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Maintenance-Free Corrosion-Resistant Steel Plate for Bridges

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Final Report VTRC 19-R21

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This study compared the fabrication requirements, corrosion resistance, mechanical properties, construction components, and cost of using traditional bridge steels and corrosion-resistant steels. Comparisons were made based on existing literature, discussions with and knowledge gained from industry experts, experimental testing, and field visits.

Types of corrosion-resistant steel plate included in the study were galvanized weathering steel; steel plate meeting the chemistry requirements of ASTM A1035CS (and AASHTO M334M, Alloy Type 1035 CS); ASTM A709 Grade 50CR steel; and duplex stainless steels, such as Grades 2101, 2202, 2304, and 2205.

The results showed that the galvanized weathering steel had performed well on a bridge for 6 years without any major issues. The steel plate meeting the chemistry requirements of ASTM A1035CS showed good corrosion resistance and material properties, and further research is recommended to determine its suitability for use as a steel bridge material. The ASTM A709 Grade 50CR steel showed good tensile and slip-critical bolted fatigue behavior that met the requirements of ASTM A709 Grade 50 steel. The study also showed that it can be successfully fabricated into a steel plate girder and has had good corrosion resistance in bridge applications.

The study recommends that the Virginia Department of Transportation (VDOT) consider further implementation of ASTM A709 Grade 50CR steel and that the Virginia Transportation Research Council (VTRC) work with the VDOT districts on two bridge projects using the steel. The duplex stainless steel showed excellent mechanical properties and has been successfully designed and fabricated for bridge use in the United States and worldwide. It is recommended that VTRC initiate a study comparing the mechanical and corrosive properties of dissimilar metal welded connections of ASTM A709 Grade 50CR and duplex stainless steels to those of conventionally used steels.

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(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

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FINAL REPORT

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INTRODUCTION

The Virginia Department of Transportation (VDOT) has transitioned from the use of coated reinforcing steels in bridge deck construction to uncoated corrosion-resistant alloyed steel in order to reduce maintenance costs associated with corrosion in bridge decks. The benefit of such a change is most clearly seen in the Progresso Pier in Mexico, which has shown that corrosion-resistant steel can successfully perform in a highly corrosive environment (RAMBØLL Consulting Engineers and Planners, 2007). VDOT's change in materials use demonstrated that one of the primary factors for selecting steels should not be solely initial costs but instead should include up-front costs plus future costs associated with steel maintenance operations. This is often calculated using life-cycle cost analysis and is especially relevant when minor changes in material costs could significantly reduce maintenance costs. VDOT is currently expanding this research effort by looking at more corrosion-resistant prestressing and post-tensioning strands with increased corrosion resistance. Using corrosion-resistant material has also expanded into the potential for steel bridges and prompted the following question: Could corrosion-resistant steel plate (CRP) be used to provide steel plate girder bridges with increased durability that would mitigate future maintenance costs? A sweep of the current literature and research efforts in Virginia and other states indicated that the answer is clearly "Yes."

Traditionally, VDOT has used painted, galvanized, or weathering steel as a means of corrosion protection for steel plate girder bridges; each has its advantages and disadvantages. Painting has a low initial cost, but paint systems (typically inorganic zinc-rich primer, epoxy mid-coat, and urethane top coat) have expected service lives of approximately 20 to 30 years before recoating is required, so several recoating operations are required to meet service life expectations.

Galvanized steel can perform well in atmospheric environments, but much of its corrosion resistance depends on the quality of the galvanizing process. Galvanizing baths are also limited in size, which limits the length of girders that can be galvanized. In many cases, this requires the designer to limit structural members to a length that can fit in a galvanizing bath. These size limitations can lead to additional members and splices, which increase the initial cost. There have also been some areas in Virginia, such as in parts of VDOT's Hampton Roads District, in which galvanized steel has not performed adequately. In cases such as these, where the galvanizing has failed, the corrosion rate of the steel is increased substantially until a coating is reapplied in the field.

Weathering steel, expected to cost minimally more than typical carbon steel, has proved successful in some areas but performs poorly in specific environments, such as in marine coastal areas, areas of prolonged wetness, or near industrial areas. Historically poor performance in such locations led to a Federal Highway Administration (FHWA) technical advisory, written in 1989, detailing areas in which uncoated weathering steel is not recommended (FHWA, 1989).

In the last decade, VDOT has begun employing creative solutions for additional corrosion protection. One such case was the Genito Road Project, which required the replacement of the old Route 604 Bridge over the Swift Creek Reservoir. The site had numerous physical and construction constraints, such as the bridge being located over a reservoir, a low water clearance of approximately 3 ft, and shallow superstructure depth requirements, because of the existing road grade. Because of geometric constraints, concrete girders were not an option. Traditional coated steel girders were not used because of concerns with deterioration of the paint over the reservoir. Neither weathering steel nor galvanized steel girders were selected because of potential corrosion concerns. This led to the decision to combine two corrosion-resistant mechanisms resulting in the fabrication and placement of galvanized weathering steel (GWS) girders on the bridge. Although these GWS girders have generally performed well to date, this demonstrates a need for additional corrosion-resistant steel options through means other than coatings.

Several years ago, Fletcher (2011) performed a study titled *Improved Corrosion-Resistant Steel for Highway Bridge Construction*. In this study, the corrosion resistances of several different steel plate products with different compositions were compared. One such material was ASTM A1010 (UNS S41003) steel, a low-alloy, cost-effective stainless steel that had originally been used in industrial applications such as coal hopper rail cars and salt spreader trucks. Because of its ferrite and tempered martensite microstructure, ASTM A1010 steel has good strength, toughness, and weldability (ASTM International [ASTM], 2013). Because of its substantially improved corrosion resistance relative to Grade 50 or 50W steel, the study's life cycle analysis showed that the ASTM A1010 steel plate material had a 90% chance of being more economical as compared to coated Grade 50 steel after 20 years in service and a 100% chance of being more economical after 40 years in service (Fletcher, 2011).

To date, ASTM A1010 steel has been used on a total of six steel bridges in the United States, including in California, Pennsylvania, Oregon, Iowa, and Virginia. In 2004, as part of the Innovative Bridge Research and Construction Program, ASTM A1010 steel was used in a multi-cell box girder design in Williams, California (Seradj, 2010). Although the bridge is not located near the coastline or exposed to de-icing salt, it is located in a foggy environment and is in close proximity to the water below the girders. These two reasons served as the rationale for selecting ASTM A1010 steel as the material of choice for the bridge. In 2007, a producer of the ASTM A1010 steel plate product used the material to fabricate the first plate girders to support a bridge at their steel mill in Coatesville, Pennsylvania. The Oregon Department of Transportation (ODOT) constructed two ASTM A1010 steel plate girder bridges, one in 2012 and the other in 2013. In doing so, ODOT made significant advancements in using ASTM A1010 steel as a more corrosion-resistant alternative to weathering steel. Work by ODOT has included investigations into the following (Seradj, 2010; Seradj, 2014; Seradj, 2015):

- groove welding (submerged arc welding process)
- fillet welding, single and double pass (submerged arc and flux-cored arc welding)
- accelerated corrosion testing
- machinability testing.

In 2016, the Iowa Department of Transportation constructed an overpass bridge in which two of the six plate girder lines (one exterior and the adjacent interior girder) were fabricated with ASTM A1010 steel; the remaining girders were fabricated with Grade 50W steel (Shuck-Britson Inc. and Iowa Department of Transportation, 2015). In 2017, VDOT constructed the Route 340 Bridge over the South River in Waynesboro using haunched steel plate girders of ASTM A1010 steel. The bridge marked the first time that ASTM A1010 steel was used in a haunched plate girder and the first time that ASTM A1010 steel was used for the secondary members. The Route 340 Bridge used stainless steel fasteners for all of the connections, including the bolted field splice, which was also a first for U.S. vehicular bridges (Provines et al., 2018).

In September 2017, ASTM A1010 steel was incorporated into ASTM A709, Standard Specification for Structural Steel for Bridges, as Grade 50CR (hereinafter “50CR”) steel, where “CR” stands for “corrosion resistant” (ASTM, 2017b). The addition of 50CR steel into ASTM A709 makes it easier for state departments of transportation to specify corrosion-resistant steel for future applications.

Aside from 50CR steel, higher alloyed, more corrosion-resistant, and higher strength duplex stainless steels have found use as structural elements in vehicular, pedestrian, and rail bridges. Europe has embraced the use of duplex stainless steel as primary structural sections for bridges. The world’s first vehicular bridge to be constructed of duplex stainless steel was the Cala Galdana Bridge in Spain built in 2005. Duplex stainless steel was selected for the primary structural beams because of the location’s marine atmosphere and the need for a longer service life. This led to other large European duplex stainless steel vehicular bridge applications, such as the arches and deck beams of the dual arch suspension Piove di Sacco Bridge in Italy built in 2006. Prior to that, the first duplex pedestrian bridge in Europe was the Suransuns Bridge, constructed in Switzerland in 1999, followed by the Millennium Bridge in the United Kingdom in 2001. The first European rail bridge to use duplex stainless steel for all of its primary structural members was the Añorga Railway Bridge in Spain, constructed in 2012.

In the United States, one major bridge application using duplex stainless steel was the Harbor Drive Pedestrian Bridge in San Diego, California, constructed in 2011 (Roads & Bridges, 2019). Duplex stainless steel was selected for this bridge because of the corrosive marine environment and aesthetics. The West 7th Street Bridge was constructed in Fort Worth, Texas, using Grade 2205 steel hangers, selected for their corrosion resistance, high strength, and pleasing aesthetics. There is much information and many specifications regarding the structural use of duplex stainless steel. The International Stainless Steel Forum (2016) maintains a website that provides additional information on structural applications of duplex stainless steel products. The *AISC Design Guide 27: Structural Stainless Steel* (Baddoo, 2013) was published by the American Institute of Steel Construction (AISC). It covers structural design procedures related specifically to duplex stainless steel. The Eurocode has also included the full range of stainless steel structural sections for some time, and the European Union recently funded a slip-critical bolted connection study.

Duplex stainless steels have also been shown to be cost-effective with regard to the cost of maintenance actions over the service life of a structure. A composite vehicular life cycle cost analysis was completed in 2012 using four scenarios and published U.K. maintenance cost data for painted carbon steel over a 60-year service life. It found that the total cost savings associated with using unpainted duplex stainless steel beams could range from 30% to 40% when compared to that of painted carbon steel.

VDOT has applications where CRP girders would likely be a competitive solution to concrete alternatives or where they are necessary to meet aesthetic requirements. One example is the use of CRP in targeted locations such as in beam ends under bridge joints that cannot be eliminated. Further, applications exist in Virginia and around the world where CRP has been successfully used as a structural component in bridge applications. As part of this study, the fabrication, corrosion resistance, mechanical properties, construction components, and cost associated with steel plate girders made from 50CR and duplex stainless steel plate were compared. These factors were compared to those of more traditional Grade 50 and galvanized Grade 50W steel girders.

PURPOSE AND SCOPE

The purpose of this study was to compare CRP girder materials to the traditional steels used by VDOT to determine CRP's suitability for bridge use in highly corrosive environments when both initial cost and maintenance costs of the life of the structure are considered. The comparison included fabrication requirements, corrosion resistance, mechanical properties, construction components, and cost analysis.

The scope of the study included a literature review, discussions with industry experts, experimental testing, and field visits to aid in the comparison of CRP materials and traditional steels; the CRP materials considered in the study are shown in Table 1.

Table 1. Traditional Versus CRP Girder Materials

Traditional Steel Girders	CRP Girders
ASTM A709 Grade 50	Galvanized ASTM A709 Grade 50W
ASTM A709 Grade 50W	ASTM A1035CS steel plate
ASTM A709 HPS Grade 50	ASTM A709 Grade 50CR
ASTM A709 HPS Grade 70	Duplex stainless steels

CRP = corrosion-resistant steel plate.

Currently, ASTM A1035 is limited to reinforcing steel (ASTM, 2016a). The ASTM A1035CS steel plate referenced in Table 1 meets the chemical requirements of ASTM A1035CS (and AASHTO M334M Alloy Type 1035 CS [American Association of State Highway and Transportation Officials (AASHTO), 2017b]) but has been manufactured into plate form. Based on earlier studies of reinforcing steel, A1035CS steel was known to have enhanced corrosion resistance, so plate material was included in this investigation to determine if it could be suitable for bridge use. In the remainder of this report, this steel plate material manufactured to meet the chemistry requirements of ASTM A1035CS is referred to as "A1035CS steel plate." The duplex stainless steels referenced in Table 1 include Grades 2101, 2202, 2304, and 2205, all of which were included in the current investigation.

METHODS

Five tasks were performed to compare CRP girder materials and traditionally used bridge steels:

1. The CRP fabrication potential was evaluated.
2. The corrosion resistances were compared.
3. The mechanical properties were compared.
4. The construction and cost were compared.
5. Case studies of bridges made from CRP were analyzed.

Task 1: Evaluation of CRP Fabrication Potential

Steel Plate Girder Fabrication

The Virginia Transportation Research Council (VTRC) identified several facilities capable of fabricating CRP girders. Much of this phase of the study focused on understanding the different welding parameters to ensure that a viable CRP girder could be successfully produced. In particular, available consumables and potential welding techniques during fabrication of CRP products were investigated. When 50CR steel was considered, the ODOT research (Seradj, 2010; Seradj, 2014; Seradj, 2015) provided initial guidance regarding that material. The work by the European Commission provided guidance for the duplex stainless steel products (Zilli et al., 2008). Further, producers of other CRP products and fabricators were contacted for information on the weldability, and they suggested welding consumables and procedures for their products.

Rolled Structural Shapes Availability

In addition to contacting steel producers about the availability of CRP, VTRC also contacted vendors to determine if rolled structural shapes are available in CRP. Rolled structural shapes, such as beams, channels, and angles, are generally considered necessary for bridges because they are used to build secondary members such as cross frames, diaphragms, etc.

Task 2: Comparison of Corrosion Resistance

Bridge Site Visits

During the study, two bridges were monitored, one constructed with CRP girders and one constructed with galvanized Grade 50W. The William G. Taylor Memorial Bridge in Coatesvillle, Pennsylvania, was constructed in 2005 using 50CR steel plate girders. The Route 604 Bridge over the Swift Creek Reservoir in Virginia was built in 2007 using GWS plate girders. A site visit was made to each bridge so that both could be assessed visually. The inspection records for the Route 604 Bridge were also reviewed and incorporated into the findings of this study.

PREN Comparison

An initial corrosion resistance comparison was conducted using the pitting resistance equivalence number (PREN) of the steels. The PREN is a common designation for estimating the relative corrosion resistance of a steel based on its chemistry; the PREN value is determined by the steel's chemical composition of chromium, molybdenum, and nitrogen. The PREN is valid only for stainless steels and does not indicate an actual relative corrosion resistance under varying atmospheric conditions.

Long-Term Corrosion Data

In addition to site visits and PREN comparisons, a review was conducted of two long-term corrosion studies in Panama (Southwell and Bultman, 1982) and North Carolina (Houska, 2014). Long-term atmospheric corrosion testing has been done in many locations around the world and is the most accurate way of comparing the actual corrosion loss of carbon and weathering steels to various stainless steels. Although these studies did not include 50CR and duplex stainless steels, steels with similar chemistry compositions were included and were used for relative comparison purposes.

Task 3: Comparison of Mechanical Properties

To characterize the mechanical properties of the different types of steel, tensile tests were performed on 50CR steel, A1035CS steel plate, and four grades of duplex stainless steel (Grades 2101, 2202, 2304, and 2205). The results of these tests were compared to those for Grade 50 steel to determine how the CRP materials differed in tensile behavior. Three thicknesses of

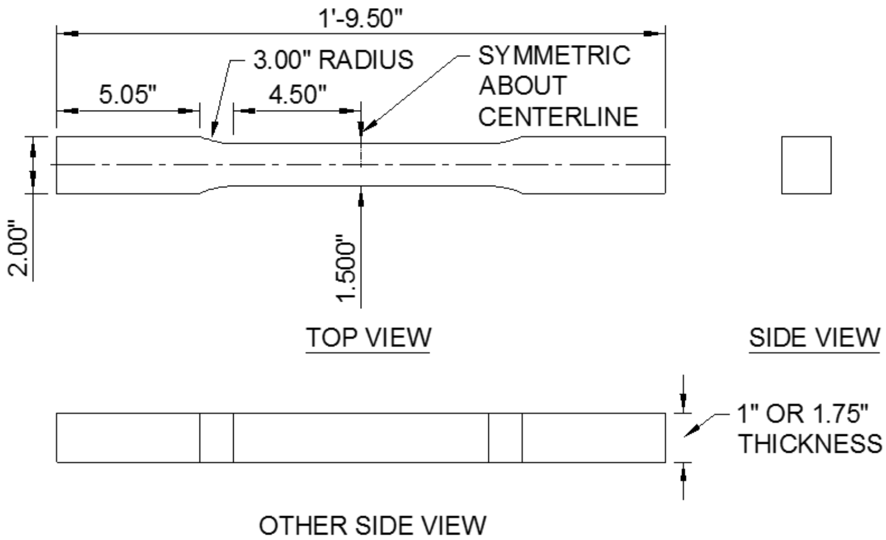
50CR steel plate were tested to investigate the effect of plate thickness on tensile strength. All tensile test specimens were tested in accordance with the ASTM A370 specifications for standard testing of metallic materials (ASTM, 2017a). The tensile tests were conducted in a variety of servo-hydraulic controlled test frames ranging from 55-kip to 550-kip capacity at either VTRC or FHWA's Turner-Fairbank Highway Research Center. Strain was measured using one of three methods: clip-on extensometer, laser extensometer, or video extensometer. The test frame and strain measurement system were selected based on the specimen size and site constraints. The tests were conducted under displacement control at loading rates in the ASTM E8 specifications (ASTM, 2016b). Stress-strain curves were plotted for each specimen to obtain mechanical properties such as modulus of elasticity, yield stress, ultimate strength, and elongation at fracture. Tensile test specimens were also used to evaluate the microstructure and fracture surface of the 50CR steel.

In addition to tensile tests, fatigue tests were performed on both bolted and welded 50CR steel plate specimens. The fatigue tests were conducted in closed loop servo-hydraulic load frames with capacities ranging from 110 kip to 220 kip. Tests were conducted under constant amplitude loading, with stress ranges selected to produce reasonable fatigue life failures and maintain elastic behavior (in the net section for bolted specimens). Both the load and number of cycles were recorded during testing. The fatigue life was evaluated in accordance with the fatigue design specifications in *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017a).

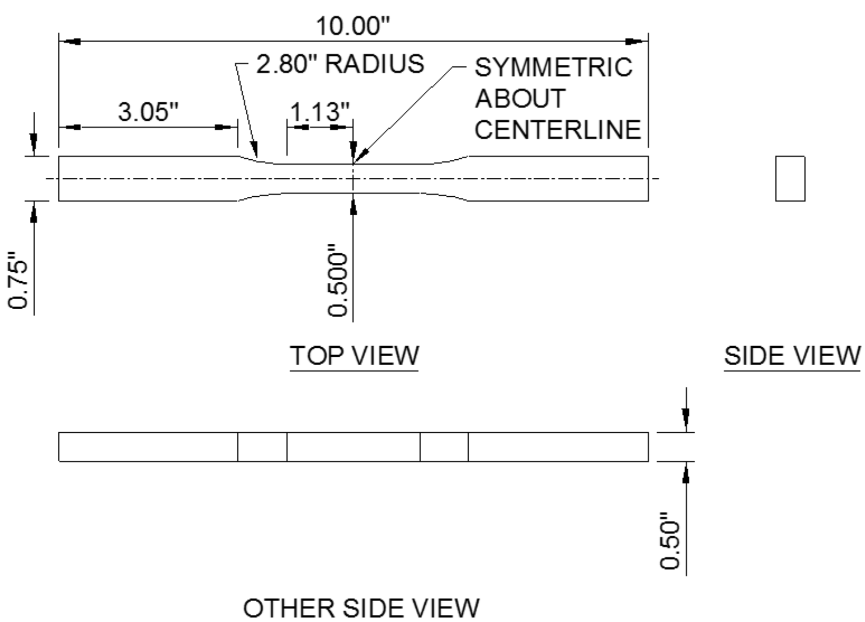
Uniaxial Testing of 50CR Steel Plates

Uniaxial tensile tests were performed on base material specimens cut from 50CR steel plates. In order to investigate the effect of plate thickness on tensile strength, specimens with three different thicknesses were tested: ½ in, 1 in, and 1¾ in. Dimensions for the 1- and 1¾-in-thick specimens are shown in Figure 1(a) and were designed as ASTM E8 plate-type standard specimens with a gauge length of 8 in. Figure 1(b) shows the ½-in-thick specimens, which were designed as ASTM E8 sheet-type standard specimens with a gauge length of 2 in.

The tensile tests were performed in accordance with Method C in the ASTM E8 specifications. The method specifies a crosshead rate of 0.015 in/in of the length of the reduced section per minute when determining yield properties and allows the rate to be increased to 0.05 to 0.5 in/in of the length of the reduced section per minute after the yield behavior has been recorded. Therefore, the speed of crosshead movement during testing of the 8-in-gauge-length specimens was set to 0.135 in/min until the yield point and was then gradually increased to 0.54 in/min until failure. Similarly, for testing specimens with a 2-in gauge length, the speed of crosshead movement was set to 0.034 in/min until the yield point and was then gradually increased to 0.135 in/min until failure. Since a video extensometer was used to record displacements and strains, a non-periodic, isotropic, and high contrast speckle surface pattern of black dots on a white background was applied on the specimens. Figure 2 shows an example of the tensile test setup.



(a)



(b)

Figure 1. Dimensions of Specimens: (a) 1-in- and 1¾-in-thick plate-type specimens; (b) ½-in-thick sheet-type specimens

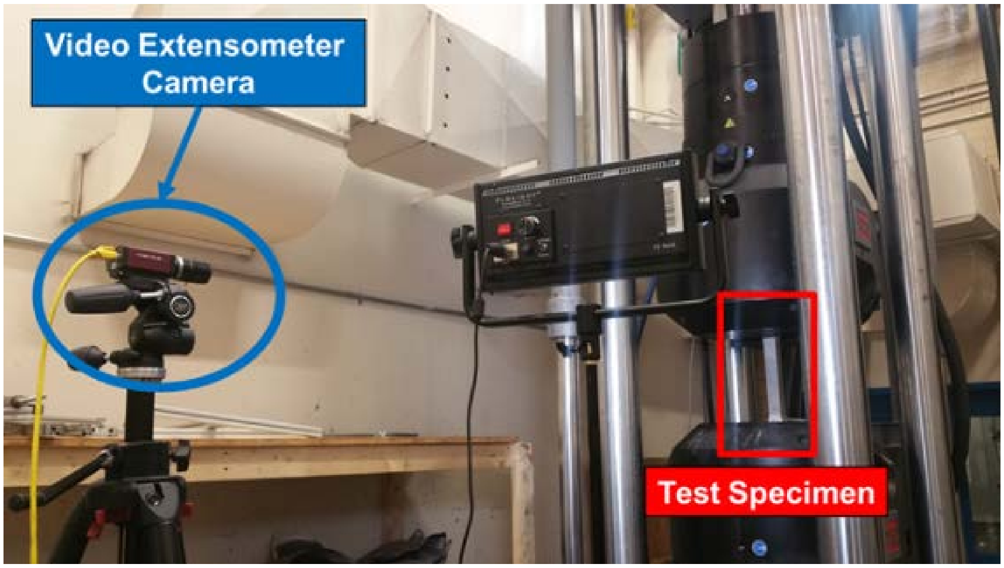


Figure 2. Tensile Test Setup

In addition to the specimens already discussed, samples made from 3-in-thick material meeting the chemistry requirements of 50CR steel were machined and tested. Although the current 50CR specification does not allow for plates more than 2 in thick, the material manufacturer has begun producing material at a thickness of 3 in while still meeting the specified chemical composition requirements. Since the 3-in-thick steel meets the chemistry requirements of the ASTM specification, it is referred to as "3-in-thick 50CR steel" material throughout the remainder of this report. In order to evaluate the strength capabilities of this 3-in-thick 50CR steel, specimens were machined into tension samples with dimensions as specified in ASTM A370 (ASTM, 2017a). This specification was used to make smaller and lighter weight samples than would have been required by the typical ASTM E8 specification. This was done to limit the overall weight of the bulk 3-in-thick material received since VTRC does not have a crane to lift heavy items. A drawing of the 3-in-thick 50CR steel tension samples is shown in Figure 3.

One specimen was oriented longitudinal (parallel) to the rolling direction, and one specimen was oriented transverse to the rolling direction. The 3-in-thick 50CR steel specimens were tested in a similar fashion as previously described for the other tension tests. Load, displacement, and strain over the specimen gauge length were recorded during testing.

Microstructure of 50CR Steel Tension Specimens

Samples of 50CR steel were cut, ground, and polished prior to the samples being etched with several different etchants and compared to 50CR steel microstructure images in a report by Fletcher (2011). In general, the use of etchants allows for a steel's microstructure to be viewed under an optical microscope. The etchants were selected with the knowledge of what features are preferentially affected by the etchant within a particular type of steel. The features and microstructure were then viewed using an optical microscope with bright-field illumination.

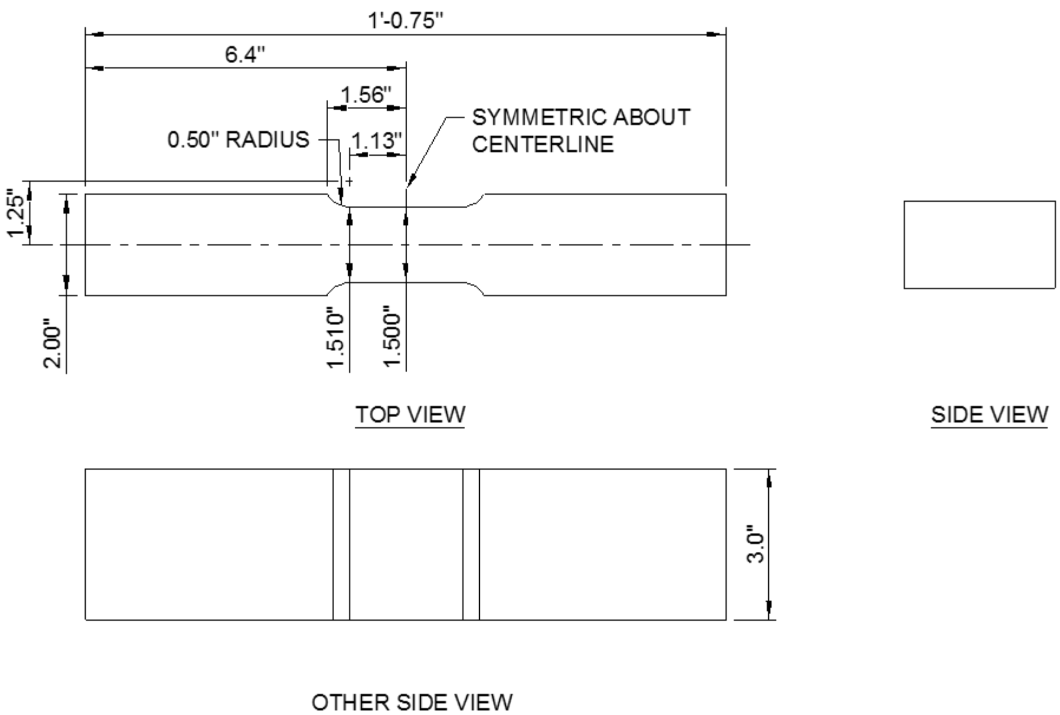


Figure 3. ASTM A709 Grade 50CR Steel Tensile Test Specimens: 3 in Thick

Although etchants such as 4% picral plus HCl, glyceregia, Ralph's reagent, and Fry's reagent have been used successfully to etch martensitic stainless steels and document microstructural features, for this work, Vilella's reagent was selected (ASM International, 2004). The composition and use for this etchant is provided in ASTM E407 (ASTM, 2015a) under Etchant #80. This etchant has been the most commonly used etchant for 50CR steel.

Fractography of 50CR Steel Tension Specimens

After the 50CR steel tension specimens had been tested, the fracture surfaces were examined using both visual inspection and a scanning electron microscope. Visual inspection was used to examine the specimen as a whole to look for any oddities that may have occurred. The scanning electron microscope was used to examine the fracture surfaces more closely to determine if the fracture was ductile or brittle in nature.

Fatigue Testing of 50CR Steel Bolted and Welded Connections

The bolted fatigue specimens were designed as slip-critical bolted joints in a double shear lap splice configuration, consisting of two ½-in-thick plates connected using two ½-in-thick splice plates with eight ⅞-in-diameter ASTM F3125 Grade A325 structural bolts. The bolts were tightened by the turn-of-nut method (Research Council on Structural Connections, 2014). Since all of the bolts were pretensioned, the specimens were expected to have a fatigue resistance equivalent to an AASHTO fatigue detail Category B (AASHTO, 2017).

Seven of nine specimens were cycled under a constant 30 ksi stress range; the remaining specimens were cycled under a constant 20 ksi stress range. Cycling at different stress ranges is common for experimental fatigue tests. In both cases, the minimum stress was 3 ksi. Stress ranges were calculated using the gross cross-sectional area of the connections because the connections were designed and constructed as slip critical.

During testing, it was common for one of the two splice plates to fail in fatigue before the other did. When this occurred, the plate that did not fail was matched with an untested splice plate and testing was resumed. Conducting the tests in this manner allowed for one stress range–number of cycles (S-N) data point to be produced per splice plate tested. The load and number of cycles were recorded until failure or until the test was stopped and was declared a runout. Figure 4 shows the geometry and test setup of the fatigue test of the 50CR steel bolted connection.

All of the welded fatigue specimens were fabricated from ½-in-thick 50CR steel plate. The specimens were designed in accordance with ASTM E466 (ASTM, 2015b). The specimens were prepared with a complete joint penetration (CJP) groove weld, which was welded using the submerged arc welding process. The welding parameters of the 50CR steel plate were set to an average of 370 amps, 32 volts, 16 in/min travel speed, 45 kJ/in heat input, and maximum interpass temperature of 300°F. The wire and flux used were Lincoln Blue Max ER309L and Lincoln 880M, respectively.

The welded specimens were cycled at an initial constant amplitude stress range; five specimens were started at a 20 ksi stress range, four were cycled at 15 ksi, and two were cycled at 11 ksi. All of the stress ranges had a minimum stress of 1 ksi. In some cases, where failure did not occur after at least 10 million cycles, the stress range was increased in the hopes of producing a fatigue failure within a reasonable time frame. In these cases, Miner's rule was used to calculate an equivalent constant amplitude stress range. The fatigue tests were cycled at a frequency of 7 Hz to 10 Hz. Test loads, displacements, and number of cycles were recorded by the testing machine.

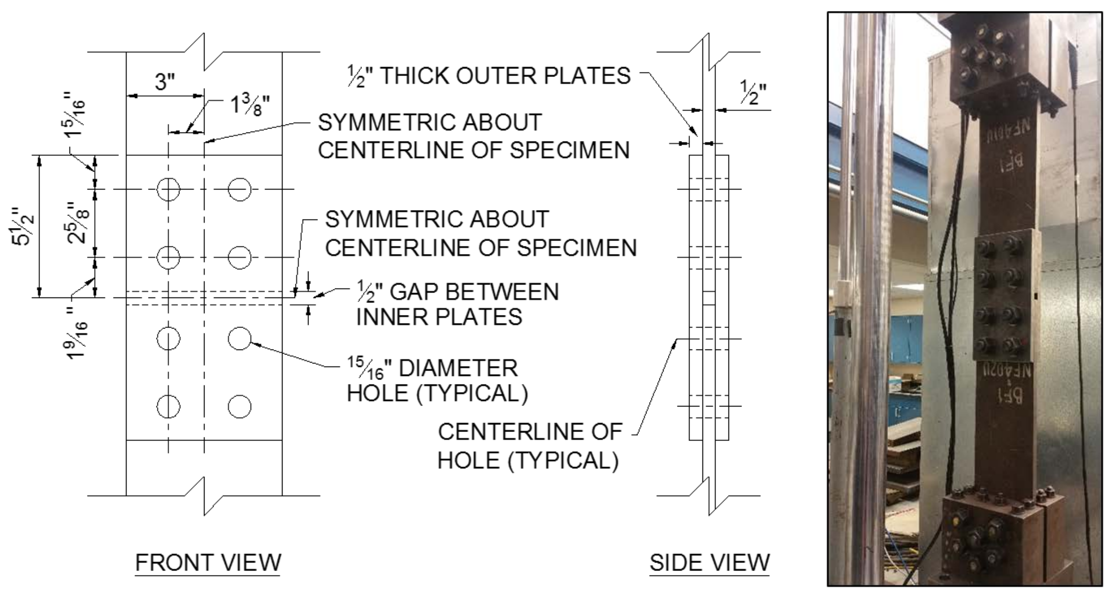


Figure 4. Geometry and Fatigue Test Setup of ASTM A709 Grade 50CR Steel Bolted Connections: left, geometry; right, test setup

Figure 5 shows the fatigue test setup of the 50CR steel welded plates. Unfortunately, after the samples were welded and the testing was conducted, the samples did not meet the AASHTO workmanship requirements to be considered fatigue detail Category B, as was originally intended.

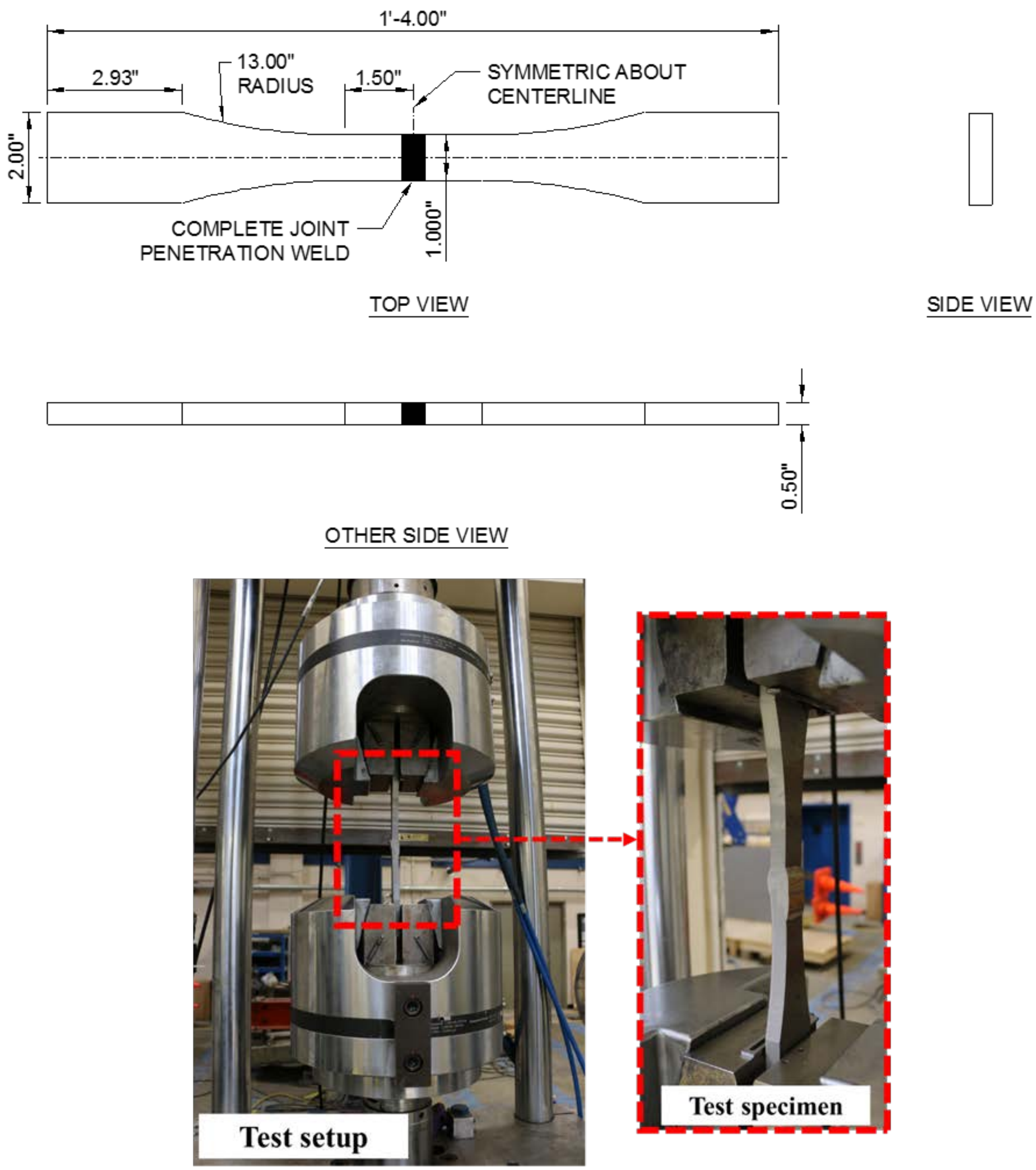


Figure 5. Geometry and Fatigue Test Setup of ASTM A709 Grade 50CR Steel Welded Specimens: top, geometry; bottom, test setup

Uniaxial Testing of A1035CS Steel Plate and Duplex Stainless Steel

Uniaxial testing was also used to evaluate several different types of stainless steel plate. The plate types evaluated are listed in Table 2 and include A1035CS steel plate and duplex stainless steels (Grades 2101, 2202, 2304, and 2205).

The stainless steel plates listed in the table were machined into tensile test specimens with the same dimensions as the 50CR steel plates shown in Figure 1(b) with one exception: tests were also conducted on ¾-in-thick Grade 2304 samples from a second steel supplier. For the duplex steels, three specimens were tested in both the longitudinal and transverse directions with respect to the plate rolling direction. For the A1035CS steel plate samples, four specimens were tested in the longitudinal direction.

Table 2. Types of CRP Evaluated and Minimum Tensile Properties

Material Type	Yield Stress (ksi)	Tensile Strength (ksi)	Minimum Elongation Over 2-in Gauge Length (%)
A1035CS steel plate	Unpublished[a]	Unpublished[a]	Unpublished[a]
Grade 2101	65	94	30
Grade 2202	65	94	30
Grade 2304	58	87	25
Grade 2205	65	95	25

CRP = corrosion-resistant steel plate.
[a] Published information exists for A1035CS reinforcing steel but not for steel plate.

Published Duplex Stainless Steel Charpy V-notch and Fatigue Data

Because of study time constraints, Charpy V-notch (CVN) or fatigue testing could not be conducted on the duplex stainless steel plate product. However, since both CVN and fatigue test results are integral to the performance of steel plate girder bridges, both properties were investigated through examination of published information.

Task 4: Comparison of Construction and Cost

Information on the CRP materials deemed important during the design phase of a project was gathered; this information included relative fabrication time, whether or not the material could meet the Buy America regulations, what type of fabricator would likely be best suited for fabricating a bridge with the respective material, and maximum available thickness (FHWA, 2017). The information deemed important was determined through discussions with the CRP producers and other organizations with extensive knowledge of duplex stainless steels.

A cost comparison of the plate materials was also conducted to determine the relative cost between typical bridge steels and CRP steels. The cost of the materials was based on the raw steel plate cost, not including any fabrication. The costs of traditional steels (Grades 50 and 50W) were determined using current market values; the cost of Grade 50CR steel plate was determined using information from the Route 340 Bridge; and the costs of the duplex steels were obtained through asking stainless steel plate producers for a cost estimate on the plate sizes and quantities used for the Route 340 Bridge. These cost data were obtained in the summer of 2018.

Using the plate quantities from the Route 340 Bridge allowed for a direct material cost comparison among traditional bridge steels and CRP materials.

Task 5: Case Studies

Numerous projects have used CRP materials for bridge applications in the United States and across the world. Several of these case studies were reviewed, and information related to the design, fabrication, and current condition of the bridge (if possible) were included in this report.

RESULTS AND DISCUSSION

Evaluation of CRP Fabrication Potential

Steel Plate Girder Fabrication

Currently, there are five steel plate girder bridges in the United States, including the Route 340 Bridge, fabricated using 50CR steel plate. At least four different U.S. steel bridge fabricators were used during the fabrication of these bridges. This demonstrates that 50CR steel plate girders can be successfully fabricated by several steel bridge fabricators.

One significant part of steel plate girder fabrication is the consumables used during the welding process. ODOT had conducted a weldability study of 50CR steel (Seradj, 2010), which provided guidance for VDOT during the fabrication of the Route 340 Bridge. To date, all of the 50CR steel plate girder bridges in the United States have been fabricated using a 309L welding consumable. This consumable is an austenitic stainless steel (Grade 309), giving superior corrosion resistance compared to 50CR steel. The “L” included in the consumable name stands for “low carbon,” which helps to prevent detrimental welding effects.

It is important to note that the welding consumables used for fabrication must be melted and manufactured in the United States to meet Buy America regulations for federal funding eligibility. This is not a concern for typical bridge steel welding consumables since they are widely produced in the United States. However, 309L is an austenitic stainless steel, which is not as readily available in the United States. This requirement should not deter agencies from pursuing 50CR steel plate girders but is provided to be a reminder that the domestic availability of the welding consumable should be checked when fabricating a CRP girder bridge. VDOT helped to alleviate this issue during fabrication of the Route 340 Bridge by electing to allow both 309L and 309L-C consumables, where the “C” in the latter stands for “cored wire.” The allowance of both consumables made it easier to locate and procure enough consumables made in the United States to fabricate the steel plate girders with U.S. consumables.

At present, an American Welding Society (AWS) task group has completed a ballot item to include 50CR steel material in the D1.5 Bridge Welding Code. The ballot was submitted to

AWS and is expected to be incorporated into a future interim edition of the 2020 AWS/AASHTO D1.5 Bridge Welding Code.

In order to allow for additional welding consumables and more efficient welding practices by steel bridge fabricators, VTRC and the University of Virginia are currently conducting a welding study on 50CR steel. The study includes several welding consumables other than the 309L and 309L-C consumables that have been successfully used for 50CR steel bridge applications. Having additional available consumables would help projects more easily meet Buy America regulations. The study also includes examining welding parameters such as heat input and interpass temperature. Examining these parameters will provide VDOT with the information necessary to develop specifications to allow fabricators to weld more efficiently yet still meet mechanical and inspection requirements.

Duplex stainless steels have a longer history of welding than does 50CR steel. Their microstructure is a careful balance of austenite and ferrite that must be maintained when these materials are welded. If duplex steels are welded incorrectly, including improper selection of filler metal, it can shift the balance of austenite and ferrite, leading to a reduction in mechanical properties and corrosion resistance. Fortunately, AWS provides guidance on welding stainless steels in *AWS D1.6/D1.6M Structural Welding Code—Stainless Steel* (hereinafter "D1.6") (AWS, 2017). Although fabricated duplex stainless steel bridge beams are not commonly used in the United States, stainless steel structural elements have found extensive use in other engineering applications that require excellent durability.

AWS D1.6 provides guidance on welded stainless steel elements that are subjected to stress (AWS, 2017). Similar to AASHTO/AWS D1.5M/D1.5:2015 (hereinafter "D1.5") (AASHTO/AWS, 2015), D1.6 covers topics such as welding connections, prequalification, qualification, fabrication, inspection, and welding of shear studs. It also highlights topics that are more important for stainless steels as compared to conventional steel, such as the importance of heat input and dilution. For example, very low heat input or high dilution can promote ferrite formation, and very high heat input can promote the formation of detrimental intermetallic compounds, all of which can lead to undesirable properties (AWS, 2017). D1.6 highlights these potential issues and provides guidance on how to avoid them by describing important aspects of filler metals and the approximate heat input values that should be avoided (AWS, 2017). Extensive information on all aspects of the fabrication of duplex stainless steels is also available from the International Molybdenum Association (2014).

Although duplex stainless steel has not been used for plate girder bridges in the United States, the fabrication of duplex stainless steel structures is not new. Duplex stainless steel has been successfully fabricated and welded for industrial structural applications for decades. Numerous fabricators have experience fabricating structures made of duplex stainless steel, including the following:

- Ameco
- Enerfab
- Vigor
- Chattanooga Boiler & Tank

- Offenhauser
- Chicago Bridge and Iron Company
- Shepard Steel
- Northern Manufacturing.

Rolled Structural Shapes Availability

VDOT elected to use 50CR steel for all of the secondary members, such as cross frames and diaphragms, on the Route 340 Bridge. This decision allowed the secondary members to have a corrosion resistance equal to that of the steel plate girders, thus providing the overall structure with excellent durability. This decision was initially challenging since rolled structural steel shapes, such as beams, channels, and angles, are currently not produced in 50CR steel. Therefore, secondary members were successfully fabricated by using bent plates to form substitutes for the necessary channels and angles. VTRC contacted the sole 50CR steel supplier in the United States after completion of the Route 340 Bridge, and at present the supplier does not have immediate plans to begin producing rolled structural shapes in 50CR steel material.

It is quite possible that for a future VDOT bridge constructed with 50CR steel plate girders, the secondary members could be constructed of either 50W or galvanized steel, as was done for the other four 50CR steel plate girder bridges, depending on the site location. Although both of these steel types have a reduced corrosion resistance when compared to 50CR steel, they could be used for secondary members on straight plate girder bridges since the secondary members are typically used only for erection purposes.

For duplex stainless steel, the potential exists for rolled structural shapes to be used for both primary and secondary members on steel plate girder bridges. This would eliminate the need for bent plates and/or welding and could provide a means for producing cost-effective structural elements. Two potential manufacturers of duplex stainless steel structural shapes were identified and contacted by VTRC: Stainless Structurals and Chatham Steel Corporation.

Stainless Structurals produces duplex and austenitic structural shapes using both conventional hot rolling and laser fusion welding. Their laser-fused beams come in standard rolled shape dimensions and are available in stock or on demand in sizes for beams up to W24 x 136; for angles up to L8 x 8 x ¾; and for channels up to C15 x 40. VTRC was provided samples of a laser fusion structural beam and angle. Although additional mechanical testing and inspection would likely be warranted because of the innovative nature of the technology, the process offers promise that duplex stainless steel shapes could be produced in a cost-effective form without the need for traditional fabrication and welding. This could potentially allow for duplex beams to be used for rolled beam bridges where galvanized rolled steel beams have previously been used. It also could allow for duplex angles and channels to be used as secondary members, such as cross frames and diaphragms, when duplex stainless steel beams are used.

Chatham Steel Corporation also provides duplex and austenitic structural beams, angles, and channels offered as hot rolled or laser fused. It is possible that some of the austenitic stainless steels, such as Type 304L or Type 316L, could be used as secondary members for bridges. Although austenitic steels have yield stresses near 30 ksi, which is less than the yield stress of typically used ASTM A36 steel (36 ksi), the superior corrosion resistance of the austenitic steel could warrant its use with duplex stainless steel primary members.

Corrosion Resistance Comparison

Bridge Site Visits

Genito Road Bridge Over Swift Creek

The current Genito Road Bridge (State Bridge No. 6119) over Swift Creek replaced a fracture-critical two-girder bridge that was designed for 12 tons, originally built in 1938 (Figure 6). Prior to replacement, the bridge had been classified as functionally obsolete because of the narrow 20-ft-wide roadway and was also considered structurally deficient.

Figure 6. Original Bridge Over Swift Creek: left, 3 ton posting in spring 2010; right, underside of bridge

After an inspection in the spring of 2010, deterioration of the structure led the bridge to be load rated for a capacity of 3 tons, which necessitated that all school and emergency vehicles be detoured around the bridge. Although the original bridge had provided access across Swift Creek for 72 years, new physical and construction constraints required changes in the bridge design.

The physical and construction constraints that had to be addressed for the new bridge design were as follows:

- low clearance over water, with the girders having a low chord of 3 ft above design high water
- shallow depth of water (approximately 5 ft maximum)
- limited superstructure depth
- limited causeway width, so crane size was limited

- no impacts to waterline

- minimal permanent impacts to reservoir

- water quality maintained during and after construction, which limited concrete work in and over water

Although the original steel girders had provided a suitable service life of 72 years, corrosion of the structure had reduced the load-carrying capacity from 12 tons to 3 tons on the bridge. In addition, concerns with the future impact of the reservoir dictated the need for a minimal maintenance design. Concrete girders would normally be acceptable in this application, but the low water clearance limited the possible girder depth, thus preventing their use. The low water clearance also precluded the use of uncoated Grade 50W girders, since the material is not recommended in this environment (FHWA, 1989). VDOT elected not to use painted, metallized, or galvanized Grade 50 steel to minimize potential access in the reservoir should these protection systems become damaged and require re-coating. Therefore, the decision was made to use a steel with increased atmospheric corrosion resistance under the galvanized coating. Since 50CR steel was not yet available at the time of construction, Grade 50W steel was selected as the steel to be galvanized. The rationale was to provide an alloyed steel under the galvanizing such that if the zinc layer cracked or failed, the base metal would still possess some inherent corrosion resistance to provide a long service life.

Immediately, discussions ensued about fabricating GWS girders, and the following concerns were raised about GWS:

- The silicon content in the steel alloy will influence the coating thickness, and the galvanizer should treat Grade 50W steel as a "reactive" steel, which causes zinc to be deposited at a faster rate.

- The galvanizer should sandblast the steel prior to galvanizing to create a rough surface, which will mitigate the growth of excessively thick galvanized coating.

- Copper from the alloyed steel can become trapped in the coating or can end up in the galvanizing bath.

- A Grade 50W steel girder is expected to increase in weight by a sizeable amount because of how the zinc reacts with the steel, as compared to Grade 50 steel, so this additional dead load must be considered.

- Unexpected galvanizing coating failures, such as cracking, could occur.

- Bolt holes will need to be reamed after galvanizing because the excessive zinc will substantially reduce the hole diameter.

- A passive zinc layer may form on the exposed zinc, and if the Grade 50W steel is exposed, a patina may also form on it. Either case will cause the corrosion rate to diminish, and the GWS girders will have a long service life, possibily more than 100 years for a temperate lake environment.

- There are no known long-term data on GWS.

In addition, Langill (2003) provided insight and assurance regarding hot-dip galvanized Grade 50W steel. The author confirmed that it was possible to galvanize Grade 50W steel; described what would be different as compared to Grade 50 steel; and stated that this approach would add to the corrosion resistance of the girder. To help ensure that galvanizing Grade 50W steel was a success, the following requirements were incorporated into the project specifications for the Genito Road Bridge:

- The contractor must identify the steel type (Grade 50W).

- The contractor must provide a piece list of all parts to be galvanized.

- The top flange and bearing locations must be masked during galvanizing to eliminate the need to remove zinc in the field to apply shear studs and prepare bearing locations.

Discussions ensued with the galvanizer to ensure the galvinizer was prepared for galvanizing a reactive steel and could plan accodingly.

After two girders were galvanized, coating thickness measurements were made on two girders. These measurements (Figure 7) clearly show an increase in thickness, with the mean values equal to 16 mils and 14 mils for Girder 1 and Girder 2, respectively. It was also clear from the measurement data that the distribution was skewed, with the median being less than the mean. Therefore, the GWS girders did exhibit a relative increase in thickness compared to Grade 50 steel, but this increase was consistant with comments from the industry. However, a notable increase in additional dead load was not observed and the bolt holes did not exhibit a substantial reduction in hole diameter.

Construction of the bridge using the GWS girders was consistent with standard construction practices. Figure 8 shows the exposed studs before placement of the concrete on the deck and the girder splice that was used. In the figure showing the splice, a variation in the color can be seen; however, the galvaning was tightly adherent. This figure also shows the underside of the bridge and a side view of the finished bridge.

After completion of the bridge, an initial inspection was performed in October 2012. Images from this inspection are shown in Figure 9. The overall condition of the structure was reported as “Good,” and there were no indications of any noteworthy deficiencies with the GWS girders. Regular inspection reports from October 2014 and 2016 also indicated that the overall condition of the structure was “Good” and there were still no indications of any noteworthy deficiencies with the GWS girders.

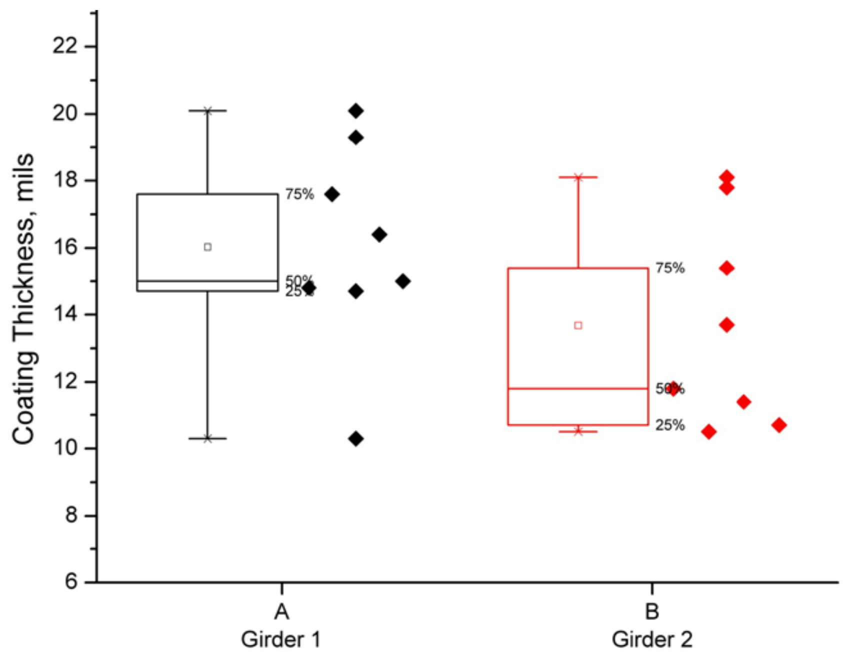
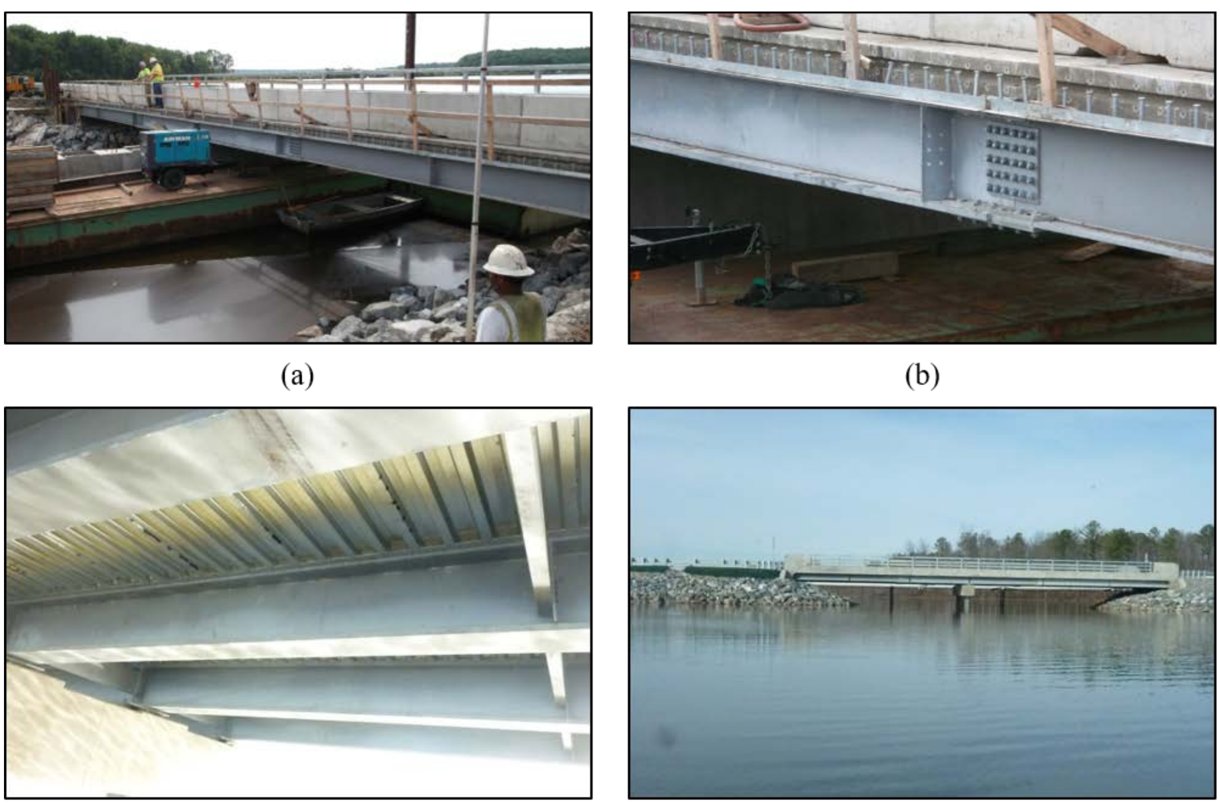


Figure 7. Initial Coating Thickness Measurements on 2 of 18 Weathering Steel Pieces That Were Coated Using Hot-Dip Galvanizing



(a) (b) (c) (d)

Figure 8. The New Genito Road Bridge Over the Swift Creek Reservoir: (a) during construction; (b) showing the studs and splice; (c) underside of the bridge; (d) after completion of construction

Figure 9. Images of the Genito Road Bridge From the Initial VDOT Inspection Report in 2012: left, upstream (north) elevation; right, downstream (south) elevation

In March 2018, a field visit by VTRC and VDOT staff was arranged for a visual assessment of the condition of the GWS girders. Most of the surface appeared to be in acceptable condition and corrosion was not evident, as shown in Figure 10. However, a small circular area was found along the bottom of a girder where the galvanizing was no longer present. It will be important to monitor this site as the bridge continues to age and to understand better the corrosion resistance of a galvanized Grade 50W steel girder in this environment.

Coating thickness measurements were also taken of the girder flanges and webs and the galvanized steel guardrail plate anchor and railing, both of which are on the southeast side of the bridge. The galvanizing thickness of the last two items was measured so that changes in the coating thickness of GWS girders could be compared to changes in conventional galvanized Grade 50 steel. The results of the coating thickness measurements are provided in Figure 11.

Figure 10. Visual Condition of Galvanized Weathering Steel Girder in March 2018: left, an example of the general condition; right, the only location exhibiting loss of galvanizing coating and showing the underlying weathering steel girder

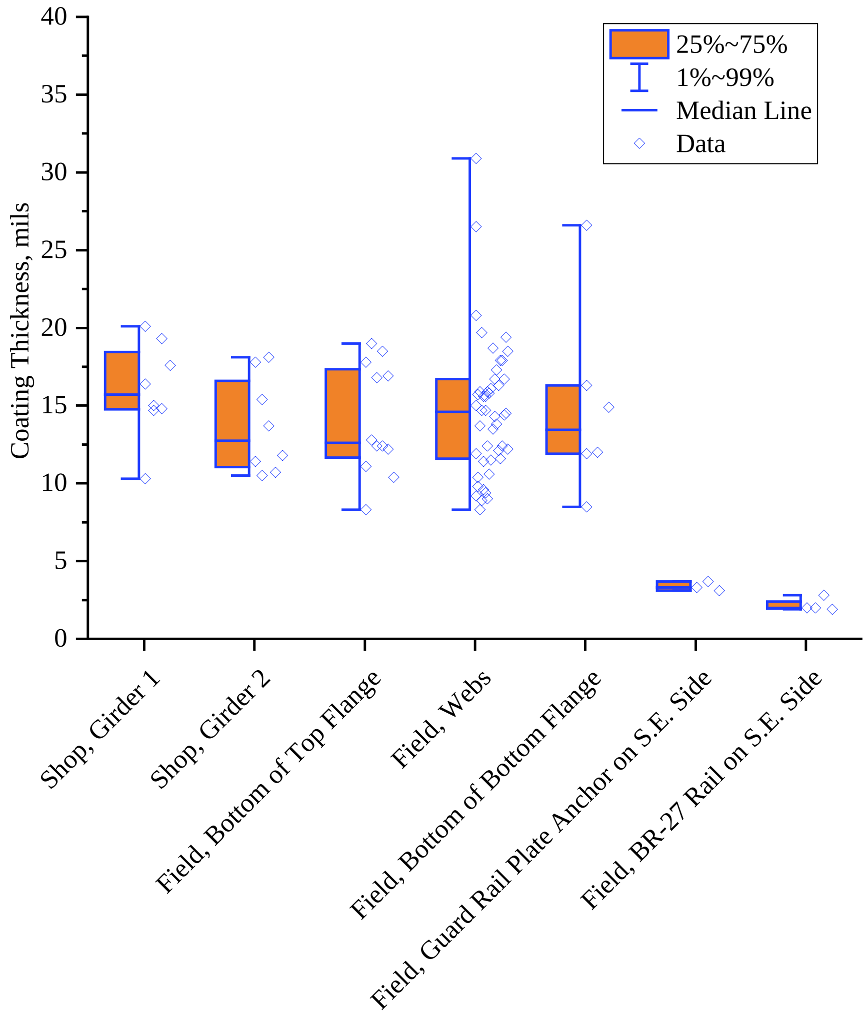


Figure 11. Distribution of Zinc Coating Thickness Measurements for Different Bridge Components and Relative to the Initial Shop Measurements That Are Also Shown in Figure 6

It is clear that the values measured in the shop were similar to those measured on the GWS girder in the field. The coating thickness of the GWS flanges and webs was similar, whereas the conventional galvanized steel components had a thinner average coating and smaller thickness ranges. These observations are consistent with the expectation that GWS will have a thicker zinc coating compared to galvanized Grade 50 steel.

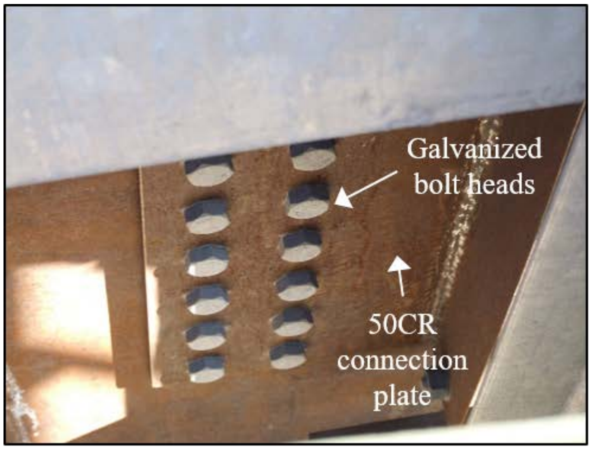
William G. Taylor Memorial Bridge

In October 2013, the producer of the 50CR steel gave a small group of personnel from VTRC and VDOT the opportunity to visit the William G. Taylor Memorial Bridge, which was constructed using 50CR steel plate and is located in Coatesville, Pennsylvania. Since its dedication in 2012, the bridge has been subjected to low volume, heavy loads because of its location between the steel belt shop and the rolling mill at the steel producer. A photograph of one of these heavy loads is shown in Figure 12.

Figure 12. William G. Taylor Memorial Bridge Constructed Using ASTM A709 Grade 50CR Steel Plate Girders

The bridge was constructed using a galvanized steel open grid deck supported by 50CR steel plate girders. As shown in Figure 13, the bridge hardware, including fastener assemblies, is also galvanized. The 309L filler metal used when welding is evident, which is why the weld beads have a shiny metallic luster and the 50CR steel plate has a darker rustic patina. As may be seen, the patina formed on the 50CR steel plate except for the region around the galvanized bolt head.

A second visit to the bridge in 2016 found that little had changed visually. Figure 14 shows examples of what was observed when the 50CR steel plate, galvanized hardware, and 309L filler material was evaluated. No noteworthy changes were observed with any of these items. In addition, the pier below the 50CR steel girders did not exhibit any rust staining associated with runoff from the girders. The shiny weld bead from the 309L filler metal can still be seen easily. The bolted connection, with galvanized fastener assemblies, also looked similar to the original visit. This was in clear contrast to the rustic patina of the 50CR steel.



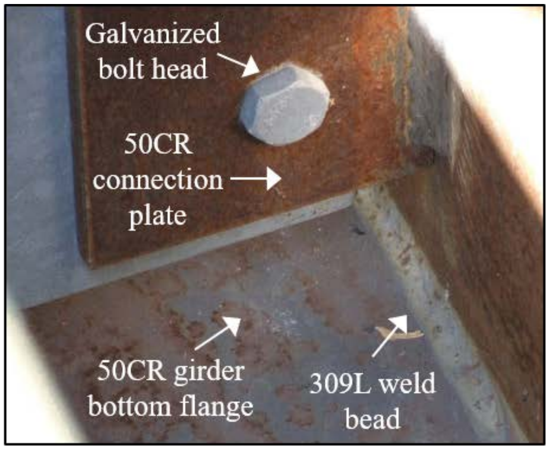


Figure 13. ASTM A709 Grade 50CR Steel Plate With Galvanized Bolts and Weld Bead: left, details; right, close-up of galvanized bolt and weld bead

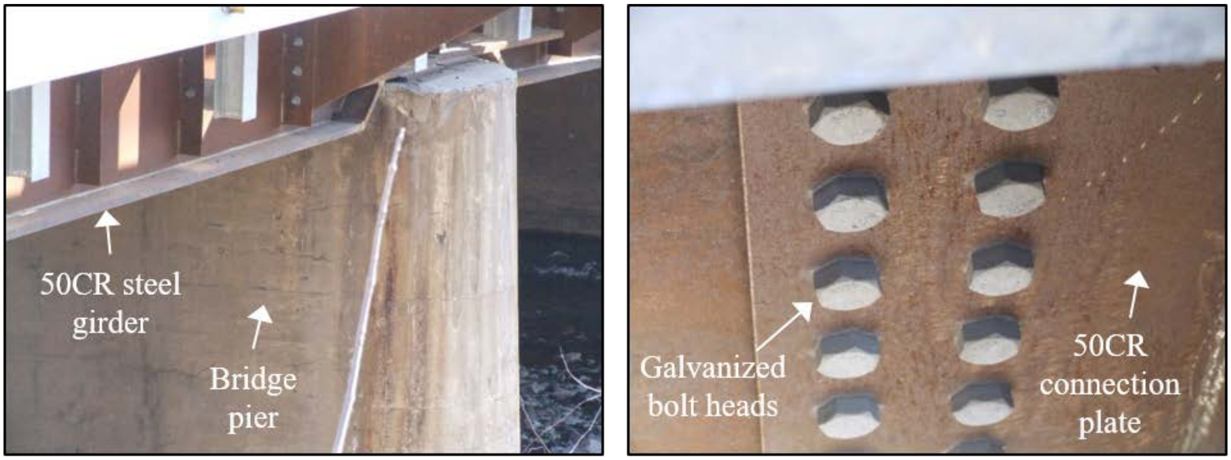


Figure 14. Photographs of William G. Taylor Memorial Bridge During a 2016 Follow-up Visit: left, lack of staining on the concrete pier; right, appearance of the galvanized bolted connection, similar to the appearance on the original visit

PREN Comparison

An initial corrosion resistance comparison of the CRP materials was conducted using their PREN values. The results are listed in Table 3.

Although carbon steels fall outside the scope of the PREN evaluation, their PREN values can be computed to provide a baseline for comparing traditional bridge steels and CRP. For example, a Grade 50 steel would have a PREN value of 0 and a Grade 50W steel would have a PREN value ranging from 0 to 1. Although not technically a stainless steel since it is not guaranteed to have 10.5% chromium by specification, the A1035CS steel plate would have a PREN value of 9 to 12. Using these values, simple relative comparisons can be made between Grade 50W steel and CRP. CRP materials have a corrosion resistance of roughly 10 times better for A1035CS steel plate and 50CR steel to roughly 40 times better for Grade 2205. The table also shows that there is a substantial difference between the corrosion rate of the stainless steels, which is dependent on surface finish, exposure to chloride salts, pollution, and weather conditions.

Table 3. Plate Materials and Associated PREN Value Based on the Typical Alloy Composition

Material Type	Estimated Corrosion Resistance (PREN Range)
ASTM A709 Grade 50CR	11-13
Grade 2101 (UNS S32101)	25-29
Grade 2202 (UNS S32202)	25-27
Grade 2304 (UNS S32304)	22-30
Grade 2205 (UNS S32205)	34-38

PREN = pitting resistance equivalent number.

Long-Term Corrosion Data

One long-term corrosion study that was reviewed was conducted in Cristobal, Panama (Southwell and Bultman, 1982). The unpainted steel samples were placed near the shorelines at an angle so they could benefit from any natural rain cleaning. The steels included in the study were carbon steel, Type 410 stainless steel, and Type 316 stainless steel. The carbon steel in the study was pickled, which should provide it with more corrosion resistance compared to the same type of steel that was not pickled. Type 410 stainless steel has a chemistry similar to 50CR steel and should have comparable corrosion resistance. Some of the duplex stainless steels, such as Grades 2101, 2202, and 2304, are similar to Type 316 in terms of corrosion resistance; as indicated by the relative PREN, Grade 2205 is more corrosion resistant. Table 4 shows the average annual corrosion rate after 16 years of exposure.

As evident in the table, Type 316 and Type 410 stainless steels have corrosion rates at least 7 times slower than the carbon steel in this particular environment. Since bridge steels are not generally pickled, it would be expected that Type 316 and Type 410 stainless steels would perform even better when compared to a carbon steel that was not pickled. Since Type 316 and Type 410 are representative of duplex stainless steels and 50CR steel, respectively, in terms of corrosion resistance, they demonstrate the potential for extended durability that CRP can provide.

The second long-term corrosion study reviewed was conducted in Kure Beach, North Carolina (Houska, 2014). Kure Beach is a low pollution environment with lower coastal salt exposure than areas further north or south based on historical United States National Atmospheric Deposition Program data. Among other steels, this study included carbon steel, galvanized steel, and Type 316 stainless steel. Similar to the previous study, Type 316 was used to represent some of the duplex steels, including Grades 2101, 2202, and 2304. Table 5 shows the exposure time for each steel sample and the average annual corrosion rate.

As shown in the table, the galvanized steel has a corrosion rate approximately 8.5 times better than uncoated carbon steel, and Type 316 stainless steel has a corrosion rate of about 6,000 times better than carbon steel. Again, these data demonstrate how CRP steels can provide a significant increase in durability over conventional carbon steels. When the results of both studies are examined, the significant variation in corrosion rates illustrates the importance of assessing the severity of the service environment and avoiding conclusions based on locations that are substantially different with regard to pollution, chloride salt, moisture, and time of wetness.

Table 4. Average Annual Corrosion Rate of Steel Samples After 16 Years in Cristobal, Panama

Steel Type	Average Annual Corrosion Rate (mm/yr)
Type 316 stainless	<0.0003
Type 410 stainless	0.0003
Carbon steel, pickled	0.0021

Table 5. Average Annual Corrosion Rate of Steel Samples After 13+ Years in Kure Beach, North Carolina

Steel Type	Exposure Time (yr)	Average Annual Corrosion Rate (mm/yr)
Type 316 stainless	15	<0.000025
Galvanized steel	13	0.0173
Carbon steel	16	0.147

Mechanical Properties Comparison

Uniaxial Testing of 50CR Steel Plates

Using the test loads and strains collected during testing, stress-strain curves were developed for each specimen. Overall, the stress-strain curves for the specimens were not drastically different from those for typical bridge steels. A linear elastic portion of the curve is observed until yielding, at which point strain hardening begins and continues until the ultimate tensile strength is reached. At this point on the curve, necking was observed in the cross section of the specimen, and loading continued until fracture of the plate. Whenever multiple specimens were tested, they produced nearly identical curves, so only the average stress-strain curves and values are reported herein. Figure 15 shows average complete stress-strain curves for the ½-in-, 1-in-, and 1¾-in-thick specimens tested, and Figure 16 shows enlarged plots of the stress-strain curves from 0 to 0.015 strain.

One slight difference between the 50CR steel and traditional steels is the continuous yielding behavior, which is most notably seen in the plots in Figure 16. This phenomenon, common to stainless steels, is indicated on the stress-strain curve where the plots are gradually rounded in the transition from elastic behavior to strain hardening. This behavior was expected based on experience with 50CR steel. The continuous yielding behavior is a difference in the material behavior when compared to that of Grade 50 steel. The bridge design process would be similar when these two materials are considered.

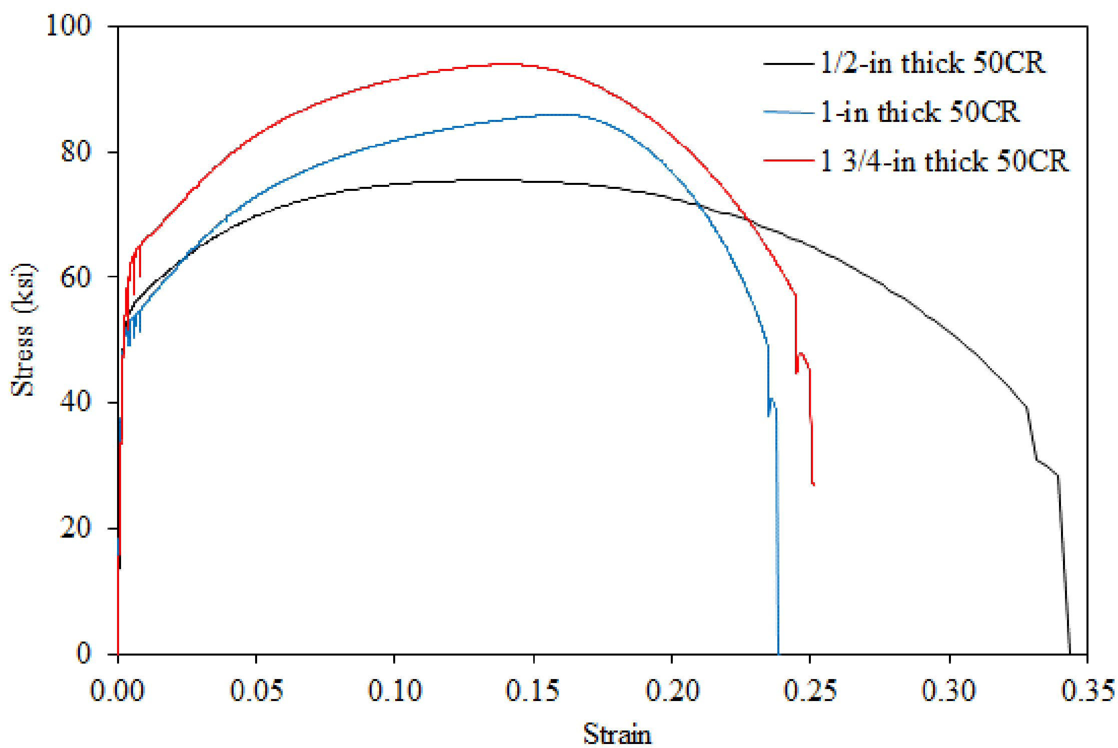


Figure 15. Stress-Strain Curves of ASTM A709 Grade 50CR Steel Specimens ½ in, 1 in, and 1¾ in Thick

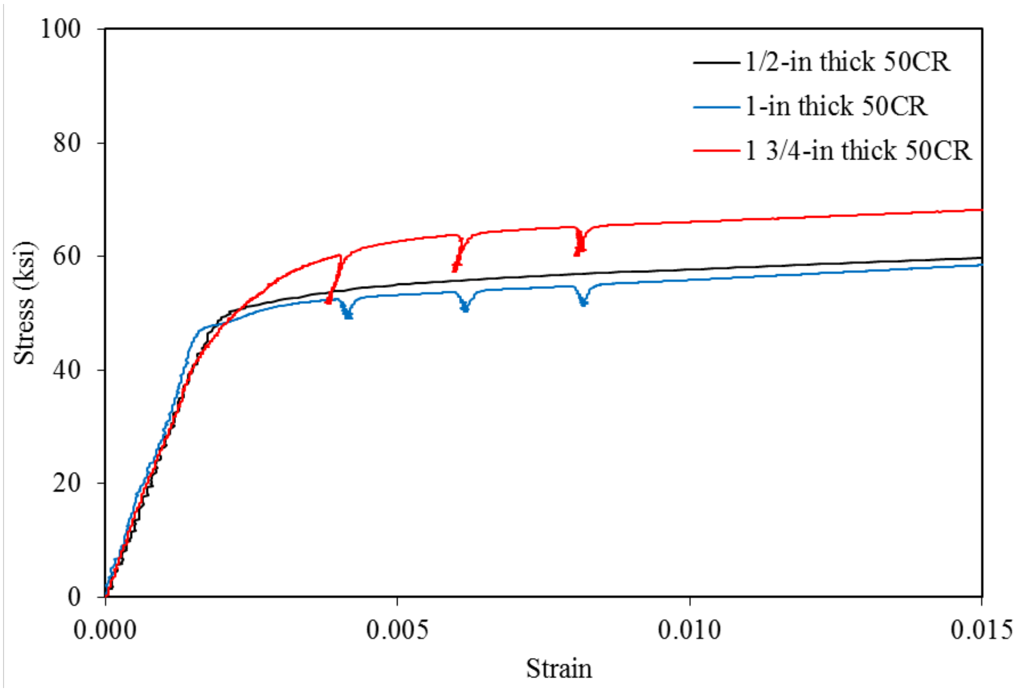


Figure 16. Enlarged Plots of the Stress-Strain Curves (Up to 0.015 Strain) of ASTM A709 Grade 50CR Steel Specimens ½ in, 1 in, and 1¾ in Thick

It is also important to note that some of this testing was conducted at FHWA's Turner-Fairbank Highway Research Center; their standard testing protocol imposes static holds during the testing process, which are used to determine the static yield point. These static holds resulted in the stress drops observed in the initial portions of the stress-strain curves shown in the plots for the 1-in- and 1¾-in-thick 50CR steel specimens in Figure 16. These static holds are characteristic of the loading protocol, not the material behavior. The ½-in-thick 50CR steel was tested at VTRC; therefore, the plot for the ½-in-thick 50CR steel in Figure 16 does not show these static holds.

Table 6 provides the average elastic modulus, yield stress, ultimate strength, and elongation at fracture for each of 50CR steel specimens tested. The yield stress was calculated using the 0.2% offset method as described in the ASTM E8/E8M specifications. Table 7 shows the specified tensile requirements for 50CR steel provided in ASTM A709.

Table 6. Tensile Test Results of ASTM A709 Grade 50CR Steel Plate

Thickness (in)	Yield Stress (ksi)	Ultimate Strength (ksi)	Elongation at Fracture (%)
1/2	54.0	75.7	34[a]
1	52.3	74.7	25[b]
1 3/4	59.8	82.4	24[b]

[a] Elongation was measured over a 2-in gauge length.
[b] Elongation was measured over an 8-in gauge length.

Table 7. Tensile Test Requirements for ASTM A709 Grade 50CR Steel

Grade	Yield Stress (ksi)	Tensile Strength (ksi)	Elongation at Fracture (%) 8-in Gauge Length	2-in Gauge Length
ASTM A709 Grade 50CR	50 min	70 min	18	21

By comparing the test results in Table 6 to the material requirements in Table 7, it is clear that all of the 50CR steel tests exceeded their specified values for yield stress, ultimate strength, and elongation at fracture. The elastic modulus of the 50CR steel also followed the widely accepted value of 29,000 ksi used for typical carbon steel. The test results confirm that 50CR steel has a behavior similar to that other bridge steels so that designing a bridge with 50CR steel would not require any changes to the design process.

Table 8 provides a summary of the results from the tensile testing of 3-in-thick 50CR steel. Overall the 3-in-thick 50CR steel showed good tensile properties, having a yield stress of 81 ksi and an ultimate strength of 97.1 ksi. When compared to the 50CR steel specifications, although it is outside the allowable thickness limits, the 3-in-thick material tested meets all of the tensile requirements. Because of project timeline requirements, CVN testing was not conducted on the 3-in-thick 50CR steel. CVN testing would need to be conducted to determine further if the 3-in-thick 50CR steel can meet all of the ASTM A709 requirements. VTRC currently possesses extra 3-in-thick 50CR steel, which could be machined into CVN samples and tested for a future project.

Table 8. Tensile Test Results of 3-in-Thick ASTM A709 Grade 50CR Steel Plate

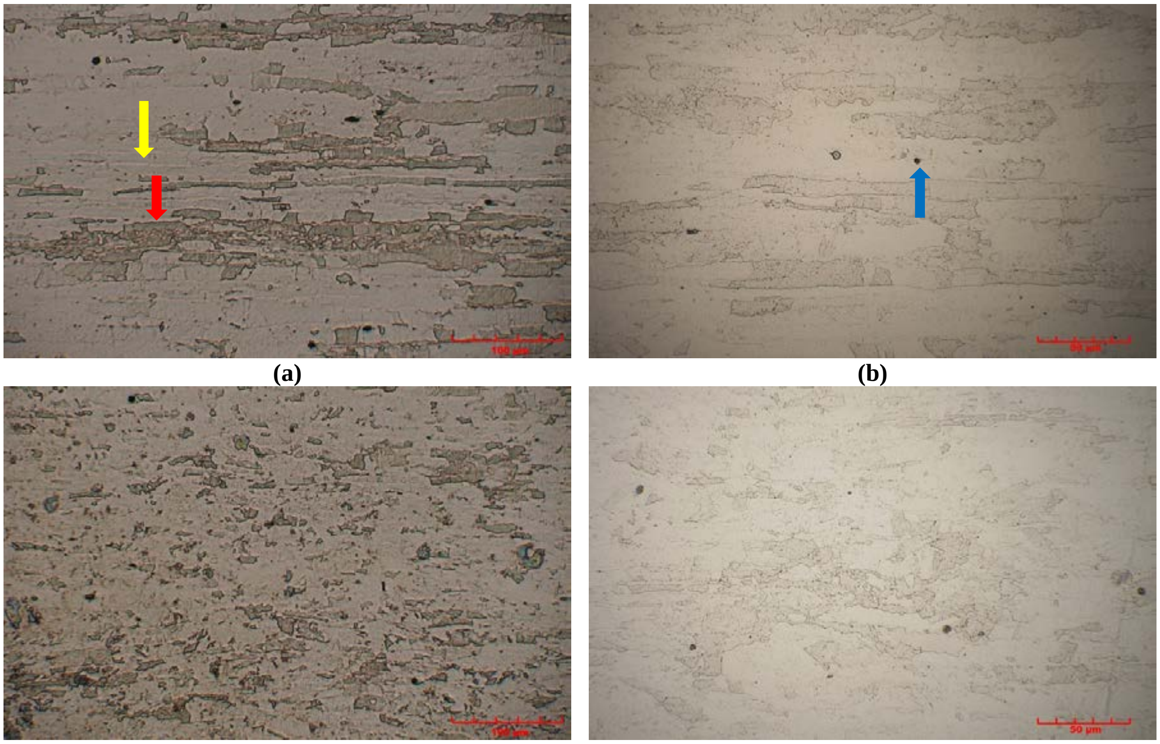
Thickness (in)	Yield Stress (ksi)	Ultimate Strength (ksi)	Elongation at Fracture (%)
3	81.0	97.1	36

Microstructure of 50CR Steel Tension Specimens

The etched micrographs shown in Figure 17 are for 50CR steel. The Vilella's reagent etchant used to evaluate the microstructure of the different 50CR steel samples is for steel that was produced in the as-rolled and tempered condition. Figure 17 shows both longitudinal and transverse sections that were etched with Vilella's reagent. This etchant reveals the ferrite-carbide structure in the microstructure and is commonly used for this type of steel. As is evident in these images, Vilella’s reagent reveals the darker martensitic and lighter ferritic phases in the microstructure, along with the smaller dark carbides that are present

The 1-in sample material of 50CR steel shown in Figure 18 was produced by normalizing and tempering in accordance with the guidelines in ASTM 709. This heat treatment process has been found to improve the consistency of the mechanical properties. The 1-in plate was sectioned longitudinally and etched with Vilella’s reagent.

All of the samples etched with Vilella's reagent exhibited similar features upon comparison with different samples that had been rolled in same direction. A comparison between two samples rolled in different directions appeared to show different features; this was expected because of the influence of rolling. However, when Figures 17 and 18 are compared, it is easy to see the thicker horizontal martensitic structure with thinner ferritic grains between the bands of martensite in Figure 18.



(a) (b) (c) (d)

Figure 17. Micrographs From ½-in ASTM A709 Grade 50CR Steel Plate Etched With Vilella's Reagent: (a) longitudinal sample at 600X magnification; (b) longitudinal sample at 1000X magnification; (c) transverse sample at 600X magnification; (d) transverse sample at 1000X magnification. Darker martensitic (red arrow) and lighter ferritic (yellow arrow) phases can be seen with smaller dark carbides (blue arrow).

(a) (b)

Figure 18. Micrographs From an ASTM A709 Grade 50CR Steel Plate Etched With Vilella's Reagent: (a) longitudinal sample at 1000X magnification; (b) transverse sample at 1000X magnification

Fractography of 50CR Steel Tension Specimens

The dominant failure mode in all of the 50CR steel tension specimens was necking, followed by a delamination parallel to the loading direction in the reduced section, and immediately followed by fracture at the reduced section. Similar delaminations were observed in tensile tests on welded 50CR steel performed by Seradj (2010). These delaminations occurred within less than 1% strain of the final elongation before final fracture, so they are not a concern for design. This delamination could be the result of segregation, but further research is needed to determine the exact cause. A photograph of a delamination observed during tensile testing is shown in Figure 19.

The fracture surface of the tensile specimens was investigated using a scanning electron microscope to analyze the fracture morphology and determine the type of failure. An overall view of the fracture surface is shown in Figure 20(a), and a magnified image of the fracture surface is shown in Figure 20(b). The surface in Figure 20(b) shows microvoid coalescence, which is indicative of a ductile failure.

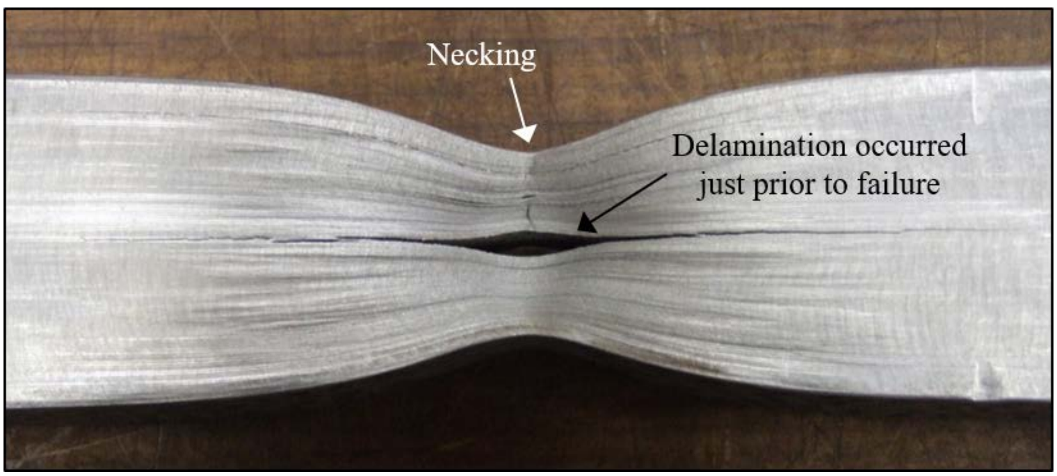
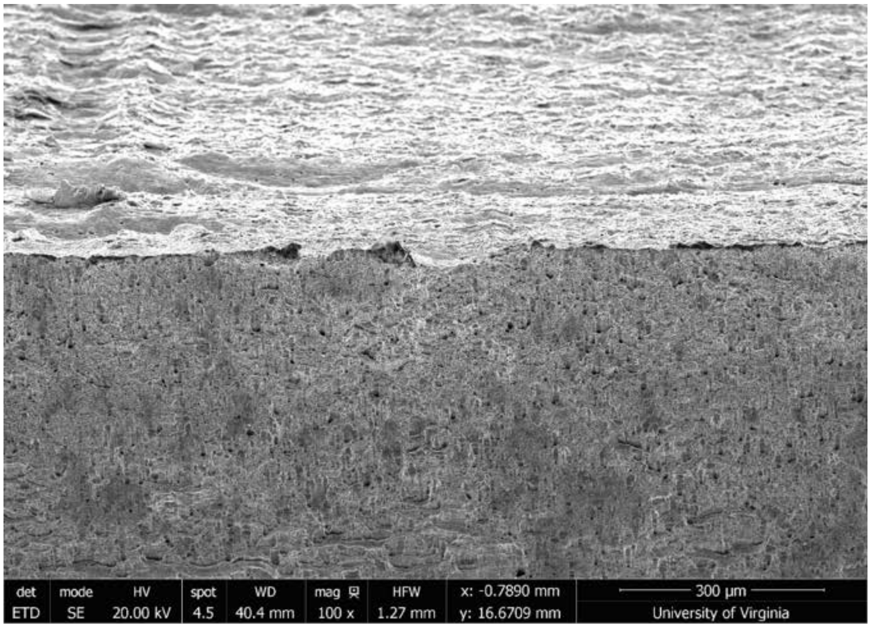
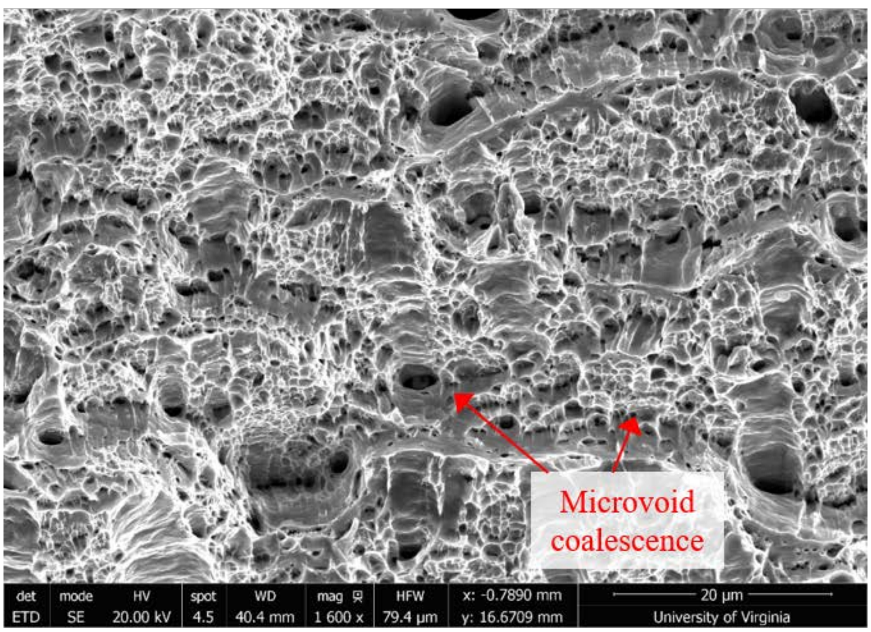


Figure 19. Delamination of ASTM A709 Grade 50CR Steel During Tensile Testing



(a)



(b)

Figure 20. Fracture Surface of ASTM A709 Grade 50CR Steel Tensile Test Sample: (a) overview; (b) close-up

Fatigue Testing of 50CR Steel Bolted and Welded Connections

Table 9 summarizes the bolted fatigue test data including the stress range and number of cycles until failure. All of the tests conducted at a 30 ksi stress range produced fatigue failures, whereas the plates cycled at a stress range of 20 ksi reached more than 15 million cycles without producing a failure. At that point, cyclic loading was stopped and both plates were considered runouts.

During the fatigue testing, some of the bolted plates failed because of fretting fatigue, which occurs because of microscopic motion at the contact surface between two components and is typically a result of some initial surface damage. For these specimens, the cracks initiated in the connection plates at the region between the pretensioned bolts and the edge of the splice plates. The cracks then propagated horizontally and in some cases reached the edge of the plate. Figure 21 shows some of the typical fatigue failures encountered during testing. The photographs in the upper left of the figure show an example of a net section fatigue failure; the remaining photographs show examples of fretting fatigue failures.

According to Benhamena et al. (2010), the presence of fretting fatigue in a bolted connection depends on the level of clamping force in the bolt. Under smaller clamping forces, cyclic fatigue failures occur at the bolt hole edge. For medium values of clamping forces, failure occurs near the bolt hole because of the gross sliding of the specimen surfaces. At greater clamping forces, fretting fatigue failure is the dominant mode of failure. For the current tests, all of the bolts were tightened using the turn-of-nut method, so the exact level of pretension in the bolts was unknown.

The fatigue data were plotted on a typical S-N plot and were compared to the AASHTO fatigue design curves, as shown in Figure 22. The fatigue detail categories A through E are included on the figure. As noted previously, the stress ranges were calculated based on gross section properties of the connections since the connections were designed and constructed as slip critical. A regression analysis was conducted on the experimental data to determine the mean and lower 95% confidence interval regression lines, which are also included in the figure.

Table 9. Fatigue Test Results of ASTM A709 Grade 50CR Steel Slip-Critical Bolted Connections

Plate No.	Stress Range (ksi)	Cycles to Failure
1	30	2,259,345
2	30	3,238,799
3	30	2,546,022
4	30	8,048,272
5	30	9,905,753
6	30	2,586,003
7	30	14,434,442
8	20	15,244,403[a]
9	20	15,244,403[a]

[a] Cyclic loading was stopped and the test was considered a runout.

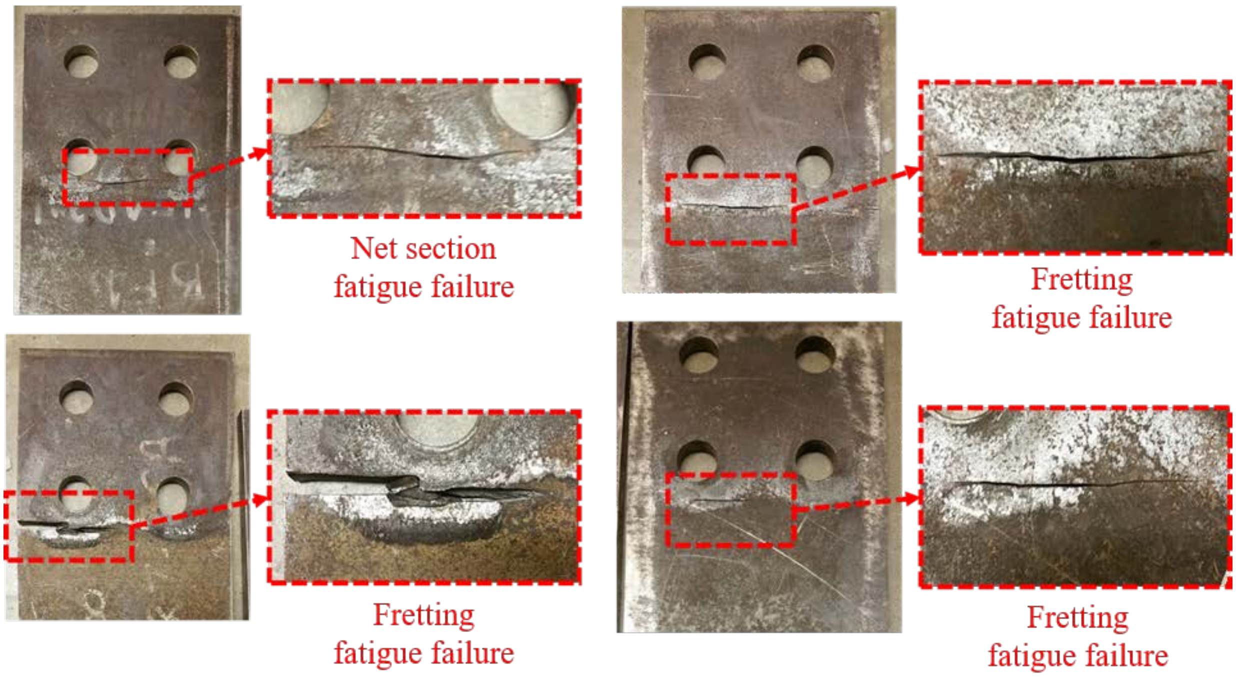


Figure 21. Typical Fatigue Failures of ASTM A709 Grade 50CR Steel Bolted Connection Plates

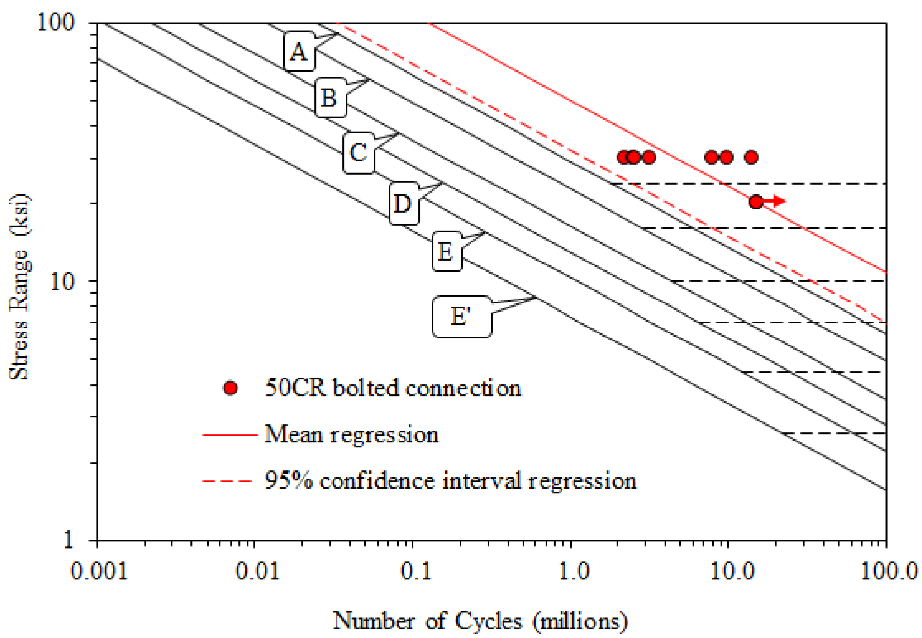


Figure 22. Fatigue Results for ASTM A709 Grade 50CR Steel Bolted Connections Compared to AASHTO Design Curves

From examination of the figure, it is clear that the lower 95% confidence interval regression line exceeds the AASHTO fatigue design Category B curve, which is the detail category for slip-critical bolted connections. This shows that when a slip-critical bolted connection with 50CR steel is designed, it can be treated the same as a typical Grade 50 or 50W steel. Further research is ongoing at VTRC regarding other aspects of 50CR steel bolted connections, including the slip coefficient and fastener assembly mechanical properties.

Table 10 summarizes the fatigue test results for 50CR steel welded connections. As may be seen in the table, two specimens, Nos. 5 and 8, had an undercut located near the weld toe that caused the specimens to fail prematurely. Neither of these specimens would have passed the current AASHTO/AWS D1.5 workmanship criteria for welds in tension. Thus these two specimens were excluded from any additional analysis since the focus of the testing was to determine the fatigue behavior of 50CR steel plate without any undercut welds. It is also important to note that none of the weld reinforcement had been ground smooth on any of the samples, so they would not have met the AASHTO fatigue detail Category B requirements.

As shown in Table 10, Specimen 10 was initially cycled at a stress range of 11 ksi and did not fail after 13 million cycles. At this point, the stress range was increased to 25 ksi to produce an eventual fatigue failure. Miner's rule was used to calculate an equivalent constant amplitude stress range for the specimen. A similar process was also applied to Specimen 11, which began cycling at 15 ksi, before the stress range was eventually increased to 20 ksi. Even after the stress range was increased, this specimen did not produce a fatigue failure after more than 52 million cycles and was declared a runout.

The test data, aside from the specimens with undercuts, were plotted on an S-N plot to compare the experimental data to the AASTHO fatigue design curves. This is shown in Figure 23. Similar to the 50CR steel bolted fatigue data, the mean and lower 95% confidence interval regression lines are also shown.

Since the welds on the welded specimens were not ground smooth and weld soundness was not established using non-destructive testing (NDT), the welded samples had been expected to behave as a fatigue detail Category C. This highlights the importance of good quality specifications, welding, and NDT practices during steel bridge fabrication. However, from examination of the placement of the lower 95% confidence interval line, the samples behaved in a manner more similar to that of a fatigue detail Category D. This was likely due to a combination of how the welds were specified and the weld quality.

Table 10. Fatigue Test Results of ASTM A709 Grade 50CR Steel Welded Connections

Specimen No.	Stress Range (ksi)	Cycles to Failure
1	20	992,280
2	20	818,767
3	20	787,726
4	20	472,816
5	20	182,216[a]
6	15	3,704,090
7	15	906,095
8	15	491,733[a]
9	11	10,473,466
10	11.7[b]	13,244,548
11	17.5[b]	52,250,319[c]

[a] Undercut was present in the weld not passing current AASHTO/AWS D1.5 workmanship criteria for welds in tension.
[b] Specimen was cycled at multiple constant amplitude stress ranges; thus, Miner's rule was used to calculate an equivalent constant amplitude stress range.
[c] Cyclic loading was stopped and the specimen was considered a runout.

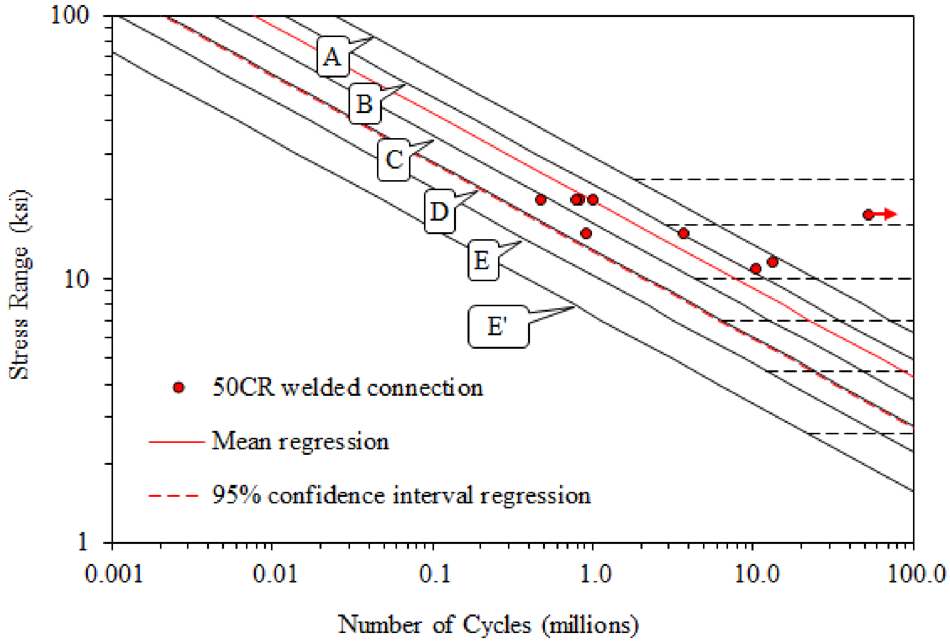


Figure 23. S-N Test Results of Welded ASTM A709 Grade 50CR Steel Connections

The presence of undercuts in some of the welded samples indicates that when the samples were ordered from the fabricator, the order specifications had not stated that the welds be ground smooth or pass any NDT requirements, including visual inspection. For all steel bridges, including the Route 340 Bridge, CJP welds, such as those in these specimens, would have to be ground smooth and pass NDT requirements. Therefore, it was concluded that the welds tested in this study did not comprise a representative sample of 50CR steel bridge welds. It is recommended that additional research be conducted on 50CR steel welded samples that meet the fatigue detail Category B requirements (i.e., weld reinforcement ground smooth and NDT requirements passed) since these details are commonly used for steel bridges.

It is important to note that all of the welded 50CR steel fatigue specimens were fabricated using a three-pass weld necessary to achieve the weld; one side of the specimen had one pass, and the other side had two passes. All of the fatigue failures initiated at the weld toe on the two-pass side of the specimen.

Uniaxial Testing of A1035CS Steel Plate and Duplex Stainless Steel

The results of the tensile tests on the remaining stainless steels, including duplex and A1035CS steel plate, are shown in Figure 24. As previously mentioned, the steels tested included Grades 2101, 2202, 2304, and 2205 and A1035CS steel plate. All tests were conducted on ½-in-thick samples except for a second group of Grade 2304 samples from another steel producer, which were conducted on ¾-in-thick samples.

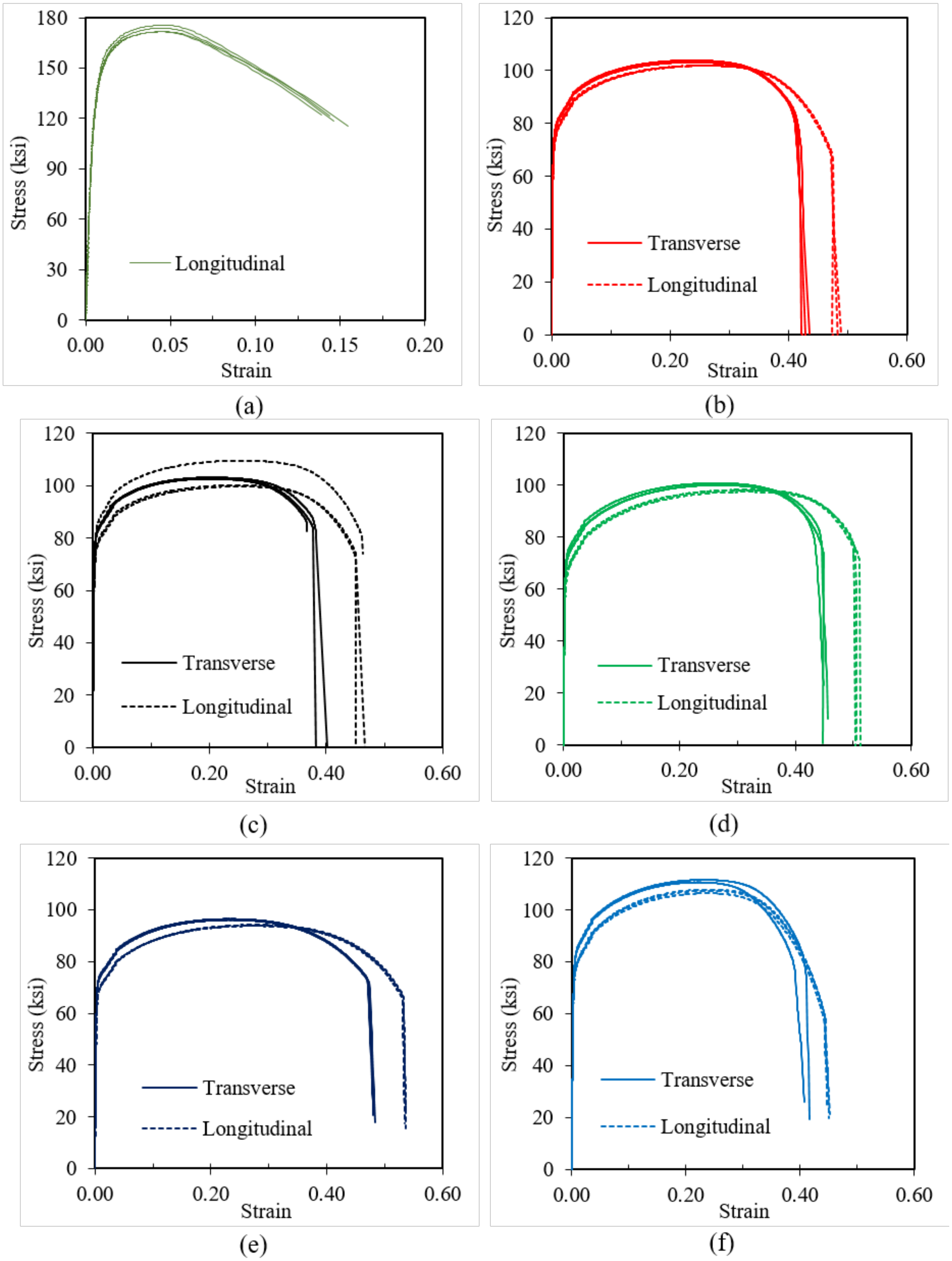


Figure 24. Tensile Test Results: (a) ½-in-thick A1035CS steel plate, (b) ½-in-thick Grade 2101; (c) ½-in-thick Grade 2202; (d) ½-in-thick Grade 2304; (e) ¾-in-thick Grade 2304; (f) ½-in-thick Grade 2205

Overall, the stress-strain behavior of the stainless steels was relatively similar to that of carbon steel. Similar to the 50CR steel, there was no discontinuous yielding or yield point elongation. The stress-strain curve of the stainless steels transitioned gradually from elastic to plastic behavior. There were some slight differences between the longitudinal and transverse samples of each steel type, such as the longitudinal samples tended to produce slightly smaller yield stress values and slightly greater elongations. Overall, however, the differences between the two orientations were not significant. The A1035CS steel plate exhibited similar behavior to that of the duplex steels, although the transition from elastic to plastic behavior occurred more rapidly. In order to provide a more clear comparison among all of the A1035CS steel plate and duplex stainless steels, Figure 25 shows a plot with a representative curve for each steel type. For the duplex steels, a representative longitudinal curve was selected since the A1035CS steel consisted of only longitudinal samples. For comparison purposes, plots of Grade 50 and Grade 50CR steels were also included in the figure. Table 11 also provides a numerical summary of the tensile test results for A1035CS and duplex stainless steels.

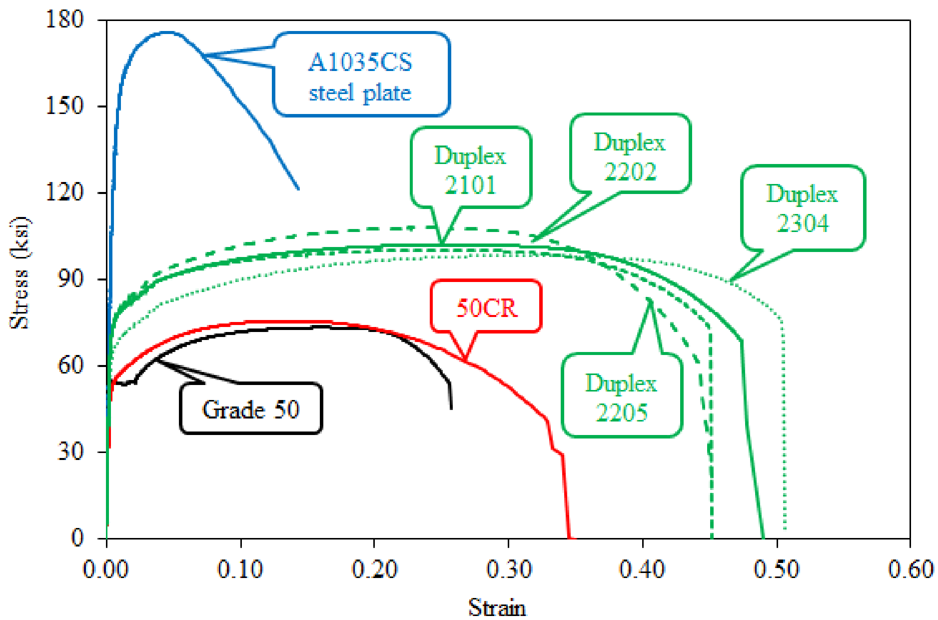


Figure 25. Comparison of Tensile Test Results for Corrosion-Resistant Steels

Table 11. Tensile Test Results for A1035CS and Duplex Stainless Steel Plate

Steel Type	Thickness (in)	Orientation to Rolling Direction	Modulus of Elasticity (ksi)	Yield Stress (ksi)	Ultimate Strength (ksi)	Elongation at Fracture (%)
A1035CS plate	½	Longitudinal	29,100	133	173	15
Grade 2101	½	Transverse	30,200	75	104	43
Grade 2202	½	Transverse	31,400	78	103	38
Grade 2304	½	Transverse	29,900	69	100	45
Grade 2304	¾	Transverse	30,000	70	96	48
Grade 2205	½	Transverse	29,400	65	111	41
Grade 2101	½	Longitudinal	29,700	71	102	48
Grade 2202	½	Longitudinal	30,700	75	103	46
Grade 2304	½	Longitudinal	28,100	63	98	51
Grade 2304	¾	Longitudinal	27,900	68	94	54
Grade 2205	½	Longitudinal	28,000	73	107	45

The A1035CS steel plate possesses tensile properties that are attractive for use on steel bridges. Similar to the duplex steels, the modulus of elasticity is essentially the same as for other carbon steels. The yield stress of 133 ksi and ultimate strength of 173 ksi are much greater than the respective values for carbon steels and put A1035CS steel plate into a category more similar to that of the high performance steels (HPS), which have yield stresses of 50, 70, and 100 ksi, depending on the grade. The fracture elongation of A1035CS steel plate is slightly less than the HPS requirements of 18% to 21%, but the steel likely has similar ductility because of its increased yield stress. Of course, in order to meet the requirements of an HPS, A1035CS steel plate would need to meet toughness and weldability requirements, neither of which is discussed in this report. Nevertheless, the enhanced corrosion resistance and strength of A1035CS steel plate warrant further investigation into how this material can be used in plate form for steel bridge applications. Further research is recommended on A1035CS steel plate in the areas of fracture toughness and weldability. Once research on the fracture toughness has been completed, A1035CS steel plate could potentially be used for bolted repairs until the weldability of the material has been determined.

As shown in Table 11, the duplex steels tested have tensile properties that easily meet or exceed those required for steel bridges. The modulus of elasticity for the duplex steels is in line with the commonly used value of 29,000 ksi for typical carbon steels. The yield stresses, calculated using the 0.2% offset method, for the duplex steels ranged from 65 to 78 ksi, and the ultimate strengths ranged from 94 to 111 ksi, both of which exceed values required for typical ASTM A709 bridge steels. The fracture elongation values for the duplex steels, ranging from 38% to 54%, also show how much more ductility the steels possess. Overall, the tensile test properties illustrate that duplex stainless steels could be designed using the standard steel bridge design properties used for typical carbon steels. Based on the tensile test results, bridge designs using duplex stainless steel grades could be used by using a yield stress of 65 ksi and an ultimate strength of 90 ksi. The additional yield stress provided by duplex steels could even result in less steel required for design, which could result in a cost savings.

Published Duplex Stainless Steel Charpy V-notch and Fatigue Data

The CVN data for duplex stainless steels were gathered from steel producers and other published data as a means of comparing these properties of duplex steels to those of typical bridge steels. There were no known published CVN values of A1035CS steel plate, and project time constraints did not allow for CVN testing to be conducted on either A1035CS or 50CR steel. Since the CVN properties of a steel are temperature dependent, it is important to keep both the CVN results and test temperature in mind when the results are considered. The CVN values for the base metal of typical bridge steels are generally reported at temperatures of 70°F, 40°F, and 10°F, depending on the temperature zone specified. Since the lowest expected temperature in Virginia likely ranges from -1°F to -30°F, this would classify VDOT bridges as being in AASHTO LRFD Zone 2 with the requirements shown in Table 12 (AASHTO, 2017).

Table 12. CVN Requirements for Typical Bridge Steels in AASHTO LRFD Zone 2

Grade	Thickness (in)	Minimum Test Value (ft-lb)	Fracture-Critical Zone 2 Requirement (ft-lb at °F)	Nonfracture-Critical Zone 2 Requirement (ft-lb at °F)
50/50W	t ≤ 2	20	25 at 40	15 at 40
	2 < t ≤ 4	24	30 at 40	20 at 40

It is clear from Table 12 that CVN testing for typical bridge steels for Zone 2 must be conducted at 40°F. Unfortunately, duplex CVN values could not be found for this specific temperature. Since CVN values decrease with decreasing temperature, the decision was made to find CVN values at lower temperatures since this would represent a worse case than necessary for bridge steels. Therefore, CVN values were gathered from steel producers for Grades 2202, 2304, and 2205 duplex stainless steels at -40°F (80°F less than typically specified). The results are shown in Table 13.

Table 13 shows that the Grade 2304 and Grade 2205 steels have a greater toughness at -40°F than what is specified by AASHTO at 40°F for both fracture-critical and nonfracture-critical applications. Since this is the case, both steels would clearly meet the CVN requirements when tested at 40°F. The CVN value for Grade 2202 steel at -40°F meets the Zone 2 nonfracture-critical requirements but falls slightly below the fracture-critical toughness requirements; however, it is important to keep the temperature difference in mind when the two are compared. It is quite possible that if CVN tests were conducted at 40°F, the results would meet both specifications. Further CVN testing of duplex stainless steels would provide the information necessary for a more direct comparison to bridge specifications. Nonetheless, this information shows that these duplex stainless steel have sufficient toughness for use on VDOT bridge structures.

Similar to fracture toughness, the fatigue properties of duplex steels were documented through examining existing literature and specifications. Although there are no known standard S-N curves for duplex stainless steels, a limited number of experimental tests have shown that some typical bridge welded details made from duplex steel have performed as well or better than their carbon steel counterparts (Lihavainen et al., 2000; Steel Construction Institute [SCI], 2017). Historical test specimens were made from Grade 2205 stainless steel (and other austenitic stainless steels) and were fabricated into longitudinal and transverse fillet welded specimens using the shielded metal arc welding procedure. Constant amplitude fatigue loading was then applied to the axial specimens. A linear regression was conducted on the results, and the 95% confidence interval for the duplex fatigue life was determined to meet or exceed those of carbon steel for the corresponding Eurocode fatigue details (SCI, 2018).

Table 13. Duplex CVN Values at -40°F

Steel Type	CVN Value (ft-lb at °F)
Grade 2202	19+ at -40
Grade 2304	30+ at -40
Grade 2205	40+ at -40

CVN = Charpy V-notch.

A later study (Hechler and Collin, 2008) conducted more fatigue tests on welded details made from Grade 2205 stainless steel. Specimens in the test program included typical bridge details such as longitudinal welded attachments, transverse CJP welds with and without the backing removed, transverse welded attachments, and a shear stud welded to steel plate. These small-scale specimens were all loaded axially under constant amplitude loading until a fatigue failure occurred. The test program also included orthotropic deck details such as a partial joint penetration rib-to-deck weld and a CJP rib splice weld with a backing bar. These orthotropic details were tested in component form under constant amplitude loading until failure. The tests on the duplex specimens produced 95% confidence intervals that were either nearly identical or exceeded those of the corresponding carbon steel details (SCI, 2018).

Construction and Cost Comparison

The cost of a CRP girder is expected to be greater than one of conventional steel because of the higher material cost and the increase in time necessary for fabrication. Much of the greater initial costs of these CRP materials is due to the higher alloying requirements, such as nickel, chromium, and molybdenum. However, this increased initial cost for using CRP materials may be warranted when a life cycle cost analysis is considered since CRP girders are expected to require little to no maintenance, including no initial or re-painting, over their service life. Table 14 shows the comparative cost between Grade 50W steel and other CRP materials. The cost values provided in the table are the budgetary unit costs of the steel material and do not include any surcharge cost values.

As may be seen in Table 14, the material costs of traditional Grade 50 and Grade 50W steels are nearly identical. When the material costs of the CRP materials are compared, it is important to keep in mind the relative corrosion resistance of the materials. The 50CR steel is approximately 2.5 times more expensive than Grade 50W steel but is also approximately 10 times more corrosion resistant. Of the duplex steels, Grade 2205 is the least expensive. From discussions with experts in the stainless steel community, this is because it is the most commonly used duplex stainless steel for structural applications. Grade 2205 is approximately 3.3 times more expensive than Grade 50W but has approximately 40 times the corrosion resistance. Therefore, it is important to consider the life cycle benefits gained by using CRP materials. The design strengths gained when using duplex stainless steels could also be included in a life cycle cost analysis since the strength increase gained from using these materials could lead to less material being needed in the design.

Table 14. Material Comparison of Traditional and CRP Steels

Grade	PREN	Material Cost ($/lb)	Relative Fabrication Time	Meets Buy America Requirements?	Fabricator Type	Maximum Thickness Available (in)
50	N/A	$0.51	1x	Yes	Traditional	4
50W	N/A	$0.54	1x	Yes	Traditional	4
50CR	11-13	$1.35	2x	Yes	Traditional	3
2101	25-29	$2.11	2x	Yes	Stainless	2
2304	22-30	$1.87	2x	Yes	Stainless	2
2205	34-38	$1.79	2x	Yes	Stainless	4

CRP = corrosion-resistant steel plate.
PREN = pitting resistance equivalent number.

The remaining information in Table 14 was gathered from discussions with CRP producers and other experts in the stainless steel community. It is expected that the fabrication time when using CRP will be approximately twice that of a typical bridge steel, which will also likely increase the total cost of a project. However, it is important to keep in mind that when CRP is used, although the upfront cost will be increased, the life cycle cost of a CRP structure is expected to be much less than of a traditional steel structure because of the drastic reduction in maintenance costs. All of the CRP materials listed in Table 14 are expected to meet Buy America regulations without any concern. Duplex stainless steel filler metals are readily available domestically, whereas fillers for 50CR steel are comparatively limited.

Some experts in the stainless steel community mentioned that if VDOT were to build a bridge with duplex stainless steel, it could be advantageous to use a specialized stainless steel fabricator. There are many stainless steel fabricators, and they have fabricated large structures for the nuclear, pipeline, or architectural sectors. These fabricators would have extensive knowledge in welding duplex stainless steels, and some also have steel bridge experience.

If a traditional steel bridge fabricator without experience in duplex stainless steels were used, a sharp learning curve in welding and fabricating a stainless steel girder would be expected. Using a stainless steel fabricator would likely reduce relative cost because of his or her knowledge and experience working with duplex stainless steels. Currently, CRP materials are being produced in plate thicknesses of 2 to 4 in, which is sufficient for use in steel bridge plate girders.

Case Studies

There are several examples of corrosion-resistant steel being used in infrastructure in the United States and across the world. This includes road bridges, pedestrian bridges, rail links, road tunnels, underground stations, car parks, energy supply stations, harbors, airports, and water and sewage applications, just to name a few (International Stainless Steel Forum, 2016). In particular, the use of an equivalent Type 304 stainless steel reinforcement was proven successful through its earlier use in the Progreso Pier in the Gulf of Mexico, which was completed in the early 1940s and is still in use today. The longevity of this structure showed that CRP can provide substantial benefits when used in the proper applications. The following cases highlight applications where corrosion-resistant steels, including 50CR and duplex stainless steels, have been used and the rationales for doing so.

Case 1: Route 340 Bridge

The Route 340 Bridge carries Route 340 (Main Street) over the South River in Waynesboro, Virginia. The previous structure was a coated steel girder bridge. Grade 50CR steel was chosen as the material for the new bridge for three reasons. First, there was an industrial site upstream of the bridge. Second, the bridge is in close proximity to standing water and can be inundated after storms. Third, Waynesboro desired an aesthetically pleasing, rustic look for the bridge because it is located in a park near downtown. Aside from the aesthetics, these are areas in which FHWA does not recommend the use of uncoated weathering steel (Sharp et al., 2018).

The new girders were successfully designed, fabricated, and constructed by January 2017. The Route 340 Bridge was the first 50CR steel bridge to include haunched girders, a bolted field splice with stainless steel fasteners, 50CR steel cross frames and diaphragms, and the use of a non-metallic blast cleaning media. The entire project was a success and provided the public with access across the river and trails connecting a park-like setting, shown in Figure 26. The bridge also received the 2018 Prize Bridge Award given by the National Steel Bridge Alliance (AISC, 2018).

Figure 26. Route 340 Bridge With ASTM A709 Grade 50CR Steel Girders

Case 2: Onancock Bridge Repairs

The Onancock Bridge carries Route 1002 over Onancock Creek in Onancock, Virginia, on the Virginia eastern shore. The original bridge was built in 1951 and has a water clearance of only a few feet at high tide. The environment is highly corrosive because of the proximity of the Chesapeake Bay and Atlantic Ocean. In 2017-2018, repairs were necessary to five steel beams. The repairs consisted of using both 50W and 50CR steel cover plates for web and flange repairs. The 50W steel repairs used Type 3 fastener assemblies, and the 50CR steel repairs used ASTM F3125 Grade A325 galvanized fastener assemblies. This was VDOT's first dissimilar metal bolted connection using 50CR steel. VTRC will monitor the bridge to evaluate the viability of this repair type.

Case 3: Jenkins Bridge Road Bridge Replacement

The Jenkins Bridge Road Bridge crosses Holdens Creek on the Virginia eastern shore. The existing bridge has four 15-ft spans constructed with painted steel beams, a timber deck, and an asphalt wearing surface. The bridge has a low water clearance over Holdens Creek and is frequently submerged, making it a highly corrosive environment. The rolled steel beams, timber deck, and railing were replaced in 1993. A 2012 bridge inspection report noted that the coating on the steel beams was failing. By 2016, another inspection report stated that significant corrosion and section loss had taken place. The section loss included 1/16 to 1/8 in of thickness loss throughout the steel beams. Web perforations had occurred in seven areas of the bridge. In summary, the paint system had failed after 19 years and the uncoated steel had failed in approximately 4 years.

Grade 50CR steel was selected as the material for the new bridge design. This decision was made because galvanized steel and hardware had not performed well in selected areas of VDOT's Hampton Roads District. The new bridge will consist of plate girders made of 50CR steel. One benefit of using plate girders over rolled steel beams in this application is that the web thickness can be increased. In some cases, water and chloride deposits on the bottom flange caused accelerated corrosion in the web. Grade 50CR steel is inherently corrosion resistant, and increasing the thickness of the web will help extend the service life of the structure. These changes to the traditional practice of using carbon steel rolled beams will provide VDOT with a bridge that can be expected to perform much better than its predecessor.

Case 4: Cala Galdana Bridge in Minorca, Spain

Constructed in 2005, the Cala Galdana Bridge on the island of Minorca, Spain, is the world's first vehicular bridge built using stainless steel as its primary structural elements (SCI, 2010). The bridge was constructed with Grade 2205 duplex stainless steel, and its structural systems consist of two parallel arches with a free span of 147.6 ft and an intermediate deck. The prior bridge was a reinforced concrete bridge that had been in place for 30 years, but the marine environment had led to large amounts of deterioration; therefore, a material with excellent durability was required. Aesthetics also played a part in the material selection because of the importance of tourism in the area. Although carbon steel, austenitic stainless, and duplex stainless steels were considered, Grade 2205 duplex stainless steel was selected because of its enhanced corrosion resistance and its high strength and ductility.

The bridge was designed using Eurocode 3 and the Spanish code for steel bridges (SCI, 2010). No significant differences in the design process were encountered. Welding processes used on the bridge included shielded metal arc welding, gas metal arc welding, flux-cored arc welding, and submerged arc welding with an interpass temperature of 300°F. After welding occurred, all of the welded areas were pickled to remove any contaminants from the welding process; pickling also encourages the formation of the passive layer that protects against corrosion. Once the fabrication process was complete, the bridge was erected. Duplex steel shear studs were welded to the girders to provide composite action with the concrete deck. All of the welds were inspected using traditional methods including visual inspection, radiographic testing, and magnetic particle testing.

Case 5: Harbor Drive Pedestrian Bridge in San Diego, California

The Harbor Drive Pedestrian Bridge crosses Harbor Drive at Park Boulevard in downtown San Diego, California. The curved span of the bridge is supported by a unique 130-ft-tall inclined pylon that supports stainless steel–wrapped suspension cables attached along one edge of the bridge deck. This was the first U.S. pedestrian bridge to use duplex stainless steel components (Roads & Bridges, 2018). Grade 2205 duplex stainless steel was selected because of its inherent corrosion resistance; it was expected to perform well in the coastal marine environment. Grade 2205 duplex steel was selected over other austenitic or ferritic stainless steels because of its increased strength; this allowed less material to be used, which saved both weight and money. The increased strength and toughness of Grade 2205 steel also met the fracture-critical requirements of the bridge. It also has a much better resistance to chloride stress corrosion cracking than other stainless steels. Aesthetics were also important in the material selection since pedestrians would be traveling over the bridge for many years. Overall, Grade 2205 stainless steel was expected to provide a 100-year service life for the structure.

In 2016, after 5 years of being in service, a medium-term inspection was conducted to determine how the stainless steel was performing. Key components included the Grade 2205 suspenders; Grade 2205 railing posts and pipe made from steel plates up to 5 in thick; Type 316 cable and Type 316L railing mesh; and Type 316 spider fittings. Overall, the results of the inspection were promising. The Grade 2205 suspenders and steel plates showed no signs of corrosion. There were signs of physical scarring in the plates because of embedded metal from skateboards, but since this was only surface corrosion, these areas are not cause for concern. The Type 316 and Type 316L railing mesh showed local staining and crevice corrosion attributable to the high salt exposure and regular dampening by salt fog. When the bridge was designed, a higher alloyed cable and mesh than Type 316 could not be obtained, so some staining was expected on these elements. The Type 316 spider fittings showed no corrosion on the exposed elements. Photographs of the bridge from 2018 are shown in Figure 27.

(a) (b)

Figure 27. Harbor Drive Pedestrian Bridge: (a) overall view; (b) close-up of Grade 2205 duplex stainless steel suspender. Photograph courtesy of Catherine Houska.

Case 6: West 7th Street Bridge in Ft. Worth, Texas

In 2013, the Texas Department of Transportation completed the West 7th Street Bridge in Ft. Worth as a replacement for the 100-year-old bridge that existed previously (International Molybdenum Association, 2018). Aside from being the world's first precast concrete network arch bridge, the structure is unique because of its Grade 2205 duplex stainless steel hanger bars. Nearly 110 tons of steel was used to construct the 1¾-in-diameter angled hanger bars that connect the top of the bridge arch to the tie. The link plate and pin and hanger components were also fabricated from Grade 2205 steel plate, whereas the clevises were cast from Type 316 austenitic stainless steel. The corrosion-resistant steels were chosen for this bridge because of their excellent corrosion resistance and structural properties found during initial testing conducted by the Texas Department of Transportation in addition to their aesthetics. One of the designers of the bridge stated that the use of the duplex stainless steel and concrete in compression brings the bridge closer toward "infinite durability" (International Molybdenum Association, 2018).

CONCLUSIONS

Galvanized Weathering Steel

- *GWS has performed well after 6 years on the Genito Road Bridge.* Overall, the galvanizing coating is in satisfactory condition without any major concerns. There is one minor imperfection on the galvanized coating that will continue to be monitored. It is important to inform the galvanizer that Grade 50W steel can be a "reactive" steel and to consider the composition of the steel when galvanizing the steel to ensure a detrimental increase in the coating thickness is not formed. Many of the initial concerns regarding GWS appeared to be based on a lack of evidence rather than on poor experience.

A1035CS Steel Plate

- *Because of its inherent corrosion resistance and excellent yield stress and tensile strength, steel plate meeting the chemistry requirements of ASTM A1035CS has the potential for use in steel bridge applications.* Further research is necessary to develop a full understanding of the mechanical properties of A1035CS steel plate, including CVN toughness. The steel is expected to have good toughness, with values that either meet or exceed those of Grade 50 steel. Currently, there are no known published studies on the weldability of A1035CS steel plate. (It should be noted that ASTM A1035CS is a reinforcing steel specification. The plate tested in this study met the chemistry requirements of ASTM A1035CS reinforcing steel but was tested in plate form.)

50CR Steel Plate

- *ASTM A709 Grade 50CR steel possesses good tensile mechanical and fatigue properties that have met or exceeded the requirements of Grade 50 steel.* The bridge design process using

50CR steel will be similar to that using Grade 50 steel. Grade 50CR steel falls under the ASTM A709 specifications and meets the same mechanical property requirements of Grade 50 steel. The fatigue performance of slip-critical bolted connections made with 50CR steel showed similar or better performance compared to Grade 50 steel. Unfortunately, the 50CR steel welded fatigue samples exhibited undercuts and did not meet the workmanship requirements for steel bridge fabrication, so direct comparisons to Grade 50 steel could not be made.

- *Grade 50CR steel can be successfully fabricated by a traditional U.S. steel bridge fabricator.* To date, the steel has been successfully fabricated and used for five plate girder bridges in the United States and two in Canada. Although there seemed to be a learning curve in welding and fabricating with 50CR steel, all of the challenges were overcome with relative ease.

- *Grade 50CR steel has good corrosion resistance in steel bridge applications.* This corrosion resistance was seen in site visits to a 50CR steel plate girder bridge. The steel is performing well and showed no signs of overall or pitting corrosion in a location with large amounts of de-icing salts and low volume, heavily loaded truck traffic.

Duplex Stainless Steel

- *Duplex stainless steels have better mechanical properties compared to Grade 50 steel.* Duplex stainless steel bridge designs using a yield stress of 65 ksi and an ultimate strength of 90 ksi may be warranted. Bridge designs using duplex stainless steels would be similar when compared to typical bridge steels. The increased strength of duplex steels could be leveraged to reduce material, thereby reducing cost. Of course, designs with smaller girder sections would still need to satisfy all limit states, including buckling requirements. The toughness and fatigue performance of duplex stainless steels also exceed those of Grade 50 steel.

- *Because of its excellent corrosion resistance, duplex stainless steels have successfully been used as critical structural members on bridges.* The initial cost of duplex steels will be more than for Grade 50 steel, but in the pedestrian bridges in the United States, duplex stainless steel was used only for select members that required enhanced corrosion resistance. By using duplex steel for targeted locations, the material can be used where it is most beneficial while still reducing overall cost. Similar to what VDOT has experienced with corrosion-resistant reinforcing steel, duplex stainless steel can provide excellent durability for steel plate members.

- *Current guidance exists for the design and fabrication of duplex stainless steel structural members. AWS D1.6/D1.6M Structural Welding Code—Stainless Steel* (AWS, 2017) provides guidance on the welding and fabrication of duplex steel. This includes information about which filler materials should be used when duplex stainless steels are being welded. The *AISC Design Guide 27: Structural Stainless Steel* (Baddoo, 2013) also provides guidance on the design and fabrication of duplex steel structural members. Commercial rolled or welded structural shapes currently exist for duplex stainless steels and could be attractive options for secondary members used with duplex primary members.

RECOMMENDATIONS

1. *VDOT's Structure and Bridge Division should consider implementing the use of ASTM A709 50CR steel in regions where uncoated weathering steel is not recommended in FHWA's Technical Advisory 5140.22 (FHWA, 1989) or where site limitations do not allow the use of concrete girders.*

2. *VTRC should work with VDOT's Structure and Bridge Division, Hampton Roads District, and Fredericksburg District to identify locations and bridge types that would benefit from the use of corrosion-resistant steels and identify strategies for minimizing cost when these materials are used.*

3. *VTRC should work with VDOT's Hampton Roads District during the design, fabrication, and construction of the Jenkins Bridge Road Bridge over Holdens Creek to be constructed of 50CR steel, including long-term monitoring of the durability of the structure.*

4. *VTRC should work with VDOT's Structure and Bridge Division and Hampton Roads District to evaluate the dissimilar metal bolted repairs to the Onancock Bridge and to initiate development of guidelines for the use of 50CR steel bolted to other ASTM A709 steels.*

5. *VTRC should initiate a study on the use of dissimilar metal welded connections consisting of ASTM A709 steels, including 50CR, and duplex stainless steels.*

6. *VTRC should initiate a study on steel plate meeting the chemistry requirements of A1035CS and AASHTO 334M Alloy Type 1035 CS or similar-type steel plate for use as a steel bridge material.*

IMPLEMENTATION AND BENEFITS

Implementation

Implementation of Recommendation 1 will include future discussions with members of VDOT's Structure and Bridge Division, VDOT's Materials Division, and VTRC that continue to evaluate locations where 50CR steel should be used, including areas where uncoated weathering steel beams are currently not recommended for use or where concrete beams cannot be used. Implementation of Recommendation 1 will occur within 1 year of the publication of this report.

Implementation of Recommendation 2 will include meetings, either in person or by telephone, with VDOT's Structure and Bridge Division, Hampton Roads District, and Fredericksburg District to determine potential bridge projects using corrosion-resistant steels in ways to minimize project cost. One possibility for this could be a steel beam bridge with a timber deck. Typically, these types of structures have been constructed with coated rolled steel beams. Implementation of Recommendation 2 will occur within 2 years of the publication of this report.

Implementation of Recommendation 3 will include VTRC initiating a study to assist VDOT's Hampton Roads District with evaluating the design, construction, and long-term evaluation of the Jenkins Bridge Road Bridge using 50CR steel. Implementation of Recommendation 3 will occur within 1 year of the publication of this report.

Implementation of Recommendations 4, 5, and 6 will be similar. VTRC will assemble a technical review panel and identify a champion within VDOT to initiate a study for the three recommendations. Implementation of Recommendations 4, 5, and 6 will occur within 2 years of the publication of this report.

Benefits

Implementation of Recommendation 1 will provide VDOT more options when replacing structures and promote competition with corrosion-resistant concrete girders in areas where steel has not been traditionally used. Grade 50CR steel is expected to provide VDOT with adequate corrosion resistance, lower dead load, architectural aesthetics, and girder depth flexibility and will be easier to inspect when compared to concrete girders. Similar to Grade 50W steel, 50CR steel can also be spliced in the field to allow for long span structures.

Implementation of Recommendation 2 will allow VDOT to use corrosion-resistant steels where they are most needed. Using corrosion-resistant steels in an application where rolled steel beams and a timber deck would allow for the option of using a corrosion-resistant plate girder with a service life much better than its predecessor, because of both the material's inherent corrosion resistance and the ability to increase thickness as compared to a rolled beam. There is the potential for significant initial cost savings by maximizing the material ordered and fabricating a group of beams under a single purchase order.

Implementation of Recommendation 3 will allow VDOT and VTRC to learn about the design, fabrication, construction, and cost of using a 50CR steel plate girder in a highly corrosive environment on Virginia's eastern shore.

Implementation of Recommendations 4 and 5 will provide VDOT a cost savings by using corrosion-resistant steel in areas where it is most suited and provides the greatest benefit. Dissimilar metal bolted and welded connections can be used for both repairs and maintenance-free designs. For example, VDOT can use 50CR and duplex steels in highly corrosive areas, such as repair plates for corroded beam ends. For new designs, VDOT can elect to use 50CR or duplex steels in areas close to water and can use either 50W or galvanized steel in other areas. This allows for corrosion-resistant steels to be used only in areas where it is most needed, resulting in a cost savings for the overall structure.

Implementation of Recommendation 6 will provide VDOT with a cost savings by gaining information about additional CRP types that could promote more competition in plate girders that require additional corrosion resistance and a rustic patina for aesthetics. The proposed study could include investigating the CVN, fatigue, weldability, and formability of other CRP types.

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