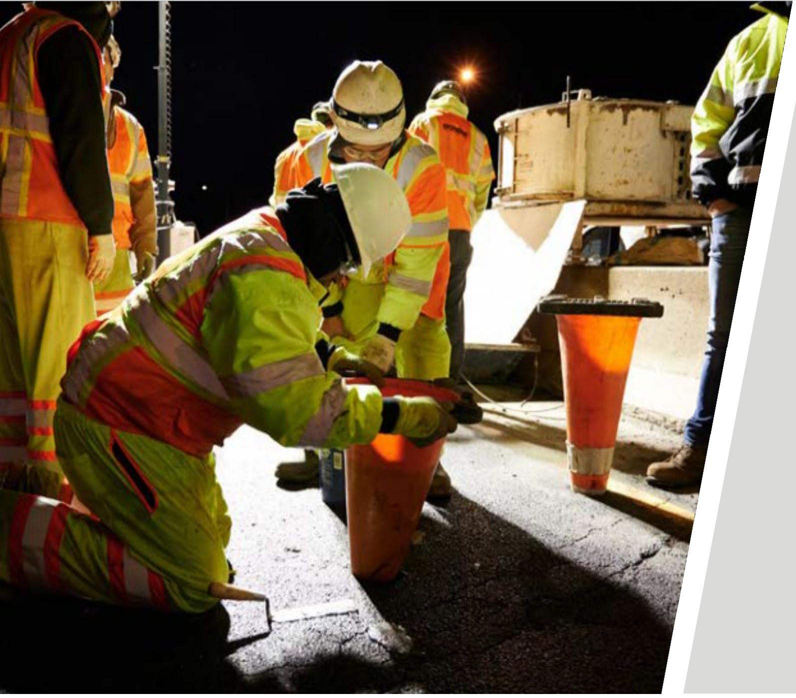

Guidelines for the Utilization of Ultra-High Performance Concrete in the Rehabilitation of Steel Bridge Girder Ends

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1 INTRODUCTION

1.1 SCOPE OF THE GUIDELINES

The provisions of these guidelines aim to provide guidance on the repair of corrosion damaged steel bridge girder ends with Ultra-High Performance Concrete (UHPC). All references to "repair" in this document represent repair of corrosion-damaged steel bridge girder ends using UHPC. The provisions in this document only address the aspects of the repair, and do not supersede any other standard design guidelines or standard documents. They are intended to provide guidance on the effective means and practices to implement this repair method.

The provisions of these guidelines fit within the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) framework. All the load and resistance factors are borrowed from the AASHTO LRFD Specifications.

These guidelines only address the static capacity and fatigue resistance of the repair design. All other design aspects, such as seismic considerations, are not part of the scope of these guidelines.

The commentary is not intended to provide a complete historical background on the development of these guidelines, nor is it intended to provide a detailed summary of all research. References to important research data are provided.

C1.1

The term "shall" denotes a requirement for compliance with these guidelines.

The term "should" indicates a strong preference for a given criterion.

The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criteria can also be used in a manner consistent with the LRFD approach to bridge design.

The term "recommend" is used for suggestions based on prior experience.

1.2 OVERVIEW OF REPAIR

The repair is an alternative means of restoring bearing/shear capacity lost due corrosion damage on the web and stiffeners in steel bridge girders. The repair involves casting a panel of UHPC over each side of the web of the damaged girder. The UHPC panel is attached to the girder through steel shear studs that are welded to the undamaged portions of the web. The UHPC panel restores bearing/shear capacity by providing an alternate load path through the UHPC panels for shear and bearing forces and protects the underlying steel from further corrosion.

C1.2

This repair is the result of a collaboration between the Connecticut Department of Transportation and the University of Connecticut through a three-phase research program spanning five years.

Phase I of the research developed a proof-of-concept of the efficacy of repairing corroded girders with UHPC [1]. A series of half-scale experiments were conducted to determine the undamaged, damaged, and repaired capacity of rolled steel beams. Corrosion was simulated on the damaged and repaired specimens using machining to provide a uniform thickness reduction. This research demonstrated that a 33% reduction in web thickness resulted in a 75% reduction of bearing capacity. The web buckling failure of the damaged specimen was limited to the reduced section. The undamaged specimen failed in traditional web buckling. A partial height UHPC repair on the girder end was able to exceed the capacity of the undamaged girder and shift the failure from end buckling to flexural yielding. Finite element models were validated with test data and used for a parametric study on full size girders.

This repair provides many benefits when compared to the conventional method of restoring section loss by adding steel cover plates, followed by painting. The noted impermeability of UHPC means that the underlying steel will be protected against future corrosion. The high compressive strength means that the failure of the UHPC will not control the design, and so the design shall focus on the shear studs. The ability of the UHPC to gain strength quickly allows for the roadway to be reopened quickly.

Phase II of the research developed design methods and investigated the effectiveness of the repair. Full scale push-off tests evaluated procedures to weld shear studs on both sides of a thin web [2], and the overall design and configuration of the repair [3]. Results from the push-off tests using UHPC established limits for stud size, spacing, cover, surface preparation, and eccentricity. A formulation for stud capacity based on stud diameter was developed for use in design. The research also verified the superior performance of UHPC over High Strength Concrete (HSC). Full scale tests of full-height and partial-height repairs on plate girders were conducted to examine performance. The pattern and level of damage on the girders were informed by inspection report data. In addition, two UHPC mix designs were evaluated. The corrosion damage in all specimens was limited to 5 inches of the web above the bottom flange and extended at least 2 inches past the end of the bearing. A 66% loss of the web and 50% loss of the bearing stiffener was targeted. The results indicated that each repair could reach a bearing capacity 30% higher than the design capacity with the recommended shear stud configuration [3].

Phase III studied the implementation of the repair on an in-service bridge on a critical part of Connecticut's major highway system. The bridge crossed over a highly serviced rail line allowing minimal access. The repair was completed successfully, prompting the development of these guidelines to support a wider use of the repair method.

The critical component of the repair is the UHPC, an advanced composite concrete material with specially engineered properties. It has a higher compressive strength, typically greater than 22 ksi (150 MPa). This is achieved by having significantly smaller particles with a maximum size less than 0.02 inch (0.5 mm), and a water-to-cement ratio of 0.24 and below. This results in an extremely dense matrix and permeability usually below 100 coulombs. UHPC is self-consolidating and flowable due to the use of high-range water reducers and superplasticizers. Set accelerators may be added and can help the material gain over 10 ksi (69 MPa) within 12 hours. UHPC has a first cracking stress typically around 1.2 ksi (8.5 MPa) and can sustain tensile loads and form multiple cracks up to a tensile strain of around 2.5%. This is achieved by dosing the mix with small (0.0078 inch (0.2 mm) diameter and 0.5 inch (13 mm) long) high-strength (>250 ksi (1,700 MPa)) steel fibers, typically around 2% by volume. These fibers bridge microcracks as they form and allow multiple cracks to form. This is only a summary of the key properties of UHPC that make it an ideal material for this repair. There are more in-depth discussions on the constitutive materials, mechanical properties, structural design, and durability of UHPC [4] [5] [6] [7].

A series of push-off tests were performed on UHPC (both with and without fibers) versus traditional high strength (8 ksi [55 MPa]) concrete (both unreinforced and reinforced) to compare the performance and investigate whether this repair could be performed at lower cost with alternate materials [2]. The tests made it clear that fiber reinforced UHPC was the only material capable of performing this repair. Specimens cast with the unreinforced HSC and the UHPC without fibers failed due to splitting of the panel at a load about 70% of the fiber reinforced UHPC. Even when reinforced at 0.5% the HSC failed by spalling the panel rather than rupturing the shear studs. Being able to achieve full rupture capacity of the studs while maintaining a panel small enough to fit on the beam is important to this repair. Impermeability is another crucial aspect of the repair that traditional HSC is not able to provide. As steel corrodes, it expands in volume, this can lead to premature spalling of the concrete and failure of the studs. There are several properties of UHPC that make it necessary for this repair.

The design shall ensure that the repair does not make any other part of the structure vulnerable to corrosion. The design of the repair should be performed such that it does not allow the collection of water.

Allowing water to pool on any part of the structure, or on top of the panels if not full-height, as a result of implementing this repair could expose other parts of the structure to corrosion.

The designer should be aware of the implication of casting the repair under the ambient vibration of the roadway. UHPC is self-consolidating and flowable and relies on the uniform distribution of fibers to perform as expected. The designer may wish to provide means to help mitigate vibrations.

These design guidelines represent the current state of the art of this repair and incorporate findings from the most current research findings. The guidelines will require updating if future research alters current findings or enhances the technical recommendations presented here.

Research is ongoing with regards to several aspects of this repair. Alternate means of shear transfer, including high-strength threaded rod and UHPC lugs are being investigated. These may help to provide alternatives to shear studs if the studs are difficult to apply.

Situations may occur where corrosion extends along the interface between the bottom flange and web beyond the bearing stiffener. In these cases, the repair should be designed for both bearing forces and interface shear. These guidelines do not include provisions for the design or detailing of repairs that include flange studs.

Interface shear should be addressed by welding an appropriate number of shear studs to the bottom flange to address the reduction in the interface shear capacity.

1.3 CTDOT DESIGN DOCUMENTATION

These design guidelines for UHPC repair of corroded steel girder ends are intended as a supplement to the existing documentation on design, materials, and construction in the state of Connecticut.

C1.3

The current version of the AASHTO LRFD Bridge Specifications at the time of this document is the 9th edition (2020) [8].

1.3.1 Connecticut Bridge Design Guide

The CTDOT bridge design guide can be found CTDOT Bridge Design Webpage.

C1.3.1

The current version of the CTDOT Bridge Design Manual at the time of this document is the 2003 edition with 12/19 revisions [9].

1.3.2 Material and Construction Standard Specifications

The CTDOT Material and Construction Standard Specifications can be found on CTDOT Bridge Design Webpage.

C1.3.2

The current version of the CTDOT Material and Construction Standard Specifications at the time of this document is the 2020 edition [10].

2 GENERAL DESIGN INFORMATION

2.1 SCOPE

This section lists all terms unique to this repair, all abbreviations contained within the guidelines, and symbols included within the equations in the guidelines.

C2.1

2.2 TERMS

Attachment Stud	A stud that is not considered to resist either the static or fatigue load.
Load Bearing Stud	A stud that is considered to resist both the static and fatigue loads.
Panel	The block of UHPC that is cast over the beam
Repair	The rehabilitation of corroded girders using UHPC panels attached with shear studs welded primarily to the web.

C2.2

2.3 ABBREVIATIONS

AASHTO	American Association of State Highway Transportation Officials
ADTT	Average Daily Truck Traffic
AWS	American Welding Society
CTDOT	Connecticut Department of Transportation
DC	Dead Load of structural components and nonstructural attachments
DW	Dead Load of wearing surface and utilities
HSC	High Strength Concrete
ICR	Instantaneous Center of Rotation
IM	Dynamic Load Allowance
LRFD	Load Resistance Factor Design
UHPC	Ultra-High Performance Concrete

C2.3

2.4 SYMBOLS

$(ADTT)_{SL}$	Single lane average daily truck traffic (AASHTO LRFD Section 3.6.1.4) [vpd]
A	Constant, 1040.0 x 10^8 [ksi^{3m}]
A_{sc}	Area of the shear stud shank [$inch^2$ (mm^2)]
d_b	Diameter of the shear stud [inch (mm)]
h_b	Length of the shear stud [inch (mm)]
h_w	Height of the girder web [inch (mm)]
f'_c	Compressive strength of the UHPC [ksi (MPa)]
F_u	Ultimate strength of the shear stud [ksi (MPa)]
N	Number of vehicle cycles in fatigue life
N_s	Number of studs required for repair based on Section 4.3.2 of this guideline
N_{sf}	Final number of studs required for design
n	Number of fatigue cycles per vehicle crossing
P	Factored design load from Section 4.2.1 of this guideline
P_u	Ultimate capacity of the shear stud [kip (kN)]
t_w	Thickness of the girder web [inch (mm)]

C2.4

x_s	Distance from the end of girder to the bearing stiffener [inch (mm)]
Y	Design life of the repair life [years]
β	Modification factor for compression factor for the value of η {0.0822 [ksi] (0.0199 [MPa])}
η	Modification factor for stud capacity based on the weld collar
ρ	Conversion Factor ksi to MPa {6.89}
φ_b	Reduction factor for bearing
φ_c	Reduction factor for compression
φ_{sc}	Reduction factor for shear connectors
φ_v	Reduction factor for shear

3 DEFINING LOADS, LOAD FACTORS, AND LOAD COMBINATIONS

3.1 SCOPE

This section defines the minimum requirements for loads, load factors, and load combinations used for the design of the repair. These guidelines are not intended to design for extreme events.

C3.1

3.2 DEFINITION OF LOADS

Loads shall be defined according to the current AASHTO LRFD Section 3.

C3.2

3.3 LOAD FACTORS

Load factors are based on those given in the AASHTO LRFD Specifications.

C3.3

The load factors presented in this section are intended to create maximum bearing load in the girder for the discussed design scenarios. As a result, only a limited number of load combinations need to be considered. See commentary provided in Section 4.2.1 of these guidelines for guidance on factors that may contribute to the selection of the static design load.

3.3.1 Dynamic Load Allowance

Impact factors are based on those given in the AASHTO LRFD Specifications

C3.2.1

The impact factors presented in this section are the factors presented in the AASHTO LRFD Table 3.6.2.1-1.

3.3.2 Redundancy

No redundancy modifier is permitted to be used.

C3.2.4

No Redundancy modifier is permitted per CTDOT Design Code Section 3.1.2.

3.4 LOAD COMBINATIONS

This section specifies the definition of load combinations that may be used to define the loads which should be resisted by the repair design. Section 4.1 governs selection of design load.

C3.4

If the design loads or preexisting bearing capacity for the girder being repaired are known, they may be utilized in place of the calculations defined in this section.

3.4.1 Live Load Only I

The HL93 live loading is considered. This load combination considers the maximum shear of the design truck or tandem and lane load (LL, AASHTO LRFD Section 3.6.1.2) and the dynamic load allowance (IM, AASHTO LRFD Section 3.6.2), it is calculated using:

$$1.75(LL + IM) \qquad (3.4.1\text{-}1)$$

where:

$IM = 33\%$

The live load calculated as part of the original beam design may be used in place of this calculation.

C3.4.1

This simplification of the live load is intended to identify the most common and critical loads that would be expected to be encountered at the beam end. These are the loads directly experienced by traffic. The braking force is not considered as this acts horizontally.

In the event that the live load calculations are available from the initial design, they may be used unless the loading conditions have changed since the initial construction.

3.4.2 Strength I

This load should be determined according to the load

C3.4.2

factors in Strength I AASHTO LRFD Table 3.4.1-1 considering the live load and dead load components.

The live load utilized shall be that defined in Section 3.4.1.

This simplification of the live load is intended to identify the most common and critical loads that would be expected to be encountered at the beam end. These are the loads directly experienced by traffic.

The dead load shall be the contribution of DC and DW, as defined in the AASHTO LRFD Section 3.5.1.

This simplification of the dead load is intended to identify the most common loads expected to be encountered at the beam end. These are the loads from the structure and wearing surface.

The Strength I load calculated as part of the original design may be used in place of this calculation.

In the event that the Strength I calculations are available from the initial design, they may be used unless the loading conditions have changed since the initial construction.

3.4.3 As-Built Capacity

The As-Built Capacity should be the lowest capacity of the as-built girder dimensions as defined in AASHTO LRFD Section 6.10.9, supplemented by AASHTO LRFD Section 6.9.4.1.2 and 6.10.11.2 if bearing stiffeners are present or AASHTO LRFD Section D6.5 if bearing stiffeners are not present.

C3.4.3

This load is intended to satisfy that the entire initial capacity of the girder can be carried by the repair. This is the most conservative of the options and assumes that the web is incapable of carrying any load.

3.4.4 Fatigue Load

This load should be determined in accordance with AASHTO LRFD Section 3.6.1.4. Fatigue I or II should be considered based on the $(ADTT)_{SL}$ of the bridge per standard practice.

C3.4.4

4 DESIGN METHOD

4.1 SCOPE

This section defines the selection of the design load category to be used, the design capacities for the repair, limits on stud sizing, as well as spacing and cover requirements.

The capacity design of the studs will assume full plastic capacity of the shear studs. All limits specified, as a result, will be based on this assumption.

The repair shall be designed for Static Design Load as noted in 4.2.1 and Fatigue Load as noted in 4.2.2 of these guidelines.

C4.1

4.2 DETERMINING DESIGN LOAD

This section shall define the load combinations found in Section 3.4 based on the design considerations determined by the designer.

C4.2

The static capacity defines the number of studs required for the repair. The designer selects the category for static design load based on the needs of a given project.

The commentary for the subsequent section will help to describe what each design load is considered to resist. This will help the designer select the most applicable design load for the project.

In addition to the static design loads, the controlling Fatigue I or II shall be checked for all designs.

4.2.1 Static Design Load

There are three categories for static design load: Live Load Only I, Strength I, and As-Built Capacity. The designer selects the category for design. The controlling Fatigue limit sate (Fatigue I or II) must be checked for all designs.

The loads selected shall be resisted by the capacity of the shear studs defined in Section 4.3.2 of these guidelines.

If the repair should carry only the live load, load provision of Section 3.4.1 of these guidelines should be used.

If the repair should carry the entire design load, the load provision of Section 3.4.2 of these guidelines should be used.

If the repair should support the original designed

C4.2.1

This section will describe the considerations for using each of the three static design loads. This should be used to inform the selection of the design load to adequately support the present loads, have the proper longevity to avoid future repairs, and provide the best economy possible. Fatigue will be discussed in the following section.

The Live Load Only I category represents only the active live loading being resisted by the repair. It is assumed that the remaining section of the damaged girder resists the dead load. This could represent a girder where the corrosion is not very severe and there are no signs of local buckling or distress, a rural bridge that does not experience heavy or frequent loading, or a repair with a short design life. Of the three design loads, it is the least conservative. It is applicable in many situations, particularly if there are no signs of distress in the girder.

The Strength I category represents the repair carrying both the live and the dead load. This assumes that the web of the girder will no longer need to carry any structural load. This could represent a girder that has sustained enough section loss that visible deformations are present in the corroded section, routes that experience a larger volume of traffic, or repairs with a longer design life.

The As-Built Capacity category represents the repair capacity, the load provision of Section 3.4.3 of these guidelines should be used.

carrying the entire original bearing capacity of the section before it was damaged by corrosion. This could be utilized on a highly damaged girder, on a bridge with a significant level of traffic, or on a repair that is expected to be in service indefinitely. This is the most conservative of the three design loads.

4.2.2 Fatigue Load

Fatigue I and Fatigue II loads shall be calculated in accordance with Section 3.4.4 of these guidelines.

The determination of whether Fatigue I or Fatigue II controls is based on the $(ADTT)_{SL}$ threshold of 11,320.

- $(ADTT)_{SL} \geq$ 11,320, Fatigue I load combination and factored shear resistance shall be used.
- $(ADTT)_{SL}$<11,320, Fatigue II load combination shall be used. The design life of the repair should be calculated using the required number of studs from the Live Load Only I, Strength I, or As-Built Capacity.

C4.2.2

Of the limited studies on the fatigue performance of UHPC [11] [12] [13] [14] [15], UHPC has been shown to have superior fatigue performance when compared to conventional concrete. Until results demonstrate that there is a concern for the fatigue performance of UHPC over the base metal and studs in this repair, fatigue shall be governed by these provisions.

This threshold is to comply with 10th edition of the AASHTO LRFD Bridge Design Specifications that is forthcoming, Section 6.10.10._ *Fatigue Resistance*.

4.3 SHEAR STUD REQUIREMENTS

This section provides guidance on the required size of studs, their static resistance, number of studs to be used in the design, and fatigue life calculation.

The standard installation of studs should be on the undamaged portion of the web using a standard stud welding gun.

C4.3

The designer will ultimately decide the size, length, and positioning of the studs that satisfy the requirements herein.

The undamaged web is the preferred location of stud installation, this ensures a direct load path from the panel to the bearing without having to consider condition of the stiffener. The use of a stud welding gun is the preferred method of stud installation. The weld collar at the base of the stud is a critical part of the repair in terms of generating capacity and ensuring that the stud is adequately fixed to the beam. More research is required on alternate means of welding shear studs in this repair before alternate means are endorsed. Section 4.3.4 of these guidelines contains information on alternative locations and means of welding studs to the beam.

4.3.1 Determining Stud Size

Maximum stud size should be defined based on the thickness of the web. The maximum length of the shear studs should be governed by their diameter.

C4.3.1

4.3.1.1 d_b/t_w Ratio

The stud diameter and the web thickness should satisfy the ratio of $d_b/t_w \leq 2.0$.

C4.3.1.1

A series of finite element simulations were performed to investigate the effects of diameter on the capacity and performance of studs welded to thin webs embedded in UHPC [2]. It was found when the d_b/t_w ratio exceeded 2.0, yielding or buckling of the web controlled the capacity rather than rupture of the stud. This ratio is put in place to limit the stress in the web so the full rupture capacity of the shear studs is maintained.

It is critical that stud rupture control the design. If web yielding or bucking controls, a variety of issues are introduced when considering the design of the repair. First, the calculated stud capacity will not represent the capacity calculated and may lead to premature failure of the repair. Second, if the studs become too large, the fatigue performance may decrease due to the increase in the stress in the plate relative to the stress in the stud. The fatigue design is limited by the stress in the stud, but the expected failure is from the toe of the weld through the base metal. If the stress is higher in the base metal relative to what is expected, the fatigue life will be effected.

4.3.1.2 h_b/d_b Ratio

The length of the stud and the diameter of the stud should satisfy the ratio of $h_b/d_b \geq 5.0$.

C4.3.1.2

This limit is to comply with 10th edition of the AASHTO LRFD Bridge Design Specifications that is forthcoming., Section 6.10.10.1._. Testing performed utilized h_b/d_b ratios of smaller than 4.0 to investigate the worst-case scenario [2]. Until a more thorough investigation on the effect of stud length on capacity is performed, the AASHTO limit should remain in place.

4.3.2 Stud Capacity

This Section contains the static capacity of the studs.

There are two classifications for studs: load-bearing studs and attachment studs.

Load-bearing studs are counted as supporting the static load.

Attachment studs hold the panel to the girder and should not be counted towards the static capacity.

C4.3.2

There are two major considerations for determining the classification for the studs: weld collar quality/welding process and the location of the stud. These considerations are explained in Sections 4.3.4 of this guideline.

Ideally, all studs should be load-bearing studs, however girder geometry and the field welding process may prevent this. For this reason, the other classification was created.

4.3.2.1 Static Capacity

The factored resistance of one shear connector embedded in UHPC is calculated using the following equation:

$$P_u = \varphi_s P_n \quad (4.3.2.1\text{-}1)$$

where:

$$P_n = 0.7 A_s Fu \quad (4.3.2.1\text{-}2)$$

φ_{sc} = shear connector resistance factor, 1.0
A_{sc} = cross-sectional area of the stud shank [inch^2 (mm^2)]
F_u = ultimate strength of stud [ksi (MPa)]

C4.3.2.1

The stud capacity equation is consistent with the 10th edition of the AASHTO LRFD Bridge Design Specifications. This equation was modified from that developed during the research study for consistence with AASHTO LRFD. The AASHTO LRFD equation is simpler to use and produces similar results that are slightly more conservative.

The equation for the nominal stud capacity developed during research (Eq. C4.3.2.1-2) includes two terms two terms: the first is the rupture capacity of the stud, and the second is the bearing strength of the weld collar bearing on the UHPC. The designer may select to remove the latter factor to be conservative, although it has been well-documented by welded shear studs embedded in UHPC [16] [17] [18]. Push-out tests performed led to the development of an equation for η based on the compressive strength of the UHPC [2].

The factored resistance of one shear connector embedded in UHPC is calculated using the following equation:

$$P_u = \varphi_{sc} P_n \qquad \text{(C4.3.2.1-1)}$$

where:

$$P_n = A_{sc} F_u + 0.16 \eta f'_c d_b^2 \qquad \text{(C4.3.2.1-2)}$$

$$\eta = \beta f'_c - 1 \qquad \text{(C4.3.2.1-3)}$$

φ_{sc} = shear connector resistance factor, 0.85
A_{sc} = cross-sectional area of the stud shank [inch^2 (mm^2)]
F_u = ultimate strength of stud [ksi (MPa)]
f'_c = compressive strength of UHPC [ksi (MPa)]
d_b = diameter of shear stud [inch (mm)]
β = 0.082 [ksi] (0.012 [MPa])

4.3.2.2 Fatigue Resistance

4.3.2.2.1 Fatigue I

The fatigue shear resistance shall be calculated in accordance with AASHTO LRFD Eq. 6.10.10.2-1.

4.3.2.2.2 Fatigue II

When Fatigue II controls, the design life of the repair should be checked using:

$$Y = \frac{N}{365n(ADTT)_{SL}} \qquad \text{(4.3.2.2.2-1)}$$

where:

$$N = \frac{A}{\left(\frac{P}{N_s A_{sc}}\right)^m} \qquad \text{(4.3.2.2.2-2)}$$

Y = design life of the repair [years]
n = number of stress range cycles per truck passage from AASHTO LRFD Table 6.6.1.2.5-2
$(ADTT)_{SL}$ = single-lane ADTT as specified in AASHTO LRFD article 3.6.1.4
A = constant, 1040.0 x 10^8 [ksi^{3m}]
P = factored design load from Section 4.2.1 of these guidelines
N_s = number of studs required for repair based on

C4.3.2.2

C4.3.2.2.1

C4.3.2.2.2

This approach is to comply with 10th edition of the AASHTO LRFD Bridge Design Specifications that is forthcoming, Section 6.6.1.2.5.

4.3.2.1 of this guideline.
A_{sc} = cross-sectional area of the stud shank [$inch^2$]
m = Fatigue growth constant, 5

Additional studs may be used to extend the fatigue life of the repair by reducing the stress range in each stud.

4.3.2.3 Increase Factor

A minimum increase factor of 1.2 shall be used unless otherwise approved by the department. This factor may be increased at the discretion of the designer.

$$N_{sf} = 1.2N_s \qquad (4.3.2.3\text{-}1)$$

where:
N_s = number of studs required for repair based on 4.3.2 of this guideline.
N_{sf} = final number of studs required for design.

C4.3.2.3

An increase factor is included to account for uncertainties in the field application of studs and field implementation errors. This includes, but is not limited to, challenges with shooting studs on vertical surfaces, imperfections in the base metal due to inadequate surface preparation, residual paint, or the presence of rust material, geometric constraints, or access complexities. The challenges noted are based on limited field observations by the research team during two implementation efforts in CT.

4.3.2.4 Minimum Number of Studs

Each panel should have a minimum of four studs. If there are fewer than four load-bearing studs, connectivity of opposite panels should be provided to ensure a balanced distribution of stresses in the panels.

C4.3.2.4

A load-bearing stud is a stud that has a fully intact and complete weld collar from a standard welding gun procedure. Non-load-bearing studs, or attachment studs, are only intended to hold the panel onto the girder and are not counted to carry load. Some panels, for example those between a double bearing stiffener, those with high skew, or those with a large extent of corrosion damage, may not be able to have studs properly welded, or may not permit a stud gun to properly access the web. In these cases, some welds may prove inadequate or may require that the studs be stick welded. Section 4.3.4 in this guideline discuss alternate welding of studs.

The situation where severe corrosion effects stud placement is very common, particularly in front of the bearing stiffener. For situations where severe corrosion exists, the highly damaged portion of the web and/or stiffener may be cut out to provide connectivity of the panels on both sides of the plate to ensure a balanced distribution of stresses.

4.3.3 Eccentricity Effects

4.3.3.1 Single Sided

Application of this repair to a single side should not be performed.

C4.3.3.1

Single sided repairs are not permitted as the underlying steel remains exposed to corrosion, which can compromise the web. Continued corrosion on the web could undercut the studs and cause the failure of the repair to shift from the rupture of the shear studs to the yielding or buckling of the web below the studs.

4.3.3.2 Double Sided

The designer should position the centroid of the applied studs over the bearing plate of the girder.

If the centroid of studs is not positioned over the bearing plate, the designer should account for the effects of eccentricity.

C4.3.3.2

This limit is put in place to limit the potential eccentricity in the repair. By positioning the centroid over the bearing, if possible, the force in the studs will be centered over the bearing and induce no rotation.

When the centroid of the studs applied does not fall over the bearing plate, a torsional moment is induced that increases the forces in each stud applies. Push-off tests have been performed on two levels of eccentricity: 2 inch (50 mm) and 4 inch (100 mm) using 0.5 inch (12 mm) studs [2]. It was noted that the capacities of these configurations were 92% and 0.33% of the centered configuration, respectively. The instantaneous center of rotation (ICR) method was used to determine the stud capacity when experiencing this torsion. The test configuration used did not account for the influence of stiffeners to limit rotation induced forces. The designer must account for the effects of eccentricity when they are present if they cannot design the repair as centric.

4.3.4 Alternative Means of Stud Attachment

The standard installation of studs should be on the undamaged portion of the web using a standard stud welding gun.

If stud welding gun cannot be used on certain portions of the girder due to access constrains, studs may be fillet welded. When using fillet welding, the weld shall be sized to ensure failure of the stud shank rather than the weld. The AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* shall apply.

Minimum weld size for the studs shall be governed by AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*, Table 7.2.

If enough studs cannot be placed on the web, studs may be shot on bearing stiffeners. Studs placed on the stiffeners may be considered load bearing.

If additional attachment studs are required, studs may be attached to the bottom flange. Studs placed on the bottom flange shall not be considered load-bearing studs.

C4.3.4

Attachment of studs using a standard stud welding gun is preferred as all experimental studies for the repair had studs welded in this manner. There have been no tests to date performed on the static resistance of shear studs fillet welded to thin webs encased in UHPC.

Fillet welding may also be used for attachment studs and to touch up studs with inadequate weld collars in accordance with the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

For studs $^{3}/_{8}$ inch to 1 inch, the minimum size of a fillet weld is defined as $^{5}/_{16}$ inch.

In cases where geometry or severe corrosion prevents applying studs to the web using a stud gun, stiffeners may be considered as welding surfaces. This option is intended to be used for a minimal number of studs to supplement the studs welded on the web. Studs on the stiffener are expected to be applied using the standard installation procedure with a stud welding gun. When applying studs on both the web and stiffener, clearance of the studs on perpendicular planes should be considered.

As studs placed on the flanges would not be a part of the load path, they cannot be counted towards resisting the load.

Limited research has been performed on alternate means of attaching the UHPC panel to the beam, including high-strength threaded rod and UHPC dowels. At this point there is insufficient data to recommend the use of any means of shear transfer other than shot shear studs.

4.4 STUD LAYOUT

4.4.1 Minimum Spacing

The minimum spacing of the studs should be four stud diameters ($4d_b$), if smaller spacing is required, the minimum spacing shall be three times the stud diameter ($3d_b$) or 1 inch (25 mm).

Studs on perpendicular surfaces, such as the stiffener and web, minimum spacing should be four times the stud diameter ($4d_b$), if smaller spacing is required, the minimum spacing shall be three times the stud diameter ($3d_b$) or 1 inch (25 mm).

4.4.2 Maximum Spacing

Maximum spacing between studs should be 6 inches (152 mm).

4.4.3 Stud Orientation Between Sides

Studs should be staggered between the two sides of the web.

4.5 DETERMINING PANEL SIZING

The UHPC panel should be sized to facilitate the installation and to satisfy the cover requirements in all directions to prevent premature localized failure of the concrete around the studs.

C4.4

C4.4.1

Minimum spacing requirements are based on prior research [2] and spacing requirements given in AASHTO/AWS Bridge Welding Code Section 7.4.5.

Force transfer from the stud to UHPC is dispersed in a conical zone of increased stress. As spacing between adjacent studs becomes smaller, the likelihood of interaction of their zones increases. As these high-stress zones interact, premature failure of the concrete may occur before the full rupture capacity of the studs are reached. The minimum spacing limitations given in this section ensure the full rupture capacity of the studs.

Staggered spacing is defined where there is both a vertical and a horizontal space between adjacent studs. Push-off tests have been performed with staggered spacing as small as two stud diameters ($2d_b$) [2]. Spacing this narrow may lead to issues fitting the stud welding gun to achieve a proper weld.

Non-staggered spacing is defined where there is only a vertical or a horizontal space between adjacent studs. Push-off tests have been performed with staggered spacing as small as three stud diameters ($3d_b$) [2]. Spacing this narrow may lead to issues fitting the stud welding gun to achieve a proper weld.

This spacing is to help to facilitate the welding of all studs between adjacent surfaces.

C4.4.2

This spacing is intended to avoid overstressing a stud that may be isolated from the remaining group of studs.

C4.4.3

Staggering studs on alternate sides of the web is recommended to reduce the stress present on the web. Having a pair of studs welded back to back may negatively affect fatigue life by increasing the relative stress on the underlying web.

C4.5

Due to the strength of the UHPC, the stress in the panel is not of concern when compared to the capacity of the studs, so long as sufficient cover is provided.

The UHPC panel need not occupy the entire width of the bottom flange nor extend the full height of the web. Tests have been successfully performed on panels that were both full and partial height [3]. For optimal protection from further corrosion, it is suggested that the panel width be close to the bottom flange width, and the panel height extend to the top flange.

The designer selects the panel size. The contractor selects means of forming the panels to ensure the panel shall reach the full design height

4.5.1 Side Cover

Side cover should be a minimum of four stud diameters ($4d_b$).

C4.5.1

Push-off tests for side cover ratios of two and four stud diameters ($2d_b$ and $4d_b$) [2] showed that studs with four diameters ($4d_b$) side cover were capable of obtaining full capacity as defined by AASHTO. Those with only two stud diameters ($2d_b$) side cover failed by splitting of the UHPC.

4.5.2 Top Cover

The cover at the top of the panel should be a minimum of six stud diameters ($6d_b$).

The distance between the stud and the top of the top of the corrosion damaged web should be a minimum of four stud diameters ($4d_b$).

C4.5.2

The limit for top cover is taken as 150% the side cover. This increase accounts for two aspects that can reduce the capacity. The first is that the force acting to fail the cover is the full bearing force of the stud, rather than the radial expansion caused by the Poisson effect as is with side cover. The second aspect is air voids that may be present in the top of the UHPC from the casting process. UHPC forms a surface skin soon after casting that may trap air. This reduces the cover of the stud. Additional testing should be performed to verify this cover and identify the failure mode.

The limit for offset from the damaged section of the web was taken as the same as that of side cover. This is intended to provide for load dispersion through the web before encountering the damaged portion of the web. If viable weldable area is limited, this may be relaxed at the discretion of the designer to expand the weldable area.

4.5.3 Clear Cover

The clear cover is defined as the distance from the end of the shear stud head to the face of the UHPC. The clear cover should be the larger of two times the fiber length or 1 inch (25 mm).

C4.5.3

Push-off tests with a minimum cover of 0.25 inch (6 mm) on 0.5-inch (12-mm) shear studs achieved full rupture capacity [2]. However, to ensure that the UHPC can flow through the repair without fibers clumping between the studs and the formwork, a clear cover of two times the fiber length used. Standard fibers are 0.5 inch (12 mm) long, so the standard clear cover would be 1 inch (25 mm).

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