NEHRP RECOMMENDED PROVISIONS SEISMIC DESIGN OF STEEL STRUCTURES

- Context in *NEHRP Recommended 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

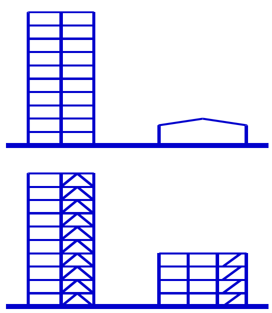
<u>Using the 2003 *NEHRP Recommended Provisions*:</u>

Load combination	Chap. 4
Seismic load analysis	Chap. 5
Components and attachments	Chap. 6
Design of steel structures	Chap. 8
	AISC Seismic
	and others

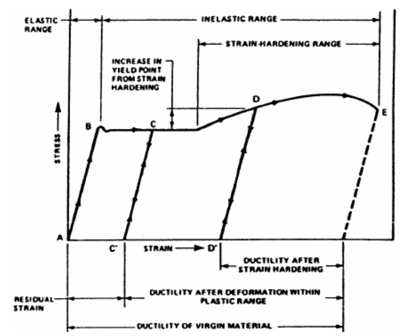
Seismic Resisting Systems

Unbraced Frames
- Joints are:
  Rigid/FR/PR/
  Moment-resisting
- Seismic classes are:
  Special/intermediate/
  Ordinary/not detailed

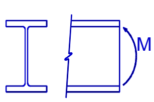
Braced Frames
- Concentric bracing
- Eccentric bracing



Monotonic Stress-Strain Behavior



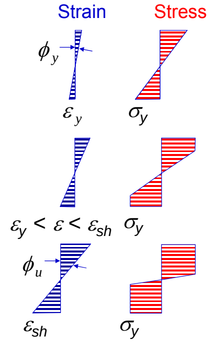
Bending of Steel Beam



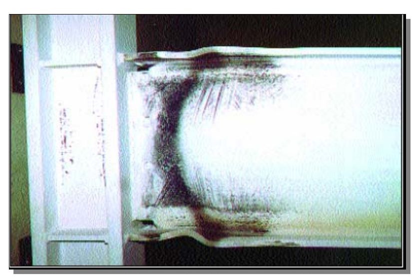
Extreme fiber reaches yield strain and stress

Strain slightly above yield strain

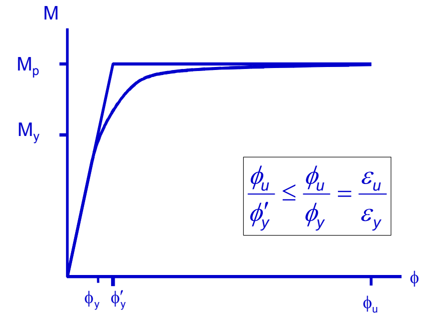
Section near “plastic”



Plastic Hinge Formation

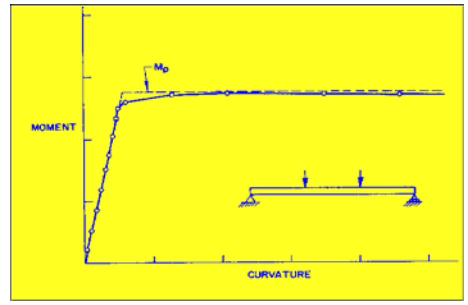


Cross - section Ductility
Conceptual moment - curvature

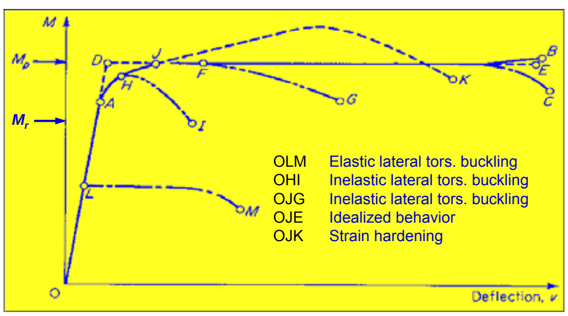


$$\frac{\phi_u}{\phi'_y} \le \frac{\phi_u}{\phi_y} = \frac{\varepsilon_u}{\varepsilon_y}$$

Moment Curvature
Laboratory Test -- Annealed W Beam



Behavior Modes For Beams

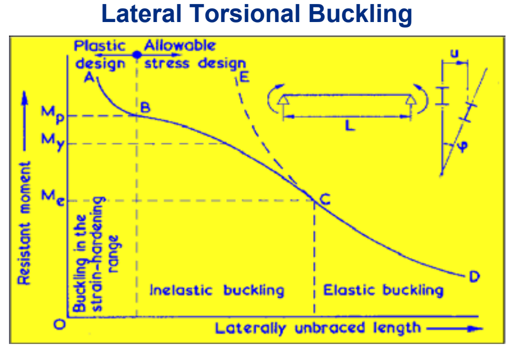


Flexural Ductility of Steel Members
Practical Limits

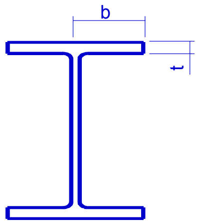
1 Lateral torsional buckling
  Brace well
2 Local buckling
  Limit width-to-thickness ratios
  for compression elements
3 Fracture
  Avoid by proper detailing

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

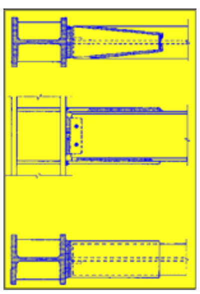
With the plate buckling coefficient taken as 0.7 and an adjustment for residual stresses, the expression for b/t becomes:

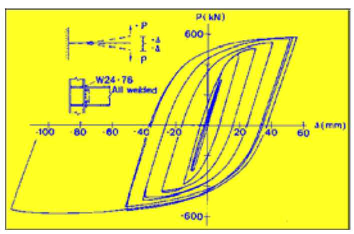
$$\frac{b}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$$

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

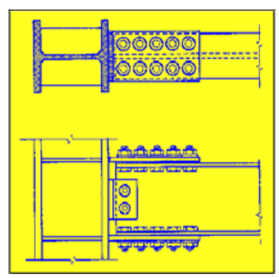
$$\frac{b}{t} \leq 0.3\sqrt{\frac{E}{F_y}}$$

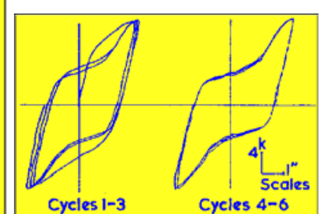
Welded Beam to Column Laboratory Test - 1960s



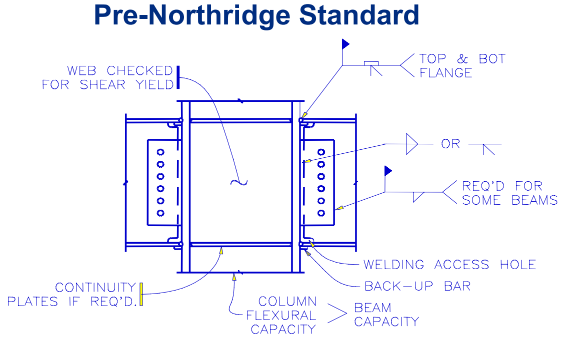


Bolted Beam to Column Laboratory Test - 1960s





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 research effort known as 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

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Beam Bottom Flange Weld Fracture Causing a Column Divot Fracture

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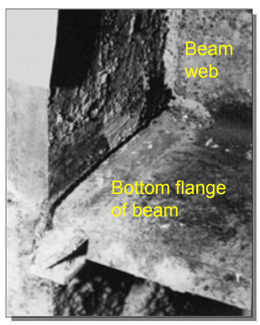
Northridge Failure

- Crack through weld
- Note backup bar and runoff tab



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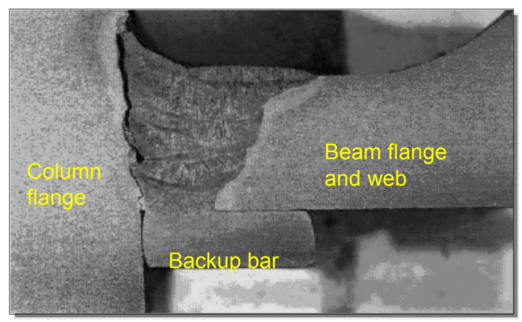
Northridge Failure



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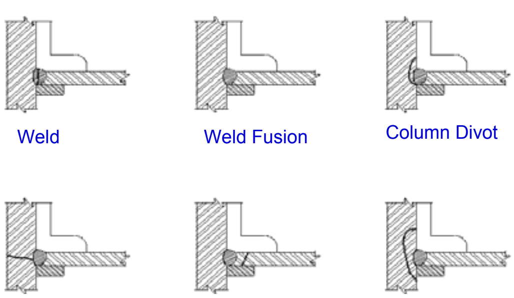
Northridge Failures



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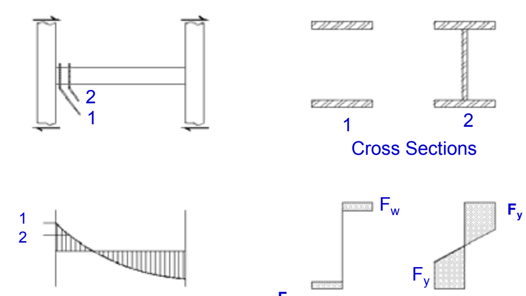
Flexural Mechanics at a Joint



$F_w \cdot Z_1 > F_y \cdot Z_2$

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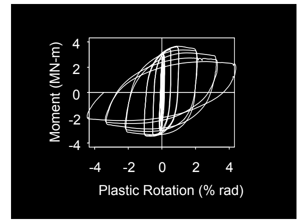
Welded Steel Frames

- 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, “dog bone”)
  - Verify through tests
- SAC Project: FEMA Publications 350 through 354

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

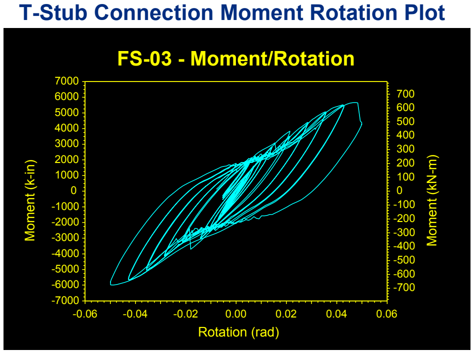
T-Stub Failure Mechanisms

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

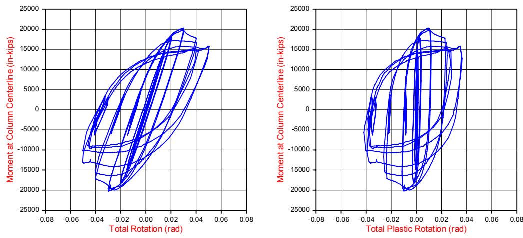


Graphic courtesy of Professor Roberto Leon, Georgia Institute of Technology

Extended Moment End-Plate Connection Results

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

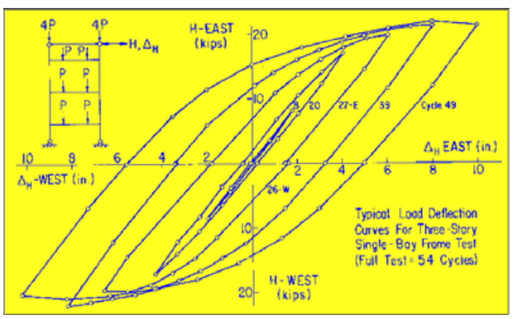
Ductility of Steel Frame Joints
Limits

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

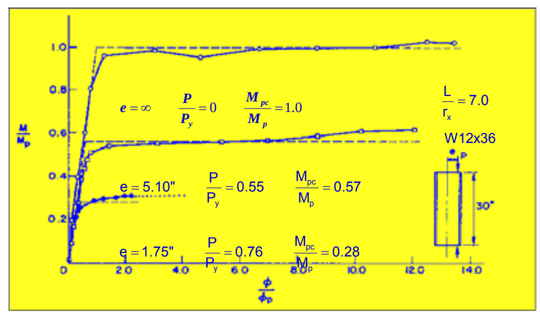
Bolted Joints
- Fracture at net cross-section
- Excessive slip

Joint Too Weak For Member
- Shear in joint panel

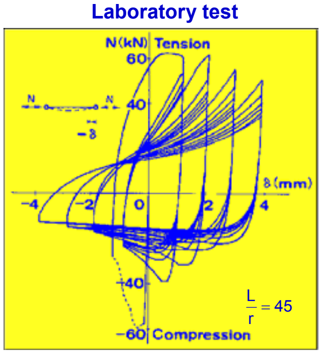
Multistory Frame
Laboratory Test



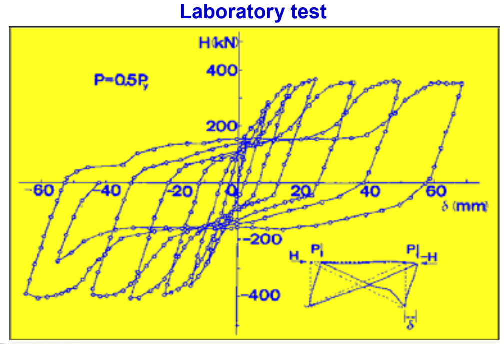
Flexural Ductility
Effect of Axial Load



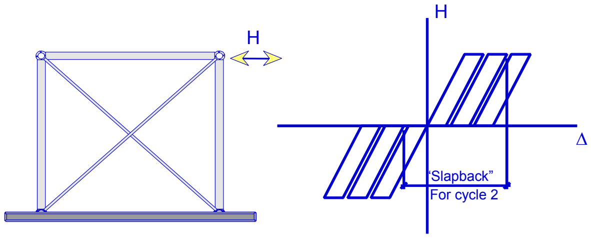
Axial Strut
Laboratory test



Cross Braced Frame
Laboratory test

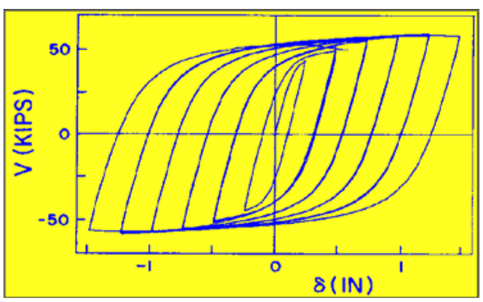


Tension Rod (Counter) Bracing
Conceptual Behavior



Eccentrically Braced Frame

Eccentrically Braced Frame
Lab test of link



Steel Behavior

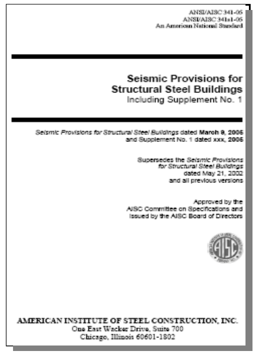
- Ductility
  - Material inherently ductile
  - Ductility of structure < ductility of material
- Damping
  - Welded structures have low damping
  - More damping in bolted structures due to slip at connections
  - Primary energy absorption is yielding of members

Steel Behavior

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

NEHRP Recommended Provisions Steel Design

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



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*.

Other Steel Members

Steel Joist Institute
Standard Specifications, 2002

Steel Cables
ASCE 19-1996

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

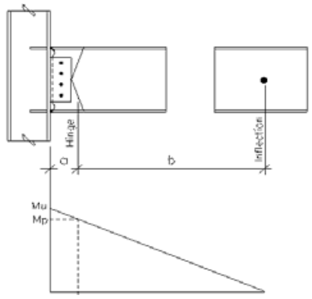
NEHRP Recommended Provisions
Steel Design

- Context in *NEHRP Recommended 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	NA	Few

Steel Moment Frame Joints



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

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

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

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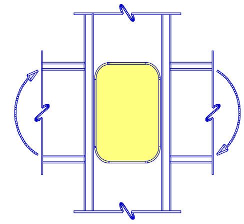
Panel Zones

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



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Steel Moment Frames

- Beam shear: $1.1 R_y M_p$ + gravity
- Beam local buckling
  - Smaller b/t than LRFD 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-SRS columns

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Steel Moment Frames

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

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Prequalified Connections

See FEMA 350: *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*

-Welded Unreinforced Flange
-Welded Free Flange Connection
-Welded Flange Plate Connection
-Reduced Beam Section Connections
-Bolted Unstiffened End Plate Connection
-Bolted Stiffened End Plate Connection
-Bolted Flange Plate Connection

See 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

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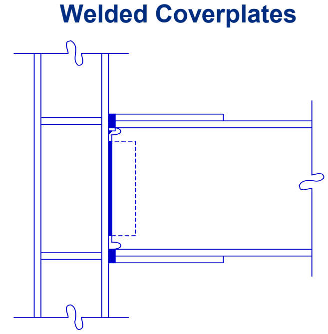
Welded Coverplates



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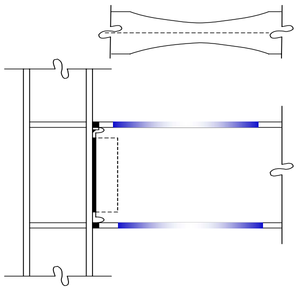
Reduced Beam Section (RBS)



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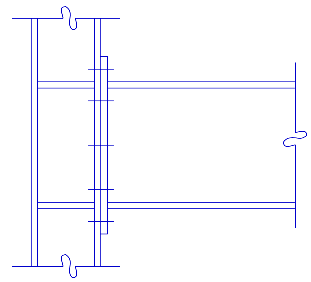
Extended End Plate



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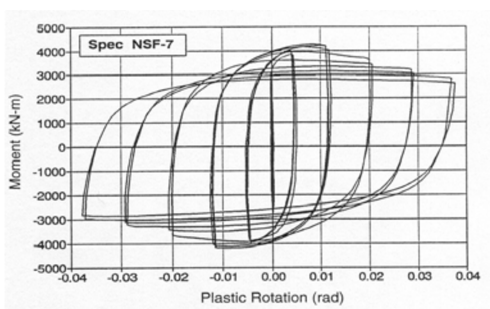
Excellent Moment Frame Behavior



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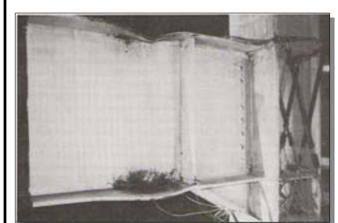
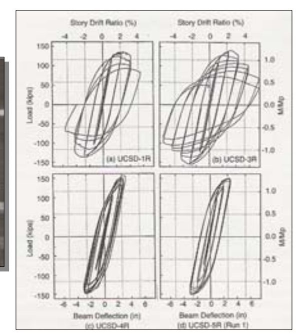
Excellent Moment Frame Behavior





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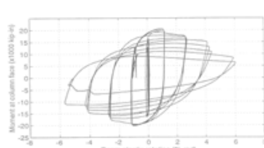
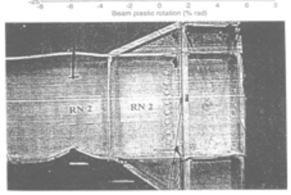
Excellent Moment Frame Behavior





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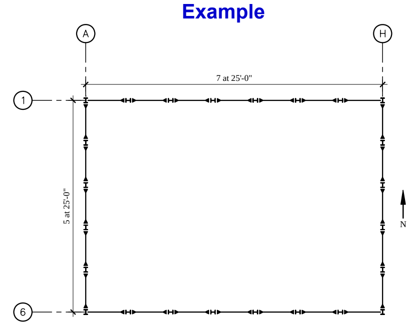
Special Moment Frames

Example



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Special Moment Frames

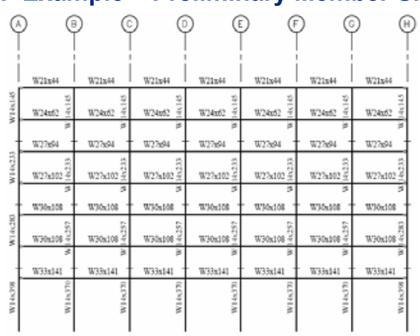
The following design steps will be reviewed:

- 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

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 3D stiffness model in the program RAMFRAME. As will be discussed in a subsequent slide, the drift requirements controlled the design of these members.

SMF Example – Preliminary Member Sizes



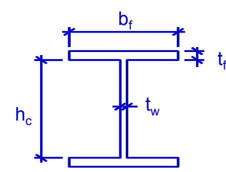
SMF Example – Check Member Local Stability

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

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

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

Upper limit: $3.76\sqrt{\dfrac{E}{F_y}} = 90.6$ OK



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:

$$\frac{C_d\Delta_{x\ story\ 2}}{C_d\Delta_{x\ story\ 3}} = \frac{\left(\dfrac{5.17in.}{268in.}\right)}{\left(\dfrac{3.14in.}{160in.}\right)} = 0.98 < 1.3$$

Therefore, there is no vertical irregularity.

SMF Example – Check Torsional Amplification

The torsional amplification factor is given below. If Ax < 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 451, indicate that none of the $\delta_{max} / \delta_{avg}$ ratios exceed 1.2; therefore, there is no torsional amplification.

SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_u/\varphi P_n$ 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 $\quad \dfrac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0$

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 – W14x370; beam – W33x141

$$\Sigma M^*_{pc} = \Sigma Z_c\left(F_{yc} - \frac{P_{uc}}{A_g}\right) = 2\left[736in^2\left(50ksi - \frac{500kips}{109in^2}\right)\right]$$

$$\Sigma M^*_{pc} = 66{,}850in - kips$$

Adjust this by the ratio of average story height to average clear height between beams.

$$\Sigma M^*_{pc} = 66{,}850in - kips\left(\frac{268in. + 160in.}{251.35in. + 128.44in.}\right) = 75{,}300in - kips$$

SMF Example – Column-Beam Moment Ratio

For beams:

$$\Sigma M^*_{pb} = \Sigma(1.1R_yM_p + M_v)$$

$$\text{with } M_v = V_pS_h$$

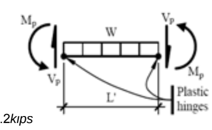
$$S_h = \text{dist. from col. centerline to plastic hinge}$$

$$= d_c/2 + d_b/2 = 25.61in.$$

$$V_p = \text{shear at plastic hinge location}$$

$$V_p = \left[2M_p + \left(wL'^2/2\right)\right] = \frac{2M_p + \frac{wL'^2}{2}}{L'}$$

$$= \frac{(2)(25{,}700in - kips) + \left(\frac{(1.046klf)}{12}\frac{(248.8in.)^2}{2}\right)}{248.8in.} = 221.2kips$$



SMF Example – Column-Beam Moment Ratio

$$M_v = V_pS_h = (221.2kips)(25.61in.) = 5{,}665in - kips$$

and

$$\Sigma M^*_{pb} = \Sigma(1.1R_yM_p + M_v)$$

$$= 2[(1.1)(1.1)(25{,}700in - kips) + 5{,}665in - kips] = 73{,}500in - kips$$

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

$$Ratio = \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = \frac{75{,}300in - kips}{73{,}500in - kips} = 1.02 > 1.00 \quad \therefore OK$$

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 1,883 kips.

$$R_v = 0.6F_yd_ct_p\left[1 + \frac{3b_{cf}t_{cf}^2}{d_bd_ct_p}\right] = (0.6)(50ksi)(17.92in.)(t_p)\left[1 + \frac{(3)(16.475in.)(2.66)^2}{(33.3in.)(17.92in.)(t_p)}\right]$$

$$R_v = 537.6t_p + 315$$

The required total (web plus doubler plate) thickness is determined *by*:

$$\phi R_v = R_u$$

$$(1.0)(537.6t_p + 315) = 1{,}883kips$$

$$t_{p_{required}} = 2.91in.$$

The column web thickness is 1.66*in.*, *therefore the required doubler plate thickness is*:

$$t_{p_{doubler}} = 1.25in. \quad (\text{therefore use one } 1.25in. \text{ plate or two } 0.625in. \text{ plates})$$

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.

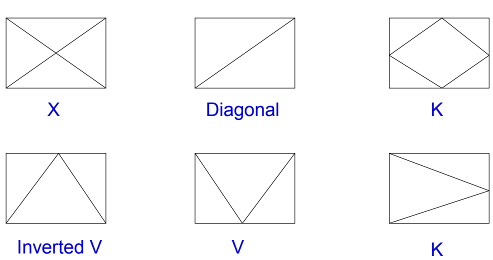
Special Moment Frames
Summary

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

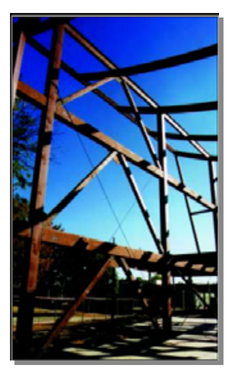
NEHRP Recommended Provisions
Steel Design

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

Concentrically Braced Frames
Basic Configurations



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 $\frac{KL}{r} < 4\sqrt{\frac{E}{F_y}}$
- Balanced tension and compression

Concentrically Braced Frames
Special concentrically braced frames

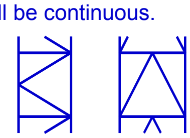
Brace connections
Axial tensile strength > smallest of:
- Axial tension strength = $R_y F_y A_g$
- Maximum load effect that can be transmitted to brace by system.

Axial compressive strength ≥ $1.1 R_y P_n$ 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 $A_g F_{CR}$

Concentrically Braced Frames

V bracing:
- Design beam for D + L + unbalanced brace forces, using $0.3\phi P_c$ for compression and $R_y F_y A_g$ in tension
- Laterally brace the beam
- Beams between columns shall be continuous.

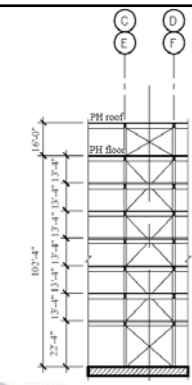
K bracing:
- Not permitted



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



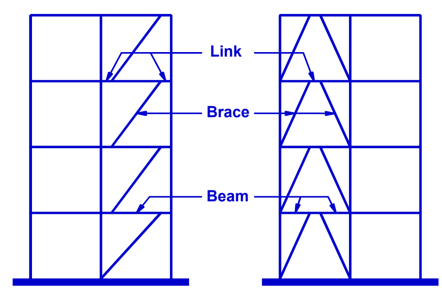
Concentrically Braced Frame Example

E-W direction

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

Eccentrically Braced Frames



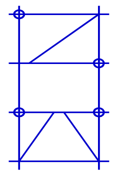
Eccentrically Braced Frame Under Construction

Eccentrically Braced Frame Under Construction

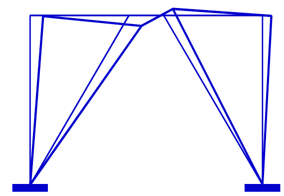
Eccentrically Braced Frames

Eccentric bracing systems	R	C_d
Building frame system or part of dual system w/ special moment frame		
With moment resisting connections at columns away from links	8	4
Without moment resisting connections at columns away from links	7	4

These connections determine classification

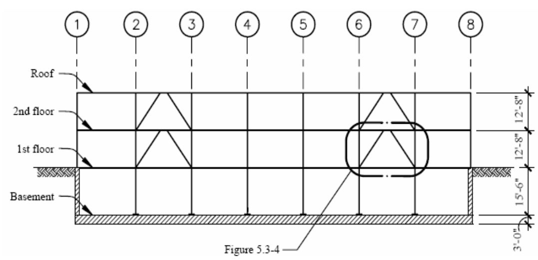


Eccentrically Braced Frames
Design Procedure



1. Elastic analysis
2. Check rotation angle; reproportion as required
3. Design check for strength
4. Design connection details

Eccentrically Braced Frames
Example



Eccentrically Braced Frames
Rotation Angle

1. Compute total $\Delta = C_d \Delta_E$
2. Deform model as rigid-plastic mechanism with hinges at ends of line
3. Compute rotation angle at end of link
4. Check limits (Sec. 15.2g)

$\alpha \leq 0.08$ radians when $L \leq \frac{1.6M_p}{V_p}$

$\alpha \leq 0.02$ radians when $L \geq \frac{2.6M_p}{V_p}$

Interpolate for α when $\frac{1.6M_p}{V_p} < L < \frac{2.6M_p}{V_p}$

Eccentrically Braced Frames
Rotation Angle Example

From computer analysis:

$\Delta_e = 0.247\ in$

Total drift:

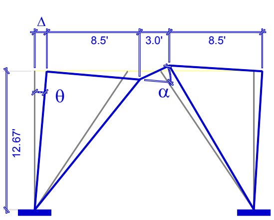
$\Delta = C_d \Delta_e = 4(0.247) = 0.99\ in.$

From geometry:

$\alpha = \left(\frac{L}{e}\right)\theta = \left(\frac{20}{3}\right)\left(\frac{0.99}{12.67(12)}\right) = 0.043\ rad$

Because $e = 3.0' < \frac{1.6M_p}{F_y} = 3.52'$

$\alpha_{max} = 0.08\ rad > 0.043\ rad$ *OK*



Eccentrically Braced Frames
Rotation Angle

- Rotation angle limits based on link beam equivalent length
  - Short links yield in shear and are allowed greater rotation
- Rotation angle may be reduced in design by:
  - Increasing member size (reducing Δ_e)
  - Changing geometric configuration (especially changing length of link beam)

Eccentrically Braced Frames
Link Design

- Provide strength V and M per load combinations
- Check lateral bracing per AISC L_{pd}
- Local buckling (width to thickness of web and flange) per AISC Seismic
- Stiffeners (end and intermediate) per AISC Seismic

Eccentrically Braced Frames
Brace Design

$$\text{Strength} > 1.25R_y \cdot \begin{pmatrix} \text{axial force from design} \\ \text{shear strength of link} \end{pmatrix}$$

Eccentrically Braced Frames
Brace Design Example

Check axial strength of 15.26 ft long TS 8 x 8 x 5/8 F_y = 46 ksi:

$$\frac{KL}{r} = \frac{(1)(15.26)(12)}{2.99} = 61.2$$

$$61.2 < 4.71\sqrt{\frac{E}{F_y}} = 118.3 \qquad \therefore F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000)}{61.2^2} = 76.4\ ksi$$

$$F_{cr} = \left(0.658^{\frac{46}{76.4}}\right)46 = 35.8 ksi$$

$$\phi_c P_n = \phi_c A_g F_{cr} = 0.9(16.4)(35.8) = 528\ kip$$

Eccentrically Braced Frames
Brace Design Example

$$\phi V_n = 0.9(0.6F_y)d\,t_w = 0.9\left[0.6(50)(16.4)(0.43)\right] = 190\ kip$$

or

$$\phi V_n = 2(0.9)M_p / e = \frac{2(0.9)(50)(105)}{3(12)} = 262.5\ kip$$

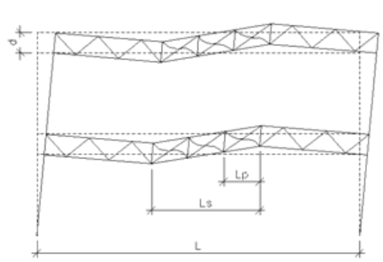
$$V_{e(link)} = 85.2\ kip \quad and \quad P_{e(brace)} = 120.2\ kip$$

$$\therefore P_u = 1.25(1.1)\left(\frac{190}{85.2}\right)(120.2) = 369 < 528 \quad OK$$

NEHRP Recommended Provisions
Steel Design

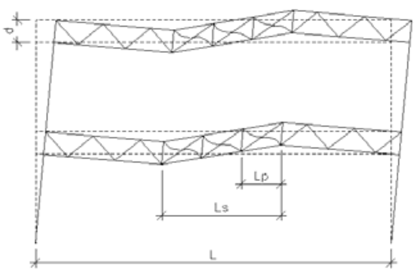
- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics

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:

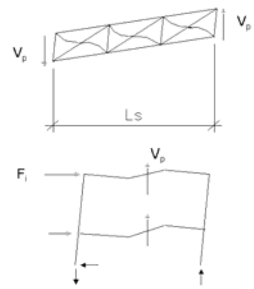
$L \le 65'$ $d \le 6'$

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

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

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

Special Truss Moment Frame

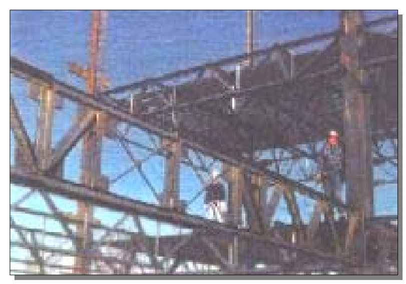


$$V_p = 2\left(\frac{2\,M_{pc}}{L_s}\right) + \sin\alpha\,\left(P_{nt} + 0.3P_{cd}\right)$$

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

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

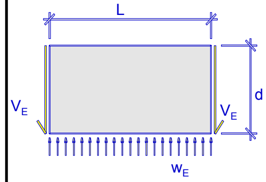
Steel Diaphragm
Example

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

$\phi = 0.6$

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

Steel Deck Diaphragm Example



$L = 80'$ $d = 40'$

$w_D = w_L = 0$ $w_E = 500$ plf

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

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

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

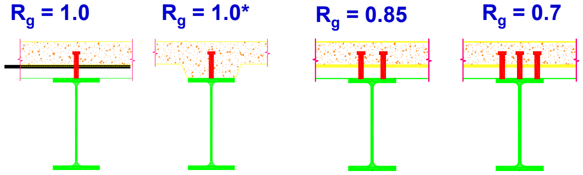
R_g = stud geometry adjustment factor
R_p = stud position adjustment factor

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.

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

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

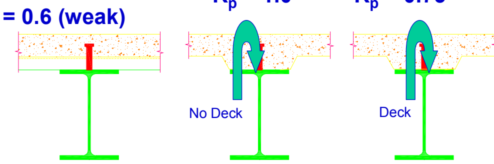


*0.85 if $w_r/h_r < 1.5$

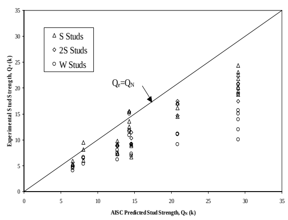
Shear Studs – Position Adjustment Factor

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

$R_p = 0.75$ (strong) $= 0.6$ (weak) $R_p = 1.0$ $R_p = 0.75$

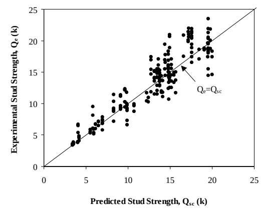


Shear Studs – Strength Calculation Model Comparison



AISC Seismic prior to 2005

Virginia Tech strength model



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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.

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Inspection and Testing
Inspection Requirements

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

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Inspection and Testing
Shop Certification

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

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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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NEHRP Recommended Provisions Steel Design

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

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