Composite Steel and Concrete

Clinton O. Rex, P.E., PhD

Originally developed by
James Robert Harris, P.E., PhD, and Frederick R. Rutz, P.E., PhD
Introduction

Outline For The Presentation

• Background and Overview
• Example Building
• PR Composite Connection Design
• Loads and Load Combinations
• Analysis
• ASCE 7-10 and AISC Design Checks
• Questions
Background & Overview

Composite Steel & Concrete Systems in AISC 341-16

- Composite Ordinary, Intermediate, and Special Moment Frame
- Composite Partially Restrained Moment Frame
- Composite Ordinary and Special Concentrically Braced Frame
- Composite Eccentrically Braced Frame
- Composite Ordinary and Special Shear Walls
- Composite Plate Shear Walls
Background & Overview

Composite Partially Restrained Moment Frame (C-PRMF)

• Typically Consists Of:
  • Standard W-Shape Steel Columns
  • Composite W-Shape Steel Beams
  • Partially Restrained Composite Connections (PRCC)
Background & Overview

**Different From A Typical Moment Frame**

- The PRCC is what makes the difference
  - Not as strong (Partial Strength Vs. Full Strength)-Typically designed as partial strength
  - Not as stiff (Angle between beam end and column does not stay at right angle)
  - Can’t neglect the connection behavior through simplifying assumptions in the analysis

![Diagram of Moment-rotation behavior](image-url)
Background & Overview

Different From A Typical Moment Frame

• The connections become the seismic fuses for seismic energy dissipation (need ductile connections)
• Limited yielding of beams and columns
• Required to engage more frames because of the reduced stiffness and strength
• Highly redundant lateral force resisting system
• Typically simple fabrication and erection details are used allowing quicker fabrication and erection
• Cannot delegate the design to the fabricator
• PRCC connections are not fully effective until the concrete hardens – similar to waiting for moment connections to get welded
### Background & Overview

#### ASCE 7-16 Provisions

Table 12.2-1

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section Where Detailing Requirements Are Specified</th>
<th>Response Modification Coefficient, R&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Overstrength Factor, Ω&lt;sub&gt;ds&lt;/sub&gt;</th>
<th>Deflection Amplification Factor, C&lt;sub&gt;d&lt;/sub&gt;&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Structural System Limitations Including Structural Height, h&lt;sub&gt;s&lt;/sub&gt; (ft) Limits&lt;sup&gt;c&lt;/sup&gt;</th>
<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>C. MOMENT-RESISTING FRAME SYSTEMS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Steel special moment frames</td>
<td>14.1 and 12.2.5.5</td>
<td>8</td>
<td>3</td>
<td>5½</td>
<td>NL</td>
<td>NL</td>
</tr>
<tr>
<td>2. Steel special truss moment frames</td>
<td>14.1</td>
<td>7</td>
<td>3</td>
<td>5½</td>
<td>NL</td>
<td>NL 160 100 NP</td>
</tr>
<tr>
<td>3. Steel intermediate moment frames</td>
<td>12.2.5.7 and 14.1</td>
<td>4½</td>
<td>3</td>
<td>4</td>
<td>NL</td>
<td>NL 35&lt;sup&gt;b&lt;/sup&gt; NP&lt;sup&gt;b&lt;/sup&gt; NP&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>4. Steel ordinary moment frames</td>
<td>12.2.5.6 and 14.1</td>
<td>3½</td>
<td>3</td>
<td>3</td>
<td>NL</td>
<td>NL NP&lt;sup&gt;b&lt;/sup&gt; NP&lt;sup&gt;b&lt;/sup&gt; NP&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>5. Special reinforced concrete moment frames&lt;sup&gt;a&lt;/sup&gt;</td>
<td>12.2.5.5 and 14.2</td>
<td>8</td>
<td>3</td>
<td>5½</td>
<td>NL</td>
<td>NL NP&lt;sup&gt;b&lt;/sup&gt; NP&lt;sup&gt;b&lt;/sup&gt; NP&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>6. Intermediate reinforced concrete moment frames</td>
<td>14.2</td>
<td>5</td>
<td>3</td>
<td>4½</td>
<td>NL</td>
<td>NL NP NP NP</td>
</tr>
<tr>
<td>7. Ordinary reinforced concrete moment frames</td>
<td>14.2</td>
<td>3</td>
<td>3</td>
<td>2½</td>
<td>NL</td>
<td>NP NP NP NP</td>
</tr>
<tr>
<td>8. Steel and concrete composite special moment frames</td>
<td>12.2.5.5 and 14.3</td>
<td>8</td>
<td>3</td>
<td>5½</td>
<td>NL</td>
<td>NL NP NP NP</td>
</tr>
<tr>
<td>9. Steel and concrete composite intermediate moment frames</td>
<td>14.3</td>
<td>5</td>
<td>3</td>
<td>4½</td>
<td>NL</td>
<td>NL NP NP NP</td>
</tr>
<tr>
<td>10. Steel and concrete composite partially restrained moment frames</td>
<td>14.3</td>
<td>6</td>
<td>3</td>
<td>5½</td>
<td>160</td>
<td>160 100 NP NP</td>
</tr>
</tbody>
</table>
Background & Overview

**ASCE 7-16 Provisions**

14.3 – Composite Steel and Concrete Structures

14.3.1 Reference Documents

The design, construction, and quality of composite steel and concrete members that resist seismic forces shall conform to the applicable requirements of the following:

1. AISC 341
2. AISC 360
3. ACI 318, excluding Chapter 22

14.3.2 General

Systems of structural steel acting compositely with reinforced concrete shall be designed in accordance with AISC 360 and ACI 318, excluding Chapter 22. Where required, the seismic design of composite steel and concrete systems shall be in accordance with the additional provisions of Section 14.3.3.
14.3 – Composite Steel and Concrete Structures

14.3.3 Seismic Requirements for Composite Steel and Concrete Structures

Where a response modification coefficient, R, in accordance with Table 12.2-1 is used for the design of systems of structural steel acting compositely with reinforced concrete, the structures shall be designed and detailed in accordance with the requirements of AISC 341.
Section G4. COMPOSITE PARTIALLY RESTRAINED (PR) MOMENT FRAMES (C-PRMF)

G4.1. Scope

This Section is applicable to frames that consist of structural steel columns and composite beams that are connected with partially restrained (PR) moment connections that meet the requirements in Specification Section B3.4b(b). Composite partially restrained moment frames (C-PRMF) shall be designed so that under earthquake loading yielding occurs in the ductile components of the composite PR beam-to-column moment connections. Limited yielding is permitted at other locations, such as column base connections. Connection flexibility and composite beam action shall be accounted for in determining the dynamic characteristics, strength and drift of C-PRMF.
Background & Overview

AISC 341-16 Provisions

Section G4

G4.5a. Columns
Steel columns shall satisfy the requirements of Sections D1.1 for moderately ductile members.

G4.5b. Beams
Composite beams shall be unencased, fully composite and shall meet the requirements of Section D1.1. A solid slab shall be provided for a distance of 12 in. (300 mm) from the face of the column in the direction of moment transfer.

G4.6. Connections
The required strength of the beam-to-column PR moment connections shall be determined considering the effects of connection flexibility and second-order moments. In addition, composite connections shall have a nominal strength that is at least equal to 50 percent of $M_p$, where $M_p$ is the nominal plastic flexural strength of the connected structural steel beam ignoring composite action. Connections shall meet the requirements of Chapter I and shall have a total interstory drift angle of 0.02 radians that is substantiated by cyclic testing as described in Section G4.6d.
Other Design Guidance

- AISC Design Guide 8 – 1996

DESIGN GUIDE FOR PARTIALLY RESTRAINED COMPOSITE CONNECTIONS

By the ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete

Instructional Material Complementing FEMA P-1051, Design Examples
Example Building

Building Plan
Example Building

**Building Elevations**

**East and West Side Elevation**

**Typical North Elevation**
Example Building

Building 3-D SAP 2000 Model
Typical W18x35 PRCC

- Standard double angle shear connection w/ A36 2L4x4x1/4 w/ 4-3/4" dia. A325-N-SC bolts
- One row 3/4" dia. studs spaced @ 6"
- Place transverse reinforc. below top of studs, See 9.2-3 for number and layout
- 8 - #4 Long bars see 9.2-3 for layout
- A36 L8x6x1/2x0'-9" (SLV) w/ 4-1" dia. A490-X-SC bolts in horiz. leg & 2-1" dia. A490-X-SC bolts in vert. leg
- OVS holes in column at seat angle

Centerline beam
Centerline connection

Instructional Material Complementing FEMA P-1051, Design Examples
Composite Steel R/C - 16
PR Composite Connection Design

Connection Moment-Rotation Curves

Two Curves Required:

**Service** Analysis Curves From ASCE TC Paper

(ASCE TC, Eq. 4) Negative moment-rotation behavior (slab in tension):

$$M_c^- = C_1 \left( 1 - e^{-C_2 \theta} \right) + C_3 \theta$$

(ASCE TC, Eq. 3) Positive moment-rotation behavior (slab in compression):

$$M_c^+ = C_1 \left( 1 - e^{-C_2 \theta} \right) + (C_3 + C_4) \theta$$
Connection Moment-Rotation Curves

Two Curves Required:

**Strength** Analysis Curves Based on Modifications of the Service Analysis Curves to Account for Direct Analysis Approach to Design

Elastic Stiffness Reduction:

\[
\theta_{cDA}\text{M} = \theta_c + \frac{M_c}{4 \times K_{ci}}
\]

\[
K_{ci} = \frac{M_{c@2.5\ mrad}}{2.5\ mrad}
\]

Strength Reduction:

\[
M_{cDA} = 0.85\ M_c
\]
PR Composite Connection Design

Connection Moment-Rotation Curves

W18x35 PRCC Moment Rotation Curves
### PR Composite Connection Design

**Connection Moment-Rotation Curves**

**W18x35 PRCC Key Connection Curve Values**

<table>
<thead>
<tr>
<th></th>
<th>W18x35 PRCC</th>
<th>W21x44 PRCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{ci}^-$ (kip-in/rad) (nominal)</td>
<td>704,497</td>
<td>1,115,253</td>
</tr>
<tr>
<td>$K_{ci}^+$ (kip-in/rad) (nominal)</td>
<td>338,910</td>
<td>554,498</td>
</tr>
<tr>
<td>$M_c^- @ 20$ mrad (kip-ft) (nominal/modified)</td>
<td>232/206</td>
<td>367/326</td>
</tr>
<tr>
<td>$M_c^+ @ 10$ mrad (kip-ft) (nominal/modified)</td>
<td>151/127</td>
<td>240/202</td>
</tr>
</tbody>
</table>
PR Composite Connection Design

Longitudinal Reinforcing Steel

- 4-#4 longitudinal
- 4-#4 cont. longitudinal lap w/ 4#5 @ end bay
- 4-#5 longitudinal
- 4-#5 cont. longitudinal
- 8-#4 transverse
- #4x4'-0" @ 24" service
PR Composite Connection Design

Longitudinal Reinforcing Steel

- Minimum 6 Bars Distributed Evenly Over An Effective Width of 7 Column Flanges
- Choose Sufficient Reinforcing To Reach Desired Strength Goals
- Limit Reinforcing To Ensure Ductile Failure of Reinforcing Before Other Possible Non-Ductile Connection Failures
- Use #6 or Smaller Bar Size
- Distribute As Close to Equal on Each Side of Column Center Line As Possible
  - Minimum Of 33% of Reinforcing On One Side Of Column
  - Minimum Of 3 Bars On One Side Of Column
- All Bars Should Extend ¼ Of The Beam Span Or 24 Bar Diameters Past the Inflection Point
- For Seismic Design – Detail 50% of Bars As Continuous
- Continuous bars should be spliced with Class B lap per ACI 318
- Minimum cover per ACI 318
PR Composite Connection Design

Transverse Reinforcing Steel

4-#5 longitudinal

4-#4 longitudinal

4-#5 cont. longitudinal

4-#4 cont. longitudinal lap w/ 4#5 @ end bay

#4x4'-0" @ 24" service

4-#5 transverse

8-#4 transverse
Transverse Reinforcing Steel

- Provided to promote the force transfer from longitudinal steel to column and to beam flange.
- Allows development of strut and tie fields for force transfer to column – basis for selection of reinforcing.
- Prevents longitudinal splitting over beam to allow force transfer to studs then to beam flange – reason to keep transverse steel below top of shear studs.
- Use #6 or smaller bars.
- Typically same number as longitudinal steel.
- Extend 12 bar diameters or 12” past outside longitudinal bars.
PR Composite Connection Design

Concrete to Column Force Transfer

- Transfer Slab Compression Force From One Side Of Connection Combined With Slab Tension Force From Other Side Of Connection Through Bearing Of Concrete On Column Flanges
  - Typically Taken As The Combination Of $M_c^-$ And $M_c^+$ Of The Connection
- Limit Bearing Stress to $1.8 f'_c$
- Check Flange Local Bending
- Check Web Local Yielding
- Concrete Between Flanges
- Provide Sufficient Studs To Develop Slab Forces Within Inflection Points Each Side Of Column
PR Composite Connection Design

Connection Moment Capacity Limits

- AISC 341 Section G4 Requires A Minimum Nominal Connection Capacity Of 50% Of Nominal Bare Steel Beam, $M_p$
- ASCE TC Recommends 75% As The Target With 50% Lower Limit And 100% Upper Limits
- Determine Nominal Connection Strengths At Target Rotations:
  - 20 mrad For Negative Connection Moment Capacity
  - 10 mrad For Positive Connection Moment Capacity
- Not Clear If This Requirement Applies To Both Negative And Positive Capacity When A Connection Has Different Capacity In Each Direction – Commentary of AISC 341 – 16 Suggests That Limit Is Applicable To Both Directions
- Example:
  - W21x44 Negative Ratio 0.92
  - W21x44 Positive Ratio 0.60
PR Composite Connection Design

Seat Angle Design

• Minimum Width Of 9” To Allow Typical Bolt Gage Of 5.5” and 1.75”
Minimum Edge Distance For Sheared Edges At 1” Bolts

• Minimum Horizontal Angle Leg Area:
  - \( A_{samin} = 1.33 \times F_{yrd} \times A_{rb} / F_{ya} \)

• Minimum Horizontal Leg Length Of 8” To Allow 4 – 1” Diameter Bolts

• Vertical Leg Length Chosen To Allow Ductile Hinging Of The Seat Angle
  When Connection Is In Full Positive Bending

• Limit \( t_{sa} / b' \) To < 0.5 To Allow
Development Of Proper Hinges And
To Ignore Shear Bending Interaction
Seat Angle Bolts

- Bolts In Vertical Leg:
  - Oversize Holes For Mill Tolerance
  - Bolts In Vertical Leg Should Be Designed For Maximum Bolt Tension Including Prying Action (See Paper) Magnified By $R_y$ From AISC 341 Table I-6-1.
    - $R_y \times B_{sa}$
- Bolts In Horizontal Leg:
  - Generally This Is The Limiting Factor In The PRCC Design
  - Maximize Bolt Capacity By Using A490-X Bolts
  - 1” Diameter Is A Practical Upper Limit On Bolt Size For The Typical Angle Size and Connected Beams
  - Check Against Full Plastic Capacity Of Longitudinal Steel And Web Angles Magnified By Appropriate $R_y$ Factors
    - $R_y \times T_{wa} + R_y \times F_{yrd} \times A_{rb}$
PR Composite Connection Design

Double Angle Web Connection

- Primary Purpose To Carry The Connection Shear Demand
- Seismic Shear Demand Associated With Full Connection Strength At Each End Should Be Added To Gravity Shear Demand With Appropriate Load Factors To Determine Total Shear Demand
- Detail Gage With Same Ductility Concept As Seat Angle Of $t_{wa} / b'$ To $<$ 0.5 To Allow Web Angles To Hinge As The Connection Rotates
### Loads And Load Combinations

**Gravity Loads and Seismic Weight**

<table>
<thead>
<tr>
<th>Description</th>
<th>Non-Composite Dead Loads ($D_{nc}$)</th>
<th>Composite Dead Loads ($D_{c}$)</th>
<th>Live Loads ($L$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.5-in. Slab on 0.6-in. Form Deck (4.5-in. total thickness) plus Concrete Ponding</td>
<td>Fire Insulation</td>
<td>Typical Area Live and Partitions (Reducible)</td>
</tr>
<tr>
<td></td>
<td>58 psf 58 psf</td>
<td>4 psf 4 psf</td>
<td>70 psf 10 psf</td>
</tr>
<tr>
<td></td>
<td>6 psf 6 psf</td>
<td>Mechanical and Electrical</td>
<td>Records Storage Area Live (Non-Reducible)</td>
</tr>
<tr>
<td></td>
<td>2 psf 2 psf</td>
<td>Ceiling</td>
<td>200 psf 100 psf</td>
</tr>
<tr>
<td></td>
<td>Total: 66 psf 66 psf</td>
<td>Total: 12 psf 12 psf</td>
<td></td>
</tr>
<tr>
<td></td>
<td>800 plf 800 plf</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Seismic Loads

### Table 12.2-1

**Seismic Design Category**

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section Where Detailing Requirements Are Specified</th>
<th>Response Modification Coefficient, $R^a$</th>
<th>Overstrength Factor, $\Omega_0^a$</th>
<th>Deflection Amplification Factor, $C_d^a$</th>
<th>Structural System Limitations Including Structural Height, $h_s$ (ft) Limits(^b)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>C. MOMENT-RESISTING FRAME SYSTEMS</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Steel special moment frames</td>
<td>14.1 and 12.2.5.5</td>
<td>8</td>
<td>3</td>
<td>5½</td>
<td>NL NL NL NL NL</td>
</tr>
<tr>
<td>2. Steel special truss moment frames</td>
<td>14.1</td>
<td>7</td>
<td>3</td>
<td>5½</td>
<td>NL NL 160 100 NP</td>
</tr>
<tr>
<td>3. Steel intermediate moment frames</td>
<td>12.2.5.7 and 14.1</td>
<td>4½</td>
<td>3</td>
<td>4</td>
<td>NL NL 35 NP NP NP</td>
</tr>
<tr>
<td>4. Steel ordinary moment frames</td>
<td>12.2.5.6 and 14.1</td>
<td>3½</td>
<td>3</td>
<td>3</td>
<td>NL NL NP NP NP NP</td>
</tr>
<tr>
<td>5. Special reinforced concrete moment frames(^a)</td>
<td>12.2.5.5 and 14.2</td>
<td>8</td>
<td>3</td>
<td>5½</td>
<td>NL NL NL NL NL</td>
</tr>
<tr>
<td>6. Intermediate reinforced concrete moment frames</td>
<td>14.2</td>
<td>5</td>
<td>3</td>
<td>4½</td>
<td>NL NL NP NP NP</td>
</tr>
<tr>
<td>7. Ordinary reinforced concrete moment frames</td>
<td>14.2</td>
<td>3</td>
<td>3</td>
<td>2½</td>
<td>NL NP NP NP NP</td>
</tr>
<tr>
<td>8. Steel and concrete composite special moment frames</td>
<td>12.2.5.5 and 14.3</td>
<td>8</td>
<td>3</td>
<td>5½</td>
<td>NL NL NL NL NL</td>
</tr>
<tr>
<td>9. Steel and concrete composite intermediate moment frames</td>
<td>14.3</td>
<td>5</td>
<td>3</td>
<td>4½</td>
<td>NL NL NP NP NP</td>
</tr>
<tr>
<td>10. Steel and concrete composite partially restrained moment frames</td>
<td>14.3</td>
<td>6</td>
<td>3</td>
<td>5½</td>
<td>160 160 100 NP NP</td>
</tr>
</tbody>
</table>

---

Equivalent Lateral Force Procedure Permitted Per Table 12.6-1
Loads And Load Combinations

**Seismic Loads – ELF Building Period Determination**

- Approximate Period – Per Equation 12.8-7 & Table 12.8-2
- \( T_a = 0.66 \text{ sec} \)

\[
T_a = C_t h_n^x
\]

### Table 12.8-2 Values of Approximate Period Parameters \( C_t \) and \( x \)

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>( C_t )</th>
<th>( x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel moment-resisting frames</td>
<td>0.028 (0.0724)²</td>
<td>0.8</td>
</tr>
<tr>
<td>Concrete moment-resisting frames</td>
<td>0.016 (0.0466)²</td>
<td>0.9</td>
</tr>
<tr>
<td>Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1</td>
<td>0.03 (0.0731)²</td>
<td>0.75</td>
</tr>
<tr>
<td>Steel buckling-restrained braced frames</td>
<td>0.03 (0.0731)²</td>
<td>0.75</td>
</tr>
<tr>
<td>All other structural systems</td>
<td>0.02 (0.0488)²</td>
<td>0.75</td>
</tr>
</tbody>
</table>
Seismic Loads – ELF Building Period Determination

- Upper Limit Period – Per Table 12.8-1
- $S_{D1} = 0.14$ so $C_u = 1.62$
- $T_{max} = 0.66 \times 1.62 = 1.07$ sec

### Table 12.8-1 Coefficient for Upper Limit on Calculated Period

<table>
<thead>
<tr>
<th>Design Spectral Response Acceleration Parameter at 1 s, $S_{D1}$</th>
<th>Coefficient $C_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\geq 0.4$</td>
<td>1.4</td>
</tr>
<tr>
<td>0.3</td>
<td>1.4</td>
</tr>
<tr>
<td>0.2</td>
<td>1.5</td>
</tr>
<tr>
<td>0.15</td>
<td>1.6</td>
</tr>
<tr>
<td>$\leq 0.1$</td>
<td>1.7</td>
</tr>
</tbody>
</table>
Loads And Load Combinations

Seismic Loads – ELF Building Period Determination

- Dynamic Analysis Period From 3-D SAP Model
- Use $K_{ci}$ For Connection Stiffness To Estimate Shortest Possible Analytical Building Period
- $T_{\text{dynamic}} = 2.13$ sec North-South And 1.95 sec East-West
- Summary of Periods:
  - $T_a = 0.66$ sec
  - $T_{\text{max}} = 1.07$ sec
  - $T_{\text{dynamic}} = 2.13$ sec North-South And 1.95 sec East-West
  - Use $T = 1.07$ sec To Determine Strength Level Forces
  - Could Use $T_{\text{dynamic}}$ For Drift Check Forces If Need Be
Loads And Load Combinations

Notional Loads

- AISC 360 Requires The Application Of Notional Loads To the Gravity-Only Load Combination For This Example Building
- Notional Loads Are Taken As 0.2% Of Gravity Loads
- Example Building
  \[ ND_{nc} = 4,258 \text{ kips} \times 0.002 = 8.516 \text{ kips} / 4 \text{ floors} = 2.13 \text{ kips/floor} \]
  \[ ND_c = 2,393 \text{ kips} \times 0.002 = 4.786 \text{ kips} / 4 \text{ floors} = 1.20 \text{ kips/floor} \]
  \[ NL = 4,469 \text{ kips} \times 0.002 = 8.938 \text{ kips} / 4 \text{ floors} = 2.23 \text{ kips/floor} \]
- Because The Center Of Building Loading Corresponds To The Centroid Of The Building For This Example, These Notional Loads Can Be Applied At Building Centroid
  - Typically Notional Loads Are Determined On A Column By Column Basis Thus Capturing The Actual Gravity Load Distribution
Loads And Load Combinations

Load Combinations

- Basic Load Combinations Considered From Section 2.3.2:
  
  Load Combination 2: \( 1.2D + 1.6L \)
  
  Load Combination 5: \( 1.2D + 0.5L + 1.0E \)
  
  Load Combination 7: \( 0.9D + 1.0E \)

- Expand For Vertical And Horizontal EQ Effects, Breakout Non-Composite And Composite Dead Loads, And Add Notional Loads

  Load Combination 2: \( 1.2(D_{nc} + ND_{nc}) + 1.2(D_c + ND_c) + 1.6(L + NL) \)
  
  Load Combination 5: \( 1.2D_{nc} + 1.2D_c + 0.5L + 1.0E_h + 1.0E_v \)
  \[
  E_v = 0.2S_{DS} (D_{nc} + D_c) = 0.2(0.33)(D_{nc} + D_c) = 0.067(D_{nc} + D_c) \\
  1.267D_{nc} + 1.267D_c + 0.5L + 1.0E_h
  \]
  
  Load Combination 7: \( 0.9D_{nc} + 0.9D_c + 1.0E_h - 1.0E_v \)
  \[
  0.833D_{nc} + 0.833D_c + 1.0E_h
  \]
Loads And Load Combinations

Load Combinations

- Two-Stage Connection Behavior Requires Two-Stage Building Analysis And Thus Different Load Combinations For Each Phase And Separate Applications Of Dead Loads To Beams And Columns

Stage 1 Analysis:

Load Combinations 2 and 5: 1.2 $D_{ncb}$

Load Combination 7: 0.9 $D_{ncb}$

Stage 2 Analysis:

Load Combination 2: 1.2($D_{ncc} + ND_{nc}$) + 1.2($D_{c} + ND_{c}$) + 1.6($L + NL$)

Load Combination 5: 1.2$D_{ncc}$ + 0.067$D_{ncb}$ + 1.267$D_{c}$ + 0.5$L$ + 1.0$E_{h}$

Load Combination 7: 0.9$D_{ncc}$ - 0.067$D_{ncb}$ + 0.833$D_{c}$ + 1.0$E_{h}$

- Column Design From Stage 2
- Beam Design From Linear Combination Of Stage 1 & 2
Loads And Load Combinations

Load Combinations

• Seismic And Wind Drift Checks Based On Non-Linear Analysis Thus You Need Full Load Combinations In Order To Capture Connection Behavior And Building Second Order Effects:

  Seismic Drift: \(1.0D_{nc} + 0.067D_{ncb} + 1.0D_c + 0.5L + 1.0E_h\)

  Wind Drift: \(1.0D_{nc} + 1.0D_c + 0.5L + 1.0W\)

• All Typical Permeations Of Above Load Combinations For North-South And East-West Directions Have To Be Generated
Analysis

Preliminary Design

• Design All Framing For Pure Gravity Loading Assuming All PR Connections As Pins
• PR Frame Beams To Be Designed As 100% Composite
• Filler Beams Designed As Typical
• Perform 1st Order Lateral Analysis Assuming All PR Connections As Rigid
  • Review Beam End Moments From EQ And Wind And Compare To Previously Determined Connection Design Capacity (Would Like To Be Below About 75% Of Connection Design Capacity)
  • Review Building Drift Assuming PR Building Drift Will Be Approximately 2 x Above Model Drift
• If Fail:
  • Increase Number Of Frames
  • Increase Column / Beam Sizes (& Thus Connections)
Analysis

Application of Load

- Pre-Composite PR Connection Behavior Assumed As Pinned
  - Stage 1 Analysis Applies Pre-Composite Gravity Loads To Beams Only
- Post-Composite PR Connection Behavior Based On Previously Determined Curves
  - Stage 2 Analysis Applies Pre-Composite Gravity Loads To Columns Only And Post-Composite Gravity Loads To Beams (With Exception Of Seismic Vertical Loading Associated With Pre-Composite Loads)
Analysis

Application of Load

- Stage 2 Load Combination 5

1.2 $D_{nc}$ + 1.267 $D_c$ + 0.5 $L$

1.0 $E_h$ + 0.067 $D_{ncb}$
Analysis

Application of Load

- Stage 2 Load Combination 7

\[ 0.9 \, D_{nc} \quad + \quad 0.833 \, D_c \quad + \quad 1.0 \, E_h \]

\[ - \quad 0.067 \, D_{ncb} \]
Analysis

Beam & Column Moment of Inertia

- Beam Moment Of Inertia
  - \( I_{eq} = 0.6I_{LB+} + 0.4I_{LB-} \) For Drift
    - \( I_{LB} \) = Lower Bound Moment Of Inertia Of Composite Beam
    - \( I_{LB+} \) Positive Bending – From AISC Manual
    - \( I_{LB-} \) Negative Bending – Typical Assume Bare Steel
    - 0.8 x \( I_{eq} \) For Strength (Direct Analysis)
- Column Moment Of Inertia
  - \( I_{col} \) For Drift
  - 0.8 x \( t_b \) x \( I_{col} \) For Strength (Direct Analysis)
    - \( t_b \) Only Applies If \( P_r/P_y > 0.5 \)
Analysis

Connection Behavior Modeling

- For Each Connection There Are 4 Connection Models
  - Linear Spring Using $K_{ci}$ For Dynamic Analysis For Determining Building Period
  - Non-Linear Spring Using Nominal Connection Behavior Curve For Service Level Analysis
  - Pin For Stage 1 Gravity Analysis Of Beams
  - Non-Linear Spring Using Reduced Connection Behavior Curve For Stage 2 Strength Level Analysis
- SAP 2000 Allows 2 Connection Behavior Definitions – One For Linear Analysis And One For Non-Linear Analysis When Using The Multi-Linear Elastic Link Option
- Conduct A Path Independent Analysis Where Connection Behavior Always Remains On Connection Curve
Analysis

Connection Behavior Modeling

- Path Independent Vs. Path Dependent

- Reason For Not Reducing Beam Sizes
ASCE 7-16 & AISC Design Checks

*Building Drift and P-Δ Checks*

- Use Service Moment-Rotation Curves and Full Member Moment of Inertia
- Nonlinear Analysis Required Because of Connection Curves (Requires Non-Linear Load Combinations)
- Check Wind and Seismic Drift Using P-Δ Analysis
  - Wind Drift: $h_{sx}/400$ Limit Applied
  - Torsional Irregularity Check ASCE 7-16 Table 12.3-1
  - Seismic Drift Check ASCE 7-16 Table 12.12-1
- Check P-Δ Effect and Story Stability Coefficient ($q$) By Comparing and Contrasting Analysis Results With and Without P-Δ.
  - Check For P-Δ Effect to Be Less Than 1.5 to Meet AISC 360 Requirement For Using Notional Loads As Minimum Lateral Load (Factored Load Check)
  - Check For $\theta$ per Section 12.8.7 of ASCE 7-10 (No Load Factor > 1.0)

\[
\theta = 1 - \frac{1}{P\Delta_{amp}} \\
\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25
\]
ASCE 7-16 & AISC Design Checks

Beam Design

• Design As Composite Beam For 100% Composite Action
• Shear Stud Distribution Needs To Allow For Development of Slab Reinforcing Between Column and Inflection Point But Does Not Need to Develop 100% Composite Action Between These Points
• AISC 341 Section G4 Does Not Specifically Address Beam Compactness Criteria – Suggest Using AISC 360 Compact Criteria (Note AISC 341-16 Will Require Seismic Compact Criteria)
ASCE 7-16 & AISC Design Checks

Beam Design

Because Beam Is Now Part Of Lateral System, It May Go Into Negative Moment At One End Resulting In Possibly Considering The Entire Beam As Un-braced For Lateral-Torsional Buckling Checks – Consider Using Alternative $C_b$ Equations For Special Beam Cases (Yura, Helwig – Beam Buckling and Bracing)

$C_b$ - TOP FLANGE BRACED

1. If neither end moment cause compression on the bottom flange there is no buckling.

2. When one or both end moments cause compression on the bottom, use $C_b$ with $L_b$.

\[
C_b = 3.0 - \frac{2}{3} \left( \frac{M_1}{M_0} \right) - \frac{8}{3} \left( \frac{M_c}{M_0 + M_1} \right) \]

*Take $M_1 = 0$ in this term if $M_1$ is positive

$M_0 = \text{end moment that gives the largest comp. stress on the bottom flange}$

$M_1 = \text{the other end moment}$

$M_c = \text{moment at midspan}$
ASCE 7-16 & AISC Design Checks

Column Design

- AISC 341 Section G4.5 Requires Columns Meet Requirements Of AISC 341 Section D2
- Material Requirements of Section 6 Are Met By All W10 Columns of A992 Steel
- AISC 341 Requires Special Load Combination If $P_d/\phi c P_n$ Exceeds 0.4.
- AISC 360 Allow Use Of K = 1.0 By Direct Analysis Method
- AISC 341 Does Require Compactness Criteria.
- Strong Column Weak Beam Concept For C-PRMF Not Addressed By AISC 341 Currently But ASCE TC Recommends:

$$\sum M_{p, col} \left(1 - \frac{P_d}{P_y}\right) \geq 1.25 \left(M^-_{cu} + M^+_{cu}\right)$$
ASCE 7-16 & AISC Design Checks

Connection Checks

Because Using Non-Linear Curve, A Check Against Connection Capacity Is Not Really Necessary; However, It Is Of Interest To Understand How Hard The Connections Are Being Pushed.

• Example Problem:

<table>
<thead>
<tr>
<th>Table 9.4-5 Connection Moment Demand vs. Capacity (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W21 PRCC</td>
</tr>
<tr>
<td>(-) M-θ</td>
</tr>
<tr>
<td>Demand</td>
</tr>
<tr>
<td>Capacity</td>
</tr>
<tr>
<td>Ratio</td>
</tr>
</tbody>
</table>

• This Is Not The Case If Using Linear Springs To Model The Connection Behavior
Questions
DISCLAIMER

• NOTICE: Any opinions, findings, conclusions, or recommendations expressed in this publication do not necessarily reflect the views of the Federal Emergency Management Agency. Additionally, neither FEMA nor any of its employees make any warranty, expressed or implied, nor assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, product or process included in this publication.

• The opinions expressed herein regarding the requirements of the NEHRP Recommended Seismic Provisions, the referenced standards, and the building codes are not to be used for design purposes. Rather the user should consult the jurisdiction’s building official who has the authority to render interpretation of the code.

• Any modifications made to the file represent the presenters' opinion only.