Designing Built-Up Columns

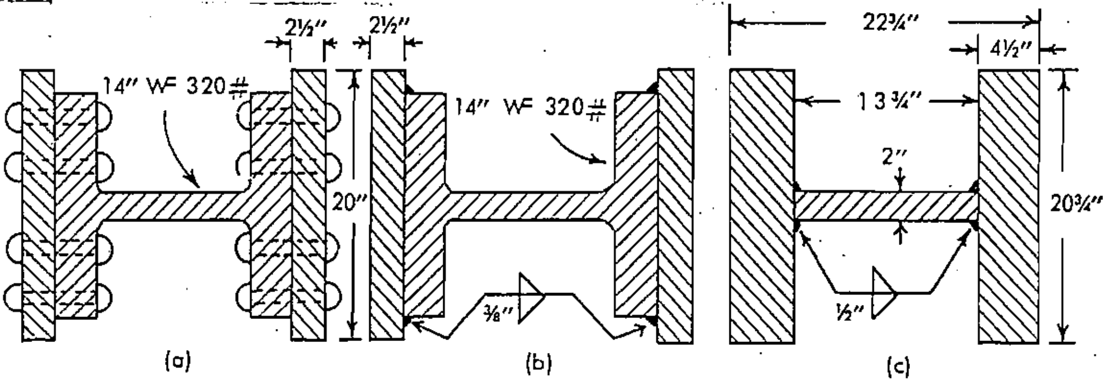


FIGURE 1

1. ADVANTAGES OF WELDED BUILT-UP COLUMNS

In the past, when engineers required steel columns of heavier section than those commercially available, they designed the columns to be made by riveting cover plates to the flanges of 14″ WF rolled sections. See Figure 1(a). The cover plates were sized to produce the required additional section area.

In recent years, fabricating shops have simply substituted fillet welds for rivets and produced the same column section; Figure 1(b). This practice has presented a design problem in getting an efficient transfer of tensile force from the beam flange through the cover plate into the column without pulling the cover plate away from the column flange. The cover plate, being attached only along its two outer edges, tends to bow outward; Figure 2. This results in uneven distribution of forces on the beam-to-column weld.

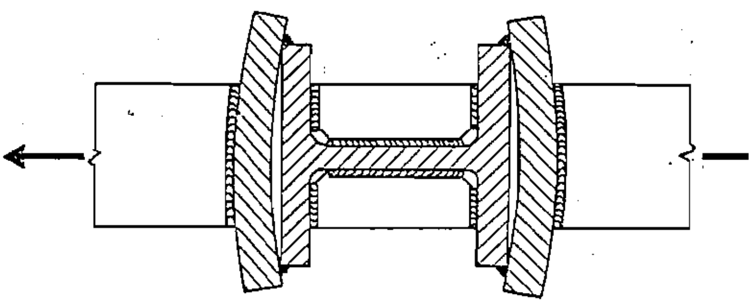
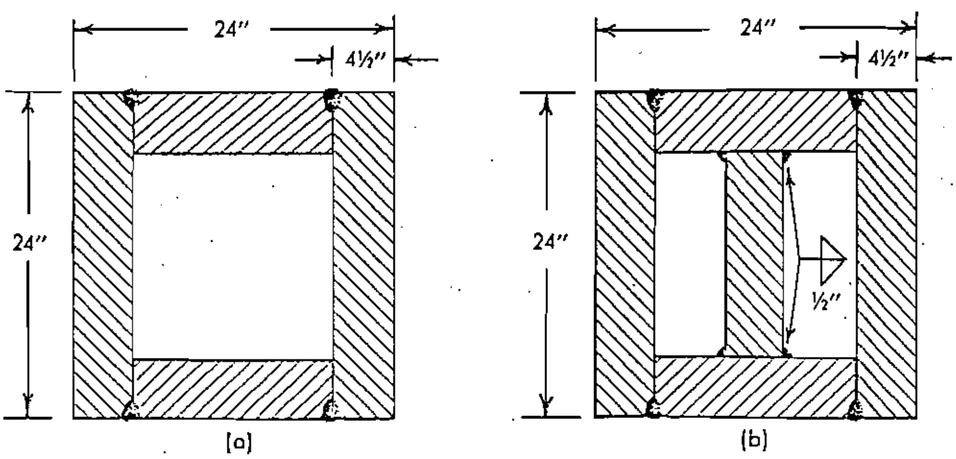


FIGURE 2

The best design is a completely welded built-up column; Figure 1(c). This gives the exact section required without any increase in welding, and there is no problem in transferring tensile forces from the beam flange through the column.

FIGURE 3



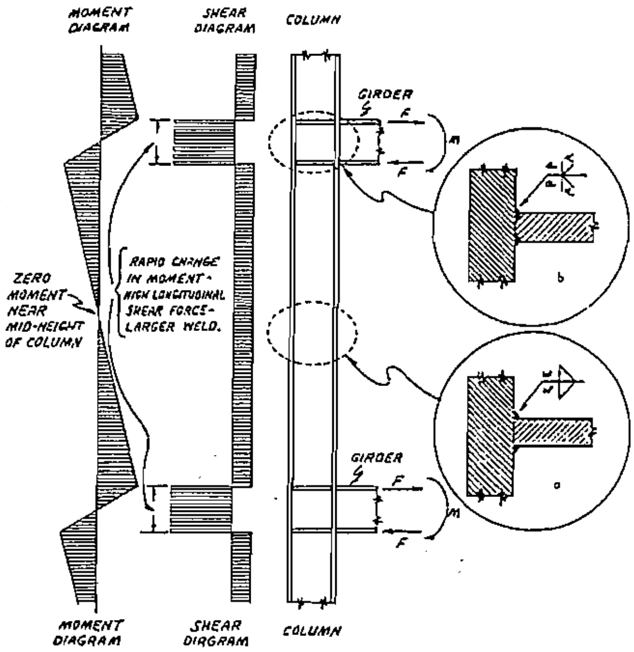


FIGURE 4

For very large column sections, 4 plates can be welded together to form a box section; Figure 3(a). Sometimes a web plate is added to this box for additional area in the lower part of a building; Figure 3(b). Moving up the building, the point is reached where this web plate can be omitted without changing the outer section dimensions.

2. WELD REQUIREMENTS

There are two general requirements for the welds holding the plates of the columns together; Figure 4.

a. The entire length of the column must have sufficient welds to withstand any longitudinal shear resulting from moments applied to the column from wind or beam loads; Figure 4(a). Notice at the left the rather low change in moment along most of the column length.

b. Within the region where the beams connect to the column, this longitudinal shear is much higher because of the abrupt change in moment within this region; Figure 4(b). Also the tensile force from the beam flange will be transferred through a portion of this weld. These two conditions require heavier welds in the connection region.

Various types of welds are employed in fabricating:

a. *Fillet welds* (Fig. 5) require no plate preparation. They can be made to any size simply by making more passes. However, since the amount of weld metal varies as the square of the leg size, these welds can require a large amount of weld metal for the larger sizes. For nominal size welds (approx. ½″ to ¾″), fillet welds are usually used. When their size becomes too large, they are replaced with some type of groove weld because less weld metal is required.

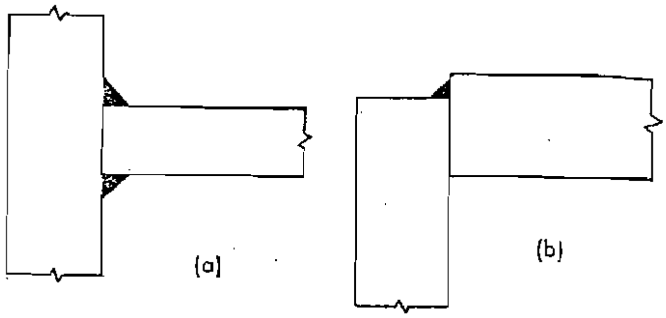


FIGURE 5

b. *Bevel and Vee groove welds* (Fig. 6) require joint edges of the plate to be beveled, usually by the oxygen cutting process. On larger size welds, this additional preparation cost is offset by the reduction in weld metal required. AWS and AISC deduct the first ⅛″ of weld to compensate for any slight lack of penetration into the very bottom of the bevel joint, if welded manually.

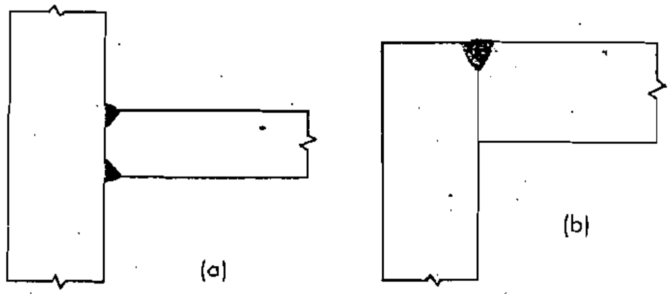


FIGURE 6

c. *J and U groove welds* (Fig. 7) require the plates to be gouged or machined. Machining is seldom used in the structural field, although air carbon-arc gouging is becoming more popular. The J and U welds may not require as much weld metal as the bevel or Vee weld. AWS and AISC allow the full throat or depth of groove to be used.

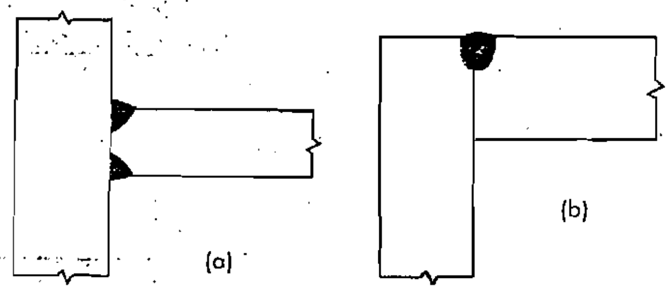
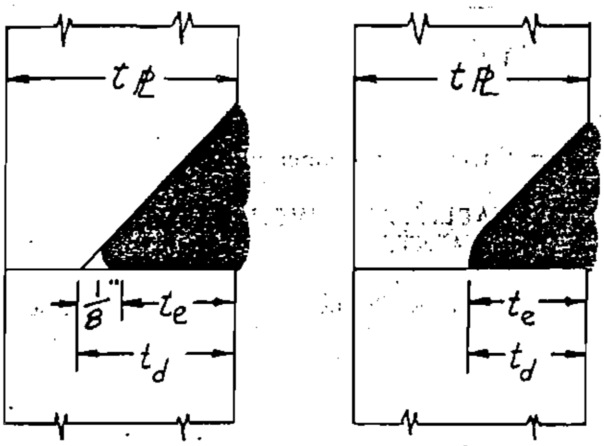


FIGURE 7



a FIGURE 8 b

3. PARTIAL-PENETRATION GROOVE WELDS

Partial-penetration groove welds are allowed in the Building field. They have many applications; for example, field splices of columns, built up columns, built-up box sections for truss chords, etc.

If a vee J or U groove is used, it is assumed the welder can easily reach the bottom of the joint. Thus, the effective throat of the weld (t_e) is equal to the actual throat of the prepared groove (t), see Fig. 8(b).

If a bevel groove is used, it is assumed that the weldor may not quite reach the bottom of the groove, therefore AWS and AISC deduct ⅛" from the prepared groove. Here the effective throat (t_e) will equal the throat of the groove (t) minus ⅛", see Fig. 8(a).

TABLE 1—AISC Allowables for Weld Metal

1.5.3.1 1.5.3.2 1.17.2	A36 E60 & SAW-1 A7, A373 steel E60 & SAW-1 E70 & SAW-2	A36, A242*, A441* steel E70 & SAW-2
FILLET WELDS		
for any direction of force	τ = 13,600 psi f = 9600 ω	τ = 15,800 psi f = 11,300 ω
PARTIAL PENETRATION GROOVE WELDS		
shear	τ = 13,600 psi	τ = 15,800 psi
‡tension transverse to axis of weld	σ = 13,600	σ = 15,800 psi
tension parallel to axis of weld	same as plate	same as plate
compression bearing	same as plate	same as plate
COMPLETE PENETRATION GROOVE WELDS		
tension compression bending shear bearing	same as plate	same as plate

* low hydrogen E60 & SAW-1 may be used for fillet welds & partial penetration groove welds on A242 or A441 steel. (at the lower allowable τ = 13,600 psi)

‡ only for splices or connections of columns or other members subject primarily to axial compression stress

TABLE 2—Partial-Penetration Groove Welds and Fillet Welds

depth of groove or leg size of fillet weld	WEIGHT	FORCE	WEIGHT (20°)	FORCE	WEIGHT (60°)	FORCE	WEIGHT
½"	.482	4,800 5,600	.850	6,800 7,900	.536	5,100 5,925	.425
⅝"	.754	6,000 7,000	1.12	8,500 9,875	.837	6,800 7,900	.664
¾"	1.085	7,200 8,400	1.40	10,200 11,850	1.21	8,500 9,875	.956
⅞"	1.427	8,400 9,800	1.71	11,900 13,825	1.64	10,200 11,850	1.30
1"	1.93	9,600 11,200	2.03	13,600 15,800	2.15	11,900 13,825	1.70
1⅛"	2.44	10,800 12,600	2.37	15,300 17,775	2.72	13,600 15,800	2.15
1¼"	3.02	12,000 14,000	2.74	17,000 19,750	3.35	15,300 17,775	2.66
1⅜"	3.65	13,200 15,400	3.12	18,700 21,725	4.06	17,000 19,750	3.21
1½"	4.34	14,400 16,800	3.52	20,400 23,700	4.83	18,700 21,725	3.82
1⅝"	5.09	15,600 18,200	3.94	22,100 25,675	5.66	20,400 23,700	4.49
1¾"	5.91	16,800 19,600	4.38	23,800 27,650	6.57	22,100 25,675	5.21
1⅞"	6.79	18,000 21,000	4.84	25,500 29,625	7.55	23,800 27,650	5.98
2"	7.72	19,200 22,400	5.32	27,200 31,600	8.58	25,550 29,625	6.80
2⅛"	8.71	20,400 23,800	5.82	28,900 33,575	9.69	27,200 31,600	7.68
2¼"	9.76	21,600 25,200	6.33	30,600 35,550	10.88	28,900 33,575	8.61
2⅜"	10.88	22,800 26,600	6.87	32,300 37,525	12.10	30,600 35,550	9.59
2½"	12.06	24,000 28,000	7.42	34,000 39,500	13.30	32,300 37,525	10.62
2⅝"	13.32	25,200 29,400	8.00	35,700 41,475	14.80	34,000 39,500	11.72
2¾"	14.60	26,400 30,800	8.72	37,400 43,450	16.23	35,700 41,475	12.87
2⅞"	15.96	27,600 32,200	9.21	39,100 45,425	17.75	37,400 43,450	14.07
3"	17.37	28,800 33,600	9.84	40,800 47,400	19.300	39,100 45,425	15.30

force — lbs per linear inch — upper value A7, A373 steel & E60 welds
lower value A36, A441 steel & E70 welds

weight of weld metal — lbs per foot.

Tension applied parallel to the weld's axis, or compression in any direction, has the same allowable stress as the plate.

Tension applied transverse to the weld's axis, or shear in any direction, has reduced allowable stress, equal to that for the throat of a correspanding fillet weld.

Just as fillet welds have a minimum size for thick plates because of fast cooling and greater restraint, so partial-penetration groove welds have a minimum effective throat (t_e) of—

$$t_e \geq \sqrt{\frac{t_p}{6}}$$

where:

t_p = thickness of thinner plate

4. ALLOWABLES AND WELD METAL REQUIREMENTS

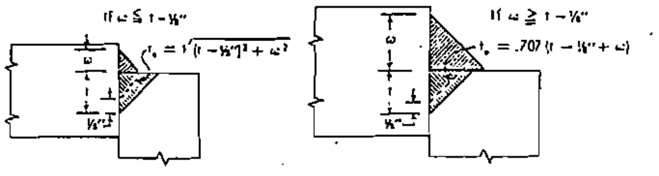
Table 1 lists the AWS and AISC allowable stresses in welds used on Buildings. Values for both partial-penetration and full-penetration groove welds and for fillet welds are included.

Table 2 translates the Table 1 values into allowable forces (lbs/linear in.) and required weld metal (lbs/ft) for fillet welds and several types of partial-penetration groove welds. These values cover weld sizes from ½″ to 3″.

Table 3 provides allowable forces for partial-penetration groove welds reinforced by a fillet weld.

Table 4 directly compares a number of joints to carry a given force, illustrating their relative requirements in weight of weld metal.

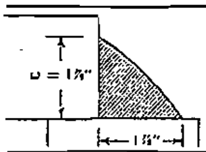
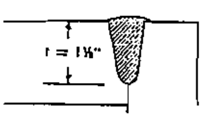
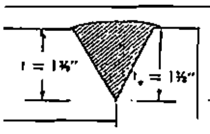
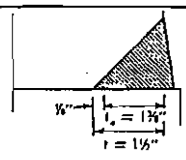
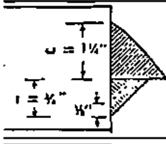
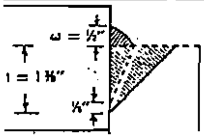
TABLE 3—Partial-Penetration Groove Weld *Reinforced by a Fillet Weld*



depth of groove	leg size of fillet weld ½″	⅝″	¾″	⅞″	1″	1⅛″	1¼″
½″	8,400 9,770 .970	9,610 11,160 1.18	10,810 12,540 1.51	12,010 13,930 1.85	13,010 15,320 2.36	14,410 16,720 2.87	15,620 18,130 3.44
⅝″	9,610 11,160 1.15	10,810 12,540 1.42	12,010 13,930 1.75	13,210 15,320 2.09	14,410 16,720 2.59	15,620 18,130 3.10	16,820 19,530 3.68
¾″	10,870 12,620 1.44	12,010 13,930 1.71	13,200 15,320 2.04	14,410 16,720 2.38	15,620 18,130 2.89	16,820 19,530 3.40	18,020 20,930 3.97
⅞″	12,250 14,220 1.78	13,270 15,410 2.05	14,410 16,720 2.39	15,620 18,130 2.73	16,820 19,530 3.23	18,020 20,930 3.74	19,220 22,310 4.32
1″	14,000 16,270 2.18	14,620 16,980 2.45	15,660 18,170 2.79	16,820 19,530 3.13	18,020 20,930 3.63	19,220 22,310 4.14	20,420 23,700 4.72
1⅛″	15,200 17,650 2.63	16,030 18,620 2.90	17,000 19,740 3.24	18,080 21,000 3.58	19,220 22,310 4.08	20,420 23,700 4.59	21,610 25,100 5.17
1¼″	16,710 19,400 3.14	17,480 20,320 3.41	18,350 21,300 3.75	19,380 22,530 4.09	20,520 23,800 4.59	21,610 25,100 5.10	22,820 26,470 5.68
1⅜″	18,300 21,240 3.69	19,000 22,060 3.96	19,700 22,860 4.30	20,730 24,040 4.64	21,750 25,250 5.14	22,830 26,550 5.65	24,000 27,850 6.23
1½″	19,900 23,120 4.30	20,510 23,810 4.57	21,260 24,680 4.91	22,180 25,780 5.25	23,100 26,830 5.75	24,160 28,060 6.26	25,240 29,350 6.84
1⅝″	21,460 24,930 4.97	22,100 25,670 5.24	22,800 26,460 5.58	23,600 27,400 5.92	24,510 28,450 6.42	25,470 29,600 6.93	26,550 30,830 7.51
1¾″	23,100 26,830 5.63	23,650 27,500 5.96	24,350 28,300 6.30	25,100 29,200 6.64	25,950 30,170 7.14	26,850 31,220 7.65	27,900 32,400 8.23

1st value force lbs per linear inch A7, A373 steel & E60 welds
2nd value force lbs per linear inch A36, A441 steel & E70 welds
3rd value weight of weld metal lbs per foot

TABLE 4—Joints to Carry Force of 20,000 lbs./lin inch
A36 Steel E70 Welds

		ALLOWABLE FORCE	WEIGHT OF WELD METAL
	ω = 1⅞″	21,000#/in	6.79#/ft
	t = 1⅜″	21,725#/in	3.12#/ft
	t = 1⅜″	21,725#/in	4.06#/ft
	t = 1½″	21,725#/in	3.82#/ft
	ω = 1¼″ t = ¾″	20,930#/in	3.97#/ft
	ω = ½″ t = 1⅜″	21,240#/in	3.69#/ft

5. COMBINING WELD TYPES

There are several ways in which different types of welds can be combined in economically fabricating built-up columns to meet the two basic requirements: a) welds from end-to-end of column to withstand longitudinal shear resulting from (wind and beam load) applied moments, and b) heavier welds in connection regions to withstand higher longitudinal shear due to abrupt change in moment, and to carry tensile force from the beam flange. The following cases illustrate combinations that permit optimum use of automatic welding:

Case 1

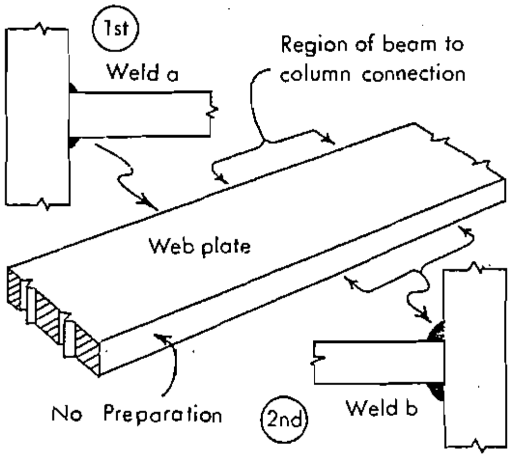


FIGURE 9

If the weld sizes are not too large, the column may be first fillet welded with weld (a) along its entire length. Second, additional passes are made in the connection region to bring the fillet weld up to the proper size for weld (b).

Case 2

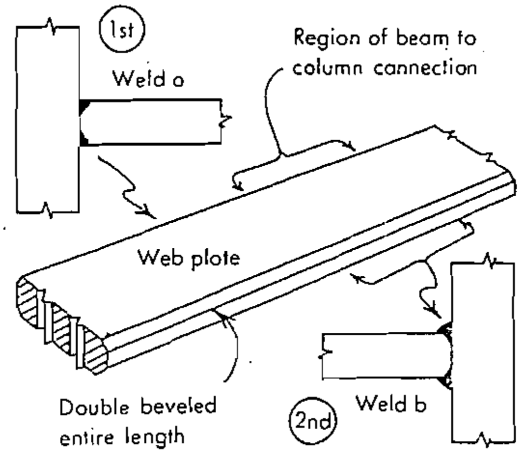


FIGURE 10

The web plate is beveled to the proper depth on all 4 edges along the entire length. Groove weld (a) is first made along the entire length. Second, fillet weld (b) is made over the groove weld within the connection region to bring it up to the proper size.

Case 3

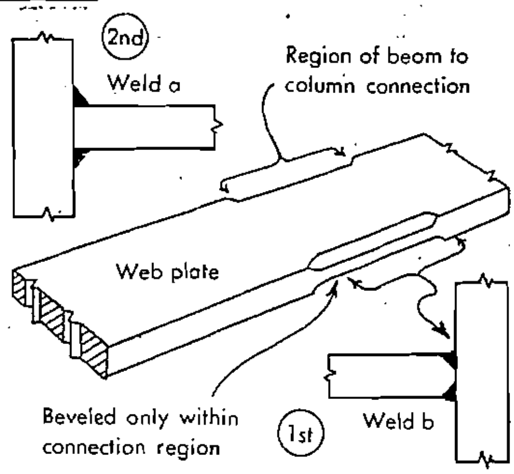


FIGURE 11

The web plate is beveled to the proper depth along short lengths within the connection region. First, groove weld (b) is made flush with the surface within the connection region. Second, fillet weld (a) is made along the entire length of the column.

Case 4

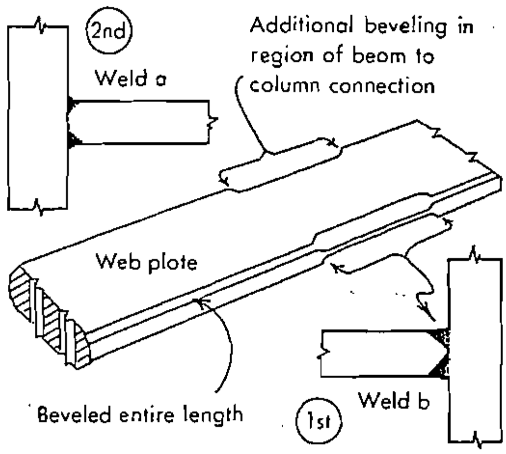


FIGURE 12

The web plate is beveled to the proper depth on all 4 edges along the entire length. Within the connection region, the web is further beveled to a deeper depth. First, groove weld (b) is made within the connection region until the plate edge is built up to the height of the first bevel. Second, groove weld (a) is made along the entire length.

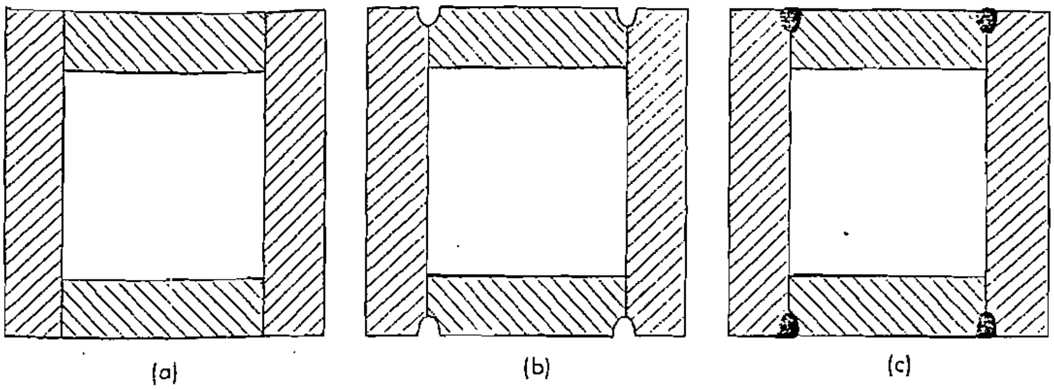


FIGURE 13

6. BOX SECTIONS

In column box sections, J and U groove welds may be substituted for bevel and Vee groove welds if the fabricator is equipped to gouge and prefers to do so rather than bevel. Since beveling is a cutting method, the plates must be beveled before assembling them together. Gouging, however, may be done either before or after assembling. Further, heavy J or U groove welds normally require less weld metal than the bevel or Vee groove welds.

Some fabricators, in making built-up box sections, have assembled and lightly tack welded the plates together without any preparation; Figure 13(a). The joints are next air carbon-arc gouged to the desired depth for very short distances and further tack welded; Figure 13(b). Next, the longer distances in between tack welds are air carbon-arc gouged. When this is completed, the entire length is automatically submerged-arc welded together; Figure 13(c).

7. BEAMS FRAMING INTO BUILT-UP BOX COLUMNS

At first glance it might be thought that the requirements for a beam flange welded to the flange of a built-up box column, Figure 14(a), would be similar to the beam flange welded to the flange of an I shaped column, Figure 14(b). This is because the box column flange is treated as a beam simply supported at its two outer edges, Figure 14(c); it has the same maximum bending moment as the WF column flange treated as a beam supported at its center, Figure 14(d).

The following analysis of a beam flange welded to

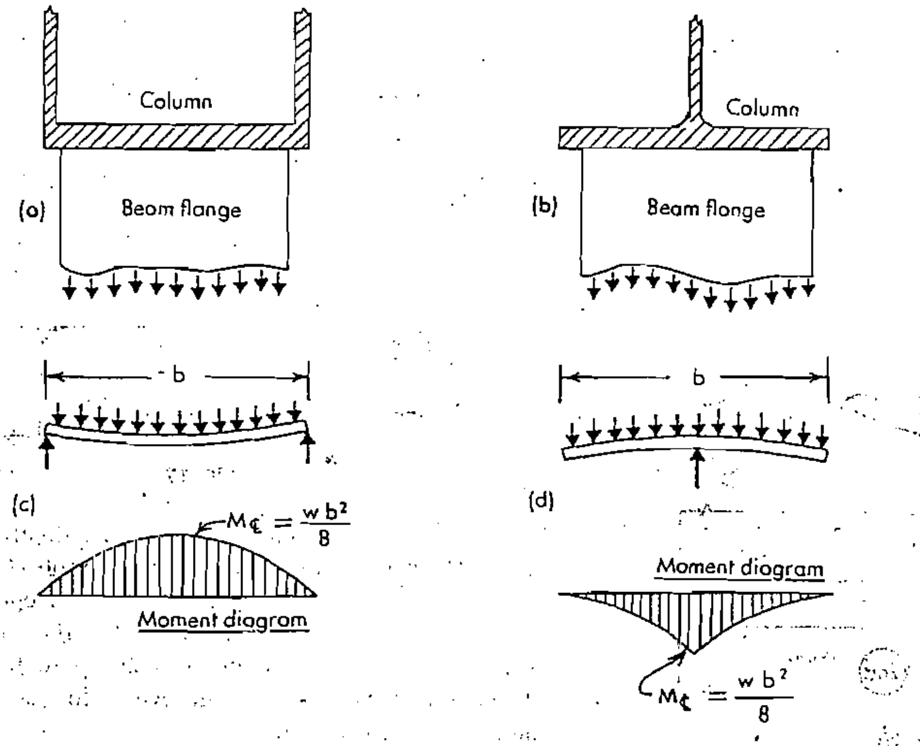


FIGURE 14

a box column, Figure 15(a), is based upon a similar analysis of a line force applied to a cover-plated WF column, Figure 15(b). The latter analysis was made by Dr. T. R. Higgins, Director of Engineering and Research of the AISC.

The following assumptions are made:

1. The length of the box column flange resisting this line force is limited to a distance equal to 6 times its thickness above and below the application of the line force. See Figure 16.

2. The edge welds offer no restraining action to this flange plate. In other words, these two edges are just supported. The upper and lower boundary of this portion of the column flange are fixed.

3. The tensile line force applied to this flange area is uniformly distributed.

At ultimate load (P_u), it is assumed that this rectangular plate has failed as a mechanism with plastic hinges forming along the dotted lines.

The internal work done by the resisting plate equals the summation of the plastic moments (M_p) multiplied by the angle change (ϕ) along these edges.

The external work done equals the ultimate load (P_u) multiplied by the virtual displacement (Δ).

By setting these two expressions equal to each other, it is possible to solve for the ultimate load (P_u) which may be applied to this portion of the flange plate.

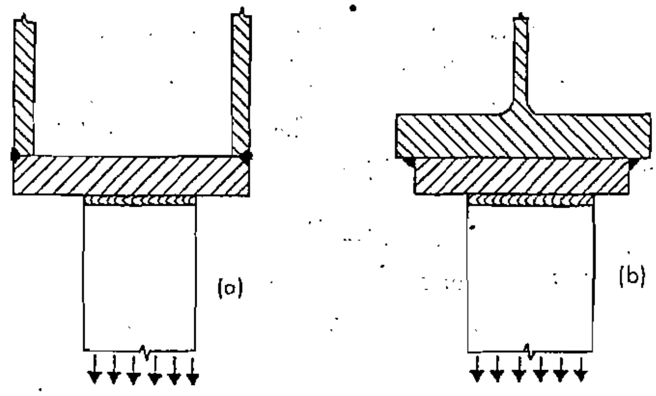


FIGURE 15

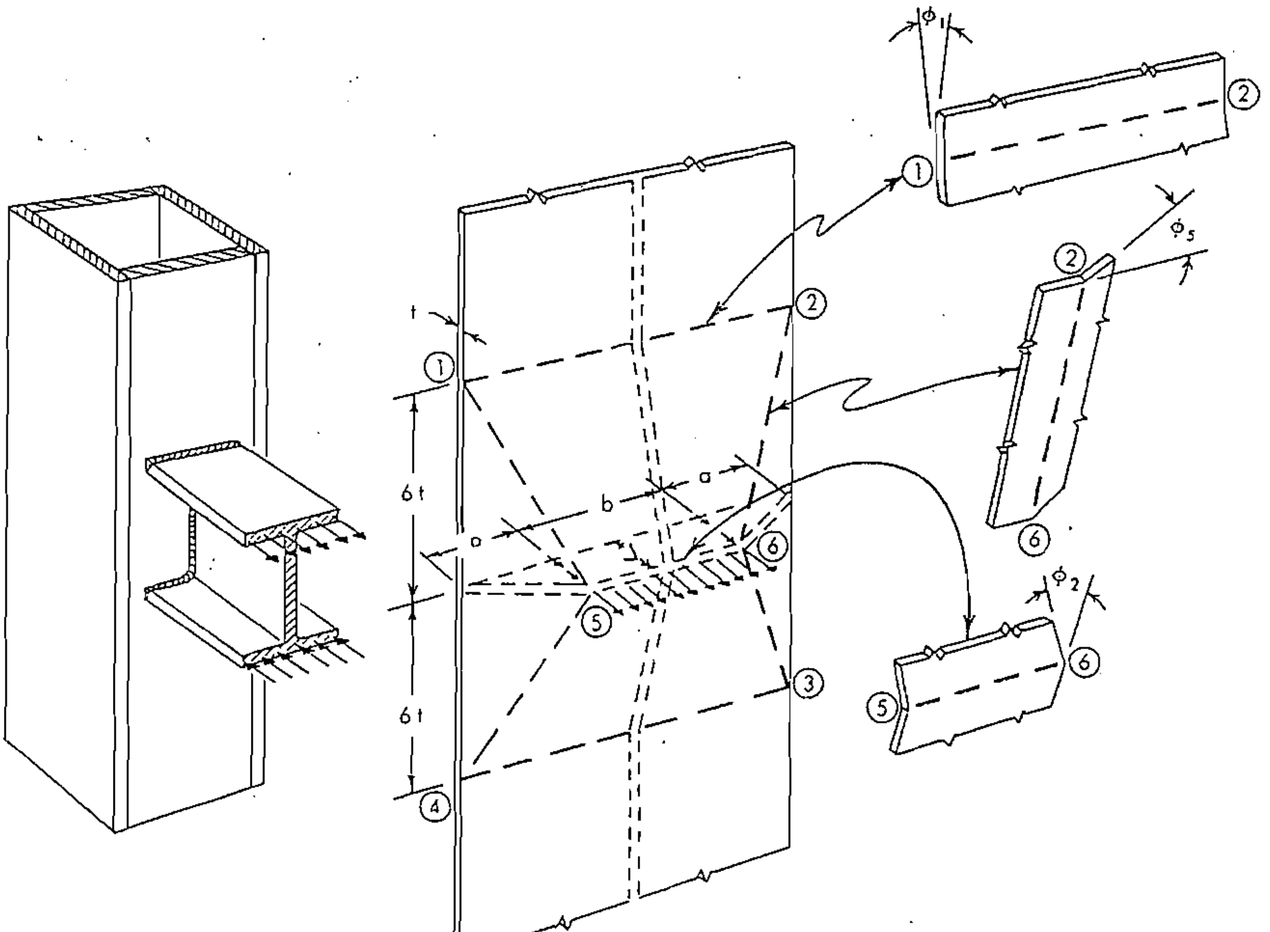


FIGURE 16

At ultimate loading (P_u), plastic moments (M_p) will build up along the dashed lines (Fig. 16) to form plastic hinges. The internal work done, when this plate is pulled out, will be the plastic moment (M_p) multiplied by the corresponding angle changes (ϕ) along these lengths:

angle ϕ_1 along ①—② & ③—④

angle ϕ_2 along ⑤—⑥

angle ϕ_5 along ①—⑤, ②—⑥, ③—⑥ & ④—⑤

With reference to Figure 17:

Distance ②—⑥ $= \sqrt{a^2 + 36\ t^2}$

$$\frac{⑥—ⓨ}{\sqrt{a^2 + 36\ t^2}} = \frac{6\ t}{a}$$

or distance ⑥—ⓨ $= \frac{a}{6\ t} \sqrt{a^2 + 36\ t^2}$

$$\tan \phi_6 = \frac{\sqrt{a^2 + 36\ t^2}}{ⓨ—ⓨ} \text{ also } = \frac{a}{6\ t}$$

or distance ⓨ—ⓨ $= \frac{6\ t}{a} \sqrt{a^2 + 36\ t^2}$

Now find the angle changes (ϕ) along the hinges at ultimate load:

$$\phi_1 = \frac{\Delta}{6\ t}$$

$$\phi_2 = 2\ \phi_1 = \frac{\Delta}{3\ t}$$

and since

$$\phi_3 = \frac{\Delta\ a}{6\ t \sqrt{a^2 + 36\ t^2}}$$

and

$$\phi_4 = \frac{6\ \Delta\ t}{a \sqrt{a^2 + 36\ t^2}}$$

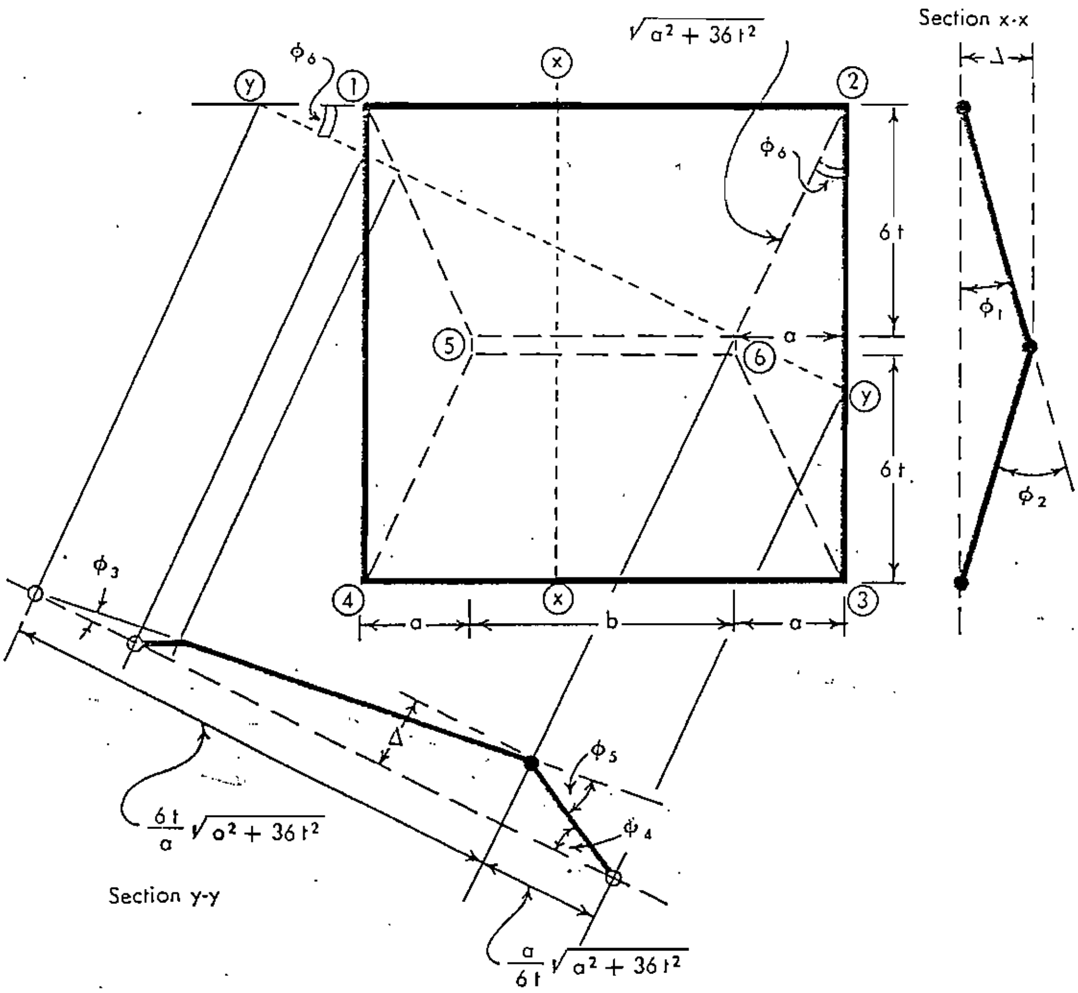


FIGURE 17

$$\therefore\ \phi_5 = \phi_3 + \phi_4 = \frac{\Delta}{6\ at}\sqrt{a^2 + 36\ t^2}$$

internal work

$$= M_p\left[\phi_1\, 2(2\,a + b) + \phi_2\, b + \phi_5\, 4\sqrt{a^2 + 36\,t^2}\right]$$

$$= M_p\left[\frac{\Delta}{6\,t}\,2(2\,a + b) + \frac{\Delta\, b}{3\,t} + \frac{\Delta}{6\ a\ t}\left(\sqrt{a^2 + 36\,t^2}\right)\left(4\sqrt{a^2 + 36\,t^2}\right)\right]$$

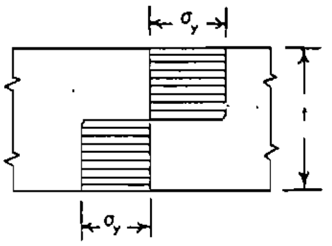
$$= M_p\,\Delta\left[\frac{2(a + b)}{3\,t} + 4\left(\frac{a^2 + 36\,t^2}{6\ a\ t}\right)\right]$$

$$= \frac{2\,M_p\,\Delta}{3\,t}\left(2\,a + b + 36\frac{t^2}{a}\right)$$

where the plastic moment (M_p), in in.-lbs/linear inch is—

$$M_p = 2\left(\sigma_y \times \frac{t}{2} \times 1'' \times \frac{t}{4}\right) = \frac{\sigma_y\, t^2}{4}$$

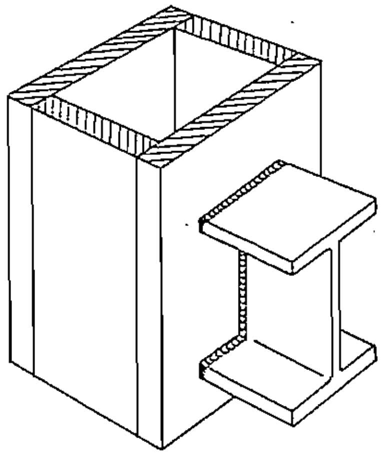
FIGURE 18



external work

$$= P_u\,\Delta$$

FIGURE 19



allowable force

external work = internal work

$$P_u\,\Delta = \left(\frac{2\,\Delta}{3\,t}\right)\left(\frac{\sigma_y\,t^2}{4}\right)\left(2\,a + b + 36\,\frac{t^2}{a}\right)$$

$$= \frac{\Delta\,\sigma_y\,t}{6}\left(2\,a + b + 36\frac{t^2}{a}\right)$$

Applying a load factor of 2, and using the yield strength (σ_y), the allowable force (P) which may be applied to the plate would be—

$$\boxed{P = \frac{t\,\sigma_y}{12}\left(2\,a + b + 36\frac{t^2}{a}\right)}$$

Example

Here:

$t = 3½''$

$a = 5''$

$b = 14''$

$\sigma = 22{,}000$ psi

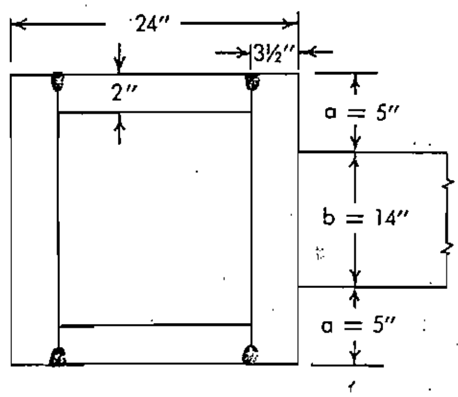
calculated tensile force on beam flange = 386 kips

The allowable force:

$$P = \frac{t\,\sigma_y}{12}\left(2\,a + b + 36\,\frac{t^2}{a}\right)$$

$$= \frac{(3½)(36\text{ kips/sq in.})}{12}\left(2 \times 5'' + 14'' + 36\,\frac{(3½)^2}{5}\right)$$

$$= 1178 \text{ kips} > 386 \text{ kips} \quad \underline{\text{OK}}$$



8. TYPICAL APPLICATIONS

Equitable Life Assurance Building

Columns for the Equitable Life Assurance building in San Francisco, an earthquake area, were built and erected in 3-story lengths. The columns were uniformly tapered $\frac{3}{32}$ in./ft from the base to the 14th story.

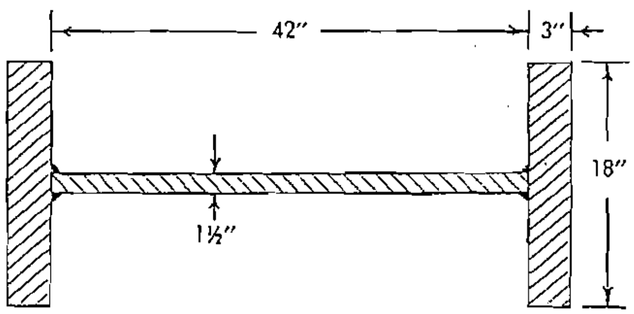


FIGURE 20

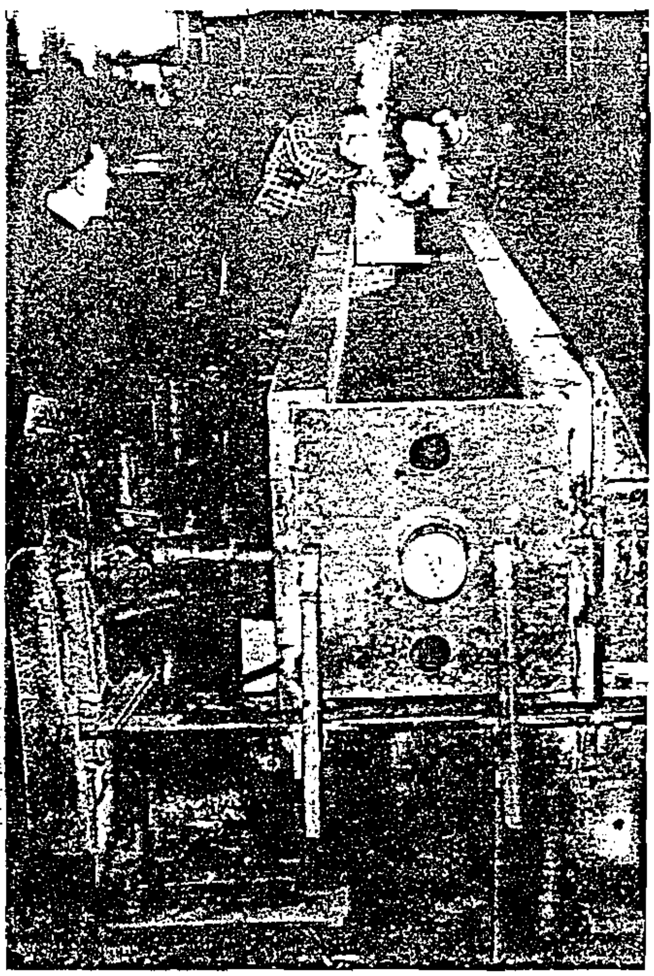


FIGURE 22

Exterior columns started with a 42″ web at the bottom, tapering to a 12″ web at the 14th story level; Figure 20. Flanges were 18″ × 3″ at the base. The tapered columns were fabricated by welding two flange plates and a web together. L-shaped columns were used at the corners of the building.

C.I.L. House

The 32-story C.I.L. House in Montreal, Canada has the heaviest "H" section columns ever constructed. The fabricated columns weigh as much as 2,000 lbs/ft. A typical column, Figure 21, consists of two 7⅛″ × 28″ flange plates welded to a 5″ × 16½″ web plate.

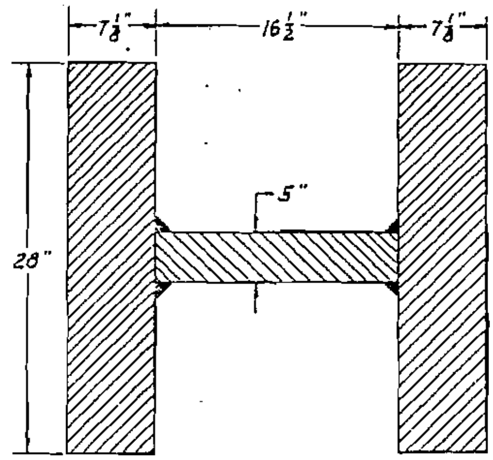


FIGURE 21

Automatic submerged-arc welding was used in fabricating these columns; Figure 22. Simple continuous fillet welds of about ¾″ leg size join the column flanges to the web. Because of the greater forces within the beam-to-column connection region, these welds were increased in size by beveling the web.

The depth of the bevel for this double beveled T-joint varied with the forces to be transferred, but ranged from a minimum of ½″ on each side of the web up to 100%. Less than 10% of these groove welds required 100% beveling. The grooved joints extended in length slightly above and below the depth of the connecting beam and ranged in length from 2′ to 5′.

Joint preparation involved beveling with oxygen cutting equipment at a 22° to 30° angle to the correct depth. After tacking the flange to the web, the weldor lightly air carbon-arc gouged the bottom of the joint prior to welding to open it up for the root pass; the result was a modified J-groove.

The columns, 2 stories high, range from 22′ to 34′ in length. Flange and web plates were clamped in heavy fixtures to maintain proper alignment during welding; Figure 22. After tack welding, trunnions were

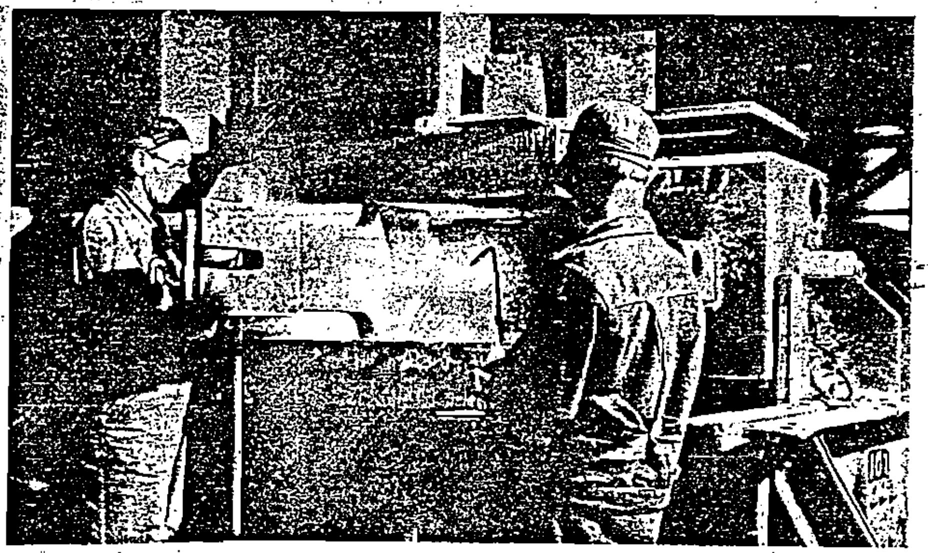


FIGURE 23

attached to the column ends so that all welds could be deposited in the flat position. The columns with trunnions attached were then transferred to the automatic welding unit. After preheating to the correct temperature, using natural gas torches, the shorter-length groove welds were made first. The remaining length of unwelded column was then fillet welded.

After welding, trunnions were removed and the column ends machine faced to proper length. Connection plates were attached after machining, with most welds positioned downhand to achieve maximum welding speed. Preheating preceded the manual welding of these plates in position, using low-hydrogen electrodes; Figure 23.

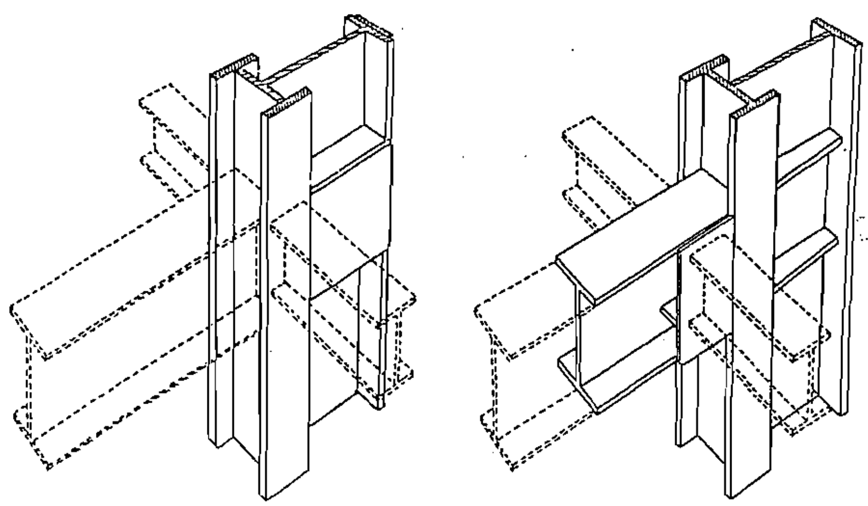
Inland Steel Building
& North Carolina National Bank Building

Elimination of interior columns in a building designed for welded construction is not unique, but usually requires the design and fabrication of special columns; Figure 24.

The column design on the right was used in the Inland Steel Building in Chicago. The inner portion of the built-up column is a standard WF section; the outer portion is a flat plate from 1" to 3" thick. A web plate, from ⅝" to 1½" thick, joins these two segments. Notice that a section of the main girder was shop welded to the fabricated column. Dotted lines show the spandrel beams and remainder of the girder that were field welded to produce a rigid connection. The main girders span 60'.

On the left is a typical column from the North Carolina National Bank Building in Charlotte. A specially rolled WF section is the main segment of this column. Wing plates have been added to one flange and a cover plate to the other to develop the needed column properties. The main girders and spandrels (dotted sections) were later attached by field welding.

FIGURE 24



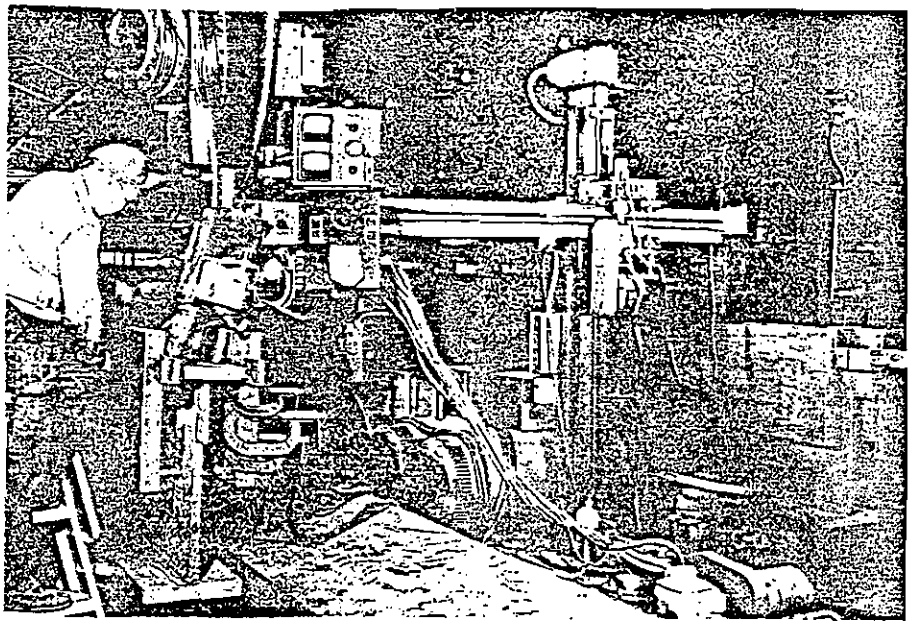


FIGURE 25

Fabrication of special column sections demand low cost, high production assembly and welding techniques. Submerged-arc automatic welding is used extensively in fabricating these columns. The welding head, Figure 25, is mounted on a universal, track traveling type welding manipulator. The manipulator, flux recovery unit, and welding generators are mounted on a self-propelled carriage having a 65 ft track travel distance. Two identical welding fixtures are positioned parallel to and on either side of the carriage track. This has reduced handling time for setup and repositioning of the columns.

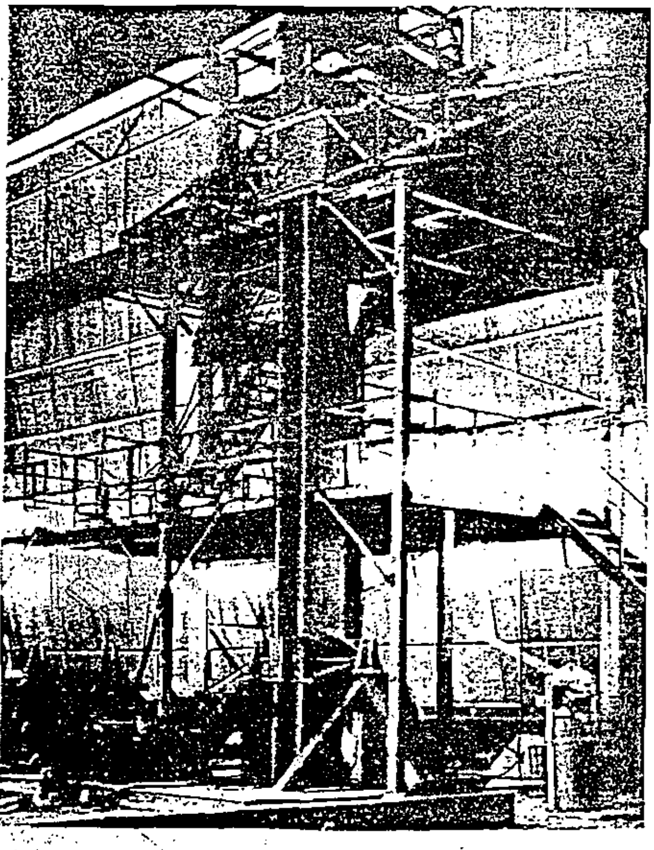


FIGURE 26

During fabrication of columns for the North Carolina National Bank Building, they were placed in a specially designed trunnion fixture; Figure 26. This stood the columns on end. Shop welding of connection details could then be performed in the flat and horizontal position. This facilitated use of semi-automatic, submerged-arc welding and minimized weld costs.

Commerce Towers Building

Columns of similar section configuration were used in the 32-floor Commerce Towers Building in Kansas City. Here, heavy floor loading due to the modern electronic business machines to be installed necessitated very heavy sections.

Column sections were built up by first welding plates into an I section and a T section, and then joining the end of the T section web to the middle of the I section web. The typical column length is 34′ and the lower columns use 5″ flange plates and 5″ web plates.

Tandem-arc automatic submerged-arc welding was used in joining the flange plates to web; Figure 27. The basic weld was a ½″ fillet deposited at 32-36 ipm. Preheat torches ran ahead of the arc.

In joining together the I and T sections, they are assembled in an air-clamping fixture and tack welded; Figure 28. Automatic submerged-arc welding is then used, with the fixture on a rail-mounted carriage.

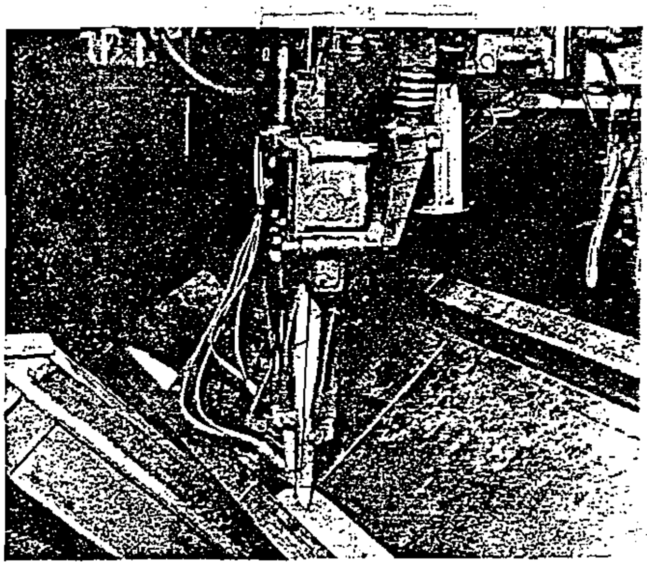


FIGURE 27

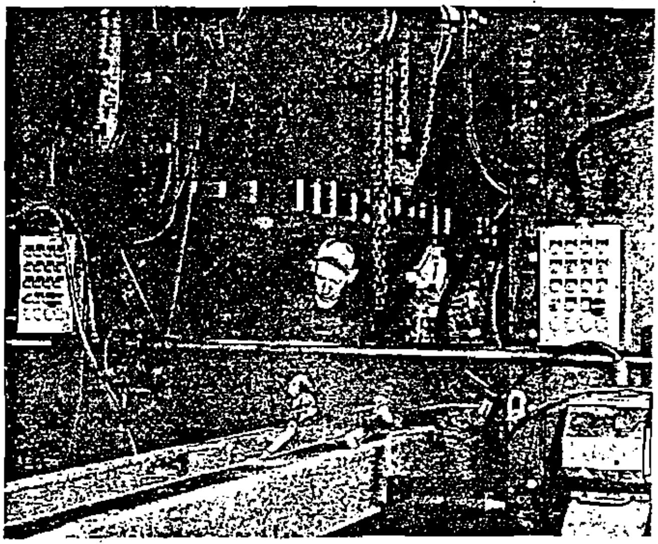
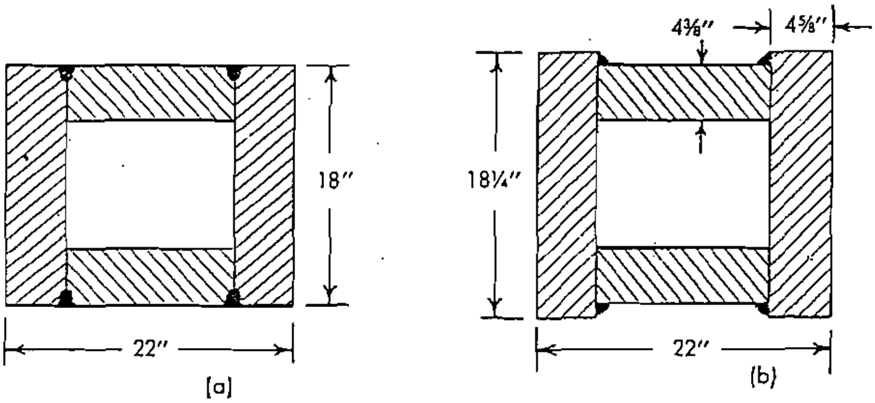


FIGURE 28

FIGURE 29



First Federal Savings & Loan Co. Building

On this project in Detroit, Michigan, the engineer originally detailed the fabricated columns to the 17th floor as built-up box sections, flush around the outside periphery. U-groove welds were to be used; Figure 29(a). This would have meant grooving the plates for the entire length of the column.

The fabricator, chose to set one set of plates slightly in or out; Figure 29(b). This would allow use of continuous fillet welds for the basic welding. The fabricator obtained permission to exceed the original outside column dimension in one direction by ¼″. Any further adjustment was precluded because of the already detailed curtain walls, etc.

The original outside dimensions of the columns were 18″ × 22″ to the 5th floor, 18″ × 20″ to the 11th floor, 18″ × 19″ to the 13th floor, and 18″ × 18″ to the 17th floor. Above the 17th floor, WF sections were used. The modified box section on the lower floors were then built up from two 18¼″ × 4⅝″ flange plates, with two 12¾″ × 4⅝″ web plates recessed slightly to permit the fillet welding. Above the 5th floor, the smaller plates were set out slightly.

In general, these full-length welds were ½″ fillets; with ⅜″ fillets for plates 2¼″ or less in thickness. This eliminated plate preparation except for short distances in the region of the beam-to-column connections. Here the plates were previously beveled, to the required depth, varying from ⅜″ to 5/16″ depending upon load requirements. The typical joint consisting of the beveled groove weld topped by the continuous fillet weld extended 9″ above and below the beam-to-column connection.

9. FIELD SPLICES

Partial-penetration groove welds; either single bevel or single J, may be used for the field splicing of columns. The information presented previously under "Partial-Penetration Groove Welds" will apply here.

Attaching angles shop-welded to the columns serve to temporarily hold the column sections in alignment. For the H column in Figure 30, using high tensile bolts, this connection was considered sufficient to transfer any horizontal shear force across the

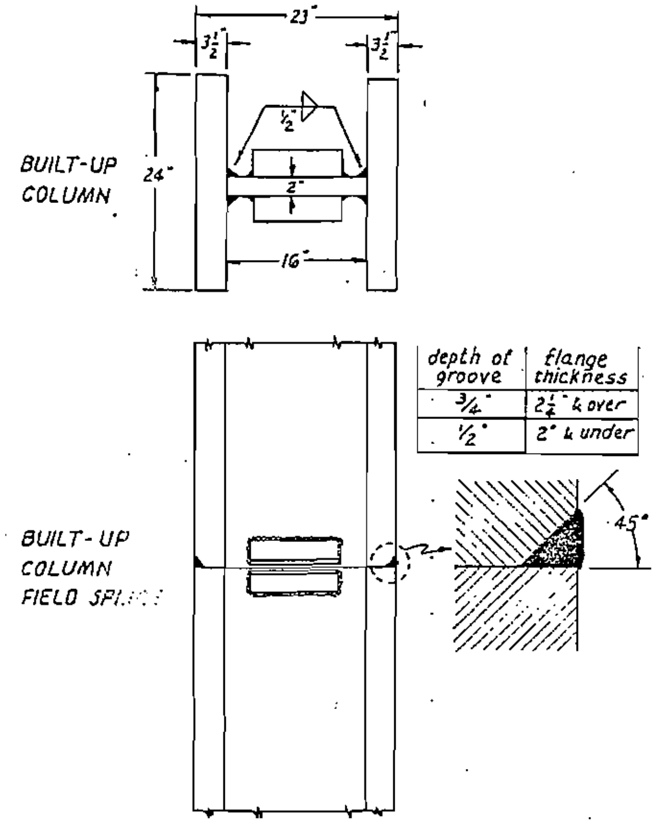


FIGURE 30

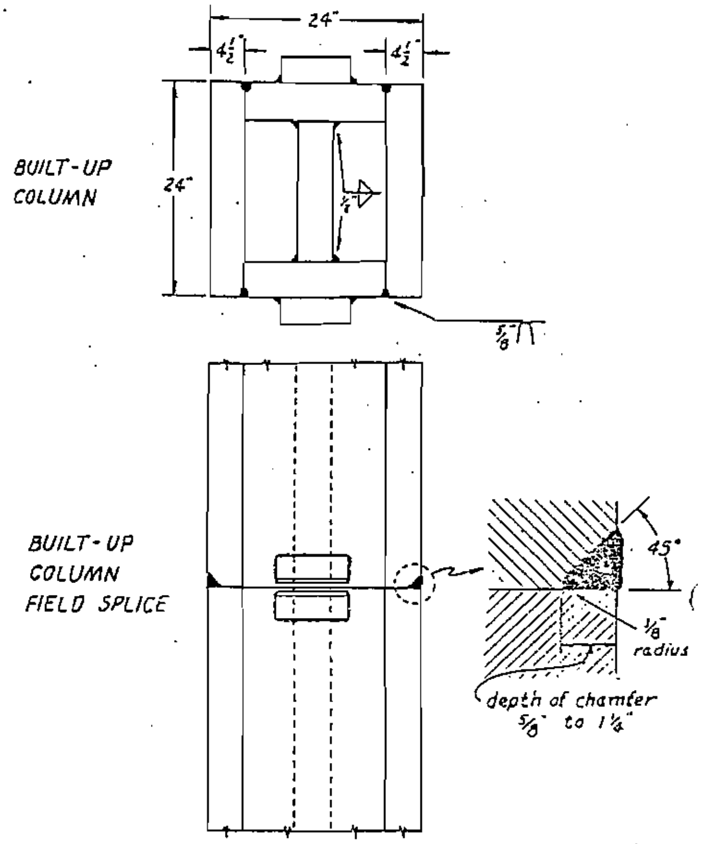


FIGURE 31

web in this direction. The column field splice, consisting of two single bevel, partial-penetration groove welds, would transfer any horizontal shear in the other direction.

FIGURE 32

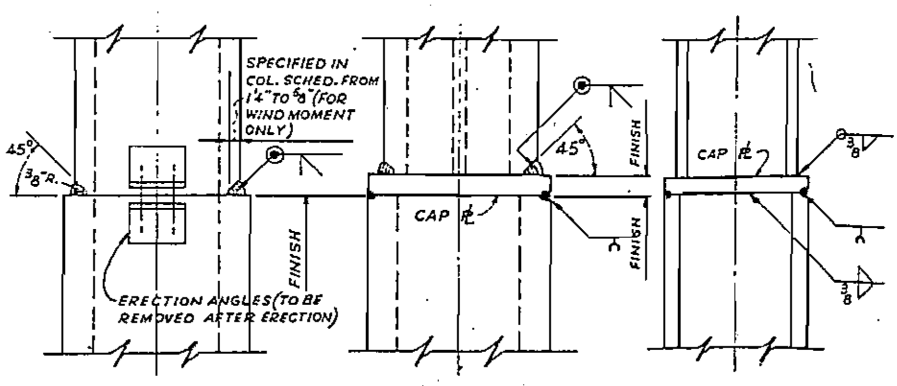
For the box column in Figure 31, the column field splice consisted of a partial-penetration J groove weld on all four sides of the column. These four welds would transfer any horizontal shear in the column splice. The attaching angles here were used simply to facilitate erection.

Partial-penetration welds on column splices permit fast semi-automatic welding techniques to be used in the field. In the Commerce Towers project, semi-automatic arc welding with self-shielding, cored electrode permitted deposition of 100 lbs/man/8-hour day; Figure 32.

10. CONCLUSION

The full economic impact of welded steel built-up columns in construction of tall multi-story buildings, can be realized by carefully considering the major cost factors. These are column design, placement of welds, joint design, weld size, and procedure. The dominating objective is the fullest use of automatic arc welding methods in the shop, with an extension of these benefits into the field by use of semi-automatic arc welding for beam-to-column connections and for field splices.

Built-up columns are a key design feature of the 28-story Michigan Consolidated Gas Co. Building in Detroit. Welding was considered to be the only practical method for fabricating these columns which carry a maximum load of approximately 6800 kips. Photo shows a field splice of the column, revealing the shop beveling that facilitated welding. Clip angles shown are for temporary use during erection.

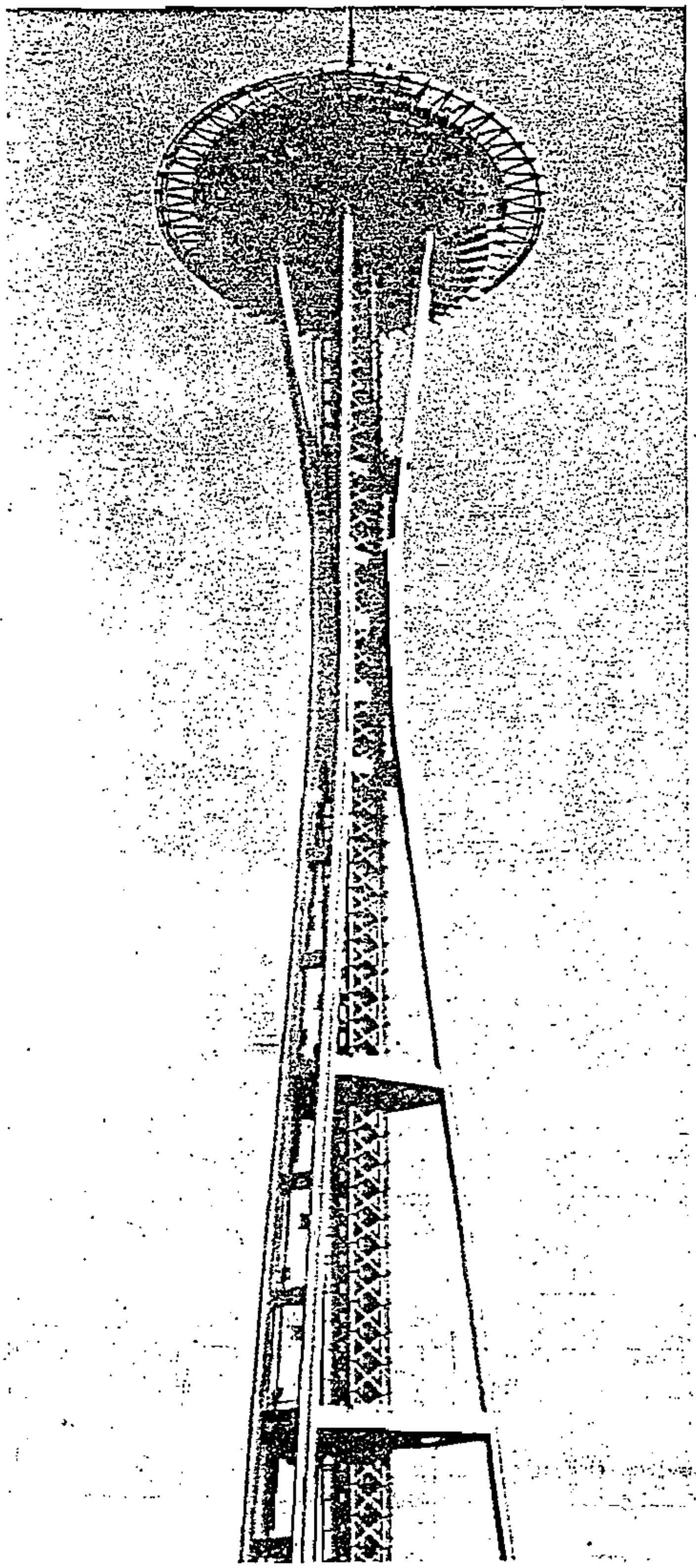


Typical splice for built-up column

Alternate splice for built-up column

Typical splice for WF column

Splice details from the Michigan Consolidated project show how maximum use was made of material at minimum weight.



Automatic submerged-arc welding was used extensively in shop fabricating the unique and complex built-up columns for the 500′ space tower which overlooked the Seattle World's Fair. Approximately 50% of all shop welding was with the submerged-arc process; 25% with self-shielding cored wire, semi-automatically; and the remainder manual stick electrode. At the top of the tower is a five-story observatory and restaurant. The structure required 3400 tons of structural steel.