Volume XV, Number 2, 1998

Welding Innovation

Advancing Arc Welding Design and Practice Worldwide

Closing the Gap Between Education and Practice

The most important question currently facing the structural engineering profession is an old one, but it looms large: "How can we do things better in the future?" Some of the challenges confronting us include:

- Extremely lengthy, complex and difficult-to-understand codes and specifications
- Inadequate understanding of the loading and deformational effects (i.e., wind or seismic) on the performance of structures
- The need to improve existing, or develop new, construction materials, and to provide for accelerated testing of such materials in order to fully qualify them for use
- Greater attention to such elements of structural design as redundancy (of resistance), stability (local and gross aspects), and torsional performance
- A need to improve the quality of the constructed product, including addressing the fact that many of our structural failures occur during construction

From where I stand, many of the most critical issues facing our profession will be addressed only when we begin to close the gap between the theories taught in our baccalaureate and graduate engineering programs and the actual nitty-gritty of structural engineering practice.

We presume that the education of a structural engineer includes a strong grounding in "the basics," including physics, chemistry, mathematics, and mechanics, as well as the language skills that enable the individual to think, write, and speak effectively. Ideally, this foundation should then support specialization in the structural elements, covering design, analysis, proportioning in different materials (steel, concrete, wood, etc.), followed by an introduction to the synthesis of a complete structure. In addition to computational training, the student should have some exposure to laboratory work. Finally, the engineer-to-be should receive a well-rounded education in disciplines such as the social sciences and the humanities; since, after all, the structural engineer's client is our society in its broadest sense.

Sadly, we must admit that few engineering students, either in the United States or in other countries, receive an education as thorough as the one described above. And yet I believe that such a curriculum represents the bare minimum that will enable us to face the problems challenging structural engineering as a profession.

Then, *to close the gap between education and practice*, we will have to go further. We will have to commit ourselves to the premise that a properly educated structural engineer has an in-depth knowledge of the properties of materials, including basic

W. J. Hall

strength properties, fatigue and fracture characteristics, and durability, as well as a basic understanding of welding and joining technologies. To this end, there is no substitute for hands-on learning in the laboratory about the properties of materials, as well as the behavioral aspects of structural elements fabricated from these materials. The educational process should emphasize the importance of analyzing the behavior of the entire structure, from the foundation up. It would be ideal if the student could gain some experience in actual structural practice, including the all-important area of nondestructive inspection.

If, as a profession, we can close that gap between the classroom and the real-life situation, it will pave the way for the development of:

- Clear, concise, performance-based specifications
- Accurate predictions of how structures will perform under adverse loading and deformation conditions
- Higher-quality, more reliable, more durable materials
- Designs that take into account resistance, stability, and torsional performance
- The expert coordination and continuous attention to detail that will produce the highest quality of construction and structural integrity

These are goals worthy of our very best efforts.

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Adapted in part from the Keynote address presented at Structural Engineers World Congress, July 1998, San Francisco, CA. Used with permission from the publisher of the Proceedings, Elsevier Science Ltd.

Welding Innovation

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The James F. Lincoln Arc Welding Foundation

Cover: This dramatic bridge in Rotterdam, the Netherlands, features a state-of-the-art movable section, or bascule bridge, to accommodate ships that exceed the customary Rhine navigation height. The construction of the Erasmus bridge, fabricated using Lincoln Smitweld products and equipment, is described in the story starting on page 19.
Cover photo: Aeroview Rotterdam

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Field Welding—Common Problems and Their Solutions

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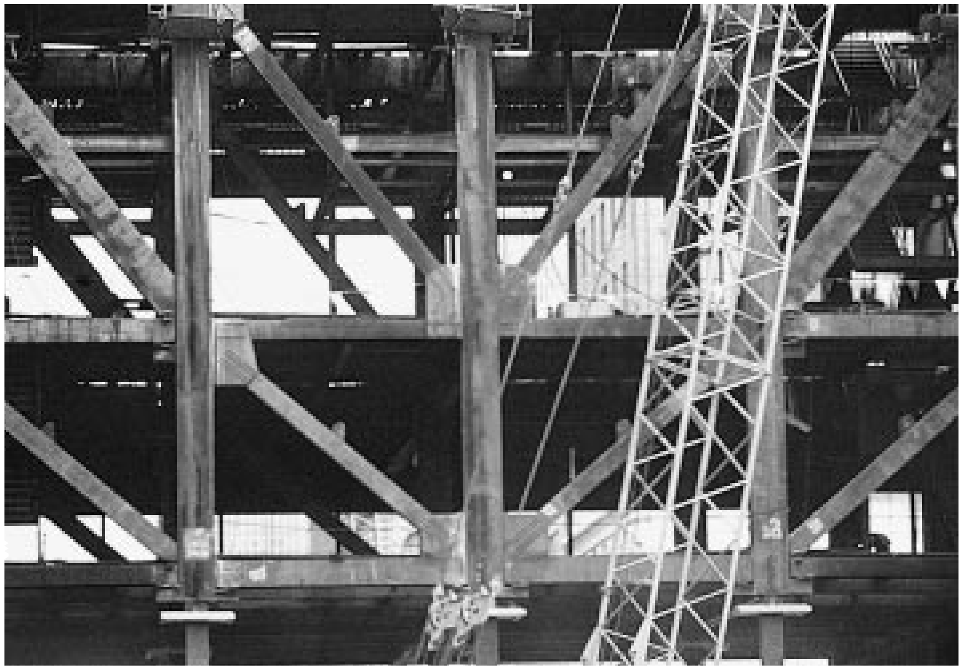

Figure 1. Transfer truss of A588 material, 80 ft (24.4 m) long by 36 ft (11 m) deep.

Introduction

More than three decades of involvement with the fabrication, construction and welding of steel structures has convinced me that the *ANSI/AWS D1.1 Structural Welding Code — Steel* (1) is a standard that works. As a consensus document, the D1.1 Code represents a wealth of experience accumulated by engineers, fabricators, erectors, inspectors and educators, over decades of working in the structural welding industry (2). This article will provide examples of field welding problems of various kinds, and describe how they are addressed in the Code. Some of the problems are quite basic, but they all pose the same question: How should the weld be made or repaired so that the product will function as the engineer intended?

The "Main Rule"

Although the D1.1 Code stipulates various rules to be followed in order to create sound welds, there is one rule that should be mandated even though it is not listed in any code or specification. I call this the "Main Rule," and I believe it may be the most important rule when one is involved with fabrication or welding:

> ***"Don't hurry too much; take the time to do the job properly, because otherwise, you may have to do it twice."***

Everyone should be interested in doing the job properly, because as fabricators or erectors we really have nothing to sell if the engineer or inspector will not accept the finished product. In other words, we are all in this together.

Preparation of Material

Whether in the shop or in the field, something that is easy to control with minimal effort is the careful preparation of material. Obviously, it is impossible to produce a good weld if the material has not been properly prepared. The tolerances and details shown in the D1.1-98 Code, Figure 5.3, "Workmanship Tolerances in Assembly of Groove Welded Joints," are there to insure that the welded joint can be made satisfactorily. A little care exercised here will save a lot of time and money for everyone.

For groove welds, the preparation starts with the cutting of the groove. Regardless of the groove type (V, bevel, U, etc.), the surface of the groove must be reasonably smooth and free of notches and gouges. All too often, one hears the comment, "I'll melt through that." Most of the time, it simply doesn't happen. Other problems associated with preparation are:

- Bevel not uniform, resulting in improper groove angle and varying root face
- Rough surfaces and gouges, which can result in slag inclusions and lack of fusion
- Cut surface not cleaned of cutting slag, with an outcome of poor fit-up, porosity, and lack of fusion

Any person claiming not to have been exposed to these conditions has not spent any time in the steel fabrication and erection business. Proper adjustment of the cutting flame, well maintained equipment, and attention to dimensions will add up to good, accurate preparation.

Probably the primary reason for porosity in welds is a joint with a dirty surface that is to be welded. For fillet welds in particular, surface cleanliness is vital to sound weld metal. Many excuses are

Return to TOC

given to avoid cleaning, but nothing excuses the result: porosity.

This particular issue is the basis for paragraph 9.21.1.6 in the *D1.5-96 Bridge Welding Code* (3), which reads as follows:

> A subsurface inspection for porosity shall be performed whenever piping porosity 2.4 mm or larger in diameter extends to the surface at intervals of 300 mm or less over a distance of 1200 mm, or when the condition of electrodes, flux, base metal or the presence of weld cracking indicates that there may be a problem with piping or gross porosity.

Heavy Trusses

On one project, very large transfer trusses made of A588 steel were used to span a city street and carry a 40-story office building (see Figure 1). The trusses were 36 ft (11 m) deep with a span of 80 ft (24.4 m), and had a top, bottom, and intermediate chord. Several welding problems were associated with these trusses, so the following specific areas will comprise our examples:

- Welded lifting lugs on flame cut edges
- Flame cut edges of the truss members
- Weld access holes
- Electroslag welds

Welded Lifting Lugs

The first example illustrates the problem of welding on diagonal member flame cut edges. The welding caused a base metal underbead crack in the diagonal member. As the tension stresses increased during the erection of the structural steel, the cracks propagated from under the lifting lug weld metal until they were visible. When the erection lugs were removed, the area ground smooth, and the member dye penetrant inspected, several cracks were found in the base metal. Obviously, the preheat was not sufficient, but the primary cause of these cracks was that the low hydrogen electrodes had not been handled properly.

When thick A588 steel (a 4.5 in [114 mm] thickness in this case) is cut, a hard skin and heat affected zone forms on the cut surface. The higher the strength of the steel, the more important it is that low hydrogen electrodes be properly handled. Keeping electrodes in the oven, getting them only from the oven, and taking only enough to last a short time are essential rules for welding situations like this one.

...surface cleanliness is vital to sound weld metal

The *D1.5-96 Bridge Welding Code*, Section 12- Fracture Control Plan, addresses welding on flame cut edges. For some applications, consideration is being given to requiring that the steel be preheated before flame cutting (4, 5, 6). This is now required prior to thermal cutting of weld access holes in ASTM Group 4 and 5 shapes and welded built-up shapes of thicknesses in excess of 1-1/2 in (38 mm). The storage of low hydrogen electrodes as described in Section 5.3 of AWS D1.1 has always been a code requirement.

Flame Cut Edges

On the flame cut edges, some cracking problems were experienced as a result of weld shrinkage stresses in areas where the surface roughness was excessive, and the surface had hardened due to the high cooling rate during the cutting operation. Random weld arc strikes further aggravated the problem. It was corrected by running hardness tests on all flame cut edges, etching the surface to locate random weld arc strikes, and grinding all edges to remove the surface hardness. All flame cut edges were checked for hardness and were judged unacceptable if the Rockwell C hardness (Rc) was higher than 30. Research has shown that oxygen cutting of steel, particularly in heavy sections, leaves a thin layer of hard, untempered, high carbon martensite at the cut surface. The carbon content of the steel at the cut surface is higher than the average base metal chemistry, because during oxygen cutting, other elements such as iron, silicon, and manganese, are burned away. The hard surface layer produced by thermal cutting and the hard heat affected zone directly below the surface are very shallow when cutting is done under normal conditions. A micro hardness, Vickers or Knoop hardness measurements in excess of the Rockwell C50 and even C60, is frequently encountered at the surface. However, normal base metal hardness is generally undisturbed at depths greater than 3/32 in (2.4 mm) from the thermal cut edge (5).

The D1.1 Code now addresses the surface roughness issue and the welding of gouges which are too deep to be removed by grinding. Paragraph 5.15.4.2 of the AWS D1.1-98 Code states:

> **5.15.4.2 Profile Accuracy.** Steel and weld metal may be thermally cut, provided a smooth and regular surface free from cracks and notches is secured...

and

> **5.15.4.4 Gouge or Notch Limitations.**....In thermal-cut surfaces, occasional notches or gouges may, with approval of the Engineer, be repaired by welding.

Weld Access Holes

Special attempts were made to prevent weld access holes from cracking in thick material. All flame cut access holes were ground smooth, and all edges were made with smooth transitions and free of notches. Paragraph 5.17.2 of the D1.1-98 Code addresses this issue:

> **5.17.2 Group 4 and 5 Shapes.** For ASTM A6 Group 4 and 5 shapes

and built-up shapes with web material thickness greater than 1.5 in (38.1 mm), the thermally cut surfaces of beam and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods.

After grinding, all tension areas were painted to protect against stress corrosion cracking from the chemicals in the spray fireproofing.

Electroslag Welds

Welding of the 4.5 in (114 mm) and 5.5 in (140 mm) thick diagonal braces and horizontal struts to the column flanges was done using the consumable guide electroslag welding process. Because the weld joints were severely restrained, making the electroslag welds introduced high residual stresses into the structure. Furthermore, developing workable electroslag welding techniques required numerous starts and stops and repairs.

A total of 36 electroslag field welds were made on each truss. For the specific weld joint (A588 base material, 4.5 in [114 mm] thick and 68 in [1.7 m] long), the material thickness mandated the use of two guide tubes, each with a diameter of 5/8 in (16 mm). Electroslag welds penetrate deep into the base metal, which can require extensive base metal repairs if run-off tabs cannot be used. In this case, the weld joint geometry did not permit the use of a run-off tab on one side of the weld joint. As a result, the weld melted into the base metal of the column flange, creating a gouge.

This melt-back area had to be repaired, and as is all too often the case, a combination of events resulted in an underbead crack and brittle failure. The 4.5 in (114 mm) flange sustained a crack approximately five feet long; in addition, cracks occurred in the 0.75 in (19 mm) web of the box column and the 4.5 in thick flange of the horizontal strut.

Figure 2. Crack repair joint configuration.

The crack resulted from the following sequence of events: the welder gouged out the area at the top of the joint and started to make a repair as he had done many times before. Because it took most of the day to get the joint ready to weld, there was only time for one or two passes before the welder quit for the day. Preheat and low hydrogen electrodes were used,

...weld repairs can be made almost anyplace, and in almost any environment

but the preheat was not high enough and the low hydrogen electrodes were not properly controlled. Furthermore, preheat was not maintained overnight, when the temperature dropped from a high of 60°F (33°C) to a low of 30°F (17°C) in approximately three hours. As a result, the cracks described previously were found when we arrived for work the next morning. Macro etched samples in the weld area showed that hydrogen underbead cracks had extended into the base metal and initiated the brittle failure.

The repair was pretty straightforward. We gouged out the joint to a configuration that would permit easy access for welding (see Figure 2). We tried to duplicate the prequalified joint detail B-U2a in Figure 3.4 of the AWS D1.1-98 Code. A back-up plate was placed on the inside of the column, and the area was preheated to 300°F and maintained 24 hours a day until the welding was complete. The repair area at the top of the gouge was larger than usual because a coupon was removed to determine the cause of the crack. This was handled by adding weld metal to the sides of the joint until the joint configuration was restored to that of the original B-U2a joint detail. The weld was radiographically and ultrasonically inspected 100%. No rejectable indications were found. The other cracks were repaired in a similar manner.

Piping Porosity

Another example of a problem associated with electroslag welds is piping porosity (see Figure 3). It was more prevalent with the old electroslag techniques developed during the 1970's than it is now with the new, improved narrow gap technique that is presently in use and being proposed for tension members on highway bridges.

The AWS D1.1-98 Code mandates additional evaluation with radiography if piping porosity is suspected in electroslag welds. A note to the Ultrasonic Acceptance-Reject Criteria Tables states as follows:

> **Electroslag or electrogas welds:** Discontinuities detected at "scanning level" which exceed 2 in (51 mm) in length shall be suspected as being piping porosity and shall be further evaluated with radiography.

Piping porosity is usually caused by moisture contamination related to shoe packing to prevent molten metal tap-outs during welding. Packing is used to fill the gaps from misalignment during fit-up and to prevent molten

Return to TOC

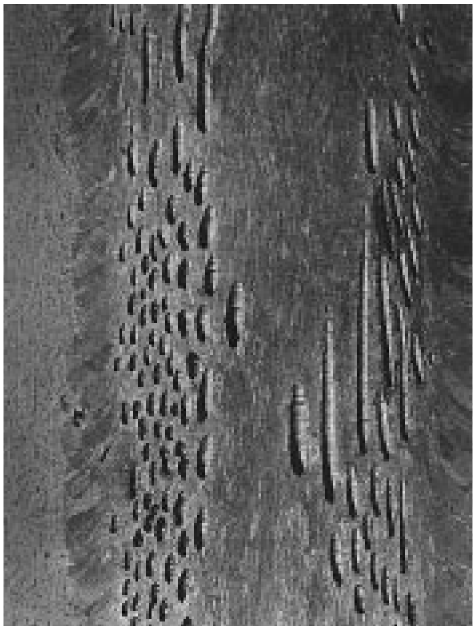

Figure 3. Piping porosity in an electroslag weld.

metal from tapping out from behind the copper water-cooled shoe. If an excessive amount of moisture is used to make the packing into a paste, the water can be drawn into the molten metal as the weld puddle travels up the joint.

Normally, piping porosity is not rejectable by the ultrasonic inspection techniques in the D1.1 Code. Ultrasonic inspection will detect piping porosity indications, but the amplitude response is not high enough to produce rejectable readings per the Code. This is the result of the sound scattering from the spherical surface of the porosity and not returning to the transducer. It can be compared to a light beam that reflects away from the light source when it strikes a curved mirror.

Beam Cope and Access Holes

Another area that has received considerable attention over the years is that of beam cope and access holes. The AWS D1.1-98 Code requires that in ASTM A6 Group 4 and 5, shapes and built-up members with web material thickness greater than 1.5 in (38 mm) get special treatment:

> **5.17.2 Group 4 and 5 Shapes**...The thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods...

> **5.17 Beam Copes and Weld Access Holes.** Radii of beam copes and weld access holes shall provide a smooth transition free of notches or cutting past the point of tangency...

ASTM A6 Group 4 and 5 shapes may crack in the cope hole because of reduced mechanical properties in the flange to web area, and the high residual stresses and micro hardness caused during the thermal cutting of the cope hole. In thick material, the weld shrinkage is restrained in the thickness direction, as well as in the width and length directions, causing triaxial stresses to develop that may inhibit the ability of ductile steel to deform in a ductile manner. Under these conditions, the possibility of brittle fracture increases.

An example of this problem was found during the investigation of the fracture of a web in a 14WF430 jumbo section. The crack started at the radius of the cope hole in the center of the 1.75 in (44 mm) thick web. The crack propagated through the web and arrested only after it ran into the compression area of the opposite flange. The stresses in the web were induced when the flanges were welded before welding the web with the FCAW externally shielded process. This problem was corrected by grinding all cope holes smooth and removing all surface hardened material created during the flame cutting of the cope.

Another example is a case where workmanship in both cutting the cope and in making the weld caused the crack. The crack started at the cope hole and extended down through the web of the girder. This crack resulted from a notch in the cope hole, and was compounded with high residual stresses from over-welding of the web joint (the weld was over 1 in [25 mm] wide). This was also a case where the design should have extended the web into the cross member that was being joined, eliminating the web weld which created the stresses that caused the crack. Further investigation of the crack showed that it extended down through the web and into the bottom flange of the 14 ft (4.3 m) deep girder. AWS D1.1-98 addresses root openings greater than those normally permitted as follows:

> **5.22.4.3 Correction.** Root openings greater than those permitted in 5.22.4.1, but not greater than twice the thickness of the thinner part or 3/4 in, whichever is less, may be corrected by welding to acceptable dimensions prior to joining the parts by welding.

Another example of cracks in the weld access holes was exhibited in a wide flange structural shape that was cut manually and had excessive surface roughness (6), as shown in Figure 4. The thermal cut was made without preheat or any other precautions to minimize the detrimental effects of

Figure 4. Wide flange section weld access hole crack.

Return to TOC

thermal cutting. This can induce large tensile residual stresses and can produce a hard, brittle martensitic layer where cracks may initiate. The size of these cracks depends on the magnitude of the residual and applied tensile stresses and on the fracture toughness of the material. In this case, large cracks were found propagating from the perimeter of the weld access holes during fabrication. These cracks resulted from thermal stresses from the cutting operation and residual tensile stress from the cutting and welding operations. The intensification of these stresses caused by the roughness of the thermally cut surfaces created a stress that was significant enough to initiate and extend cracks even in tough material.

Figure 5. 50,000 ton (50,803 m ton) Loewy forge press grinder.

Fatigue

Fatigue failure can occur even from nonstructural attachments when they are not removed, as shown by the example of an erection lug on the tension flange of a 50,000 ton forge press (see Figure 5). In this particular instance, welding cannot be blamed because the girder had a cast flange and the erection lug was part of the casting, not a welded attachment. It is included here as an example of a classic fatigue failure. The girder was one of six which made up the bottom half of the press. It was 14 ft (4.27 m) deep, 11.5 ft (3.5 m) thick and 36 ft (11 m) long. The offending lug caused no problems until the girder was loaded to its fatigue limit. In this case, that was after the press had been used for 38 years and loaded approximately two million times. Investigation revealed that almost every other lug had started to fatigue-crack through the bottom flange in the same way. Analysis of the fracture face shows that the girder crack was a brittle failure that occurred after the fatigue crack reached critical size.

The D1.1 Code mandates additonal evaluation with radiography if piping porosity is suspected in electroslag welds

The girder was repaired using a modified version of the Fracture Control Plan in Section 12 of the *D1.5-96 Bridge Welding Code.* The plan was altered to suit the actual conditions in the field. The engineer wanted the repair to be accomplished with welds made in the vertical position, which required that the girder be repositioned several times. Only 5/32 in (4 mm) diameter shielded metal arc electrodes were permitted to be used in the root of the weld joint. Then, only after enough weld metal had been deposited to provide suitable access for the welding nozzle, was the process changed to the flux cored externally shielded semi-automatic process. The material was preheated to a minimum of 375° F (190° C) and as high as 425° F (220° C) to prevent hydrogen induced cracking. The welding electrodes were controlled to the maximum extent possible to prevent hydrogen contamination.

We also found defective base metal on the flange edge which appeared to have been caused by the sand mold collapsing into the molten metal during casting. The metal surface was cracked in a pattern similar to what one would see in cracked tempered glass. The defective base metal was up to 2 in (50 mm) deep and 6 ft (1.8 m) long. The area was gouged to remove all defective base metal, ground smooth, and magnetic particle inspected. The flange was rebuilt with weld metal to its original contour. After the girder had been completely welded, post weld heat treated, and stress relieved, it was turned over to the customer. The customer had wanted the girder to last for at least two years of service after the repair. The repair was made more than six years ago, and the girder is still in service and in daily use.

Weld Repairs Can Be Made Anywhere

Repairs successfully made to a 1,200 ft (365 m) radio tower (Figure 6) prove that weld repairs can be made almost anyplace, and in almost any environment. The tower consisted of three legs made of A588 material on 12 ft (3.6 m) centers that were 8 in (200 mm) solid round bars extending up to 1,200 feet in the air. The main guy pull-offs were 2 in (50 mm) thick by 32

Return to TOC

Figure 6. Weld repairs were successfully made to this 1,200 ft (365 m) radio tower.

in (800 mm) long gusset plates and were at the 300 ft (90 m), 600 ft (180 m), 900 ft (275 m) and 1,200 ft (365 m) elevations.

The guy pull-off welds had originally been made with the Gas Metal Arc welding process, commonly called MIG welding. The weld joint angles were too small to permit the nozzle of the MIG gun to access the bottom of the joint, resulting in a lack of fusion in the

Fatigue failure can occur even from nonstructural attachments when they are not removed...

root area. There also had been a lack of fusion in the subsequent passes due to improper welding parameters and techniques caused in part by welding in the short circuiting transfer mode.

The girder was repaired using a modified version of the Fracture Control Plan in Section 12 of D1.5-95. Temporary rigging was installed to take the stresses off the guy pull-off lugs. The weld joints were gouged and repaired in a sequential manner specified by the engineer. After the repair was started, inclement weather (including freezing rainstorms and 70 mph [112 kmph] winds) ensued. We did no actual welding during this type of weather; however, we did maintain 325° F (160° C) preheat on all three legs at the same time until the repairs were complete. The finished welds were ground smooth and all corners had weld reinforcing added for grinding radiuses to enhance the fatigue characteristics of the weld joint (see Figure 7).

Conclusion

By providing the preceding examples of technically challenging field welding problems and their solutions, I hope I have made it clear that two basic elements of success are teamwork, and careful adherence to applicable code provisions. In closing, because I believe it cannot be stated too often, I will once again repeat my "Main Rule":

"Don't hurry too much; take the time to do the job properly, because otherwise, you may have to do it twice."

Figure 7. Weld joints ground smooth to enhance resistance to fatigue.

References

(1) American Welding Society, *ANSI/AWS D1.1-98 Structural Welding Code - Steel*, 1998, Miami, FL.

(2) Post, J.W. "Overcoming Problems in Welding and Fabrication," *Proceedings of the Structural and Heavy Steel Fabrication FMA/AWS Conference,* Houston, TX, November, 1995.

(3) American Welding Society, *ANSI/AASHTO/AWS D1.5-96 Bridge Welding Code*, 1996, Miami, FL.

(4) Barsom, J.M., "Properties of Bridge Steels," Volume 1, Chapter 3, *Highway Structures Design Handbook*, American Institute of Steel Construction, Chicago, IL.

(5) American Association of State Highway and Transportation Officials, "AASHTO Commentary on the ANSI/AASHTO/AWS D1.5-88 Bridge Welding Code," 1991.

(6) Barsom, J.M. "Noncolumn Application of Wide Flange Shapes," *Proceedings of the Association of Iron and Steel Engineers Engineering Conference*, Miami, FL, June 1988.

Design File

Consider Direction of Loading When Sizing Fillet Welds

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

The traditional approach used to design a fillet weld assumes that the load is resisted by the weld's throat, regardless of the direction of loading. Experience and experimentation, however, have shown that fillet welds loaded perpendicular to their longitudinal axis have an ultimate strength that is approximately 50% greater than the same weld loaded parallel to the longitudinal axis. The traditional approach, in which direction of loading is not considered, is therefore conservative. Such a philosophy was incorporated into the AWS *D1.1 Structural Welding Code - Steel*, as represented by the following provision from the 1994 edition:

2.3.2 Fillet Welds. The effective area shall be the effective length multiplied by the effective throat. Stress in a fillet weld shall be considered as applied to this effective area, *for any direction of applied load*. (Emphasis added)

The same code defines the effective throat as follows:

2.3.2.4. The effective throat shall be the shortest distance from the joint root to the weld face of the diagrammatic joint.

This definition of effective throat is also conservative. It accurately defines the theoretical failure plane for fillet welds loaded parallel to their length, but underestimates the increased effective throat that results when the failure plane moves from a 45° orientation to a 67.5° orientation, characteristic of fillet welds loaded perpendicular to their longitudinal axis.

Changes incorporated into the 1996 D1.1 Code, and subsequently repeated in the 1998 edition, offer the potential for significant savings. From D1.1 - 98, the following is found:

Correction

In the Design File column published in *Welding Innovation*, *Vol. XV, No. 1, 1998*, entitled "Consider Penetration When Determining Fillet Weld Size," two errors appeared on page 22. The last sentence of the left hand column should read "In order to help prevent centerline cracking, the w/d ratio should exceed 1:2." The caption for Figure 5 should read "A weld that cracked due to an insufficient width-to-depth ratio." We regret the errors.

2.14.4 In-Plane Center of Gravity Loading. The allowable stress in a linear weld group loaded in-plane through the center of gravity is the following:

$$F_V = 0.30\, F_{EXX}\, (1.0 + 0.50 \sin^{1.5} \Theta)$$

where:

F_V	=	allowable unit stress, ksi
F_{EXX}	=	electrode classification number, i.e., minimum specified tensile strength, ksi
Θ	=	angle of loading measured from the weld longitudinal axis, degrees

For parallel loading, Θ = 0, and the parenthetical term in the above equation becomes 1, yielding the same allowable unit stress as has been traditionally permitted. For perpendicular loading, Θ = 90°, and the parenthetical term becomes 1.5, permitting the increased allowable unit stress.

Design Example

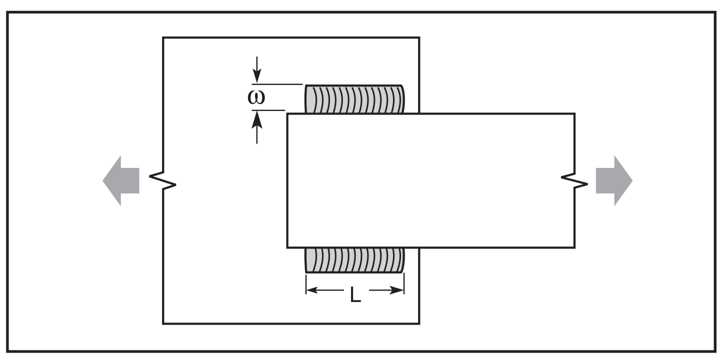

Figure 1. Lap joint with fillet welds loaded in parallel.

Consider the two assemblies shown in Figures 1 and 2. The weld sizes would be computed as follows:

Using an E70 electrode (E48), and with L = 4" (100mm, 0.1m), what weld size is needed to resist the applied load of 40 kips (180 kN)?

Return to TOC

$F_V = 0.30\ F_{EXX}\ (1 + 0.5 \sin^{1.5} \Theta)$

ENGLISH

$F_V = 0.30\ (70\ \text{ksi})\ (1 + 0.5 \sin^{1.5} 0°) = 21\ \text{ksi}$

$F = F_V\ (A) = F_V\ (2\ \text{welds})\ (L)\ (0.707)\ (\omega)$

$$\omega = \frac{F}{F_V 2L(0.707)} = \frac{40\ \text{kips}}{(21\ \text{ksi})(2)(4")(0.707)}$$

$= 0.337"$

Use 3/8" fillet

METRIC

$F_V = 0.30\ (480\ \text{MPa})\ (1 + 0.5 \sin^{1.5} 0°) = 144\ \text{MPa}$

$$\omega = \frac{180\ \text{kN}}{(144\ \text{MPa})(2)(0.1\ \text{m})(0.707)}$$

$= 0.0088\ \text{m}\ (8.8\text{mm})$

Use 10 mm fillet

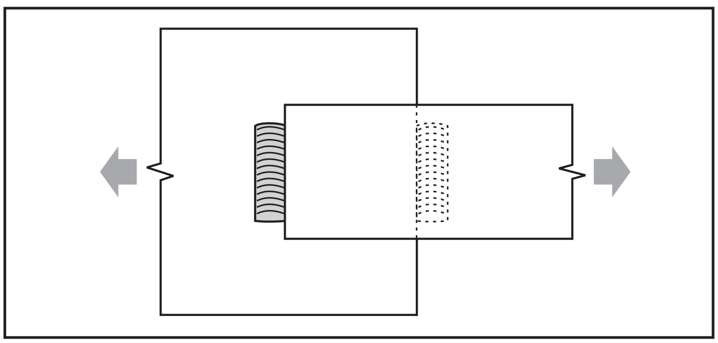

Figure 2. Lap joint with fillet welds loaded perpendicularly.

ENGLISH

$F_V = 0.30\ (70\ \text{ksi})\ (1 + 0.5 \sin^{1.5} 90°) = 31.5\ \text{ksi}$

$$\omega = \frac{40\ \text{kips}}{(31.5\ \text{ksi})(2)(4")(0.707)} = 0.224"$$

Use 1/4" fillet

METRIC

$F_V = 0.30\ (480\ \text{MPa})\ (1 + 0.5 \sin^{1.5} 90°) = 216\ \text{MPa}$

$$\omega = \frac{180\ \text{kN}}{(216\ \text{MPa})(2)(0.1\ \text{m})(0.707)}$$

$= 0.00589\ \text{m}\ (5.89\ \text{mm})$

Use 6 mm fillet

Consistent with expectations, the welds in Figure 2 are permitted to be decreased — in this case, by two standard weld sizes. The welds in Figure 2 require 55% less weld metal than the welds in Figure 1.

Decreased Deformation Capacity

Along with the increase in strength of welds loaded perpendicular to their length, the researchers found a decrease in the deformation capacity before failure. If significant post-yielding deformation capacity is desired, the assembly in Figure 1 would be preferred. Most engineered structures are expected to remain elastic under design loads, so consideration of only the strength is generally adequate. However, for structures that may be subject to overload conditions where large amounts of plastic deformation that precede failure are desired, the designer may choose to orient the welds parallel to the major applied load.

Practical Applications

In order to capitalize upon the additional allowable stress capacity, the designer must orient the welds so that they are as nearly perpendicular to the applied load as possible. Notice that the equation permits the use of any value of Θ, even though the examples have shown the extremes of 0° and 90°.

The increased deformation capacity of longitudinally loaded fillet welds may have some design advantages in certain applications. When this is the case, geometries that involve the application of loads perpendicular to the weld's longitudinal axis should be avoided.

The designer has the opportunity to review existing designs and determine whether weld sizes can be reduced. It is imperative, however, that this approach only be employed where previous designs were based upon accurate assumptions and calculations. In many applications, weld sizes have been modified over the years, increasing or decreasing in weld sizes based upon prototype behavior or field experiences. Reduction of weld sizes under these conditions would be inappropriate.

Even though the particular product that is being designed may not fall under the domain of the D1.1 Code, these principles apply and could be used on other types of welded applications other than structures.

Conclusion

The orientation of welds with respect to the primary applied load significantly affects the weld metal allowable stress, as well as the overall deformation capacity. The designer should consider these factors in order to maximize performance while minimizing costs.

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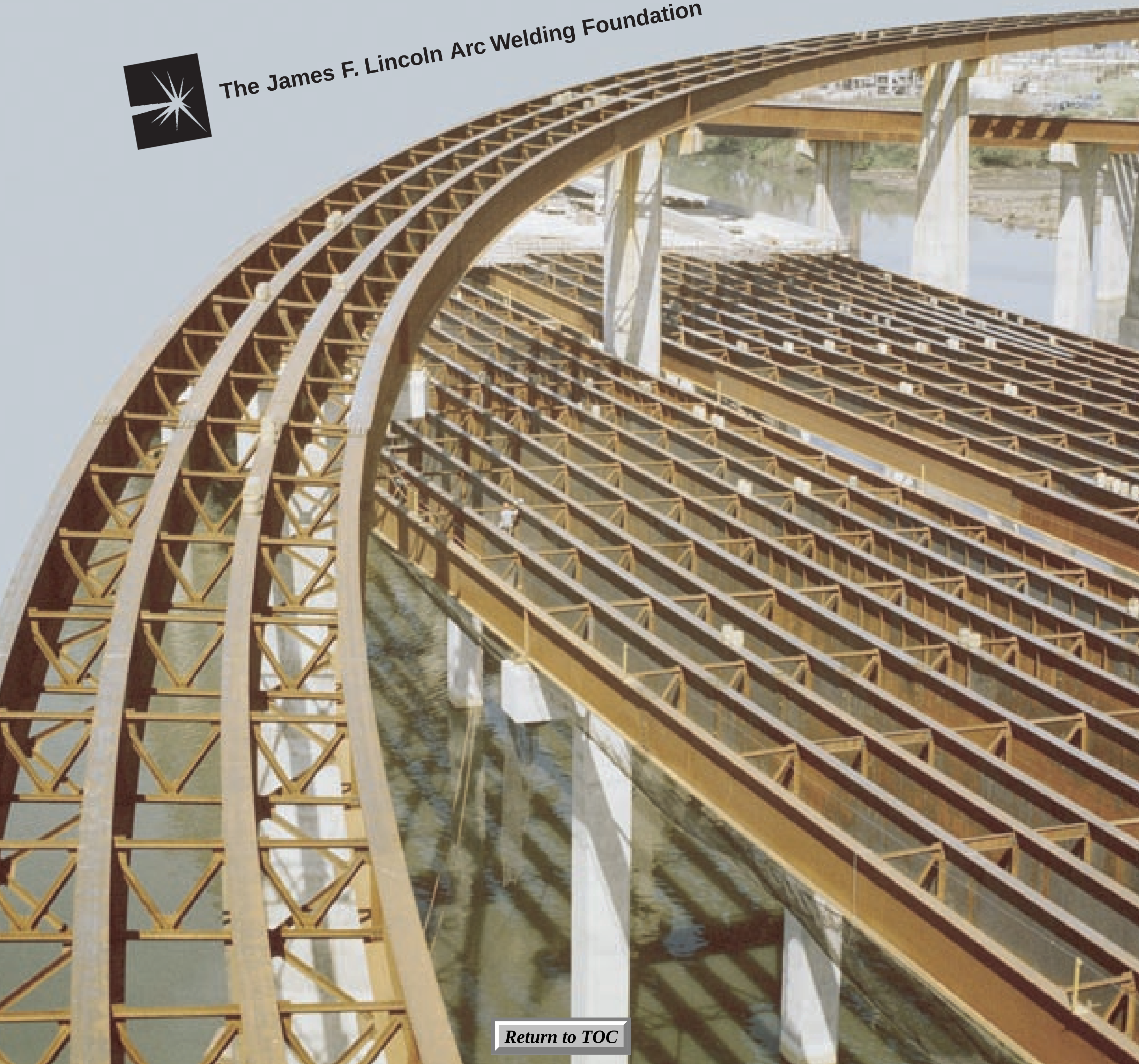

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All Entries should describe innovative ideas in welded design, engineering, research, fabrication, welding procedures or welding processes that reduced costs or achieved other specific objectives. Subject matter may relate to:

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- Any aspect of arc welding: equipment, procedures, fixturing, process, research, testing, inspection, etc.

Each Entry must include, at a minimum, a 1,000 word written description of the work, suitably illustrated. The Entry does ***not*** have to be written by the person or persons submitting it. It may be written by a public relations department, an advertising agency, or any other writer.

The Entry may describe an entire project, or only a component or part related to the use of welding. The written paper should clearly describe the problems or objectives of the work, the approaches considered for solution, a rationalization of the approach selected, a step-by-step explanation of how the results were achieved, how welding contributed to the project, and the significance of the results. Entries to the 1999 Program must describe work completed after January 1, 1997.

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One person or a group of no more than three individuals working as a team in the areas of design, engineering, architecture, research, fabricating, production or maintenance, may submit an Entry. There is no limit to the number of Entries that may be submitted by any one individual or group, provided that each Entry is on a different subject.

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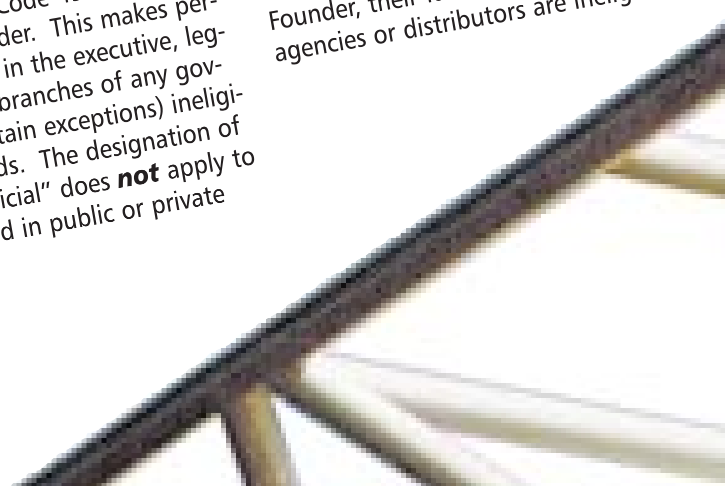

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The Jury of Awards will be selected from various branches of engineering, education, business, industry, or from any other suitable source by Chairman of the Jury Donald N. Zwiep, Department Chairman Emeritus, Worcester Polytechnic Institute, or in the event of his failure to act for any reason, by a chairman appointed by the Board of Trustees of the Foundation. The Jury will receive only an unidentified original copy of each Entry, and all Entries will be judged anonymously. If a 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. The Jury of Awards will use the following criteria in judging the Entries:

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- **Practicality**
- **Results achieved or expected**
- **Contribution of arc welding**
- **Conciseness and clarity of the Entry**
- **Professionalism of the presentation, including the quality of photos and illustrations**
- **Conformity of the Entry to these rules**

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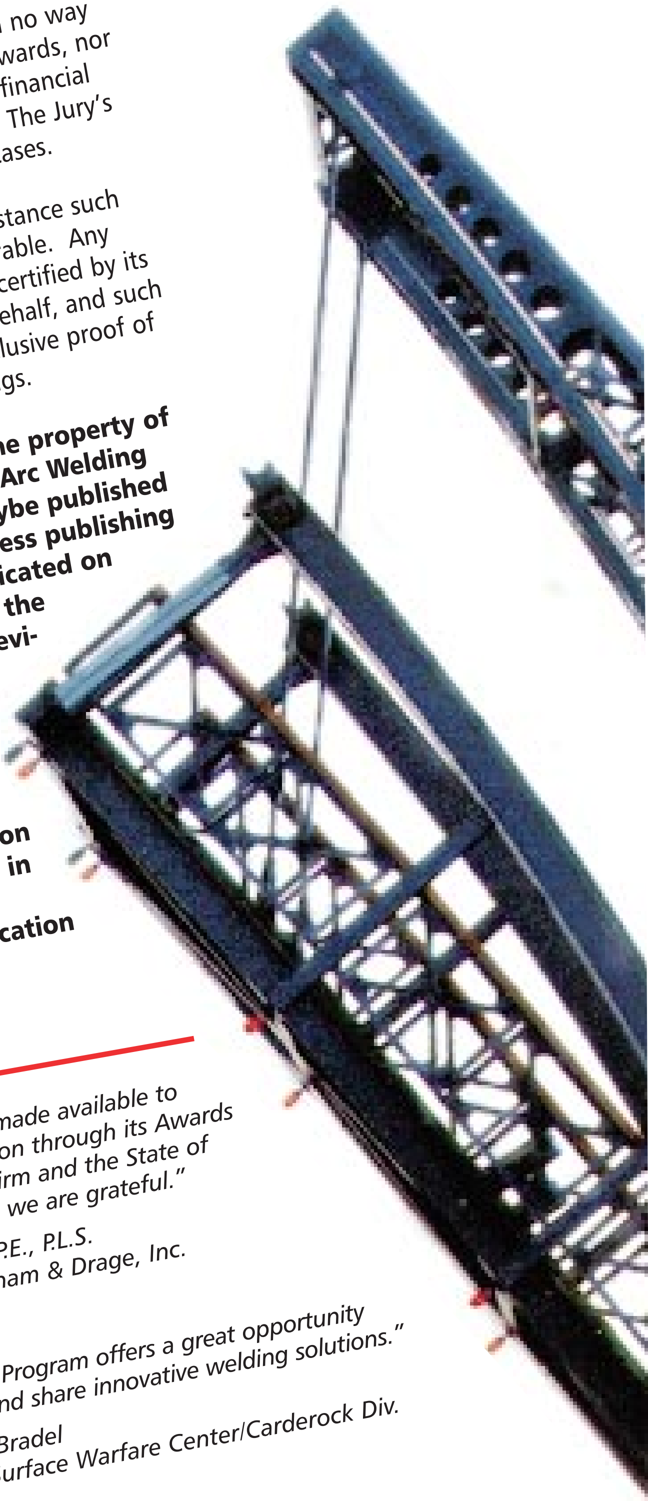

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C. Original drawings or copies in the form of prints or photostats must be clearly readable. Glossy print photographs are preferred.

D. The Entry must be bound in a suitable cover, and be prefaced with a 1-page abstract of the paper.

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H. Complete the Entry Form and place it inside the front cover of your binder. It will be removed before your Entry is sent to the judges.

1. Please submit a current photograph of each author, with the name written on the back, with the Entry.

Send questions or comments to: innovate@lincolnelectric.com

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Bell Street Pier Wave Barrier

Prefabricated Welded Steel Key to Success

By Roy Peratrovich, Jr., Principal
Jonathon B. Keiser, Senior Engineer
Jeff Gilman, Vice President
Peratrovich, Nottingham & Drage, Inc.
Seattle, Washington

Figure 1. Aerial view of Seattle's Bell Street Pier.

Introduction

Before the construction of the Bell Street Pier, there were no modern tourist-oriented waterfront facilities near the heart of downtown Seattle. Only old, generally dilapidated timber docks and abandoned buildings lined the main waterfront street, Alaskan Way. The Port of Seattle's new $67 million Gateway to the Pacific Rim (see Figure 1) features an international convention center, maritime museum, restaurants, a fish processing and retail center, numerous outdoor tourist amenities, a dual purpose commercial and cruise ship dock, and a full-service transient marina.

The marine portion of the project includes 900 ft (275 m) of a one-of-a-kind permeable wave barrier to protect the transient marina, and roughly 3 acres (12,000 m^2) of concrete dock structures with complete utilities and heavy marine fenders. The marine structures used 11 miles (17,700 m) of heavy welded steel pipe piles with 374 high capacity "spin-fin" pile tips, and over 7 miles (11,265 m) of prestressed concrete piles. The wave barrier alone used 4 million lbs (8.8 million kg) of welded structural steel, including the steel pipe piles.

The Challenge

The designer's mission was to provide a first-class transient marina to attract visiting seafarers, along with a multi-purpose dock to accommodate both cruise ships and large fishing trollers, all within restrictive physical parameters. Seattle has tidal extremes of 18 ft (5.5 m). Where the breakwater is located, water depth can reach 70 ft (21 m) at high tide. It was determined that a typical rock breakwater would cost more than $20,000 per foot ($6,100 per meter), and would virtually cover the harbor. A very special structure was required to handle large wave forces while taking up as little space as possible. The wave barrier could not extend to the ocean floor for environmental reasons, since an area had to be left open to allow daily tidal flushing action to the marina, as well as to provide a safe passageway for fish. At the same time, the wave barrier would have to protect the marina from storm waves at all tide levels. A wave barrier of the size and type required to protect the marina had never been designed or constructed; this became the primary design challenge of the project.

Wave Modeling Tests

Tests were performed, including three-dimensional and two-dimensional physical scale wave modeling, and a three-dimensional tidal flushing circulation study, from which the minimum length and geometric shape of the wave barrier were determined. The tests also determined the wave barrier's optimum location, corresponding design wave forces, and design wave pressure diagrams. The tests showed that wave forces would be equal in magnitude for both the incoming and outgoing waves. This was a major discovery, as in the past, designs only accounted for incoming waves. The 8 ft (2.4 m) design wave (5.5 second periods) was estimated to cause 8 tons (8.13 m tons) of force per horizontal foot (0.30 m) of wall. This horizontal design load was later found to be 10 times larger than the weight of the wave barrier itself.

Return to TOC

Figure 2. View of the wave barrier before the dock was installed.

The Solution

Because of the heavy, complex design wave loads, the only practical structural solution required using an all-welded steel structure. Welded steel's high strength capacity, versatility, ability to be prefabricated, and flexibility in construction were essential attributes.

Numerous structural systems were considered for the wave barrier before the final solution evolved. In order to prevent pile soil plugs from being in danger of overlapping one another, determining the proper location for the pile bents and barrier piles was critical. The wider the bent spacing, the larger the pile loads would become. This would also mean increased loads on the barrier piles, pile caps, and cap beams. The combination of high pile loads and heavy bending loads from the barrier panels led to the independent barrier pile and pile bent system. This allowed the piles in the pile bents to be designed for primarily axial loads, while the barrier piles could be designed primarily for the large moments caused by the heavy concrete wave barrier panels.

Figure 3. This close-up of the wave barrier under construction shows the concrete panels. Note the perpendicular extension of the heavy steel cap beams to the barrier piles.

The system ultimately selected (see Figure 2) uses double 24 in (600 mm) round welded steel battered pipe pile bents, spaced at 12 ft (3.6 m) centers, to brace the top of vertical 48 in (1,219 mm) round welded steel barrier piles. Each pile bent is field welded to heavy prefabricated, welded steel pile caps. Prefabricated steel box beams, formed from double-wide flange beams that run the full length of the wave barrier, were also welded in the field to the tops of the pile caps. Two heavily reinforced prestressed concrete panels (see Figure 3), each 8 in (200 mm) thick and about 8 ft (2.4 m) wide, form a 48 ft (14.6 m) deep wave barrier panel that is suspended between the barrier piles above the ocean floor. The panels are designed to slide down between reinforced vertical steel guide plates that are welded to the wave barrier piles. Panels are supported by a prefabricated welded steel bracket seat attached to the bottom of the same guide plates. The barrier piles are also spaced on 12 ft centers between the double pile bents. The wave barrier panels are located to clear the dock for ease of installation and future maintenance.

Solid prestressed concrete deck panels, with one end resting on top of the longitudinal steel cap beams, form the dock structure located above the wave barrier system. The shore side of these deck panels rests on another longitudinal steel cap beam. Prefabricated heavy marine fenders are mounted in front of the dock on the same welded steel pile cap extensions that are connected to the barrier piles. All welded connections were designed for simplicity, and all prefabricated piles were inspected at the fabrication plant by the designer.

A Special Technology

Steel fins, welded to pipe pile tips at a slight angle, impact a screwing motion to the piles when they are driven into the ground; hence they are called "spin-fin" piles (see Figure 4). The strength of the spin-fin can be compared to the relative difference of the ultimate "pull-out" capacity of the wood screw to the nail. The spin-fin piles were essential to the economic success of the wave barrier system. Piles with spin-fin tips were found to be more than five times stronger than piles without spin-fin tips. Without the spin-fins, either more piles would have been required, and/or they would have had to be larger.

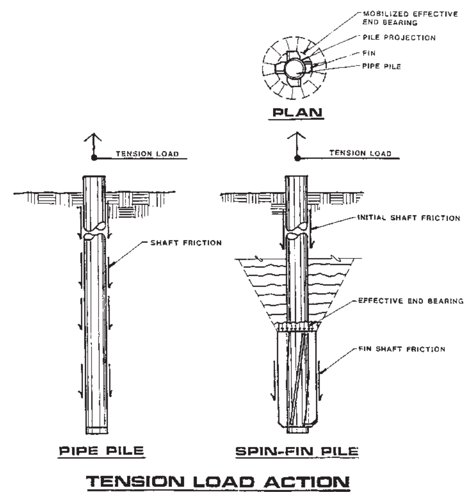

Figure 4. These drawings show how the "spin-fin" pile works. Its ultimate load capacity vs. a slick pile can be compared to the difference between the pullout strength of a wood screw and that of a nail.

Conclusion

Thanks to a successful joint partnering effort among the client, the contractor, and the designers, the entire project was completed on time and under budget. Prefabricated welded steel design was key to the success of the wave barrier. Since its completion, the Bell Street Pier has made a major contribution to the revitalization of Seattle's waterfront.

Key Concepts in Welding Engineering

by R. Scott Funderburk

Postweld Heat Treatment

What is PWHT?

Postweld heat treatment (PWHT), defined as any heat treatment after welding, is often used to improve the properties of a weldment. In concept, PWHT can encompass many different potential treatments; however, in steel fabrication, the two most common procedures used are **post heating** and **stress relieving**.

When is it Required?

The need for PWHT is driven by code and application requirements, as well as the service environment. In general, when PWHT is required, the goal is to increase the resistance to brittle fracture and relaxing residual stresses. Other desired results from PWHT may include hardness reduction, and material strength enhancements.

Post Heating

Post heating is used to minimize the potential for hydrogen induced cracking (HIC). For HIC to occur, the following variables must be present (see Figure 1): a sensitive microstructure, a sufficient level of hydrogen, or a high level of stress (e.g., as a result of highly constrained connections). In ferritic steels, hydrogen embrittlement only occurs at temperatures close to the ambient temperature. Therefore, it is possible to avoid cracking in a susceptible microstructure by diffusing hydrogen from the welded area before it cools. After welding has been completed, the steel must not be allowed to cool to room temperature; instead, it should be immediately heated from the interpass temperature to the post heat temperature and held at this temperature for some minimum amount of time. Although various code and service requirements can dictate a variety of temperatures and hold times, 450°F (230°C) is a common post heating temperature to be maintained for 1 hour per inch (25 mm) of thickness.

The need for post heating assumes a potential hydrogen cracking problem exists...

Post heating is not necessary for most applications. The need for post heating assumes a potential hydrogen cracking problem exists due to a sensitive base metal microstructure, high levels of hydrogen, and/or high stresses. Post heating, however, may be a code requirement. For example, ASME Section III and the National Board Inspection Code (NBIC) both have such provisions. The Section III requirement for P-No. 1 materials is 450 to 550°F (230 to 290°C) for a minimum of 2 hours, while the NBIC requirement is 500 to 550°F (260 to 290°C) for a minimum of 2 hours. Furthermore, post heating is often required for critical repairs, such as those defined under the Fracture Control Plan (FCP) for Nonredundant Members of the AASHTO/AWS D1.5 Bridge Welding Code. The FCP provision is 450 to 600°F (230 to 315°C) for "not less than one hour for each inch (25 mm) of weld thickness, or two hours, whichever is less." When it is essential that nothing go wrong, post heating can be used as insurance against hydrogen cracking. However, when the causes of hydrogen cracking are not present, post heating is not necessary, and unjustifiable costs may result if it is done.

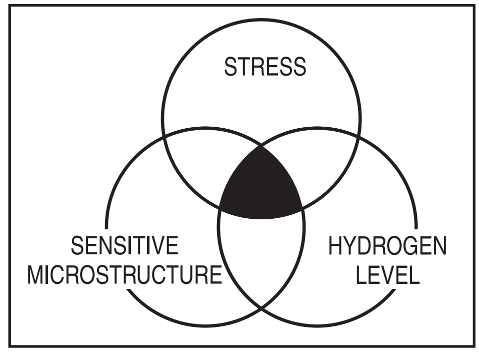

Figure 1. Criteria for hydrogen induced cracking (HIC).

Stress Relieving

Stress relief heat treatment is used to reduce the stresses that remain locked in a structure as a consequence of manufacturing processes. There are many sources of residual stresses, and those due to welding are of a magnitude roughly equal to the yield strength of the base material. Uniformly heating a structure to a sufficiently high temperature, but below the lower transformation temperature range, and then uniformly cooling it, can relax these

Return to TOC

residual stresses. Carbon steels are typically held at 1,100 to 1,250°F (600 to 675°C) for 1 hour per inch (25 mm) of thickness.

Stress relieving offers several benefits. For example, when a component with high residual stresses is machined, the material tends to move during the metal removal operation as the stresses are redistributed. After stress relieving, however, greater dimensional stability is maintained during machining, providing for increased dimensional reliability.

In addition, the potential for stress corrosion cracking is reduced, and the metallurgical structure can be improved through stress relieving. The steel becomes softer and more ductile through the precipitation of iron carbide at temperatures associated with stress relieving.

Finally, the chances for hydrogen induced cracking (HIC) are reduced, although this benefit should not be the only reason for stress relieving. At the elevated temperatures associated with stress relieving, hydrogen often will migrate from the weld metal and the heat affected zone. However, as discussed previously, HIC can be minimized by heating at temperatures lower than stress relieving temperatures, resulting in lower PWHT costs.

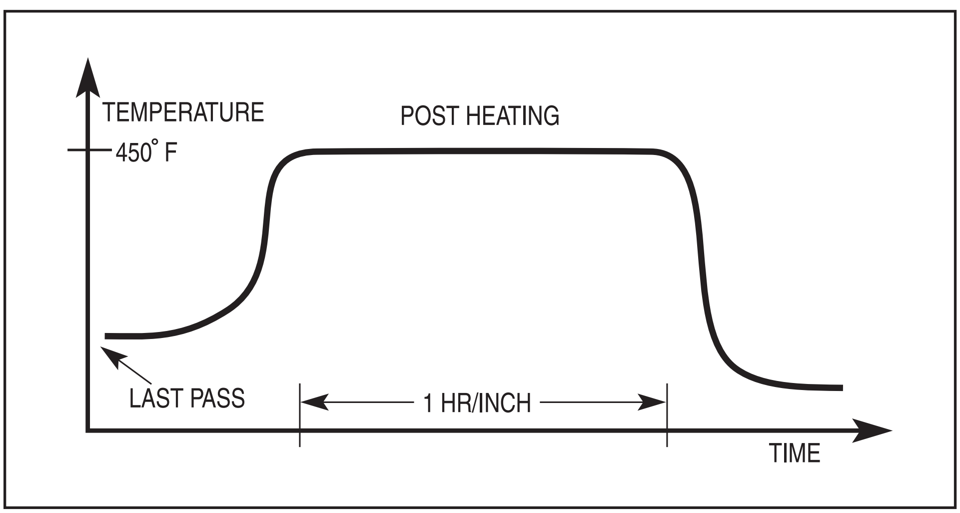

Figure 2. Post heat applied immediately after last pass.

Other Considerations

When determining whether or not to postweld heat treat, the alloying system and previous heat treatment of the base metal must be considered. The properties of quenched and tempered alloy steels, for instance, can be adversely affected by PWHT if the temperature exceeds the tempering temperature of the base metal. Stress relief cracking, where the component fractures during the heating process, can also occur. In contrast, there are some materials that almost always require PWHT. For example, chrome-molybdenum steels usually need stress relieving in the 1,250 to 1,300°F (675 to 700°C) temperature range. Thus, the specific application and steel must be considered when determining the need, the temperature and time of treatment if applied, and other details regarding PWHT.

When determining whether or not to PWHT, the alloying system and previous heat treatment of the base metal must be considered

The filler metal composition is also important. After heat treatment, the properties of the deposited weld can be considerably different than the "as welded" properties. For example, an E7018 deposit may have a tensile strength of 75 ksi (500 MPa) in the "as welded" condition. However, after stress relieving, it may have a tensile strength of only 65 ksi (450 MPa). Therefore, the stress relieved properties of the weld metal, as well as the base metal, should be evaluated. Electrodes containing chromium and molybdenum, such as E8018-B2 and E9018-B3, are classified according to the AWS A5.5 filler metal specification in the stress relieved condition. The E8018-B2 classification, for example, has a required tensile strength of 80 ksi (550 MPa) minimum after stress relieving at 1,275°F (690°C) for 1 hour. In the "as welded" condition, however, the tensile strength may be as high as 120 ksi (825 MPa).

The objective of this article is to introduce the fundamentals of postweld heat treatment; it is not meant to be used as a design or fabrication guide. For specific recommendations, consult the filler metal manufacturer and/or the steel producer.

For Further Reading

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A CITY EMBRACES ITS SWAN

Striking Bridge Design Provides Structural Challenges

By Vincent van der Mee
The Lincoln Electric Company
Cleveland, Ohio
Leo van Nassau
Lincoln Smitweld
Nijmegen, the Netherlands

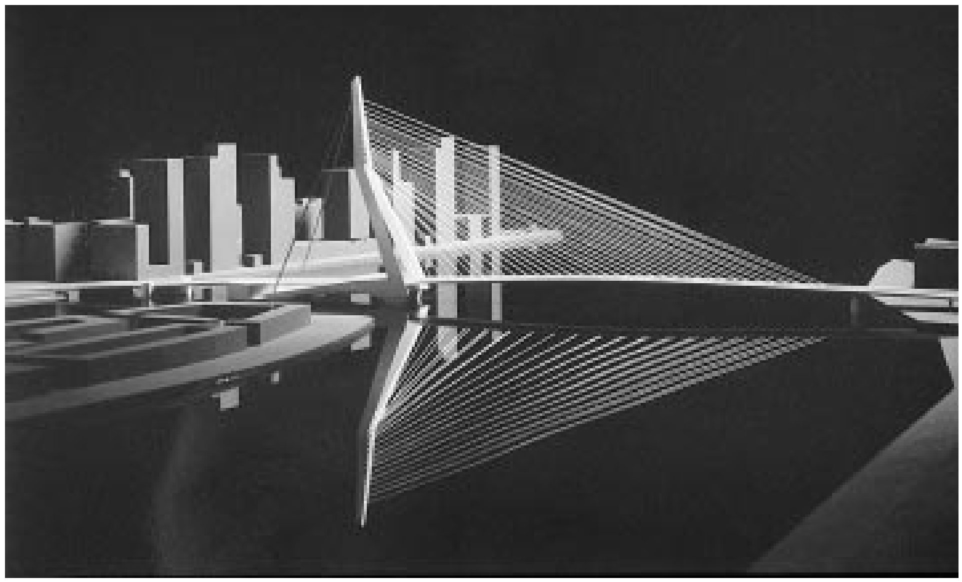


Photo: Jan Derwig

Figure 1. Model of the "Swan."

The Erasmus Bridge in Rotterdam, the Netherlands, open only since late 1996, already has met the highest objective of public art: it has been embraced by the citizenry as the symbol of a vibrant, future-oriented metropolis. The dramatic welded, cable-stayed structure is universally and fondly referred to simply as "the Swan."

Project Rationale

Rotterdam city planners had known for well over a decade that development of the south bank of the Nieuwe Maas river would require the construction of an above-ground link with the inner city. Unveiled in 1987, a comprehensive plan for the south bank area known as the Kop van Zuid called for the construction of 5,300 residences, 400,000 m^2 (4.3 million ft^2) of offices, 35,000 m^2 (0.4 million ft^2) of industrial and shop space, and 60,000 m^2 (0.6 million ft^2) of recreational facilities on a site measuring 125 hectares (300 acres). Two designs were seriously considered for the cable stayed Erasmus Bridge: one approach featured four pylons or towers, while the other was distinguished by a single, dramatic "twisted" (backward-sloping) pylon.

Choice of Design

In 1991, the Rotterdam city council selected the single pylon design conceived by architect Ben Van Berkel (see Figure 1). In his development of what he called "the Swan," Van Berkel committed himself to the following:

- Spreading the stays asymmetrically.
- Giving expression to the play of forces, in which the pylon acts as a counterweight.
- Emphasizing the fact that the base of the pylon is more than a mere vertical element, since it stretches along a horizontal plan to the bascule column.

Van Berkel meant for "the Swan" to serve as a point of reference for the city. Architecturally, this was expressed as follows:

- The height of the pylon points to the high-rise buildings in the area.
- The cable suspension structure permits a less obstructed view of the water from the quays than is possible with other types of bridges.
- The necessary asymmetry reinforces the urban orientation, since the northern and southern ends of the pylon are so different in shape; as a result, each end of the bridge affords a unique visual perspective.
- The shape of the bridge symbolizes the union of Rotterdam South with Rotterdam North.

Project Overview

Measuring 33 m (109 ft) in width, totaling over 800 m (2,630 ft) in length, and weighing 33 million kg (15 million lbs), "the Swan" carries an estimated 26,000 cars per day. The largest span of the structure (410 m [1,350 ft]) is executed in the shape of a modern cable bridge. The roadway was produced from 28 sections, each 15 m (50 ft) long and 33 m (109 ft) wide, fabricated of S355K2G3 steel.

The overall design accommodates shipping traffic while providing a roadway with a limited slope to facilitate vehicular and pedestrian traffic flow across the bridge. The headroom for ships is 12.5 m (41 ft). The bascule part of the bridge has a width of passage totaling 50 m (164 ft). “The Swan” has special lanes for cars, bicycles, and pedestrians, with the middle lane reserved for buses and streetcars.

The slim roadway, comprised of 28 sections weighing 825,000 kg (375,000 lbs) each, is suspended with 32 cables on a pylon 139 m (456 ft) high. At a height of 110 m (361 ft) above the roadway, a maximum force of 98 million N (22 million lbf) is transferred to 8 heavy back ropes, each

...each end of the bridge affords a unique visual perspective

made from 127 twines of steel and having a diameter of 280 mm (11 in). The kink in the pylon, a specific detail by the architect, was especially designed to resist the enormous forces resulting from the high bend load.

The fine welded box construction of the pylon and the rear end, each with many internal stiffener boxes, is fabricated of high strength S460ML steel (which is a thermo-mechanically treated steel with high yield point)with wall thicknesses ranging from 15 mm (0.6 in) to 70 mm (2.8 in). The moveable part of the bridge is designed as a bascule bridge in which the steel ramp is balanced with a counterweight totaling 5.3 million kg (2.4 million lbs). The turning axis of the bascule bridge is diagonal with the longitudinal axis, as shipping traffic and road traffic cross

Welding Procedure Approval Record

WPAR Kr-180.01
Rev.: 1
Ref. WPQ: 95.074

Procedure Specification

Base material	BS 7191 Grade 420 EMZ (TMCP)
Welding processes	A: SMAW B:
Manual or machine	Manual
Welding position	PA (1G)
Filler metal (trade)	1: E 8016-G 2: Kryo 1 - 180
Flux	N.A.
Filler metal classific.	2: AWS A5.5: E 8018 - G EN 499: E 46 4 1Ni B 73 H5
Shielding gas [l/min]	N.A. Flow
Backing (gas) [l/min]	N.A. Flow
Gouge method	N.A.
Current / polarity	Root DC - ; Fill & Cap DC +
Preheat temp. [°C]	min. 25 °C
Interpass temp. [°C]	max. 220 °C Lot No.:
Postheat treatment	N.A. ø 3.2 : 513308
Welder's name	- ø 4.0 : 514164
	ø 5.0 : 515193
Laboratory Test No.	-
Remarks :	Welding and testing in accordance with EEMUA 158 and EN 288-3

Test Results

MT and ultrasonic Examination: Acceptable

Reduced-section tension test

Tensile strength [MPa]		Fracture location
25.0 × 25.2	585	Base metal
25.0 × 28.0	593	Base metal

All-weld-metal tension test ø 11.2 mm

Yield point Rp0.5 [MPa] :	528
Tensile strength [MPa] :	628
Elongation, A5 [%] :	24,5
Reduction, Z [%] :	66,3

Side-bend tests Former diameter: **4 × t**

30 × 10 mm 4 × **180°** Good

Impact tests ISO - V [Joule] Test temp [°C]: **- 40**
Size of specimen: 10 × 10 × 55 mm

	Cap			av.	Root			av.
Clw	45	49	46	47	79	67	51	66
Fl	264	269	282	272	115	128	172	138
Fl + 2	279	284	281	281	275	278	281	278
Fl + 5	282	280	278	280	283	280	277	280

Welding Procedure

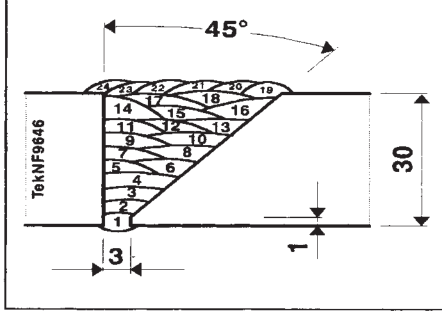
Pass No.	Consumable index	Ø [mm]	Welding Current Ampere	Volts	Speed [mm/min]	H.I. [kJ/mm]
1	A1	2.5	75	21	53	1.8
2	A2	3.2	145	32	235	1.2
3	A2	4.0	185	35	270	1.4
4	A2	5.0	225	38	292	1.8
5	A2	3.2	145	32	265	1.0
6 + 7	A2	4.0	200-230	37-38	273-285	1.6 - 1.8
8 - 16	A2	5.0	230	38	229-275	1.9 - 2.2
17	A2	3.2	145	32	258	1.0
18	A2	4.0	200	37	279	1.6
19-24	A2	3.2	145	32	227-240	1.1 - 1.2

Macro examination

Free from significant inclusions and other defects.

Ferrite Content (FN) see sketch

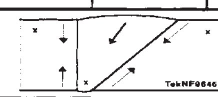
Joint Detail



Hardness see sketch
Test type : Vickers Load : 10 kg

	BM	HAZ	WM	HAZ	BM
Cap	167	186-183-181-181		170-179-175-177	171
Mid			187-191-185		
Root		190-192-194-192	213	195-187-184-182	

Sketch



We hereby, certify that the statements in this record are correct.

Project	
Manufacturer or Contractor	Heerema Havenbedrijf BV
Authorized by	Lloyd's Register
Issued by	Fred Neessen
Date	15 September 1996

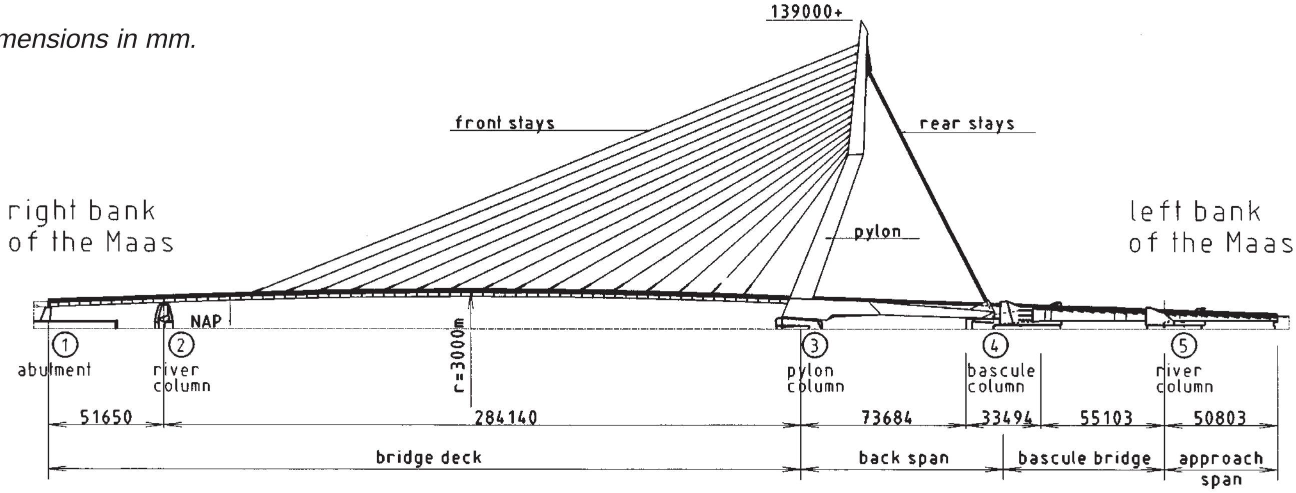
Figure 2. Welding procedure approval record.

diagonally. The box girders (with heights ranging from 4.5 to 7.9 m [15 to 26 ft], and widths ranging from 2.4 to 3 m [8 to 10 ft]) have the important function of absorbing the large torque and bending load with a minimum of distortion. The electro-hydraulic movement of the bascule bridge is very impressive, with only 2 minutes required for opening or closing at a maximum wind velocity of 47 km/hr (30 mph).

All welded connections are butt joints or fillet welds, using the SMAW, GMAW, FCAW, or SAW processes. An example welding procedure approval record is shown in Figure 2. **Lincoln Smitweld** products were used for most of the work. The Kyro 1-180 (AWS A5.5:E8018G*), a 180% recovery, extra low diffusible hydrogen content electrode with 1%Ni was used for the fillet welds in the pylon. For the roadway, the flux/wire combination of choice was Lincoln P230 flux with Lincoln LNS140A(S2Mo) filler metal (AWS A5.17/A5.23:F8A4/F8P5 EA2*). Fillet welds of the stiffener troughs in the cable bridge roadway were made using Lincoln’s LNM25 (AWS A5.18:ER70S-3) GMAW electrode with CO_2 shielding gas. Large quantities of Outershield MC 710-H (AWS A5.18:E70C-6MH4) metal cored electrode were used to fabricate the bascule bridge.

Return to TOC

All dimensions in mm.



Gemeentewerken, Rotterdam

Figure 3. Survey of the Erasmus Bridge, main dimensions and major components.

The Cable Suspension Bridge

Structural Design

The structural detailing of Van Berkel's design posed many technical challenges (Figure 3). There were virtually no right angles, and most surface areas had sloping connections. It was not possible to produce detailed structural drawings manually. From the outset, the engineers used a three-dimensional drawing package that provided automatic checks as to whether the planned corners of a given surface actually lay in a single surface.

The detailing of the rear stay anchoring in the pylon and the back span presented a particular problem. The rear stays had to support 45,000 kN (10 million lbf) each, not including the load factor. Not only were they sloping, but they also had to form a slanted connection to the exterior plating. A Plexiglas scale model was created, and the ultimate solution came after much cutting and pasting of the anchoring plates. The contractor worked out the pylon using a three-dimensional drawing program which also controlled the cutting machines directly. Because of the twisted shape of the pylon, tremendous moments are created at the location of the twist, making the use of heavy plates unavoidable. To reduce the weight of the pylon as much as possible, the plate thickness was kept to 50 mm (2 in) with the use of thermo-mechanically reinforced steel with a high yield point.

Photo: Ben Seelt, Leidschendam

Figure 4. A section of the pylon being fabricated using Lincoln Smitweld products.

The Pylon

While the pylon's dimensions originally were determined by visual relationships, the dimensions of the plates and profiles had to be modified somewhat for structural reasons. The pylon was fabricated of S460ML steel, fully welded to reduce weight (Figure 4). Internal reinforcement was provided by horizontal partitions which also serve as floors. Trapezoidal-shaped flexible reinforcements, the heaviest of which had a wall thickness of 20 mm (3/4 in), were placed longitudinally.

The partition dimensions were modified to the thickness of the plates to be reinforced, which varied from 20 mm (3/4 in) in low-load sections to 50 mm (2 in) in the highest load sections near the twist. In the location of the rear stay anchoring, plates with a thickness of up to 100 mm (4 in) were used. Two parallel partitions were inserted in the pylon to support the front stay anchor blocks, and the rear stays were anchored on three parallel partitions.

Internal stairs, ladders and platforms were built in both bases of the pylon. Along with an elevator installed in the western base, these installations guarantee good access to permit proper maintenance of the stay anchoring and the top of the pylon.

The Stays

Fan-shaped front stays support the river bridge span, while two stays connect the top of the pylon to the anchoring at the rear side of the bridge. The back span's enormous height, an essential element of the design, made it unnecessary to support the back span with stays.

A stay complex consists of strands, with each strand made up of seven thermally chromed threads. Each strand is covered with an extruded polyethylene jacket, and the space between the threads and the jacket is filled with grease. The strands are fixed in anchor blocks resting against partitions, welded into the structure. The stays are tightened by adjusting each strand separately with a jack. Should it be necessary, each strand can be individually replaced, meaning that the entire stay complex would not have to be removed in order to install a new strand.

The Bridge Deck and Back Span

The bridge deck (Figure 5) is supported by two box girders, each 2.25 m (7.2 ft) high and 1.25 m (4 ft) wide, located 20 m (66 ft) apart. Cross beams run between the main girders,

Fan-shaped front stays support the river bridge span...

4.9 m (16 ft) center to center. These are not as high as the main girders, allowing space for an inspection cart rail. As an extension of the cross beams on the outside of the box girders, consoles support cycle and pedestrian paths.

The 18 mm (0.7 in) thick orthotropic steel (S355K2G3) bridge deck is reinforced with trapezoidal stiffeners measuring 600 mm (24 in) center to center. The fully welded deck has an 8 mm (0.3 in) thick synthetic resin wear layer, providing considerable savings on the structure's own weight, compared to an asphalt mastic layer.

Supports

The bridge rests on four columns, with horizontal and vertical responses absorbed by separate supports: the vertical by rubber packages, and the horizontal by a statically determined system of a hinge on one (the bascule) column, and a roll on another column.

The bascule column has three functions:

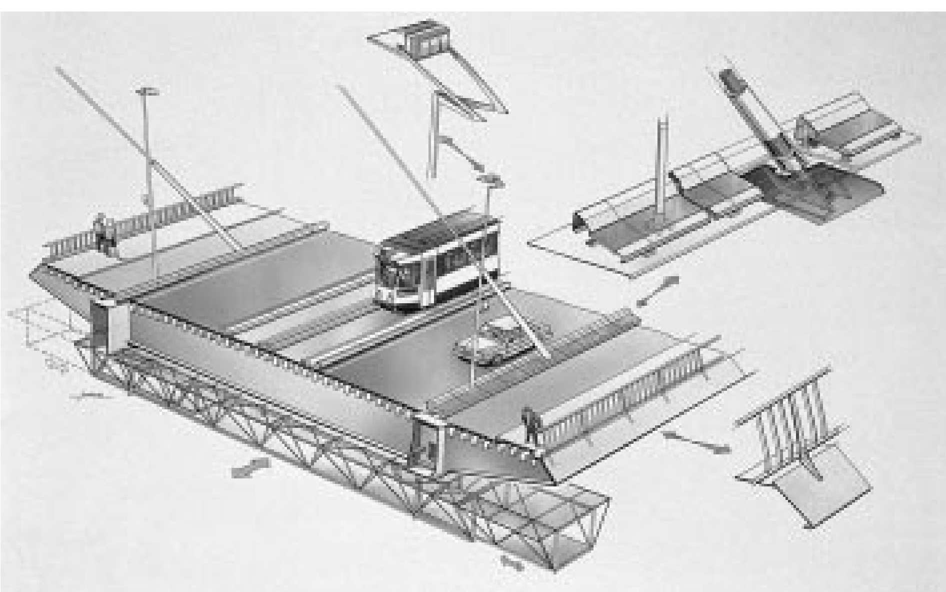
- anchoring of the cable bridge;
- providing the pivotal point of the bascule bridge; and
- housing the bascule cellar.

The pylon's total load is 80,000 kN (17 million lbf). Four 1 x 2 m (3.25 x 6.5 ft) rubber pads at each base provide for the transfer of this load. The base of the pylon has extra jacks which make it possible to jack up the pylon to replace supports or compensate for unexpected subsidence. Large horizontal displacement takes place in the other two columns, in a longitudinal direction from the bridge. Due to limited space, a combination of rubber and Teflon was used for the supports.

Fabrication

Because of the complex forms, all plates had to be cut in the proper shape within small tolerances. The same three-dimensional drawing program was used to create the drawings and to guide the cutting equipment.

The sections of the longitudinal deck were fabricated from bottom to top. First, the deck was put together from plates. The trapezoidal stiffeners were placed on top and welded onto the deck before the cross girders were added. After the trapezoidal stiffeners were welded to the cross beam, the section was turned and the welds were placed on the top side of the deck plate.



Drawing: Cas Schook Mijnsheerenland

Figure 5. Perspective with diagonal cross-section of the bridge deck.

The tube-shaped main girders were built as complete, 15 m (49 ft) long units. The box-shaped, slanting sections which comprised the pylon were put together in a horizontal position.

Assembly
The sections of the back spans were assembled into complete main girders at Vlissingen, and the longitudinal deck was then placed between them. Struts and locators for placing the pylon were added, and the complete back span was then driven onto a pontoon.

The pylon's base sections were built first, then welded to the cross piece where the sections of the base came together. The pylon was finished in a horizontal position and then driven onto a pontoon. The two pontoons were towed by sea from Vlissingen to the Caland channel, where an offshore crane vessel lifted the pylon onto the back span (Figure 6).

The bridge deck sections were prefabricated in 15 m (49 ft) lengths. Both main girders and the longitudinal deck were placed first, then adjusted and welded together prior to the addition of the consoles.

The entire bridge span between the bascule and pylon columns was put into place without any temporary support points. The pontoon navigated between the columns, and put the pylon with back span in place before it was lowered into position. After exact positioning, the supports were cast and the pull anchoring installed.

The extension of the main bridge span was accomplished in cycles of two to three weeks per section. Each section was swung into place using a floating derrick, and affixed to the bridge with a temporary connection (frame with jack screw) on the top side of each main girder to absorb the pull force, and a contact connection (butt) between the bottom flange of the main girders to absorb the pressure force. Protruding beams under the existing main girder absorbed the diagonal force. Plates were then attached and welded, followed by the adjustment and welding of the trapezoidal stiffeners.

Simultaneously, preparations were made for placing two stays per section. The tubular casing, which was supplied in sections and welded on site, was lifted and the first strand was drawn through the tube and tightened.

The kink in the pylon was designed to resist the enormous forces resulting from the high bend load

One after another, the strands were drawn through the tube, with each strand tightened in two steps: first, to 70 percent, and after all the strands had been installed, to 100 percent.

When the pylon and the extended part of the bridge were almost in equilibrium, the temporary pylon supports on the back span were removed, and the backward sloping pylon began to serve in its capacity as a counterweight. Finally, the tubular casings and strands for the rear stays were installed.

The Bascule Bridge

The bascule bridge is the movable section of "the Swan" that permits the passage of ships taller than Rhine navigation height (Figure 7). It is one of the largest of its kind, with a deck measuring 52.3 by 35.8 m (172 by 117 ft), and an apron weighing 1,560 tonnes (3.5 million lbf). In an open position, the fall stands 19 m (62 ft) "out of plumb."

Structural Design
The deck plate, trough-shaped longitudinal girders, cross and main girders form a fully welded, orthotropic deck. The cross girders and consoles were fabricated in the form of girder plates. Girders were constructed from box profiles around the rotation point, where bending and torsion moments are greatest. Toward the front, the forces and required rigidity are less, and the cross section becomes a girder plate. To limit the diagonal eccentricity of the deck, the sideward twist was placed in front of the main rotation point wherever possible.

The weight of the bascule bridge was almost completely balanced by the counterweight, except for the front bearing pressure. The ballast stands eccentrically in a diagonal direction to compensate for the obliqueness of the bridge. This equally distributes both the weight responses in the main rotation points, and the bending moments caused by the bridge's own weight in both main girders.

Rigidity against torsion and bending is provided by the square formed by the two tail girders, the ballast girder and the rear cross girder. The torsion load is created not only by the obliqueness of the bridge, but also by the protrusion of the main rotation points, the position of the ballast, the three-cylinder operation, and temperature and mobile loads. Due to its own weight, the bridge had a 300 mm (12 in) distortion on the front side. The sideward overhang of 90 mm (3.5 in) equaled the difference in deflection between both front girders. The distortion caused by the bridge's own weight was completely corrected during fabrication using a building camber.

The Moving Action
The moving time is limited to 120 and 135 seconds, respectively, during opening and closing of the bridge. An electro-hydraulic moving action, similar

Return to TOC

to that used in many movable bridges in Rotterdam, was selected. It was designed as an open hydraulic system with a controlled pump adjustment and a nonconnecting rod. When open, the bridge is pressed into the buffers at creeping speed and hydraulically pretensioned to be held rigidly in the

The box girders absorb the large torque and bending load with a minimum of distortion

open position. A mechanical holding arrangement is employed to keep the bridge open for longer periods. When the bridge is being closed, the creeping speed on the front side is 6 cm/s (2.36 in/s), with hydraulic dampers used to reduce the speed to 2 cm/s (0.79 in/s) right before it closes completely.

The Approach Span

A steel concrete composite viaduct creates the link between the bascule bridge of "the Swan," and the left bank of the Maas.

Structural Design

In the fully welded steel section of the approach span, the cross girders all lie parallel to the bascule column, which joins the main girders at different angles. The design of the edge profiles coincides with that of the other sections of the bridge. The design of the main girders was modified somewhat to meet Van Berkel's design requirements, resulting in a sharply declining construction height for each main girder. The top side of the structure was raised 130 mm (5 in) at the pedestrian path to reduce the thickness of the concrete deck.

Steel Concrete Composite

In a steel concrete composite bridge, the concrete plate functions simultaneously as a supporting floor for mobile loads and as a pressure flange for the steel girders. To bring about the fullest possible marriage of steel and concrete, approximately 20,000 peg dowels were placed on the upper flanges of the steel girders, concentrated at the location of the supports. This will guarantee reliable performance, even in the event of strongly changing moment distribution.

Because of the large plate thickness requirements and the restricted effect of fatigue and instability, the entire bridge was fabricated in S460ML steel. This is a thermo-mechanically treated steel with a high elasticity tolerance.

Assembly of the Approach Span

The approach span was completely prefabricated (Figure 8). First, the steel frame was put together from the support elements, stabilized, and welded. Then, jack supports were placed under the middle of the main girders to jack up the frame to a pressure of 30 N/mm^2 (4,350 psi) in the bottom flanges. Approximately 85 percent of the frame weight rests on the middle supports. Two extra temporary supports were placed for each main girder and the sheet piling was positioned against the bottom of the top flanges.

A total of 400 m^3 (525 yd^3) of structural concrete was used in a single pouring. Afterwards, the temporary bottom supports remained in place for a month. After the interim supports were removed, the bridge was finished using additional concrete and wear layers.

Sources

Lincoln Smitweld
Public Works Dept. of the City of Rotterdam
Contractors:
Heerema Vlissingen
Grootint Dordrecht
Ravenstein Deest
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Back cover photo: Aeroview Rotterdam

Return to TOC

Photo: Aeroview Rotterdam

Figure 6. An offshore craneship puts the pylon on the back span.

Photo: Harm Bokje, Leidschendam

Figure 7. The bascule bridge in open position.

Photo: Harm Bokje, Leidschendam

Figure 8. The complete steel construction of the bridge during building.

Return to TOC

P.O. Box 17035
Cleveland, Ohio 44117-0035

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


Fondly referred to by city residents as “The Swan,” the Erasmus Bridge enhances both the skyline and traffic flow in Rotterdam, the Netherlands. The welded, cable-stayed structure was fabricated using Lincoln Smitweld products. Story on page 19.

Return to TOC