An example of how these two frames looked (in this instance for the 4500-lb vehicle that impacted at 24°) and how the longitudinal distributions of contact pressure were deduced from them is indicated by Figure 6 (top and bottom). The depth dimension was deduced by subtracting the 3-in sill height (see Figure 5) from the height of the resultant to find D/2 or, when the resultant lay above (39/2) + 3 = 22.5 in (the mid height of the wall panels), by subtracting the resultant height from 42 in to find D/2. After integration and inversion to solve for the maximum intensity in terms of the measured resultant, one finds that q_{max} = R (π^2/4DL). The double-sine distribution for the initial impact of the 4740-lb vehicle at 59.9 mph and 24° is shown in Figure 5.

A summary of results from all tests is given in Table 1. Data for both the initial and the final phases of the impact are given for each test. The column headed Height gives the distance from the pavement surface to the resultant impact force,

whereas the column headed Contact Height gives the vertical dimension over which the force was distributed. Similarly, the contact length is the distance along the railing over which the force was distributed. These values were used to derive the data in the last two columns, which contain peak values (for the half-sine-wave distribution) of force per unit area and per unit length.

It should be noted that the force measurements were obtained from a nondeflecting barrier and represent the upper bound of forces that would be expected on service railings. Tests conducted on service railings that had typical deflection capabilities result in forces significantly lower than those shown in Table 1.

REFERENCE

1. Standard Specifications for Highway Bridges, 11th ed. AASHTO, Washington, DC, 1973.

Strength of Fillet Welds in Aluminum Lighting Poles

JAMES S. NOEL, C.E. BUTH, AND T.J. HIRSCH

Tests were performed to ascertain the inherent strength of aluminum fillet welds such as those used to make lighting support poles for highways. It was found that two sources of excess strength beyond that recognized by current design specifications were often available. One was that the strength of a fillet weld when loaded so that the resultant forces are perpendicular to its length is 35-45 percent greater than when it is loaded parallel to its length. The other, applicable only to members that are hollow and round or near-round (as are virtually all the aluminum highway lighting support poles), was that the shape factor for such cross sections was 1.31 rather than 1.12, the shape factor often used for most metal structural shapes. Examples of a near-round member include many-sided polygons and ellipses in which the major and minor axes are nearly the same length. Because the shape factor represents excess strength beyond first yield, this finding represents a [(1.31/1.12) - 1] 100 percent = 17 percent increase in load-carrying capacity. A method is suggested for amending the applicable specifications to reflect these greater strengths.

Weld sizes used by manufacturers of spun-aluminum lighting poles were established primarily on the basis of tests conducted by the individual companies. Although experience has shown that these weld sizes are satisfactory, both state and federal highway engineers have questioned whether they can be justified by using only the requirements of the American Association of State Highway and Transportation Officials (AASHTO) (1).

The AASHTO specifications refer to the Aluminum Association's Specifications for Aluminum Bridge and Other Highway Structures (2), which calls for an allowable shear stress of 30 MPa in fillet welds of filler alloy 4043 with parent alloy 6063. This allowable stress was established on the basis of longitudinal shear stress tests of fillet welds and a bridge safety factor of 2.64.

The geometry at the base of most aluminum highway support poles is similar to that shown in Figure 1. The relatively thin-walled circular pole is connected into a cast-aluminum base flange by a circumferential fillet weld or welds as shown. Bending of the pole by the forces of nature causes the fillet welds to be stressed perpendicular to their lengthwise (circumferential) direction. Part of the purpose of this study was to determine whether an

allowable stress greater than 30 MPa should be used because of the difference in strength between transverse and longitudinal loading of fillet welds.

The effect that the circular shape of the weld has on the bending strength of the joint was also included among the objectives. The published allowable stresses for bending of round and elliptical tubes take into account the greater strength of these shapes compared with that of other shapes. But these same effects are not recognized by the allowable stresses prescribed for circumferential welds.

The Tapered Aluminum Pole Group of the National Electrical Manufacturers Association elected to support a program designed to quantify the significance of these effects and, if possible, to suggest how the results could be incorporated into the existing specifications for such structures.

STRENGTH OF TRANSVERSE VERSUS LONGITUDINAL FILLET WELDS

Comparison of the results of the tests of weld splices in flat-bar specimens confirmed what has long been known by structural engineers, namely, that the load-carrying capacity of a fillet weld transverse to the direction of a tensile traction is considerably greater than that of a fillet weld parallel to the traction. Spraragen and Claussen (3) report that tests of fillet welds in the 1920s and 1930s had already determined that transverse fillets would carry up to 40 percent more load than would parallel fillets. Usually this difference in strength is ignored by design specifications, which are predicated on the weakest possible configuration for the weld and load. Associated commentaries and textbooks usually explain that the excess strength is disregarded, primarily to simplify calculations (4).

A simple strength-of-materials approach to demonstrating the excess strength, as opposed to a theory-of-elasticity approach, is quite convincing.

Figure 1. Typical pole base section.
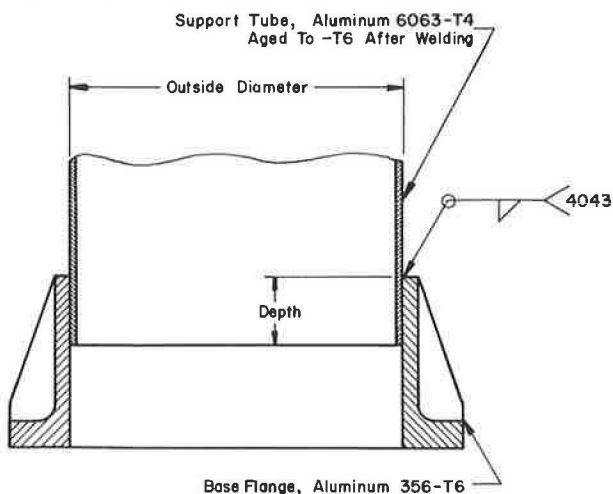
Consider a free body of a transverse weld on a double-lap shear specimen (Figure 2). For equilibrium in the horizontal direction, we have a tensile force P of the same magnitude on the horizontal face. In order to maintain complete equilibrium of the weld, a shear force P on the vertical face and a tensile force P on the horizontal face must be included. This results in a homogeneous state of plane stress everywhere in the weld such as that shown on a small cubical element in Figure 3a or, with Mohr's circle, that in Figure 3b. From the latter, one promptly sees that the maximum shear stress is τ_{max} = P/sL or τ_{max} = 0.707 (P/A_{throat}), where s is the weld leg size and L is the weld length. In other words, the maximum shear in welds loaded transversely is only 71 percent as great as the maximum shear stress in a longitudinal fillet weld subjected to the same force. So it follows that the transverse shear strength should be 1.41 times as great as the longitudinal shear strength.

Much more complete explanations of this phenomenon can be found; one of the more elaborate is that offered by Kato and others (5,6). Their analysis used the von Mises criterion for yield, assumed the direct stress on the tensile face of the transverse weld to be uniformly distributed, and neglected the geometrical changes that occur during loading. Their conclusions were that a unit length of transverse weld could carry 1.46 times the load that a symmetrically loaded unit length of longitudinal weld could. These conclusions were subsequently confirmed by using a dense mesh finite-element computer program (1). One advantage of finite-element approaches for such calculations is that elastic solutions can be compared with elastoplastic solutions, a comparison that, in this instance, revealed little difference in the relative capacities of transverse and parallel fillet welds irrespective of the assumed material behavior.

The analyses have been very carefully corroborated by tests, especially for steel (3,7,8). However, there appears to be a paucity of comparable data for aluminum. This explains the motivation for performing the tests reported here.

Data summaries of flat-bar tensile test samples that have transverse and longitudinal splice welds are given in Table 1. The statistical summary is given below:

Statistic (MPa)	Type of Weld	
	Transverse	Longitudinal
Avg, all specimens	177	132.0
SD	14	11.1
Lowest value	150	112.0
Mean minus 3 SD	135	98.6

The double-lapped, butt-joint test specimens were fabricated by using 4043 weld wire on 6063-T4 aluminum plate that was then precipitation heat treated (artificially aged) to the T6 temper after welding. All welds were terminated by using saw cuts to assure as little variation as possible in the effective lengths. The entries in the column headed Nominal Weld Stress at Failure were calculated by dividing the failure load by the weld length and the throat depth. The table indicates that a few of the welds had unequal leg lengths (sizes). The throat depth for a fillet weld that has unequal leg lengths is defined in Figure 4.

A comparison of the magnitudes of the average weld failure stresses in the two test configurations clearly and expectedly showed that the transverse weld was 177 MPa/132 MPa = 1.34 times as strong as the longitudinal weld. This result led directly to the decision to recommend that Table 7.1.3.2 [Allowable Shear Stresses in Fillet Welds for Bridge Type Structures in the Alumination Association's Specifications for Aluminum Bridge and Other Highway Structures (2)] be amended by adding a tabulation of allowable stresses for fillet welds loaded transversely in which the allowables are increased by a factor of 1.36. Thus, the allowable stress of 30 MPa (4.4 ksi) specified for 4043 fillets on 6063-T6 aluminum would be increased to 41 MPa (6.0 ksi). The justification is equally as applicable for 4043 welds on other aluminum alloys (see Figure 5).

BENDING LOADS VERSUS AXIAL LOADS

Tubes Welded to Flat Base Plate and Loaded in Axial Tension

The 12 specimens tested in this series were tubes 12.7 cm in diameter and approximately 2.1 m long welded to a flat base plate. An increasing axial tensile load was applied until failure occurred. The loading system consisted of a frame that had a hydraulic ram connected to the specimens by using a collect system. The failure mode was simultaneous failure around the circumferential weld and approximately on the plane defined by the throat of the weld based on the inscribed right triangle indicated in Figure 4.

The stresses in the welds at failure were calculated by using the following formula, which results in the conventional computation of stress on the throat of the weld:

$$f_t = P/\pi\psi(d + \psi) \tag{1}$$

where

   f_t = tensile transverse shear stress (MPa),
   P = applied load (N),
   ψ = throat depth (mm), and
   d = outside diameter of tube (mm).

Failure stresses calculated by using Equation 1 are presented in Table 2.

Tubes Welded to Flat Base Plate and Loaded in Bending

Each of the 12 specimens for this type of test consisted of a tube 12.7 cm in diameter and approximately 3 m long that had a 4.77-mm wall and was attached to a flat aluminum base plate by a single circumferential fillet weld. The specimens were loaded as cantilever beams. An increasing load was applied a measured distance from the weld until failure occurred. The loading rate was such that the time to failure was approximately 3 min. Failure mode was rupture of the weld in the region of highest tensile stress.

Stresses in the welds at failure were computed by using the following formula, which is based on

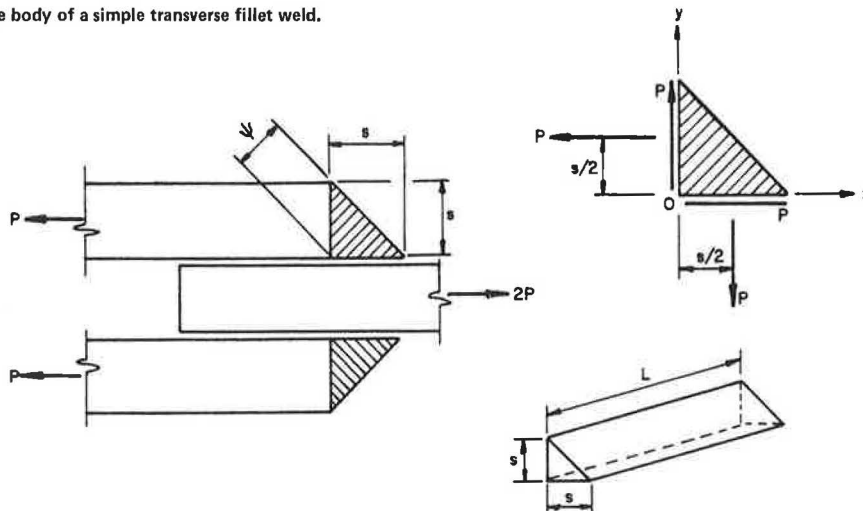Figure 2. Free body of a simple transverse fillet weld.

Figure 3. Simplified stress state hypothesized for fillet weld loaded as shown in Figure 2.
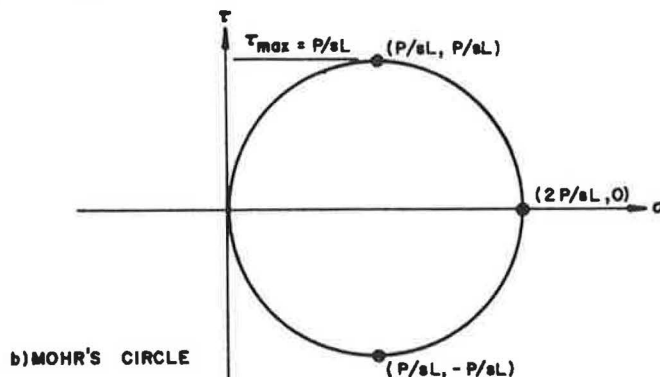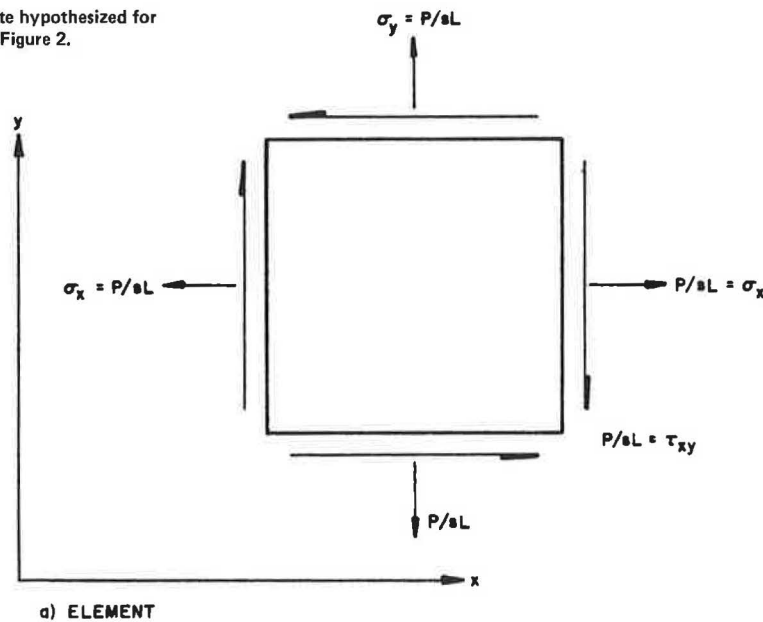

a) ELEMENT

b) MOHR'S CIRCLE

Table 1. Flat-bar specimens with transverse and longitudinal welds.

Average Weld Size (cm)	Weld Length (cm)	Failure Load (kN)	Nominal Weld Stress at Failure (MPa)
Specimen with Transverse Weld			
0.635 x 0.635	5.08	38.8	170
0.635 x 0.635	5.08	38.3	168
0.635 x 0.635	5.08	37.8	165
0.635 x 0.635	5.08	34.1	150
0.635 x 0.635	5.08	38.3	168
0.635 x 0.635	5.08	45.5	199
0.635 x 0.635	5.08	40.6	178
0.635 x 0.635	5.08	46.2	203
0.635 x 0.635	5.08	44.0	193
0.635 x 0.635	5.08	44.9	174
0.635 x 0.635	5.08	41.4	181
0.635 x 0.635	5.08	39.6	174
0.635 x 0.635	5.08	41.2	181
0.635 x 0.635	5.08	40.5	177

Specimen with Longitudinal Weld			
0.762 x 0.635	14.5	78.7	112
0.635 x 0.635	14.7	85.0	128
0.635 x 0.762	14.6	102.0	143
0.635 x 0.711	14.4	96.1	141
0.762 x 0.635	14.5	95.4	135
0.762 x 0.635	14.2	97.7	141
0.635 x 0.635	14.7	89.6	136
0.635 x 0.635	15.0	80.9	120

Figure 4. Method used to compute throat depth based on leg lengths of fillet weld.

THROAT DEPTH $= s \cos \tan^{-1} \frac{s}{h} = \psi$

elastic theory and results in the stress on the throat of the weld:
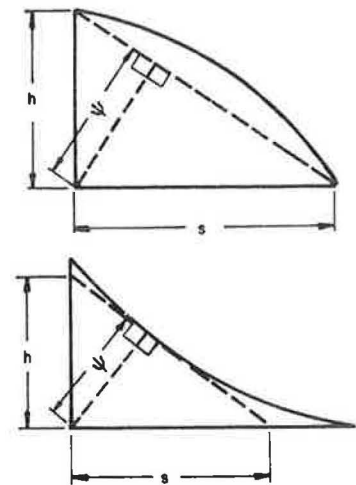
$$\sigma = M_b c/I \qquad (2)$$

where

M_b = applied bending moment (N·mm),
$c = (d + 2\psi)/2$,
d = inside diameter of weld (mm), and
$I = (\pi/64) [(d + 2\psi)^4 - d^4]$.

A comparison of the maximum stresses measured in the bending tests (summarized in Table 3) with those measured in the tension tests indicates a ratio of 164 MPa ÷ 101 MPa = 1.62. This increase is credited to the excess strength over and beyond the moment attained when the extreme fiber first yields.

Tubes Welded to a Sleeve and Loaded in Axial Tension

In this group of nine tests, the specimens consisted of two tubes 20.3 cm in diameter and approximately 1.4 m long with 4.77-mm walls. The tubes were connected at the ends by a sleeve and two circumferential welds so that tension applied to the system would cause a transverse load on the welds. Each tube was inserted 1.27 cm into the sleeve. The specimens were connected by means of a collect system to an expanding frame that applied an increasing tensile load until failure occurred. Failure occurred simultaneously around the circumferential weld. The welds consistently failed on a

plane that approximated the throat of the weld based on the inscribed right triangle.

The stresses in the welds at failure were calculated by using the same method as that for the welds between tube and base plate. Table 2 presents these values also. The statistical summary from Table 2 is shown below:

Statistic (MPa)	Type of Weld	
	To Flat Plate	To Sleeve
Avg, all specimens	102.0	113.0
SD	22.0	10.2
Lowest value	70.0	99.0
Mean minus 3 SD	35.5	82.5

Tubes Welded to a Sleeve and Loaded in Bending

The specimens for this test were two tubes 20.3 cm in diameter and approximately 1.4 m long with 4.77-mm walls. They were connected at the ends by a sleeve and two circumferential welds. The specimens were supported simply and had a span length of 2.6 m; they were loaded to failure by using two equal concentrated loads spaced 23 cm on either side of the midspan. This loading condition produced bending moment in the absence of beam shear on the weld joints. The applied load was incremented until failure occurred. The loading rate was such that the time to failure was 5 min. The welds failed consistently on the throat plane based on the inscribed right triangle. The failure mode was rupture of the weld in the region of highest tensile stress.

Weld failure stresses for these specimens were computed by using the elastic bending equation (used also for the tubes welded to a flat plate) and are presented in Table 3.

Again, the ratio of the average maximum bending stress to the average maximum axial stress is 137 MPa/113 MPa = 1.21. Although it is not 1.31, it should be noted that the maximum axial stress of 113 MPa is well above the ultimate stress expected to be nominal for this weld in simple tension. An inflated denominator would cause the ratio to appear too small.

Figure 5. Table 7.1.3.2 from Specifications for Aluminum Bridge and Other Highway Structures (2) showing changes suggested by test results.

TABLE 7.1.3.2
ALLOWABLE SHEAR STRESSES IN FILLET WELDS FOR BRIDGE TYPE STRUCTURES* –ksi

Filler Alloy ‡		1100	4043	4043 Transverse	5356 5554	5556
Parent Alloy						
Alclad	3003	2.8	4.4	6.0	–	–
	3004	–	4.4	6.0	6.5	7†
	5052	–	4.4	6.0	6.5	–
	5083	–	–	–	–	7.5
	5154	–	–	–	6.5	7.5
	5456	–	–	–	–	7.5
	6005, 6061, 6351	–	4.4	6.0	6.5	7.5
	6063	–	4.4	6.0	6†	6†

† Values controlled by the shear strength of the parent metal.
‡ Minimum expected shear strengths of filler alloys are:

Alloy 1100	7.5 ksi
4043	11.5
5356	17
5554	17
5556	20

* When the welds join circular and near circular cross-sections which are to be loaded in bending these allowables may be multiplied by 1.17.

Note: 1 ksi = 6.89 MPa.

Table 2. Tubes welded to flat base plate and to a sleeve and loaded in axial tension.

Average Weld Size (cm)	Failure Load (kN)	Nominal Weld Stress at Failure (MPa)
Tube Welded to Flat Base Plate		
0.709 x 0.640	2.35	119
0.663 x 0.643	2.56	134
0.691 x 0.681	2.52	125
0.782 x 0.686	2.44	114
0.785 x 0.749	2.22	99
0.688 x 0.663	2.37	120
0.744 x 0.627	1.55	78
0.818 x 0.782	2.06	88
0.813 x 0.777	2.65	113
0.927 x 0.792	1.90	75
0.798 x 0.757	1.61	70
0.945 x 0.879	2.24	83

0.478 cm Wall

2.13 m

12.7 cm

→ P
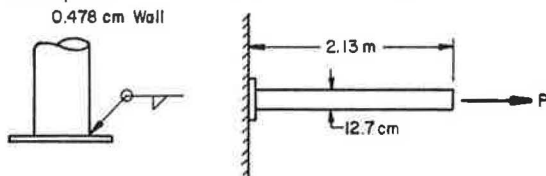
Tube Welded to Sleeve		
0.874 x 0.739	38.8	105
0.823 x 0.749	38.4	105
0.729 x 0.632	38.5	123
0.805 x 0.754	38.4	106
0.871 x 0.556	30.4	99
0.632 x 0.589	36.6	130
0.693 x 0.533	32.9	119
0.780 x 0.765	40.1	112
0.818 x 0.653	39.8	119

0.478 cm Wall

2.74 m

20.3 cm

P →   ← P
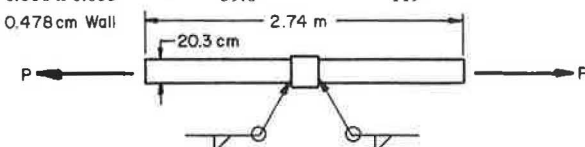
The calculated shape factor for typical lighting-pole dimensions is about 1.31. This can vary from 1.27 and more for very thin-walled circular shapes to 1.70 for solid circular shapes.

Shape factors for wide-flange shapes vary from about 1.10 to 1.18; the most frequent value is about 1.12 (8).

If one then compares the plastic moment of circular sections with the plastic moment of wide-flange sections, that for the circular section would be expected to be about 1.31/1.12 = 1.17 times greater than that for the typical wide flange.

TYPICAL LIGHTING-POLE BASES

Nine specimens configured as similar as possible to actual luminaire supports were then tested to assure the practicality of liberalizing the design allowables as suggested by theoretical considerations and laboratory tests. The cylindrical tube structures were 7.6 m long, had an outside diameter of 25.4 cm, and were 6.5 mm thick. This tube was inserted 1.27 cm into a cast-aluminum (356-T6) socket base and connected by means of a fillet weld all around the top of the base. These cantilevered beams were then subjected to a transverse end load. The transverse loads were increased at a rate that caused the average time to failure to be about 4 min. In all cases, failure was a result of a rupture of the weld in the region of maximum tensile stress. Failure occurred near the plane formed by the throat of the weld based on the inscribed right triangle.

The transverse shear stresses in the welds at failure were computed by using the following equation:

$$f_b = [M_B(d + 2\psi)/2] / [(\pi/8)(d^3\psi + 3d^2\psi^2 + 4d\psi^3 + \psi^4)] \qquad (3)$$

where f_b is the transverse shear stress on the throat of the weld in megapascals and d is the outside diameter of the tube in millimeters.

It should be noted that stresses computed in this

**Table 3. Tubes welded to flat base plate, to a sleeve, and into a support socket and loaded in bending.**

Average Weld Size (cm)	Failure Load (kN)	Failure Moment (kN·m)	Elastic Failure Stress (MPa)
Tubes Welded to Flat Base Plate			
0.663 x 0.599	4.18	11.8	201
0.663 x 0.650	4.00	11.5	189
0.841 x 0.826	4.46	12.5	160
0.815 x 0.805	4.46	12.5	165
0.704 x 0.693	4.09	11.5	177
0.693 x 0.650	3.86	10.8	174
0.704 x 0.683	4.09	11.5	179
0.785 x 0.767	4.09	11.5	159
0.879 x 0.874	4.57	12.9	156
0.838 x 0.798	3.77	10.6	139
0.973 x 0.798	4.27	12.0	145
0.886 x 0.810	3.61	10.2	128

0.478 cm Wall — 3.05 m — P — 12.7 cm
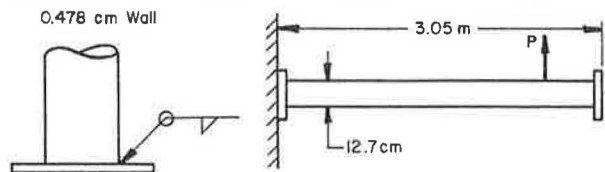
Tube Welded to Sleeve

0.843 x 0.792	43.6	23.3	121
0.846 x 0.681	49.4	26.3	150
0.693 x 0.686	44.3	23.6	146
0.734 x 0.729	46.9	25.0	146
0.810 x 0.620	48.3	25.8	170
0.925 x 0.894	46.6	24.9	116
0.752 x 0.645	39.0	20.8	128
0.719 x 0.559	32.4	17.3	119

0.478 cm Wall — 2.74 m — 20.3 cm — P P P P
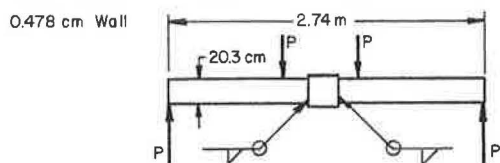
Tube Welded into Support Socket

0.787 x 0.597	6.11	44.6	182
0.762 x 0.561	6.70	48.7	209
0.978 x 0.770	8.35	60.6	193
0.813 x 0.693	7.32	53.2	195
0.747 x 0.660	7.07	51.3	201
0.935 x 0.724	7.25	52.7	178
0.930 x 0.914	7.21	52.3	154
0.869 x 0.726	7.09	51.5	179
0.919 x 0.757	7.78	56.5	187

0.556 cm Wall — 7.62 m — P — 25.4 cm
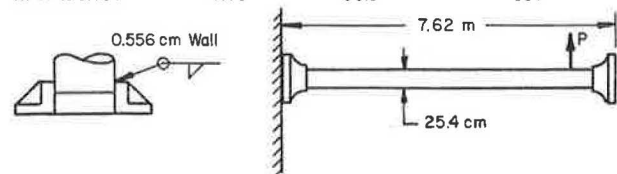
manner are based on elastic theory and a section modulus based on the weld throat, without regard to the direction of the applied load relative to the orientation of the weld. These stresses are given in Table 3. They vary from 152 to 207 MPa; the average is 186 MPa. Such luminaire bases thus demonstrate a factor of safety against their ultimate strength of 186 MPa/(1.36 x 1.31 x 30 MPa) $\sim$ 3.4, which is well above the prescribed 2.64 [we recall that the specifications (2) allowed a shear stress of 30 MPa].

The statistical summary from Table 3 is given below:

Statistic (MPa)	Type of Weld		
	To Flat Plate	To Sleeve	Into Support Socket
Avg, all specimens	164.0	137.0	186
SD	21.1	19.0	160
Lowest value	128.0	11.6	154
Mean minus 3 SD	101.0	80.1	139

CONCLUSIONS AND RECOMMENDATIONS

The data and theory presented here are in agreement with research reported in the literature and support the following conclusions:

1. The transverse shear strength of fillet welds is greater than the longitudinal shear strength. Commentaries and textbooks usually explain that the excess strength is disregarded, primarily to simplify design calculations. Tests reported here indicate that aluminum transverse welds are 34 percent stronger than longitudinal welds. A simple strength-of-materials calculation indicates that transverse fillet welds may have as much as 36 percent more strength than longitudinal fillet welds, whereas a more-sophisticated solution based on the theory of elasticity by Kato and Morita (5) suggests an even greater increase in strength, namely, 46 percent.

It is recommended that an allowable shear strength of 1.36 x 30 MPa = 41 MPa for aluminum fillet welds made of 4043 weld on 6063 parent metal aged to T6 temper after welding be permitted in typical lighting poles.

2. Round and oval tubular members and other beams of ductile materials exhibit bending strengths in excess of those predicted by elastic analysis procedures. This is due to the fact that, at ultimate loads, plasticity theory better describes the behavior of such members. Plasticity theory predicts that typical aluminum lighting-pole sections would exhibit strengths about 31 percent in excess of those predicted by elasticity theory. For wide-flange shapes and similar shapes, the excess strength is about 12 percent. This indicates that rounded and oval members are about 17 percent (1.31 $\div$ 1.12 = 1.17) stronger in bending than are wide-flange shapes and similar shapes. This additional strength is recognized by the aluminum specifications (2) for the members themselves but not for the welds.

It is recommended that this 17 percent increase in the allowable stress in these types of aluminum beams be extended to the weld metal in circumferential joints in such members.

3. The two factors recommended above are additive. In other words, in a situation in which a circular-shaped fillet weld is subjected to a transverse shear as a result of bending moment, the allowable stress based on the throat area of the weld would be 30 MPa x 1.36 x 1.17 = 48 MPa.

4. The AASHTO Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals (1) permit allowable stresses to be increased by 40 percent when stresses are produced by wind or seismic loading. Since the controlling design load for lighting poles is due to the wind, this 40 percent is especially significant and should be used. When this is done, the allowable stress would be 30 MPa x 1.36 x 1.17 x 1.40 = 68 MPa.

5. Table 7.1.3.2 of Allowable Shear Stresses in Fillet Welds for Bridge Type Structures in the Aluminum Association's Specifications for Aluminum

Bridge and Other Highway Structures (2) should be amended by adding a tabulation of allowable stresses for fillet welds loaded transversely in which the allowables are increased by a factor of 1.36. Thus, the allowable stress of 30 MPa specified for 4043 fillets on 6063-T6 parent metal shown would be increased to 41 MPa (footnoted to allow a further increase by a factor of 1.17 to 48 MPa if the fillet is joining round or near-round members subject to bending). The allowable of 41 MPa (6.0 ksi) is consistent with the factor of safety of 2.64. When it is intended that another factor of safety be used [for example, 2.34 in the Aluminum Association Specifications for Aluminum Structures (9)], this allowable could be modified accordingly if care is taken to assure that the shear strength of the parent metal is not exceeded.

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Crash Tests of Light-Post Thrie-Beam Traffic Barriers

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Thrie-beam corrugated steel rail (a W-beam that has a third corrugation) was tested as a single-rail upgrading for discontinuous bridge-rail panels and on S3×5.7 posts as a guiderail and double-faced median barrier. Tests were performed to determine rail deflection characteristics, structural adequacy, vehicle decelerations, and vehicle damage. Ten-gage Thrie beam was used for all tests. As a bridge-rail upgrading, the Thrie beam is suitable for 60-mph, 25° impacts by 4500-lb vehicles. As a guiderail or median barrier on S3X5.7 posts, it appears suitable as a longitudinal barrier, based on tests with 2250-lb and 3500-lb vehicles. Proposed design deflections for Thrie-beam guiderails and median barriers are close to those for box-beam guiderails and median barriers. Further testing of these guiderail and median-barrier designs would yield better definition of impact and redirection characteristics and would better indicate what actions could be taken to reduce the impact between the vehicle's wheel and the posts.

New York's most frequently used longitudinal traffic barrier systems consist of steel rail elements--cable, W-beam, or box beam--mounted on S3x5.7 steel posts. These light-post barriers depend primarily on rail tension or beam bending to redirect impacting vehicles because the posts yield on impact to prevent snagging of vehicles. Traffic accident studies confirm that their performance has generally been very good (1,2).

A new rail element called a Thrie beam was developed several years ago. It is a W-beam that has a third corrugation added. Tests reported by Southwest Research Institute (3) claim good performance for this rail element in strong-post designs, and other tests (4) indicate that tubular Thrie-beam bridge rail performs well as a bridge-rail upgrading system. However, before the work reported here was done, the Thrie beam had not been tested on S3x5.7 posts.

Despite the generally good performance of New York's light-post barriers, the Thrie-beam rail element seems to offer distinct advantages over current designs. The standard height of W-beam rail on S3x5.7 posts in New York State is now 33 in to the rail top. Less height increases the chances that large cars may penetrate the barrier (1). However, at the 33-in mounting height, small cars may tend to lodge beneath the rail.

To protect vehicles from snagging on rigid elements behind the 6-in vertical face of the box beam when there is a transition to a bridge parapet, a second rail element must be introduced before the transition. This second rail requires special hardware and must be terminated safely upstream well behind the main rail. Downstream, the box beam must be terminated flush with the concrete face to eliminate snag points. Very often the approach guiderail is a W-beam element that requires a complicated transition to box beam upstream of the bridge before the transition to the bridge parapet or rail.

Finally, a box-beam median barrier is troublesome to maintain. To replace any damaged posts, rail sections either 18 or 36 ft long that weigh 400 or 800 lb must be removed by using heavy mechanized equipment. Proper alignment of post paddles and rail slots and reassembly of the internal tube splices are difficult. Also, an impacted box-beam median barrier may bend at the mounting slots. Straightening damaged rails is very difficult and reassembly is impossible unless the rail elements are perfectly straight.

Because it is 20 in deep, Thrie-beam performance is much less sensitive to mounting height, and its resistance to penetration is greater for both small and large cars. At bridge parapets, the need for a transition from W-beam to box beam is eliminated. Neither the W-beam nor the Thrie beam need be terminated at concrete anchors. Instead, a commercially available transition of W-beam to Thrie beam is bolted in place to maintain rail tension. Beam