Optimizing Concrete Deck Removal in Concrete I-Girder Bridges

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**16. Abstract**

Wide flange precast/prestressed concrete I-girders have been widely used by several State Departments of Transportation (DOTs) in the last two decades. These girders have many advantages over standard AASHTO I-girders. Their wide and thick bottom flange accommodates a large number of prestressing strands and their wide and thin top flange provides a shorter deck span, reduced girder weight, greater stability in construction, and adequate platform for workers. Despite these advantages, the wide and thin top flange might be disadvantageous when it comes to deck removal, as it is more susceptible to damage. Therefore there is a need to investigate the impact of deck removal methods on the performance of the supporting wide flange I-girder. In this study, two deck removal methods are presented: saw cutting and jackhammering. These two methods were implemented on the Camp Creek Bridge over I-80 in Lancaster County, NE before demolition due to its functional obsolescence. Different saw cutting and jackhammering techniques were performed for deck removal between girders and on top of girders. Data obtained from using similar techniques on three other projects were collected and analyzed. Two girders from the Camp Creek Bridge were taken to the lab for testing in flexure after applying different levels of deck removals around shear connectors and re-decking. Test results indicated adequate performance of the new composite section even when partial deck removal around shear connectors is applied. Another investigation was conducted to evaluate the effect of top flange width on the performance of bridge I-girders. The top flange was longitudinally saw cut and its width was reduced by fifty percent. The effects on geometrical properties, flexural capacity, horizontal shear capacity, and deflection were investigated analytically and experimentally under construction loads and service loads. Investigation results indicate that in some cases top flange width does not have significant impact on the structural performance of I-girders.

**17. Key Words**

Add key words here

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Disclaimer

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Abstract

Wide flange precast/prestressed concrete I-girders have been widely used by several State Departments of Transportation (DOTs) in the last two decades. These girders have many advantages over standard AASHTO I-girders. Their wide and thick bottom flange accommodates a large number of prestressing strands and their wide and thin top flange provides a shorter deck span, reduced girder weight, greater stability in construction, and adequate platform for workers. Despite these advantages, the wide and thin top flange might be disadvantageous when it comes to deck removal, as it is more susceptible to damage. Therefore there is a need to investigate the impact of deck removal methods on the performance of the supporting wide flange I-girder.

In this study, two deck removal methods are presented: saw cutting and jackhammering. These two methods were implemented on the Camp Creek Bridge over I-80 in Lancaster County, NE before demolition due to its functional obsolesces. Different saw cutting and jackhammering techniques were performed for deck removal between girders and on top of girders. Data obtained from using similar techniques on three other projects were collected and analyzed. Two girders from the Camp Creek Bridge were taken to the lab for testing in flexure after applying different levels of deck removal around shear connectors and re-decking. Test results indicated adequate performance of the new composite section even when partial deck removal around shear connectors is applied.

Another investigation was conducted to evaluate the effect of top flange width on the performance of bridge I-girders. The top flange was longitudinally saw cut and its width reduced by fifty percent. The effects on geometrical properties, flexural capacity, horizontal shear capacity, and deflection were investigated analytically and experimentally under construction loads and
service loads. Investigation results indicate that in some cases top flange width does not have significant impact on the structural performance of I-girders.
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Chapter 1. Introduction

1.1 Background

With the evolution of precast/prestressed concrete bridge I-girders comes greater structural capacity and ability to span lengths of up to 200 ft. Figure 1.1 shows the evolution of cross sections of typical concrete bridge I-girders from the standard AASHTO girders to PCI Bulb Tee girders, and recently to wide and thin top flange I-girders (e.g. NU girders). Precast/prestressed concrete I-girders with wide and thin top flanges have unique characteristics compared to the other concrete girders. The wide and thin top flange provides an adequate platform for workers, a shorter deck span, and reduced girder weight. While the wide and thick bottom flange accommodates a high amount prestressing to improve the section capacity, the wide and thin top flange improves girder stability during construction and reduces the tendency to side sway when long spans are used.

Figure 1.1 Cross sections of standard AASHTO girders (left) and NU girders (right)
NU girders are one of the early examples of an I-girder with wide and thin top flange. These girders were developed in the mid-1990s and have been extensively used since then. Although the examples presented in this report use NU girders, all deck removal methods, conclusions, and recommendations apply to other concrete I-girders with wide and thin top flange.

1.2 Problem Statement

Despite the advantages of concrete I-girders with wide and thin top flange, several challenges could arise during deck removal operations since the top flange is more susceptible to damage in I-girders than conventional AASHTO and bulb tee girders. There are no guidelines, specifications, or deck removal experience for this generation of I-girders. Therefore, there is a need to investigate different deck removal methods and evaluate their impact on girder condition and performance. Furthermore, there is a lack of research on the efficiency and cost effectiveness of different deck removal methods as well as their impact on the environment.

1.3 Objectives

The objective of this project is to investigate different deck removal methods and their impact on the structural performance of precast/prestressed concrete I-girders with wide and thin top flange. More specifically, different saw cutting and jackhammering techniques are investigated in terms of the resulting damage to the girder, duration, cost, and impact on the environment.

1.4 Report Organization

This report is organized into six chapters as follows:

Chapter 1 presents background information, a problem statement, research objectives, and the report’s organization.

Chapter 2 reviews the literature on existing deck removal methods and the most common practices currently used by state DOTs.
Chapter 3 presents the findings of the field investigation performed on the Camp Creek Bridge.

Chapter 4 gives a brief introduction to the cost analysis of deck removal techniques.

Chapter 5 presents the analytical investigation performed. A proposed deck removal method is analyzed for two bridge examples.

Chapter 6 shows the experimental investigation and validation of the analytical work. The specimen preparation, testing, and test results for the proposed method are presented as well.
Chapter 2. Literature Review

2.1 Publications

NCHRP Report 407 discusses the rapid replacement of bridge decks and states that methods for deck replacement do not affect only the duration and the cost of the project, but also the performance of the supporting structure. Equipment that can be used to remove an old deck includes pneumatic breakers, saws, drills, breakers, splitters, crushers, and blasting charges. The main limitations are the accessibility of the elements to be removed, removal time frame, and environmental and noise restrictions. The improper application of the aforementioned equipment can result in some damage that affects the performance of the structure (Tadros & Baishya, 1998).

One method of deck removal is saw-cutting the deck into small pieces that are manageable to lift and transport. Micro-cracking in the girder’s top surface was observed when pneumatic hammers were used. Damage to the top flange can be extensive when rig-mounted breakers, wrecking balls, and blasting charges are used. New techniques, such as chemical splitters and cutters, have been used infrequently (Tadros & Baishya, 1998).

The province of Alberta in Canada has specifications for bridge construction: jackhammers heavier than 14 kg (30 lb) and chipping hammers heavier than 7 kg (15 lb) are not allowed to be used for full depth repair of bridge decks (Alberta Ministry of Transportation, 2010).

2.2 Surveys

2.2.1 Nebraska Department of Roads (NDOR) Survey

A questionnaire was sent to the state DOTs in order to investigate all the possible methods according to the DOTs’ experience. Most of the DOTs’ practices were saw cutting between the girders, followed by picking the deck and jackhammering on top of the girders to remove the remaining part of the deck. Hydro-demolition was suggested by many states, however, with this
method, it can be challenging to control the water with the concrete according to EPA requirements. A list of the 10 DOTs that responded to the survey and their responses are shown in Table 2.1.

Of the ten states that responded to the survey, there were four states that practice hydro-demolition. The response from these four states was that hydro-demolition is a noisy and costly removal method with environmental control issues, but has low risk of damage. The state of Florida mentioned that if labor cost is low, jackhammering is used, and if labor cost is high, hydro-demolition is preferred. The responses gathered revealed that all states practice conventional saw cutting and jackhammering practices.

The use of pneumatic hammers attached to mini-excavators or a backhoe is a practice used by many states for the first half depth of the bridge deck. The use of pneumatic hammers is more economical but risky because the operators need to be very careful to avoid damaging the girder top flange. The remaining concrete down to the girder top flange is removed using hand chippers and small jackhammers. Contractors typically attempt to bid this method first, such as in the state of Pennsylvania, rather than hand remove the full depth of the deck. The cost is reduced by almost 33% when pneumatic hammers are used. The cost of removal with a combination of pneumatic hammering and hand chipping is around $600-$700/c.y., whereas the cost of using only hand chipping is $900-$1000/c.y.

The DOT’s response to the survey also said that the de-bonded strip at the top flange edge is a good starting place for longitudinal saw cutting and easy lifting of deck panels. Florida’s DOT suggested vertical saw cutting 2 in. inside the top flange and lifting deck panels with a crane. The Florida DOT also recommends slope saw cutting longitudinally at the flange edge so the deck wedges itself after cutting until it is lifted out.
Table 2.1 DOTs response to the survey (1/2)

<table>
<thead>
<tr>
<th>No.</th>
<th>State</th>
<th>Contact</th>
<th>Experience</th>
<th>Methods Used/Recommended for Deck Removal on Bulb-Tee Girders</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Indiana</td>
<td>James Colonies (317) 467-3964</td>
<td>Yes</td>
<td>Hydro-demolition</td>
<td>Fast, noisy, and costly because of water control</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Small jackhammers</td>
<td>Slow (1 cft/hr), less noisy, and economical</td>
</tr>
<tr>
<td>2</td>
<td>Minnesota</td>
<td>Paul Rowekamp (651) 366-4484</td>
<td>Yes</td>
<td>Saw cutting removal of deck sections between beams</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Hand held cutting and jack hammer removal above the top flange</td>
<td>Less probable damage and slow but can be easier when good access is provided (false floor on bottom flange)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Small track mounted pneumatic hammer above the top flange</td>
<td>Faster but had more top flange damage than jackhammer</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Steel trowel finish and 6” bond breaker are applied to the newly developed I-beam that has 4 ft wide top flange.</td>
<td>None</td>
</tr>
<tr>
<td>3</td>
<td>New Mexico</td>
<td>Ray M. Trujillo <a href="mailto:raymond.trujillo@state.nm.us">raymond.trujillo@state.nm.us</a></td>
<td>Yes, BT-54 girders</td>
<td>Backhoe with a pneumatic hammer</td>
<td>Break some of the top flange.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Saw cutting a few inches beyond the edge of the top flanges, then, use chipping hammer to remove the deck above the top flanges</td>
<td>Care needs to be given as the deck removal can break off the thin flanges fairly easy.</td>
</tr>
<tr>
<td>4</td>
<td>Pennsylvania</td>
<td>Tom Macioce (717)787-2881</td>
<td>Not Specified</td>
<td>Saw cut deck with diamond saw at approximately 10 ft intervals transversely. Plunge cuts through parapets at same intervals. Break concrete over beam stirrups using mini-excavator with a small hydraulic hammer for half depth and chipping hammers for the remainder. Remove slab using a Gradall excavator with a slab grab bucket. The same procedure is applied to parapets but they need to be lifted with cables.</td>
<td>More economical but risky. Operations need to be watched closely to ensure that SIP pan clips are not damaging the flanges when pulled out</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Saw cut the deck and parapet as in the previous method. Machine break and then hand demolition over the entire width of the beam. Leave slabs hanging from some rebar. Torch pan angle welds. Engage slab grab bucket and cut remaining bars. Only chip and free enough length to stay within the lift capacity of the excavator.</td>
<td>Most contractors bid this method and try the first one. Hand chipping over the entire beam top is very expensive ($900-$1000 /cy). Combination of machine and hand is probably ($600-$700/cy). Hammering is very noisy.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>First method is used if slabs can be pulled free from SIP clips. Second method is used if slab pans are not pulling free.</td>
<td>Safety is an issue. Longitudinal fall protection will need to be installed.</td>
</tr>
</tbody>
</table>
### Table 2.1 DOTs response to the survey (2/2)

<table>
<thead>
<tr>
<th>No.</th>
<th>State</th>
<th>Contact</th>
<th>Experience</th>
<th>Methods Used/Recommended for Deck Removal on Bulb-Tee Girders</th>
<th>Results</th>
</tr>
</thead>
</table>
| 5   | Texas     | Kevin Pruski (512) 416-2306 | Yes, Not Bulb Tee Girders | Conventional jackhammer methods  
Hydro-demolition with controlling the depth of removal | Contractor had to repair beam top flange in many locations.               |
| 6   | Oregon    | Crain Shike (503) 986-3323 | No         | No methods are recommended at the meantime  
Debonding 8” wide strips at the top flange is a good start | None                                                                     |
| 7   | Virginia  | Julius F. J. Volgyi (804) 786-7537 | No         | Saw cut between girders and remove deck sections by crane  
Hydro-blasting of concrete over top flanges to below top layer of deck reinforcement and 1’ strips from edge of top flange to top of top flange of girder  
Small pneumatic hammers (15-20 lbs.) for removal of deck concrete below top reinforcement in the 2 ft wide center strip | None                                                                     |
| 8   | Florida   | Steven Plotkin (904) 360-5501 | Yes        | Concrete over beam flanges is removed using small jack hammers or hydroblasting depending on the cost. Hydroblasting can be controlled in a way that gouging the top flange is not a problem.  
Deck between beams is removed by either vertical saw cutting 10 ft sections 2 in. inside the top flange and lifting with a crane, or sloped saw cutting over the beam flange tip so the deck wedges itself after cutting until it is lifted out. | If labor cost is low, jack hammer is used. If labor cost is high, hydroblasting is preferred. They both work well.  
The bonding action over the 2 in. strip occasionally produce minor spalls on the beam flange when vertical saw cutting is used. |
| 9   | California| Susan E. Hida (916) 227-8738 | No         | Recommend full depth saw-cutting outside the limits of the top flange and high pressure water blasting to remove the concrete deck inside the limits of the top flange to prevent damage to the pre-cast bulb-tee girders. | None                                                                     |
| 10  | Missouri  | Gregory E. Sanders (573) 526-0245 | No         | Superstructure removal may be more feasible and economical option. Debonding more of the top flange will certainly help in deck removal. | None                                                                     |
2.2.2 Iowa State University (ISU) Survey

A national survey was conducted by the Iowa State University Bridge Center, and a total of 28 states responded on the methods they practice for concrete and steel bridge deck removal. The criteria by which methods were evaluated were performance, time, cost, noise, and safety. The results of the survey are summarized in this section.

Table 2.2 shows deck removal methods currently used by the 28 states that responded to the national survey. A description of tools used in each method is given.

### Table 2.2 Equipment and tools used for different methods

<table>
<thead>
<tr>
<th>Method</th>
<th>Equipment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sawing</td>
<td>Blade saws, Diamond wire saws</td>
</tr>
<tr>
<td>Drilling</td>
<td>Core drills</td>
</tr>
<tr>
<td>Breaking</td>
<td>Machine-mounted hydraulic breakers, Rig-mounted pneumatic breakers, Whihammers</td>
</tr>
<tr>
<td>Splitting</td>
<td>Mechanical splitters, Chemical Splitters</td>
</tr>
<tr>
<td>Crushing</td>
<td>Rig-mounted shears</td>
</tr>
<tr>
<td>Hydrominolition</td>
<td>High-pressure water jets</td>
</tr>
<tr>
<td>Ball and Crane</td>
<td>Wrecking ball attached to a crane</td>
</tr>
<tr>
<td>Blasting</td>
<td>Explosives (dynamite, ANFO)</td>
</tr>
</tbody>
</table>

For deck removal and the re-use of girders, three methods are considered: saw cutting, breaking, and hydro-demolition. Table 2.3 gives a generic comparison of these three methods for the criteria mentioned. Although hydro-demolition has a low risk of damaging the girders, it ranks as more costly than other methods and more dangerous for the operator. Saw cutting and jackhammering are more cost effective, but can also result in more damage to the girders.
2.3 ISU Research

The ISU Bridge Center has conducted research on the shear capacity of three different types of shear connectors with varying levels of deck removal. Three different types of shear connectors welded to I-beams were tested for shear capacity and behavior of the connection with the testing variable of 50%, 75%, or 100% of concrete removed. The three different types of shear connectors are standard shear studs, a c-channel connector, and the angle with a welded bar connector. The testing consisted of 27 specimens; three specimens for every variation of concrete removal and type of shear connector. The test setup is shown in Figure 2.1.

<table>
<thead>
<tr>
<th></th>
<th>Saw Cutting</th>
<th>Breakers (Jackhammering)</th>
<th>Hydrodemolition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost</td>
<td>Moderate</td>
<td>Moderate to Low</td>
<td>High</td>
</tr>
<tr>
<td>Duration</td>
<td>Moderate to Low</td>
<td>Moderate to Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>Safety</td>
<td>Moderate to High</td>
<td>Moderate to High</td>
<td>Moderate</td>
</tr>
<tr>
<td>Noise</td>
<td>Moderate</td>
<td>High</td>
<td>High</td>
</tr>
<tr>
<td>Risk of Damage to Steel Girders</td>
<td>Moderate to High</td>
<td>High</td>
<td>None to Low</td>
</tr>
<tr>
<td>Risk of Damage to AASHTO Girders</td>
<td>Moderate to Low</td>
<td>Moderate to Low</td>
<td>Low</td>
</tr>
<tr>
<td>Risk of Damage to Bulb-T Girders</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Low</td>
</tr>
</tbody>
</table>
It should be noted that no specific height and width dimensions of the concrete around the connector were used to classify 50%, 75%, or 100% concrete removal, but was instead classified by weight. **Figure 2.2** shows the different types of shear connectors used in testing.
Figure 2.2 Angle + bar, c-channel, and shear stud connectors (Courtesy of ISU Bridge Center)

Specimen forms were made by casting the “new” deck around the shear connectors with existing concrete on shown in Figure 2.3 and Figure 2.4.
Figure 2.3 Specimen forming (Courtesy of ISU Bridge Center)

Figure 2.4 ISU push-off test setup (Courtesy of ISU Bridge Center)

Specimen failure mode is shown in Figure 2.5. All of the shear connector types had the same resultant failure mode, which is shearing off the connector at the deck to girder interface.
The results of testing the different connectors with varying concrete deck removal levels of 50%, 75%, and 100% are shown in Figure 2.6 through Figure 2.8. The graphs show no correlation between the level of deck removal and the behavior of the connection. Therefore, it can be concluded that the amount of concrete removal around the shear connectors does not adversely affect the behavior of the connection.
Figure 2.7 Load vs average displacement for c-channel (Courtesy of ISU Bridge Center)

Figure 2.8 Load vs average displacement for angle + bar (Courtesy of ISU Bridge Center)
2.4 Workshops

The University of Nebraska-Lincoln (UNL) hosted a workshop on concrete deck removal methods for concrete I-girders on November 16, 2012. Bridge contractors, owners, and researchers discussed effective deck removal methods, procedures, and future tasks in this research project.

2.4.1 Deck Removal between Girders

The methods for deck removal between girders are determined by environmental restrictions. The most cost effective method is breaking the deck panels down to the ground after saw cutting using a hydraulic hammer mounted on backhoe. However, this method is not permitted when there is an underlying waterway, highway, or railroad. If there are environmental restrictions, transverse and longitudinal saw cutting followed by lifting deck panels with a crane or slab crab will be used. Concrete deck panels are usually 6’ x 12’ in dimension.

2.4.2 Deck Removal On Top of Girders

The use of hydro-demolition, hand-operated jackhammering, and small impact jackhammers mounted on excavators are recommended for deck removal. Since there are different methods available for removing the deck on top of girders, both efficiency and cost need to be investigated.

2.4.3 Proposed Methods for Research

Four methods were proposed for removing the deck on top of the girders. These methods include: (1) sloped saw cutting part of the top flange then forming a new deck; (2) milling part of the old deck down to shear connectors and pouring a new deck on top; (3) vertical saw cutting down to girder flange and jackhammering the concrete around the shear connectors; and (4) saw cutting deck just outside the shear connectors, followed by milling the old deck down to shear connectors and pouring new deck. Cost analyses of these methods need to be conducted, as well
as the cost for replacing the entire superstructure (girders and deck) versus removing just the deck. In some cases, the cost of precast/prestressed bridge girders per square foot can be close to the cost of deck removal.

Method 1 - Sloped Saw Cut Top Flange

A saw-cut machine with a blade that could pivot at a certain angle is needed to perform a sloped cut without the costly and time-consuming operation of using the guided rail with wall saws. In this case, using the sloped saw to cut through the top flange can be a good alternative if the girder retains its structural capacity and stability when the top flange width is reduced. **Figure 2.9** shows a sketch of this alternative in which the shaded area is jackhammered and the new deck is formed similar to the formation of decks on steel girders. The new deck can have a haunch to provide adequate cover for the exposed steel in the girder top flange. This alternative does not require the debonded zone, but the ability of cut deck panel to carry the weight of construction equipment needs to be investigated.

**Figure 2.9** Sloped saw cut alternative
Method 2- New Deck On Top of Old Deck

Another alternative is shown in Figure 2.10. Mill the top 2-3 in. of the deck over the girder, cut and lift deck panels between girders, keep the old deck around the shear connectors, pour the new deck on top of it, and connect old and new deck to achieve composite action (using new connectors on the top or the side of the old deck). This solution will result in about a 5 in. increase in deck elevation.

Figure 2.10 Alternative method of pouring new deck on top of old deck

Method 3- Vertical Saw Cut at Deboned Zone

A third alternative is shown in Figure 2.11. The method requires saw cutting the deck panels vertically at the debonded zone, using a mini-excavator to break the concrete above the girder, and using a heavy excavator to break the deck between girders. Avoid 15-kip and 30-kip jackhammers because using these small jackhammers is very time consuming and costly.
Method 4- Vertical Saw Cut outside Shear Connectors

A fourth alternative is shown in Figure 2.12. Saw cut deck transversely and longitudinally around shear connectors. Grind the top 2-3 in. of the deck over the shear connectors (highway grinder was suggested as a way of milling that 2-3 in.). Remove the remaining concrete around the shear connectors (sing small jackhammers or manual hydro-blasting. Finally, lift (pop) the slabs/panels between the girders, which should easily break the bonded area.
Figure 2.12 Alternative method vertical saw cut outside shear connectors

2.4.4 Effective Sequencing of Tasks

To minimize cost and unnecessary movements, each sequence should be planned. The amount of manual work done and idle time of equipment should be minimized. Also, saw cutting, jackhammering, and panel lifting should be sequenced so that lifting equipment will be supported on deck panels that are not yet cut and jackhammering is done before lifting adjacent panels. The recommended sequence of deck removal tasks discussed in the workshop include:

1. Saw-cut deck transversely for the full width every 10-12 ft
2. Saw-cut deck longitudinally at the debonded zone over the girder lines
3. Jackhammer/hydro-blast on top of the two girder lines
4. Lift panels using crane or hydraulic backhoe to take away deck in between girders
5. Repeat tasks 1-4 for the following girder lines
6. For the last two girders, cut, jackhammer, and lift panels section by section
Chapter 3. Field Investigations

3.1 Camp Creek Bridge

The purpose of this investigation is to experimentally evaluate the effectiveness and efficiency of different deck removal methods and their impact on the supporting girders. For deck removal between girders, three main methods were attempted using different locations for longitudinal saw cutting. For removal on top of girders, three methods were also attempted with different combinations of saw cutting and jackhammering.

Figure 3.1 shows the sectional elevation, plan, and cross section of the Camp Creek Bridge over I-80 in Lancaster County, NE. The bridge is a 170 ft long, 42 ft wide, three span (52.5-65-52.5 ft) bridge that has four NU1100 girders per span. The bridge was built in 1996 and is being demolished after only 15 years due to its functional obsolesce. This bridge is considered one of the early bridges built with precast/prestressed NU girders. It is also the first bridge with NU girder to have its deck removed. Figure 3.2 is a chart that depicts deck removal methods implemented on the Camp Creek Bridge.
Figure 3.1 Elevation and cross section views of the Camp Creek Bridge

Figure 3.2 Organizational chart of the methods implemented for deck removal
3.1.1 Evaluation of Removal Methods: Between the Girders

This procedure involved saw cutting the deck transversally into six 8-ft long panels and cutting three different longitudinal saw cuts as shown in Figure 3.3. Below lists the three different methods used for the longitudinal saw cuts:

1. Saw cutting the deck 6 in. from the edge of the top flange of the girder towards the inside of the girder, which is close to the end of the debonded zone.
2. Saw cutting the deck 2 in. from the edge of the top flange of the girder towards the inside of the girder, which is the standard practice used in conventional bridge girders.
3. Saw cutting the deck at the edge of the top flange with a 60° angle to simplify panel lifting after saw cutting.

Figure 3.3 Implemented methods between girders

Figure 3.4 and Figure 3.5 give the panel number that corresponds to the method attempted on the panel. Two panels were saw cut and lifted for each method.
Figure 3.4 Plan view of the methods implemented for deck removal in between girders

Figure 3.5 The deck while saw cutting, showing the panel numbers compared to the plan view of the proposed methods

3.1.1.1 Method 1 and 2: Vertical Cut Panels

Method 1 includes cutting panels #1 and #2 at 6 in. from the edge of the girders, while method 2 includes cutting panels #3 and #4 at 2 in. from the edge of the girder. All panels were transversely saw cut for their full depth (8 in.) at 8 ft spacing. The haunch was 1 in. at the ends of the girders, causing for a deck depth of 9 in. at these locations. All cuts were located at the debonded zone of the girder top flange.
First, 14 in. diameter blades were used for two passes to create 4-4.5 in. deep cuts. Second, 18 in. diameter blades were used for one pass to create 6-6.5 in. deep cut. Last, 24 in. diameter blades were used to create 7.5 – 8 in. deep cuts. Figure 3.6 shows the three blade sizes used for saw cutting. Each pass took about 1 minute to cut 8 ft. Three 1/8 in. blades were used in each cut, making for a 3/8 in. wide cut to simplify panel lifting.

Figure 3.6 (From left to right) 14 in. diameter, 18 in. diameter, and 24 in. diameter blades

Two brackets were anchored at the centerline of the panel 1 ft away from the panel edges. Panels were lifted from one bracket first to break the bond between the panel and the deck, and then the two brackets were used to lift the panel completely (Figure 3.7). The two panels with 2 in. overlap and the first panel with 6 in. overlap were easily lifted.
The second 6 in. overlap panel caused difficulties when the crew was performing the first lift to break the bond between the panel and the remaining part of the deck. There was difficulty because the haunch was deeper at that part of the bridge. The lifted edge was hammered extensively on both sides; however, it could not separate the panel from the deck. A hammer and a chisel were used to break the haunch from the rest of the deck (Figure 3.8). Since the chisel could not go deep enough into the concrete due to the thicker haunch, a 60 lb jackhammer was used to break the deck attached to the haunch (Figure 3.9). The crane was lifting the edge of the panel while the workers were using jackhammers on the panel, which caused the bolts holding the bracket to the concrete to slip out of the panel, resulting in the location of the bracket needing to be changed. The panel required a lot of wiggling until it was completely lifted. Despite the rough actions on the deck, the flange did not show any signs of cracks or damage.
Figure 3.8 Breaking panel from deck using a hammer and a chisel

Figure 3.9 Jackhammering the deck attached to the panel haunch
3.1.1.2 Method 3: Sloped Cut Panels

Panels #5 and #6 were longitudinally saw cut at a 60° slope at the edge of the top flange. For sloped cuts, a single 24 in. diameter blade was used to create cuts 6 in. deep in two passes, then a single 30 in. diameter blade was used to complete the full cut in one pass. This procedure took about 20 minutes for an 8 ft long cut (Figure 3.10). Another method was attempted to save the time it took to change the blades: using a 30 in. diameter blade to make the full depth cut in three passes. Even though the cutting process is easier, the process of installing the frame for the blade and anchoring it to the deck was time consuming, especially with the frame extending a maximum of only 10 ft, so for any extra length, the frame would need to be removed and re-anchored in the new location.

Figure 3.10 The two 24 in. and 30 in. diameter blades used in the sloped cuts (left) and a side view of the sloped blade mounting (right)
The two sloped saw cut panels were lifted first with no problems (Figure 3.11). Lifting those two panels was determined to easiest and fastest due to the sloped cuts, however, the saw cutting required more time. The panels were lifted without causing any damage to the girder.

![Figure 3.11 Lifting the sloped cut panel](image)

3.1.1.3 Deck Peeling

The overhang deck in Figure 3.12 was peeled from the supporting girder using the 8-in. wide debonded strip at the edges of the top flange. A CAT 330DL excavator with a 40 kip capacity was used to lift the edge of the overhang deck. Despite the powerful shaking of the deck, the girder and the deck stayed connected. It was suggested that the edge of the deck be pushed down to cause tension at the top of the deck and crack it. The deck cracked when it was pushed down, but the crack did not go deeper than the location of the top reinforcement mat. Unlike the rest of the overhang, the west edge was saw cut longitudinally; as a result, the deck was broken off of the flange at the saw cut line when it was pulled up. The side of the deck was broken, but the steel reinforcement did not break (Figure 3.13).
Figure 3.12 The CAT 330DL excavator peeling the overhang deck

Figure 3.13 Part of the deck that was peeled off

The deck was then jackhammered transversely to form 6 ft long panels, which make it easier to peel up the deck (Figure 3.14). However, the edge of the flange broke when the deck was jackhammered. When the panel was lifted by the excavator, the edge of the flange continued to
break and the process was stopped. Deck peeling proved to be a vigorous, inefficient, and damaging method.

![Figure 3.14 Jackhammering the deck using backhoe](image)

3.1.2 Evaluation of Removal Methods: On Top of the Girders

Referring to the chart in Figure 3.2, three methods were used to evaluate the most efficient way to jackhammer the deck on top of the girders. The plan view of the methods implemented is shown in Figure 3.15. Three different combinations of longitudinal saw cutting and jackhammering were attempted and labeled JH1, JH2, and JH3. The rate of removal was recorded and presented for each method.
3.1.2.1 Method 1 (JH1)

The first method was to saw cut the deck around the shear reinforcement, forming a 14 in. wide by a 5 ft long rectangle for the full depth of the deck (Figure 3.16). To jackhammer this strip, a 30 lb jackhammer was used down to the top reinforcing mat, then a 15 lb jackhammer was used down to the top of the girder. This process took two hours for a two-man crew to finish. The rate of deck removal for this method is 0.343 m/hr/ft².
3.1.2.2 Method 2 (JH2)

In the second method, a 4 ft 10 in. long by 3 ft 2 in. wide rectangle was saw cut then jackhammered (Figure 3.17). Jackhammering was performed using 60 lb and 30 lb jackhammers, which are heavier than the specified 30 lb and 15 lb jackhammers for deck removal on top of the girders. The 60 lb jackhammer was used down to the bottom reinforcing steel mat, then the 30 lb jackhammer was used down to the top flange of the girder. It took four hours for the two-man crew to finish the jackhammering process. The girder was slightly damaged when the 30 lb jackhammer slipped off and hit the top of the girder. The rate of deck removal for this method is 0.2613 mhr/ft².

**Figure 3.16** The 14” wide x 5” long full depth jackhammered strip
3.1.2.3 Method 3 (JH3)

The third method was to cut a 4 ft 10 in. long by 3 ft ½ in. wide rectangle and remove the concrete using 30 lb and 60 lb jackhammers. This method had additional saw cuts. Saw cuts were made directly outside the shear connectors and away from the shear connectors (Figure 3.18). This method was significantly more efficient because the broken deck came out in bigger chunks, making them require less time and effort to remove. The two-man crew needed 1 hour and 38 minutes to finish. The girder had only one location where the jackhammer hit the top surface and caused a 5-½ in. x 2-½ in. piece about ½ in. deep to be chipped off the girder. The rate of deck removal for this method is 0.111 mhr/ft².
Based on the observations made in this investigation, the following lessons were learned:

- For wide and thin top flange I-girders, longitudinal saw cuts can be made 6 in. from the top flange edge and achieve the same efficiency in lifting deck panels as with the panels that have a 2 in. overhang. With the saw cuts further inside the de-bonded zone, but not passed the de-bonded zone, a larger portion of deck will be lifted away, leaving less concrete on top of the girders to jackhammer. If possible, multiple blades should be used side-by-side to create a wider vertical saw cut for ease of lifting deck panels.

- With current equipment and tools used for sloped saw cutting, the process of having to re-assemble the frame proved to be very inefficient. Although lifting deck panels was very easy, this method should not be considered. Deck peeling also should not be considered because of the high risk of damaging girder top flanges.

- For deck removal on top of girders, longitudinal saw cuts proved to be much more efficient because they allowed concrete to be broken in larger chunks. With more longitudinal and transverse saw cutting, the time it takes to jackhammer is reduced.
3.2 Chappell Bridge

The purpose of this investigation was to evaluate the ease of performing deck removal methods on a bridge with stay-in-place forms that need to be re-used for the new deck. Most bridge decks in Nebraska are currently built using stay-in-place forms. Observations are made of the ease of deck removal between girders and damage to stay-in-place forms.

Constructed in 1969, the Chappell Bridge interchange re-decking operations started after 44 years of being in service. The four-span bridge consists of precast/prestressed concrete AASHTO girders with a cast-in-place deck. Saw cutting and jackhammering methods were used to remove the deck. First, longitudinal saw cuts were made 2 in. from the top flanges of the girders on each side. The longitudinal cuts were to the full depth of 7-½ in. Next, transverse saw cuts were made at 7 ft. increments. The saw cut locations can be seen as the red lines in Figure 3.19 and Figure 3.20. The center to center span between girder lines is 5 ft. 6 in. Chappell Bridge spans 48’, 88’6”, 88’6”, and 48’ from abutment 1 to pier 1, pier 1 to pier 2, pier 2 to pier 3, and pier 3 to abutment 2, respectively.

![Figure 3.19 I-80 Chappell Bridge cross sectional view and dimensions](image-url)
Figure 3.20 Four span Chappell Bridge plan view with location of saw cuts

Transverse saw cuts through the epoxy coated rebar were made starting above the middle bridge pier, and the deck panels between the girders were lifted off the stay-in-place forms. The deck panels easily lifted away from the stay-in-place forms (Figure 3.21). For the deck on top of the girder, a large jackhammer mounted on an excavator was used to break up majority of concrete and then smaller jackhammers were used to minimize the damage to girders and shear connectors (Figure 3.22).

Damage occurred with the use of the large jackhammer mounted to the excavator. Figure 3.23 shows the minor damage that occurred to the top flange of the girders, as well as a half dozen shear connectors. The broken straps between the stay-in-place forms will be replaced before casting the new deck. This operation totaled 1300 man hours with 12 crewmen working 9 hour
days to remove a 275 ft x 18.58 ft x 7.5 in. deep deck. The rate of deck removal with a 12 crewmen on this project was 0.254 mhr/ft².

Figure 3.21 Deck panel lifted from stay-in-place forms between girders
Figure 3.22 Jackhammering above the girders

Figure 3.23 Damaged shear connectors and top flange of the girder

From this investigation, it was learned that the same deck removal methods presented in the first field investigation can be applied to a bridge with stay-in-place forms. The deck panels
between girders were lifted with ease and minor damage was seen on the stay-in-place forms. Only the straps for the stay-in-place forms needed to be replaced.

3.3 Salt Creek Bridge

The purpose of this investigation was to evaluate deck removal techniques used when environmental restrictions exist. The I-80 Salt Creek Bridge over the Salt Creek waterway in Lincoln, NE was demolished in mid-March of 2013. Due to the environmental restrictions, the bridge deck needed to be removed in slabs by saw-cutting and lifting panels. The bridge had three spans from abutment No.1 to pier No.1, pier No.1 to pier No.2, and pier No.2 to abutment No.2; 105 ft, 140 ft, and 105 ft, respectively. The bridge had five girder lines with a deck width of 30 ft.

The non-composite action between deck and girders made removing the deck much easier. First, a jackhammer mounted to an excavator was used to remove the overhang of the deck from the rail. Jackhammering was the major contributor to concrete chunks that fell into the waterway underneath. Saw cutting longitudinally at the center of the deck above the center line girder was done for the whole length of the bridge. Next, transverse saw cuts were made every 5 ft. The non-composite action simplified the removal process, and deck panels easily lifted in 5 ft by 15 ft segments.
Figure 3.24 Plan view with dimensions of Salt Creek Bridge

Figure 3.25 Cross sectional view with girder span and thickness of deck

The location of transverse and longitudinal saw cuts can be seen as the red dashed lines in Figure 3.24 and Figure 3.25. The 5 ft by 15 ft concrete slabs were lifted with the slab crab, seen in Figure 3.26. Small concrete chunks fell to the ground from jackhammering the guard rail (Figure 3.27).
The lessons learned from this investigation were how to maximize efficiency with a non-composite bridge when environmental restrictions exist. Longitudinal saw cutting is not restricted by girder line with a non-composite bridge. Deck panels can be saw cut in very large segments, only restricted by the lifting capacity of the excavator/crane.
3.4 Pacific St. and 106th Street Bridge

The purpose of this investigation was to observe sequencing of tasks in an area with high traffic volume and environmental restrictions. Also, deck removal methods were evaluated when steel I-girders with short steel stud shear connectors are used. The Pacific St. Bridge at 106th Street is located over Big Papillion Creek. The operations were not only restricted environmentally, but also conducted on one side of the bridge, while the other side was open to two-way heavy traffic. Figure 3.28 shows the plan view of Pacific St. Bridge.

![Plan view and dimensions of Pacific St. Bridge](image)

**Figure 3.28** Plan view and dimensions of Pacific St. Bridge

This project included deck replacement and widening of the bridge. The bridge is 240 ft long, 83’8” wide, and has three spans; 70 ft, 100 ft, and 70 ft, respectively. The composite bridge consists of W36 steel girders. Furthermore, because of environmental restrictions, the deck panels needed to be lifted away from the waterway. For re-decking, the steel girders are reused after they are sandblasted and repainted.
The sequencing of tasks was important for efficiency and to minimize the duration of the closed roadway. After clearing the area, closing off the street, and moving equipment into place, the contractor first set up safety lines for workers. The second task performed was longitudinally saw cutting for the full depth along the full length. Two longitudinal saw cuts were made at each girder line, each cut was made in the center of the outer-most shear stud to the center shear stud. The shear connectors used for composite action are three short shear studs spaced along the full length of the girders.

After longitudinal saw cutting, the overhang on each side of the barriers or guard rail were jackhammered and broken off. This was the biggest contributor of concrete chunks that fell to the ground below. Next, transverse saw cuts were done along the full width at an increment of 5 ft, shown in Figure 3.29.

Figure 3.29 Girder layout and saw cut location of Pacific St. Bridge

Next, the deck panels were lifted using a large crane. The crane’s capabilities allowed lifting panels at the mid-span of the bridge. The deck panels popped off from the shear connectors
that were still embedded in the deck. For efficiency, a jackhammer attached to a backhoe was used to jackhammer most of the concrete above the girder. However, the remaining concrete that stayed around the shear connectors needed to be removed and cleaned up using hand chippers. The girders were then able to be removed for further sandblasting and repainting. The total deck removal process took two weeks to complete.

Figure 3.30 Workers using hand chippers to remove concrete around shear studs

Figure 3.31 View looking west after completed deck removal
The lessons learned from this investigation were the sequencing of tasks required for efficient and smooth deck removal. The environmental restrictions and high traffic area dictated the methods and equipment used.
Chapter 4. Cost Data

The purpose of this chapter is to obtain cost data for deck removal practices. This includes the cost of saw cutting, jackhammering, and hydro-demolition for deck removal. The costs are obtained from national average cost data, local contractors, and NDOR contracts.

The national average cost is obtained from *RSMeans Heavy Construction Cost Data*, which has been engaged in publishing construction costs in North America for more than 70 years (Spenser, 2010). The national average cost obtained from the 2010 cost data is shown in Table 4.1, which includes 10% overhead and profit.

<table>
<thead>
<tr>
<th>Job Type</th>
<th>Total Incl O&amp;P</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydro-demolition 4000 psi, 8” depth</td>
<td>$15.13</td>
<td>S.F.</td>
</tr>
<tr>
<td>Concrete Slab Saw Cutting</td>
<td>$7.34</td>
<td>L.F.</td>
</tr>
<tr>
<td>Concrete Jack Hammering for 8 in. deck</td>
<td>$2.38</td>
<td>S.F.</td>
</tr>
<tr>
<td>(excludes saw cutting, torch cutting, loading or hauling)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Break up into small pieces, minimum reinforcing</td>
<td>$2.38</td>
<td>S.F.</td>
</tr>
<tr>
<td>Average reinforcing</td>
<td>$3.56</td>
<td>S.F.</td>
</tr>
<tr>
<td>Maximum reinforcing</td>
<td>$7.14</td>
<td>S.F.</td>
</tr>
</tbody>
</table>

Hawkins Construction Co. is a local bridge contractor who provided estimates on bridge deck removal methods. These methods include break and fall, longitudinal saw cutting and lifting deck panels, and hand removal with jackhammering on top of girders. The break and fall method is used when no environmental restrictions exist and when bridge girders are not to be re-decked. The cost of this method averages $0.99/S.F. The saw cutting and lifting of panels between girders is used when environmental restrictions exist. This method averages $3.16/S.F. The jackhammering method is used to remove the deck on top of the girder for re-decking purposes. This method averages $15.73/S.F, which is almost five times the cost of saw cutting due to the
large number of man-hours involved. **Figure 4.1** shows the cost of each method in dollars per square foot of deck. These estimates were provided by averaging the cost incurred in several projects by the same contractor.

![Cost vs. Type of Method Used in Deck Removal](image)

**Figure 4.1** Illustration of cost vs. type of method used in deck removal

The obtained cost data is applied to Camp Creek Bridge presented in the previous chapter. The three methods used for deck removal on top of the girders are analyzed as following:

1. Removal of a 14 in. wide x 5 ft long x 8 in. deep strip with saw cuts directly outside shear connectors. This is an area of 5.83 ft² of deck to jackhammer.

2. Removal of a 3 ft 2 in. wide x 4 ft 10 in. long x 8 in. deep strip with saw cuts 1 ft away from stirrup legs. This is a deck area of 15.30 ft².
3. Removal of a 3 ft ½ in. wide x 4 ft 10 in. long x 8 in. deep strip with saw cuts directly outside shear connectors and saw cuts at approximately 1 ft away from stirrup legs. This is a deck area of 14.70 ft$^2$.

Table 4.2 shows the total cost for each method estimated using the national average prices.

The cost of deck removal varies depending on location and access to job site.

<table>
<thead>
<tr>
<th>Method</th>
<th>Amount to Saw Cut (L.F.)</th>
<th>Cost of Saw Cutting</th>
<th>Area of Deck to Jack Hammer (S.F)</th>
<th>Cost of Jack Hammering</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method 1</td>
<td>10.00</td>
<td>$73.40</td>
<td>5.83</td>
<td>$20.73</td>
<td>$94.13</td>
</tr>
<tr>
<td>Method 2</td>
<td>9.67</td>
<td>$70.98</td>
<td>15.30</td>
<td>$54.40</td>
<td>$125.38</td>
</tr>
<tr>
<td>Method 3</td>
<td>19.33</td>
<td>$141.91</td>
<td>14.70</td>
<td>$52.27</td>
<td>$194.17</td>
</tr>
</tbody>
</table>

This table indicates that Method 1 is the most cost effective method, while Method 3 is the least cost effective. This is probably due to the very small area used in this analysis. As the deck area increases, the relative cost of saw cutting to jackhammering will change, which may alter this conclusion.
Chapter 5. Analytical Investigations

5.1 Introduction

The purpose of this investigation is to analytically evaluate the effect of top flange width on the flexural capacity, horizontal shear capacity, and deflection of the girder during construction and at service. This investigation aims to present whether cutting or damaging the wide and thin top flange of bridge concrete I-girders has a significant effect on the structural performance of the bridge. A reduction in top flange width by saw cutting will significantly reduce the amount of deck to be jackhammered, which consequently reduces removal cost. Because cross bracing placed between girders after erection is not removed with the deck, the lateral stability of the girder is not a concern even when the top flange width is significantly reduced. Therefore no lateral stability analysis is conducted in this investigation.

In this chapter two examples are investigated: a bridge with a low span-to-depth ratio (14.5) and a bridge with a high span-to-depth ratio (33.53). For each example, calculations are made to compare the flexural capacity, horizontal shear capacity, and deflection of cut flange versus a full flange girder.

5.2 Comparison of Geometric Properties

Saw cutting the top flange reduces the area, inertia, and section modulus of I-girders. The geometric properties of NU girders with a full flange and cut flange are shown in Table 5.1. Geometric properties are calculated assuming reduction of the top flange by 50%, or a longitudinal cut 1 ft from the flange edge on either side, as shown in figure 5.1.
Table 5.1 NU girder full flange and cut flange properties

<table>
<thead>
<tr>
<th>Section</th>
<th>Height (in.)</th>
<th>Web Width (in.)</th>
<th>Top Flange Width</th>
<th>Bottom Flange Width</th>
<th>A (in^2)</th>
<th>Yb (in.)</th>
<th>Yt (in.)</th>
<th>I (in^4)</th>
<th>Sb (in^3)</th>
<th>St (in^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NU 900</td>
<td>35.4</td>
<td>5.9</td>
<td>48.2</td>
<td>38.4</td>
<td>648.1</td>
<td>16.1</td>
<td>19.3</td>
<td>110,262</td>
<td>6849</td>
<td>5713</td>
</tr>
<tr>
<td>NU 1100</td>
<td>43.3</td>
<td>5.9</td>
<td>48.2</td>
<td>38.4</td>
<td>694.6</td>
<td>19.6</td>
<td>23.7</td>
<td>182,279</td>
<td>9300</td>
<td>7691</td>
</tr>
<tr>
<td>NU 1350</td>
<td>53.1</td>
<td>5.9</td>
<td>48.2</td>
<td>38.4</td>
<td>752.7</td>
<td>24.0</td>
<td>29.1</td>
<td>302,334</td>
<td>12597</td>
<td>10389</td>
</tr>
<tr>
<td>NU 1600</td>
<td>63.0</td>
<td>5.9</td>
<td>48.2</td>
<td>38.4</td>
<td>810.8</td>
<td>28.4</td>
<td>34.6</td>
<td>458,482</td>
<td>16144</td>
<td>13251</td>
</tr>
<tr>
<td>NU 1800</td>
<td>70.9</td>
<td>5.9</td>
<td>48.2</td>
<td>38.4</td>
<td>857.3</td>
<td>32.0</td>
<td>38.9</td>
<td>611,328</td>
<td>19104</td>
<td>15715</td>
</tr>
<tr>
<td>NU 2000</td>
<td>78.7</td>
<td>5.9</td>
<td>48.2</td>
<td>38.4</td>
<td>903.8</td>
<td>35.7</td>
<td>43.0</td>
<td>790,592</td>
<td>22145</td>
<td>18386</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
<th>Height (in.)</th>
<th>Web Width (in.)</th>
<th>Top Flange Width</th>
<th>Bottom Flange Width</th>
<th>A (in^2)</th>
<th>Yb (in.)</th>
<th>Yt (in.)</th>
<th>I (in^4)</th>
<th>Sb (in^3)</th>
<th>St (in^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NU 900</td>
<td>35.4</td>
<td>5.9</td>
<td>24.1</td>
<td>38.4</td>
<td>583.5</td>
<td>14.1</td>
<td>21.3</td>
<td>86,550</td>
<td>6151</td>
<td>4058</td>
</tr>
<tr>
<td>NU 1100</td>
<td>43.3</td>
<td>5.9</td>
<td>24.1</td>
<td>38.4</td>
<td>629.9</td>
<td>17.2</td>
<td>26.1</td>
<td>145,744</td>
<td>8493</td>
<td>5576</td>
</tr>
<tr>
<td>NU 1350</td>
<td>53.1</td>
<td>5.9</td>
<td>24.1</td>
<td>38.4</td>
<td>687.5</td>
<td>21.2</td>
<td>32.0</td>
<td>245,547</td>
<td>11610</td>
<td>7685</td>
</tr>
<tr>
<td>NU 1600</td>
<td>63.0</td>
<td>5.9</td>
<td>24.1</td>
<td>38.4</td>
<td>745.7</td>
<td>25.3</td>
<td>37.7</td>
<td>378,927</td>
<td>14960</td>
<td>10059</td>
</tr>
<tr>
<td>NU 1800</td>
<td>70.9</td>
<td>5.9</td>
<td>24.1</td>
<td>38.4</td>
<td>792.1</td>
<td>28.7</td>
<td>42.2</td>
<td>510,897</td>
<td>17777</td>
<td>12118</td>
</tr>
<tr>
<td>NU 2000</td>
<td>78.7</td>
<td>5.9</td>
<td>24.1</td>
<td>38.4</td>
<td>837.9</td>
<td>32.2</td>
<td>46.5</td>
<td>665,007</td>
<td>20672</td>
<td>14292</td>
</tr>
</tbody>
</table>

Figure 5.1 Non-composite cross-section of a full flange and cut flange section
Table 5.2 lists the percent of reduction in all geometric properties of non-composite NU girder sections when the top flange width is reduced by 50%. The range of reduction in area is from 7.3% to 10%, while the range of reduction in moment of inertia is from 15.9% to 21.5%.

<table>
<thead>
<tr>
<th>Section</th>
<th>Percent Area Reduction</th>
<th>Percent Reduction in Centroid (Yb)</th>
<th>Percent Increase in Centroid (Yt)</th>
<th>Percent Reduction in Inertia</th>
<th>Percent Reduction in Section Modulus (Sb)</th>
<th>Percent Reduction in Section Modulus (St)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NU 900</td>
<td>10.0</td>
<td>12.6</td>
<td>10.5</td>
<td>21.5</td>
<td>10.2</td>
<td>29.0</td>
</tr>
<tr>
<td>NU 1100</td>
<td>9.3</td>
<td>12.4</td>
<td>10.3</td>
<td>20.0</td>
<td>8.7</td>
<td>27.5</td>
</tr>
<tr>
<td>NU 1350</td>
<td>8.7</td>
<td>11.9</td>
<td>9.8</td>
<td>18.8</td>
<td>7.8</td>
<td>26.0</td>
</tr>
<tr>
<td>NU 1600</td>
<td>8.0</td>
<td>10.8</td>
<td>8.9</td>
<td>17.4</td>
<td>7.3</td>
<td>24.1</td>
</tr>
<tr>
<td>NU 1800</td>
<td>7.6</td>
<td>10.2</td>
<td>8.4</td>
<td>16.4</td>
<td>6.9</td>
<td>22.9</td>
</tr>
<tr>
<td>NU 2000</td>
<td>7.3</td>
<td>9.9</td>
<td>8.2</td>
<td>15.9</td>
<td>6.7</td>
<td>22.3</td>
</tr>
</tbody>
</table>

5.3 Example 1: Analysis of Camp Creek Bridge

5.3.1 Bridge Parameters

The Camp Creek Bridge over I-80 in Lancaster County, NE, is analyzed using full and cut flange girders. The bridge has three spans with four girder lines per span.

Girder cross-sectional dimensions for composite sections are shown in Figure 5.2. The bridge has girder spacing of 12 ft. Camp Creek bridge parameters are shown in Table 5.3.
Table 5.3 Parameters of the Camp Creek Bridge

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of Girder Lines</td>
<td>x</td>
<td>-</td>
<td>4</td>
</tr>
<tr>
<td>Girder spacing</td>
<td>s</td>
<td>ft</td>
<td>12</td>
</tr>
<tr>
<td>Girder span length</td>
<td>L</td>
<td>ft</td>
<td>52.5</td>
</tr>
<tr>
<td>Girder weight</td>
<td>W_t</td>
<td>k/ft</td>
<td>0.724</td>
</tr>
<tr>
<td>Deck thickness</td>
<td>t</td>
<td>in.</td>
<td>8</td>
</tr>
<tr>
<td>Weight of concrete</td>
<td>W_c</td>
<td>k/ft^3</td>
<td>0.15</td>
</tr>
<tr>
<td>Girder compressive strength</td>
<td>f_c'girder</td>
<td>ksi</td>
<td>8</td>
</tr>
<tr>
<td>Deck compressive strength</td>
<td>f_c'deck</td>
<td>ksi</td>
<td>4</td>
</tr>
</tbody>
</table>

Cross-sectional geometric properties of non-composite and composite sections are presented in Table 5.4. Composite cross-sections to be analyzed are shown in Figure 5.2.

Table 5.4 Full and cut flange cross-sectional properties

<table>
<thead>
<tr>
<th>Girder Cross-Sectional Property</th>
<th>Girder With Full Flange</th>
<th>Girder With Cut Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Precast Section</td>
<td>Composite Section</td>
</tr>
<tr>
<td>A (in^2)</td>
<td>692</td>
<td>1844</td>
</tr>
<tr>
<td>Ig (in^4)</td>
<td>180,542</td>
<td>523,623</td>
</tr>
<tr>
<td>Yb (in.)</td>
<td>19.40</td>
<td>32.90</td>
</tr>
<tr>
<td>Yt (in.)</td>
<td>23.92</td>
<td>14.48</td>
</tr>
<tr>
<td>Sb (in^3)</td>
<td>9,306</td>
<td>15,916</td>
</tr>
<tr>
<td>St (in^3)</td>
<td>7,547</td>
<td>36,174</td>
</tr>
</tbody>
</table>
5.3.2 Effect of Flange Width Reduction on Flexural Capacity and Deflection

This section outlines the analytical investigation of the effect of a reduced flange width on flexural capacity. NU1100 girders use 22 strands that are 0.5 in. in diameter and are tensioned to 75% of an ultimate stress of 270 ksi. A conservative assumption for prestressing losses of 20% is used. The effective prestressing force after all losses is 549 kips.

Applied moments acting on the girder during construction and at service are analyzed. This includes dead load due to girder self-weight, deck-weight, weight due to guardrails, wearing
surface, and any contributing dead loads acting on the super structure. Moments from dead loads and live load are considered.

For applied moments due to live load, the HL-93 live load model proposed by AASHTO is used. Table 5.5 shows the calculated applied moments due to dead and live loads. There is no significant difference in applied moment values due to the reduction of flange width.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Full Flange Girder</th>
<th>Cut Flange Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment due to Girder Self-Weight</td>
<td>M_{s,w.}</td>
<td>k-fl</td>
<td>249</td>
<td>224</td>
</tr>
<tr>
<td>Moment due to Deck Self-Weight</td>
<td>M_{d}</td>
<td>k-fl</td>
<td>413</td>
<td>413</td>
</tr>
<tr>
<td>Moment due to Guard Rail</td>
<td>M_{r}</td>
<td>k-fl</td>
<td>68</td>
<td>68</td>
</tr>
<tr>
<td>Moment due to Wearing Surface</td>
<td>M_{w.s.}</td>
<td>k-fl</td>
<td>81</td>
<td>81</td>
</tr>
<tr>
<td>Dist. Factor</td>
<td>DF</td>
<td>-</td>
<td>1.04</td>
<td>1.04</td>
</tr>
<tr>
<td>Moment due to Live Load</td>
<td>M_{ll}</td>
<td>k-fl</td>
<td>1161</td>
<td>1160</td>
</tr>
</tbody>
</table>

The demand at each phase is calculated using AASHTO LRFD specifications. The strength limit state used is Strength I for basic load combination relating to the normal vehicular use of the bridge without wind. Load factor for permanent loads, \( \gamma_{p} \), is a value of 1.25 (AASHTO, 2007).

The demand is compared against girder capacity calculated for composite and non-composite sections. Strain compatibility concepts are used to calculate full flange girder capacity and cut flange girder capacity under each loading phase. The structural demand and capacity are compared in Table 5.6. The demand presented is at mid-span during construction and under service loads.
The percent reduction in nominal flexural capacity due to cutting of the flange is 8% of non-composite section during construction. There is no reduction in capacity after the deck and girder become a composite section.

The deflection due to live load moment at mid-span is compared for the full flange section and cut flange section. The composite section inertia values are relatively the same for the full flange and cut flange girder cross-sections, which gives close values for service load deflections. Deflection values are all within the limit of l/400.

As presented in the table, the flexural capacity of the cut flange girder highly exceeds the demand at construction, which is the critical loading stage. This is the case for the 52 ft 6 in. long span girder, whereas a NU1100 girder can span much longer.

**Figure 5.3** shows a graph of the capacity/demand ratio at each critical stage for the full flange section and cut flange section. The ratio in all loading stages is greater than 1.0, which is considered adequate.

| Table 5.6 Capacity/demand comparison of full flange and cut flange girder sections |
|---------------------------------|---------------------------------|---------------------------------|---------------------------------|---------------------------------|
| Camp Creek Bridge NU1100 Girder (52.5 ft span) | Full Flange Girder | Cut Flange Girder |
| | Non-Composite | Composite | Non-Composite | Composite |
| Nominal Moment Capacity M<sub>n</sub> (k-ft) | 2,995 | 3,636 | 2,754 | 3,636 |
| Demand (k-ft) | 712 | 2,904 | 685 | 2,875 |
| Ratio | 4.21 | 1.25 | 4.02 | 1.26 |
| L.L. Deflection at Mid-span (in.) | - | 0.216 | - | 0.217 |
Because of the short span, the girder did not experience moment values close to the nominal capacity, however, this may not always be the case.

Analytical steps were taken to satisfy serviceability requirements. Stresses at the top and bottom fibers were checked at different loading stages. Stress values were checked under construction loads and under service loads.

AASHTO LRFD specifications limit states were used. Maximum compression was checked under Service I limit states and maximum tension was checked under Service III limit states. The difference between Service I and Service III limit states is that Service I has a limit state of 1.0 for live load whereas Service III has a limit state of 0.8 for live load.

Compressive stress limits and tensile stress limits were obtained from table 5.9.4.2.1-1 and table 5.9.4.2.2-1 in the AASHTO LRFD specifications book. For compressive stress, the limit due
to the sum of effective prestress, permanent loads, transient loads, and during shipping and handling is \(0.60*f_c'\) (AASHTO, 2007). For tensile stress, the limit in prestressed concrete after losses for fully prestressed components in bridges, which include bonded prestressing tendons and are subjected to no worse than moderate corrosion conditions, shall be taken as \(0.19 * \sqrt{f_c'}\) (AASHTO, 2007).

Table 5.7 summarizes the results of the analysis. As presented in the table, the serviceability requirements, assuming non-cracked sections, are met.

<table>
<thead>
<tr>
<th>Load</th>
<th>Camp Creek Bridge Midspan Critical Service Stresses</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girder With Full Flange</td>
</tr>
<tr>
<td></td>
<td>At Construction</td>
</tr>
<tr>
<td>ft (ksi)</td>
<td>fb (ksi)</td>
</tr>
<tr>
<td>P/A</td>
<td>0.79</td>
</tr>
<tr>
<td>(Pe+e)/Snon-com</td>
<td>-1.23</td>
</tr>
<tr>
<td>Ms.w./Snon-com</td>
<td>0.40</td>
</tr>
<tr>
<td>Md/Snon-com</td>
<td>0.66</td>
</tr>
<tr>
<td>Mr/Scom</td>
<td>-</td>
</tr>
<tr>
<td>Mw.s./Scom</td>
<td>-</td>
</tr>
<tr>
<td>MLI/Scom</td>
<td>-</td>
</tr>
<tr>
<td>Combined Stresses (ksi)</td>
<td>0.61</td>
</tr>
<tr>
<td>Limiting Stresses (ksi)</td>
<td>0.45*f_c'</td>
</tr>
<tr>
<td></td>
<td>3.60</td>
</tr>
</tbody>
</table>

The results obtained for critical stresses for the Service I limit state and for the Service III limit state are all within the limit for an un-cracked section.

5.3.3 Effect of Flange Width Reduction on Horizontal Shear Capacity
This section analyzes the effect cutting girder top flange has on horizontal shear. The 2012 AASHTO bridge specifications are followed for calculations. By reducing flange width, the area of concrete engaged in horizontal shear transfer, $A_{cv}$, is reduced.

For the full flange girder, $A_{cv}$ is the full concrete surface area of the flange minus the debonded area. The effective width, $b_v$, is shown below. The effective depth, $d_v$, is the same for both cases. $A_{cv}$ is reduced by 25% when the flange is cut to a 2 ft width.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Full Flange Girder</th>
<th>Cut Flange Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b_v$</td>
<td>32 in.</td>
<td>24 in.</td>
</tr>
<tr>
<td>$d_v$</td>
<td>44.7 in.</td>
<td>44.7 in.</td>
</tr>
<tr>
<td>$A_{cv}$</td>
<td>1430 in$^2$</td>
<td>1073 in$^2$</td>
</tr>
</tbody>
</table>

The stirrup configuration of the Camp Creek girders is shown in Figure 5.4. The maximum interface reinforcement spacing is two #3 stirrups every 12 in. at mid-span and a minimum spacing of 2 in. at the ends of the girder. The stirrup reinforcement area for two #3 bars is 0.22 in$^2$.

Nominal horizontal shear capacity values are calculated for distances from the mid-span in increments of 2 ft. The results are obtained and graphed in Figure 5.5. From the results, it is
apparent that when cutting through the top flange, some horizontal shear capacity is lost due to a reduction in top flange width. The average percent loss in horizontal shear capacity over 24 ft from mid-span is 20.75%.

![Graph showing horizontal shear capacity for NU1100](image)

**Figure 5.5** Effect of cutting top flange on horizontal shear capacity for NU1100

5.4 Example 2: Oxford South Bridge

5.4.1 Bridge Parameters

The second example is of a bridge utilizing NU1350 girders. The span chosen for analysis is 140 ft long. The span-to-depth ratio is 33.53. With more than double the span-to-depth ratio from the previous example, this is a much less conservative case. The same analytical procedure is used to analyze the effect of cutting the top flange on flexural capacity, horizontal shear capacity, and mid-span deflection. The composite cross-sections of the NU1350 full flange and cut flange section is shown below in **Figure 5.6**. The bridge parameters are shown below in **Table 5.9**.
Cross-sectional geometric properties for the case of a NU1350 girder with a full flange and cut flange section are shown in Table 5.10. In this example, the larger girder has slightly less of a reduction in properties than in the previous example. Composite cross-sections to be analyzed are shown in Figure 5.6.
The Oxford South Bridge is a five span bridge with four girder lines per span. Span lengths are 110 ft, 110 ft, 140 ft, 110 ft, and 110 ft respectively from the south abutment to the north abutment.
NU1350 girders are tensioned using 34 strands with a diameter of 0.7 in. and six strands with a diameter of 0.6 in. With an assumption for prestress losses of 20%, the effective prestressing force is 1830 kips.

Dead load moments from the girder self-weight and the deck are accounted as applied moments at construction. Dead load from the guardrail, wearing surface, and live load using the AASHTO HL-93 model are accounted as applied moments at service. The values for applied moments are in Table 5.11 below. For live load moment, the distribution factor for the full flange section and cut flange section are the same despite the reduced inertia, area, and centroid of the section. The case for an interior bridge girder is assumed.

### Table 5.11 Calculated dead and live load moments

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Full Flange Girder</th>
<th>Cut Flange Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment due to Girder Self-Weight</td>
<td>Ms.w.</td>
<td>k-ft</td>
<td>1923</td>
<td>1756</td>
</tr>
<tr>
<td>Moment due to Deck Self-Weight</td>
<td>Md</td>
<td>k-ft</td>
<td>2205</td>
<td>2205</td>
</tr>
<tr>
<td>Moment due to Guard Rail</td>
<td>Mr</td>
<td>k-ft</td>
<td>490</td>
<td>490</td>
</tr>
<tr>
<td>Moment due to Wearing Surface</td>
<td>Mw.s.</td>
<td>k-ft</td>
<td>441</td>
<td>441</td>
</tr>
<tr>
<td>Dist. Factor</td>
<td>DF</td>
<td>-</td>
<td>0.68</td>
<td>0.68</td>
</tr>
<tr>
<td>Moment due to Live Load</td>
<td>Ml.l.</td>
<td>k-ft</td>
<td>3082</td>
<td>3081</td>
</tr>
</tbody>
</table>

For the full flange section, the demand is 4,438 k-ft at construction and 10,832 k-ft at service. The capacity of the full flange section and cut flange section are compared against demand in Table 5.12. At the critical stage of construction, the capacity/demand ratio is 1.37 for a full flange section and 1.13 for the cut flange section.
The effect of cutting the top flange reduced the girder flexural capacity by 20.81% during construction. After the deck and girder become a composite section, there is no difference in capacity with a full flange or cut flange section. The critical stage of this method is during the re-decking processes, where there is the weight of fresh concrete during construction. Table 5.6 graphs the capacity/demand ratio for the girder in this example, showing the ratio above 1.0 as adequately designed.

For mid-span deflection calculations, the values are the same for composite sections under live load moments at service. Deflection values are all within the limit of l/400.

Table 5.12 Capacity vs. demand of full flange and cut flange girders

<table>
<thead>
<tr>
<th></th>
<th>Oxford South Bridge NU1350 Girder (140 ft span)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Non-Composite</td>
</tr>
<tr>
<td>Nominal Moment Capacity M_n (k-ft)</td>
<td>6,085</td>
</tr>
<tr>
<td>Demand (k-ft)</td>
<td>4,438</td>
</tr>
<tr>
<td>Ratio</td>
<td>1.37</td>
</tr>
<tr>
<td>L.L. Deflection at Mid-span (in.)</td>
<td>-</td>
</tr>
</tbody>
</table>
This extreme example shows that the deck removal method of cutting the top flange to reduce manual labor may not always be adequate for re-decking. Contractors must always take precautions and checks must always be made.

For serviceability requirements, AASHTO LRFD specifications limit states are used. Maximum compression is checked under Service 1 limit states and maximum tension is checked under Service III limit states. For compressive stress, the limit during construction loading stages shall be taken as $0.45 f'_c$, and at final loading conditions as $0.60 f'_c$ (AASHTO, 2007). For tensile stress, the limit shall be taken as $0.19 \sqrt{f'_c}$ (AASHTO, 2007). The critical service stresses exceed the limit for an uncracked section (Table 5.13).
Table 5.13 Midspan critical stresses for 140 ft long N1350 girder

<table>
<thead>
<tr>
<th>Load</th>
<th>Oxford Bridge Midspan Critical Service Stresses</th>
<th></th>
<th>Girder With Cut Flange</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At Construction</td>
<td>At Service</td>
<td>At Construction</td>
<td>At Service</td>
</tr>
<tr>
<td></td>
<td>ft (ksi)</td>
<td>fb (ksi)</td>
<td>ft (ksi)</td>
<td>fb (ksi)</td>
</tr>
<tr>
<td>P/A</td>
<td>2.60</td>
<td>2.60</td>
<td>2.60</td>
<td>2.60</td>
</tr>
<tr>
<td>(Pe*e)/Snon-com</td>
<td>-3.91</td>
<td>3.15</td>
<td>-3.91</td>
<td>3.15</td>
</tr>
<tr>
<td>Ms.w./Snon-com</td>
<td>2.27</td>
<td>-1.83</td>
<td>2.27</td>
<td>-1.83</td>
</tr>
<tr>
<td>Md/Snon-com</td>
<td>2.60</td>
<td>-2.10</td>
<td>2.60</td>
<td>-2.10</td>
</tr>
<tr>
<td>Mr/Scm</td>
<td>-</td>
<td>-</td>
<td>0.15</td>
<td>-0.33</td>
</tr>
<tr>
<td>Mw.s./Scm</td>
<td>-</td>
<td>-</td>
<td>0.14</td>
<td>-0.29</td>
</tr>
<tr>
<td>M11/Scm</td>
<td>-</td>
<td>-</td>
<td>0.96</td>
<td>-1.64</td>
</tr>
<tr>
<td>Combined Stresses</td>
<td>3.56</td>
<td>1.81</td>
<td>4.82</td>
<td>-0.44</td>
</tr>
<tr>
<td>(ksi)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limiting Stresses</td>
<td>0.45*fc'</td>
<td>0.45*fc'</td>
<td>0.60*fc'</td>
<td>0.19*(fc')</td>
</tr>
<tr>
<td>(ksi)</td>
<td>4.05</td>
<td>4.05</td>
<td>5.40</td>
<td>-0.57</td>
</tr>
</tbody>
</table>

Service I limit state for compressive stresses is exceeded in the top flange at construction and service due to a reduction in flange width.

5.4.3 Effect of Flange Width Reduction on Horizontal Shear Capacity

This section analyzes the effect of horizontal shear on the Oxford South Bridge NU1350 girder. The analytical steps are the same as in the previous section, the only difference being that the effective depth, \( d_v \), is 53.81 in. instead of 44.7 in. and the stirrup distribution is slightly different. However, the effective width, \( b_v \), is reduced the same amount by cutting the top flange, and therefore the engaged area \( A_{cv} \) is reduced by 25%.
Table 5.14 Parameters for horizontal shear analysis on NU1350

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Full Flange Girder</th>
<th>Cut Flange Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b_v$</td>
<td>32 in.</td>
<td>24 in.</td>
</tr>
<tr>
<td>$d_v$</td>
<td>53.81 in.</td>
<td>53.81 in.</td>
</tr>
<tr>
<td>$A_{cv}$</td>
<td>1430 in$^2$</td>
<td>1073 in$^2$</td>
</tr>
</tbody>
</table>

The stirrup configuration of the Oxford South Bridge girders is shown in Figure 5.8. The maximum interface reinforcement spacing is two #3 stirrups every 12 in. at midspan and minimum spacing of 2 in. at the ends of the girder. Stirrup reinforcement area for two #3 bars is 0.22 in$^2$.

Figure 5.8 NU1350 stirrup distribution to midspan of girder

Nominal horizontal shear capacity values are calculated for distances from the mid-span in increments of 2 ft. The results are obtained and graphed in Figure 5.9. From the results, the average percent loss in horizontal shear capacity over 24 ft from mid-span is 21.87%.
Effect of cutting top flange on horizontal shear capacity for NU1350

5.5 Conclusion

From the analytical investigations of this chapter, the following conclusions can be made:

1. For a low span-to-depth ratio scenario, a reduction in top flange width is not detrimental to girder capacity under service loads. However, there is a significant enough difference in capacity in a large span-to-depth ratio scenario to be inadequate.

2. There is a significant reduction in horizontal shear capacity of about 20-22% due to flange width reduction of 50%.
Chapter 6. Experimental Investigation

The purpose of the experimental investigation is threefold:

1. Evaluate the effect of the saw cutting/damaging the thin top flange on the girder flexural capacity and validating the analytical investigation presented in Chapter 5.

2. Evaluate the effect of the saw cutting/damaging the thin top flange on the horizontal shear capacity of the girder due to reduced interface surface area. The effect of partial removal of concrete around the shear connected will also be evaluated.

3. Evaluate the efficiency of saw cutting and jackhammering operations to estimate the cost and duration of different removal methods.

Two 52 ft long girders were removed from the Camp Creek Bridge in the spring of 2012 and stored at Concrete Industries (CI) in Lincoln, NE to be used for this experimental investigation. The bridge deck was saw cut 8 in. from the edge of the top flange and the intermediate and end diaphragms were broken. A cross-sectional view of the bridge and bridge layout can be seen in the deck removal field investigation discussed in section 3.1. Figure 6.1 shows one of the two NU1100 bridge girders used in the experimental investigation being lifted. Figure 6.2 shows the locations of the concrete cores drilled from the girder ends to evaluate the effectiveness of the debonding agents used between the girder top flange and the deck to facilitate deck removal without damaging the thin top flange.
For each girder, four cores were drilled; two on each side of the two girders. On each side of the girders, one core was drilled in the bonded area, and one in the debonded area between the deck and the girder, shown in Figure 6.3. The cores taken in the bonded area remained in one piece, the cores taken in the debonded area split at the deck-to-girder interface. The orange-colored surface shown in Figure 6.4 is due to the use of a debonding agent. Concrete cores were also used to estimate compressive strength of the two girders.
Figure 6.3 Cores taken from bonded zone (left) and from debonded zone (right)

Figure 6.4 Bonded concrete surface (left) and debonded concrete surface (right)

6.1 Specimen Preparation

6.1.1 Saw Cutting Sequence

The first step in preparing the two 52 ft long NU1100 girders for testing is to make longitudinal full depth saw cuts. The “full flange girder” was saw cut to the full depth of the deck while keeping the full flange intact. The “cut flange girder” was saw cut the full depth of the deck and through the flange. The effect of reduced flange width on the flexural and horizontal shear
capacities will be determined via testing. The dimensions of the girders with old deck are given in Figure 6.5. The deck is 5 ft 4in. wide and 8 in. thick with an additional 1 in. haunch. The debonded zone is 8 in. wide strip from the edges of the 4 ft wide top flange.

![Diagram of girder dimensions](image)

**Figure 6.5** Composite dimensions of girders taken from Camp Creek Bridge

6.1.1.1 Full Flange Girder

For the full flange girder, saw cuts were made at the end of the debonded zone for the full depth of the deck to make the removal of the overhung portions of the deck easy to remove without damaging the girder top flange. Saw cutting closer to the shear connectors will minimize the amount of jackhammering needed, however removing the bonded deck would have been very difficult without damaging the top flange. Therefore, longitudinal saw cuts were made 1 ft 4 in. from the center of the girder on either side. Figure 6.6 shows the location of each saw cut. The cuts were made to the full depth of about 9 in.
The 9 in. deep longitudinal saw cuts required three passes, cutting down about 2-3 in. per pass. **Figure 6.7** shows the longitudinal saw cuts being made on the full flange girder. **Figure 6.8** shows a cross-sectional view after all longitudinal saw cuts were made.
6.1.1.2 Cut Flange Girder

For the cut flange girder, the saw cuts were made to the full depth of the deck and through the top flange. The locations of saw cuts were dictated by the width of the saw cutting machine, which is 25 in. Therefore, the flange width was reduced from 48 in. to 25 in. Figure 6.9 shows the location of longitudinal saw cuts on the cut flange girder, which were made 12.5 in. from the center of the girder. The cuts were made to a depth of about 1 ft.
The 12 in. deep longitudinal cut required four passes, cutting down about 3 in. per pass. A larger blade was required for the last pass. The blade was switched from an 18 in. diameter blade to a 24 in. diameter blade, as shown in Figure 6.10. Saw cutting was most difficult when the blade hit a longitudinal bar because the bar had to be cut longitudinally, which slowed down the process. Figure 6.11 shows a cross-sectional view of the cut flange girder after saw cutting was completed. The total time required to finish the saw cutting on both girders was five hours. Two hours were spent on the full flange girder, and three hours on the cut flange girder.
Figure 6.10 Separation of deck and flange after falling down

Figure 6.11 Finished saw cuts for cut flange girder
6.1.2 Jackhammering Sequence

The next step is to remove the deck above the girder and around the shear connectors by jackhammering. The jackhammering plan for both girders is to have the deck completely removed for half of the girder (100% deck removal), and partially removed around the connectors in the other half (50% deck removal). The purpose of doing this is to investigate the effect of deck removal level on horizontal shear capacity. By not requiring the contractor to completely remove the old deck around the connectors, significant savings in the removal cost and duration can be achieved.

There is no specific volume or dimensions of the concrete deck to leave around the connector; jackhammering is not very precise and this would defeat the purpose. However, it is important to leave some space, about 1-2 in. underneath each stirrup leg. This provides space for the new deck to latch onto the bar, as well as the necessary clearance. Adequate space should be provided in between the shear connectors to lay transverse reinforcement of the new deck. Three workers from a local concrete removal contractor were hired to do this job. Observations were made, activity logs were recorded, and different removal methods were attempted.

Figure 6.12 and Figure 6.13 shows a plan view, cross-sectional view, and side view of the jackhammering plan with the orange hatched area denoting the area to be jackhammered. After saw cutting, the remaining deck on the full flange girder is a 2 ft 8 in. wide, 9 in. deep, and 52 ft long. The remaining deck for the cut flange girder is 25 in. wide, 9 in. deep, and 52 ft long. The job took five days to complete, with the fifth day being a half day.
Figure 6.12 Jackhammering plan for girder #1
Figure 6.13 Jackhammering plan for girder #2
Two sizes of jackhammers were used. The 90 lb jackhammer was used down to the depth of the shear connectors. This top 4 in. of concrete took the least time per unit volume to remove. Once the shear connectors were reached, the 60 lb jackhammer was used to remove the remaining concrete, which was much more difficult, especially at the girder ends where the spacing between shear connectors is small, as shown in Figure 6.14.

![Figure 6.14 Difficulty removing concrete under longitudinal bars](image)

For the full flange girder, approximately 88 ft$^3$ total volume of concrete deck needed to be removed. For the cut flange girder, approximately 62 ft$^3$ needed to be removed. At the end of the first day, workers removed approximately 51 ft$^3$ of concrete. This was mostly the volume of concrete above the stirrups, shown in Figure 6.15, which is the easiest to remove.
Using the 90 lb jackhammer caused vibration that resulted in minor damage to the sides of the top flange, shown in Figure 6.16. Stirrups were also damaged from the blunt force of the 90 lb jackhammer (Figure 6.17). A few instances caused delays in jackhammering, such as the generator shutting off and the jackhammer getting stuck.
At the end of the second day, the crew suggested they saw cut the deck transversely to speed up the deck removal with the jackhammer. A hand held saw was used. Transverse saw cuts were made every 5 in. for about a 4 in. depth, as seen in Figure 6.18 and Figure 6.19. This method is much more efficient. The concrete came out easier in bigger chunks. It was especially efficient with simultaneous jackhammering and use of an electric hand chipper. Jackhammers were used vertically from the top of the deck, while an electric chipper was used from the side, resulting in the concrete coming out in larger blocks. At the end of the second day, an additional 46 ft$^3$ of concrete had been removed. At the end of the third day, the “fully removed” half of the cut flange girder was completed, leaving only the “partially removed” half. This is approximately 26 ft$^3$ of concrete. The full flange girder only had concrete around the stirrups for the full length of the girder.
Figure 6.18 Transverse saw cutting every 5 in.

Figure 6.19 Depth of saw cuts about 4 in.; 1 in. from top flange
Deck removal continued with the technique of first transverse saw cutting, then jackhammering and chipping. Approximately 1-2 in. of space were to be left underneath the stirrup legs. **Figure 6.20** and **Figure 6.21** show simultaneous jackhammering and electric chipping while keeping the necessary clearance underneath stirrup legs. At the end of the fourth day, the jackhammering plan was completed on the full flange girder.

**Figure 6.20** Leaving space underneath stirrup legs and removing transverse bars on full flange girder

**Figure 6.21** 1-2 in. of space underneath stirrup legs
On the fifth day, the jackhammering job was completed on the cut flange girder. There was damage observed on the top flange of the girder, which can be seen in Figure 6.22. The stirrups were damaged, one which completely detached from the flange (Figure 6.23). The total time to finish the last portion of the job, about 25 ft$^3$ of concrete, was 3.5 hrs. A view of the prepared NU1100 specimen after saw cutting and jackhammering procedures is seen in Figure 6.24.

![Figure 6.22 Damaged flange on girder #2](image)

![Figure 6.23 Broken stirrup leg](image)
A man-hour log was made for the full flange girder and cut flange girder. The results show that spending 1 extra hour on saw cutting through the top flange will save approximately 20% of the jackhammering on top of the girder (Table 6.1).

<table>
<thead>
<tr>
<th>Time spent of Manual Labor</th>
<th>Full Flange Girder</th>
<th>Cut Flange Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saw Cutting</td>
<td>2 hours</td>
<td>3 hours</td>
</tr>
<tr>
<td>Jack Hammering</td>
<td>60 hours</td>
<td>48 hours</td>
</tr>
<tr>
<td>Total MHrs</td>
<td><strong>62 hours</strong></td>
<td><strong>51 hours</strong></td>
</tr>
</tbody>
</table>

6.2 Specimen Testing

The two NU1100 girders were shipped to the structural laboratory at the Peter Kiewit Institute in Omaha, NE on November 20th, 2013. Since there are no lifting points, girders were
handled by wrapping lifting chains around the girders to place them on the supports for flexure testing.

6.2.1 Re-Decking

For the full flange girder, a 4 ft wide and 7.5 in. thick deck was formed using plywood sheets and threaded rod ties as shown in Figure 6.25 and Figure 6.27. The new deck was reinforced transversely in two layers with #5@12 in. and longitudinally with 4 #8 at the mid-thickness, which is equivalent to two layers of #4@12 in. over an 8 ft wide deck.

Figure 6.25 Ties with PVC pipe spaced 3 ft apart to hold form sides
The deck was poured using 8 ksi self-consolidating concrete (SCC) with 25 in. slump. This is equivalent to a 4 ksi concrete for 8 ft wide deck. **Figure 6.27** shows the new deck after forms have been stripped.

**Figure 6.27** View of fully composite deck for full flange girder

The same procedures were followed for re-decking the cut flange girder. It should be noted that two pairs of shear connectors on each half of the girder were cut off from the flange on both
the fully removed and partially removed halves, as shown in figure 6.29. This was suggested in order to simulate the situation when some shear connectors are damaged during the jackhammering process.

![Figure 6.28 Cut shear connectors in partially removed section](image)

6.2.2 Test Setup

Girders were tested in flexure after the desired concrete compressive strength was achieved. The test setup is shown in Figure 6.29. Girders were simply supported on concrete blocks and roller supports that are spaced 40 ft on center. Two threaded rods 10 ft apart were used to anchor the steel loading I-beam to the strong floor. A 400 kip capacity hydraulic jack and load cell were used for loading each girder at mid-span. Rotation of the loading frame was prevented by using a strap on both sides that were anchored into the ground 12 ft from the center of the girder on either side, as shown in Figure 6.30
Each girder was instrumented with four horizontal and two vertical linear variable differential transformers (LVDTs) to measure displacement, one string potentiometer to measure
deflection at mid-span, and three strain gauges to measure strains at the critical section. Figure 6.31 shows the instrumentations used during testing. A view of the completed test setup on the full flange girder is shown in Figure 6.32.

Figure 6.31 LVDTs (left), strain gauges (right)

Figure 6.32 Test setup on full flange girder
6.3 Analysis of Results

6.3.1 Concrete Compressive Strength

The compressive strength testing results of the cores extracted from the girders and the cylinders made from the newly poured deck are shown in Table 6.2. These results indicate that the measured strength significantly exceeded the specified strength for the new deck.

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>Full Flange Deck poured on Nov 27th</th>
<th>Cut Flange Deck poured on Dec 19th</th>
<th>Camp Creek Girder cored cylinders</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4-day strength</td>
<td>16-day strength</td>
<td>7-day strength</td>
</tr>
<tr>
<td>1</td>
<td>8,897</td>
<td>10,765</td>
<td>9,679</td>
</tr>
<tr>
<td>2</td>
<td>8,885</td>
<td>11,093</td>
<td>10,052</td>
</tr>
<tr>
<td>3</td>
<td>8,903</td>
<td>11,209</td>
<td>9,234</td>
</tr>
<tr>
<td>Ave. (psi)</td>
<td>8,895</td>
<td>11,022</td>
<td>9,655</td>
</tr>
</tbody>
</table>

6.3.2 Full Flange Girder Test

The full flange girder was tested on December 10th, 2013. Horizontal and vertical LVDTs will be referred to as H-NW and H-NE for the horizontal North West and North East LVDTs, and as V-N and V-S for the vertical north and south LVDTs. Strain gauges were labeled as deck, top flange, and bottom flange strain gauges. The same notations were used for the cut flange girder. The full flange girder failed in flexure at an ultimate load of 379 kips and had a cracking load of 224 kips. The vertical deflection at mid-span was 0.45 in. at the cracking load, and 5.83 in. at the ultimate load. Flexural failure mode is shown in Figure 6.33.
Figure 6.33 Failure mode of full flange girder

The load-deflection curve for the test of the full flange girder is shown in Figure 6.34. This figure indicates the elastic behavior of the composite girder up to the cracking load, and the leveling off of the curve up to the ultimate load.
Figure 6.35 plots strain data at the critical section. This plot indicates the increase in the compression strains at the top fibers and tension strains at the bottom fibers up to failure load, which demonstrates the composite action of the girder.

![Strain Gauges](image)

**Figure 6.35 Full flange - load vs. strain graph**

The relative displacement data gathered from LVDTs is shown in Figure 6.36 and Figure 6.37. These two graphs indicate that horizontal and vertical displacements of the new deck relative to the girder are negligible, as the maximum value for horizontal displacement is 0.001 in. and 0.0035 in for vertical displacement, which are less than 0.01 in., the minimum acceptable initial slip. Furthermore, the displacements of the fully removed half of the girder and the partially removed half are identical. This indicates that the level of deck removal around the shear connectors does not affect the horizontal shear capacity of the full flange girder.
**Figure 6.36** Horizontal displacement for partially removed vs. fully removed

**Figure 6.37** Vertical displacement for partially removed vs. fully removed
6.3.3 Cut Flange Girder Test

The cut flange girder was tested on January 9th, 2014. The girder failed in flexure, however, the strands did not rupture as shown in Figure 6.38. The ultimate load was 378 kips and the cracking load was 230 kips. The vertical deflection at mid-span was 0.35 in. at the cracking load and 5.96 in. at ultimate load, as shown in the load-displacement curve plotted in Figure 6.39. Figure 6.40 plots strain data at the critical section. This plot indicates the increase in the compression strains at the top fibers and tension strains at the bottom fibers up to failure load, which demonstrates the composite action of the girder.

Figure 6.38 Cut flange girder failure mode
Figure 6.39 Cut flange - load vs. deflection graph

Figure 6.40 Cut flange - load vs. strain graph
Figure 6.41 and Figure 6.42 show the horizontal and vertical displacements of the new deck relative to the girder. The amount of displacement was negligible since the maximum value is 0.002 in., which is lower than the minimum acceptable value for initial slip (0.01 in.). Furthermore, the displacement of the fully removed half of the girder and the partially removed half are identical. This indicates that the level of deck removal around the shear connectors does not affect the horizontal shear capacity of the cut flange girder.

![Horizontal LVTDs](image)

**Figure 6.41** Horizontal displacement for partially removed vs. fully removed
Figure 6.42 Vertical displacement for partially removed vs. fully removed

6.3.4 Comparison

Comparing the test results of the full flange girder and cut flange girder indicates that the top flange width has no effect on the flexural capacity, which confirms the results of the analytical investigation presented in chapter 4. Figure 6.43 shows the load-deflection curves of the two girders, while figure 6.44 and figure 6.45 plot the measured cracking load, ultimate load, and deflection versus predicted values. This plot indicates that measured values are very close to the predicted values, which were calculated assuming a fully composite section. This means that tested girders, whether full flange, cut flange, with partial deck removal, or with full deck removal, behaved as fully composite with the new deck.
**Figure 6.43** Load vs. deflection curve of investigated NU1100 girders

![Graph showing load vs. deflection curve]

**Figure 6.44** Bar graph comparing actual to predicted load

![Bar graph showing load comparison]

- Measured Full Flange
- Measured Cut Flange
- Predicted
Figure 6.45 Bar graph comparing actual to predicted deflection.
Chapter 7. Conclusions

Based on the results of the field investigations, analytical investigation, and experimental investigation presented in this report, the following conclusions can be made:

1. Saw cutting, jackhammering, and hydro-demolition are the most common methods of deck removal for re-decking.

2. Debonding the edges of the top flange is an effective method for lifting saw cut deck panels between girders without damaging the thin top flange of the girders.

3. The most cost-effective method of deck removal is highly dependent on the quantity, environmental restrictions, and type of girder and its shear connectors.

4. Leaving approximately 50% of the old deck concrete around shear connectors does not significantly affect the horizontal shear capacity of the new composite section.

5. The effect of cutting approximately 50% of the girder top flange width on the structural performance of the girder is highly dependent on the span-to-depth ratio. In girders with a low span-to-depth ratio, this effect is negligible; however, in girders with high span-to-depth ratio, this effect could be significant. Flexural capacity, horizontal shear capacity, and deflections should be checked during construction and at final stages.
Chapter 8. Recommendations

Based on the observations made in this project and the outcomes of the literature search and analytical and experimental investigations, the following recommendations can be made:

1. Extend the width of the debonded strip for wide and thin top flange girders, such as NU girders, so they are at least 12 in. instead of 8 in., as shown in figure 8.1. This will minimize the amount of deck that needs to be jackhammered, which is a tedious and costly process, and ensure easy lifting of deck panels.

2. Saw cut deck panels at angles to simplify the deck removal process rather than using vertical saw cuts as shown in figure 8.1. However, pivoted saw cut machines are needed to improve the efficiency of this option compared to using the frame mounted saws that slow down the process.

Figure 8.1 Recommended practice for debonding and saw cutting
3. For existing bridges with low span-to-depth ratio and narrow debonded zone, saw cut the deck outside the shear connectors and through the wide and thin girder top flange as shown in figure 8.2. The remaining concrete around the shear connectors could be manually removed with 60 lb jackhammers up to the level of the shear connectors, and 30 lb jackhammers below the shear connectors.

4. Approximately 50% of the concrete around the shear connectors can be left intact as long as the concrete is not contaminated and will not affect the durability of the new deck. Also, several transverse cuts are recommended to maximize the efficiency of deck removal over the girder flange.

Figure 8.2 Recommended practice for existing bridges
Works Cited


