Seismic Design Requirements in ACI 318-08

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Chapter 1
General Requirements

- Modifications in
  - Scope
  - Terminology

R1.1.9 – Provisions for earthquake resistance

- Commentary was expanded to:
  - Explain changes in terminology used
  - Simplify adoption and interaction of ACI 318-08 with model codes and other documents

R1.1.9 – Provisions for earthquake resistance

- In this version of ACI 318 (2008), for the first time, earthquake resistance requirements are defined in function of the Seismic Design Category — SDC required for the structure and not directly associated with the seismic risk zone.

- The minimum SDC to use is governed by the legally adopted general building code of which ACI 318 forms a part.
TABLE R1.1.9.1 — 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>Uniform Building Code 1991, 1994, 1997</td>
<td>Seismic Zone 0, 1</td>
</tr>
</tbody>
</table>

*SDC = Seismic Design Category as defined in code, standard, or resource document.
*SPC = Seismic Performance Category as defined in code, standard, or resource document.

Chapter 2

Notation and Definitions

- There were important changes in notation of the whole document and all individual Chapter notation was moved to Chapter 2.
- There are a few new definitions related to Chapter 21. All definitions, old and new, were moved to Chapter 2.

Chapter 21

Earthquake-resistant structures

- Chapter 21 was reorganized in function of Seismic Design Categories (SDC) A, B, C, and D, E, F in incremental order from ordinary to special:

  A → B → C → D, E, F

Seismic Design Category and Energy Dissipation Capacity

<table>
<thead>
<tr>
<th>SDC Seismic Design Category</th>
<th>Denomination (Energy dissipation capacity)</th>
<th>Must comply with in ACI 318-08</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Ordinary</td>
<td>Chapters 1 to 19 and 22</td>
</tr>
<tr>
<td>B</td>
<td>Intermediate</td>
<td>Chapters 1 to 19, 22, and 21.2</td>
</tr>
<tr>
<td>C</td>
<td>Special</td>
<td>Chapters 1 to 19, 22, and 21.3and 21.4</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Special</td>
<td>Chapters 1 to 19, 22, and 21.5to 21.13</td>
</tr>
</tbody>
</table>
Chapter 21 contains provisions considered to be the minimum requirements for a cast-in-place or precast concrete structure capable of sustaining a series of oscillations into the inelastic range of response without critical deterioration in strength.

Therefore, the objective is to provide energy dissipation capacity in the nonlinear range of response.

In several earthquake resistance regulations this is defined through parameter $R$.
Elastic vs. Nonlinear Demand

Current seismic design strategy

- Given an energy dissipation capacity for the structural material and structural system, defined through an $R$ value depending on the detailing scheme the design horizontal seismic force is obtained from:

$$F_x = \frac{F_{\text{e}}}{R}$$

- and the maximum elastic force demand is in turn obtained using Newton’s 2nd Law:

$$F_{\text{e}} = \text{mass} \times S_a(T_s, \xi)$$

What would happen if energy dissipation capacity is not available?
Nonstructural wall panel in contact with the structure

ACI 318-08 requires (21.1.2) that interaction between structural and nonstructural elements that may affect the response during the earthquake must be taken into account. Rigid members assumed not to be a part of the seismic-force-resisting system are permitted provided their effect on the response of the system is considered and accommodated 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.

C.21.1 – General Requirements

- Compressive strength of concrete $f'_c \geq 21$ MPa
- Specified compressive strength of lightweight concrete $\leq 35$ MPa
- For computing the amount of confinement reinforcement $f_y \leq 700$ MPa ($= 100,000$ psi $= 7000$ kgf/cm$^2$)
- Reinforcing steel must meet ASTM A706. If ASTM A615 is used, it must meet:
  - The actual yield strength based on mill tests does not exceed $f_y$ by more than 125 MPa.
  - The ratio of the actual tensile strength to the actual yield strength is not less than 1.25
21.2 – Ordinary moment frames

- Corresponds to SDC B
- Beams must have at least two continuous longitudinal bars along both top and bottom faces. These bars shall be developed at the face of support.
- Columns having clear height less than or equal to five times the dimension \( c_1 \) must be designed for shear in accordance with 21.3.3.
  (shear requirements for intermediate SDC C)

21.3 - Intermediate moment frames

- Requirements for this Section are equivalent to the rest of Chapter 21, but are less strict and have a lesser scope.
- Two alternatives are presented for shear design of beams and columns:
  - Obtain design shear forces as function of nominal end moments as done for special elements, or
  - use twice the shear from analysis. This is equivalent to using the following load combinations:
    \[
    U = 1.2D + 1.0L + (1.0E) \times 2.0
    \]
    \[
    U = 0.9D + (1.0E) \times 2.0
    \]
- When \( P_u \) is greater, reinforcing details must meet column requirements.
- When a slab-column system without beams is part of the seismic-force-resisting system, reinforcement details in any span resisting moments caused by \( E \) must satisfy 21.3.6.
21.3 - Intermediate moment frames

For beams:
Moment strength must comply with:

\[ M_{n} = \frac{1}{5}(M_{n})_{\text{max face}} \]
\[ M_{n} = \frac{1}{3}M_{n} \]

At both ends of the beam, hoops shall be provided over lengths not less than \(2h\) measured from the face of the supporting member toward midspan. The first hoop shall be located not more than 50 mm from the face of the supporting member. Spacing of hoops shall not exceed the smallest of \(\frac{d}{4}, 8d_{h},\) 24\(d_{b}\) of the smaller longitudinal bar, 24\(d_{b}\) of hoop, or 300 mm. Stirrups shall be spaced not more than \(d/2\) throughout the length of the beam.

For columns

Two-way slabs without beams (slab-column frames)

- Reinforcement provided to resist \(M_{\text{slab}}\) shall be placed within the column strip.
- Not less than 50% of the reinforcement in the column strip at supports shall be placed within the effective slab width defined by lines drawn parallel to the span at 1.5 slab depths from the column face.
- Continuous bottom reinforcement in the column strip shall be not less than 33% of the top reinforcement at the support in the column strip.
- Not less than 25% of the top reinforcement at the support in the column strip shall be continuous throughout the span.
21.3 - Intermediate moment frames

Two-way slabs without beams (slab-column frames)

- At the critical sections for punching shear, shear caused by factored gravity loads shall not exceed, $0.4V$, where $V$ must be calculated as defined in Chapter 11 for prestressed and non prestressed slabs.

- This requirement may be waived if the slab complies with 21.13.6

21.4 — Intermediate precast structural walls

- Requirements of 21.4 apply to intermediate precast structural walls forming part of the seismic-force resisting systems.

- In connections between wall panels, or between wall panels and the foundation, yielding must be restricted to steel elements or reinforcement.

- Elements of the connection that are not designed to yield must develop at least $1.5S_y$.  

21.5 — Flexural members of special moment frames

- Axial force $P_u$ must not exceed $0.10f_yA_y$

- Clear span of element $l_n$ must be larger than $4d$

- Ratio $b_w/h > 0.3$

- Width $b_w$ must comply with:
  - $b_w > 250$ mm
  - larger than the width of the supporting element plus $3h/4$ at each side
21.5 — Flexural members of special moment frames

**Longitudinal reinforcement**

Steel ratio for negative and positive reinforcement must not be less than:

\[
\rho \geq \frac{\sqrt{f_t}}{4f_y} \geq 1.4 \frac{f_y}{f_t}
\]

But:

\[
\rho \leq 0.025
\]

At least two bars continuous top and bottom.

Longitudinal reinforcement

Moment strength at each section must be at least:

- Lap splices are permitted if hoops are provided throughout the splice length. Maximum hoop spacing must not exceed \(d/4\) or 100 mm.

- No lap splices are permitted in joints or within 2h of column face or where inelastic action is expected.
21.5 — Flexural members of special moment frames

**Hoops must be provided:**

\[
\frac{d}{3} \leq s \leq \frac{4d}{3}
\]

Shear design:

\[
\Delta V_e = \frac{M_{pr}^{+ \text{izq}} + (M_{pr})_{\text{der.}}}{\ell_n} - \sum P_a \frac{1}{\ell_n}
\]

\[
M_{pr} \text{ computed using } f_{yp} = 1.25 f_y \text{ and } \phi = 1.0
\]

21.6 — Special moment frame members subjected to bending and axial load

**General**

- Axial force greater than \(0.10 \cdot f'_c \cdot A_g\)
- The least section dimension that passes through the centroid must be greater than 300 mm.
- Ratio \(b/h > 0.4\)
21.6 — Special moment frame members subjected to bending and axial load

- Column flexural strength must comply with:
  \[ \sum M_{nc} \geq 1.2 \sum M_{nb} \]

- Transverse reinforcement in confining zones must comply with:
  - Spiral columns:
    \[ \rho_s = 0.12 \frac{f'_c}{f_{yt}} \]
  - Columns with hoops:
    \[ A_{sh} = 0.3 \cdot s \cdot b_c \cdot f'_c \left( \frac{A_e}{A_{cm}} \right) \]
    \[ A_{sh} = 0.09 \cdot s \cdot b_c \cdot f'_c \left( \frac{A_e}{A_{cm}} \right) \]

21.6 — Special moment frame members subjected to bending and axial load

- Shear design
  \[ V_e = \frac{M_{pr} \text{arriba} + M_{pr} \text{abajo}}{h_n} \]
  \[ M_{pr} \text{corresponds to the maximum moment strength for the axial load range on the element (1.25f_y and } \phi = 1 \text{). } V_e \text{ cannot be less than the one obtained from analysis.} \]
  \[ V_e = 0 \text{ if } V_e \text{ is more than 50% of the required shear or the axial force is less than } 0.05f'_c A_p \]
21.7 — Joints of special moment frames

**General requirements**

- When computing shear strength within the joint in special frames all longitudinal reinforcement must be presumed to be stressed at \(1.25 \sigma_y\).
- Longitudinal reinforcement terminating at a joint must be extended to the far face of the column confined core and anchored in tension.
- When the beam longitudinal reinforcement passes through the joint, the column dimension parallel to the reinforcement cannot be less than \(20d_b\), largest longitudinal bar, for normal weight concrete and \(26d_b\) for lightweight concrete.

**Shear strength**

- Joints confined in all four faces
  \[ V_u = 1.70 \sqrt{f_y'^2 - A_j} \]
- Joints confined in three faces or in opposite faces
  \[ V_u = 1.25 \sqrt{f_y'^2 - A_j} \]
- Other joints
  \[ V_u = 1.00 \sqrt{f_y'^2 - A_j} \]
21.7 — Joints of special moment frames

- Development for hooks embedded in the confined core

\[ L_{dh} = \frac{f_y \cdot d_h}{5.4 \sqrt{f_c'}} \]

21.8 — Special moment frames constructed using precast concrete

- The requirements of 21.8 apply for special moment frames built using precast concrete forming part of the seismic-force-resistant system.

- The detailing provisions in 21.8.2 and 21.8.3 are intended to produce frames that respond to design displacements essentially like monolithic special moment frames.

- The provisions of 21.8.4 indicate that when not satisfying 21.8.2 or 21.8.3 they must satisfy the requirements of ACI 374.1

21.8 — Special moment frames constructed using precast concrete

- Special precast moment frames with ductile connections must comply with all requirements for special cast-in-place frames and \( V_u \) should not be less than 2\( V_{ur} \).

- Special precast moment frames with strong connections are intended to experience flexural yielding outside the connections.

These requirements are applicable independently of any of these two situations.

21.9 — Special structural walls and coupling beams

- Terminology
21.9 — Special structural walls and coupling beams – General requirements

- Cover

- Maximum reinforcement spacing

\[ s \leq 3h \leq 450 \text{ mm} \]

21.9 — Special structural walls and coupling beams

**Flexure design**

- Design for flexure and flexure and axial load for structural walls must be performed using the requirements of Chapter 10.

- Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective.

- The effects of openings shall be considered.

21.9 - Special structural walls and coupling beams

**Flexure design**

Unless a more detailed analysis is performed, effective flange widths of flanged sections (I, L, C or T) may be supposed to extend from the face of the web a distance equal to the smaller of:

(a) 1/2 the distance to an adjacent wall web,

and

(b) 25 percent of the total wall height.

21.9 - Special structural walls and coupling beams

**21.9.2 – Reinforcement**

The distributed web reinforcement ratios, \( \rho_t \) and \( \rho_A \), for structural walls shall not be less than 0.0025, except that if \( V_u \) does not exceed 0.083 \( A_f \lambda_c \sqrt{f_c'} \) (MPa) = 0.27 \( A_f \lambda_c \sqrt{f_c'} \) (kgf/cm²), \( \rho_t \) and \( \rho_A \) may be reduced to the values given in 14.3.

Separation of reinforcement must not exceed 450 mm.
Minimum steel ratio

- 14.3.2 – Minimum steel ratio of vertical reinforcement $\rho_v$ computed over gross section is:
  - 0.0012 for deformed bars not larger than № 5 (5/8") Ø 16M (16 mm), with $f_y$, not less than 420 MPa.
  - 0.0015 for other deformed bars.
  - 0.0012 for welded wire reinforcement with diameter not larger than 16 mm.

- 14.3.3 – Minimum ratio of horizontal reinforcement area to gross concrete area, $\rho_h$:
  - 0.0012 for deformed bars not larger than Nº 5 (5/8") Ø 16M (16 mm), with $f_y$, not less than 420 MPa.
  - 0.0025 for other deformed bars.
  - 0.0012 for welded wire reinforcement with diameter not larger than 16 mm.

Difference between wall and column

- 14.3.6 — Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

21.9 - Special structural walls and coupling beams

At least two curtains of reinforcement must be used in a wall if $V_u$ exceeds:

$$0.17 \lambda A_{cv} \sqrt{f'_{ct}} (\text{MPa}) = 0.53 \lambda A_{cv} \sqrt{f'_{ct}} (\text{kgf/cm}^2)$$

21.9 - Special structural walls and coupling beams

Nominal shear strength must not exceed:

$$V_{n} = A_{cv} \left[ \alpha_c \lambda \sqrt{f'_{ct}} + \rho_l f_y \right]$$

where $\alpha_c$ is:
Wall boundary elements
- Boundary elements must be placed at edges and around openings when inelastic response is expected. ACI 318-08 gives two alternatives to define if boundary elements are needed:
  1) Section 21.9.6.2 presents a displacement-based procedure. Boundary elements are needed or not depending on the compressive strain at the edge of wall caused by the seismic lateral deflection, or
  2) Section 21.9.6.3 requires boundary elements when the compressive stress at the edge of wall caused by the seismic forces exceeds a threshold value.

Displacement-based boundary element procedure in ACI 318 (21.9.6.2)
- This procedure is based on the compressive strain demand at edges of wall when the wall is deformed under the maximum expected lateral displacement caused by the design earthquake ground motion.
- Section 21.9.6.2 is based on the assumption that inelastic response of the wall is dominated by flexural action at a critical, yielding section.
- The wall should be proportioned so that the critical section occurs at the base of the wall and is applicable only to walls continuous from base to top of the structure.

Displacement-based boundary element procedure in ACI 318 (21.9.6.2)
- The wall should have a single critical section under flexure and axial load at the base of the wall.
- The zones of the wall in compression must be provided with specially reinforced boundary elements when the depth of the neutral axis at nominal strength, \( c \), is greater than:
  \[
  c \geq \frac{\ell_{w}}{600.0 \left( \frac{\delta_u}{h_{w}} \right)} \quad \text{and} \quad \frac{\delta_u}{h_{w}} \geq 0.007
  \]

Nonlinear response of a wall
- The plasticification length is shown in the diagram.
**Nonlinear response of wall**

Using Moment-area theorems it is possible to show that the lateral deflection caused by curvature up to yield (green zone) is:

\[
\delta_y = \frac{1}{E} \int_{0}^{h_y} M(x) dx
\]

and the additional deflection caused by nonlinear rotation (orange zone) is:

\[
\delta_n = \frac{h_y}{E} \left( \phi_n - \phi_y \right)
\]

Total lateral deflection is then:

\[
\delta = \delta_y + \delta_n = \frac{1}{E} \int_{0}^{h_y} M(x) dx + \frac{h_y}{E} \left( \phi_n - \phi_y \right)
\]

**Nonlinear wall deflection**

The total deflection is:

\[
\delta = \frac{1}{E} \int_{0}^{h_y} M(x) dx + h_y \left( \phi_n - \phi_y \right)
\]

We can solve for the ultimate curvature demand and obtain:

- At level of displacement demand
- At level of nominal strength
- At level of yield in tension of extreme reinforcement

**Moment-curvature diagram for wall section**

**What happens at section?**

- Strain
- At level of displacement demand
- At level of nominal strength
- At level of yield in tension of extreme reinforcement

- Strain

- At level of displacement demand
- At level of nominal strength
- At level of yield in tension of extreme reinforcement
The rotation at the plastic hinge when the displacement demand ($\delta_u$) takes place is:

$$\theta_p = \frac{\delta_u}{b_p}$$

With a plastic hinge length equal to half the wall horizontal length:

$$\theta_p = \frac{\delta_u}{2}$$

Then the curvature at the wall base when the displacement demand occurs is:

$$\phi_u = \frac{\theta_p}{2} = \frac{\delta_u}{2b_p}$$

The concrete strain at the extreme fiber in compression at ultimate is:

$$\varepsilon_{cu} = \phi_u c$$

We can then obtain the strain at ultimate for the displacement demand:

$$\varepsilon_u = \frac{2}{\varepsilon_{cu}} \left( \frac{\delta_u}{b_p} \right)$$

The value of $c$ for an ultimate strain of $\varepsilon_{cu} = 0.003$ is:

$$c = \varepsilon_{cu} \left( \frac{2}{\varepsilon_u} \frac{\delta_u}{b_p} \right)$$

If a 600 value parameter is used instead of 666 in the last equation and it is solved for $\varepsilon_{cu}$, a value of $\varepsilon_{cu} = 0.0033$ is obtained, which in turn leads to the following equation:

$$c' = \frac{\varepsilon_u}{600} \left( \frac{2}{\varepsilon_{cu}} \frac{\delta_u}{b_p} \right)$$

If the maximum strain at the extreme compression fiber exceeds $\varepsilon_{cu} = 0.0033$ then the value of $c'$ obtained from the last equation would be exceeded. Thus the form ACI 318 presents it:

$$c' = \frac{\varepsilon_u}{600} \left( \frac{2}{\varepsilon_{cu}} \frac{\delta_u}{b_p} \right)$$

If $c'$ is greater than the value obtained, boundary elements must be placed along the length where it is exceeded and a little more.

Need for boundary elements in displacement-based procedure

If equation (21-8) indicates that the value of $c'$ is exceeded, this is a symptom that strains greater than $\varepsilon_{cu} = 0.0033$ must be expected and the need to confine the edge of the wall is warranted in order to prevent spalling of the concrete there.

In that case ACI 318 prescribes the same type and amount of confining transverse reinforcement that for columns.
Boundary elements displacement-base procedure

- **Displacement-based boundary element procedure in ACI 318 (21.9.6.32)**
  - Boundary elements must be placed from the critical section up for a distance not less than the larger of \( \ell_w \frac{M_u}{4V_u} \).
  - The evaluation is performed for the wall when subjected to the nonlinear horizontal design displacements corresponding to the design earthquake.
  - The value of \( \delta_u \) corresponds to the nonlinear roof horizontal displacement.

Stress-based boundary element procedure in ACI 318 (21.9.6.3)

- **Stress-based boundary element procedure in ACI 318 (21.9.6.3)**
  - Boundary elements must be provided at edges and around openings of walls when the maximum stress at the extreme fiber in compression caused by factored loads that include seismic effects exceeds \( 0.2 f'_{c} \) unless that whole wall is confined as a column.
    \[
    f_{cu} = \frac{P_o \pm \frac{M_o}{I_w} \ell_w}{A_w} > 0.2 f'_{c}
    \]

  - This procedure had been part of ACI 318 since the 1971 version.
  - In the 1999 version of 318 a modification was introduced in which the need to resist all flexural forces from seismic effects with just the boundary elements was suppressed.
**Old (pre-1999) procedure**

Boundary elements resisting all flexural effect that include seismic forces

\[ P_{ew} = \frac{P_k}{\phi} \left( \frac{M_k}{f_y} \right) \leq 0 \]

\[ P_{cu} = \frac{P_k}{\phi} \left( \frac{M_k}{f_y} \right) \leq 0 \]

\[ P_{ew} = \phi \cdot P_{cu} \]

\[ P_{cu} = 4 \cdot 0.85 \cdot f_y \left( \frac{A_k - A_v}{A_k} \right) + A_v \cdot f_y \]

\[ P_{cu} \leq 0.85 \cdot f_y \phi \cdot A_v \]

Z

**21.9 - Special structural walls and coupling beams**

**Boundary elements – Both procedures**

- When boundary elements are needed (under any of the two procedures) these boundary elements must extend horizontally from the maximum compression fiber a distance equal to the greater of: \( c - 0.1 \cdot A_w \) or \( c/2 \).
- In section with flanges the boundary element must include the effective flange width and must extend at least 300 mm into the web.
- Transverse reinforcement must be that required for column, but there is no need to comply with equation 21-3.
- Special transverse reinforcement in the boundary element must extend into the foundation element supporting the wall.
- Wall horizontal transverse reinforcement must be anchored into the confined boundary element core.

**21.9 - Special structural walls and coupling beams**

- In ACI 318-08, there are modifications in the requirements for coupling beams in walls.

**Coupling beams**
21.10 — Special structural walls constructed using precast concrete

- Scope — These requirements apply to special structural walls constructed using precast concrete forming part of the seismic-force-resisting system.

- Special structural walls constructed using precast concrete shall satisfy all requirements of special cast-in-place structural walls plus those of section 21.10.

- Special structural walls constructed using precast concrete and unbonded post-tensioning tendons and not satisfying the requirements of 21.10.2 are permitted provided they satisfy the requirements of ACI ITG-5.1.

21.11 — Structural diaphragms and trusses

This section contains:

- Requirements for slabs-on-grade, floor and roof slabs when they are part of the seismic-force-resisting system must comply with this section.
- Minimum thickness for diaphragms are given.
- Gives minimum reinforcement for diaphragms.
- Indicates shear strength for these elements.
- Defines when boundary elements must be used in diaphragms.
- Includes requirements for construction joints within the diaphragm.
21.12 — Foundations

This section contains:

- 21.12.1 — Scope - Foundations resisting earthquake induced forces or transferring earthquake-induced forces between structure and ground.
- 21.12.2 — Footings, foundation mats, and pile caps – Gives requirements for the anchoring of reinforcement in vertical elements of the seismic-force-resisting system to these foundation elements.
- 21.12.3 — Grade beams and slabs-on-ground – Sets minimum dimension s and minimum reinforcement for these elements,
- 21.12.4 — Piles, piers, and caissons – Indicates the type of effects to take into account in design and the minimum reinforcement allowable for these elements.

21.13 — Members not designated as part of the seismic-force-resisting system

This section a response to the extended practice by structural designers of designating arbitrarily some of the structural elements as being part of the seismic-force-resisting system and part not. Northridge Earthquake affecting the City of Los Angeles in 1994 pointed out great deficiencies in this practice. In ACI 318-85 this section was totally revised and it was updated in 1999, 2002, 2005, and now in 2008.

In essence it is a call to the designer to check the deformation levels that so called “non participating” elements are subjected and the minimum reinforcement they should comply with.

21.13 — Members not designated as part of the seismic-force-resisting system

This Section contains two procedures to check non-participating elements that are not part of the seismic-force-resisting system:

- When the forces induced by the design displacement combined with the gravity forces do not exceed the design strength of the elements, this section indicates the minimum reinforcement to use.
- If the strength is exceeded the sections of Chapter 21 that are mandatory for these elements are indicated.

21.13 — Members not designated as part of the seismic-force-resisting system

Slab-column frames have shown repeatedly their vulnerability under seismic demands. This vulnerability is specially associated with the punching shear strength of the slab-column joint.

The new procedure in ACI 318-08 (Section 21.13.6) indicates when shear reinforcement must be provided in the slab-column joint as a function of the story drift.
21.13 — Members not designated as part of the seismic-force-resisting system

Story drift cannot exceed the larger of:

- 0.005
- \( \left( 0.035 - 0.05 \frac{V_{ug}}{\phi V_c} \right) \)

where \( V_{ug} \) is the factored gravity punching shear demand and \( V_c \) is the punching shear strength.

The End