Volume XIX, Number 1, 2002

Welding Innovation

Advancing Arc Welding Design and Practice Worldwide

A publication of the James F. Lincoln Arc Welding Foundation

Continuing the Tradition...

It was with humility that I recently accepted Duane Miller's offer to assist in the editing of *Welding Innovation*. This was true because I did not think it would be possible for me to fill the position left vacant by my predecessor Scott Funderburk. He covered *Welding Innovation* for several years as assistant editor, and he mentored well under Duane's watchful eye. I would expect that Scott will be sorely missed here in the application engineering department and by our readership. We all wish him the very best in his new assignment.

As is certainly the case with the roles of most professionals employed in industry today, we are required to wear many hats, and my primary position as senior application engineer has dovetailed nicely with my additional responsibility to *Welding Innovation.* The irony of this is that nearly twenty years ago, I was asked to assist the former editor, Richard Sabo, by suggesting a name for this publication. We finally settled on *The Welding Innovation Quarterly*, and that name became synonymous with high-quality, creative design engineering concepts coupled with fundamentally strong welding principles. If you needed answers to tough problems, then in return we were prepared as a group to provide high quality information. In principle, it is our objective to continue the rich tradition of *Welding Innovation,* and I would like to suggest that the articles in this issue of the magazine reflect our commitment to that objective.

Professor Henry Petroski's contribution to this issue, "The Fall of Skyscrapers," provides an analysis of the collapse of the World Trade Center towers resulting from the September 11 terrorist attack. In addition, Professor Petroski provides us with ideas regarding future design considerations, and the practicality of future skyscraper construction. In contrast, contributing writer Carla Rautenberg provides an article that presents Frank Gehry's free-form design of the Peter B. Lewis Building at the Weatherhead School of Management located on the Case Western Reserve University campus. The compelling design of the newly constructed Gehry design is a clear break from the historical constraints of rectangular designed structures.

We are confident that this issue of *Welding Innovation* will be thought-provoking, and we encourage your innovative input for future issues. I especially urge you to consider sharing with our readers the value of your own experience, in the form of a submission to the "Lessons Learned in the Field" column initiated by our senior design consultant, Omer W. Blodgett. See page 18 for another of Omer's timeless and valuable engineering lessons.

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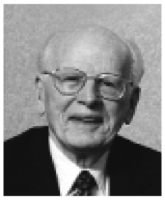
Welding INNOVATION

Cover: With a shimmering stainless steel skin concealing its intricate welded structure, the roof of the almost-completed Peter B. Lewis Building at Case Western Reserve University definitely makes an architectural statement. *See story on page 20.*



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Orthotropic Design Meets Cold Weather Challenges

An Overview of Orthotropic Steel Deck Bridges in Cold Regions

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Introduction

Initially developed by German engineers following World War II, orthotropic bridge design was a creative response to material shortages during the post-war period. Lightweight orthotropic steel bridge decks not only offered excellent structural characteristics, but were also economical to build (Troitsky 1987). Moreover, they could be built in cold climates at any time of the year. Engineers from around the world utilize this practical and economic system for all types of bridges. While concrete must be at or above 5 degrees Celsius to properly cure, it is physically possible to encapsulate and heat the concrete construction process; admittedly, this adds to construction costs (Mangus 1988), (Mangus 1991).

Orthotropic steel deck bridges have proven to be durable in cold regions. The orthotropic steel deck integrates the driving surface as part of the superstructure, and has the lowest total mass of any practicable system. In Europe, where the advantages of orthotropic design have been embraced with enthusiasm, there are more than 1,000 orthotropic steel deck bridges. In all of North America, there are fewer than 100 bridges of orthotropic design.

This article will give an overview of imaginative steel deck bridges currently in operation in Norway, Russia, Sweden, and Ukraine. The examples cover a matrix of rib types, superstructure types and various bridge types. Russia has developed a mass manufactured panelized orthotropic deck system and has devised special launching methods for cold regions. Russian engineers prefer the open rib design and have industrialized this system, while most other engineers prefer the closed rib. Researchers, as well as the owners of orthotropic steel deck bridges, continue to monitor the performance of various rib types (Figure 1).

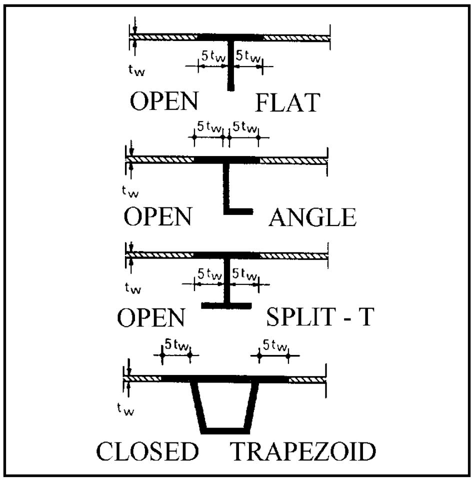

Figure 1. Rib designs.

In Norway

Nordhordland Floating Bridge

The Nordhordland Bridge across the Salhus fjord is Norway's second floating bridge and the world's largest floating bridge (Meaas, Lande, and Vindoy, 1994). The bridge was opened for traffic in 1994. The total bridge length is 1615 m and consists of a high level 369 m long cable stayed bridge and a 1246 m long floating bridge (Figure 2). The floating bridge consists of a steel box girder, which is supported on ten concrete pontoons and connected to abutments with transition elements in forged steel. The main elements are a high-level cable stayed bridge providing a ship channel and a floating bridge between the underwater rock Klauvaskallen and the other side of the fjord. The cable stayed bridge provides a clear ship channel. A 350 m long ramp is required to transition from the higher bridge deck on the cable

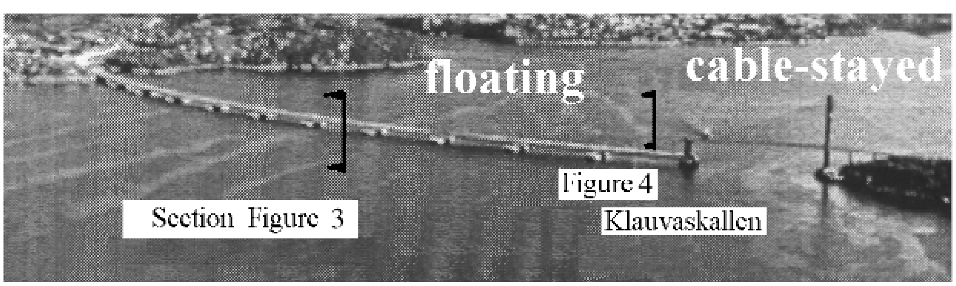

Figure 2. Nordhordland Floating Bridge across Salhus fjord of Norway.

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stayed bridge to the bridge deck 11 m above the waterline. The steel box girder of the floating bridge forms a circular arch with a radius of 1,700 m in the horizontal plane. The supporting pontoons are positioned with a center distance of 113 m and act as elastic supports for the girder, which is designed without internal hinges. The bridge follows the tidal variations by elastic deformations of the girder.

The steel box girder is the main load-carrying element of the bridge (Figure 3). The octagon girder is 5.5 m high and 13 m wide. The free height below the girder down to the waterline is 5.5 m and this allows for passage of small boats. The plate thickness varies from 14 mm to 20 mm. The plate stiffeners are in the traditional trapezoidal shape and they span in the longitudinal direction of the girder. The stiffeners are supported by cross-frames with center distance of maximum 4.5 m. At the supports on the pontoons, bulkheads are used instead of cross-frames. This is done because the loads in these sections are significantly larger than in the cross-frames. The plate thickness

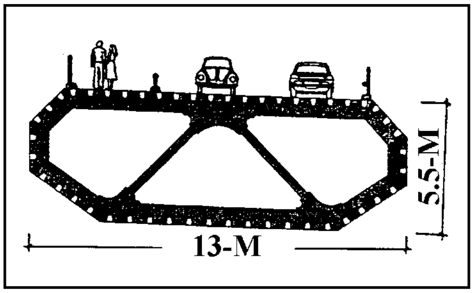

Figure 3. Nordhordland Floating Bridge.

in the bulkheads varies from 8 mm to 50 mm. The box girder is constructed in straight elements with lengths varying from 35 m to 42 m. The elements are welded together with a skew angle of 1.2° to 1.3° to accommodate the arch curvature in the horizontal plane. The cross section dimensions of the octagonal box girder are constant for the length of the bridge.

The stress level varies significantly over the length of the bridge. In the areas with the highest stresses, steel with a yield strength of 540 MPa is used.

Orthotropic bridge design was a creative response to material shortages during the postwar period

In the remainder of the bridge (in the cross-frames and bulkheads) normalized steel with yield strength of 355 MPa is used. The total steel weight of the box girder is 12,500 tons, of which approximately 3,000 tons are high strength steel. The elevated ramp is approximately 350 m long and has a grade of 5.7 percent (Figure 4). The elevated ramp is constructed with an orthotropic plate deck 12 mm thick and has 8 mm and 10 mm thick trapezoidal ribs 800 mm deep. T-shaped crossbeams support the ribs with a maximum center distance of 4.5 m. The main 1,200 mm deep box girders are located one at each edge of the ramp in order to maximize the stiffness about a vertical axis. The steel weight of the ramp is 1,600 tons.

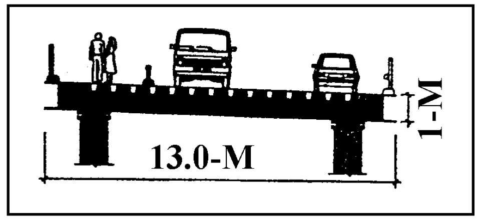

Figure 4. Nordhordland Floating Bridge.

Bybrua Bridge

The Bybrua cable stayed bridge has a main span of 185 m. The 15.5 m wide roadway superstructure was fabricated in the shop in 9.0 m sections (Figure 5). There is a combined pedestrian plus bicycles area on each side of the three traffic lanes. The cross section of the main span has a deck-plate 12 mm thick, but this increases to 16 and 20 mm at the cable anchorage. The bottom plate varies between 8 mm and 10 mm thickness, and the webs between 12 mm and 20 mm. At intervals of 3.0 m there are frames supporting the longitudinal stiffening system. In the bridge deck this is made up of standard trapezoidal ribs

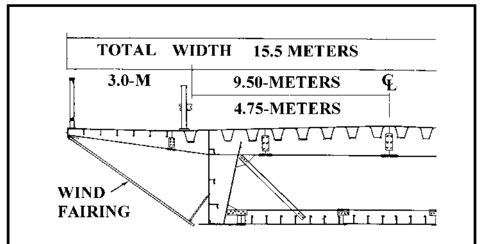

Figure 5. Bybrua Bridge.

from the German steel company Krupp, and in the bottom flange box section of bulb flats open ribs were utilized (Aune and Holand 1981). In the longitudinal direction the deck was divided into six fabricated sections, two of which were welded to the web sections. The box bottom was fabricated as three sections. The total steel weight is about 1,100 tons. All elements prefabricated in the shop were welded, as were the field splices in the deck, whereas "Huck" high tensile bolts were used in all other field joints. All field joints were calculated as friction connections. The whole steel structure is metallized with zinc and painted according to the specifications of the Norwegian Public Roads Administration. The superstructure received the maximum live load stresses during the erection of the bridge. The wearing surface of the bridge deck is the same as that developed by the Danish State Road Laboratories for the Lillebelt Bridge of Denmark.

Storda and Bomla Bridges

The "Triangle Link" project connects three islands off the Norwegian coast south of Belgen to the mainland with three bridges (Larson and Valen 2000). The entire project was completed in April 2001. The two orthotropic steel deck suspension bridges are known as the Storda Bridge and the Bomla Bridge. The Storda Bridge is 1,076 m long, has a main span of 677 m, with towers 97 m high and a vertical clearance of 18 m (Figure 6). The Bomla Bridge is 990 m long with a main span of 577 m and the tower height is 105 m. The roadway of both bridges is 9.7 m wide. Scanbridge AS of Norway fabricated the Bomla Bridge's steel approach superstructure, which was launched out over the tops of the columns from the shore. The steel components for the main span superstructure of the Storda Bridge were prefabricated in the Netherlands (Figure 6) and the main span superstructure of the Bomla Bridge was prefabricated in Italy. The orthotropic ribs for the Storda Bridge were prefabricated in France. The orthotropic sections were transported to the site by barge, and were lifted into position by a crane.

Researchers continue to monitor the performance of various rib types

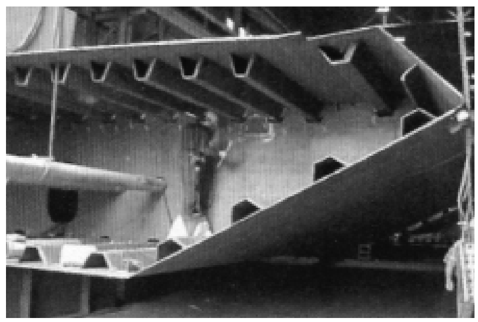

Figure 6. Storda Bridge.

In Russia

Russian engineers have standardized their orthotropic deck plates using open or flat plate ribs as shown in Figure 1. They have several launching solutions or standardized methods for pushing the superstructure across a river or gorge. There are a limited number of bridge case histories documented in English, but they provide an overall view of Russian techniques (Blank, Popov, and Seliverstov 1999). In the city of Arkhangel, Russia, a vertical lift record span bridge of 120.45 m was completed in 1990 (Stepanov 1991). The Berezhkovsky twin parallel bridges are multi-cell box girder bridges consisting of three spans of 110 m + 144.5 m +110 m. Each bridge has four traffic lanes 3.75 m wide. These bridges were the first to be launched with inclined webs (Surovtsev, Pimenov, Seliverstov, and Iourkine 2000).

Oka Bridge

The four-lane orthotropic twin box girder bridge crossing the Oka River on the bypass freeway around the city of Gorki, Russia, was opened to traffic in 1991 (Figure 7). The 966 m long superstructure consists of 2 spans x 84 m + 5 spans x 126 m + 2 spans x 84 m (Design Institute Giprotransmost 1991). This bridge is a single continuous superstructure with a fixed bearing 420 m away from one of the abutments. The total bridge width (29.5 m including steel traffic barriers) provides two sidewalks 1.5 m wide each, four traffic lanes, four safety shoulders and a center median. The total weight of steel for the superstructure is 10,635 tons, or 373 kg/m2. The orthotropic steel superstructure comprises five basic elements (Figure 7). There are two main box girders assembled from two L-shaped sections for the bottom face and sides. The intermediate orthotropic plate sections were used for the top flange of the two box girders, as well as the majority of the deck. The end sections of the orthotropic plate were panelized with tapered ends, because only sidewalk loading is required. The transverse diaphragms are steel trusses between the box girders. The diaphragms required extra steel beams at the bottom flange of the box girder above the bearings. The main box girder was shop fabricated in L-shaped sections that are 21 m long and 3.6 m deep. The intermediate orthotropic welded steel deck plate was shop fabricated in panelized sections 2.5 m wide and 11.5 m long.

The longitudinal ribs of the orthotropic deck and steel box girders are flat rib plates spaced at 0.35 m, and the spacing of transverse ribs is 3.0 m for both components. The stiffening ribs of the main girder are located on both sides of every web. The vertical split-T ribs of the box girders were aligned with the transverse ribs of the ribbed plate, thus creating the integral internal diaphragms. The longitudinal stiffening ribs are at a constant spacing

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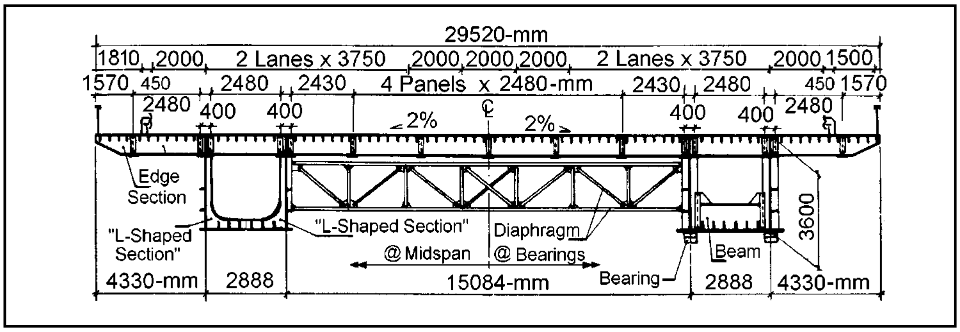

Figure 7. Twin box girder bridge crossing the Oka River, Russia (split–section).

along the bridge. Depending on the web thickness, additional vertical stiffening ribs were required between the diaphragms. The superstructure was erected using "continuous launching" from one bank of the Oka River. The shop-fabricated elements were added piece by piece to form a continuous structure at the "erection slip" area on this riverbank. The joints of the horizontal sections of the orthotropic deck and ribbed plates, as well as the joints of the web of the main girder, were automatically welded. The joints of the longitudinal ribs of the ribbed plate were manually welded. All the remaining joints used high strength bolts.

In Sweden

High Coast Suspension Bridge

The High Coast Suspension Bridge of Sweden is almost the same size as San Francisco's Golden Gate Bridge (Merging with Nature 1998). The main span is 1,210 m long with suspended

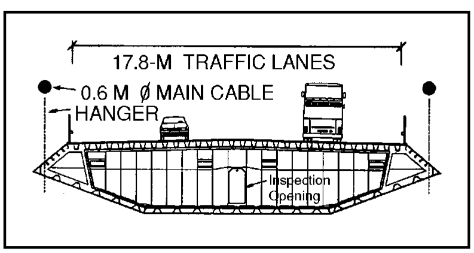

Figure 8. High Coast, Sweden, section.

side spans of 310 m and 280 m. The width of the roadway is 17.8 m, allowing for a possible future extension to 4 lanes (Figure 8). The distance between the main cables is 20.8 m and there are 20 m between the hangers. The girder is continuous through the towers extending 1,800 m from abutment to abutment where expansion joints and hydraulic buffers are located. The 48 box girder sections were fabricated at a shipyard in Finland (Pedersen 1997). The standard section is 20 m long with two

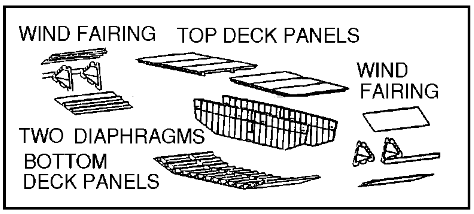

Figure 9. High Coast, Sweden components.

sets of hangers each and weighs 320 tons (Figure 9). The 20 m long panels for the deck, sides and bottom were fabricated with a maximum width of 10 m. They were fabricated from steel plates, typically 9 to 14 mm thick,10 m long and 3 to 3.3 m wide. The ribs were 20 m long trapezoidal ribs with a plate thickness of 6 to 8 mm.

The plates were placed on a plane and welded in the transverse and longitudinal direction and the trough stiffeners were fitted and welded longitudinally. Plates connecting the panels and the diaphragms were welded between ribs. The 20 m long edge sections and units for the transverse diaphragm, or bulkhead, were prefabricated. The bottom and inclined sides were placed first. Each 4 m deep transverse diaphragm or bulkhead was fitted. The edge sections were installed and finally the two deck panels were placed on top, completing a 20 m long subsection. The 31 bridge girder sections for the main span were transported from the fabrication yard in Finland on the three barges in the same way as the sections for the side spans, and erected with a floating sheerleg crane, 130 m boom, starting from mid-span and proceeding towards the towers (Edwards and Westergren 1999).

In Ukraine

South Bridge over the Dnipro River

The 1992 signature span of the South Bridge over the Dnipro [Dnepr] River in Kyiv [Kiev], Ukraine is an unsymmetrical cable stayed bridge with a main span of 271 m (Korniyiv and. Fuks 1994). The main span side of the H-tower is a continuous three-span steel box girder with orthotropic steel deck. The back-span superstructure on the opposite side of the H-tower is a segmental prestressed concrete box section. Concrete construction for the shorter back span of 60 m was used as a counter-weight mass equal to the longer orthotropic main span. The bridge carries a six-lane roadway, two rail tracks and four large-diameter water pipes (Figure 10). The total live

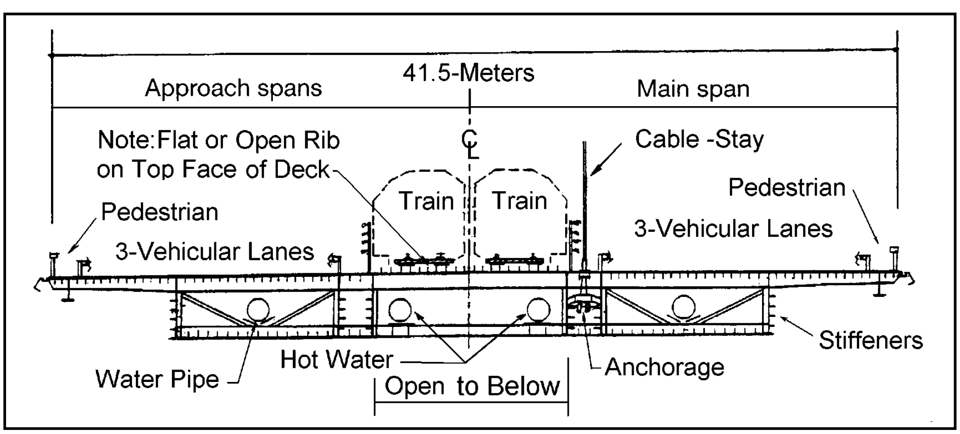

Figure 10. South Bridge (cable stayed) of Ukraine (split–section).

load is about 246 kN/m. The three-span (80.5 m + 90 m + 271 m) continuous steel box girders are made of low-alloy steel with a minimum yield strength of 390 MPa.

The bridge was divided into segments that were shop-welded. Field splices were either welded or joined by high-strength bolts. Bolting was used where automatic welding was impractical because of the short length of the weld or difficult access. The cross section of the twin two-cell box girder bridge consists of six vertical webs, the upper deck plate and the lower box flanges. The narrow cell is for the cable stayed bridge anchorage. In the central portion of the cross section that carries the two hot water supply pipes, the lower flange was omitted to preclude the undesirable effects of unequal heating inside a closed box. The bearings at the piers permit lateral displacement of the superstructure because of the 41.5 m bridge width. The orthotropic decks, bottom flanges of the boxes and the webs have open flat-bar stiffening ribs, a common feature in Ukrainian and Russian bridges. Longitudinal flat or open ribs were placed on the top face of the orthotropic deck plate under the rail tracks, thus avoiding intersections of longitudinal and transverse stiffeners.

This facilitated fabrication, at the same time precluding stress concentrations at crossing welds that would be susceptible to fatigue under dynamic train loading. Longitudinal ribs under the train tracks have a depth in excess of the design requirements, which permitted longitudinal profile adjustments of the tracks after erection of the bridge superstructure. The steel girders were pre-assembled on the bank of the south and erected by launching. The twin two-cell box girders were equipped with a launching nose and stiffened with a temporary strut system (Rosignoli1999). Single erection rollers were used at the tops of supports and had a friction factor of less than 0.015. The erection of the 271 m main span was accomplished with two false work supports providing three equal spans of about 90 m. At the H-tower of the cable stayed bridge, hinged conditions are provided by supports with limited rotational capability in the vertical and the horizontal planes. The torsional rigidity of the bridge is supplied by the two planes of cable stays, plus the stiffness of the closed box sections. Under one-sided loading of the bridge (three traffic lanes), the deck cross slope of 0.3% was measured in field tests, less than the calculated value of 0.35%. Eccentric hinged connections between the bottom flanges of the stiffening girders and the H-tower were constructed, considerably reducing the bending moments in the girders.

Conclusion

The foregoing examples illustrate a range of creative responses to the challenge of designing and constructing orthotropic steel deck bridges in cold weather regions. The versatility, economy and structural integrity of welded orthotropic design undoubtedly will continue to inspire bridge designers and structural engineers in the 21st century.

Figure Credits

Figure 1 from Ballio, G., Mazzolani, F. M. 1983, "Theory and Design of Steel Design Structures," Chapman & Hall Ltd. courtesy of Dr. Mazzolani; Figures 2, 3, & 4 courtesy of Dr. Ing. A-Aas–Jakobsen, AS Structural Engineering Consultants, Oslo, Norway; Figure 5 adapted from Aune, Petter, and Holand, Ivar (1981); Figure 6 courtesy of Mr. L. Adelaide of Profilafroid, France; Figures 8 & 9 courtesy of Claus Pedersen of Mondberg & Thorsen AS, Copenhagen, Denmark; Figures 7 & 10 courtesy of IABSE International Association of Bridge Structural Engineers.

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Design File

Designing Fillet Welds for Skewed T-joints—Part 1

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

Introduction

Detailing fillet welds for 90-degree T-joints is a fairly straightforward activity. Take the 90-degree T-joint and skew it—that is, rotate the upright member so as to create an acute and obtuse orientation, and the resultant geometry of the fillet welds becomes more complicated (see Figure 1). The greater the degree of rotation, the greater the difference as compared to the 90-degree counterpart.

A series of equations can be used to determine weld sizes for various angular orientations and required throat dimensions. Since the weld sizes on either side of the joint are not necessarily required to be of the same size, there are a variety of combinations that can be used to transfer the loads across the joint. While there are theoretical savings to be seen by optimizing the combinations of weld sizes, rarely do such efforts result in a change in fillet weld size of even one standard size.

Codes prescribe different methods of indicating the required weld size. These are summarized herein.

When acute angles become smaller, the difficulty of achieving a quality weld in the root increases. The *AWS D1.1 Structural Welding Code* deals with this issue by requiring the consideration of a Z-loss factor.

This edition of Design File addresses the situation where the end of the upright member in the skewed T-joint is parallel to the surface of the other member. A future Design File column will consider the situation in which the upright member has a square cut on the end, resulting in a gap on the obtuse side. Also to be addressed in the future are weld options other than fillet welds in skewed T-joints.

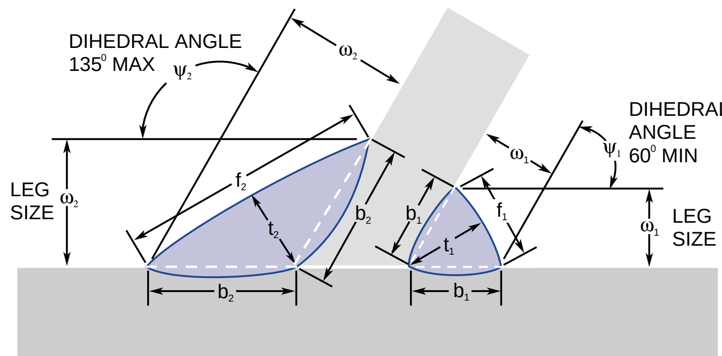

Figure 1. Equal throat sizes ($t_1 = t_2$).

The Geometry

Figure 1 provides a visual representation of the issue. For the 90-degree orientation, the weld throat is 70.7% of the weld leg dimension. This relationship does not hold true for fillet welds in skewed joints. On the obtuse side, the weld throat is smaller than what would be expected for a fillet weld of a similar leg size in a 90-degree joint, and the opposite is the case for the acute side. These factors must be considered when the fillet weld leg size is determined and specified.

Careful examination of the fillet welds on the skewed joint raises this question: What is the size of the fillet weld in a skewed joint?

Figure 1 illustrates the fillet weld leg size for a skewed T-joint, and is designated by "ω." This, however, is inconsistent with *AWS Terms and Definitions* (AWS A3.0-94) which defines a "fillet weld leg" as "The distance from the joint root to the toe of the fillet weld." According to this definition, and as shown in Figure 1, the fillet weld leg is dimension "b." The dimension that is labeled "ω" is the distance from a member to a parallel line extended from the bottom weld

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toe. While not technically correct according to AWS A3.0, it is the dimension and terminology used when fillet welds in skewed joints are discussed in the *AWS D1.1 Structural Welding Code*, as well as other AWS publications (i.e., *The Welding Handbook*, ninth edition, volume 1). Such terminology will be used here.

This raises an additional question: What would a weld inspector actually measure when dealing with a fillet weld in a skewed T-joint? Conventional fillet weld gauges could be used to measure the obtuse side's fillet weld leg dimension "ω" as shown in Figure 1. Dimension "b" would be difficult to measure directly since the location of the weld root cannot be easily determined. Welds on the acute side are impossible to measure using conventional fillet weld gauges. The face dimension "f," however, offers an easy alternative: when this dimension is known for the weld size and the dihedral angle, the welder and inspector can easily determine what the actual size is by using a pair of dividers. Alternately, a series of simple gauges of various widths could be made to directly compare the requirements to the actual weld size. Thus, dimension "f" may be important for controlling weld sizes in skewed T-joints.

When sizing a fillet weld for 90-degree T-joints or skewed T-joints, the starting point is to determine the required throat size needed to resist the applied loads. From the throat dimension, the fillet weld leg size can be determined. Three options will be considered:

Where the throat size is the same on either side of the joint (see Figure 1)

To determine the required fillet weld size for a given throat, the following relationship can be used:

$$\omega = 2\,t \sin\left(\frac{\Psi}{2}\right) \qquad (1)$$

The width of the face of the weld ("f") can be found from this equation:

$$f = 2\,t \tan\left(\frac{\Psi}{2}\right) \qquad (2)$$

Dimension "b," that is, the 'true' fillet leg size, can be found from this relationship:

$$b = \frac{t}{\cos\left(\frac{\Psi}{2}\right)} \qquad (3)$$

Finally, the cross-sectional area of the weld metal can be determined from the following:

$$A = t^2 \tan\left(\frac{\Psi}{2}\right) \qquad (4)$$

Where the leg size is the same on both sides (see Figure 2)

If the designer decides to make both welds with the same leg size (as is illustrated in figure 2), the first required step is to determine the composite total dimension of the two

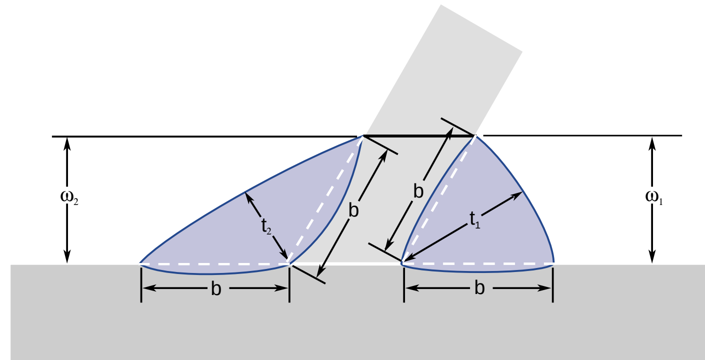

Figure 2. Equal fillet weld leg sizes ($\omega_1 = \omega_2$).

throat sizes. This dimension "t_T" is then inserted into the following equations to determine the two throats "t_1" and "t_2."

$$t_1 = t_T \frac{\cos\left(\frac{\Psi_1}{2}\right)}{\cos\left(\frac{\Psi_1}{2}\right) + \cos\left(\frac{\Psi_2}{2}\right)} \qquad (5)$$

$$t_2 = t_T \frac{\cos\left(\frac{\Psi_2}{2}\right)}{\cos\left(\frac{\Psi_1}{2}\right) + \cos\left(\frac{\Psi_2}{2}\right)} \qquad (6)$$

Equations 1 – 4 can be used to find the corresponding fillet weld leg size, face dimension, "b" dimension, and cross-sectional area. These calculations will be made using the applicable throat dimension "t" determined from equations 5 and 6, not the total throat dimension "t_T" used in equations 5 and 6.

Where a minimum quantity of weld metal is used (see Figure 3)

Even a casual review of Figure 1 shows that, when fillet weld leg sizes are specified to be of the same size on either side of the skewed T-joint, the use of weld metal is as efficient as could be. Minimum weld metal can be obtained by taking advantage of the more favorable condition that results on the acute side where a greater weld throat can be obtained for the same quantity of metal that would be placed on the obtuse side.

To minimize the volume of weld metal used in the combination of the two welds, the following equations may be used once the total throat dimension "t_T" is known:

$$t_1 = \frac{t_T}{1 + \tan^2\left(\frac{\Psi_1}{2}\right)} \qquad (7)$$

$$t_2 = \frac{t_T}{1 + \tan^2\left(\frac{\Psi_2}{2}\right)} \qquad (8)$$

Although the preceding calculations are not particularly difficult, Table 1 has been provided to simplify the process. Columns A and B are used to determine fillet weld leg sizes and face widths for various dihedral angles. To obtain the required fillet weld size, the calculated throat is multiplied by the factor in Column A. Face widths can be found following the same procedure.

If the same leg size is desired on either side of the joint, columns C-E are used. In this case the total weld throat "t_T" is used, as opposed to what was done with columns A and B.

For the minimum weld volume, columns F–H can be used. Again, the total weld throat "t_T" is used.

As will be discussed below, for dihedral angles of 30–60 degrees, the D1.1 Code requries the application of a Z-loss factor. Thus, the values in Table 1 that apply to dihedral angles where this applies are shown in blue numbers to remind the user to incorporate this factor into the weld throat sizes.

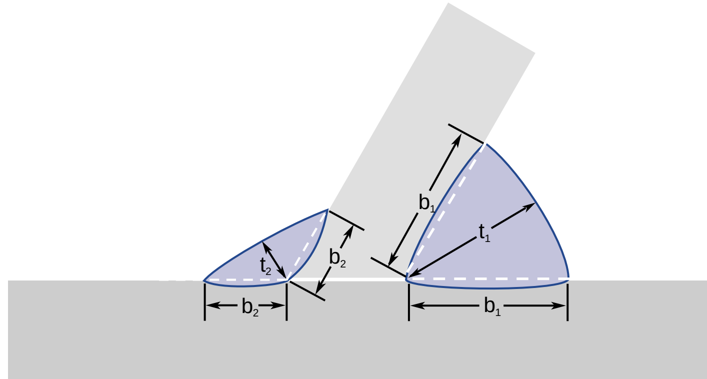

Figure 3. Minimum weld volume.

Influence of Dihedral Angle

AWS D1.1 Structural Welding Code–Steel provides for five groupings of skewed T-joints, depending on the range of sizes of the dihedral angle: a) Obtuse angles greater than 100 degrees, b) angles of 80–100 degrees, c) acute angles of 60–80 degrees, d) acute angles of 30–60 degrees, and e) acute angles of less than 30 degrees. Each is dealt with in a slightly different manner.

Table 1.

Dihedral Angle	Leg & Face Dimensions Multipy by t		Same Leg Size Multipy by t_T			Minimum Weld Volume Multipy by t_T		
Ψ phi1 deg	A leg size	B face width	C throat	D leg size	E face width	F throat	G leg size	H face width
30	0.517	0.536	0.788	0.408	0.422	0.933	0.483	0.536
35	0.601	0.630	0.760	0.457	0.479	0.910	0.547	0.630
40	0.684	0.728	0.733	0.501	0.533	0.883	0.604	0.728
45	0.765	0.828	0.707	0.541	0.585	0.854	0.653	0.828
50	0.845	0.932	0.682	0.576	0.635	0.822	0.694	0.932
55	0.923	1.04	0.657	0.607	0.684	0.787	0.726	1.04
60	1.00	1.15	0.634	0.634	0.731	0.750	0.750	1.15
65	1.07	1.27	0.611	0.656	0.778	0.712	0.764	1.27
70	1.15	1.40	0.588	0.674	0.823	0.671	0.770	1.40
75	1.22	1.53	0.566	0.689	0.868	0.630	0.766	1.53
80	1.29	1.68	0.544	0.699	0.912	0.587	0.755	1.68
85	1.35	1.83	0.522	0.705	0.956	0.544	0.735	1.83
90	1.41	2.00	0.500	0.707	0.999	0.500	0.707	2.00
95	1.47	2.18	0.478	0.705	1.043	0.457	0.673	2.18
100	1.53	2.38	0.456	0.699	1.087	0.414	0.633	2.38
105	1.59	2.60	0.434	0.689	1.131	0.371	0.589	2.60
110	1.64	2.85	0.412	0.675	1.175	0.329	0.540	2.85
115	1.69	3.14	0.389	0.656	1.221	0.289	0.488	3.14
120	1.73	3.46	0.366	0.634	1.267	0.250	0.434	3.46
125	1.77	3.84	0.343	0.608	1.314	0.214	0.379	3.84
130	1.81	4.28	0.318	0.577	1.363	0.179	0.324	4.28
135	1.85	4.82	0.293	0.542	1.413	0.147	0.271	4.82
140	1.88	5.48	0.267	0.502	1.465	0.117	0.221	5.48
145	1.91	6.33	0.240	0.458	1.519	0.091	0.173	6.33
150	1.93	7.44	0.212	0.409	1.576	0.067	0.130	7.44

Blue numbers indicate that Z-loss factors must be considered.

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Obtuse angles greater than 100 degrees
For this category, contract drawings should show the required effective throat. Shop drawings are to show the required leg dimension, calculated with equation 1, or by using columns C or D of Table 1 (AWS D1.1-2002, para 2.2.5.2, 2.3.3.2).

Angles of 80–100 degrees
For this group, shop drawings are required to show the fillet leg size (AWS D1.1-2002, para 2.2.5.2). While not specifically stated in the code, the assumption is that contract drawings also show this dimension.

Angles of 60–80 degrees
For this category, contract drawings should show the required effective throat. Shop drawings are to show the required leg dimension (AWS D1.1-2002, para 2.2.5.2, 2.3.3.2)

Angles of 30–60 degrees
Contract drawings are to show the effective throat. Shop drawings are required to "show the required leg dimensions to satisfy the required effective throat, increased by the Z-loss allowance ... " (AWS D1.1-2002, para 2.2.5.2, 2.3.3.3). The Z-loss factor is used to account for the likely incidence of poor quality welding in the root of a joint with a small included angle. The amount of poor quality weld in the root of the joint is a function of the dihedral angle, the welding process, and the position of welding. Table 2.2 of D1.1 summarizes this data as contained below:

Table 2. Z-loss dimension.

Dihedral Angle Ψ	Position of Welding V or OH			Position of Welding H or F		
	Process	Z (in.)	Z (mm)	Process	Z (in.)	Z (mm)
$60^\circ > \Psi \geq 45^\circ$	SMAW	1/8	3	SMAW	1/8	3
	FCAW-S	1/8	3	FCAW-S	0	0
	FCAW-G	1/8	3	FCAW-G	0	0
	GMAW	N/A	N/A	GMAW	0	0
$45^\circ > \Psi \geq 30^\circ$	SMAW	1/4	6	SMAW	1/4	6
	FCAW-S	1/4	6	FCAW-S	1/8	3
	FCAW-G	3/8	10	FCAW-G	1/4	6
	GMAW	N/A	N/A	GMAW	1/4	6

Once the Z-loss dimension has been determined, it is added to the required throat dimension. Even though part of the weld in the root is considered to be of such poor quality as to be incapable of transferring stress, the resultant weld will contain sufficient quality weld metal to permit the transfer of imposed loads. Figure 4 illustrates this concept.

The data in Table 1 that applies to dihedral angles of 30–60 degrees has been shown in blue numbers because these values must be modified to account for the Z-loss factors. Such a modification has not been done for the data in the Table.

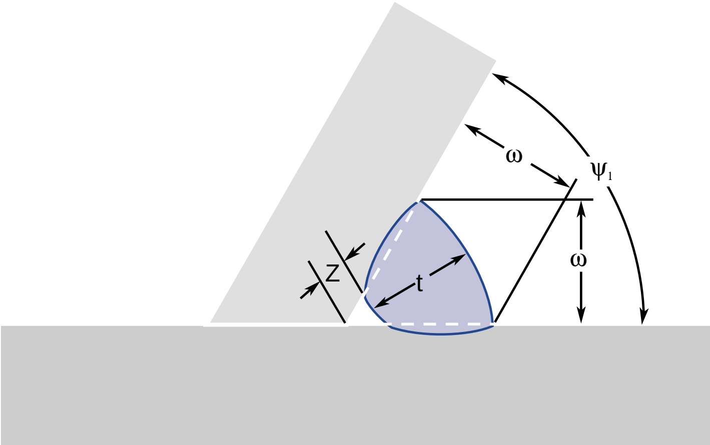

Figure 4. Z-loss.

Acute angles less than 30 degrees
The D1.1 code says that welds in joints with dihedral angles of less than 30 degrees "shall not be considered as effective in transmitting applied forces …" and then goes on to discuss a single exception related to tubular structures. In that exception, with qualification of the welding procedure specification, such welds may be used for transferring applied stresses. For plate (e.g., non-tubular) applications, such an option is not presented in the code.

The practical application of this principle is that when welds are placed on the acute side, no capacity is assigned to the weld. Rather, the full load is assumed to be transferred with the weld on the obtuse side.

Practical Considerations

The most straightforward, and easiest, approach to determining the required weld size is to assume two welds with equal throat dimensions will be used, calculate the required weld throat dimension, and then calculate the required fillet weld leg size, using either equation 1 or Table 1, columns A and B. Simple? Yes. Best? Let's see.

The optimizing method that uses equations 6 and 7 will result in reduced weld metal volumes. But, reduced how much? The significance increases with greater rotations from the 90-degree T-joint orientation. For angles of 80, 70, and 60 degrees, the differences in weld volume are approximately 3, 12 and 25%. However, note that these differences are functions of the leg size squared. Accordingly, the change in leg size is approximately 1, 6, and 13%. In practical terms, for dihedral angles between 60 and 120 degrees, there will not be a standard fillet weld leg size until the welds become quite large. In the case of a 70-degree dihedral angle, for example, and assuming a 1/8 in. increment for standard sizing of fillet welds over 1 in. leg size, the leg size would need to be 2 in. before the optimized weld size would result in a smaller weld. For a 2 mm standard size, this would equal a 34 mm fillet.

For angles less than 60 degrees, there can be significant differences in weld volume. These are situations where the Z-loss factor must be considered as well. Thus, for angles of 30–60 degrees, optimization of weld size makes sense, and the Z-loss factors can be considered at the same time.

It must be recognized that other code provisions may further affect these results. For example, when optimized for minimum weld volume, welds on the obtuse side may be smaller than minimum fillet weld sizes as contained in Table 5.8 of D1.1. The calculated sizes, if less than these minimums, must be increased to comply with this requirement.

There does not appear to be any intrinsic value in having welds on opposite sides of the skewed T-joint be of the same size. If this approach is used, the resultant weld volumes will fall somewhere between the results for the same sized throat and the optimized sizes.

After the welds are detailed, the joint must be welded. Practical considerations apply here too. It must be recognized that the ratio of the face width “f” to the throat dimension “t” constitutes the equivalent of a width-to-depth ratio for the root pass. On the obtuse side, this ratio is large, exceeding 1:6 for dihedral angle of 106 degrees or more. It is very difficult to get a single weld bead to “wash” out this wide without electrode manipulation (weaving). On the acute side, the ratio is less than 1:2 for angles of 62 degrees. This can lead to width-to-depth ratio cracking.

Recommendations

When determining fillet weld details for skewed T-joints with dihedral angles from 60–120 degrees, it rarely matters which method of proportioning of weld sizes is used. Using equal throat dimensions is a straightforward method, similar to what is typically done for fillet welds on either side of a 90-degree T-joint. Unless the weld size is large, optimizing it will probably not result in a smaller specified weld size.

For fillet welds on skewed T-joints with dihedral angles from 30–60 degrees, the Z-loss factor must be considered. Based on the specific dihedral angle, the welding process, and the position of welding, the Z-loss factor can be determined, and this dimension added to the required weld throat dimension.

It is important to consider how these dimensions should be communicated between the designer, fabricator, welder and inspector. The face dimension is a practical means of verifying that the proper weld size has been achieved.

Opportunities

Lincoln Electric Technical Programs

Welding Technology Workshop

June 10-14, 2002
July 29 – August 2, 2002
The purpose of this program is to introduce or enhance knowledge of current thinking in arc welding safety, theory, processes, and practices. The course is designed primarily for welding instructors, supervisors and professional welders. Fee: $395.

Welding of Aluminum Alloys, Theory and Practice

October 15-18, 2002
Designed for engineers, technologists, technicians and welders who are already familiar with basic welding processes, this technical training program provides equal amounts of classroom time and hands-on welding. Fee: $595.

Space is limited, so register early to avoid disappointment. For full details, see

www.lincolnelectric.com/knowledge/training/seminars/

Or call 216/383-2240, or write to Registrar, Professional Programs, The Lincoln Electric Company, 22801 St. Clair Avenue, Cleveland, OH 44117-1199.

The Fall of Skyscrapers

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Editor's Note: This article first appeared in American Scientist*, Volume 90, January-February 2002. It is reprinted here with permission. Copyright, Henry Petroski, 2002. The article is reprinted as written earlier this year. In the ensuing months, many investigations have been performed, and the level of understanding of some of the technical aspects of the World Trade Center collapse have increased. The results of the FEMA investigation discussed in this article are now available at www.house.gov/science/hot/wtc/wtcreport.htm*

Figure 1. With the collapse of the World Trade Center towers, the fate of future skyscraper projects has come into question.

The terrorist attacks of September 11, 2001, did more than bring down the World Trade Center towers. The collapse of those New York City megastructures, once the fifth and sixth tallest buildings in the world, signaled the beginning of a new era in the planning, design, construction and use of skyscrapers . For the foreseeable future, at least in the West, there are not likely to be any new super-tall buildings proposed, and only those currently under construction will be added to the skylines of the great cities of the world. Even the continued occupancy of signature skyscrapers may come under scrutiny by their prime tenants.

Since two hijacked airplanes loaded with jet fuel were crashed within about 15 minutes of each other into the two most prominent and symbolic structures of lower Manhattan, the once reassuringly low numbers generated by probabilistic risk assessment seem irrelevant. What happened in New York ceased being a hypothetical, incredible or ignorable scenario. From now on, structural engineers must be prepared to answer harder questions about how skyscrapers will stand up to the impact of jumbo jets and, perhaps more important, how they will fare in the ensuing conflagration. Architects will likely have to respond more to questions about stairwells and evacuation routes than to those about facades and spires. Because of the nature of skyscrapers, neither engineers nor architects will be able to find answers that will satisfy everyone.

Inclination, Not Economy

Although the idea of the skyscraper is modern, the inclination to build upward is not. The Great Pyramids, with their broad bases, reached heights unapproached for the next four millennia. But even the great Gothic cathedrals, crafted of bulky stone into an aesthetic of lightness and slenderness, are dwarfed by the steel and reinforced concrete structures of the 20th century. It was modern building materials that made the true skyscraper structurally possible, but it was the mechanical device of the elevator that made the skyscraper truly practical. Ironically, it is also the elevator that has had so much to do with limiting the height of most tall buildings to about 70 or 80 stories. Above that, elevator shafts occupy more than 25 percent of the volume of a tall building, and so the economics of renting out space argues against investing in greater height.

The World Trade Center towers were 110 stories tall, but even with an elaborate system of local and express elevators, the associated sky lobbies and utilities located in the core still removed almost 30 percent of the towers' floor area from the rentable space category. By all planning estimates, the World Trade Center towers should have been viewed as a poor investment and so might not have been undertaken as a strictly private enterprise. In fact, it was the Port of New York Authority, the bi-state governmental entity now known as the Port Authority of New York and New Jersey, that in the 1960s undertook to build the towers. With its ability to issue bonds, the Port Authority could afford to undertake a financially risky project that few corporations would dare.

Sometimes private enterprise does engage in similarly questionable investments, balancing the tangible financial risk with the intangible gain in publicity, with the hope that it will translate ultimately into profit. This was the case with the Empire State Building, completed in 1931 and now the seventh tallest building in the world. Although it was not heavily occupied at first, the cachet of the world's tallest building made it a prestigious address and added to its real-estate value. The Sears Tower stands an impressive 110 stories tall, the same count that the World Trade Center towers once claimed. This skyscraper gained for its owner the prestige of having its corporate name associated with the tallest building in the world. The Sears Tower, completed in 1974, one year after the second World Trade Center tower was finished, held that title for more than 20 years-until the twin Petronas Towers were completed in Kuala Lumpur, Malaysia, in 1998, emphasizing the rise of the Far East as the location of new megastructures.

The Port Authority could undertake a financially risky project that few corporations would dare

Building Innovation

It is not only the innovative use of elevators, marketing and political will that has enabled super-tall buildings to be built. A great deal of the cost of such a structure is in the amount of materials it contains, so lightening the structure lowers its cost. Innovative uses of building materials can also give a skyscraper more desirable office space. Now more than 70 years old, the steel frame of the Empire State Building has closely spaced columns, which break up the floor space and limit office layouts. In contrast, the World Trade Center employed a tubular-construction principle, in which closely spaced steel columns were located around the periphery of the building. Sixty-foot-long steel trusses spanned between these columns and the inner structure of the towers, where further columns were located, along with the elevator shafts, stairwells and other non-exclusive office space. Between the core and tube proper, the broad columnless space enabled open, imaginative and attractive office layouts.

The tubular concept was not totally new with the World Trade Center, it having been used in the diagonally braced and tapered John Hancock Center, completed in Chicago in 1969. The Sears Tower is also a tubular structure, but it consists of nine 75-foot(23-meter)-square tubes bundled together at the lower stories. The varying heights of the tubes give the Sears Tower an ever-changing look, as it presents a different profile when viewed from the different directions from which one approaches it when driving the city's expressways. When new to the Manhattan skyline, the unrelieved 209-foot(64-meter)-square plans and unbroken 1,360-foot(415-meter)-high profiles of the twin World Trade Center buildings came in for considerable architectural criticism for their lack of character. Like the Sears Tower, however, when viewed from different angles, the buildings, especially as they played off against each other, enjoyed a great aesthetic synergy. The view of the towers from the walkway of the Brooklyn Bridge was especially striking, with the stark twin monoliths echoing the twin Gothic arches of the bridge's towers.

Although the World Trade Center towers did look like little more than tall prisms from afar, the play of the ever-changing sunlight on their aluminum-clad columns made them new buildings by the minute. From a closer perspective, the multiplicity of unbroken columns corseting each building also gave it an architectural texture.

The tubular concept was not totally new with the WTC

The close spacing of the columns was dictated by the desire to make the structure as nearly a perfect tube as possible. A true tube, like a straw, would be unpunctured by peripheral openings, but since skyscrapers are inhabited by people, windows are considered a psychological must. At the same time, too-large windows in very tall buildings can give some occupants an uneasy feeling. The compromise struck in the World Trade Center was to use tall but narrow windows between the steel columns. In fact, the width of the window openings was said to be less than the width of a person's shoulders, which was intended by the designers to provide a measure of reassurance to the occupants. Since the terrorist attack, however, one of the most haunting images of those windows is of so many people standing sideways in the openings, clinging to the columns and, ultimately, falling, jumping or being carried to their death.

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Failure Analyses

Terrorists first attempted to bring the World Trade Center towers down in 1993, when a truck bomb exploded in the lower-level public garage, at the base of the north tower. Power was lost in the tower and smoke rose through it. It was speculated that the terrorists were attempting to topple the north tower into the south one, but even though several floors of the garage were blown out, the structure stood. There was some concern among engineers then that the basement columns, no longer braced by the garage floors, would buckle, and so they were fitted with steel bracing before the recovery work proceeded. After that attack, access to the underground garage was severely restricted, and security in the towers was considerably increased. No doubt the 1993 bombing was on the minds of many people when the airplanes struck the towers last September.

As they had in the earlier bombing, the World Trade Center towers clearly survived the impact of the Boeing 767 airliners. Given the proven robustness of the structures to the earlier bombing assault, the thought that the buildings might actually collapse was probably far from the minds of many of those who were working in them on September 11. It certainly appears not to have been feared by the police and fire fighters who rushed in to save people and extinguish the fires. Indeed, the survival of the World Trade Center after the 1993 bombing seems to have given an unwarranted sense of security that the buildings could withstand even the inferno created by the estimated 20,000 gallons (76,000 liters) of jet fuel that each plane carried. (That amount of fuel has been estimated to have an energy content equivalent to about 2.4 million sticks of dynamite.)

The survival of the WTC after the 1993 bombing seems to have given an unwarranted sense of security

Steel buildings are expected to be fireproofed, and so the World Trade Center towers were. However, fireproofing is a misnomer, for it only insulates the steel from the heat of the fire for a limited period, which is supposed to be enough time to allow for the fire to be brought under control, if not extinguished entirely. Unfortunately, jet fuel burns at a much higher temperature than would a fire fed by normal construction materials and the customary furniture and contents found in an office building. Furthermore, conventional fire-fighting means, such as water, have little effect on burning jet fuel. The World Trade Center fire, estimated to have produced temperatures as high as of the order of the melting point of steel, continued unabated. It has been speculated that some of the steel beams and columns of the structure that were not destroyed by the impact eventually may have been heated close to if not beyond their melting point, but this appears to have been unlikely.

Even if it did not melt, the prolonged elevated temperatures caused the steel to expand, soften, sag, bend and creep. The intense heat also caused the concrete floor, no longer adequately supported by the steel beams and columns in place before the impact of the airplane, to crack, spall and break up, compromising the synergistic action of the parts of the structure. Without the stabilizing effect of the stiff floors, the steel columns still intact became less and less able to sustain the load of the building above them. When the weight of the portion of the building above became too much for the locally damaged and softened structure to withstand, it collapsed onto the floors below. The impact of the falling top of the building on the lower floors, whose steel columns were also softened by heat transfer along them, caused them to collapse in turn, creating an unstoppable chain reaction. The tower that was struck second failed first in part because the plane hit lower, leaving a greater weight to be supported above the damaged area. (The collapse of the lower floors of the towers under the falling weight of the upper floors occurred for the same reason that a book easily supported on a glass table can break that same table if dropped on it from a sufficient height.)

Within days of the collapse of the towers, failure analyses appeared on the Internet and in engineering classrooms. Perhaps the most widely circulated were the mechanics-based analysis of Zdenek Bazant of Northwestern University and the energy approach of Thomas Mackin of the University of Illinois at Urbana-Champaign. Each of these estimated that the falling upper structure of a World Trade Center tower exerted on the lower structure a force some 30 times what it had once supported. Charles Clifton, a New Zealand structural engineer, argues that the fire was not the principal cause of the collapse. He thinks that it was the damaged core rather than the exterior tube columns that succumbed first to the enormous load from above. Once the core support was lost on the impacted floors, there was no stopping the progressive collapse, which was largely channeled by the structural tube to occur in a vertical direction. In the wake of the World Trade Center disaster, the immediate concerns were, of course, to rescue as many people as might have survived. Unfortunately, even to recover most of the bodies proved an ultimately futile effort. The twin towers were gigantic structures. Each floor of each building encompassed an acre, and the towers enclosed 60 million cubic feet each. Together, they contained 200,000 tons of steel and 425,000 cubic yards (325 cubic meters) or about 25,000 tons of concrete. The pile of debris in some places reached as high as a ten-story building. A month after the terrorist

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attack, it was estimated that only 15 percent of the debris had been removed, and it was estimated that it would take a year to clear the site.

Forensic Engineering

Among the concerns engineers had about the clean-up operation was how the removal of debris might affect the stability of the ground around the site. Because the land on which the World Trade Center was built had been part of the Hudson River, an innovative barrier had to be developed at the time of construction to prevent river water from flowing into the basement of the structures. This was done with the construction of a slurry wall, in which the water was held back by a deep trench filled with a mudlike mixture until a hardened concrete barrier was in place. The completed structure provided a watertight enclosure, which came to be known as the "bathtub" within which the World Trade Center was built. The basement floors of the twin towers acted to stabilize the bathtub, but these were crushed when the towers broke up and collapsed into the enclosure. Early indications were that the bathtub remained intact, but in order to be sure its walls do not collapse when the last of the debris and thus all the internal support is removed, vulnerable sections of the concrete wall were being tied back to the bedrock under the site even as the debris removal was proceeding.

Atop the pile of debris, the steel beams and columns were the largest and most recognizable parts in the wreckage. The concrete, sheetrock and fireproofing that were in the building were largely pulverized by the collapsing structure, as evidenced by the ubiquitous dust present in the aftermath. (A significant amount of asbestos was apparently used only in the lower floors of one of the towers, bad publicity about the material having accelerated during the construction of the World Trade Center. Nevertheless, in the days after the collapse, the once-intolerant Environmental Protection Agency declared the air safe.) The grille-like remains of the buildings' facades, towering precariously over what came to be known as Ground Zero, became a most eerie image. Though many argued for leaving these cathedral wall-like skeletons standing as memorials to the dead, they posed a hazard to rescue workers and were in time torn down and carted away for possible future reuse in a reconstructed memorial. As is often the case following such a tragedy, there was also some disagreement about how to treat the wreckage generally. Early on, there was clearly a need to remove as much of it from the site as quickly as possible so that what survivors there might be could be uncovered. This necessitated cutting up steel columns into sections that could fit on large flatbed trucks. Even the disposal of the wreckage presented a problem. Much of the steel was marked for immediate recycling, but forensic engineers worried that valuable clues to exactly how the structures collapsed would be lost.

All of the speculations about the mechanism of collapse are in fact hypotheses

All of the speculations of engineers about the mechanism of the collapse are in fact hypotheses, theories of what might have happened. Although computer models will no doubt be constructed to test those hypotheses and theories, actual pieces of the wreckage may provide the most convincing confirmation that the collapse of the structures did in fact progress as hypothesized. Though the wreckage may appear to be hopelessly jumbled and crushed, telltale clues can survive among the debris. Pieces of partially melted steel, for example, can provide the means for establishing how hot the fire burned and where the collapse might have initiated. Badly bent columns can give evidence of buckling before and during collapse. Even the scratches and scars on large pieces of steel can be useful in determining the sequence of collapse. This will be the task of teams of experts announced shortly after the tragedy by the American Society of Civil Engineers and the Federal Emergency Management Agency. Also in the immediate wake of the collapse, the National Science Foundation awarded eight grants to engineering and social science researchers to assess the debris as it is being removed and to study the behavior of emergency response and management teams.

Analyzing the failure of the towers is a Herculean task, but it is important that engineers understand in detail what happened so that they incorporate the lessons learned into future design practices. It was the careful failure analysis of the bombed Federal Building in Oklahoma City that led engineers to delineate guidelines for designing more terrorist-resistant buildings. The Pentagon was actually undergoing retrofitting to make it better able to withstand an explosion when it was hit by a third hijacked plane on September 11. Part of the section of the building that was struck had in fact just been strengthened, and it suffered much less damage than the old section beside it, thus demonstrating the effectiveness of the work.

Understanding how the World Trade Center towers collapsed will enable engineers to build more attack-resistant skyscrapers. Even before a detailed failure analysis is completed, however, it is evident that one way to minimize the damage to tall structures is to prevent airplanes and their fuel from being able to penetrate deeply into the buildings in the first place. This is not an impossible task. When a B-25 bomber struck the Empire State

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Building in 1945, its body stuck out from the 78th and 79th floors like a long car in a short garage. The building suffered an 18-by-20-foot (5.5-by-6-meter) hole in its face, but there was no conflagration, and there certainly was no collapse. The greatest damage was done by the engines coming loose and flying like missiles through the building. The wreckage of the plane was removed, the local damage repaired and the building restored to its original state. Among the differences between the Empire State Building and World Trade Center incidents was that in the former case, relatively speaking, a lighter plane struck a heavier structure. Furthermore, the propeller-driven bomber was on a short-range flight from Bedford, Massachusetts to Newark airport and

The towers might have stood after the attack if the fires had been extinguished quickly

so did not have on board the amount of fuel necessary to complete a transcontinental flight or to bring down a skyscraper.

Modern tall buildings can be strengthened to be more resistant to full penetration by even the heaviest of aircraft. This can be done by placing more and heavier columns around the periphery of the structure, making the tube denser and thicker, as it were. The ultimate defense would be to make the facade a solid wall of steel or concrete, or both. This would eliminate windows entirely, of course, which would defeat some of the purpose of a skyscraper, which is in part to provide a dramatic view from a prestigious office or board room. The elimination of that attraction, in conjunction with the increased mass of the structure itself, would provide space that would command a significantly lower rent and yet cost a great deal more to build. Indeed, no one would likely even consider building or renting space in such a building. Hence, the solution would be a Pyrrhic victory over terrorists.

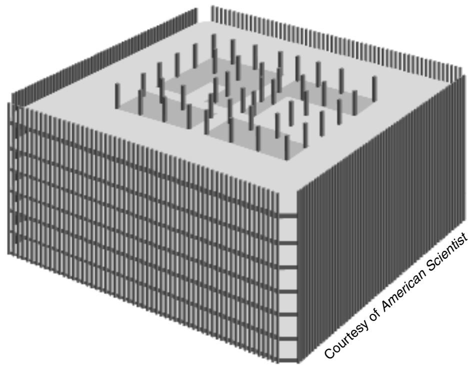

Figure 2. Structural design of the World Trade Center towers was a tube with a central core.

The World Trade Center towers might have stood after the terrorist attack if the fires had been extinguished quickly. But even if the conventional sprinkler systems had not been damaged, water would not have been effective against the burning jet fuel. Perhaps skyscrapers could be fitted with a robust fire-fighting system employing the kind of foam that is laid down on airport runways during emergency landings, or fitted with some other oxygen-depriving scheme, if there could also be a way for fleeing people to breathe in such an environment. Such schemes would need robustness and redundancy to survive tremendous impact forces, so any such system might be unattractively bulky and prohibitively expensive to install. Other approaches might include more effective fireproofing, such as employing ceramic-based materials, thus at least giving the occupants of a burning building more time to evacuate.

The evacuation of tall buildings will no doubt now be given much more attention by architects and engineers alike. Each World Trade Center tower had multiple stairways, but all were in the single central core of the building. In contrast, stairways in Germany, for example, are required to be in different corners of the building. In that configuration, it is much more likely that one stairway will remain open even if a plane crashes into another corner. But locating stairwells in the corners of a building means, of course, that prime office space cannot be located there. In other words, most measures to make buildings safer also make them more expensive to build and diminish the appeal of their office space. This dilemma is at the heart of the reason why the future of the skyscraper is threatened.

It is likely that, in the wake of the World Trade Center collapses, any super-tall building currently in the development stage will be put on hold and reconsidered. Real-estate investors will want to know how the proposed building will stand up to the crash of a fully fueled jumbo jet, how hot the ensuing fire will burn, how long it will take to be extinguished and how long the building will stand so that the occupants can evacuate. The investors will also want to know who will rent the space if it is built.

Potential tenants will have the same questions about terrorist attacks. Companies will also wonder if their employees will be willing to work on the upper stories of a tall building. Managers will wonder if those employees who do agree to work in the building will be constantly distracted, watching out the window for approaching airplanes. Corporations will wonder if clients will be reluctant to come to a place of business perceived to be vulnerable to attack. The very need to have workers grouped together on adjacent floors in tall buildings is also being called into question.

After the events of September 11, the incentive to build a signature structure, a distinctive super-tall building that sticks out in the skyline, is greatly diminished. In the immediate future, as leases come up for renewal in existing skyscrapers, real-estate investors will be watching closely for trends. It is unlikely that our most familiar skylines will be greatly changed in the foreseeable future. Indeed, if companies begin to move their operations wholesale out of the most distinctive and iconic of super-tall buildings and into more non-descript structures of moderate height, it is not unimaginable that cities like New York and Chicago will in time see the reversal of a long-standing trend. We might expect no longer to see developers buying up land, demolishing the low-rise buildings on it, and putting up a new skyscraper. Instead, owners might be more likely to demolish a vacant skyscraper and erect in its place a building that is not significantly smaller or taller than its neighbors. Skylines that were once immediately recognizable even in silhouette for their peaks and valleys may someday be as flat as a mesa.

There is no imperative to such an interplay between technology and society. What really happens in the coming years will depend largely on how businesses, governments and individuals react to terrorism and the threat of terrorism. Unfortunately, the image of the World Trade Center towers collapsing will remain in our collective consciousness for a few

Skylines once recognizable in silhouette may someday be as flat as a mesa

generations, at least. Thus, it is no idle speculation to think that it will be at least a generation before skyscrapers return to ascendancy, if they ever do. Developments in micro-miniaturization, telecommunications, information technology, business practice, management science, economics, psychology and politics will likely play a much larger role than architecture and engineering in determining the immediate future of macro-structures, at least in the West.

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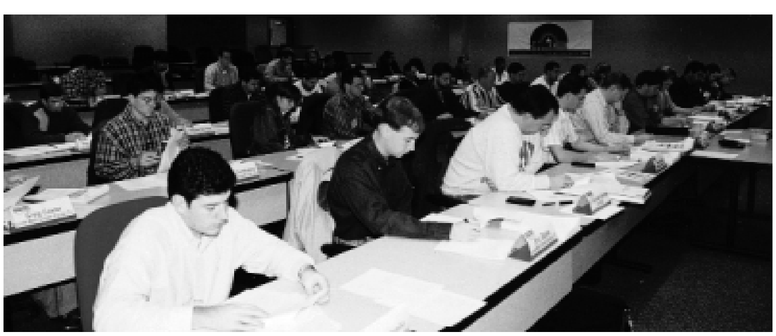

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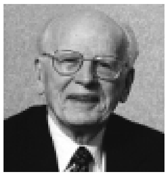

Lessons Learned in the Field

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

Consider the Transfer of Stress through Members

Introduction

Here I am, back again. In the second issue of 2001 (Vol. XVIII, No. 2), the editors of *Welding Innovation* were delighted to publish an excellent piece in this space: “Persistence Pays Off” by Rob Lawrence of Butler Manufacturing. Now, where are the submissions from the rest of you out there?

I started this column a couple of years ago with the idea of providing a forum in which our readers could share the important principles gleaned from the everyday challenges of working in the field. It seems to me that often the “evident” solution to a problem turns out to be a dead end. I call these my “ah-ha!” moments. Surely they’ve happened to many of you. Think about what you actually learned from these experiences, that you were able to apply again in other situations. Then send an email describing your column idea to Assistant Editor Jeff Nadzam at Jeffrey_Nadzam@lincolnelectric.com. Don’t worry about preparing a finished, illustrated article. Our writers, editors and artists can help with that. We’re just looking for a description of the real-life circumstances, and a statement of what you learned.

All right, then, here are some more lessons I learned, not in school, but working in the field.

Provide a Path for Transfer of Stress

A common design oversight is the failure to provide a path so that a transverse force can enter that part of the member (section) that lies parallel to the force.

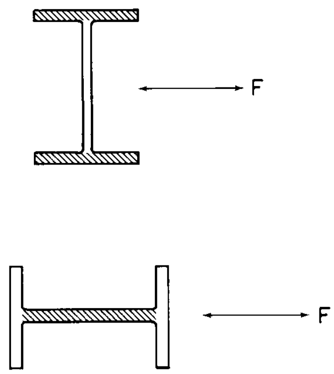

Figure 1.

Given what is needed for the proper transfer of force (as shown in Figure 1), let’s consider some examples.

The top of Figure 2 shows a lug that has been welded to a flanged beam in the simplest and most efficient manner—so the force goes into the web, the part parallel to it. In the center sketch of Figure 2, the lug is placed across the bottom flange, necessitating the use of either rectangular or triangular stiffeners to transfer the load to the web. If, for some reason, the circumstances require the lug to be placed in this manner, the stiffeners (with the attendant increase in welding and material usage they entail) are mandatory. Merely welding the lug across the more flexible flange could result in an uneven load on the weld. Note that the stiffeners are not welded to the top flange. There would be no reason to weld them there, since the flange will not take the force. At the bottom of Figure 2, the member is in a different position, and the lug is correctly welded to the flanges that will take the load. It is not welded to the web, since that would serve little purpose in transferring the force.

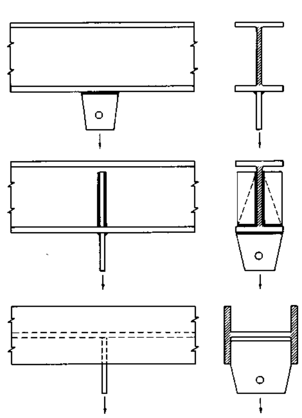

Figure 2.

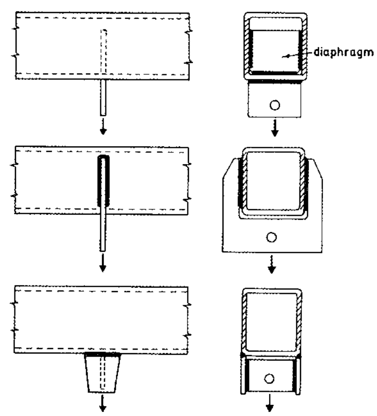

Figure 3.

Figure 3 illustrates how a lug might be welded to a box section so as to transfer force to the parts parallel to it. The sketch at the top, of course, is not applicable to the rolled section shown, since there would be no way of getting the diaphragm inside the box. But if it were a fabricated box section, the diaphragm could be welded in before welding the top plate on. The center

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and bottom drawings in Figure 3 show additional ways to attach a lug to a box section. In the center, the lug is shaped as a sling and directly welded to the flange. At the bottom, the lug is designed so it will transfer the force into the two webs. This is a very efficient way to transfer the force on the lug into the webs.

When the Member Is Circular

Figure 4 illustrates two methods of applying a transverse force to a circular member. The rationale for these methods of attachment is shown in

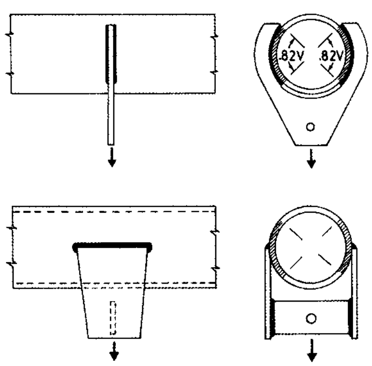

Figure 4.

Figure 5. At the top of Figure 5, the beam is welded to a support. In standard practice, it is assumed that the flanges transfer the bending moments and the web transfers vertical shear. In the case of the circular member at the bottom of Figure 5, however, it is difficult to decide which part of the member is flange, and which part is web. Mathematical analysis has shown that if a tube is divided into four quadrants, the top and bottom quadrants will transfer 82% of the bending moment, and the side quadrants, 82% of the vertical shear. The methods of attaching the lug shown in Figure 4, therefore, are methods that transfer force tending to cause vertical shear into the areas of the circular section most closely parallel to the force.

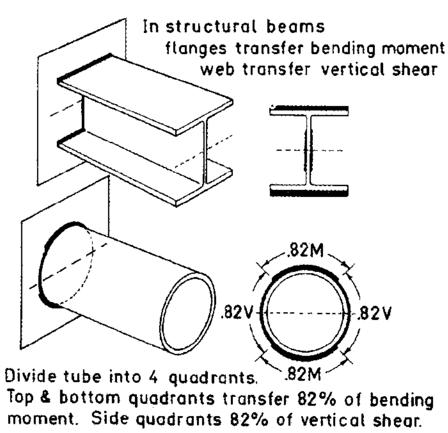

Figure 5.

More Complicated Examples

Figure 6 provides a more complicated example of force transfer. A tank to haul water on a truck is made up of 1/4 in. (6.4 mm) thick plate, with the sides overlapping the ends so as to provide fillet welds. Considering the forces from the water pressure on the tank ends, the only place for them to

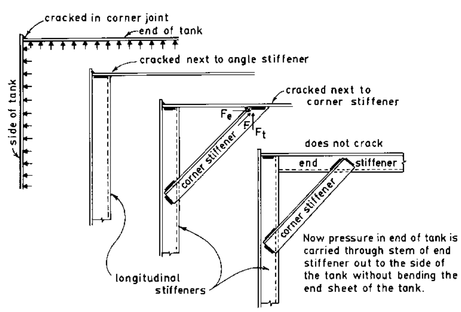

Figure 6.

go is through the welds and into the sides—the parts parallel to their direction. The forces get there by bending the end plate. In service, the welds cracked. Three remedies were tried successively, as shown in Figure 6, using longitudinal and corner stiffeners, and finally both longitudinal and end stiffeners with corner stiffeners.

Figure 7 shows the center sill of a piggyback railroad car to which a bracket is welded to carry a 500 lb. (227 kg) air compressor unit. There are no interior diaphragms. The vertical

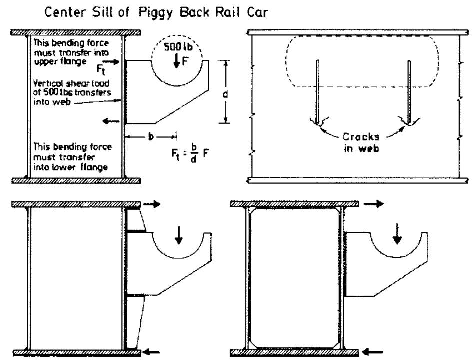

Figure 7.

force from the weight of the unit is transferred as moment into the bracket, creating bending at the web. The two horizontal bending forces must eventually transfer to the parallel flanges, but with an open box section there are no ready pathways. As a result, the web flexes and fatigue cracks appear in the web. The sketches at the bottom of Figure 7 illustrate two possible means for correcting the faulty design. In one, a stiffener is added before the web opposite the bracket side is welded into the assembly. The stiffener is welded to both flanges and to one web. There are now paths for the bending forces to get to the flanges. The second way to correct the design is to shape the bracket so it can be welded directly to the sill flanges in new fabrications, or to add pieces to the bracket on existing cars to accomplish the same purpose.

Conclusion

The foregoing are just a few examples intended to illustrate the importance of considering the transfer of force through members. Sometimes we engineers act a little like horses with blinders on: we concentrate so single-mindedly on the problem at hand, that we can't see what is going on around us. The ideas discussed in this column should demonstrate how critical it is for us as engineers to take our blinders off, expand our limited views, and test our assumptions.

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Gleaming "Waterfall" Refreshes Urban Campus

By Carla Rautenberg
Welding Innovation Contributing Writer
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Cleveland, Ohio

Controversy and intense interest have surrounded The Peter B. Lewis Building of the Weatherhead School of Management at Case Western Reserve University ever since architect Frank Gehry unveiled his design. But many Cleveland area residents who shuddered when they first saw photographs of the model in the city's paper, *The Plain Dealer*, have been won over as they watched the shining sculptural curves of the roof take solid shape and form.

According to *Plain Dealer* architecture critic Steven Litt, the building depicts "Gehry's vision of a gleaming waterfall splashing over boulders in a mountain stream." Sure enough, the stainless steel skin that slinks over and around the sensuous curves of the steel structure glistens in the sun like so much rushing water. The sight, etched against a blue sky, can be breathtaking.

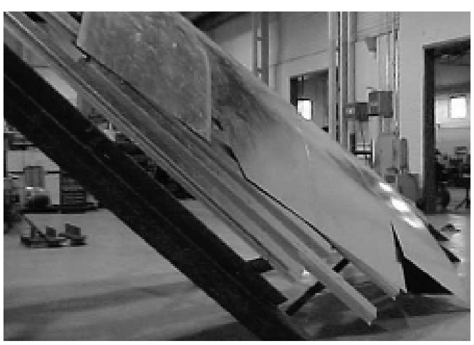

Figure 1.

What no longer shows is the meticulous planning, shop fabricating and field welding work that went into creating the structural steel supports for that elegant silvery "gown" of shingles the $61.7 million building now wears. When the project's lead contractor, Hunt Construction Group of Indianapolis, called for bids to fabricate and erect the structural steel, the response was apparently less than overwhelming. However, Mariani Metal Fabricators, Ltd., based across Lake Erie from Cleveland in Toronto, Ontario, answered the call.

Figure 2.

Software Jumps Industries

Greg Kern, vice president of the 16-year-old firm, readily admits that the project was a challenge, not only to build, but to price. "This was one of the first uses of CATIA software in the steel construction industry," he points out. CATIA, which was developed for automotive design and drafting applications, is employed by architect Frank Gehry. "Therefore, the geometry was there for us, because [Gehry] had worked out the models," says Kern. After winning the $6 million structural steel contract, Mariani Metal hired a drafting and software training company with automotive industry expertise to produce project models and about 1,200 drawings using CATIA. Parallel CATIA software stations were established in the Mariani fabrication shop and at the construction site to provide design and fabrication adjustments in real-time, a step which prevented many potential disruptions in production.

Devising a Practical Approach

When considered in the light of traditional steel construction concepts, creating the three-dimensional negative and positive curves that comprise the roof structure posed practical problems both structurally and in terms of cost. After analyzing the complex geometry from a real-world erection standpoint, Mariani Metal proposed fabricating the structural framework in a series of ladders, infills, truss panels and support members, which would be shop-fabricated (Figure 1) and then assembled and field-welded on site. "It was basically a modular approach," notes Kern.

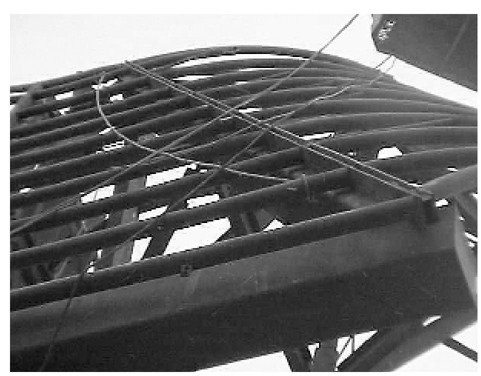

Figure 3.

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Another challenge was devising a method of bending the hundreds of pipes which form the "lines of ruling" (Figures 2 and 3). Almost every pipe utilized in the roof structure had a unique curvature and length. With the help of the CATIA software, Mariani Metal developed a system which permitted unique members to be created within a production line. This allowed stockpiling of pipe sections which could then efficiently feed the shop fabrication process.

Standard AWS D1.1 connection details were employed to weld the 700 tons of Grade 50 pipe and structural steel used to create the framework for the roof. Mariani Metals, which has a full-time workforce of thirty, employed eight welders on the job in the shop; field welding was done by a crew that averaged between eight and sixteen welders. The process most used was shielded metal arc, with semi-automatic flux cored arc welding in selected applications. "We kept the field welding operation as straight-forward as possible, doing the most complex welding in the shop," says Kern. He proudly states that "the skin lies directly on our pipes, with the pipes themselves creating the geometry of the surface," and adds that the whole process required the precision of "building a Swiss watch in full scale." That kind of precision is apparently something the folks at Mariani thrive on; Kern maintains that they would hasten to work on additional Gehry projects.

Across the street from the construction site in Cleveland's University Circle, the stone caryatids of Case Western Reserve's gothic style Mather Memorial Building have silently watched a monument to 21st century architecture take dramatic form. When the 149,000 ft.^{2} (13,843 m^2) Peter B. Lewis Building is dedicated later this year, the city of Cleveland will have a new landmark.

This soaring structural steel framework is now hidden by the outer skin and inner walls of the building.

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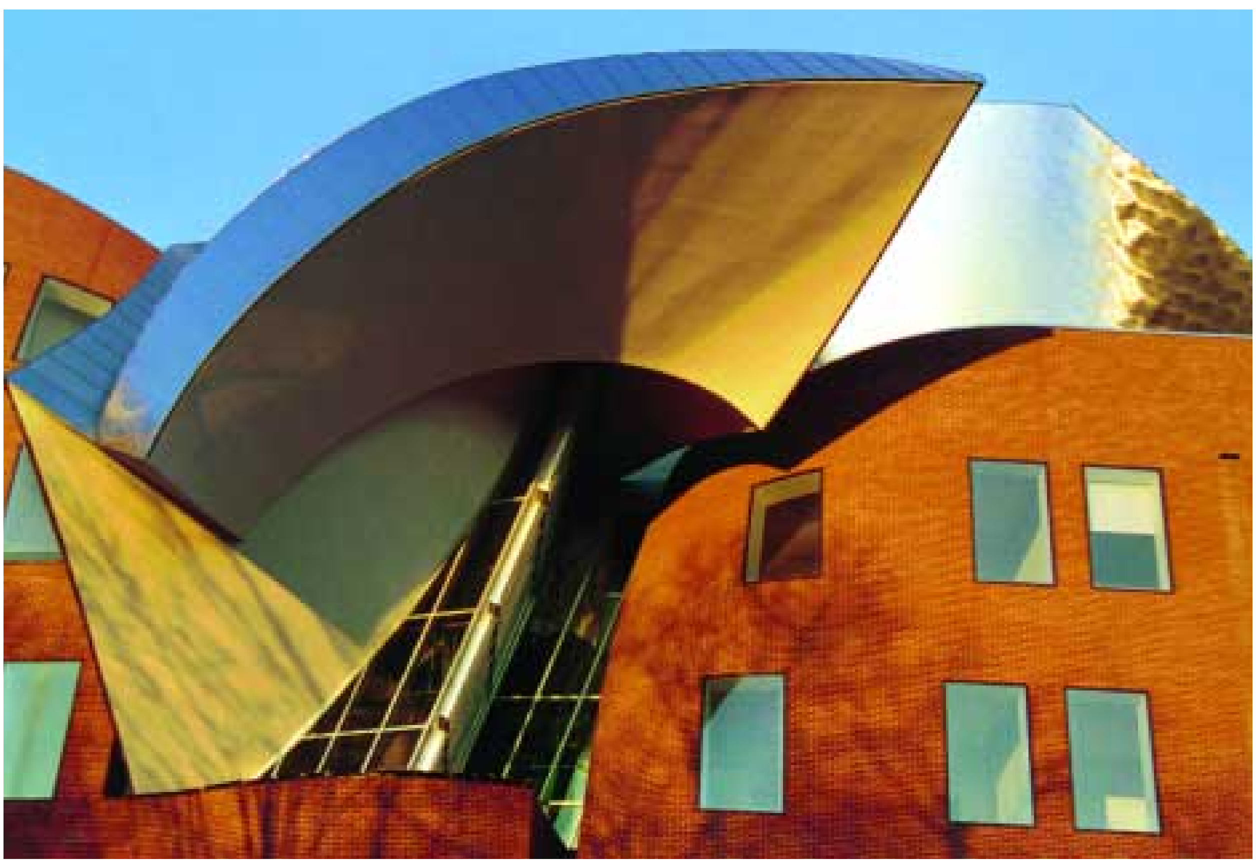

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