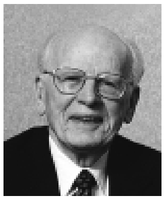
The Challenge of Welding Jumbo Shapes

Reprinted Articles from Welding Innovation Magazine

by Omer W. Blodgett, Sc.D., P.E. and Duane K. Miller, Sc.D., P.E.

The James F. Lincoln Arc Welding Foundation

Welding Innovation



Omer W. Blodgett, Sc.D., P.E.
Design Consultant

Volume X
Number 1, 1993

Editor
Duane K. Miller, Sc.D., P.E.

Assistant Editor
R. Scott Funderburk

The James F. Lincoln
Arc Welding Foundation

The Challenge of Welding Jumbo Shapes

3 **Part I: The AISC Specifications**

6 **Part II: Increasing Ductility of Connections**

10 **Part III: Case Study**

12 **Consider Welded Vs. Bolted Connections for Jumbo Sections**

Visit *Welding Innovation* online at http://www.lincolnelectric.com/innovate.htm

INTERNATIONAL ASSISTANT SECRETARIES

Australia and New Zealand
Raymond K. Ryan
Phone: 61-29-772-7222
Fax: 61-29-792-1387

Croatia
Prof. Dr. Slobodan Kralj
Phone: 385-1-6168306
Fax: 385-1-6157124

Hungary
Dr. Géza Gremsperger
Phone: 361-156-3306

India
Dr. V.R. Krishnan
Phone: 91-11-247-5139
Fax: 91-124-321985

Japan
Dr. Motoomi Ogata
Phone: 81-565-48-8121
Fax: 81-565-48-0030

People's Republic of China
Dai Shu Hua
Phone: 022-831-4170
Fax: 022-831-4179

Russia
Dr. Vladimir P. Yatsenko
Phone: 07-095-238-5543
Fax: 07-095-238-6934

United Kingdom
Dr. Ralph B.G. Yeo
Phone & Fax:
44-1709-379905

The Challenge of Welding Jumbo Shapes

Part I: The AISC Specifications

By Duane K. Miller, P.E.
Welding Design Engineer
The Lincoln Electric Company,
Cleveland Ohio

Introduction

In 1989, the American Institute of Steel Construction developed specifications for welding jumbo shapes. The next three articles will review the AISC specifications, discuss additional welding engineering principles and structural details that will contribute to the success of similar projects, and present a case study of a recent successful project which utilized these principles extensively.

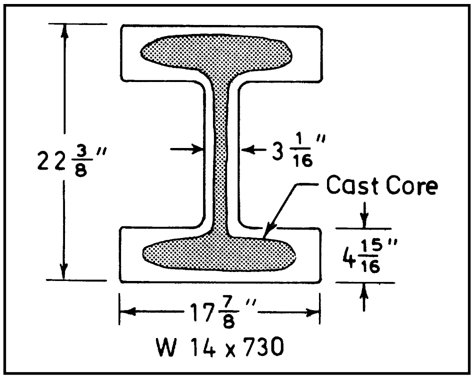


Figure 1. Group 4 and 5 shapes can weigh up to 848 pounds per linear foot.

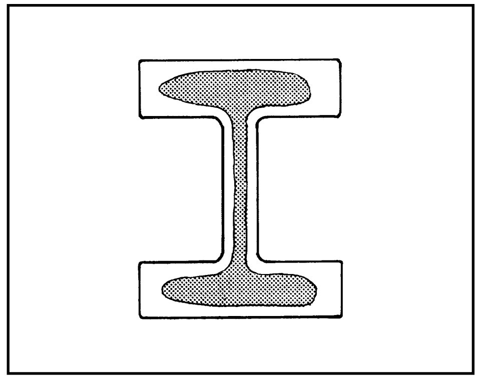


Figure 2. A "cast" structure is found in the middle of the section.

Special Problems of Welding Jumbo Shapes

During fabrication and erection of welded assemblies that utilized Group 4 and 5 shapes, commonly called "jumbo" sections, cracking problems were experienced on a number of projects. Particularly alarming were complete, through-section cracks that occurred in the tension cords of large trusses. The failures were classic brittle-type fractures that occurred in the complete absence of service loads. The typical crack would begin in the region of the weld access hole and propagate through the web, or the flange, or both. The fracture was usually in the base metal, with the weld unaffected. Fortunately, since these incidents occurred during construction, their impact was minimized. Nevertheless, these experiences caused many engineers to look to alternate materials or revert to bolted connections when jumbo shapes were used.

Group 4 and 5 shapes are very heavy rolled sections that weigh up to 848 pounds per linear foot (see Figure 1). During the initial solidification of the ingot used to make these shapes, it is possible for carbon and other alloys to segregate, causing an enriched concentration of elements in the center of the ingot. Upon rolling, the outer surfaces of the shape being formed will be cooled by the mill rolls and other external cooling methods. In addition, the surfaces of the shape receive significant mechanical working, improving the toughness of these surfaces, but the center region receives minimum mechanical working. As shown in Figure 2, a "cast" structure is formed in the middle of the section which may exhibit poor notch toughness. Where failures have been experienced in the past, poor toughness in this region has been characteristic.

The greatest problems with welding on jumbo shapes occurred when these materials, originally contemplated for compression (or column) applications, were used in tension applications. Since the failures occurred before the structures were subject to service loads, the difference was not inherent to the application, but rather to the type of weld details used in the two types of applications. In primary tensile applications, Complete Joint Penetration (CJP) groove welds typically are required. For compression applications, Partial Joint Penetration (PJP) groove welds generally are sufficient. When CJP groove welds are used, weld access holes (colloquially known as "rat holes") are required. For PJP groove welds, they are not. The presence or absence of weld access holes, and the difference in residual stresses experienced by CJP welds (which have greater weld metal volume than PJP groove welds) explain the difference in behavior between tension and compression applications. The PJP groove weld may not intersect the as-cast core structure in the center of the section, while the weld access hole used with CJPs automatically intrudes into this region.

A common method of preparing the various rolled sections for welding is to use the oxy fuel thermal cutting process, which generates a change in microstructure. The surface may be enriched in carbon content, and the hot steel on the surface of the cut is rapidly cooled by the conduction of heat into the surrounding steel. As a result, a small, thin layer of relatively hard, brittle microstructure may form. Under some conditions, microcracks have resulted in this zone. Good workmanship is also required in this area. In some cases, ragged edges resulted in crack-like notches in the weld access hole region. Further complicating the cutting process is the semicircular nature of the weld access hole, where good manual dexterity is required in order to create a uniform surface. Finally, as the cut approaches the region of the web-to-flange interface, the natural radius that occurs in rolled shapes makes precise cutting difficult. In many examples involving fracture, poor workmanship resulted in inadequate preparation of the weld access holes.

When welds cool from elevated temperatures, they must shrink in size due to the thermal contraction that takes place.

As the hot, but already solidified, weld metal shrinks in size, it induces shrinkage strains on the surrounding materials. These strains induce stresses that cause localized yielding. As the weld metal cools to near-room temperature, the remaining strain may be insufficient to cause yielding, but will result in residual stresses that are present after welding is complete. For example, a weld that is used to join flanges will establish a residual stress pattern that is transverse and longitudinal to the direction of welding. The web weld will similarly set up a longitudinal and transverse shrinkage stress. Surrounding the regions of high residual tensile stress, there will be a region of residual compressive stress. When weld access holes are small in size, the residual stresses from these two welds can combine and develop into a triaxial state of stress. Under these conditions, the steel may be unable to exhibit its normal ductility, although the same material under uniaxial conditions may behave in a very ductile fashion. The formation of this high triaxial stress in the region of the web-to-flange interface is the driving force behind the initiation of these brittle cracks.

Figure 3 illustrates the interaction of weld type (CJP vs. PJP), weld access holes, and the cast core region. In the top figures, the CJP preparation automatically intersects the region of the shape with questionable microstructure. For the PJP, the absence of access holes, plus the reduced depth of bevel, minimizes or eliminates the amount of core region that is intersected by the cutting process. As a result, weld metal, and the shrinkage stress it will induce, does not directly act on the core region.

Fracture mechanics methods may be used to determine the required notch toughness to prevent brittle fracture. In the case of the jumbo section failures, the cracks located in the weld access holes were of sufficient size, and the residual stresses from welding of a sufficient level, that they exceeded the resisting force, or the fracture toughness, of the base material, particularly in the cast core region. To ensure reliable fabrication, the AISC Task Group addressed all three issues: notch toughness of the base material, quality of the weld access holes, and a reduction in the residual stresses developed by examining the geometry of the weld access hole.

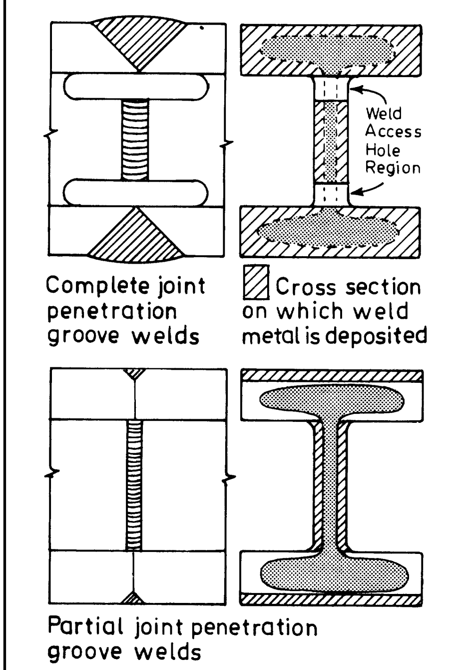


Figure 3. The interaction of weld type, weld access holes, and the cast core region is shown here.

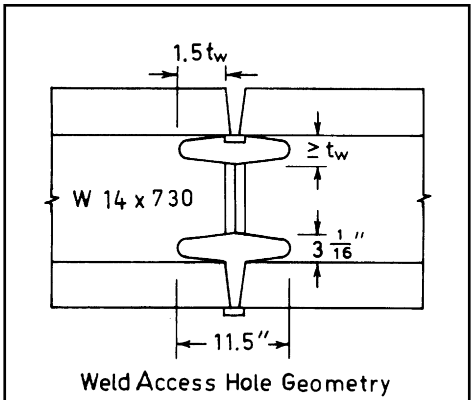


Figure 4. The root opening and overall width of the weld joint further add to the width of the access hole.

The AISC Response

The AISC task group set up to study this matter established, and the Specification Committee adopted, a series of controls to permit problem-free welding of jumbo sections. These were initially published in 1989 as Supplements 1 and 2 to the *AISC Steel Construction Manual*. They have now been incorporated into the ninth edition of the ASD Manual. The first was a requirement that the base metal exhibit a minimum notch toughness of 20 foot-pounds at +70°F. It is required that the Charpy specimens be taken from the web-to-flange interface, the region expected to have the poorest toughness. A variety of methods may be employed to improve the notch toughness in this area, including the use of semi-killed steel, fine-grain practice, and special cooling techniques.

To provide additional resistance to cracking during the thermal cutting of weld access holes, the specification now requires a preheat of 150°F before the thermal cutting is to be performed. This slows the cooling rate experienced by the cut surface, providing increased resistance to cracking. After thermal cutting, the surfaces must be ground to bright metal and inspected with either magnetic particle testing (MT) or liquid penetrant testing (PT), further assuring smooth transitions that are free of notches and cracks.

In order to reduce the level of residual stresses in the area of concern, specific minimum dimensional requirements were imposed on the size and shape of the weld access holes. The minimum size for the width of the weld access hole is required to be 1.5 times the thickness of the web on either side of the joint. The root opening and overall width of the weld joint further add to the width of the access hole, as shown in Figure 4. If a B-U2-S type prequalified welding joint is used on a W 14 x 730 rolled section, the resulting total weld access hole dimension is required to be a minimum of 11.4 inches. These minimum dimensions are required for several reasons. First, generously sized weld access holes prohibit the residual stress patterns from the flange welds and the web weld from interacting with each other. When the radius which forms the end of the weld access hole is placed away; from the flange weld, the radius is located in a region of residual compressive stress, or a region of nearly no stress. In either case, ductility of the steel is enhanced. Finally, the separation of the web from the flange for this distance permits the unrestricted yielding of the web region. When small access holes are used, the web, rigidly attached to the flanges, is forced to absorb all the shrinkage stresses in a very small area, concentrating stresses right in the region of concern: the weld access hole. The requirement of minimum dimensions for the weld access hole reduces

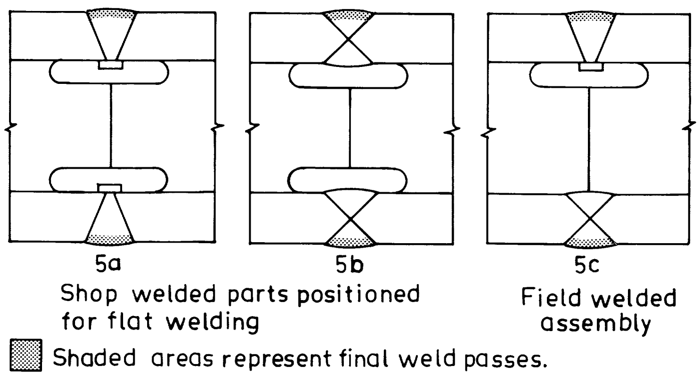


Figure 5. Final weld passes should be made on the outer surfaces of the weld flanges wherever possible.

residual stress levels. This is discussed in the following article entitled "Increasing Ductility of Connections."

The preheat requirement for welding on jumbo sections has been increased to 350°F. The preheat temperature level should be extended to a minimum interpass temperature as well, although this is not detailed in the AISC specification.

Further Recommendations

The new AISC guidelines apply only to applications where the members are subject to primary tensile stresses and spliced with full penetration groove welds. These requirements do not apply to situations where members are not subject to primary tensile stresses. However, as noted, these failures were not associated with service loading. If the types of details that are typically used for tension applications are applied to compression components, e.g., CJP groove welds, the same types of cracking problems may occur.

Additional techniques that minimize the accumulation of residual stresses should be employed when welding on jumbo shapes, even though they are not enumerated in the AISC specification. Selection of the specific welding joint detail is important. Double-sided joints reduce the amount of weld metal required by a factor of two. Reducing the amount of weld metal proportionately reduces the residual stresses. Double-sided joints require access from both sides, which in the case of field welding would dictate overhead welding. While out-of-position welding is generally discouraged, it may be advisable in this case to minimize the amount of residual stresses.

Furthermore, welding sequence can be important. On jumbo sections that involved cracking, when the flanges were welded first, the crack would form in the web; when the web was welded first, the cracking typically occurred in the flanges. Finally, cracking was more likely when the final weld passes were applied on the inside surfaces of the flanges (closest to the weld access hole). The following should be observed with regard to welding sequence:

1. As much as is practical, do not weld any specific joint to completion. Weld no more than 1/3 of the depth of any joint before moving on to a separate joint.
2. Utilize joint details that permit the application of the final weld passes on the outer surfaces of the weld flanges where possible. For shop fabrication where single vee groove welds may be used, as shown in Figure 5a, the last welds will automatically be made on the outer surfaces. If double vee groove welds are preferred (because of the reduced weld volume and reduced shrinkage stresses), the last passes should be on the outer flanges, as shown in Figure 5b. For field work where flanges are horizontal, a combination of these joints may be desirable. The top flange can be prepared with a single vee groove, while the bottom flange is prepared as a double vee, as illustrated in 5c. This necessitates some overhead welding, but the final passes occur on the other flanges, reducing cracking tendencies.

The AISC specifications do not impose any new requirements on welding processes or the consumables used to join these jumbo shapes. The failures that had been experienced were not weld metal failures, but rather were located in the base materials. True, the driving force for cracking was due to the residual stresses from welding, but the primary problem was one of inadequate base metal toughness.

Some confusion has resulted regarding the requirements for filler metals on jumbo shapes. The new specifications do not impose notch toughness requirements on the welding materials. This was confusing because the supplements were printed with an additional comment regarding the use of "mixed weld metal." This provision is applicable under circumstances where notch toughness has been specified for the weld metal, and the composite weld metal consisting of different compositions must have a composite notch-tough weld metal. However, the requirement applies only to situations where notch toughness has been specified for the weld metal – typically, in dynamic structures, since static structures rarely require the use of weld metal with increased notch toughness properties.

Conclusion

The New AISC specification requirements successfully addressed the variables that have been associated with the fracture of welded jumbo sections in the past. In addition, proper selection of the welding joint detail and careful consideration of welding sequence will contribute to the successful use of welded jumbo sections in tension applications.

Referenced Documents

American Institute of Steel Construction: *Supplement No. 2 to the Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings (8th Edition ASD Manual) January 1, 1989.*

American Institute of Steel Construction: *Supplement No. 1 to the Load and Resistance Factor Design Specification for Structural Steel Buildings (1st Edition LRFD Manual)*, January 1, 1989.

American Institute of Steel Construction: *Specification for Structural Steel Building, Allowable Stress Design and Plastic Design (9th Edition ASD Manual)*, Paragraph A3.1.c, June 1, 1989.

The Challenge of Welding Jumbo Shapes

Part II: Increasing Ductility of Connections

By Omer W. Blodgett, P.E.
Senior Design Engineer
The Lincoln Electric Company,
Cleveland Ohio

Introduction

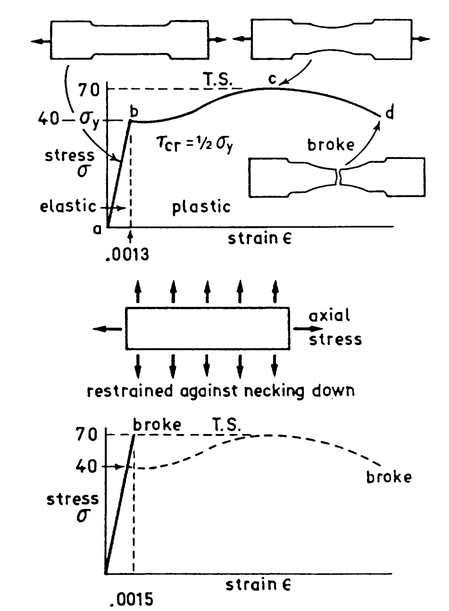
Materials used in steel structures are becoming increasingly thicker and heavier. A greater chance of cracking during welding of beams in columns, for example, has resulted due to increased thickness of material. With weld shrinkage restrained in the thickness, width, and length, triaxial stresses develop that may inhibit the ability of steel to exhibit ductility. This article attempts to explain why these cracks may occur, and what can be done to prevent them, by expanding on information presented in the AISC Supplement No. 2 entitled "To the Specification for the Design, Fabrication & Erection of Structural Steel for Buildings."

Field Results

Engineers have been taught that the yield point property of the material is the prime factor relating to ductility. This, however, offers a limited view. Figure 1 shows a stress-strain curve applied to a steel specimen which is loaded in tension parallel to its length (a). In this type of test, the specimen is free to neck-down once the yield strength is reached (b). As it plastically yields, it strain-hardens to a higher strength (b to c). This stress continues to increase to (d), but because of a reduction in the cross-section, its apparent strength drops from (c) to (d).

If the load is removed, the specimen will not return to its original dimensions. Within the limit of elastic behavior occurring from (a) to (b), however, movement is small and would not be noticed unless measured. If the specimen's load is removed, it will return to its original dimensions with a springlike movement. For example, if a steel flange plate has a yield strength of 40 ksi, elastic movement would be:

$$\epsilon = \frac{\sigma}{E} = \frac{40{,}000 \text{ psi}}{30{,}000{,}000 \text{ psi}} = 0.0013 \text{ in/in}$$



Stress–Strain Curve for Steel

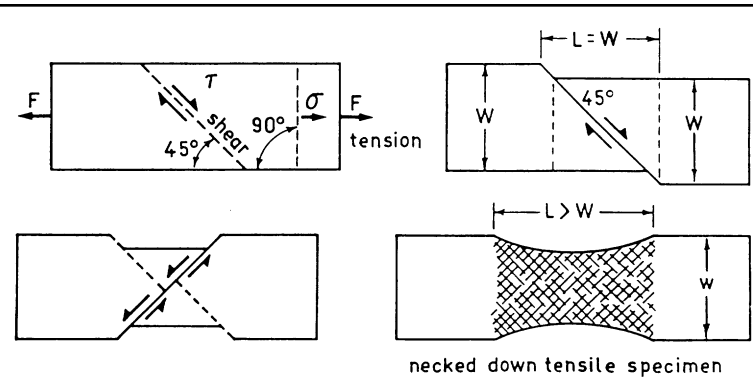
Figure 1.

Laboratory Results

In the laboratory, it is typical to think of applying a force to a tensile specimen so that its resulting strain or movement may be observed. But this is not what really happens with a tensile laboratory testing machine. When the machine is turned on, a motor gradually turns a screw feed which slowly strains or stretches the specimen in the longitudinal direction. The resisting force of the specimen against this straining movement is indicated on a gauge. Yield strength is reached when the applied stress exceeds the critical point, and the specimen is free to plastically neck-down. If the specimen is restrained, as it usually is in the field, the stress-strain curve indicated in Figure 1 may continue to the point of ultimate tensile strength in an almost straight path, until it ultimately fails without exhibiting much apparent ductility.

When an axial force (F) is applied to a test specimen, it will cause a normal stress (σ) on a plane 90 degrees to the direction of the force. It also causes a shear stress (τ), which reaches its maximum on a plane 45 degrees to this force, and is equal to one-half the value of normal or tensile stress. If this shear exceeds a critical value, a sliding action takes place, allowing the specimen to become longer in the direction of the force and narrow across its width. If the resulting shear value is low, based on design, and the critical shear stress point cannot be exceeded, then an increased load will mean failure when the critical tensile point is exceeded.

Sliding action can also take place on the 45 degree plane in the other direction. If the action continues, a necked-down elongation results in a tensile-tested specimen as Figure 2 indicates. The slip plane lies at 45 degrees, forming a reduced section, initially having a square outline. If the unrestrained length (L) of this section is at least equal to or greater than the width (W), the specimen



Ductility of Steel

Figure 2.

will be free to neck-down and show full ductility. If the unrestrained length (L) is less than the width (W), the shear component (τ) will decrease. A greater applied force will be necessary for the critical shear value to be exceeded, reducing its ductility. This is one reason AISC Supplement 2 requires the weld-access hole to extend a distance on each side of the weld, equal to three times the web thickness. Doing so provides an unrestrained length of web, giving the specimen sufficient ductility.

Field Application

In the field, specimens do not usually exist independently. Steel plates are often restrained and not free to neck-down. The weld solidifies and shrinks as it cools, similar to a steel casting. When this shrinkage or strain is restricted, a high residual tensile stress results, sometimes sufficient to cause some part of the joint to pull apart and crack.

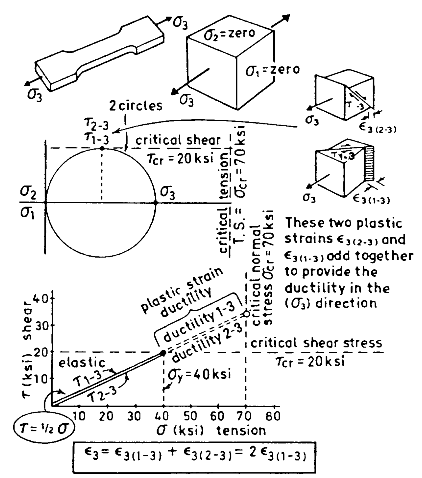
Instead of looking for stresses which might cause such a crack in the welded joint, it is better to consider strains and how they can be reduced to avoid cracking. Application of distortion and residual stress factors will help reduce these strains when a restrained member is welded.

Mohr's Circle Explains Stress

A biaxial stress condition in a steel plate can be explained using Mohr's circle of stress as Figure 3 indicates. The tensile specimen will be stressed in one direction only. Stresses σ_2 and σ_1 equal zero. Their circle has zero radius and zero shear stress ($\tau_{1\text{-}2}$) along the vertical access.

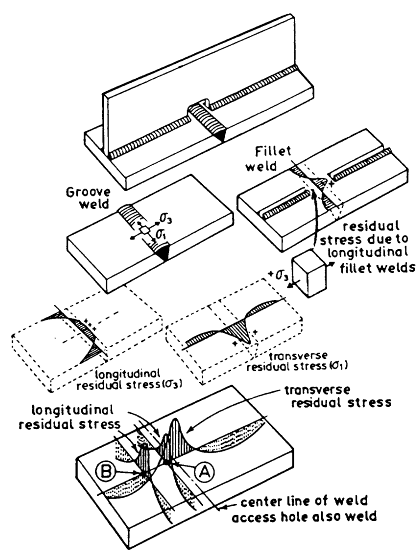
The two cubes on the right part of Figure 3 show that the resulting shear stresses ($\tau_{1\text{-}3}$) and ($\tau_{2\text{-}3}$) from the normal stress (σ) cause a sliding action which produces movements $\varepsilon_{3(1\text{-}3)}$ and $\varepsilon_{3(2\text{-}3)}$ in the direction of (σ_3).

The lower figure plots shear stress (τ) versus normal stress (σ_3). In this particular load condition – a simple tensile test – the shear stress is always equal to one-half the applied normal stress (σ). Notice that this load line will increase upward and to the right until it reaches the critical shear value of 20 ksi, at which time the yield strength is reached



Ductility of Simple Tensile Specimen

Figure 3.



Resultant Residual Stress of Welding

Figure 4.

(σ_y = 40 ksi). Ductility or plastic strain takes place until the critical normal stress (70 ksi) is reached. Failure then occurs immediately. During this time, plastic straining $\varepsilon_{3(1\text{-}3)}$ and $\varepsilon_{3(2\text{-}3)}$ from two different shear stresses ($\tau_{1\text{-}3}$) and ($\tau_{2\text{-}3}$) results in a very ductile condition.

The other two resulting shear stresses ($\tau_{1\text{-}3}$) and ($\tau_{2\text{-}3}$) are each equal to one-half applied normal stress. Shear stress, if incorporated into a structure's original design, accommodates yield, if needed. For example, if the yield point is 40 ksi, the critical shear stress value would be 20 ksi. Below 20 ksi, only elastic strain exists; above 20 ksi, there is plastic strain.

At the critical shear stress point of 20 ksi, plastic straining begins; the steel specimen starts to neck-down. With two sheer planes involved (two similar circles), ductility doubles. Applied tensile stress at this point is called the yield point (40 ksi). When the section necks-down to the extent that the resultant normal tensile stress exceeds the critical value of 70 ksi, tensile failure occurs. Between these two points, ductility occurs.

Two Residual Stresses Isolated

Figure 4 illustrates that two important residual stresses exist in the weld's termination zone. The butt weld in the flange has a residual stress longitudinal to the length of the flange (σ_3) as well as a stress transverse to the flange (σ_1). Longitudinal stress is tensile along the center line of the flange where the weld-access hole terminates. It can be compared to tightening a cable lengthwise in the center in tension, with compression spread on both sides. The transverse stress is tensile in the weld zone, with a portion of the adjacent plate, going through zero, and then compression, beyond the adjacent plate. A transverse stress is also similar to tightening a cable.

In addition, because some restraint exists through the thickness of the flange in the region where the flange connects to the web at the termination of the weld access hole, a triaxial stress may be introduced. Terminating near the side of the flange weld (A), the access hole is subjected to residual tensile stress (σ_1) transverse to the flange, as well as to residual tensile stress (σ_3) longitudinal to the flange, with full ductility restricted. If the weld access hole is made wider (B), terminating at a point where the residual stress (σ_1) transverse to the flange is compressive, a more ductile condition results. The result is similar to one person pulling on a tube of toothpaste, while another squeezes it. It can stretch more easily. Otherwise, the combination of transverse and longitudinal stresses results in two tensile stresses at 90 degrees.

Residual Stresses Applied

These residual stresses may be applied to a weld detail having a narrow weld-access hole, Figure 5. The hole terminates at point (A), resulting in (σ_1) and (σ_3) being in tension. Although ($\tau_{1\text{-}2}$) may be high, the right portion of Figure 5 indicates that this strain, $\varepsilon_{1\,(1\text{-}2)}$, does not act in the direction of (σ_3). It offers little help in producing ductility in this direction.

Normal stresses, (σ_1) and (σ_3), produce shear stresses which act in opposite directions, so the final value will be small and not helpful in the prevention of cracking. This leaves only shear stress ($\tau_{2\text{-}3}$) which will be effective in providing ductility.

As Figure 6 suggests, it is possible that this does not fully represent the problem. Since the web at the edge of the weld-access hole offers some restraint against movement in the through-thickness direction of the flange plate, stress in the σ_2 direction may have an appreciable tensile value. For example, by moving stress σ_2 from zero up to a tensile value, resulting circles and their corresponding shear stresses become similar. For the same value of (σ_3), both ($\tau_{2\text{-}3}$) and ($\tau_{1\text{-}3}$) will probably never intersect with the critical shear stress value, and plastic strain or ductility will not occur as the lower portion of Figure 6 illustrates.

If the weld access hole can be made wider – as recommended by AISC Supplement 2 – so that it terminates in a zone where the transverse residual stress (σ_1) is compressive (see Figure 7), then a more favorable stress condition will result in greater ductility in the (σ_3) direction. In this case, shear stress ($\tau_{1\text{-}3}$) will be high as shown by Mohr's Circle of Stress.

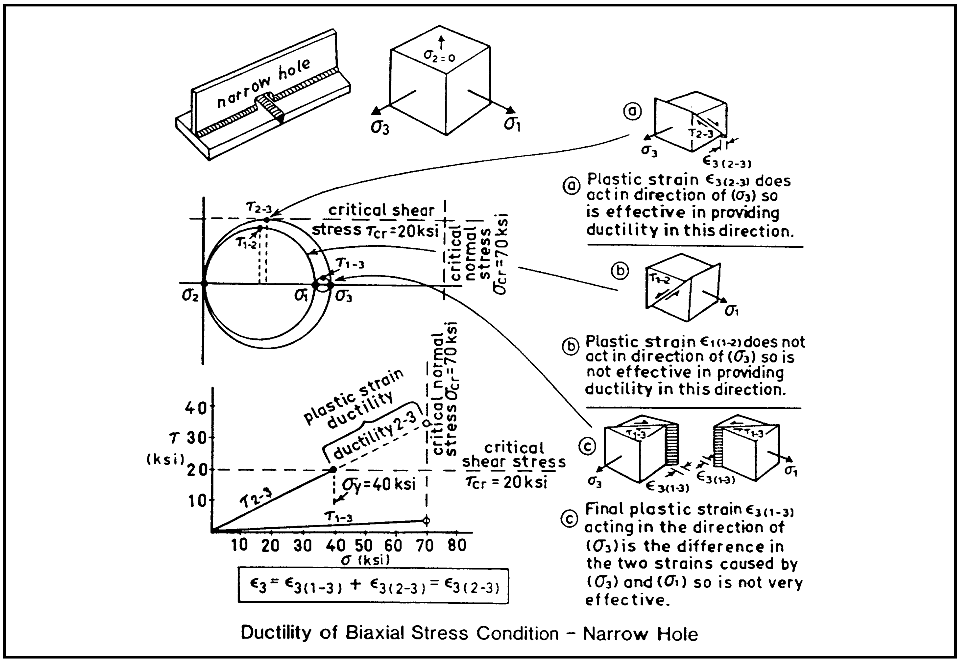


Figure 5.

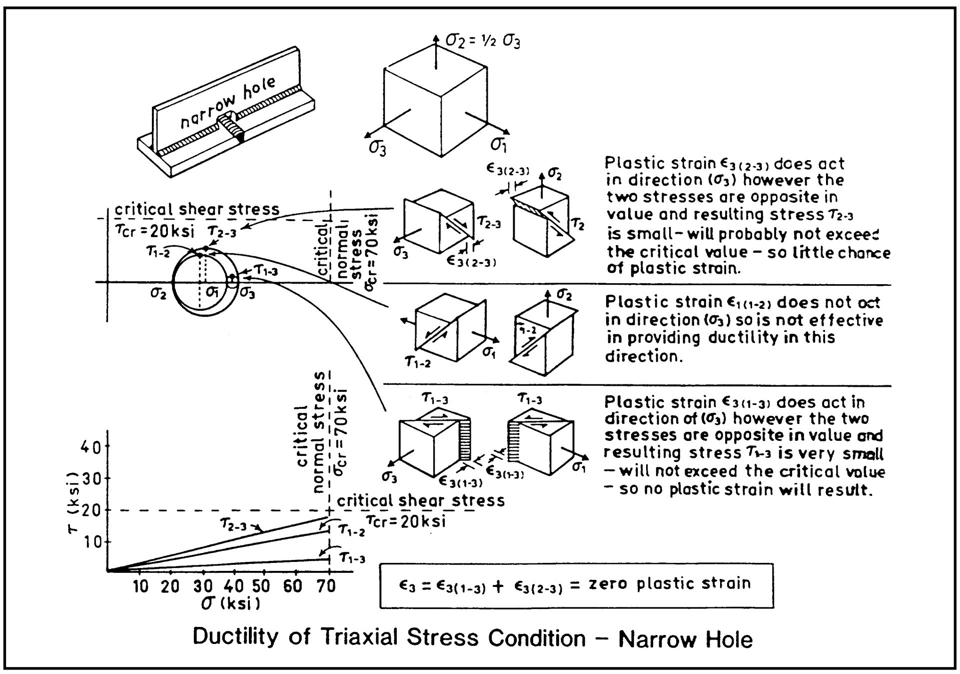
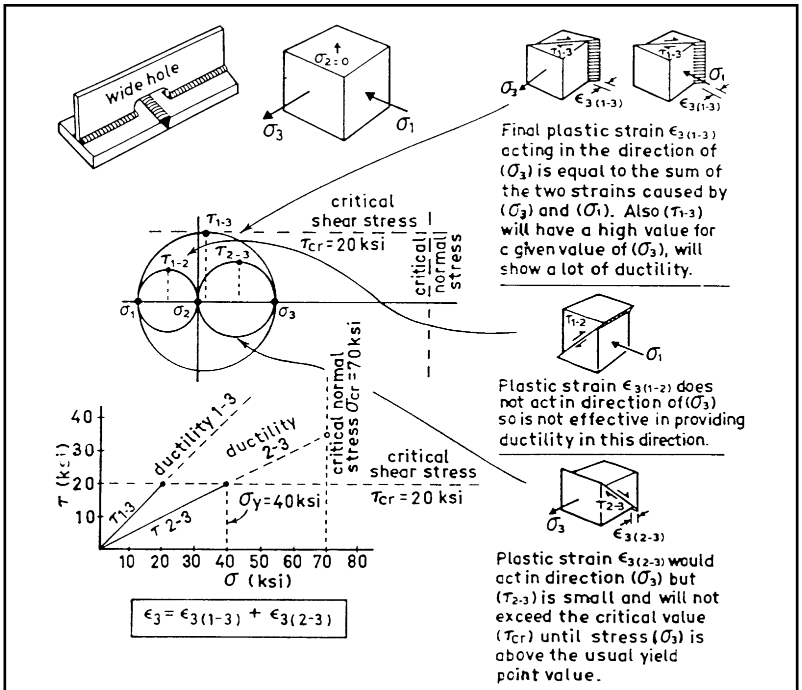


Figure 6.

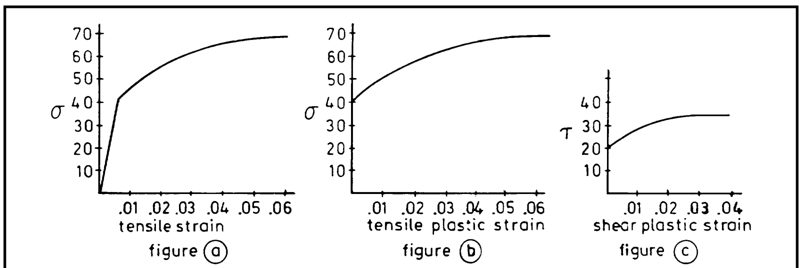
The diagram on the right of Figure 7 shows that (σ_1) and (σ_3) produce shear stress ($\tau_{1\text{-}3}$) which, in turn, produces two strains or movements $\varepsilon_{3\,(1\text{-}3)}$, acting in the same direction. Not only is the plastic movement larger, but as the lower diagram shows, the critical shear value is reached at a much lower normal stress or load value. This produces more ductility in the (σ_3) direction, greatly reducing the chance of a transverse crack in the flange at the termination of the weld access hole.

If there is some restraint in the through-thickness of the flange plate enabling σ_2 to have an appreciable residual tensile stress, this would simply move the point on Mohr's Circle of Stress from its initial value of zero, to the right, to some tensile value. Circle (2-3) would become smaller, reducing shear stress ($\tau_{2\text{-}3}$) and lowering load line (2-3) shown in the lower diagram. This would not, however, change circle (1-3) or its shear stress ($\tau_{2\text{-}3}$). It also should not affect the position of the load line (1-3) in the lower figure, and should still result in good ductility.



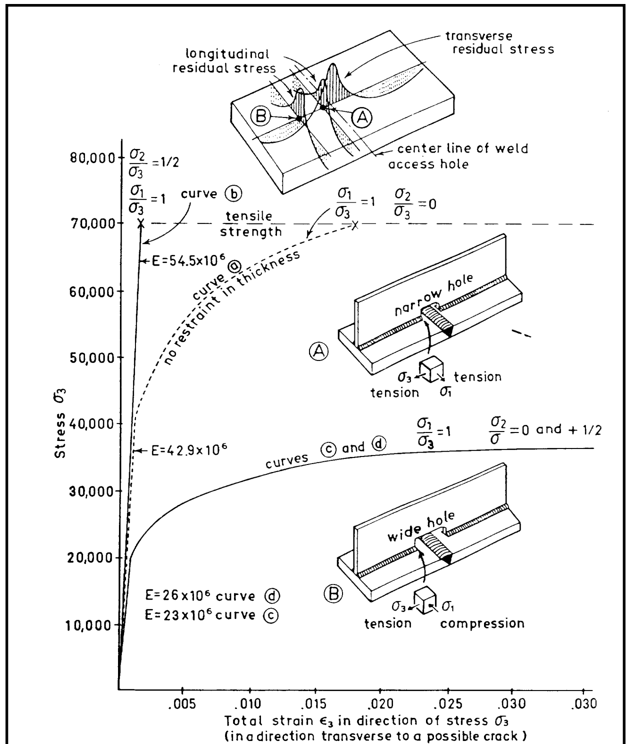
Ductility of Biaxial Stress Condition – Wide Hole

Figure 7.



Shear Plastic Strain Curve

Figure 8.



Constructed Stress–Strain Curve For Various Weld Details

Figure 9.

Using a method proposed by F. R. Shanley, it is possible to take a representative stress-strain curve for mild steel (Figure 8a), separate the plastic strain portion from it, (Figure 8b), and convert this into a shear stress-plastic strain for any given shear stress (τ) once it exceeds the critical shear value. The shear stresses (τ_{1-3}) and (τ_{2-3}) can now be converted into plastic strain plus elastic strain as the value of the applied normal stress (σ_3) is increased. A stress-strain curve for any combination of triaxial stresses may be constructed. Figure 9 contains curves for the conditions already discussed. This makes it possible to "see" the ductile behavior of these details. Notice the beneficial effects of the wide access hole as recommended by AISC Supplement 2.

Conclusion

The way in which a designer selects structural details under particular load conditions greatly influences whether the condition provides enough shear stress component so that the critical shear value may be exceeded first, producing sufficient plastic movement before the critical normal stress value is exceeded. This will result in a ductile detail and minimize the chances of cracking.

References

American Institute of Steel Construction: *Supplement No. 2 to the Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings (8th Edition ASD Manual)*, January 1, 1989.

Blodgett, Omer W. *Weight of Weld Metal*, The James F. Lincoln Arc Welding Foundation, Bulletin D417, April 1978.

Gensamer, Maxwell, "Strength of Metals Under Combined Stresses," American Society of Metals, 1941, pg. 10.

Bjorhovde, Brozzetti, Alpsten and Tall. "Residual Stresses in Thick Welded Plates," AWS Welding Journal, August 1972, pg. 397.

Estuar and Tall. "Experimental Investigation of Welded Built-Up Columns," AWS Welding Journal, April 1963, pg. 170.

Parker, Earl R., *Brittle Behavior of Engineering Structures*, John Wiley and Sons, Inc., 1957, pg. 19.

Gayles and Willis. "Factors Affecting Residual Stresses in Welds," AWS Welding Journal, August 1940, pg. 303.

Shanley, F.R., "Plastic Strain – Combined Loading," *Strength of Materials*, McGraw-Hill Book Co., 1957; Chapter 11, pgs. 178 – 200.

The Challenge of Welding Jumbo Shapes

Part III: Case Study

By Duane K. Miller, P.E.
Welding Design Engineer
The Lincoln Electric Company,
Cleveland Ohio

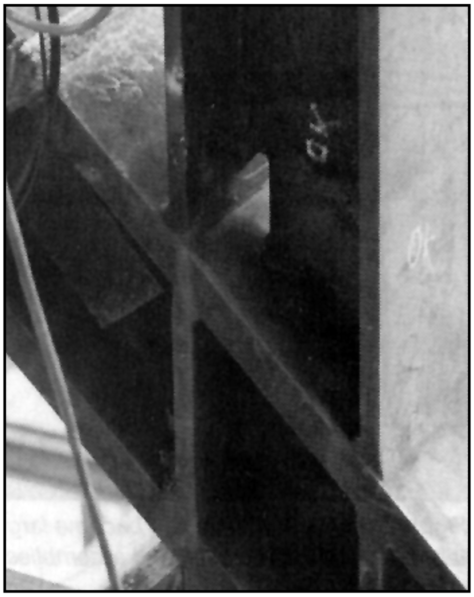


Figure 1. Good workmanship is exhibited in this access hole. Extension tabs have already been removed.

Introduction

When EDS, Inc. (Plano, Texas) wanted to expand their world headquarters, they chose a design that involved suspending a four-story building between two six-story structures. Two, two-lane roads would run below the elevated four-story section. W&W Steel (Oklahoma City, Oklahoma) was awarded the contract to fabricate the 8,000 tons of steel; because of the size of the project, they sublet about 50% of the work to AFCO Steel (Little Rock, Arkansas). Initially, both fabricators were concerned about a number of articles that had documented problems associated with the welding of jumbo sections. In addition to their determination to create a structurally sound building, they adopted a further goal: to successfully fabricate by welding a structure that would utilize jumbo members in tension applications.

Design Details

The design called for four catenary trusses, with two double catenaries located on opposite sides of vertical columns. The end columns were W14x500, while the catenary section ranged from W14x370 to W14x398. Interior columns were typically W14x145, and floor beams were W27x146. The upper cord of the truss was W14x283. On the exterior trusses, the catenary was to be the continuous member, with the columns cut to fit around the diagonal catenary member. Stiffeners were required for continuity and the columns were then to be welded to the catenary.

Knowing they were facing a real challenge, the fabricators sought and obtained consultation advice from The Lincoln Electric Company. Reviews of past failures were studied. The AISC specifications, their implications, and the background justifications for them were carefully reviewed. Particular attention was focused on the weld access hole geometries and steel toughness requirements. Welding procedures and increased preheat requirements were discussed. There were a dozen different geometric configurations because of the ever-changing orientation of the catenary to the trusses and/or floor beams. This dictated that each specific geometry be carefully evaluated. Because of the unusual geometries involved, direct application of the specifications did not always generate an acceptable configuration that would assure freedom from intersection of the various residual stress patterns in the vicinity of the weld access hole. Under these conditions, engineering judgment was used to extend the minimum requirements to a more liberally sized access hole.

The specification requirements dictate the removal of weld extension tabs and weld backing. While all the extension tabs were removed, it was impossible to remove the weld backing in every case due to access problems. In these situations, engineering approval was sought and gained to leave the backing in place.

The assembly sequence of the more complex geometries had to be carefully planned in order to assure access for the deposition of quality welds and subsequent inspection. After a feasible plan was developed, a full sized mock-up was fabricated. Although some of the configurations were extremely difficult to execute, none proved to be impossible.

Figure 2. The non-planar orientation of the catenary would have made bolted connections difficult or impossible to achieve.

The weld access hole geometries were detailed on the shop drawings to ensure complete communication. Instead of burning or thermally cutting the access holes, both fabricators elected to drill a hole to serve as the radius. Thermal cutting was then used to extend the hole to the weld joint. This eliminated grinding of the access hole, as well as subsequent nondestructive testing of the surface. An example of the excellent workmanship is shown in Figure 1.

Fabrication Details

Both fabricators utilized conventional welding processes with standard welding procedures. One fabricator chose gas-shielded flux-core (E70T-1), while the other used self-shielded flux-core (E71T-8Ni1). Excellent results were achieved in each case. Shop welding was employed as much as possible, although the size of the parts dictated that extensive field assembly would be required.

On many projects of this type, there is considerable debate regarding the relative advantages of field bolting vs. field welding. On the EDS project, however, there was little question. The non-planar orientation of so many of the catenary members would have made the use of field bolted splices difficult if not impossible. The lap splices would have required bending, and it would have been nearly impossible to assure adequate contact between the surfaces. Field welding was performed using self-shielded flux-cored electrodes, as well as manual shielded metal arc welding.

During construction, the field erector utilized Lincoln Electric LN22 and LN25 portable wire feeders because of their versatility and portability. DC600 multi-process power sources were advantageous since they permitted the use of both constant current (e.g., SMAW) and constant voltage (e.g., FCAW) welding from one power supply. To simplify electrical installation and handling of the power supplies, Lincpacks were used. With this arrangement, a common frame accommodated six electrically connected power supplies.

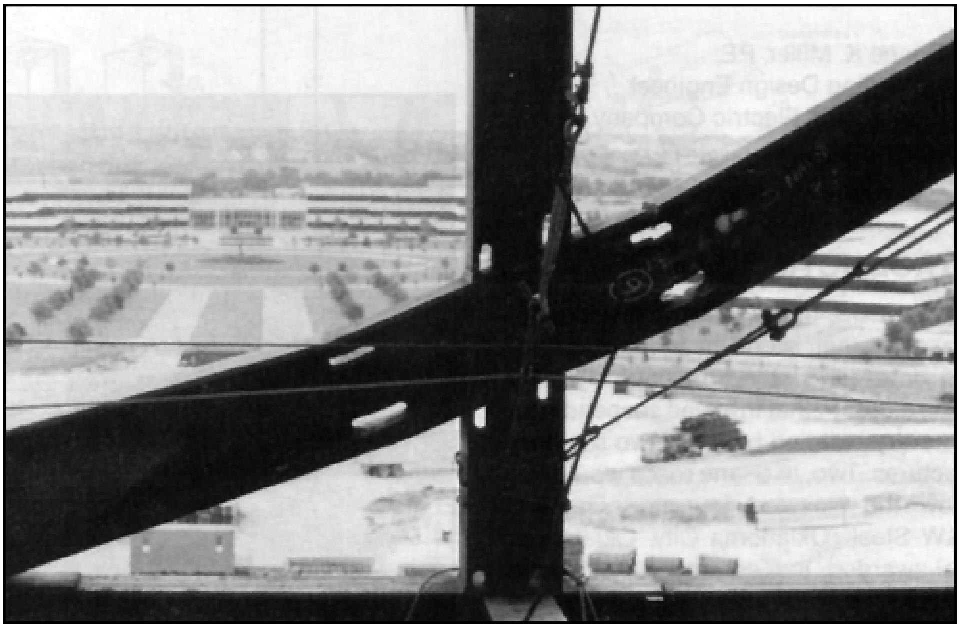


Figure 3. Access holes can become large. The location of field splices allows for shop fabrication of the most difficult assemblies. Note the EDS campus in the background.

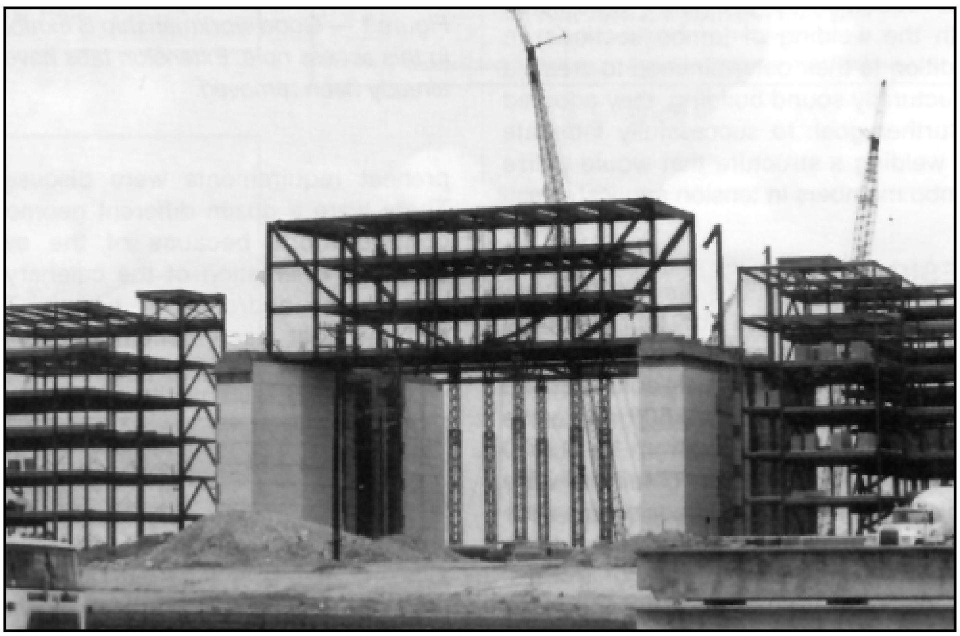


Figure 4. The catenaries can be clearly seen as two of the four trusses near completion.

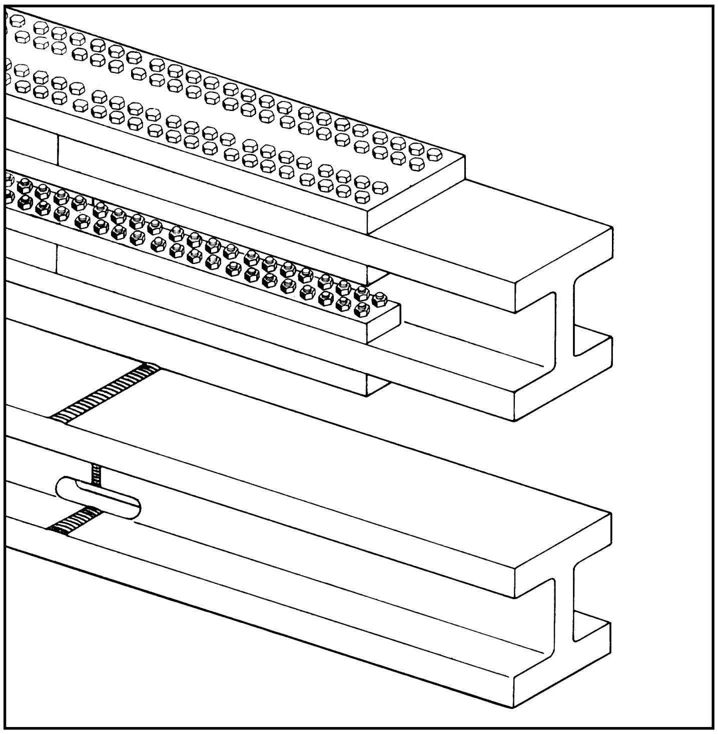
Conclusion

Today, the EDS structure in Plano, Texas stands as a dramatic illustration of engineering progress. The fabricators are to be commended for having employed current industry requirements to overcome problems experienced by others I the past. To the steel industry, the EDS project offers welcome proof that designers can use jumbo sections in welded construction with confidence.

Design File

Consider Welded Vs. Bolted Connections for Jumbo Sections

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



COST COMPARISON

		Hours	Dollars
Bolted:	Labor		
	• Cut & prep	15.5	
	• Drill	46.9	
	• Assemble & Torque	21.0	
	Total Hours	83.4	
	Labor Costs		
	• 83.4 hrs at $37.50/hr		$3127
	Material Costs		
	• Steel		
	2 - 18″x4″x8′5″ 6900 lbs		
	4 - 7″x3½″x8′5″ at $0.28/lb		1936
	• Bolts		
	256 HSB 1⅛″x1′2″ at $3.00 ea.		768
	TOTAL COST PER SPLICE		$5800
Welded:	Labor		
	• Cut & prep	7.0	
	• Weld flanges and web	17.6	
	• Remove tabs, grind	4.0	
	Total Hours	28.6	
	Labor Costs		
	• 28.6 hrs. at $37.50/hr		$1072
	Material Costs		
	• 70 lbs electrode at $0.60/lb		42
	• 140 lbs flux at $0.55/lb		77
	TOTAL COST PER SPLICE		$1200
SAVINGS PER SPLICE			**$4600**

Background

Other articles in this issue of *The Welding Innovation Quarterly* cover the new American Institute of Steel Construction (AISC) specifications for welding on jumbo shapes, as well as the associated development of sound fabrication principles. This Design File column will demonstrate that welding jumbo sections can provide a safe and very economical alternate to bolted construction.

The fabrication of any quality welded connection requires specific preparatory activities which are labor-intensive, and therefore expensive. Furthermore, when the connections involve joining jumbo sections, AISC specifications impose additional requirements, again increasing costs. It must be noted, however, that the time required to prepare the work is not directly proportional to the thickness of the materials involved. Therefore, for thin members (such as Group 1 or 2 shapes), preparation and assembly time may take more time than the actual welding, while in the case of jumbo shapes, the welding time will constitute a higher percentage of the total fabrication time.

When connections are bolted, the amount of time required for cutting, drilling, bolt installation and torquing dramatically increases as the material involved becomes thicker. Welded connections use material more efficiently, since they do not require splice plates. Therefore, the relative economics of bolting vs. welding change as the beam weights change. Welded connections gain an economic advantage as the members become thicker.

Case Study

Taking a W14x730 jumbo section made of A572 Grade 50 as an example, two full-strength butt splice connections will be compared: a bolted design and the welded alternative.

The bolted design required a total of 256, 1 1/8" diameter A325 bolts in double shear, with a bolt length of 1'2". The outer plate measured 18" x 4" x 8'5", and the inner plates were 7" x 3 ½" x 8'5". Only the flanges were bolted since the web had inadequate space to develop the required connection.

The welded alternative utilized prequalified CJP groove welds: B-U3a-S for the flanges, and B-U2-S for the web. Hand held semi-automatic submerged arc welding at a rate of 16 lbs/hr, at a 25% operating factor, yielded a net deposit rate of 4 lbs/hr.

Although the requirements for welding jumbo sections are more complicated than those for other materials, the necessary controls will assure product integrity. In this case, a cost savings of $4600 per splice more than justifies the extra effort.