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SEISMIC STRUCTURAL DESIGN ASSOCIATES, INC.

RATIONALE FOR THE DESIGN OF SLOTTED BEAM -TO - COLUMN CONNECTIONS IN STEEL MOMENT FRAMES

This rationale is based upon:

- **The IBC and AISC/LRFD Specifications and the principles of plastic design: the elements of the connection are proportioned on the basis of their maximum strength.**

- **ATC-24 protocol test results using beams ranging from W24x94 to W36x393 and columns ranging from W14x176 to W14x550 and W27x307 to W30x235.**

- **Inelastic finite element analyses of the stress and strain distributions and buckling modes.**

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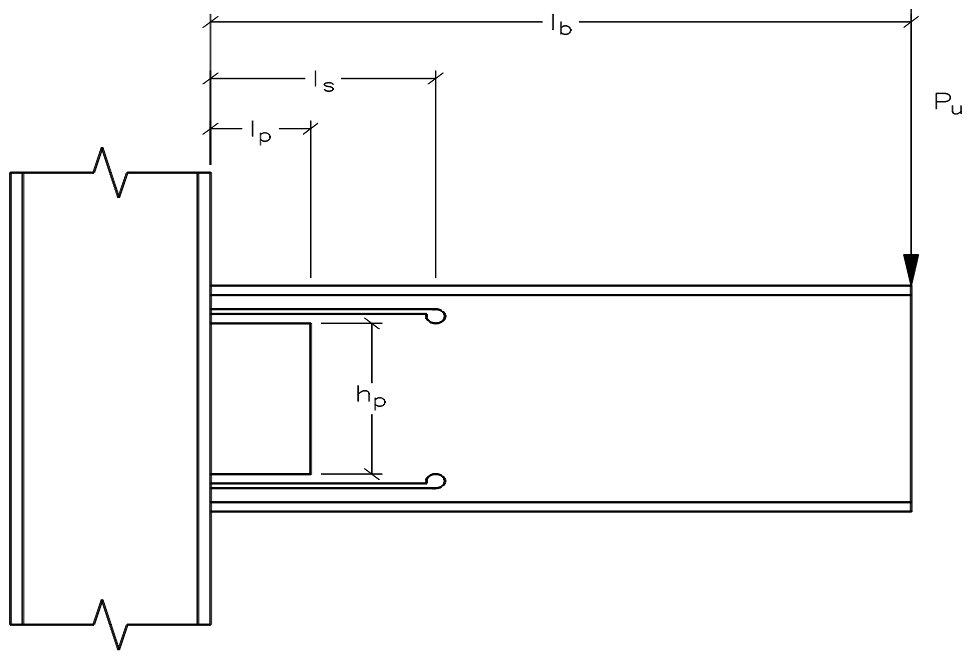
Slot and Shear Plate Designs for Slotted Beams



ATC-24 Test Connection Details

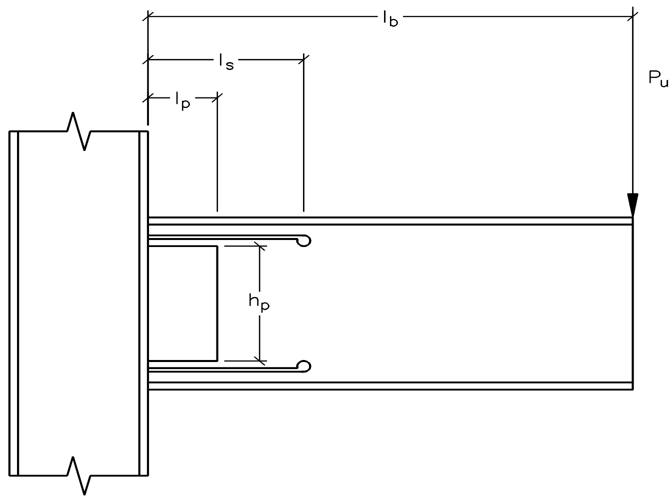
showing beam length, slot length, and shear plate height and width

Beam Slot Designs

- **Beam slots are designed to:**
  - » **Force the beam shear at the connection to be carried by the beam web which greatly reduces the beam flange shear.**
  - » **Provide a nearly uniform stress and strain distribution horizontally across and vertically through the beam flanges from the column face to the end of the beam slot**
  - » **Allow plastic beam flange and beam web buckling to occur independently in the region of the slot. This eliminates the lateral-torsional mode of buckling of non-slotted beams.**
  - » **Insure plastic beam flange buckling so that the full plastic moment capacity of the beam is developed:**

$$l_s / t_f \leq 0.60 \times (E/F_y)^{1/2}$$

Beam Slot Length Designs (Continued)



- **Beam slot length is equal to the <u>shortest</u> of the following:**

1) l_s = 1.5 x (Nominal Beam Flange Width)
2) $l_s = 0.60 \times t_f \times (E/F_y)^{1/2}$
3) l_s = 1/2 x (Nominal Beam Depth)
4) $l_s = l_p + (l_b - l_p)/10$

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Beam Slot Length Computation

Example: **For a W36x194 with b_f = 12.12", t_f = 1.26", l_p= 6", l_b= 168" and Grade 50 steel, use F_y= 55 ksi :**

l_s = 1.5 x 12" = 18" or

l_s = 0.60 x 1.26 x $(29000/55)^{1/2}$ = 17.3" or

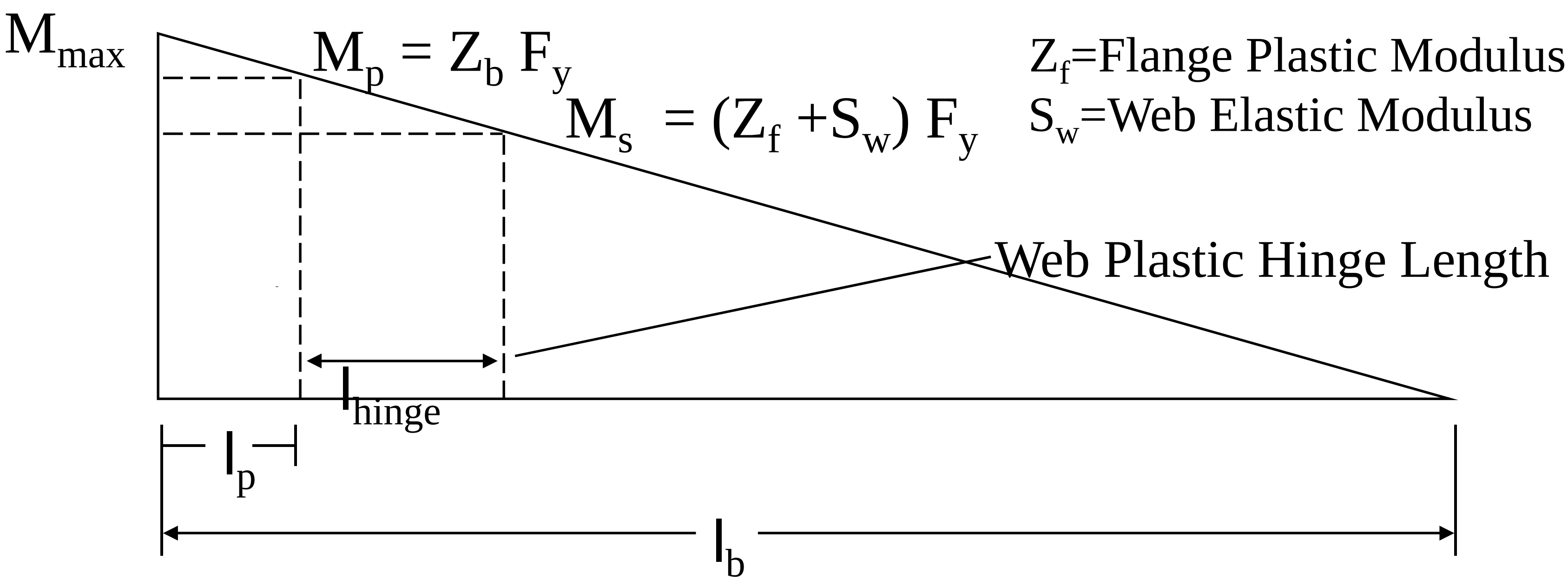
l_s = 1/2 x 36" = 18" or

l_s = 6" + (168" - 6")/10 = 22.2"

USE SLOT LENGTH = 17.5"

Length of Beam Web Plastic Hinge

- **Use the ATC-24 moment diagram to compute the beam <u>web</u> plastic hinge length measured from end of the shear plate.**



With $(Z_f + S_w) = (Z_b - Z_w/3)$ where $Z_w = (Z_b - Z_f)$ then,

$$l_{hinge} = (l_b - l_p)\,[(Z_b - Z_f)/(3 Z_b)] = (l_b - l_p)/10 \text{ for } Z_f/Z_b = 0.70$$

In the special cases of short spans, typically less than 15', and with deep beams, the slot length is limited to the length of the shear plate plus the web plastic hinge length, that is:

$$l_s = l_p + (l_b - l_p)/10$$

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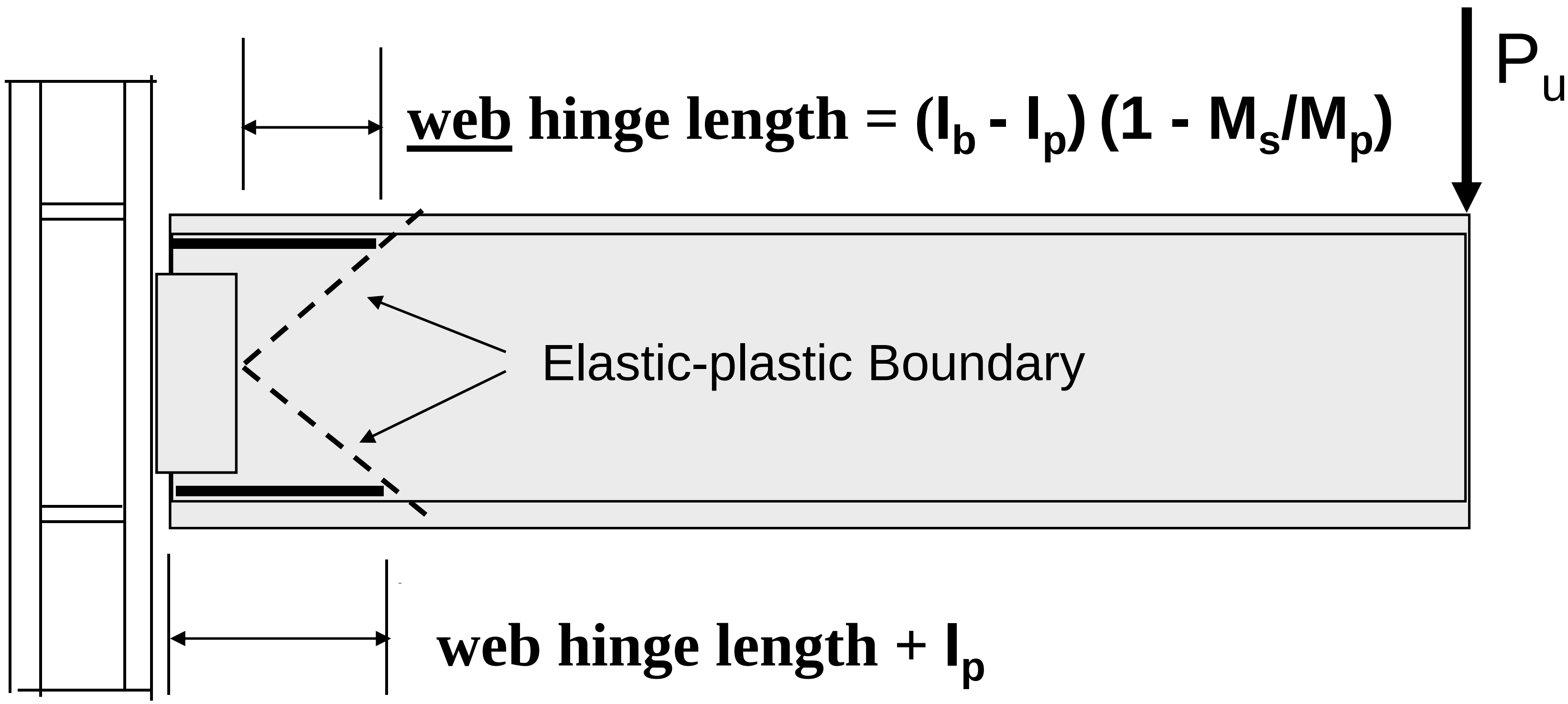
BEAM WEB PLASTIC HINGE LENGTH



The location of the centroid of the plastic hinge in the beam is located at the outboard edge of the shear plate.

Beam Slot Designs (Continued)

- **Beam Slot Location and Width**

  » **Slot is cut in the beam web at the toe of the beam flange fillet (along the “k” line) from the end of the beam to a drilled 1-1/16” slot termination hole.**

  » **Slot width = 1/8” to 1/4”**

Effect of Beam Slots on Connection Stiffness

- **Finite Element Analyses using high fidelity models of the ATC-24 test assemblies have shown that the beam slots do not change the stiffness or elastic force-deflection behavior of these assembles.**

- **Therefore standard finite element/structural programs may be used to design steel frames subjected to static and seismic loading when slotted beams are used**

Shear Plate Width and Height Design

- **Plate width, l_p, is typically 4" to 6"**
- **Use the maximum height that allows plate weldment and beam web slots.**

Typically, the plate height,

$$h_p = T - 2" \quad \text{(T from the AISC Manual)}$$

Example:

For a W36x194 beam, T= 32 1/8"

Use h_p = 32 1/8 - 2 = 30"

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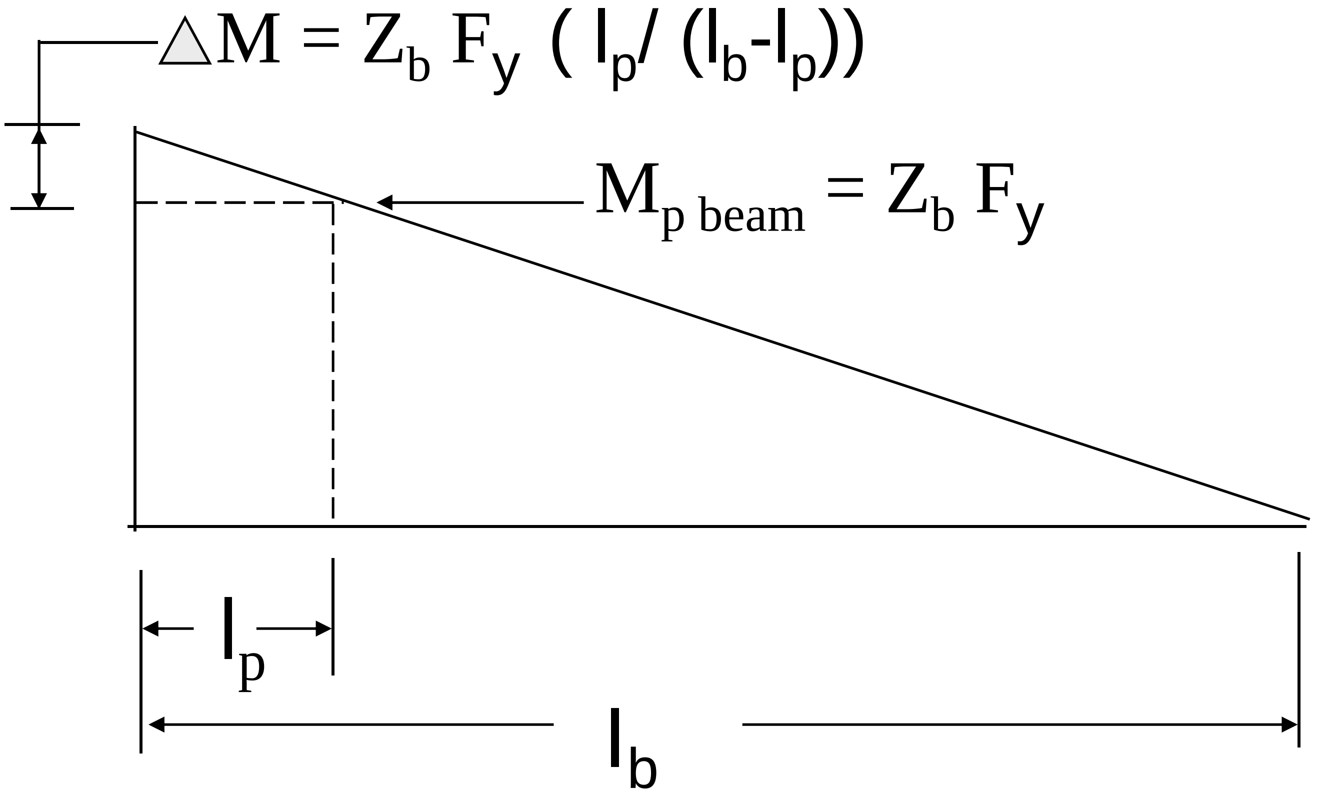
Shear Plate Thickness Design

- **Develop required plate strength at the column face using ATC-24 moment diagram**



Shear Plate Thickness Design (Continued)

- **At the face of the column the shear plate of height h_p must resist moment ΔM at the column face.**
- **Determine minimum plate thickness required based upon (conservatively) its <u>elastic</u> section modulus.**

$$t_{p\ req'd} = 6/h_p^2[\ Z_b I_p/(I_b - I_p)] \qquad \text{(for Grade 50 steel)}$$

- **Use this shear plate thickness to design the shear plate weld to the beam web.**
- **The shop welded shear plate is the backing for the field groove beam web-to-column flange weld and stabilizes the beam web from buckling near the column flange.**

Shear Plate Thickness Design (Continued)

Example using a Grade 50 Shear Plate:

For a W36x194 with l_b= 168", l_p= 6"

Z_b = 767 in^3, h_p= 30", t_{web} = 0.765";

$t_{p\ req'd} = 6/30^2[\ 767x6/(168-6)] = 0.189"$

= t_p in the following design eqs

Note: l_b = (beam clear span)/2

Shear Plate to Beam Web Weld Design

- **Even though the beam web is welded to the column flange, design the shear plate weld to develop the ratio of $t_p/(t_w+t_p)$ of the beam web moment and shear <u>at the face of the column</u>. This design moment is greater than △M and keeps the web plastic hinge outboard of the shear plate.**

$$M_{plate} = t_p/(t_w + t_p)\ [\ Z_{web}\ F_y]\ \text{with}\ \ Z_{web} = t_w T^2/4$$

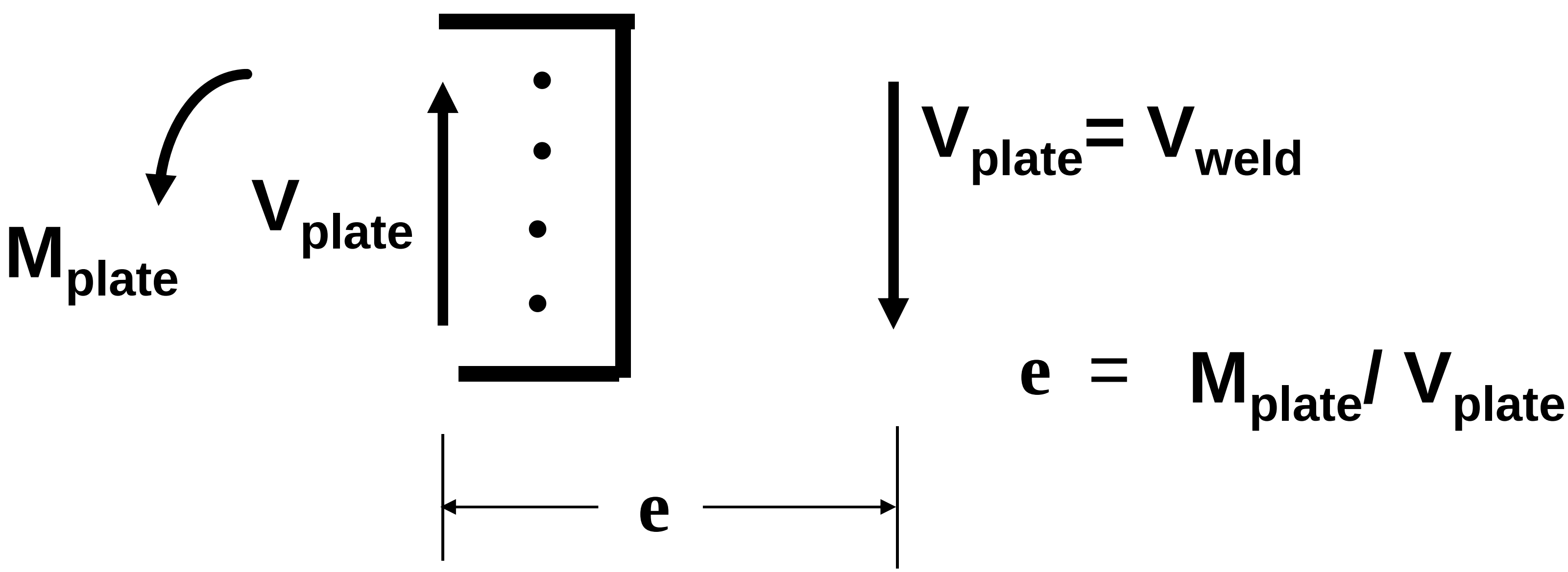
$$V_{beam} = Z_b\ F_y\ /(l_b - l_p)\ +\ [\ 1.2\ D + 0.5\ L]/\ 2$$

$$V_{plate} = t_p/(t_w + t_p)\ [\ V_{beam}\ ]$$

Use $F_y = 1.1 \times F_{ye}$ to include strain hardening and expected yield

Shear Plate Weld Design (Continued)

- **Use AISC LRFD Table 8-43 to design the "C" shear plate weld.**
- **Use fully tensioned high strength erection bolts.**



Shear Plate Weld Design (Continued)

Example: W36x194 with l_b= 168”, l_p= 6”, Z_{web} = 197 in^3, h_p=30”, t_w=0.765”, F_{ye}= 55ksi, and D=72k and L=160k

- With $t_{p\ req'd} = t_p$ = 0.189” then

  $t_p/(t_p + t_w)$ = 0.189/(0.189+ 0.765) = 0.198

Shear plate moment and shear:

- M_{plate} = 0.198 (197) (1.1x 55) = 2360 in-k

  V_{beam}= 767 (1.1 x 55) /(168-6)

  + [1.2(72) + 0.5(160)] / 2 = 370 k

- V_{plate} = 0.198 (370) = 73.3 k

  then e = M_{plate} / V_{plate} = 32.2”

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Shear Plate Weld Design (Continued)

- **For this W36x194 beam, use 7/8" A325 fully tensioned erection bolts at 6" o-c.**
- **Design the shear plate "C" fillet weld to resist all of the plate moment and shear.**
- **<u>The Weld Design</u>**

  V_{weld} = 73.3 k and e = 32.2"

From LRFD Table 8-43 with a = 1.07, k = 0.20; C= 0.694

D = 73.3 / (.694 x 30) = 3.52 Use a 5/16" fillet weld (see note below)

- **Select a 5/8" shear plate with 1/8" land for a PJP shear plate-to-column flange weld. This PJP weld matches the shear plate-to-beam web fillet weld strength.**

Note: To stabilize the beam web at the column flange use at least a minimum plate thickness of 2/3 beam web thickness or $t_{p\ req'd}$ plus 1/4" with a minimum 5/16" plate to beam web fillet weld.

Commentary on the Shear Plate Design

The shear plate and welds as designed herein provide the additional connection strength to keep the centroid of web plastic hinging region, verified by both the finite element analyses and the ATC-24 tests, outboard of the shear plate.

Moreover, the further recommended minimum plate thickness of 2/3 the beam web thickness or the required plate thickness plus 1/4” is to provide lateral beam web stiffness only, not additional strength, to the connection at the column face.

Check Beam Shear Capacity

- **Check the Design Example W36x194 Beam:**

  Beam Shear V_u= 370 kips

- **Use AISC *Specifications* Beam Shear Strength Equation to check the beam shear strength for $V_n \geq V_{beam}$**

$$V_n = 0.60\ F_y\ A_{web}$$

with $A_{web} = T \times t_w = A_{slotted\ web}$

- $V_n = 0.60 \times F_y\ T\ t_w$

  $= 0.60 \times 55 \times 32.12 \times 0.765$

  = 810 kips > 370 kips OK

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Seismic Stress Concentration and Ductility Demand Factors

- **Baseline Connection:**
  - » **CJP Beam-to-Column Welds**
  - » **No Continuity Plates**
- **Slotted Connection:**
  - » **CJP Beam-to-Column Welds**
  - » **Beam Slots and Continuity Plates as required**
- **Seismic Stress Concentration Factor (SCF):**
  - » **SCF = Computed Elastic Stress/Yield Stress**
- **Ductility Demand Factor (DDF):**
  - » **DDF = Strain/ Yield Strain - 1 = SCF - 1**

Stress Concentration and Ductility Demand Factors (continued)

- Typical SCF for Baseline Connection = 4.6
  - » DDF = 4.6 - 1 = 3.6
- Typical SCF for a Slotted Connection = 1.4
  - » DDF = 1.4 - 1 = 0.4
- Slotted Connection Design Reduces the Flange/Weld Ductility Demand Factor by a Ratio of 3.6/0.4 = 9.

This large reduction in the connection ductility demand improves the seismic fatigue life of the slotted connection very significantly as shown by low cycle fatigue tests .

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Stress Concentration and Ductility Demand Factors