SEISMIC DESIGN OF REINFORCED CONCRETE STRUCTURES
NEHRP Recommended Provisions
Concrete Design Requirements

• Context in the *NEHRP Recommended Provisions*
• Concrete behavior
• Reference standards
• Requirements by Seismic Design Category
• Moment resisting frames
• Shear walls
• Other topics
• Summary
Context in *NEHRP Recommended Provisions*

Design basis: Strength limit state

**Using *NEHRP Recommended Provisions***:
- Structural design criteria: Chap. 4
- Structural analysis procedures: Chap. 5
- Components and attachments: Chap. 6
- Design of concrete structures: Chap. 9 and ACI 318
Seismic-Force-Resisting Systems
Reinforced Concrete

Unbraced frames (with rigid “moment resisting” joints):
Three types
- Ordinary
- Intermediate
- Special

R/C shear walls:
- Ordinary
- Special

Precast shear walls:
- Special
- Intermediate
- Ordinary
NEHRP Recommended Provisions
Concrete Design

• Context in the Provisions
• Concrete behavior
Unconfined Concrete Stress-Strain Behavior

![Graph showing stress-strain behavior for different concrete strengths.]

- 4500 psi
- 8800 psi
- 13,500 psi
- 17,500 psi
Idealized Stress-Strain Behavior of Unconfined Concrete

\[ f_c = f'_c \left( \frac{2\varepsilon_c}{\varepsilon'_o} - \left( \frac{\varepsilon_c}{\varepsilon'_o} \right)^2 \right) \]

\[ \varepsilon'_o = \frac{2f'_c}{E_t} \]

\[ E_t = 1.8\times10^6 + 460f'_c, \text{ psi} \]
Confinement by Spirals or Hoops

Confinement from spiral or circular hoop

Forces acting on 1/2 spiral or circular hoop

Confinement from square hoop
Confinement

- Rectangular hoops with cross ties
- Confinement by transverse bars
- Confinement by longitudinal bars
Opened 90° hook on hoops
Confined Concrete Stress-Strain Behavior

Stress, psi

Average strain on 7.9 in. gauge length

no confinement
4.75 in.
3.5 in.
2.375 in.
1.75 in.

Pitch of ¼ in. dia. spiral

Tests of 6 in. x 12 in. cylinders

FEMA
Instructional Material Complementing FEMA 451, Design Examples
Design for Concrete Structures 11 - 11
Idealized Stress-Strain Behavior of Confined Concrete

Kent and Park Model

![Graph showing idealized stress-strain behavior of confined concrete. The graph plots stress in psi on the y-axis against strain in in./in. on the x-axis. Different hoop spacing configurations are represented by different lines. The confined area is 12" x 16".](image)

Confined Area 12” x 16”

Strain, in./in.
Reinforcing Steel Stress-Strain Behavior

- **Grade 40**
  - Strain hardening: ~1-3%
  - Rupture: ~18-20%

- **Grade 60**
  - Rupture: ~10-12%

- **Grade 75**
  - E = 29,000 ksi

Stress, ksi vs. Microstrain

- Stress values: 20, 40, 60, 80, 100
- Microstrain values: 1000, 2000, 3000, 4000, 5000, 6000, 7000, 8000
Reinforced Concrete Behavior

Load

Mid-Point Displacement, $\Delta$

- Uncracked
- Cracked-elastic
- Cracked-inelastic
- Steel yields
- Failure
Behavior Up to First Yield of Steel

\[ E < f_s y \]

\[ f_c \]

\[ \varepsilon_s \]

Strain

\[ \varepsilon_c \]

Stress

\[ \varepsilon_s E_s \leq f_y \]
Behavior at Concrete Crushing

\[ M_n = A_s f_y j d \]
Typical Moment Curvature Diagram

- Strain hardening
- Without strain hardening

Parameters:
- $f'_c = 4\text{ ksi}$
- $f_y = 60\text{ ksi}$
- $b = 8\text{ in}$
- $d = 10\text{ in}$
- $\rho = 0.0125$
Influence of Reinforcement Ratio

- $f'_c = 4$ ksi
- $f_y = 60$ ksi
- $b = 10$ in
- $d = 18$ in
- $\rho = 2.5\%$
- $\rho = 1.5\%$
- $\rho = 0.5\%$

**Graph Details:**
- $M$, in-kip on the vertical axis
- $\phi \times 10^{-5}$ in$^{-1}$ on the horizontal axis
Influence of Compression Reinforcement

<table>
<thead>
<tr>
<th>Beam</th>
<th>$\rho$</th>
<th>$\rho'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0375</td>
<td>0.0250</td>
</tr>
<tr>
<td>2</td>
<td>0.0375</td>
<td>0.0125</td>
</tr>
<tr>
<td>3</td>
<td>0.0375</td>
<td>0.0</td>
</tr>
<tr>
<td>4</td>
<td>0.0250</td>
<td>0.0125</td>
</tr>
<tr>
<td>5</td>
<td>0.0250</td>
<td>0.0</td>
</tr>
<tr>
<td>6</td>
<td>0.0125</td>
<td>0.0125</td>
</tr>
<tr>
<td>7</td>
<td>0.0125</td>
<td>0.0</td>
</tr>
</tbody>
</table>
Moment-Curvature with Confined Concrete

\[
\begin{align*}
\varepsilon_{c,\text{max}} \\
\varepsilon_s > \varepsilon_y \\
\text{Strain} \\
\phi \\
f_y \\
\text{Stress} \\
f'_c
\end{align*}
\]
Moment-Curvature with Confined Concrete

Beam - 24 in. x 36 in.
Tension Steel - 12 ea. #10
Compression Steel - 5 ea. #8
Confining Steel - #4 hoops at 4 in. c-c

![Graph showing moment-curvature relationship with and without confining. The graph includes labels for the beam dimensions and steel quantities.]
Plastic Hinging

\[ \phi = \phi_u - \phi_y \]

- **idealized**
- **actual**

plastic rotation
Strategies to Improve Ductility

- Use low flexural reinforcement ratio
- Add compression reinforcement
- Add confining reinforcement
Other Functions of Confining Steel

- Acts as shear reinforcement
- Prevents buckling of longitudinal reinforcement
- Prevents bond splitting failures
Structural Behavior Frames

Story Mechanism

Sway Mechanism
Story Mechanism
Structural Behavior - Walls

- Flexural failure
- Horizontal tension
- Sliding on flexural cracks
- Sliding on construction joint

Symbols:
- $T$  
- $V$  
- $H$  
- $\Delta_s$
Structural Behavior
Walls

\[ V_u \]

\[ h_w \]

\[ l_w \]

Compression

\[ 45^\circ \]
Structural Behavior
Columns

Ultimate yield

Axial load, P, kip

Moment, M, in-kip

Curvature, \( \phi \), rad/in

14 in square
4-#11 bars
f'_c = 4 ksi
f_y = 45 ksi

1.75” bending axis
Influence of Hoops on Axial Strength

Before spalling -

\[ P = A_g f'_c \]

After spalling -

\[ P = A_{core}(f'_c + 4 f_{lat}) \]

After spalling \( \geq \) Before spalling
Column with Inadequate Ties
Well Confined Column
Hysteretic Behavior of Well Confined Column
V = \frac{M_1 + M_2}{L} = \frac{2M_u}{L}

Range of P
Column Shear Failure
Structural Behavior
Joints

Max. shear force
\[ V_j = T - V \]
Hysteretic Behavior of Joint with Hoops

\[ \frac{M}{M_u} \]

Drift, %

-1 0.5 1.0

-0.5 5 6
Hysteretic Behavior of Joint with No Hoops

\[ \frac{M}{M_u} \]

-0.5

0.5

1.0

Drift, %

-1

-0.5

5

6
Joint Failure – No Shear Reinforcing
Anchorage Failure in Column/Footing Joint
Summary of Concrete Behavior

• Compressive Ductility
  – Strong in compression but brittle
  – Confinement improves ductility by
    • Maintaining concrete core integrity
    • Preventing longitudinal bar buckling

• Flexural Ductility
  – Longitudinal steel provides monotonic ductility at low reinforcement ratios
  – Transverse steel needed to maintain ductility through reverse cycles and at very high strains (hinge development)
Summary of Concrete Behavior

• **Damping**
  - Well cracked: moderately high damping
  - Uncracked (e.g. prestressed): low damping

• **Potential Problems**
  - Shear failures are brittle and abrupt and must be avoided
  - Degrading strength/stiffness with repeat cycles
    • Limit degradation through adequate hinge development
NEHRP Recommended Provisions
Concrete Design

• Context in the Provisions
• Concrete behavior
• Reference standards
Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)

An ACI Standard

Reported by ACI Committee 318

ACI 318-05
Use of Reference Standards

• **ACI 318-05**
  - Chapter 21, Special Provisions for Seismic Design

• **NEHRP Chapter 9, Concrete Structures**
  - General design requirements
  - Modifications to ACI 318
  - Seismic Design Category requirements
  - Special precast structural walls
  - Untopped precast diaphragms (Appendix to Ch.9)
Detailed Modifications to ACI 318

- Modified definitions and notations
- Scope and material properties
- Special moment frames
- Special shear walls
- Special and intermediate precast walls
- Foundations
- Anchoring to concrete
NEHRP Recommended Provisions
Concrete Design

• Context in the Provisions
• Concrete behavior
• Reference standards
• Requirements by Seismic Design Category
### Design Coefficients - Moment Resisting Frames

<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>
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</tbody>
</table>
# Design Coefficients

## Shear Walls (Bearing Systems)

<table>
<thead>
<tr>
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<th>Deflection Amplification Factor, $C_d$</th>
</tr>
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<tbody>
<tr>
<td>Special R/C Shear Walls</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Ordinary R/C Shear Walls</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Intermediate Precast Shear Walls</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Ordinary Precast Walls</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>
# Design Coefficients

## Shear Walls (Frame Systems)

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<tbody>
<tr>
<td>Special R/C Shear Walls</td>
<td>6</td>
<td>5</td>
</tr>
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<td>Ordinary R/C Shear Walls</td>
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<td>Ordinary Precast Walls</td>
<td>4</td>
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# Design Coefficients

## Dual Systems with Special Frames

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<tr>
<th>Seismic Force Resisting System</th>
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<th>Deflection Amplification Factor, $C_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dual System w/ Special Walls</td>
<td>8 (7)</td>
<td>6.5 (5.5)</td>
</tr>
<tr>
<td>Dual System w/ Ordinary Walls</td>
<td>6</td>
<td>5</td>
</tr>
</tbody>
</table>

(ASCE 7-05 values where different)
## Frames

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Minimum Frame Type</th>
<th>ACI 318 Requirements</th>
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<tbody>
<tr>
<td>A and B</td>
<td>Ordinary</td>
<td>Chapters 1 thru 18 and 22</td>
</tr>
<tr>
<td>C</td>
<td>Intermediate</td>
<td>ACI 21.2.1.3 and ACI 21.12</td>
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</table>
# Reinforced Concrete Shear Walls

<table>
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<tr>
<th>Seismic Design Category</th>
<th>Minimum Wall Type</th>
<th>ACI 318 Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>A, B and C</td>
<td>Ordinary</td>
<td>Chapters 1 thru 18 and 22</td>
</tr>
<tr>
<td>D, E and F</td>
<td>Special</td>
<td>ACI 21.2.1.4 and ACI 21.2 and 21.7</td>
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</tbody>
</table>
## Precast Concrete Shear Walls

<table>
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<th>Seismic Design Category</th>
<th>Minimum Wall Type</th>
<th>ACI 318 Requirements</th>
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<tr>
<td>C</td>
<td>Intermediate</td>
<td>ACI 21.2.1.3 and ACI 21.13</td>
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<tr>
<td>D, E and F</td>
<td>Special</td>
<td>ACI 21.2.1.4 and ACI 21.2, 21.8</td>
</tr>
</tbody>
</table>
Additional *Provisions Requirements*

- **Category C**
  - Discontinuous members
  - Plain concrete
    - Walls
    - Footings
    - Pedestals (not allowed)
NEHRP Recommended Provisions
Concrete Design

• Context in the Provisions
• Concrete behavior
• Reference standards
• Requirements by Seismic Design Category
• Moment resisting frames
Performance Objectives

- **Strong column**
  - Avoid story mechanism

- **Hinge development**
  - Confined concrete core
  - Prevent rebar buckling
  - Prevent shear failure

- **Member shear strength**

- **Joint shear strength**

- **Rebar development**
Frame Mechanisms
“strong column – weak beam”

Story mechanism

Sway mechanism
Required Column Strength

\[ \sum M_{nc} \geq 1.2 \sum M_{nb} \]
Hinge Development

- **Tightly Spaced Hoops**
  - Provide confinement to increase concrete strength and usable compressive strain
  - Provide lateral support to compression bars to prevent buckling
  - Act as shear reinforcement and preclude shear failures
  - Control splitting cracks from high bar bond stresses
Hinge Development

Before spalling

After spalling
Hinge Development

Bidirectional cracking

Spalled cover
ACI 318-05, Overview of Frames: Beam Longitudinal Reinforcement

\[ \frac{200}{f_y} \leq \rho \leq 0.025 \]

At least 2 bars continuous top & bottom

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

Splice away from hinges and enclose within hoops or spirals
ACI 318-05, Overview of Frames: Beam Transverse Reinforcement

Closed hoops at hinging regions with “seismic” hook

135° hook, 6d_h ≥ 3” extension

Maximum spacing of hoops:

d/4   8d_b   24d_h    12”

Longitudinal bars on perimeter tied as if column bars

Stirrups elsewhere, s ≤ d/2
ACI 318-05, Overview of Frames: Beam Shear Strength

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

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

Then \( V_c = 0 \)

and \( P_u < \frac{A_g f_c'}{20} \)
ACI 318-05, Overview of Frames: Beam-Column Joint

\[ V_j = T + C - V_{\text{col}} \]

\[ T = 1.25 f_y A_{s,\text{top}} \]

\[ C = 1.25 f_y A_{s,\text{bottom}} \]
ACI 318-05, Overview of Frames: Beam-column Joint

\[ V_n = \begin{cases} 20 \\ 15 \\ 12 \end{cases} \sqrt{f_c'} A_j \]

- \( V_n \) controls size of columns
- Coefficient depends on joint confinement
- To reduce shear demand, increase beam depth
- Keep column stronger than beam
ACI 318-05: Overview of Frames: Column Longitudinal Reinforcement

\[ M_{nc1} \quad M_{nb1} \quad M_{nc2} \quad M_{nb2} \]

0.01 \leq \rho \leq 0.06

\[ \sum M_{nc} \geq 1.2 \sum M_{nb} \]

At joints

(strong column-weak beam)

\( M_{nc} \) based on factored axial force, consistent with direction of lateral forces
ACI 318-05, Overview of Frames: Column Transverse Reinforcement at Potential Hinging Region

Spirals
\[ \rho_s = 0.45 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \]

and
\[ \rho_s \geq 0.12 \frac{f'_c}{f_{yt}} \]

Hoops
\[ A_{sh} \geq 0.3 \left( s_b c \frac{f'_c}{f_{yt}} \right) \left( \frac{A_g}{A_{ch}} - 1 \right) \]

and
\[ A_{sh} \geq 0.09 s_b c \frac{f'_c}{f_{yt}} \]
ACI 318-05, Overview of Frames: Column Transverse Reinforcement at Potential Hinging Region

\[ s_o = 4 + \left( \frac{14 - h_x}{3} \right) \]

Spacing shall not exceed the smallest of:
- \( b/4 \) or \( 6 \, d_b \) or \( s_o \) (4” to 6”)
- Distance between legs of hoops or crossties, \( h_x \leq 14” \)
ACI 318-05, Overview of Frames: Potential Hinge Region

• For columns supporting stiff members such as walls, hoops are required over full height of column if

\[ P_e > \frac{f'_c A_g}{10} \]

• For shear strength- same rules as beams (concrete shear strength is neglected if axial load is low and earthquake shear is high)

• Lap splices are not allowed in potential plastic hinge regions
Splice in Hinge Region

Terminating bars
ACI 318-05, Overview of Frames: Potential Hinge Region

\[ l_o \geq \left\{ \frac{d}{6} \right\} \text{clear height} \]

\[ \left\{ \begin{array}{c}
18''
\end{array} \right\} \]
Moment Frame Example

7 @ 30' = 210'
5 @ 20' = 100'

A A' B C C' D
1
2
3
4
5
6
7
8

N
Frame Elevations

Column Lines 2 and 7

Column Lines 3 to 6
Story Shears: Seismic vs Wind

- Seismic E-W
- Seismic N-S
- Wind E-W
- Wind N-S
Story Shears: E-W Loading

includes shearwall

frame 1
frame 2
frame 3
Story Shears: 25% rule

Frame 1
25% Frame 1

w/o walls
w/ walls
Layout of Reinforcement
Bending Moment Envelopes: Frame 1 Beams

- Seismic
- Dead
- Combined:

1.42D + 0.5L + E
0.68D - E
1.2D + 1.6L
Beam Reinforcement: Longitudinal

Max negative $M_u = 5834$ in-kips

$b = 22.5'' \quad d = 29.6'' \quad f'_c = 4$ ksi \quad f_y = 60$ ksi

$$A_{s \text{ req'd}} = \frac{M_u}{\phi f_y (0.875d)} = \frac{5834}{0.9 \cdot 60 \cdot 0.875 \cdot 29.6} = 4.17 \text{in}^2$$

Choose: 2 #9 and 3 #8 \quad A_s = 4.37 \text{ in}^2

$\rho = 0.0066 < 0.025 \quad \text{OK}$

$\phi M_n = 6580$ in-kips \quad \text{OK}$
Beam Reinforcement: Longitudinal (continued)

Positive $M_u$ at face of column = 4222 in-kips (greater than $\frac{1}{2}(5834) = 2917$)

$b$ for negative moment is the sum of the beam width (22.5 in.) plus $1/12$ the span length (20 ft x 12 in./ft)/12, $b = 42.5$ in.

$$A_{s \text{req'd}} = \frac{M_u}{\phi f_y(0.9d)} = \frac{4222}{60 \cdot 0.9 \cdot 29.6} = 2.94 \text{in}^2$$
Beam Reinforcement: Longitudinal (continued)

Choose 2 #7 and 3 #8  \( A_s = 3.57 \text{ in}^2 \)
\( \phi M_n = 5564 \text{ in-kips} \)  OK

Run 3 #8s continuous top and bottom
\( \phi M_n = 3669 \text{ in-kips} \)
This moment is greater than:
  25\% of max negative \( M_n = 1459 \text{ in-kips} \)
Max required \( M_u = 834 \text{ in-kips} \)
Beam Reinforcement: Preliminary Layout

- **A** 2 #8
- **A’** 2 #8, 2 #9
- **B** 2 #9, 3 #8
- **C** 2 #9
- **C’** 2 #8
- **D** 2 #7
- **2 #7**
- **2 #7**
- **3 #8**
- **3 #8**
- **3 #8**
Moments for Computing Shear

Hinging mechanism

Plastic moments (in-kips)

Girder and column shears (kips)
Joint Shear Force

\[ T = 1.25f_y A_{s,\text{top}} = 355.5 \text{ kips} \]
\[ C = 1.25f_y A_{s,\text{bot}} = 355.5 \text{ kips} \]
\[ V_j = T + C - V_{\text{col}} = 560.5 \text{ kips} \]
\[ v_j = 15\sqrt{f'_c} = 949 \text{ psi} \]
\[ V_n = v_j A_j = 949 \cdot 30 \cdot 30 = 854 \text{ kips} \]
\[ \phi V_n = 0.85 \cdot 854 = 726 \text{ kips} > 560.5 \text{ kips} \]
Beam Shear Force

Seismic shear

Factored gravity shear

Design shear

A A' B C

79.4 75.6 82.7 82.7

29.5 29.5 29.5 29.5

49.9 108.9 53.2 112.2

105.1 108.9 49.9 112.2

Instructional Material Complementing FEMA 451, Design Examples
Beam Reinforcement: Transverse

\[ V_{\text{seismic}} > 50\% \ V_u \text{ therefore take } V_c = 0 \]
\[ 82.7 \text{ kips} = 73\% (112.2) \]

Use 4 legged #3 stirrups

\[ V_s = \frac{A_v f_y d}{s} \]

At ends of beam \( s = 5.5 \text{ in.} \)
Near midspan \( s = 7.0 \text{ in.} \)
Beam Reinforcement: Transverse

• Check maximum spacing of hoops within plastic hinge length (2d)
  – $d/4 = 7.4$ in.
  – $8d_b = 7.0$ in.
  – $24d_h = 9.0$ in.
Column Design Moments

Girder moments (Level 7)

\[ \sum M_{nc} = 1.2(7311 + 6181) \]

= 16190 in – k

Column moments (Level 7)
Column Design Moments

\[
\text{if} \quad P_u > \frac{f'_c A_g}{10} \\
\sum M_{nc} > 1.2 \sum M_{nb}
\]

Distribute relative to stiffness of columns above and below:

\[
M_{nc} = 8095 \text{ in-kips (above)} \\
M_{nc} = 8095 \text{ in-kips (below)}
\]
# Design Strengths

<table>
<thead>
<tr>
<th>Design Aspect</th>
<th>Strength Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam rebar cutoffs</td>
<td>Design strength</td>
</tr>
<tr>
<td>Beam shear reinforcement</td>
<td>Maximum probable strength</td>
</tr>
<tr>
<td>Beam-column joint strength</td>
<td>Maximum probable strength</td>
</tr>
<tr>
<td>Column flexural strength</td>
<td>1.2 times nominal strength</td>
</tr>
<tr>
<td>Column shear strength</td>
<td>Maximum probable strength</td>
</tr>
</tbody>
</table>
Column Transverse Reinforcement

\[ A_{sh} = 0.3 \left( sbc \frac{f'_c}{f_{yt}} \right) \left[ \left( \frac{A_g}{A_{ch}} \right) - 1 \right] \]

and

\[ A_{sh} = 0.09 sbc \frac{f'_c}{f_{yt}} \]

\( A_g \) = gross area of column
\( A_{ch} \) = area confined within the hoops
\( b_c \) = transverse dimension of column core
  measured center to center of outer legs

Second equation typically governs for larger columns
Column Transverse Reinforcement

Maximum spacing is smallest of:
  • One quarter of minimum member dimension
  • Six times the diameter of the longitudinal bars
  • $s_o$ calculated as follows:

$$s_o = 4 + \frac{14 - h_x}{3}$$

$h_x$ = maximum horizontal center to center spacing of cross-ties or hoop legs on all faces of the column, not allowed to be greater than 14 in.
Column Transverse Reinforcement

For max s = 4 in.

\[ A_{sh} = 0.3 \left( \frac{f'_c}{f_{yt}} \right) \left[ \frac{A_g}{A_{ch}} - 1 \right] = 0.3 \left( 4 \cdot 26.5 \cdot \frac{4}{60} \right) \left( \frac{900}{702} - 1 \right) \]

A_{sh} = 0.60 \text{ in}^2

and

\[ A_{sh} = 0.09s b_c \frac{f'_c}{f_{yt}} = 0.09 \cdot 4 \cdot 26.5 \cdot \frac{4}{60} = 0.64 \text{ in}^2 \]

Use 4 legs of #4 bar – A_{sh} = 0.80 \text{ in}^2
Determine Seismic Shear

\[ M_{pr,1} \quad M_{pr,2} \quad M_{pr,3} \quad M_{pr,4} \]

\[ V_{seismic} \]

\[ \ell_n \]
Column Transverse Reinforcement Shear Demand from $M_{pr}$ of Beams

$M_{pr,1} = 9000$ in-k (2 #9 and 3 #8)  
$M_{pr,2} = 7460$ in-k (2 #7 and 3 #8)

Assume moments are distributed equally above and below joint

$$V_{seismic} = \frac{8230 \cdot 2}{(12.5 \cdot 12) - 32} = 139 \text{ kips}$$

Note $V_{seismic} \approx 100\%V_u$

$V_c = 0$, if $P_{min} < \frac{f'_c A_g}{20} = 180 \text{ kips}$

For 30 in. square column  
$P_{min} = 266 \text{ kips} \quad \text{OK}$
Column Transverse Reinforcement
Shear Demand from $M_{pr}$ of Beams

\[
\phi V_c = \phi 2\lambda \sqrt{f'_{c}} bd = 0.75 \cdot 2 \cdot 0.85 \sqrt{4000} \cdot 30 \cdot 27.5 = 66.5 \text{ kips}
\]

\[
\phi V_{s,\text{required}} = 139 - 66.5 = 72.5 \text{ kips}
\]

\[
\phi V_{s,\text{provided}} = \phi \frac{A_v f_y d}{s} = 0.75 \frac{4 \cdot 0.2 \cdot 60 \cdot 29.6}{4} = 266.4 \text{ kips}
\]

Hoops

4 legs #4
s = 4”
Column Reinforcement

7 @ 4"
5"
8 @ 6"
5"
7 @ 4"

#4 hoops:

12 #8 bars
# Levels of Seismic Detailing for Frames

<table>
<thead>
<tr>
<th>Issue</th>
<th>Ordinary</th>
<th>Intermediate</th>
<th>Special</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hinge development and confinement</td>
<td></td>
<td>minor</td>
<td>full</td>
</tr>
<tr>
<td>Bar buckling</td>
<td></td>
<td>lesser</td>
<td>full</td>
</tr>
<tr>
<td>Member shear</td>
<td></td>
<td>lesser</td>
<td>full</td>
</tr>
<tr>
<td>Joint shear</td>
<td>minor</td>
<td>minor</td>
<td>full</td>
</tr>
<tr>
<td>Strong column</td>
<td></td>
<td></td>
<td>full</td>
</tr>
<tr>
<td>Rebar development</td>
<td>lesser</td>
<td>lesser</td>
<td>full</td>
</tr>
<tr>
<td>Load reversal</td>
<td>minor</td>
<td>lesser</td>
<td>full</td>
</tr>
</tbody>
</table>
NEHRP Recommended Provisions
Concrete Design

- Context in the Provisions
- Concrete behavior
- Reference standards
- Requirements by Seismic Design Category
- Moment resisting frames
- Shear walls
Performance Objectives

• Resist axial forces, flexure and shear
• Boundary members
  — Where compression strains are large, maintain capacity
• Development of rebar in panel
• Discontinuous walls: supporting columns have full confinement
Design Philosophy

• Flexural yielding will occur in predetermined flexural hinging regions

• Brittle failure mechanisms will be precluded
  – Diagonal tension
  – Sliding hinges
  – Local buckling
ACI 318-05, Overview of Walls: General Requirements

\[ \rho_t = \text{parallel to shear plane} \]
\[ \rho_\ell = \text{perpendicular to shear plane} \]

Shear plane, \( A_{cv} = \text{web thickness} \times \text{length of wall} \)
ACI 318-05, Overview of Walls: General Requirements

• \( \rho_l \) and \( \rho_t \) not less than 0.0025 unless
  \[ V_u < A_{cv} \sqrt{f'_c} \]
  then as allowed in 14.3

• Spacing not to exceed 18 in.

• Reinforcement contributing to \( V_n \) shall be continuous and distributed across the shear plane
ACI 318-05, Overview of Walls: General Requirements

- Two curtains of reinforcing required if:
  \[ V_u > 2A_{cv} \sqrt{f'_c} \]

- Design shear force determined from lateral load analysis
ACI 318-05, Overview of Walls: General Requirements

• Shear strength:

\[ V_n = A_{cv} \left( \alpha_c \sqrt{f'_c} + \rho_t f_y \right) \]

\[ \alpha_c = 3.0 \text{ for } h_w/\ell_w \leq 1.5 \]
\[ \alpha_c = 2.0 \text{ for } h_w/\ell_w \geq 2.0 \]

Linear interpolation between

• Walls must have reinforcement in two orthogonal directions
ACI 318-05, Overview of Walls: General Requirements

- For axial load and flexure, design like a column to determine axial load – moment interaction
ACI 318-05, Overview of Walls: Boundary Elements

For walls with a high compression demand at the edges – Boundary Elements are required

Widened end with confinement

Extra confinement and/or longitudinal bars at end
ACI 318-05, Overview of Walls: Boundary Elements

• Boundary elements are required if:

\[ c \geq \frac{\ell_w}{600(\frac{\delta_u}{h_w})} \]

\[ \delta_u = \text{Design displacement} \]
\[ c = \text{Depth to neutral axis from strain compatibility analysis with loads causing } \delta_u \]
ACI 318-05, Overview of Walls: Boundary Elements

• Where required, boundary elements must extend up the wall from the critical section a distance not less than the larger of:

\[ \ell_w \quad \text{or} \quad \frac{M_u}{4V_u} \]
Boundary elements are required where the maximum extreme fiber compressive stress calculated based on factored load effects, linear elastic concrete behavior and gross section properties, exceeds 0.2 $f'_c$

Boundary element can be discontinued where the compressive stress is less than 0.15$f'_c$
ACI 318-05: Overview of Walls
Boundary Elements

• Boundary elements must extend horizontally not less than the larger of $c/2$ or $c-0.1\ell_w$

• In flanged walls, boundary element must include all of the effective flange width and at least 12 in. of the web

• Transverse reinforcement must extend into the foundation
Wall Example

A A' B C C' D

11 @ 12.5

15' 18'
Wall Cross-Section
Story Shears
E-W Loading

includes shearwall
Boundary Element Check

Required if: \( f_c > 0.2 f'_c \) based on gross concrete section

Axial load and moment are determined based on factored forces, including earthquake effects

At ground \( P_u = 5550 \text{ kip} \)

\( M_u \) from analysis is 268,187 in-kip

The wall has the following gross section properties:

\[ A = 4320 \text{ in}^2 \quad S = 261,600 \text{ in}^3 \]

\[ f_c = 2.3 \text{ ksi} = 38\% \text{ of } f'_c = 6 \text{ ksi} \]

\[ \therefore \text{ Need boundary element} \]
Boundary Element Design

Determine preliminary reinforcing ratio in boundary elements by assuming only boundary elements take compression

\[ M = 268,187 \text{ in-k} \]

\[ P = 5550 \text{ k} \]

\[
B_1 = \frac{P}{2} + \frac{M}{240} = 3892 \text{ kip}
\]

\[
B_2 = \frac{P}{2} - \frac{M}{240} = 1658 \text{ kip}
\]

Need

\[
0.8P_o = 0.8 \left(0.7\right)A_g \left[0.85 f'_c \left(1 - \rho\right) + \rho f_y \right] > 3892 \text{ kip}
\]

For \( A_g = 30(30) = 900 \text{ in}^2 \)

For \( f'_c = 4 \text{ ksi} \) \( \Rightarrow \) \( \rho = 7.06\% \) Too large

For \( f'_c = 6 \text{ ksi} \) \( \Rightarrow \) \( \rho = 4.18\% \) Reasonable; 24 #11
Boundary Element Confinement

Transverse reinforcement in boundary elements is to be designed essentially like column transverse reinforcement

\[ A_{sh} = 0.09 \cdot s \cdot b_c \cdot \frac{f_c'}{f_y} = 1.08 \text{ in}^2 \text{ at } s = 4" \]

4 legs of #5
Shear Panel Reinforcement

\[ V_n = A_{cv} \left( 2 \lambda \sqrt{f'_c} + \rho_t f_y \right) \]

\[ V_u = 539 \text{ kips (below level 2)} \]

\[ \phi = 0.6 \text{ (per ACI 9.3.4(a))} \]

\[ \rho_t = 0.0036 \text{ for } f_y = 40 \text{ ksi} \]

\[ \text{Min } \rho_l \text{ (and } \rho_t) = 0.0025 \]

2 curtains if \[ V_u > 2 \sqrt{f'_c A_{cv}} \]
Shear Panel Reinforcement

Select transverse and longitudinal reinforcement:

longitudinal:

\[
\#4@12" \Rightarrow \frac{0.2 \cdot 2}{12 \cdot 12} = 0.0028 > 0.0025
\]

transverse:

\[
\#4@9" \Rightarrow \frac{0.2 \cdot 2}{12 \cdot 9} = 0.0037 > 0.0036
\]
Check Wall Design

![Diagram showing nominal and factored combinations of axial load and moment for wall design.](image-url)
Shear Wall Reinforcement

- Shear wall sections labeled R, B, C, 8, 7, 6, 5, 4, 3.
- Reinforcement details for each level:
  - Level 8: #4@18" ver. E.F., #4@16" hor. E.F., #4@12" E.W. E.F., #4 @ 4".
  - Level 7: 24 #9.
  - Level 6: 24 #10.
  - Level 5: #4@12" ver. E.F., #4@6" hor. E.F.
  - Level 4: #5 @ 4".
  - Level 3: 24 #11.
  - Level 2: 24 #10.
  - Level 1: #4@12" ver. E.F., #4@6" hor. E.F.

Concrete strengths:
- \( f_c' = 6 \text{ ksi} \)
- \( f_c' = 4 \text{ ksi} \)
NEHRP Recommended Provisions
Concrete Design

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- Shear walls
- Other topics
Members Not Part of SRS

• In frame members not designated as part of the lateral-force-resisting system in regions of high seismic risk:
  – Must be able to support gravity loads while subjected to the design displacement
  – Transverse reinforcement increases depending on:
    Forces induced by drift
    Axial force in member
Diaphragms

- Load from analysis in accordance with design load combinations

Check:
- Shear strength and reinforcement (min. slab reinf.)
- Chords (boundary members)
  - Force = M/d Reinforced for tension
  (Usually don’t require boundary members)
Struts and Trusses
performance objectives

• All members have axial load (not flexure), so ductility is more difficult to achieve

• Full length confinement
Precast performance objectives

Field connections at points of low stress

Strong connections
• Configure system so that hinges occur in factory cast members away from field splices

Field connections must yield

Ductile connections
• Inelastic action at field splice
Quality Assurance
Rebar Inspection

• Continuous
  – Welding of rebar

• Periodic
  – During and upon completion of placement for special moment frames, intermediate moment frames and shear walls
Shear panel reinforcement cage
Quality Assurance: Reinforcing Inspection - Prestressed

• **Periodic**
  - Placing of prestressing tendons (inspection required upon completion)

• **Continuous**
  - Stressing of tendons
  - Grouting of tendons
Quality Assurance: Concrete Placement Inspection

- **Continuous**
  - Prestressed elements
  - Drilled piers
  - Caissons

- **Periodic**
  - Frames
  - Shear walls
Quality Assurance:
Precast Concrete (plant cast)

- Manufacturer may serve as special inspector if plant’s quality control program is approved by regulatory agency
- If no approved quality control program, independent special inspector is required
Quality Assurance: PCI Certification Program

• Review of plant operations
  – Scheduled and surprise visits
  – Qualified independent inspectors
  – Observed work of in-plant quality control
  – Check results of quality control procedures
  – Periodic – specific approvals requiring renewal
Quality Assurance: ACI Inspector Certification

• Specialized training available for:
  - Laboratory and in situ testing
  - Inspection of welding
  - Handling and placement of concrete
  - Others
Quality Assurance: Reinforcement Testing

• **Rebar**
  – Special and intermediate moment frames
  – Boundary elements

• **Prestressing steel**

• **Tests include**
  – Weldability
  – Elongation
  – Actual to specified yield strength
  – Actual to specified ultimate strength
Quality Assurance: Concrete Testing

- Sample and test according to ACI 318-05
  - Slump
  - Air content
  - 7 and 28 day strengths
  - Unit weight

- Rate
  - Once per day per class
NEHRP Recommended Provisions: Concrete Design

- Context in the Provisions
- Concrete behavior
- Reference standards
- Requirements by Seismic Design Category
- Moment resisting frames
- Shear walls
- Other topics
- Summary