Design of Seismic-Resistant Steel Building Structures

6. Special Plate Shear Walls

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with the support of the
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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

6 - Special Plate Shear Walls

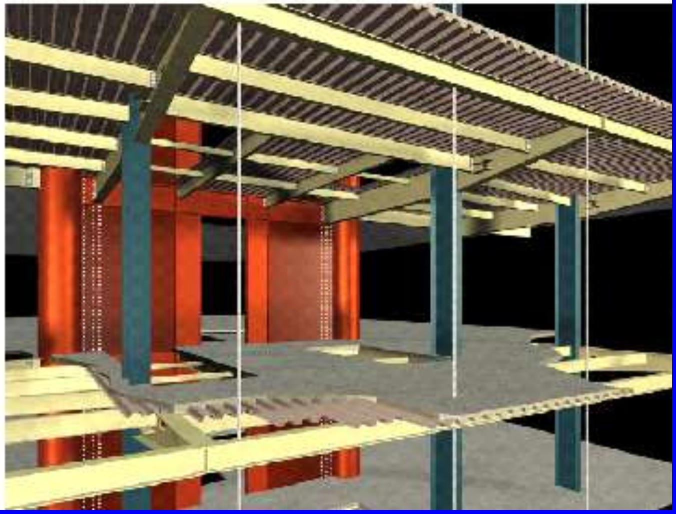
- Introduction and background
- Mechanics of slender-web shear walls
- Design of Special Plate Shear Walls
- AISC Seismic Provisions for Special Plate Shear Walls
- History: Research and Applications
- Materials, Serviceability, and Configurations

Steel plate shear wall panel in Japan: Wall with horizontal panel stiffeners

Steel plate shear wall panel in Japan: Wall with horizontal and vertical stiffeners

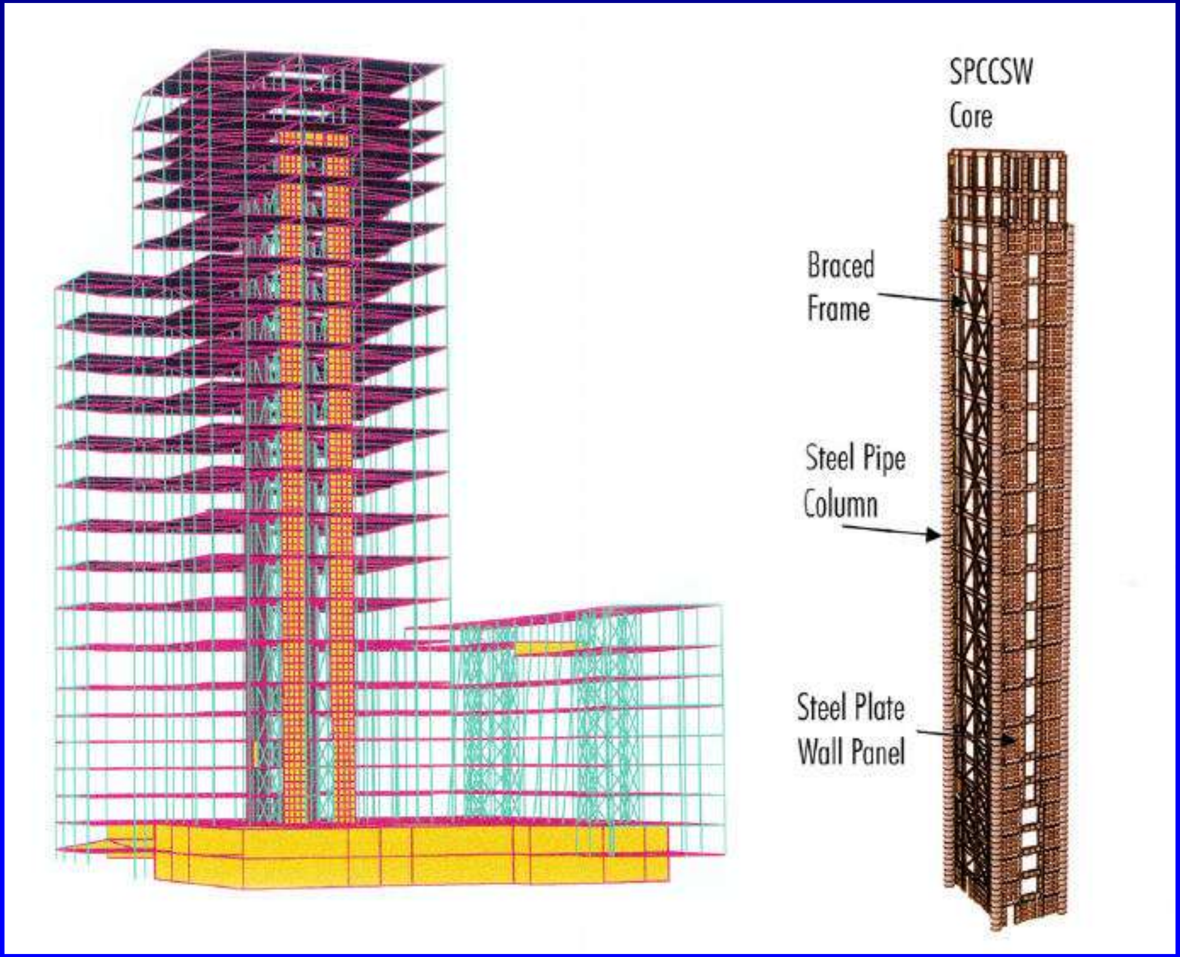
Courtesy of Nippon Steel

U.S. Federal Courthouse, Seattle



Courtesy of John Hooper, MKA Seattle

Structural System for U.S. Federal Courthouse, Seattle



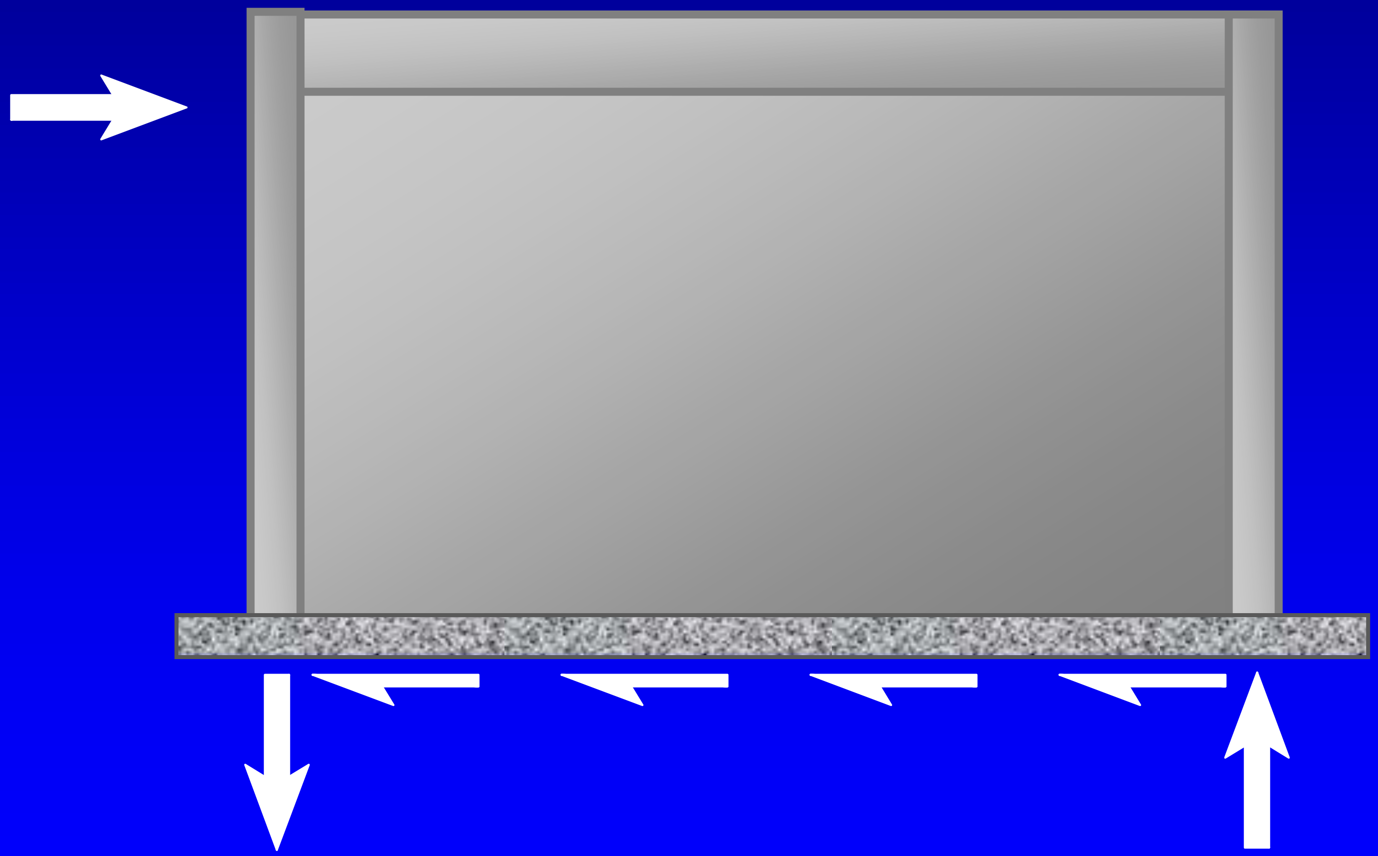
Courtesy of John Hooper, MKA Seattle

Steel plate shear walls in residential light-frame construction

Courtesy of Matt Eatherton, GFDS

Courtesy of Jon Brody Structural Engineers

Simplified Wall Analogy



Tension-Only Braced Frame Analogy

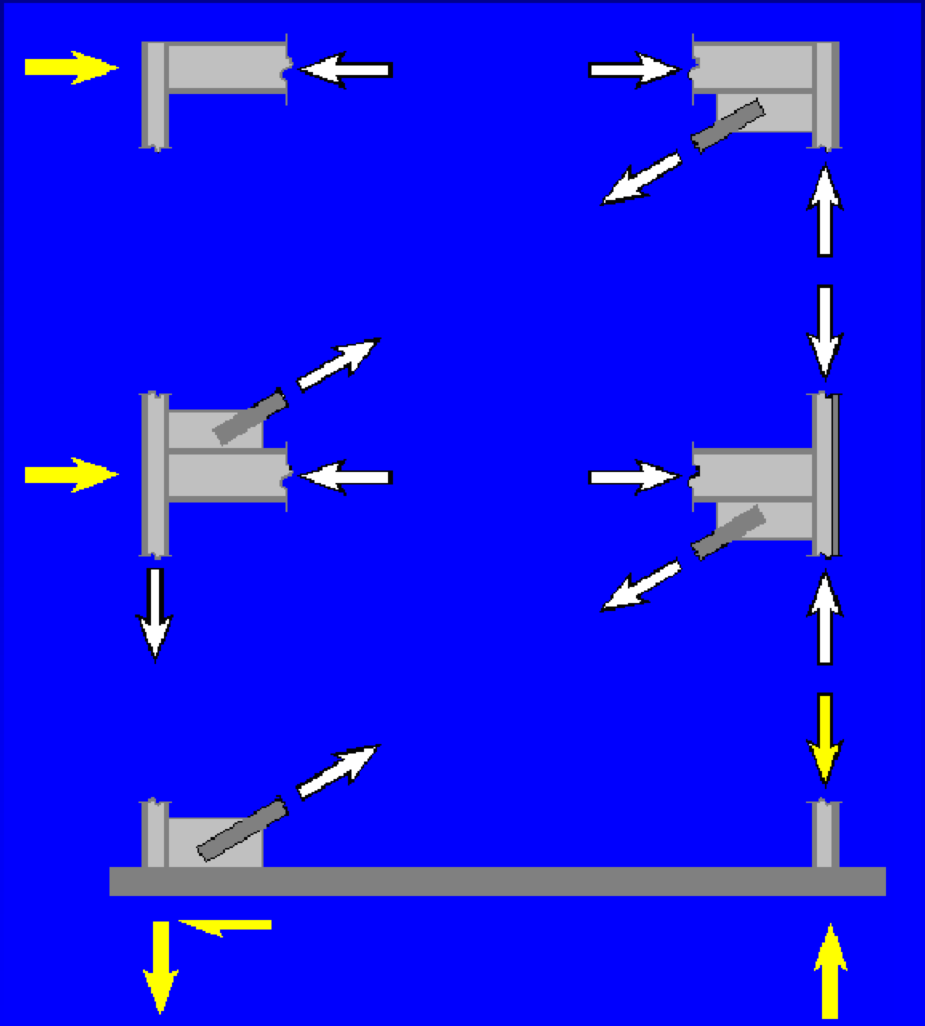


Plate-Girder Analogy

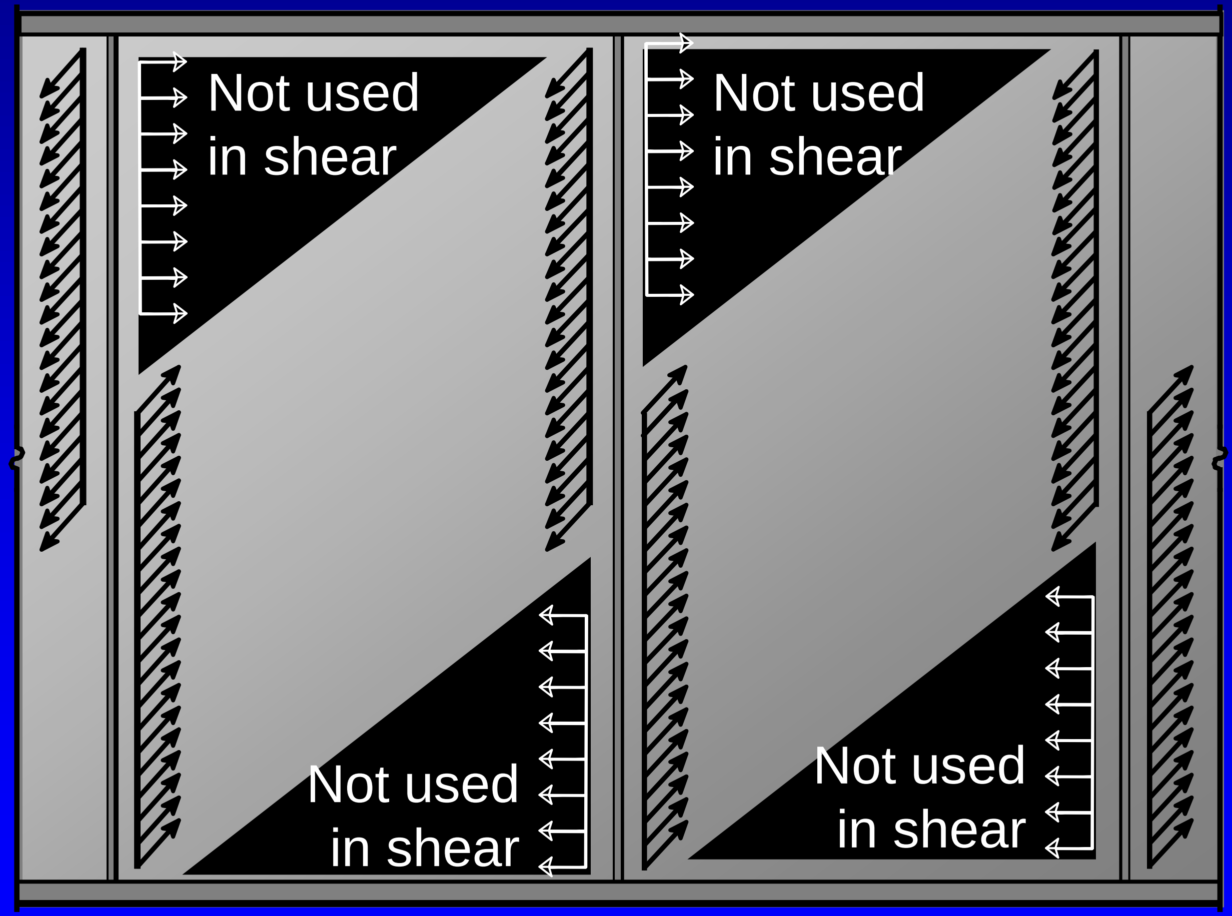
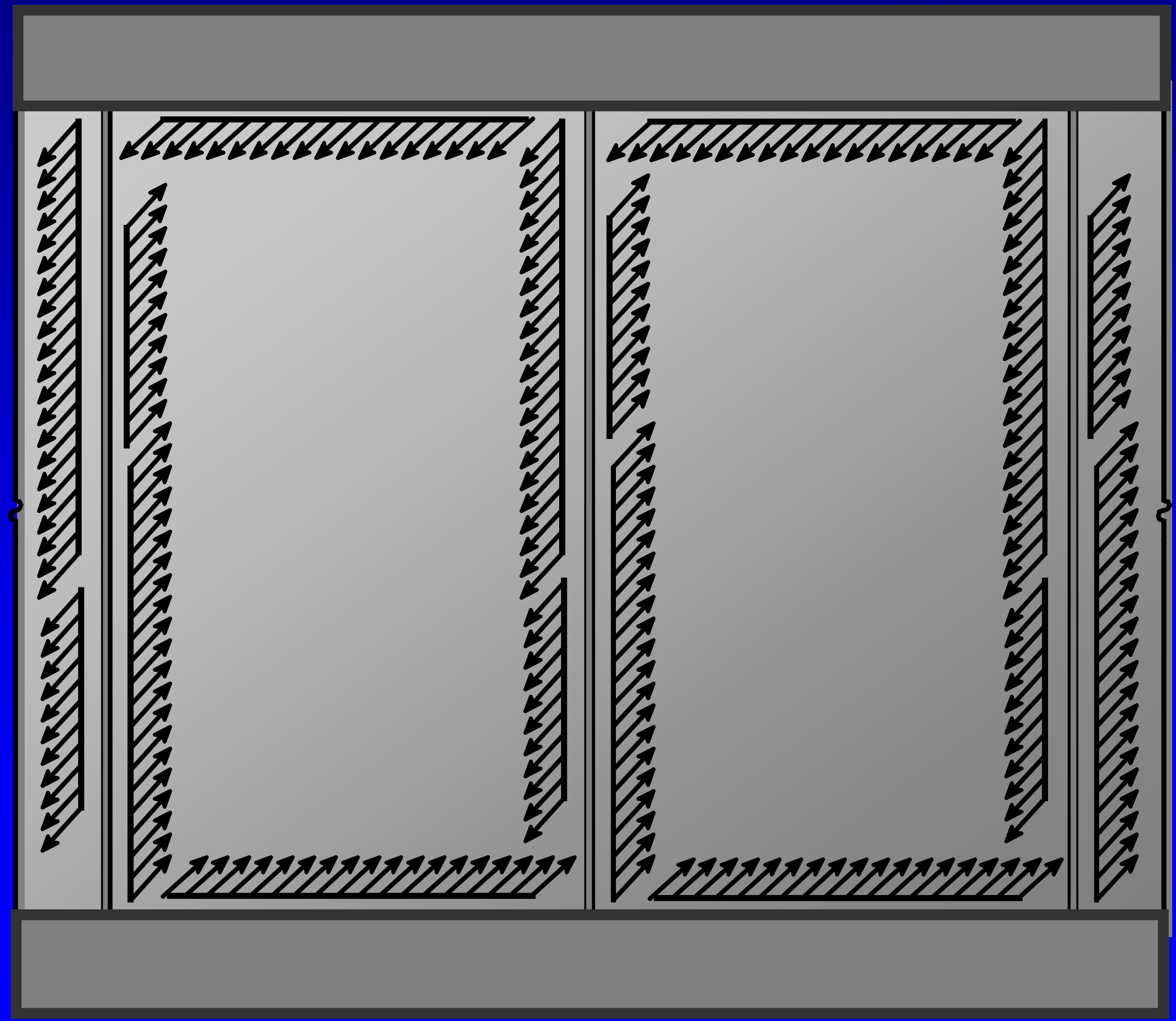
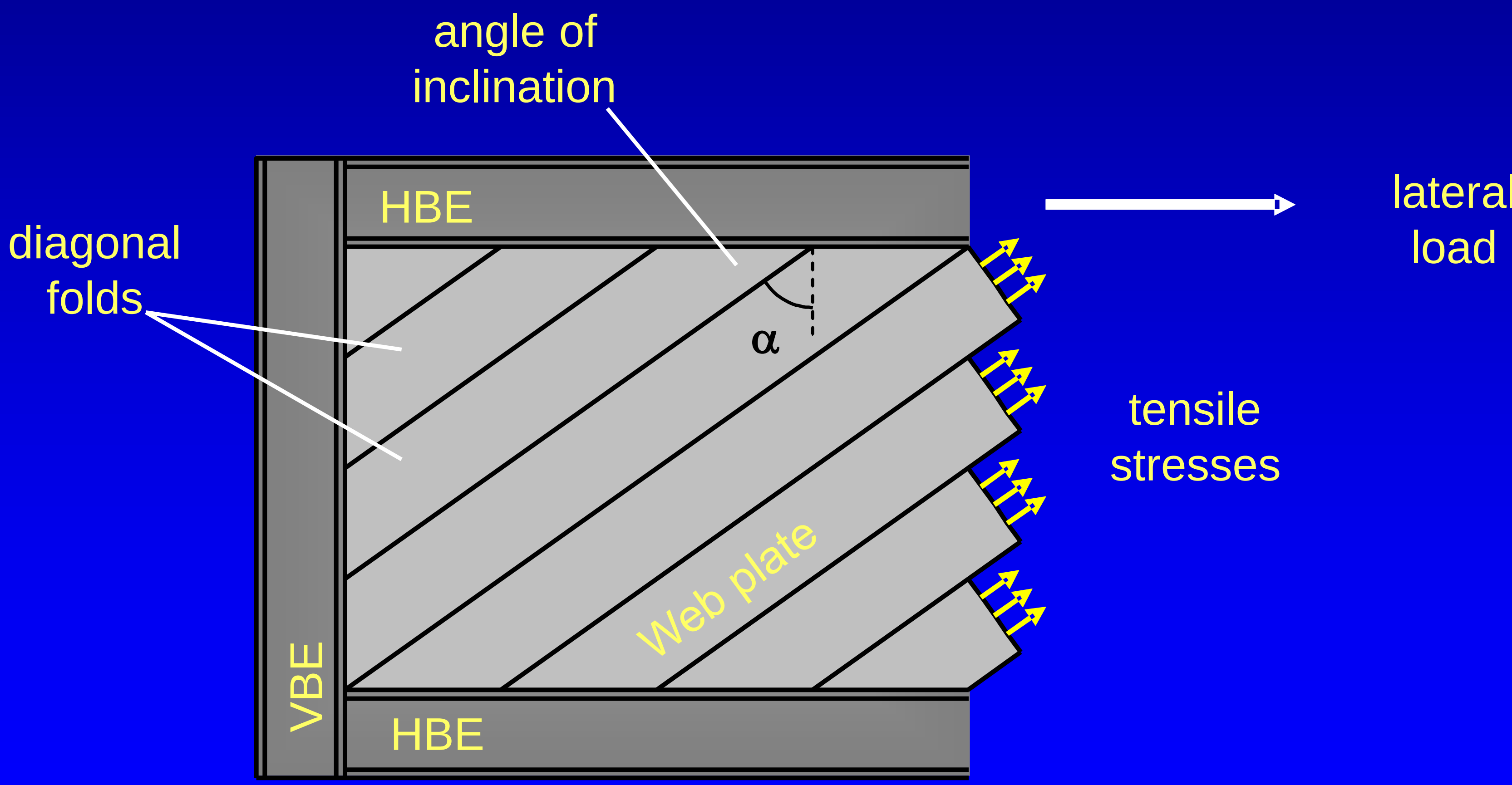


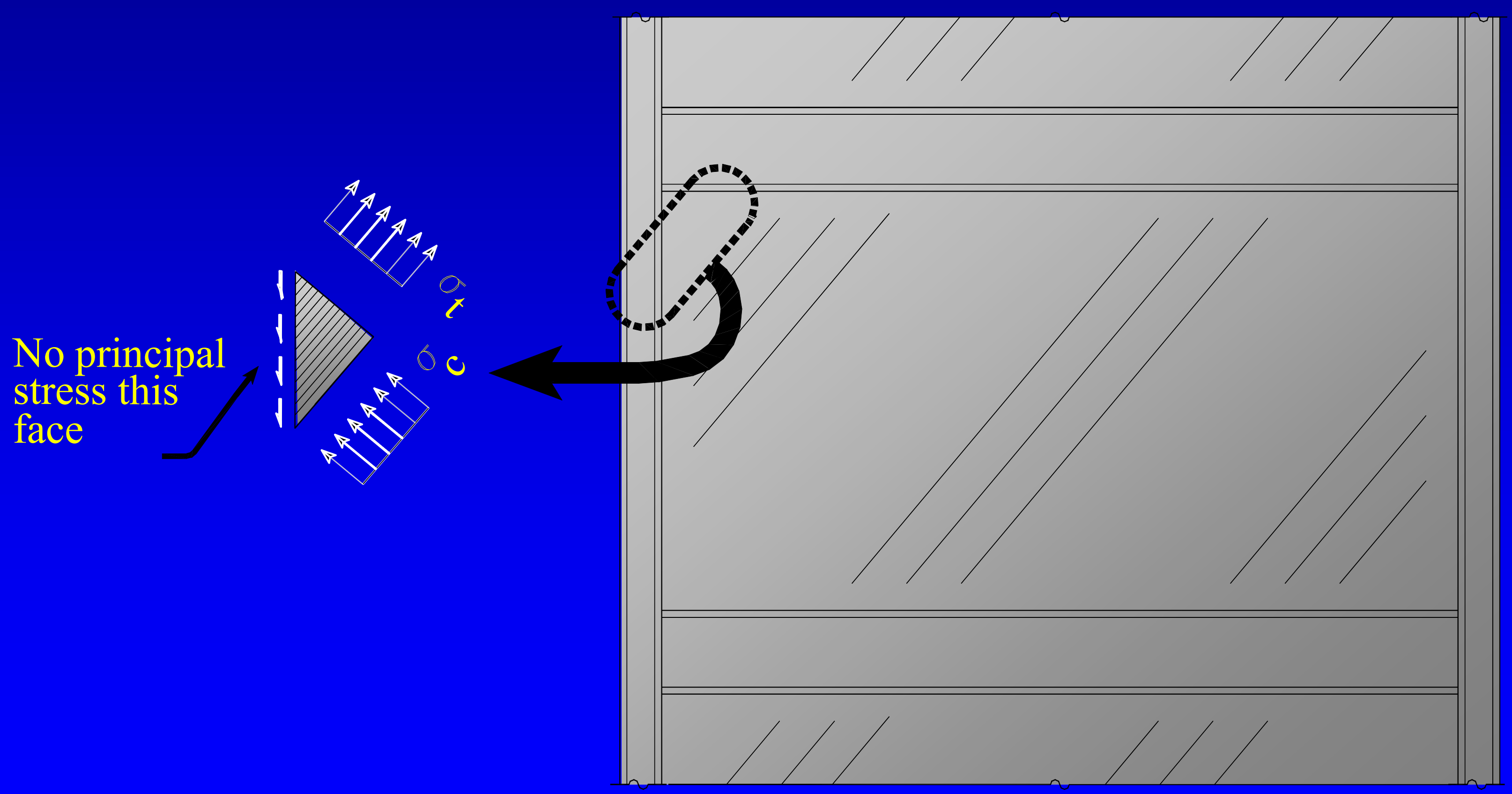
Plate-Girder Analogy



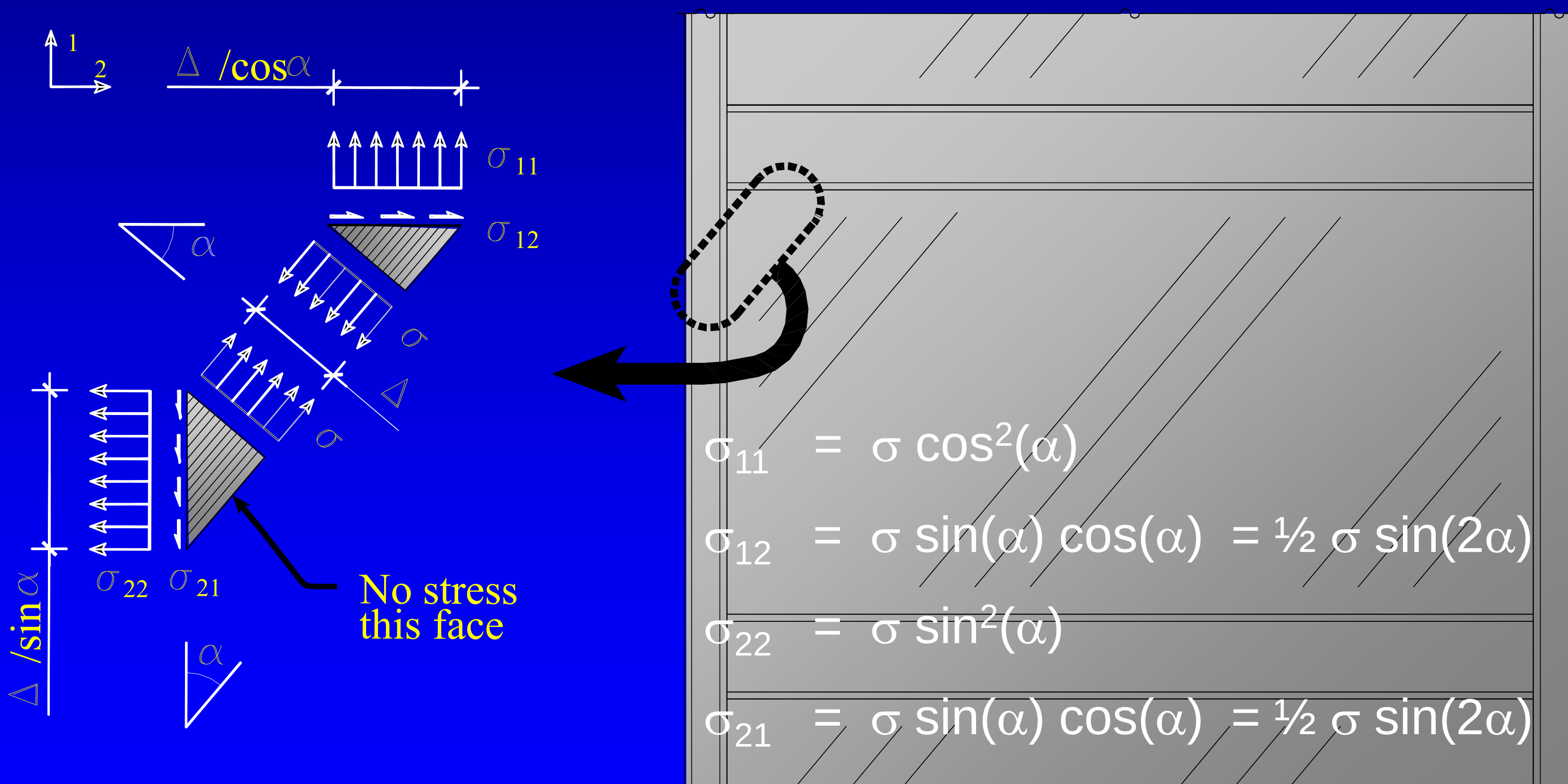
Diagonal tension in steel plate shear wall web plate



Boundary Tension and Shear Stresses (pure shear)



Boundary Tension and Shear Stresses (diagonal tension)



$$\sigma_{11} = \sigma \cos^2(\alpha)$$

$$\sigma_{12} = \sigma \sin(\alpha) \cos(\alpha) = \tfrac{1}{2}\, \sigma \sin(2\alpha)$$

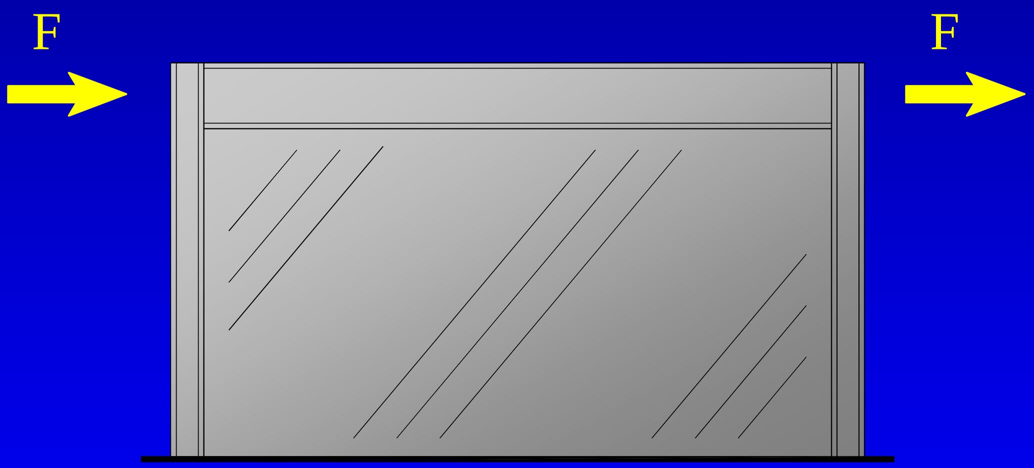
$$\sigma_{22} = \sigma \sin^2(\alpha)$$

$$\sigma_{21} = \sigma \sin(\alpha) \cos(\alpha) = \tfrac{1}{2}\, \sigma \sin(2\alpha)$$

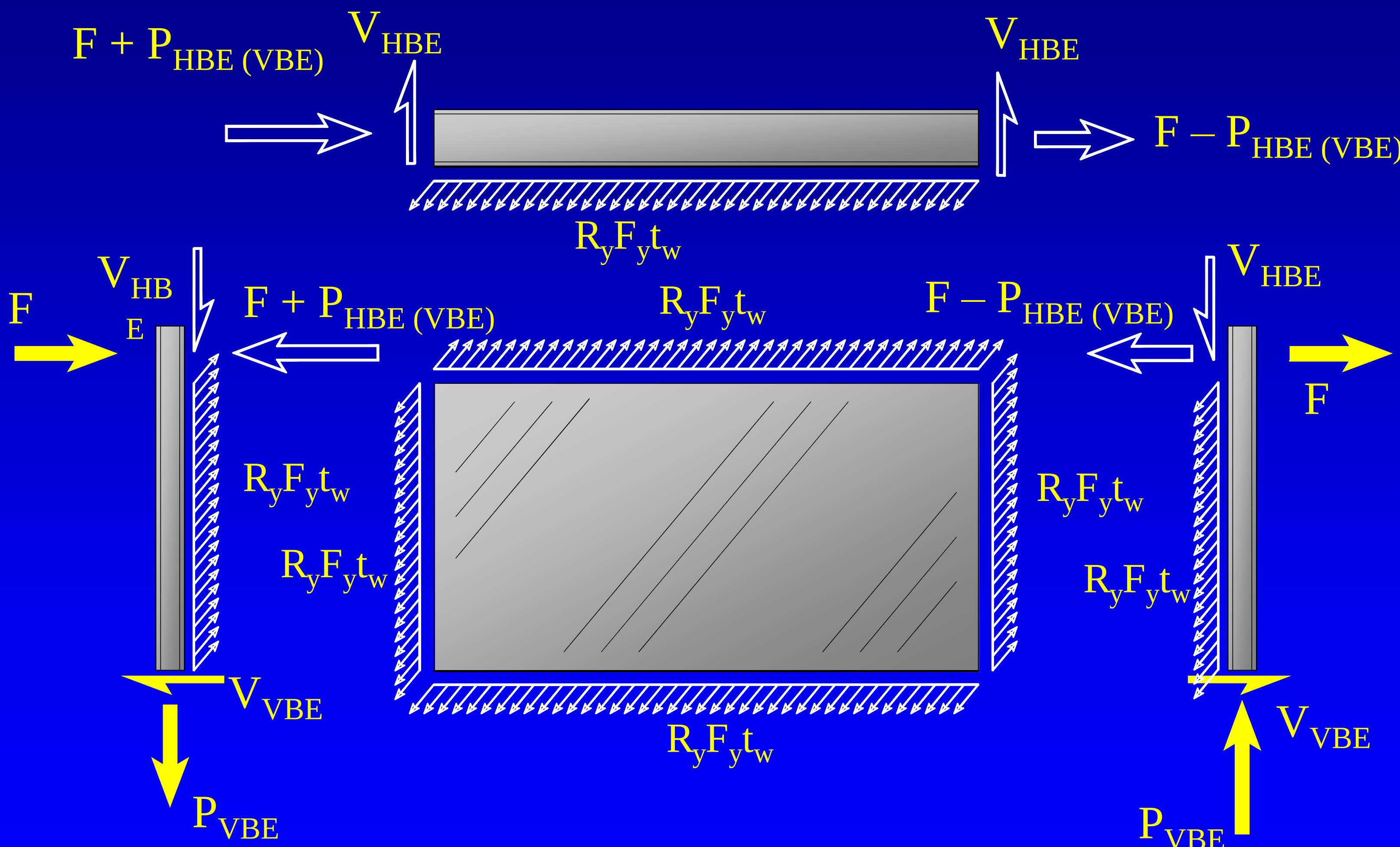
Angle α

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w L}{2 A_c}}{1 + t_w h \left[\dfrac{1}{A_b} + \dfrac{h^3}{360 I_c L} \right]}$$

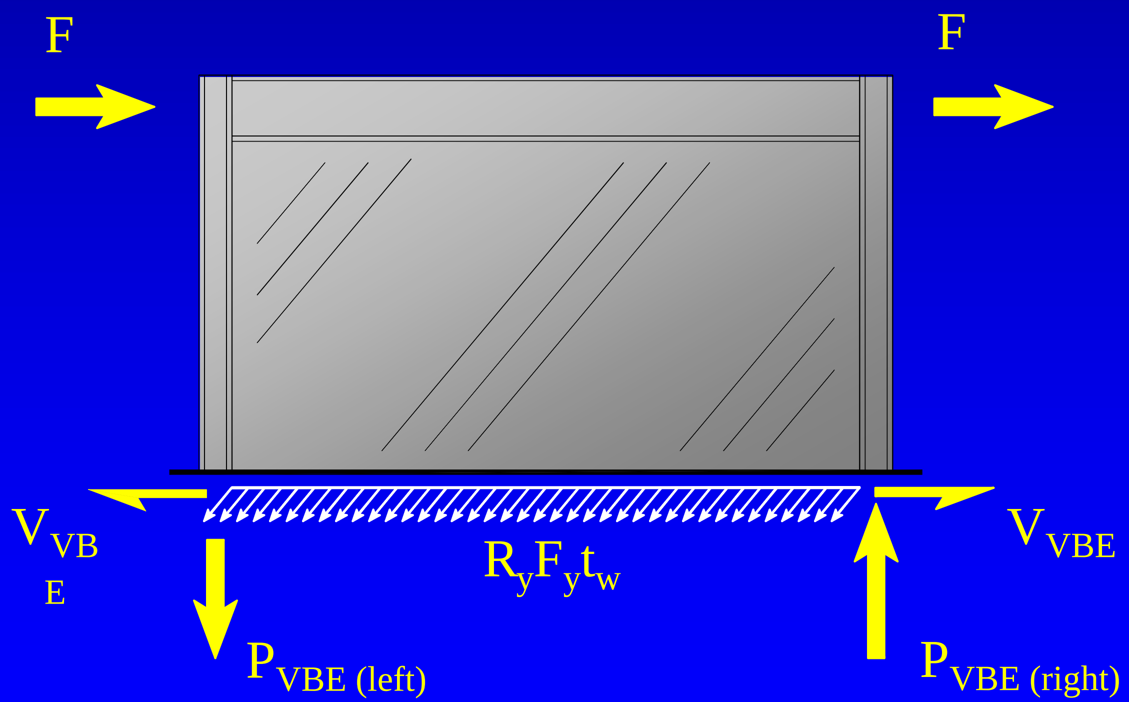
Mechanics of Slender Web Plates



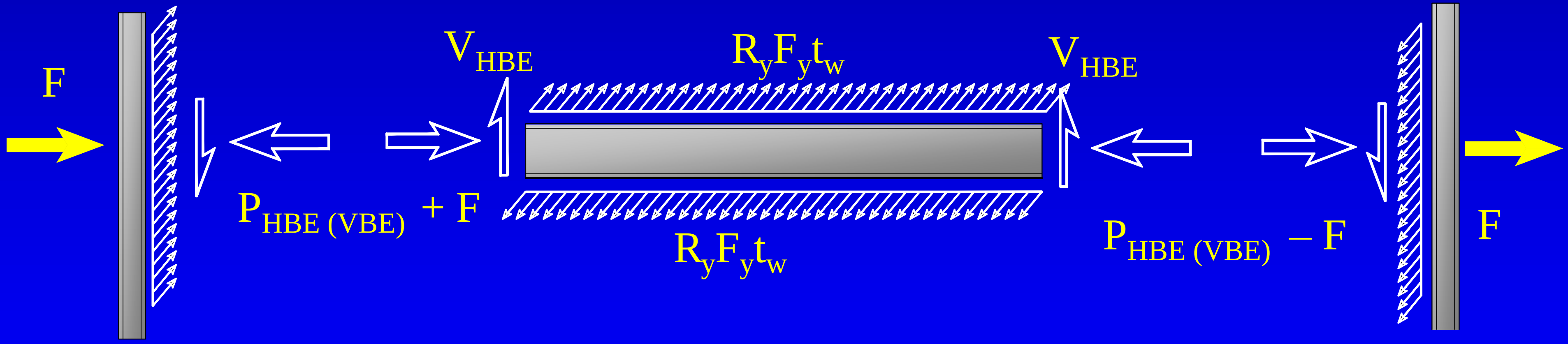
Internal Forces



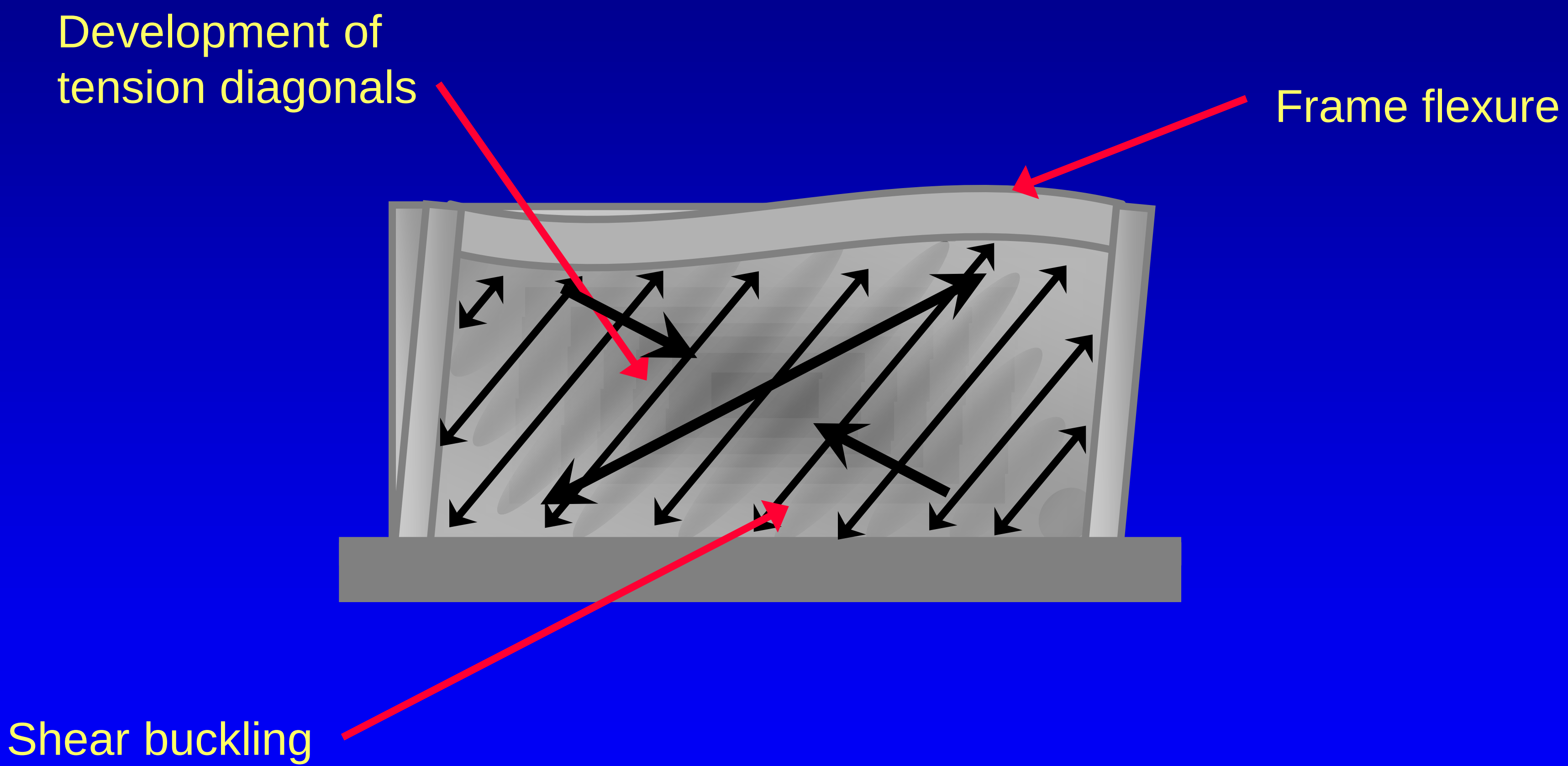
Reactions



Multi-Story Internal Forces



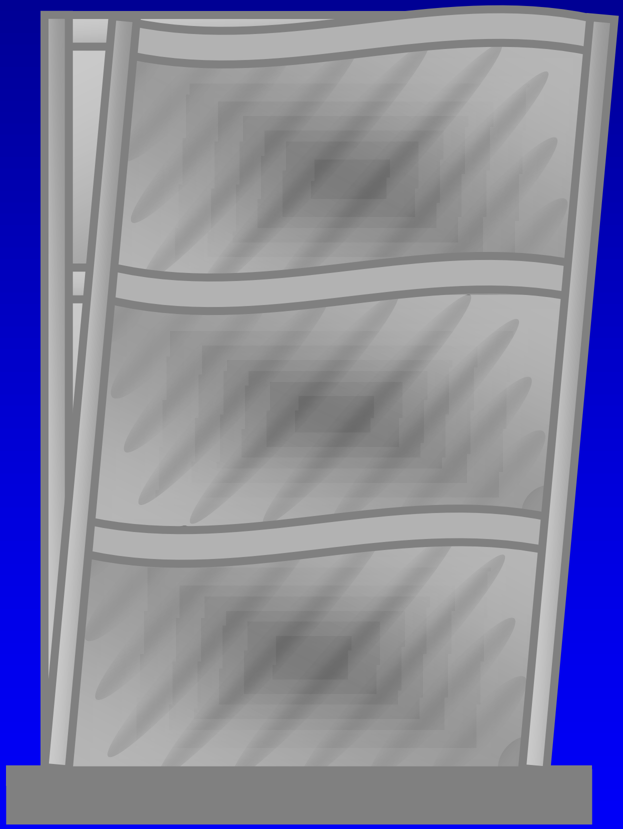
Expected Yield Mode



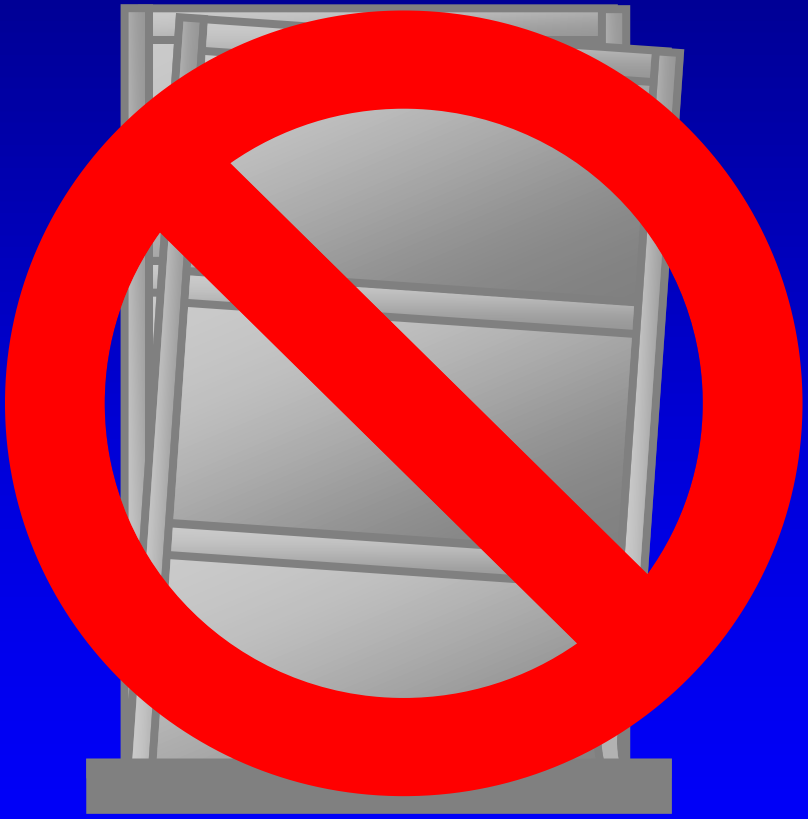
Buckling of steel plate shear wall web plate at 1.82% Drift

Courtesy of Berman and Bruneau

Expected Yield Mode



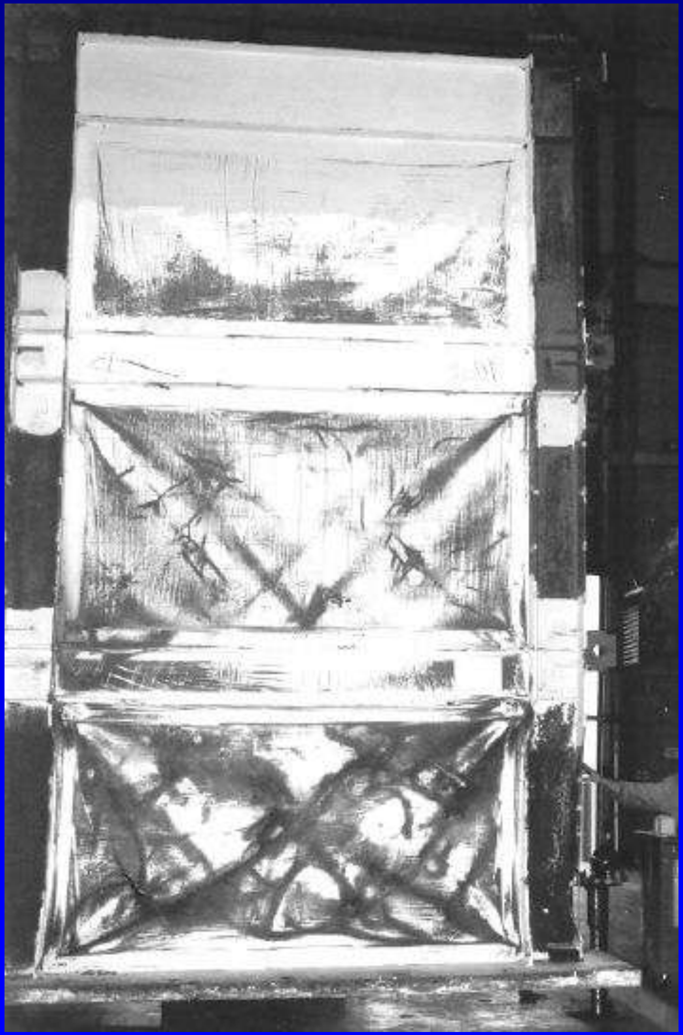
Multi-story shear mode

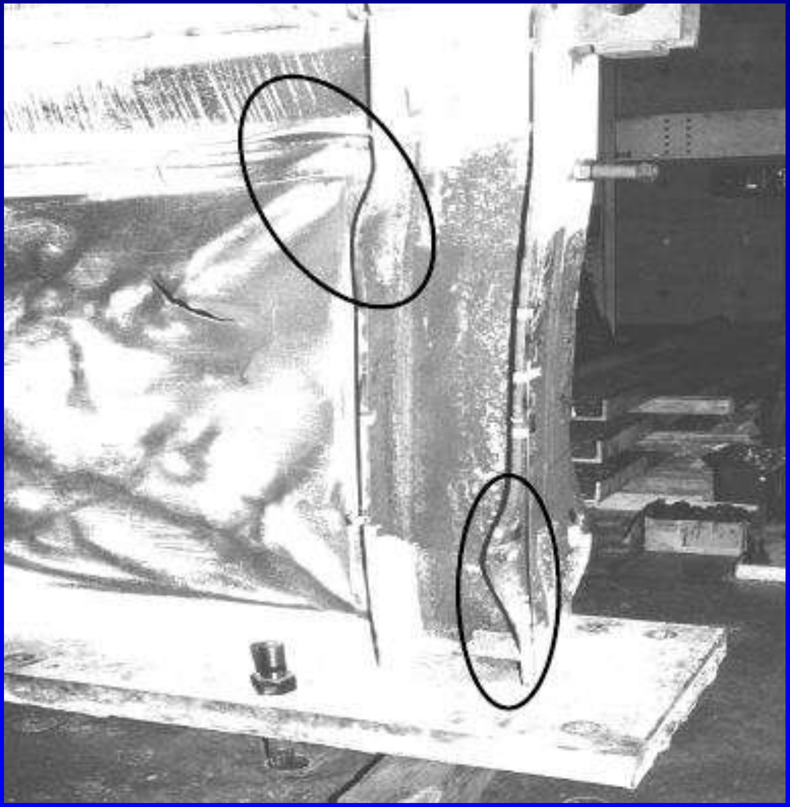


“Flexural” mode:

Axial yield at base

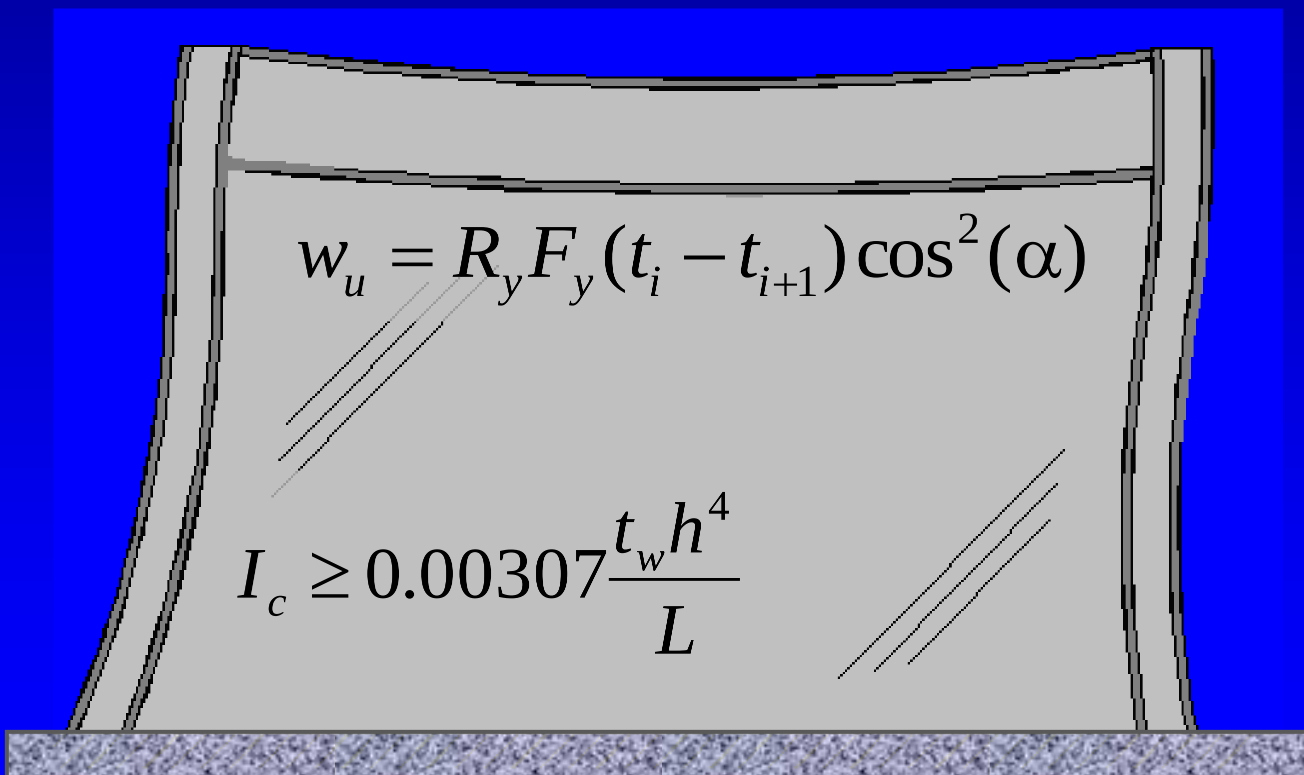
Testing of a multi-story steel plate shear wall





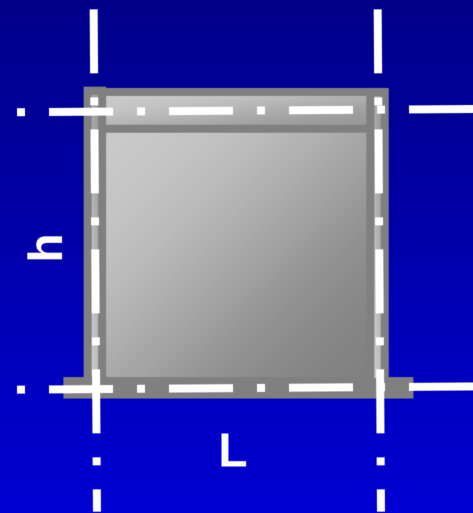
Courtesy of Behbahanifard

Inward Flexure of Boundary Elements

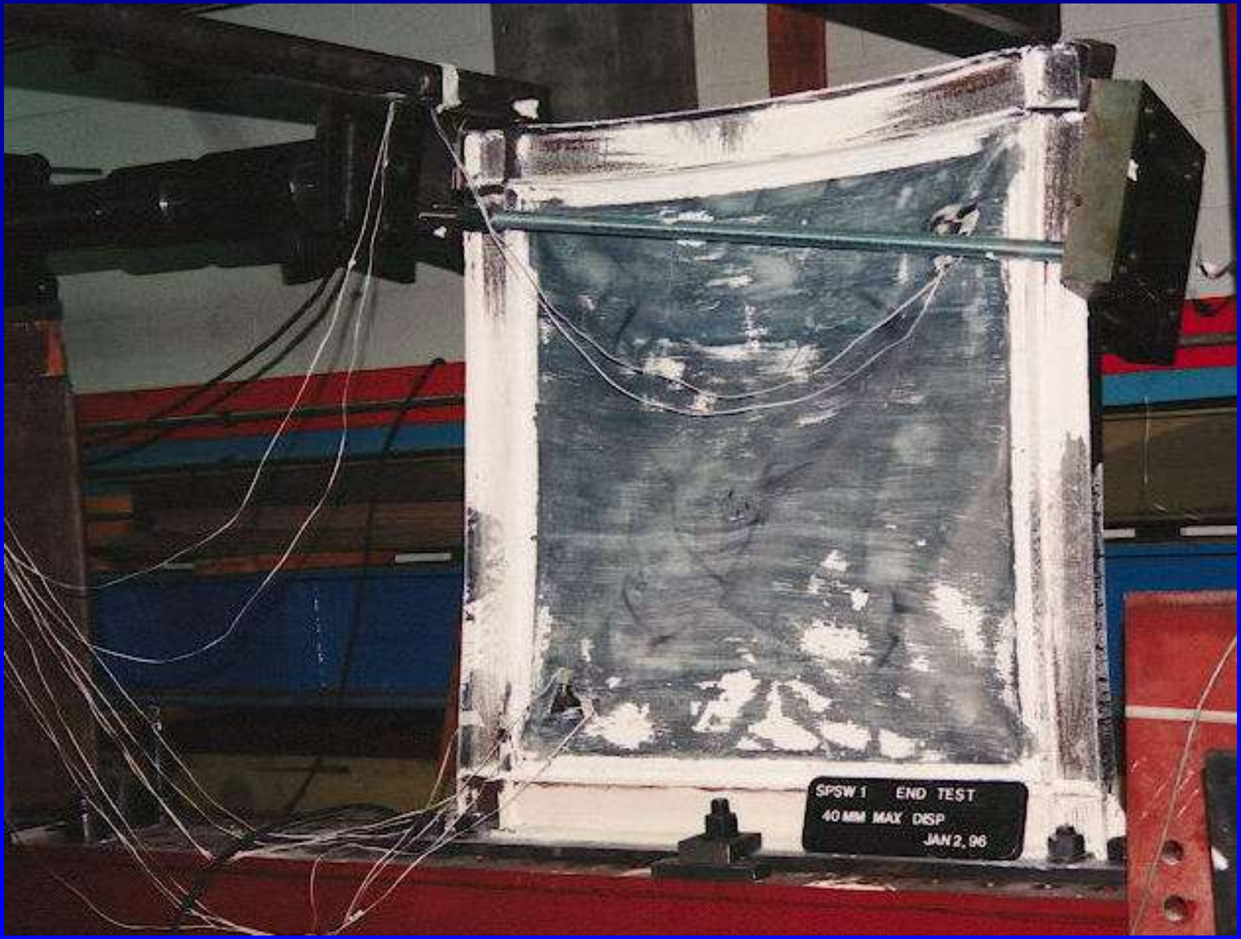


$$w_u = R_y F_y (t_i - t_{i+1}) \cos^2(\alpha)$$

$$I_c \geq 0.00307 \frac{t_w h^4}{L}$$

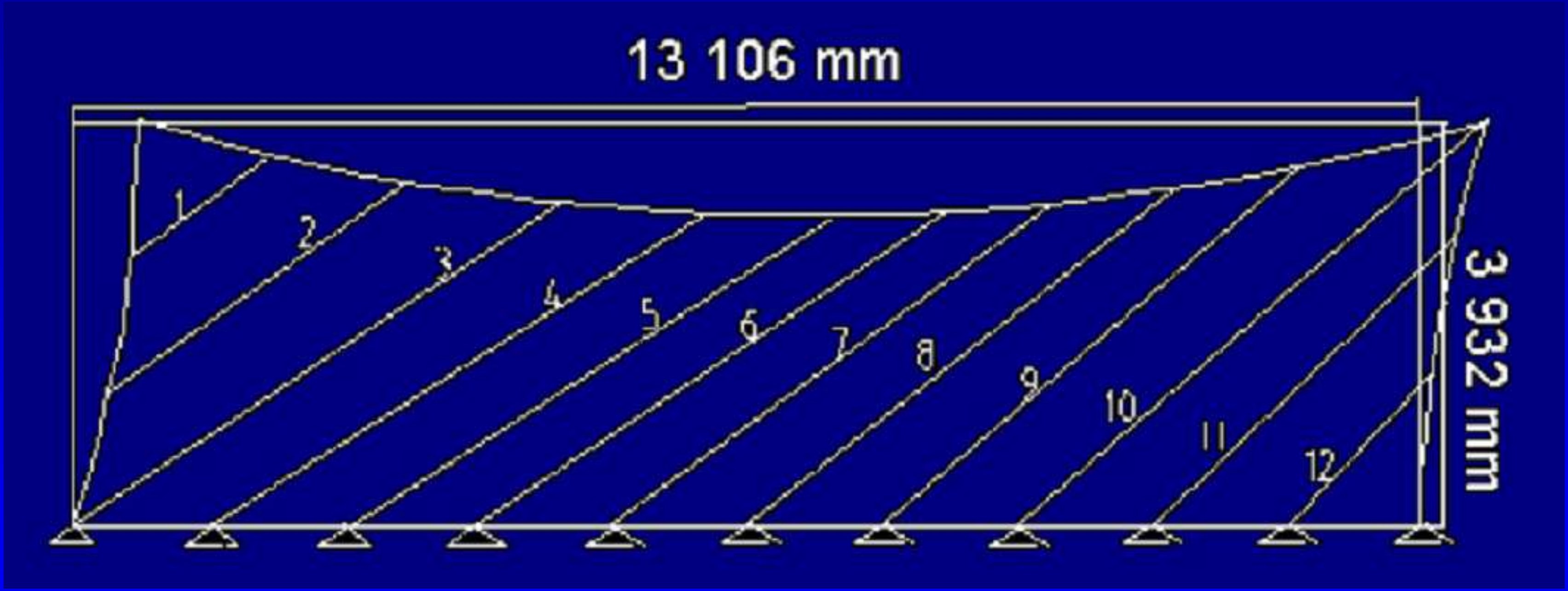


Inward flexure of steel plate shear wall beams and columns

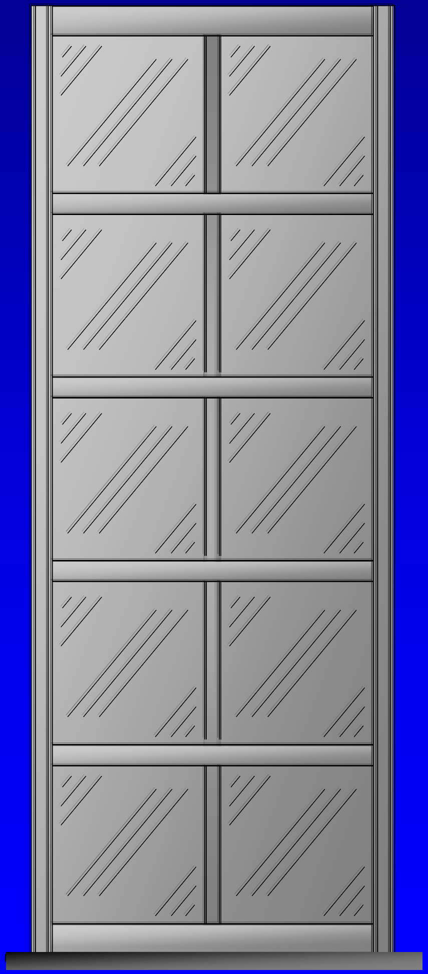


Courtesy of Carlos Ventura, University of British Columbia, Vancouver, Canada

Effect of beam flexibility in long steel plate shear walls

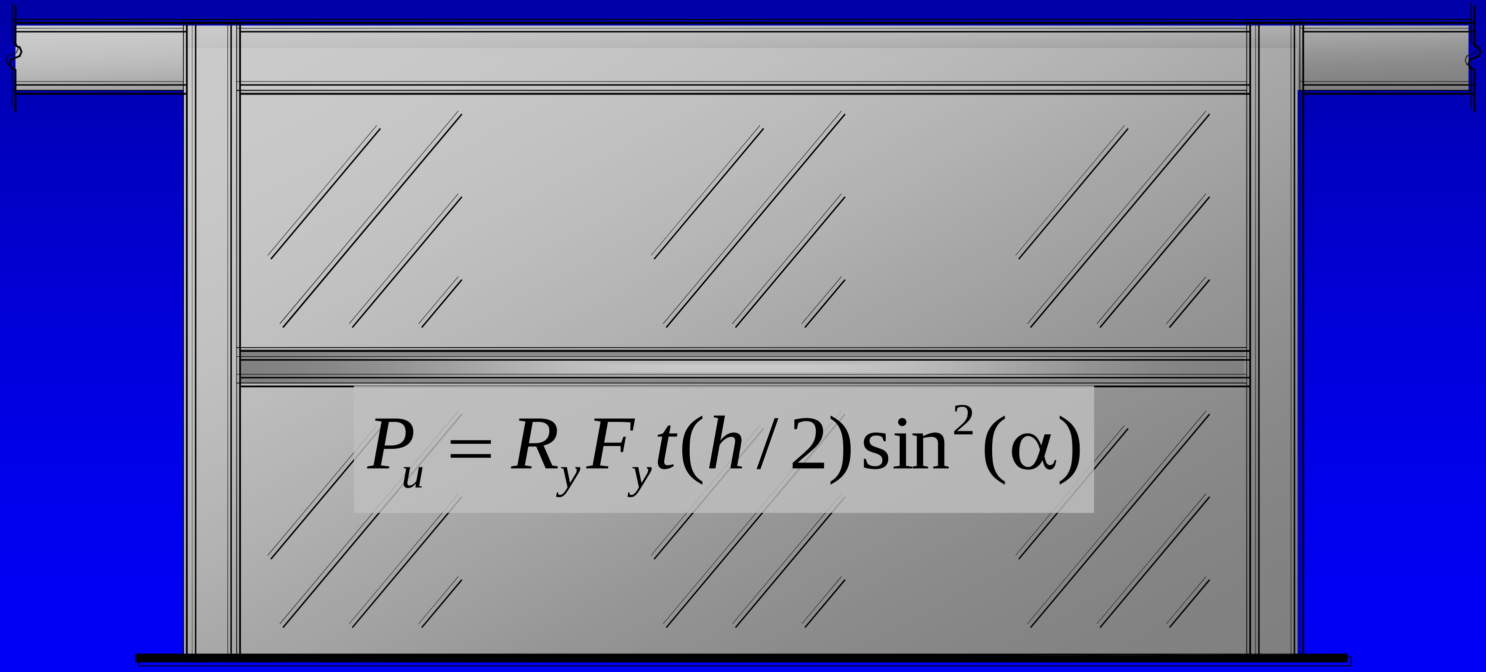


Vertical Struts



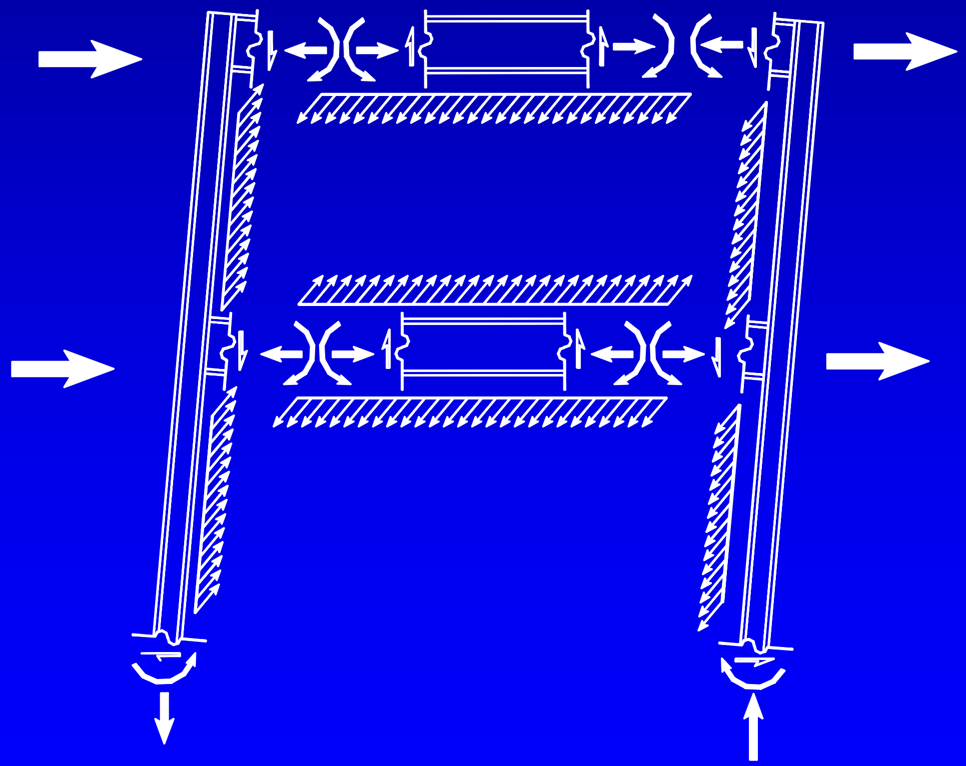
$$P_{u(i)} = \sum_{i}^{n} \tfrac{1}{2} w_{u(i)} L_{cf}$$

Horizontal Strut

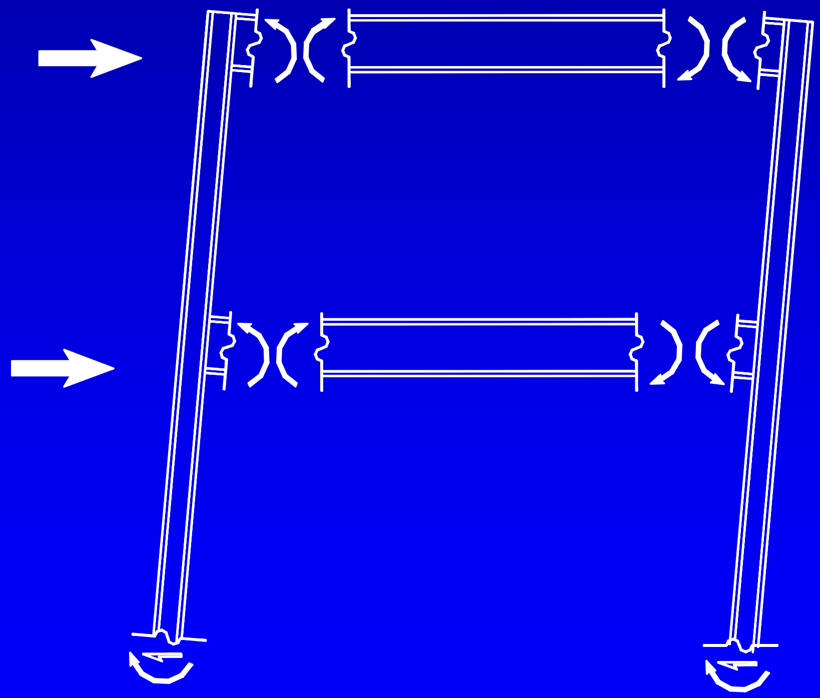


$$P_u = R_y F_y t(h/2)\sin^2(\alpha)$$

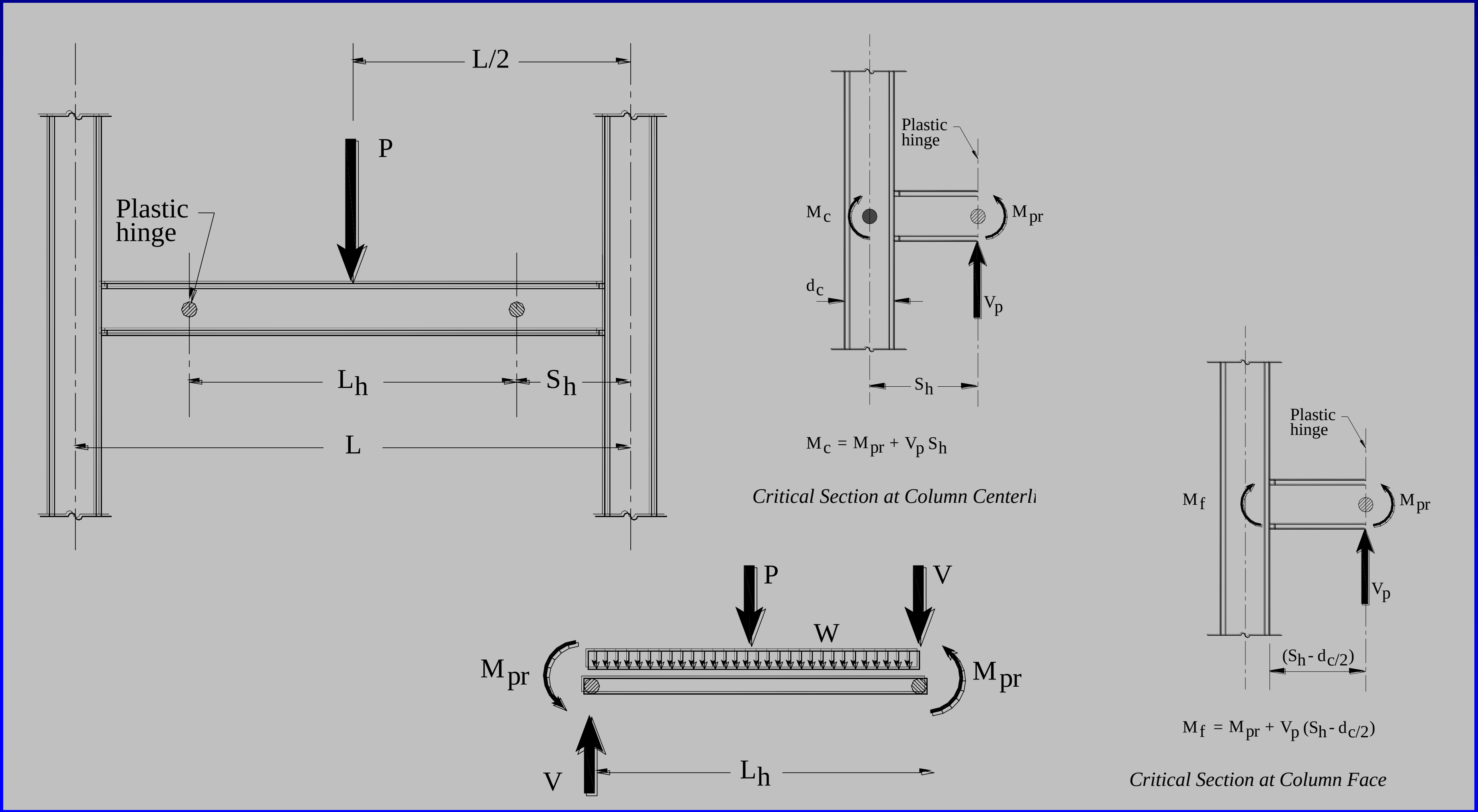
Flexural Forces



Frame Behavior



Frame Behavior



Tension-Strip Model

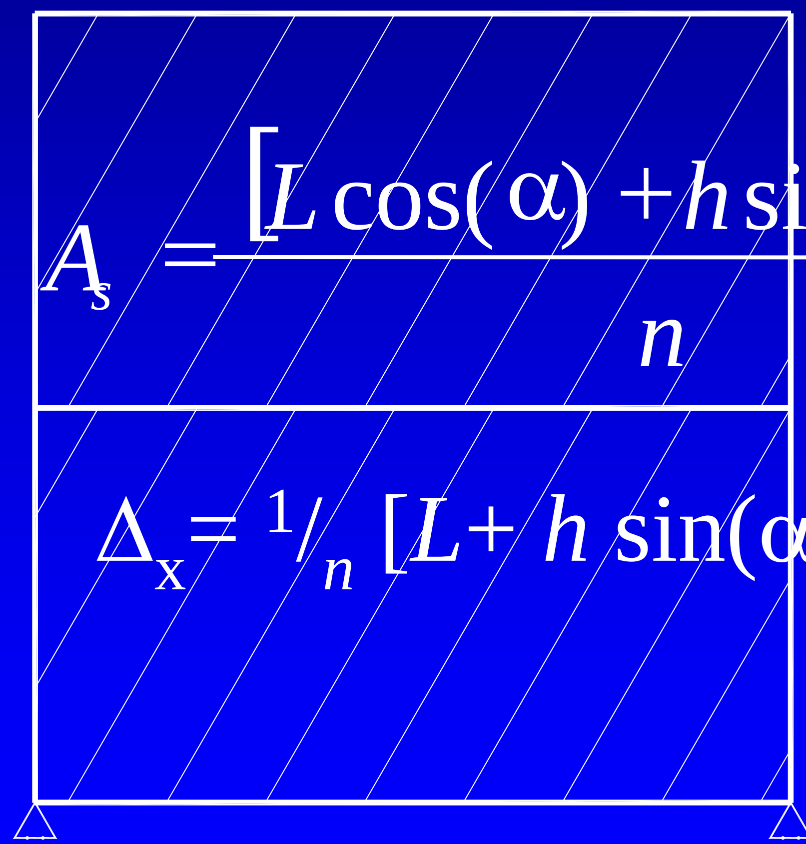
Calculate angle α

- Use average of all stories (unless there is a wide range)

Divide plate into sufficient number of strips (10)

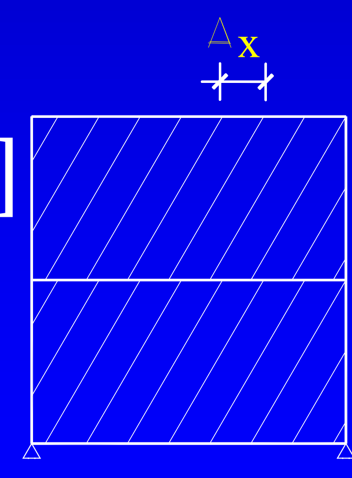
Calculate strip area

Locate intersection points

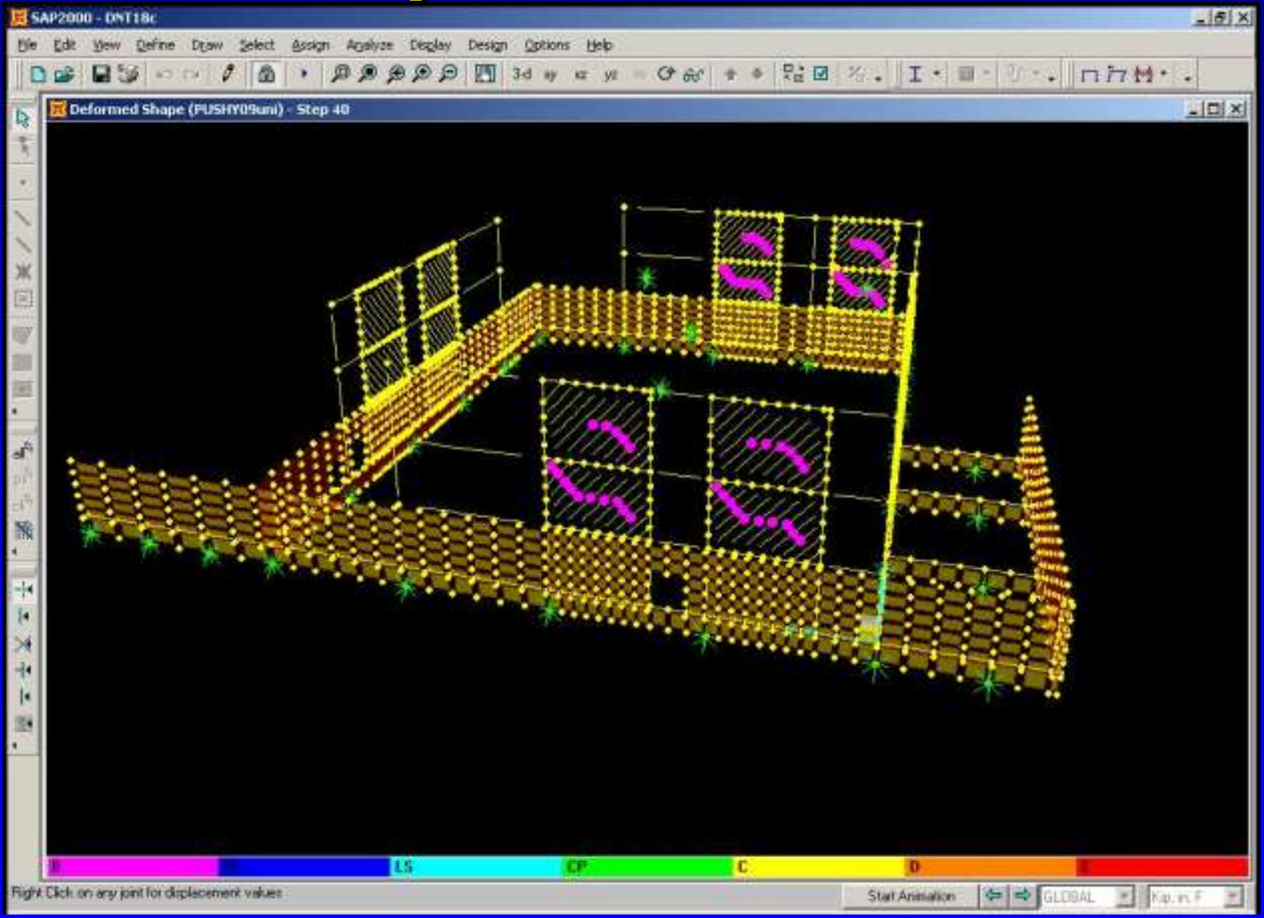


$$A_s = \frac{[L\cos(\alpha) + h\sin(\alpha)]t_w}{n}$$

$$\Delta_x = {}^1/_n\,[L + h\sin(\alpha)]$$

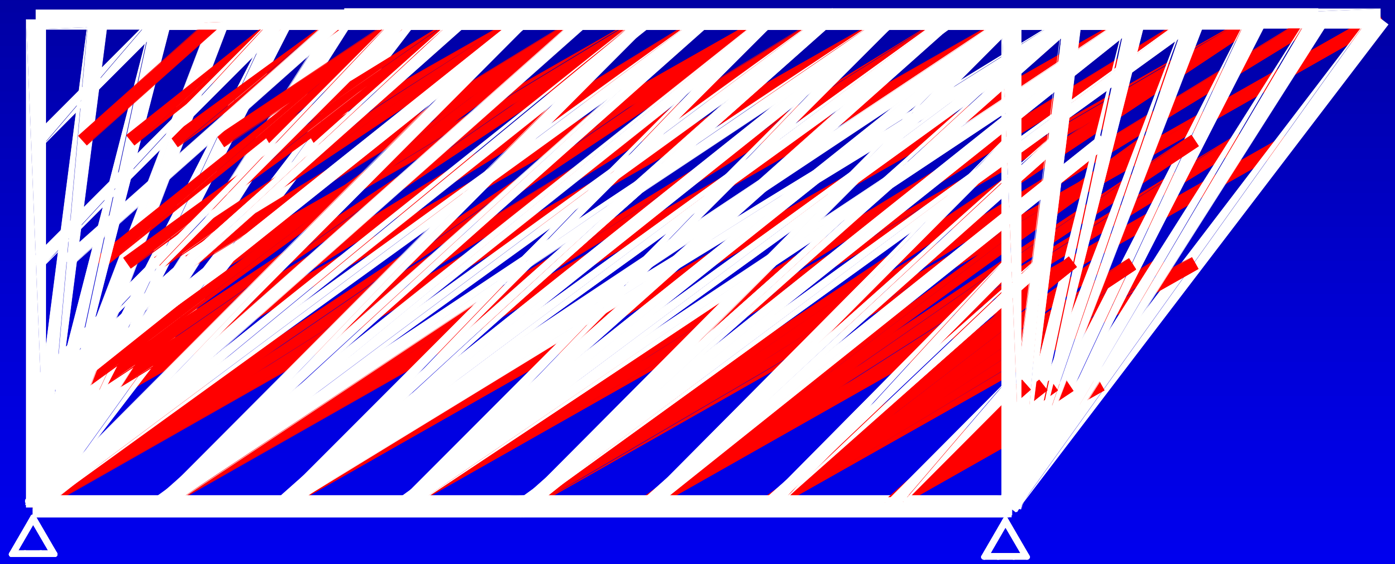


Strips models used in retrofit project using steel plate shear walls



Courtesy of Jay Love, Degenkolb Engineers

Progression of yielding across strips



Orthotropic Plate Model

Rotate local axes to α

Model diagonal tension behavior

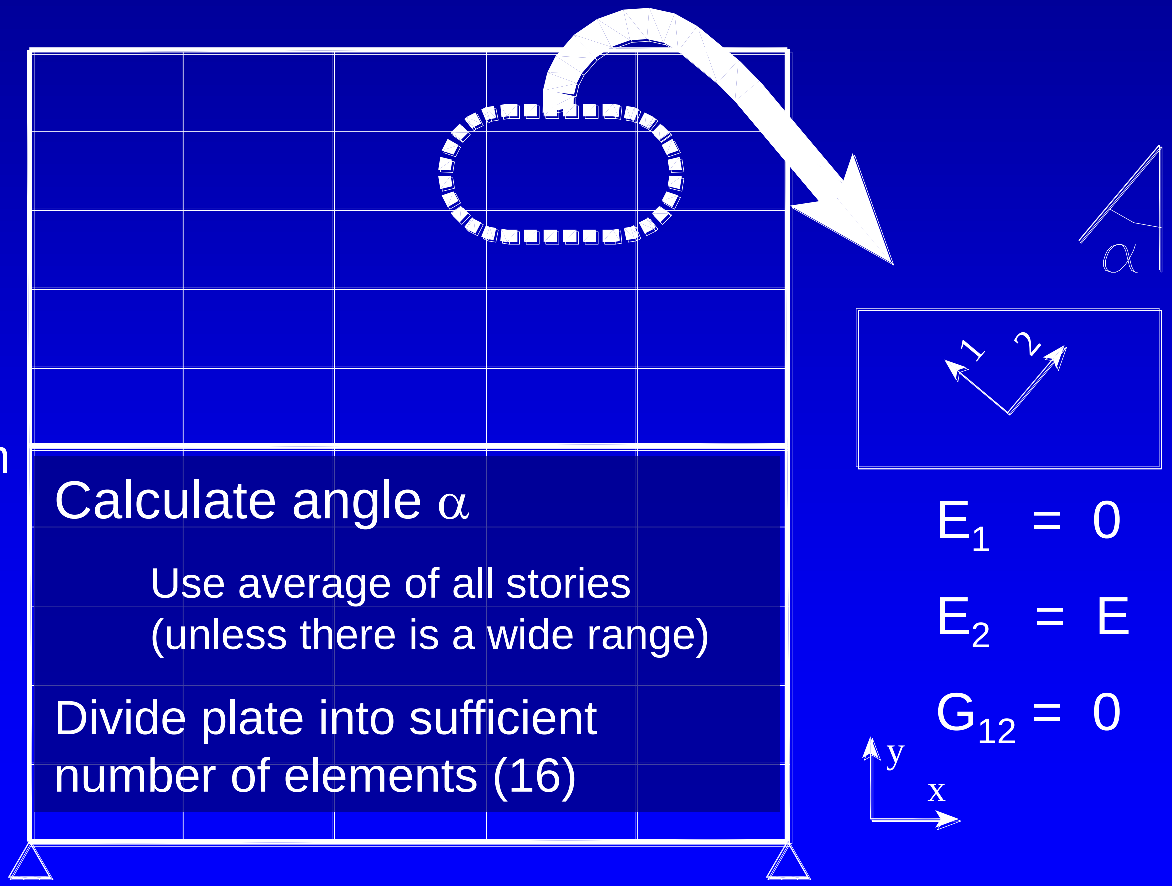
- Set modulus of elasticity in tension direction to 29,000 ksi

Remove plate diagonal compression resistance from model

- Set modulus of elasticity in compression direction to 0 ksi

Remove plate overturning resistance from model

- Set shear modulus to 0 ksi



Calculate angle α

- Use average of all stories (unless there is a wide range)

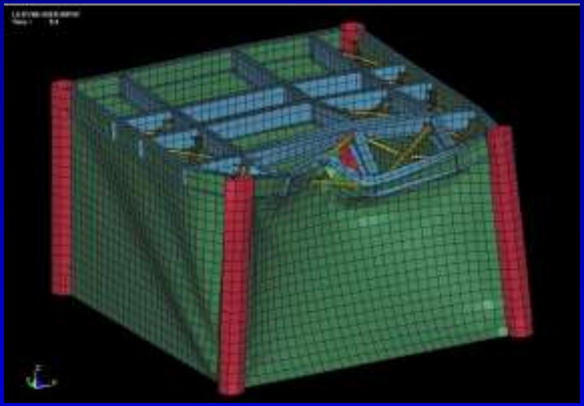
Divide plate into sufficient number of elements (16)

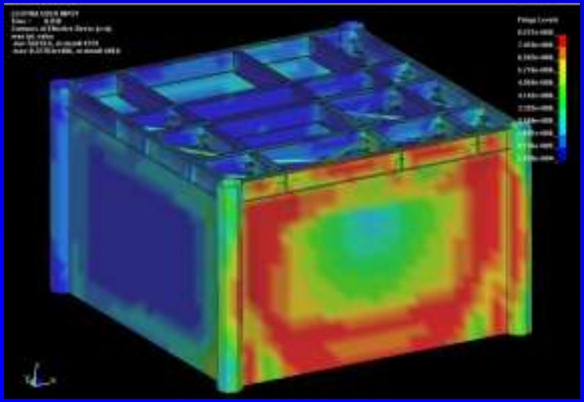
$E_1 = 0$

$E_2 = E$

$G_{12} = 0$

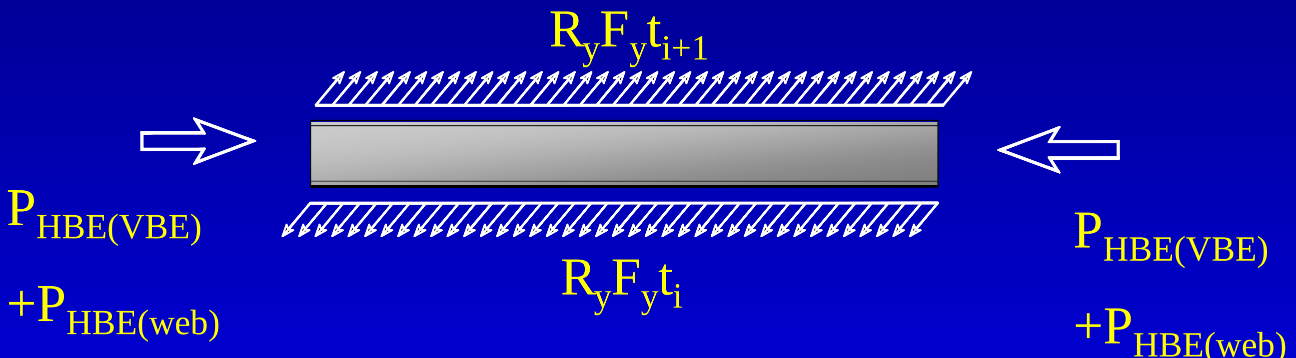
Proposed blast- and impact-resistant air traffic control towers using steel plate shear walls





Courtesy of John Pao, BPA Group, Structural Engineers, Bellevue, Washington

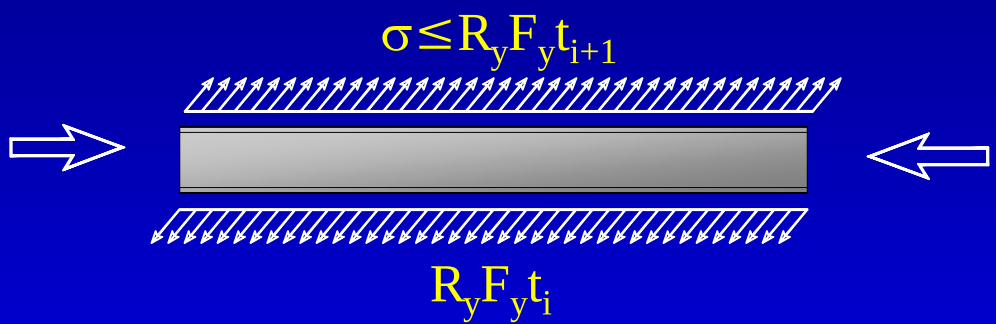
HBE Design Axial Forces



$$P_{HBE\,(web)} = \tfrac{1}{2} R_y F_y \left[t_i \sin(2\alpha_i) - t_{i+1} \sin(2\alpha_{i+1}) \right] L_{cf}$$

$$P_{HBE\,(VBE)} = \sum \tfrac{1}{2} h_c R_y F_y \sin^2(\alpha) t_w$$

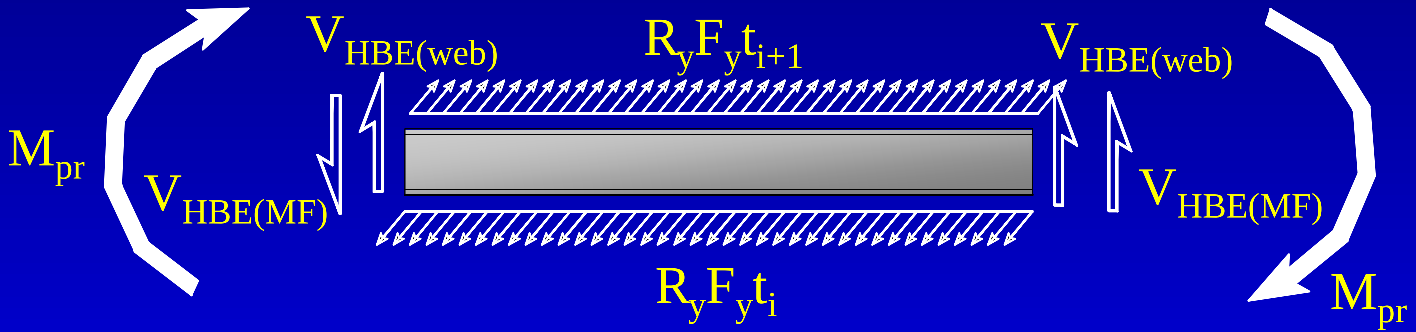
HBE to VBE Design Axial Forces



$$R_{u(horiz)} \geq P_{HBE(VBE)} + \Omega_o P_{collector}$$

$$R_{u(horiz)} \geq P_{HBE(VBE)} + P_{HBE(web)}$$

HBE Design Flexural Forces

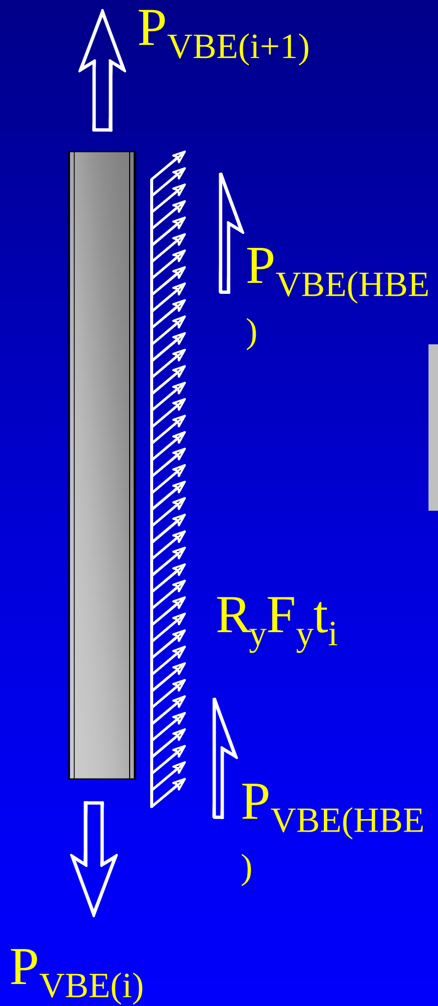


$$M_u = R_y F_y \left[t_i \cos^2(\alpha_i) - t_{i+1} \cos^2(\alpha_{i+1}) \right] L_h^2 \Big/ 8$$ (at midspan)

$$V_{HBE(web)} = R_y F_y \left[t_i \cos^2(\alpha_i) - t_{i+1} \cos^2(\alpha_{i+1}) \right] L_h \Big/ 2$$

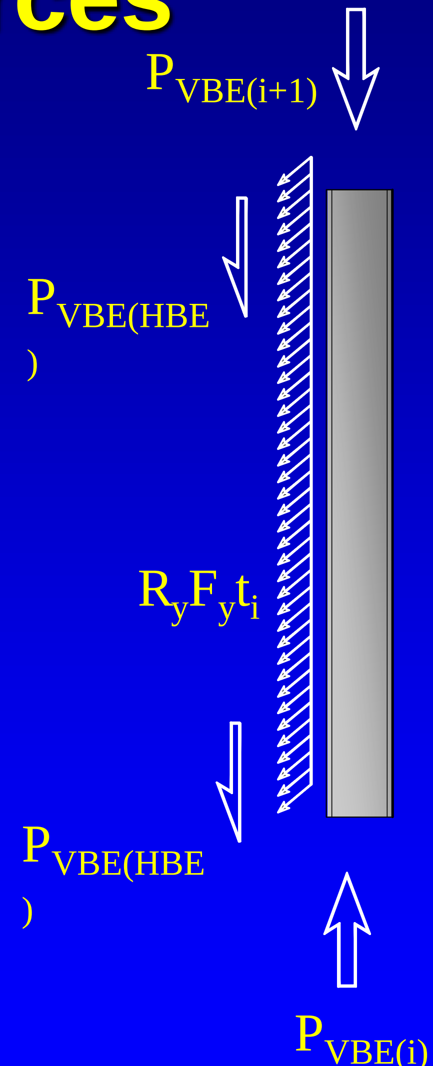
$$V_{HBE(MF)} = 2M_{pr} \Big/ L_h$$

VBE Design Axial Forces

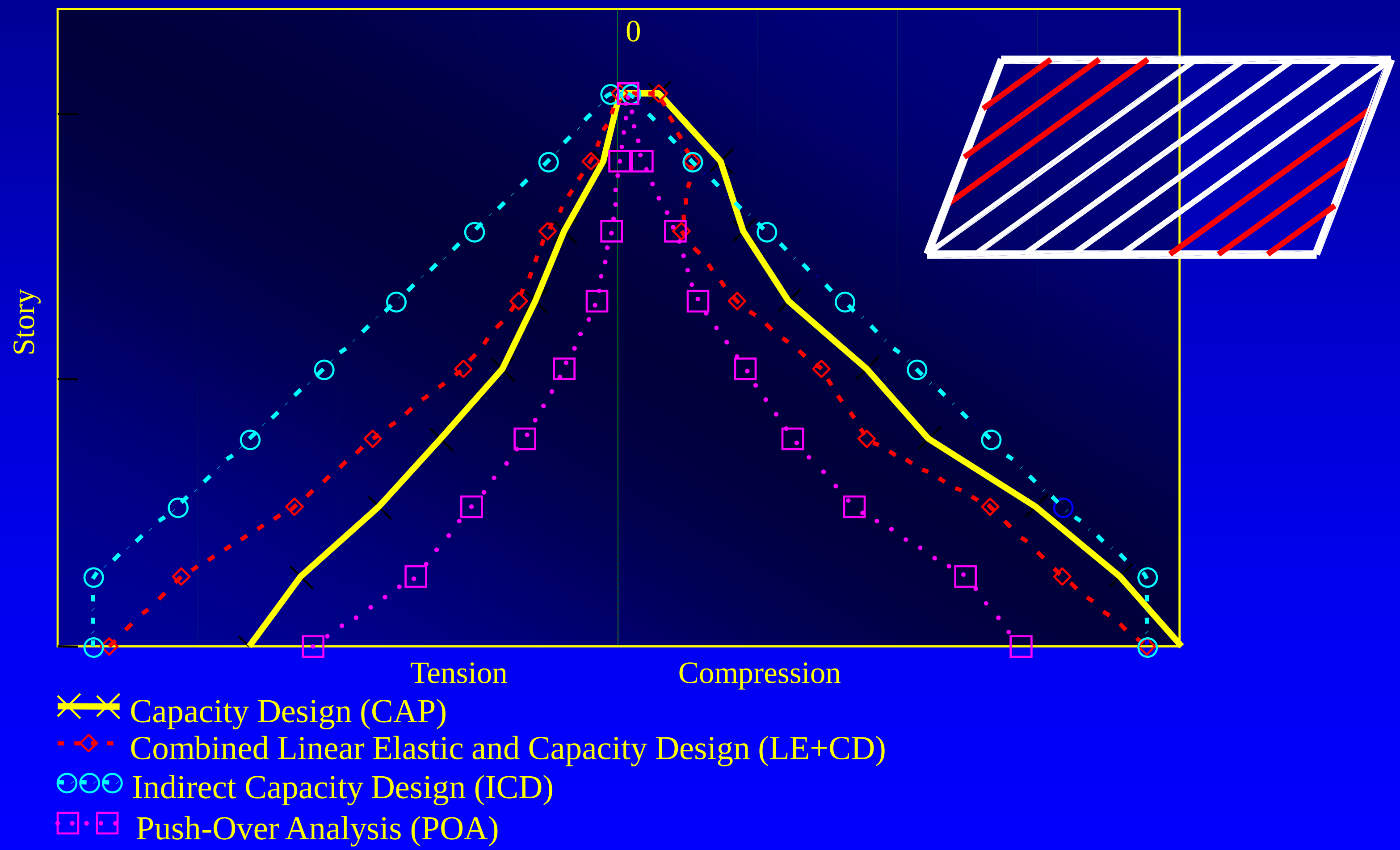


$$P_{VBE\,(web)} \le \sum_{i}^{n} {}^{1}\!/_{2}\, R_y F_y t_i \sin(2\alpha_i) h_c$$

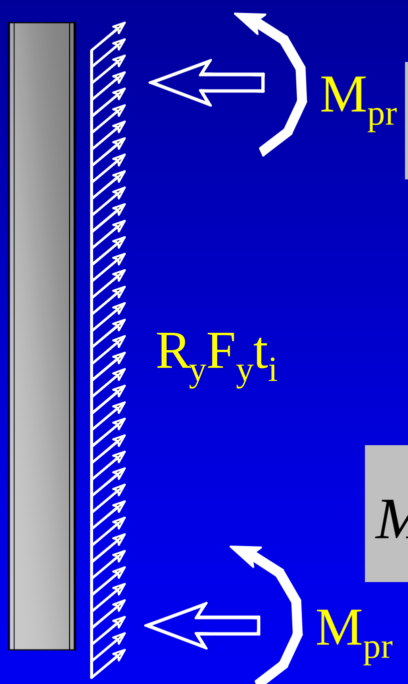
$$P_{VBE\,(HBE\,)} = V_{HBE\,(MF\,)} \pm V_{HBE\,(web)}$$



Approximations of column axial force



VBE Design Flexural Forces

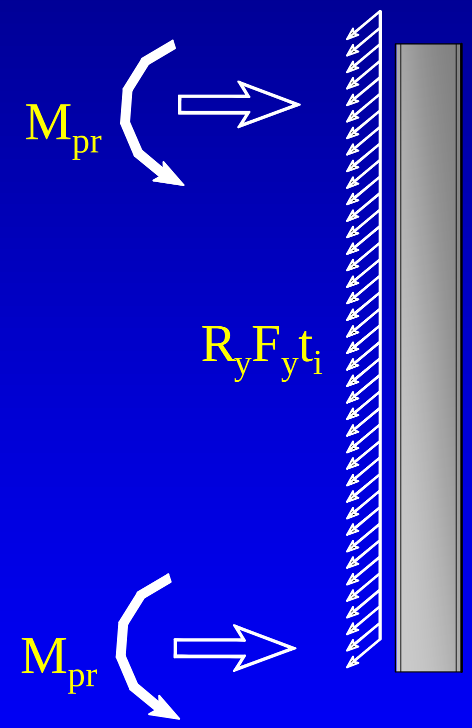


$$V_{VBE(web)} = \tfrac{1}{2} h_c R_y F_y \sin^2(\alpha) t_w$$

$$V_{VBE(MF)} = \frac{\sum \tfrac{1}{2} M^*_{pb}}{h_c}$$

$$M_{VBE(web)} = R_y F_y \sin^2(\alpha) t_w \frac{h_c^2}{12}$$

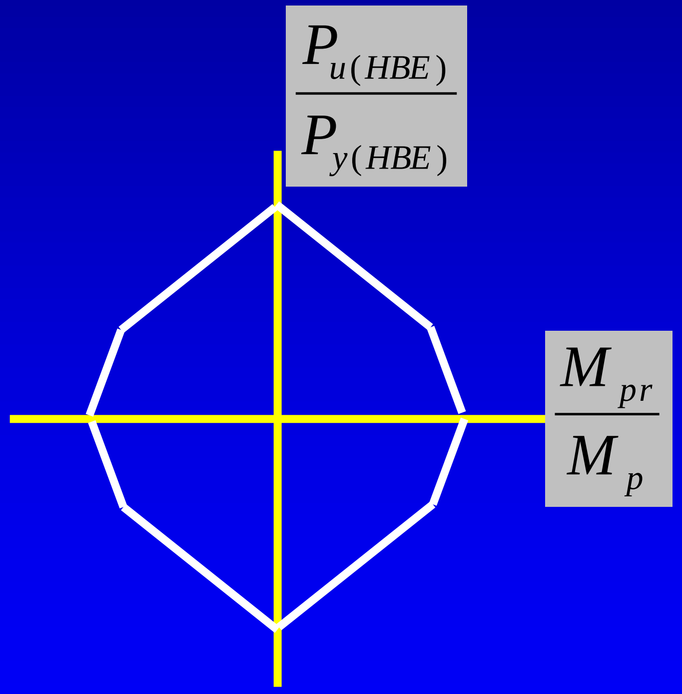
$$M_{VBE(MF)} \leq \frac{\tfrac{1}{2} \sum M^*_{pb}}{1.1 R_y}$$



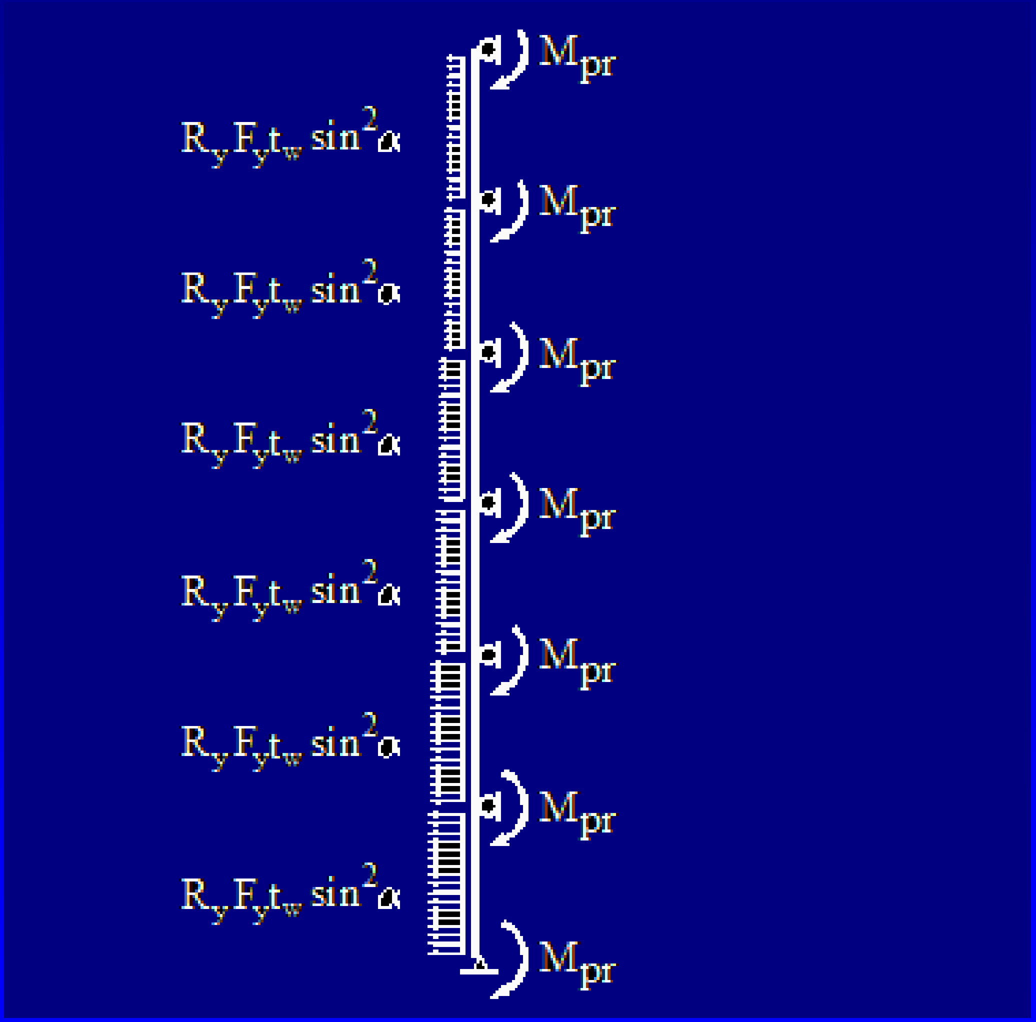
Flexural strength in the presence of high axial force.

$$M_{pr} \le \left[1.1 R_y F_y Z\right]\left[1 - \tfrac{1}{2}\frac{P_{u(HBE)}}{P_{y(HBE)}}\right]$$

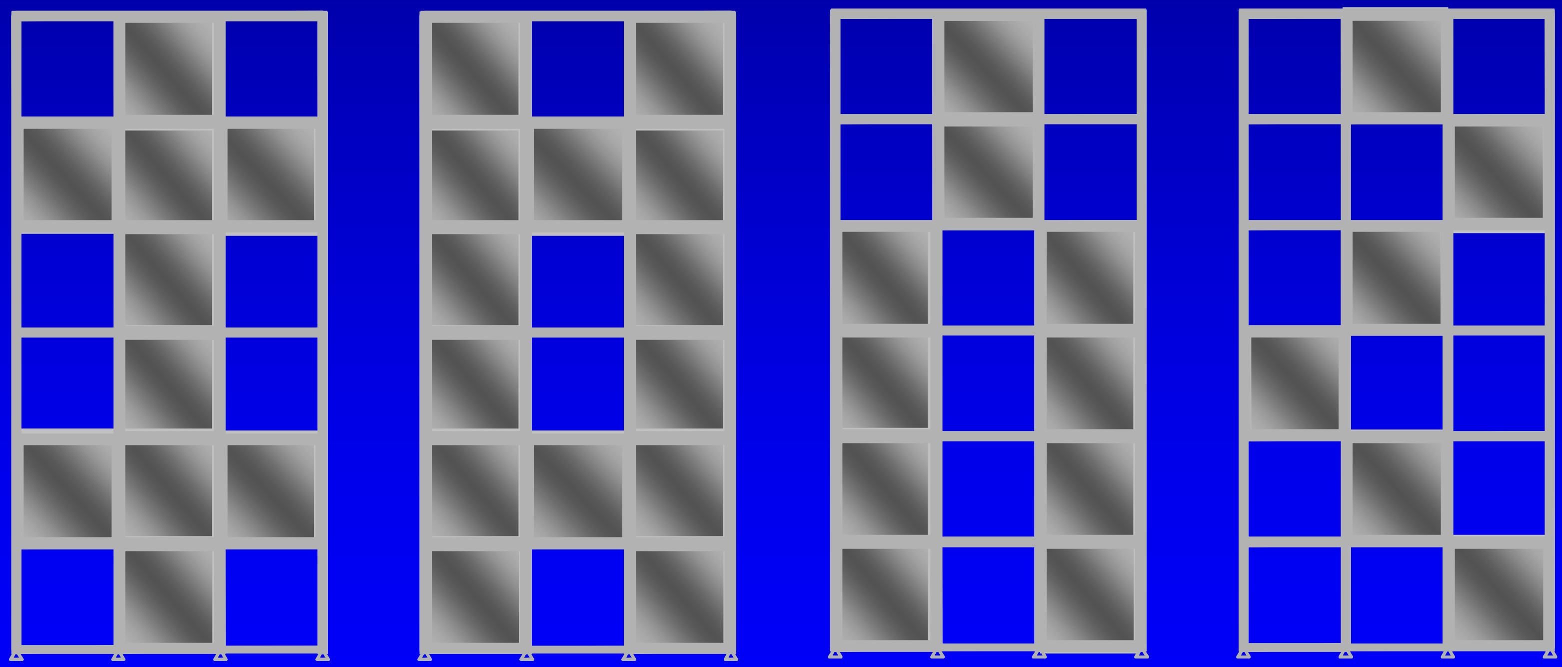
$$M_{pr} \le \tfrac{9}{8}\left[1.1 R_y F_y Z\right]\left[1 - \frac{P_{u(HBE)}}{P_{y(HBE)}}\right]$$



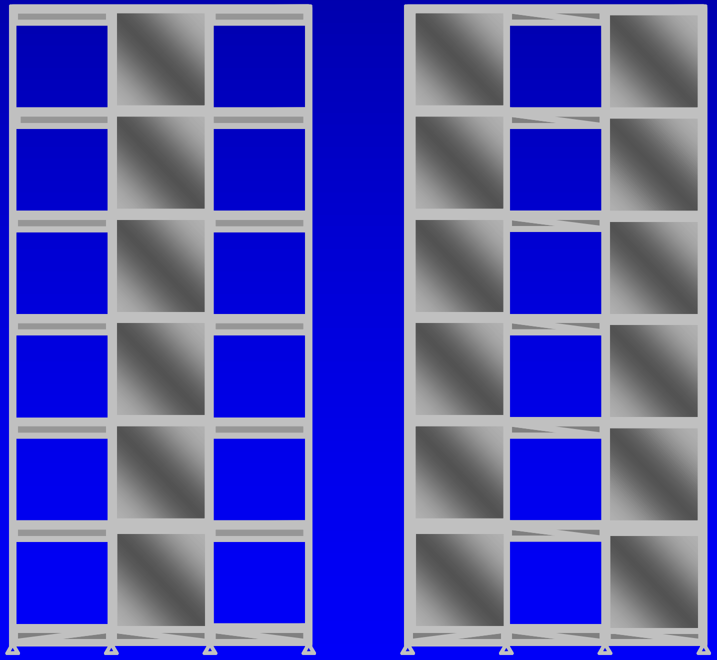
More exact column analysis



Irregular wall configurations to reduce overturning forces on columns

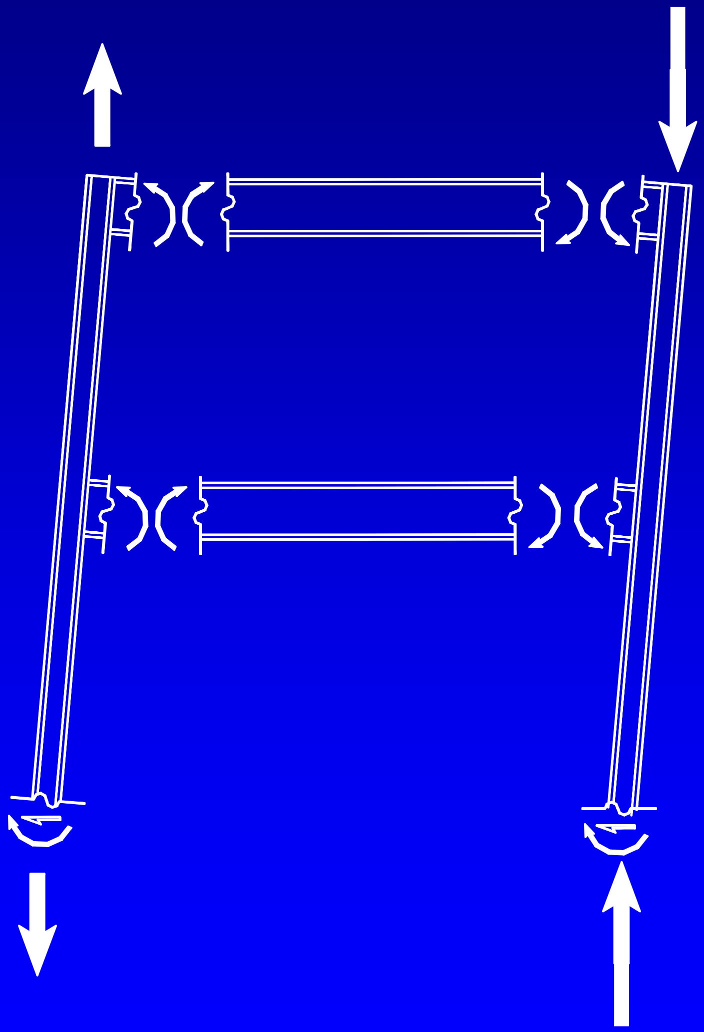


Outrigger and coupling beams used to reduce overturning forces on columns



Strong Column/Weak Beam

$$Z_c \geq \frac{1}{2} \frac{\sum M_{pb}^{*}}{\left[2F_{yc} - \frac{|P_{uC}| + |P_{uT}|}{A_g} \right]}$$



Section 17
Special Plate Shear Walls (SPSW)

17.1 Scope

17.2 Webs

17.3 Connections of Webs to Boundary Elements

17.4 Horizontal and Vertical Boundary Elements

17.1 Scope

Significant inelastic strain expected in web plates

HBE and VBE expected to be “essentially elastic” when subject to forces from fully yielded web plates

- Exception: plastic hinges expected at each end of HBE

Consult Building Code for:

- R
- Ω_o
- C_d

If Building Code does not have these, consult Appendix R

17.2 Webs

17.2a Shear Strength

17.2b Panel Aspect Ratio

17.2c Openings in Webs

17.2a Shear Strength

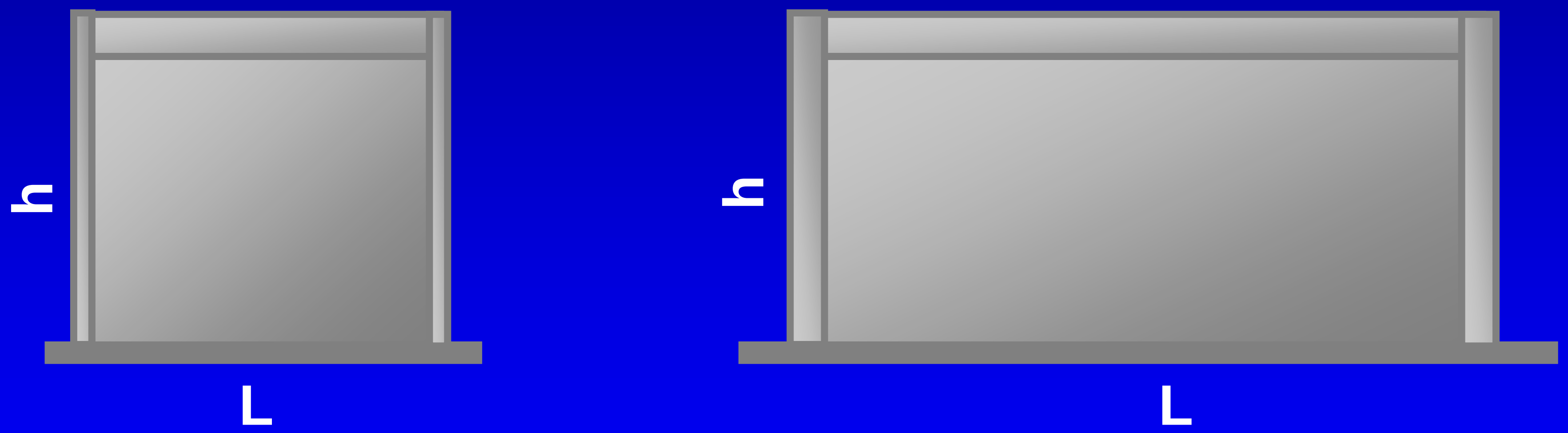
$V_n = 0.42\, F_y\, t_w\, L_{cf} \sin(2\alpha)$

$\phi = 0.9 \qquad \Omega = 1.67$

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w L}{2 A_c}}{1 + t_w h \left[\dfrac{1}{A_b} + \dfrac{h^3}{360\, I_c L} \right]}$$

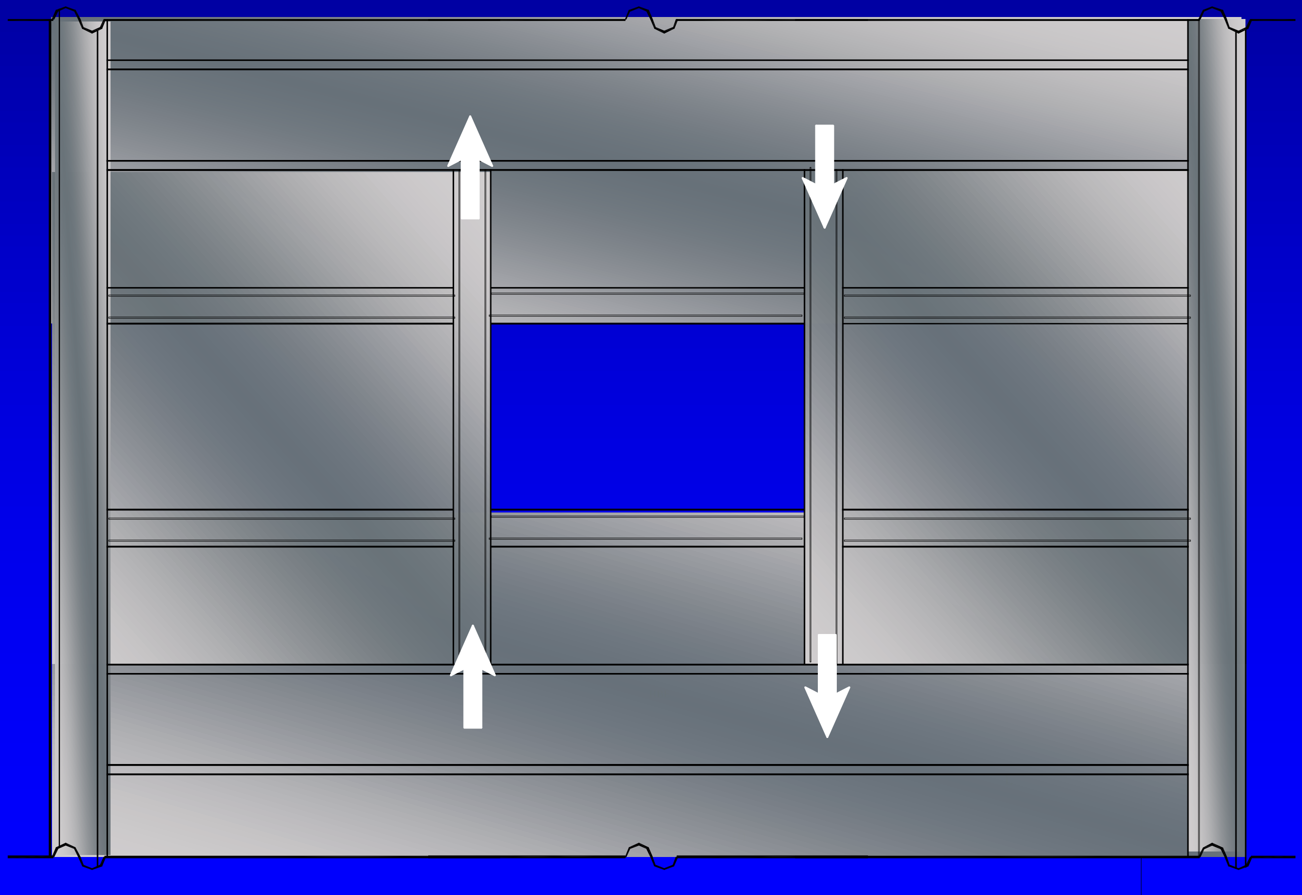
17.2b Panel Aspect Ratio

$0.8 \leq {}^{L}/_{h} \leq 2.5$



17.2c Openings in Webs

Local boundary elements around the opening are required



17.3 Connections of Webs to Boundary Elements

Must develop expected web strength

Connection to VBE

Tension: $R_y F_y t_w h_c \sin^2(\alpha)$

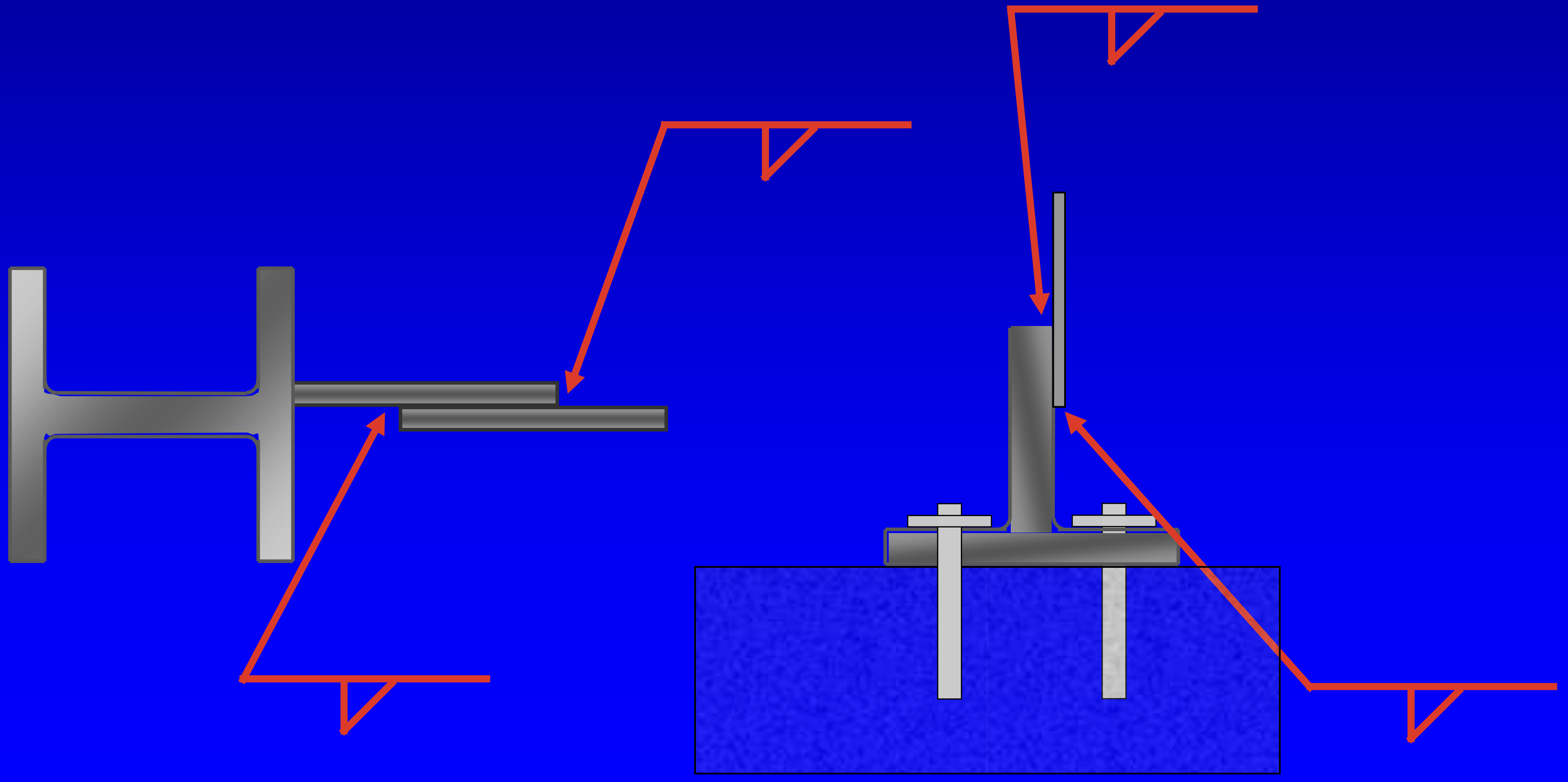
Shear: ½ $R_y F_y t_w h_c \sin(2\alpha)$

Connection to HBE

Tension: $R_y F_y t_w L_{cf} \cos^2(\alpha)$

Shear: ½ $R_y F_y t_w L_{cf} \sin(2\alpha)$

Typical connections of web plates



17.4 Horizontal and Vertical Boundary Elements

17.4a Required Strength

17.4b HBE-to-VBE Connections

17.4c Width-Thickness Limitations

17.4d Lateral Bracing

17.4e VBE splices

17.4f Panel Zones

17.4g Stiffness of Vertical Boundary Elements

17.4a Required Strength

Must develop expected web strength

VBE

Transverse load: $R_y\, F_y\, t_w \sin^2(\alpha)$

Distributed axial load:

$\tfrac{1}{2}\, R_y\, F_y\, t_w\, h_c \sin(2\alpha)$ [+ force from above]

HBE

Transverse load:

$R_y\, F_y\, [t_{w(i)} — t_{w(i+1)}]\, L_{cf} \cos^2(\alpha)$

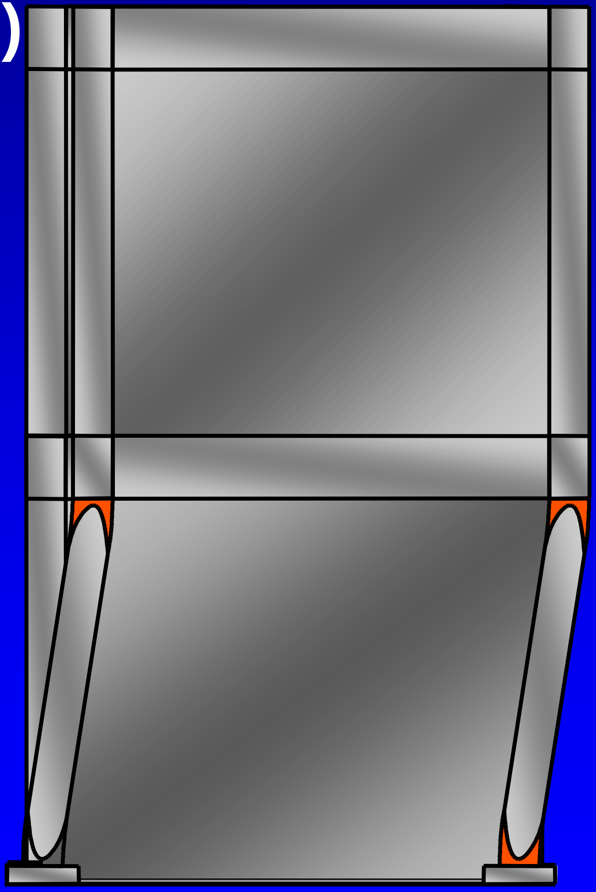
Distributed axial load:

$\tfrac{1}{2}\, R_y\, F_y\, [t_{w(i)} — t_{w(i+1)}]\, L_{cf} \sin(2\alpha)$

17.4a Required Strength

HBE to VBE connection must satisfy SMF Section 9.6

(Strong-column/weak-beam)



Purpose of strong column - weak girder requirement:

Prevent Soft Story Collapse

AISC Seismic Provisions - SMF

9.6 Column-Beam Moment Ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0 \qquad \text{Eqn. (9-3)}$$

9.6 Column-Beam Moment Ratio

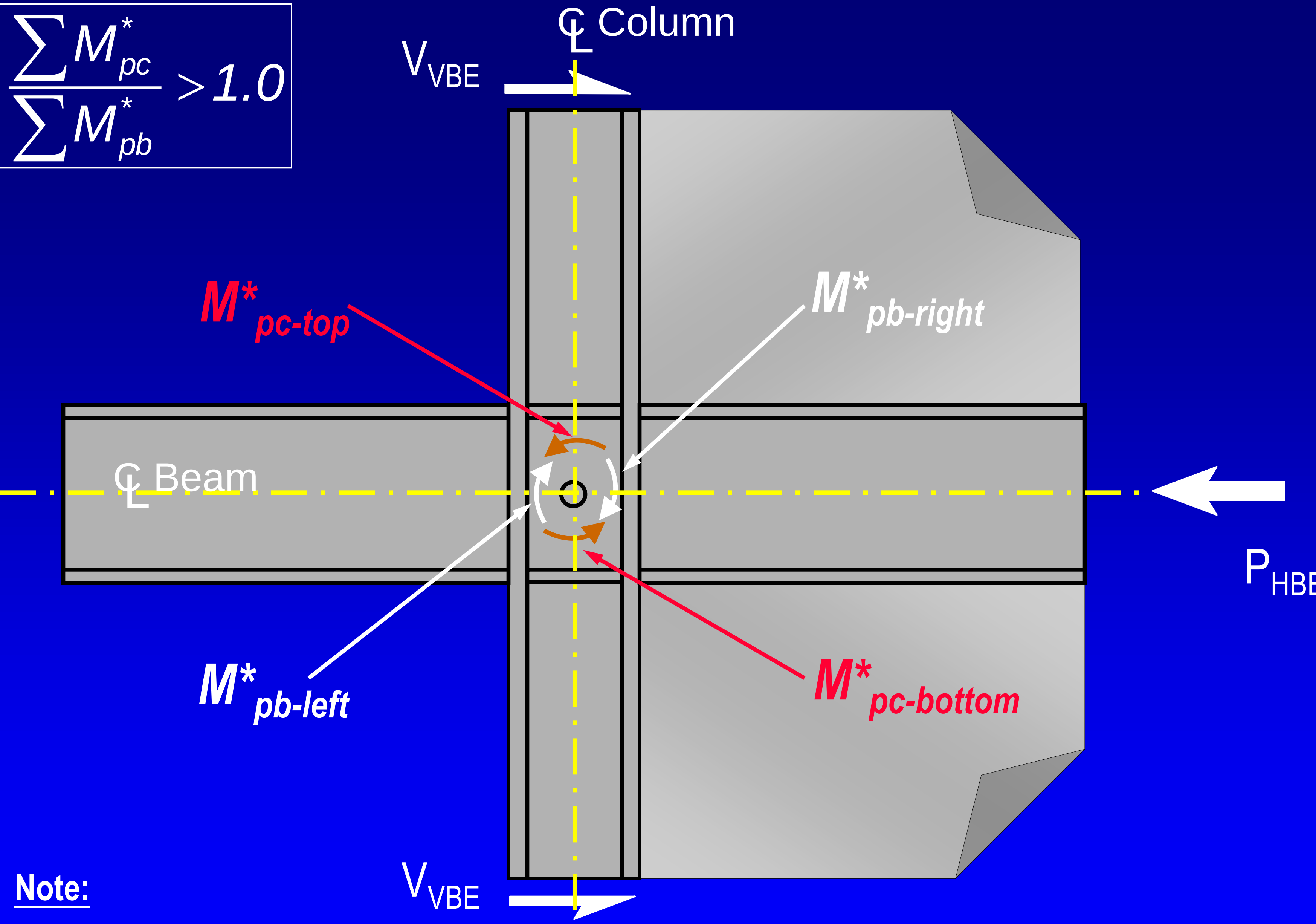
$$\boxed{\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0}$$

$\sum M^*_{pc} =$ the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\sum M^*_{pc}$ is determined by summing the projections of the nominal flexural strengths of the columns above and below the joint to the beam centerline with a reduction for the axial force in the column.

It is permitted to take $\sum M^*_{pc} = \sum Z_c\,(F_{yc} - P_{uc}/A_g)$

$\sum M^*_{pb} =$ the sum of the moments in the beams at the intersection of the beam and column centerlines. $\sum M^*_{pb}$ is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. $\sum M^*_{pb} = \sum [Z_b\,(R_y F_{yb} - P_{ub}/A_g) + V_p(s_h + d_{col}/2)]$

$$\frac{\sum M_{pc}^{*}}{\sum M_{pb}^{*}} > 1.0$$

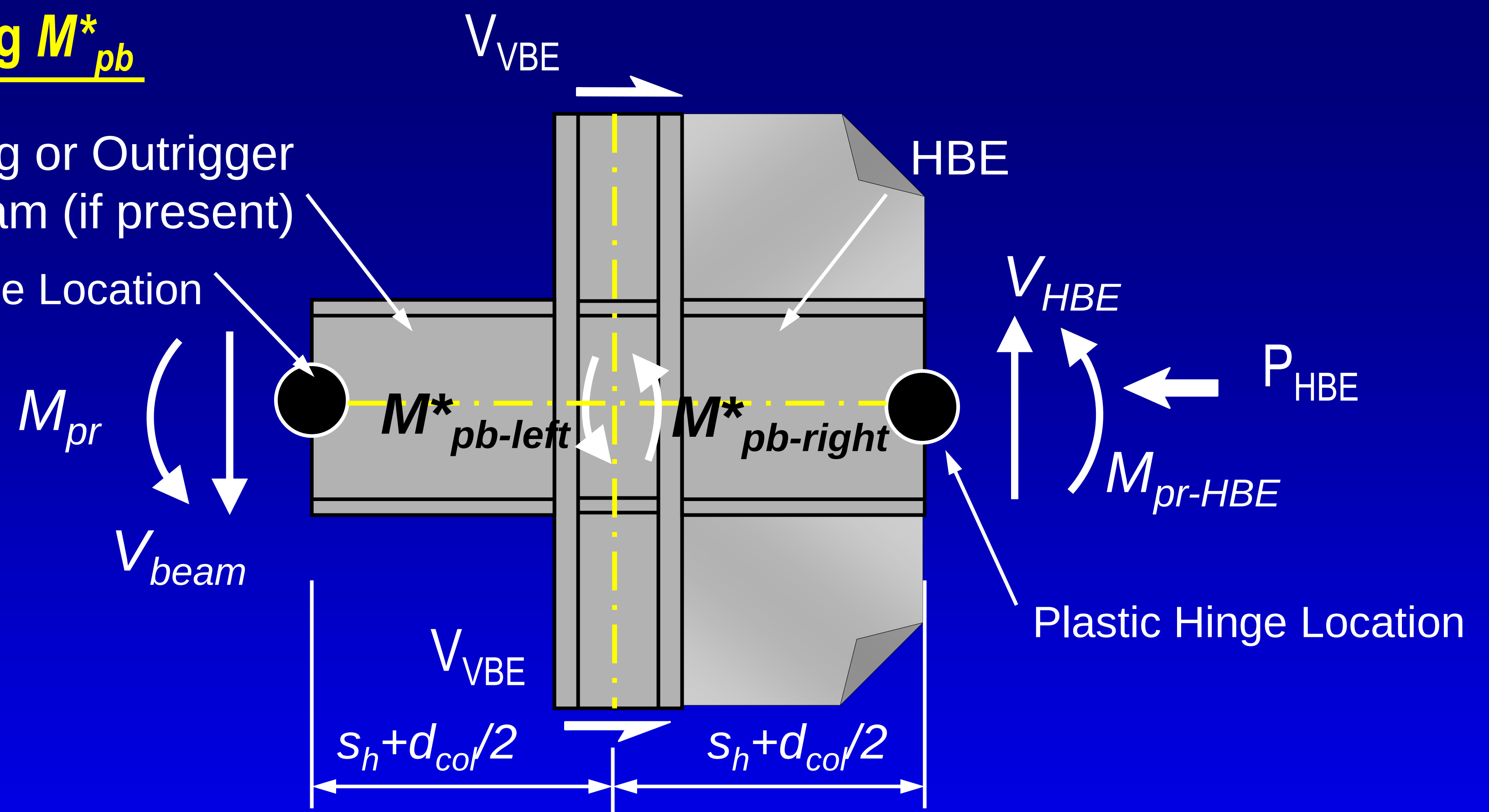


Note:

M^*_{pc} is based on minimum specified yield stress of column

M^*_{pb} is based on expected yield stress of beam and includes allowance for strain hardening

Computing M^*_{pb}



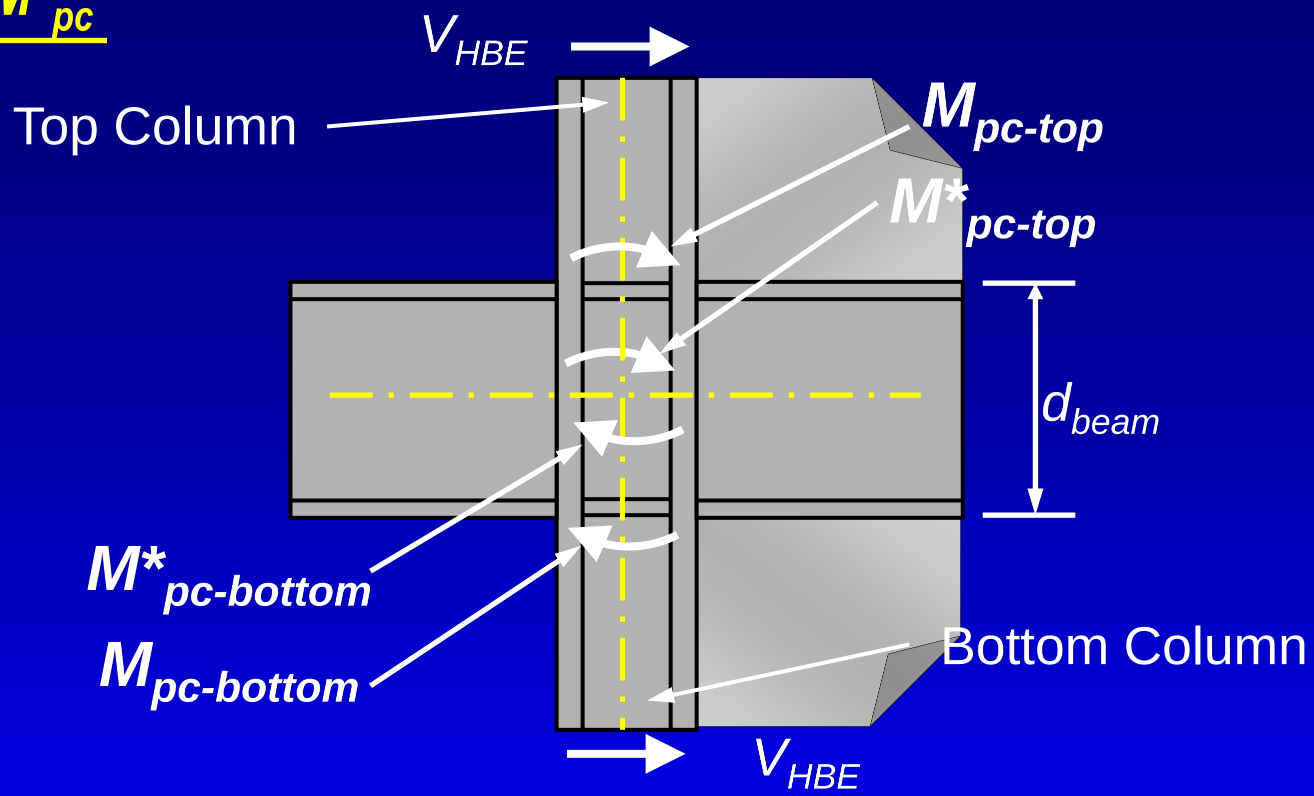
M_{pr} = expected moment at plastic hinge

V_{HBE} = beam shear (includes effect of web plate)

s_h = distance from face of column to beam plastic hinge location (specified in ANSI/AISC 358)

$$M^*_{pb} = Z_b\,(R_yF_{yb} - P_{HBE}/A_g) + V_{HBE}(s_h + d_{col}\,/2)$$

Computing M^*_{pc}



M_{pc} = nominal plastic moment capacity of column, reduced for presence of axial force; can take $M_{pc} = Z_c\,(F_{yc} - P_{uc} / A_g)$ [or use more exact moment-axial force interaction equations for a fully plastic cross-section]

V_{HBE} = column shear

$$\boxed{M^*_{pc} = M_{pc} + V_{HBE\,(top)}\,(d_{beam}\,/2\,)}$$

17.4b HBE-to-VBE Connections

OMF beam-column connection per 11.2 (or better)

Minimum shear:

$$V_u = V_{HBE(web)} + V_{HBE(MF)}$$

$$V_{HBE(web)} = R_y\, F_y\, [t_{w(i)} - t_{w(i+1)}]\, L_{cf} \cos^2(\alpha)\ L_h/2$$

$$V_{HBE(MF)} = 2\ M_{pr}/L_h$$

$$M_{pr} = 1.1\ R_y\ Z_b\ (F_{yb} - P_{HBE}/A_g)$$

17.4c Width-Thickness Limitations

Flanges

$$\frac{b_f}{2t_f} \leq 0.30\sqrt{\frac{E_s}{F_y}}$$

Web

$$\frac{P_u}{\phi P_y} \leq 0.125 \qquad \frac{h}{t_w} \leq 3.14\sqrt{\frac{E_s}{F_y}}\left[1-1.54\frac{P_u}{\phi P_y}\right]$$

$$\frac{P_u}{\phi P_y} > 0.125 \qquad \frac{h}{t_w} \leq 1.12\sqrt{\frac{E_s}{F_y}}\left[2.33-\frac{P_u}{\phi P_y}\right] > 1.49\sqrt{\frac{E_s}{F_y}}$$

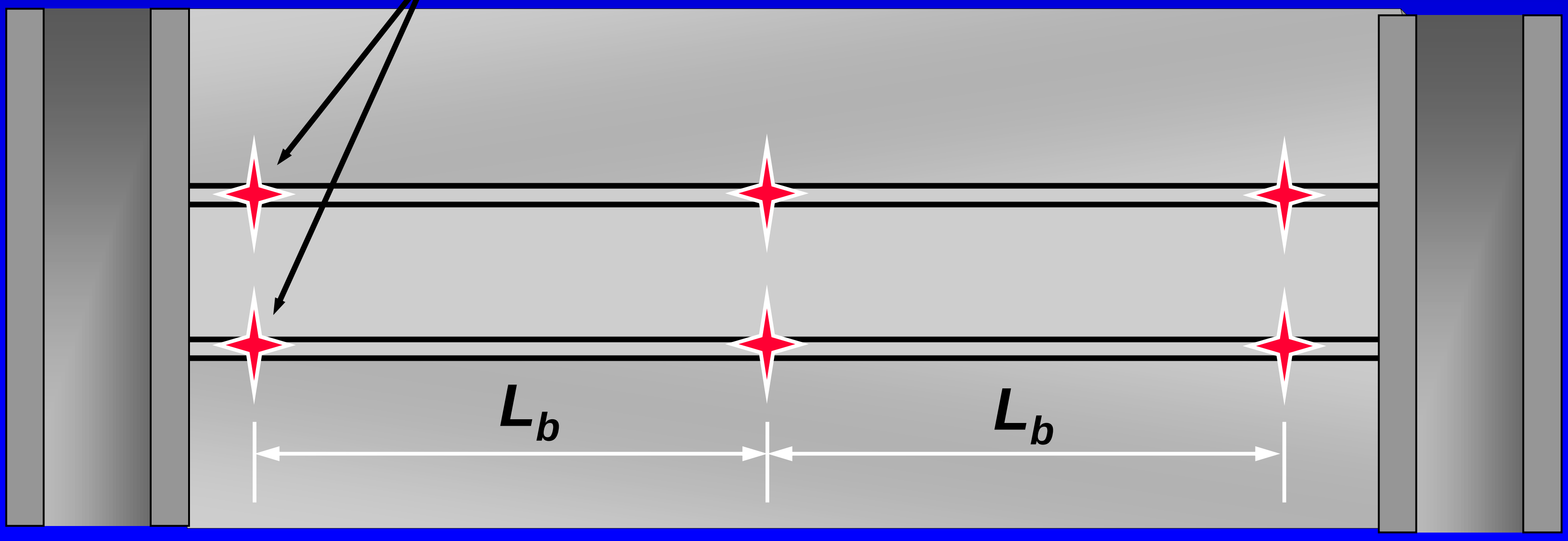
17.4d Lateral Bracing

Lateral torsional buckling controlled by: $\dfrac{L_b}{r_y}$

L_b = distance between beam lateral braces

r_y = weak axis radius of gyration

Beam lateral braces (top & bottom flanges)



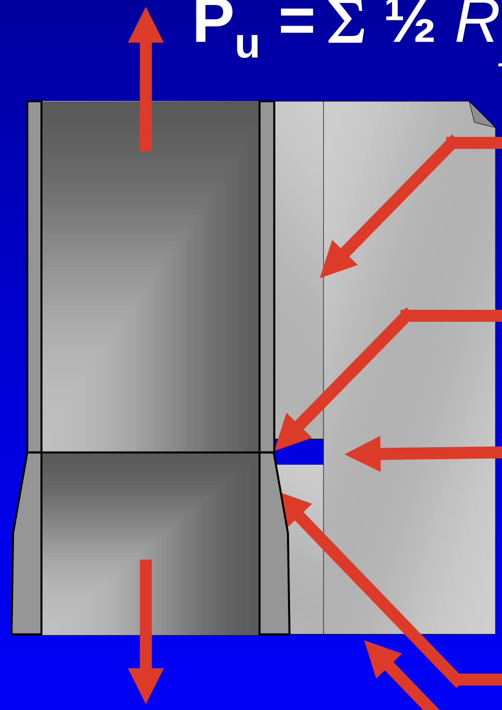
17.4d Lateral Bracing

Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = 0.086\, r_y\, E / F_y$

$$L_b \le 0.086\left(\frac{E}{F_y}\right) r_y \qquad \left(= 50 r_y \ \ for\ \ F_y = 50\, ksi\right)$$

17.4e VBE splices

$\mathbf{P_u} = \Sigma\ ½\ R_y\ F_y\ t_w\ h_c\ \sin(2\alpha)$ — ΣHBE reactions



Fish plate

$\mathbf{R_u = ½\ R_y F_y A_f}$

Small gap

For PJP, use $\mathbf{R_u \geq 2\ \Omega_o\ Q_E}$

Transition per AWS D1.1 (2.7.1)

Web Plate

17.4f Panel Zones

Comply with SMF requirement 9.3

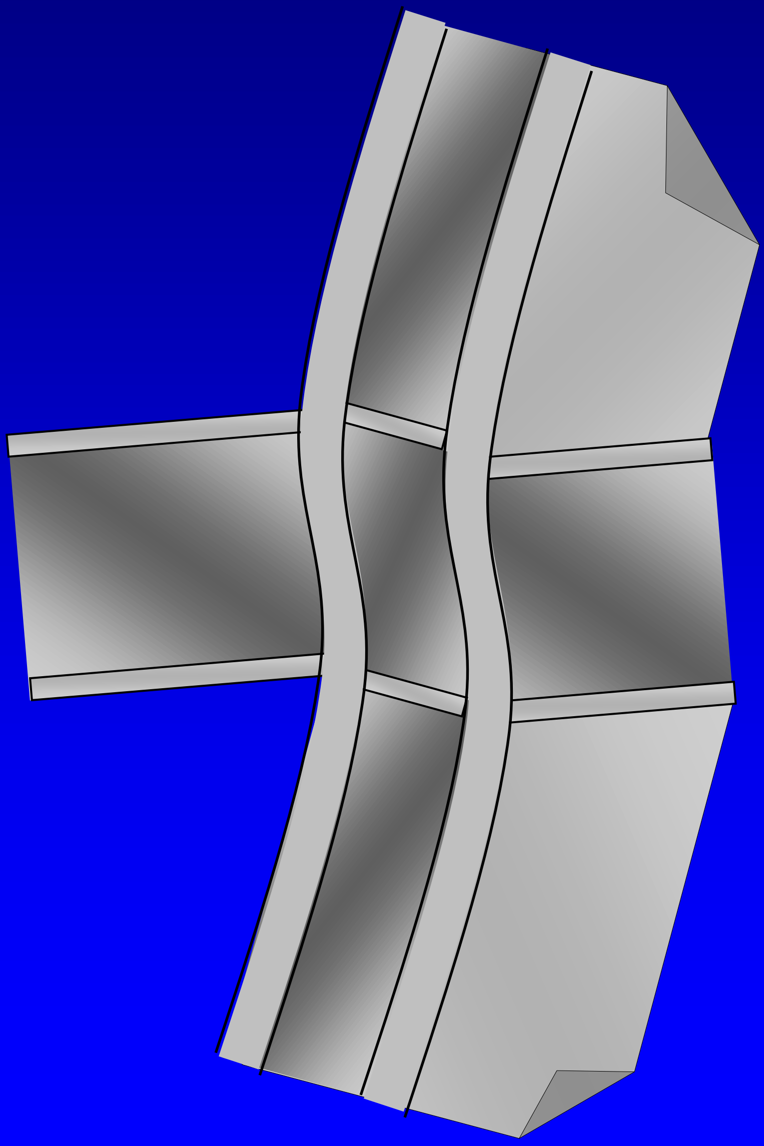
AISC Seismic Provisions - SMF

9.3 Panel Zone of Beam-to-Column Connections

9.3a Shear Strength

9.3b Panel Zone Thickness

9.3c Panel Zone Doubler Plates



AISC Seismic Provisions - SMF - Panel Zone Requirements

9.3a Shear Strength

The minimum required shear strength, R_u , of the panel zone shall be taken as the shear generated in the panel zone when plastic hinges form in the beams.

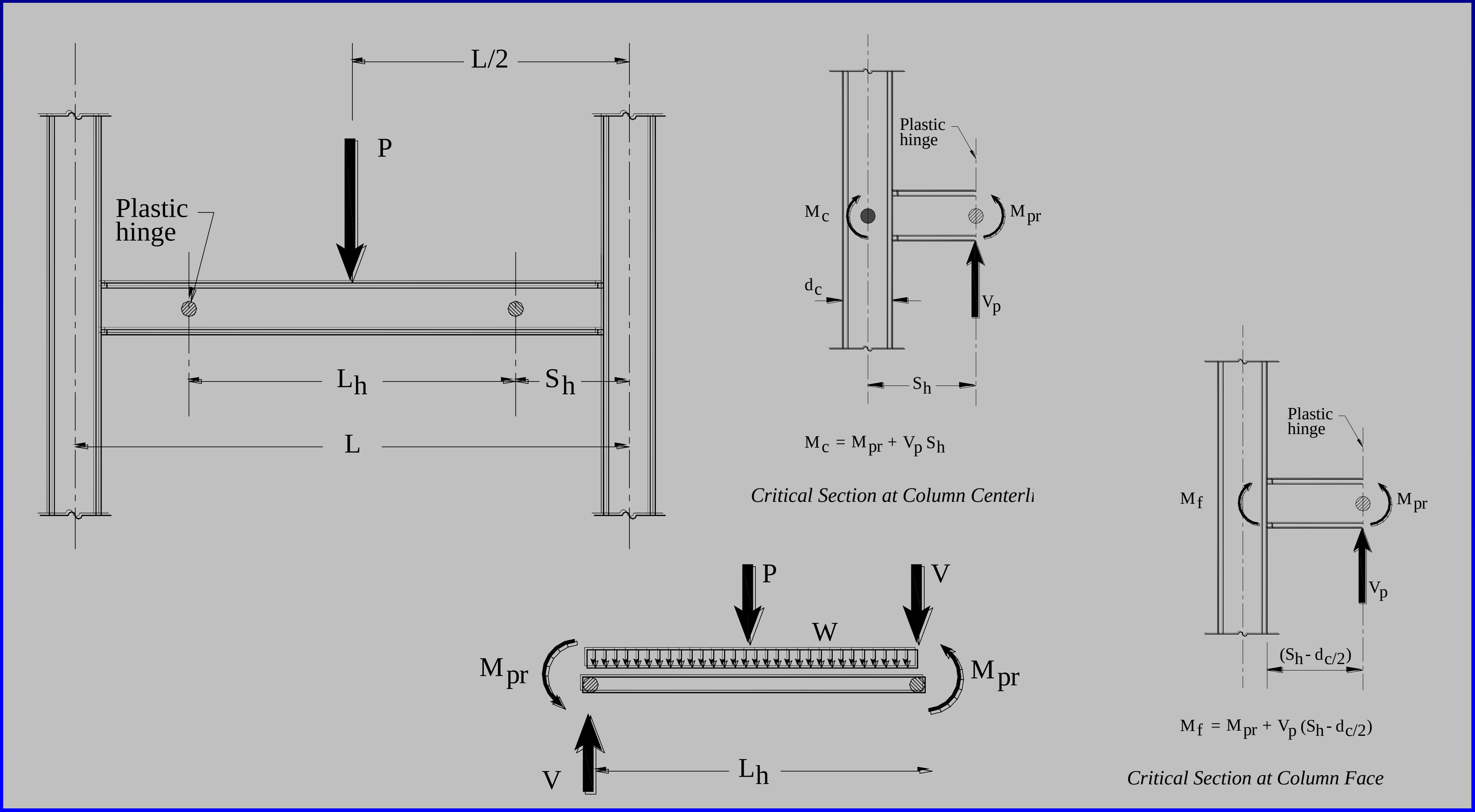
To compute panel zone shear.....

Determine moment at beam plastic hinge locations

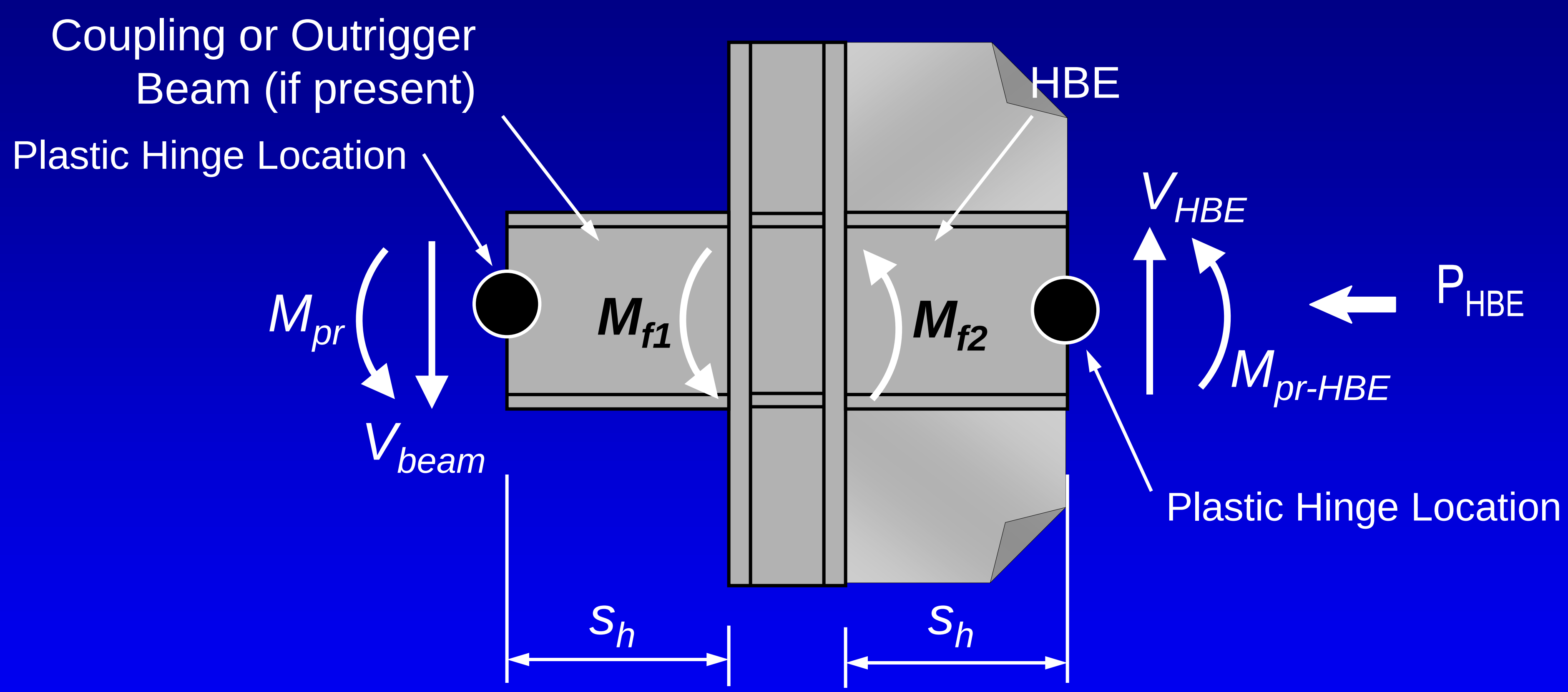
Project moment at plastic hinge locations to the face of the column (based on beam moment gradient, including the effects of web-plate induced HBE shear)

Compute panel zone shear force.

Projecting Moment to Column Face



Panel Zone Shear Strength (cont)

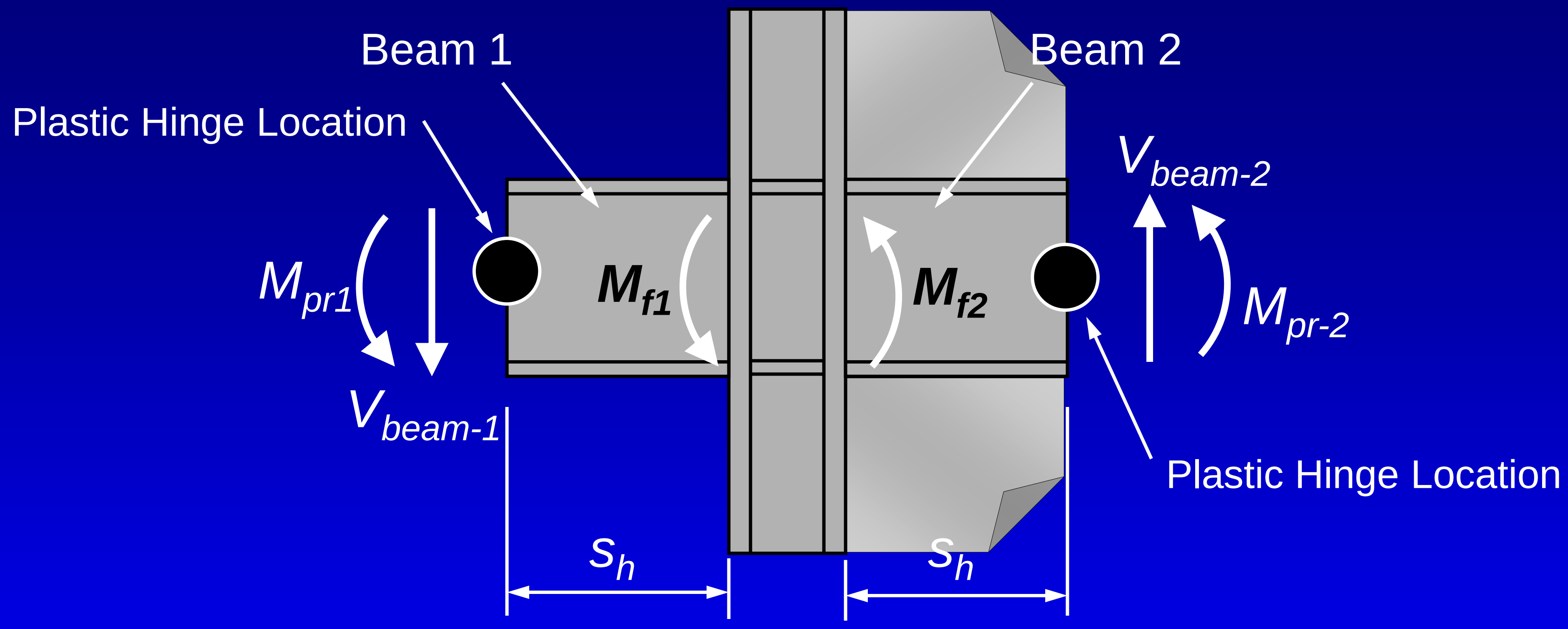


M_{pr} = expected moment at plastic hinge

V_{HBE} = beam shear (includes web-plate effect)

s_h = distance from face of column to beam plastic hinge location (specified in ANSI/AISC 358, or $d_{beam}/2$)

Panel Zone Shear Strength (cont)



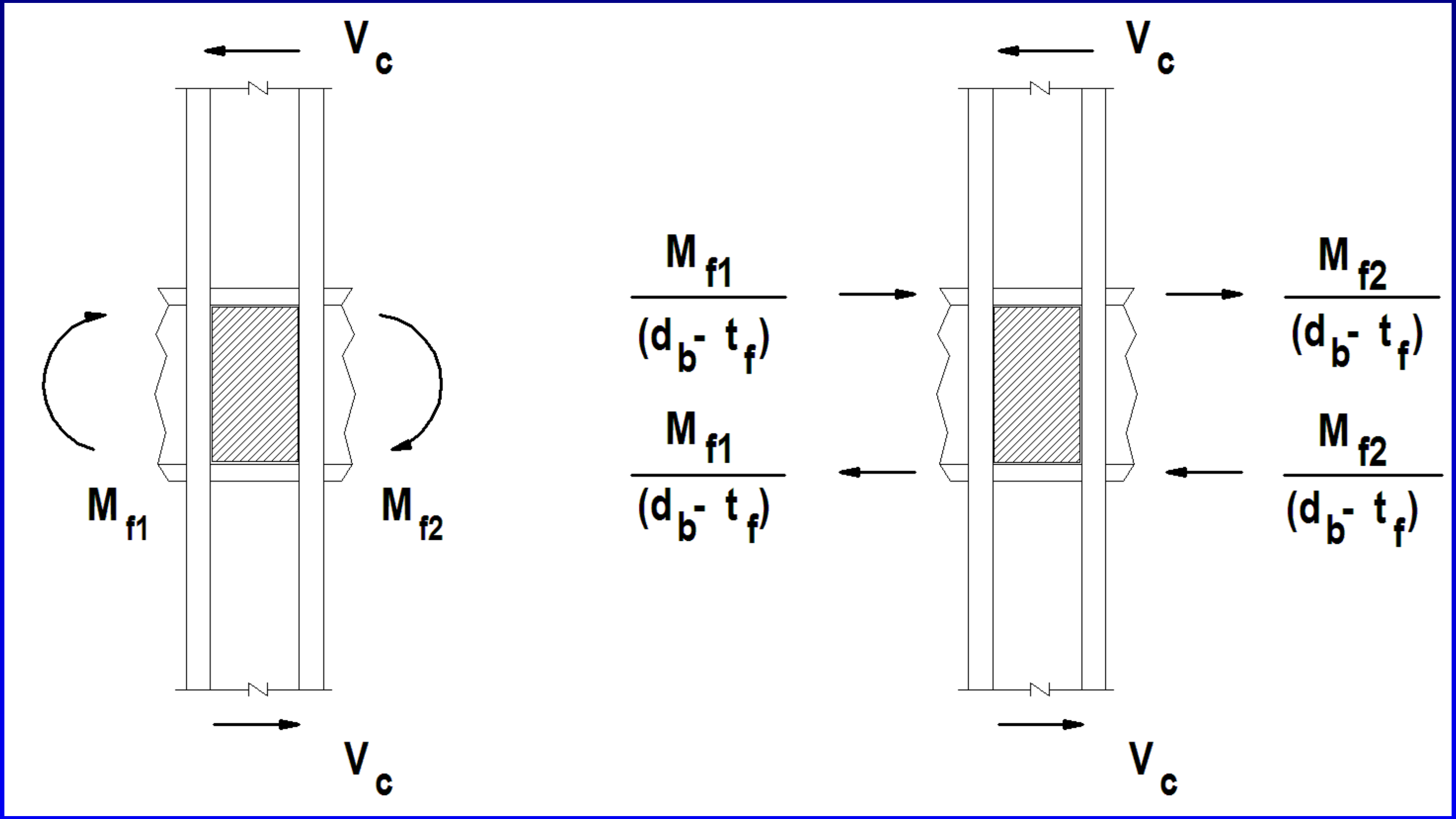
M_f = moment at column face

$$M_f = M_{pr} + V_{beam} \times s_h$$

$$M_{pr} = 1.1\ R_y\ Z_b\ (F_{yb} - P_{HBE}/A_g)$$

Panel Zone Shear Strength (cont)

$$V_c = \tfrac{1}{2} R_y F_y t_w h_c/2 \sin(2\alpha)$$



Panel Zone Required Shear Strength = $R_u = \dfrac{\sum M_f}{\left(d_b - t_f\right)} - V_c$

Panel Zone Shear Strength (cont)

Panel Zone Design Requirement:

$$R_u \leq \phi_v R_v \quad \textit{where } \phi_v = 1.0$$

R_v = nominal shear strength, based on a limit state of shear yielding, as computed per Section J10.6 of the AISC *Specification*

Panel Zone Shear Strength (cont)

To compute nominal shear strength, R_v, of panel zone:

When $P_u \leq 0.75\ P_y$ in column:

$$R_v = 0.6 F_y d_c t_p \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p} \right]$$

(AISC Spec EQ J10-11)

Where:

Where:	d_c	=	**column depth**
	d_b	=	**beam depth**
	b_{cf}	=	**column flange width**
	t_{cf}	=	**column flange thickness**
	F_y	=	**minimum specified yield stress of column web**
	t_p	=	**thickness of column web including doubler plate**

Panel Zone Shear Strength (cont)

To compute nominal shear strength, R_v, of panel zone:

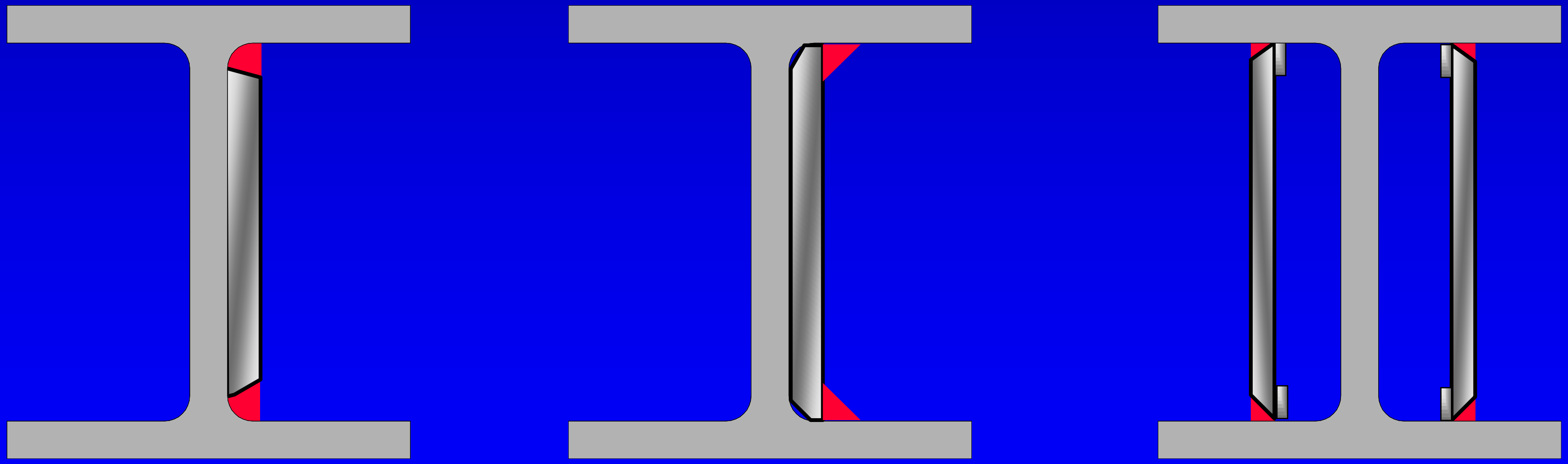
When $P_u > 0.75\ P_y$ in column (not recommended):

$$R_v = 0.6 F_y d_c t_p \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_p} \right] \left[1.9 - \frac{1.2 P_u}{P_y} \right]$$

(AISC Spec EQ J10-12)

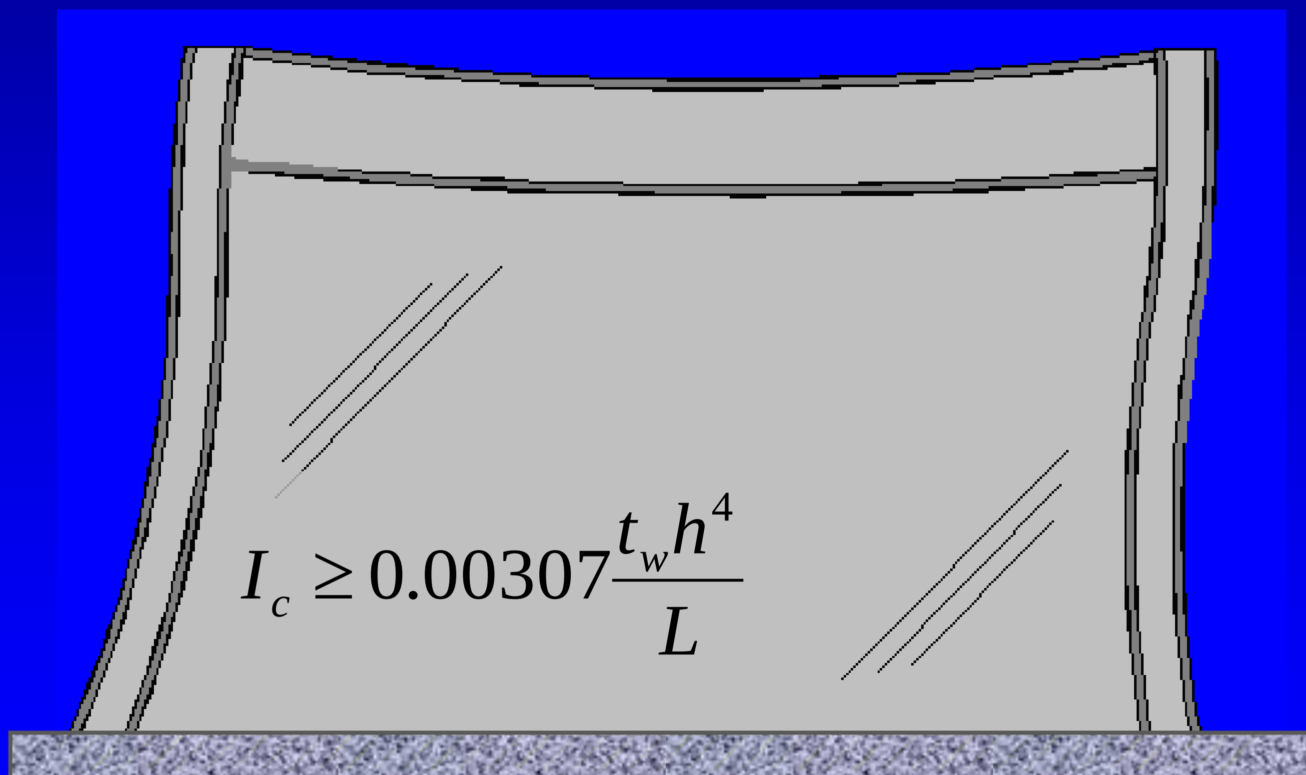
If shear strength of panel zone is inadequate:

- Choose column section with larger web area
- Weld doubler plates to column

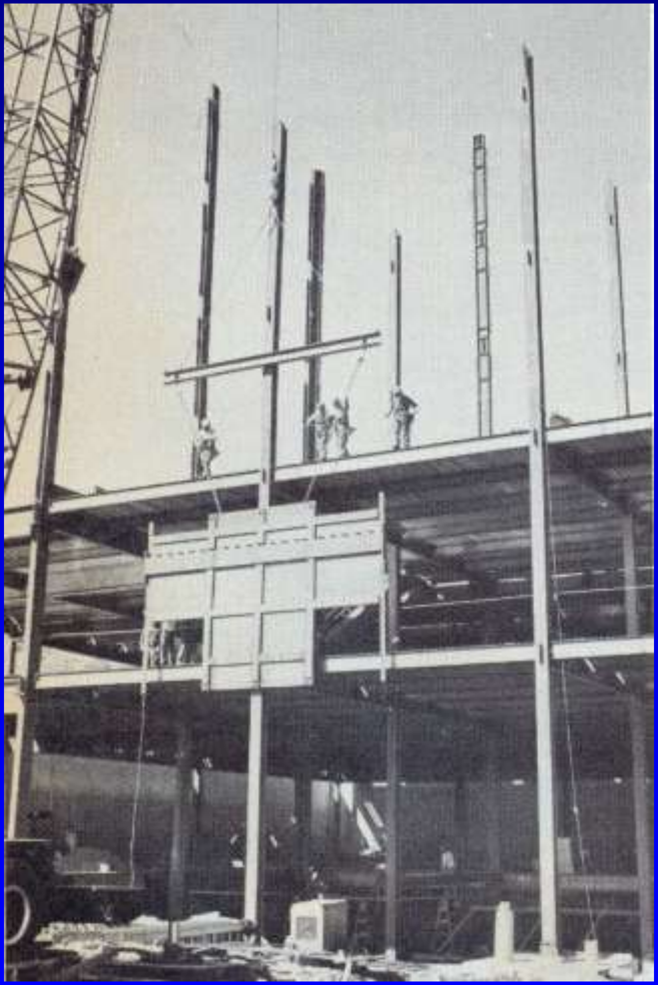


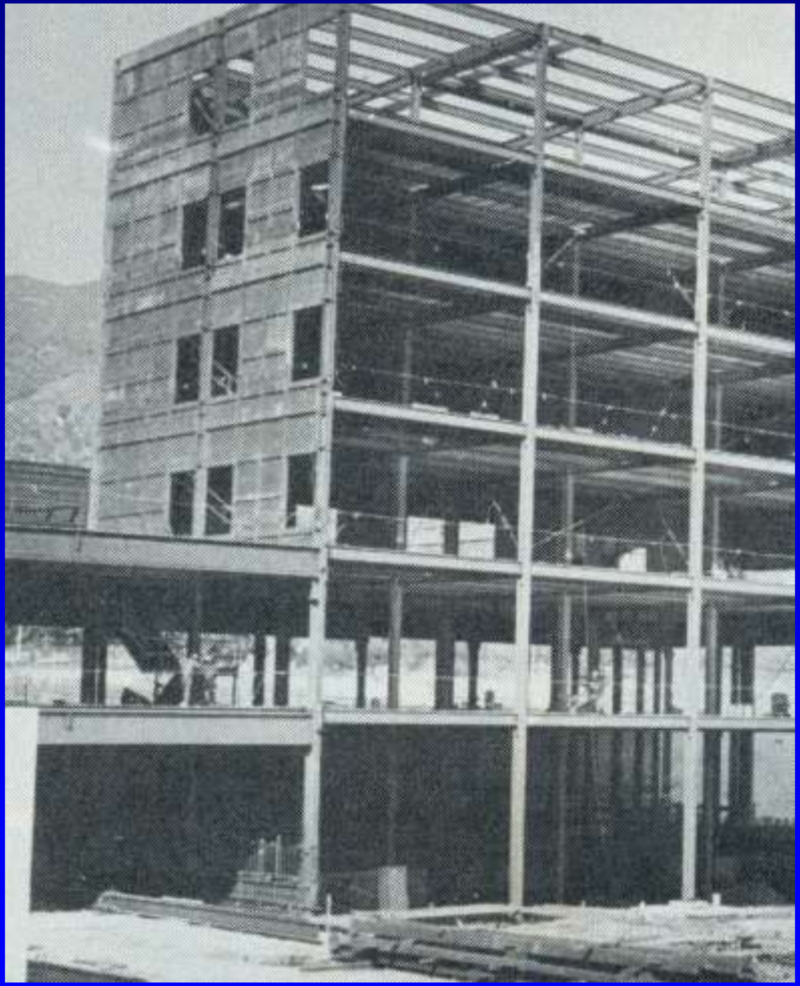
Options for Web Doubler Plates

17.4g Stiffness of Vertical Boundary Elements



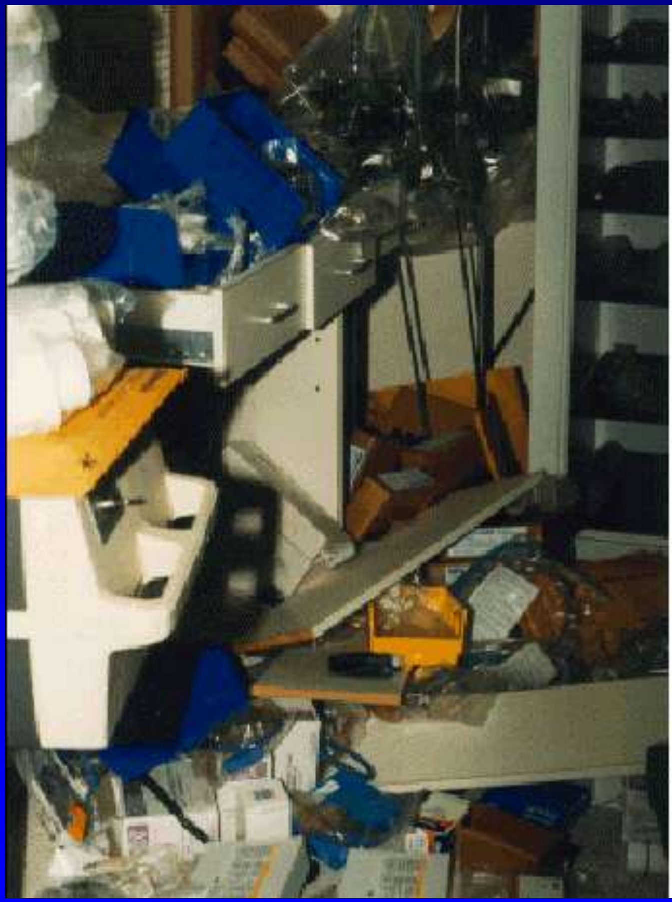
Olive View Hospital





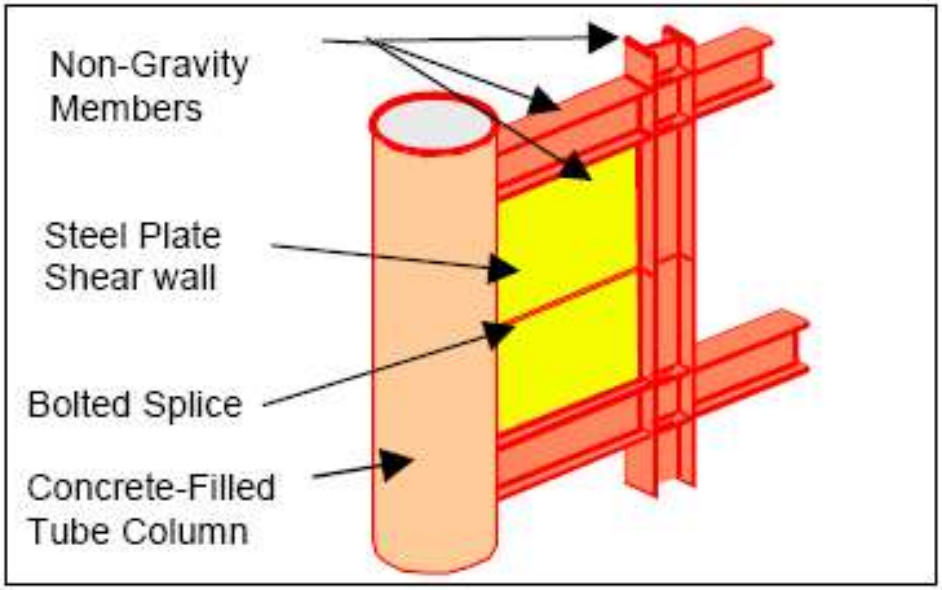
Courtesy of ENR

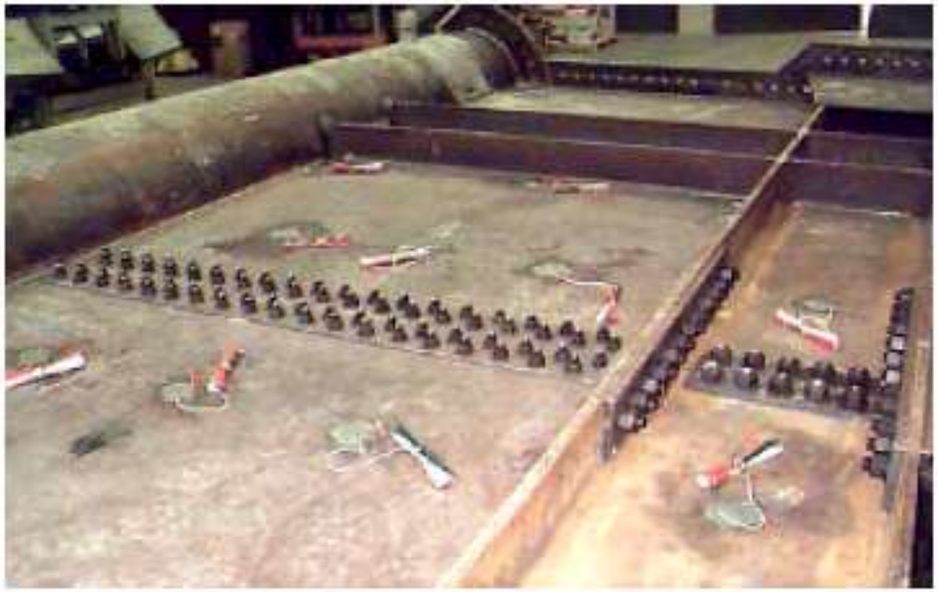
Olive View Hospital



Courtesy of Naeim and Lobo

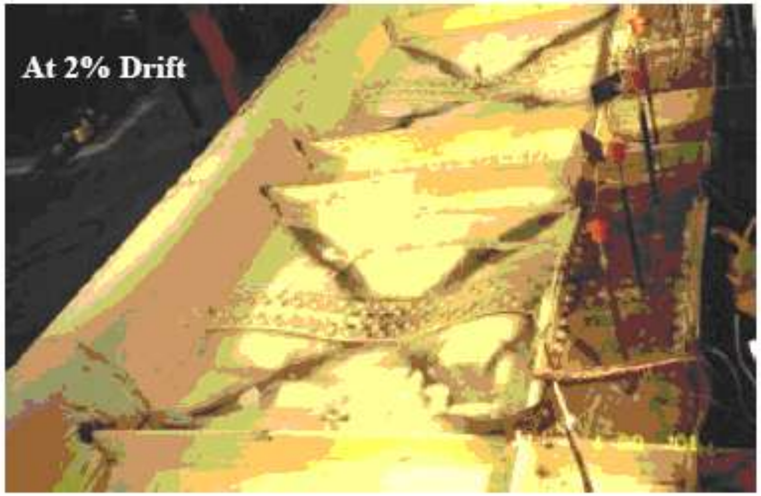
Steel plate shear wall with very strong column

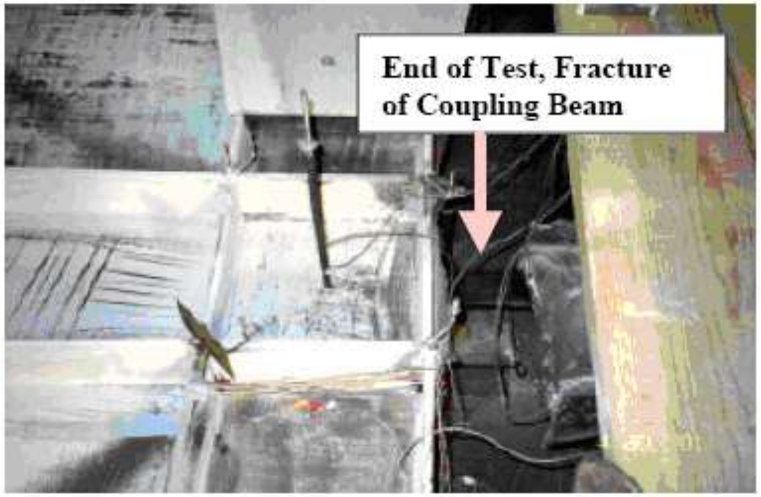


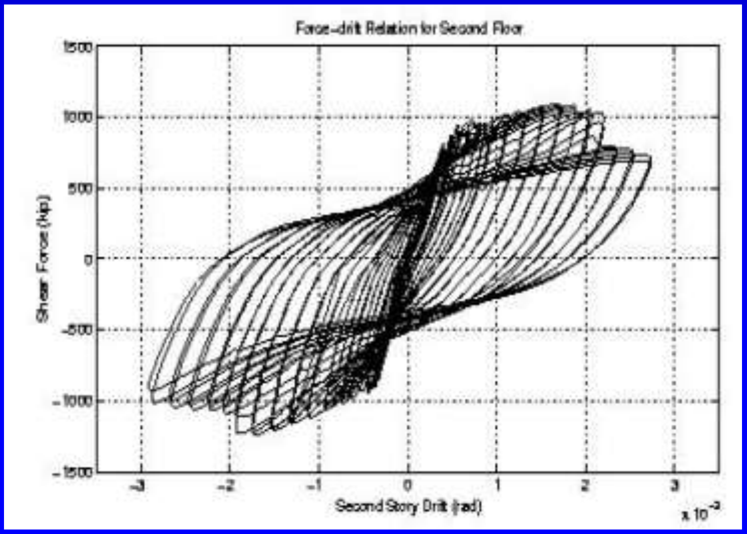


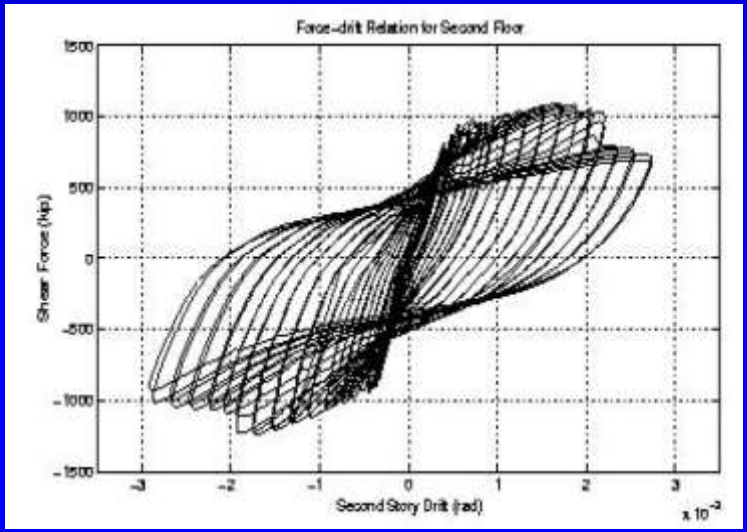
Courtesy of Astaneh-Asl and Zhao

Testing of steel plate shear wall









Courtesy of Astaneh-Asl and Zhao

Steel plate shear walls in Canam Manac Group headquarters expansion

Courtesy of Richard Vincent, Canam Manac Group, St. George, Quebec, Canada

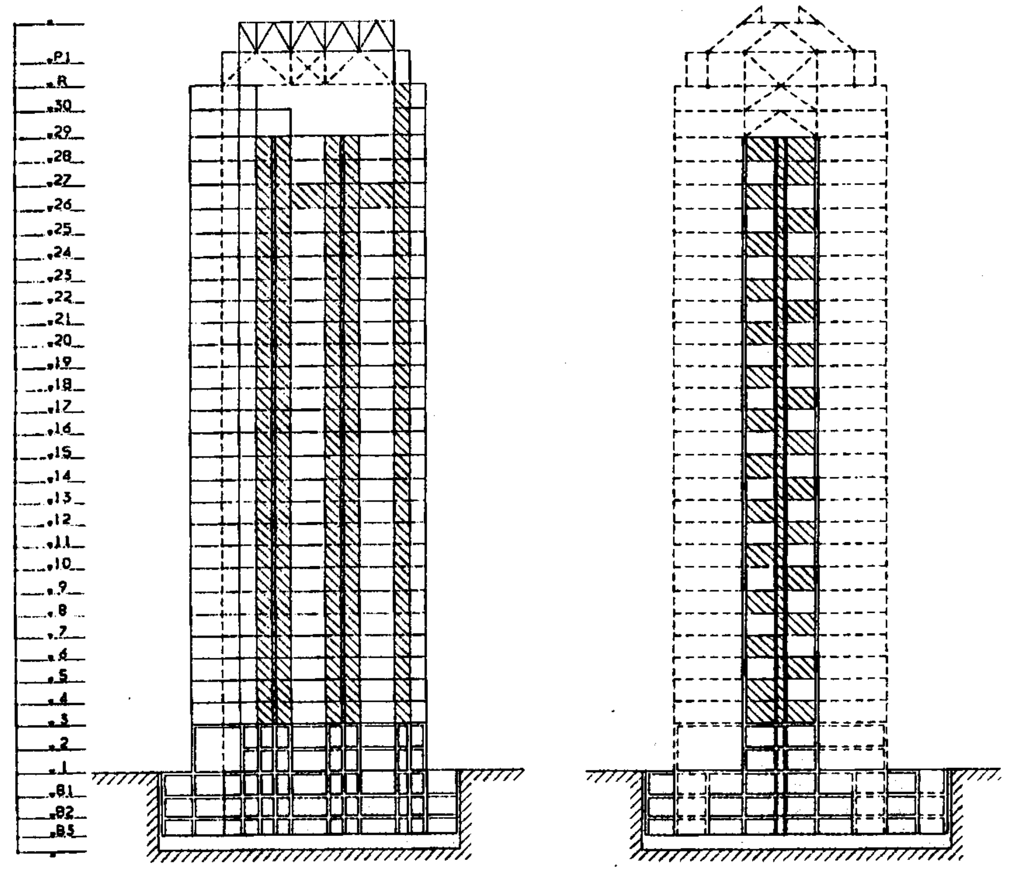
Steel plate shear walls and details at base of SPW, ING building

Courtesy of Louis Crepeau and Jean-Benoit Ducharme, Groupe Teknika, Montreal, Canada

Steel plate shear wall in ICRM building

Courtesy of Louis Crepeau and Jean-Benoit Ducharme, Groupe Teknika, Montreal, Canada

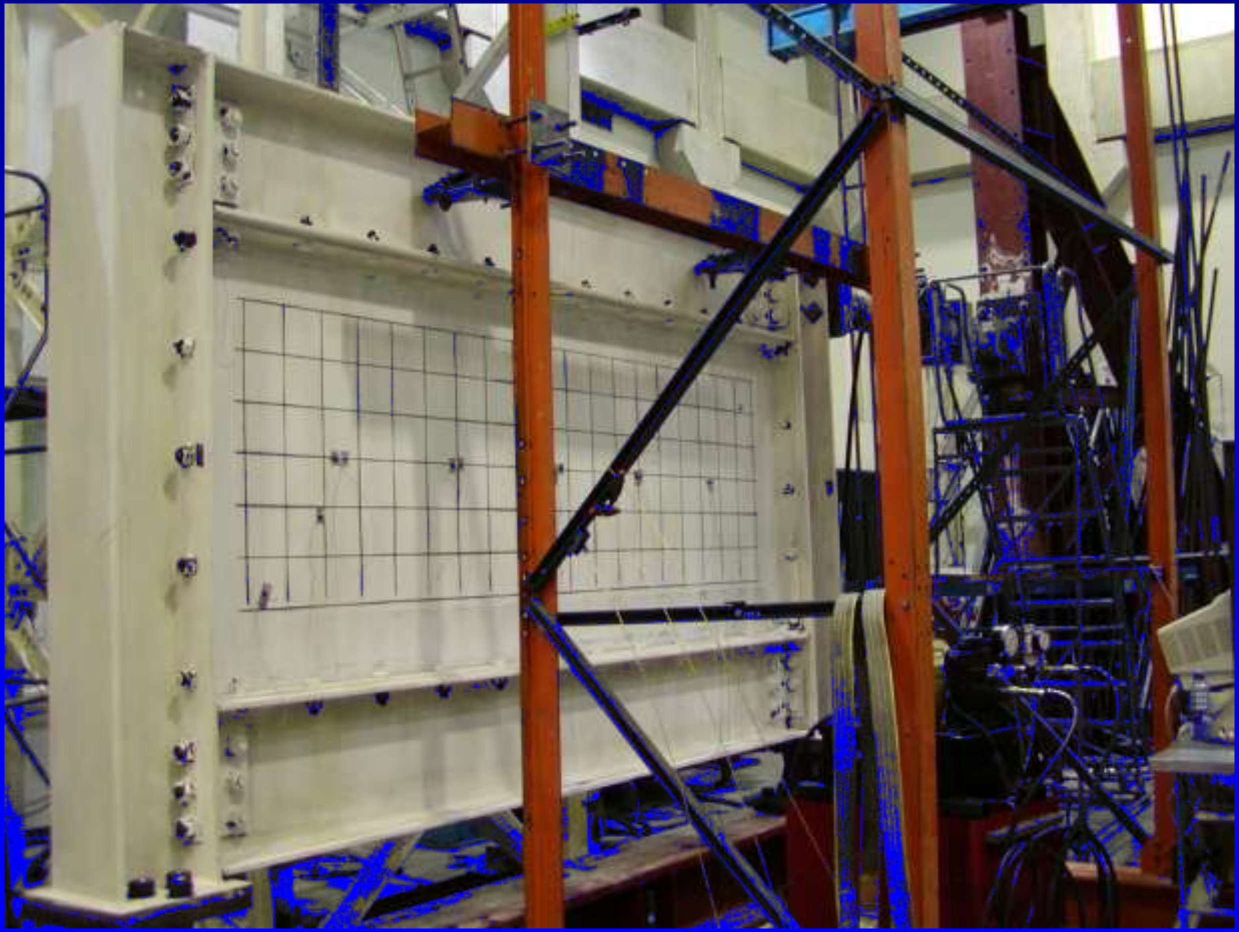
Elevation of Kobe City Hall



(a) Frame elevation (X(N-S)-direction) (b) Frame elevation (Y(E-W)-direction)

Courtesy of Fujitana

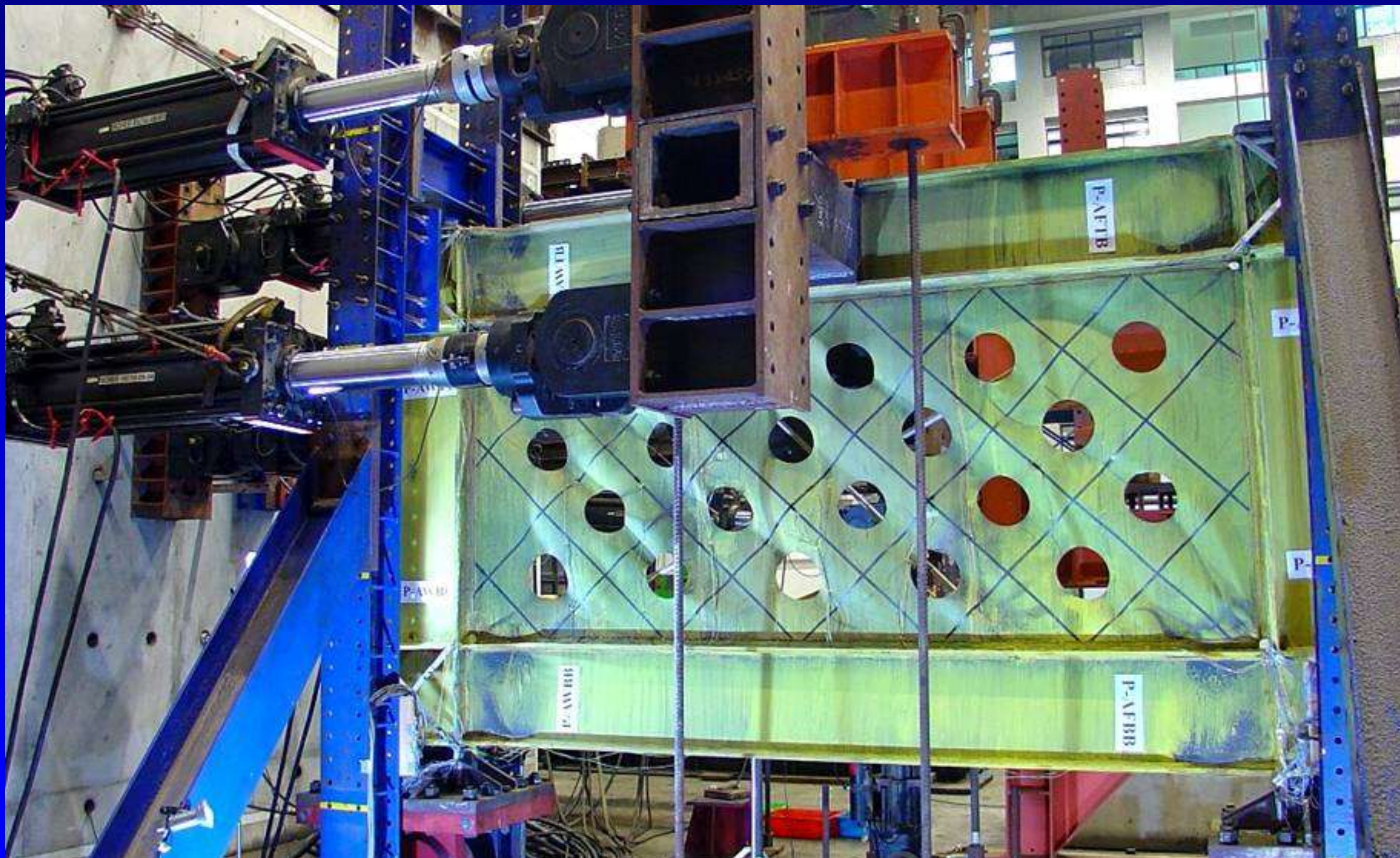
Testing of SPSW



Fracture of steel plate shear wall web plate corner at 3.07% Drift

Courtesy of Berman and Bruneau

Perforated steel plate shear wall

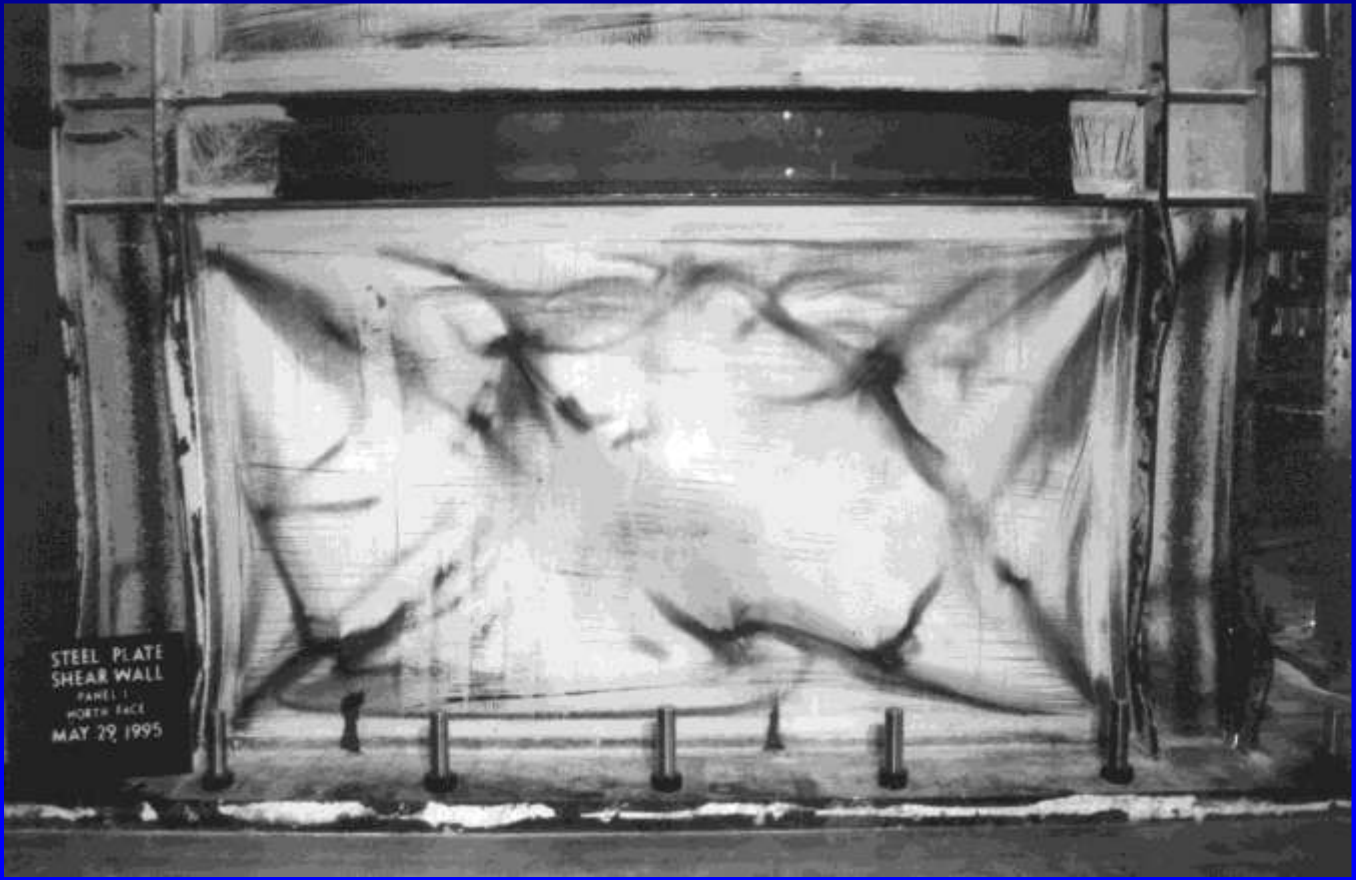


Courtesy of Vian and Bruneau

Steel plate shear wall with corner openings

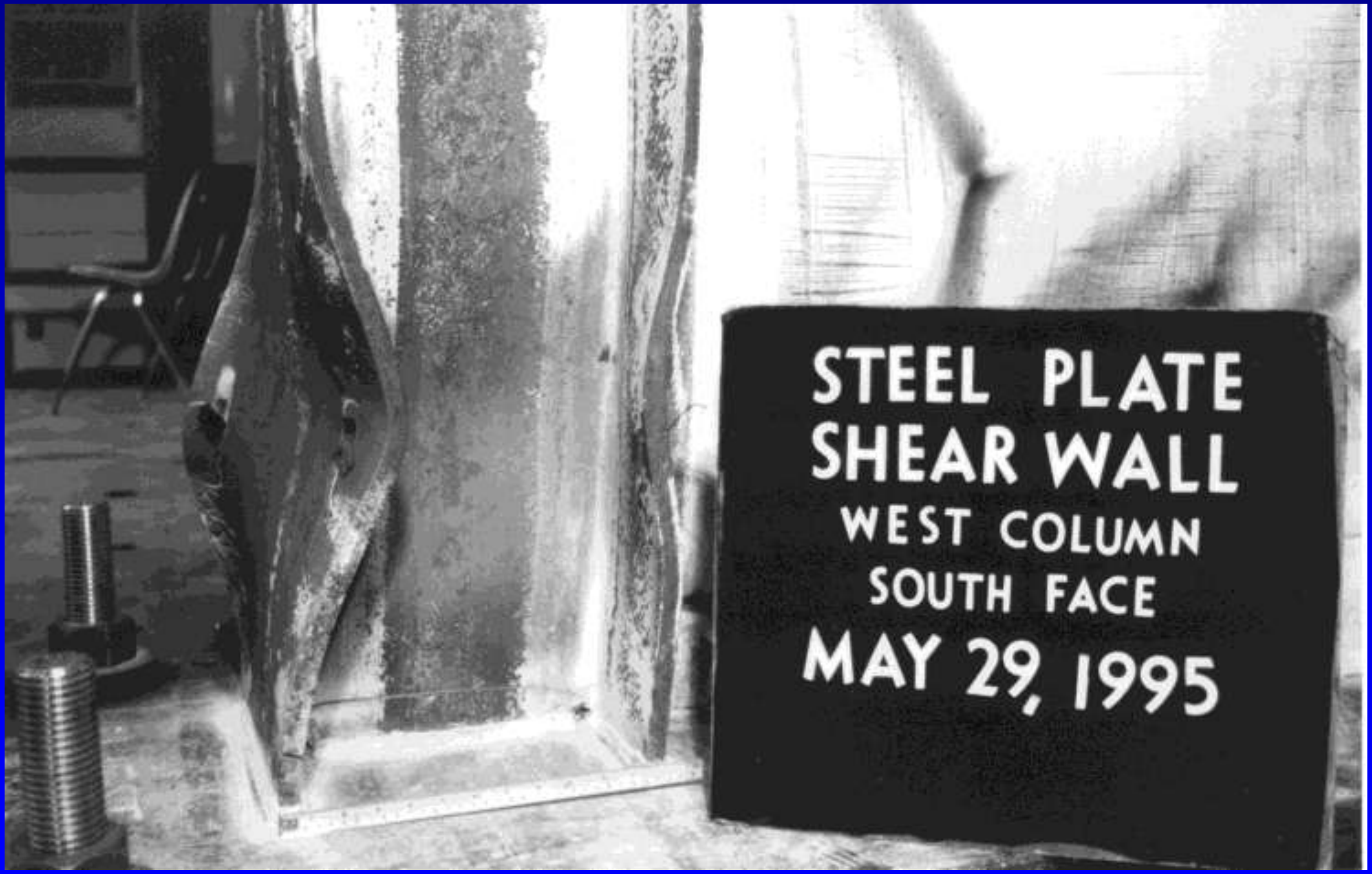
Courtesy of Vian and Bruneau

Web plate buckling



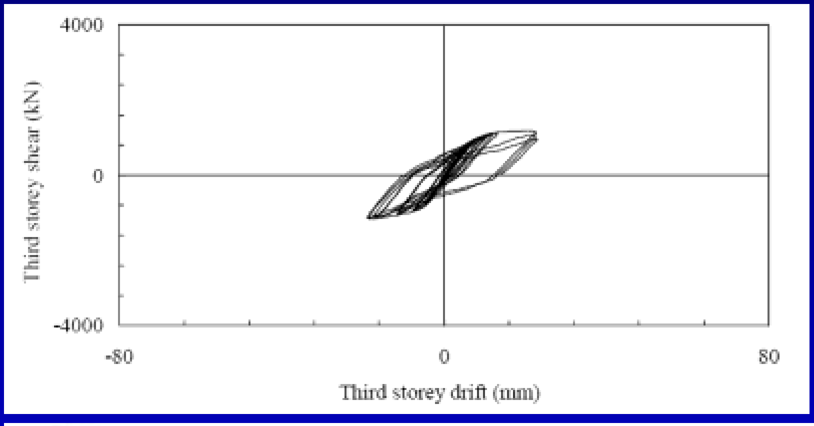
Courtesy of Robert Driver, University of Alberta, Edmonton, Canada

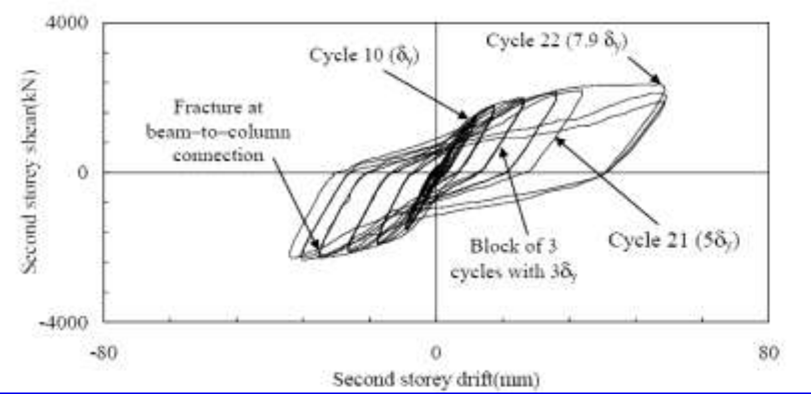
Local buckling and fracture of column

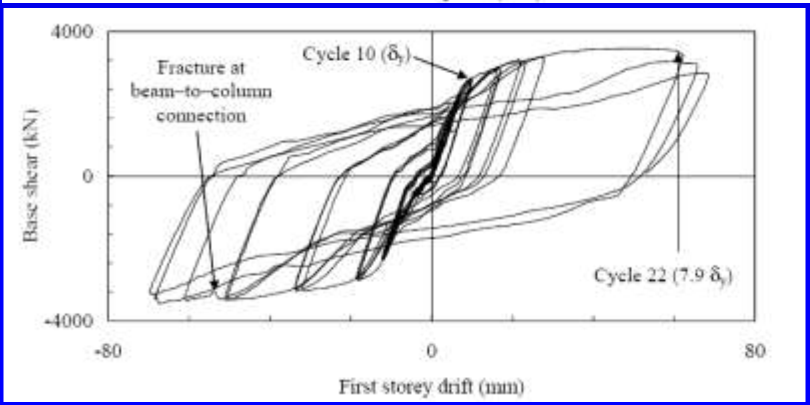


Courtesy of Robert Driver, University of Alberta, Edmonton, Canada

Testing of a multi-story steel plate shear wall







Courtesy of Behbahanifard

ASTM Designation	Specified Minimum Yield Stress F_y (ksi)	Specified Minimum Tensile Stress F_u (ksi)	Minimum Elongation in 2 in. Gage per ASTM A370 (percent)	Listed in AISC 341?	Ratio of Expected Yield to Specified Minimum R_y
A36	36	58	23	Yes	1.3
A572 Gr. 42	42	60	24	Yes	Not defined
A572 Gr. 50	50	65	21	Yes	1.1
A588 Gr. 50	50	70	21	Yes	1.1
A709 Gr. 36	36	58	23	Yes	Not defined
A709 Gr. 50	50	65	21	Yes	Not defined
A1011 CS	(30-50)[1]	Not defined	(25)[1]	No	Not defined
A1011 DS	(30-45)[1]	Not defined	(28)[1]	No	Not defined
A1011 SS Gr. 30	30	49	25[2]; 24[3]; 21[4]	No	Not defined
A1011 SS Gr. 33	33	52	23[2]; 22[3]	No	Not defined
A1011 SS Gr. 36 Type 1	36	53	22[2]; 21[3]	No	Not defined
A1011 SS Gr. 36 Type 2	36	58	21[2]; 20[3]	No	Not defined
A1011 SS Gr. 40	40	55	21[2]; 20[3]	No	Not defined
A1011 HSLAS Gr. 45 Class 1	45	60	25[5]; 23[6]	No	Not defined
A1011 HSLAS Gr. 45 Class 2	45	55	25[5]; 23[6]	No	Not defined
A1011 HSLAS Gr. 50 Class 1	50	65	22[5]; 20[6]	No	Not defined
A1011 HSLAS Gr. 50 Class 2	50	60	22[5]; 20[6]	No	Not defined

Thickness (in.)	Standard Gage	ASTM Designation															
		A36	A572 Gr. 42	A572 Gr. 50	A588	A709 Gr. 36	A709 Gr. 50	A1011 CS	A1011 DS	A1011 SS Gr. 30	A1011 SS Gr. 33	A1011 SS Gr. 36 Type 1	A1011 SS Gr. 36 Type 2	A1011 HSLAS Gr. 45 Type 1	A1011 HSLAS Gr. 45 Type 2	A1011 HSLAS Gr. 50 Type 1	A1011 HSLAS Gr. 50 Type 2
0.0598	16							•	•	•	•	•	•	•	•	•	•
0.0625		•															
0.0673	15							•	•	•	•	•	•	•	•	•	•
0.0747	14	•						•	•	•	•	•	•	•	•	•	•
0.0897	13							•	•	•	•	•	•	•	•	•	•
0.1046	12	•						•	•	•	•	•	•				
0.1196	11							•	•	•	•	•	•				
0.1250		•															
0.1345	10	•						•	•	•	•	•	•				
0.1495	9							•	•	•	•	•	•				
0.1644	8							•	•	•	•	•	•				
0.1793	7							•	•	•	•	•	•				
0.1875		•	•	•	•	•	•			•	•	•	•				
0.1943	6									•	•	•	•				
0.2092	5									•	•	•	•				
0.2242	4									•	•	•	•				
0.2391	3									•	•	•	•				
0.2500		•	•	•	•	•	•										
0.3125		•	•	•	•	•	•										
0.3750		•	•	•	•	•	•										
0.5000		•	•	•	•	•	•										