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Welding Innovation

Advancing Arc Welding Design and Practice Worldwide

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Mentoring Part V: "More than a Saying"

In several previous issues of *Welding Innovation*, starting in 1996, a four-part series on mentoring in the engineering profession was published (and is now easily accessed on our new web site at www.WeldingInnovation.com.) As a continuation on that theme, I would like to share with you my experiences in a mentoring relationship.

It has been a real privilege to have not only one, but two excellent mentors to help me jump-start my professional career. Both Omer Blodgett and Duane Miller have outstanding reputations in the structural welding industry. What has made the greatest impression on me on a daily basis are the little sayings and illustrations that they both use to explain concepts and express ideas. At first glance, these comments may seem to be only catchy clichés. However, these quotes are more. They are words to live by – golden nuggets of truth to help guide us like road signs along life's highway. These "power phrases" are at the heart of our mentoring relationship. The very word mentor means a trusted counselor or guide – a coach!

One example of Duane's sayings is "what are the facts? I can deal with the facts; just give me the facts." This may seem like an obvious request from a supervisor, and it is. But the principle behind the statement is much larger than the question. When faced with a new challenge or a difficult situation, what do you do? How should you respond? As we learned in our first undergrad engineering class, before you can solve any problem you need to clearly define it. You need "the facts!" Decisions are easier and situations are more manageable when we get to the facts. I am learning that "disasters" can be handled if you understand the facts. This bit of wisdom has helped me to be able to focus and solve problems more quickly.

Omer also has plenty of sage advice, and one of my favorite examples is his philosophy of "don't design with your heart!" This is another variation on the "get to the facts" principle. In our design seminars, Omer tells a story of a young engineer (and no, it was not me!) who was challenged to reduce the deflection of an accelerating steel component on a piece of equipment. The engineer decided to use aluminum rather than steel to reduce the weight. However, in this example the change to aluminum did not solve the problem because he never addressed the real issue – deflection. Yes, the density was decreased to one-third that of steel, but the modulus also changed to one-third that of steel, and nothing was accomplished to reduce deflection. Omer calls this "designing with your heart." The idea is simple yet profound: take time to rationally think through a problem and minimize your assumptions. In other words, get to the facts and use mathematics and engineering to make the decisions for you. Don't design with your heart!

Other examples that I have picked up are…
- "Don't solve one problem by creating another."
- "One plus one doesn't necessarily have to equal two, but it better be close."
- "Under-promise and over-deliver."

These "sayings" are so important to me because they all contain a lesson, a principle that can be applied in many areas of life, both technical and non-technical.

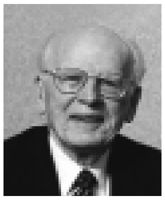
Both of my mentors have shared with me the benefits of sales experience to building a career. I am now about to follow their advice and their example. After the first of the year, I will be transferring to the Houston office of Lincoln Electric and will not be directly involved with the James F. Lincoln Arc Welding Foundation or *Welding Innovation* for the next couple of years.

My parting message to our readers is simple: please consider sharing the wealth of your experience by creating one or more meaningful mentoring relationships. And if you are in a mentoring position already, whether it is formally defined or unspoken, I encourage you to consider teaching through your own power phrases. Believe me, the impact can be long-lasting and dynamic!

Scott Funderburk, P.E.
Assistant Editor, Welding Innovation

Welding Innovation

Cover: This chemical tanker, named for the opera composer Verdi, was one of six built by the Shipyard K. Damen of Rotterdam, the Netherlands, featuring cargo tanks fabricated of duplex stainless steel. See story on page 2.



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Tankers—A Composition in Duplex Stainless

By Fred Neessen
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Introduction

Duplex stainless steel is finding an increasing frequency of application in the shipbuilding sector, mainly due to its high yield strength and corrosion resistance properties. The design and fabrication of a recent chemical tanker project illustrates the trend.

The higher yield strength and superior corrosion resistance of duplex stainless governed the choice of material

The ship owner Gesellschaft fur Oltransport (GEFO) of Hamburg, Germany, contracted the Shipyard K. Damen of Rotterdam, the Netherlands, to build six ships designed for both inland and seagoing navigation, featuring cargo tanks fabricated of duplex stainless steel. The resulting double-hull tankers, designed by GEFO to transport up to 2,750 tonnes (2700 tons) or 3,250 m^3 (4,250 yd.^{3}) of liquid in twelve separate tanks, are 95 m (312 ft.) long and 6.35 m (21 ft.) high with a 12.5 m (41 ft.) beam. The separate cargo tanks allow fully independent loading and emptying, permitting the simultaneous transportation of different chemicals. On an interesting note, each ship was named for a famous musical composer: Rossini, Puccini, Verdi, Bellini, Mozart, and Donizetti.

Choice of Material

The cargo tanks were fabricated of duplex stainless steel (WNr 1.4462), which has a higher alloy content than the austenitic AISI 316LN grade often used in the construction of similar inland navigation tankers. The higher yield strength and superior corrosion resistance of duplex stainless governed the choice of the material. These two properties increased the number of different chemical products that can be loaded and transported by the tankers. While the ultimate tensile strength of WNr 1.4462 is approximately 20 percent higher than that of 316L, its yield strength is 120 percent higher. Since European shipbuilding codes are based on yield strength, not tensile strength, WNr 1.4462 was particularly attractive in this application. Furthermore, the lower nickel content of WNr 1.4462 made it a more economical choice for this application than either 316LN or 317LN.

Another factor taken into consideration was the resistance of the base materi-

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Table 1. Mechanical properties of base materials according to ASTM A 240.

UNS	AISI	Yield (MPa)	Tensile (MPa)	A4 (%)
S 31653	316LN	≥ 205	≥515	≥40
S 31753	317LN	≥ 240	≥550	≥40
S 31803	DSS (1.4462)	≥ 450	≥620	≥ 25

al to pitting corrosion, as expressed by the "Pitting Resistance Equivalent" or PRE. The PRE may be expressed with or without the influence of nitrogen (N), as shown in the following formula:

$$PRE(N) = \%Cr + 3.3 * \%Mo\ (+16 * \%N)$$

This formula clearly shows that molybdenum (Mo) makes an important contribution to pitting resistance. The higher the PRE number, the higher the resistance to pitting and crevice corrosion.

Specific comparisons of the mechanical properties and chemical compositions of the three grades of steel are shown in Tables 1 and 2. To sum up, the duplex stainless was chosen for reasons of economy, high strength, and excellent resistance to both chloride corrosion cracking and pitting corrosion. The material's high yield strength translated to reduced plate thickness and reduced weight, which really means increased cargo carrying capacity.

Process and Consumable Selection

For both CrNi and CrNiMo stainless steels, any conventional welding process can produce welds of optimum quality, provided that the correct welding parameters are maintained, and that the correct consumables are used. For this chemical tanker project, Shipyard K. Damen considered the total cost of various processes, including the costs of any necessary pre- and post-weld treatment, before deciding to use a combination of GMAW, FCAW and SAW. Welding positions, base material combinations, and the selection of welding processes and consumables were all decided in accordance with Germanischer Lloyd rules. Lincoln Smitweld provided technical support and assistance with development of the welding procedures, process and consumables selection, welder qualification and test-

Table 2. Chemical composition of base materials according to ASTM A 240.

Base material		Chemical composition					Pitting Resistance Equivalent %Cr + 3.3 * %Mo (+ 16 * %N)		
UNS	AISI	C max.	Cr	Ni	Mo	N	min.	max.	average
S 31653	316LN	0.030	16 - 18	10 - 14	2 - 3	0.10 - 0.16	24.2	30.5	27.3
S 31753	317LN	0.030	18 - 20	11 - 15	3 - 4	0.10 - 0.22	29.5	36.7	33.1
S 31803	(1.4462)	0.030	21 - 23	4.5 - 6.5	2.5 - 3.5	0.08 - 0.20	30.5	37.8	34.2

Table 3. Duplex stainless welding consumables.

Product	AWS classification	EN classification	C	Mn	Si	Cr	Ni	Mo	N	FN
Arosta 4462	A5.4: E 2209-16*	EN 1600: E 22 9 3 N L R 3 2	0.02	0.8	1.0	22.5	9.5	3.2	0.16	30–55
Arosta 4462-145	A5.4: E 2209-16*	EN 1600: E 22 9 3 N L R 5 3	0.025	0.7	1.0	22.5	9.5	3.0	0.16	30–55
LNM 4462	A5.9: ER 2209	EN 12072: G 22 9 3 N L	0.018	1.5	0.5	22.7	8.5	3.0	0.15	
Cor-A-Rosta 4462	A5.22: E 2209T0-4	EN 12073: T 22 9 3 N L R M 3	0.03	0.9	0.6	22.9	9.3	3.4	0.14	40
Cor-A-Rosta P 4462	A5.22: E 2209T1-4	EN 12073: T 22 9 3 N L P M 2	0.03	0.7	0.6	22.9	9.2	3.4	0.14	40
LNS 4462	A5.9: ER 2209	EN 12072: S 22 9 3 N L	0.03	0.9	0.7	22	8	3.0	0.15	30–50
P 2000	-	EN 760: S A AF 2 6 3 DC								
Cor-A-Rosta 309L	A5.22: E 309LT0-1/4	EN 12073: T 23 12 L R C/M 3	0.03	1.4	0.6	24	12.6	-		15
Cor-A-Rosta P 309L	A5.22: E 309LT1-1/4	EN 12073: T 23 12 L P C/M 2	0.03	1.2	0.6	23.3	12.6	-		15

Table 4. Overview of welding methods.

Reference No. (fig. 2)	Material	Welding position	Welding process	Welded joint	Testing
1	Duplex / Duplex	PB (2F)	FCAW (P 4462)	Double fillet weld throat = 4 mm	Dye check HV10 Fracture
2		2PD (4F)			
3		PA (1G)	SAW	I-joint (square)	As 5-6 and 7
4		PB (2F - 2G)	FCAW (P 4462)	1/2 V-50° incl. fillet weld	Dye check HV10
5		PA (1G)		V-60° with ceramic backing	X - Ray Corrosion Ferrite Mechanical
6		PF (3G up)			
7	Duplex / Grade A	PA (1G)	FCAW (P 309L)	Double fillet weld throat = 4 mm	Dye check HV10 Fracture
8		PB (2F) manual			
9		PB (2F) machined			
10		PD (4F)			
11		PF (3F up)			
12	Duplex / Duplex	PG + PF (3Gd + 3Gu)	GMAW + FCAW	V-60°	As 5-6 and 7

ing, and welder training. The welders needed training and qualification (t&q) on duplex stainless steel as well as on welding of dissimilar materials joints.

Pulsed gas metal arc welding was used to create a root run in a V-60° joint in the vertical down position on a ceramic backing strip. The shielding gas employed was a three part $ArHeCO_2$ blend.

Stainless flux cored electrode accounted for most of the welding of the tankers. The Lincoln Smitweld Cor-A-Rosta® range of products was used, as follows:

- Cor-A-Rosta 4462 was employed for downhand welding of grooves and horizontal-vertical fillets. Shielding gas selections include 100% CO_2, as well as 80% Argon + 20% CO_2.

The material's high yield strength translated to reduced plate thickness and reduced weight

- Cor-A-Rosta P 4462 was employed for out-of-position welding. The shielding gas is restricted to 80% Argon + 20% CO_2.

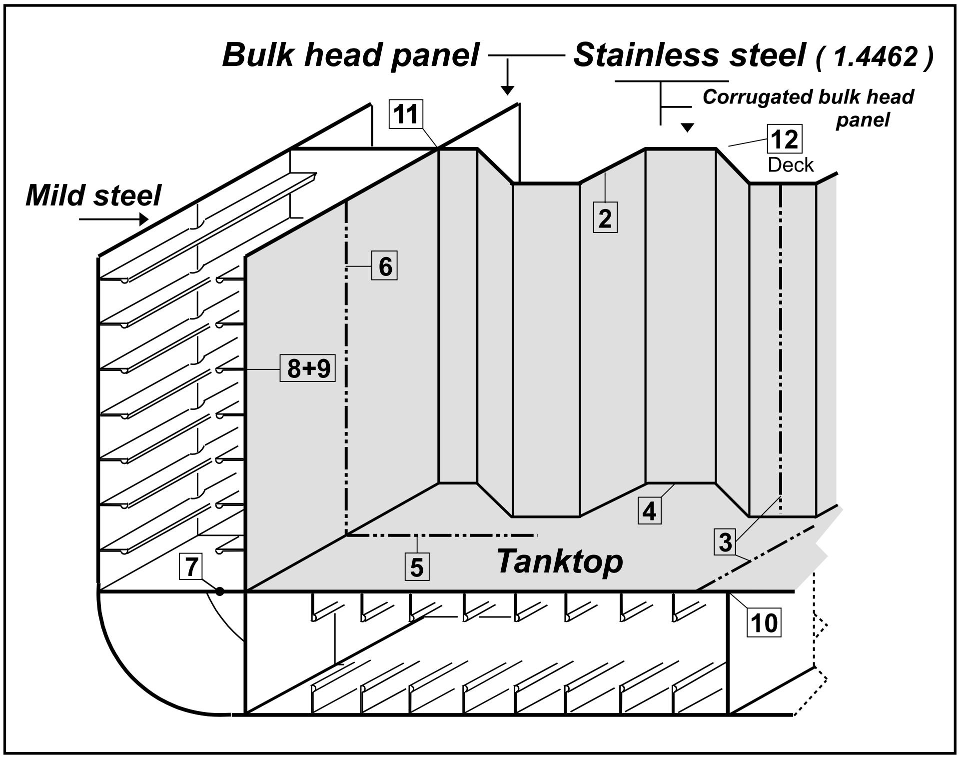

Figure 1. Schematic cross section of a chemical tanker.

The use of stainless steel flux cored electrode offered the following advantages over solid electrode:

- Weldable using conventional MIG/MAG power sources
- Wide current setting
- 30% higher deposition rate
- Smooth bead surface
- Fewer undercuts and less oxidation of adjacent areas
- Less spatter; less post-weld cleaning
- Better wetting properties
- Out-of-position welding capability
- Less expensive shielding gas ($Ar + CO_2$ or 100% CO_2)
- High operator appeal

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Figure 2. Actual view of the layout of the tanks.

Submerged arc welding, although offering very high productivity, is usually limited to welding in the flat position. Because of this, its use on this project was limited to the butt weld joining of sheets. Cor-A-Rosta 4462 wire and a neutral flux were selected for the SAW process.

Manual metal arc welding was employed in those areas of the fabrication that could not be welded with mechanized processes. The covered electrodes selected were Arosta 4462 and Arosta 4462-145 (145% efficiency). Tack welds were made using Arosta 4462 (without high efficiency).

For further details of welding methods, consult Table 4, with its references keyed to Figure 1.

Testing

Fillet welds were given Vickers hardness and fracture tests as prescribed by Germanischer Lloyd (GL) rules.

Butt welds were subjected to mechanical tests per GL rules, as follows:
- Vickers hardness
- Ferrite content measured with Magne Gage
- Reduced-section tensile test
- Root and face bend tests
- Impact test: center line weld, fusion line and fusion line + 2 mm (0.08 in.) Charpy samples

Butt welds were also corrosion-tested in accordance with GL rules, which for chemical tankers require:

- Intergranular corrosion attack according to DIN 50914. There were no defects.

- ASTM G48 method A during 24 hours @20 – 22 - 23° C (68 – 72-73° F). No pitting was observed.

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Design File

Designing Welded Lap Joints

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

Welded connections involve two components that are both under the direct control of the designer: the joint type, and the weld type. Failures in or near the weld may be the result of an improperly designed joint. In this Design File, the principles that should be applied when designing lap joints are presented.

Superficially, a lap joint looks very simple, and it may seem odd that this plain configuration of material would need to be carefully considered. The complication stems from the fact that loads do not instantaneously transfer from one member to another. The three joints in Figure 1—one butt joint, and two lap joints—show the differences in the flow of stress through the two joints. The butt joint includes a groove weld while the lap joints use fillet welds. The difference is, stress flow is more associated with the joint type, as opposed to the weld type. The resultant differences in stress distribution result in the need for rules to proportion the lap connection components.

Forces applied to the ends of lap joints result in eccentric loads in the connection area. This can cause joint rotation, as illustrated in Figure 2. This same eccentricity can cause the root of a fillet weld to tear when only one transverse fillet is applied to a lap joint that is permitted to deflect laterally, as can be seen in Figure 3.

In summary, the simple lap joint inherently offers two broad challenges to the designer:

1. How to deal with the non-uniform stress distribution, and
2. How to deal with the eccentricity.

While many welded applications are not contractually governed by *AWS D1.1 Structural Welding Code–Steel*, the designer of any product can find helpful provisions in that code that address these conditions.

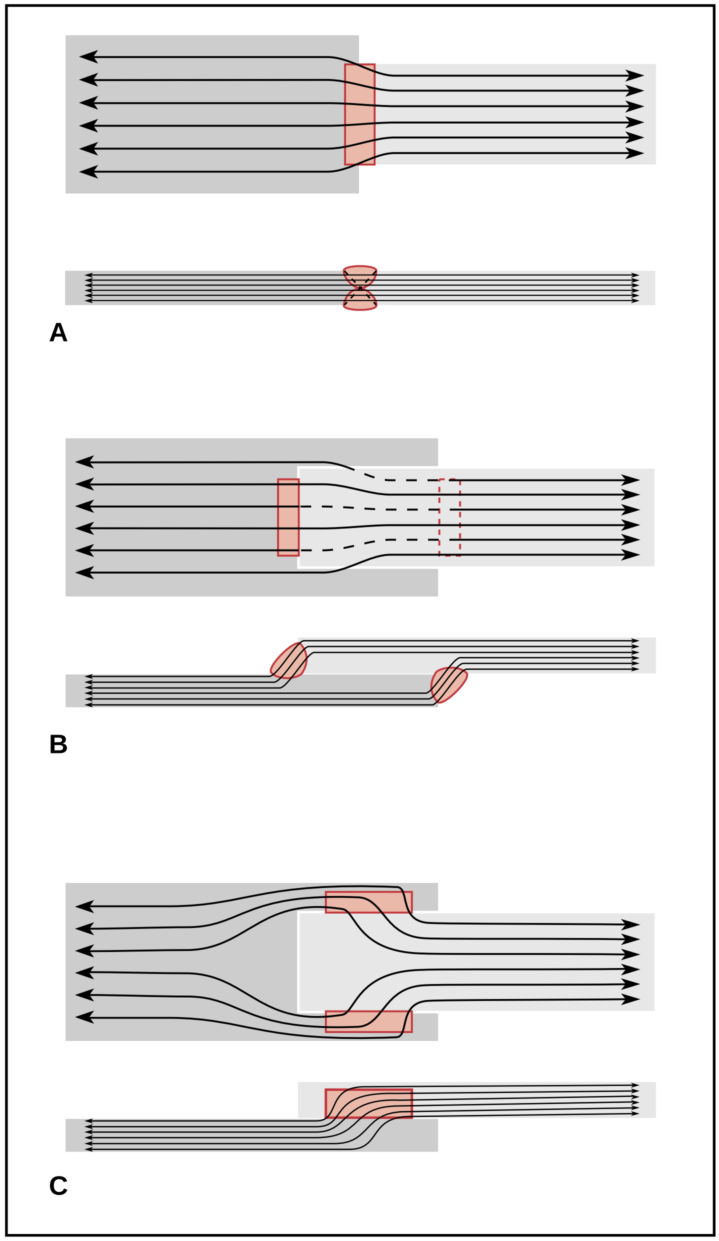

Figure 1

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Non-Uniform Stress Distribution

D1.1 paragraph 2.14.1 requires that, when longitudinal fillet welds are used alone (such as in figure 1c), the length of the fillet weld shall be no less than the perpendicular distance between them. This is illustrated in Figure 4. Even though the weld length "L" may be acceptable for the transfer of force "F," the complicated stress flow pattern of Figure 4b will generate unacceptable stress concentrations.

The Code goes further in paragraph 2.14.1 and requires that the distance between the welds (shown as "D" in Figure 4) be no greater than 8 in. (200 mm) if only longitudinal welds are used (as shown in Figure 1c). For distances greater than 8 in. (200 mm), transverse welds or intermediate plug or slot welds are permitted to overcome this restriction. While the code does not specifically identify the option, bolts could also be used to accomplish this function.

Paragraph 2.32.1 in Part C for Cyclically Loaded (i.e., susceptible to fatigue failure) Connections additionally requires that this distance not exceed 16 times the thickness of the thinner member, and gives the following reason for the need for the intermediate plug or slot welds: to prevent buckling or separation of the parts. Such separation would strain the root of the longitudinal fillet welds, and could lead to tearing. In cyclic loading, it could lead to fatigue failure, initiating from the weld root.

The role of the 16 times plate thickness would only be applicable for material less than 1/2 in. (12.5 mm); otherwise, the 8 in. (200 mm) requirement from paragraph 2.14.1 would govern.

Eccentric Loads

D1.1 requires that at least two lines of longitudinal or transverse welds be applied to lap joints (paragraph 2.4.8, 2.4.8.1). This eliminates the concerns shown in Figure 3. There is a caveat: this requirement does not apply when "the joint is sufficiently restrained to prevent it from opening under load" (paragraph 2.4.8.1). Whatever the external restraint, if rotation is prevented, the concerns of eccentricity are eliminated.

To prevent the condition illustrated in Figure 2, paragraph 2.4.8.2 requires a minimum overlap of five times the thickness of the thinner part, but not less than 1 in. (25 mm). Double fillet welds in lap joints with proper overlap is sufficient to prevent such rotation.

If restrained, the five times overlap provision does not apply. Any sufficient restraint is acceptable, and this is conceptually illustrated in Figure 5.

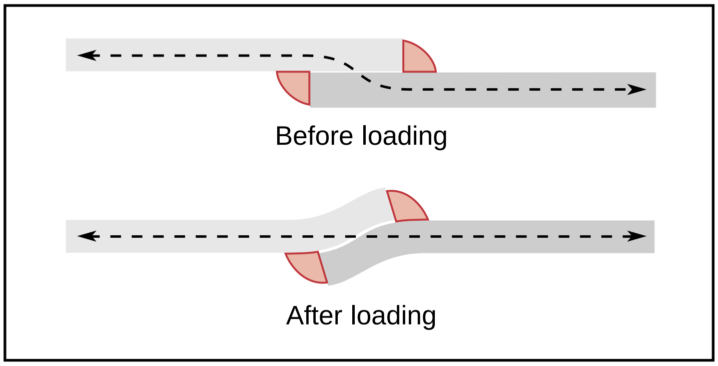

Figure 2

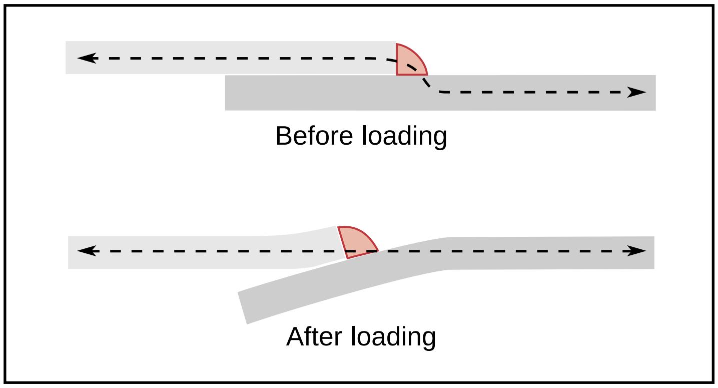

Figure 3

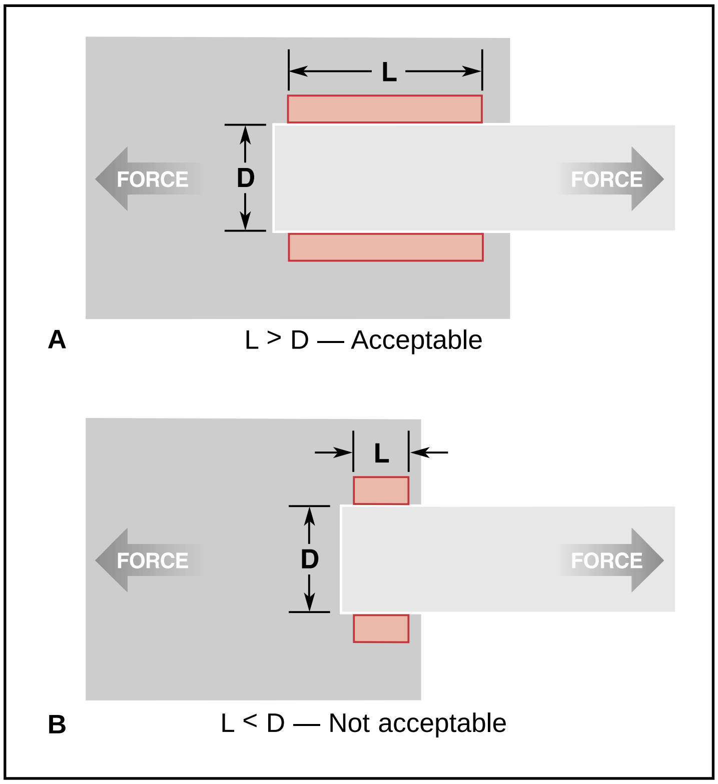

Figure 4

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Other Issues

The code also provides requirements for the details of the fillet welds that are typically used in these connections. For example, the fillet welds are to terminate "not less than the size of the weld from the start of the extension" (paragraph 2.4.7.2). See Figure 6. This is primarily a workmanship concern. Carrying the weld out to the end of the part (where there is little material to conduct away the heat of the weld) often leads to undercut, or melting away of the edges, creating a weak spot in the lapped attachment.

Often, the lap joint lends itself to welds being applied on either side of the joint. Illustrated in Figure 6, the code describes this as welding on "opposite sides of a common plane," and in paragraph 2.4.7.5, requires that the welds be interrupted at the corners. Again, this is to avoid undercut and unacceptable melting of the edges.

The provisions of paragraph 2.4.5 also apply to lap joints. This provision restricts the maximum fillet weld size to the thickness of the base metal for material less than 1/4 in. (6 mm) thick, and for heavier material, to the thickness of the part less 1/16 in. (2 mm), "unless the weld is designated on the drawings to be built out to obtain full throat thickness." See figure 7. This is to avoid the situation where a "nothin' " weld can be generated—that is, a weld that appears to be full size, but in fact lacks the required weld throat. (See Design File, *Welding Innovation* Vol. XVI, Number 1, 1999.)

The selection criteria for longitudinal versus transverse fillet welds could consider the increased allowable strength associated with the transverse option, reducing the required size (see "Consider Direction of Loading When Sizing Fillet Welds," Vol. XV, No. 2, 1998). While this option will result in a higher allowable strength, it comes at the cost of reduced ductility *in the weld.* The ductility of the connected material, typically the point where inelastic strains are designed to be concentrated, would be unchanged with either weld orientation.

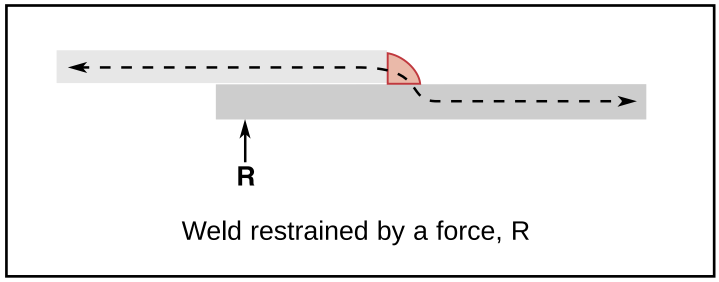

Figure 5

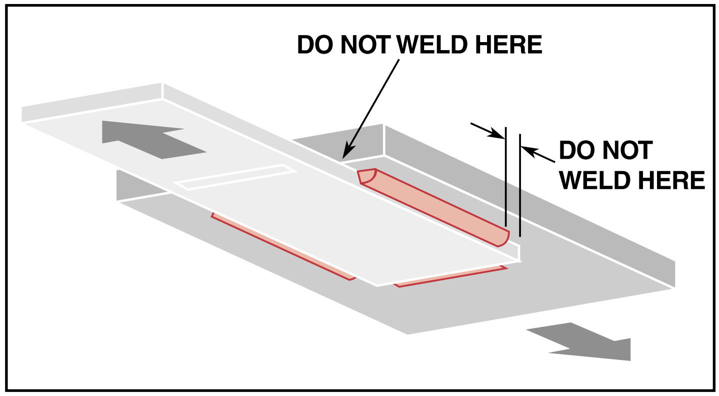

Figure 6

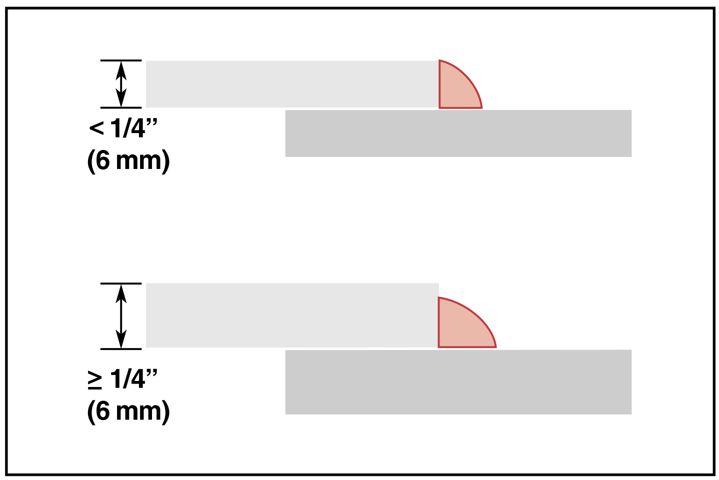

Figure 7

Conclusion

Superficially, detailing a lap joint and the corresponding welds may seem simple, but a variety of important details need to be considered. The following checklist may be helpful:

Are the parts sufficiently restrained to prevent joint rotation? If not, use at least two rows of welds.
Is the overlap at least five times the thickness of the thinner part? And, is it at least 1 in. (25 mm)?
For longitudinal welds, are they at least as long as the distance between them?
For lap joints with only longitudinal welds, is the distance between the welds less than 8 in. (200 mm)? For cyclically loaded members, is this distance also less than 16 times the thinner member?
For material thicknesses of 1/4 in. (6 mm) or more, has the fillet weld leg size been reduced by 1/16 in. (2 mm)?
Have the fillet welds been detailed to terminate at least one weld size from the end of the piece? Are they detailed to avoid tying the welds together on opposite sides of the common plane of contact?

One final note: these provisions are intended to be applied to lap joints designed to transfer stresses between members. For situations involving lap joints but where the joint is more associated with the assembly of a member, and not with transfer of calculated forces, the principles presented above are not necessarily applicable.

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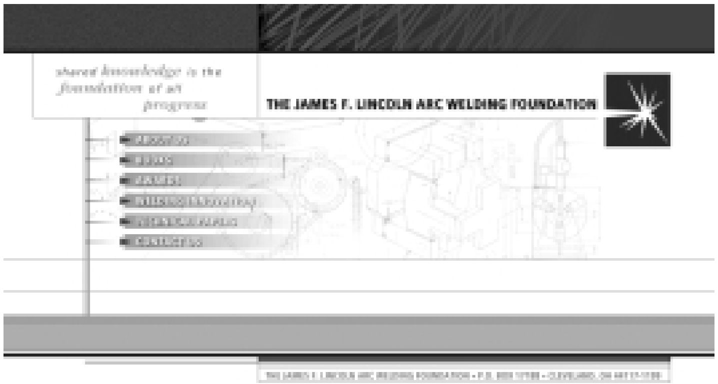

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Engineering for Rehabilitation of Historic Metal Truss Bridges

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Editor's Note: An earlier version of this paper was presented at the 7th Historic Bridges Conference in Cleveland, Ohio, in September 2001, and was published in the proceedings of that conference.

Figure 1. The rehabilitated 133rd Avenue bridge, installed at the Calhoun County Historic Bridge Park.

Introduction

The Calhoun County Historic Bridge Park southeast of Battle Creek, Michigan, displays a collection of rehabilitated metal truss bridges for use and enjoyment by pedestrians. From the perspective of a structural engineer, it was instructive to investigate the general feasibility of rehabilitating century-old metal truss highway bridges for pedestrian service consistent with modern standards for safety[1,2,3] and historic integrity[14]. Engineering aspects of rehabilitation are discussed for bridges that are now in the Park, specifically:

- 133rd Avenue bridge (Figure 1), a pin-connected half-hip Pratt pony truss spanning 64 ft. (19.5 m), erected in 1897 by the Michigan Bridge Company to cross the Rabbit River in Allegan County, Michigan.
- Twenty Mile Road bridge (Figure 2), a 70 ft. (21 m) long riveted Pratt pony truss that spanned the St. Joseph River in Calhoun County. Physical features hint that this bridge was designed for railway service. The builder has not been identified and several sources date construction to the early twentieth century.
- Gale Road bridge, a pin-connected skewed Pratt through truss built in 1897 by the Lafayette Bridge Company. Originally spanning 122 ft. (37 m) over the Grand River in Ingham County, Michigan, this bridge currently is being re-erected in the Park.

Six other bridges have been procured and are awaiting rehabilitation before being put in the Park, including these that also will be discussed

- Tallman Road and Bauer Road bridges, nearly identical pin-connected Pratt through trusses that spanned about 90 ft. (27 m) over the Looking Glass River in Clinton County. Manufactured by the Penn Bridge Company and erected in 1880, they are two of Michigan's oldest through trusses[9].
- Charlotte Highway bridge, manufactured by the Buckeye Bridge Company and erected in 1886. Prior to its recent removal (Figure 3), it crossed the Grand River in Ionia County with a span of 177 ft. (54 m) and was one of very few double-intersection Pratt truss bridges remaining in Michigan[9].

Feasibility

Investigation of feasibility involves comparing historic and modern specifications for bridge design, particularly those governing materials and loads. During the period when the project bridges were built, standards were promulgated by individual iron and steel producers, bridge designers and manufacturers, owners (typically municipal governments) and textbook authors. These standards were numerous and varied; those cited are representative rather than comprehensive.

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Strength of Metals

Although the quality of structural steel has been perfected over the past century, the strength of low carbon steels usually used in bridges has not changed significantly (Table 1). However, the allowable stresses used by bridge designers increased as confidence and understanding developed. This is reflected in the trend toward lower factors of safety illustrated by Tables 1 and 2. Early bridge designers used factors of safety as high as six to compensate for lack of quantitative information. Today, based on results of a century of research and experience, factors of safety of two or less are typical. Modern specifications may allow larger stresses in the old steel and wrought iron members of a historic bridge than did its designer.

Figure 2. The rehabilitated Twenty Mile Road bridge, shown in its new position at the Historic Bridge Park.

Live Load

An old highway bridge may have become deficient in strength due to the increased weight of trucks. In 1916 Waddell[17] advocated designing Class C bridges for a single 6 ton (53 kN) truck weight, and Class A bridges for an 18 ton (160 kN) truck, noting that "Almost all of the old highway bridges are incapable of carrying these new live loads with safety." The smallest design vehicle load currently recognized is a two-axle truck weighing 15 tons[1] (133 kN). However, historic metal highway bridges were designed to carry uniformly distributed loads in addition to, or in lieu of, concentrated axle loads to assure safety for lines of wagons or automobiles, livestock, and crowds of people, the latter being the larger, or governing, distributed load.

Modern specs may allow larger stresses in the members of a historic bridge than did its designer

Figure 3. Lifting the Charlotte Highway bridge from its original abutment. This end was lowered onto a barge prior to hauling the bridge across the river and up the other bank.

Table 3 traces the trend and variations in design values for distributed live loads on highway bridges as well as listing current design values for pedestrian bridges[2]. Ranges reflect levels of service. This table demonstrates that, in general, the published design loads for old highway bridges exceed the current requirement for pedestrian bridges. Bridges with long spans and designed for rural service may be exceptions.

Wind Load

In contrast to distributed live loads, design wind loads have increased significantly. In 1901 Waddell advocated design loads of 250 and 150 lb/ft. (3.65 and 2.19 kN/m) on the loaded and unloaded chords, respectively, for class A bridges with spans of 150 ft. (46 m) or less[16], but by 1916 he had

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Table 1. Tensile strengths of steel and factors of safety for tension fracture at net section.

Source	Year	Grade of Steel	Yield stress, minimum, ksi (MPA)	Ultimate stress, minimum, ksi (MPA)	Allowable stress on net section, ksi (MPA)	Factor of safety for fracture
Pottsville Iron & Steel Co.[7]	1887				15.6 (108)	
Carnegie Phipps & Co.[7]	1889-1893	for bridges			12.5 (86)	
IATM[10]	1900	medium	35 (241)	60 (414)		
Waddell[16]	1901	medium	35 (241)	60 (414)	16 (110) 18 (124)	3.8 3.3
Burr and Falk[4]	1901					3.5 to 6.0@
Copper[12]	1909	medium			10 to 25 (69 to 720)#	2.4 to 6.0#
Michigan[13]	1910	medium	30 (207)	60 (414)	15 (103)	4.0
Bethlehem Steel Co.[7]	1907-11	moving loads			12.5 (86)	
Waddell[17]	1916	medium	35 (241)	60 (414)	16 (110)	3.8
Ketchum[12]	1920	medium			16 (110)	
AASHTO[3]	pre 1905 1905-36		26 (179) 30 (207)	52 (358) 60 (414)	26 (179)* 30 (207)*	2.0* 2.0*
AASHTO[1]	current	ASTM A36	36 (248)	58 (400)	29 (200)*	2.0*

* for inventory rating, less than 100,000 load cycles
@ depending on span # depending on type of load, including impact factor

Table 2. Tensile strengths of wrought iron and factors of safety for tension fracture.

Source	Year	Grade of Steel	Yield stress, minimum, ksi (MPA)	Ultimate stress, minimum, ksi (MPA)	Allowable stress ksi (MPA)	Factor of safety for fracture
Carnegie Kloman & Co.[7]	1873	wrought iron			14 (97)	3
Waddell[15]	1883	iron	26 (179)	50 (345)	8 to 12.5 (55 to 86)#	4.0 to 6.2#
Phoenix Iron Co.[7]	1885				12 (83)	
IATM[11]	1900	refined iron test iron class A test iron class B stay-bolt iron	25 (172) 25 (172) 25 (172) 25 (172)	48 (331) 48 (331) 50 (345) 46 (317)		
Waddell[16]	1901	wrought iron	26 (179)	50 (345)	13 (90)	3.8
AASHTO[3]		wrought iron			14.6 (101)*	

* for inventory rating # depending on service class and influence area

increased those values to 320 and 180 lb/ft.[17] (4.67 and 2.63 kN/m). The Illinois Highway Department designed for the larger of 25 lb/ft.^{2} (1.2 kN/m^2) on the vertical projection of each truss and of the deck, or 300 and 150 lb/ft. (4.38 and 2.19 kN/m) on the loaded and unloaded chords, respectively[12]. Modern specifications[1,2] are much more demanding, requiring design for wind loads of 75 lb/ft.^{2} (3.6 kN/m^2) on the vertical projection of each truss and of the deck, plus 300 and 150 lb/ft. (4.38 and 2.19 kN/m) on the loaded and unloaded chords, respectively (this lineal load is not required for pedestrian bridges), plus 20 lb/ft.^{2} (0.96 kN/m^2) upward on the deck. Clearly, historic bridges are unlikely to have been designed for the wind loads currently mandated.

Structural Analysis and Design

The components of each of the rehabilitated project bridges were analyzed to estimate design stresses associated with internal forces caused by specified combinations of loads[1] and acting on the original uncorroded member cross-sections. Allowable stresses were computed from assumed material properties[3] and specified factors of

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Table 3. Uniformly distributed design live loads for highway bridge trusses in pounds per square foot (kN/m²).

Source	Year	Span		
		50 feet (15.2 m)	100 feet (30.5 m)	200 feet (61.0 m)
Whipple[5]	1846	100 (4.79)	100 (4.79)	100 (4.79)
ASCE[5]	1875	100-70 (4.79-3.35)	75-50 (3.59-2.39)	60-40 (2.87-1.92)
Waddell[15]	1883	100-80 (4.79-3.83)	90-80 (4.31-3.83)	70-60 (3.35-2.87)
Waddell*[16]	1901	170-113 (8.14-5.41)	149-98 (7.13-4.69)	120-80 (5.75-3.83)
American Bridge Co.*[4]	1901	125-100 (5.99-4.79)	125-94 (5.99-4.50)	100-69 (4.79-3.30)
Michigan Highway Comm.[13]	1910	100 (4.79)	100 (4.79)	100 (4.79)
Waddell*#[17]	1916	161-107 (7.71-5.12)	144-95 (6.89-4.55)	119-80 (5.70-3.83)
Ketchum*[12]	1920	151-116 (7.23-5.55)	126-89 (6.03-4.26)	103-60 (4.93-2.87)
Illinois Highway Comm.[12]	1920	125 (5.99)	100 (4.79)	85 (4.07)
Wisconsin Highway Comm.[12]	1920	120 (5.74)	93 (4.45)	50 (2.39)
AASHTO (pedestrian)#[2]	1997	67 (3.21)	65 (3.11)	65 (3.11)

* Prescribes an impact factor, which is included in the tabulated values # For 16 foot (4.88 m) deck width

safety[1]. For each component and load combination, the allowable stress was divided by the design stress. A ratio less than unity indicates need for modification, while a ratio greater than unity suggests that an acceptable level of safety may be achieved without completely restoring corroded sections (in general, significant damage was repaired in the interest of historic integrity and esthetics). The three rehabilitated project bridges were found to have adequate capacity for pedestrian loading.

Unusual Features

The structural analysis of a truss usually is a routine procedure. To simplify computations, the structural engineer assumes that each member transmits force only in the direction of its longitudinal axis. That is, the member is not subject to transverse force (shear) or bending. This assumed behavior is achieved if the members are straight and connected at their ends by frictionless pins, longitudinal axes of members are concentric at connections, and loads are applied to the truss only at connections. Real trusses conform to this idealization only approximately but member forces may be computed with sufficient accuracy if the design approaches the ideal conditions.

The Tallman Road bridge displays two peculiar details that are contrary to the ideal conditions and to subsequent practice. The most obvious is the hip joint, which has two pins rather than one. One pin carries the vertical eyebar and the other carries the diagonal eyebar pair. Because the longitudinal axes of the inclined end post, top chord, vertical and diagonal members do not meet at a common point, bending is induced in the end post and top chord.

The second peculiarity of the Tallman Road bridge is that each lower chord eyebar spans two deck panels and has three eyes: one at each end and one in the middle. When gravity load is applied to a truss, the panel points near midspan typically deflect downward more than those near the ends. If the truss conforms to the ideal conditions, the members rotate but remain straight as the panel points deflect. Obviously this behavior cannot be achieved by a three-hole eyebar. Thus, these unusual lower chord eyebars are subject to bending as well as axial tension.

Strength Not Predicted by Conventional Truss Analysis

Conventional analysis predicts that the lower chord of a single-span through truss is always in tension when the bridge is carrying gravity load. However, the lower chords in the end panels of the Charlotte Highway bridge were observed to be slack (i.e., subjected to compression rather than tension) when the bridge was in service in its original location. Those members remained slack after the vehicular railings and deck were removed in preparation for moving the bridge from its masonry abutments. However, when the bridge was freed from its inoperative expansion bearings, that end appeared to move inland several inches and cracks opened where the wingwalls join the abutments. Apparently the upper chord and end posts had been functioning as an arch as well as restraining displacement of the heavy abutments and fill.

Design wind loads have increased significantly

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Figure 4. Severely corroded sections of the Twenty Mile Road bridge were replaced by welding new steel to sound original material.

Prior to lifting the six-panel Bauer Road bridge from its original abutments, the contractor removed railings, decking and stringers. Then a lifting sling was attached to the upper lateral struts at the third points of the span. Conventional truss analysis predicts that the bottom chord will be compressed when the bridge is lifted in this manner. Since the bottom chord consists of eyebars, which have negligible resistance to compression, it seemed likely that the trusses would collapse. The fact that the lift was accomplished without damage attests that the upper chord, hip joints and end posts possess significant bending strength.

Conventional truss analysis may underestimate the strength of a metal truss bridge. More comprehensive analysis techniques coupled with detailed modeling of connections may make it possible to quantify additional strength.

Inadequate Resistance to Wind Load

By modern design standards, the rehabilitated project bridges had inadequate resistance to wind load. It was necessary to employ a provision[1] that permits design wind speed to be adjusted from a nominal 100 MPH (45 m/s) to reflect favorable local conditions. The inland location of the Park and the low and sheltered sites of the project bridges justify a design wind velocity of 70 MPH (31 m/s). Despite the resulting 50% reduction of wind force, the original anchor bolts typically were inadequate, and each of the three bridges manifested other deficiencies.

Analysis of the 133rd Avenue bridge predicted that modern design wind loads would cause net axial compression of the windward lower chord eyebars. Since eyebars have negligible resistance to compression, they would buckle and the truss would become unstable. This was corrected by installing an unusually heavy deck to create enough tension in the lower chord to counteract the compression induced by wind. Alternatively, it may have been possible to rely on the deck or upper chord to stabilize the trusses as suggested in the preceding section.

The deck lateral ties of the Twenty Mile Road bridge were evaluated using the assumed strength of steel produced before 1905[3] and found to be inadequate. The ties, like other parts of this bridge (Figure 4) were too badly corroded to be salvaged. Replacing them with new steel, in the original sizes, was sufficient to provide the required wind resistance.

Structural analysis showed that the original portal braces of the Gale Road bridge were inadequate. Vertical struts had been arc welded to the lattice panels sometime after construction, apparently to correct perceived weak-

Figure 5. Forge-welded loop eyebars like these are obsolete.

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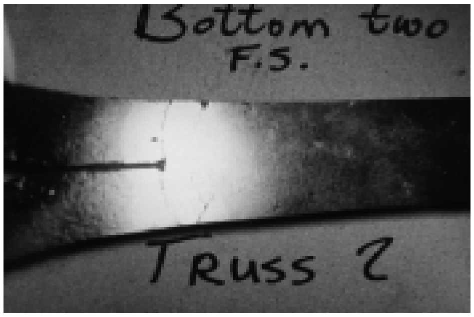

Figure 6. Dye penetrant inspection of a forge weld.

ness, and localized bending of horizontal members occurred after these reinforcements were installed. The original portal braces will be retained

Conventional truss analysis may underestimate the strength of a bridge

for display but not installed on the rehabilitated bridge. The replacement portal braces have larger connection gussets than the originals, and the lattice is steel angles of the same width as the original flat bars. The configuration and overall dimensions of the original portal braces are duplicated.

Features Not Covered in Current Specifications

Pony trusses and loop eyebars (Figure 5) are obsolete, and there are no current standards to guide assessment of these features. Pony trusses are prone to lateral instability of the top chords. That is, the bridge tends to fold inward under heavy load. The two rehabilitated pony trusses were checked for stability by Holt's method[8] and both were found to have adequate factors of safety for pedestrian loading.

Single-load tests of seventeen wrought iron loop eyebars reported by Ellerby et al[6] demonstrated that fracture may occur at a forge weld rather than in the body of a bar, sometimes at a load significantly less than the design strength of the bar. As part of the same investigation, twenty-six wrought iron loop eyebars were repeatedly loaded to working stress level. The number of load cycles to failure suggests that the bars could have remained in highway service for many more decades. When fatigue fractures finally did occur, they were in the loops (except for two bars, which initially had large cracks at forge welds). The investigators speculated that repeated flexing of the loops was a critical factor and noted the deleterious effect of poor fit on the pin.

The usual practice for the project bridges is to inspect eyebar eyes and forge welds visually and by ultrasonic and dye penetrant methods (Figure 6). Cracks are ground out and bars are built back to original profile by arc welding. Testing has shown that careful arc welding restores full strength[6].

Conclusion

Selected historic metal truss bridges that are rehabilitated to near-original condition can satisfy modern safety standards for pedestrian service. This is demonstrated by the bridges on display in the Calhoun County Historic Bridge Park.

Acknowledgements

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Artistic Precision

By Carla Rautenberg
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Many readers of this publication regard welding as an art—for the welders at Mt. Vernon Machine & Tool, that is literally true. The precision metalworking firm located in Mt. Vernon, Ohio, provides the essential welding and cutting services needed to realize the artistic vision of sculptor Barry Gunderson, Professor of Art at Kenyon College in Gambier, Ohio.

According to president Gail Stenger, working with Gunderson is a welcome change of pace for his company's five full-time welders. "We rotate the work so that all of the welders have a chance to work on Barry's pieces. He brings a breath of fresh air into our weld shop."

A Collaborative Effort

The relationship began a little more than a decade ago, when Gunderson's sculpture students started to frequent the metalworking shop looking to purchase round bar and pieces of flat steel for their studio projects. "We really got to know Barry through his students," Stenger recalls. To date, Mt. Vernon Machine & Tool has fabricated five of Gunderson's sculptures, four in aluminum and one in stainless steel.

The collaborative effort begins after Gunderson is notified that he is a finalist in the competition for a commission. "At that point," Stenger says, "he contacts me. He brings in a small model of the piece so we can discuss his ideas for it, and talk about how he would like to see it fabricated. I do a cost breakdown so he can submit a budget."

The working relationship is now so well-established that Gunderson has his own key to the Mt. Vernon weld shop. He sometimes spends evenings there, grinding or painting sections of the current project. "As we get closer to the end of a commission, he's pretty much here all the time," Stenger notes, "grinding so he can get exactly the texture he wants." Gunderson also does much of the plasma cutting work on his sculptures.

Understorms

The first piece Stenger's company welded for Gunderson was "Understorms," which represented the sculptor's first commission from the Ohio Arts Council Percent for Art Program, a statewide initiative that places public art in state facilities. This work, like the other aluminum sculptures created at his shop, was welded using "gas metal arc, spool-style, with assist guns on the welders," according to Stenger, who added that GTAW is commonly used on the smaller parts. Mt. Vernon Machine & Tool installed the sculpture, with Gunderson's oversight, at Franklin Park Conservatory in Columbus, Ohio, in 1991.

Spountain

Several years later, Gunderson was a finalist for another Ohio Percent for Art commission. This time, he approached Stenger with a 2 ft. (0.6 m) cardboard model for another piece he proposed fabricating out of aluminum, this one called "Spountain." The 30-ft. (9 m) tall sculpture would be, said Gunderson, "an abstraction of water." Due to the size of the piece and in consideration of wind shears, Stenger advised that it be constructed not of aluminum, but of stainless steel. The advice duly taken, Mt. Vernon Machine & Tool proceeded to fabricate the piece, using SMAW to create the structure, and GMAW to finish the skin surface.

"Spountain" was too tall to fit inside the Mt. Vernon shop, so a cement pad was built outside to accommodate it during fabrication. The final product, weighing almost 10,000 lbs. (4,500 kg), took 15 months to create, from conception to installation. "Spountain" now resides in front of the George V. Voinovich Livestock and Trade Center on the Ohio State Fairgrounds in Columbus.

Coventry Arch

Gunderson's most recent collaboration with Mt. Vernon Machine & Tool resulted in a work that serves as the symbolic gateway to a neighborhood (see back cover). The Coventry PEACE Public Art Committee, a community group in Cleveland Heights, Ohio, wanted to enhance a newly landscaped park area in front of a local branch of the Cleveland Heights-University Heights Public Library.

Ten regional artists responded to a call for entries; of these, Gunderson was among the three finalists. In his proposal, he explained: "I have been fascinated with the complex invention of turning industrial materials—pipes and structures—into anatomical forms … My intent here is to use 12 in. (300 mm) diameter aluminum pipe rolled into a 180° arch to form a passage way of greeting—two abstract figurative forms

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on either side … four figures, two on each side, will thus form two arches, one slightly higher than the other … My hope is that this figurative cluster will serve as a symbol of the community's interactions with each other and with visitors …"

Describing the Coventry Arch, Stenger reported, "If Barry had a favorite project of the ones we've done together, that was it." He explained that Gunderson conducted art workshops with the children attending nearby Coventry Elementary School, and that the children's involvement and interest added an extra dimension to the project for all concerned.

The collaboration between artist and welding shop is one of mutual commitment and respect. "Barry knows we'll go the extra mile to give him what he needs. We've cut the parts apart and rewelded them when he wanted us to," says Stenger. For his part, Gunderson states, "I feel very fortunate to have such a special relationship with Gail and his workers. It truly helps me see my artistic visions come to life."

Mt. Vernon Machine & Tool, in business since 1924, employs a total of thirty people. Gail Stenger represents the fourth generation of his family in the business.

"Understorms"

"Spountain"

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**The
James F. Lincoln
Arc Welding
Foundation**

This welded aluminum arch provides a visual “gateway” to a neighborhood. Story on page 16.

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