Night School 26: Developing an Eye for Connection Design

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Session 7 – Extended Shear Tabs

August 24, 2021 | Larry Muir

Smarter. Stronger. Steel.

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Course Description

Extended Shear Tabs
August 24, 2021

This session will take a deep dive into extended shear tabs and will draw heavily from a design example. Key design considerations will be reviewed including strength of a bolted connection, block shear patterns, shear lag models, rotational ductility of plate, yield line analysis and shear rupture strength of column web at connection, plate stability check, and more.

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Learning Objectives

1. Describe the different block shear patterns in a bolted connection.
2. List the different shear lag models that can be used for a bolted connection.
3. Explain how the full magnitude of the shear and axial loads being applied simultaneously may not always be the worst condition.
4. Explain why the presenter prefers only using extended shear tabs when a slab or deck is present.

Night School 26: Developing an Eye for Connection Design

Session 7: Extended Shear Tabs
August 24, 2021

Larry Muir, PE, Consultant

Smarter.
Stronger.
Steel.

Design Examples

Friends or Foes

8

Design Examples

Friends ~~or~~ and Foes

9

Full Disclosure

- I have come to believe that Design Examples are far more dangerous than they are useful.
- I regret producing virtually every design example that I have published or presented.
- Virtually every design example that I have published or presented has been misused in some manner.

10

Full Disclosure

- The *Design Examples Companion to the AISC Steel Construction Manual* states, “The design examples provide coverage of all applicable limit states, whether or not a particular limit state controls the design of the member or connection.”

- In my opinion this statement likely is not true, and even if it is true it is misleading and not useful.

11

Full Disclosure

- I will shortly engage in a pretty brutal critique of one design example.

- Though the journey will be long and arduous, I will still not address every infrequently encountered problem, which might occur in the full range of structural design.

- For the record I think the design example is pretty good.

12

Full Disclosure

My work tends to connect me to many more problems than successes. This inevitably skews my perspective.

13

Full Disclosure

- Engineers often say they want design examples that include the entire process, not just the final result.
- This is my attempt to do that.

DESIGN EXAMPLES

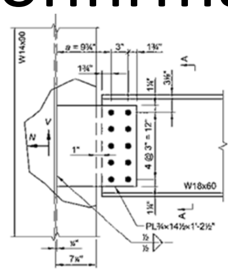
Companion to the AISC *Steel Construction Manual*

Version 15.0

15

Beware: Confirmation Bias

- The fabricator wants it
- It is economical
- It is a cool idea



- It is weird
- It is not what I specified
- I don't understand it

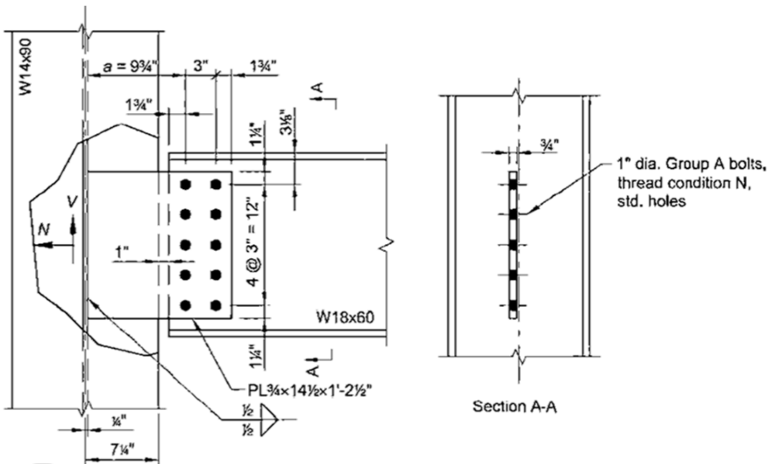
Extended tabs are cool. They are economical and solve a lot of problems. They are simple to fabricate.

Vigorous debate

Extended tabs are not structurally intuitive. They are torsionally and axially weak. There are dangers.

16

Example II.A-19B Extended Single-Plate Connection Subject to Axial and Shear Loading



Verify available strength

ASTM A992 members:

W18x60 beam

W14x90 column

70-ksi electrodes

ASTM A572 Grade 50 plate

LRFD

Shear, *Vu* = 75 kips

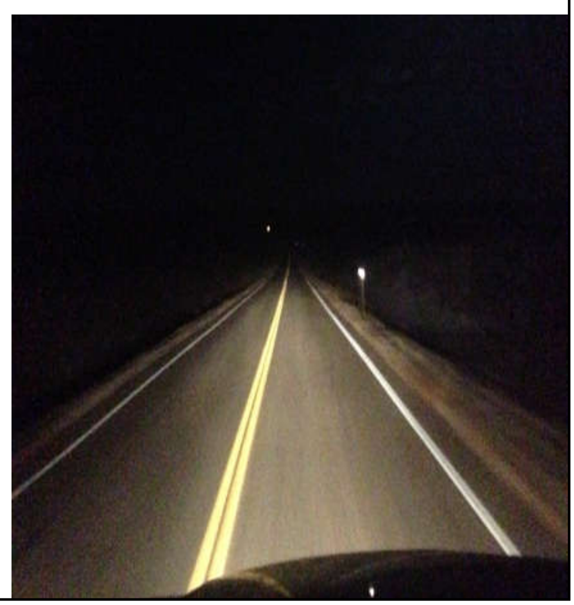
Axial tension, *Nu* = 60 kips

Putting Things Into Context

Things I did when using extended tabs:

- Only with slab or deck
  - preferably with slab.
- Not governed by buckling – $\lambda \leq 0.7$.
- Full depth
- Recognized extended shear tabs are not always the best option.

λ no longer used in Manual procedure, but you can still use it for this.

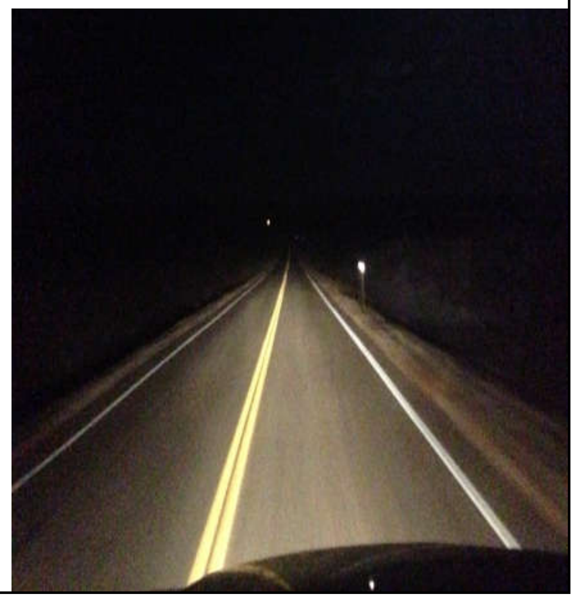


Putting Things Into Context

Most of the time steel design is easy.

Sometimes steel design is complicated.

Oftentimes the difference is not your destination, but rather the path you choose to get there. Sometimes it's easiest if you keep it between the lines.



The Plan

We will walk through this design example identifying some (but not all) of the assumptions made and examining them in an explicit manner that is unusual in practice.

In design all assumptions should be considered. Most will be evaluated "by inspection" "based on engineering judgment".

To help you follow along statements taken directly from the design example are in DARK RED.

20

Starting in the Middle

- We are designing a connection.
- We have very little information about the rest of the structure.
- In practice we would be able to see the connection in the context of the rest of the project.
- Many assumptions will have been made to get us to this point. To reflect this, we will arbitrarily call our “first” assumption “Assumption 51”.

21

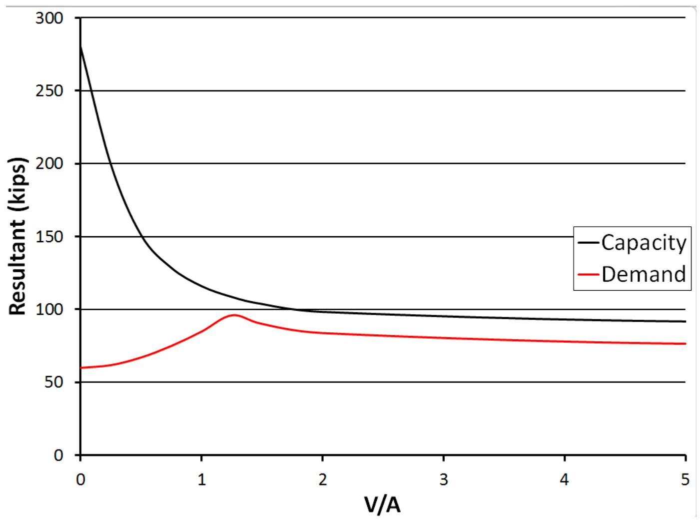
Assumption 51

When considering interaction between the beam axial and shear reactions the design example assumes that the worst case is represented by the full magnitude of the shear and axial loads being applied simultaneously.

This may not always be the worst condition.

22

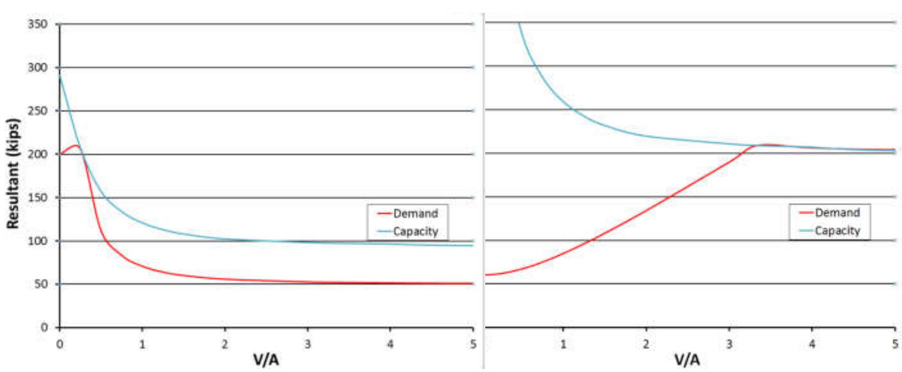
Assumption 51 – Worst Load Case



This graph shows the demand and capacity related to the bolt group for various ratios of shear-to-axial load consistent with the requirements in the example.

23

Assumption 51 – Worst Load Case



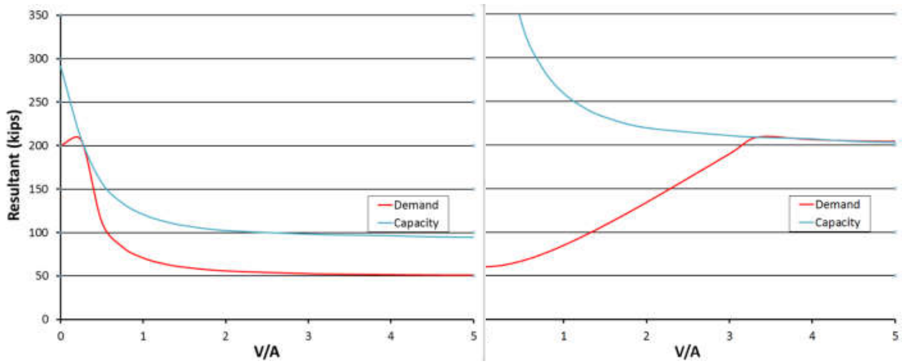
The worst case is represented by the full magnitude of the shear and axial loads being applied simultaneously over the full range of possibilities.

High Max. Axial
Low Max. Shear

High Max. Shear
Low Max. Axial

24

Assumption 51 – Worst Load Case



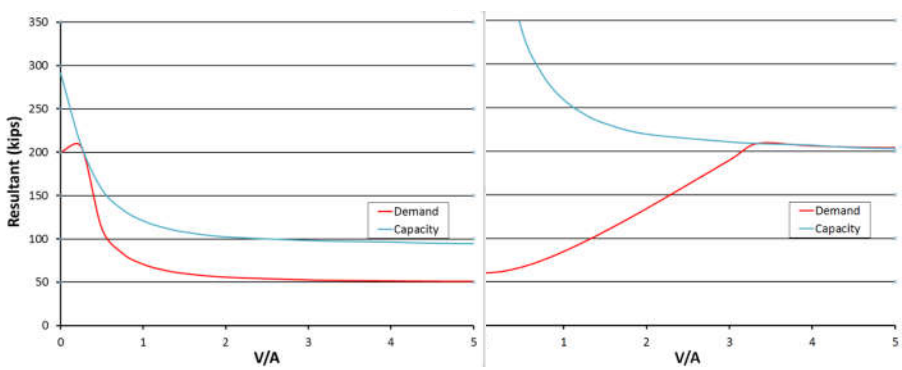
If this is all obvious to you, then there should never be a need to perform such analysis – unless you need to prove it to a reviewer.

High Max. Axial
Low Max. Shear

High Max. Shear
Low Max. Axial

25

Assumption 51 – Worst Load Case



Will this sort of relationship always apply?

What if there was a directional strength increase???

High Max. Axial
Low Max. Shear

High Max. Shear
Low Max. Axial

26

Assumption 52 – C-value

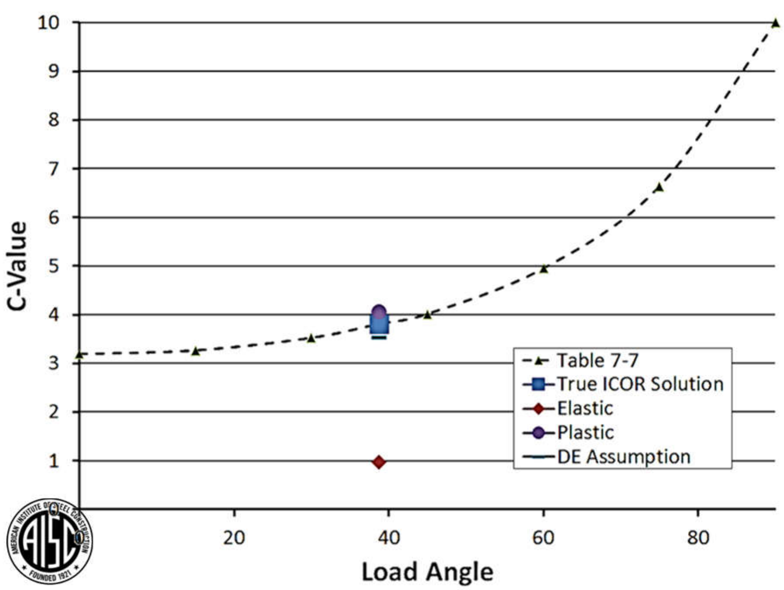
Strength of Bolted Connection—Beam Web

The strength of the bolt group is determined by interpolating AISC Manual Table 7-7 for Angle = 30° and n = 5.

Note that 30° is used conservatively in order to employ AISC Manual Table 7-7.

27

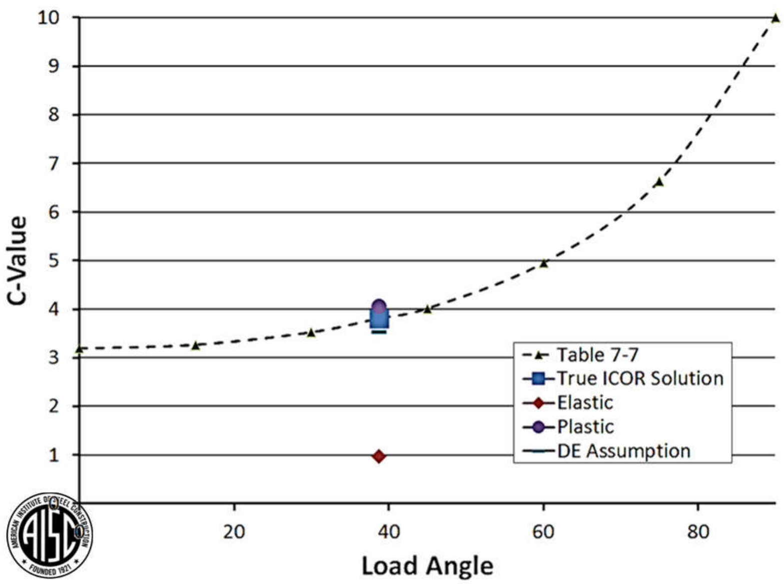
Assumption 52 – C-value



The Manual states, "Linear interpolation within a given table between adjacent values of e_x is permitted…Straight-line interpolation between values for loads at different angles may be significantly unconservative."

28

Assumption 52 – C-value



The dashed line is "straight-line interpolation between values for loads at different angles" – exactly what you are told not to do.

NOTE: For illustration purposes only. Follow the recommendations of the Manual – unless you are willing to do the work to do otherwise.

29

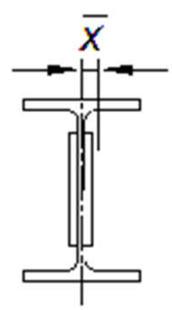
Assumption 53 – Shear Lag

Tensile Rupture Strength of Beam

From AISC *Specification* Section J4.1, determine the available tensile rupture strength of the beam. The effective net area is $A_e = A_n U$, where U is determined from AISC *Specification* Table D3.1, Case 2.

U = 0.607

ϕR_n = 447 kips > 60 kips **o.k.**



30

Assumption 53 – Shear Lag

Alternative 1

From AISC *Specification* Section D3: "For open cross sections… the shear lag factor, *U*, need not be less than the ratio of the gross area of the connected element(s) to the member gross area."

$$U = \frac{dt_w}{A} = \frac{(18.2\ in.)(0.415\ in.)}{17.6\ in.^2} = 0.429$$

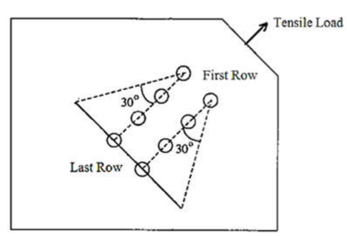
ϕR_n = 316 kips > 60 kips **o.k.**

31

Assumption 53 – Shear Lag

Alternative 2

Whitmore Section



$$L_w = 12\ in. + 2\tan(30°)\,(3\ in.) = 15.5\ in.$$

$$\phi R_n = 0.9(15.5\ in.)(0.415\ in.)(50\ ksi) = 289^k > 60^k \textbf{ o.k.}$$

32

Assumption 53 – Shear Lag

Alternative 3

I have done enough connection design that if I were designing this by hand, shear lag would be one of the last things I would check.

In the substantiating connection information I would check this as shown in the design example.

If I were reviewing shop drawings, then I probably would deem shear lag okay by inspection.

33

Assumption 54 – Whitmore Section

The design example implicitly assumes that yielding of the Whitmore section is not a concern.

I am okay with this. Why?

"The routine calculations associated with block shear and Whitmore are sufficient in practice to eliminate the consideration of any sections other than the gusset to column and gusset to beam sections."

~~~ Bill Thornton and Tom Kane
Steel Design Handbook (Tamboli 1997)

34
~~~

Assumption 55 – Block Shear

Block Shear Rupture of Beam Web

Block shear rupture is only applicable in the direction of the axial load because the beam is uncoped and the limit state is not applicable for an uncoped beam subject to vertical shear.

People can dream up all sorts of block shear failure modes. Many are pure fantasy as no similar behavior has ever been observed.

If one were to consider block shear for an uncoped beam subject to vertical shear, it would make sense to include the flange in some manner.

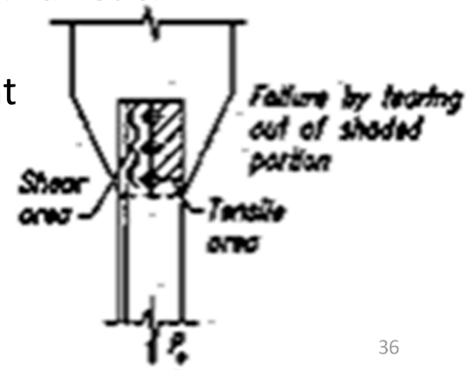
35

Assumption 55 – Block Shear

Various models of block shear have been included in the *Specification* over the years. Some assume fracture on both the tension and the shear surfaces. Some assume yield on one and fracture on the other. Some tend to be more empirical and some more mechanistic.

To my knowledge they all tend to be somewhat conservative for certain conditions.

It is what it is.

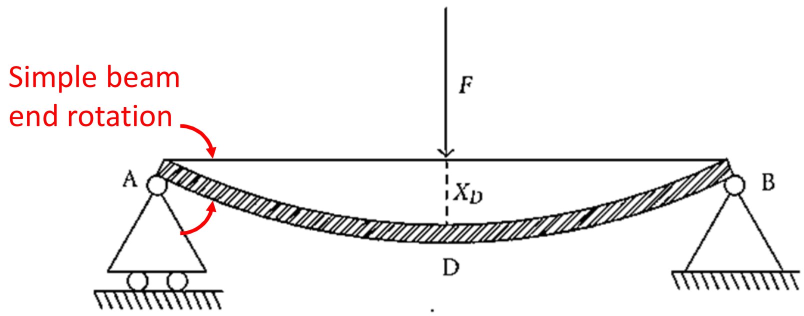


36

Assumption 56 – Rotational Ductility

Maximum Plate Thickness

Determine the maximum plate thickness, t_{max}, that will result in the plate yielding before the bolts shear.



37

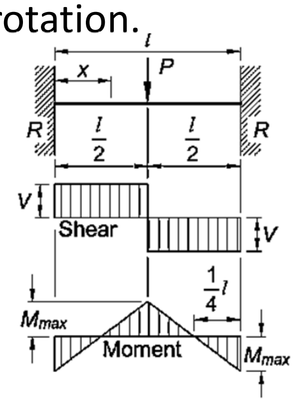
Assumption 56 – Rotational Ductility

Part 9 of the *Manual* states, "Connections satisfying the parameters discussed in the foregoing can be expected to accommodate rotations in the range of 0.03 rad." 0.03 radians is a very large end rotation.

Let's assume beam span is 18d = 27 feet.

For a non-composite steel beam:

$$\theta = \frac{PL^2}{16EI} = \frac{2(75\ kips)(324\ in.)^2}{16(29{,}000\ ksi)(984\ in.^4)} = 0.034\ rads.$$



38

Assumption 56 – Rotational Ductility

There are other means of convincing oneself that sufficient rotational ductility exists "to accommodate the required rotation determined by the analysis of the structure" as required by *Specification* Section B3.4a.

They require a good bit of judgment and can be labor intensive.

It is probably best to stick to simple and established methods whenever possible.

39

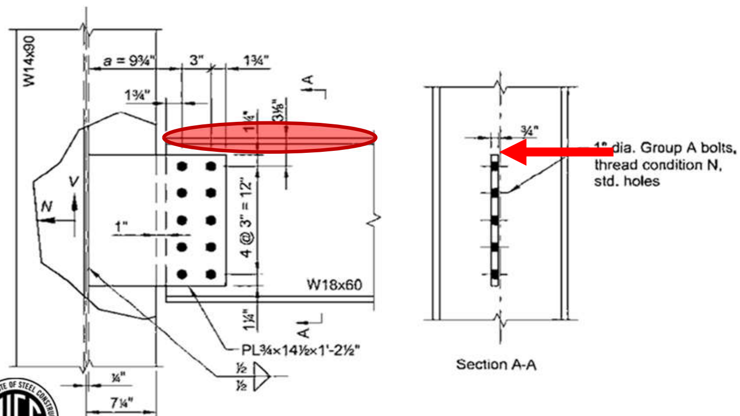
Assumption 57 – Bracing

Flexure Strength of Plate

The plate is checked for the limit state of buckling using the double-coped beam procedure as given in AISC *Manual* Part 9, where the unbraced length for lateral-torsional buckling, L_b, is taken as the distance from the first column of bolts to the supporting column web and the top cope dimension, d_{ct}, is conservatively taken as the distance from the top of the beam to the first row of bolts.

40

Assumption 57 – Bracing

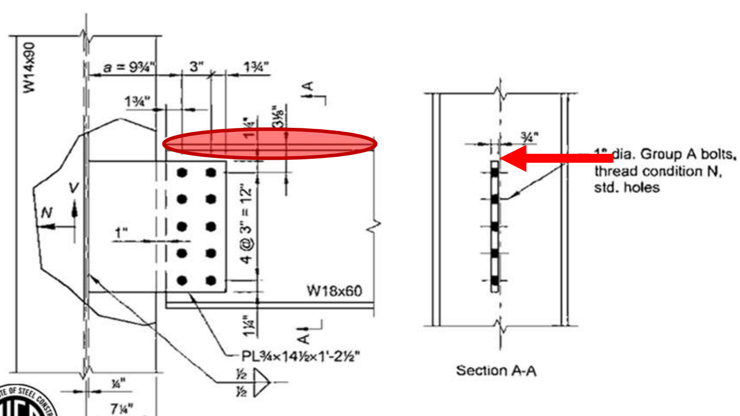


Manual Part 9: "For a coped beam laterally braced at the end of the uncoped section…"

Manual Part 10: "Ensure that the supported beam is braced at points of support."

41

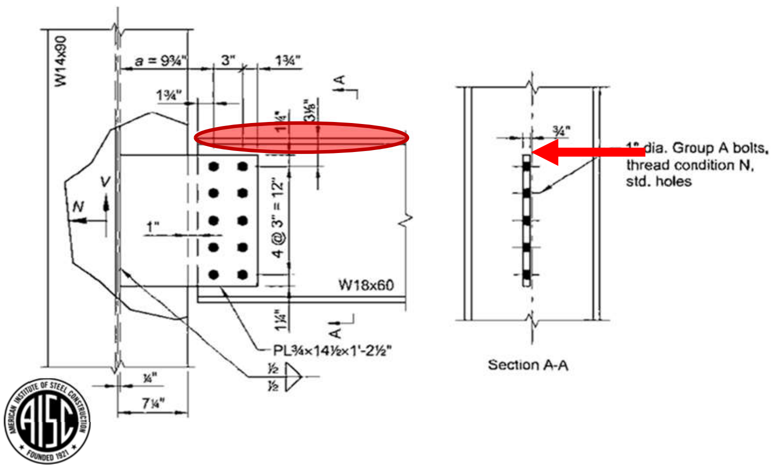
Assumption 57 – Bracing



Specification Chapter F is not applicable unless "points of support for beams and girders are restrained against rotation about their longitudinal axis".

42

Assumption 57 – Bracing



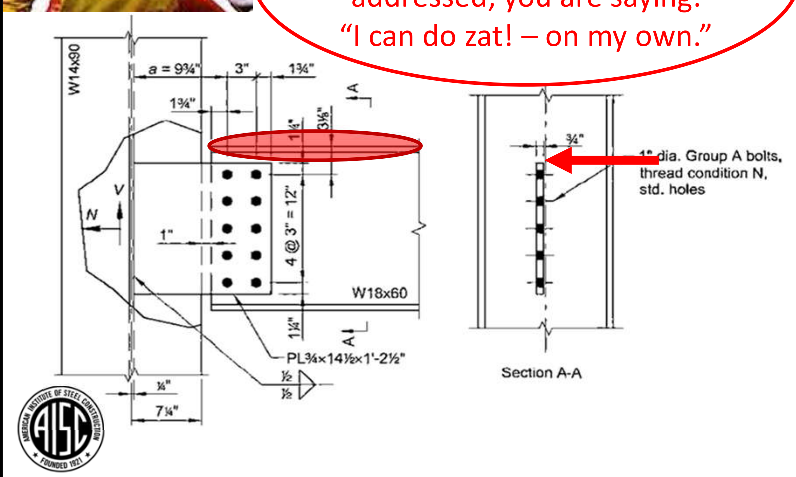
If there is not sufficient bracing then the *Manual* and *Specification* may not adequately address:

- Stability of plate
- Flexural strength of the beam

43

Assumption 57 – Bracing

When you choose to do things that are not addressed, you are saying:
"I can do zat! – on my own."



Specification Section A1:
"Where conditions are not covered… designs are permitted to be based on tests or analysis… Alternative methods of analysis and design are permitted…"

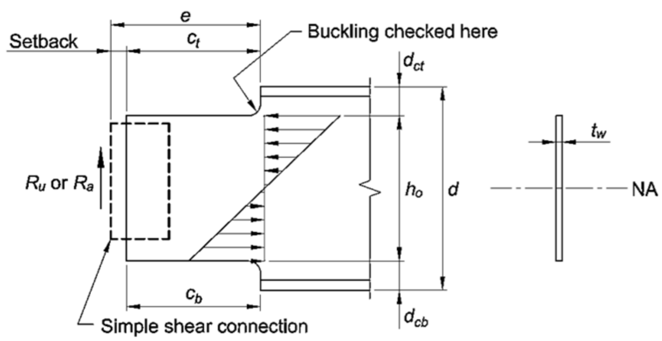
You are on your own.

44

Assumption 58 – Plate Flexure Strength

Flexure Strength of Plate

The plate is checked for the limit state of buckling using the double-coped beam procedure as given in AISC *Manual* Part 9…



45

Assumption 58 – Plate Flexure Strength

Remember Assumption 51??? When considering interaction between the beam axial and shear reactions the design example assumes that the worst case is represented by the full magnitude of the shear and axial loads being applied simultaneously.

The tension, if coexisting with the shear, will tend to reduce the tendency of the plate to buckle – in other words…

as stated previously the full magnitude of the shear and axial loads being applied simultaneously may not always be the worst condition.

46

A Digression --- Does not Compute

Assumption 51 and Assumption 58 are diametrically opposed.

If you turn this over to a computer what will it do?

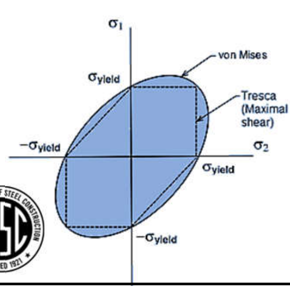
Will it check every possible combination of loads relative to every limit state that is “applicable”? How does your computer know what is applicable?

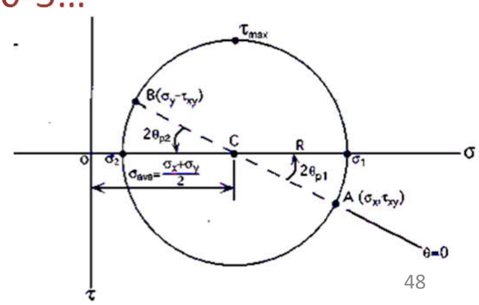
How do you know what your computer will do?

Assumption 59 – Stress Interaction

Interaction of Axial, Flexure and Shear Yielding in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjunction with AISC *Manual* Equation 10-5…





48

Assumption 59 – Stress Interaction

If I were trapped on a desert island, I would probably not use Chapter H to combine bending and axial. I might square the shear term in *Manual* Equation (9-1) instead of raising it to the fourth power.

Since $V_r / V_c \leq 0.40$ in this case, I might just neglect shear.

The plate is very stable 3,750 kip-in. >> 1,970 kip-in. and the flexural strength was governed by the plastic moment strength of the plate.

49

Assumption 59 – Stress Interaction

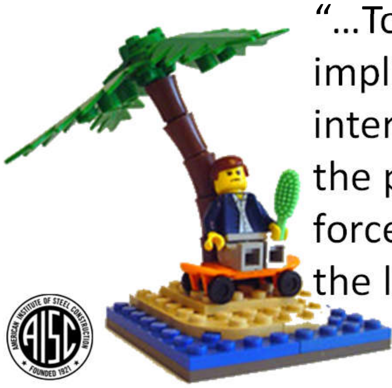
Why do we even check stress interaction for extended single-plate shear connections?

"…researchers showed that single-plate connections maintain the majority of their torsional rigidity until the plate yields, at which time… torsional rigidity of the connection decreases and the twist of the connection becomes apparent…"

50

Assumption 59 – Stress Interaction

Why do we even check stress interaction for extended single-plate shear connections?



"…To design for this effect, the limit state of twist… is implicitly checked considering the von Mises interaction of forces on the plate, which ensures that the plate does not yield in the presence of shear forces. Effects of twisting can be further mitigated by the lateral bracing of the beam when a slab is present."

~~~ Some guys named Muir & Hewitt

51

Assumption 59 – Stress Interaction

Just between us if I were trapped on a desert island, I would probably not check interaction for extended tabs at all, but remember I only use extended tabs with slab or deck - preferably with slab.

52
~~~

Assumption 60 – Tensile Rupture

Tensile Rupture Strength of Plate

AISC *Specification* Table D3.1, Case 1, applies in this case because the tension load is transmitted directly to the cross-sectional element by fasteners; therefore, U = 1.0.

53

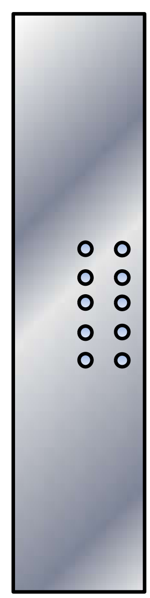
Assumption 60 – Tensile Rupture

Because the tension load is transmitted directly to the cross-sectional element by fasteners U = 1.0.

What if the plate looked like this?

Whitmore check?

What about compression?



54

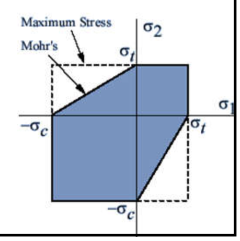
Assumption 61 – Rupture Interaction

Interaction of Axial, Flexure and Shear Rupture in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjunction with AISC *Manual* Equation 10-5…

To my knowledge there is little (and possibly no) rational basis for this check.

Not saying this is bad… just sayin'…

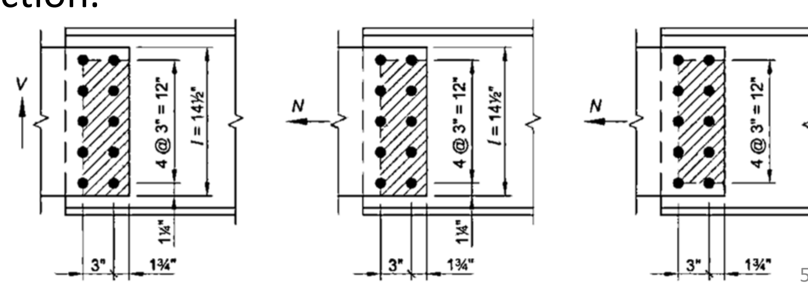


Assumption 62 – More Block Shear

See Assumptions 55 and 61.

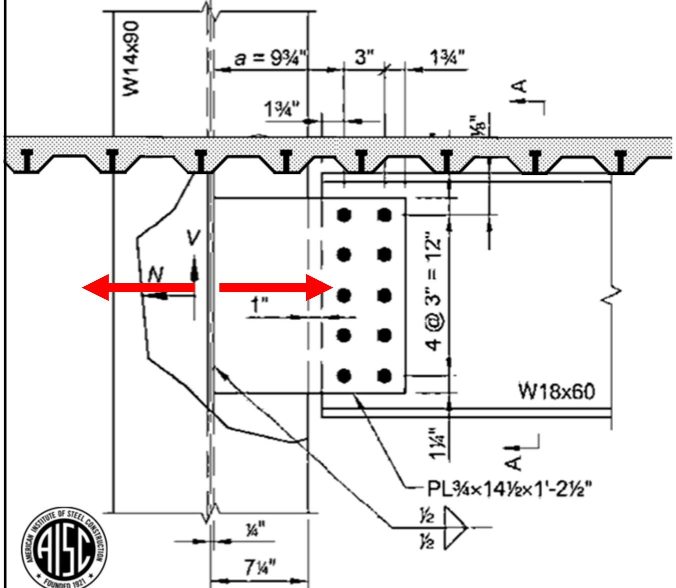
Three pages of calculations for: 0.137 < 1 **o.k.**

Alternative: Okay by inspection.



56

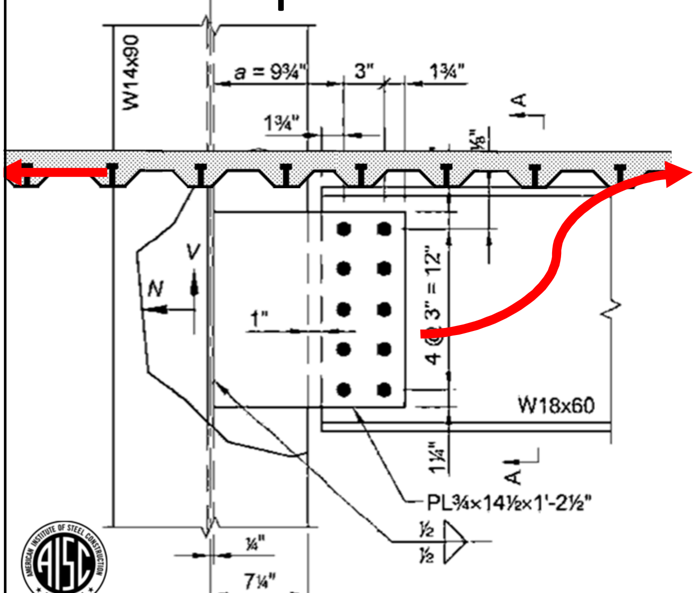
Assumption 63 – The Tension Load Exists



Why would the load go here…

57

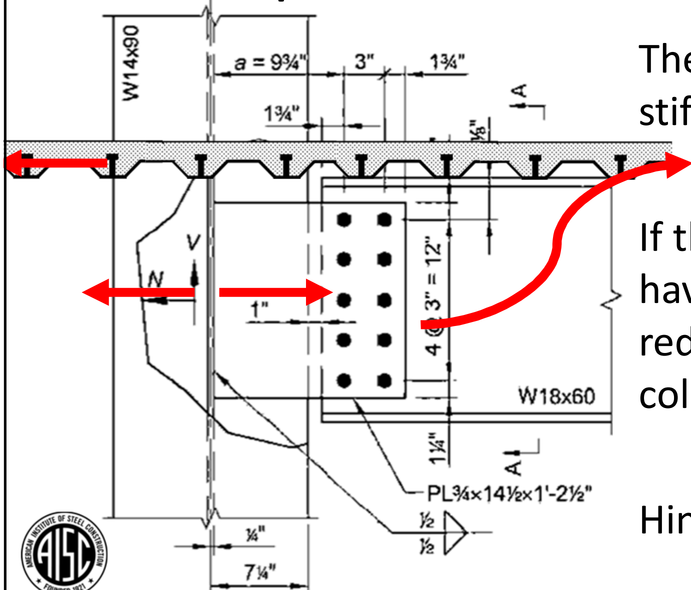
Assumption 63 – The Tension Load Exists



…when it could go here?

58

Assumption 63 – The Tension Load Exists



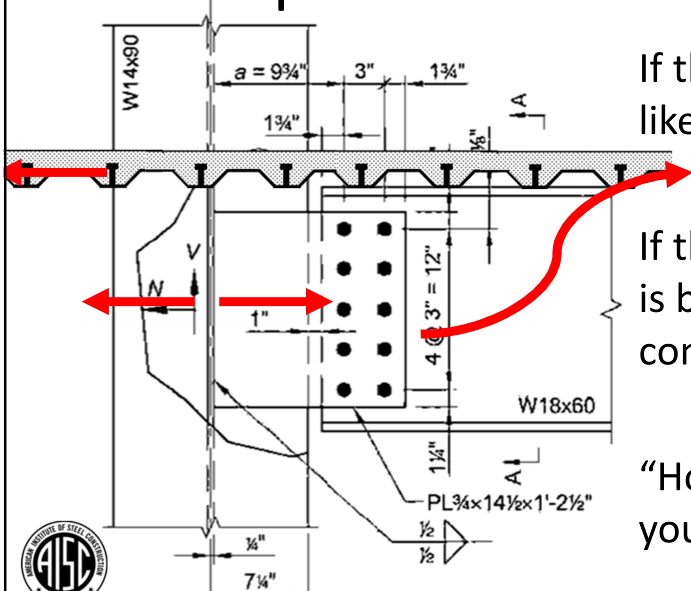
The load will distribute based on stiffness.

If the slab is stiffer, then the slab will have to yield and soften in order to redistribute load to the less stiff column web.

Hint: Concrete doesn't yield well.

59

Assumption 63 – The Tension Load Exists



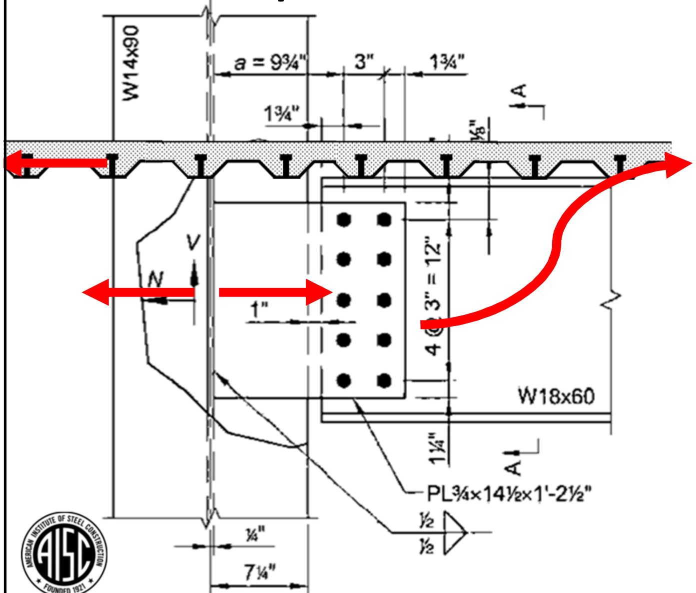
If there is a diaphragm, it seems likely the load will go there.

If there is not a diaphragm then what is bracing the beam end and connection?

"How can you have any pudding if you don't eat yer meat?"

~~~ Roger Waters
~~~

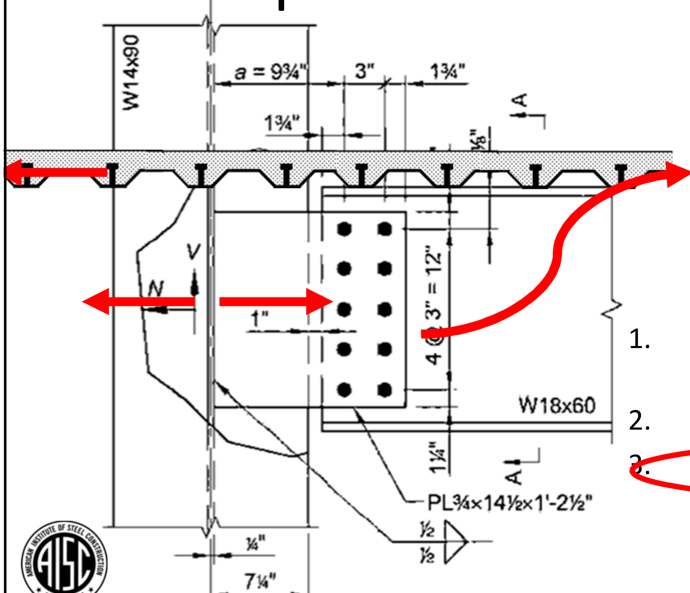
Assumption 63 – The Tension Load Exists



As a general rule it makes more sense to distribute all of the load to the stiffer element – this is especially true if the stiffer is also the less (least) ductile element.

61

Assumption 63 – The Tension Load Exists



As a general rule it makes more sense to distribute all of the load to the stiffer element – this is especially true if the stiffer is also the less (least) ductile element.

1. Choose a force distribution that satisfies equilibrium.
2. Do not exceed any limit states.
3. Take reasonable measures to ensure ductility.

Requires some judgment and faith.

62

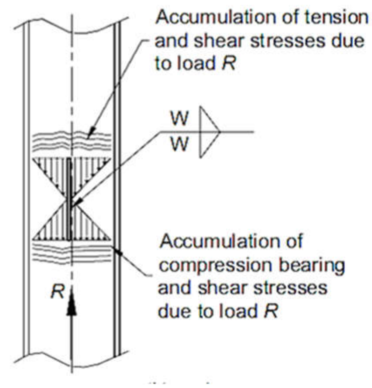
Assumption 64 – Shear at Weld

Shear Rupture Strength of Column Web at Weld

The design example assumes two shear planes along each side of the shear tab.

Other models have been proposed.

"Guidance on Shear Rupture, Ductility and Element Capacity in Welded Connections" Fortney, Muir and Thornton (EJ 2nd Qtr. 2019)



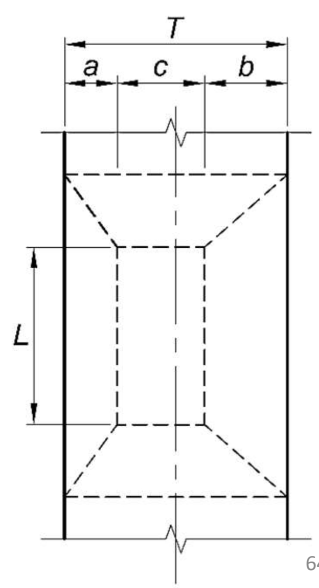
63

Assumption 65 – Tension into Web

Yield Line Analysis on Supporting Column Web

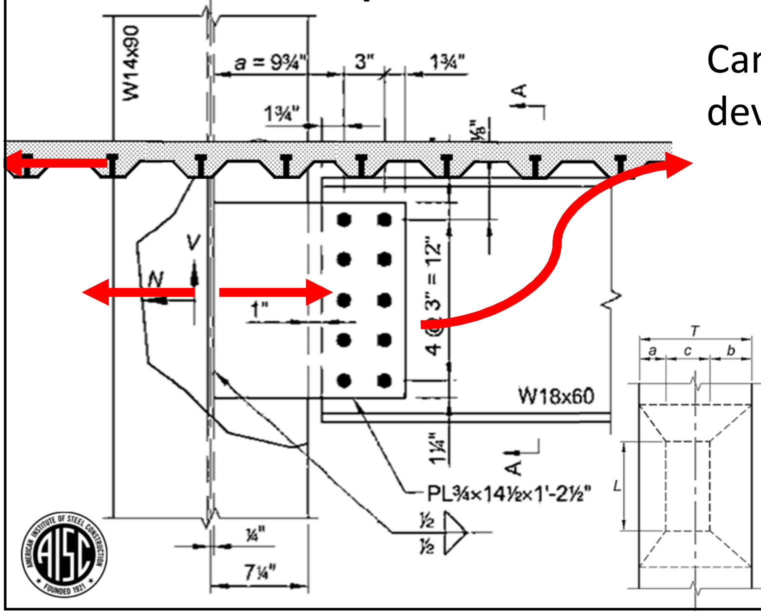
A yield line analysis is used to determine the strength of the column web in the direction of the axial tension load.

The analysis assumes each of the dashed lines will yield. This could take a lot of deformation.



64

Assumption 65 – Tension into Web



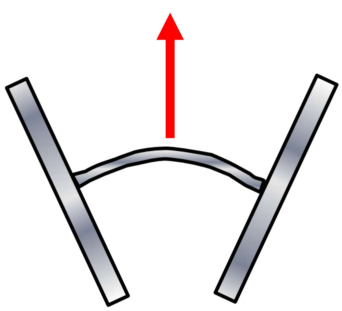
Can the yield line strength develop before the slab fails???

What condition is being addressed in the design example?

Can we, as users, know?

65

Assumption 65 – Tension into Web

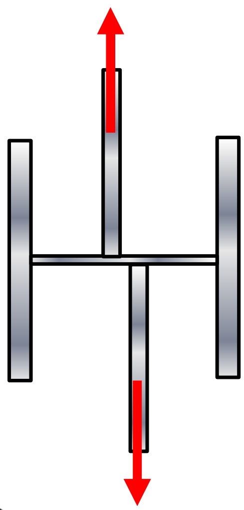


From the *Manual: "*For wide-flange sections, the edges of the column web are generally assumed to be pinned."

This assumes the column flanges just go along for the ride.

66

Assumption 65 – Tension into Web



If there are connections to both side of the web, then there is no need to do a yield line check.

If offset:

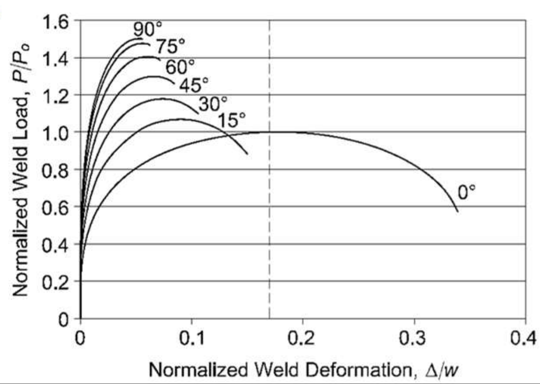
- Is there shear on the column web?
- Does it make sense to assume the welds are equally loaded?
- Does it make a difference that the plate is developed?

67

Assumption 66 – Weld Strength

Strength of Weld

The available weld strength is determined using AISC *Manual* Equation 8-2a or 8-2b, incorporating the directional strength increase from AISC *Specification* Equation J2-5…



Assumption 66 – Weld Strength

The checks in the design example make sense – as long as a significant moment cannot be developed at the weld, which is the case for the condition considered.

However, the statement, "A two-sided fillet weld with size of $(^5/_8)t_p$ = 0.469 in. (use 2-in. fillet welds) is used. As discussed in AISC *Manual* Part 10, this weld size will develop the strength of the shear plate used because the moment generated by this connection is indeterminate."

It may be indeterminate, but it will also be small, so why use $(^5/_8)t_p$?

69

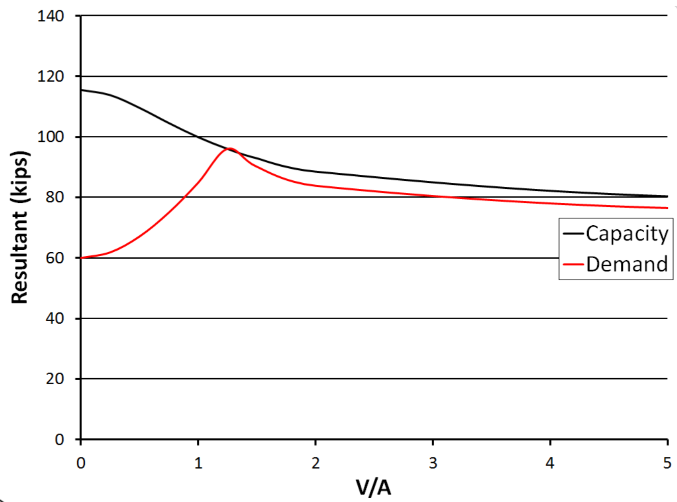
Assumption 66 – Weld Strength

If the moment generated by this connection is indeterminate, but maybe not small, it may make sense to recheck the weld for the given loads assuming a moment equal to R(e) at the weld – or not.

We will neglect the eccentricity as we tie up some loose ends relative to Assumption 51.

70

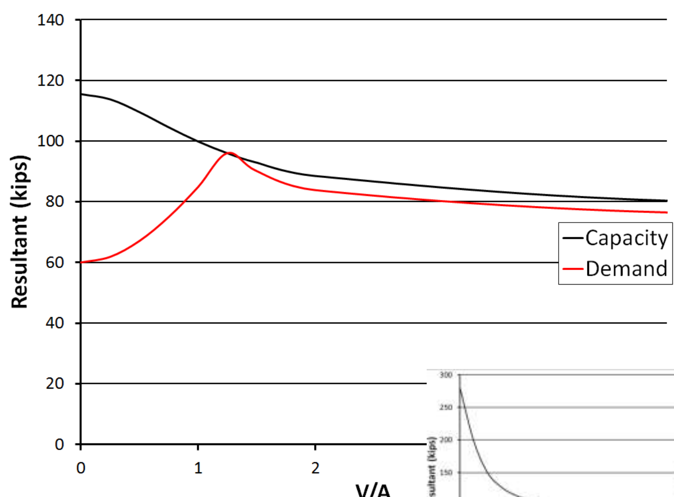
Assumption 51 Revisited – Worst Load Case

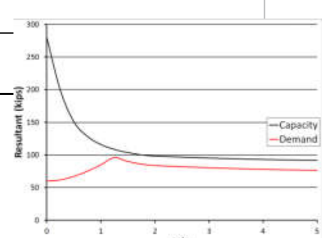


This graph shows the demand and capacity related to the weld group for various ratios of shear-to-axial load consistent with the requirements in the example.

71

Assumption 51 Revisited – Worst Load Case





The result for the welds is closer to the result for the bolts than I would have expected since the welds have a directional strength increase and the bolts don't.

Why?

72

Assumption 67 – Compression

The given for the design example indicates that the axial load is in tension.

The design example concludes, but them comments on a fictional compression case.

Assumption 67 – Compression

Compression Buckling

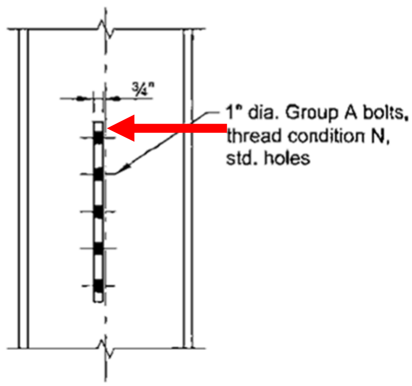
> If the applied axial load were in compression, the connection plate would need to be checked for compressive flexural buckling strength… This is required in the case of the extended configuration of a single-plate connection...

From AISC *Specification* Table C-A-7.1, Case c:

K = 1.2

Disclaimer: Other possibilities exist. The beam could be braced relative to lateral-torsional buckling but could be subject to constrained-axis buckling relative to compression. We really do not know what condition we are addressing.

Assumption 67 – Compression



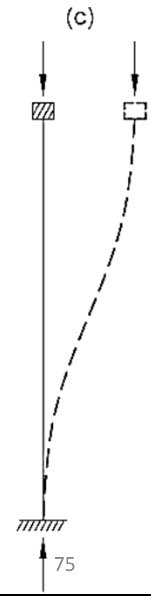
Section A-A

If the plate is braced, then how can it sidesway buckle?

If the plate can sidesway buckle, then how can it be braced?

Again - "How can you have any pudding if you don't eat yer meat?"

~~~ Roger Waters



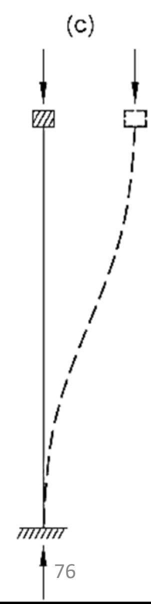
75

Assumption 68 – Compression Interaction?

This check is shown in the conclusion as a standalone check with the calculated available strength compared to the axial beam end reaction.

It is not clear that the intent is to use this result in the *Specification* Chapter H in conjunction with AISC *Manual* Equation 10-5 interaction check… but I assume this is the intent.

Another approach would be to somehow combine the axial and flexural compression and perform a buckling check.



76
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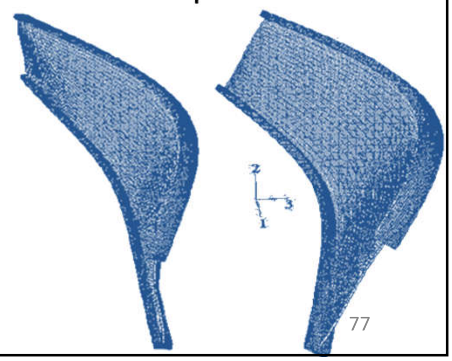
Assumption 69 – Erection Loads

Until the deck is on the connection will not be braced at its end so the Part 9 cope checks are not applicable.

It is also not obvious that the supported beam is braced at points of support.



77

Assumption 69 – Erection Loads

A rule-of-thumb is that sufficient torsional end restraint exists (relative to the beam) if the end torsional restraint stiffness equals at least 20 times the torsional stiffness of the beam.

Again the Design Example does not provide enough information for use to check the condition provided, so we will assume the span is 18 times the nominal beam depth or 27 feet.

78

Assumption 69 – Erection Loads

Torsional stiffness of beam:

Assumes point load at mid-span. More conservative than uniform load.

$$\frac{T}{\theta} = \left(\frac{GJ}{l}\right)\left[1 + \frac{\pi^2 E C_w}{GJl^2}\right]$$

$$= \left(\frac{(11{,}200\ ksi\ )(2.17\ in.^4\ )}{(324\ in.)}\right)\left[1 + \frac{\pi^2(29{,}000\ ksi)(3{,}850in.^6\ )}{(11{,}200\ ksi\ )(2.17\ in.^4\ )(324\ in.)^2}\right] = 107\ \frac{kip - in}{rad.}$$

79

Assumption 69 – Erection Loads

Torsional stiffness of connection:

$$J = \frac{dt^3}{3} = \frac{(14.5\ in.)(0.75\ in.)^3}{3} = 2.04\ in.^4$$

$$\frac{T}{\theta} = \left(\frac{GJ}{l}\right) = \left(\frac{(11{,}200\ ksi\ )(2.04\ in.^4\ )}{(9.75\ in.)}\right) = 2{,}340\ \frac{kip - in}{rad.}$$

80

Assumption 69 – Erection Loads

$$\frac{K_{conn}}{K_{bm}} = \frac{2{,}260\,\frac{kip-in}{rad.}}{107\,\frac{kip-in}{rad.}} = 21.1 > 20$$

This indicates that the Chapter F equations are applicable. If the EoR (or anyone else) checked the beam for erection loads using Chapter F, then this confirms these checks were sufficient.

81

Assumption 69 – Erection Loads

Let's approach this another way using Appendix 6:

$$\beta_T = \frac{1}{\Phi}\frac{1.24L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right) = \frac{1}{0.75}\left(\frac{1.24(27\,ft)}{2(29{,}000\,ksi)(50.1\,in^4)}\right)(163\,kip-ft) = 1{,}365\,\frac{kips-in}{rad}$$

I think the rule of thumb considers top flange loading. From the Commentary C_{tt}=1.2:

$$\beta_T = 1{,}365\,\frac{kips-in}{rad}(1.2^2) = 1{,}966\,\frac{kips-in}{rad} \approx 2{,}340\,\frac{kip-in}{rad.}$$

Not too bad for a rule of thumb.

Disclaimer: It has been a long time since I have done this sort of analysis.
Do NOT plug-and-chug,

82

Assumption 69 – Erection Loads

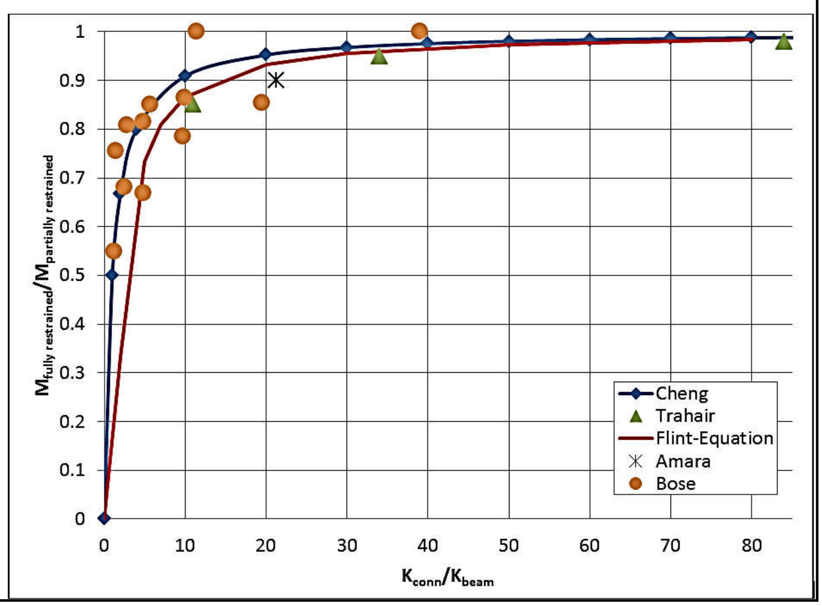
If $\frac{K_{conn}}{K_{bm}}$ was less than 20, this would not necessarily mean that the condition is insufficient. It would, however, indicate that the Chapter F equations would tend to overestimate the strength of the beam.

Erection loads tend to be small relative to typical design floor loads. Therefore, some reduction in the flexural strength below that assumed in Chapter F is likely okay.

83

Assumption 69 – Erection Loads

This plot indicates the reduction in strength due to insufficient torsional stiffness at the beam ends.



Assumption 69 – Erection Loads

This is also an apples-to-oranges comparison to begin with. The torsional stiffness analysis assumes top flange loading. Chapter F assumes "that the loads are applied along the beam centroidal axis" as stated in the *Commentary*.

If loads are applied to the top flange, Chapter F is not directly applicable to begin with. On the other hand, it is difficult to apply load without something (like a deck) to apply it to. It is also difficult to apply load without also providing some level of restraint.

85

Assumption 69 – Erection Loads

The stiffness of the end connection calculated is unconservative in some respects. Namely it neglects distortion at the web. The stiffness of the connection can be calculated as a series of springs:

$$\frac{1}{K_{conn}} = \frac{1}{K_{tab}} + \frac{1}{K_{web}} + \frac{1}{anything\ else\ that\ distorts}$$

My preference to use only full depth connections is intended to reduce the flexibility of the web to the point where its effect is negligible.

86

Assumption 69 – Erection Loads

“Bracing of Steel Beams in Bridges”(1992 Yura, Phillips, Raju, and Webb) states, “…if the connection extends over at least one-half the beam depth, then cross section distortion will not be important…”

This discussion relates to the derivation of the term β_{sec} in *Specification* Appendix 6, which presumably only applies away from the end of the member and a condition that likely does more to stiffen the web.

β_{sec} and my K_{web} are addressing similar behaviors, but should not be confused.

87

Assumption 69 – Erection Loads

I do not do this sort of analysis in practice. I use full depth extended tabs, look at the final condition, and apply engineering judgment.

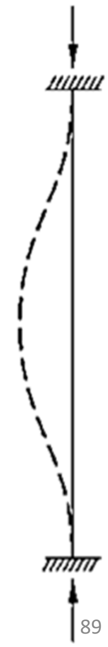
My engineering judgment is informed by performing this sort of analysis often enough that I have a “feel” for the parameters involved.

I cannot assess “by inspection” the adequacy of marginal conditions, but I can distinguish between conditions that are very much okay and those that are very much no good.

88

Assumption 70 – Column Bracing

Since the design example does not check the adequacy of the condition to brace the column, it must be assumed the connection is not required to brace the column at this point. Otherwise the check would be included in “all applicable limit states, whether or not a particular limit state controls the design of the member or connection”.



In many instances beams are assumed to provide restraint when they frame to columns, so let’s go ahead and check the condition.

89

Assumption 70 – Column Bracing

Again the Design Example does not provide enough information for use to check the condition provided, so we will again assume the span is 18 times the nominal beam depth or 27 feet and this time we will also assume that the column is pinned-pinned 13 feet long and at its full compression strength, 1,050 kips from Manual Table 4-1a.

From Appendix 6 the required strength of the brace is $0.01P_y$ = 10.5 kips, which we know from the example to be okay.

90

Assumption 70 – Column Bracing

From Appendix 6 the required stiffness of the brace is:

$$\beta_{br} = \frac{1}{\varphi}\left(\frac{8P_r}{L_{br}}\right) = \frac{1}{0.75}\left(\frac{8(1{,}050\ kips)}{156\ in.}\right) = 71.8\ \frac{kips}{in}$$

91

Assumption 70 – Column Bracing

The stiffness of the W18x60 is:

$$\beta_{W18x60} = \frac{AE}{L} = \frac{(17.6\ in.^2)(29{,}000\ \ ksi)}{324\ in.} = 1{,}580\ \frac{kips}{in}$$

Some engineers stop here and assume sufficient bracing.

This is not correct. It neglects the connection.

92

Assumption 70 – Column Bracing

The axial stiffness of the plate is:

$$\beta_{plate} = \frac{AE}{L} = \frac{(10.9\ in.^2)(29{,}000\ \ ksi)}{9.75\ in.} = 32{,}400\,\frac{kips}{in}$$

The plate is much stiffer than the beam, so this should not be a problem but…

93

Assumption 70 – Column Bracing

What about the bending stiffness of the column web?

Let's check it as a simply supported beam with a width equal to the connection depth.

$$\beta_{web} = \frac{48EI}{T^3} = \frac{48(29{,}000\ \ ksi)\left(\frac{(14.5\ in.)(0.440\ in.)^3}{12}\right)}{(10\ in.)^3}$$

$$= 143\ \frac{kips}{in}$$

94

Assumption 70 – Column Bracing

No flexibility in the bolts or welds is accounted for. It is assumed that the bolt group will behave as slip critical relative to the 10.5 kips bracing load. This assumption could be controversial.

Assuming the same connection at each end we get:

$$\frac{1}{\beta_{br}} = \frac{1}{\beta_{beam}} + 2\left(\frac{1}{\beta_{plate}}\right) + 2\left(\frac{1}{\beta_{web}}\right)$$

$$= \frac{1}{1{,}580\ \frac{kips}{in}} + 2\left(\frac{1}{32{,}400\frac{kips}{in}}\right) + 2\left(\frac{1}{143\ \frac{kips}{in}}\right)$$

95

Assumption 70 – Column Bracing

Assuming the same connection at each end we get:

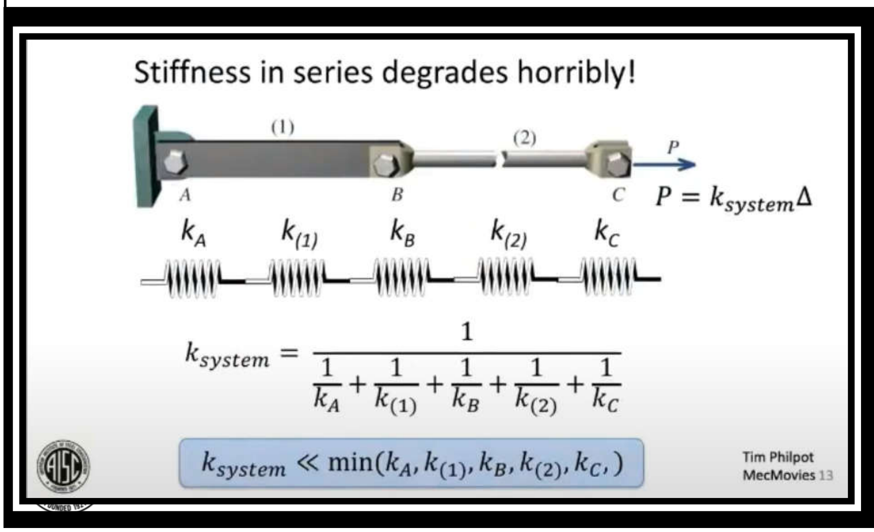
$$\frac{1}{\beta_{br}} = \frac{1}{\beta_{beam}} + 2\left(\frac{1}{\beta_{plate}}\right) + 2\left(\frac{1}{\beta_{web}}\right)$$

$$= \frac{1}{1{,}580\ \frac{kips}{in}} + 2\left(\frac{1}{32{,}400\ \frac{kips}{in}}\right) + 2\left(\frac{1}{143\ \frac{kips}{in}}\right)$$

$$\beta_{br} = 68.1\frac{kips}{in} < 71.9\ \frac{kips}{in}$$

96

Assumption 70 – Column Bracing



Want More?

"Structural Stability - Letting the Fundamentals Guide Your Judgment" by Ron Ziemian in the AISC Continuing Education Archives.

Assumption 70 – Column Bracing

It does not take much to brace a member. Even in this case the assumption, that the column can be modelled as a simply supported beam with an effective width equal to the connection depth, is very conservative.

With a slightly less onerous assumption even this connection can be shown to be sufficient.

I suspect it is okay, but it is not obvious to me by inspection, so I would check it – probably "off to the side".

98

Assumption 70 – Column Bracing

Even if the connection is sufficient to brace the column laterally in the weak-axis, it will not be sufficient to brace the column torsionally.

"When the torsional effective length is larger than the lateral effective length, Section E4 may control the design of wide-flange and similarly shaped columns."

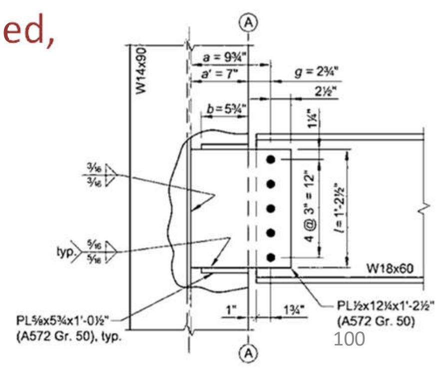
The *Commentary* states, "Recommendations for torsional bracing of columns can be found in Helwig and Yura (1999)."

99

Assumption 71 – Stiffeners

Column Reinforcement

Use stiffener or stabilizer plates—also called continuity plates. This is probably the most viable option, but changes the nature of the connection, because the stiffener plates will cause the column to be subjected to a moment. This cannot be avoided, but may be used advantageously.

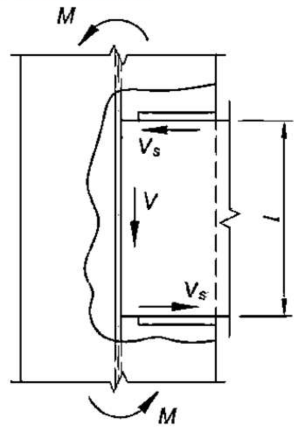


100

Assumption 71 – Stiffeners

Column Reinforcement

The column design needs to be reviewed to ensure that this moment does not overload the column.



101

Assumption 71 – Stiffeners

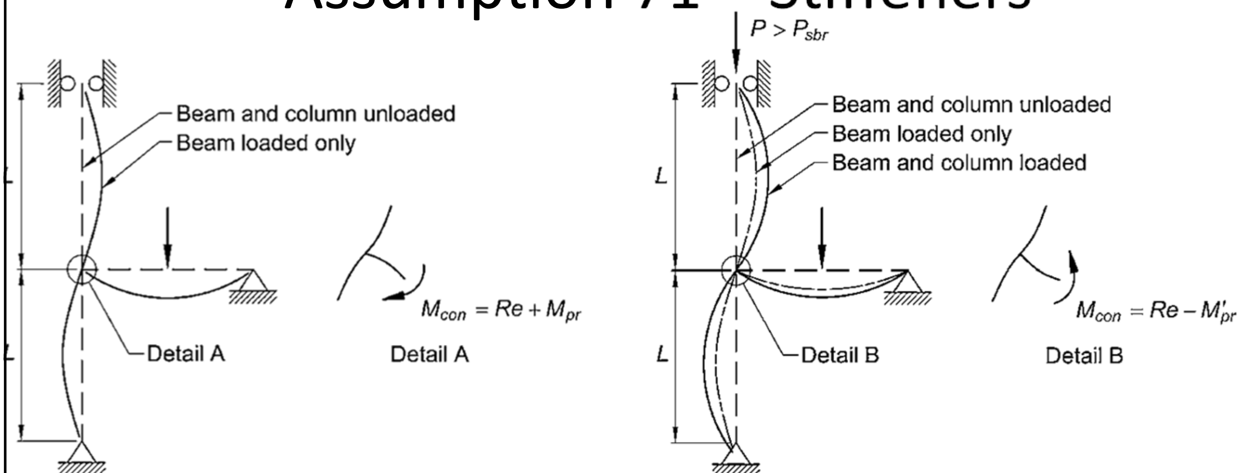


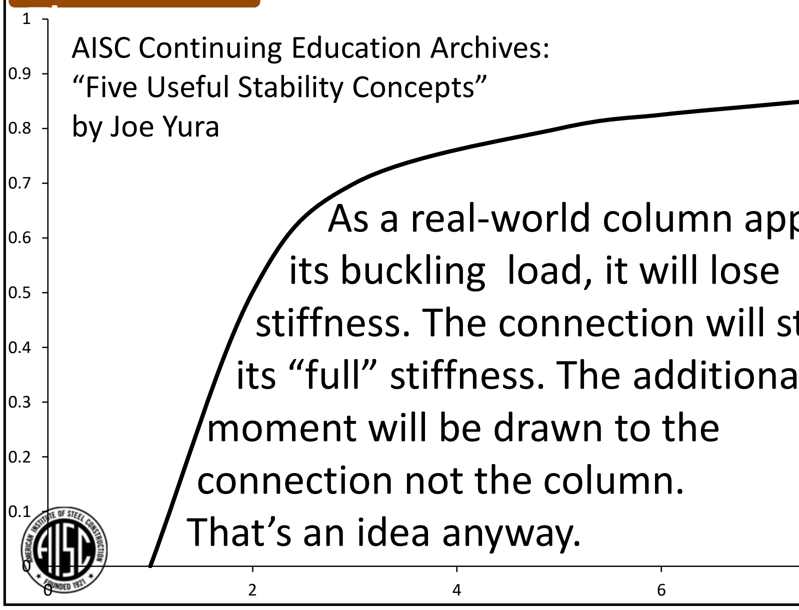
Illustration of beam, column and connection behavior under loading of beam only.

Illustration of beam, column and connection behavior under loading of beam and column.

102

Lower Bound

Column Buckling



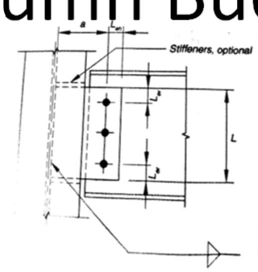
AISC Continuing Education Archives:
"Five Useful Stability Concepts"
by Joe Yura

As a real-world column approaches its buckling load, it will lose stiffness. The connection will still have its "full" stiffness. The additional moment will be drawn to the connection not the column. That's an idea anyway.

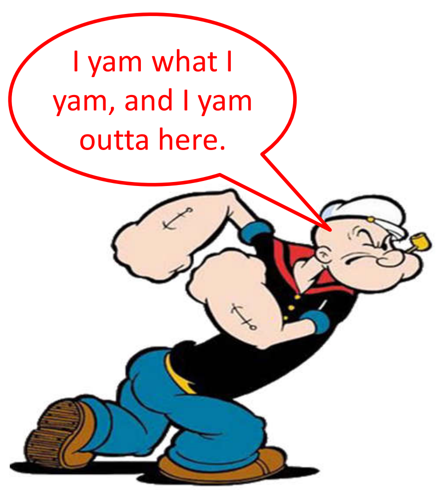
Lower Bound

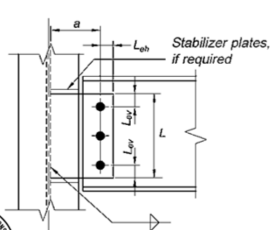
Column Buckling


Design of Unstiffened Extended Single-Plate Shear Connections

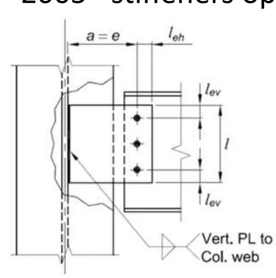


2005 –stiffeners optional





2010 – if req'd



2016 – no stiffeners

104

Assumption 71 – Stiffeners

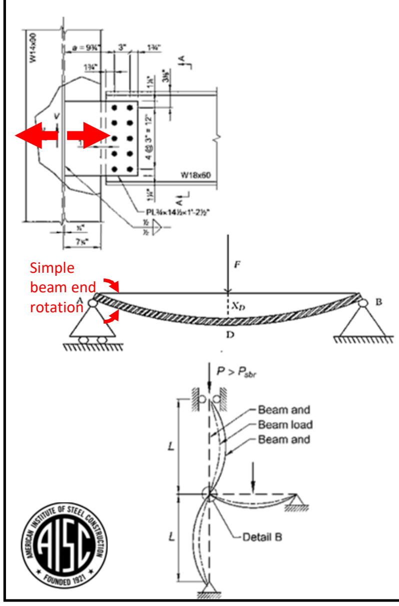
Will the introduction of stiffeners increase the moment delivered to the column? Yes.

Will the introduction of stiffeners decrease the overall strength of the structure???

Judgment and Faith

105

Assumption 71 – Stiffeners



How do you provide enough strength and stiffness to deliver significant beam end axial load…

and still accommodate this…

but not do stuff like this???

"How can you have any pudding if you don't eat yer meat?"
~~~ Roger Waters
~~~

Assumption 72 – You

I have just gone through one example in nauseating detail.

If for every connection you design you consider all 20 assumptions, you will still NOT be applying sufficient engineering judgment.

- Master the basics – Statics & Strength of Materials
- Stick with what you know – as you seek to learn more
- Do what makes sense to you
- Welcome a challenge
- Question your assumptions – especially the implicit ones

"It ain't what you don't know that gets you into trouble.
It's what you know for sure that just ain't so."
~~~Mark Twain

107

Assumption 72 – You

Is it strong enough?

Is it stable enough?

How would I build it?

Should I really be designing this?

Is it ductile enough?

Do I understand it?

Does it look right?

"I will say that at my first reading of your email advice I was taken aback. Upon reflection however, I smiled and said yes that's probably what I would have said in a similar situation. Your observations and warnings were appropriate here. With that in mind I am going to recommend to the fabricator that he find another consultant…"

108
~~~

Thank you!

AISC | Questions

Smarter.
Stronger.
Steel.

Individual Session Registrants

PDH Certificates

- All WFH individuals associated with a group registration will be issued a certificate.
- All individuals attending at your connection: you will receive an email on how to report their attendance from: registration@aisc.org.
  - Be on the lookout: Check your spam filter! Check your junk folder!
  - Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

8-Session Registrants

PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.

8-Session Registrants

Access to the quiz

Information for accessing the quiz will be emailed to you by Thursday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

Quiz and attendance records

Posted Thursday mornings. www.aisc.org/nightschool -- Click on Current Course Details.

Reasons for quiz

- EEU – You must take all quizzes and the final exam to receive EEU.
- PDHs – If you watch a recorded session, you must pass quiz for PDHs.
- REINFORCEMENT – Reinforce what you learn tonight. Get more out of the course.

Note: If you attend the live presentation, you do not have to take the quizzes to receive PDHs

8-Session Registrants

Access to the recording

Information for accessing the recording will be emailed to you by Thursday. The recording will be available for four weeks. (For 8-session registrants only.) EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

PDHs via recording

If you watch a recorded session, you must take *and pass* the quiz for PDHs.

8-Session Registrants

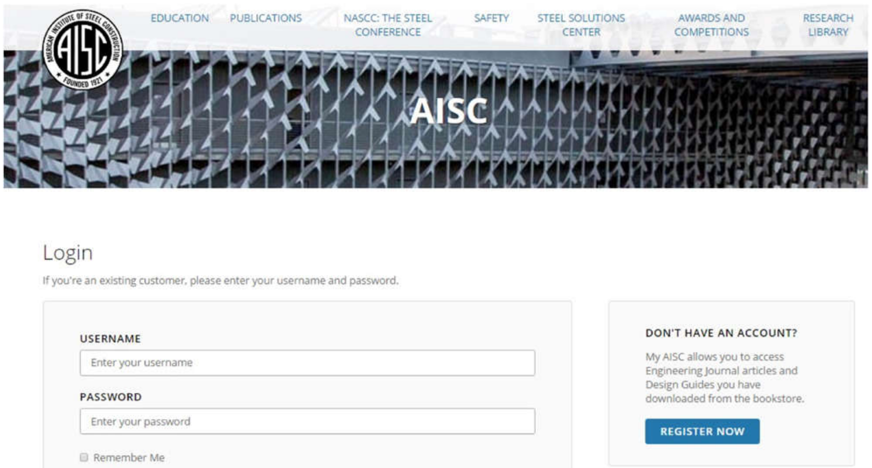
Night School Resources

Find all your handouts, quizzes and quiz scores, recording access, and attendance information all in one place!

8-Session Registrants

Night School Resources

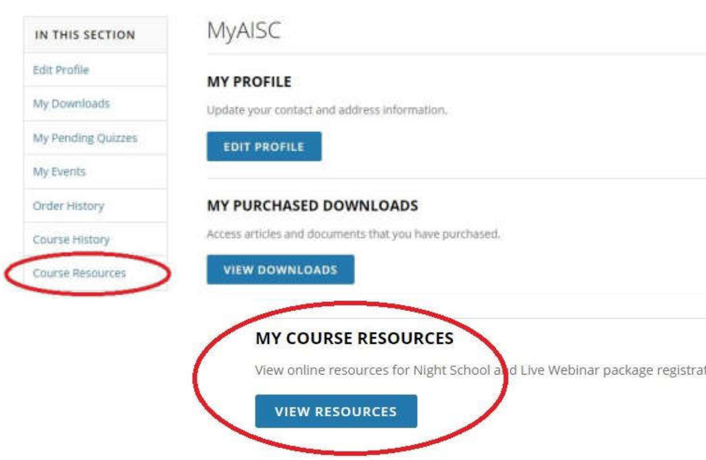
Go to www.aisc.org and sign in.



8-Session Registrants

Night School Resources

Go to www.aisc.org and sign in.



8-Session Registrants

Night School Resources

Course Resources

Event	Start Date
NS 13 8-Session Package-Night School 13 - Design of Industrial Buildings	1/30/2017 7:00:00 PM
NS 14 8-Session Package-Night School 14 - Fundamentals of Stability	6/5/2017 7:00:00 PM

8-Session Registrants

Night School Resources

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dsn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending

8-Session Registrants

Night School Resources

- Weekly “quiz and recording” email.
- Weekly updates of the master quiz and attendance record, found at www.aisc.org/nightschool26. Scroll down to Quiz and Attendance records.
    - Updated on Friday mornings.

8-Session Registrants

Night School Resources

- Webinar connection information
    - Reminder email sent out Tuesday mornings
- Links to handouts also found here

AISC | Thank you

Smarter.
Stronger.
Steel.