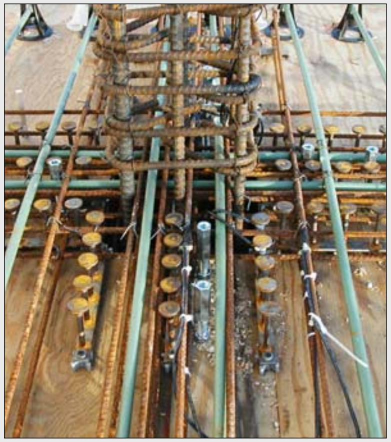
Slab – Column Frames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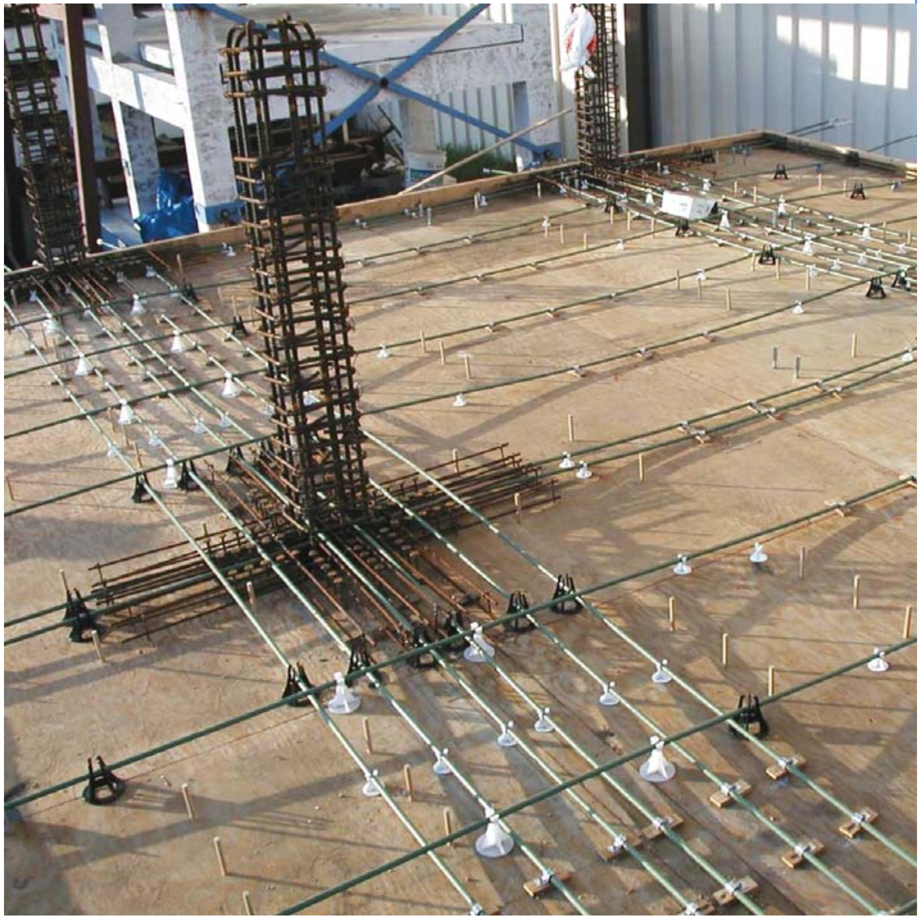
Presentation Overview

- Current Practice
  - Modeling & analysis
  - Connection design
  - Progressive collapse
  - Deformation compatibility
- Existing Construction
  - Post-earthquake observations
  - Modeling and Model Assessment
  - Backbone curves/Rehabilitation
- Shear reinforcement

Current Practice

- Non-participating or "gravity" system
- Post-tensioned slab-column frame
- Span-to-depth ratios typically ~40+
- Use of shear reinforcement at slab-column connection to allow for thinner slabs or to eliminate drop panels



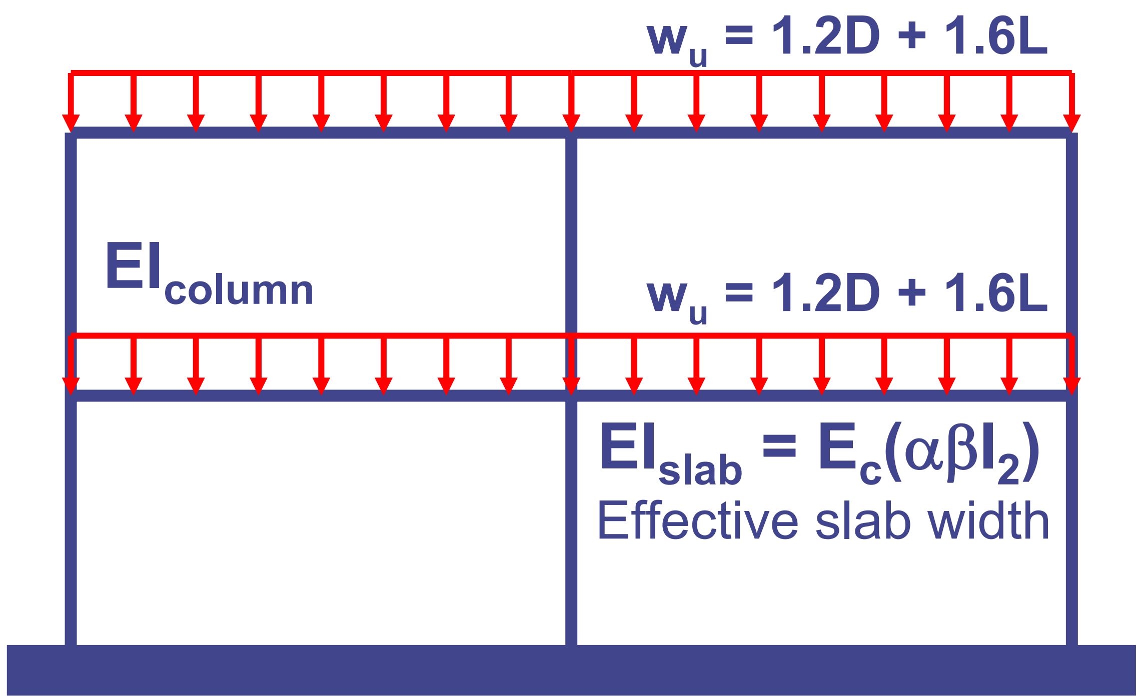
~1/3 scale shake table test specimen

Shear Reinforcement

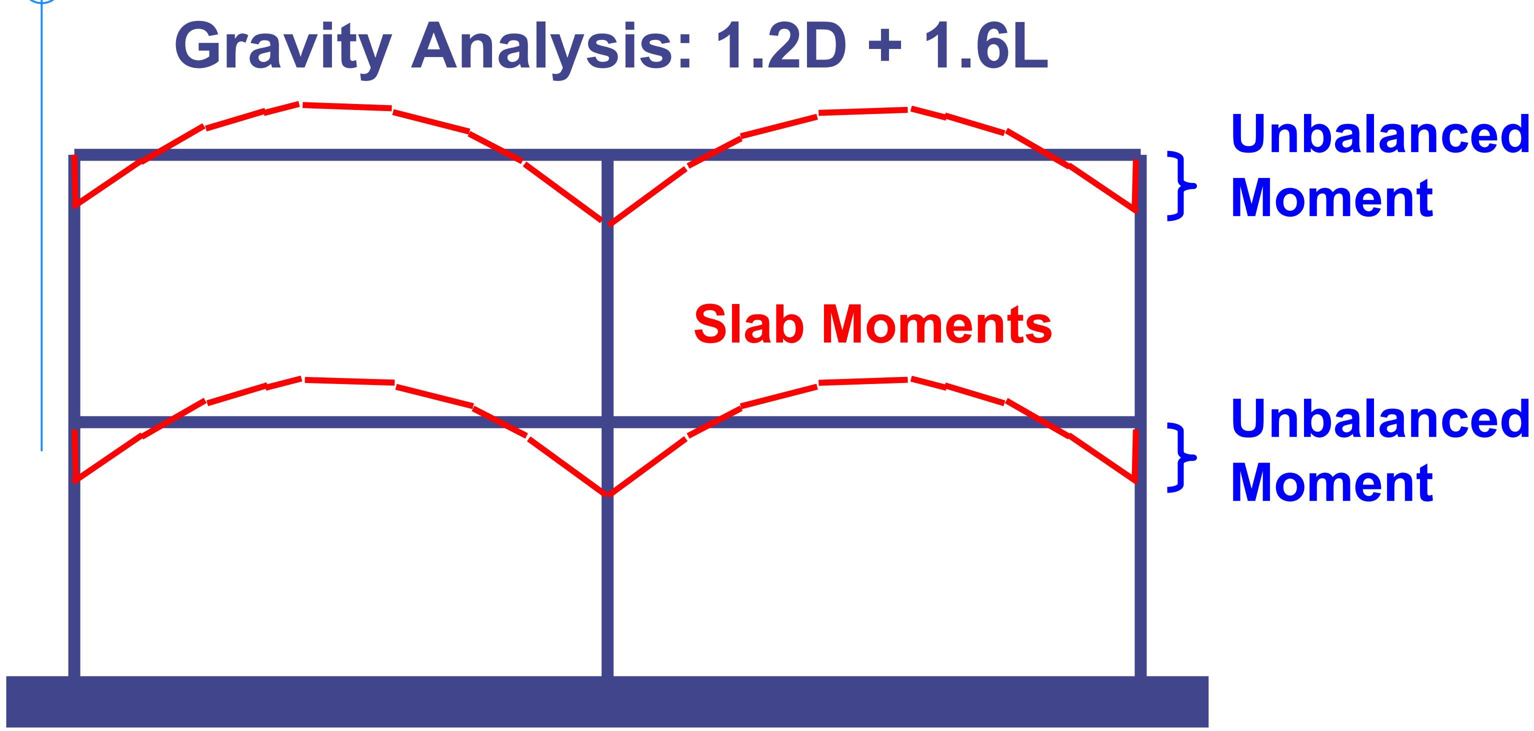
~1/3 scale shake table tests: Kang & Walalce, ACI SJ, Sept-Oct. 2005

Gravity Load Analysis & Design

- ACI 318 Chapter 11, 13, & 21 Materials
  - Slab moments: Use direct design, Equivalent frame, or computer program
  - Connection design – Chapter 11 & 13



Gravity Load Analysis - Moments



- Design slab-column connection to transfer unbalanced moment to column
- FEMA 356 refers to ACI 318 provisions

Unbalanced Moment Transfer

- Unbalanced moment at the slab-column connection is transferred by two mechanisms:
  - Moment transfer (flexure) over a transfer width of c + 3h centered on the column
  - Eccentric shear on a critical section around the slab-column connection
  - Code provisions are covered in Chapter 13 (13.5) and Chapter 11 (11.12) of ACI 318

$$M_f = \gamma_f M_{unbalanced}$$

$$\text{where} \quad \gamma_f = \frac{1}{1 + (2/3)\sqrt{b_1/b_2}}$$

$b_1, b_2 =$ widths of critical section defined in 11.12.1.2

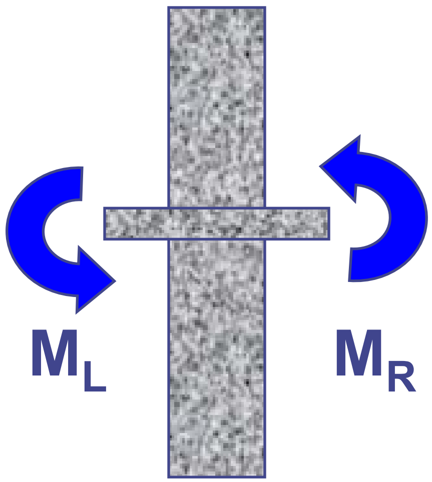
$$M_v = (1 - \gamma_f) M_{unbalanced} = \gamma_v M_{unbalanced}$$

If $b_1 = b_2$, then:

$$\gamma_f = 0.6 \quad \text{and} \quad \gamma_v = 0.4$$

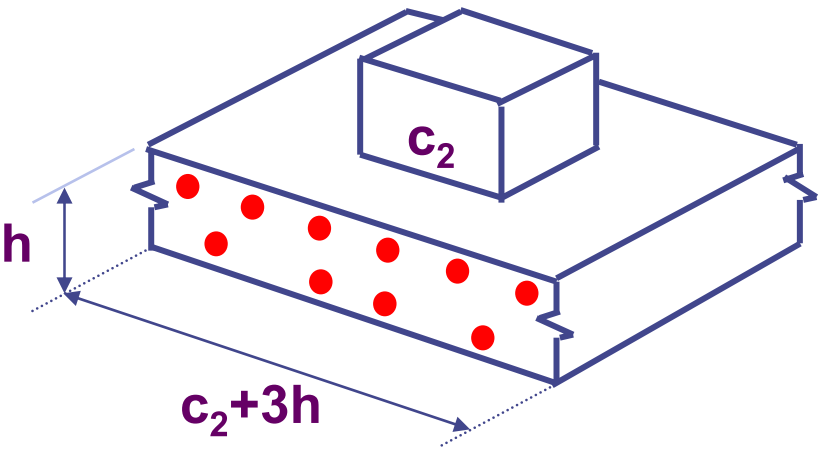
Unbalanced Moment Transfer

Unbalanced Moment (Interior connection)



$M_{unb} = M_L + M_R$

- Flexural Transfer: c_2 + 3h
  - $\gamma_f M_{unb}$ where γ_f is typically ~0.6 for square columns
  - Ratio of top to bottom reinforcement of 2:1 recommended in ACI 318 (R13.5.3.3)



- FEMA 356 6.5.4.3(2) allows use of c_2+5h

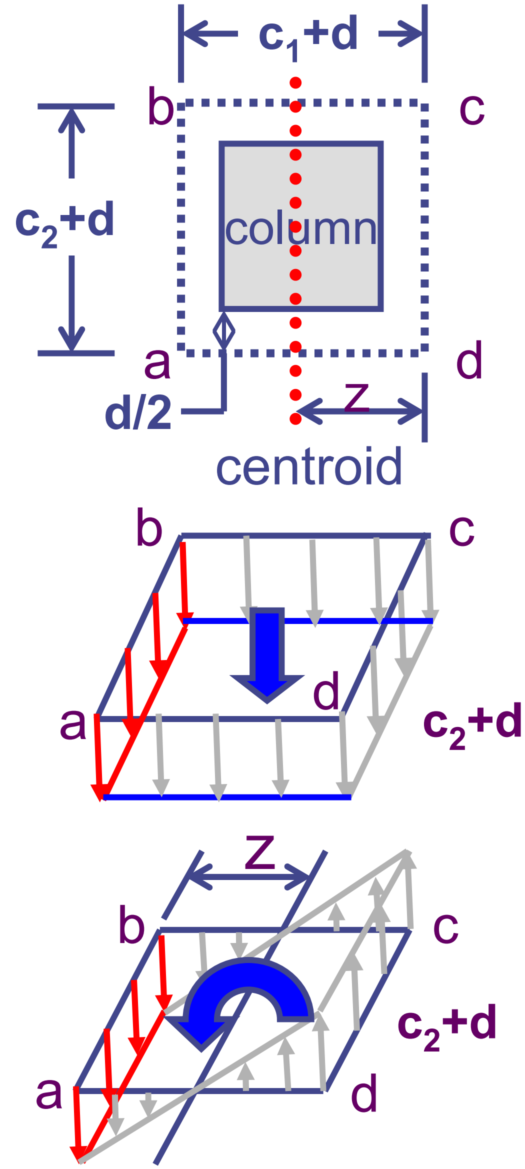
Unbalanced Moment Transfer

- Eccentric Shear transfer
  - Critical section is defined d/2 from column face
  - Direct shear stress
    - b_0 = perimeter of critical section

$$v_{gravity} = \frac{V_{u(direct)}}{b_o d}$$

  - Eccentric shear stress due to $(1-\gamma_f)M_{unb} = \gamma_v M_{unb}$

$$v_{unb} = \gamma_v \frac{M_{unb} z}{J}$$



Unbalanced Moment Transfer

- Combined shear stresses
- Check punching failure per 318

$$(\phi v_n = \phi v_c)\phi v_n \geq v_u \quad \text{where } \phi = 0.75$$

$$v_c = Min\left\{\begin{array}{l}\sqrt{f_c'}\left(2+\dfrac{4}{\beta_c}\right)\\ 2\sqrt{f_c'}\left(2+\dfrac{\alpha_s+d}{b_0}\right)\\ 4\sqrt{f_c'}\end{array}\right\}$$

ACI Eq. 11-33, -34, -35

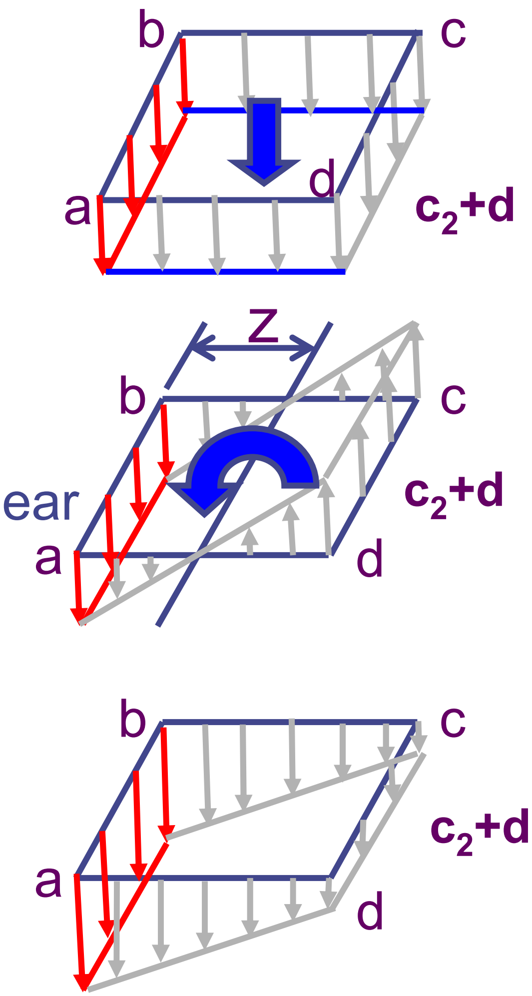
Direct shear stress

+

Eccentric shear stress

=

Total shear stress



$$v_{u,\max} \leq \phi v_n$$

Laboratory Studies

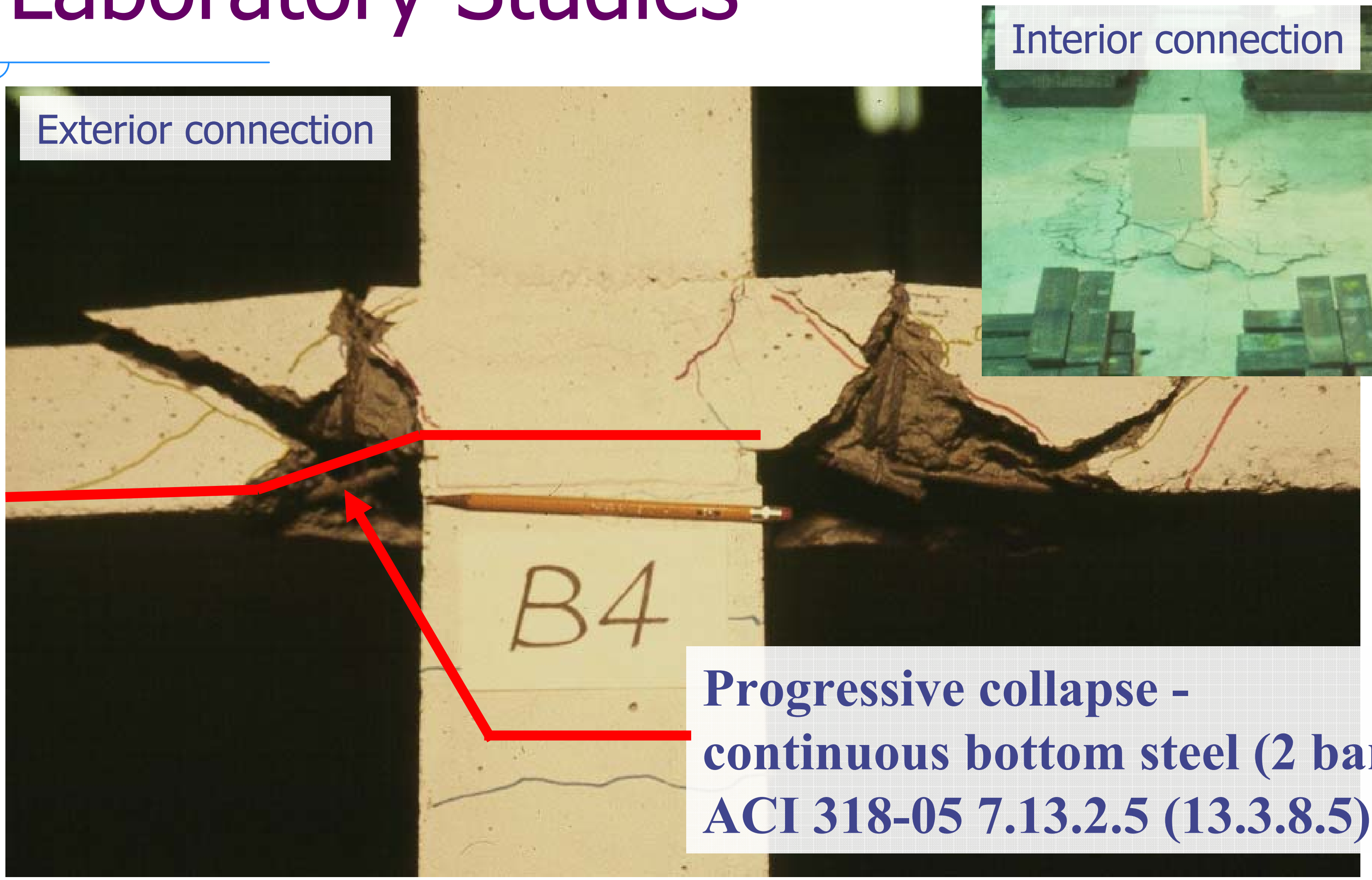


Photo: Hwang and Moehle, ACI SJ, March-April 2000.

ACI Committee 352.1R89

Slab – Column Report

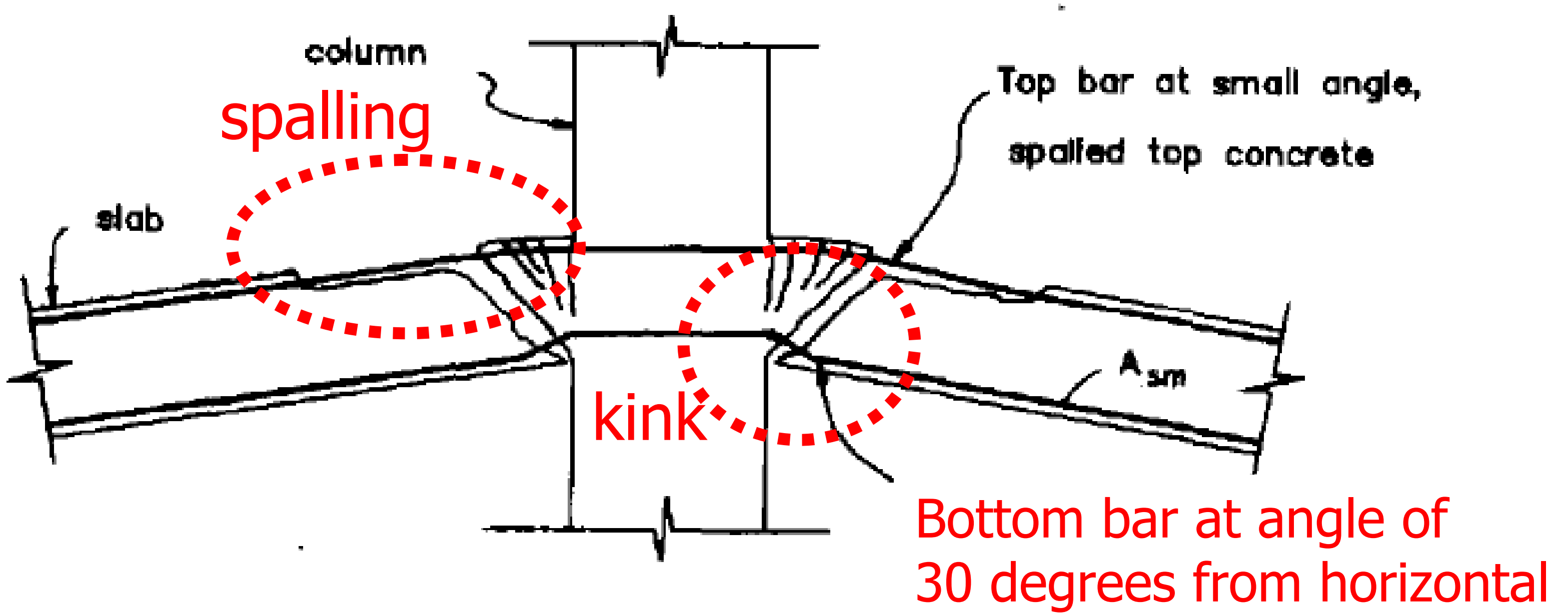


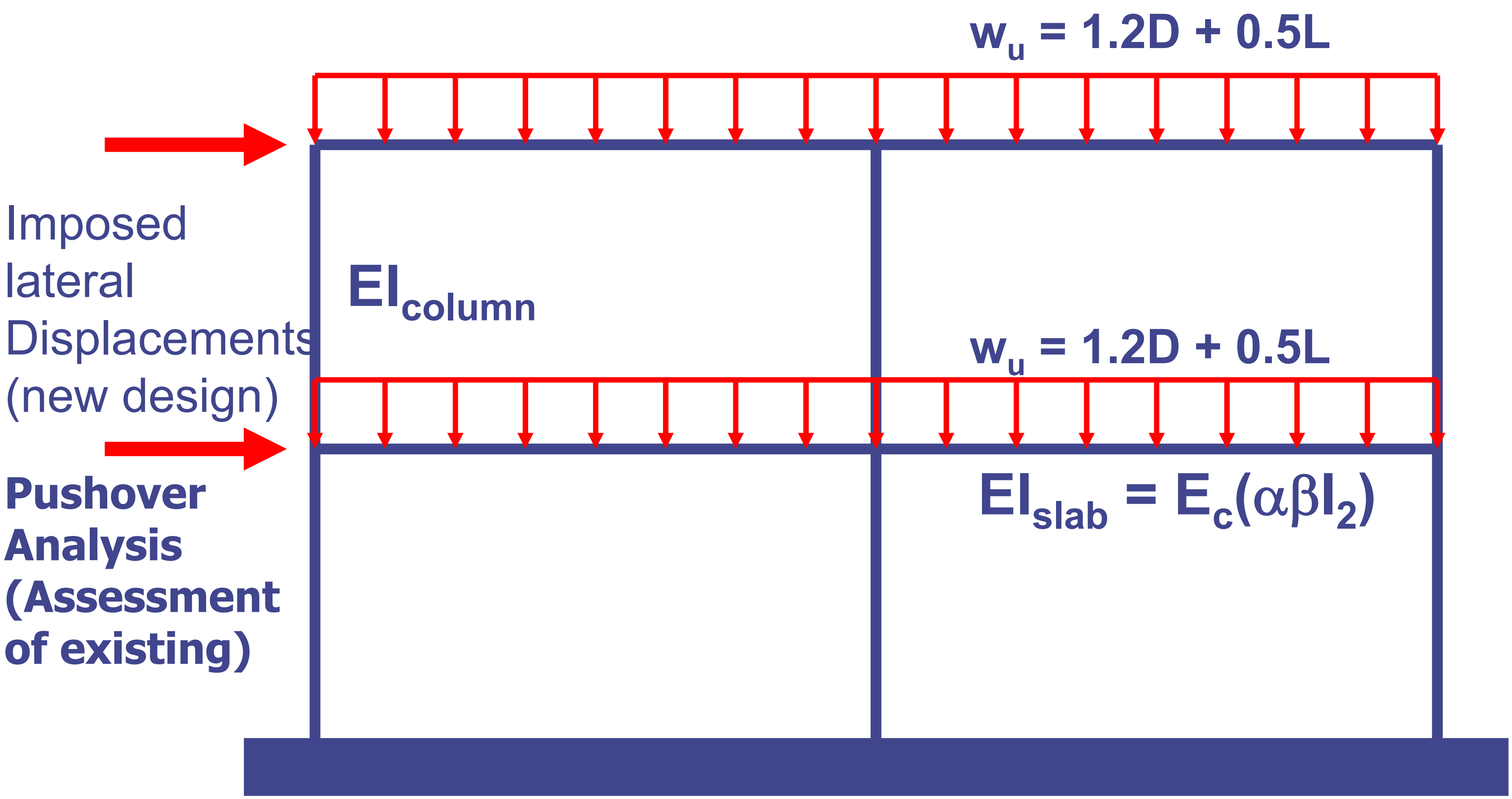
Fig. 5.4-Model of connection during punching failure

$$A_{sm} = 0.5\frac{w_u l_1 l_2}{\phi f_y}$$

Recommendations for the design of slab-column connections in monolithic RC Structures, ACI-ASCE Committee 352, Report 352.1R-89 (reapproved 1997)

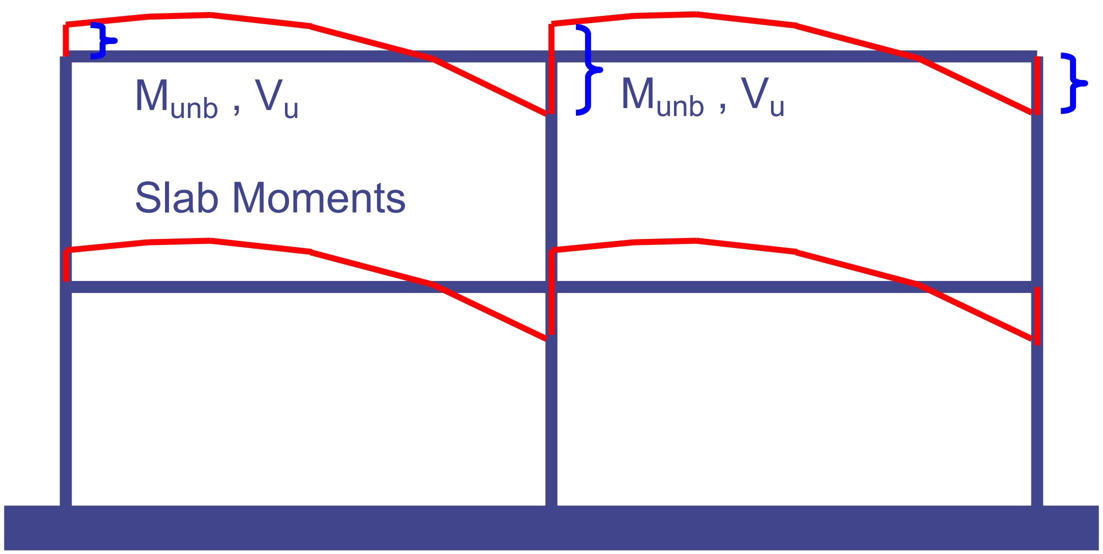
Deformation Compatibility

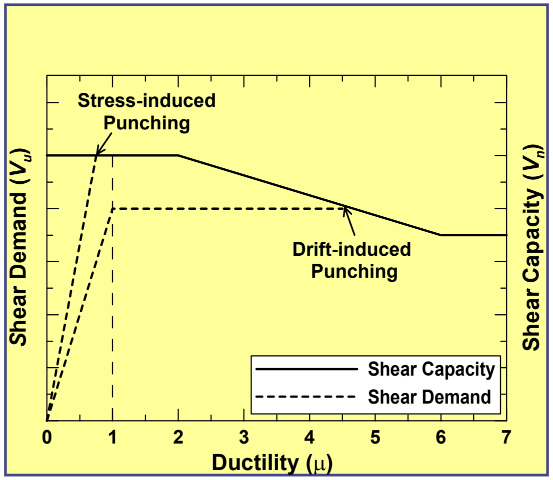
- Slab – column (gravity) frame assessment
  - Included in the model with the lateral system



Deformation Compatibility

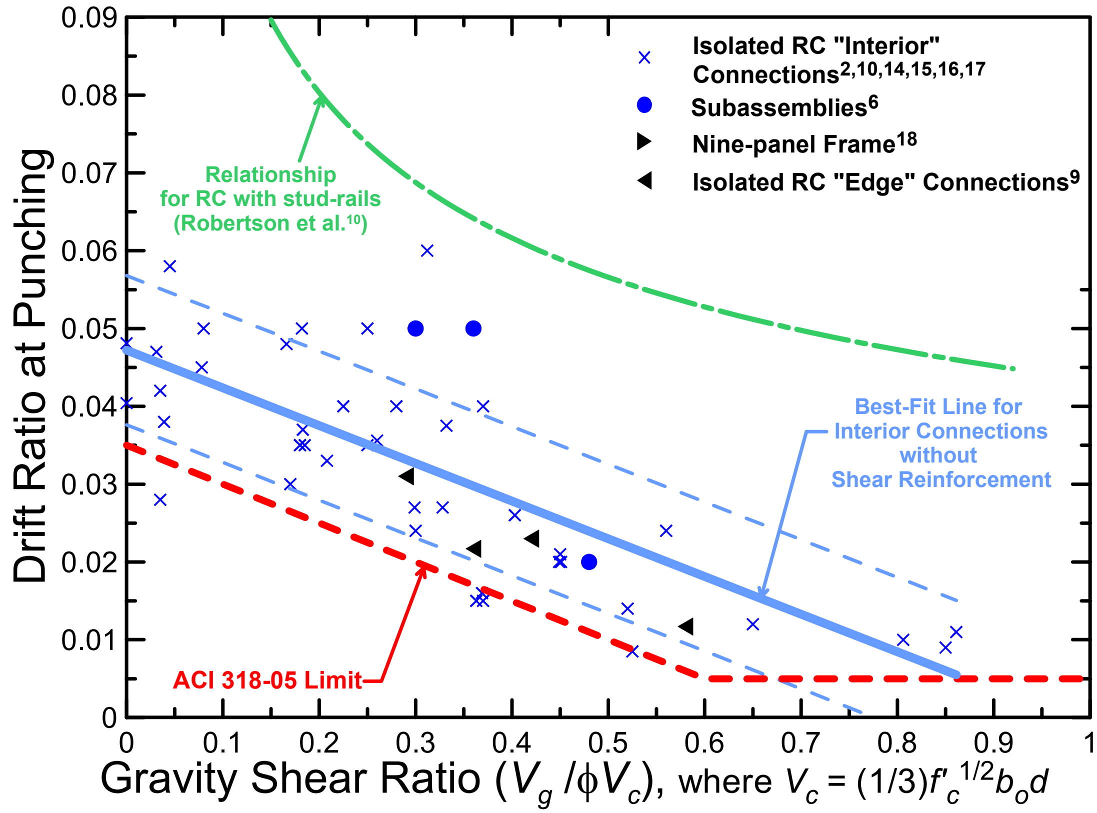
- Determine if the connection can resist the V_u & M_{unb} without punching failure – Adequate strength. (ACI 318-05 21.11.5)
  - Flexural transfer, eccentric shear stress model
  - Limit analysis approach – for connections with a fuse
  - this does not consider the potential for shear strength degradation.





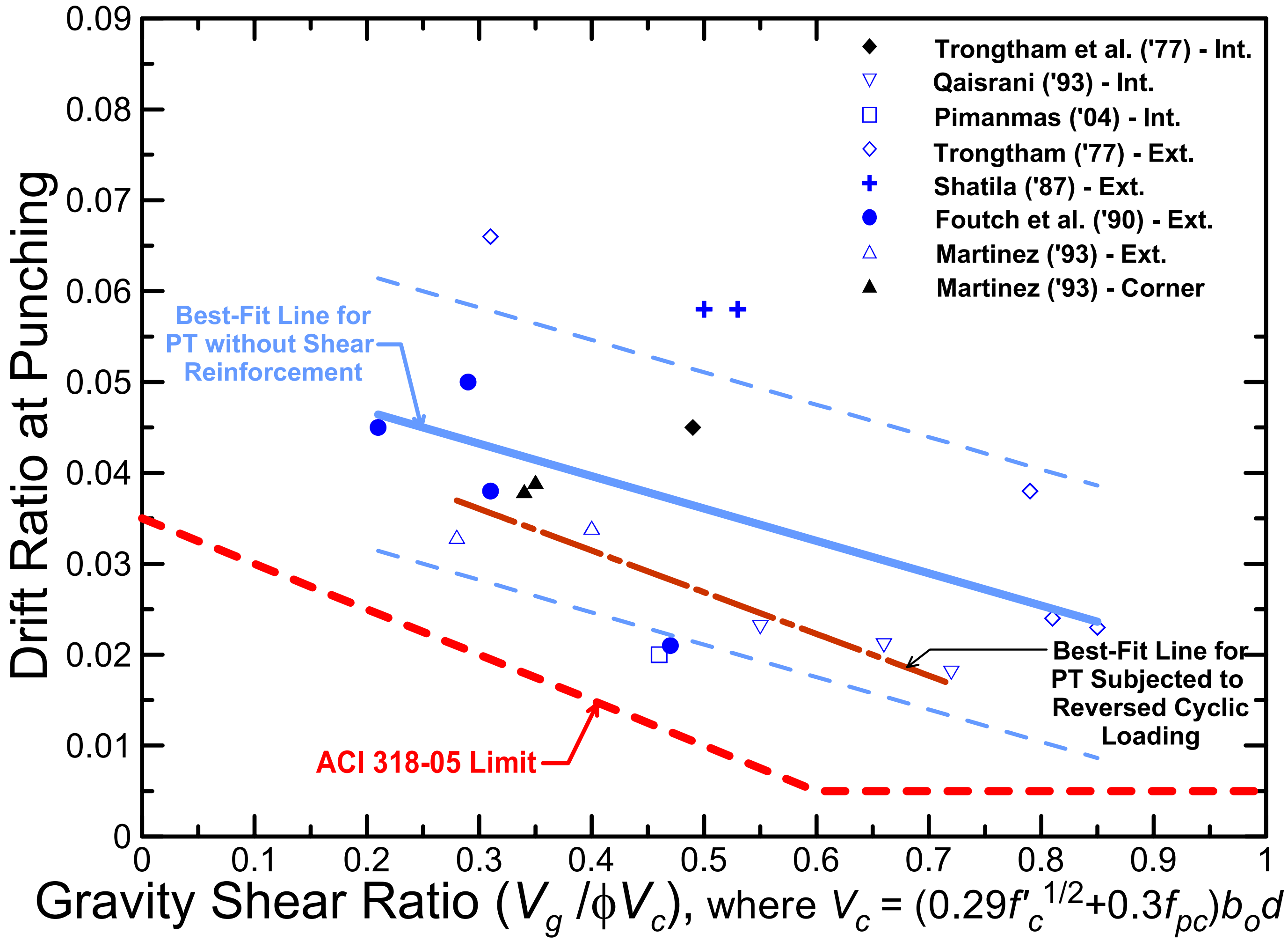
Alternative - Deformation Compatibility

- Verify that punching failures do not occur for gravity shear combined with imposed interstory displacement for Δ_M (new) or δ_{target} (Rehab). Adequate deformability. (ACI 318-05 21.11.5)
- RC interior and exterior (limited data) connections



Deformation Compatibility

- PT Connections without shear reinforcement



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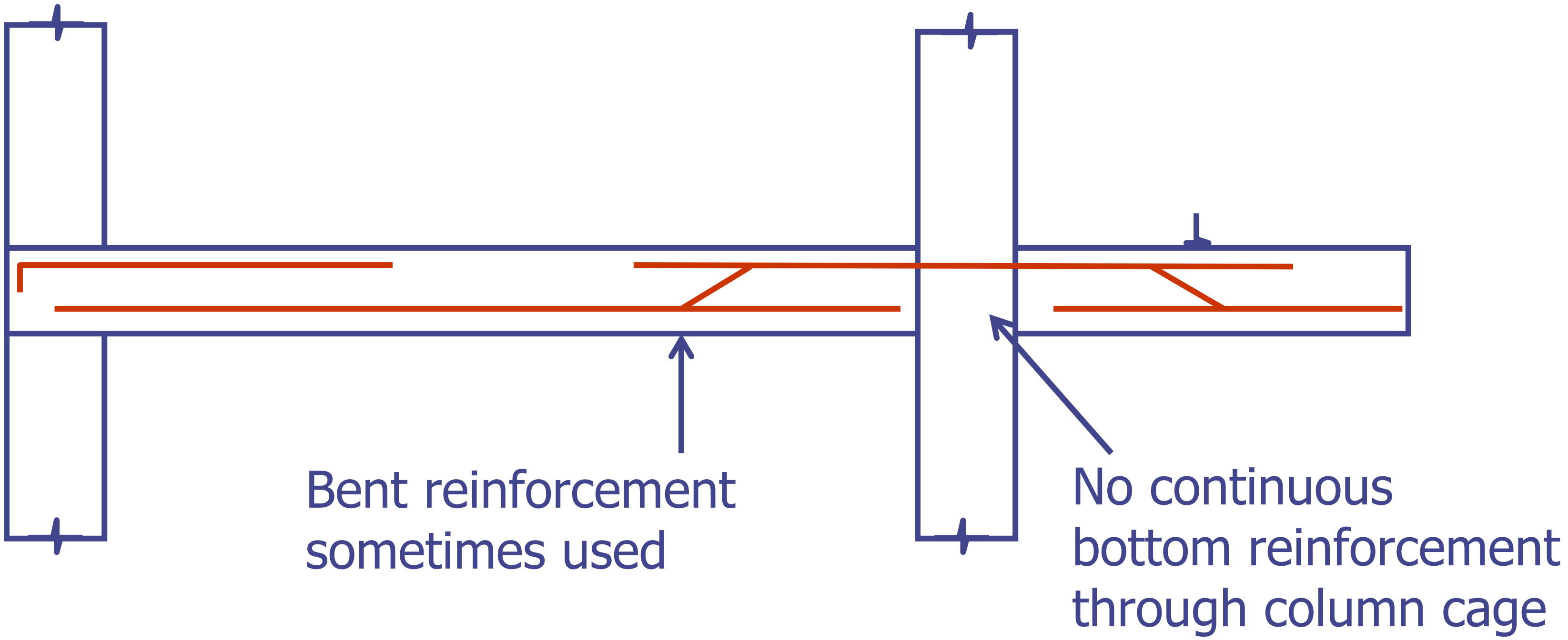
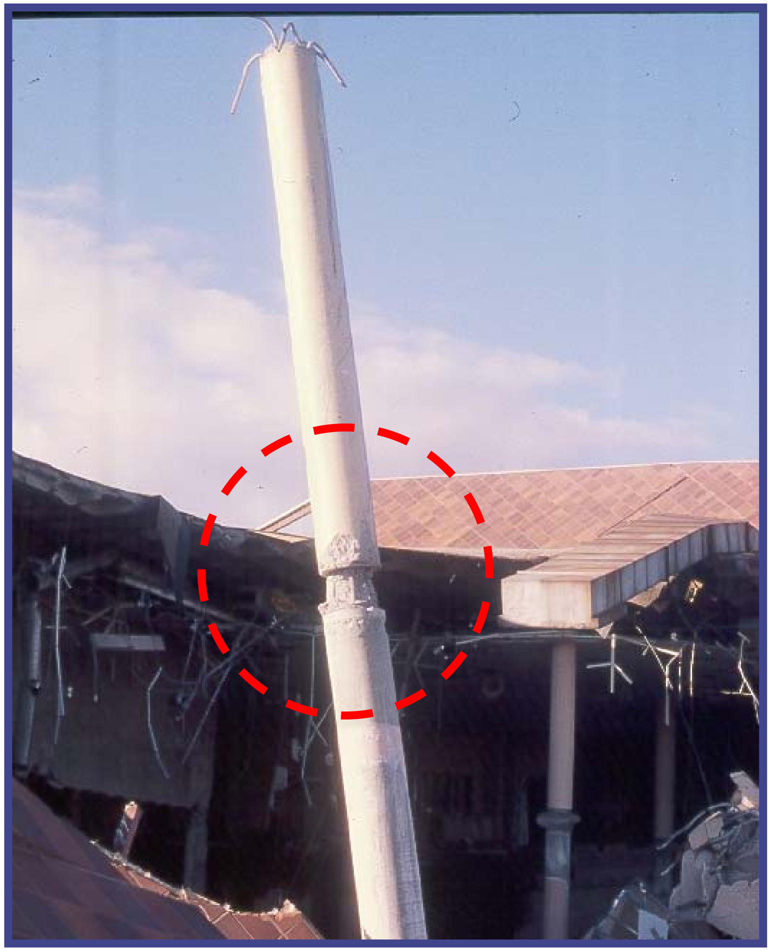
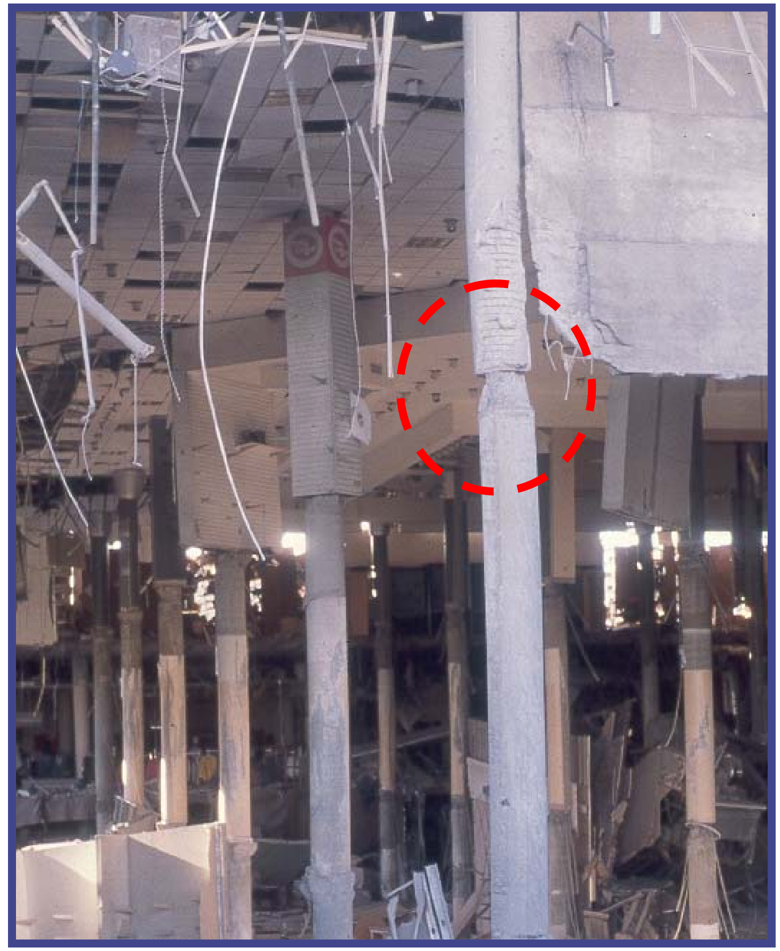
Older Construction

- Gravity design, or relatively low lateral forces used for design



Post-Earthquake Observations





Bullock's Department Store - Northridge Fashion Mall

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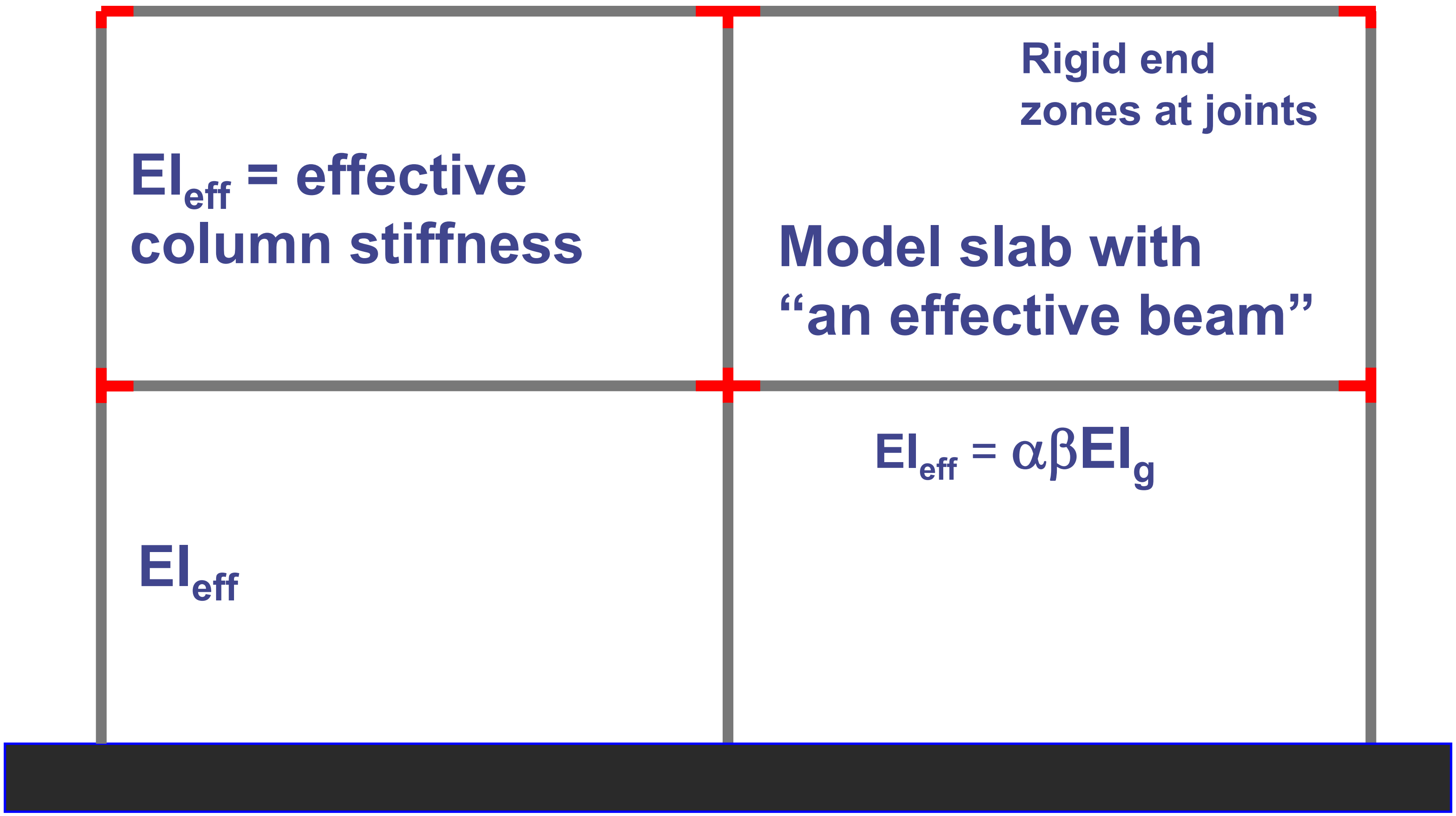
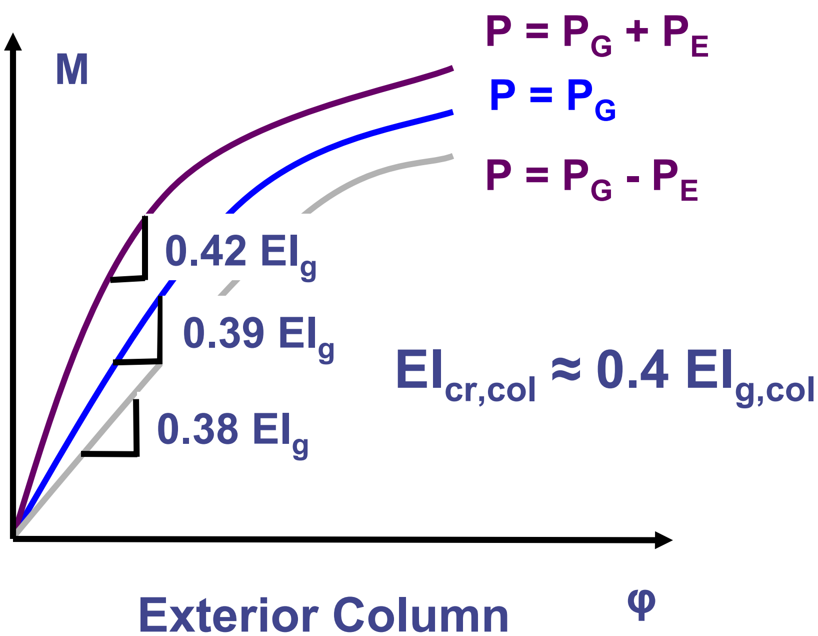
Modeling Overview

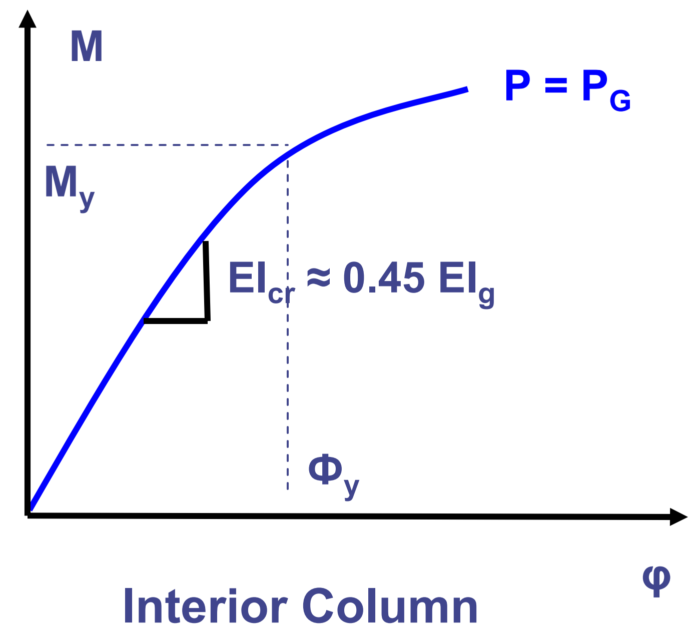
- How to model…
  - Lateral stiffness?
  - Connection behavior?
- How good are our models?
  - Shake table studies
- FEMA 356 backbone curves
  - Basis of existing curves
  - New information?

Modeling Assumptions - Typical



Analytical Model - Column Stiffness

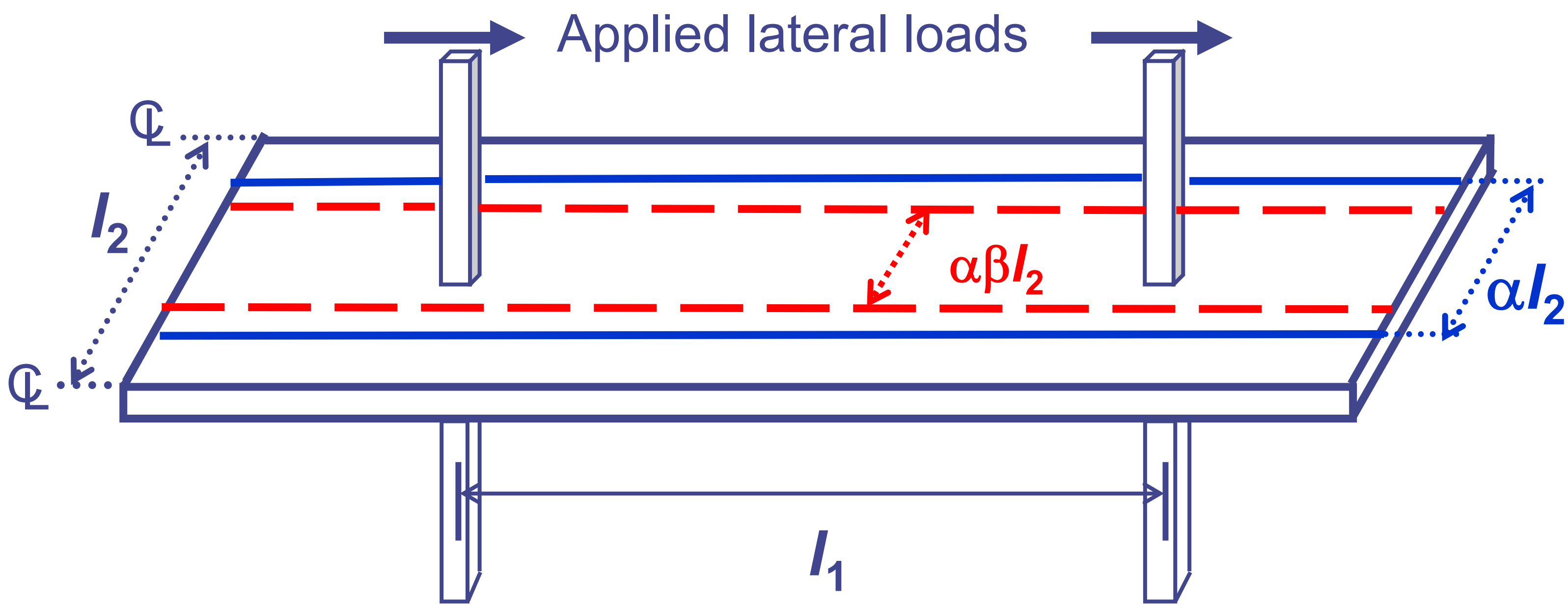




- P_G = axial from gravity   and P_E = axial from earthquake
- Anchorage slip – not likely as significant as noted for beam – column frames (see Elwood presentation)

Analytical Model – Slab Flexural Stiffness

Effective Beam Width Model



α: Effective Beam Width Factor

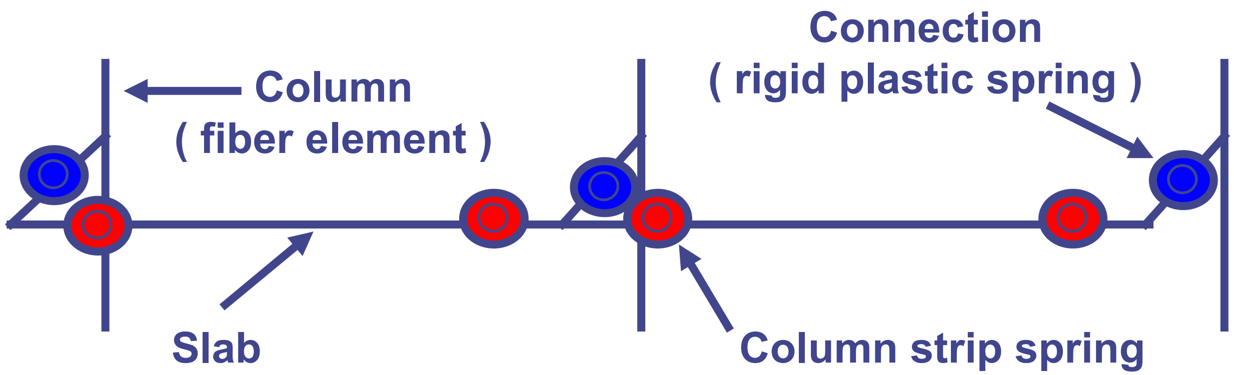
β: Coefficient accounting for Cracking

Allen & Darvall, ACI 74(7), 1977.
Grossman, ACI 94(2), 1997.
Hwang & Moehle, ACI 97(1), 2000.
Kang & Wallace, ACI 102(5), 2005.

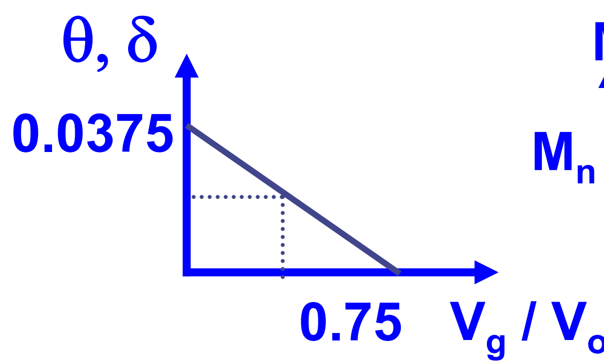
Kang & Wallace (2005)

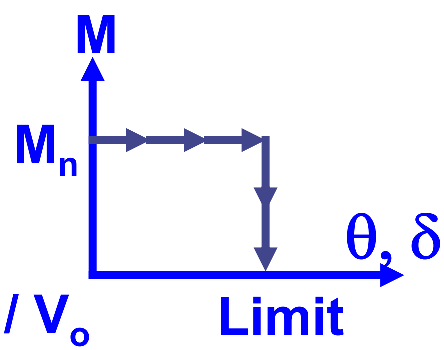
	RC	PT
α	**0.75**	**0.65**
β	**0.33**	**0.5**

Analytical Modeling - Connections



- $M_{unbalanced}$ @connection
  - Flexure c_2+3h (5h)
  - Eccentric shear
  - $M_n = M_f/0.6$
- M^-_n / M^+_n @ column strip



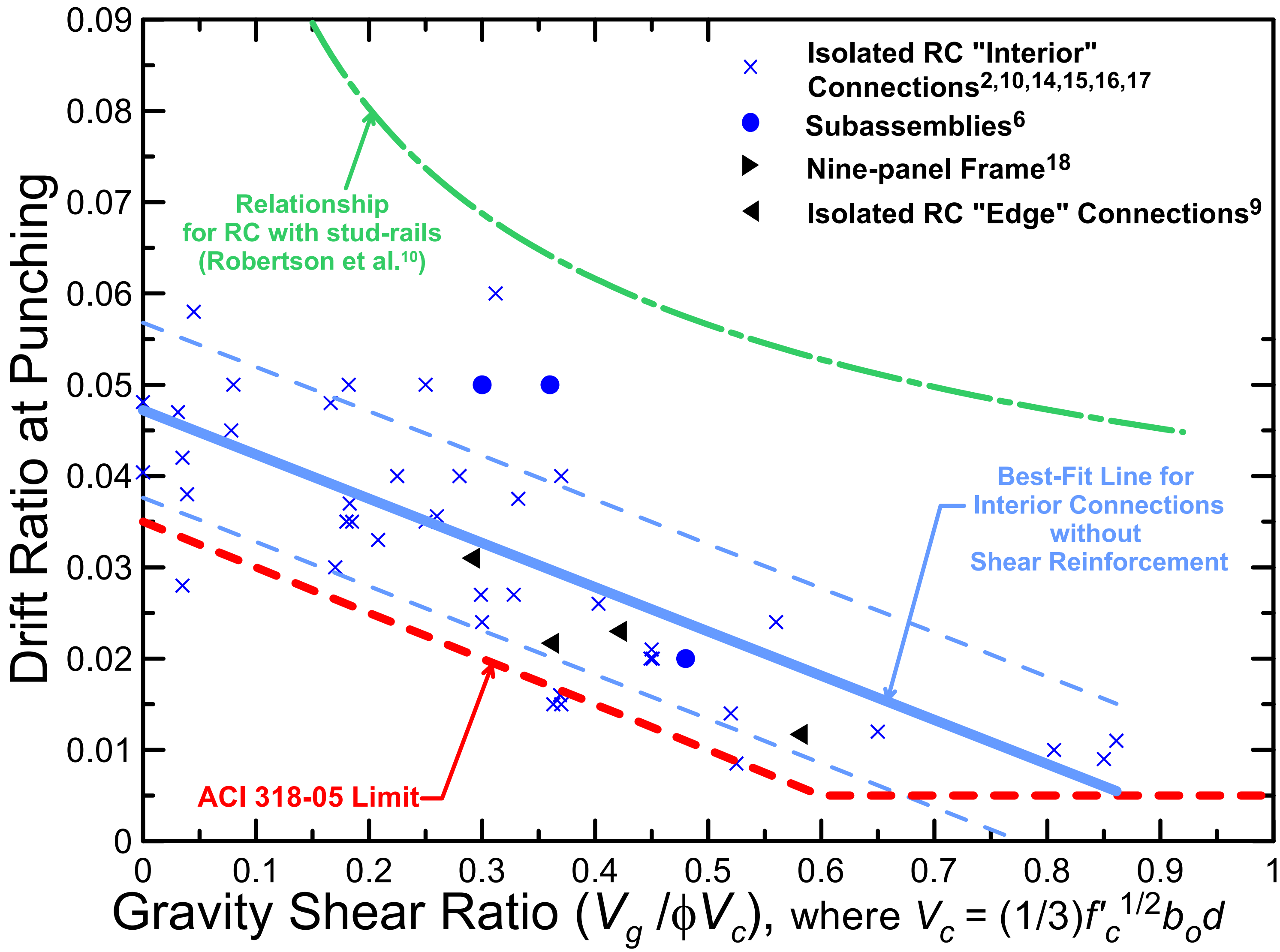


Punching before or after yielding
Kang, 13WCEE, Aug. 2004, Paper 1119

Limit State Model: Mean – 1 σ

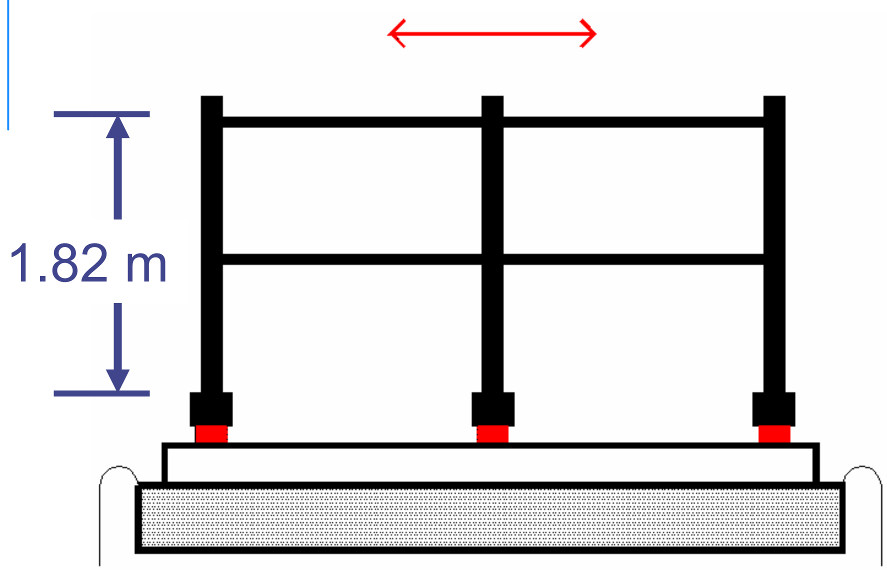
This model satisfies FEMA 356 6.5.4.2.2, which states that the connection must be modeled separately from slab and column elements.

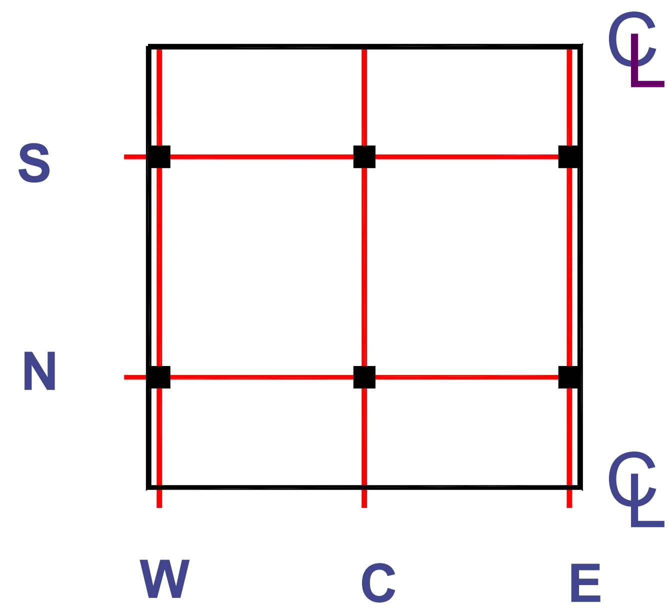
Connection Modeling - Punching



Shake Table Studies

- Two stories, 2 × 2 bays
  Approximately 1/3 scale

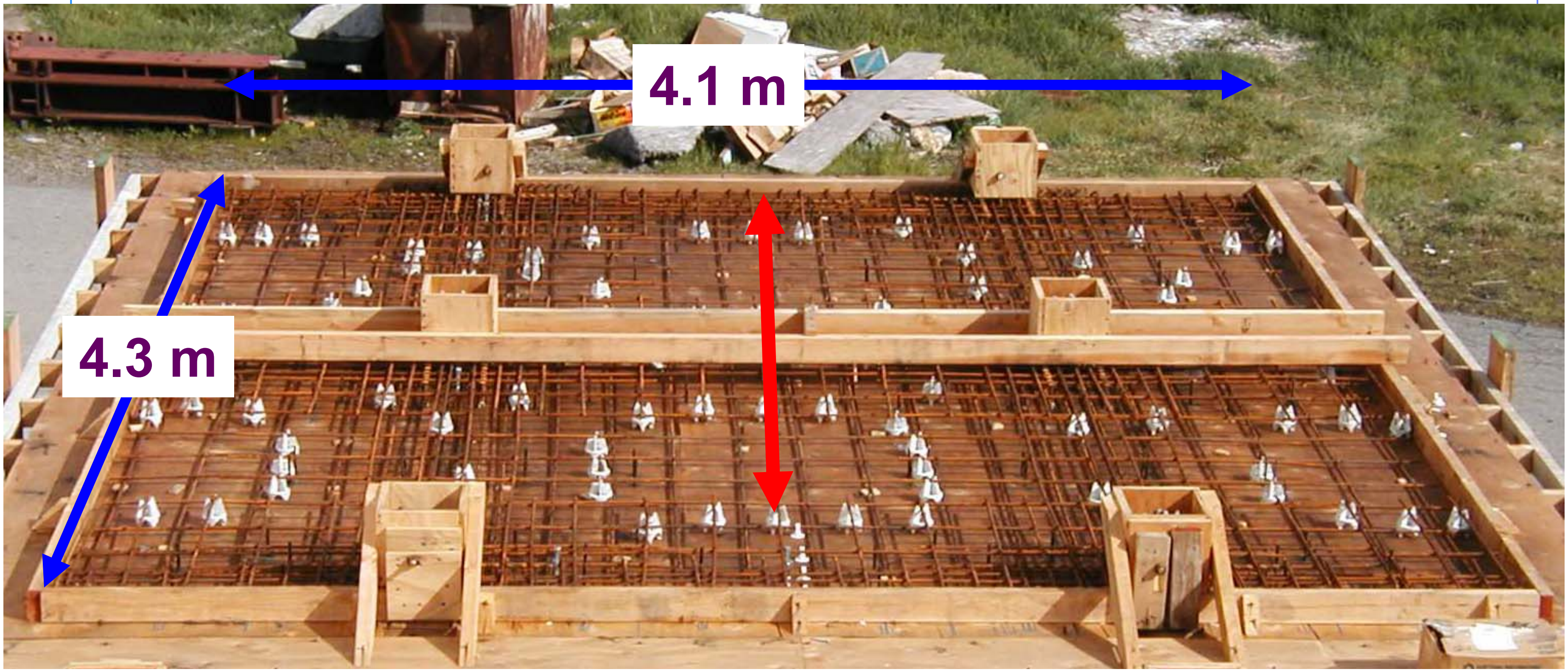




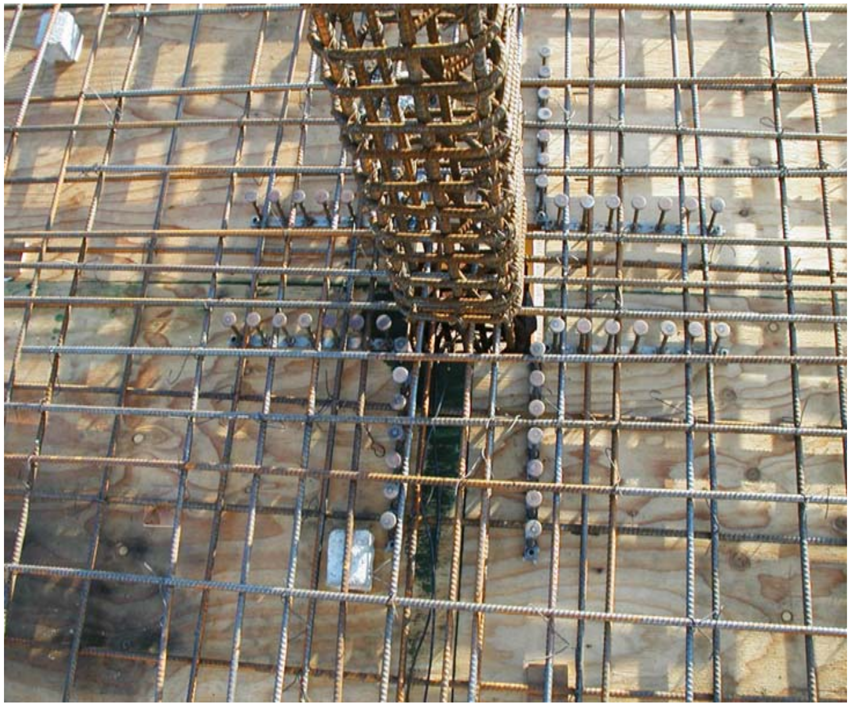
Kang and Wallace, ACI SJ, Sept-Oct, 2005, another paper in-press.

RC Specimen

- Six 200 x 200 mm columns
- 90 mm thick slab
- 9.5 mm rebar f_y = 414 MPa
- f'_c = 28 MPa



RC Specimen - Reinforcement

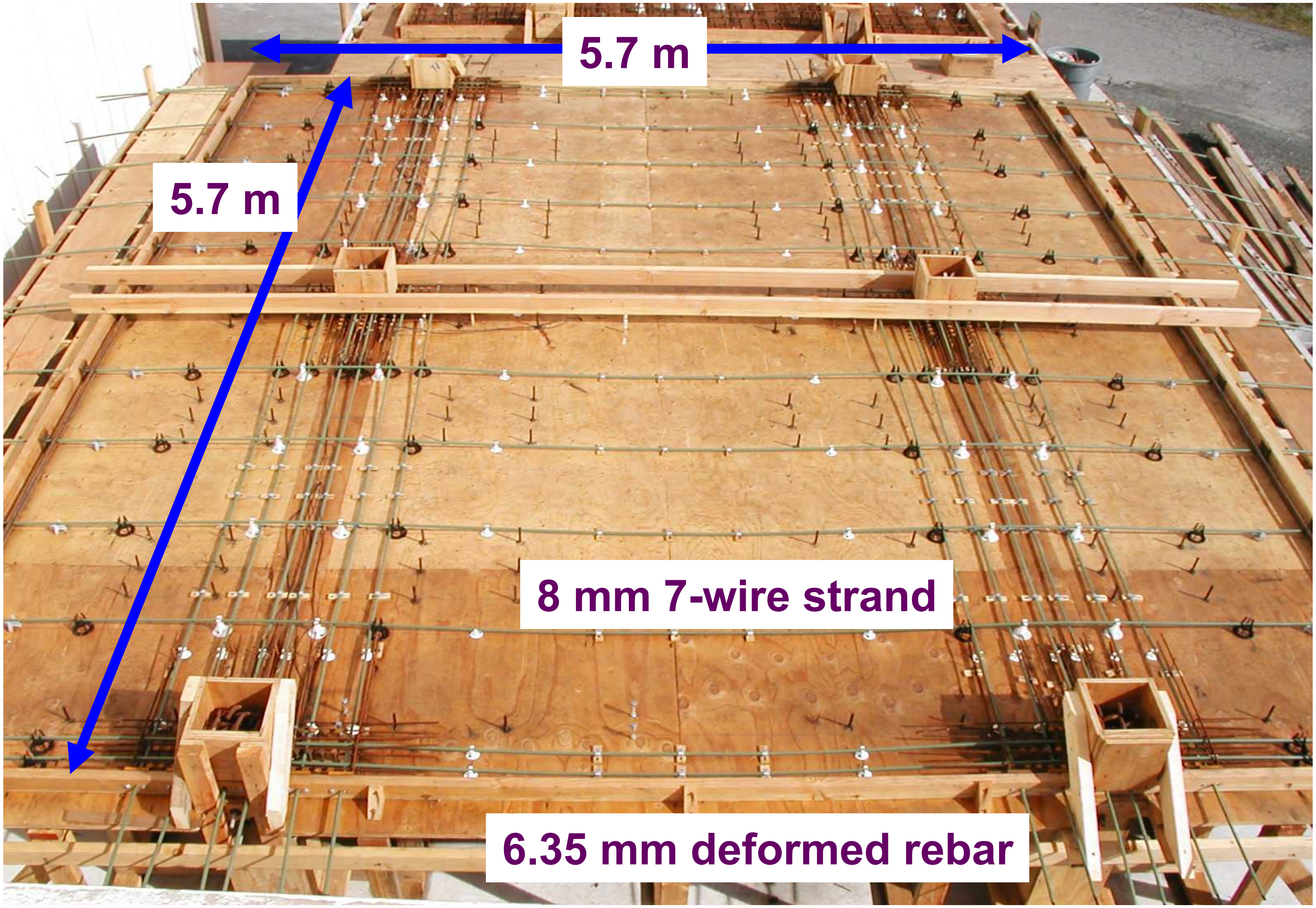


Interior Connection

Shear Reinforcement

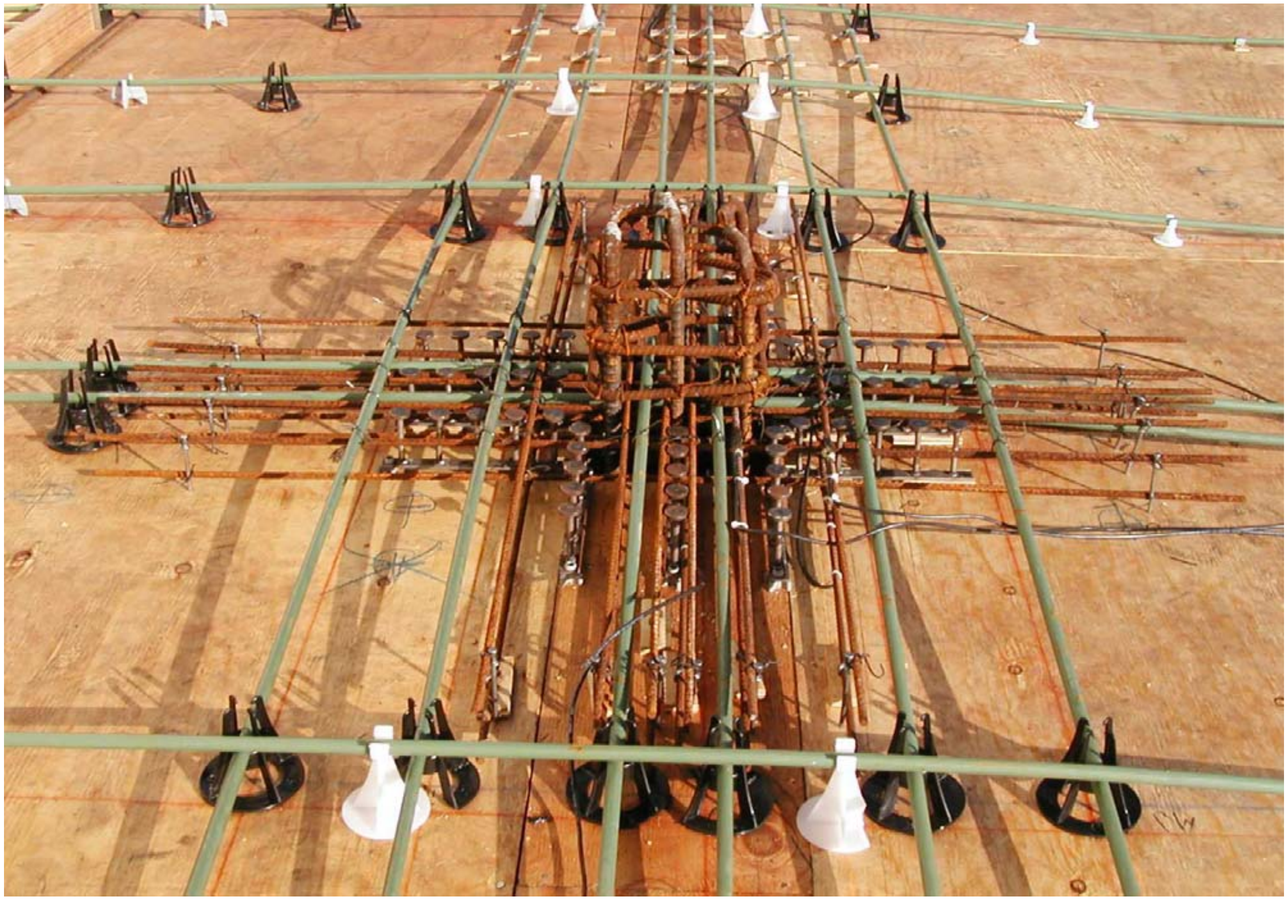
Expected connection behavior: Flexural yielding, followed by punching

PT Specimen

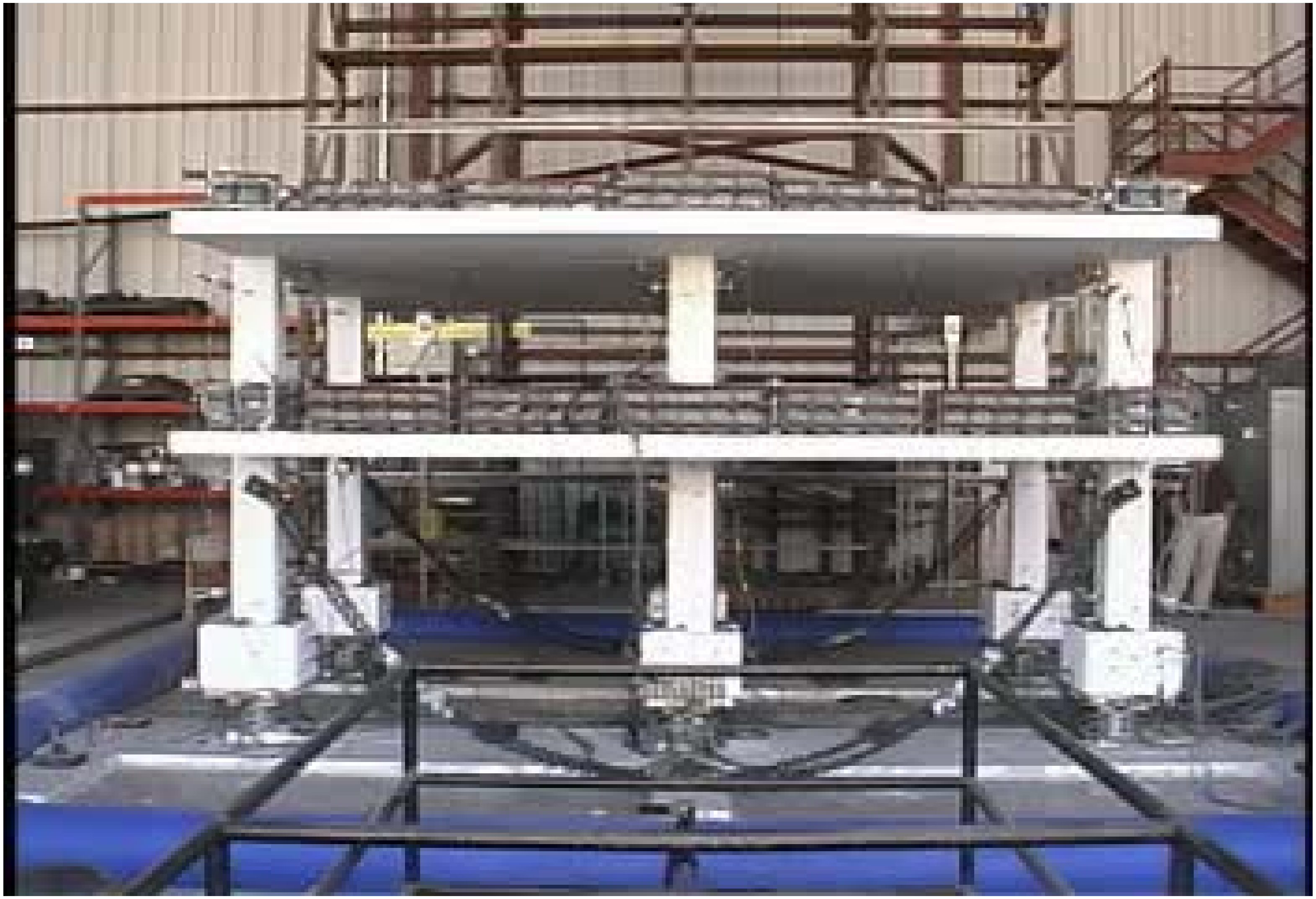


PT Specimen – Interior

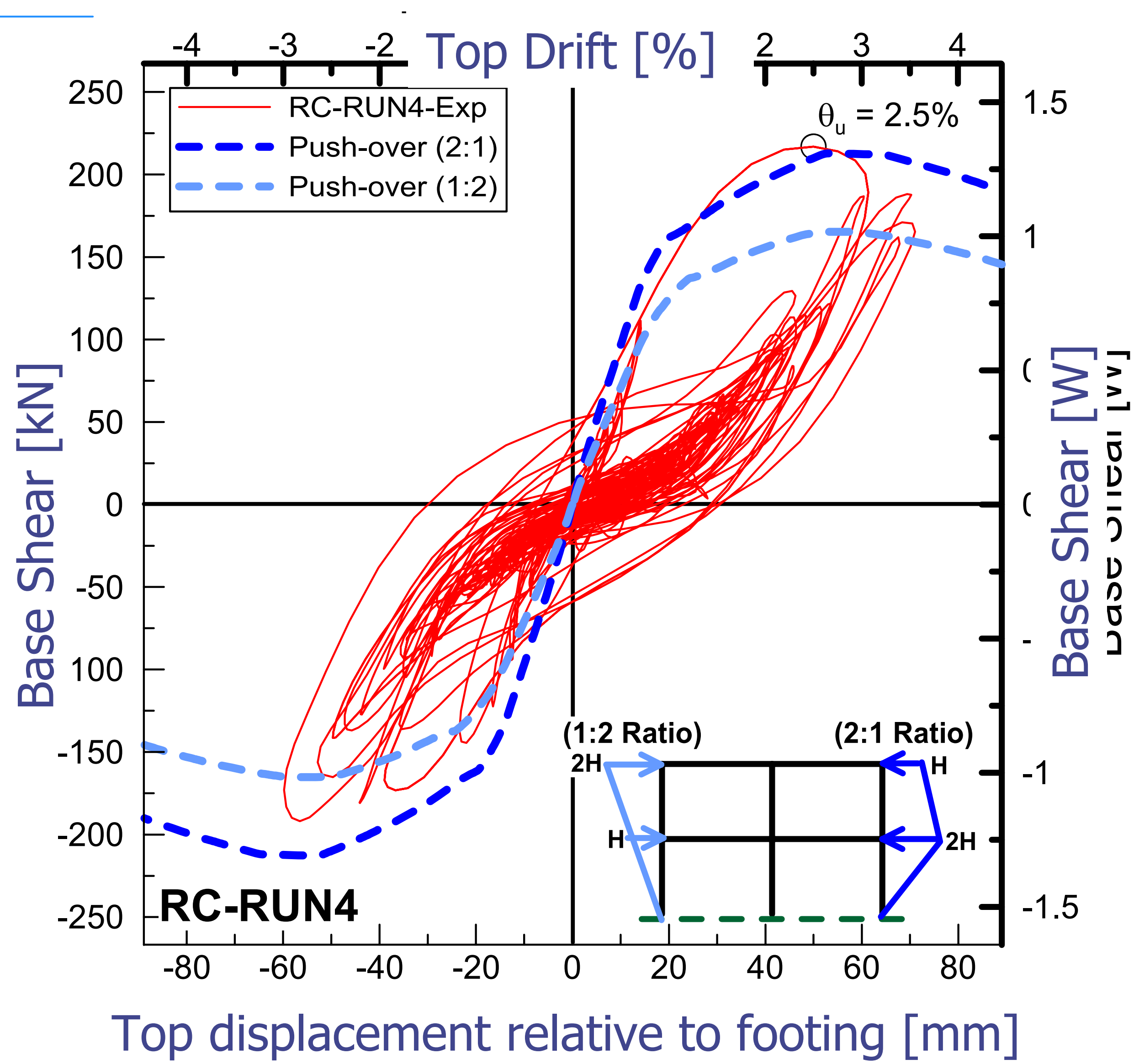
ACI318-05 Requires only bottom (integrity) reinforcement



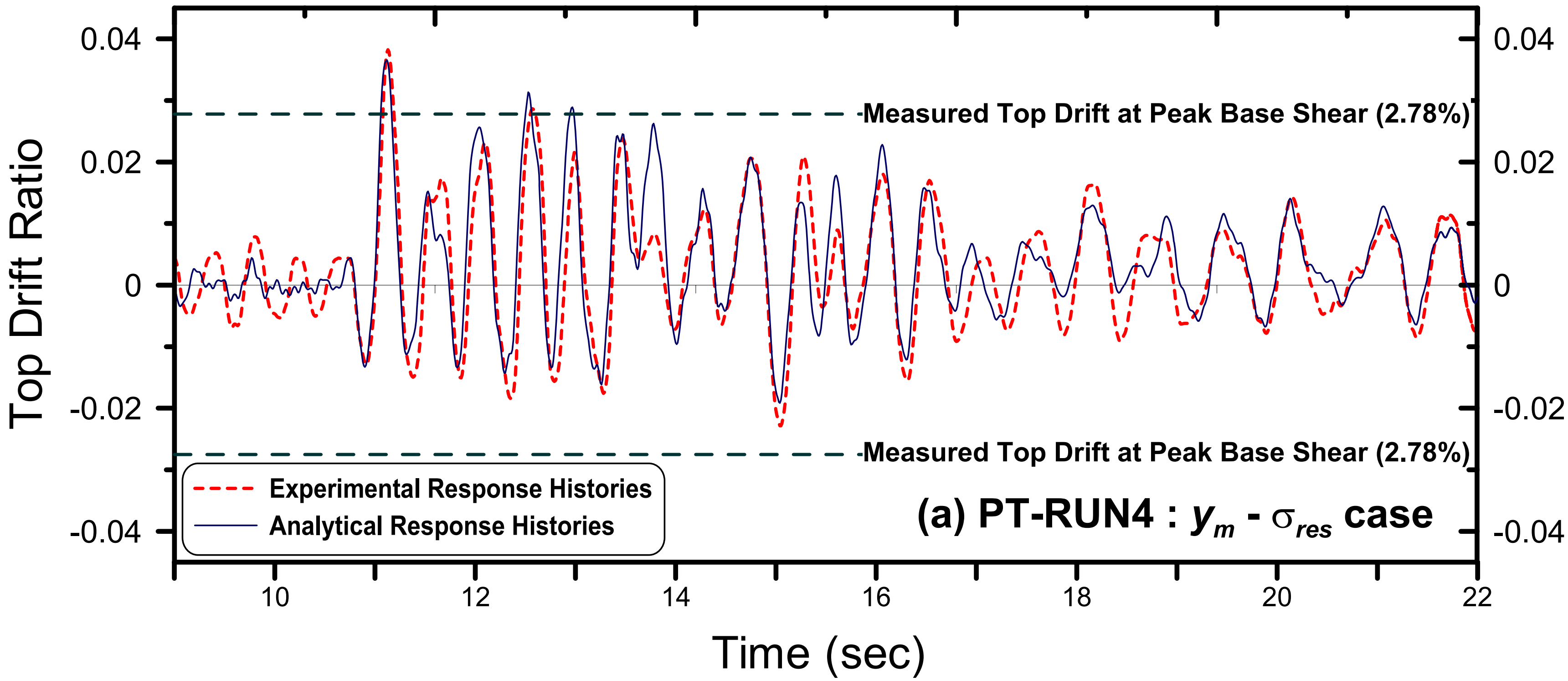
PT Video – Run 5



Model Assessment - NSP



Model Assessment - PT



See Kang et al., 13WCEE, August 2004, paper 1119
Direct measurement of footing rotations

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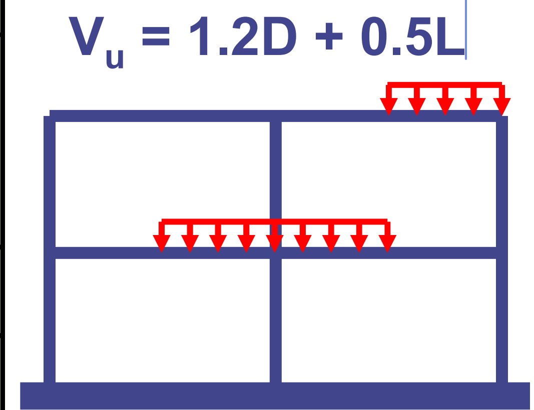
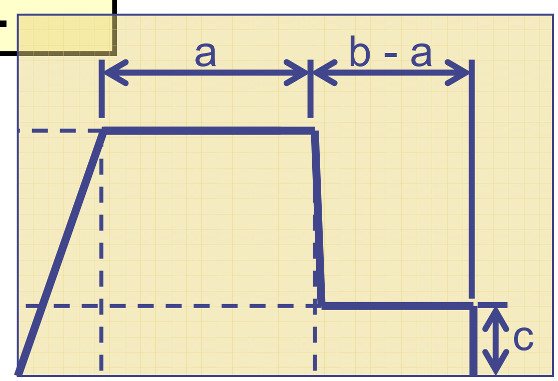
Deformation – Backbone Curves

Slabs Controlled by Flexure		Model Parameters, Radians		
$\frac{V_{gravity}}{V_0}$	Continuity Reinforcement	Plastic Hinge a	Plastic Hinge b	Residual Strength c
≤ 0.2	Yes	0.02	0.05	0.2
> 0.4	Yes	0.0	0.04	0.2
≤ 0.2	No	0.02	0.02	--
≥ 0.25	No	0.0	0.0	--

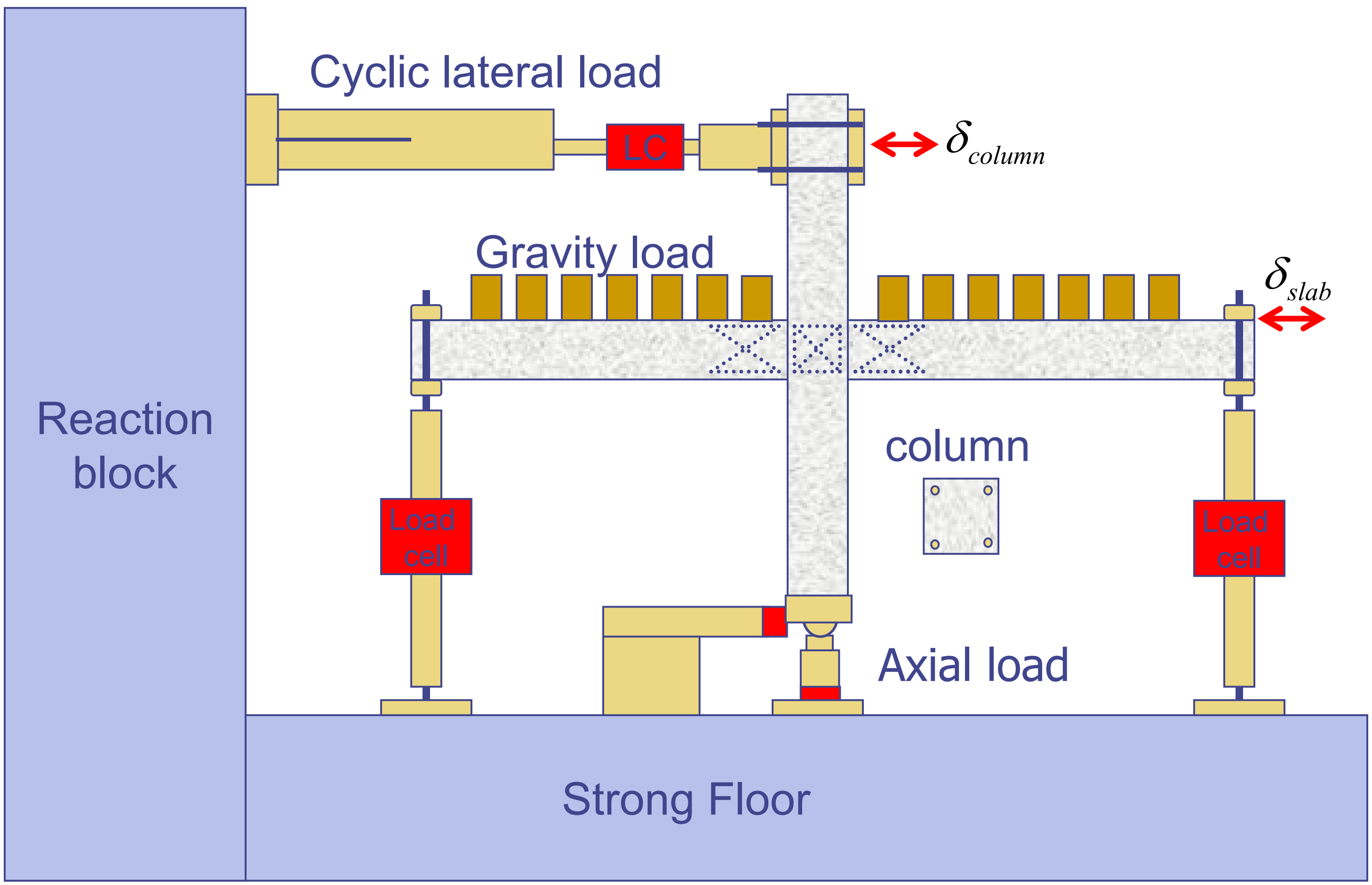


Continuity reinforcement defined as at least one bottom bar or pt bar continuous through the column cage in each direction



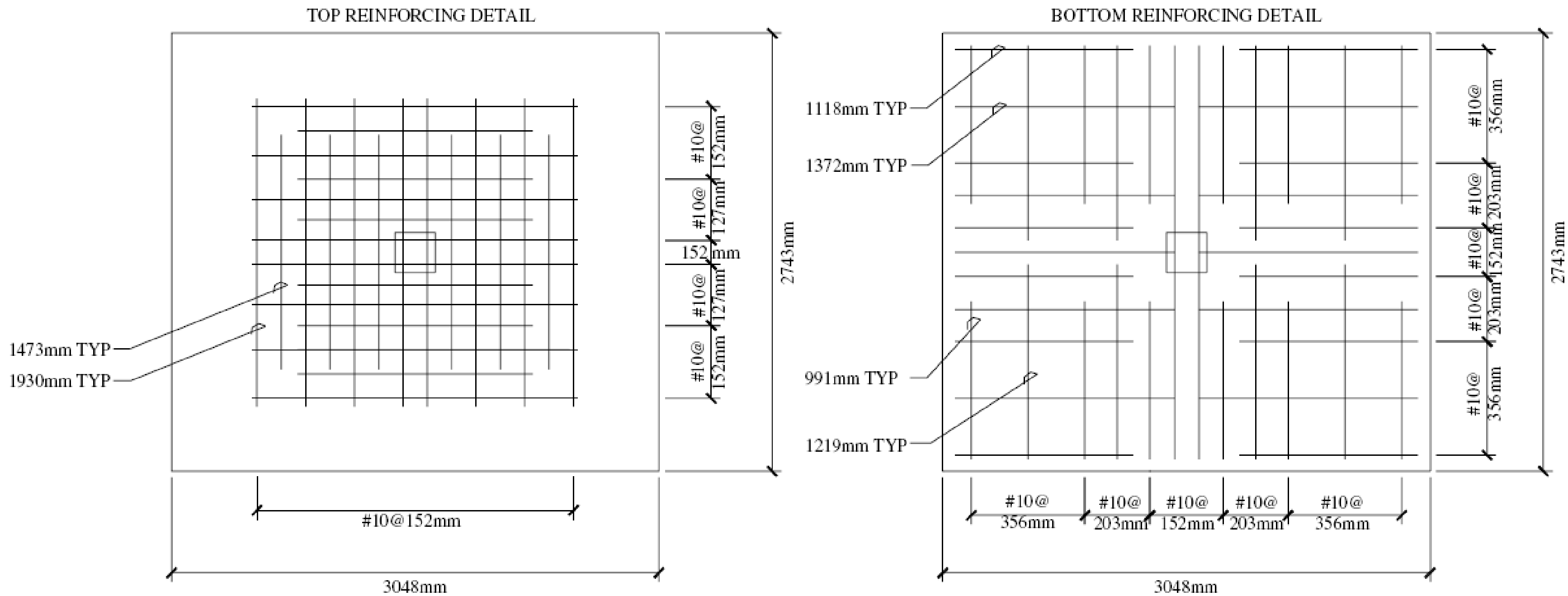
Slab – Column Tests

- Typical test setup



New Data – Test Results #1

- Slab reinforcing details



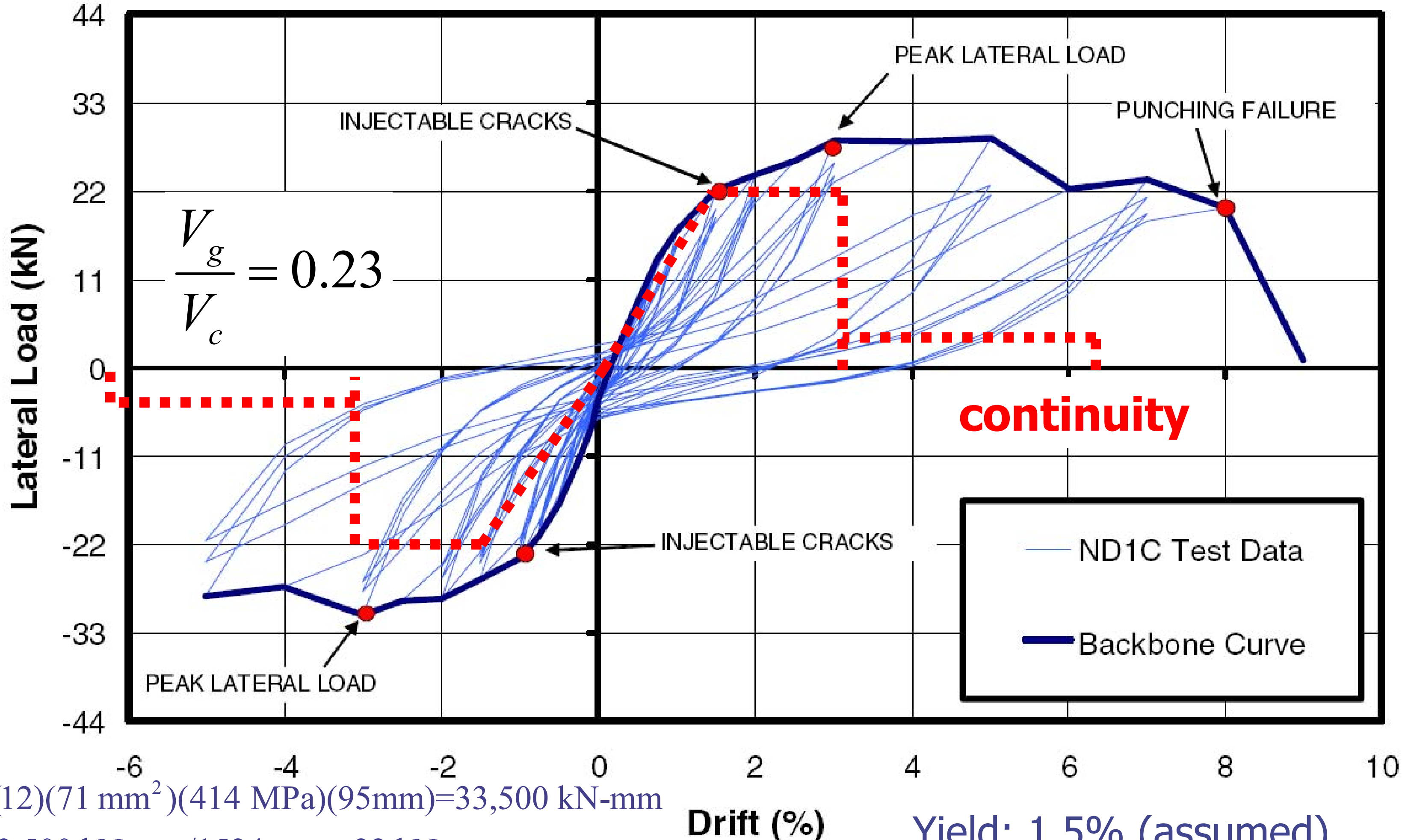
10ft x 10 ft x 4.5”

2 Continuous bottom bars

Figure 2-Slab Reinforcement for ND1C, ND4LL, and ND5XL.

Robertson & Johnson, 13WCEE, August 2004, Paper 143

Test Results #1 – Control Specimen



$M_n = (12)(71\ \text{mm}^2)(414\ \text{MPa})(95\text{mm}) = 33{,}500\ \text{kN-mm}$

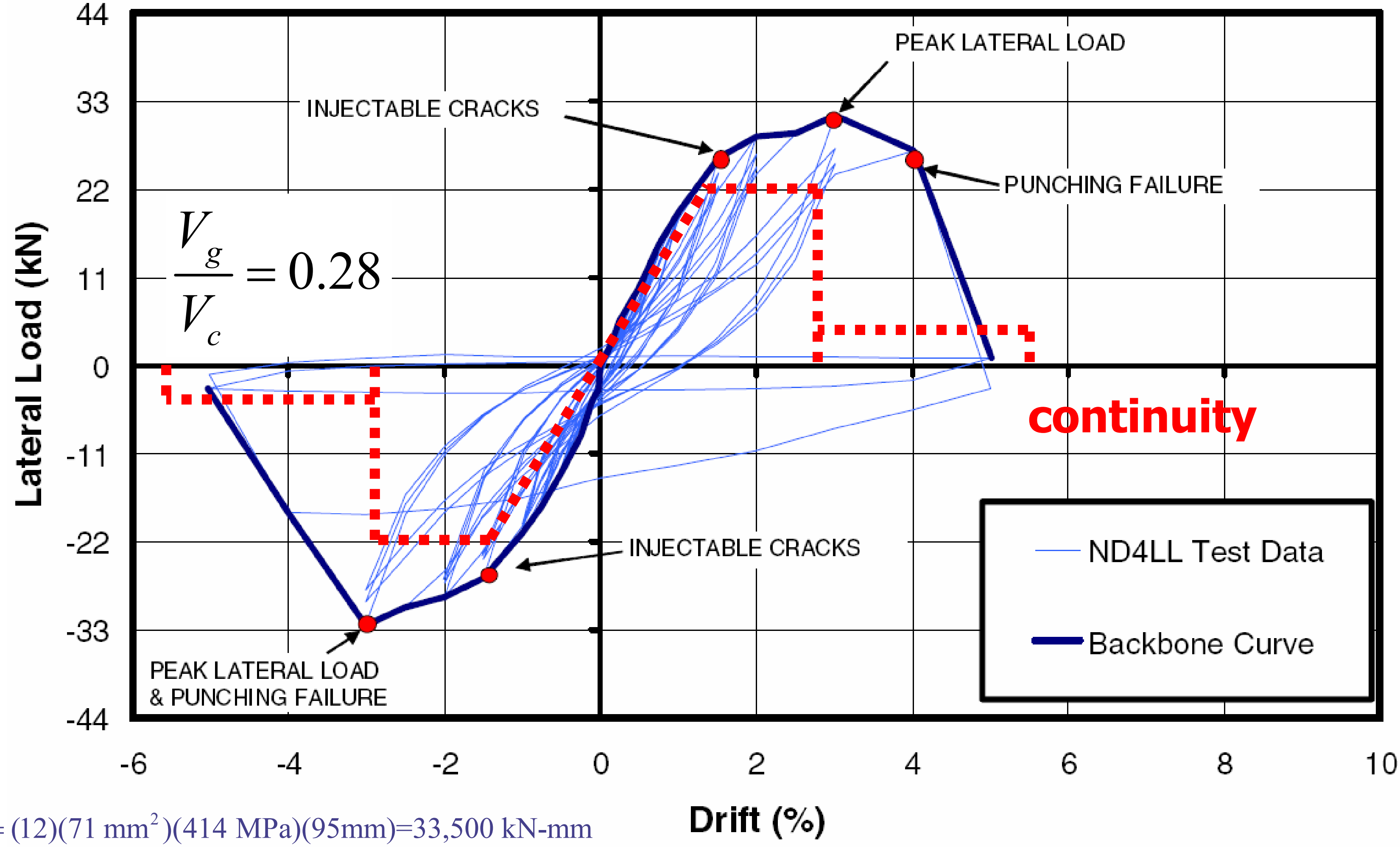
$P_n = 33{,}500\ \text{kN-mm}/1524\text{mm} = 22\ \text{kN}$

$\theta_e = 0.015; \quad \theta_p = 0.02(17/20) = 0.017$

Yield: 1.5% (assumed)

Robertson & Johnson, 13WCEE, August 2004, Paper 143

Test Results #1

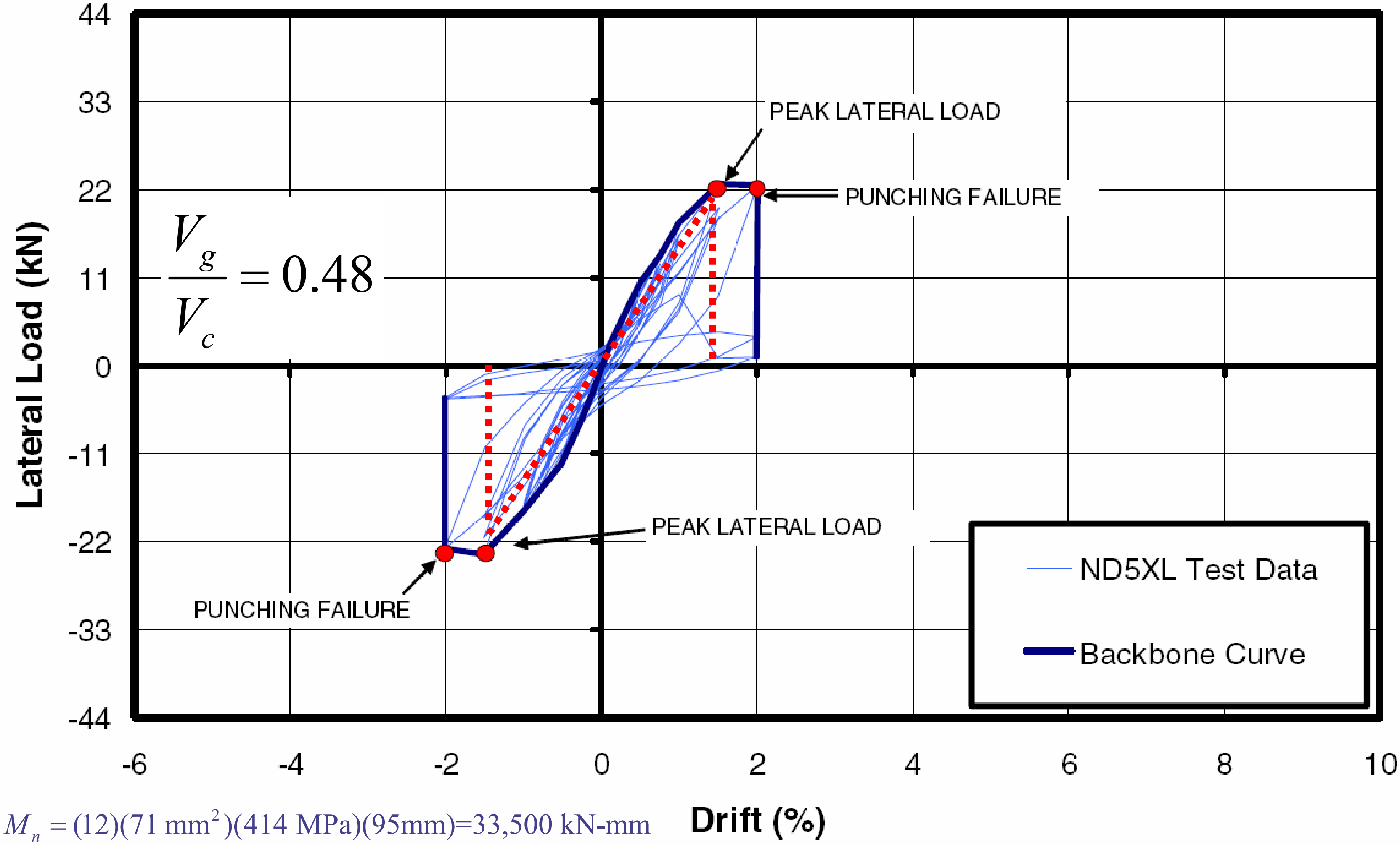


$M_n = (12)(71\ \text{mm}^2)(414\ \text{MPa})(95\text{mm}) = 33{,}500\ \text{kN-mm}$

$P_n = 33{,}500\ \text{kN-mm}/1524\text{mm} = 22\ \text{kN}$

$\theta_e = 0.015;\quad \theta_p = 0.02(12/20) = 0.012$

Test Results #1



$M_n = (12)(71\ \text{mm}^2)(414\ \text{MPa})(95\text{mm}) = 33{,}500\ \text{kN-mm}$

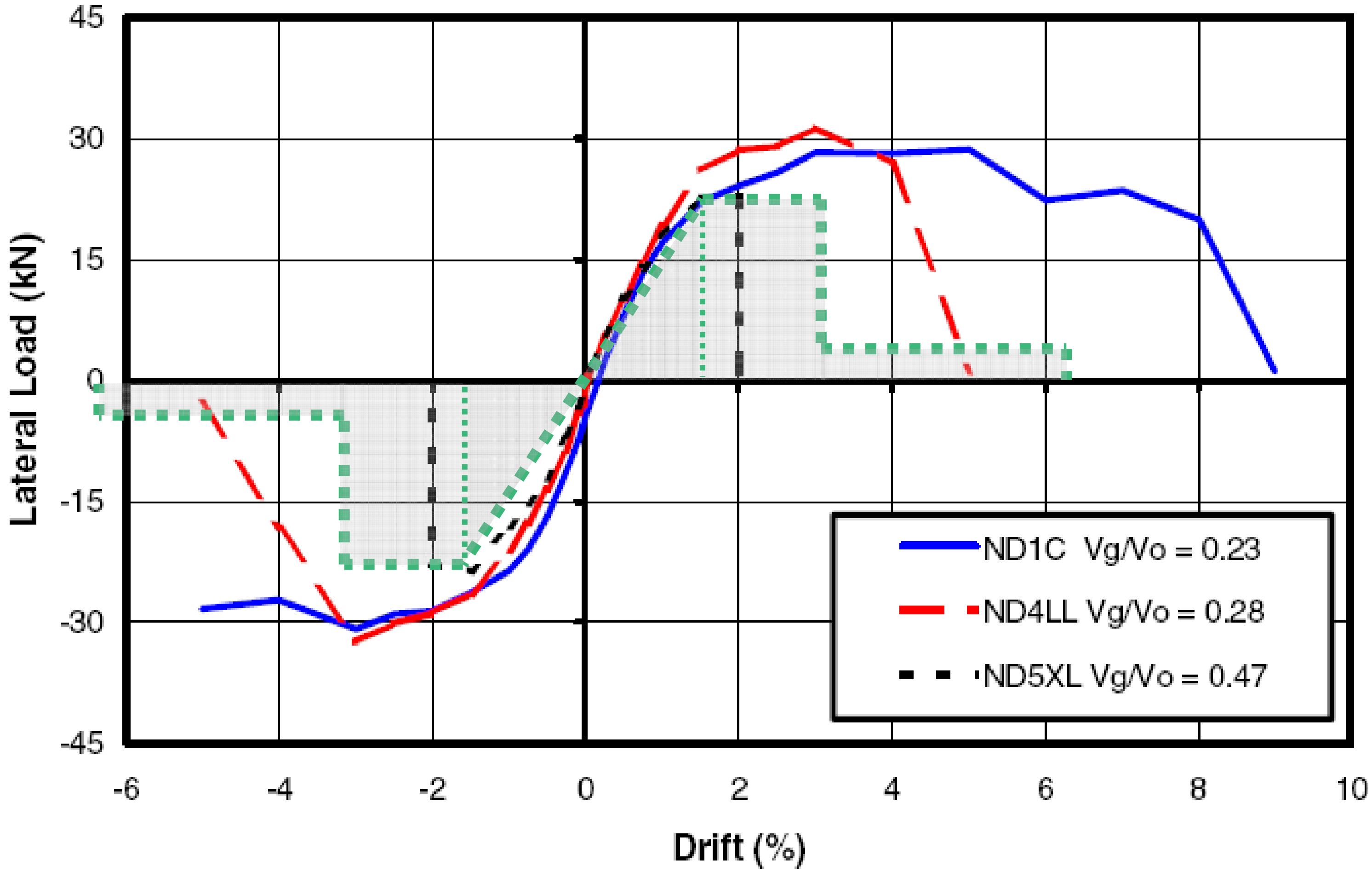
$P_n = 33{,}500\ \text{kN-mm}/1524\text{mm} = 22\ \text{kN}$

$\theta_e = 0.015; \quad \theta_p = 0$

Robertson & Johnson, 13WCEE, August 2004, Paper 143

Test Results #1 - Summary

- FEMA 356 – Overall comparison



Robertson & Johnson, 13WCEE, August 2004, Paper 143

Test Results - #2

- Slab reinforcing details – less reinforcement

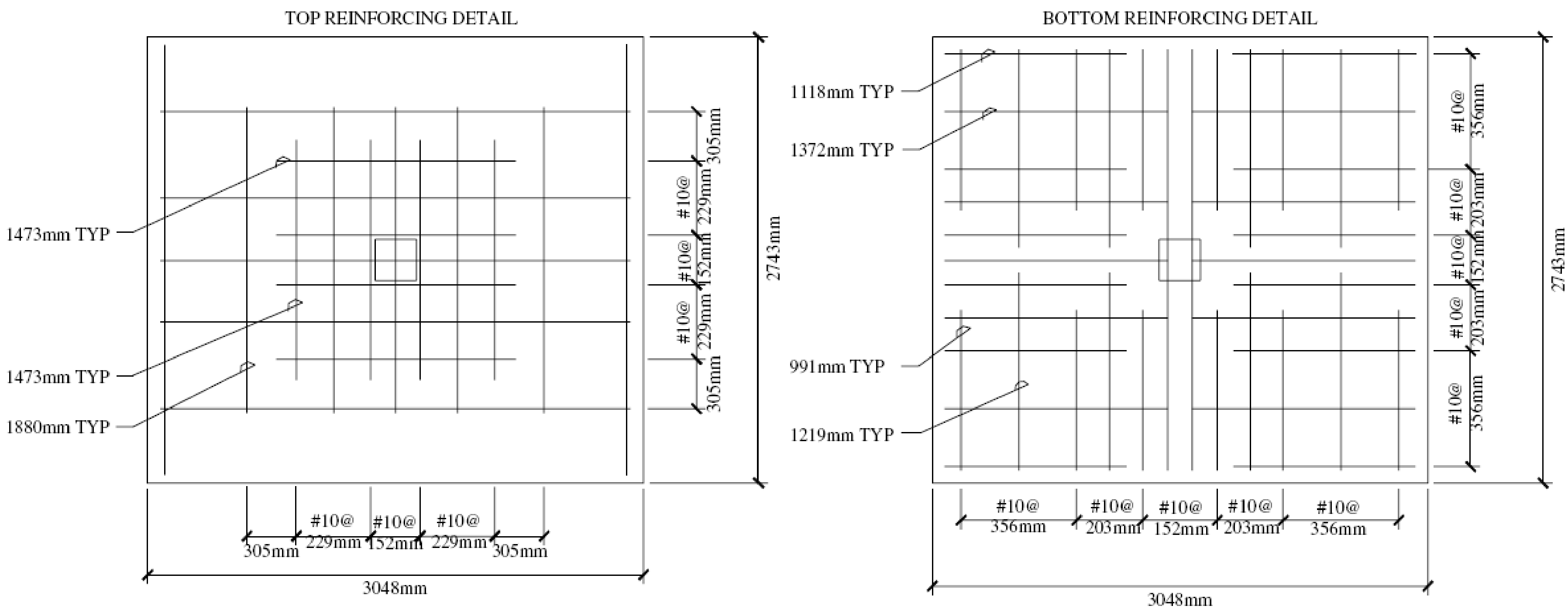


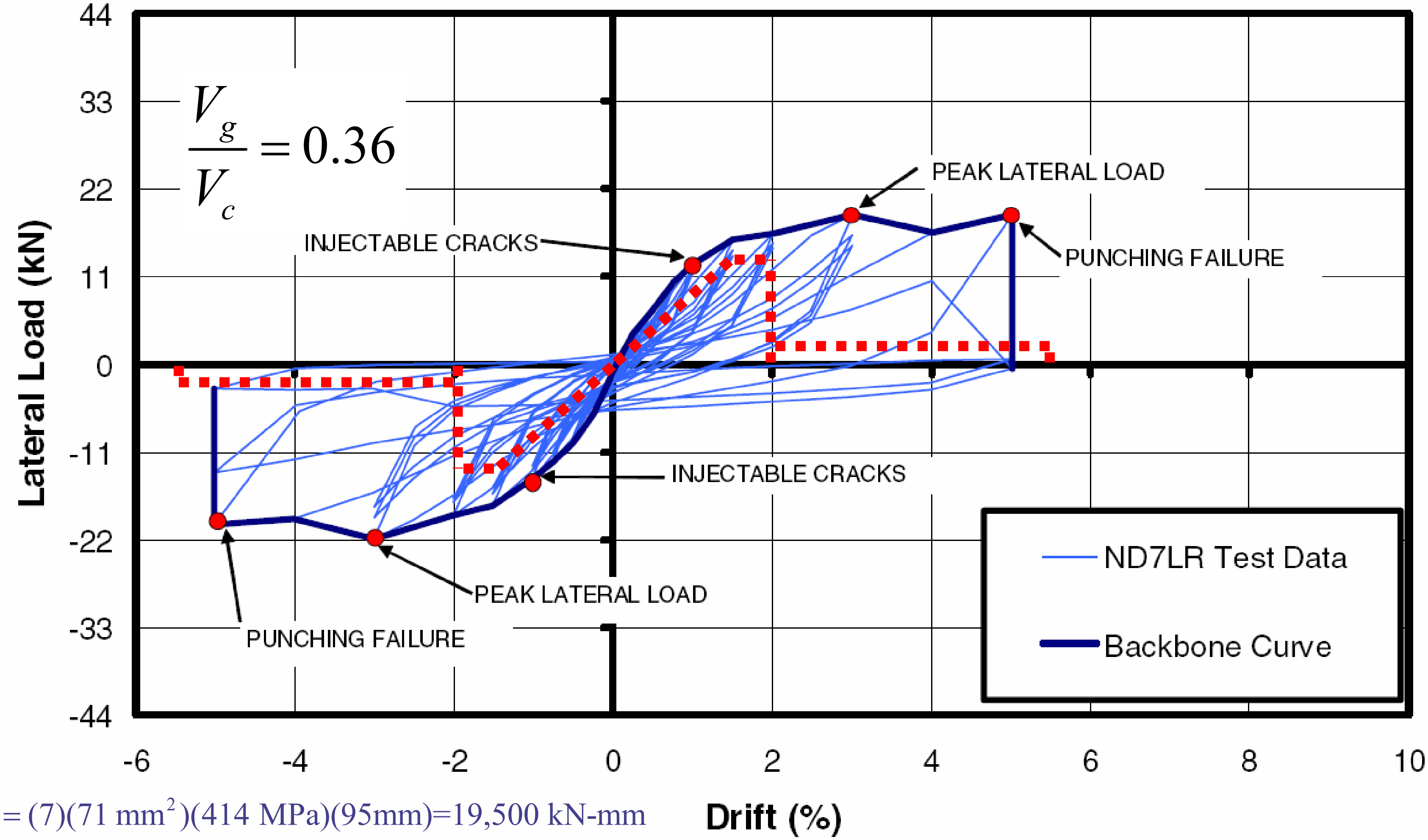
Figure 4-Slab Reinforcement for ND7LR.

10ft x 10 ft x 4.5”

2 Continuous bottom bars

Robertson & Johnson, 13WCEE, August 2004, Paper 143

Test Results



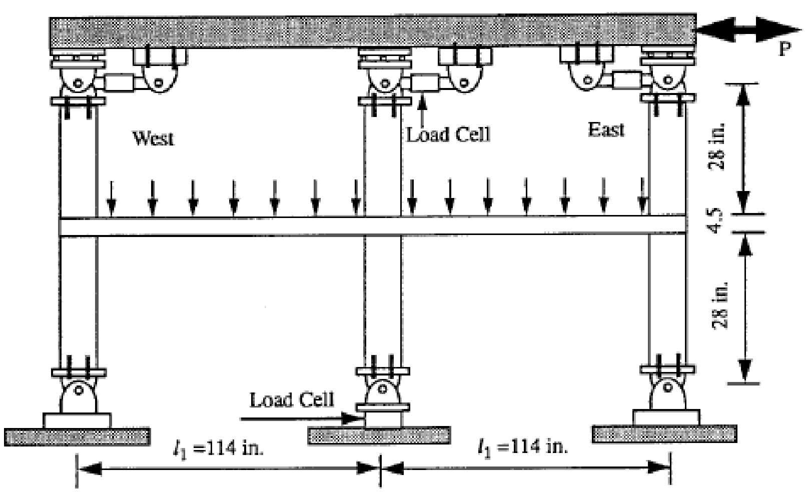
$M_n = (7)(71\ \text{mm}^2)(414\ \text{MPa})(95\text{mm}) = 19{,}500\ \text{kN-mm}$

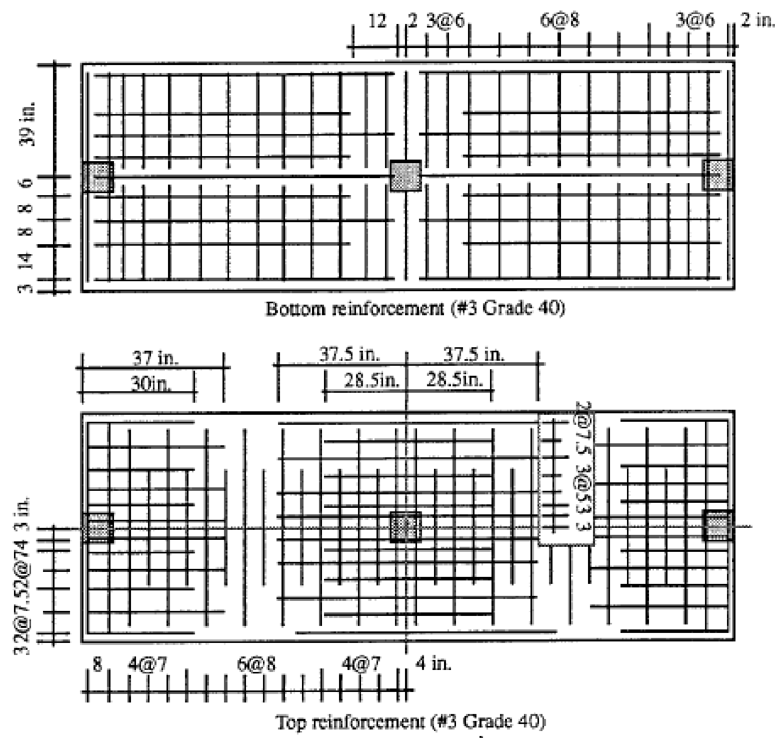
$\text{P}_\text{n} = 19{,}500\ \text{kN-mm}/1524\text{mm} = 13\ \text{kN}$

$\theta_e = 0.015; \quad \theta_\text{p} = 0.02(4/20) = 0.004$

Subassembly Test

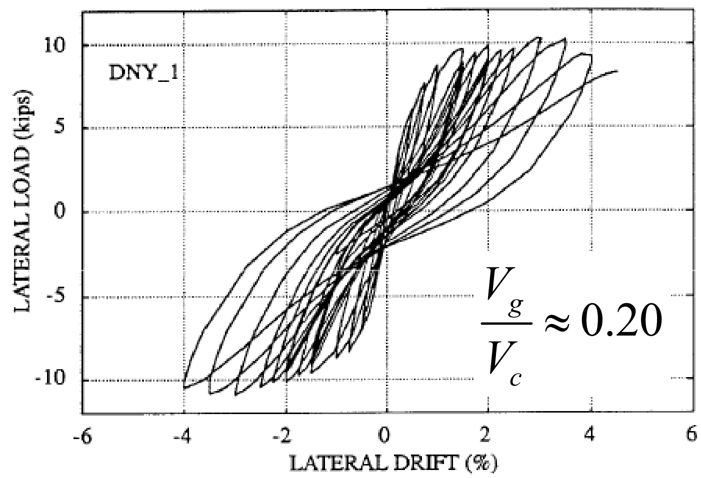
- Test specimens ~1/2 scale
- 4 specimens
  - DNY_1, DNY_2, DNY_3, DNY_4 (spandrel beam)
  - Bent-up (1,2,4), Straight (3)
  - $\dfrac{V_g}{V_c} = 0.2, 0.3, 0.24, 0.28$

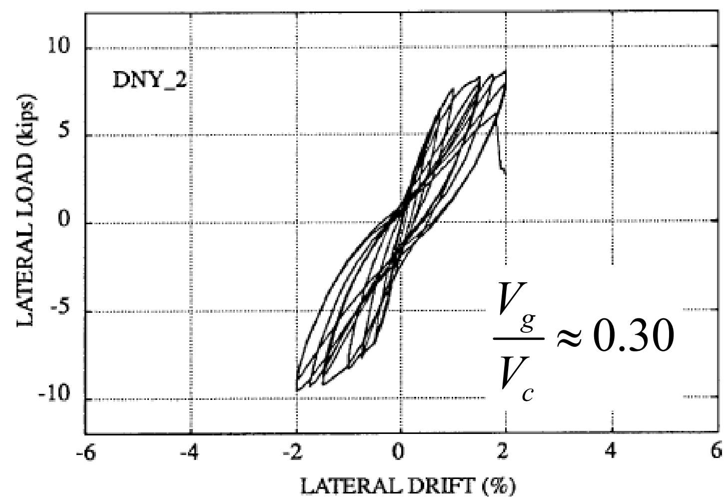


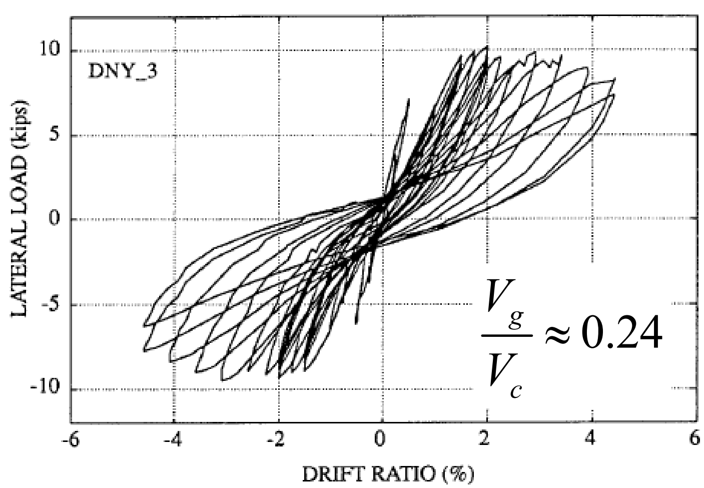


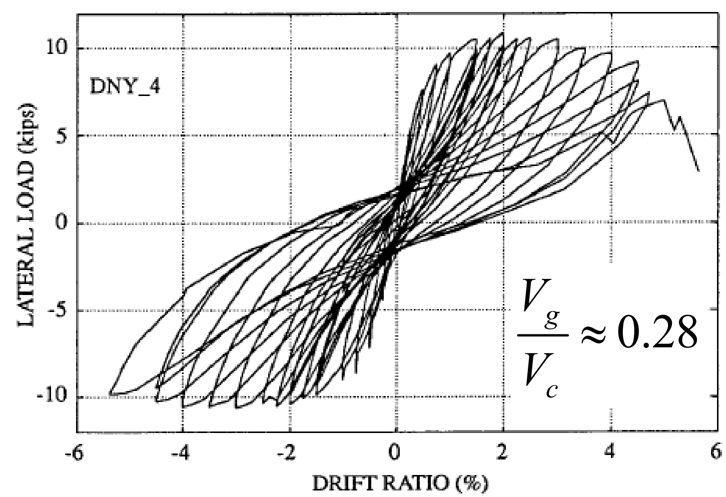
Durrani, Du, Luo, ACI SJ, July-Aug. 1995

Test Results









Spandrel beam

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Test Results - Summary

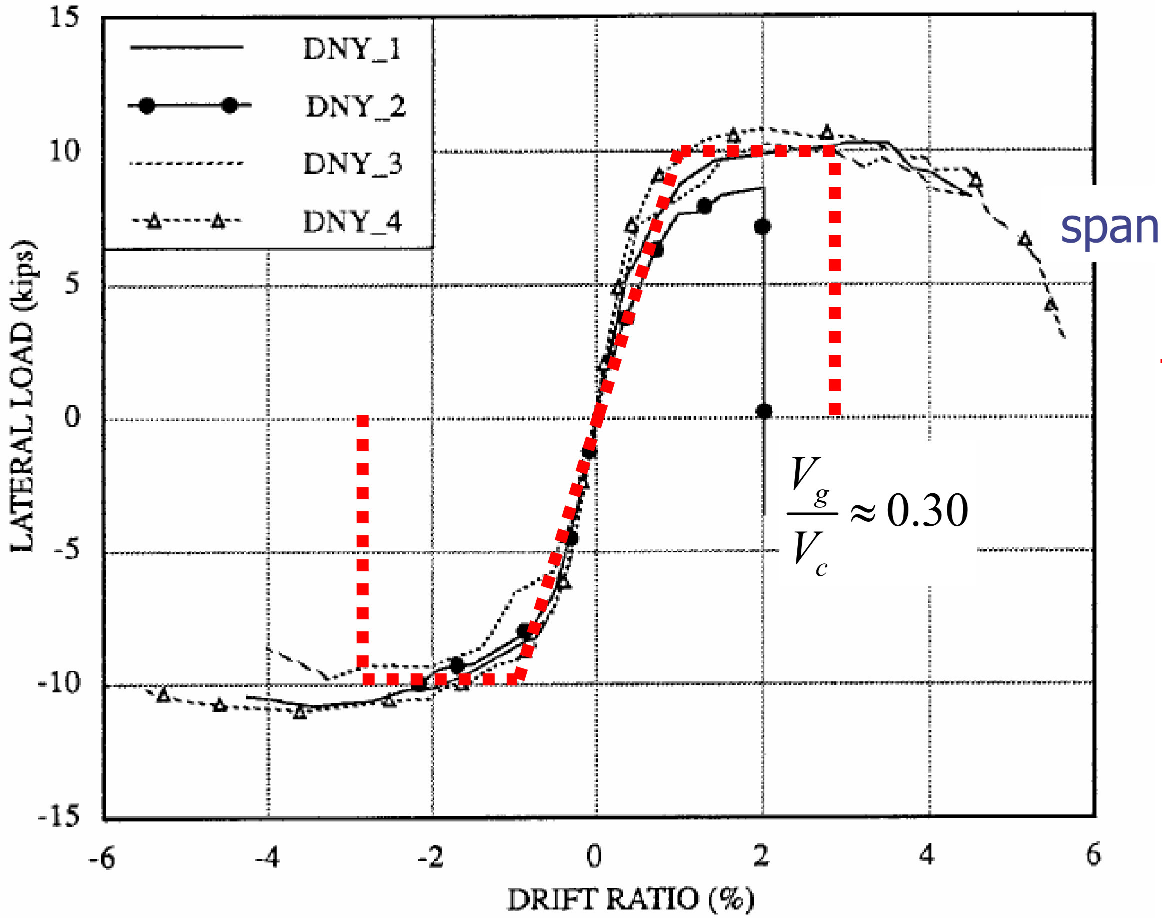

Fig. 5—Envelopes of load-drift hysteresis loops for subassemblies

Backbone relation:

P = 10 kip (arbitrary)

$\theta_e \approx 0.01 \qquad \theta_p = 0.02$

Straight bars vs
Bent up bars
- no difference
- except for collapse

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Presentation Overview

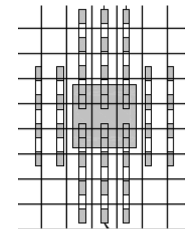
- Current Practice
  - Modeling & analysis
  - Connection design
  - Progressive collapse
  - Deformation compatibility
- Existing Construction
  - Background & observations
  - Modeling and Model Assessment
  - Backbone curves/Rehabilitation
- Shear reinforcement

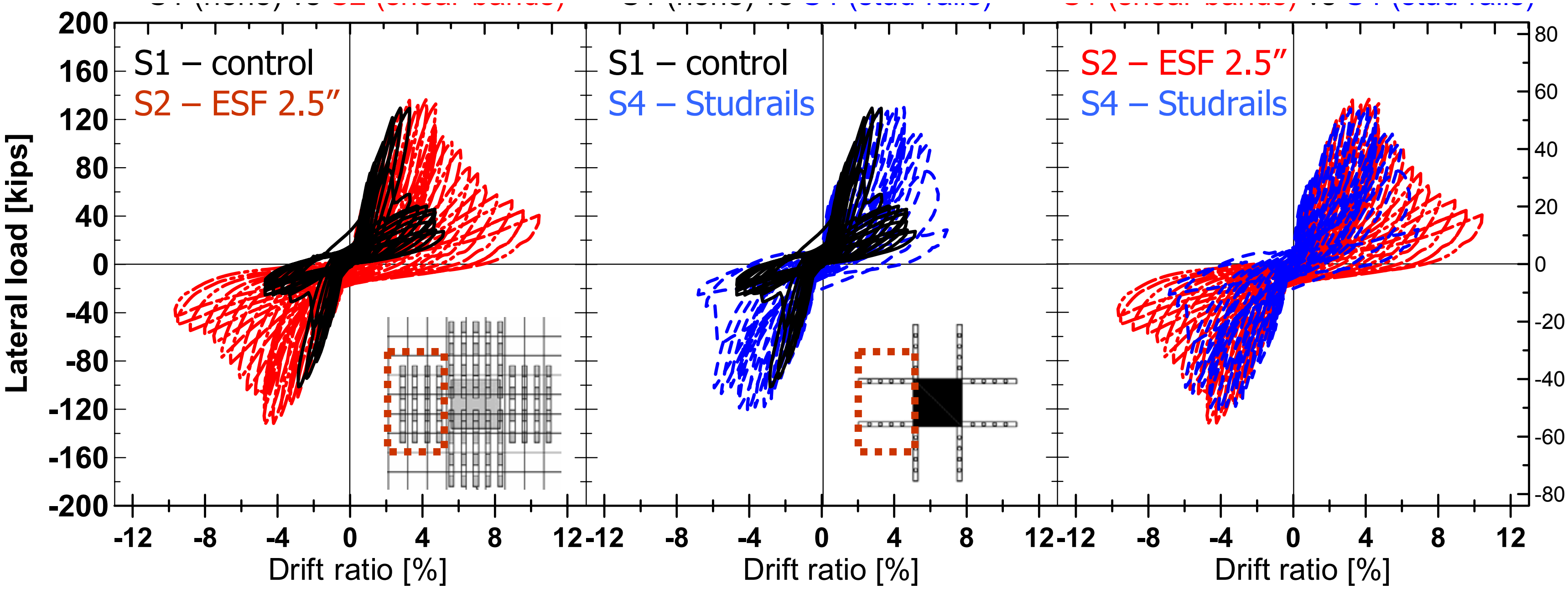
Recent Tests – ERICO Fortress Steel

0.6 Scale model tests (6” thick slab) – Smith Emery

Preliminary Results – ERICO Steel Fortress

- Fortress steel appears to be very effective in improving the punching resistance
- Appear as effective as stud-rails





A_v = 1.44 in^2 f_y =72.5 ksi A_v = 0.88 in^2 f_y =60 ksi

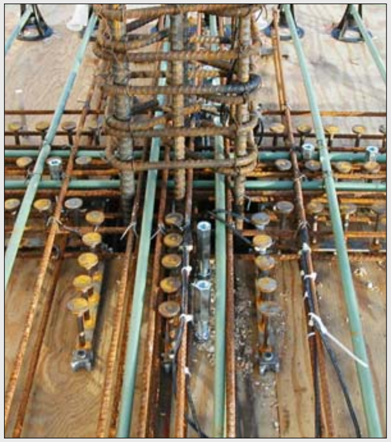
Summary

- Modeling
  - Effective beam width model
  - Connection behavior – Limit state model
- Backbone curves - RC
  - Conservative – In general
  - Review allowable plastic rotation for low gravity stress ratios < 0.2, mean - σ
  - Potential to increase plastic rotation for low reinforcement ratios
  - Remove residual capacity for RC connections

Summary

- Backbone curves - PT
  - Conservative
  - Increase plastic rotation from 0.02 (RC) to 0.03 at gravity shear ratio of 0.2
  - Review higher gravity shear ratios – allowable plastic rotation of 0.01 at a gravity shear ratio of 0.5
  - Allow residual capacity of 0.2 up to drifts of about 5% where one strand pass within the column cage in both directions.

Slab – Column Frames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