
# Important Considerations in Design of Primary Structural Components: Part 1

Organized by:

AIT Asian Institute of Technology

AIT Solutions
Technology · Engineering · Environment · Development · Management

ACECOMS

In collaboration with:

CSi COMPUTERS & STRUCTURES, INC.

Supported by:

Council on Tall Buildings and Urban Habitat

# Shear Walls

# Shear Walls

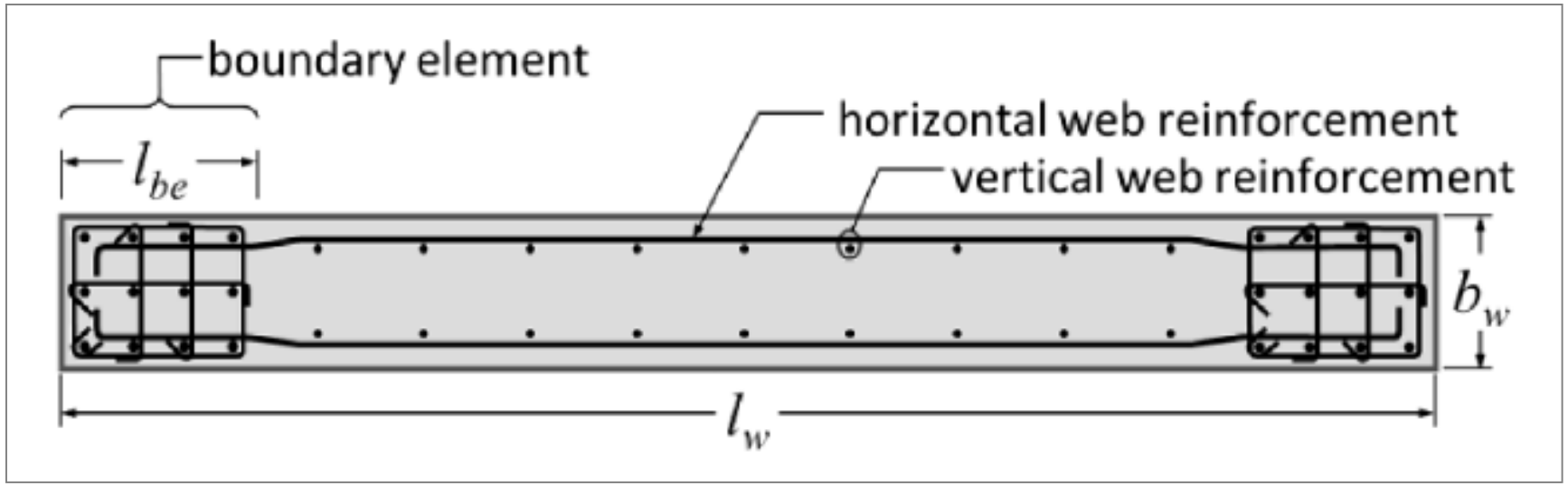
- Proportioned and detailed to resist shear, moment and axial force as a building sways through multiple displacement cycles during strong earthquake ground shaking
- Capable of resisting strong earthquake shaking without critical loss of stiffness or strength

# Wall Proportioning

- ACI 318 has no prescriptive minimum thickness.
- Minimum thickness of 200 mm (8 in.) is a practical lower limit for special structural walls.
- Construction and performance are generally improved if wall thickness is at least 300 mm (12 in.) where special boundary elements are used.
- Walls with conventional coupling beams require minimum thickness of 350 mm (14 in.).
- Walls with diagonal reinforced coupling beams require minimum thickness of 400 mm (16 in.)

# Wall Reinforcement

- Minimum reinforcement ratio , $\rho_l$ and $\rho_t = 0.0025$
- $\rho_l$ and $\rho_t$ are permitted to be reduced if $V_u \leq 0.5\ \Phi V_c$
- If $h_w/l_w \leq 2$, $\rho_t$ is not to be less than $\rho_l$




Source: NEHRP Seismic Design Technical Brief No. 6

# Minimum Reinforcement in Walls

**Table 11.6.1—Minimum reinforcement for walls with in-plane $V_u \leq 0.5\phi V_c$**

| Wall type | Type of nonprestressed reinforcement | Bar/wire size | $f_y$, psi | Minimum longitudinal[1], $\rho_\ell$ | Minimum transverse, $\rho_t$ |
|---|---|---|---|---|---|
| Cast-in-place | Deformed bars | ≤ No. 5 | ≥ 60,000 | 0.0012 | 0.0020 |
| | | | < 60,000 | 0.0015 | 0.0025 |
| | | > No. 5 | Any | 0.0015 | 0.0025 |
| | Welded-wire reinforcement | ≤ W31 or D31 | Any | 0.0012 | 0.0020 |
| Precast[2] | Deformed bars or welded-wire reinforcement | Any | Any | 0.0010 | 0.0010 |

[1]Prestressed walls with an average effective compressive stress of at least 225 psi need not meet the requirement for minimum longitudinal reinforcement $\rho_\ell$.

[2]In one-way precast, prestressed walls not wider than 12 ft and not mechanically connected to cause restraint in the transverse direction, the minimum reinforcement requirement in the direction normal to the flexural reinforcement need not be satisfied.

AIT Solutions
Technology · Engineering · Environment · Development · Management

Source: Table 11.6.1, ACI 318-14

# Principles for Special Structural Wall Design

- **Slender walls ($h_w/l_w \geq 2$)**
  - Tend to behave much like flexural cantilevers
  - Preferred inelastic behavior is ductile flexure yielding, without shear failure.

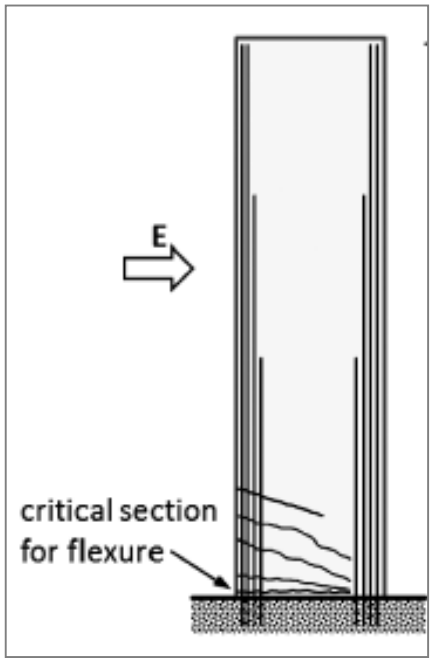
- **Squat walls ($h_w/l_w \leq 0.5$)**
  - Resist lateral force in diagonal strut mechanism
  - Concrete and distributed horizontal and vertical reinforcement resist shear

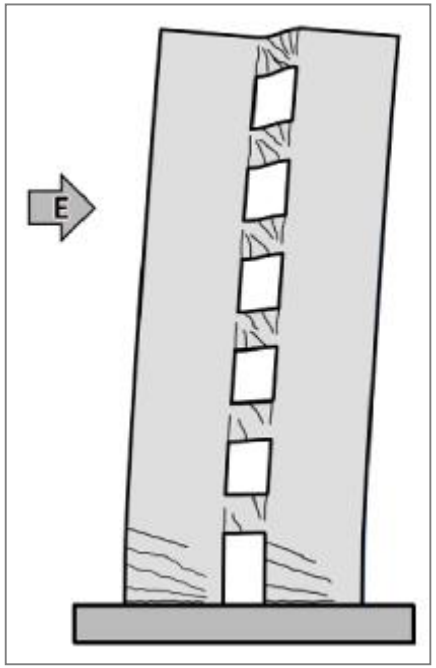
- Wall behavior transitions between above extremes for intermediate aspect ratios.

# Slender Walls

# Yield Mechanism

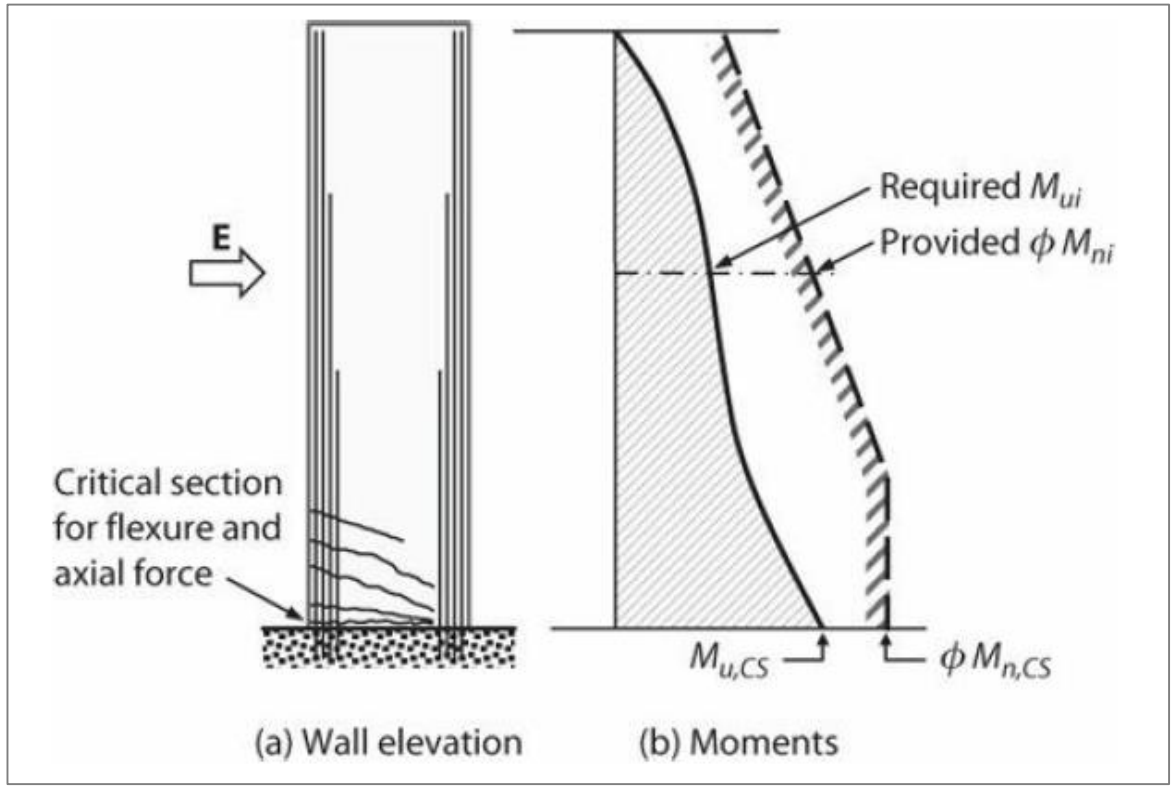
- For slender walls, ductile flexural yielding at the base of wall
- For slender coupled walls, ductile yielding of coupling beams plus ductile flexural yielding at the base of wall.
- Wall shear failure, diaphragm failure and foundation failure should be avoided.




# Yield Mechanism

- Design selected critical section to have strength in flexure and axial closely matching the required strengths, with some overstrength provided at other locations.
- Special details for ductile response can be concentrated around the selected critical section, with relaxed detailing elsewhere.
- In very tall buildings, higher mode response may cause some flexural yielding in intermediate stories.




Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Achieve Ductile Flexural Yielding

- Key factors for improving cyclic ductility
  - Keep global compressive and shear stresses low.
  - Design confined, stable flexural compression zones.
  - Avoid splice failures.
- Axial force well below the balanced point
- UBC 97
  - $P_u \leq 0.35\ P_0$, which corresponds approximately to the balanced axial force in a symmetric wall.
- ACI 318
  - No limit for wall axial force.
  - Avoid concrete reaches strain of 0.003 before tension reinforcement yields.

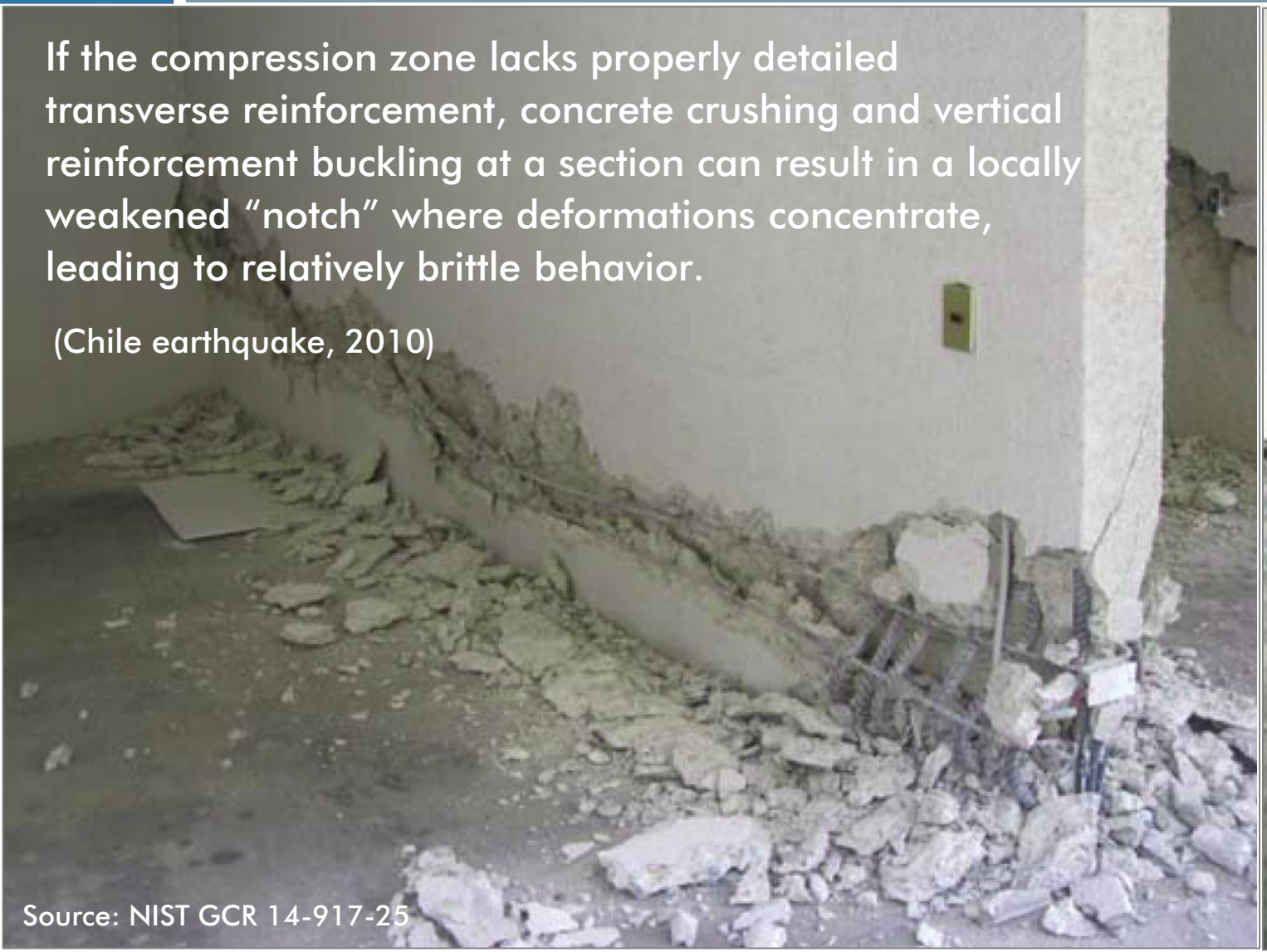
# Achieve Ductile Flexural Yielding

- Good design practice aims for design shear not exceeding **$0.33\Phi\sqrt{f'_c}\,A_{cv}$ to $0.5\Phi\sqrt{f'_c}\,A_{cv}$** so that ductility capacity is not overly compromised.
- Where flexural ductility demands are low, higher nominal shear stresses can be tolerated.

# Concrete Crushing and Reinforcement Buckling

If the compression zone lacks properly detailed transverse reinforcement, concrete crushing and vertical reinforcement buckling at a section can result in a locally weakened "notch" where deformations concentrate, leading to relatively brittle behavior.

(Chile earthquake, 2010)

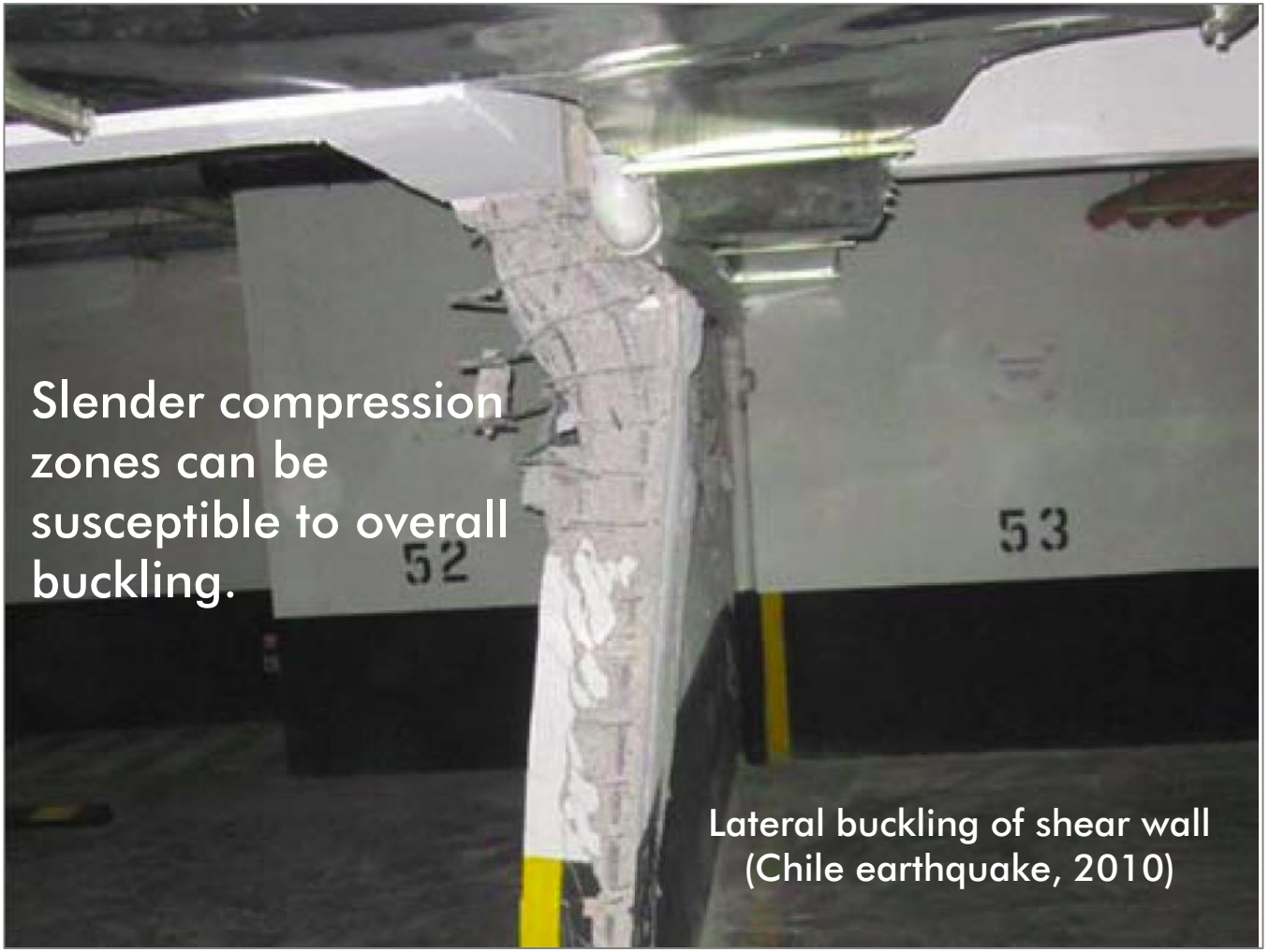


Source: NIST GCR 14-917-25

# Lap Splices

- Lap splices of vertical reinforcement can result in a locally strengthened section, such that yielding, if it occurs, may be shifted above or below the lap.
- Lap splices subjected to multiple yielding cycles can "unzip" unless they are confined by closely spaced transverse reinforcement.
- ACI 318 requires splice lengths at least 1.25 times lengths calculated for $f_y$ in tension, with no requirement for confinement.
- Better practice either prohibits lap splices in the intended hinge zone or provides confining transverse reinforcement along the splice length.

# Lateral Buckling




- Although global wall buckling occurs when the wall boundary is in compression, buckling may be influenced by residual tensile strain in the wall due to prior loading in the opposite direction.
- If the boundary yields in tension, cracked sections are produced, with crack width dependent on the amplitude of the reinforcement tensile strain $\varepsilon_{sm}$ during the tension excursion.

Source: NIST GCR 14-917-25

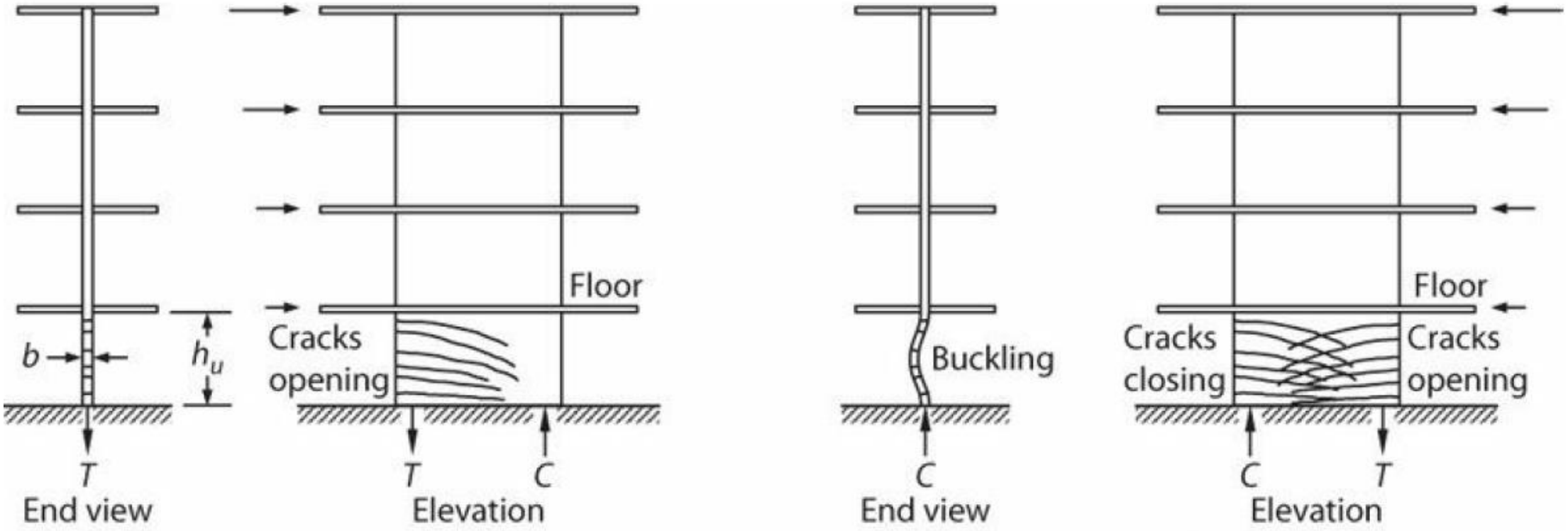
# Lateral Buckling

- In a previously yielded wall, crack closure under deformation reversal may require yielding of the longitudinal reinforcement in compression.
- In a wall with two curtains of reinforcement, any slight asymmetry in the reinforcement will result in one curtain yielding before the other, leading to out-of-plane curvature and a tendency to buckle out of plane.

Lateral buckling of shear wall (Christchurch earthquake, 2011)

Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Lateral Instability of Wall Boundary Previously Yielded in Tension




(a) Crack opening under tension cycle

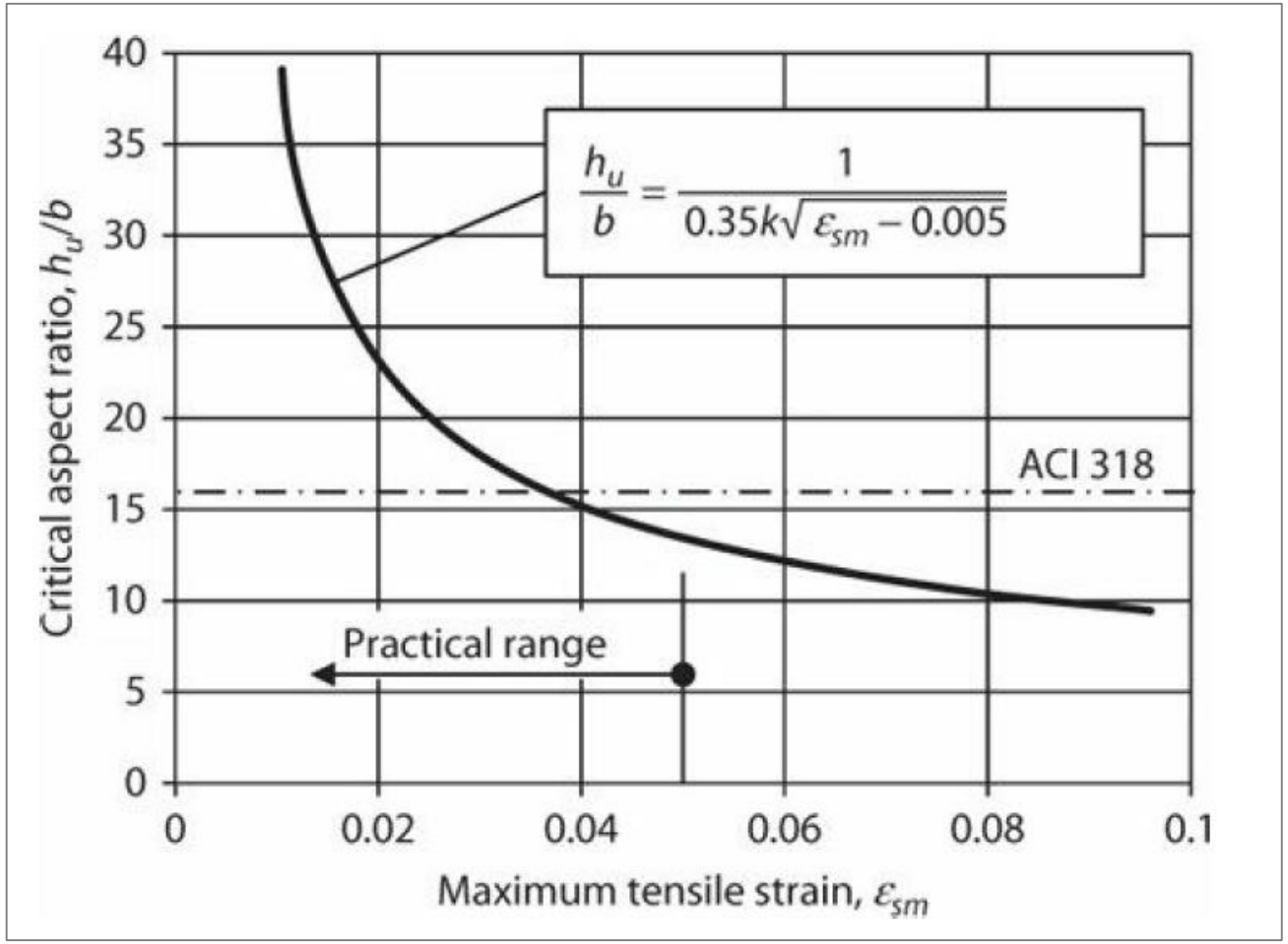
(b) Crack closing under compression cycle

Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Critical Slenderness Ratio for Wall Boundaries

In U.S. practice,

- $h_u$ / b ≤ 10 within intended hinge region
- $h_u$ / b ≤ 16 elsewhere

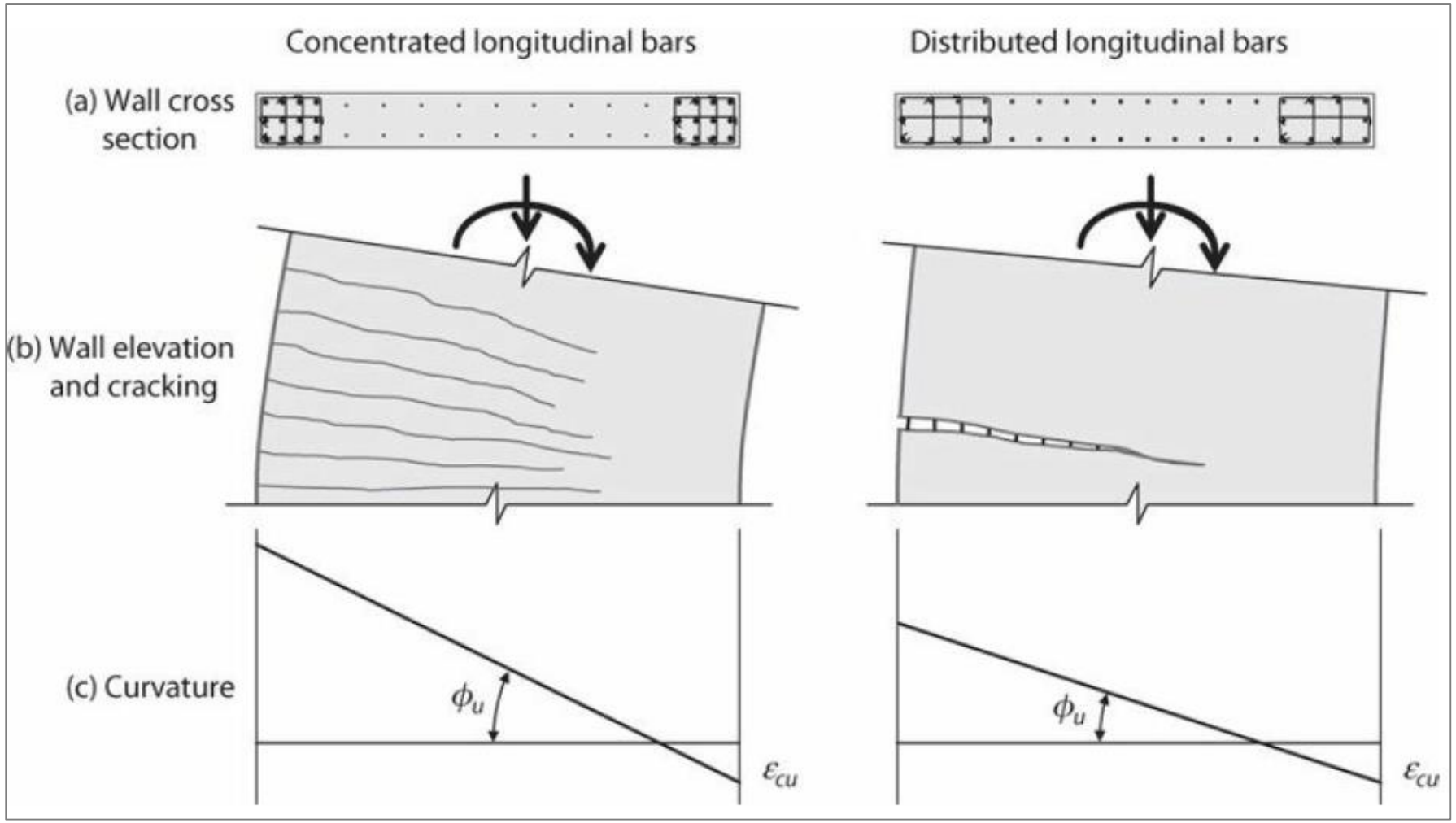



Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Advantages to Concentrating Reinforcement at Boundaries

- Provides a convenient zone for securing transverse reinforcement for confinement.
- Enhance curvature capacity moderately.
- Local reinforcement ratio at the wall edge is increased, promoting better distribution of flexural cracks, yielding over greater wall height, and increased displacement capacity.

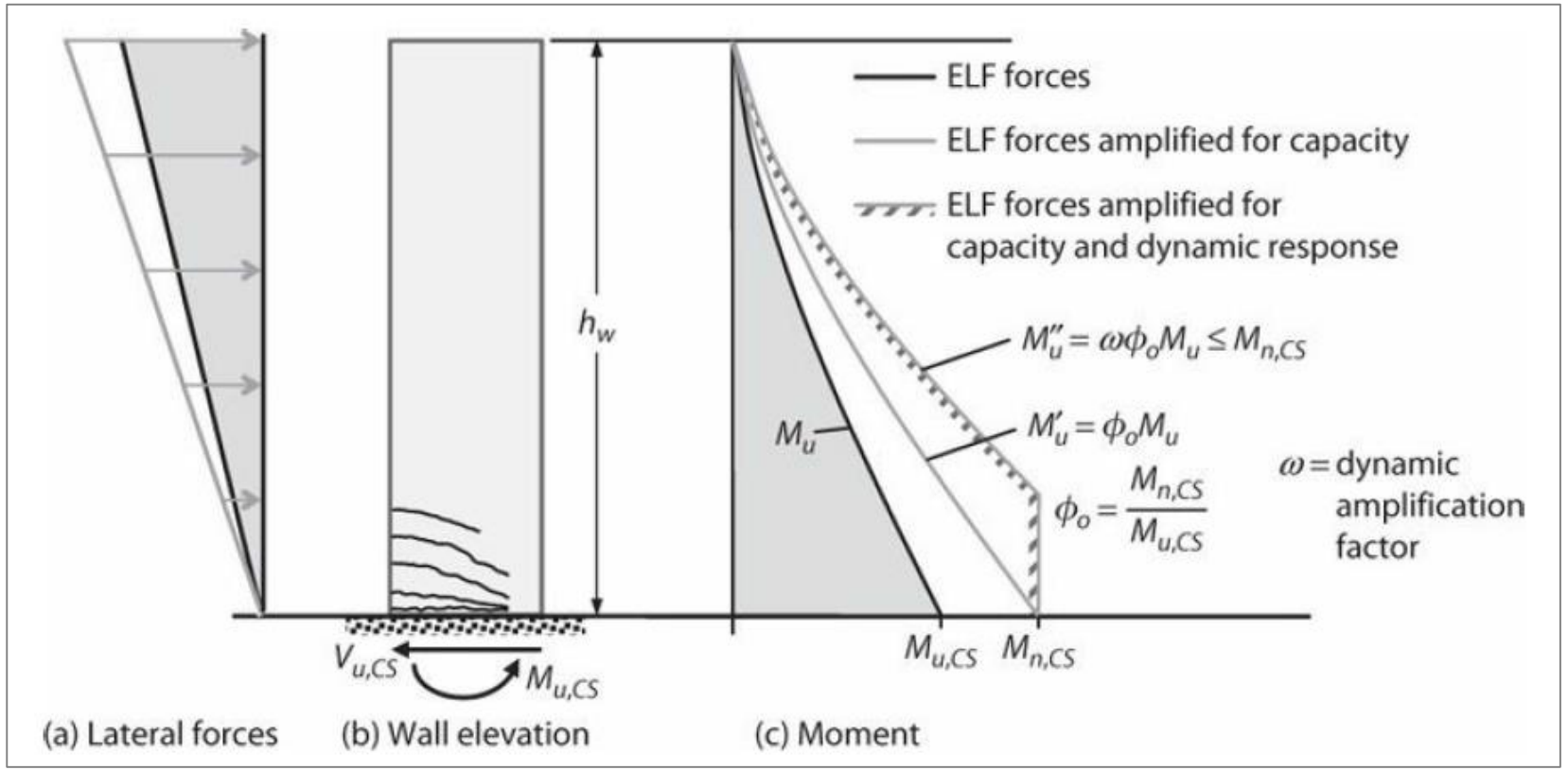
# Advantages to Concentrating reinforcement at Boundaries




- Where reinforcement is uniformly distributed, the local reinforcement ratio at the wall edge is likely to be low.
- In extreme cases, the tensile strength of reinforcement at a cracked section may be less than the concrete tensile strength in adjacent sections, such that only one or few cracks form, leading to localized yielding.

Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Moment Design




Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

Determine design moment, $M_U$.

↓

Select location for primary flexural yielding.

↓

Design that critical section to satisfy $M_{pr,CS} \ge M_{u,CS}$

↓

Define a flexural overstrength factor, $\Phi_0 = M_{n,CS}/M_{u,CS}$.

↓

Amplify design moments, based on overstrength, $M'_u = \Phi_0 M'_u$

↓

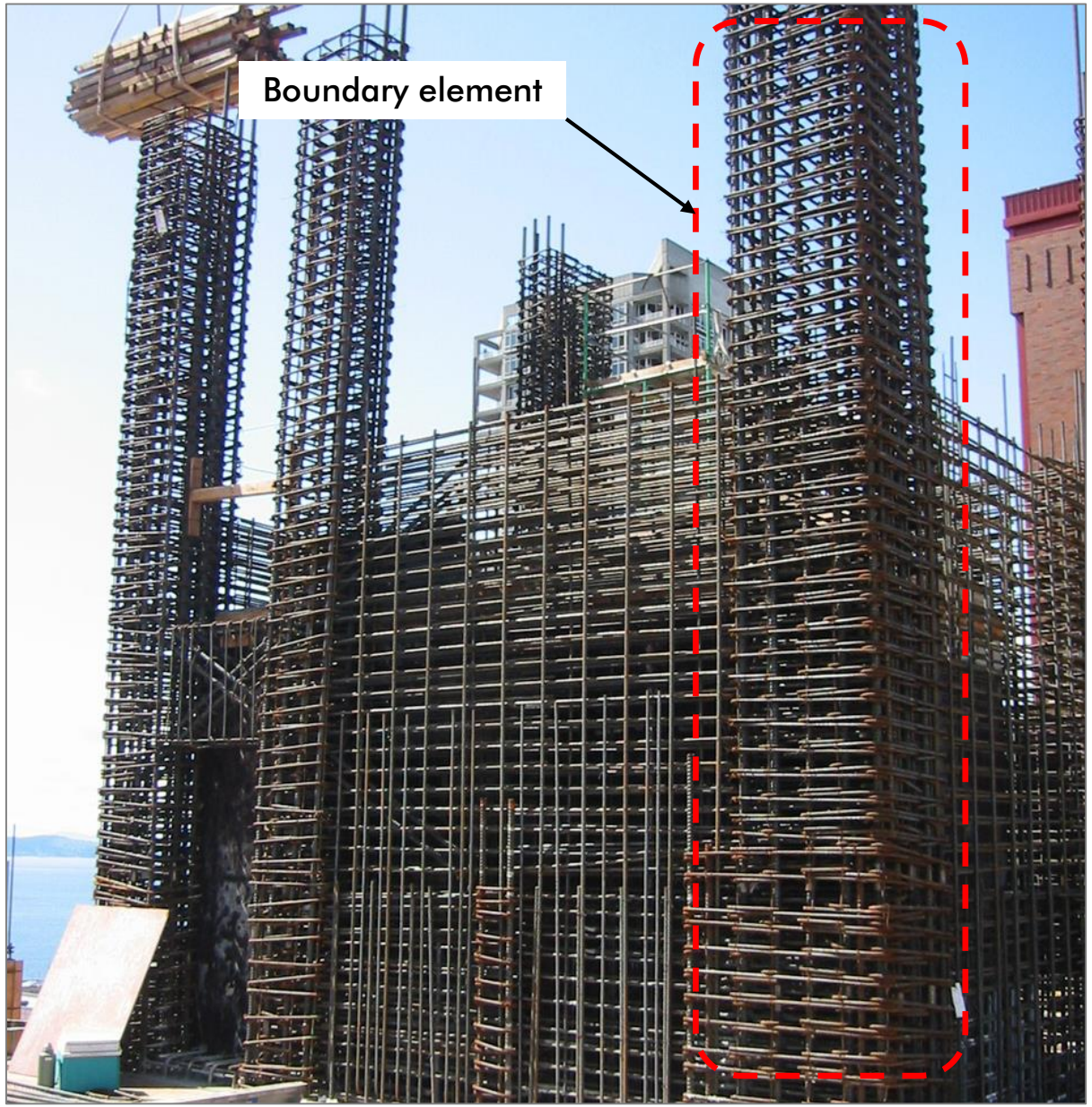
Amplify design moments by a dynamic amplification factor $\omega$, $M''_u = \omega\Phi_0 M'_u$

↓

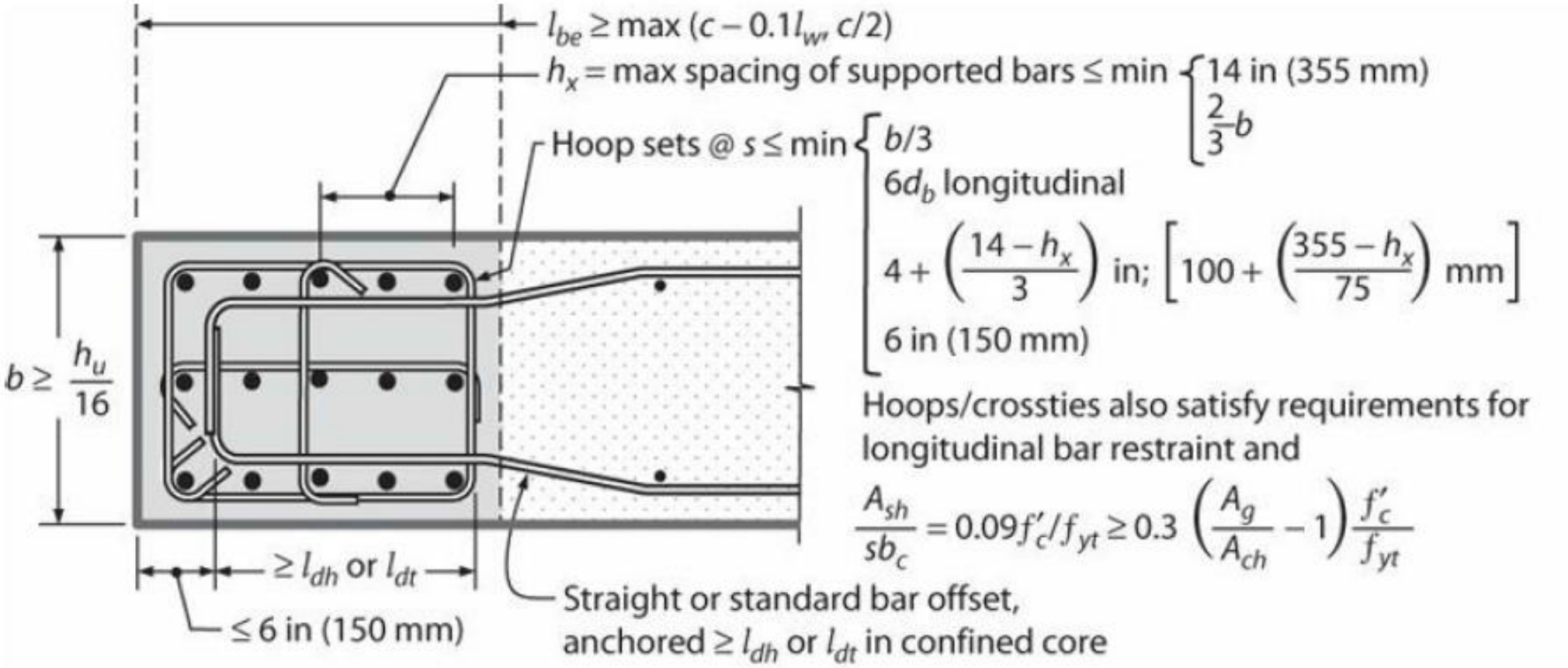
Provide $M_n \ge M''_u$ at every elevation outside the intended plastic hinge zone.

# Boundary Elements

- Where compressive demands are higher on the edge, ACI 318 requires a special boundary element.
- Where compressive demands are lower, special boundary elements are not required, but ordinary boundary element transverse reinforcement still is required if the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$, psi ($2.8/f_y$, MPa).
- If nominal concrete stress exceeds $0.2\ f'_c$, boundary element required.
- Extend until compressive stress drops below $0.15\ f'_c$.

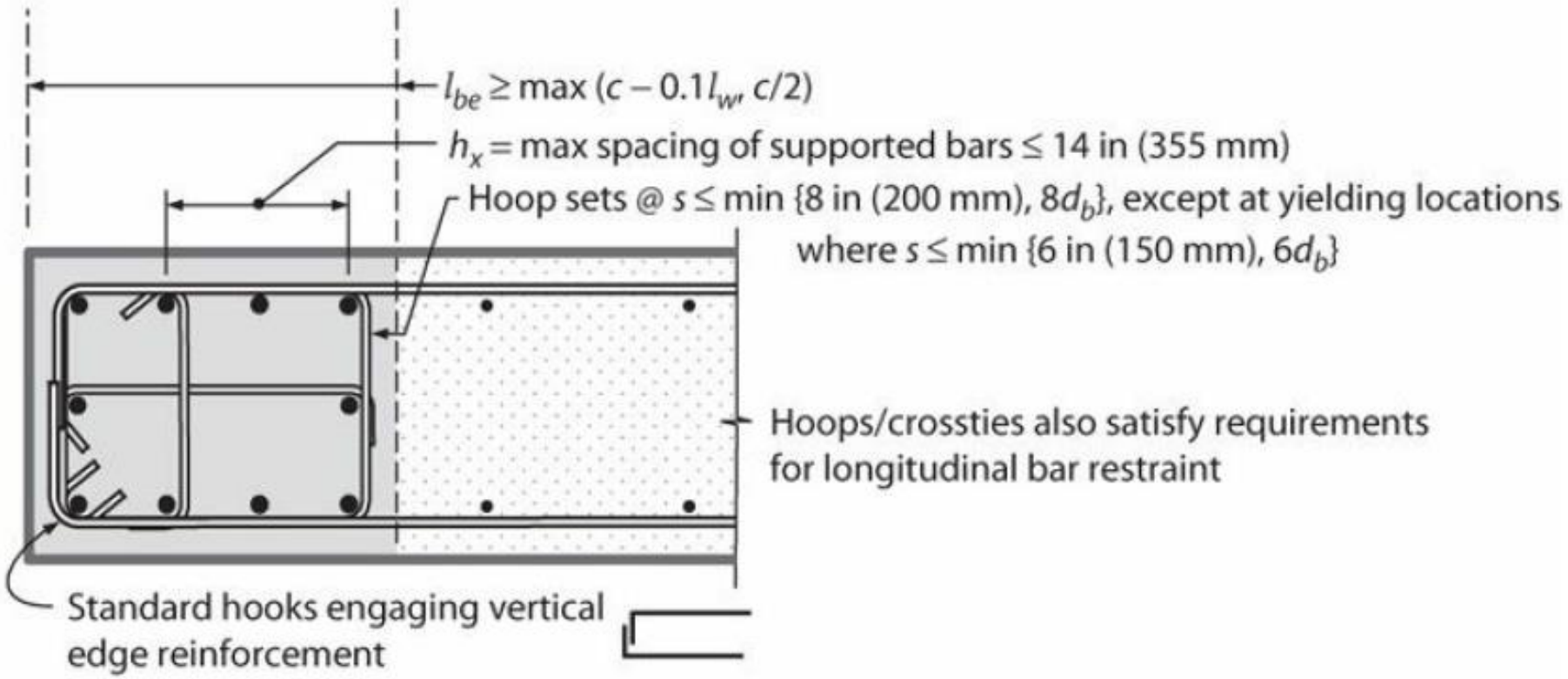


# Special Boundary Element



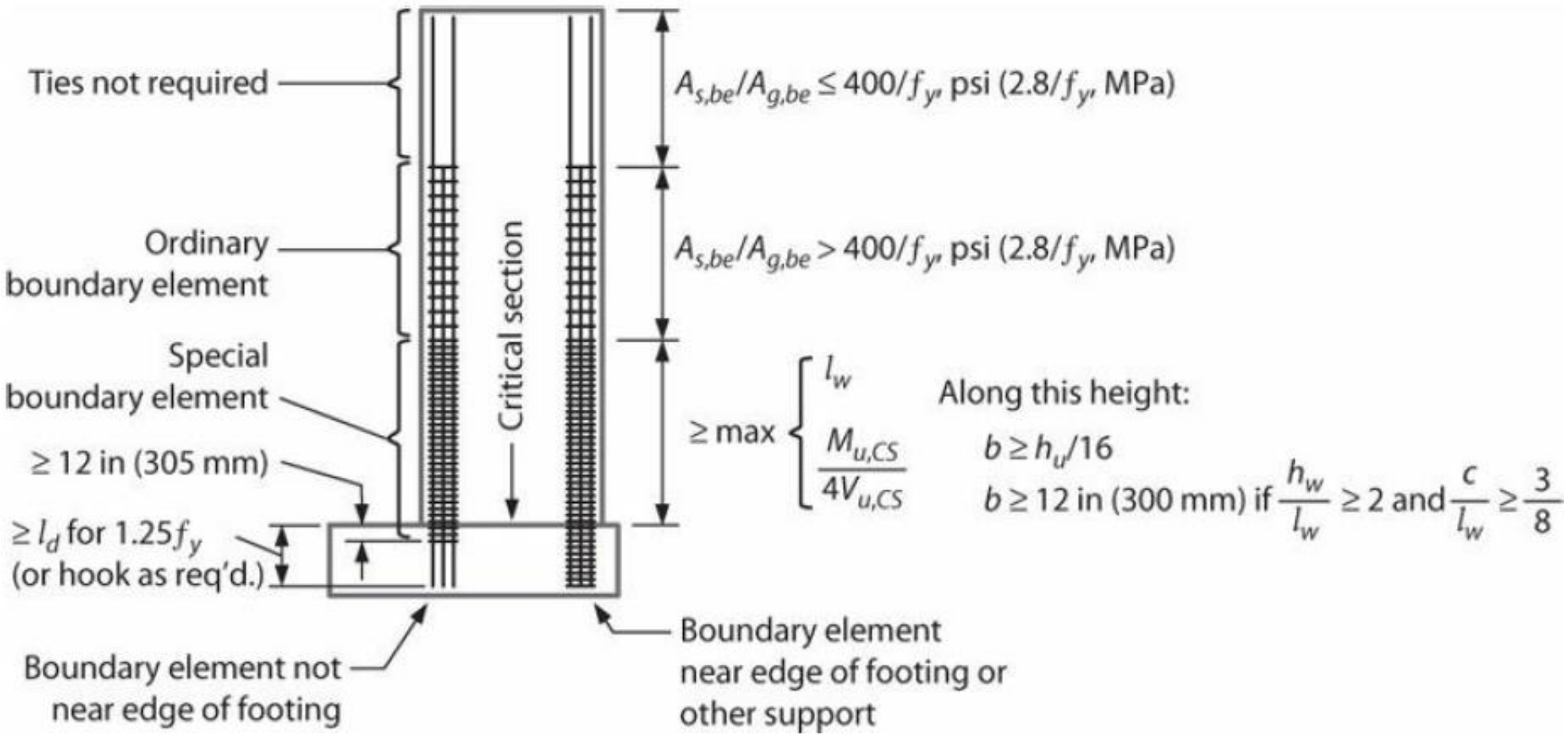

Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Ordinary Boundary Element




Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Boundary Element




Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Vertical Reinforcement Layout

Determine type of boundary element required.

↓

Determine boundary element length

↓

Select trial boundary element reinforcement size and spacing

↓

Select trial size and spacing of vertical reinforcement

↓

Determine P-M strength

↓

Use P-M analysis to check assumed boundary element length

# Alternative Procedure

- Select the vertical reinforcement and spread it within the required boundary element length.
- Layout the transverse reinforcement to support verticals and confine the core.
- Iterate until all requirements are met.

# Shear Design

- Wall shears $V_u$ are determined either from modal response spectrum analysis or equivalent lateral force (ELF) analysis, with seismic forces reduced by response modification coefficient R.
  - If these shears are used for design without further modification, use $\Phi = 0.6$.
  - If the shears are amplified by overstrength factor $\Phi_0 = M_{n,CS}/M_{u,CS}$, $\Phi = 0.75$.
- Considering that a yielding wall is likely to develop a moment closer to probable moment strength, overstrength factor $\Phi_0 = M_{n,CS}/M_{u,CS}$, would be more appropriate.
- Typical values of the ratio $M_{pr,CS}/M_{u,CS}$ are 1.5 or greater, such that use of this amplification factor along with $\Phi = 0.75$ generally will result in much more conservative design than the other option (unamplified shears with $= 0.6$).

# Shear Design

- $V_n = A_{cv} (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y)$
  - $A_{cv} = l_w b_w$
  - $\alpha_c = 0.25$ MPa if $h_w/l_w \leq 1.5$
  - $\alpha_c = 0.17$ MPa if $h_w/l_w \geq 2.0$
- Upper limit
  - $0.66A_{cv} \sqrt{f'_c}$, for all vertical wall segments resisting common lateral force
  - $0.83A_{cv} \sqrt{f'_c}$, for individual vertical wall segments

# Wall or Column?

- Referring to ACI 318-14, Chapter 2, wall is defined as a vertical element designed to resist axial load, lateral load, or both, with a horizontal length-to-thickness ratio greater than 3.
- In U.S. practice, a wall pier is defined as a relatively narrow vertical wall segment that is essentially a column, but whose dimensions do not satisfy requirements of special moment frame columns (ratio of width to depth is at least 0.4).

# Wall or Column?

- If $l_w/b_w \leq 2.5$, wall segment shall be designed in accordance with Section 18.10.8.1, satisfying the special moment frame requirements for columns of 18.7.4, 18.7.5, and 18.7.6 (but alternate procedure is not allowed (a) to (f) of Section 18.10.1).
- If $2.5 < l_w/b_w \leq 6$, wall segment shall be designed in accordance with Section 18.10.8.1, satisfying specified column design requirements or alternate requirements (a) to (f) of Section 18.10.8.1.
- If $l_w/b_w > 6$, design as wall.

# Wall or Column

## Table R18.10.1—Governing design provisions for vertical wall segments[1]

| Clear height of vertical wall segment/length of vertical wall segment, $(h_w/\ell_w)$ | Length of vertical wall segment/wall thickness $(\ell_w/b_w)$ | | |
|---|---|---|---|
| | $(\ell_w/b_w) \leq 2.5$ | $2.5 < (\ell_w/b_w) \leq 6.0$ | $(\ell_w/b_w) > 6.0$ |
| $h_w/\ell_w < 2.0$ | Wall | Wall | Wall |
| $h_w/\ell_w \geq 2.0$ | Wall pier required to satisfy specified column design requirements; refer to 18.10.8.1 | Wall pier required to satisfy specified column design requirements or alternative requirements; refer to 18.10.8.1 | Wall |

[1] $h_w$ is the clear height, $\ell_w$ is the horizontal length, and $b_w$ is the width of the web of the wall segment.

Source: Table R18.10.1, ACI 318-14

# Alternate Procedure for Wall Piers ($2.5 < l_w/b_w \leq 6$)

a) Design shear force shall be calculated in accordance with 18.7.6.1 with joint faces taken as the top and bottom of the clear height of the wall pier. If the general building code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed $\Omega_o$ times the factored shear calculated by analysis of the structure for earthquake load effects.

b) $V_n$ and distributed shear reinforcement shall satisfy 18.10.4.

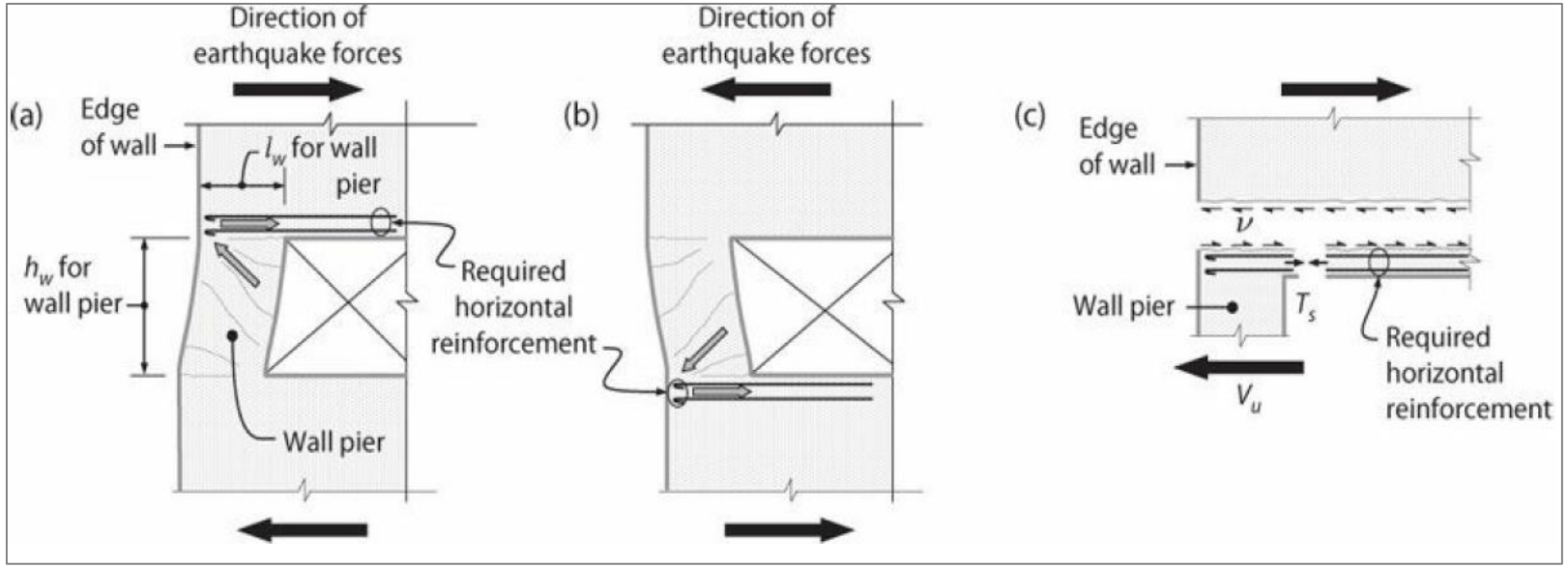
c) Transverse reinforcement shall be hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to $\ell_w$ where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180-degree bends at each end that engage wall pier boundary longitudinal reinforcement.

d) Vertical spacing of transverse reinforcement shall not exceed 150 mm.

e) Transverse reinforcement shall extend at least 300 mm. above and below the clear height of the wall pier

f) Special boundary elements shall be provided if required by 18.10.6.3.

# Wall Piers




Source: NIST GCR 14-917-25

# Wall Panel Zones

- A wall region in which forces from adjacent wall segments are resolved

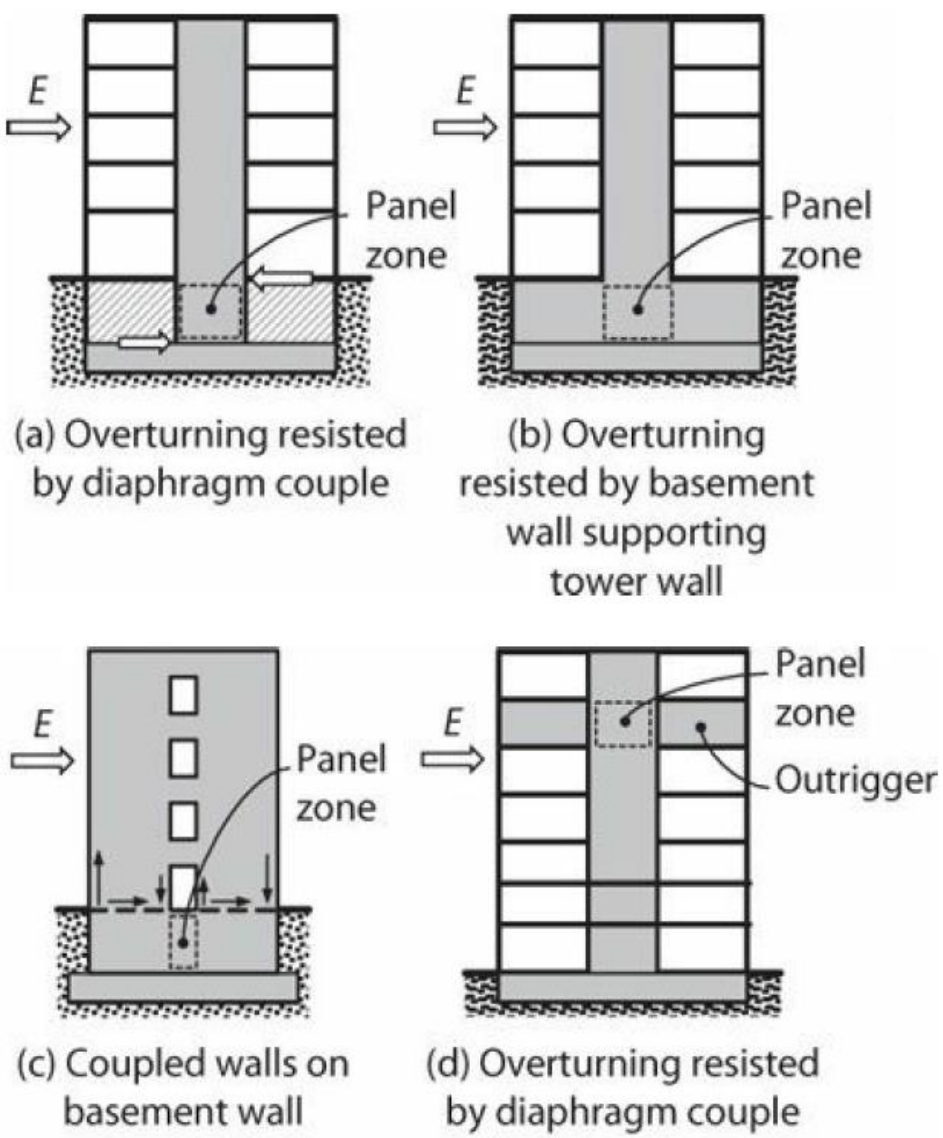
- **Panel zone shear strength**
  (recommended in "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle)

$$V_n = 0.25\sqrt{f'_c} + \rho_{smin} f_y \leq 0.2 f'_c$$

$\rho_{smin}$ = lesser of distributed reinforcement ratios in the vertical and horizontal directions

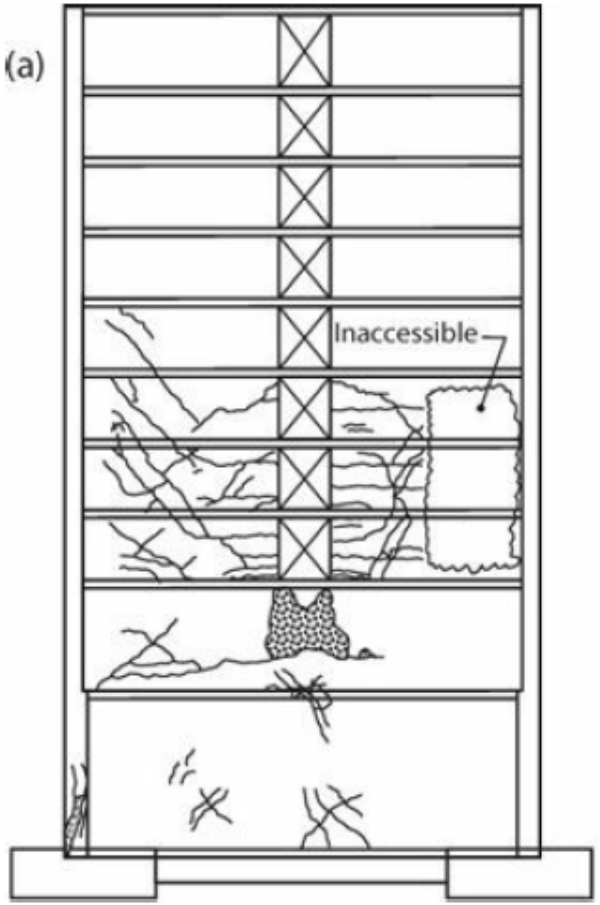
- **Shear stress limit**

$$v_u = \Phi\, 0.83\sqrt{f'_c}$$

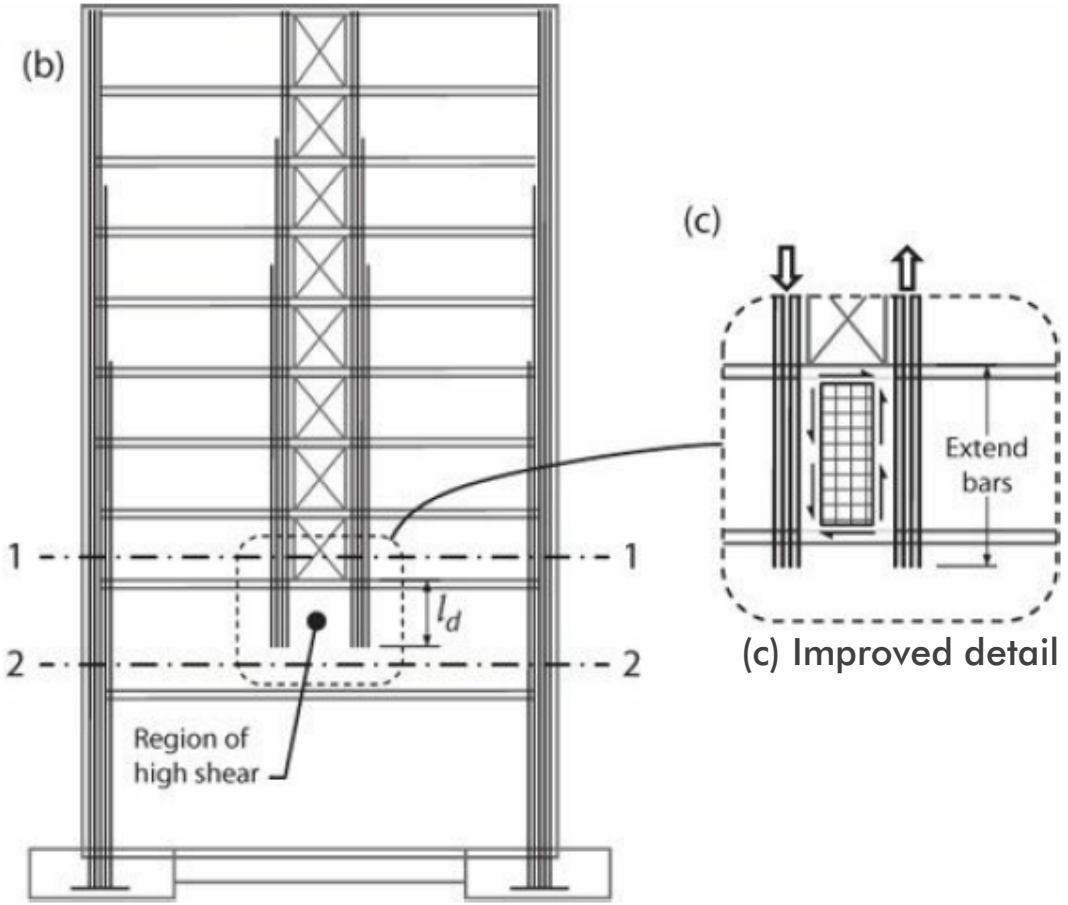



Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Wall Panel Zones




(a) Damage in wall from Loma Prieta, California Earthquake (1989)

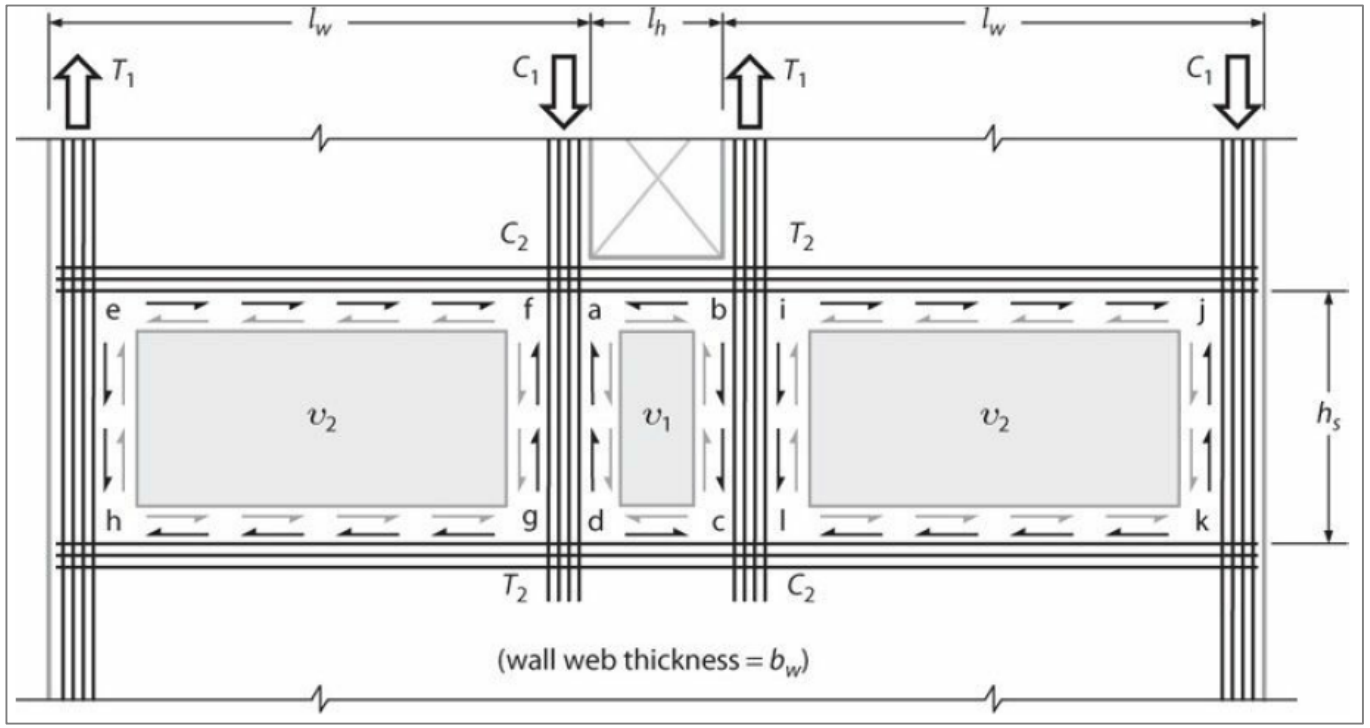



(b) Inappropriate termination of coupled wall boundary element

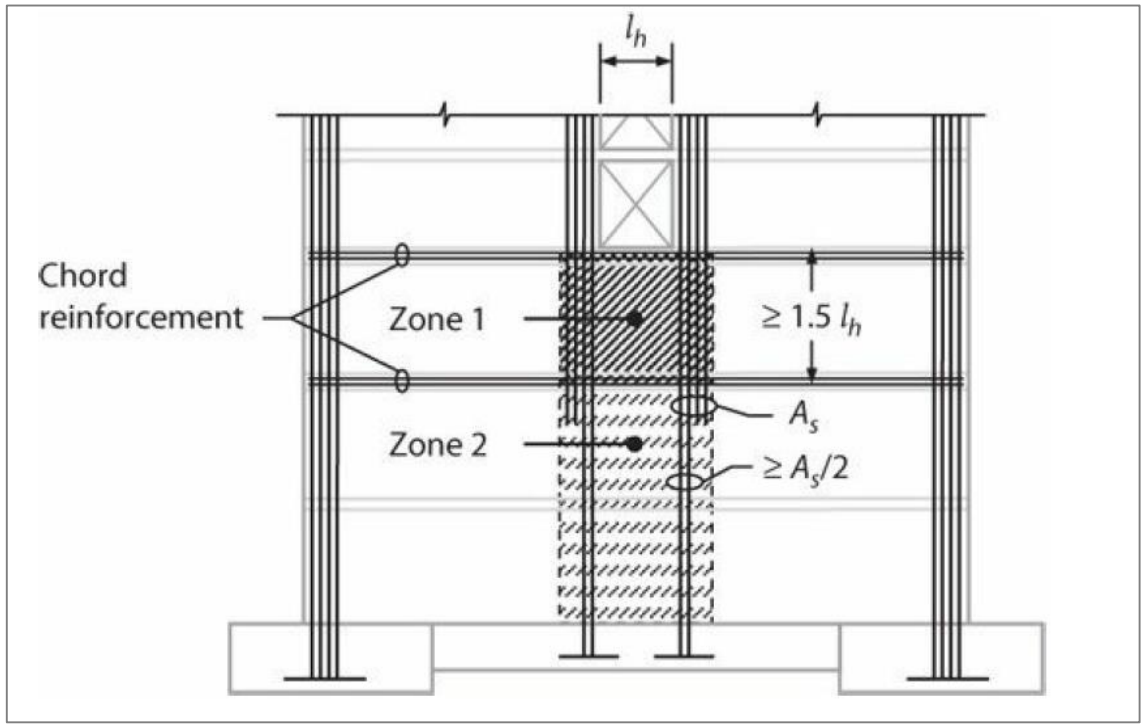
(c) Improved detail

Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Wall Panel Zones




Idealized resolution of wall forces



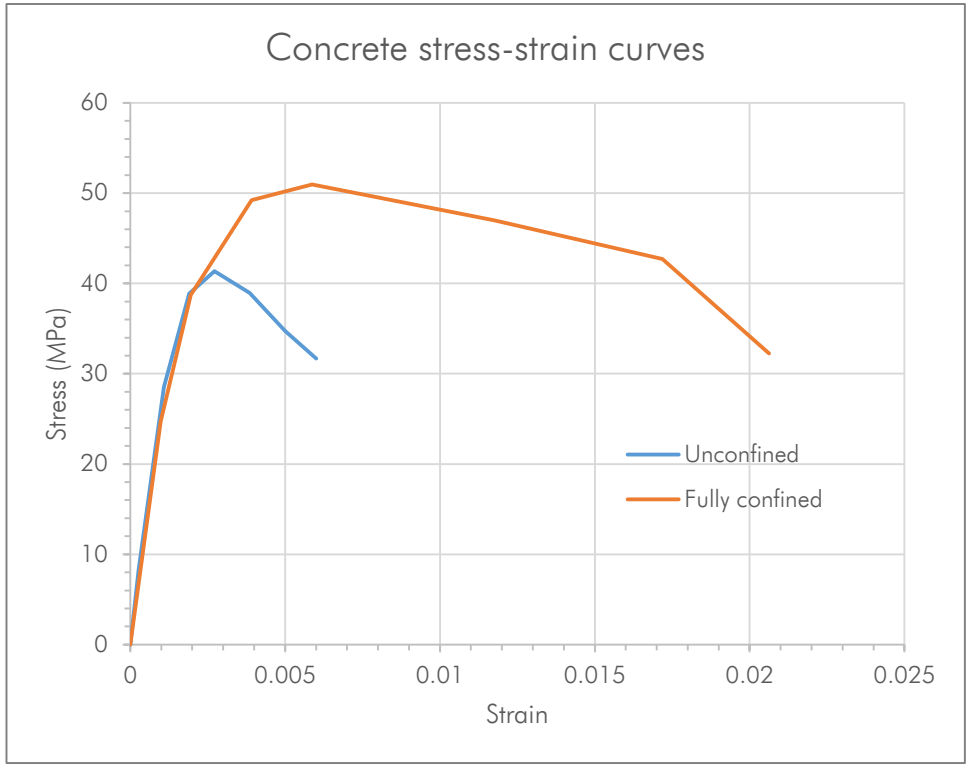

Suggested reinforcement details

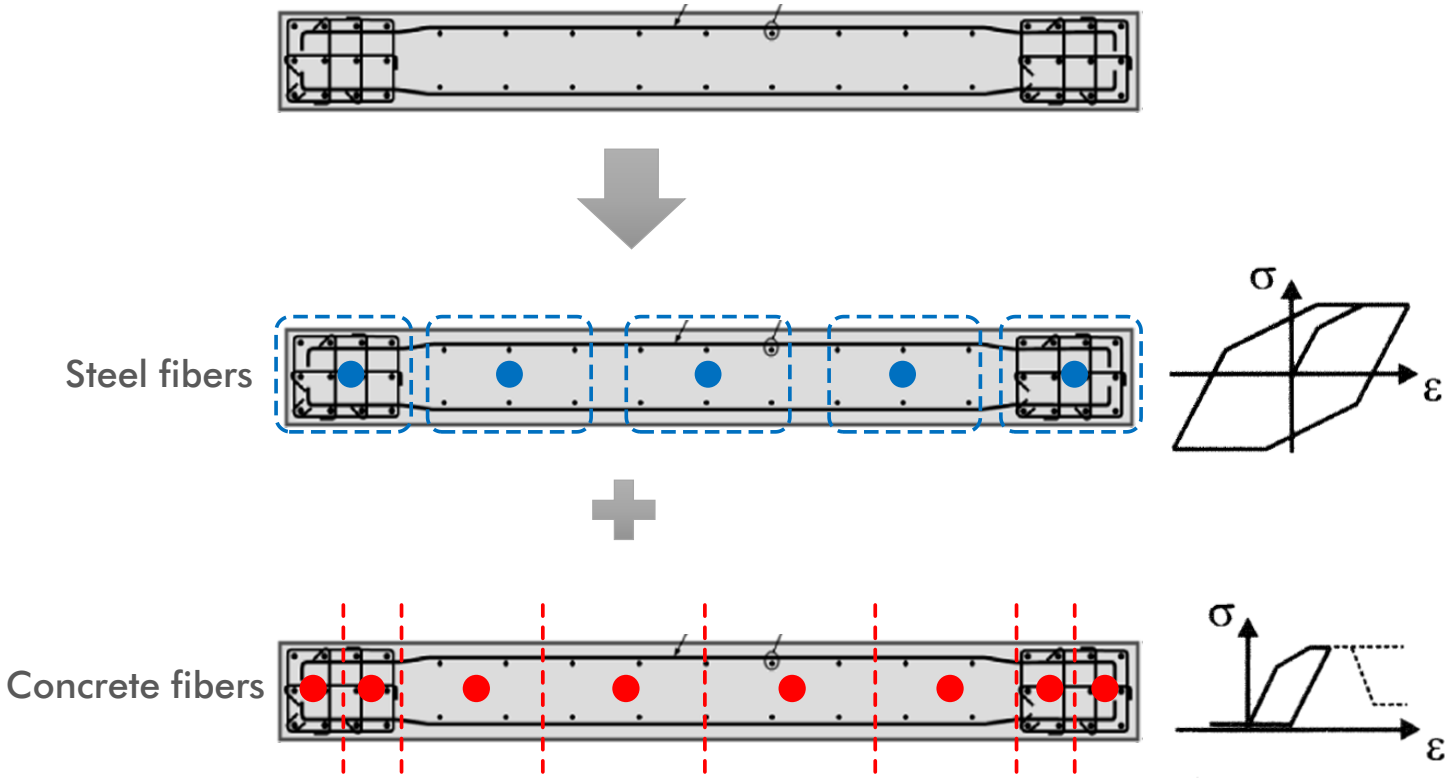
Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

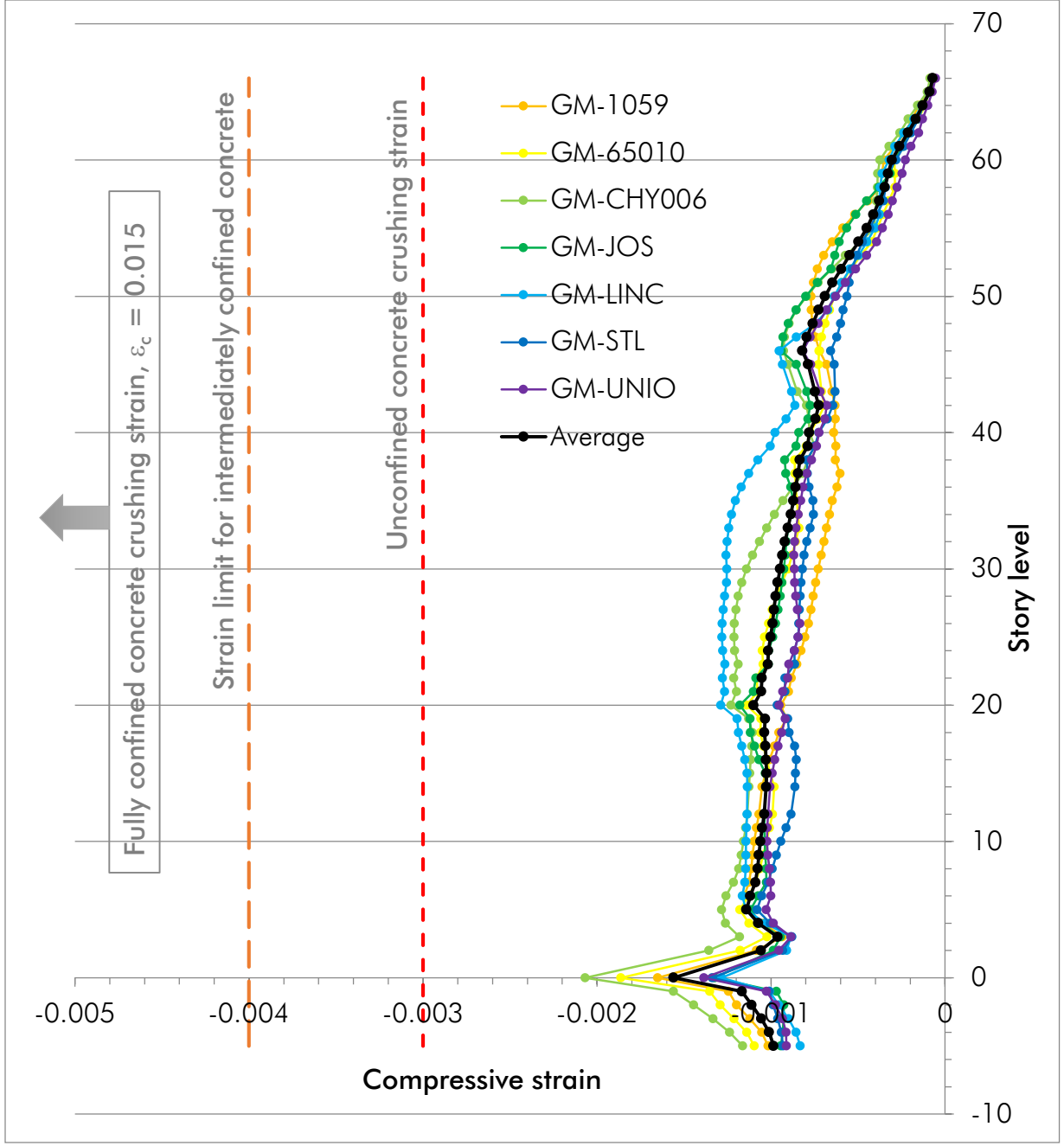
# Performance-based Evaluation of Shear Walls

- Modeling
  - Inelastic shear wall element
  - In-plane flexural response is considered as nonlinear, out-of-plane response is linear.
  - Single element with several fibers can be used.
  - Confinement effect in concrete is considered.
  - Shear response is modeled as linear.
- Evaluation
  - Axial strain of concrete and reinforcement (Flexural response)
  - Shear capacity check

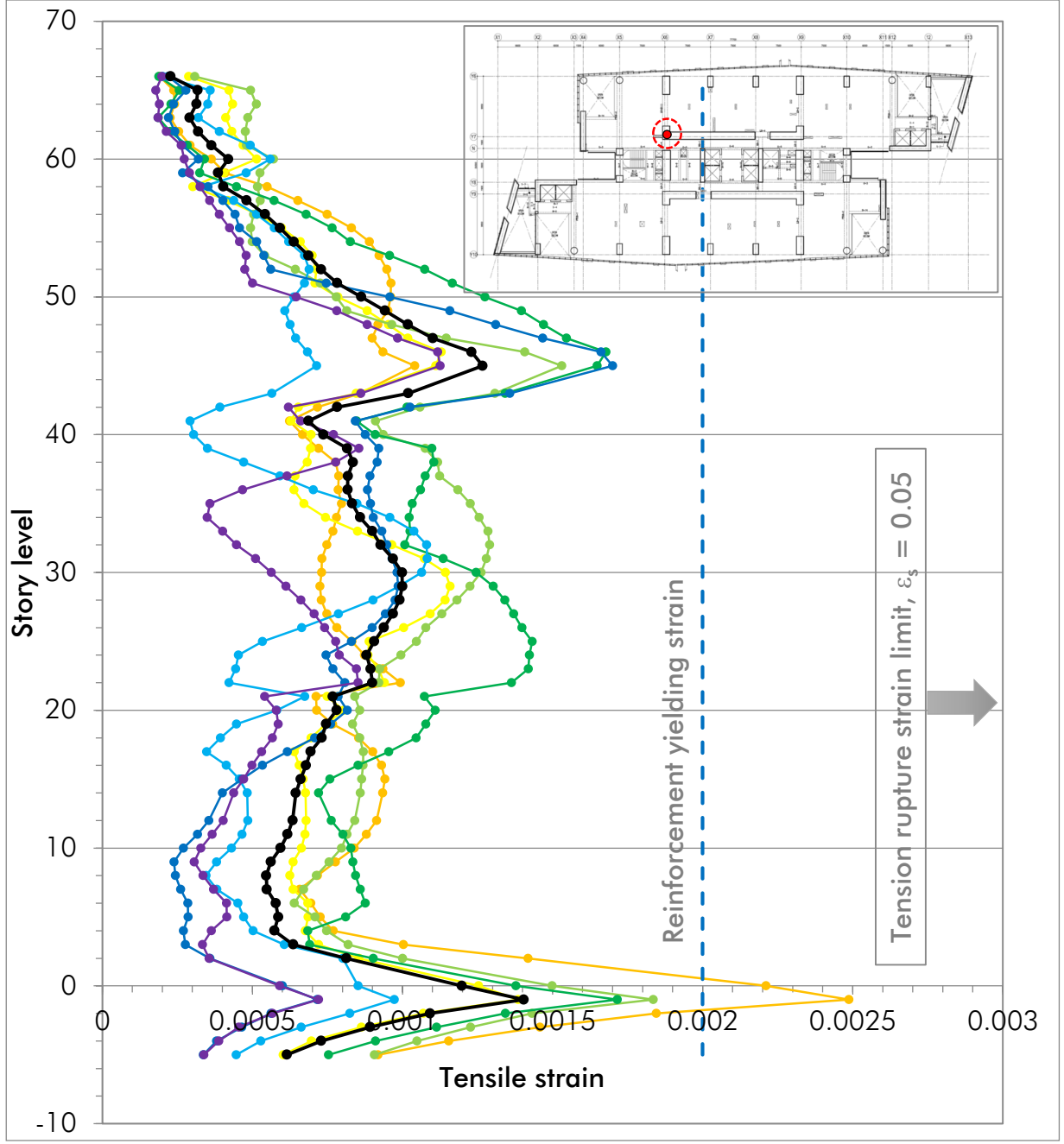
# Nonlinear Shear Wall Modeling



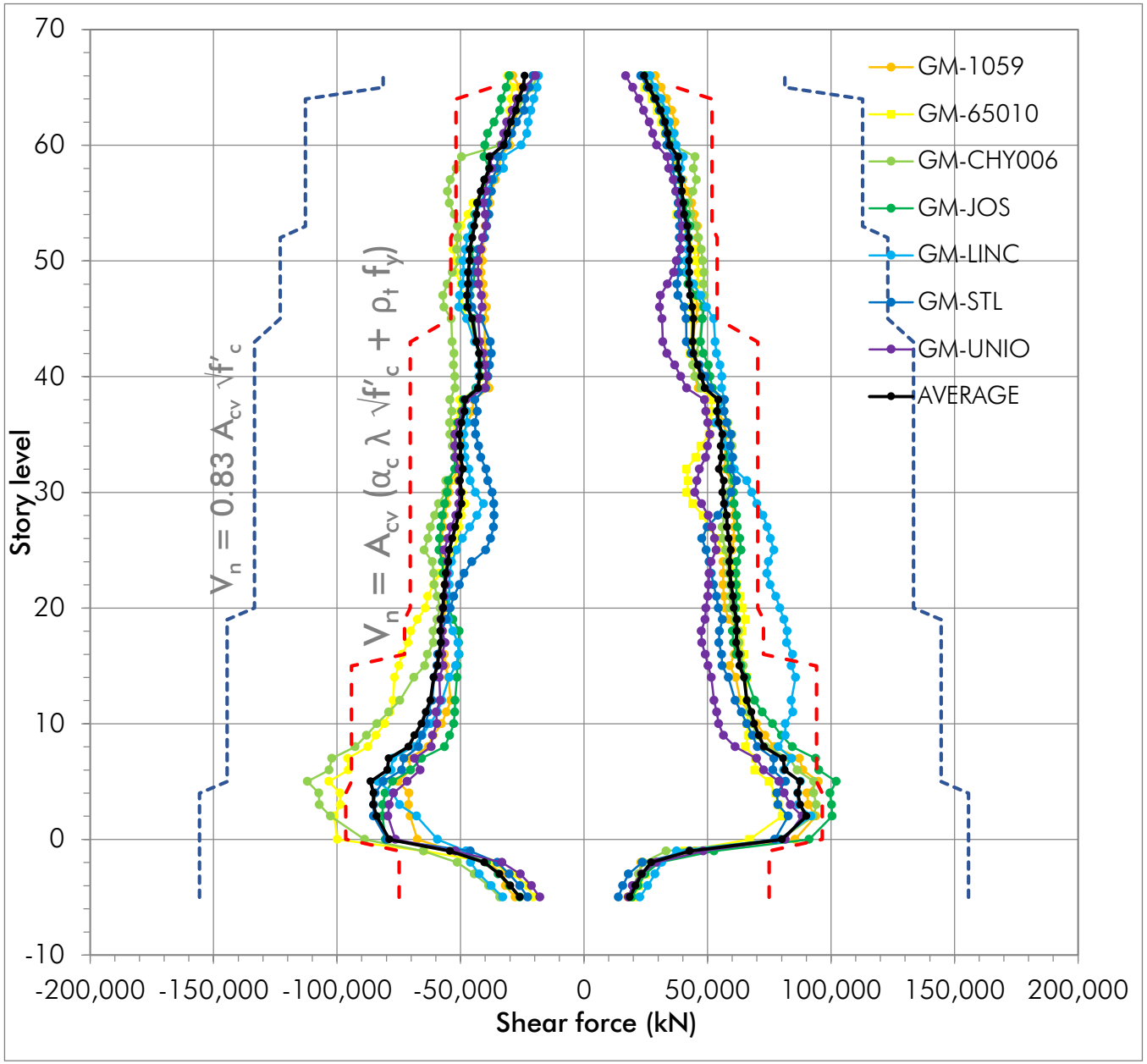



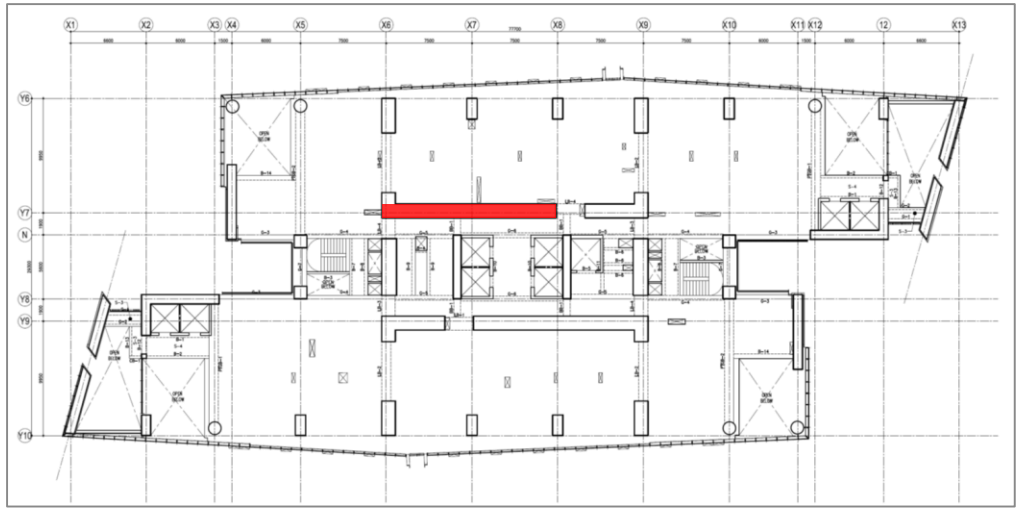



Compressive axial strain of concrete




Tensile axial strain of vertical rebar



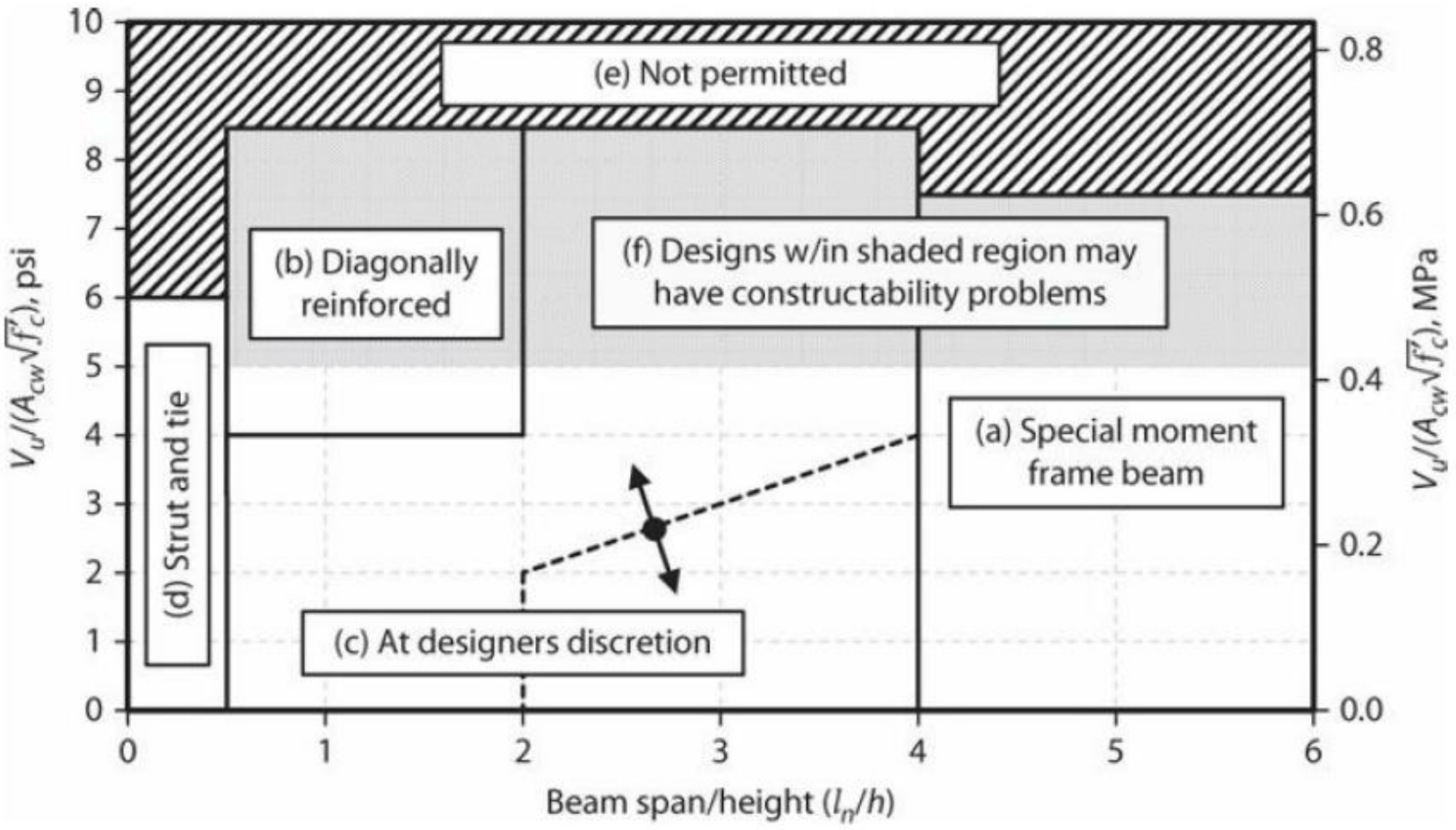
## Shear design check


# Coupling Beams

# Coupling Beams

- **Coupling beams ($l_n/h \geq 4$)**
  - Design as special moment frame beam (Conventional reinforced)
- **Coupling beams ($l_n/h < 2$ and $V_u > 0.33\lambda\sqrt{(f'_c)}\, A_{cw}$)**
  - Design as diagonal reinforced beams
- **Other coupling beams (Not falling within above two limits)**
  - Design as either special moment frame beam or diagonal reinforced beam

# Coupling Beams

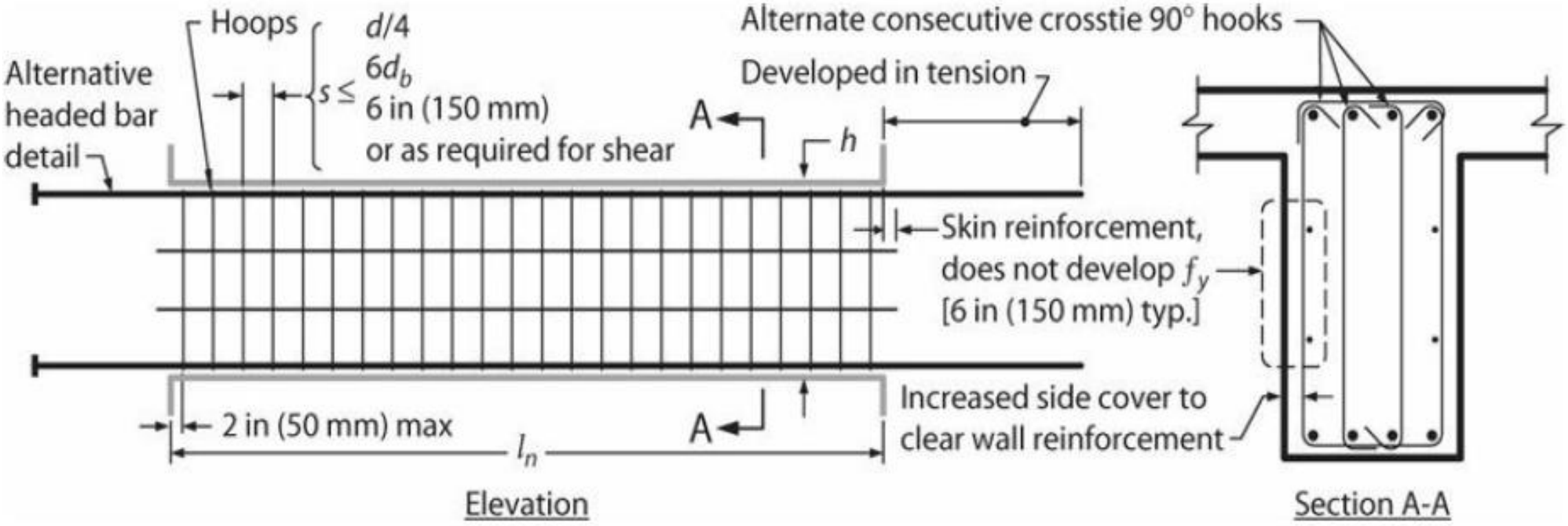



Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Conventional Reinforced Beam

- **Flexure Design**
  - Design as special moment frame beams
  - Reinforcement placed horizontally at top and bottom
- **Shear Design**
  - Shear design is based on probable moment strength (use 1.25 $f_y$)
  - Within 2h of member ends, shear strength is calculated based on steel only
- **Confinement**
  - Hoops confine at end regions
  - If ln/h is relatively small, longitudinal cannot be lapped
  - Easier to use closed hoops over entire beam rather than only 2h at each end

# Conventional Reinforced Beam

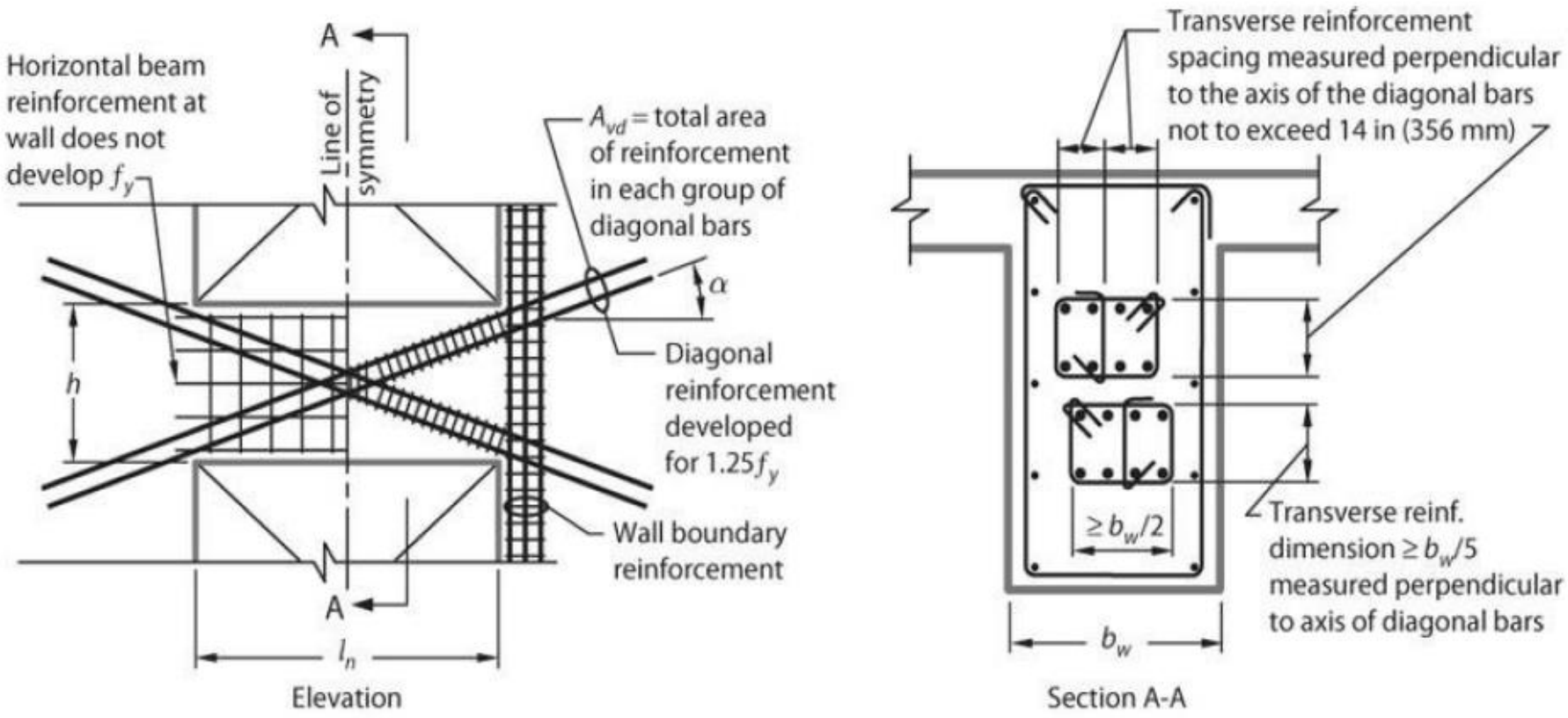



Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Diagonal Reinforced Beam

- **Shear Design**
  - $V_n = 2A_{vd}f_y \sin\alpha \leq 0.83\sqrt{f'_c}\, A_{cw}$ (ACI 318-14, Eq. 18.10.7.4)
  - Minimum four bars in each group of diagonal
- **Confinement (2 Options)**
  - Confine individual diagonals
  - Confine entire beam cross section

# Confine Individual Diagonals

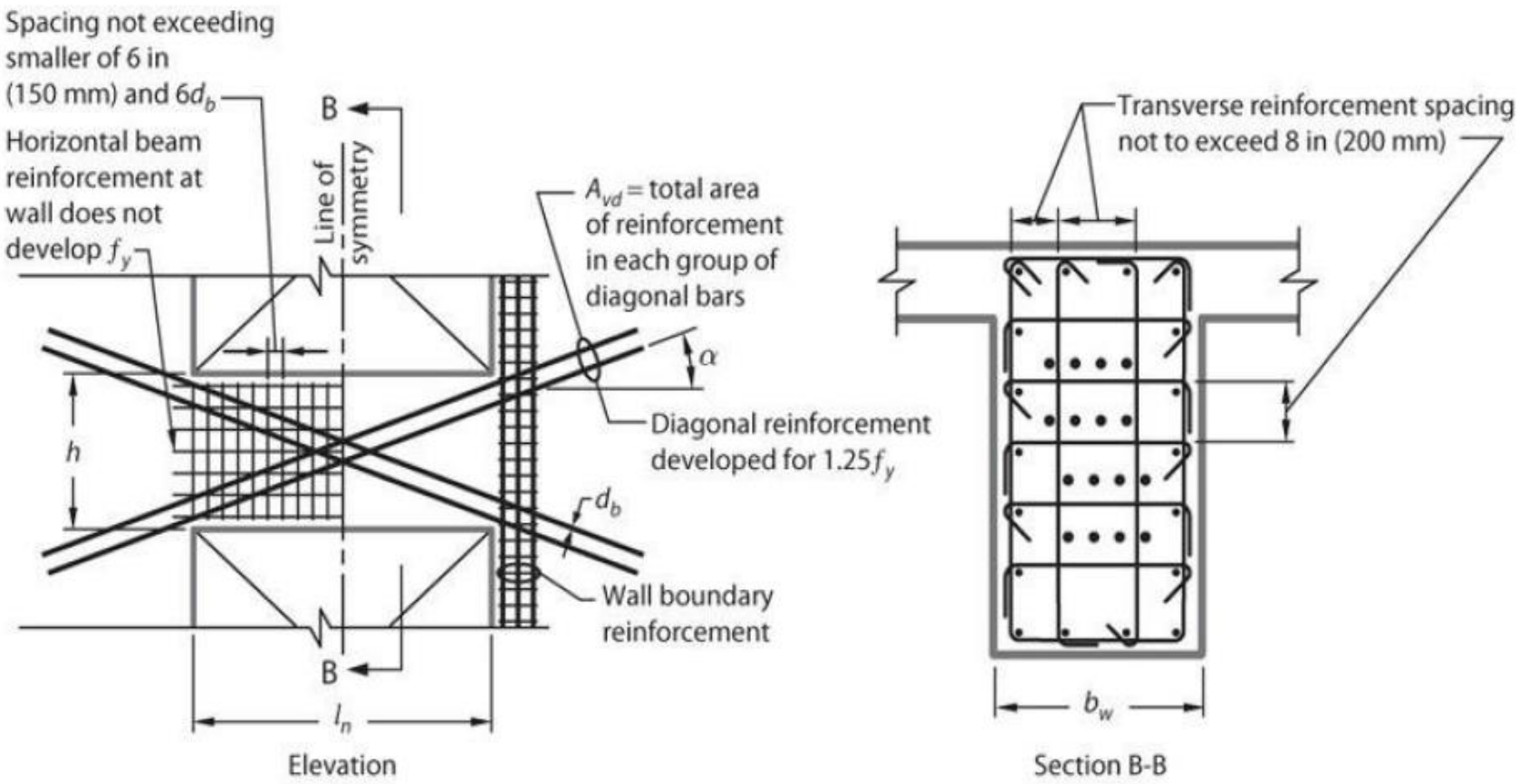



**(a) Confinement of individual diagonals**

**Notes:** - Consecutive crossties have their 90° hooks alternating end for end both along and around the beam.
- For clarity, only part of the required reinforcement is shown on each side of the line of symmetry.

Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Confine Entire Beam Cross-section




**(b) Confinement of entire beam section**

Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

# Performance-based Evaluation of Coupling Beams

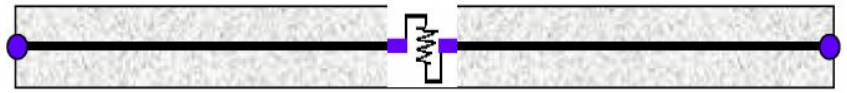
- **Conventional reinforced beam**
  - Flexural rotations (ASCE 41-13, Table 10-19)
  - Shear capacity to satisfy probable shear demand based on moment capacity
- **Diagonal reinforced beam**
  - Shear rotations (ASCE 41-13, Table 10-19)

# Nonlinear Modeling

- **Conventional reinforced beam**
  - Moment hinges at the ends of the beam
  - Shear response is modeled as linear

- **Diagonal reinforced beam**
  - Shear hinges at the mid span of the beam (Diagonal reinforced beam)
  - Considering that there is no shear deformation along the length of the beam

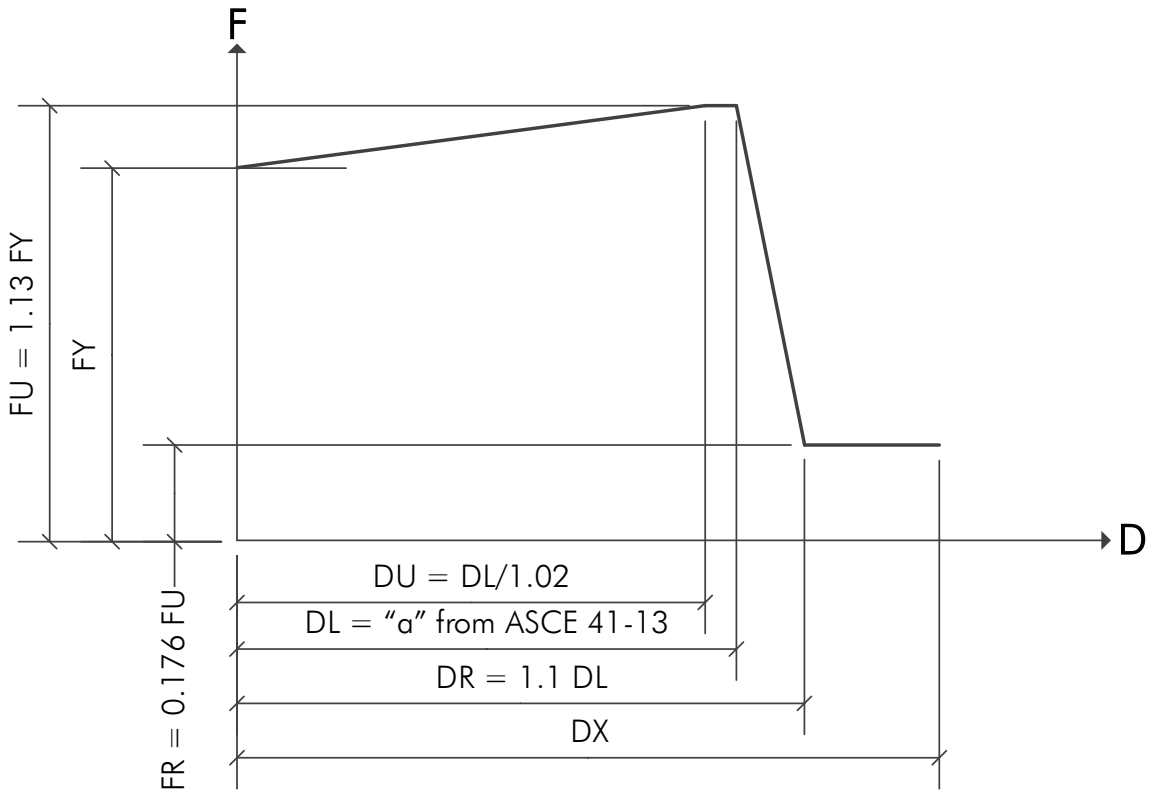
Moment hinge (rotational springs) at the ends



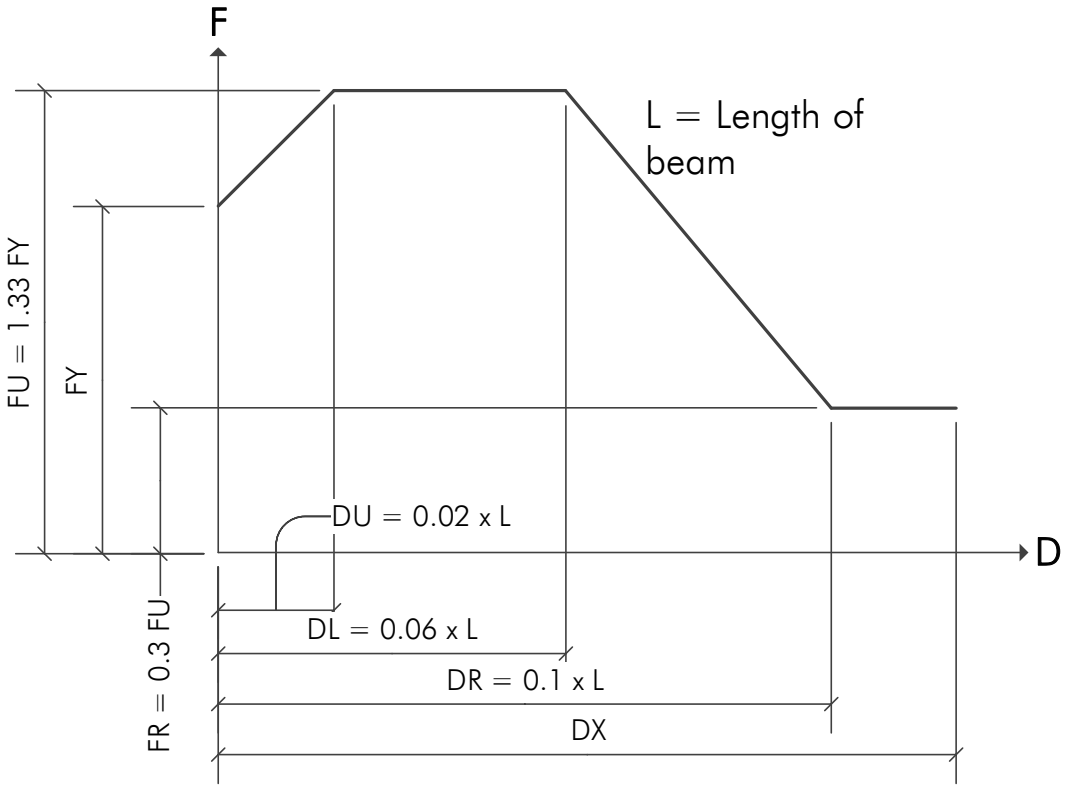
Shear-displacement hinge at the mid span

Source: PEER/ATC 72-1 Report

# Force-deformation Relationship




Conventional reinforced beam

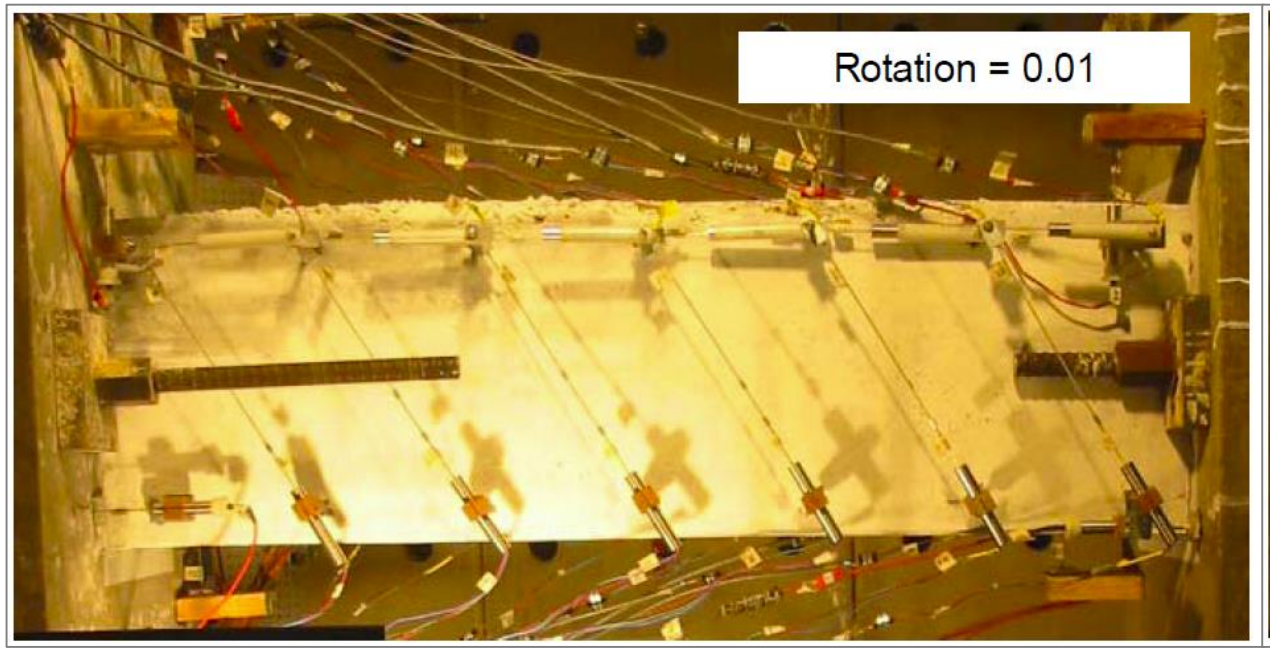


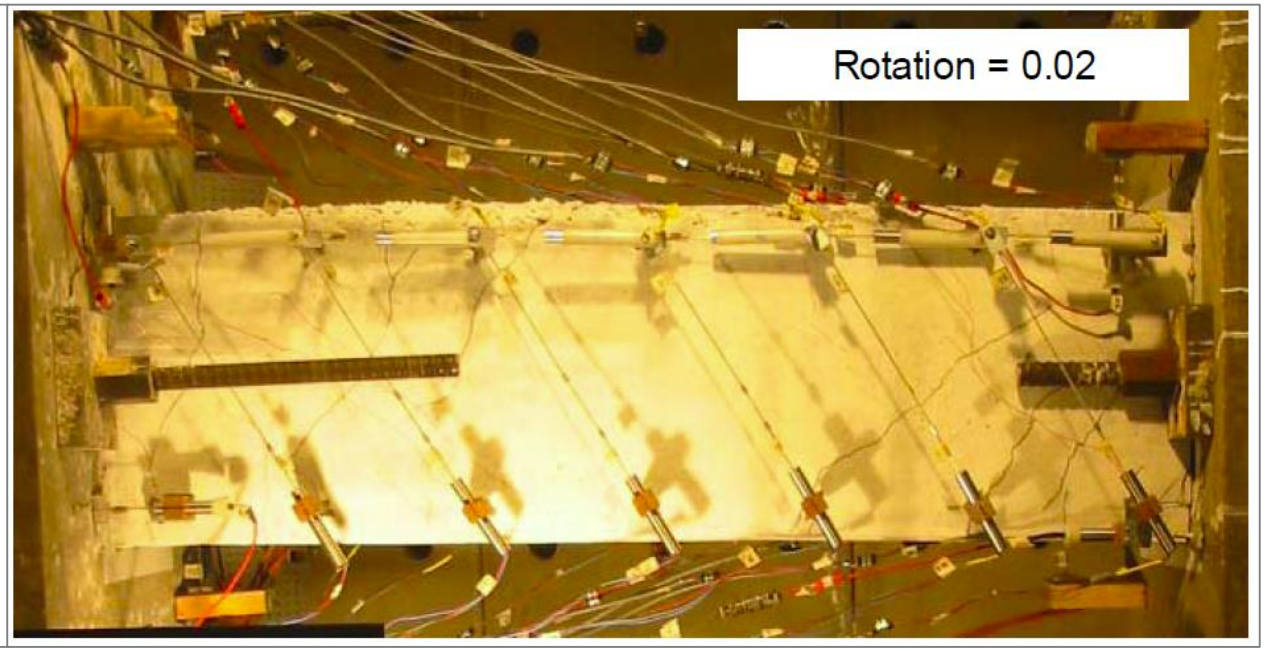

Diagonal reinforced beam

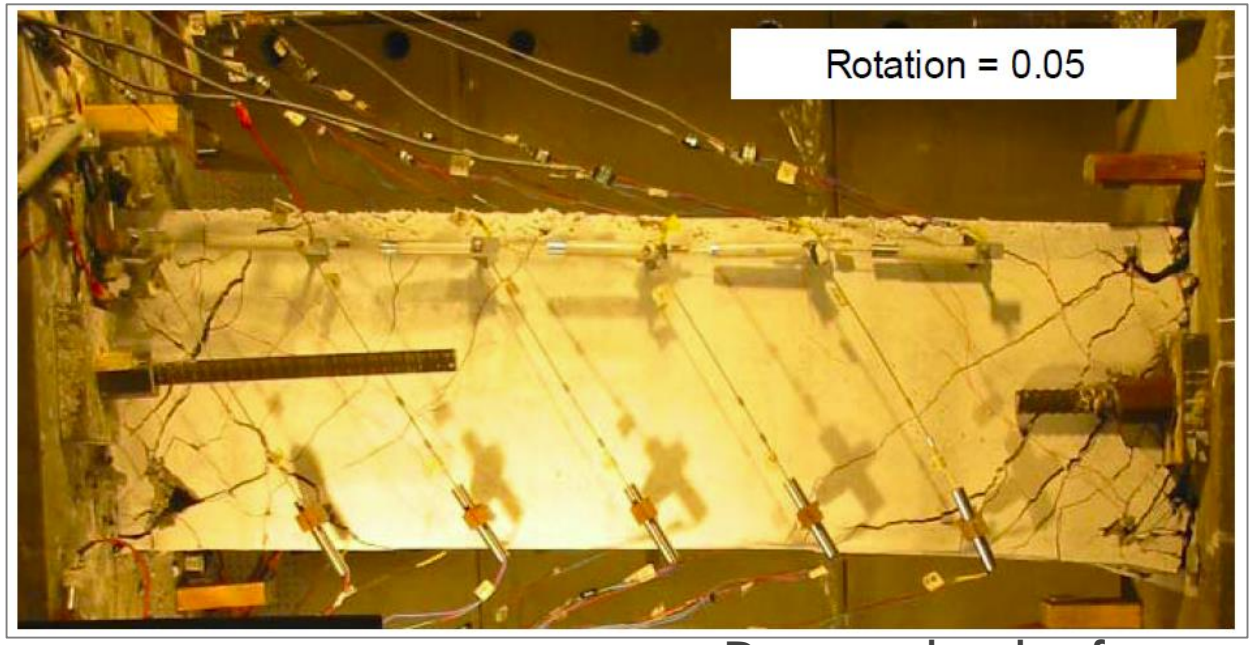
**Table 10-19. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—R/C Shear Walls and Associated Components Controlled by Flexure**

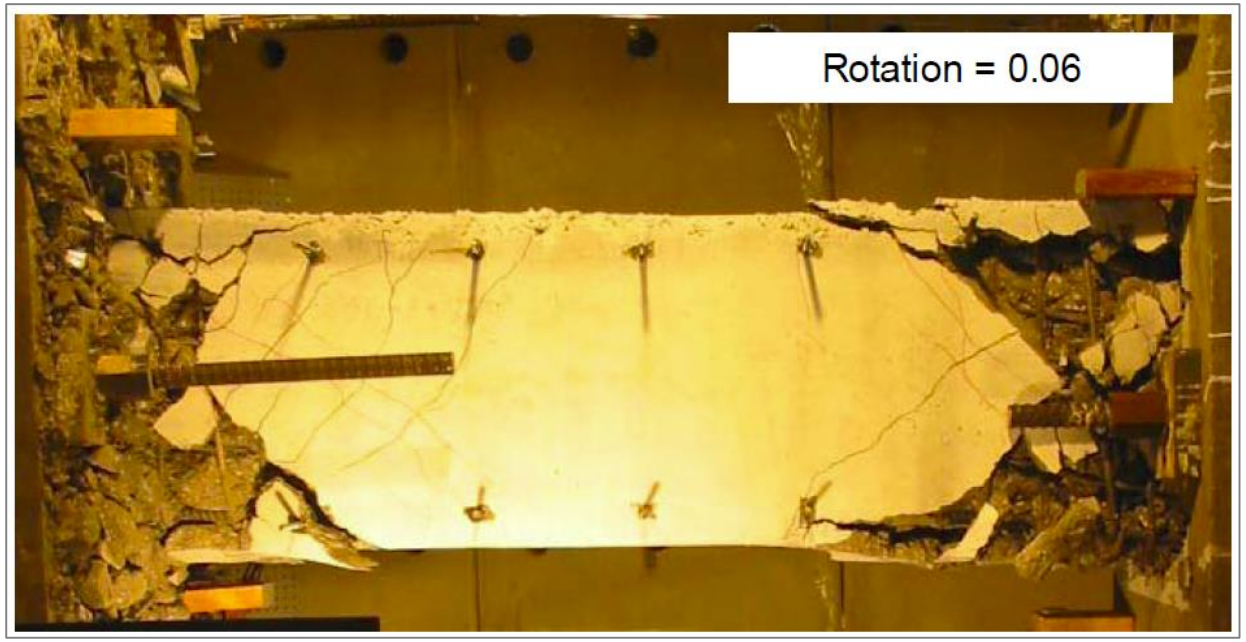
| Conditions | | | Plastic Hinge Rotation (radians) | | Residual Strength Ratio | Acceptable Plastic Hinge Rotation[a] (radians) Performance Level | | |
|---|---|---|---|---|---|---|---|---|
| | | | *a* | *b* | *c* | IO | LS | CP |
| i. Shear walls and wall segments | | | | | | | | |
| $\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'}$ | $\frac{V}{t_w l_w \sqrt{f_c'}}$ | Confined Boundary[b] | 0.015 | | | | | |
| ≤0.1 | ≤4 | Yes | 0.010 | 0.020 | 0.75 | 0.005 | 0.015 | 0.020 |
| ≤0.1 | ≥6 | Yes | 0.009 | 0.015 | 0.40 | 0.004 | 0.010 | 0.015 |
| ≥0.25 | ≤4 | Yes | 0.005 | 0.012 | 0.60 | 0.003 | 0.009 | 0.012 |
| ≥0.25 | ≥6 | Yes | 0.008 | 0.010 | 0.30 | 0.0015 | 0.005 | 0.010 |
| ≤0.1 | ≤4 | No | 0.006 | 0.015 | 0.60 | 0.002 | 0.008 | 0.015 |
| ≤0.1 | ≥6 | No | 0.003 | 0.010 | 0.30 | 0.002 | 0.006 | 0.010 |
| ≥0.25 | ≤4 | No | 0.002 | 0.005 | 0.25 | 0.001 | 0.003 | 0.005 |
| ≥0.25 | ≥6 | No | 0.002 | 0.004 | 0.20 | 0.001 | 0.002 | 0.004 |
| ii. Shear wall coupling beams[c] | | | | | | | | |
| Longitudinal reinforcement and transverse reinforcement[d] | | $\frac{V}{t_w l_w \sqrt{f_c'}}$ | | 0.050 | | | | |
| Conventional longitudinal reinforcement with conforming transverse reinforcement | | ≤3 | 0.025 | 0.040 | 0.75 | 0.010 | 0.025 | 0.050 |
| | | ≥6 | 0.020 | 0.035 | 0.50 | 0.005 | 0.020 | 0.040 |
| Conventional longitudinal reinforcement with nonconforming transverse reinforcement | | ≤3 | 0.020 | 0.025 | 0.50 | 0.006 | 0.020 | 0.035 |
| | | ≥6 | 0.010 | 0.050 | 0.25 | 0.005 | 0.010 | 0.025 |
| Diagonal reinforcement | | NA | 0.030 | 0.050 | 0.80 | 0.006 | 0.030 | 0.050 |

Source: Table 10-19, ASCE/SEI 41-13



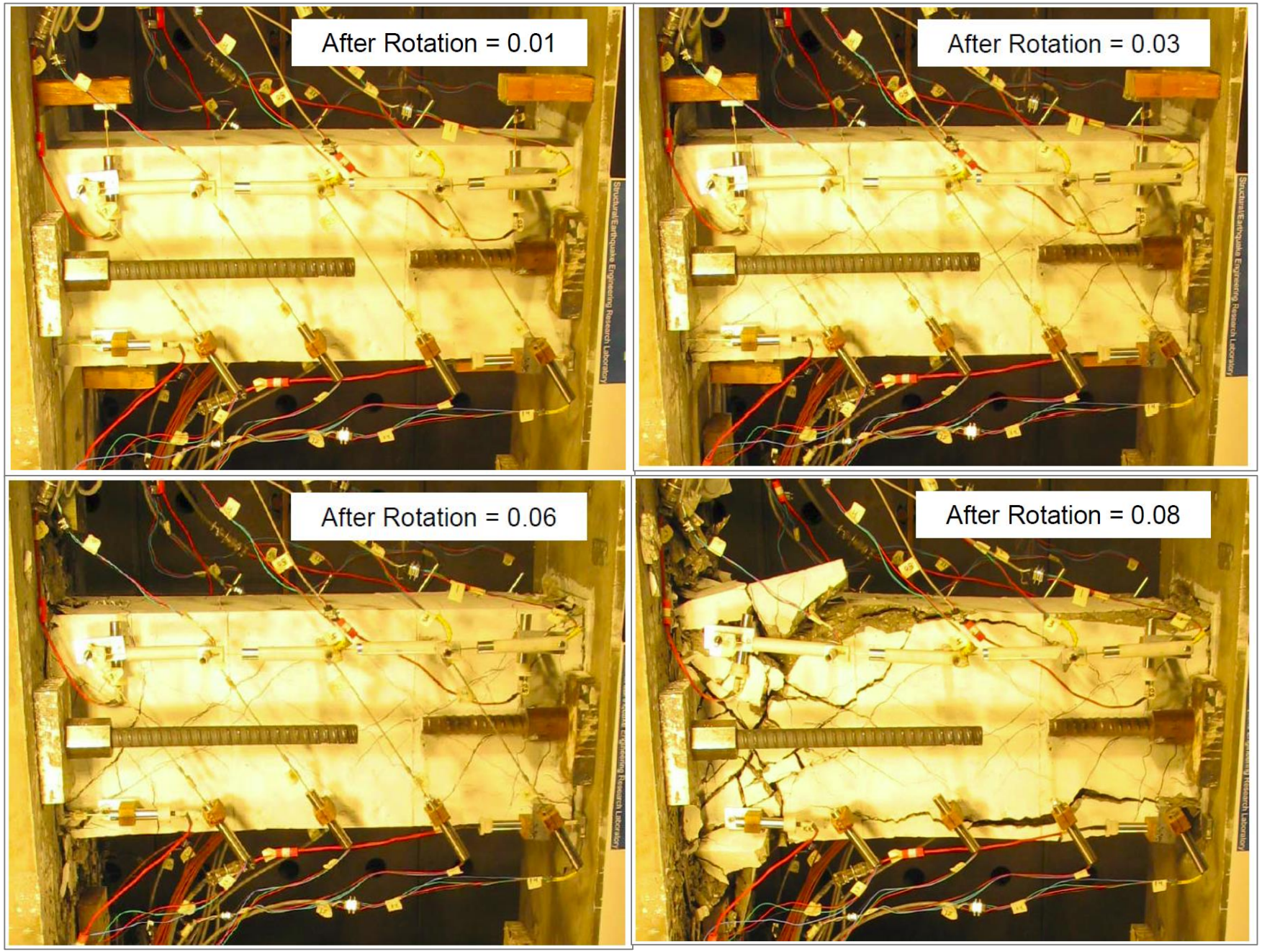



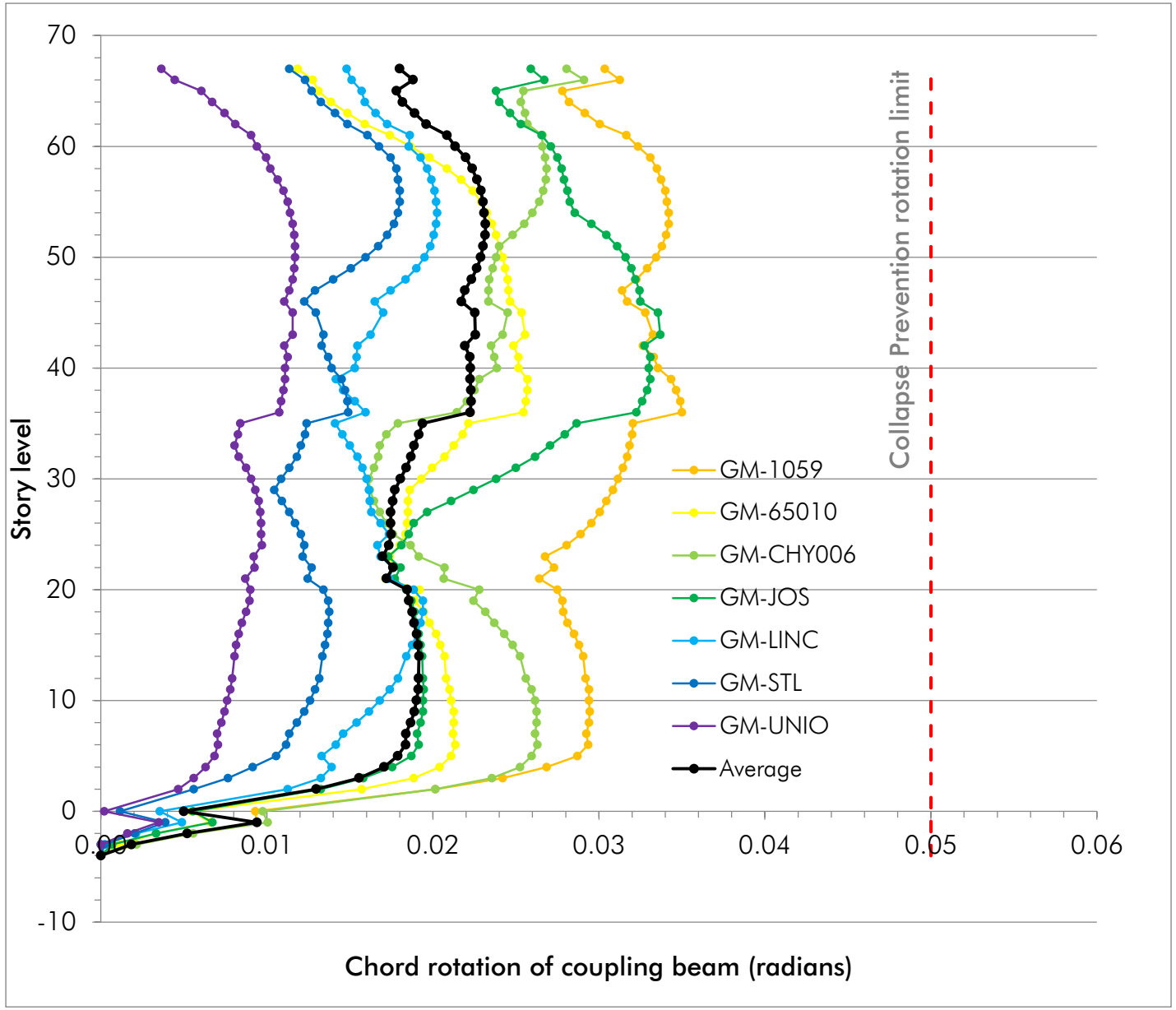

Damage levels of conventional reinforced beam

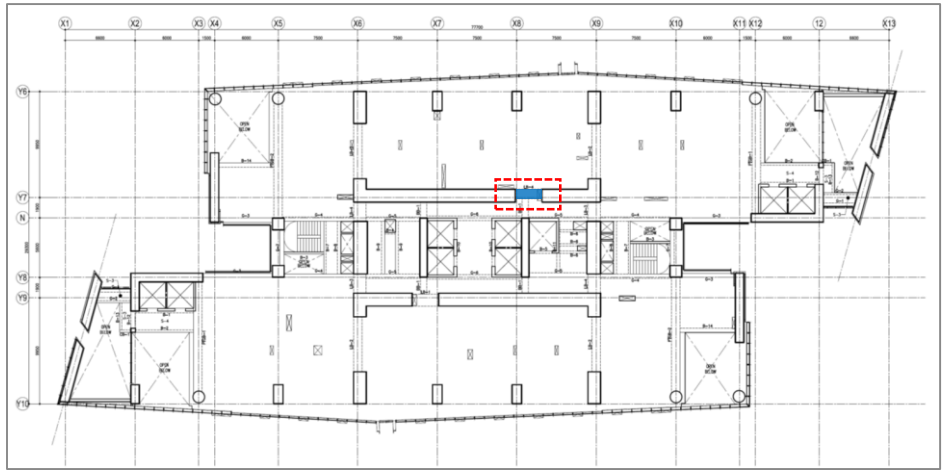
Source: UCLA-SGEL Report 2009/06




Damage levels of diagonal reinforced beam

Source: UCLA-SGEL Report 2009/06



## Coupling beam rotation check

# Post-tensioned Slabs in Seismic Regions

# Post-tensioned Slabs in Seismic Regions

- In high seismic regions, post-tensioned (PT) slab-column frames can be used to support gravity loads in conjunction with a lateral-force resisting system (LFRS; e.g., a core wall).
- The LFRS is designed to resist 100% of the design lateral forces as well as to limit lateral displacements to an acceptable level, whereas the slab-column frame must sustain the gravity loads under the expected (design) displacements.

# Post-tensioned Slabs in Seismic Regions

- According to the seismic provisions in ACI 318, structural systems are either designated to resist earthquake forces (i.e., be part of the LFRS) or they are referred to as "nonparticipating" systems or "gravity" force resisting systems (GFRS).
- The lateral displacements imposed on the slab-column frame are likely to introduce significant unbalanced moments on the slab-column connections, increasing the potential for punching failures.

# Code-based procedures (ACI 318-14, Section 18.14.5)

- Check gravity load punching shear D/C ratio vs. story drift.
- If gravity load shear D/C does not have margin of safety, allowable story drift under earthquake will be small.

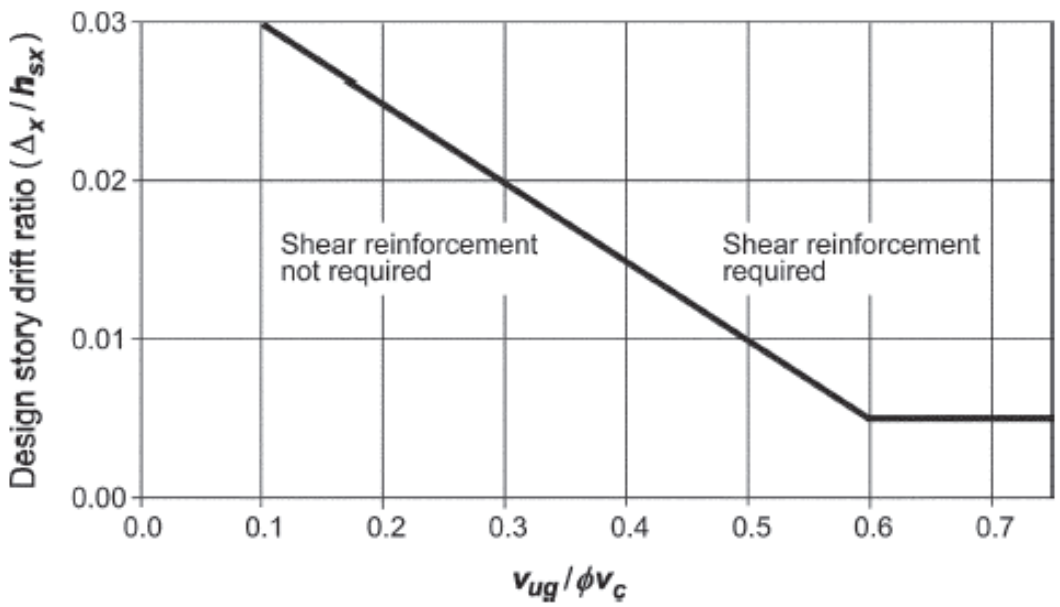
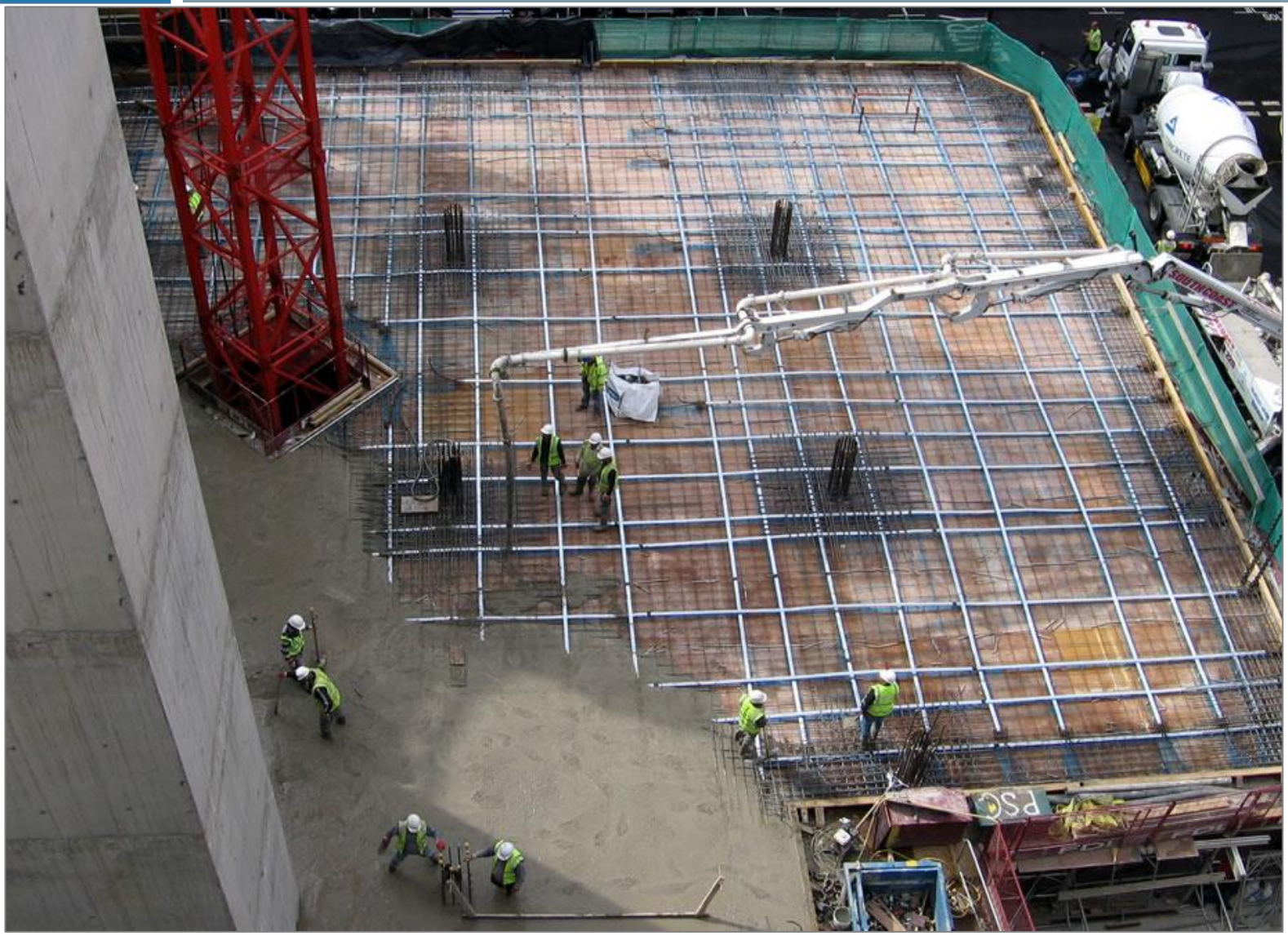



*Fig. R18.14.5.1—Illustration of the criterion of 18.14.5.1.*

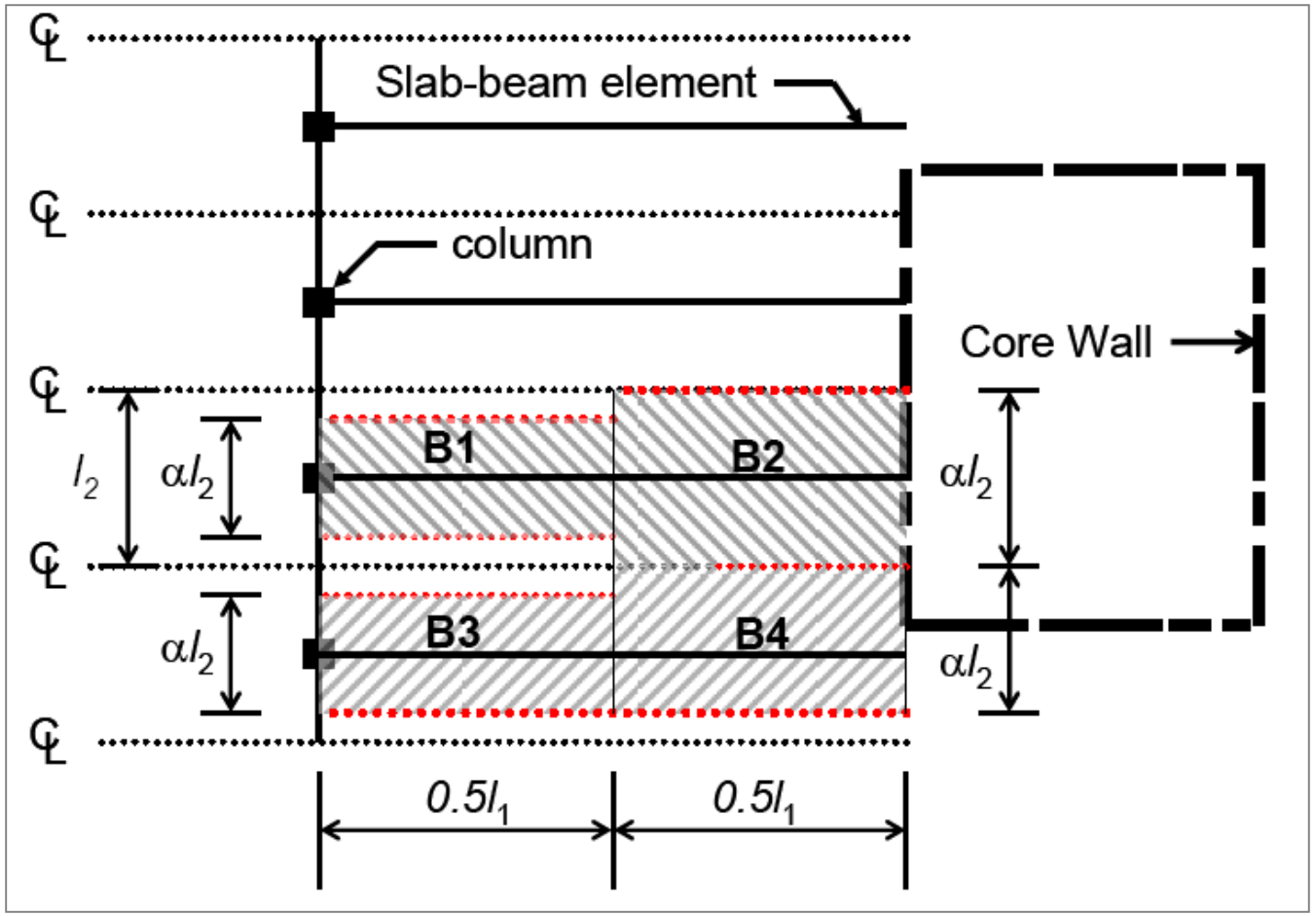
Source: Fig. R18.14.5.1, ACI 318-14

# Performance-based Procedures (ASCE 41-13, Section 10.4.4)



- Inelastic rotation of the slab is checked.
- The allowable rotation limit depends on the gravity load shear D/C ratio of the slab.

# Nonlinear Modeling

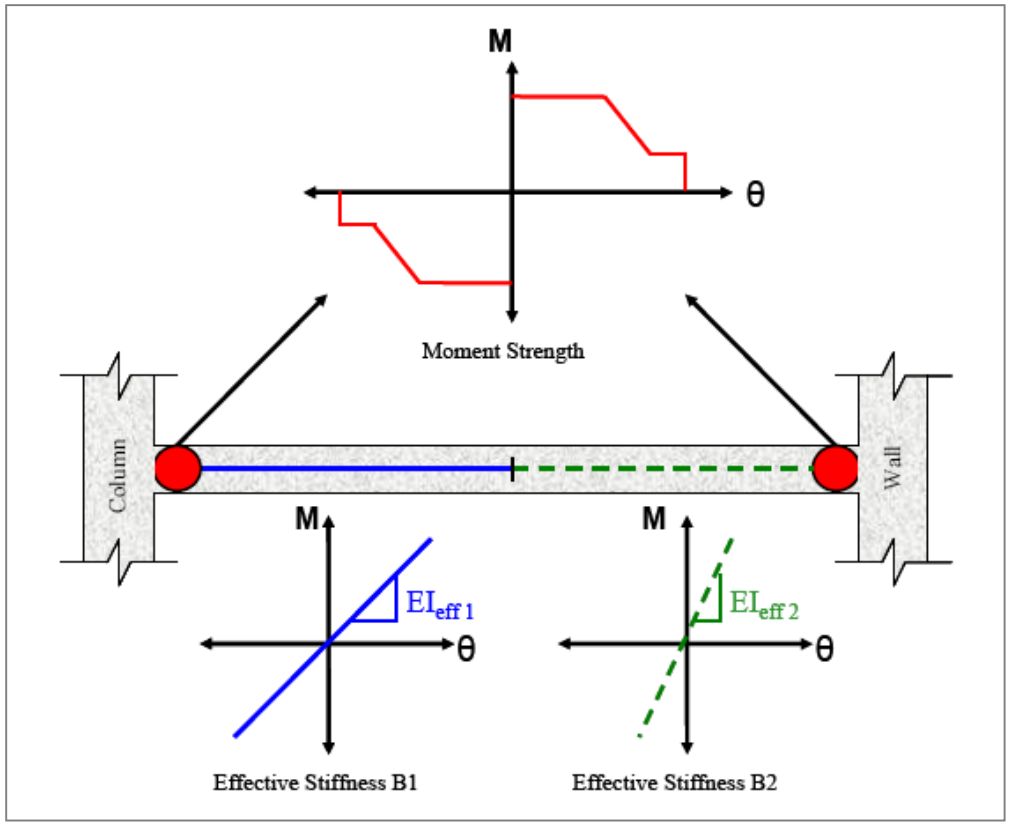



Source: PEER/ATC 72-1 Report

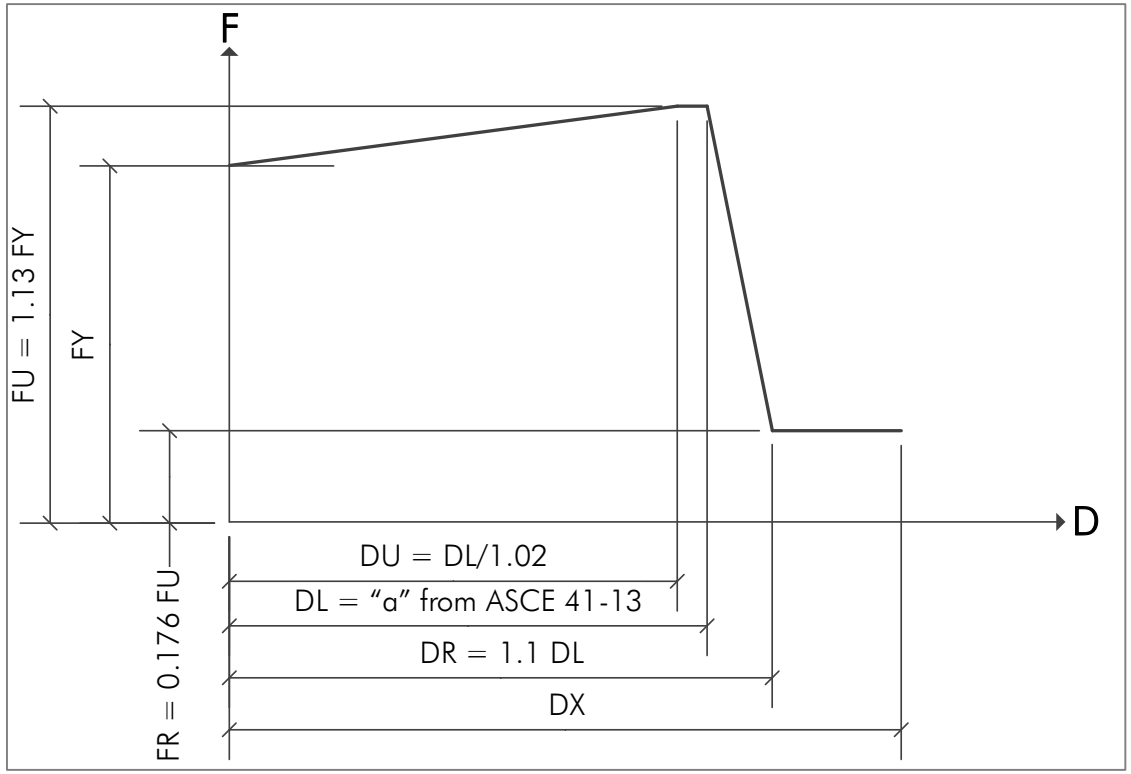
- **Elastic slab effective width**
  - $\alpha l_2 = 2c_1 + l_1/3$, for interior frames
  - $\alpha l_2 = c_1 + l_1/6$, for exterior frames
  - $\alpha = 1/2 \sim 3/4$ for interior frames (Non-prestressed construction)
  - $\alpha = 1/2 \sim 2/3$ for interior frames (PT construction)
- **Stiffness reduction**
  - $\beta = 4c_1 / l_1$ (Non-prestressed construction)
  - $\beta = 1/2$ (PT construction)

$c_1$ is column dimension parallel to slab.

# Nonlinear Modeling




Source: PEER/ATC 72-1 Report




Force-deformation relationship

**Table 10-15. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Two-Way Slabs and Slab–Column Connections**

| | | Modeling Parameters[a] | | | Acceptance Criteria[a] Plastic Rotation Angle (radians) | | |
|---|---|---|---|---|---|---|---|
| | | Plastic Rotation Angle (radians) | | Residual Strength Ratio | Performance Level | | |
| | | | | | | Secondary | |
| Conditions | | a | b | c | IO | LS | CP |
| Condition i. Reinforced concrete slab–column connections[b] | | | | | | | |
| $\frac{V_g}{V_o}$[c] | Continuity reinforcement[d] | | | | | | |
| 0 | Yes | 0.035 | 0.05 | 0.2 | 0.01 | 0.035 | 0.05 |
| 0.2 | Yes | 0.03 | 0.04 | 0.2 | 0.01 | 0.03 | 0.04 |
| 0.4 | Yes | 0.02 | 0.03 | 0.2 | 0 | 0.02 | 0.03 |
| ≥0.6 | Yes | 0 | 0.02 | 0 | 0 | 0 | 0.02 |
| 0 | No | 0.025 | 0.025 | 0 | 0.01 | 0.02 | 0.025 |
| 0.2 | No | 0.02 | 0.02 | 0 | 0.01 | 0.015 | 0.02 |
| 0.4 | No | 0.01 | 0.01 | 0 | 0 | 0.008 | 0.01 |
| 0.6 | No | 0 | 0 | 0 | 0 | 0 | 0 |
| >0.6 | No | 0 | 0 | 0 | —[e] | —[e] | —[e] |
| Condition ii. Posttensioned slab–column connections[b] | | | | | | | |
| $\frac{V_g}{V_o}$[c] | Continuity reinforcement[d] | | | | | | |
| 0 | Yes | 0.035 | 0.05 | 0.4 | 0.01 | 0.035 | 0.05 |
| 0.6 | Yes | 0.005 | 0.03 | 0.2 | 0 | 0.025 | 0.03 |
| >0.6 | Yes | 0 | 0.02 | 0.2 | 0 | 0.015 | 0.02 |
| 0 | No | 0.025 | 0.025 | 0 | 0.01 | 0.02 | 0.025 |
| 0.6 | No | 0 | 0 | 0 | 0 | 0 | 0 |
| >0.6 | No | 0 | 0 | 0 | —[e] | —[e] | —[e] |
| Condition iii. Slabs controlled by inadequate development or splicing along the span[b] | | | | | | | |
| | | 0 | 0.02 | 0 | 0 | 0.01 | 0.02 |
| Condition iv. Slabs controlled by inadequate embedment into slab–column joint[b] | | | | | | | |
| | | 0.015 | 0.03 | 0.2 | 0.01 | 0.02 | 0.03 |

Source: Table 10-15, ASCE/SEI 41-13

# Continuity Reinforcement

- **RC slab**
  - Area of effectively continuous main bottom bars passing through the column cage in each direction is greater than or equal to **0.5 $V_g$ /(Φ$f_y$)**
- **PT slab**
  - At least one of the posttensioning tendons in each direction passes through the column cage

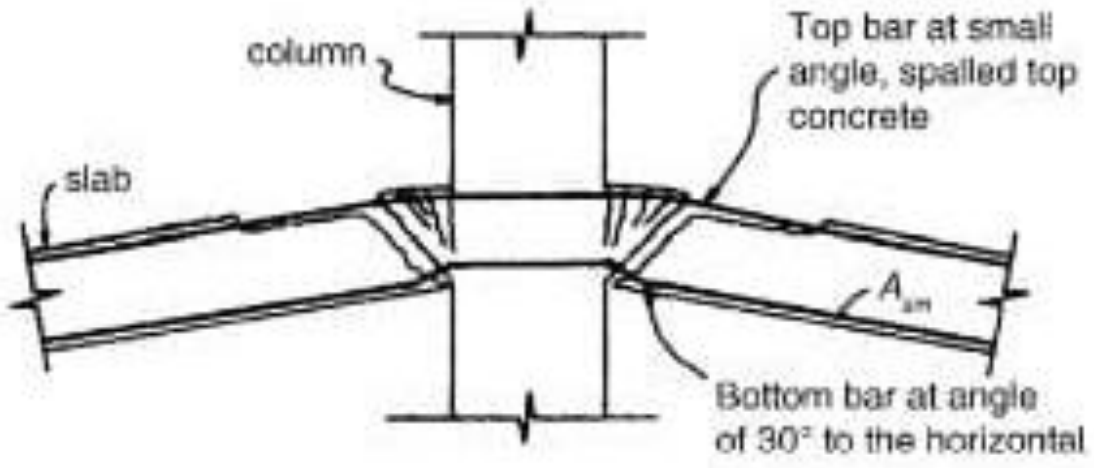
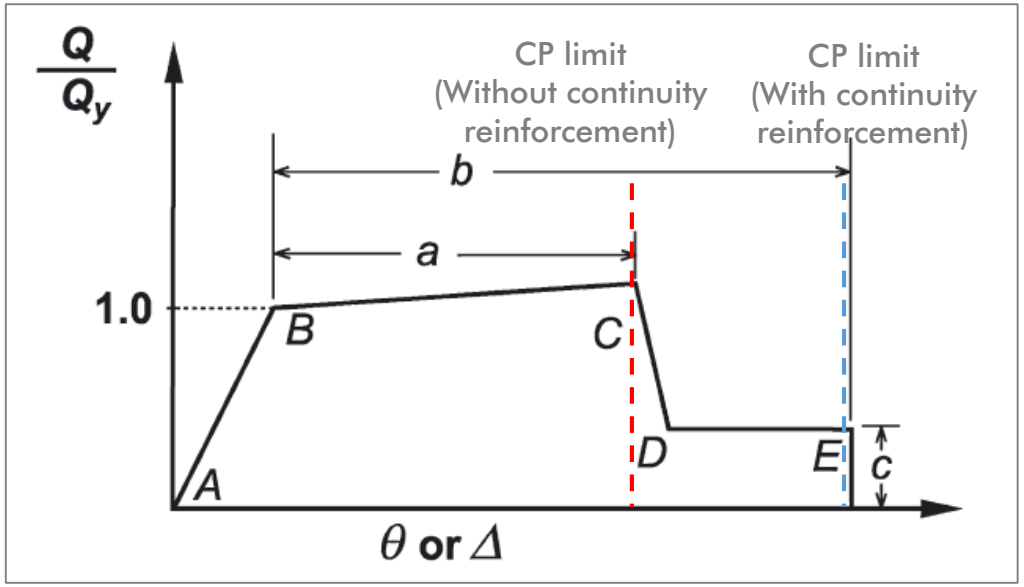
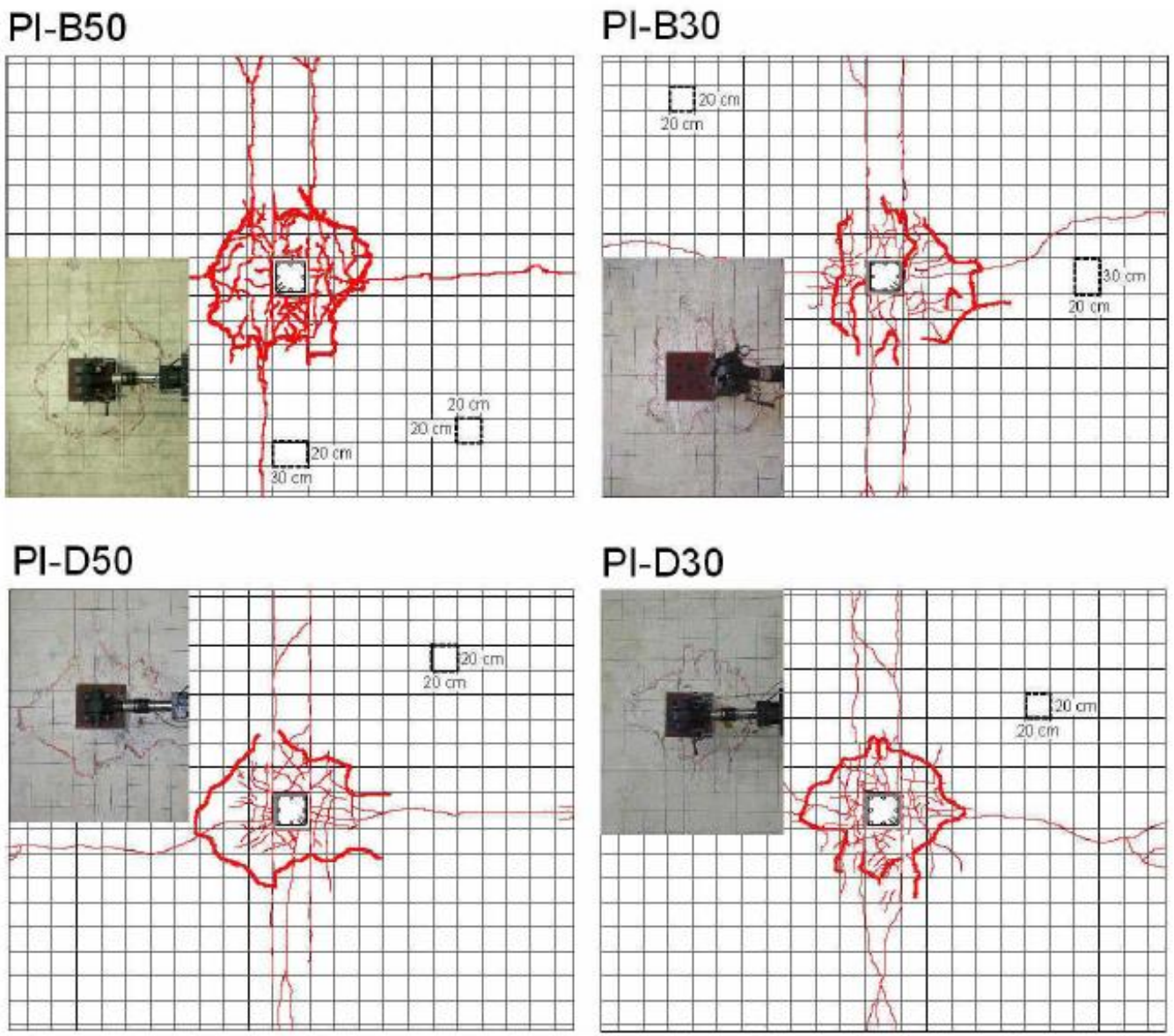



*Fig. 6.3.1—Model of connection during punching failure.*

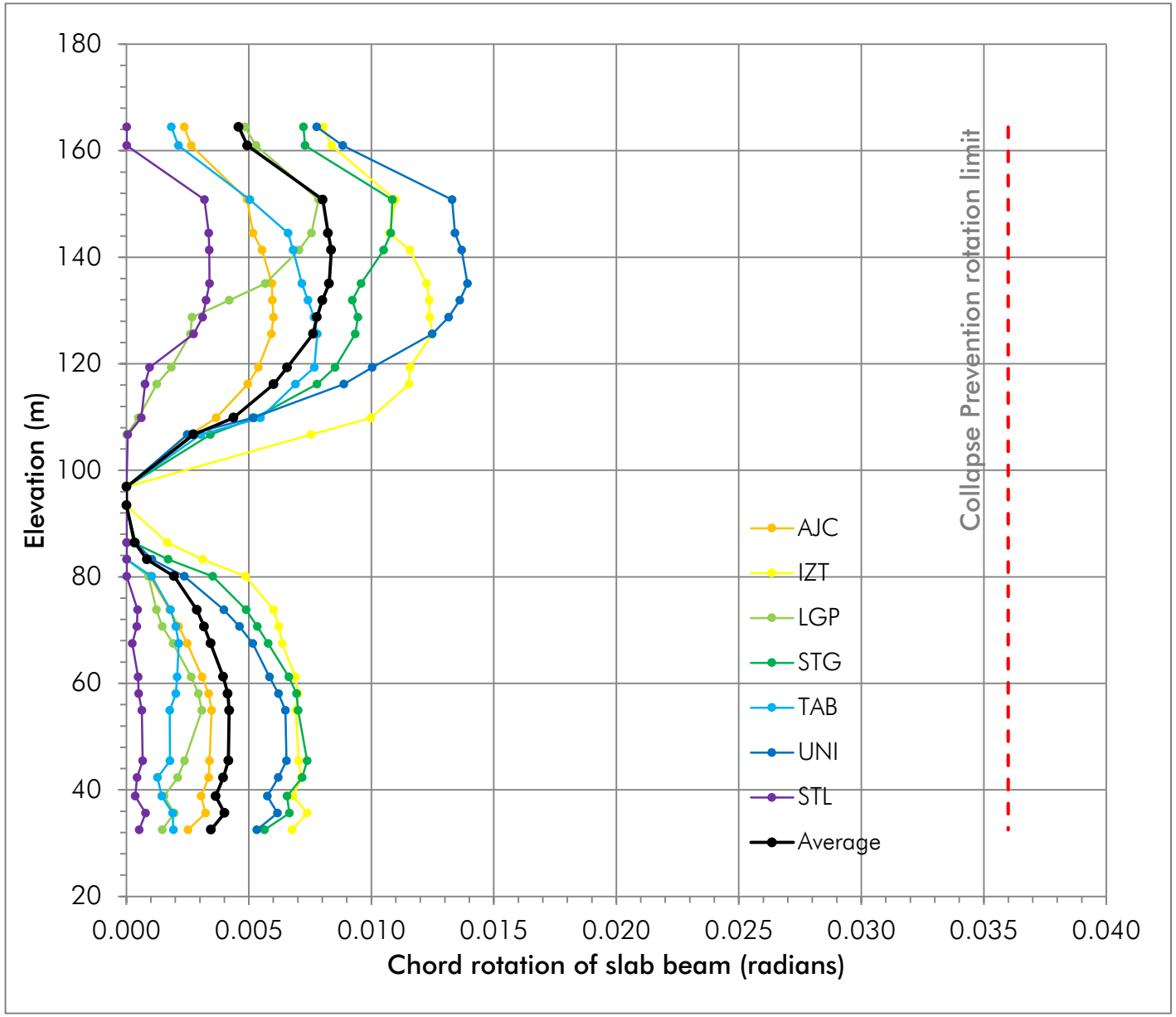
Source: ACI 352.1R-11

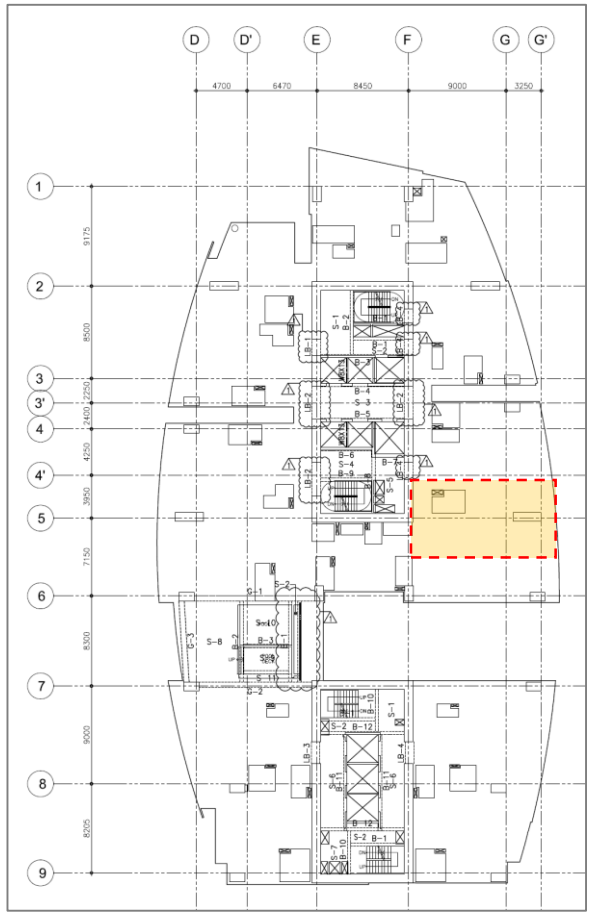





Damage pattern in PT slab at story drift > 3%

Source: Interior "Post-tensioned Slab-column Connections Subjected to Cyclic Lateral Loading", by Thomas H.-K. Kang, Seong-Hoon Kee, Sang Whan Han, Li-Hyung Lee, and John W. Wallace



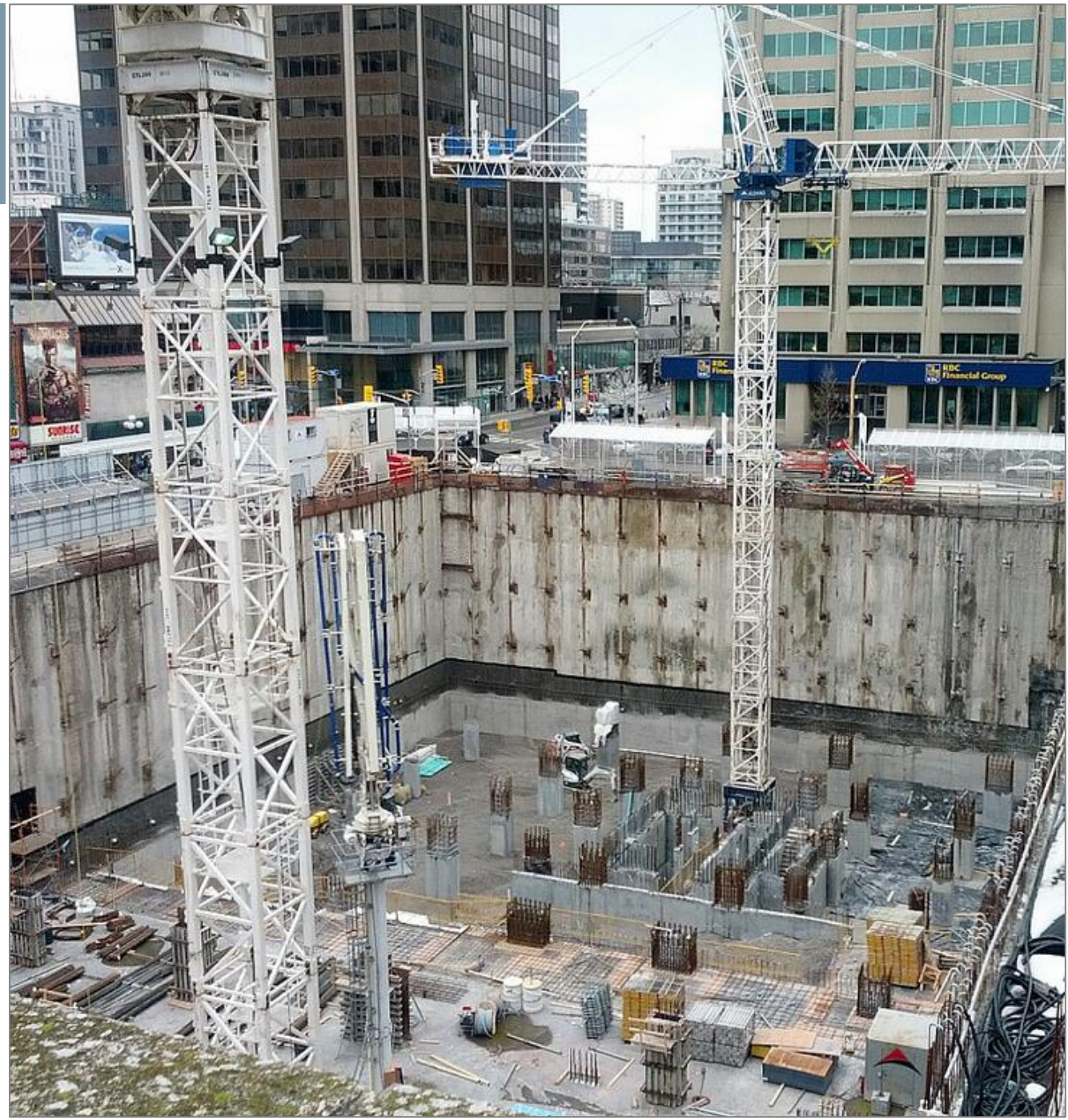

# Slab-beam rotation check


# Basement Walls

# Basement Walls

- **Out-of-plane flexure and shear** (Lateral pressure from soil)
  - Inertia component
  - Kinematic component

- **In-plane shear** (force transferred from ground and basement level diaphragms)




AIT Solutions
Technology · Engineering · Environment · Development · Management

# Seismic Earth Pressure

- **Inertia component**
  - Passive earth pressure due to movement of basement
  - Can be determined from lateral displacement of soil springs x soil spring stiffness

- **Kinematic component**

  $\Delta p = 0.4\ k_h\ \gamma_h\ H_{rw}$

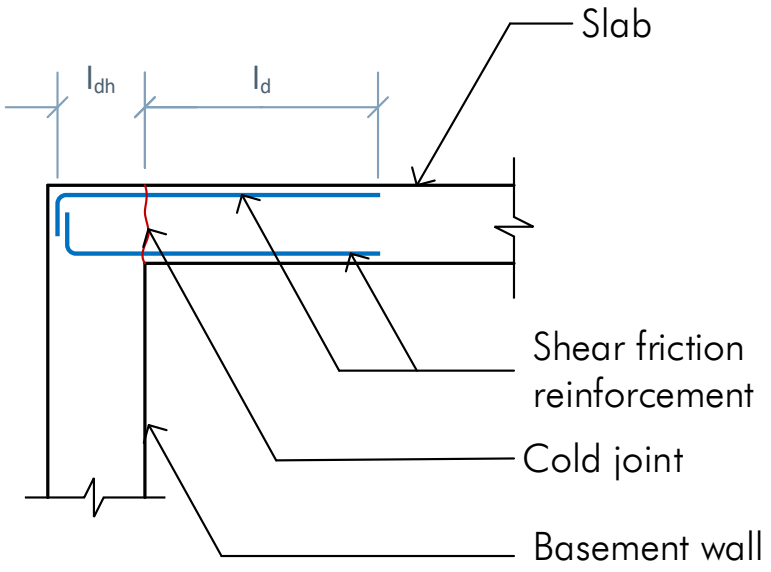
  - $\Delta p$ = additional earth pressure caused by seismic shaking
  - $k_h$ = horizontal seismic coefficient of soil
  - $H_{rw}$ = height of basement wall

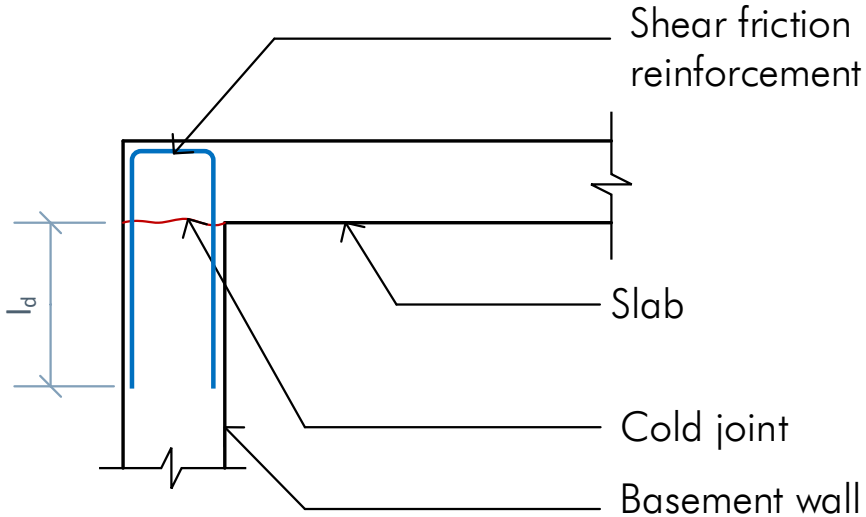
# Shear Friction

- Transfer in-plane shear force of diaphragm to basement wall.

- Shear friction reinforcement

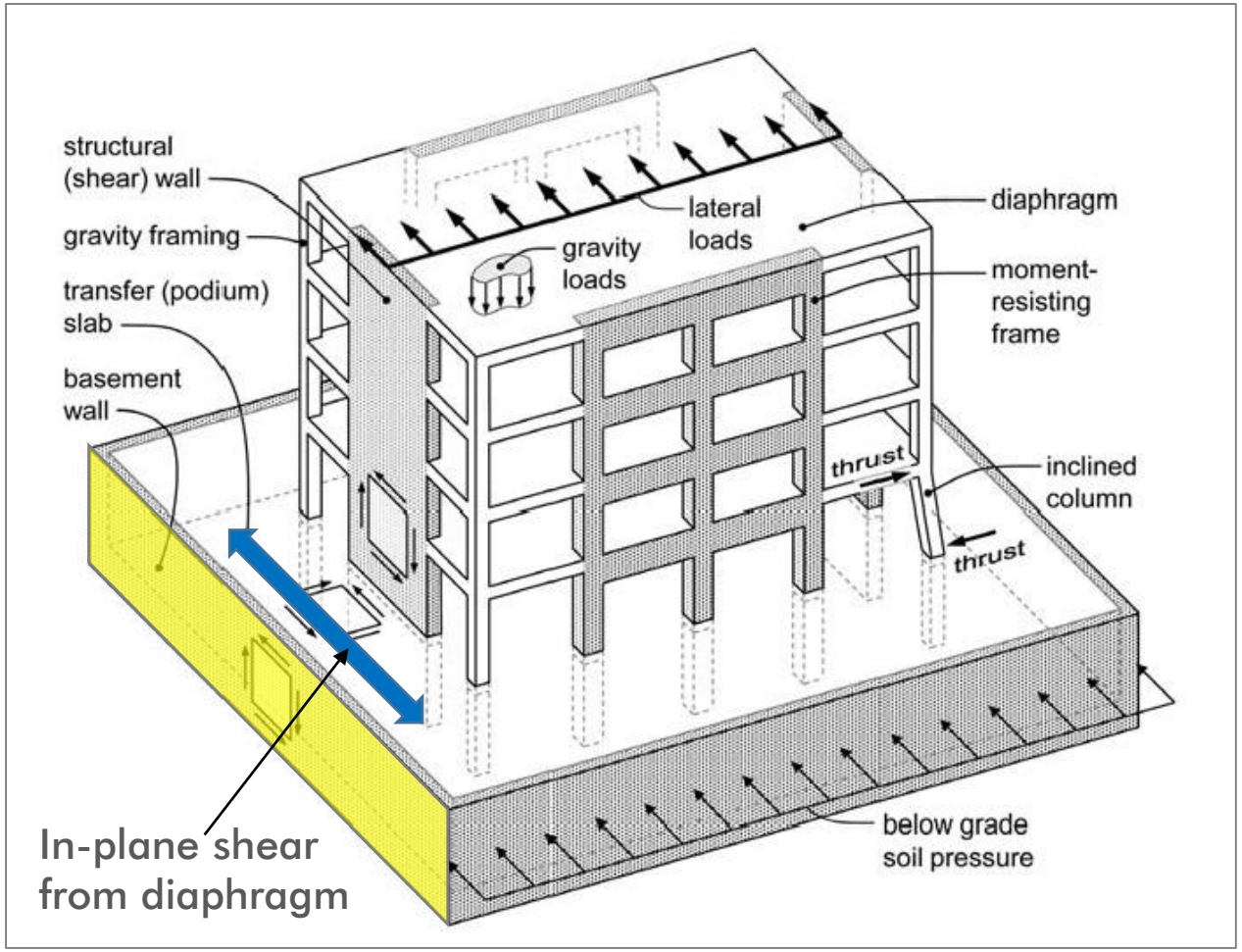
$$V_n = \mu A_{vf} f_y$$



# In-plane Shear

- Shear capacity is determined same as shear wall shear capacity.
- Horizontal reinforcement in basement wall is considered as shear reinforcement.



Technology Engineering Environment Development Management

# Thank You