Framing Connections for Square and Rectangular Structural Tubing

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THE SQUARE AND RECTANGULAR structural tube was introduced as a new structural shape several years ago. In this short period, these shapes have found application in many structures; they have been particularly popular for columns in low-rise buildings such as schools and factories. Other recent applications include space frame and Vierendeel girder construction.

Two of the outstanding characteristics of the tubular shape are (a) high efficiency in carrying compressive loads, and (b) clean appearance for exposed framing. In addition, it has exceptional torsional rigidity in comparison to open structural shapes such as the wide flange beam and channel section. Square and rectangular tubes are now readily available in many sizes and in several grades of steel.

While conventional steel design procedures are appropriate for sizing tubular members, there has been considerable question as to the best methods for connecting tubes to tubes and wide flange beams to tubes. The purpose of this paper is to attempt to resolve some of these questions by presenting the results of recent research conducted at Cornell University. Primary emphasis has been on studying the characteristics and behavior of simple connections (AISC Type 2 framing); although some pilot studies have been conducted on rigid connections (AISC Type 1 framing). Both classes of connections will be discussed herein. All tests were conducted on square tubing, but the results should be applicable to rectangular tubing with no significant error.

SIMPLE FRAMING CONNECTIONS

Requirements—The primary requirements for adequate simple framing connections can be summarized as follows:

(a) Sufficient strength in direct shear to carry safely the maximum reactions of the connected beam.

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(b) Flexibility, or lack of rigidity, such that bending moment transmitted to the column is at a minimum. This flexibility requirement is reflected in Section 1.15.4 of the 1963 AISC Specification in which a minimum beam end rotation is specified.

(c) Ability to carry safely the moment that is imposed on any connection which is not a perfect hinge. As nearly all simple connections do possess some bending rigidity, a moment will be induced on the connection as the ends of the connected beam rotate under dead and live loadings. Therefore, even though a simple connection is not designed to carry bending moment, it must be sufficiently strong to carry this induced moment. Unless some part of the connection can deform, either elastically or plastically, possible failure can occur in the connection itself or in the connectors (bolts, welds, or rivets).

(d) A basic connection configuration which does not produce extensive deformation of the column becomes an important factor for connections to tubes because of the inherent flexibility of the flat tube walls.

(e) The connection should not require complex design, fabrication, or erection procedures, and should use a minimum of materials.

Testing Program—The five simple connection types discussed herein are shown in Fig. 1; they have been termed Type A, B, C, D, and E for identification purposes. Moment-rotation characteristics were determined for all connections by testing symmetrical double cantilever specimens (two connections per specimen) as shown in Figs. 2a and 2b. Shear strengths were determined by inverting the specimen at the conclusion of the moment test and loading it such that each connection was subjected to essentially pure shear (Fig. 2c).

The moment-rotation tests were conducted to check each connection against requirements (b), (c), and (d)

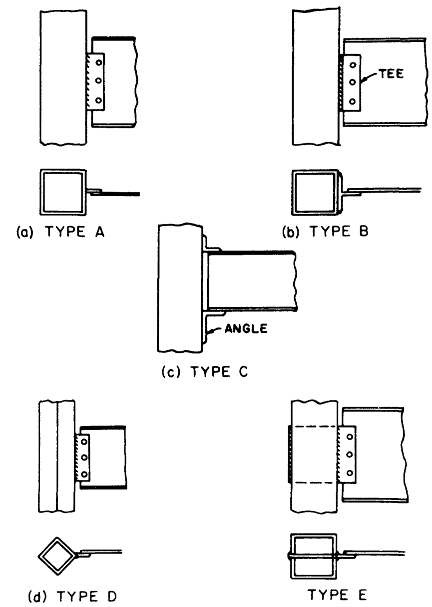

Fig. 1. Simple connection types tested

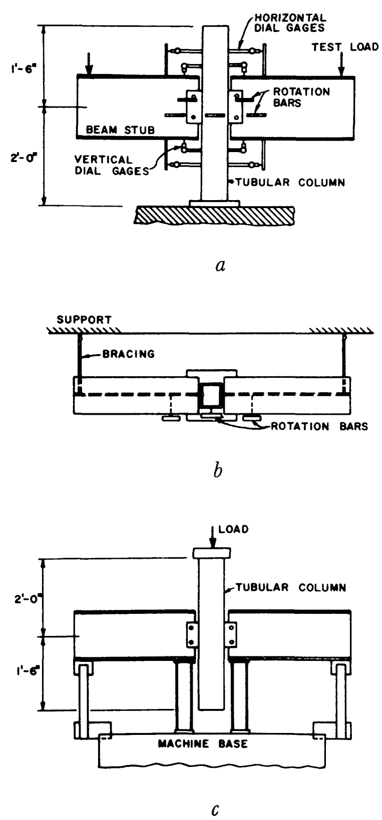

Fig. 2. Test specimen configurations: (a) M-φ test, (b) M-φ test (plan), and (c) shear test

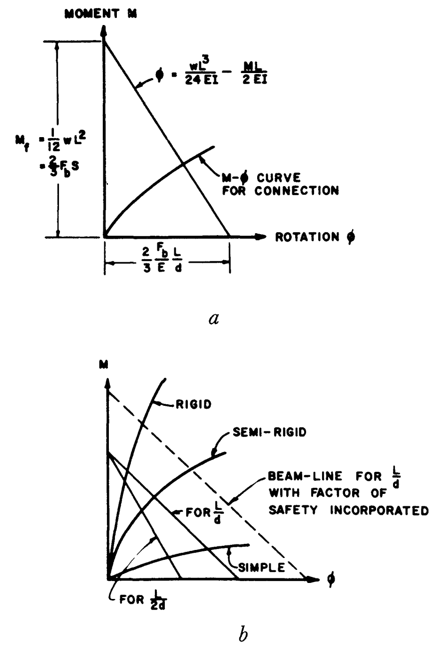

Fig. 3. Beam-line concept: (a) Beam-line for uniformly loaded beam and (b) M-φ relations for different types of connections

above. Flexibility and moment capacity can best be investigated by plotting values of applied moment M vs. measured connection rotation ϕ; typical M-ϕ plots for the three classes of AISC connections are given in Fig. 3b. To judge the behavior of the connection when used with a particular beam, the so-called beam line is superimposed on the M-ϕ curve. The beam line simply represents the end rotation of the beam (which must be compatible with connection rotation ϕ provided no slippage occurs in the connection) as a function of the end moment M. The beam line for a uniformly loaded beam is defined in Fig. 3a, in which the intercepts have been determined by successively setting $\phi = 0$ and $M = 0$ to represent completely fixed and perfectly hinged ends, respectively, and incorporating the relation $M = wL^2/8 = F_bS$. The intersection of the M-ϕ curve and the beam line defines the amount of end moment on the beam and the rotation which it undergoes. It is evident that considerably higher rotations are needed for long spans than for short spans because the ϕ intercept is directly proportional to the length-to-depth (L/d) ratio of the beam.

The beam line used for checking requirement (c) was chosen as that for $L/d = 25$. An overload factor of 1.65, which shifts the beam line up and to the right (Fig. 3b) was used to establish the "safe beam line". If the M-ϕ curve for any particular beam reaches this beam line, as for the semi-rigid and rigid connections in Fig. 3b, then the connection is satisfactory; if not, as for the simple connection in Fig. 3b, then the connection is unsatisfactory according to requirement (c).

Selection of test specimens and dimensions of connections are summarized in Table 1. Connections were welded to the tubes and bolted to the beams, with A325 high strength bolts being used on all connections except Type C. All material used met the mechanical property requirements for A36 steel. Additional discussion on choice of test specimens is given in Reference 1.

Design of weldments was by conventional procedures. Eccentricity of load was included in sizing welds for connection Types B and C, but not for the other types.

A summary of test results is given in Table 2. Two stiffnesses are given for each specimen. The first, expressed as a percentage, is the ratio of M/M_f, where M is the moment developed on the connection when it is used with a uniformly loaded beam with a span-to-depth (L/d) ratio of 25, and M_f is the fixed end moment developed for the same beam by the uniform loading w. Beam size is determined from the simple beam moment $wL^2/8$. The second stiffness, which is independent of beam size, is given as the value of moment required to produce a rotation of 0.01 rad. Values of $M_{0.01 \text{ rad}}$ are tabulated in column (4) of Table 2.

Shear tests results are also given in Table 2. The ultimate load factor, or safety factor against failure, is the ratio of the maximum load reached in the test to the design load.

Type A Connections—The Type A connection (Fig. 1a), consisting of a plate welded to the tube face, is immediately attractive from the fabrication standpoint, as it uses a minimum of plate material and weld length. This important characteristic is offset, unfortunately, by the undesirable deformations and stresses induced in the flexible tube wall as the connection plate rotates under load. Most of the rotational capacity of the connection arises from deformation of the tube rather than from deformation of the connection itself. It is evident that this behavior could produce a serious weakening effect on the axial load capacity of the column.

Moment-rotation (M-ϕ) curves for the eight specimens tested are shown in Fig. 4. The specimens, tabulated in Table 1, ranged from $4 \times 4 \times 3/16$ in. tubes with 8 in. deep beams to $8 \times 8 \times 1/2$ in. tubes with 18 in. deep beams. Plotted M-ϕ curves all exhibit the same basic shape, with the connections on the thicker walled tubes being considerably stiffer than those on thin walled tubes. Connections on the thin 8 in. tubes remained nearly elastic up to the safety beam line. The degree of inelastic action of the stiffer connections is emphasized by the divergence of the actual M-ϕ curves from the theoretical elastic curves obtained from an analytical solution (Ref. 1). Stiffness varied from a low of 2.9 percent to a high of 32.8 percent. Theoretical stiffnesses, which compare favorably with measured stiffnesses except for the thick walled 4 in. tubes, are given for a variety of plate lengths and tube sizes in Fig. 5.

Strain gage readings showed that high stresses developed in the connected tube wall at low load levels, particularly at the ends of the connection plate where yielding commenced very early in the load history. The warping action on the connected tube wall was easily discernible by eye and was of serious proportions for the thinner tubes.

All eight specimens were also tested under shear loading. Various failure modes were noted, including local tube wall buckling (A1), web crippling of con-

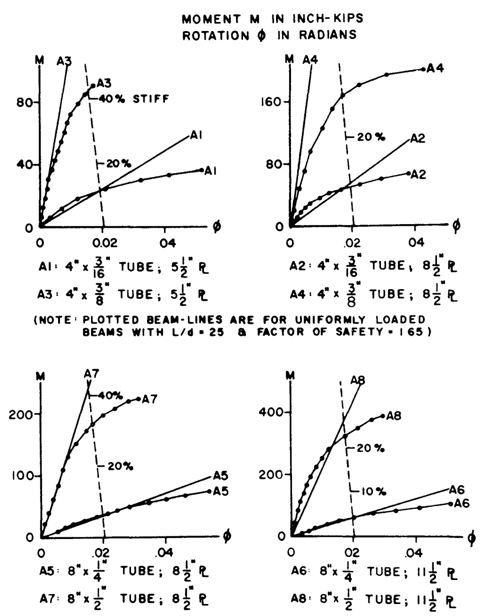

Fig. 4. M-ϕ curves for Type A connections

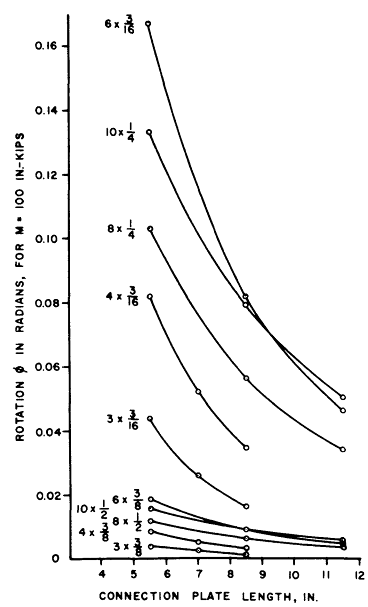

Fig. 5. Predicted stiffnesses for Type A connections

Table 1. Simple Connection Specimens

Specimen	Nominal Tube Size	Nominal w/t Ratio	Connection	Connection Length, in.	Welds, in.	Beam Size	Number and Size of Bolts
A1	4 × 4 × 3/16	21.3	3/8 in. plate	5½	3/16	8WF17	2—7/8 in.
A2	4 × 4 × 3/16	21.3	3/8 in. plate	8½	3/16	12WF31	3—7/8 in.
A3	4 × 4 × 3/8	10.6	3/8 in. plate	5½	3/16	8WF17	2—7/8 in.
A4	4 × 4 × 3/8	10.6	3/8 in. plate	8½	3/16	12WF31	3—7/8 in.
A5	8 × 8 × 1/4	32.0	3/8 in. plate	8½	3/16	12WF31	3—7/8 in.
A6	8 × 8 × 1/4	32.0	3/8 in. plate	11½	5/16	18WF55	4—1 in.
A7	8 × 8 × 1/2	16.0	3/8 in. plate	8½	3/16	12WF31	3—7/8 in.
A8	8 × 8 × 1/2	16.0	3/8 in. plate	11½	5/16	18WF55	4—1 in.
B1	4 × 4 × 3/16	21.3	ST 3.5 I 7.6	8½	5/16	12WF27	3—7/8 in.
B2	4 × 4 × 3/8	10.6	ST 3.5 I 7.6	8½	5/16	12WF27	3—7/8 in.
B3	8 × 8 × 1/4	32.0	(Piece of	11½	7/16	18WF55	4—1 in.
B4	8 × 8 × 1/2	16.0	18 I 54.7)	11½	7/16	18WF55	4—1 in.
B′1	4 × 4 × 3/16	21.3	(Piece of	8½	5/16	12WF27	3—7/8 in.
B′2	4 × 4 × 3/8	10.6	7 I 15.3)	8½	5/16	12WF27	3—7/8 in.
C1	6 × 6 × 3/16	32.0	(Angle	—	7/16	12WF27	—
C2	6 × 6 × 1/2	12.0	5 × 3½ × 3/4)	—	7/16	12WF27	—
D1	6 × 6 × 3/16	32.0	3/8 in. plate	8½	3/16	12WF27	3—7/8 in.
D2	6 × 6 × 1/2	12.0	3/8 in. plate	8½	3/16	12WF27	3—7/8 in.
E1	6 × 6 × 3/16	32.0	3/8 in. plate	8½	3/16	12WF27	3—7/8 in.
E2	6 × 6 × 1/2	12.0	3/8 in. plate	8½	3/16	12WF27	3—7/8 in.

Table 2. Test Results for Simple Connections

Specimen	Tube Size	Moment-Rotation Test				Shear Test		Safety Factor
		(1) M_f, in.-k.	(2) M_{25}, in.-k.	(3) Percent Stiff	(4) $M_{0.01\ \mathrm{rad}}$ in.-k.	Design Load, kips	Test Load, kips	
A1	4 × 3/16	207	17	8.2	16	19.6	60	3.06
A2	4 × 3/16	578	40	6.9	37	34	58[a]	1.71
A3	4 × 3/8	207	68	32.8	74	19.6	62	3.16
A4	4 × 3/8	578	127	22.0	132	34	54[a]	1.59
A5	8 × 1/4	578	25	4.3	22	34	108	3.18
A6	8 × 1/4	1440	42	2.9	36	69	135.8[a]	1.97
A7	8 × 1/2	578	135	23.4	141	34	113	3.32
A8	8 × 1/2	1440	260	18.1	263	69	176.5	2.56
B1	4 × 3/16	500	120	24.0	123	—	—	—
B2	4 × 3/8	500	150	30.0	161	30.6	83.5	2.73
B3	8 × 1/4	1440	230	16.0	240	—	—	—
B4	8 × 1/2	1440	360	25.0	375	69	156	2.64
B′1	4 × 3/16	500	127	25.4	131	—	—	—
B′2	4 × 3/8	500	160	32.0	166	—	—	—
C1	6 × 3/16	500	33	6.6	30	20	83	4.15
C2	6 × 1/2	500	40	8.0	37	20	70	3.5
D1	6 × 3/16	500	130	26.0	140	—	—	—
D2	6 × 1/2	500	185	37.0	183	—	—	—
E1	6 × 3/16	500	178	35.6	185	—	—	—
E2	6 × 1/2	500	190	38.0	191	30.6	111.5	3.64
CT2	3 × 3/16	207	25.5	12.3	27.5	—	—	—
CT6, 7	6 × 3/16	500	20	4.0	18	—	—	—

[a] Shear test stopped because of web crippling.

(1) Fixed end moment for connected beam

(2) Connection moment for beam with $L/d = 25$

(3) Ratio of (2)/(1).

(4) Connection moment for rotation of 0.01 rad

nected beams (A2, A4, and A6), excessive beam web bearing deformations around the bolt holes (A8), and weld tearing (A3). The latter specimen is shown in Fig. 6. Shear capacities were in excess of 2.5 times the design loads in all tests which were not prematurely terminated because of beam web crippling.

The only requirement not met by the Type A connection was requirement (d) having to do with distortion of the connected column. It was deemed necessary to conduct additional tests on column specimens loaded with Type A connections in order to study the amount of reduction of axial load capacity as caused by the connection-induced column deformations. Seven columns with slenderness ratios from 40.8 to 166 were tested as shown in Fig. 7. The test configuration represents a rather severe case—that of connections at the mid-height of a laterally unbraced column. Beam loads were applied through jacks mounted on trolleys that permitted the load to move laterally with the column when it buckled. The entire program is described in detail in Reference 1.

Reduction in column capacity varied from a low of 10–15 percent for $3 \times 3 \times \frac{3}{16}$ in. tubes with slenderness ratios of 53 and 106, and $6 \times 6 \times \frac{3}{16}$ in. tubes with $l/r = 80$, to a high of 30–40 percent for $4 \times 4 \times \frac{3}{8}$ in. tubes with l/r values of 67 and 132. It appears that maximum reduction in strength occurs with smallest width-thickness ratios of tube walls and with lowest values of slenderness ratios, although more extensive testing is needed to substantiate this observation.

Because of the serious tube weakening effects discussed above, the Type A connection cannot be recommended for general usage. Granted that the connection will ordinarily be used at a braced point rather than an unbraced point as in the test situation, one must still be quite conservative in formulating possible applications for this connection. Additional study of the effects of the connection on the tube when attached at a braced point might relax the restrictions imposed below.

A possible use for the connection is at locations where full column strength is either not utilized or not needed, such as in the upper story of a two or three story building constructed with constant cross section tubular columns. In this situation only the bottom story of the column is fully stressed.

In summary, it is recommended that Type A connections be restricted to (a) lightly loaded secondary connections, and (b) those situations where the total design load on the column, on a section immediately below the connection area, is not more than about 60 percent of the safe column capacity, and where the column is laterally supported against buckling at the location of the connection.

Type A connections can be safely designed by proportioning the plate to carry the beam reaction in shear on its net section and by sizing the fillet welds at the plate-tube interface to carry shear only.

Type B Connections—The Type B connection consists of a length of structural tee, shop welded to the tube along both flange edges. This configuration (Fig. 1b) permits reasonable rotation through distortion of the structural tee, and unlike the Type A connection, does not induce high stresses and distortions in the tube wall. Its basic behavior is similar to that of conventional framing angles in that it tends to pull away from the column face on the tension side of the connection, but it is inherently stiffer because of the continuity of the tee flange across the tube face.

This connection has been used in building construction, although it is expensive to fabricate. A variation of the connection, consisting of a single angle welded at its heel to the tube mid-face and at its toe to the tube corner, has also found use in tubular construction.

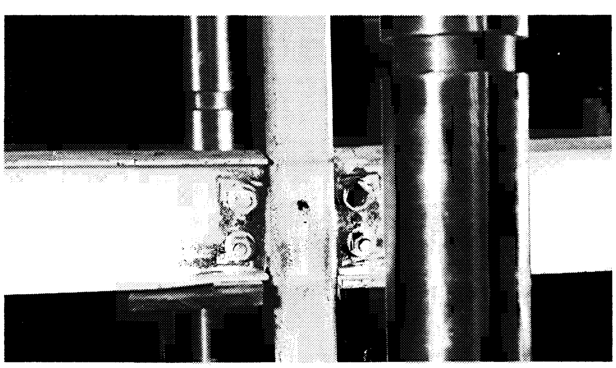

Fig. 6. Shear test, connection A3

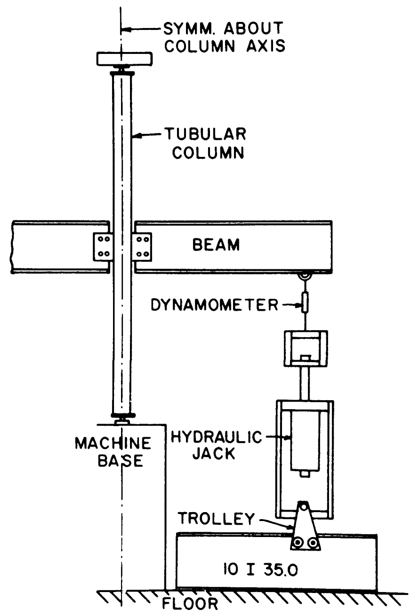

Fig. 7. Column test with Type A connection at mid-length

Four specimens were tested for moment-rotation characteristics; tube sizes ranged from 4 × 4 × 3/16 in. to 8 × 8 × 1/2 in., as tabulated in Tables 1 and 2. The M-ϕ curves of Fig. 8 yielded stiffnesses ranging from 16 to 30 percent. The increased stiffness of the specimens with the thicker walled tubes is clearly apparent.

Strain gage readings indicated that tube stresses remained low during the M-ϕ test. In all specimens the most highly stressed portion of the tee was in the vicinity of the bolt hole at the bottom of the tee (compression side of connection). Although the M-ϕ test on connection B2 was terminated because of excessive slipping and bolt bearing deformations, it is believed that this basic connection configuration will always meet requirement (c) discussed previously.

Two specimens, B2 and B4, were tested in shear. Ultimate shear strength was about 2.7 times the design shear. Connection B2 failed by shearing through the web of one tee, while connection B4 failed by tearing of the welds at the tension end of the connection.

The Type B connection is suitable for simple framing even though it may tend to be stiffer than desired, particularly when used on a thick walled tube. Pertinent design factors include:

(a) The tee flange width should be such that welding can be done along the corners of the tube. If a narrow tee is welded to the middle of a wide tube face, appreciable tube deformation might result because the connection is approaching the Type A connection in behavior. There should be no welding along the ends of the tee flanges.

(b) The width-thickness ratio of the tee flange must be reasonably high in order to reduce rigidity of the connection. The connections tested, all of which had both adequate strength and satisfactory flexibility, had flange width-to-thickness ratios of about 9. In the absence of any better information, it appears that using a width-thickness ratio of about 10 would be a proper design rule.

(c) Tee web thickness and weld dimensions are to be sized by conventional techniques. Load eccentricity was accounted for in designing welds for the test specimens.

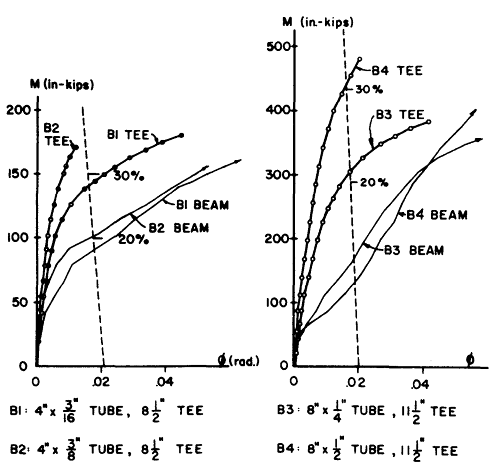

Fig. 8. M-ϕ curves for Type B connections

Type C Connection—The Type C connection (Fig. 1c) is familiar to all structural engineers—it is the welded, unstiffened, angle seat connection. Its basic behavior has been well documented in previous tests.

Seat angle connections on 6 × 6 × 3/16 in. and 6 × 6 × 1/2 in. tubes were tested and were very flexible (6.6 and 8.0 percent, respectively), as shown in Fig. 9. Ultimate load factors measured in the shear tests, in which failure was initiated by tearing of the welds along both sides of the length of angle, were 4.15 and 3.50, the highest of any connections tested.

This connection produces no undesirable effects in the tube; its behavior is essentially independent of tube size and thickness. Usual design procedures should be used for determining angle and weld dimensions. Minimum angle length should be approximately equal to the width of the connected tube wall to ensure a minimum of load transfer into the flexible tube face. Angle lengths greater than the tube width can be accommodated by welding on the back of the angle at the tube corners.

The unstiffened seat has a rather low inherent load capacity, which often leads to the modified version called the "stiffened seat" (Fig. 10a). Although no stiffened seat connections were studied, a few comments are appropriate because nearly all forms of the stiffened seat involve a vertical element fastened to the center of the connected column face. As in the Type A connection,

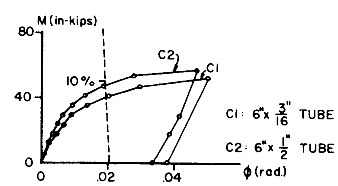

Fig. 9. M-ϕ curves for Type C connections

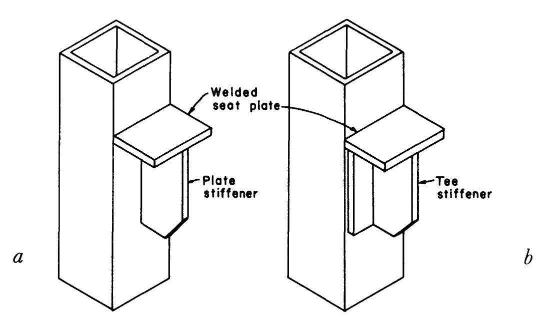

Fig. 10. Stiffened seat connections

the stiffened seat can cause serious distortion of the tube, and it is recommended that the stiffener be designed such that it also stiffens the tube wall. A length of structural tee with flange thickness not less than the tube thickness should suffice (Fig. 10b).

Type D Connection—The configuration of the Type D connection (Fig. 1d) resulted from an attempt to utilize the single plate of Type A in a manner that would reduce tube distortion to an acceptable level. Fillet welded to the tube corner at 45 degrees to the principal axes of the tube cross-section, the connection is limited to situations where the required 45 degree rotation of the tube (with respect to normal orientation of columns) would not be objectionable.

$8\frac{1}{2}$ in. deep connections on $6 \times 6 \times \frac{3}{16}$ and $6 \times 6 \times \frac{1}{2}$ in. tubes were tested, with moment-rotation characteristics as shown in Fig. 11. Both connections were quite stiff (26 and 37 percent, respectively); connection D2 did not attain sufficient rotation because excessive beam web yielding and connector slippage forced a premature stoppage of the test. No shear tests were conducted, as this configuration should certainly have shear strength not less than the ample shear capacity of the Type A connection.

Distortions and stresses were significant in the thinner tube as maximum load was approached. The thicker tube had no visible distress at any stage of loading.

No final recommendations can be formulated on the basis of the two tests described above—more tests are needed to clarify the dependence of flexibility on tube dimensions and to delineate the possible seriousness of the tube distortions produced by the connection, particularly on a thin tube. The connection certainly has some potential as a practical simple connection, even though the designer faces a dual problem—if the connected tube is thick, excessive rigidity is encountered, and if the tube is thin, distortions may become excessive.

Type E Connection—The fifth connection type investigated consisted of a single flat plate welded in position after being inserted through a pair of slots in the tube wall (Fig. 1e). Because of the continuity of the connection through the tube, local tube bending and distortion is at a minimum and can be safely ignored. The continuity is also disadvantageous, however, in that it makes the connection excessively stiff.

$8\frac{1}{2}$ in. deep connections on $6 \times 6 \times \frac{3}{16}$ in. and $6 \times 6 \times \frac{1}{2}$ in. tubes were 35.6 and 38 percent stiff (Fig. 12). Nearly all of the rotation capacity of the connection results from bearing deformations around the bolt holes in the plate and beam web. Moment testing of the thicker tube specimen was terminated before the safe beam line had been crossed because of excessive bearing deformations. A large amount of slipping occurred in both specimens as seen by the differences in connection and beam rotations in Fig. 12.

As would be expected, shear capacity of this type of connection is high. Specimen E2 ($\frac{1}{2}$ in. tube) is shown in Fig. 13 at the conclusion of the shear test. Failure occurred by tearing of the connection plate at a load equal to 3.64 times the design load. Note that the plate distorted about $1\frac{1}{4}$ in. vertically before failure—an excellent example of the ductility of steel. The tubes remained intact, with no visible yielding or distortion, throughout all stages of both moment and shear testing.

Although this connection has been used in some buildings, it is not recommended for general usage as a flexible framing connection because of excessive stiffness. If used for a one-sided connection, appreciable bending moment leading to a reduction in axial load capacity of the tube could easily result. Perhaps it could be used as a two-sided connection with beams of identical length and loading, in which case the net moment on the column would be zero. If any case, however, the Type E connection cannot even approach the flexibility requirement in Section 1.15.4 of the AISC Specification.

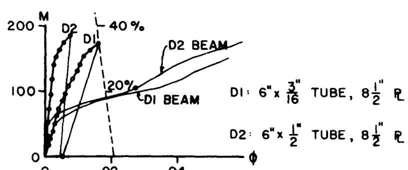

Fig. 11. M-φ curves for Type D connections

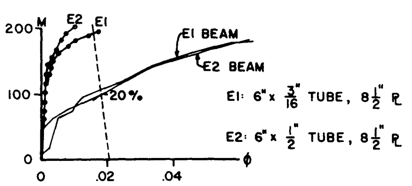

Fig. 12. M-φ curves for Type E connections

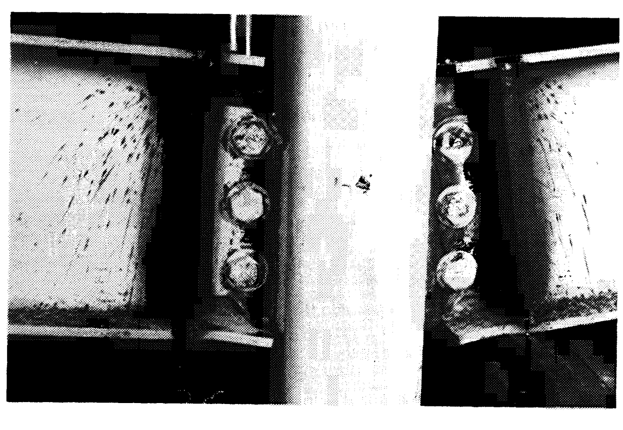

Fig. 13. Shear test, connection E3

RIGID FRAMING CONNECTIONS

Although most tubular construction has utilized simple framing methods, the availability of larger tubular sections makes the possibility of using them for rigid frame construction increasingly attractive. It may be desirable in certain cases to have rigid connections in low-rise framing made up of tubular columns and wide flange beams. Other applications of rigid connections are being made in construction of Vierendeel trusses from tubular members, and in reticulated shells and space trusses.

Requirements for rigid connections can be summarized as follows:

(a) The connection should preferably be able to carry the full plastic moment of the connected beam.
(b) Its stiffness or rigidity should be equal to or greater than that of an equivalent length of the rolled sections to be joined.
(c) If used in plastic design, the connection must be able to sustain the large inelastic rotations occurring in the members at the connections.
(d) Fabrication of the connection should be economical, and field erection procedures should not be complicated by the connection configuration.
(e) The appearance of the connection is often quite important, particularly in exposed construction.

The following discussion is limited to continuous interior beam-to-column connections. Problems similar to those outlined below also exist, however, for other types of rigid connections such as a square corner connection for a single story rigid frame.

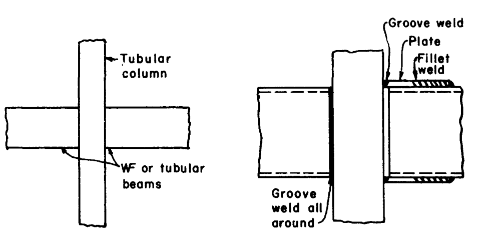

Fig. 14. Continuous interior beam-to-column connections

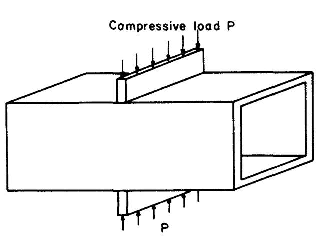

Fig. 15. Model used to predict performance of rigid connections

CONTINUOUS INTERIOR BEAM-TO-COLUMN CONNECTIONS

The framing situation shown in Fig. 14 is a typical interior connection for tubular or wide flange beams framing into tubular columns. Alternate methods of making a direct connection to the tube are shown in Fig. 14b. The left beam is welded directly to the tube with a full penetration groove weld all around, and the right beam is connected to flange plates welded to the tube. In order to meet the requirements listed above, the tubular column must be capable of carrying the flange and web forces in the beam without appreciable deformation of the tube cross section.

The behavior of a tube subjected to concentrated flange forces can be studied by utilizing a simple physical model of a portion of the total beam-column configuration. The model, shown in Fig. 15, consists of a tube loaded transversely in compression through steel plates simulating the flanges. The maximum load capacity P is an excellent indication of the maximum flange force that the section could carry, while a measurement of tube deformation (δ) in the direction of loading (P) as a function of loading enables one to compute a predicted M-ϕ curve for any particular beam. The latter construction assumes that the same P-δ curve would result if P were tensile instead of compressive, and also assumes that the tensile flange force does not influence the behavior of the tube under the compression flange, and vice versa. The first assumption would be adequate for low values of P (tube still elastic or nearly elastic), but would be grossly inaccurate for values of P near P_{max}, where inelastic buckling is governing the failure of the compressed tube.

Plots of P vs. δ for three tube sizes are given in Fig. 16. The significance of the magnitudes of P_{max} can be demonstrated by comparing P_{max} to the forces developed at full plastification of the flanges for beams of the size that would be framed into these three tubular columns; Table 3 tabulates the appropriate forces. Values of P_{max}

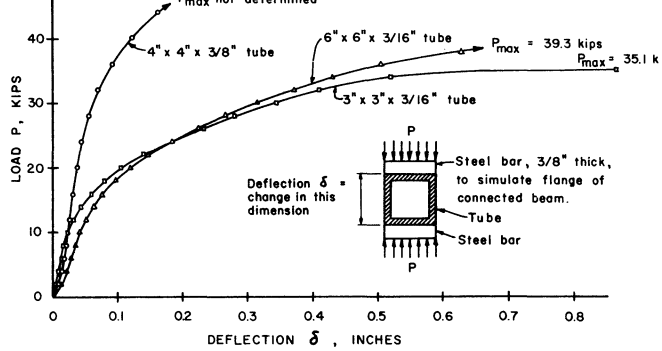

Fig. 16. Load-deflection response of model in Fig. 15.

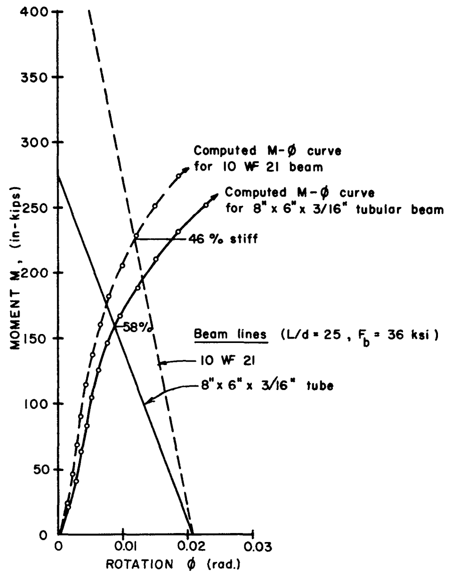

Fig. 17. M-ϕ curves for unstiffened 6 × $\frac{3}{16}$ in. tubular column connections

measured from the tests of Fig. 16 occurred at values of deflection δ of approximately 1 in. Therefore, although P_{max} exceeds the required flange force, as it does for both beams on the 3 in. tube, a plastic moment cannot be developed in the connected beam without having excessively high rotation at the beam end. Also, the force required to bring the web of the connected beam to full plastification in bending has been neglected; additional tube strength would be required to provide the resistance against this force.

Computed M-ϕ curves and beam lines for two different size beams framing into the 6 × 6 × $\frac{3}{16}$ in. tube are shown in Fig. 17. The difference in the M-ϕ curves is due to the difference in depths of the two beams, and also to the difference in percentage of total cross section area concentrated in the flanges. Both connections must be classed as semi-rigid. Connections on the smaller tubes are somewhat stiffer, but not as stiff as desired for a rigid connection.

CONCLUSIONS

The major conclusion to be drawn from the above discussion is that a tube, particularly a thin-walled tube, must be reinforced if it is to be used as a column in an interior continuous connection situation. The best method of reinforcing the connection has yet to be determined, but several alternatives seem feasible, including:

(a) Extend the flange plates of the right side connection in Fig. 14b through slots cut in the tube, as for the Type E simple connection, and weld in place.

(b) Fill the tubular column with concrete. This technique would make the tube extremely resistant to compressive flange forces, and should also increase lateral stiffness of the tube at the tensile flange because the unconnected tube walls (tube walls parallel to the connected beam web) could not move inward as the tensile load was applied.

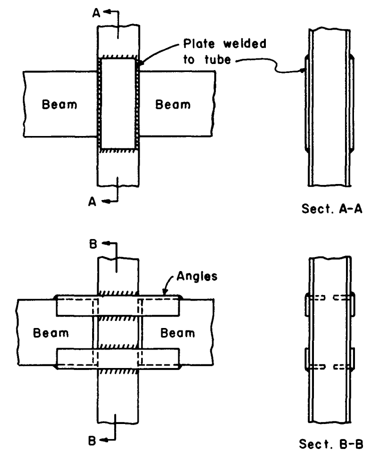

Fig. 18. Stiffened interior connections

(c) Provide external stiffening with plates welded to the critical tube walls or with angles welded to the beam flanges and coped to fit around the column (Fig. 18). The latter method seems quite attractive in that the bottom pair of angles could be shop welded to the column, thereby providing a seat for the beams in the field erection process.

In conclusion, it must be remembered that the above comments on rigid connections are based on a minimum of experimental work. It is hoped, however, that the brief discussion presented has at least pointed out some critical factors in the behavior of continuous connections, and that it will serve as a stimulant to continued thought and discussion. A comprehensive study of the problems arising in continuous connections for tubular members is being planned; if conducted, it should provide answers to many of the questions posed above.

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