Volume XIV, Number 2, 1997

Welding Innovation

Advancing Arc Welding Design and Practice Worldwide

Technology Transfer—The Basics

Like many readers of this publication, I have enjoyed a rewarding career in the welding business. Over the many years of my association with welding, I seem to have picked up an amazing number of "tricks of the trade"—everyday principles that I tend to take for granted. Some of them may not be particularly sophisticated, or technically complex, but I am tempted to assume that since they are so familiar to me, these principles are common knowledge. Unfortunately, this is often far from the truth.

On a recent visit to Lincoln Electric, I commented to *Welding Innovation* editor Duane Miller that I had especially appreciated one of his recent Design File columns. I confessed to Duane that many of the principles outlined in Design File were beyond me, but this particular one I understood because it was one of those "basics" that I had learned many years ago. I was so pleased to see it in print that I copied the article and circulated it throughout the engineering department of our company. The particular column had identified one of the less sophisticated, less technically complex principles of welding engineering, but one that nonetheless must be communicated to a new generation of welding engineers, who need to absorb all of the fundamentals that their more experienced colleagues have accumulated over several decades of experience in this industry.

Which brings me to my point: the basics of technology transfer. In these days of CAD/CAM, finite element analysis, and such emerging technologies as neural networks, the next generation of engineers still must be able to grasp such fundamental concepts as the benefits of using fillet welds vs. groove welds, the fact that such manufacturing concerns as access for welding equipment have to be considered, and the reality that poor preparation and fitup can be barriers to making good welds!

Think about the wealth of knowledge and experience that your senior employees now hold. Are we, as managers, effectively facilitating the transfer of this precious technology to younger employees? It's a challenge for all of us, isn't it?

Herb Cable

As Duane and I talked about the importance of passing the basics of welding engineering along to the next generation, we came up with the idea of a new *Welding Innovation* column emphasizing fundamentals. Therefore, Duane has asked me to introduce this publication's newest department, Key Concepts, authored by assistant editor R. Scott Funderburk. The topic of the inaugural column is "preheat," hardly a recent high-tech development, but one of those essential "tricks of the trade" that must be understood by every welding engineer. Come to think of it, I think I'll be sending copies of this column around to my engineering department, too!

Herb Cable
Chairman
Weld Tooling Corp.
Pittsburgh, PA

Welding Innovation

Cover: An innovative repair procedure was used to restore the precision support track for the hydrodynamic test carriage at the U.S. Navy's Naval Surface Warfare Center. See story on page 2.

Omer W. Blodgett, P.E.
Design Consultant

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Precision Weld Repair of Worn Joints in a Towing Basin Track

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Introduction

The Naval Surface Warfare Center, Carderock Division (NSWC/CD), located in Bethesda, MD, is the Navy's principal research, development, test, and evaluation laboratory. As instrumented carriages travel along a one kilometer (5/8 mile) long precision railroad-type track at the Center's Model Towing Basin, researchers measure ship model hydrodynamic characteristics. After 50 years of service, the track had worn unevenly, with greater wear occurring at the joints between adjacent 10 m (33 ft) rails.

This track is much smoother and more nearly level than conventional railroad applications, yet the rail is very heavy, 90 kg/m (180 lb/yd), comparable with the heaviest rail rolled in the United States. When constructed, in 1939 and 1940, it was level within 0.1 mm (0.004 in) over a length of more than 0.3 km (1000 ft) (Ref. 1). To avoid distortion due to thermal expansion and to provide a smooth transition between rail sections, the rail ends were designed with a small gap, cut on a diagonal, ranging in width from 0.5 to 4 mm (0.02 to 0.15 in), as shown in Figure 1.

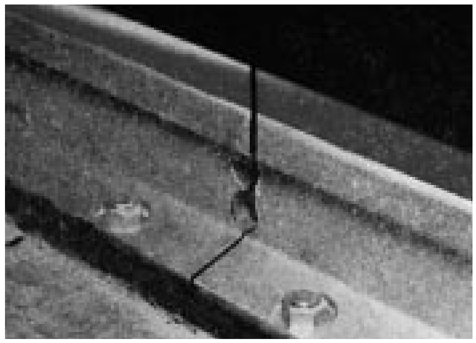

Figure 1. Gap between adjacent rail ends.

The precision of the track is even more impressive in the context of the massive size of the basin and the carriage. Figure 2 shows the basin and one of the carriages. A substantial carriage structure is required simply to span the 16 m (52 ft) width of the basin. In addition, the carriage must support large ship models, test equipment, operating personnel, and a drive system capable of rapidly accelerating to 13 m/s (40 ft/s). The most used carriage, shown in Figure 2, weighs approximately 57 t (125,000 lb). The weight of this carriage, combined with over 50 years of tests, caused wear of the track joints.

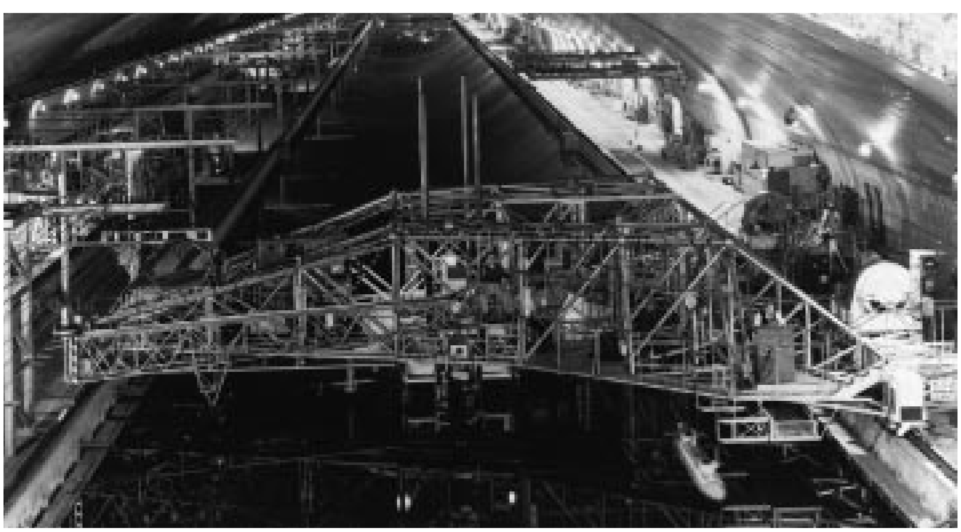

Figure 2. View of basin and carriage.

Precise measurements showed that the track surfaces near some of the joints had worn preferentially about 0.25 mm (0.01 in) (Ref. 2). The wear decreased with distance away from the joints. The worn region was approximately the shape of an inverted isosceles triangle, with a height near 0.25 mm and a base near 100 mm. Periodic impacts were imparted to the carriage as the wheels hit the worn regions at high speeds.

Not all joints had worn equally. To identify locations for the first repairs, the carriage was instrumented for measuring both the position and vertical-plane vibration. Vibration was measured at various carriage speeds, from about 1.5 m/s to the maximum carriage speed of about 13 m/s. The vibration record showed the severity of the wear at each joint.

Because of its special shape and precise installation, estimates to replace the track were in the range of $5 million to $20 million. Replacing the original track was not economically feasible and an alternative solution was pursued.

A repair procedure was developed to rebuild the rail ends with weld metal of the same wear resistance (hardness). Matching of the surface hardness was considered to be important in minimizing future localized wear, reducing the likelihood of subsequent repairs. Since the basin roof had been back fitted with insulation, stabilizing the temperature in the basin, the gaps between the track sections were thought to be no longer needed. Therefore, our goal was to develop a welding procedure to allow joining the track ends and to eliminate the expansion gap. After using this repair approach on the first few joints, we found that an expansion gap (with a diagonal kerf) was necessary to prevent stress cracking during cooling of the weld.

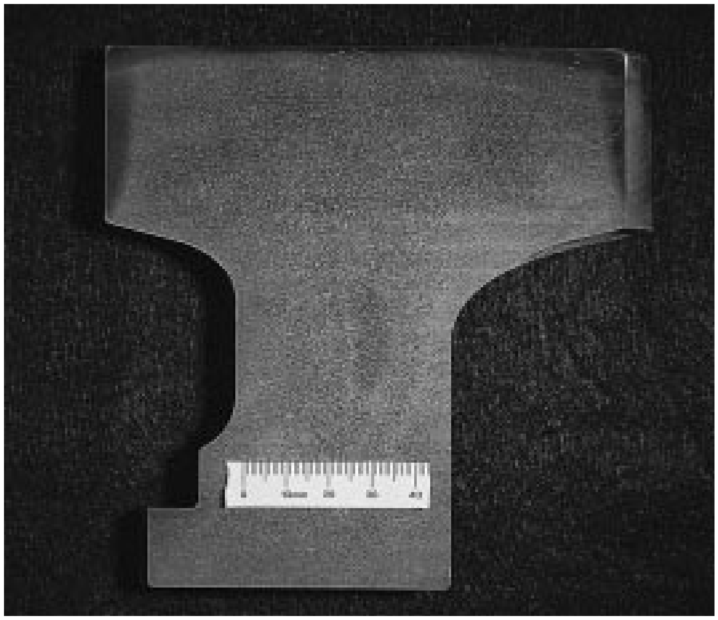

Figure 3. Polished and etched cross-section of NSWC rail (on 45-degree angle to rail length). Notice the dark-etching hardened region on the top and sides.

Characterization of Material

A. The Rails

Only a single, 1 m section of the track could be spared for metallurgical characterization and procedure development. A polished and etched cross-section from this rail revealed the hardened layer on the face and sides of the rail (Figure 3). The depth of this layer, 2 to 6 mm (0.1 to 0.25 in), agreed closely with the original surface hardening specifications for the track (Refs. 1, 3, 4). In general, surface hardness was near HRC 30 to 35 and the remainder of the rail near HRC 25. These hardnesses are typical for the rail composition, 0.75 wt-% C and 0.80 wt-% Mn, listed in the original specifications (Ref. 3) when flame hardened (HRC 30 to 35) and as rolled (HRC 25). The rail was specified in the traditional units of weight as 180 lb/yd (approximately 90 kg/m) (Ref. 5).

B. The Electrodes

Electrodes recommended for rail repair have a wide range of compositions and mechanical properties. The primary requirement for the repair of this track was to closely match the HRC 35 hardness of the face and sides of the track head. Welds much harder than this were undesirable as they could reduce the coefficient of friction between the wheel and track, resulting in slippage of the carriage wheels during acceleration and deceleration. A soft repair weld would allow rapid wear of a joint.

Electrodes used to repair tracks were found to fall into the following three general alloy categories: Hadfield manganese, austenitic stainless steel, and martensitic chromium-carbon alloys. Both the manganese and stainless steel varieties are metastable austenitic structures that achieve good wear resistance through work-hardening of the surface. As the hardened surface layer slowly wears away, a new hardened layer forms. For many conventional track applications, this work-hardening mechanism produces a repair with a low hardness bulk (for high toughness) and a wear resistant hard surface. But, this work-hardening mechanism was unacceptable for repair of this precision track because the surface-hardening deformation necessary could deform the surface beyond its dimensional tolerances or require repeated deformation and surface grinding operations. In addition, the final surface hardness would be too high.

...we found that an expansion gap was necessary to prevent stress cracking during cooling of the weld.

The chromium-carbon alloys produce a stable martensitic structure with chromium providing hardenability and carbon increasing the maximum hardness. These alloys range from a 0.05C-2.3Cr (wt-%) type up to a 6C-27Cr type, with hardnesses ranging from about HRC 20 up to HRC 65. Since the various electrode manufacturers formulate their versions to slightly different strengths, a survey was needed to locate electrodes that matched the hardness of the flame-hardened track when diluted with the track composition.

Return to TOC

Depth from Weld Interface (mm)[a]	High-Heat Input	Low-Heat Input
-4.5	32	-
-4	34	-
-3	35	33
-2	36	34
-1	33	37
-0.5	32	-
0	32	34
1	32	28
3	28	26
5	21	23[b]
6	23[b]	23

(a) Defined as a negative number in the weld metal, 0 at weld interface, and positive in the HAZ and unaffected base metal.
(b) Approximate edge of HAZ.

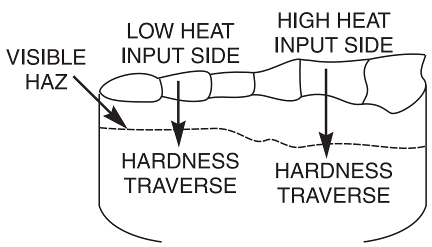

Table 1. Effect of Heat Input on Weld Hardness for Flux-cored Electrode (Hardness in HRC).

All of the common repair processes were considered: thermal spray, shielded-metal arc, flux-cored arc welding, and submerged-arc welding. For this application, a thermal-spray coating was considered unsuitable because a rolling wheel can induce tensile stresses to a few millimeters below the track surface. Although the thermal-spray process thermally affects the track least, the tensile stresses produced during service were expected to debond at the overlay-rail interfaces and spall the repair deposit.

The submerged arc welding process was unsuitable because of its deeply penetrating arc. When the electrode hardness (about 0.4 wt.% C) is mixed with a large quantity of base metal (0.8 wt.% C), the resultant hardness would be unacceptably high.

One flux-cored arc welding (FCAW) and two shielded-metal arc (SMAW) electrodes were selected for evaluation, producing welds of the approximate composition 0.5C-2.3Cr-1Mo. The two SMAW electrodes chosen had the smallest diameters available, 3.2 and 5 mm, and were used to evaluate the effect of heat input and bead size on the deposit quality. The self-shielded FCAW electrode was also chosen in the smallest diameter available (1.6 mm). This electrode's globular-transfer mode provides a relatively shallow penetration, with low dilution.

Development of the Weld Procedure

The welding procedure was developed by evaluating a) effect of heat input on the weld morphology and hardness, b) effect of preheat, and c) the best combination of these variables.

A. Effect of Heat Input

Initial welds were produced with the three electrodes at two heat inputs. The heat input was adjusted by varying the travel speed, producing one heat input near the top of the operating range and one near the bottom of the operating range.

The rail that was purchased for the initial tests was prepared by milling a flat surface to match the head profile of the NSWC track. To allow deposition of a relatively thick weld layer, a second cut on a 45-degree angle removed material at the worn joints. Also, a copper mold was constructed to constrain the molten metal and to allow the production of a weld that matched the small radius at the rail corner.

When a rail steel is welded at or below room temperature, a brittle, hard microstructure is formed. To avoid the creation of this structure, rails are typically heated before welding. This preheating promotes the formation of a softer and tougher structure. A temperature of 260°C (500°F) was chosen for the first two tests.

Welds were made with the three electrodes using various conditions (Ref. 6). The welds were sectioned for etching and hardness traverses, the results of which are presented in Tables 1 and 2. The schematic drawings in Tables 1 and 2 also show the orientation of the different welding electrodes and heat inputs, which allowed evaluation of multiple variables with only two welds. All three electrodes produced welds with hardnesses that were acceptable, but they varied in their ease of use. Also, the higher deposition rate of the FCAW electrode, for such a small area as this repair, forced the welder to move rapidly. Even though we selected an FCAW electrode with the least penetration, it nevertheless had a deeper penetration than the SMAW electrodes, which was judged to be unacceptable. Thus, although an electrode with a globular transfer was selected so the penetration would be minimized, the inherently high penetration of FCAW caused unacceptable dilution for this application.

Depth from Fusion Line (mm)	SMAW-1		SMAW-2	
	High-Heat Input	Low-Heat Input	High-Heat Input	Low-Heat Input
-3	29.5	29	38	37
-2	30	30	42	38
-0.5	32.5	30	43.5	40
0.5	39	39	32	39
2	39	37	-	37
3	36	36	-	-
5	34	31	29[a]	29[a]
6	25[a]	25[a]	25	25

(a) Approximate edge of HAZ.

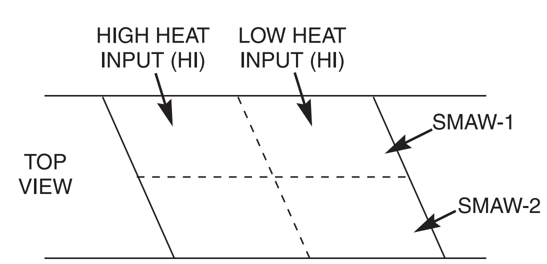

Table 2. Effect of Heat Input On Weld Hardness for Electrodes SMAW-1 and SMAW-2 (Hardness in HRC).

Return to TOC

Depth from Fusion Line (mm)	Preheat 370°C	Preheat 205°C
-3	36	34
-2	35	44
-0.5	37	37
0.5	39	30
2	-	36
3	35	35
5	25[a]	26[a]
6	-	25

(a) Approximate edge of HAZ.

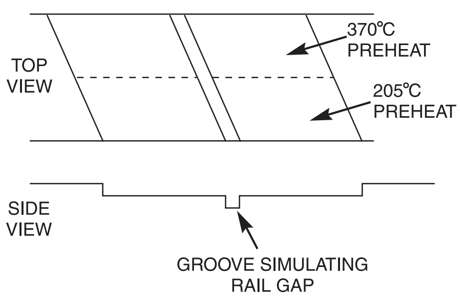

Table 3. Effect of Preheat on Weld Hardness (Hardness in HRC).

Of the two SMAW electrodes, electrode SMAW-2 had a slightly smoother arc and was selected for further investigation.

B. Effect of Preheat

A preheat at lower temperatures produces less thermal expansion in the rail material and can be achieved more easily, while preheat at higher temperatures tempers the brittle martensite to a more ductile structure. To evaluate the effect, a weld was produced with electrode SMAW-2 that included a half with each extreme of preheat recommended for rail steels, 205°C and 370°C. We measured the hardness of the sectioned and etched weld. Table 3 indicates great similarity between the two preheats. A preheat at 205°C gave a weld with HRC 44, but the other preheats all yielded welds between HRC 30 and 40. Thus, a 205°C preheat was sufficient to temper the brittle martensite, while minimizing thermal expansion for the rail.

C. Laboratory Evaluation of Preliminary Procedure

Based on the acceptable results in the preheat test, the rail was preheated to 205°C, and welding was begun using the 5 mm (0.157 in) diameter 0.05C-2.3Cr-1Mo SMAW electrode. The interpass temperature was maintained between 205°C and 260°C (400°F and 500°F). After cooling, the rail surface was flattened by grinding and the hardness of the bead was measured. After the hardness confirmed that the weld met the criteria, a welding procedure record was prepared from the weld results (Ref. 6).

Implementation of the Repair

Individual joints were repaired as the hydrodynamic test schedule permitted. Each repair was carefully scheduled to allow sufficient time for the joint preparation, preheating, welding, and grinding operations and to allow for minimal disruption to the ongoing model testing. High demand for model testing required a repair procedure that could be completed over a weekend. A portable grinder was built to resurface the rails to their original dimensions and tolerances. An indentation hardness test is very precise but leaves a permanent indentation on the surface of the part. This was considered unacceptable for the track, where we had made every effort to produce a very smooth and uniform surface. A rebound hardness tester was substituted for measurement of the final surface hardness.

Post-Repair Vibration Evaluation

A schematic of the hydrodynamic testing carriage is shown on the far left of Figure 4. This carriage is designed with a light load on the rail to the left and the major load, including the drive wheels and motors, on the rail to the right. Only the rails under the right side had shown wear, so their joints were the subject of the repair.

Because both the worn rails and the massive drive wheels (whose interaction caused the vibration) were on the rail to the right, we instrumented only the right side with accelerometers, locating them near the farthest east drive wheel and the inboard drive wheel (Figure 4). The records from both accelerometers provided similar data, so a single record is sufficient to represent the worn condition. As stated earlier, vibration was measured over the speed range of 1.5 to 13 m/s. The data in Figure 4 cover 100 m of track and are representative of the data taken at all speeds. Additional identifiers were added to the figure to correlate the data to the location units used by the carriage operators. Each of the sharp spikes (about 0.012 to 0.023-g in magnitude) in the left record in Figure 4 corresponds to a worn rail joint. The background vibration in the record between the spikes comes from sources of vibration inherent in the carriage. Since this background vibration does not interfere with the experiments, only the spikes needed to be removed.

About 60 joints have been successfully repaired with this procedure. The methods and equipment developed are available if the joints need repair again.

After the joints in this area were repaired, the carriage was run down the track again. The same accelerometers and recording system that identified the vibrations earlier, measured the difference. The background vibration (Figure 4, far right) is the same (confirming the proper replication of the accelerometers), but the spikes from the worn joints are absent.

Return to TOC

Service History of Repaired Joints

About 60 joints have been successfully repaired with this procedure. The methods and equipment developed are available if the joints need repair again.

Conclusions

1. Procedures for grinding and welding were developed which permitted restoration of the original rail dimensions, while matching the original surface hardness.

2. The procedure did not interfere with the scheduled model testing; it was implemented during weekends when the facility was not in use.

3. The repair procedure reduced the vibration in the carriage by over 75 percent, removing traces of the worn joints from the vibration record.

4. Welding the rail surfaces, as opposed to replacing the rail, cost $240,000 vs. millions of dollars, and minimally disrupted ongoing model testing.

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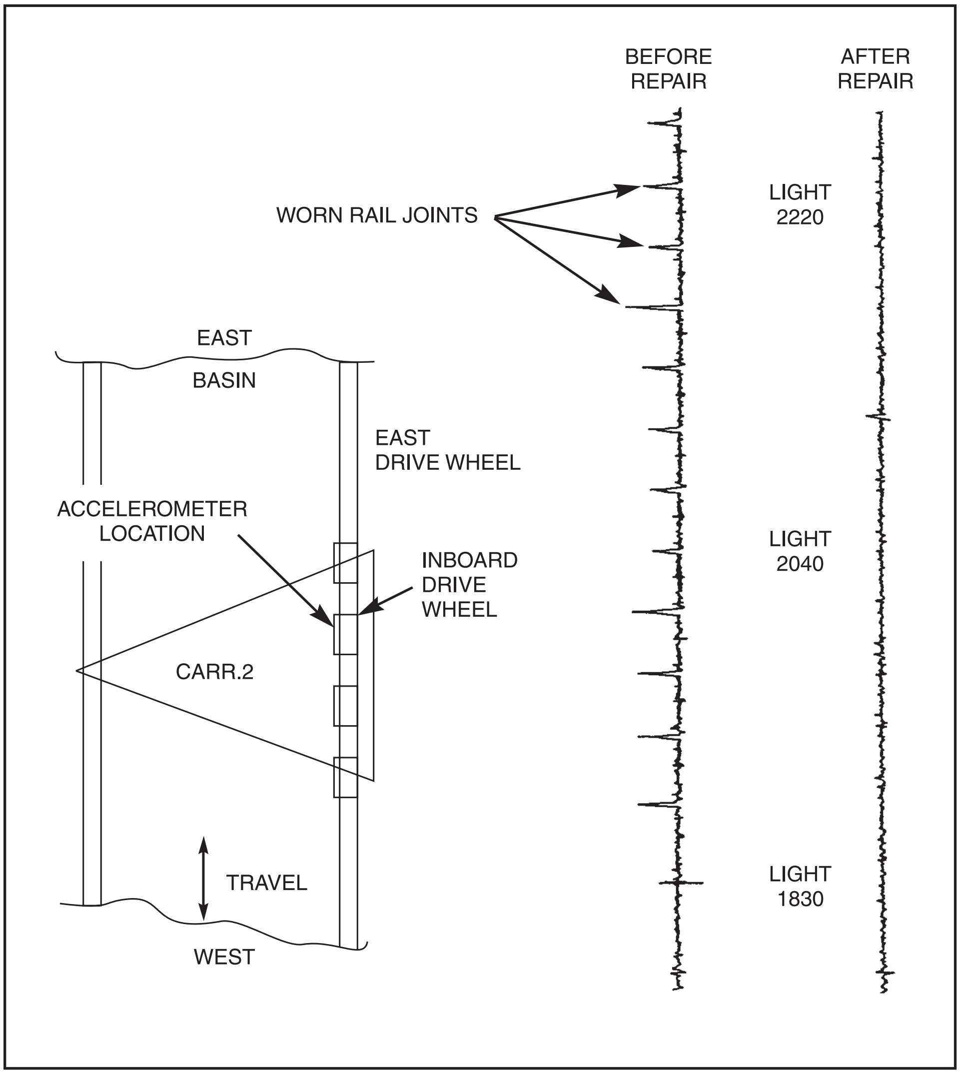


Figure 4. Schematic of carriage and record of vibration before and after the weld repair. Data taken at about a 3 m/s (10 knot) carriage speed.

Return to TOC

Design File

Thicker Steel Permits the Use of Opposing Arcs

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

An Efficient Technology

A welding system featuring opposing arcs is commonly used to weld stiffeners to webs on bridge members. In concept, opposing arc systems can be used to fabricate any tee joint configuration requiring fillet welds on both sides of the vertical member. Originally developed by Ogden Engineering and promoted as the “Dart Welder,” this highly efficient technology permits horizontal fillet welds to be made simultaneously on either side of a tee joint. Since welding is done in the horizontal position, the maximum fillet weld leg size is typically 5/16 in (8 mm). It is commonly applied in stiffener-to-web connections, and may also be used for web-to-flange connections. The system is illustrated in Figure 1.

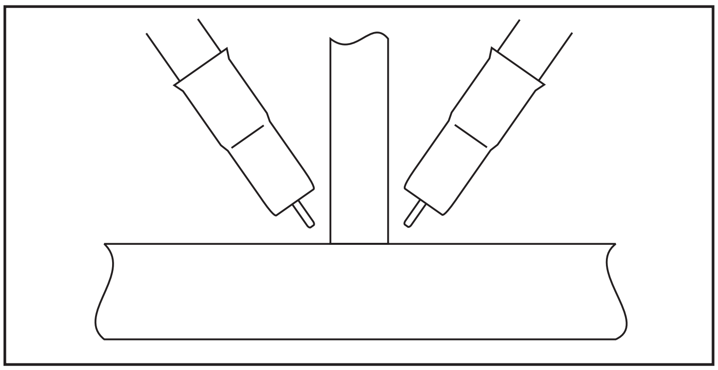


Figure 1.

Cracking Problems

The drawback of this approach is that a unique type of cracking can occur when high energy opposing arcs are applied to relatively thin vertical members. The crack is longitudinal in nature, and occurs slightly below the upper toe of the fillet weld, as shown in Figure 2.

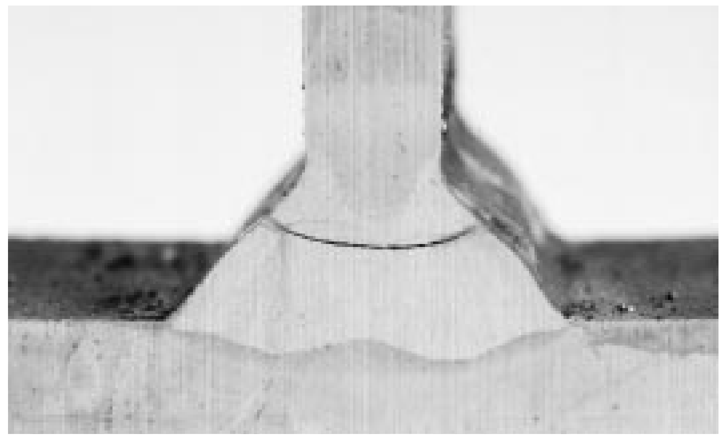


Figure 2.

The cause of this type of crack is fairly simple to understand when the cross-section is analyzed, as depicted in Figure 3. While the two welds are simultaneously made, a tremendous amount of thermal energy is imposed upon the parts being joined. The conduction of heat into the horizontal member is always biaxial in nature. Heat flow is rapid, generating a typical penetration pattern. Conduction through the vertical member, however, is nearly uniaxial in nature, and the thermal energy from two welds must be conducted up through the relatively thin vertical member. As the energy is conducted into the vertical member, the temperature of the base metal rises, decreasing the thermal conductivity. This has a compounding effect, causing less heat to be conducted up through the narrow member. As a result, it is common for the thermal energy to become concentrated in the region between the two beads, raising the temperature of the base

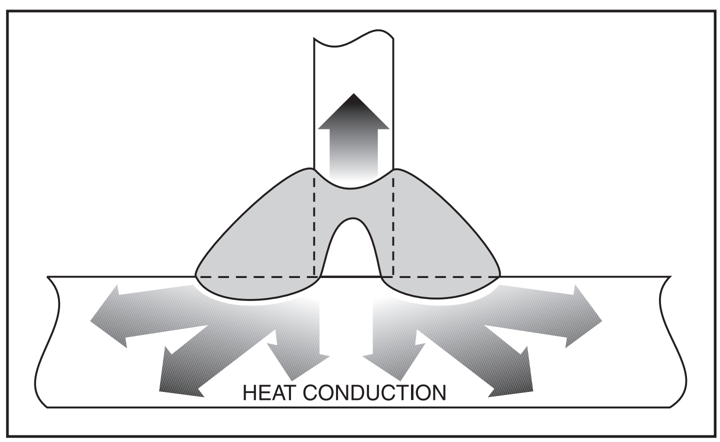


Figure 3.

metal to the melting point. When this occurs, the two weld beads may actually join together, forming a “bridge” of molten metal between the two fillet welds as shown in Figure 4.

When this molten material begins to solidify, grains of solidified material grow roughly perpendicular to the surface, as shown in Figure 4. As the grains growing down from the top surface intersect with those growing up from the region

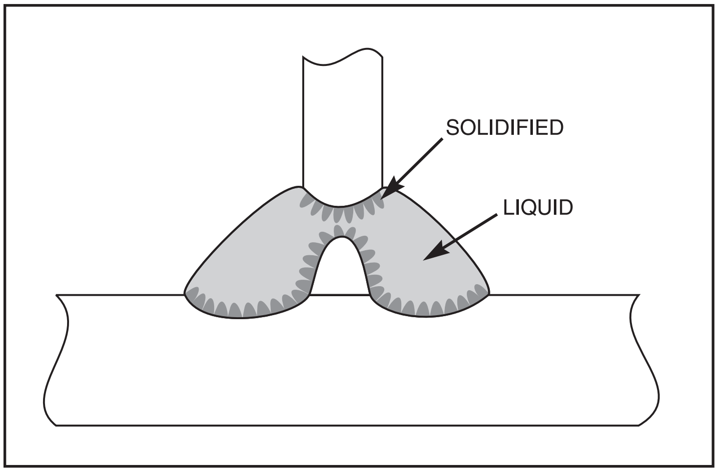


Figure 4.

below, fusion may not be achieved between the two approaching, solidifying planes. This is not unlike the problems associated with improper width-to-depth ratio weld beads that are subject to center line cracking. In this type of cracking, illustrated in Figure 5, the grains form planes that approach from either side and fail to fuse across the center, resulting in a characteristic center line crack. Another look at Figure 4 shows that similar physical principles can cause a crack to develop in the region slightly below the upper toe.

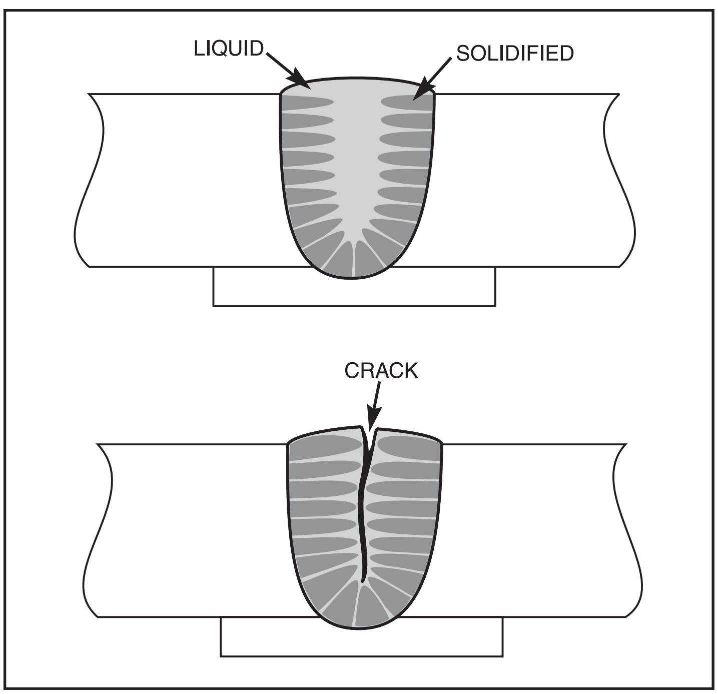


Figure 5.

Eliminating Cracking

To avoid this type of cracking, the “bridging” of metal between the two beads must be eliminated. This is best accomplished by utilizing heavier material for the vertical member. Experience has shown that this condition is quite common when 1/4 in (6 mm) stiffeners are used, but almost never occurs when the stiffeners are 3/8 in (10 mm) thick. When the stiffener is 5/16 in (8 mm) thick, on the other hand, the probability of cracking is directly related to the welding procedure used.

Designers should be encouraged to specify at least 3/8 in (10 mm) thick steel for stiffeners wherever possible. Indeed, many fabricators have found that substituting heavier stiffeners for thinner materials is an economical solution in the long run.

When thinner members must be welded, the “bridging” usually can be overcome by utilizing one or more of the following techniques:

- Directing the energy of the arc more toward the bottom plate by moving the electrode away from the joint root.
- Moving the electrode to a more vertical orientation.
- Lowering the welding current.
- Utilizing negative polarity procedures for submerged arc welding.

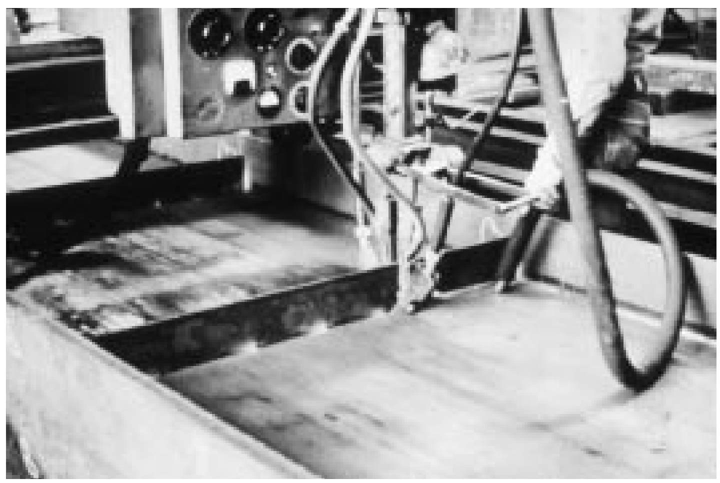


Figure 6.

Conclusion

The use of opposing arcs as shown in Figure 6 can reduce welding costs by as much as 50 percent. A slight increase in the thickness of the vertical member, often only 1/16th (1.5 mm) of an inch, can eliminate significant production problems. The increase in material costs can be easily justified by the avoidance of ongoing problems.

1997 Professional Awards Program

The Jury of Awards, meeting in July of 1997, selected the entries described here for their respective awards. The Trustees of the Foundation appreciate the effort and expertise the Jury brought to this task.

(Left to right) Larry Leifer, Vincent Wilczynski, Burford Furman

Jury of Awards:

Burford Furman
Professor of Engineering
San Jose State University

Larry Leifer
Professor of Mechanical Engineering
Stanford University

Vincent Wilczynski
Professor of Engineering
U.S. Coast Guard Academy

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

BEST OF PROGRAM — $10,000

Precision Weld Repair of Worn Joints in a Towing Basin Track

An innovative procedure was developed to repair the precision support track for the hydrodynamic test carriage at the U.S. Navy's largest ship model towing facility, the Naval Surface Warfare Center in Bethesda, Maryland. Fifty years of use had caused localized wear at the track joints which was limiting the precision of hydrodynamic tests. Estimates for replacing the track ranged from $5 million to $20 million. A weld repair procedure was developed that rebuilt the track ends, and the complete repair, including welding and resurfacing, was accomplished for $240,000.

Jeffrey A. Bradel
Engineer
Naval Surface Warfare Center
West Bethesda, Maryland

Thomas A. Siewert
Group Leader
National Institute of Standards & Technology
Boulder, Colorado

GOLD AWARD — $5,000

Welding One-Watt Heater Units for the Cassini Spacecraft

Loading and GTA closure welding of Light Weight Radioisotope Heater Units (LWRHUs) for the Cassini spacecraft and its probe Huygens was performed on an automated, digitally controlled welding system built specifically for this task. The results of the Cassini capsule welding operation were far superior to those obtained in the production of LWRHUs for the Galileo mission to Jupiter in 1988. For example, weld rejects were reduced from 10.7% to 0.6% for a cost saving of $300,000.

E.A. Franco-Ferreira
Senior Program Manager
Oak Ridge National Laboratory
Oak Ridge, Tennessee

G.H. Rinehart
Project Leader
Los Alamos National Laboratory
Los Alamos, New Mexico

SILVER AWARDS — $2,500

Design, Fabrication and Erection of Braced Frame Stainless Steel Pin Base Connections for the Sather Gate Garage Seismic Upgrade

As part of a seismic upgrade for a concrete garage structure, steel braced frames with stainless steel pin connections mounted on structural steel tapered pedestals were erected to form a new lateral force resisting system. The project required the implementation of proper fabrication techniques and welding procedures to control distortion during the welding of the stainless steel. Project design issues led to the use of pin connections and the selection of stainless steel.

Adam M. Greco
Design Engineer

David W. Cocke
Principal

Degenkolb Engineers, Los Angeles, California

Steel Gateways for a Community Commercial District

Two all-welded, all-steel sculptures in the form of huge Puerto Rican flags provide visually striking "gateways" to an inner-city Puerto Rican commercial district in a major American city. Design wind-loading led to a double-latticed structural solution, which was then realized in flowing "pipe-waves" and hundreds of beautifully ground full-penetration welds. The joining end-to-end of the many pipe segments forming the waves of the flag was the most difficult fabrication challenge of the project.

James R. DeStefano
Principal in Charge

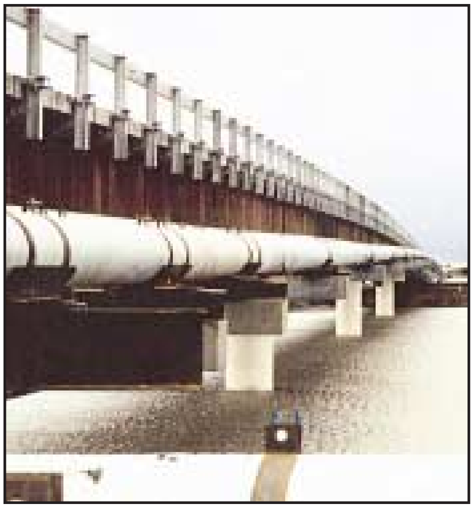
John Adams Dix
Management Principal

John E. Windhorst
Project Architect

DeStefano & Partners, Chicago, Illinois

Return to TOC

SILVER AWARDS — $2,500



West Dock Causeway Bridge

Concern about migrating fish populations along the coast of the Beaufort Sea, off the North Slope of Alaska, prompted federal agencies to mandate that a 650-foot breach be provided in the existing West Dock gravel causeway. Design of a bridge to span this breach while meeting unusual environmental load conditions relied upon modular components of welded steel for the most flexible and cost effective solution.

Alan B. Christopherson
Senior Vice President

John W. Pickering
Senior Engineer

John L. DeMuth
Senior Engineer

Peratrovich, Nottingham & Drage, Anchorage and Juneau, Alaska

BRONZE AWARDS — $1,500

Tudor Road Trail Crossing, Anchorage, Alaska

Arc welded steel and a unique structural design made it possible to bridge the busy, five-lane Tudor Road in Anchorage with an 80-degree curved, torsionally stable span, with no supports other than at the abutments. Field welding connected six prefabricated double box-girder sections and deck plates into a single, 160 foot long weldment which was moved into position and erected with road closure of less than 24 hours.

Dennis Nottingham
President

Todd Sean Nottingham
Senior Engineer

Peratrovich, Nottingham & Drage, Anchorage, Alaska and Seattle, Washington

Movable Seating, the Pyramid, California State University

To fully utilize the interior space of the Pyramid, a unique structure on the campus of California State University at Long Beach, a dramatic new type of welded steel movable seating system was designed and fabricated. The system was required to provide 6,500 spectator seats for college basketball, concerts, or other large assemblies, yet the seats had to be completely removable to allow a clear floor space for a central gymnasium/ arena for student sports and recreation. An all-welded steel cantilever type seating system provided the solution.

James Robert MacIntyre
President
Rollway Grandstand Corp.
Los Angeles, California

Return to TOC

BRONZE AWARDS — $1,500

Bell Street Pier Wave Barrier, Port of Seattle

The 900-foot Bell Street Pier wave barrier, located in over 50 feet of water, was necessary to protect the full-service transient marina that is one of the major attractions to the Port of Seattle's tourist-oriented waterfront facility. The wave barrier, shown here under construction, utilized 4 million pounds of welded structural steel members and piles to resist 8-foot design waves. Key to the success of the project was the use of the designer's "spin-fin" piles. The relative strength of the spin-fin pile to a smooth pile can be likened to comparing a wood screw to a nail.

Roy Peratrovich, Jr.
Principal

Jonathan B. Keiser
Senior Engineer

Jeff Gilman
Vice President

Peratrovich, Nottingham & Drage, Seattle, Washington

Mooring Buoy Seabed Anchor Pile

The failure of an improperly installed connection between a marine mooring buoy and the buoy anchor pile near Valdez, Alaska, created the need to design and construct a new anchorage for the buoy. The new design consisted of an innovative, lightweight steel anchor pile which incorporated features that minimized the weight of the steel, but maximized the mooring resistance of the anchor. Almost all of the welding was designed to be completed in the fabrication process, minimizing field construction time and cost.

Alan B. Christopherson
Senior Vice President

Kenton W. Braun
Staff Engineer

Peratrovich, Nottingham & Drage, Anchorage, Alaska

MERIT AWARDS — $500 EACH

Hawk Inlet Dolphin

Dennis Nottingham
Principal

Christopher M. Gianotti
Senior Engineer

James W. Gute
Engineer in Training

Peratrovich, Nottingham & Drage, Anchorage & Juneau, Alaska

MERIT AWARDS — $500 EACH

Pier A Wave Barrier



Todd Nottingham
Principal Engineer

Dave Pierce
Principal Engineer

Jonathan B. Keiser
Senior Engineer

Peratrovich, Nottingham & Drage, Seattle, Washington

Widening of the Rainbow Bridge


Andrew C. Coates
Principal Associate

Anne M. Zweibel
Associate

Hardesty & Hanover, LLP, New York, New York

Arc Welding on the World's Tallest Building



Len Joseph
Vice President

Chris A. Christoforou
Senior Project Engineer

Lan-Cheng Pan, Ph.D.
Project Engineer

Thornton-Tomasetti Engineers, New York, New York

East River Round-About


Aine M. Brazil
Vice President
Thornton-Tomasetti Engineers
New York, New York

Mark J. Tamaro
Design Engineer
Cagley & Associates
Rockville, Maryland

South Franklin Cruise Ship Dock


John L. DeMuth
Senior Engineer

Christopher M. Gianotti
Senior Engineer

Peratrovich, Nottingham & Drage, Juneau, Alaska

Welded Girder Extensions Used to Facilitate Replacement of Bridge Bearings


Jack H. Stewart, P.E.
Director of Engineering

Scott F. Easter, E.I.T.
Structural Engineer

URS Greiner, Inc., Virginia Beach, Virginia

Return to TOC

Key Concepts in Welding Engineering

by R. Scott Funderburk

Fundamentals of Preheat

Preheating involves heating the base metal, either in its entirety or just the region surrounding the joint, to a specific desired temperature, called the *preheat temperature*, prior to welding. Heating may be continued during the welding process, but frequently the heat from welding is sufficient to maintain the desired temperature without a continuation of the external heat source. The *interpass temperature*, defined as the base metal temperature at the time when welding is to be performed between the first and last welding passes, cannot be permitted to fall below the preheat temperature. Interpass temperature will not be discussed further here; however, it will be the subject of a future column. Preheating can produce many beneficial effects; however, without a working knowledge of the fundamentals involved, one risks wasting money, or even worse, degrading the integrity of the weldment.

Why Preheat?

There are four primary reasons to utilize preheat: (1) it slows the cooling rate in the weld metal and base metal, producing a more ductile metallurgical structure with greater resistance to cracking; (2) the slower cooling rate provides an opportunity for hydrogen that may be present to diffuse out harmlessly, reducing the potential for cracking; (3) it reduces the shrinkage stresses in the weld and adjacent base metal, which is especially important in highly restrained joints; and (4) it raises some steels above the temperature at which brittle fracture would occur in fabrication. Additionally, preheat can be used to help ensure specific mechanical properties, such as weld metal notch toughness.

When Should Preheat Be Used?

In determining whether or not to preheat, the following should be considered: code requirements, section thickness, base metal chemistry, restraint, ambient temperature, filler metal hydrogen content and previous cracking problems. If a welding code must be followed, then the code generally will specify the minimum preheat temperature for a given base metal, welding process and section thickness. This minimum value must be attained regardless of the restraint or variation in base metal chemistry; however, the minimum value may be increased if necessary.

When there are no codes governing the welding, one must determine whether preheat is required, and if so, what preheat temperature will be appropriate. In general, preheat usually is not required on low carbon steels less than 1 in (25 mm) thick. However, as the chemistry, diffusible hydrogen level of the weld metal, restraint or section thickness increases, the need for preheat also increases.

What Preheat Temperature Is Required?

Welding codes generally specify minimum values for the preheat temperature, which may or may not be adequate to prohibit cracking in every application. For example, if a beam-to-column connection made of ASTM A572-Gr50 jumbo sections (thicknesses ranging from 4 to 5 in [100-125 mm]) is to be fabricated with a low-hydrogen electrode, then a minimum prequalified preheat of 225°F (107°C) is required (AWS D1.1-96, Table 3.2). However, for making butt splices in jumbo sections, it is advisable to increase the preheat temperature beyond the minimum prequalified level to that required by AISC for making butt splices in jumbo sections, namely 350°F (175°C) (AISC LRFD J2.8). This conservative recommendation acknowledges that the minimum preheat requirements prescribed by AWS

Return to TOC

D1.1 may not be adequate for these highly restrained connections.

When no welding code is specified, and the need for preheat has been established, how does one determine an appropriate preheat temperature? Consider AWS D1.1-96, Annex XI: "Guideline on Alternative Methods for Determining Preheat" which presents two procedures for establishing a preheat temperature developed primarily from laboratory cracking tests. These techniques are beneficial when the risk of cracking is increased due to the chemical composition, a greater degree of restraint, higher levels of hydrogen or lower welding heat input.

The two methods outlined in Annex XI of AWS D1.1-96 are: (1) heat affected zone (HAZ) hardness control and (2) hydrogen control. The *HAZ hardness control method*, which is restricted to fillet welds, is based on the assumption that cracking will not occur if the hardness of the HAZ is kept below some critical value. This is achieved by controlling the cooling rate. The critical cooling rate for a given hardness can be related to the carbon equivalent of the steel, which is defined as:

$$CE = C + \frac{(Mn + Si)}{6} + \frac{(Cr + Mo + V)}{5} + \frac{(Ni + Cu)}{15}$$

From the critical cooling rate, a minimum preheat temperature can then be calculated. AWS D1.1-96 states that "Although the method can be used to determine a preheat level, its main value is in determining the minimum heat input (and hence minimum weld size) that prevents excessive hardening" (Annex XI, paragraph 3.4).

The *hydrogen control method* is based on the assumption that cracking will not occur if the amount of hydrogen remaining in the joint after it has cooled down to about 120°F (50°C) does not exceed a critical value dependent on the composition of the steel and the restraint. This procedure is extremely useful for high strength, low-alloy steels that have high hardenability. However, the calculated preheat may be somewhat conservative for carbon steels.

The three basic steps of the hydrogen control method are: (1) Calculate a composition parameter similar to the carbon equivalent; (2) Calculate a susceptibility index as a function of the composition parameter and the filler metal diffusible hydrogen content; and (3) Determine the minimum preheat temperature from the restraint level, material thickness, and susceptibility index.

How Is Preheat Applied?

The material thickness, size of the weldment and available heating equipment should be considered when choosing a method for applying preheat. For example, small production assemblies may be heated most effectively in a furnace. However, large structural components often require banks of heating torches, electrical strip heaters, or induction or radiant heaters.

Preheat must be used whenever applicable codes so specify...

Preheating carbon steel to a precise temperature generally is not required. Although it is important that the work be heated to a minimum temperature, it usually is acceptable to exceed that temperature by approximately 100°F (40°C). However, this is not the case for some quenched and tempered (Q&T) steels such as A514 or A517, since welding on overheated Q&T steels may be detrimental in the heat affected zone. Therefore, Q&T steels require that maximum and minimum preheat temperatures be established and closely followed.

When heating the joint to be welded, the AWS D1.1 code requires that the minimum preheat temperature be established at a distance that is at least equal to the thickness of the thickest member, but not less than 3 in (75 mm) in all directions from the point of welding. To ensure that the full material volume surrounding the joint is heated, it is recommended practice to heat the side opposite of that which is to be welded and to measure the surface temperature adjacent to the joint. Finally, the interpass temperature should be checked to verify that the minimum preheat temperature has been maintained just prior to initiating the arc for each pass.

Summary

- Preheat can minimize cracking and/or ensure specific mechanical properties such as notch toughness.
- Preheat must be used whenever applicable codes so specify; when no codes apply to a given situation, the welding engineer must determine whether or not preheat is needed, and what temperature will be required for a given base metal and section thickness.
- Annex XI of AWS D1.1-96 provides guidelines for alternative methods of determining proper amounts of preheat: the HAZ hardness control method, or the hydrogen control method.
- Preheat may be applied in a furnace, or by using heating torches, electrical strip heaters, or induction or radiant heaters. Carbon steels do not require precise temperature accuracy, but maximum and minimum preheat temperatures must be followed closely for quenched and tempered steels.

For Further Reading...

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Graville, B.A. *The Principles of Cold Cracking Control in Welds.* Dominion Bridge Company, Ltd., 1975.
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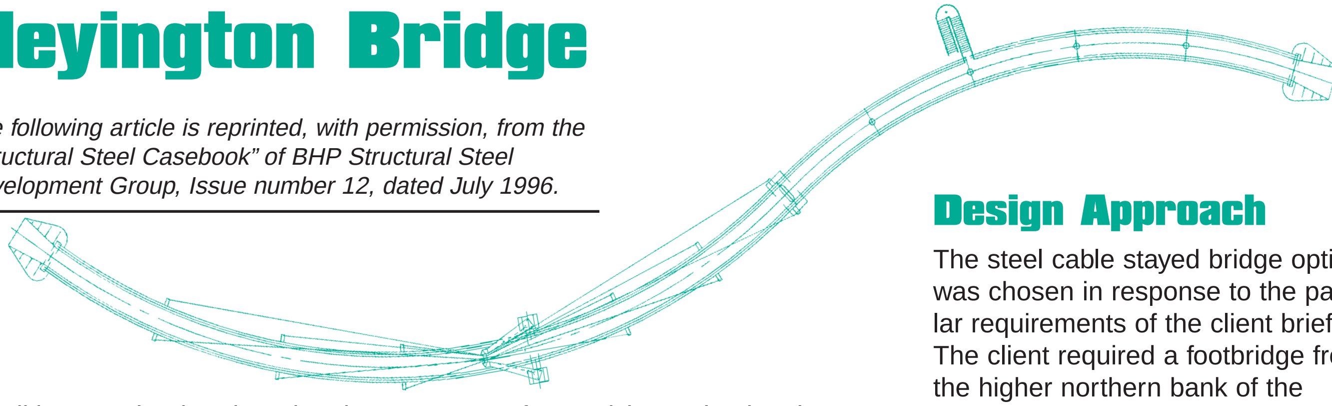
Heyington Bridge

The following article is reprinted, with permission, from the "Structural Steel Casebook" of BHP Structural Steel Development Group, Issue number 12, dated July 1996.



A striking new landmark pedestrian bridge spans the new Eastern Freeway extension and adjacent wetlands in Doncaster, an eastern suburb of Melbourne, Australia. The 172 meter Heyington footbridge provides a vital link for pedestrians, cyclists and school children crossing the new freeway, and easy access to the wetlands for recreation, whilst visually enhancing the landscape.

Bridge Description

Heyington Avenue footbridge is a curved cable stayed bridge with curved approach spans supported on single column piers.

The cable stayed deck has a 75 meter radius curve in plan and a slight crest vertical curve. Main supporting elements are twin steel box beams with tubular transoms (406CHS6.4) at cable stay anchorage points. These transoms cantilever beyond the edges of the deck to ensure the cable stays do not interfere with the clearance box. The steel deck is braced with plan diagonal bracing (200x200x13EA) and intermediate transoms (200UB18) which also support timber deck joists.

Jarrah timber deck planks and hand rails are a feature of the bridge. Jarrah was selected for its long life, visual appearance and tactile properties.

Central to the visual impact of the bridge is the steel tower, a leaning 'A' frame which was designed to be self supporting during erection and construction. The tower's legs are triangular in cross-section and taper to an upper frame giving a slender elegant appearance. Cable stay anchorage points are accommodated in the tower's upper frame where stiffened gusset plates accept clevis pin anchorages.

Approach spans are twin, simply supported steel edge beams, curved in plan to a 65 meter radius. Universal beam transoms are provided at pier locations to support the twin box shaped edge beams on a single column pier. The external shape of the box beams is the same for both the

cable stayed and simply supported sections, ensuring visual continuity. All rolled sections are Grade 300PLUS. Steel circular hollow sections are Grade 350. Plates are Grade 250 steel with Grade 350 steel used in some locations.

Design Approach

The steel cable stayed bridge option was chosen in response to the particular requirements of the client brief. The client required a footbridge from the higher northern bank of the Eastern Freeway reserve to the low lying southern area where an existing shared path was to be maintained.

The client's specifications state in brief: "The objective of the bridge is to maintain continuity and quality of pedestrian space throughout the valley...The bridge is to appear to the pedestrian as a continuation of the natural landscape...it should not create a negative, uncomfortable and threatening space over the road."

The architect worked in conjunction with the contractor and designers to achieve the brief's objectives.

The bridge was to curve down from its northern point, over the future Eastern Freeway, and curve back around a proposed wetlands area. This alignment required a reverse curve on the bridge and a second curve allowing users to look back to the west, over future wetlands. A stairway was also required to link the footbridge to a boardwalk to traverse these wetlands.

Architecturally, the response to these requirements was a smoothly curving cable stayed structure featuring a single mast reaching towards the higher ground and over the new freeway. This form was chosen for:

1. its structural efficiency; it requires a minimum of support points allowing the bridge to touch the earth lightly.
2. its mast and cable pattern, providing a landmark and orientation point;
3. the visual effects upon the mast of changing light conditions which vary with time of day and season; and
4. its physical 'aliveness' and response to the user's presence.

Return to TOC

While the support structure is steel, chosen for its efficiency and strength, the 'human' components of deck, balustrade and handrail are of timber chosen for its acoustic, visual and tactile qualities.

Heyington Avenue footbridge has a color scheme to enhance identification of its location, and to be compatible with future noise walls which will be constructed near the bridge and parallel to the freeway alignment.

Spotlights will be installed near the base of the steel mast to highlight balustrades, cables and mast. Highlighting will create the impression of the structure floating in space.

Structural design presented a challenge. Horizontal curvature of the bridge added much complexity to the already difficult task of analyzing a cable stayed bridge with multi-stays. The system is highly redundant, with significant interdependence of structural components such as the tower, deck and cables. The engineers used a three dimensional model to analyze the structure, taking into account non-linear behavior of the cable stays and the stiffness of each member which significantly affected structural behavior. Careful considerations in design, detailing and documentation were key factors in the successful assembly of the bridge.

Steelwork Fabrication

Shop drawings were generated using CAD. According to Eric Hanschmann, Associated Iron Industries senior estimator, "It was important to engage a competent drafting service with CAD capability. CAD drafting was faster and more efficient. The CAD file, on a disk, was forwarded to...(the) steel distributor for precutting all the plates to the various shapes and curves. The CAD file was input directly to CNC equipment for accurate and economical cutting of rather complex shapes."

Fabrication of the tower was the most challenging task, consisting of triangular legs tapering from 1,212 mm at the base, to 635 mm at the top. Sub-arc welding was used throughout.

Surface treatment for steelwork consisted of sandblast to Class 2.5, 75 microns of inorganic zinc silicate primer topped by a 125 micron high build epoxy tiecoat and 50 microns polyurethane final coat. The tower color is known as Desert Flower Red and the remainder of the steelwork is in Metallic Gray.

Construction

The 70-meter approach to the bridge was erected in five sections (5 spans of 14 meters). Carpentry work to install timber decking and handrails was undertaken in the structural steel painter's yard and then transported to the site, except for one bay of timber decking across the joint. The units were lifted from the trucks into position and spliced together using bolted connections. Erection was by a 50 t crane and was completed in one working day.

Falsework for the cable stayed deck consisted of five support towers, with the bridge sections being supported on screw jacks permitting fine tolerance to be achieved during erection.

The 'A' frame tower was transported to site in one piece, requiring a special route which allowed the unit measuring over 35 meters to negotiate corners. The tower was lifted off the transporter using a 200 t crane and a 50 t tailing crane. Erection was carried out over one morning and the third section of the cable stayed deck, which is supported by the transom located between the tower legs, was erected that afternoon. The next day, the remaining four sections of the cable stayed deck were erected on falsework. Assembly and erection of the twelve steel stress bar tie rod was completed over five days. Each tie rod was assembled in one piece on the ground along the alignment of the bridge and then installed using two cranes.

Rods were stressed using hydraulic jacks in a predetermined staged sequence. Falsework was dismantled and bridge carpentry, abutments and finishing touches completed within two months from the commencement of bridge erection.

Summary

Ian Bryant, John Holland's Engineering Manager, said, "Despite the difficult geometry of the bridge, meticulous detailed planning of construction ensured a very smooth operation with no hold ups. The bridge was assembled quickly and we are very pleased with the end result."

Gary Liddle, Vic Roads Project Manager, said, "The bridge has provided a feature in this section of the valley that fits the environment being developed with the adjacent wetlands. The bridge combines aesthetic appeal with the functionality of providing for pedestrian movements across the valley."

Project Participants

Client:	*Vic Roads*
Architect:	*Brian Stafford*
Engineer:	*Maunsell Pty Ltd*
Contractor:	*John Holland Construction & Engineering Pty Ltd*
Steel Fabricator:	*Associated Iron Industries*

Mentoring in the Engineering Profession

Passing It On...

By Jeffrey W. Post, P.E.
J.W. Post & Associates, Inc.
Humble, Texas

This article concludes a four-part series in which four different engineers recount their unique experiences as protégés of recognized experts in the steel fabrication industry.

Ray Stitt and I first met at an American Welding Society D1.1 Code meeting in Atlanta in 1977. It was my first code meeting attendance. At that point, he was 70 years old, but still very active and regularly attending AWS meetings. I remember that he struck me as one of the old-timers, amongst many old-timers. In those days, the committee was much smaller than it is now.

Ray Stitt, mentor, and Jeff Post.

Show and Tell

Every time I saw him at a code meeting, Mr. Stitt would reach into his pocket and take out a packet or two of snapshots of his latest flame straightening job. He was always eager to show them to people, and to explain the techniques and challenges of a particular project. Over the years, I began to look forward to these "show and tell" sessions, and perhaps I expressed a little more interest than some of the others. My only hands-on exposure to flame straightening up to that point had been the experience of cambering a couple of bridge beams, and at the time, I hadn't really understood the principles behind the work.

Ray Stitt had retired from R.C. Mahon in Detroit, but was very active as an independent consultant by the time I met him. Part of our initial camaraderie may have stemmed from the fact that I was a 1966 graduate of Ohio State's welding engineering program, which he actually had started in 1938, with some help from The James F. Lincoln Arc Welding Foundation.

Our relationship grew very gradually. Perhaps he was consciously looking for someone to whom he could pass on his vast store of knowledge, but if that was the case, I certainly was not aware of it at the time. I do know he recognized that I was interested in doing hands-on work; such an interest would be essential for any protege of his. So during the years from 1977 until 1985, I looked forward to the semi-annual code meetings, and Ray Stitt's latest illustrated lesson in distortion control.

Mentoring by Phone

In 1985, a colleague in Houston asked me to try to straighten a bridge girder in his shop. My long distance call asking for Mr. Stitt's help and advice turned out to be the first of many I would make over the ensuing nine years. After that first job in '85, with

Perhaps he was consciously looking for someone to whom he could pass on his vast store of knowledge

my mentor's recommendation, I bought my first torch and the necessary accessories. I went to a local shop, gathered some scrap material, and started to practice. Then I would telephone Mr. Stitt, and we would talk about what I had seen, and what I had measured.

"Jeff's Post"

Eventually, I decided to create a demonstration project. Our church needed to replace a wooden basketball goal that was falling apart. I thought, "Why not bend a piece of pipe and make a permanent fixture?" Of course, it was the most expensive possible way to hold up a basketball hoop, but I learned a lot doing it, and I still

Return to TOC

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Memo 5/30/89.

To Jeff –

Print 1 "Before" This damage to the 7/8" thick bearing stiffener was done in transit when the 149,300 lb. girder for a R.R. Bridge, in Dearborn, MI. shifted in the car. Note the "BIG" torch we used on the 7/8" thick PL. The soapstone mark is along the tangent point where we start heating.

Print 2 "After" The bearing stiffener is back in position, "as-good-as-new" and repainted.

Print 3 "Before" This is the right hand end of the 149,300 lb girder. Note that this bridge is on a skew with the other attached girder offset to fit the skew.

Print 4 "After" This is the same right hand end of the 149,300 lb girder, like new and painted.

Print 5 "Before" This is one of the other seven bearing stiffeners, but this time on a 152,000 lb shipping piece.

Print 6 "After" This is the same left-hand end fully restored and painted.

There were 9 bearing stiffeners and 16 – ½" thick other stiffeners (some of which were bent to 45° from the web) to be straightened. It took us 4½ – 8 hour days working with a crew of 4 Good Iron workers.

— 11 —

Compare that with arc gouging the welds of the 25 stiffeners, grinding the residue of the fillet welds so as not to gouge into the web PLs, obtaining new stiffener material with proper bolt holes, making the new layout, fitting and vertical and overhead welding.

J Stitt

5

6

J. Ray Stitt

Born in Youngstown, Ohio, in 1907, Ray Stitt held a B.S. in civil engineering from Penn State. His first position after graduation, as a construction field engineer for the Austin Company, led to a lifetime interest in welding and, ultimately, in his specialty area of distortion control.

In 1938, Mr. Stitt was invited to join the faculty of the Engineering College of Ohio State University to organize and teach the first complete curriculum in Welding Engineering, leading to the establishment of a Bachelor of Science degree in the field. During World War II, he worked on welding and metallurgical problems for the Office of Scientific Research and Development. In 1950, he became the first individual to register as a professional welding engineer in the State of Ohio.

From 1945 to 1971, Mr. Stitt was in charge of research and technical service at the R.C. Mahon Company in Detroit. He was very active in the Detroit Section of the American Welding Society, serving as chairman and for nine years as secretary-treasurer of the Section. He was a member of the AWS National Board of Directors and in 1963 was appointed to the Structural Welding Committee of AWS. He was posthumously elected a Fellow of the American Welding Society in 1994.

During the course of a long and productive career, Ray Stitt perfected methods and techniques for the torch heat straightening of weldments or members that had been distorted by shrinkage stresses, overloads, or fire. He published many articles and frequently lectured on the importance of controlling distortion by proper preplanning and the correct execution of welding procedures. Following his retirement from R.C. Mahon in 1971, he became an international "trouble-shooter" in this area, and was known all over the world by the motto printed on his business card, "Have Torch, Will Travel."

As he approached his eighties, Mr. Stitt began to share his expertise in distortion control with a younger colleague, Jeffrey Post. He increasingly referred clients to Jeff, to whom he eventually, symbolically, "passed the torch." Ray Stitt passed away in 1994 at the age of 87, after a brief illness.

use photos of the project to demonstrate the principles of flame straightening. Mr. Stitt, who advised me every step of the way on the project, liked to make puns on my name, so he dubbed it “Jeff’s Post.” With his penchant for photos, he made a collage of the project snapshots I sent him; I still show potential clients that collage today. He also enjoyed reminding me that he was giving me a “Post-graduate” course in flame straightening and distortion control.

“I’m Sending a Young Fella”

As time went on, Ray Stitt continued to field calls from all over the country requesting his expertise. After all, his business card read “Have Torch, Will Travel.” But the “will travel” part became more and more difficult. By 1985, he was 78, and problems with his legs had begun to restrict his ability to climb. When clients called, he started to respond, “Yes, we can do this. I’m going to send you a young fella I’m teaching.”

...the thought is always there, that I need to find the right person, who can accept the heritage of Mr. Stitt’s knowledge and transfer it to yet another generation.

In the beginning, we had extensive phone conferences before each job. We shared photos or drawings of the damage, and he would suggest the best line of attack. As time went by, he refrained from suggesting solutions, saying instead, “Well, what do you think you’re going to do?” And eventually, he began turning it over entirely, saying “You figure it out and let me know.”

Rules of Distortion Control

I was uniquely blessed to learn the lessons of distortion control from a master. Some of the rules Ray Stitt repeated until they became a part of me included:

- Plan to prevent distortion through a detailed fabrication and welding sequence. Don’t let the member get out of shape in the first place.
- Study the distorted member thoroughly in order to determine how to shrink it back to its original configuration. (However it got into that shape, reverse that action.)
- Always heat on the bend lines (yield lines), i.e., convex sides.
- Patience and perseverance are essential. Never let the contractor or anybody else rush you on the job.

Sink-or-Swim

On a typical job, we would confer by mail and telephone beforehand, agreeing on an approach to the problem at hand. Then I would fly out to the job site and start the actual flame straightening. I’d call Mr. Stitt at lunchtime and say, “Man, this isn’t responding like I thought it would,” and he’d say, “Why don’t you change the pattern a little bit? Try this...” Then when I got back to the motel room at night, I’d call him again and say “Well, it’s still not working exactly right,” and he would offer some more suggestions. During the next day’s lunch hour, I’d call him to report, “Now it’s working the way you told me it should!”

It would have been better to have had him there, but the fact that he wasn’t there put me in a sink-or-swim situation. I had to do it on my own each morning, and every afternoon, but it was great knowing I could talk to him and get immediate feedback twice a day. On every job, I’d take a set of photos—before, during, and after. I’d always order double prints, and number each set of photos. After I sent him one set of the snapshots, he’d call me and say “O.K. Jeff, look at number 6. What were you doing right there?” It was sort of a debriefing, after every job, and it was incredibly helpful.

I doubt that anyone ever had a better “Post-graduate” course than the one Ray Stitt gave me. It is not only an obligation now, but it will be a privilege to pass it on. Although at the age of 54, I feel I am far from retirement, none of us knows how much time we have. So the thought is always there, that I need to find the right person, who can accept the heritage of Mr. Stitt’s knowledge and transfer it to yet another generation.

A Sudden Loss

Ray Stitt died somewhat unexpectedly. As often happens with elderly people, he had a couple of bad falls, developed pneumonia, and was suddenly gone. A few months later, I went to his home to help his family sort out his files. I would open a file, and there would be a note to me: “Jeff, on this case, we did....” He hadn’t known whether or not we’d have the time get to certain subjects, so he’d left me notes, just in case.

At the end of our phone calls, it was his habit to sign off by saying, “More power to you.” I was always glad to hear that from him. He was a great guy, and I really miss him. He died three years ago, and there are still times, on the toughest jobs, when I talk to him, and dearly wish I could hear his answers.

Return to TOC

WALHALLA TOURIST ROAD BRIDGE

The Design Department of Vic Roads, Australia

1996 Recipient of the A$5,000 First Prize Award in the Biennial Australasian Steel Bridge Awards Program co-sponsored by the Australian Institute of Steel Construction and the James F. Lincoln Arc Welding Foundation

Walhalla Tourist Road truss bridge over the Thomson River.

Project Description

A through steel truss bridge 98 metres long and 9.9 metres wide was constructed to take the Walhalla Tourist Road over the Thomson River. The bridge is the gateway to the tourist township of Walhalla; it replaced a timber bridge, and was designed to improve the poor vertical and horizontal alignment of the existing road.

The five-span bridge has a skew of 35.3°, with spans of 9.42, 22.04, 22.04, 22.04 and 21.82 metres. The first span consists of steel universal beams with a composite concrete deck. The other four spans consist of steel through-trusses using square hollow sections with a composite concrete deck supported by ‘universal’ beam stringers and ‘welded beam’ crossbeams. The width of the bridge comprises a 1.5 metre wide footpath and 7.5 metres for traffic lanes. The trusses are supported by reinforced concrete ‘hammer head’ shaped piers ranging in height from 5.1 to 13.4 metres.

Structural Details

The bridge was designed using NAASRA Bridge Design Specification 1976 supplemented with references to technical papers on recent designs for square hollow sections. The design was reasonably straightforward, but the skew of 35.3° presented a number of problems in the detailing. The skew angle was chosen to provide a longitudinal displacement of the upstream truss in relation to the downstream truss of two panel points. This allowed the crossbeams to be perpendicular to the trusses, thereby simplifying the connection details. The design philosophy was to simplify construction procedures as much as possible, with the steelwork designed to be bolted together on site, and on site welding kept to a minimum. In fact, the only on site welding was of the cross frames used in span 1.

Conclusion

The Walhalla Tourist Road Bridge was designed to achieve an efficient use of steelwork and to simplify on site construction. Fabrication relied primarily on fillet welds and simple connection detailing. During the tender process, bridges with concrete superstructures had been proposed as alternatives, but the cost of this steel truss bridge was lower. The location, limited site access and efficient design were the main factors contributing to the very competitive cost of the truss bridge.

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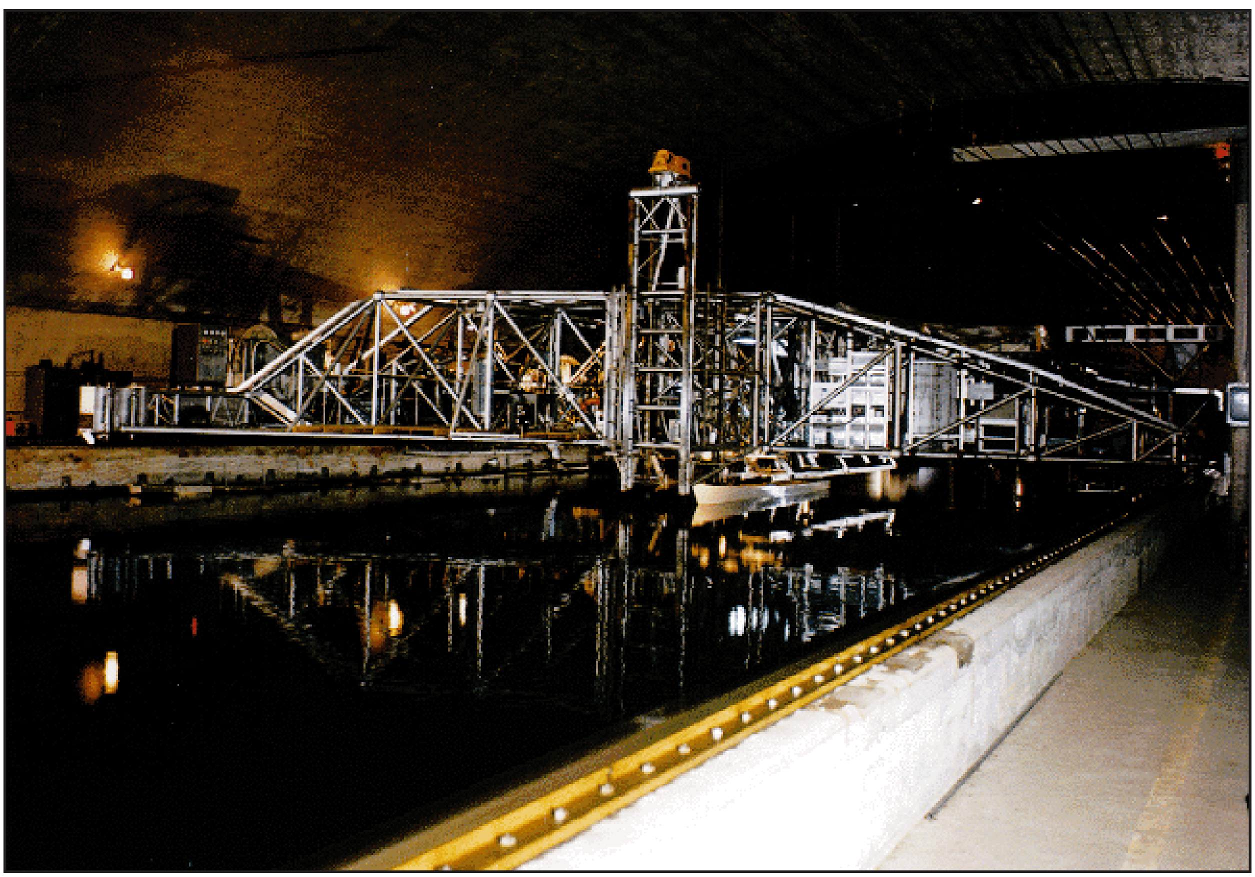

The support track for the hydrodynamic test carriage at the U.S. Navy's Naval Surface Warfare Center was restored by welding. *See story on page 2.*

Return to TOC