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Welding Innovation

Advancing Arc Welding Design and Practice Worldwide

A publication of the James F. Lincoln Arc Welding Foundation

A Foundation for Progress

The James F. Lincoln Arc Welding Foundation celebrates its 65th birthday this year. Far from preparing to retire, the Foundation is actually reinvigorated with a new sense of purpose and energy. We remain the only organization in the United States *solely* devoted to educating the public about the art and science of arc welding. Now, with the aid of electronic technology and the Internet, we are poised to take a major step into the international arena by sponsoring a new James F. Lincoln Arc Welding Foundation web site. Development work is already underway. Until the new, independent site is up and running, information about current Foundation activities is available on the Lincoln Electric web site: www.lincolnelectric.com

When I had my first contact with the Foundation in the 1950s, I would hardly have imagined the organization and its programs going online! Neither the word nor the concept existed. Many technological advances have been made since that time, but as we contemplate the challenges of the future, perhaps we should also review our history.

Born in the depths of the Great Depression, the Foundation was the idea of its namesake, James F. Lincoln, whose innovative approach to industrial management eventually led The Lincoln Electric Company to world prominence in the arc welding field. The original Deed of Trust stated: "The object and purpose of such fund and foundation … is to encourage and stimulate scientific interest in the development of the arc welding industry … and to that end to provide for awards to those persons who by reason of the excellence of their papers upon said subject may be selected … as the most worthy to receive such awards."

The first award, granted in 1936, was for $5,000, an amount about equivalent to the Nobel Prize of that day. Over the last six-and-a-half decades, the individual cash awards granted by the Foundation have ranged from a $50 Merit Award in the School/Shop Program to a $25,000 Best of Program Award in the Professional Program. Literally thousands of students and engineers have benefited from participating in the award program.

The value of the program to the welding industry has been, if anything, even greater than the sum of its importance to individuals, however. Early on, the Trustees of the Foundation realized the importance of collecting and publishing the discoveries of those who had entered their arc welding projects in the competition. The first book, *Arc Welding in Design, Manufacture and Construction*, ran to 1,402 pages and when it was published in 1939, sold for $1.50, *postage included!* The value of the knowledge that has been shared from the dissemination of this information over the years is incalculable. We continue to publish award winning papers in this magazine today (including the cover story of this issue).

In 1948, the Engineering Student Design Competition was initiated. Since I have spent most of my career in academia, I will admit that this program is very close to my heart. In fact, it was in 1956 when I was a young assistant professor at Colorado State University that I first learned of the Foundation, and the college awards served as my introduction. I have more fully described the program's background, objectives and significance in an article on page 13 of this issue, entitled "A Unique Mechanism for Enhancing Engineering Education."

As we take The James F. Lincoln Arc Welding Foundation into cyberspace to support a new, international level of programming, President Roy Morrow, Executive Director Duane Miller and I are focused on the standards of excellence that have always defined this organization. We look forward to receiving and responding to your feedback on our award programs, book publication activities and, of course, *Welding Innovation*.

Donald N. Zwiep
Chairman, The James F. Lincoln Arc Welding Foundation

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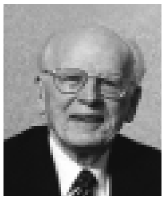
Welding Innovation

Cover: Welded trusses like this one support the retractable roof of Seattle's new Safeco Field stadium.



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The James F. Lincoln Arc Welding Foundation

Features

The serviceability of a product or structure utilizing the type of information presented herein is, and must be, the sole responsibility of the builder/user. Many variables beyond the control of The James F. Lincoln Arc Welding Foundation or The Lincoln Electric Company affect the results obtained in applying this type of information. These variables include, but are not limited to, welding procedure, plate chemistry and temperature, weldment design, fabrication methods, and service requirements.

Departments

Out-of-Plane Fatigue Cracking in Welded Steel Bridges

Why It Happened and How It Can Be Repaired

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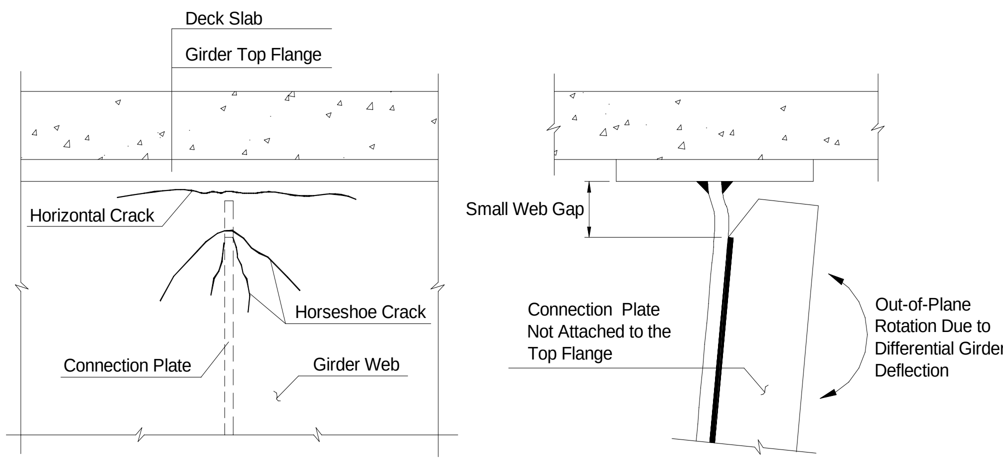

Figure 1. Formation of out-of-plane distortion-induced fatigue cracking.

Background

The Interstate construction boom from the late 1950s through the 1970s built many of the steel highway bridges currently in service in the United States. However, due to the lack of in-depth research on the fatigue performance

Out-of-plane distortion accounts for the largest category of fatigue cracking nationwide

of both the structural components and the connection details, a large portion of the bridges constructed during that era have developed fatigue cracks in service. Often, welded bridge details are more susceptible to fatigue cracking than bolted or riveted ones. Discontinuities in the welds form crack initiation sites at imperfections such as entrapped porosity, lack of fusion or penetration, or incomplete removal of slag. Fractures can also initiate from geometrical stress risers, such as fillet weld toes. Subsequent crack propagation would occur if the surrounding material is exposed to a cyclic tensile stress field. Unfavorable residual stresses can exacerbate the already severe condition of stress concentration and accelerate the process of fatigue crack propagation in these localized regions. Since attached plates are fused together by welding, a continuous path is provided for crack growth from one plate to another. Of the various crack types observed in welded steel bridges, those caused by out-of-plane distortion have been recognized as the largest category of fatigue cracking nationwide [Fisher and Menzemer, 1990].

Out-of-Plane Distortion

Out-of-plane fatigue cracking occurs mostly at locations where transverse structural components such as floorbeams, diaphragms, or cross-frames are framed into longitudinal girders through connection plates. Before and during the early 1980s, the connection plate detail was designed by following the early European practice of not welding to the girder tension flange to avoid having a category C fatigue detail. Sometimes the connection plate was not attached to the compression flange, either. However, as shown in Figure 1, an unstiffened portion of the web gap was then left during service and was susceptible to being pulled out-of-plane when the end of the transverse structural member rotated under traffic loading. Distortion-induced cracks developed unexpectedly at both the web-to-flange and web-to-connection-plate fillet

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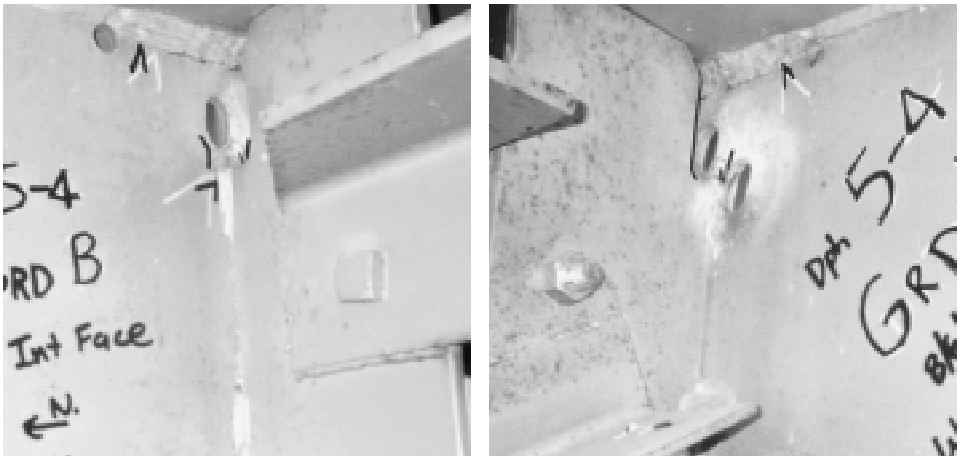

(a) north side of connection plate (b) south side of connection plate

Figure 2. Cracking and repair condition on each side of a connection plate in the Fancy Creek Bridge.

welds, typically as horizontal or horseshoe cracks, as indicated in Figure 1.

A research project currently underway at the University of Kansas is studying the fatigue behavior and repair approaches for the out-of-plane distortion-driven cracks experienced by many Kansas Department of Transportation (KDOT) welded plate girder bridges. Figures 2 and 3 exhibit development of typical horizontal and horseshoe cracks in two KDOT bridges. Web gaps near the girder top flanges are the most common location for these cracking problems. The top flange is held rigid by the deck slab above, so a more abrupt stiffness change occurs than that at the bottom flange, which is relatively free to move laterally. Cracks most frequently occur in the positive moment regions of the bridge girders, where the differential girder deflections are the largest and the out-of-plane bending moments are the highest. The common conditions observed in KDOT bridges that have led to web gap cracking are: 1) no positive attachments provided between the connection plates and the girder flanges; and 2) no additional stiffener plates erected on the other side of the girder web as would have been done at bearing stiffeners. If either one of these two countermeasures had been carried out, a rigid load path could have been formed between the transverse members and the longitudinal girders, and the chances of forming out-of-plane fatigue cracking would have been slight.

In order to better understand the history of the distortion-induced fatigue and to obtain more information about crack repair solutions and experiences, the authors of this article reviewed the different editions and interims of the AASHTO bridge design specifications published in the past twenty years, and conducted two surveys among different DOTs and others with an interest in steel bridges. The first survey was carried out in 1999 within the North Central States and Federal Highway Administration Region 3, and the second one was performed in 2000 through the email list of AASHTO/NSBA Steel Bridge Collaboration (thelist@steelbridge.org). The input from the surveys provided both valuable insights into the retrofit mechanism of the out-of-plane fatigue cracking and detailed implementations employed in the repair of other DOTs' bridges.

Cracks most frequently occur in the positive moment regions of the bridge girders

Evolution of Connection Plate Design Detail Specs

Generally speaking, the detailing of connection plates has never been specified independently as an individual section in the AASHTO *Standard Specifications for Highway Bridges*. From the first time it was mentioned in the specifications (1982 Interim), design of connection plates has been always included in either the section covering transverse intermediate stiffeners or the section covering diaphragms and cross-frames. It was not until the issuance of the first AASHTO LRFD edition in 1994 that the rationale of distortion-induced fatigue was fully explained and the connection plate design detail was clearly and correctly specified in a separate section.

The story of the connection plate detail should date back to the 1981 Interim, which states that *"Intermediate stiffeners … may be in pairs … with a tight fit at the compression flanges … When stiffeners are used on one side only of the web plate, they shall be fastened to the compression flange"* and *"Transverse intermediate stiffeners need not be in bearing with the tension flange."* Strictly speaking, stiffeners and connection plates are different concepts in terms of their structural purposes. However, the same plate can fulfill both functions. Since distortion-induced fatigue was not a widely recognized problem at that time, the specifications were normally

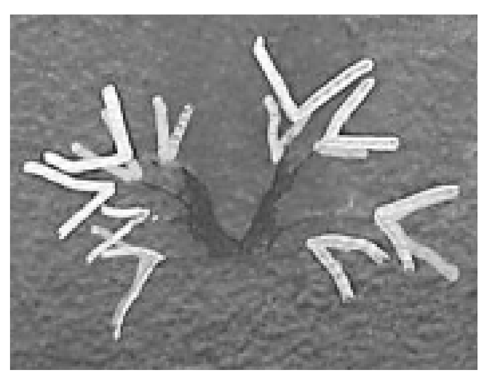

Figure 3. Horseshoe cracks observed in the Hump Yard Bridge.

interpreted as having the stiffener details requirements also applying to connection plate details. In other words, the connection plate function was seen as subordinate to the intermediate stiffener function.

The 1982 Interim mentioned connection plate details explicitly for the first time in AASHTO. The aforementioned statement for the stiffener-to-compression-flange connection was revised to *"Stiffeners provided on only one side of the web must be in bearing against but need not be attached to the compression flange for the stiffener to be effective; however, consideration shall be given to the need for this attachment if the location of the stiffener or its use as a connector plate for a diaphragm or cross-frame will produce out-of-plane movements in a welded web to flange connection."* The authors understand this statement to mean that the connection plate was allowed, but was not required, to be attached to the compression flange. The connection plate to tension flange detail was still not explicitly addressed. By default, the relationship between a stiffener and the tension flange would be applied, implying that no welded or bolted connection was needed.

Stiffeners and connection plates are different concepts in terms of their structural purposes

In 1983, the 13th AASHTO edition changed to the now current format. The former description of the stiffener-to-compression-flange connection appeared in section 10.34.4.6, and that of the stiffener-to-tension-flange connection appeared in section 10.34.4.9. The contents of these two sections were the same as in the 1982 Interim and were kept unchanged until 1995. Design of diaphragms and cross-frames was specified in section 10.20. No information about connection plate details was mentioned in the 1983 Interim.

The 1985 Interim added the important statement to section 10.20.1 that *"Vertical connection plates such as transverse stiffeners which connect diaphragms or cross-frames to the beam or girder shall be rigidly connected to both top and bottom flanges."* This is the first time AASHTO required that connection plates be attached to both girder flanges. However, those related provisions previously covered in section 10.34.4 for transverse intermediate stiffeners remained the same, which made the specifications very unclear. Unwillingness to change the old design habit, in addition to the ambiguity of the specifications, delayed the process of preventing or eliminating out-of-plane fatigue cracking in newly built bridges. For example, KDOT started welding or bolting connection plates to both girder top and bottom flanges in early 1989. Fatigue cracking has not been observed to date in bridges designed since this practice was adopted. However, almost all those welded plate girder bridges built with the pre-1989 detail were found with fatigue cracks in the web gap area.

Finally, in the 1995 Interim, the connection plate detail was made clear and the following revised statement was repeated both in section 10.34.4.6 for the compression flange connection and in section 10.34.4.9 for the tension flange connection. *"... However, transverse stiffeners which connect diaphragms or cross-frames to the beam or girder shall be rigidly connected to both the top and bottom flanges."*

The AASHTO LRFD *Bridge Design Specifications*, available since 1994, clearly specify that *"Connection plates shall be welded or bolted to both the compression and tension flanges of the cross-section."* Explanation of distortion-induced fatigue is given in section 6.6.1.3 and its corresponding commentary, and the requirement of rigid attachment between connection plates and girder flanges is addressed in section 6.6.1.3.1 for transverse connection plates, section 6.7.4.1 for diaphragms and cross-frames, and section 6.10.8.1.1 for transverse intermediate stiffeners.

Retrofitting Out-of-Plane Fatigue Cracks

Different repair methods, either having already been used in actual bridge retrofits by DOTs, or still being researched, are described as below. This is a summary based on responses to the two surveys previously mentioned.

Hole drilling

The traditional repair method shown in Figure 4 consists of drilling a hole at the crack tip. The hole diameter is sized to be at least 2ρ, where ρ is determined by Equation 1 [Barsom and Rolfe, 1999].

$$\frac{\Delta K}{\sqrt{\rho}} < 4\sqrt{\sigma_y} \quad (\text{for } \sigma_y \text{ in ksi}) \tag{1}$$

ΔK is the stress intensity factor range and σ_y is the yield strength of the specified steel. This repair is especially effective when arresting crack propagation in low stress regions. However, cracking may recur if the hole size is not large enough or the stress range at the crack location increases. If this is the case, a

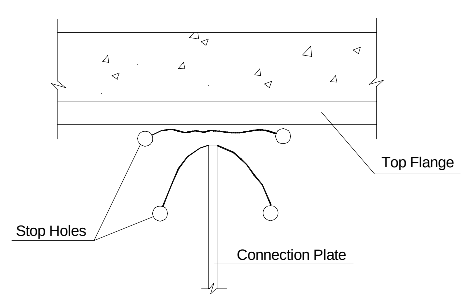

Figure 4. Repair by drilling stop holes at the crack tips.

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supplemental step can be taken either by cold working the hole or by filling the hole with a pretensioned high strength bolt, as will be described in the sections to follow, so that the crack front is restrained from further propagation. Hole drilling is easy to perform and should be used wherever possible even when other repairs are also employed at the same time.

Cold expansion

Cold expansion is an approach mostly used in aircraft and railway rails for fatigue life enhancement of rivet or bolt holes. It is often performed by pulling a tapered mandrel, such as is used in the split sleeve process [Cannon et al., 1986], through one side of the hole to the other, in order to expand the hole diameter and to produce plastic deformation in the periphery. A zone of residual compressive stresses, both radially and circumferentially, is then formed, so that the initial fatigue resistance of the area surrounding the hole can be greatly improved. However, this method has not been seen in use for crack repair by any bridges. The stop holes in bridge repairs are often drilled, intercepting the fatigue cracks at the very ends. Cold expansion, therefore, would not be effective unless the holes were placed a certain distance away from the crack tips to provide room for the formation of the compressive stress field. In addition, the crack locations, which often form at plate-to-plate connection fillet welds, make it almost impossible to accommodate tools (such as a sleeve) needed for cold expanding. This method still can be used in bridge fatigue cracking repair if the cracks have extended to a free surface away from member intersections and the repair holes are drilled at a small distance away from the crack ends. But in most cases, installing pretensioned bolts is a more cost-effective and widely used method of strengthening the repair holes.

To release the constraints at the cracked area, the diaphragms were lowered to rest on the bottom flanges

Filling drilled holes with pretensioned bolts

Preloaded high-strength bolts are often used to prevent crack reinitiation from the drilled holes. This method forms local compressive stresses perpendicular to the member surface around the hole, creates friction between the faying surfaces, and effectively keeps the crack from recurring. It has been used very often for the repair of out-of-plane fatigue cracking in the web gap region, in addition to the hole drilling approach.

Stiffening the web gap

Hole drilling alone can only stop the growth of existing cracks, not the formation of new cracks. Some other measures have to be taken to make the floor-beam (or diaphragm, cross-frame) to girder connection either more rigid or more flexible, so that not only are the existing cracks arrested, but also that no more cracks develop. If stiffening the web gap is desired, then either a welded or a bolted connection plate detail (Figures 5 and 6, respectively) may be used. The welded detail is the simplest, but it can only resist stress ranges up to AASHTO fatigue detail Category C. Weld quality is a concern if an overhead position is required. A bolted detail repair can improve the fatigue resistance to detail Category B. As shown in Figures 6(a) and 6(b), either an angle or a T-section can be used to bolt the connection plate to the girder flange. However, if the repair is performed at the top flange, part of the deck slab has to be removed for bolt installation.

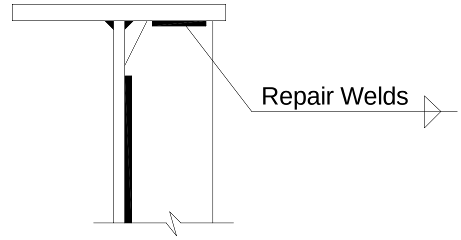

Figure 5. Stiffening web gap by welding connection plate to girder flange.

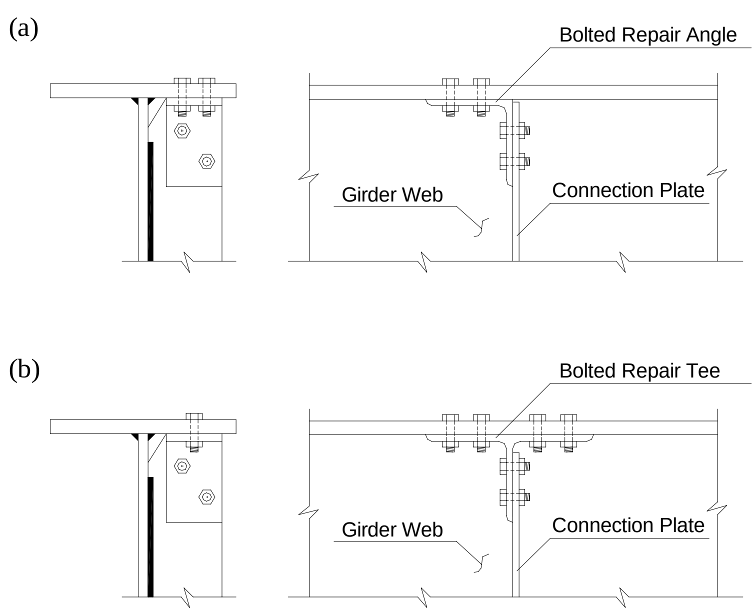

Figure 6. Stiffening web gap by bolting connection plate to girder flange.

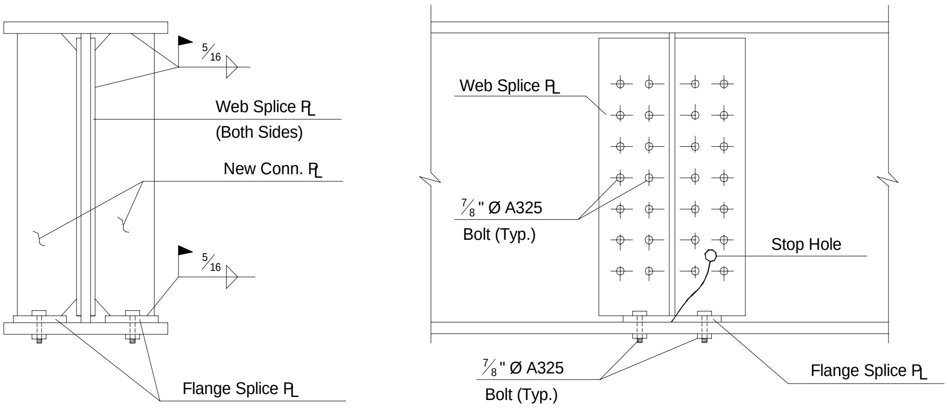

Figure 7. Repair of girder web by using splice plates.

Bolted splices
If large fatigue cracks have developed deep into the girder web, the load-carrying capacity of the main structural member is impaired, which may affect the structural integrity of the bridge. This is especially of concern when the cracks are located in a tension zone. As shown in Figure 7, the repair can be performed by removing the original connection plates and bolting reinforcing splices (or coverplates) on both sides of the web. New connection

Rewelding, if well performed, could at least restore the original member capacity

plates also need to be connected rigidly to girder flanges, either by welding or bolting. Thus the cracked web is stiffened and the girder section properties are restored by this retrofit.

Cutting the connection plate back
This method was used in 1980 for the retrofit of Des Moines Bridge [Fisher, 1984]. It has since been used by Iowa DOT on about 50 two-girder bridges experiencing small web gap cracks. None of them have yet experienced cracking problems after the repair. Bridges in other states, such as the Lexington Avenue Bridge (Minnesota), the Poplar Street Bridge (Illinois), and the Midland County Bridge (Texas) [Keating et al., 1996], were also repaired by employing this approach at the web gap locations. As illustrated in Figure 8, part of the connection plate is cut back so that the area of the girder web below the flange is sufficiently flexible to accommodate the out-of-plane rotation. Both field and laboratory tests showed that the secondary stress is significantly reduced after the connection is softened. The cut surface should be well finished to prevent crack reinitiation. To efficiently release the restrained web, a minimum cut-short dimension of 12 in. (300 mm) is recommended for the connection plate [Fisher et al., 1990].

Diaphragm removal
Diaphragms and cross-frames are important during construction because they provide lateral bracing to the girders and stabilize the entire structural system. Once the deck slab is placed, they are no longer needed if construction stability is their only function. Removing interior diaphragms can completely eliminate the secondary stresses that cause fatigue cracks in the girder web, but it can also increase the in-plane bending stresses in the main girders. Stallings et al. [1996 & 1998] performed field testing of both completely and partially removing diaphragms of two Alabama DOT bridges. The findings indicated that a 15% increase in girder stress can reasonably be expected after the repair. Thus it is recommended that repair of out-of-plane fatigue by diaphragm removal only be considered for bridges with rating factors exceeding 1.15.

This repair method should be used with caution since it would increase the girder stresses and decrease the structural resistance against unexpected loading conditions such as earthquake or vehicle collisions. Care should also be taken to make sure that any subsequent removal of the concrete slab considers girder stability.

Bolt loosening
Wipf et al. [1998] investigated the effect of repair by loosening the cross-frame-to-connection-plate bolts on five Iowa DOT bridges. Field measurement indicated that the maximum web gap stress ranges at the tested locations were reduced by 25–85%, the maximum out-of-plane distortion was reduced by 20–88%, and the maximum forces in the diaphragm diagonals were reduced by 73–95%. Compared with the diaphragm removal method, bolt

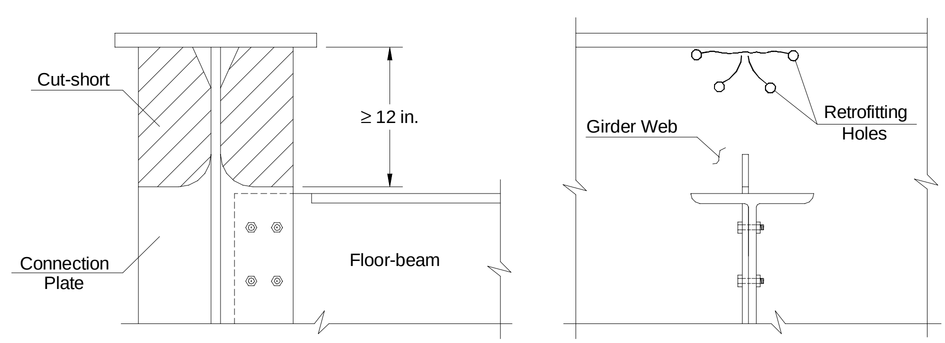

Figure 8. Repair by cutting connection plate back.

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loosening has advantages in that it is easier to perform on the site, does not increase girder bending stresses, provides lateral resistance in case of extreme events, and by retightening bolts stabilizes the structure when the deck needs to be replaced.

Diaphragm repositioning

This method was used in the repair of a Minnesota DOT bridge that experienced fatigue cracking in web gaps. Diaphragms were originally located near the girder top flanges. To release the constraints at the cracked area, the diaphragms were lowered to rest on the bottom flanges. At a minimum, stress is decreased in the affected areas by a factor of two. This repair option has similar advantages to bolt loosening when compared to diaphragm removal.

Rewelding

This repair method usually requires gouging out the existing cracked welds before the new welds are applied, and grinding smooth the rewelded surface after the new welds are filled. Although not recommended by many DOTs due to the expensive labor required to guarantee sound weld quality and smooth surface finishing, it is the last choice if other repair methods cannot effectively stop the crack growth. NCHRP Report 321 [Gregory et al., 1989] studied the repair of fatigue cracking by welding and provided guidance for achieving good quality welds. The experimental work conducted for this research showed that rewelding, if well performed, could at least restore the original member capacity and provide the same fatigue life as the original shop welds.

Peening

Peening is used to inhibit cracking process by impacting the weld toes with pneumatic hammer or automatic shot peening equipment. Residual compressive stresses are introduced and fatigue resistance can be improved by one category at the weld termination area. It has been used for many cover plate end reinforcements [Welsch, 1990] and was most effective when arresting propagation of shallow cracks (less than 3 mm) [Fisher, 1998].

Gas tungsten arc (TIG) remelting

This method reduces the stress concentration at the weld toes. Fatigue resistance can be increased by one category after the repair [Fisher, 1998]. However, it is difficult to perform under vibration once the bridge is in service.

Using composite materials

Bassetti et al. [2000] studied retrofitting fatigue cracks by using newly developed composite materials such as prestressed carbon fiber laminates (CFRP – Carbon Fiber Reinforced Polymers). Prestressed CFRP strips oriented perpendicular to the crack faces could slow down or even completely stop crack propagation. Experimental testing of a retired riveted railway bridge is currently underway in Switzerland. The application of this material is still in the research phase.

Summary

Distortion-induced fatigue cracking has been observed in many KDOT welded steel bridges due to the past use of fatigue-prone connection plate details. This article explains the formation of the out-of-plane fatigue, reviews the changes in the AASHTO provisions for the connection plate detail design, and briefly describes different kinds of methods that have been used for crack retrofit.

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Lessons Learned in the Field

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Persistence Pays Off

Figure 1. The design of the roof beams had to be such that this would not happen!

Editorial Note: Last year, Omer W. Blodgett initiated a new forum for Welding Innovation readers, "Lessons Learned in the Field." Inviting readers to contribute their own accounts of lessons they learned on the job, rather than in the classroom, Mr. Blodgett cited the opportunity to "take our blinders off, expand our limited world view, and test our assumptions." Rob Lawrence of Butler Manufacturing was the first to respond, with his submission of this piece. The editors look forward to hearing from more of you who have similar "ah-ha!" experiences to share.

The Project

At Butler Manufacturing, we received an order for a metal factory building to be used for the manufacture of candy. The design of the roof beams posed a special challenge due to the possibility that factory dust might accumulate on the top side of the bottom flanges over the years, and then fall as a clump into a batch of candy while it was being made, ruining it (Figure 1). To eliminate this possibility, the design engineer had specified that the bottom flanges must be covered with sloping covers. The original specification required welding the full length of the steeply sloped covers over the bottom flanges of the roof beams. According to the design specifications, the covers had to be welded; they could not be fastened using adhesive.

The heat would expand the cover faster than either the web or the flange

Fabrication Challenges

Due to the welding requirement and to simplify the erection of the building, we decided to weld the covers offsite in our fabrication plant. The original design called for a simple 1/8 in. (3 mm) thick flat plate welded to the vertical web of the beam and to the horizontal bottom flange at the outer edge (Figure 2). We attempted to weld samples to develop the welding procedure specification. These weld samples quickly revealed four problems with the design:

1. It was very difficult to get the pieces to fit at the outside corner of the flange since the flange was rounded.

2. Due to the steep slope of the plate, it was difficult to find the weld joint at the edge of the flange, as it was almost a butt joint.

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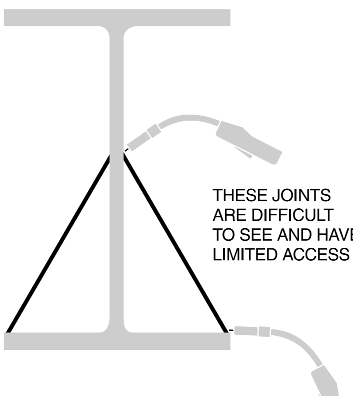


Figure 2. The original design posed some problems.

3. If the cover was welded to one side first, the beam would sweep in the direction of the welded side.

4. Attempts to weld the joint resulted in separation of the cover plate from the web or flange due to the different thermal expansion rates for the web, flange and cover.

The cover was ⅛ in. (3 mm) thick, the web was ¼ in. (6 mm) thick, and the flange was ½ in. (13 mm) thick. When this joint was being welded, the heat would expand the cover faster than either the web or the flange. After some length of welding, usually about 8 in. (200 mm), the cover would bow away from the tack welded joint. More frequent tacking was attempted, but the expansion would break the tack welds. Furthermore, the tack welds caused visually unacceptable "lumps" in the welds. By using a back-stepping weld sequence, we could get the welding done, but the fastest travel speed attainable by hand was about 12 ipm (300 mm/min.) and the welding had to be performed in 8 in. (200 mm) increments. There were covers on all the roof beams and mezzanine beams. This was going to require an unacceptable amount of time.

Discussions with the design engineer resulted in the cover design being changed to a formed shape with a lap joint to the web and a flare-vee joint to the flange (Figure 3). These design changes made it much easier to fit the covers and provided better weld joints both to the web and to the flange. The attachment to the web was now a "real" fillet weld, and the attachment to the flange was a flare vee that made it much easier to see and follow the joint when welding.

There was still the problem of the required back-stepping welding technique that was very slow. Further experimentation, using a side beam seam welder in an attempt to speed up the welding process and reduce the amount of time that was going to be required in the fabrication shop, showed that if the weld travel speed was increased to 20 ipm (500 mm/min.), the weld length that could be made before the cover separated from the web or flange was longer. When I observed that, I suspected that the thermal expansion caused by the welding was traveling through the thin cover faster than the thicker web and flange and was causing the cover to lengthen, in turn causing the cover to bow away from the heavier web and flange due to the different rates of thermal expansion (Figure 4). Further experiments showed that still faster travel speeds further increased the length that could be welded before the cover bowed away.

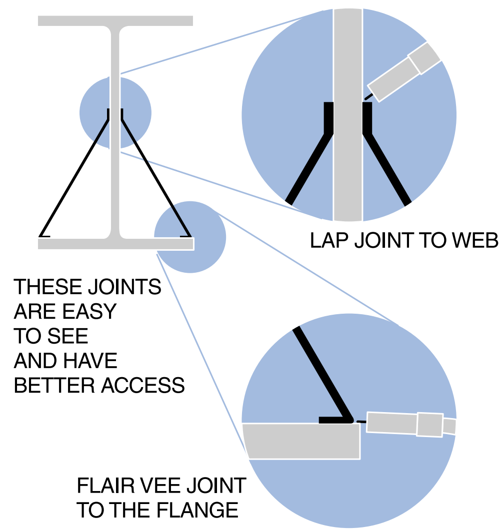


Figure 3. The revised design made it easier to fit the covers and provided better weld joint configurations.

Faster travel speeds further increased the length that could be welded before the cover bowed away

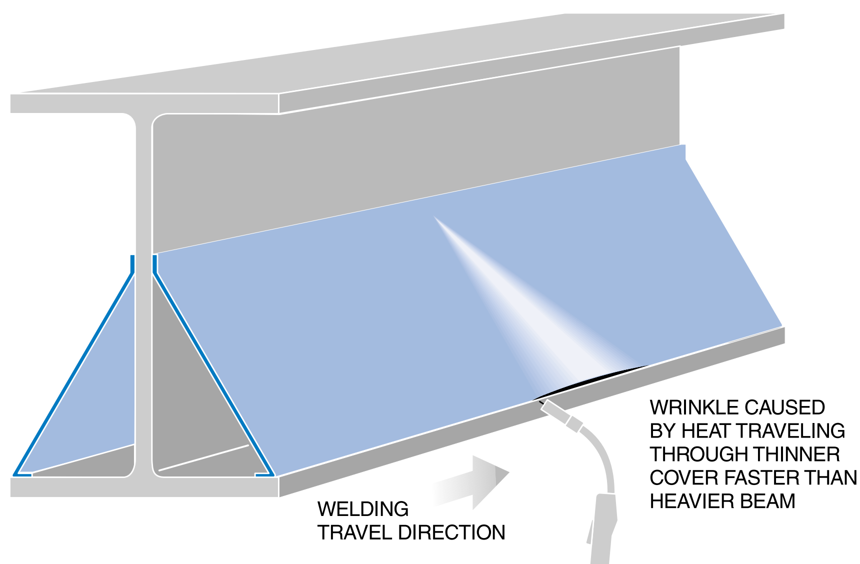


Figure 4. During welding, different rates of thermal expansion caused the thinner material of the cover to bow away from the heavier web and flange.

The Answer: Outrunning Thermal Expansion

By continuing to experiment, we learned that once the weld travel speed exceeded the rate of heat transfer through the thinner cover, the cover did not bow away from the beam at all. The travel speed, in this case, was about 45 ipm (1,100 mm/min.). This theoretically solved one problem, but at the same time created another. A hand welder simply cannot operate at a travel speed of 45 ipm. The weld travel speed that was required to eliminate the bowing problem was further complicated by the 40-ft. (12-m) length of the beams. Plus, we still had the sweep problem.

To apply this knowledge about the travel speed and to solve the other problems, we needed to mechanize the process using two welding units, each running on opposite sides of the beam in a mirrored configuration. To accomplish this, we purchased a Bug-O® portable travel carriage, 40 ft. (12 m) of track, and two sets of torch holders. We mounted two welding torches on opposite sides of the travel carriage using rack and pinion adjusters, and hung an opaque welding curtain with an inserted welding shade outside the welding torches so the welder-operators wouldn't have to use welding helmets (Figure 5).

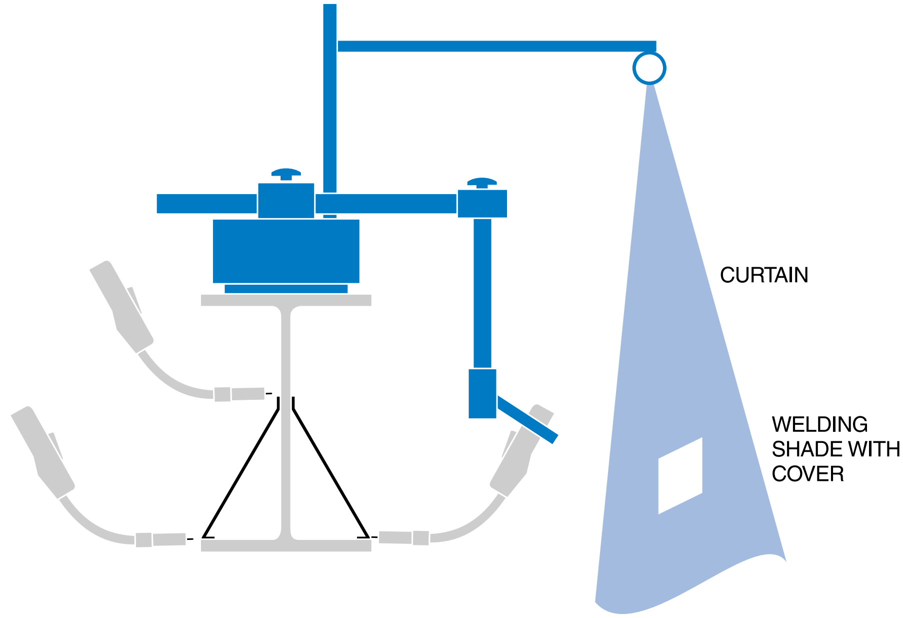


Figure 5. This set-up allowed the welding operators to weld faster than the rate of thermal expansion.

The DC-600 welding machines were connected to remote pendant controls for each weld head that were wired to separately start the weld, but collectively to stop the welding. That way if either welder had difficulty, he could stop both welders at the same time. The travel carriage was set for a 45 ipm (1,100 mm/min.) travel speed.

After production started, the supervisor noticed that it took too much time to disassemble the sections of track, move them to the next beam, reassemble them, then re-align the track with the beam. To solve this problem, we mounted the entire Bug-O assembly on a light-weight fabricated rectangular tube with edge guides to align the entire system with the top flange of the beam in a single move.

Using this set-up, we were able to safely make the welds in a timely fashion, with a good degree of operator comfort, without generating sweep in the beams.

Key Concepts in Welding Engineering

By R. Scott Funderburk, P.E.

Selecting Filler Metals: Electrodes for Stress Relieved Applications

Introduction

This is the third and final installment in a series on selecting filler metals. Many filler metals are classified in the "as-welded" condition. This simply means that no subsequent heat treating operation was performed following welding and prior to mechanical testing. Other electrodes are classified in the stress relieved condition. The choice of an appropriate electrode should be based on the actual condition of the welded part; either as-welded or stress relieved.

What Is Stress Relieving?

Thermal stress relieving is a postweld heat treating operation to reduce residual stresses. The weldment is heated to a temperature below the transformation temperature, approximately 1350°F (730°C) for ferritic steels, and held at this temperature for a predetermined amount of time, followed by uniform cooling.[1]

Stress relieving is often used to reduce distortion and to control dimensional stability and tolerances. For example, presses require precise dimensional control and are typically stress relieved after welding. Stress relieving may also be performed to prevent stress corrosion cracking or other deleterious results of residual stresses.

Which Electrodes Are Stress Relieved?

Table 1 contains the AWS filler metal specifications where deposited weld metal can be classified in the stress relieved condition.

If the filler metal classification includes one of the suffixes listed in Table 2, then that product is classified in the stress relieved condition. For example, Lincoln Outershield 81B2-H (E81T1-B2) is classified with a postweld heat treatment of 1275°F (675°C) for 1 hour. The B2 suffix alone is enough information to know that the deposit is stress relieved. Notice that in Table 2 the stress relieving time and temperature vary for each suffix, and in some cases they vary between the different filler metal specifications. For specific requirements, the filler metal specifications should be reviewed.[2]

*Table 1. AWS Specifications with Filler Metal Classified in the Stress Relieved Condition**

Specification	Application
A5.5	Low alloy SMAW
A5.23	Low alloy SAW
A5.28	Low alloy GMAW
A5.29	Low alloy FCAW

* The filler metal specifications use the term "Postweld Heat Treatment" rather than stress relieved.

Table 2. Stress Relieving Electrode Suffixes[a]

AWS Classification Suffix	PWHT Temperature, °F (°C)
A1	1150 (620)
A2	1150 (620)
A3	1150 (620)
A4	1150 (620)
B1, B1L	1275 (690)[b]
B2, B2L, B2H	1275 (690)[c]
B3, B3L, B3H	1275 (690)
B4, B4L	1275 (690)
B5	1275 (690)[d]
B6, B6L, B6H	1375 (740)
B7, B7L	1375 (740)
B8, B8L	1375 (740)
B9	1375 (740)
C1, C1L	1125 (605)
C2, C2L	1125 (605)
C5L	1075 (579)
D1	1150 (620)
D2	1150 (620)
D3	1150 (620)
Ni1	1150 (620)
Ni2	1150 (620)
Ni3	1150 (620)
Ni4	1150 (620)
Ni5	1150 (620)
F1	1150 (620)
F2	1150 (620)
F3	1150 (620)

a) The PWHT hold time is generally one hour, except the A5.29-98 Specification (FCAW) requires 2 hours for B6, B6L, B8 and B8L.
b) A5.23-97 requires 1150°F (620°C) for B1.
c) A5.28-96 requires 1150°F (620°C) for B2 and B2L.
d) A5.23 requires 1150°F (620°C) for B5.

In the classification designation for submerged arc, the third character identifies the postweld heat treatment condition: either as-welded or stress relieved. An "A" indicates as-welded

"as-welded" simply means that no subsequent heat treating operation was performed

and a "P" designates postweld heat treatment. For example, an F7A4-EG-Ni1 flux/electrode combination is classified in the as-welded condition, while an F7P4-EG-Ni1 is classified in the stress relieved condition. In some cases, the same product can be classified in both the as-welded and stress relieved conditions (e.g., Lincoln LA85/882 is classified as F7A4-EG-Ni1 and F7P4-EG-Ni1).

Potential Problems

Three situations can arise where the "wrong" electrode is used.

- An electrode classified in the stress relieved condition is used in an application that does not get stress relieved.
- An electrode classified in the as-welded condition gets stress relieved.
- The actual postweld heat treatment time and/or temperature differ from that of the classification.

If one of these scenarios occurs, it does not necessarily mean that is the result will be a "bad" weld. However, the situation should be reviewed to determine any influence on mechanical properties and quality.

Influence on strength

Stress relieving typically reduces weld strength by about 10 to 15%. For example, the tensile strength in the as-welded condition may be 80 ksi, while in the stress relieved condition it may only be 70 ksi. Therefore, if an elec-

Stress relieving typically reduces weld strength by 10 to 15%

trode classified in the as-welded condition is stress relieved, the final tensile strength could fall below the minimum classification tensile strength. This situation would create a weld that is weaker than intended.

On the other hand, if a weld is made with an electrode classified in the stress relieved condition and is not stress relieved after welding, then an overmatching strength relationship may exist. This situation is not necessarily detrimental. However, higher strength welds generally lead to higher residual stresses, lower ductility and greater crack sensitivity. In addition, the *AWS D1.1 Structural Welding Code–Steel* requires that the welding procedure be qualified by test if overmatching strength is used.

Influence on notch toughness

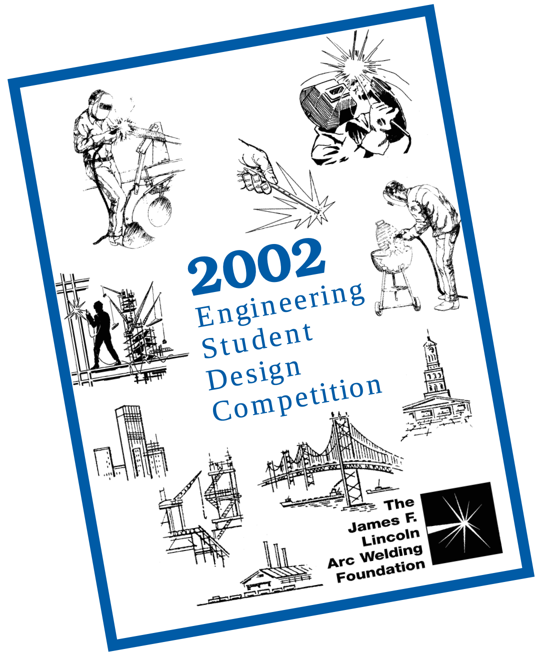
In most cases, notch toughness is increased by stress relieving. If an as-welded product is stress relieved, the notch toughness will most likely go up. However, if the product is classified as stress relieved, and the Charpy V-Notch (CVN) properties are only slightly above the minimum values, this could be a problem if the weld deposit is not stress relieved. In this case, the as-welded CVN energy values could fall below the minimum requirements. Furthermore, excessively high stress relieving temperatures can reduce the measured CVN toughness values. Therefore, during stress relieving care should be taken to control the temperature and time at temperature.

Conclusions

If stress relieving heat treatment is to be conducted, the final weld properties and quality should be evaluated and the filler metal should be one that is classified in the stress relieved condition. The influence of the heat treatment on the weld metal, heat-affected zone and base metal properties should be assessed. Finally, if the heat treatment time and temperature are different than the filler metal classification, then the possible effects of these differences should be evaluated.

1) R. Scott Funderburk. "Postweld Heat Treatment," *Welding Innovation*, Vol XV, No. 2, 1998.
2) Copies of the filler metal specifications can be ordered from AWS at http://www.aws.org

A Unique Mechanism for Enhancing Engineering Education



By Donald N. Zwiep
Professor Emeritus, Worcester Polytechnic Institute and
Chairman, James F. Lincoln Arc Welding Foundation

Editor's Note: Much of the content of this article was also included in a paper Prof. Zwiep delivered at the Fourth International Conference on Engineering Education, held in Sheffield, England, 17–20 April, 2000.

Introduction

Since 1948, The James F. Lincoln Arc Welding Foundation has provided monetary awards to college engineering students, at both the undergraduate and graduate levels, to encourage the solution of design, engineering or fabrication problems. Students compete by submitting papers that describe and illustrate their projects, which may relate to any type of building, bridge or other structure, any type of machine, products or mechanical apparatus, or to arc welding research, testing, procedure or process development. Reports or projects prepared for course work, including theses and dissertations, are eligible to be submitted as entries. Each student or team of students must list on the entry form the name of the professor or faculty advisor who oversaw the work.

Solving design, engineering or fabrication problems fully supports the outcomes Assessments associated with the program criteria of the Accreditation Board for Engineering and Technology (ABET), which accredits all engineering and technology college programs in the United States.

Scope of Program

The subject of the entries is design, and the topics covered are virtually limitless. For example, the range of winning titles in the 2000 competition included:

- Seatbelt Hypertensioner
- Disposable Cassette System for ATM Currency
- Ski Resort Parking Structure Design
- A Simple Anastomosis Device for Coronary Artery Bypass Grafting
- Voluntary Milking System
- Behavior of Square CFT Beam-Columns with High Strength Concrete Under Seismic Loading
- Design and Fabrication of a Retractable Wheelchair Foot Tray
- Improvement of Durability and Ergonomics for Packaging

In 2000, 35 projects won awards, and there were 13 departmental honorariums. A total of 114 students were involved in producing the winning projects (31 in the graduate division and 83 in the undergraduate division). Students from the University of Illinois, Stanford University, and Worcester Polytechnic Institute, colleges which give strong emphasis to project oriented education, have been consistent winners of the Foundation's awards over the past decade.

Faculty Plays Key Role

The success of the program is highly dependent on the faculty's enthusiasm, encouragement, and mentoring of students. Professors are able to help young people look beyond the classroom to envision real-world applications for their design efforts. And just as in the work world, the program provides a cash incentive to students for expending the effort required to submit entries of professional quality.

Studies of the success of students at Stanford University by Professor Emeritus Doug Wilde, and at the University of Illinois by Professor James Carnahan, strongly commend the Foundation's programs. The General Engineering Department of the University of Illinois publication "Senior Design Project, G.E. 242," describes the Foundation's program as follows:

" … the Lincoln awards are the nation's most prestigious undergraduate engineering awards and represent the excellence in engineering upon which the senior design project is based."

Judging

Members of the Jury of Awards are selected from various branches of engineering, education, business, industry, or from any other suitable source by the Chairman of the Foundation. Jury members receive an honorarium and reimbursement for their travel expenses.

Each entry is read independently by each Juror, who receives only an unidentified original copy of the entry. If the Juror, through prior knowledge, is able to recognize any of the entrants, that Juror will not participate in any judging related to that particular entry. Jury activities take place over a nominal two weeks of active review of the entries with a final meeting of all the Jurors to select the winning entries. A distinctive feature of the Jury process is the thoroughness of the review given each entry.

The Jurors use the following criteria to guide their decision-making process:

- Originality or Ingenuity
- Feasibility
- Results Achieved or Expected
- Engineering Competence
- Clarity of the Presentation

These criteria are entirely consistent with the expectations for the outcomes associated with the new accreditation methodology being implemented in the United States. ABET's EC2000 accreditation process relies on engineering departments establishing expected outcomes, measuring the accomplishment of those outcomes, and correcting the curriculum to minimize the difference between expectation and accomplishment.

The expected outcomes necessarily include the "realization" process. This means that students must succeed in specifying, designing, building, and testing a "product" appropriate to their discipline. It is, of course, obvious that such student design activities often provide the basis for entries to the Foundation's award programs. What may not be as obvious is the feedback that such entries provide for continuous improvement of curricula. In this way, the program's Jurors, as constituents, provide valuable feedback to academic programs.

The existence of patents in no way affects Jury ratings, nor does the Foundation have any financial interest in patent rights. The Jury's decision is final in all cases. Any action of the Jury may be certified by its Chairman for and on its behalf, and such certification shall be conclusive proof of all action and proceedings.

Presentation

The Foundation encourages the preparation of a professional quality paper based on a student's regular college activities in courses and projects as they are related to engineering design. The entries normally meet most or all of the ABET requirements associated with a capstone design activity. The entries also demonstrate, contrary to an oft-heard criticism, that engineering students are articulate in their written expressions. Software programs such as CAD, CAM, and FEM are commonly used. Drawings, photographs and similar illustrations are encouraged.

The rules of the program stipulate that identifying marks such as the names of students, faculty or their schools must not appear on the entries proper. Title pages and entry data are removed from each entry prior to its review by members of the Jury of Awards in order to avoid any possibility of bias.

Rewards for Participation

Faculty and students find that just the process of preparing an entry provides intangible rewards of its own. When an entry actually wins, there is a cash prize (in 2001 the prizes ranged from $250 to $2000 each), the prestige of recognition, and the confirmation of having developed and described an effective solution to a challenging problem.

The educational process that the students and faculty undertake as they work on the entries enables young people to experience the stimulation of a professionally competitive atmosphere. The process constitutes an outstanding preparation for the global challenges of the engineering profession.

Colleges note that having national award winners affirms the excellence of their academic programs, while developing and recognizing the project-advising capabilities of their faculty. Accrediting agencies, both national and regional, consider national award winning entries evidence of the quality of engineering programs. Entries to the Foundation's college program are considered excellent examples of how to meet the engineering design requirements of ABET.

Conclusion

The Engineering Student Design Competition, sponsored by The James F. Lincoln Arc Welding Foundation and now in its 52nd year of operation, is recognized as one of the most prestigious engineering award programs in the United States. The Foundation's programs are exclusively supported by The Lincoln Electric Company of Cleveland, Ohio, and are presently limited by the organization's Deed of Trust to the United States. However, the Foundation is currently exploring avenues for expanding the global scope of many of its activities, including, the award programs.

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2002

COLLEGE ENGINEERING AND TECHNOLOGY ENTRY FORM

ENTRY MUST BE POSTMARKED ON OR BEFORE MIDNIGHT, JUNE 15, 2002

Secretary, The James F. Lincoln Arc Welding Foundation, P.O. Box 17188, Cleveland, Ohio 44117-0035

A PHOTOGRAPH MUST BE SUBMITTED OF EACH AUTHOR, WITH THE NAME WRITTEN ON THE BACK, WITH THE ENTRY.

Fill in information requested and attach to title page of paper being entered in the Program. **ENTRY MUST BE TYPED.**

DO NOT PUT YOUR NAME OR NAME OF SCHOOL ANYWHERE IN YOUR ENTRY OTHER THAN ON THE ENTRY FORM.

Subject or title of paper(put this title on Page 1 of paper also). ________________

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To be entered in the following division (check only one):

❑ Undergraduate Division ❑ Graduate Division

Department ________________________________

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Professor or Faculty Advisor ________________________

School Name ________________________________

School Address ______________________________

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This paper completed by the undersigned author(s) within the period of June 16, 2001 to June 14, 2002.

Entrant(s) signature(s) ____________________________

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NOTE: In case of joint authorship of an award winning Entry, each author will receive a check dividing the total award amount equally unless otherwise indicated. Please check one of the boxes below.

❑ Divide Equally ❑ Divide as follows:

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STATEMENT OF CERTIFICATION

(Must be signed by faculty member)

Following statement must be signed by a professor, instructor or the faculty advisor.

The author(s) of this paper ___________________________

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(is/are) (was/were) registered at (school) ___________________

and to the best of my knowledge this paper was originated and prepared by the student(s) named on this Entry Form.

Signed ____________________________________

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CHECK WILL BE SENT TO HOME ADDRESS. NOTIFICATION 10/2002.
Home address must be address at which you will receive mail October 2002.

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Entries may not be submitted by more than five students.

Application and Rules can be found at www.lincolnelectric.com

The James F. Lincoln
Arc Welding Foundation

Design File

Mixing Welds and Bolts, Part 1

Practical Ideas for the Design Professional by Duane K. Miller, Sc.D., P.E.

Introduction

There are a variety of circumstances in which the engineer may need to assess the strength of a connection that is composed of both welds and mechanical fasteners. Today, mechanical fasteners are typically bolts, but older structures may include rivets. Such situations may be encountered during the course of rehabilitation, repair or strengthening projects. For new construction, bolts and welds may be combined in connections where the materials being joined are initially secured with bolts, and then welded to gain the full connection strength. As will be seen, calculating the total capacity of the connection is not as simple as totaling the arithmetical sum of the individual components (welds, bolts, and rivets). Such an assumption is unconservative, and the consequences could be disastrous.

Part 1 of this two-part edition of "Design File" will deal with snug-tightened and pretensioned mechanical fasteners combined with welds. Part 2 will address combining welds with slip-critical, high-strength bolted connections.

Some Background on Bolted Connections

Bolted joints are described in the *AISC Specification for Structural Joints Using ASTM A325 or A490 Bolts* (June 23, 2000) as either snug-tightened, pretensioned, or slip-critical (p. 23). A snug-tightened joint has the condition of "tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into *firm contact*."(p. xi). A pretensioned joint is one in which the bolts have been installed in a manner so that the bolts are under significant tensile load with the plates under compressive load (p. x). Four acceptable methods are listed in Section 8.2: turn-of-nut, calibrated wrench, twist-off-type tension-control bolts, and direct-tension-indicators. Slip-critical joints have bolts installed just as they would be in a pretensioned joint, but also have "faying surfaces that have been prepared to provide a calculable resistance against slip." (p. xi).

In simple terms, in snug-tightened joints and pretensioned joints, the bolts act as pins. Slip-critical joints work by friction: the pretension forces create clamping forces and the friction between the faying surfaces work together to resist slippage of the joint.

ASTM A325 bolts have a minimum tensile strength of 105–120 ksi (725–830 MPa) depending upon the bolt diameter, while A490 bolts must fall between 150 and 170 ksi (1035–1175 MPa) tensile strength. Riveted joints behave more like snug-tightened joints, but the "pins" in this case are the rivets, and are typically about half the strength of A325 bolts.

When a mechanically fastened joint is loaded in shear, one of two types of behavior is possible. The joint may have the bolts or rivets bear against the sides of the holes in the connected material, concurrently putting the bolt or rivet into shear. The second possible behavior is that friction, introduced by the clamping forces provided by the pretensioned fastener, resists the shear loading. No slippage is expected in this joint, but the possibility exists nonetheless.

Snug-tight joints are acceptable for many applications since minor slippage may not adversely affect the performance of the connection. When there is significant load reversal, pretensioned joints may be required. When joints are subject to fatigue loads with reversal of direction, slip-critical joints are required.

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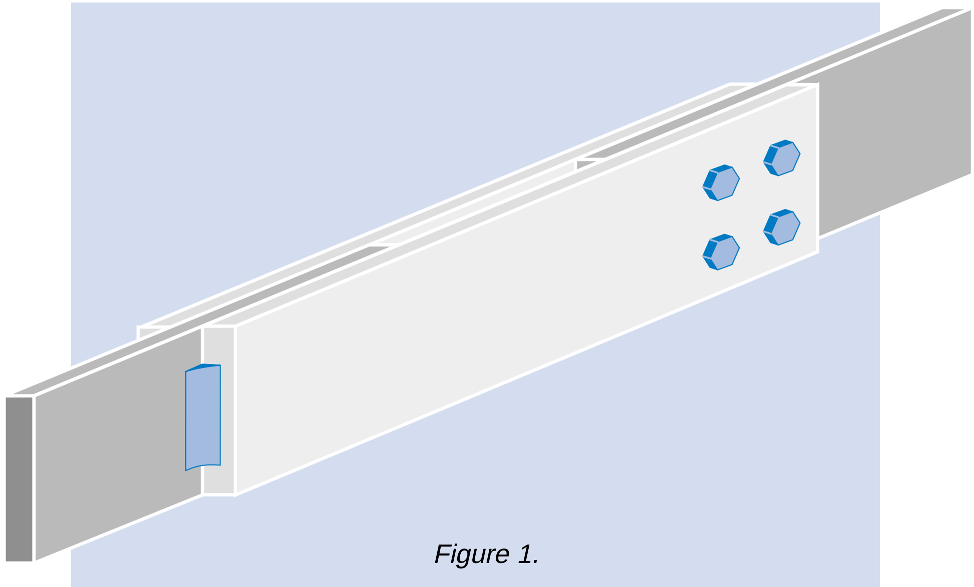

Figure 1.

Thus, an existing bolted connection may have been designed and built to any of these criteria. Riveted joints would be considered the snug-tight type.

Adding Welds to Mechanically Fastened Joints

While a weld may be composed of metal that is capable of demonstrating an elongation of 20% or more in an all-weld-metal tensile specimen, the same metal in a restrained joint may be incapable of delivering any significant deformation prior to fracture, due to the interaction of triaxial stresses. In other words, welded connections are rigid. Welded connections are stiff. Unlike snug-tightened bolted joints that may slip as they are loaded, welds are not expected to stretch and distribute the applied load to any great extent. In most cases, welds and bearing-type mechanical fasteners will not deform equally. The load is transferred through the stiffer part, and therefore the weld will carry virtually all the load, sharing little with the bolts. And that's why caution needs to be taken when welds and bolts and rivets are combined.

Code Provisions

The issue of mixing mechanical fasteners and welds is addressed in the *AWS D1.1:2000 Structural Welding Code—Steel*. Provision 2.6.3 states:

> "Welds with Rivets or Bolts. Rivets or bolts used in bearing type connections shall not be considered as sharing the load in combination with welds. Welds, if used, shall be provided to carry the entire load in the connection. However, connections that are welded to one member and riveted or bolted to the other member are permitted. High-strength bolts properly installed as a slip-critical-type connection prior to welding may be considered as sharing the stress with the welds."

The first three sentences of this provision address the topic discussed here. The fourth sentence will be addressed in part 2.

When the mechanical fasteners are of the bearing type and a weld is added, the capacity of the bolt is essentially ignored. The weld must be designed to transfer all the load, according to this provision. This is, in essence, the same as the requirement of AISC LRFD-1999, provision J1.9. However, the Canadian standard CAN/CSA-S16.1-M94 also permits the use of the capacity of the mechanical fastener or the bolts alone when this is higher than the capacity of the welds. All three standards are in agreement on this issue: the capacities of the bearing-type mechanical fasteners and the welds cannot be added together.

AWS D1.1, paragraph 2.6.3, goes on to discuss an acceptable situation in the third sentence. Bolts and welds can be combined in the situation where a connection consists of two separate components, as illustrated in Figure 1. On the left is a welded connection, and on the right, a bolted one. This is acceptable. Each part of the overall connection behaves independently, and thus, the Code provides an exception to the principles as contained in the first part of 2.6.3.

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The previous provisions are applicable for new construction. For existing structures, D1.1 paragraph 8.3.7, goes on to say:

> "Use of Existing Fasteners. When design calculations show rivets or bolts will be overstressed by the new total load, only existing dead load shall be assigned to them. If rivets or bolts are overstressed by dead load alone or are subject to cyclic loading, then sufficient base metal and welding shall be added to support the total load."

The first sentence permits sharing of loads between mechanical fasteners and welds if the structure is preloaded (i.e., any slip has already occurred), but only the dead load can be assigned to the mechanical fastener. Welds must be used to take up all the applied or live load. No such sharing of loads is permitted when the mechanical fasteners are already overloaded. When cyclic loading is involved, no load sharing is permitted.

An Illustration

Consider a lap joint originally connected with snug-tight bolts, as shown in Figure 2a. Additional capacity is being added to the structure, and the connection and the attached members must be increased to provide twice as much strength. Figure 2b illustrates the basic plan to strengthen the members. What should be done to the connection?

Since the new steel is going to be joined to the old with fillet welds, the engineer decides to add some fillet welds to the connection. Since the bolts are still in place, the initial thought is to add only the welds required to transfer the additional capacity of the new steel, expecting 50% of the load to go through the bolts, and 50% through the new welds. Will this be acceptable?

Let's first assume there is no dead load currently applied to the connection. In this case, D1.1 paragraph 2.6.3 applies. In this bearing type connection, the welds and bolts cannot be "considered as sharing the load." Thus, the specified weld size must be large enough to carry the entire dead and live load. The capacity of the bolts cannot be considered in this example.

Next, let's assume a dead load is applied. Further, let's assume that the existing connection is adequate to transfer the existing dead load. D1.1 paragraph 8.3.7 applies in this case and the new welds are only required to carry the increased dead load and the total live load. The existing dead load can be assigned to the existing mechanical fasteners.

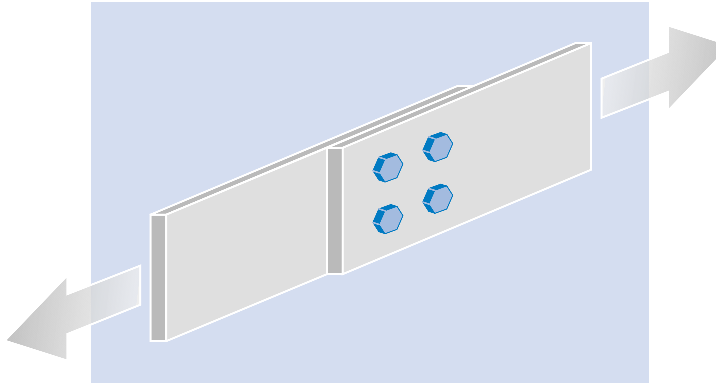

Figure 2a.

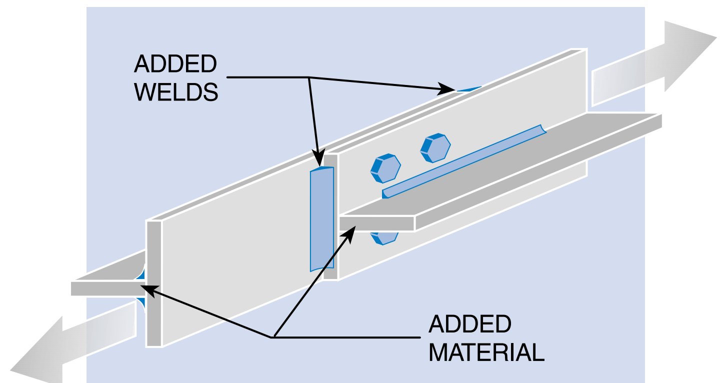

Figure 2b.

Conclusion

In summary, the answer to the question "Is this acceptable?" depends on the loading conditions. In the first case where no dead load was assumed, the answer is "no." Under the specific conditions of the second scenario, the answer is "yes." It cannot be concluded that the answer will always be "yes" simply because dead load is applied. The level of dead load, the adequacy of the existing mechanical connection, and the nature of final loading (whether static or cyclic) could change the answer.

All of the above apply to mechanical fasteners of the pin type. Part 2 will deal with high strength, slip-critical bolted connections in combination with welds. The technical aspects of the content of part 2 are currently being evaluated by technical committees. The work of the committees may not be complete in time for the next issue of Welding Innovation. Part 2 will be forthcoming just as soon as all the technical information is available.

Tri-Chord Roof Trusses Enhance Safeco Field

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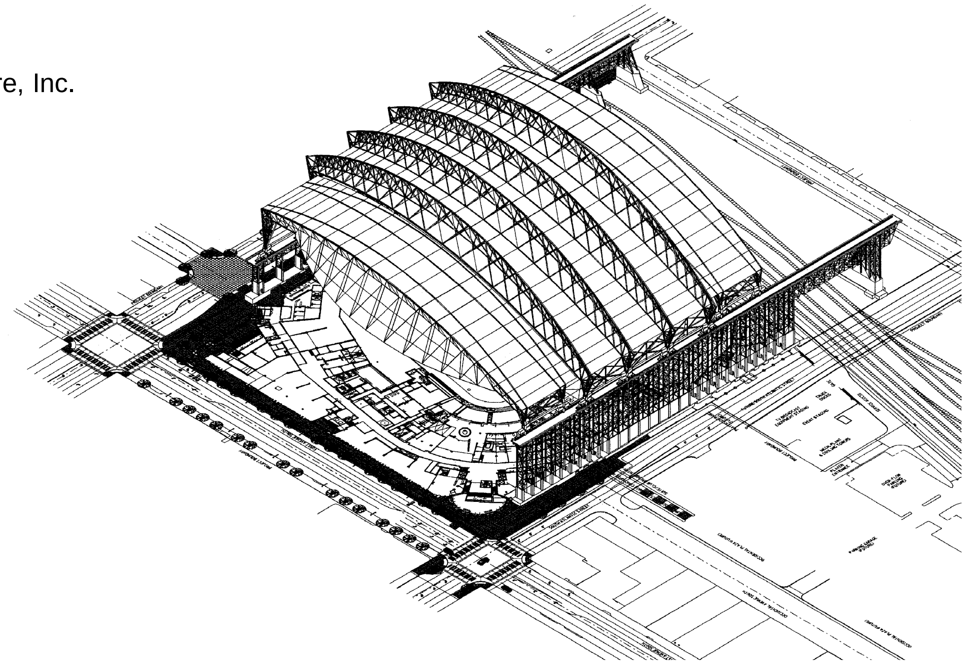

Figure 1. Aerial view of computer model with the roof in the field position.

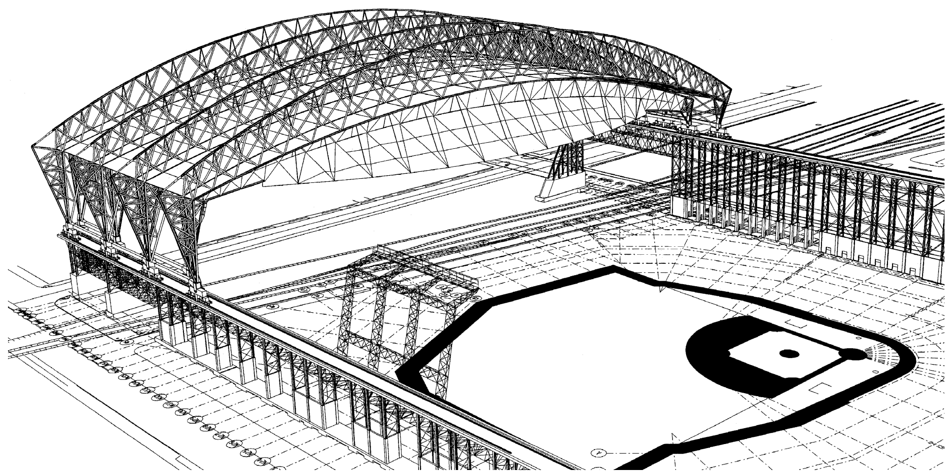

Figure 2. Computer model of the Safeco Field roof in the retracted position.

Introduction

Safeco Field, the new home of the Seattle Mariners baseball team, features a unique retractable roof. Modern arc welding enabled the design and construction of the stadium's spectacular tri-chord roof trusses, which contribute both structurally and aesthetically to the stadium. Arc welding was also used for other critical elements throughout the stadium, and turned out to be the only suitable method to meet the demanding construction schedule.

Project Description

The retractable roof spans 655 ft. (200 m) over the stadium, providing the ballpark with a giant umbrella (Figure 1) to prevent the potential for rained-out games in Seattle's wet climate. In its retracted position (Figure 2), the roof is stored completely off the stadium, over the Burlington National Railroad right-of-way adjacent to the ballpark.

The roof rides atop two 800-ft.-long (244 m) runway structures, standing 100 ft. (30 m) tall on the south side and 50 ft. (15 m) tall on the north side of the stadium. The shorter runway on the north side allows spectacular views of Seattle and Puget Sound from the seating bowl.

The three independent roof panels cover 8.7 acres (3.5 hectares) and weigh approximately 12,000 tons (10,900 m tons). The largest, center panel is 275 ft. (84 m) above the playing field. The two lower panels that slide underneath the center panel in the retracted position have down-turned trusses with a bottom chord 165 ft. (50 m) above the field.

The use of the Burlington Northern Railroad's right-of-way and the linear tracking system for the roof allowed the roof trusses to be assembled on a stationary, temporary platform, located outside the main stadium footprint. The large, stable tri-chord roof trusses were assembled one at a time on top of the temporary staging platform, and rolled off to the side, making way for the erection of the next truss. This allowed the erection of the roof structure to be done completely outside of the stadium bowl, taking it off the critical path for the rest of the project.

One of the key design objectives was to support the roof on moveable "legs." This makes the roof unlike any other retractable roof that has ever been built. When the roof retracts, the supporting walls move with it to truly make an open air ballpark—not just a building with a sunroof.

The architect insisted that these legs be as slender and elegant as possible, while also, of course, being resistant to all wind and earthquake forces. The only way to achieve this was to use welded steel plate in the legs.

Maximizing Use of Shop Welding

The roof structures had to be designed so that they could be quickly erected in order to meet a very aggressive construction schedule for the ballpark. As designers, we were also concerned about the use of field welding on a structure that would be erected primarily during the wet, windy part of the year. Therefore, we decided to make maximum use of shop welding to minimize the amount of welding that would have to be done outdoors in the elements. This allowed us to take advantage of cost effective shop welding processes, as well as the higher production rates and better quality control attainable in a shop environment. It also eliminated the time, expense and safety issues attendant to having welders work in relatively inaccessible areas, high above the Burlington Northern Railway's right-of-way.

The use of modern welding and testing technology allowed us to design extremely complex roof truss and leg connections, which were prefabricated in a controlled shop environment. This reduced the amount of time and work required to assemble the trusses in the field.

Curved Geometry of the Trusses

One of the key elements required to attain the overall curved geometry of the trusses was the ability to articulate the truss chords at each work point where the four diagonals attach to the chord. The most economical and practical way to accomplish this was by using full-penetration welds of the jumbo sections, ranging in size from W14x145 to W14x730. Extra precautions were taken to ensure the integrity of the joints in these large sections (Figure 3). Welding electrodes were required to have a minimum specified Charpy V-Notch (CVN) rating of 20 foot-pounds (27.2 joules) at minus 20°F (-29°C). CVN tests were performed on the structural members to ensure that the base metal also had high toughness properties. All base materials around the full-penetration splices had ultrasonic and magnetic particle tests before welding of the joints to ensure that there were no flaws in the base metal that might propagate into the welds.

When the roof retracts, the supporting walls move with it

Based on an AISC advisory on the potential for problems with rotary-straightened members yielding low CVN numbers in the "k" zone, all members requiring full-penetration butt splices were specified as gag straightened. All of the truss chord members were specified as A913 Grade 65 material. Despite the manufacturer's claims of no required preheating for this material, a minimum preheat of Category B was specified for all critical welds.

Welding Procedures Minimize Distortion

It was critical that the fabricators maintain the correct geometry, due to the nature of the very large and complex weldments that are attached to the truss chords. The fabricators developed detailed welding procedures to minimize the distortion of the plates and chord elements during the welding process. As a result of their diligent effort, the fit-up of the trusses in the field was outstanding (Figure 4), with a minimum number of field modifications required.

Welding Prevents Corrosion

The entire roof structure is exposed to the wet environment of the Pacific Northwest. The welding details were developed with special consideration given to minimizing the potential corrosion of the structural steel. Joints were designed so that all surfaces could be prepainted before assembly. Field welding was minimized to prevent potential corrosion of heat-affected areas where the paint would be burned off in the welding process. Where field welding was required, special details were provided to seal-weld inaccessible areas that could not be painted after welding. Shop seal-welds (Figure 5) were used in many applications, such as large, round lid enclosures which were welded to the diagonal braces as they pass through the roof membrane. These lids act as a top to a round PVC enclosure that is part of the roof membrane (Figure 6).

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Summary

On this project, welding was used instead of bolting wherever possible because of its superior construction fit-up and corrosion resistance. Only arc welding provided the designers the latitude to create such a complex and elegant structure, the first of its kind in the world.

Figure 3. Bottom chord assembly at the splice location with welded diagonal connections.

Figure 4. Erection crew assembling truss.

Figure 5. Shop photo showing seal welds being placed around the lid assemblies used for waterproofing.

Figure 6. Lower chord of panel 3 with diagonals and waterproof lid assemblies.

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Seattle's Safeco Field features a unique retractable roof. See story on page 19.

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