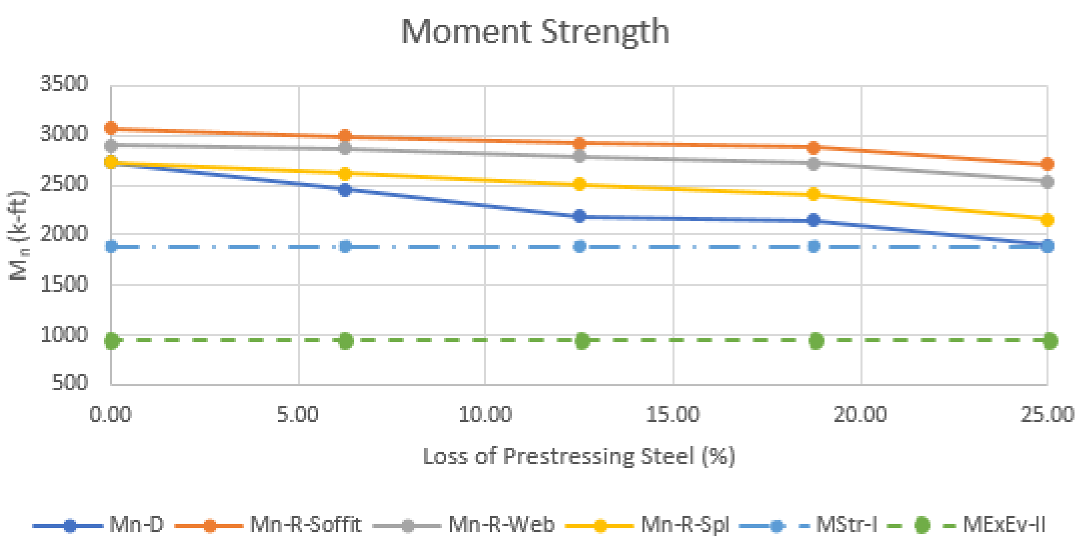


Figure 7.10. Structure 2 – variation of moment strength with loss in prestressing steel

Figure 7.11. Structure 2 – variation of inventory rating factor with loss in prestressing steel

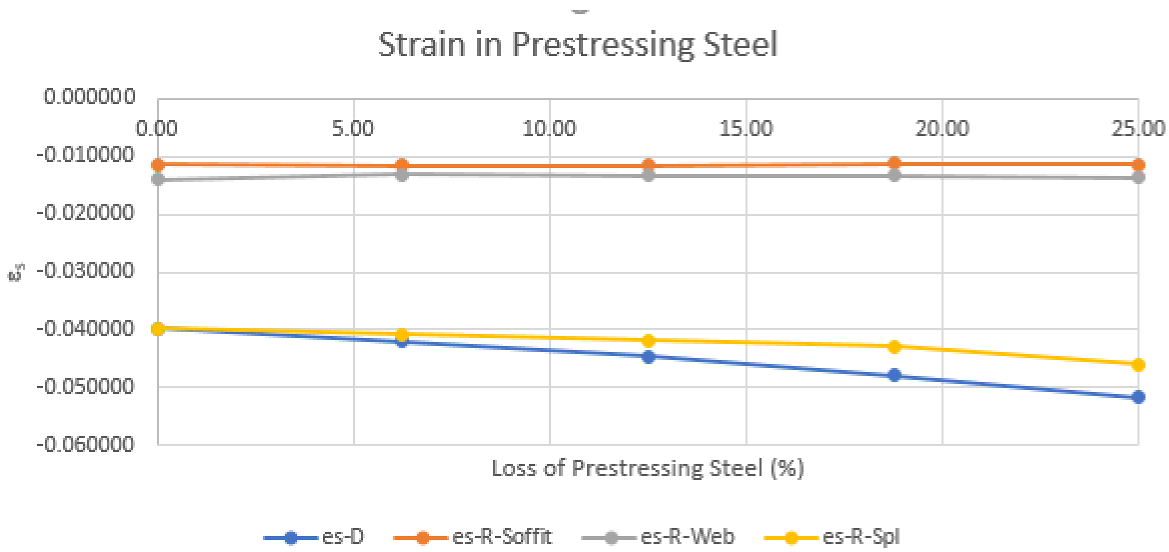


Figure 7.12. Structure 2 – variation of prestressing strain (at flexural failure) with loss in prestressing steel

7.3.3 Structure 1 Top Damage

In top damage cases, the deck slab is considered removed from the top of the girder in the PreBARS program. Then damage is introduced on the non-composite girder that would remain. The patching and the deck is then "placed" on the girder and strength calculations are made for the composite section. It should be noted that the strength of the girder that has sustained damage to the top flange, is essentially unchanged from the undamaged condition following the patching of the flange and deck placement. It is important, nonetheless, to check service stresses in the damaged non-composite state, and to check strength of the non-composite damaged girder with respect to the weight of the fresh concrete slab that would be placed on it. These are done within the PreBARS program.

It is important to check the sweep of the top flange of the girder when the girder is in a non-composite state and the top flange is damaged, especially when the damage is substantial across the section and is widespread along the length of the girder. Furthermore, as the top flange of the non-composite girder is damaged, the tendency would be for girder to lift up. As it does, depending on the restraint (to rotation) that may exist at the supports, the girder may develop cracking near the supports. Therefore, the girder sweep (lateral displacement) and cracking should be monitored. Temporary horizontal bracing, in addition to existing diaphragms, should be provided when required. It should be noted that the bracing system should be a competent system in light of the fact that there would not be a deck to transmit the brace forces to the piers (through the end diaphragm).

Figures 7.13 and 7.14 show the moment strength and rating factors for the 72W" girder as damage to the top flange is increased from 0 to 50%. It should be noted that the rating factor in this case is calculated assuming the "live load" is the weight of fresh concrete slab with a load factor of 1.75. Figure 7.15 shows that the maximum compressive stress (at service load) at the bottom of the non-composite beam increases as the extent of damage to the top flange increases.

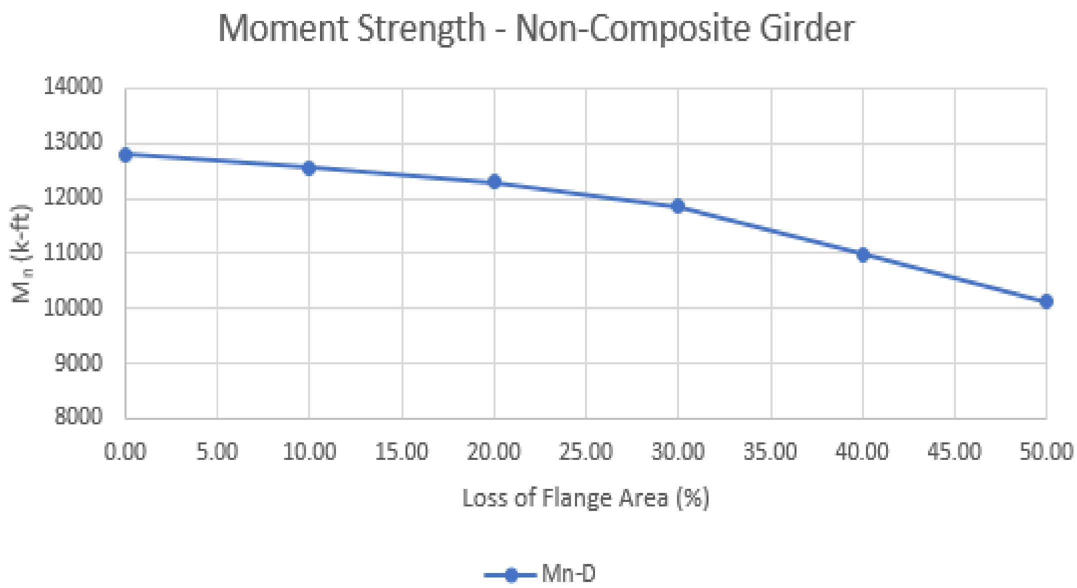


Figure 7.13. Structure 1 – variation of moment strength of non-composite girder as a function of extent of damage to top flange

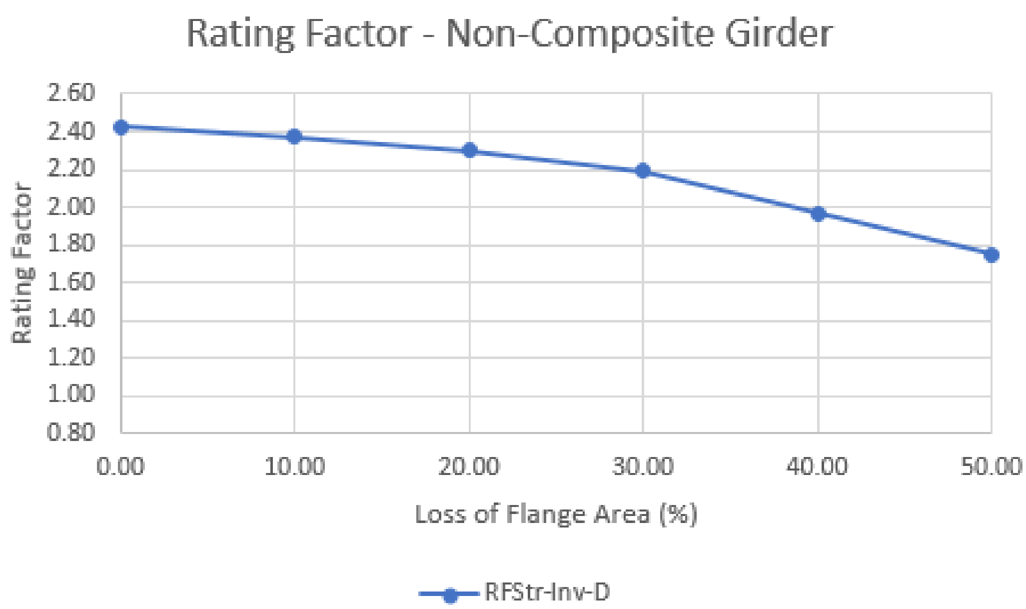


Figure 7.14. Structure 1 – variation of inventory rating factor for the non-composite girder as a function of extent of damage to top flange

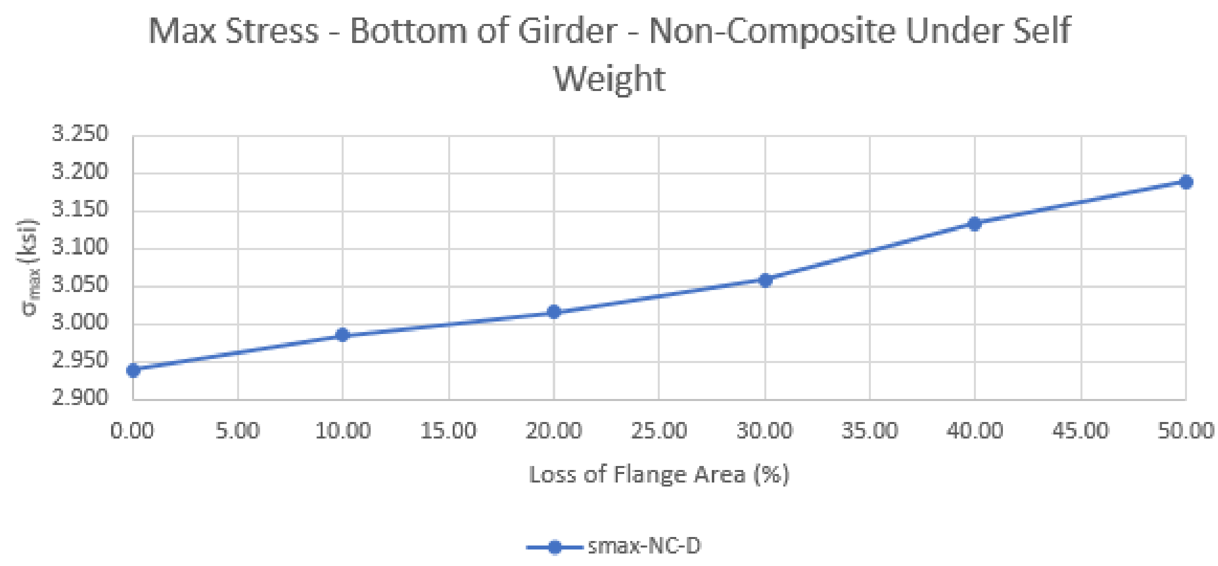


Figure 7.15. Structure 1 – variation of maximum compressive stress at the bottom flange for the non-composite girder as a function of extent of damage to top flange

7.3.4 Structure 2 Top Damage

Figures 7.16 through 7.18 shows the corresponding figures for Structure 2. A pattern similar to that observed for Structure 1 is also observed for Structure 2.

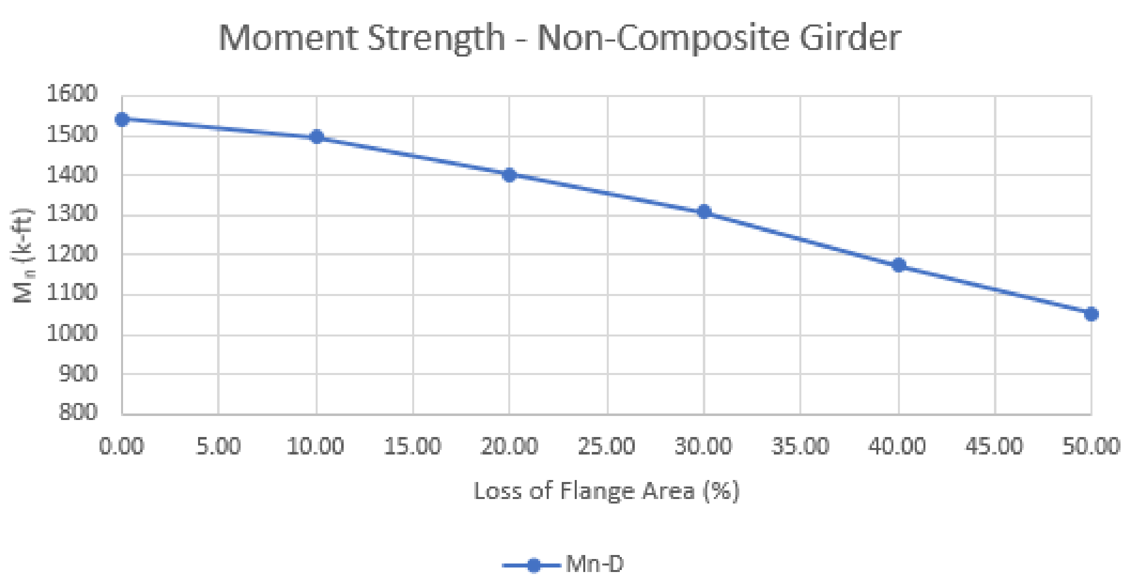


Figure 7.16. Structure 2 – variation of moment strength of non-composite girder as a function of extent of damage to top flange

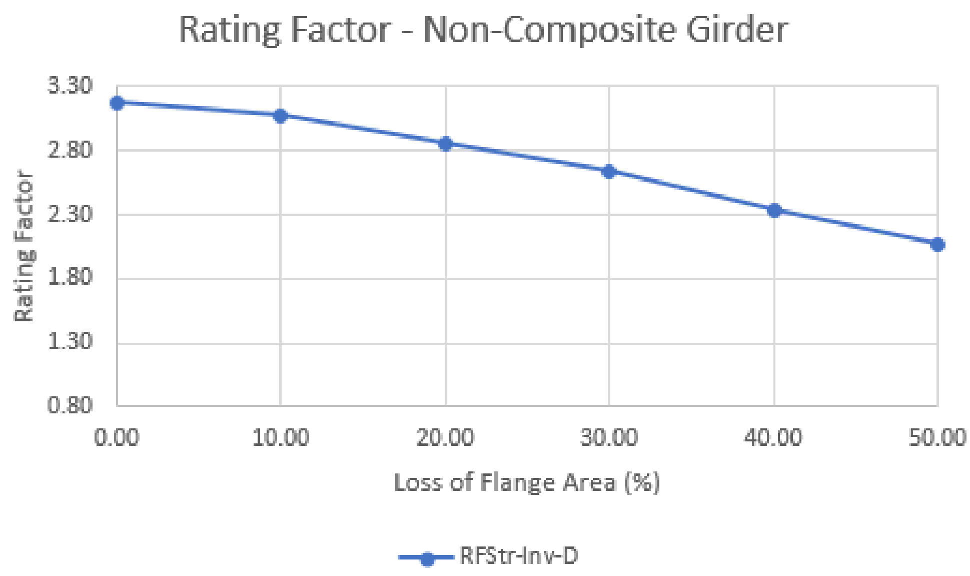


Figure 7.17. Structure 2 – variation of inventory rating factor for the non-composite girder as a function of extent of damage to top flange

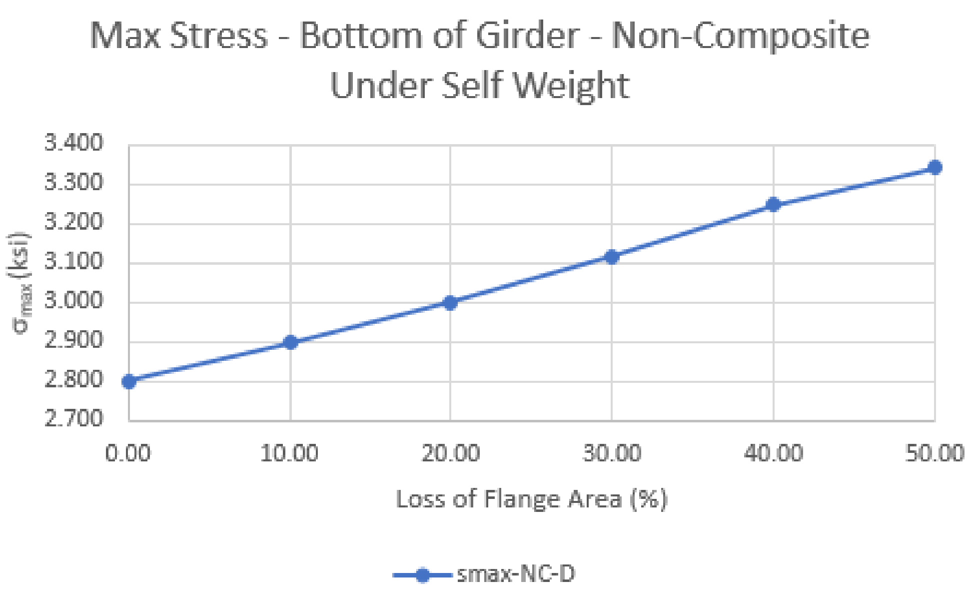


Figure 7.18. Structure 2 – variation of maximum compressive stress at the bottom flange for the non-composite girder as a function of extent of damage to top flange

Chapter 8. Summary and Conclusions

This study addressed the assessment and repair of damaged prestressed bridge girders due to either accidental impact by over-height vehicles on the bottom flange of the girder, or damage to the top flange of the girder during deck removal operations. To properly assess the structural condition of damaged bridge girders, it is important that the damage conditions be carefully assessed in the field and calculations be performed for both serviceability and strength limit states. Based on the inspection results and the structural calculations, the damage conditions could be categorized, and appropriate repair/replacement decisions could be made.

There is substantial prior work on damage to bottom flanges of prestressed bridge girders due to impact. However, most such prior works focus on sectional strength issues alone, and do not address the serviceability stress checks that are part of the design requirements in the AASHTO LRFD bridge design specifications. To address the need to calculate changes in service stresses due to damage, undamaged and damaged transformed section properties must first be calculated. Then, procedures must be developed to calculate changes in stress under the service conditions. The damaged section would consist of the undamaged section minus all spalled concrete, severed strands, and cracks that effectively reduce the sectional areas. This would likely result in an unsymmetrical cross section, thus further complicating the calculation process.

In this study (chapter 4), a set of procedures are developed to calculate changes in sectional stress due to loss of section that results in a generalized unsymmetrical cross section. These procedures apply equally to cases of bottom or top damage in prestressed girders. Verification of the procedures are provided through a 3-dimensional finite element model.

Considering the irregular pattern of damage in typical field damage cases, it is necessary to develop tools to accurately calculate the irregular section properties for stress calculations. An approach involving the use of spreadsheets was adopted in this study. Each cell in the spreadsheet represented a 0.5-in x 0.5-in square in the cross section. For all standard prestressed girder sections used in Wisconsin, the undamaged cross sections were generated within the spreadsheet. Slab is similarly added to make a composite section. The appropriate strand patterns would then be incorporated into the section. The numerical content of a cell would determine whether it is girder concrete, deck concrete, strand, CFRP, strand splice or patch repair material. The section properties utilize the cell contents and positions for calculations. For damaged properties, the user would “zero-out” all the cells that have been damaged/spalled/severed by deleting the corresponding cell contents. The damaged section properties would then be automatically calculated within the spreadsheet.

The strength calculations, although rigorous, are well established and involve using the strain compatibility method allowed in the AASHTO LRFD specifications. This would also allow, within the same calculation process, consideration of loss of strand in the tensile zone, loss of concrete in the compression flange, external CFRP reinforcement, and strand splices.

A software program, Prestressed Bridge Assessment, Repair, and Strengthening (PreBARS), was developed in this study. The program runs within the Excel spreadsheet and involves visual Basic and spreadsheet calculations. Based on the user input, the program calculates AASHTO HL-93 moments, distribution factors, prestress losses (refined and approximate), serviceability and strength limit state loads, etc. for any point on a bridge with up to three continuous spans. The program calculates sectional service stresses and strengths for undamaged, damaged and repaired conditions. Repairs may involve patches, strand splices, and external CFRP reinforcement. The CFRP reinforcement may be installed at the soffit, the webs, or on the sloped and vertical faces of the bottom flange. The preloading option (involving a loaded dump truck) is also incorporated into the program. The features of PreBARS are described in detail in Chapter 6, and a user guide is provided in the Appendix.

Using the PreBARS program, several case studies were conducted on two example prestressed I-girder bridges, one with a long span (146 ft) and the other with a short span (50 ft). Different levels of loss of strands in the bottom flange (up to 25% loss) were simulated on both structures, and various repairs were applied in each case. Similarly, different levels of top flange damage were introduced in both structures up to 50% loss of the top flange. CFRP repairs applied on the soffit and the webs were examined in conjunction with the partial splicing of the severed strands for bottom damage cases. Results of these case studies are presented in Chapter 7.

Various damage categories have been defined for both top and bottom flange damage scenarios. For bottom damage, the damage categories include minor, moderate, significant, serious, and severe. For top damage, the categories are minor, moderate, and significant. Table 8.1

provides a description for each damage category. Several detailed recommendations have been made regarding inspections, assessment, and repair of damaged prestressed girders in Chapter 9 for bottom and top damage scenarios.

Table 8.1. Damage categories for bottom and top flange damage cases

Bottom Flange Damage	
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 at 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.
Top Flange Damage	
Damage Category	Description
Minor	Concrete nicks, gouges, scrapes, and cracks that are less than 0.006 in wide. Spalled area less than 25% of top flange area; no cracking near supports due to girder uplift; girder sweep <1/8-in over 10 ft length; no damage to interface shear reinforcement.
Moderate	Spalled area more than 25% and less than 50% of top flange area; no cracking near supports due to girder uplift; girder sweep <1/8-in over 10 ft length; little to no damage to interface shear reinforcement.
Significant	Spalled area more than 50% of top flange area; or cracking near supports due to girder uplift; or girder sweep >1/8-in over 10 ft length; or extensive damage to interface shear reinforcement.

Chapter 9. Recommendations

In this chapter, recommendations are made regarding inspection and reporting, structural assessment and repair requirements for damage to prestressed concrete bridge girders. Impact damage to the bottom of the beams as well as damage to the top flange of the girder during deck removal is considered. In addition, discussions on the choice of patch repair materials and deck removal procedures are provided.

9.1 Bottom Damage

The types of damage to bottom of the girder is divided into five categories: Minor, Moderate, Significant, Serious, and Severe. In the following, definition of each category of damage as well as associated inspection/reporting, assessment, and repair suggestions are provided.

9.1.1 Minor Damage:

Definition: This category of damage involves nicks, gouges, scrapes, cracks (less than 0.006 in wide), and/or limited spalling without any exposed or partially exposed prestressing strands. Any spalling would thus be limited to near surface areas.

Inspection and reporting: Close visual inspection is needed to evaluate the condition of concrete (including spalled and loose concrete), cracking (including crack widths), and localized girder sweep (out-of-plane deformation). Location of damage and size/extent of spalls and loose concrete should be quantified and documented. A sketch of the cross section with the largest loss of section should be provided. Out-of-plane deformation should be measured and reported as relative lateral deformation over a 10-ft length of girder. A crack map showing the orientation and width of all cracks (including those in the web, deck slab, diaphragms, and barrier walls) should be included. Bearings should be inspected for any signs of damage. Detailed pictures (with accompanying explanations) should be provided from all affected areas.

Assessment: The information obtained during inspection should be carefully examined by the Engineer.

Repair: When needed, hand tools alone should be used in removing any loose concrete. If deemed necessary, nicks and small gouges can be repaired using an approved low-modulus epoxy material that could accommodate feathered edges. Oven-dried aggregates may be added to the epoxy (in accordance with the manufacturer's recommendations) to achieve a stiffer mix for deeper areas. When patching is required (other than nicks and gouges), the limits of patch area should be delineated using a shallow saw cut. Care should be taken to avoid damage to strands or stirrups. An appropriate approved patch material (such as a polymer-modified patch material with an epoxy bonding agent applied using manufacturer's recommendations) could be used to repair the spalled area under this damage category. It should be noted that polymeric patch materials with modulus of elasticity that is lower than the substrate concrete would not restore the full sectional properties. Higher modulus patch materials (as discussed in the Significant damage category) may also be used. Careful consideration should be given to the bond properties of patch material to avoid debonding in the future.

9.1.2 Moderate Damage:

Definition: This category of damage involves cracking and spalling that would partially or fully expose at least one prestressing strand. However, no severed strands should be present. It should be noted that strands with damaged, deformed, or kinked wire(s) should be considered severed and thus a Significant damage category should be selected. Under the Moderate damage category, the moment strength would generally be unchanged despite the damage, but service level stresses would undergo changes.

Inspection and reporting: In addition to the items listed under Minor damage, the condition of exposed strands including dents, cracks, distortions, and corrosion should be examined and noted.

Assessment: Follow the steps provided for the Minor damage. After review of all field data and structural analyses, a determination should be made whether the initially assigned damage category is appropriate or should be modified.

Patch repairs would be needed under this and other more severe damage categories. Therefore, an appropriate patch material should be selected. An assessment should be made regarding the application of preloading. In general, preloading would be recommended to restore prestress into the patch material, and to maximize the structural contribution of the cross section under service loading. If pre-loading is required, the patching material should have a modulus of elasticity that is compatible with the substrate concrete

Repair: Follow the recommendations made under the Minor damage category, where applicable. If the out-of-plane sweep of the girder (that is a result of the impact) exceeds the specified tolerance, a decision should be made as to whether repairs could address the problem. Otherwise a replacement of the girder should be considered.

According to Wipf et al. (2004), the most important parameters in good patch repair materials are modulus of elasticity and bond strength, followed by thermal compatibility and compressive strength. The compatibility of modulus of elasticity is particularly important when preloading is performed. A low-modulus (compared to substrate) patch material would develop lower prestress levels in the patch, thus reducing the effectiveness of preload.

Remove all loose and damaged concrete from the damaged area. Hand tools alone should be used in removing any loose concrete to avoid damage to steel, or further damage to concrete. Established and appropriate concrete repair practices should be followed. Shallow saw cuts should delineate the patch boundary (it is vitally important to avoid damage to strand during saw cutting). The concrete all around exposed strands should be removed (3/4" gap) to allow full encasement of the exposed strand with the patch material. Follow patch manufacturer's recommendations and sound industry practices for substrate surface preparations.

All cracks that exceed a width of 0.006 in should be injected with epoxy. Other cracks may be treated with a sealer. If there are any cracks that appear to be open below the surface, they should be carefully assessed regarding possible yielding of the underlying reinforcement. Injection of epoxy should be done after the application of preload (where applicable) and prior to patching.

When the patch area is not small (above the minor category), consideration should be given to the use of CFRP U-wraps over the entire patch area to contain and confine the patch material over the long-term. Fiber fabric or sheet impregnated with resin can be used in the wet layup process to provide the U-wraps. The U-wraps may be extended to the web. The fiber orientation would be perpendicular to the direction of the girder, and thus the U-wrap would not directly contribute to flexural strength of the girder.

9.1.3 Significant Damage:

Definition: This damage category applies when, in addition to cracking and spalling of concrete within the impact zone, not more than 15% of all strands are severed. This type and level of damage would result in a reduction in structural strength of the girder and increases in stresses under service load.

Inspection and reporting: Bridge information including plan/elevation and cross section details, continuity conditions, girder types, girder spacing, slab thickness, build-up over girder (at damage location), skew angle, strand type, size and patterns, tie-down location and information (if any is located within 5 feet of the edge of damage location), location of damaged girder (exterior or interior). These would be needed for structural calculations. In addition to the applicable items listed under the Minor and Moderate damage scenarios, identify the location (within the cross section) and exposed lengths of all severed strands. The location of severed strands in important in calculating stress distribution and in assessing the number and location of internal splices that can be effectively used. It should be stated whether the cracks appear to be open or closed below the surface.

Assessment: Follow the steps listed under Minor and Moderate damage scenarios, when applicable. Service and strength limit state conditions should be checked considering the information provided. Therefore, structural analyses would be needed. The AASHTO-specified design forces should be calculated for the damage location. The software tool (PreBARS) can assist in the analysis. A determination should be made whether the initially assigned damage category is appropriate or should be modified. If there is spalling (i.e. patching is needed), an assessment should be made whether preloading would be needed. If service stresses are within the allowable limits outside of the patch zone (they would likely exceed tension limit due to live load within the patch zone), and the extent of patching is limited, then a preload may not be required. Without pre-loading, however, the patch would not restore the full pre-damage sectional properties.

Structural repair options should be evaluated to restore the member strength and serviceability to the required/desired level. If the damage is located near the point of maximum moment (i.e. within a distance equivalent to the development length of strand), the structural condition and repairs may need to be assessed at two locations: the damage location and the point of maximum moment. The severed strand affects the attainable sectional strength away from the damage location (at the point of maximum moment) through a proportional reduction in stress at flexural failure (ratio of the distance from the point of maximum moment to the end of strand to the development length of strand). Damage at or near the tie-down location is another area of consideration. Damage to the tie-down location should be carefully examined. Loss of concrete above the harped strands in the tie-down location can lead to detensioning of harped strands and, after careful examination, may be a basis for replacement of the girder in lieu of repairs.

Repair: Follow the procedures and discussions outlined under the minor and moderate damage scenarios. Internal strand splices have long been used in the repair of impact-damaged prestressed concrete beams. They can restore prestress in the strand (not the patch) and make it mostly effective for its share of contribution to moment strength. There is however a recommended reduction factor of 0.85 for internal strand splices. The maximum diameter of strand for which internal splices are commercially available at the present (2018) is ½ in. If the strand is 0.6 in, then the manufacturer recommends using a reducer to convert it to a ½ in strand, and then use the ½ in splice. Considering the ratio between areas of the ½ in and 0.6 in strands, and the recommended reduction factor of 0.85, a factor of 0.6 can be used when 0.6 in strands are spliced. These factors are coded into the PreBARS program.

Another consideration is the congested nature of the strand layout in the beam cross sections. It may not be feasible to splice all severed strands that are located adjacent to each other. Torque devices used to introduce tension in the device require use of wrenches that may not fit in a congested situation. In general, about one-half of severed strands may be practically spliced in adjacent layouts. This should be considered in the design of repairs. In general, splices should be used to restore strands to the extent possible.

In assessing strength, it is important to recognize that the design of prestressed concrete bridge I-girders is typically controlled by the service stress checks, which would result in typically higher member strengths that are required by the AASHTO strength I combination (for I-girders). This would allow design of repairs that do not necessarily aim at restoring the original girder strength. However, damaged girder should be repaired, the service stress conditions should be addressed, and the strength should be restored to meet all code requirements and to the extent that is economically justifiable.

Another repair option that could be used in conjunction with the internal splices is the use of longitudinally oriented CFRP to enhance strength and stiffness of the damaged girder. There are two applicable design standards that govern the design of CFRP for repair of beams (AASHTO, 2012 and ACI 440.2R, 2017).

Each damage/repair case is a unique problem, and broad generalizations cannot be made across different bridge configurations and damage scenarios. Therefore, the specific condition of the girder as well as repair options must be considered through an analytical process. The PreBARS program is designed to address such cases where individual repair techniques, or a combination of methods, may be considered and assessed to arrive at a possible repair option.

If preloading is required, the modulus of elasticity of the patch should be comparable to the substrate concrete’s modulus of elasticity to allow development of anticipated compression in the patch material. The application of any preload can be in the form of strategic placement of a loaded truck or other forms of load application directly above the damage location. Analyses are required to ensure that the preload is not excessive for the damaged state of the girder, and to determine service stresses before and after patching with preload. The PreBARS program can assist in that process. The program assumes that a 4-axle dump truck (with a total weight of 80 kips) could be used. The middle axle in the back group of three axles would be placed directly over the damaged area with the center of one wheel directly over the affected girder. PreBARS would provide guidance as to the direction that the truck should face to maximize the imposed moment. The preload should remain in place until the patch material has gained sufficient strength and stiffness.

Crack injection, and patching issues must be considered as discussed in the Moderate damage scenario. Durability of repairs is an important consideration. CFRP materials should be coated with a protective coating for outdoor use as suggested by the manufacturer. In general, this damage category may be addressable through patching with internal strand splices or a combination of splices with CFRP application. Consideration should be given to using CFRP U-Wraps to contain and confine the patch area, and to provide anchorage for longitudinal CFRP reinforcement (if used).

Although early studies on prestressed girder repairs included a steel jacket repair (Shanafelt and Horn, 1980 and 1985), the modern development of CFRP repairs have limited the recent use of this approach. Problems include the need for field welds, drilling into concrete, and injection of epoxy or grout into the space between the jacket and concrete. The thickness of the steel plate used over the soffit could also encroach on an already limited vertical clearance under the bridge. Despite the noted deficiencies with the steel jacket approach, there may still be opportunities for the use of adhesively attached steel plates (with some mechanical fasteners) to the vertical and sloped faces of the bottom flange, or the web to increase flexural strength without the use of CFRP.

The use of post-tensioning systems to restore the strength of prestressed girders have been discussed in detail by Shanafelt and Horn (1980 and 1985). Although these methods can be effectively used in cases of substantial damage to the girders, they also have disadvantages including cost, anchorage requirements and aesthetics.

9.1.4 Serious Damage:

Definition: This damage category applies when, in addition to cracking and spalling of concrete within the impact zone, more than 15% and less than 25% of all strands are severed. This type and level of damage would result in a significant reduction in structural strength of the girder and substantial increases in stress under service load.

Inspection and reporting: In addition to the applicable items listed under the Minor, Moderate, and Serious damage scenarios, the vertical (loss of camber) and horizontal (sweeping) deformations of the girder should also be recorded. Any cracking on the deck should also be noted.

Assessment: Detailed structural analyses are required to assess damaged and repair conditions, and to compare various repair scenarios. Strength and serviceability limit states must be checked. Service stresses in the concrete and strain at the centroid of prestressing steel should be checked. The need for temporary traffic restrictions above and/or below the bridge may be assessed. A combination of internal strand splices and external CFRP reinforcement may provide the additional strength or service stress reductions that may be required. Preloading would be recommended.

External CFRP reinforcement offers enhanced strength and stiffness to the girder. However, there is an important limitation that must be considered when evaluating strength of CFRP reinforced concrete members. The governing limit state could be due to debonding failure of the CFRP at strain levels that are comparatively lower than the strains achieved by steel. The AASHTO Guide Specifications (2012) limits the strain in the CFRP layer to 0.005, while the ACI 440.2R (2017) standard provides an equation for the maximum strain in the CFRP that is a function of thickness, stiffness of each CFRP layer, and the number of layers of CFRP used. Increasing the number of layers would reduce the achievable maximum strain. Two types of CFRP reinforcement is typically used: wet layup involving use of flexible fiber fabrics or sheets, and preformed CFRP strips that are rigid and are attached to the surface using adhesives. The commercial preformed strips typically have higher fiber content and strength per layer, but multi-layer (over 2-layer) applications are typically not recommended by the manufacturer for preformed strips.

The debonding strain limit is partly because of appearance of cracks in the concrete at the CFRP interface. This can lead to sudden bond failure in beams reinforced with CFRP when the strain limit is reached. ACI 440.2R recommends that the CFRP reinforcement not be used when the beam without CFRP does not have a specified minimum level of strength.

The external CFRP reinforcement is typically applied to the beam's soffit (below the centroid of prestressing steel) to increase its effectiveness. However, the CFRP debonding strain limit would necessarily limit the strain in the prestressing strands along with a reduction in the full potential of the prestressing steel. The net effect is nonetheless beneficial for both strength and stiffness of the girder. Attachment to the soffit is an overhead installation that is more difficult than vertical installation.

In this research, both soffit and web installations of CFRP (both faces of web) were examined. The amount of CFRP used is increased when it is installed on the webs instead of soffit, but similar or better strengths are achieved for longer span bridges, while achievable ductility is increased. When the debonding strain limit is reached at the soffit CFRP, the entire CFRP layer could theoretically debond from the surface at once. Because of strain variations, however, the entire length of web CFRP would not be subject to debonding strain limit at once, thus providing for a possibly more gradual debonding failure. On the other hand, the increased stiffness and reduced steel strain associated with the soffit installation of the CFRP could reduce the risk of fatigue in the internal strand splices. The soffit CFRP installations also provide more strength in shorter span bridges. The choice of various CFRP/splice/patch options and their effects can be studied through the PreBARS program for the specific problem at hand.

Repair: Follow the procedures and discussions outlined under the Minor, Moderate damage scenarios. Select the type and location of repairs based on a structural analysis. External CFRP reinforcement should be applied after the patch installation is complete and sufficiently aged (based on CFRP manufacturer requirements). Use CFRP U-wraps (using CFRP fabric in wet layup process) to contain and confine the patch, and to provide anchorage for the longitudinal CFRP reinforcement and enhance its debonding strain. Apply CFRP protective coating per manufacturer's recommendations.

9.1.5 Severe Damage:

Definition: This damage category applies when, in addition to cracking and spalling of concrete within the impact zone, more than 25% of all strands are severed. This type and level of damage would result in a major reduction in structural strength of the girder. A number of researchers and the Washington State DOT use the 25% limit as a repair/replace transition point. Although it may be theoretically possible to go beyond this limit, at least in some cases, any such step must be taken with great caution and be viewed as experimental. Therefore, loss of over 25% of strands would require replacement of the girder.

Inspection and reporting: In addition to the applicable items listed under the Minor, Moderate, Serious, and Severe damage scenarios, safety and stability issues must be considered in light of the observed conditions in the field. Consideration should be given to restricting traffic as needed.

Assessment: Design for the removal and replacement of damaged girder should consider safety and durability considerations. The joint between the old barrier and the new deck at or near the curb has been reported to be an issue for long-term durability. The design of the new girder should consider the potential for future impacts unless steps are taken to address any clearance shortfalls that may exist.

Repair: Although repair may be possible in some cases under the severe category, any such repairs should be considered experimental and involve detailed studies, and follow up evaluations. Otherwise, replacement of the girder(s) should be considered.

9.2 Top Damage

The types of damage to top flange of the girder is divided into three categories: Minor, Moderate, and Significant. In the following, definition of each category of damage as well as associated inspection, reporting, assessment, and repair suggestions are provided.

Damage to the top flange typically occurs during deck removal process. A major study by Li et al. (2017) addressed this issue and suggested details for new construction to facilitate deck removal in the future. In addition, recommendations were made regarding removal of deck in existing bridges. Kansas, Nebraska and South Dakota also provide guidance in their construction specifications. These provisions, which are reproduced in Chapter 2 of this report, can be a basis for new design requirements and construction procedures to prevent damage to top flange of girders.

The NCHRP 407 report (Tadros and Baishya, 1998) studied rapid deck replacement methods. They concluded that localized loss of the top flange during removal would not have any practical effect on the composite slab-girder flexural strength if “full composite connection is preserved”. They also state that “experimental studies have demonstrated that concrete girders with up to 50 percent of the top flange width damaged at the maximum positive moment location caused no noticeable structural deficiency…”

Since the bottom flange of the girder (and thus the strands within them) is not directly affected by damage to the top flange, the placement of a new deck on the girder would provide the necessary top flange for the composite girder (including the required balancing compression force) such that the final moment strength of the composite girder would not be diminished, despite damage to the top flange concrete. However, this is contingent upon preservation of the interface shear reinforcement during removal.

Although the loss of part of the top flange may not result in a reduction in the composite strength, it could substantially reduce the strength of the girder as a non-composite beam. The non-composite girder must resist the weight of fresh concrete during slab construction. Therefore, the remaining girder strength must be assessed and compared against the factored loads from the deck slab and other construction loads. The deck slab can be assumed conservatively to act as a live load applied on the non-composite girder (i.e. with a load factor of 1.75). In the analyses performed in this study, the strength of the non-composite beam in the damaged state was found to be sufficient. However, this cannot be assumed to be the case under all circumstances. Therefore, an analysis should be performed. The PreBARS program provides a rating factor for the damaged non-composite beams assuming the slab dead weight to be the live load.

There are other potential issues that may arise because of damage to the top flange. These include service level stresses in the non-composite beam that could exceed the allowable stresses. Considering that these overages would likely be temporary, and the placement of slab would counteract and reduce these stresses, moderate (say 10%) increases beyond the allowable stresses may be considered as determined by the Engineer after reviewing all data. The girder could exhibit vertical displacement and cause cracking near the support if the rotation of the girder at the support is restrained. The upward vertical displacement of non-composite girder, beyond that is associated with the removal of slab dead load, can be an indicator of the extent of widespread distribution of damage to the top flange. Restraint of girder rotation could occur due to continuity reinforcement at the bottom of the girder extending into the diaphragm, or due to other unintended restraint of rotation at the support. If the stresses are considered excessive or damage is observed near the supports, then temporary preload may be applied to reduce the service stresses and upward displacement in the non-composite beam.

There is also potential for out-of-plane sweep of the girder because of the reduction in transverse stiffness of the top flange and shift in the positions of the centroid of the cross section and the centroid of the prestressing steel, especially when the damage to the top flange is widespread. This should be mitigated by monitoring the lateral sweep as the deck removal proceeds, by ensuring that the existing diaphragms are in place, and by providing temporary lateral support if deemed necessary.

9.2.1 Minor Damage:

Definition: This category of damage involves nicks, gouges, scrapes, cracks (less than 0.006 in wide), localized loss of not more than 25% of the area of top flange, no damage to the interface shear reinforcement, no cracking observed near the supports due to girder vertical deformation, and girder sweep (at the top flange) that is not more than 1/8 in per 10 ft.

Inspection and reporting: Close visual inspection is needed to evaluate the condition of concrete (including spalled and loose concrete), cracking (including crack widths), and localized girder sweep (out-of-plane deformation). Location of damage and size/extent of spalls and loose concrete should be quantified and documented. Sketches of one or more cross sections with the largest loss of sections, especially those near the point of maximum moment, should be provided. The percent loss of flange in a section is determined by sketching the cross section at the point of maximum loss, including non-overlapping losses that may occur at other sections within a distance equal to the width of top flange (undamaged) on either side of the section being sketched. Out-of-plane deformation should be measured and reported as relative lateral deformation over a 10-ft length of girder. A crack map showing the orientation and width of all cracks anywhere along the girder should also be provided. It should be noted whether cracks appear closed/open below the surface. Detailed pictures should be provided.

Assessment: The information obtained during inspection should be carefully examined by the Engineer. Calculation of stresses and strength are not specified for this category, but should be performed if the inspection information warrant. Service and strength limit state conditions should be checked considering the information provided. The AASHTO-specified design forces should be calculated for the largest damage location(s), especially those near points of maximum moment. A determination should be made as to whether the initially assigned damage category is appropriate or should be modified.

Repair: When needed, hand tools alone should be used in removing any loose concrete. If deemed necessary, isolated nicks and small gouges can be repaired using an approved low-modulus epoxy material. Repair of the damaged flange (other than nicks and gouges) can occur simultaneous with the placement of the deck slab, unless calculations show that a prior repair of the top flange should be performed.

9.2.2 Moderate Damage:

Definition: This category of damage involves localized loss of between 25% and 50% of the cross section of the top flange, no damage to the interface shear reinforcement, no cracking observed near the supports due to girder vertical deformation, and girder sweep (at the top flange) that is less than 1/8 in per 10 ft.

Inspection and reporting: Follow the inspection and reporting requirements for Minor damage. Bridge information including plan/elevation and cross section details, continuity conditions, girder types, girder spacing, slab thickness, skew angle, strand type, size and patterns, and tie-down location. These would be needed for structural analysis calculations.

Assessment: Follow the steps provided for the Minor damage. Structural analyses should be performed to assess the damaged non-composite member strength, service stresses in the non-composite girder, and the secondary moment due to restraint of vertical movement of the girder at the support. The software tool PreBARS can assist in the analysis. After review of all field data and structural analyses, a determination should be made as to whether the initially assigned damage category is appropriate or should be modified. If the service stresses exceed the allowable, the engineer may consider patching of the flange prior to casting of the deck with or without an application of preload. Under the Moderate damage condition, it is anticipated that the strength of the non-composite girder may be sufficient to resist the weight of slab, and prior patching of the top flange may not be required. However, the adequacy of the strength of damaged girder should be verified and appropriate steps (repair or replacement) taken if a deficiency is found. If interface shear reinforcement is damaged, calculate the remaining strength and evaluate its impact on the strength of the composite section. If the strength of damaged girder is deemed insufficient to resist the weight of slab (as live load), repair or replacement of the non-composite girder may be considered. In case of excessive service stresses, early (prior to casting of the deck) repair of flange with or without preloading may be evaluated. Cracks should be evaluated to see if they are open or closed below the surface.

Repair: Follow the recommendations made under the Minor damage category, where applicable. If prior patching is required, follow patch procedures outlined for the bottom damage cases. Inject all cracks that are wider than 0.006 in with epoxy. If the interface shear reinforcements are damaged, they should be repaired or restored. "When it is determined that prior patching of the top flange is required, and there is no existing transverse steel in the top flange, adhesive anchorage bars may be provided as required (based on structural calculations) prior to patching. As a general guide (pending calculations based on actual field conditions), ½-in-diameter adhesive anchors at 6-in-spacing may be considered."

9.2.3 Significant Damage:

Definition: This category of damage involves localized loss of greater than 50% of the cross section of the top flange, extensive damage to the interface shear reinforcement, or out-of-tolerance sweep of the top flange of the girder.

Inspection and reporting: In addition to the applicable items listed under the Minor and Moderate, damage scenarios, safety and stability issues must be considered in light of the observed conditions in the field. A detailed assessment must be made to determine if repairs are warranted or the girder(s) should be replaced.

Assessment: Detailed structural evaluation of the girder(s) should be performed. Removal and replacement of damaged girders should consider safety and durability considerations.

Repair: Although repair may be possible in some cases under this damage category, any such repairs should be considered only after careful consideration of structural strength, serviceability and stability considerations. Otherwise, replacement of the girder(s)should be considered. In case of excessive sweep of girder, replacement may be the only practical option.

9.3 Patch Repair Materials

The choice of patch repair material is an important consideration in repair of prestressed girders. In a study for the Iowa Department of Transportation, Wipf et al. (2004) (volume 3 of 3) report on a comprehensive study on patch materials for repair of concrete bridge structures. They conducted tests on five different patch materials in addition to extensive review of prior works on this topic.

The authors of this Iowa study emphasize the importance of modulus of elasticity as the most important material property. The authors identify the following key parameters for selection of repair materials (in order of importance): modulus of elasticity, bond strength, coefficient of thermal expansion, and compressive strength. They suggest comparing different materials by creating a table with these test parameters listed. Give numerical ratings to each parameter based on the reported test results. Considering the importance of modulus of elasticity and bond strength, a weighing factor of two is suggested for these two parameters, with the other parameters receiving a weighing factor of one. The weighted ratings are then added for each material, and a ranking is determined. Based on the results of their tests, they reported the following two generically-defined materials as best performing among the five materials tested:

“[A] rheoplastic shrinkage-compensated cement-based material with silica fume, fibers, and a corrosion inhibitor”;

“[A] dry hydraulic cement material”.

The authors suggest using materials with modulus of elasticity that is similar to concrete and discourage selecting repair materials based solely on compressive strength. They make several recommendations for field application of repair materials.

In addition to the reasons noted by Wipf et al. (2004) for the importance of modulus of elasticity, there is an additional factor that points to its importance in the repair of damaged prestressed bridge girders. When preloading is used to introduce compression into the patch material, a patch with a lower modulus of elasticity than the girder concrete could significantly diminish the amount of prestress that could be introduced. Therefore, the study being reported here concurs with the conclusions of the Wipf et al. (2004) study in this regard.

The use of CFRP U-wraps should be considered to contain and confine the patch, especially when larger patch areas are used. When a decision is made to patch smaller gouges, the cementitious patches may come loose over time. In such cases, the use of low-modulus epoxy may achieve better long-term bond but would contribute little to structural stiffness.

9.4 Deck Removal

Detailed discussions on the various studies on this topic are given in Chapter 2. Below is a summary of items related to deck removal practices suggested by others. It should be noted that

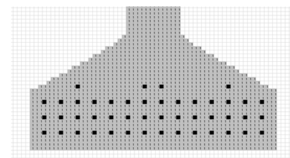
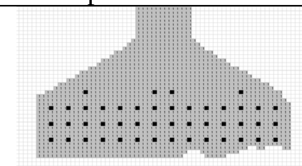
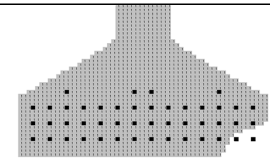
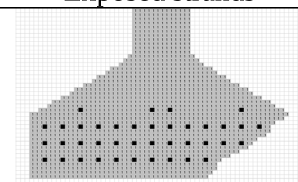
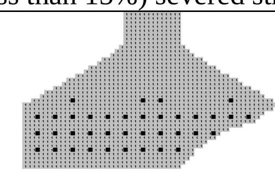
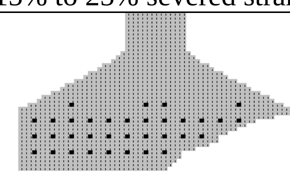
WisDOT does not favor methods that involve saw cutting of the flange during the removal processes and wishes to preserve the entire top flange during deck removal to the extent possible. WisDOT currently specifies a 2- to 15-in-wide (depending on girder size) smooth area along each edge of the flange to achieve easier debonding during deck removal.

A study of deck removal procedures was not within the scope of this study. However, the Procedure A recommended by Li et al. (2017) (discussed in detail in Chapter 2 and reproduced below) may be in line with the WisDOT current preferences and practices. In addition, consideration should be given to using roofing felt over the 2- to 15-in smooth areas to further facilitate future removals.

<u>Procedure A: For Girders with a Roughened and or Troweled Top Flange (Li et al., 2017)</u>

1. *"Perform a series of saw-cuts between girders transverse to the girder axes to create a series of panels to facilitate lifting and disposal. Through thickness cuts are appropriate when clear of girders. When near to the girder top flanges, limit the saw-cut depth to 0.5 in. less than the deck thickness.*
2. *Using a crane or other piece of lifting equipment in tandem with saws, separate the ends of each panel from the deck concrete over the girders and lift clear for disposal.*
3. *Use 30-lb demolition hammers to remove the concrete over the bridge girders down to the level of the bottom layer of deck reinforcement. The 30-lb demolition hammers should be used at an angle not exceeding 45 degrees from horizontal.*
4. *Once the bottom layer of deck reinforcement is exposed, 15-lb demolition hammers should be used to remove the remaining concrete and install a roughness in compliance with project specifications. Where the deck concrete immediately over the flange is sound, it may not be necessary to fully remove all deck concrete. In this study, it was observed that the deck and girder concrete form a strong bond where the concrete was roughened prior to casting of the deck that makes complete removal of deck concrete difficult. The judgement of the engineer should govern the extent of removal necessary considering the condition of the system and risk of damage to the underlying girders."*

Table 9.1. Categorical definitions for bottom flange damage

Damage category	Definition	Bottom Damage Examples
Minor Damage	Nicks, gouges, scrapes, cracks (less than 0.006 in wide), and/or limited spalling without any exposed or partially exposed prestressing strands	 Undamaged bottom flange  Damaged bottom flange/ no severed strand
Moderate Damage*	Cracking and spalling that would partially or fully expose at least one prestressing strand. Strands with damaged, deformed, or kinked wire(s) should be considered severed	 Exposed strands
Significant Damage	Cracking and spalling of concrete within the impact zone, not more than 15% of all strands severed	 (Less than 15%) severed strands
Serious Damage	Cracking and spalling of concrete within the impact zone, more than 15% and less than 25% of all strands severed	 15% to 25% severed strands
Severe Damage	Cracking and spalling of concrete within the impact zone, more than 25% of all strands severed	 More than 25% severed strands

* In cases where there are no severed strands but at least 25% of concrete in the flange and/or web is missing, higher condition categories should be considered.

Table 9.2. Categorical definition for top flange damage

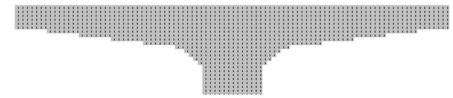
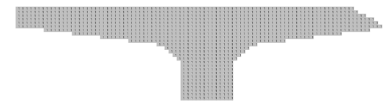
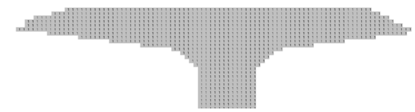
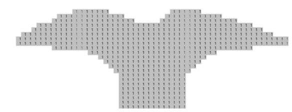
Damage category	Definition	Top Damage example
Minor Damage	Nicks, gouges, scrapes, cracks (less than 0.006 in wide), localized loss (less than 25% of the area of top flange), no damage to the interface shear reinforcement, no cracking near the supports, girder sweep (at the top flange) that is not more than 1/8 in per 10 ft.	 Undamaged top flange  Minor Damage to top flange
Moderate Damage	Localized loss of between 25% and 50% of the cross section of the top flange, no damage to the interface shear reinforcement, no cracking observed near the supports due to girder vertical deformation, and girder sweep (at the top flange) that is less than 1/8 in per 10 ft.	 Moderate Damage to top flange
Serious Damage	Localized loss of greater than 50% of the cross section of the top flange, extensive damage to the interface shear reinforcement, or out-of-tolerance sweep of the top flange of the girder.	 Serious Damage to top flange

Table 9.3. Suggested repairs for bottom flange damage*

Damage category	Inspection and Assessment	Repair methods
Minor Damage	• Conduct close visual inspection. • Record bridge information and location of damaged girder (exterior or interior). • Evaluate condition of concrete (including spalled and loose concrete). • Note cracking (including crack widths) and map orientation of all cracks including those in the web, deck slab, diaphragms, and barrier walls. • Record any girder sweep (out-of-plane deformation). • Record location of damage and size/extent of spalls and loose concrete. • Provide a sketch of the cross section with the largest loss of section. • Measure out-of-plane deformation. • Inspect bearings for any signs of damage. • Provide detailed pictures (with accompanying explanations) from all affected areas. • An engineer should carefully examine the information obtained during inspection. • Reevaluate the damage category based on analysis of data.	• Use only hand tools in removing any loose concrete • Delineate the limits of patch area using a shallow saw cut • Avoid damage to strands or stirrups • Use low-modulus epoxy material for nicks and gouges • When patching, use higher modulus patch materials • Investigate bond properties of patch material to avoid debonding in the future • Follow patch manufacturer's recommendations and sound industry practices for substrate surface preparations
Moderate Damage	In addition to items listed for minor damage; • Evaluate and record condition of exposed strands including dents, cracks, distortions, and corrosion. • Review all field data and analyze the structure. • Assessment should be made regarding the application of preloading. • In case of pre-loading application, use a patching material with modulus of elasticity compatible with the substrate concrete.	In addition to items listed for minor damage where applicable; • Remove concrete around exposed strands (3/4" gap) to allow full encasement of the exposed strand with the patch material. • Inject all cracks that exceed a width of 0.006 with epoxy. Treat other cracks with a sealer. • Epoxy application should be after the application of preload (where applicable) and prior to patching.

	• Investigate if repairs can address the problem in case of out-of-plane sweep of girder (that is a result of the impact) exceeding the specified tolerance	• Use CFRP U-wraps over the entire patch area if patch area is not small (above the minor category). • If repairs cannot address the out of plane sweep, replace the girder.
Significant Damage	In addition to items listed for minor and moderate damage; • Identify the location and exposed lengths of all severed strands. • Analyze the structure and check service and strength limit state conditions. • Calculate the AASHTO-specified design forces for the damage location. • Assess service stresses before and after patching with preload, if applicable. • The software tool (PreBARS) can assist in the analyses. • Assess structural condition and repairs at damage location and point of maximum moment, damage at or near the tie-down location. • Investigate loss of concrete above the harped strands in the tie-down location. • Evaluate structural repair options to restore member strength and serviceability to the required/desired level.	In addition to items listed for minor and moderate damage, consider the following where applicable; • Use of internal strand splices • Use of longitudinally oriented carbon fiber composites (CFRP) • Application of preloading (if required) before patching that remains in place until the patch material has gained sufficient strength and stiffness. • Use of post-tensioning systems • Coating CFRP materials with a protective coating for outdoor use as suggested by the manufacturer • Patching with internal strand splices or a combination of splices and CFRP application • Attaching steel plates to the bottom flange or the web
Serious Damage	In addition to applicable items listed for minor, moderate, and significant damage; • Record the vertical (loss of camber) and horizontal (sweeping) deformations of the girder. • Record any cracking of the deck. • Check service stresses in the concrete and strain at the centroid of prestressing steel. • Investigate the need for temporary traffic restrictions above and/or below the bridge.	In addition to items listed for minor and moderate damage, consider the following where applicable; • A combination of internal strand splices and external CFRP reinforcement • Preloading (recommended) • Application of external CFRP reinforcement after patch installation is complete and sufficiently aged • Use of CFRP U-wraps (with CFRP fabric in wet layup process) • Application of CFRP protective coating per manufacturer's recommendations
Severe Damage	In addition to applicable items listed for minor, moderate, and significant damage;	• Any repair in lieu of replacement should be considered experimental and carefully examined.

	• Consider safety and stability issues. • Design for the removal and replacement of damaged girder. • Consider safety and durability for the design of new girder. • Evaluate joint condition between the old barrier and the new deck at or near the curb. • Consider the potential for future impacts unless steps are taken to address any clearance shortfalls that may exist.	

*For a complete list, see the full description in Chapter 9.

Table 9.4. Repair recommendation for top flange damage*

Damage category	Inspection and Assessment	Repair methods
Minor Damage	• Conduct close visual inspection • Record bridge information and location of damaged girder (exterior or interior). • Evaluate condition of concrete (including spalled and loose concrete). • Note cracking (including crack widths) and map orientation of all cracks. • Record any girder sweep (out-of-plane deformation). • Record locations of damage and size/extent of spalls and loose concrete. • Provide a sketch of the cross section with the largest loss of section. • Provide detailed pictures (with accompanying explanations) from all affected areas. • Check service and strength limit state conditions if warranted. • The software tool PreBARS can assist in the analyses. • An engineer should carefully examine the information obtained during inspection. • Reevaluate the damage category based on analysis of data.	• Use only hand tools in removing any loose concrete. • Use low-modulus epoxy for repair of isolated nicks and small gouges. • Repair damaged flange (other than nicks and gouges) simultaneously with the placement of the deck slab, unless calculations show that a prior repair of the top flange should be performed.
Moderate Damage	In addition to items listed for minor damage; • Assess damaged non-composite member strength, service stresses in the non-composite girder, and the secondary moment due to restraint of vertical movement of the girder at the support.	In addition to items listed for minor damage where applicable;

	• If the service stresses exceed the allowable, the engineer may consider patching of the flange prior to casting of the deck with or without the application of a preload. • Evaluate the strength of interface shear reinforcement and calculate the remaining strength if damaged. • Evaluate girder strength to resist the weight of slab. • In case of excessive service stresses, early (prior to casting of the deck) repair of flange with or without preloading may be evaluated. • Cracks should be evaluated to see if they are open or closed below the surface.	• If prior patching is required, follow patch procedures outlined for the bottom damage cases. • Inject all cracks that are wider than 0.006 in with epoxy. • Repair or restore interface shear reinforcements if they are damaged. • Repair the girder before casting the deck if the damaged girder strength is insufficient.
Significant Damage	In addition to items listed for minor and moderate damage; • Consider damage scenarios, safety and stability issues in light of the observed conditions in the field. • A detailed assessment must be made to determine if repairs are warranted or the girder(s) should be replaced. • Consider safety and durability in case of removal and replacement of damaged girders.	• Any repairs should be considered only after careful consideration of structural strength, serviceability and stability considerations. Otherwise, replacement should be considered. In case of excessive sweep of girder, replacement may be the only practical option.

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Appendix

User Guide

In this chapter, a user guide for using the software is developed through examples for both bottom and top damage.

The software is consisting of two spreadsheets that are integrated and internally linked. Therefore, it is necessary to keep both files next to each other in a same folder and directory of a computer. The spreadsheet that users need to directly word with is called "UWM_DamagedBridgeGirder_Analysis_V1.0" and load calculations are done in a linked workbook called "UWM-Bridge-PS-Girder-LoadCalculations".

The user guide is constructed based on bridge design data taken from an example in WISDOT bridge manual, example "E19-1 Single Span Bridge, 72W″ Prestressed Girder-LRFD".

"UWM_DamagedBridgeGirder_Analysis_V1.0" workbook is consisting of several worksheets, each one designed for a specific purpose including:

- User Input
- Report
- Section Properties
- Strength and Stress
- Undamaged Section
- Damaged Section
- Repaired Section
- Strand Configuration

A.1 User Input

Input data including designed bridge, damage and repair information should be provided by user in “User Input” sheet. All required data are highlighted in yellow. These data are shown and discussed in detail in the following sections.

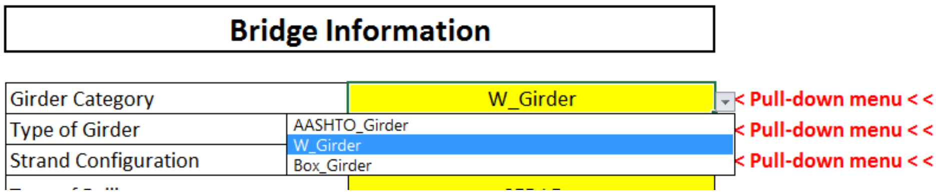
A.1.1 Bridge Information

Designed bridge information should be provided in “Bridge Information” table in the “User Input” sheet. Items included in this table are shown below.

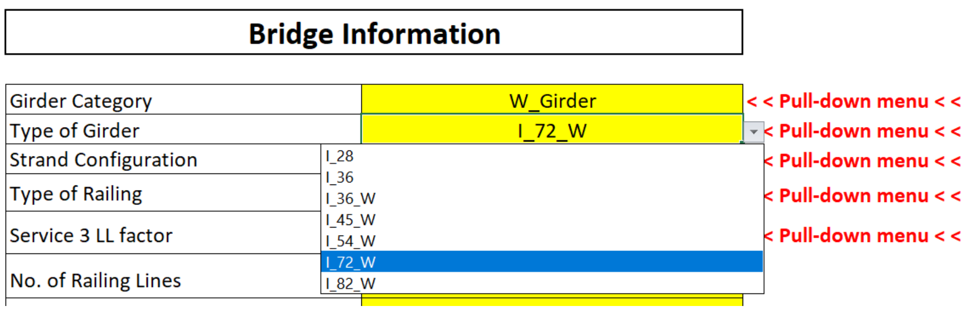
Bridge Information

Girder Category	W_Girder	< < Pull-down menu < <	
Type of Girder	I-72W"	< < Pull-down menu < <	
Strand Configuration	46 STRANDS-WITH DRAPED	< < Pull-down menu < <	
Type of Railing	SFP LF	< < Pull-down menu < <	
Service 3 LL factor	0.8	< < Pull-down menu < <	
No. of Railing Lines	2		
Exterior / Interior Girder?	Interior	< < Pull-down menu < <	
Sidewalk Cross Sectional Area (ft^2)	0	Enter 0 if no sidewalk	
Span 1	146	ft	
Span 2	0	ft	
Span 3	0	ft	
Girder Spacing	7.5	ft	
thickness of slab	7.5	in	Structural thickness only
Initial wearing surface	0.5	in	or existing wearing surface
Girder - f'c	8	ksi	
Slab f'c	4	ksi	
Girder - f'ci	6.8	ksi	
No. of strands	46		
Size of strands	0.6	in	
Type of strand	Low Relaxation	< < Pull-down menu < <	
Grade of strand	270	ksi	< < Pull-down menu < <
f_y of Strand	243	ksi	
d_e	1.25	ft	Assumed the same in all spans
Skew angle	20	degrees	
N_b	6	No. of beams in cross section	
Build-up	2	in	
Curb-to-curb width	40	ft	
Average annual humidity	70	%	
Concrete age at transfer	1	days	
Concrete age at deck placement	90	days	
Concrete age - current	20000	days	
Prestress Loss Method	Approximate Method	< < Pull-down menu < <	
Girder concrete unit weight	0.15	kcf	
Deck concrete unit weight	0.15	kcf	
Damage Location (x)	73	ft	

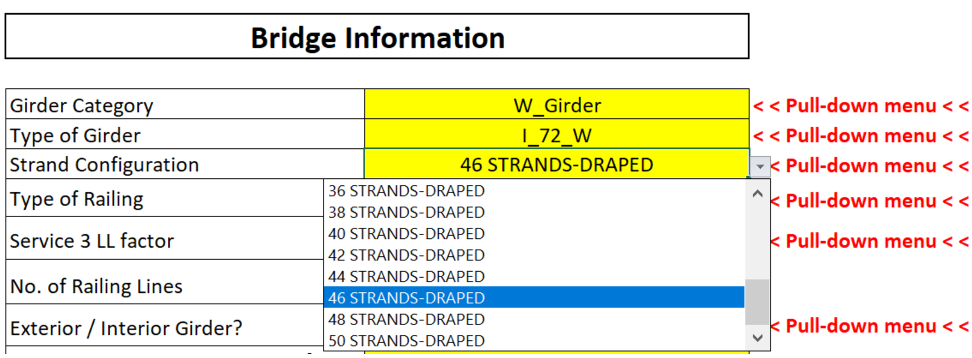
- Girder category: can be selected from a drop-down menu and indicates standard girders for the state of Wisconsin girders, AASHTO girders, and box girders. This information is needed for the section properties calculations.



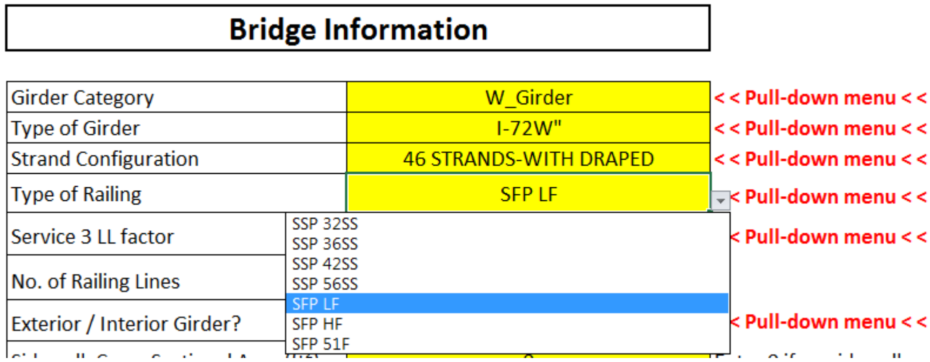
- Type of girder: is selected from a drop-down menu. This is an indication of shape of the girders and divides them into two categories: I-shaped girders and box girders and is required for load calculation spreadsheet.



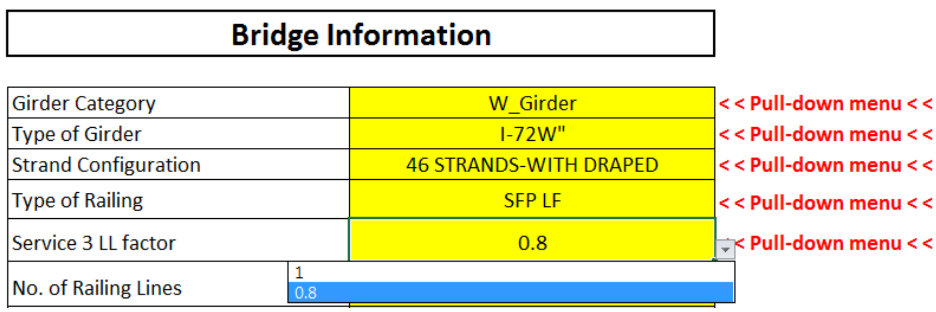
- Strand Configuration: should be selected based on the pattern considered in the designed girder.



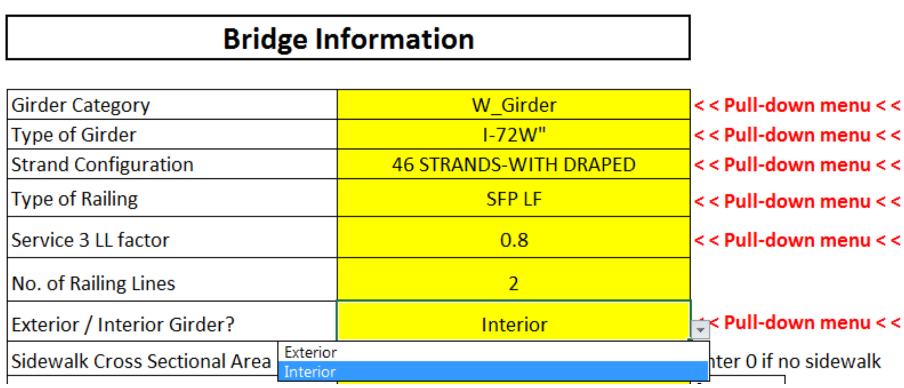
- Type of Railing: defines different type of railing that affects the dead load. That can be selected from a drop-down menu.



- Service 3 LL factor: is the factor for the effect of live load. This can be either 1 or 0.8 based on the design specification.

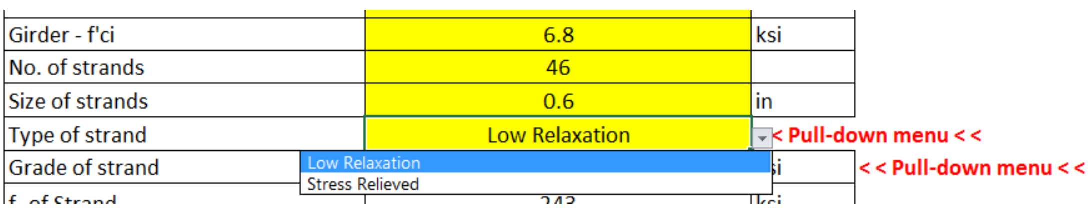


- No. of Railing Lines
- Exterior / Interior Girder?: This can be either interior girder or exterior girder.

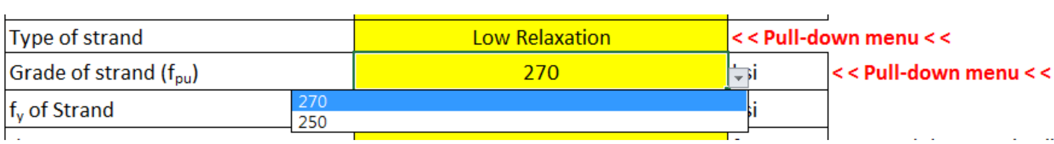


- Sidewalk Cross Sectional Area (ft^2)
- Span 1: length of span 1 of the bridge (ft).
- Span 2: length of span 2 of the bridge (ft).

- Span 3: length of span 3 of the bridge (ft).
- Girder spacing: is distance of center to center of girders in the bridge cross section (ft)
- Thickness of slab: is structural thickness of the slab (in).
- Initial wearing surface
- Girder - f_c':specified compressive strength of concrete for girder(ksi)
- Slab - f_c': specified compressive strength of concrete for slab (ksi)
- Girder - f_{ci}': specified compressive strength of concrete for girder at the time of release (ksi)
- No.of strands: number of strands in the girder section.
- Size of strands: nominal size of strands that can be 0.5 in, 0.6 in, or 5/16 in.
- Type of strands: can be low relaxation or stress relieved.

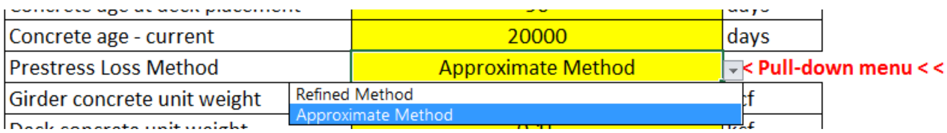


- Grade of strands (f_{pu}): can be 250 ksi or 270 ksi.



- f_y of Strand: is defined as yield stress of prestressing strands which is $0.9f_{pu}$ for strand grade 270 ksi and $0.85f_{pu}$ for strand grade 250 ksi.
- d_e: is length of cantilever of each side of the bridge deck.
- Skew angle: (degree)
- N_b: number of beams in the bridge cross section.

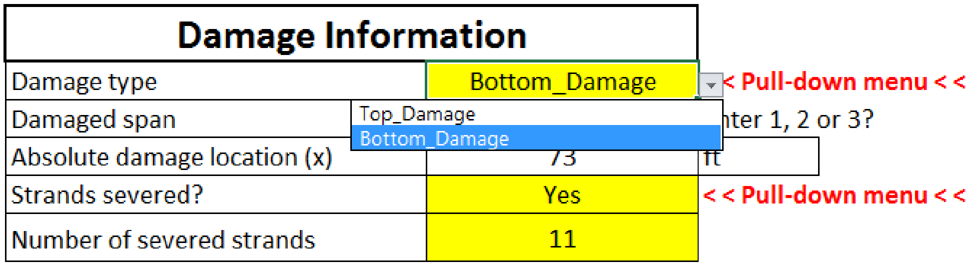
- Build-up:
- Crub-to-crub width
- Average annual humidity
- Concrete age at transfer: should be in days.
- Concrete age at deck placement: should be in days.
- Concrete age-current: should be in days.
- Prestress loss method: can be approximate method or refined method and needs to be determined for long term shortening and effective prestress force calculation.



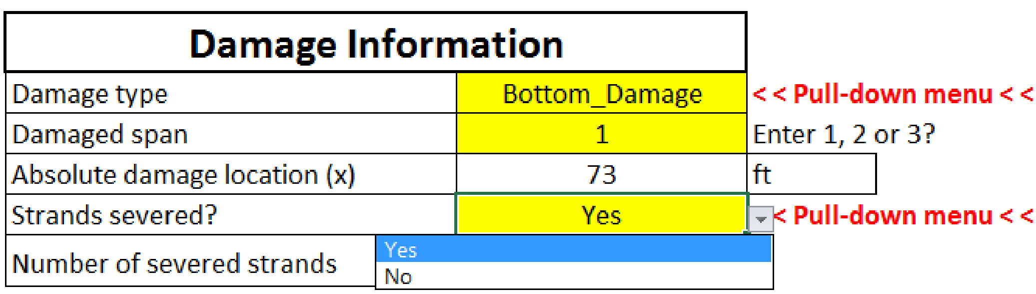
- Girder concrete unit weight: should be in kips per cubic feet.
- Deck concrete unit weight: should be in kips per cubic feet.
- Relative damage location (x): is desired location for load calculation and in case of damage analysis, where the damage happens. It should be calculated according to the discussion in chapter 6.

A.1.2. Damage Information

Damage type: is indication of top flange damage or bottom flange damage. This should be determined since (different parameters) section properties are used for stress calculations. Each damage case is discussed in more detail in the section pertaining to the damage simulation.



- Damaged span: is the span that the damage girder is located. It is always span 1 if the bridge is single span. In case of continuous spans, bridges with two spans or three spans, it can be identified according to section 6.4.
- Absolute damage location: is absolute distance of damage location from the start of the bridge with respect to the direction of stationing.
- Strand severed? (Yes or No): implies the severity of damage and indicates if strands are severed.

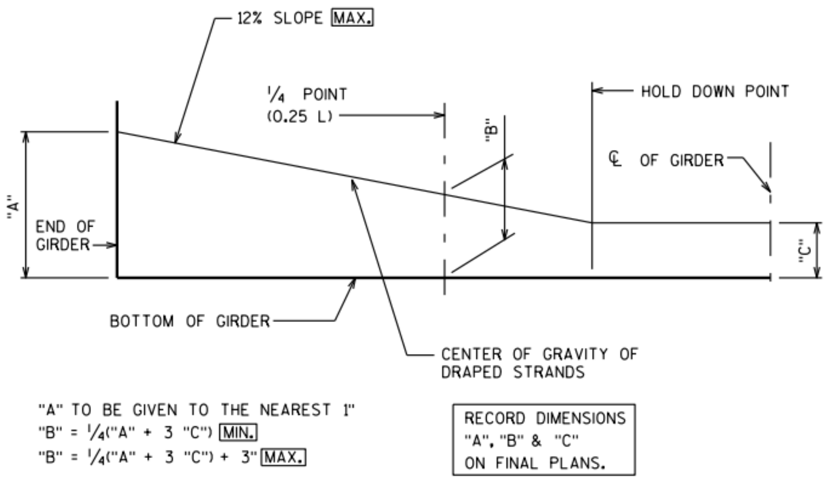


- Number of severed strands: is required for remaining effective prestressing force in case of damage with severed strands.

A.1.3. Drape Information for Damaged Span

- No. of Draped strands: number of draped strands in case of draping.
- A: is the location of the centroid of draped strands at the end of the span.
- B: is the location of the centroid of draped strands at $1/4L$ of the span.
- C: is the location of the centroid of draped strands at the hold-down point.

- Slope: is the slope of the draping according to the parameter A and B. This doesn't need to be entered by user and would be calculated automatically.



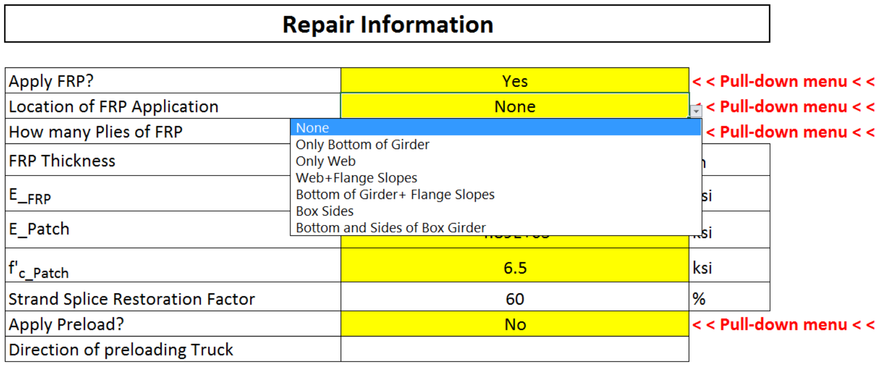
A.1.4. Repaired Information

- Apply FRP?: In case of repair through FRP or in case of retrofit this option should be "Yes" and if no FRP applied on the repaired section it should be "No".

Repair Information

Apply FRP?	No		< Pull-down menu < <
Location of FRP Application	Yes		< Pull-down menu < <
How many Plies of FRP	No 1		< < Pull-down menu < <
FRP Thickness	0.047	in	
E_FRP	2.32E+04	ksi	
E_Patch	4.89E+03	ksi	
f'_{c_Patch}	6.5	ksi	
Strand Splice Restoration Factor	60	%	
Apply Preload?	No		< < Pull-down menu < <
Direction of preloading Truck			

- Location of FRP Application: the location of FRP depends on the user decision and can be selected from a drop-down menu created in the software.



- How many Plies of FRP: number of FRP plied to be applied on the repaired girder at a specified location.
- FRP Thickness: thickness of FRP which is predefined as 0.047in. This can be changed by the user based on FRP properties that will be applied on the section.
- E_{FRP}: modulus of elasticity of FRP (ksi)
- E_Patch: modulus of elasticity of patch materials (ksi)
- f'_{c_Patch}: specified compressive strength of patch materials (ksi)
- Strand Splice Restoration Factor: this is defined as 60% for strands with nominal diameter of 0.6 in and 80% for all other strands.
- Apply Preload?: preload can be applied in case of bottom damage. “Yes” or “No” options are provided for preloading as shown below.

Repair Information

Apply FRP?	No	< < Pull-down menu < <
Location of FRP Application	None	< < Pull-down menu < <
How many Plies of FRP	1	< < Pull-down menu < <
FRP Thickness	0.047	in
$E_{\_FRP}$	2.32E+04	ksi
E_Patch	4.89E+03	ksi
f'_{c_Patch}	6.5	ksi
Strand Splice Restoration Factor	60	%
Apply Preload?	Yes	< Pull-down menu < <
Direction of preloading Truck	Yes No	

- Direction of preloading Truck: If preload is considered for repair of bottom damage, the direction that preloading truck should be positioned is specified as shown below.

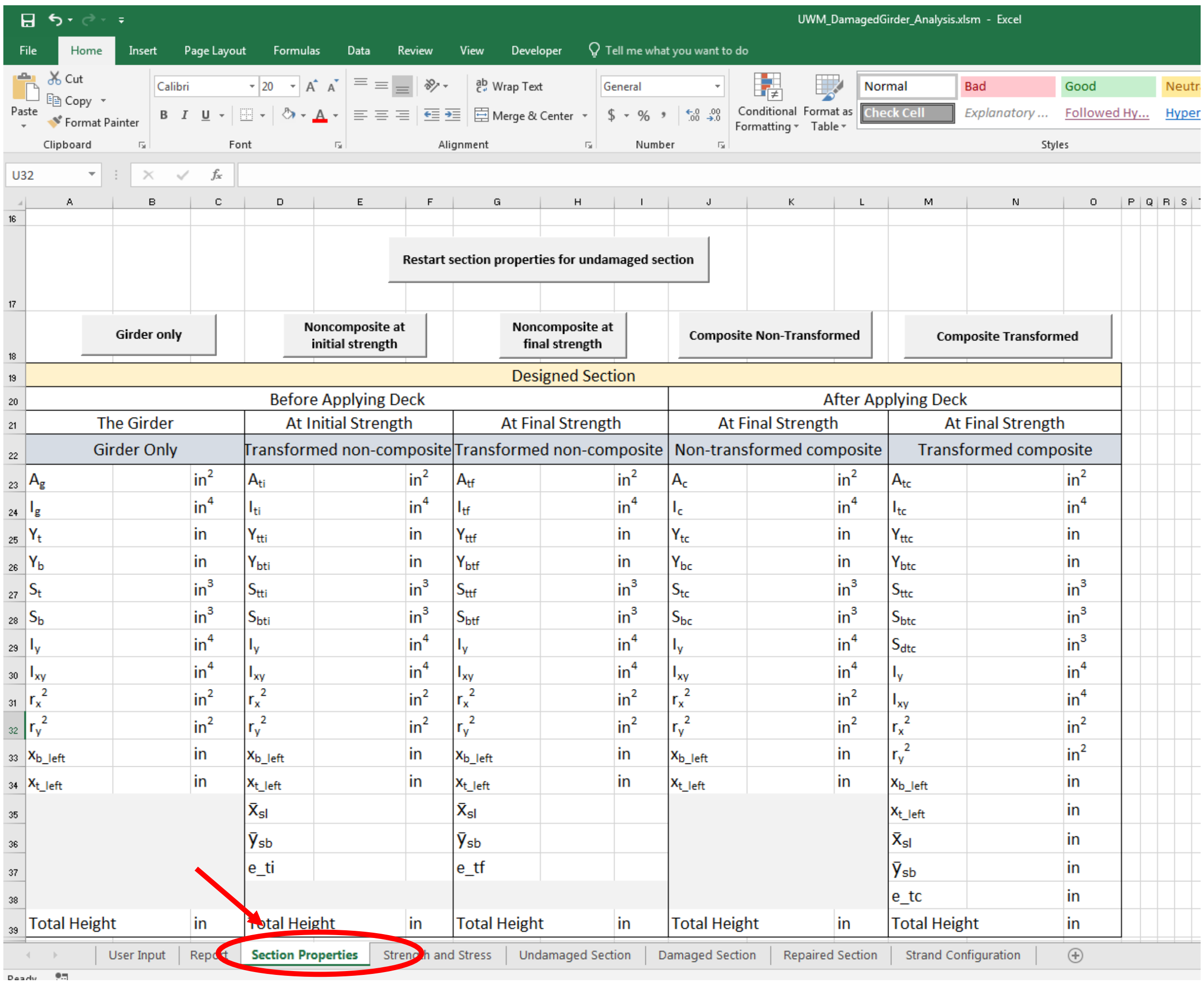
Repair Information

Apply FRP?	No	< < Pull-down menu < <
Location of FRP Application	None	< < Pull-down menu < <
How many Plies of FRP	1	< < Pull-down menu < <
FRP Thickness	0.047	in
$E_{\_FRP}$	2.32E+04	ksi
E_Patch	4.89E+03	ksi
f'_{c_Patch}	6.5	ksi
Strand Splice Restoration Factor	60	%
Apply Preload?	Yes	< Pull-down menu < <
Direction of preloading Truck	----->	

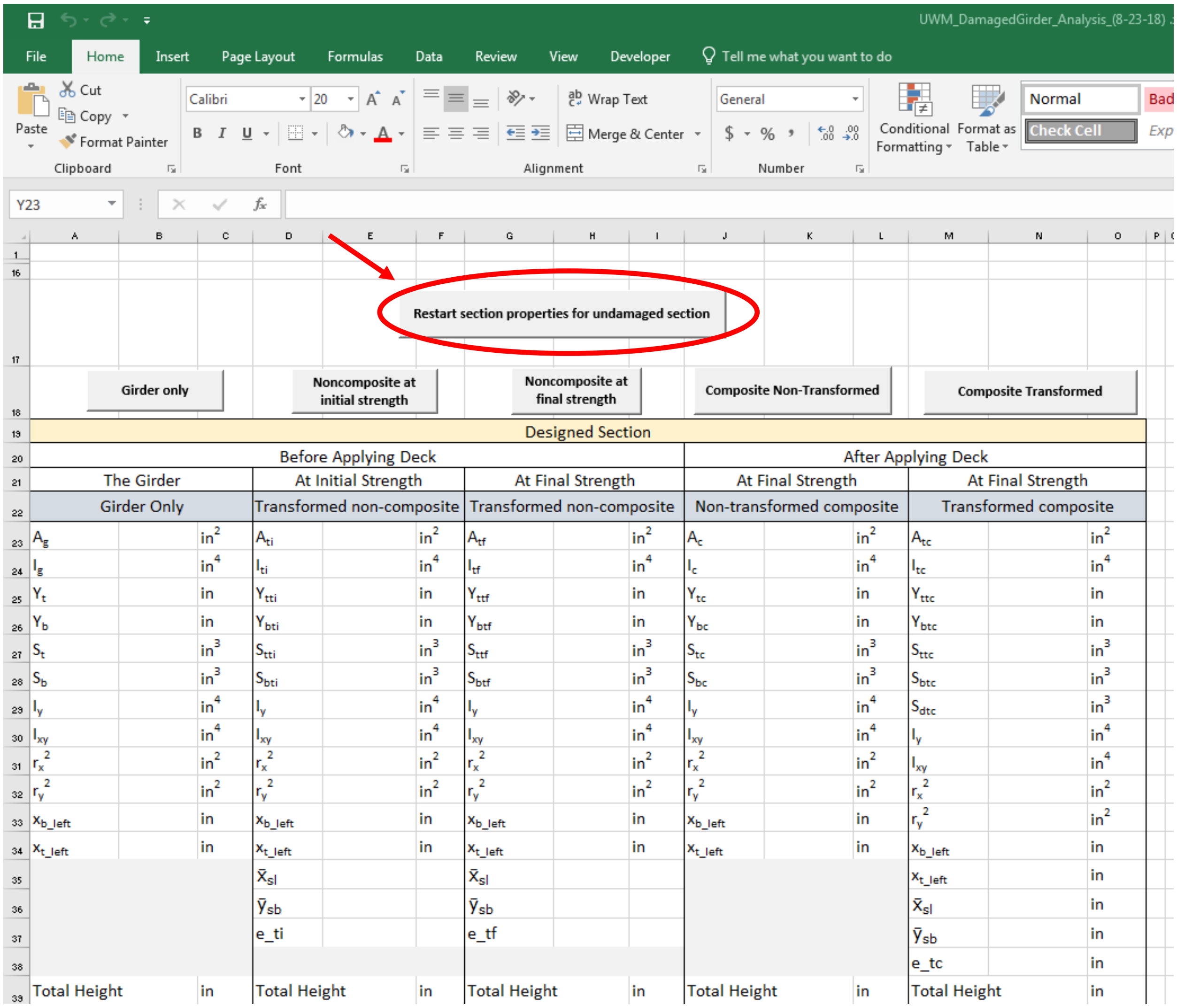
A.2 Section Properties

Prior to apply bottom damage or top damage, section properties for different states of girder, non-composite, and composite sections should be calculated and the results would be transferred to the linked workbook for load calculations.

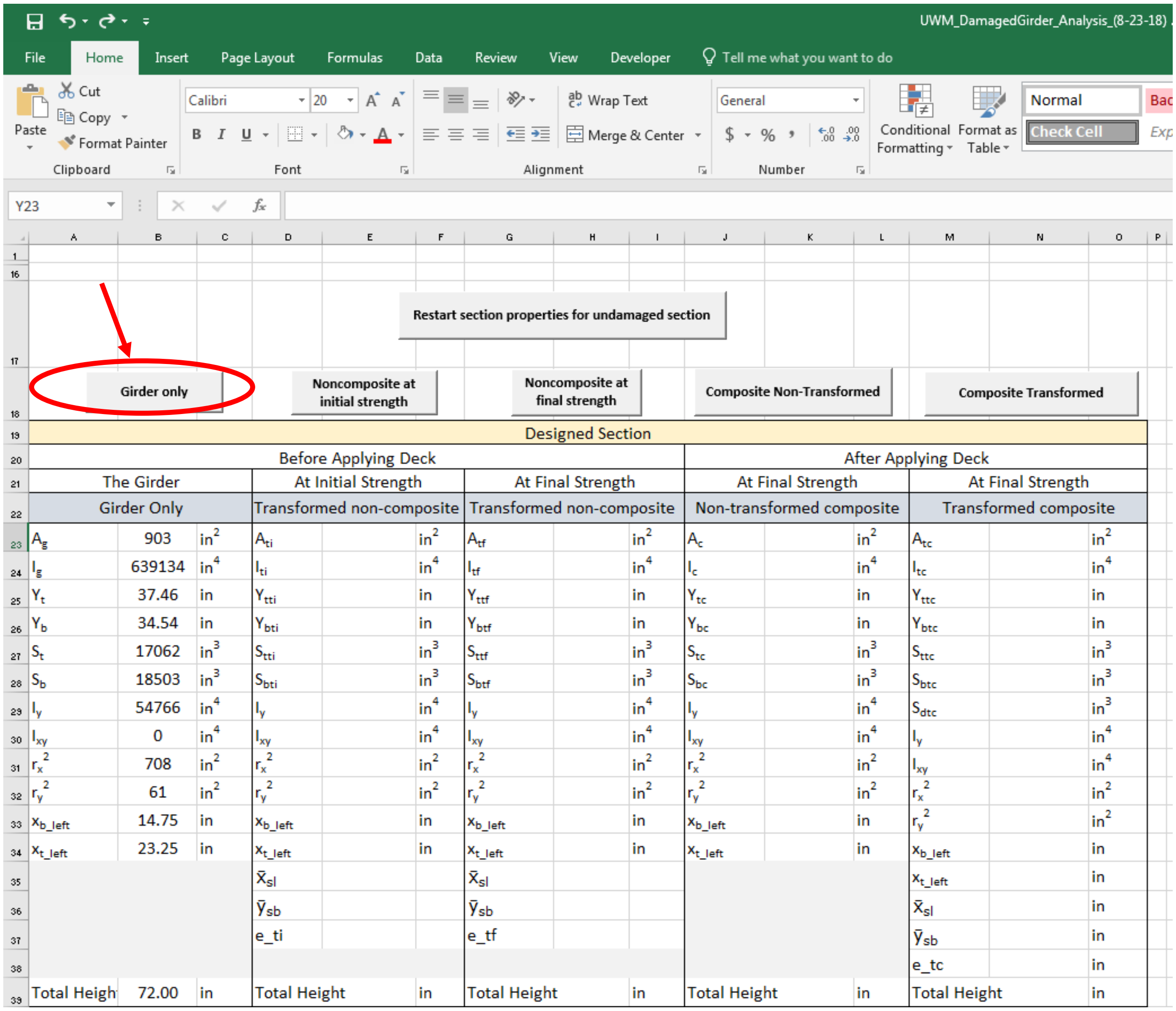
To calculate section properties, click on sheet “Section Properties”.



A button to restart all calculation is located on the top of the "Section Properties" sheet, in case of data existed from a previous analysis.

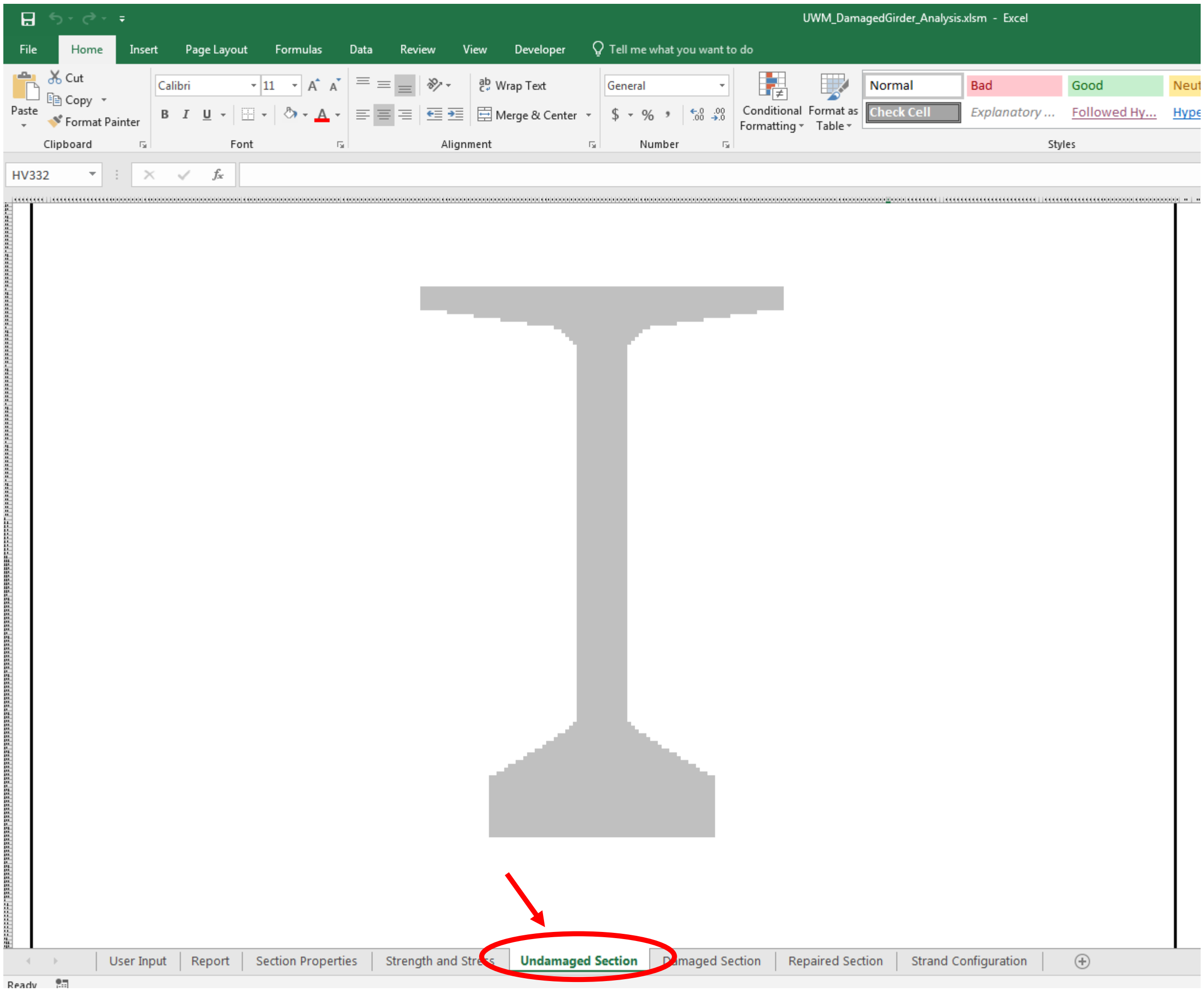


To calculate properties for the girder section alone, click on the button shown below.

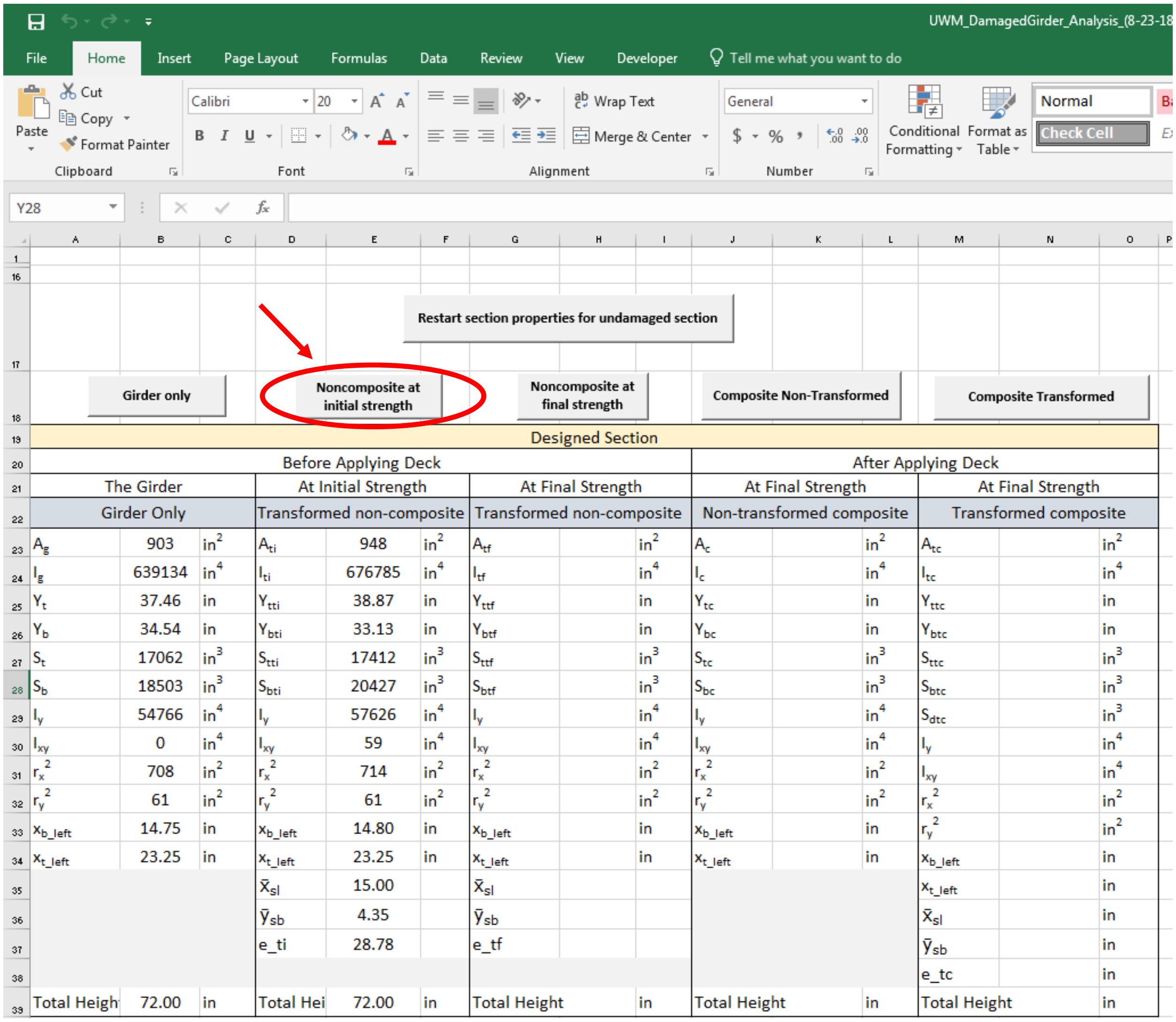


Designed Section										
Before Applying Deck							After Applying Deck			
The Girder			At Initial Strength		At Final Strength		At Final Strength		At Final Strength	
Girder Only			Transformed non-composite		Transformed non-composite		Non-transformed composite		Transformed composite	
A_g	903	in^2	A_{ti}	in^2	A_{tf}	in^2	A_c	in^2	A_{tc}	in^2
I_g	639134	in^4	I_{ti}	in^4	I_{tf}	in^4	I_c	in^4	I_{tc}	in^4
Y_t	37.46	in	Y_{tti}	in	Y_{ttf}	in	Y_{tc}	in	Y_{ttc}	in
Y_b	34.54	in	Y_{bti}	in	Y_{btf}	in	Y_{bc}	in	Y_{btc}	in
S_t	17062	in^3	S_{tti}	in^3	S_{ttf}	in^3	S_{tc}	in^3	S_{ttc}	in^3
S_b	18503	in^3	S_{bti}	in^3	S_{btf}	in^3	S_{bc}	in^3	S_{btc}	in^3
I_y	54766	in^4	I_y	in^4	I_y	in^4	I_y	in^4	S_{dtc}	in^3
I_{xy}	0	in^4	I_{xy}	in^4	I_{xy}	in^4	I_{xy}	in^4	I_y	in^4
r_x^2	708	in^2	r_x^2	in^2	r_x^2	in^2	r_x^2	in^2	I_{xy}	in^4
r_y^2	61	in^2	r_y^2	in^2	r_y^2	in^2	r_y^2	in^2	r_x^2	in^2
x_{b_left}	14.75	in	x_{b_left}	in	x_{b_left}	in	x_{b_left}	in	r_y^2	in^2
x_{t_left}	23.25	in	x_{t_left}	in	x_{t_left}	in	x_{t_left}	in	x_{b_left}	in
			$\bar{x}_{sl}$		$\bar{x}_{sl}$				x_{t_left}	in
			$\bar{y}_{sb}$		$\bar{y}_{sb}$				$\bar{x}_{sl}$	in
			e_ti		e_tf				$\bar{y}_{sb}$	in
									e_tc	in
Total Heigh	72.00	in	Total Height	in	Total Height	in	Total Height	in	Total Height	in

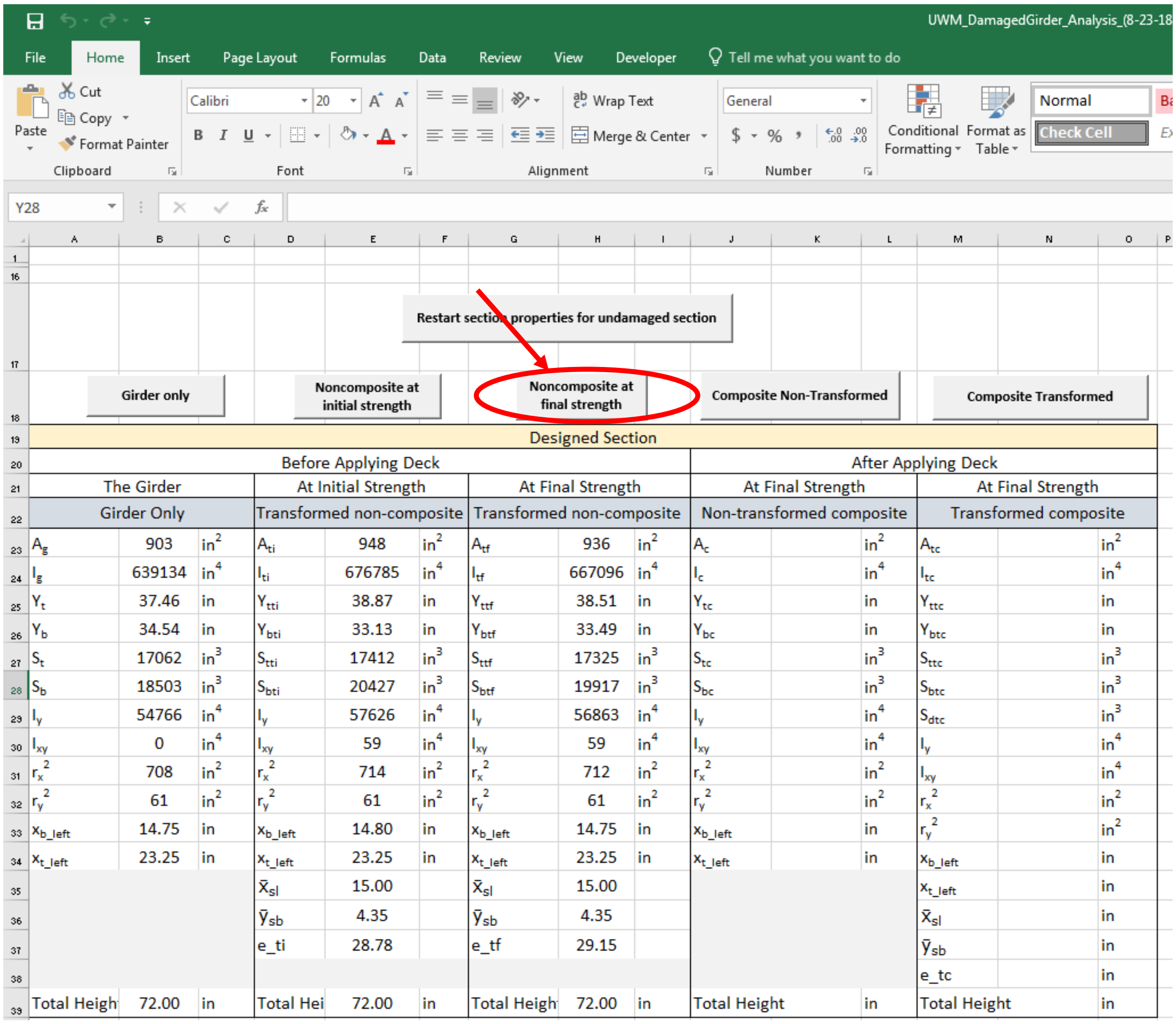
Generated section can be seen in the sheet “Undamaged Section”, as shown below.



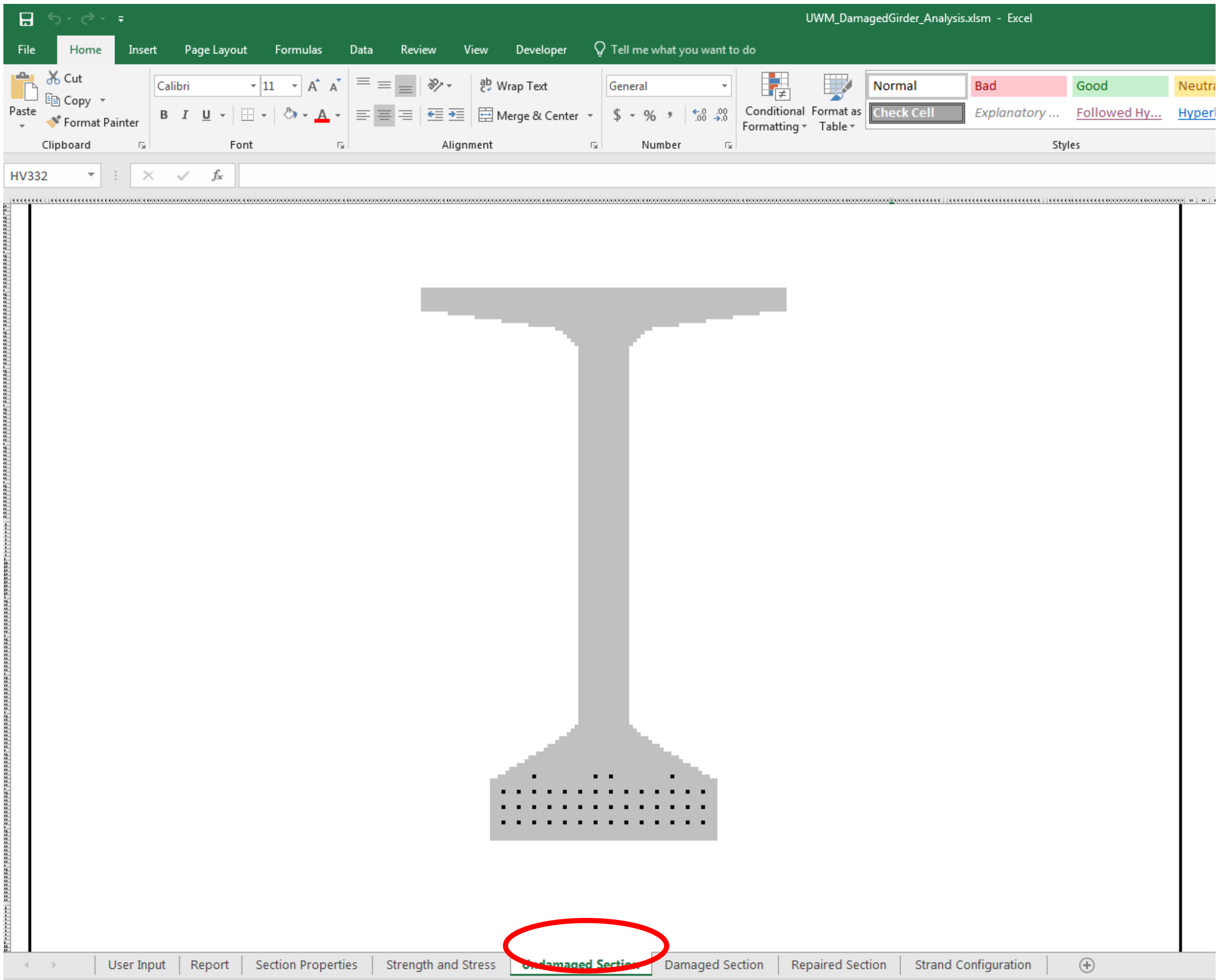
To calculate properties for transformed non-composite section at transfer (initial strength of the girder), click on the button shown below.



To calculate properties for transformed non-composite section at final strength, click on the button shown below.



Designed Section														
Before Applying Deck									After Applying Deck					
The Girder			At Initial Strength			At Final Strength			At Final Strength			At Final Strength		
Girder Only			Transformed non-composite			Transformed non-composite			Non-transformed composite			Transformed composite		
A_g	903	in^2	A_{ti}	948	in^2	A_{tf}	936	in^2	A_c		in^2	A_{tc}		in^2
I_g	639134	in^4	I_{ti}	676785	in^4	I_{tf}	667096	in^4	I_c		in^4	I_{tc}		in^4
Y_t	37.46	in	Y_{tti}	38.87	in	Y_{ttf}	38.51	in	Y_{tc}		in	Y_{ttc}		in
Y_b	34.54	in	Y_{bti}	33.13	in	Y_{btf}	33.49	in	Y_{bc}		in	Y_{btc}		in
S_t	17062	in^3	S_{tti}	17412	in^3	S_{ttf}	17325	in^3	S_{tc}		in^3	S_{ttc}		in^3
S_b	18503	in^3	S_{bti}	20427	in^3	S_{btf}	19917	in^3	S_{bc}		in^3	S_{btc}		in^3
I_y	54766	in^4	I_y	57626	in^4	I_y	56863	in^4	I_y		in^4	S_{dtc}		in^3
I_{xy}	0	in^4	I_{xy}	59	in^4	I_{xy}	59	in^4	I_{xy}		in^4	I_y		in^4
r_x^2	708	in^2	r_x^2	714	in^2	r_x^2	712	in^2	r_x^2		in^2	I_{xy}		in^4
r_y^2	61	in^2	r_y^2	61	in^2	r_y^2	61	in^2	r_y^2		in^2	r_x^2		in^2
x_{b_left}	14.75	in	x_{b_left}	14.80	in	x_{b_left}	14.75	in	x_{b_left}		in	r_y^2		in^2
x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}		in	x_{b_left}		in
			$\bar{x}_{sl}$	15.00		$\bar{x}_{sl}$	15.00					x_{t_left}		in
			$\bar{y}_{sb}$	4.35		$\bar{y}_{sb}$	4.35					$\bar{x}_{sl}$		in
			e_ti	28.78		e_tf	29.15					$\bar{y}_{sb}$		in
												e_tc		in
Total Heigh	72.00	in	Total Hei	72.00	in	Total Heigh	72.00	in	Total Height		in	Total Height		in


UWM_DamagedGirder_Analysis.xlsm - Excel
File
Home
Insert
Page Layout
Formulas
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Developer
Tell me what you want to do
Cut
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Format Painter
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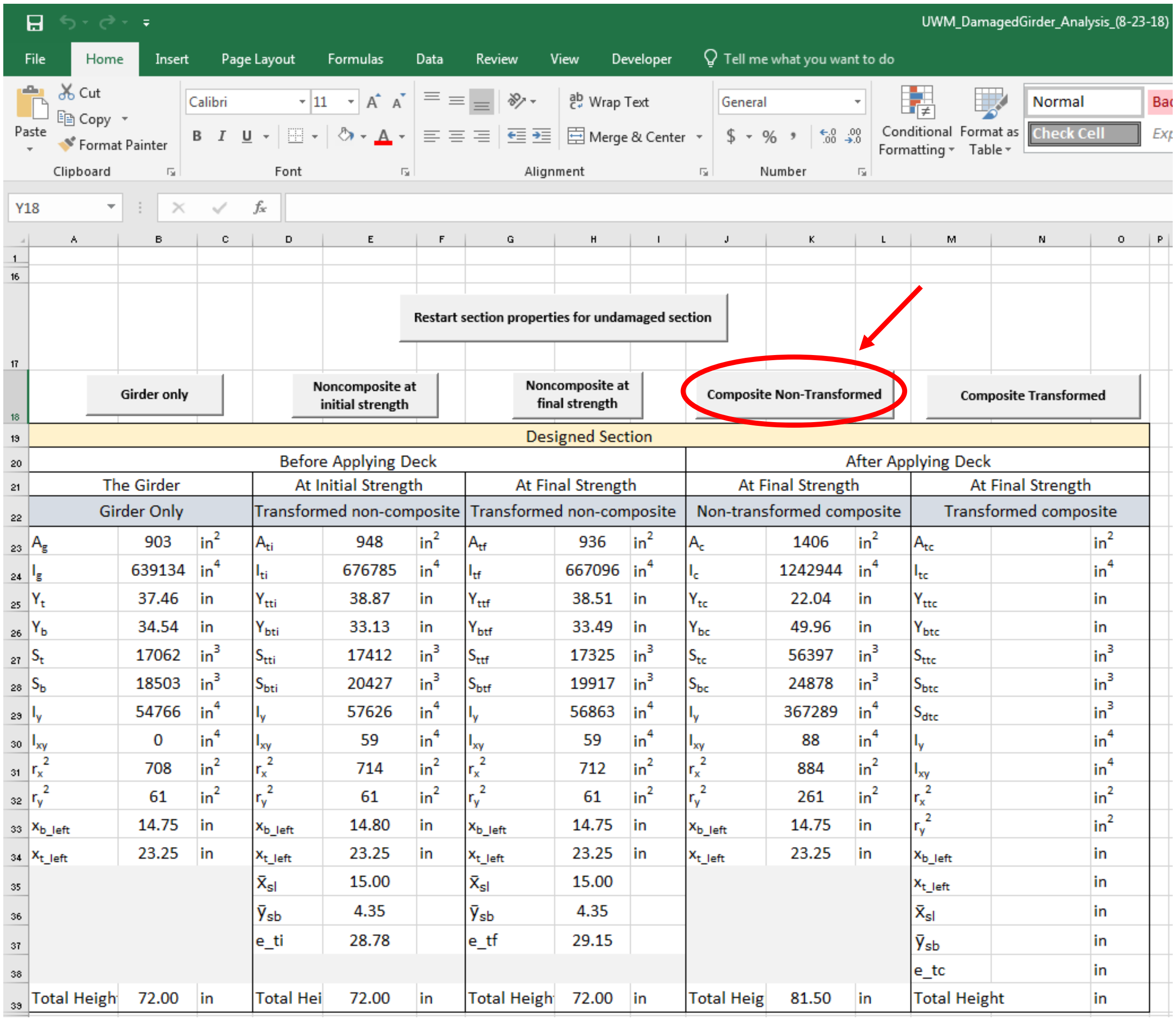
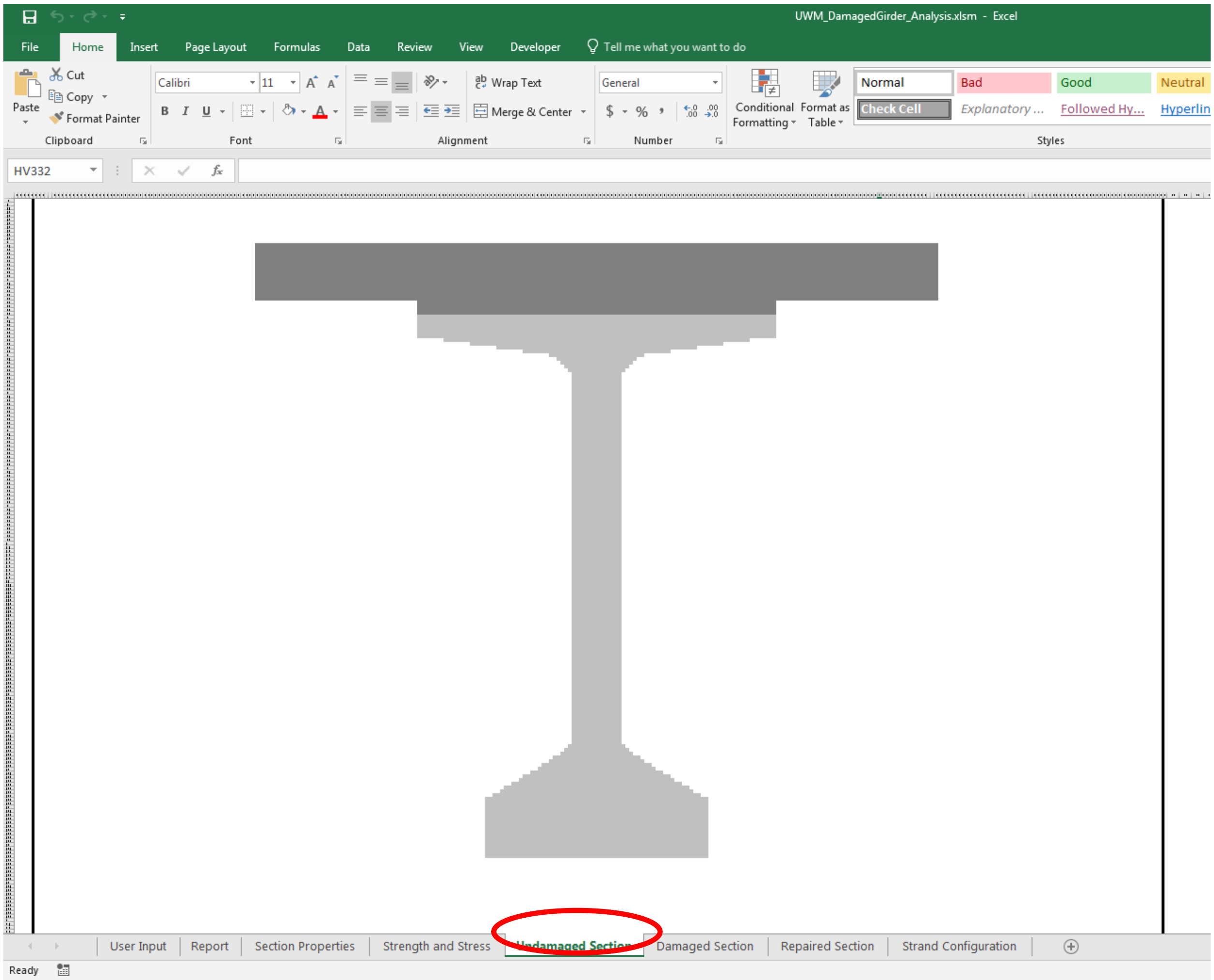
Styles
HV332
User Input
Report
Section Properties
Strength and Stress
Undamaged Section
Damaged Section
Repaired Section
Strand Configuration

To calculate properties for non-transformed composite section at final strength, click on the button shown below.



Designed Section														
Before Applying Deck									After Applying Deck					
The Girder			At Initial Strength			At Final Strength			At Final Strength			At Final Strength		
Girder Only			Transformed non-composite			Transformed non-composite			Non-transformed composite			Transformed composite		
A_g	903	in^2	A_{ti}	948	in^2	A_{tf}	936	in^2	A_c	1406	in^2	A_{tc}		in^2
I_g	639134	in^4	I_{ti}	676785	in^4	I_{tf}	667096	in^4	I_c	1242944	in^4	I_{tc}		in^4
Y_t	37.46	in	Y_{tti}	38.87	in	Y_{ttf}	38.51	in	Y_{tc}	22.04	in	Y_{ttc}		in
Y_b	34.54	in	Y_{bti}	33.13	in	Y_{btf}	33.49	in	Y_{bc}	49.96	in	Y_{btc}		in
S_t	17062	in^3	S_{tti}	17412	in^3	S_{ttf}	17325	in^3	S_{tc}	56397	in^3	S_{ttc}		in^3
S_b	18503	in^3	S_{bti}	20427	in^3	S_{btf}	19917	in^3	S_{bc}	24878	in^3	S_{btc}		in^3
I_y	54766	in^4	I_y	57626	in^4	I_y	56863	in^4	I_y	367289	in^4	S_{dtc}		in^3
I_{xy}	0	in^4	I_{xy}	59	in^4	I_{xy}	59	in^4	I_{xy}	88	in^4	I_y		in^4
r_x^2	708	in^2	r_x^2	714	in^2	r_x^2	712	in^2	r_x^2	884	in^2	I_{xy}		in^4
r_y^2	61	in^2	r_y^2	61	in^2	r_y^2	61	in^2	r_y^2	261	in^2	r_x^2		in^2
x_{b_left}	14.75	in	x_{b_left}	14.80	in	x_{b_left}	14.75	in	x_{b_left}	14.75	in	r_y^2		in^2
x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{b_left}		in
			$\bar{x}_{sl}$	15.00		$\bar{x}_{sl}$	15.00					x_{t_left}		in
			$\bar{y}_{sb}$	4.35		$\bar{y}_{sb}$	4.35					$\bar{x}_{sl}$		in
			e_ti	28.78		e_tf	29.15					$\bar{y}_{sb}$		in
												e_tc		in
Total Heigh	72.00	in	Total Hei	72.00	in	Total Heigh	72.00	in	Total Heig	81.50	in	Total Height		in



UWM_DamagedGirder_Analysis.xlsm - Excel
File
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Formulas
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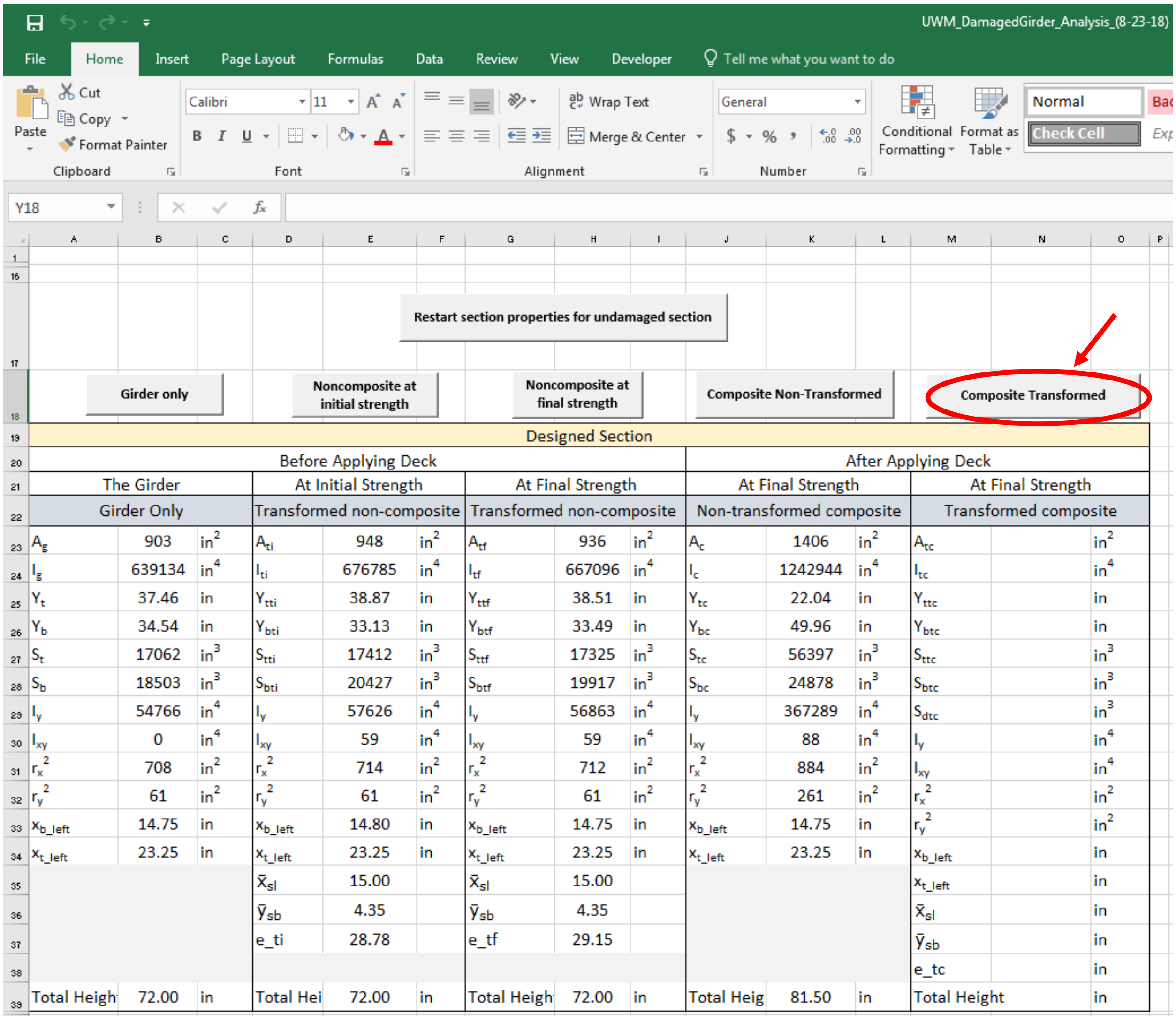
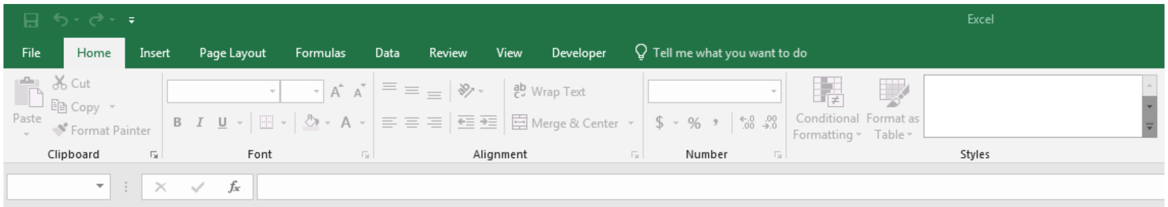
View
Developer
Tell me what you want to do
Cut
Copy
Paste
Format Painter
Clipboard
Calibri
11
Font
Wrap Text
Merge & Center
Alignment
General
Number
Conditional Formatting
Format as Table
Normal
Bad
Good
Neutral
Check Cell
Explanatory ...
Followed Hy...
Hyperlin
Styles
HV332
User Input
Report
Section Properties
Strength and Stress
Undamaged Section
Damaged Section
Repaired Section
Strand Configuration
Ready

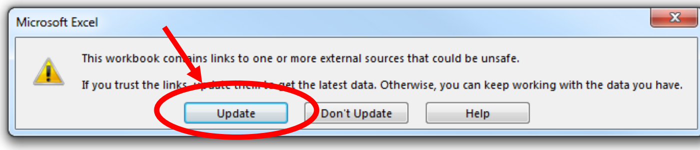
To calculate properties for transformed composite section at final strength, click on the button shown below.

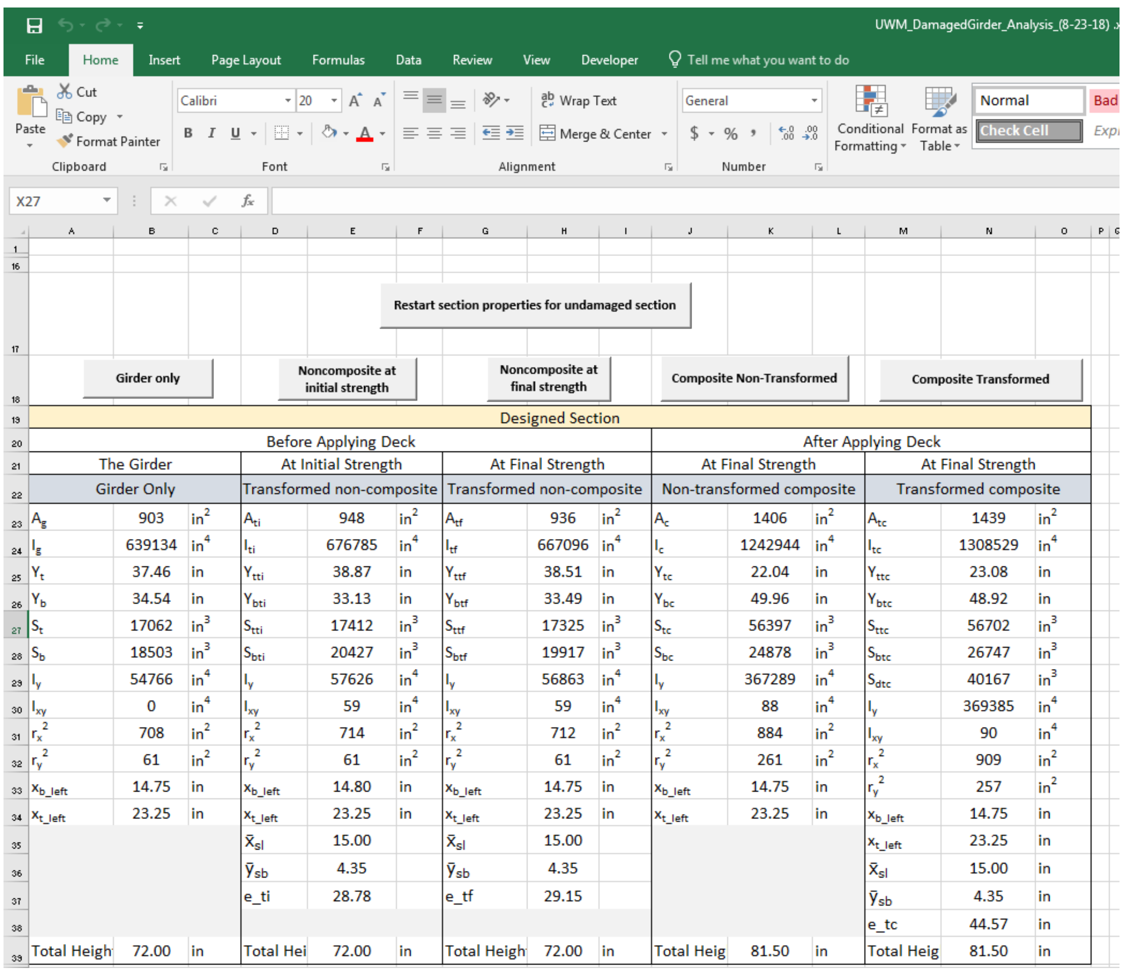


Designed Section													
Before Applying Deck									After Applying Deck				
The Girder			At Initial Strength			At Final Strength			At Final Strength			At Final Strength	
Girder Only			Transformed non-composite			Transformed non-composite			Non-transformed composite			Transformed composite	
A_g	903	in^2	A_{ti}	948	in^2	A_{tf}	936	in^2	A_c	1406	in^2	A_{tc}	in^2
I_g	639134	in^4	I_{ti}	676785	in^4	I_{tf}	667096	in^4	I_c	1242944	in^4	I_{tc}	in^4
Y_t	37.46	in	Y_{tti}	38.87	in	Y_{ttf}	38.51	in	Y_{tc}	22.04	in	Y_{ttc}	in
Y_b	34.54	in	Y_{bti}	33.13	in	Y_{btf}	33.49	in	Y_{bc}	49.96	in	Y_{btc}	in
S_t	17062	in^3	S_{tti}	17412	in^3	S_{ttf}	17325	in^3	S_{tc}	56397	in^3	S_{ttc}	in^3
S_b	18503	in^3	S_{bti}	20427	in^3	S_{btf}	19917	in^3	S_{bc}	24878	in^3	S_{btc}	in^3
I_y	54766	in^4	I_y	57626	in^4	I_y	56863	in^4	I_y	367289	in^4	S_{dtc}	in^3
I_{xy}	0	in^4	I_{xy}	59	in^4	I_{xy}	59	in^4	I_{xy}	88	in^4	I_y	in^4
r_x^2	708	in^2	r_x^2	714	in^2	r_x^2	712	in^2	r_x^2	884	in^2	I_{xy}	in^4
r_y^2	61	in^2	r_y^2	61	in^2	r_y^2	61	in^2	r_y^2	261	in^2	r_x^2	in^2
x_{b_left}	14.75	in	x_{b_left}	14.80	in	x_{b_left}	14.75	in	x_{b_left}	14.75	in	r_y^2	in^2
x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{b_left}	in
			$\bar{x}_{sl}$	15.00		$\bar{x}_{sl}$	15.00					x_{t_left}	in
			$\bar{y}_{sb}$	4.35		$\bar{y}_{sb}$	4.35					$\bar{x}_{sl}$	in
			e_ti	28.78		e_tf	29.15					$\bar{y}_{sb}$	in
												e_tc	in
Total Heigh	72.00	in	Total Hei	72.00	in	Total Heigh	72.00	in	Total Heig	81.50	in	Total Height	in

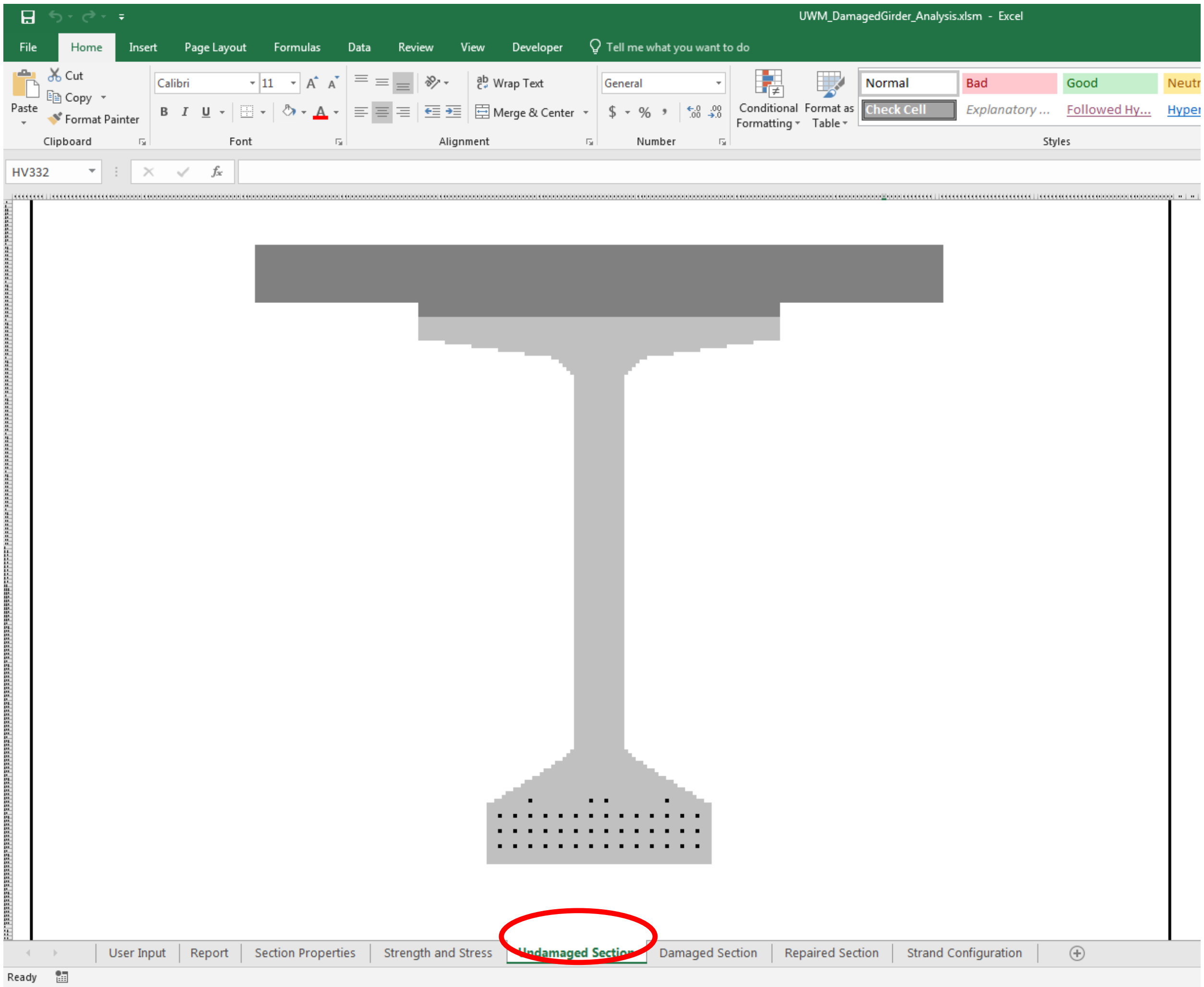
At this stage, the section properties and input data would be transferred to the linked workbook for load calculations. Click on "Update" in the figure below to successfully transfer data. Section properties at each step will be printed in a table as shown below.







Designed Section														
Before Applying Deck									After Applying Deck					
The Girder			At Initial Strength			At Final Strength			At Final Strength			At Final Strength		
Girder Only			Transformed non-composite			Transformed non-composite			Non-transformed composite			Transformed composite		
A_g	903	in^2	A_{ti}	948	in^2	A_{tf}	936	in^2	A_c	1406	in^2	A_{tc}	1439	in^2
I_g	639134	in^4	I_{ti}	676785	in^4	I_{tf}	667096	in^4	I_c	1242944	in^4	I_{tc}	1308529	in^4
Y_t	37.46	in	Y_{tti}	38.87	in	Y_{ttf}	38.51	in	Y_{tc}	22.04	in	Y_{ttc}	23.08	in
Y_b	34.54	in	Y_{bti}	33.13	in	Y_{btf}	33.49	in	Y_{bc}	49.96	in	Y_{btc}	48.92	in
S_t	17062	in^3	S_{tti}	17412	in^3	S_{ttf}	17325	in^3	S_{tc}	56397	in^3	S_{ttc}	56702	in^3
S_b	18503	in^3	S_{bti}	20427	in^3	S_{btf}	19917	in^3	S_{bc}	24878	in^3	S_{btc}	26747	in^3
I_y	54766	in^4	I_y	57626	in^4	I_y	56863	in^4	I_y	367289	in^4	S_{dtc}	40167	in^3
I_{xy}	0	in^4	I_{xy}	59	in^4	I_{xy}	59	in^4	I_{xy}	88	in^4	I_y	369385	in^4
r_x^2	708	in^2	r_x^2	714	in^2	r_x^2	712	in^2	r_x^2	884	in^2	I_{xy}	90	in^4
r_y^2	61	in^2	r_y^2	61	in^2	r_y^2	61	in^2	r_y^2	261	in^2	r_x^2	909	in^2
x_{b_left}	14.75	in	x_{b_left}	14.80	in	x_{b_left}	14.75	in	x_{b_left}	14.75	in	r_y^2	257	in^2
x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{t_left}	23.25	in	x_{b_left}	14.75	in
			$\bar{x}_{sl}$	15.00		$\bar{x}_{sl}$	15.00					x_{t_left}	23.25	in
			$\bar{y}_{sb}$	4.35		$\bar{y}_{sb}$	4.35					$\bar{x}_{sl}$	15.00	in
			e_ti	28.78		e_tf	29.15					$\bar{y}_{sb}$	4.35	in
												e_tc	44.57	in
Total Heigh	72.00	in	Total Hei	72.00	in	Total Heigh	72.00	in	Total Heig	81.50	in	Total Heig	81.50	in


UWM_DamagedGirder_Analysis.xlsm - Excel
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Conditional Formatting
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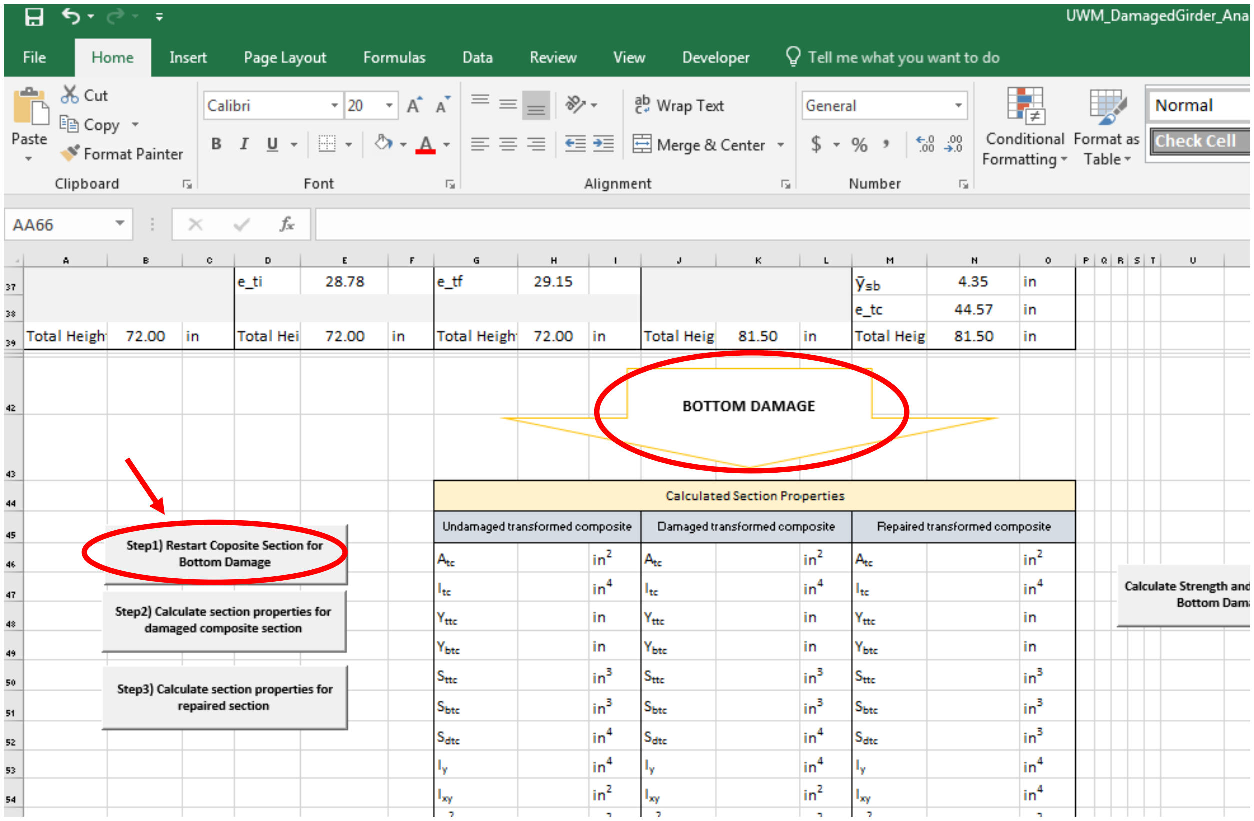
Normal
Bad
Good
Check Cell
Explanatory ...
Followed Hy...
Styles
HV332
User Input
Report
Section Properties
Strength and Stress
Undamaged Section
Damaged Section
Repaired Section
Strand Configuration
Ready

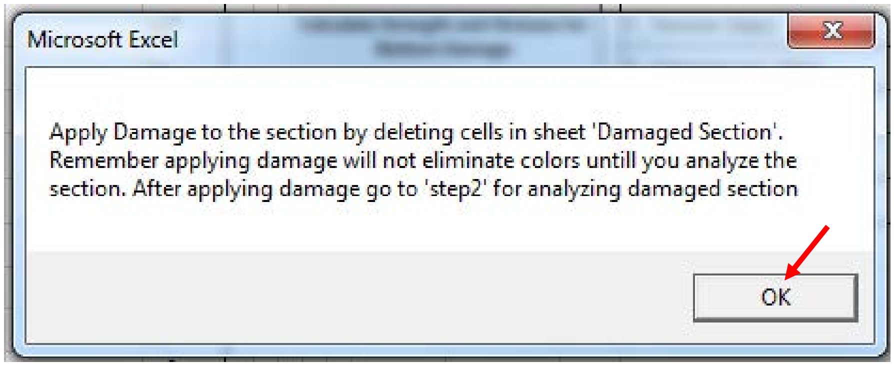
A.3 Bottom Damage

Bottom damage can be simulated in PreBARS in 3 steps.

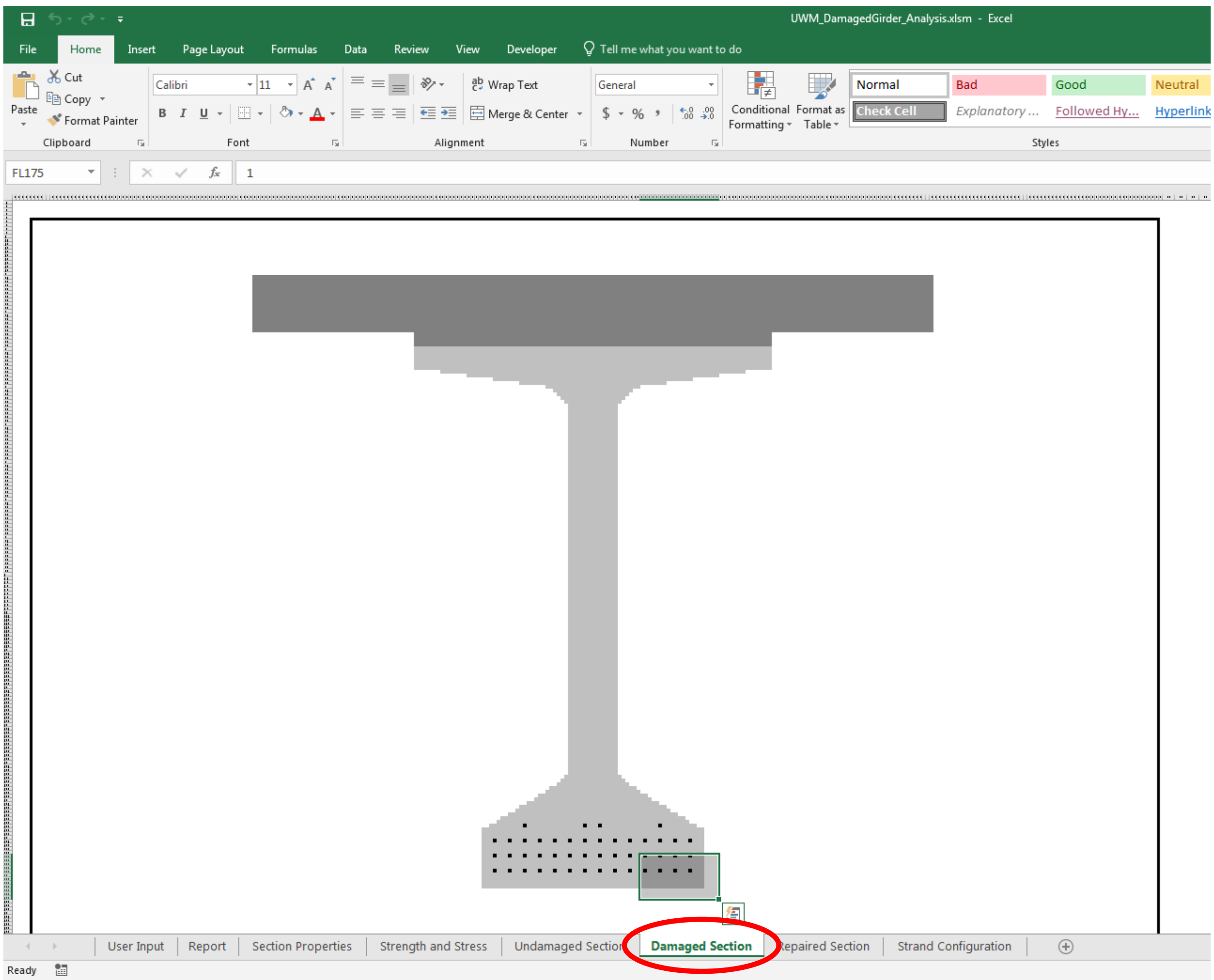
Step1. In sheet "Section Properties", go to bottom damage section and click on Step 1.



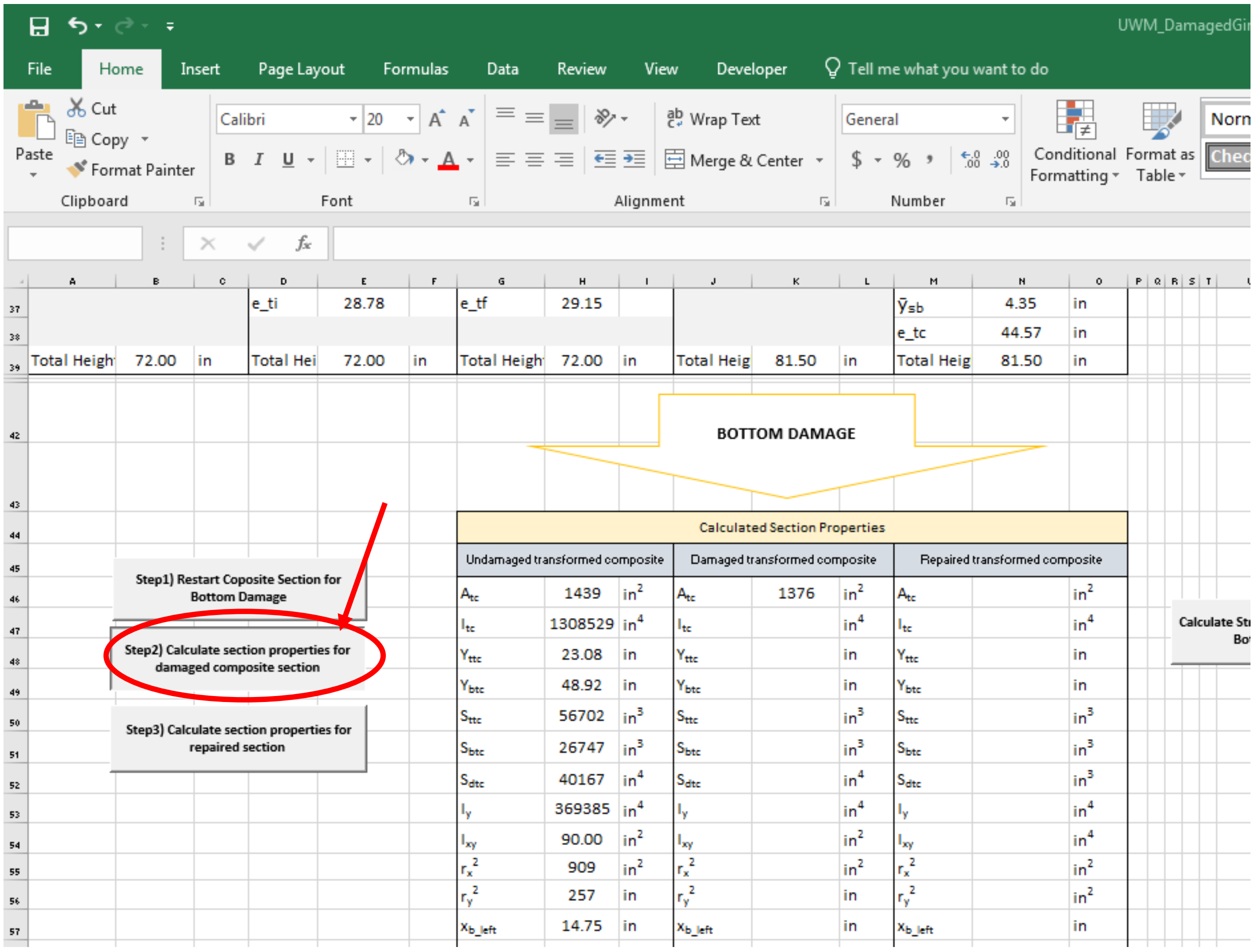
At this stage, the software generates undamaged transformed composite section in both "Undamaged Section" and "Damaged Section" sheets and calculates section properties for undamaged section. These data would be transferred to the load calculation workbook. A message box will pop up asking the user to apply damage in "Damaged Section" sheet.



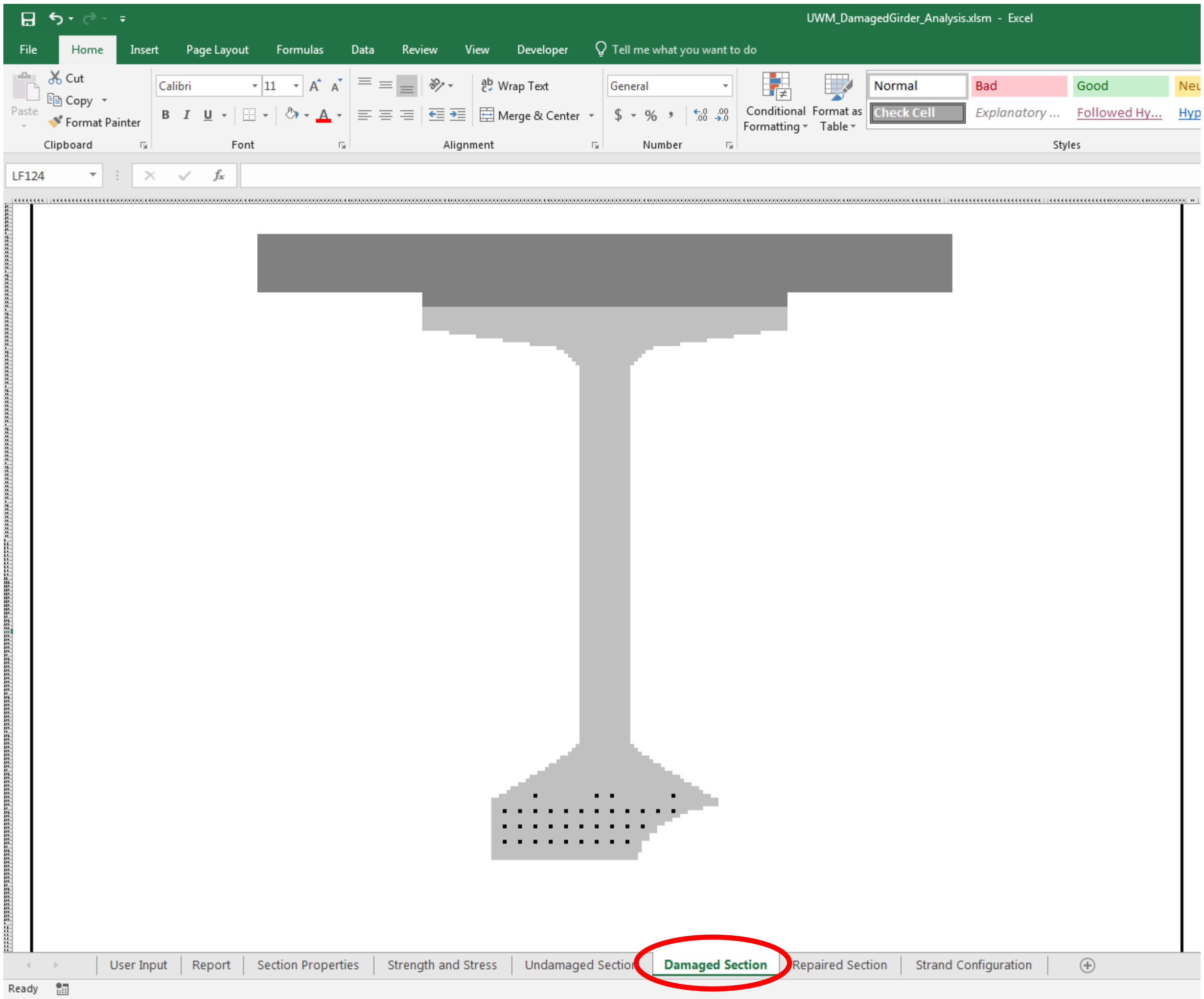
Click on the tab "Damages Section" and apply damage to the damaged area by deleting cell values including severed strands.



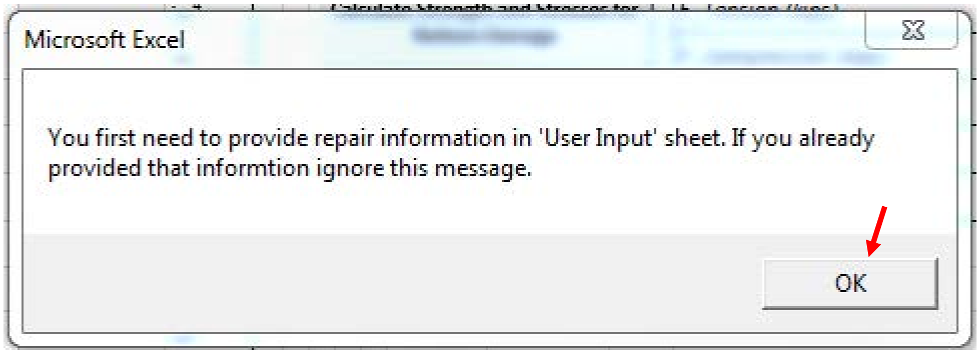
Step2. Go back to tab “Section Properties” to calculate damaged section properties.



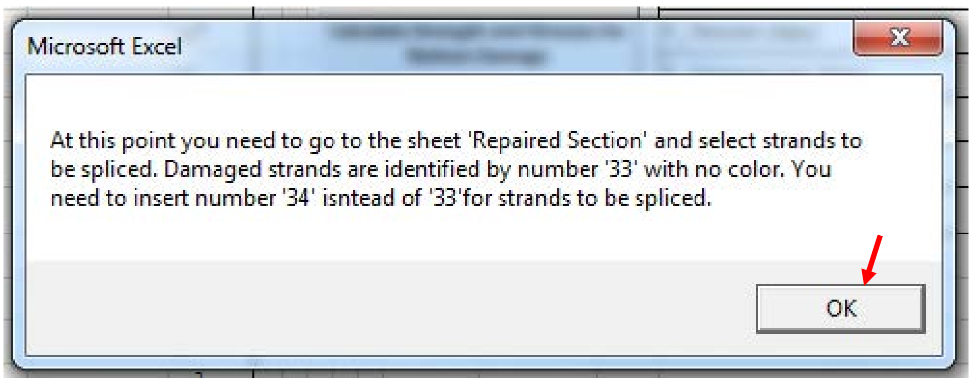
In this example, 11 strands are severed and deleted from the model.

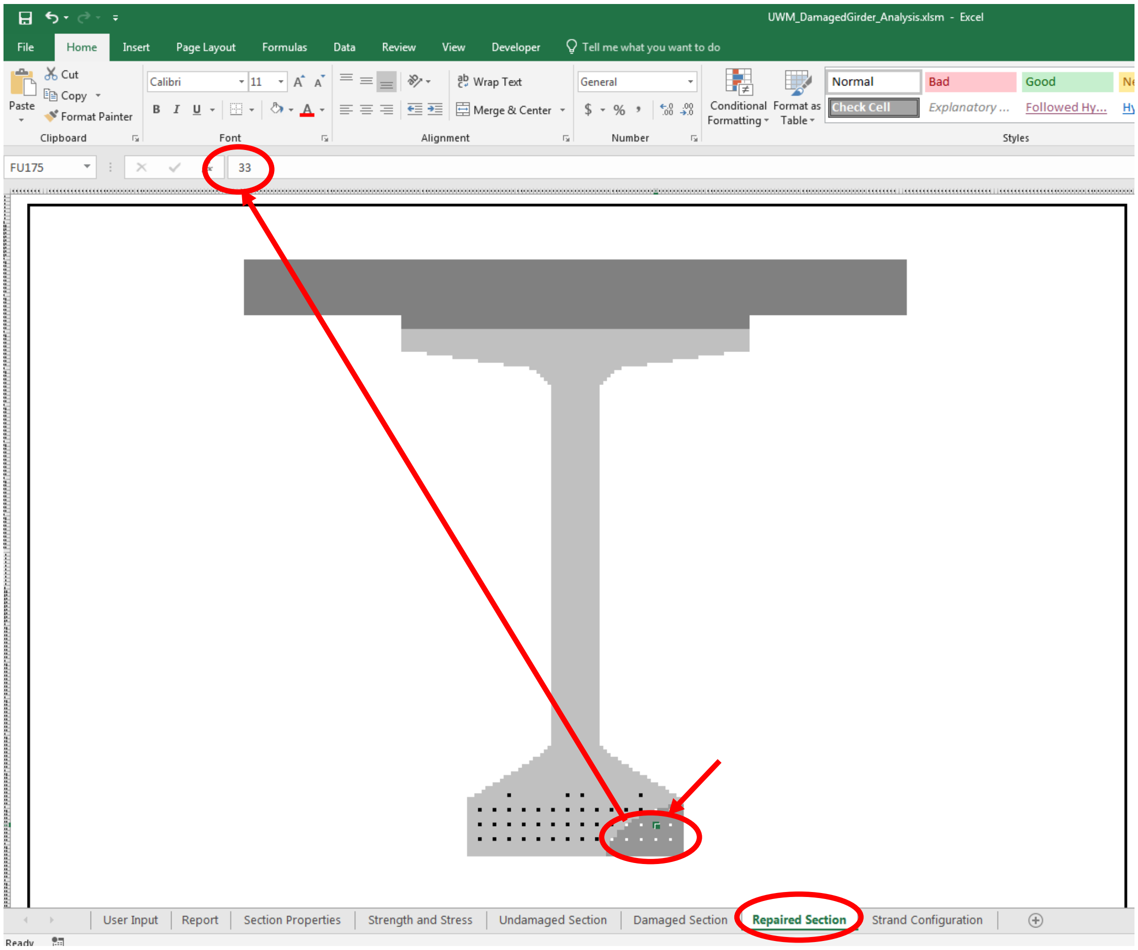


As the section properties for damaged section is calculated, a section for applying repair would be generated in the "Repair Section" sheet. A message box asking for repair information will pop up according to Figure 34.

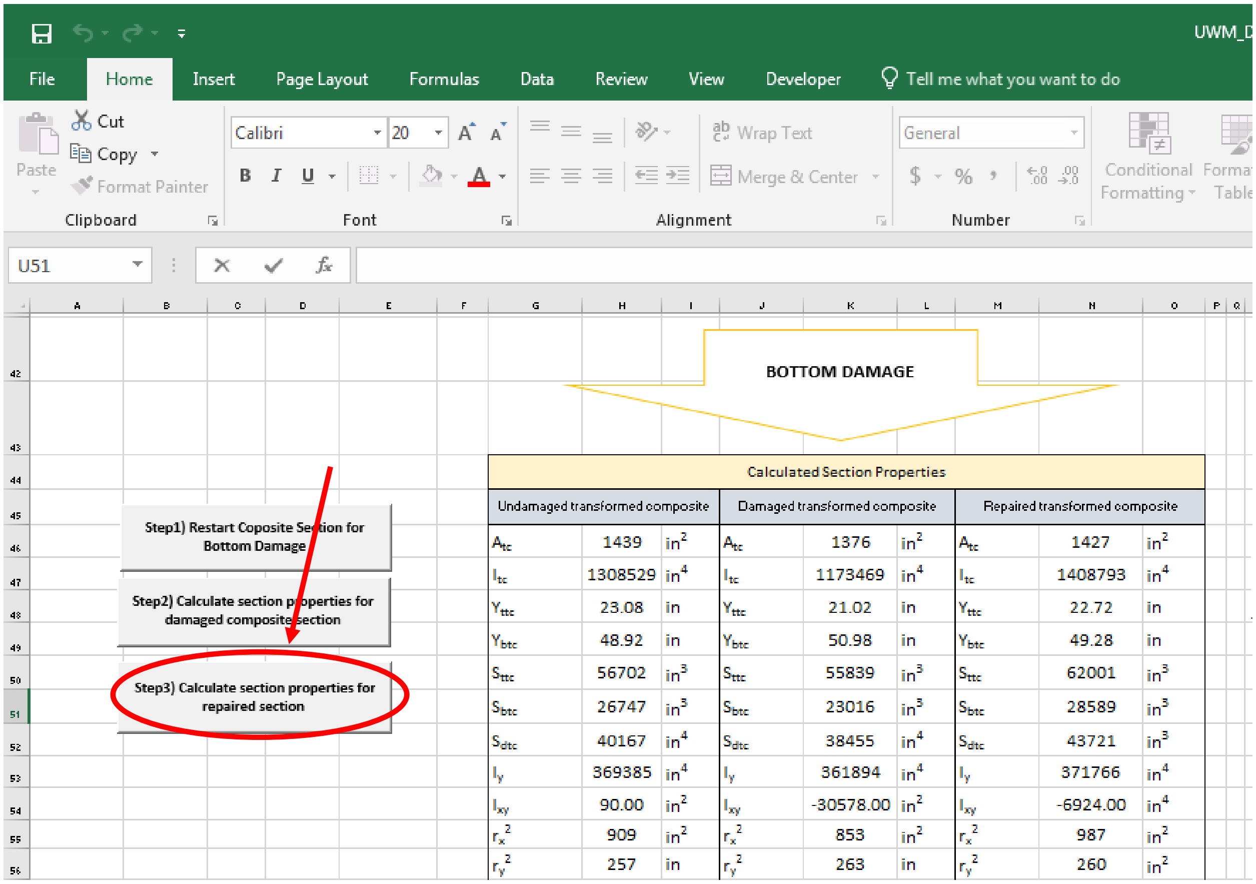


When the repaired section is generated, the severed strands values are replaced by value of "33", indicating severed strands. In the next step the user should specify spliced strands by entering value of "34", instead of "33" in the corresponding cells, as shown in the following figures.



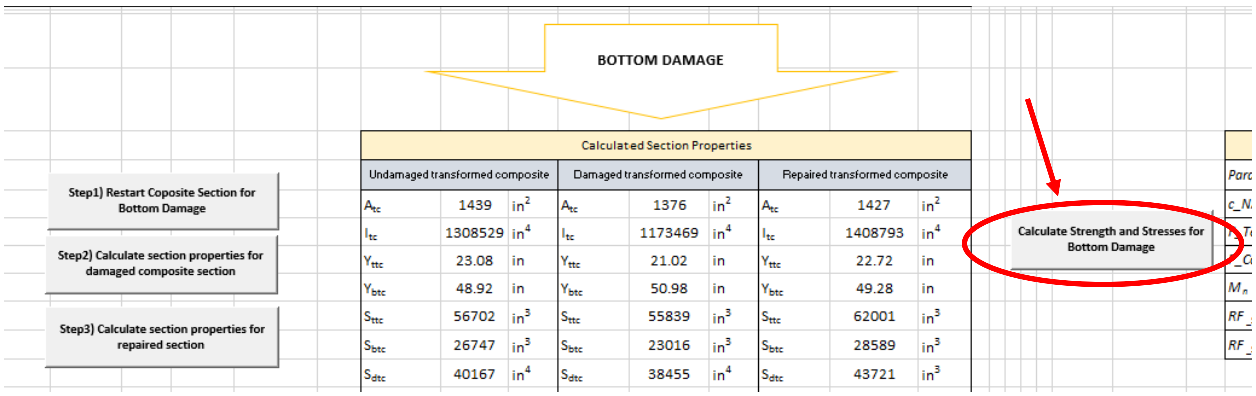

UWM_DamagedGirder_Analysis.xlsm - Excel
File
Home
Insert
Page Layout
Formulas
Data
Review
View
Developer
Tell me what you want to do
FU175
33
User Input
Report
Section Properties
Strength and Stress
Undamaged Section
Damaged Section
Repaired Section
Strand Configuration

Step 3. Click on the button for step 3 to calculate section properties for repaired section.



Calculated Section Properties								
Undamaged transformed composite			Damaged transformed composite			Repaired transformed composite		
A_{tc}	1439	in^2	A_{tc}	1376	in^2	A_{tc}	1427	in^2
I_{tc}	1308529	in^4	I_{tc}	1173469	in^4	I_{tc}	1408793	in^4
Y_{ttc}	23.08	in	Y_{ttc}	21.02	in	Y_{ttc}	22.72	in
Y_{btc}	48.92	in	Y_{btc}	50.98	in	Y_{btc}	49.28	in
S_{ttc}	56702	in^3	S_{ttc}	55839	in^3	S_{ttc}	62001	in^3
S_{btc}	26747	in^3	S_{btc}	23016	in^3	S_{btc}	28589	in^3
S_{dtc}	40167	in^4	S_{dtc}	38455	in^4	S_{dtc}	43721	in^3
I_y	369385	in^4	I_y	361894	in^4	I_y	371766	in^4
I_{xy}	90.00	in^2	I_{xy}	-30578.00	in^2	I_{xy}	-6924.00	in^4
r_x^2	909	in^2	r_x^2	853	in^2	r_x^2	987	in^2
r_y^2	257	in	r_y^2	263	in	r_y^2	260	in^2

To calculate strength and stresses click on the button shown below.



Calculated Section Properties								
Undamaged transformed composite			Damaged transformed composite			Repaired transformed composite		
A_{tc}	1439	in^2	A_{tc}	1376	in^2	A_{tc}	1427	in^2
I_{tc}	1308529	in^4	I_{tc}	1173469	in^4	I_{tc}	1408793	in^4
Y_{ttc}	23.08	in	Y_{ttc}	21.02	in	Y_{ttc}	22.72	in
Y_{btc}	48.92	in	Y_{btc}	50.98	in	Y_{btc}	49.28	in
S_{ttc}	56702	in^3	S_{ttc}	55839	in^3	S_{ttc}	62001	in^3
S_{btc}	26747	in^3	S_{btc}	23016	in^3	S_{btc}	28589	in^3
S_{dtc}	40167	in^4	S_{dtc}	38455	in^4	S_{dtc}	43721	in^3

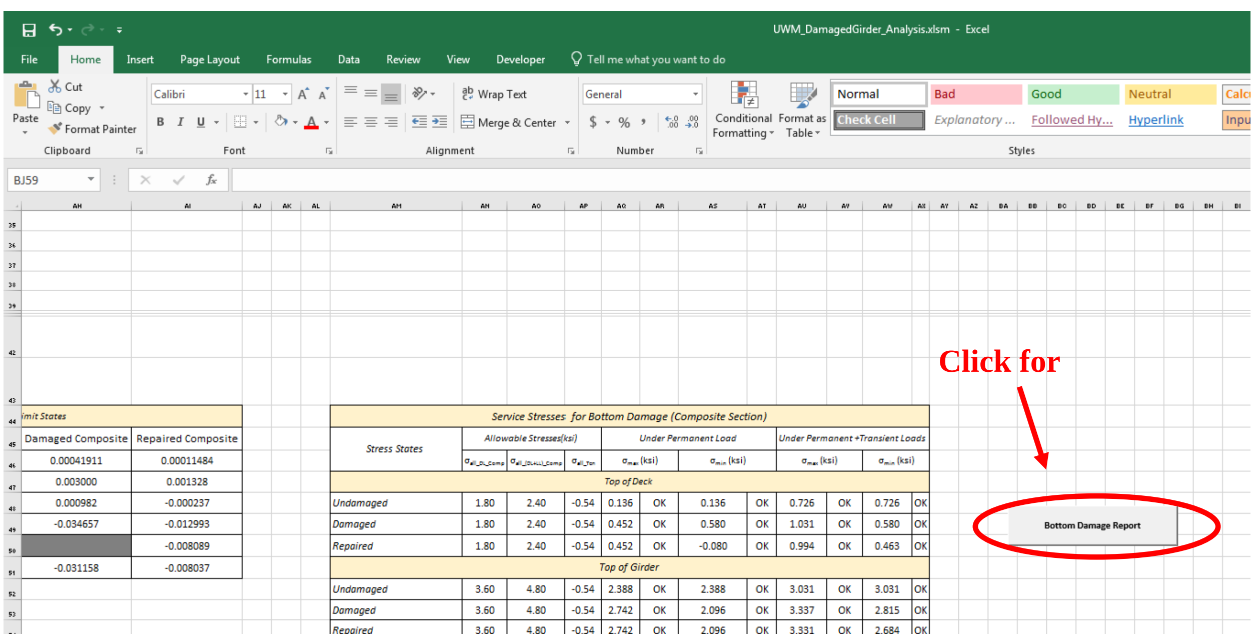
Following tables show calculations for strength and stresses.

Bottom Damage Strength Calculations			
Parameters	Undamaged Composite	Damaged Composite	Repaired Composite
c_NA (in)	9.83	7.16	11.57
F_Tension (kips)	2627	1946	2370
F_Compression (kips)	2687	2037	2549
M_n (ft-kips)	16142	12210	14845
$RF_{\_Str\_Inv}$	1.72	0.98	1.48
$RF_{\_Str\_Oper}$	2.23	1.28	1.92

Curvature and Strain at Strength Limit States			
Strain and Curvature	Undamaged Composite	Damaged Composite	Repaired Composite
Curvature	0.00030527	0.00041911	0.00011484
Strain at the top of the deck	0.003000	0.003000	0.001328
Strain at the top of the girder	-0.000100	0.000982	-0.000237
Strain at the centroid of strands (tension)	-0.026013	-0.034657	-0.012993
Strain at the bottom of FRP (tension)			-0.008089
Strain at the bottom of the beam (tension)	-0.021880	-0.031158	-0.008037

Service Stresses for Bottom Damage (Composite Section)											
Stress States	Allowable Stresses(ksi)			Under Permanent Load				Under Permanent +Transient Loads			
	$\sigma_{all\_DL\_Comp}$	$\sigma_{all\_(DL+LL)\_Comp}$	σ_{all_Ten}	σ_{max} (ksi)		σ_{min} (ksi)		σ_{max} (ksi)		σ_{min} (ksi)	
Top of Deck											
Undamaged	1.80	2.40	-0.54	0.136	OK	0.136	OK	0.726	OK	0.726	OK
Damaged	1.80	2.40	-0.54	0.452	OK	0.580	OK	1.031	OK	0.580	OK
Repaired	1.80	2.40	-0.54	0.452	OK	-0.080	OK	0.994	OK	0.463	OK
Top of Girder											
Undamaged	3.60	4.80	-0.54	2.388	OK	2.388	OK	3.031	OK	3.031	OK
Damaged	3.60	4.80	-0.54	2.742	OK	2.096	OK	3.337	OK	2.815	OK
Repaired	3.60	4.80	-0.54	2.742	OK	2.096	OK	3.331	OK	2.684	OK
Bottom of Girder											
Undamaged	3.60	4.80	-0.54	1.285	OK	1.285	OK	0.205	OK	0.205	OK
Damaged	3.60	4.80	-0.54	0.728	OK	0.000	OK	0.000	OK	-0.763	NG
Repaired	3.60	4.80	-0.54	0.728	OK	0.000	OK	-0.283	OK	-0.778	NG
* Permanent load for undamaged and damaged girder (at the top and bottom of girder) includes girder+slab+future wearing surface+barriers dead load											
* Permanent load for repaired section (at the top and bottom of girder) includes girder+slab+future wearing surface+barriers dead load											
* Permanent load on top of deck includes only dead load from future wearing surface+barriers											
* Permanent+transient loads at each point include the corresponding dead load + live load											

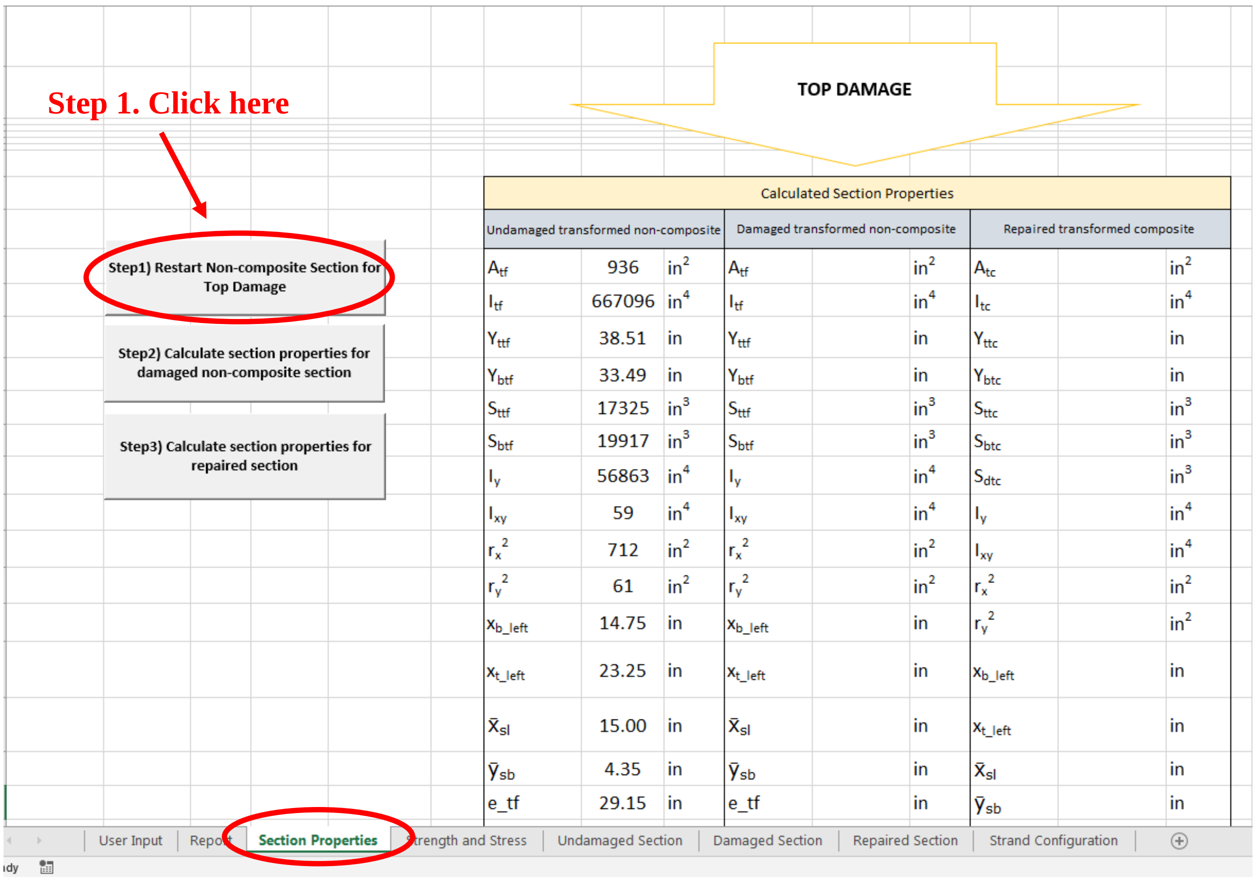
A button is created for s summary of all calculations including load calculation and section properties for undamaged, damaged, and repaired sections copied to the "Report" sheet.



A.4 Top Damage

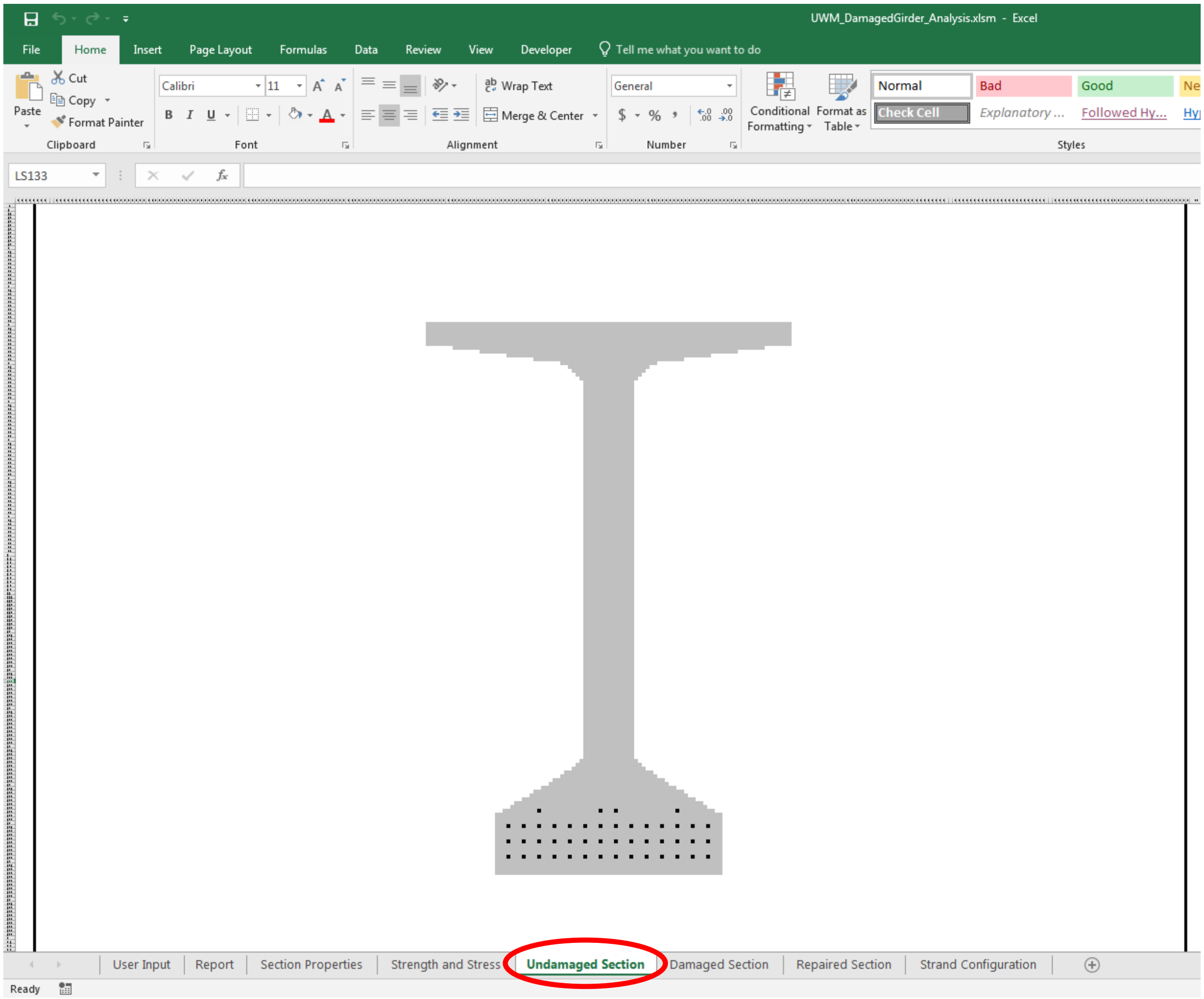
To simulate damage to a girder top flange, follow Step 1 through Step 3 in the section for top damage in the PreBARS software.

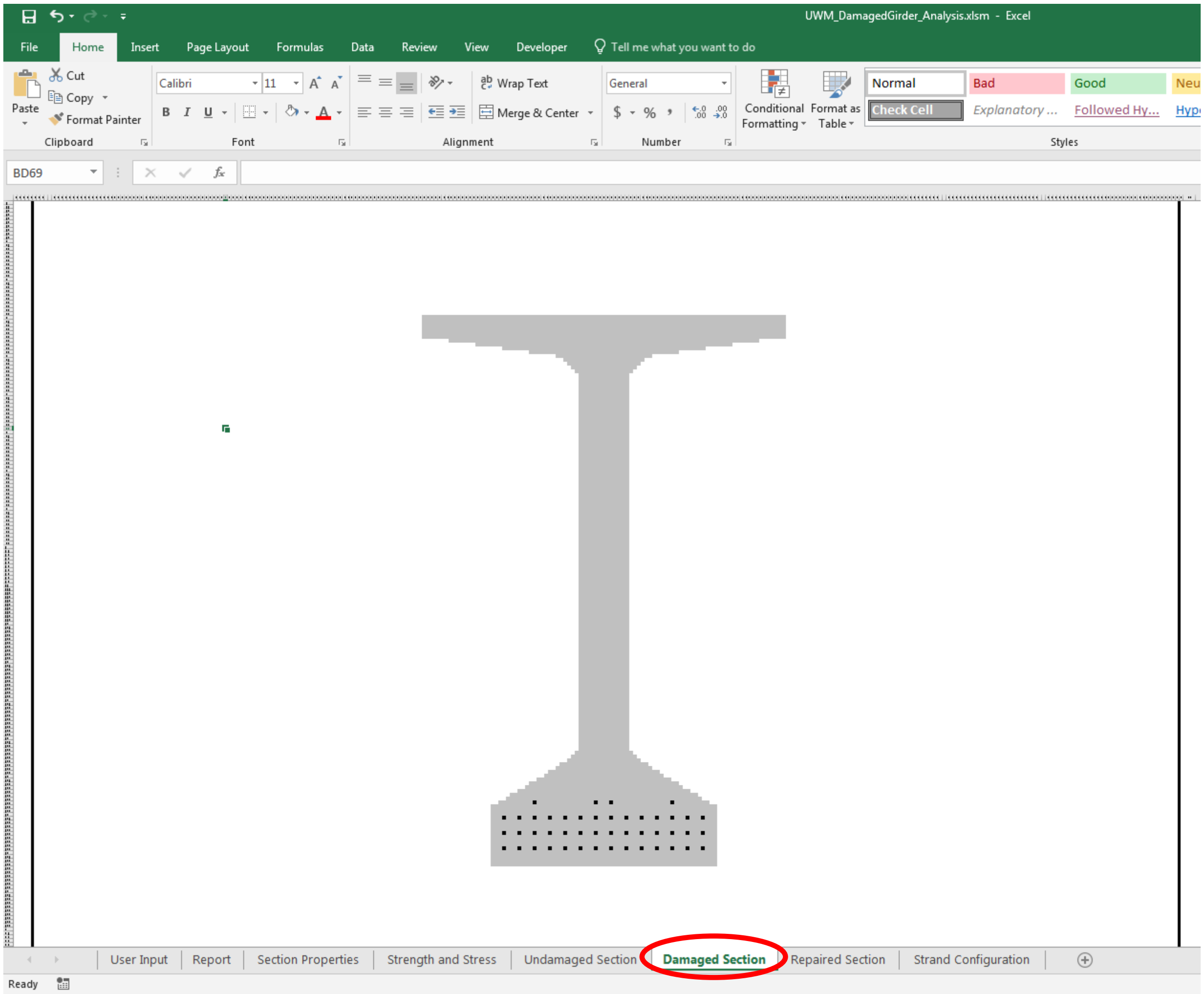
Step 1. Start to generate non-composite section to apply top damage in the tab “Section Properties”.



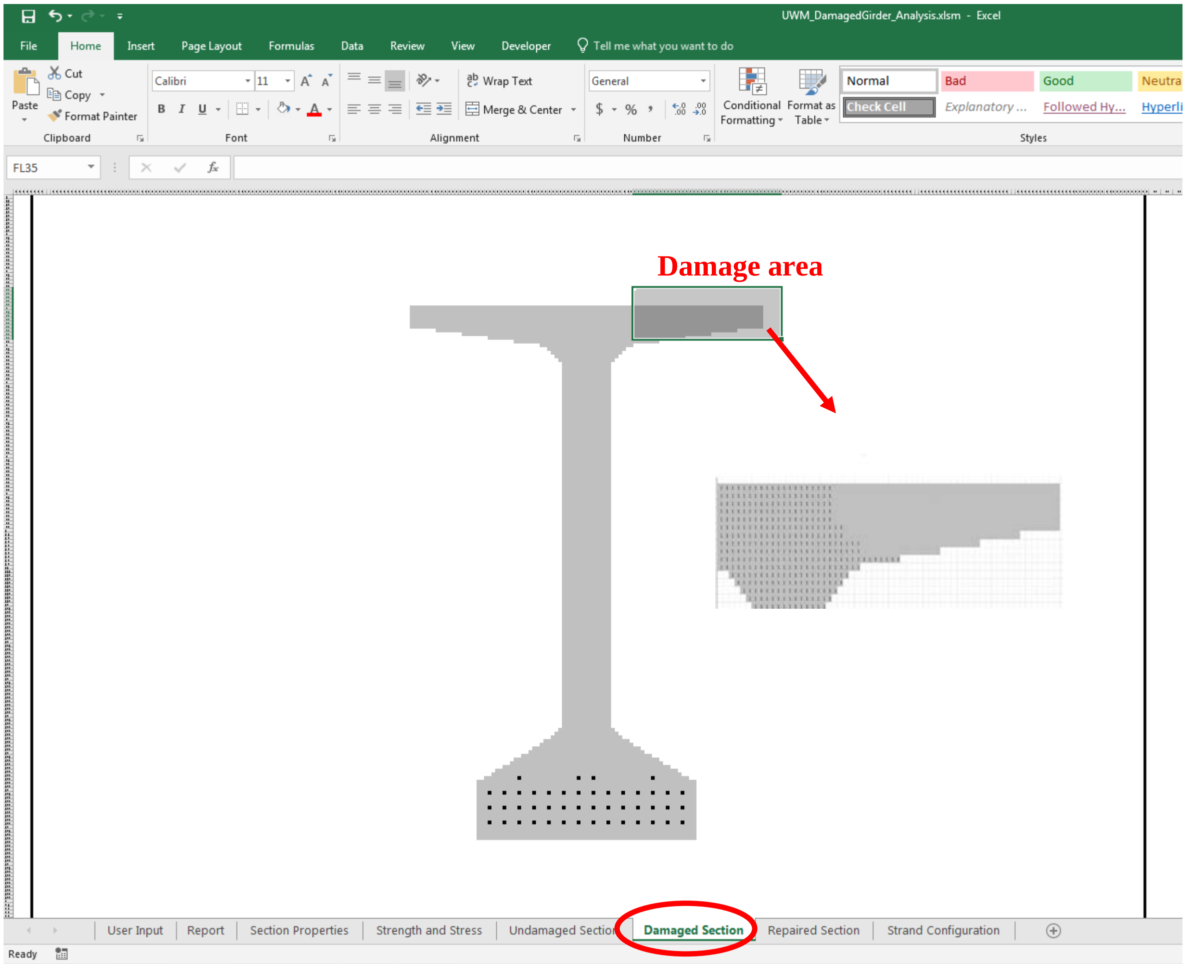
Calculated Section Properties						
Undamaged transformed non-composite			Damaged transformed non-composite		Repaired transformed composite	
A_{tf}	936	in^2	A_{tf}	in^2	A_{tc}	in^2
I_{tf}	667096	in^4	I_{tf}	in^4	I_{tc}	in^4
Y_{ttf}	38.51	in	Y_{ttf}	in	Y_{ttc}	in
Y_{btf}	33.49	in	Y_{btf}	in	Y_{btc}	in
S_{ttf}	17325	in^3	S_{ttf}	in^3	S_{ttc}	in^3
S_{btf}	19917	in^3	S_{btf}	in^3	S_{btc}	in^3
I_y	56863	in^4	I_y	in^4	S_{dtc}	in^3
I_{xy}	59	in^4	I_{xy}	in^4	I_y	in^4
r_x^2	712	in^2	r_x^2	in^2	I_{xy}	in^4
r_y^2	61	in^2	r_y^2	in^2	r_x^2	in^2
x_{b_left}	14.75	in	x_{b_left}	in	r_y^2	in^2
x_{t_left}	23.25	in	x_{t_left}	in	x_{b_left}	in
$\bar{x}_{sl}$	15.00	in	$\bar{x}_{sl}$	in	x_{t_left}	in
$\bar{y}_{sb}$	4.35	in	$\bar{y}_{sb}$	in	$\bar{x}_{sl}$	in
e_tf	29.15	in	e_tf	in	$\bar{y}_{sb}$	in

Non-composite sections would be generated in both “Undamaged Section” and “Damaged Section”.


UWM_DamagedGirder_Analysis.xlsm - Excel
File
Home
Insert
Page Layout
Formulas
Data
Review
View
Developer
Tell me what you want to do
Cut
Copy
Paste
Format Painter
Clipboard
Font
Wrap Text
Merge & Center
Alignment
General
Number
Conditional Formatting
Format as Table
Normal
Bad
Good
Check Cell
Explanatory ...
Followed Hy...
Styles
LS133
User Input
Report
Section Properties
Strength and Stress
Undamaged Section
Damaged Section
Repaired Section
Strand Configuration
Ready

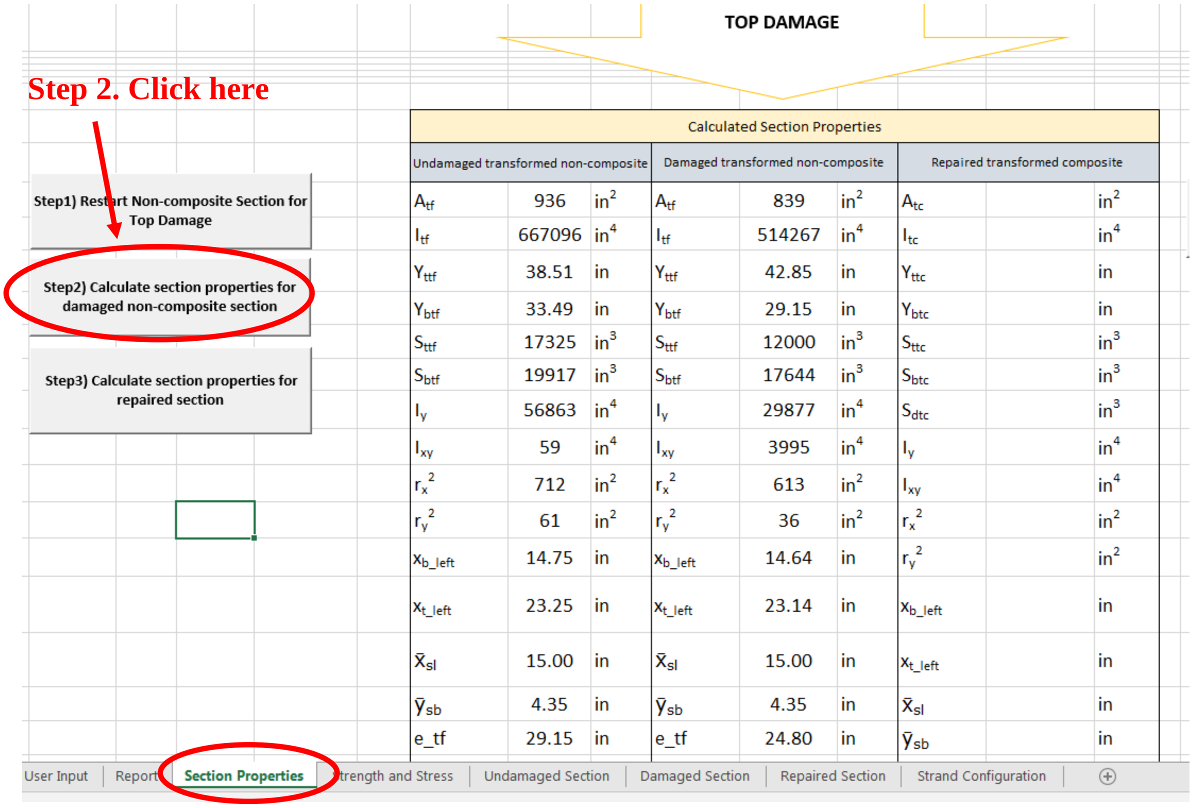


Go to the “Damaged Section” to apply top flange damage. For applying damage, delete cell values in the damaged area.



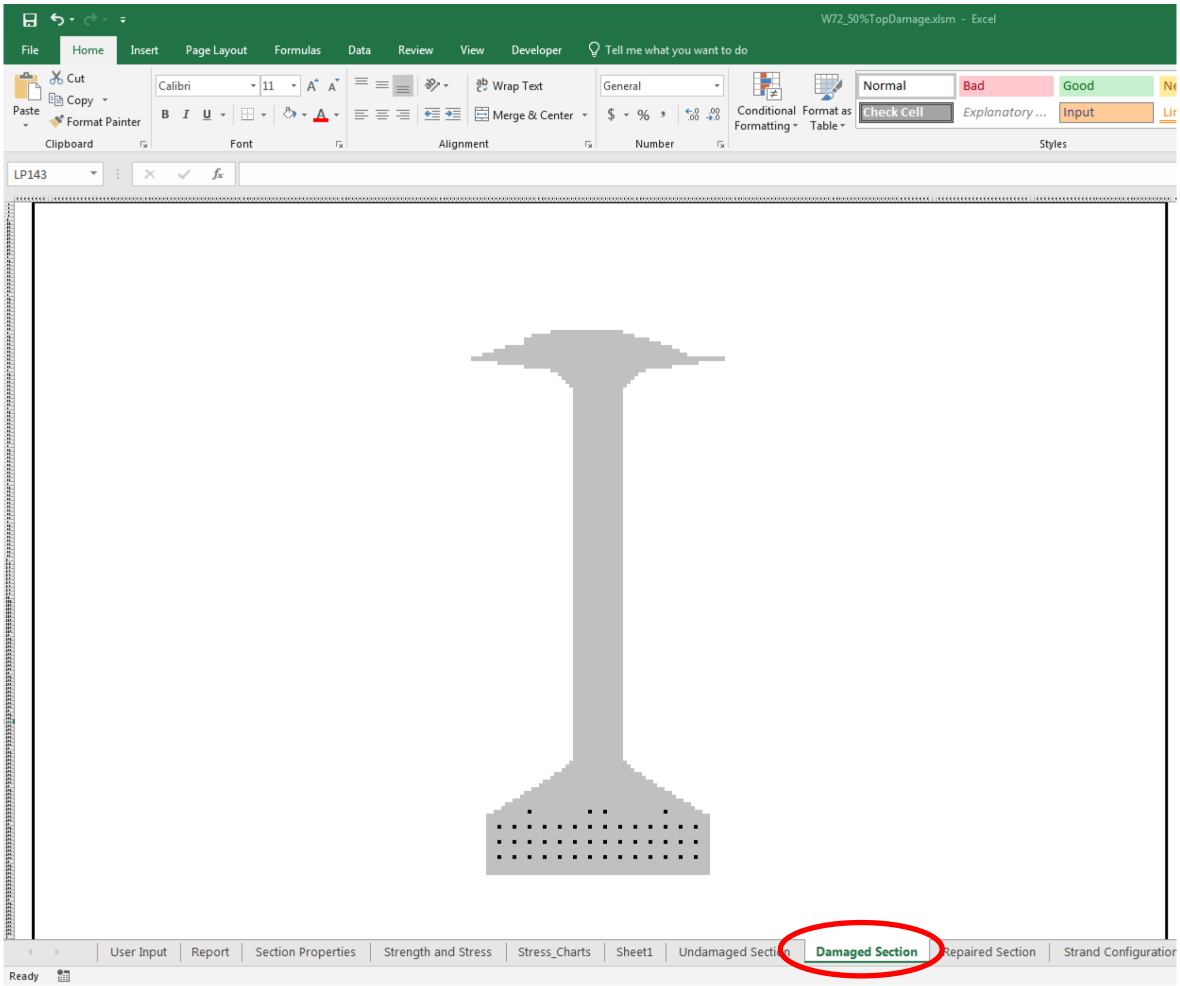
UWM_DamagedGirder_Analysis.xlsm - Excel
Damage area
User Input
Report
Section Properties
Strength and Stress
Undamaged Section
Damaged Section
Repaired Section
Strand Configuration

Step 2. Calculate damaged section properties in Step2, as shown below.



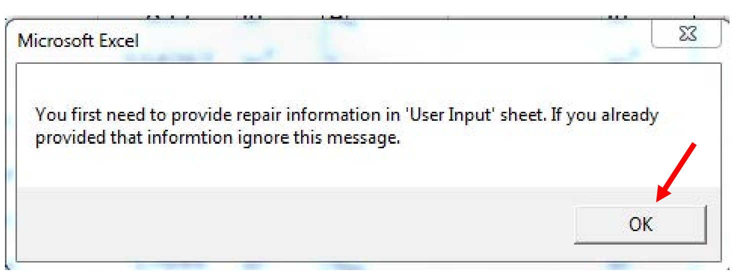
Calculated Section Properties							
Undamaged transformed non-composite			Damaged transformed non-composite			Repaired transformed composite	
A_{tf}	936	in^2	A_{tf}	839	in^2	A_{tc}	in^2
I_{tf}	667096	in^4	I_{tf}	514267	in^4	I_{tc}	in^4
Y_{ttf}	38.51	in	Y_{ttf}	42.85	in	Y_{ttc}	in
Y_{btf}	33.49	in	Y_{btf}	29.15	in	Y_{btc}	in
S_{ttf}	17325	in^3	S_{ttf}	12000	in^3	S_{ttc}	in^3
S_{btf}	19917	in^3	S_{btf}	17644	in^3	S_{btc}	in^3
I_y	56863	in^4	I_y	29877	in^4	S_{dtc}	in^3
I_{xy}	59	in^4	I_{xy}	3995	in^4	I_y	in^4
r_x^2	712	in^2	r_x^2	613	in^2	I_{xy}	in^4
r_y^2	61	in^2	r_y^2	36	in^2	r_x^2	in^2
x_{b_left}	14.75	in	x_{b_left}	14.64	in	r_y^2	in^2
x_{t_left}	23.25	in	x_{t_left}	23.14	in	x_{b_left}	in
$\bar{x}_{sl}$	15.00	in	$\bar{x}_{sl}$	15.00	in	x_{t_left}	in
$\bar{y}_{sb}$	4.35	in	$\bar{y}_{sb}$	4.35	in	$\bar{x}_{sl}$	in
e_tf	29.15	in	e_tf	24.80	in	$\bar{y}_{sb}$	in

Following figure shows an example of approximately 50% top flange damage.

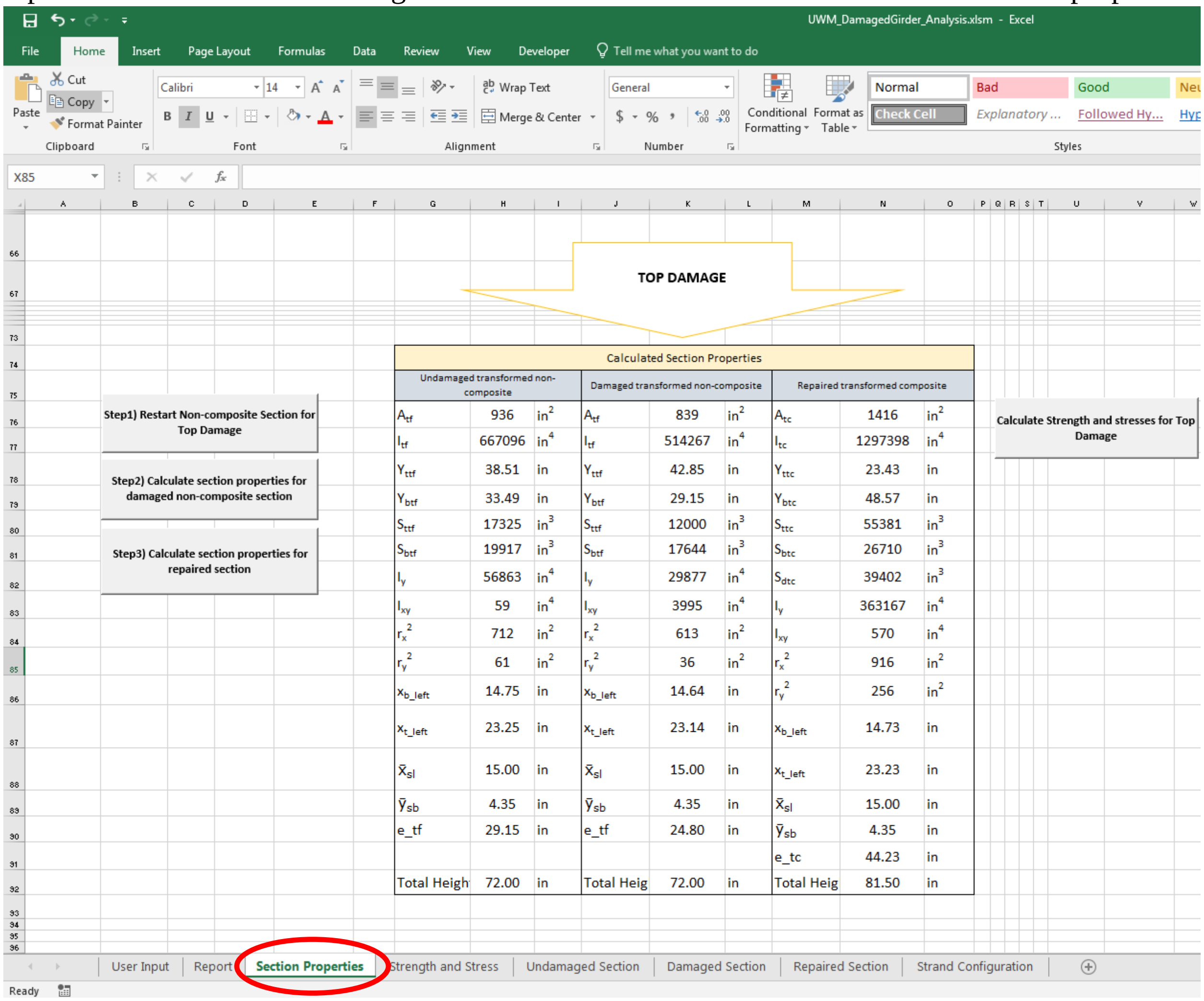


Step 3. Calculate section properties for repaired section. Repaired section is a composite section with slab applied on the girder (with or without repair).

A message box regarding repair information would be popped up at this step.

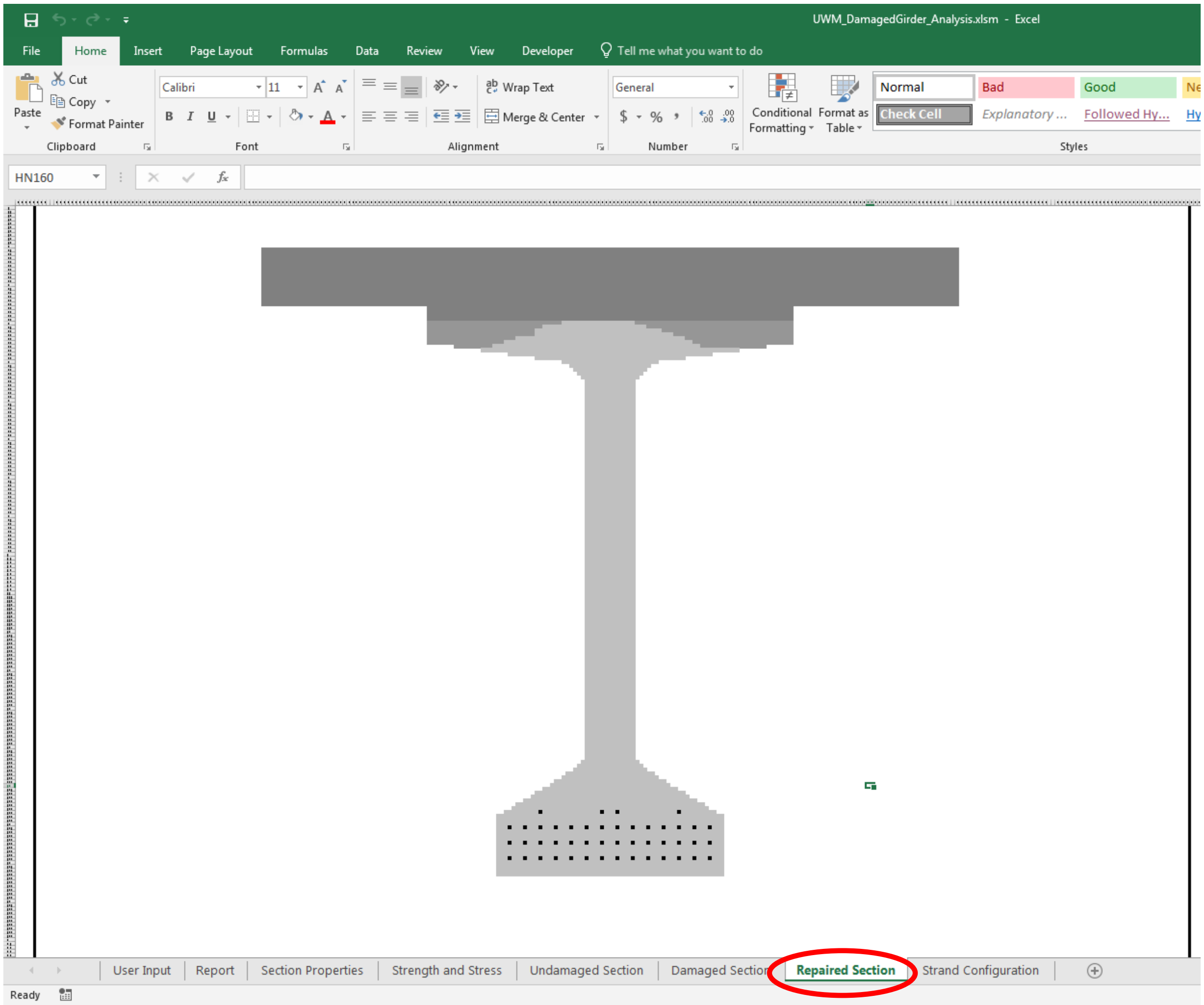


Section properties for undamaged damaged, and repaired section is now printed in a table for top damage section properties.

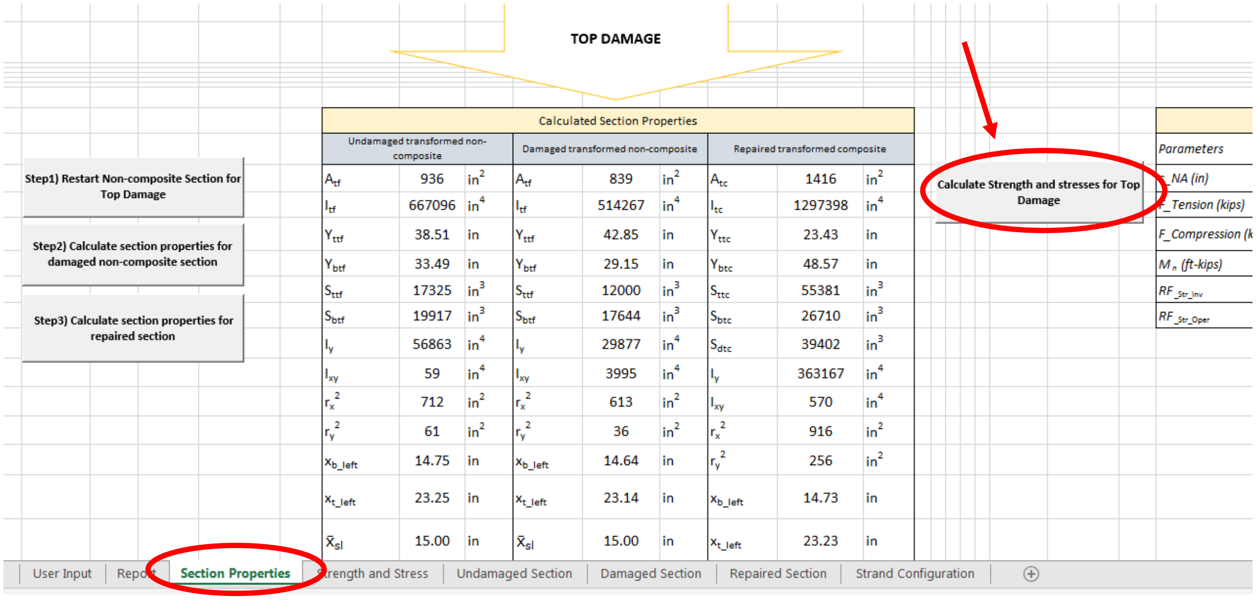


Calculated Section Properties								
Undamaged transformed non-composite			Damaged transformed non-composite			Repaired transformed composite		
A_{tf}	936	in^2	A_{tf}	839	in^2	A_{tc}	1416	in^2
I_{tf}	667096	in^4	I_{tf}	514267	in^4	I_{tc}	1297398	in^4
Y_{ttf}	38.51	in	Y_{ttf}	42.85	in	Y_{ttc}	23.43	in
Y_{btf}	33.49	in	Y_{btf}	29.15	in	Y_{btc}	48.57	in
S_{ttf}	17325	in^3	S_{ttf}	12000	in^3	S_{ttc}	55381	in^3
S_{btf}	19917	in^3	S_{btf}	17644	in^3	S_{btc}	26710	in^3
I_y	56863	in^4	I_y	29877	in^4	S_{dtc}	39402	in^3
I_{xy}	59	in^4	I_{xy}	3995	in^4	I_y	363167	in^4
r_x^2	712	in^2	r_x^2	613	in^2	I_{xy}	570	in^4
r_y^2	61	in^2	r_y^2	36	in^2	r_x^2	916	in^2
x_{b_left}	14.75	in	x_{b_left}	14.64	in	r_y^2	256	in^2
x_{t_left}	23.25	in	x_{t_left}	23.14	in	x_{b_left}	14.73	in
$\bar{x}_{sl}$	15.00	in	$\bar{x}_{sl}$	15.00	in	x_{t_left}	23.23	in
$\bar{y}_{sb}$	4.35	in	$\bar{y}_{sb}$	4.35	in	$\bar{x}_{sl}$	15.00	in
e_tf	29.15	in	e_tf	24.80	in	$\bar{y}_{sb}$	4.35	in
						e_tc	44.23	in
Total Heigh	72.00	in	Total Heig	72.00	in	Total Heig	81.50	in

The repaired section can be seen in the tab “Repaired Section”.



Strength and stress calculations for top damage can be done clicking on a button as shown below.



Strength and stress calculations would be calculated and reported in tables as shown below.

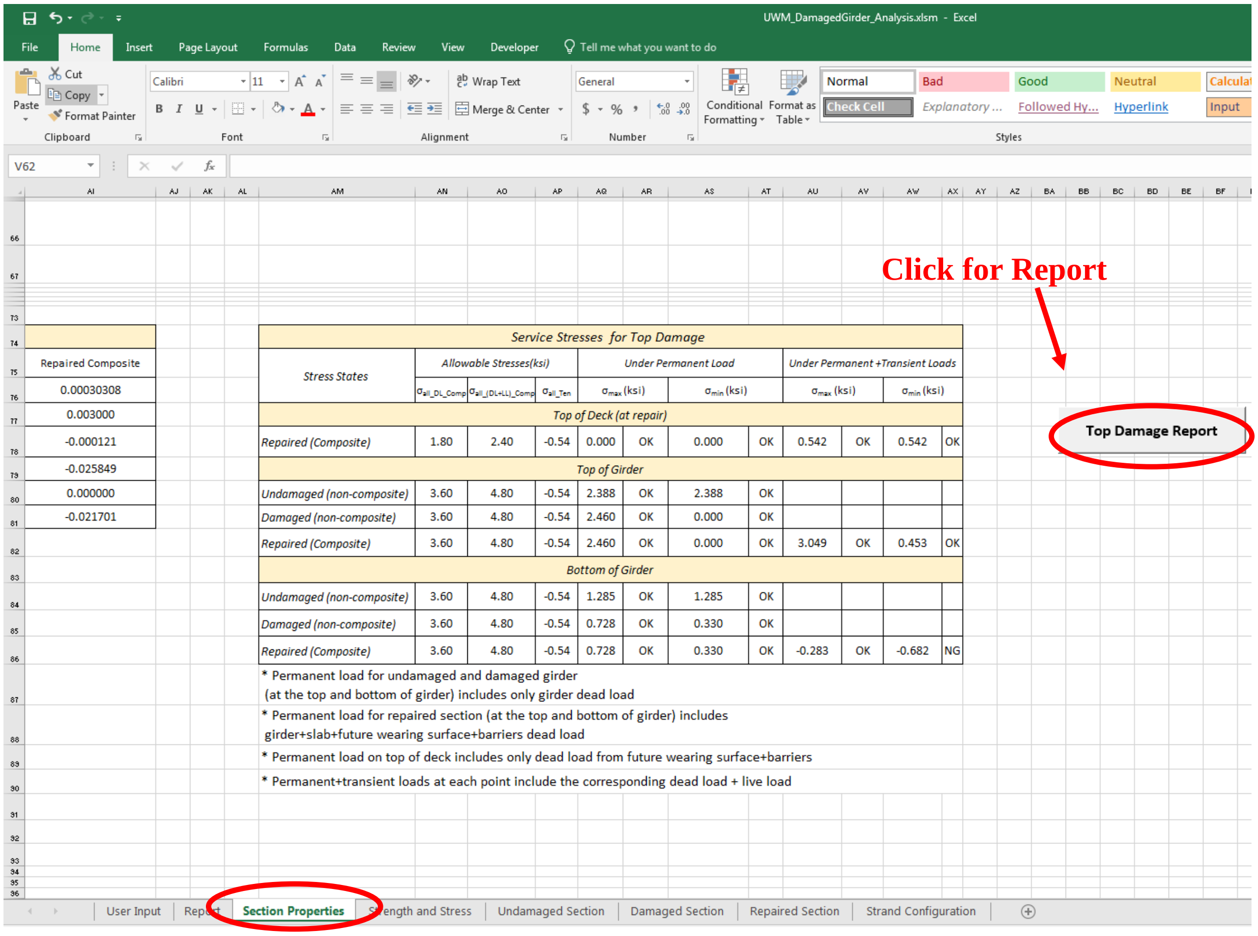
Top Damage Strength Calculations			
Parameters	Undamaged Non-composite	Damaged Non-composite	Repaired Composite
c_NA (in)	27.14	37.60	9.90
F_Tension (kips)	2519	2207	2646
F_Compression (kips)	2539	2241	2676
M_n (ft-kips)	12805	10110	16179
$RF_{\_Str\_Inv}$	2.43	1.75	1.73
$RF_{\_Str\_Oper}$	3.15	2.27	2.24

Curvature and Strains at Strength Limit States			
Strain and Curvature	Undamaged Non-composite	Damaged Non-composite	Repaired Composite
Curvature	0.00011052	0.00007980	0.00030308
Strain at the top of the deck			0.003000
Strain at the top of the girder	0.003000	0.000000	-0.000121
Strain at the centroid of strands (tension)	-0.009938	-0.005440	-0.025849
Strain at the bottom of FRP (tension)			0.000000
Strain at the bottom of the beam (tension)	-0.004958	0.000000	-0.021701

Service Stresses for Top Damage											
Stress States	*Allowable Stresses(ksi)*			*Under Permanent Load*				*Under Permanent +Transient Loads*			
	$\sigma_{all\_DL\_Comp}$	$\sigma_{all\_(DL+LL)\_Comp}$	σ_{all_Ten}	σ_{max} (ksi)		σ_{min} (ksi)		σ_{max} (ksi)		σ_{min} (ksi)	
Top of Deck (at repair)											
Repaired (Composite)	1.80	2.40	-0.54	0.000	OK	0.000	OK	0.542	OK	0.542	OK
Top of Girder											
Undamaged (non-composite)	3.60	4.80	-0.54	2.388	OK	2.388	OK				
Damaged (non-composite)	3.60	4.80	-0.54	2.460	OK	0.000	OK				
Repaired (Composite)	3.60	4.80	-0.54	2.460	OK	0.000	OK	3.049	OK	0.453	OK
Bottom of Girder											
Undamaged (non-composite)	3.60	4.80	-0.54	1.285	OK	1.285	OK				
Damaged (non-composite)	3.60	4.80	-0.54	0.728	OK	0.330	OK				
Repaired (Composite)	3.60	4.80	-0.54	0.728	OK	0.330	OK	-0.283	OK	-0.682	NG

* Permanent load for undamaged and damaged girder (at the top and bottom of girder) includes only girder dead load
* Permanent load for repaired section (at the top and bottom of girder) includes girder+slab+future wearing surface+barriers dead load
* Permanent load on top of deck includes only dead load from future wearing surface+barriers
* Permanent+transient loads at each point include the corresponding dead load + live load

A button is created for s summary of all calculations including load calculation and section properties for undamaged, damaged, and repaired sections copied to the “Report” sheet.



Repaired Composite
0.00030308
0.003000
-0.000121
-0.025849
0.000000
-0.021701

Service Stresses for Top Damage											
Stress States	Allowable Stresses(ksi)			Under Permanent Load				Under Permanent +Transient Loads			
	$\sigma_{all\_DL\_Comp}$	$\sigma_{all\_(DL+LL)\_Comp}$	σ_{all_Ten}	σ_{max} (ksi)		σ_{min} (ksi)		σ_{max} (ksi)		σ_{min} (ksi)	
Top of Deck (at repair)											
Repaired (Composite)	1.80	2.40	-0.54	0.000	OK	0.000	OK	0.542	OK	0.542	OK
Top of Girder											
Undamaged (non-composite)	3.60	4.80	-0.54	2.388	OK	2.388	OK				
Damaged (non-composite)	3.60	4.80	-0.54	2.460	OK	0.000	OK				
Repaired (Composite)	3.60	4.80	-0.54	2.460	OK	0.000	OK	3.049	OK	0.453	OK
Bottom of Girder											
Undamaged (non-composite)	3.60	4.80	-0.54	1.285	OK	1.285	OK				
Damaged (non-composite)	3.60	4.80	-0.54	0.728	OK	0.330	OK				
Repaired (Composite)	3.60	4.80	-0.54	0.728	OK	0.330	OK	-0.283	OK	-0.682	NG

* Permanent load for undamaged and damaged girder (at the top and bottom of girder) includes only girder dead load
* Permanent load for repaired section (at the top and bottom of girder) includes girder+slab+future wearing surface+barriers dead load
* Permanent load on top of deck includes only dead load from future wearing surface+barriers
* Permanent+transient loads at each point include the corresponding dead load + live load