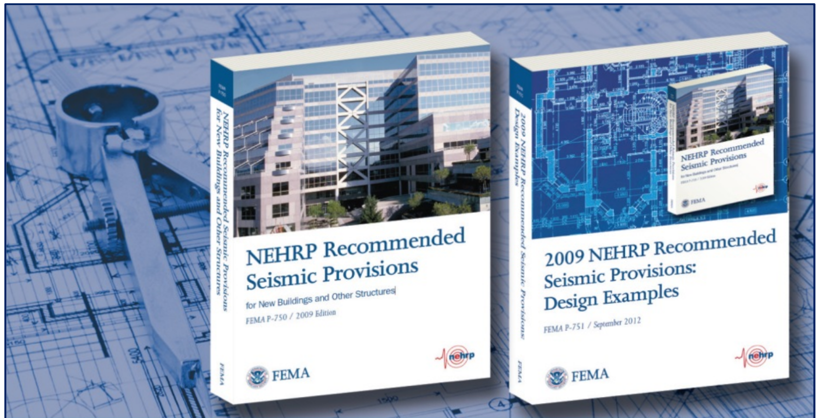


2009 NEHRP Recommended Seismic Provisions:
Training and Instructional Materials

FEMA P-752 CD / June 2013

FEMA

nehrp

6

Structural Steel Design

Rafael Sabelli, S.E., and Brian Dean, P.E.

Originally developed by
James R. Harris, P.E., PhD, Frederick R. Rutz, P.E., PhD, and Teymour Manouri, P.E., PhD

SEISMIC DESIGN OF STEEL STRUCTURES

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics
- Summary

Steel Design: Context in Provisions

Design basis: Strength limit state

Using the 2009 *NEHRP Recommended Provisions*, Refer to ASCE 7 2005:

Seismic Design Criteria	Chap. 11
Seismic Design Requirements	
Buildings	Chap. 12
Nonstructural components	Chap. 13
Design of steel structures	Chap. 14
	AISC Seismic
	and others

Seismic Resisting Systems

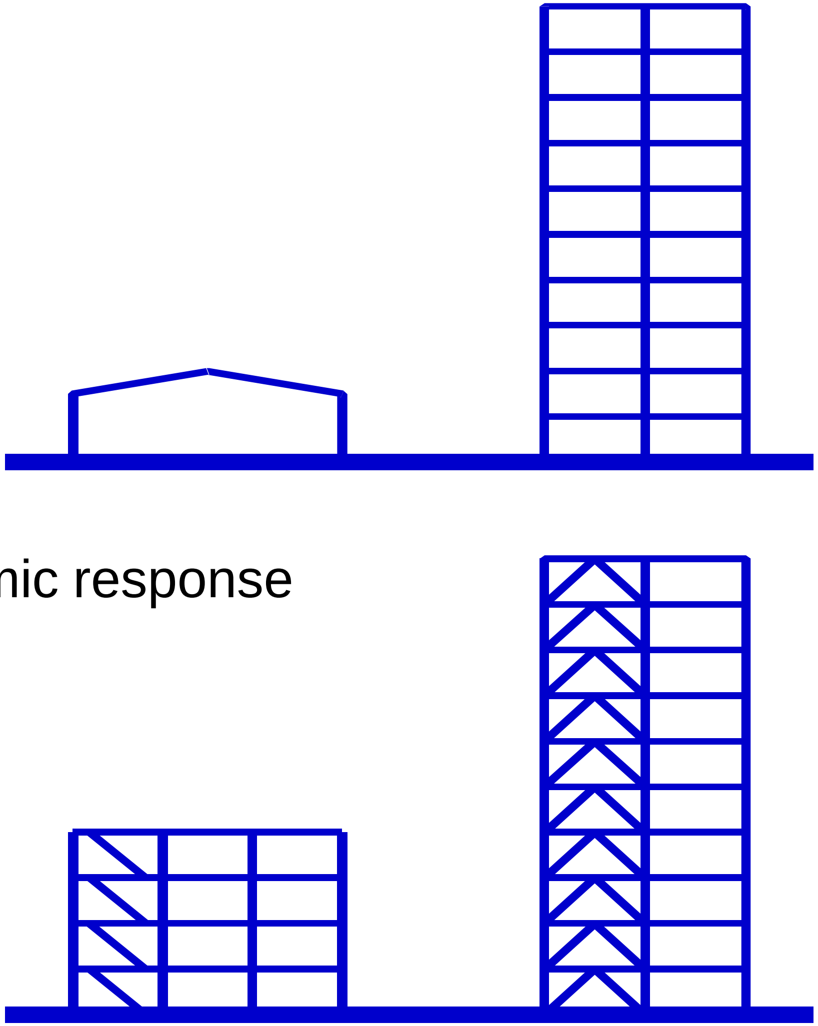
Unbraced Frames

Connections are:

- Fully Restrained Moment-resisting
- Partially Restrained Moment-resisting

Seismic classes are:

- Special Moment Frames
- Intermediate Moment Frames
- Ordinary Moment Frames
- Systems not specifically detailed for seismic response



Braced Frames

Ordinary Concentric Braced Frames

Special Concentric Braced Frames

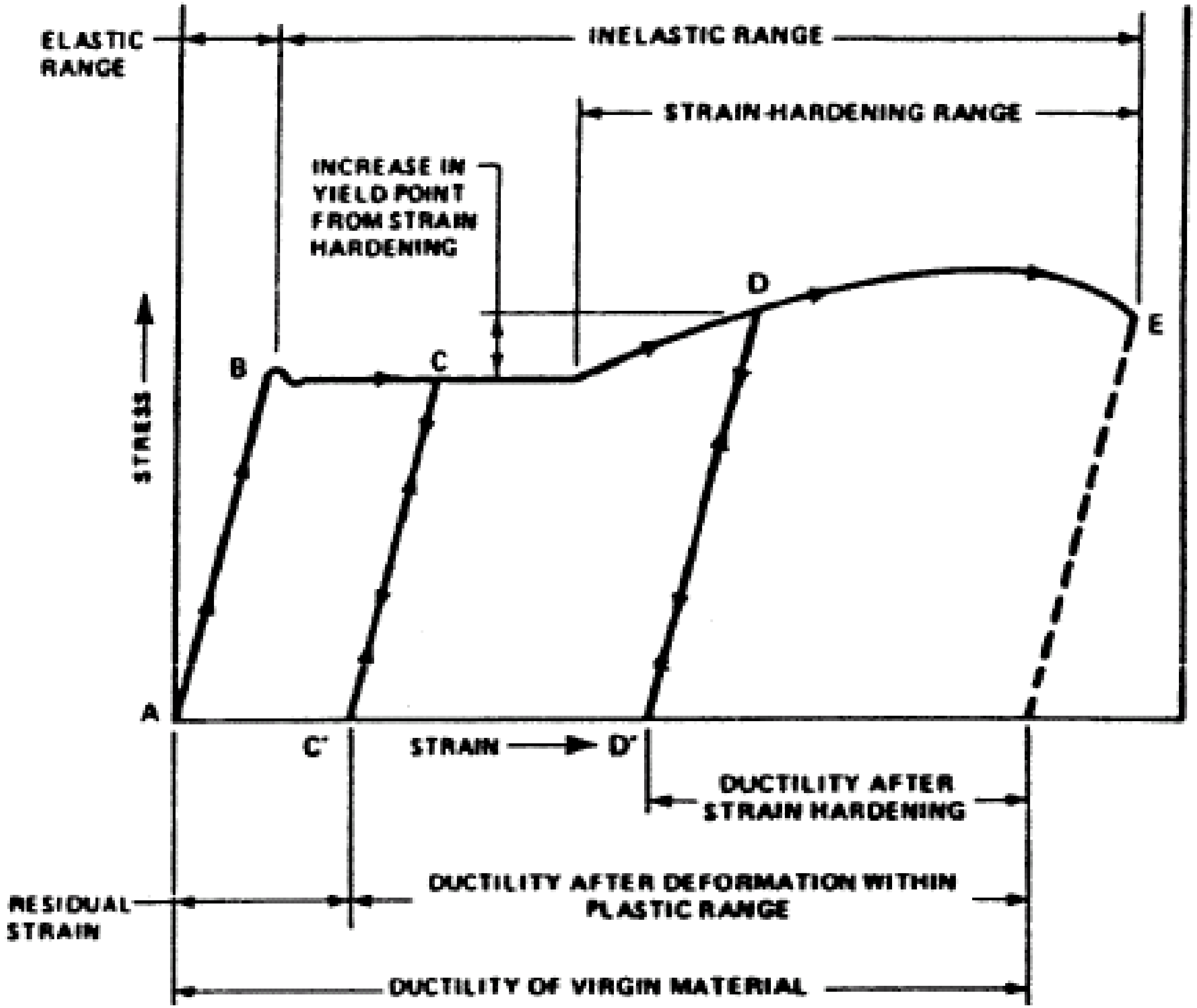
Eccentrically Braced Frames

Buckling Restrained Braced Frames

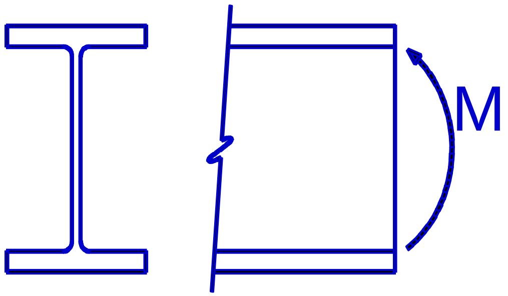
Special Plate Shear Walls

Systems not specifically detailed for seismic response

Monotonic Stress-Strain Behavior



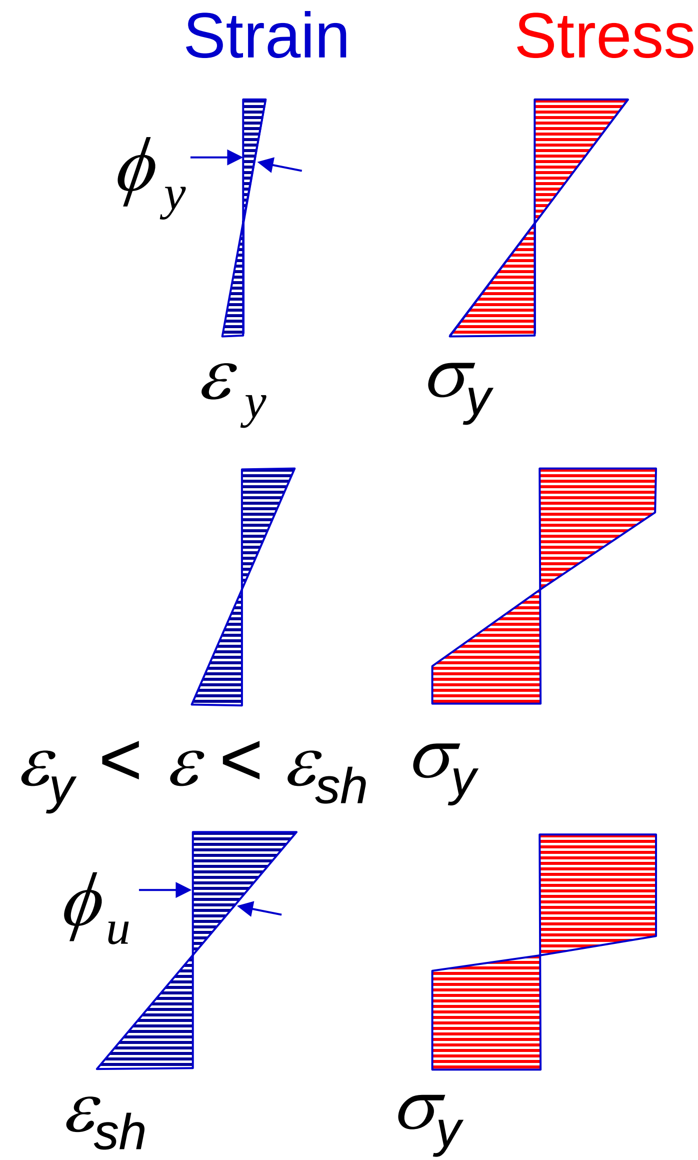
Bending of Steel Beam



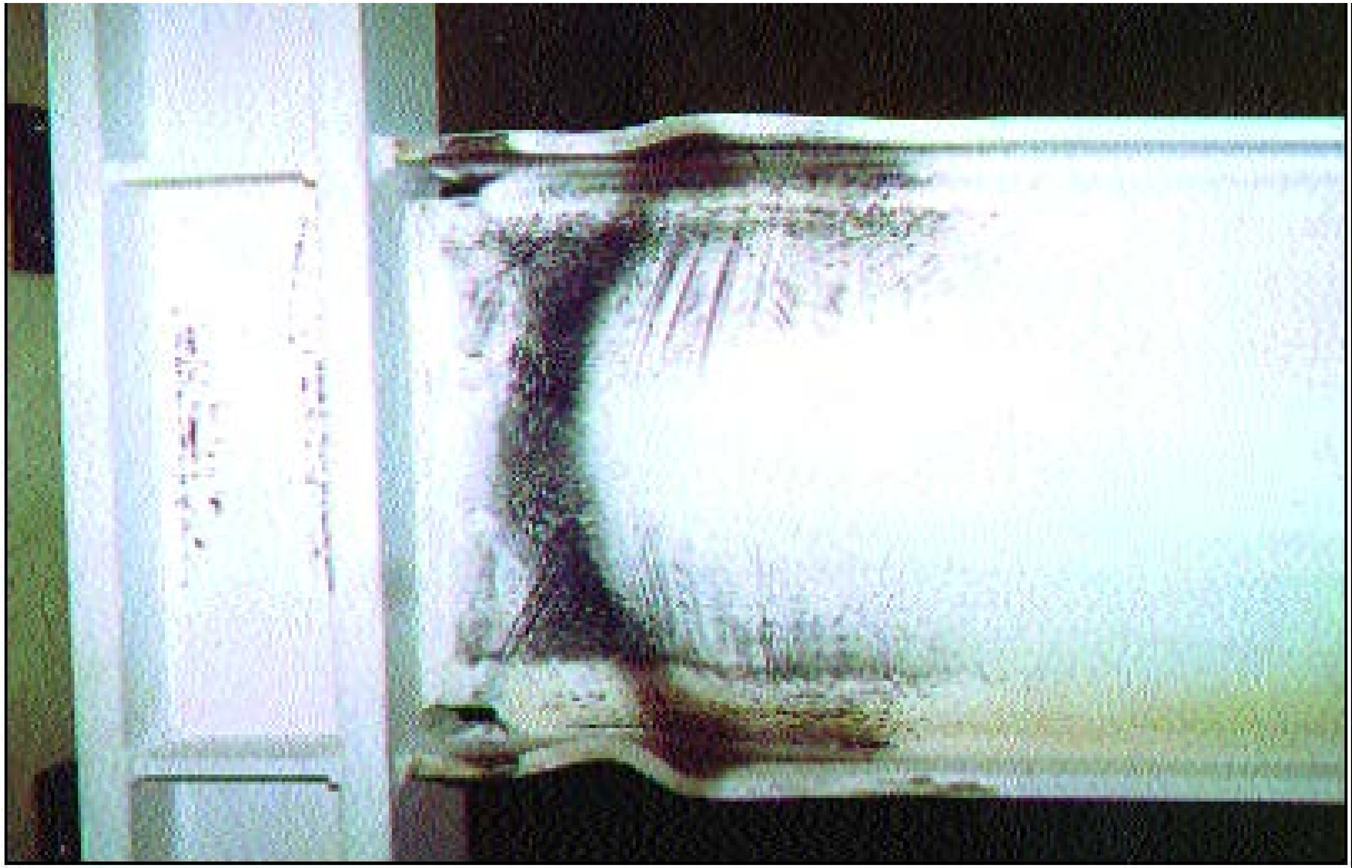
Extreme fiber reaches yield strain and stress

Strain slightly above yield strain

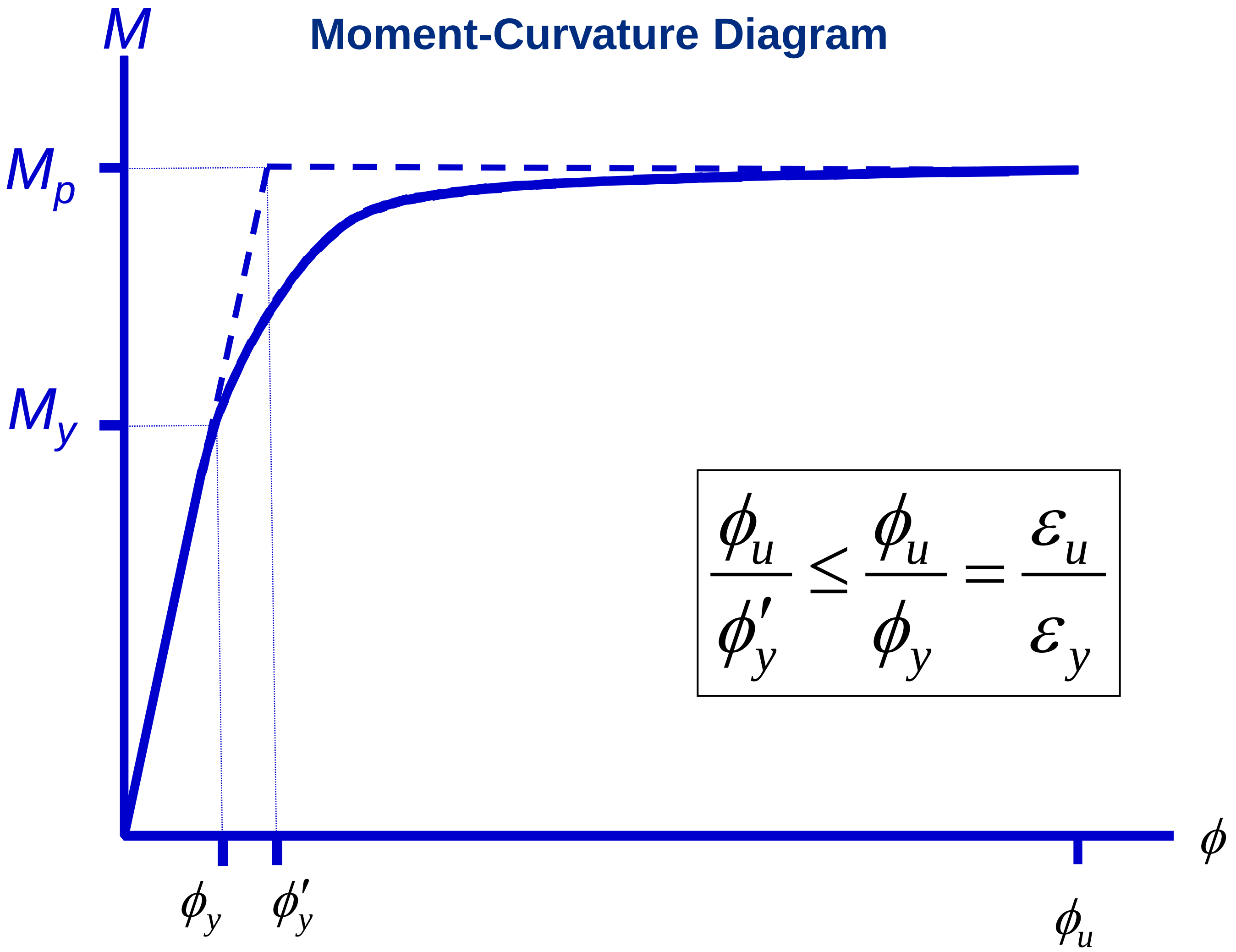
Section near "plastic"



Plastic Hinge Formation

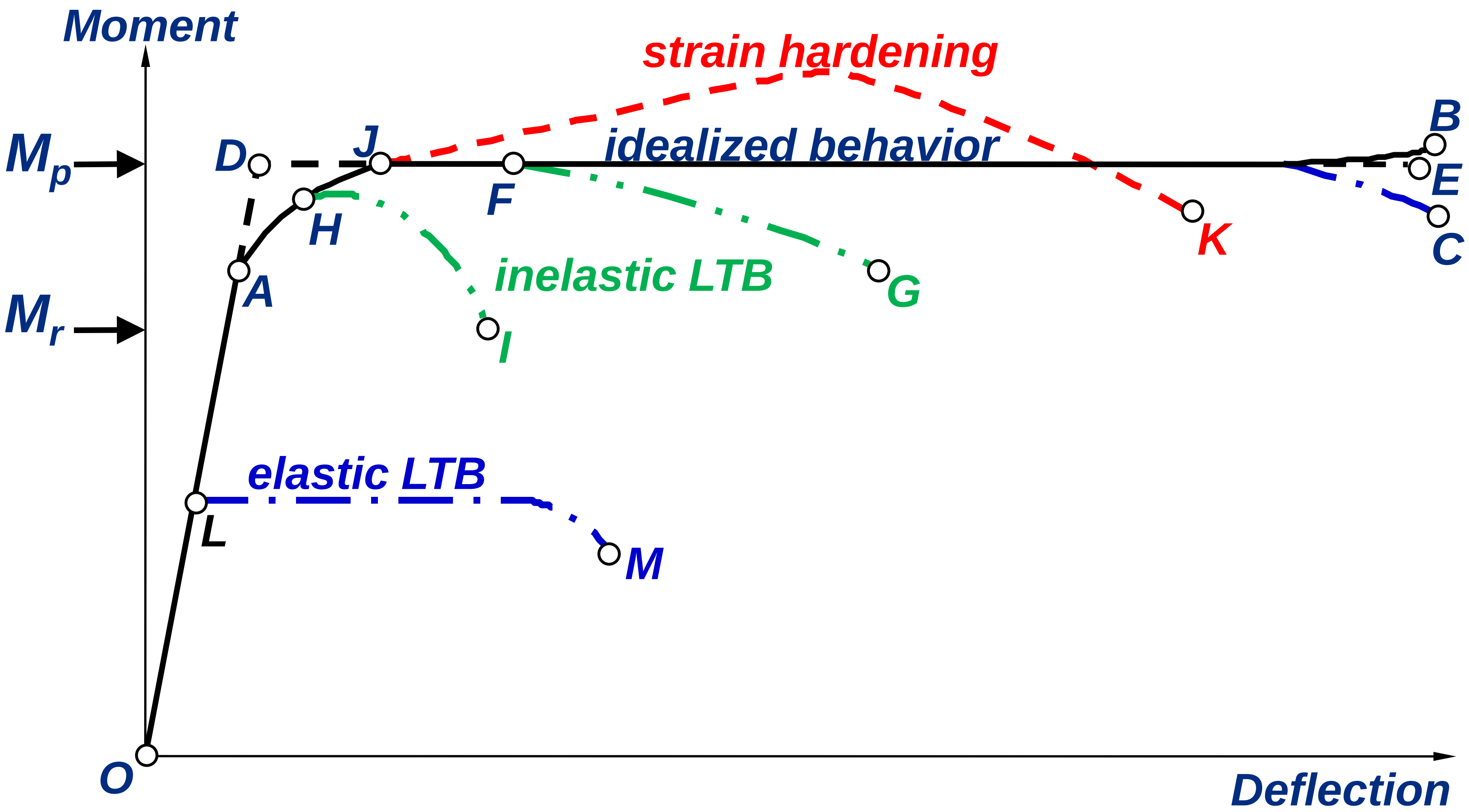


Cross - section Ductility



$$\frac{\phi_u}{\phi'_y} \le \frac{\phi_u}{\phi_y} = \frac{\varepsilon_u}{\varepsilon_y}$$

Behavior Modes For Beams



Flexural Ductility of Steel Members
Practical Limits

1 Lateral torsional buckling

Brace well

2 Local buckling

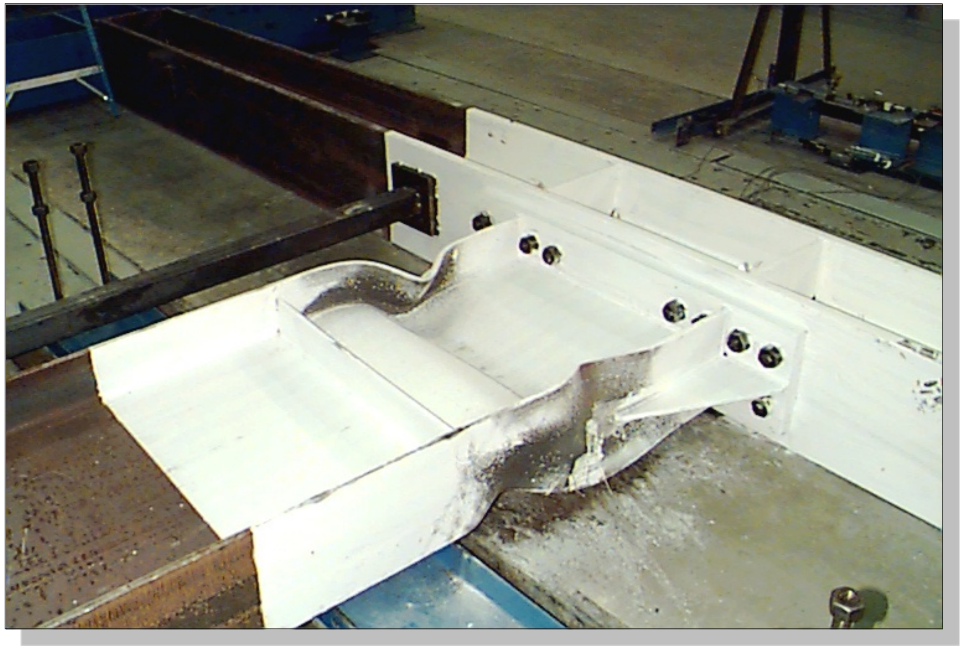
Limit width-to-thickness ratios

for compression elements

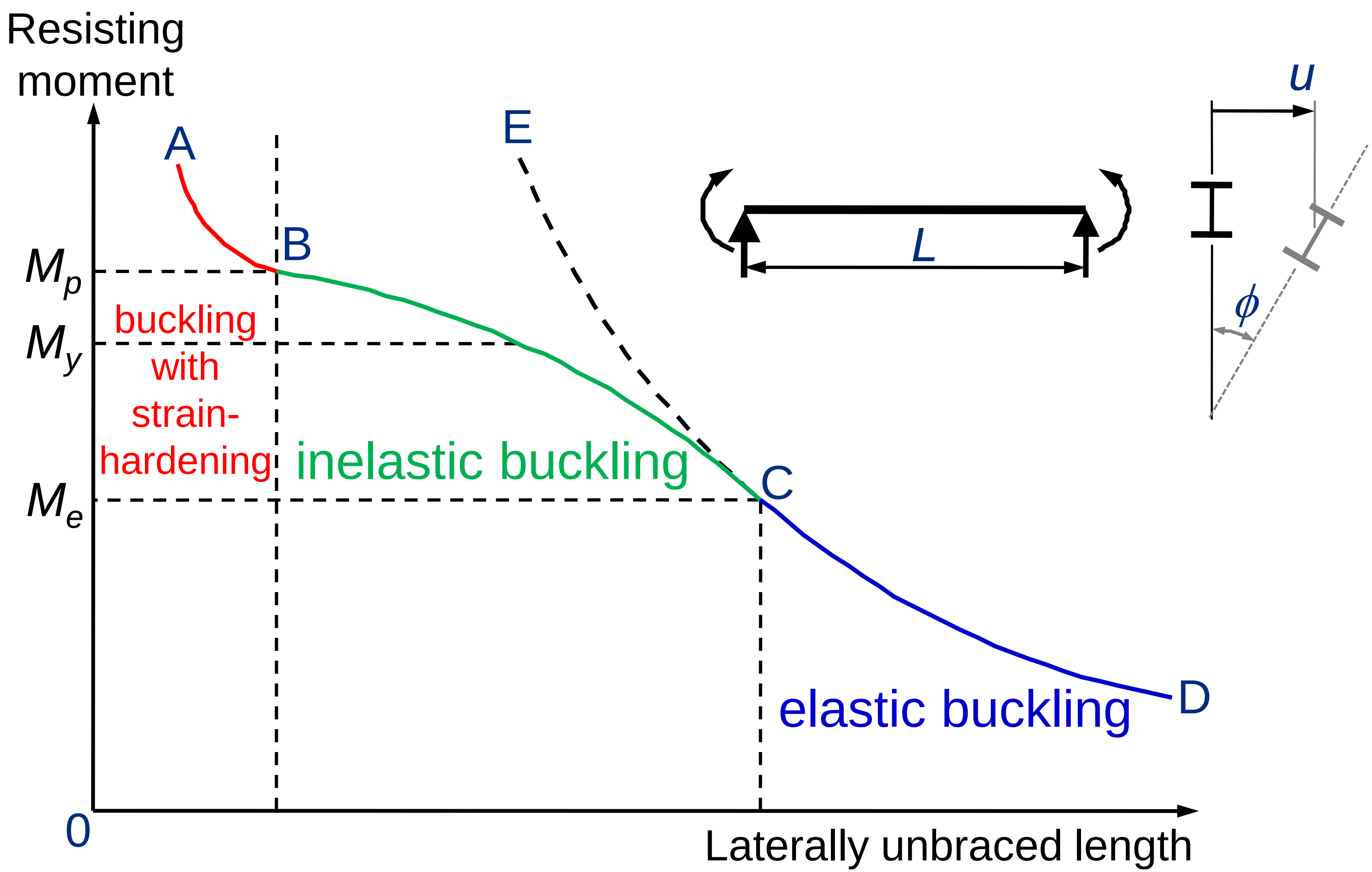
3 Fracture

Avoid by proper detailing

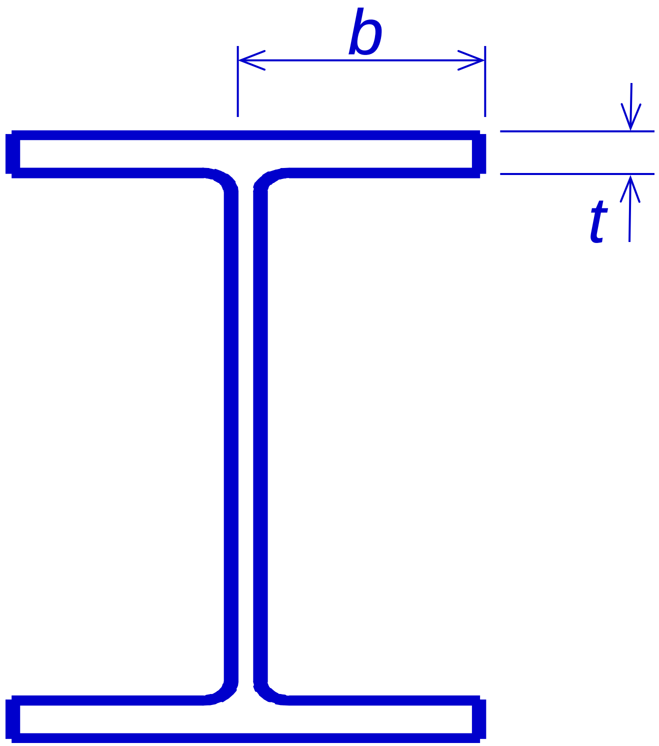
Local and Lateral Buckling



Lateral Torsional Buckling



Local Buckling



Classical plate buckling solution:

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(b/t)^2} \leq \sigma_y$$

Substituting μ = *0.3 and rearranging*:

$$\frac{b}{t} \leq 0.95\sqrt{\frac{kE}{F_y}}$$

Local Buckling
continued

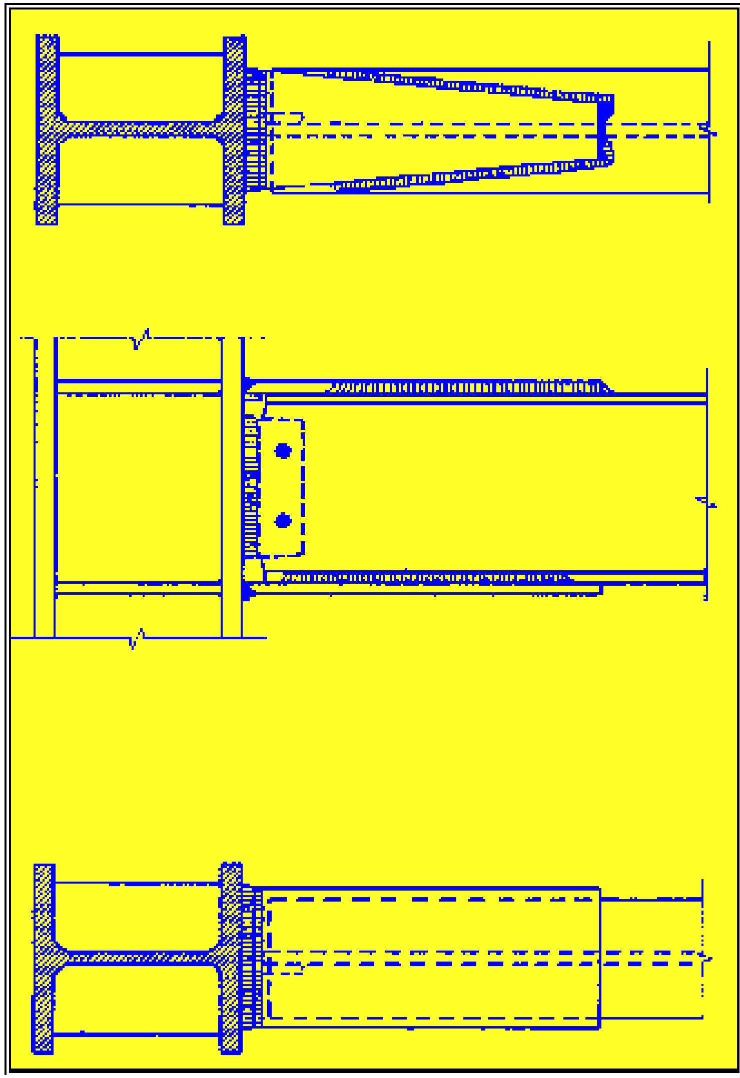
With the plate buckling coefficient, *K*, taken as 0.7 and an adjustment for residual stresses, the expression for b/t becomes:

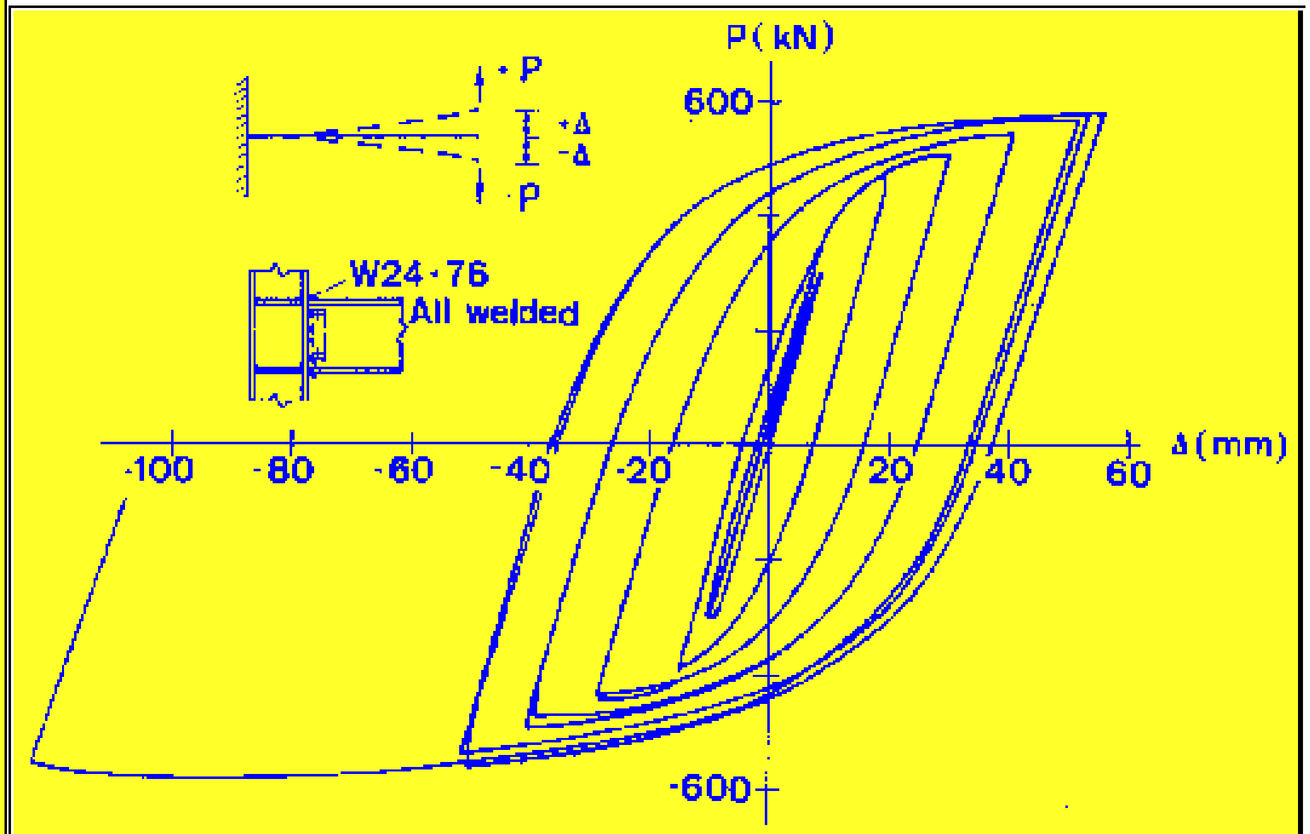
$$\frac{b}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$$

This is the slenderness requirement given in the AISC specification for compact flanges of I-shaped sections in bending. The coefficient is further reduced for sections to be used in seismic applications in the AISC Seismic specification

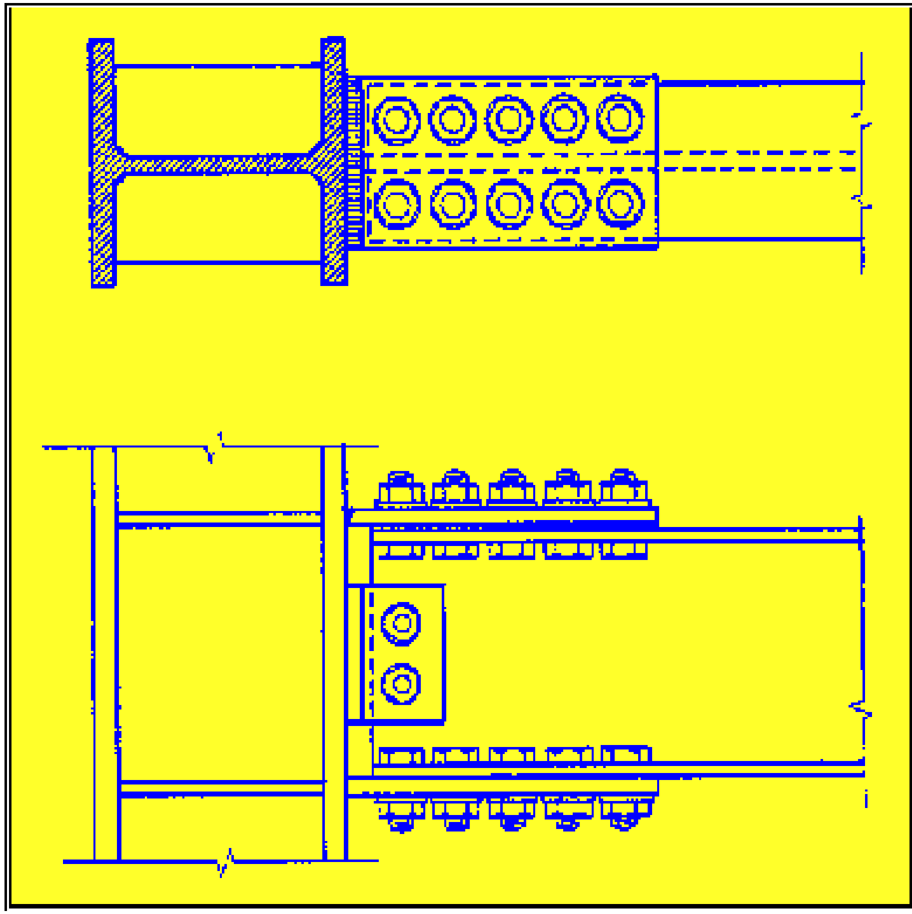
$$\frac{b}{t} \leq 0.3\sqrt{\frac{E}{F_y}}$$

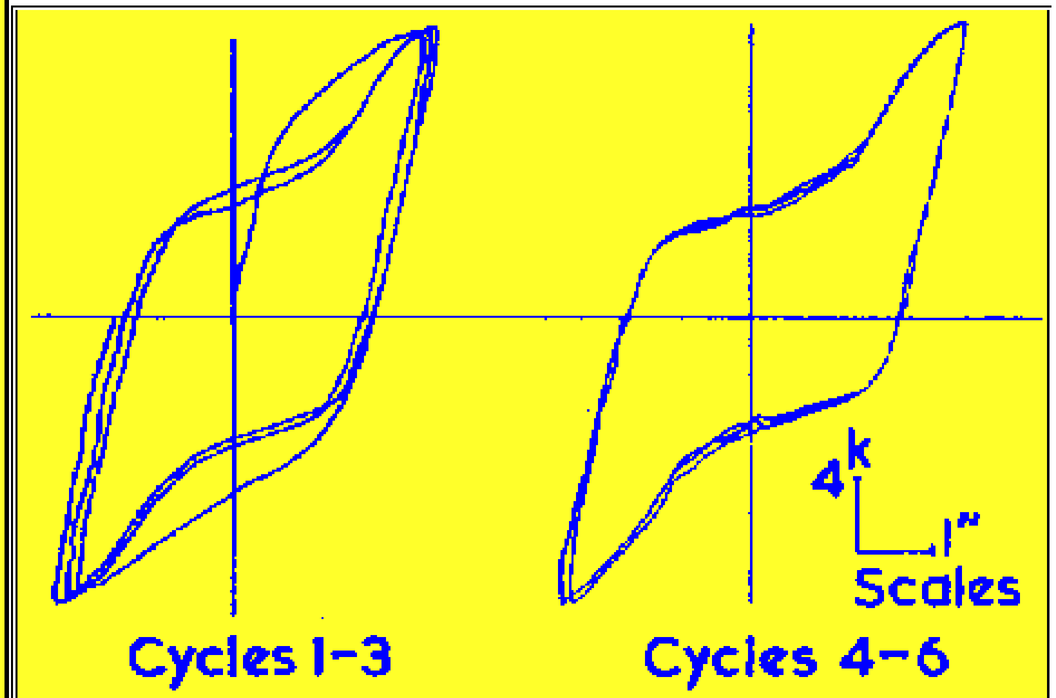
Welded Beam to Column Laboratory Test - 1960s



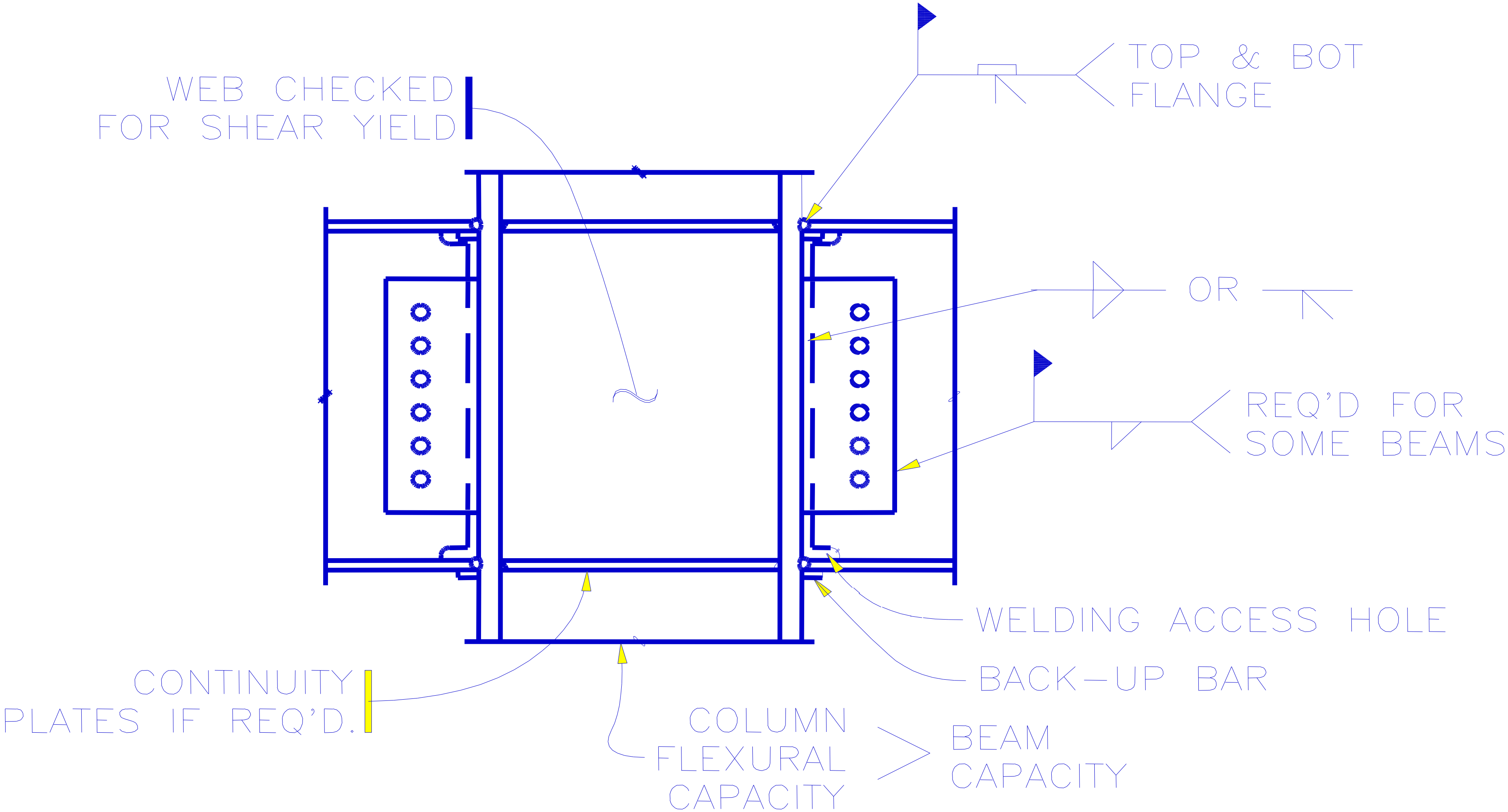


Bolted Beam to Column Laboratory Test - 1960s





Pre-Northridge Standard



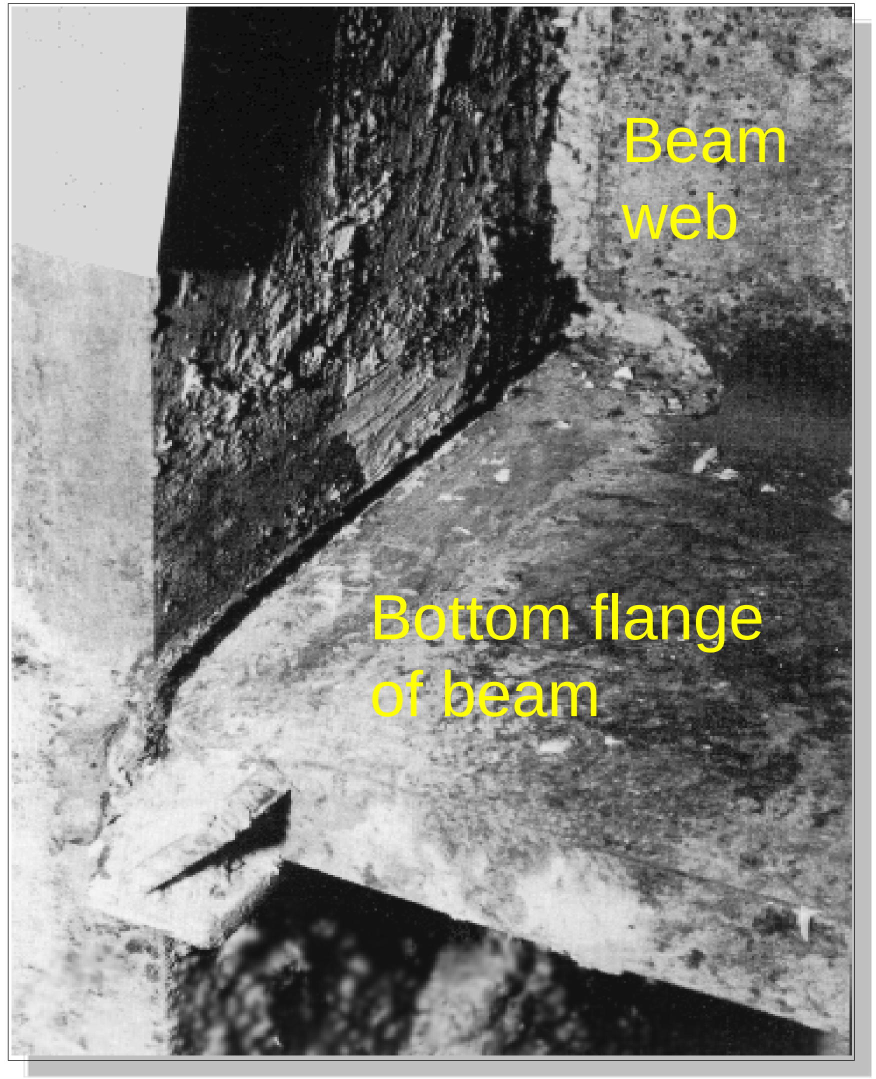
Following the 1994 Northridge earthquake, numerous failures of steel beam-to-column moment connections were identified. This led to a multiyear, multimillion dollar FEMA-funded problem-focused study undertaken by the SAC Joint Venture. The failures caused a fundamental rethinking of the design of seismic resistant steel moment connections.

Bottom Flange Weld Fracture Propagating Through Column Flange and Web

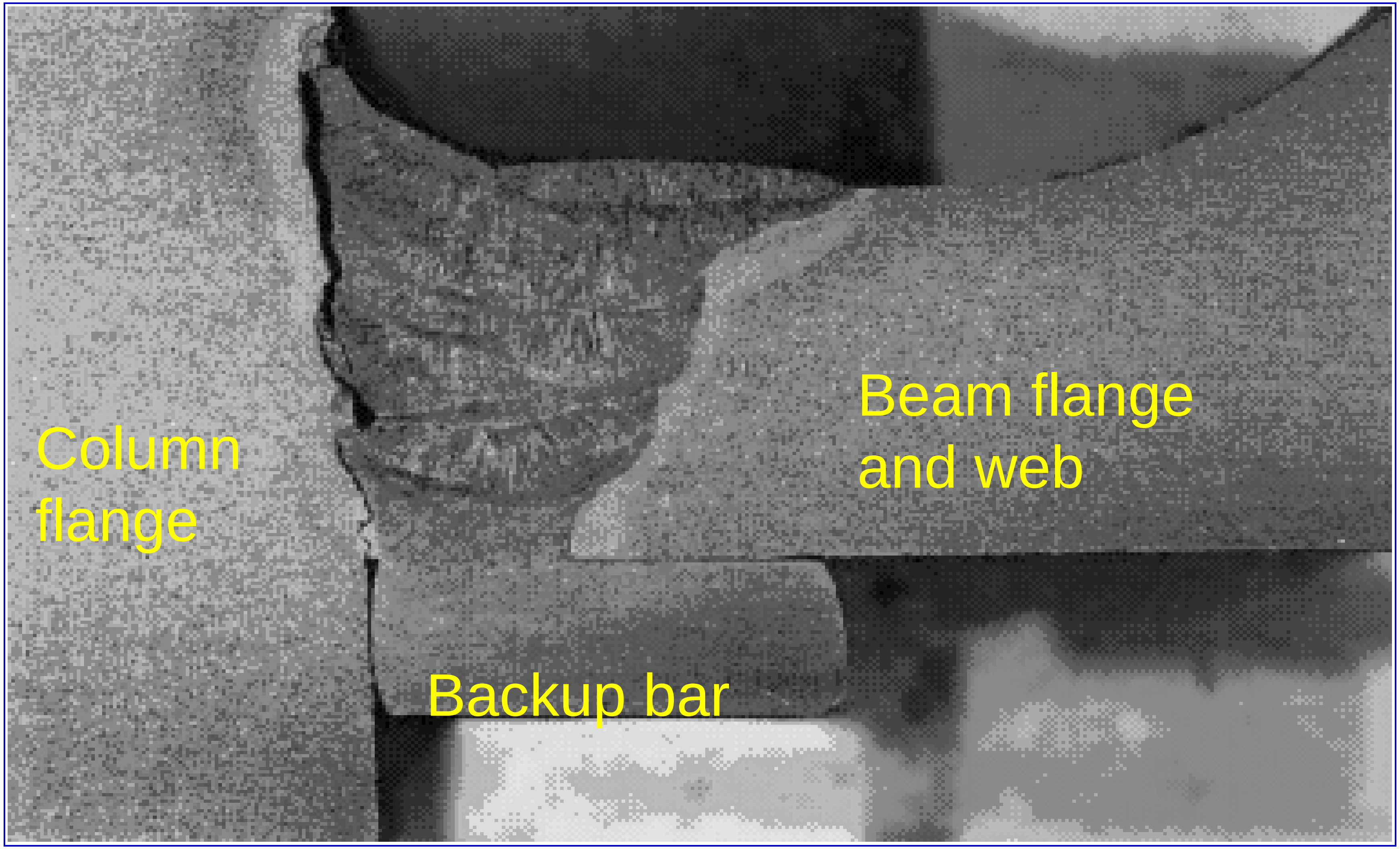
Beam Bottom Flange Weld Fracture Causing a Column Divot Fracture

Northridge Failure

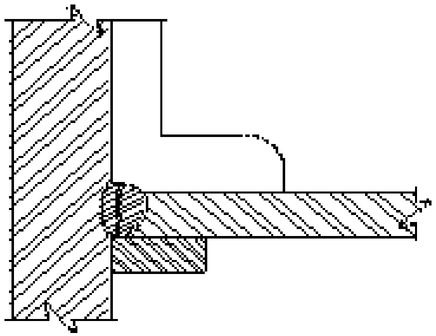
- Crack through weld
- Note backup bar and runoff tab



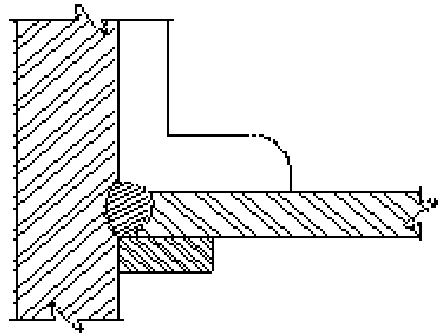
Northridge Failure



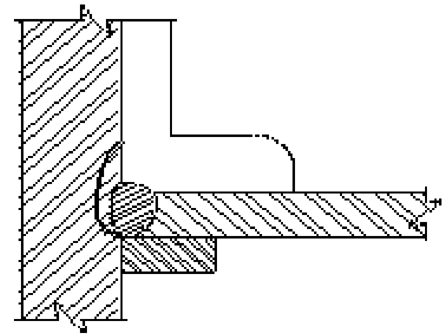
Northridge Failures



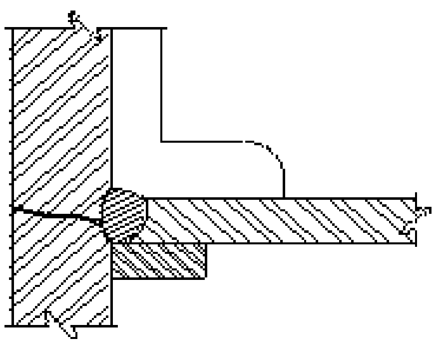
Weld



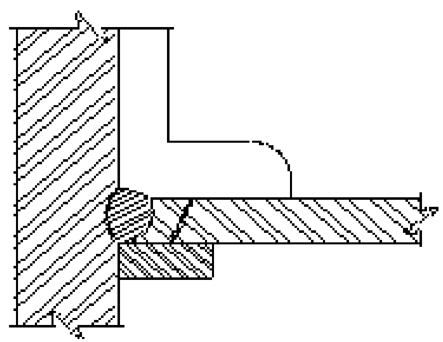
Weld Fusion



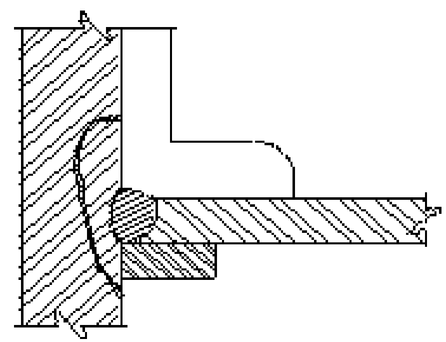
Column Divot



Column Flange

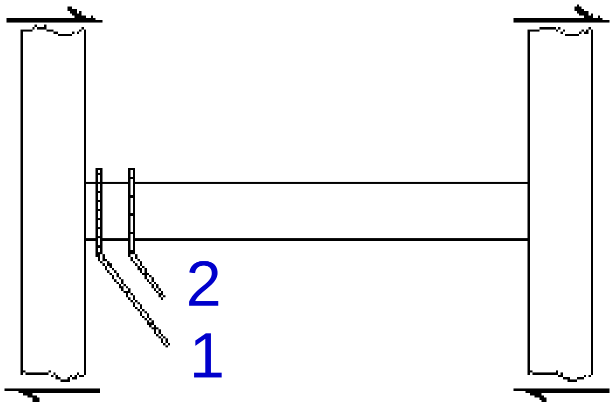


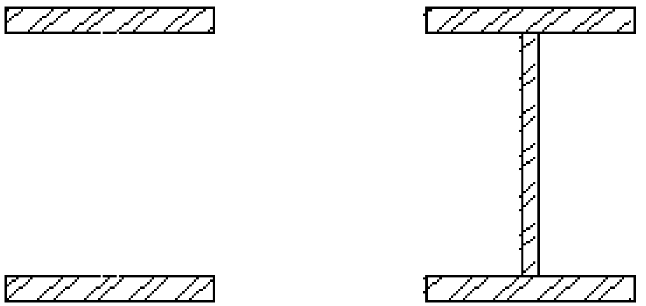
Heat Affected Zone



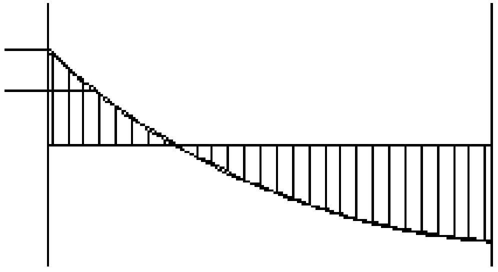
Lamellar Tear

Flexural Mechanics at a Joint

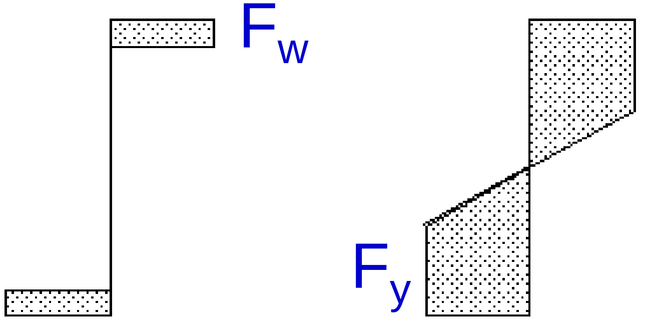




Cross Sections



Beam Moment

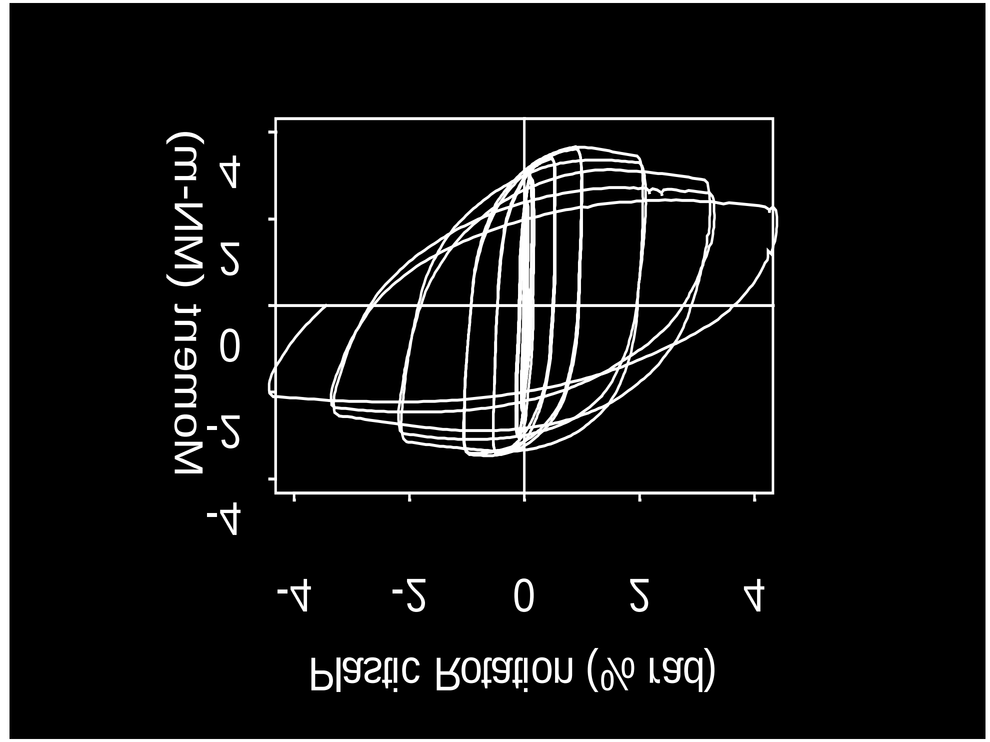


$$F_w \cdot Z_1 > F_y \cdot Z_2$$

Welded Steel Frames

- Northridge showed serious flaws. Problems correlated with:
  - Weld material, detail concept and workmanship
  - Beam yield strength and size
  - Panel zone yield
- Repairs and new design
  - Move yield away from column face
    (cover plates, haunches, reduced beam section)
  - Verify through tests
- SAC Project: FEMA Publications 350 through 354
- AISC 358

Reduced Beam Section (RBS) Test Specimen
SAC Joint Venture

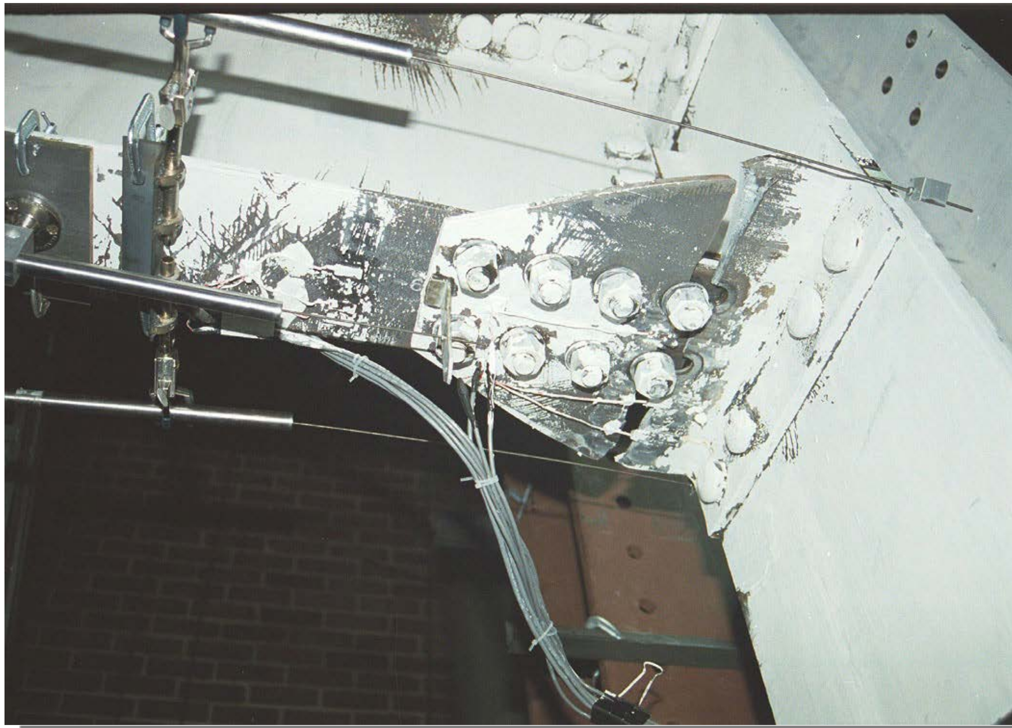


Graphics courtesy of Professor Chia-Ming Uang, University of California San Diego

T-stub Beam-Column Test SAC Joint Venture

Photo courtesy of Professor Roberto Leon, Georgia Institute of Technology

T-Stub Failure Mechanisms

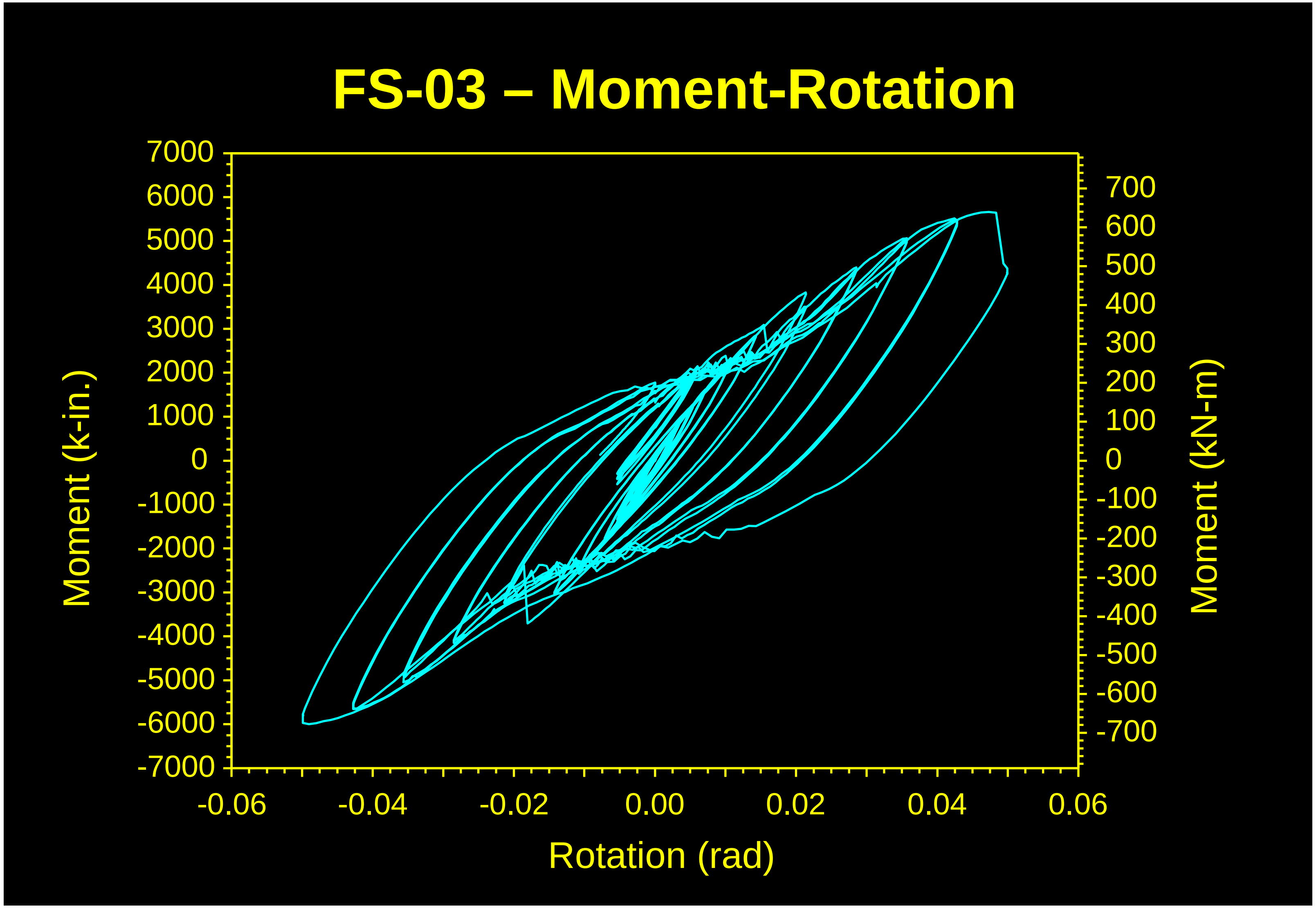


Net section fracture in stem of T-stub

Plastic hinge formation -- flange and web local buckling

Photos courtesy of Professor Roberto Leon, Georgia Institute of Technology

T-Stub Connection Moment Rotation Plot

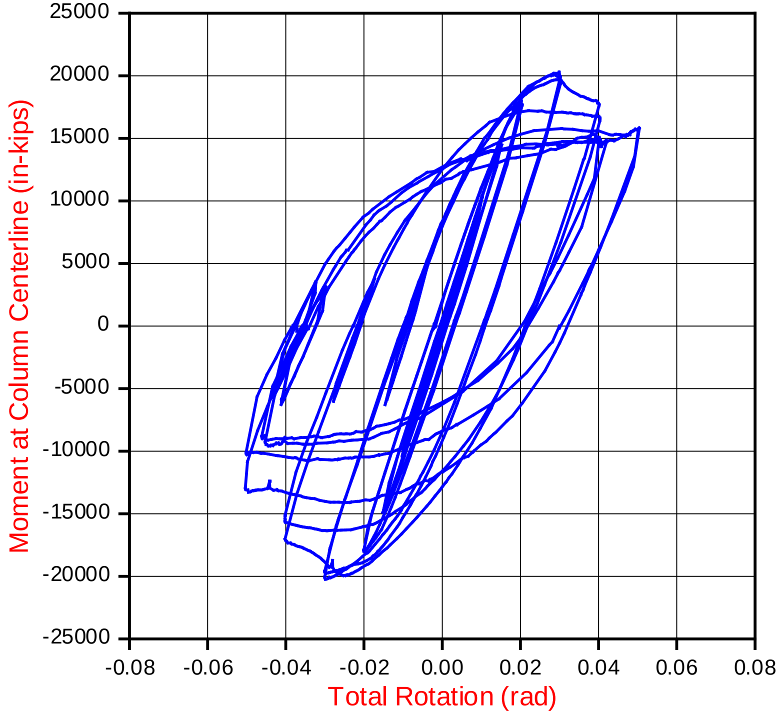


Graphic courtesy of Professor Roberto Leon, Georgia Institute of Technology

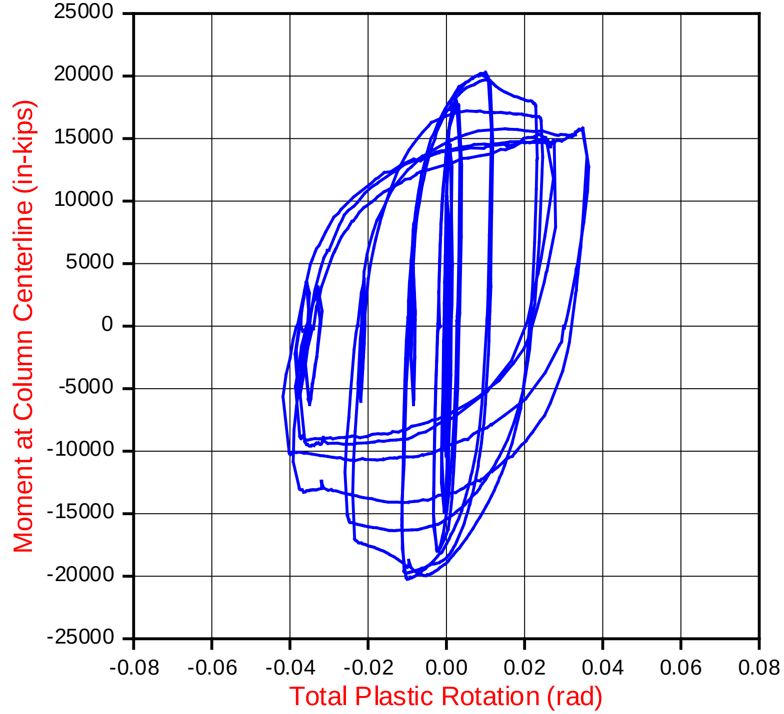
Extended Moment End-Plate Connection Res*ults*

Photo courtesy of Professor Thomas Murray, Virginia Tech

Extended Moment End-Plate Connection Results



(a) Moment vs Total Rotation



(b) Moment vs Plastic Rotation

Graphics courtesy of Professor Thomas Murray, Virginia Tech

Ductility of Steel Frame Joints

Limit States

Welded Joints

- Brittle fracture of weld
- Lamellar tearing of base metal
- Joint design, testing, and inspection

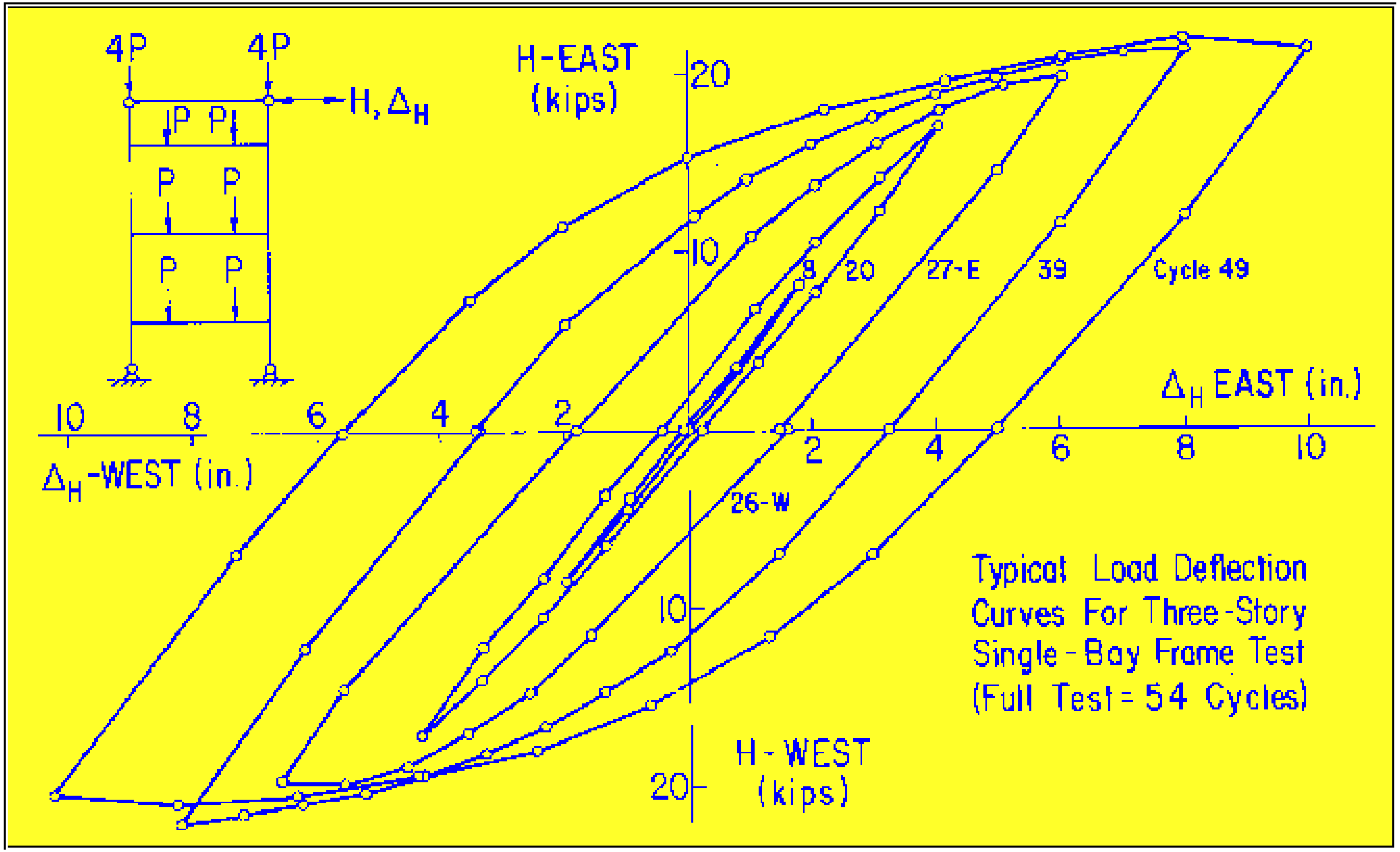
Bolted Joints

- Fracture at net cross-section
- Excessive slip

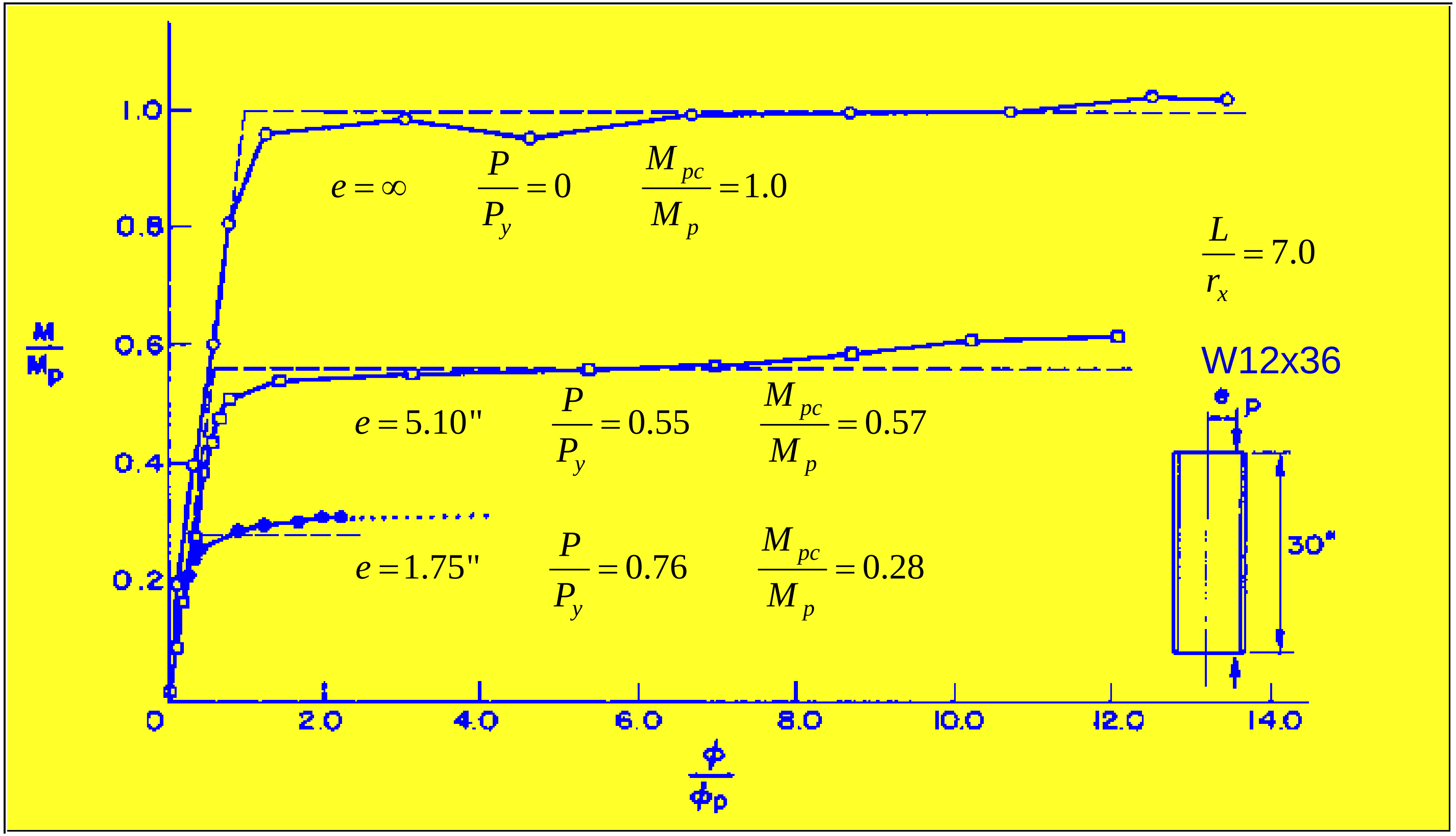
Joint Too Weak For Member

- Shear in joint panel

Multistory Frame
Laboratory Test

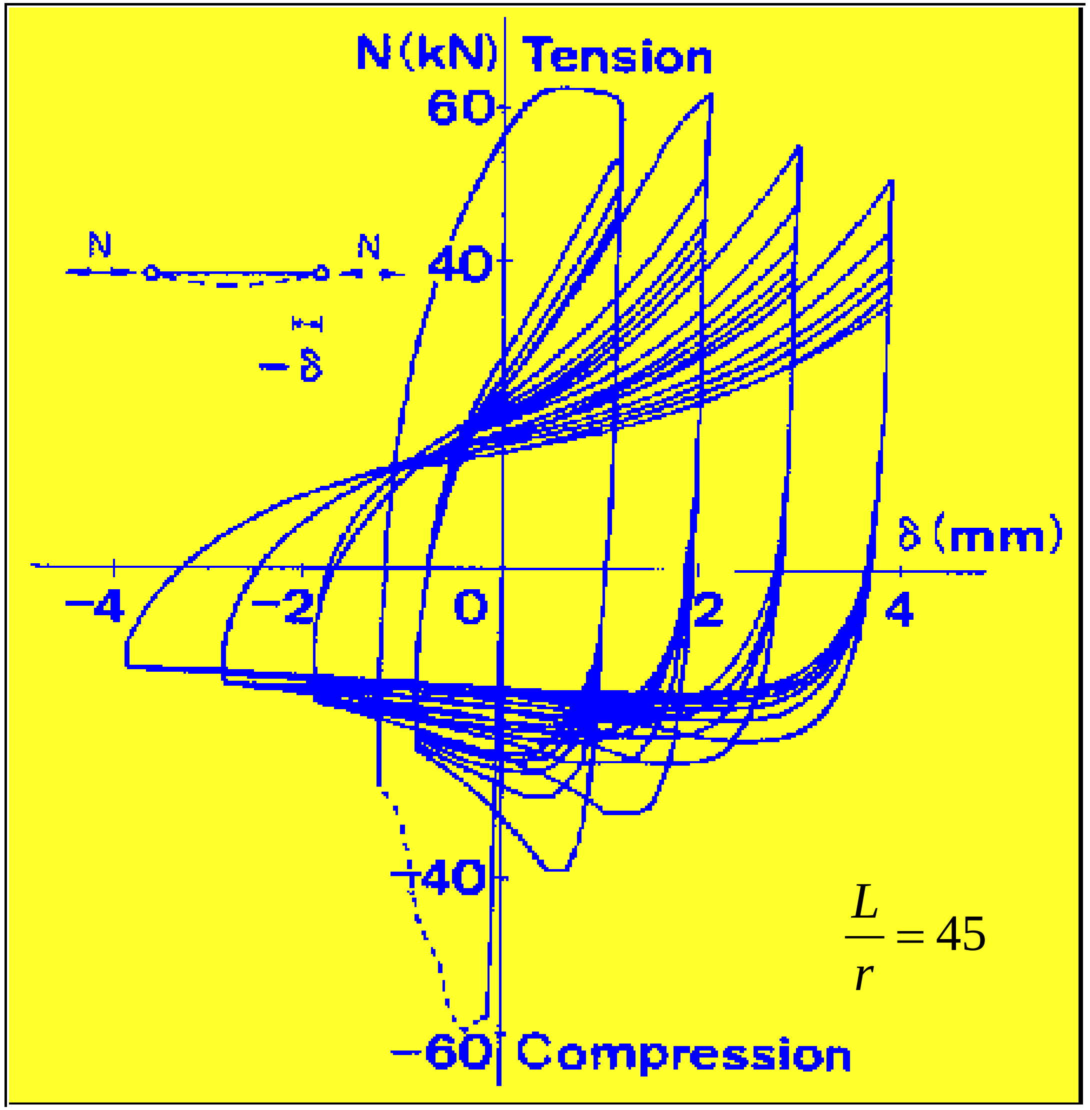


Flexural Ductility
Effect of Axial Load



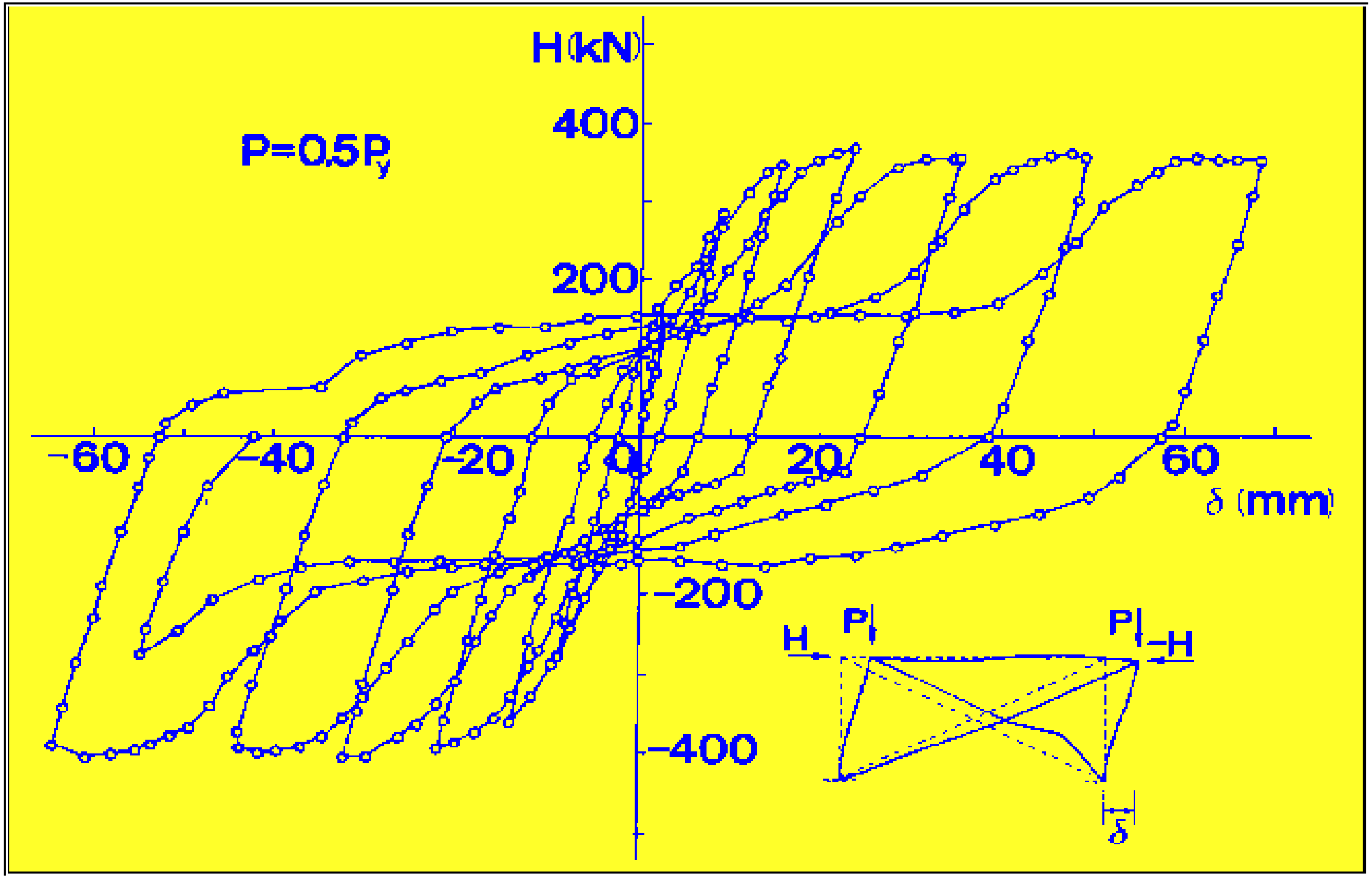
Axial Strut

Laboratory test



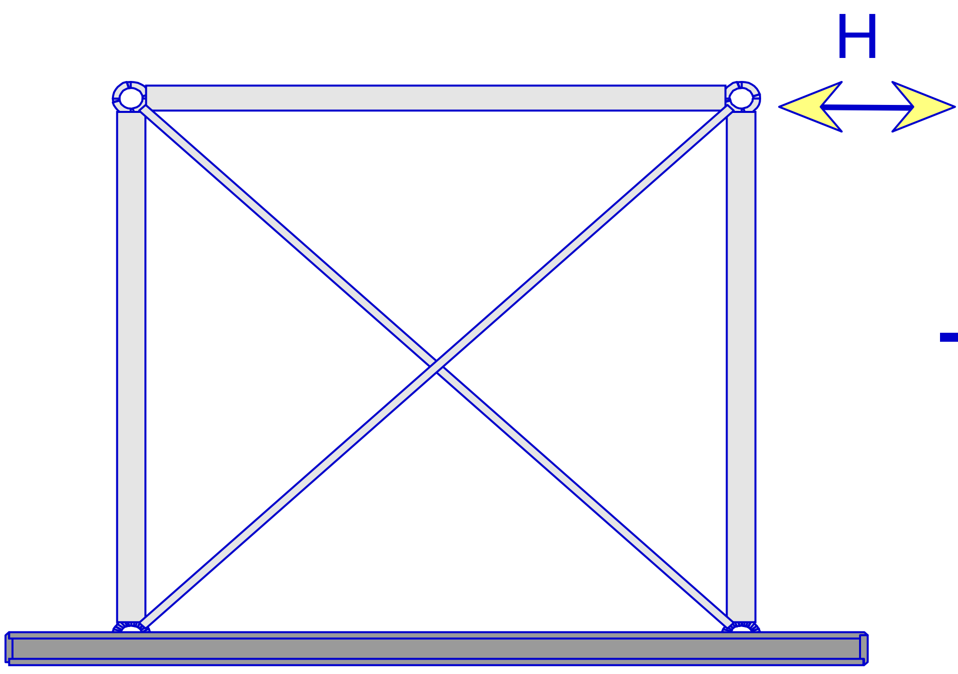
Cross Braced Frame

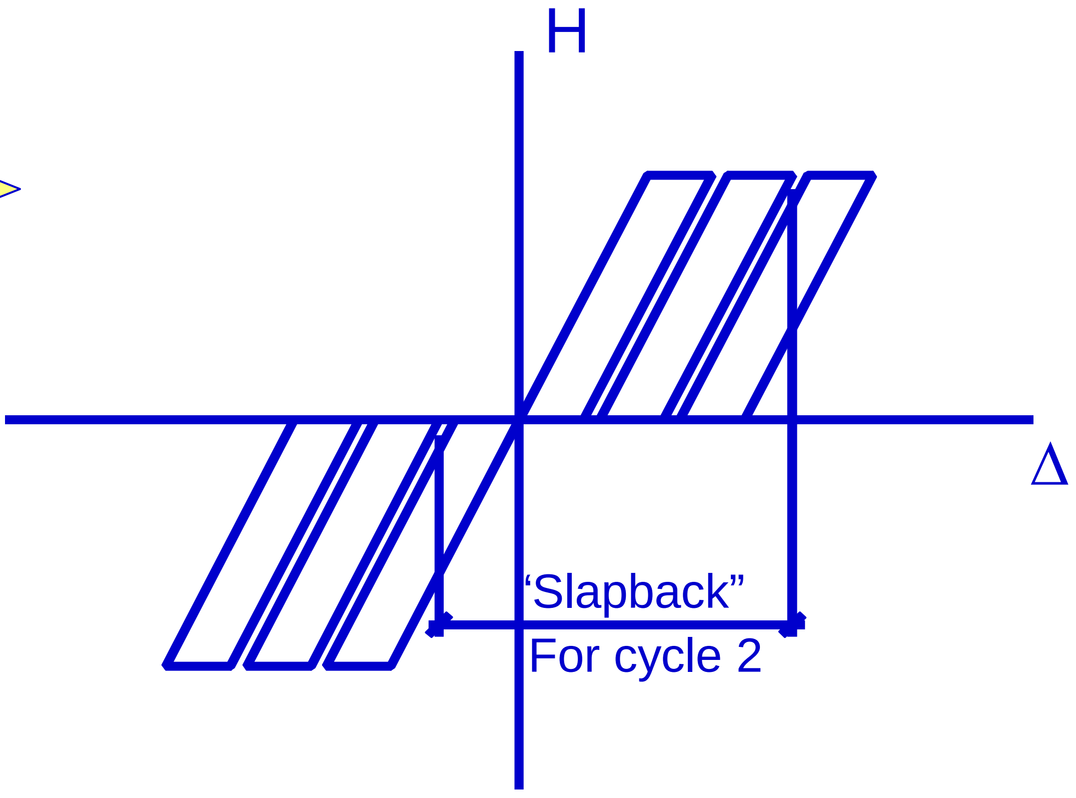
Laboratory test



Tension Rod (Counter) Bracing

Conceptual Behavior

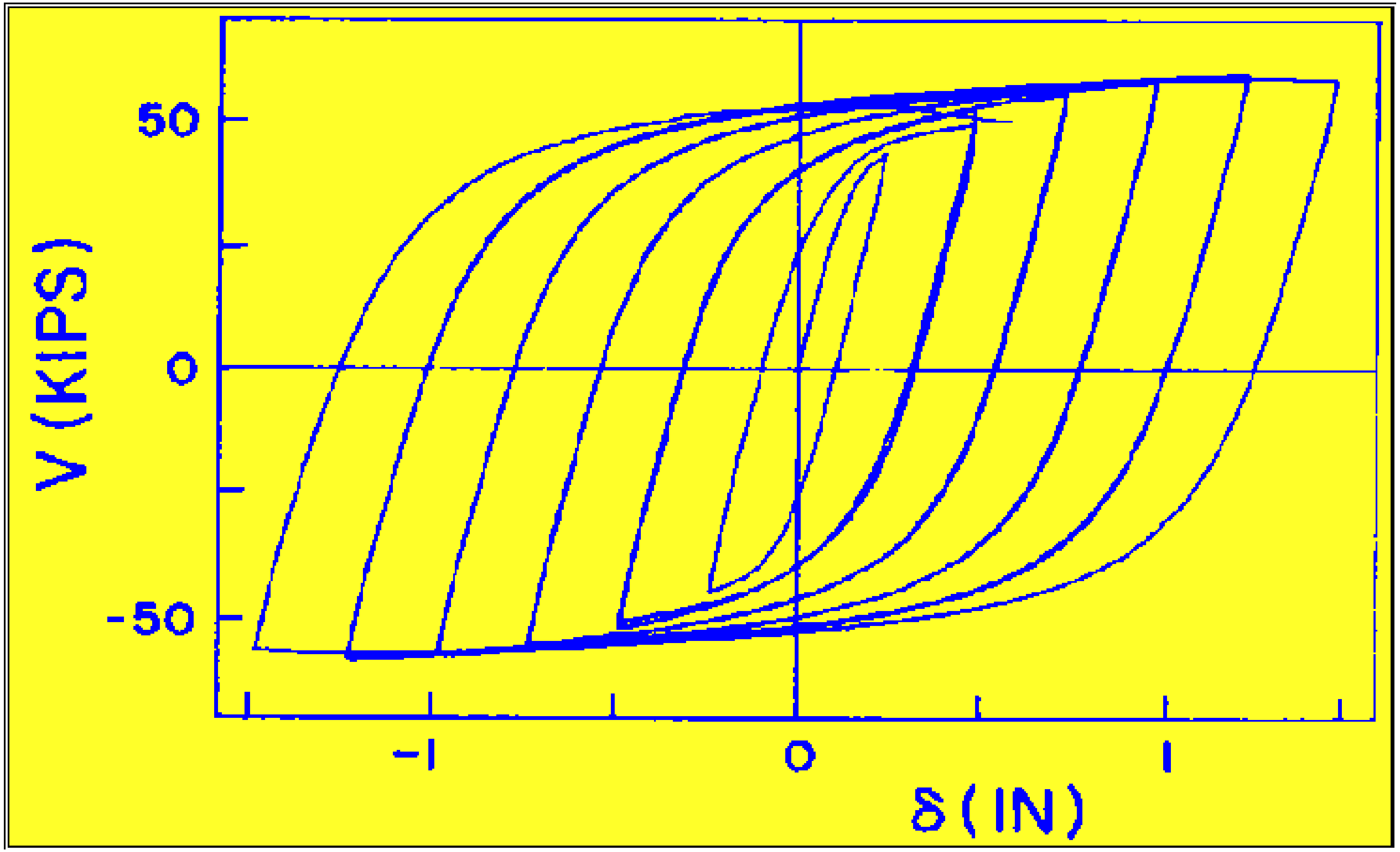




Eccentrically Braced Frame

Eccentrically Braced Frame

Lab test of link



Steel Behavior – Summary

- Ductility
  - Material inherently ductile
  - Ductility of structure < ductility of member < ductility of material
  - Achieved through detailing
- Damping
  - Welded structures have low damping
  - More damping in bolted structures due to slip at connections
  - Primary energy absorption is yielding of members

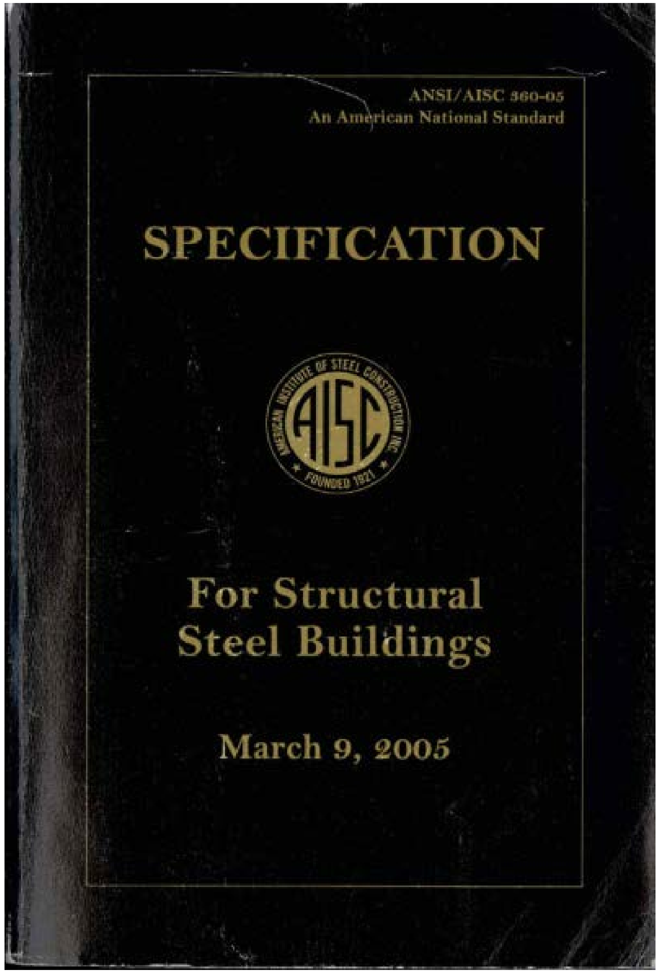
Steel Behavior – Summary

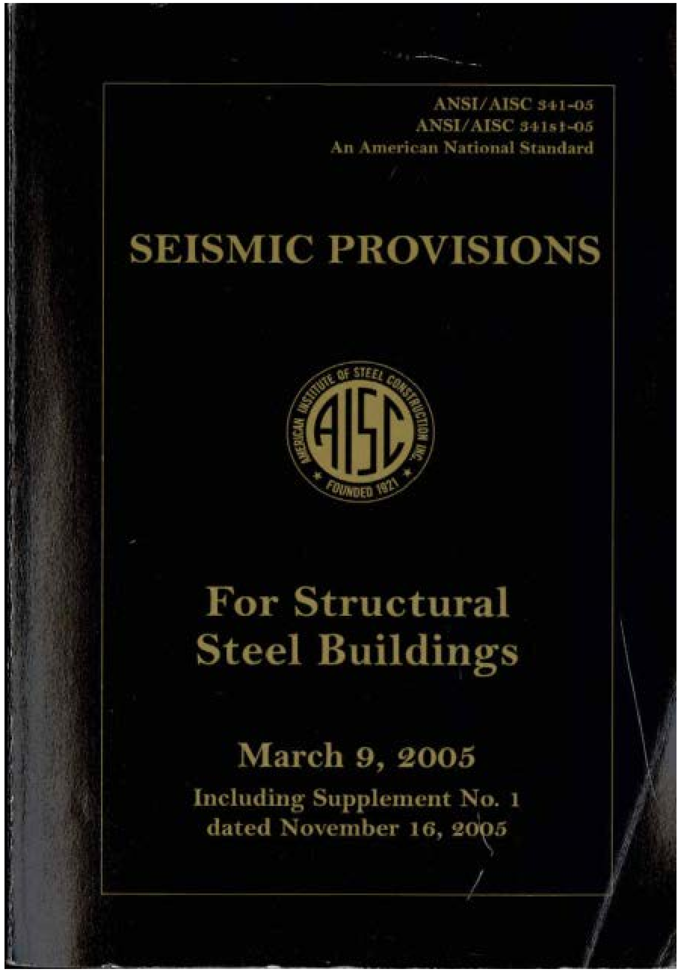
- Buckling
  - Most common steel failure under earthquake loads
  - Usually not ductile
  - Local buckling of portion of member
  - Global buckling of member
- Fracture
  - Nonductile failure mode under earthquake loads
  - Heavy welded connections susceptible
  - Net section rupture

NEHRP Recommended Provisions Steel Design

- Context in *Provisions*
- Steel behavior
- **Reference standards and design strength**

Steel Design Specifications

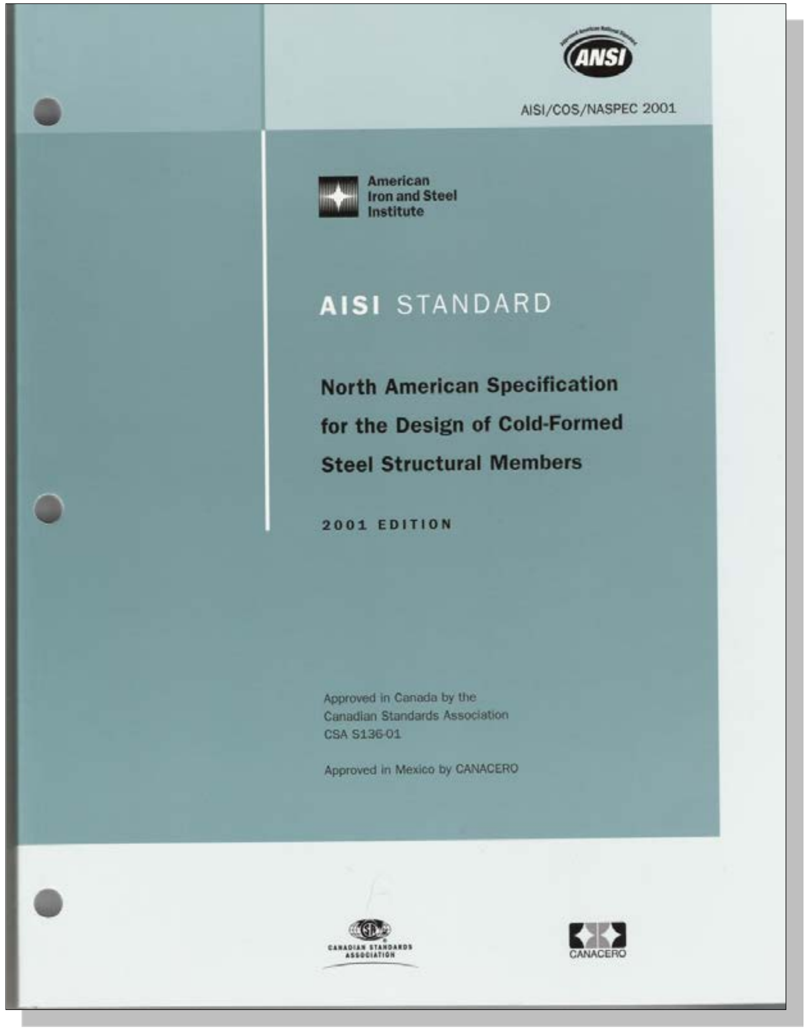




Using Reference Standards
Structural Steel

Both the AISC LRFD and ASD methodologies are presented in a unified format in both the *Specification for Structural Steel Buildings* and the *Seismic Provisions for Structural Steel Buildings*.

Cold Formed Steel Standard



Other Steel Members

Steel Joist Institute

Standard Specifications, 2002

Steel Cables

ASCE 19-1996

Steel Deck Institute

Diaphragm Design Manual, 3rd Ed., 2005

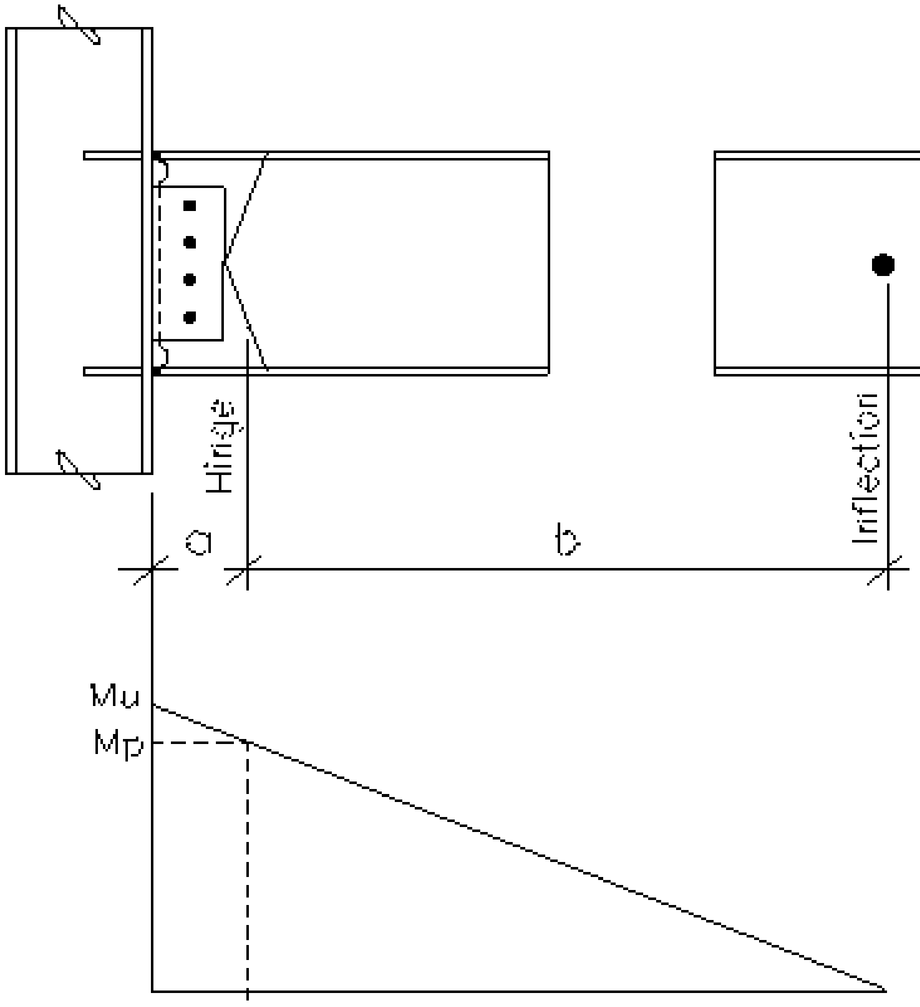
NEHRP Recommended Provisions
Steel Design

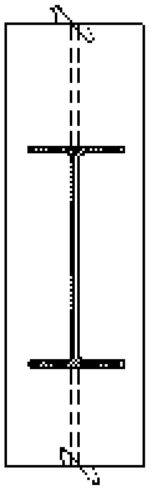
- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- **Moment resisting frames**

Steel Moment Frame Joints

Frame	Test	θ_i	Details
Special	Req'd	0.04	Many
Intermediate	Req'd	0.02	Moderate
Ordinary	Allowed	N.A.	Few

Steel Moment Frame Joints





$$M_u \approx M_p \cdot \frac{a+b}{b}$$

$$F_y^* = R_y \cdot F_y$$

$$F_u \approx F_y^* Z \cdot \frac{a+b}{b} \cdot \frac{1}{A_f d} \approx 1.7 F_y^*$$

Panel Zones

Special and intermediate moment frame:

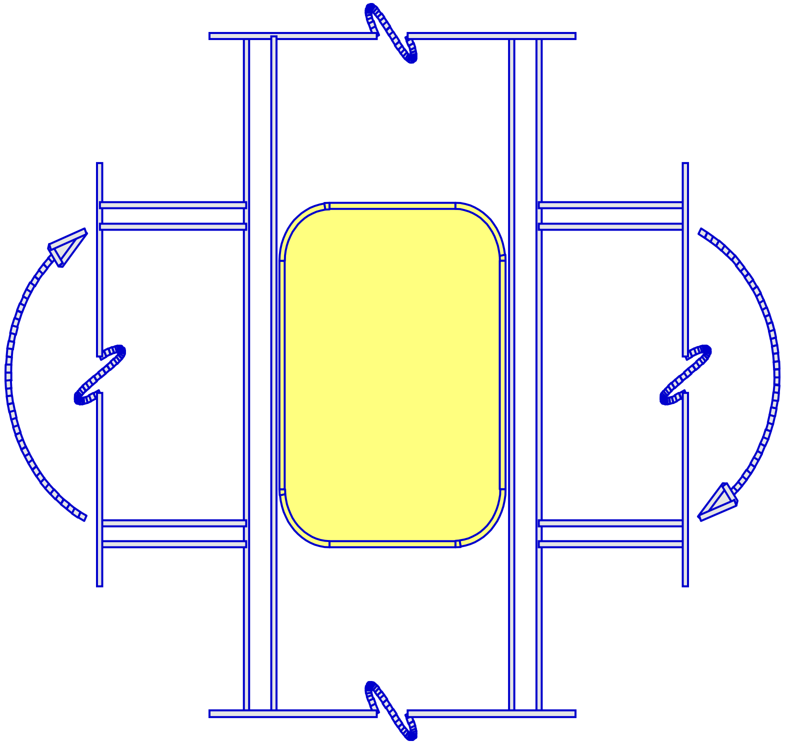
- Shear strength demand:

  Basic load combination

  or

  $\phi R_y M_p$ of beams

- Shear capacity equation
- Thickness (for buckling)
- Use of doubler plates (not economical, try to increase col. size instead)



Steel Moment Frames

- Beam shear: $1.1 R_y M_p$ + gravity
- Beam local buckling
  - Smaller *b*/*t* than for plastic design
- Continuity plates in joint per tests
- Strong column - weak beam rule
  - Prevent column yield except in panel zone
  - Exceptions: Low axial load, strong stories, top story, and non-SFRS columns

Steel Moment Frames

- Lateral support of column flange requirements
  - Top of beam if column elastic
  - Top and bottom of beam otherwise
  - Amplified forces for unrestrained

- Lateral support of beams requirements
  - Both flanges
  - Spacing < $0.086 r_y E/F_y$

Prequalified Connections

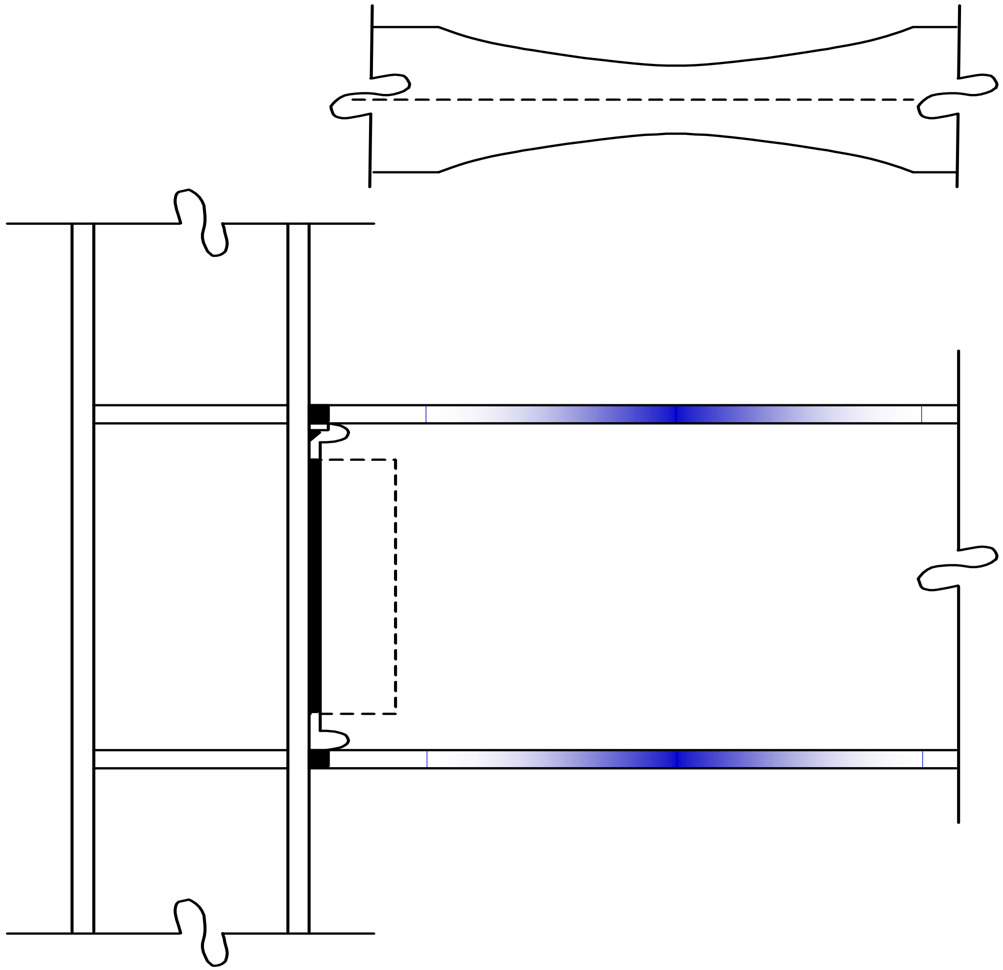
ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*

- Reduced Beam Section Connections
- Bolted Stiffened and Unstiffened Extended Moment End Plate Connections

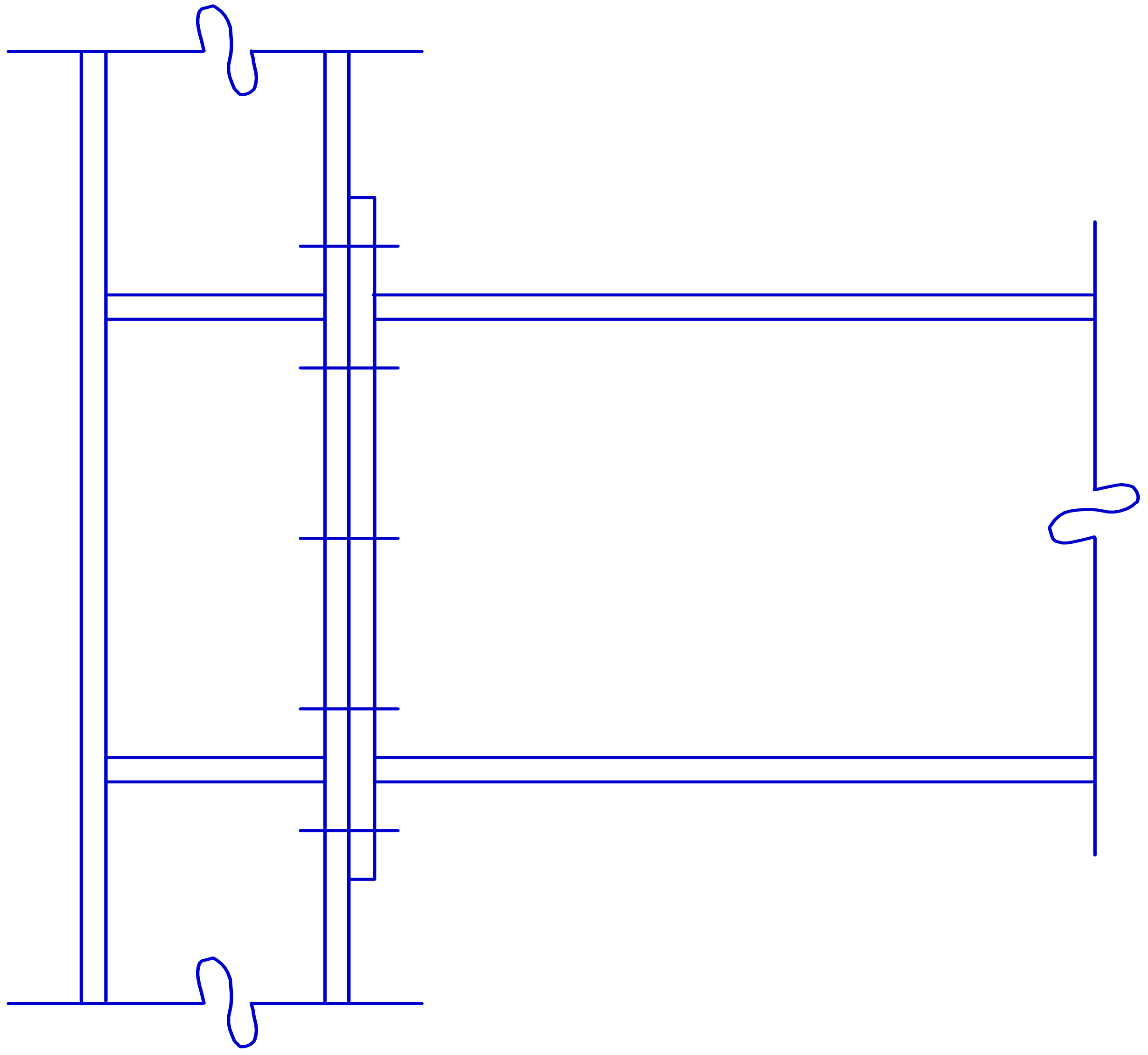
Additional connections addressed in FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*:

- Welded Unreinforced Flange
- Welded Free Flange Connection
- Welded Flange Plate Connection
- Bolted Flange Plate Connection

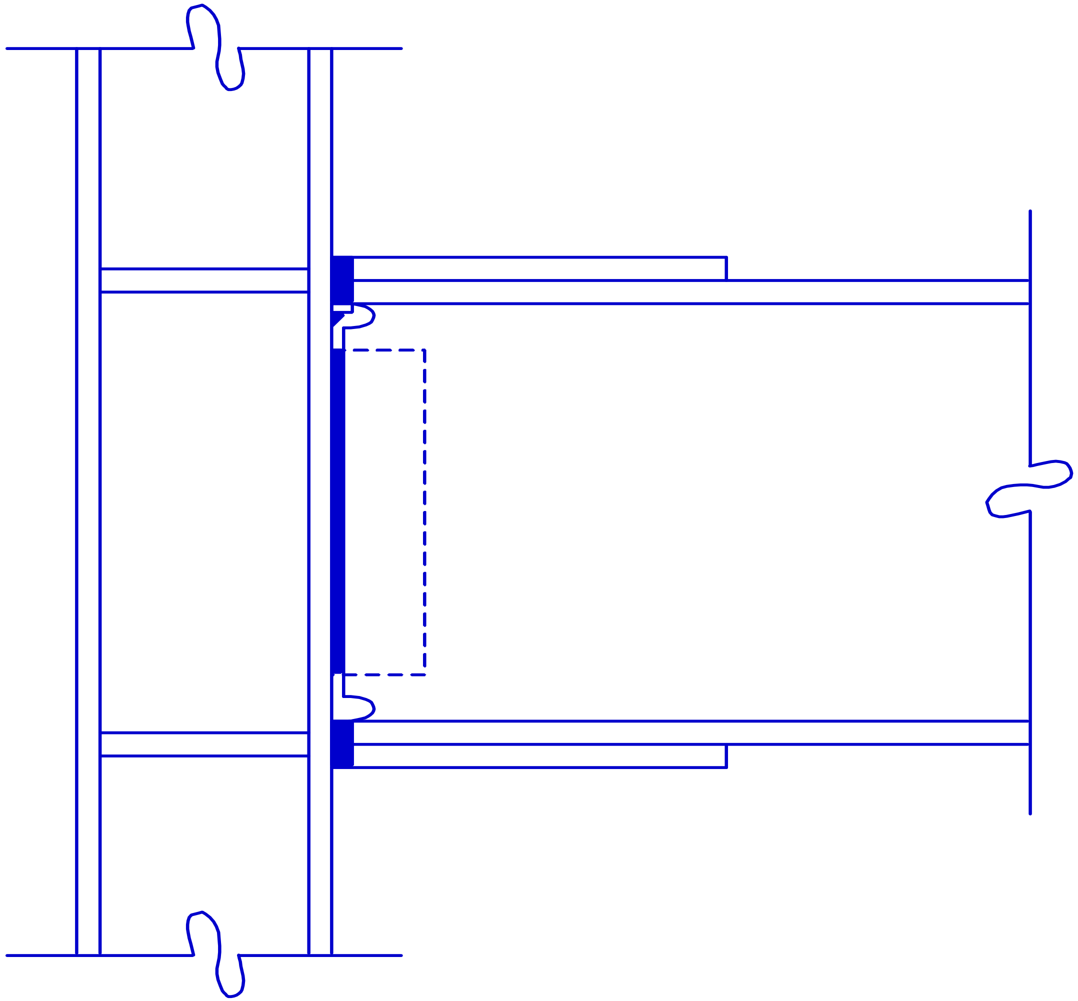
Reduced Beam Section (RBS)



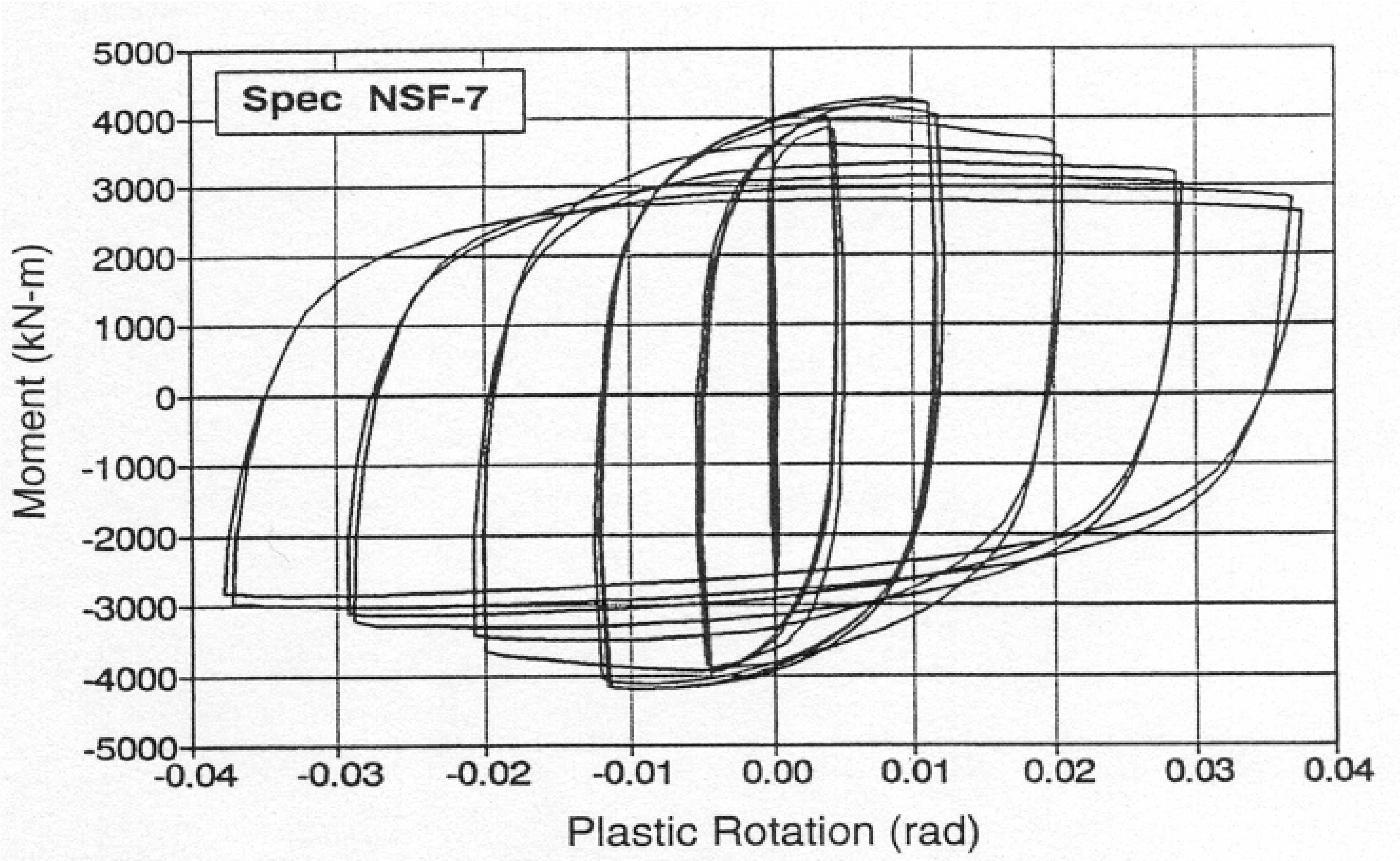
Extended End Plate



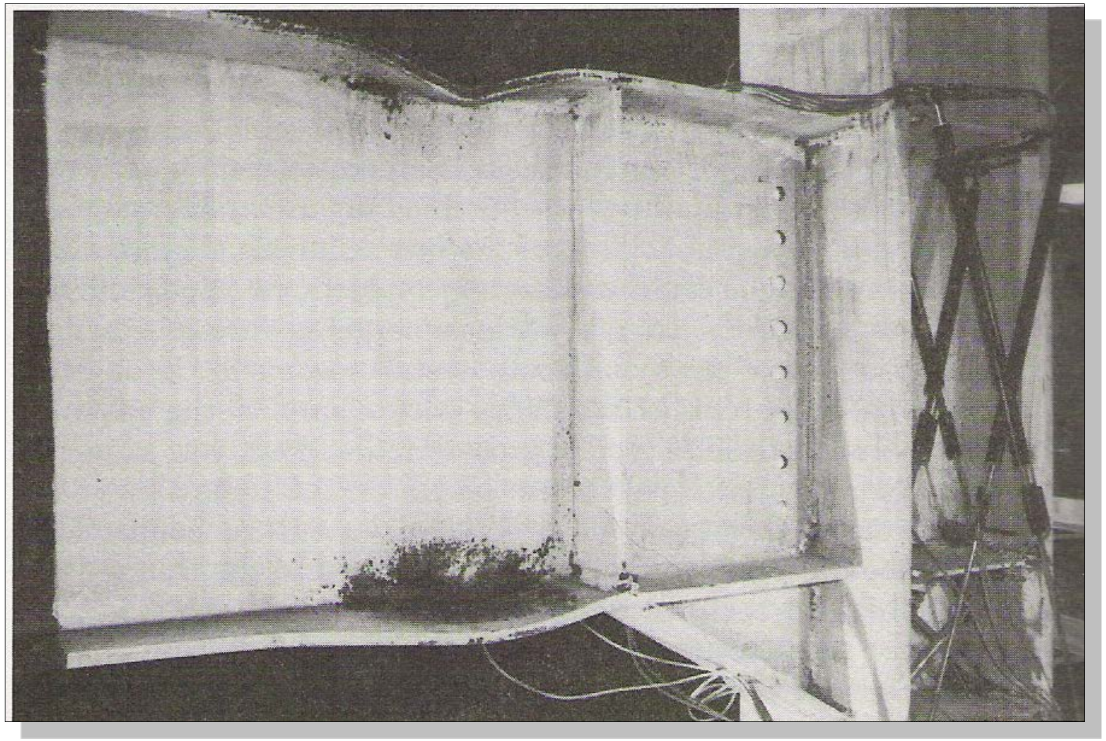
Welded Flange Plates

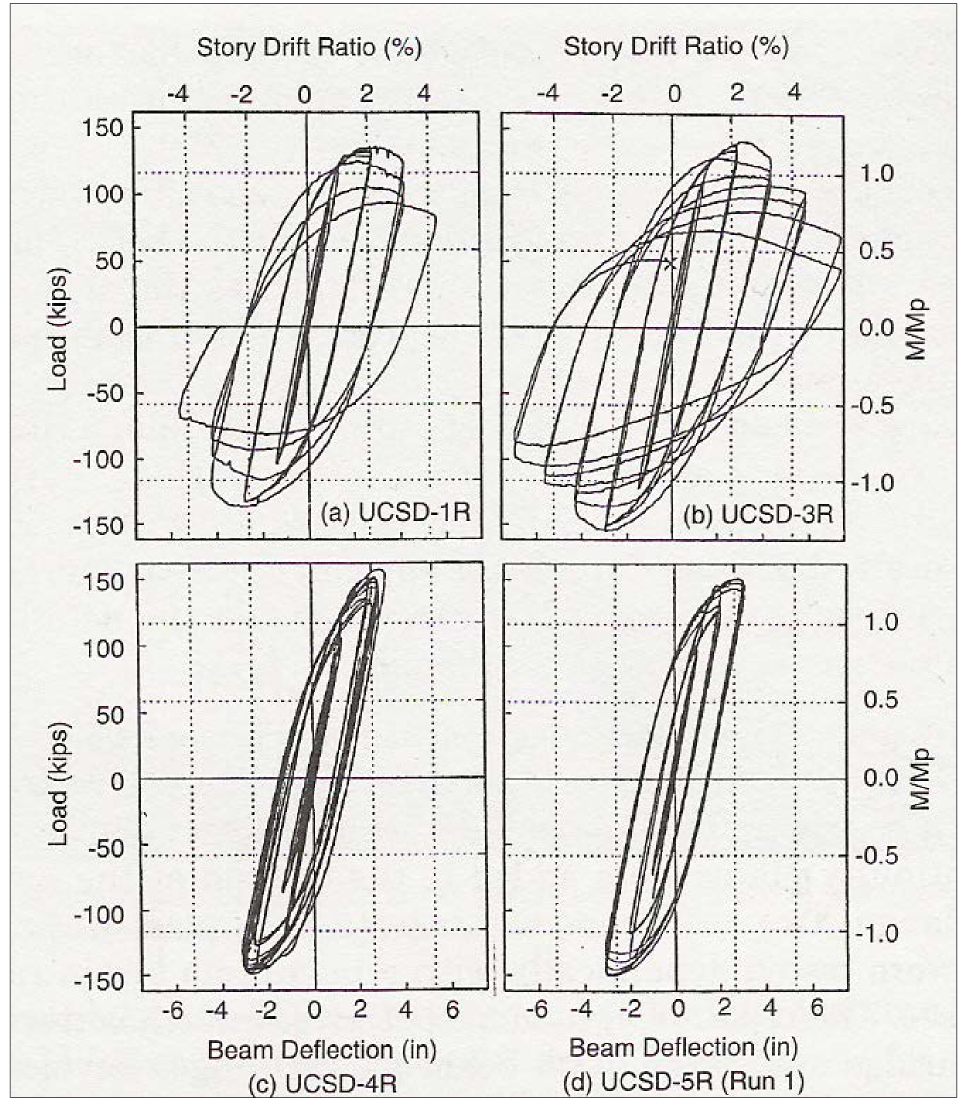


Excellent Moment Frame Behavior

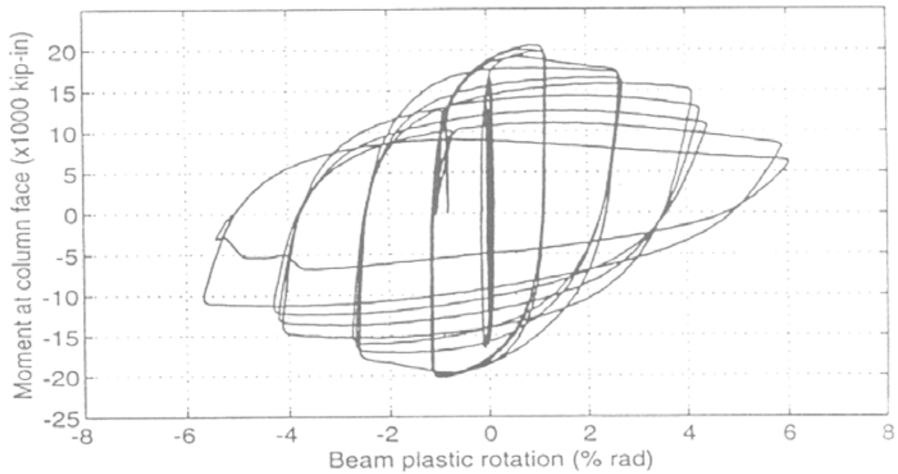


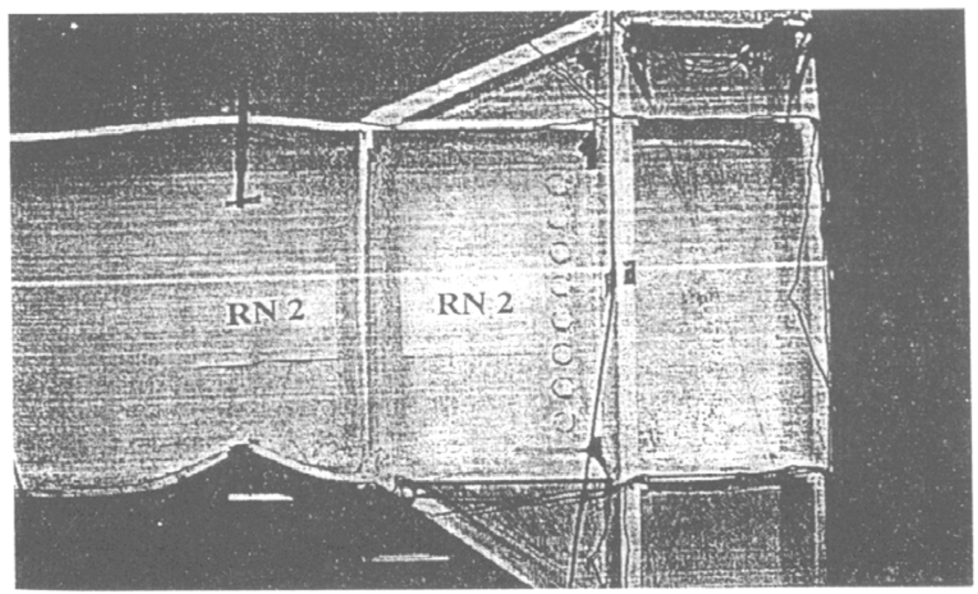
Excellent Moment Frame Behavior





Excellent Moment Frame Behavior





Special Moment Frames

Seven Story Office Building, Los Angeles

Perimeter Moment Frames, all bays engaged

S_{DS}=1.0

S_{D1}=0.6

Occupancy Category II

Seismic Design Category D

Design Parameters (Table 12.2-1)

- R=8
- C_d=5.5
- Ω_0=3.0

Special Moment Frame Example

Structural Materials:

Concrete (all floors) = 3.0 ksi lightweight

Other Concrete = 4.0 ksi normal weight

Steel:

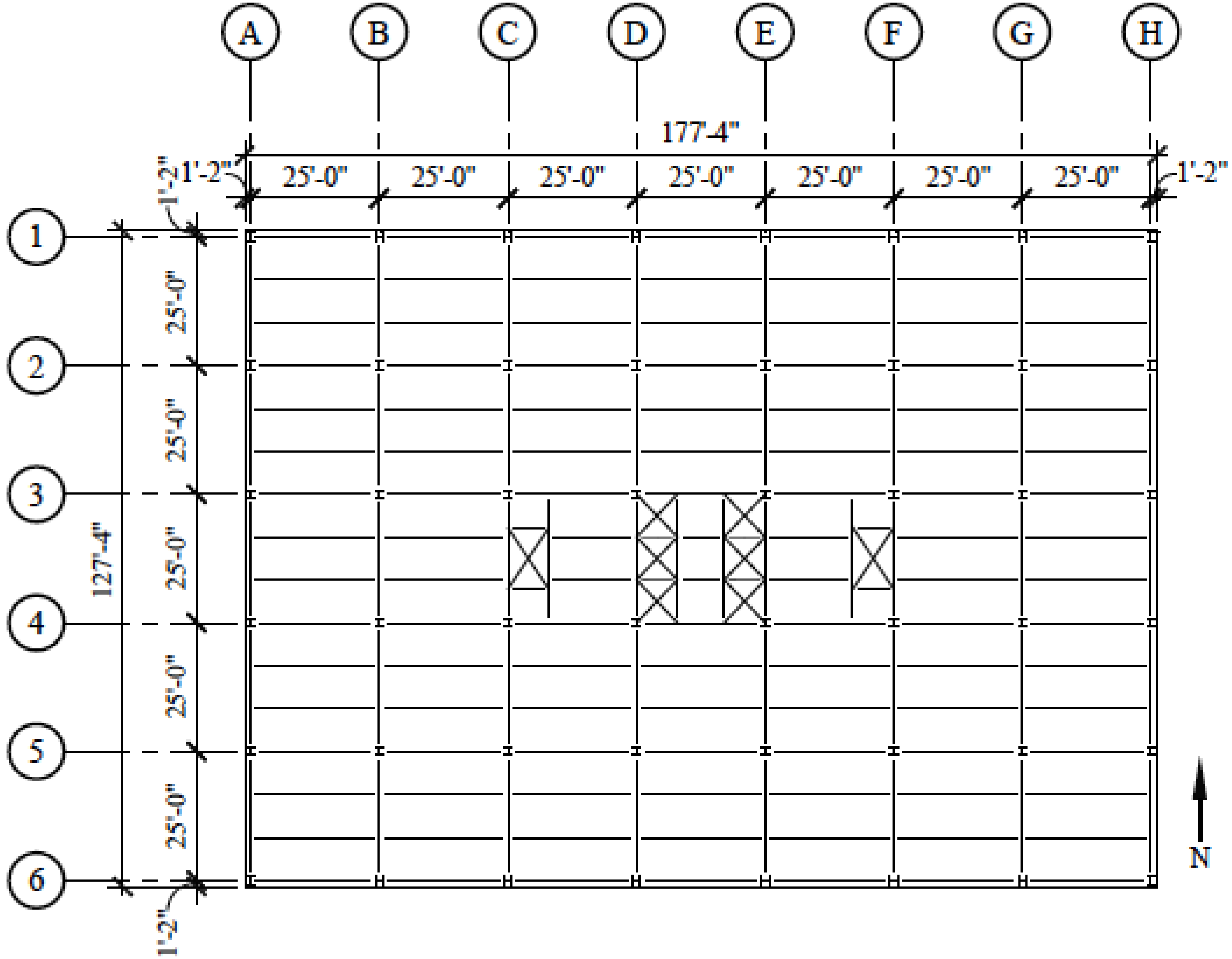
Wide Flange Sections= ASTM A992 Grade 50

HSS= ASTM A500 Grade B

Plates= ASTM A36

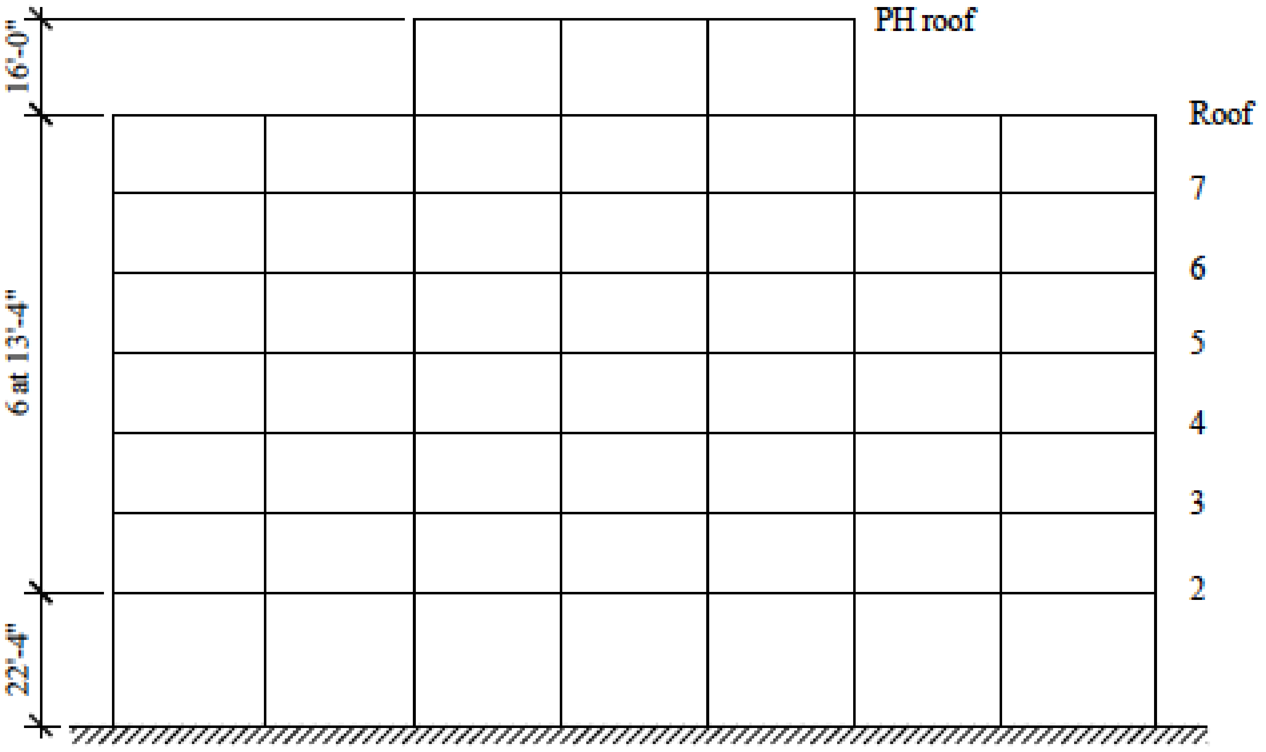
Special Moment Frames

Plan of Building



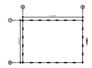
Special Moment Frames

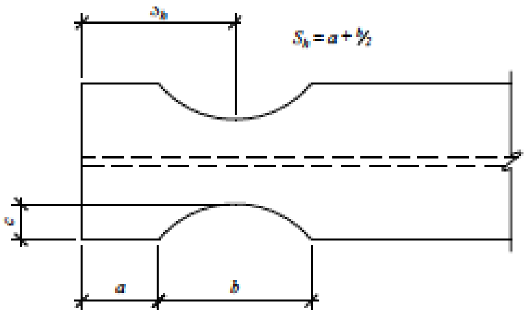
Elevation of Building



Special Moment Frames

Perimeter Moment Frames with RBS





Reduced Beam Section

System Designed

Special Moment Frames

The following design steps will be reviewed:

- Compute Lateral Loads
- Select preliminary member sizes
- Check member local stability
- Check deflection and drift
- Check torsional amplification
- Check the column-beam moment ratio rule
- Check shear requirement at panel zone
- Select connection configuration

Special Moment Frames

Building Weight:

Penthouse Roof = 94 kips
Lower Roof = 1,537 kips
Typical Floor = 1,920 kips
Total = 94 + 1,537 + 6(1,920) = 13,151 kips

Building Period:

$T_a = C_t h_n^x = (0.028)\ (102.3)^{0.8} = 1.14$ sec.
$T = C_u T_a = (1.4)(1.14) = 1.596$ sec.

Design Base Shear:

$C_s = S_{D1}/(T/(R/I) = 0.6/(1.596/(8/1)) = 0.047$ <<< CONTROLS
$C_{s,min} = 0.044\ S_{DS} I = 0.044(1.0)(1) = 0.044$
$V = C_s W = 0.047(13,151) = 618$ kips.

Special Moment Frames

Select preliminary member sizes – The preliminary member sizes are given in the next slide for the frame in the East-West direction. These members were selected based on the use of a 3-Dimensional model analyzed using the program *ETABS*. As will be discussed in a subsequent slide, the drift requirements controlled the design of these members.

SMF Example – Preliminary Member Sizes

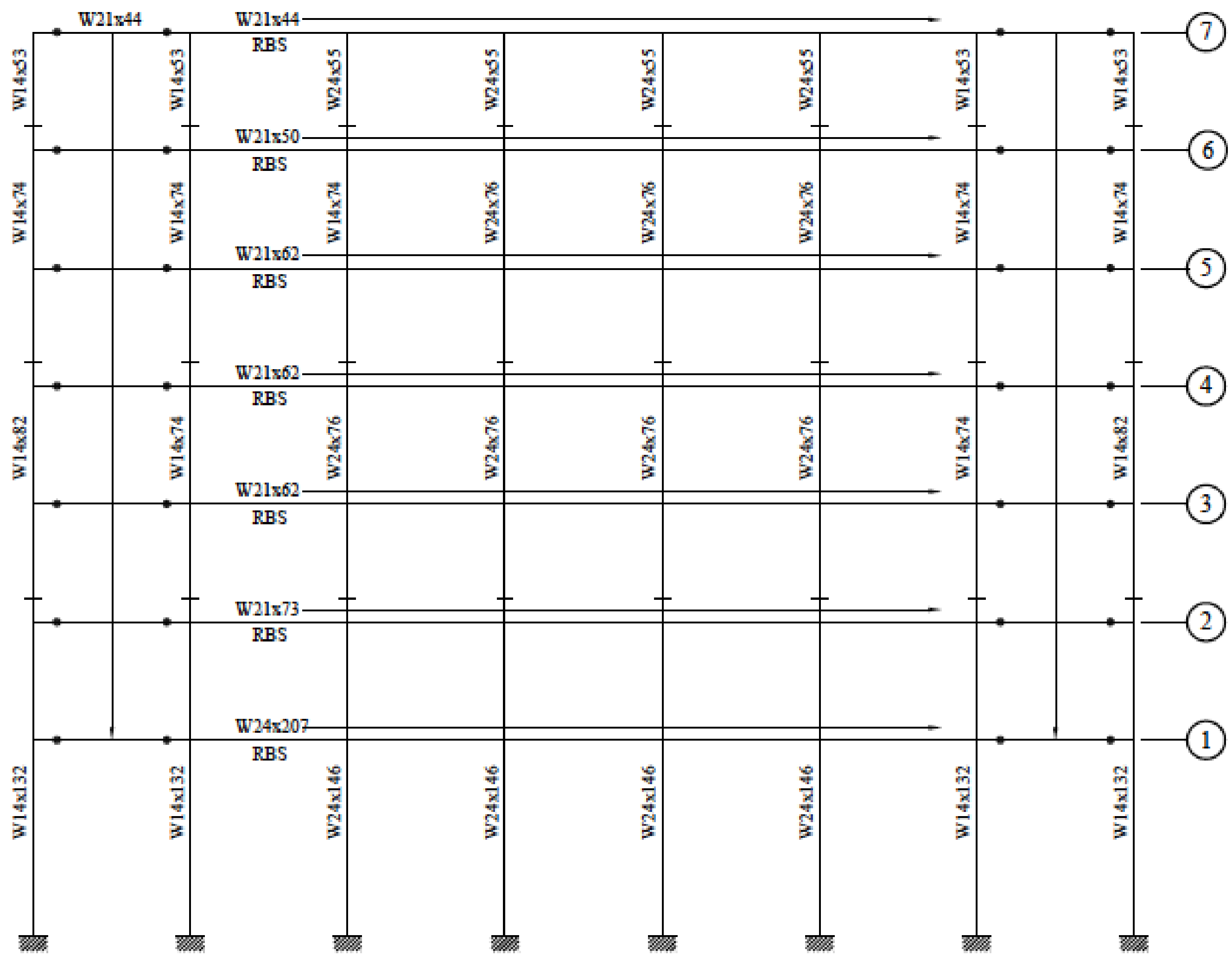


Figure 6.2-3 SMRF frame in E-W direction (penthouse not shown)

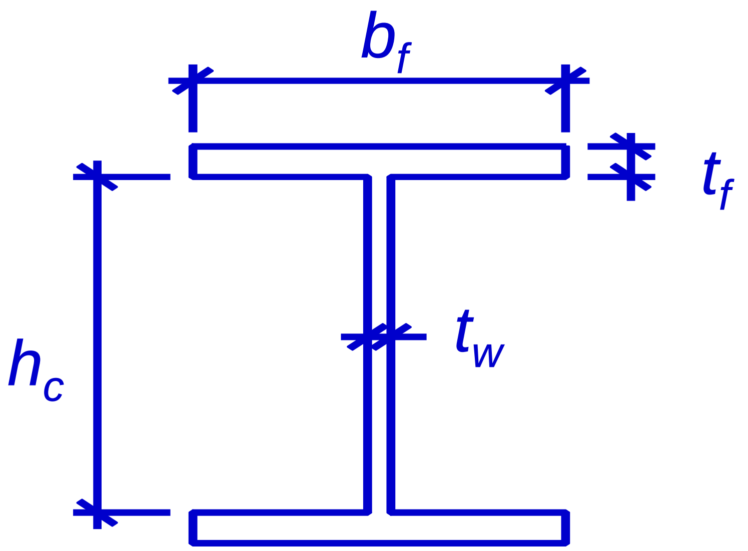
SMF Example – Check Member Local Stability

Check beam flange: $\frac{b_f}{2t_f} = 6.01$
(W33x141 A992)

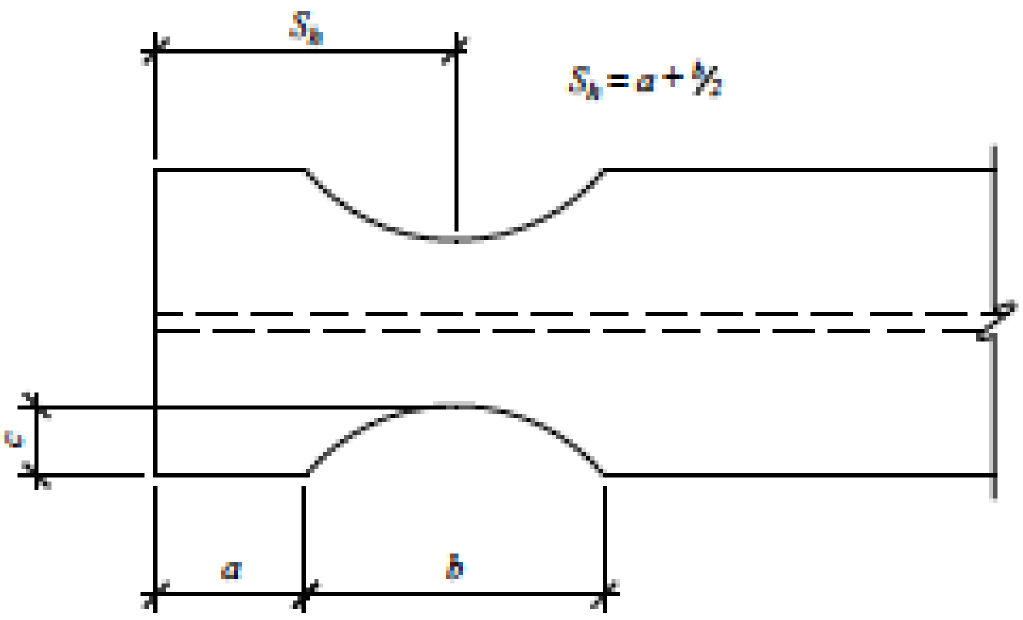
Upper limit: $0.3\sqrt{\frac{E}{F_y}} = 7.22$ OK

Check beam web: $\frac{h_c}{t_w} = 49.6$

Upper limit: $2.45\sqrt{\frac{E}{F_y}} = 59.0$ OK



SMF Example – RBS Details



a=0.625b_f

b=0.75d_b

c=0.20b_f

SMF Example – Check Deflection and Drift

The frame was checked for an allowable story drift limit of $0.020h_{sx}$. All stories in the building met the limit. Note that the *NEHRP Recommended Provisions* Sec. 4.3.2.3 requires the following check for vertical irregularity:

$$\frac{C_d \Delta_{x,story2}}{C_d \Delta_{x,story3}} = \frac{(1.2)}{(1.8)} = 0.67 < 1.3$$

Therefore, there is no vertical irregularity.

SMF Example – Check Deflection and Drift

Table 6.2-1 Alternative A (Moment Frame) Story Drifts under Seismic Loads

	Elastic Displacement at Building Corner, From Analysis		Expected Displacement ($=\delta_e C_d$)		Design Story Drift Ratio		Allowable Story Drift Ratio
Level	δ_eE-W (in.)	δ_eN-S (in.)	δ E-W (in.)	δ N-S (in.)	Δ E-W/h (%)	Δ N-S/h (%)	Δ/h (%)
Level 7	2.92	3.18	16.0	17.5	1.2	1.2	2.0
Level 6	2.66	2.89	14.7	15.9	1.4	1.7	2.0
Level 5	2.33	2.47	12.8	13.6	1.6	2.0	2.0
Level 4	1.91	1.95	10.5	10.7	1.9	2.0	2.0
Level 3	1.41	1.40	7.76	7.70	1.8	1.8	2.0
Level 2	0.90	0.88	4.96	4.85	1.2	1.2	2.0
Level 1	0.55	0.52	3.04	2.89	1.1	1.1	2.0

1.0 in. = 25.4 mm.

Building Satisfies Drift Limits

SMF Example – Check Torsional Amplification

The torsional amplification factor is given below. If $A_x < 1.0$ then torsional amplification is not required. From the expression it is apparent that if $\delta_{max} / \delta_{avg}$ is less than 1.2, then torsional amplification will not be required.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2$$

The 3D analysis results, as shown in FEMA P-751, indicate that none of the $\delta_{max} / \delta_{avg}$ ratios exceed 1.2; therefore, torsional amplification is not required.

SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_u/\phi P_n$ is typically less than 0.4 for the columns, combinations involving Ω_0 factors do not come into play for the special steel moment frames (re: AISC Seismic Sec. 8.3). In sizing columns (and beams) for strength one should satisfy the most severe value from interaction equations. However, the frame in this example is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

SMF Example – Column-Beam Moment Ratio

Per AISC Seismic Sec. 9.6

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

where ΣM^*_{pc} = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. ΣM^*_{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.

ΣM^*_{pb} = the sum of the moments in the beams at the intersection of the beam and column centerlines.

SMF Example – Column-Beam Moment Ratio

Column – W24x146; beam – W21x73

$$\Sigma M^{*}_{pc} = \Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right) + \frac{\left(M_{BF\,i} + M_{BF\,i+1} \right)}{h_c} \frac{d_b}{2}$$

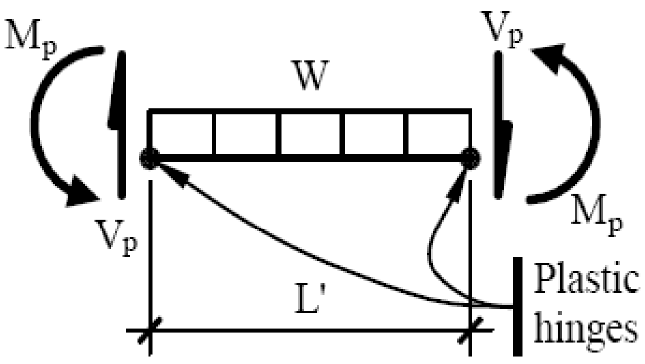
$$= 39400 \text{ in.-kips}$$

SMF Example – Column-Beam Moment Ratio

For beams:

$$\Sigma M^{*}_{pb} = \Sigma\left[M_{pr} + V_e\left(S_h + \frac{d_c}{2}\right)\right]$$

$$= 17749 \text{ in.-kips}$$



where

$M_{pr}=C_{pr}R_yF_y\,Z_e$

Z_e = effective Z (RBS)

V_e= 2 M_{pr} / L'

S_h= dist. from col. centerline to plastic hinge

SMF Example – Column-Beam Moment Ratio

The ratio of column moment strengths to beam moment strengths is computed as:

$$\text{Ratio} = \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = \frac{39400 \text{ in.-kips}}{17749 \text{ in.-kips}} = 2.22 > 1.0$$

Other ratios are also computed to be greater than 1.0

SMF Example – Panel Zone Check

The 2005 AISC Seismic specification is used to check the panel zone strength. Note that FEMA 350 contains a different methodology, but only the most recent AISC provisions will be used. From analysis shown in the NEHRP *Design Examples* volume (FEMA 451), the factored strength that the panel zone at Story 2 of the frame in the EW direction must resist is 794 kips. The shear transmitted to the joint from the story above, V_c, opposes the direction of R_u and may be used to reduce the demand. Previously calculated, this is 102 kips at this location. Thus the frame R_u = 794 – 102 = 692 kips.

$$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w} \right)$$

$$= 547 \text{ kips}$$

Since $\phi_v = 1$, $\phi_v R_b = 547$ kips, which is less than the required resistance, 692 kips. Therefore, doubler plates are required. The required additional strength from the doubler plates is 692 – 547 = 145 kips. The plates must be at least ¼" thick as the strength of the double plates is:

$$\phi_v R_n = 0.6 t_{doub} d_c F_y$$

SMF Example – Connection Configuration

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

(3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h \qquad (9\text{-}1)$$

SMF Example – Connection Configuration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 9.2a by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Appendix P.

(c) Provision of qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

Special Moment Frames
Summary

Beam-to-column connection capacity

Select preliminary member sizes

Check member local stability

Check deflection and drift

Check torsional amplification

Check the column-beam moment ratio rule

Check shear requirement at panel zone

Select connection configuration

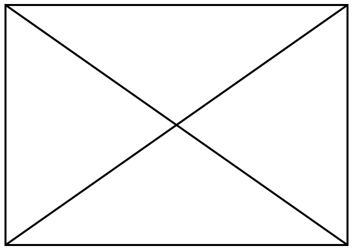
- Prequalified connections
- Testing

NEHRP Recommended Provisions
Steel Design

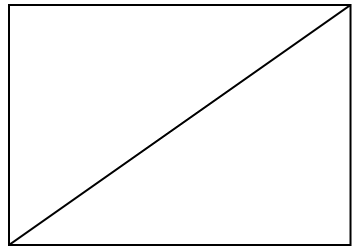
- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Seismic design category requirement
- Moment resisting frames
- **Braced frames**

Concentrically Braced Frames

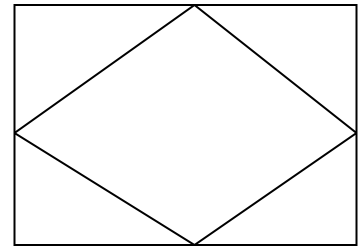
Basic Configurations



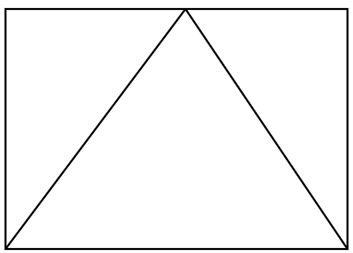
X



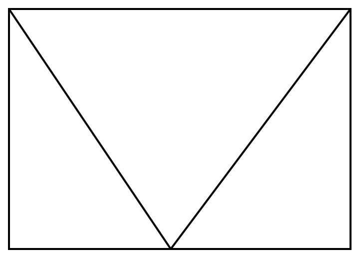
Diagonal



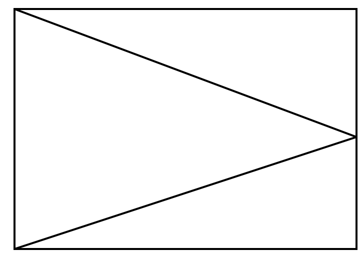
K



Inverted V

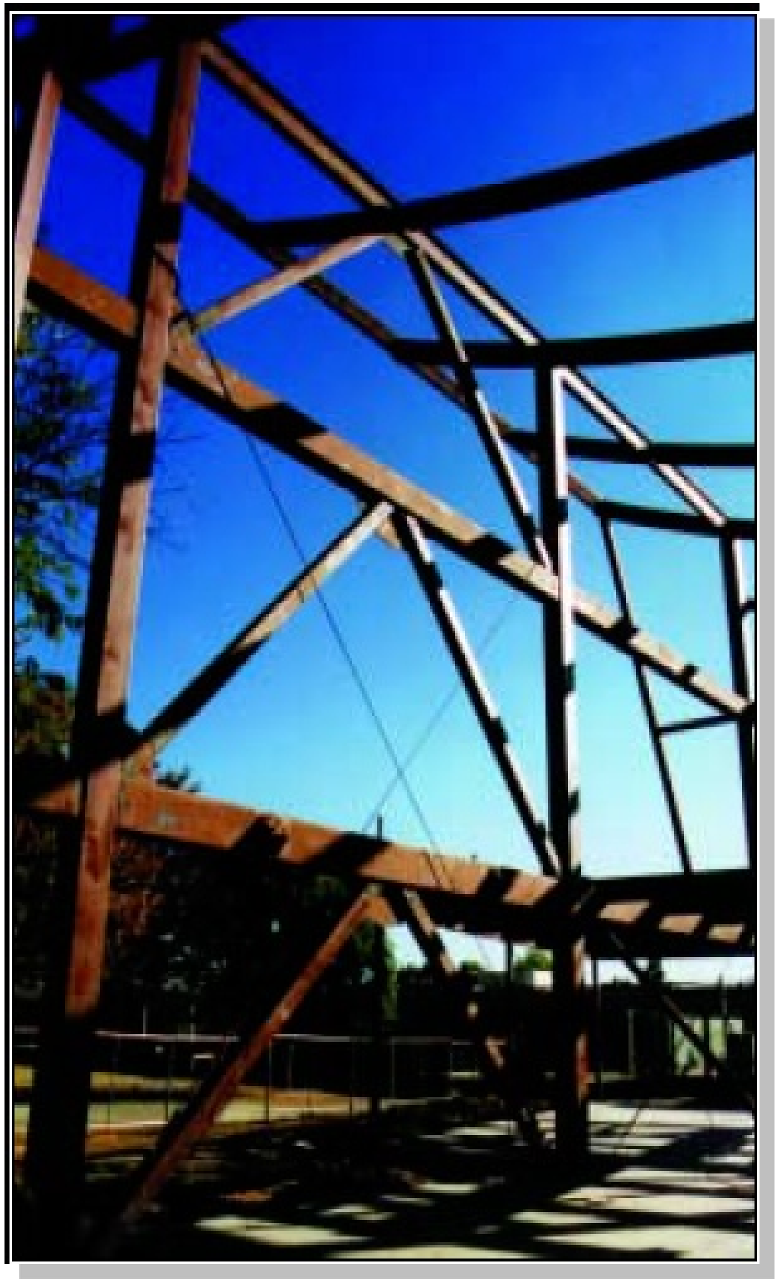


V



K

Braced Frame Under Construction



Braced Frame Under Construction

Concentrically Braced Frames

Special AISC Seismic	R = 6
Chapter 13	
Ordinary AISC Seismic	R = 3.25
Chapter 14	
Not Detailed for Seismic	R = 3
AISC LRFD	

Concentrically Braced Frames

Dissipate energy after onset of global buckling by avoiding brittle failures:

- Minimize local buckling
- Strong and tough end connections
- Better coupling of built-up members

Concentrically Braced Frames

Special and Ordinary

Bracing members:

- Compression capacity = $\phi_c P_n$
- Width / thickness limits

  Generally compact

  Angles, tubes and pipes very compact
- Overall $\frac{KL}{r} < 4\sqrt{\frac{E}{F_y}}$ < 200 for SCBF
- Balanced tension and compression

Concentrically Braced Frames

Special concentrically braced frames

<u>Brace connections</u>

Axial tensile strength > smallest of:

- Axial tension strength = $R_y F_y A_g$
- Maximum load effect that can be transmitted to brace by system

Axial compressive strength ≥ $1.1 R_y P_n$, where P_n is the nominal compressive strength of the brace.

Flexural strength > $1.1 R_y M_p$ <u>or</u> rotate to permit brace buckling while resisting $A_g F_{CR}$

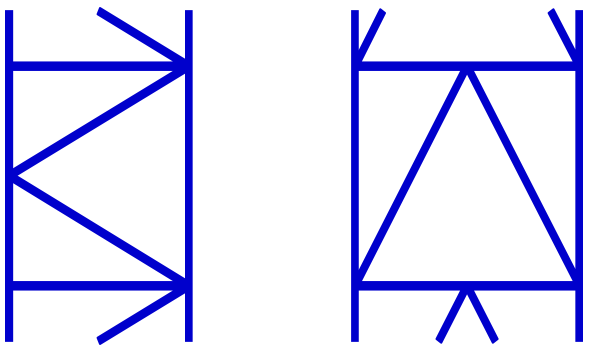
Concentrically Braced Frames

V bracing:

- Design beam for D + L + unbalanced brace forces, using $0.3P_c$ for compression and $R_yF_yA_g$ in tension
- Laterally brace the beam
- Beams between columns must be continuous

K bracing:

- Not permitted



Concentrically Braced Frames

Built-up member stitches:

- Spacing < 40% KL/r
- No bolts in middle quarter of span
- Minimum strengths related to P_y

Column in CBF:

- Same local buckling rules as brace members
- Splices resist moments

Special Concentrically Braced Frame Example

Seven Story Office Building, Los Angeles

Perimeter Moment Frames, all bays engaged

S_{DS}=1.0

S_{D1}=0.6

Occupancy Category II

Seismic Design Category D

Design Parameters (Table 12.2-1)

- R=6
- C_d=5.0
- Ω_0=2.0

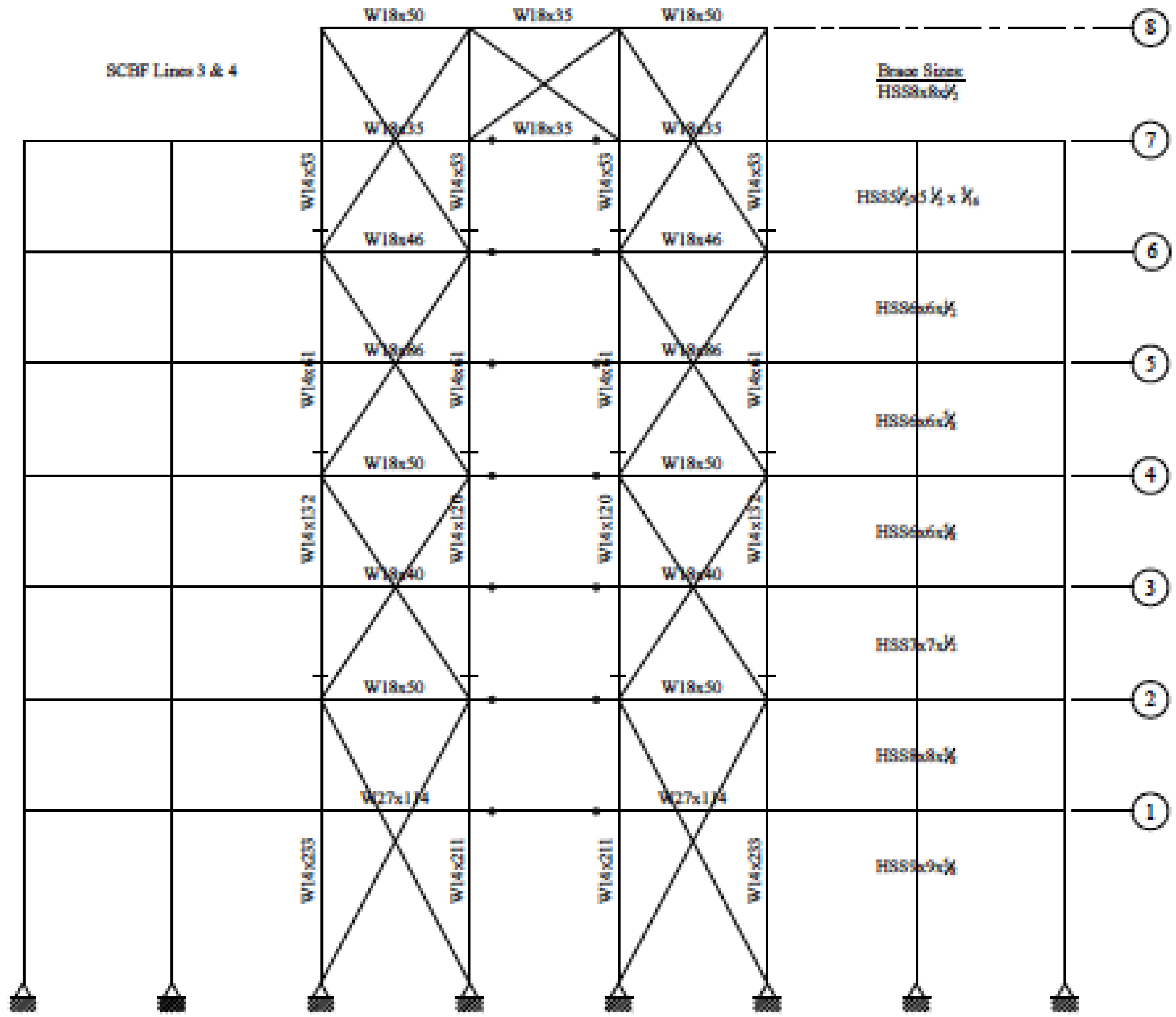
Concentrically Braced Frame Example

The following general design steps are required:

- Selection of preliminary member sizes
- Check strength
- Check drift
- Check torsional amplification
- Connection design

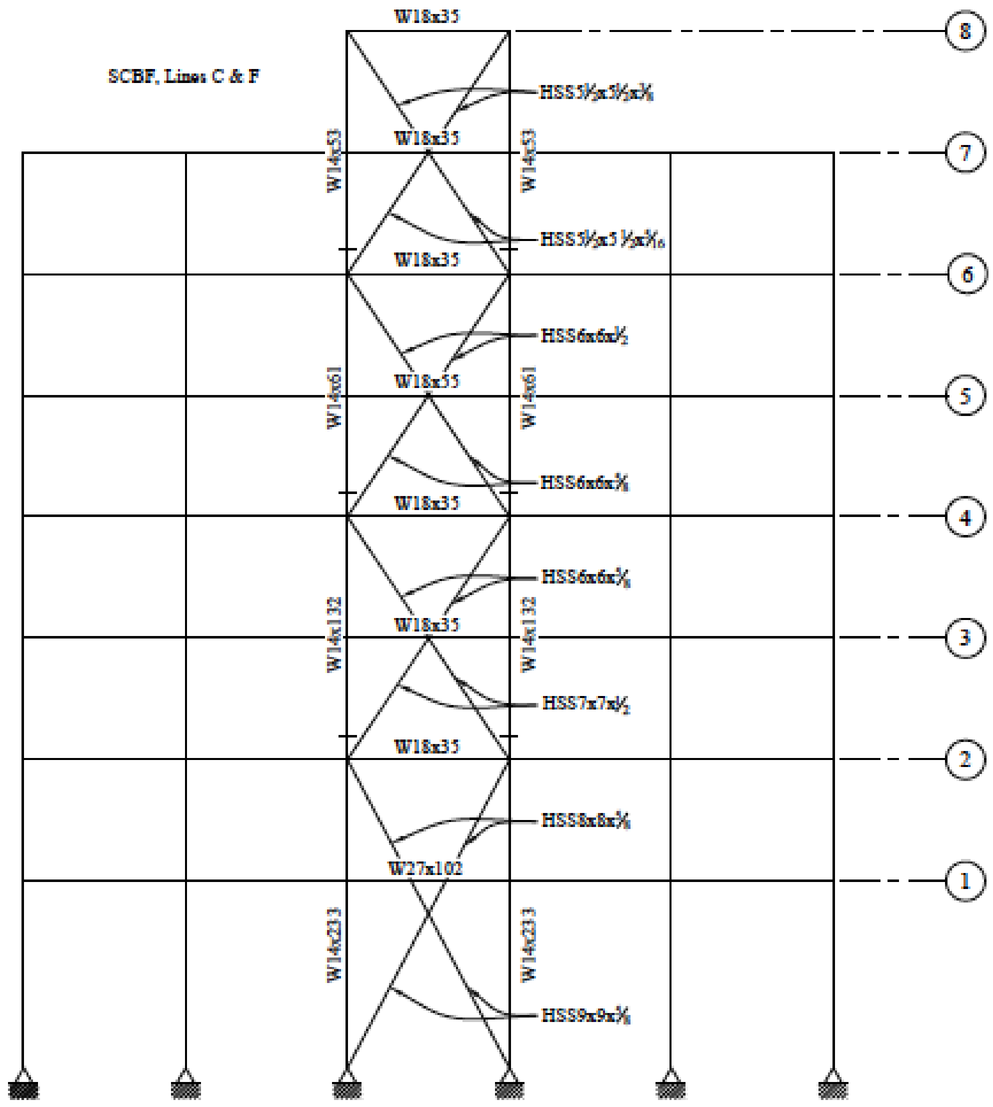
E-W Direction Framing

and Preliminary Member Sizes



N-S Direction Framing

and Preliminary Member Sizes



CBF Example

Building Weight:

Penthouse Roof = 94 kips

Lower Roof = 1,537 kips

Typical Floor = 1,920 kips

Total = 94 + 1,537 + 6(1,920) = 13,151 kips

Building Period:

$T_a = C_t h_n^x = (0.02)(102.3)^{0.75} = 0.64$ sec.

$T = C_u T_a = (1.4)(0.64) = 0.896$ sec.

Design Base Shear:

$C_s = S_{D1}/(T/(R/I)) = 0.6/(0.896/(6/1)) = 0.112$ <<< CONTROLS

$C_{s,min} = 0.044\, S_{DS} I = 0.044(1.0)(1) = 0.044$

$V = C_s W = 0.112(13{,}151) = 1{,}473$ kips.

CBFF Example – Check Deflection and Drift

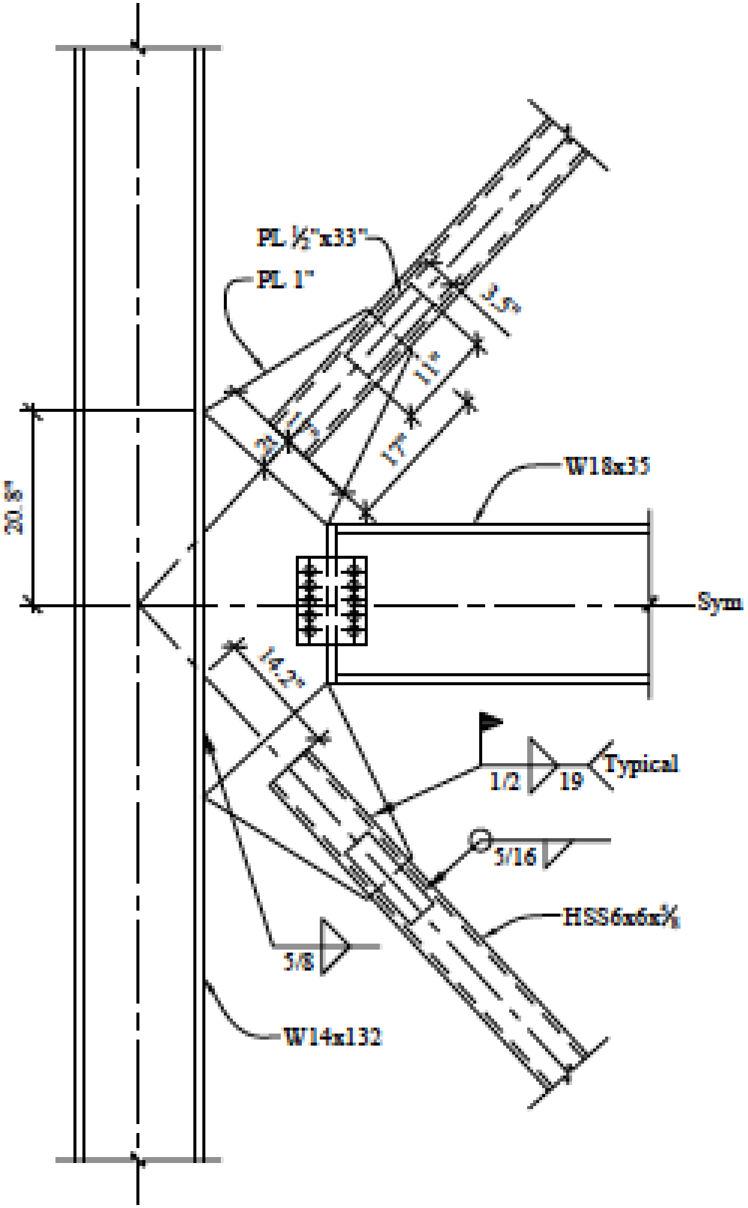
Table 6.2-2 Alternative B Story Drifts under Seismic Load

	Elastic Displacement at Building Corner, From Analysis		Expected Displacement (=$\delta_e C_d$)		Design Story Drift Ratio		Allowable Story Drift Ratio
Level	δ_e E-W (in.)	δ_e N-S (in.)	δ E-W (in.)	δ N-S (in.)	Δ E-W/h (%)	Δ N-S/h (%)	Δ/h (%)
Level 7	1.63	1.75	8.14	8.76	0.72	0.93	2.0
Level 6	1.41	1.48	7.07	7.38	0.74	0.94	2.0
Level 5	1.19	1.20	5.97	5.99	0.76	0.84	2.0
Level 4	0.96	0.94	4.80	4.72	0.81	0.85	2.0
Level 3	0.71	0.69	3.56	3.43	0.72	0.71	2.0
Level 2	0.49	0.47	2.44	2.33	0.60	0.59	2.0
Level 1	0.30	0.28	1.49	1.40	0.56	0.52	2.0

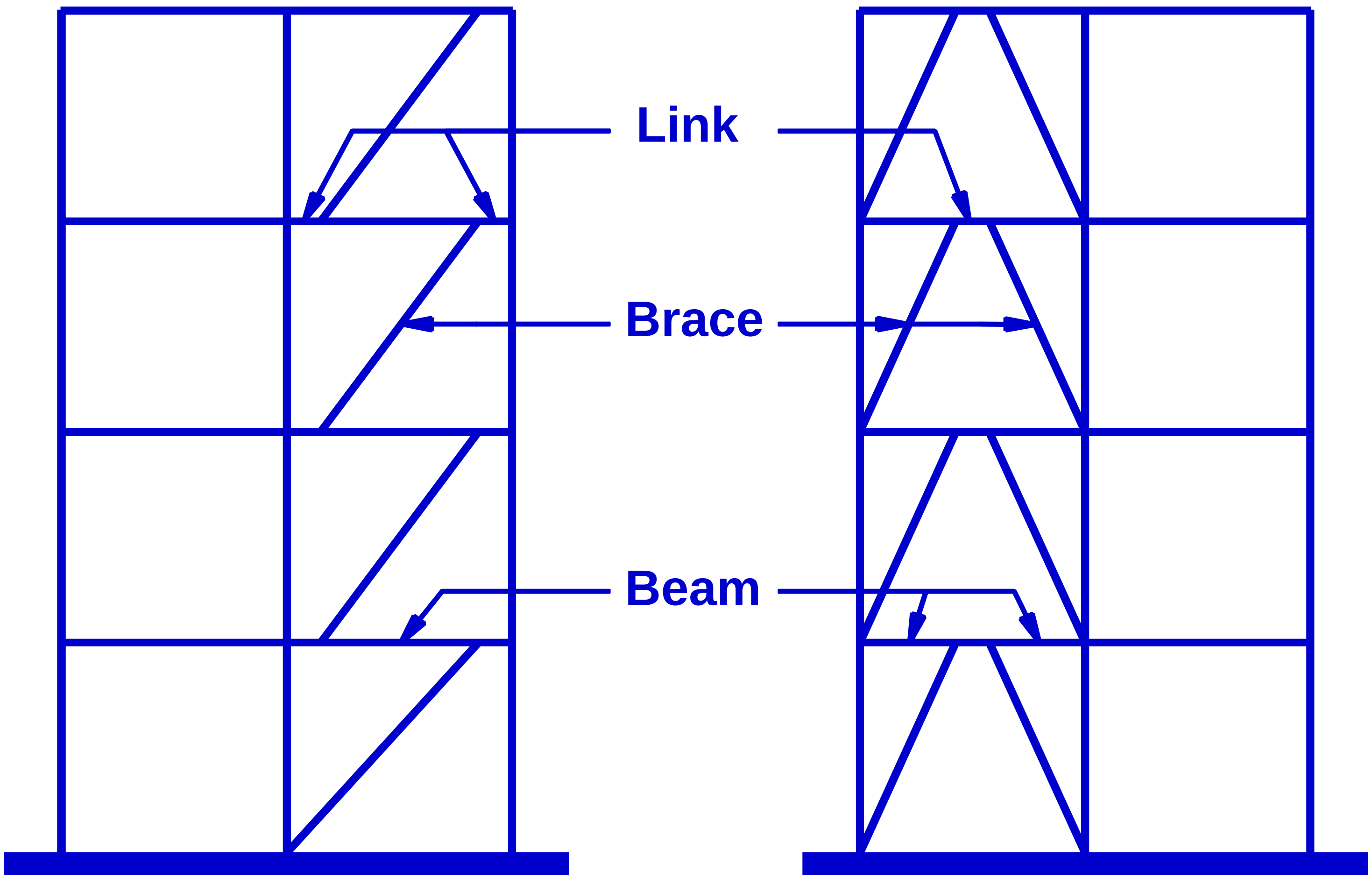
1.0 in. = 25.4 mm

Building Easily Satisfies Drift Limits

CBF Example – Connection Detail



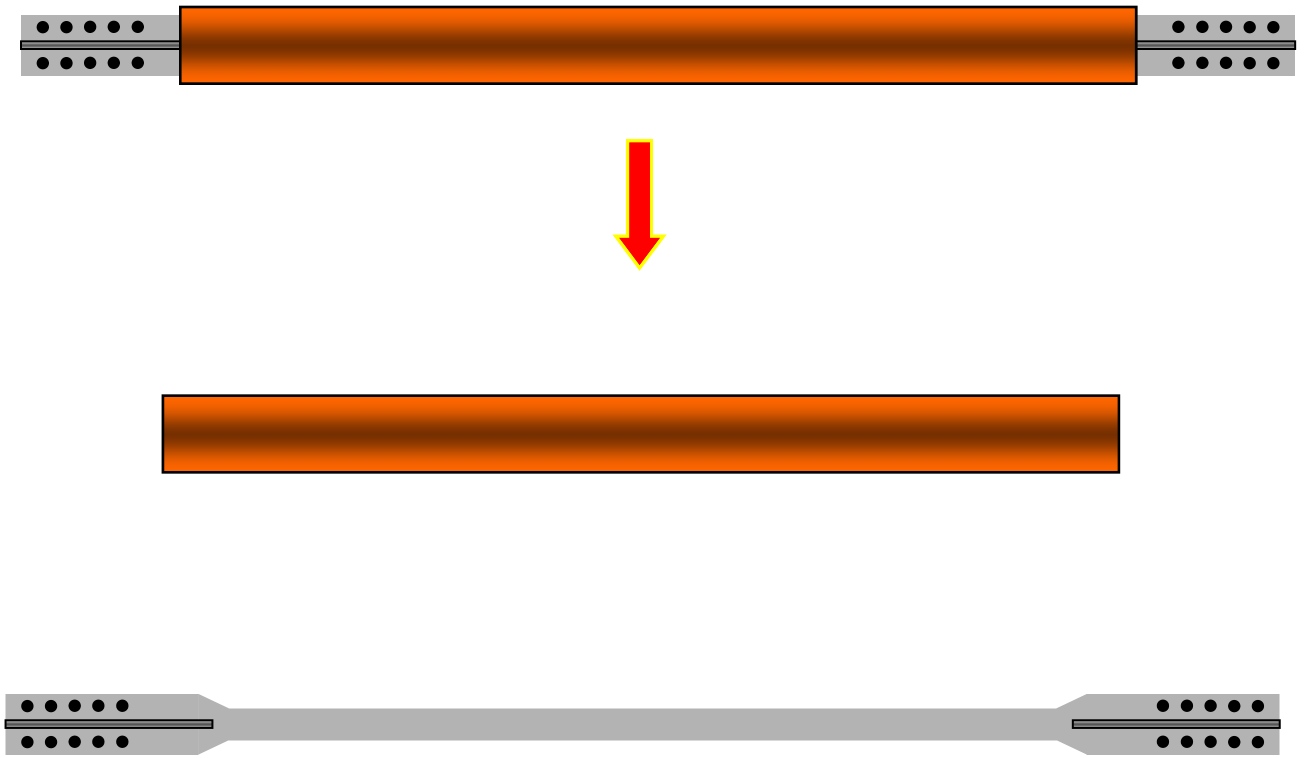
Eccentrically Braced Frames



Buckling-Restrained Braced Frames (BRBFs)

- Type of concentrically braced frame
- Beams, columns and braces arranged to form a vertical truss. Resist lateral earthquake forces by truss action
- Special type of brace members used: *Buckling-Restrained Braces (BRBs)*. BRBS yield both in tension and compression - *no buckling !!*
- Develop ductility through inelastic action (cyclic tension and compression yielding) in BRBs.
- System combines high stiffness with high ductility

Buckling-Restrained Braced Frames (BRBFs)

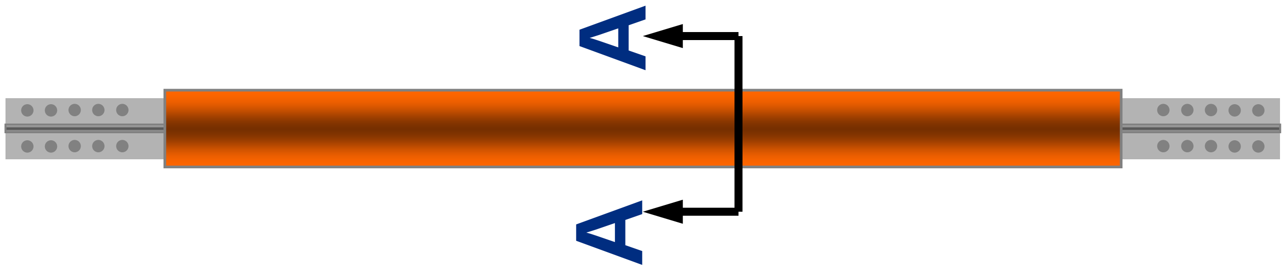


Buckling- Restrained Brace:
Steel Core
+
Casing

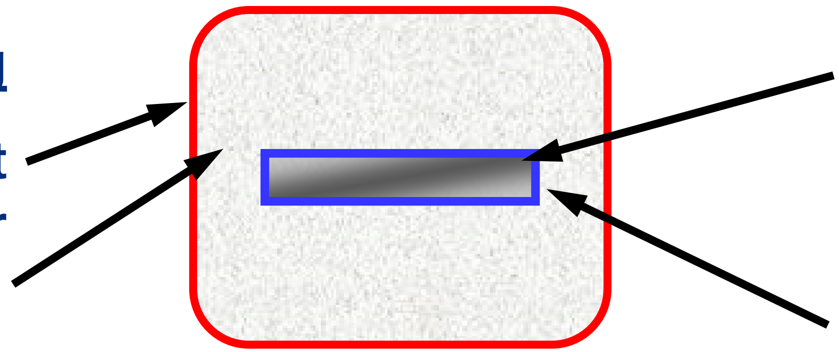
Casing

Steel Core

Buckling-Restrained Braced Frames (BRBFs)



Buckling-Restrained Brace: Steel Core + Casing



Section A-A

Buckling-Restrained Braced Frames (BRBFs)

Steel core resists entire axial force P

Casing is debonded from steel core

- casing does not resist axial force P
- flexural stiffness of casing restrains buckling of core

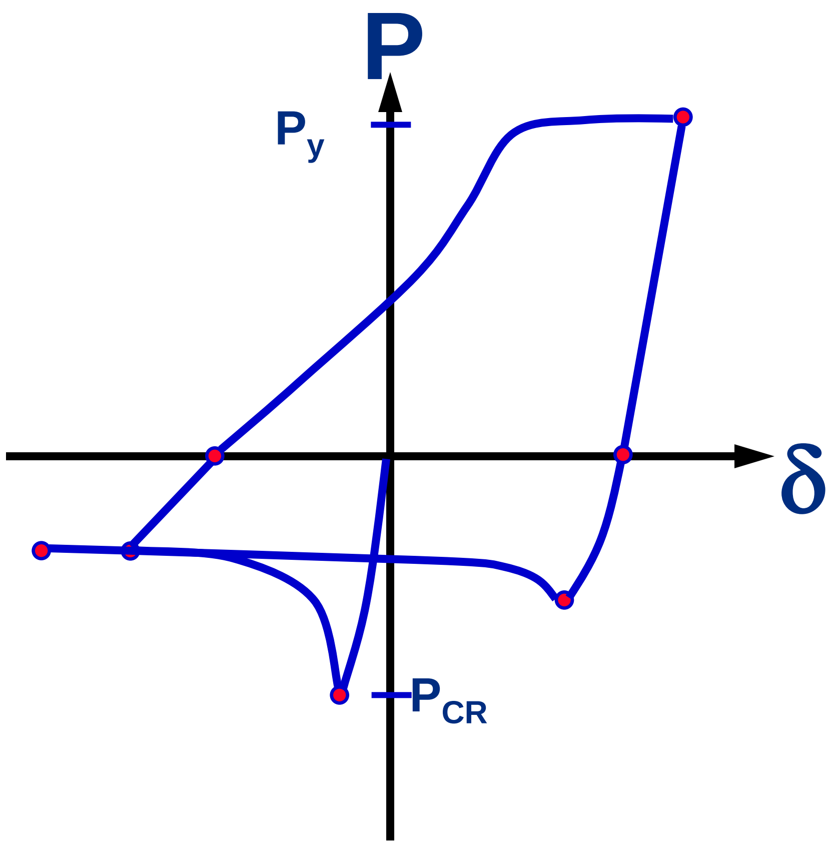
Buckling-Restrained Braced Frames (BRBFs)

Buckling-Restrained Brace: Steel Core + Casing

Steel Core

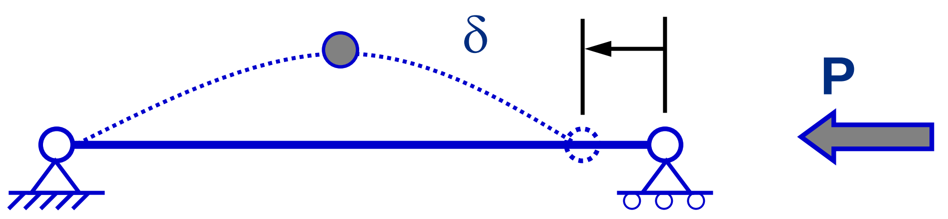
Core projection and brace connection segment

Brace Behavior Under Cyclic Axial Loading

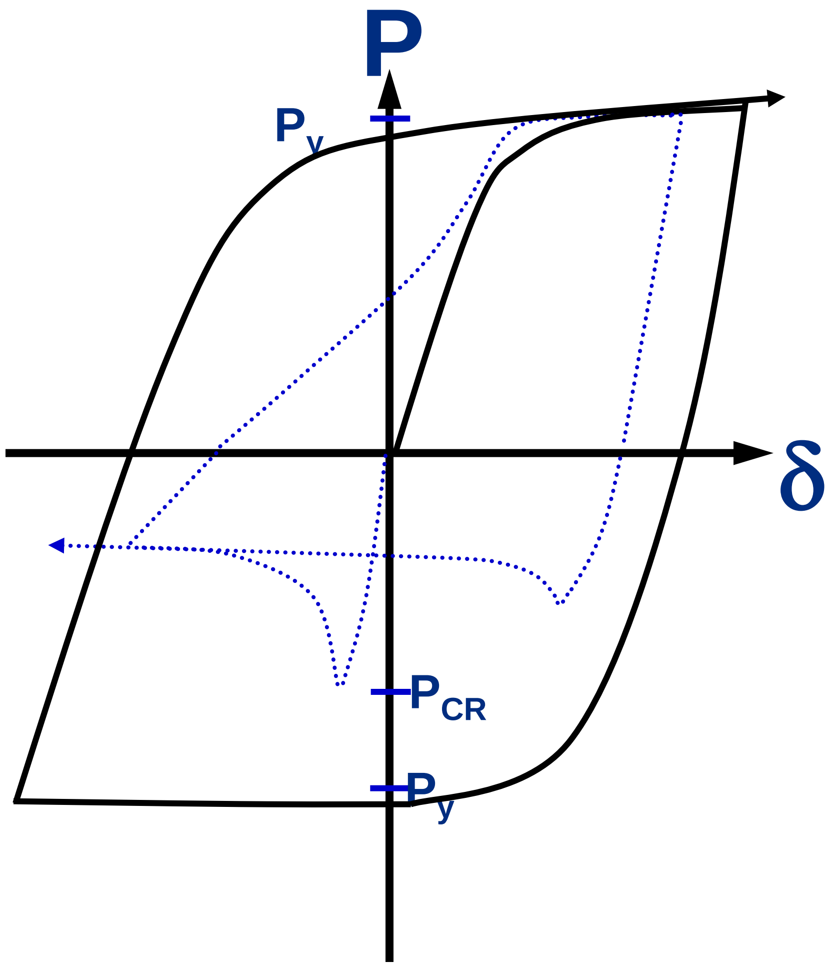


Conventional Brace:

- yields in tension (ductile)
- buckles in compression (nonductile)
- significantly different strength in tension and compression

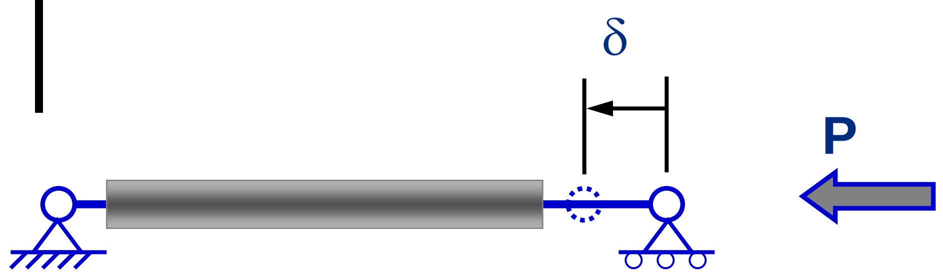


Brace Behavior Under Cyclic Axial Loading

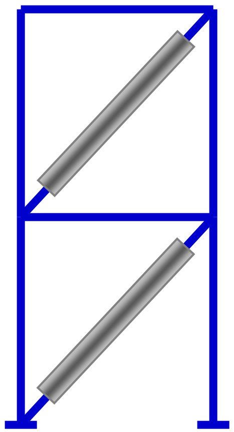


Buckling-Restrained Brace:

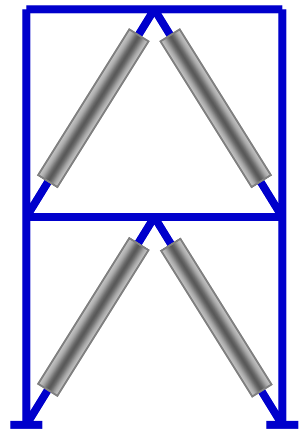
- yields in tension (ductile)
- yields in compression (ductile)
- similar strength in tension and compression (slightly stronger in compression)



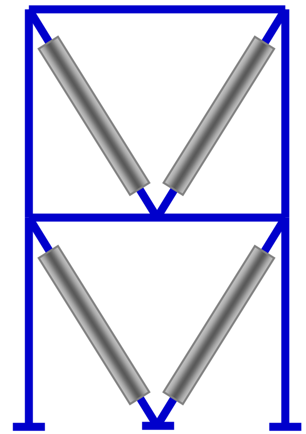
Bracing Configurations for BRBFs



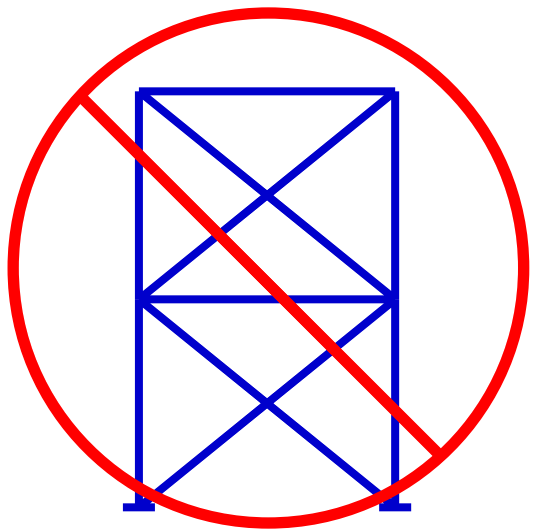
Single Diagonal



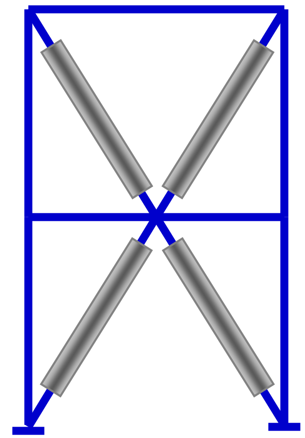
Inverted V- Bracing



V- Bracing

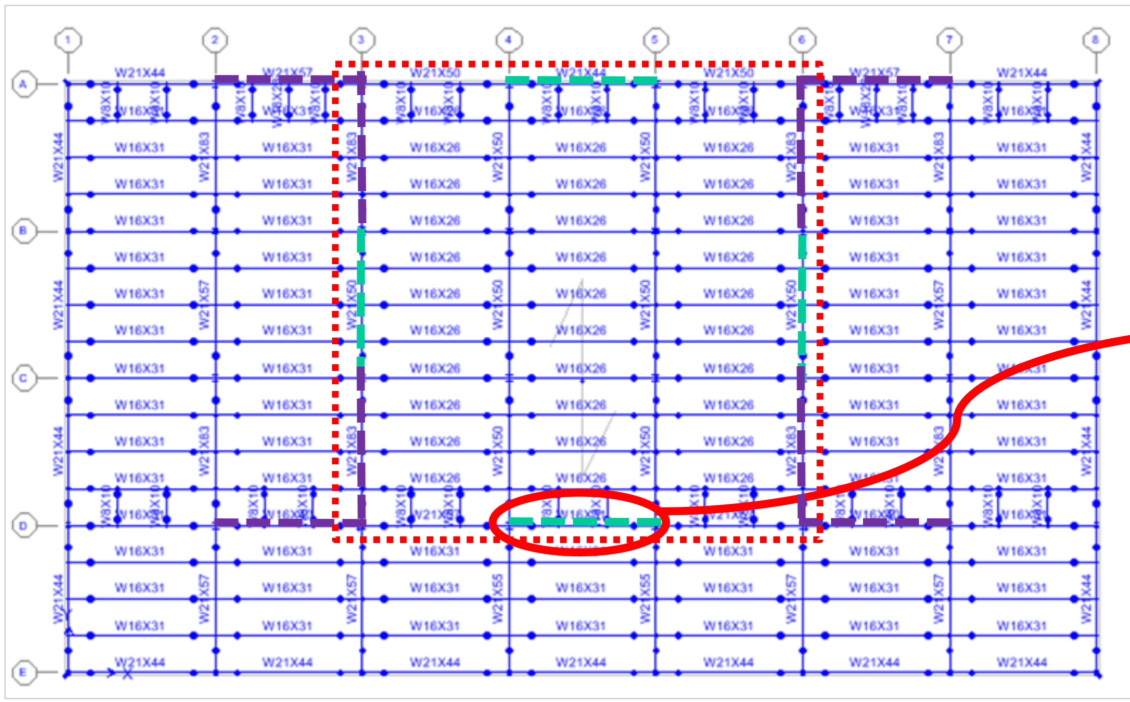


X- Bracing



Two Story X-Bracing

BRB Design Example

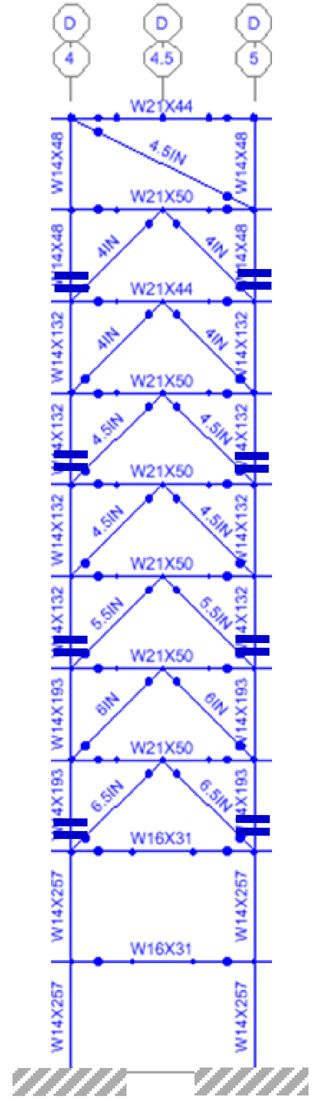


Plan View – 3rd Level (roof of podium, base of tower)

Indicates extent of tower floorplate (above)

Indicates bay of BRBs in tower (above)

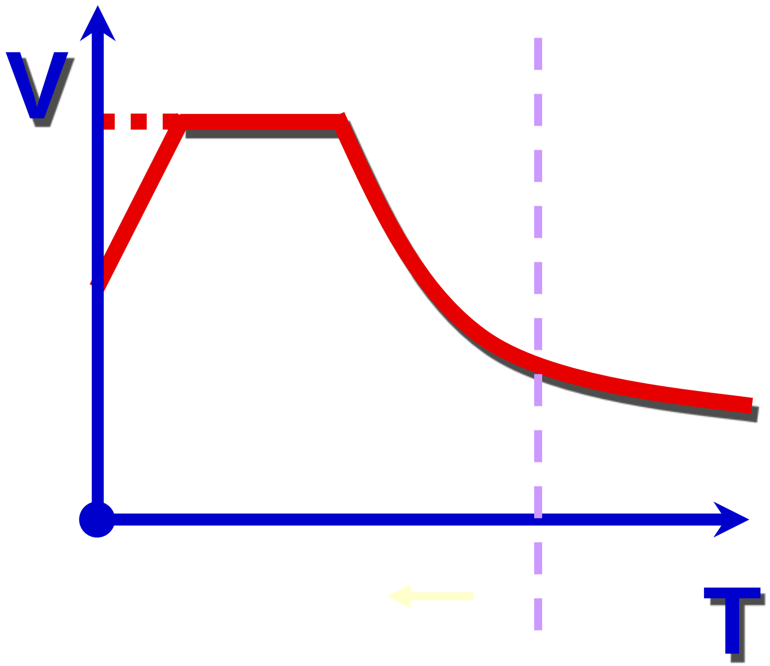
Indicates bay of BRBs in podium (below)



R = 8

ASCE 7 2005
AISC Seismic 2005

Base Shear



Hazard

$S_{ds} = 0.859$

$S_{d1} = 0.433$

$T_a = 1.32$ sec.

$V = 0.057\ W$

Load Combinations

Basic

$1.2D + f_1L + 0.2S + E$

$0.9D \pm E$

Special (Amplified Seismic Load)

$1.2D + f_1L + 0.2S + E_m$

$0.9D \pm E_m$

$f_1 = 0.5$

$E = \rho Q_E + 0.2S_{DS} D$

$E_m = \Omega_o Q_E + 0.2S_{DS} D$

$1.37D + 0.5L + 0.2S + \rho Q_E$

$0.73D \pm \rho Q_E$

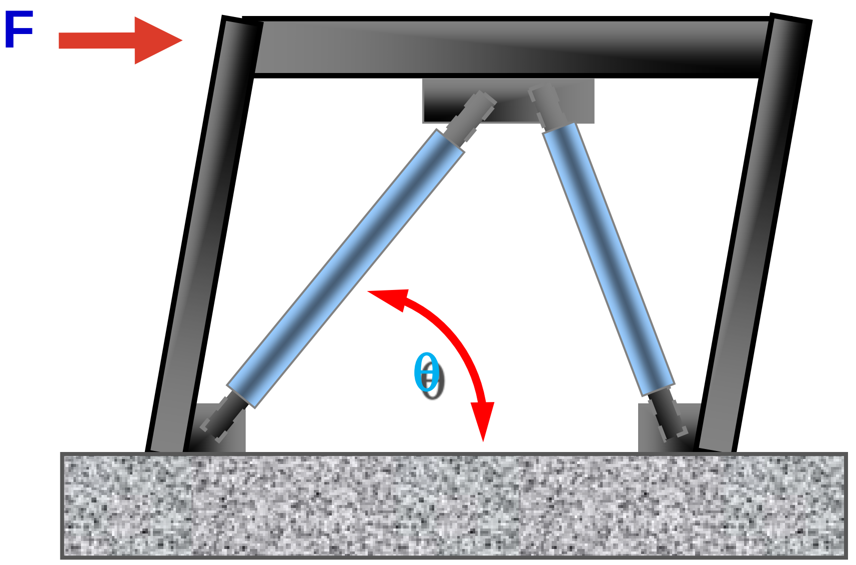
$1.37D + 0.5L + 0.2S + \Omega_o Q_E$

$0.73D \pm \Omega_o Q_E$

Vertical Distribution of Forces

Diaphragm Level	Story Force (kips)	Brace Level	Story Shear (kips)	% of Total Base Shear
Roof	210	10	210	24%
10	152	9	362	42%
9	126	8	488	56%
8	102	7	590	68%
7	80	6	670	77%
6	60	5	730	84%
5	43	4	772	89%
4	28	3	800	92%
3	51	2	851	98%
2	14	1	865	100%

Preliminary Design of Braces



$$P_u = \frac{F}{2\cos\theta}$$

Assume braces resist
100% of story shear

$$A_{sc} = \frac{P_u}{\phi F_y}$$

Design braces precisely to calculated capacity ($P_u = \phi P_n = \phi F_y A_{sc}$)

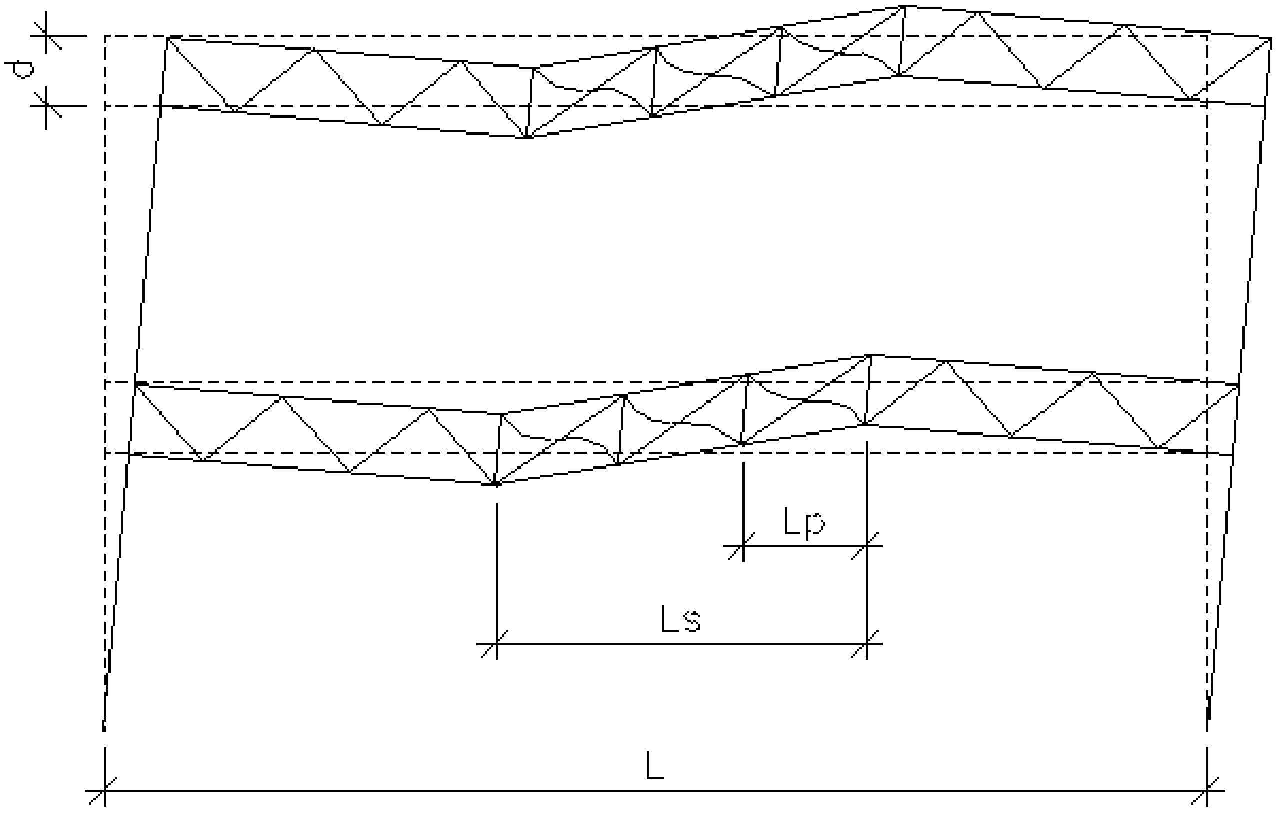
BRB Design – Gridline D

Brace Level	Brace Angle α (deg)	Brace Force P_u (kips)	Required Core Area A_{sc}	Provided Core Area A_{sc}
10	26.6	142	4.21	4.5
9	45.0	126	3.71	4.0
8	45.0	133	3.98	4.0
7	45.0	141	4.12	4.5
6	45.0	151	4.42	4.5
5	45.0	170	5.12	5.5
4	45.0	197	5.70	6.0
3	45.0	210	6.26	6.5
2	50.1	187	5.76	6.0
1	50.1	195	6.81	7.0

NEHRP Recommended Provisions
Steel Design

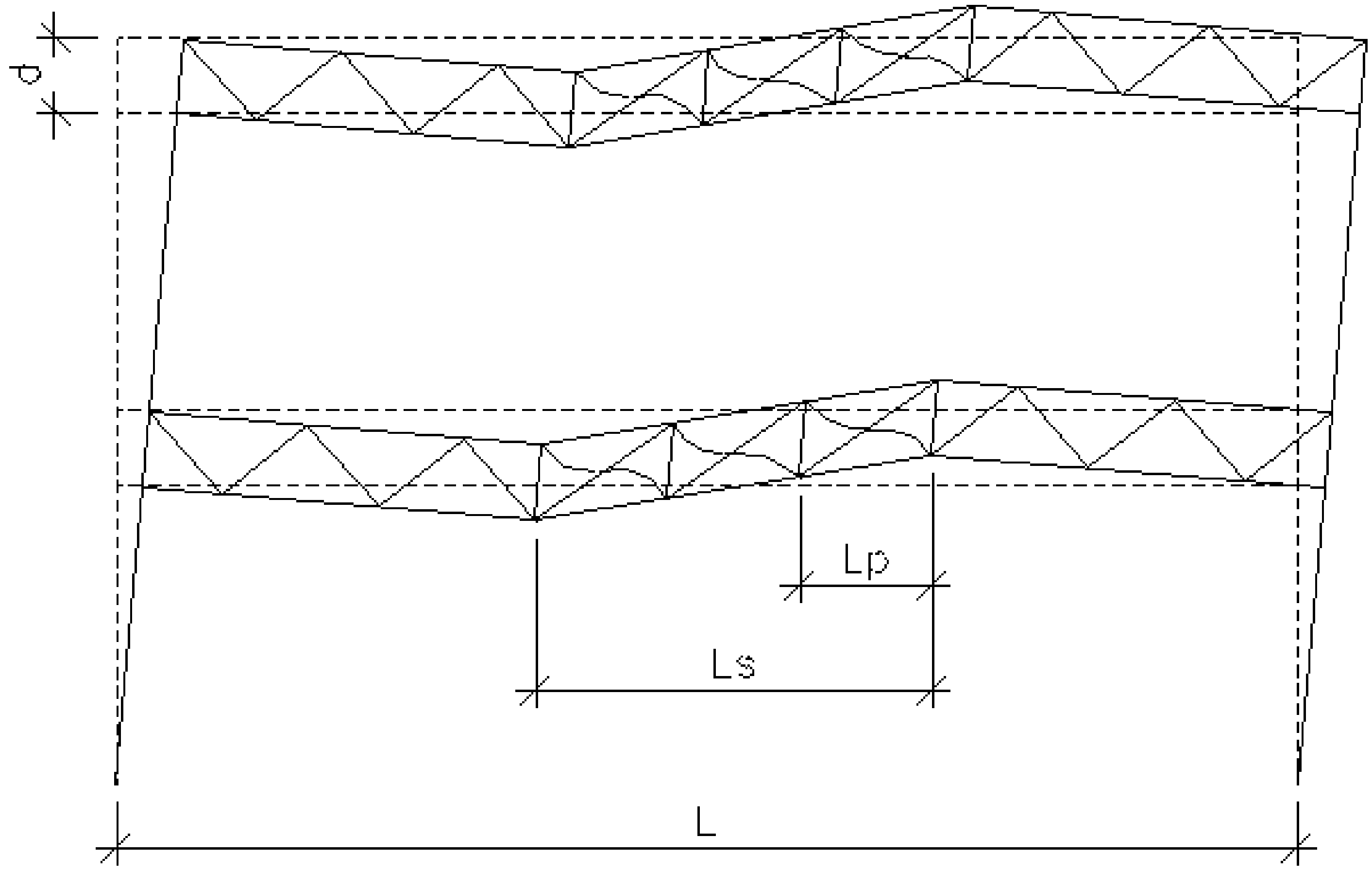
- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- **Other topics**

Special Truss Moment Frame



- Buckling and yielding in <u>special</u> section
- Design to be elastic outside special section
- Deforms similar to EBF
- Special panels to be symmetric X or Vierendeel

Special Truss Moment Frame



Geometric Limits:

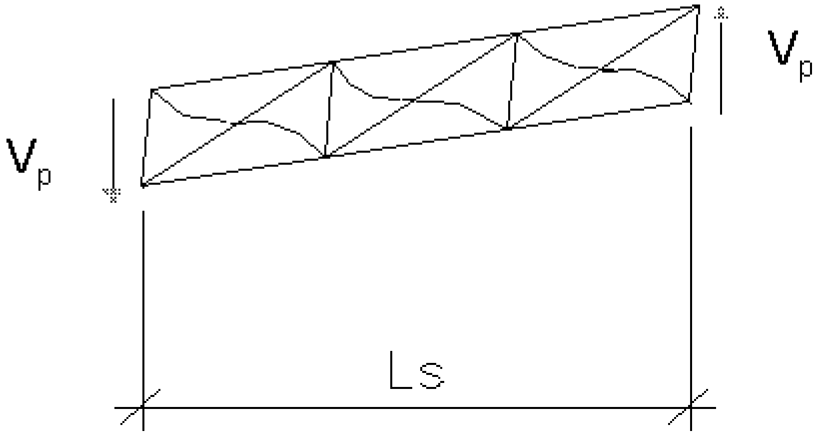
$L \leq 65'$ $\quad d \leq 6'$

$$0.1 < \frac{L_s}{L} < 0.5$$

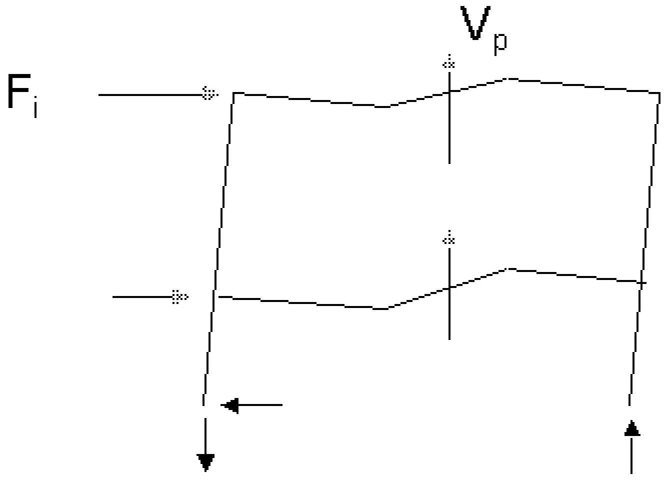
$$\frac{2}{3} < \frac{L_p}{d} < \frac{3}{2}$$

Flat bar diagonals, $\frac{b}{t} \leq 2.5$

Special Truss Moment Frame



$$V_p = 2\left(\frac{2\,M_{pc}}{L_s}\right) + \sin\alpha\left(P_{nt} + 0.3P_{cd}\right)$$



$$\sum F_i\,h_i = \sum V_p\,L$$

Special Truss Moment Frame

Special Truss Moment Frame

General Seismic Detailing

Materials:

- Limit to lower strengths and higher ductilities

Bolted Joints:

- Fully tensioned high strength bolts
- Limit on bearing

General Seismic Detailing

Welded Joints:

- AWS requirements for welding procedure specs
- Filler metal toughness
  - CVN > 20 ft-lb @ -20°F, or AISC Seismic App. X
- Warning on discontinuities, tack welds, run offs, gouges, etc.

Columns:

- Strength using Ω_o if $P_u / \phi P_n > 0.4$
- Splices: Requirements on partial pen welds and fillet welds

Steel Diaphragm

Example

$\phi V_n = \phi$ (approved strength)

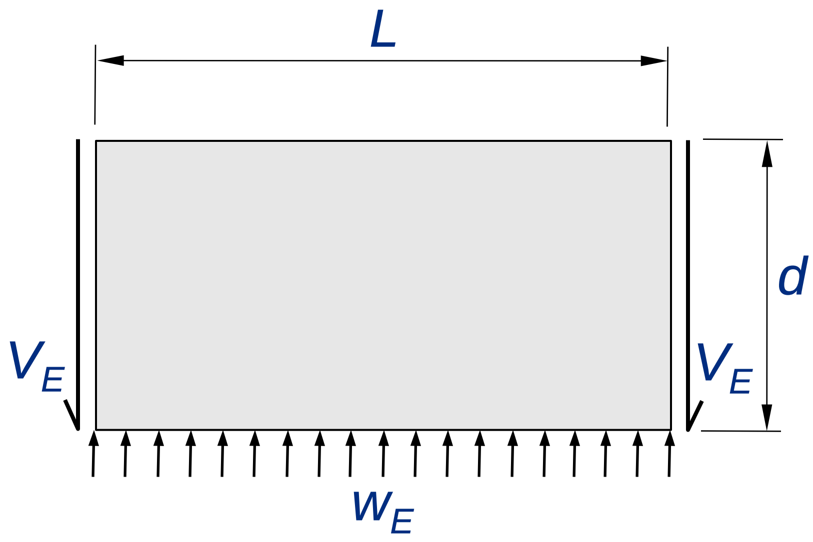
$\phi = 0.6$

For <u>example only:</u>

Use approved strength as 2.0 × working load in SDI *Diaphragm Design Manual*

Steel Deck Diaphragm

Example



$$L = 80' \qquad d = 40'$$

$$w_D = w_L = 0 \qquad w_E = 500 \text{ plf}$$

$$V_E = \frac{w_E L}{2} = 20 \text{ kip}; \quad v_E = \frac{20000}{40} = 500 \text{ plf}$$

$$v_{SDI} = \frac{v_E}{2\phi} = \frac{500}{2(0.6)} = 417 \text{ plf}$$

Deck chosen:

1½”, 22 gage with welds on 36/5 pattern and 3 sidelap fasteners, spanning 5’-0”

Capacity = 450 > 417 plf

Welded Shear Studs

Shear Stud Strength - AISC 2005 Specification

$$Q_n = 0.5\, A_{sc} \left(f_c'\, E_c \right)^{1/2} \leq R_g\, R_p\, A_{sc}\, F_u$$

R_g = stud geometry adjustment factor
R_p = stud position adjustment factor

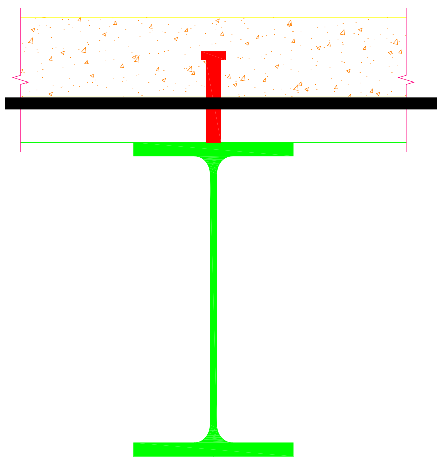
Note that the strength reduction factor for bending has been increased from 0.85 to 0.9. This results from the strength model for shear studs being more accurate, although the result for Q_n is lower in the 2005 specification.

Shear Studs – Group Adjustment Factor

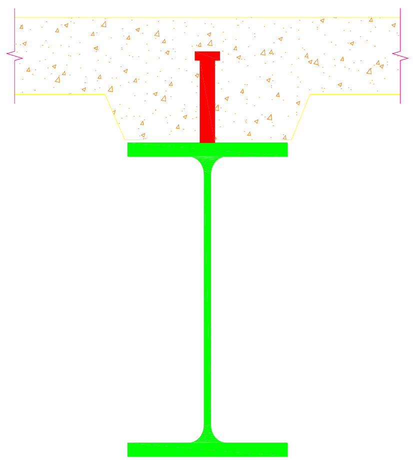
$$Q_n = 0.5\, A_{sc}\, (\, f_c'\, E_c)^{1/2} \leq R_g\, R_p\, A_{sc}\, F_u$$

R_g = stud group adjustment factor

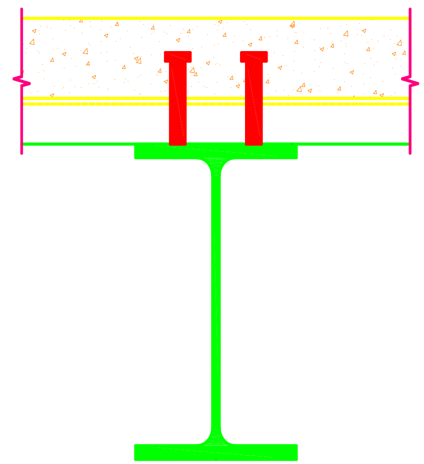
$R_g = 1.0$



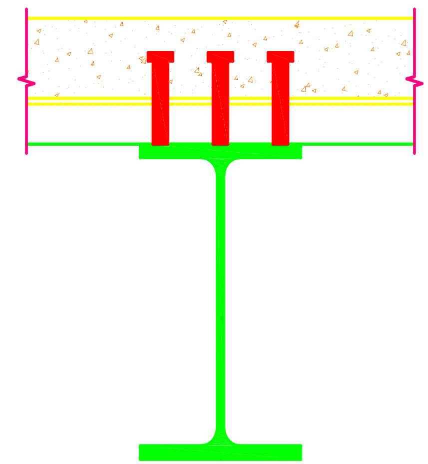
$R_g = 1.0$*



$R_g = 0.85$



$R_g = 0.7$



*0.85 if $w_r\, /h_r < 1.5$

Shear Studs – Position Adjustment Factor

$$Q_n = 0.5\, A_{sc}\, (\, f_c'\, E_c)^{1/2} \leq R_g\, R_p\, A_{sc}\, F_u$$

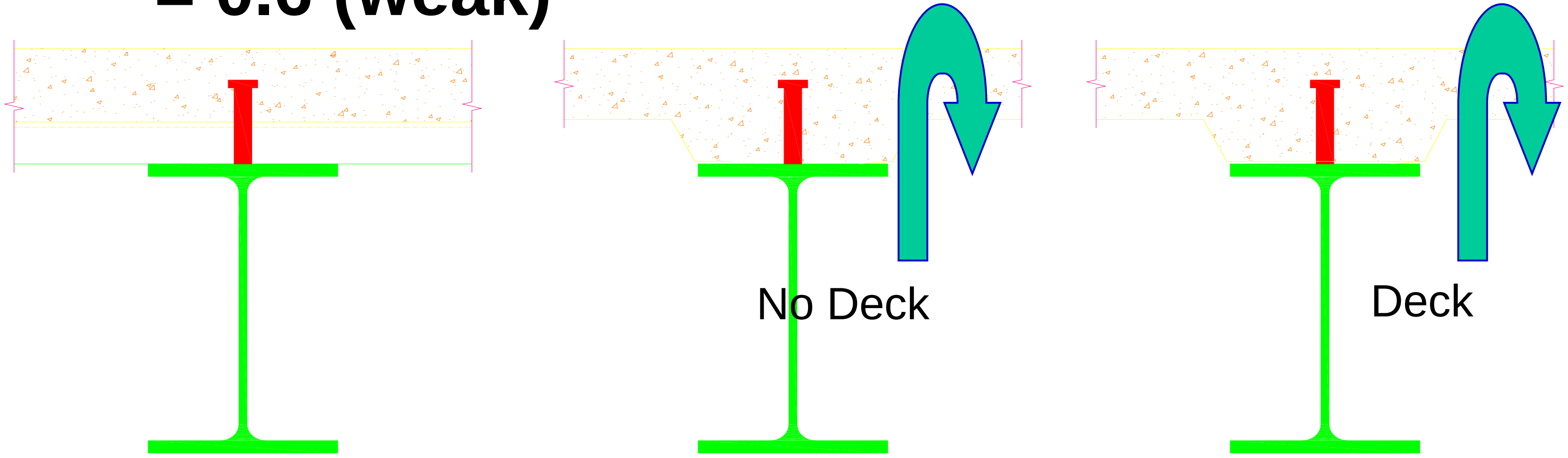
R_p = stud position adjustment factor

R_p = 0.75 (strong)

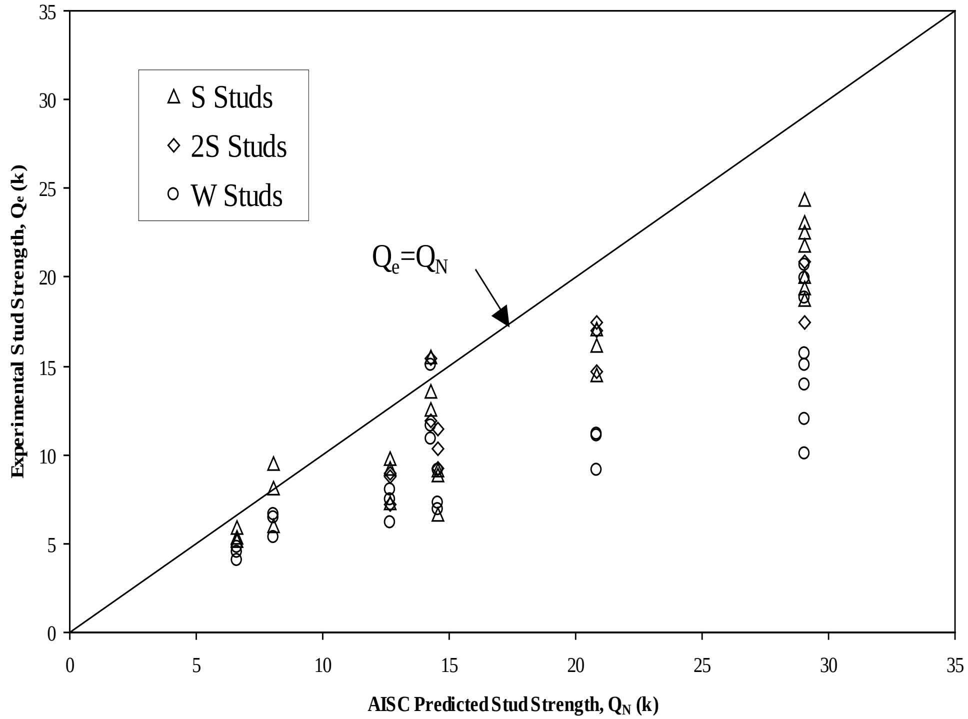
= 0.6 (weak)

R_p = 1.0

R_p = 0.75

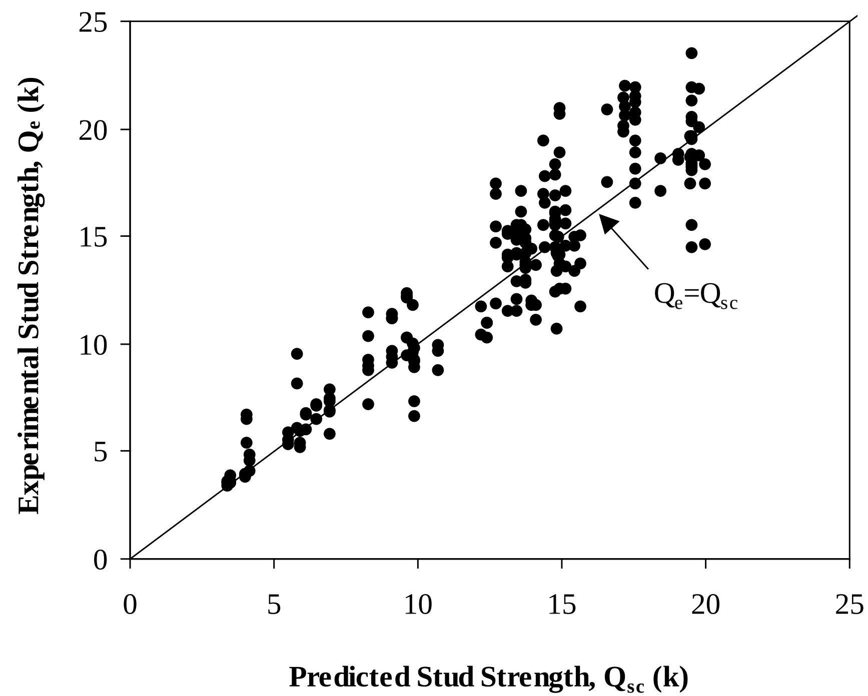


Shear Studs – Strength Calculation Model Comparison



AISC Seismic prior to 2005

Virginia Tech strength model



Shear Studs – Diaphragm Applications

Shear studs are often used along diaphragm collector members to transfer the shear from the slab into the frame. The shear stud calculation model in the 2005 AISC specification can be used to compute the nominal shear strengths. A strength reduction factor should be used when comparing these values to the factored shear. There is no code- established value for the strength reduction factor. A value of 0.8 is recommended pending further development.

Inspection and Testing

Inspection Requirements

- Welding:
  - Single pass fillet or resistance welds

    PERIODIC
  - All other welds

    CONTINUOUS

- High strength bolts:

  PERIODIC

Inspection and Testing

Shop Certification

- Domestic:
  - AISC
  - Local jurisdictions
- Foreign:
  - No established international criteria

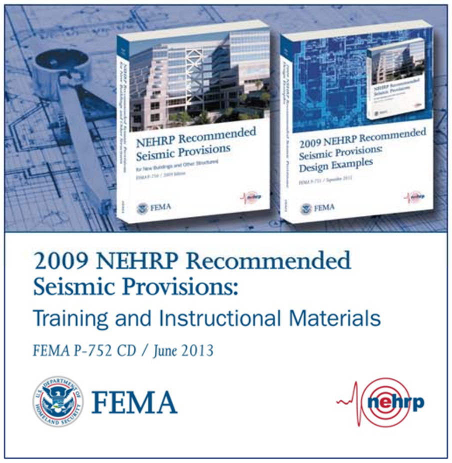
Inspection and Testing
Base Metal Testing

- More than 1-1/2 inches thick
- Subjected to through-thickness weld shrinkage
- Lamellar tearing
- Ultrasonic testing

NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics

Questions



6

Structural Steel Design

Rafael Sabelli, S.E., and Brian Dean, P.E.

Originally developed by
James R. Harris, P.E., PhD, Frederick R. Rutz, P.E., PhD, and Teymour Manouri, P.E., PhD

Title slide.

SEISMIC DESIGN OF STEEL STRUCTURES

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics
- Summary

Table of contents. Note that most of the substance, except the first two items, is taken from the AISC 341 *Seismic Provisions for Steel Buildings*, which is referenced by the *Standard.*

Some of the examples in this topic draw heavily on the FEMA 751 Design Examples CD. Please see this CD for additional details.

Steel Design: Context in Provisions

Design basis: Strength limit state

Using the 2009 *NEHRP Recommended Provisions*, Refer to ASCE 7 2005:

Seismic Design Criteria	Chap. 11
Seismic Design Requirements	
Buildings	Chap. 12
Nonstructural components	Chap. 13
Design of steel structures	Chap. 14
	AISC Seismic
	and others

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Structural Steel Design - 3

The *Provisions* requirements affect design loads, limit states, and specific details required for members and connections. The bulk of the detailing rules are in the reference documents; the BSSC steel technical committee and the AISC seismic committee have been structured to work very closely together; thus, BSSC's recommendations are incorporated into AISC Seismic very quickly.

Seismic Resisting Systems

Unbraced Frames

Connections are:

- Fully Restrained Moment-resisting
- Partially Restrained Moment-resisting

Seismic classes are:

- Special Moment Frames
- Intermediate Moment Frames
- Ordinary Moment Frames
- Systems not specifically detailed for seismic response

Braced Frames

Ordinary Concentric Braced Frames

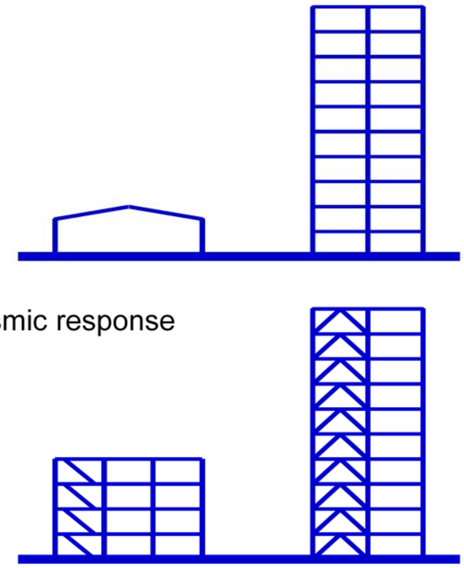
Special Concentric Braced Frames

Eccentrically Braced Frames

Buckling Restrained Braced Frames

Special Plate Shear Walls

Systems not specifically detailed for seismic response



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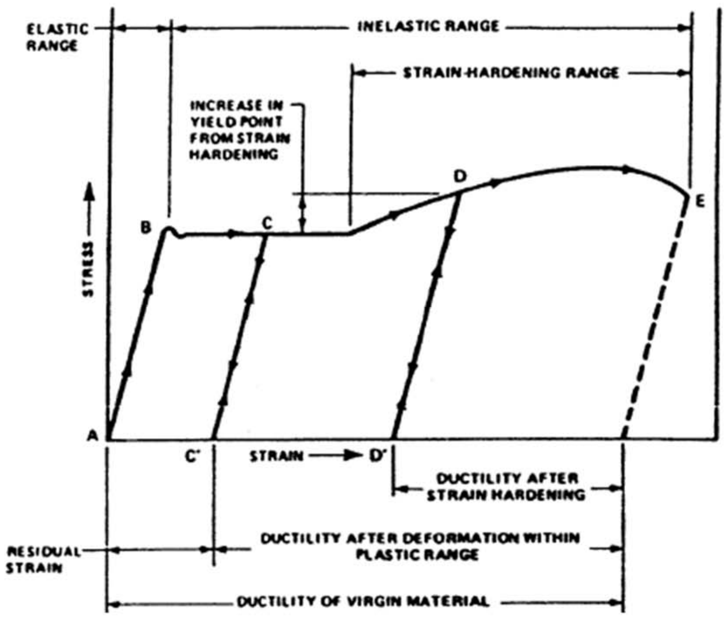
Structural Steel Design - 4

Unbraced ("moment resisting") frames resist lateral forces through flexural actions of members framing into (fully or partially) rigidly connected joints. Concentrically braced frames (no moment resisting connections) resist lateral forces through truss action that causes axial forces in members. Eccentric bracing creates high moments and shears in short links intended to yield first. Overall structural deformation in tall buildings: "shear building" pattern for unbraced frame but flexural pattern in braced frame.

Monotonic Stress-Strain Behavior



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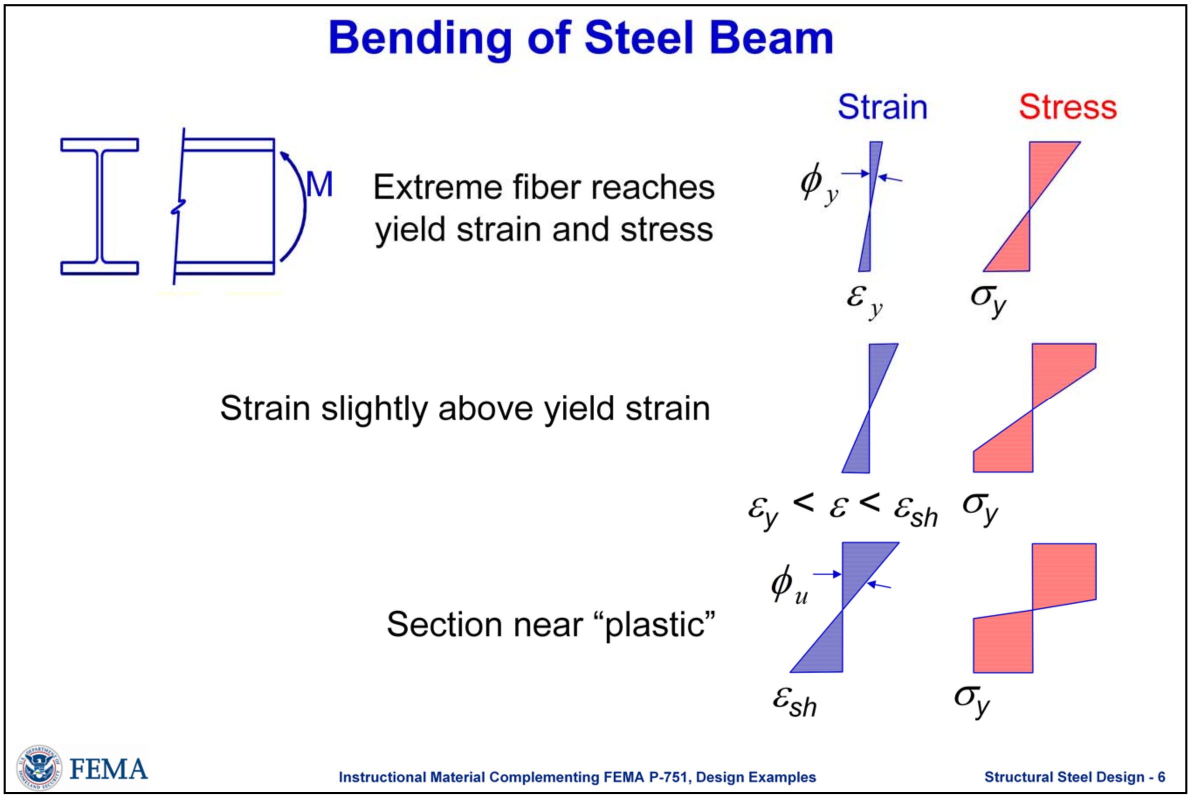
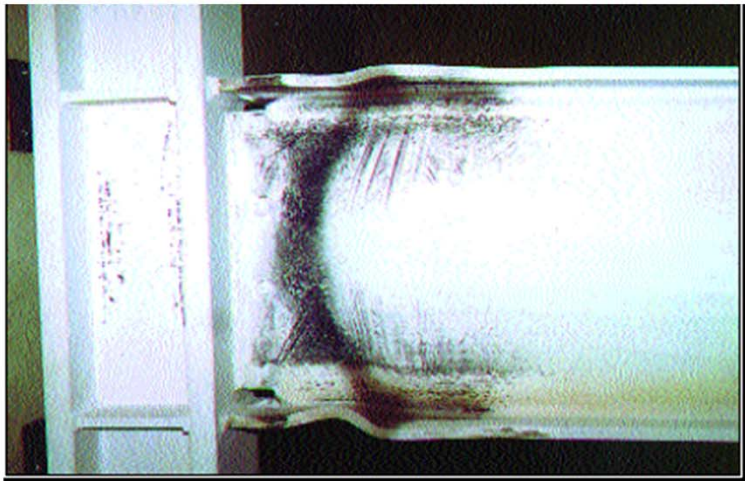
Stress-strain diagram for mild steel. Note the elastic range, yield point, plastic range, and region of strain hardening. Define ductility ratio; note values of 100 or more. Compare high- and low-strength steels.

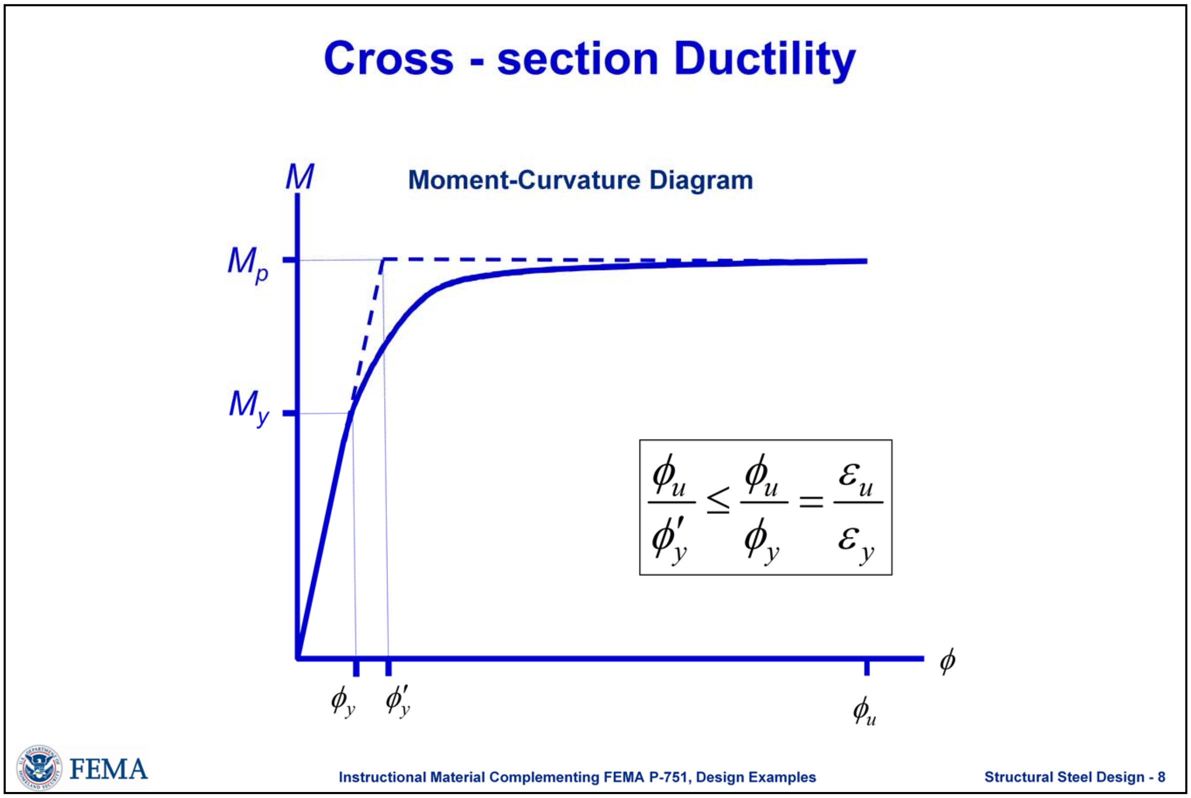


For the elastic-plastic model, regardless of strain increases at extreme fibers, stress never exceeds yield stress. However, more material yields as strain increases; stress distribution becomes partially plastic. Note that the strain diagram near plastic is prior to the onset of strain hardening.

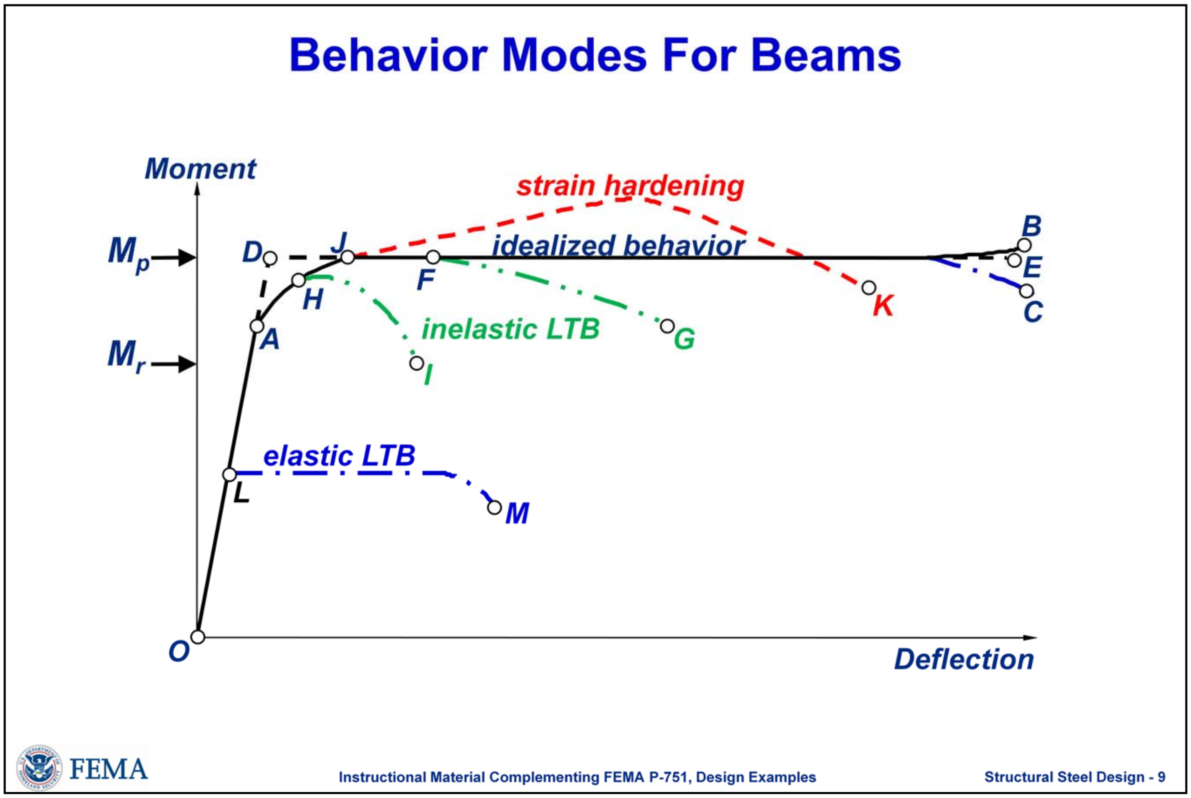
Plastic Hinge Formation



From *Modern Steel Construction* – Photograph formation of a plastic hinge in a laboratory test of a RBS (reduced beam section) specimen. Yielding may be observed from flaking of whitewash from surface of steel. Very distinctive flange buckling is also visible.



Moment-curvature diagram shows definitions for various curvatures. Yield curvature: at first yield (at M_y). As moment approaches M_p, curvature increases rapidly. At M_p, curvature increases without additional moment. Ultimate curvature: curvature at failure. Behavior often idealized by extending elastic and plastic portions of curve to intersection. Note that strain hardening is not reflected in the plot.



Note the various possible limit states (as on slide).

Strain hardening behavior is typical if hinge occurs in a region of significant shear (moment diagram is not constant). This is the typical situation, whereas the constant moment diagram in previous slide is really a laboratory idealization. M_r is the yield moment that includes the effects of residual stress.

Flexural Ductility of Steel Members
Practical Limits

1. Lateral torsional buckling
   - Brace well
2. Local buckling
   - Limit width-to-thickness ratios for compression elements
3. Fracture
   - Avoid by proper detailing

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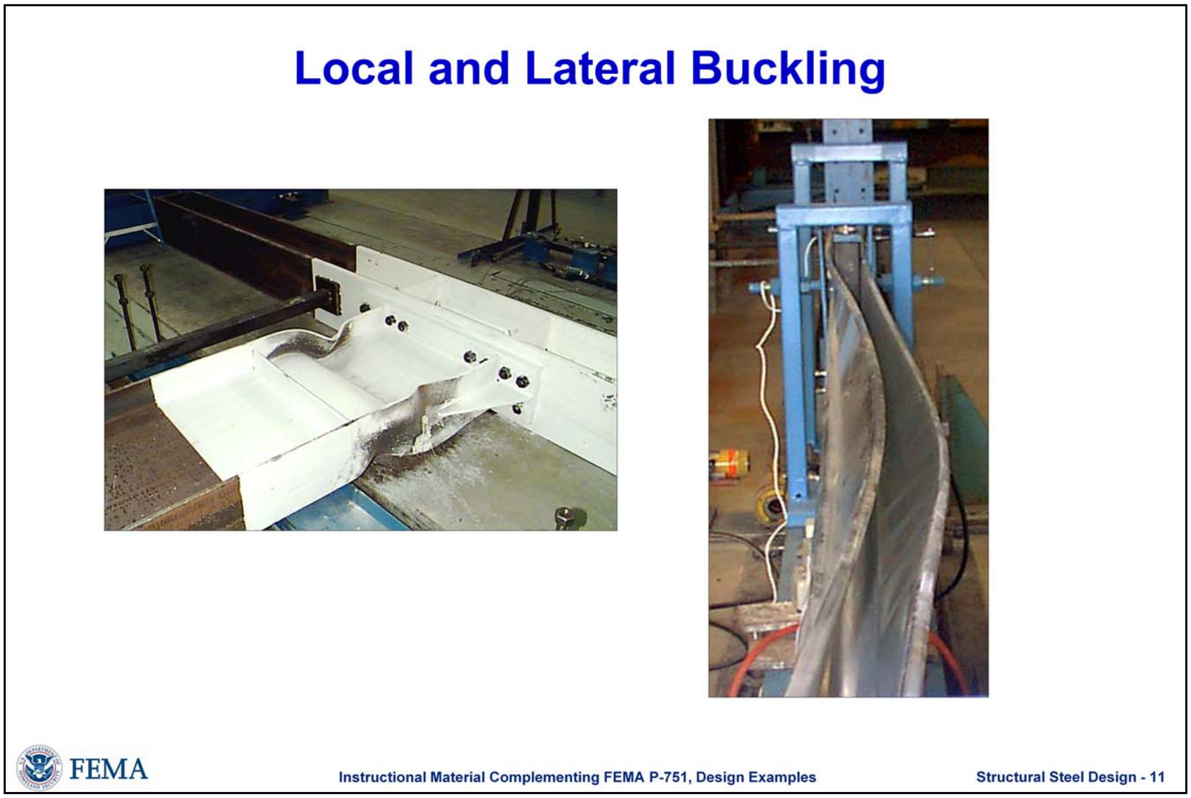
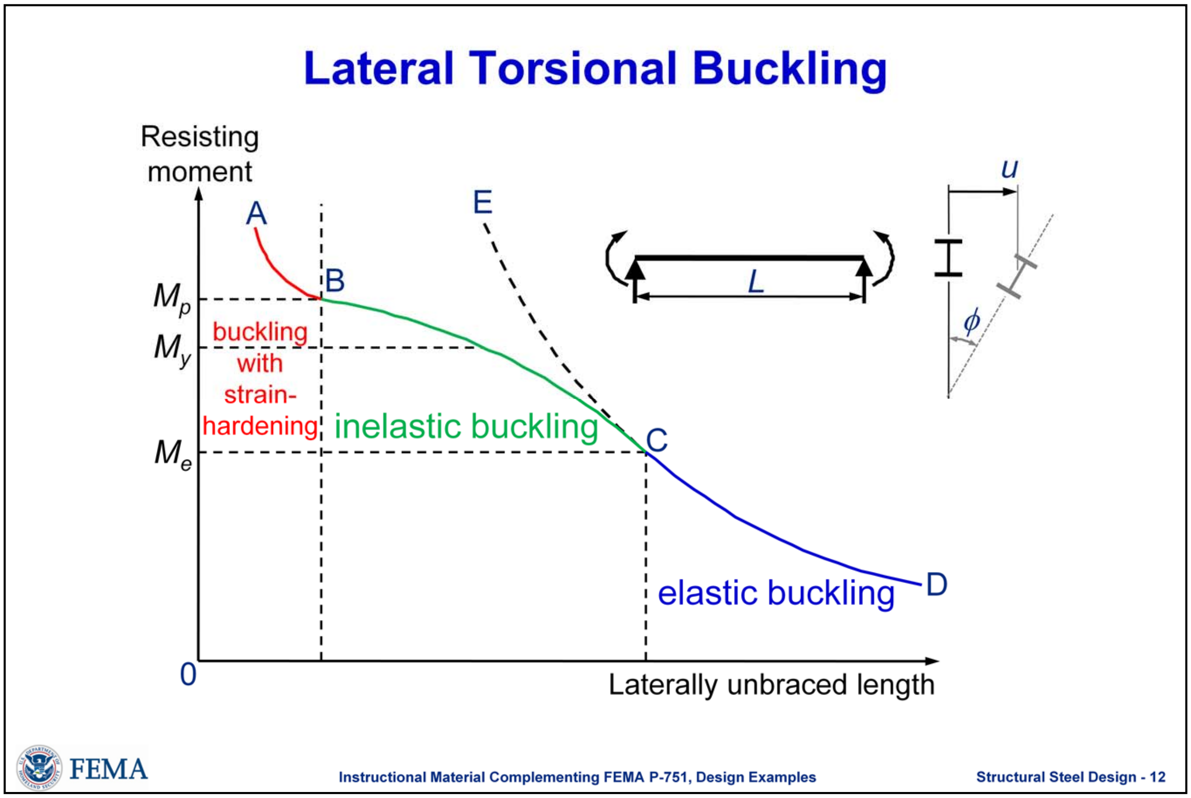
Recall from prior lesson (inelastic behavior) that structural ductility < section ductility < material ductility. Structural ductility of **steel** members further limited, as noted on slide, by local and lateral torsional buckling.

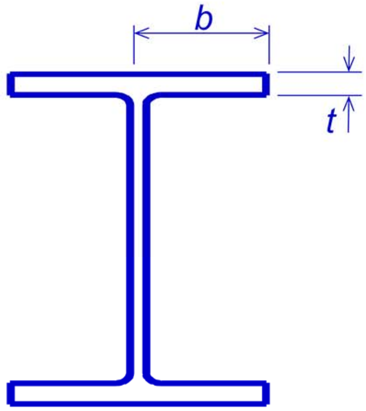


Photograph on the left is a laboratory test of a moment end-plate connection test exhibiting local buckling. Photo courtesy of Professor T. M. Murray, Virginia Tech. Photo on right illustrates lateral torsional buckling of a cold-formed steel hat section test specimen.



Effect of lateral-torsional buckling on moment capacity. Diagram for uniform bending of simple beam. Capacity above M_p due to strain hardening.

Local Buckling



Classical plate buckling solution:

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(b/t)^2} \le \sigma_y$$

Substituting μ = *0.3 and rearranging*:

$$\frac{b}{t} \le 0.95\sqrt{\frac{kE}{F_y}}$$

Example correlates width-to-thickness limits for compression flanges with ductility ratios. The mathematical derivation is not truly correct because linear mechanics are extrapolated beyond the yield level by using strain as a substitute for stress. The point is to show that stability depends on width-to-thickness ratios and that high strains require stubby elements.

Local Buckling
continued

With the plate buckling coefficient, *K*, taken as 0.7 and an adjustment for residual stresses, the expression for b/t becomes:

$$\frac{b}{t} \le 0.38\sqrt{\frac{E}{F_y}}$$

This is the slenderness requirement given in the AISC specification for compact flanges of I-shaped sections in bending. The coefficient is further reduced for sections to be used in seismic applications in the AISC Seismic specification

$$\frac{b}{t} \le 0.3\sqrt{\frac{E}{F_y}}$$

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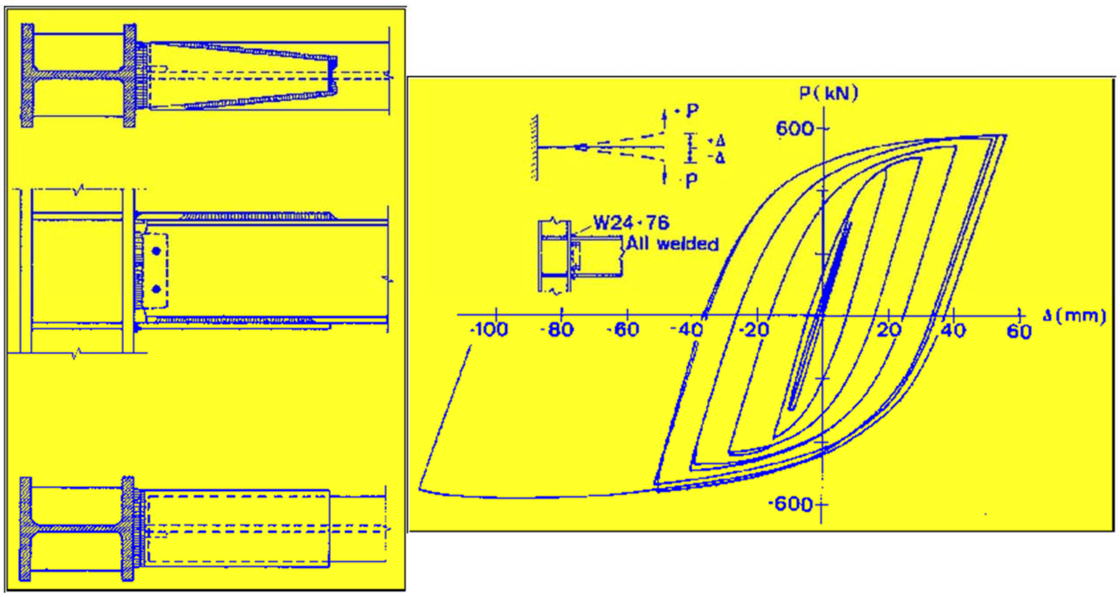
Structural Steel Design - 14

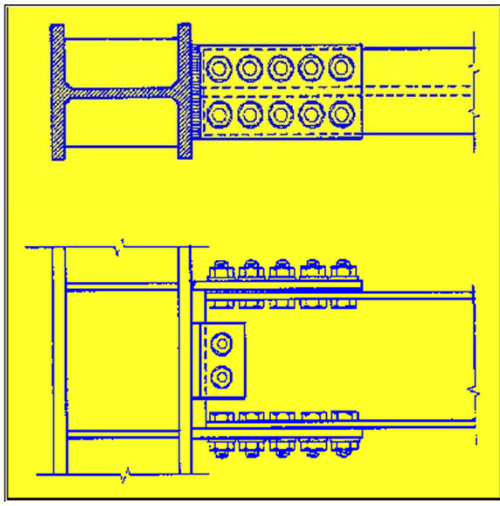
Note that AISC limits are for members expected to undergo significant inelastic deformation in a seismic event. The limits for plastic design are more liberal. These limits correspond very roughly to ductility ratio of about 10. Also note that several sections do not satisfy the limit for grade 50 steel.

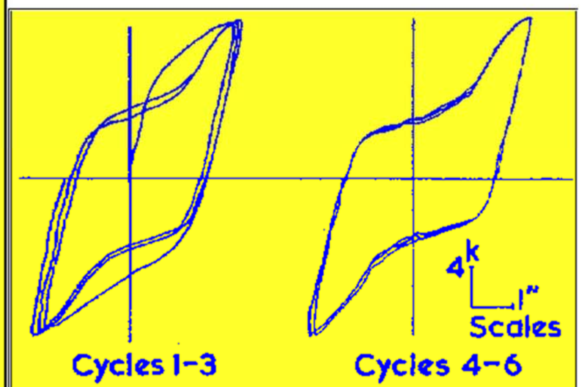
Welded Beam to Column Laboratory Test - 1960s



Load-deflection diagram for repeated reversed loads on welded joints performed under laboratory conditions in late 1960s and early 1970s. It shows relative stability of connection and its large capacity to absorb energy. Tests of joints with welded flanges and bolted webs showed similar stability and essentially same capacity. Northridge earthquake and subsequent laboratory testing have shown serious problems with particular welded joints. It turns out that it is somewhat difficult to actually achieve the behavior represented on this diagram.

Bolted Beam to Column Laboratory Test - 1960s





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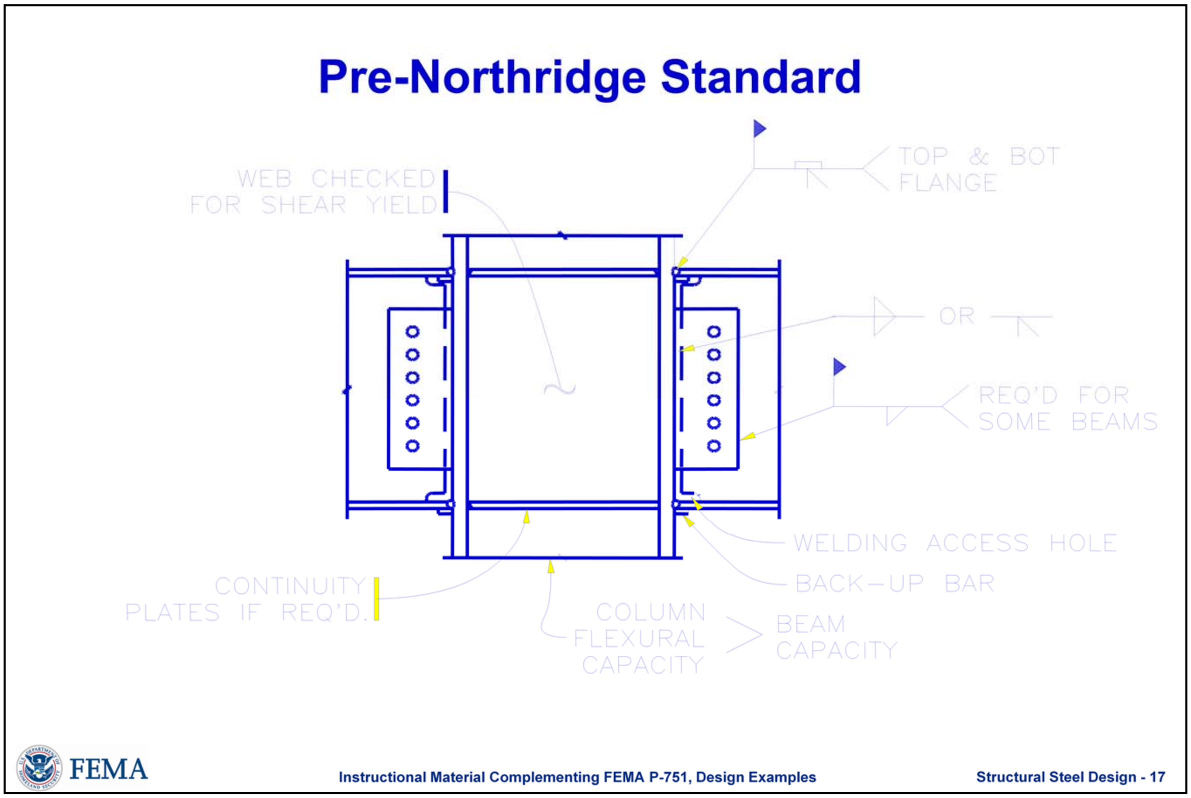
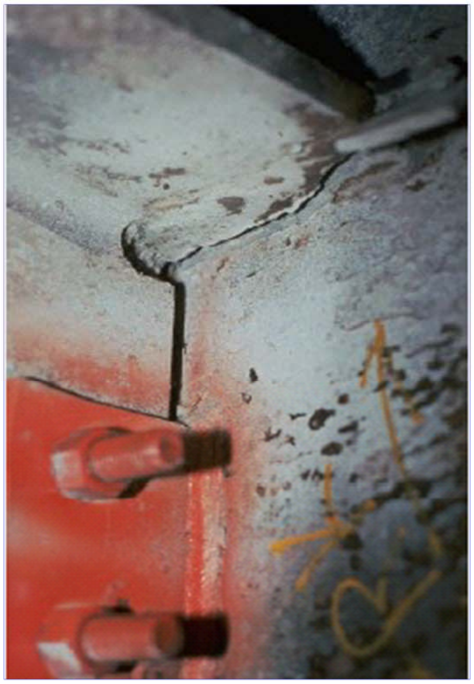
Note mildly pinched hysteresis from slipping of bolts in holes.



The Northridge earthquake revealed serious flaws in the performance of welded joints in steel moment frames. The drawing illustrates the particular detail that had received prescriptive acceptance for seismic-resistant design in the decade prior to Northridge. The only real variable was the weld from the shear tab to the beam web, which was required in beams in which the web represented more than 30% of the plastic flexural capacity.



Following the 1994 Northridge earthquake, numerous failures of steel beam-to-column moment connections were identified. This led to a multiyear, multimillion dollar FEMA-funded problem-focused study undertaken by the SAC Joint Venture. The failures caused a fundamental rethinking of the design of seismic resistant steel moment connections.

The photograph shows a fracture resulting in a divot being removed from the column flange adjacent to the toe of the full penetration weld at the beam flange.

Bottom Flange Weld Fracture Propagating Through Column Flange and Web

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Structural Steel Design - 19

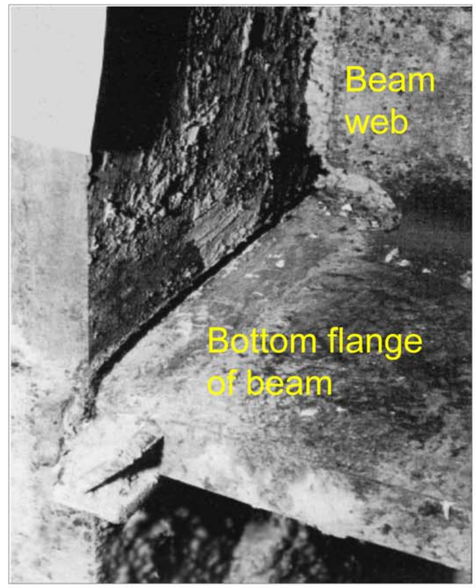
Many of the buildings that experienced failures similar to that shown in the slide exhibited very little obvious signs of distress.

Beam Bottom Flange Weld Fracture Causing a Column Divot Fracture

Photo showing crack in column flange at beam flange weld location. Note that the beam web and the weld access hole for the beam appear at the right side of the photo.

Northridge Failure

- Crack through weld
- Note backup bar and runoff tab



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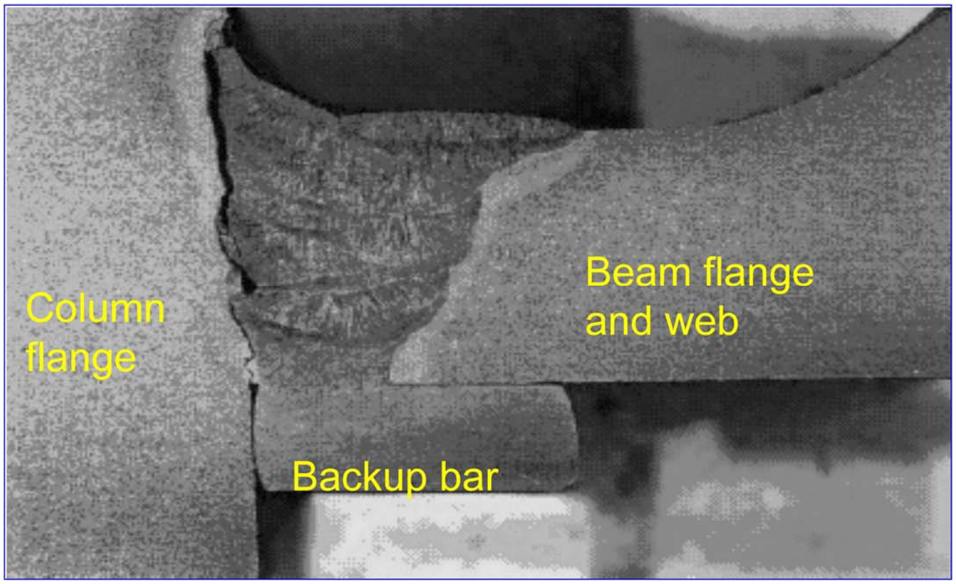
The photo is a closeup view of the weld between the bottom flange of a beam and the flange of a column. Note the crack between the weld and the column flange. Also note the backup bar and the runoff tab (the diagonal bar) at the left edge of the beam flange. (The joint had been covered with spray-on fireproofing.)

Northridge Failure



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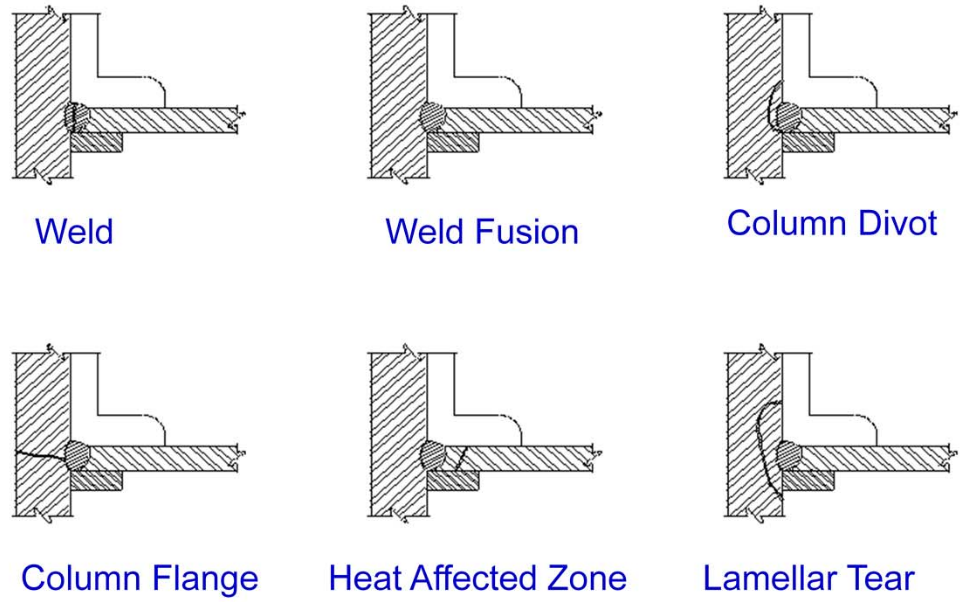
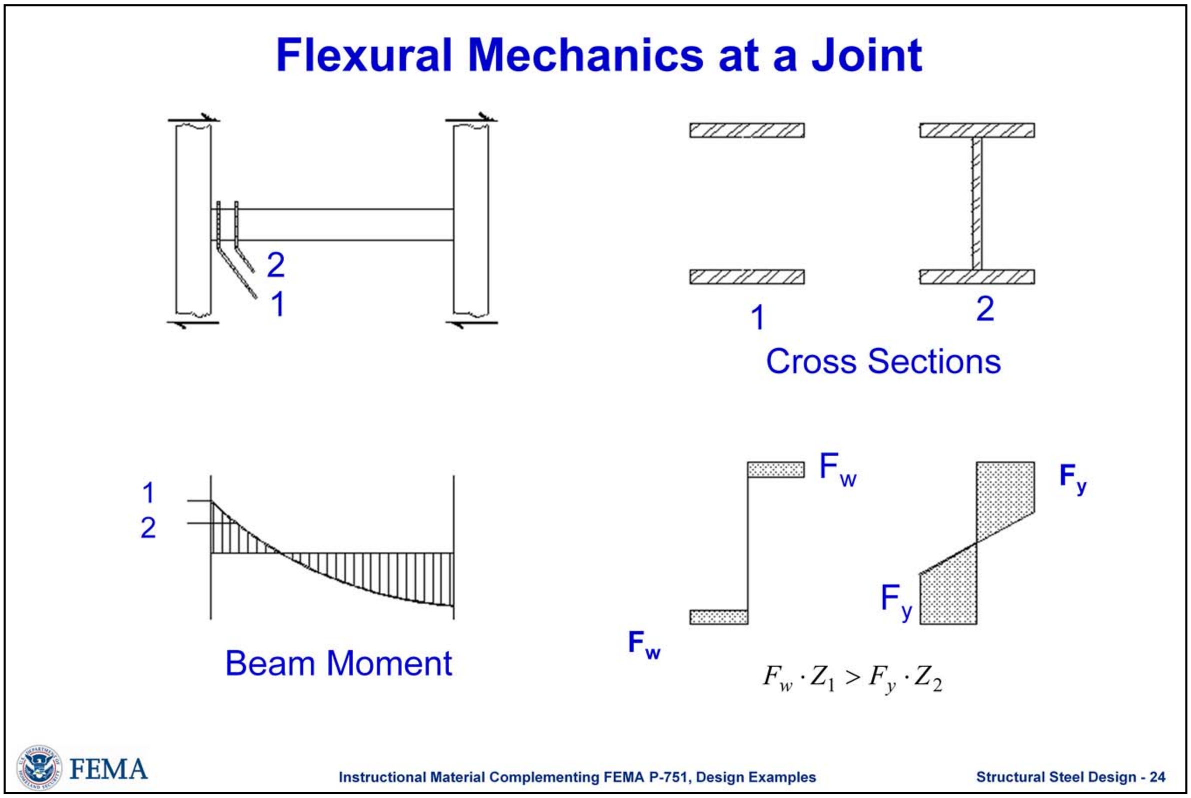
The photo shows a cut and polished specimen with a crack at the fusion line between the weld and the column flange. Note that the backup bar is not attached to the column flange, which creates a notch normal to the stress in the flange. The section is cut at the centerline of the beam; thus, the beam web beyond the weld access hole appears as the upward curving line at the right. Note that at the instant the beam flange is in tension due to lateral sway, the column flange at the weld is also likely to be in tension.

Northridge Failures



FEMA   Instructional Material Complementing FEMA P-751, Design Examples   Structural Steel Design - 23

The figures are abstracted from *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, the early SAC report (95-02) on the problems found at Northridge. It is also available as FEMA 267, August 1995. The majority of the cracks began at the root of the weld directly above the notch at the end of the backup bar. Subsequent investigation determined that significant cracks exist in such joints that have not been exposed to seismic ground motion -- due to a combination of restraint of weld shrinkage, a ready-made notch, use of materials that are not "notch tough," difficulties in inspection at the point in question, and other factors.



One of the inherent weaknesses of the joint is that the flange welds must be strong enough to develop substantial yielding in the beam away from the flange in order to dissipate significant energy. The plastic section modulus is less at the welds and the gradient in the moment as one moves away from the column flange both combine to require that the ultimate strength of the weld be substantially more than the yield of the beam. Another factor is that most available beams now have yield strengths of about 50 ksi (even though sold under A36 specification) whereas the tests done at the time the joint was developed were made on material much closer to 36 ksi yield; the strength of weld metal has not changed in the same fashion.

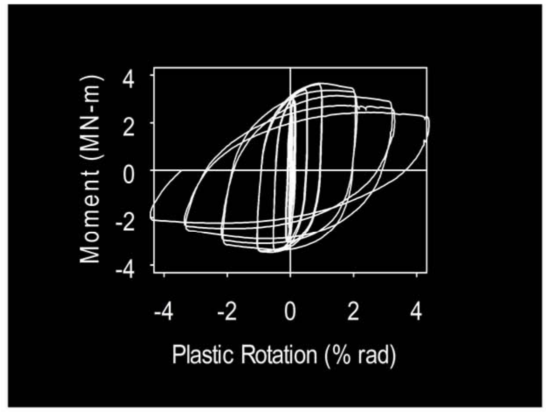
Welded Steel Frames

- Northridge showed serious flaws. Problems correlated with:
  - Weld material, detail concept and workmanship
  - Beam yield strength and size
  - Panel zone yield
- Repairs and new design
  - Move yield away from column face (cover plates, haunches, reduced beam section)
  - Verify through tests
- SAC Project: FEMA Publications 350 through 354
- AISC 358

FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 25

The slide summarizes the major points learned in the first phase of the SAC project (a FEMA-funded joint venture of SEAOC, ATC, and CUREE, which is California Universities for Earthquake Engineering). The second phase of the SAC project, also funded by FEMA, was a very substantial undertaking, and much more was learned. The profession and the steel industry will continue to feel profound effects of the Northridge event for years to come. For the reduced beam section (RBS) the beam flange width is reduced (by cutting) at a point away from the column face in order to force first yield at that point.

Reduced Beam Section (RBS) Test Specimen
SAC Joint Venture



Graphics courtesy of Professor Chia-Ming Uang, University of California San Diego

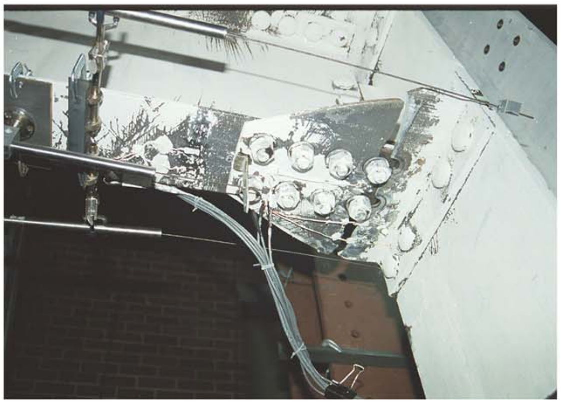
A beam-to-column joint test that utilizes the reduced beam section (RBS) detail is shown in the photograph. The experimental hysteresis plot is also shown. The RBS detail has been recognized as a pre-qualified connection in FEMA 350 and subsequent AISC documents. The behavior of this connection (actually member) detail is ductile and forces the inelasticity to occur in the beam section away from the beam-to-column welding and bolting. Graphics courtesy of Professor Chia-Ming Uang, University of California San Diego.

T-stub Beam-Column Test
SAC Joint Venture

Photo courtesy of Professor Roberto Leon, Georgia Institute of Technology

This slide shows a photo of a T-stub connection prior to testing. A hydraulic actuator was used to load the system, and the column was restrained by load sensing pins. A series of six beam-column tests that incorporated T-stubs very similar to those tested in a component test series were tested to failure. The beam-column tests were intended to provide benchmarks for relating the component test data to actual connections. Courtesy of Professor Roberto Leon, Georgia Institute of Technology and Professor James Swanson, University of Cincinnati.

T-Stub Failure Mechanisms



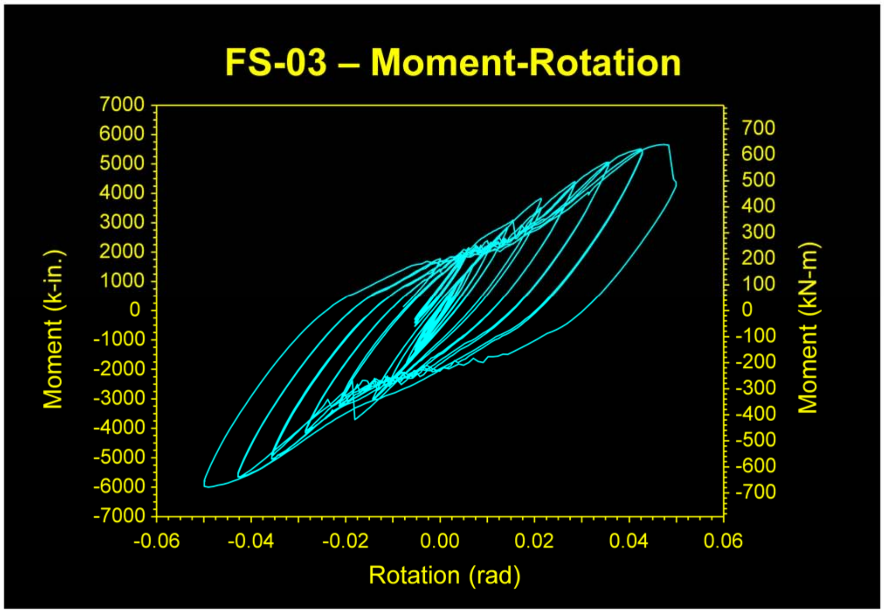
Net section fracture in stem of T-stub

Plastic hinge formation -- flange and web local buckling

Photos courtesy of Professor Roberto Leon, Georgia Institute of Technology

Two of the beam-column tests failed when the stems of the T-stubs fractured as is shown in the left photo on this slide. Because the T-stubs failed before the beams, these two connections would be classified as partial strength. Four of the connections failed with plastic hinges forming in the beam as is shown in the photo on the right in this slide; these connections would be considered full strength. Also notice the severe flange and web local buckling of the beam. Courtesy of Professor Roberto Leon, Georgia Institute of Technology and Professor James Swanson, University of Cincinnati.

T-Stub Connection Moment Rotation Plot



Graphic courtesy of Professor Roberto Leon, Georgia Institute of Technology

This slide shows the moment-rotation curve obtained during the first beam-column test. The connection failed with a net section fracture of the bottom T-stub. The connection exhibited substantial rotation capacity and ductility. Notice, too, that there is relatively little pinching and that the loss of slip resistance is not as pronounced as it was in the component tests. Courtesy of Professor Roberto Leon, Georgia Institute of Technology and Professor James Swanson, University of Cincinnati.

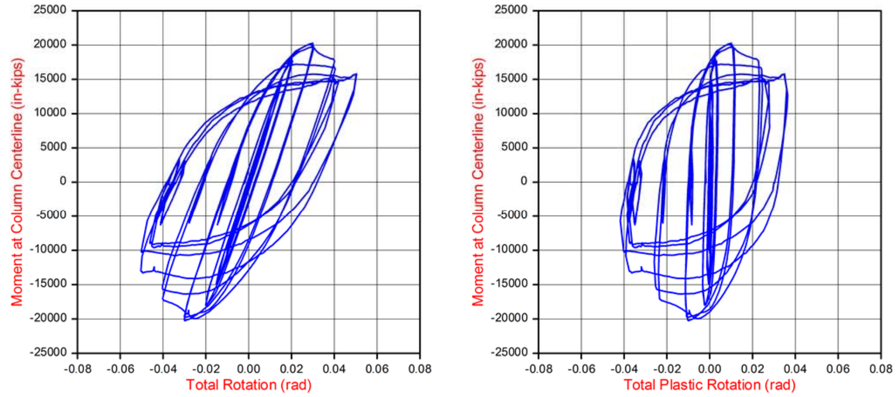
Extended Moment End-Plate Connection Results

Photo courtesy of Professor Thomas Murray, Virginia Tech

FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 30

Photo is of a laboratory test specimen of an extended moment end-plate connection. The “thick plate” behavior results in the development of a plastic hinge at the end of the stiffened section of the beam as the significant local flange and web buckling illustrate. The photograph is courtesy of Professor Thomas Murray, Virginia Tech.

Extended Moment End-Plate Connection Results



(a) Moment vs Total Rotation **(b) Moment vs Plastic Rotation**

Graphics courtesy of Professor Thomas Murray, Virginia Tech

FEMA

Hysteresis plots from an extended, stiffened, moment end-plate connection. Graphics courtesy of Professor Thomas Murray, Virginia Tech

Ductility of Steel Frame Joints

Limit States

Welded Joints
- Brittle fracture of weld
- Lamellar tearing of base metal
- Joint design, testing, and inspection

Bolted Joints
- Fracture at net cross-section
- Excessive slip

Joint Too Weak For Member
- Shear in joint panel

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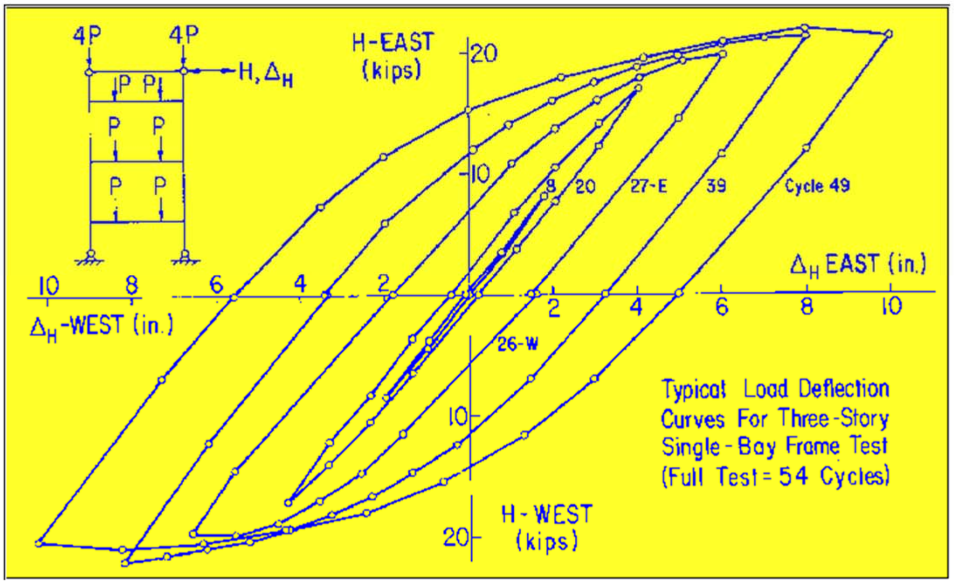
Other failures not on slide were due to crushing or buckling of column web, distortion of column flange, or panel zone shear yielding or buckling.

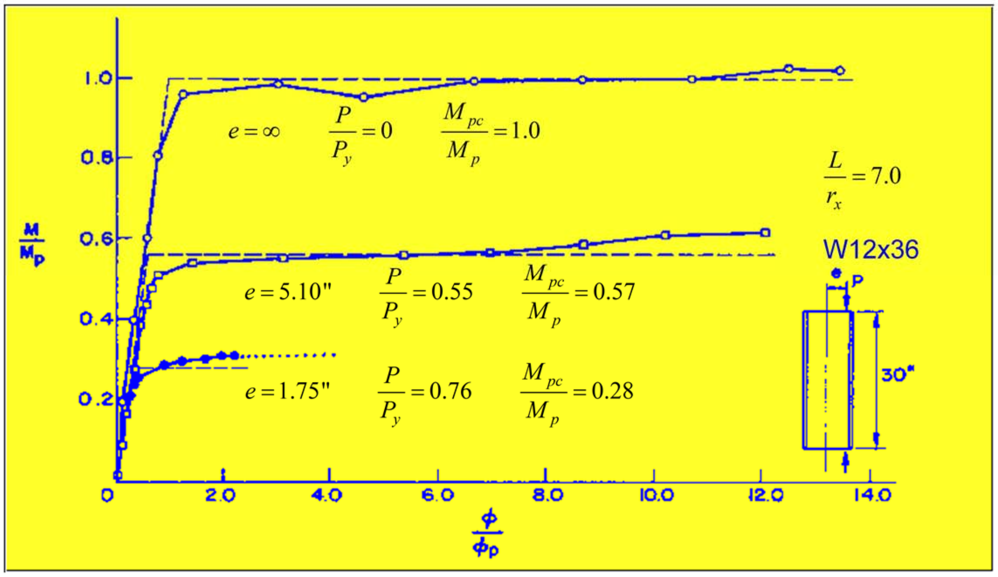
Multistory Frame
Laboratory Test



Load deflection plot for three-story, single-bay frame test. Heavier axial loads would limit stability at large lateral deflections.

Flexural Ductility

Effect of Axial Load



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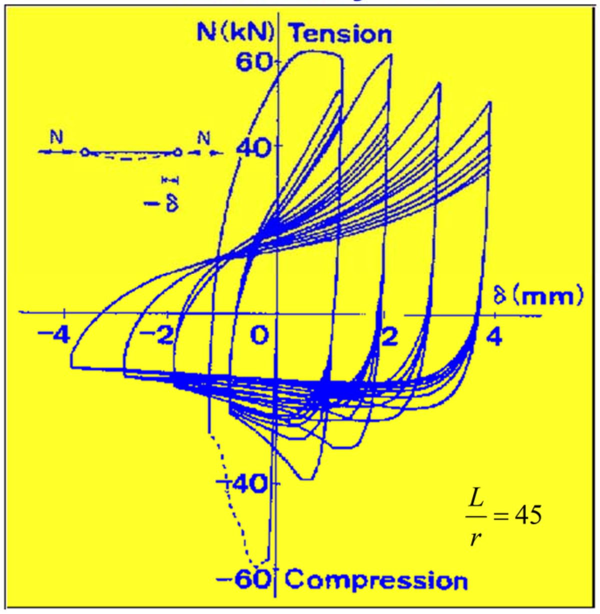
Moment capacity depends on axial load. Diagram shows moment-curvature for three different eccentricities on short column.

Axial Strut

Laboratory test



FEMA

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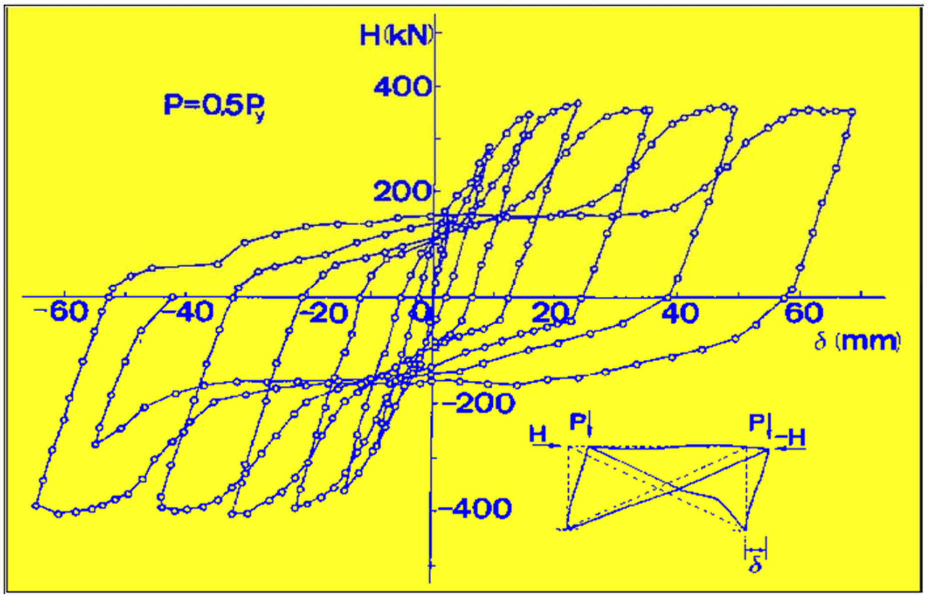
Structural Steel Design - 35

Plot representative of column or brace. Plot of axial load versus axial deflection for low-to-medium slenderness member (*L*/*r* = 45). Tension in upper right quadrant and compression in lower left. Theoretical buckling load reached on only the first cycle. Note the degrading stiffness, decreasing compressive strength, and permanent set caused by tension yielding. Other testing has shown prevention of local buckling is crucial to maintaining any post-buckling strength.

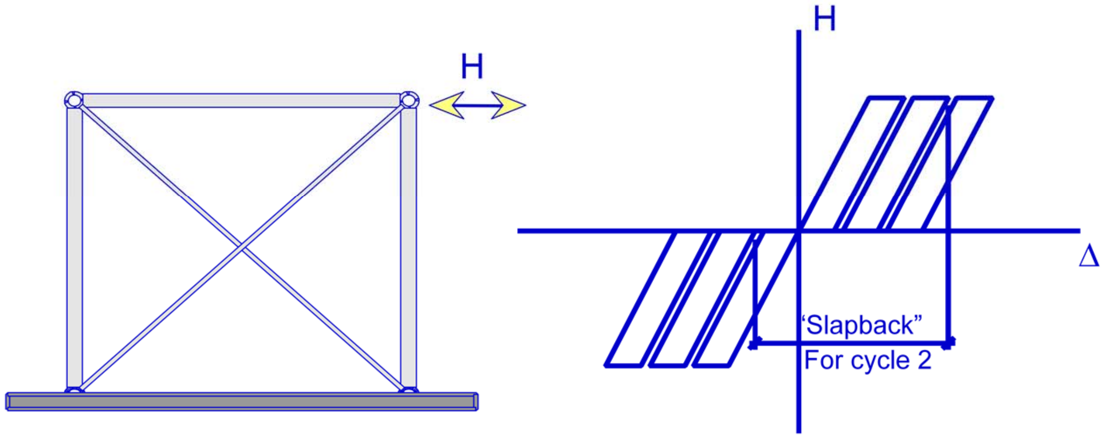
Cross Braced Frame

Laboratory test



Overall performance of concentrically braced frame governed by axial behavior of braces. Frame has braces in both directions. Fatness of loops depends on slenderness of bracing members; lower slenderness ratios give fatter loops. Note the degrading stiffness.

Tension Rod (Counter) Bracing
Conceptual Behavior



Tension ties stressed beyond yield experience permanent deformation. Each load reversal results in a further increment of deformation; structure moves, unrestrained, through larger range of motion. If braces do not have equal strength, structure will accumulate deflections in one direction, "racheting" sideways as cycles go on.

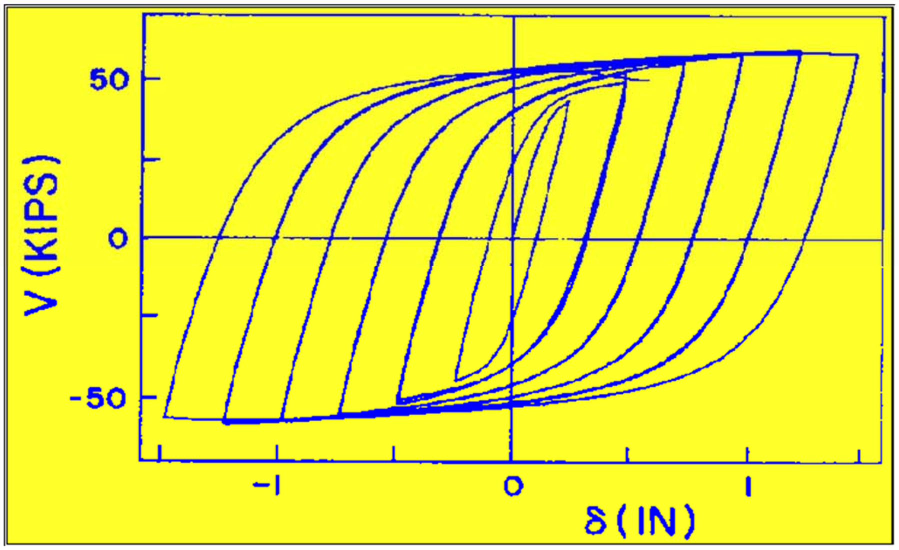
Eccentrically Braced Frame

FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 38

Photograph of eccentrically braced frame (EBF) under construction. Photo from *Modern Steel Construction.*

Eccentrically Braced Frame

Lab test of link



Yield in eccentrically braced frames occurs in link (short portion of beam between braces). Shear yielding produces very fat and stable hysteresis loops. Braces, beams, and columns designed to remain elastic at load level that causes link yielding.

Steel Behavior − Summary

- Ductility
  - Material inherently ductile
  - Ductility of structure < ductility of member < ductility of material
  - Achieved through detailing
- Damping
  - Welded structures have low damping
  - More damping in bolted structures due to slip at connections
  - Primary energy absorption is yielding of members

FEMA

Steels used in construction are typically extremely ductile materials. Ductility of the steel structure or system is less (sometimes many times less) than the ductility of the material because only a portion of material in the system will actually experience yielding. Damping is typically not high in steel structures, especially in welded structures. Bolted structures exhibit a higher degree of damping due to slip at connections. Unlike structures of other materials, the primary energy absorption mode in steel structures is yielding of members, usually in bending but sometimes in shear, axial tension, or compression.

Steel Behavior − Summary

- Buckling
  - Most common steel failure under earthquake loads
  - Usually not ductile
  - Local buckling of portion of member
  - Global buckling of member
- Fracture
  - Nonductile failure mode under earthquake loads
  - Heavy welded connections susceptible
  - Net section rupture

FEMA

Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 41

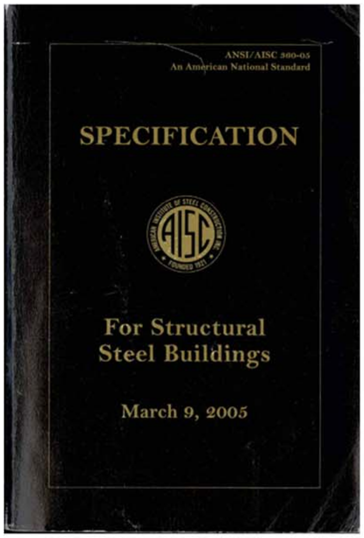
Typical failure in steel structure subjected to earthquake is buckling, either global buckling of member, local buckling of portion of member, or global buckling of entire structure. Another potential weakness in steel structures is susceptibility to fracture; primarily a concern in heavy welded connections, members with notches, and in cold environments. The Northridge earthquake caused substantially more concern about fracture.

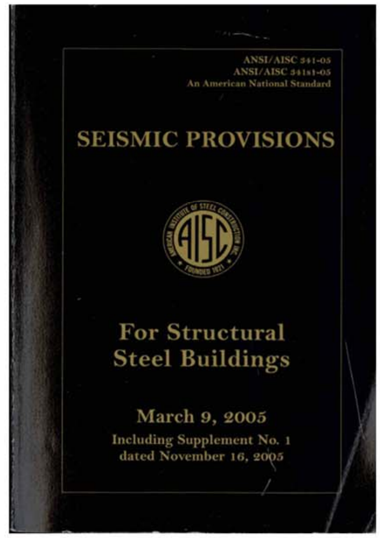
NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- **Reference standards and design strength**

Table of contents: Reference standards and design strength. The *Provisions* references commonly accepted codes for each material and indicates how requirements in those standards must be modified for use with the *NEHRP Recommended Provisions* seismic design procedure. Special additional provisions are also included in each materials chapter.

Steel Design Specifications





FEMA

Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 43

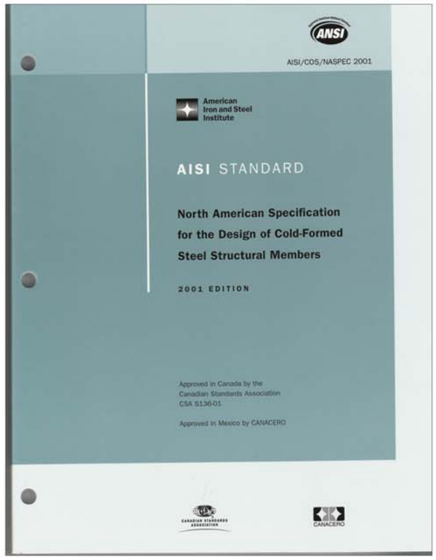
The referenced standard for structural steel (hot rolled shapes and plates, and sections built up from hot rolled shapes and plates) is the *Specification for Structural Steel Buildings*, March 2005. The specification governing seismic design of steel buildings is *Seismic Provisions for Structural Steel Buildings*, March 2005 and the Supplement, which was not complete at the time of this writing. These documents supersede the earlier specifications from AISC.

Using Reference Standards
Structural Steel

Both the AISC LRFD and ASD methodologies are presented in a unified format in both the *Specification for Structural Steel Buildings* and the *Seismic Provisions for Structural Steel Buildings*.

Slide discusses reference standards.

Cold Formed Steel Standard

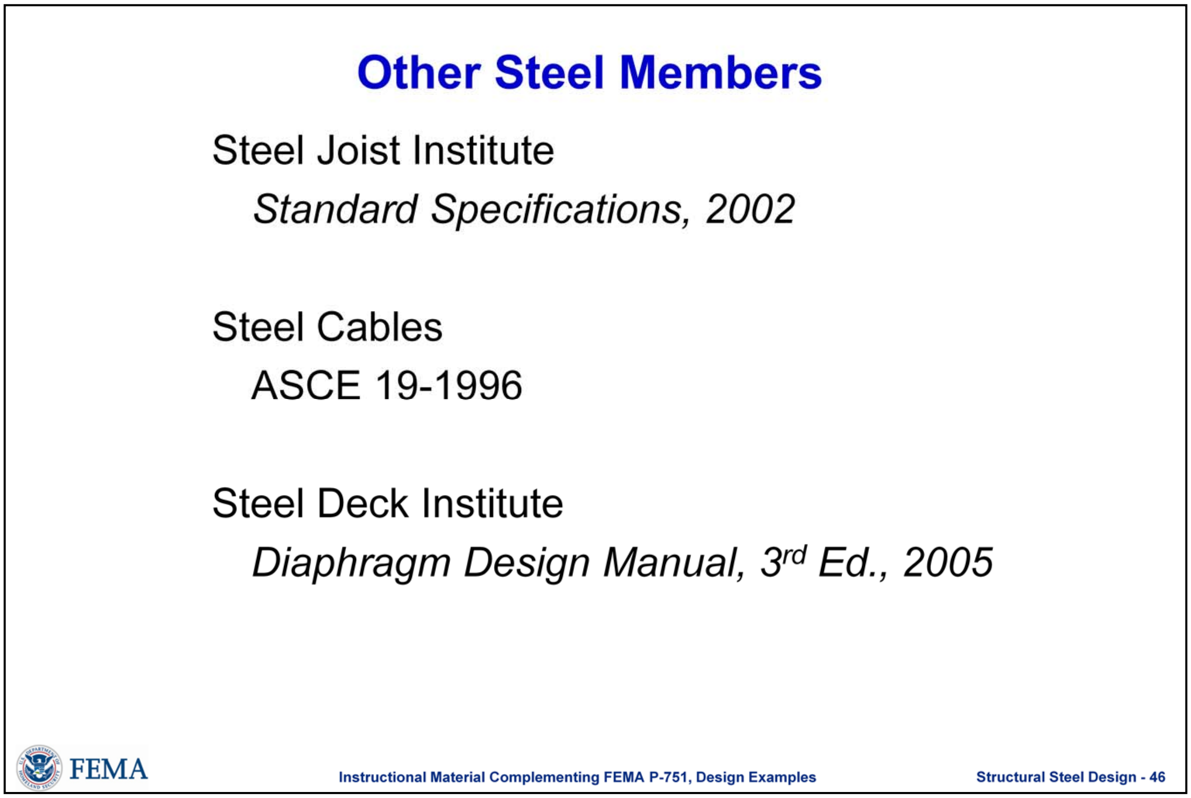


FEMA

Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 45

Reference standard for cold formed steel is the AISI *North American Specification for the Design of Cold-formed Steel Structural Members* (shown on slide). It includes both ASD and LRFD. ASCE 8-90 is a similar standard (LRFD only) for cold-formed stainless steel. The *Provisions* adjusts the load factor to be compatible.



For design of other specialized steel members, the *Provisions* refers user to standards published by Steel Joist Institute and the ASCE 19 standard for cables. The *Provisions* indicates how SJI allowables should be increased to maximum strengths for use in seismic resistant design and give guidelines for modification of cable allowables. Note the Steel Deck Institute's *Diaphragm Design Manual* is no longer referenced in the *Provisions* although it is a good source (do **not** use the 2.75 "factor of safety"). There are also other sources for steel deck diaphragms.

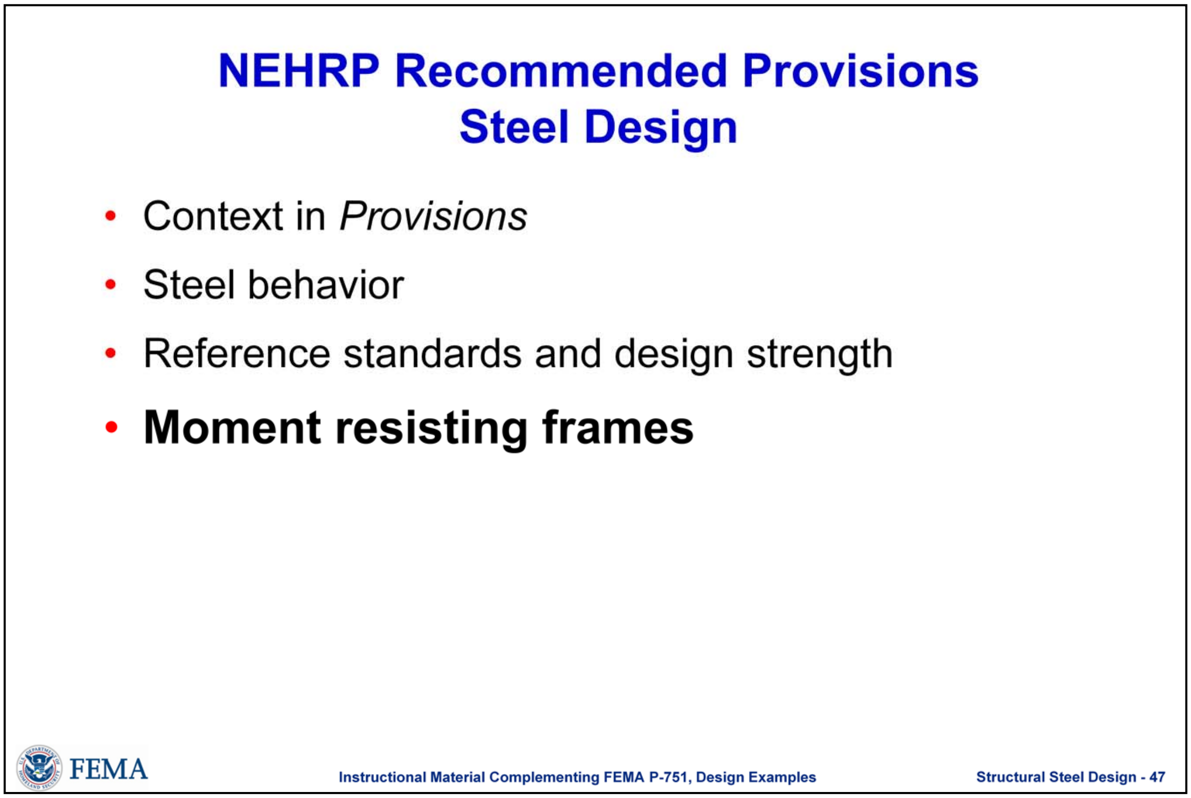


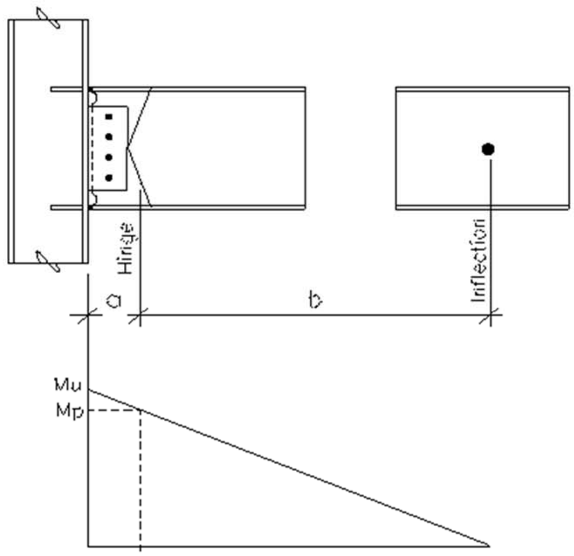
Table of contents: Moment resisting frames.

Steel Moment Frame Joints

Frame	Test	θ_i	Details
Special	Req'd	0.04	Many
Intermediate	Req'd	0.02	Moderate
Ordinary	Allowed	N.A.	Few

θ_i is shorthand for the total (elastic plus inelastic) rotation at the beam-to-column connection. Elastic drift at yield is on the order of 0.01 radians. The capacity is very sensitive to detail; thus, the requirement for testing representative joints. AISC Seismic includes an appendix with a testing protocol.

Steel Moment Frame Joints



$$M_u \approx M_p \cdot \frac{a+b}{b}$$

$$F_y^* = R_y \cdot F_y$$

$$F_u \approx F_y^* Z \cdot \frac{a+b}{b} \cdot \frac{1}{A_f d} \approx 1.7\, F_y^*$$

This illustrates one aspect of the problem with the welded joint. The old connection usually must develop its flexural strength solely through the flange welds at the face. The moment is somewhat higher than M_p in order to develop an appreciable hinge in the beam. The larger moment and the reduced section both increase the demand stress above the actual yield of the beam. The number 1.7 is derived from common proportions for the contribution of the flange to the total plastic section modulus and the ratio of hinge length to inflection point distance plus overstrength commonly found in A36 steel (which is no longer the steel of choice for hot-rolled W shapes.) Additional issues include the geometric stress concentration, weld imperfections, shear lag from beam web to flange tip, residual stresses from welding heat, loss of ductility where triaxial tension exists, etc.

Panel Zones

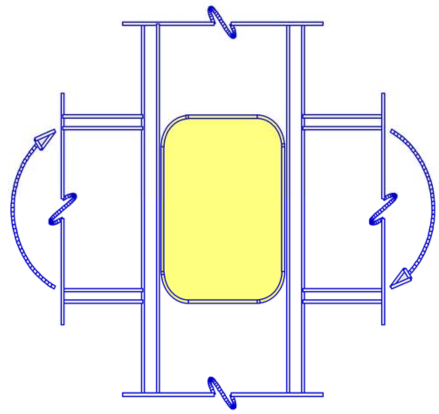
Special and intermediate moment frame:

- Shear strength demand:

  Basic load combination

  or

  $\phi R_y M_p$ of beams
- Shear capacity equation
- Thickness (for buckling)
- Use of doubler plates (not economical, try to increase col. size instead)



Panel shear often controls, and this may require larger column or "doubler" plate on web, which is expensive to fabricate. SAC testing has shown that panel zone yield contributes to fracture of beam flange welds, apparently due to the concentration of strains at that location. However, some yielding in the panel zone is considered to be a good way to dissipate energy. The final SAC recommendations require that the yield strength of the shear panel zone be at least strong enough to reach the beginning of yield in the beam flexural hinge areas but not full development of the flexural hinge.

Steel Moment Frames

- Beam shear: $1.1R_yM_p$ + gravity
- Beam local buckling
  - Smaller *b*/*t* than for plastic design
- Continuity plates in joint per tests
- Strong column - weak beam rule
  - Prevent column yield except in panel zone
  - Exceptions: Low axial load, strong stories, top story, and non-SFRS columns

Note the tighter restrictions on *b*/*t* and *h*/*t* than required for plastic design. Beam flange continuity plates are now required if included in the test specimens. It is not an analytical check. The *b*/*t* limit rules out a few common beam shapes for grade 50 steel.

Steel Moment Frames

- Lateral support of column flange requirements
  - Top of beam if column elastic
  - Top and bottom of beam otherwise
  - Amplified forces for unrestrained
- Lateral support of beams requirements
  - Both flanges
  - Spacing < $0.086 r_y E / F_y$

Limits to prevent member buckling. The limit is for special frames and it is different from the general limits in the main spec, which applies to the compression flange and depends on the square root of F_y. Refer to Section 9.8 of the 2005 AISC Seismic specification for additional details. The Commentary to the 2005 AISC specification includes the following:

"Spacing of lateral braces for beams in SMF systems is specified not to exceed 0.086 ry E / Fy. This limitation, which is unchanged from previous editions, was originally based on an examination of lateral bracing requirements from early work on plastic design and based on limited experimental data on beams subject to cyclic loading. Lateral bracing requirements for SMF beams have since been investigated in greater detail in Nakashima, Kanao and Liu (2002). This study indicates that a beam lateral support spacing of 0.086ryE/Fy is appropriate, and slightly conservative, to achieve an interstory drift angle of 0.04 radian."

Prequalified Connections

ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*

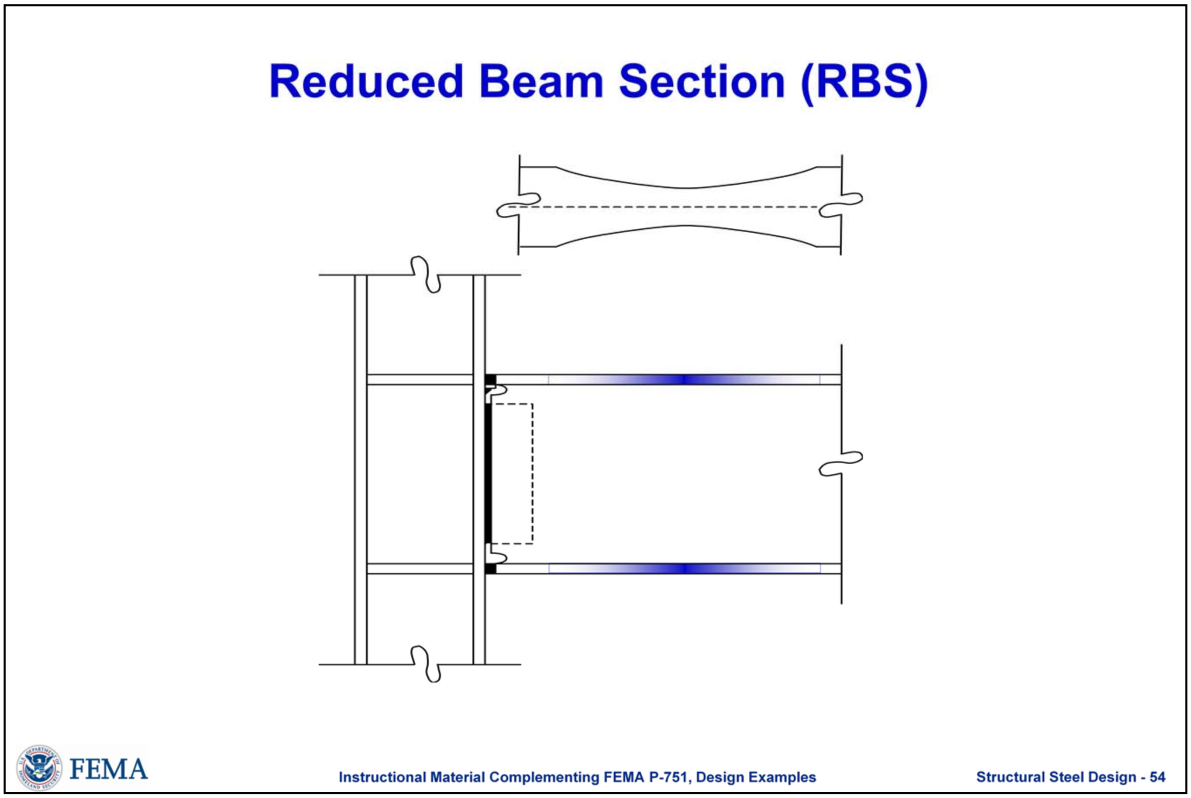
- Reduced Beam Section Connections
- Bolted Stiffened and Unstiffened Extended Moment End Plate Connections

Additional connections addressed in FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*:

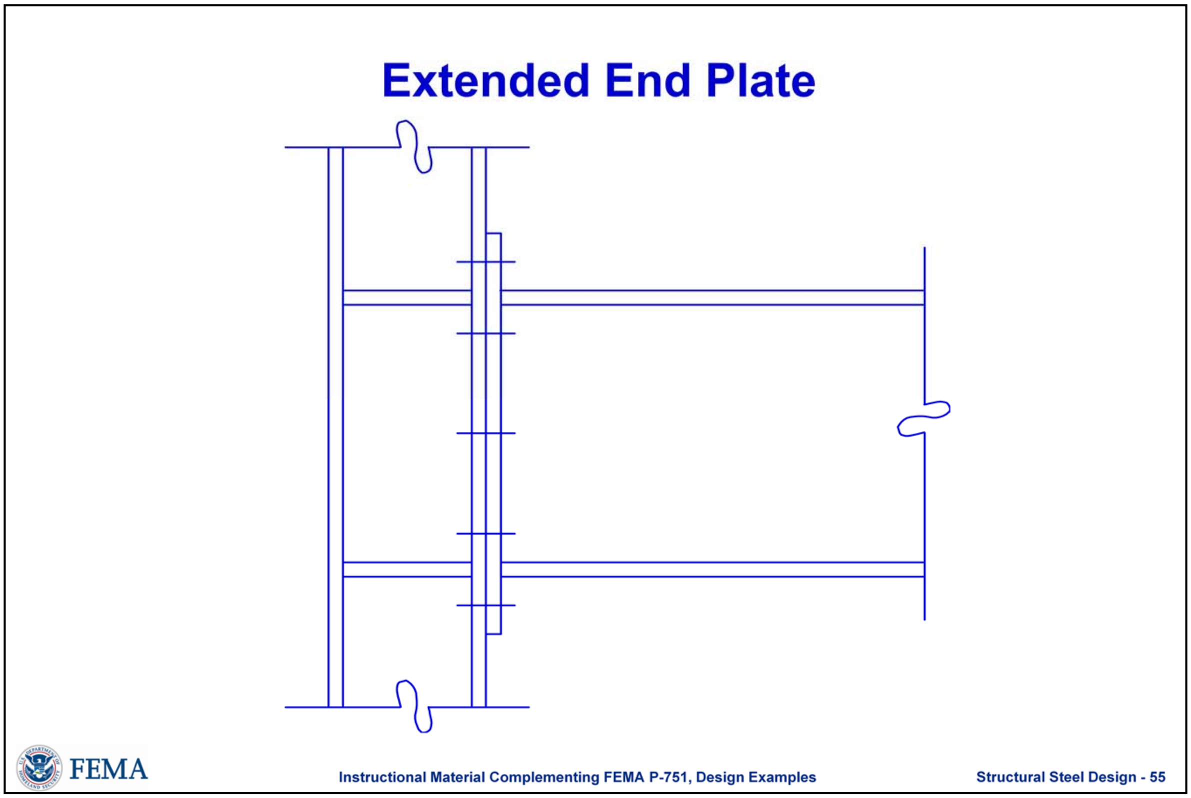
- Welded Unreinforced Flange
- Welded Free Flange Connection
- Welded Flange Plate Connection
- Bolted Flange Plate Connection

FEMA

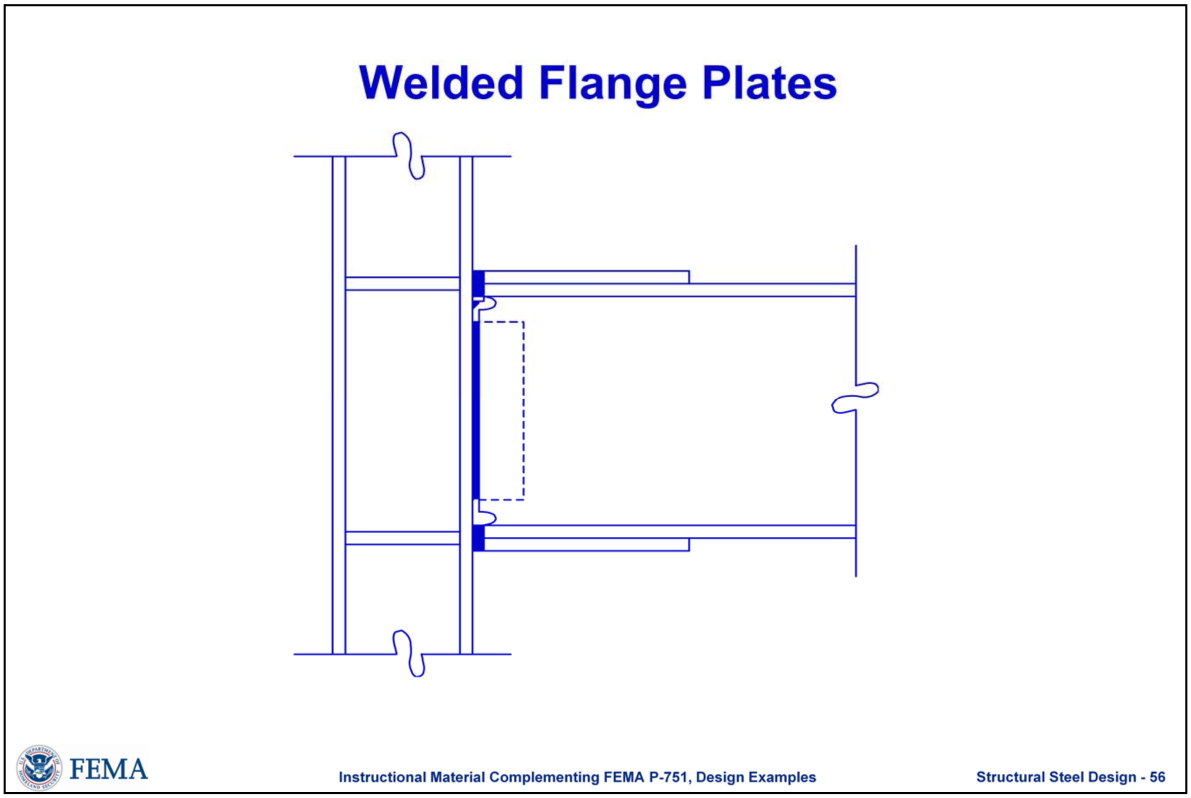
FEMA 350 resulted from the SAC Joint Venture project. Subsequently, an ANSI/AISC committee was established to create ANSI/AISC 358-05. The work by this committee is continuing and additional connections are being addressed. The ANSI/AISC 358 standard is available for download from the AISC website.



The slide illustrates a beam-to-column connection using the reduced beam section (RBS) connection. The beam flanges and shear plate are welded to the column. The reduction of the beam section forces hinging to occur in the beam away from these connection details.

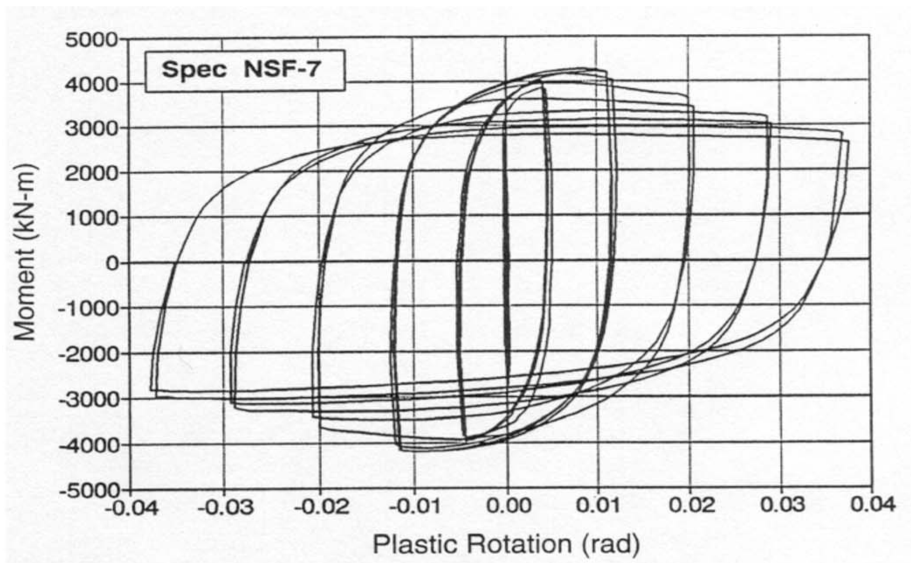


The slide illustrates an extended moment end-plate connection.



Slide illustrates a beam-to-column connection using welded cover plates on the beam flanges.

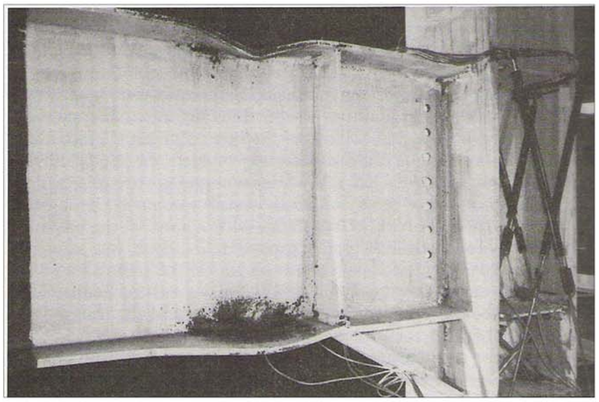
Excellent Moment Frame Behavior

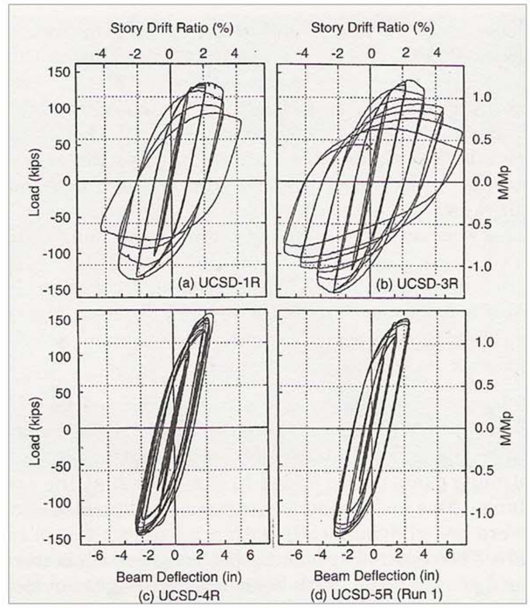


FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 57

This slide illustrates excellent moment frame behavior as noted by the large, full hysteresis loops that exhibit very little strength degradation out to a plastic rotation of approximately 0.04 rad. The test was of a cover plated beam with fully welded web conducted at the University of Texas under the direction of Professor Michael Englehardt

Excellent Moment Frame Behavior

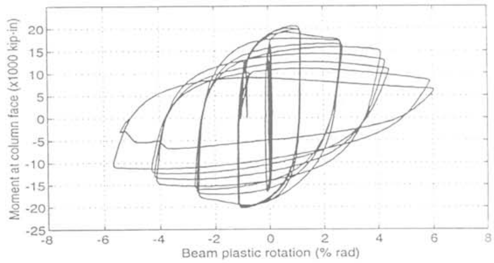


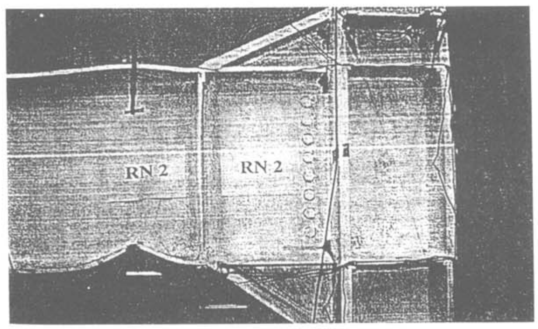


FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 58

The photograph is of a welded bottom haunch connection that exhibits excellent hysteretic behavior. The test was conducted at the University of California at San Diego under the direction of Professor Chia-Ming Uang.

Excellent Moment Frame Behavior





FEMA

Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 59

The photograph is of a welded top and bottom haunch connection that exhibits excellent hysteretic behavior. The test was conducted at the University of California at Berkeley under the direction of Andrew Whitaker and Vitelmo Bertero.

Special Moment Frames

Seven Story Office Building, Los Angeles

Perimeter Moment Frames, all bays engaged

S_{DS}=1.0

S_{D1}=0.6

Occupancy Category II

Seismic Design Category D

Design Parameters (Table 12.2-1)

- R=8
- C_d=5.5
- Ω_0=3.0

FEMA

Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 60

The information on the following several slides are taken from Example 6.2 (Alternate A) of the NEHRP *Design Examples* volume (FEMA 751). FEMA 751 was written based on the 2009 *NEHRP Recommended Provisions*. The example complies with the 2005 AISC Seismic specification.

Special Moment Frame Example

Structural Materials:

Concrete (all floors) = 3.0 ksi lightweight

Other Concrete = 4.0 ksi normal weight

Steel:

Wide Flange Sections= ASTM A992 Grade 50

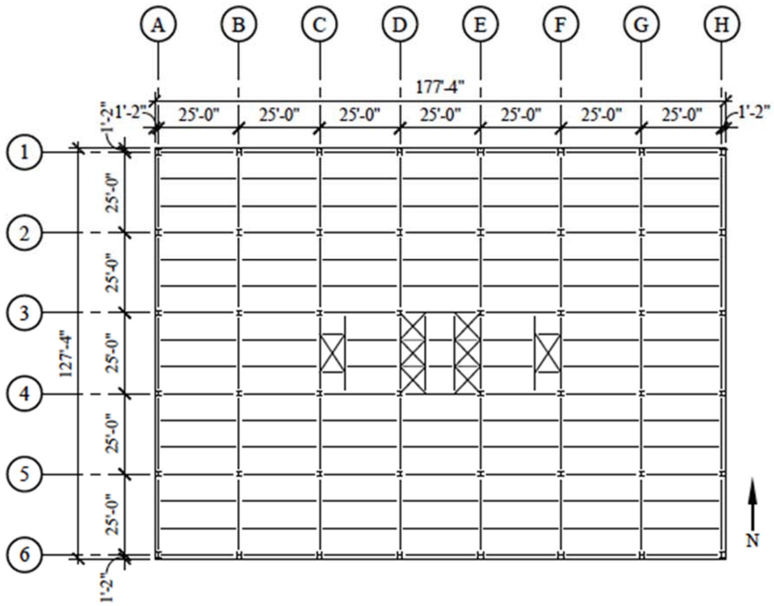
HSS= ASTM A500 Grade B

Plates= ASTM A36

Structural Materials

Special Moment Frames

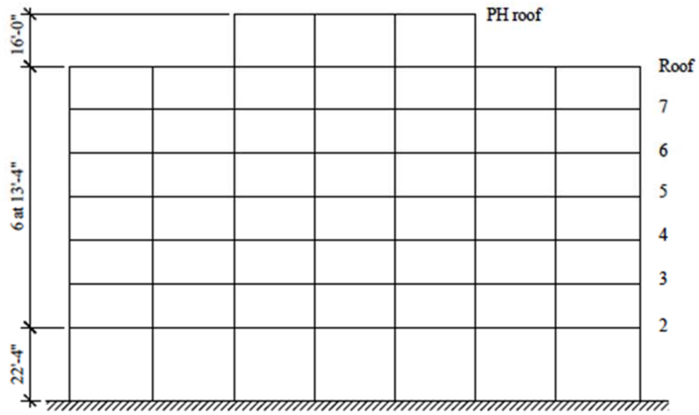
Plan of Building



FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 62

This diagram is a full floor plan of the building. There are seven bays in the E-W direction and five bays in the N-S direction. The design is illustrated for loads acting in the E-W direction.

Special Moment Frames
Elevation of Building

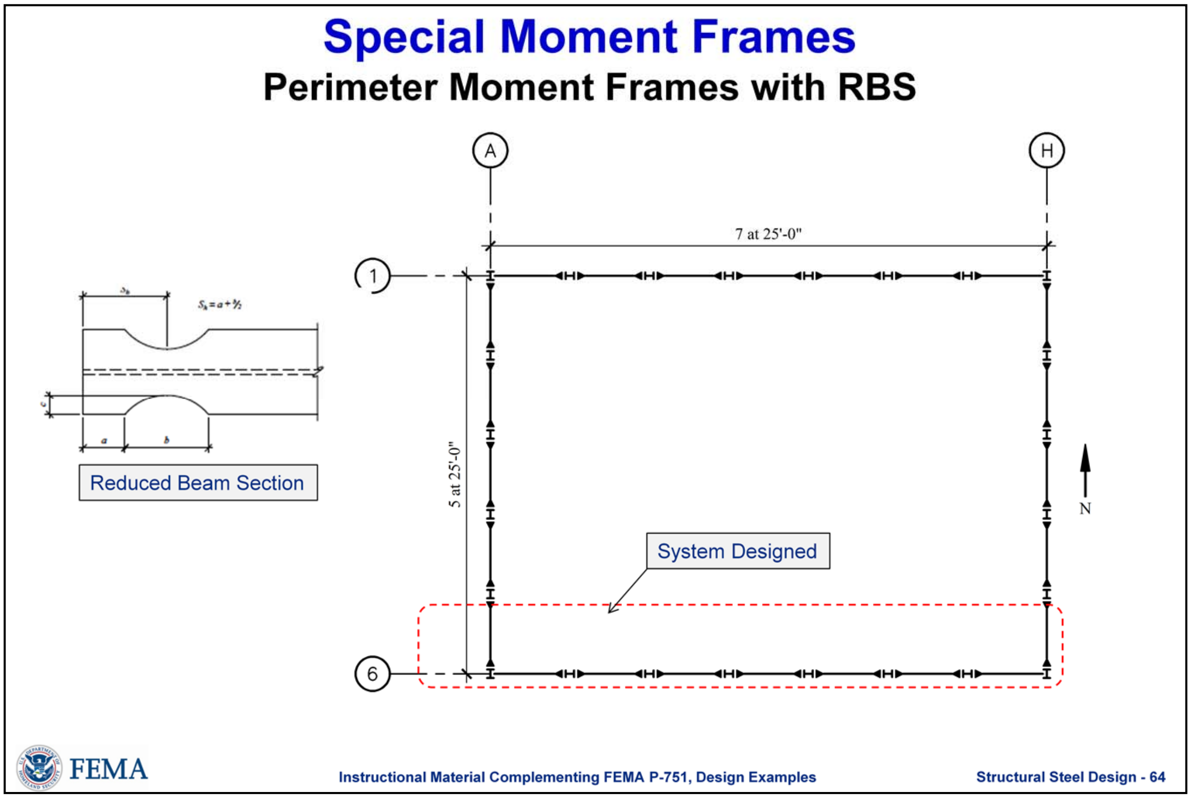


FEMA

Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 63

This Diagram is an elevation showing the North face of the building. There are seven full stories, plus a partial penthouse.



The perimeter special moment frames are shown on this diagram. The seven-bay frame that acts in the E-W direction is designed in this example.

Special Moment Frames

The following design steps will be reviewed:

- Compute Lateral Loads
- Select preliminary member sizes
- Check member local stability
- Check deflection and drift
- Check torsional amplification
- Check the column-beam moment ratio rule
- Check shear requirement at panel zone
- Select connection configuration

FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 65

The design steps in this slide will be briefly reviewed in the subsequent slides. Additional explanation and detail is available in the FEMA 751. A portion of the East-West frame at level two will be used to illustrate various calculations.

Special Moment Frames

Building Weight:

Penthouse Roof = 94 kips
Lower Roof = 1,537 kips
Typical Floor = 1,920 kips
Total = 94 + 1,537 + 6(1,920) = 13,151 kips

Building Period:

$T_a = C_t h_n^x = (0.028)(102.3)^{0.8} = 1.14$ sec.
$T = C_u T_a = (1.4)(1.14) = 1.596$ sec.

Design Base Shear:

$C_s = S_{D1}/(T/(R/I) = 0.6/(1.596/(8/1)) = 0.047$ <<< CONTROLS
$C_{s,min} = 0.044\ S_{DS} I = 0.044(1.0)(1) = 0.044$
$V = C_s W = 0.047(13,151) = 618$ kips.

FEMA

Building weights are given for the entire building.

Special Moment Frames

Select preliminary member sizes – The preliminary member sizes are given in the next slide for the frame in the East-West direction. These members were selected based on the use of a 3-Dimensional model analyzed using the program *ETABS*. As will be discussed in a subsequent slide, the drift requirements controlled the design of these members.

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Instructional Material Complementing FEMA P-751, Design Examples

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Checking preliminary sizes for special moment frames.

SMF Example – Preliminary Member Sizes

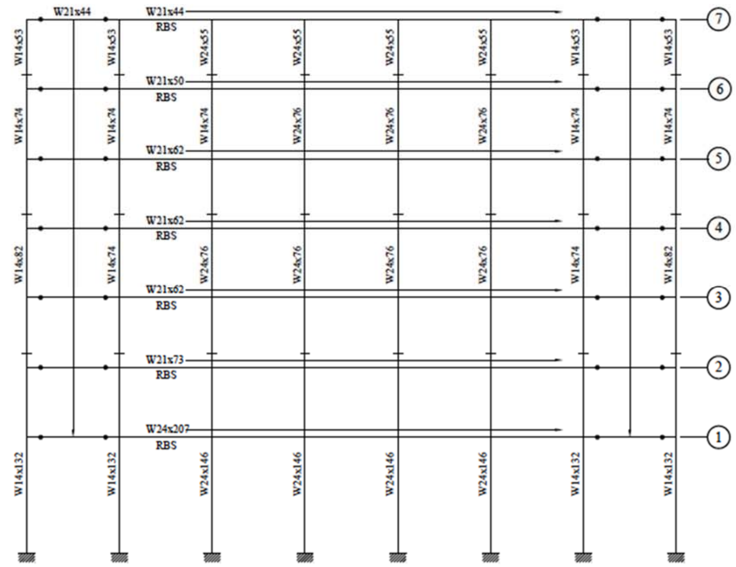

Figure 6.2-3 SMRF frame in E-W direction (penthouse not shown)

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The preliminary member sizes were determined from the use of a structural analysis model and were based on satisfying the drift limits for this design. FEMA 751 indicates that the drift requirements govern over the strength requirements, which is often the case for SMF designs.

Note that this example is based on Chapter 6.2 of the FEMA 751 Examples document.

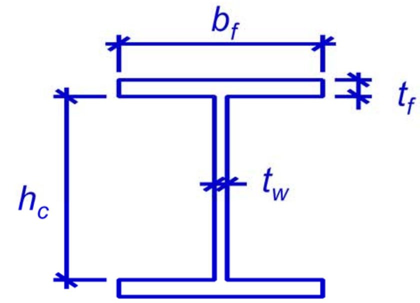
SMF Example – Check Member Local Stability

Check beam flange: $\dfrac{b_f}{2t_f} = 6.01$
(W33x141 A992)

Upper limit: $0.3\sqrt{\dfrac{E}{F_y}} = 7.22$ OK

Check beam web: $\dfrac{h_c}{t_w} = 49.6$

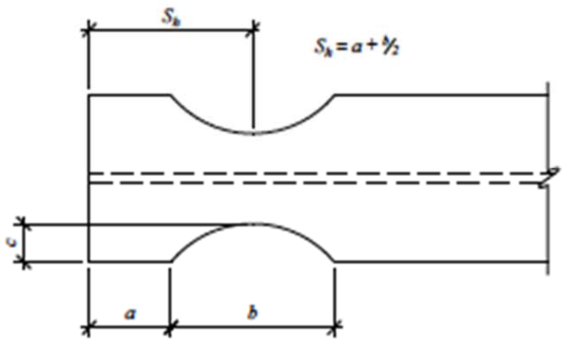
Upper limit: $2.45\sqrt{\dfrac{E}{F_y}} = 59.0$ OK



FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 69

The beam flange local stability limits are prescribed in the AISC Seismic specification, which is more stringent for the W shape in the SMF application. The web local stability limits are prescribed in the AISC *Specification for Structural Steel Buildings*. The column check is similar. Note that the provision for checking the web slenderness under combined bending and compression no longer appears in the AISC *Specification for Structural Steel Buildings*.

SMF Example – RBS Details



$a = 0.625 b_f$

$b = 0.75 d_b$

$c = 0.20 b_f$

The Reduced Beam Section details are shown in this slide.

SMF Example – Check Deflection and Drift

The frame was checked for an allowable story drift limit of $0.020h_{sx.}$ All stories in the building met the limit. Note that the *NEHRP Recommended Provisions* Sec. 4.3.2.3 requires the following check for vertical irregularity:

$$\frac{C_d \Delta_{x,story2}}{C_d \Delta_{x,story3}} = \frac{(1.2)}{(1.8)} = 0.67 < 1.3$$

Therefore, there is no vertical irregularity.

Calculation showing drift check.

SMF Example – Check Deflection and Drift

Table 6.2-1 Alternative A (Moment Frame) Story Drifts under Seismic Loads

	Elastic Displacement at Building Corner, From Analysis		Expected Displacement ($=\delta_e C_d$)		Design Story Drift Ratio		Allowable Story Drift Ratio
Level	δ_eE-W (in.)	δ_eN-S (in.)	δ E-W (in.)	δ N-S (in.)	Δ E-W/h (%)	Δ N-S/h (%)	Δ/h (%)
Level 7	2.92	3.18	16.0	17.5	1.2	1.2	2.0
Level 6	2.66	2.89	14.7	15.9	1.4	1.7	2.0
Level 5	2.33	2.47	12.8	13.6	1.6	2.0	2.0
Level 4	1.91	1.95	10.5	10.7	1.9	2.0	2.0
Level 3	1.41	1.40	7.76	7.70	1.8	1.8	2.0
Level 2	0.90	0.88	4.96	4.85	1.2	1.2	2.0
Level 1	0.55	0.52	3.04	2.89	1.1	1.1	2.0

1.0 in. = 25.4 mm.

Building Satisfies Drift Limits

Calculation showing drift check. Note that drifts are checked at the building corners, although this is not required for a torsionally regular building. P-Delta effects are directly included in the computations.

SMF Example – Check Torsional Amplification

The torsional amplification factor is given below. If A_x < 1.0 then torsional amplification is not required. From the expression it is apparent that if δ_{max} / δ_{avg} is less than 1.2, then torsional amplification will not be required.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2$$

The 3D analysis results, as shown in FEMA P-751, indicate that none of the δ_{max} / δ_{avg} ratios exceed 1.2; therefore, torsional amplification is not required.

The torsional amplification equation given in the slide is described FEMA 751. As noted, the structural analysis results indicate that torsional amplification need not be considered in this design. This is not surprising given the regularity (symmetry) of the floors and frames (placed around the perimeter.) The same regular and symmetrical building with braced frames in the core (and no moment frames) falls into the “extreme torsional irregularity” category.

SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_u/\phi P_n$ is typically less than 0.4 for the columns, combinations involving Ω_0 factors do not come into play for the special steel moment frames (re: AISC Seismic Sec. 8.3). In sizing columns (and beams) for strength one should satisfy the most severe value from interaction equations. However, the frame in this example is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

The content of this slide is taken directly from FEMA 751.

SMF Example – Column-Beam Moment Ratio

Per AISC Seismic Sec. 9.6

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

where $\Sigma \boldsymbol{M}^*_{pc}$ = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\boldsymbol{\Sigma M^*_{pc}}$ 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.

$\Sigma \boldsymbol{M}^*_{pb}$ = the sum of the moments in the beams at the intersection of the beam and column centerlines.

It is permitted to take $\Sigma M^*_{pc} = \Sigma Zc(F_{yc}\text{-}P_{uc/}A_g)$ (LRFD). ΣM^*_{pb} is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\Sigma M^*_{pb} = \Sigma(1.1R_yF_{yb}Z_b+M_{uv})$. Refer to AISC Seismic specification Sec. 9.6 for remainder of definitions.

SMF Example – Column-Beam Moment Ratio

Column – W24x146; beam – W21x73

$$\Sigma M^*_{pc} = \Sigma Z_c\left(F_{yc} - \frac{P_{uc}}{A_g}\right) + \frac{\left(M_{BF\,i} + M_{BF\,i+1}\right)}{h_c}\frac{d_b}{2}$$

$$= 39400\ \text{in.-kips}$$

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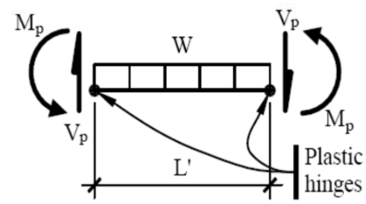
The story height for the first story is 268 in. (top of concrete to mid-depth of first floor beam and the clear height is 251.35 in. The values for the second story are 160 in. and 128.44 in. For diagrams and additional details the reader is referred to FEMA 751 and the AISC Seismic specification.

SMF Example – Column-Beam Moment Ratio

For beams:

$$\Sigma M^*_{pb} = \Sigma\left[M_{pr} + V_e\left(S_h + \frac{d_c}{2}\right)\right]$$

$$= 17749 \text{ in.-kips}$$



where

M_{pr}=$C_{pr}R_yF_y\ Z_e$

Z_e = effective Z (RBS)

V_e= 2 M_{pr} / L'

S_h= dist. from col. centerline to plastic hinge

Calculations for V_p. Figure from FEMA 751. The calculation for the factored uniform dead load, w, is given and is found to be 1.406 klf.

SMF Example – Column-Beam Moment Ratio

The ratio of column moment strengths to beam moment strengths is computed as:

$$\text{Ratio} = \frac{\Sigma M_{pc}^{*}}{\Sigma M_{pb}^{*}} = \frac{39400\,\text{in.-kips}}{17749\,\text{in.-kips}} = 2.22 > 1.0$$

Other ratios are also computed to be greater than 1.0

The column-beam strength ratio for all the other stories are determined in a similar manner.

SMF Example – Panel Zone Check

The 2005 AISC Seismic specification is used to check the panel zone strength. Note that FEMA 350 contains a different methodology, but only the most recent AISC provisions will be used. From analysis shown in the NEHRP *Design Examples* volume (FEMA 451), the factored strength that the panel zone at Story 2 of the frame in the EW direction must resist is 794 kips. The shear transmitted to the joint from the story above, V_c, opposes the direction of R_u and may be used to reduce the demand. Previously calculated, this is 102 kips at this location. Thus the frame R_u = 794 – 102 = 692 kips.

$$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)$$

$$= 547 \text{ kips}$$

Since ϕ_v = 1, $\phi_v R_b$ = 547 kips, which is less than the required resistance, 692 kips. Therefore, doubler plates are required. The required additional strength from the doubler plates is 692 – 547 = 145 kips. The plates must be at least ¼" thick as the strength of the double plates is:

$$\phi_v R_n = 0.6 t_{doub} d_c F_y$$

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The calculations shown use the 2005 AISC Seismic specification methodology.

SMF Example – Connection Configuration

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

(3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h \qquad (9\text{-}1)$$

The text on this slide is taken from the 2005 AISC Seismic specification. Connections that accommodate the required interstory drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified above are permitted. In addition to satisfying the requirements noted above, the design must demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design must include analysis for stability effects of the overall frame, including second-order effects.

SMF Example – Connection Configuration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 9.2a by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Appendix P.

(c) Provision of qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

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The following is taken from the Commentary to the 2005 AISC Seismic specification. This provision specifically permits the use of prequalified connections meeting the requirements of ANSI/AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2005a) to facilitate and standardize connection design. Other prequalification panels may be acceptable but are subject to the approval of the authority having jurisdiction. Use of connections qualified by prior tests or project specific tests may also be used although the engineer of record is responsible for substantiating the connection. Published testing, such as that conducted as part of the SAC project and reported in FEMA 350 and 355, or project-specific testing may be used to satisfy this provision.

Special Moment Frames
Summary

Beam-to-column connection capacity
Select preliminary member sizes
Check member local stability
Check deflection and drift
Check torsional amplification
Check the column-beam moment ratio rule
Check shear requirement at panel zone
Select connection configuration

- Prequalified connections
- Testing

Review SMF requirements. FEMA 350 is the current best resource for prequalified connections. AISC will eventually become the official repository for such information. FEMA 350 is available electronically for download at the AISC ePubs site (www.aisc.org).

NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Seismic design category requirement
- Moment resisting frames
- **Braced frames**

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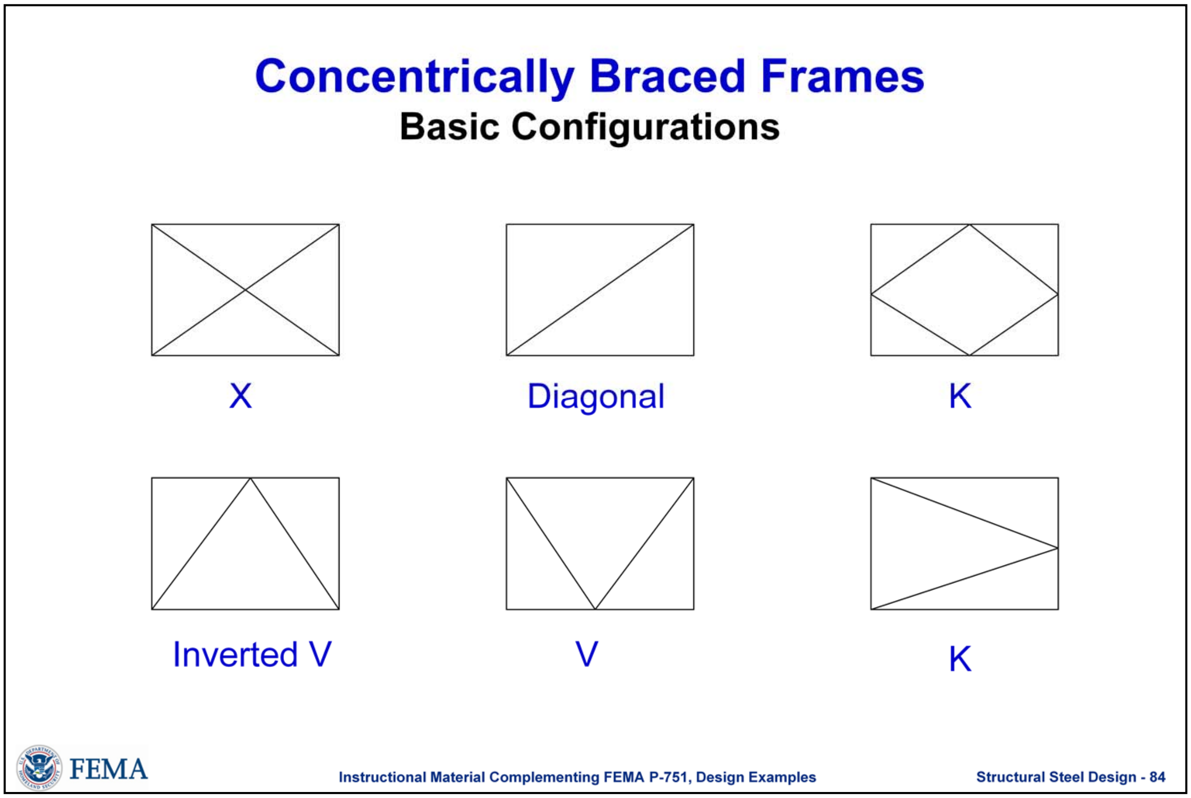
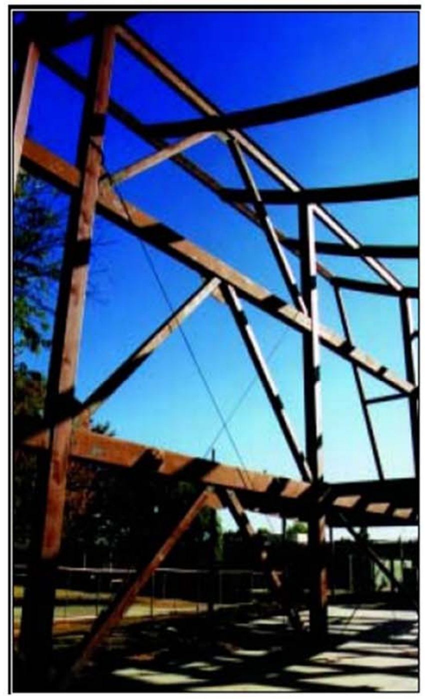
Table of Contents: braced frames.



K bracing will introduce bending into columns following buckling of compression brace and is often prohibited. V and inverted V bracing require continuity of beams.

Braced Frame Under Construction



A braced frame using inverted V type bracing is shown under construction.

Braced Frame Under Construction

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Braced frame using diagonal bracing under construction.

Concentrically Braced Frames

Special AISC Seismic	R = 6
Chapter 13	
Ordinary AISC Seismic	R = 3.25
Chapter 14	
Not Detailed for Seismic	R = 3
AISC LRFD	

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The special and ordinary systems are very similar. Chapter 13 and Chapter 14 refer to sections in the 2005 AISC Seismic specification.

Concentrically Braced Frames

Dissipate energy after onset of global buckling by avoiding brittle failures:

- Minimize local buckling
- Strong and tough end connections
- Better coupling of built-up members

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Most common failures were from: (1) fracture at local buckling where global buckling created a hinge and (2) fracture at connections.

Concentrically Braced Frames

Special and Ordinary

Bracing members:

- Compression capacity = $\phi_c P_n$
- Width / thickness limits

  Generally compact

  Angles, tubes and pipes very compact
- Overall $\frac{KL}{r} < 4\sqrt{\frac{E}{F_y}}$ < 200 for SCBF
- Balanced tension and compression

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Previous versions of these the AISC Seismic specification have required that the members of OCBF be designed for the amplified seismic load, effectively reducing the effective *R* factor by half. To make the design of OCBF consistent with other systems, this requirement has been eliminated from the 2005 AISC Seismic specification consistent with a corresponding reduction in the *R* factor for these systems in SEI/ASCE 7-05, Supplement Number 1. The required strength of the members of OCBF will now be determined using the loading combinations stipulated by the applicable building code (and the reduced *R* factors prescribed in SEI/ASCE 7-05, Supplement Number 1) without the application of the amplified seismic load.

"Balanced" tension and compression means that the sum of horizontal components of tension braces is between 30% and 70% of total horizontal force. This check is a design level, not post-buckling level. It is intended to prevent the "racheting" accumulation of inelastic deformation in one direction and the "impact" possible in a "slapback" phenomenon.

Concentrically Braced Frames

Special concentrically braced frames

Brace connections

Axial tensile strength > smallest of:

- Axial tension strength = $R_y F_y A_g$
- Maximum load effect that can be transmitted to brace by system

Axial compressive strength ≥ $1.1 R_y P_n$, where P_n is the nominal compressive strength of the brace.

Flexural strength > $1.1 R_y M_p$ or rotate to permit brace buckling while resisting $A_g F_{CR}$

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These requirements are all for special concentrically braced frames. Block shear and tensile rupture on net section must both be considered. Refer to the 2005 AISC Seismic specification for ordinary concentrically braced frame details.

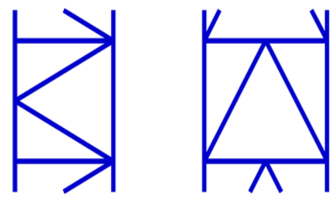
Concentrically Braced Frames

V bracing:

- Design beam for $D + L$ + unbalanced brace forces, using $0.3P_c$ for compression and $R_yF_yA_g$ in tension
- Laterally brace the beam
- Beams between columns must be continuous

K bracing:

- Not permitted



Limits on use and design of V and K braces. Refer to Sec. 13.4a in the 2005 AISC Seismic specification for additional description. The following text is taken from the Commentary to the 2005 AISC Seismic specification. Refer to the Commentary for additional information and description of the behavior.

“V-braced and inverted-V-braced frames exhibit a special problem that sets them apart from braced frames in which both ends of the braces frame into beam-column connections. The expected behavior of SCBF is that upon continued lateral displacement as the brace in compression buckles, its force drops while that in the brace in tension continues to increase up to the point of yielding. In order for this to occur, an unbalanced vertical force must be resisted by the intersecting beam as well as its connections and supporting members. In order to prevent undesirable deterioration of lateral strength of the frame, the SCBF provisions require that the beam possess adequate strength to resist this potentially significant post-buckling load redistribution (the unbalanced load) in combination with appropriate gravity loads.”

Concentrically Braced Frames

Built-up member stitches:

- Spacing < 40% KL/r
- No bolts in middle quarter of span
- Minimum strengths related to P_y

Column in CBF:

- Same local buckling rules as brace members
- Splices resist moments

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The following text is taken from the Commentary to the 2005 AISC Seismic specification. Closer spacing of stitches and higher stitch strength requirements are specified for built-up bracing members in SCBF than those required for OCBF. These are intended to restrict individual element bending between the stitch points and consequent premature fracture of bracing members. Wider spacing is permitted under an exception when buckling does not cause shear in the stitches. Bolted stitches are not permitted within the middle one-fourth of the clear brace length as the presence of bolt holes in that region may cause premature fractures due to the formation of a plastic hinge in the post-buckling range.

Special Concentrically Braced Frame Example

Seven Story Office Building, Los Angeles

Perimeter Moment Frames, all bays engaged

S_{DS}=1.0

S_{D1}=0.6

Occupancy Category II

Seismic Design Category D

Design Parameters (Table 12.2-1)

- R=6
- C_d=5.0
- Ω_0=2.0

The information on the following several slides are taken from Example 6.2 (Alternate A) of the NEHRP *Design Examples* volume (FEMA 751). FEMA 751 was written based on the 2009 *NEHRP Recommended Provisions*. The example complies with the 2005 AISC Seismic specification.

Concentrically Braced Frame Example

The following general design steps are required:

- Selection of preliminary member sizes
- Check strength
- Check drift
- Check torsional amplification
- Connection design

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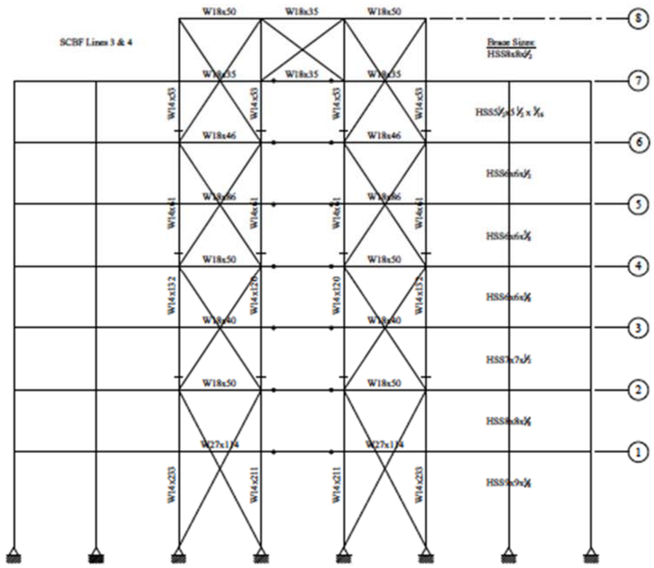
The general design steps for a SCBF are indicated on the slide. The load and analysis calculations, along with a summary of each of these steps are given in FEMA 751.

E-W Direction Framing

and Preliminary Member Sizes



Instructional Material Complementing FEMA P-751, Design Examples

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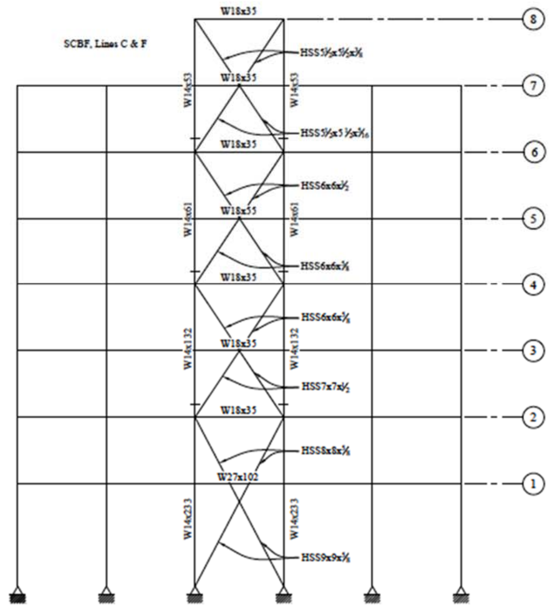
This example is the same building as was considered in the SMF example, which is Example 6.2 in FEMA 751.

N-S Direction Framing

and Preliminary Member Sizes



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Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 96

This example is the same building as was considered in the SMF example, which is Example 6.2 in FEMA 751.

CBF Example

Building Weight:

Penthouse Roof = 94 kips
Lower Roof = 1,537 kips
Typical Floor = 1,920 kips
Total = 94 + 1,537 + 6(1,920) = 13,151 kips

Building Period:

$T_a = C_t h_n^x = (0.02)(102.3)^{0.75} = 0.64$ sec.
$T = C_u T_a = (1.4)(0.64) = 0.896$ sec.

Design Base Shear:

$C_s = S_{D1}/(T/(R/I) = 0.6/(0.896/(6/1)) = 0.112$ <<< CONTROLS
$C_{s,min} = 0.044\, S_{DS} I = 0.044(1.0)(1) = 0.044$
$V = C_s W = 0.112(13,151) = 1,473$ kips.

FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 97

Building weights are given for the entire building.

CBFF Example – Check Deflection and Drift

Table 6.2-2 Alternative B Story Drifts under Seismic Load

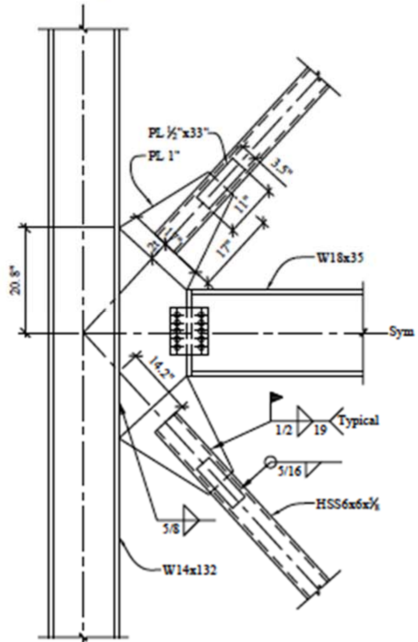
	Elastic Displacement at Building Corner, From Analysis		Expected Displacement (=$\delta_e C_d$)		Design Story Drift Ratio		Allowable Story Drift Ratio
Level	δ_e E-W (in.)	δ_e N-S (in.)	δ E-W (in.)	δ N-S (in.)	Δ E-W/h (%)	Δ N-S/h (%)	Δ/h (%)
Level 7	1.63	1.75	8.14	8.76	0.72	0.93	2.0
Level 6	1.41	1.48	7.07	7.38	0.74	0.94	2.0
Level 5	1.19	1.20	5.97	5.99	0.76	0.84	2.0
Level 4	0.96	0.94	4.80	4.72	0.81	0.85	2.0
Level 3	0.71	0.69	3.56	3.43	0.72	0.71	2.0
Level 2	0.49	0.47	2.44	2.33	0.60	0.59	2.0
Level 1	0.30	0.28	1.49	1.40	0.56	0.52	2.0

1.0 in. = 25.4 mm

Building Easily Satisfies Drift Limits

Calculation showing drift check. Note that drifts are checked at the building corners, although this is not required for a torsionally regular building. P-Delta effects are directly included in the computations.

CBF Example – Connection Detail



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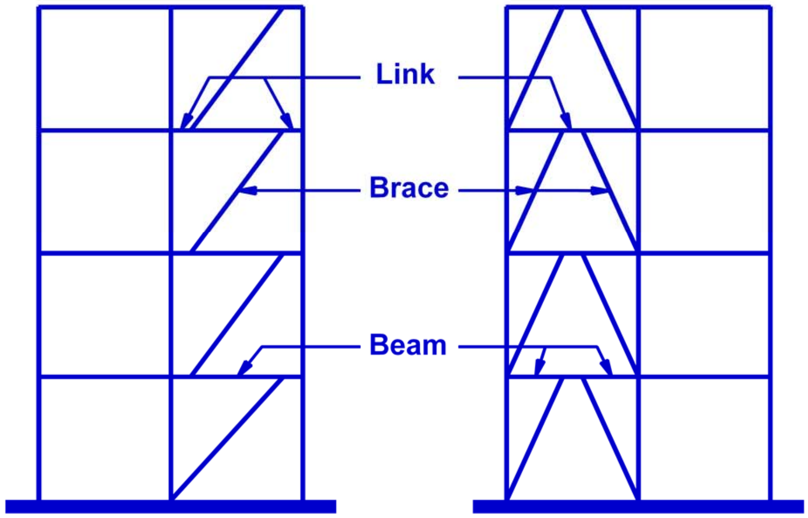
Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 99

Calculation showing drift check. Note that drifts are checked at the building corners, although this is not required for a torsionally regular building. P-Delta effects are directly included in the computations.

Eccentrically Braced Frames



Eccentrically braced frames. Define terms.

BRBFs are concentrically braced frames, consisting of beams, columns and braces arranged to form a truss. BRBFs resist lateral load primarily through truss action, with member response dominated by axial force (tension and compression).

Unlike conventional concentrically braced frames, BRBFs are constructed with a special type of brace member: Buckling-Restrained Braces (BRBs).

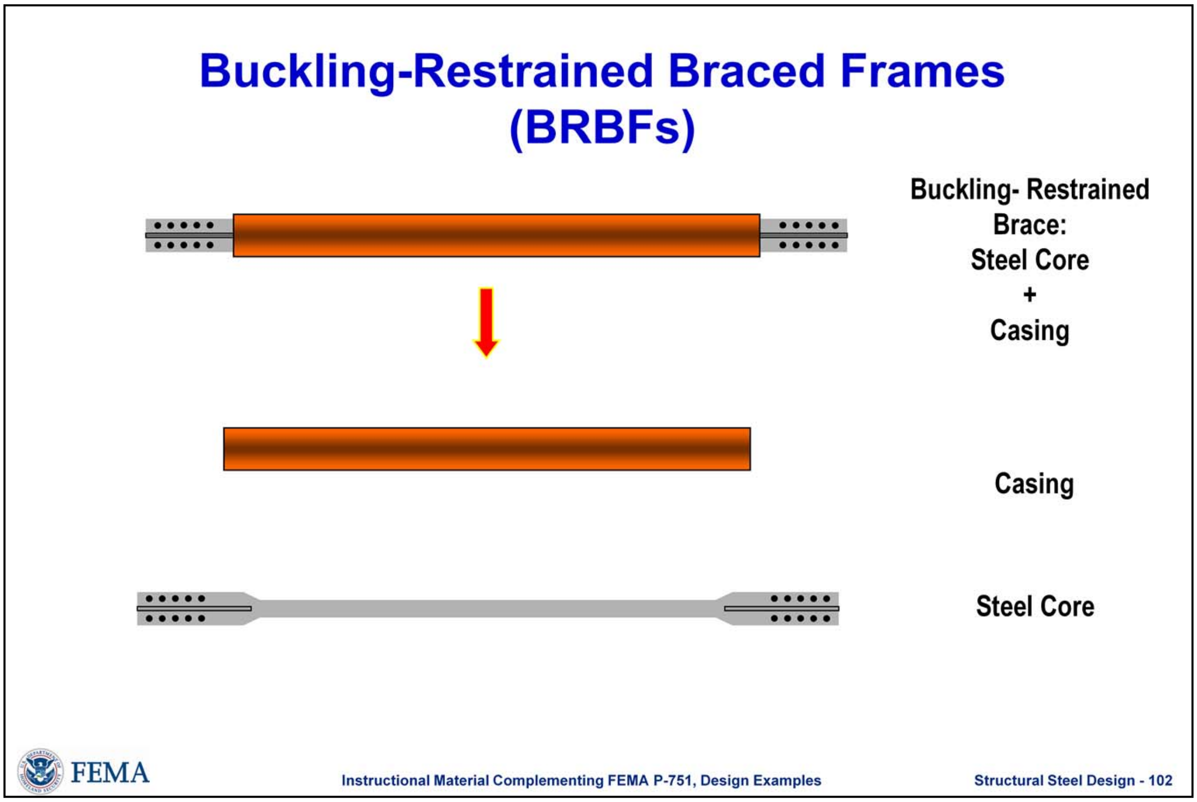
BRBs have the unique quality that they do not buckle when loaded in compression. Consequently, BRBs yield in a ductile manner in both tension and compression. The use of BRBs results in a frame that combines high elastic stiffness with excellent ductility.

BRBFs are considered to be a high ductility framing system, and is assigned high R-factors in ASCE-7.

Per ASCE-7, the R-factors for BRBFs are:
R=7 for BRBF with non-moment resisting beam-to-column connections.

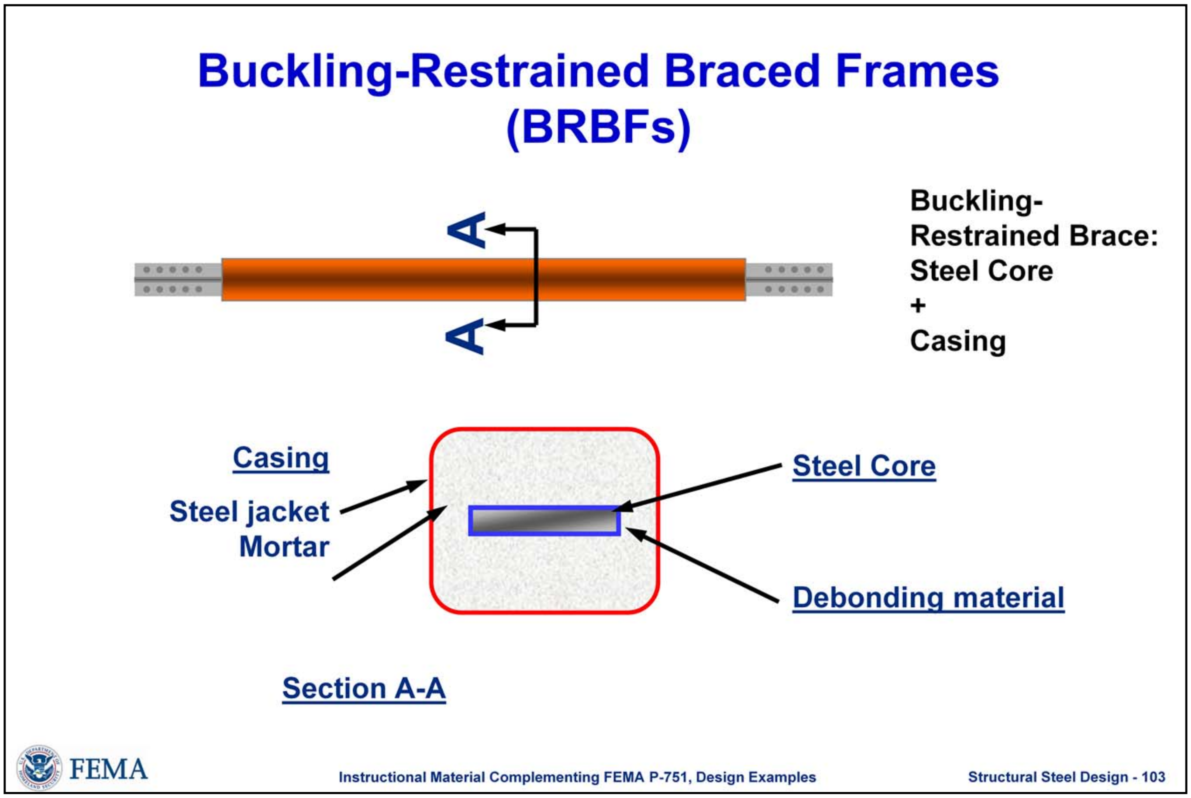
R=8 for BRBF with moment resisting beam-to-column connections.

These are the same as the R-factors assigned to EBFs.



The key feature of a BRBF is the use of Buckling-Restrained Braces (BRBs).

A BRB consists of a steel core and a casing. The casing covers the steel core.



The are a number of different configurations for BRBs. This slide shows one possibility. The steel core is simply a rectangular plate. The casing, which restrains buckling of the core, is typically constructed of a mortar filled steel tube. The steel core is surrounded by a debonding material that decouples the steel core from the casing, for purposes of resisting axial force. That is, the debonding material is intended to prevent transfer of axial stress from the steel core to the casing. Consequently, the casing essentially "floats" on the steel core.

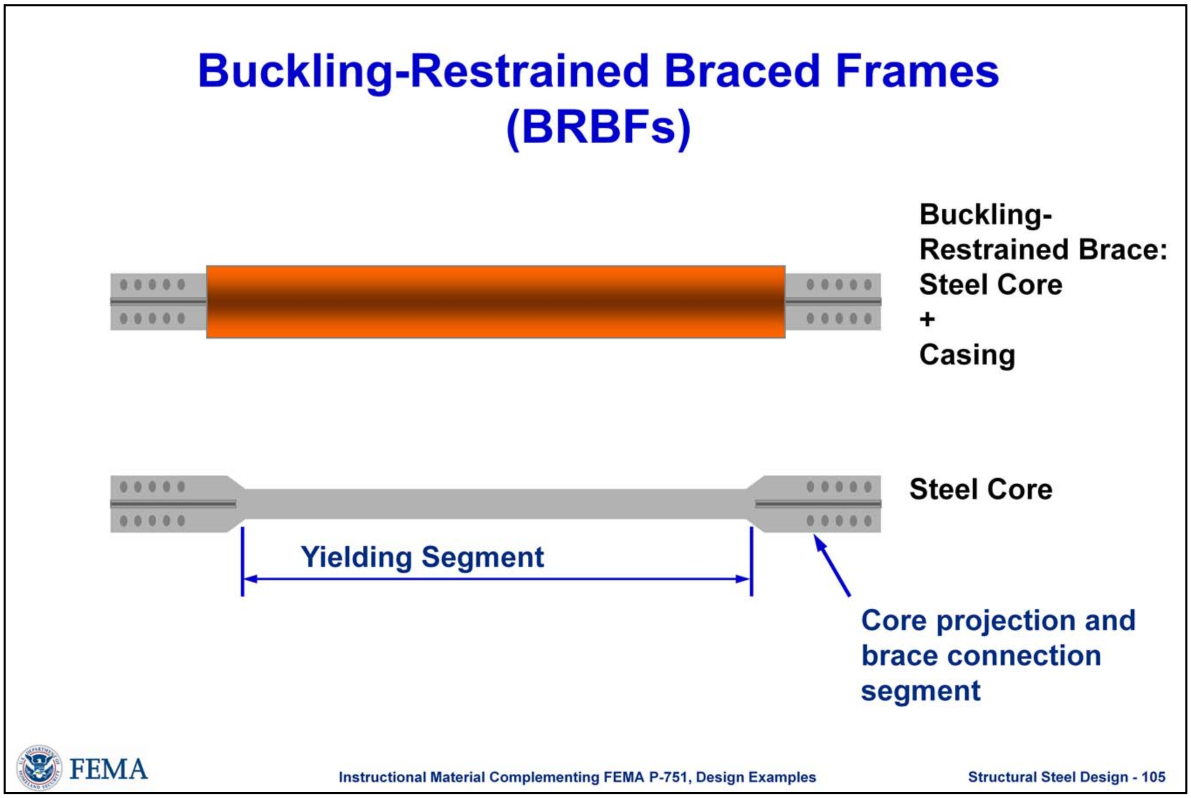
There are a number of possible configurations for the shape of the steel core. In addition to a single plate, BRBs have been constructed using cruciform shaped plates, or a pair of plates or shapes.

Buckling-Restrained Braced Frames (BRBFs)

Steel core resists entire axial force P

Casing is debonded from steel core
- casing does not resist axial force P
- flexural stiffness of casing restrains buckling of core

The concept of the BRB is that the steel core resists the applied axial force P. The debonding material that surrounds the core is intended to prevent the casing from carrying axial force. Since the casing has no (or very little) axial force, its full flexural stiffness is available to resist buckling of the steel core.



The central section of the steel core, as shown in the slide, is intended to be the portion of the core that yields in tension and in compression. At the ends of the BRB, the core projects outside of the casing. This core projection typically has a significantly larger cross-sectional area than the yielding segment, and is usually reinforced to prevent buckling of the projection. Further, the core projection is normally prepared with bolt holes or other features that allow connection of the BRB to the remainder of the frame.

A building designer using a BRBF does not normally develop the detailed design of a buckling restrained brace, such the shape of the core and core projections, the details of the casing and debonding material, and the connection details provided on the core projection.

Rather, there are companies that manufacture and sell BRBs. These are proprietary and patented designs which have undergone design development and testing by the manufacturers. Thus, the building designer will normally specify the required strength and stiffness of the BRBs, and the manufacturer will supply BRBs that meet these specifications.

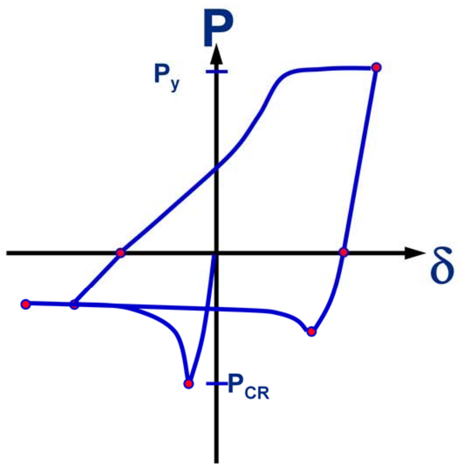
At present (early 2007) there are three manufacturers that market BRBs in the US:

- CoreBrace (http://www.starseismic.net/)

- Star Seismic (http://www.corebrace.com/)

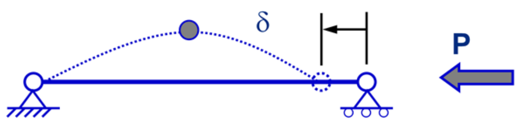
- Nippon Steel Corporation (http://www.unbondedbrace.com/)

Brace Behavior Under Cyclic Axial Loading



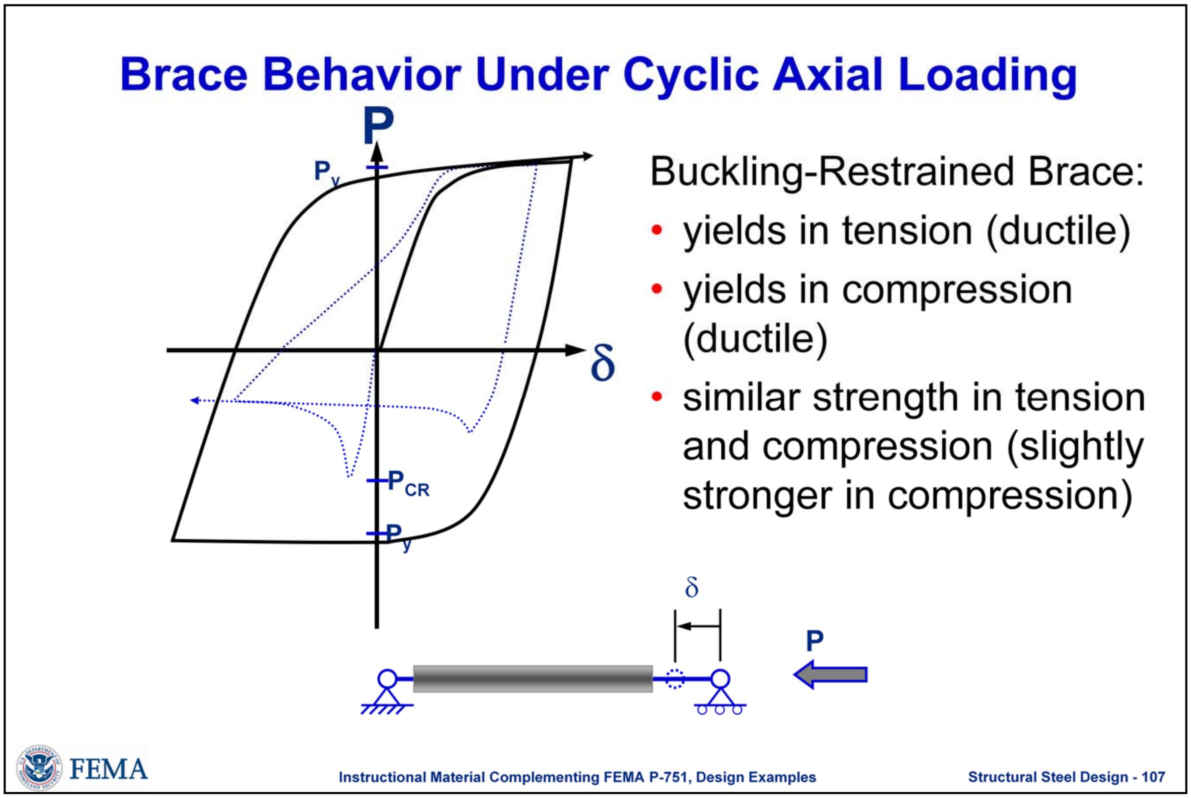
Conventional Brace:

- yields in tension (ductile)
- buckles in compression (nondductile)
- significantly different strength in tension and compression



This slide reviews the behavior of a conventional brace when subjected to cyclic axial load.

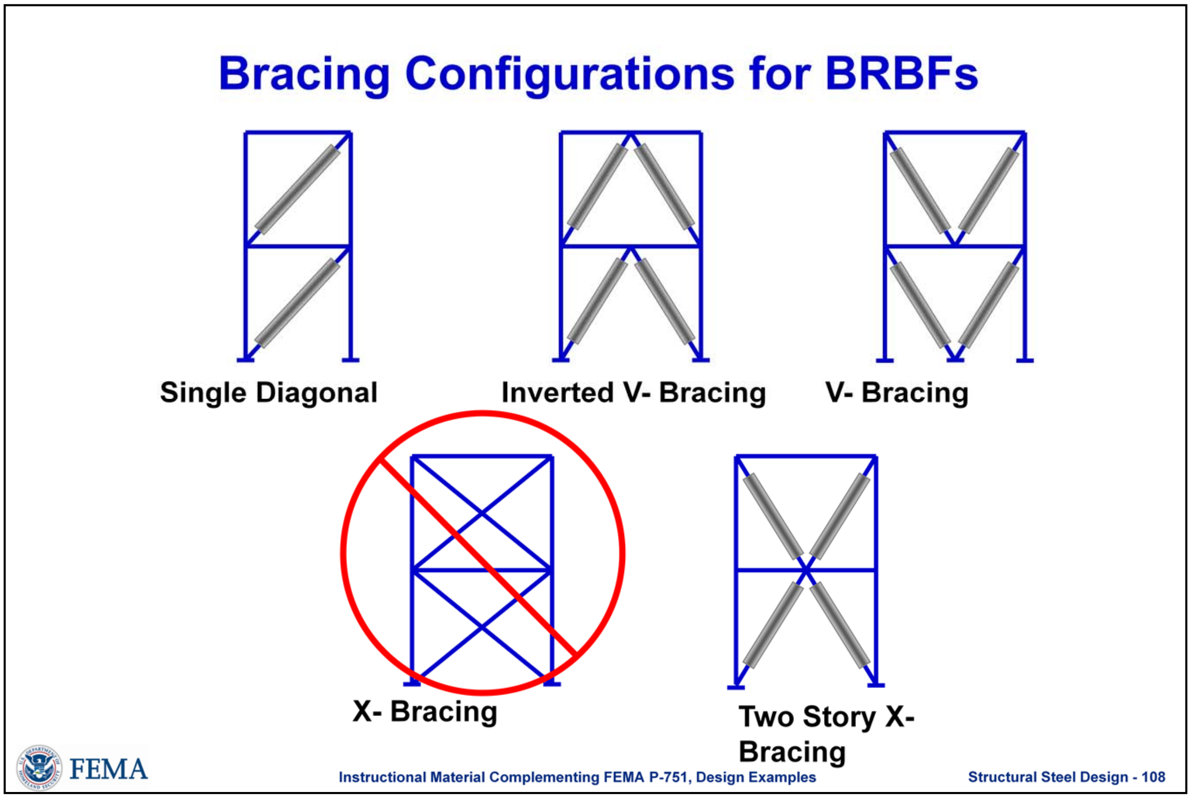
A conventional brace buckles when loaded in compression, resulting in deterioration of strength and stiffness. A conventional brace exhibits significantly different strengths in compression and tension.



A buckling-restrained brace yields in a ductile manner, both in tension and in compression. As a result, a BRB exhibits highly desirable behavior as an energy-dissipating element in a seismic-resistant frame.

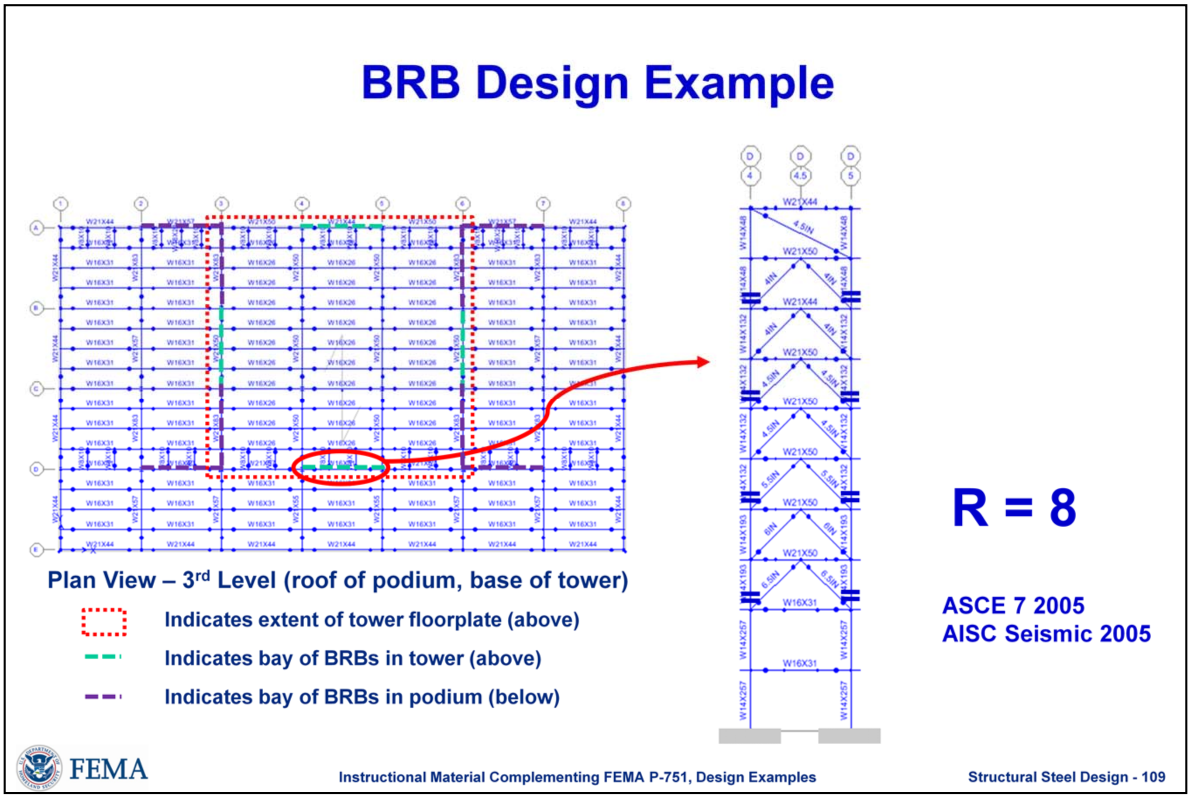
A BRB also exhibits very similar strengths in compression and in tension. This avoids many of the difficulties that result from large unbalanced brace forces in SCBFs, such as the difficulty in designing beams in inverted-V braced SCBFs.

Tests show that BRBs exhibit slightly higher strength in compression than in tension. The maximum compression strength of a BRB may be on the order of 3 to 10-percent greater than the maximum tension strength of the BRB. This higher measured compression strength may be due to partial engagement of the casing in resisting axial force, when the BRB is loading in compression. This partial engagement of the casing may be due to imperfect debonding, resulting from expansion of the core in compression (Poisson's effect) and higher mode buckles (ripples) in the core that casue the core to press against the casing.



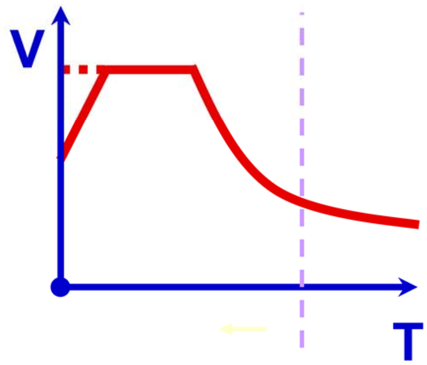
Various bracing arrangements are possible for BRBFs, as is the case with conventional braced frames.

Single story X-bracing is generally not possible for BRBFs because of the difficulty in crossing the two BRBs at their centers. Also, K-braced BRBFs are not permitted (see Section 16.4).



Slides shows location of BRB Frame in plan of building.

Base Shear



Hazard

S_{ds} = 0.859

S_{d1} = 0.433

T_a = 1.32 sec.

V = 0.057 W

Calculating the base shear.

Load Combinations

Basic

$1.2D + f_1L + 0.2S + E$

$0.9D \pm E$

$f_1 = 0.5$

$E = \rho\, Q_E + 0.2S_{DS}\, D$

$1.37D + 0.5L + 0.2S + \rho Q_E$

$0.73D \pm \rho Q_E$

Special (Amplified Seismic Load)

$1.2D + f_1L + 0.2S + E_m$

$0.9D \pm E_m$

$E_m = \Omega_o\, Q_E + 0.2S_{DS}\, D$

$1.37D + 0.5L + 0.2S + \Omega_o Q_E$

$0.73D \pm \Omega_o Q_E$

Determining load combinations.

Vertical Distribution of Forces

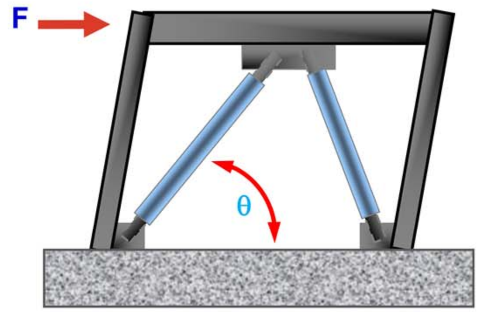
Diaphragm Level	Story Force (kips)	Brace Level	Story Shear (kips)	% of Total Base Shear
Roof	210	10	210	24%
10	152	9	362	42%
9	126	8	488	56%
8	102	7	590	68%
7	80	6	670	77%
6	60	5	730	84%
5	43	4	772	89%
4	28	3	800	92%
3	51	2	851	98%
2	14	1	865	100%

Seismic forces along height.

Preliminary Design of Braces

$$P_u = \frac{F}{2\cos\theta}$$

Assume braces resist 100% of story shear



$$A_{sc} = \frac{P_u}{\phi F_y}$$

Design braces precisely to calculated capacity ($P_u = \phi P_n = \phi F_y A_{sc}$)

Preliminary design of braces.

BRB Design – Gridline D

Brace Level	Brace Angle α (deg)	Brace Force P_u (kips)	Required Core Area A_{sc}	Provided Core Area A_{sc}
10	26.6	142	4.21	4.5
9	45.0	126	3.71	4.0
8	45.0	133	3.98	4.0
7	45.0	141	4.12	4.5
6	45.0	151	4.42	4.5
5	45.0	170	5.12	5.5
4	45.0	197	5.70	6.0
3	45.0	210	6.26	6.5
2	50.1	187	5.76	6.0
1	50.1	195	6.81	7.0

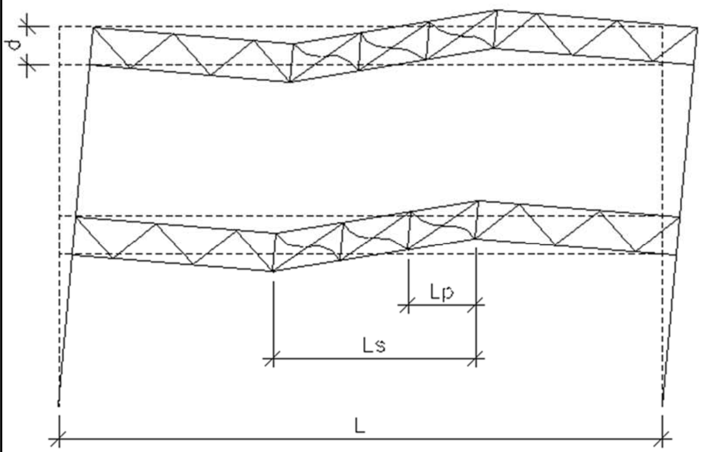
Design forces in braces.

NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- **Other topics**

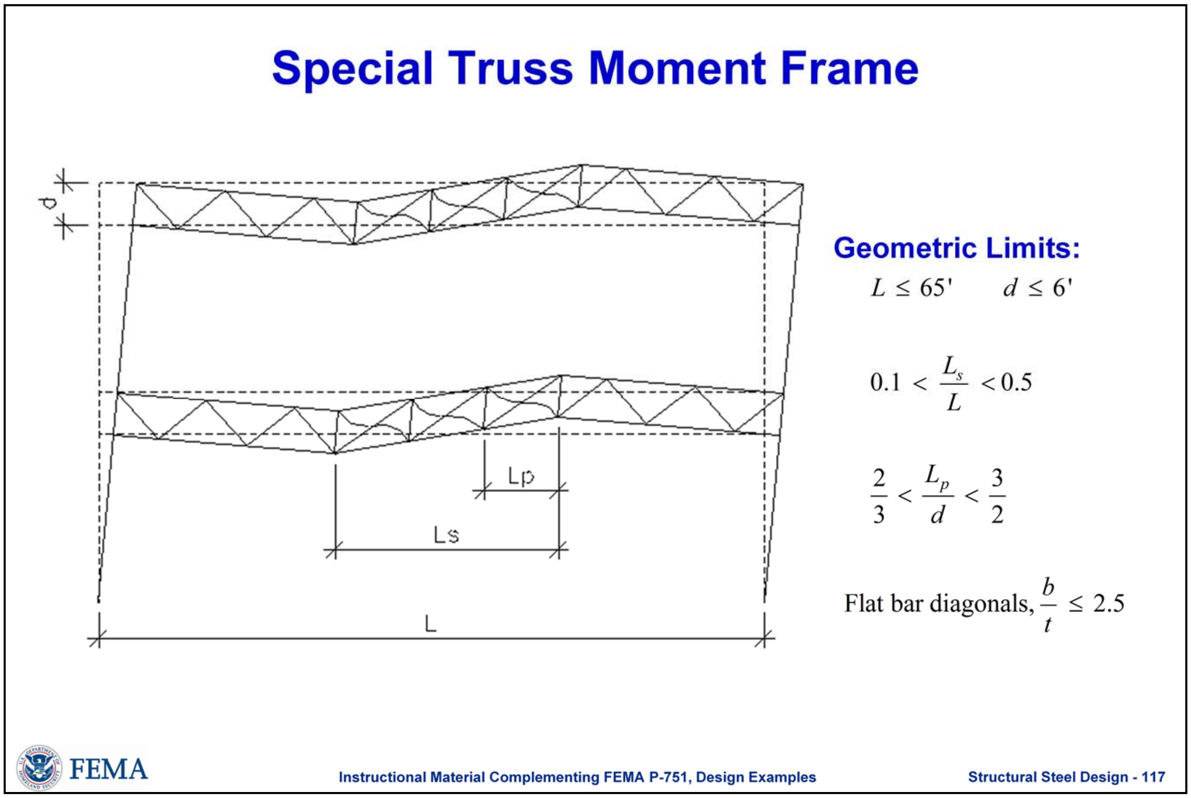
Table of contents: other topics.

Special Truss Moment Frame



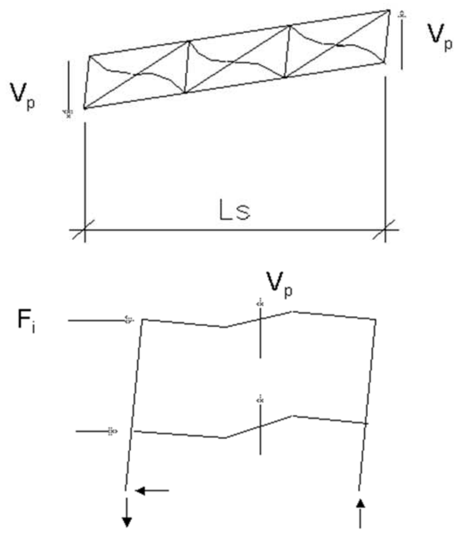
- Buckling and yielding in <u>special</u> section
- Design to be elastic outside special section
- Deforms similar to EBF
- Special panels to be symmetric X or Vierendeel

Special truss moment frames are a new introduction to seismic resistant construction. The concept is that a set of panels near midspan of horizontal trusses, where the gravity load shear is low, are design to yield and buckle in a controlled manner to dissipate energy of strong ground shaking. It should be anticipated that the bucked members may have to be replaced following strong ground shaking.



The system has been tested, but only a few real buildings have yet been constructed. The geometric limits are mostly based upon the tested specimens. The use of flat bar diagonals in the special panels prevents local buckling problems.

Special Truss Moment Frame



$$V_p = 2\left(\frac{2\,M_{pc}}{L_s}\right) + \sin\alpha\,\left(P_{nt} + 0.3P_{cd}\right)$$

$$\sum F_i\,h_i = \sum V_p\,L$$

The special section is shown. The plastic shear capacity depends on flexural hinging of the chords at the ends of the section and the buckling and tensile yield capacities of the webs. The plastic shear demand is directly related to the horizontal forces on the structure. The angle between the diagonal and the chord is given as α.

Special Truss Moment Frame

Special truss moment frame experimental test specimen. This specimen was part of a research program at the University of Michigan under the direction of Professor Subash Goel. This research led to the inclusion of the STMF as a recognized structural system in the *NEHRP Recommended Provisions*. Photo courtesy of Nucor Research and Development

Special Truss Moment Frame

Special truss moment frame under construction.

General Seismic Detailing

Materials:

- Limit to lower strengths and higher ductilities

Bolted Joints:

- Fully tensioned high strength bolts
- Limit on bearing

FEMA

Instructional Material Complementing FEMA P-751, Design Examples

Structural Steel Design - 121

Seismic detailing per AISC Seismic is more than simply the rules for the specific systems. There are also general rules for several subjects that apply to all types of steel seismic resisting systems.

General Seismic Detailing

Welded Joints:

- AWS requirements for welding procedure specs
- Filler metal toughness
  - CVN > 20 ft-lb @ -20°F, or AISC Seismic App. X
- Warning on discontinuities, tack welds, run offs, gouges, etc.

Columns:

- Strength using Ω_o if $P_u / \phi P_n > 0.4$
- Splices: Requirements on partial pen welds and fillet welds

FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 122

The requirements for weld metal toughness are new. The reference to AWS welding procedure specs is a new emphasis. It is likely that structural engineers will have to become more well educated about welding in the future. The rules on columns are intended to prevent premature failure in these vulnerable components.

Steel Diaphragm
Example

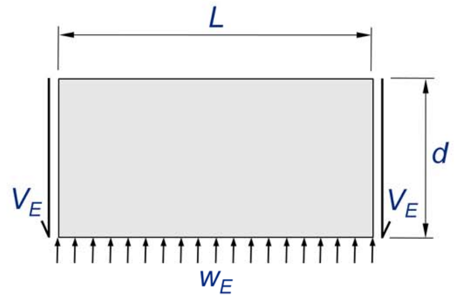
$\phi V_n = \phi$ (approved strength)

$\phi = 0.6$

For <u>example only:</u>
Use approved strength as 2.0 × working load in SDI *Diaphragm Design Manual*

This example is not in FEMA 751; for example only, use capacities from SDI *Diaphragm Design Manual*.

Steel Deck Diaphragm
Example



$$L = 80' \qquad d = 40'$$

$$w_D = w_L = 0 \qquad w_E = 500 \text{ plf}$$

$$V_E = \frac{w_E L}{2} = 20 \text{ kip}; \quad v_E = \frac{20000}{40} = 500 \text{ plf}$$

$$v_{SDI} = \frac{v_E}{2\phi} = \frac{500}{2(0.6)} = 417 \text{ plf}$$

Deck chosen:
1½", 22 gage with welds on 36/5 pattern and 3 sidelap fasteners, spanning 5'-0"

Capacity = 450 > 417 plf

Shear in diaphragm due to dead and live loads is zero. Given the seismic load, w_E=500 plf, calculate total shear at end of diaphragm; compute shear flow at end of diaphragm. Required shear capacity is shear demand divided by 2.0ϕ. Choose deck and fasteners. Fasteners and their spacing greatly affect diaphragm capacity; must carefully inspect deck welds and sidelap fasteners. Inspector needs to make sure that deck has not been burned through, and that connection has been made.

Welded Shear Studs

FEMA Instructional Material Complementing FEMA P-751, Design Examples Structural Steel Design - 125

Welded shear studs have been a part of the AISC Specifications since 1961. Beginning in the late 1980s, the strength model for shear studs used in cold-formed steel deck begin to come under scrutiny. In particular, the reduction factor, which was a function of deck geometry, in the AISC specifications was thought to be unconservative and in need of change. Research teams from several places around the world were working on the problem. This was in part due to the fact that the AISC procedures had been widely adopted. A series of beam tests and several hundred push-out tests were conducted at Virginia Tech over a number of years. This comprehensive research program led to changes in the 2005 AISC specification. These changes will be highlighted in the following slides.

Shear Stud Strength - AISC 2005 Specification

$$Q_n = 0.5\,A_{sc}\,(f_c'\,E_c)^{1/2} \le R_g\,R_p\,A_{sc}\,F_u$$

R_g = stud geometry adjustment factor

R_p = stud position adjustment factor

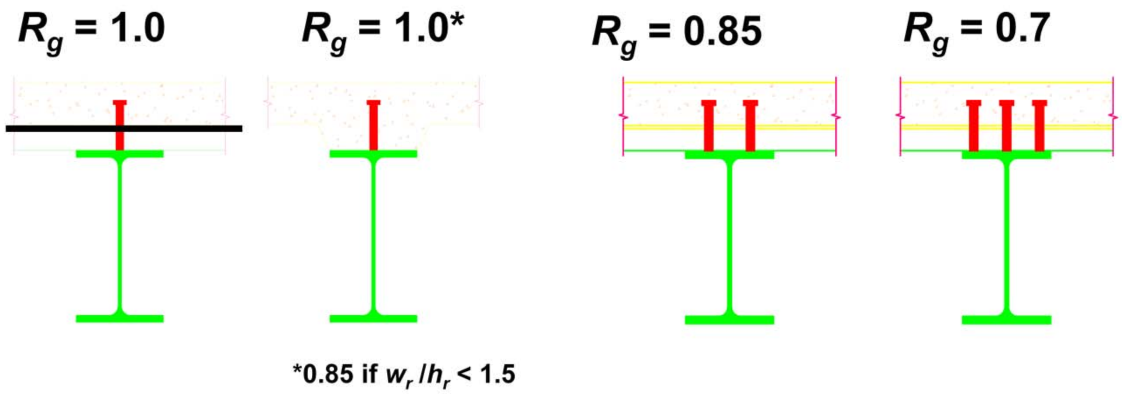
Note that the strength reduction factor for bending has been increased from 0.85 to 0.9. This results from the strength model for shear studs being more accurate, although the result for Q_n is lower in the 2005 specification.

The shear stud strength model is no longer a direct function of the deck geometry, although the limitations of the empirical based formulation is a function of the deck depths that were used in the study. The reader is referred to the 2005 AISC specification and commentary for a more complete discussion. The flexural design procedure is unchanged, other than the calculation of the shear stud strength. The strength reduction factor for bending has been increased from 0.85 to 0.9. This reflects the use of a more accurate shear stud strength model. As in past specifications, there is no strength reduction factor applied separately to the shear studs.

Shear Studs – Group Adjustment Factor

$$Q_n = 0.5\, A_{sc}\, (f_c'\, E_c)^{1/2} \leq R_g\, R_p\, A_{sc}\, F_u$$

R_g = stud group adjustment factor



Instructional Material Complementing FEMA P-751, Design Examples

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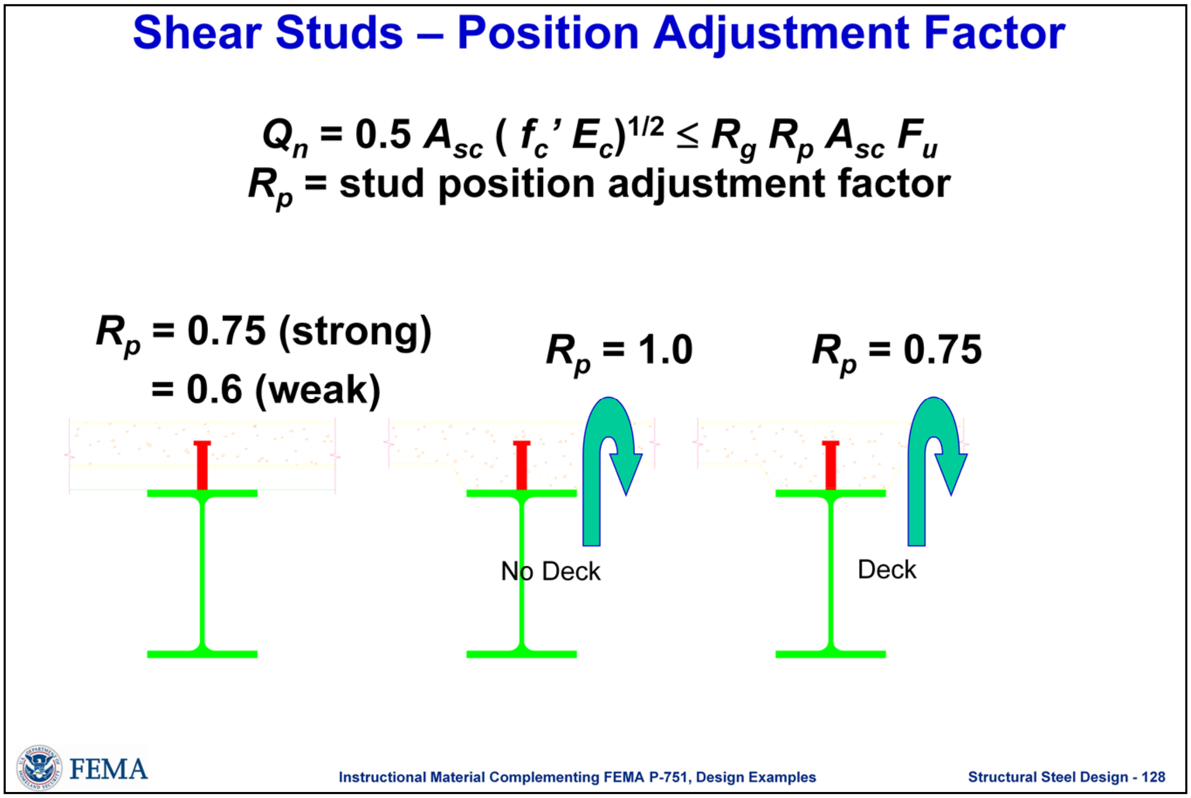
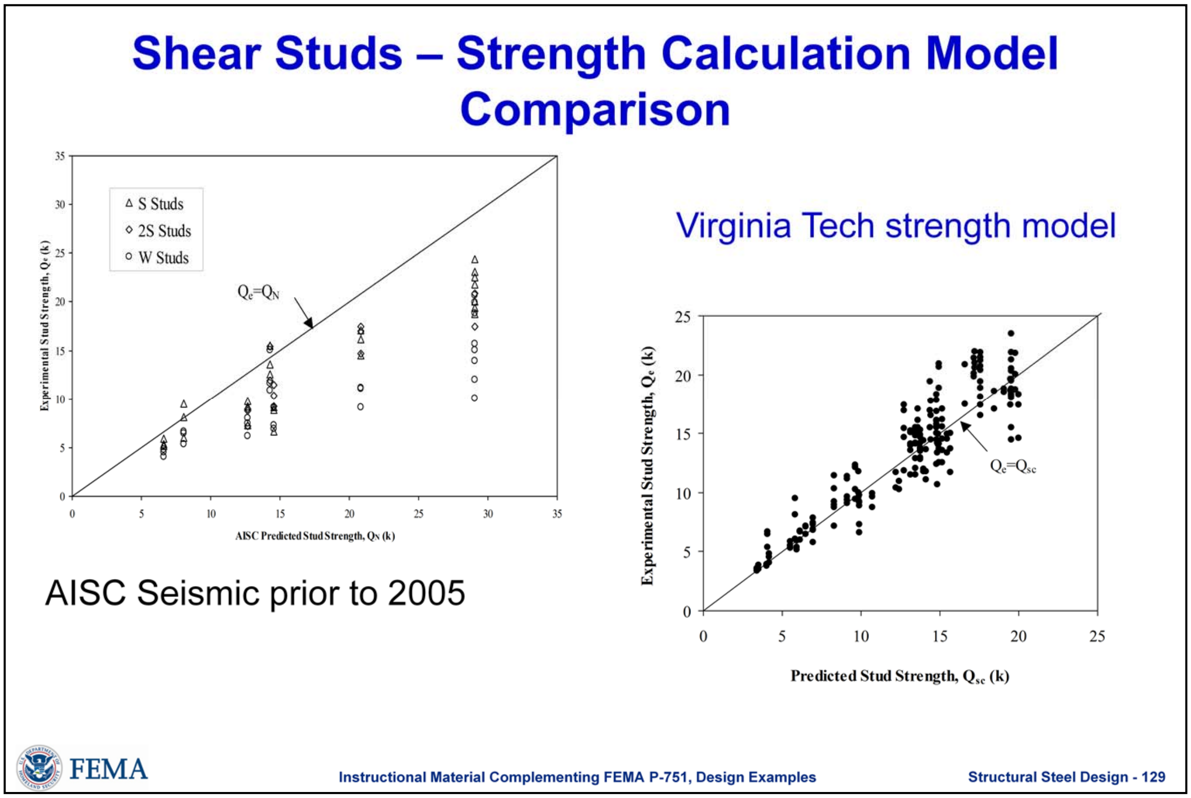
This slide illustrates the application of the group adjustment factor, R_g. The first, third, and fourth figures illustrate "beam" applications – that is the deck is perpendicular to the steel section. The second figure illustrates a "girder" for which the deck is parallel to the steel section. A narrow rib in the girder application, as defined by the w_r/h_r ratio being less than 1.5, results in a reduction in R_g to a value of 0.85.



This slide illustrates the applications of the position adjustment factor. Refer to AISC for a further description of the "no deck" and "deck" cases for the "girder" applications in the second and third figure. The recommended value of R_p for the beam case, illustrated in the first figure, is the weak position case of 0.6. Use of the higher value of 0.75 requires close control and inspection during the stud installation process during construction. This control is difficult to obtain.



The two graphs illustrate shear stud strength calculation model comparisons with experimental push-off test data. The graph on the left is a comparison between experimental shear stud strengths and the AISC calculation model prior to 2005. Note that virtually all the data indicates that the model is unconservative by as much as 50% in a number of cases. The reader should note that the relationship between shear stud strength and flexural strength is nonlinear. A reduction of shear stud strength by 50% may equate to a reduction in flexural strength of approximately 20%. The graph on the right is a comparison between a large data base of push out tests with what is referred to as the Virginia Tech calculation model. The Virginia Tech model formed the basis for the 2005 AISC specification provisions. Note that the comparison with the new model is much better.

Shear Studs – Diaphragm Applications

Shear studs are often used along diaphragm collector members to transfer the shear from the slab into the frame. The shear stud calculation model in the 2005 AISC specification can be used to compute the nominal shear strengths. A strength reduction factor should be used when comparing these values to the factored shear. There is no code- established value for the strength reduction factor. A value of 0.8 is recommended pending further development.

Notes regarding shear studs.

Inspection and Testing

Inspection Requirements

- Welding:
  - Single pass fillet or resistance welds
    PERIODIC
  - All other welds
    CONTINUOUS
- High strength bolts:
  PERIODIC

Inspection requirements: Periodic inspection of single pass fillet welds and continuous inspection of all other welds. AWS D1.1 is a good reference for what items should be noted during inspection of welding. Additional inspection methods include ultrasonic inspection, x-ray inspection, and other non-destructive methods. High strength bolts must be inspected periodically.

Inspection and Testing
Shop Certification

- Domestic:
  - AISC
  - Local jurisdictions
- Foreign:
  - No established international criteria

AISC can certify fabricating shops. Program includes scheduled and surprise inspections to review quality control procedures, personnel, and results. Some local jurisdictions have adopted similar certification procedures. As there is no internationally accepted quality certification, foreign fabricated steel should be given close inspection.

Inspection and Testing
Base Metal Testing

- More than 1-1/2 inches thick
- Subjected to through-thickness weld shrinkage
- Lamellar tearing
- Ultrasonic testing

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Base metal testing required: where base metal more than 1-1/2 in. thick subjected to through-thickness weld shrinkage (lamellar tearing is possible); ultrasonic testing required to identify presence of discontinuities in base metal.

NEHRP Recommended Provisions Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics

Summary

Questions

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Slide to initiate questions from the participants.

6

Structural Steel Design

Rafael Sabelli, S.E., and Brian Dean, P.E.

Originally developed by
James R. Harris, P.E., PhD, Frederick R. Rutz, P.E., PhD, and Teymour Manouri, P.E., PhD

SEISMIC DESIGN OF STEEL STRUCTURES

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics
- Summary

Steel Design: Context in Provisions

Design basis: Strength limit state

Using the 2009 *NEHRP Recommended Provisions*, Refer to ASCE 7 2005:

Seismic Design Criteria	Chap. 11
Seismic Design Requirements	
Buildings	Chap. 12
Nonstructural components	Chap. 13
Design of steel structures	Chap. 14 AISC Seismic and others

Seismic Resisting Systems

Unbraced Frames

Connections are:

- Fully Restrained Moment-resisting
- Partially Restrained Moment-resisting

Seismic classes are:

- Special Moment Frames
- Intermediate Moment Frames
- Ordinary Moment Frames
- Systems not specifically detailed for seismic response

Braced Frames

Ordinary Concentric Braced Frames

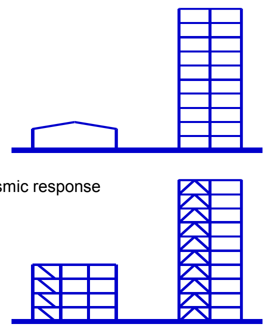
Special Concentric Braced Frames

Eccentrically Braced Frames

Buckling Restrained Braced Frames

Special Plate Shear Walls

Systems not specifically detailed for seismic response



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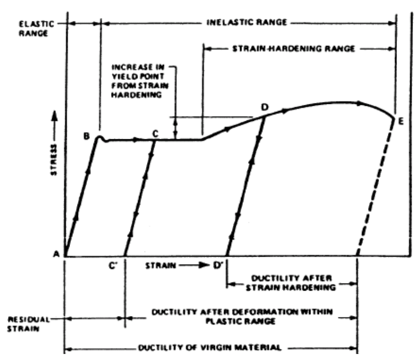
Monotonic Stress-Strain Behavior



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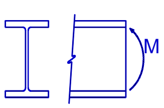
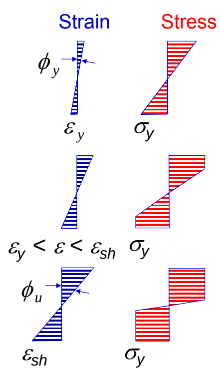
Bending of Steel Beam



Extreme fiber reaches yield strain and stress

Strain slightly above yield strain

Section near "plastic"



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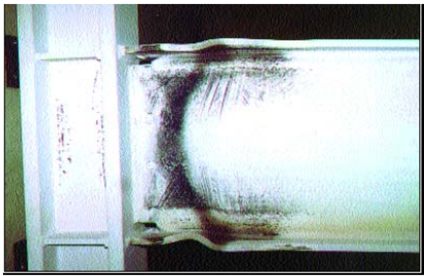
Plastic Hinge Formation



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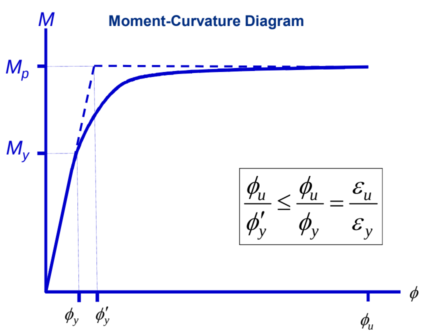
Cross - section Ductility



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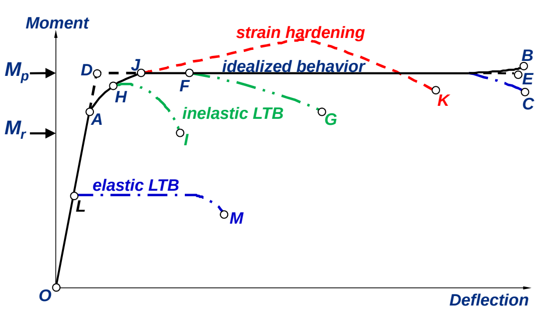
Behavior Modes For Beams



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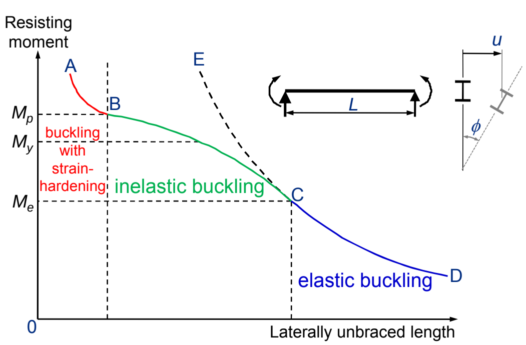
Flexural Ductility of Steel Members
Practical Limits

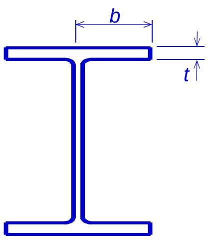
1 Lateral torsional buckling
   Brace well
2 Local buckling
   Limit width-to-thickness ratios
   for compression elements
3 Fracture
   Avoid by proper detailing

Local and Lateral Buckling

Lateral Torsional Buckling



Local Buckling



Classical plate buckling solution:

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(b/t)^2} \le \sigma_y$$

Substituting μ = 0.3 and rearranging:

$$\frac{b}{t} \le 0.95\sqrt{\frac{kE}{F_y}}$$

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Local Buckling
continued

With the plate buckling coefficient, *K*, taken as 0.7 and an adjustment for residual stresses, the expression for b/t becomes:

$$\frac{b}{t} \le 0.38\sqrt{\frac{E}{F_y}}$$

This is the slenderness requirement given in the AISC specification for compact flanges of I-shaped sections in bending. The coefficient is further reduced for sections to be used in seismic applications in the AISC Seismic specification

$$\frac{b}{t} \le 0.3\sqrt{\frac{E}{F_y}}$$

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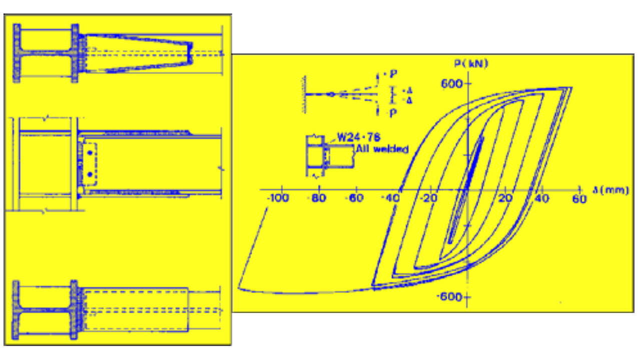
Welded Beam to Column Laboratory Test - 1960s



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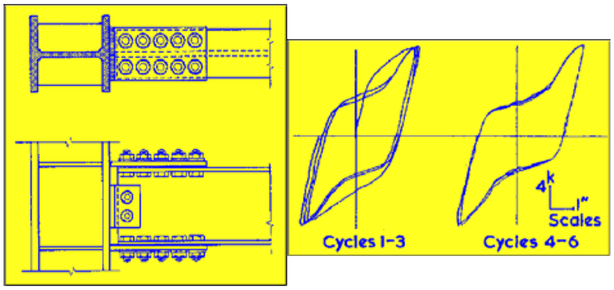
Bolted Beam to Column Laboratory Test - 1960s



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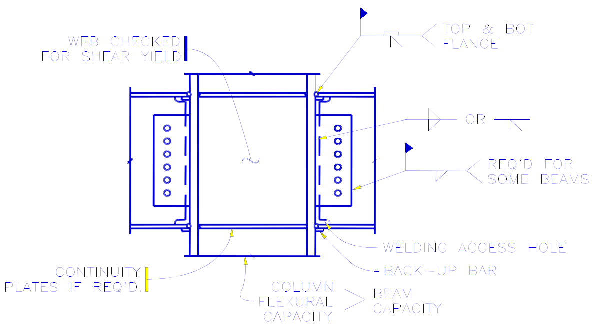
Pre-Northridge Standard



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Following the 1994 Northridge earthquake, numerous failures of steel beam-to-column moment connections were identified. This led to a multiyear, multimillion dollar FEMA-funded problem-focused study undertaken by the SAC Joint Venture. The failures caused a fundamental rethinking of the design of seismic resistant steel moment connections.

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Bottom Flange Weld Fracture Propagating Through Column Flange and Web

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Beam Bottom Flange Weld Fracture Causing a Column Divot Fracture

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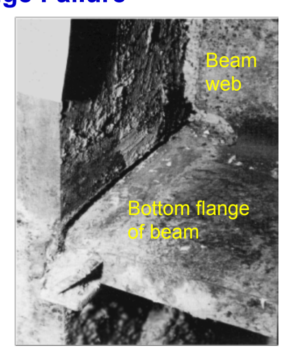
Northridge Failure

- Crack through weld
- Note backup bar and runoff tab



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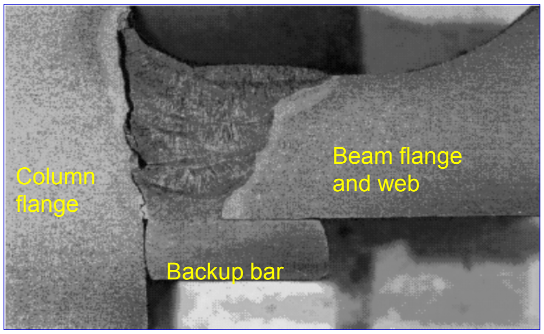
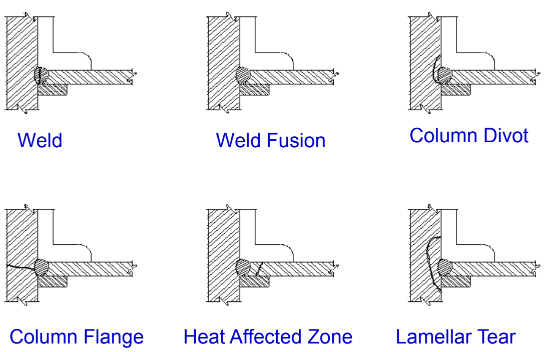
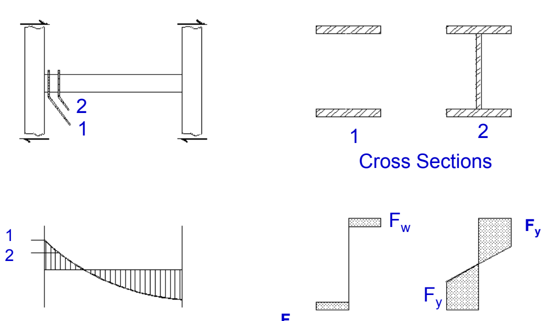
Northridge Failure



Northridge Failures



Flexural Mechanics at a Joint

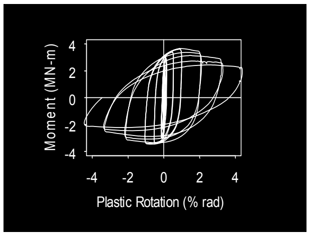


$F_w \cdot Z_1 > F_y \cdot Z_2$

Welded Steel Frames

- Northridge showed serious flaws. Problems correlated with:
  - Weld material, detail concept and workmanship
  - Beam yield strength and size
  - Panel zone yield
- Repairs and new design
  - Move yield away from column face (cover plates, haunches, reduced beam section)
  - Verify through tests
- SAC Project: FEMA Publications 350 through 354
- AISC 358

Reduced Beam Section (RBS) Test Specimen SAC Joint Venture

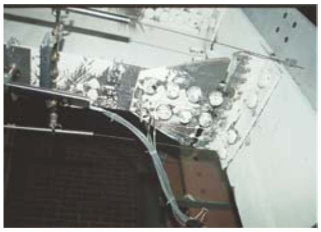


Graphics courtesy of Professor Chia-Ming Uang, University of California San Diego

T-stub Beam-Column Test SAC Joint Venture

Photo courtesy of Professor Roberto Leon, Georgia Institute of Technology

T-Stub Failure Mechanisms



Net section fracture in stem of T-stub

Plastic hinge formation -- flange and web local buckling

Photos courtesy of Professor Roberto Leon, Georgia Institute of Technology

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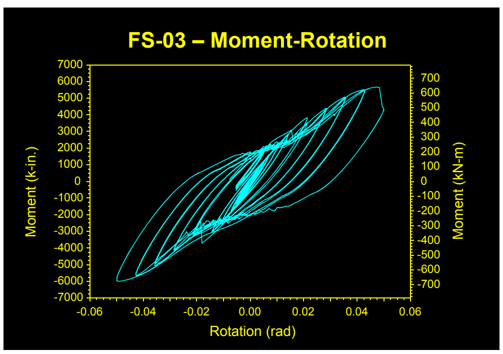
T-Stub Connection Moment Rotation Plot



Graphic courtesy of Professor Roberto Leon, Georgia Institute of Technology

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Extended Moment End-Plate Connection Res*ults*

Photo courtesy of Professor Thomas Murray, Virginia Tech

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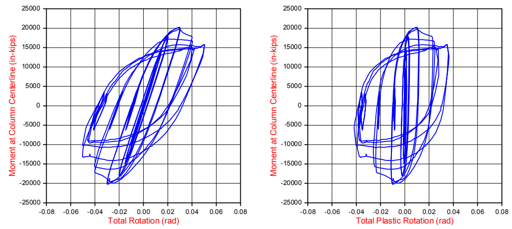
Extended Moment End-Plate Connection Results



(a) Moment vs Total Rotation

(b) Moment vs Plastic Rotation

Graphics courtesy of Professor Thomas Murray, Virginia Tech

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Ductility of Steel Frame Joints

Limit States

Welded Joints
- Brittle fracture of weld
- Lamellar tearing of base metal
- Joint design, testing, and inspection

Bolted Joints
- Fracture at net cross-section
- Excessive slip

Joint Too Weak For Member
- Shear in joint panel

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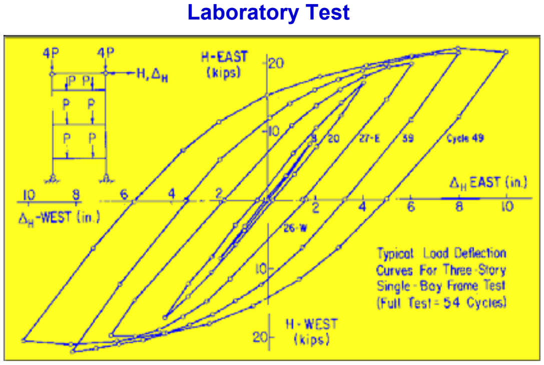
Multistory Frame
Laboratory Test



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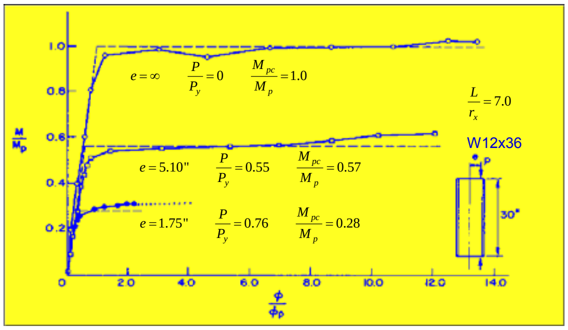
Flexural Ductility
Effect of Axial Load



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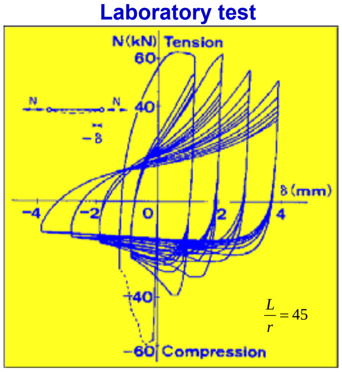
Axial Strut
Laboratory test



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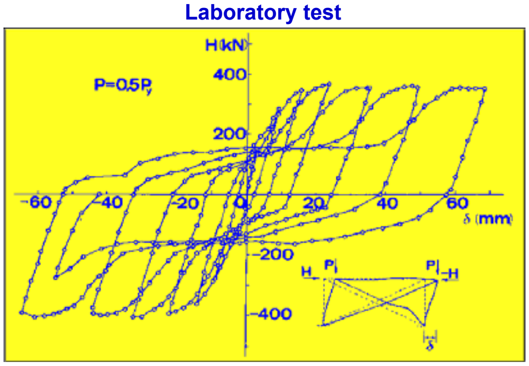
Cross Braced Frame
Laboratory test



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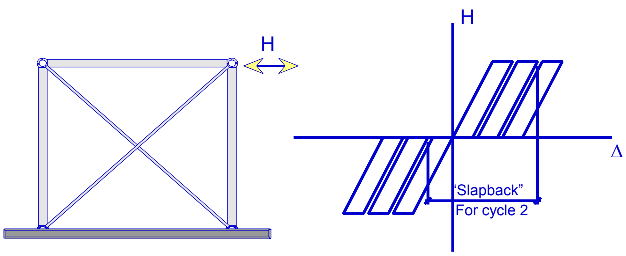
Tension Rod (Counter) Bracing
Conceptual Behavior



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Eccentrically Braced Frame

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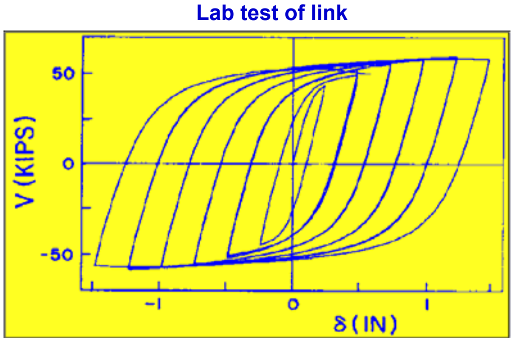
Eccentrically Braced Frame
Lab test of link



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Steel Behavior – Summary

- Ductility
  - Material inherently ductile
  - Ductility of structure < ductility of member < ductility of material
  - Achieved through detailing
- Damping
  - Welded structures have low damping
  - More damping in bolted structures due to slip at connections
  - Primary energy absorption is yielding of members

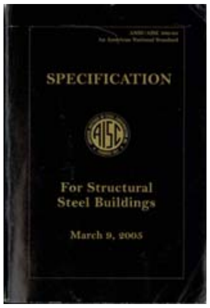
Steel Behavior – Summary

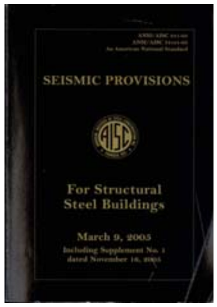
- Buckling
  - Most common steel failure under earthquake loads
  - Usually not ductile
  - Local buckling of portion of member
  - Global buckling of member
- Fracture
  - Nonductile failure mode under earthquake loads
  - Heavy welded connections susceptible
  - Net section rupture

NEHRP Recommended Provisions Steel Design

- Context in *Provisions*
- Steel behavior
- **Reference standards and design strength**

Steel Design Specifications





Using Reference Standards
Structural Steel

Both the AISC LRFD and ASD methodologies are presented in a unified format in both the *Specification for Structural Steel Buildings* and the *Seismic Provisions for Structural Steel Buildings*.

Cold Formed Steel Standard

Other Steel Members

Steel Joist Institute
Standard Specifications, 2002

Steel Cables
ASCE 19-1996

Steel Deck Institute
Diaphragm Design Manual, 3rd Ed., 2005

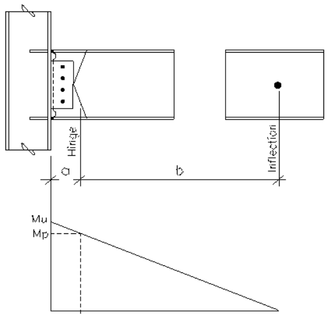
NEHRP Recommended Provisions Steel Design

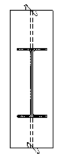
- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- **Moment resisting frames**

Steel Moment Frame Joints

Frame	Test	θ_i	Details
Special	Req'd	0.04	Many
Intermediate	Req'd	0.02	Moderate
Ordinary	Allowed	N.A.	Few

Steel Moment Frame Joints





$$M_u \approx M_p \cdot \frac{a+b}{b}$$

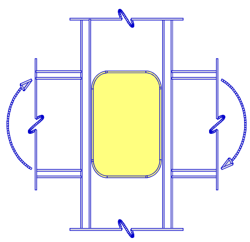
$$F_y^* = R_y \cdot F_y$$

$$F_u \approx F_y^* Z \cdot \frac{a+b}{b} \cdot \frac{1}{A_f d} \approx 1.7 F_y^*$$

Panel Zones

Special and intermediate moment frame:

- Shear strength demand:
  - Basic load combination
  - or
  - $\phi R_y M_p$ of beams
- Shear capacity equation
- Thickness (for buckling)
- Use of doubler plates (not economical, try to increase col. size instead)



Steel Moment Frames

- Beam shear: $1.1 R_y M_p$ + gravity
- Beam local buckling
  - Smaller *b*/*t* than for plastic design
- Continuity plates in joint per tests
- Strong column - weak beam rule
  - Prevent column yield except in panel zone
  - Exceptions: Low axial load, strong stories, top story, and non-SFRS columns

Steel Moment Frames

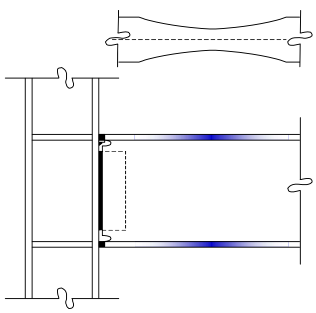
- Lateral support of column flange requirements
  - Top of beam if column elastic
  - Top and bottom of beam otherwise
  - Amplified forces for unrestrained
- Lateral support of beams requirements
  - Both flanges
  - Spacing < $0.086 r_y E/F_y$

Prequalified Connections

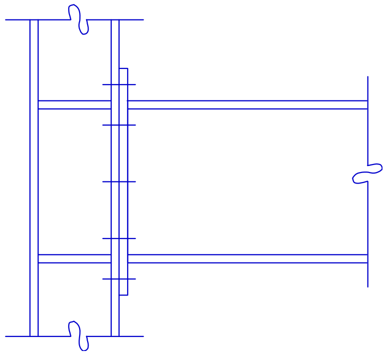
ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*
- Reduced Beam Section Connections
- Bolted Stiffened and Unstiffened Extended Moment End Plate Connections

Additional connections addressed in FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*:
- Welded Unreinforced Flange
- Welded Free Flange Connection
- Welded Flange Plate Connection
- Bolted Flange Plate Connection

Reduced Beam Section (RBS)

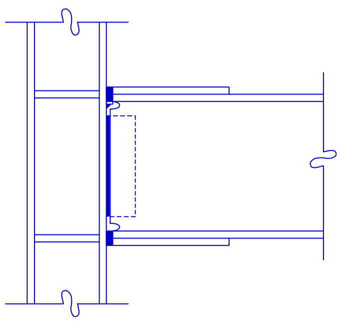


Extended End Plate



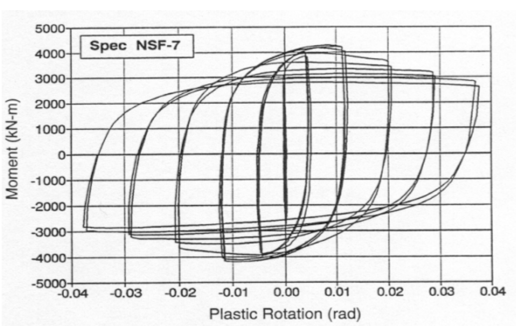
FEMA

Welded Flange Plates



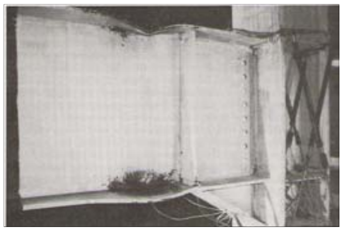
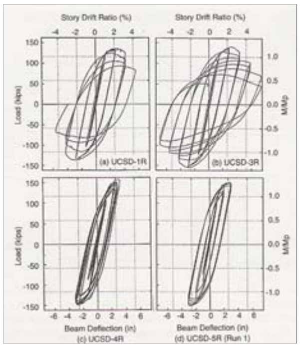
FEMA

Excellent Moment Frame Behavior

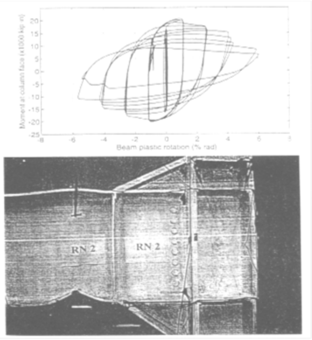


FEMA

Excellent Moment Frame Behavior

Excellent Moment Frame Behavior



Special Moment Frames

Seven Story Office Building, Los Angeles

Perimeter Moment Frames, all bays engaged
S_{DS}=1.0
S_{D1}=0.6
Occupancy Category II
Seismic Design Category D
Design Parameters (Table 12.2-1)
- R=8
- C_d=5.5
- Ω_0=3.0

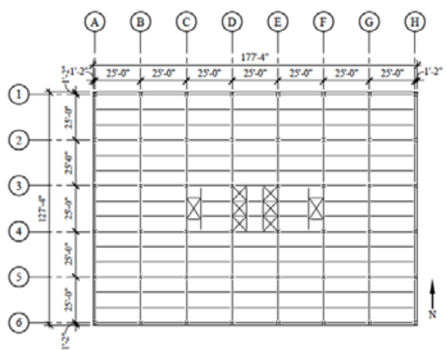
Special Moment Frame Example

Structural Materials:
- Concrete (all floors) = 3.0 ksi lightweight
- Other Concrete = 4.0 ksi normal weight

Steel:
- Wide Flange Sections= ASTM A992 Grade 50
- HSS= ASTM A500 Grade B
- Plates= ASTM A36

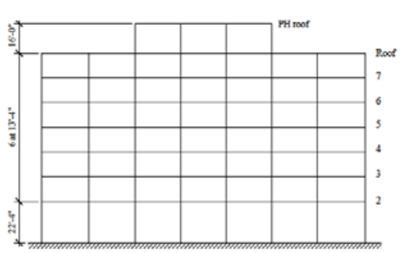
Special Moment Frames

Plan of Building



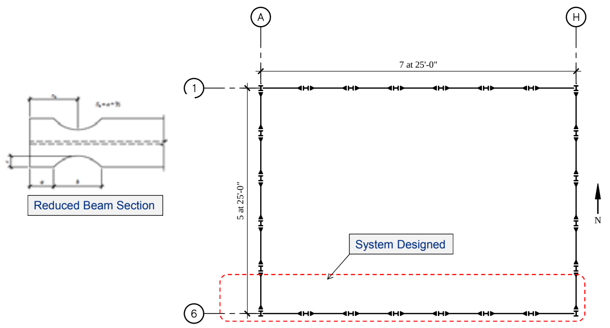
Special Moment Frames

Elevation of Building



Special Moment Frames

Perimeter Moment Frames with RBS



Special Moment Frames

The following design steps will be reviewed:

- Compute Lateral Loads
- Select preliminary member sizes
- Check member local stability
- Check deflection and drift
- Check torsional amplification
- Check the column-beam moment ratio rule
- Check shear requirement at panel zone
- Select connection configuration

Special Moment Frames

Building Weight:

Penthouse Roof = 94 kips
Lower Roof = 1,537 kips
Typical Floor = 1,920 kips
Total = 94 + 1,537 + 6(1,920) = 13,151 kips

Building Period:

$T_a=C_t h_n^x=(0.028)\,(102.3)^{0.8} = 1.14$ sec.
$T=C_u T_a = (1.4)(1.14)=1.596$ sec.

Design Base Shear:

$C_s=S_{D1}/(T/(R/I))=0.6/(1.596/(8/1))=0.047$ <<< CONTROLS
$C_{s,min} = 0.044\ S_{DS}I=0.044(1.0)(1)=0.044$
$V=C_sW=0.047(13,151)=618$ kips.

Special Moment Frames

Select preliminary member sizes – The preliminary member sizes are given in the next slide for the frame in the East-West direction. These members were selected based on the use of a 3-Dimensional model analyzed using the program *ETABS*. As will be discussed in a subsequent slide, the drift requirements controlled the design of these members.

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SMF Example – Preliminary Member Sizes

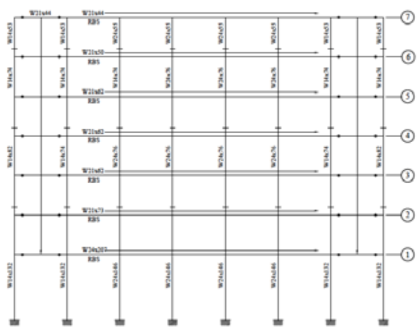

Figure 6.2-3 SMRF frame in E-W direction (penthouse not shown)

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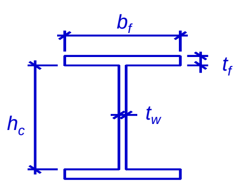
SMF Example – Check Member Local Stability

Check beam flange: $\frac{b_f}{2t_f} = 6.01$
(W33x141 A992)

Upper limit: $0.3\sqrt{\frac{E}{F_y}} = 7.22$ OK

Check beam web: $\frac{h_c}{t_w} = 49.6$

Upper limit: $2.45\sqrt{\frac{E}{F_y}} = 59.0$ OK



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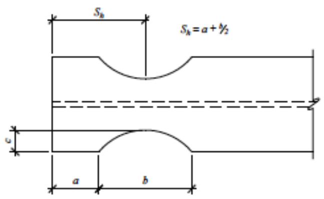
SMF Example – RBS Details



$a = 0.625b_f$
$b = 0.75d_b$
$c = 0.20b_f$

SMF Example – Check Deflection and Drift

The frame was checked for an allowable story drift limit of $0.020h_{sx}$. All stories in the building met the limit. Note that the *NEHRP Recommended Provisions* Sec. 4.3.2.3 requires the following check for vertical irregularity:

$$\frac{C_d \Delta_{x,story2}}{C_d \Delta_{x,story3}} = \frac{(1.2)}{(1.8)} = 0.67 < 1.3$$

Therefore, there is no vertical irregularity.

SMF Example – Check Deflection and Drift

Table 6.2-1 Alternative A (Moment Frame) Story Drifts under Seismic Loads

	Elastic Displacement at Building Corner, From Analysis		Expected Displacement ($=\delta_e C_d$)		Design Story Drift Ratio		Allowable Story Drift Ratio
Level	δ_eE-W (in.)	δ_eN-S (in.)	δ E-W (in.)	δ N-S (in.)	Δ E-W/h (%)	Δ N-S/h (%)	Δ/h (%)
Level 7	2.92	3.18	16.0	17.5	1.2	1.2	2.0
Level 6	2.66	2.89	14.7	15.9	1.4	1.7	2.0
Level 5	2.33	2.47	12.8	13.6	1.6	2.0	2.0
Level 4	1.91	1.95	10.5	10.7	1.9	2.0	2.0
Level 3	1.41	1.40	7.76	7.70	1.8	1.8	2.0
Level 2	0.90	0.88	4.96	4.85	1.2	1.2	2.0
Level 1	0.55	0.52	3.04	2.89	1.1	1.1	2.0

1.0 in. = 25.4 mm.

Building Satisfies Drift Limits

SMF Example – Check Torsional Amplification

The torsional amplification factor is given below. If $A_x < 1.0$ then torsional amplification is not required. From the expression it is apparent that if δ_{max} / δ_{avg} is less than 1.2, then torsional amplification will not be required.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2$$

The 3D analysis results, as shown in FEMA P-751, indicate that none of the δ_{max} / δ_{avg} ratios exceed 1.2; therefore, torsional amplification is not required.

SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_u/\phi P_n$ is typically less than 0.4 for the columns, combinations involving Ω_0 factors do not come into play for the special steel moment frames (re: AISC Seismic Sec. 8.3). In sizing columns (and beams) for strength one should satisfy the most severe value from interaction equations. However, the frame in this example is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

SMF Example – Column-Beam Moment Ratio

Per AISC Seismic Sec. 9.6

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

where ΣM^*_{pc} = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. ΣM^*_{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.

ΣM^*_{pb} = the sum of the moments in the beams at the intersection of the beam and column centerlines.

SMF Example – Column-Beam Moment Ratio

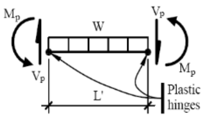
Column – W24x146; beam – W21x73

$$\Sigma M^*_{pc} = \Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right) + \frac{\left(M_{BF\,i} + M_{BF\,i+1} \right)}{h_c} \frac{d_b}{2}$$
$$= 39400 \text{ in.-kips}$$

SMF Example – Column-Beam Moment Ratio

For beams:

$$\Sigma M^*_{pb} = \Sigma \left[M_{pr} + V_e \left(S_h + \frac{d_c}{2} \right) \right]$$
$$= 17749 \text{ in.-kips}$$



where

- $M_{pr} = C_{pr} R_y F_y Z_e$
- Z_e = effective Z (RBS)
- $V_e = 2\, M_{pr} / L'$
- S_h = dist. from col. centerline to plastic hinge

SMF Example – Column-Beam Moment Ratio

The ratio of column moment strengths to beam moment strengths is computed as:

$$\text{Ratio} = \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = \frac{39400 \text{ in.-kips}}{17749 \text{ in.-kips}} = 2.22 > 1.0$$

Other ratios are also computed to be greater than 1.0

SMF Example – Panel Zone Check

The 2005 AISC Seismic specification is used to check the panel zone strength. Note that FEMA 350 contains a different methodology, but only the most recent AISC provisions will be used. From analysis shown in the NEHRP *Design Examples* volume (FEMA 451), the factored strength that the panel zone at Story 2 of the frame in the EW direction must resist is 794 kips. The shear transmitted to the joint from the story above, V_c, opposes the direction of R_u and may be used to reduce the demand. Previously calculated, this is 102 kips at this location. Thus the frame R_u = 794 – 102 = 692 kips.

$$R_n = 0.60 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)$$

$$= 547 \text{ kips}$$

Since ϕ_v = 1, $\phi_v R_b$ = 547 kips, which is less than the required resistance, 692 kips. Therefore, doubler plates are required. The required additional strength from the doubler plates is 692 – 547 = 145 kips. The plates must be at least ¼" thick as the strength of the double plates is:

$$\phi_v R_n = 0.6 t_{doub} d_c F_y$$

SMF Example – Connection Configuration

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

(3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1 R_y M_p]/L_h \qquad (9\text{-}1)$$

SMF Example – Connection Configuration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 9.2a by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Appendix P.

(c) Provision of qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

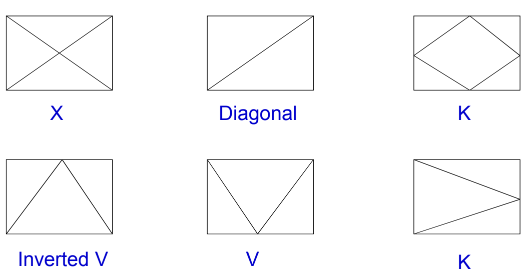
Special Moment Frames
Summary

Beam-to-column connection capacity
Select preliminary member sizes
Check member local stability
Check deflection and drift
Check torsional amplification
Check the column-beam moment ratio rule
Check shear requirement at panel zone
Select connection configuration
- Prequalified connections
- Testing

NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Seismic design category requirement
- Moment resisting frames
- **Braced frames**

Concentrically Braced Frames
Basic Configurations



Braced Frame Under Construction

FEMA

Braced Frame Under Construction

FEMA

Concentrically Braced Frames

Special AISC Seismic R = 6
Chapter 13

Ordinary AISC Seismic R = 3.25
Chapter 14

Not Detailed for Seismic R = 3
AISC LRFD

FEMA

Concentrically Braced Frames

Dissipate energy after onset of global buckling by avoiding brittle failures:

- Minimize local buckling
- Strong and tough end connections
- Better coupling of built-up members

Concentrically Braced Frames

Special and Ordinary

Bracing members:

- Compression capacity = $\phi_c P_n$
- Width / thickness limits
  - Generally compact
  - Angles, tubes and pipes very compact
- Overall $\frac{KL}{r} < 4\sqrt{\frac{E}{F_y}}$ < 200 for SCBF
- Balanced tension and compression

Concentrically Braced Frames

Special concentrically braced frames

Brace connections

Axial tensile strength > smallest of:

- Axial tension strength = $R_y F_y A_g$
- Maximum load effect that can be transmitted to brace by system

Axial compressive strength ≥ $1.1R_yP_n$, where P_n is the nominal compressive strength of the brace.

Flexural strength > $1.1R_yM_p$ or rotate to permit brace buckling while resisting A_gF_{CR}

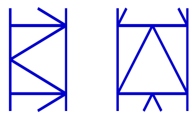
Concentrically Braced Frames

V bracing:

- Design beam for D + L + unbalanced brace forces, using $0.3P_c$ for compression and $R_y F_y A_g$ in tension
- Laterally brace the beam
- Beams between columns must be continuous

K bracing:

- Not permitted



Concentrically Braced Frames

Built-up member stitches:

- Spacing < 40% KL/r
- No bolts in middle quarter of span
- Minimum strengths related to P_y

Column in CBF:

- Same local buckling rules as brace members
- Splices resist moments

Special Concentrically Braced Frame Example

Seven Story Office Building, Los Angeles

Perimeter Moment Frames, all bays engaged

S_{DS}=1.0

S_{D1}=0.6

Occupancy Category II

Seismic Design Category D

Design Parameters (Table 12.2-1)

R=6

C_d=5.0

Ω_0=2.0

Concentrically Braced Frame Example

The following general design steps are required:

- Selection of preliminary member sizes
- Check strength
- Check drift
- Check torsional amplification
- Connection design

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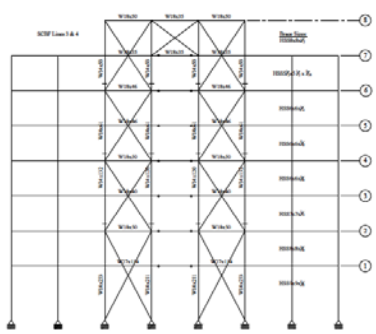
E-W Direction Framing

and Preliminary Member Sizes



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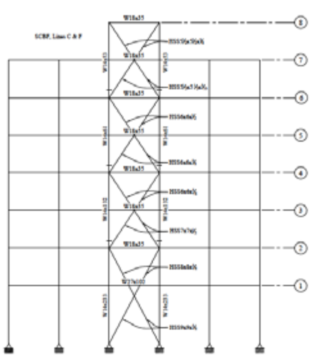
N-S Direction Framing

and Preliminary Member Sizes



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CBF Example

Building Weight:

Penthouse Roof = 94 kips
Lower Roof = 1,537 kips
Typical Floor = 1,920 kips
Total = 94 + 1,537 + 6(1,920) = 13,151 kips

Building Period:

$T_a = C_t h_n^x = (0.02)(102.3)^{0.75} = 0.64$ sec.
$T = C_u T_a = (1.4)(0.64) = 0.896$ sec.

Design Base Shear:

$C_s = S_{D1}/(T/(R/I)) = 0.6/(0.896/(6/1)) = 0.112$ <<< CONTROLS
$C_{s,min} = 0.044\, S_{DS} I = 0.044(1.0)(1) = 0.044$
$V = C_s W = 0.112(13,151) = 1,473$ kips.

CBFF Example – Check Deflection and Drift

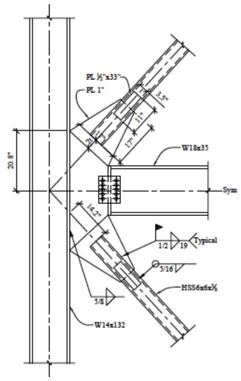
Table 6.2-2 Alternative B Story Drifts under Seismic Load

	Elastic Displacement at Building Corner, From Analysis		Expected Displacement ($=\delta_e C_d$)		Design Story Drift Ratio		Allowable Story Drift Ratio
Level	δ_eE-W (in.)	δ_eN-S (in.)	δ E-W (in.)	δ N-S (in.)	Δ E-W/h (%)	Δ N-S/h (%)	Δ/h (%)
Level 7	1.63	1.75	8.14	8.76	0.72	0.93	2.0
Level 6	1.41	1.48	7.07	7.38	0.74	0.94	2.0
Level 5	1.19	1.20	5.97	5.99	0.76	0.84	2.0
Level 4	0.96	0.94	4.80	4.72	0.81	0.85	2.0
Level 3	0.71	0.69	3.56	3.43	0.72	0.71	2.0
Level 2	0.49	0.47	2.44	2.33	0.60	0.59	2.0
Level 1	0.30	0.28	1.49	1.40	0.56	0.52	2.0

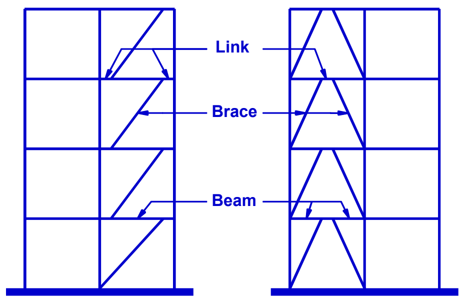
1.0 in. = 25.4 mm.

Building Easily Satisfies Drift Limits

CBF Example – Connection Detail



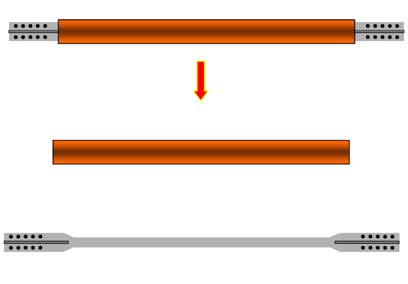
Eccentrically Braced Frames



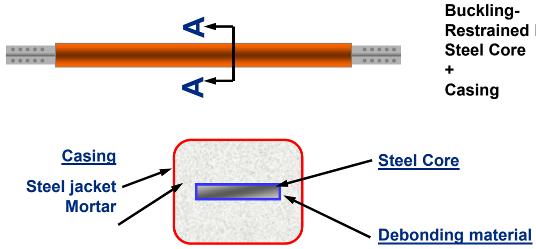
Buckling-Restrained Braced Frames (BRBFs)

- Type of concentrically braced frame
- Beams, columns and braces arranged to form a vertical truss. Resist lateral earthquake forces by truss action
- Special type of brace members used: *Buckling-Restrained Braces (BRBs)*. BRBS yield both in tension and compression - *no buckling !!*
- Develop ductility through inelastic action (cyclic tension and compression yielding) in BRBs.
- System combines high stiffness with high ductility

Buckling-Restrained Braced Frames (BRBFs)



Buckling-Restrained Braced Frames (BRBFs)



Buckling-Restrained Brace: Steel Core + Casing

Section A-A

Buckling-Restrained Braced Frames (BRBFs)

Steel core resists entire axial force P

Casing is debonded from steel core
- casing does not resist axial force P
- flexural stiffness of casing restrains buckling of core

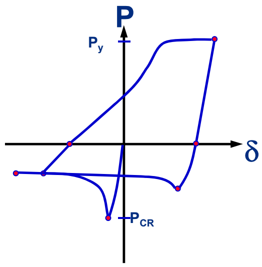
Buckling-Restrained Braced Frames (BRBFs)

Buckling-Restrained Brace: Steel Core + Casing

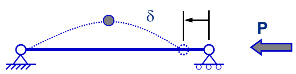
Steel Core

Core projection and brace connection segment

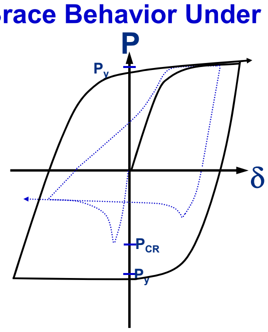
Brace Behavior Under Cyclic Axial Loading



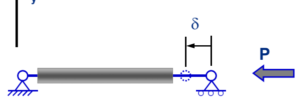
Conventional Brace:
- yields in tension (ductile)
- buckles in compression (nonductile)
- significantly different strength in tension and compression



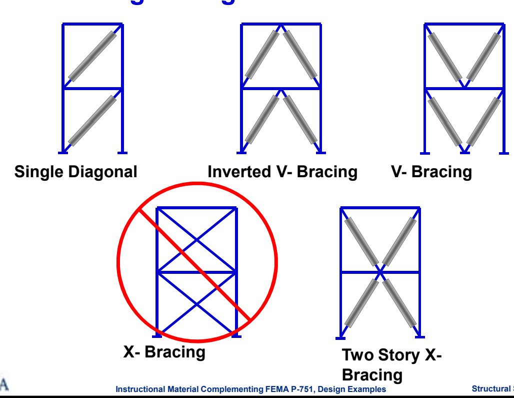
Brace Behavior Under Cyclic Axial Loading



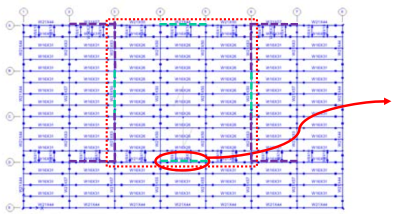
Buckling-Restrained Brace:
- yields in tension (ductile)
- yields in compression (ductile)
- similar strength in tension and compression (slightly stronger in compression)



Bracing Configurations for BRBFs

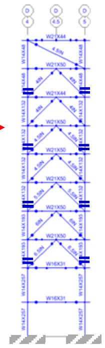


BRB Design Example



Plan View – 3rd Level (roof of podium, base of tower)

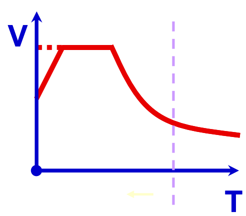
- Indicates extent of tower floorplate (above)
- Indicates bay of BRBs in tower (above)
- Indicates bay of BRBs in podium (below)



R = 8

ASCE 7 2005
AISC Seismic 2005

Base Shear



Hazard

S_{ds} = 0.859

S_{d1} = 0.433

T_a = 1.32 sec.

V = 0.057 W

Load Combinations

Basic	Special (Amplified Seismic Load)
$1.2D + f_1L + 0.2S + E$	$1.2D + f_1L + 0.2S + E_m$
$0.9D \pm E$	$0.9D \pm E_m$

$f_1 = 0.5$

$E = \rho Q_E + 0.2S_{DS} D$ $\qquad$ $E_m = \Omega_o Q_E + 0.2S_{DS} D$

$1.37D + 0.5L + 0.2S + \rho Q_E$	$1.37D + 0.5L + 0.2S + \Omega_o Q_E$
$0.73D \pm \rho Q_E$	$0.73D \pm \Omega_o Q_E$

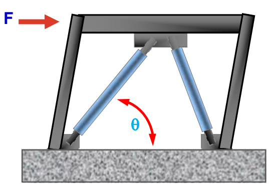
Vertical Distribution of Forces

Diaphragm Level	Story Force (kips)	Brace Level	Story Shear (kips)	% of Total Base Shear
Roof	210	10	210	24%
10	152	9	362	42%
9	126	8	488	56%
8	102	7	590	68%
7	80	6	670	77%
6	60	5	730	84%
5	43	4	772	89%
4	28	3	800	92%
3	51	2	851	98%
2	14	1	865	100%

Preliminary Design of Braces

$$P_u = \frac{F}{2\cos\theta}$$

Assume braces resist 100% of story shear



$$A_{sc} = \frac{P_u}{\phi F_y}$$

Design braces precisely to calculated capacity
($P_u = \phi P_n = \phi F_y A_{sc}$)

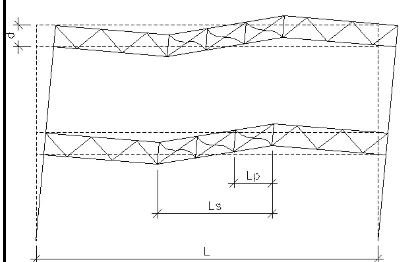
BRB Design – Gridline D

Brace Level	Brace Angle α (deg)	Brace Force P_u (kips)	Required Core Area A_{sc}	Provided Core Area A_{sc}
10	26.6	142	4.21	4.5
9	45.0	126	3.71	4.0
8	45.0	133	3.98	4.0
7	45.0	141	4.12	4.5
6	45.0	151	4.42	4.5
5	45.0	170	5.12	5.5
4	45.0	197	5.70	6.0
3	45.0	210	6.26	6.5
2	50.1	187	5.76	6.0
1	50.1	195	6.81	7.0

NEHRP Recommended Provisions
Steel Design

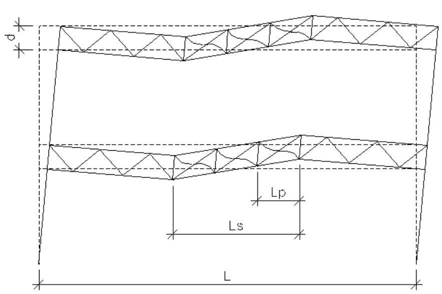
- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- **Other topics**

Special Truss Moment Frame



- Buckling and yielding in special section
- Design to be elastic outside special section
- Deforms similar to EBF
- Special panels to be symmetric X or Vierendeel

Special Truss Moment Frame



Geometric Limits:

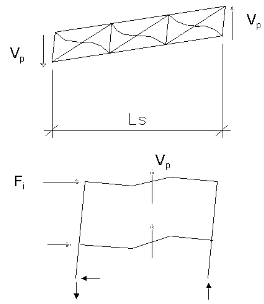
$L \le 65'$ $\quad d \le 6'$

$$0.1 < \frac{L_s}{L} < 0.5$$

$$\frac{2}{3} < \frac{L_p}{d} < \frac{3}{2}$$

Flat bar diagonals, $\frac{b}{t} \le 2.5$

Special Truss Moment Frame



$$V_p = 2\left(\frac{2\,M_{pc}}{L_s}\right) + \sin\alpha\,\left(P_{nt} + 0.3P_{cd}\right)$$

$$\sum F_i\,h_i = \sum V_p\,L$$

Special Truss Moment Frame

Special Truss Moment Frame

General Seismic Detailing

Materials:
- Limit to lower strengths and higher ductilities

Bolted Joints:
- Fully tensioned high strength bolts
- Limit on bearing

General Seismic Detailing

Welded Joints:
- AWS requirements for welding procedure specs
- Filler metal toughness
  - CVN > 20 ft-lb @ -20°F, or AISC Seismic App. X
- Warning on discontinuities, tack welds, run offs, gouges, etc.

Columns:
- Strength using Ω_o if $P_u / \phi P_n > 0.4$
- Splices: Requirements on partial pen welds and fillet welds

Steel Diaphragm

Example

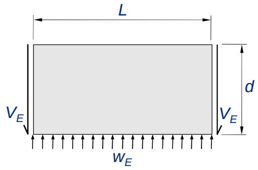
$\phi V_n = \phi$ (approved strength)

$\phi = 0.6$

For example only:
Use approved strength as 2.0 × working load in SDI *Diaphragm Design Manual*

Steel Deck Diaphragm

Example



$$L = 80' \qquad d = 40'$$

$$w_D = w_L = 0 \qquad w_E = 500 \text{ plf}$$

$$V_E = \frac{w_E L}{2} = 20 \text{ kip}; \quad v_E = \frac{20000}{40} = 500 \text{ plf}$$

$$v_{SDI} = \frac{v_E}{2\phi} = \frac{500}{2(0.6)} = 417 \text{ plf}$$

Deck chosen:
1½", 22 gage with welds on 36/5 pattern and 3 sidelap fasteners, spanning 5'-0"

Capacity = 450 > 417 plf

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Welded Shear Studs

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Shear Stud Strength - AISC 2005 Specification

$Q_n = 0.5\, A_{sc}\, (\, f_c'\, E_c)^{1/2} \leq R_g\, R_p\, A_{sc}\, F_u$

R_g = stud geometry adjustment factor
R_p = stud position adjustment factor

Note that the strength reduction factor for bending has been increased from 0.85 to 0.9. This results from the strength model for shear studs being more accurate, although the result for Q_n is lower in the 2005 specification.

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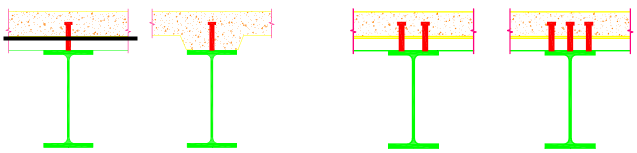
Shear Studs – Group Adjustment Factor

$Q_n = 0.5\, A_{sc}\, (f_c'\, E_c)^{1/2} \le R_g\, R_p\, A_{sc}\, F_u$
R_g = stud group adjustment factor

R_g = 1.0 R_g = 1.0* R_g = 0.85 R_g = 0.7



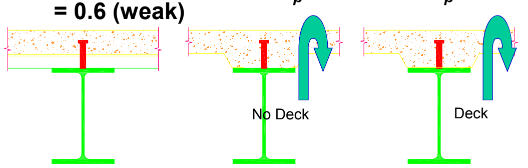
*0.85 if w_r / h_r < 1.5

Shear Studs – Position Adjustment Factor

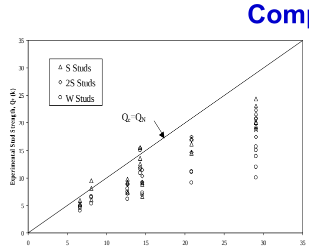
$Q_n = 0.5\, A_{sc}\, (f_c'\, E_c)^{1/2} \le R_g\, R_p\, A_{sc}\, F_u$
R_p = stud position adjustment factor

R_p = 0.75 (strong)
= 0.6 (weak)

R_p = 1.0 R_p = 0.75

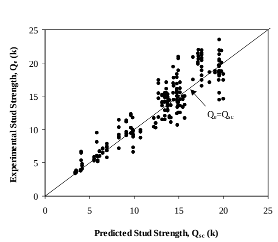


Shear Studs – Strength Calculation Model Comparison



AISC Seismic prior to 2005

Virginia Tech strength model



Shear Studs – Diaphragm Applications

Shear studs are often used along diaphragm collector members to transfer the shear from the slab into the frame. The shear stud calculation model in the 2005 AISC specification can be used to compute the nominal shear strengths. A strength reduction factor should be used when comparing these values to the factored shear. There is no code- established value for the strength reduction factor. A value of 0.8 is recommended pending further development.

Inspection and Testing

Inspection Requirements

- Welding:
  - Single pass fillet or resistance welds
    PERIODIC
  - All other welds
    CONTINUOUS
- High strength bolts:
  PERIODIC

Inspection and Testing

Shop Certification

- Domestic:
  - AISC
  - Local jurisdictions
- Foreign:
  - No established international criteria

Inspection and Testing
Base Metal Testing

- More than 1-1/2 inches thick
- Subjected to through-thickness weld shrinkage
- Lamellar tearing
- Ultrasonic testing

NEHRP Recommended Provisions Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics

Questions