Seismic Design of Structures
RC Seismic Load Resisting Systems
RC Systems

- **Load Path**
- **Adequacy:** is implemented by ensuring that, at any point along its path, it can withstand the actions occurring at that point.

- In designating a load path, the engineers must ensure that the structure has reliable strength and adequate ductility.
RC Systems

- **Gravity Systems**
- **Floor Systems**
- 1- Flat Plates:
  - Concrete slabs are often used to carry vertical loads directly to walls and columns without the use of beams and girders
  - Flat plates can be used with irregularly spaced column layouts
RC Systems

- **Gravity Systems**
- **Floor Systems**
- **2- Flat Slabs:**
  - Flat slab is also a two-way system of beamless construction but incorporates a thickened slab in the region of columns and walls
  
  - Drop panels and columns capitals, reduce shear and negative bending stresses around the columns
RC Systems

• **Gravity Systems**
• **3-Waffle Systems**
• This system also called a two-way joist system.

• In contrast to a joist which carries loads in a one-way action, a waffle system carries the loads simultaneously in **two directions**
RC Systems

- **Gravity Systems**

4- One-Way Concrete Ribbed Slabs
- The joists are designed as one-way T-beams for the full-moment tributary to its width.

5- Skip Joist System

6- Beam And Slab System
This system consists of a continuous slab supported by beam
RC Systems

- **Lateral Load Resisting Systems**
- Most of the systems can be grouped into three basic types:
  1. Shear wall system
  2. Frame system
  3. Combination of the two, the shear wall-frame system (dual system)
RC Systems

- **Lateral Load Resisting Systems**
- The seismic-force-resisting system as being composed of
1. Vertical Elements
2. Horizontal Elements
3. Foundation
RC Systems

• **Lateral Load Resisting Systems**
  • The seismic-force-resisting system as being composed of
    1- Vertical Elements  2- Horizontal Elements  3- Foundation

• **Diaphragms:** Make up the horizontal elements of the seismic-force-resisting system

• These act to transmit inertial forces from the floor system to the vertical elements of the seismic-force-resisting system.

• They also tie the vertical elements together, and thereby transmit forces between these elements as may be required during earthquake shaking.
RC Systems

- **Lateral Load Resisting Systems**

1. As the ground shakes, the motion is transferred to the foundation and into the superstructure.

2. The resulting motion of the superstructure leads to inertial forces (product of mass and acceleration).

3. The seismic-force-resisting system must be designed to provide a balanced and continuous load path from the source of the inertial forces back down to the foundation.

- Locate the vertical elements so the center of resistance is close to the center of mass.
RC Systems

- **Lateral Load Resisting Systems**
  - Acceleration of the floor diaphragm results in inertial forces within the plane of the diaphragm that must be transmitted to the vertical elements of the seismic-force-resisting system.

- Excessive flexibility, inelastic response, or failure of inadequate diaphragm components contributed to the collapse of parking structures during the 1994 Northridge earthquake
RC Systems

- **Lateral Load Resisting Systems**
- The vertical elements of the seismic-force-resisting system are required to transmit the accumulated seismic forces to the foundation system.

- It is preferable for the vertical elements to be *continuous* over height.


**RC Systems**

- **Lateral Load Resisting Systems**
  - In conventional buildings, the intended inelastic response ideally is restricted to the vertical elements of the seismic-force-resisting system.
  - This can be accomplished by first sizing the vertical elements for expected earthquake demands (reduced for anticipated inelastic response).
  - Then designing the diaphragm and foundation elements to have sufficient strength to avoid significant inelastic response (overstrength using a factor $\Omega_0$).

- Inelastic response also is permitted for elements not designated as part of the seismic-force resisting system, such as the gravity framing.

- **but** it must be checked to be certain the deformation capacity is adequate.
RC Systems

- Lateral Load Resisting Systems

(1) Shear wall system
- Buildings engineered with structural walls are almost always stiffer than framed structures, reducing the possibility of excessive deformations and hence damage.
- By adopting special detailing measures, dependable ductile response can be achieved under major earthquakes.
- Lateral forces cause shear and overturning moments in walls.
1 - Shear wall system

- Cast-in-Place
  - Ordinary Plain
  - Detailed Plain
  - Ordinary
  - Intermediate
  - Special
- Precast
  - Ordinary
  - Intermediate
**Lateral Load Resisting Systems**

1. **Shear wall system- Coupled Shear walls**
   - The magnitude of the axial force, $T = C$, is given by the sum of the shear forces occurring in the coupling beams.
   - If coupling beams are stiff, major moment resistance is by the couple generated by the equal and opposite axial focus in the wall piers.

\[ M = M_1 + M_2 + Td \]
RC Systems

• Lateral Load Resisting Systems

2- Moment Resisting Frames

• The lateral load resistance is provided by the interaction of girders and the columns

• The ACI 318 requires that the flexural strengths of columns be at least 20% more than the sum of the corresponding strength of the connecting beams at any story
RC Systems

2- Moment-Resisting Frames

- Cast-in-Place
  - Ordinary
  - Intermediate
  - Special

- Precast
  - Special
RC Systems

• **Lateral Load Resisting Systems**

3- **Dual Systems**

- Reinforced concrete frames interacting with shear walls together provide the necessary resistance to lateral forces,
- Each system carries its appropriate share of the gravity load

**Shear wall- Frame Interaction**
RC Systems

- Lateral Load Resisting Systems - ASCE 7-10

Bearing Wall Systems

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>ASCE 7 Section Where Detailing Requirements Are Specified</th>
<th>Response Modification Coefficient, $R^d$</th>
<th>Overstrength Factor, $O_{M}^d$</th>
<th>Deflection Amplification Factor, $C_{d}$</th>
<th>Structural System Limitations Including Structural Height, $h_{s}$ (ft) Limits</th>
<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. BEARING WALL SYSTEMS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Special reinforced concrete shear walls&lt;sup&gt;6&lt;/sup&gt;</td>
<td>14.2</td>
<td>5</td>
<td>2½</td>
<td>5</td>
<td>NL  NL  160  160  100</td>
<td></td>
</tr>
<tr>
<td>2. Ordinary reinforced concrete shear walls&lt;sup&gt;6&lt;/sup&gt;</td>
<td>14.2</td>
<td>4</td>
<td>2½</td>
<td>4</td>
<td>NL  NL  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>5. Detailed plain concrete shear walls&lt;sup&gt;5&lt;/sup&gt;</td>
<td>14.2</td>
<td>2</td>
<td>2½</td>
<td>2</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>4. Ordinary plain concrete shear walls&lt;sup&gt;5&lt;/sup&gt;</td>
<td>14.2</td>
<td>1½</td>
<td>2½</td>
<td>1½</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>5. Intermediate precast shear walls&lt;sup&gt;5&lt;/sup&gt;</td>
<td>14.2</td>
<td>4</td>
<td>2½</td>
<td>4</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>6. Ordinary precast shear walls&lt;sup&gt;5&lt;/sup&gt;</td>
<td>14.2</td>
<td>3</td>
<td>2½</td>
<td>3</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>7. Special reinforced masonry shear walls&lt;sup&gt;7&lt;/sup&gt;</td>
<td>14.4</td>
<td>5</td>
<td>2½</td>
<td>3½</td>
<td>NL  160  160  100</td>
<td></td>
</tr>
<tr>
<td>8. Intermediate reinforced masonry shear walls&lt;sup&gt;7&lt;/sup&gt;</td>
<td>14.4</td>
<td>3½</td>
<td>2½</td>
<td>2½</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>9. Ordinary reinforced masonry shear walls&lt;sup&gt;7&lt;/sup&gt;</td>
<td>14.4</td>
<td>2</td>
<td>2½</td>
<td>1½</td>
<td>NL  160  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>10. Detailed masonry shear walls&lt;sup&gt;7&lt;/sup&gt;</td>
<td>14.4</td>
<td>2</td>
<td>3½</td>
<td>1½</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>11. Ordinary plain masonry shear walls&lt;sup&gt;7&lt;/sup&gt;</td>
<td>14.4</td>
<td>1½</td>
<td>2½</td>
<td>1½</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>12. Prestressed masonry shear walls&lt;sup&gt;7&lt;/sup&gt;</td>
<td>14.4</td>
<td>1½</td>
<td>2½</td>
<td>1½</td>
<td>NL  NP  NP  NP  NP</td>
<td></td>
</tr>
<tr>
<td>13. Ordinary reinforced AAC masonry shear walls&lt;sup&gt;7&lt;/sup&gt;</td>
<td>14.4</td>
<td>2</td>
<td>2½</td>
<td>2</td>
<td>NL  35  NP  NP  NP</td>
<td></td>
</tr>
</tbody>
</table>
RC Systems

- **Lateral Load Resisting Systems - ASCE7-10**

### Building Frame Systems

<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, $\Omega^b_0$</th>
<th>Deflection Amplification Factor, $C^b_d$</th>
<th>Structural System Limitations Including Structural Height, $h_s$ (ft) Limits$^c$</th>
<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>4. Special reinforced concrete shear walls$^f$</td>
<td>14.2</td>
<td>6</td>
<td>2½</td>
<td>5</td>
<td>NL NL 160 160 100</td>
<td></td>
</tr>
<tr>
<td>5. Ordinary reinforced concrete shear walls$^f$</td>
<td>14.2</td>
<td>5</td>
<td>2½</td>
<td>4½</td>
<td>NL NL NP NP NP</td>
<td></td>
</tr>
</tbody>
</table>

- **Moment-Resisting Frame Systems**

<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, $\Omega^b_0$</th>
<th>Deflection Amplification Factor, $C^b_d$</th>
<th>Structural System Limitations Including Structural Height, $h_s$ (ft) Limits$^c$</th>
<th>Seismic Design Category</th>
</tr>
</thead>
<tbody>
<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 NL</td>
<td></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 NP</td>
<td></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 NP</td>
<td></td>
</tr>
</tbody>
</table>
**RC Systems**

- **Lateral Load Resisting Systems - ASCE7-10**
- **Dual Systems with Special Moment Frames**
  
  CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3. Special reinforced concrete shear walls(d)</td>
<td>14.2</td>
<td>7</td>
<td>2½</td>
<td>5½</td>
<td>NL</td>
</tr>
<tr>
<td>4. Ordinary reinforced concrete shear walls(d)</td>
<td>14.2</td>
<td>6</td>
<td>2½</td>
<td>5</td>
<td>NL</td>
</tr>
</tbody>
</table>

- **Dual Systems with Intermediate Moment Frames**
  
  CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES

<p>| | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2. Special reinforced concrete shear walls(d)</td>
<td>14.2</td>
<td>6½</td>
<td>2½</td>
<td>5</td>
<td>NL</td>
</tr>
<tr>
<td>3. Ordinary reinforced masonry shear walls</td>
<td>14.4</td>
<td>3</td>
<td>3</td>
<td>2½</td>
<td>NL</td>
</tr>
</tbody>
</table>
RC Systems

- **Lateral Load Resisting Systems - ASCE7-10**
- **Dual Systems with Special Moment Frames**
  CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES

| 3. Special reinforced concrete shear walls | 14.2 | 7 | 2½ | 5½ | NL | NL | NL | NL | NL |
| 4. Ordinary reinforced concrete shear walls | 14.2 | 6 | 2½ | 5 | NL | NL | NP | NP | NP |

- **Dual Systems with Intermediate Moment Frames**
  CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES

| 2. Special reinforced concrete shear walls | 14.2 | 6½ | 2½ | 5 | NL | NL | 160 | 100 | 100 |
| 3. Ordinary reinforced masonry shear walls | 14.4 | 3 | 3 | 2½ | NL | 160 | NP | NP | NP |
RC Seismic Load Resisting Systems

Moment Resisting Frame
Overview

• Introduction
• Reinforced Concrete
• General Requirements
• Moment Resisting Frame (MRF)
• Shear Wall
Topics

- Codes
- Seismic Design Category Requirements
- Special Moment Resisting Frame
  - Seismic Design Basis
  - General Requirements
  - Beams
  - Columns
  - Joints
References

- ASCE 7-10, Minimum Design Loads for Buildings and Other Structures
- ACI 318-14, Building Code Requirements for Structural Concrete And Commentary
- NIST 8-917-1, Seismic Design of RC Special Moment Frame, 2008
- NIST GCR 11-917-11 V-1, Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams, 2012
- PCA Notes on ACI 318-11
Special Moment Resisting Frame

Codes

RC Systems
Reference Codes

- Reference standards
  - ASCE 7-10
  - ACI 318-14

An ACI Standard and Report

Building Code Requirements for Structural Concrete (ACI 318-14)

Commentary on Building Code Requirements for Structural Concrete (ACI 318R-14)

Reported by ACI Committee 318
Reference Codes

- **ASCE 7-10**
  - Determine Loads
  - Define Systems and Classifications
  - Provides Design Coefficients

- **ACI 318-14**
  - Provides System Design
  - Chapter 18 Earthquake-Resistant Structures
  - Includes Detailing Requirements
  - Some Modifications are required ASCE 7 Section 14.2
    - presents some modifications to ACI 318
    - some additional reinforced concrete structure requirements
Special Moment Resisting Frame

Seismic Design Category Requirements
Design Coefficients

- **Moment-Resisting Frames**
  - ASCE 7-10 Table 12.2-1

<table>
<thead>
<tr>
<th>Seismic Force Resisting System</th>
<th>Response Modification Coefficient, R</th>
<th>Deflection Amplification Factor, C_d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special R/C Moment Frame</td>
<td>8</td>
<td>5.5</td>
</tr>
<tr>
<td>Intermediate R/C Moment Frame</td>
<td>5</td>
<td>4.5</td>
</tr>
<tr>
<td>Ordinary R/C Moment Frame</td>
<td>3</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Design Coefficients

- **Dual Systems with Special Frames**
  - ASCE 7-10 Table 12.2-1
  - Dual systems include a special moment resisting frame

<table>
<thead>
<tr>
<th>Seismic Force Resisting System</th>
<th>Response Modification Coefficient, R</th>
<th>Deflection Amplification Factor, $C_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dual System w/ Special Walls</td>
<td>7</td>
<td>5.5</td>
</tr>
<tr>
<td>Dual System w/ Ordinary Walls</td>
<td>6</td>
<td>5</td>
</tr>
</tbody>
</table>
Seismic Terminology

- Seismic-related terminology in model codes

5.2.2 Loads and Seismic Design Categories (SDCs) shall be in accordance with the general building code, or determined by the authority having jurisdiction.

- Seismic Design Categories (SDCs) in ACI Code are adopted directly from ASCE/SEI 7.

Table R5.2.2—Correlation between seismic-related terminology in model codes

<table>
<thead>
<tr>
<th>Code, standard, or resource document and edition</th>
<th>Level of seismic risk or assigned seismic performance or design categories as defined in the Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318-05 and previous editions</td>
<td>Low seismic risk</td>
</tr>
<tr>
<td>Uniform Building Code 1991, 1994, 1997</td>
<td>Seismic Zone 0, 1</td>
</tr>
</tbody>
</table>

[1] SDC = seismic design category as defined in code, standard, or resource document.
Load Combinations

• The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated and the variation that might be expected in the load during the lifetime of the structure.

• Variability in the structural analysis used to calculate moments and shears.

5.3—Load factors and combinations

5.3.1 Required strength $U$ shall be at least equal to the effects of factored loads in Table 5.3.1, with exceptions and additions in 5.3.3 through 5.3.12.

<table>
<thead>
<tr>
<th>Load combination</th>
<th>Equation</th>
<th>Primary load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$U = 1.4D$</td>
<td>(5.3.1a)</td>
<td>$D$</td>
</tr>
<tr>
<td>$U = 1.2D + 1.6L + 0.5(L_r or S or R)$</td>
<td>(5.3.1b)</td>
<td>$L$</td>
</tr>
<tr>
<td>$U = 1.2D + 1.6(L_r or S or R) + (1.0L or 0.5W)$</td>
<td>(5.3.1c)</td>
<td>$L_r or S or R$</td>
</tr>
<tr>
<td>$U = 1.2D + 1.0W + 1.0L + 0.5(L_r or S or R)$</td>
<td>(5.3.1d)</td>
<td>$W$</td>
</tr>
<tr>
<td>$U = 1.2D + 1.0E + 1.0L + 0.2S$</td>
<td>(5.3.1e)</td>
<td>$E$</td>
</tr>
<tr>
<td>$U = 0.9D + 1.0W$</td>
<td>(5.3.1f)</td>
<td>$W$</td>
</tr>
<tr>
<td>$U = 0.9D + 1.0E$</td>
<td>(5.3.1g)</td>
<td>$E$</td>
</tr>
</tbody>
</table>
System Selection

• **Moment-Resisting Frame**
  
  □ **Ordinary moment frames**
  
  • Have very few requirements of **ACI 318 Chapter 18 Section 18.3**
  
  • For the most part, they are designed in accordance with the non-seismic chapters of **ACI 318**

  □ **Intermediate moment frames**
  
  • Must meet requirements of **ACI 318 section 18.4**
  
  • Requirements are more stringent detailing than for ordinary frames but less severe than for special frames

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Minimum Frame Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Ordinary</td>
</tr>
<tr>
<td>C</td>
<td>Intermediate</td>
</tr>
<tr>
<td>D, E and F</td>
<td>Special</td>
</tr>
</tbody>
</table>
System Selection

• Moment-Resisting Frame
  ▫ Special moment frames
    • Special moment frames must meet detailed requirements in various sections of ACI 318, Chapter 18
    • Sections 18.6 to 18.8 should be satisfied
    • Requirements includes detailing to ensure ductility, stability, and minimum degradation of strength during cyclic loading

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Minimum Frame Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Ordinary</td>
</tr>
<tr>
<td>C</td>
<td>Intermediate</td>
</tr>
<tr>
<td>D, E and F</td>
<td>Special</td>
</tr>
</tbody>
</table>
System Selection

- Shear Walls (Structural Wall)
  - Reinforced Ordinary Shear Walls
    - They are designed in accordance with the non-seismic chapters of ACI 318
  - Reinforced Special Shear Walls
    - Special moment frames must meet detailed requirements in various sections of ACI 318, Chapter 18
    - Section 18.10 should be satisfied
  - Plain concrete walls
    - Are designed per Chapter 14
    - Are permitted in SDC B for some circumstances

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Minimum Wall Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>B and C</td>
<td>Ordinary</td>
</tr>
<tr>
<td>D, E and F</td>
<td>Special</td>
</tr>
</tbody>
</table>
Design Requirements

- **Seismic Design Category Requirements**
  - Some general requirements for concrete buildings based on Seismic Design Category and independent of specific lateral force resisting system
  - Consistent throughout the Provisions the design scope is more detailed for higher Categories

---

Table R18.2—Sections of Chapter 18 to be satisfied in typical applications[^1]

<table>
<thead>
<tr>
<th>Component resisting earthquake effect, unless otherwise noted</th>
<th>A (None)</th>
<th>B (18.2.1.3)</th>
<th>C (18.2.1.4)</th>
<th>D, E, F (18.2.1.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis and design requirements</td>
<td>18.2.2</td>
<td>18.2.2</td>
<td>18.2.2, 18.2.4</td>
<td></td>
</tr>
<tr>
<td>Materials</td>
<td>None</td>
<td>None</td>
<td></td>
<td>18.2.5 through 18.2.8</td>
</tr>
<tr>
<td>Frame members</td>
<td>18.3</td>
<td>18.4</td>
<td></td>
<td>18.6 through 18.9</td>
</tr>
<tr>
<td>Structural walls and coupling beams</td>
<td>None</td>
<td>None</td>
<td></td>
<td>18.10</td>
</tr>
<tr>
<td>Precast structural walls</td>
<td>None</td>
<td>18.5</td>
<td></td>
<td>18.5[2], 18.11</td>
</tr>
<tr>
<td>Diaphragms and trusses</td>
<td>None</td>
<td>None</td>
<td></td>
<td>18.12</td>
</tr>
<tr>
<td>Foundations</td>
<td>None</td>
<td>None</td>
<td></td>
<td>18.13</td>
</tr>
<tr>
<td>Frame members not designated as part of the seismic-force-resisting system</td>
<td>None</td>
<td>None</td>
<td></td>
<td>18.14</td>
</tr>
<tr>
<td>Anchors</td>
<td>None</td>
<td>18.2.3</td>
<td>18.2.3</td>
<td></td>
</tr>
</tbody>
</table>

[^1]: In addition to requirements of Chapters 1 through 17, 19 through 26, and ACI 318.2, except as modified by Chapter 18. Section 14.1.4 also applies in SDC D, E, and F.
Historic Development

Reinforced concrete special moment frame concepts were introduced in the U.S. starting around 1960 (Blume, Newmark, and Corning 1961).

In 1973 the Uniform Building Code first required use of the special frame details in regions of highest seismicity.

- The earliest detailing requirements have many similarities to those in use today, though there are notable differences.

- In most early applications, special moment frames were used in all framing lines of a building.

- A trend that developed in the 1990s was:
  - to use SMF in fewer framing lines of the building.
  - the remainder comprising gravity-only framing that was not designated as part of the seismic force resisting system.
Introduction

- **Historic Development**
  - Some of these *gravity-only frames* did not perform well in the 1994 Northridge Earthquake
    - Leading to more stringent requirements for proportioning and detailing these frames
    - The provisions for members not designated as part of the seismic force-resisting system are contained in ACI 318 Section 18.14 and apply wherever special moment frames are used in Seismic Design Category D, E, or F
  - The detailing requirements for the *gravity-only elements* are similar to the requirements for the SMRs
    - Some economy may be achieved if the *gravity-only frames* can be made to qualify as part of the seismic force-resisting system
Introduction

• Historic Development
  • Special moment frames have also found use in dual systems that combine special moment frames with shear walls or braced frames.

• In current U.S. codes, if a seismic-force-resisting system is designated as a dual system it is required that:
  ▫ the moment frame be capable of resisting at least 25% of the design seismic forces.
  ▫ While the total seismic resistance is provided by the combination of the moment frame and the shear walls or braced frames in proportion with their relative stiffnesses.
Application

When to Use Special Moment Frames?

• Moment frames are generally selected as the seismic force resisting system when architectural space planning flexibility is desired

• When concrete moment frames are selected for buildings in Seismic Design Categories D, E, or F, they are required to be detailed as special reinforced concrete moment frames

• Proportioning and detailing requirements for a special moment frame will enable the frame to
  ▫ Safely undergo extensive inelastic deformations that are anticipated in these seismic design categories
  ▫ SMF may be used in Seismic Design Categories A, B, and C, though this may not lead to the most economical design
Application

When to Use Special Moment Frames?

• Dual systems combining walls or braced frames with special moment frames:

  1- For tall buildings. Some building codes limit the height of certain seismic-force-resisting systems such as special reinforced concrete shear walls when such systems provide the entire seismic force resistance. These height limits do not apply when special moment frames are added to create a dual system.

  2- Where buildings are constructed on poor soils requiring expensive foundations. By using a dual system rather than a special shear wall without frames, the design forces may be reduced.
Frame Proportioning

Typical RC Moment-Resisting Frames

• **Cross-sectional dimensions for Beams**
• Typical economical beam spans for special moment frames are in the range of 6 to 9 m
  ▪ In general, this range will result in beam depths that will support typical gravity loads and the seismic forces without overloading the adjacent beam-column joints and columns

• The clear span of a beam must be at least four times its effective depth per *ACI 318 - 18.6.2*
• Beams are allowed to be wider than the supporting columns as noted in *ACI 318 - 18.6.2*
• Beam width normally does not exceed the width of the column, for constructability
• Provisions for special moment frames exclude use of slab-column framing as part of the seismic force-resisting system
Frame Proportioning

Typical RC Moment-Resisting Frames

- Minimum beam width is 0.3hb, but not less than 250 mm.

- Cross-sectional dimensions for columns
  - The ratio of the cross-sectional dimensions for columns shall not be less than 0.4 per ACI 318 – 18.7.2 to limit the cross section to a more compact section, not a long rectangle
  - ACI 318 – 21.6.1.1 sets the minimum column dimension to 300 mm, which is often not practical to construct
  - A minimum dimension of 400 mm is suggested, except for unusual cases or for low-rise buildings
Design Principals

- The proportioning and detailing goals are
  1. Design a strong-column/weak-beam system
  2. Detail beams and columns for ductile flexural response
  3. Avoid more brittle failure modes such as shear, axial, connection, and splice failures
  4. Avoid interaction with nonstructural components

- The $R$ factor for special moment frames is 8
- A special moment frame should be expected to sustain multiple cycles of inelastic response if it experiences design-level ground motion
Design Principals

- Design a Strong-column / Weak-beam Frame
  - The distribution of damage over height depends on the distribution of lateral drift
    - If the building has weak columns, drift tends to concentrate in a few stories and may exceed the drift capacity of columns
    - If columns provide a stiff and strong spine over the building height, drift will be more uniformly distributed and localized damage will be reduced
Design Principals

- Design a Strong-column / Weak-beam Frame
  - The columns in a given story support the weight of the entire building above those columns
  - The beams only support the gravity loads of the floor
  - Failure of a column is of greater consequence than of a beam
  - This strong-column/weak-beam principle is fundamental to achieving safe behavior of frames during strong earthquake ground shaking

(a) Story mechanism  (b) Intermediate mechanism  (c) Beam mechanism
Design Principals

- Design a Strong-column /Weak-beam Frame
- Achieving a complete beam mechanism may require column moment strengths several times beam moment strengths, increasingly so for taller buildings, which may prove uneconomical
- Therefore, some yielding of the columns has to be anticipated, and reinforcement details consistent with this anticipated behavior must be provided
Design Principals

- Avoid Non-ductile Failure Modes
  - Ductile response requires that
    - members yield in flexure
    - shear failure be avoided

1- Column and Beam Shear
  - Shear failure, especially in columns is relatively brittle and can lead to rapid loss of lateral strength and axial load-carrying capacity
    - Column shear failure is the most frequently cited cause of concrete building failure and collapse in earthquakes
  - Shear failure is avoided through use of a capacity-design approach

Shear failure can lead to a story mechanism and axial collapse
Design Principals

1- Column
2- Column Axial Load

- Column axial failure can trigger progressive collapse in which axial loads from the overloaded column are transferred to adjacent columns.
- Overloading them in turn and leading to collapse of an entire story or building.
Design Principals

3- Connections

- In reinforced concrete special moment frame construction, we are concerned with connections between horizontal and vertical elements.

- Beam-column joints are especially vulnerable to failure at the perimeter of buildings because exterior faces are not confined by adjacent concrete framing members.

- Transverse reinforcement is required in special moment frame joints to confine the joint concrete and participate in the resistance of joint forces.
Design Principals

- **Detail for Ductile Behavior**
  - Ductile behavior of reinforced concrete members is based on these principles
    - Confinement for heavily loaded sections
    - Ample shear reinforcement
    - Avoidance of anchorage or splice failure
Design Principals

- **Detail for Ductile Behavior**
  - Ample shear reinforcement
    - Shear strength degrades in members subjected to multiple inelastic deformation reversals, especially if axial loads are low
  - In such members ACI 318 requires that the contribution of concrete to shear resistance be ignored, \((V_c = 0)\)
  - Shear reinforcement is required to resist the entire shear force
Design Principals

• **Detail for Ductile Behavior**
  ▫ **Avoidance of anchorage or splice failure**
    - Severe seismic loading can result in loss of concrete cover
    - This will reduce development and lap-splice strength of longitudinal reinforcement
  ▫ **Lap splices, must be located away from sections of maximum moment (that is, away from ends of beams and columns) and must have closed hoops to confine the splice in the event of cover spalling**
    - Bars passing through a beam-column joint can create severe bond stress demands on the joint; for this reason, ACI 318 restricts beam bar sizes
  ▫ **Bars anchored in exterior joints must develop yield strength using hooks located at the far side of the joint**
Analysis

- **Stiffness Recommendations**
  - It is important to appropriately model the cracked stiffness of the beams, columns, and joints.
    - This stiffness determines the resulting building periods, base shear, story drifts, and internal force distributions.
  - Table shows the range of values for the effective, cracked stiffness for each element per ACI 318 - 6.6.3.
    - More detailed analysis may be used based on applied loading.

<table>
<thead>
<tr>
<th>Member and condition</th>
<th>Moment of Inertia</th>
<th>Cross-sectional area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>$0.70I_g$</td>
<td></td>
</tr>
<tr>
<td>Walls</td>
<td>$0.70I_g$</td>
<td>$1.04g$</td>
</tr>
<tr>
<td>Cracked</td>
<td>$0.35I_g$</td>
<td></td>
</tr>
<tr>
<td>Beams</td>
<td>$0.35I_g$</td>
<td></td>
</tr>
<tr>
<td>Flat plates and flat slabs</td>
<td>$0.25I_g$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Member</th>
<th>Alternative value of $I$ for elastic analysis</th>
<th>$I$</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns and walls</td>
<td>$0.35I_g$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$0.80 + 25 \left( \frac{A_u}{A_g} \right) \left[ 1 - \frac{M_u}{P_s h} - 0.5 \frac{P_s}{P_e} \right]$</td>
<td></td>
<td>$0.875I_g$</td>
</tr>
<tr>
<td>Beams, flat plates, and flat slabs</td>
<td>$0.25I_g$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$(0.10 + 25p) \left( 1.2 - 0.2 \frac{b_v}{d} \right) I_g$</td>
<td></td>
<td>$0.5I_g$</td>
</tr>
</tbody>
</table>
Analysis

- Stiffness Recommendations

6.6.3.1.1 Moment of inertia and cross-sectional area of members shall be calculated in accordance with Tables 6.6.3.1.1(a) or 6.6.3.1.1(b), unless a more rigorous analysis is used. If sustained lateral loads are present, \( I \) for columns and walls shall be divided by \((1 + \beta_d)\), where \( \beta_d \) is the ratio of maximum factored sustained shear within a story to the maximum factored shear in that story associated with the same load combination.

6.6.3.1.2 For factored lateral load analysis, it shall be permitted to assume \( I = 0.5I_g \) for all members or to calculate \( I \) by a more detailed analysis, considering the reduced stiffness of all members under the loading conditions.

**Table 6.6.3.1.1(a)—Moment of inertia and cross-sectional area permitted for elastic analysis at factored load level**

<table>
<thead>
<tr>
<th>Member and condition</th>
<th>Moment of Inertia</th>
<th>Cross-sectional area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>0.70I_g</td>
<td></td>
</tr>
<tr>
<td>Walls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uncracked</td>
<td>0.70I_g</td>
<td></td>
</tr>
<tr>
<td>Cracked</td>
<td>0.35I_g</td>
<td>1.0A_g</td>
</tr>
<tr>
<td>Beams</td>
<td>0.35I_g</td>
<td></td>
</tr>
<tr>
<td>Flat plates and flat slabs</td>
<td>0.25I_g</td>
<td></td>
</tr>
</tbody>
</table>

**Table 6.6.3.1.1(b)—Alternative moments of inertia for elastic analysis at factored load**

<table>
<thead>
<tr>
<th>Member and condition</th>
<th>Alternative value of ( I ) for elastic analysis</th>
</tr>
</thead>
</table>
| Columns and walls    | 0.35I_g  
                   | \( 0.80 + 25 \left( \frac{A_e}{A_g} \right) \left( 1 - \frac{M_n}{P_n h} - 0.5 \frac{P}{P_o} \right) I_e \)  
                   | 0.875I_g  |
| Beams, flat plates, and flat slabs  | 0.25I_g  
                   | \( 0.10 + 25p \left( 1 - 0.2 \frac{h}{d} \right) I_g \)  
                   | 0.5I_g  |

Notes: For continuous flexural members, \( I \) shall be permitted to be taken as the average of values obtained for the critical positive and negative moment sections. \( P_o \) and \( M_n \) shall be calculated from the load combination under consideration, or the combination of \( P_o \) and \( M_n \) that produces the least value of \( I \).
Analysis

• **Stiffness Recommendations**

6.3.2.1 For nonprestressed T-beams supporting monolithic or composite slabs, the effective flange width $b_f$ shall include the beam web width $b_w$ plus an effective overhanging flange width in accordance with Table 6.3.2.1, where $h$ is the slab thickness and $s_w$ is the clear distance to the adjacent web.

Table 6.3.2.1—Dimensional limits for effective overhanging flange width for T-beams

<table>
<thead>
<tr>
<th>Flange location</th>
<th>Effective overhanging flange width, beyond face of web</th>
</tr>
</thead>
<tbody>
<tr>
<td>Each side of web</td>
<td>Least of:</td>
</tr>
<tr>
<td></td>
<td>$8h$</td>
</tr>
<tr>
<td></td>
<td>$s_w/2$</td>
</tr>
<tr>
<td></td>
<td>$\ell_n/8$</td>
</tr>
<tr>
<td>One side of web</td>
<td>Least of:</td>
</tr>
<tr>
<td></td>
<td>$6h$</td>
</tr>
<tr>
<td></td>
<td>$s_w/2$</td>
</tr>
<tr>
<td></td>
<td>$\ell_n/12$</td>
</tr>
</tbody>
</table>

6.3.2.2 Isolated nonprestressed T-beams in which the flange is used to provide additional compression area shall have a flange thickness greater than or equal to $0.5b_w$ and an effective flange width less than or equal to $4b_w$. 
Analysis

• **Stiffness Recommendations**
  - For beams cast monolithically with slabs, it is acceptable to include the effective flange width of ACI 318 - 6.3.2
    - It is generally sufficiently accurate to take $I_g$ of a T-beam as $2I_g$ for the web, $2(b_wh^3/12)$
  - ACI 318 does not contain guidance on modeling the stiffness of the beam-column joint
    - In a special moment frame the beam-column joint is stiffer than the adjoining beams and columns, but it is *not* perfectly rigid
    - As described in ASCE 41 the joint stiffness can be adequately modeled by extending the beam flexibility to the column centerline and defining the column as rigid within the joint
Overview

- Chapter 18 Earthquake-Resistant Structures

1. Scope
2. General
   1. Structural systems
   2. Analysis and proportioning of structural members
   3. Anchoring to concrete
   4. Strength reduction factors
   5. Concrete in special moment frames and special structural walls
   6. Reinforcement in special moment frames and special structural walls
   7. Mechanical splices in special moment frames and special structural walls
   8. Welded splices in special moment frames and special structural walls
3. Ordinary moment frames
Overview

• Chapter 18 Earthquake-Resistant Structures

4. Intermediate moment frames
   1. Scope
   2. Beams
   3. Columns
   4. Joints
   5. Two-way slabs without beams

5. Intermediate precast structural walls

6. Beams of special moment frames
   1. Scope
   2. Dimensional limits
   3. Longitudinal reinforcement
   4. Transverse reinforcement
   5. Shear strength
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- Chapter 18 Earthquake-Resistant Structures

  7. Columns of special moment frames
     1. Scope
     2. Dimensional limits
     3. Minimum flexural strength of columns
     4. Longitudinal reinforcement
     5. Transverse reinforcement
     6. Shear strength

  8. Joints of special moment frames
     1. Scope
     2. General
     3. Transverse reinforcement
     4. Shear strength
     5. Development length of bars in tension
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• Chapter 18 Earthquake-Resistant Structures
  9. Special moment frames constructed using precast concrete
  10. Special structural walls
  11. Special structural walls constructed using precast concrete
  12. Diaphragms and trusses
  13. Foundations
  14. Members not designated as part of the seismic-force-resisting system

1. Scope
2. Design actions
3. Cast-in-place beams, columns, and joints
4. Precast beams and columns
5. Slab-column connections
6. Wall piers
General Requirements

18.1 Scope

- Chapter 18
  - Does not apply to structures assigned to Seismic Design Category SDC-A
  - For SDC B and C, applies to structural systems designated as part of the seismic-force-resisting system SFRS
  - For SDC D through F, applies to both structural systems designated as part of SFRS and structural systems not designated as part of the SFRS
- The design philosophy in Chapter 18 is for cast-in-place concrete structures to respond in the nonlinear range when subjected to design-level ground motions, with decreased stiffness and increased energy dissipation but without critical strength decay
**General Requirements**

18.2 General

- 18.2.1 Structural systems
  - The combination of reduced stiffness and increased energy dissipation tends to reduce the response accelerations and lateral inertia forces relative to values that would occur were the structure to remain linearly elastic and lightly damped
  - Seismic design categories are adopted directly from ASCE/SEI 7, and relate to seismic hazard level, soil type, occupancy, and use
  - SDC B through F must satisfy requirements of Chapter 18 in addition to all other applicable requirements of this Code

18.2—General
18.2.1 Structural systems

18.2.1.1 All structures shall be assigned to a SDC in accordance with 4.4.6.1.

18.2.1.2 All members shall satisfy Chapters 1 to 17 and 19 to 26. Structures assigned to SDC B, C, D, E, or F also shall satisfy 18.2.1.3 through 18.2.1.7, as applicable. Where Chapter 18 conflicts with other chapters of this Code, Chapter 18 shall govern.

18.2.1.3 Structures assigned to SDC B shall satisfy 18.2.2.

18.2.1.4 Structures assigned to SDC C shall satisfy 18.2.2 and 18.2.3.

18.2.1.5 Structures assigned to SDC D, E, or F shall satisfy 18.2.2 through 18.2.8 and 18.12 through 18.14.

4.4.6.1 Every structure shall be assigned to a Seismic Design Category in accordance with the general building code or as determined by the authority having jurisdiction in areas without a legally adopted building code.
General Requirements

18.2 General

18.2.1 Structural systems

- Structures assigned to SDC D, E, or F may be subjected to strong ground motion

- It is the intent of ACI Committee 318 that the SFRS of structural concrete buildings assigned to SDC D, E, or F be provided by special moment frames, special structural walls, or a combination

- In addition to 18.2.2 through 18.2.8, these structures also are required to satisfy requirements for continuous inspection (26.13.1.4), diaphragms (18.12), foundations (18.13), and gravity-load-resisting elements that are not designated as part of the SFRS (18.14)

18.2.1.6 Structural systems designated as part of the seismic-force-resisting system shall be restricted to those designated by the general building code, or determined by other authority having jurisdiction in areas without a legally adopted building code. Except for SDC A, for which Chapter 18 does not apply, (a) through (h) shall be satisfied for each structural system designated as part of the seismic-force-resisting system, in addition to 18.2.1.3 through 18.2.1.5:

(a) Ordinary moment frames shall satisfy 18.3
(b) Ordinary reinforced concrete structural walls need not satisfy any detailing provisions in Chapter 18, unless required by 18.2.1.3 or 18.2.1.4
(c) Intermediate moment frames shall satisfy 18.4
(d) Intermediate precast walls shall satisfy 18.5
(e) Special moment frames shall satisfy 18.2.3 through 18.2.8 and 18.6 through 18.8
(f) Special moment frames constructed using precast concrete shall satisfy 18.2.3 through 18.2.8 and 18.9
(g) Special structural walls shall satisfy 18.2.3 through 18.2.8 and 18.10
(h) Special structural walls constructed using precast concrete shall satisfy 18.2.3 through 18.2.8 and 18.11
18.2 General

18.2.1 Structural systems

- The proportioning and detailing requirements in Chapter 18 are based predominantly on field and laboratory experience with monolithic reinforced concrete building structures.

- Precast concrete building structures designed and detailed to behave like monolithic building structures.

- Extrapolation of these requirements to other types of cast-in-place or precast concrete structures should be based on evidence provided by field experience, tests, or analysis.

The acceptance criteria

- For moment frames given in ACI 374.1 can be used to demonstrate that the strength, energy dissipation capacity, and deformation capacity of a proposed frame system equals or exceeds that provided by a comparable monolithic concrete system.

- ACI ITG-5.1 provides similar information for precast wall systems.

The toughness requirement in 18.2.1.7 refers to the requirement to maintain structural integrity of the entire SFRS at lateral displacements anticipated for the MCE motion.
**General Requirements**

18.2 General

18.2.1 Structural systems

- Table R18.2 summarizes the applicability of the provisions of Chapter 18 as they are typically applied when using the minimum requirements in the various seismic design categories.
General Requirements

• 18.2 General

• 18.2.2 Analysis and proportioning of structural members

  ▫ It is assumed that distribution of required strength to the various components of a SFRS will be determined from the analysis of a linearly elastic model of the system under the factored forces

  ▫ For lateral displacement calculations, assuming all the structural members to be fully cracked is likely to lead to better estimates of the possible drift than using uncracked stiffness for all members

  ▫ In selecting member sizes for earthquake-resistant structures,
    • it is important to consider constructibility problems related to congestion of reinforcement
    • The design should be such that all reinforcement can be assembled and placed in the proper location and that concrete can be cast and consolidated properly
    • Using the upper limits of permitted reinforcement ratios may lead to construction problems
General Requirements

18.2 General

18.2.2 Analysis and proportioning of structural members

- The intent of 18.2.2.1 and 18.2.2.2 is to draw attention to the influence of nonstructural members on structural response and to hazards of falling objects.

- Section 18.2.2.3 serves as an alert that the base of structure as defined in analysis may not necessarily correspond to the foundation or ground level.

- Details of columns and walls extending below the base of structure to the foundation are required to be consistent with those above the base of structure.

18.2.2 Analysis and proportioning of structural members

18.2.2.1 The interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

18.2.2.2 Rigid members assumed not to be a part of the seismic-force-resisting system shall be permitted provided their effect on the response of the system is considered in the structural design. Consequences of failure of structural and nonstructural members that are not a part of the seismic-force-resisting system shall be considered.

18.2.2.3 Structural members extending below the base of structure that are required to transmit forces resulting from earthquake effects to the foundation shall comply with the requirements of Chapter 18 that are consistent with the seismic-force-resisting system above the base of structure.
General Requirements

• 18.2 General
• 18.2.3 Anchoring to concrete

18.2.3 Anchoring to concrete

18.2.3.1 Anchors resisting earthquake-induced forces in structures assigned to SDC C, D, E, or F shall be in accordance with 17.2.3.

17.2.3 Seismic design

17.2.3.1 Anchors in structures assigned to Seismic Design Category (SDC) C, D, E, or F shall satisfy the additional requirements of 17.2.3.2 through 17.2.3.7.
General Requirements

18.2 General

18.2.4 Strength reduction factors

The 21.2.4.1 provision addresses shear-controlled members
- Such as low-rise walls, portions of walls between openings, or diaphragms
- For which nominal shear strength is less than the shear corresponding to development of nominal flexural strength for the loading conditions

The 21.2.4.2 provision is intended to increase strength of the diaphragm and its connections in buildings for which the shear strength reduction factor for walls is 0.60, as those structures tend to have relatively high overstrength

18.2.4.1 Strength reduction factors shall be in accordance with Chapter 21.

For structures that rely on elements in (a), (b), or (c) to resist earthquake effects $E$, the value of $\phi$ for shear shall be modified in accordance with 21.2.4.1 through 21.2.4.3:

(a) Special moment frames
(b) Special structural walls
(c) Intermediate precast structural walls in structures assigned to Seismic Design Category D, E, or F

21.2.4.1 For any member designed to resist $E$, $\phi$ for shear shall be 0.60 if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal moment strength of the member. The nominal moment strength shall be calculated considering the most critical factored axial loads and including $E$.

21.2.4.2 For diaphragms, $\phi$ for shear shall not exceed the least value of $\phi$ for shear used for the vertical components of the primary seismic-force-resisting system.
General Requirements

- 21.2—Strength reduction factors for structural concrete members and connections

21.2.4.3 For beam-column joints and diagonally reinforced coupling beams, $\phi$ for shear shall be 0.85.

<table>
<thead>
<tr>
<th>Table 21.2.1—Strength reduction factors $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Action or structural element</strong></td>
</tr>
<tr>
<td>(a) Moment, axial force, or combined moment and axial force</td>
</tr>
<tr>
<td>(b) Shear</td>
</tr>
<tr>
<td>(c) Torsion</td>
</tr>
<tr>
<td>(d) Bearing</td>
</tr>
<tr>
<td>(e) Post-tensioned anchorage zones</td>
</tr>
<tr>
<td>(f) Brackets and corbels</td>
</tr>
<tr>
<td>(g) Struts, ties, nodal zones, and bearing areas designed in accordance with strut-and-tie method in Chapter 23</td>
</tr>
<tr>
<td>(h) Components of connections of precast members controlled by yielding of steel elements in tension</td>
</tr>
<tr>
<td>(i) Plain concrete elements</td>
</tr>
<tr>
<td>(j) Anchors in concrete elements</td>
</tr>
</tbody>
</table>
General Requirements

18.2 General

18.2.5 Concrete in special moment frames and special structural walls

Requirements of this section refer to concrete quality in frames and walls that resist earthquake induced forces

- The maximum specified compressive strength of lightweight concrete to be used in structural design calculations is limited to 35 MPa, primarily because of paucity of experimental and field data on the behavior of members made with lightweight concrete subjected to displacement reversals in the nonlinear range.

- The minimum specified compressive strength of concrete to be used in structural design calculations is limited to 21 MPa

Table 19.2.1.1—Limits for $f'_c$

<table>
<thead>
<tr>
<th>Application</th>
<th>Concrete</th>
<th>Minimum $f'_c$, MPa</th>
<th>Maximum $f'_c$, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td>Normalweight and lightweight</td>
<td>17</td>
<td>None</td>
</tr>
<tr>
<td>Special moment frames and special structural walls</td>
<td>Normalweight</td>
<td>21</td>
<td>None</td>
</tr>
<tr>
<td>Special moment frames and special structural walls</td>
<td>Lightweight</td>
<td>21</td>
<td>35[^1]</td>
</tr>
</tbody>
</table>

[^1]: The limit is permitted to be exceeded where demonstrated by experimental evidence that members made with lightweight concrete provide strength and toughness equal to or exceeding those of comparable members made with normalweight concrete of the same strength.
18.2 General

18.2.6 Reinforcement in special moment frames and special structural walls

- Use of longitudinal reinforcement with strength substantially higher than that assumed in design will lead to higher shear and bond stresses at the time of development of yield moments.
- These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design.
- Therefore, an upper limit is placed on the actual yield strength of the steel.

ASTM A706 for low-alloy steel reinforcing bars includes both Grade 420 and Grade 560:
- Only Grade 420 is generally permitted because of insufficient data to confirm applicability of existing code provisions for structures using the higher grade.
- Grade 560 is permitted if results of tests and analytical studies are presented in support of its use.
General Requirements

• 20.2- Nonprestressed bars and wires

  ▫ The requirement for the tensile strength to be greater than the yield strength of the reinforcement by a factor of 1.25 is based on the assumption that the capability of a structural member to develop inelastic rotation capacity is a function of the length of the yield region along the axis of the member.

  ▪ the length of the yield region has been related to the relative magnitudes of probable and yield moments.

  ▪ the greater the ratio of probable-to-yield moment, the longer the yield region.

20.2.2.4 Types of nonprestressed bars and wires to be specified for particular structural applications shall be in accordance with Table 20.2.2.4a for deformed reinforcement and Table 20.2.2.4b for plain reinforcement.

20.2.2.5 Deformed nonprestressed longitudinal reinforcement resisting earthquake-induced moment, axial force, or both, in special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers shall be in accordance with (a) or (b):

(a) ASTM A706M, Grade 420
(b) ASTM A615M Grade 280 reinforcement if (i) and (ii) are satisfied and ASTM A615M Grade 420 reinforcement if (i) through (iii) are satisfied.

(i) Actual yield strength based on mill tests does not exceed $f_y$ by more than 125 MPa
(ii) Ratio of the actual tensile strength to the actual yield strength is at least 1.25
(iii) Minimum elongation in 200 mm shall be at least 14 percent for bar sizes No. 10 through No. 19, at least 12 percent for bar sizes No. 22 through No. 36, and at least 10 percent for bar sizes No. 43 and No. 57.
General Requirements

- 20.2- Nonprestressed bars and wires
  - The restrictions on the values of $f_y$ and $f_{yt}$ apply to all types of transverse reinforcement, including spirals, circular hoops, rectilinear hoops, and crossties
  - The restrictions on the values of $f_y$ and $f_{yt}$ for calculating nominal shear strength are intended to limit the width of shear cracks
18.2 General

18.2.7 Mechanical splices in special moment frames and special structural walls

The additional requirement for a Type 2 mechanical splice is intended to result in a mechanical splice capable of sustaining inelastic strains through multiple cycles.

18.2.7.1 Mechanical splices shall be classified as (a) or (b):

(a) Type 1 – Mechanical splice conforming to 25.5.7
(b) Type 2 – Mechanical splice conforming to 25.5.7 and capable of developing the specified tensile strength of the spliced bars

18.2.7.2 Type 1 mechanical splices shall not be located within a distance equal to twice the member depth from the column or beam face for special moment frames or from critical sections where yielding of the reinforcement is likely to occur as a result of lateral displacements beyond the linear range of behavior. Type 2 mechanical splices shall be permitted at any location, except as noted in 18.9.2.1(c).

25.5.7 Mechanical and welded splices of deformed bars in tension or compression

25.5.7.1 A mechanical or welded splice shall develop in tension or compression, as required, at least 1.25fy of the bar.
18.2 General

Welded splices in special moment frames and special structural walls

- The locations of welded splices are restricted because reinforcement tension stresses in yielding regions can exceed the strength.
- The restriction on welded splices applies to all reinforcement resisting earthquake effects, including transverse reinforcement.
- Welding of crossing reinforcing bars can lead to local embrittlement of the steel.

18.2.8.1 Welded splices in reinforcement resisting earthquake-induced forces shall conform to 25.5.7 and shall not be located within a distance equal to twice the member depth from the column or beam face for special moment frames or from critical sections where yielding of the reinforcement is likely to occur as a result of lateral displacements beyond the linear range of behavior.

18.2.8.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design shall not be permitted.

25.5.7 Mechanical and welded splices of deformed bars in tension or compression

25.5.7.1 A mechanical or welded splice shall develop in tension or compression, as required, at least $1.25f_y$ of the bar.
Beams of SMRF

• 18.6 Beams of special moment frames
  • 18.6.1 Scope
    ▫ In previous Codes,
      • any frame member subjected to a factored axial compressive force exceeding \(A_g f_c'/10\) under any load combination was to be proportioned and detailed as described in 18.7
    ▫ In the 2014 Code,
      • all requirements for beams are in 18.6 regardless of the magnitude of axial compressive force
    ▫ This Code is written assuming that special moment frames comprise horizontal beams and vertical columns interconnected by beam-column joints

18.6—Beams of special moment frames

18.6.1 Scope

18.6.1.1 This section shall apply to beams of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure and shear.

18.6.1.2 Beams of special moment frames shall frame into columns of special moment frames satisfying 18.7.

• It is acceptable for
  ▫ beams and columns to be inclined provided the resulting system behaves as a frame
  ▫ beams of SMF to cantilever beyond columns, but such cantilevers are not part of the SMF
  ▫ beams of a special moment frame to connect into a wall boundary if the wall boundary is reinforced as a SMF column in accordance with 18.7
Beams of SMRF

- 18.6 Beams of special moment frames
  - 18.6.1 Scope
    - In special moment frames, it is acceptable to design beams to resist combined moment and axial force as occurs in beams that act both as moment frame members and as chords or collectors of a diaphragm
    - A concrete braced frame, in which lateral resistance is provided primarily by axial forces in beams and columns, is not a recognized seismic-force-resisting system

18.6—Beams of special moment frames
18.6.1 Scope

18.6.1.1 This section shall apply to beams of special moment frames that form part of the seismic-force-resisting system and are proportioned primarily to resist flexure and shear.

18.6.1.2 Beams of special moment frames shall frame into columns of special moment frames satisfying 18.7.
Beams of SMRF

18.6 Beams of special moment frames

18.6.2 Dimensional limits

- Experimental evidence indicates that, under reversals of displacement into the nonlinear range, behavior of continuous members having length-to-depth ratios of less than 4 is significantly different.
- Design rules derived from experience with relatively slender members do not apply directly to members with length-to-depth ratios less than 4, especially with respect to shear strength.

18.6.2 Dimensional limits

18.6.2.1 Beams shall satisfy (a) through (c):

(a) Clear span $l_n$ shall be at least $4d$
(b) Width $b_w$ shall be at least the lesser of $0.3h$ and $250\, \text{mm}$
(c) Projection of the beam width beyond the width of the supporting column on each side shall not exceed the lesser of $c_2$ and $0.75c_1$.

- Geometric constraints indicated in 18.6.2.1(b) and (c) were derived from practice and research on reinforced concrete frames resisting earthquake-induced forces.
- The limits in 18.6.2.1(c) define the maximum beam width that can effectively transfer forces into the beam-column joint.
Beams of SMRF

- 18.6 Beams of special moment frames
- 18.6.2 Dimensional limits

18.6.2 Dimensional limits

18.6.2.1 Beams shall satisfy (a) through (c):

(a) Clear span $l_n$ shall be at least 4d
(b) Width $b_w$ shall be at least the lesser of 0.3h and 250 mm
(c) Projection of the beam width beyond the width of the supporting column on each side shall not exceed the lesser of $c_2$ and 0.75$c_1$.
Beams of SMRF

• 18.6 Beams of special moment frames
  • 18.6.3 Longitudinal reinforcement
    □ The limiting reinforcement ratio of 0.025 is based primarily on considerations of reinforcement congestion and, indirectly, on limiting shear stresses in beams of typical proportions
    • Continuous bars in top and bottom are required due to reversal of seismic motions and variable live load
    • The reinforcement ratio limits insure a tension controlled failure mode in bending
    • The maximum 0.01 is more practical for constructability and for keeping joint shear forces within reasonable limits

18.6.3 Longitudinal reinforcement

18.6.3.1 Beams shall have at least two continuous bars at both top and bottom faces. At any section, for top as well as for bottom reinforcement, the amount of reinforcement shall be at least that required by 9.6.1.2 and the reinforcement ratio $\rho$ shall not exceed 0.025.

9.6.1.2 $A_{s,\text{min}}$ shall be the greater of (a) and (b), except as provided in 9.6.1.3. For a statically determinate beam with a flange in tension, the value of $b_w$ shall be the lesser of $b_f$ and $2b_w$.

\[
\begin{align*}
(a) & \quad \frac{0.25\sqrt{f'_c}}{f_y} b_w d \\
(b) & \quad \frac{1.4}{f_y} b_w d
\end{align*}
\]

\[\frac{1.4}{f_y} \leq \rho \leq 0.025\]

At least 2 bars continuous top & bottom
Beams of SMRF

- 18.6 Beams of special moment frames
- 18.6.3 Longitudinal reinforcement
- If top reinforcement area significantly exceeds bottom reinforcement area, cracks that open when the beam is flexed in negative moment (top in tension) will not close when moment is reversed.

18.6.3 Longitudinal reinforcement

18.6.3.1 Beams shall have at least two continuous bars at both top and bottom faces. At any section, for top as well as for bottom reinforcement, the amount of reinforcement shall be at least that required by 9.6.1.2 and the reinforcement ratio $\rho$ shall not exceed 0.025.

9.6.1.2 $A_{s,min}$ shall be the greater of (a) and (b), except as provided in 9.6.1.3. For a statically determinate beam with a flange in tension, the value of $b_w$ shall be the lesser of $b_f$ and $2b_w$.

\[
\begin{align*}
(a) & \quad 0.25\sqrt{f'_c} \frac{b_w d}{f_y} \\
(b) & \quad \frac{1.4}{f_y} b_w d \\
\end{align*}
\]

\[
\frac{1.4}{f_y} \leq \rho \leq 0.025
\]

At least 2 bars continuous top & bottom
Beams of SMRF

18.6 Beams of special moment frames

18.6.3 Longitudinal reinforcement

- Because the design of other frame elements depends on the amount of beam flexural reinforcement, the designer should take care to optimize each beam and minimize excess capacity.

- An objective in the design of special moment frames is to restrict yielding to specially detailed lengths of the beams.

18.6.3.2 Positive moment strength at joint face shall be at least one-half the negative moment strength provided at that face of the joint. Both the negative and the positive moment strength at any section along member length shall be at least one-fourth the maximum moment strength provided at face of either joint.

Joint face $M_{n}^{+}$ not less than 50% $M_{n}^{-}$

Min. $M_{n}^{+}$ or $M_{n}^{-}$ not less than 25% max. $M_{n}$ at joint face
Beams of SMRF

- 18.6 Beams of special moment frames
- 18.6.3 Longitudinal reinforcement
  - Lap splices of reinforcement are prohibited along lengths where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range
  - Transverse reinforcement for lap splices at any location is mandatory because of the potential of concrete cover spalling and the need to confine the splice
  - Generally, if lap splices are used, they are placed near the mid-span of the beam

18.6.3.3 Lap splices of deformed longitudinal reinforcement shall be permitted if hoop or spiral reinforcement is provided over the lap length. Spacing of the transverse reinforcement enclosing the lap-spliced bars shall not exceed the lesser of \( d/4 \) and \( 100 \text{ mm} \). Lap splices shall not be used in locations (a) through (c):

(a) Within the joints
(b) Within a distance of twice the beam depth from the face of the joint
(c) Within a distance of twice the beam depth from critical sections where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior

18.6.3.4 Mechanical splices shall conform to 18.2.7 and welded splices shall conform to 18.2.8.
Beams of SMRF

- 18.6 Beams of special moment frames
- 18.6.3 Longitudinal reinforcement
Beams of SMRF

18.6 Beams of special moment frames

18.6.4 Transverse reinforcement

- Beams in special moment frames are required to have either hoops or stirrups along the entire length.
  - Hoops fully enclose the beam cross section and are provided to confine the concrete, restrain longitudinal bar buckling, improve reinforcing bar bond, and resist shear.
  - Stirrups, which generally are not closed, are used where only shear resistance is required.
- Transverse reinforcement is required primarily to confine the concrete and maintain lateral support for the bars in regions where yielding is expected.

Beams of special moment frames can be divided into three different zones when considering where hoops and stirrups can be placed:

- The zone at each end of the beam where flexural yielding is expected to occur.
- The zone along lap-spliced bars, if any.
- The remaining length of the beam.

18.6.4.1 Hoops shall be provided in the following regions of a beam:

(a) Over a length equal to twice the beam depth measured from the face of the supporting column toward midspan, at both ends of the beam.
(b) Over lengths equal to twice the beam depth on both sides of a section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior.
Beams of SMRF

18.6 Beams of special moment frames

18.6.4 Transverse reinforcement

- An objective in the design of special moment frames is to restrict yielding to specially detailed lengths of the beams.
- If the beam is relatively short and/or the gravity loads relatively low compared with seismic design forces, beam yielding is likely to occur at the ends of the beams adjacent to the beam-column joints Figure (a).
- In contrast, if the span or gravity loads are relatively large compared with earthquake forces, then a less desirable behavior can result Figure (b).

- For members with varying strength along the span or if the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span.
Beams of SMRF

• 18.6 Beams of special moment frames

• 18.6.4 Transverse reinforcement

18.6.4.2 Where hoops are required, primary longitudinal reinforcing bars closest to the tension and compression faces shall have lateral support in accordance with 25.7.2.3 and 25.7.2.4. The spacing of transversely supported flexural reinforcing bars shall not exceed 350 mm. Skin reinforcement required by 9.7.2.3 need not be laterally supported.

25.7.2.3 Rectilinear ties shall be arranged to satisfy (a) and (b):

(a) Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees
(b) No unsupported bar shall be farther than 150 mm clear on each side along the tie from a laterally supported bar

25.7.2.4 Circular ties shall be permitted where longitudinal bars are located around the perimeter of a circle.
Beams of SMRF

- 18.6 Beams of special moment frames

18.6.4.3 Hoops in beams shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the beam, the 90-degree hooks of the crossties shall be placed on that side.
Beams of SMRF

• 18.6 Beams of special moment frames
• 18.6.4 Transverse reinforcement
  ▪ Using beam stirrups with crossties rather than closed hoops is often preferred for constructability so that the top longitudinal beam reinforcement can be placed in the field, followed by installation of the crossties

25.3.4 Seismic hooks used to anchor stirrups, ties, hoops, and crossties shall be in accordance with (a) and (b):

(a) Minimum bend of 90 degrees for circular hoops and 135 degrees for all other hoops
(b) Hook shall engage longitudinal reinforcement and the extension shall project into the interior of the stirrup or hoop

25.3.5 Crossties shall be in accordance with (a) through (e):

(a) Crosstie shall be continuous between ends
(b) There shall be a seismic hook at one end
(c) There shall be a standard hook at other end with minimum bend of 90 degrees
(d) Hooks shall engage peripheral longitudinal bars
(e) 90-degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end, unless crossties satisfy 18.6.4.3 or 25.7.1.6.1

25.7.4 Hoops

25.7.4.1 Hoops shall consist of a closed tie or continuously wound tie, which can consist of several reinforcement elements each having seismic hooks at both ends.
Beams of SMRF

• 18.6 Beams of special moment frames
• 18.6.4 Transverse reinforcement
  ▫ The upper limits were changed in the 2011 edition because of concerns about adequacy of longitudinal bar buckling restraint and confinement in large beams

18.6.4.4 The first hoop shall be located not more than 50 mm from the face of a supporting column. Spacing of the hoops shall not exceed the least of (a) through (c):

(a) $d/4$
(b) Six times the diameter of the smallest primary flexural reinforcing bars excluding longitudinal skin reinforcement required by 9.7.2.3
(c) 150 mm

18.6.4.5 Where hoops are required, they shall be designed to resist shear according to 18.6.5.
Beams of SMRF

- 18.6 Beams of special moment frames
- 18.6.4 Transverse reinforcement
  - Hoops are required along the beam end zones (where flexural yielding is expected) and along lap splices, with spacing limits
  - Elsewhere, transverse reinforcement is required at a spacing not to exceed \( d/2 \) and is permitted to be in the form of beam stirrups with seismic hooks

18.6.4.5 Where hoops are required, they shall be designed to resist shear according to 18.6.5.

18.6.4.6 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than \( d/2 \) throughout the length of the beam.
Beams of SMRF

- 18.6 Beams of special moment frames
- 18.6.4 Transverse reinforcement
Beams of SMRF

- 18.6 Beams of special moment frames
- 18.6.4 Transverse reinforcement

18.6.4.7 In beams having factored axial compressive force exceeding $\frac{A_{g}f_{c}'}{10}$, hoops satisfying 18.7.5.2 through 18.7.5.4 shall be provided along lengths given in 18.6.4.1. Along the remaining length, hoops satisfying 18.7.5.2 shall have spacing $s$ not exceeding the lesser of six times the diameter of the smallest longitudinal beam bars and 150 mm. Where concrete cover over transverse reinforcement exceeds 100 mm, additional transverse reinforcement having cover not exceeding 100 mm and spacing not exceeding 300 mm shall be provided.
Beams of SMRF

• 18.6 Beams of special moment frames
• 18.6.5 Shear strength
  ▫ Unless a beam possesses a moment strength that is on the order of 3 or 4 times the design moment, it should be assumed that it will yield in flexure in the event of a major earthquake
  ▫ The design shear force should be selected so as to be a good approximation of the maximum shear that may develop in member
• Required shear strength for frame members is related to flexural strengths of the designed member rather than to factored shear forces indicated by lateral load analysis.

18.6.5 Shear strength

18.6.5.1 Design forces—The design shear force $V_e$ shall be calculated from consideration of the forces on the portion of the beam between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable flexural strength, $M_{pr}$, act at the joint faces and that the beam is loaded with the factored tributary gravity load along its span.
Beams of SMRF

18.6 Beams of special moment frames

- For a typical beam in a special moment frame, the resulting beam shears do not trend to zero near midspan as they would in a gravity-only beam.

- Typical practice for gravity-load design of beams is to take the design shear at \( d \) away from the column face.

- For special moment frames, the shear gradient typically is low. Thus, for simplicity the design shear value usually is evaluated at the column face.
Beams of SMRF

18.6 Beams of special moment frames

18.6.5 Shear strength

- Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, required shear strengths are determined using a stress of at least 1.25$\,f_y$ in the longitudinal reinforcement.

- For simplicity the design shear value usually is evaluated at the column face.

- Probable moment strength $M_{pr}$ is calculated from conventional flexural theory considering the as-designed cross section, using $\phi = 1.0$, and assuming reinforcement yield strength equal to at least 1.25 $f_y$

$$M_{pr} = M_n \quad \text{with} \quad f_s = 1.25f_y, \quad \phi = 1.0$$
Beams of SMRF

18.6 Beams of special moment frames

18.6.5 Shear strength

- For beams (small axial load), concrete shear strength is neglected when the earthquake-induced shear force represents one half or more of the design shear strength of the beam.

- Experimental studies of RC members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure if the member is subjected to alternating nonlinear displacements than if the member is loaded in only one direction.

18.6.5.2 Transverse reinforcement—Transverse reinforcement over the lengths identified in 18.6.4.1 shall be designed to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

(a) The earthquake-induced shear force calculated in accordance with 18.6.5.1 represents at least one-half of the maximum required shear strength within those lengths.

(b) The factored axial compressive force $P_u$ including earthquake effects is less than $A_g f'_c / 20$.

\[
\text{If earthquake-induced shear force} > \frac{1}{2} V_e \quad \text{and} \quad P_u < \frac{A_g f'_c}{20}
\]

- Along the beam end zones, the shear design requirement typically is $\phi V_s > V_e$, where $\phi = 0.75$.

- Outside the end zones, design for shear is done using the conventional design equation $\phi (V_c + V_s) > V_e$. 
Beams of SMRF

• 18.6 Beams of special moment frames
• 18.6.5 Shear strength

\[
V_e = \frac{M_{pr1} + M_{pr2}}{\ell_n} + \frac{w_u \ell_n}{2} \geq V_e \quad \text{by analysis}
\]

If earthquake-induced shear force \( > \frac{1}{2} V_e \)

then \( V_c = 0 \)

and \( P_u < \frac{A_g f_c'}{20} \)
Columns of SMRF

18.7 Columns of special moment frames

18.7.1 Scope

- This section applies to columns of special moment frames regardless of the magnitude of axial force
- Before 2014, the Code permitted columns with low levels of axial stress to be detailed as beams
Columns of special moment frames

18.7.2 Dimensional limits

The geometric constraints in this provision follow from previous practice:

- The ratio of the cross-sectional dimensions for columns shall not be less than 0.4
- This limits the cross section to a more compact section rather than a long rectangle
- The minimum column dimension to 300 mm, which is often not practical to construct
- A minimum dimension of 400 mm is suggested, except for unusual cases or for low-rise buildings

18.7.2.1 Columns shall satisfy (a) and (b):

(a) The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall be at least 300 mm
(b) The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall be at least 0.4.
Columns of SMRF

- 18.7 Columns of special moment frames
- 18.7.3 Minimum flexural strength of columns
  - The intent of 18.7.3.2 is to reduce the likelihood of yielding in columns that are considered as part of the seismic-force-resisting system
  - If columns are not stronger than beams framing into a joint, there is increased likelihood of inelastic action
  - In the worst case of weak columns, flexural yielding can occur at both ends of all columns in a given story, resulting in a column failure mechanism that can lead to collapse

18.7.3 Minimum flexural strength of columns

18.7.3.1 Columns shall satisfy 18.7.3.2 or 18.7.3.3.

18.7.3.2 The flexural strengths of the columns shall satisfy

$$\sum M_{nc} \geq (6/5)\sum M_{nb}$$

(18.7.3.2)

where

- $\sum M_{nc}$ is the sum of nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.
- $\sum M_{nb}$ is the sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width defined in accordance with 6.3.2 shall be assumed to contribute to $M_{nb}$ if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation (18.7.3.2) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.
Columns of SMRF

18.7 Columns of special moment frames

18.7.3 Minimum flexural strength of columns

- In 18.7.3.2, the nominal strengths of the beams and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (18.7.3.2)
- The 1995 and earlier Codes required design strengths to be compared at the center of the joint, which typically produced similar results but with added calculation effort
- In determining the nominal moment strength of a beam section in negative bending (top in tension), longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the beam increases the beam strength
  - The effective flange widths defined in 6.3.2 gives reasonable estimates of beam negative moment strengths of interior connections at story displacements approaching 2 percent of story height
  - This effective width is conservative where the slab terminates in a weak spandrel
Columns of SMRF

18.7 Columns of special moment frames

18.7.3 Minimum flexural strength of columns

- This check must be verified independently for sway in both directions and in each of the two principal framing directions
- When this flexural strength check is done, consideration needs to be given to the maximum and minimum axial loads in the column, because the column flexural strength is dependent on the axial load
Columns of SMRF

18.7 Columns of special moment frames
18.7.3 Minimum flexural strength of columns
  - T-beam geometry

6.3.2 T-beam geometry

6.3.2.1 For nonprestressed T-beams supporting monolithic or composite slabs, the effective flange width $b_f$ shall include the beam web width $b_w$ plus an effective overhanging flange width in accordance with Table 6.3.2.1, where $h$ is the slab thickness and $s_w$ is the clear distance to the adjacent web.

Table 6.3.2.1—Dimensional limits for effective overhanging flange width for T-beams

<table>
<thead>
<tr>
<th>Flange location</th>
<th>Effective overhanging flange width, beyond face of web</th>
</tr>
</thead>
<tbody>
<tr>
<td>Each side of web</td>
<td>Least of: $8h$, $s_w/2$, $\ell_n/8$</td>
</tr>
<tr>
<td>One side of web</td>
<td>Least of: $6h$, $s_w/2$, $\ell_n/12$</td>
</tr>
</tbody>
</table>
18.7 Columns of special moment frames
- 18.7.3 Minimum flexural strength of columns
  - In some cases it may not be practical to satisfy the strong column/weak-beam provisions for a small number of columns
  - If 18.7.3.2 cannot be satisfied at a joint, 18.7.3.3 requires that any positive contribution of the column or columns involved to the lateral strength and stiffness of the structure is to be ignored
  - This column must be provided with transverse reinforcement to increase its resistance to shear and axial forces

18.7.3.3 If 18.7.3.2 is not satisfied at a joint, the lateral strength and stiffness of the columns framing into that joint shall be ignored when calculating strength and stiffness of the structure. These columns shall conform to 18.14.

- Negative contributions of the column or columns should not be ignored
  - For example, ignoring the stiffness of the columns ought not to be used as a justification for reducing the design base shear
  - If inclusion of those columns in the analytical model of the building results in an increase in torsional effects, the increase should be considered as required by the general building code
Columns of SMRF

• 18.7 Columns of special moment frames

• 18.7.4 Longitudinal reinforcement
  ▫ The lower limit of the area of longitudinal reinforcement is to control time dependent deformations and to have the yield moment exceed the cracking moment.
  ▫ The upper limit of the area reflects concern for reinforcement congestion, load transfer from floor elements to column (especially in low-rise construction) and the development of high shear stresses.

18.7.4 Longitudinal reinforcement

18.7.4.1 Area of longitudinal reinforcement, $A_{st}$, shall be at least 0.014g and shall not exceed 0.064g.

18.7.4.2 In columns with circular hoops, there shall be at least six longitudinal bars.

▫ ACI 318 allows the longitudinal reinforcement to reach 6 % of the gross section area, but this amount of reinforcement results in very congested splice locations.
▫ The use of mechanical couplers should be considered where the reinforcement ratio is in excess of 3 %.
Columns of SMRF

18.7 Columns of special moment frames

18.7.4 Longitudinal reinforcement

- Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in these locations vulnerable.

- If lap splices are to be used at all, they should be located near the midheight where stress reversal is likely to be limited to a smaller stress range than at locations near the joints.

- Transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height and the need for confinement of lap splices subjected to stress reversals.

18.7.4.3 Mechanical splices shall conform to 18.2.7 and welded splices shall conform to 18.2.8. Lap splices shall be permitted only within the center half of the member length, shall be designed as tension lap splices, and shall be enclosed within transverse reinforcement in accordance with 18.7.5.2 and 18.7.5.3.
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

- This section is concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.
- This section stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the column ends, where flexural yielding normally occurs.

- Research results indicate that the length should be increased by 50 percent or more in locations, such as the base of a building, where axial loads and flexural demands may be especially high.

18.7.5.1 Transverse reinforcement required in 18.7.5.2 through 18.7.5.4 shall be provided over a length $l_o$ from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of lateral displacements beyond the elastic range of behavior. Length $l_o$ shall be at least the greatest of (a) through (c):

(a) The depth of the column at the joint face or at the section where flexural yielding is likely to occur
(b) One-sixth of the clear span of the column
(c) 450 mm
Columns of SMRF

• **18.7 Columns of special moment frames**
  
  18.7.5 Transverse reinforcement
  
  - **Crossties with a 90-degree hook are not as effective as either crossties with 135-degree hooks or hoops in providing confinement**
  - **For lower values of \(P_u/A_g f_c\)′ and lower concrete compressive strengths, crossties with 90-degree hooks are adequate if the ends are alternated along the length and around the perimeter of the column.**

---

**18.7.5.2 Transverse reinforcement shall be in accordance with (a) through (f):**

(a) Transverse reinforcement shall comprise either single or overlapping spirals, circular hoops, or rectilinear hoops with or without crossties.

(b) Bends of rectilinear hoops and crossties shall engage peripheral longitudinal reinforcing bars.

(c) Crossties of the same or smaller bar size as the hoops shall be permitted, subject to the limitation of 25.7.2.2. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section.

---

**25.7.2.2 Diameter of tie bar shall be at least (a) or (b):**

(a) No.10 enclosing No. 32\(^1\) or smaller longitudinal bars
(b) No.12 enclosing No. 36 or larger longitudinal bars or bundled longitudinal bars
Columns of SMRF

• 18.7 Columns of special moment frames
  • 18.7.5 Transverse reinforcement
    ▪ For higher values of $P_u/A_g f'_c$, for which compression-controlled behavior is expected, and for higher compressive strengths, for which behavior tends to be more brittle, the improved confinement provided by having corners of hoops or seismic hooks supporting all longitudinal bars is important to achieving intended performance
      • Crossties with seismic hooks at both ends are required
      • The 200 mm limit on $h_x$ is also intended to improve performance under these critical conditions

(d) Where rectilinear hoops or crossties are used, they shall provide lateral support to longitudinal reinforcement in accordance with 25.7.2.2 and 25.7.2.3.
(e) Reinforcement shall be arranged such that the spacing $h_x$ of longitudinal bars laterally supported by the corner of a crosstie or hoop leg shall not exceed 350 mm around the perimeter of the column.
(f) Where $P_u > 0.3 A_g f'_c$ or $f'_c > 70$ MPa in columns with rectilinear hoops, every longitudinal bar or bundle of bars around the perimeter of the column core shall have lateral support provided by the corner of a hoop or by a seismic hook, and the value of $h_x$ shall not exceed 200 mm $P_u$ shall be the largest value in compression consistent with factored load combinations including $E$.

25.7.2.3 Rectilinear ties shall be arranged to satisfy (a) and (b):

(a) Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees
(b) No unsupported bar shall be farther than 150 mm clear on each side along the tie from a laterally supported bar
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

- For bundled bars, bends or hooks of hoops and crossties need to enclose the bundle, and longer extensions on hooks should be considered.
- Column axial load $P_u$ should reflect factored compressive demands from both earthquake and gravity loads.
- In the 2014 edition of the Code, $h_x$ refers to the distance between longitudinal bars supported by those hoops or crossties.
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

- Column hoops should be configured with at least three hoop or crosstie legs restraining longitudinal bars along each face
- A single perimeter hoop without crossties, legally permitted by ACI 318 for small column cross sections, is discouraged because confinement effectiveness is low

![Diagram of different confinement levels: Poorly confined, Improved confinement, Well confined]
18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

- The requirement that spacing not exceed one fourth of the minimum member dimension is to obtain adequate concrete confinement
- The requirement that spacing not exceed six bar diameters is intended to restrain longitudinal reinforcement buckling after spalling
- The 100 mm spacing is for concrete confinement; the equation permits this limit to be relaxed to a maximum of 150 mm if the spacing of crossties or legs of overlapping hoops is 200 mm or less.

\[ s_o = 100 + \left( \frac{350 - h_x}{3} \right) \]  

Equation (18.7.5.3)

The value of \( s_o \) from Eq. (18.7.5.3) shall not exceed 150 mm and need not be taken less than 100 mm.
Columns of SMRF

• 18.7 Columns of special moment frames
• 18.7.5 Transverse reinforcement

<table>
<thead>
<tr>
<th>Transverse reinforcement</th>
<th>Conditions</th>
<th>Applicable expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{sh}/sb_c$ for rectilinear hoop</td>
<td>$P_u \leq 0.3A_{g}f'_c$ and $f'_c \leq 70$ MPa</td>
<td>Greater of (a) and (b)</td>
</tr>
<tr>
<td></td>
<td>$P_u &gt; 0.3A_{g}f'_c$ or $f'_c &gt; 70$ MPa</td>
<td>Greatest of (a), (b), and (c)</td>
</tr>
<tr>
<td></td>
<td>$A_{sh}/sb_c$ for rectilinear hoop</td>
<td>$0.3\left(\frac{A_{sh}}{A_{ch}} - 1\right)\frac{f'<em>c}{f</em>{yl}}$ (a)</td>
</tr>
<tr>
<td></td>
<td>$P_u \leq 0.3A_{g}f'_c$ and $f'_c \leq 70$ MPa</td>
<td>$0.09\frac{f'<em>c}{f</em>{yl}}$ (b)</td>
</tr>
<tr>
<td></td>
<td>$P_u &gt; 0.3A_{g}f'_c$ or $f'_c &gt; 70$ MPa</td>
<td>$0.2k_f k_n \frac{P_u}{f_{yl} A_{ch}}$ (c)</td>
</tr>
<tr>
<td>$\rho_s$ for spiral or circular hoop</td>
<td>$P_u \leq 0.3A_{g}f'_c$ and $f'_c \leq 70$ MPa</td>
<td>Greater of (d) and (e)</td>
</tr>
<tr>
<td></td>
<td>$P_u &gt; 0.3A_{g}f'_c$ or $f'_c &gt; 70$ MPa</td>
<td>Greatest of (d), (e), and (f)</td>
</tr>
<tr>
<td></td>
<td>$A_{sh}/sb_c$ for rectilinear hoop</td>
<td>$0.45\left(\frac{A_{sh}}{A_{ch}} - 1\right)\frac{f'<em>c}{f</em>{yl}}$ (d)</td>
</tr>
<tr>
<td></td>
<td>$0.12\frac{f'<em>c}{f</em>{yl}}$ (e)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$0.35k_f \frac{P_u}{f_{yl} A_{ch}}$ (f)</td>
<td></td>
</tr>
</tbody>
</table>

18.7.5.4 Amount of transverse reinforcement shall be in accordance with Table 18.7.5.4.

The concrete strength factor $k_f$ and confinement effectiveness factor $k_n$ are calculated according to Eq. (18.7.5.4a) and (18.7.5.4b).

(a) $k_f = \frac{f'_c}{175} + 0.6 \geq 1.0$ 0

(b) $k_n = \frac{n_l}{n_l - 2}$

where $n_l$ is the number of longitudinal bars or bar bundles around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks.
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

Expressions (a), (b), (d), and (e) have historically been used in ACI 318 to calculate the required confinement reinforcement to ensure that spalling of shell concrete does not result in a loss of column axial load strength.

Expressions (c) and (f) were developed from a review of column test data and are intended to result in columns capable of sustaining a drift ratio of 0.03 with limited strength degradation.

Expressions (c) and (f) are triggered for axial load greater than \(0.3A_g f_c'\), which corresponds approximately to the onset of compression-controlled behavior for symmetrically reinforced columns.

<table>
<thead>
<tr>
<th>Transverse reinforcement</th>
<th>Conditions</th>
<th>Applicable expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A_{zh}/5b_c) for rectilinear hoop</td>
<td>(P_u \leq 0.3A_g f_c') and (f_c' \leq 70) MPa</td>
<td>Greater of (a) and (b)</td>
</tr>
<tr>
<td></td>
<td>(P_u &gt; 0.3A_g f_c') or (f_c' &gt; 70) MPa</td>
<td>0.3 (\frac{A_z}{A_{zh}} - 1) (\frac{f_c'}{f_{cy}}) (a)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.09 (\frac{f_c'}{f_{cy}}) (b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.2k'k_n (P_u) (\frac{f_{cy}}{f_{cy A_{zh}}}) (c)</td>
</tr>
<tr>
<td>(\rho_c) for spiral or circular hoop</td>
<td>(P_u \leq 0.3A_g f_c') and (f_c' \leq 70) MPa</td>
<td>Greater of (d) and (e)</td>
</tr>
<tr>
<td></td>
<td>(P_u &gt; 0.3A_g f_c') or (f_c' &gt; 70) MPa</td>
<td>0.45 (\frac{A_z}{A_{zh}} - 1) (\frac{f_c'}{f_{cy}}) (d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.12 (\frac{f_c'}{f_{cy}}) (e)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.35k' (P_u) (\frac{f_{cy}}{f_{cy A_{zh}}}) (f)</td>
</tr>
</tbody>
</table>
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

- The $k_n$ term decreases the required confinement for columns with closely spaced, laterally supported longitudinal reinforcement because such columns are more effectively confined than columns with more widely spaced longitudinal reinforcement.

- The $k_f$ term increases the required confinement for columns with $f_{c'} > 70$ MPa because such columns can have brittle failure if not well confined.

Concrete strengths greater than 100 MPa should be used with caution given the limited test data for such columns.

<table>
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<tr>
<td>$A_{zh}/sb_c$ for rectilinear hoop</td>
<td>$P_u \leq 0.3A_yf_{c'}$ and $f_{c'} \leq 70$ MPa</td>
<td>$0.3\left(\frac{A_y}{A_{zh}} - 1\right) \frac{f_{c'}}{f_{yv}}$ (a)</td>
</tr>
<tr>
<td></td>
<td>$P_u &gt; 0.3A_yf_{c'}$ or $f_{c'} &gt; 70$ MPa</td>
<td>$0.09\frac{f_{c'}}{f_{yv}}$ (b)</td>
</tr>
<tr>
<td></td>
<td>$P_u \leq 0.3A_yf_{c'}$ and $f_{c'} \leq 70$ MPa</td>
<td>$0.2k_fk_n\frac{P_u}{f_{yv}A_{zh}}$ (c)</td>
</tr>
<tr>
<td>$\rho_f$ for spiral or circular hoop</td>
<td>$P_u &gt; 0.3A_yf_{c'}$ or $f_{c'} &gt; 70$ MPa</td>
<td>$0.45\left(\frac{A_y}{A_{zh}} - 1\right) \frac{f_{c'}}{f_{yv}}$ (d)</td>
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<tr>
<td></td>
<td>$P_u \leq 0.3A_yf_{c'}$ and $f_{c'} \leq 70$ MPa</td>
<td>$0.12\frac{f_{c'}}{f_{yv}}$ (e)</td>
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<tr>
<td></td>
<td>$P_u &gt; 0.3A_yf_{c'}$ or $f_{c'} &gt; 70$ MPa</td>
<td>$0.35k_f\frac{P_u}{f_{yv}A_{zh}}$ (f)</td>
</tr>
</tbody>
</table>
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

Expressions (a), (b), and (c) are to be satisfied in both cross-sectional directions of the rectangular core

- For each direction, $b_c$ is the core dimension perpendicular to the tie legs that constitute $A_{sh}$
- The column transverse reinforcement should initially be selected based on the confinement requirements of 18.7.5
  - $A_g$ = gross area of column
  - $A_{ch}$ = area confined within the hoops
  - $b_c$ = transverse dimension of column core, center to center of outer legs
  - $s$ = hoop spacing

Table 18.7.5.4—Transverse reinforcement for columns of special moment frames

<table>
<thead>
<tr>
<th>Transverse reinforcement</th>
<th>Conditions</th>
<th>Applicable expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{sh}/s b_c$ for rectilinear hoop</td>
<td>$P_u \leq 0.3 A_{eff} f' c$ and $f' c \leq 70$ MPa</td>
<td>Greater of (a) and (b): $0.3 \left( \frac{A_{sh}}{A_{ch}} - 1 \right) \frac{f' c}{f_{yt}}$ (a)</td>
</tr>
<tr>
<td></td>
<td>$P_u &gt; 0.3 A_{eff} f' c$ or $f' c &gt; 70$ MPa</td>
<td>Greatest of (a), (b), and (c): $0.09 \frac{f' c}{f_{yt}}$ (b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.2 k_r k_n \frac{P_u}{f_{yt} A_{ch}}$ (c)</td>
</tr>
</tbody>
</table>
Columns of SMRF

• 18.7 Columns of special moment frames
  • 18.7.5 Transverse reinforcement
    ▪ This provision is intended to provide reasonable protection to the midheight of columns outside the length \( l_0 \)
    ▪ Observations after earthquakes have shown significant damage to columns in this region, and the minimum hoops or spirals required should provide more uniform strength of the column along its length

18.7.5.5 Beyond the length \( l_0 \) given in 18.7.5.1, the column shall contain spiral or hoop reinforcement satisfying 25.7.2 through 25.7.4 with spacing \( s \) not exceeding the lesser of six times the diameter of the smallest longitudinal column bars and 150 mm, unless a greater amount of transverse reinforcement is required by 18.7.4.3 or 18.7.6.

25.7.2 Ties

25.7.2.1 Ties shall consist of a closed loop of deformed bar with spacing in accordance with (a) and (b):

(a) Clear spacing of at least \((4/3)d_{agg}\)
(b) Center-to-center spacing shall not exceed the least of \(16d_b\) of longitudinal bar, \(48d_b\) of tie bar, and smallest dimension of member

25.7.2.2 Diameter of tie bar shall be at least (a) or (b):

(a) No. 10 enclosing No. 321 or smaller longitudinal bars
(b) No. 12 enclosing No. 36 or larger longitudinal bars or bundled longitudinal bars
Columns of SMRF

18.7 Columns of special

25.7.3.2 For cast-in-place construction, spiral bar or wire diameter shall be at least 30 mm.

25.7.3.3 Volumetric spiral reinforcement ratio \( \rho_s \) shall satisfy Eq. (25.7.3.3).

\[
\rho_s \geq 0.45 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}
\]  \hspace{1cm} (25.7.3.3)

where the value of \( f_{yt} \) shall not be taken greater than 70 MPa.

25.7.4 Hoops

25.7.4.1 Hoops shall consist of a closed tie or continuously wound tie, which can consist of several reinforcement elements each having seismic hooks at both ends.

25.7.4.2 The ends of the reinforcement elements in hoops shall be anchored using seismic hooks that conform to 25.3.4 and engage a longitudinal bar. A hoop shall not be made up of interlocking headed deformed bars.

25.7.2.3 Rectilinear ties shall be arranged to satisfy (a) and (b):

(a) Every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees
(b) No unsupported bar shall be farther than 150 mm. clear on each side along the tie from a laterally supported bar

25.7.2.4 Circular ties shall be permitted where longitudinal bars are located around the perimeter of a circle.

25.7.3 Spirals

25.7.3.1 Spirals shall consist of evenly spaced continuous bar or wire with clear spacing conforming to (a) and (b):

(a) At least the greater of 25 mm and \( (4/3) d_{agg} \)
(b) Not greater than 75 mm

25.7.3.2 For cast-in-place construction, spiral bar or wire diameter shall be at least 10 mm.
Columns of SMRF

18.7 Columns of special moment frames

- Longitudinal reinforcement satisfies $0.01 \leq A_{vl}/A_g \leq 0.06$.
- Transverse reinforcement is spirals, circular hoops, or rectilinear hoops and crossties, designed to resist shear as required.
- Rectilinear hoops and crossties engage at least corner and alternate longitudinal bars, with no unsupported bar more than 6 in (150 mm) clear from a supported bar, and with spacing $h_x$ of supported longitudinal bars not exceeding 14 in (360 mm) on center.
- For columns with $P_u > 0.3A_g f_v^*$ or $f_v^* > 10,000$ psi (70 MPa), every longitudinal bar around the perimeter shall be supported by the corner of a hoop or by a crosstie seismic hook, in either case having included angle not exceeding $135^\circ$, with $h_x \leq 8$ in (200 mm).
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

- Columns supporting discontinued stiff members, such as walls or trusses, may develop considerable inelastic response
- These columns have the specified reinforcement throughout their length

---

18.7.5.6 Columns supporting reactions from discontinued stiff members, such as walls, shall satisfy (a) and (b):

(a) Transverse reinforcement required by 18.7.5.2 through 18.7.5.4 shall be provided over the full height at all levels beneath the discontinuity if the factored axial compressive force in these columns, related to earthquake effect, exceeds $A_{gf} c'/10$. Where design forces have been magnified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $A_{gf} c'/10$ shall be increased to $A_{gf} c'/4$.

(b) Transverse reinforcement shall extend into the discontinued member at least $l_d$ of the largest longitudinal column bar, where $l_d$ is in accordance with 18.8.5. Where the lower end of the column terminates on a wall, the required transverse reinforcement shall extend into the wall at least $l_d$ of the largest longitudinal column bar at the point of termination. Where the column terminates on a footing or mat, the required transverse reinforcement shall extend at least 300 mm into the footing or mat.
Columns of SMRF

18.7 Columns of special moment frames

18.7.5 Transverse reinforcement

- The unreinforced shell may spall as the column deforms to resist earthquake effects
- Separation of portions of the shell from the core caused by local spalling creates a falling hazard
- The additional reinforcement is required to reduce the risk of portions of the shell falling away from the column

18.7.5.7 If the concrete cover outside the confining transverse reinforcement required by 18.7.5.1, 18.7.5.5, and 18.7.5.6 exceeds 100 mm, additional transverse reinforcement having cover not exceeding 100 mm and spacing not exceeding 300 mm shall be provided.
Columns of SMRF

18.7 Columns of special moment frames

18.7.6 Shear strength

Three distinct procedures for calculating design shear of columns are given:

a) $V_e$ shall not be taken less than the shear obtained by analysis of the building frame $V_{\text{code}}$ considering the governing design load combinations.

b) $V_e$ can be determined using the capacity design approach.

c) By this alternative column design shear can be taken equal to the shear determined from joint strengths based on $M_{pr}$ of the beams framing into the joint.

18.7.6.1 Design forces

18.7.6.1.1 The design shear force $V_e$ shall be calculated from considering the maximum forces that can be generated at the faces of the joints at each end of the column. These joint forces shall be calculated using the maximum probable flexural strengths, $M_{pr}$, at each end of the column associated with the range of factored axial forces, $P_u$, acting on the column. The column shears need not exceed those calculated from joint strengths based on $M_{pr}$ of the beams framing into the joint. In no case shall $V_e$ be less than the factored shear calculated by analysis of the structure.
Columns of SMRF

- 18.7 Columns of special moment frames
- 18.7.6 Shear strength
Columns of SMRF

- 18.7 Columns of special moment frames
- 18.7.6 Shear strength
Columns of SMRF

- 18.7 Columns of special moment frames
- 18.7.6 Shear strength
  - Above the ground floor, the moment at a joint may be limited by the flexural strength of the beams framing into the joint
  - Where beams frame into opposite sides of a joint, the combined strength is the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint
  - Distribution of the combined moment strength of the beams to the columns above and below the joint should be based on analysis
  - A common assumption is to distribute the moments to the columns in proportion with column flexural rigidity

- Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcement with an effective yield stress equal to at least 1.25fy
- $M_{pr}$ is to be taken equal to the maximum value associated with the anticipated range of axial forces
Columns of SMRF

• 18.7 Columns of special moment frames
  • 18.7.6 Shear strength
    ▫ The probable moment strength is to be the maximum consistent with the range of factored axial loads on the column
    ▪ Sidesway to the right and to the left must both be considered
    ▪ It is obviously conservative to use the probable moment strength corresponding to the balanced point
Columns of SMRF

18.7 Columns of special moment frames

18.7.6 Shear strength

Assuming that Mpr develops at both ends of the column simultaneously may be excessively conservative,

An alternative sometimes used is to assume that the frame develops its intended beam-yielding mechanism, and then calculate the column shear corresponding to development of Mpr of the beams framing into the joints.
Columns of SMRF

• 18.7 Columns of special moment frames

• 18.7.6 Shear strength
  ▫ The design shear strength for the column is \( \phi (V_c + V_s) > V_e \), with \( \phi = 0.75 \)
  • \( V_c \) must be set to zero over the length of \( l_0 \), for any load combination for which the column has low axial load (\(< A_g f_c/20\)) and high seismic shear demand (\( V_e = V_u/2 \))
  • Note that both of these conditions must occur to require \( V_c = 0 \).

18.7.6.2 Transverse reinforcement

18.7.6.2.1 Transverse reinforcement over the lengths \( l_o \), given in 18.7.5.1, shall be designed to resist shear assuming \( V_c = 0 \) when both (a) and (b) occur:

(a) The earthquake-induced shear force, calculated in accordance with 18.7.6.1, is at least one-half of the maximum required shear strength within \( l_o \).

(b) The factored axial compressive force \( P_u \) including earthquake effects is less than \( A_g f_c'/20 \).
Joints of SMRF

• 18.8 Joints of special moment frames

• 18.8.1 Scope
  ▫ The overall integrity of a structure is dependent on the behavior of the beam-column joint
  ▫ Degradation of the joint can result in large lateral deformations which can cause excessive damage or even failure
  ▫ Joint shear is a critical check and will often govern the size of the moment frame columns

18.8—Joints of special moment frames

18.8.1 Scope

18.8.1.1 This section shall apply to beam-column joints of special moment frames forming part of the seismic-force-resisting system.

As part of the frame design, it is assumed that the beams framing into the column will yield and develop their probable moment strengths at the column faces

This action determines the demands on the column and the beam column joint
18.8 Joints of special moment frames

18.8.2 General

- Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain.
- Joint shear force generated by the flexural reinforcement is calculated for a stress of $1.25f_y$ in the reinforcement.
- Assuming the beam to have zero axial load, the flexural compression force in the beam on one side of the joint is taken equal to the flexural tension force on the same side of the joint.
Joints of SMRF

- 18.8 Joints of special moment frames
  - 18.8.2 General
    - Use a free body diagram is made by cutting through the beam plastic hinges on both sides of the column and cutting through the column one-half story height above and below the joint.
    - In this figure, subscripts A and B refer to beams A and B on opposite sides of the joint, and $V_{e2,A}$ and $V_{e1,B}$ are shears in the beams at the joint face corresponding to development of $M_{pr}$ at both ends of the beam.
    - For a typical story, the column midheight provides good approximation to the point of contraflexure.
Joints of SMRF

18.8 Joints of special moment frames

18.8.2 General

- The design provisions for hooked bars are based mainly on research and experience for joints with standard 90-degree hooks
- Standard 90-degree hooks generally are preferred to standard 180-degree hooks unless unusual considerations dictate use of 180-degree hooks
- For bars in compression, the development length corresponds to the straight portion of a hooked or headed bar measured from the critical section to the onset of the bend for hooked bars and from the critical section to the head for headed bars

18.8.2.2 Beam longitudinal reinforcement terminated in a column shall extend to the far face of the confined column core and shall be developed in tension in accordance with 18.8.5 and in compression in accordance with 25.4.9.

25.4.9 Development of deformed bars and deformed wires in compression

25.4.9.1 Development length $l_{de}$ for deformed bars and deformed wires in compression shall be the greater of (a) and (b)

(a) Length calculated in accordance with 25.4.9.2
(b) 200 mm

25.4.9.2 $l_{de}$ shall be the greater of (a) and (b), using the modification factors of 25.4.9.3:

\[
(a) \left( \frac{0.24f_y\psi_r}{\lambda\sqrt{f'_c}} \right) d_b
\]

(b) $0.043f_y\psi_r d_b$
Joints of SMRF

18.8 Joints of special moment frames

18.8.2 General

- This requirement is to ensure the full depth of the joint is used to resist the joint shear generated by anchorage of the hooked bars
- For exterior joints, beam longitudinal reinforcement usually terminates in the joint with a standard hook
- The tail of the hook must project toward the mid-depth of the joint so that a joint diagonal compression strut can be developed

- It is common practice to hold the hooks back 25 mm from the perimeter hoops of the joint to improve concrete placement
Joints of SMRF

- 18.8 Joints of special moment frames
- 18.8.2 General

18.8.2.2 Beam longitudinal reinforcement terminated in a column shall extend to the far face of the confined column core and shall be developed in tension in accordance with 18.8.5 and in compression in accordance with 25.4.9.
18.8 Joints of special moment frames

18.8.2 General

- This requirement helps improve performance of the joint by resisting slip of the beam bars through the joint.
- Some slip, however, will occur even with this column dimension requirement.
- Tests has shown that straight beam bars may slip within the beam-column joint during a series of large moment reversals.
- The bond stresses on these straight bars may be very large.

18.8.2.3 Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall be at least 20 times the diameter of the largest longitudinal beam bar for normalweight concrete or 26 times the diameter of the largest longitudinal bar for lightweight concrete.

- To reduce slip substantially during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 32, which would result in very large joints.
- On reviewing the available tests, the required minimum ratio of column depth to maximum beam longitudinal bar diameter was set at 20 for normalweight concrete and 26 for lightweight concrete.
Joints of SMRF

- 18.8 Joints of special moment frames
- 18.8.2 General

18.8.2.3 Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall be at least 20 times the diameter of the largest longitudinal beam bar for normalweight concrete or 26 times the diameter of the largest longitudinal bar for lightweight concrete.
18.8 Joints of special moment frames

18.8.2 General

- The requirement on joint aspect ratio applies only to beams that are designated as part of the seismic-force-resisting system.
- Joints having depth less than half the beam depth require a steep diagonal compression strut across the joint, which may be less effective in resisting joint shear.
- Tests to demonstrate performance of such joints have not been reported in the literature.

**18.8.2.4** Depth $h$ of the joint shall not be less than one-half of depth $h$ of any beam framing into the joint and generating joint shear as part of the seismic-force-resisting system.
Joints of SMRF

18.8 Joints of special moment frames

18.8.3 Transverse reinforcement

- Joint transverse reinforcement is provided to confine the joint core and improve anchorage of the beam and column longitudinal reinforcement
- The amount of transverse hoop reinforcement in the joint is to be the same as the amount provided in the adjacent column end regions
- The Code requires transverse reinforcement in a joint regardless of the magnitude of the calculated shear force
Joints of SMRF

18.8 Joints of special moment frames

18.8.3 Transverse reinforcement

- The amount of confining reinforcement may be reduced and the spacing may be increased if beams of adequate dimensions frame into all four sides of the joint.

- Where beams frame into all four sides of the joint, and where each beam width is at least three-fourths the column width, then transverse reinforcement within the depth of the shallowest framing member may be relaxed to one-half the amount required in the column end regions, provided the maximum spacing does not exceed 150 mm.
Joints of SMRF

- 18.8 Joints of special moment frames
- 18.8.3 Transverse reinforcement
  - The required transverse reinforcement, or transverse beam if present, is intended to confine the beam longitudinal reinforcement and improve force transfer to the beam-column joint.

18.8.3.3 Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of 18.6.4.4, and requirements of 18.6.4.2, and 18.6.4.3, if such confinement is not provided by a beam framing into the joint.
Joints of SMRF

- 18.8 Joints of special moment frames
  - 18.8.4 Shear strength
    ▫ The requirements for proportioning joints are based on ACI 352R in that behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint
    ▫ Because tests of joints and deep beams indicated that shear strength was not as sensitive to joint (shear) reinforcement
    ▪ The strength of the joint has been set as a function of only the compressive strength of the concrete and requires a minimum amount of transverse reinforcement in the joint

- 18.8.4 Shear strength

  - 18.8.4.1 $V_n$ of the joint shall be in accordance with Table 18.8.4.1.

<table>
<thead>
<tr>
<th>Joint configuration</th>
<th>$V_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>For joints confined by beams on all four faces$^{[1]}$</td>
<td>$1.7\lambda\sqrt{f'_c A_j}$ $^{[2]}$</td>
</tr>
<tr>
<td>For joints confined by beams on three faces or on two opposite faces$^{[1]}$</td>
<td>$1.2\lambda\sqrt{f'_c A_j}$ $^{[2]}$</td>
</tr>
<tr>
<td>For other cases</td>
<td>$1.0\lambda\sqrt{f'_c A_j}$ $^{[2]}$</td>
</tr>
</tbody>
</table>

$^{[1]}$ Refer to 18.8.4.2.
$^{[2]}$ $\lambda$ shall be 0.75 for lightweight concrete and 1.0 for normalweight concrete. $A_j$ is given in 18.8.4.3.

- The strength is a function of how many beams frame into the column and confine the joint faces
- A circular column should be considered as a square section of equivalent area
Joints of SMRF

18.8 Joints of special moment frames

18.8.4 Shear strength

- The shear design strength \( \varphi V_n \geq V_j \) the required strength
- The design strength is defined as \( \varphi V_n = \varphi \gamma \sqrt{f_c} A_j \) in which
  - \( \varphi \) equals 0.85
  - \( A_j \) is the joint effective area
  - \( \gamma \) is a strength coefficient
- ACI 318 does not define different strengths for roof and typical floor levels

<table>
<thead>
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<th>Joint configuration</th>
<th>( V_n )</th>
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<tr>
<td>For joints confined by beams on all four faces(^{[1]})</td>
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</tr>
<tr>
<td>For other cases</td>
<td>( 1.0 \lambda \sqrt{f_c} A_j ) [2]</td>
</tr>
</tbody>
</table>

**Diagrams**
- Interior \( \gamma = 1.7 \)
- Exterior \( \gamma = 1.2 \)
- Corner \( \gamma = 1.0 \)
- Interior Roof \( \gamma = 1.2 \)
- Exterior Roof \( \gamma = 1.0 \)
- Corner Roof \( \gamma = 0.7 \)

21.2.4.3 For beam-column joints and diagonally reinforced coupling beams, \( \varphi \) for shear shall be 0.85.
Joints of SMRF

• 18.8 Joints of special moment frames

• 18.8.4 Shear strength
  ▫ Cyclic loading tests of joints with extensions of beams with lengths at least equal to their depths have indicated similar joint shear strengths to those of joints with continuous beams
  ▫ These findings suggest that extensions of beams, when properly dimensioned and reinforced with longitudinal and transverse bars, provide effective confinement to the joint faces, thus delaying joint strength deterioration at large deformations

18.8.4.2 In Table 18.8.4.1, a joint face is considered to be confined by a beam if the beam width is at least three-quarters of the effective joint width. Extensions of beams at least one overall beam depth $h$ beyond the joint face are considered adequate for confining that joint face. Extensions of beams shall satisfy 18.6.2.1(b), 18.6.3.1, 18.6.4.2, 18.6.4.3, and 18.6.4.4.

▫ If a beam covers less than three quarters of the column face at the joint, it must be ignored in determining which coefficient γ applies
Joints of SMRF

• 18.8 Joints of special moment frames

• 18.8.4 Shear strength
  ▫ The effective joint width, is not to be taken any larger than the overall width of the column

18.8.4.3 Effective cross-sectional area within a joint, $A_j$, shall be calculated from joint depth times effective joint width. Joint depth shall be the overall depth of the column, $h$. Effective joint width shall be the overall width of the column, except where a beam frames into a wider column, effective joint width shall not exceed the lesser of (a) and (b):

(a) Beam width plus joint depth
(b) Twice the smaller perpendicular distance from longitudinal axis of beam to column side

Definition of beam-column joint dimensions
Joints of SMRF

- 18.8 Joints of special moment frames
- 18.8.4 Shear strength

Note:
Effective area of joint for forces in each direction of framing is to be considered separately. Joint illustrated does not meet conditions of 18.8.3.2 and 18.8.4.1 necessary to be considered confined because the beams do not cover at least ¾ of the width of each of the faces of the joint.
Joints of SMRF

- **18.8 Joints of special moment frames**
- **18.8.5 Development length of bars in tension**
  - The requirement applies to beam and column bars terminated at a joint with a standard hook
  - Minimum embedment length in tension for deformed bars with standard hooks is determined using Eq. (18.8.5.1), which is based on the requirements of 25.4.3
    - The embedment length of a bar with a standard hook is the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the hook
  - This expression assumes that the hook is embedded in a confined beam-column joint
  - The requirement for the hook to project into the joint is to improve development of a diagonal compression strut in the joint

\[
\ell_{dh} = f_y d_b / (5.4\lambda \sqrt{f'_c})
\] (18.8.5.1)
Joints of SMRF

• 18.8 Joints of special moment frames

• 18.8.5 Development length of bars in tension

  ▫ Minimum development length in tension for straight bars is a multiple of the length indicated by 18.8.5.1

18.8.5.3 For bar sizes No. 10 through No. 36, $l_d$, the development length in tension for a straight bar, shall be at least the greater of (a) and (b):

  (a) 2.5 times the length in accordance with 18.8.5.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm
  (b) 3.25 times the length in accordance with 18.8.5.1 if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm
Joints of SMRF

- **18.8 Joints of special moment frames**
- **18.8.5 Development length of bars in tension**
  - If the required straight embedment length of a reinforcing bar extends beyond the confined volume of concrete the required development length is increased on the premise that the limiting bond stress outside the confined region is less than that inside

\[ \ell_{dm} = 1.6(\ell_d - \ell_{dc}) + \ell_{dc} \]

or

\[ \ell_{dm} = 1.6\ell_d - 0.6\ell_{dc} \]

- **18.8.5.4** Straight bars terminated at a joint shall pass through the confined core of a column or a boundary element. Any portion of \( \ell_d \) not within the confined core shall be increased by a factor of 1.6.
  - \( \ell_{dm} \) is the required development length if bar is not entirely embedded in confined concrete
  - \( \ell_d \) is the required development length in tension for straight bar as defined in 18.8.5.3
  - \( \ell_{dc} \) is the length of bar embedded in confined concrete