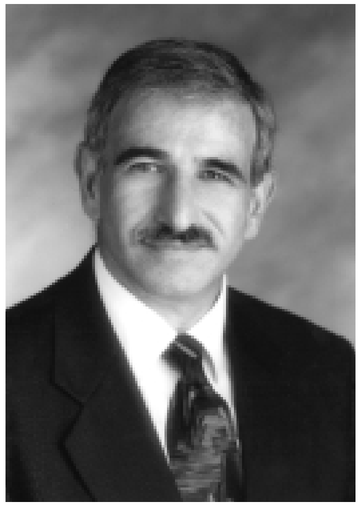
Volume XVIII, Number 1, 2001

Welding INNOVATION

Advancing Arc Welding Design and Practice Worldwide

A publication of the James F. Lincoln Arc Welding Foundation

Fighting Destruction By Building Consensus



On January 17, 1994, both the San Fernando Valley near Los Angeles and much of the building construction industry were shaken by the Northridge earthquake. For those of us involved in the technical investigations that followed, the past seven years have been a whirlwind of discovery, analysis, controversy and learning.

I had the privilege, responsibility, frustration and professional satisfaction of serving as Director of Product Development in the FEMA-sponsored SAC project, which ultimately became a six-year, approximately $12 million effort to better understand the behavior of the steel moment frame connections that fractured during the Northridge earthquake and to develop new practices to avoid these problems. I am pleased to announce that the findings of the SAC project are now available in published form from FEMA. See page 10 of this issue for details.

Recently, in an interview for *Structural Engineer*, I was asked, “Did SAC participants 'take sides' on certain issues?” I responded that the participants did indeed take sides, based on legitimate technical opinion and also influenced by four less altruistic factors:

- Litigation issues. A variety of lawsuits had been filed as a result of damage to buildings. Since the SAC project sought to gather the best technical experts in the field, and attorneys on the both sides consulted the same individuals as expert witnesses, views expressed in the courtroom inevitably cropped up during technical sessions.
- Economic issues. These arose because steel suppliers, fabricators, erectors and others had understandable concerns that SAC recommendations might unjustly increase the cost of steel construction, and give other materials a market advantage.
- Job security. For example, some inspectors became concerned that SAC recommendations might limit the scope of their involvement on projects.
- Proprietary designs. Participants’ knowledge of proprietary connection design information complicated the kind of group interaction that has characterized much of the research in the steel construction field.

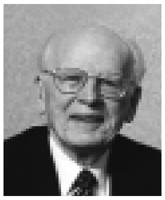
Despite these complications, or perhaps even because of them, what has emerged in these new FEMA documents is the best, most comprehensive set of recommendations compiled to date to address the discoveries that resulted from the post-Northridge investigations. Yet these guidelines lack one element common to U.S. building codes: consensus. These are simply guidelines, not codes. By themselves, they have no legal authority. FEMA documents cannot be incorporated directly into building codes because they have not gone through the consensus process.

I am pleased to report that this process is underway. The AISC Seismic task committee, under the capable leadership of my SAC colleague Jim Malley of Degenkolb Engineers, continues to work SAC developments into the *Seismic Provisions for Structural Steel Buildings*. Welding-related issues from the SAC project are being considered by the AWS D1 Seismic Welding subcommittee, led by Duane Miller, Chair, with myself as Vice Chair. Not only are we considering the SAC work, but we are following developments from Japan and New Zealand, as well as the IIW commission chaired by Hardy Campbell of AWS. Also, we have taken steps to bring the views of some of the strongest critics of the SAC recommendations to the table for consideration as well.

Of course, along with consensus comes the reality of dealing with differences of opinion, again. When these are based upon honestly differing points of view, we all must be patient, respectful and try to see each other’s perspectives. This is when the best of the consensus process emerges. Uneducated opinions, self-serving perspectives and ‘head-in-the-sand’ denials of the need for change only hinder the process.

As I viewed damage from the recent earthquake in Washington state, the inevitability of earthquakes and their destructive power emphasized to me the need to incorporate into consensus standards the best and most current science for dealing with these mighty forces of nature. I urge my colleagues to use the consensus process in the best manner, for the good of all members of society whose lives are touched by the standards we help develop. When we do so, ours will be a lasting contribution.

Ronald O. Hamburger, S.E.
Senior Vice President
EQE International Inc.



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Volume XVIII
Number 1, 2001

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The James F. Lincoln
Arc Welding Foundation

Welding Innovation

Cover: The Maritime Off-Ramp Bridge in San Francisco is the first curved welded steel orthotropic bridge in North America. See story on page 2. Cover photo by Lynn Harrison of Caltrans.

Features

Departments

North America's First Curved Welded Steel Orthotropic Bridge

by James E. Roberts, Tony Marquez, Carl Huang, Alfred Mangus
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ICF Kaiser Engineers, Oakland, California
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Introduction

The $22 million, 2,356ft (718 m) long "Maritime Off-Ramp," also known as the Horseshoe Line or "HS" Line because of its 250 ft (76 m) radius horseshoe shape, is the first curved welded steel orthotropic bridge in North America. It is one of eleven bridges in a $130 million interchange located at the intersection of two freeways I-80 and I-880 (Figure 1), which was one construction contract. The $1.1 billion replacement project was designed and then built in seven separate construction contracts to minimize impact on traffic in this important transportation corridor.

There are fewer than 50 orthotropic bridges in North America and about 1,000 in Europe. "Orthotropic" is a combination of two words, orthogonal and isotropic. When such a bridge is built entirely of steel, the term "orthotropic bridge" is used, but if only the deck is orthotropic, the term "orthotropic deck" is used to describe the bridge. The jargon "tub" girder is sometimes used to describe a reinforced concrete deck placed on top of steel components consisting of two webs and bottom plate. The Maritime Off-Ramp bridge was more difficult to fabricate than a tub girder because the welders and painters would then be required to work inside a closed cellular structure.


Lynn Harrison, Caltrans

Figure 1. Morning rush hour traffic under the Maritime Off-Ramp.

Bridge Design Criteria

- Load factor design was used and 1983 AASHTO with interims and revisions by CALTRANS.
- Live Loading: HS20-44 and alternative and CALTRANS permit design load.
- Seismic Design: The reconstruction project design package with a site-specific response curve.
- Structural Steel: ASTM A709 Grade 50T2 was specified for the tension components of the orthotropic box girders. The "T2" is specified to ensure adequate toughness (CVN Impacts) for improved fatigue performance.
- ASTM A709 Grade 50 for box girders, flat plate stiffeners, bearing stiffeners and shear keys.
- ASTM A709 Grade 36 for bearing shim plates, manhole door covers including stiffeners and hinges and latch components and ladders.
- ASTM A500 Grade B for structural tubing.

- ASTM A668 Class G with supplemental Charpy requirement for heavy wall cylindrical forging for shear keys.

- ASTM A252 Grade 3 (minimum yield strength of 45 ksi) for steel piles.

- ASTM A-325 for structural steel connections. All bolt heads shall be located on the exterior face of all exterior steel plates, including the top deck flange plate, bottom flange plate, exterior inclined web plates and end diaphragm plates.

- ASTM A-449 for masonry plate anchor bolts embedded in concrete.

- ASTM A-307 for miscellaneous anchor bolts unless noted otherwise.

- Hexagonal shaped reinforced concrete footings with reinforced concrete columns using the latest CALTRANS detailing methods.

Orthotropic Superstructure

Type Selection

Steel bridges were selected by CALTRANS to cross over the busy freeway to minimize travel delays or lane closures during bridge erection to commuters and highway traffic. Kaiser Engineers selected an orthotropic bridge with trapezoidal ribs. The unique 180-degree curve or horseshoe bend shape of the bridge made a closed cell structure the most economical shape to resist the torsional forces. The bridge utilized reinforced concrete "T" bents with a single column with spiral reinforcing ties. Two special bearings connect the superstructure to each "T" bent.

Special Issues

Orthotropic Deck Stiffness

Some of the details used on the "HS" Line were first utilized on the Golden Gate Bridge. Many welding details for the trapezoidal rib were repeated for the "HS" Line. The transverse flexural stiffness of the orthotropic steel deck is critical for the long-term behavior of the asphalt overlay. The weight-saving concerns, which guide the seismic design, dictate minimal overlay thickness. This leads to a compliant deck system, which is very sensitive to temperature and to local deflections imposed by concentrated wheel loads. The cyclic nature of this loading only increases the problem of compliance between overlay and steel underlayment. The long-term delamination danger of the overlay is greatly reduced by providing a uniform stiffening pattern with relatively closely spaced components, and a relatively stiff top deck plate.

"Orthotropic" is a combination of two words, orthogonal and isotropic

Fatigue

The over-all stress ranges under global loads are very low for this type of multicell box girder. The longitudinal continuity and the 180° in plane "curve" further minimize longitudinal tensile stress ranges. The torsional stress components are also low for these large "cell perimeters." The resultant equivalent stress intensity values (von Mises yield criterion) are very low.

The local orthotropic deck stress ranges generated by localized dynamic wheel loading are more critical for fatigue, particularly at points of greater restraint and stiffness, such as the intersection of deck plating with the longitudinal webs and the transverse diaphragms. Inclusion of the various stress raisers and consideration of the weld types still leads to excellent long term fatigue capacities.

Fracture Control Plan

The optimization of the safety and performance of the structure under minimal cost is the basic aim of a rational "fracture control plan." While buckling and general yield are considered in the basic design approach, special consideration is given to the danger of subcritical crack growth and unstable crack extension. Special attention, at the design stage and subsequently at the fabrication stage, is given to the factors affecting these fatigue-induced micro cracks.

The main elements of the fracture control plan considered are:

- the identification of the main tributary factors such as local loads, dynamic amplifications, stress risers, residual stresses, and two-dimensional states of "hydrostatic tensile stresses;"

- the establishment of the relative importance and contribution of each of these tributary factors;

- the determination of the various strategies in design and fabrication to mitigate the most important "fracture-causing" elements; and

- the recommendation of an optimal design and fabrication procedure, including choice of materials, quality control and inspection methodology.

The three primary factors affecting local failure and ensuing major damage, and hence the life span of a structure undergoing fatigue loading are:

- the tensile stress range amplitude;

- the material flaw or discontinuity sizes, which relates to quality control and assurance;

- the material toughness properties which means choice of specific steel.

Robert Colin, Caltrans

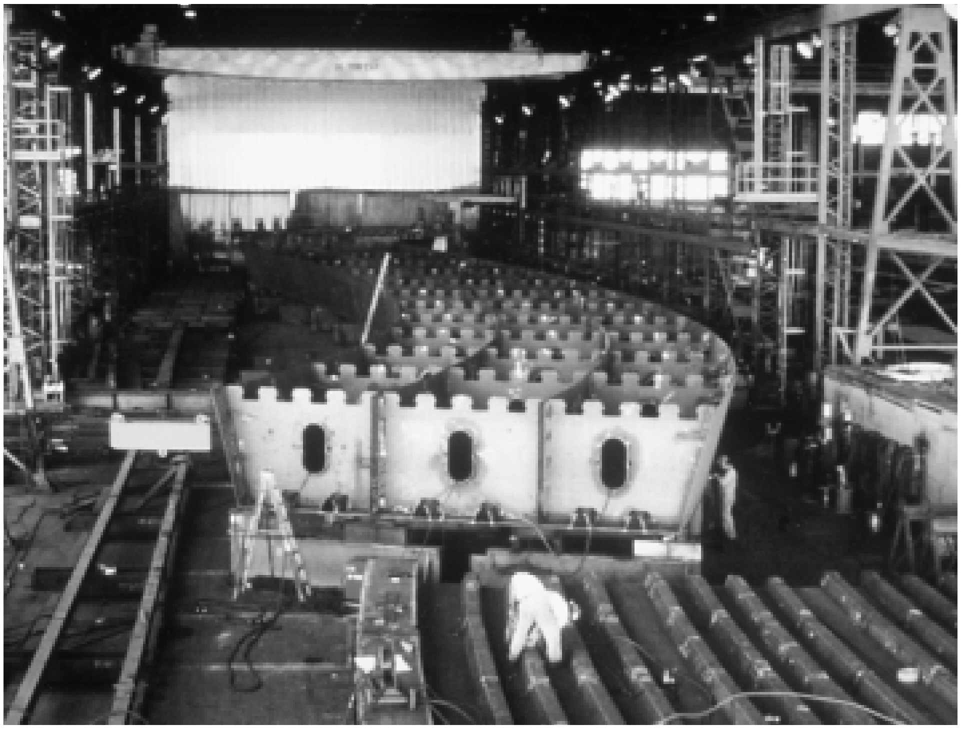

Figure 2. The unusually large components of the orthotropic box girder sections, shown in the fabricator's shop.

The design philosophy consists of providing for "minimal" stress raisers through a careful choice of details and weld types. Redundancy is used with caution to minimize global and local stress range amplitudes and also to provide multiple load paths. Crack arresters are considered in various fracture propagation scenarios.

Buckling

Web and flange local buckling are considered in a "limit load analysis" approach with given transverse imperfections. Indeed, it is essential to include actual fabrication conditions simulating imperfect geometry and welding residual stresses.

Shear Lag

The web "slenderness" leads to some shear lag effect which needs to be considered in the local fatigue, as well as in the local buckling capacity of the most critical web panels at midspan and near supports.

Analysis

Various task-specific programs were used to simulate the static and dynamic behavior of the bridge structure global static analysis.

Kaiser Engineers developed a global static model for the complete structure using the computer software "SAP-90". Dedicated pre- and post- processors allowed Kaiser Engineers' bridge designers to produce the governing

The design philosophy consists of providing for "minimal" stress raisers...

load combinations for service and for ultimate conditions for service, and for ultimate conditions corresponding to the standard LRFD approach, in accordance with AASHTO-NCHRP and CALTRANS policies.

The cross sectional properties were derived from local sectional models. The use of substructuring by means of the "super-element" technique was contemplated and subsequently abandoned because of the need for live loads applied on to every span and the development of envelope curves for "forces" and "displacements." The global dynamic model was developed with the main requirement of capturing 90 percent of the total modal mass participation in all three orthogonal directions.

Special attention was given to the expansion joints and to the foundations. Upper and lower bounds for foundation impedance's were considered. The pier columns were also considered under two distinct states, elastic uncracked and cracked as well.

The seismic analysis consists of the spectral response approach based on the CQC modal superposition. These linear analyses are performed for a site-specific "mud site" spectrum, corresponding to the maximum credible earthquake occurring on the closest fault with an 8.0 M magnitude and, alternatively, on the nearby fault with an 8.5 M magnitude.

The seismic design philosophy is a displacement-ductility driven approach, which permits controlled plastic straining to occur at the various pier foundation bases. Soil-structure interaction analyses were performed by means of the GROUP finite-difference program. The very deep mud layers were simulated under different material assumptions to detain meaningful sensitivity curves, and to verify the compliant pile behavior.

Dedicated non-linear ABAQUS finite element models were developed for local web and diaphragm buckling analyses for local wheel loading studies on the orthotropic deck. The steel impact barrier was studied by means of a dedicated non-linear ABAQUS model as well in order to simulate the

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ultimate energy absorbing capability and the impact load reaction applied to the main superstructure.

Seismic Detailing Features

The bridge has several unique seismic detailing features including the use of rubber dock fenders as seismic shock absorbers reducing forces between completed bridge sections. The rubber delta-shaped “dock fenders” were used to reduce kinetic energy occurring during a seismic event at the hinges in the superstructure. The systems were developed for this bridge by Kaiser Engineers and are unique to the CALTRANS.

Poly-tetra-fluoro-ethylene (PTFE) spherical bearings were used to allow for rotation and expansion of members. These bearings can resist higher lateral forces including seismic forces. A central shear key pipe was added for additional lateral capacity.

Fabrication of Orthotropic Bridge Sections

The unusually large dimensions and weight of the orthotropic box girders would limit their fabrication to only the largest fabricators (Figure 2). Both transverse and longitudinal bolted splice alternatives were included in the design to allow the fabrication of the girders in the smallest sections possible. The complexities of the welding and fit-up of component pieces required experienced steel bridge fabricators. Fabrication was subcontracted to Universal Structural Inc. of Vancouver, WA, an established steel bridge fabricator and a certified AISC category III shop. They had previously supplied fabricated steel components for many bridges owned by CALTRANS.

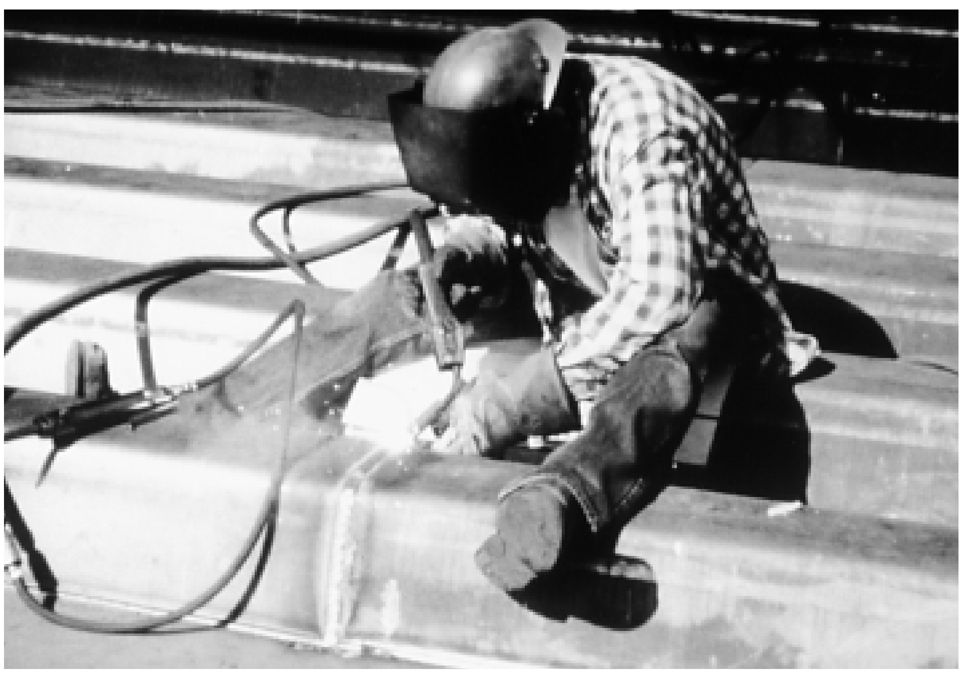
The large size and weight of individual girder components created some difficult handling and fit-up problems. This was further complicated due to the curved configuration of the girders. The top deck plates with the welded rib sections proved difficult to handle. Once all sub-assemblies were fitted together, this required that most welding be performed in the vertical and horizontal positions within the closed box sections. The girder sections were too large to turn to allow welding in the optimum flat and horizontal welding positions. Both shop and inspection personnel had to work in the interior confinement of the box sections using artificial lighting and utilizing the various manhole openings for access.

Universal Structural Inc. provided in-house quality control inspection and testing. This was augmented by the CALTRANS QA shop inspectors who provided continuous oversight during the period of fabrication and painting. All inspections and tests were performed in accordance with the requirements of CALTRANS standard specifications and added special provision requirements. With the exception of the added welding requirements for the rib-to-deck welds, welding was performed in accordance with the requirements of the AWS D1.5-88 Bridge Welding Code (Figure 3). Non-destructive testing was performed on all critical weld joints. The contractor fabricated thirteen full bridge width orthotropic sections 7'-0" (2.13 m) deep by 35'-6" (10.7 m) to 37'-6" (11.3 m) wide, with

All sections were shipped with a steel orthotropic deck...

lengths varying from 123 to 219 ft (37.5 to 66.75 m) per section. The sections ranged in weight from 250 tons (227 m tons) to a maximum of 459 tons (416 m tons). All sections were shipped with a steel orthotropic deck and with the installed steel barrier rails. The total weight of all fabricated steel equaled 5,014 tons (4,548 m tons).

All the steel plate for the superstructure was manufactured in the USA by Bethlehem Steel Corporation Burns



Robert Colin, Caltrans

Figure 3. Semi-automatic welding was needed to provide continuity at ends of ribs fabricated in 20 ft (6 m) long straight segments tangent to radius.

Robert Colin, Caltrans

Figure 4. Special heavy-lift hydraulic platforms (SHLHP) pull a 450-ton (408 m ton) all-steel orthotropic section across freeway lanes.

Harbor Indiana Division. The combination of their 110" and 160" (2,794 mm and 4,064 mm) plate mills were capable of the full range of plate sizes required for this bridge. The bridge sections were fabricated by and barged down in thirteen sections to a docking area about a mile (1.6 km) from the bridge site.

Dramatic Midnight Erection Process

The bidders were given the option to fabricate and erect the bridge in sections that would allow the maximum number of bidders and fabricators. A minimum section size was specified on the contract plans. The smaller the sections, the more field bolting that would be required. The smaller pieces, however, allowed smaller steel fabricators to bid on the project. The successful low-bidder was a general contracting joint venture of Kiewit-Marmolejo. The steel subcontractor Shaughnessy and Company with Crowley Maritime Services Inc. was responsible to supply the equipment and trained operators to lift and position all the steel sections. Kiewit-Marmolejo's assignment was to assist with the jacking, do the traffic control coordination with the CALTRANS, and perform all of the work associated with making the splices and preparing the bearings. A creative solution for the installation was conceived using a special heavy-lift hydraulic platform (SHLHP), which consists of two self-propelled hydraulic platforms braced in tandem with a strut beam. Universal Structural Inc. had the capacity to build the thirteen full bridge width sections as one welded unit "arc" shaped pieces. A scale model was built to try out the various methods to pre-plan the most effective use of the SHLHP.

Closure of 50 percent of the freeway lanes was required for setting three of the thirteen sections

The first three of thirteen bridge sections were loaded on a 20-line Scheuerle platform trailer. A transition ramp was placed between the barge and the Universal Structural Inc.'s dock to load the sections onto the barge. Three barge trips were conducted using a 400 ft (122 m) long by 100 ft (30.5 m) wide barge, one of the largest available in the United States.

A 20-line, two-file Scheuerle hydraulic trailer with a custom-designed spacer frame was used to off-load the girders at the docking facilities near the bridge site. The frame between the axles distributed the load over a wider path to meet structural requirements on the dock. The Scheuerle hydraulic trailers have two unique features. First, each pair of tires can be individually steered. Second, each pair has a hydraulic cylinder that allows the pair of tires to move up and down (over uneven surfaces) to a maximum of 18 in. (457 mm), while maintaining uniform loads across every pair of tires.

The sections were set on temporary cribbing in the staging area. The off-load sequence and staging locations required careful planning since the sections could not pass each other in the staging area once they were off-loaded. Therefore, each section was moved three times: first from the fabrication facility to the barge; second from the barge to the staging area beside the freeway; and finally erected.

Closure of 50 percent of the freeway lanes (at one time) was required for setting three of the thirteen sections. Approximately 500,000 vehicles cross below the Maritime Off-Ramp each day. The sections were staged on the east side of the freeway and crossed over during night erection. This could only take place in a ten-hour window beginning at midnight on a Saturday night. A stiff financial fine was stipulated for each minute that the contractor exceeded this time limit. This fine would be paid to CALTRANS. The first section

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Robert Colin, Caltrans

Figure 5. Two welders each field-weld a top bearing plate to connect the bottom of the section to the previously installed bearing.

was erected at the west abutment and allowed the team to practice for the erection over the freeway. This gave Kiewit-Marmolejo, CALTRANS and The California Highway Patrol confidence that things would go according to plan. The hydraulics of the "SHLHP" worked smoothly to position the section and lower it onto the steel bearings anchored to the top of the concrete pier. The section over the westbound lanes was erected about midnight on a Saturday. The section over the eastbound freeway was erected the following Saturday night.

The final transport vehicle for the sections, the SHLHP consisted of 24 lines of Scheuerle hydraulic trailers, which were assembled into two six-line, four-file platforms (Figure 4). Two hydraulic support towers on an adjustable frame were mounted on hydraulic powered sliding turntables. The SHLHP's ability to make various motions allowed the section to move on six axes.

Surveyors called out offsets as the SHLHP's six degrees of hydraulic-activated controls aligned the boltholes and set the other end for grade and station. Before the first bolt could be secured, all holes had to be aligned. The bolting process began less than two hours from the start of the operation and was completed in another three hours. The section's placement and bolting procedure totaled six hours, which was well under the ten-hour minimum. Once 25 percent of the bolts were installed plus the field welding of the section to the bearings (Figure 5), then the dead weight of the section was transferred to the splice and bearings. Finally the SHLHP was driven away. The SHLHP have been used to hoist eleven-story structures into place at oil fields. "This is not a test," the erector's manager said. "It's actually something we've done many times." The sections were field bolted together. "Some pieces weigh up to 500 tons (453 m tons)," said the erector's manager. The remaining ten sections were not located over freeway lanes and were erected during the daytime. After the thirteen sections were erected, the joint seal assemblies were installed and an epoxy asphalt overlay wearing was placed prior to opening the bridge to traffic.

Lessons Learned

- Pile "load indicator program" eliminated piling driving claims and established pile tip elevations for seismic loading. Largest pipe piling used was 42 in. (1067 mm) diameter.

- Although the orthotropic steel bridge system is complex, it is a buildable system. An orthotropic superstructure is under fabrication for the Carquinez Suspension Bridge to replace the 1924 Carquinez Cantilever Truss Bridge. The orthotropic superstructure is currently under final design by the East Spans replacement for San Francisco-Oakland Bay Bridge design team. This self-anchoring bridge will be about a mile (1.6 km) west of the Maritime Off-Ramp.

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Lessons Learned in the Field

by Omer W. Blodgett, Sc.D., P.E.

Don’t Design with Your Heart

Introduction

My introduction to welding and the field of welded design really began in my childhood. My grandfather owned twenty-three wooden ships, steamboats and barges on the Great Lakes. They carried salt and limestone up the lakes, and pulp wood down the lakes to the paper mills. In 1917, the year I was born, my grandfather purchased a 200 amp Lincoln welder. Ten years later, I learned to weld using that machine.

Our last ship burned in 1931, putting us out of business. It was the Great Depression, so in our home town of Duluth, Minnesota, we started a welding shop using that Lincoln welder. My brother, my father and I all welded in the shop, and my mother kept the books. Our work in the shop provided a variety of experience, and soon I was welding on steel structures. I became an iron worker, joining local No. 563 of the International Association of Bridge Structure and Ornamental Iron Workers in Duluth. Later, I graduated from the University of Minnesota with the degree Bachelor of Metallurgical Engineering in 1941.

By the beginning of World War II, I had become welding superintendent at the Globe Shipbuilding Company in Superior, Wisconsin. From 1941 to 1945, we built and delivered twenty-nine all-welded oceangoing vessels for the U.S. Maritime Commission. From 1931 to 1945, the use of welding electrode in the United States increased almost one hundred-fold.

In 1945, I wrote an article for the Globe Shipbuilding Company newsletter in which I predicted “a far greater use of welding than anything which we can now imagine.” However, the writers and codes and structural specifications were unfortunately not aware of my predictions! For example, prior to 1953, the AASHTO Specifications for Highway Bridges listed only 13 places where welding could be used on a steel bridge; a welded plate girder was not among them. Such oversights—and there were many more of this kind—undermined the true potential of welding technology.

Welding is not a fastener—it is a method of design

At *Welding Innovation,* we decided to develop this column as a forum to discuss and illuminate key principles of design that are not commonly taught in the classroom. I do hope that our readers will contribute their own “lessons learned in the field.”

In all of the examples I will cite in this column, one cardinal truth overrides every other: if the engineer makes the mistake of considering welding to be just another type of fastener, alongside such fasteners as rivets and bolts, the item or structure as designed will fall far short of its potential capabilities. Welding is not a fastener; it is a method of design which, properly used, takes full advantage of the versatility of the material.

Don’t Design with Your Heart

What do I mean by a statement like “Don’t design with your heart?” Well, all too often, before taking the time to rationally think through a problem, engineers make assumptions based on past experiences. These assumptions may or may not be applicable to a given circumstance. Although my illustrations of this lesson are not structural examples, the basic design principles I will discuss can (and should!) be applied to structural design.

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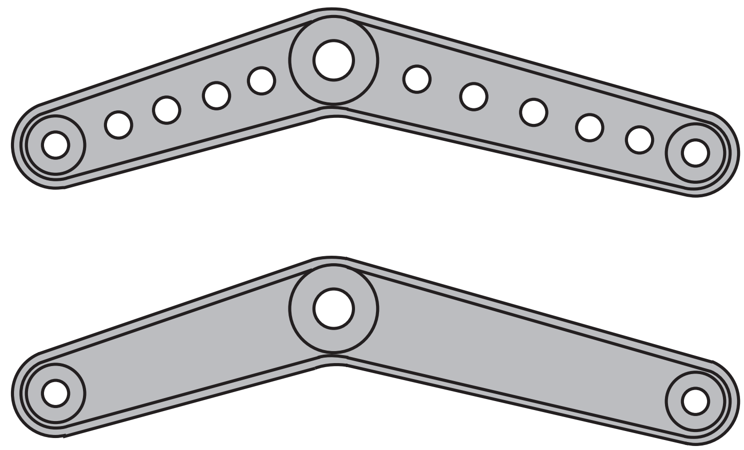

A salmon canning plant was having trouble with a cast steel lever that put the tops on the cans. When the lever operated rapidly, inertia forces (F = ma) were created, causing deflection and putting the lever out of alignment. An engineer (thinking with his heart) immediately had the idea of making the lever out of aluminum, which has one-third of the density of steel, in order to reduce the mass by one-third, thus reducing the inertial forces. This was a good idea, but would not have alleviated the deflection problem.

To solve the problem the variables that influence deflection must be studied. The following equation defines the lever deflection as a function of the material properties and cross-section.

$$\Delta = \frac{a \ \delta \ A \ L^4}{3 \ E \ I}$$

…where E/δ is the property of the material, and I/A = r^2 is the property of the section.

When the engineer switches to aluminum, he changes the density to one-third that of steel, but he fails to realize that the modulus also changes to one-third that of steel. Remember that for structural metals such as steel, stainless steel, aluminum, magnesium and titanium, the modulus of elasticity is proportional to the density. So although this solution has reduced the density by one-third, it has also reduced the stiffness property by one-third—in other words, nothing has been accomplished. The engineer is right back where he started. The solution will never be found by changing the material, but only by focusing on the geometry of the cross-section.

We can't pick just the qualities we want and throw the rest away

The design solution to this problem will be found by maximizing I over A, where I = r^2A. So the designer must maximize "r," which is the radius of gyration. Increasing the radius of gyration can be accomplished one of two ways: by putting more material out away from the neutral axis, or by subtracting material near the neutral axis. The latter can be achieved by drilling holes in the lever, as shown in the drawing. This simultaneously decreases the area (and subsequently the mass) and the moment of inertia, while increasing the radius of gyration. Even though the moment of inertia is reduced, the part will be slightly more rigid in bending because the rate of decrease for the moment of inertia is less that that of the area.

Opportunities

SAC Seismic Publications Now Available from FEMA

The results of the six-year research and development effort conducted by SAC following the 1994 Northridge earthquake have been published in four volumes by the Federal Emergency Management Agency (FEMA). In addition, a CD-ROM to be made available in the future will contain the Background Reports, which include the results of the various investigations that were conducted in order to develop the recommendations.

The SAC Joint Venture is a partnership of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREe). The following publications are available *free of charge* by calling the FEMA hotline at 800/480-2520:

FEMA 350 Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings

FEMA 351 Recommended Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings

FEMA 352 Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings

FEMA 353 Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications

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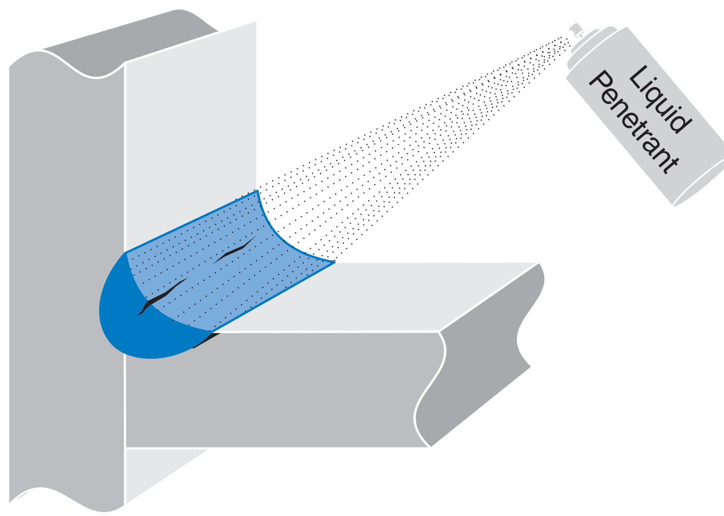
Nondestructive Inspection (NDI) and Responsibility

By Hardy H. Campbell III
Senior Staff Engineer
American Welding Society
Miami, Florida



Liquid penetrant inspection

This article was published in Inspection Trends, *a publication of The American Welding Society, and is reprinted here with permission.*

Definition of Terms

Nondestructive Inspection (NDI)–any technology used to determine the homogeneity of a weldment without affecting that weldment's physical properties. In this article, the primary focus will be on magnetic particle (MT), liquid penetrant (PT), ultrasonic (UT) and radiographic (RT) inspection techniques.

Engineer—the individual primarily responsible for the design of a welded assembly. In this article, the capitalized term "Engineer" will specifically denote the individual acting on behalf of the project client to make all decisions that modify the original design or approve changes proposed by the Fabricator, or accept the Inspector's report on NDI results.

Fabricator—refers to the contracting company that performs the welded fabrication; the term may used for either shop or site welding operations.

Inspector—refers to the individual responsible for supervision of the NDI operations, and who will communicate with the Fabricator and the Engineer about the results of the NDI.

What Engineers Need to Know about NDI

Engineers typically graduate from school with little knowledge of the sometimes bewildering science involved with NDI, which sometimes seems more akin to voodoo magic than classical physics. Therefore, they have a limited understanding of the strengths and weaknesses associated with each method. All too frequently, engineers use wishful thinking to equate methods, for example, considering UT and RT to be essentially identical.

In addition to their lack of knowledge about NDI, engineers without a great deal of experience in welding may believe that all welds, if made in accordance with welding standards, will provide the same level of integrity. Therefore, if a weld is inspected only by unaided methods and is deemed acceptable, it would be just as sound if examined by a subsurface NDI method, such as UT. The problem, of course, lies with the definition of integrity and with the lack of understanding not only of NDI, but also of the economics and business of fabricating metal.

To an engineer, "integrity" means the ability of a weld to perform its service function. But can NDI predict that? Indeed, how can anyone have confidence in that definition unless the weld is actually put into service, well after fabrication and inspection have been completed?

So, we must conclude that we can only use an indirect method to determine "integrity." NDI provides this by letting us know the extent to which a weld has failed to achieve homogeneity with the base metal being connected; i.e., the degree to which the weld and base metal have failed to form a continuous, uninterrupted mass, free of internal (e.g., porosity) and external (e.g., undercut) flaws.

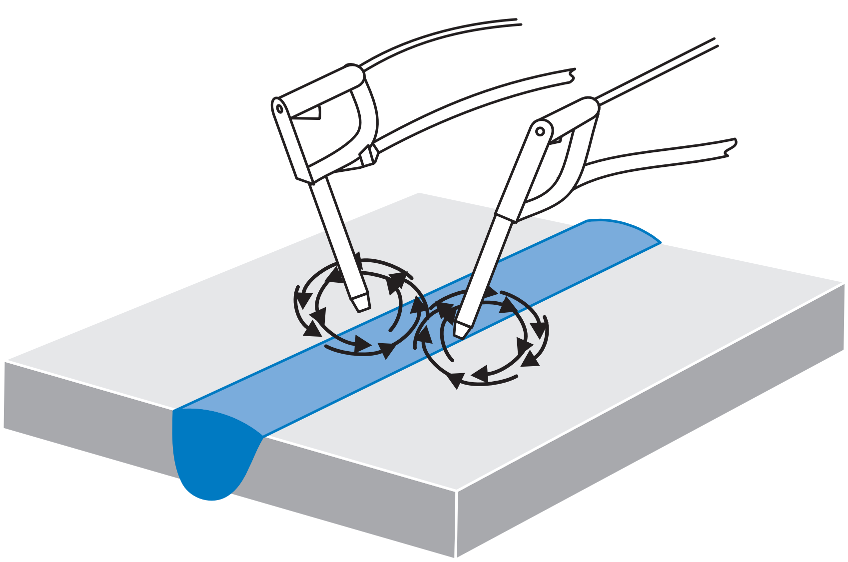
Although the achievement of a "perfect" weld is ideal, it is not necessary in most circumstances in order to provide service function. So welding specifications typically permit a variety of flaw types, sizes and frequencies. Some, such as porosity, have relatively little effect on load-resisting adequacy; others, such as lack-of-fusion, can have a significant impact.

Fabricators can control flaws, but the extent of this control will depend on the contract requirements. The knowledgeable Engineer, familiar with the fabrication business, will realize that the way a contract is bid will materially affect the way a Fabricator tries to minimize these internal and external flaws. If welds are only to be inspected with surface inspection methods, whether unaided visual or MT or PT, Fabricators will take the measures necessary to ensure that the surface satisfies these visual or NDI acceptance criteria. This may include employing experienced welders or using small electrodes for the root and capping passes, as well as careful grinding and/or the use of GTAW toe remelting.

However, the fill passes may not receive the same degree of cleaning, or less experienced welders may be used on the project. This should not be interpreted to mean that Fabricators are trying to hide anything,

NDI sometimes seems more akin to voodoo magic than classical physics

but simply that they are making optimum use of time and available resources to comply with the weld quality standard involved. The use of qualified or prequalified weld procedures, in addition to the use of qualified welders, will minimize the size and frequency of subsurface flaws. However, without a method to investigate the internal volume of a weld, confidence that major flaws have been avoided cannot be high.



Magnetic particle inspection

The knowledgeable Engineer will know that if he or she wants a subsurface quality standard, the contract should call for a subsurface method, at least for critical connections or some randomly selected percentage of total welds. This will ensure that the Fabricator devotes special attention to these welds, employing the best welders, and taking extra care in preparing joints prior to welding and in cleaning the weld while welding is in progress.

Naturally, this extra level of precaution will cost the Engineer's client more, but it will provide the added confidence that the weldment has achieved the necessary degree of homogeneity.

All too often, though, the Engineer specifies a cheaper visual or surface NDI method in the contract documents, but expects a more expensive NDI weld quality. This is sort of like buying a Volkswagen Beetle and expecting it to win a Grand Prix race. Naturally, the consequences of this unrealistic expectation can be nasty and expensive.

In the United States, there is a provision in the AWS D1.1 Structural Welding Code for Steel that attempts to address this issue. The provision states that, if the client wants additional NDI other than the contracted visual inspection, the client must absorb all of the subsequent costs, *including* the cost of repair for flaws that are outside the acceptance levels for subsurface NDI. This appears to be reasonable; after all, the client should have specified this NDI in the original bid documents, which would have allowed the Fabricator to devoted more stringent quality control standards to the welds in question. If these welds fail to live up to this new, higher standard, the Fabricator can hardly be blamed.

However, the D1.1 Code adds another clause, which requires the Fabricator to absorb all the costs if this new NDI reveals "an attempt to defraud" or

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"gross nonconformance." This last clause prompts all kinds of arguments about what these words mean. Naturally, the client, who expects sound work, may argue that *any* degree of nonconformance with the higher NDI flaw acceptance criteria is "gross" or "an attempt to defraud."

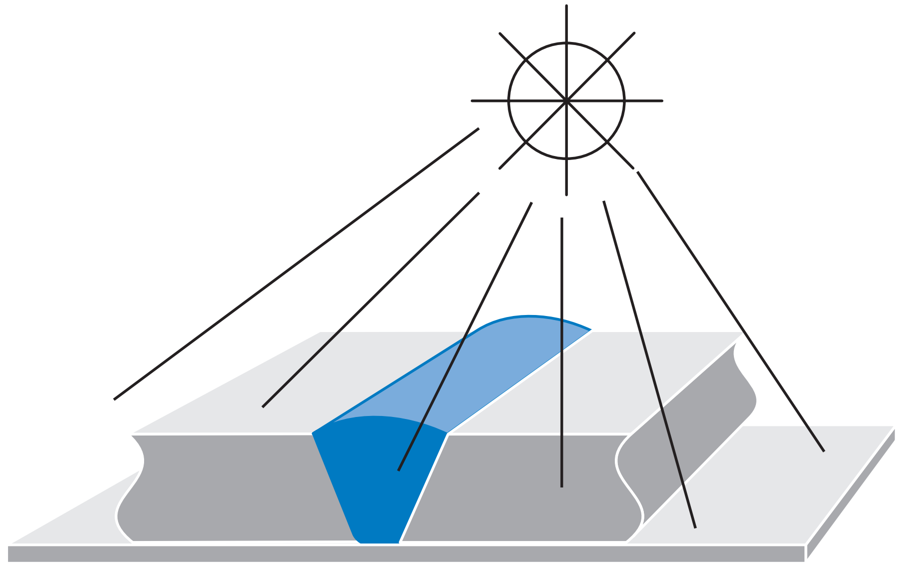
This may be a disingenuous argument, if the Engineer acting on the client's behalf is truly cognizant of the differences in NDI capabilities. However, more often than not, the Engineer is ignorant of these limitations and is apt to use the following logic.

"The Code says that if I visually inspect a weld, it's acceptable if it satisfies the visual flaw standard. The Code also says that if I use subsurface NDI on a weld, it's acceptable if it satisfies the NDI flaw standard. Therefore, both welds being acceptable must mean that both flaw standards are equivalent. So if I specify a visual standard only, I should get an NDI standard too. And if I ask for NDI and don't get it, it must mean that the weld is grossly nonconforming."

"Integrity" means the ability of a weld to perform its service function...

When I get calls in my office in Miami on this subject, I suggest that the two disputing parties agree to a mutually satisfying settlement, because neither the Code nor D1 Structural Welding Committee will ever attempt to define what is "an attempt to defraud" or "gross nonconformance."

I also suggest that future imbroglios can be avoided if some percentage of random subsurface NDI is specified in the contract, just to keep the Fabricator vigilant and ensure that extra care is devoted to critical joints.



Radiographic inspection

It may be said that such an approach could also invite abuse–why not do this all the time, and make sure that "X-ray quality" welds are made all the time? Engineers must realize that not all welds require the same level of attention in order to adequately perform their intended service function. Trying to make all welds achieve the highest quality standard only unnecessarily drives up the cost of fabrication. Naturally, the Engineer is most concerned about his or her interests, such as protection from litigation, but judgment is needed in order to provide economy as well as integrity for all the interests involved. The central question becomes, "For what weld types are the various NDI methods appropriate?" The generic answer is that this depends on both service function and weld geometry. The service function will involve issues such as criticality of the connection (i.e., consequences of load-carrying failure), redundancy (alternative load paths), loading type (e.g., cyclic tension, static compression), stress level (e.g., sub-yield), and brittle fracture resistance (is toughness specified?).

The geometry of a weld will determine the effectiveness and practicality of using some NDI methods. Some weld types are thus more conducive to reliable flaw detection than others.

Groove welds can be inspected with either UT or RT. However, partial penetration groove welds will always provide flaw-like indications or images at the unfused weld roots. Unless proper NDI procedures are written, such "false" flaws can result in high reject rates. Fillet weld geometry creates reflective problems with UT and image problems with RT. However, the use of partial penetration groove and fillet welds is usually limited to non-fatigue, non-tensile or low tensile applications, so that the need for confidence in subsurface quality is not great. The concern is usually restricted to detecting surface flaws, and so MT or PT are the preferred NDI techniques.

Likewise, engineers should specify to what extent welds of a particular type need to be inspected. Logically, welds that are subject to low stresses, or are in compression, or in highly redundant structures, could have less coverage than welds subject to high tensile stresses in low redundancy structures. It is also important to consider the advantages of automated NDI over purely operator-controlled NDI; generally, NDI operated by humans is considered less reliable than automated, computerized NDI.

Recommendations

Weld and Load Types Appropriate for MT or PT Inspection

1. Fillet welds.

2. Complete or partial penetration groove welds in compression or shear.

3. Complete or partial penetration groove welds in low static tension or fatigue loads and structures with low criticality and/or high redundancy.

4. Complete or partial penetration groove welds in high static tension or fatigue loads and structures with high criticality and/or low redundancy.

Recommended Test Frequency – Random testing for substantially less than the total weld length.

Weld and Load Types Appropriate for UT or RT Inspection

1. Complete penetration groove welds in high static tension or fatigue loads and structures with high criticality and/or low redundancy.

2. Complete penetration groove welds in low static tension or fatigue and structures with high criticality with low redundancy and materials with low or unspecified toughness.

Recommended Test Frequency – 100% manual inspection of welds or some random percentage for automated NDI.

Additional Comments for Engineers

This is not to say that either list of weld or load types is definitive, but this listing serves to highlight typical industry practice. Notice that MT or PT would be applied to complete penetration groove welds, even if these would also justify UT or RT. This is because such welds, when subject to high tensile loads in critical applications, would need to be checked for potential crack starters on the surface, which UT in particular can have trouble detecting.

Many welding standards will use different flaw acceptance limits for different load types, so it is imperative that this information be conveyed to the Fabricators and Inspectors (the latter may work for the Fabricator or the Engineer). This can be done in a number of ways, but probably the most direct method is by indicating on the design drawings whether a connection is subject to shear, compression or tension. It must be understood that this does not refer to the load in the weld proper, but the load in the attaching members. This is an important distinction to make for fillet welds, which are always considered to be loaded in shear.

The Engineer's job doesn't stop there, of course. If an Inspector reports unacceptable flaws to the Fabricator, two options exist:

1. Fix it; or

2. Get the Engineer to accept the in-situ flaw.

Most Engineers will opt for A, but often conditions will warrant exploring B.

This will require that the Engineer evaluate structural integrity based on a flaw's existence. Such fitness-for-purpose evaluations frequently require fracture-mechanics-based analyses, which, in turn, require information about 1.) service stress, 2.) flaw dimensions, location and orientation, and 3.) material toughness. For the purposes of this article, only item 2 is relevant, highlighting the question, "How confident is the Engineer of the flaw parameters reported by NDI?"

The use of ultrasonic energy to detect flaws is a tried and true method that does have drawbacks, the biggest being its reliability in flaw detection. There are many horror stories about UT detecting rejectable flaws, only to discover upon excavation that the flaw was acceptable or nonexistent. This may undermine confidence in UT to some degree, because it is also true that rejectable flaws may also go undetected. The reasons for these false alarms or missed flaws can vary from operator error to poor calibration to malfunctioning equipment. But as long as we understand that *no* NDI method is guaranteed to detect 100% of all rejectable flaws, we can appreciate the wisdom of requiring safety factors, redundant designs and specified toughness in critical connections. These all assist in bolstering our confidence in overall performance.

Too often, the Engineer specifies a cheaper visual or surface NDI method...but expects a more expensive NDI weld quality

Various welding standards use different approaches to UT, each with its inherent limitation to accurately size flaws. UT that uses beam reflection amplitude only can undersize flaws

Ultrasonic inspection

significantly, while UT that employs beam-boundary techniques is somewhat more reliable, but not foolproof. Even more sophisticated techniques, such as Time-of-Flight-Diffraction method, do not provide a guarantee of accuracy, so Engineers must be prudent when using NDI-derived flaw dimensions.

RT is, of course, an indirect, visual method of detecting and sizing flaws, but its weakness is flaw orientation—planar flaws normal to radiation may go undetected. Naturally, planar flaws, such as lack of fusion or penetration,

Many welding standards will use different flaw acceptance limits for different load types...

are the more significant ones from a fracture-mechanics standpoint; for this reason UT is preferred, since its strength is detecting planar flaws normal to acoustic energy. Additionally, UT is cheaper than RT and does not involve safety-hazard issues. For these reasons, RT is not generally preferred for structural work, though many contracts will specify RT in addition to UT for critical connections.

What Fabricators Need to Know about NDI

Fabricators fabricate. That statement may be a tautology, but it emphasizes their primary focus, which is not design or inspection per se. They are expected to deliver a product that meets the standard of quality required in their contract with the Engineer's client. If they employ NDI, it is to verify that that standard has been achieved. Additionally, the client may have a third party Inspector peering over the Fabricator's shoulder during fabrication to provide an extra level of confidence that the required quality is being achieved.

But Fabricators have to be aware of their obligations vis-à-vis NDI. With reference to the situation described above, should a Fabricator who knows that a critical structure is being bid for visual inspection only remain silent at the bidding stage, and anticipate that the client may get wise and require NDI later?

Such anticipation may require going to the Engineer and pointing out the wisdom of specifying NDI, but Fabricators may be reluctant to tell Engineers what their responsibilities are. It is a bit of a conundrum, but one that should be addressed at the bid stage, rather than risking a potentially nasty, litigious struggle ahead if "gross nonconformance" is discovered.

Fabricators also need to be aware of how NDI can affect integrity. For example, using prods for MT can cause arc strikes on base metal. While the Fabricator may be diligent in removing arc strikes caused by welding, the same concern may not be shown for prod-induced strikes. Both, however, can have potentially harmful hardening effects on steel.

The Fabricator also needs to be aware of the NDI complications that can arise from the selection of certain welding details. Frequently, Engineers will allow Fabricators to specify the type of detail, after indicating the design requirement of weld type (typically for complete penetration groove welds). For example, complete penetration groove welds made with steel backing that must be ultrasonically inspected will produce a large number of false indications arising from the small air gap between the unfused portion of the backing and the base metal. Too often, inexperienced UT operators will report these as rejectable flaws. This

The most important aspect of Fabricator responsibility lies in the area of quality control

can be avoided if removable nonfusible backing is used, or a backgouged two-sided weld is employed, but it behooves all parties to be aware of the potential NDI problems associated with a selected detail.

Other problems associated with NDI include the residual magnetism produced by MT, which may affect subsequent welding or even the service function of the part. Appropriate degaussing techniques may have to be implemented.

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The most important aspect of Fabricator responsibility lies in the area of quality control; i.e., the methods used to ensure conformance with a given quality standard in practice. Quality control adds another layer to an Engineer's confidence that, even if some major flaws go undetected, the overall system has been soundly fabricated and the consequences of localized substandard quality are minimal. Naturally, this does impose on the Engineer the need to verify in some fashion that a quality control system is in place and functioning.

Inspectors should resist the temptation to supplant the Engineer in accepting or rejecting detected flaws

The use of quality documentation systems such as ISO 9001 and its myriad variants may offer some security in this regard, but Engineers should be wary of relying exclusively on paper empires. It is the actual implementation of written quality systems that determines effectiveness, and for this reason, third party inspection teams are frequently used.

Inspectors and Their Responsibilities

NDI personnel have education, training and experience in the esoterica of their craft. They know the basic physics and fundamental topics associated with the various methods. They will work according to procedures established by supervisory personnel to adequately examine the prescribed weldments. And it is these supervisory personnel, who will typically be the Inspector or people who report to the Inspector, who will need to liaise closely with Engineers and Fabricators.

As mentioned earlier, fused steel backing can become a source of disagreement as the result of the air gap interface. The good Inspector realizes the potential for this to happen and writes a procedure for the NDI personnel that does not reject these indications but instead predicts them. The Inspector includes this information in the report to the Engineer, who may decide to accept these indications as innocuous, or insist on removal of the backing and re-inspection, or propose some other alternative.

Similarly, if an Inspector sees that partial penetration groove welds are to be subjected to UT, a procedure must be developed that recognizes the reflectors at the not-fully-penetrated weld root so that these are not grounds for automatic rejection.

Inspectors must insist on receiving information regarding the tensile or compressive nature of the loads on inspected welds when the applicable standards require this. Frequently, Engineers fail to provide this information, and the Inspector is tempted to assume a load-type rather than seek the Engineer's input; this should be avoided.

Inspectors should liaise with Fabricators before fabrication or erection even begins in order to determine the accessibility of welds during the construction process. This is particularly true for welds that will be enclosed after final assembly is complete. Sometimes artificial access, such as plate cut-outs, have to be provided for tight and congested areas. For details that are to be radiographed, it is particularly important for a schedule to be established in order to minimize the safety hazards.

Inspectors should also resist the temptation to supplant the Engineer in accepting or rejecting detected flaws. Inspectors are primarily reporters of such information, who should of course provide their assessment of acceptability to the Fabricator. However, the Fabricator is free to pursue Option B as described earlier (persuading the Engineer to accept the in-situ flaw) and should not feel hamstrung by an Inspector's opinion. The Engineer should be the final arbiter of any disputes that arise.

What Do We Get Out of NDI?

Can NDI, in and of itself, guarantee anything about a product? Can it make up for inadequate design? Can it even detect all aspects of poor construction? The answer to all three questions is *"No."* What NDI can do is enhance overall confidence in the product's capacity to fulfill its intended function, but it is only part of a process that involves design and construction quality control.

I have used the word "confidence" throughout this article, because this unquantifiable emotion is the very basis for all the conservative design, construction and inspection standards we use. Compliance with all of these cannot guarantee performance, but each project phase can add layers of confidence that the end objective, in-situ service performance, can be achieved. NDI is the last and perhaps the most visible layer, but cannot stand in isolation from design and quality construction. Only when those responsible for all three project disciplines cooperate and communicate effectively can maximum confidence be obtained.

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Design File

Fillet Welds That Are "Too Long"

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

When fillet welds exceed a certain leg size to length ratio, and when such welds are "end loaded," they can become "too long." That is, the added length may not add strength that is proportional to the increase in length. This situation rarely occurs, as will be seen, but the designer should be aware of when it occurs, why the capacity is diminished, and how to mitigate the effects.

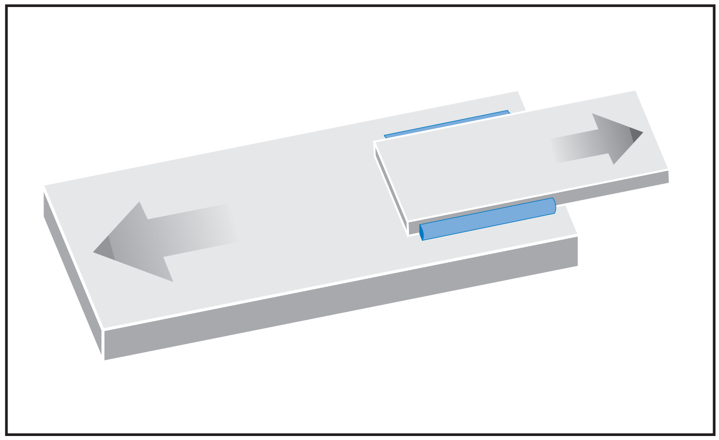

Figure 1.

"End loaded" applies to connections where the load is transferred to the end of a weld. Figure 1 illustrates one such example. Many lap joints with longitudinal welds would have end loaded fillet welds, as would bearing stiffeners. Welds subject to shear loading due to bending forces, such as those shown in Figure 2, are not included in end loaded applications. In addition, transversely loaded welds are not considered end loaded.

The distribution of stress at the end of welds, such as the one shown in Figure 1, is far from uniform. The relative stiffness of the weld versus the two lapped members may be significantly different. Shear lag further complicates the stress distribution. Due to these factors, and perhaps others as well, the full length of the weld may not be uniformly loaded. At some length, it becomes unconservative to assume the full length of the weld is equally effective in transferring stress. For the purposes of this article, it is at that point that the weld is considered to be "too long."

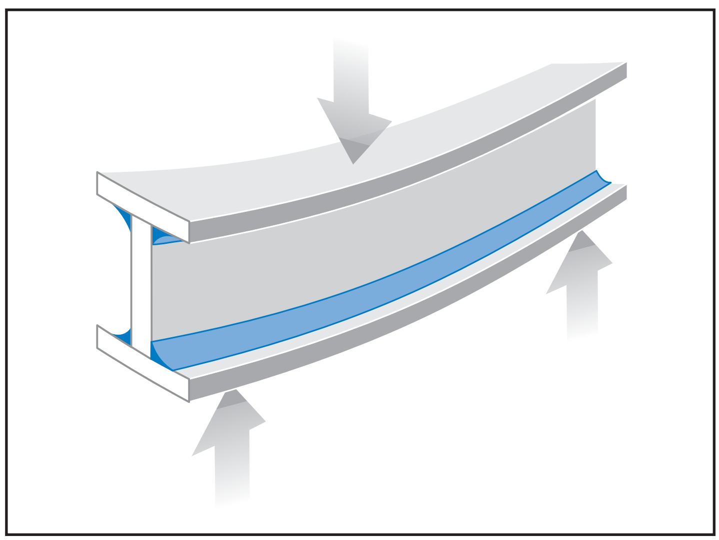

Figure 2.

Based on experience and research, a ratio of the weld leg size to weld length has been determined to be a critical factor in determining the effective length. When this ratio is 100 or less, the entire length can be considered effective. Thus, ¼ in. (6 mm) welds less than 25 in. (600 mm) long, and 3/8 in. (10 mm) welds less than 37.5 in. (1000 mm) long are no problem and can be treated in the conventional manner. Therefore, for many applications, concern about welds that are "too long" will not occur due to practical considerations.

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For longer welds, however, the additional length may not be proportionally stronger. To address this, the AISC LRFD 2000 Specification has added an equation to calculate a β (beta) factor, which reduces the effective weld length as follows:

$$\beta = 1.2 - 0.002\ (L/w) < 1.0$$

$$L_{eff} = \beta \times L$$

where,

β = length reduction factor

L = actual length of end-loaded weld, in. (mm)

w = weld leg size, in. (mm)

L_{eff} = effective length, in. (mm).

When the length of the weld exceeds 300 times the leg size, the value of β shall be taken as 0.60.

Consider a weld with a w/L ratio of 200: a ¼ in. (6 mm) fillet weld that is 50 in. (1200 mm) long. Beta is 0.8 in this example, and the effective length is reduced to 40 in. (960 mm).

Note for w/L less than 100, the equation would generate an invalid value of β that is greater than 1.0.

Once w/L is greater than 300, β remains fixed at 0.6, according to the above equation.

Table 1 summarizes key issues surrounding the leg size to weld length ratio. Columns 2 and 3 simply show the 100w and 300w values for the different weld sizes. Welds less than 100w are never "too long" and β = 1.0. Welds that are longer than 300w will have their length adjusted by β = 0.6. Between these two values, the simple equation shown above must be used.

In the design process, before the weld size or length is determined, the load transferred through the connection is calculated. Then, the corresponding weld length and size is determined for the electrode strength classification that will be used. Columns 4 and 5 show the maximum load that can be end loaded on a fillet weld of length 100w, assuming the use of an E70 (E48) electrode. Column 4 assumes the unusual case where only one fillet weld is involved, while Column 5 considers the more typical situation where a pair of welds is involved.

Columns 6 and 7 examine the applications of the equation described above in yet another manner; that is, by considering the size of the connected materials. Assuming the use of a 50 ksi (350 MPa) steel, and a maximum allowable stress of 60% of yield, Column 6 provides the maximum cross sectional area of the connected material that can be joined by one fillet weld of 100w length. Column 7 provides the same data for a pair of such fillet welds.

Careful examination of the data in this table demonstrates that the need to consider an adjustment on the weld length will not arise often. The 300w ratio will only occur in very unique circumstances. Nevertheless, the designer should be aware of the situations where the weld is "too long" and adjust the effective length in accordance with the equation shown above.

ENGLISH

Weld Size, w in.	Critical Length, in. 100w	300w	Capacity, kips 1 weld	2 welds	Member Size, in^2 1 weld	2 welds
1/16	6.3	18.8	5.8	11.6	0.2	0.4
1/8	12.5	37.5	23.2	46.4	0.8	1.5
3/16	18.8	56.3	52.2	104.3	1.7	3.5
1/4	25.0	75.0	92.8	185.5	3.1	6.2
5/16	31.3	93.8	144.9	289.8	4.8	9.7
3/8	37.5	112.5	208.7	417.4	7.0	13.9
1/2	50.0	150.0	371.0	742.0	12.4	24.7
5/8	62.5	187.5	579.7	1,159.4	19.3	38.7
3/4	75.0	225.0	834.8	1,669.5	27.8	55.7
7/8	87.5	262.5	1,136.2	2,272.4	37.9	75.8
1	100.0	300.0	1,484.0	2,968.0	49.5	99.0

METRIC

Weld Size, w mm	Critical Length, mm 100w	300w	Capacity, kN 1 weld	2 welds	Member Size, mm^2 1 weld	2 welds
2	200	600	40.7	81.4	0.2	0.4
4	400	1,200	162.8	325.6	0.8	1.6
6	600	1,800	366.3	732.7	1.7	3.5
8	800	2,400	651.3	1,302.5	3.1	6.2
10	1,000	3,000	1,017.6	2,035.2	4.8	9.7
12	1,200	3,600	1,465.3	2,930.7	7.0	14.0
14	1,400	4,200	1,994.5	3,989.0	9.5	19.0
16	1,600	4,800	2,605.1	5,210.1	12.4	24.8
18	1,800	5,400	3,297.0	6,594.0	15.7	31.4
20	2,000	6,000	4,070.4	8,140.8	19.4	38.8
22	2,200	6,600	4,925.2	9,850.4	23.5	46.9
24	2,400	7,200	5,861.4	11,722.8	27.9	55.8
26	2,600	7,800	6,879.0	13,758.0	32.8	65.5

Kuparuk River Submersible Bridges

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Figure 1. Aerial view of the completed East Channel Bridge during spring ice breakup.

Introduction

The strength, resiliency and construction flexibility of welded steel were keys to the design and construction of innovative submersible bridges on the North Slope of Alaska. Representing a $10 million investment, they cost about 50 percent less than elevated bridges to cross two river channels in a flood plain nearly two miles (3.2 km) wide. Design challenges included:

- extreme environmental conditions
- design vehicle weights approaching 4 million lbs (1.8 million kg)
- impact from river ice 5 ft (1.5 m) thick
- discontinuous permafrost soil conditions

Background

Concerns for fish habitat as well as further industry expansion beyond the Kuparuk Oil Field were the major factors driving the Kuparuk River East and West Channel Crossings project. For the previous nineteen years, these crossings had consisted of gravel roads and large multi-plate culverts. The crossings were breached and allowed to wash out during annual spring ice breakup, eliminating all road access to the Kuparuk Field for six to eight weeks per year. The new Kuparuk River submersible bridges, with a total length of 360 ft (110 m), reduce the closure period of this critical road link to a maximum of one week per year, and eliminate the need to reconstruct the road annually. The project's completion is expected to save the owner and other Alaska North Slope contractors more than $3 million per year in oil field operational and advanced material purchase costs. The submersible bridges provide a permanent crossing between the Kuparuk Field and infrastructure at Prudhoe Bay, Alaska (Figure 1).

The piece is a woven shell, in which the inside is outside, and the outside is inside

The crossings pass the peak spring breakup flows (typically more than seven times greater than summer flows) through and across the existing Spine Road by using a combination of welded steel submersible bridges and paved low-water roadways. Traditional elevated bridges serving the same function would have cost about twice as much as the submersible solution, even though the submersible structures are subject to substantially greater environmental forces. The short-span, stout, welded steel structures proved to be the most practical and cost-effective answer to permanently crossing the dynamic arctic coastal plain rivers.

Design and Construction

The owner requested that the bridges be designed to support any oil field vehicle currently in operation on the North Slope. The largest of these vehicles weighs about 3.8 million lbs (1.7 million kg). The entire load carrying capacity of each bridge is provided by the welded steel structure. The concrete deck is used only to provide the driving surface, lateral buckling support, and ensure composite action for horizontal loads.

Environmental loads for the bridges consisted of wind, seismic, river current, buoyant and river ice loading. For this design, ice loading was by far the most significant consideration.

The entire load carrying capacity of each bridge is provided by the welded steel structure

Design ice thickness was 52 in. (1300 mm) of hard structural ice capable of imposing tremendous loads on the bridge deck and ice breakers.

The substructure of each bridge consists of large-diameter, heavy wall, welded steel pipe piles. Each pile bent, spaced at 30 ft (9 m), consists of four vertical 36 in. (900 mm) diameter, 1 in. (25.4 mm) wall API 5LX-52 pipe piles driven to 80 ft (24 m) penetration to support the heavy vehicle loads and provide lateral support for ice loads.

The piles were spliced near the site to 100 ft (30 m) lengths with complete joint penetration groove welds in order to eliminate expensive and time-consuming splices in the field. The driven piles required welded cutting shoe tips for anticipated hard driving through permafrost at refusal. Because standard pile driving techniques cannot be used in permafrost conditions, a full depth pilot hole was drilled and steamed to thermally modify the permafrost prior to driving each pile.

Required capacity (design load) of each pile was 750 kips (375 tons) which was easily achieved using a Delmag D-62 diesel impact hammer, rated at 165,000 ft-lbs (223,700 joules) of energy.

The pipe piles were slotted at cut-off to receive the 36 in. deep x 20 in. (900 mm x 500 mm) wide welded steel box pile cap (Figure 2). The welded connection between the piles and pile cap provided both a full moment connection for lateral load resistance and flexibility in the field critical to successful fit-up of the girder splices. The slotted pile-to-pile cap connection also provided the load transfer for vertical loads, again eliminating the need for bearing stiffeners. All of the welded joint designs were AWS pre-qualified, an instrumental point in reducing cost, saving time and providing construction flexibility in the field.

Each in-stream pile bent has a welded steel ice breaking pipe on the upstream side. The ice breaker is a fabricated 36 in. (900 mm) diameter, 1 in. (25.4 mm) wall API 5LX-52 pipe installed at 45 degrees. It was field-welded to the pile cap with fillet welds to the cap webs only, which provides a pinned connection. The base was welded to a vertical pile using a 7/8 in. (22 mm) partial joint penetration groove weld with a 5/16 in. (8 mm) fillet cover pass. When impacted by an ice floe, this design will fail the ice sheet in bending rather than crushing, substantially reducing the lateral force on a single ice breaker from approximately 750 kips to 320 kips (3.34 million newtons to 1.4 million newtons). This unique design saved time and expense by reducing the ice loads to the structure, providing reasonable tolerances for field fit-up, and utilizing a vertical pile in lieu of more expensive and impractical batter piles.

The submersible bridge design utilizes 30 ft (9 m) spans that allow an extremely shallow, high-capacity deck and girder system. The superstructure consists of eleven welded 22-1/2 in. (6,858 mm) deep steel plate girders at 3 ft (0.9 m) spacing in cast-in-place structural concrete. The steel structure served as formwork for the superstructure, eliminating the need for falsework and minimizing on-site construction time and costs. A 26 in. (7,925 mm) diameter fabricated steel half-pipe deck nose was welded to the edge of the upstream girder to provide a round surface that further limits the ice load to the deck (Figure 3). The deck nose is fitted with guardrail support pipes that are welded to the outside girders. A rolled plate welded to the downstream girder provided the concrete formwork on the other side of the bridge. Steel plate placed on the bottom flange of the plate girders and

The unique design saved time and expense by reducing the ice loads to the structure...

attached with intermittent fillet welds served as the underside concrete form for the bridge.

The bridge design required an easily removable guardrail for the crossing of ultra-wide oilfield vehicles (up to 60 ft [18 m] wide). Welded steel pipe sleeves at the edges of the deck provide support for the removable guardrails. The identical sections of guardrail are easily removed and replaced as the pipe legs slide into the pipe supports in the bridge deck.

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Fabrication

All of the components of the bridge were shop-fabricated into sections that were easily transported to the site to minimize costs. Inspections by the engineer were performed throughout the fabrication process, and all welds were 100% visually inspected per AWS D1.1. In addition, 20% of all welds and all critical joints and plates were UT examined. The ends of girders were beveled in the shop for the 1-1/2 in. (460 mm) complete joint penetration groove field welds. This accelerated the project schedule and held costs down by minimizing the amount of preparation and weld that had to be performed in extreme weather conditions. Careful match marking of each fabricated member allowed the structure to be erected without field modifications even given windy and extremely cold conditions.

Conclusion

This unique and innovative design permitted the creation of permanent structures across the Kuparuk River East and West Channels in a very challenging natural environment. The combination of durable, submersible bridges and paved low water roadways kept the total expense of the project to about half of what traditional, elevated bridge designs would have cost. This successful design promises to serve as a model for future expansion of the infrastructure of the North Slope of Alaska and that of other similarly challenging environments.

Figure 2. Fabrication of the pile cap. Angle braces were welded to the base of the webs to minimize distortion/warpage and to support the bottom flange during fabrication.

Figure 3. Five-ft (1.5 m) thick fresh water ice impacting the bridge. The thin superstructure with rounded upstream nose and the sloped ice breaking pipes were all designed to minimize the ice loading on the submersible bridge, and to provide ice passage over or under the bridge.

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Cross-section conceptual drawing of a submersible bridge designed and fabricated for the North Slope of Alaska. See story on page 19.

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