Lightly-Reinforced Wall Segments

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with contributions from
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Presentation Overview

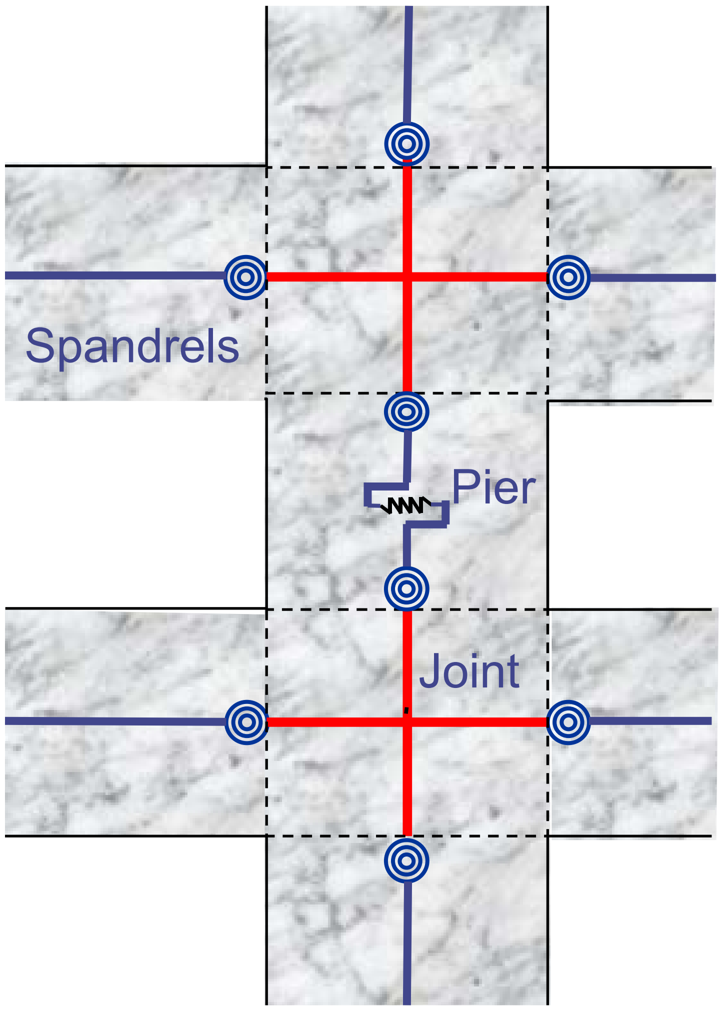
- FEMA 356 Requirements
- P-M-V Modeling
- Preliminary test results
- Axial load issues

Modified Beam - Column Model

- Use of modified beam-column element with added shear spring for both horizontal and vertical wall segments

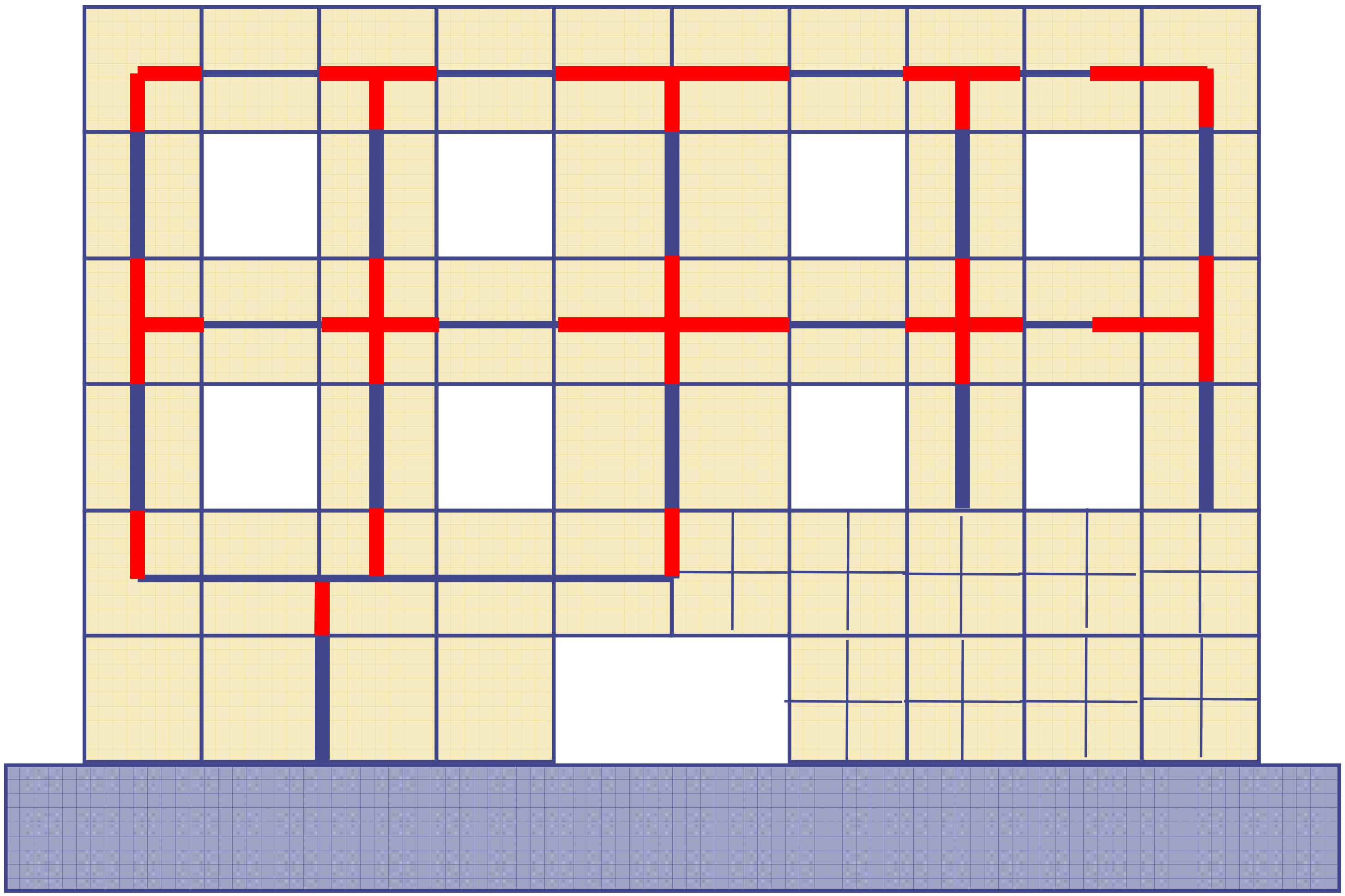
- Fiber model or general wall model with nonlinear shear backbone curve (uncoupled flexure/shear)

- If plastic hinge model – $EI_{effective}$ might be less than $0.5EI_g$ lightly-reinforced wall segments



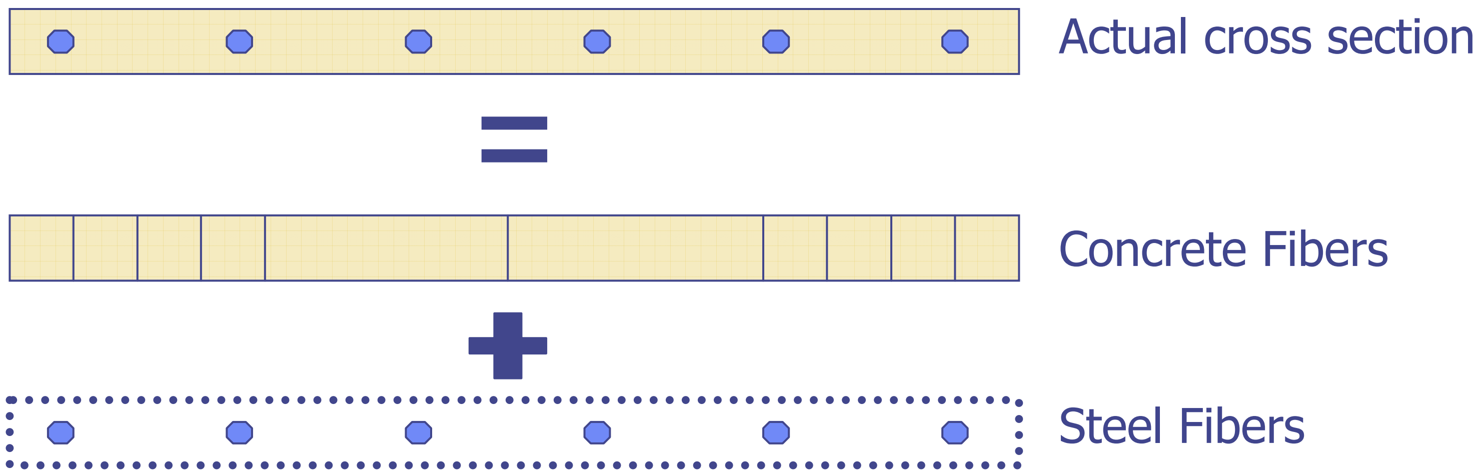
Modeling Approaches

- Frame and General Wall Models



P-M (flexural) Strength Provisions

- P_n- M_n for ε_c =0.003
- Fiber model or general wall model

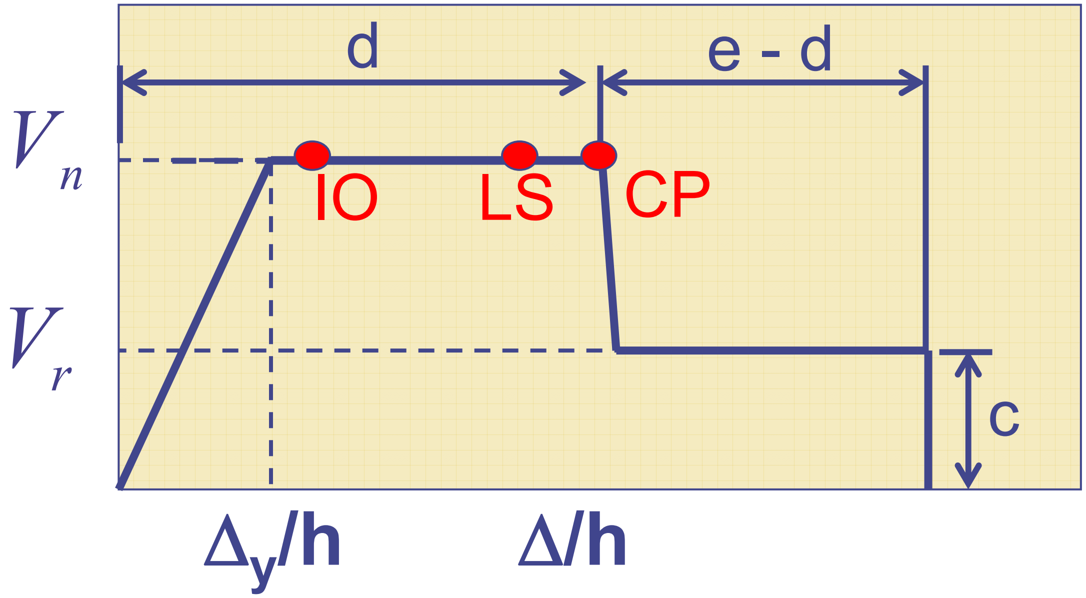


- Typically use a more refined mesh where yielding is anticipated
- However, in this case, where nonlinear shear behavior is anticipated, use enough elements to capture moment gradient.
- Nonlinear “backbone” relations (force – displacement) relations are commonly used to capture the shear behavior.

FEMA Modeling Parameters

- FEMA 356 Tables 6-19: Wall segments

Modeling Parameters, Drift %			Acceptable Drift %		
			Performance Level		
d	e	c	Immediate Occupancy	Life Safety	Collapse Prevention
0.75	2.0	0.40	0.40	0.60	0.75



Shear Strength Provisions

- V_n per ACI 318-99,02,05 Equation 21-7

$$V_n = A_{cv}\left[\alpha_c \sqrt{f_c^{'}} + \rho_t f_y\right]$$

$$\alpha_c = 3.0 \quad \text{for } h_w / l_w \leq 1.5$$

$$\alpha_c = 2.0 \quad \text{for } h_w / l_w \geq 2.0$$

Linear interpolation allowed for intermediate values

If axial load exceeds $0.15A_g f'_c$; then force controlled

ρ need not be taken less than 0.15% (Wood, ACI SJ, 1990)

Shear Strength Database

		# of Curtains	
Researcher	**Protocol**	**2**	**1**
Sugano (1973)	Monotonic	7	1
Barda	Cyclic	6	0
Cardenas	Monotonic	0	2
Hidalgo (2002)	Cyclic	0	7
Hirosawa (1975)	Cyclic	1	0
Aoya	Cyclic*	5	0

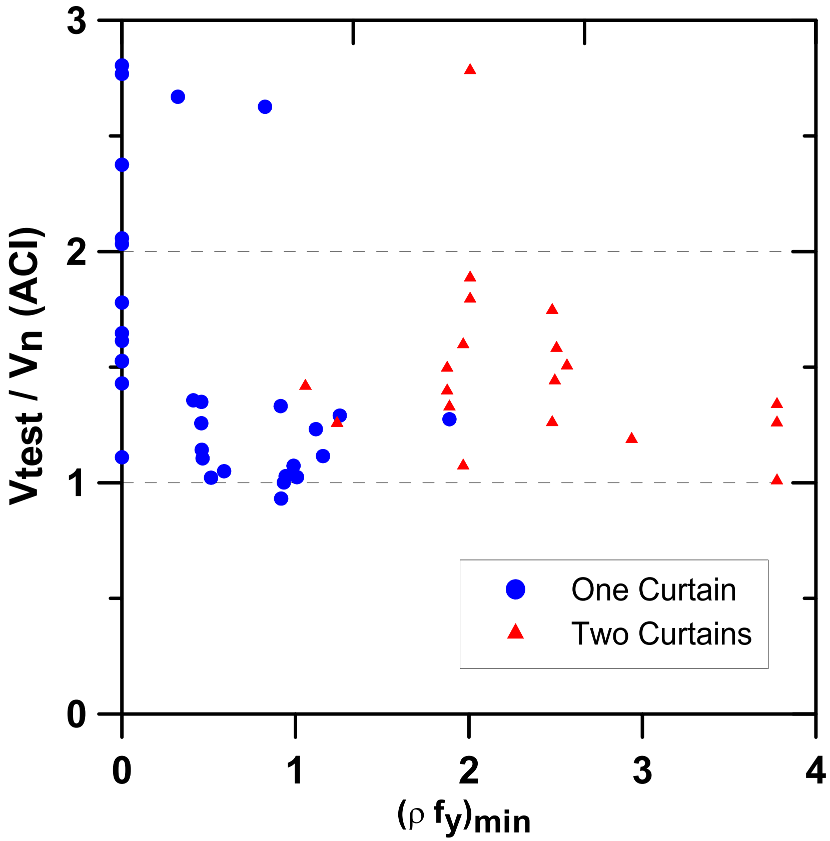
* One full cycle, then monotonic to failure

$t_w = 3.15$ to 6.3 inches $\quad 0.25\% \le \rho \le 0.67\%$

$f_c' = 3.3$ ksi, $\sigma = 1$ ksi $\quad f_y = 64$ ksi, $\sigma = 14$ ksi

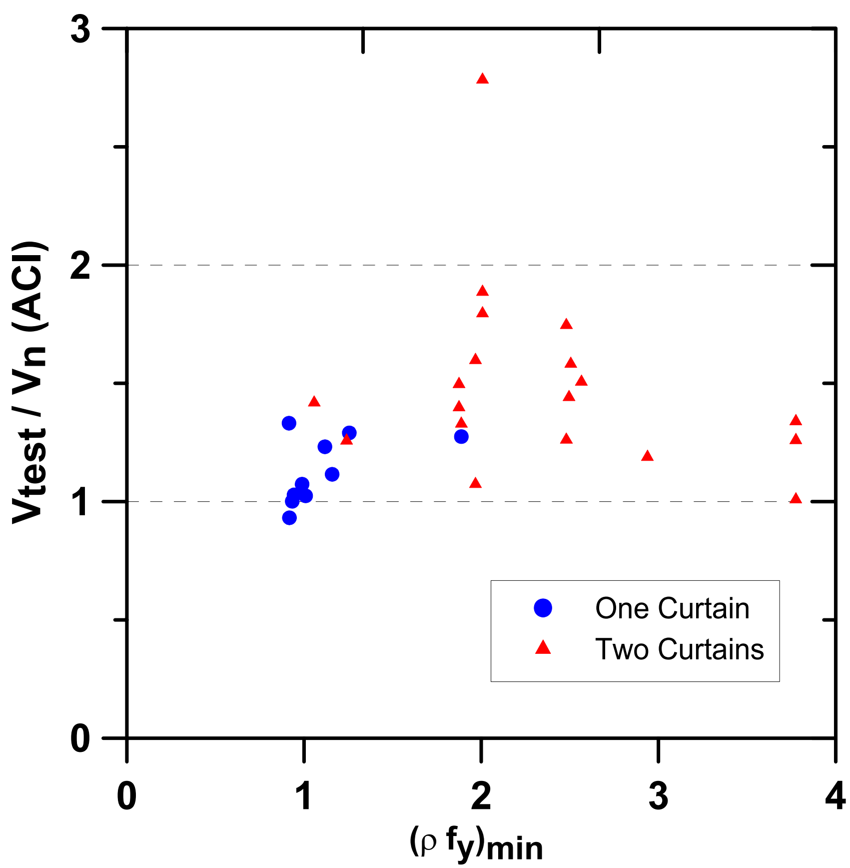
$(7) < 0.12 A_g f_c'$, $\quad (1) = 0.15 A_g f_c'$, $\quad (1) = 0.22 A_g f_c'$

Shear Strength – Expanded Database



- ρ need not be taken less than 0.15% (Wood, 1990)
- Shear strength is relatively insensitive to the web reinforcement
- For relatively thin walls, use of one or two curtains of web reinforcement, strength is similar
- Results similar for monotonic and cyclic tests

Shear Strength – Restricted Database



Tests with at least minimum reinforcement

- ρ need not be taken less than 0.15% (Wood, 1990)
- Shear strength is relatively insensitive to the web reinforcement
- For relatively thin walls, use of one or two curtains of web reinforcement, strength is similar
- Results similar for monotonic and cyclic tests

FEMA Modeling Parameters

- FEMA 356 Tables 6-19: Wall segments

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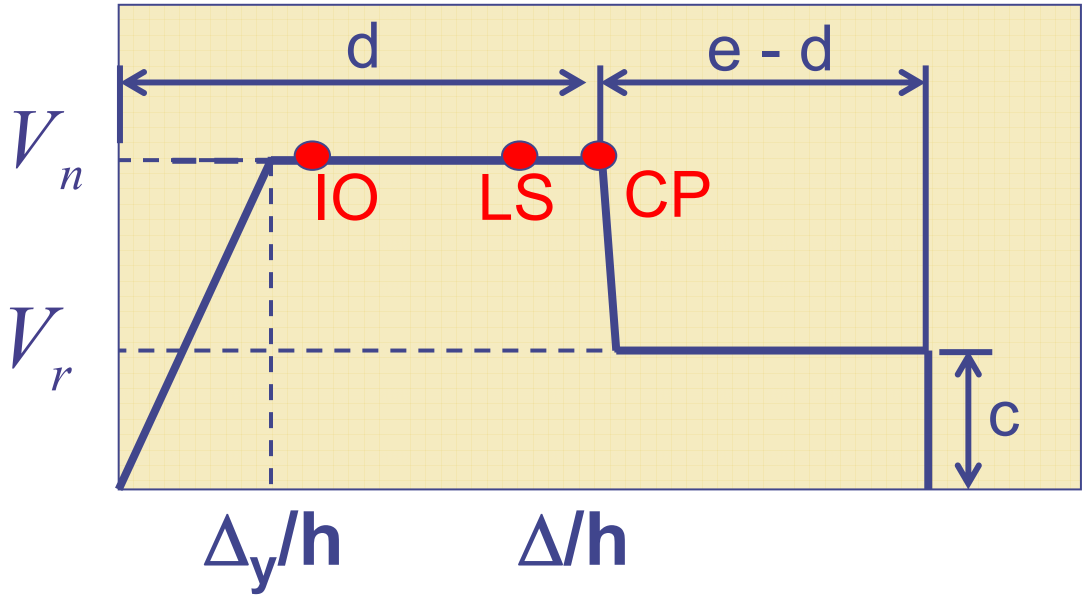


Shear Force-Deformation Behavior

- Shear backbone curve

Strength of materials

$$\tau = G\gamma \quad \sigma = \mathrm{E}\varepsilon$$

$$\Delta_y = \left(\frac{V_y}{\left(G_c = 0.4E_c\right) A} \right) h$$

$$V_y = V_n \quad \text{(i.e., no hardening)}$$

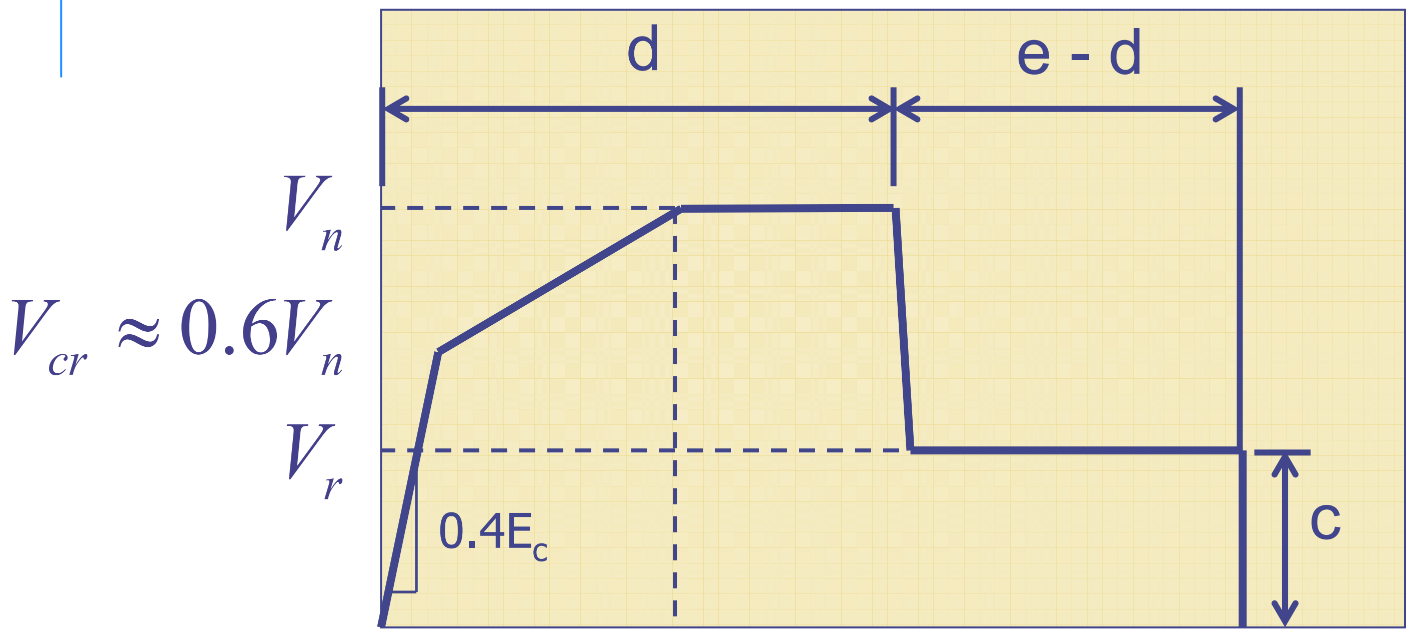
$$G_c = E_c \left(\frac{1}{1+2\nu} \right) = 0.4\mathrm{E_c}$$

$$V_{cr} = f_t \left[1 + \frac{P / A_g}{f_t} \right] \approx 0.6 V_n$$

$$f_t = \left(4 \text{ to } 6\right) \sqrt{f_c^{'}}$$ Sozen & Moehle, 1993 EPRI Report

Revised Backbone Relation

- Based on prior tests (limited database):
  - 5WCEE, Rome, 1973, pp. 1157-1166
  - 9WCEE, Tokyo, 1988, pp. IV 517-522
  - Hidalgo et al, 2002, EERI Spectra
  - Hirosawa, 1975, Japanese Report



$\Delta/h = 0.004$ to 0.005

Observations

- Limited test data
  - Stiffness and Deformation capacity – specimens tend to be stiff and strong, test control is challenging and reported stiffness and deformation values may be suspect
  - Residual strength – most tests not continued beyond modest strength degradation (~20%)
  - One row in FEMA table 6-19
- Nominal Strength
  - Test results indicated nominal strength in the range of 100 to 200% of the ACI value

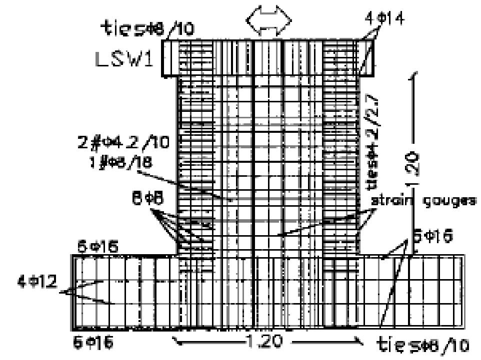
New Data Since ~1995

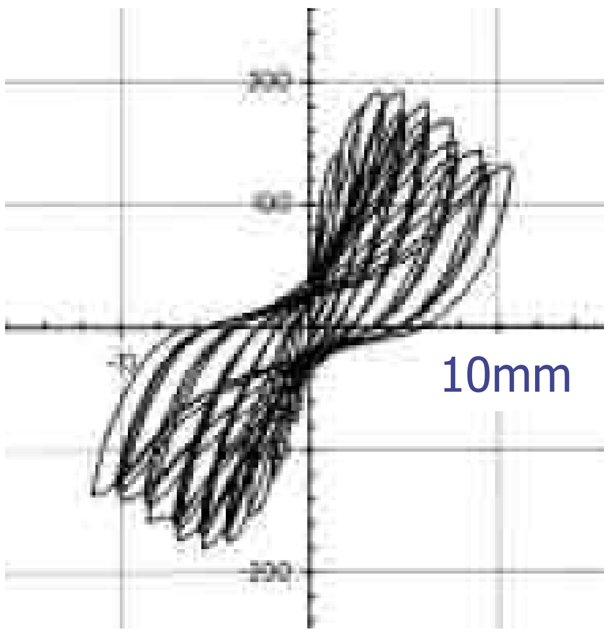
- Salonikios, Thomas N.; et al. (1999)
  - 11 tests on cantilever walls with axial load of 0.0 and $0.07A_gf'_c$
  - Aspect ratios of 1.0 (1.2m tall) and 1.5 (1.8m tall)
  - Cross section: 1.2m x 100mm (4 ft x 4")
  - 4 tests with diagonal web bars for sliding – Eurocode 8 requires 50%
  - Reasonably-well detailed (Eurocode 8)
- Hidalgo, Pedro A.; Ledezma, Christian A.; Jordan, Rodrigo M., (2002)
  - 26 tests for reverse bending (zero moment at mid-height), no axial load
  - M/Vl ratios: 1.0(3), 0.69(9), 0.5(7), 0.35(7): 1m x 2m tall; 1.5m x 1.05m tall
  - Cross section: 80 to 120 mm (3.15" to 4.72") by 1.0m to 1.7m (40" to 67")
  - Light web reinforcement: 0%, 0.125%, 0.25%, 0.375% (only one)
- Greifenhagen, H.; Lestuzzi, P, (2005)
  - 4 tests on cantilever walls with axial load (0.027, 0.027, 0.043, $0.094A_gf'_c$)
  - M/Vl ratio: 0.69
  - Cross section: 1 m x 100 mm (40" x 4")
  - Light web reinforcement: 0.3%, 0% (one case with no horizontal web bars)
- Massone, Orakcal, Wallace (2005, 2006)

Salonikious et al. 1999

Aspect ratio 1.0 tests

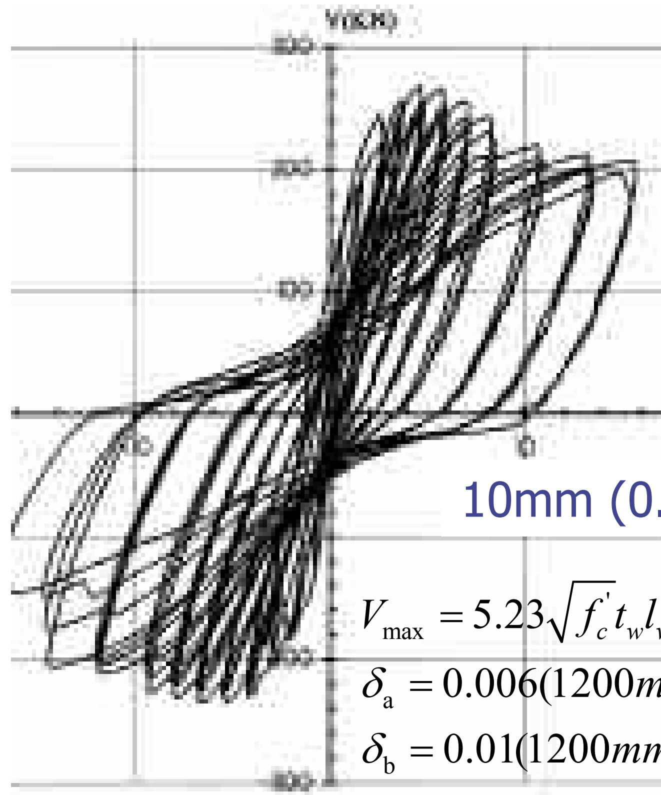
V_n = 342 kN per ACI 318
Flexural yielding ($F_{max,\,1} = 1.5 * F_{max,\,1.5}$)
Sliding failure





10mm

LSW2: 0.28% H & V and P=0



10mm (0.0083)

$$V_{max} = 5.23\sqrt{f_c'}\,t_w l_w \quad \text{non-conforming}$$

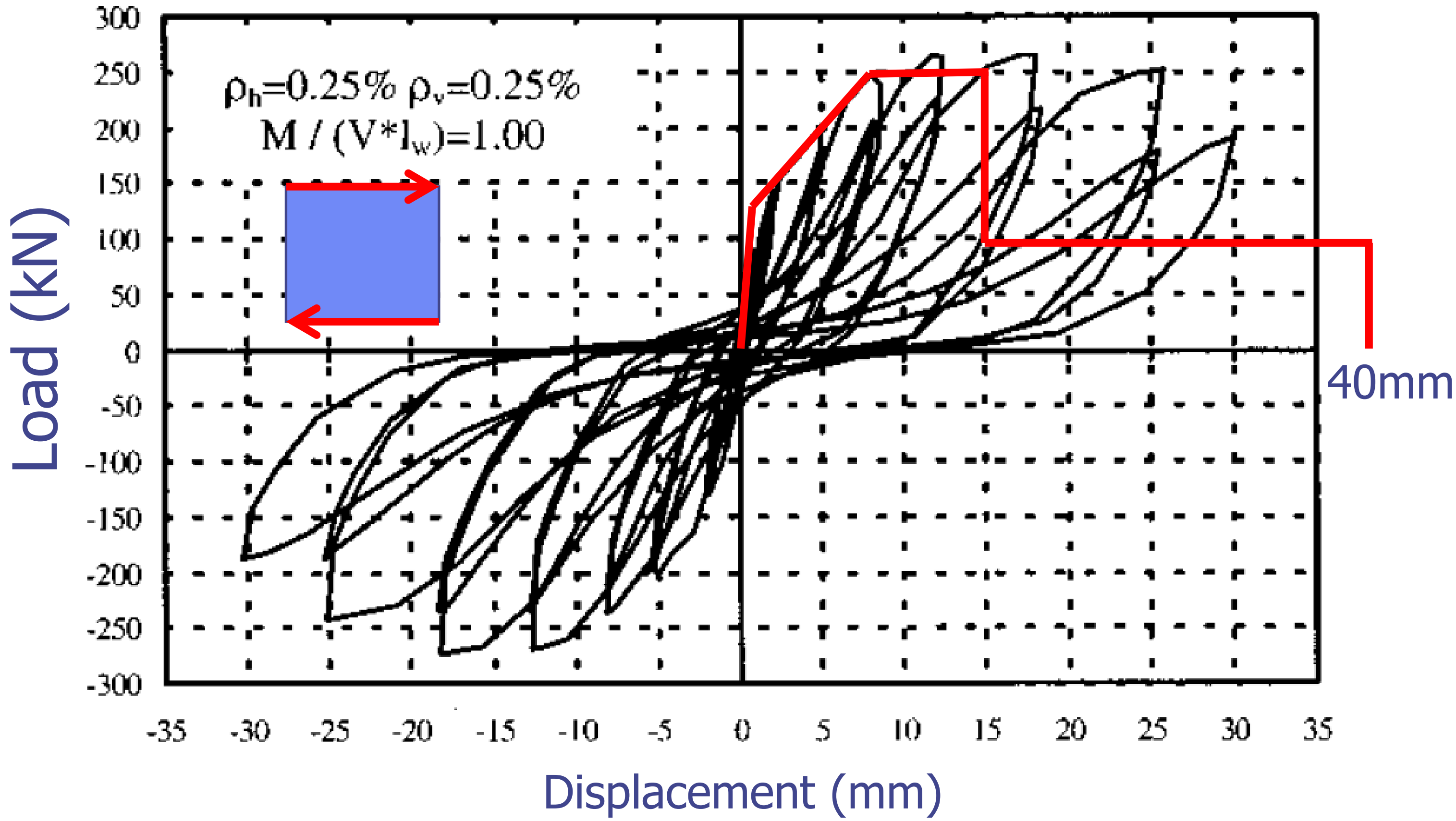
$$\delta_a = 0.006(1200mm) = 7.2 \text{ mm}$$

$$\delta_b = 0.01(1200mm) = 12.0 \text{ mm}$$

LSW3: 0.28% H & V and P=0.07Agf'c

Hidalgo et al. 2002

M/Vl$_w$ = 1.0 Specimen #2



$$V_n = 5.74\sqrt{f_c'}\, t_w l_w = 57 \text{ kips (253 kN)}$$

$$V_{crack} = 0.5 V_n = 30 \text{ kips (133 kN)}$$

$$\delta_y = \frac{V_n h_w}{GA} = \frac{(56.86 \text{ kips})(78.74")}{0.4(3040 \text{ ksi})(186 \text{ in}^2)} = 0.02" \ (0.5 \text{ mm})$$

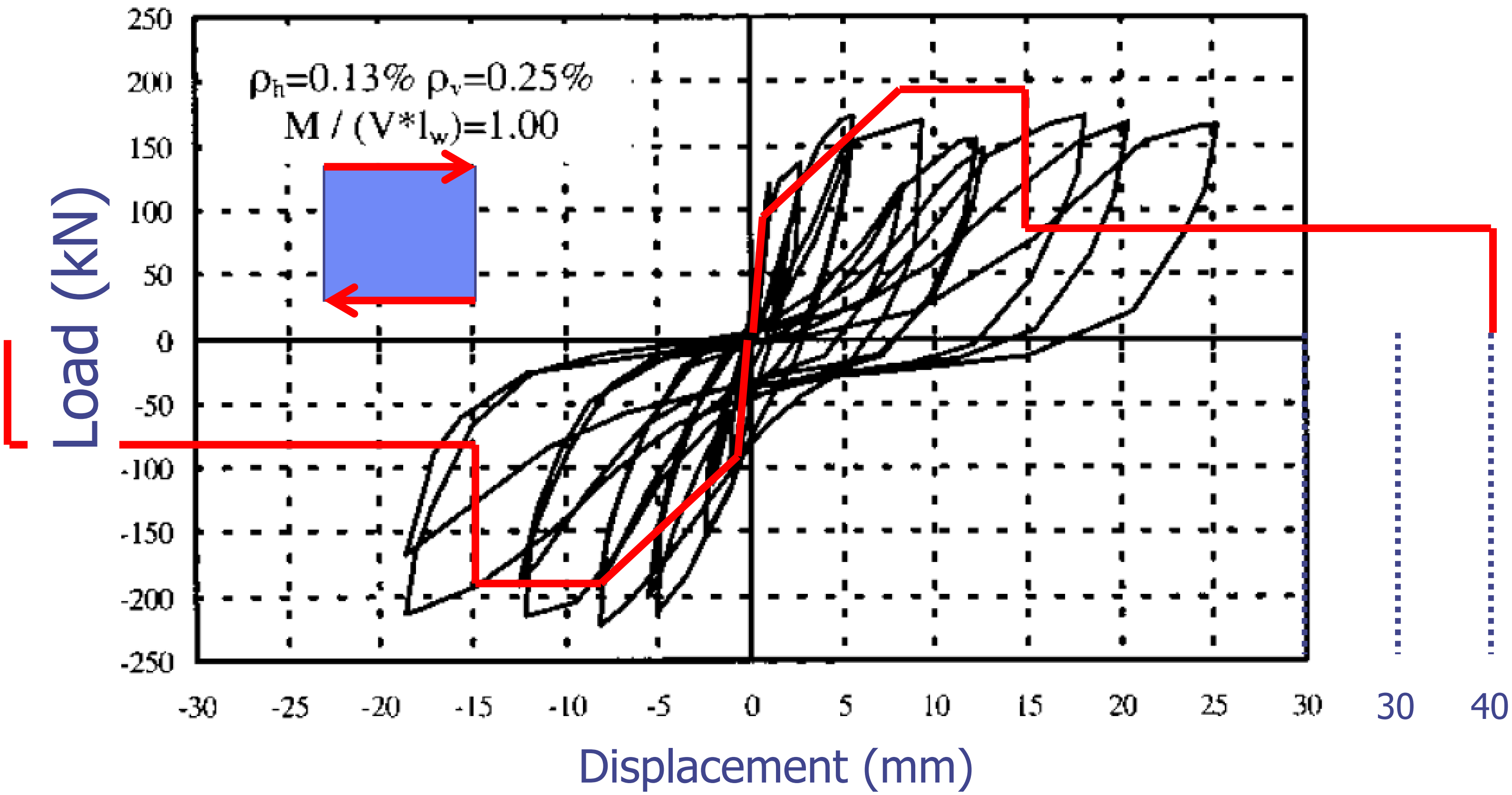
$$\delta_y = 0.004 h_w = .004(78.74") = 0.31" \ (8 \text{ mm})$$

$$\delta_d = 0.0075(2000mm) = 15 \text{ mm}$$

$$\delta_e = 0.02(2000mm) = 40 \text{ mm}$$

Hidalgo et al. 2002

M/Vl$_w$ = 1.0 Specimen #1



$V_n = 4.4\sqrt{f_c'}\,t_w l_w$=43 kips (193 kN) $f_c' = 2.81$ ksi

$V_{crack} = 0.5V_n = 22$ kips (98 kN)

$\delta_y = \dfrac{V_n h_w}{GA} = \dfrac{(43\text{ kips})(78.74")}{0.4(3020\text{ ksi})(186\text{ in}^2)} = 0.015"$ (0.4 mm)

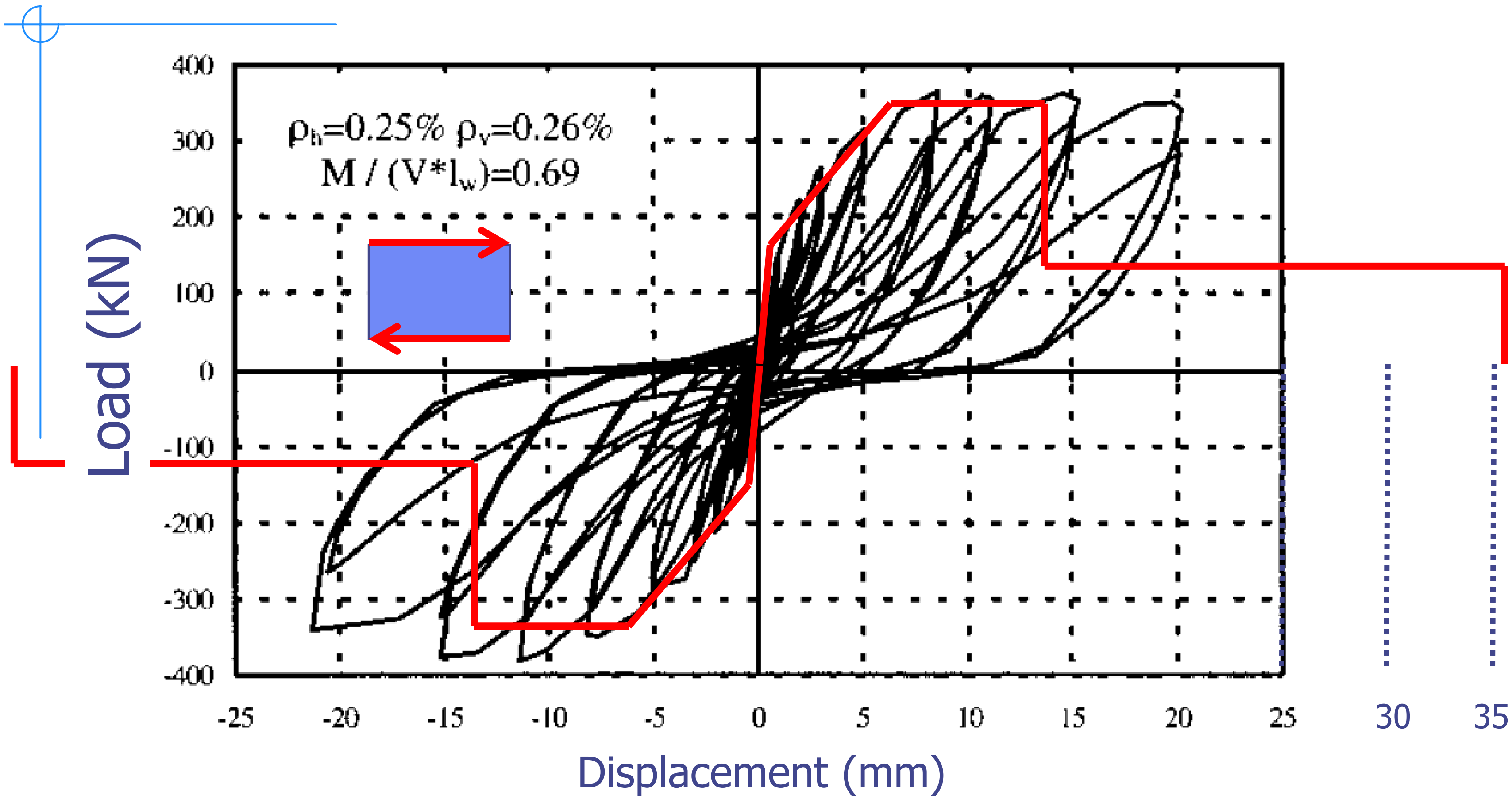
$\delta_y = 0.004h_w = .004(78.74") = 0.31"$ (8 mm)

$\delta_d = 0.0075(2000mm) = 15$ mm

$\delta_e = 0.02(2000mm) =$ 40 mm

Hidalgo et al. 2002

M/Vl_w = 0.69   Specimen #8



$$V_n = 6.6\sqrt{f_c'}\,t_w l_w = 76 \text{ kips } (337 \text{ kN})$$

$$V_{crack} = 0.5V_n = 38 \text{ kips } (169 \text{ kN})$$

$$\delta_y = \frac{V_n h_w}{GA} = \frac{(76 \text{ kips})(70.9")}{0.4(2720 \text{ ksi})(242 \text{ in}^2)} = 0.0205" \ (0.52 \text{ mm})$$

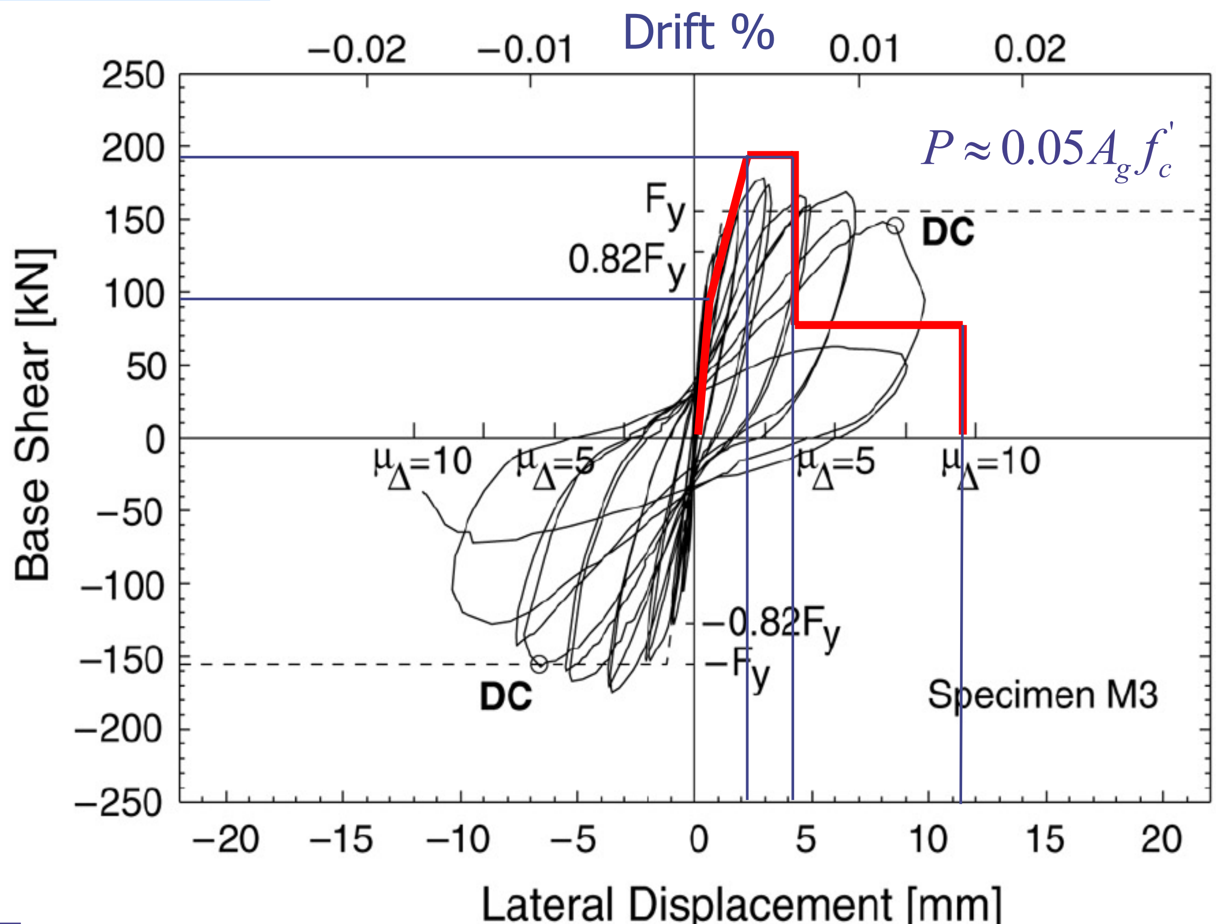
$$\delta_y = 0.004h_w = .004(70.9") = 0.284" \ (7.2 \text{ mm})$$

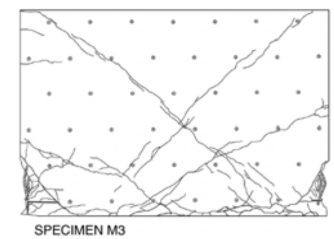
$$\delta_d = 0.0075(1800 \text{ mm}) = 13.5 \text{ mm}$$

$$\delta_e = 0.02(1800 \text{ mm}) = \ 36 \text{ mm}$$

Greifenhagen & Lestuzzi 2005

$M/Vl_w = 0.69$ Specimen M3





Diagonal tension
Sliding failure

$V_n = 7.06\sqrt{f_c'}\,t_w l_w$=42.6 kips (189 kN) f_c' =2915 psi

$V_{crack} = 0.5V_n = 21.3$ kips (95 kN)

$$\delta_y = \frac{V_n h_w}{GA} = \frac{(42.6\text{ kips})(22.24")}{0.4(3077\text{ ksi})(111.6\text{ in}^2)} = 0.0069"\ (0.175\text{ mm})$$

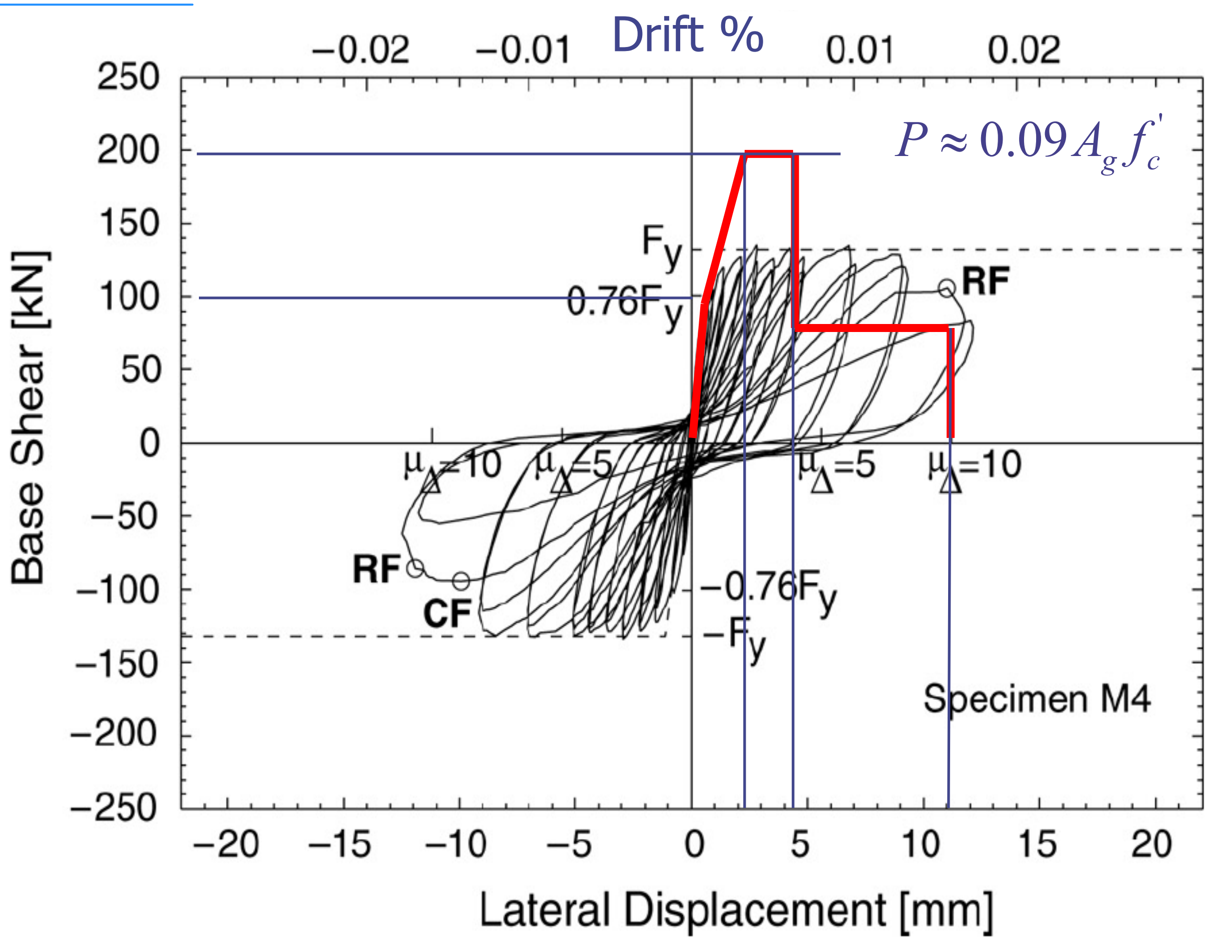
$\delta_y = 0.004h_w = .004(565\text{ mm}) = 2.26\text{ mm}$

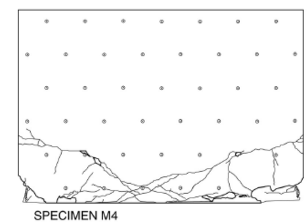
$\delta_d = 0.0075(565\text{ mm}) = 4.24\text{ mm}$

$\delta_e = 0.02(565\text{ mm}) = 11.3\text{ mm}$

Greifenhagen & Lestuzzi 2005

M/Vl_w = 0.69 Specimen M4





Sliding failure

$V_n = 6.7\sqrt{f_c'}t_w l_w$=44.4 kips (198 kN) f_c' =3539 psi

$V_{crack} = 0.5V_n = 22.2$ kips (99 kN)

$$\delta_y = \frac{V_n h_w}{GA} = \frac{(44.4\text{ kips})(22.24")}{0.4(3390\text{ ksi})(111.6\text{ in}^2)} = 0.0065"\ (0.166\text{ mm})$$

$\delta_y = 0.004h_w = .004(565\text{ mm}) = 2.26$ mm

$\delta_d = 0.0075(565\text{ mm}) = 4.24$ mm

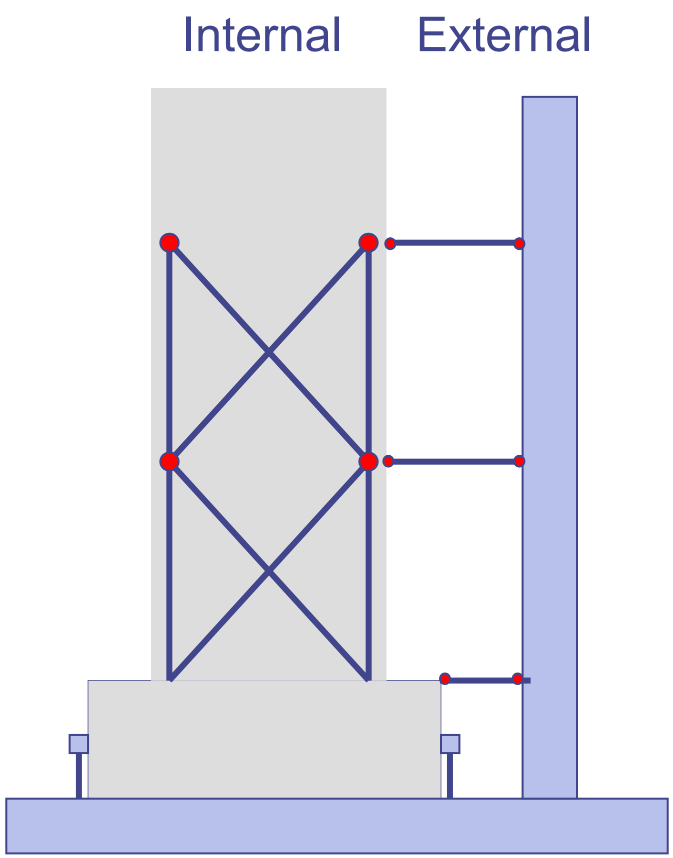
$\delta_e = 0.02(565\text{ mm}) =$ 11.3 mm

Presentation Overview

- FEMA 356 Requirements
- P-M-V Modeling
- Preliminary test results
- Axial load issues

Slender Wall Tests - Results

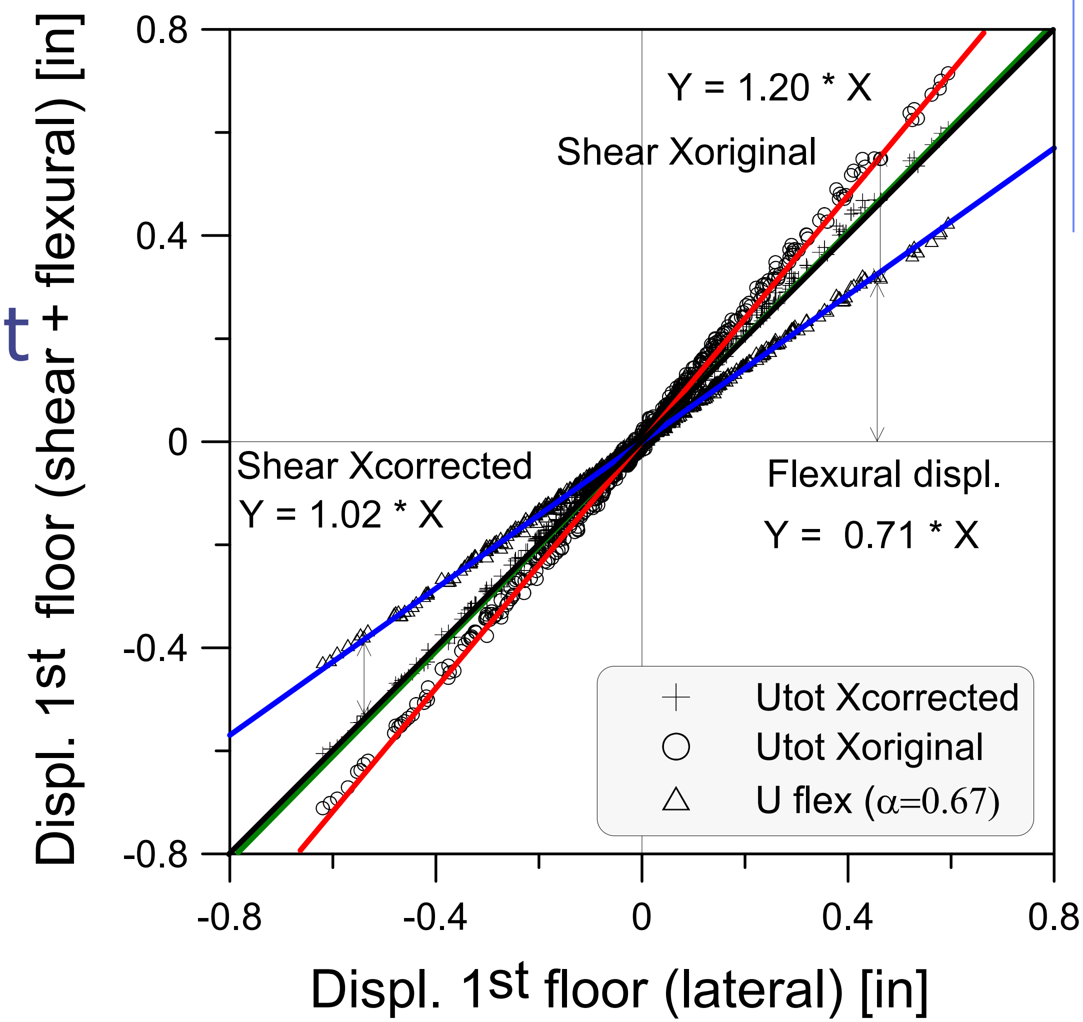
- External Instrumentation
  - Lateral displacement at different floor levels
- Internal Instrumentation
  - Shear deformation at different floor levels
  - Flexural deformation at different floor levels
- Uncouple deformations
  - Shear/Flexure
  - Assess data reliability



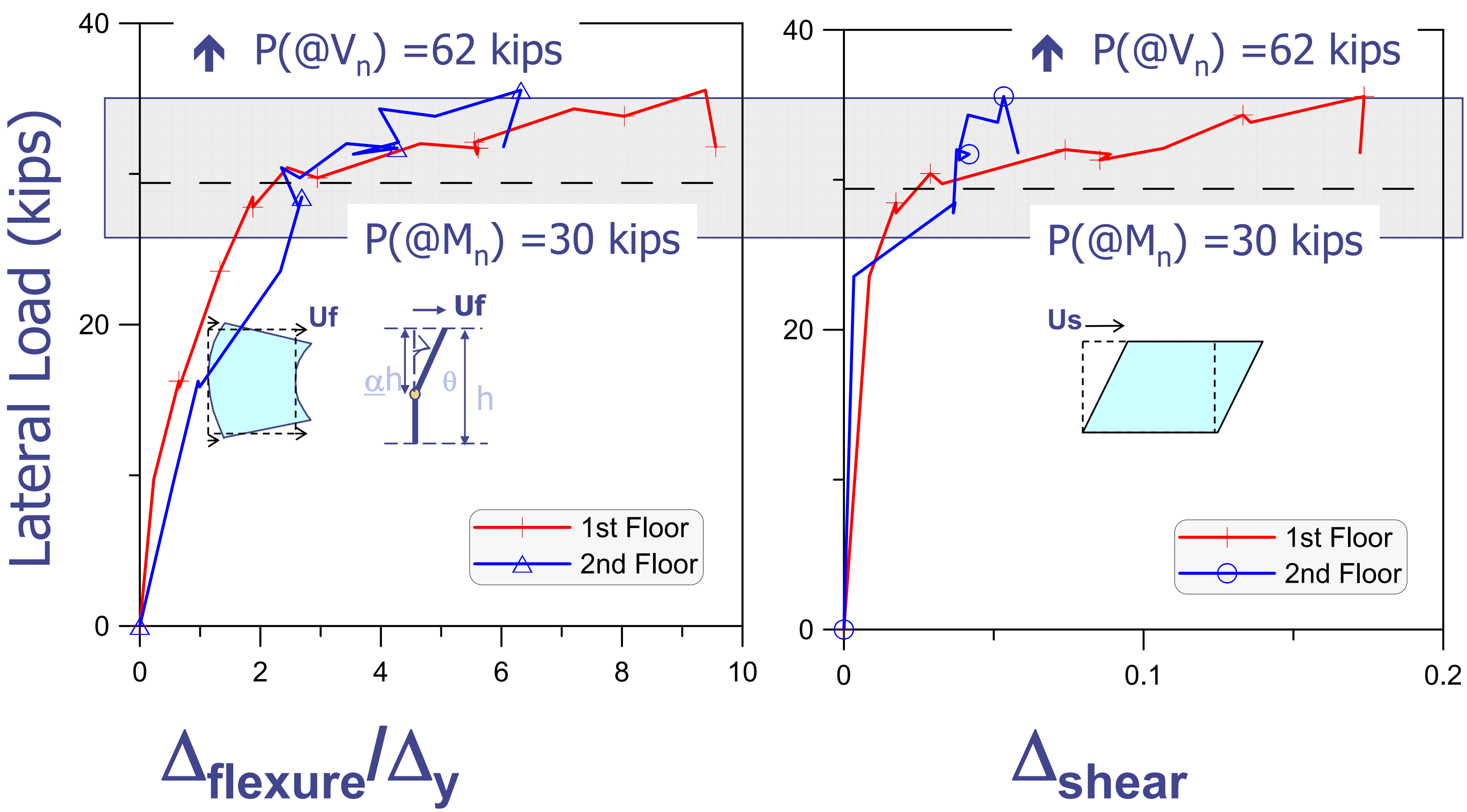
Wall base instrumentation

Tests Results: Observations

- Consistent and repeatable results
- Top displacement
  - Small shear contribution, about 5%
- 1st Story Displacement
  - 4-story walls
  - 30% shear contribution



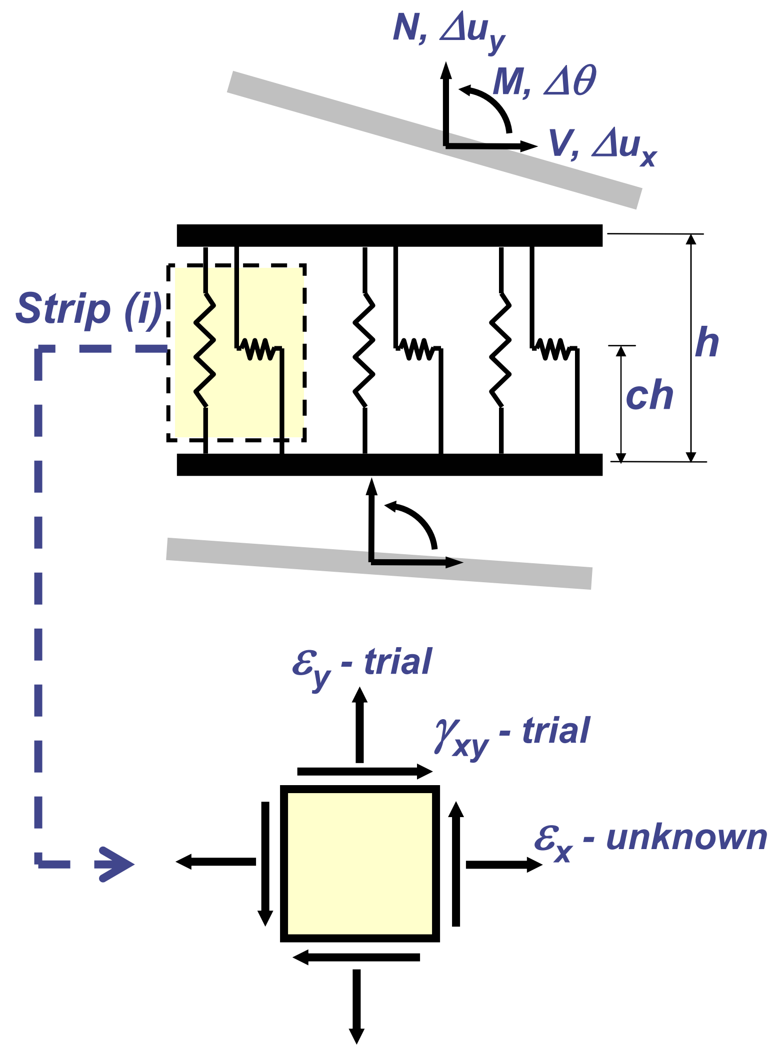
Test Results - Observations



Interaction between nonlinear flexure and shear deformations is evident even for relatively slender walls where $V_{max} \sim ½V_n$

Modeling P-M-V Interaction

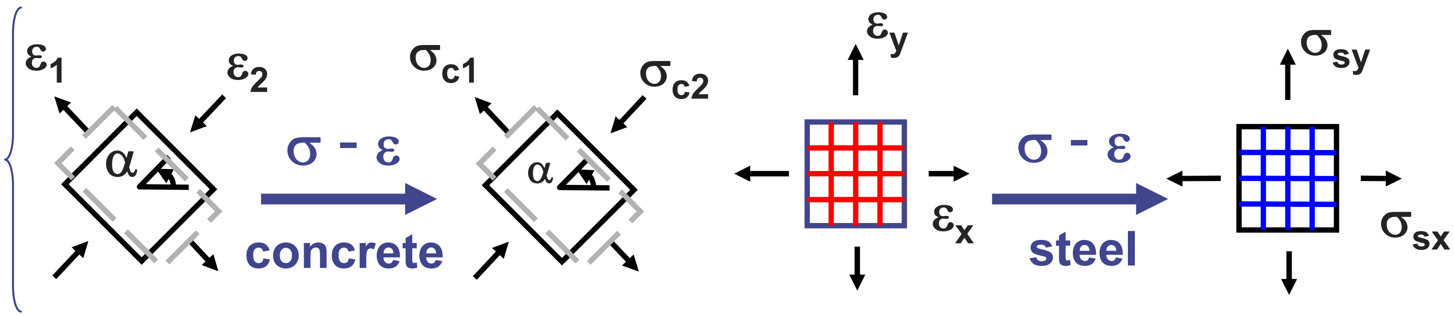
1. Modified MVLE model to incorporate shear – flexure interaction
2. Parallel pairs of "flexure" and "shear" fibers are used
3. Behavior of each set of springs described by a constitutive RC rotating-angle panel model (e.g., MCFT or RA-STM), that incorporates axial-shear interaction
4. Requires additional model iterations to establish equilibrium condition



Local Iteration Scheme

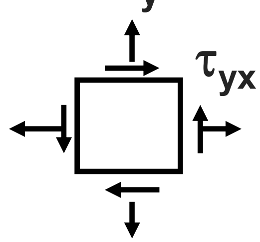
Assigning Iteration Variable

Constitutive Material Models



Horizontal (Transverse) Equilibrium

$$\sigma_y = \sigma_{cy} + \rho_y \sigma_{sy}$$



$$\sigma_x = \sigma_{cx} + \rho_x \sigma_{sx}$$

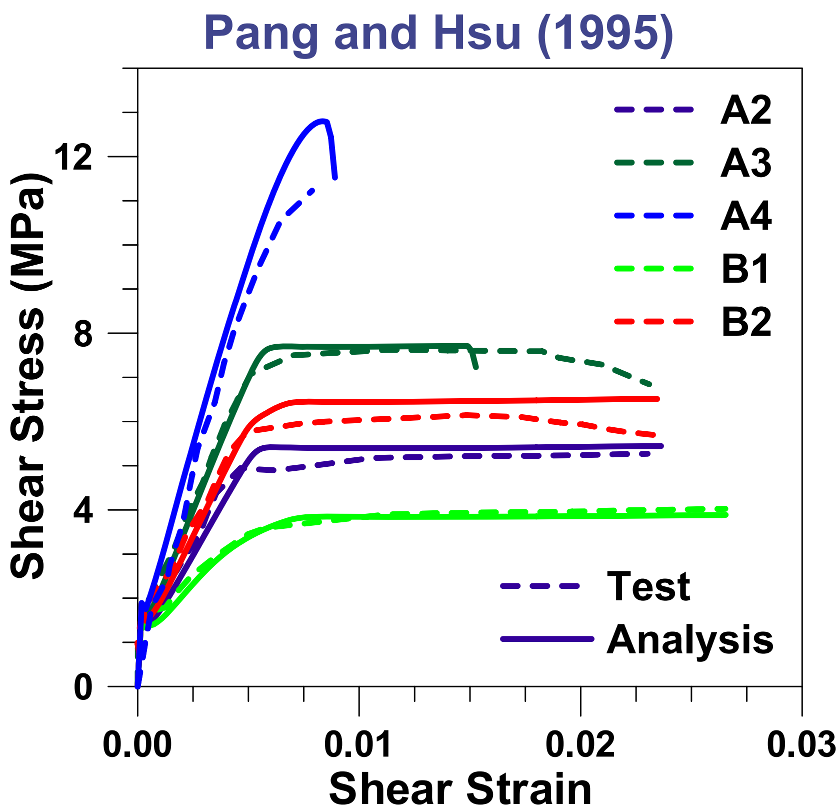
$$\sigma_x = 0$$

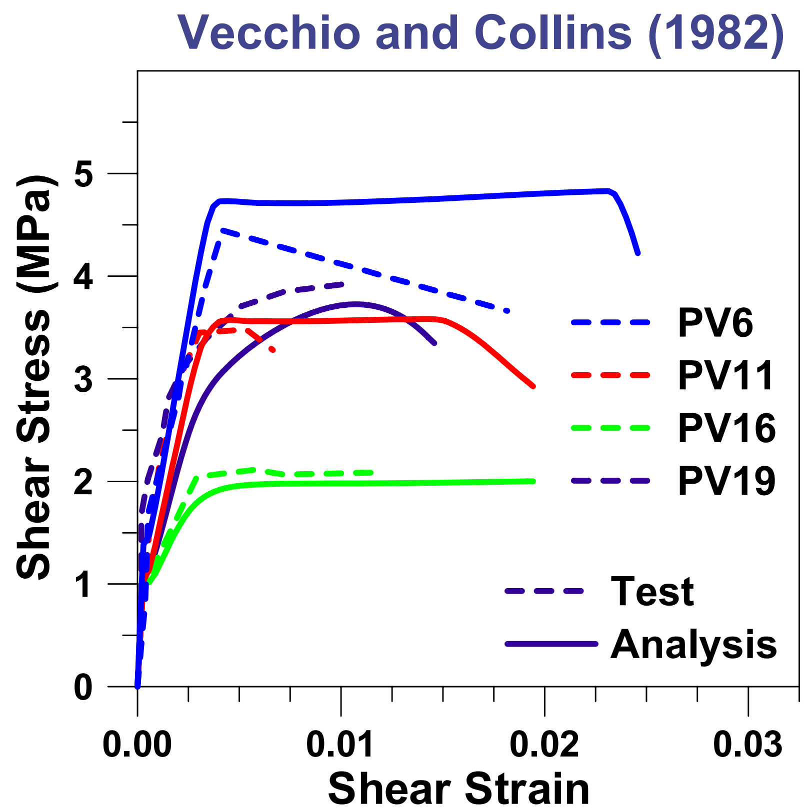
iterations

α

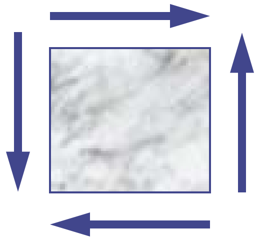
ε_x

Constitutive Panel Element Behavior

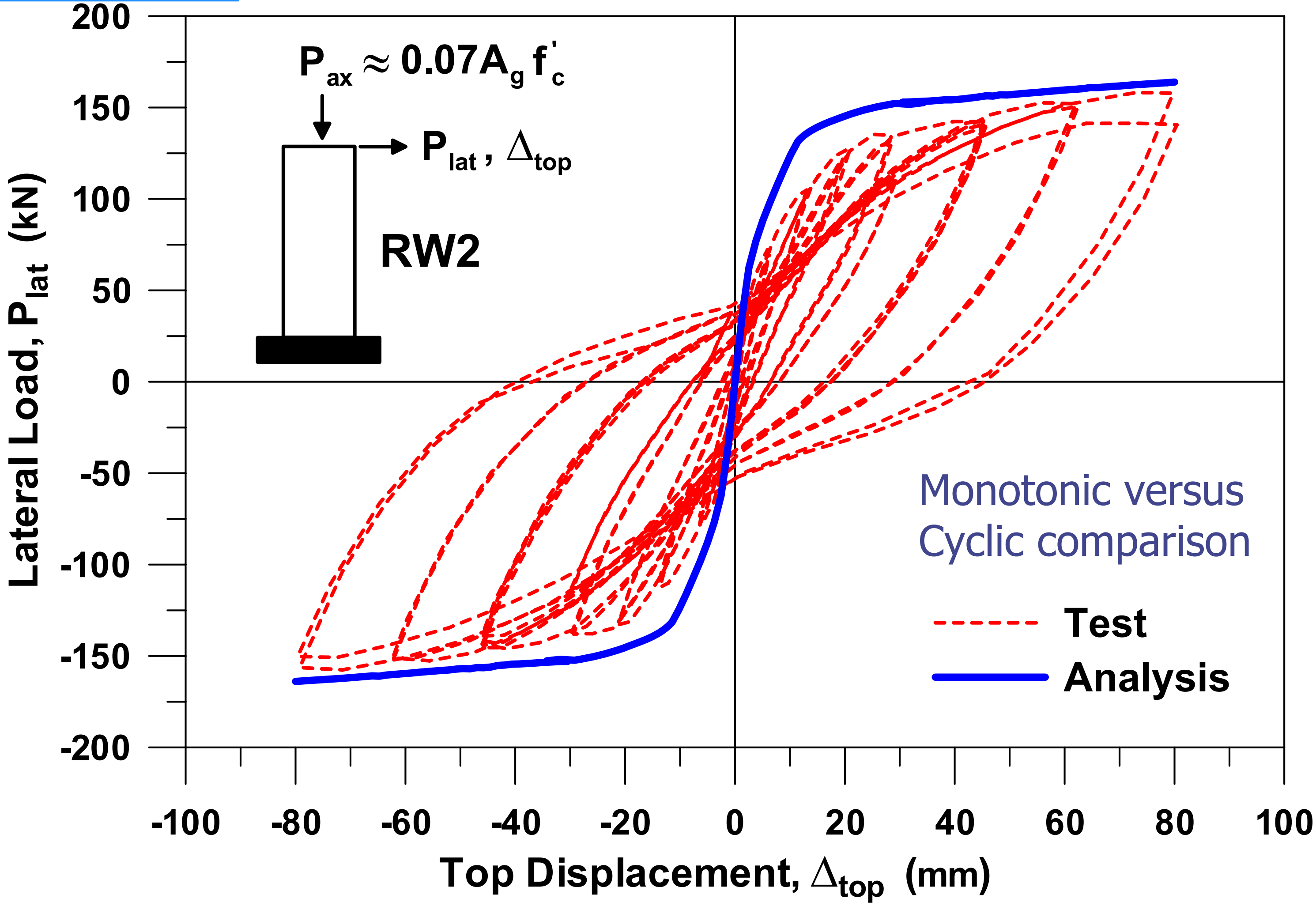




- RC Panel Specimens tested under pure shear

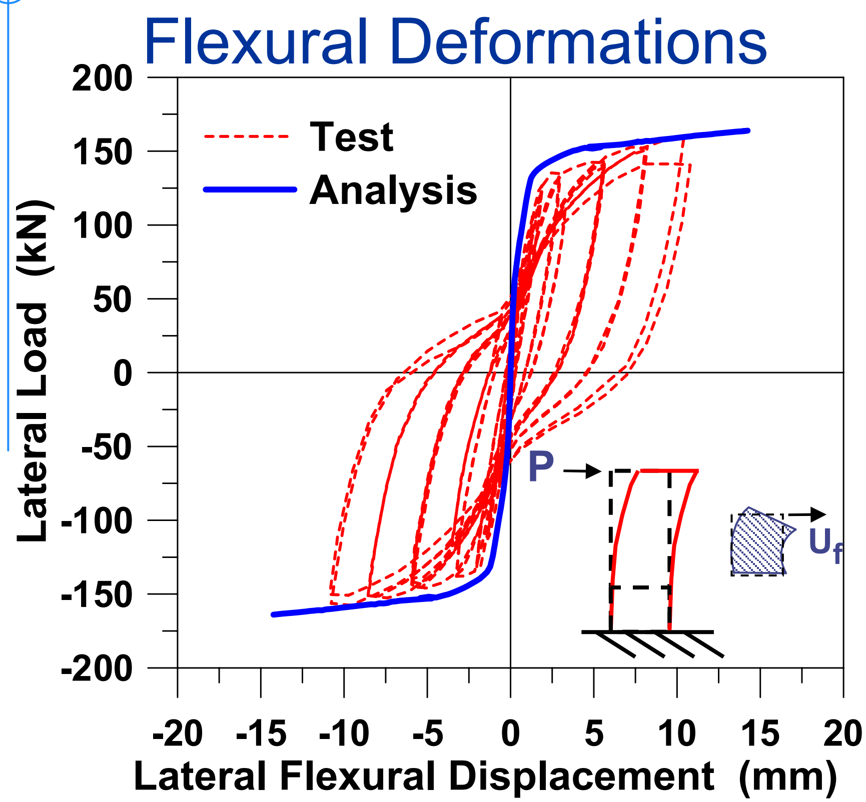


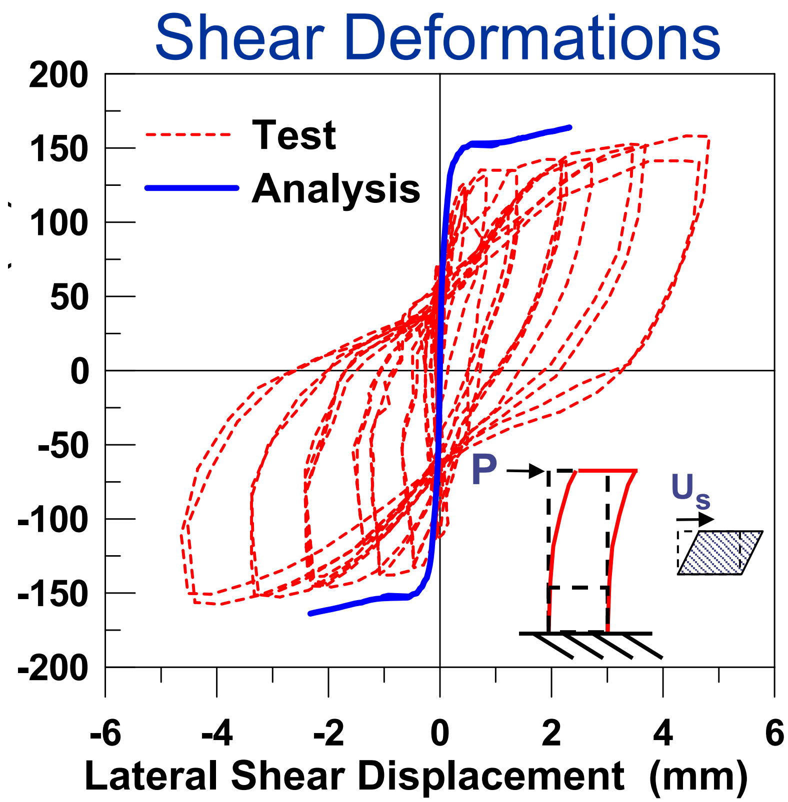
Model Assessment – RW2



Thomsen & Wallace, ASCE JSE, April 2004; Massone et al, 13WCEE & 8NCEE

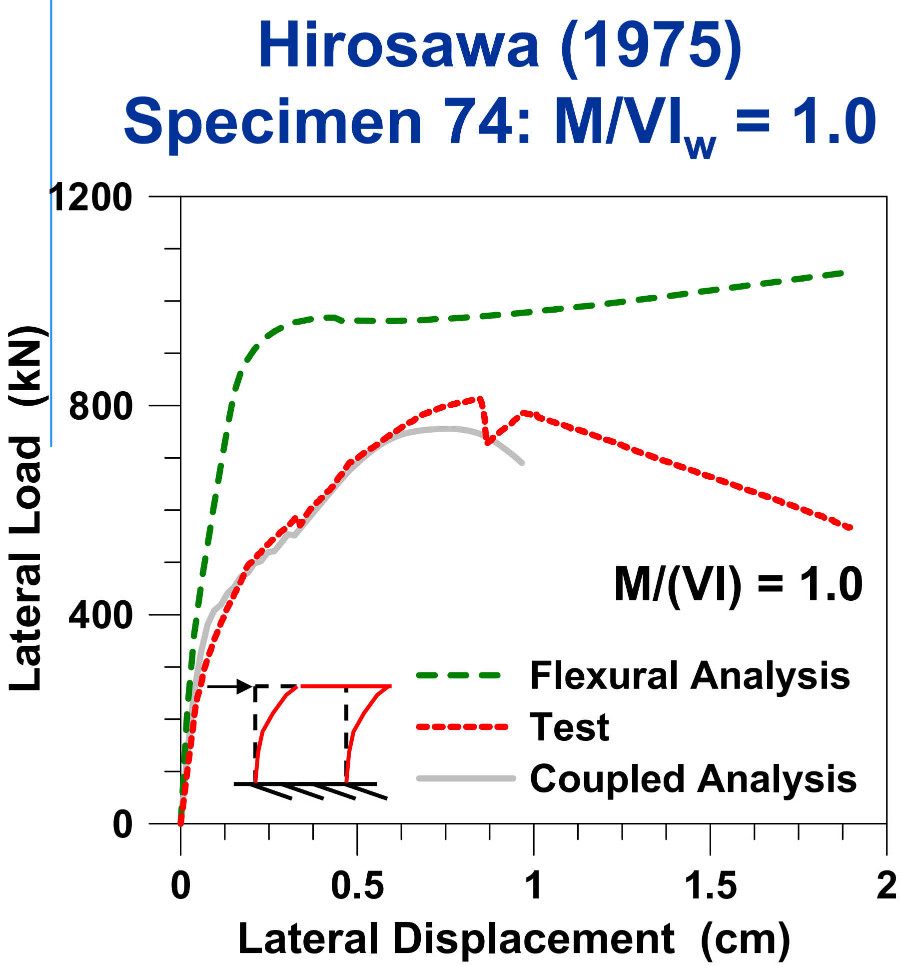
Model Assessment – RW2

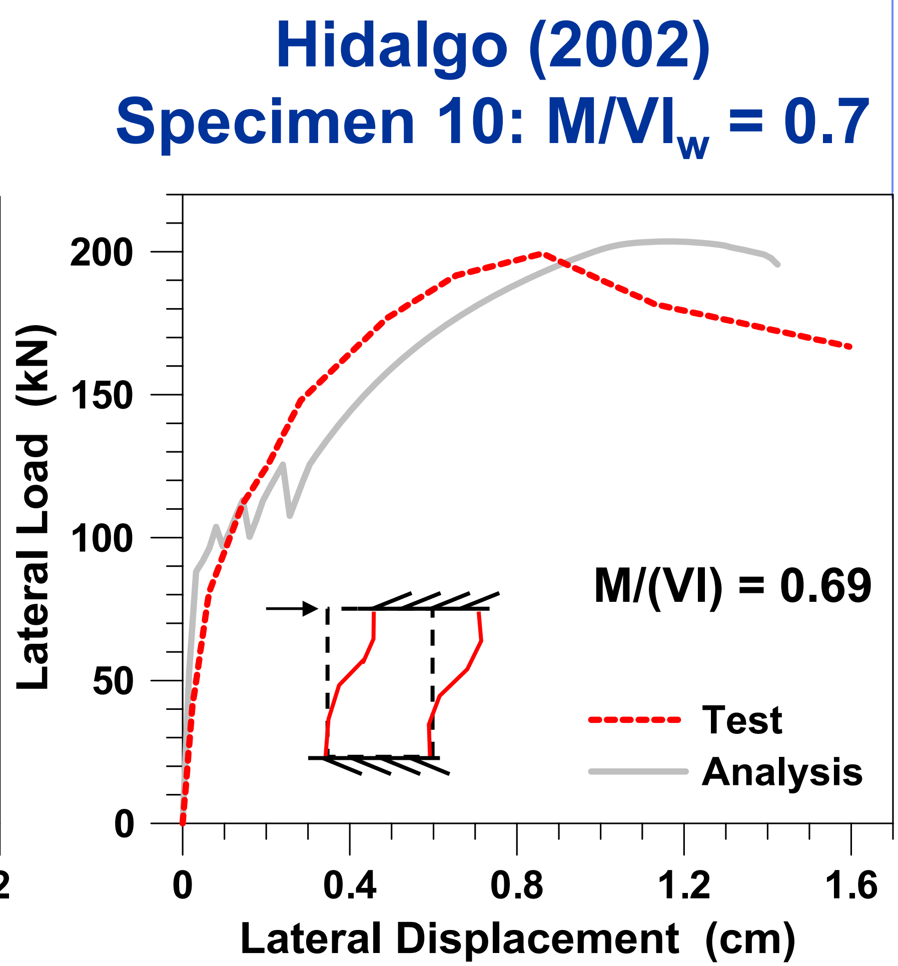




- **flexural and shear displacements at first story level of RW2**
- **coupled nonlinear flexural and shear deformations**

Model Assessment





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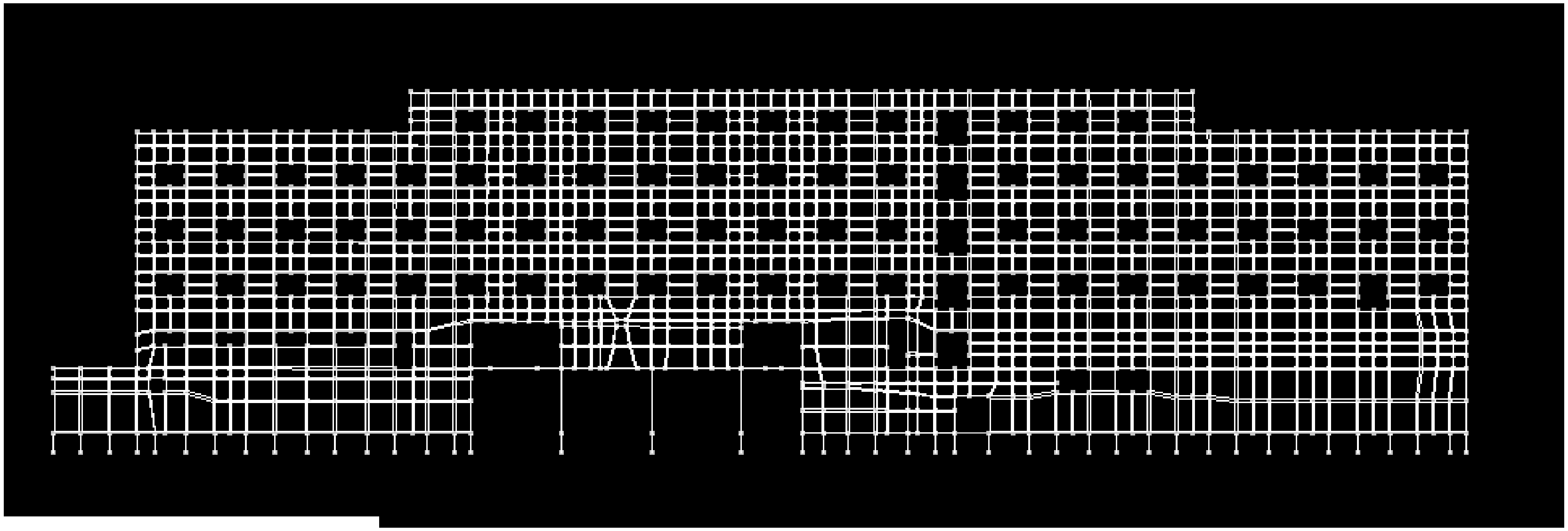
Research Motivation & Sponsors

Sponsors:
St. Joseph Health System
KPFF Consulting Engineers

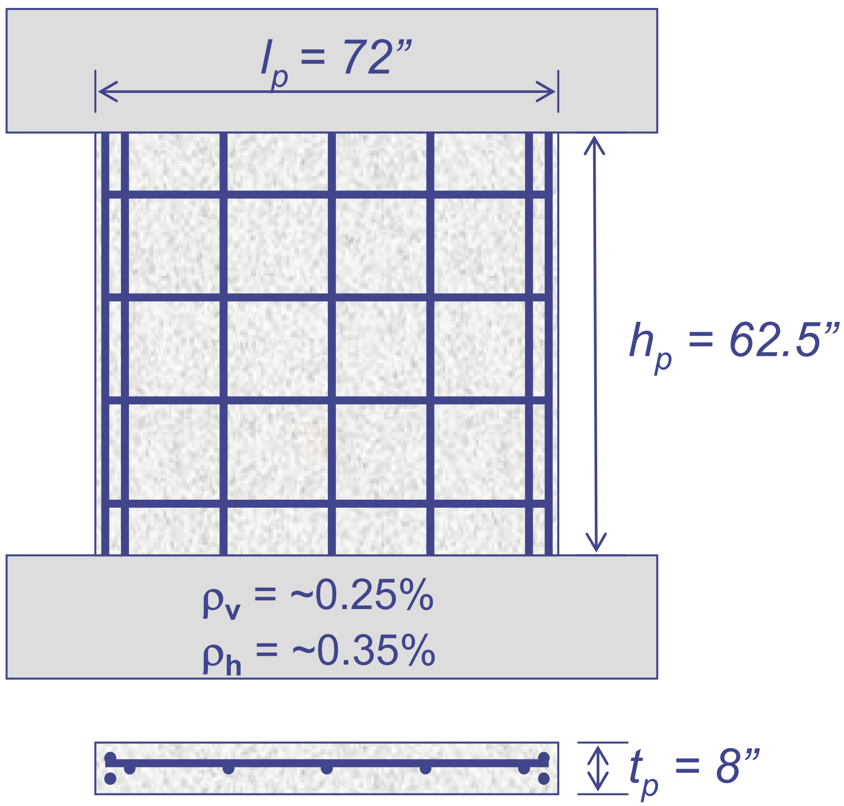
In collaboration with:
California Office of Statewide Health Planning & Development (OSHPD)



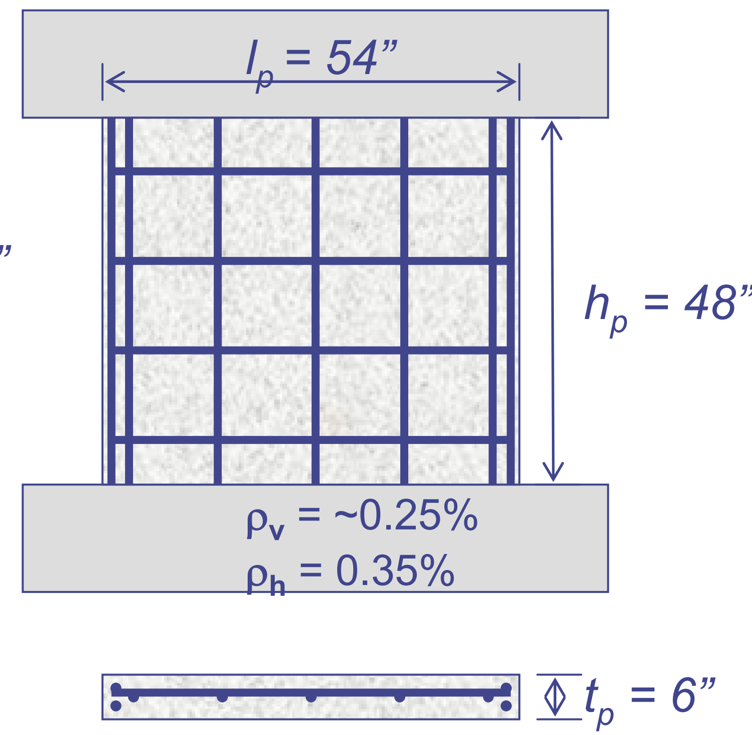
Example pushover

Test Specimens - Piers

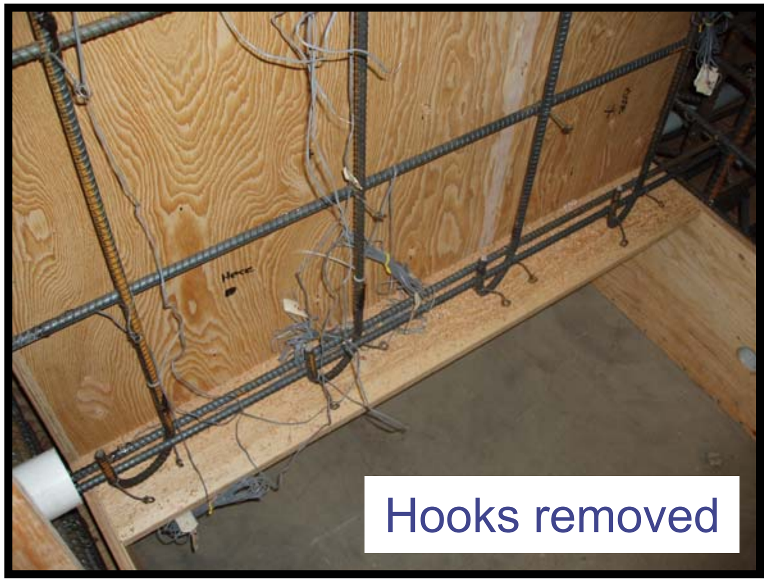
Prototype (Actual Building)



¾ Scale Test Specimen

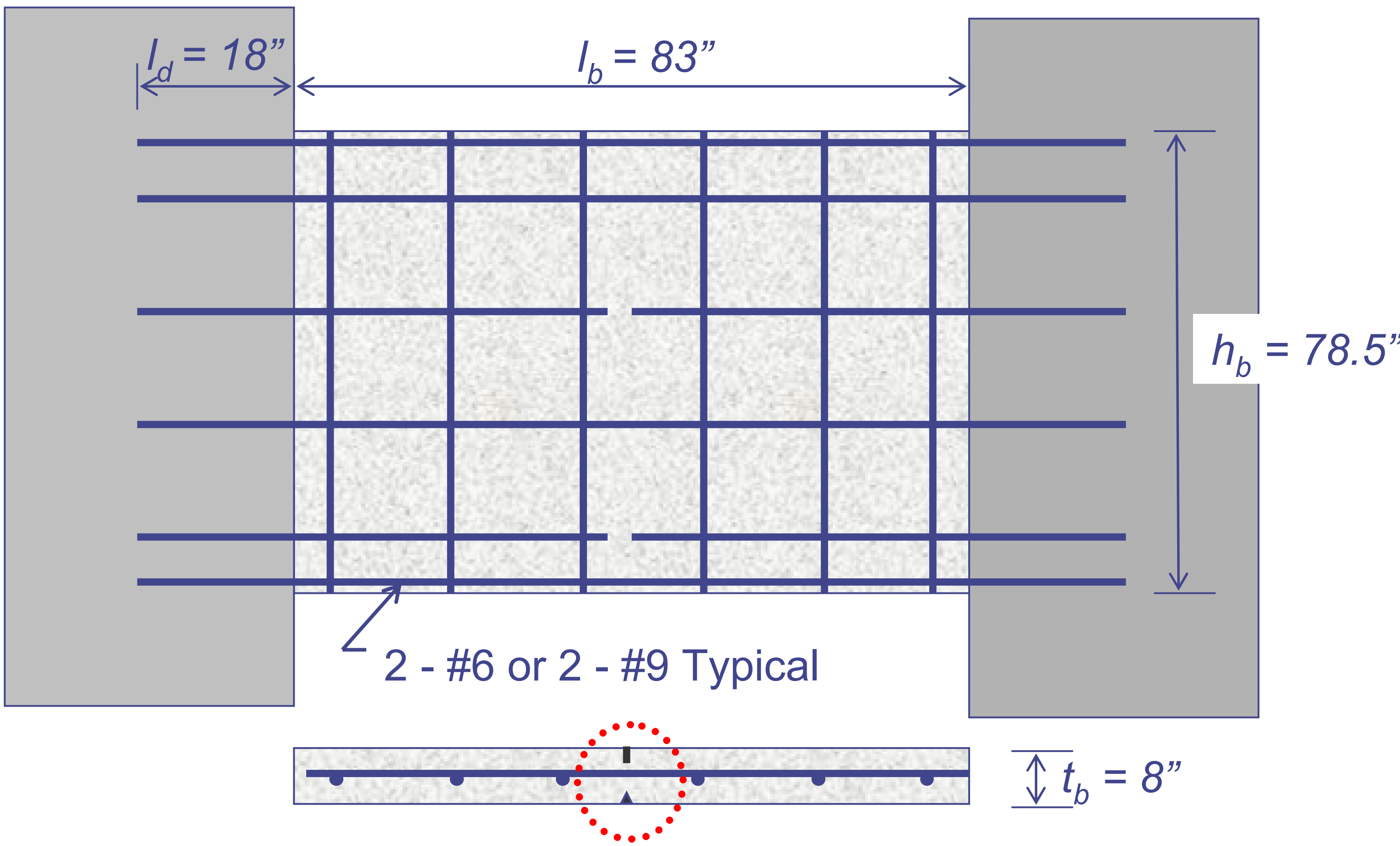


Test Specimens – Piers



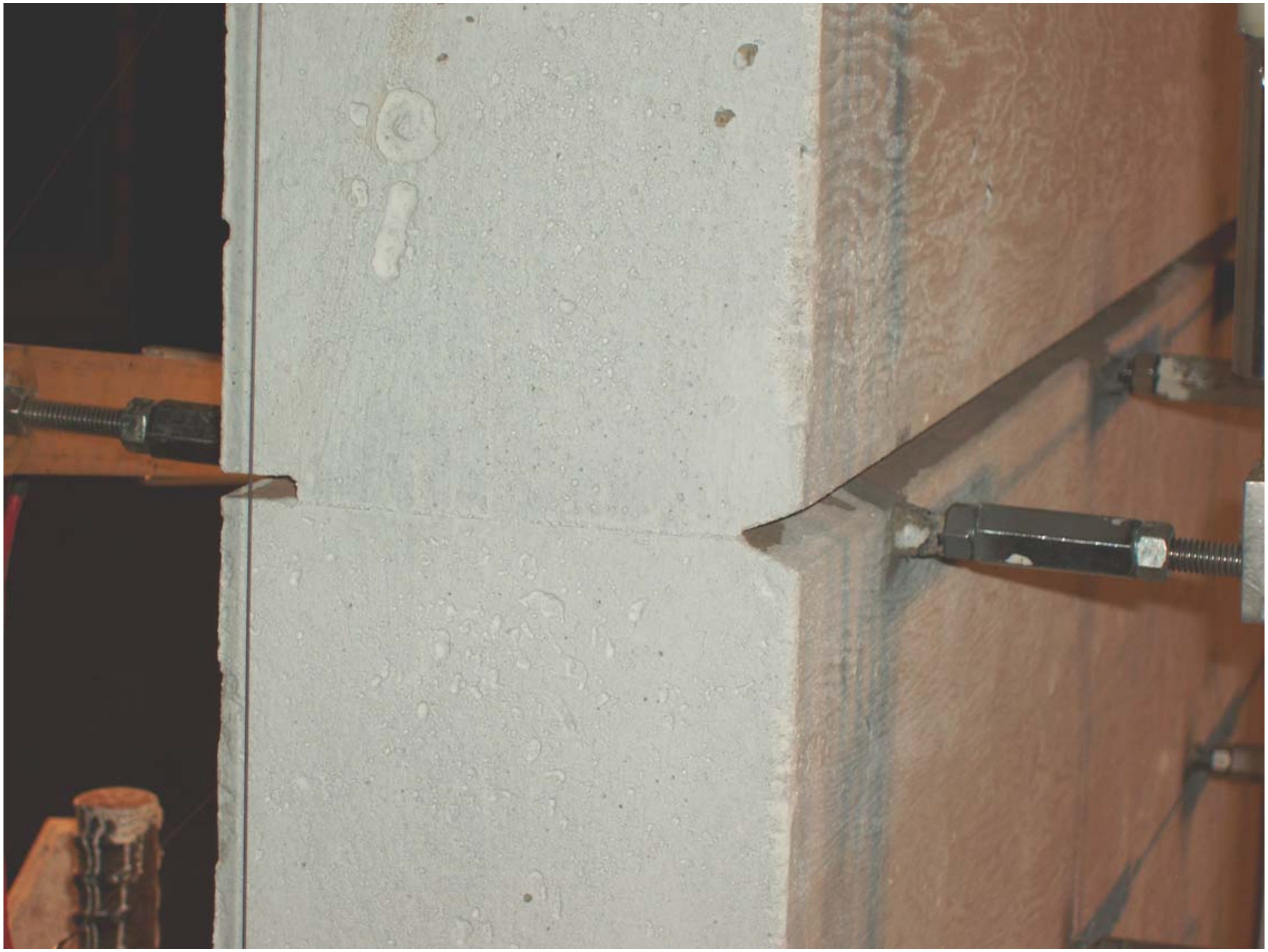
Specimen	Geometry (inches)			Reinforcement[3]			$P/A_g f'_c$[4]	Specimens
ID	Height	Length	Thickness	Edge[1]	Vert. Web[2]	Horiz. Web[2]	(kips)	(#)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
WP1-1-10	48	54	6	2 - #4	0.26%	0.35%	0.10	2
WP2-1-05	48	54	6	2 - #4	0.26%	0.35%	0.05	2
WP3-1-00	48	54	6	2 - #4	0.26%	0.35%	0.00	2
WH1-1-0	60	60	6	1-#4 1-#5	0.35%	0.26%	0.0	2
WH2-1-0	60	60	6	4 - #5	0.35%	0.26%	0.0	2

Prototype Horizontal Wall Segment



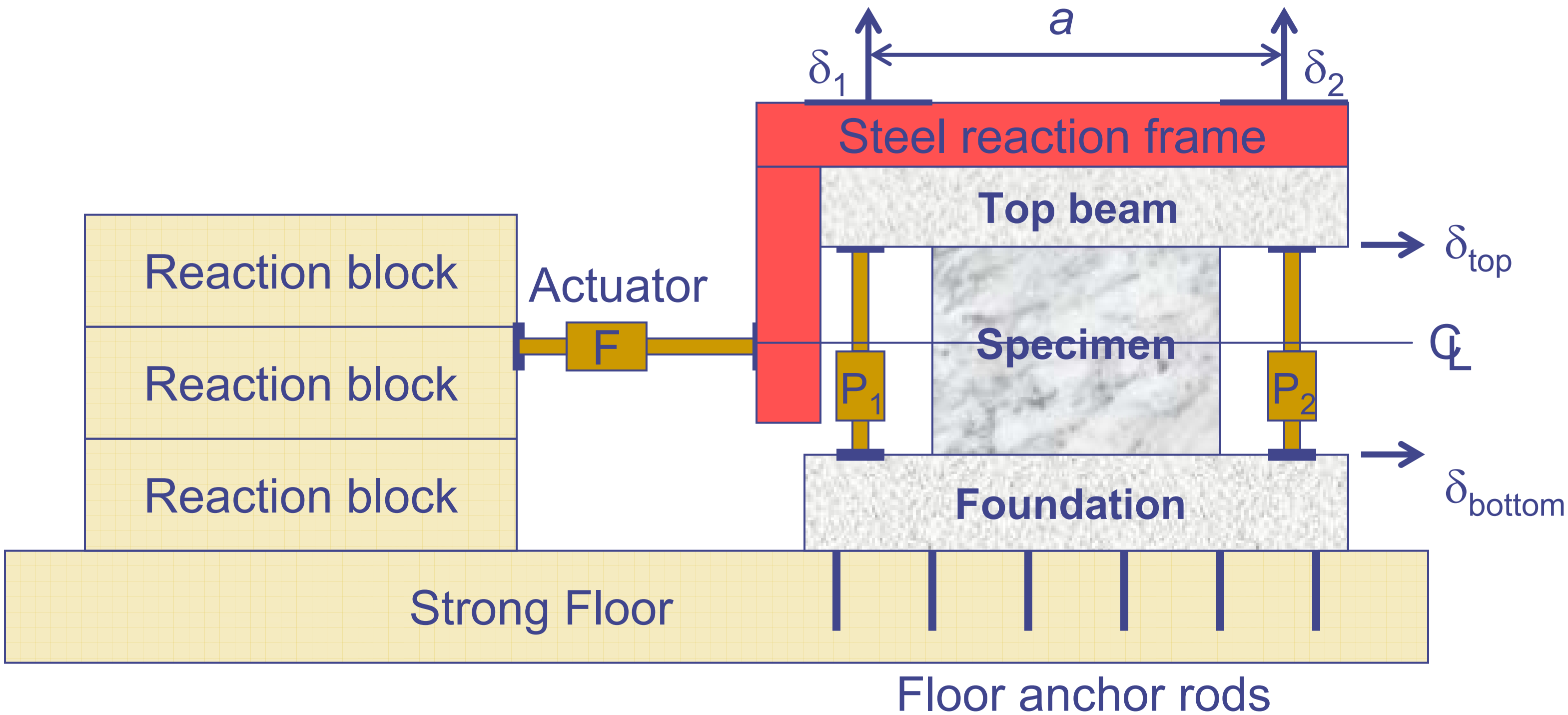
Weakened plane joint at mid-span: ½ to 2/3 of web bars cut and grooves introduced on both sides of panel

Spandrel – Weakened Plane Joint



Test Program - Construction

Test Program - Setup



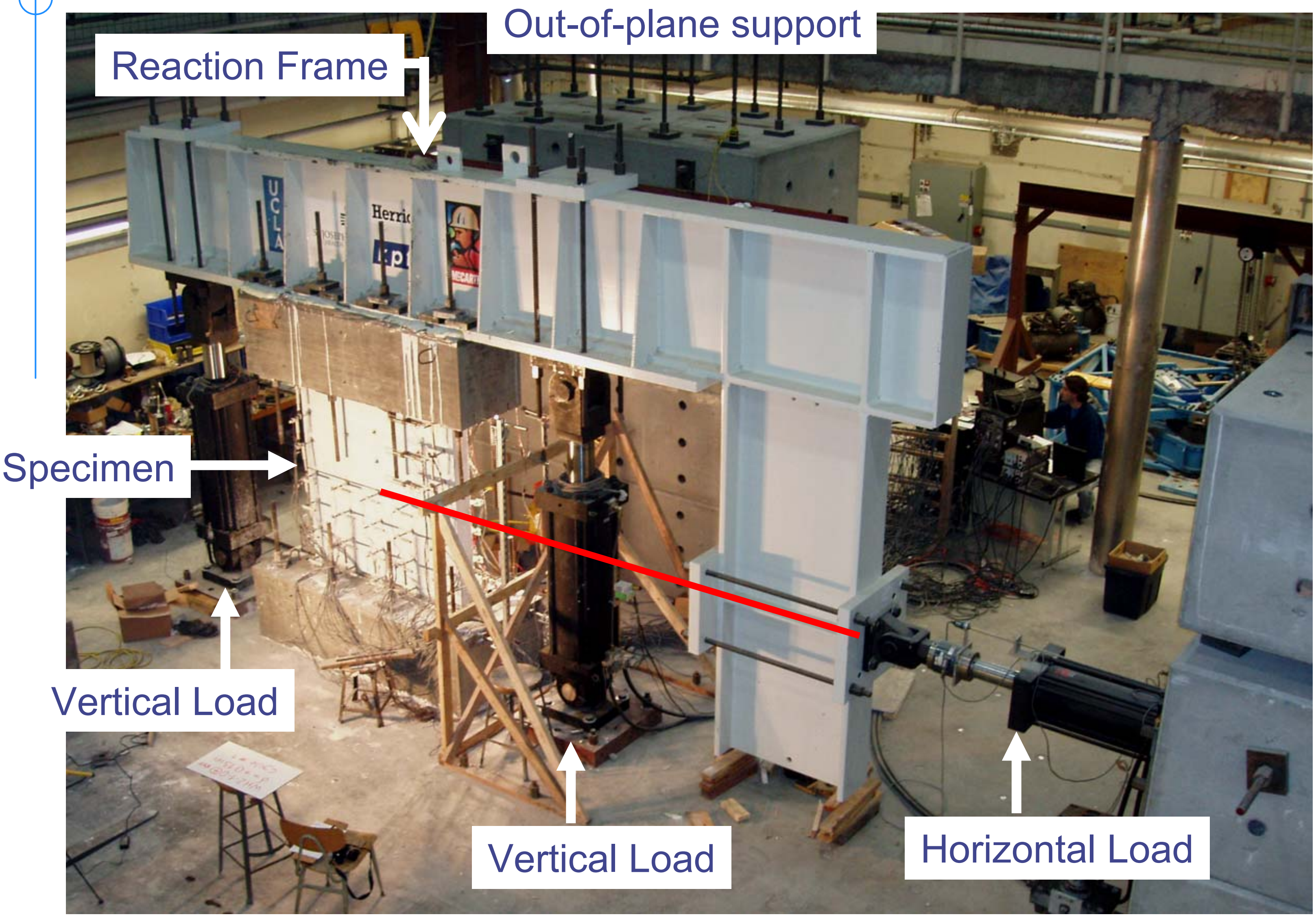
Axial Load = P = $P_1 + P_2$ (controlled)

Lateral Load = F (controlled for the first two levels)

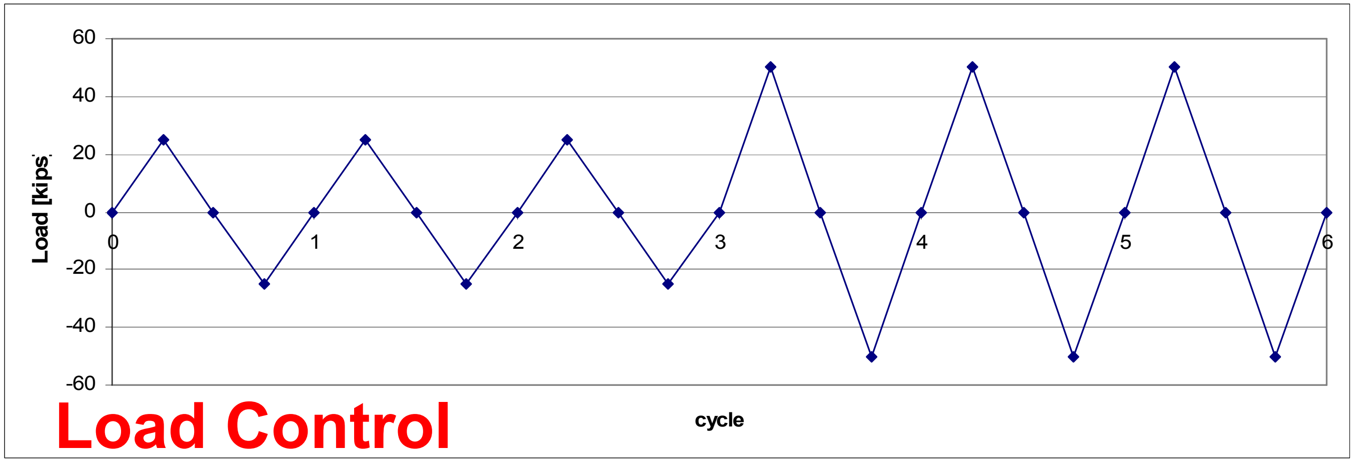
Lateral Displacement = $(\delta_{top} - \delta_{bottom})$ (controlled after first two levels)

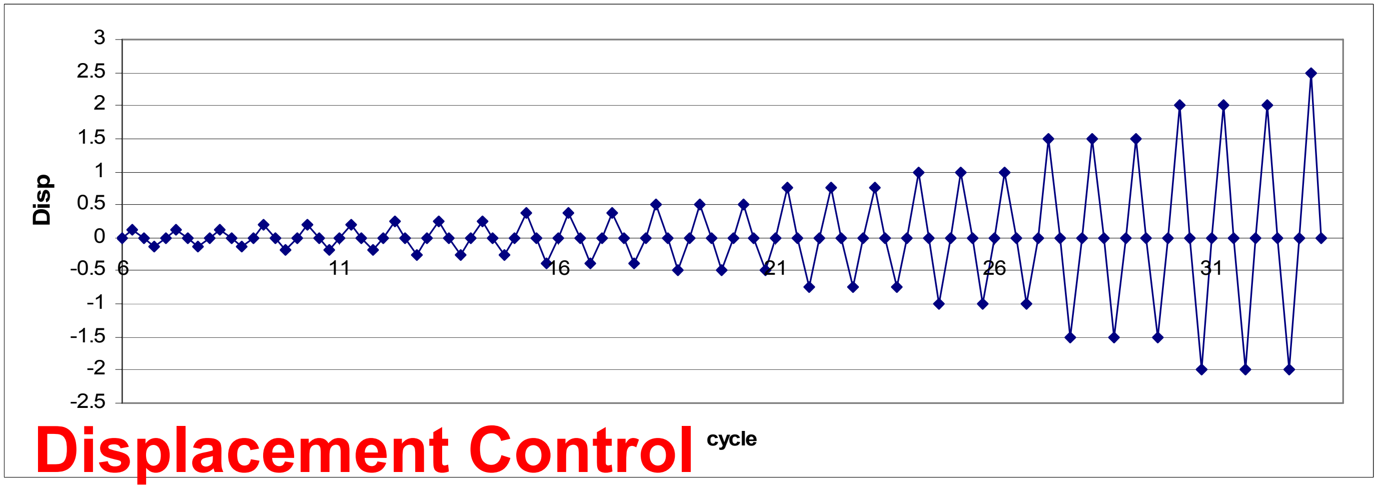
Top Rotation $\theta = (\delta_1 - \delta_2)/a = 0$ (controlled)

Test Program - Setup

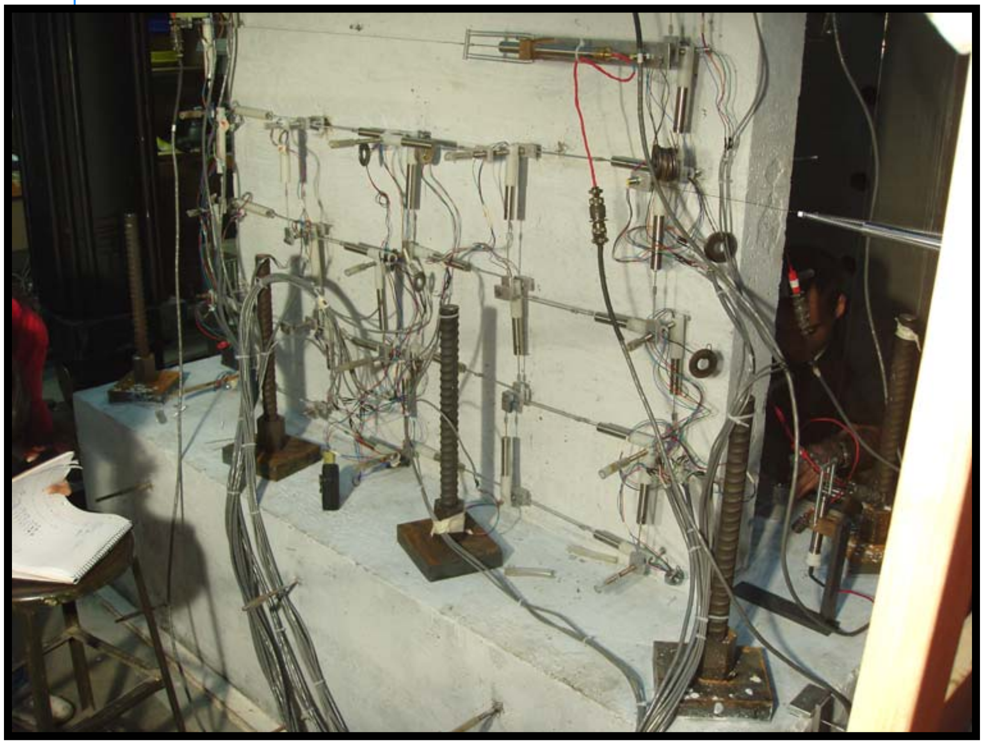


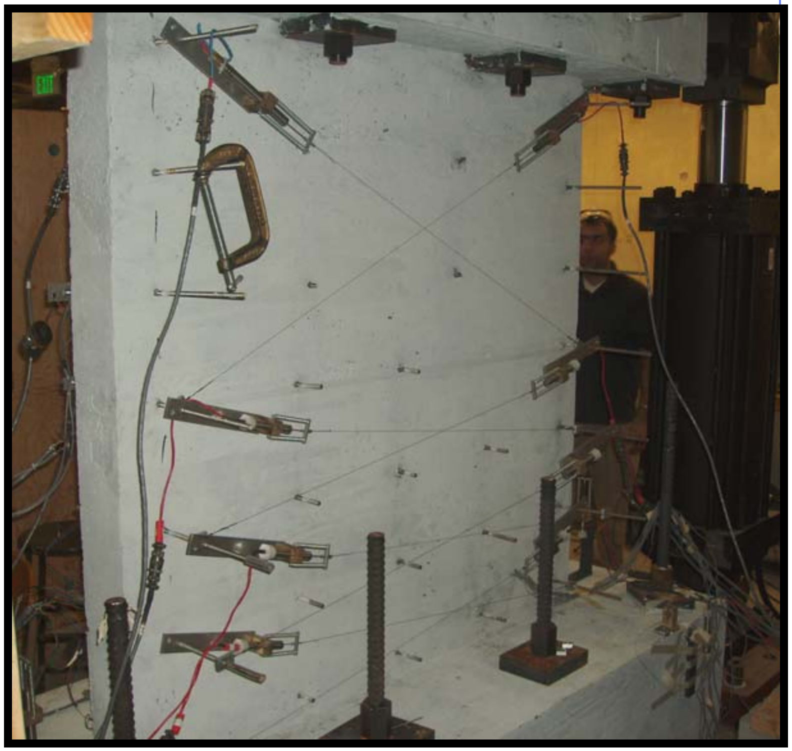
Test Program – Load History





Test Program - Instrumentation
~ 100 Sensors (load, strain, displacement)





West Face Instrumentation (flexural deformations)

East Face Instrumentation (shear and anchorage deformations)

Pedestal sliding and uplift measured
Variation of measurements used on repeated tests

Test Program - Objectives

- Backbone Relations
- Failure mode
- Influence of details
  - Jamb bars
  - No hooks
  - No Hoops/Ties

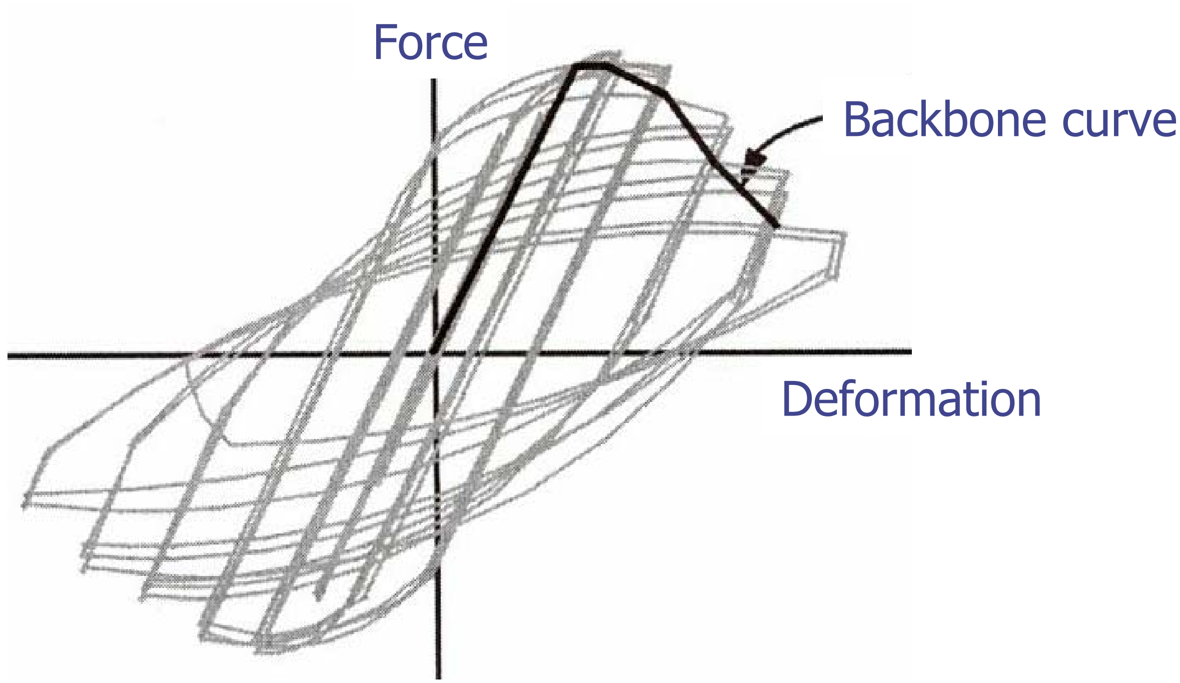
Axial load failure

FEMA 356 – Section 2.8

- Alternative modeling parameters and acceptance criteria
  - 2.8.1 Experimental setup
  - 2.8.2 Data reduction and reporting
  - 2.8.3 Design parameters and acceptance criteria
- Observations
  - For the right owner/building, can be highly productive process
  - Caveats (uncertainty, surprises, etc)
  - Satisfaction, but ultimately, it's about

FEMA 356 – Backbone Curves

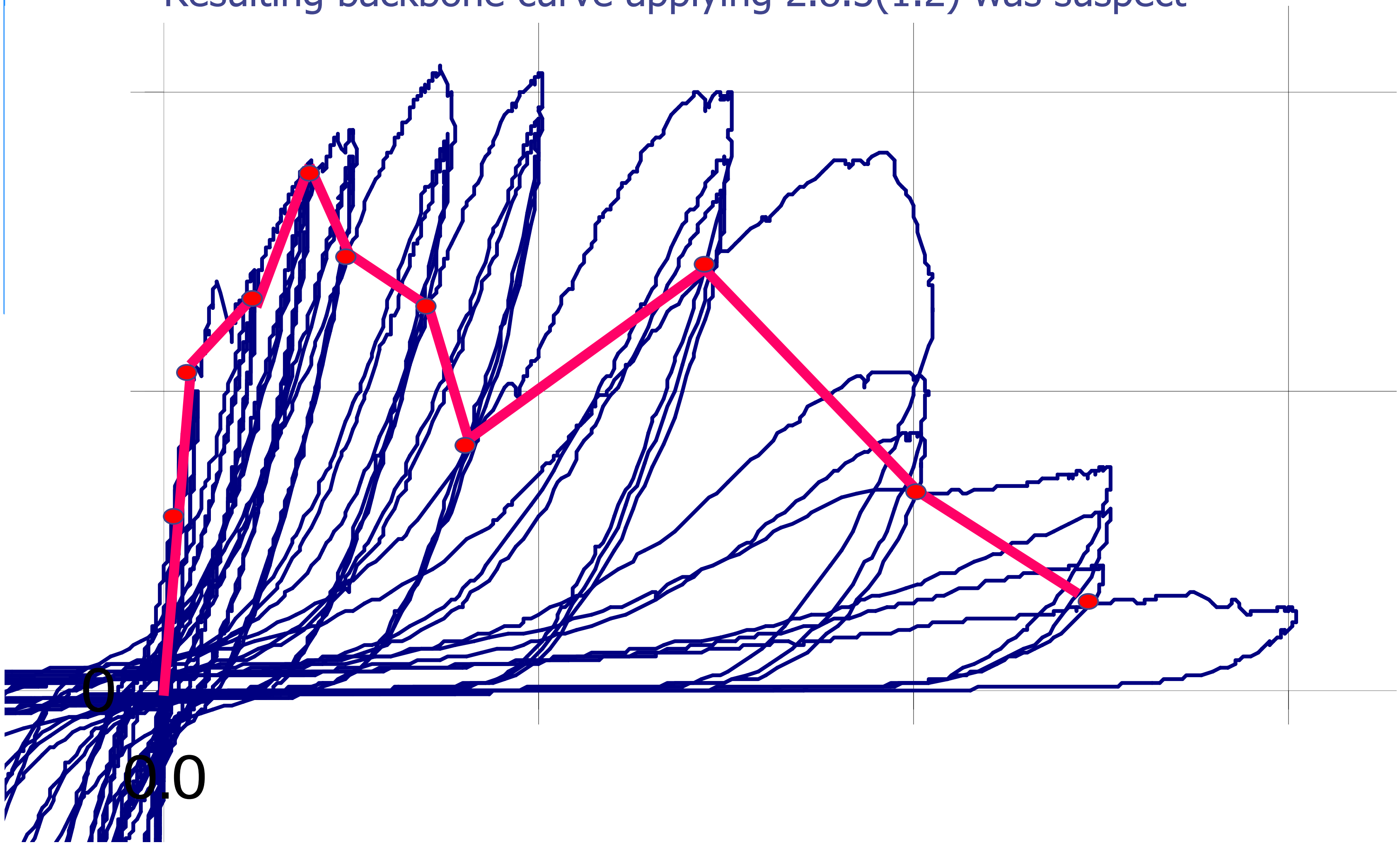
- 2.8.3(1.2): Smooth “backbone” curve shall be drawn through the intersection of the first cycle curve for the (i)th deformation step with the second cycle curve of the (i-1)the deformation step, for all i steps.



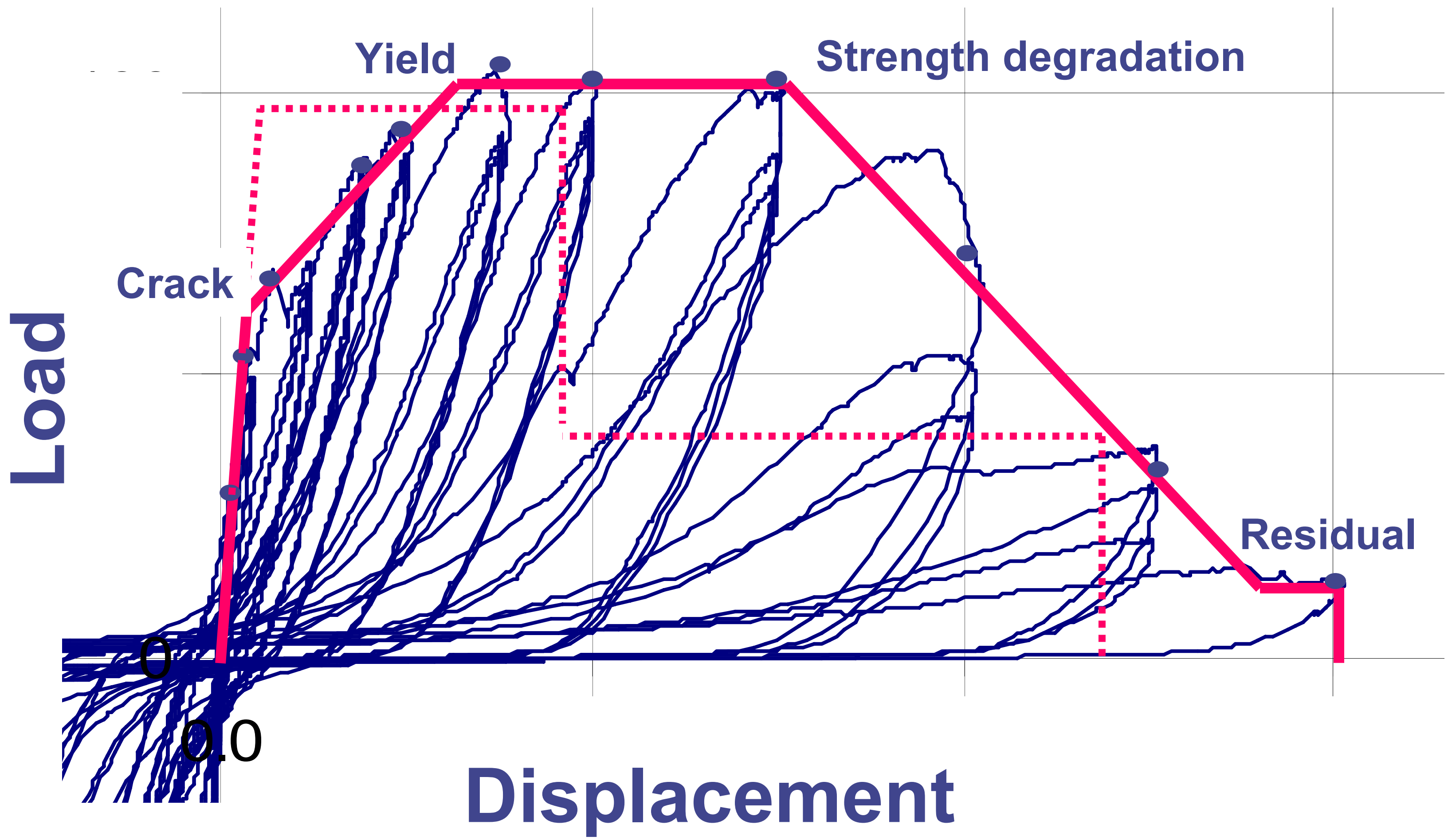
FEMA 356 Figure 2-4

FEMA 356 2.8.3(1.2) Approach

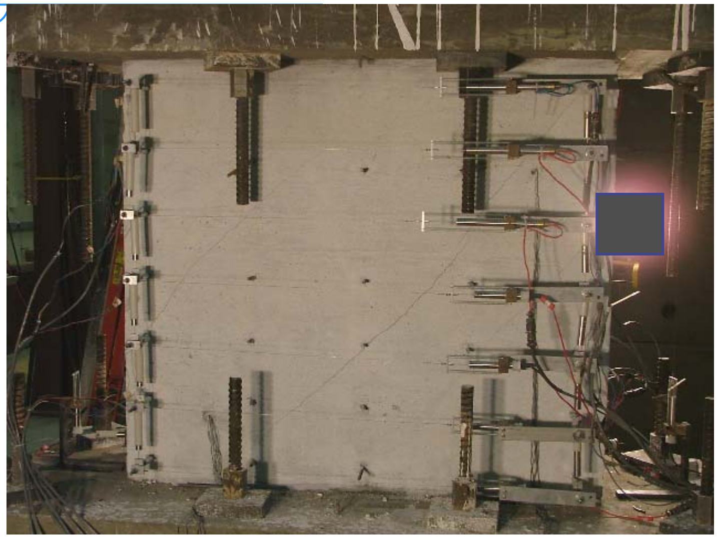
Resulting backbone curve applying 2.8.3(1.2) was suspect



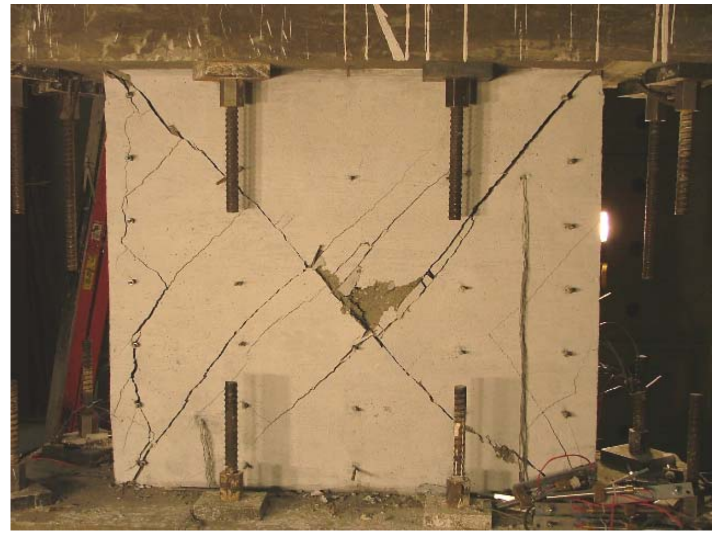
Test-Derived Backbone Curves



Test Photos ~5% Axial Load



Yield level

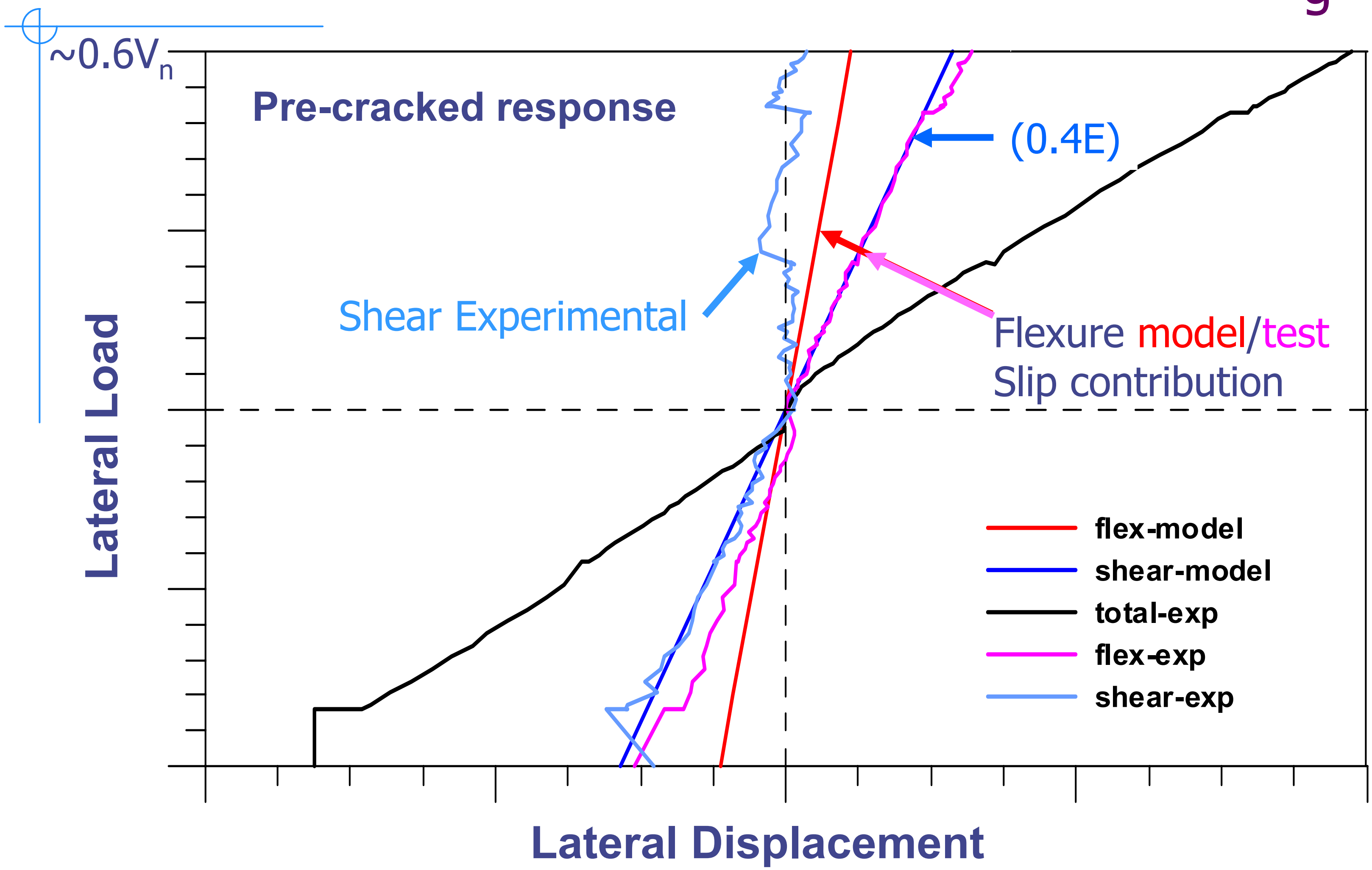


3 x Yield

Axial collapse

Axial Failure

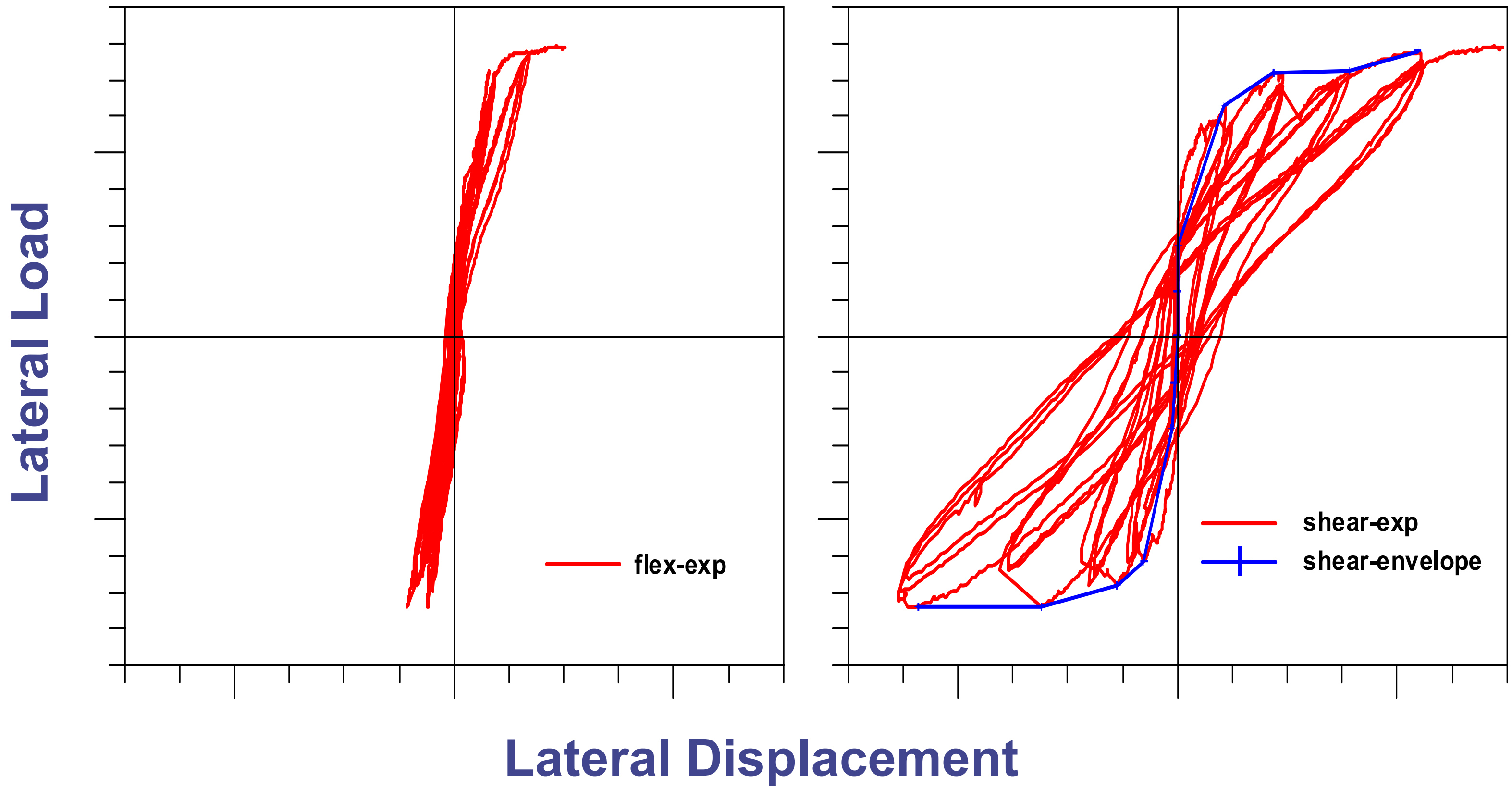
Initial Stiffness: Pier test: $P=0.05A_g f'_c$



$0.4E_c$ is reasonable for uncracked shear stiffness
Flexural stiffness appears impacted by slip.

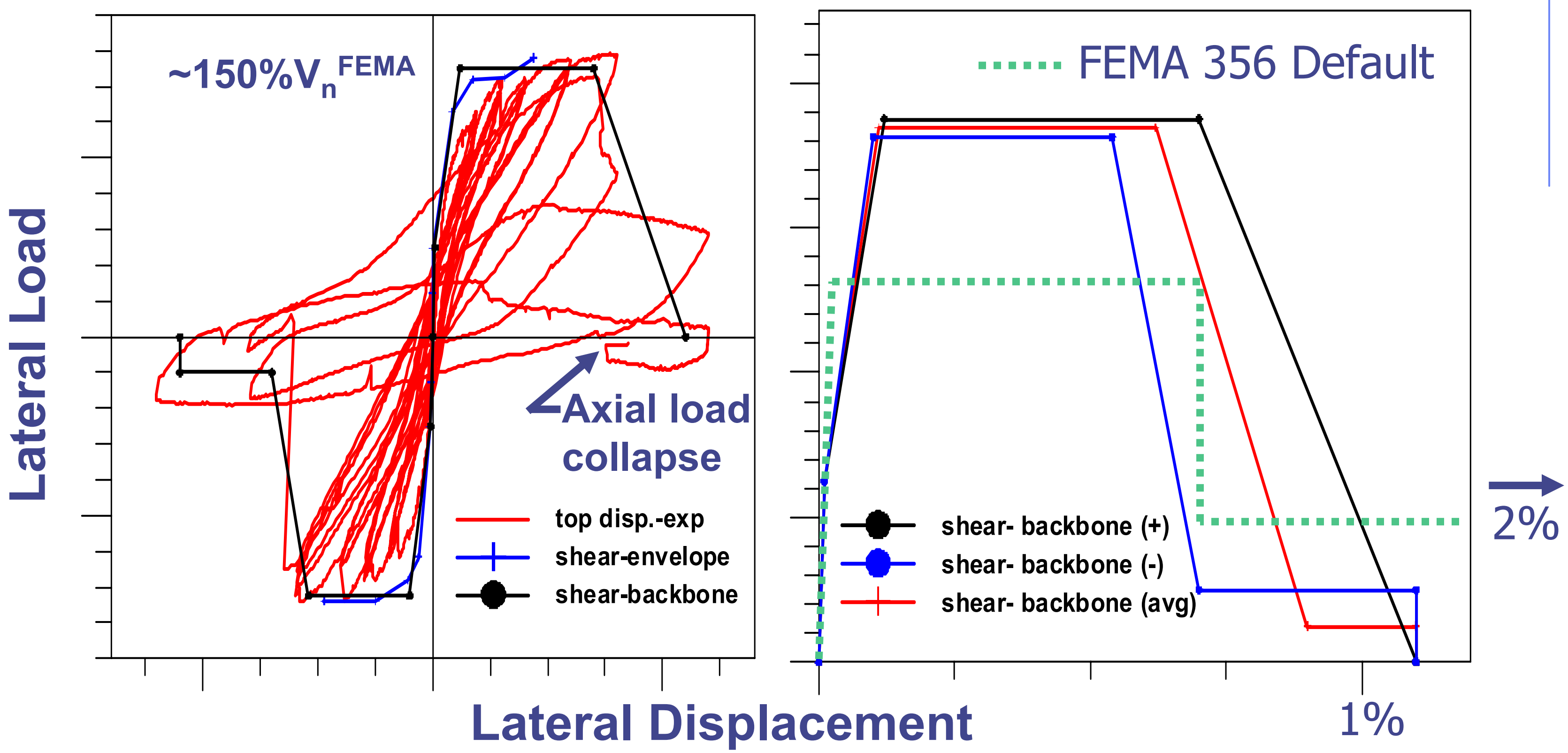
Deformations – Flexure/Shear

- Same Scale



Flexural deformations are essentially elastic, nonlinear shear

Test Derived Backbone Relations (Pier)

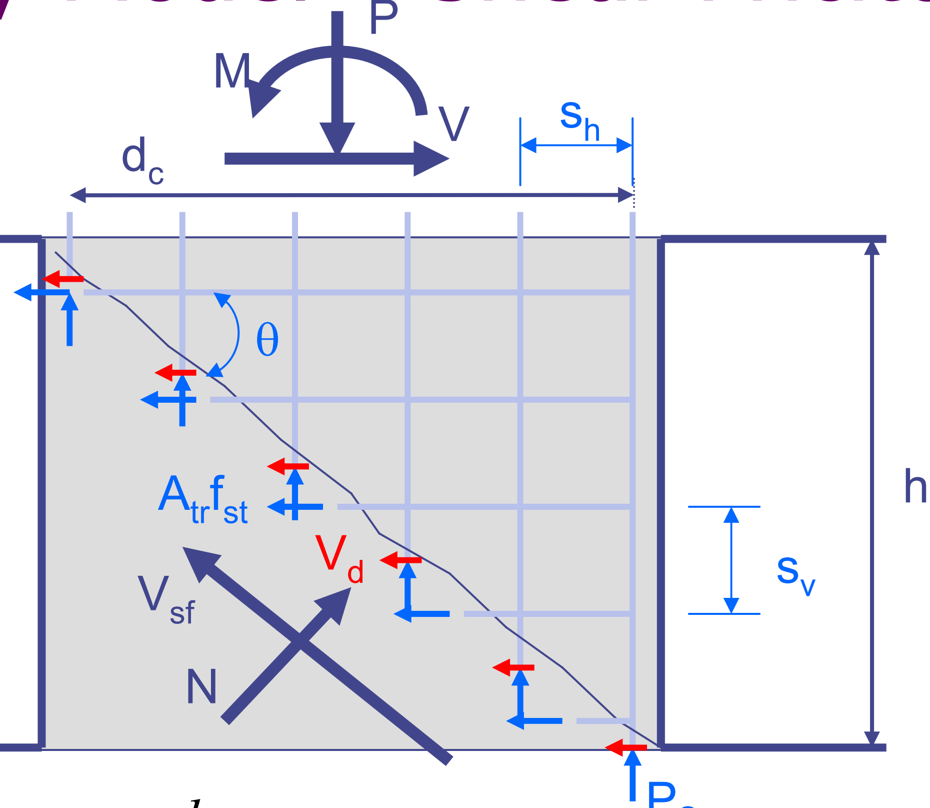


- Not as stiff in the post-cracked range as FEMA relation
- Post-cracked stiffness ~1/10 to 1/20 of the initial stiffness
- Peak strength (85 to 175%) of V_n – Consistent with prior tests
- Less pronounced strength degradation, less residual strength
- Deformation capacity > FEMA at initiation of strength degradation

Presentation Overview

- FEMA 356 Requirements
- P-M-V Modeling
- Preliminary test results
- Axial load modeling

Axial Capacity Model – Shear Friciton



V_d = dowel force
$A_{tr}f_{st}$ = force in horizontal steel
P_s = force in vertical steel
V_{sf} = force due to shear friction
N = normal force

$$V + N\sin\theta = V_{sf}\cos\theta + A_{st}f_{st}\frac{d_c}{s_v}\tan\theta + n_{bars,web}V_{d,web} + n_{bars,boundary}V_{d,boundary}$$

$$P = N\cos\theta + V_{sf}\sin\theta\frac{d_c}{s_h}P_{s,web} + n_{bars,boundary}P_{s,boundary} + n_{bars,web}P_{s,web}$$

Axial Capacity Model

- Axial capacity (Equilibrium and shear friction)

$$P_m = \left(\frac{A_s f_{yt} h}{s_v}\right)\left(\frac{1 + \mu_m \tan\theta}{\tan\theta - \mu_m}\right)$$

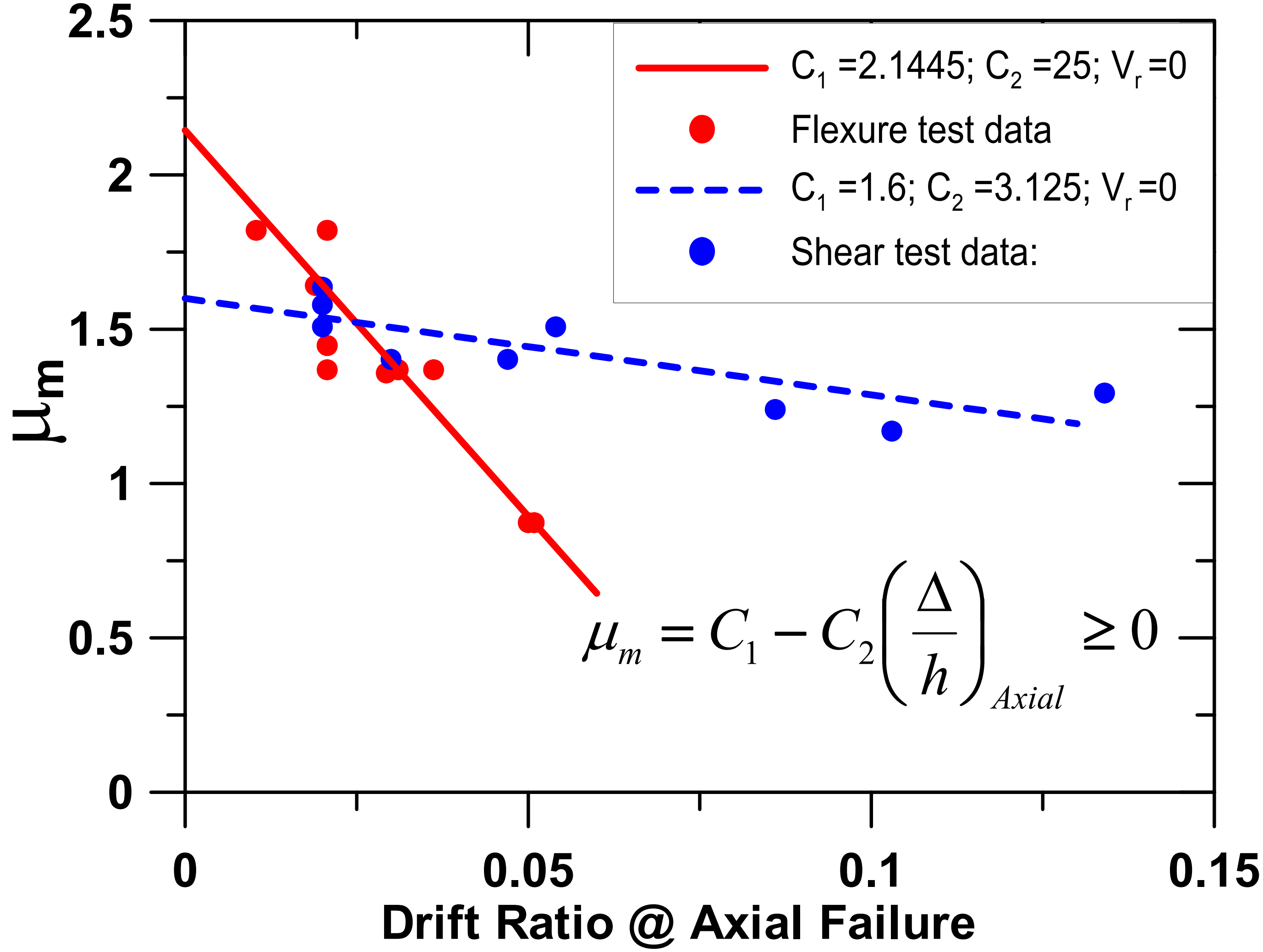
- Shear friction vs drift at axial failure

$$\mu_m = C_1 - C_2 \left(\frac{\Delta}{h}\right)_{Axial} \geq 0$$

- Drift at axial failure (column test data)

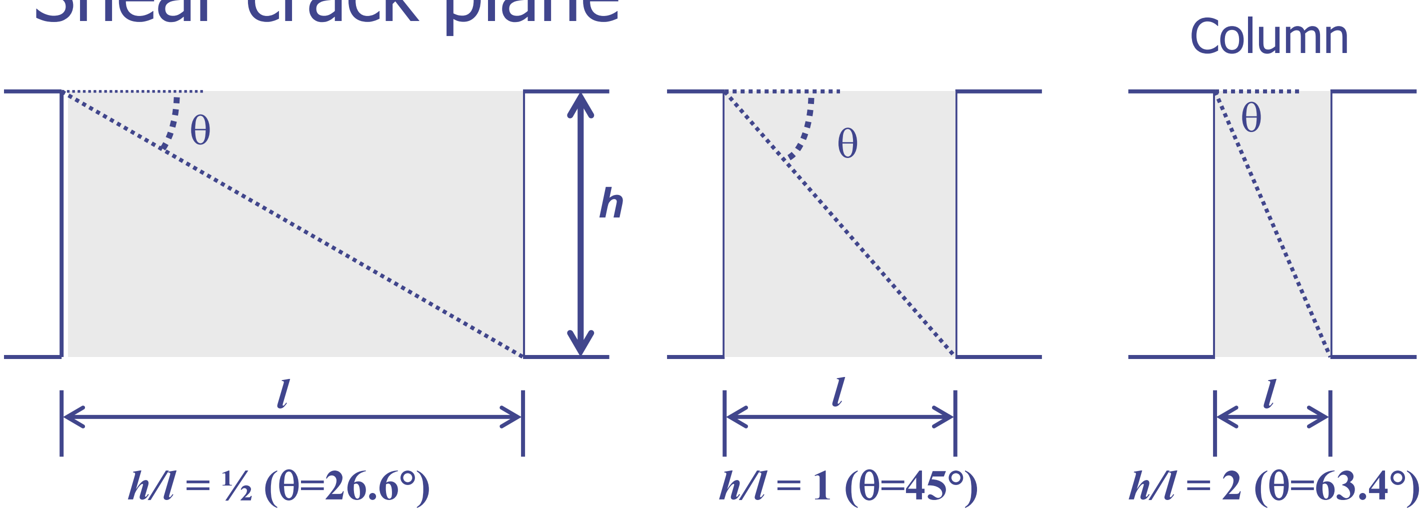
$$\left(\frac{\Delta}{L}\right)_{Axial} = \frac{(1 + C_1 \tan\theta) + \left(P / A_{st} f_{yt} h / s_v \right)(C_1 - \tan\theta)}{C_2 \left(\frac{P}{A_{st} f_{yt} h / s_v} + \tan\theta \right)}$$

Shear Friction - Columns

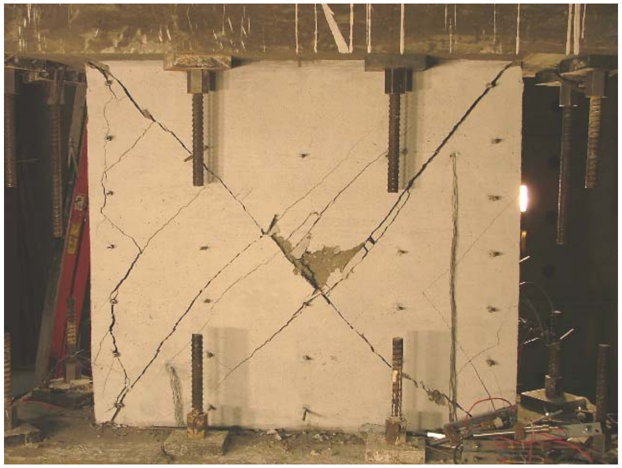


Influence of Pier Geometry

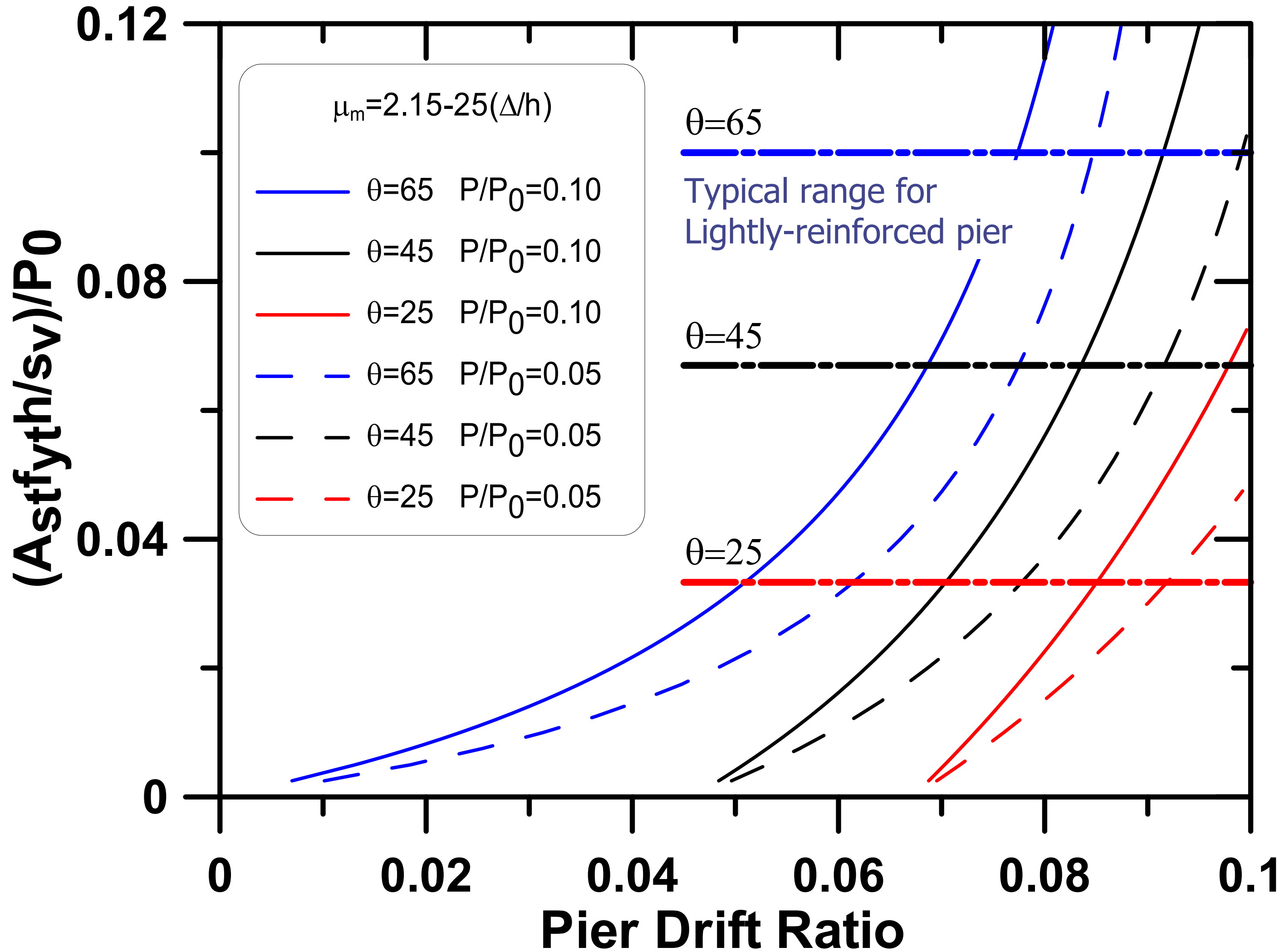
- Shear crack plane



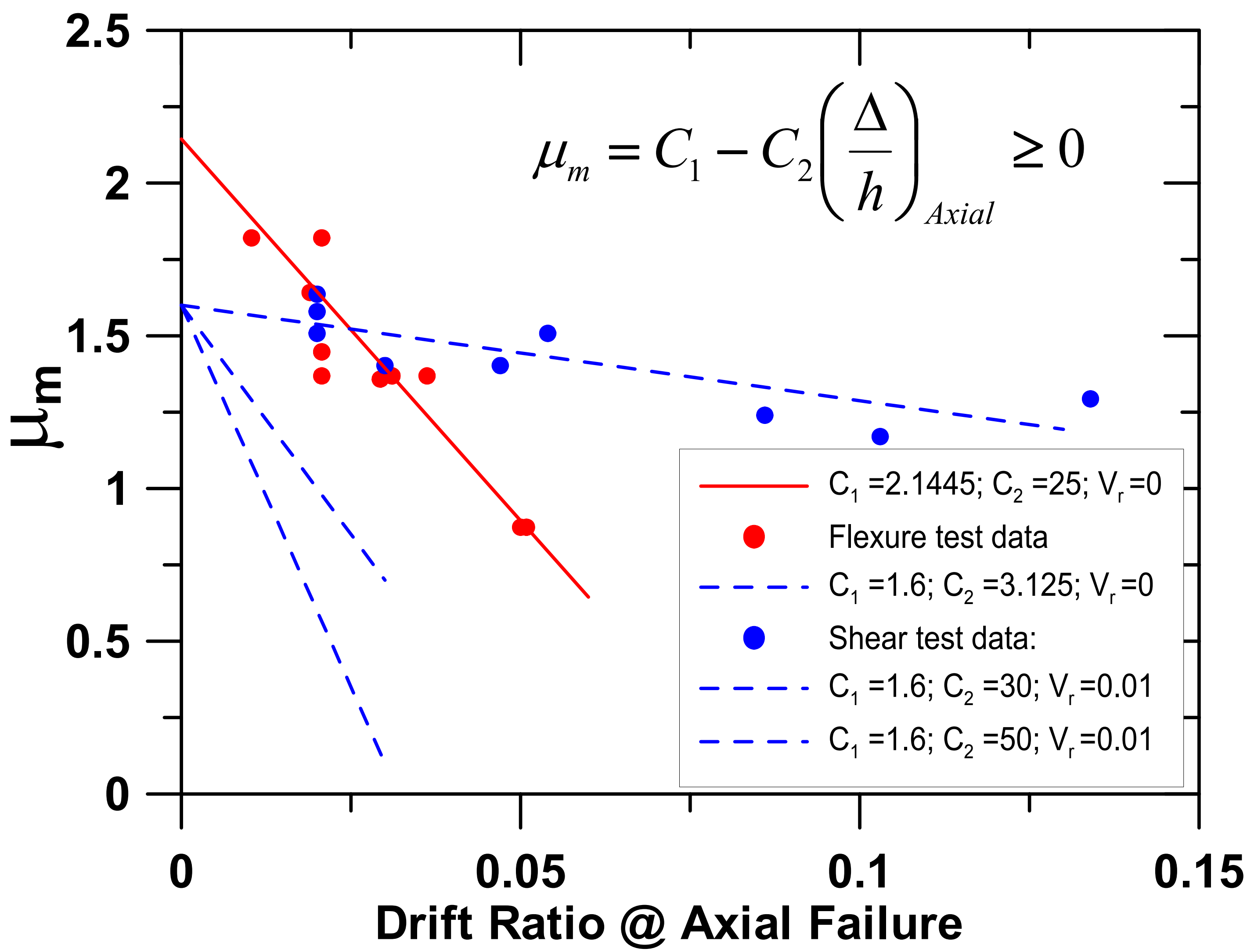
  - Assumed to extend full pier height, from corner-to-corner



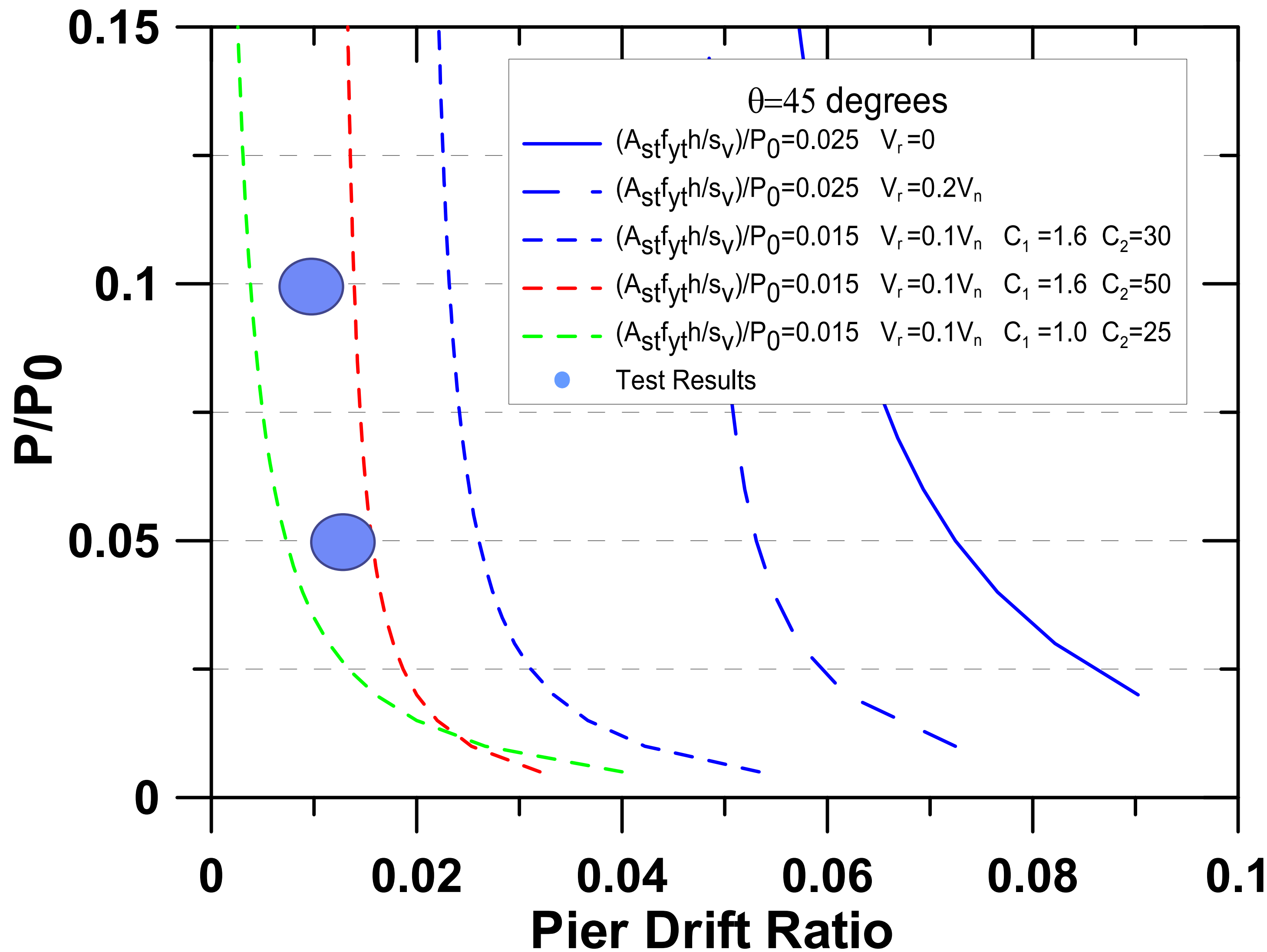
Axial Capacity Model – Wall Piers



Shear Friction – Column Tests



Axial Capacity Model – Test Results



Lightly-Reinforced Wall Segments

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University of California, Los Angeles

with contributions from

Mr. Leonardo Massone & Dr. Kutay Orakcal

University of California, Los Angeles

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Bold, <u>underlined</u>: Test results presented