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Beam-to-column connections, proposed paper, 1955

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Revisions
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GENERAL NOTES ON ILLUSTRATIONS

Fig. 1 - Typical Welded Top Plate Beam-Columns Connection
Compromise between my front-piece illustration and the sketch recently sent to you. Include beams framing into web as well as flanges; also include basic dimensions involved - L', W, W', t- and description of the basic welds. Enlarge enough to concentrate on the two top-plates in the foreground.

Fig. 2 - Moment Diagrams
Reorientate all M*Diag. as per my paper to simulate deflection curves
a) Uniformly Loaded Beam - Elastic Zone
b) Uniformly Loaded Beam - Plastic Zone
c) Wind Moment Diagram
d) Combined Loading - two diagrams; one showing reversal in one top-plate

Fig. 3 - Top Plate Designs
Same as my Fig. 7 showing Designs A and B only

Fig. 4 - Details of the Connection Test Specimen
Same as my Fig. 18 showing all dimensions to right of ℄ and mounted gages to left of ℄

Fig. 5 - Moment-Rotation Diagram
Same as my Fig. 23

Fig. 6 - Variation in the Location of the Center of Rotation. Same as my Fig. 26

Fig. 7 - Closeup of Connection After Test
Same as my Fig. 24

Fig. 8 - Tension Test Setup
Same as my Fig. 10

Fig. 9 - Mounted Tension Specimen for Plate Design "A"
Same as my Fig. 11 (More contrast in enlarging print)

Fig. 10 - Stress-Elongation Curves for Tension Tests
Same as my Fig. 12 - revise to average stress in ksi. Don't think it is necessary to include area encompassed by Fig. 13, my report

Fig. 11 - Details of Plates at Butt Weld
Same as my Fig. 14

Fig. 12 - Details of Plates for Proposed Design Method

Fig. 13 - Moment-Rotation for Proposed Design Method

Fig. A1 - a & b Connection sketch for Rotation at Center Line and at Bottom for Derivation

Fig. A2 - Per cent R Beam Curves?

INTRODUCTION

In the field of structural design it has long been known that considerable economy can be achieved by incorporating beams which are "fixed" at the ends. However, the practicalities of shop and field fabrication and the uncertainty of various design practices have tended to curtail the use of this means of beam-column connection.

Application of welding in the structural field has brought about the development of welded beam-column connections of the type shown in Fig. 1. These are known to possess considerable merit and their application to restrained joints promises to prove economical. Analysis of these joints under the action of gravity and wind loads is one of the objects of the research program currently under way at Lehigh University.

The objects in this work are several-fold:

1. To investigate the analysis of this joint type;
2. To attempt to devise a simplified approach to the design so that the structural engineer may employ the economies involved; and,
3. To devise an economical testing procedure.

This paper will discuss the project in three parts:

1. The analysis;
2. Experiments to prove the analysis;
3. Potentialities of the connection.

ANALYSIS

There are several basic assumptions used in the following analysis, all of which are practical in nature. The first assumption is that beams frame into the column from opposite sides and that the column remains vertical under loading. Later the analysis can be expanded to include the unsymmetrical situations. As a temporary expedient, it is suggested that, in the unsymmetrical case, a flexible connection be substituted, or that the column be designed to take the moments involved.

The second assumption is that column stiffeners be used when a pair of beams frame into the column flanges instead of the column web. The researches of Brandes and Mains[1] brought out the fact that when the beams framed into the column flanges and the stiffeners were omitted, local deformations appreciably reduced the restraining moments carried by the connection. This reduction brings about two bad results: (1) the analysis no longer applies, i.e., the strength of and deformations in the connection are no longer determinate; and (2) the strength of the column is impaired due to these local flange deformations.

Previous laboratory tests indicated some doubt as to the location of the center of rotation for the connection as the load was applied. This factor raised the question as to whether the tests were indicative of field

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1. J.L. Brandes & R.M. Mains - REPORT OF TESTS OF WELDED TOP-PLATE AND SEAT BUILDING CONNECTIONS from the "Welding Journal", March 1944, pp. 146s to 165s

conditions. In the development of the analysis here, two cases were studied: (1) Rotation about the base of the beam; and (2) Rotation about mid-height of the beam. Actually, the two cases give almost the same results (variation of some 5-6%) and therefore will cause no particular problem. When testing the actual connection (described later in this report) particular attention was paid to this problem and a curve plotted to show the results.

The first logical step in the analysis is to consider the action of the top-plate in the connection under loading conditions creating elastic strains in said plate. This would represent the more conservative approach to the design of the connection since designing in the plastic range is still relatively new. Using basic applications of the methods given in any standard Mechanics of Materials texts the following formulas were obtained (derivations are given in the appendix):

<u>Case 1</u>: Rotation about Base

$$T = \frac{WL^2/24\ I}{\frac{L'}{Ad} + \frac{dL}{2I}} \quad - - - - - - - - \quad (1)$$

$$\sigma = \frac{WL^2 d}{12}\left[\frac{1}{2IL' + Ad^2L}\right] - - - - - \quad (2)$$

$$A = \frac{WL}{12d\sigma} - \frac{2IL'}{d^2L} \quad - - - - - - - \quad (2a)$$

$$\%R = \frac{100\ Ad^2L}{2IL' + Ad^2L} \quad \text{(Good for elastic strains only)} \quad - \quad (6)$$

When: T = Tension in the top plate in kips

W = Total distributed load on beam in kips

L = Length of beam span in inches

I = Moment of inertia of beam in inches^4

L' = Effective length of top plate in inches (See Fig. 1)

A = Sectional area of top plate in inches^2

d = Depth of beam in inches

σ = Unit stress in top plate in ksi

$\%R$ = Per cent restraint in terms of that afforded by a fully fixed connection

<u>Case 2:</u> Rotation about Mid-height

$$T = \frac{WL^2/24\ I}{\frac{2L'}{Ad} + \frac{dL}{2I}} \quad \text{- - - - - - - - -} \quad (3)$$

$$\sigma = \frac{WL^2 d}{12}\left[\frac{1}{4IL' + Ad^2L}\right] \quad \text{- - - - - -} \quad (4)$$

$$A = \frac{WL}{12d\sigma} - \frac{4IL'}{d^2L} \quad \text{- - - - - - -} \quad (4a)$$

$$\%R = \frac{100\ Ad^2L}{4IL' + Ad^2L} \quad \text{(Good for elastic strains only)} \quad \text{-} \quad (7)$$

The discussion of the derivation in the appendix and normal application of mechanics will bring to mind the elastic action of the top-plate.

The analysis can be extended to include the action of the plate under plastic strains. Mr. LaMotte Grover[2] has a clear word description of the assumed action in this case. Basically, as the gravity load W increases, the

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2. LaMotte Grover - MANUAL OF DESIGN FOR ARC WELDED STEEL STRUCTURES, pp. 55-56, Air Reduction Co., New York 1949

stress in top-plate increases up to the yield point stress, σ_y, and, thereafter, remains stationary if the customary simplified straight line yield zone is applied. Note that the tension in the top-plate at this point is given by $T_y = A\sigma_y$, and that the moment carried by the connection may be expressed in terms of the plate tension and beam depth by $M = Td$. This situation may be better visualized by considering a moment diagram for a typical loading system; then the principles can be expanded to any loading. Fig. 2a shows the normal moment diagram (solid line) for a uniformly loaded beam (total distributed load W and span L) with fully fixed ends. Since the actual top-plate connection will not provide full restraint (the per cent end restraint being dependent on the plate design) the end moment will be something less than $WL/12$ as shown by the dotted curve. Now consider Fig. 2b where we may suppose the load to be increased first until the top-plate just reaches the yield point (the dotted moment diagram) after which the connection may take no additional moment; and, second, until the midspan moment also reaches the yield point (solid line) at which time the beam has reached its capacity. Since, for any given uniformly distributed load the total height of the moment curve will be $WL/8$, the moment at mid-span, when the connection is in the plastic zone, will be $WL/8 - T_yd$, the connection moment remaining constant at T_yd.

The case of wind loads in combination with vertical loading can be approached in somewhat the same manner. Fig. 2c shows a typical wind moment diagram while Fig. 2d combines same with the gravity load (solid line). The top plate will only effectively carry moment until it reaches the plastic zone, $M = T_y d$, as before. Any remaining additional wind moment will be transferred to the opposite end of the beam as indicated in the Figure, where it will reduce the gravity moment and possibly reverse it as indicated by the dashed curve. Reversing the moment will put the top plate in compression.

Theoretically, the wind moment may increase until a second "plastic hinge" (the left hand top plate in the Figure acts as the first one) is created. This may occur in one of two ways. First, when the midspan moment reaches the yield point as mentioned before, or, second, when the right hand top plate fails to act as a strut in carrying the compressive load set up by the moment reversal.

This is a purposely brief discussion of basic analysis. Further details will be brought out in the Joint Potentialities and Appendix sections of this report.

EXPERIMENTS

A testing program was conducted by the junior author with several objectives:

1. to confirm the method of analysis previously put forward in this report;

2. to check a proposed design procedure;

3. to attempt to devise an economical test to predict the joint action without the cost of testing actual built-up connections.

It should be noted here that these tests are the beginning of a test program and more will have to be run to give conclusive results.

The first problem was to set up a typical design procedure for the connection - particularly the top-plate. A procedure similar to that discussed under POTENTIALITIES OF THE CONNECTION, Proposed Design Method I, was used as a starting point.

The following typical problem was used as a basis of design:

15' span and 50^k total distributed load

20 ksi allowable working stress in steel and full penetration butt welds

13.6 ksi allowable shear in steel and on throat of fillet welds (1200 lb/in. per 1/8-in. weld size)

75% end-restraint for connection design and 50% end-restraint for beam design

Beam Design:

$$m = WL/8 - 0.5\ WL/12 - WL/12 = 62.5 \text{ k-ft.}$$

$$I/c = 62.5 \times 12/20 = 37.5 \text{ in}^3$$ Use 14WF30, $I/c = 41.8 \text{ in}^3$

Plate Design:

$$M = 0.75\ WL/12 = WL/16 = 562 \text{ k-in.}$$

$$T = M/d = 562/14 = 40.2 \text{ K}$$

Note: The actual per cent end-restraint is anticipated to approach 90 per cent based on previous research

Required Plate cross-sectional area at 75% end-restraint:

$40.2/20 = 2.01$ in^2 Use 5 -1/2 x 3/8 plate (2.06 in^2)
Actual stress at working load assuming 90% restraint = 20 x 90/75 = 24 ksi

Welds cannot be overstressed:

Butt welds: widen plate end: 5.5 x 24/20 = 6.6
Use 6-3/4 in. end width

Fillet welds: 3/8 plate, hence use 5/16 weld (3000 lb/in.)
40.2 x 90/75 ÷ 3.0 = 16.1" Use 16-1/2"

Fig. 3 shows the details of two plates chosen to meet the foregoing design factors. Various suggested proportions are indicated in the Figure and are based on previous research results 1, 2. Particular note should be made of some items. A curved transition from the widened end to the basic plate width is used to reduce stress concentrations. In Plate Design A, it was anticipated that considerable stress concentration might be created at the ends of the fillet welds. To possibly eliminate this problem which might give bad results, Plate Design B was created for comparison. A third design, not shown, might be devised using a slot in the middle of a rectangular plate to give the same results.

Seat Design

A standard design procedure adapted from Mr. Grover's book[2] was used to design the seat. Since "T" seats usually interfere with the interior wall line, a seat angle was used. In a moment connection some means must be provided to carry the thrust of the bottom flange (opposite in direction and equal in magnitude to the tension in the top-plate). This was done by specifying a square butt weld between the end of the beam flange and the column.

The results of the connection design are shown in Fig. 4 showing the details of a connection test specimen using Plate Design A. In way of general comments it will be noted that all welds are economically positioned - flat or horizontal shop fillet welds for the seat angle and flat fillet and butt field welds made by a man sitting on the beam. Punching erection bolt holes in the beam flanges is the only work required on the beam. (There are several erection devices now in use which would even eliminate this). The plates are designed to be fabricated from standard plate or bar stock by automatic machine flame cutting, an economical process. One suggestion is that the plates be cut with their center lines parallel to the direction of rolling of the stock plate or bar.

One of the best methods for visualizing the analysis and action of a moment connection is a Moment-Rotation Curve. Using the formulas 6 & 7 presented in this report, the per cent end-restraint (%R) for the above design may be calculated. If the analysis is correct, results of the tests of the connection should coincide closely. Using formula 6, assuming rotation about the bottom of the beam, with the following values for Plate Design A ($A = 2.06$ in^2, $L = 15' \times 12 = 180$ in., $L' = 7$ in., $d = 14$ in., and $I = 289.6$ in^4 for a 14WF30) the %R = 94.8%. In a similar manner, assuming rotation about the mid-height of the beam and using formula 7, the %R = 90.0%. For construction of a Moment-Rotation Curve the predicted "Plastic Moment", $M_y = \sigma_y Ad$ is required. From standard ASTM E8-46, 8-in. gage length coupon tests on samples cut from stock plate adjacent and parallel to Plate Design A specimens, $\sigma_y = 37.5$ ksi and, therefore, $M_y = 90$ k-ft. The "Beam Lines" for design requirements and two times design requirements may be figured in the usual way.

The Moment-Rotation curve predicted from the foregoing analysis is shown in Fig. 5 as dashed curve. The M-θ curve gives a visual interpretation of the connection action. One criterion to observe in this case is where the predicted curve intersects the 50% end-restraint line, the latter being the basis of our beam design. The curves shown indicate a fairly conservative design.

It should be noted here that some confusion has arisen as to the use of θ and ϕ to stand for the connection rotation. ϕ is more commonly associated with discussion of beam properties such as M/EI. Since it is felt that a distinction should be made between the two uses, θ, expressed in radians, will be used in this work to stand for end-rotation of the beam with respect to the column.

The built-up connection test specimen was fabricated by a fitter and a welder, both having considerable experience. All assembly of welding was done under conditions simulating field or shop work as closely as possible. A 5/32-in. E 6012 electrode was used for the first pass of the butt weld and all remaining welding was done with E 6020 electrodes. Straight polarity (the electrode positive) was used throughout. The butt weld was specified to have a 3/16-in. to 1/4-in. root gap with a 45° pointed bevel on the plate end. This was done to achieve a full penetration weld.

Fig. 4 shows the location of all gages and rotation bars on the specimen. The specimen was placed in the 800,000-lb. testing machine at Lehigh University in an upside-down position to facilitate loading (see Fig. 4). The test was run at as slow a speed as practical, and, throughout the elastic range, readings were taken at predetermined load increments. In the plastic range, readings were taken at predetermined strain increments as indicated by the elongation dial gages mounted on the top plates. Running plots were made of elongation in top plate vs. the load and vs. the actual moment in the connection which was indicated by the Huggenberger gages. These curves gave very close agreement with calculated values determined before the tests.

Some question has been raised in the past as to which rotation bars would actually indicate the connection rotation. Referring to Fig. 4, it might be logical to assume that the connection rotation is the sum of rotations indicated as No. 1 and No. 3 minus one-half of any rotation which may occur between the column flanges, No. 0. The latter proved to be negligible throughout the test which indicates that adequate stiffeners were provided. Rotation No. 3 will be the rotation of the beam itself plus possible rotation in the connection. The beam rotation may be computed for any loading by $\theta_b = \phi(l) = M\,l/EI$, where θ_b is the rotation in radians, ϕ is the unit rotation throughout the area enclosed by the rotation bars (No. 3 in this case), l is the distance between the mountings (14"), M is the average moment, and EI is a property of the beam. Thus the rotation due to the connection will be that indicated at No. 3 minus the rotation normally expected in the beam itself if it were fully fixed at the end, θ_b, as indicated above. These calculations were run for many points throughout the test range and the following interesting fact was noted. In all cases the rotation indicated at No. 3 was equal to that calculated for the beam itself and, therefore, the entire connection rotation was indicated at No. 1. In effect this means that the connection was as strong as the beam itself in this respect. However, less conservative top plate designs might not follow this pattern.

The M-θ curve resulting from this test is shown in Fig. 5 as a solid line curve. Notice should be taken of its close correlation with the predicted curves.

As mentioned previously, a close check was made on the actual position of the center of rotation and it was found that it did not remain constant. Fig. 6 is a plot of the per cent of beam depth from the bottom flange to the center of rotation vs. θ, the connection rotation. During the initial rotation the center moves upward from the bottom flange as it begins to deform, sharing more of the load with the top plate at each increment of loading. However, once yielding in the top plate starts, its elongation far exceeds the contraction possible in the bottom flange and the center moves downward again.

Fig. 7 shows a closeup view of the joint after completion of the test. The yielding indicated at the bottom flange did not occur until late in the test and did not influence the test results in the zone of usefullness for the connection.

Several conclusions may be drawn from the results of this test: (1) the connection appears more than adequate to meet the design requirements; (2) stiffeners selected as comparable in size to the beam flange are sufficient to eliminate deformations in the column; (3) the seat designed as shown proved to be entirely adequate and may be said to be somewhat too stiff since the beam flange was caused to bend over it in the later stages of the test; (4) the connection rotation for this particular design seems to be indicated entirely by the rotation indicated between the column flange and the end of the beam; and, (5) predicted end restraint by either of the two proposed formulas (rotation about mid-height and about bottom) is in close agreement with results.

One of the objectives of the testing program was to devise a more economical test to predict that joint action without the cost of testing the actual built-up connection. The need for this, economy wise, is clearly evident. Also a multitude of tests may be required for the various types of connections which will evolve from this program.

Two criteria were deemed necessary in setting up a tension test of the top plate to predict the joint capacity:

1. The tension test specimen (top plate) must be mounted in such a way as to duplicate the situation in the actual connection as closely as possible;
2. The testing "rig" should be designed in such a way as to permit repeated usage in future tension tests.

With these criteria in mind and a glance back at Fig. 4, the problems encountered may be visualized. One of the main concerns was to duplicate the eccentric loading on the plate caused by the fillet welding. It was decided to use two duplicate plates in each test assembly which would permit close duplication of the actual conditions. If care is taken in the fabrication and assembly, the two plates should share equally in carrying the test load and a plot of the average load vs. the average elongation would closely approximate a similar curve for each plate individually. The final design of the test setup is shown in Fig. 8 and Fig. 9 shows Plate Design "A" mounted in the 300,000-lb. hydraulic testing machine in Fritz Engineering Laboratory at Lehigh University.

Standard gage points were placed along the center line of the plate at 2-in. intervals starting at a point 1/2-in. from the butt welded end of the plate. A 2-in. Berry gage was used to determine the relative strains at various points along the plate, and a 10-in. Whittemore gage was used on the first 10 in. of the plate. When the Whittemore gage went out of range, a direct reading 10-in. dial gage was substituted. The dial gage shown in Fig. 9 was used to estimate strain increments when in the plastic zone, determining when readings of the strain gages should be taken.

The welders were instructed to do all welding of the plate to the mounting units just as they would do in the field. As with the built-up connection, a 5/32-in. E 6012 electrode was used for the first pass of the butt weld, and the remaining welding was done with E 6020 electrodes. Straight polarity (electrode positive) was used throughout.

It was anticipated that the specimens would follow the curve based on the results of coupon tests rather closely. However, plate design "A" would probably vary from the typical straight line in the elastic zone at a point below the expected proportional limit due to the high concentrations of stress at the ends of the fillet welds. All tension tests were run at as slow a speed as practical. Fig. 3 shows the plate design "A" and "B" which were tested.

The results of the tension tests of plate designs "A" and "B" are shown on the stress-elongation diagram, Fig. 10. The predicted M-Θ curve for plate design "A" based on the tension test results is shown in Fig. 5 as a dash-dot curve.

In the first test (plate design "A") some difficulty was encountered due to misalignment of the specimens in the testing rig pointing out the necessity of accurate assembly. However, the average curve came out close to what was predicted from the coupon tests.

The initial failure in the first test was in the butt weld and was entirely unpredicted. Investigations showed that the weld lacked 1/8-in. of full penetration, weakening it excessively. Fig. 11 shows the details of the bad weld and the changes made to help correct the situation. The revised weld proved entirely sufficient in subsequent tests which were checked. This is another of numerous examples observed by the senior author where a welded joint has been penalized because of insufficient root gap.

As expected, initial yield lines appeared near the ends of the fillet welds in plate design "A" denoting the stress concentrations there, and final failure was across this point and typically ductile in nature. In contrast, as was desired, initial yield lines appeared in the mid-region of the reduced section of plate design "B" and did not progress to either end until quite late in the test. Final failure was a ductile tear in the plate at the ends of the curved fillets leading into the reduced section of the plate.

Several important conclusions may be drawn from the tension tests and their setup: (1) Careful fitting and welding is essential. Use of wedges is recommended in the fitting operation. For the butt weld it is recommended that the plate be tapered to a point and a 3/16-in. to 1/4-in. root gap be used; (2) If the above is followed, the average curve obtained from the test will closely approximate the curves for the individual plates; (3) Plate Design "B" appears to eliminate most of the stress concentration problems encountered in Design "A" (however it might be wise to increase the radius of the curved fillets somewhat).

The important feature of these tension tests was to check the possibility of predicting the moment-rotation curve for a connection using a similar top plate. This M-θ curve is easily derived from the tension tests results if it is remembered that the tension in the top plate is a function of the moment on the connection and that the elongation in the plate is a function of the beam end rotation, θ. In other words, $M = Td$ and $\theta = e/d$. The resulting values are plotted as the predicted M-θ curve (shown as a dash-dot curve) in Fig. 5. The result is that the M-θ curve predicted from the tension tests and the actual connection M-θ curve are closely correlated. This indicates the adequacy of the tension tests for prediction of the connection actions for this particular plate design at least.

POTENTIALITIES OF THE CONNECTION

In studying the potentialities of this top plate type of connection it is felt that one should be governed by Section 1 of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings:

> "Type 3 (semi-rigid) construction will be permitted only upon evidence that the connections to be used are capable of resisting definite moments without overstress of the welds. The

proportioning of main members joined by such connections shall be predicated upon no greater degree of end restraint than the minimum known to be effected by the respective connections. Types 2 and 3 construction (flexible and semi-rigid connections respectively) may necessitate some non-elastic but self-limiting deformation of a structural steel part, but under forces which do not overstress the rivets, bolts or welds".

The above quotation could be interpreted to mean that the top plates must have an effective contact width with the column that is in excess of the effective width of the top plate through the critical section by the ratio of the yield point stress to the working stress of 20 ksi. Thus if the yield point stress is assumed as 33 ksi, and referring to Fig. 3, Plate Design A or B:

$$w' = w \times 33/20$$

The fillet welds at the right end of the top plates in Fig. 3 would require a length of $A_{eff} \times 33$ divided by the permissible load per inch of weld.

It is observed in the above that the amount of welding is directly dependent on the effective area of the top plate. If the connection under consideration relates to beams framing onto the flanges of the columns it is further noted that the column stiffeners will have a size about the equivalent of the top plate and that the cost of welding these stiffeners is likewise influenced by the selected effective area of the top plate. From these considerations it appears that the smallest top plate that satisfies the conditions will be the economical one. Another consideration is that the smaller the top plate the smaller the moments thrown into the columns from the beams. The economical aspects will be more clearly shown by several designs of a typical case.

PROPOSED DESIGN METHOD I

Assumptions:

The wind moments are carried by a suitable diagonally trussed frame so that only gravity loads need be considered in designing the beams, columns, and the connections.

Design Procedure:

Design the beams (presumably for a uniformly distributed load) for WL/12 on the basis of partially restrained joints with a guaranteed end restraint of 50% when an overload of 1.67 x working load is on the beam. Thus, under working load, the actual end restraint will be somewhere between WL/12 and 75% of WL/12 or WL/16. The moment at the center of the beam will be between WL/24 and WL/16. Should an overload of 1.67 x working load be applied and should the end connections have the minimum guaranteed value of 50% restraint the midspan moment would become WL/12. This would represent the ultimate capacity of the beam. Considering that, if the top connection is either stiffer or stronger than the bare design requirements, the beam will carry a correspondingly greater load, <u>the use of WL/12 to select beam sizes may well be considered as very conservative</u>.

Therefore, design the top plate on the assumption of 75% restraint, that is for WL/16 at the critical cross-section. (Note also that 50% of full end restraint for an overload of 1.67 is 1.67 x WL/24 = 5W L/72 where W is working load.)

Thus, if A is the critical area of the plate

d is the depth of the beam

σ_a is the allowable tensile stress in the top plate

σ_y is the yield point stress in the top plate

$$\text{Required: } A = \frac{WL}{16d\sigma_a} \qquad \text{------- Eq. (D-1)}$$

To satisfy the guarantee of 50% restraint under 1.67, the above required A must be equal to or greater than:

$$A = \frac{5WL}{72d\sigma_y} \qquad \text{-------- Eq. (D-2)}$$

To comply with the design requirement that the welds must not be overstressed the following procedure is suggested:

A. Widening of the plate for attachment to the column. Compute the per cent restraint from formula 6 or 7 (7 preferred because it is on the safe side). Label as R'. (With experience this value can be estimated and often does not vary greatly from 90%). The top plate must be widened, in order not to overstress the butt weld, by the ratio of R'/75 (say by 90/75). If this is done and if a full butt weld is made, the design requirement will be met.

B. Fillet welds between top plate and beam. The critical area A times σ_a gives the apparent tension in the top plate. However, this could be adjusted by the ratio R'/75 to get the true tension. The remainder of the design is routine.

The above design procedure assumes that stiffeners will be used between the column flanges opposite the beam flanges to prevent local distortion or buckling of the column flanges, and to insure that the connection behaves in accordance with the formulas developed. This design method may be thought of as a safe, conservative method, that should be as easy to use, after a little practice, as fully-restrained connections. It is probably somewhat competitive with direct welding of beams to columns, dependent on the equipment available in the shops. However, it should be noted that, in contrast to direct welding, in the top plate type of connection all field welding is in a down hand position easily and economically placed by a welder sitting on the beam. Also, fabrication and erection tolerances will be greater than for direct welding.

Example of Design Method I

Assumed Design Problem:

15' span and 50-kip total distributed load

20 ksi allowable stress for full penetration butt welds, also allowable stress for the top plate

13.6 ksi allowable shear in steel and on throat of fillet weld (1200 lb/in. in 1/8-in. weld size)

75% end-restraint for connection design and 50% end-restraint for beam design

Beam Design:

$M = WL/12 = 50 \times 15/12 = 62.5$ k-ft.

$I/c = M/S = 62.5 \times 12/20 = 37.50 \text{ in}^3$

Most economical section: 14 WF 30 ($I/c = 41.8$)

$$\text{Web shear} = \frac{50/2}{0.27 \times 13.86} = 6.69 \text{ ksi} < 13.6 \quad \therefore \text{OK}$$

USE: 14 WF 30

Top Plate Design:

$$\text{By Eq. D-1, } A = WL/16d\sigma_a = \frac{50 \times 15 \times 12}{16 \times 14 \times 20} = 2.01 \text{ in}^2$$

Choose plate "Design B" for illustration (See Fig.3)

Make critical section 5.0" x 7/16" ($A = 2.19 \text{ in}^2$) by 6" long

Equation D-2 is amply satisfied

$$A = \frac{5\ WL}{72d\sigma_y} = \frac{5 \times 50 \times 15 \times 12}{72 \times 14 \times 33} = 1.35 \text{ in}^3 < 2.01 \text{ in}^2 \quad \therefore \text{OK}$$

Widening of top plate for connection to column:

By Eq. 7, $\%R = \dfrac{100 \times 2.19 \times 192 \times 15 \times 12}{(4 \times 290 \times 7) + (2.19 \times 192 \times 180)}$

$= 89.9\%$ (say 90)

Minimum width = 5.0" x 90/75 = 6.0"

Widen plate to 6.00" for column connection

Widen plate for beam connection to 5.75"

(Beam flange is 6.75" wide)

Required length of filled weld (5/16" at 3k/inch)

$$\frac{2.01 \times 30 \times 90/75}{3} = 16.1''$$

Weld 5.75" on end of top plate and 5.5" on each edge. See Fig. 12 for details of plate design.

Seat Design:

Any standard method may be used to pick the seat based on the vertical reaction to be carried. However, provision must be made to carry the thrust of the bottom flange of the beam to the column. This may be done by using a square butt weld between the flange end and the column or by using fillet welds between the flange edges and the top surface of the seat. See discussion under EXPERIMENTS.

PROPOSED DESIGN METHOD II

Assumptions: No wind moments as in Method I.

Take full advantage of the rule permitting "non-elastic but self-limiting deformation" of the top plate.

Raise σ in Equation (D-1) to such a value that Equation (D-2) is also exactly satisfied, provided only that the stress in the top plate does not exceed the yield point of the material when working load is on the beam.

Design Procedure:

By equating D-1 and D-2,

$$\sigma_a = 0.9\sigma_y = 29.7 \text{ ksi for A-7 steel.}$$

However, the actual stress in the top plate, which is given by $\sigma_a \times R'/75$, must not exceed σ_y. For the average case where $R' = 90\%$, σ_a must not exceed 33 x 75/90 or 27.5 ksi. Therefore use $\sigma_a = 27.5$ ksi for design of the top plate.

Example Of Design Method II

Assumed Design Problem:

Same as for Method I except permit stress in top plate under working load to increase to, but not exceed, the yield point.

Beam and Seat Design:

Same as for Method I

Top Plate Design:

Use $\sigma_a = 27.5$ ksi

By Eq. D-1, $A = WL/16\,d\sigma_a = \dfrac{50 \times 15 \times 12}{16 \times 14 \times 27.5} = 1.46 \text{ in}^2$

Make critical section 4.0 x 3/8" ($A = 1.5 \text{ in}^2$) by 5" long

Widening of top plate for connection to column:

Minimum width = 4.0 x 33/20 = 6.6" Use 6-3/4".

Widen plate for beam connection to 5.75" to reduce stress concentrations at ends of box fillet welds.

Required length of fillet weld (5/16" at 3k/inch)

$$= \frac{1.50 \times 33}{3} = 16.5''$$

See Fig. 12 for details of plate design.

PROPOSED DESIGN METHOD III

Assumptions:

Include wind moments in the connection design.

Design Procedure:

1. Find wind moment at beam-column connection by some acceptable procedure.

2. Assuming the top plate connection will be similar to those in previous examples, the gravity moment at the connection will be very closely equal to: $M_g = 0.90\ WL/12$.

3. The total maximum end moment will then be: $M_{wg} = M_w + M_g$. Since the probability of full wind and full gravity moments occurring simultaneously are remote, the permitted procedure will be used by designing for $0.75\ M_{wg}$.

4. The permissible unit stress for the top plate is not specified in AISC Specifications but will be taken as 24 ksi, although a higher value would undoubtedly be safe. Hence, required A of the top plate will be:

$$A = \frac{0.75\ M_{wg}}{\sigma d} = \frac{0.75\ M_{wg}}{24d} \text{ or } \frac{M_{wg}}{32d}$$

5. Beam design will be based on: Required $I/c = \dfrac{0.75\ M_{wg}}{\sigma}$. The proper value for σ is debatable as some designers might use 20 ksi while others might argue that the AISC permits 24 ksi.

Examples Of Design Method III

Assumed Design Problem:

Same span and gravity loads as in previous examples.

Assume: for Case A a wind moment of 30 kip-ft.

for Case B a wind moment of 60 kip-ft.

Assume $M_g = 0.90\ WL/12$ for practical purposes.

(Note: for greater precision formula 7 may be used to compute the actual %R for use in place of the 90% assumed.)

Beam Design:

$M_g = 0.90 \times 50 \times 15/12 = 56.25$ kip-ft.

Case A: Required $I/c = \dfrac{0.75\ M_{wg}}{\sigma} = \dfrac{0.75(56.25 + 30)12}{20} = 38.8 \text{ in}^3$

USE 14WF30 ($I/c = 41.8 \text{ in}^3$)

Case B: Required $I/c = \dfrac{0.75(56.25 + 60)}{20} = 52.3 \text{ in}^3$

USE 14 WF 38 ($I/c = 54.6 \text{ in}^3$)
or 16 WF 36 ($I/c = 56.3 \text{ in}^3$)

Top Plate Design:

Case A; Required $A = \dfrac{0.75\ M_{wg}}{\sigma d} = \dfrac{0.75(56.25 + 30)12}{24 \times 14} = 2.31 \text{ in}^2$

Case B: Required $A = \frac{0.75\ (56.25 + 60)\ 12}{24 \times 14} = 3.11\ in^2$

It should be noted that use of $\sigma = 33$ ksi for the top plate design would be close but safe if small plastic deformations are acceptable. Also, if 24 ksi is permitted in the beam design some savings in beam weight would be achieved.

SUMMARY

Once the plastic theory of design and the provision for plastic deformations are accepted, it is readily seen that the top plate beam-column connection has many economic potentialities. As mentioned previously, however, additional tests must be run before acceptance of the latter proposed design procedures will be practiced to any great extent. The project is well worth prusuing when one realizes the potential economies and the simplified design procedures which will evolve. The United States is noted for its very conservative design practices and the added costs relative thereto. The foregoing is one place where those costs can be cut down, and structural steel made to work to its full potential capacity.

APPENDIX

WELDED BEAM-COLUMN CONNECTIONS

TOP PLATE AND SEAT TYPE

I. Derivations of equations for the relation between stress in the top plate and the load on the beam.

Case A: Rotation is assumed to take place about the base of the beam (which is the approximate case for connections incorporating light top plates).

Notations: A = sectional area of top plate, in^2

L' = effective length of top plate, inches

W = total distributed load on beam, kips

L = beam span, inches

d = depth of beam, inches

T = tension in top plate, kips

σ = stress in top plate, ksi

θ = end slope of beam, radians

e = elongation in the top plate, inches

E = modulus of elasticity = σ/ϵ

In Fig. A1(a): The actual end slope may be expressed as:

$$\theta = e/d \text{ where } e = \epsilon L' = \frac{\sigma L'}{E}$$

$$\theta = \frac{\sigma L'}{Ed} \quad \text{or} \quad = \frac{TL'}{AEd}$$

In Fig. A1(c): The actual end slope may be expressed by area-moment theorem as:

$$\theta = \frac{WL^2}{24EI} - \frac{TdL}{2EI}$$

Since θ must be the same in both cases:

$$\frac{TL'}{AEd} = \frac{\sigma L'}{Ed} = \frac{WL^2}{24EI} - \frac{TdL}{2EI}$$

From which the following useful equations evolve:

$$T = \frac{WL^2/24I}{\frac{L'}{Ad} + \frac{dL}{2I}} \quad - - - - - - - - \quad (1)$$

$$\sigma = \frac{WL^2/24I}{\frac{L'}{d} + \frac{AdL}{2I}} = \frac{WL^2d}{12}\left[\frac{1}{2IL' + Ad^2L}\right] - \quad (2)$$

$$A = \frac{WL}{12d\sigma} - \frac{2IL'}{d^2L} \quad - - - - - - \quad (2a)$$

<u>Case B</u>: Rotation is assumed to take place about the mid-height of the beam (which is the approximate case for connections incorporating heavy top plates).

In Fig. A1(b): The actual end slope may be expressed as:

$$\theta = \frac{e}{d/2}$$

$$\theta = \frac{2\sigma L'}{Ed} \text{ or } = \frac{2TL'}{Aed}$$

In Fig. A1(c): As in Case A: $\theta = \frac{WL^2}{24EI} - \frac{TdL}{2EI}$

From which the following useful equations evolve:

$$T = \frac{WL^2/24I}{\frac{2L'}{Ad} + \frac{dL}{2I}} \quad - - - - - - - - \quad (3)$$

$$\sigma = \frac{WL^2/24I}{\frac{2L'}{d} + \frac{AdL}{2I}} = \frac{WL^2d}{12}\left[\frac{1}{4IL' + Ad^2L}\right] - (4)$$

$$A = \frac{WL}{12d\sigma} - \frac{4IL'}{d^2L} \quad - - - - - \quad (4a)$$

II. Derivations of equations for the Per Cent Restraint provided by the top plate connection.

In Fig. A2(a): the resisting moment provided by a fully fixed end condition = $WL/12$

In Fig. A2(b): the resisting moment provided by the top plate end condition = Td

Hence, regardless of the location of the center of rotation, the per cent restraint may be expressed as:

$$\%R = \frac{Td}{WL/12}(100) = \frac{1200\,Td}{WL} = \frac{1200A\sigma d}{WL} \qquad (5)$$

Case A: Rotation about base of beam.

Substitute value of σ from equation (2) into (5):

$$\%R = \frac{1200Ad}{WL} \times \frac{WL^2d}{12}\left[\frac{1}{2IL' + Ad^2L}\right] = \frac{100Ad^2L}{2IL'+Ad^2L} \qquad (6)$$

Case B: Rotation about mid-height of beam.

Substitute value of σ from equation (4) into (5):

$$\%R = \frac{1200Ad}{WL} \times \frac{WL^2d}{12}\left[\frac{1}{4IL' + Ad^2L}\right] = \frac{100Ad^2L}{4IL'+Ad^2L} \qquad (7)$$

III. Estimation of connection moment at commencement of plastic action for prediction M-θ curve.

Assuming that the top plate yields at 33 ksi and responds as a uni-axially stressed member, the plastic action will begin when the connection moment = Td, where $T = A\sigma_y$.

Hence: $M_y = A\sigma_y d$ - - - - - - - - (8)

If σ_y = 33 ksi: $M_y = 33\,Ad$ = plastic moment - - (8a)