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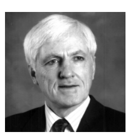
High-Strength Bolts: The Basics



Presented by
Geoff Kulak, Ph.D.
Professor Emeritus at the University of Alberta

There's always a solution in steel.

High-Strength Bolts: The Basics

- Fundamentals and Behavior
- Specification Requirements (AISC 2010)

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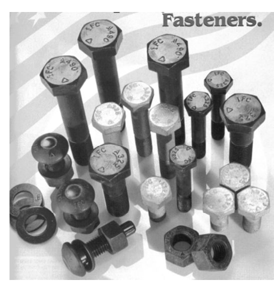
Role of the Structural Engineer…



- Selection of suitable bolt types and grades
- Design of the fasteners
- Responsibility for installation
- Responsibility for inspection

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ASTM A307 Bolts

- often a good choice when loads are static
- strength level inferior to high-strength bolts (60 ksi tensile ult.)
- pretension indeterminate

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ASTM A325 Bolts

- Type 1 or Type 3 (weathering steel)
- ASTM Spec. ↔ RCSC Spec.
- Minimum tensile strength: 120 ksi
- Pretension can be induced if desired

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ASTM A490 Bolts

- Types 1 or Type 3 (weathering steel)
- Minimum tensile strength: 150 ksi, (maximum 170 ksi)
- ASTM Spec. ⟷ RCSC Spec.
- Pretension can be induced if desired

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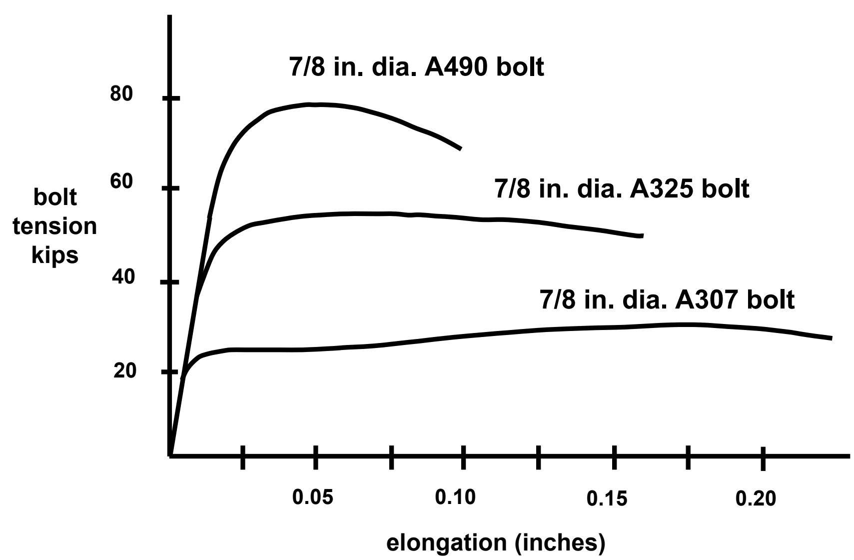
Comparison of Bolts: Direct Tension



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Comments…

- **Note: we quote the ultimate tensile strength of the bolt**
  - **benchmark for strength statements (e.g. shear strength is some fraction of ultimate tensile strength)**
- **What about yield strength?**
- **What is "proof load"**

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…comments cont'd

- **Nuts: ASTM A563**
- **Washers: if needed, ASTM F436**
- **Bolt – nut – washer sets implied so far, but other configurations available**
- **Bolt notation: Group A (A325, F1852) and Group B (A490, F2280)**

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Loading of Bolts

- Shear
  - load transfer by shear in bolt and bearing in connected material OR
  - load transfer by friction (followed by shear and bearing)
- Tension
- Combined Tension and Shear

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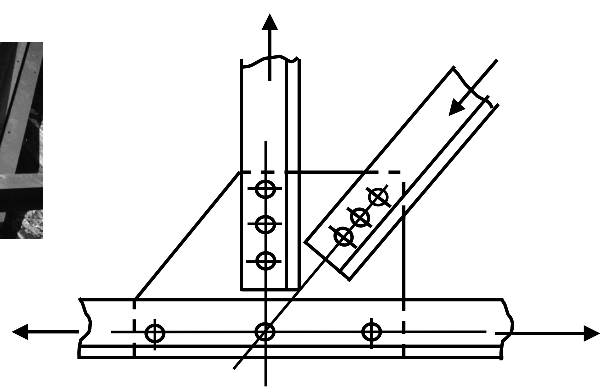
Shear Loading



Truss Joint

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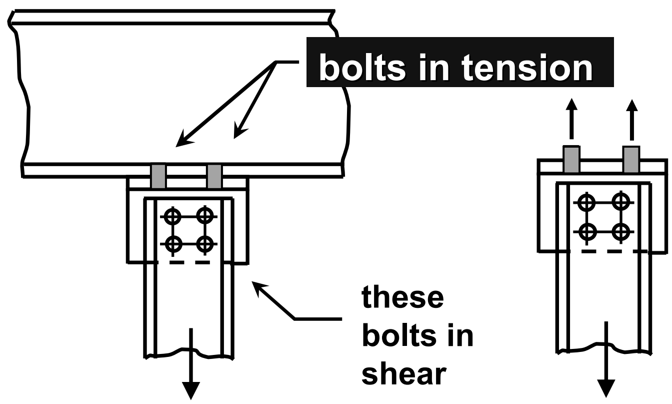
Bolts Loaded in Tension



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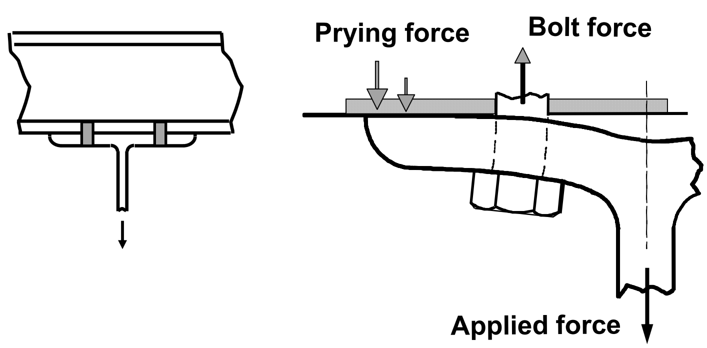
Bolts in Tension – prying



High-strength bolts in tension can be a source of problems

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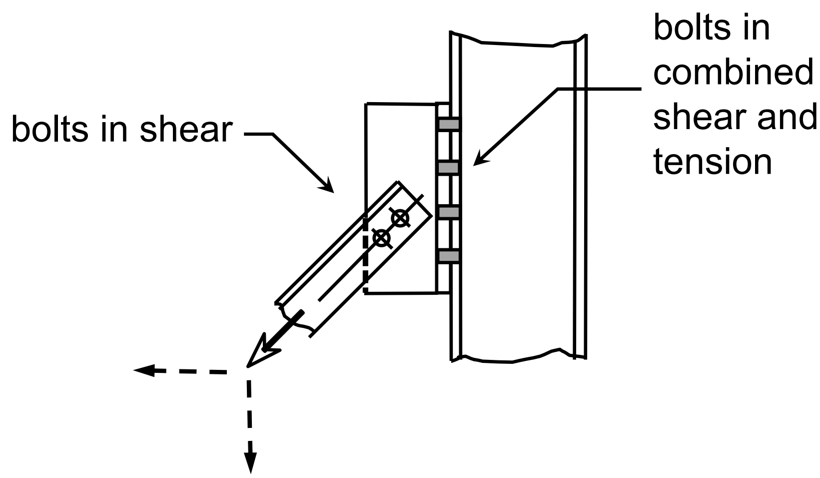
Bolts in combined tension and shear



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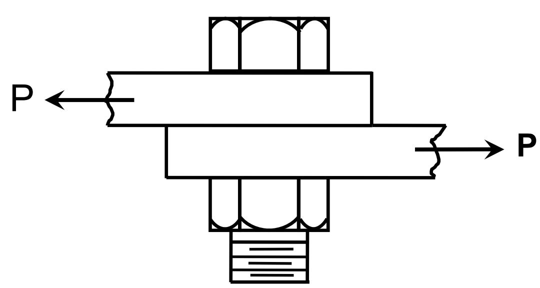
Consider a simple joint —



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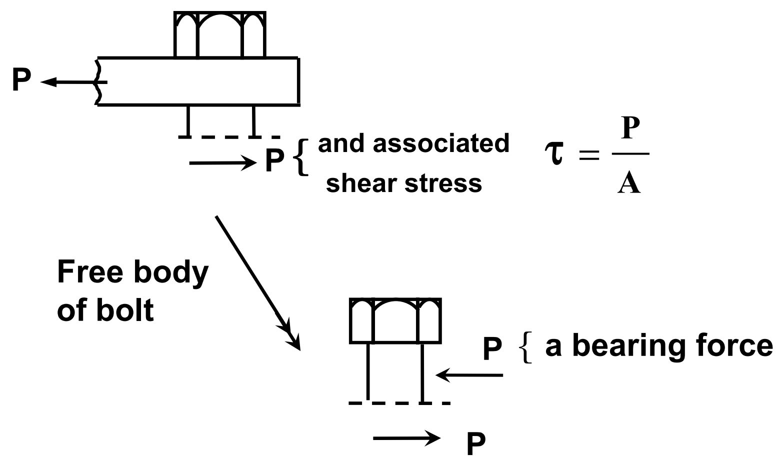

$$\tau = \frac{P}{A}$$

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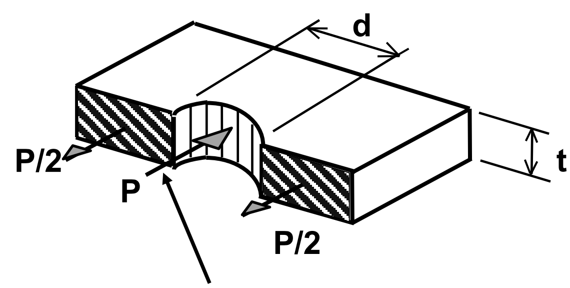
Finally...



this force is equal and opposite to the bearing force shown previously

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In the example, we identified…

- force in the bolt (a shear force)
- force that the bolt imposed on the plate (a bearing force)
- force in the plate itself (a tensile force)
- force transfer could also be by friction: not included in this illustration

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AISC Standard 2010

- Parallel LRFD and ASD rules
- LRFD uses a resistance factor, Ø
- ASD uses a safety factor, Ω
- Loads as appropriate:
  - factored loads for LRFD
  - non-factored loads for ASD

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AISC Specification cont'd

LRFD: req'd strength LRFD $\leq \phi R_n$

ASD: req'd strength ASD $\leq R_n / \Omega$

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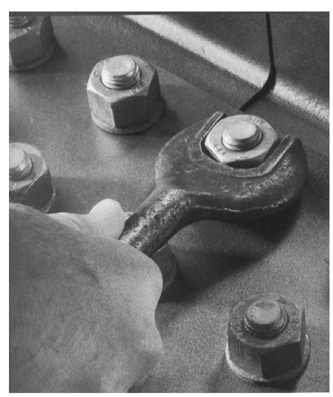
Installation —



- **Snug-tight only**
- **Pretensioned**
  - Calibrated wrench
  - Turn-of-nut
  - Other means:
    - ✓ Tension control bolts
    - ✓ Load-indicator washers

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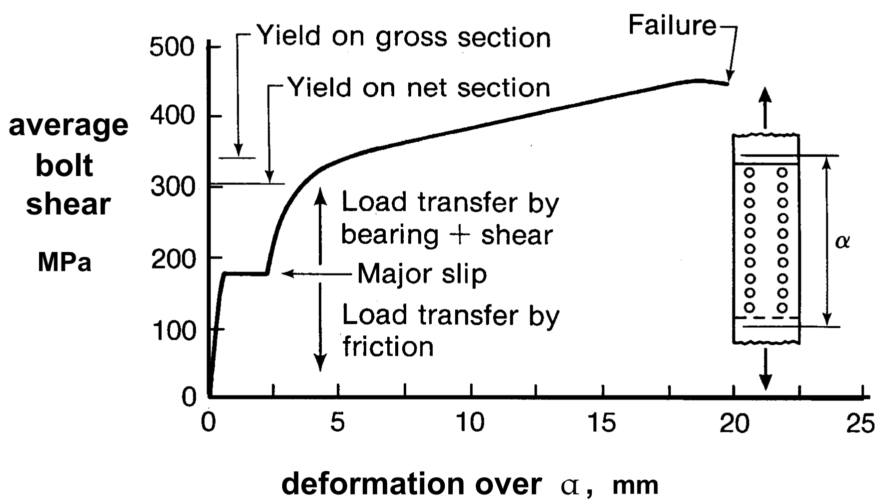
Behavior of a large joint (shear splice) —



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Bolts in Shear: Issues

- Shear strength of bolt (single shear or double shear, threads in shear plane?)
- Bearing capacity of bolt (never governs)
- Bearing capacity of plate
- Tensile (comp.) capacity of plate

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Slip in bolted joints…

- Can be as much as two hole clearances
- Some bolts will already be in bearing at start of loading
- Both laboratory tests and field measurements indicate that slip is more like 1/2 hole clearance

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Bolts in shear-type connections:

Specifications include information for:

- bearing type connections
- slip-critical connections

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Bearing-type connections:

- Issues
  - bolt shear strength
  - bearing capacity of connected material
  - member strength
- Shear strength of bolts is <u>not dependent</u> on presence or absence of pretension. (How come?)

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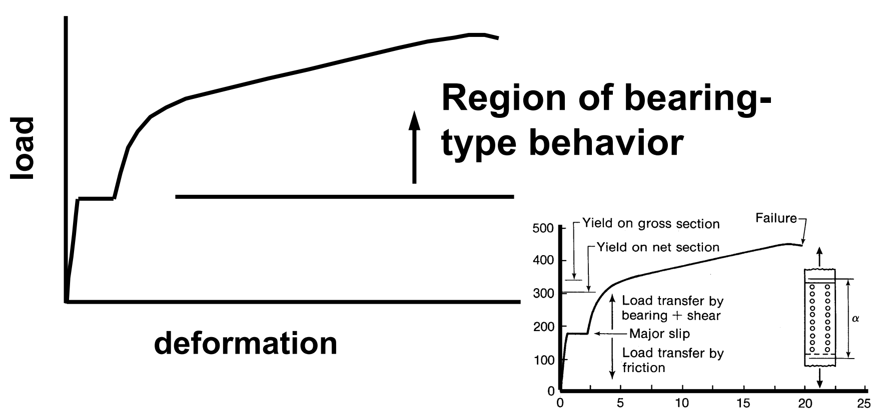
Bolts in bearing-type connections…



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Bolt Shear Strength

- Bolt shear strength ≈ 62% of bolt ultimate tensile strength (theory + tests)
  - Design rule takes 90% of this value
  - Threads in shear plane?
  - Long joint effect: another discount applied.

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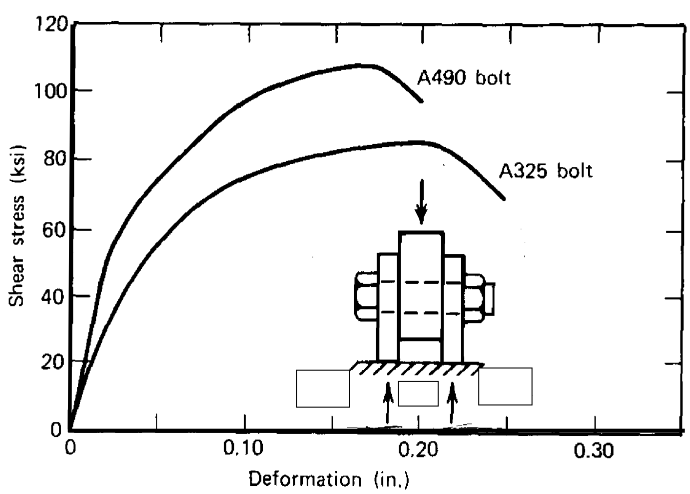
Individual bolt in shear



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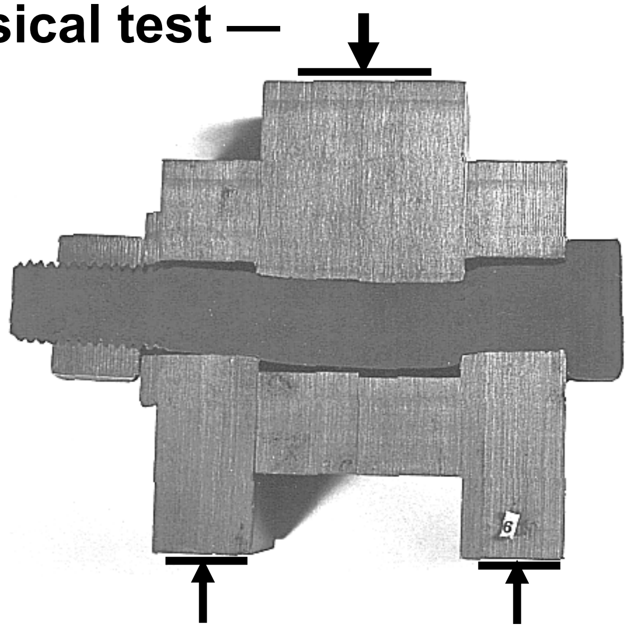
Physical test —



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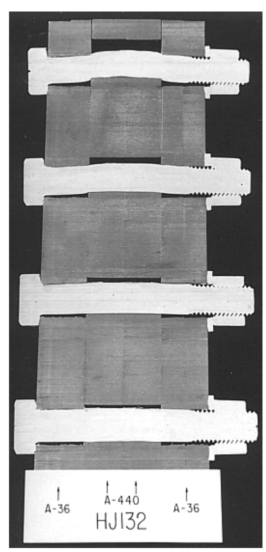
Uneven loading of bolts –

(End four bolts of 13)



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Bolt Pretension v. Shear

- The bolt pretension is attained as a result of small axial elongations introduced as nut is turned on
- These small elongations are relieved as shear deformations and shear yielding take place
- Confirmed by both bolt tension measurements and shear strength tests
- So, bolt shear strength NOT dependent on pretension in the bolt.

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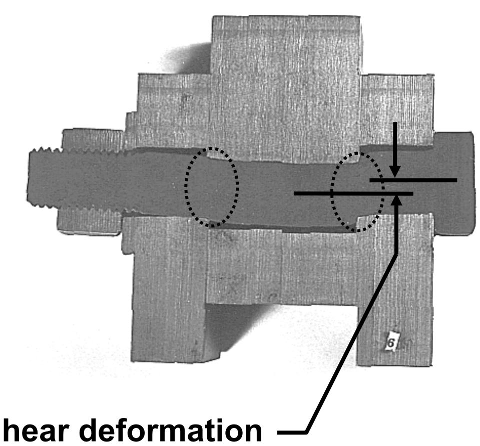
Back to bolt in shear —

Shear strength of single bolt (tests) —

$$\tau = 0.62\ \sigma_{u\ bolt}$$



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Bolts in Shear — AISC

$$\phi R_n = \phi \ F_{nv} \ A_b$$

$$\phi R_n = \text{design shear strength}$$

$$F_{nv} = \text{nominal shear strength, ksi}$$

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nominal shear strength …

$$F_{nv} = 90\% \ (0.625 \times F_u) = 0.563 \ F_u$$

e.g. A325 bolt, no threads in shear plane,
Group A : see tabulated value in Table J3.2
$(0.563 \text{ ksi } F_u = 0.563 \text{ x } 120 \text{ ksi} = 68 \text{ ksi})$

38

and…

For threads included, the tabulated values are 80% of the above.

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Comments…

- **The discount for length (use of 90%) is conservative**
- **If joint length > 38 in., a further reduction, to 83%**
- **The ø – value used for this case (0.75) is conservative**

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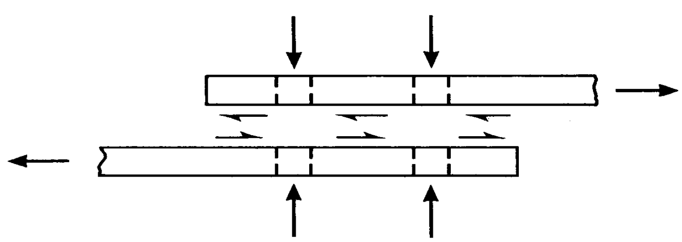
Let's return now to slip-critical connections…

Slip-Critical Connection

Clamping force from bolts (bolt pretension)



Load at which slip takes place will be a function of …?

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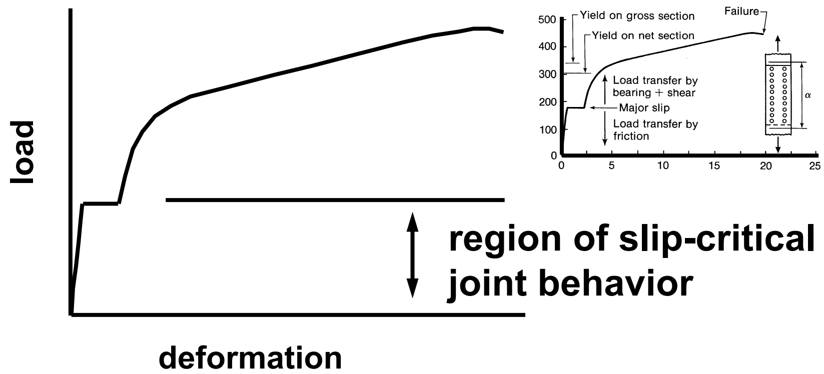
Bolts in slip-critical connections…



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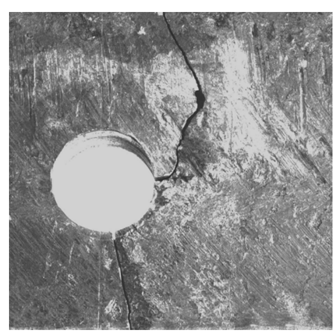
Slip-critical joints specified when…

- Load is repetitive and changes from tension to compression (fatigue by fretting could occur.)
- Change in geometry of structure would affect its performance.
- Certain other cases.
- Comment: for buildings, slip-critical joints should be the exception, not the rule (but, see also seismic rules)



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First principles, slip resistance is —

$$P = k_s \; n \; \Sigma T_i$$

k_s = slip coefficient (μ)

n = number of slip planes (usually 1 or 2)

T_i = clamping force (i.e., bolt pretension)

45

Design slip resistance, AISC

$$R_n = \mu \; D_u \; h_f \; T_b \; n_s$$

no. slip planes

clamping force

slip coefficient

…terms h_f and D_u need to be defined and a value inserted for Ø

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and the modifiers …

h_f = modifier re fills : either 1.0 or 0.85

$\phi = 1.0$ for std. holes and for short slots $\perp$

$= 0.85$ for oversize and short slots parallel

$= 0.70$ for long slotted holes

$D_u = 1.13$, ratio of installed bolt tension to specified minimum bolt tension

$\mu = 0.30$ clean mill scale, hot – dipped galvanized and roughened, etc. (Class A surfaces)

$\mu = 0.50$ unpainted and blast – cleaned, etc. (Class B surfaces)

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A note for advanced readers!!

D_u is a statistical parameter that results in a probability of slip of 5% at the service load level when the joint is designed using factored loads.

The resistance factor reflects the consequence of exceeding the "slip limit state." As the consequence of slip gets more severe, the resistance factor is decreased.

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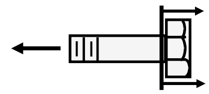
Bolts in Tension



- Capacity of a bolt in tension: product of the ultimate tensile strength of the bolt and the tensile stress area of the bolt (i.e. $F_u \ A_{st}$)
- Specifications directly reflect this calculated capacity (…to come)
- Force in bolt must reflect any prying action effect

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Bolts in Tension – some comments

- Preference: avoid joints that put bolts into tension, especially if fatigue is an issue
- Use A325 bolts rather than A490 bolts
- Minimize the prying action

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Question…

- pretensioned bolt in a connection
- apply external tension force to the connection
- do the bolt pretension and the external tension add?

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Bolt tension + external tension

1. Pretension the bolt → tension in the bolt, compression in the plates
2. Add external tension force on connection →
   - Bolt tension increases
   - Compression between plates decreases

Examine equilibrium and compatibility…

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And the result is…

- The bolt force does increase, but not by very much ($\cong$ 7%)
- This increase is accommodated within the design rule.

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AISC rule, bolts in tension—

$$\phi R_n = \phi \; F_{nt} \; A_b$$

bolt area for nominal diameter

nominal tensile strength

$$\phi R_n = \text{design tensile strength}$$

54

What is nominal tensile strength, F_{nt} ?

$$P_{ult} = F_u\ A_{st} = F_u\ (0.75 A_b)$$

Adjusted area

$$\text{or,}\quad P_{ult} = \underbrace{0.75\,F_u}_{\text{Call this } F_{nt}}\ A_b$$

55

So, the AISC rule for bolts in tension…

$$\phi\, R_n = \phi\, F_{nt}\ A_b$$

where $F_{nt} = 0.75\ F_u$ as tabulated in the Specification

As we now know, the 0.75 really has nothing to do with F_u

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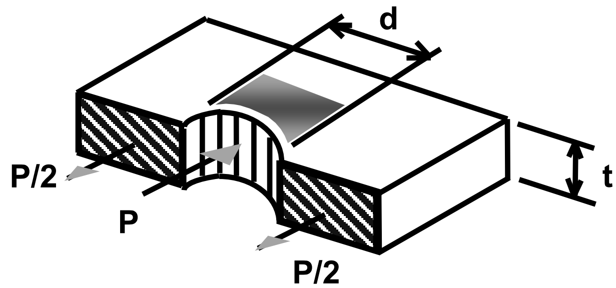
Returning to shear splice joints, we still have to deal with the bearing capacity of the connected material.



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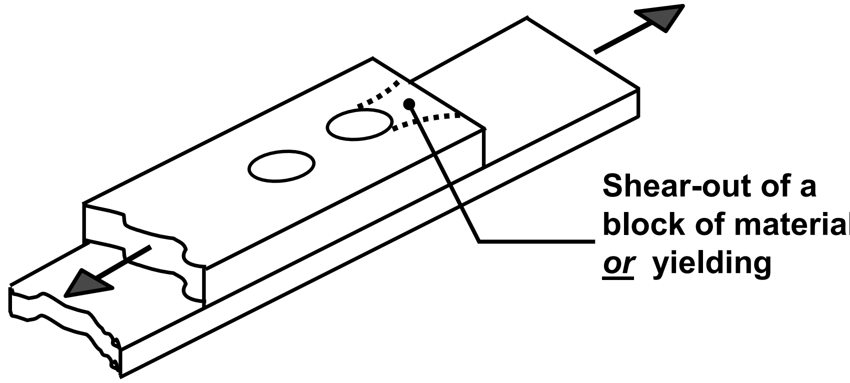
Bearing capacity (of connected material)



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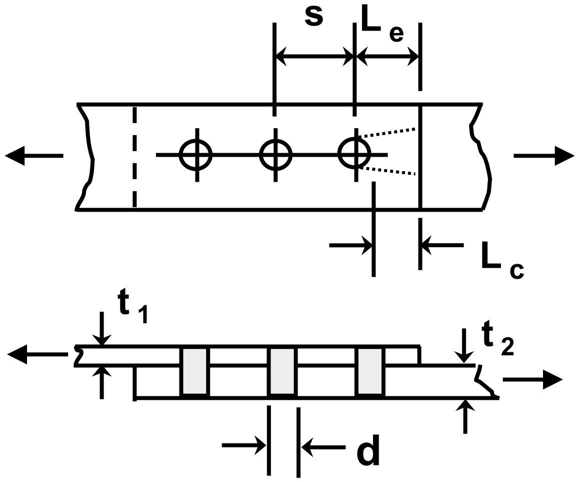
Bearing stresses at bolt holes…



Needed:

1. **shear-out rule**
2. **yield rule (deformation)**

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Shear-out rule…

$$\text{Shear-out is } 2\,(\tau_{ult} \times L_c \times t)$$

$$\text{or, } R_n = 2\,(0.75\,\sigma_u \times L_c \times t)$$

and AISC rule is: $R_n = 1.5\, F_u\, L_c\, t$

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Plate bearing…

from tests: $\dfrac{\sigma_b}{\sigma_u^{pl}} = \dfrac{L_e}{d}$

..after some arithmetic $R_n = \sigma_b \ d \ t = \sigma_u^{pl} \left(\dfrac{L_e}{d} \right) d \ t$

valid for $L_e \geq 3 \ d$

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Plate bearing…

Making the substitution and using

$F_u \equiv \sigma_u^{pl}$

$$R_n = 3 \ d \ t \ F_u \longleftarrow$$

62

Finally, the AISC rule for plate bearing capacity is …

$$R_n = 1.5\, F_u\, L_c t \quad \leq\ 3.0\, d\, t\, F_u$$

(with a ϕ-value still to be inserted)

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Further note re bearing…

When deformation a consideration, use

$$R_n = 1.2\, F_u\, L_c t \quad \leq 2.4\, d\, t\, F_u$$

Why this difference, and when do we use the latter? (value of ϕ still to be applied)

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Block shear rupture

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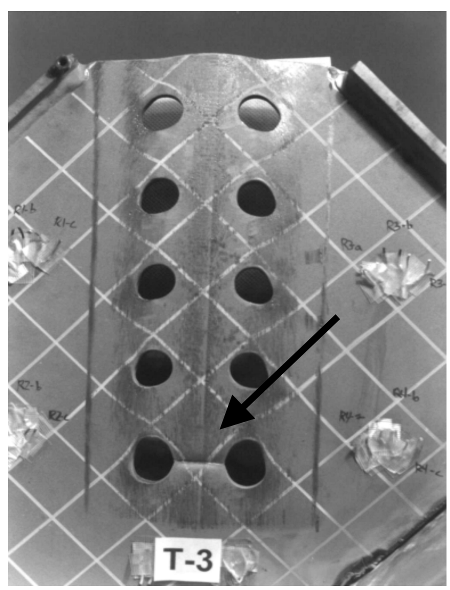
Failure (ult. load) is by tensile fracture at location shown, regardless of geometric proportions.

Shear yield along vertical planes.

Failure is controlled by *ductility* – not strength.



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Basics…

$$T_r + V_r = \underbrace{\phi A_{nt} F_u}_{\text{tension fracture}} + \underbrace{0.60 \phi A_{gv} F_y}_{\text{shear yield}}$$

where A_{nt} = net area in tension

and A_{gv} = gross area in shear

…and some other requirements, including specific case of coped beams, limit on shear

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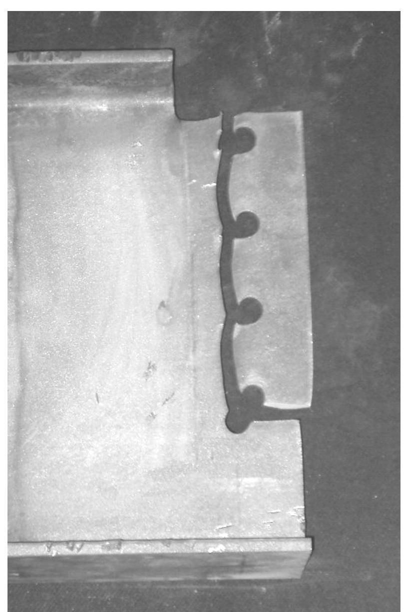
An example of shear + tension failure in a coped beam…



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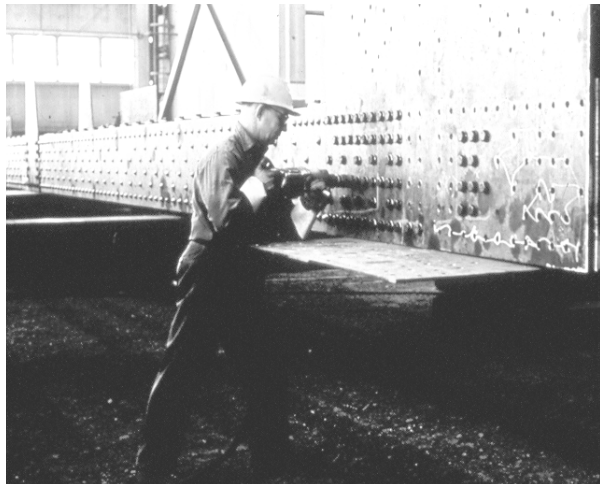
Back to installation…



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Bearing-Type Connections—Installation of Bolts

- Bolts can be installed to “snug-tight condition — ordinary effort of worker using a spud wrench. (Pretension unknown, but usually small)

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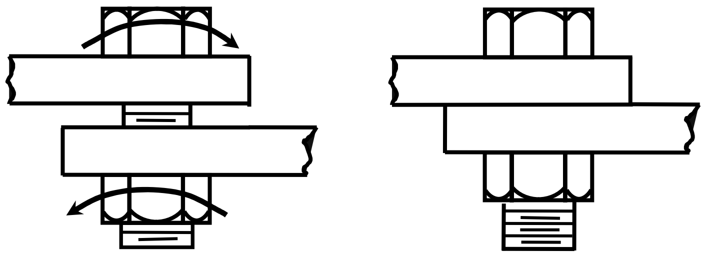
Installation —



– bring parts together, continue turning nut, bolt elongates, tension develops in bolt, and clamped parts compress

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1. Calibrated Wrench Installation

- **Reliable relationship between torque and resultant bolt tension?**
  NO ! (and forbidden by RCSC)
- **Establish relationship by calibration of the installing wrench.**

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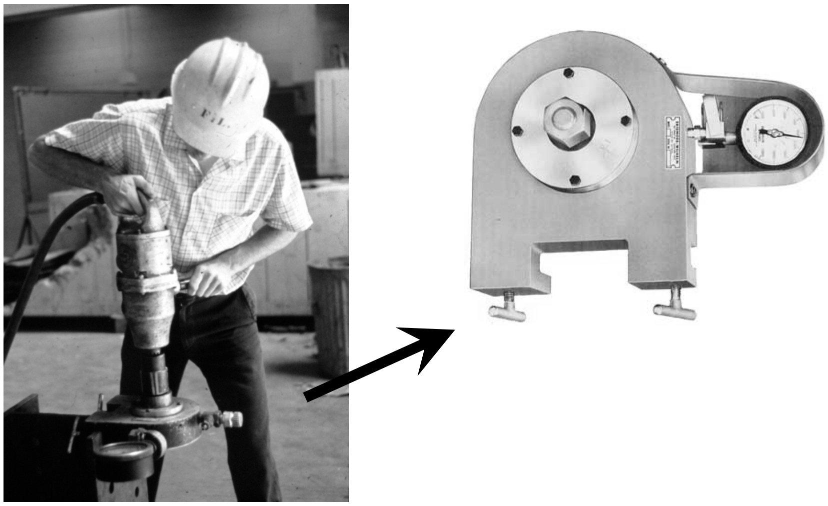
Hydraulic calibrator –



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Calibrated wrench, cont’d

- Adjust wrench to stall or cut out at desired level of bolt pretension
- Target value of pretension (RCSC) is 1.05 times specified min. value
- Calibrate using at least three bolts
- Calibration is unique to bolt lot, length, diameter, grade of bolt
- Washers must be used

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2. Turn-of-Nut Installation

- Run nut down, bring parts into close contact
- Work from stiffer regions to edges
- Establish "snug-tight" condition (first impact of impact wrench or full effort of worker using a spud wrench)
- Apply additional one-half turn (or other value, depending on bolt length)

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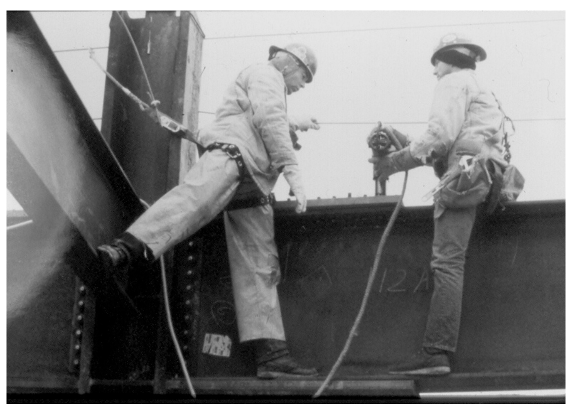
Does this definition of snug-tight seem a little vague?



How influential is "snug-tight?"

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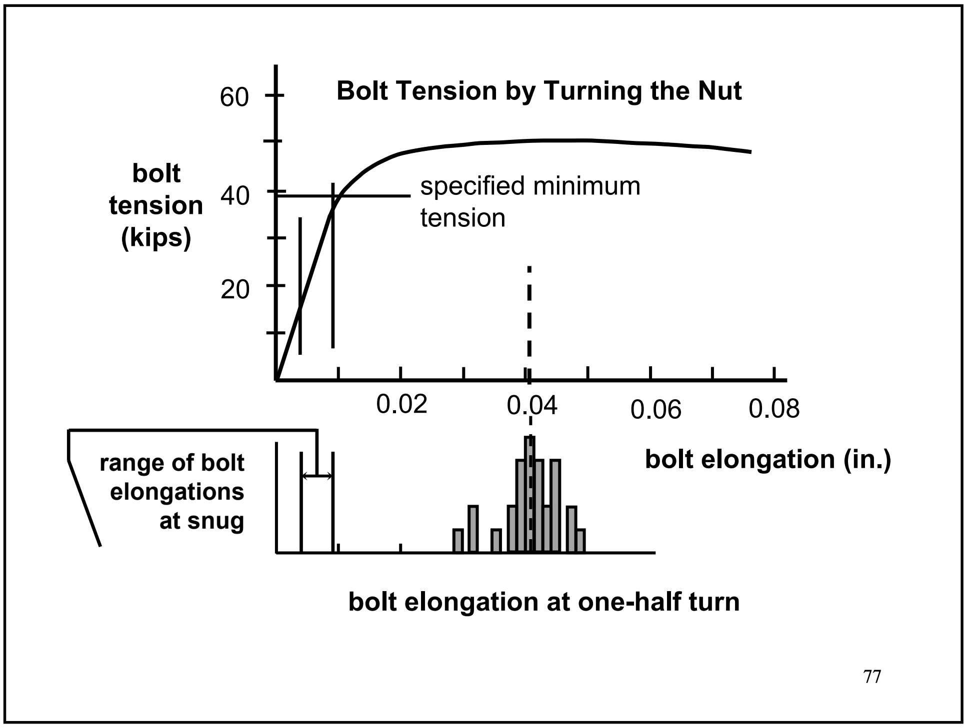
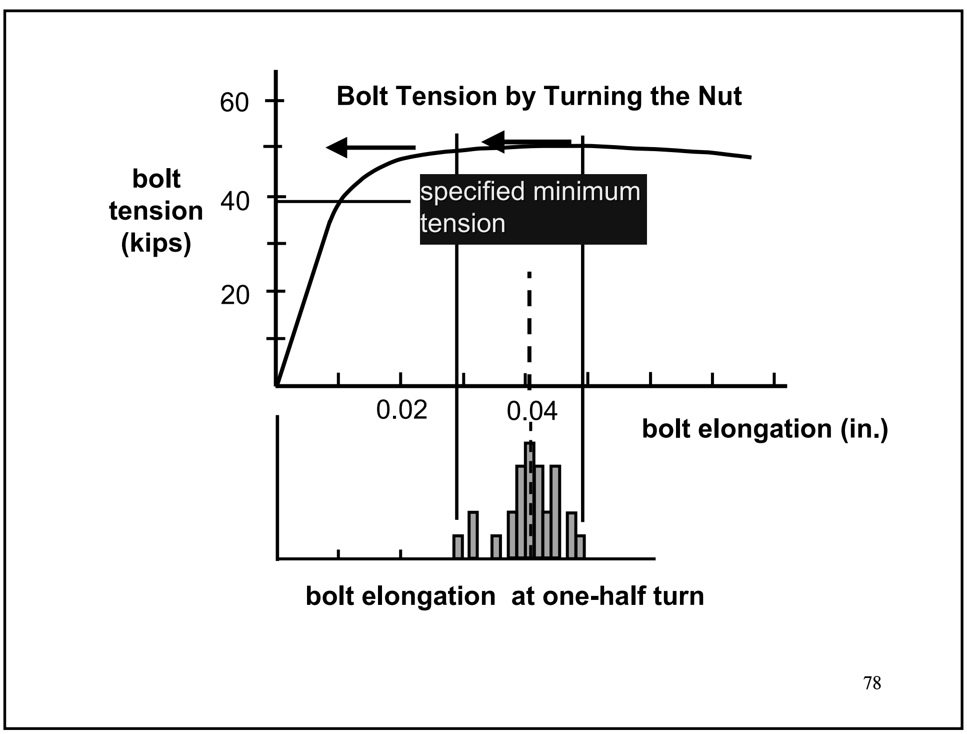

Bolt Tension by Turning the Nut
bolt tension (kips)
60
40
20
specified minimum tension
0.02
0.04
0.06
0.08
bolt elongation (in.)
range of bolt elongations at snug
bolt elongation at one-half turn
77


Bolt Tension by Turning the Nut
bolt tension (kips)
60
40
20
specified minimum tension
0.02
0.04
bolt elongation (in.)
bolt elongation at one-half turn
78

Inspection of Installation

- **Principles:**
  - **Determination of the bolt pretension after installation is <u>not practical</u>**
  - **Understand the requirements e.g., are pretensioned bolts required?**
  - **Monitor the installation on the site**
  - **Proper storage of bolts is required**

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Inspection of Installation

- **Is bolt tension required? — if not, why inspect for it !**
- **Know what calibration process is required and monitor it on the job site**
- **Observe the work in progress on a regular basis**

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Inspection of installation:

Consider the following AISC cases —

1. **Bolts need be snug-tight only**
2. **Bolts are pretensioned (but not a slip-critical joint)**
3. **Slip-critical joint**

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Snug tight only req'd….

- **Bearing-type connections**
- **Bolts in tension (A325 only)**
  - **only when no fatigue or vibration (bolt could loosen)**

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Inspection – snug tight

- Bolts, nuts, and washers (if any) must meet the requirements of the specifications
- Hole types (e.g., slotted, oversize) must meet specified requirements
- Contact surfaces are reasonably clean
- Parts are in close contact after bolts snugged
- All material within bolt grip must be steel

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Inspection: if pretensioned bolts required…

- All of requirements for snug-tight case
- Observe the pre-installation verification process
  - turn of nut, or;
  - calibrated wrench, or;
  - other (direct tension washers, tension-control bolts)
- Calibration process done minimum once per day
- Calibration process done any time conditions change

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Inspection: for slip-critical joints

- All of the above, plus
- Condition of faying surfaces, holes, etc.
- In addition to observing the calibration process, the inspection must ensure that the same process is applied to the field joints

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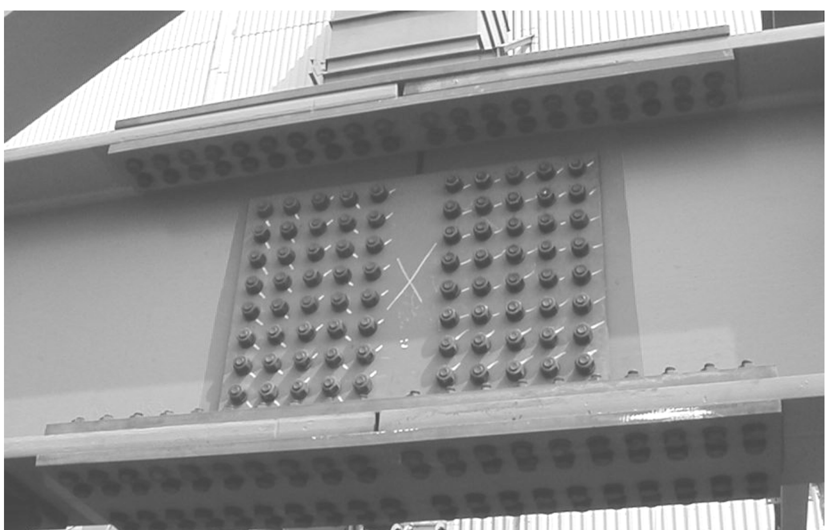
An inspected joint (turn-of-nut)



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and some other comments…

- Pretension values <u>greater</u> than those specified are not cause for rejection.
- <u>Rotation</u> tests are useful for short-grip bolts or coated fasteners (requirement is in ASTM A325 spec. and is for galvanized bolts)

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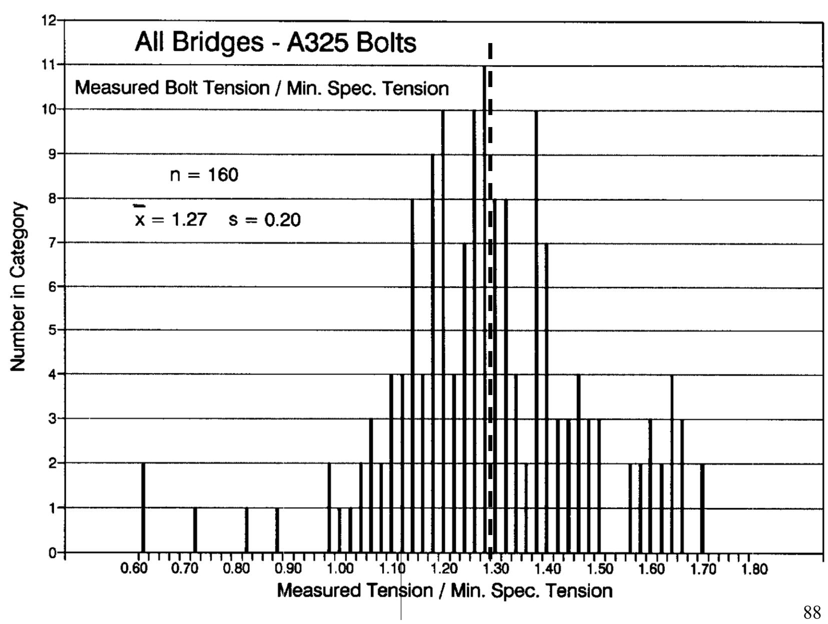


88

Actual pretensions, cont'd

- **For A325 bolts, turn-of-nut:**
  - **Average <u>tensile strength</u> exceeds spec. min. tensile by about 1.18**
  - **Average <u>pretension force</u> is 80% of actual tensile**
  - **Result is that actual bolt tension is about 35% greater than specified bolt tension**

89

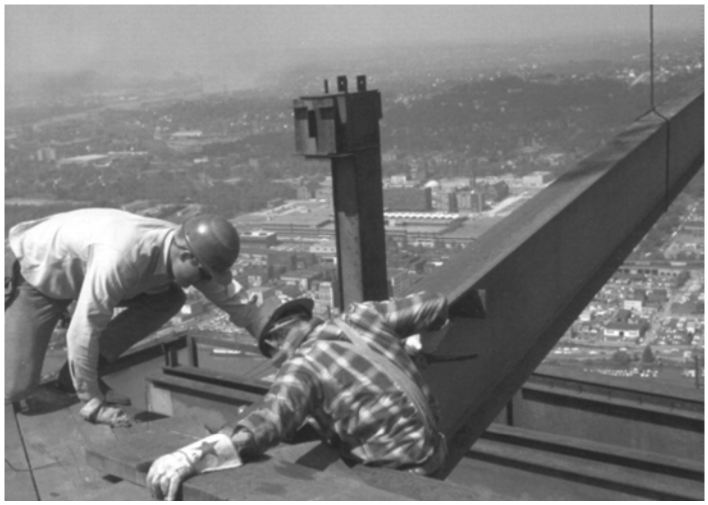
Actual pretensions, cont'd

- **A325, ½ turn-of-nut: 35% increase**
- **A490, ½ turn-of-nut: 26% increase**
- **A325 and A490, calibrated wrench: 13% increase**
- **etc. for other cases**

Note: these increased pretensions are embodied in the specification rules

90

Some other options for bolts —



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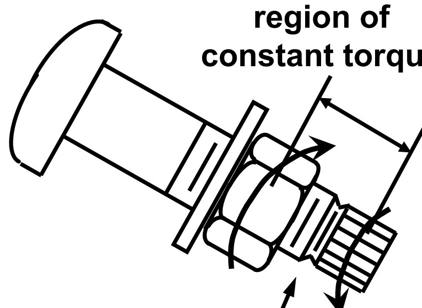
Tension Control Bolts

ASTM F1852,
F2280



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Tension control bolts….

- NOTE: evidence that tips have sheared off is not in itself evidence that desired pretension is present
- Consider limits:
  - Friction conditions are very high…
  - Friction conditions are very low…
- Hence, calibration is essential!

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Tension-Control Bolts

- Advantages
  - Installation is from one side
  - Electric wrench is used
  - Installation is quiet
- Disadvantages
  - More expensive
  - Pre-installation calibration required

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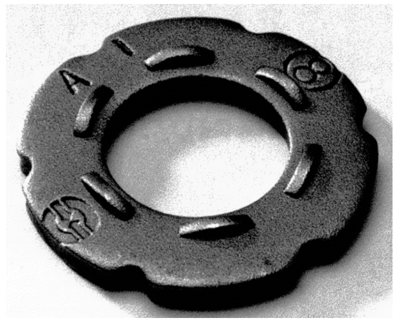
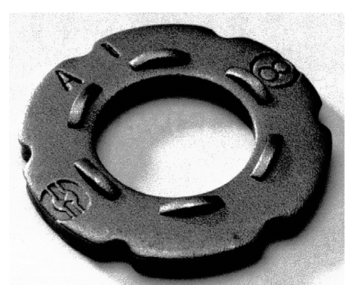
Direct tension indicators—



95

Direct Tension Indicators

- Protrusions formed in special washer
- Protrusions compress as force in bolt is developed
- Use feeler gage to measure gap (or refusal)
- User must verify the process (like calibrated wrench)



ASTM 959

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Reliability of these...

- Calibration required
- Reliability same as calibrated wrench
- Tension-control bolt is torque-dependent
- Load-indicating washer is elongation-dependent

97

Some additional topics …

- Details, other topics
  - washers (but not today!)
  - slotted or oversize holes (but not today!)
  - seismic design

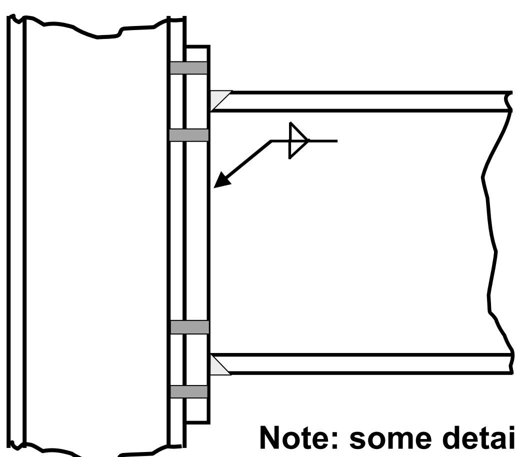
98

Seismic design of connections

- **Analyze structure in order to compute the forces**
  - **Use FEMA 350 and/or AISC Seismic Design Spec.**
- **With forces now known, design connectors**
- **Advisable to use pre-qualified configurations**

99

Pre-qualified bolted connections



Note: some details not shown, e.g., continuity plates

100

All-bolted connection

101

…bolted joints, seismic design

- All bolts pretensioned
- Faying surfaces as per slip-critical
- Use bearing values for bolts
  - moderate quakes: no slip
  - major quakes: slip will occur and bolts go into bearing
- Normal holes or short slotted only (perpendicular)
- No bolts + welds in same faying surface

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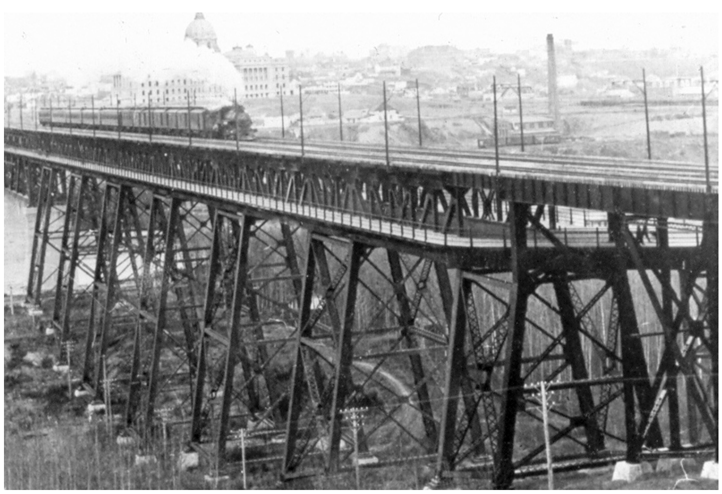
Seismic design, cont'd

- **Non-ductile limit state in either member or connection must not govern.**
- **Calculate bolt shear strength as per bearing type but use 2.4 d t F_u bearing rule**
- **Must use <u>expected</u> yield and ultimate strengths, not the specified values**

$$\text{e.g. A36 plate: use } 1.3\,\sigma_{y\ \text{spec.}}$$

103

It all started with rivets….



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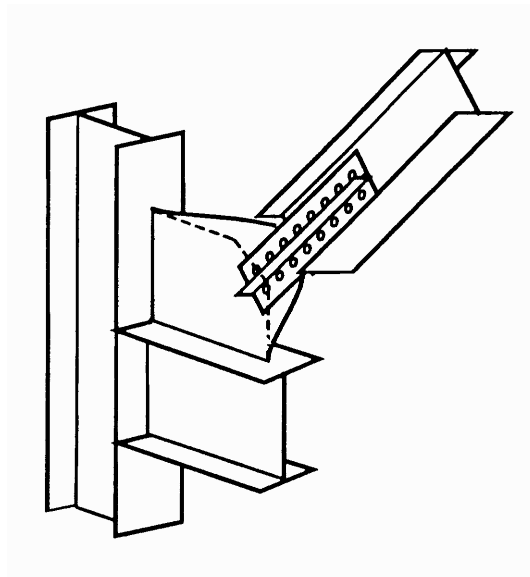
Design example: gusset plate connection



105

Determine ultimate load for this gusset plate (which is one that was tested)

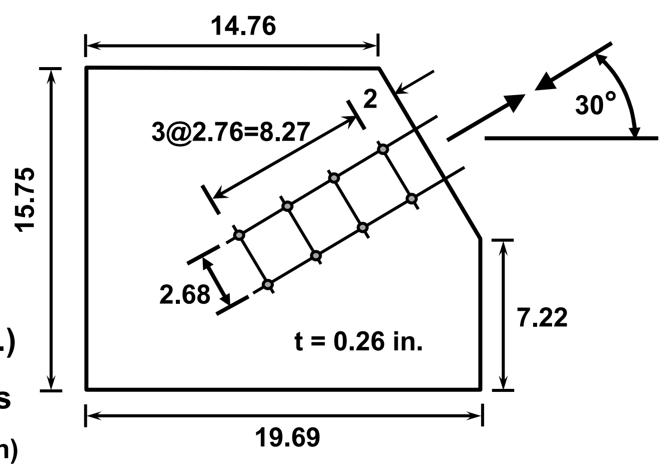
F_y = 39.9 ksi

F_u = 69.0 ksi

7/8 A325 bolts

(holes 15/16 in.)

$P_{u\ test}$ =164 kips (compression)



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Set out the issues…

- **Brace force in tension–**
  - **slip load of bolts (no slip at service load)**
  - **shear load of bolts**
  - **bearing capacity of plate**
  - **block shear**

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Continuing…

- **Brace force in compression**
  - **slip capacity of bolts (already checked for load in tension)**
  - **shear capacity of bolts (already checked for load in tension)**
  - **bearing capacity of plate (already checked)**
  - **block shear (doesn't apply)**
  - **capacity of gusset plate in compression (New)**

108

Slip load (calculate at factored load level)

$$R_n = \mu\, D_u\, h_f\, T_b\, n_s \quad \text{(per bolt)}$$

$\mu = 0.30$ (clean mill scale) $\quad h_f = 1.0$ (no fills))

$A_b = \pi\, d^2 / 4 = 0.60\ \text{in.}^2$ (7/8 in.dia.)

$F_u = 120$ ksi (A325 bolts)

$n_s = 2$ slip planes

$T_b =$ spec. min. bolt pretension $= (0.75 \times A_b)(F_u)70\%$

$= 0.75 \times 0.60\ \text{in.}^2 \times 120\ \text{ksi} \times 70\% = 37.88$ kips

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Slip load calculation cont'd.

$$R_n = \mu\, D_u\, h_f\, T_b\, N_s \quad \text{(per bolt)}$$

$= 0.30 \times 1.13 \times 1.0 \times 37.88\ \text{kip} \times 2$ slip planes (std.holes) :

$= 25.68$ kips / bolt

or, for 8 bolts, $(\phi = 1.0)$; $R_n = 205$ kips

110

Shear resistance of bolts

$$\phi R_n = \phi \; F_v \; A_b$$

Use ø =1.0 so that we can compare this load with the test load, assume threads in shear plane, no joint length effect

$$F_v = 90\% \, [0.62 \times 120 \text{ ksi}] = 68 \text{ ksi}$$

$$\phi R_n = 1.0 \times 68 \text{ ksi} \times 0.60 \text{ in.}^2 = 41.0 \text{ kips (per bolt)}$$

or, for 8 bolts, 2 shear planes, threads in shear plane

$$= (41.0 \times 8 \times 2)\text{kips} \times 0.80 = 525 \text{ kips}$$

111

Bearing resistance (use ϕ = 1.0)

$$R_n = 1.5 \; F_u \; L_c t \quad \le 3.0 \; d \; t \; F_u$$

$$3 \; d \; t \; F_u =$$

$$3 \times 7/8 \text{ in.} \times 0.26 \text{ in.} \times 69.0 \text{ ksi} = 47.1 \text{ k/bolt}$$

$$1.5 \; L_c \; t \; F_u =$$

$$1.5 \times 1.53 \text{ in.} \times 0.26 \text{ in.} \times 69.0 \text{ ksi} = 41.2 \text{ k}$$

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Bearing resistance…

…the governing value is 41.2 kips/bolt and, for 8 bolts—

Bearing resistance is 330 kips

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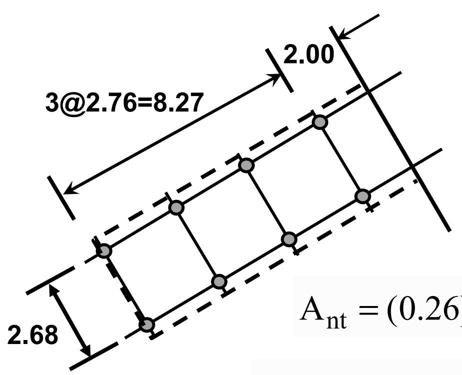
Block shear



$$A_{nt} = (0.26)(2.68 - 15/16) = 0.45 \text{ in.}^2$$

$$A_{gv} = (8.27 + 2.00)2 \times 0.26 = 5.34 \text{ in.}^2$$

$$T_r + V_r = \phi\, A_{nt}\, F_u + 0.60\, \phi\, A_{gv}\, F_y$$

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Block shear, cont'd

$$T_r = 0.45\text{ in.}^2 \times 69.0\text{ ksi} = 31.0\text{ kips}$$

$$V_r = 0.60 \times 5.34\text{ in.}^2 \times 39.9\text{ ksi} = 127.8\text{ kips}$$

and the total block shear resistance (unfactored) is (31 +128) =159 kips

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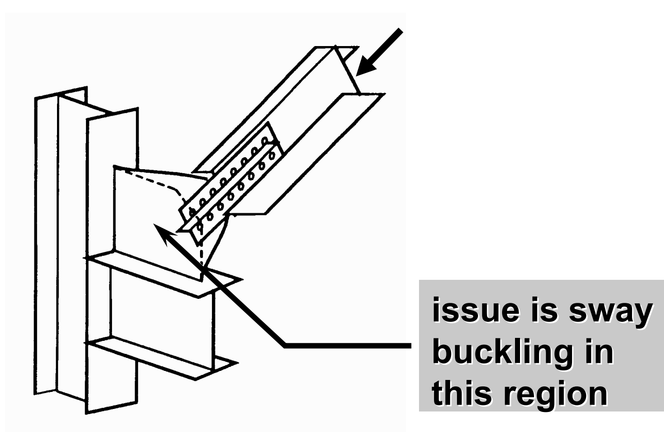
Brace force in compression:



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Checking the buckling…

- Whitmore method (checks yield)
- Thornton method (checks buckling)
- Modified Thornton method (checks buckling)

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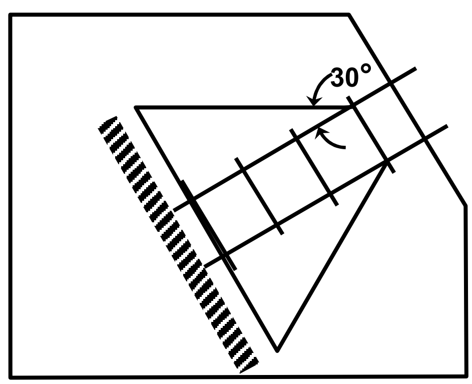
Whitmore method….

- Use beam formulae to check perceived critical sections
- Use 30^{o} , as shown to check yielding at location shown.
- Does not predict ultimate capacity very well, usually conservative but sometimes non-conservative



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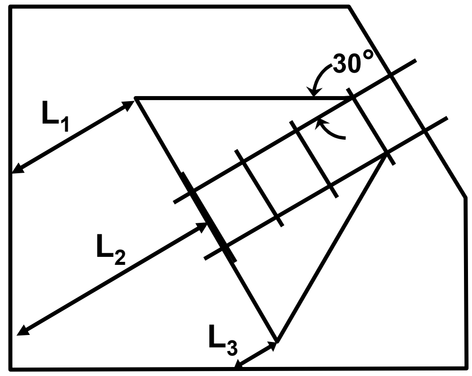
Thornton method…

- Use longest (or average) of L_1, L_2, L_3 to compute a buckling load on a unit width column, then apply this to the total width.
- Use k = 0.65 in the column formulae



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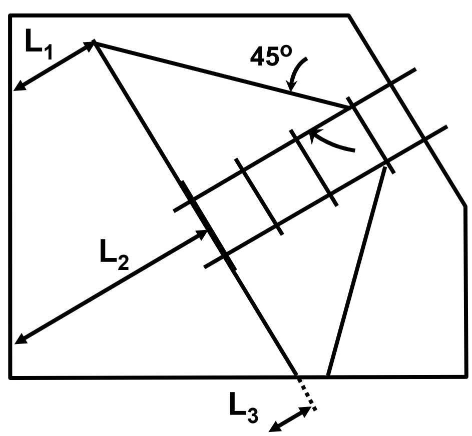
Thornton method, modified

As per Thornton method but spread load out at 45°



120

Yam & Cheng gusset plate tests (U of A, 13 tests)

	$\frac{P_u}{P_W}$	$\frac{P_u}{P_T}$	$\frac{P_u}{P_{T'}}$
mean	1.33	1.67	1.06
std. dev.	0.26	0.12	0.08

we'll use this method

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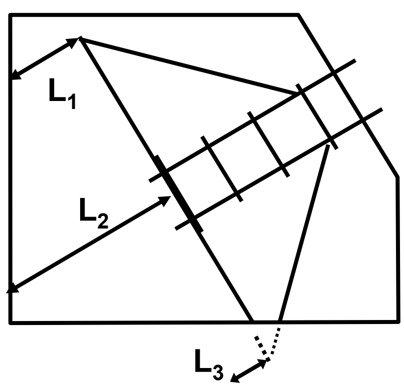
Calculations for buckling capacity:



Using scale dwg.
L_2 = 9.65 in.

Width of the 45° base is 19.2 in.

$$\phi_c\ P_n = \phi_c\ A_g\ F_{cr} \quad (\text{use } \phi_c = 1.0)$$

$$F_{cr} = (0.658^{\,F_y/F_e})\ F_y \qquad \text{use } k = 0.65$$

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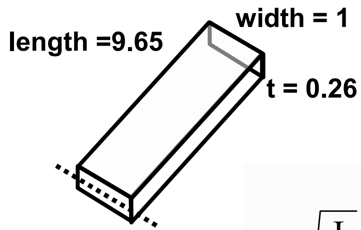
Consider a 1 in. wide strip that is 9.65 in. long



$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{1}{12}\times 1\times 0.26^3}{0.26\times 1}} = 0.0751 \text{ in.}$$

and then completing the calculations,
P_n = 6.91 kips (on a 1 in. wide strip)

123

And applying this to the total width…

P_u = (6.91 k/in.) (19.2 in.) = 132 kips

and the test ultimate load on this particular specimen was 164 kips

so, P_u / $P_{T'}$ = 1.23

(The corresponding ratios for Whitmore and Thornton for this specimen were 1.31 and 1.80)

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Summary of our calculations

Brace Force	slip load	bolt shear	plate bearing	block shear	buckling	test load
Tension	205	525	330	159	—	—
Compress.	—	—	—	—	132	164

125

Some references —

Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts, Research Council on Structural Connections, 2004 (RCSC)

(free download available at boltcouncil.org)

126

References, cont'd.

- G.L. Kulak, J.W. Fisher, and J.A.H. Struik, ***Guide to Design Criteria for Bolted and Riveted Joints*, Second Edition, John Wiley, New York, 1987 (free download at RCSC website)**
- **Bickford, John H., "An Introduction to the Design and Behavior of Bolted Joints," Second Edition, Marcel Dekker Inc., New York, 1990**
- **G.L. Kulak, *A Bolting Primer for Structural Engineers*, AISC Design Guide 17, Chicago, 2002**
- **Larry Kloiber and Larry Muir, "The 2010 AISC Specification: Changes in Design of Connections," Modern Steel Construction, Sept. 2010**

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