Proposed Design Criteria for Stiffened Seated Connections to Column Webs

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Summary
In multi-story braced frame construction, the connection of choice for many fabricators for Type 2 or Type PR connections between beams and column webs is the seated connection. This connection lends itself to ease of erection because of its greater tolerance when compared to framing angles or a "knife" connection. Because the beam may be cut short, inserting the beam between the column flanges is a simpler procedure. The additional advantage of the seat providing a stable erection platform for the beam before bolts are installed is an advantage for seated connections over framed connections.

The strength and stability of the column web supporting these connections has been questioned at times, both by design engineers and code enforcement officials.

Research, sponsored by the American Institute of Steel Construction, has been undertaken to study the behavior of this connection, and to provide design guidance to designers and detailers. Forty-seven connections were tested as part of a two year study. A limit state for column web strength has been noted as a result of this testing.

Proposed design guidelines for both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) for this connection have been developed.
PROPOSED DESIGN CRITERIA FOR STIFFENED SEATED CONNECTIONS TO COLUMN WEBS

By
Thomas Sputo and Duane S. Ellifritt

INTRODUCTION

In multistory frames, Type 2 or Type PR simple connections of beam to column web are often made by beam seats. Seated connections have advantages over framed connections in that they possess larger erection and fabrication tolerance and provide a stable erection platform for the beam before any bolts are installed.

The design of stiffened seated connections to column webs is an area where no definitive design guidance is currently available. This lack of guidance is acknowledged in both manuals [1,2] in the connection design section by noting that special connections must be designed for supporting members (columns) with thin webs. The strength and stability of the column web supporting these connections has been questioned by design engineers, fabricators, and code enforcement officials.

A number of studies of a simpler, somewhat related situation, that of a welded tab bracket on a column web have been undertaken [3,4,5,6,7,8], using a yield line analysis to model the column web behavior and determine the ultimate connection strength. These studies, while helpful, are not directly applicable to the stiffened seat connection.

A two year research program was undertaken at the University of Florida under the sponsorship of the American Institute of Steel Construction (AISC) to study the behavior of this connection and to develop design guidelines for its safe use.

BRIEF SUMMARY OF LABORATORY TESTING

Phase One Testing

Phase One testing consisted of 32 reduced scale connections. The intent of this phase was to study column web strength, observe column web/flange interaction, to determine the interaction between column web bending and column axial capacity, and to study the interaction between beam curvature and column web out-of-plane deformations.

Full details of this testing are contained in a report to AISC [10]. Some important conclusions were:

1. Contrary to concerns of some engineers, the bottom tip of the stiffener will not punch through the
column web. This was verified on webs as thin as 1/8 inch.

2. The connection will rotate more than the beam, reducing the eccentricity of the applied load, thereby reducing the stress on the welds.

3. Yield line mechanism formation in the column web is a valid concern. While no connection failed in this manner, the mechanism was observed forming prior to the failure of the column by weak-axis column failure.

4. The rotation of the connection and the column web makes this a very flexible connection, approximating a fully simple condition.

5. The flexibility of the connection, coupled with the small eccentricity of the load makes it unnecessary to consider any eccentricity of load or applied moment in the design of the column.

Phase Two Testing

Phase Two testing consisted of 16 connections, 15 to column webs, and one to a column flange as a baseline test. Column sections (W10X33,W12X40,W14X61) were chosen to be representative of normal column sections with relatively slender webs.

The connection chosen had a stiffener length (L) of 8 inches, a seat plate length (Bₜ) of 6-1/2 inches, and a stiffener width (W) of 6 inches. The erection bolts were 7/8 inch A-325 bolts, placed 3 inches out from the column web face, installed snug-tight. While this connection probably would not be typically encountered, it was chosen as a "worst case situation" of a short stiffener length combined with a wide stiffener width. Most usual connections would not be this severe.

The beam was a welded girder of Grade 70 steel, proportioned to rotate similar to a realistically sized beam which might frame into one of the chosen test columns, while being strong enough not to yield under the full capacity of the test equipment.

Failure loads and material properties are shown in Tables 1 and 2. Full details of this testing are contained in a report to AISC [11].

The predominant mode of failure was weld shear, as shown in Figures 1 and 2. Weld failure began at the corners of the seat, then rapidly spread, leading to total loss of load capacity. It was assumed that this was because of a stress concentration due to shear lag effects. Shear lag is caused by the force in the seat plate and weld migrating towards the stiffer column flanges. Test W14X61 TA-R had strain gauges installed on the seat. Figure 3 shows the stress gradient in the seat plate.
A yield line collapse mechanism was observed to be forming in the column web prior to failure of the welds. Evidence of the mechanism forming is shown in Figure 4.

DEVELOPMENT OF DESIGN PROCEDURES

Introduction

Based on the observed behavior of the test specimens, the following failure modes were established:
1. Weld shear failure
2. Yield line failure of the column web

These design recommendations are applicable to columns which meet the following criteria:

1. \( T/t_w \leq 36.1 \)
2. Nominal depth \( \leq 14 \) inches
3. \( \frac{d}{b_f} t_f^3 \leq 0.362 \)

The first criterion is derived from test results to ensure that the effects of shear lag on the welds do not cause the values for weld strength listed in the Manual of Steel Construction [1,2] to be rendered unconservative. This is shown in Figure 5.

The second criterion is empirical, based on the limits of experimental testing of column sections of nominal depth of 14 inches or less. As no sections of greater depth were tested, no assurances as to their performance can be made.

The third criterion is also empirical, designed to ensure that the column flanges are torsionally stiff enough with relation to the web to allow the yield line mechanism to proceed to failure, if the welds were not to fail first in shear. This criterion is derived from a ratio of the moment of inertia of the web to the torsional stiffness of the flange.

\[
\frac{EI_{web}}{GJ_{flange}} = \frac{29000d t_w^3}{12} / \frac{11200b_f t_f^3}{3}
\]

Removing constants produces criterion three. The uppermost limit for tested sections in Phase Two was 0.362 for the W10X33. As this section did not produce excessive flange rotations, and the more flexible sections of Phase One did, the limit of 0.362 was chosen.

These criteria allow the use of all standard column sections:
- W14X43 - W14X730
- W12X40 - W12X336
- W10X33 - W10X112
- W8X24 - W8X67
- W6X20 - W6X25
- W5X16 - W5X19
Weld Strength

The load tables in both LRFD and ASD editions of the Manual of Steel Construction [1,2] are based on weld strength with the load located at an eccentricity of 0.8 times the stiffened width from the welds, or 0.8W. When the connection is located on the web, the web rotates more than the beam end, thereby decreasing the effective eccentricity to a value of less than 0.30W, reducing to some extent the theoretical weld stress.

But as previously noted, the stress in the seat plate is decidedly nonuniform due to shear lag effects. Therefore, the weld stress at the outside corners of the seat plate, where fracture is initiated, is magnified. These two actions of shear lag and decreased eccentricity tend to counterbalance themselves, rendering the existing weld tables somewhat conservative. Based on the limits of testing, this assumption should not be extended to cases where the seat erection bolts are located more than 0.5 times the stiffener width (0.50W) or 2-5/8 inches (greater value) out from the column web face.

Yield Line Analysis

It is a well known fact that the elastic limit is not the true strength of a material such as steel which is ductile and able to redistribute stresses. For example, the plastic moment capacity of a rectangular beam is 150 percent of the first yield moment of that same beam.

Ultimate strength analysis of plate structures which primarily resist load through flexure may be analysed by the yield line method. A yield line is a continuous plastic hinge formed between two plate segments. The goal of yield line analysis is the same as that of plastic analysis of framed structures, that is to determine an ultimate inelastic collapse load. A complete discussion of the yield line method is beyond the scope of this paper and the reader is referred to any reference on the topic. [12]

The least work collapse mechanism for a T-shaped seat, as shown in Figure 6, has been calculated [3] as:

\[ Pu = k \frac{L m}{e} \]  

(1)

where:

- \( Pu \) = Ultimate applied load
- \( k \) = Yield line factor
  - \( k = A \left[ B(C) + D + E \right] \)
  - \( A = 2 / [2T-B_s] \)
  - \( B = 2 + [0.866T/L] \)
\[ C = [(T-B_s)(3T+B_s)]^{1/2} \]
\[ D = T(T-B_s)/2L \]
\[ E = 4L + 3.464T \]

- \( B_s \) = Seat plate length
- \( L \) = Stiffener length
- \( e \) = Eccentricity of applied load
- \( m \) = Ultimate moment capacity of a unit width of plate
  \[ = F^* \frac{t^2}{4} \]
- \( F^* \) = Limiting stress
- \( t \) = Column web thickness

It must be noted that that the yield line method does not consider any factors other than bending, such as axial or shearing forces, membrane action, or the effects of large deformations or strain hardening. The beneficial effect of membrane action and strain hardening has been taken into account in various ways by different researchers. One possible method \[9\], which tended to provide reasonably conservative results when compared to test data, is to use a modified ultimate stress value of:

\[ F^* = F_y + \frac{2}{3} (F_u - F_y) \quad (2) \]

This modified stress value explicitly takes into account the increased strength caused by strain hardening at large plate rotations, and implicitly considers the effects of membrane strengthening of plates at large deflections.

**PROPOSED DESIGN METHOD**

**Fabrication and Erection Criteria**

The following design should be followed in fabricating and erecting this connection.

1. Permanent high strength (A325 or A490) erection bolts of no less than 3/4 inch should be used to secure the beam to the seat. Welds should not be used as they lack the necessary ductility.
2. Erection bolts should be located no further from the column web face than the greater value of 0.50\(W\) or 2-5/8 inch. This is shown in Figure 7.
3. Seat plate should not be welded to the column flanges. To do so will negate this design procedure and induce relatively large moments into the column cross section.
Weld Design

Welds should be designed from the applicable weld design tables in the Manual of Steel Construction [1,2], and fabricated as shown in the accompanying manual figures, with the exception of erection bolts substituted for erection welds.

Column Web Yielding

From the basic yield line equation, the two following equations are suggested:

\[ P = \left[ 0.60 \left( kL \right) \right] m / e \]  \hspace{1cm} (3)

and

\[ \Omega P = \left[ \Phi_b \left( kL \right) m \right] / e \]  \hspace{1cm} (4)

where

- \( P \) = Factored load
- \( \Omega P \) = Unfactored load
- \( \Phi_b \) = Stiffener length
- \( \Phi \) = Plastic plate strength
- \( \Phi_b \) = \( \frac{F^*}{4} \)
- \( t_w \) = Column web thickness
- \( e \) = Load eccentricity
- \( B \) = Distance from column web face to center of erection bolt
- \( F^* \) = \( F_y + \frac{2}{3} \left( F_u - F_y \right) \)
- \( \Phi_b \) = 0.90
- \( k \) = Yield line factor

The calculation of \( k \) can be somewhat complicated. A chart for the value of \( kL \) is provided in Table 3. The chart assumes that the seat plate width (and length of weld beneath the seat) is equal to \( 0.4L + 1/2 \) inch. The chart is slightly conservative for all possible larger seat widths.

REVISED DESIGN CHARTS FOR MANUAL OF STEEL CONSTRUCTION

Modifications to the design aids in the Manual of Steel Construction were produced to allow direct selection of connections without resorting to calculation of the column web strength. The restrictions noted will ensure that weld failure will occur before column web failure by yielding. The revised tables and charts are provided in an Appendix to this paper.
SUMMARY AND CONCLUSIONS

Design criteria for stiffened seated connections to column webs was presented. The procedure outlined is simple and requires little change from present design, fabrication, and erection practice. The procedure recognizes two failure modes and uses accepted engineering principles in their solution.

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REFERENCES


EXAMPLE

Given: Column - W12X40  \( t_w = 0.295 \text{ in.} \)
\( T = 9-1/2 \text{ in.} \)
Grade A-36 steel

Beam - W16X26  DL = 5.75 kips
LL = 17.25 kips
TL = 23.00 kips
Grade A-36 steel
ASD Solution:

1. Determine N

   From Manual of Steel Construction (MSC)

   \[
   \begin{align*}
   R1 &= 15.8 \\
   R2 &= 5.94 \\
   R3 &= 15.0 \\
   R4 &= 1.77
   \end{align*}
   \]

   \[
   \begin{align*}
   N &= \frac{(23.0-15.8)}{5.94} = 1.21 \text{ inches} \\
   &= \frac{(23.0-15.0)}{1.77} = 4.52 \text{ inches}
   \end{align*}
   \]

   Say N = 4-1/2 inches

   Say W = 5 inches

2. Select Seat

   From MSC,

   \[
   \begin{align*}
   W &= 5 \text{ in.} \\
   L &= 7 \text{ in.} \\
   B &= 2-5/8 \text{ in.} \\
   \text{Weld} &= 1/4 \text{ in.} \\
   \text{Load} &= 25.0 \text{ kips}
   \end{align*}
   \]

   NOTE: Yield line criteria need not be checked as this connection meets requirements of revised weld tables (see Appendix).

3. Check Yield Line Criteria

   \[
   \begin{align*}
   B &= 2-5/8 \text{ inches} \\
   e &= 2.625/2 + 0.25 = 1.56 \text{ in.} \\
   \text{From Table 3, } (k L) &= 98 \\
   F^* &= 36 + 2/3(58-36) = 50.67 \text{ ksi} \\
   m &= 0.25(.295)^2(50.67) = 1.102 \text{ in-k} \\
   P &= 0.60(98)(1.102)/1.56 = 41.5 \text{ kips}
   \end{align*}
   \]

   \[41.5 > 23 \text{ OK}\]

LRFD Solution:

\[\Omega P = 1.2(5.75) + 1.6(17.25) = 34.5 \text{ kips}\]

1. Determine N

   From Manual of Steel Construction (MSC)

   \[
   \begin{align*}
   \Omega R1 &= 23.9 \\
   \Omega R2 &= 9.00 \\
   \Omega R3 &= 22.5 \\
   \Omega R4 &= 2.65
   \end{align*}
   \]

   \[
   \begin{align*}
   N &= \frac{(34.5-23.9)}{9.00} = 1.18 \text{ inches} \\
   &= \frac{(34.5-22.5)}{2.65} = 4.52 \text{ inches}
   \end{align*}
   \]

   Say N = 4-1/2 inches

   Say W = 5 inches
2. Select Seat
   From MSC,
   \[ W = 5 \text{ in.} \]
   \[ L = 7 \text{ in.} \]
   \[ B = 2-5/8 \text{ in.} \]
   \[ \text{Weld} = 1/4 \text{ in.} \]
   \[ \text{Load} = 37.5 \text{ kips} \]

NOTE: Yield line criteria need not be checked as this connection meets requirements of revised weld tables (see Appendix).

3. Check Yield Line Criteria
   \[ B = 2-5/8 \text{ inches} \]
   \[ e = 2.625/2 + 0.25 = 1.56 \text{ in.} \]
   From Table 3, \((k L) = 98\)
   \[ F* = 36 + 2/3(58-36) = 50.67 \text{ ksi} \]
   \[ m = 0.25(.295)^2(50.67) = 1.102 \text{ in-k} \]
   \[ \Omega P = 0.90(98)(1.102)/1.56 = 62.3 \text{ kips} \]
   \[ 62.3 > 34.5 \text{ OK} \]

Table 1. Phase Two Material Properties

<table>
<thead>
<tr>
<th>Section</th>
<th>(F_y) (ksi)</th>
<th>(F_u) (ksi)</th>
<th>(F^*) (ksi)</th>
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</thead>
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<tr>
<td>W10X33</td>
<td>51.5</td>
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<td>62.4</td>
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<td>W12X40</td>
<td>50.6</td>
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<tr>
<td>W14X61</td>
<td>61.3</td>
<td>80.2</td>
<td>73.9</td>
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</table>

\[ F^* = F_y + \frac{2}{3}(F_u - F_y) \]

Note: Test coupons taken from the column webs
Table 2. Phase Two Test Results

<table>
<thead>
<tr>
<th>SECTION</th>
<th>$P_{FAIL}$</th>
<th>$P_{ASD}$</th>
<th>$P_{ULT}$</th>
<th>$P_{YL}$</th>
<th>$P_{FAIL}/P_{ULT}$</th>
<th>$P_{FAIL}/P_{ASD}$</th>
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<td>W10X33 NA</td>
<td>146.6</td>
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NA = No erection angle installed
TA = Top angle installed
R = Weld return of 1/2" on seat
WAA = Weld across both top and bottom of seat
W = Beam welded to seat in addition to erection bolts
FLA = Connection attached to flange rather than web

$P_{ASD}$ = ASD design load, kips
$P_{FAIL}$ = Test failure load, kips
$P_{ULT}$ = Ultimate load based on weld strength, kips
$P_{YL}$ = Ultimate load based on yield line strength, kips

Note: 1. All welds are 1/4" E70 fillet welds except for W14X61 TA-FLA, which was 3/16" fillet.
2. (*)Test terminated prior to failure due to equipment malfunction.
Table 3. Values of $kL$

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<th>L</th>
<th>4 3/4&quot;</th>
<th>6 1/8&quot;</th>
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Figure 1. Column Web Rotation at Ultimate Load
Figure 2. Weld Fracture in Seat Plate

Figure 3. Shear Lag in Seat Plate
Figure 4. Development of Yield Line Mechanism

Figure 5. Column Web Slenderness Limit
Figure 6. Yield Line Pattern

- Yield Line Pattern

Figure 7. Connection Diagram

- Connection Diagram

\[ e = B/2 + 1/4 \text{ in.} \]

- Connection Diagram

\[ B \leq W/2 \text{ greater value} \]

\[ B \leq 2-5/8 \text{ in.} \]

- Connection Diagram
APPENDIX
STIFFENED SEATED CONNECTION DESIGN AIDS

Revisions to the Stiffened Seated Beam Connection section of the ASD and LRFD Manual of Steel Construction are noted here. Restrictions noted here will ensure that weld failure will occur before column web failure by yield line analysis. Direct design of stiffened seated connections to column webs is therefore possible without referring to other sources, except in the most extreme and rare cases.

Notes for Stiffened Seated Connections to Column Webs

1. Design criteria is applicable to the following column sections:
   - W14X43 - W14X730
   - W12X40 - W12X336
   - W10X33 - W10X112
   - W8X24 - W8X67
   - W6X20 - W6X25
   - W5X16 - W5X19

2. Beam must be connected to seat by high-strength erection bolts (A325 or A490). Centerline of bolts are located no further from the column web face than the greater value of 0.50W or 2-5/8". Welding beam to seat plate is not recommended.

3. For seated connections where W=8" or W=9" and 3-1/2" < B < 0.50W, or W=7" and 3" < B < 0.50W for a W14X43 column, see Sputo, Thomas and Duane S. Ellifritt, "Proposed Design Criteria for Stiffened Seated Connections to Column Webs," AISC Engineering Journal, Vol. ??, No. ?, ??th Quarter, 19??.

4. Top angle is welded or bolted in place, 1/4" minimum thickness.

5. Seat plate should not be welded to column flanges.

6. Except as noted, maximum weld size is limited to column web thickness (t_w) for connections to one side of the web. For connections in line on opposite sides of a column web, limit E70XX weld size to 0.50t_w for F_y=36ksi and 0.67t_w for F_y=50ksi.
STIFFENED SEATED BEAM CONNECTIONS
Welded—E70XX electrodes

TABLE VIII
Seated connections should be used only when the beam is supported by a top angle placed as shown above, or in the optional location as indicated.

Design loads in Table VIII are based on the use of E70XX electrodes. The table may be used for other electrodes, provided that the tabular values are adjusted for the electrodes used (e.g., for E60XX electrodes, multiply tabular values by 60/70 or 0.86, etc.) and the welds and base metal meet the provisions of LRFD Specification Sect. J2.

Design weld capacities in Table VIII are computed using traditional vector analysis.

Based on \( F_y = 36 \) ksi bracket material, minimum stiffener plate thickness, \( t \), for supported beams with unstiffened webs should not be less than the supported beam web thickness for \( F_y = 36 \) ksi beams, and not less than 1.4 times the beam web thickness for beams with \( F_y = 50 \) ksi. Based on bracket material of \( F_y = 50 \) ksi or greater, the minimum stiffener plate thickness \( t \) for supported beams with unstiffened webs should be the beam web thickness multiplied by the ratio of \( F_y \) of the beam to \( F_y \) of the bracket [e.g., \( F_y \) (beam) = 65 ksi; \( F_y \) (bracket) = 50 ksi; \( t = t_w \) (beam) x 65/50, minimum]. The minimum stiffener plate thickness, \( t \), should be at least two times the required E70XX weld size when \( F_y \) of the bracket is 36 ksi, and should be at least 1.5 times the required E70XX weld size when \( F_y \) of the bracket is 50 ksi.

Thickness \( t \) of the horizontal seat plate, or tee flange, should not be less than 3/8".

If seat and stiffener are separate plates, finish stiffener to bear against seat. Welds connecting the two plates should have a strength equal to, or greater than, the horizontal welds to the support under the seat plate.

Welds attaching beam to seat may be replaced by bolts.

ASTM A307 bolts may be used in seated connections, provided the stipulations of LRFD Specification Sect. J1.9 are observed.

For stiffener seats in line on opposite sides of a column web of \( F_y = 36 \) ksi material, select E70XX weld size no greater than 0.50 of column web thickness. For column web of \( F_y = 50 \) ksi, select E70XX weld size no greater than 0.67 of column web thickness.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

LRFD MANUAL

8-20
## STIFFENED SEATED BEAM CONNECTIONS

Welded—E70XX electrodes

### TABLE VIII Ultimate loads in kips

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<th>Width of Seat $W$, in.</th>
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<td>Weld Size, In.</td>
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### Connections to column webs

<table>
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<tr>
<th>B</th>
<th>2-5/8” max</th>
<th>2-5/8” max</th>
<th>3” max</th>
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<tbody>
<tr>
<td></td>
<td>W12X40, W14X43</td>
<td>for L ≥ 9”</td>
<td>limit weld ≤ $\frac{1}{4}$</td>
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</table>

Note: Loads shown are for E70XX electrodes. For E60XX electrodes, multiply tabular loads by 0.86, or enter table with 1.17 times the given reaction. For E80XX electrodes, multiply tabular loads by 1.14, or enter table with 0.875 times the given reaction.

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### STIFFENED SEATED BEAM CONNECTIONS

Welded—E70XX electrodes

#### TABLE VIII Ultimate loads in kips

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Connections to column webs

<table>
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<tr>
<th>B</th>
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<th>3-1/2&quot; max</th>
<th>3-1/2&quot; max</th>
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<td>W14X43, limit</td>
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<td>B ≤ 3&quot;</td>
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Note: Loads shown are for E70XX electrodes. For E60XX electrodes, multiply tabular loads by 0.86, or enter table with 1.17 times the given reaction. For E80XX electrodes, multiply tabular loads by 1.14, or enter table with 0.875 times the given reaction.
STIFFENED SEATED BEAM CONNECTIONS
Welded-E70XX Electrodes

TABLE VIII

Seated connections should be used only when the beam is supported by a top angle placed as shown above, or in the optional location as indicated.

Allowable loads in Table VIII are based on the use of E70XX electrodes. The table may be used for other electrodes, provided the tabular values are adjusted for the electrodes used (e.g., for E60XX electrodes, multiply tabular values by 0.92 or 0.86, etc.) and the welds and base metal meet the provisions of AISC ASD Specification Sect. J2.4.

Allowable weld capacities in Table VIII are computed using traditional vector analysis.

Based on $F_y = 36$ ksi bracket material, minimum stiffener plate thickness, $t$, for supported beams with unstiffened webs should not be less than the supported beam web thickness for $F_y = 36$ ksi beams, and not less than 1.4 times the beam web thickness for beams with $F_y = 50$ ksi. Based on bracket material of $F_y = 50$ ksi or greater, the minimum stiffener plate thickness, $t$, for supported beams with unstiffened webs should be the beam web thickness multiplied by the ratio of $F_y$ of the beam to $F_y$ of the bracket (e.g., $F_y$ (beam) = 65 ksi; $F_y$ (bracket) = 50 ksi; $t = t_w (beam) \times 65/50$, minimum). The minimum stiffener plate thickness, $t$, should be at least two times the required E70XX weld size when $F_y$ of the bracket is 36 ksi, and should be at least 1.5 times the required E70XX weld size when $F_y$ of the bracket is 50 ksi.

Thickness $t$ of the horizontal seat plate, or tee flange, should not be less than $\frac{3}{8}$ in.

If seat and stiffener are separate plates, finish stiffener to bear against seat. Welds connecting the two plates should have a strength equal to, or greater than, the horizontal welds to the support under the seat plate.

Welds attaching beam to seat may be replaced by bolts.

ASTM A307 bolts may be used in seated connections, if the stipulations of AISC ASD Specification Sect. J1.12 are observed.
Should combinations of material thickness and weld size selected from Table VIII exceed the limits set by AISC ASD Specification Sects. J2.2 and J2.4, increase the weld size or material thickness as required.

In addition to the welds shown, temporary erection bolts may be used to attach beams to seats (optional).

To permit selection of the most economical connection, the reaction values should be given on the contract drawings. If the reaction values are not given, the connections must be selected to support the beam end reaction calculated from the Allowable Uniform Load Tables for the given shape, span, and steel specification of the beam in question. The effect of concentrated loads near an end connection must also be considered.

**EXAMPLE 19**

**Given:**
- Beam: W 30 × 116 (flange = 10.495 in. × 0.85 in.; web = 0.565 in.)
- ASTM A36 steel (Fy = 36 ksi)
- Welds: E70XX
- Reaction: 100 kips

Design a two-plate welded stiffener seat using ASTM A36 steel.

**Solution:**

From the $F_y = 36$ ksi, Allowable Uniform Load Table for W30 x 116, note that $R_1$, $= 54.5$ kips, $R_2 = 13.4$ kips/in., $R_3 = 79.9$ kips, $R_4 = 4.33$ kips/in.

For yielding $N_{reqd} = (R - R_1)/R_2$

$= (100 - 54.5)/13.4 = 3.40$ in.

For buckling $N_{reqd} = (R - R_3)/R_4$

$= (100 - 79.9)/4.33 = 4.64$ in.

Stiffener width = 4.64 + 0.5 (setback) = 5.14 in.

Use $W = 6$ in.

Enter Table VIII with $W = 6$ in. and a reaction of 100 kips; select a $\frac{3}{16}$-in. weld with $L = 15$ in., which has a capacity of 103 kips. From this, the minimum length of weld between seat plate and support is 2 x 0.2$L = 6$ in. This also establishes the minimum weld between the seat plate and the stiffener as 6 in. total, or 3 in. on each side of stiffener.

Stiffener plate thickness $t$ to develop welds is 2 x $\frac{3}{16} = \frac{3}{8}$ in., or 0.625 in. This is greater than the beam web thickness of 0.565 in.; thus, the stiffener plate thickness need not be increased.

Use: $\frac{3}{8}$-in. plate for the stiffener and $\frac{3}{8}$-in. plate for seat.
## STIFFENED SEATED BEAM CONNECTIONS

**Welded—E70XX electrodes**

### TABLE VIII Allowable loads in kips

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<tr>
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<td>2-5/8&quot; max</td>
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<td>3&quot; max</td>
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**Connections to column webs**

- **B 2-5/8" max**
- **2-5/8" max**
- **3" max**

W12X40, W14X43 - for L ≥ 9", limit weld ≤ 1/4"

Note: Loads shown are for E70XX electrodes. For E60XX electrodes, multiply tabular loads by 0.86, or enter table with 1.17 times the given reaction. For E80XX electrodes, multiply tabular loads by 1.14 or enter table with 0.875 times the given reaction.

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ASD MANUAL

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STIFFENED SEATED BEAM CONNECTIONS
Welded—E70XX electrodes

TABLE VIII Allowable loads in kips

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<thead>
<tr>
<th>Width of Seat W. in.</th>
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Connections to column webs

<table>
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<th>3-1/2&quot; max</th>
<th>3-1/2&quot; max</th>
<th>3-1/2&quot; max</th>
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<td>See Note 3.</td>
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<td>See Note 3.</td>
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Note: Loads shown are for E70XX electrodes. For E60XX electrodes, multiply tabular loads by 0.86, or enter table with 1.17 times the given reaction. For E80XX electrodes, multiply tabular loads by 1.14 or enter table with 0.875 times the given reaction.