Steel Structures: Context in Provisions

Design basis: Strength limit state

Using the 2003 NEHRP Recommended Provisions:
- 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

Strain $\varepsilon_y < \varepsilon < \varepsilon_{sh}$

Stress $\sigma_y$
Cross-section Ductility
Conceptual moment - curvature

\[ M_y \phi \leq M_p \phi \]

Moment Curvature
Laboratory Test -- Annealed W Beam

Behavior Modes For Beams

OMI Elastic lateral tors. buckling
OHI Inelastic lateral tors. buckling
OJG Inelastic lateral tors. buckling
OJE Idealized behavior
OJK Strain hardening

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

Plastic
Allowable stress design

Plastic
Allowable stress design

Inelastic buckling
Elastic buckling
L laterally unbraced length
Local Buckling

Classical plate buckling solution:

\[ \sigma_{cr} = \frac{b y F_y}{12(1-\mu^2)(b/t)^2} \leq \sigma_y \]

Substituting \( \mu = 0.3 \) 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.

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

Beam Bottom Flange Weld Fracture Causing a Column Divot Fracture

Northridge Failure

- Crack through weld
- Note backup bar and runoff tab

Northridge Failure

- Column flange
- Beam flange and web
- Backup bar

Northridge Failures

- Weld
- Weld Fusion
- Column Divot
- Column Flange
- HAZ
- Lamellar Tear

Flexural Mechanics at a Joint

1. Cross Sections
2. Beam Moment

\[ F_w \cdot Z_w > F_y \cdot Z_y \]
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 Chiing Yeng, 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

Plastic hinge formation -- flange and web local buckling

Net section fracture in stem of T-stub

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

- Total Plastic Rotation (rad)
- Moment at Column Centerline (in-kips)

Graphic courtesy of Professor Thomas Murray, Virginia Tech

Ductility of Steel Frame Joints

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

H

Δ

“Slapback” of cycle 2

Eccentrically Braced Frame

Lab test of link

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

<table>
<thead>
<tr>
<th>Frame</th>
<th>Test</th>
<th></th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special</td>
<td>Req’d</td>
<td>0.04</td>
<td>Many</td>
</tr>
<tr>
<td>Intermediate</td>
<td>Req’d</td>
<td>0.02</td>
<td>Moderate</td>
</tr>
<tr>
<td>Ordinary</td>
<td>Allowed</td>
<td>NA</td>
<td>Few</td>
</tr>
</tbody>
</table>
Steel Moment Frame Joints

\[ M_a = M_j \cdot \frac{a + b}{b} \]

\[ F'_y = F_y - F' \]

\[ F = F'_y \cdot \frac{b + b}{A_d} = 1.7F'_y \]

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

Steel Moment Frames

- Beam shear: \( 1.1R_y M_p + gravity \)
- 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

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\( r_y E / F_y \)

Prequalified Connections

See FEMA 350: Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings
- Welded Unreinforced Flange - Bolted Unstiffened End Plate Connection
- Welded Flat Connection - Bolted Stiffened End Plate Connection
- Welded Flange Plate Connection - Bolted Flange Plate Connection
- Reduced Beam Section Connections

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

Welded Coverplates
Reduced Beam Section (RBS)

Extended End Plate

Excellent Moment Frame Behavior

Excellent Moment Frame Behavior

Excellent Moment Frame Behavior

Special Moment Frames Example
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

SMF Example – Preliminary Member Sizes

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 – Check Member Local Stability

Check beam flange: \( \frac{b_t}{2t} = 6.01 \)

(W33x141 A992)

Upper limit: \( 0.3 \frac{E}{F_y} = 7.22 \text{OK} \)

Check beam web: \( \frac{h_t}{t_w} = 49.6 \)

Upper limit: \( 3.76 \sqrt[3]{\frac{E}{F_y}} = 90.6 \text{ OK} \)

Therefore, there is no vertical irregularity.

SMF Example – Check Deflection and Drift

The frame was checked for an allowable story drift limit of 0.020\( h_{syst} \). 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:

\[
\frac{C_t A_t}{C_p A_p} < \frac{5.17 \text{in.}}{268 \text{in.}} = 0.98 < 1.3
\]

Therefore, there is no vertical irregularity.

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 \( \frac{\delta_{max}}{\delta_{avg}} \) 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 \( \frac{\delta_{max}}{\delta_{avg}} \) ratios exceed 1.2; therefore, there is no torsional amplification.
SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_e / \phi P_n$ is typically less than 0.4 for the columns, combinations involving $Q_{\theta}$ 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

$$\frac{\sum M_{\theta}}{\sum M_{pl}} > 1.0$$

where $\sum M_{\theta}$ is the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\sum M_{pl}$ 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.

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.

$$R_e = 0.6F\phi \left[ \frac{3b_t}{4d_t} \right] \left( 0.6 \times 50 \times 17.92 \times 1 \right) \left( 0.1645 \times 2.66 \right) \left( 0.32 \times 1.792 \times 1 \right)$$

$$R_e = 537.6 \times 315$$

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

$$R_e = \frac{R_i}{(1.0)(537.6 \times 315)} = 1.883\text{kips}$$

$$t = 2.91\text{in.}$$

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

$$t_{doub} = 1.25\text{in.}$$

(following use one 1.25 in. plate or two 0.625 in. 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 0.85$M_m$ 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 $E$:

\[ E = 2[1.1R_m]L_0 \]  

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

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

Bracing members:
- Compression capacity = φ_n P_n
- Width / thickness limits
  Generally compact
  Angles, tubes and pipes very compact
- Overall \( \frac{K_1}{F} < 4 \sqrt{\frac{F}{P_n}} \)
- Balanced tension and compression

Brace connections
Axial tensile strength > smallest of:
- Axial tension strength = \( R_y F A_g \)
- Maximum load effect that can be transmitted to brace by system.
Axial compressive strength \( \geq 1.1 R_y P_n \) 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 \( A_y F_{CR} \)

V bracing:
- Design beam for \( D + L + \) unbalanced brace forces, using \( 0.3 \phi P_c \) for compression and \( R_y F A_g \) 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

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 Frames

**Steel Structures 10 - 91**

**Instructional Material Complementing FEMA 451, Design Examples**

Eccentrically Braced Frames

- Eccentric bracing systems
- Building frame system or part of dual system w/ special moment frame

With moment resisting connections
- 8
- 4

Without moment resisting connections
- 7
- 4

These connections determine classification

---

Eccentrically Braced Frames

**Design Procedure**

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

---

Eccentrically Braced Frames

**Example**

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)

---

Eccentrically Braced Frames

**Rotation Angle Example**

From computer analysis:
- $\Delta_u = 0.247 \text{ in}$
- Total drift:
  - $\Delta = C_d \Delta_u = 4(0.247) = 0.99 \text{ in}$.

From geometry:
- $\alpha = \left( \frac{L}{E} \right) = \left( \frac{20}{3} \right) \left( \frac{0.99}{12.87(12)} \right) = 0.043 \text{ rad}$

Because $e = 3.0' < 1.6M_pL_e$ = 3.52'
- $\alpha_{ex} = 0.08 \text{ rad} > 0.043 \text{ rad}$ 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 $\Delta_u$)
  - 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 Lpd
- Local buckling (width to thickness of web and flange) per AISC Seismic
- Stiffeners (end and intermediate) per AISC Seismic

Eccentrically Braced Frames

Brace Design

Strength > 1.25Ry (axial force from design)

Eccentrically Braced Frames

Brace Design Example

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

\[
\frac{Kl}{F_i} = \frac{(1)(15.26)(12)}{2.99} = 61.2
\]

\[
61.2 < 4.71 \frac{E}{V_{F_i}} = 118.3 \quad \therefore F_i = \left(0.658\right)F_y
\]

\[
F_y = \frac{\pi^2 E}{KL} = \frac{61.2}{61.2} = 76.4 \text{ kpsi}
\]

\[
F_p = \left(0.658\right)46 = 35.8 \text{ kpsi}
\]

\[
\delta P = \delta A_f F_p = 0.9(16.4)(35.8) = 528 \text{ kip}
\]

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 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:
\[ L \leq 60^\circ \quad d \leq 60^\circ \]
\[ 0.1 \leq \frac{L}{L} < 0.5 \]
\[ \frac{2}{3} \leq \frac{L}{d} \leq \frac{3}{2} \]
Flat bar diagonals \( \frac{d}{t} \leq 2.5 \)

\[ V_d = 2 \left( \frac{2 M_{pl}}{L} \right) + \sin \alpha \left( P_{vl} + 0.3 P_{vl} \right) \]
\[ \sum F_t h_t = \sum V_t L \]

General Seismic Detailing

Materials:
- Limit to lower strengths and higher ductilities

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

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 \( \Omega_{vc} \) if \( P_v / \phi_P > 0.4 \)
- Splices: Requirements on partial pen welds and fillet welds
Steel Diaphragm Example

\[ \phi V_n = \phi (\text{approved strength}) \]

\[ \phi = 0.6 \]

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

Steel Deck Diaphragm Example

\[ L = 80' \quad d = 40' \]

\[ w_o = w_c = 0 \quad w_d = 500 \text{ pfl} \]

\[ V_c = \frac{w_d}{2} = 20 \text{ kip} \quad v = \frac{20000}{40} = 500 \text{ pfl} \]

\[ V_{sd} = \frac{V_c}{20} = 417 \text{ pfl} \]

Deck chosen:
1½”, 22 gage with welds on 36/5 pattern and 3 sidelap fasteners, spanning 5'-0'
Capacity = 450 > 417 pfl

Welded Shear Studs

Shear Stud Strength - AISC 2005 Specification

\[ Q_n = 0.5 A_{sc} \left( f_{c}^* E_c \right)^{1/2} \leq R_g R_p A_{sc} F_u \]

\[ R_g = \text{stud geometry adjustment factor} \]

\[ R_p = \text{stud position adjustment factor} \]

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

\[ Q_n = 0.5 A_{sc} \left( f_{c}^* E_c \right)^{1/2} \leq R_g R_p A_{sc} F_u \]

\[ R_g = \text{stud group adjustment factor} \]

\[ R_g = 1.0 \]

\[ R_g = 1.0^* \]

\[ R_g = 0.85 \]

\[ R_g = 0.7 \]

\*0.85 if \( w_{d}/w_{c} < 1.5 \)

Shear Studs – Position Adjustment Factor

\[ Q_n = 0.5 A_{sc} \left( f_{c}^* E_c \right)^{1/2} \leq R_g R_p A_{sc} F_u \]

\[ R_p = \text{stud position adjustment factor} \]

\[ R_p = 0.75 \text{ (strong)} \]

\[ R_p = 0.75 \text{ (weak)} \]
Shear Studs – Strength Calculation Model Comparison

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
- Summary