ANALYSIS AND CONSTRUCTION OF THE UNITED STATES' FIRST COMPLETELY STAINLESS STEEL BOLTED SPLICE ON A STEEL GIRDER HIGHWAY BRIDGE

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SUMMARY

In 2017, construction was completed on the Virginia Department of Transportation's (VDOT's) first ASTM A709 Grade 50CR (50CR) (formerly referenced as ASTM A1010) stainless steel bridge in Waynesboro, Virginia. In order to realize a truly low-maintenance structure, VDOT elected to use Grade 50CR steel for the bridge girders and cross frames, and to use stainless steel bolts for the bolted connections. Until this point, most of the bolts used on the other five 50CR steel bridges in the country had been either Type 3 (weathering steel) or galvanized bolts, although Oregon Department of Transportation (ODOT) used stainless steel bolts for cross frame connections. For its first 50CR steel bridge, VDOT decided to use stainless steel bolts for the steel girder bolted field splice connections, making it the first of its kind in the country.

Based on corrosion tests, mechanical properties, and cost considerations, VDOT elected to use ASTM A193 Grade B8 Class 2 stainless steel bolts, which had also been used by ODOT. This paper will describe the analysis and construction of the splice and how it differed due to the presence of stainless steel bolts. Key aspects covered in the paper include how the smaller clamping force and ultimate tensile strength affected the splice design and the modified bolt acceptance and installation procedures necessary for constructing the splice.

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Introduction

Corrosion of steel and concrete girder bridges and other transportation structures is a concern for the Virginia Department of Transportation (VDOT) both near the coastline and inland. Structures near the coastline are exposed to saltwater, while those inland are exposed to de-icing salts which are applied during snow and ice events. In both cases, the chlorides from the salt cause corrosion damage of the steel. This deterioration can lead to costly repairs. For example, a recent study indicated that VDOT spends approximately $105 million per year on bridge maintenance, with approximately 10% of this cost being used for bridge coating maintenance (1).

Traditionally, corrosion has been mitigated through the use of coatings or weathering steel. Coatings typically consisted of either a three-coat paint system or galvanized steel. Painted steel has shown poor performance in areas of high salt exposure. Once the paint system begins to fail, costly maintenance actions must take place or the underlying steel will corrode relatively quickly. Galvanized steel can perform better in salt laden areas but does include other challenges. Its corrosion resistance is often dependent on the quality of the processing during the galvanizing procedure, which can be variable across galvanizing plants and even projects within a plant. The galvanizing baths can also often limit the size of steel members which can be galvanized.

Uncoated weathering steel has been successful at reducing corrosion in some climates, but can exhibit poor performance in others. This led FHWA to publish a technical advisory in 1989 which detailed areas in which weathering steel is not expected to perform well, such as areas near marine coastlines, those with a high time of wetness, and those near industrial locations (2). The combination of poor corrosion resistance in some areas and the FHWA technical advisory led some agencies to have a negative view on the use of uncoated weathering steel.

More recently, VDOT and other state agencies across the country have begun pursuing other options for providing enhanced corrosion resistance. One such option is the use of a low grade stainless steel, ASTM A709 Grade 50CR steel (50CR), formerly referenced as ASTM A1010 (A1010) (3, 4). In this case, "low grade" refers to the fact that Grade 50CR contains less chromium, nickel, and other alloying elements found in higher grade stainless steels, such as austenitic and duplex steels. Although both of these higher grade stainless steels have better corrosion resistance than Grade 50CR, they are much more expensive. Grade 50CR allows for additional corrosion resistance compared to traditional bridge steels, while still remaining cost effective.

In accelerated corrosion tests, Grade 50CR has shown excellent corrosion resistance, with a corrosion rate of 1/10 that of weathering steel in the vertical direction and 1/4 that of weathering steel in the horizontal direction, with the difference being attributed to the increased time of wetness when placed horizontally (5, 6). This enhanced corrosion resistance has led to six 50CR steel bridges built since 2004 in the following states: California, two in Oregon, Pennsylvania, Iowa, and Virginia.

VDOT is the most recent agency to have constructed a bridge made of 50CR steel; the Rt. 340 Bridge in Waynesboro, VA was opened to traffic in June 2017. Grade 50CR steel was selected for the bridge over uncoated weathering steel because of its close proximity over the South River, a chemical plant located upstream of the bridge, and for general aesthetic concerns (7). In most of the previous 50CR bridges, the cross frames and bolts had been made of weathering or galvanized steel. In order to provide a truly corrosion resistant structure, VDOT elected to use 50CR steel for the cross frames and stainless steel fasteners for all bolted connections. The Oregon DOT (ODOT) had used stainless steel fasteners for the bolted connections between the girders and cross frames, but VDOT's Rt. 340 Bridge was the first to use stainless steel fastener assemblies in a slip-critical bolted girder field splice application. This paper will describe the process of analyzing and constructing the nation's first completely stainless steel bolted field splice in a steel girder highway bridge.

Selection of Stainless Steel Bolted Fastener Assemblies

Once the decision was made to use stainless steel fasteners, the Virginia Transportation Research Council (VTRC) and VDOT conducted a research study to evaluate the mechanical properties, availability, and cost of various stainless steel bolted fasteners to determine which type would be most suitable for use on the Rt. 340 Bridge (8). A summary of this research study is as follows.

Since VDOT practices specify the use of ASTM F3125 Grade A325 (A325) bolts to be used in bridge applications, the researchers wanted to find stainless steel bolts that possessed similar mechanical properties and were readily available (9). A literature review revealed that ASTM A193 (A193) bolts met the desired strength requirements (10). Within the A193 specification, the following bolt grades were selected for testing and analysis: B6, B8 Class 2 (B8-2), and B8M Class 2 (B8M-2). The B8 and B8M Class 2 bolts were selected because they are strain hardened so they provide greater tensile strengths than their Class 1 counterparts. Once it was determined that these three types of stainless steel bolts met the requirements of Buy America, bolts were procured and testing commenced (11). Type 1 (uncoated) and galvanized A325 bolts were also included in the tests for comparison.

The experimental testing and analysis included uniaxial tensile tests, modified rotational capacity tests, hardness, optical and scanning electron microscopy, and sensitization testing for the Grade B8-2 and B8M-2 bolts. The results of these analyses concluded that the Grade B8 Class 2 fastener assemblies were most suited for use on the Rt. 340 Bridge. This is the same grade of stainless steel bolts that ODOT had used with success on one of their bridges constructed with Grade 50CR steel. The B8-2 bolts showed a combination of high strength and ductility, with minimal variability between individual tests. The nominal tensile strength of these bolts was 125 ksi based on their ASTM specification. Experimental tensile test results showed that these bolts had an ultimate strength of approximately 135-140 ksi.

During the modified rotational capacity tests, the bolts were tightened in 20° increments and tensile and torque measurements were recorded at each increment. This test data was then evaluated to determine that a B8-2 bolt did not develop tensile force as great as a standard A325 bolt did; a 7/8 in. diameter (VDOT's standard size) B8-2 bolt was able to safely develop a clamping force of 30 kips, compared to the specified 39 kips for a A325 bolt of the same diameter.

In most cases, the B8-2 bolts were able to develop forces greater than 30 kips, but the decision was made to limit the design clamping force to this value due to instances of galling between the nut and bolt threads. Galling is a known phenomenon between stainless steel threaded parts in which they essentially become cold-welded together. When galling occurs during the tensioning process, the additional friction drastically increases the torsional stresses in the bolt. The resulting combined torsion and tensile stresses decrease the tension that the fastener can develop. The use of Never-Seez® High Temperature Lubricating Compound was shown to reduce the effects of galling. Limiting the design clamping force to 30 kips also reduced the potential for galling between the threaded parts.

Microscopy of the bolt fracture surfaces revealed that the B8-2 bolts failed in a ductile fashion, which agreed with the necking observed during tensile testing. The B8-2 bolts were also evaluated to determine if the as-received bolts were subject to sensitization. Sensitization refers to poor heat treatment of stainless steel where chromium particles migrate to the grain boundaries to form chromium carbides. This leaves chromium-depleted zones near the grain boundaries, which can cause premature intergranular corrosion. Microscopy evaluation revealed that none of the B8-2 bolts in the study showed signs of sensitization or intergranular corrosion.

These test results and analyses led to the following fastener assembly to be recommended for the Rt. 340 Bridge bolted connections: A193 Grade B8 Class 2 bolts, ASTM A194 (A194) Grade 8 nuts, and grade 304 hardened washers (12).

Rt. 340 Bridge Bolted Splice Analysis

Fastener Assembly Design Properties

The results of the B8-2 bolt testing provided foundational information for use in the Rt. 340 Bridge splice design. Although ASTM A193 specifies a nominal tensile strength of 125 ksi for B8-2 bolts and uniaxial tensile tests supported this value, the design tensile strength of the bolts was reduced to 100 ksi to provide an additional margin of caution. The tensile testing had included only a small number of bolts from the same lot, so the tensile strength was reduced to account for possible differences in the multiple bolt lots likely to be used on the Rt. 340 Bridge. This additional conservatism was also provided since this was the first application in which Grade B8-2 bolts were to be used in slip-critical bolted field splice for a highway girder bridge.

The modified rotational capacity testing of 7/8 in. diameter B8-2 bolts demonstrated they could consistently achieve a clamping force of 30 kips, which is less than the clamping force of 39 kips for the standard A325 bolt of the same diameter. For an A325 bolt, a 39 kip clamping force corresponds to 70% of the nominal ultimate strength. For the B8-2 bolt, the 30 kip clamping force is approximately 65% of the nominal ultimate strength (using 100 ksi). This difference is due to both the conservative reduction in nominal ultimate strength and because modified rotational capacity test data showed that the B8-2 bolts could safely achieve a clamping force of 30 kips, rather than the standard 39 kips. Table 1 summarizes the project findings in design properties for a 7/8 in. diameter A193 Grade B8-2 bolt and a 7/8 in. diameter A325 bolt.

Table 1. Design properties comparison for standard 7/8 in. diameter bolt

Bolt Type	Nominal Tensile Strength, F_{ub} (ksi)	Design Clamping Force, P_t (k)	% F_{ub} For P_t
B8-2	100	30	65%
A325	120	39	70%

Aside from the difference in the bolts, the difference in the plate material had to be accounted for in the design of the splice. The current AASHTO LRFD Bridge Design Specifications provide tabulated values for the surface condition factor used in the calculation of the slip resistance of a bolt in a slip-critical connection (13). These tabulated values were developed based on experimental testing of plain, painted, weathering, and galvanized steel, both in the unblasted and blasted condition. Since the majority of these tests were conducted well before Grade 50CR steel had been considered for use in steel bridges, no tests had been conducted on 50CR to determine its surface condition factor.

The 50CR steel used on the Rt. 340 Bridge was also blast cleaned using a non-metallic garnet blast media. The tests of blast cleaned surfaces that were used to develop the AASHTO surface condition factor values had been blasted with either sand or steel shot media. Since garnet blast-cleaned 50CR steel surfaces were not addressed with the AASHTO specifications, VDOT elected to assume a Class B surface condition for the Rt. 340 Bridge. This surface condition class corresponds to unpainted blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings, and it was consistent with the requirements for the 50CR on the bridge, i.e., unpainted and blast-cleaned.

As previously stated, the bolt research study had originally recommended that grade 304 washers be used on the Rt. 340 Bridge. However, cost and availability ultimately resulted in the washers being specified as grade 303. Although grade 303 steel has slightly decreased corrosion resistance relative to grade 304, it is also an austenitic stainless steel, and the substitution is not expected to affect the overall durability of the fasteners or the bridge. Table 2 shows the fastener assemblies used on the Rt. 340 Bridge and the specification or grade of steel used for each part.

Table 2. Fastener assemblies used on Rt. 340 Bridge

Fastener Assembly Part	Specification or Grade Used
Bolt	ASTM A193 Grade B8 Class 2
Nut	ASTM A194 Grade 8
Washer	Grade 303

Analysis of Additional Stainless Steel Bolts Required

All of the analyses herein were conducted after the Rt. 340 Bridge was constructed, with the goal of these analyses being to illustrate the differences in the bolt quantity required for the splice for using the B8-2 bolts instead of conventional A325 bolts, as well as using the 8th Edition AASHTO LRFD specifications, rather than the 7th Edition (13, 14). The difference in specification edition is notable because significant changes were made to the bolted splice design provisions between the 7th and 8th editions of the AASHTO LRFD. Originally, the intent of these analyses was only to highlight the differences in the splice design due to the use of the B8-2 bolts rather than conventional A325 bolts, in hopes of providing useful information to designers using stainless steel bolts for future bolted splice connections. However, since the Rt. 340 Bridge was designed shortly before the 8th Edition AASHTO LRFD was published, the authors opted to include analyses comparing the splice design using both specification editions. This was done so that any conclusions reached would be applicable to future structures since the most recent AASHTO LRFD specification was considered.

The actual Rt. 340 Bridge bolted splice design drawings were used to develop the analysis models to ensure the geometry in the models would match the actual splice geometry in the bridge. The design loads acting on the splice in the analyses were determined using inputs from the bridge design drawings and from National Steel Bridge Alliance (NSBA) program, LRFD Simon v10.2.0.0. The design drawings also provided the bolt quantity on the actual splice which could be used as a comparison to the results of the analyses.

Once the splice geometry and design loads were known, the Rt. 340 Bridge splice was analyzed using two versions of NSBA software: NSBA Splice v1.0 and NSBA Splice v2.02. Two versions of the NSBA Splice software were used because each program follows a different edition of the AASHTO LRFD; Splice v1.0 follows the 7th Edition, while Splice v2.02 follows the 8th Edition. Both versions of NSBA Splice have predefined inputs allowed for some of the fields used to design a bolted splice. One of these predefined fields is for the grade of steel used for the girder and splice plates. Since 50CR steel was not available as an allowable input, Grade 50W was selected as a proxy since both steels have the same yield stress and ultimate strength. Another field that had to be changed was the type of bolt used in the connection. Both versions of Splice initially only allow for input of the two types of bolts typically used for bridges in the United States: an A325 bolt or an ASTM F3125 Grade A490 (A490) bolt. Since Splice v1.0 is an executable program, the bolt properties are not allowed to be changed by the user. For this version of Splice, a 3/4" in. diameter A325 bolt was selected to serve as proxy for a 7/8 in. diameter B8-2 bolt in the program. Table 3 shows the design properties of both bolts.

Table 3. Design properties of 7/8" B8-2 and 3/4" A325 (simulated B8-2) bolt

Diam. & Bolt Type	Design Clamping Force, P_t (k)	Design Shear Strength, R_n (k)
7/8" diam. B8-2	30.0	26.8
3/4" diam. A325	28.1	23.2

As shown in the table, both the design clamping force and design shear strength are relatively similar for the two bolts, having differences of 6.3% and 13.4%, respectively. In both design properties, the simulated B8-2 bolt design values are slightly less than actual B8-2 bolt values. This provides a slightly conservative comparison between the two bolt types when using the 7th Edition of the AASHTO LRFD, while not grossly over-predicting the number of simulated B8-2 bolts needed to satisfy the design requirements. Unlike v1.0, Splice v2.02 is a spreadsheet-based program, and therefore could be more easily modified by the user; the tensile strength and design clamping force of an actual 7/8 in. diameter B8-2 bolt were input into the program. This provided a more direct comparison between the two bolt types when using the 8th Edition of the AASHTO LRFD. Hand calculations were also necessary as a supplement in a few instances due to limitations of the software programs. Since neither Splice v1.0 nor v2.02 allows for bolt stagger, hand calculations were necessary since stagger was present in the Rt. 340 Bridge flange splices. These hand calculations were conducted on design checks that included a net section area, such as net section fracture and block shear rupture.

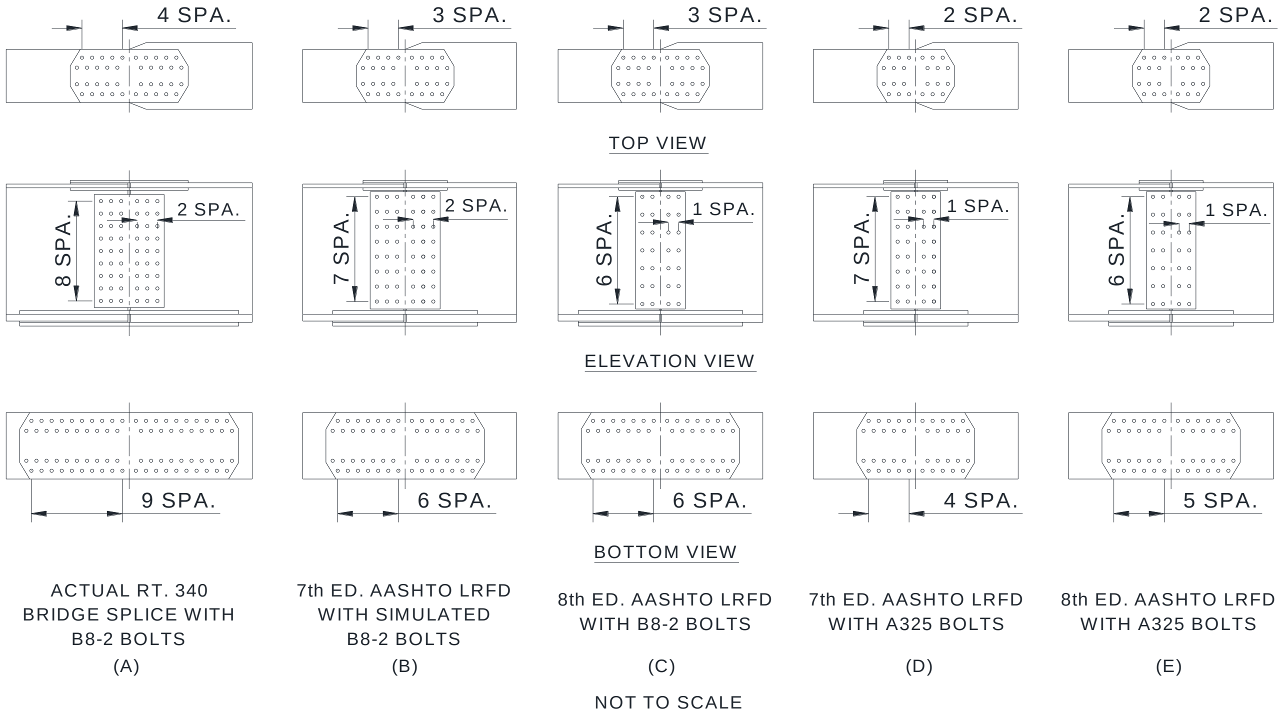


Figure 1. Illustrative drawing of bolt quantity required for each design and analysis case: (A) Actual Rt. 340 Bridge splice with B8-2 bolts, (B) 7th Edition AASHTO LRFD with simulated B8-2 bolts, (C) 8th Edition AASHTO LRFD with B8-2 bolts, (D) 7th Edition AASHTO LRFD with A325 bolts, and (E) 8th Edition AASHTO LRFD with A325 bolts.

In addition to analyzing the Rt. 340 Bridge splice design according to the two most recent AASHTO LRFD splice design specifications, analyses were performed with the two versions of NSBA Splice using typical 7/8 in. diameter A325 bolts instead of the A193 bolts that were actually used in the bridge. This comparison would indicate how many more bolts were needed in the splice due to the reduced design strength of the B8-2 bolts. Figure 1 shows a drawing of the design to illustrate the number of bolts needed for each analysis case.

Table 4 provides a comparison of the required bolt quantity per splice side in the top flange, web, and bottom flange of the Rt. 340 Bridge splice design. The first line in the table shows the actual number of B8-2 bolts in the Rt. 340 Bridge splice, as determined using the bridge design drawings. The remaining lines in the table show the various analysis cases to illustrate the difference in bolt quantity required for the same splice geometry and loading when considering both the difference in bolt type and splice design specifications.

Table 4. Required bolt quantity per splice side comparison between actual Rt. 340 Bridge splice design and analysis cases using different bolts and design specifications

Bridge / Specs and Bolt Type	# TF[1] Bolts	# Web Bolts	# BF[2] Bolts
Actual Rt. 340 Bridge with B8-2 bolts	20	27	40
7th Ed. specs with simulated B8-2 bolts	16	24	28
8th Ed. specs with B8-2 bolts	16	14	28
7th Ed. specs with A325 bolts	12	16	20
8th Ed. specs with A325 bolts	12	14	24

[1]TF = Top Flange
[2]BF = Bottom Flange

One clear observation from examination of Figure 1 and Table 4 is that the actual Rt. 340 Bridge splice was conservatively designed; it is likely that additional bolt rows were added as a measure of precaution since this type of bolt had not been used in a highway bridge splice application prior to this project. Moreover, comparisons can also be made between the bolt quantities determined under the 7th and 8th Editions of the AASHTO LRFD. Table 5 shows the percentage increase in number of B8-2 bolts required in the top flange, web, bottom flange, and total number per each side of the splice under each splice design provision.

Table 5. Increase in bolt quantity when using B8-2 instead of A325 bolts on Rt. 340 Bridge splice

Specifications Used	TF Bolts	Web Bolts	BF Bolts	Total Bolts
7th Ed. specs	+33%	+50%	+40%	+42%
8th Ed. specs	+33%	+0%	+17%	+16%

Overall, both the 7th and 8th Editions require an increase in bolt quantity when using B8-2 bolts rather than A325 bolts, which is expected due to the reduced nominal ultimate strength and clamping force. When examining the 7th Edition specifications, there is a 42% increase in the total number of bolts required for the Rt. 340 Bridge splice. A large portion of this difference comes from the 8 additional bolts (+50%) required in the web. The quantity of additional bolts required more than compensates for the percentage reduction in either tensile strength or clamping force between the two bolt types. An examination of Figure 1 (B) and (D) shows that this increase was due to an entire additional column of required B8-2 bolts in the web. This overcompensation shows that, in addition to the reduced strength of the B8-2 bolts, the geometry of the splice will determine the additional quantity of B8-2 bolts needed when compared to using A325 bolts. Similar observations can be made about the number of bolts required in the flanges using the 8th Edition specifications.

Finite Element Analysis of Bolted Splice

Finite element analysis was conducted on the actual bolted splice used in the Rt. 340 Bridge to demonstrate its effectiveness relative to the design assumptions. Due to time constraints, only one splice design could be analyzed using finite element analysis; the decision was made to analyze the Rt. 340 Bridge splice so that results could be obtained for the splice that was actually built.

The modeling technique used as part of this analysis is thoroughly described in Ocel (15); work presented in this reference was used, in part, as justification in balloting of the new bolted splice design provisions in the 8th Edition of the AASHTO LRFD. In short, the technique was to represent the deck, girders, splice plates and fill plates as shell elements. However, individual bolts are represented with fastener elements that strictly connect two surfaces together with nonlinear stiffness properties.

The existing model was revised to the geometry of the Rt. 340 splice, and revisions were also made to the material models for the base material and bolts. The material model of the 50CR base material was not too unlike that reported in Ocel (15), though it was based on actual tension test data of 50CR steel and scaled to ideal yield and tensile strengths of 50 ksi and 70 ksi, respectively.

Since B8-2 bolts were used in the Rt. 340 Bridge, new submodels of lap splices with a single bolt were created to form the shear force vs. displacement behavior of the fastener elements in the splice model. Further details of the submodeling can be found in Ocel (15), though they were modified to use the same 50CR steel material and the bolt material was based on tension testing of an actual B8-2 bolt supplied from a lot used in Rt. 340 bridge, though the measured stresses were scaled to have the ideal 100 ksi tensile stress assumed in design.

Three submodels were constructed, with one for the top flange, bottom flange, and web splice bolts because of the differences in plate thickness in those three locations. The resulting nonlinear behavior of the fastener elements from the three respective submodels is shown in Figure 2. Again, these three shear force vs. shear displacement curves define the nonlinear behavior of the fastener elements used to connect the various shell elements together in the model of the entire bolted splice.

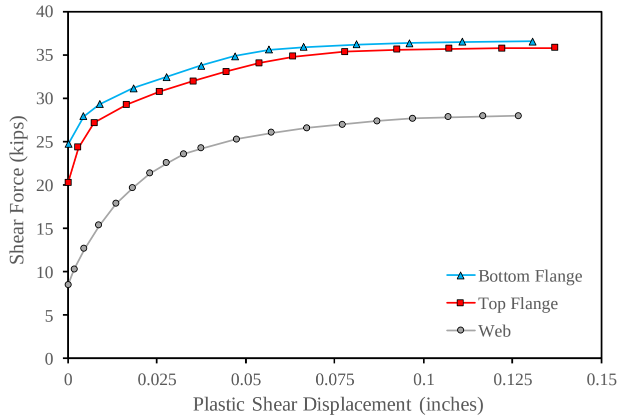
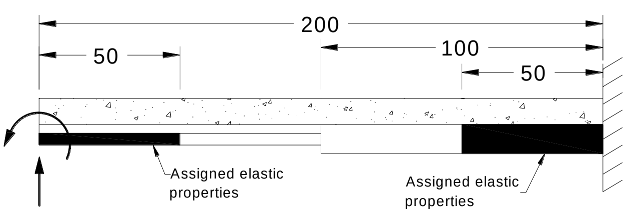


Figure 2. Shear force vs. shear displacement behavior of bolt fastener elements.

The top and bottom flange models result in similar behavior, with an expected maximum of approximately 35 kips, i.e., $0.58*(1/4\pi d^2)*100$. The web bolt demonstrated reduced strength and lower stiffness than the flange bolts. This was due to extreme bearing deformation in the 1/2 in. thick web plate in the submodel, which must be accounted for because the full spliced girder model with shell elements does not have the fidelity to capture bearing deformation.

The dimensions of the full model are shown in Figure 3. The figure also shows that the two girder sections and bolted splice were modeled as a cantilevered beam with loads applied at the tip. Two loading scenarios were analyzed based on the loading arrangement shown in Figure 3.



Units=inches

Figure 3. Details of bolted splice model.

The first scenario applied 442 kips of shear and 19,369 kip-in. of moment at the tip of the cantilever; this is referred to as the "positive moment" scenario. The second, "negative moment" scenario used a tip shear load of -442 kips and -26,345 kip-inch of moment. These two loading scenarios were selected to produce the design moment and design shear at the location of the splice on the actual bridge. The concrete deck was not present in the model under negative moment. A view of the actual model, without the concrete deck, is shown in Figure 4.

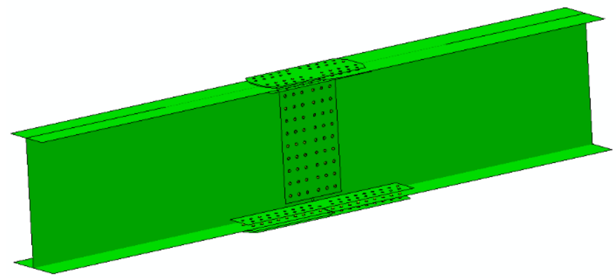


Figure 4. Snapshot of bolted splice model.

The model uses the Von Mises yield criterion to capture nonlinear material behavior. Figure 5 shows the Von Mises stress patterns around the splice under the positive moment scenario. The contours are plotted from 0 to 50 ksi, therefore, according to the bounds of the legend, if yielding was occurring it would be represented by a grey color. From the figure, the highest stressed portion is the girder web, while the flanges and all the various splice plates do not exceed roughly half their yield resistance. The high Von Mises stress in the web is dominated by the shear stresses since the 442 kips of shear is near the design limit of the web.

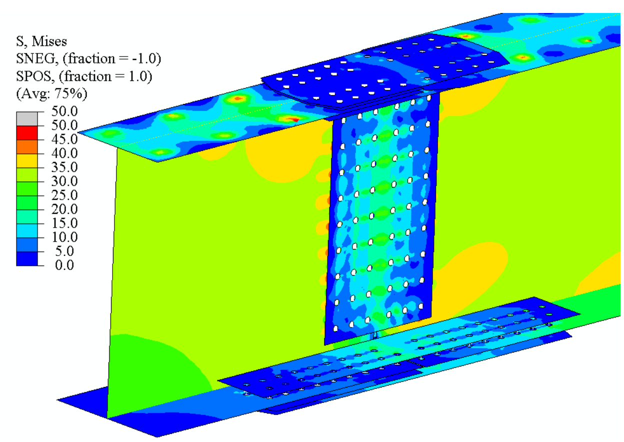


Figure 5. Von Mises stress under positive moment scenario.

The bolt forces in individual shear planes are shown in Figure 6. This plot shows the bolt forces do not exceed roughly 14 kips, which is well below the limit state of an individual B8-2 fastener.

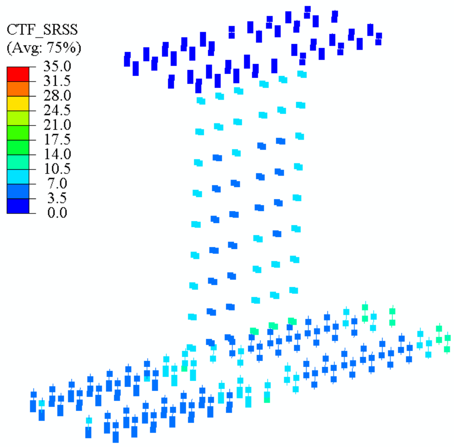


Figure 6. Bolt shear forces under positive moment scenario.

The Von Mises stresses under the negative moment scenario are shown in Figure 7. The negative moment scenario results are relatively similar to the positive moment scenario; the girder webs are the highest stressed and the splice plates themselves are not stressed more than half their yield resistance. Likewise, the 442 kips of shear in the girder webs dominated the Von Mises stress.

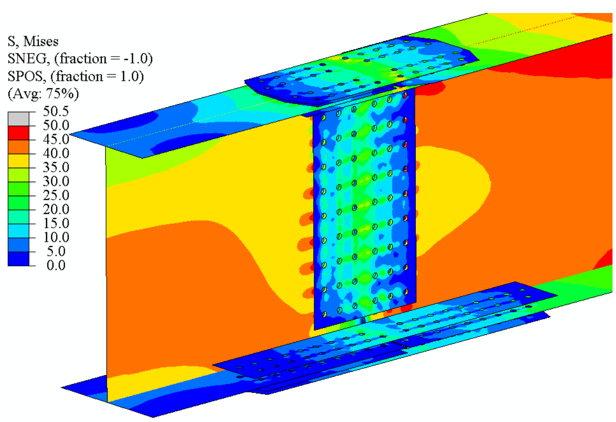


Figure 7. Von Mises stress under negative moment scenario.

The bolt forces under the negative moment scenario are shown in Figure 8. Similar to the positive moment scenario, the individual shear forces under negative moment do not exceed roughly 14 kips, well below the limit the B8-2 bolts could sustain.

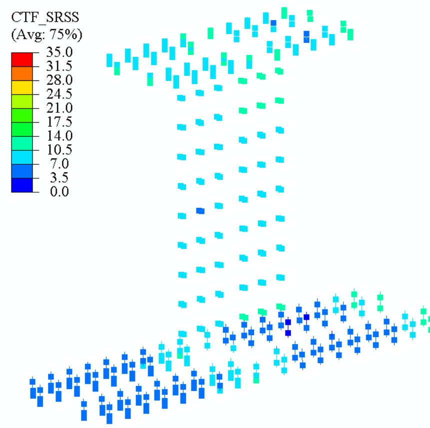


Figure 8. Bolt shear forces under negative moment scenario.

Results from the finite element modeling indicate that the Rt. 340 Bridge bolted splice is conservatively designed for the design loading scenario, with notably low stresses in the splice plates and B8-2 bolts. The conservative nature of the splice design is not surprising, given that it was the first time that stainless steel bolts had been used in a steel girder highway bridge splice application in the United States.

Installation of Stainless Steel Fasteners

Development of Modified Procedures

Since a B8-2 bolt was only able to achieve a clamping force of 30 kips, rather than the 39 kips of a standard A325 bolt, modified bolt acceptance and installation procedures had to be developed. The test results from the bolt research study were used as a starting point, and additional tests were also conducted on a calibrated Skidmore-Wilhelm bolt tension measuring device to aid in the development of these procedures.

The modified acceptance and installation procedures are similar to those used for standard A325 bolts, but with different values used for the parameters such as clamping force, maximum allowable torque, and required nut rotation. The turn-of-nut pretensioning method was used for tightening the bolts in both methods. Multiple meetings and demonstrations were held between VDOT officials, the contractor, and erector such that the modified procedures could be discussed prior to acceptance and installation of the bolts on-site at the Rt. 340 Bridge. The key differences between the B8-2 and A325 bolts with respect to both of these procedures are discussed next.

Acceptance Testing

A calibrated Skidmore-Wilhelm device and a calibrated torque wrench were used throughout the acceptance testing procedure. First, a small amount of Never-Seez® High Temperature Lubricating Compound lubricant was applied evenly to the threads of the bolt, and additional lubricant was used to cover all of the bolt, nut, and washer contact surfaces. Next, the nut was tightened to a standard initial tension loading of 4 kips and collinear lines were marked on the wrench socket and testing apparatus.

The nut was then tightened until a tensile force of 30 kips was achieved in the B8-2 bolt. This value corresponds to the consistently repeatable clamping force found in the bolt research study. At this point, the angle of nut rotation and torque values were recorded. The maximum allowable torque value was given by the following equation: $T = 0.25\ P \times d_b$, where T is the torque in ft.-lbs, P is the tensile force of the bolt in lbs, and d_b is the bolt diameter in ft. For a 7/8 in. diameter B8-2 bolt under a 30 kips tensile load, this maximum allowable torque is equal to 550 ft.-lbs. For reference, VDOT allows a maximum torque value of 710 ft.-lbs for A325 bolts (16). This difference in allowable torque is due to the difference in clamping force between the two bolt types.

The nut was then tightened to the rotation listed in Table 6. The levels of rotation listed in this table are those required to produce the design clamping force in the B8-2 bolts. For comparison, the table also provides the required rotation for installing A325 bolts (17). In general, the nut rotation required to produce the design clamping force in the B8-2 bolts is slightly greater than those for the A325 bolts.

Table 6. Required rotation necessary to meet design clamping force

Bolt Type	$L^1 \leq 4d_b^2$	$4d_b > L \geq 8d_b$	$8d_b > L \geq 12d_b$
B8-2	1/2 turn	2/3 turn	1 turn
A325	1/3 turn	1/2 turn	2/3 turn

[1]L = Length of bolt

[2]d_b = Bolt diameter

At this point in the acceptance testing, the tensile force and torque in the bolt were recorded again, with the maximum allowable torque still given as 550 ft.-lbs. The nut was then tightened to the rotation level shown in Table 7. The rotation levels are also provided for A325 bolts for reference (16).

Table 7. Required rotation necessary to produce at least 1.15 times clamping force in bolt

Bolt Type	$L \leq 4d_b$	$4d_b > L \geq 8d_b$	$8d_b > L \geq 12d_b$
B8-2	1 turn	1-1/3 turn	1-1/2 turn
A325	2/3 turn	1 turn	1-1/3 turn

These rotation levels in Table 7 are designed to produce a tension force in the bolt of at least 1.15 times the required clamping force of 30 kips, or 34.5 kips. Once the required rotation levels were obtained, the tension and torque values were recorded, and the tension level in the B8-2 bolt was checked to ensure it met the minimum required tensile force of 1.15 times the clamping force. Once the bolt was verified to meet the required clamping force, the B8-2 bolt was removed from the Skidmore-Wilhelm device. The thread condition was then checked by rethreading the nut onto the bolt by hand to determine if it could be tightened to the location at the end of the test. Figure 9 shows a photograph taken during a demonstration of the acceptance testing of the B8-2 bolts.

Figure 9. Demonstration of B8-2 bolt acceptance testing.

One important observation from the demonstration acceptance testing is that the type of lubrication used can greatly affect the torque level in the B8-2 bolts.

In one of the demonstration acceptance test meetings, the erection contractor compared two different types of lubricant on separate bolts during testing. When the contractor's alternative lubricant was used, each of the bolts surpassed the maximum allowable torque level prior to reaching their specified tensile clamping force due to galling between the nut and bolt threads. When the recommended Never-Seez® lubricant was used, each of the bolts successfully reached its design clamping force without exceeding the maximum allowable torque. These results were consistent with those found during the bolt research study (8). ODOT had also reported success using the same lubricant when using B8-2 bolts for their cross frame connections in one of their 50CR steel bridges. For these reasons, VDOT elected to specify the use of the Never-Seez® lubricant for all bolted connections on the Rt. 340 Bridge.

Field Installation

Once the acceptance testing was successfully conducted, the B8-2 bolts were installed into the cross frame and bolted splice connections on the Rt. 340 Bridge. First, Never-Seez® lubricant was applied to all threaded parts and the washer contact surfaces. The nuts were initially tightened to a snug-tight position. Collinear lines were then marked with a paint marker on the steel plate being connected, nut, and wrench socket used to tighten the nuts. Since the turn-of-nut pretensioning method was used, finish marks were also drawn on the steel plate to indicate the stopping position of the nut when the bolt could be considered pretensioned. The location of these finish marks were determined from the rotation level given in Table 6 for B8-2 bolts. Once the bolts were installed, the construction inspector then used a calibrated torque wrench to ensure that torque values in the bolts did not exceed the maximum allowable torque of 550 ft.-lbs. Figure 10 shows one of the bolted splice connections on the Rt. 340 Bridge after installation of the B8-2 bolts.

As an aside, Figure 10 clearly shows that the girders on either side of the splice are different colors. This is due to the surface finish on the girders and the conditions in which each were shipped. The 50CR girders and splice plates were blast-cleaned with non-metallic, garnet blast media prior to leaving the fabricator's facility. The girder on the left was shipped to the job site on a clear, sunny day. The girder on the left was shipped through snow on roadways in which de-icing salt had been applied. This de-icing salt accelerated the formation of the patina. The girder on the right is expected to be a similar color of brown once its patina forms.

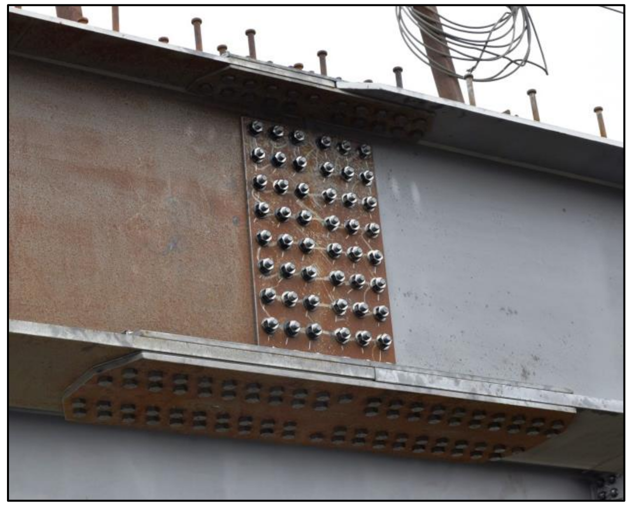


Figure 10. Rt. 340 Bridge bolted splice after installation of B8-2 bolts.

The bolt installation process went relatively smoothly. Although the rotation level requirement for tightening was different for the B8-2 bolts than what is typically used for A325 bolts, the erection contractor did not have any problems with this modified tightening procedure. One unexpected issue that occurred early in the tightening process was that the lines, made with a paint marker to indicate the starting and ending rotation position, would not adhere to the stainless steel nuts. Several marks began to rub off as the nuts were tightened. This issue was resolved by marking the face of the nuts with a center punch tool so that a permanent mark would remain on the nut. Once the nuts were marked with a center punch, the bolt installation process continued without additional issues.

Cost of Stainless Steel Fastener Assemblies

During the initial bolt study that determined that Grade B8-2 bolts were well suited for the Rt. 340 Bridge, the procurement process enabled costs to be recorded (8). The Grade B8-2 fastener assembly order pursuant to the bolt study consisted of 30 B8-2 bolts, 30 A194 Grade 8 nuts, and 60 Grade 304 washers. A similarly small quantity of standard Grade A325 bolts and accompanying nuts and washers, available off the shelf, were also ordered for the tests. The cost of the stainless steel and carbon steel fastener assemblies for the bolt study are noted in Table 8. All bolts shown in the table had a diameter of 7/8 in. and were 3.5 in. long and all nuts and washers were sized for a 7/8 in. bolt.

Table 8. Small quantity cost comparison of stainless and standard steel fastener assemblies

Item	A325 Fastener Assembly Cost	B8-2 Fastener Assembly Cost in Study
Bolt	$ 1.88	$14.65
Nut	$ 1.90	$16.50
Washer	$ 0.31	$ 2.60
Total Cost	$ 4.09	$33.75

Table 8 shows that the cost of a Grade B8-2 fastener assembly was approximately 8 times the cost of a standard Grade A325 fastener assembly in the "small quantity" order for the study, with each item having a cost increase of similar proportion.

In discussions with the Rt. 340 Bridge fabricator at the close-out meeting after bridge erection was complete, the fabricator stated that since a large quantity of the B8 fastener assemblies was purchased, the actual final cost of the assemblies reflected the potentially significant savings to be realized with economies of scale in order volume and therefore production processes. For the approximately 3,000 fastener assemblies installed on the Rt. 340 Bridge, the fabricator reported the final cost to be only some 20% greater than the cost of standard A325 fastener assemblies.

Conclusions

The following conclusions were reached during this study:

- ASTM A193 Grade B8 Class 2 bolts and accompanying nuts and washers are readily available in the United States and can meet the Buy America requirements.
- The B8-2 bolts can be successfully used in a steel bridge bolted splice application, using a nominal tensile strength of 100 ksi and a design clamping force of 30 kips for a 7/8 in. diameter bolt.
- Due to the reduced design forces when compared with standard Grade A325 bolts, additional B8-2 bolts are likely to be required when designing a bolted splice, though the percentage increase in bolt quantity will depend on the splice geometry.
- Finite element analysis showed that the actual Rt. 340 Bridge was conservatively designed, with low stresses seen in the splice plates and B8-2 fasteners.
- Modified acceptance and installation procedures were successfully developed for B8-2 bolts, including the level of rotation required for a 7/8 in. diameter bolt to reach its design clamping force.
- The type of lubrication used during the tightening process for a B8-2 bolt has a significant effect on the achievement of design clamping force before reaching maximum allowable torque due to galling between the stainless steel bolt and nut.
- Paint markers were not sufficient for recording the initial mark on the nuts before tightening the bolts using the turn-of-nut pretensioning method. The face of the nuts should be physically marked (e.g., by means of a center punch) or an alternative method of effectively marking the nuts should be employed.
- In the small quantity procured for the bolt study, a B8-2 bolt and accompanying fastener assembly cost approximately 8 times more than a standard A325 bolt fastener assembly. In the large quantity procured for the Rt. 340 Bridge, B8-2 fastener assemblies cost about 20% more than standard A325 assemblies.
- The use of B8-2 fastener assemblies for additional corrosion resistance will add cost to a project because of the increased number of B8-2 fasteners required relative to A325 fasteners and because of the increased costs of B8-2 fastener assemblies. For example, if the Route 340 Bridge were designed anew according to the 8th Edition AASHTO LRFD, the required number of B8-2 bolts would be 16% greater than if A325 bolts were used, due to the specific splice geometry in this bridge. If the B8-2 bolts carry the reported cost premium of 20% over A325 bolts, the substitution of B8-2 bolts could boost the fastener cost component by approximately 40% over the alternative with standard A325 fasteners.

Future Work

Currently there are two research projects being conducted at VTRC related to the use of stainless steel bolted connections. The first project involves slip coefficient testing on 50CR steel, with various types of surfaces finishes. These tests will aid in determining the surface condition classification of 50CR for future bolted splices. Tests are also being conducted on hybrid steel bolted connections in which 50CR steel is bolted to either weathering or galvanized steel; this is in anticipation of using hybrid 50CR girders or girder sections, where 50CR will be used only where necessary for corrosion resistance to realize cost savings.

The second project at VTRC involves a series of mechanical and corrosion tests on corrosion resistant bolts. Bolts tested in the study include the B8-2 bolts used on the Rt. 340 Bridge, Grade 2205 stainless steel bolts, and bolts manufactured from a martensitic stainless steel. Mechanical tests include those necessary to qualify bolts as ASTM F3125 Grade A325 to determine if corrosion resistant bolts can meet the same criteria. Various types of lubrication will also be tested to determine criteria for selecting lubricant for stainless steel bolts. Long term corrosion tests will compare the durability among the different fastener assembly types.

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