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Beam-to-Column Connections

PROPOSAL FOR TESTS OF MOMENT-RESISTANT BEAM-TO-COLUMN CONNECTIONS

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Fritz Engineering Laboratory Report No. 333.12

~~(1st Draft)~~

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by

Joseph S. Huang

and

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This work has been carried out as part of an investigation sponsored jointly by the American Iron and Steel Institute and the Welding Research Council.

Department of Civil Engineering

Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania

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1. INTRODUCTION

One of the most influential elements in the cost of steel multi-story building frames is the moment resisting beam-to-column connection. Should the engineer use a bolted joint, a welded joint, or a combination of both? Selection of connections is often based upon the consideration that some particular connection assembly used is more economical and better than any other fabrication. In general, the cooperation of both the consulting engineer and the fabricators is essential in order to attain the most efficient and economical design for beam-to-column connections.

The various types of connections which have been used extensively in steel building frames and are of particular interest to designers are summarized in Ref. 1. It is an interim report prepared to indicate the basic problems pertaining to connection study and to suggest some possible areas of needed research work. Ref. 2 contains detailed evaluation of different types of connections from the viewpoint of a practicing engineer.

Previous investigations on moment resisting beam-to-column connections conducted at Cambridge University, Cornell University, and Lehigh University are summarized and discussed in Ref. 3. The types of connections studied are: fully welded connection, welded top plate and angle seat connection, bolted top plate and angle seat connection, end plate connection, and T-stub connection. In addition, the behavior of welded corner connection, bolted lap splices in beams, and end plate type beam splices was discussed. The connecting media for these specimens were welding, riveting, and bolting. Only A325 high-strength bolts were used. The most important result of these tests is that for all properly designed and detailed welded and bolted moment connections the plastic moment of the adjoining member was reached, and the connections were able to develop large plastic rotation capacity. There were no premature failures except those which could have been predicted and prevented[3].

Currently, a research program on beam-to-column connections is under way at Lehigh University under the guidance of the Welding Research Council. One of the objectives of the investigation is to study those types of beam-to-column connections which have been suggested to be of higher priority and require immediate attention.

In this proposal three types of beam-to-column connections are considered: flange-welded web-bolted connection (Problem 1.6 of Ref. 1), flange-welded connection (Problem 1.7) and bolted top and bottom moment plate connection (Problem 1.9B). One of the topics included in this study is the weld requirements for horizontal stiffeners (Problem 1.5B). This is to determine whether or not fillet welds can be used in lieu of groove welds.

2. PROPOSAL

2.1 OBJECTIVE OF RESEARCH

The objective of this study is to investigate the performance of those suggested beam-to-column connections that are under symmetrical loading condition. Primary attention is focused upon the moment capacity and deformation capacity of these connections.

Fig. 1 shows the behavior of a beam-to-column connection under symmetrical loads. If a connection is properly proportioned and the weld and bolts are properly designed, it will be able to carry the plastic moment of the beam and allow the beam to rotate inelastically through a very large angle as indicated by Curve A. Curves B, C, and D show that the performance of connection is unsatisfactory, obviously, because some detail has been underdesigned.

In this study, the connections are proportioned to meet the plastic design requirements. The results should provide the basis for further modifications in design procedures of safe, efficient, and economical beam-to-column connections.

2.2. THEORETICAL ANALYSIS

Usually the behavior of a beam-to-column connection can be judged by its moment-rotation relationship. Fig. 2 shows the intermediate steps which facilitate the explanation of the method of analysis. The axial load in the column, Fig. 2(a), will introduce a bending moment diagram as depicted in Fig. 2(b). Research on the behavior of beams under moment gradient indicates that, due to the presence of shear force which accelerates the strain-hardening in the yielded parts of the beam, the beam moment will increase beyond the plastic moment value. Utilizing the stress-strain curve from Fig. 2(c), the curvature along the beam can be obtained by the theory of strength of materials taking into consideration the property of member cross-section. The curvature diagram is shown in Fig. 2(d). Finally, the moment-rotation curve can be readily obtained by performing integration of the curvature diagram.

Two prediction curves are shown in Fig. 2(e). Curve A is predicted by using simplified plastic theory and Curve B, by considering strain-hardening effect. In plastic analysis it is assumed that the connection is able to sustain large amount of rotation near maximum load. It is the primary goal of this test program to examine the design rules of connection details which will assure the adequate performance of beam-to-column connections.

2.3 DESIGN OF SPECIMENS

The specimens are designed according to Section 2.8, entitled, "Connections," of AISC Specification[4]. The connections are proportioned to resist the moment and shear generated by the full factored load. Since the loading condition resembles gravity type loading (dead load plus live load), the load factor used is 1.7. The stresses used in proportioning welds, shear plates, and top and bottom plates are then equal to 1.7 times those given in Section 1.5 of the AISC Specification(4). For A325 and A490 high-strength bolts in bearing-type connections the design shear stresses used are equal to 1.7 times 30 ksi and 40 ksi, respectively, instead of 22 ksi and 32 ksi suggested in current Specification. The concept for this procedure will be discussed later.

1. Member Size and Beam Span

The connection specimens are chosen to have an appropriate combination of beam section and a column section which represents the real interior beam-to-column connections in a multi-story frame. Two cases are considered: one at upper level and one at lower level of a multi-story frame. Another factor considered in design is the way a wide-flange shape resists bending moment and shear force. It is well known that the flanges resist a large portion of bending moment, and the web carries almost entirely the shear force. The ratio of flange area to web area, then, indicates the degree of interaction between moment and shear resisting ability.

With regard to the effect of shear force on the full plastic moment of a wide-flange section, it has been suggested that no modification of the plastic moment M_p is required if the magnitude of shear force at the maximum load does not exceed the shear value producing full yielding of web[5]. This limitation of shear force V_p can be written as

$$V_p = \frac{\sigma_y}{\sqrt{3}} w \, d_w \qquad (1)$$

where σ_y, w, and d_w are yield-stress level, web thickness, and web depth of wide-flange shape, respectively.

The specimens are proportioned such that the section at beam-column juncture can resist M_p and V_p. Beam span, then, is simply the ratio of these two values. A W14 x 74 beam connected to a W10 x 60 column and a W27 x 94 beam connected to a W14 x 314 column are used in this test program. The ratios of one flange area to web area A_f/A_w for W14 x 74 and W27 x 94 sections are 1.39 and 0.60, respectively. The behavior of these two sections should be representative of all wide-flange sections.

The material of the specimens is ASTM A572 Grade 55 steel.

2. Fasteners and Holes

ASTM A325 and A490 bolts are used to assemble the joint. In bearing-type connections, the allowable shear stresses used in design for A325 and A490 bolts are 30 ksi and 40 ksi, respectively. These values reflect the logical design criterion which would result if an adequate factor of safety was applied against the shear strength of the fasteners. This design criterion is based upon the results of study of A7 and A440 steel lap and butt joints fastened with A325 bolts, and A440 steel joints connected with A490 bolts[6]. Tests have been subsequently carried out to substantiate the suggested design criterion, especially the use of A490 bolts in A440 and A514 steel joints[7,8].

Since both the oversize holes and slotted holes are desirable to facilitate erection adjustments, experimental justification is required for beam-to-column connections assembled with high-strength bolts with enlarged or slotted holes. Previous research has indicated that oversize holes, size according to bolt diameter, do not adversely affect the slip behavior of friction-type joints or cause undesirable bolt-tension losses[9]. It was also observed that slotted holes did not affect the strength of bearing-type joints. However, slots did decrease the slip resistance of friction-type joints by 22 to 33 per cent.

Provisions based upon these findings are now included in the Specifications for Structural Joints Using ASTM A325 or A490 Bolts as approved by the Research Council on Riveted and Bolted Structural Joints of the

Engineering Foundation (endorsed by American Institute of Steel Construction and Industrial Fasteners Institute)[10].

In the test program proposed herein, 5/16 in. oversize holes are used in top and bottom plates fastened with 1 1/4 in. diameter A490-F bolts (Test C9 shown in Fig. 11). Slotted holes are used in two types of shear connections (bolts subjected to shear): top and bottom plates bolted with A490-F bolts (C8 shown in Fig. 10), and one-sided shear plates fastened with either A325-X bolts (C8 and C9) or A490-X bolts (C3 shown in Fig. 5).

3. PROPOSED TEST PROGRAM

The proposed test program, designated as C-series, is summarized in Table 1. This series of nine tests is designed to investigate those problems suggested in Phases 10 and 11 of the research project. Detailed discussion of the test program is given as follows.

3.1 DESCRIPTION OF SPECIMENS

Fig. 3 shows the joint detail of Test C1. Beam flanges are directly welded to the column flanges providing for plastic moment capacity. A one-sided shear plate bolted with three 1 in. diameter A490-X bolts is used to resist vertical shear. Horizontal stiffeners are designed according to Section 1.15.5 of the AISC Specification(4). Since the connection panel zone is under symmetrical loads, a clearance of 1/2 in. is provided between horizontal stiffeners and column web. The size of fillet welds for horizontal stiffeners is determined by computing the force taken by stiffeners when plastic moment is attained at beam-column juncture.

Test C2 is shown in Fig. 4. Its connection type is similar to Test C1, the only difference being that horizontal stiffeners are not required. The one-sided shear plate of Test C3, shown in Fig. 5, has slotted holes. The dimensions of these slots conform to the provision in current Specification(10).

Fig. 6 shows the joint detail of Test C4. Moment capacity is provided through direct butt welding of the beam flanges to the column flanges. Vertical shear is resisted by a two-plate welded stiffener seat. The strength of this connection should be greater than that of Test C5, shown in Fig. 7.

In the case of Test C5, both the moment and shear are resisted by the butt welds. An equilibrium analysis has been made to estimate the moment and shear value. Results show that about 75 per cent of M_p and 74 per cent of V_p can be supplied by this connection. Further analytical study will be made to evaluate the performance of this type of connection.

Fig. 8 shows bolted top and bottom moment plate connection, Test C6. The plastic moment is carried by flange plates which are fastened with 1 in. diameter A490-X bolts. The design procedure follows the example given on page 4-92 of the AISC Manual of Steel Construction(4). The bracket stiffeners are designed with the aid of Table VIII of the AISC Manual(4). Design alternatives for stiffener detail can be found in Refs. 11 and 12.

Test C7 is shown in Fig. 9. The vertical shear is supplied by a one-sided shear plate connected to the beam web by three 1 in. diameter A490-X bolts. Test C6 and C7 are designed for the same amount of moment and shear, and, therefore, their behavior should be comparable.

Test C8 (see Fig. 10) and C9 (see Fig. 11) are designed as friction-type connections having slotted and oversize holes in top and bottom plates. The one-sided shear plates have slotted holes and are designed as bearing-type connections. The design for top and bottom plates of Test C8 takes into account 30 per cent decrease in slip resistance due to parallel slotted holes. The use of 5/16 in. oversize holes in Test C9 is permitted by the Specification[10].

Provisions concerning the use of oversize and slotted holes in current Specification[10] are based upon the results of tests conducted on lap joints[9]. Test results of C8 and C9 should provide justification for this provision when the location of the joint occurs where a plastic hinge forms and is subjected to substantial inelastic deformation.

The 1 1/2 in. and 1 1/4 in. high-strength bolts used in Test C8 and C9 are impractical in the field. They are used here because the use of smaller size bolts will increase joint length. A long joint is not feasible because an appropriate relationship must be maintained between joint length and beam span. In laboratory tests, the maximum beam span allowed is the laterally unsupported distance as permitted in AISC Specification[4].

The beam span for Tests C8 and C9 is 6 ft. 9 in., which is slightly less than the maximum permitted value. The beam shear force, the ratio of M_p to L, is reduced to 47.8 per cent of V_p. Five 1 in. diameter A325-X bolts are used to connect the shear plate to the beam web.

3.2 TEST SETUP

The proposed test setup is shown in Fig. 12. The axial load in the column will be applied by a 5,000,000 pound-capacity hydraulic universal testing machine. The crosshead of the testing machine is indicated. The beam will be supported by two pedestals resting on the floor. Rollers will be used to simulate simply supported end conditions. In the compression region of beam-column juncture, lateral bracing will be used to provide stability. A member resting against the testing machine column will support the lateral bracing as indicated in Fig. 12.

3.3 INSTRUMENTATION

The specimen will be instrumented with electrical resistance strain gages at locations on the beam, the column, and the horizontal stiffeners, as indicated in Fig. 13. Those strain gages mounted at Section A-A will provide information for constructing the moment diagram. The resultant axial force will be computed by the strain readings taken at Section B-B. The two strain gages mounted on horizontal stiffeners, shown in Section C-C, are used to indicate the effectiveness of the stiffeners in resisting the forces coming from beam flanges.

The absolute rotations of beam ends and beam-column juncture will be measured with level bars attached to the member webs directly inside the flanges. The locations of the level bars are shown in Fig. 13.

Finally, the over-all deflection of the connection will be monitored by a dial gage placed directly under the column.

4. SUMMARY

Steel framing costs can be substantially reduced if proper attention is given to moment-resistant beam-to-column connections. Realistic design rules for connections should consider not only strength and rigidity but also economical fabrication and ease of erection. With the advent of high-strength steels being used in tall steel buildings, there is a need to evaluate the connection design procedures, taking into account the properties of connecting media (welds and bolts) and the details of connection components (stiffeners and plates).

In this report a series of nine tests is proposed on full-size beam-to-column connections made of ASTM A572 Grade 55 steel, fastened with A325 and A490 bolts. The results of these tests will be utilized to formulate design procedures so that building construction costs might be reduced substantially.

5. ACKNOWLEDGEMENTS

The project is sponsored jointly by the American Iron and Steel Institute and the Welding Research Council. Research work is carried out under the technical advice of the WRC Task Group, of which Mr. John A. Gilligan is Chairman. (See Appendix for Task Group Roster).

This proposal was prepared under the supervision of Dr. Lynn S. Beedle, Project Director. Other staff members are Dr. George C. Driscoll, Jr., Messrs. David J. Fielding, David E. Newlin, and John E. Regec.

The proposed test program has been reviewed by Dr. John W. Fisher. The suggestions and comments of Messrs. William E. Edwards and John H. A. Struik have also been incorporated.

This manuscript was typed by Miss Phyllis Raudenbush.

The authors are thankful to all of these.

6. APPENDIX

Welding Research Council

Task Group on Beam-to-Column Connections

(AISI Project 137, Lehigh University Project 333)

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Victor V. Bertero, University of California

Omer W. Blodgett, Lincoln Electric Company

Hubert C. Crick, Mosher Steel Company

Carson F. Diefenderfer, Bethlehem Steel Corporation

Norman W. Edwards, Pittsburgh-Des Moines Steel Company

William E. Edwards, Bethlehem Steel Corporation

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Theodore R. Higgins, American Institute of Steel Construction

Ira M. Hooper, Seelye-Stevenson-Value-Knecht

Carl L. Kreidler, Bethlehem Fabricators, Inc.

Hugh A. Krentz, Canadian Institute of Steel Construction

William A. Milek, Jr., American Institute of Steel Construction

Clarkson W. Pinkham, S. B. Barnes and Associates

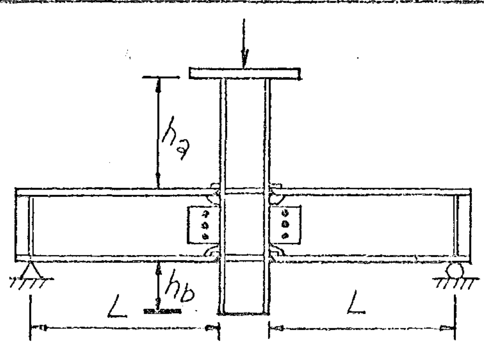
Arch N. Sherbourne, University of Waterloo

Charles F. Larson, Secretary, Welding Research Council

TABLE 1 PROPOSED TEST PROGRAM FOR C-SERIES

PHASE	TEST	COLUMN SIZE	BEAM SIZE	PROBLEM IN 333.7 (Ref. 1)	DESIGN: MOMENT	DESIGN: SHEAR	STIFFENING	DIMENSION: L (in)	DIMENSION: h_a (in)	DIMENSION: h_b (in)
10	C1	W10x60	W14x74	1.6	M_p = 6930 k-in Beam Flange Groove Weld	$V = 178^k$ (98.5% V_p) One-sided Shear Plate w/3-1"ϕ A490-X Bolts Round Holes	1.5B Horiz. Stiffener 3/4x4x8 7/8	39	30	15
	C2	W14x314	W27x94	1.6	M_p = 15290 k-in Beam Flange Groove Weld	$V = 392^k$ (99.2% V_p) One-sided Shear Plate w/7-1"ϕ A490-X Bolts Round Holes	—	39	30	27
	C3	W14x314	W27x94	1.6	Ditto	Ditto Shear Plate has Slotted Holes	—	39	30	27
	C4	W10x60	W14x74	1.7	M_p = 6930 k-in Beam Flange Groove Weld	$V = 178^k$ (98.5% V_p) Two-plate Welded Stiffener Seat	1.5B Horiz. Stiffener 3/4x4x8 7/8	39	30	15
	C5	W10x60	W14x74	1.7	Estimate M = 5220 k-in (75% M_p) Beam Flange Groove Weld	Estimate V = 134 k (74% V_p) Beam Flange Groove Weld	1.5B Horiz. Stiffener 5/8x4x8 7/8	39	30	15
11	C6	W10x60	W14x74	1.9B	M_p = 6930 k-in Top and Bottom Plates w/8-1"ϕ A490-X Bolts Round Holes	$V = 178^k$ (98.5% V_p) Stiffener Plate 3/4x9x19	1.5B Horiz. Stiffener 1x4x8 7/8	39	30	15
	C7	W10x60	W14x74	1.9B	Ditto	$V = 178^k$ (98.5% V_p) One-sided Shear Plate w/3-1"ϕ A490-X Bolts Round Holes	Ditto	39	30	15
	C8	W14x314	W27x94	1.9B	M_p = 15290 k-in Top and Bottom Plates w/14-1 1/2" ϕ A490-F Bolts Parallel Slotted Holes	$V = 189^k$ (47.8% V_p) One-sided Shear Plate w/5-1"ϕ A325-X Bolts Slotted Holes	—	81	30	27
	C9	W14x314	W27x94	1.9B	M_p = 15290 k-in Top and Bottom Plates w/14-1 1/4"ϕ A490-F Bolts 5/16" Oversize Holes	Ditto	—	81	30	27

Note:
All specimens are made of ASTM A572 Grade 55 steel.



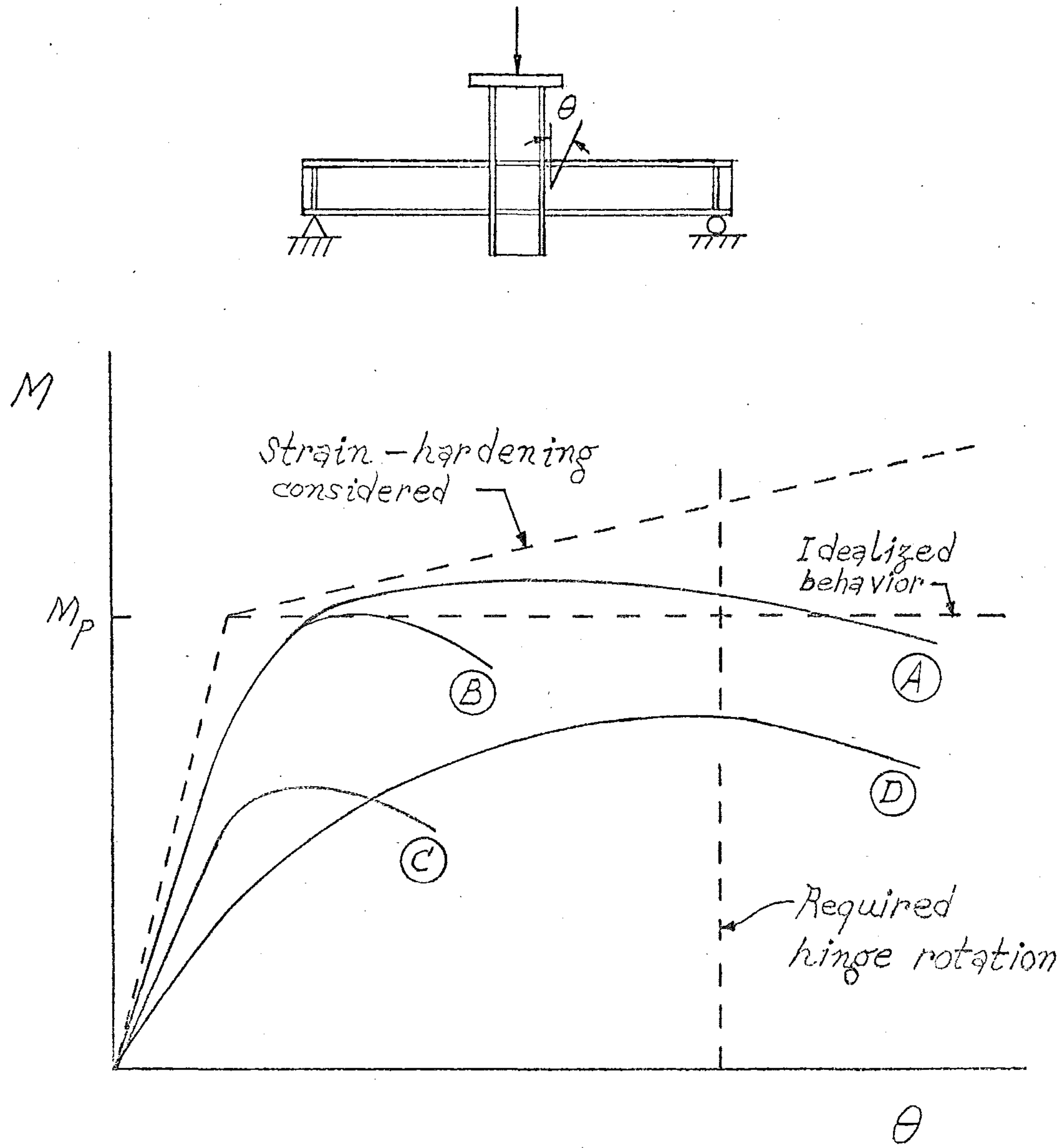


Fig. 1 Moment-Rotation Curves

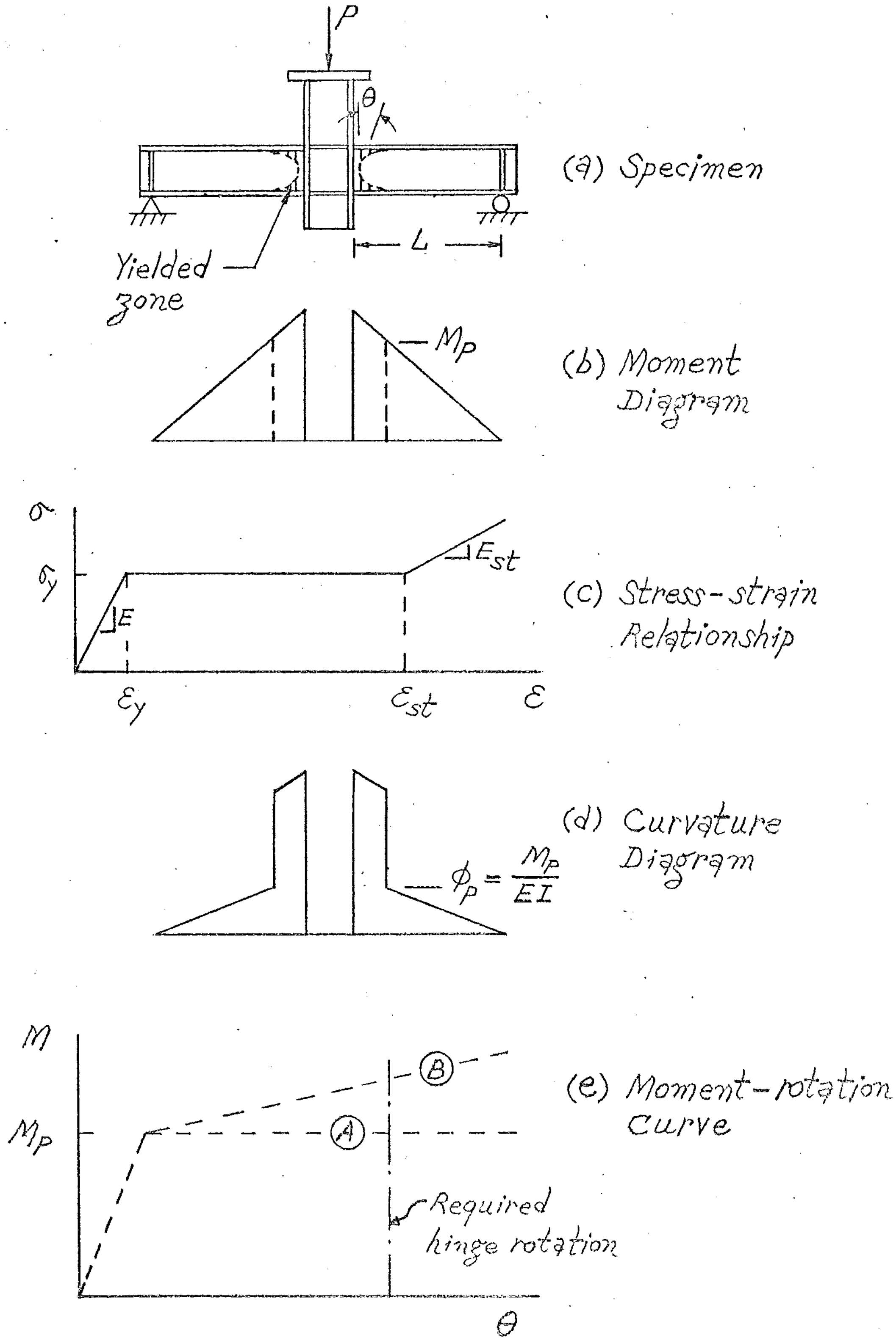


Fig. 2 Procedures of Theoretical Analysis

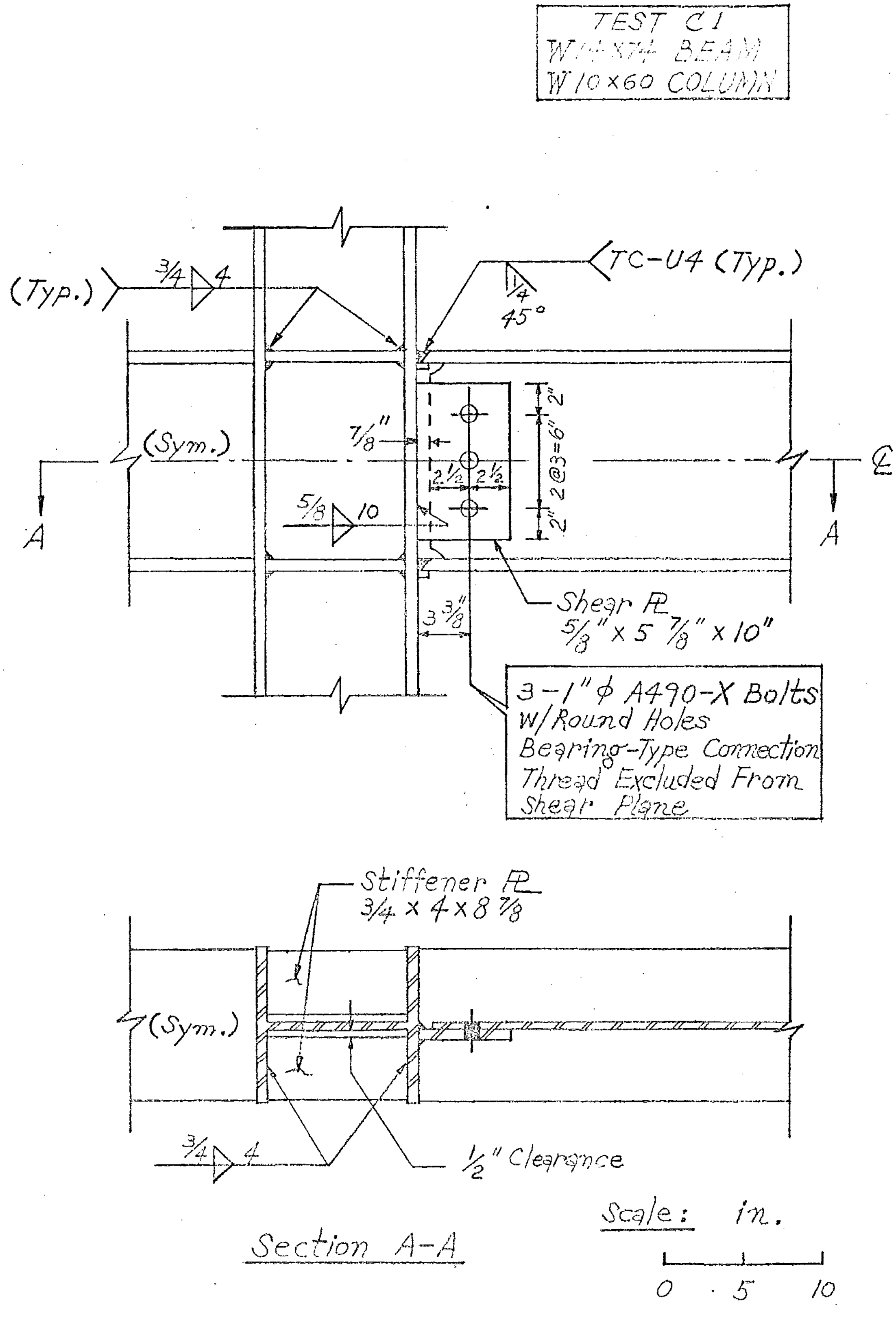

TEST C1
W14X74 BEAM
W10x60 COLUMN
(Typ.)
3/4 4
TC-U4 (Typ.)
1/4
45°
7/8"
(Sym.)
℄
A
A
2 1/2
2 1/2
2"
2 @3=6"
2"
5/8 10
Shear ℞
5/8" x 5 7/8" x 10"
3 3/8"
3-1"ϕ A490-X Bolts
W/Round Holes
Bearing-Type Connection
Thread Excluded From
Shear Plane
Stiffener ℞
3/4 x 4 x 8 7/8
(Sym.)
3/4 4
1/2" Clearance
Scale: in.
0 5 10
Section A-A

Fig. 3 Test C1

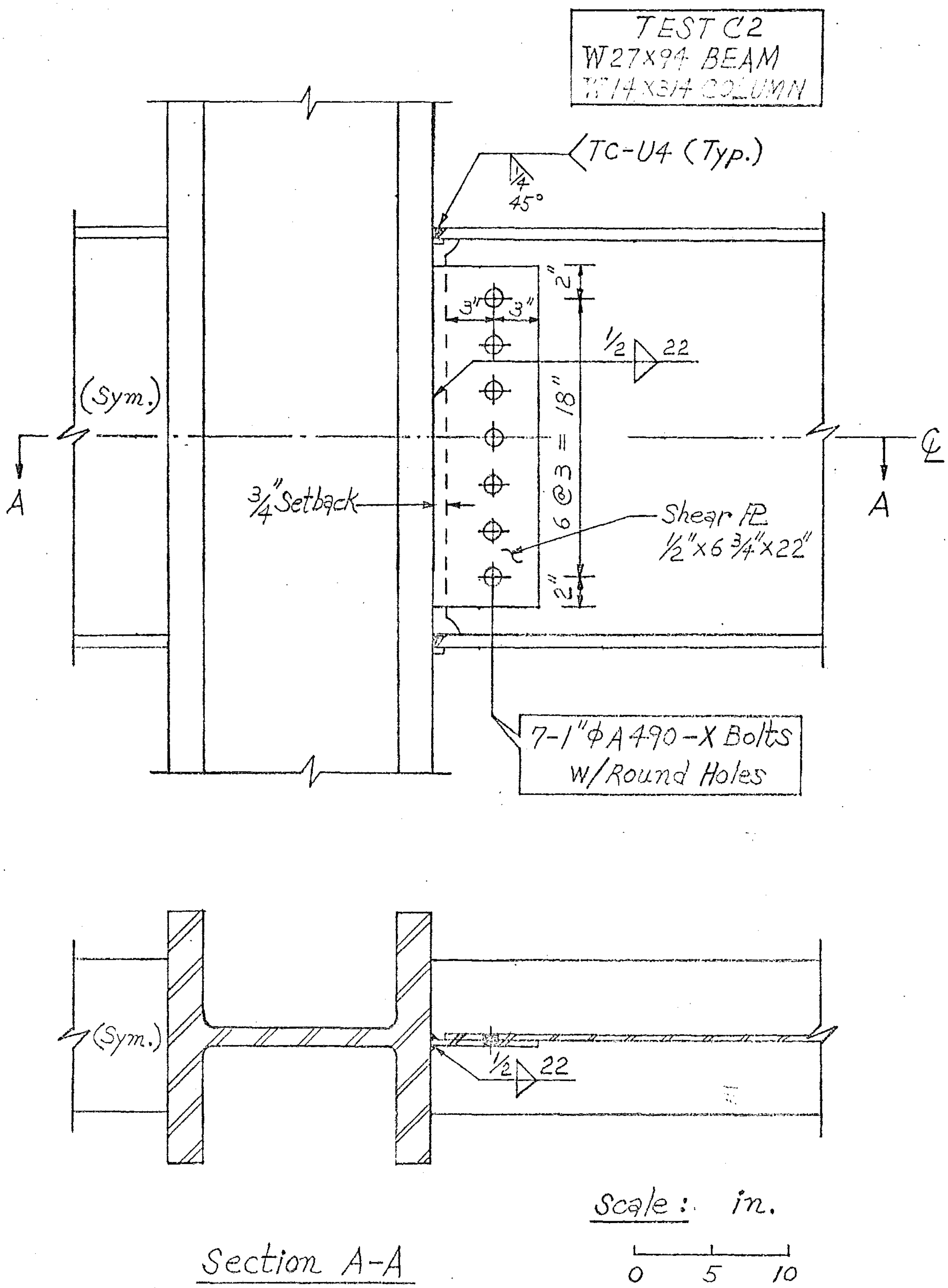

TEST C2
W27X94 BEAM
W14X314 COLUMN
TC-U4 (Typ.)
1/4
45°
3"
3"
2"
1/2
22
(Sym.)
6 @ 3 = 18"
A
A
℄
3/4" Setback
Shear PL
1/2"x6 3/4"x22"
2"
7-1"φ A490-X Bolts
W/Round Holes
(Sym.)
1/2
22
Section A-A
Scale: in.
0
5
10

Fig. 4 Test C2

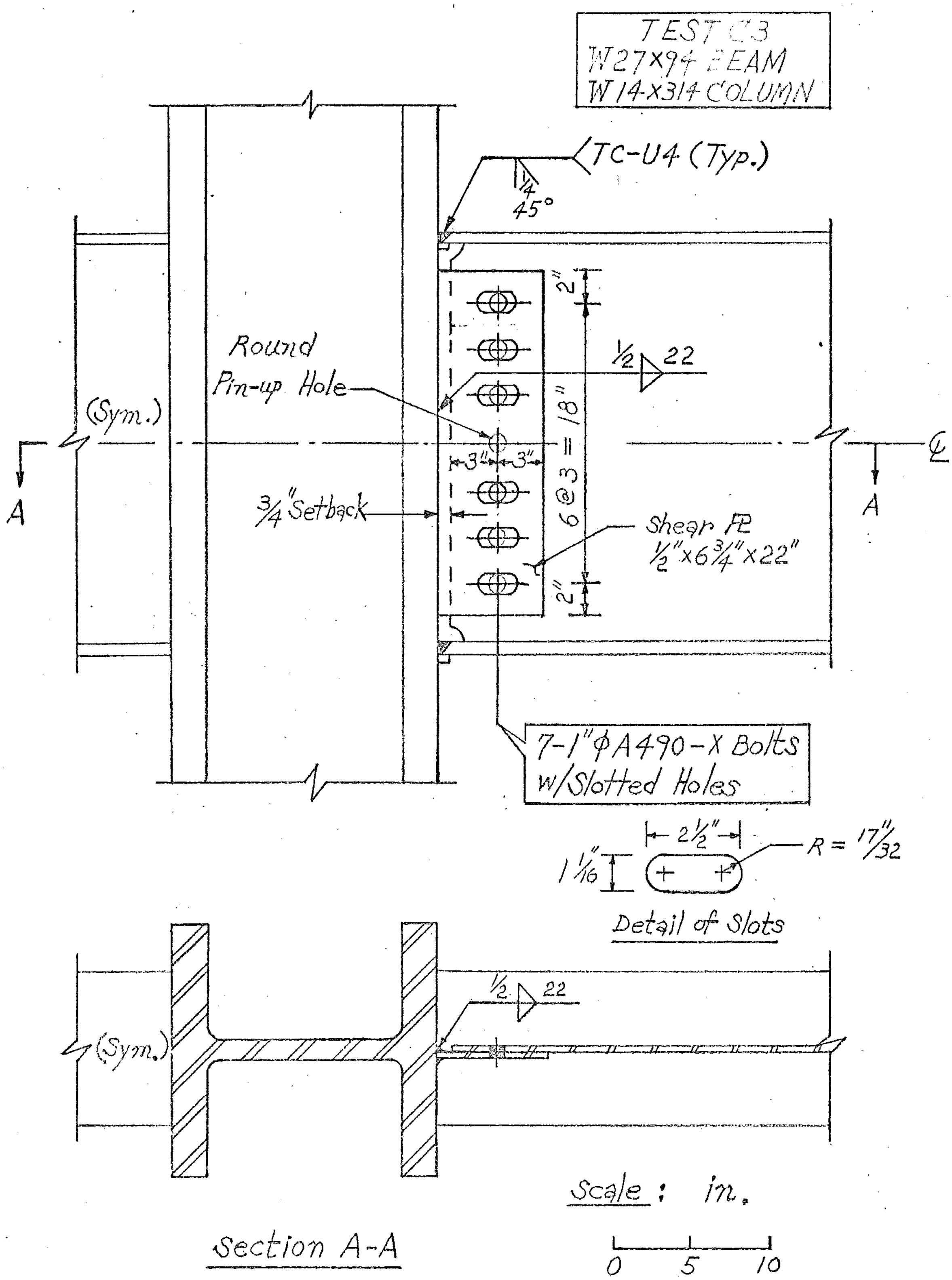

TEST C3
W27x94 BEAM
W14x314 COLUMN
TC-U4 (Typ.)
1/4
45°
2"
1/2 22
Round
Pin-up Hole
(Sym.)
6@3 = 18"
3" 3"
A
A
3/4" Setback
Shear PL
1/2"x6 3/4"x22"
2"
7-1"φ A490-X Bolts
w/Slotted Holes
2 1/2"
R = 17/32
1 1/16
Detail of Slots
1/2 22
(Sym.)
Section A-A
Scale: in.
0 5 10

Fig. 5 Test C3

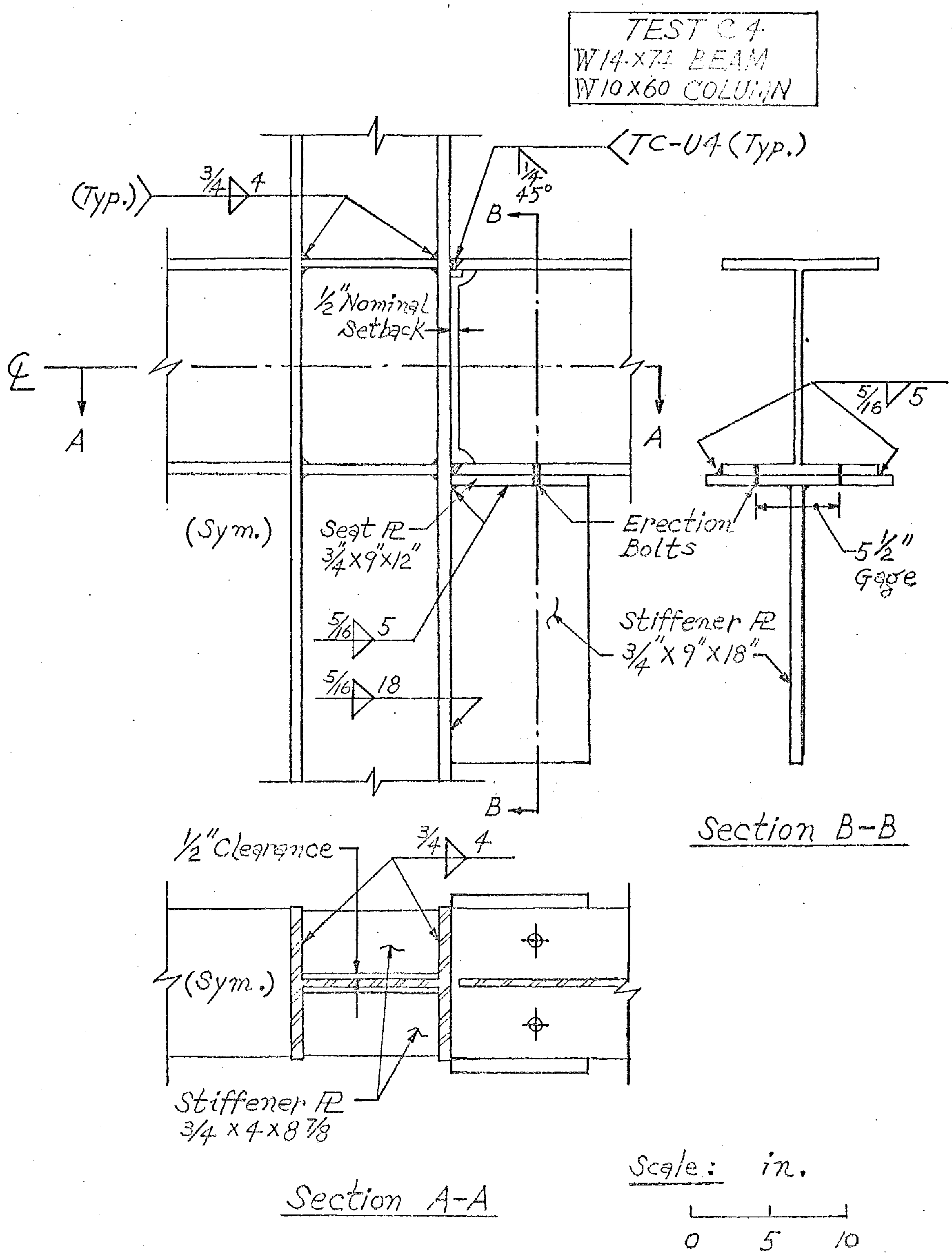

TEST C4
W14X74 BEAM
W10X60 COLUMN
TC-U4 (Typ.)
(Typ.)
3/4 4
1/4 45°
B
1/2" Nominal Setback
℄
A
A
(Sym.)
Seat PL 3/4"x9"x12"
5/16 5
5/16 18
Erection Bolts
Stiffener PL 3/4"x9"x18"
5/16 5
5 1/2" Gage
B
Section B-B
1/2" Clearance
3/4 4
(Sym.)
Stiffener PL 3/4 x4x8 7/8
Section A-A
Scale: in.
0 5 10

Fig. 6 Test C4

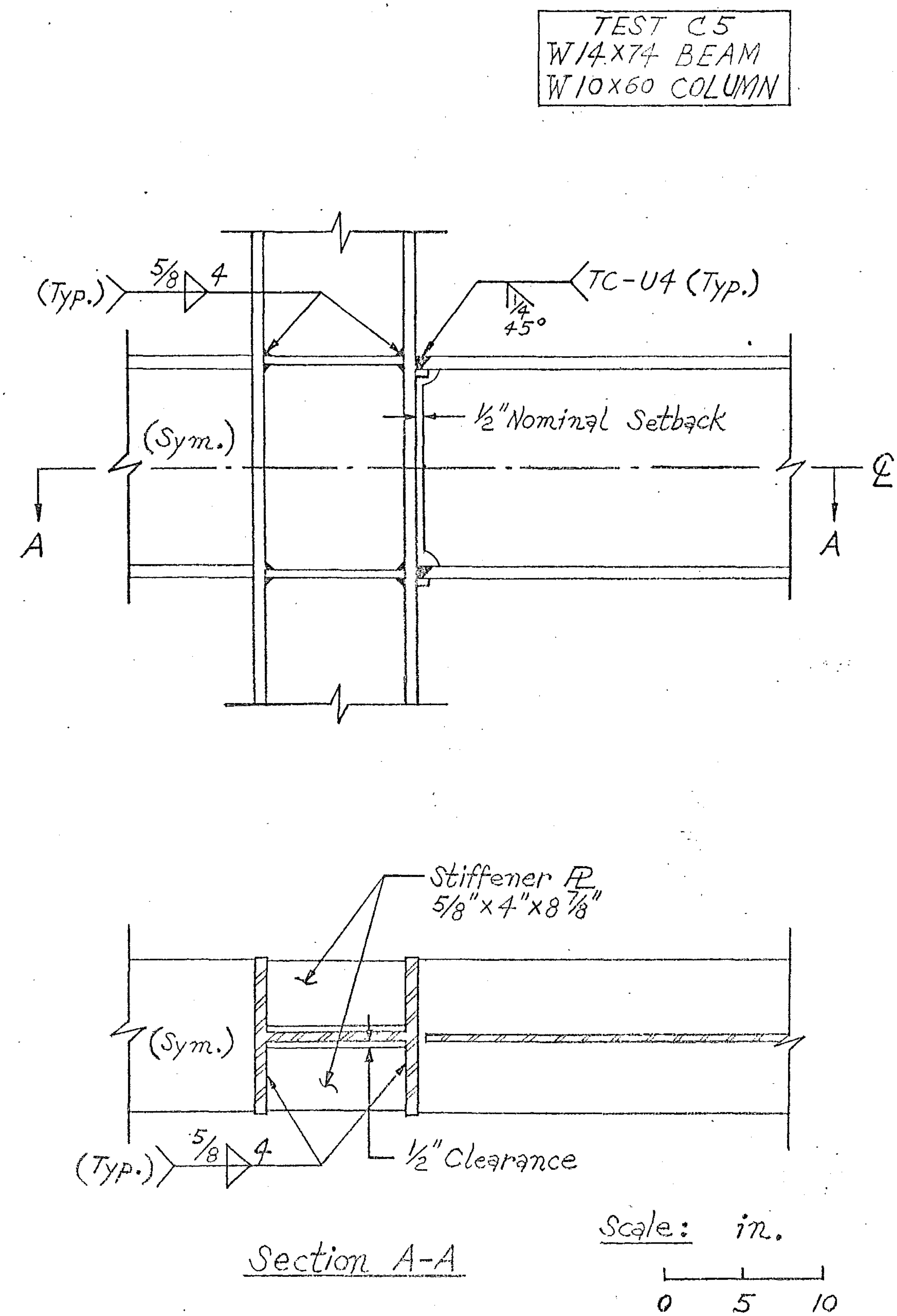

TEST C5
W14x74 BEAM
W10x60 COLUMN
(Typ.)
5/8
4
TC-U4 (Typ.)
1/4
45°
1/2" Nominal Setback
(Sym.)
A
A
℄
Stiffener PL
5/8"x4"x8 7/8"
(Sym.)
(Typ.)
5/8
4
1/2" Clearance
Section A-A
Scale: in.
0
5
10

Fig. 7 Test C5

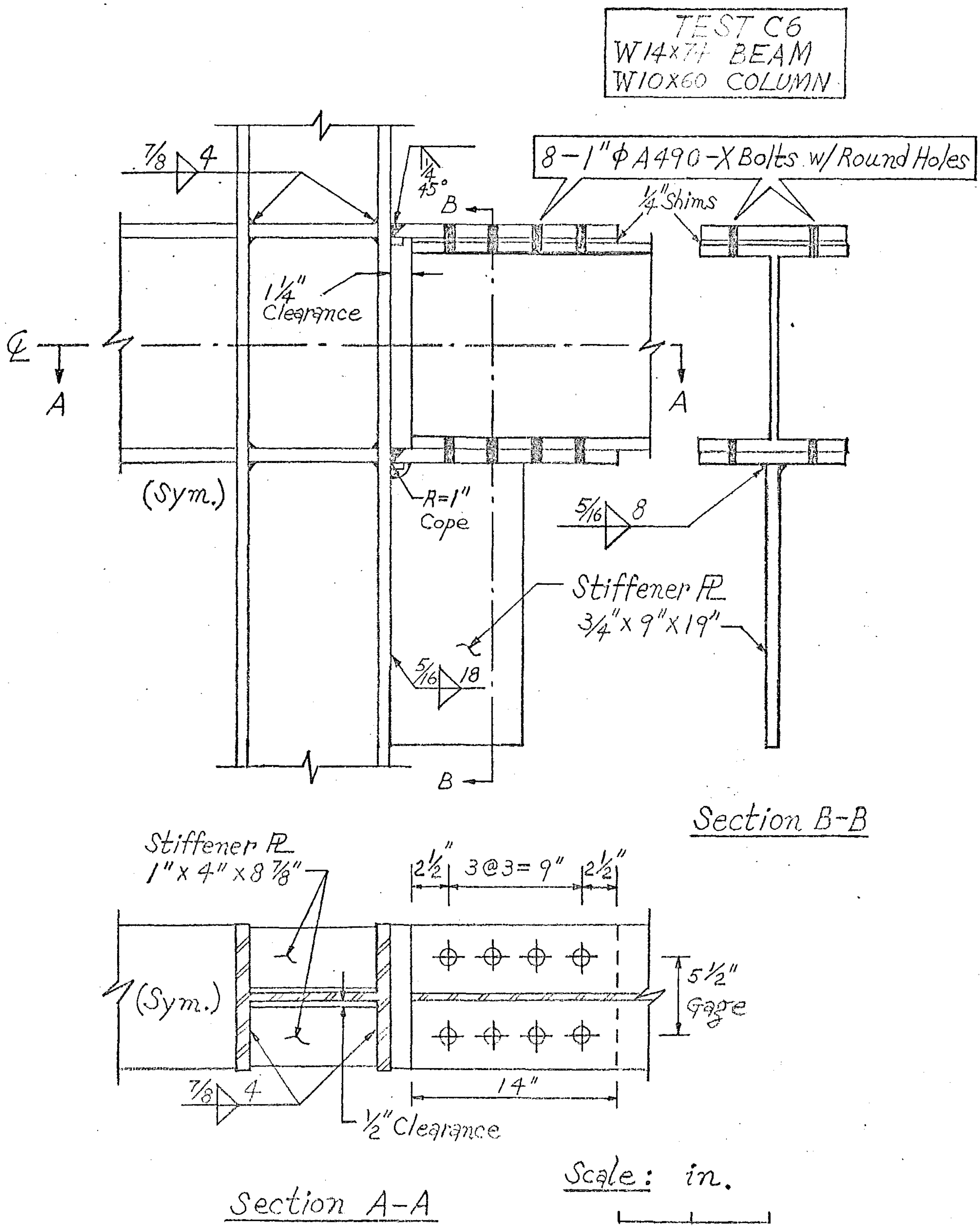

TEST C6
W14x74 BEAM
W10x60 COLUMN
8-1"φ A490-X Bolts w/Round Holes
7/8 4
1/4 45°
B
1/4" Shims
1 1/4" Clearance
℄
A
A
(Sym.)
R=1" Cope
5/16 8
Stiffener PL 3/4"x9"x19"
5/16 18
B
Section B-B
Stiffener PL 1"x4"x8 7/8"
2 1/2" 3@3=9" 2 1/2"
5 1/2" Gage
(Sym.)
14"
7/8 4
1/2" Clearance
Scale: in.
0 5 10
Section A-A

Fig. 8 Test C6

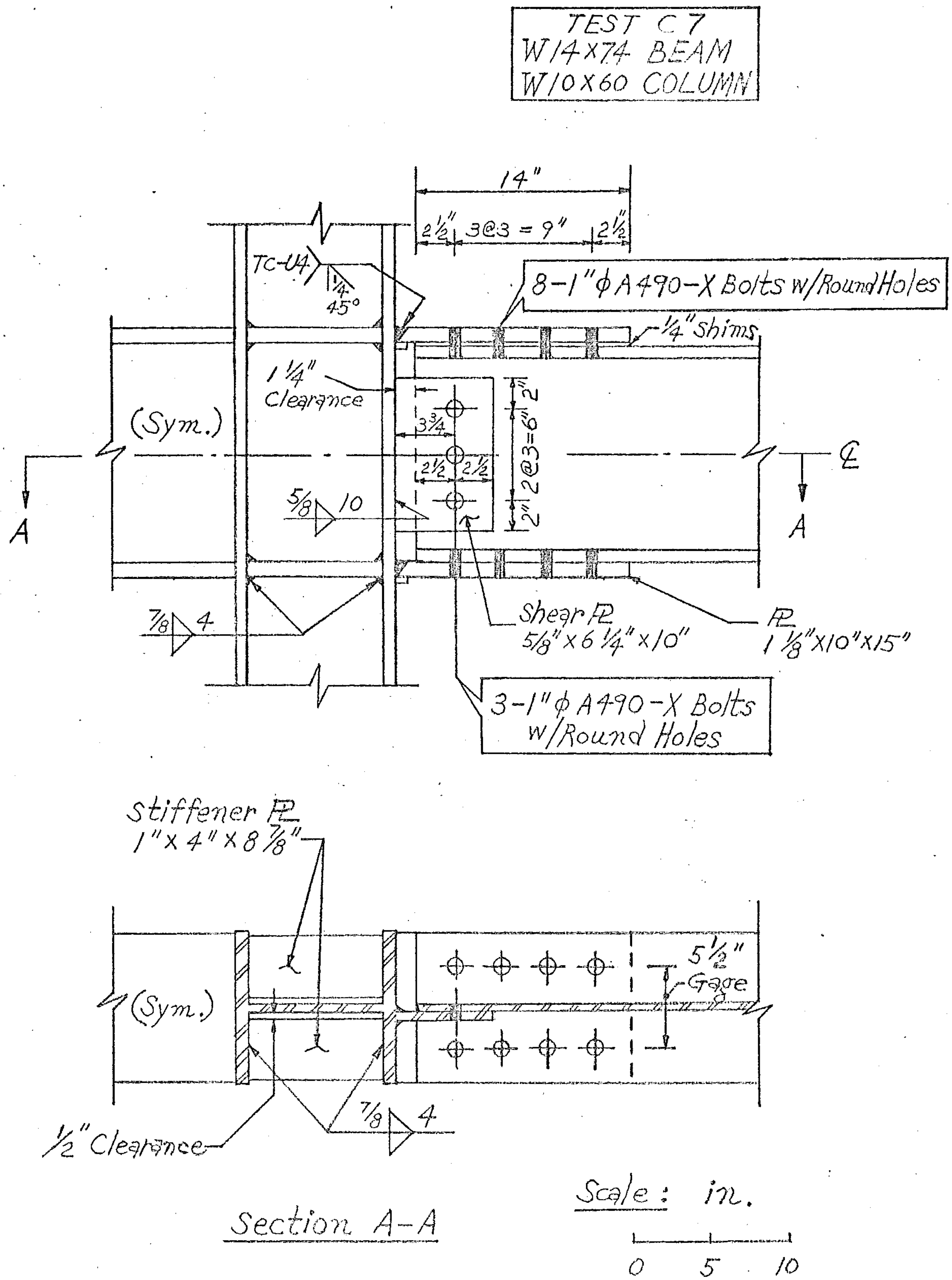

TEST C7
W14X74 BEAM
W10X60 COLUMN
14"
2½" 3@3 = 9" 2½"
8-1"φ A490-X Bolts w/Round Holes
¼" Shims
TC-U4
¼
45°
1¼" Clearance
(Sym.)
3¾
2½ 2½
2" 2@3=6" 2"
℄
A
A
⅝ 10
⅞ 4
Shear PL 5/8"x6 1/4"x10"
PL 1 1/8"x10"x15"
3-1"φ A490-X Bolts w/Round Holes
Stiffener PL 1"x4"x8 7/8"
(Sym.)
5½" Gage
⅞ 4
½" Clearance
Section A-A
Scale: in.
0 5 10

Fig. 9 Test C7

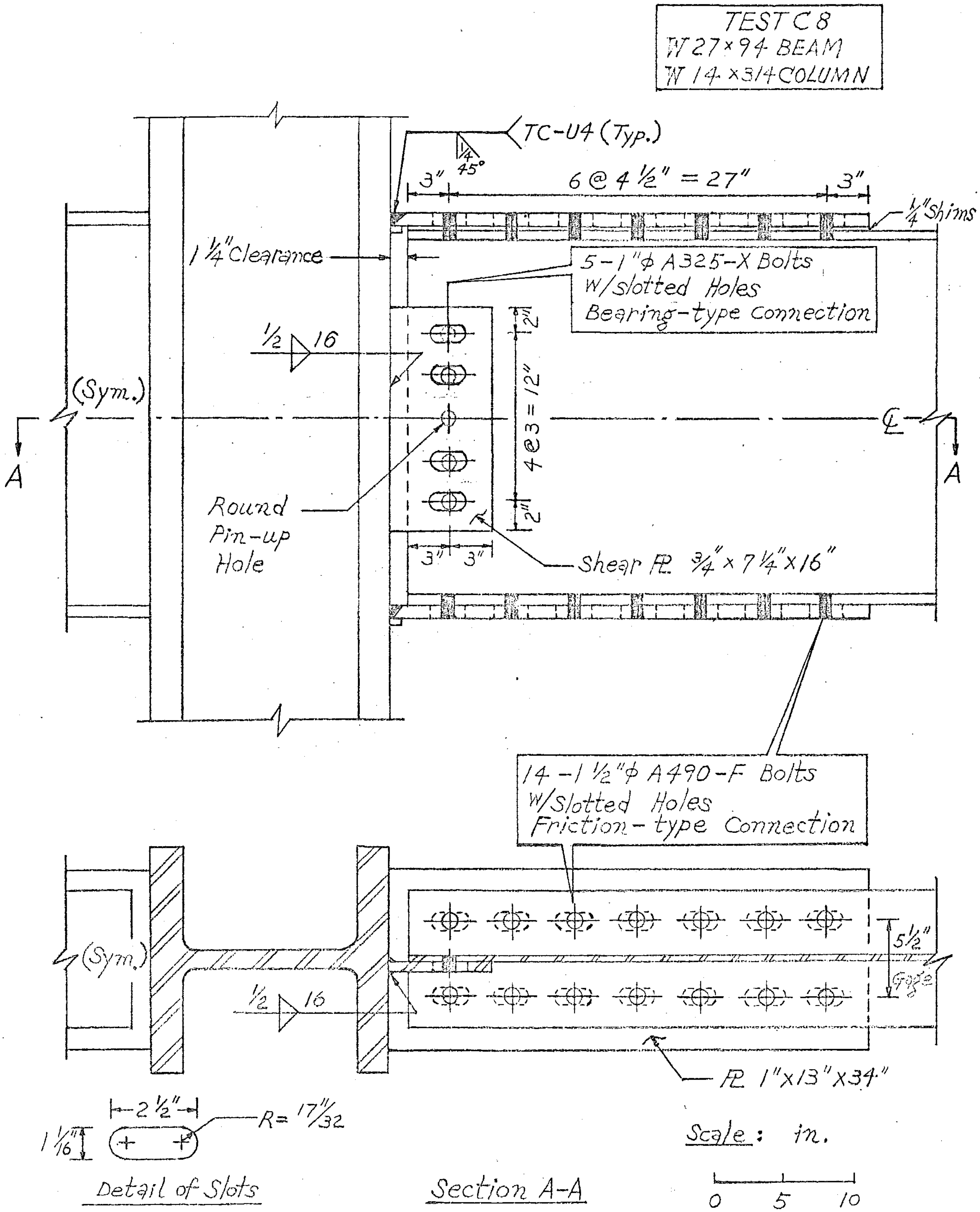


Fig. 10 Test C8

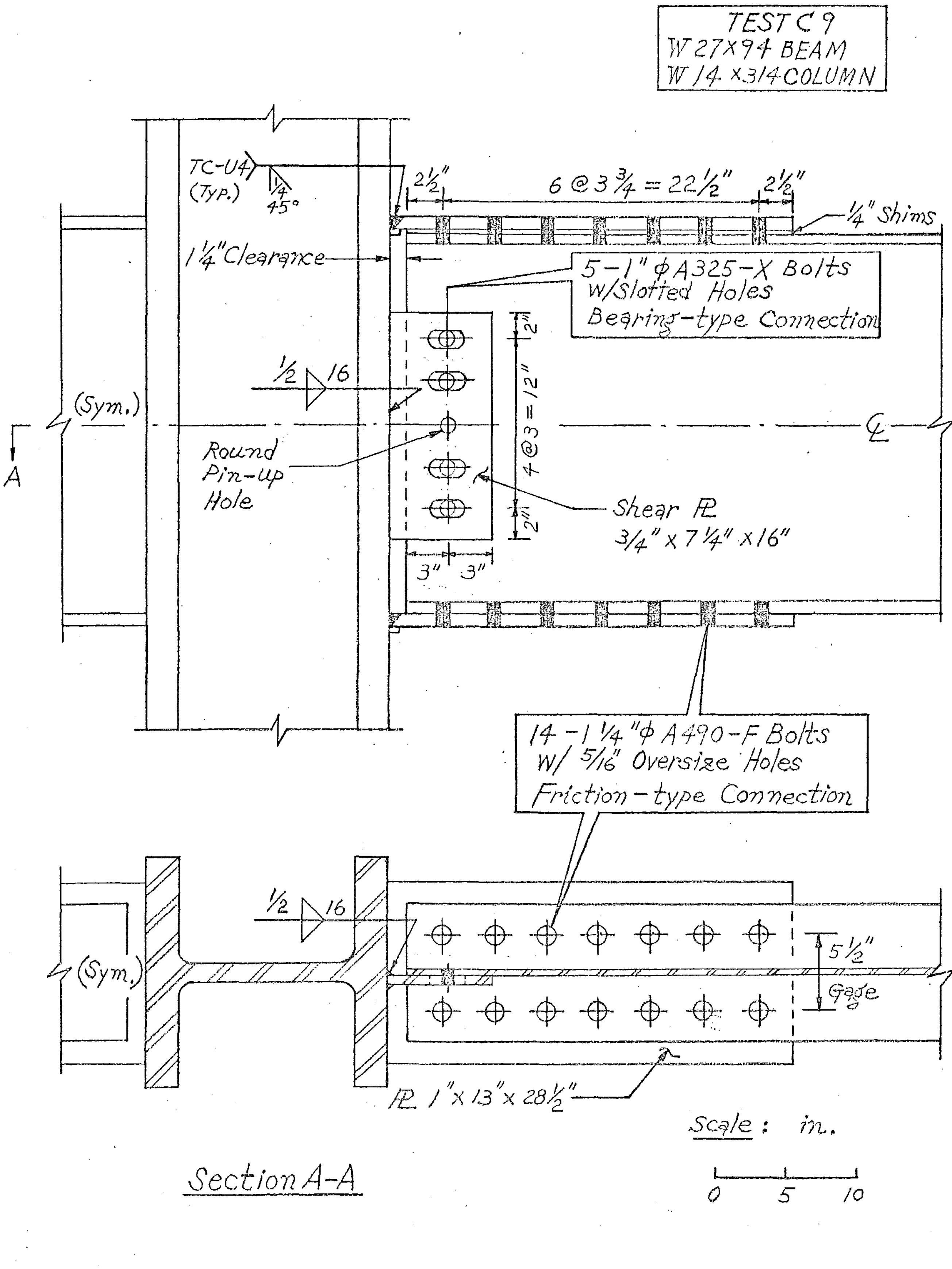


Fig. 11 Test C9

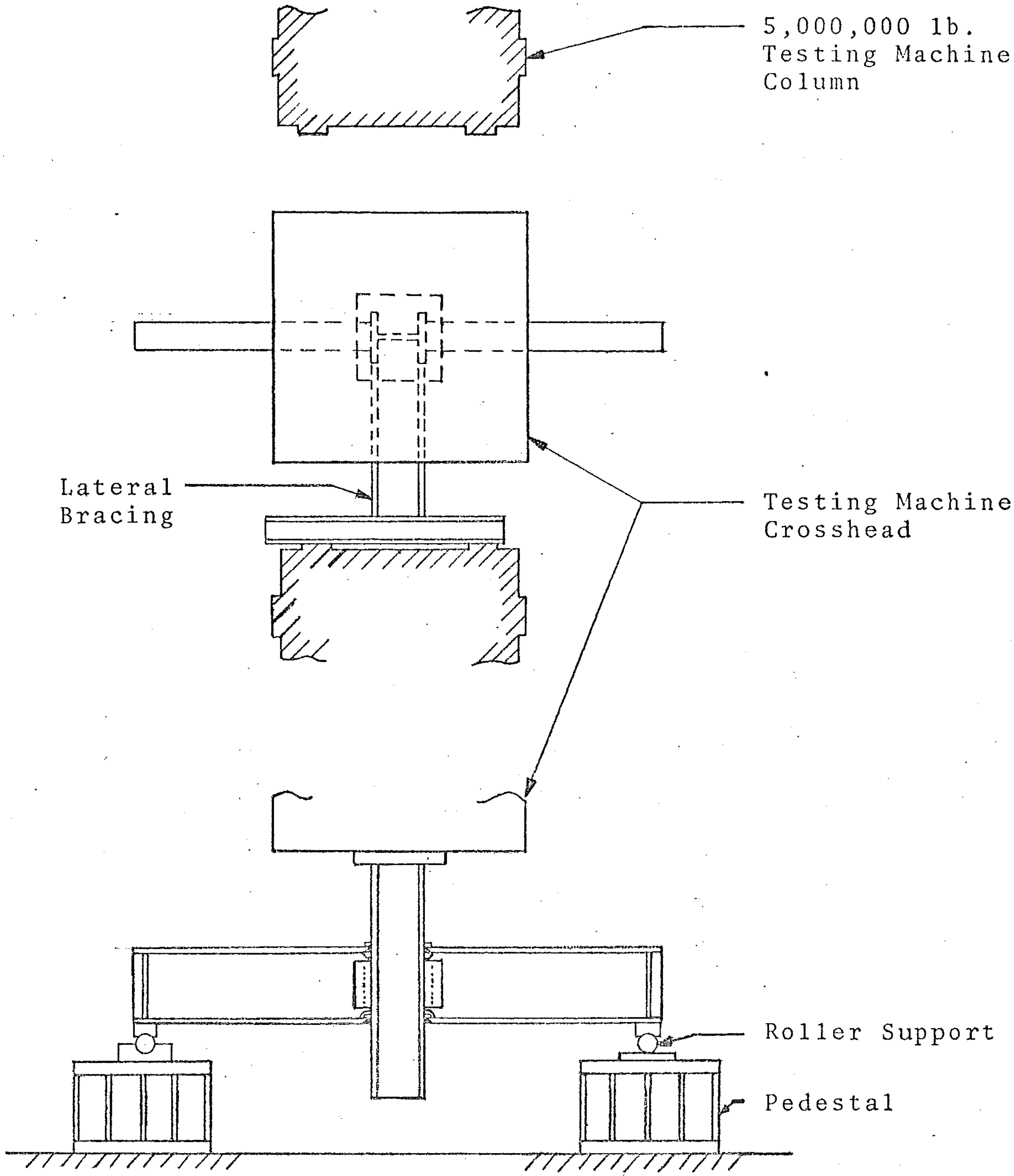
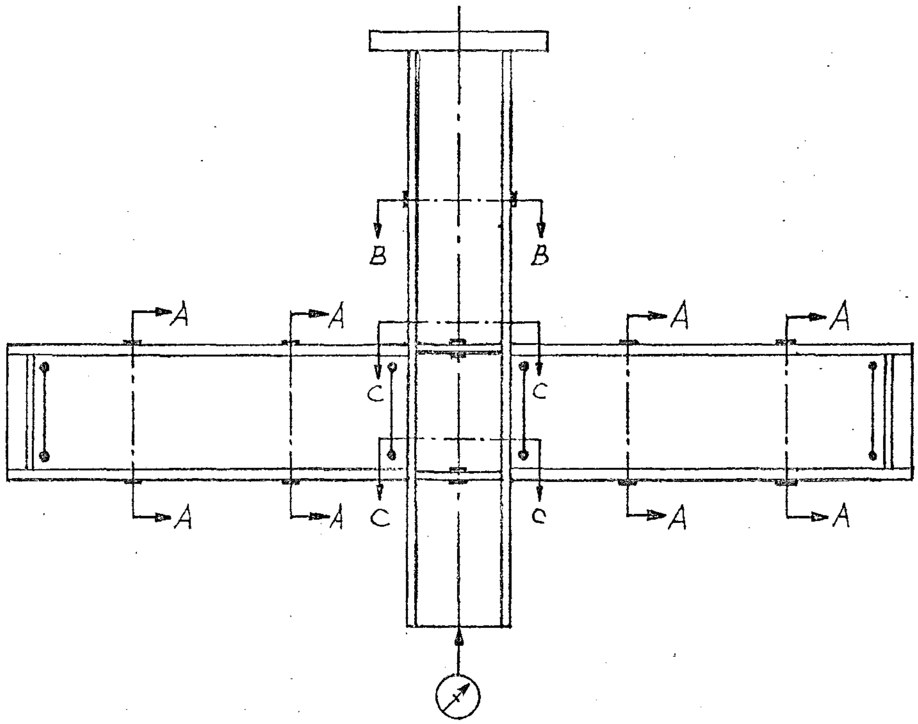
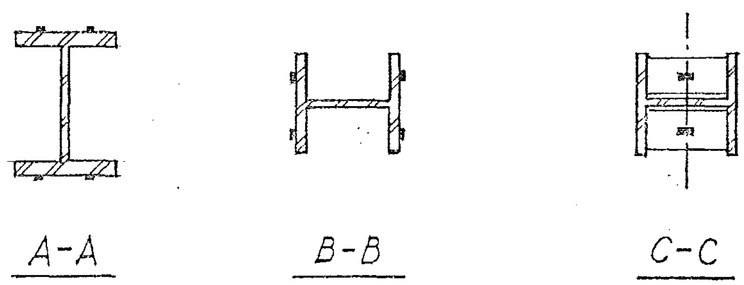


Fig. 12 Proposed Test Setup





SR-4 Strain Gages

Lever Bars for Rotation Measurement

Deflection Dial Gage

Fig. 13 Instrumentation

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