Design of Seismic-Resistant Steel Building Structures

3. Concentrically Braced Frames

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with the support of the
American Institute of Steel Construction.

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Design of Seismic-Resistant Steel Building Structures

1 - Introduction and Basic Principles

2 - Moment Resisting Frames

3 - Concentrically Braced Frames

4 - Eccentrically Braced Frames

5 - Buckling Restrained Braced Frames

6 - Special Plate Shear Walls

3 - Concentrically Braced Frames

- Description and Types of Concentrically Braced Frames
- Basic Behavior of Concentrically Braced Frames
- AISC Seismic Provisions for Special Concentrically Braced Frames

Concentrically Braced Frames

- Description and Types of Concentrically Braced Frames
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Concentrically Braced Frames (CBFs)

Beams, columns and braces arranged to form a vertical truss. Resist lateral earthquake forces by truss action.

Develop ductility through inelastic action in braces.

- braces yield in tension
- braces buckle in compression

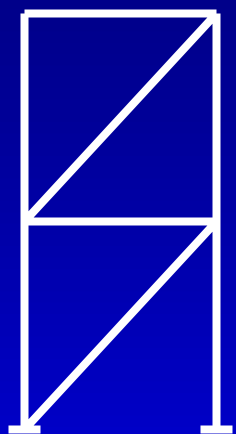
Advantages

- high elastic stiffness

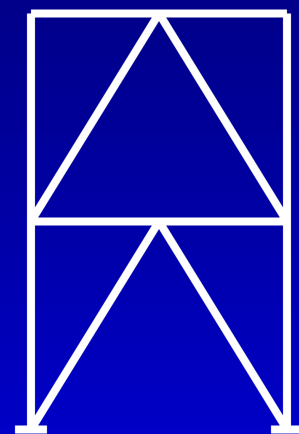
Disadvantages

- less ductile than other systems (SMFs, EBFs, BRBFs)
- reduced architectural versatility

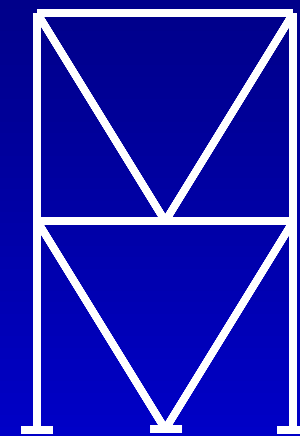
Types of CBFs



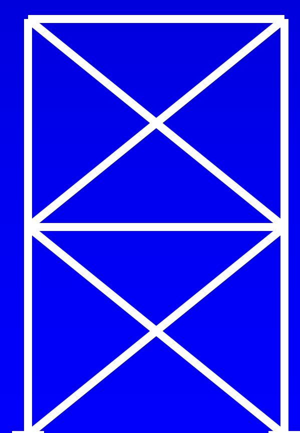
Single Diagonal



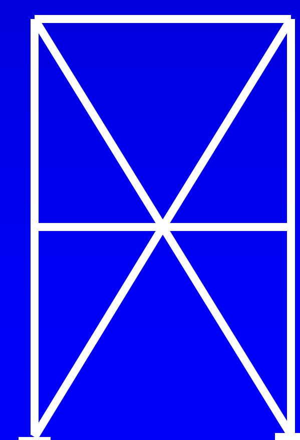
Inverted V- Bracing



V- Bracing



X- Bracing

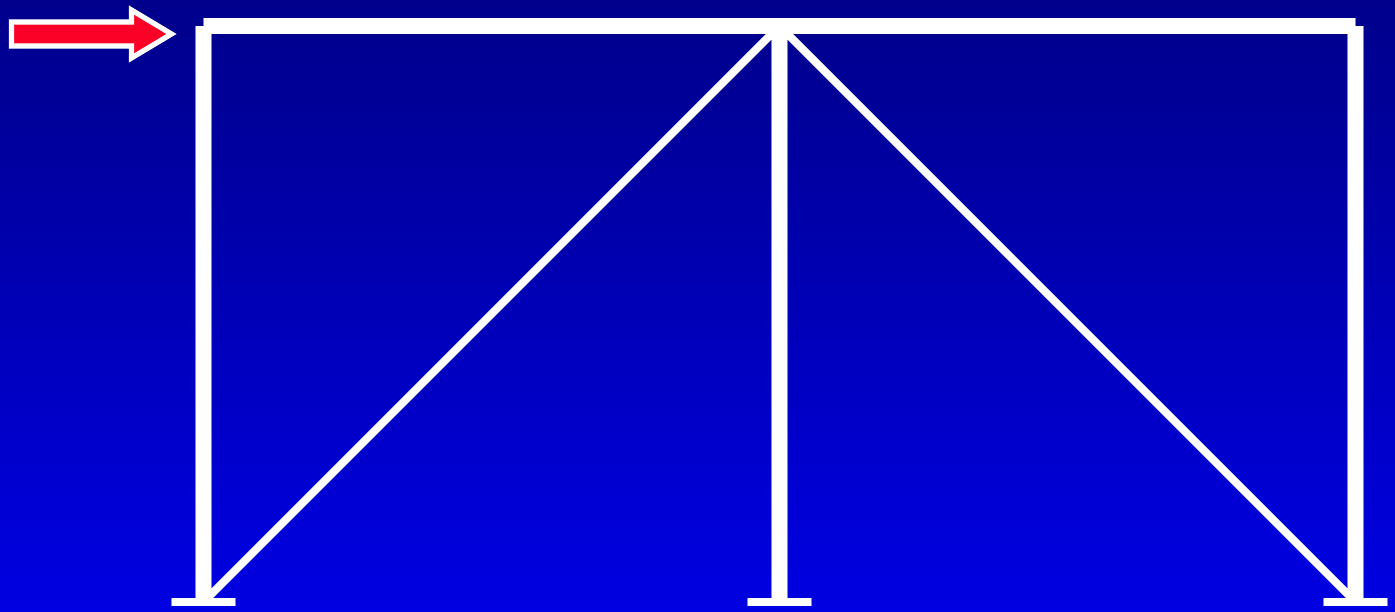


Two Story X- Bracing

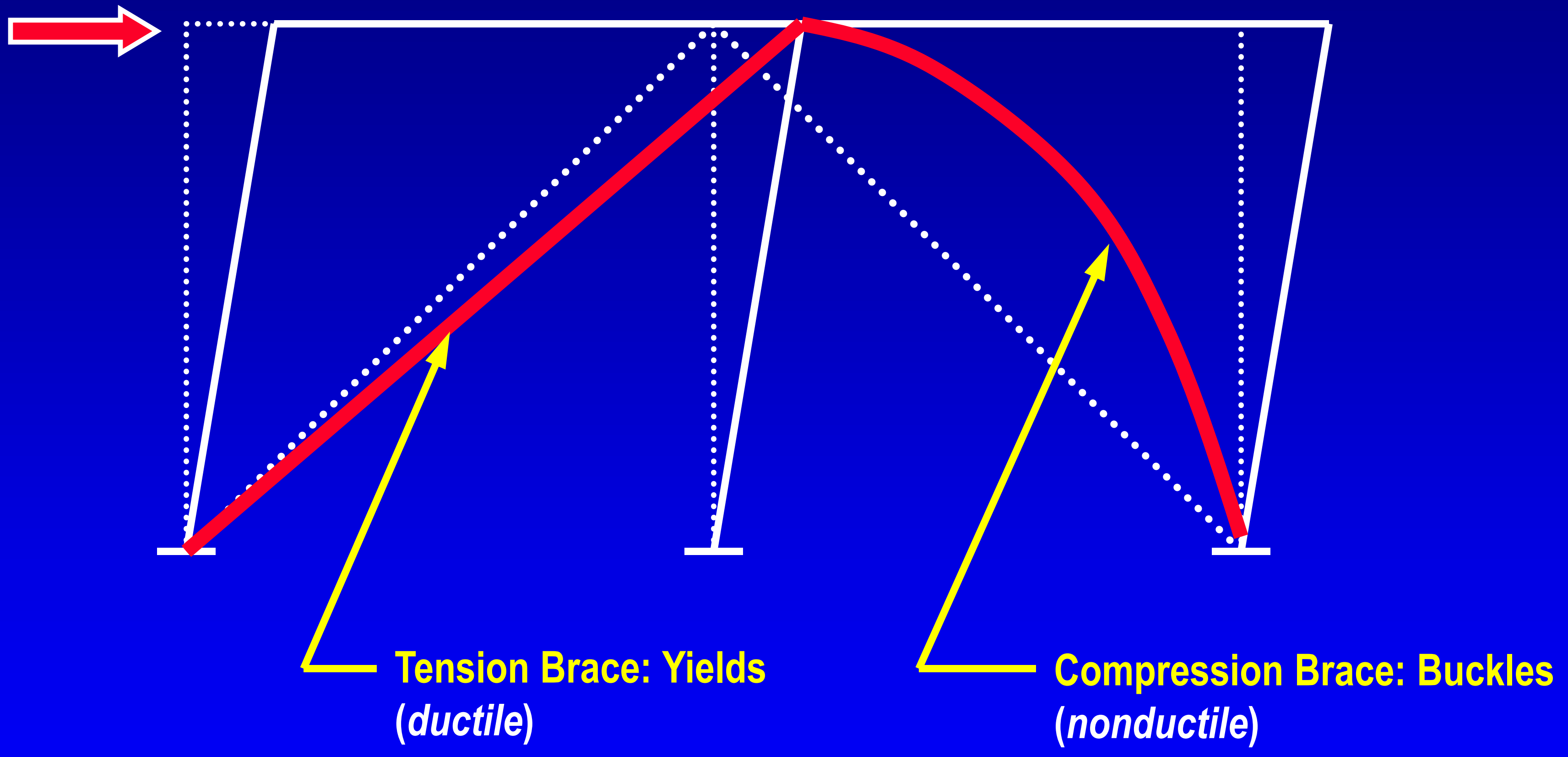
Concentrically Braced Frames

- Description and Types of Concentrically Braced Frames
- **Basic Behavior of Concentrically Braced Frames**
- AISC Seismic Provisions for Special Concentrically Braced Frames

Inelastic Response of CBFs under Earthquake Loading

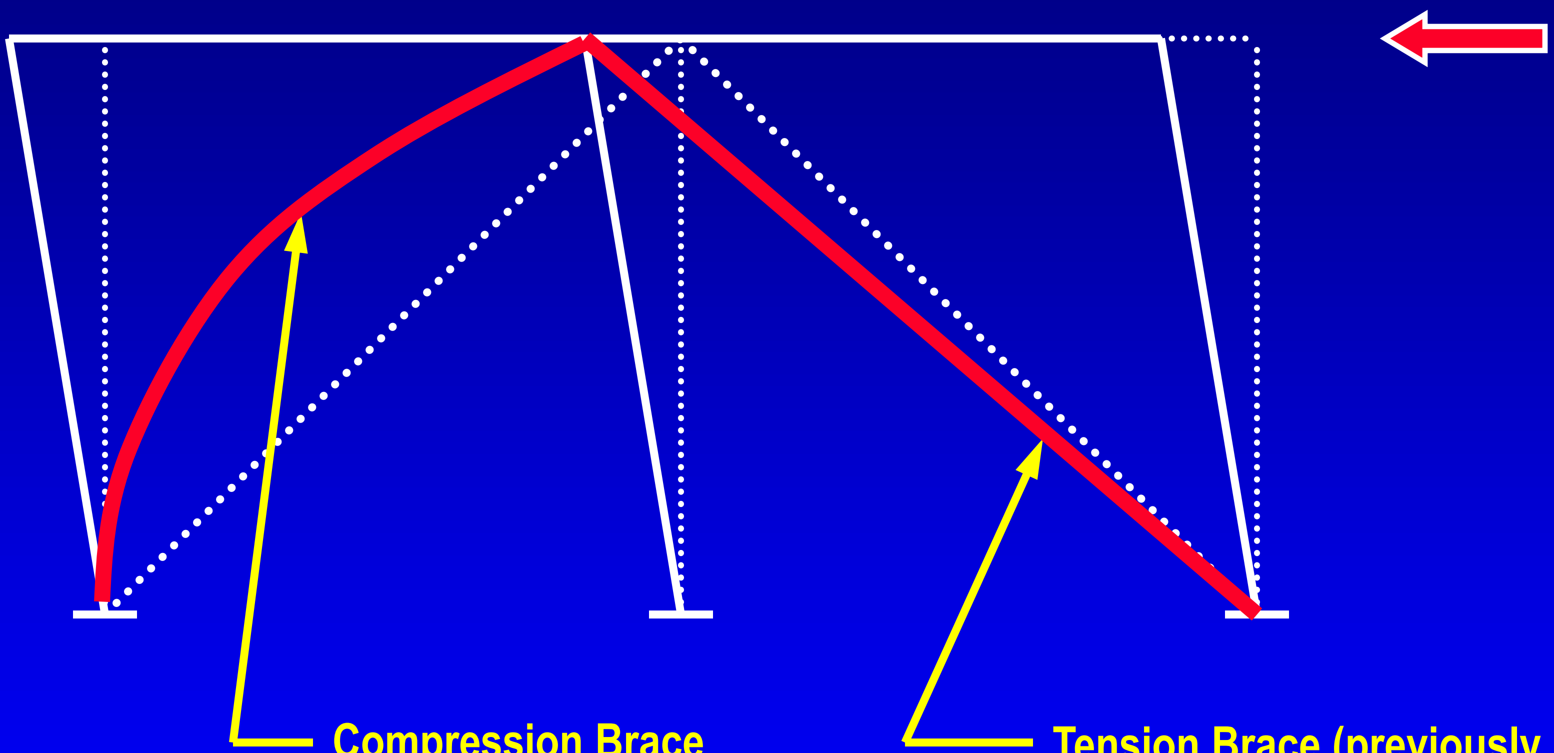


Inelastic Response of CBFs under Earthquake Loading



Columns and beams: remain essentially elastic

Inelastic Response of CBFs under Earthquake Loading

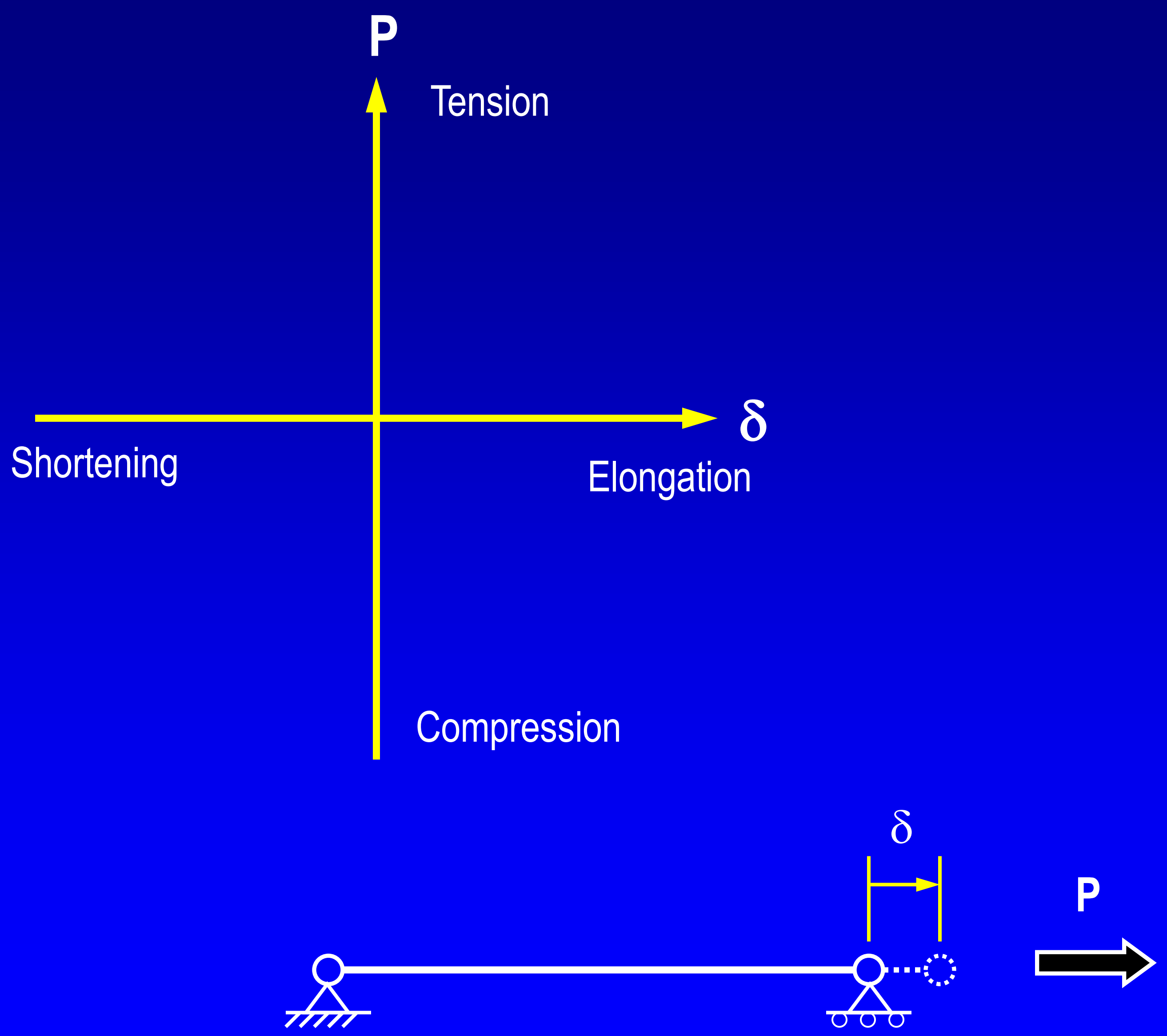


Compression Brace (previously in tension): Buckles (*nonductile*)

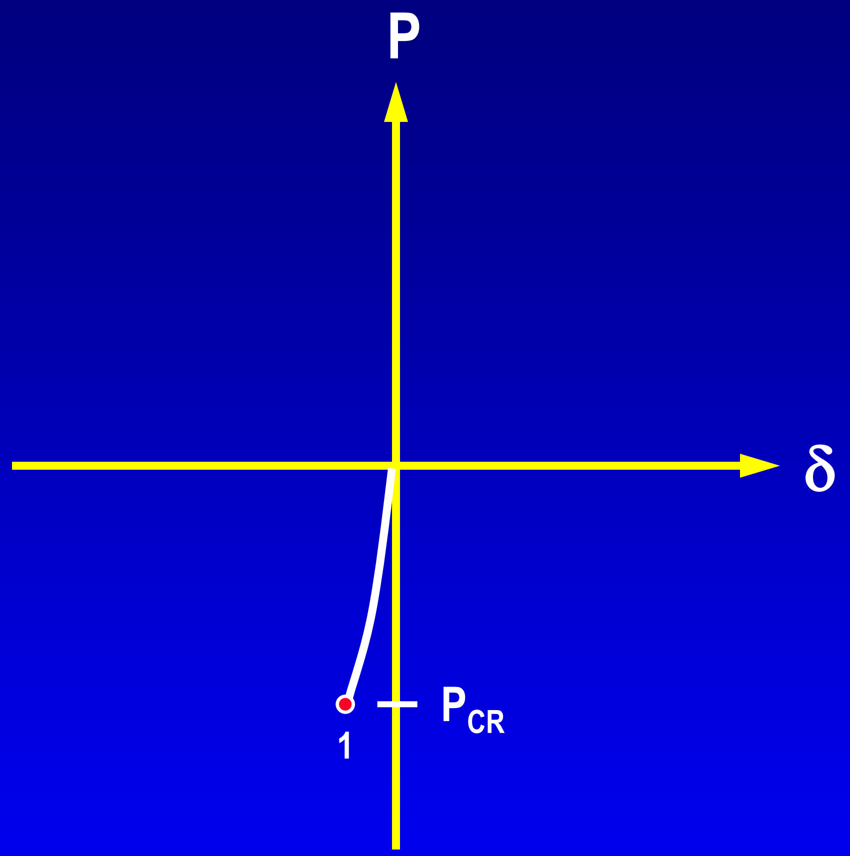
Tension Brace (previously in compression): Yields (*ductile*)

Columns and beams: remain essentially elastic

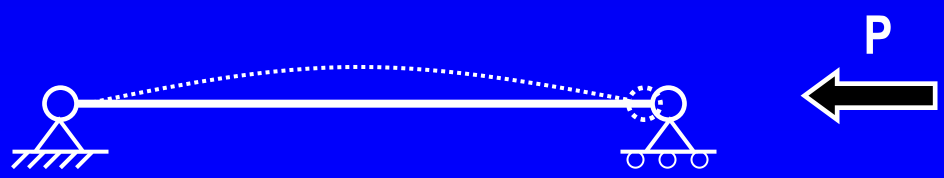
Brace Behavior Under Cyclic Axial Loading



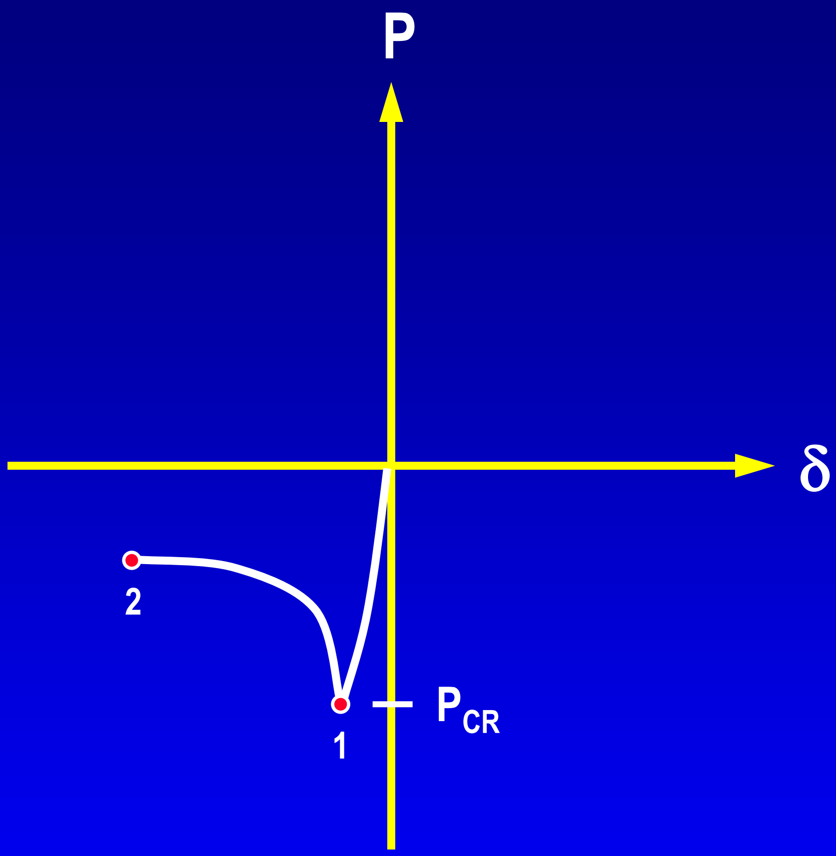
Brace Behavior Under Cyclic Axial Loading



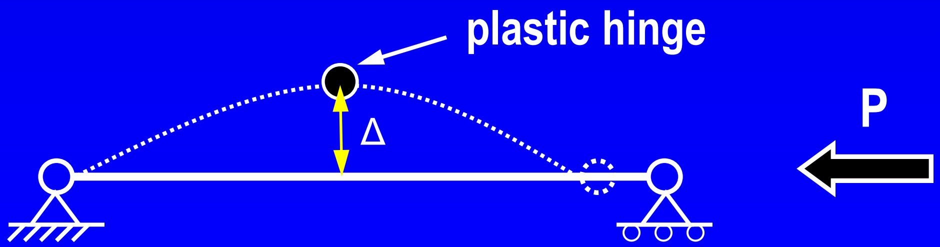
1. Brace loaded in compression to peak compression capacity (buckling).



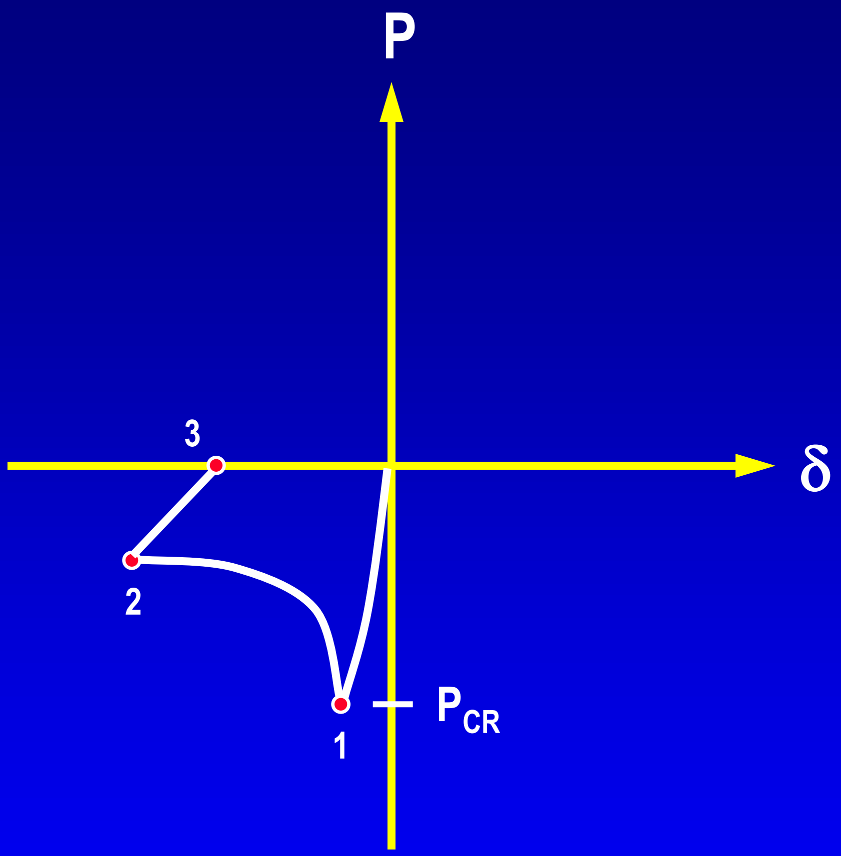
Brace Behavior Under Cyclic Axial Loading



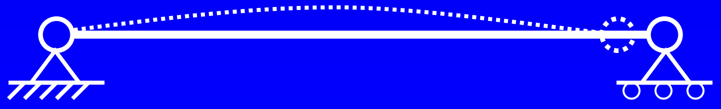
1. Brace loaded in compression to peak compression capacity (buckling).
2. Continue loading in compression. Compressive resistance drops rapidly. Flexural plastic hinge forms at mid-length (due to P-Δ moment in member).



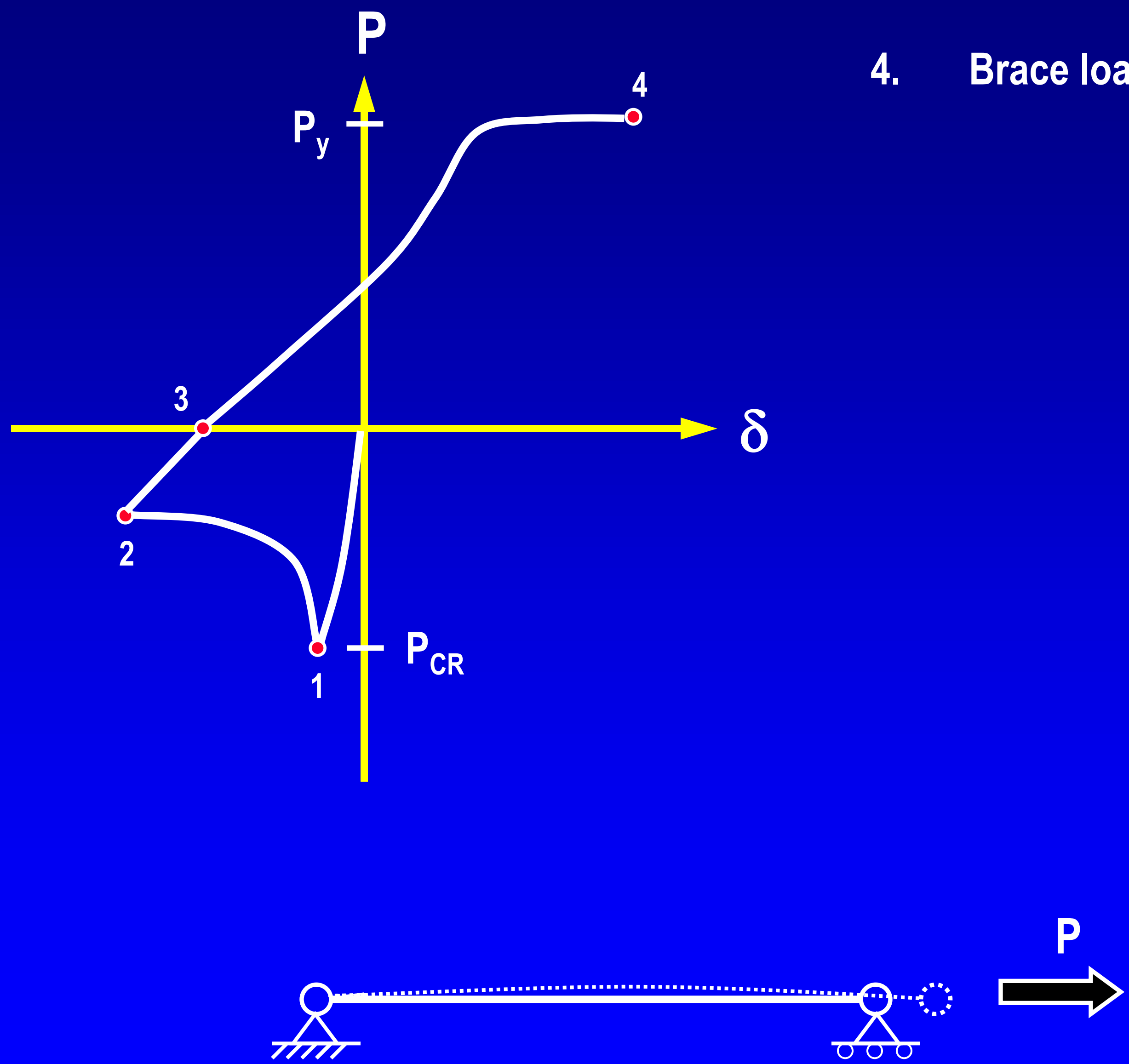
Brace Behavior Under Cyclic Axial Loading



1. Brace loaded in compression to peak compression capacity (buckling).
2. Continue loading in compression. Compressive resistance drops rapidly. Flexural plastic hinge forms at mid-length (due to P-Δ moment in member).
3. Remove load from member (P=0). Member has permanent out-of-plane deformation.

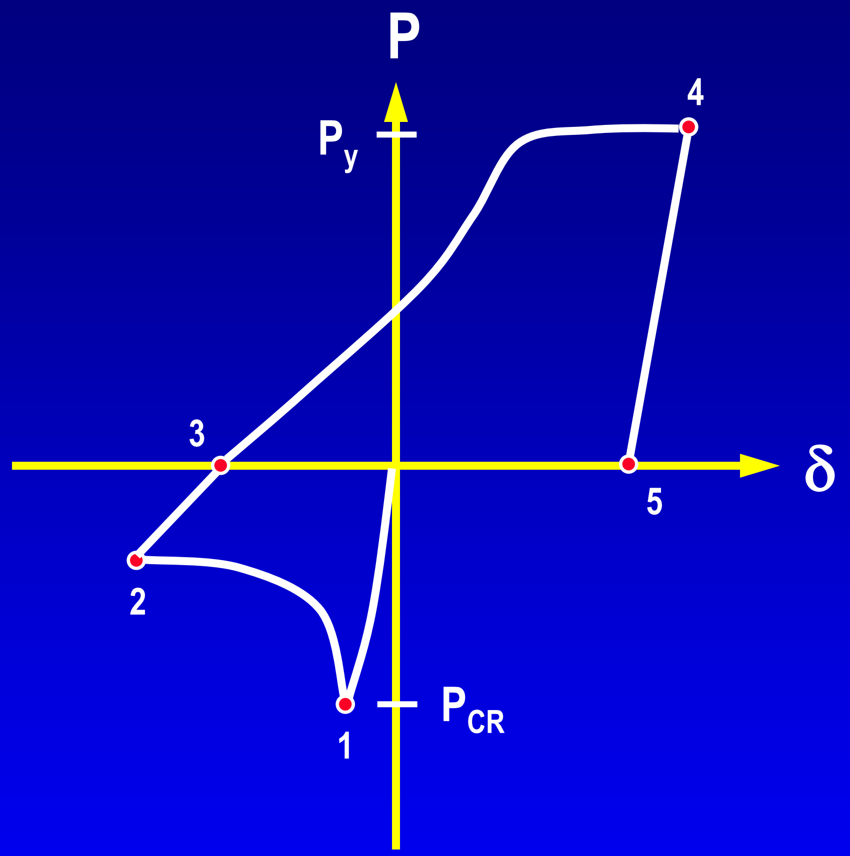


Brace Behavior Under Cyclic Axial Loading

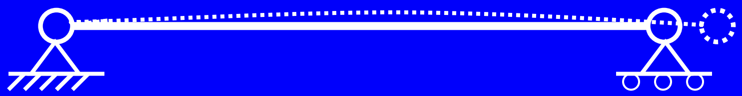


4. Brace loaded in tension to yield.

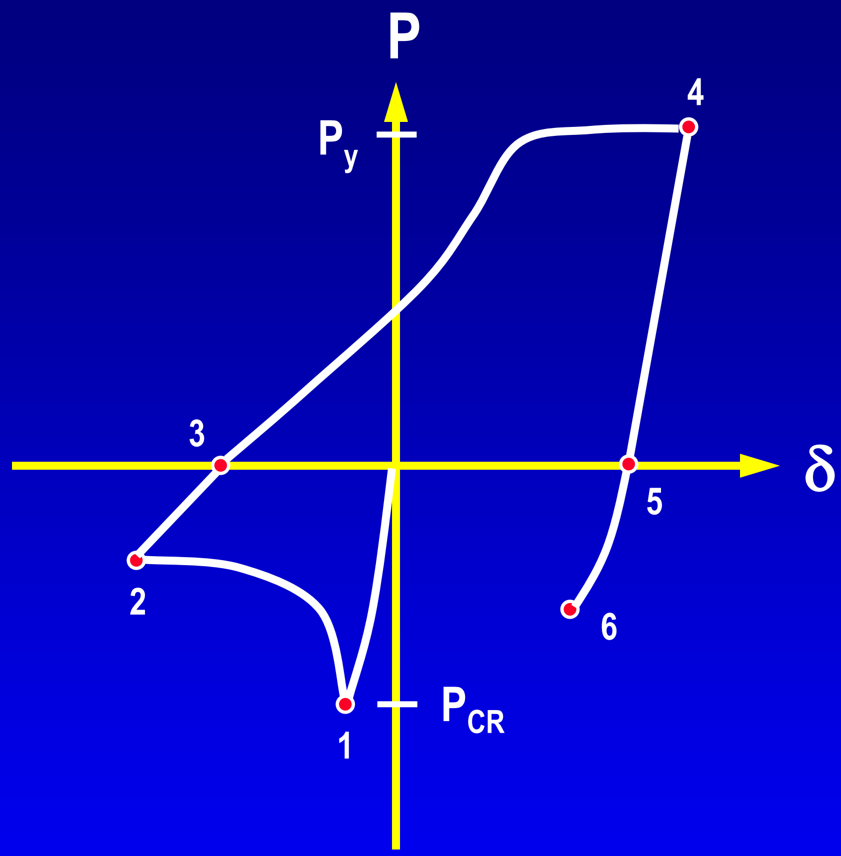
Brace Behavior Under Cyclic Axial Loading



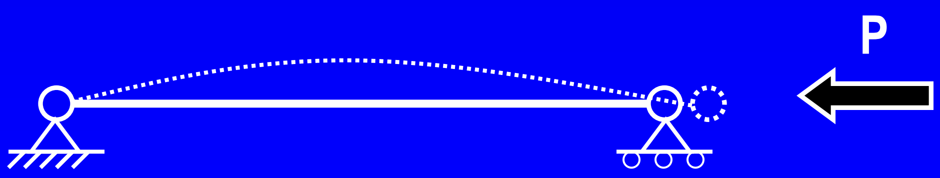
4. Brace loaded in tension to yield.
5. Remove load from member (P=0). Member still has permanent out-of-plane deformation.



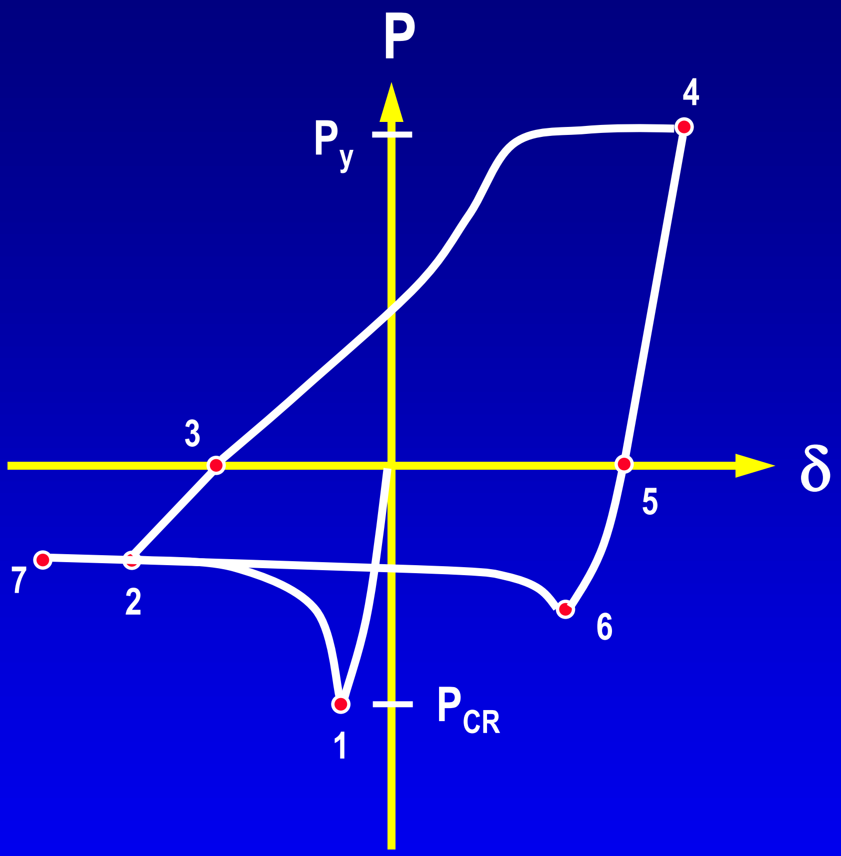
Brace Behavior Under Cyclic Axial Loading



4. Brace loaded in tension to yield.
5. Remove load from member (P=0). Member still has permanent out-of-plane deformation.
6. Brace loaded in compression to peak compression capacity (buckling). Peak compression capacity reduced from previous cycle.



Brace Behavior Under Cyclic Axial Loading

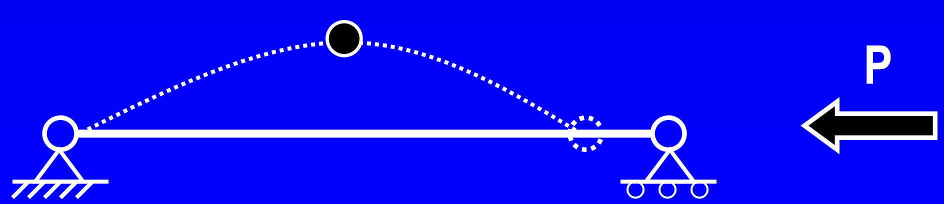


4. Brace loaded in tension to yield.

5. Remove load from member (P=0). Member still has permanent out-of-plane deformation.

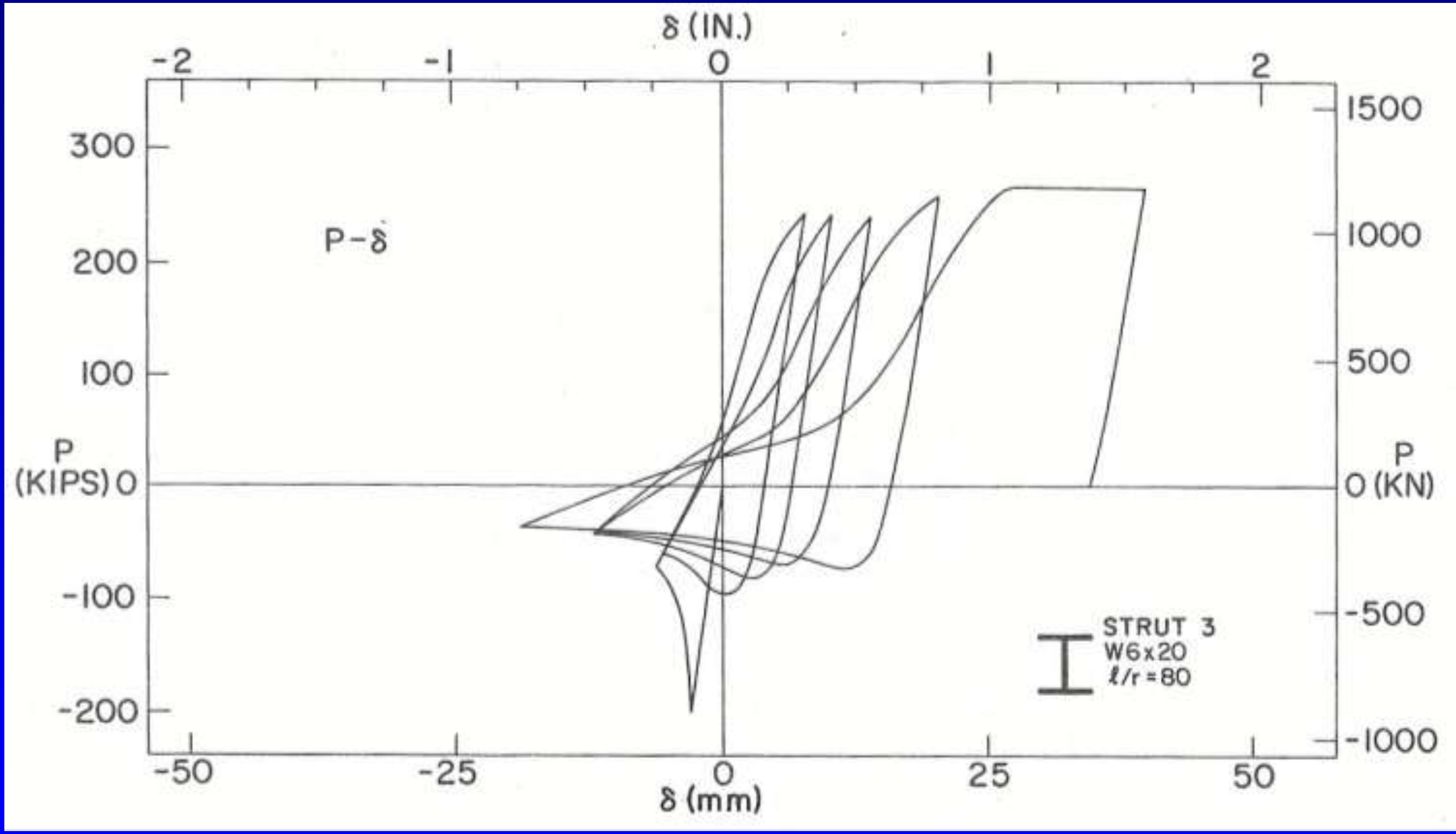
6. Brace loaded in compression to peak compression capacity (buckling). Peak compression capacity reduced from previous cycle.

7. Continue loading in compression. Flexural plastic hinge forms at mid-length (due to P-Δ moment in member).



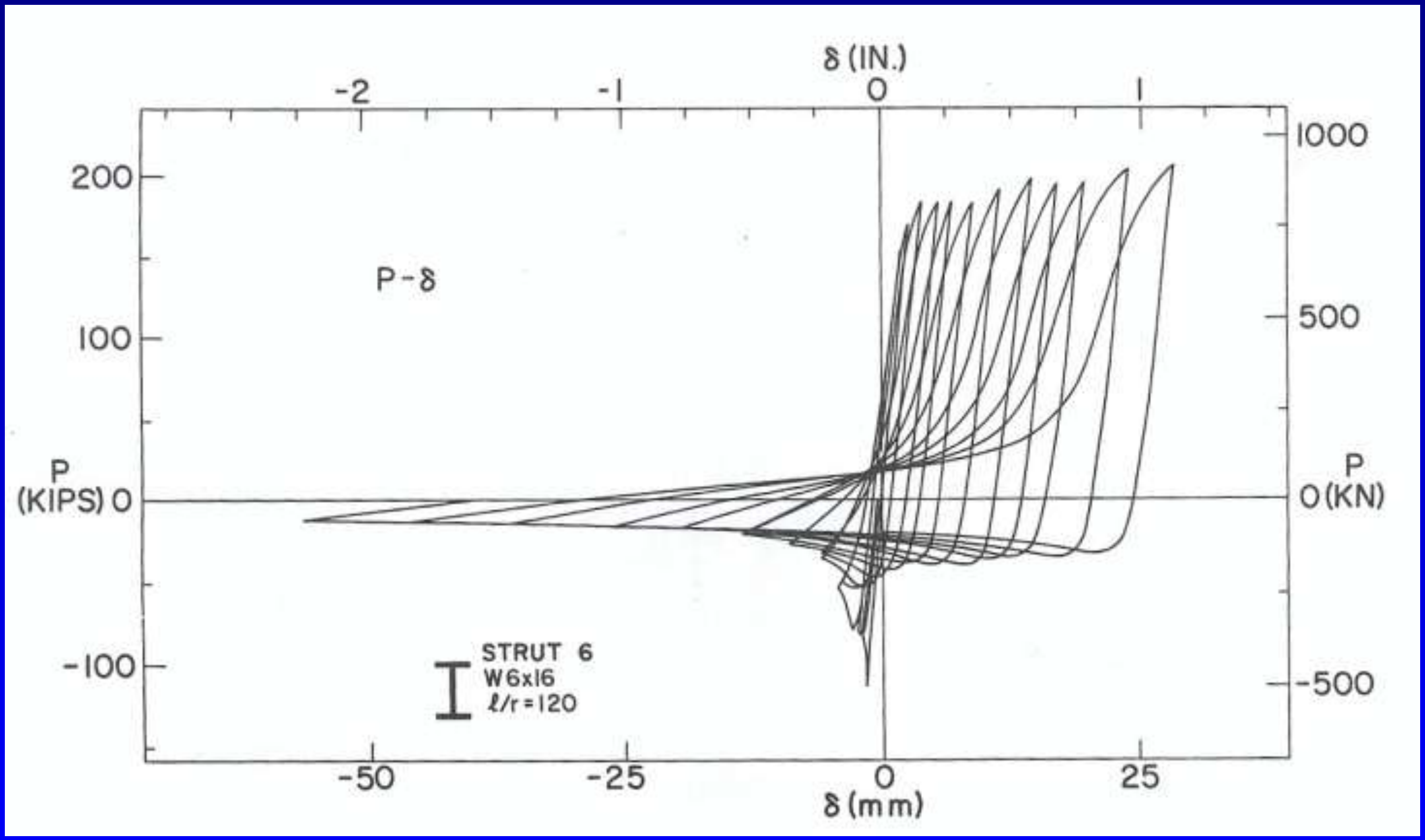
Experimental Behavior of Brace Under Cyclic Axial Loading

W6x20 Kl/r = 80

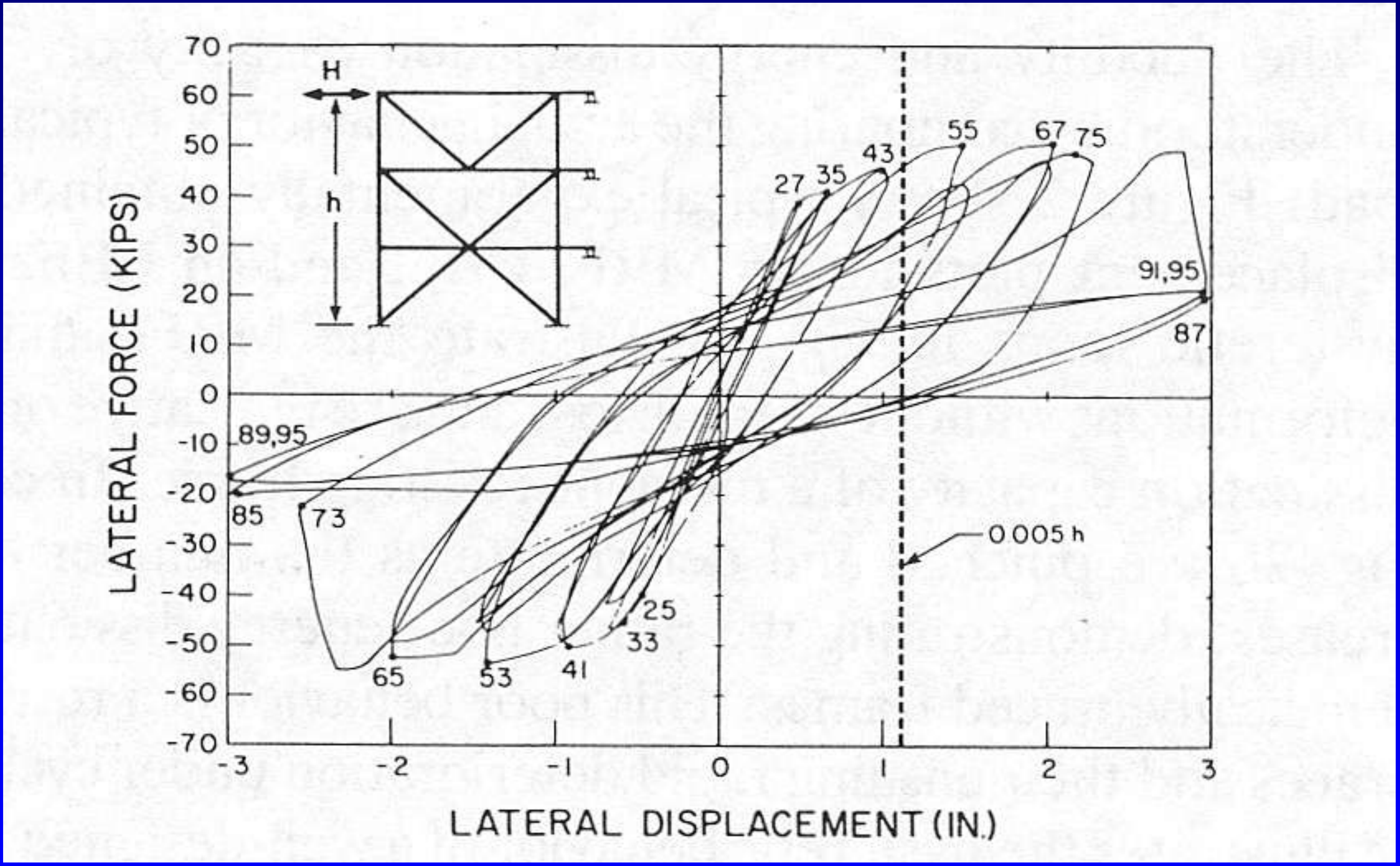


Experimental Behavior of Brace Under Cyclic Axial Loading

W6x16   Kl/r = 120

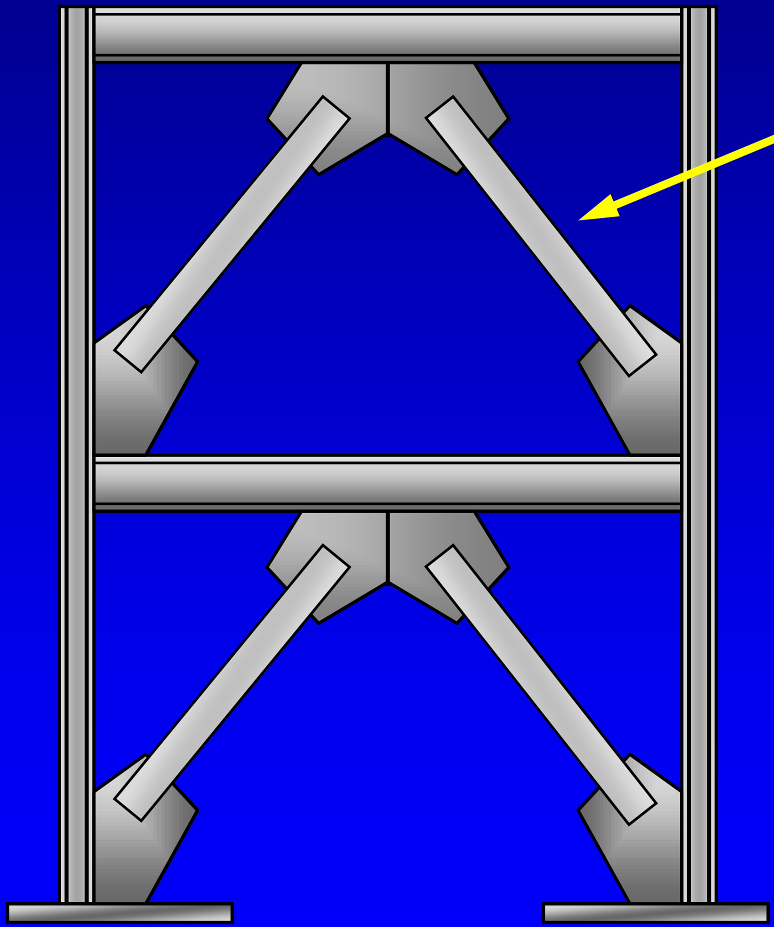


Experimental Behavior of Braced Frame Under Cyclic Loading



Developing Ductile Behavior in CBFs

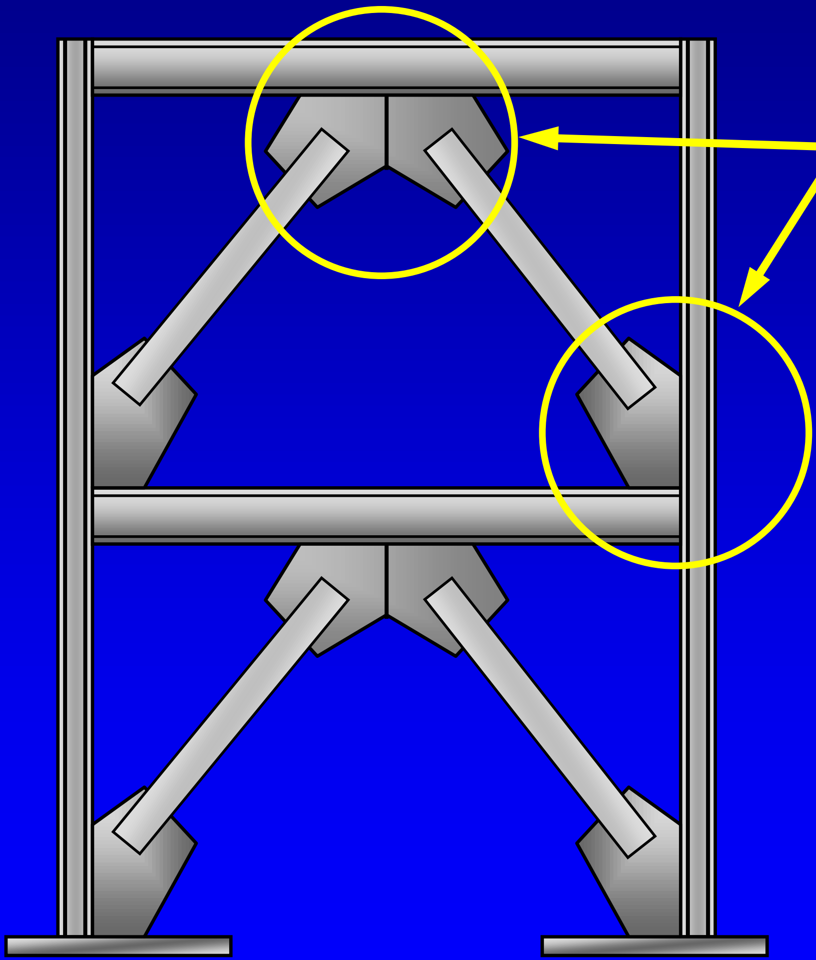
General Approach



- Design frame so that inelastic behavior is restricted to braces.
  - Braces are "fuse" elements of frame.
  - Braces are weakest element of frame. All other frame elements (columns, beams, connections) are stronger than braces.
- Choose brace members with good energy dissipation capacity and fracture life (limit kL/r and b/t).

Developing Ductile Behavior in CBFs

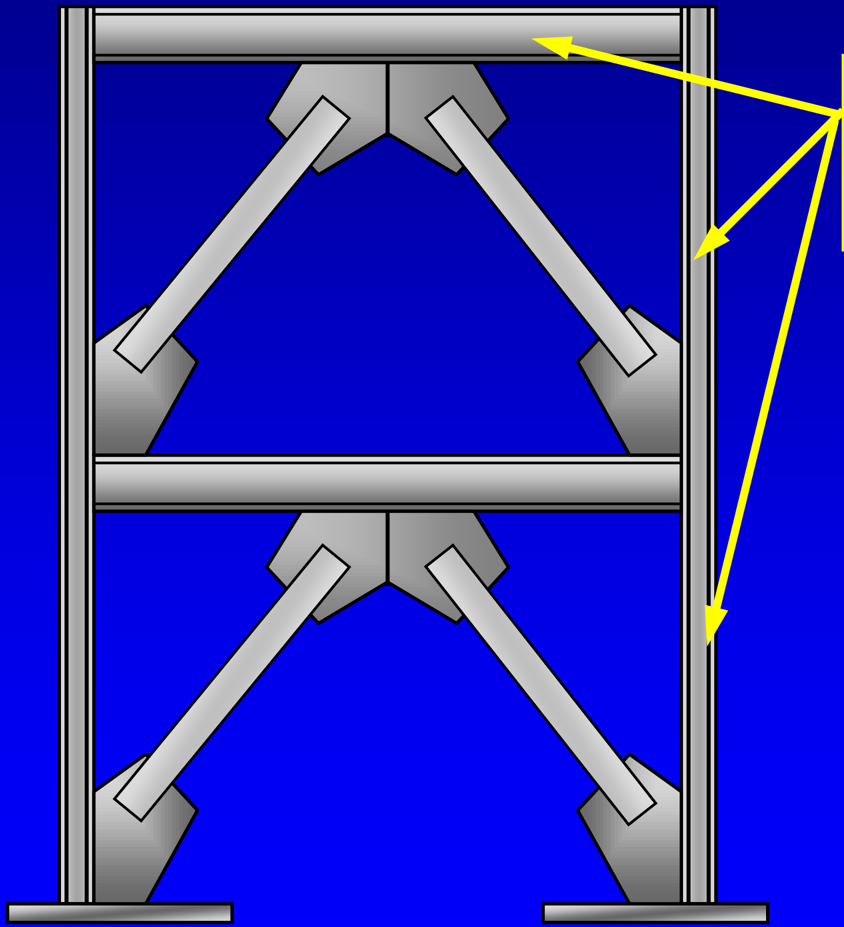
General Approach



- Design brace connections for maximum forces and deformations imposed by brace during cyclic yielding/buckling

Developing Ductile Behavior in CBFs

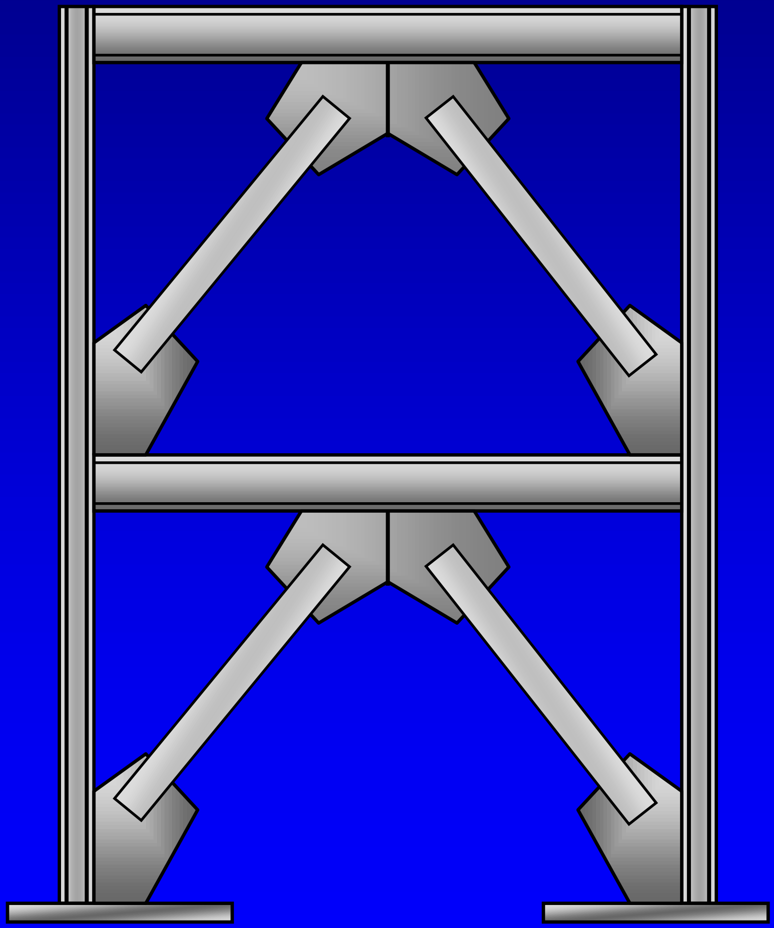
General Approach



- Design beams and columns (and column splices and column bases) for maximum forces imposed by braces

Developing Ductile Behavior in CBFs

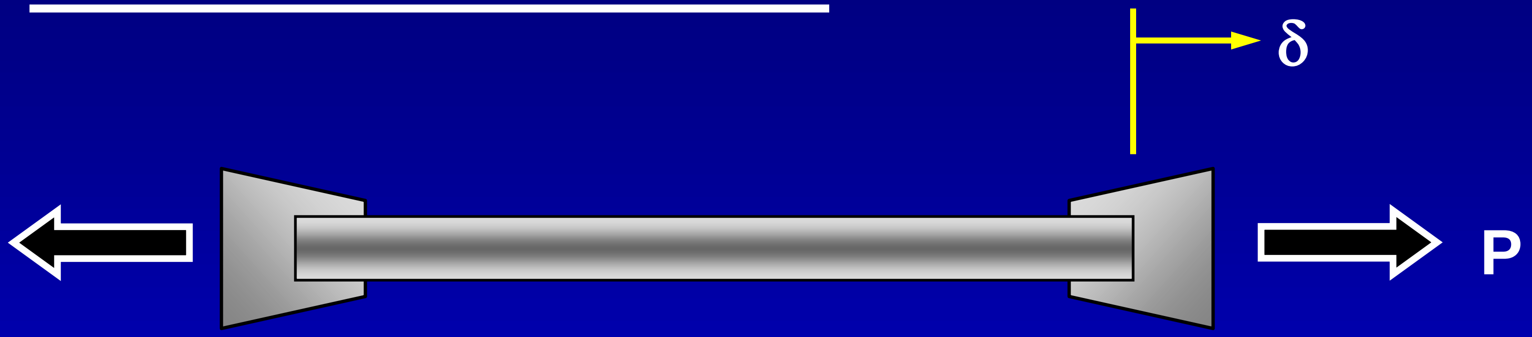
General Approach

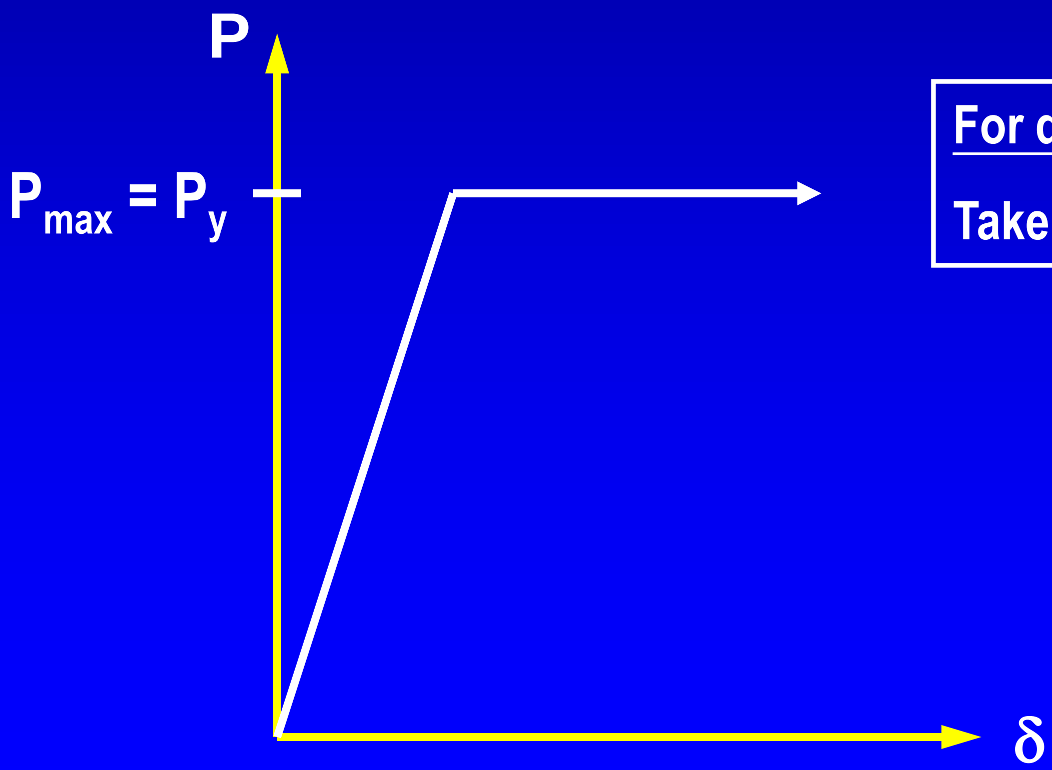


- Design braces based on code specified earthquake forces.
- Design all other frame elements for maximum forces that can be developed by braces.

Maximum Forces Developed by Braces

Braces in Tension - Axial Force:



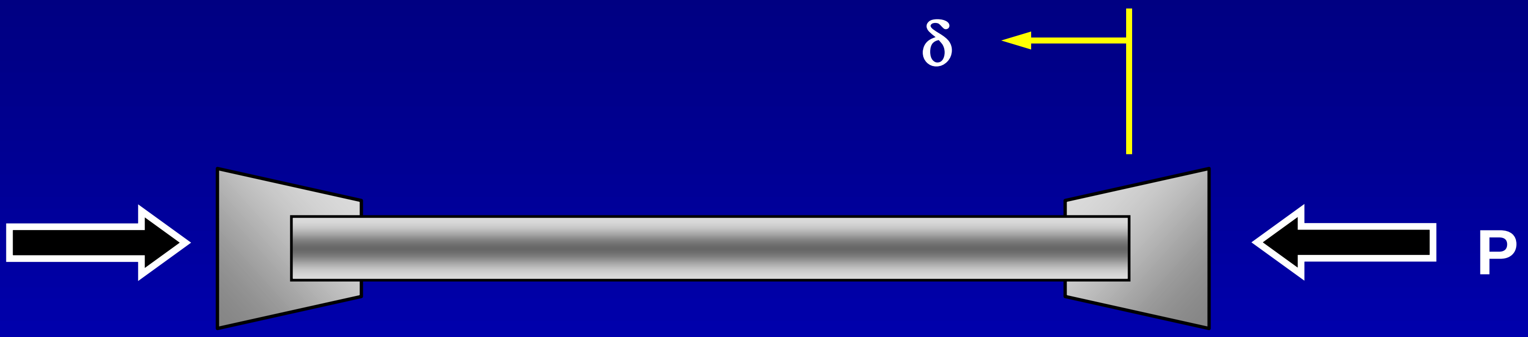


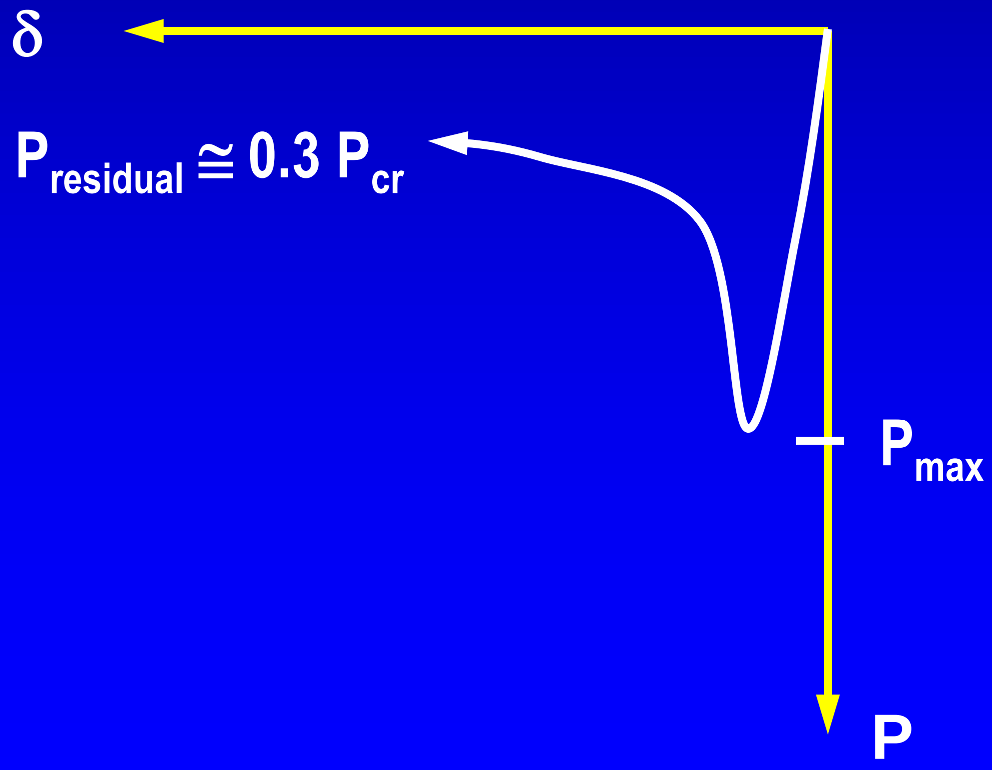
For design:

Take $P_{max} = R_y F_y A_g$

Maximum Forces Developed by Braces

Braces in Compression - Axial Force





For design:

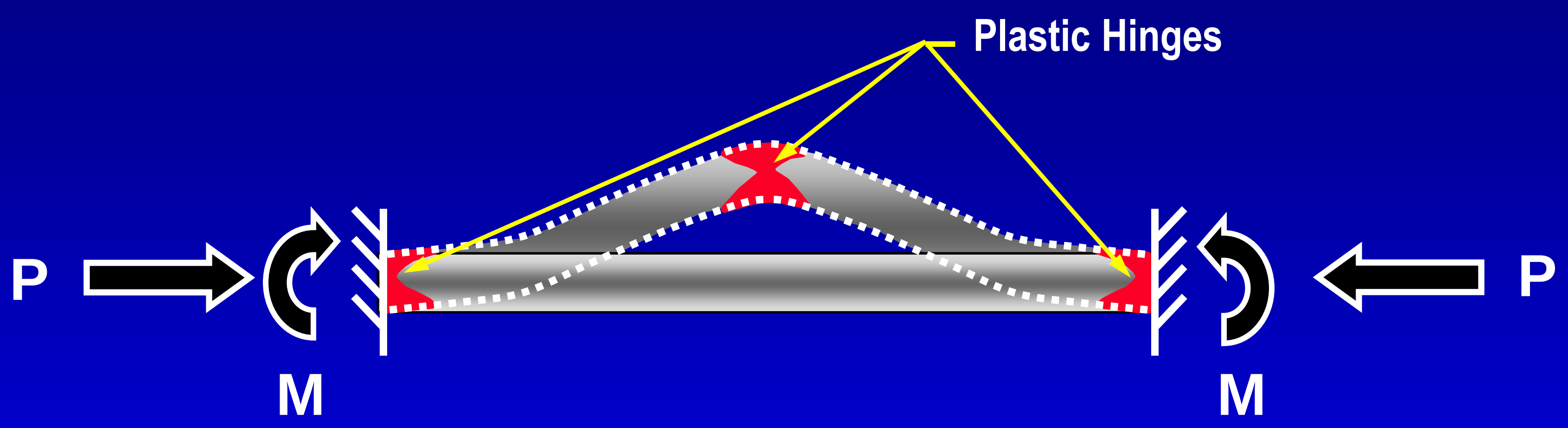
Take $P_{max} = 1.1\ R_y\ P_n$

$(\ P_n = A_g\ F_{cr}\ )$

Take $P_{residual} = 0.3\ P_n$

Maximum Forces Developed by Braces

Braces in Compression - Bending Moment:



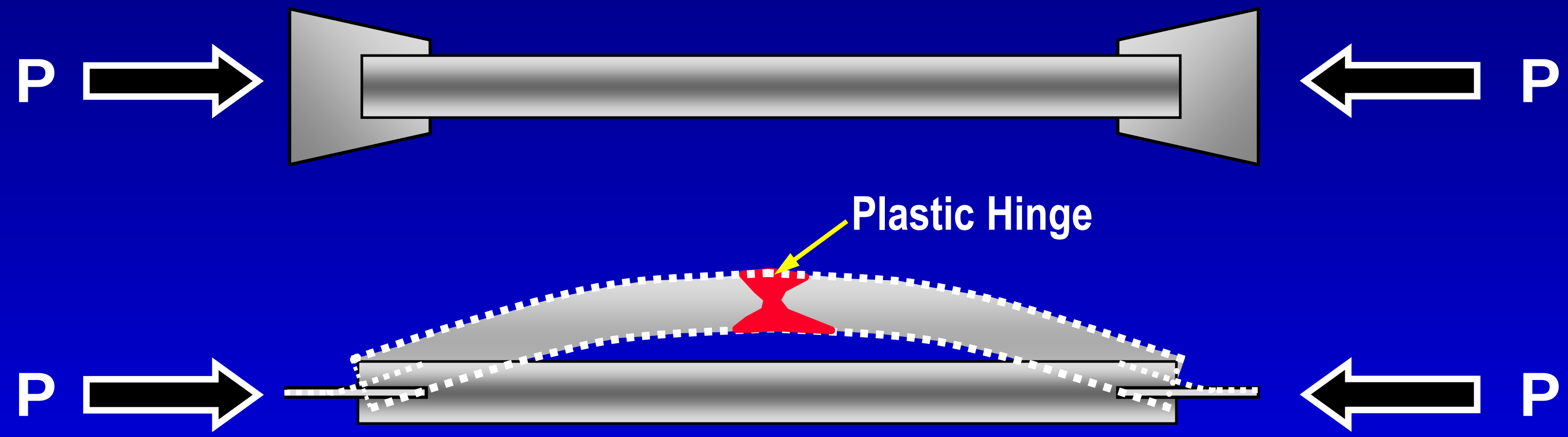
For "fixed" end braces: flexural plastic hinges will form at mid-length and at brace ends. Brace will impose bending moment on connections and adjoining members.

For design:

Take $M_{max} = 1.1\ R_y\ F_y\ Z_{brace}$ (for critical buckling direction)

Maximum Forces Developed by Braces

Braces in Compression - Bending Moment:



For "pinned" end braces: flexural plastic hinge will form at mid-length only. Brace will impose no bending moment on connections and adjoining members.

Must design brace connection to behave like a "pin"

Maximum Forces in Columns and Beams

To estimate maximum axial forces imposed by braces on columns and beams:

Braces in tension:

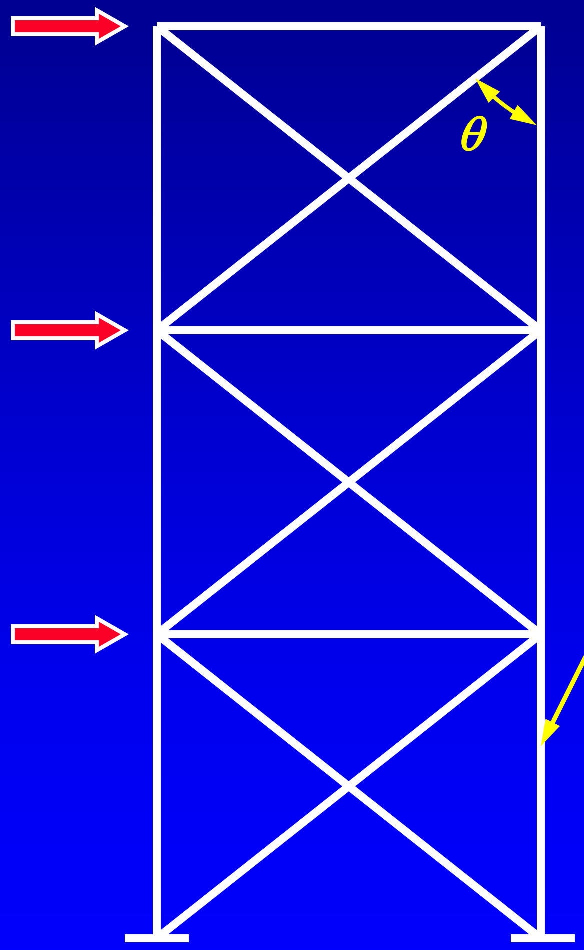
Take $P = R_y F_y A_g$

Braces in compression:

Take $P = 1.1\ R_y P_n$ or $P = 0.3\ P_n$

whichever produces critical design case

Example

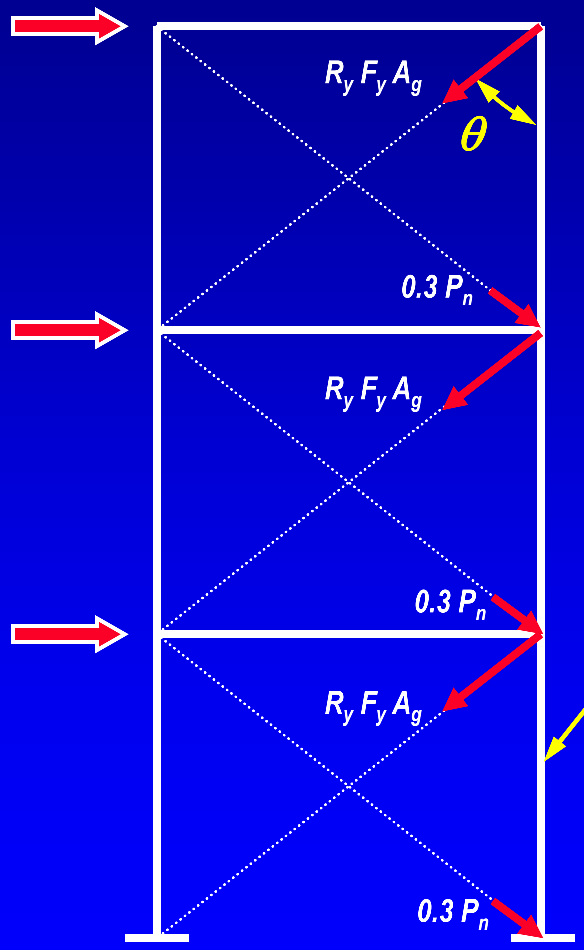


Find maximum axial compression in column.

Tension Braces:
Take $P = R_y F_y A_g$

Compression Braces:
Take $P = 0.3\ P_n$

Example



Column Axial Compression =

$[\ \Sigma\ (R_y\ F_y\ A_g\ ) \cos\theta + \Sigma\ (0.3\ P_n) \cos\theta\ ] + P_{gravity}$

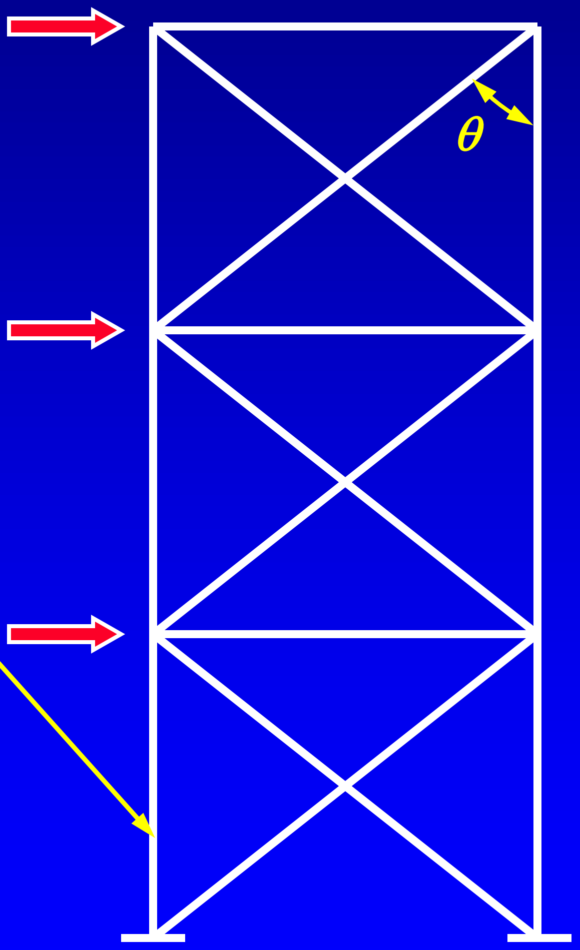
(sum brace forces for all levels above column)

Example

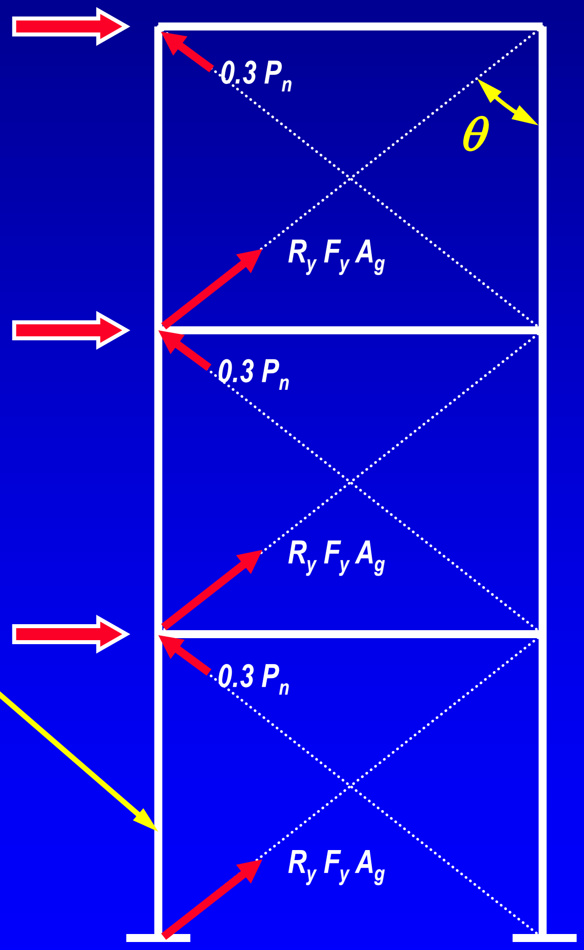
Find maximum axial tension in column.

Tension Braces:
Take $P = R_y F_y A_g$

Compression Braces:
Take $P = 0.3\ P_n$



Example

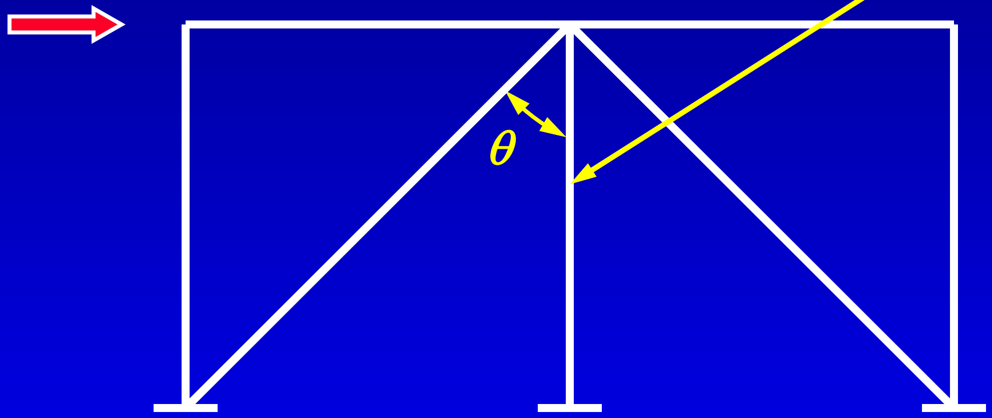


Column Axial Tension =

$[\ \Sigma\ (R_y\ F_y\ A_g\ )\ \cos\theta + \Sigma\ (0.3\ P_n)\ \cos\theta\ ] - P_{gravity}$

(sum brace forces for all levels above column)

Example

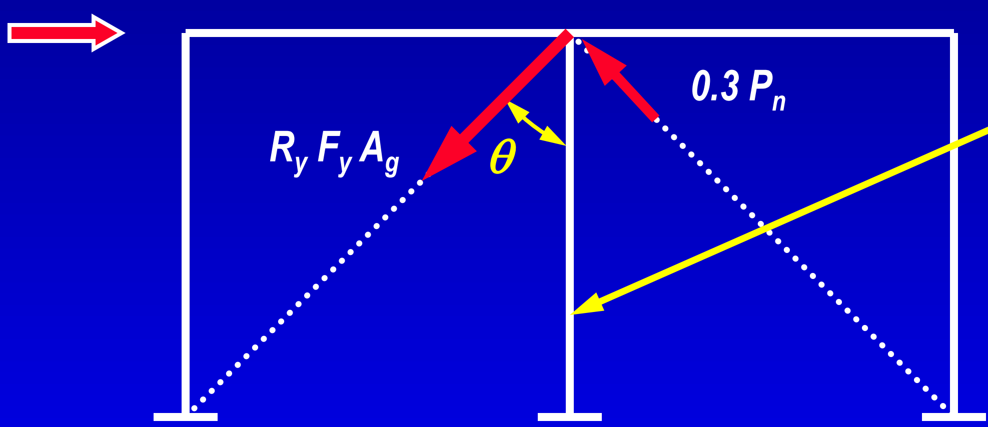


Find maximum axial compression in column.

Tension Brace:
Take $P = R_y F_y A_g$

Compression Brace:
Take $P = 0.3\, P_n$

Example



Column Axial Compression =

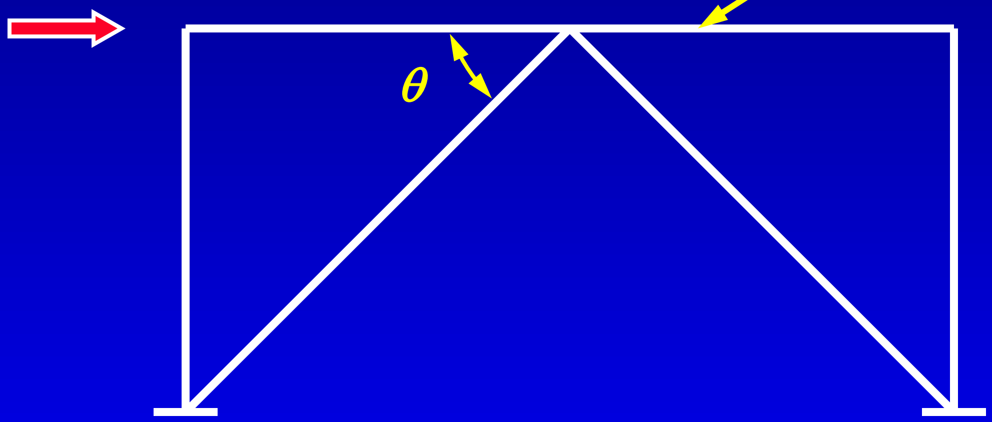
$(R_y\ F_y\ A_g)\cos\theta + (0.3\ P_n)\cos\theta + P_{gravity}$

Note

Based on elastic frame analysis:

Column Axial Force = $P_{gravity}$

Example

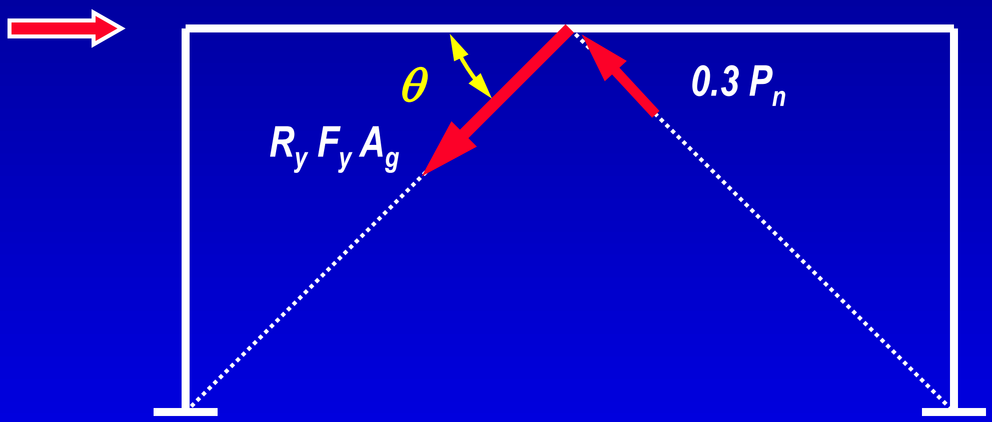


Find maximum bending moment in beam.

Tension Brace:
Take $P = R_y F_y A_g$

Compression Brace:
Take $P = 0.3 P_n$

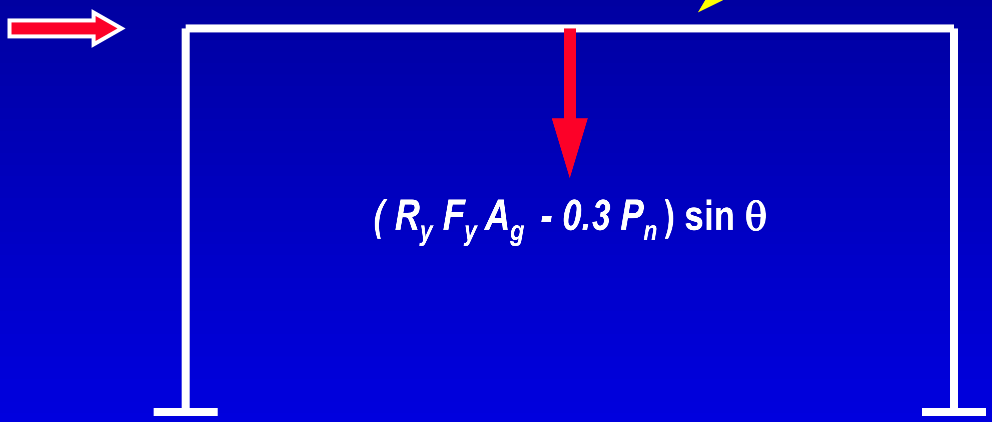
Example



Example

Compute moment in beam resulting from application of concentrated load at midspan of $(R_y F_y A_g + 0.3 P_n) \sin\theta$

and add moment due to gravity load

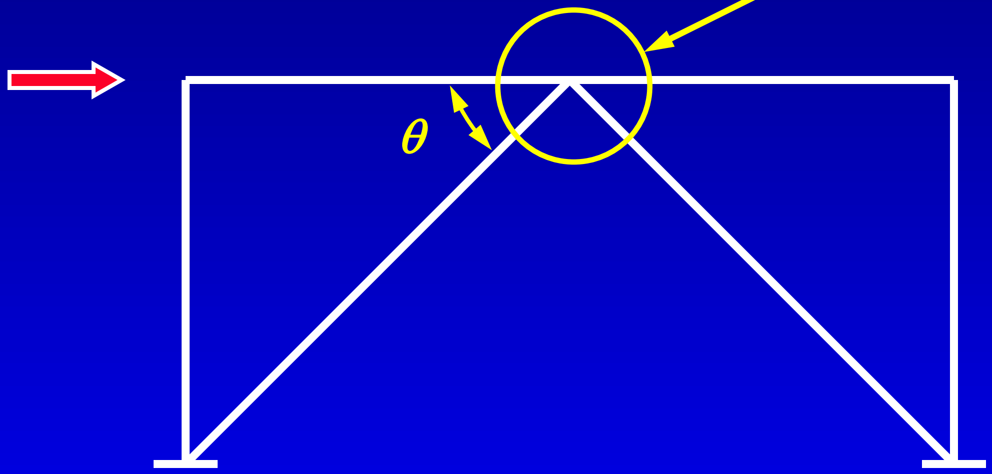


Note

Based on elastic frame analysis:

Moment in beam $\cong 0$

Example



Find maximum axial tension and compression that will be applied to gusset plate.

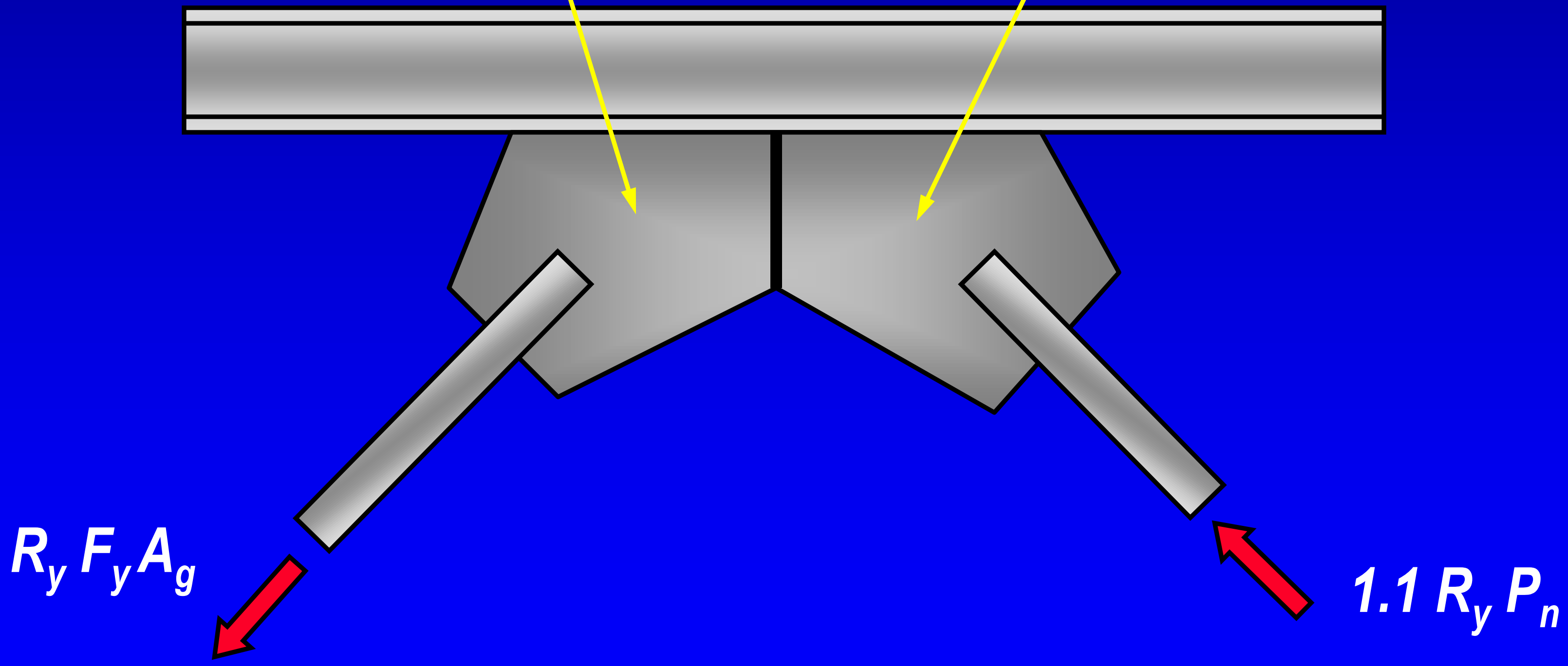
Tension Brace:
Take $P = R_y F_y A_g$

Compression Brace:
Take $P = 1.1\ R_y P_n$

Example

Check gusset yield, gusset net section fracture, gusset block shear fracture, local beam web yielding, etc.

Check gusset buckling, beam web crippling, etc.



Concentrically Braced Frames

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- AISC Seismic Provisions for Special Concentrically Braced Frames

2005 AISC Seismic Provisions

Section 13 **Special Concentrically Braced Frames (SCBF)**

Section 14 **Ordinary Concentrically Braced Frames (OCBF)**

Section 13
Special Concentrically Braced Frames (SCBF)

13.1 Scope

13.2 Members

13.3 Required Strength of Bracing Connections

13.4 Special Bracing Configuration Requirements

13.5 Column Splices

13.6 Protected Zone

AISC Seismic Provisions - SCBF

13.1 Scope

Special concentrically braced frames (SCBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake.

13.2 Members

13.2a Slenderness

Bracing members shall have: $$\frac{KL}{r} \leq 4\sqrt{\frac{E}{F_y}}$$

F_y = 36 ksi:	$KL/r \leq 114$
F_y = 42 ksi:	$KL/r \leq 105$
F_y = 46 ksi:	$KL/r \leq 100$
F_y = 50 ksi:	$KL/r \leq 96$

13.2a Slenderness

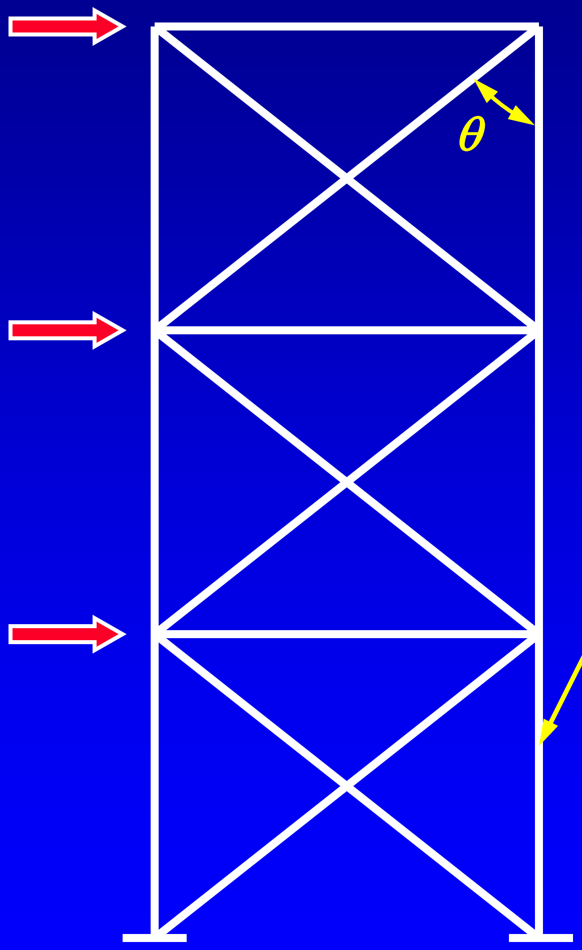
Bracing members shall have: $\dfrac{KL}{r} \leq 4\sqrt{\dfrac{E}{F_y}}$

Exception:

Braces with: $4\sqrt{\dfrac{E}{F_y}} \leq \dfrac{KL}{r} \leq 200$

are permitted in frames in which the available strength of the columns is at least equal to the maximum load transferred to the column considering R_y times the nominal strengths of the brace elements.

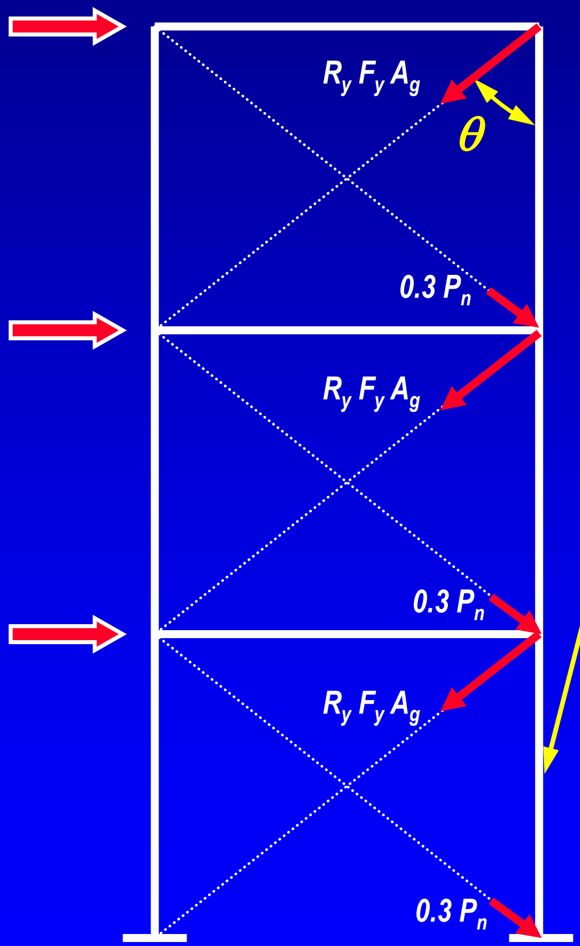
Example



Find required axial compression strength of column.

Example

All bracing members: $\dfrac{KL}{r} \le 4\sqrt{\dfrac{E}{F_y}}$



Required column axial compression strength =

$[\ \Sigma\ (R_y F_y A_g\ ) \cos\theta + \Sigma\ (0.3\ P_n) \cos\theta\ ]$
$+$
$\Sigma\ [(1.2 + 0.2S_{DS})\ D + 0.5L]$

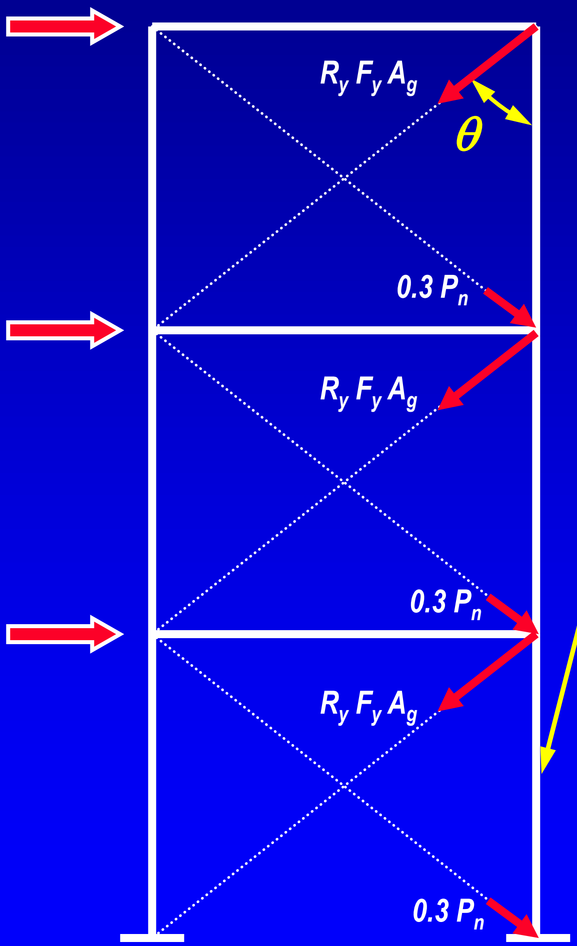
OR

$\Omega_0\ Q_E$
$+$
$\Sigma\ [(1.2 + 0.2S_{DS})\ D + 0.5L]$

Note: $\Omega_0 = 2$ for SCBF and OCBF

Example

Bracing members with: $4\sqrt{\frac{E}{F_y}} \le \frac{KL}{r} \le 200$



Required column axial compression strength =

$$[\ \Sigma\ (R_y F_y A_g)\cos\theta - \Sigma\ (0.3\ P_n)\cos\theta\ ]$$

$$+$$

$$\Sigma\ [(1.2 + 0.2S_{DS})\ D + 0.5L]$$

$$\cancel{\Omega_0\ Q_E + \Sigma\ [(1.2 + 0.2S_{DS})\ D + 0.5L]}$$

NOT PERMITTED

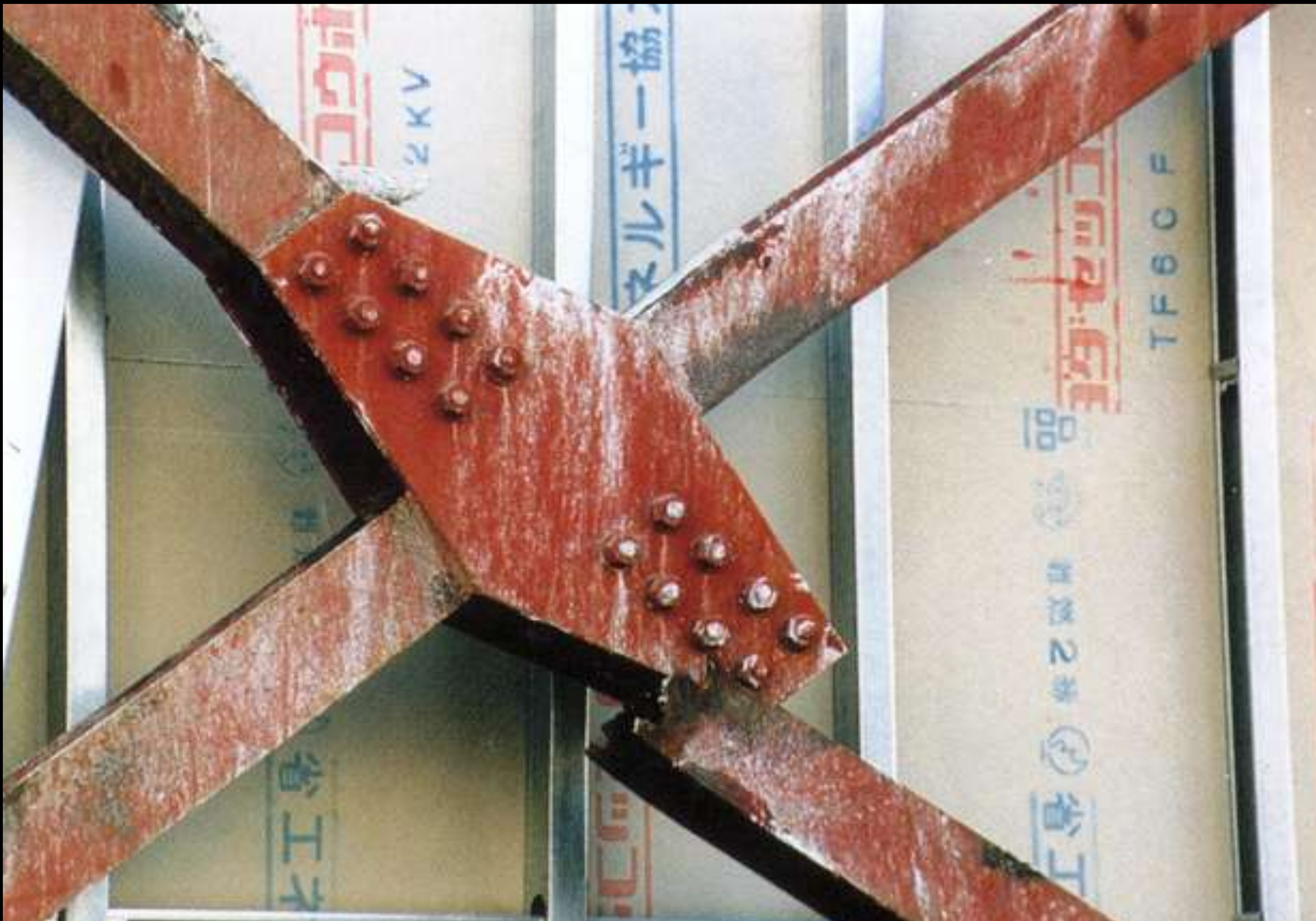
13.2 Members

13.2b Required Strength

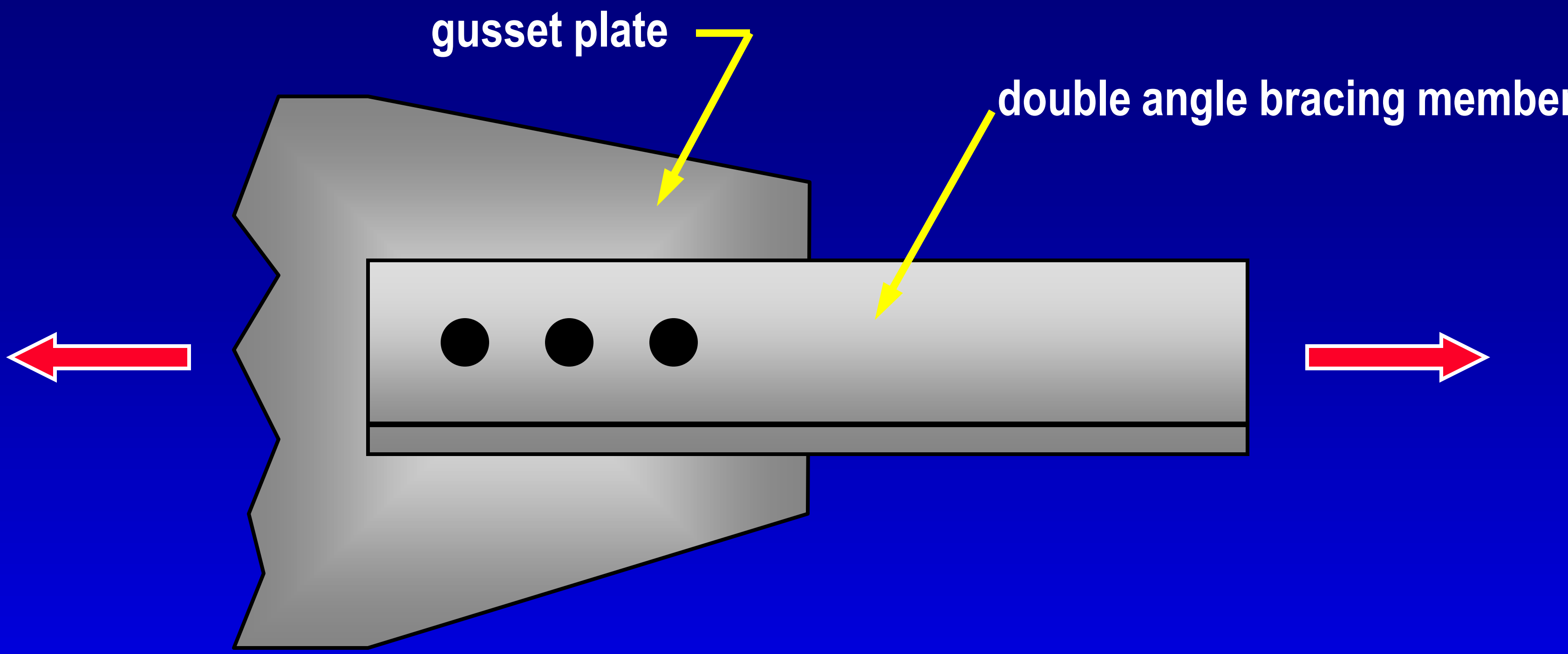
Where the effective net area of bracing members is less than the gross area, the *required tensile strength* of the brace, based on a limit state of fracture of the net section shall be at least $R_y F_y A_g$ of the bracing member.

Objective: yield of gross section of brace prior to fracture of net section

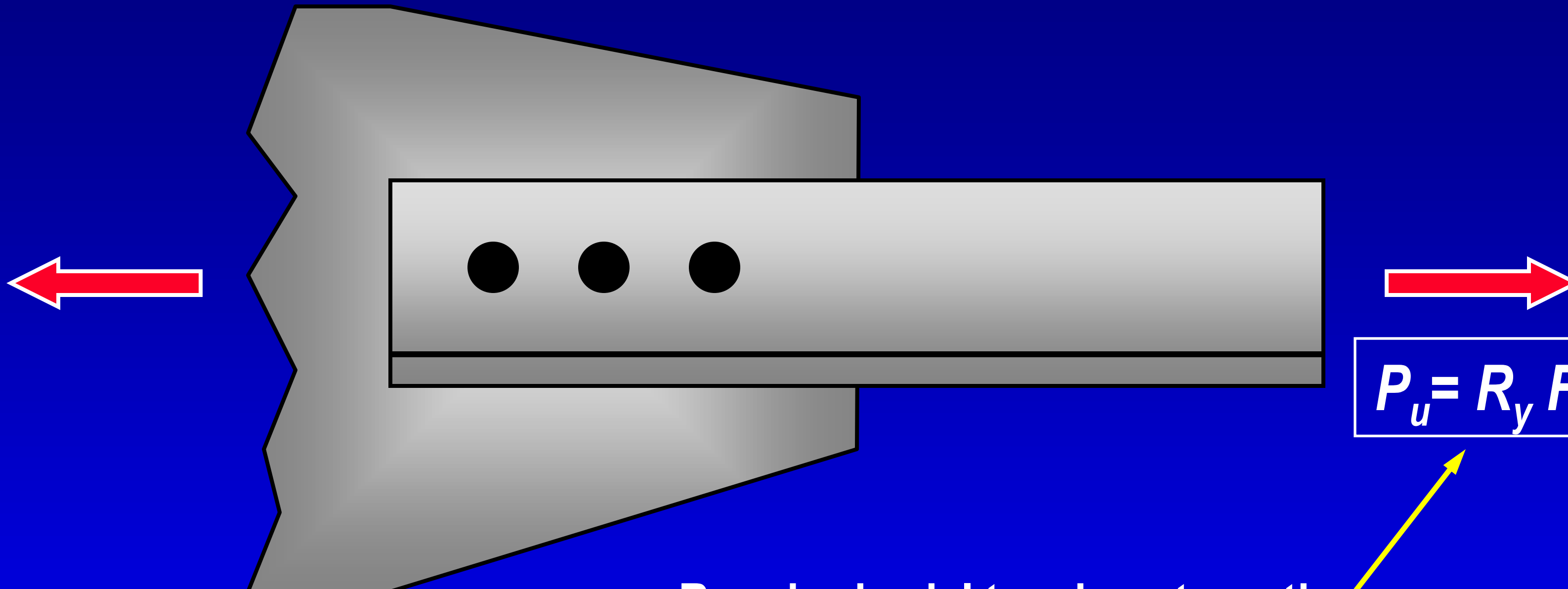

95 2 17


ネルギー協
TF6CF

Example

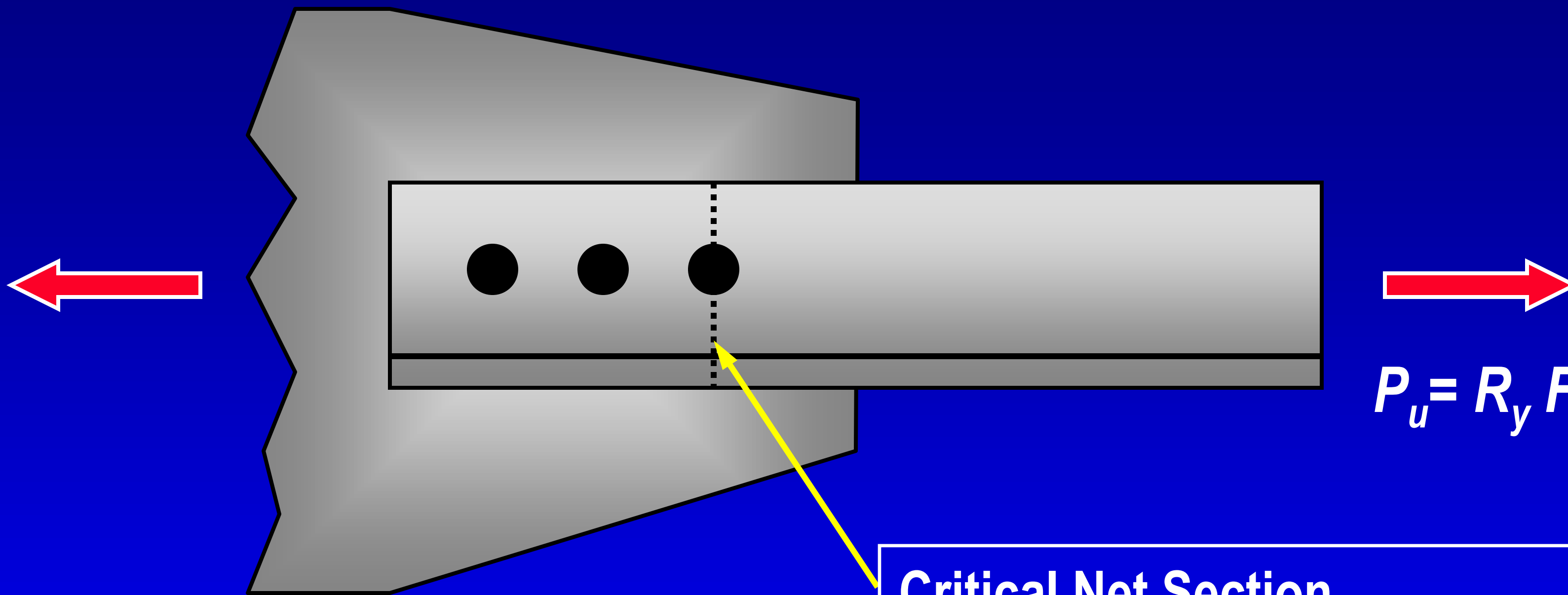


Check double angle bracing member for limit state of net section fracture



$$P_u = R_y F_y A_g$$

Required axial tension strength of brace for limit state of fracture of the net section



$P_u = R_y \, F_y \, A_g$

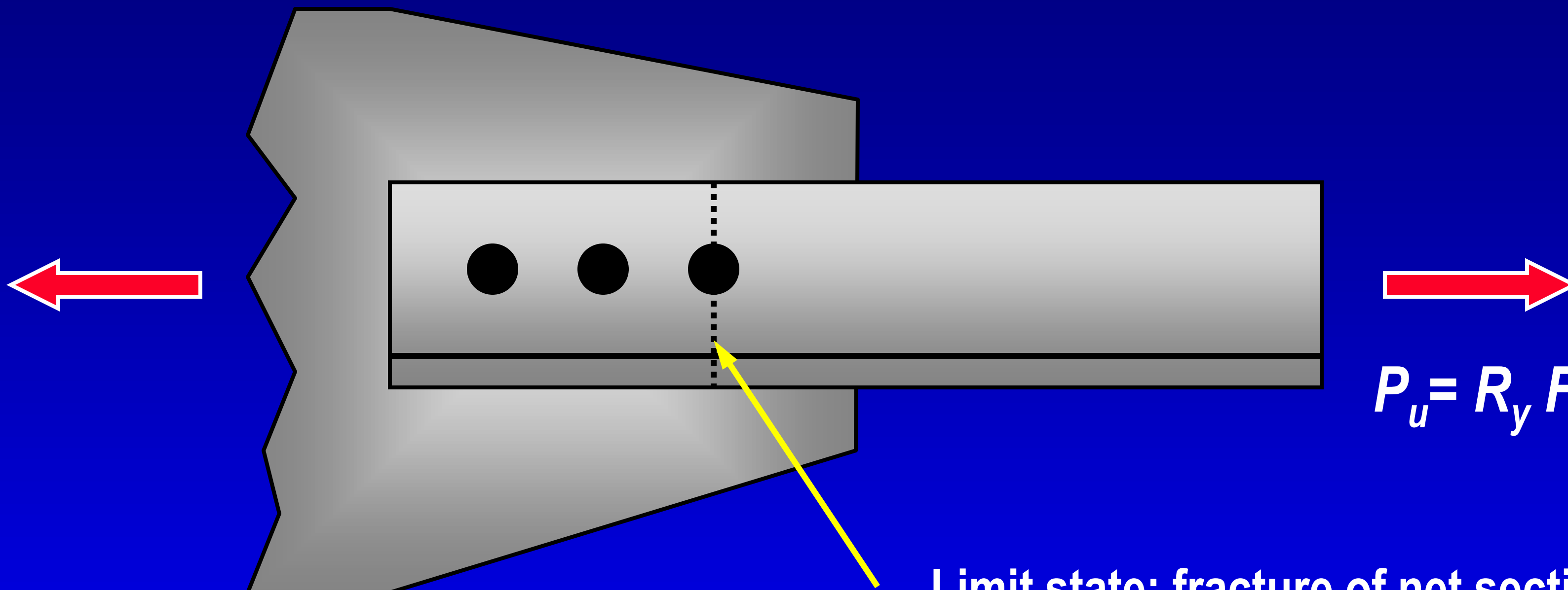
Critical Net Section

$A_e = U \, A_n$

$A_e < A_g$ due to:

bolt hole ($A_n < A_g$), and

shear lag ($U < 1$)

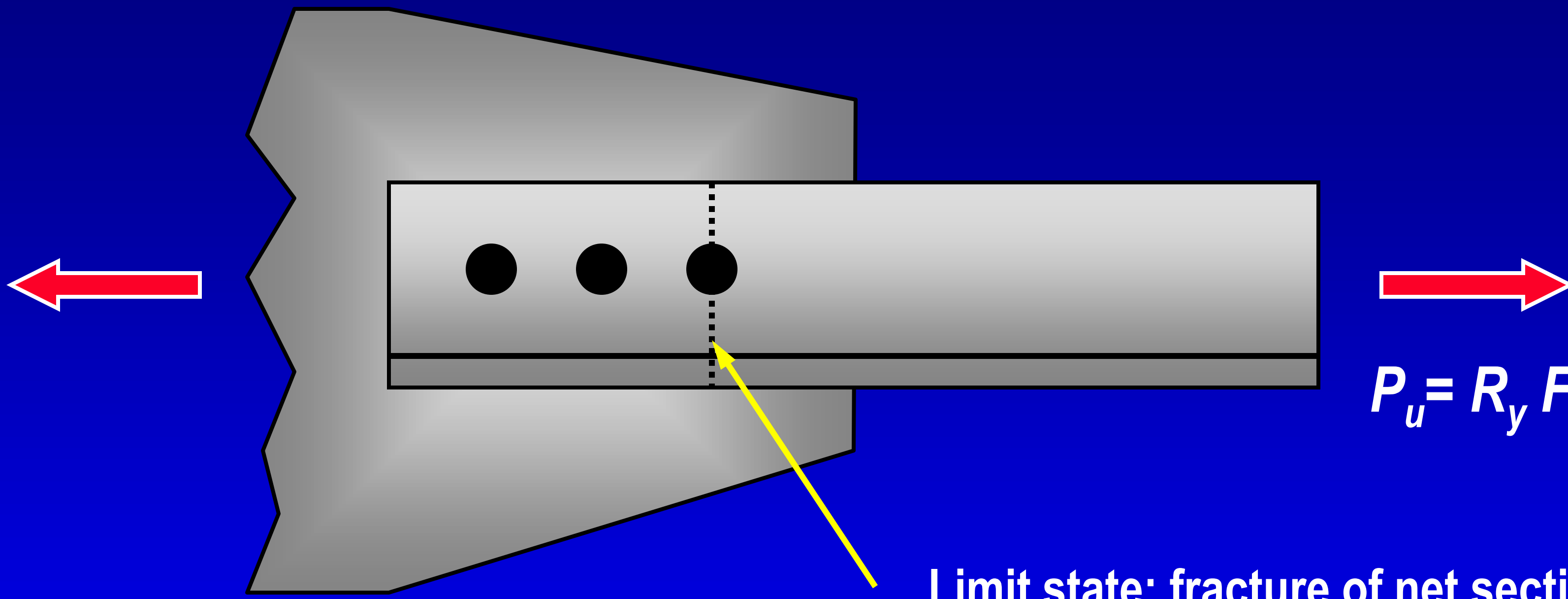


$$P_u = R_y F_y A_g$$

<u>Limit state: fracture of net section</u>

$$\phi P_n = \underbrace{(0.75)}_{\phi} A_e \, (\underbrace{R_t F_u})$$

Per Section 6.2: use *expected tensile strength* $R_t F_U$ when checking net section fracture of bracing member, since $R_y F_y$ of the same member is used to computed the required strength



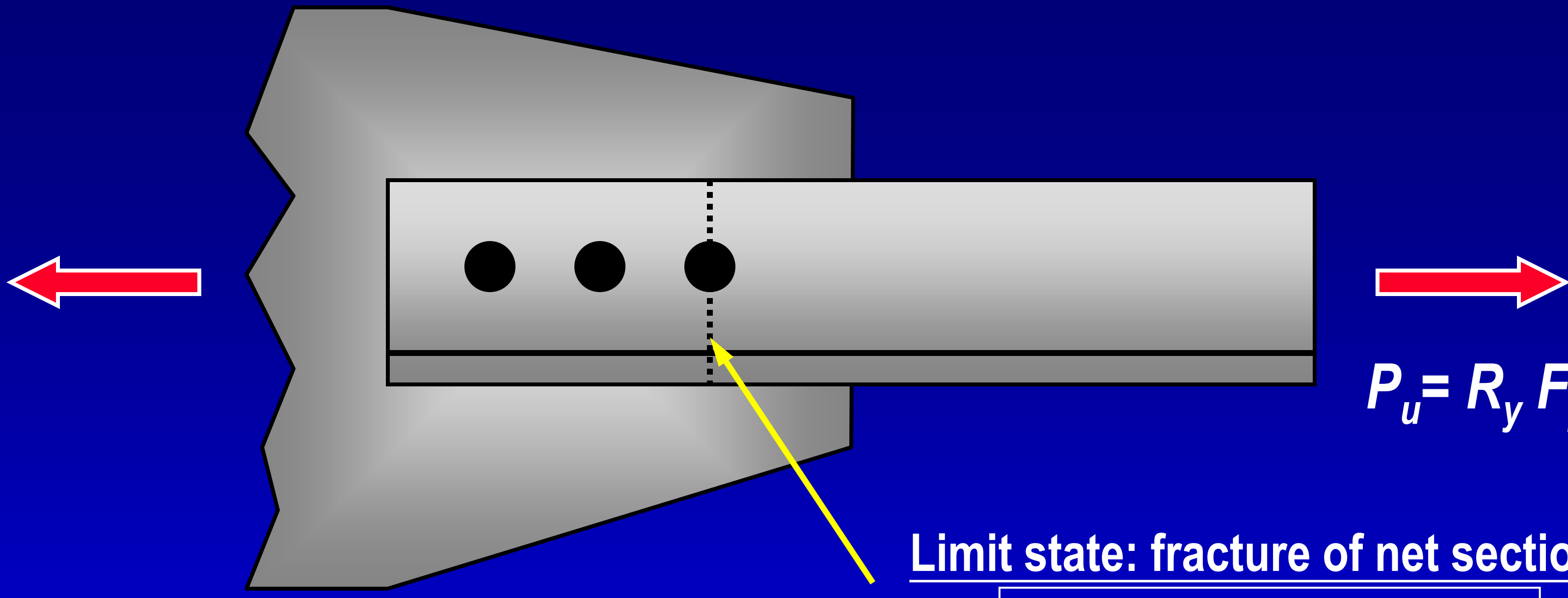
$$P_u = R_y\,F_y\,A_g$$

Limit state: fracture of net section

$$(0.75)\,A_e\,(R_t\,F_u) \geq R_y\,F_y\,A_g$$

OR:

$$\boxed{\frac{A_e}{A_g} \geq \frac{R_y F_y}{(0.75)\,R_t F_u}}$$



$P_u = R_y F_y A_g$

Limit state: fracture of net section

$$\frac{A_e}{A_g} \geq \frac{R_y F_y}{(0.75) R_t F_u}$$

For A36 Angles:

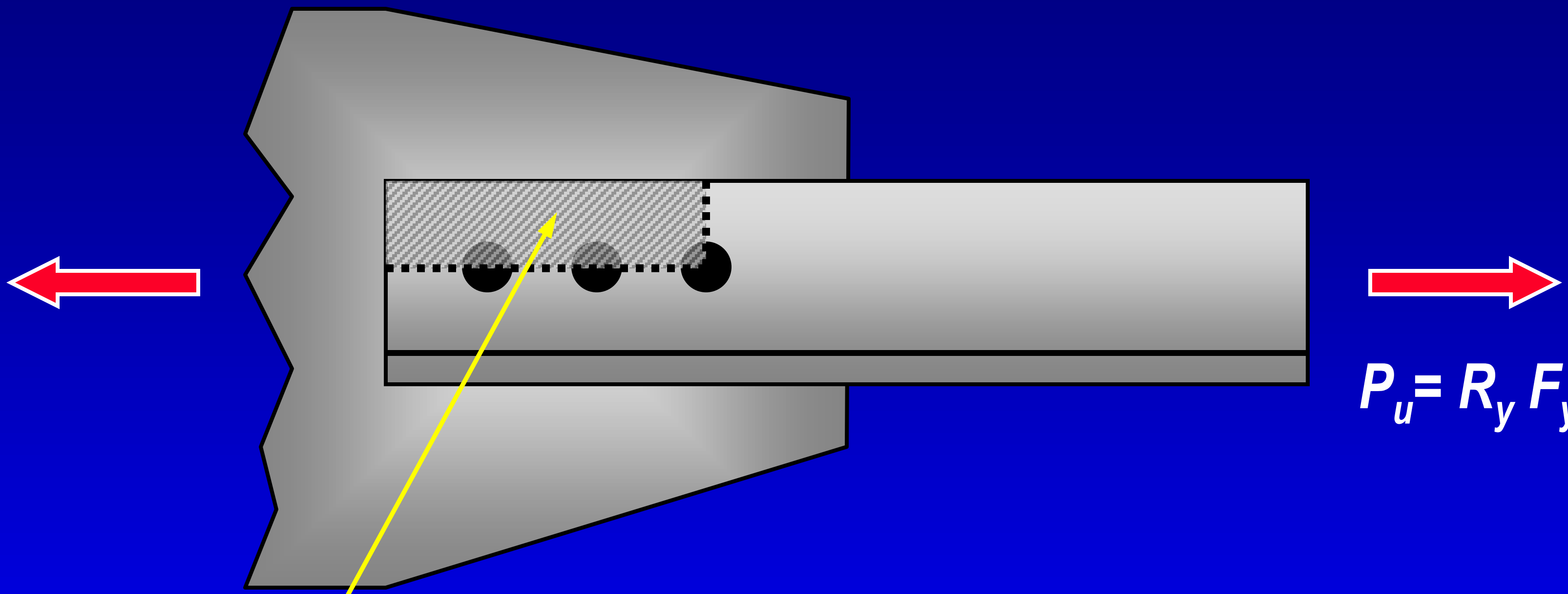
$$\frac{A_e}{A_g} \geq \frac{1.5 \times 36ksi}{(0.75)1.2 \times 58ksi} = 1.03$$

For A572 Gr. 50 Angles:

$$\frac{A_e}{A_g} \geq \frac{1.1 \times 50ksi}{(0.75)1.1 \times 65ksi} = 1.03$$

Need to Reinforce Net Section *(A_e need not exceed A_g)*

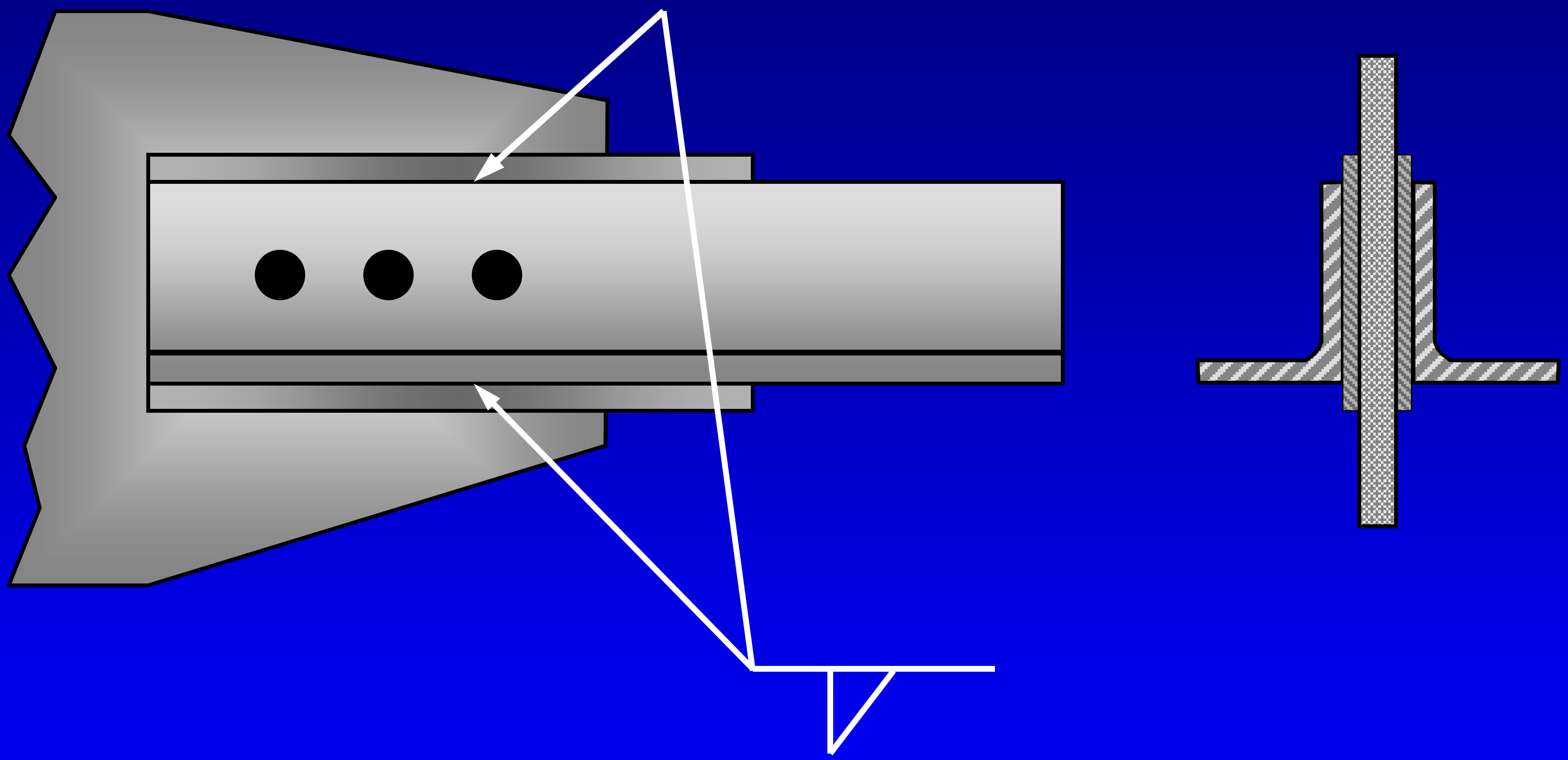
Also check block shear rupture of bracing member....



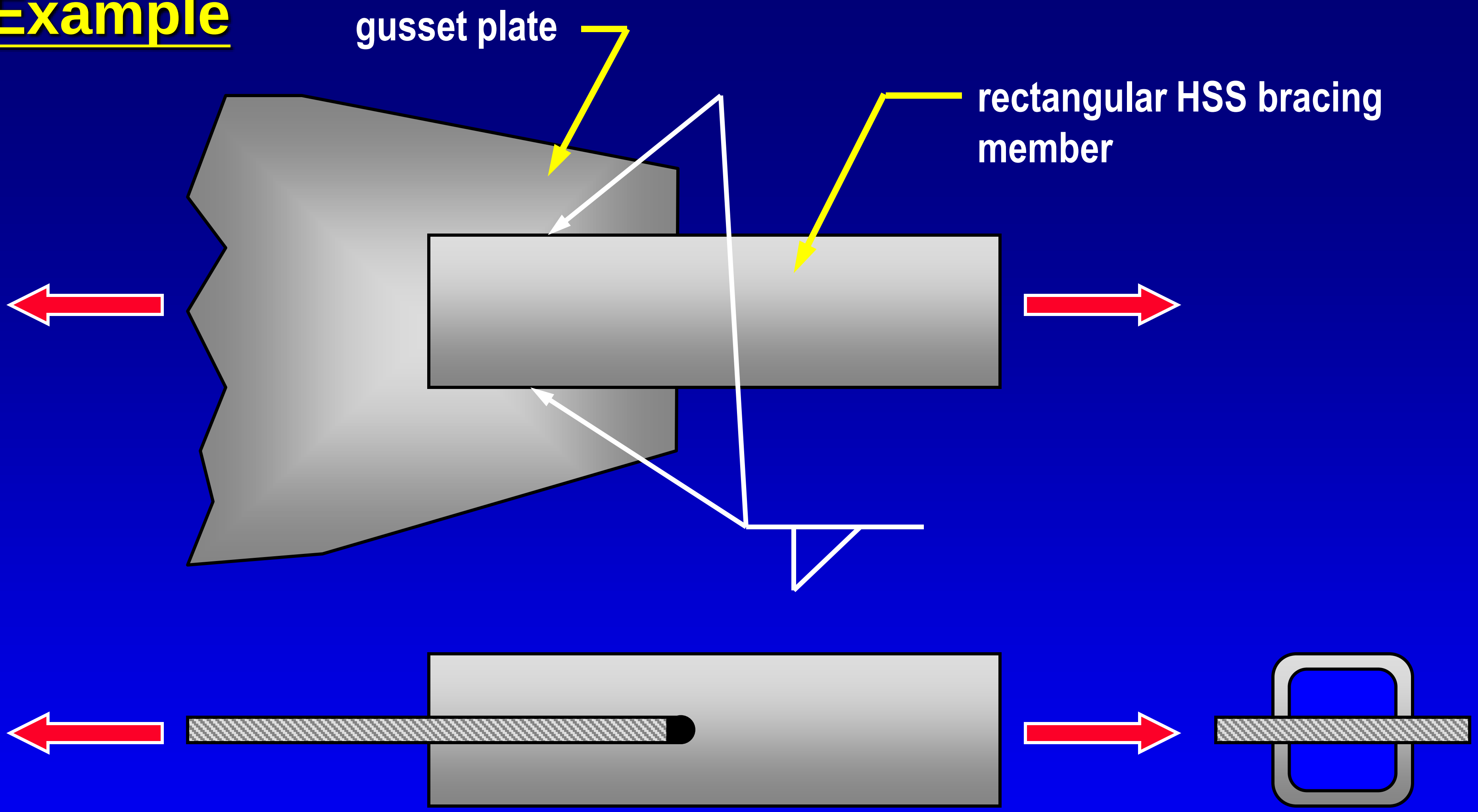
$$P_u = R_y F_y A_g$$

$$\phi P_n = (0.75)\left[U_{bs} A_{nt} R_t F_u + \textit{lesser of} \begin{Bmatrix} 0.6 A_{nv} R_t F_u \\ 0.6 A_{gv} R_y F_y \end{Bmatrix} \right]$$

Reinforcing net section of bracing member....

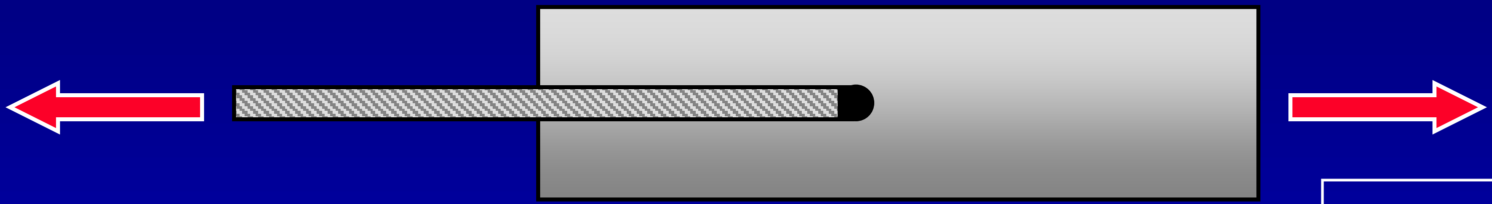


Example



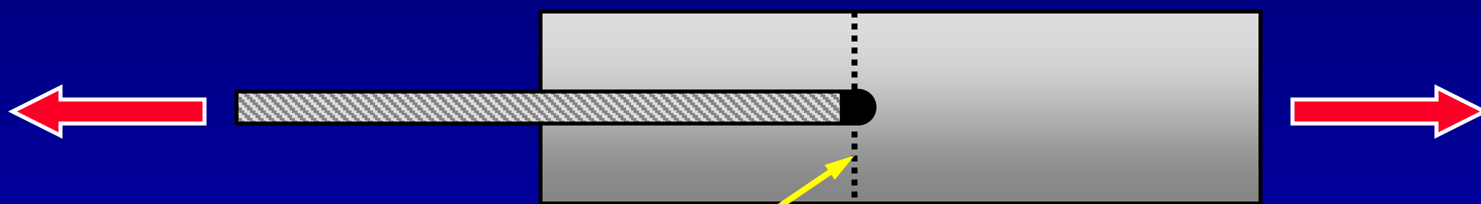
Check HSS bracing member for limit state of net section fracture


13 A



$$P_u = R_y F_y A_g$$

Required axial tension strength of brace for limit state of fracture of the net section



$P_u = R_y F_y A_g$

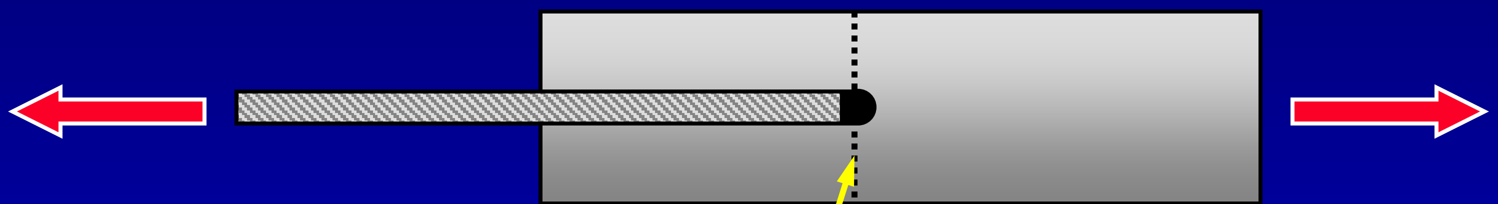
Critical Net Section

$A_e = U A_n$

$A_e < A_g$ due to:

slot ($A_n < A_g$), and

shear lag ($U < 1$)



$P_u = R_y\,F_y\,A_g$

Limit state: fracture of net section

$(0.75)\,A_e\,(R_t\,F_u) \geq R_y\,F_y\,A_g$

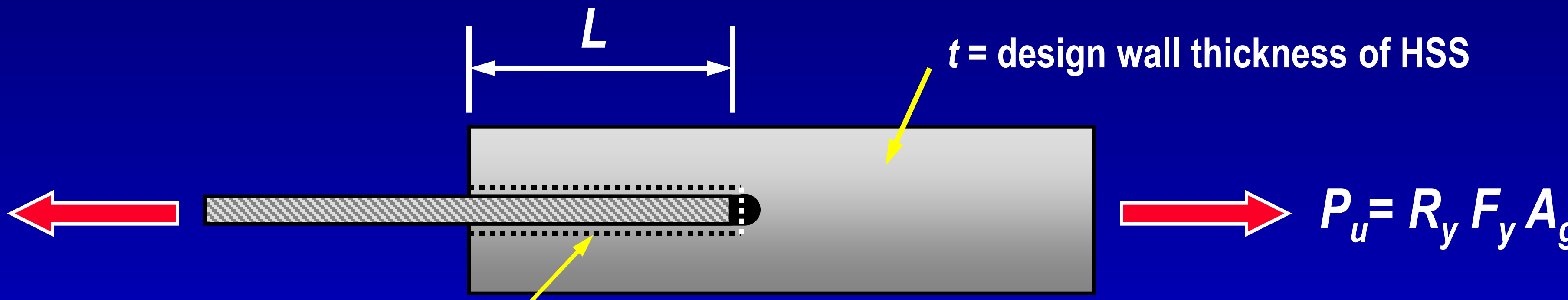
OR: $\boxed{\dfrac{A_e}{A_g} \geq \dfrac{R_y F_y}{(0.75)\,R_t F_u}}$

For A500 Gr B rectangular HSS:

$$\frac{A_e}{A_g} \geq \frac{1.4 \times 46ksi}{(0.75)\,1.3 \times 58ksi} = 1.14$$

Need to Reinforce Net Section *(A_e need not exceed A_g)*

Also check block shear rupture of bracing member....



$$\phi P_n = (0.75)\left[U_{bs} A_{nt} R_t F_u + \text{lesser of} \begin{Bmatrix} 0.6\, A_{nv} R_t F_u \\ 0.6\, A_{gv} R_y F_y \end{Bmatrix} \right]$$

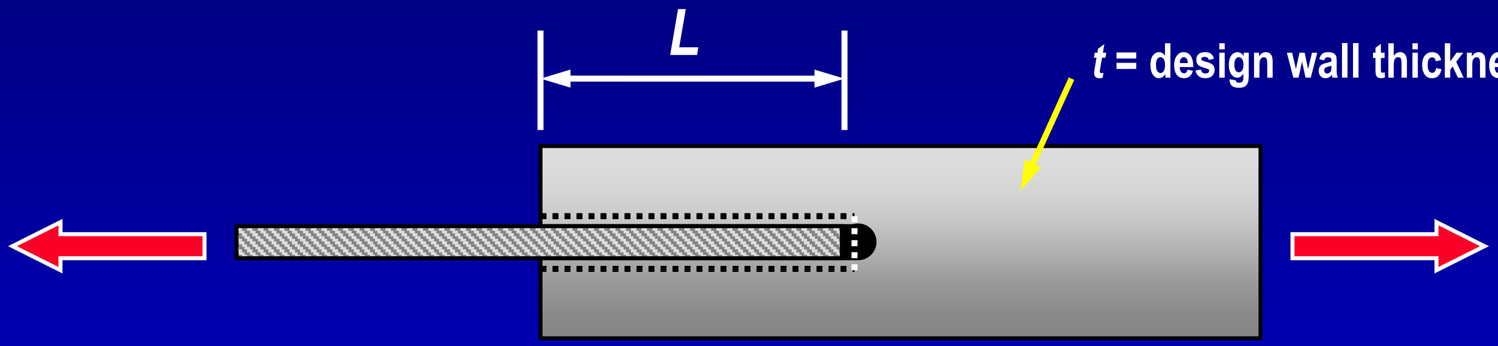
$$A_{nt} \cong 0$$

$$A_{nt} = A_{gv} = 4\, L\, t$$

For A500 Gr B rectangular HSS: $R_t F_u = 1.3 \times 58 \text{ ksi} = 75.4 \text{ ksi}$

$R_y F_y = 1.4 \times 46 \text{ ksi} = 64.2 \text{ ksi}$

Also check block shear rupture of bracing member....



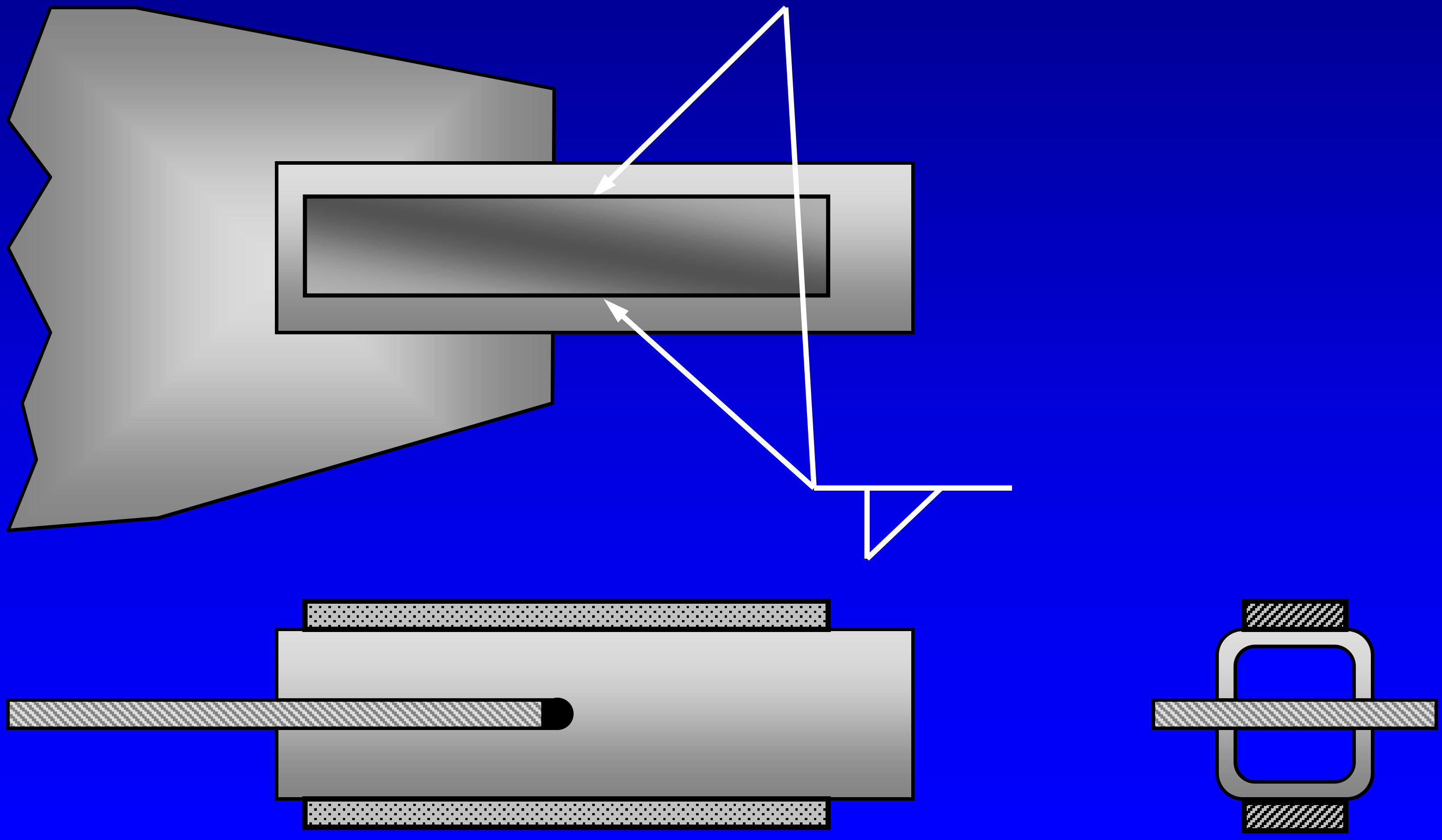
$P_u = R_y F_y A_g$

$$\phi P_n = (0.75)\,(4\,L\,t \times 0.6 \times 64.2\text{ ksi}) \geq 1.4 \times 46\text{ ksi} \times A_g$$

$$L \geq \frac{0.557 \times A_g}{t}$$

= minimum length of welded overlap needed based on block shear rupture in HSS bracing member

Reinforcing net section of bracing member....



13.2 Members

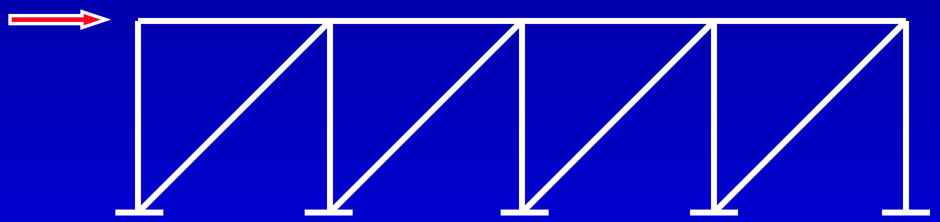
13.2c Lateral Force Distribution

Along any line of bracing, braces shall be **deployed in opposite directions** such that, for either direction of force parallel to the bracing, at least 30 percent but not more than 70% of the total horizontal force along that line is resisted by braces in tension..

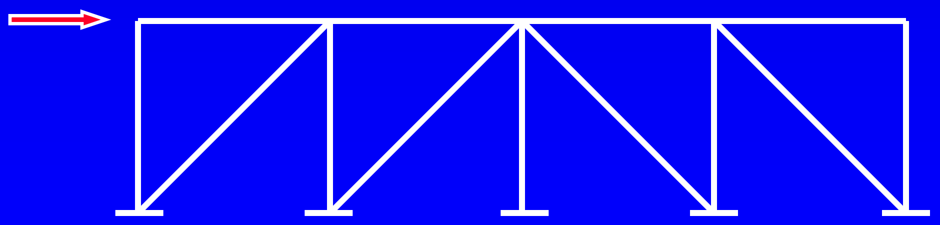
13.2 Members

13.2c Lateral Force Distribution

Deploy braces so that about half are in tension (and the other half in compression)



All braces in tension (or compression) ***NG***



OK

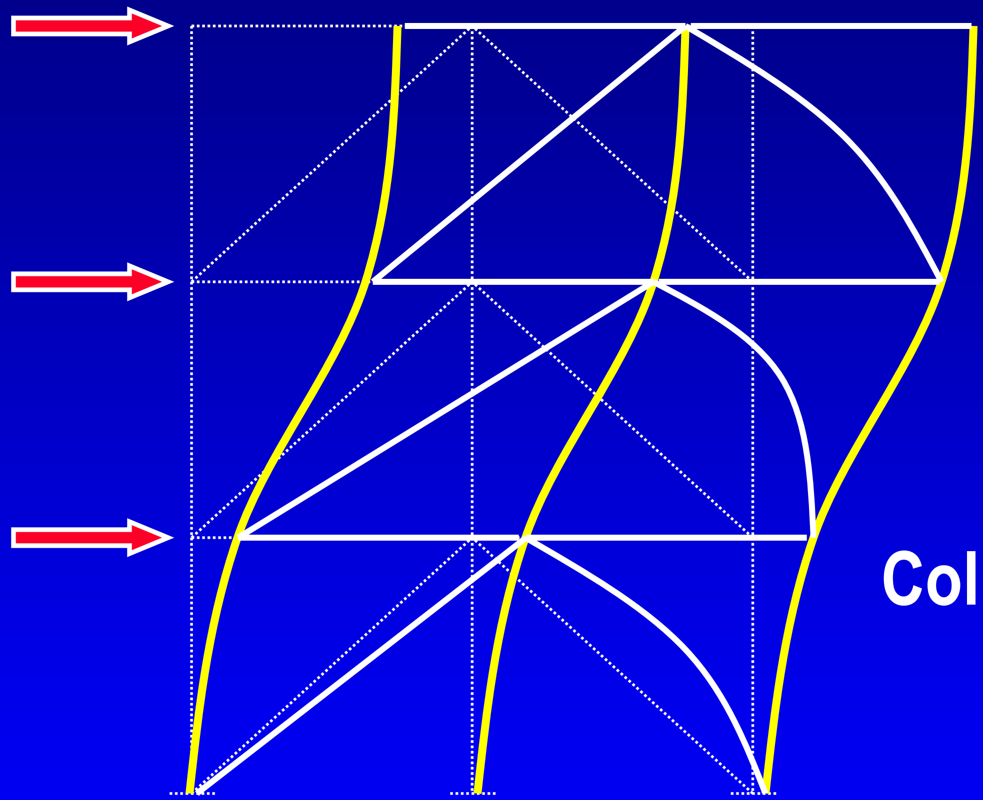
13.2 Members

13.2d Width-Thickness Limitations

Columns and braces shall meet requirements of Section 8.2b.

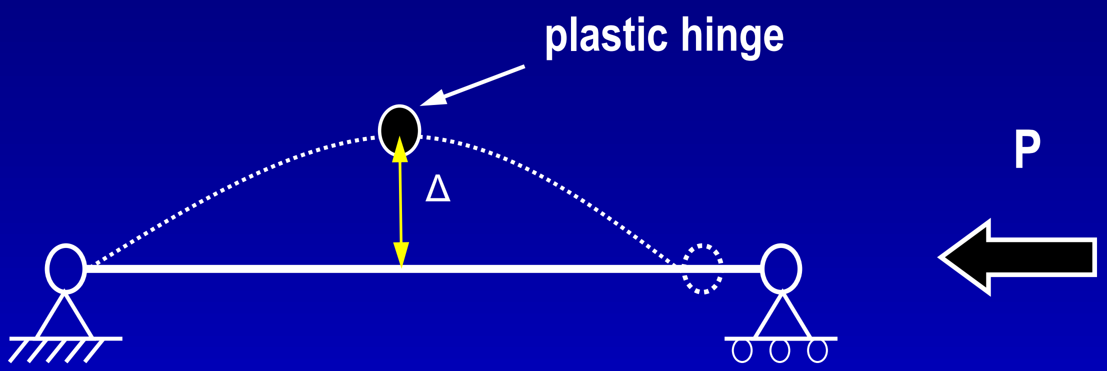
i.e. columns and braces must be *seismically compact :* $\lambda \leq \lambda_{ps}$

13.2d Width-Thickness Limitations



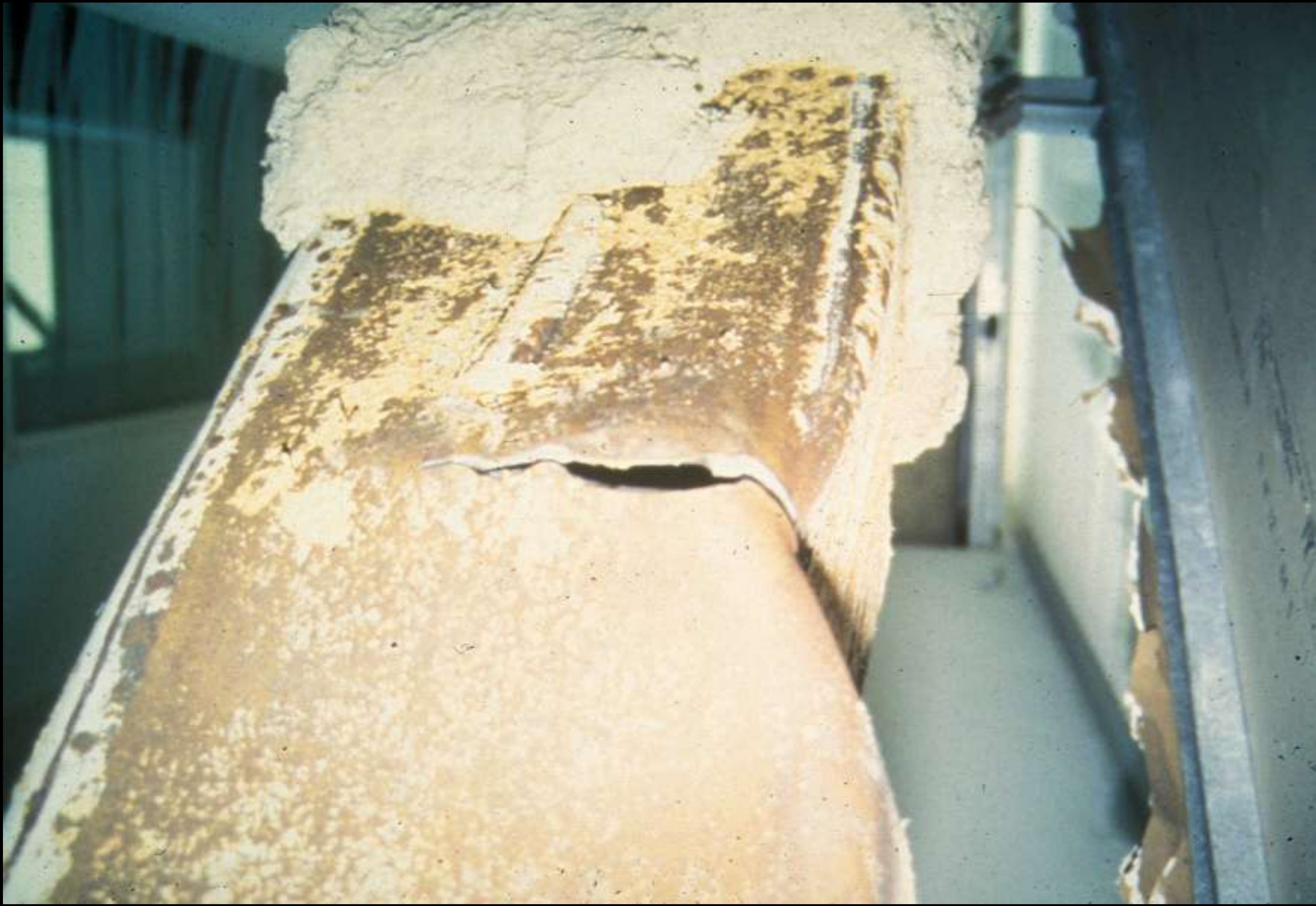
Columns: $\lambda \leq \lambda_{ps}$

13.2d Width-Thickness Limitations



Braces: form *plastic hinge* during buckling

With high b/t's - local buckling and possibly fracture may occur at plastic hinge region



13.2d Width-Thickness Limitations

Bracing Members: $\lambda \le \lambda_{ps}$

For rectangular HSS (A500 Gr B steel):

$$\frac{b}{t} \le 0.64\sqrt{\frac{E}{F_y}} = 0.64\sqrt{\frac{29000\ ksi}{46\ ksi}} = 16.1$$

Table 1–12
Square HSS
Dimensions and Properties

HSS16–HSS8

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	b/t	h/t	I	S	r	Z	Workable Flat	Torsion J	Torsion C	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS16×16×$\frac{5}{8}$	0.581	127.00	35.0	24.5	24.5	1370	171	6.25	200	$13\frac{3}{16}$	2170	276	5.17
×$\frac{1}{2}$	0.465	103.00	28.3	31.4	31.4	1130	141	6.31	164	$13\frac{3}{4}$	1770	224	5.20
×$\frac{3}{8}$	0.349	78.45	21.5	42.8	42.8	873	109	6.37	126	$14\frac{5}{16}$	1350	171	5.23
×$\frac{5}{16}$	0.291	65.82	18.1	52.0	52.0	739	92.3	6.39	106	$14\frac{5}{8}$	1140	144	5.25
HSS14×14×$\frac{5}{8}$	0.581	110.00	30.3	21.1	21.1	897	128	5.44	151	$11\frac{3}{16}$	1430	208	4.50
×$\frac{1}{2}$	0.465	89.55	24.6	27.1	27.1	743	106	5.49	124	$11\frac{3}{4}$	1170	170	4.53
×$\frac{3}{8}$	0.349	68.24	18.7	37.1	37.1	577	82.5	5.55	95.4	$12\frac{5}{16}$	900	130	4.57
×$\frac{5}{16}$	0.291	57.31	15.7	45.1	45.1	490	69.9	5.58	80.5	$12\frac{5}{8}$	759	109	4.58
HSS12×12×$\frac{5}{8}$	0.581	93.14	25.7	17.7	17.7	548	91.4	4.62	109	$9\frac{3}{16}$	885	151	3.83
×$\frac{1}{2}$	0.465	75.94	20.9	22.8	22.8	457	76.2	4.68	89.6	$9\frac{3}{4}$	728	123	3.87
×$\frac{3}{8}$	0.349	58.03	16.0	31.4	31.4	357	59.5	4.73	69.2	$10\frac{5}{16}$	561	94.6	3.90
×$\frac{5}{16}$	0.291	48.81	13.4	38.2	38.2	304	50.7	4.76	58.6	$10\frac{5}{8}$	474	79.7	3.92
×$\frac{1}{4}$	0.233	39.40	10.8	48.5	48.5	248	41.4	4.79	47.6	$10\frac{7}{8}$	384	64.5	3.93
×$\frac{3}{16}$	0.174	29.82	8.15	66.0	66.0	189	31.5	4.82	36.0	$11\frac{3}{16}$	290	48.6	3.95
HSS10×10×$\frac{5}{8}$	0.581	76.13	21.0	14.2	14.2	304	60.8	3.80	73.2	$7\frac{3}{16}$	498	102	3.17
×$\frac{1}{2}$	0.465	62.33	17.2	18.5	18.5	256	51.2	3.86	60.7	$7\frac{3}{4}$	412	84.2	3.20
×$\frac{3}{8}$	0.349	47.82	13.2	25.7	25.7	202	40.4	3.92	47.2	$8\frac{5}{16}$	320	64.8	3.23
×$\frac{5}{16}$	0.291	40.30	11.1	31.4	31.4	172	34.5	3.94	40.1	$8\frac{5}{8}$	271	54.8	3.25
×$\frac{1}{4}$	0.233	32.60	8.96	39.9	39.9	141	28.3	3.97	32.7	$8\frac{7}{8}$	220	44.4	3.27
×$\frac{3}{16}$	0.174	24.72	6.76	54.5	54.5	108	21.6	4.00	24.8	$9\frac{3}{16}$	167	33.6	3.28
HSS9×9×$\frac{5}{8}$	0.581	67.62	18.7	12.5	12.5	216	47.9	3.40	58.1	$6\frac{3}{16}$	356	81.6	2.83
×$\frac{1}{2}$	0.465	55.53	15.3	16.4	16.4	183	40.6	3.45	48.4	$6\frac{3}{4}$	296	67.4	2.87
×$\frac{3}{8}$	0.349	42.72	11.8	22.8	22.8	145	32.2	3.51	37.8	$7\frac{5}{16}$	231	52.1	2.90
×$\frac{5}{16}$	0.291	36.05	9.92	27.9	27.9	124	27.6	3.54	32.1	$7\frac{5}{8}$	196	44.0	2.92
×$\frac{1}{4}$	0.233	29.19	8.03	35.6	35.6	102	22.7	3.56	26.2	$7\frac{7}{8}$	159	35.8	2.93
×$\frac{3}{16}$	0.174	22.16	6.06	48.7	48.7	78.2	17.4	3.59	20.0	$8\frac{3}{16}$	121	27.1	2.95
×$\frac{1}{8}$	0.116	14.95	4.09	74.6	74.6	53.5	11.9	3.62	13.6	$8\frac{7}{16}$	82.0	18.3	2.97
HSS8×8×$\frac{5}{8}$	0.581	59.11	16.4	10.8	10.8	146	36.5	2.99	44.7	$5\frac{3}{16}$	244	63.2	2.50
×$\frac{1}{2}$	0.465	48.72	13.5	14.2	14.2	125	31.2	3.04	37.5	$5\frac{3}{4}$	204	52.4	2.53
×$\frac{3}{8}$	0.349	37.61	10.4	19.9	19.9	100	24.9	3.10	29.4	$6\frac{5}{16}$	160	40.7	2.57
×$\frac{5}{16}$	0.291	31.79	8.76	24.5	24.5	85.6	21.4	3.13	25.1	$6\frac{5}{8}$	136	34.5	2.58
×$\frac{1}{4}$	0.233	25.79	7.10	31.3	31.3	70.7	17.7	3.15	20.5	$6\frac{7}{8}$	111	28.1	2.60
×$\frac{3}{16}$	0.174	19.61	5.37	43.0	43.0	54.4	13.6	3.18	15.7	$7\frac{3}{16}$	84.5	21.3	2.62
×$\frac{1}{8}$	0.116	13.25	3.62	66.0	66.0	37.4	9.34	3.21	10.7	$7\frac{7}{16}$	57.3	14.4	2.63

Table 1–12 (continued)
Square HSS
Dimensions and Properties

HSS7–HSS4 1/2

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	b/t	h/t	I	S	r	Z	Workable Flat	Torsion J	Torsion C	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS7×7×5/8	0.581	50.60	14.0	9.05	9.05	93.4	26.7	2.58	33.1	4 3/16	158	47.1	2.17
×1/2	0.465	41.91	11.6	12.1	12.1	80.5	23.0	2.63	27.9	4 3/4	133	39.3	2.20
×3/8	0.349	32.51	8.97	17.1	17.1	65.0	18.6	2.69	22.1	5 5/16	105	30.7	2.23
×5/16	0.291	27.54	7.59	21.1	21.1	56.1	16.0	2.72	18.9	5 5/8	89.7	26.1	2.25
×1/4	0.233	22.39	6.17	27.0	27.0	46.5	13.3	2.75	15.5	5 7/8	73.5	21.3	2.27
×3/16	0.174	17.08	4.67	37.2	37.2	36.0	10.3	2.77	11.9	6 3/16	56.1	16.2	2.28
×1/8	0.116	11.55	3.16	57.3	57.3	24.8	7.09	2.80	8.13	6 7/16	38.2	11.0	2.30
HSS6×6×5/8	0.581	42.10	11.7	7.33	7.33	55.2	18.4	2.17	23.2	3 3/16	94.9	33.4	1.83
×1/2	0.465	35.11	9.74	9.90	9.90	48.3	16.1	2.23	19.8	3 3/4	81.1	28.1	1.87
×3/8	0.349	27.41	7.58	14.2	14.2	39.5	13.2	2.28	15.8	4 5/16	64.6	22.1	1.90
×5/16	0.291	23.29	6.43	17.6	17.6	34.3	11.4	2.31	13.6	4 5/8	55.4	18.9	1.92
×1/4	0.233	18.99	5.24	22.8	22.8	28.6	9.54	2.34	11.2	4 7/8	45.6	15.4	1.93
×3/16	0.174	14.51	3.98	31.5	31.5	22.3	7.42	2.37	8.63	5 3/16	35.0	11.8	1.95
×1/8	0.116	9.85	2.70	48.7	48.7	15.5	5.15	2.39	5.92	5 7/16	23.9	8.03	1.97
HSS5 1/2×5 1/2×3/8	0.349	24.85	6.88	12.8	12.8	29.7	10.8	2.08	13.1	3 13/16	49.0	18.4	1.73
×5/16	0.291	21.16	5.85	15.9	15.9	25.9	9.43	2.11	11.3	4 1/8	42.2	15.7	1.75
×1/4	0.233	17.28	4.77	20.6	20.6	21.7	7.90	2.13	9.32	4 3/8	34.8	12.9	1.77
×3/16	0.174	13.23	3.63	28.6	28.6	17.0	6.17	2.16	7.19	4 11/16	26.7	9.85	1.78
×1/8	0.116	9.00	2.46	44.4	44.4	11.8	4.30	2.19	4.95	4 15/16	18.3	6.72	1.80
HSS5×5×1/2	0.465	28.30	7.88	7.75	7.75	26.0	10.4	1.82	13.1	2 3/4	44.6	18.7	1.53
×3/8	0.349	22.30	6.18	11.3	11.3	21.7	8.68	1.87	10.6	3 5/16	36.1	14.9	1.57
×5/16	0.291	19.03	5.26	14.2	14.2	19.0	7.62	1.90	9.16	3 5/8	31.2	12.8	1.58
×1/4	0.233	15.58	4.30	18.5	18.5	16.0	6.41	1.93	7.61	3 7/8	25.8	10.5	1.60
×3/16	0.174	11.96	3.28	25.7	25.7	12.6	5.03	1.96	5.89	4 3/16	19.9	8.08	1.62
×1/8	0.116	8.15	2.23	40.1	40.1	8.80	3.52	1.99	4.07	4 7/16	13.7	5.53	1.63
HSS4 1/2×4 1/2×1/2	0.465	24.90	6.95	6.68	6.68	18.1	8.03	1.61	10.2	2 1/4	31.3	14.8	1.37
×3/8	0.349	19.75	5.48	9.89	9.89	15.3	6.79	1.67	8.36	2 13/16	25.7	11.9	1.40
×5/16	0.291	16.91	4.68	12.5	12.5	13.5	6.00	1.70	7.27	3 1/8	22.3	10.2	1.42
×1/4	0.233	13.88	3.84	16.3	16.3	11.4	5.08	1.73	6.06	3 3/8	18.5	8.44	1.43
×3/16	0.174	10.68	2.93	22.9	22.9	9.02	4.01	1.75	4.71	3 11/16	14.4	6.49	1.45
×1/8	0.116	7.30	2.00	35.8	35.8	6.35	2.82	1.78	3.27	3 15/16	9.92	4.45	1.47

AISC Seismic Provisions - SCBF

13.3 Required Strength of Bracing Connections

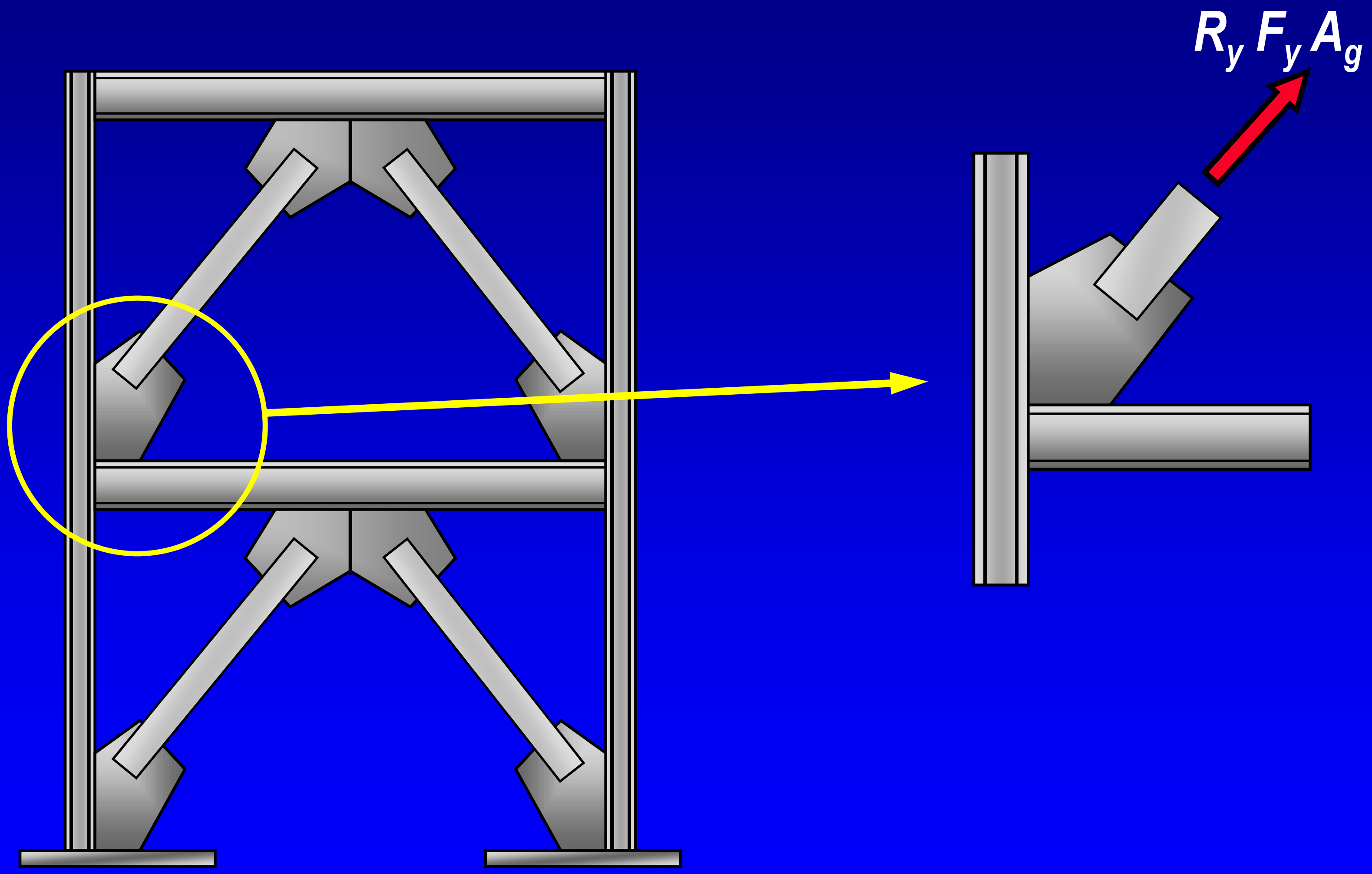
13.3a Required Tensile Strength

The *required tensile strength* of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:

1. $R_y F_y A_g$ of the bracing member.
2. The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

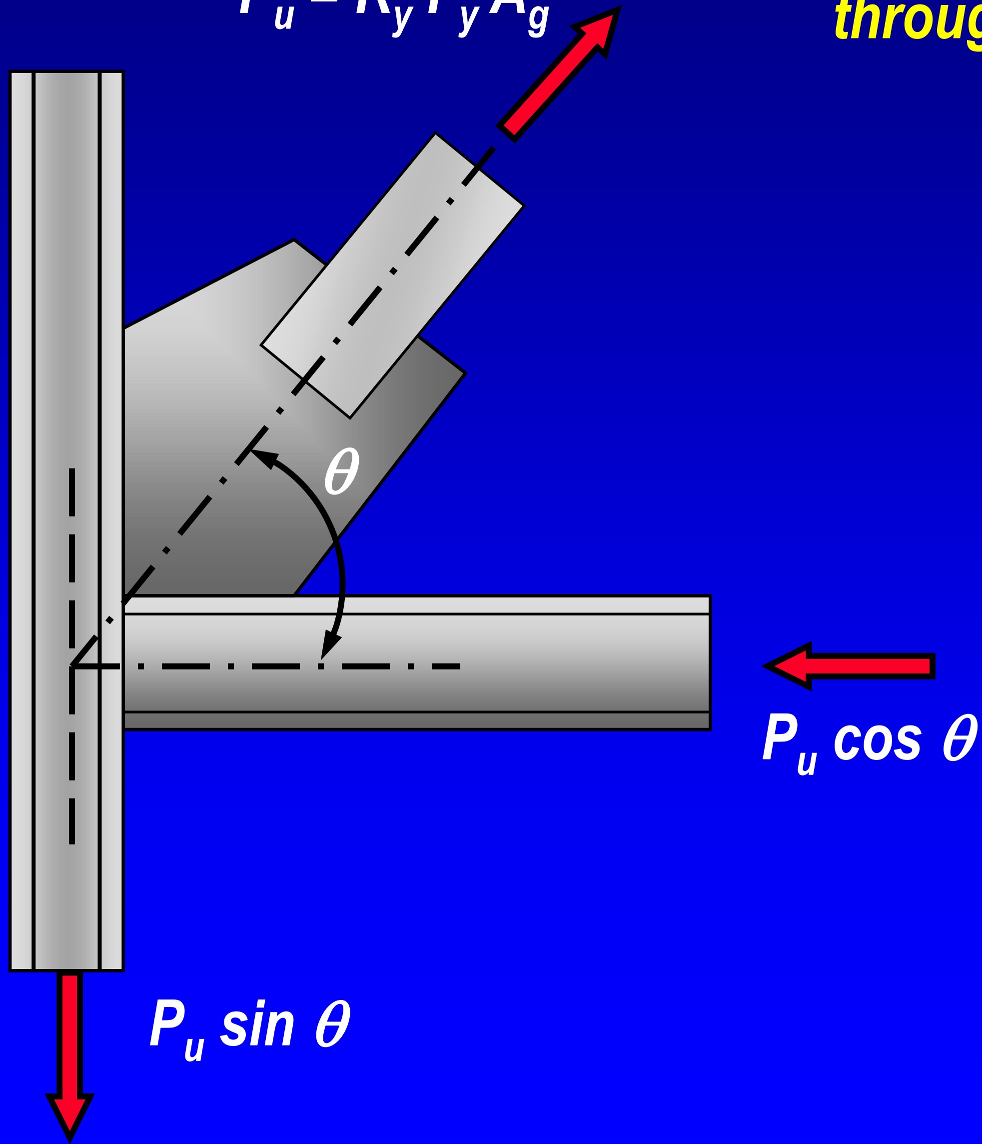
Few practical applications of Item 2.

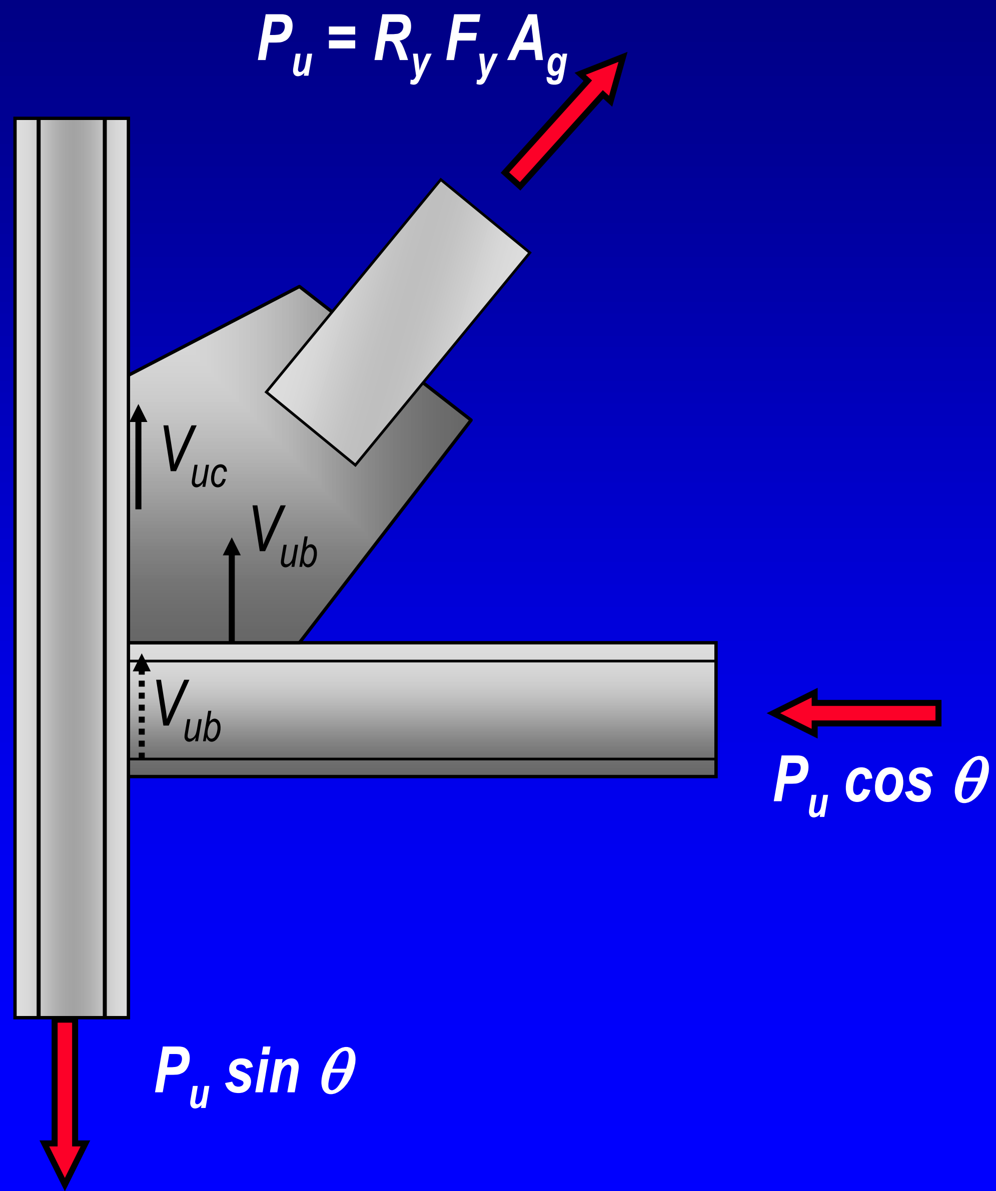
Note that $\Omega_o Q_E$ is NOT an acceptable method to establish "maximum load effect"



$R_y F_y A_g$

Consider load path
through connection region





Consider load path through connection region:

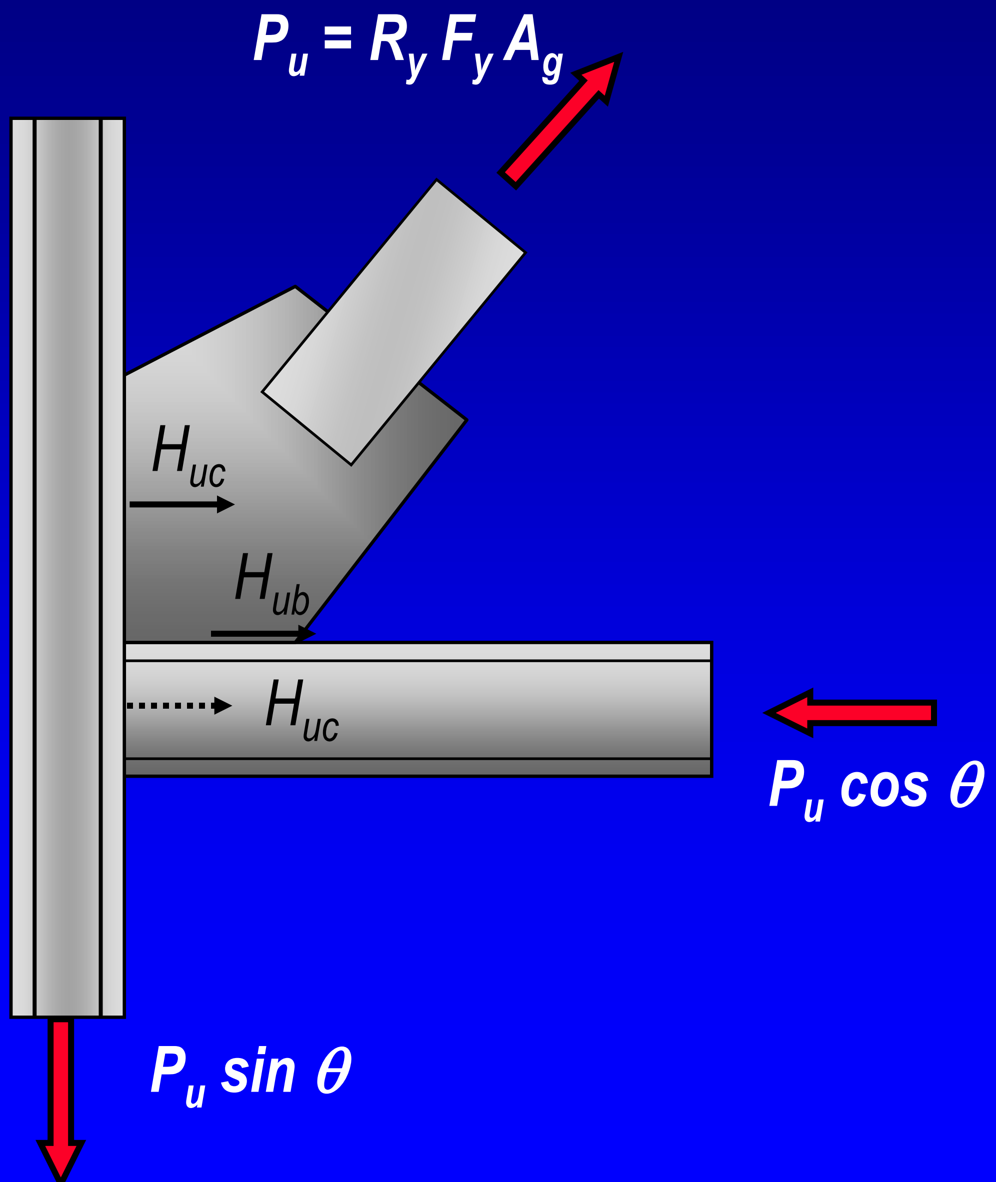
Uniform Force Method -

Vertical Component of P_u transferred to column.

$$V_{uc} + V_{ub} = P_u \sin\theta$$

V_{uc} is transferred directly to column

V_{ub} is transferred indirectly to column through beam and beam to column connection



Consider load path through connection region:

Uniform Force Method - *Horizontal Component of P_u transferred to beam.*

$$H_{uc} + H_{ub} = P_u \cos\theta$$

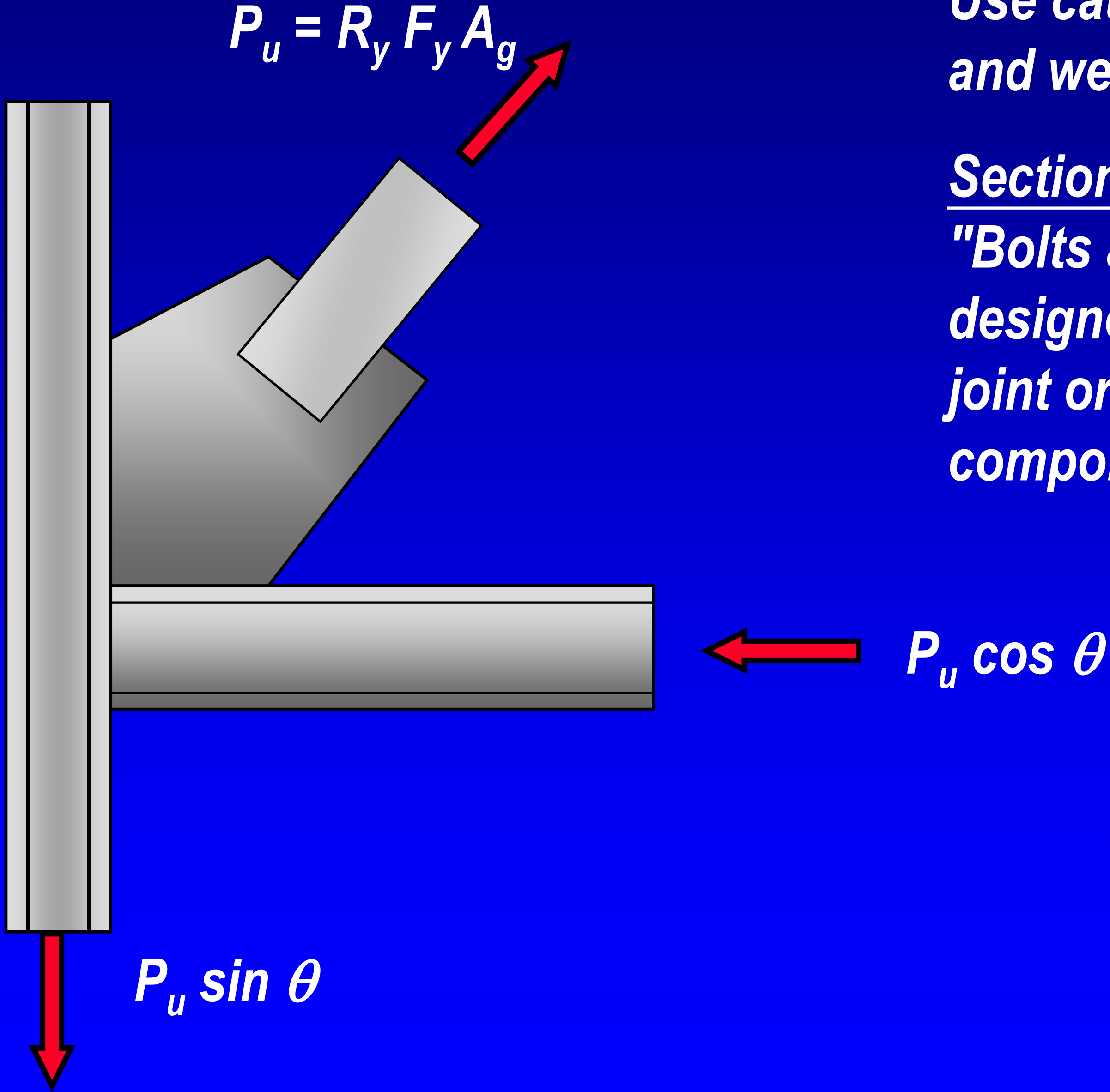
H_{ub} is transferred directly to beam

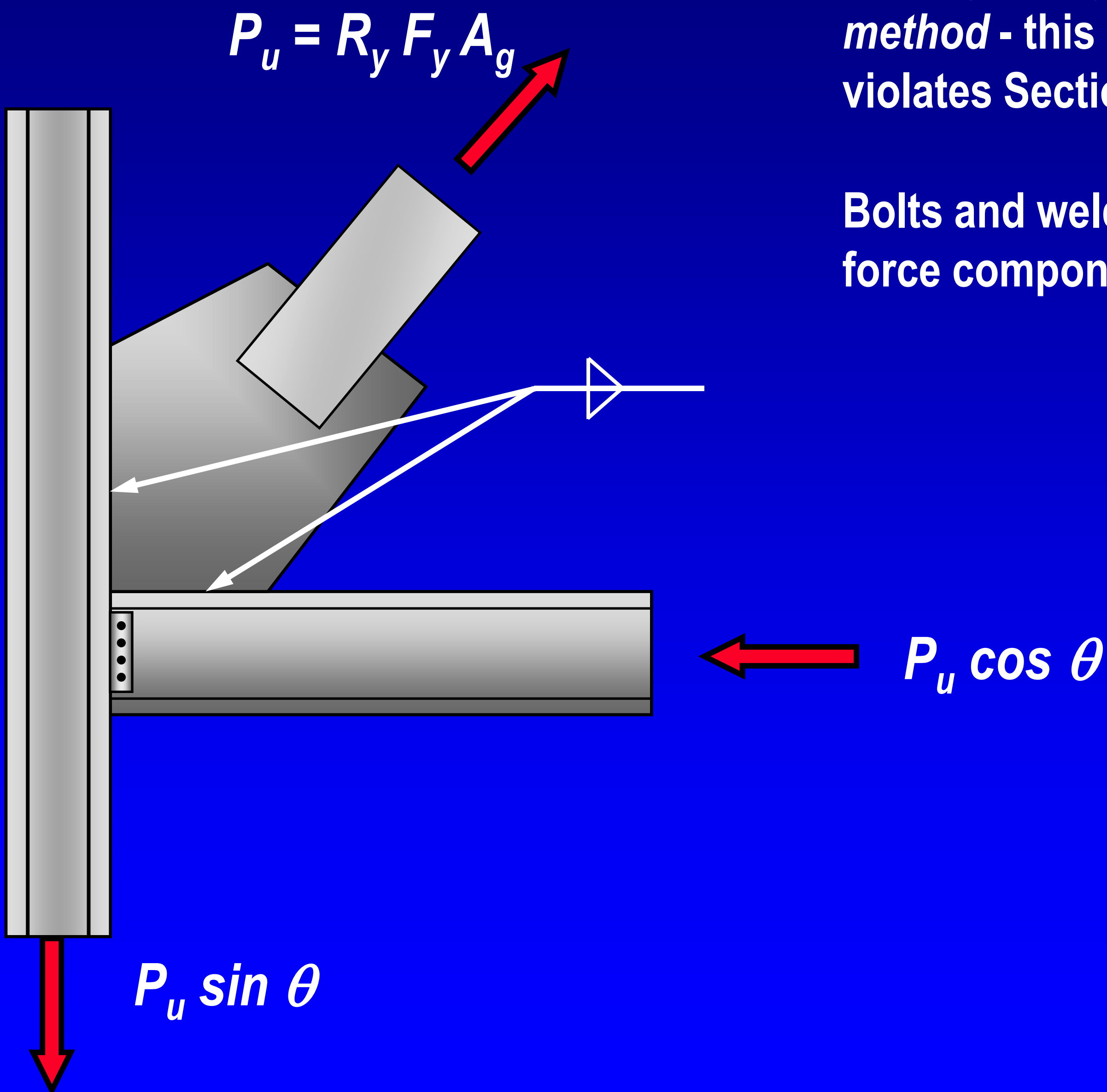
H_{uc} is transferred indirectly to beam through column and beam to column connection

Consider load path through connection region:

Use caution in use of bolts and welds.

<u>Section 7.2:</u>
"Bolts and welds shall not be designed to share force in a joint or the same force component in a connection."





If designed by *uniform force method* - this connection violates Section 7.2

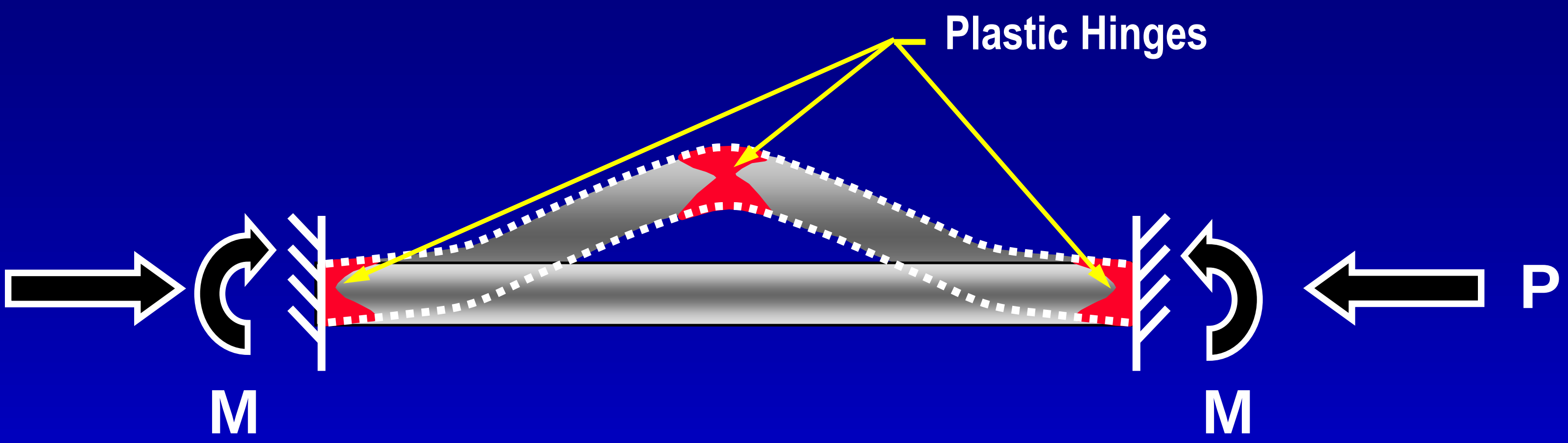
Bolts and welds must transfer same force components.

AISC Seismic Provisions - SCBF

13.3 Required Strength of Bracing Connections

13.3b Required Flexural Strength

The *required flexural strength* of bracing connections is $1.1\ R_y\ M_p$ of bracing member.



For "fixed" end braces: flexural plastic hinges will form at mid-length and at brace ends. Brace will impose bending moment on connections and adjoining members.

$M_u = 1.1\ R_y\ M_p = 1.1\ R_y\ F_y\ Z_{brace}$
(for critical buckling direction)

$1.1\ R_y\ M_{p\text{-brace}}$


33.14 T

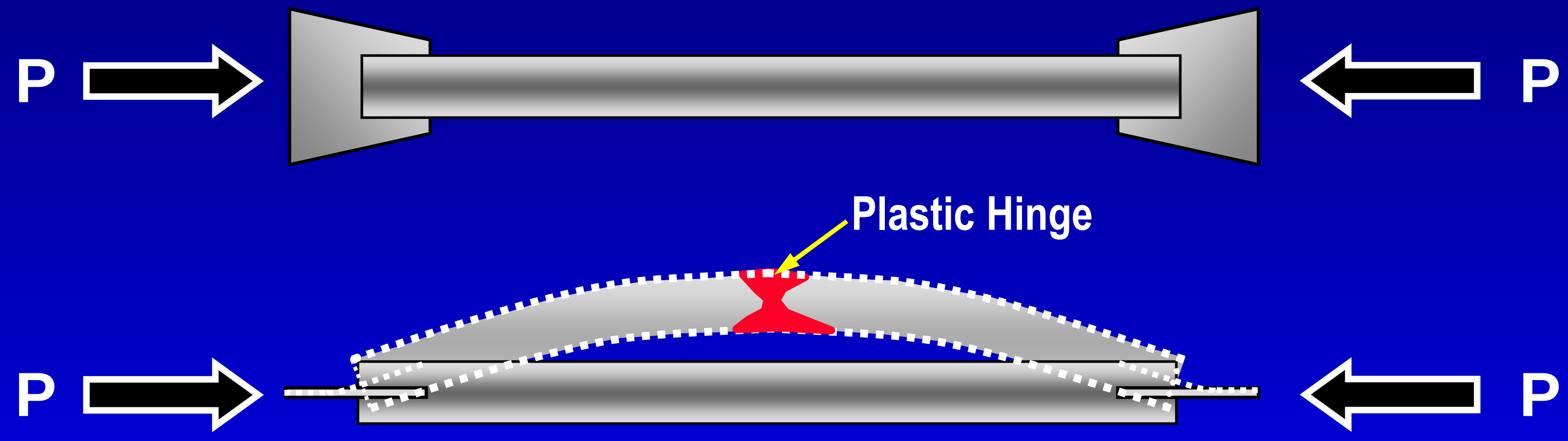
13.3 Required Strength of Bracing Connections

13.3b Required Flexural Strength

The ***required flexural strength*** of bracing connections is **1.1 $R_y M_p$** of bracing member.

Exception:

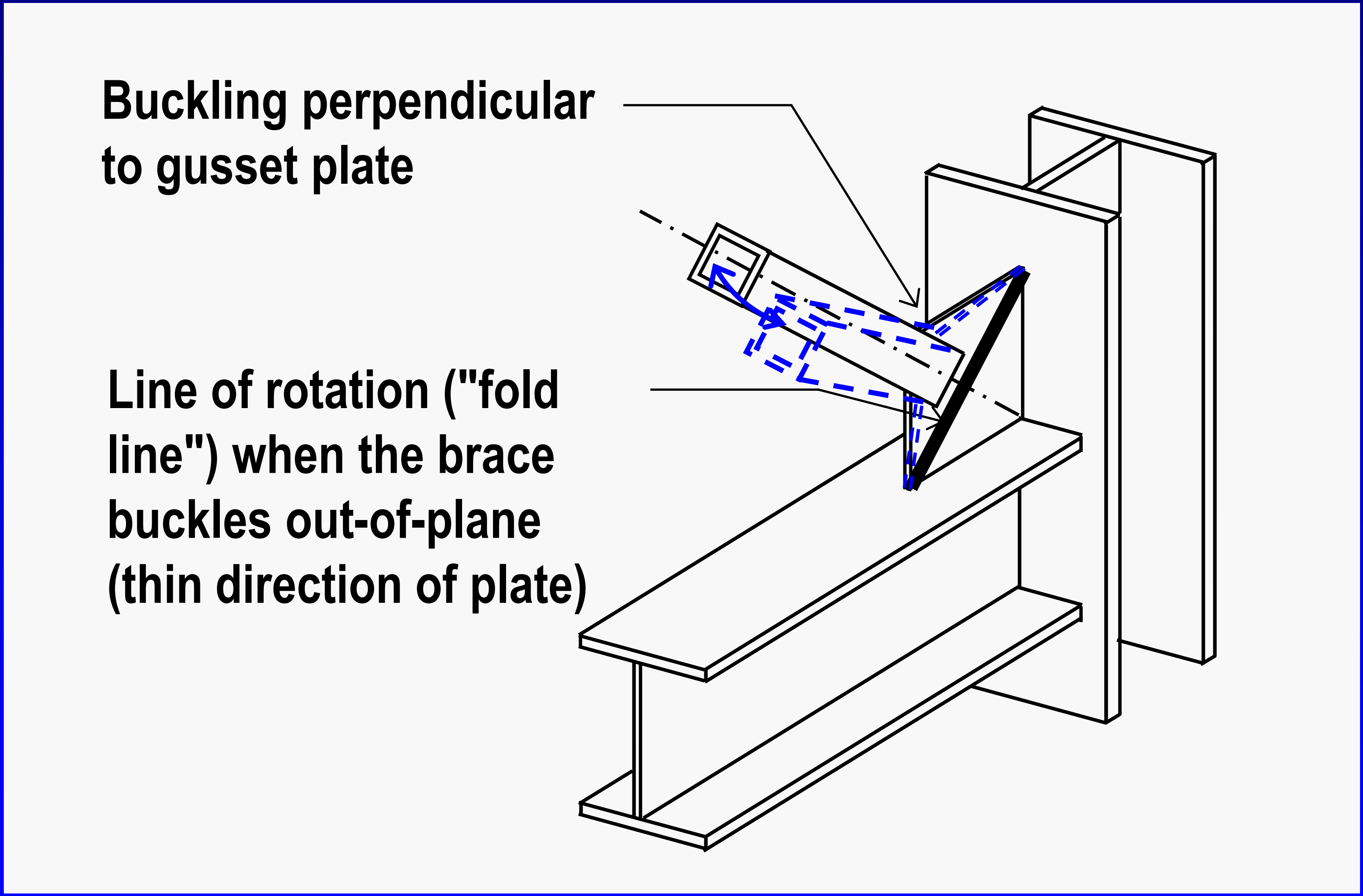
Brace connections that can accommodate the inelastic rotations associated with brace post-buckling deformations need not meet this requirement.

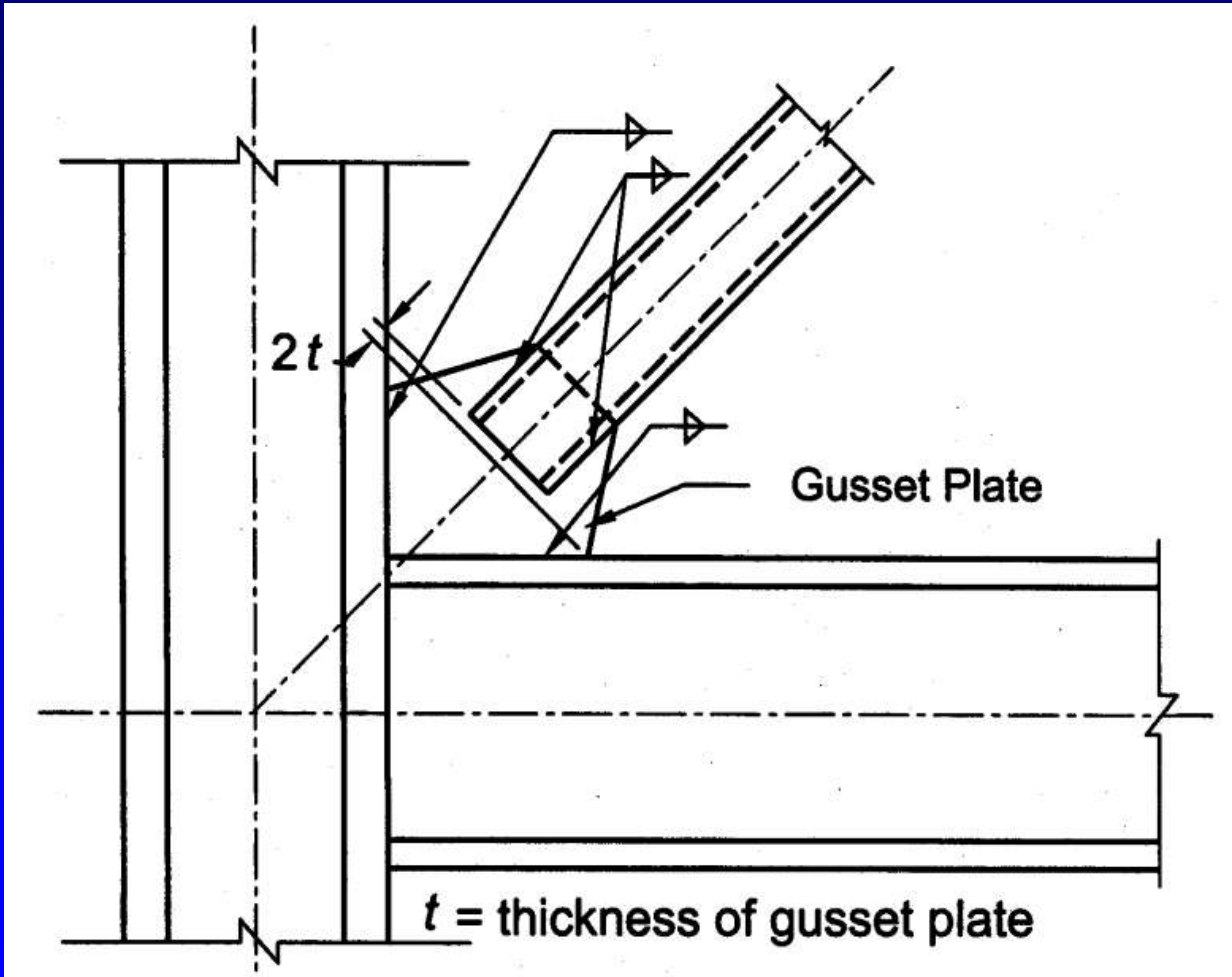


For "pinned" end braces: flexural plastic hinge will form at mid-length only. Brace will impose no bending moment on connections and adjoining members.

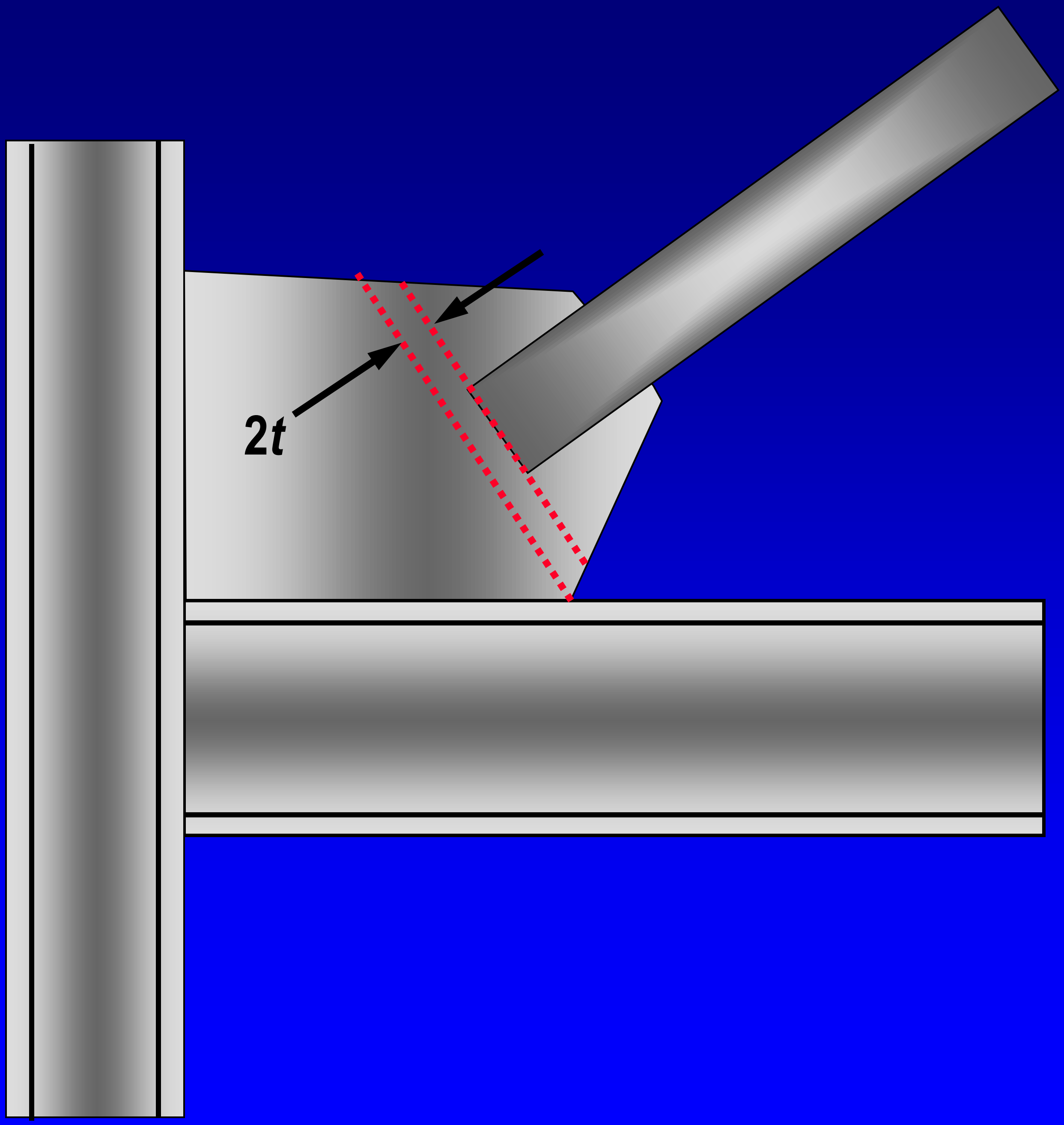
Must design brace connection to behave like a "pin"

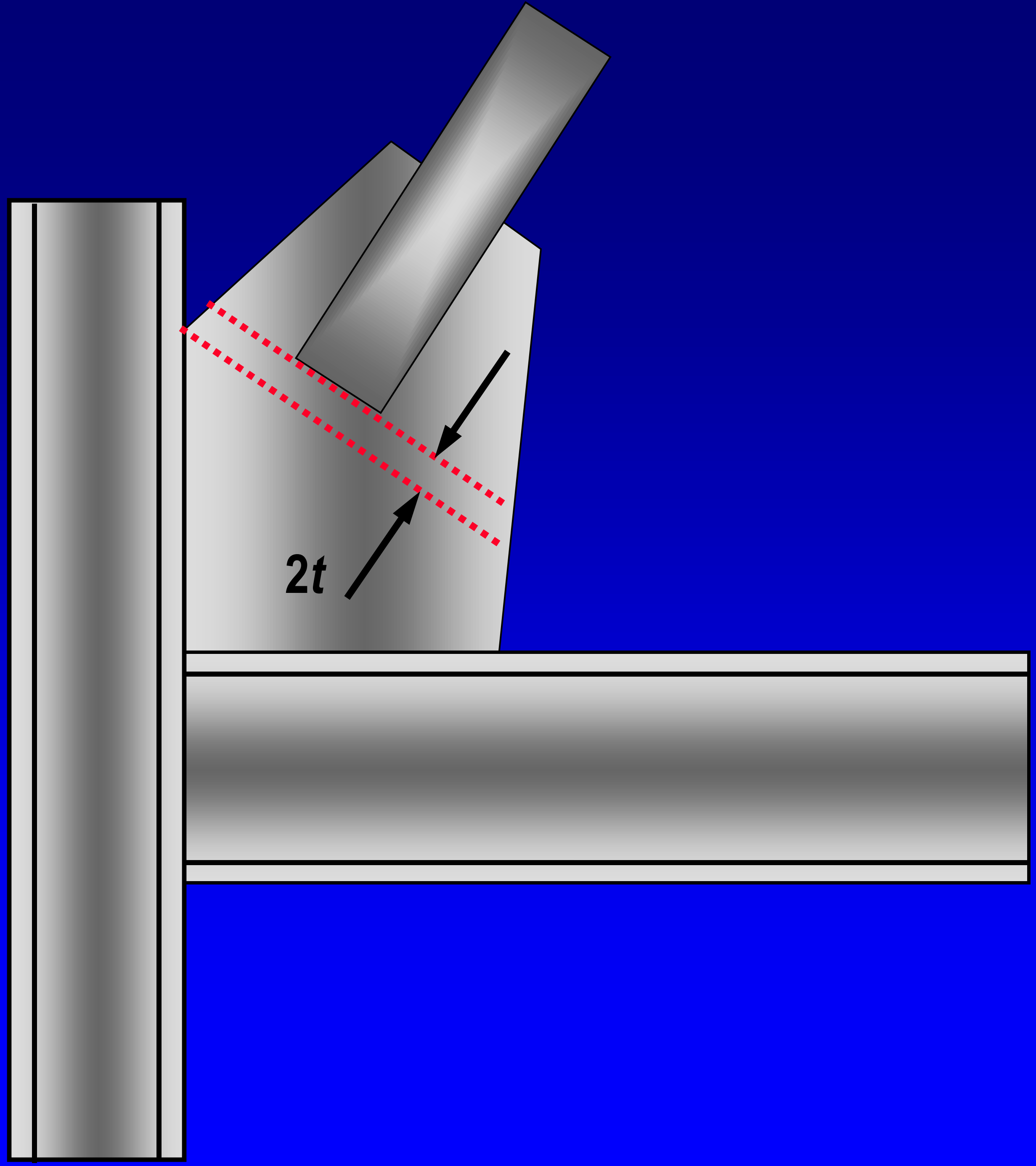
To accommodate brace end rotation: provide "fold line"





2t
Gusset Plate
t = thickness of gusset plate


2t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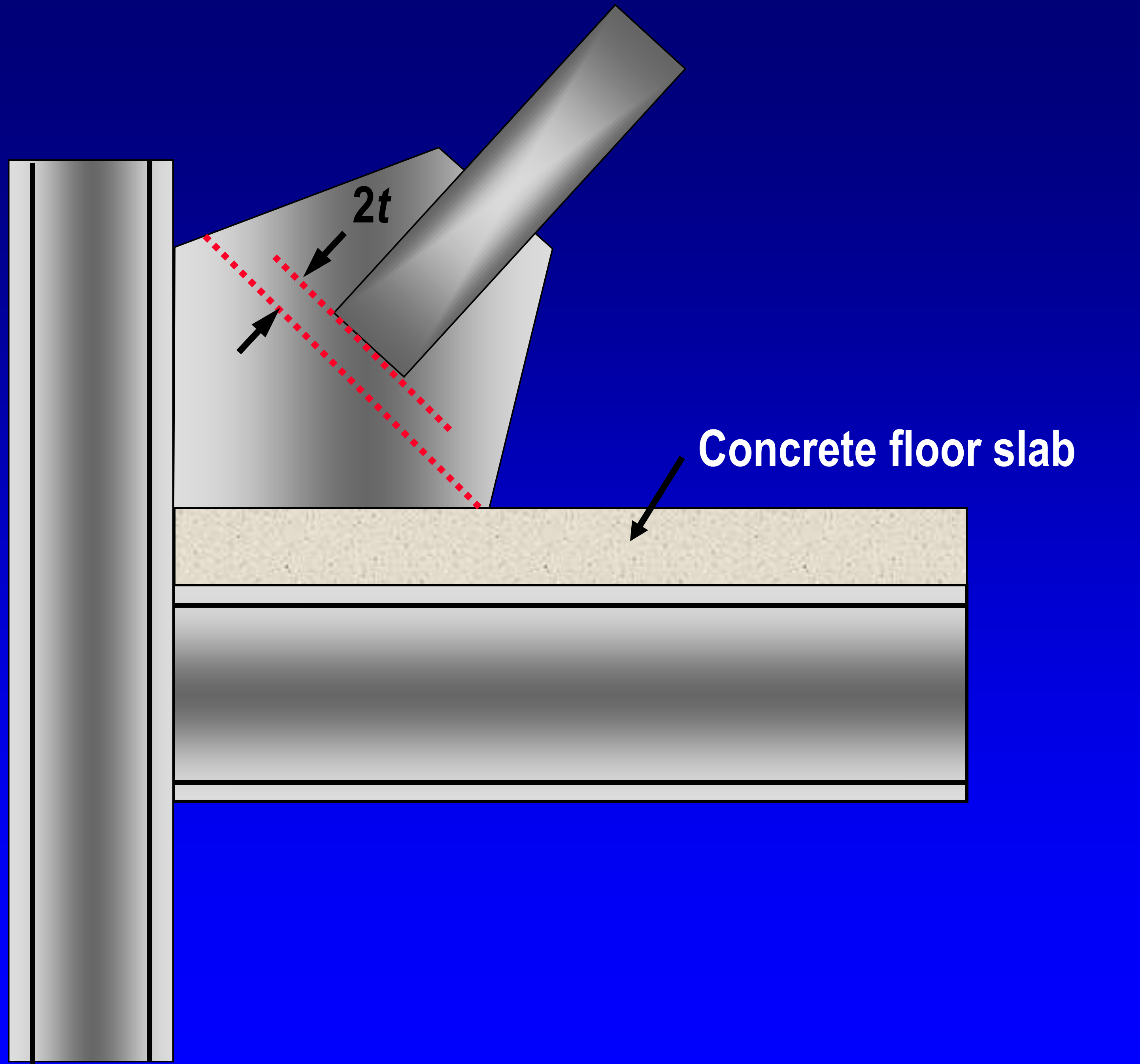
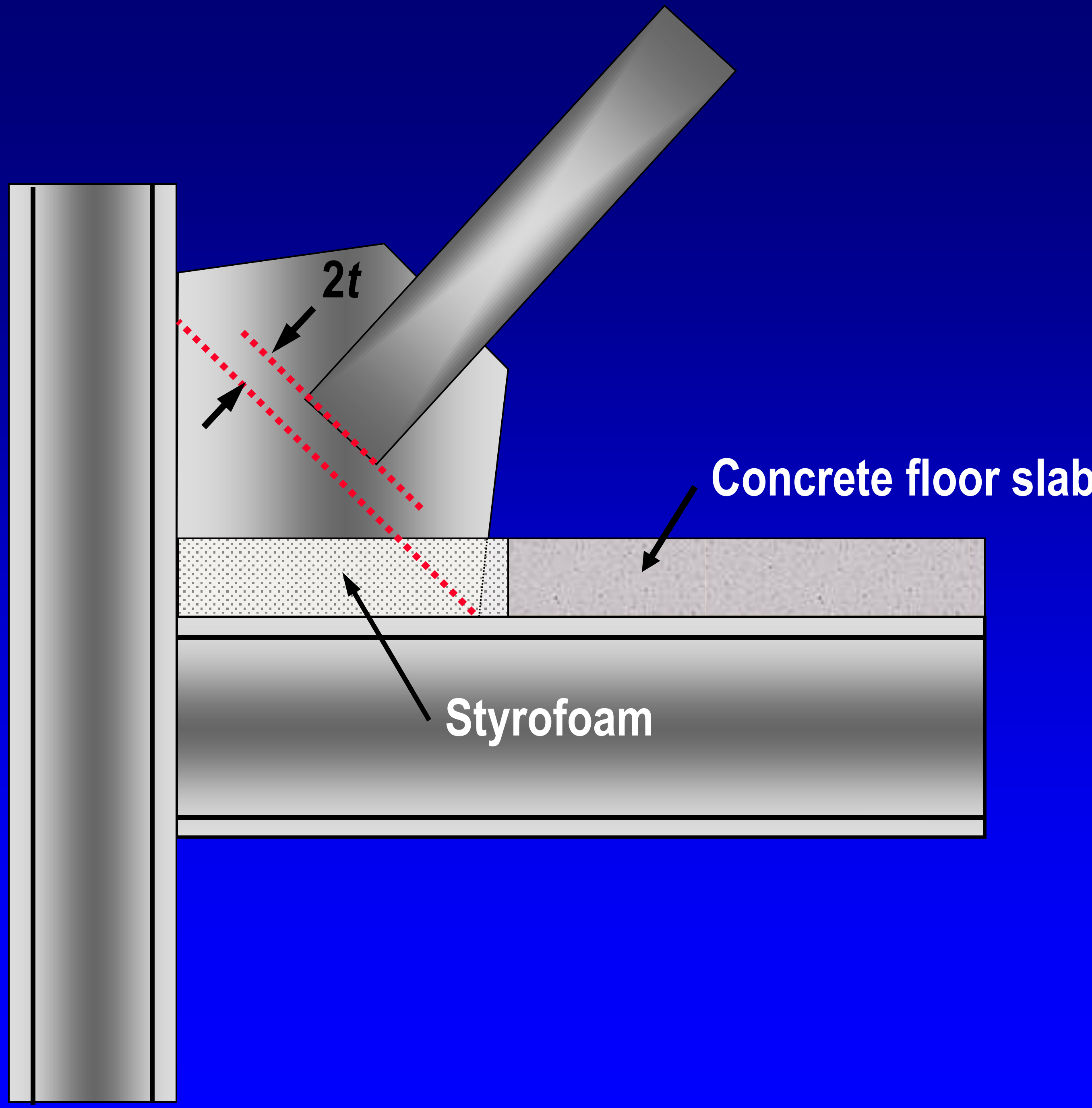
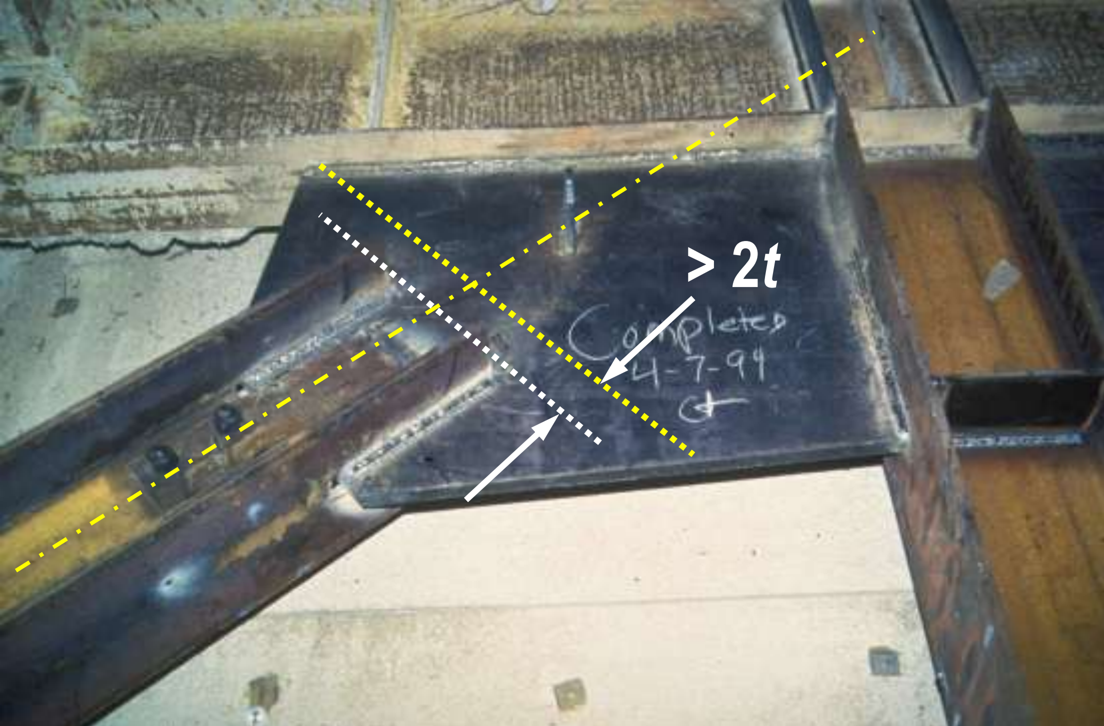

2t
Concrete floor slab

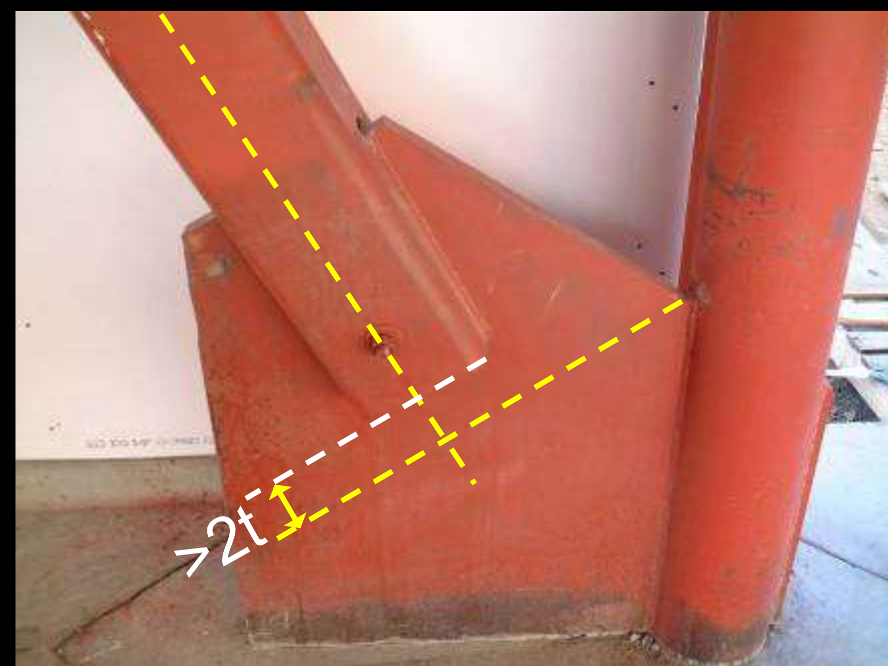


2t
Concrete floor slab
Styrofoam



105D1


> 2t
Completed
4-7-99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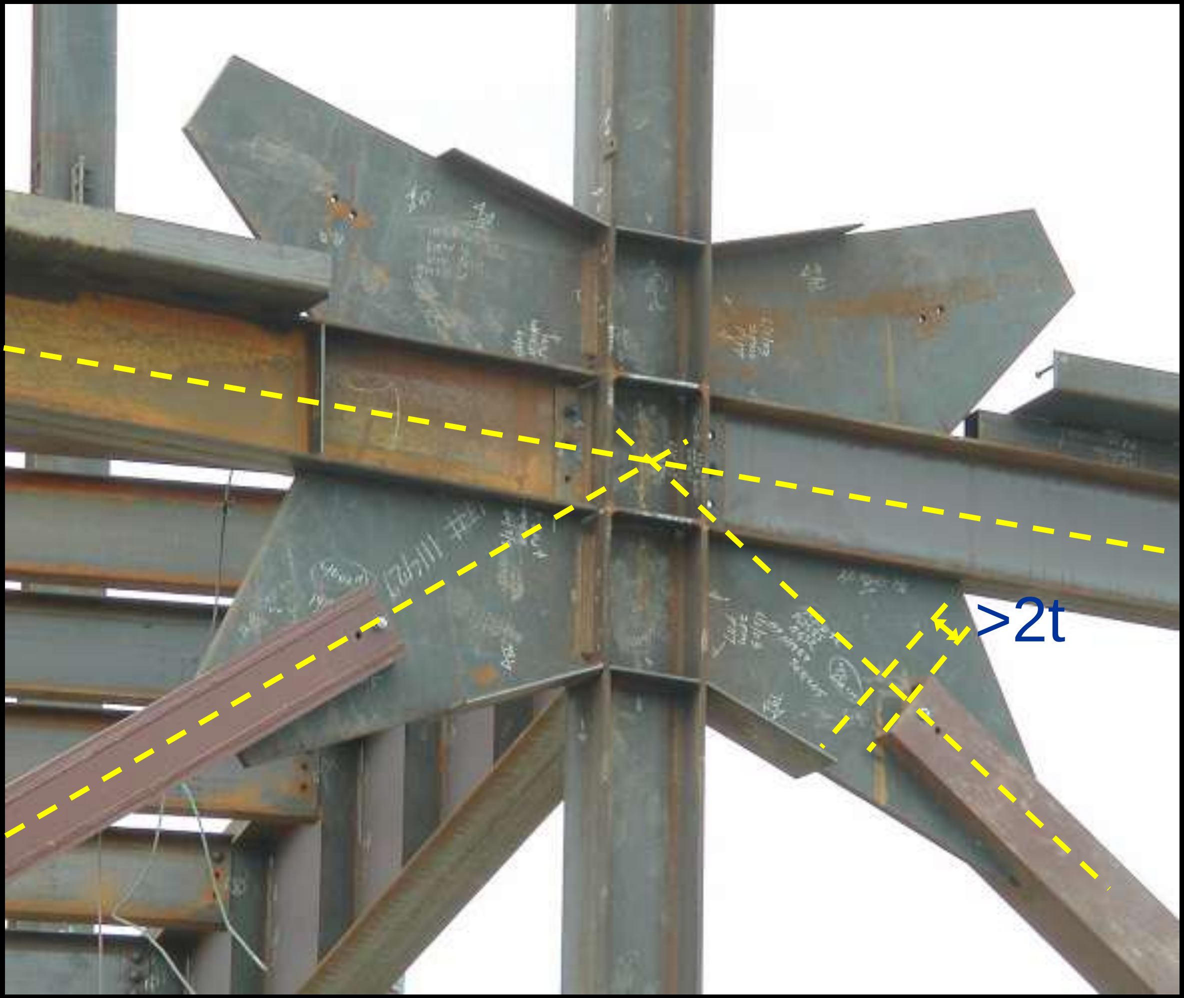

>2t

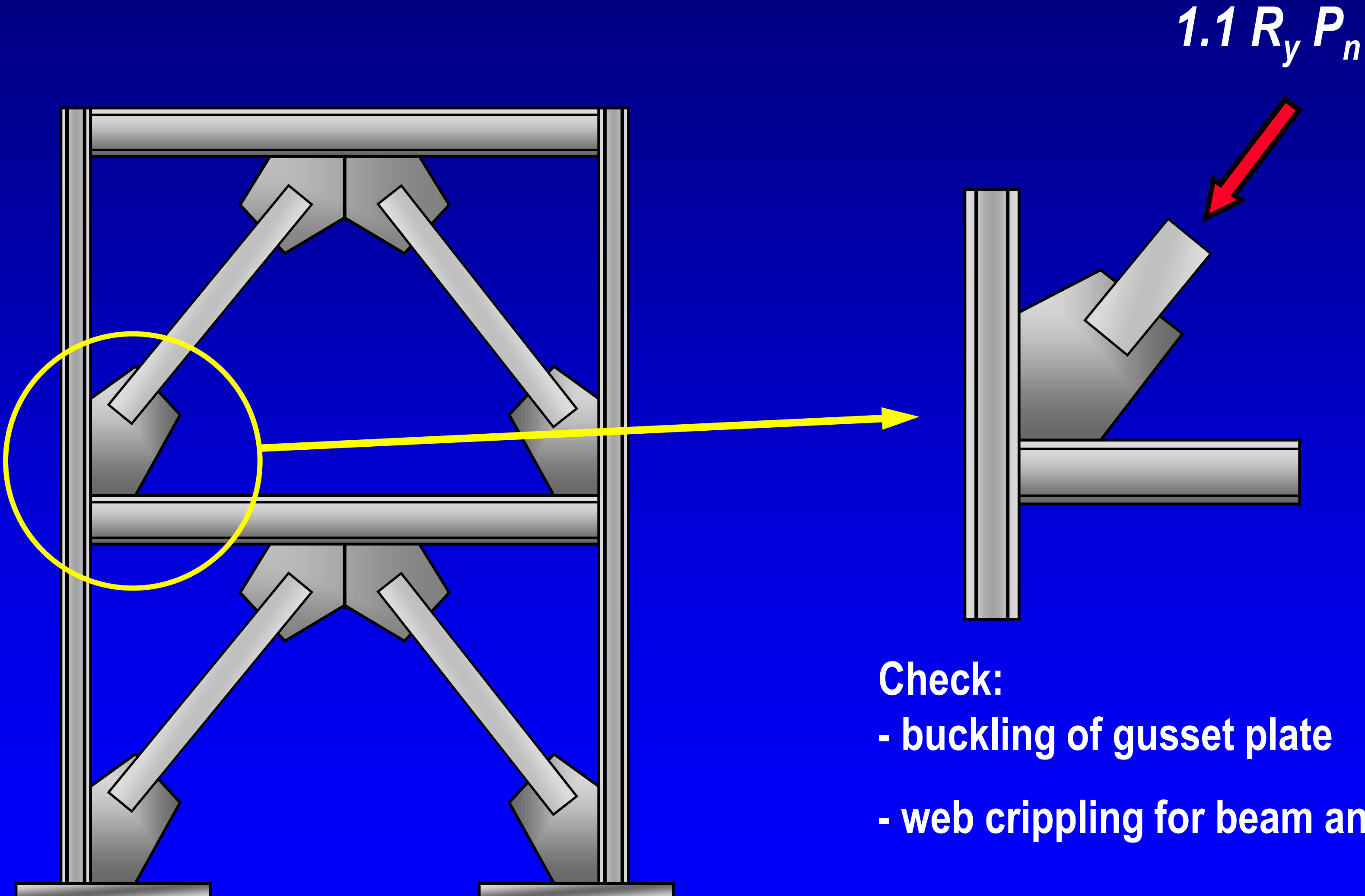

> 2t

13.3 Required Strength of Bracing Connections

13.3c Required Compressive Strength

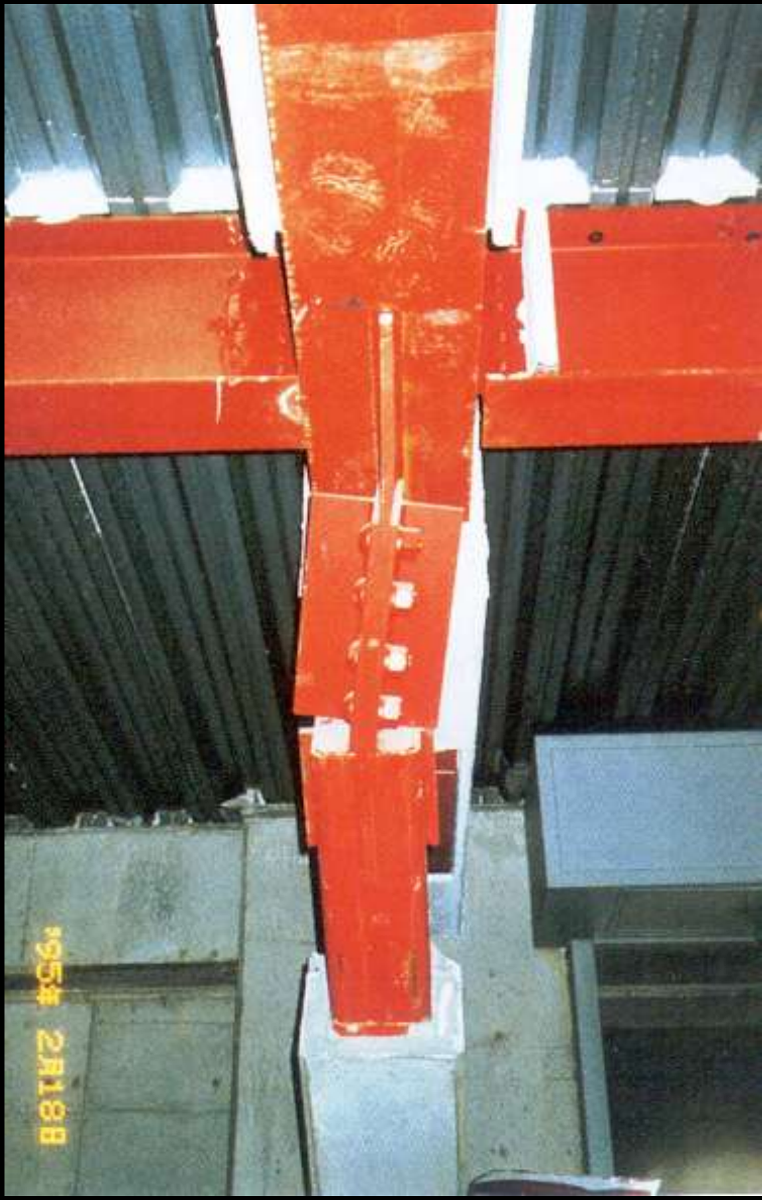
The *required compressive strength* of bracing connections shall be at least 1.1 $R_y P_n$

$P_n = A_g F_{cr}$ of bracing member
(per Chapter E of AISC Main Specification)



Check:

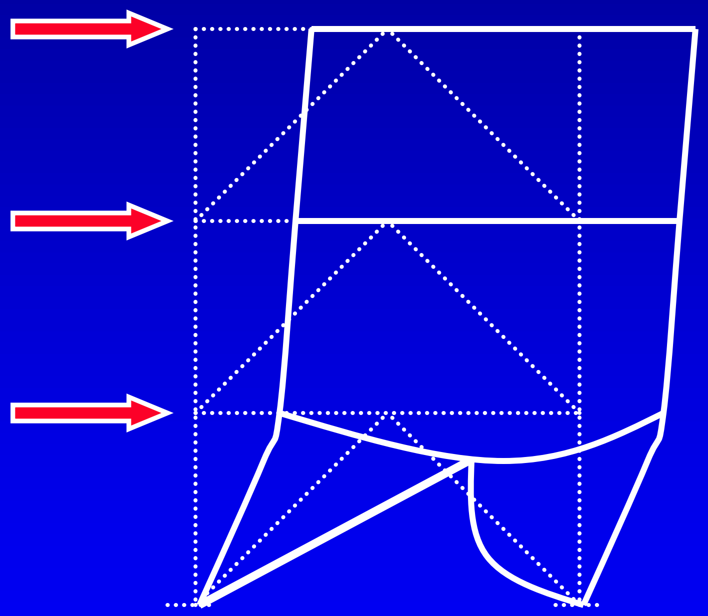
- buckling of gusset plate
- web crippling for beam and column



AISC Seismic Provisions - SCBF

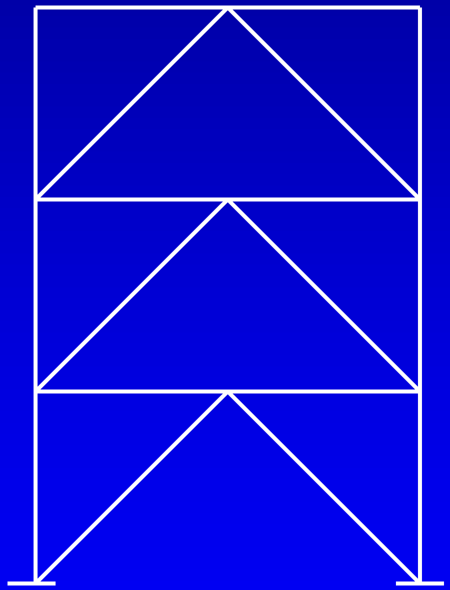
13.4 Special Bracing Configuration Requirements

13.4a V-Type and Inverted V-Type Bracing



13.4 Special Bracing Configuration Requirements

13.4a V-Type and Inverted V-Type Bracing



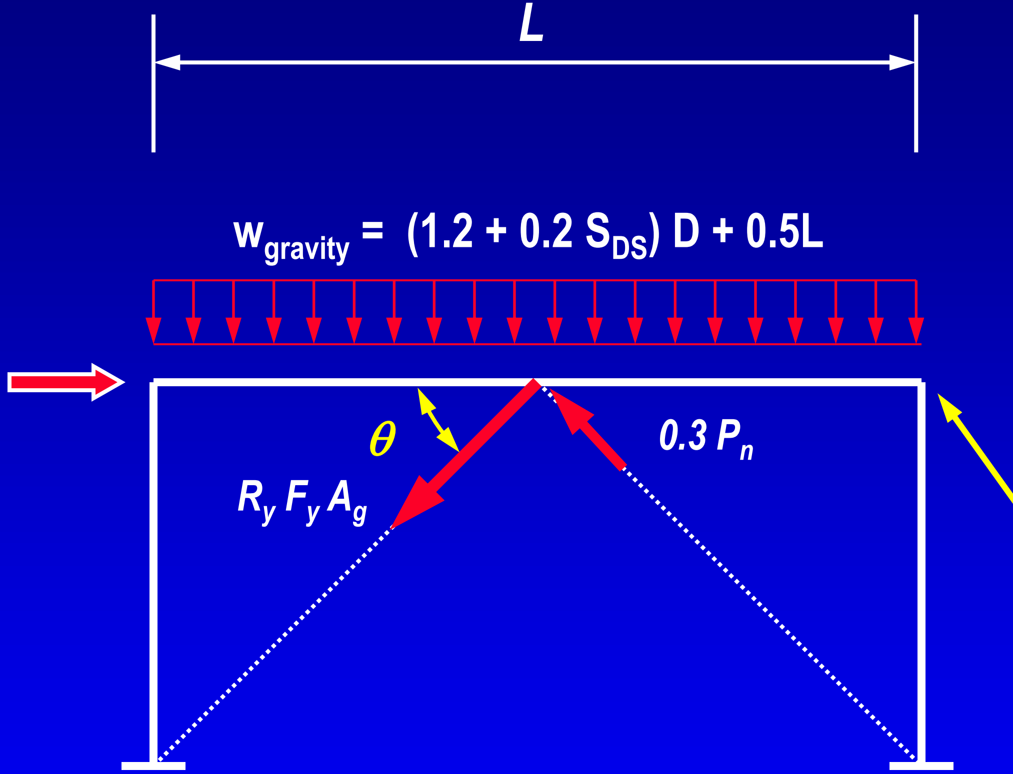
(1) Design beams for unbalanced load that will occur when compression brace buckles and tension brace yields.

Take force in tension brace: $R_y F_y A_g$

Take force in compression brace: $0.3 P_n$

Assume beam has no vertical support between columns.

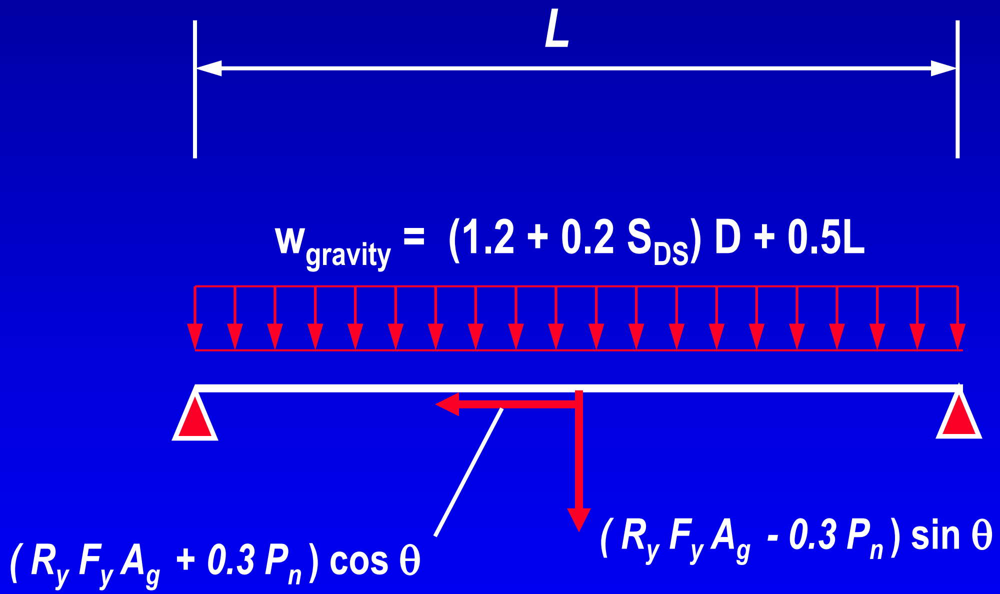
Example



Beam-to-column connections: simple framing

Example

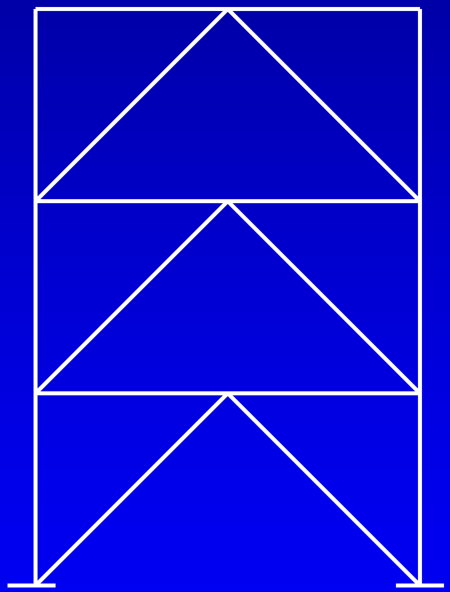
Forces acting on beam:



AISC Seismic Provisions - SCBF

13.4 Special Bracing Configuration Requirements

13.4a V-Type and Inverted V-Type Bracing



(2) Both flanges of beams must be provided with lateral braces with a maximum spacing of L_{pd}

and

Both flanges of the beam must be braced at the point of intersection of the braces.

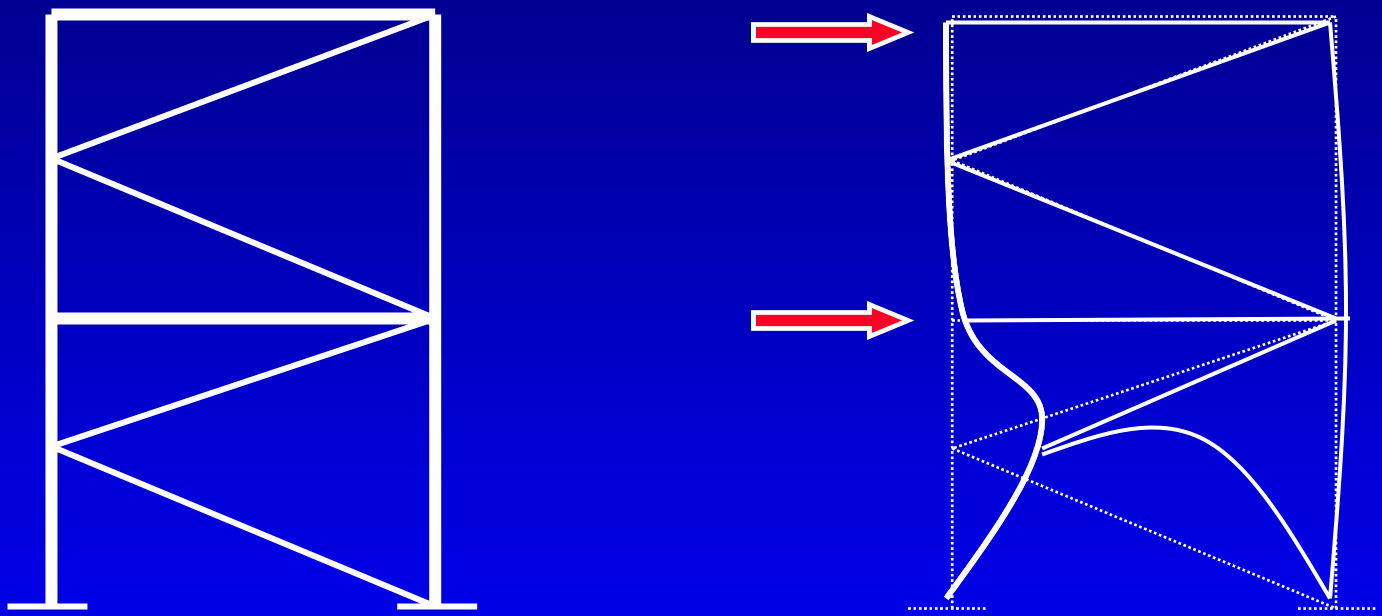
Per Main AISC Specification (Appendix 1):

$$L_{pd} = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y$$

AISC Seismic Provisions - SCBF

13.4 Special Bracing Configuration Requirements

13.4b K-Type Bracing



K-Type Braces are not Permitted for SCBF

Section 13
Special Concentrically Braced Frames (SCBF)

13.1 Scope

13.2 Members

13.3 Required Strength of Bracing Connections

13.4 Special Bracing Configuration Requirements

13.5 Column Splices

13.6 Protected Zone