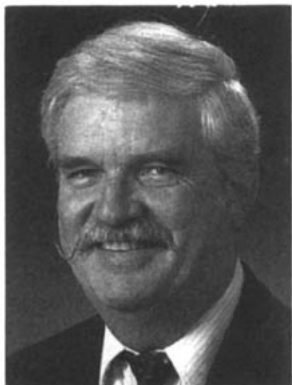# Proposed Design Criteria for Stiffened Seated Connections to Column Webs

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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 (B_s) 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. $dt_w^3/b_f t_f^3 \leq 0.362$

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.

$$EI_{web}/GJ_{flange} = 29000dt_w^3/12 \ / \ 11200b_f t_f^3/3$$

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 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:

$$Pu = k \, L \, m \, / \, e \tag{1}$$

where:

Pu = Ultimate applied load

k = Yield line factor

= A [B(C) + D + E]

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

   = $F* t^2 / 4$

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:

$$F* = F_y + 2/3 (F_u - F_y) \qquad (2)$$

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 0.50W 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:

ASD $\quad$ **P = [0.60 (k L) m] / e** $\qquad\qquad$ (3)

and

LRFD $\quad$ **ΩP = [Φ$_b$ (k L) m] / e** $\qquad\qquad$ (4)

where

ΩP	=	Factored load
P	=	Unfactored load
L	=	Stiffener length
m	=	Plastic plate strength
	=	$t_c^2 F*/4$
t$_w$	=	Column web thickness
e	=	Load eccentricity
	=	(B/2 + 1/4) inches
B	=	Distance from column web face to center of erection bolt
F*	=	$F_y + 2/3(F_u - F_y)$
Φ$_b$	=	0.90
k	=	Yield line factor

The calculation of k can be somewhat complicated. A chart for the value of (k L) 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.

ACKNOWLEDGMENTS

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EXAMPLE

Given: Column - W12X40 t_w = 0.295 in.
                             T = 9-1/2 in.
                             Grade A-36 steel

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

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ASD Solution:
    1.   Determine N
        From <u>Manual of Steel Construction</u> (MSC)
$$R1 = 15.8$$
$$R2 = 5.94$$
$$R3 = 15.0$$
$$R4 = 1.77$$

$$N = (23.0-15.8)/5.94 = 1.21 \text{ inches}$$
$$= (23.0-15.0)/1.77 = 4.52 \text{ inches}$$
Say N = 4-1/2 inches
Say W = 5 inches

    2.   Select Seat
        From MSC,
$$W = 5 \text{ in.}$$
$$L = 7 \text{ in.}$$
$$B = 2-5/8 \text{ in.}$$
$$Weld = 1/4 \text{ in.}$$
$$Load = 25.0 \text{ kips}$$

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
$$F^* = 36 + 2/3(58-36) = 50.67 \text{ ksi}$$
$$m = 0.25(.295)^2(50.67) = 1.102 \text{ in-k}$$

$$P = 0.60(98)(1.102)/1.56 = 41.5 \text{ kips}$$
$$41.5 > 23 \text{ OK}$$

LRFD Solution:
$$\Omega P = 1.2(5.75) + 1.6(17.25) = 34.5 \text{ kips}$$

    1.   Determine N
        From <u>Manual of Steel Construction</u> (MSC)
$$\Omega R1 = 23.9$$
$$\Omega R2 = 9.00$$
$$\Omega R3 = 22.5$$
$$\Omega R4 = 2.65$$

$$N = (34.5-23.9)/9.00 = 1.18 \text{ inches}$$
$$= (34.5-22.5)/2.65 = 4.52 \text{ inches}$$
Say N = 4-1/2 inches
Say W = 5 inches

2.   Select Seat
            From MSC,
                        W   = 5 in.
                        L   = 7 in.
                        B   = 2-5/8 in.
                        Weld = 1/4 in.
                        Load = 37.5 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 inches
            e = 2.625/2 + 0.25 = 1.56 in.
            From Table 3, (k L) = 98
            F* = 36 + 2/3(58-36) = 50.67 ksi
            m = 0.25(.295)^2(50.67) = 1.102 in-k

            ΩP = 0.90(98)(1.102)/1.56 = 62.3 kips
                        62.3 > 34.5 OK

        Table 1.   Phase Two Material Properties

Section	Fy(ksi)	Fu(ksi)	F*(ksi)
W10X33	51.5	67.9	62.4
W12X40	50.6	69.9	63.5
W14X61	61.3	80.2	73.9

            F* = Fy + 2/3 (Fu - Fy)

    Note: Test coupons taken from the column webs

8-12

Table 2. Phase Two Test Results

SECTION	P_{FAIL}	P_{ASD}	P_{ULT}	P_{YL}	P_{FAIL}/P_{ULT}	P_{FAIL}/P_{ASD}
W10X33 NA	146.6	36.8	73.6	103.1	1.99	3.98
W12X40 NA	101.5	36.8	73.6	98.7	1.38	2.76
W14X61 NA	105.3	36.8	73.6	177.4	1.43	2.86
W10X33 TA	95.5	36.8	73.6	103.1	1.30	2.59
W12X40 TA	80.5	36.8	73.6	98.7	1.09	2.19
W14X61 TA	120.3	36.8	73.6	177.4	1.63	3.27
W10X33 TA-R	111.3	36.8	73.6	103.1	1.51	3.02
W12X40 TA-R	112.0	36.8	73.6	98.7	1.52	3.04
W14X61 TA-R	48.9*	36.8	73.6	177.4	*	>1.33
W10X33 TA-WAA	83.4	57.2	114.4	103.1	0.73	1.46
W12X40 TA-WAA	85.0	57.2	114.4	98.7	0.74	1.49
W14X61 TA-WAA	90.2	57.2	114.4	177.4	0.79	1.58
W10X33 TA-R-W	45.1	36.8	73.6	103.1	0.61	1.22
W12X40 TA-R-W	51.1	36.8	73.6	98.7	0.69	1.44
W14X61 TA-R-W	45.1	36.8	73.6	177.4	0.61	1.22
W14X61 TA-FLA	57.9	27.6	55.2	N/A	1.05	2.09

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.

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Table 3. Values of k L

T

L	4 3/4"	6 1/8"	7 5/8"	9 1/2"	11"	11 1/4"
6	102	91	86	82	82	82
7	130	112	103	98	96	96
8	164	137	123	114	111	110
9	203	166	146	133	127	127
10	246	199	171	153	146	145
11	***	235	199	176	165	164
12	***	277	231	200	186	185
13	***	321	265	227	209	207
14	***	360	303	256	234	231
15	***	***	345	287	261	257
16	***	***	390	322	289	285
17	***	***	436	359	320	315
18	***	***	***	398	353	347
19	***	***	***	441	388	381
20	***	***	***	486	426	418
21	***	***	***	533	467	457
22	***	***	***	578	509	499
23	***	***	***	***	555	543
24	***	***	***	***	601	589
25	***	***	***	***	649	637
26	***	***	***	***	689	683

Figure 1. Column Web Rotation at Ultimate Load
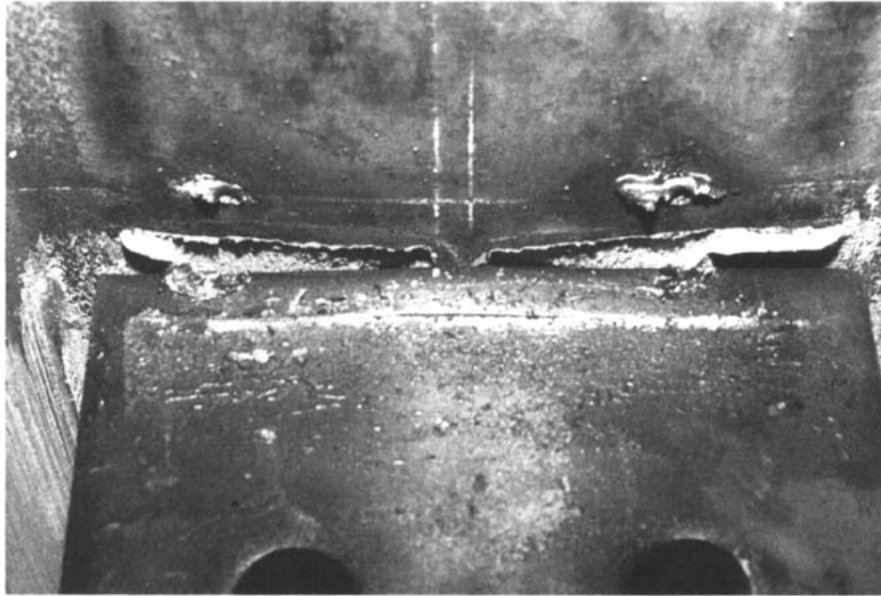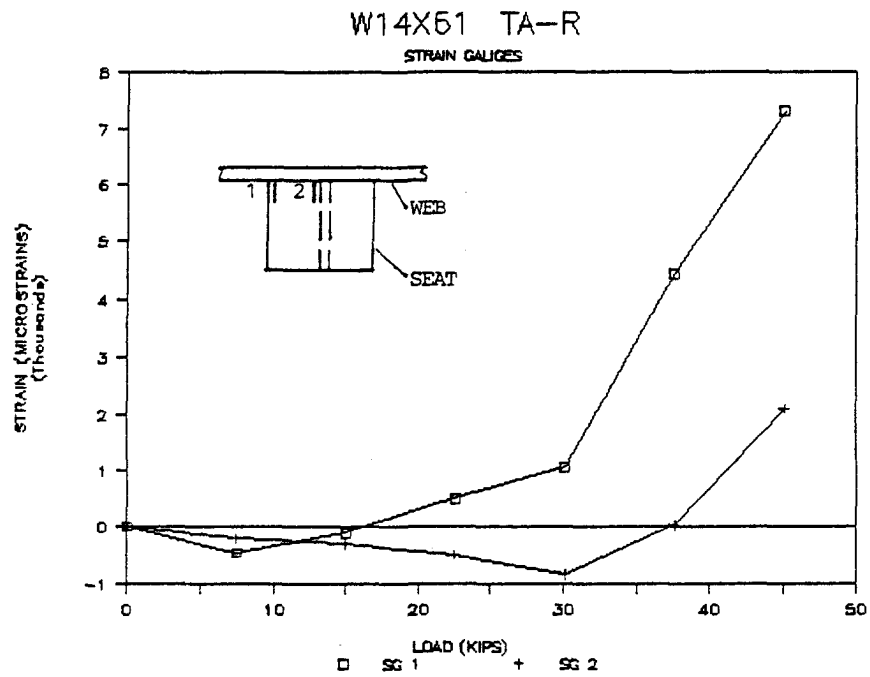
Figure 2.   Weld Fracture in Seat Plate

Figure 3.   Shear Lag in Seat Plate

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Figure 4.   Development of Yield Line Mechanism

COLUMN WEB SLENDERNESS LIMIT
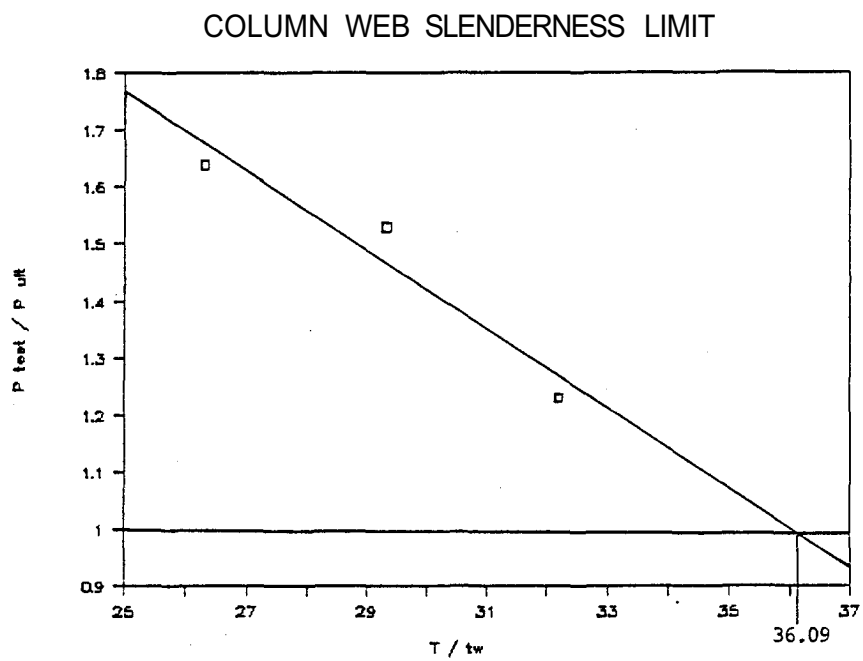
Figure 5.   Column Web Slenderness Limit

8-17

$$e = B/2 + 1/4 \text{ in.}$$
$$B \leq W/2$$
$$B \leq 2\text{-}5/8 \text{ in.}$$
$$\left.\begin{array}{l} \text{greater} \\ \text{value} \end{array}\right\}$$

Figure 7. Connection Diagram

Figure 6.
Yield Line Pattern

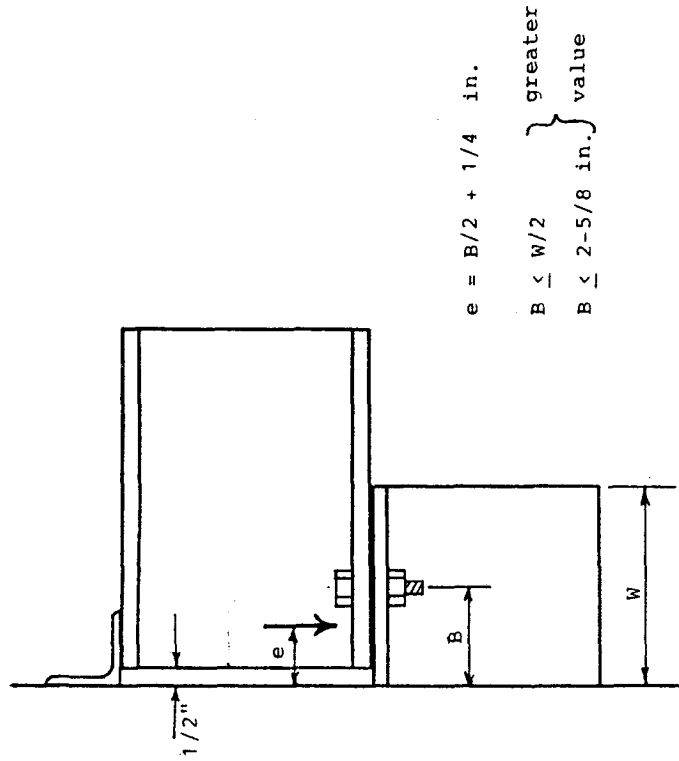Revisions to the Stiffened Seated Beam Connection section of the ASD and LRFD Manual of Steel Construction are noted here. Restrictions noted here will ensure that weld failure will occur before column web failure by yield line analysis. Direct design of stiffened seated connections to column webs is therefore possible without refering 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, 199?.

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.
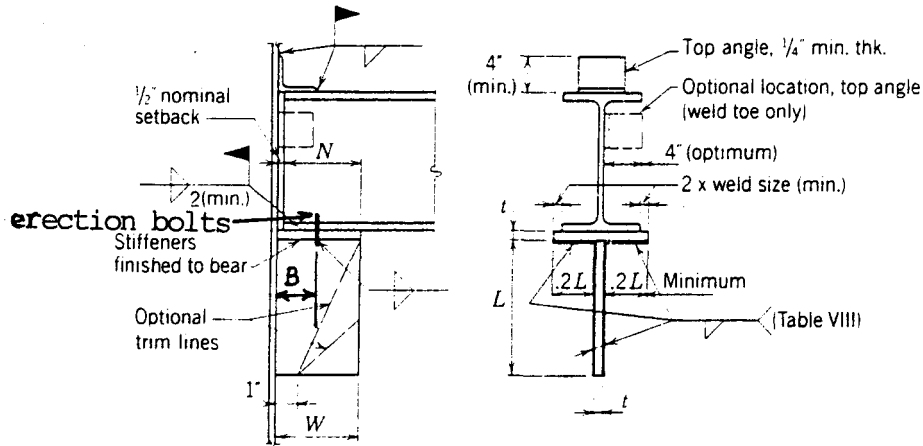
TABLE VIII

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

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STIFFENED SEATED BEAM CONNECTIONS
Welded—E70XX electrodes

TABLE VIII Ultimate loads in kips

L, In.	Width of Seat W, in.											
	4				5				6			
	Weld Size, In.				Weld Size, In.				Weld Size. In.			
	¼	5⁄16	3⁄8	7⁄16	5⁄16	3⁄8	7⁄16	½	5⁄16	3⁄8	7⁄16	½
6	34.0	42.5	51.1	59.6	35.2	42.2	49.3	56.3	29.9	35.9	41.9	47.8
7	44.9	56.1	67.3	78.6	46.9	56.2	65.6	75.0	40.1	48.1	56.1	64.1
8	56.7	70.8	85.0	99.2	59.8	71.7	83.7	95.6	51.4	61.7	72.0	82.2
9	69.2	86.5	104	121	73.7	88.5	103	118	63.8	76.6	89.3	102
10	82.3	103	123	144	88.5	106	124	142	77.2	92.6	108	123
11	95.8	120	144	168	104	125	146	167	91.3	110	128	146
12	110	137	165	192	120	144	168	192	106	127	149	170
13	124	155	186	217	137	164	192	219	122	146	170	195
14	138	173	207	242	154	185	216	246	138	165	193	220
15	152	191	229	267	171	206	240	274	154	185	216	247
16	167	209	250	292	189	227	265	302	171	205	240	274
17	181	227	272	318	207	248	290	331	188	226	264	301
18	196	245	294	343	225	270	315	360	206	247	288	329
19	211	263	316	369	243	291	340	388	223	268	313	357
20	225	281	338	394	261	313	365	417	241	289	337	386
21	240	300	359	419	279	335	391	446	259	311	362	414
22	254	318	381	445	297	357	416	476	277	332	388	443
23	269	336	403	470	315	378	442	505	295	354	413	472
24	283	354	425	495	334	400	467	534	313	376	438	501
25	297	372	446	520	352	422	492	563	331	397	464	530
26	312	390	468	546	370	444	518	592	349	419	489	559
27	326	408	489	571	388	466	543	621	368	441	515	588

Connections to column webs		
B 2-5/8" max	2-5/8" max	3" max
W12X40, W14X43 - for L ≳ 9", limit weld ≲ ¼"		

Note: Loads shown are tor 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 Ultimate loads in kips

L, In.	Width of Seat W, In.											
	7				8				9			
	Weld Size, In.				Weld Size, In.				Weld Size. In.			
	5/16	3/8	7/16	1/2	5/16	3/8	1/2	5/8	5/16	3/8	1/2	5/8
11	81.0	97.2	113	130	72.5	87.1	116	145	65.6	78.7	105	131
12	94.7	114	133	151	85.1	102	136	170	77.1	92.5	123	154
13	109	131	153	174	98.3	118	157	197	89.3	107	143	179
14	124	149	174	198	112	135	180	224	102	123	164	204
15	139	167	195	223	127	152	203	253	116	139	185	232
16	155	186	217	249	142	170	227	283	130	156	208	260
17	172	206	240	275	157	189	251	314	144	173	231	289
18	188	225	264	301	173	208	277	346	159	191	255	319
19	205	246	287	329	189	227	303	378	175	210	280	350
20	223	267	312	356	206	247	329	411	191	229	305	381
21	240	288	336	384	222	267	356	445	207	248	331	413
22	258	309	361	412	240	287	383	479	223	268	357	446
23	275	330	385	440	257	308	411	514	240	288	384	480
24	293	352	410	469	274	329	439	548	257	308	411	513
25	311	373	435	498	292	350	467	584	274	329	438	548
26	329	395	461	526	309	371	495	619	291	349	466	582
27	347	417	486	555	327	393	524	655	308	370	494	617
28	365	438	511	584	345	414	552	690	326	391	522	652
29	383	460	537	613	363	436	581	726	344	412	550	687
30	402	482	562	643	381	457	610	762	362	434	578	723
31	420	504	588	672	399	479	639	799	379	455	607	759
32	438	526	613	701	417	501	668	835	397	477	636	795

Connections to column webs

B	3-1/2" max	3-1/2" max	3-1/2" max
	W14X43, limit B ≤ 3" See Note 3.	See Note 3.	See Note 3.

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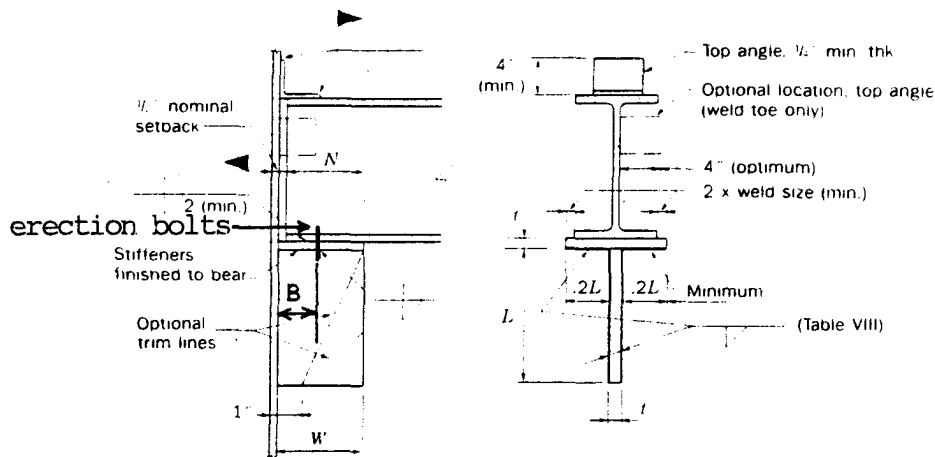
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Seated connections should be used only when the beam is supported by a top angle placed as shown above, or in the optional location as indicated.

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 $^{18}/_{21}$, 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) 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 $^3/_8$ 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.

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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 (F_y = 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, req'd = $(R - R_1)/R_2$

$$= (100 - 54.5)/13.4 = 3.40 \text{ in.}$$

For buckling N, req'd = $(R - R_3)/R_4$

$$= (100 - 79.9)/4.33 = 4.64 \text{ in.}$$

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 ⁵⁄₁₆-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 0.2L = 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 ⁵⁄₁₆ = ⅝ 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: ⅝-in. plate for the stiffener and ⅜-in. plate for seat.

STIFFENED SEATED BEAM CONNECTIONS
Welded—E70XX electrodes

TABLE VIII Allowable loads in kips

L In.	Width of Seat W, In.														
	4					5						6			
	Weld Size, In.					Weld Size, In.						Weld Size, In.			
	$3/16$	$1/4$	$5/16$	$3/8$	$7/16$	$3/16$	$1/4$	$5/16$	$3/8$	$7/16$	$1/2$	$5/16$	$3/8$	$7/16$	$1/2$
6	17.0	22.7	28.4	34.0	39.7	14.1	18.8	23.5	28.1	32.8	37.5	19.9	23.9	27.9	31.9
7	22.4	29.9	37.4	44.9	52.4	18.7	25.0	31.2	37.5	43.7	50.0	26.7	32.0	37.3	42.7
8	28.3	37.8	47.2	56.7	66.1	23.9	31.9	39.8	47.8	55,8	63.7	34.3	41.1	48.0	54.8
9	34.6	46.1	57.6	69.2	80.7	29.5	39.3	49.1	59.0	68.8	78.6	42.5	51.1	59.6	68.1
10	41.1	54.9	68.6	82.3	96.0	35.4	47.2	59.0	70.8	82.6	94.4	51.4	61.7	72.0	82.3
11	47.9	63.9	79.8	95.8	112	41.6	55.5	69.4	83.3	97.1	111	60.9	73.1	85.2	97.4
12	54.8	73.1	91.4	110	128	48.1	64.1	80.2	96.2	112	128	70.8	85.0	99.2	113
13	61.9	82.5	103	124	144	54.8	73.0	91.3	110	128	146	81.1	97.4	114	130
14	69.0	92.0	115	138	161	61.6	82.1	103	123	144	164	91.9	110	129	147
15	76.2	101	127	152	178	68.5	91.4	114	137	160	183	103	123	144	165
16	83.5	111	139	167	195	75.6	100	126	151	176	202	115	138	160	183
17	90.7	121	151	181	212	82.7	110	138	165	193	221	126	151	176	201
18	98.0	131	163	196	229	89.9	119	150	180	210	240	137	164	192	219
19	105	140	175	211	246	97.1	129	162	194	227	259	149	179	208	238
20	112	150	188	225	263	104	139	174	209	243	278	161	193	225	257
21	119	160	200	240	280	111	148	189	223	260	298	173	207	242	276
22	127	169	212	254	296	118	158	198	238	277	317	185	221	258	295
23	134	179	224	269	313	126	168	210	252	294	336	197	236	275	315
24	141	189	236	283	330	133	177	222	267	311	356	209	250	292	334
25	148	198	248	297	347	140	187	234	281	328	375	221	265	309	353
26	155	208	260	312	364	148	197	247	296	345	394	233	279	326	373
27	163	217	272	326	380	155	206	259	310	362	414	245	294	343	392

Connections to column webs

B	2-5/8" max	2-5/8" max	3" max
	W12X40,W14X43 - for L≥ 9", limit weld ≤ ¼"		

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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TABLE VIII Allowable loads in kips

L in.	Width of Seat W. In.											
	7				8				9			
	Weld Size, in.				Weld Size, in.				Weld Size, in.			
	5/16	3/8	7/16	1/2	5/16	3/8	1/2	5/8	5/16	3/8	1/2	5/8
11	54.0	64.8	75.6	86.3	48.4	58.0	77.3	96.6	43.7	52.4	69.9	87.4
12	63.1	75.7	88.4	101	56.7	68.1	90.7	113	51.4	61.7	82.2	103
13	72.7	87.2	102	117	65.5	78.7	105	131	59.6	71.5	95.3	119
14	82.6	99.1	116	132	74.8	89.8	120	149	68.2	81.8	109	136
15	92.9	112	130	149	84.4	101	135	169	77.2	92.6	123	154
16	104	124	145	166	94.4	113	151	189	86.5	104	138	173
17	114	137	160	183	105	126	167	209	96.2	115	154	192
18	126	151	176	201	115	138	184	230	106	127	170	212
19	137	164	192	219	126	151	202	252	117	140	186	233
20	148	178	208	237	137	165	219	274	127	152	203	254
21	160	192	224	256	148	178	237	296	138	165	220	276
22	172	206	240	274	159	192	255	319	149	178	238	297
23	183	220	257	293	171	205	274	342	160	192	256	320
24	195	234	274	312	183	219	292	365	171	205	274	342
25	207	249	290	331	195	233	311	389	182	219	292	365
26	219	263	307	351	206	248	330	412	194	233	310	388
27	231	278	324	370	218	262	349	436	206	247	329	411
28	243	292	341	389	230	276	368	460	217	261	348	435
29	256	307	358	409	242	291	387	484	229	275	367	458
30	268	321	375	428	254	305	406	508	241	289	386	482
31	280	336	392	447	266	319	426	532	253	303	405	506
32	292	350	409	467	278	334	445	556	265	318	424	530

Connections to column webs

B	3-1/2" max	3-1/2" max	3-1/2" max
	W14X43, limit B ≤ 3" See Note 3.	See Note 3.	See Note 3.

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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