Design Criteria for Stiffened Seated Connections to Column Webs

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INTRODUCTION

In multistory braced frames, simple, or Type PR connections of beams to columns are often made using beam seats. These may be made with heavy angles or, if loads are large, a *stiffened* seat may be used. The stiffened seat is usually made by welding two plates in the form of a tee, as shown in Figure 1. A seated connection has an advantage over a framed connection in that it can permit larger fabrication and erection tolerances, requires only two bolts, and provides a stable erection platform for the beam before any bolts are installed.

When attached to a column flange, the capacity of the seat is calculated by checking the fillet welds in combined shear and tension. There are tables in Volume II of the AISC Manual (AISC, 1995) that already have these values worked out for various seat and stiffener sizes. Because of the column flange stiffness, the seat will rotate very little under load, usually less than the end of the beam which rests on it.

However, when the seat is welded to a column *web*, the behavior is very different and it was not clear whether the AISC tables were still applicable to this situation. Figure 2 shows this difference in behavior. A research program was undertaken at the University of Florida under the sponsorship of the American Institute of Steel Construction to study the behavior of this connection and to develop design guidelines for its safe use. This research was conducted in three distinct phases.

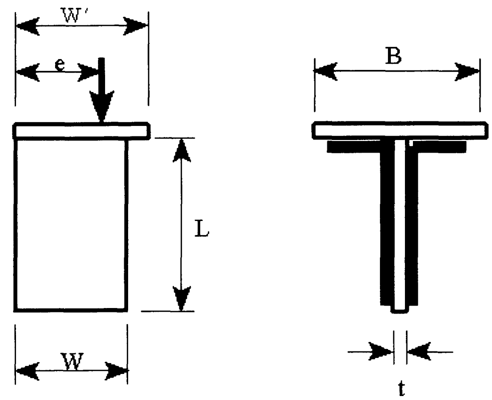

Fig. 1. Stiffened seat dimensions and nomenclature.

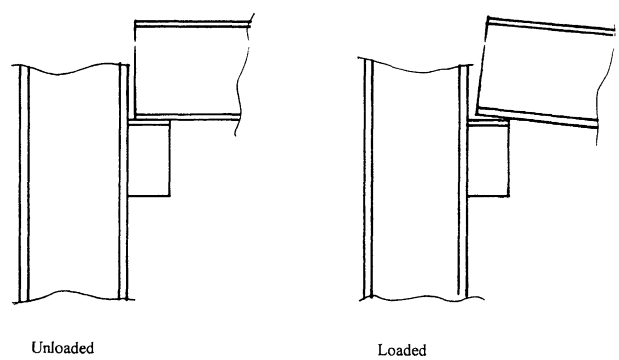

Fig. 2a. Seat attached to flange.

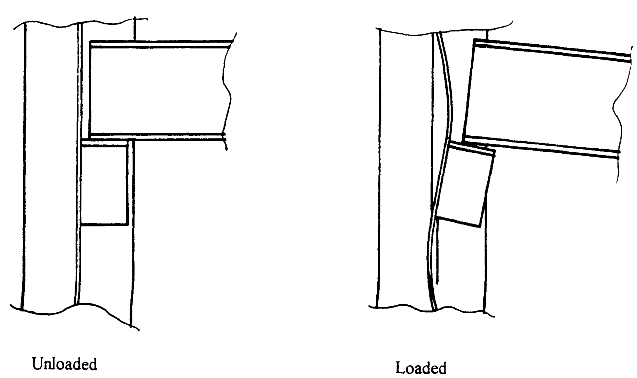

Fig. 2b. Seat attached to web.

PHASE I

Analysis and Behavior

Under load, the column web with attached seat may deform to the point of yielding, but because it is confined by the flanges, it possesses post-yielding strength. In this case, the failure load may be determined by using a yield-line analysis, similar to concrete slabs.

A number of studies of a simpler, but somewhat related, problem have been undertaken. (Abolitz and Warner, 1965; Hoptay and Ainso, 1981; and Hopper, Batson, and Ainso,

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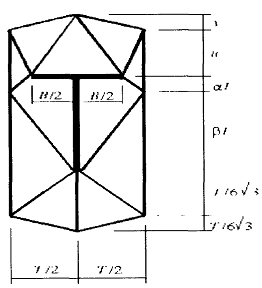
1985). These involved a single plate welded to the mid-line of the web parallel to the flanges. Yield-line analyses were used to model the column web behavior and determine the ultimate connection strength. Stockwell (1974) developed a yield-line analysis for a W-shape with flanges welded to a column web. These studies, while helpful, were not directly applicable to the stiffened seat connection.

However, they did suggest a possible solution for inelastic behavior of the web and a yield-line procedure for a "tee" attached to a web and loaded in bending was developed. Several possible yield line patterns were investigated in the present work. The one shown in Figure 3 gives the minimum failure load and was somewhat corroborated by observed yielding in the white-washed test specimens.

It has been noted by several researchers that the ultimate capacity of a plate is far in excess of the yield strength. Some have suggested that the ultimate strength (F_u) be used instead of the yield strength (F_y). Packer and Bruno (Packer and Bruno, 1986) have suggested the following "effective" yield stress:

$$F^* = F_y + \frac{2}{3}(F_u - F_y) \tag{1}$$



$$\alpha = \frac{T-B}{2T-B}\left[1 - \frac{T}{4L}\sqrt{\frac{3T+B}{T-B}} - \sqrt{3}\right]$$

$$\beta = 1 - \alpha$$

$$u = \frac{T+B}{2}\sqrt{\frac{T-B}{3T+B}}$$

$$v = \frac{T}{2}\sqrt{\frac{T-B}{3T+B}}$$

$$k = \frac{2}{2T-B}X$$

where

$$X = \left[(2 + \frac{T\sqrt{3}}{2L})\sqrt{T-B\,(3T+B)} + \frac{T(T-B)}{2L} + 4L + 2T\sqrt{3}\right]$$

Fig. 3. Yield line pattern for stiffened seat on web.

The nominal strength of the seated connection, as controlled by web strength, can be calculated as,

$$P_n = \frac{kLm}{e} \tag{2}$$

where,

L = stiffener length
m = moment capacity of a unit width of plate
= $\frac{1}{4}t_w 2\ F^*$
t_w = column web thickness
e = load eccentricity
k = yield line factor = $A[C(D) + E + G]$
A = $2/(2T\text{-}B)$
C = $2 + (0.866T/L)$
D = $[(T\text{-}B)(3T\text{+}B)]^{1/2}$
E = $T(T\text{-}B)/2L$
G = $4L + 3.464T$
T = clear distance between column web fillets
B = seat width

The calculation of k can be somewhat cumbersome, but there are a limited number of T distances for standard rolled shapes. Tabulated values of kL for various T dimensions can be generated, making the calculation of P_n easier.

One key element in calculating the strength in this manner is the way that eccentricity, e, is evaluated. In the AISC procedure, which was originally made for the *flange* seated connection, the eccentricity has been taken as 0.8 times the stiffener width, W. However, the behavior of a beam seat under load is quite different when attached to the web. Figure 2 shows this difference. When the web deforms, the point of

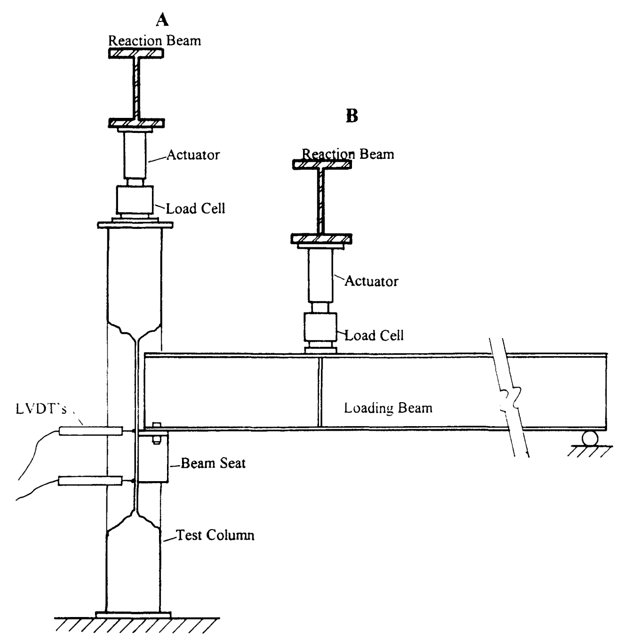


Fig. 4. Experimental set-up for testing beam seats.

Table 1. Results of Phase I Tests							
Column Section	Seat $B \times L$	Loads at Failure, kips		Eccentricity at Failure, in.	Tensile Properties of Column Web, ksi		
		P_a	P_c		F_y	F_u	F^*
W10x12-A	6x6	45.0	60.0	1.586	60.5	69.6	66.6
W10x12-N	6x10	50.0	57.0	1.107	49.4	69.4	62.8
W10x12-A	6x10	30.0	57.5	1.454	52.1	70.3	64.2
W10x12-N	4x6	30.0	60.0	1.412	53.1	70.1	64.4
W10x12-N	4x10	70.0	60.0	1.214	58.3	74.6	69.1
W10x12-A	4x10	50.0	60.0	1.436	58.2	73.5	68.4
W8x10-N	4x10	30.0	60.0	0.997	49.1	67.4	61.3
W8x10-A	4x10	26.0	57.5	1.089	48.2	66.5	60.4
M8x6.5-A	6x6	11.0	45.0	0.422	49.1	65.6	60.1
M8x6.5-A	6x10	11.0	40.0	0.460	46.1	60.8	55.9
M8x6.5-N	4x10	2.0	37.5	0.637	48.7	66.4	60.5
M8x6.5-A	4x10	11.0	36.0	0.649	50.9	65.3	60.5
M8x6.5-N	4x6	2.0	45.0	0.421	45.0	62.2	56.5
M8x6.5-A	4x6	11.0	40.0	0.490	47.6	65.1	59.3
W8x10-N	4x6	38.0	60.0	0.886	49.2	67.2	61.2
W8x10-A	4x6	35.0	60.0	0.959	49.8	67.5	61.6
W8x10-N	6x6	35.0	60.0	1.073	52.3	71.3	65.0
W8x10-A	6x6	29.0	60.0	1.033	49.6	66.2	60.7
W8x10-N	6x10	43.0	60.0	0.834	49.7	68.6	62.3
W8x10-A	6x10	40.0	60.0	0.856	49.2	68.4	62.0
W8x13-A	6x6	72.0	60.0	1.248	52.0	66.0	61.3
W8x13-A	6x10	70.0	60.0	1.161	50.1	65.6	60.4
W8x13-N	4x10	69.0	60.0	1.194	50.4	66.3	61.0
W8x13-A	4x10	69.0	60.0	1.310	52.3	66.2	61.5
W8x13-N	4x6	70.0	60.0	1.054	48.4	64.4	59.1
W8x13-A	4x6	70.0	60.0	1.167	50.2	65.5	60.4

Notes: P_a = Column axial load applied at failure
P_c = Connection load applied at failure
$F^* = F_y + \frac{2}{3}(F_u - F_y)$
N = 1-ksi axial load applied to column initially, then increased until failure
A = Large axial load applied to column, then seat load increased until failure

loading migrates toward the web and the eccentricity is reduced. The magnitude of this eccentricity is crucial to any formula for calculating ultimate capacity. This will be discussed in more detail later in this paper.

Test Program

Phase I testing consisted of 26 connections. The intent of this phase was to study column web strength, observe column web/flange interaction, determine the interaction between column web bending and column axial capacity and study the relationship between beam curvature and column web out-of-plane deformations.

The test arrangement and loading device are shown in Figure 4. The load was delivered to a horizontal beam which rested on the beam seat. Two ⅞-in. A325 bolts were used to attach the load beam to the test specimen. The bolts were not pretensioned. Every attempt was made to simulate the real end rotation of a typical floor beam, since end rotation is crucial to the eccentricity of load on the seat. In a real installation, a column would be loaded from the floors above and so the web would already be under considerable compression, even without the load transmitted by the seat. In Phase I, duplicate tests were made both with and without axial load. The results of these tests are shown in Table 1.

The tests were made on very light column sections, purposely to get large web deformations. The failure modes were always a beam-column failure about the weak axis below the seat, sometimes accompanied by local flange buckling. In *none* of the tests did a weld failure occur. In the tests shown in Table 1 having an "N" suffix, a light axial load of 1 ksi was applied to the column, then the seat was loaded to around 40 to 60 kips, depending on the size of the column. Then the column axial load was increased until failure occurred.

In those tests with an "A" suffix, a large axial load was initially applied to the column, then the load on the seat was gradually increased until failure occurred. The loading sequence seemed to make little difference in the results. The failure modes of some of these specimens can be easily seen in Figure 5.

Since the purpose of this phase of testing was to study the web bending, column sections were used which had very high h/t_w ratios. Some sections had very narrow flanges as well, which led to flange buckling.

Conclusions to be drawn from Phase I are as follows:

1. There was no evidence that the bottom of the stiffener would punch through a thin web, even in webs as thin as ⅛-in. (3 mm).
2. The connection will rotate more than the beam end, thereby reducing the eccentricity of the applied load and reducing stress on the welds.
3. The flexibility of the connection, coupled with the small eccentricity of load, makes it unnecessary to consider any eccentricity of load in the design of the column itself.

Fig. 5. Failure of Phase I tests in a combined web and flange buckling mode.

PHASE II

Introduction

There was some justifiable criticism of the Phase I work. A few persons thought the sections used for the columns were unrealistic and would never be found in such applications in practice. They suggested that more testing be done on realistic column sizes, which were then selected by the AISC review committee. This became the Phase II test program.

Test Program

The column sizes selected for testing were W10×33, W12×40, and W14×61. These sections were chosen as representative of normal column sections with relatively high web slenderness ratios. The connection had a stiffener length of 8 in., a seat width of 6.5 in., and an outstanding leg of 6 in. The erection bolts were ⅞-in. A325 bolts, snug tightened, placed 3 in. out from the column face.

The reaction beam was a welded girder made of 70 ksi steel. The flange plates were 6 in. by ¾-in. and the web plate was 14 in. by ½-in. The same test set-up was used as in the Phase I tests, with the exception that no axial load was put on the column at A (see Figure 4). The results of the Phase I tests showed very little difference in ultimate load whether or not axial load was present in the column, so it was omitted from the Phase II tests. Failure loads for all 16 tests are shown in Table 2. The initial series of three tests, one with each column size, had the beam connected to the seat by the erection bolts, but without a top angle. The second series were duplicates of the first except with a welded 4×4×¼ top angle.

Large rotations of the column web were evident during the testing. These rotations, which exceeded the end rotation of the beam, may contribute to web crippling and/or web yielding of the beam. Figure 6 shows the amount of rotation in the beam seat.

It was noted from the initial six tests that the weld failure began at the corners of the seat nearest the column flanges. It was assumed that this was because of a stress concentration due to a shear lag effect. This effect would be caused by the force in the seat plate and weld "migrating" towards the stiffer column flanges. Test W14×61 TA-R had strain gages installed across the seat, which confirmed the stress gradient, with the highest stress at the seat corners. Based on this observation, the third series of tests had a weld return of ½-in. placed around the corner of the seat plate. This resulted in slightly higher strengths.

In the previous nine tests, the weld failed in combined shear and bending before the yield line mechanism could proceed to failure. Evidence of the mechanism beginning to develop was shown by the flaked whitewash in the regions where yielding was predicted to develop. The fourth series of tests had the weld on the seat carried fully around the seat, on the top as well as on the bottom, hoping to provide enough weld strength to allow the yield line mechanism to fully develop to failure. Surprisingly, the ultimate loads on this series of tests were far below the previous tests. The mode of failure was tearing of the column web, which was too thin to fully develop the strength of the welds. AISC recommends only welding across the *bottom* of the seat and these tests confirmed the wisdom of that recommendation.

PHASE III

Introduction

After reviewing the results of Phases I and II, the AISC Research Committee felt there were a few other variables that needed to be explored. These included seat plates that extended beyond the leading edge of the stiffener, varying seat thicknesses, and varying stiffener lengths. The decision was made to extend the test program to include these additional variables.

(a) Before loading.

(b) Rotation of seat and web after loading.

Fig. 6. Phase II experiments, Test #1. Column is W10×33 (W250×49).

Table 2. Phase II Test Results

Section		P_{fail}, kips	P_{ult}, kips	P_{fail}/P_{ult}
W10x33	NA	146.6	73.6	1.99
W12x40	NA	101.5	73.6	1.38
W14x61	NA	105.3	73.6	1.43
W10x33	TA	95.5	73.6	1.30
W12x40	TA	80.5	73.6	1.09
W14x61	TA	120.3	73.6	1.63
W10x33	TA-R	111.3	73.6	1.51
W12x40	TA-R	112.0	73.6	1.52
W14x61*	TA-R	48.9	—	—
W10x33	TA-WAA	83.4	114.4	0.73
W12x40	TA-WAA	85.0	114.4	0.74
W14x61	TA-WAA	90.2	114.4	0.79
W10x33	TA-R-W	45.1	73.6	0.61
W12x40	TA-R-W	51.1	73.6	0.69
W14x61	TA-R-W	45.1	73.6	0.61
W14x61	TA-FLA	57.9	55.2	1.05

NA=No erection angle installed
TA=Top angle installed
R =Weld return of 1/2-in. (13 mm) 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_{FAIL}=Test failure load, kips (kN)
P_{ULT}=Calculated ultimate design load, kips (kN)
*Test terminated prior to failure due to equipment malfunction

Note: 1. All welds are 1/4-in. (6.4 mm) E70 fillet welds except for W14x61 (W360X91) TA-FLA, which was 3/16-in. (4.8 mm) fillet.

Test Program

Six more tests were conducted, some with the seat extending beyond the stiffener and one with a much longer stiffener than had previously been tested. Since one of the concerns was excessive seat rotation in the web, it was felt that a longer stiffener would reduce this rotation. Table 3 shows the test dimensions and Table 4 shows the results.

Evaluation of Results

A curious situation developed during the testing of the specimens used in Phase III. As always, the researchers had supplied drawings to a steel fabricator who made the specimens and donated them free of charge to the University. During testing, the usual seat rotations occurred, but unlike all the Phase II tests, a failure in the weld was never achieved.

All the specimens were loaded to 200 kips, which was the capacity of the actuator, with no failure. The location of the

Table 3. Seat Dimensions for Phase III Tests (refer to Figure 1 for nomenclature)			
Test No.	Seat Thickness t_s, in.	Stiffener Length, L, in.	W', in.
1	3/8	8	W
2	3/8	16	W
3	3/8	8	7
4	1/2	8	7
5	3/4	8	7
6	3/8	8	W

All columns are W14x61
All stiffeners are 3/8-in. thick
W is 4 in.
B is 6 in.

Table 4. Phase III Test Results Compared with AISC Calculated Values			
Test	P_{Test}, kips	$P_{Calc.}$, kips	P_{Test} / $P_{Calc.}$
1	133.1**	102.6	1.30
2	177.2*	302.2	0.59
3	147.6	65.0	2.27
4	148.7	65.0	2.29
5	182.5	65.0	2.81
6	166.9	102.6	1.63

* =Stiffener buckled at 130 kips.
**=No top angle used.
P_{test}=Failure load from tests.
$P_{calc.}$=Calculated weld failure load according to AISC procedure. (actual values with 3/8-in. and/or 5/16-in. welds)

actuator had been chosen as one-quarter of the beam span away from the seat to simulate the proper end slope for a typical floor beam (see Figure 4). This meant that the actual maximum load on the seat was 150 kips.

After the first round of tests produced no weld failure, a careful examination of the welds revealed that they were much larger than the ¼-in. welds called for on the drawings. Fillet welds were found to be 5⁄16-in. to ⅜-in. and some were not symmetrical. It was decided to retest all the specimens by moving the actuator closer to the seat. However, doing so would mean that the beam end rotation would be significantly reduced. To counter this, the far end of the beam was lowered to produce the same end slope at the seat as the earlier tests. The tests were then all repeated and loaded until the welds failed.

The extended plates, Tests 3, 4, and 5, performed about the same as those in which the seat was flush with the stiffener. The P_{test}/P_{calc} values shown in Table 4 for these tests seem to be much higher than the rest, but this is influenced by the larger eccentricity. Since the bolts were farther from the column web, beyond the end of the stiffener, the eccentricity, e, was taken as $0.8W'$. This makes P_{calc} much smaller and increases the ratio of P_{test}/P_{calc}.

Tests 3, 4 and 5 varied only in the seat thickness, which was ⅜, ½, and ¾, respectively. Failure loads for 3 and 4 were almost identical, while 5 was about 23 percent higher. Based on this limited data, it seems that neither increasing the seat thickness nor extending the seat beyond the end of the stiffener makes a significant difference in the ultimate strength of the connection.

Test 6 was identical to Test 1, except that 6 had a 4× 4 × ¼ top angle and test 1 had none. The presence of a top angle, generally added only for torsional stability, does improve the ultimate capacity.

Specimen 2 was made with a 16-in. long stiffener, whereas all the others had 8 in., to see if this would reduce the seat rotation. It did indeed reduce rotation in the web, but the stiffener buckled at 130 kips, then loading was continued until the weld failed at 177 kips. A picture of this buckled stiffener can be seen in Figure 7. An obvious solution to this problem would be to increase the stiffener thickness. According to Table 9-9 in Volume II of the AISC Manual, the capacity of this connection on the flange with a 5⁄16-in. fillet weld is 209 kips. The stiffener *thickness* is *not* a variable in these tables.

BEAM WEB YIELDING

Because the tests reported herein were taken to failure, very large web and seat rotations occurred. In fact, the slope of the seat was much greater than the slope of the beam end. [See again Figure 2(b)]. This meant that the bearing surface was essentially reduced to the area between the bolt and the beam end. As a result of this high concentration of force at the beam end, web yielding occurred in the loading beam.

One could always use a web stiffener on the beams that are attached by stiffened seats to column webs to prevent web yielding. However, in these tests, no web yielding was observed at service loads and even at ultimate load the yielding of the web at the beam end did not seem to affect the ultimate capacity of the seat. In fact, we continued to use the same loading beam with the "wrinkled" end on the remaining tests.

Fig. 7. Buckled stiffener in Test 2 of Phase III.

This raises a question of just how serious a limit state web yielding is in this context. In a well-braced floor system, it will probably not lead to collapse nor even loss in load capacity, since the beam end slope will conform more closely to the slope of the seat. About the only consequence could be a slight vertical deflection at the end of the beam. Perhaps, *for this application*, beam web yielding should be considered a *serviceability* limit state.

COLUMN WEB DEFORMATIONS

Up to this point, this paper has only discussed *strength* criteria, either weld failure or web yielding. However, it is apparent that there should be some limitations on the rotation of the seat on the column web, especially if the seat is only on one side. Just as the design of a simple beam may be governed by strength or deflections, the design of a stiffened seat attached to a column web should have some similar serviceability requirement.

As has already been shown, large rotations occur quite early in the loading sequence, well below the failure load. These rotations do not appear to affect the ultimate strength of the seat and so should not be considered a *strength* limit state. As a serviceability limit state, an old question is encountered: how much is too much? What limiting rotation is appropriate? It has been suggested that the seat rotation should be limited to the rotation at the end of the beam which rests on it, but that seems to be unnecessarily punitive. That stage is reached very early in the loading sequence. The test specimen shown in Figure 6(b) has not yet failed, but there is a sizeable rotation apparent.

There are two factors at work here that make the situation far better than it appears. First, the eccentricity of the load gets smaller as the seat rotates on the web, thereby reducing the moment on the seat. Second, the column web, after its initial deformation actually gets stiffer as it begins to pick up membrane tension. In the University of Florida tests, LVDT's were used to measure the deformations in the web. These can be seen in Figures 4 and 6(b). A typical load-deflection curve for deformation at the base of the stiffener is shown in Figure 8.

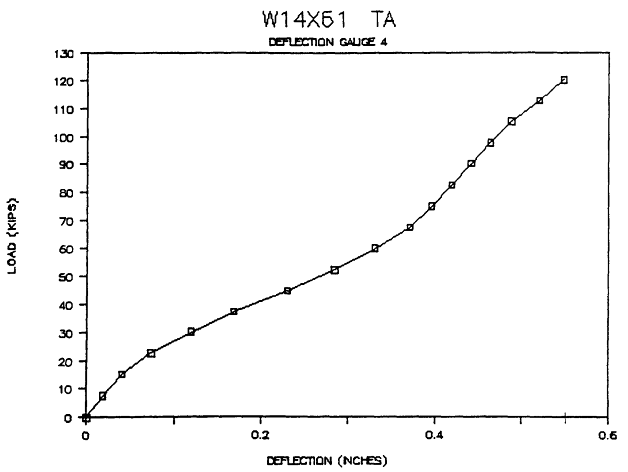


Fig. 8. Typical load vs. deflection curve measured at the base of the stiffener.

SUMMARY AND CONCLUSIONS

The purpose of this study was to examine the behavior of a stiffened beam seat when it is attached to a column web rather than the flange and to make design recommendations. AISC has a procedure for calculating the capacity of stiffened seats attached to the flange and tables to aid the designer. A secondary purpose of this research was to see if the same tables were also applicable for the case of the web attachment.

Nearly fifty full-scale tests were performed at the University of Florida on stiffened seats welded to various column webs. Two failure modes were identified: weld failure and column web yielding. As a result of this research, it was determined that the tables in Volume II of the AISC Manual, which are based on weld strength only, are appropriate for the web-attached seat, with a few exceptions noted at the bottom of the tables. These tables are, of course, still valid for the flange-attached seats. For the case where stiffened seats are attached on opposite sides of the column web at the same elevation, the tabulated values for the flange-attached seats can be used.

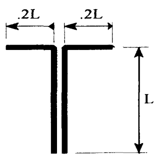
If a designer chooses to use a "beam" section as a column, not an uncommon practice, the yield-line procedure must be checked also. A yield-line analysis procedure is presented herein. Examples 3 and 4 show this calculation. Example 3 is for a typical "column" section, while Example 4 shows the procedure for a "beam" type section.

Whether checking the weld strength or the yielding failure of the web, one critical issue is the magnitude of the eccentricity, e. As the seat rotates under the load, the point of load application migrates toward the column web and the eccentricity is reduced.

In Phase I the eccentricity, e, was calculated by working backwards from the failure load and using the beam-column interaction equation. Values so calculated are shown in Table 1. In Phase II none of the tested specimens failed as a beam-column, so e was simply estimated from test observations. It was assumed that the point of action of the load at failure was one-quarter of the stiffener width plus ¼-in.

Example 1. Stiffened seat attached to column flange.

The strength of this connection is based on the weld capacity in combined shear and tension. This is the procedure that is used to develop the load tables in Volume II of the AISC LRFD *Manual of Steel Construction,* Vol. II, pp. 9-145, 146. In this method the weld is treated as a line element for the purpose of generating section properties and the weld beneath the seat is taken as the minimum recommended length of $0.2L$.



$$\bar{y} = \frac{2L \times L/2}{2L + 0.4L} = \frac{L}{2.4}$$

$$I_x = \frac{2L^3}{12} + .4L\left(\frac{L}{2.4}\right)^2 + 2L\left(\frac{L}{2} - \frac{L}{2.4}\right)^2 = \frac{L^3}{4}$$

$$S_x = \frac{I_x}{\bar{y}} = \frac{L^3/4}{L/2.4} = 0.6L^2$$

Let P_u be the factored shear load on the seat and R_{nw} the nominal resistance of the weld.

$$\phi R_{nw} = \sqrt{\left(\frac{P_u e}{S_x}\right)^2 + \left(\frac{P_u}{2.4L}\right)^2} \qquad (1)$$

Substituting $S_x = 0.6L^2$ into the above leads to:

$$P_u = \frac{2.4L^2\,\phi R_{nw}}{\sqrt{16e^2 + L^2}} \qquad (2)$$

Now let $L = 8$, $\phi R_{nw} = 6.975$ k/in., $W = 6$, $B = 6$, $t_s = 3/8$, and $t_w = 5/16$ and in keeping with AISC practice, take eccentricity, e, as $0.8W$.

Substituting these values into Equation (2) gives $P_u = 51.4$ kips, which is the number found in the table on page 9-145 of Volume II.

Example 2. Stiffened seat attached to column web.

Note that in Example 1 the eccentricity, e, was taken as $0.8W$, which is what AISC recommends. It has been shown in this paper, however, that the eccentricity is *reduced* when the seat is on the web. (Refer again to Figure 2.) It is difficult to establish a specific number for this eccentricity, because it varies with load. Based on the test results of Phases I and II, the eccentricity when the seat is attached to the web can be taken as:

$$e = \frac{B'}{2} + \frac{1}{4}\text{ in.}$$

where $B' = \frac{W}{2}$ or $2\frac{5}{8}$ in., whichever is larger

The same welding pattern will be assumed as in Example 1, so nothing changes except eccentricity. (Incidentally, the $0.2L$ minimum weld recommended by AISC cannot be met if the stiffener length, L, is more than $2\frac{1}{2}$ times the seat width, B).

$$B' = \frac{6}{2} = 3\text{ in.} > 2\tfrac{5}{8}\text{ in.} \quad \text{Use } B' = 3\text{ in.}$$

$$e = \frac{B'}{2} + \frac{1}{4}\text{ in.} = \frac{3}{2} + \frac{1}{4}\text{ in.} = 1.75\text{ in.}$$

$$P_u = \frac{2.4L^2\,\phi R_{nw}}{\sqrt{16e^2 + L^2}} = \frac{2.4\,(8)^2\,6.96}{\sqrt{16(1.75)^2 + (8)^2}} = 100.6\text{ k}$$

Note that the strength of this connection has been about doubled over that of the same seat attached to the flange. This is a direct result of the reduced eccentricity. However, the web must also be checked for yield line formation.

Example 3. Yield-line analysis

Assume column is W14×61
$F_y = 50$ ksi, $F_u = 70$ ksi
$t_w = 0.375$ in., $T = 11$ in.

$$A = \frac{2}{2T - B} = \frac{2}{2 \times 11 - 6} = \frac{1}{8}$$

$$C = 2 + (0.866T/L) = 2 + (0.866 \times \frac{11}{8}) = 3.191$$

$$D = \sqrt{(T - B)(3T + B)} = \sqrt{(11 - 6)(3 \times 11 + 6)} = 13.96$$

$$E = \frac{T(T - B)}{2L} = \frac{11(11 - 6)}{2 \times 8} = 3.438$$

$$G = 4L + 3.464\,T = 4 \times 8 + 3.464 \times 11 = 70.10$$

$$k = A(CD + E + G) = \tfrac{1}{8}(3.191 \times 13.96 + 3.438 + 70.1) = 14.76$$

$$F^* = F_y + \frac{2}{3}(F_u - F_y) = 50 + \frac{2}{3}(70 - 50) = 63.3\text{ ksi.}$$

$$m = \frac{t_w^2 F^*}{4} = \frac{(0.375)^2 \times 63.3}{4} = 2.225$$

Let eccentricity, e, be 1.75 in.

$$P_n = \frac{k\,L\,m}{e} = \frac{14.76 \times 8 \times 2.225}{1.75} = \underline{150\text{ kips}}$$

Example 4. Yield-line analysis with a non-typical column section.

Assume column is W16×31
$F_y = 50$ ksi, F_u=70 ksi
t_w=0.275 in., T=13.625 in.

Stiffened seat and weld are the same as in previous examples.

$$A = \frac{2}{2T - B} = \frac{2}{2 \times 13.625 - 6} = 0.0941$$

$$C = 2 + (0.866T/L) = 2 + (0.866 \times 13.625/8) = 3.475$$

$$D = \sqrt{(T - B)(3T + B)} = \sqrt{(13.625 - 6)(3 \times 13.625 + 6)} = 18.91$$

$$E = \frac{T(T-B)}{2L} = \frac{13.625(13.625-6)}{2 \times 8} = 6.493$$

$$G = 4L + 3.464\,T = 4 \times 8 + 3.464 \times 13.625 = 79.20$$

$$k = A(CD + E + G) = 0.0941$$
$$(3.475 \times 18.91 + 6.493 + 79.2) = 14.25$$

$$m = \frac{t_w^2 F^*}{4} = \frac{(0.275)^2 \times 63.3}{4} = 1.197$$

$$P_n = \frac{k\,L\,m}{e} = \frac{14.25 \times 8 \times 1.197}{1.75} = 78.0 \text{ kips}$$

Note that weld strength is 105 kips

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