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Nonlinear Analysis of Concrete Wall Buildings Using OpenSees

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University of Washington

Acknowledgements

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  - Joshua Pugh, EDG Inc.
  - Dawn Lehman, University of Washington
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  - The National Science Foundation
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  - The Charles Pankow Foundation

Why Concrete Walls

Damage to Chilean Walled Buildings

- Compression-controlled flexural failure for walls with poorly confined boundary elements

CM: 18+2 stories, residential building, 2006 construction, Concepcion, Chile

AH: 15+2 stories, mixed-use, 2009 construction, Concepcion, Chile (J. Moehle)

PR: 12-story, residential building, 2006 construction, Concepcion, Chile

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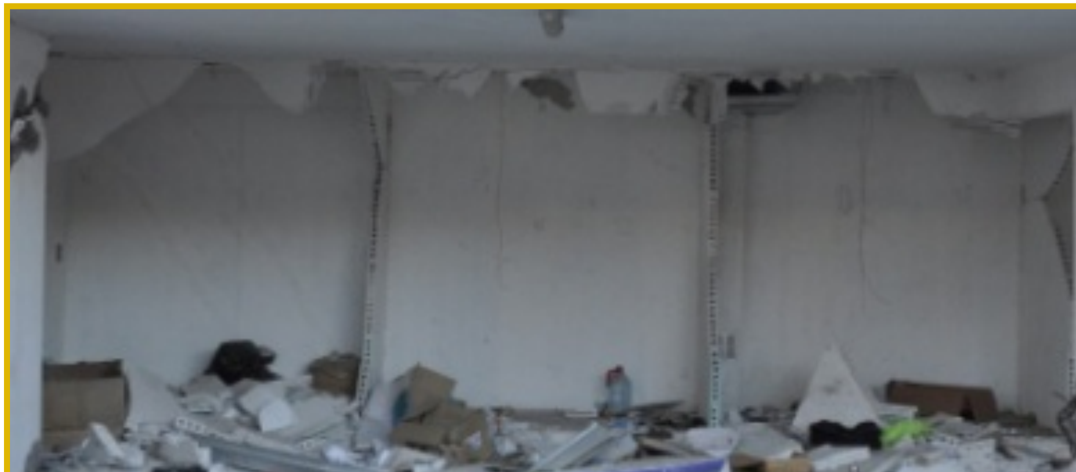
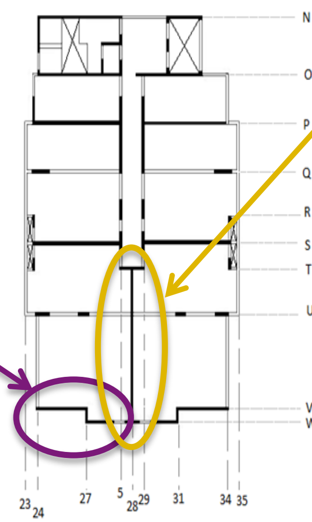
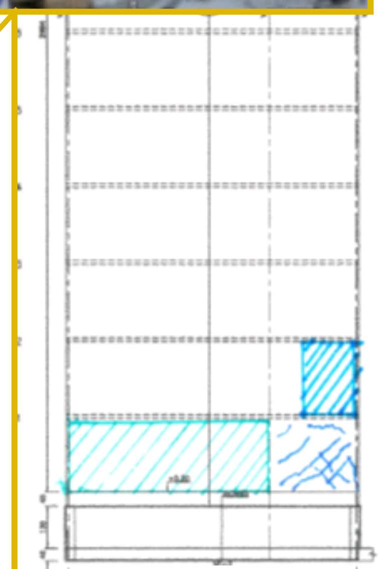
Damage to Chilean Walled Buildings

- Compression and shear damage in lightly reinforced walls





Plan View

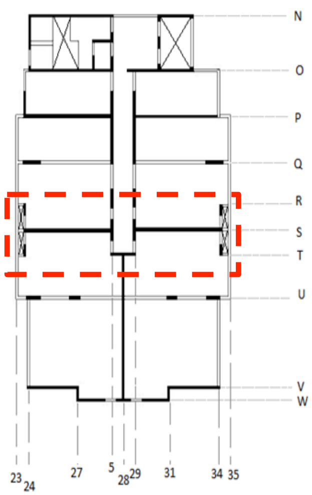


Elevation View

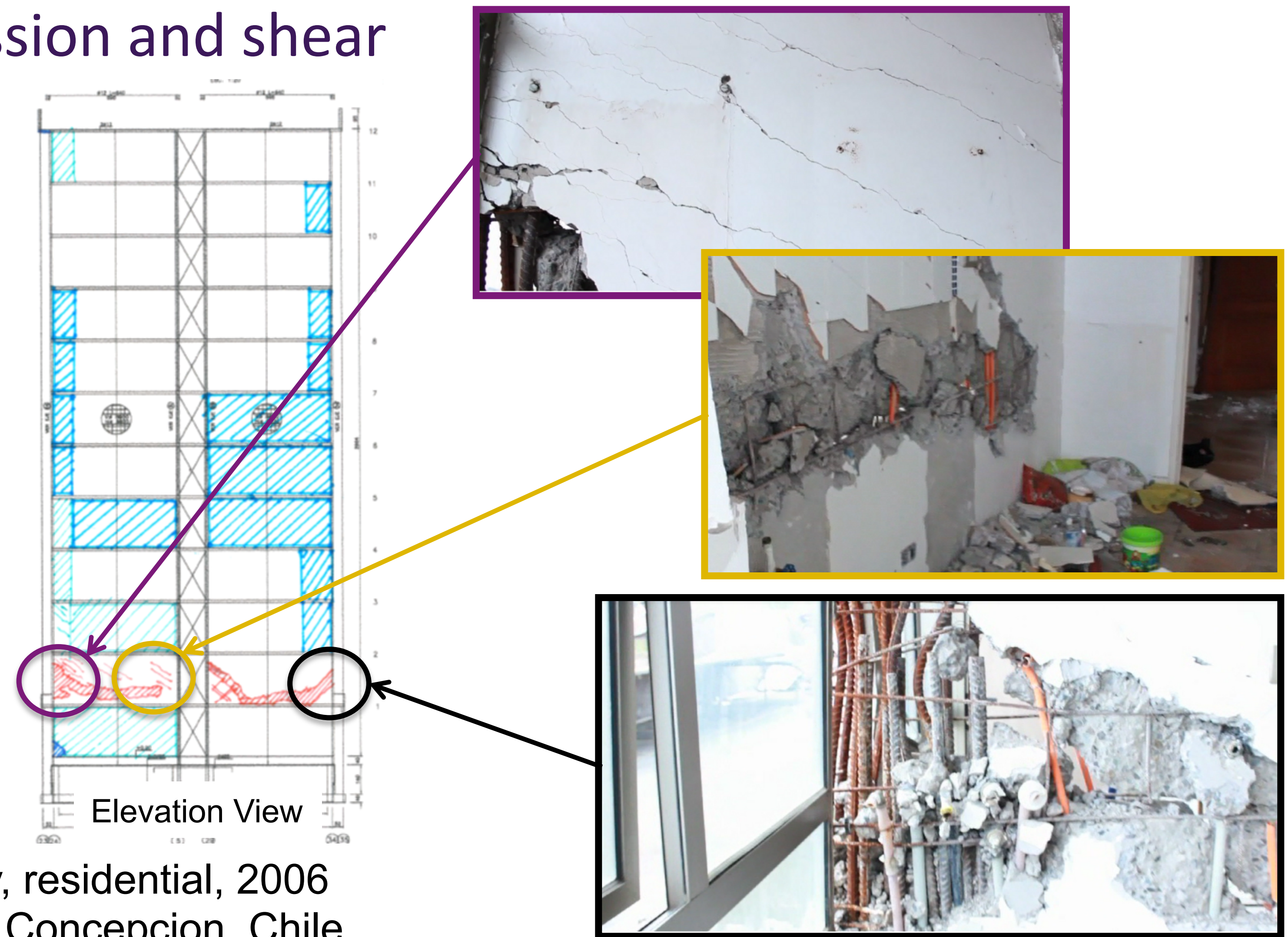
PR: 12-story, residential, 2006 construction, Concepcion, Chile

Damage to Chilean Walled Buildings

- Compression and shear damage



Plan View



PR: 12-story, residential, 2006 construction, Concepcion, Chile

Damage to Walled Buildings in Christchurch, NZ

- Damage to modern walls
- Modern walls exhibiting compression-controlled flexural failure and shear-compression failure

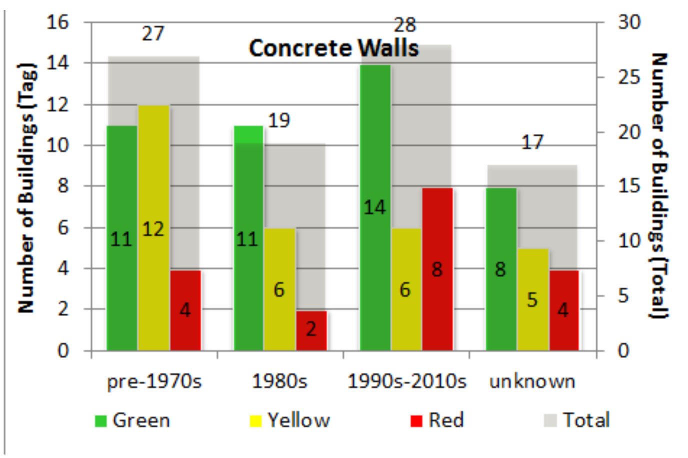


Figure 33: Seven-storey 1980s office block with significant compression failure of the V-shaped RC shear wall.

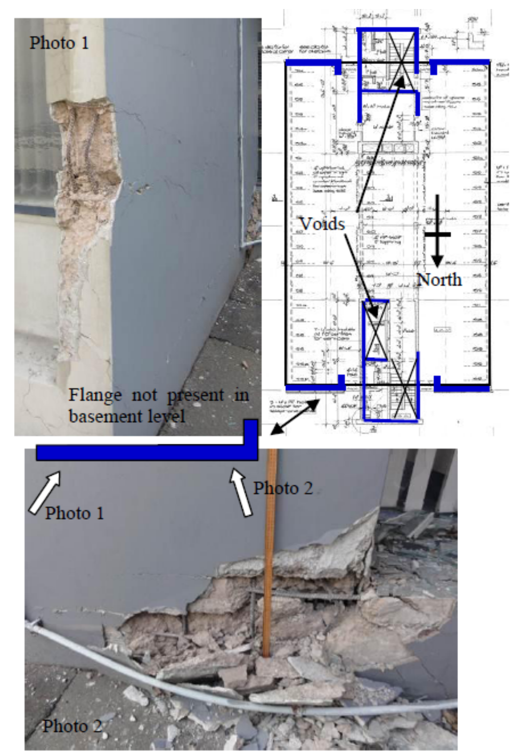


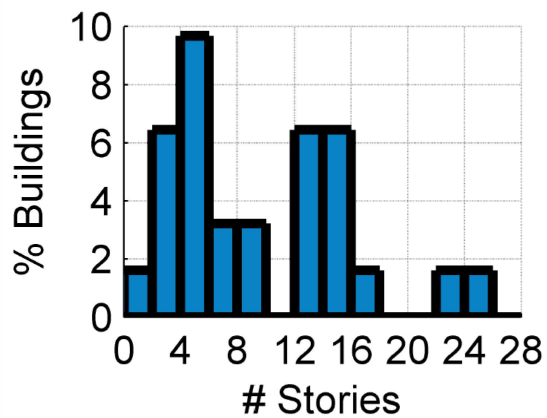
Figure 17: Wall boundary compression zone crushing and buckling failure.

(Figures from Kim, Pampanin and Elwood 2011)

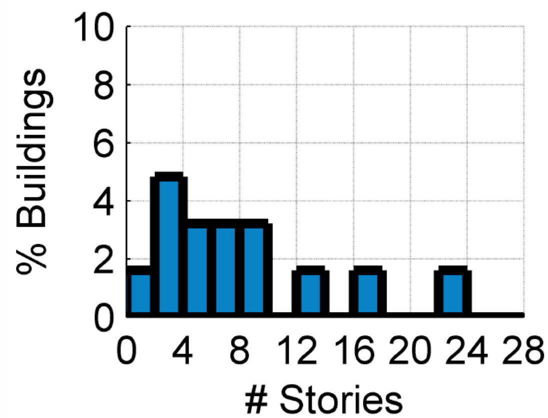
Earthquake Damage to Walled Buildings in the Past

- Consider earthquake reconnaissance data from 22 earthquakes around the world going back as far as the 1957 Mexico City earthquake.
- Reports document damage to 97 walled buildings.

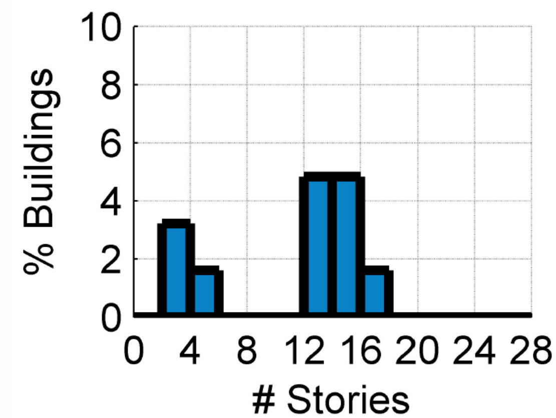
Failure Mode versus Building Height



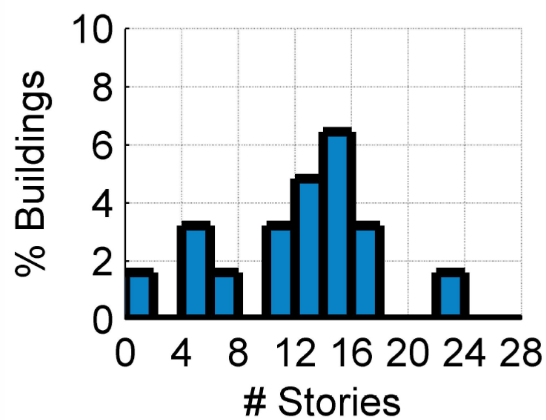
(a) B.E. compressive failure



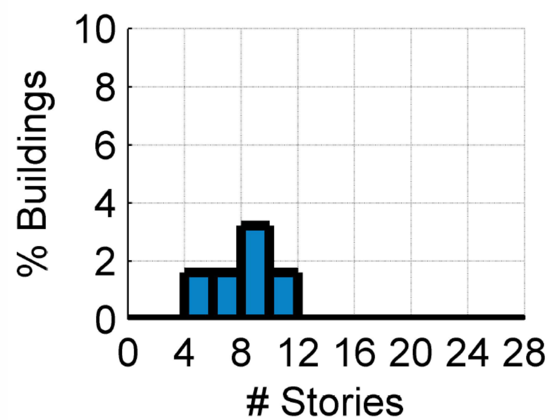
(b) Diagonal shear failure



(c) Web crushing failure



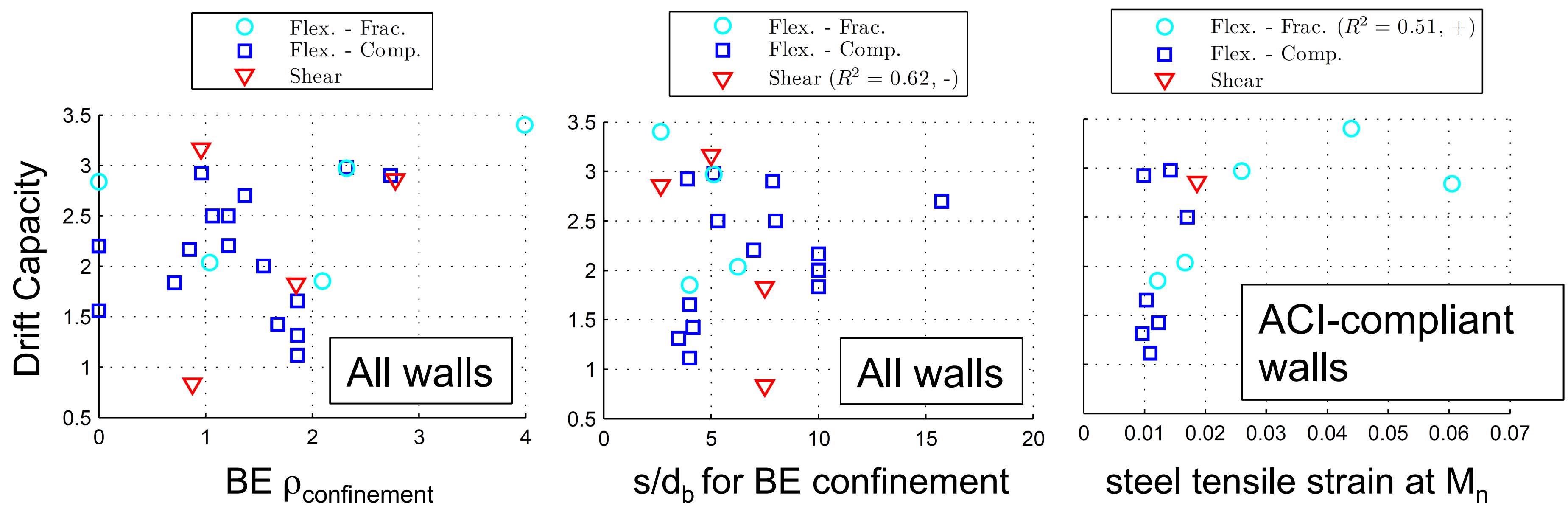
(d) Horizontal failure plane



(e) Collapse

Damage to Slender Walls Tested in the Laboratory

- Approximately 50% of ACI-compliant walls fail in compression.
- ACI-compliant walls fail in compression even if steel strain at M_n is well in excess of 0.005.
- Drift capacity not correlated with confinement ratio or s/d_b



Implications for the Analyst

- Concrete walls, both modern and older, may exhibit brittle compression-controlled failure at relatively low drift levels.

- Thus, assessment of wall performance requires accurate simulation of this failure mechanism.

Research Questions

1. For walls and walled buildings designed using current US Codes and standards of practice?
   - What is the expected failure mode for a wall? Flexure or shear? Compression- or tension-controlled flexure?
   - What is the collapse risk for a walled building for various levels of earthquake demand?
2. How can we improve design to achieved desired performance:
   - Achieve desired failure mechanism.
   - Achieve acceptable collapse risk.

Research Process

1. Develop a numerical modeling approach for slender concrete walls that enables accurate simulation of response through failure, **including accurate simulation of failure mode and drift capacity.**
2. Use this model to
   - Evaluate the earthquake performance of concrete walled building designed using current US design codes.
   - Develop recommendations to improve wall design.

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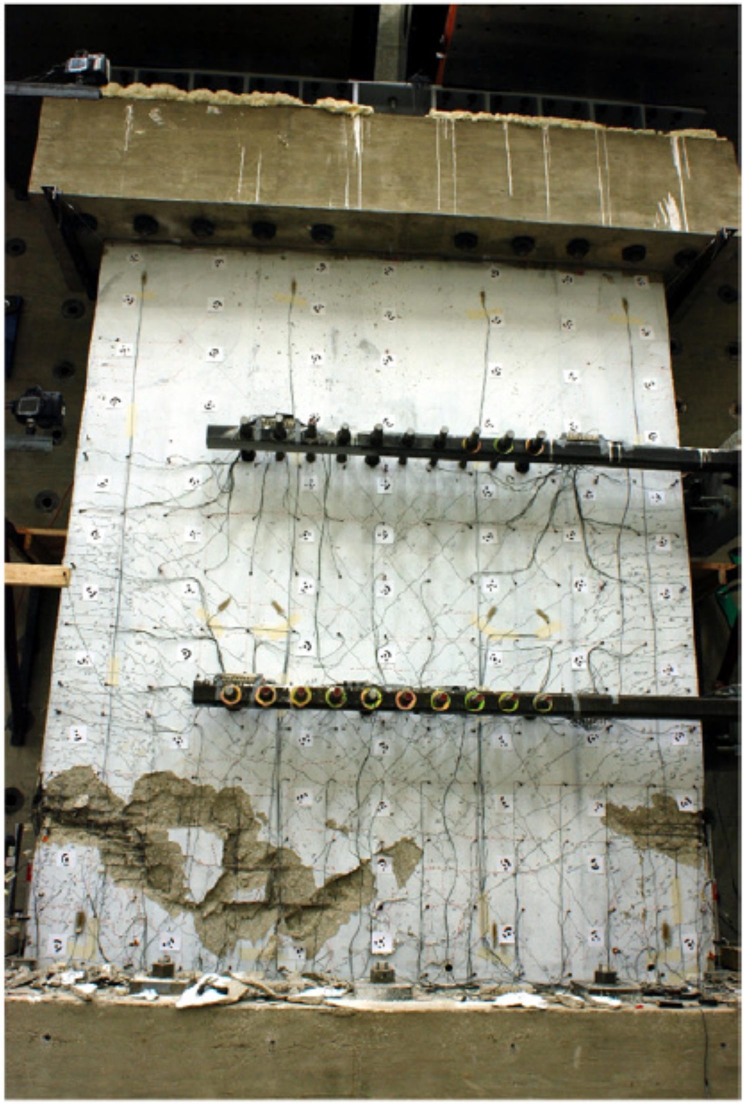
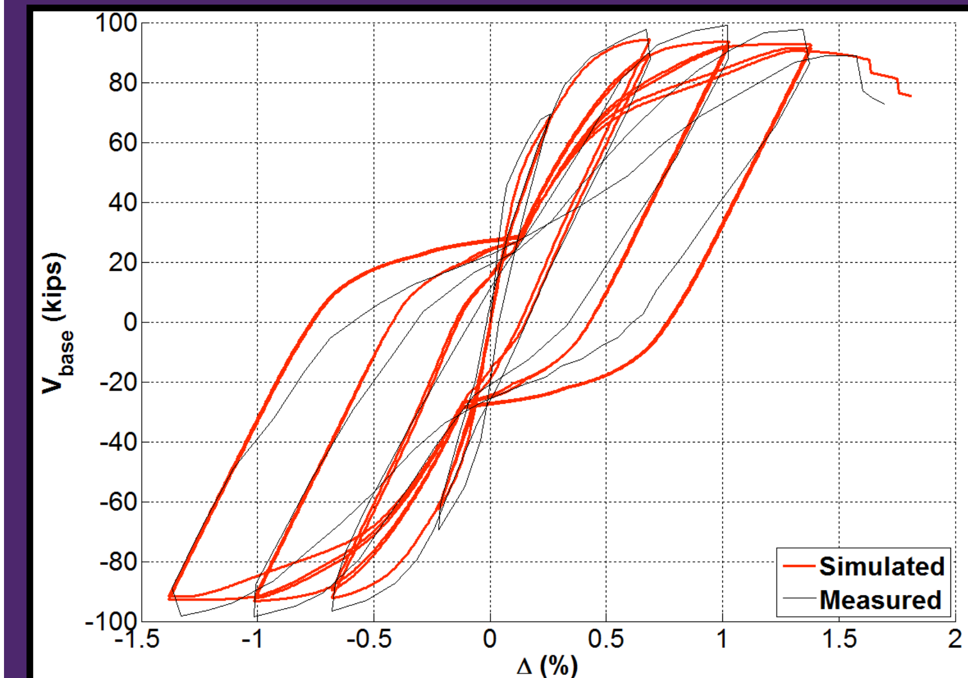
Using OpenSees to Simulate Wall Response





Modeling the Earthquake Response of Concrete Wall Buildings

- Objectives
  - Accurate simulation of response
  - Computational efficiency and robustness
  - Script-based input to facilitate parameter studies
- Models considered
  - Continuum (Abaqus, VecTor2,ATENA): too computationally demanding for system analyses, not numerically robust (Abaqus), no script-based input (VecTor2).
  - PERFORM 3D Fiber Wall Element: too computationally demanding, no script-based input.
  - Line-element models with flexure/shear interaction: inaccurate/not calibrated for a range of wall designs OR not readily available for use.
  - OpenSees fiber-type beam-column elements with distributed plasticity: greatest potential to meet all modeling objectives

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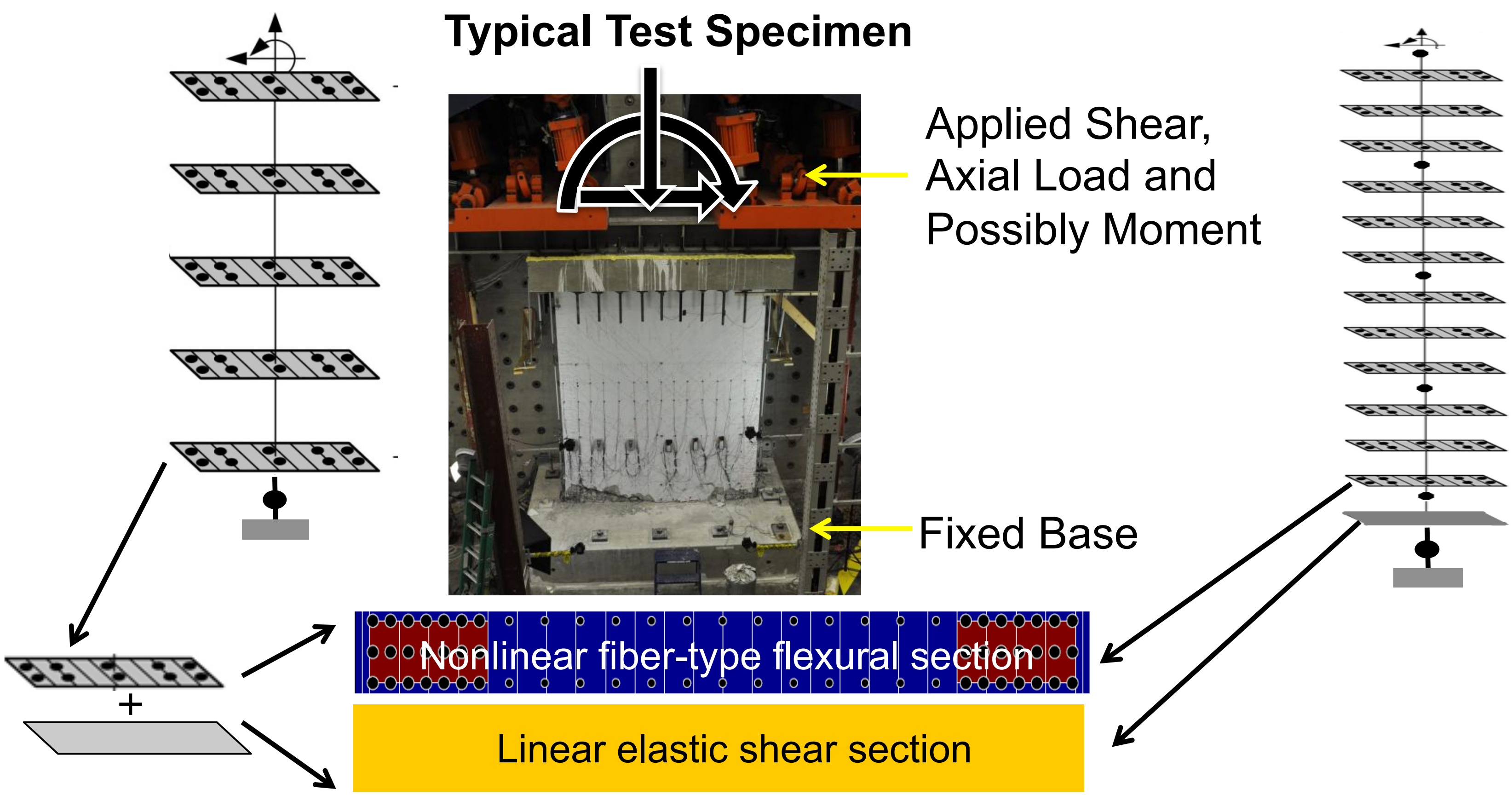
Evaluating Fiber-Type Beam-Column Elements for Modeling Wall Response

- **Force-Based Element:**
- **Displacement-Based Element:**



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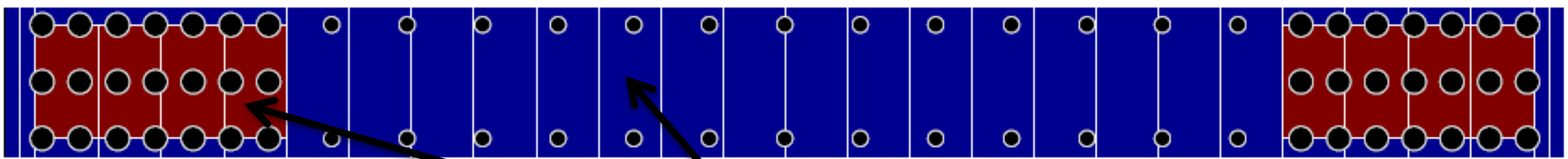
Fiber Section:
Concrete 02 model used for concrete

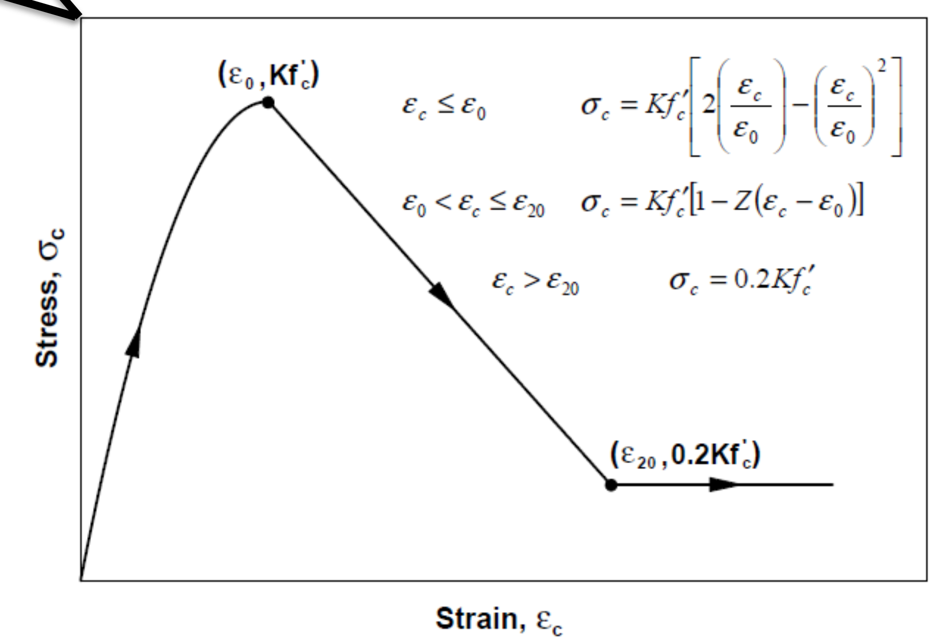


Unconfined Fibers:

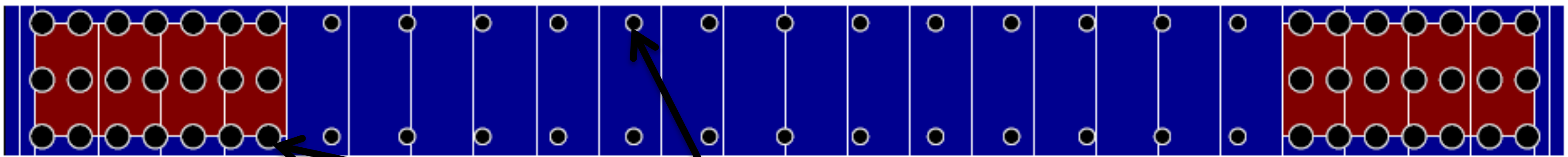
$$\varepsilon_0 = \frac{2f_c'}{57000\sqrt{f_c'}} \qquad \varepsilon_{20} = 0.008$$

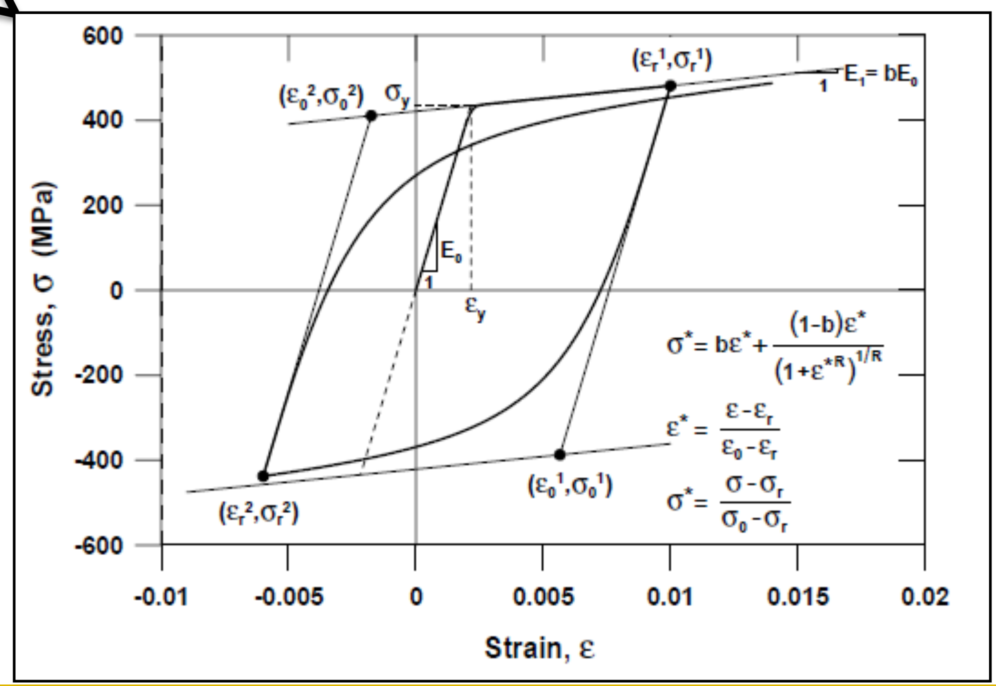
Confined Fibers:

$$\left.\begin{array}{l} K \\ \varepsilon_0 \\ \varepsilon_{20} \end{array}\right\}$$ ***Saatcioglu & Razvi (1992)***



Fiber Section:
Steel 02 used for reinforcing steel

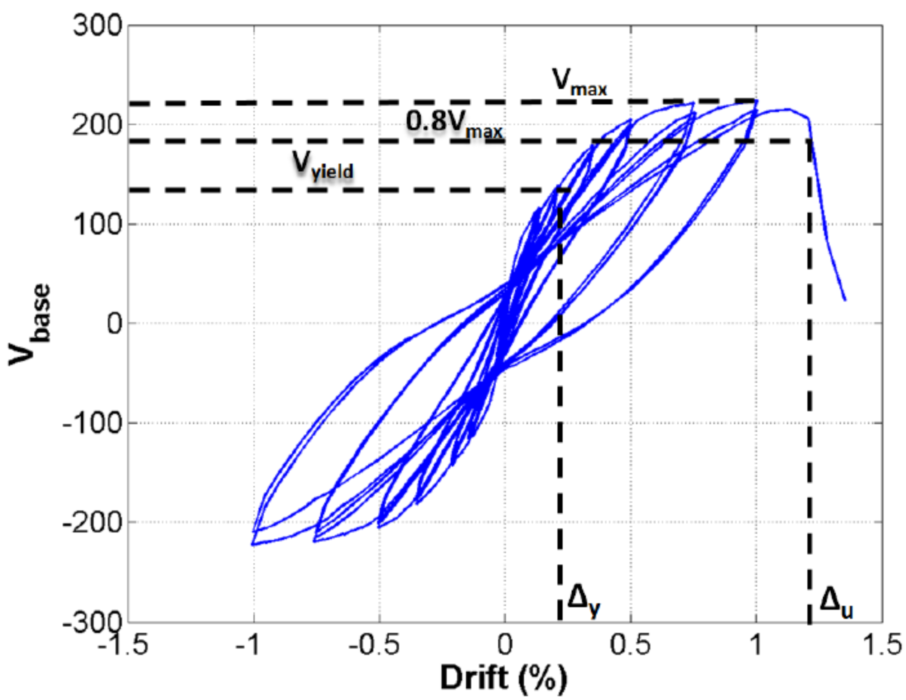




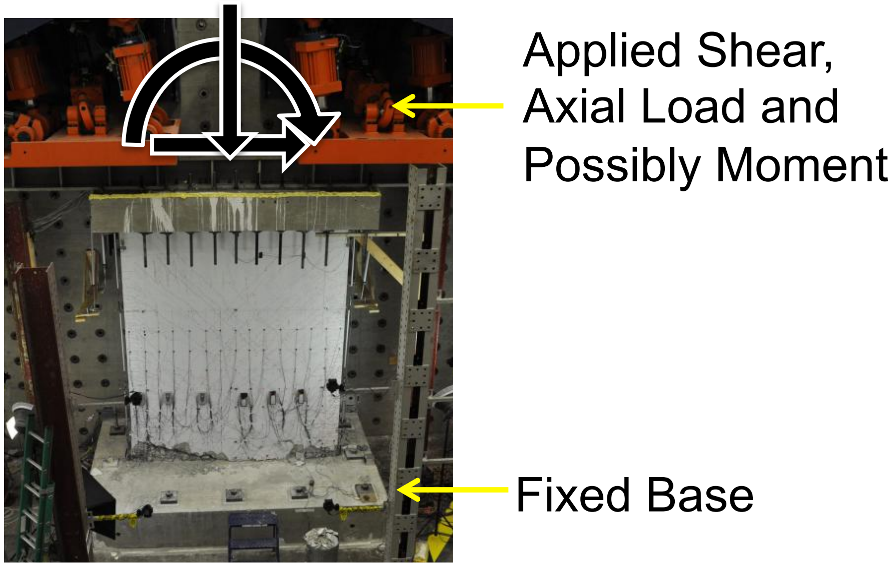
Experimental Data Used for Model Evaluation, Calibration & Validation

- 19 rectangular, 3 barbell, 6 c-shape, 4 t-shaped specimens from 10 test programs
- All walls are slender with $(M/V)/l_w > 2$
- All walls exhibit flexural failure mechanisms
  - Crushing of boundary-element concrete, buckling and/or rupture of long. reinforcement
  - Walls exhibiting web crushing (barbell walls) not included
- All wall have scale = t_w/12 in. > 1/3
- Axial load ratios: $0.01f_cA_g$ - $0.16f_cA_g$
- Shear stress demands: $1.0\sqrt{f_c}\,A_{cv} - 6.0\sqrt{f_c}\,A_{cv}$ psi

Quantities Used for Model Evaluation, Calibration & Validation



Typical Test Specimen

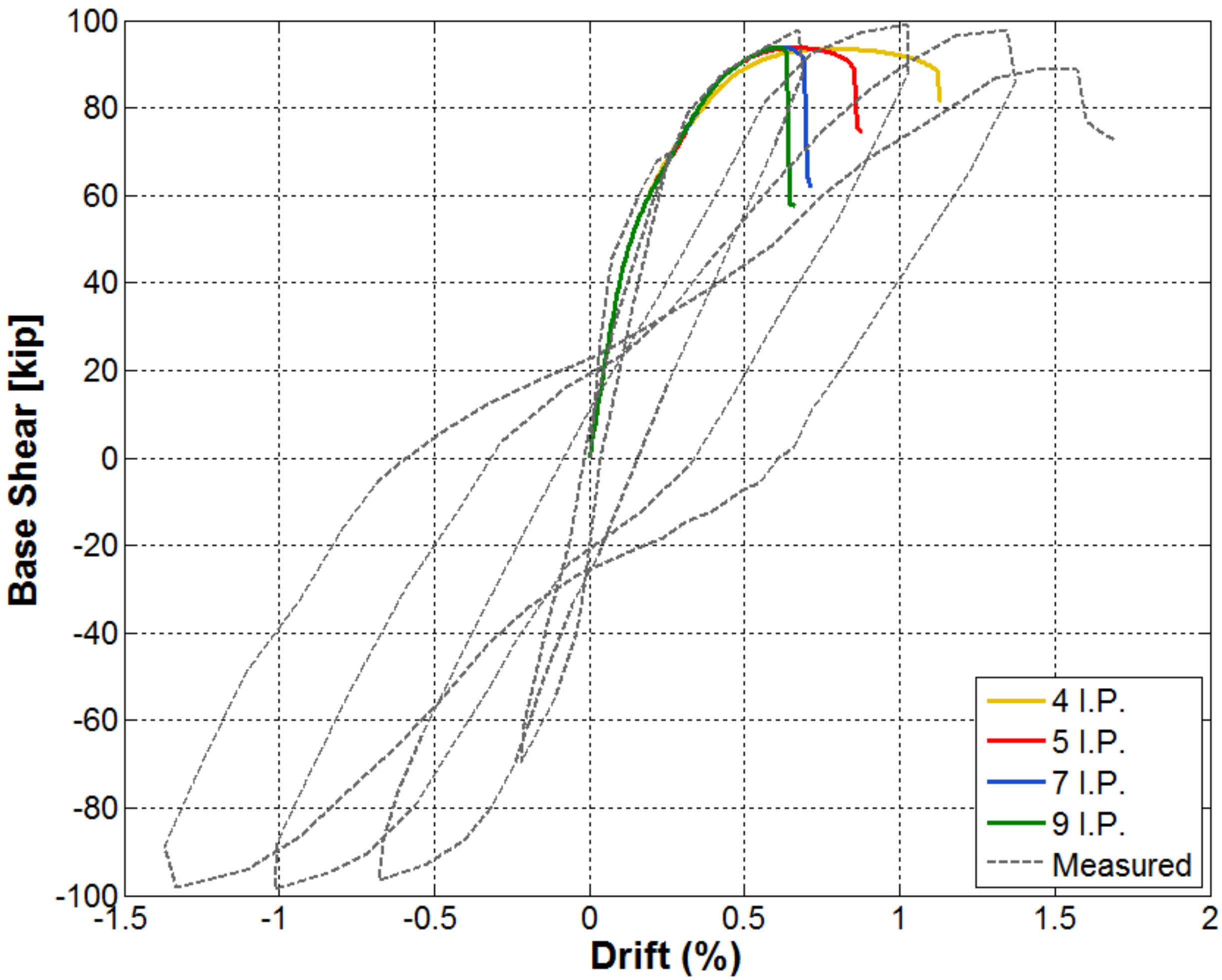


Force-Based Distributed-Plasticity Beam-Column Element:

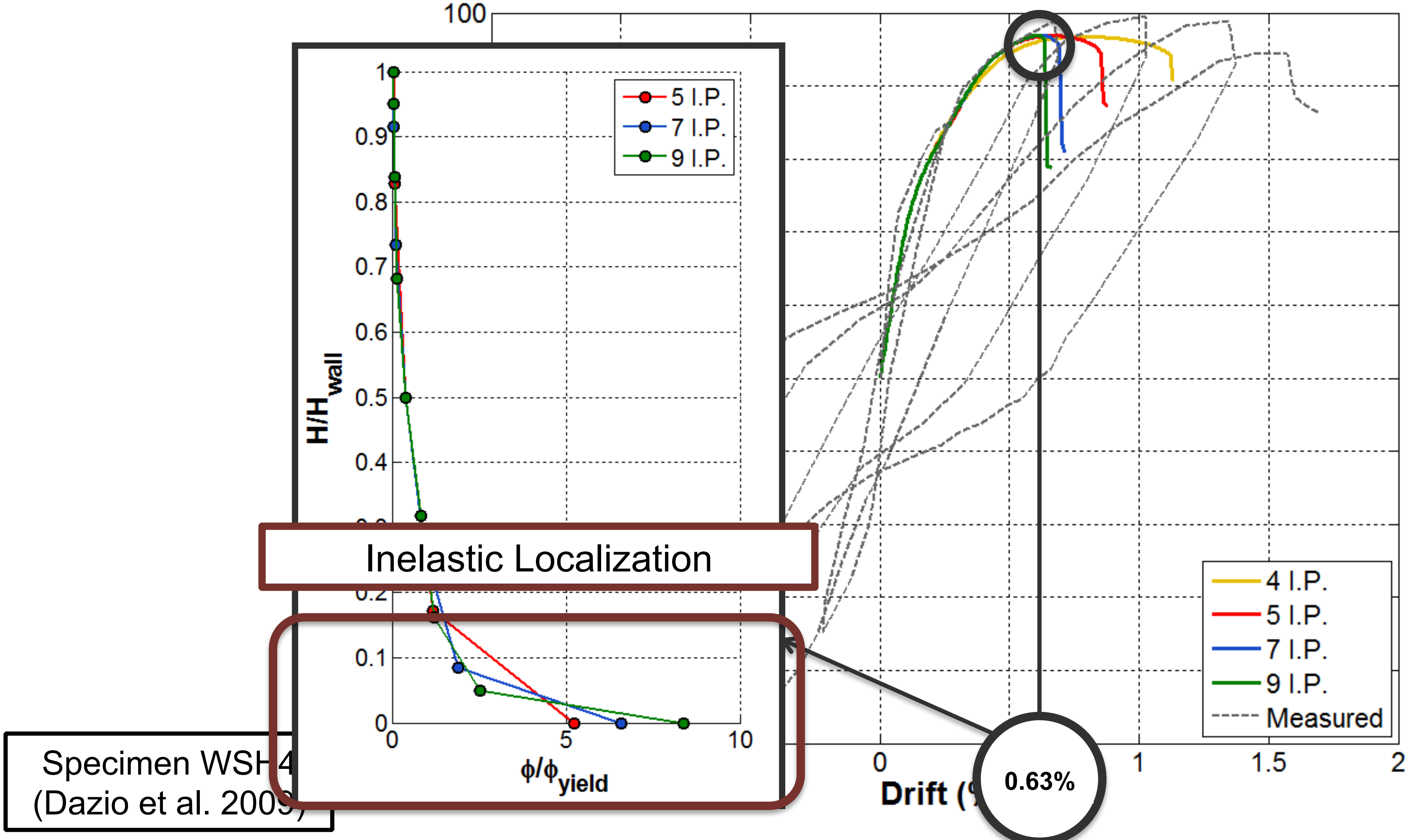
Evaluation, Calibration and Validation

Model Evaluation

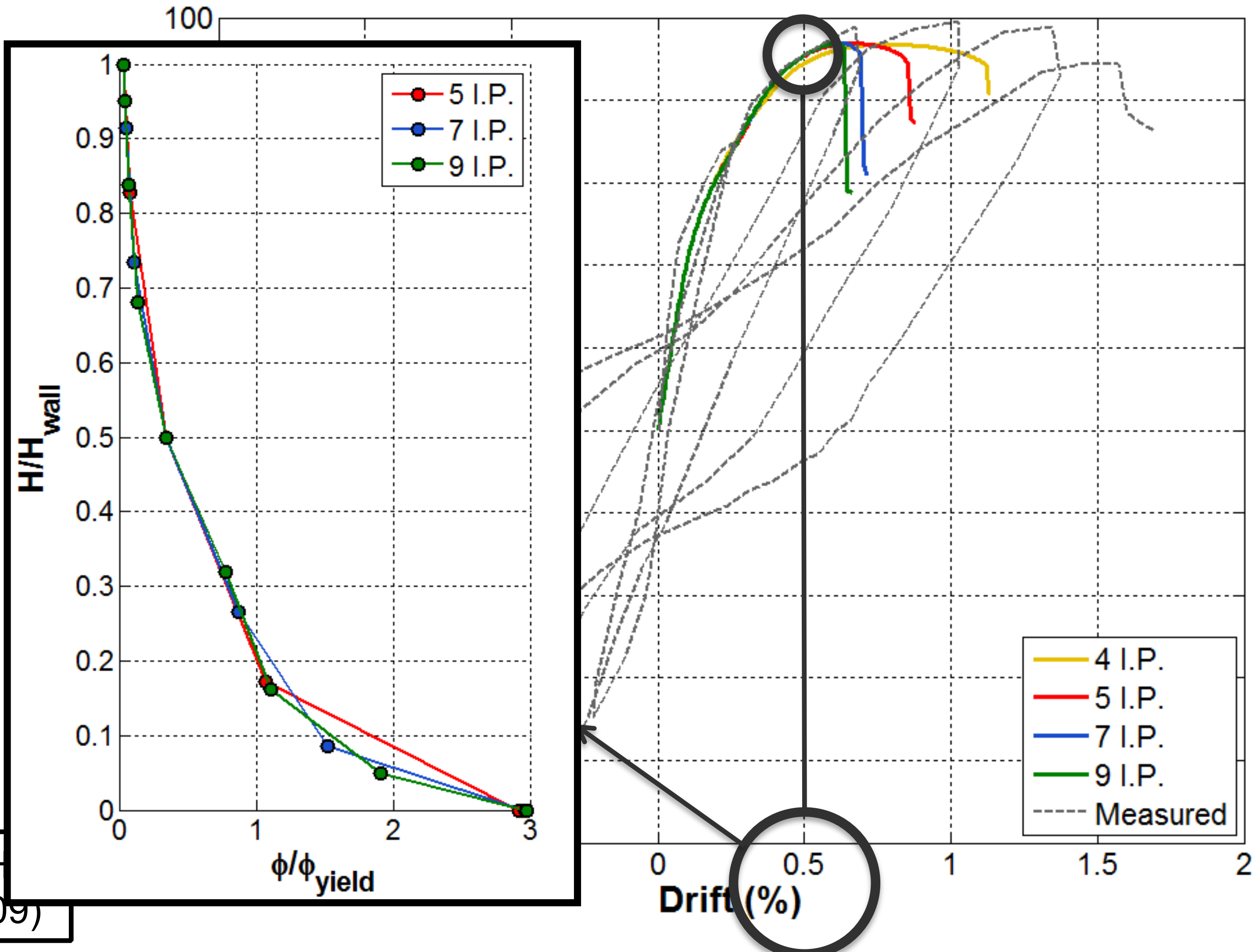
No. of I.P.	No. of Spec's	Me
3	23	0.9
5	23	0.9
7	23	0.9



Localization of Damage / Deformation



No Localization Prior to Strength Loss



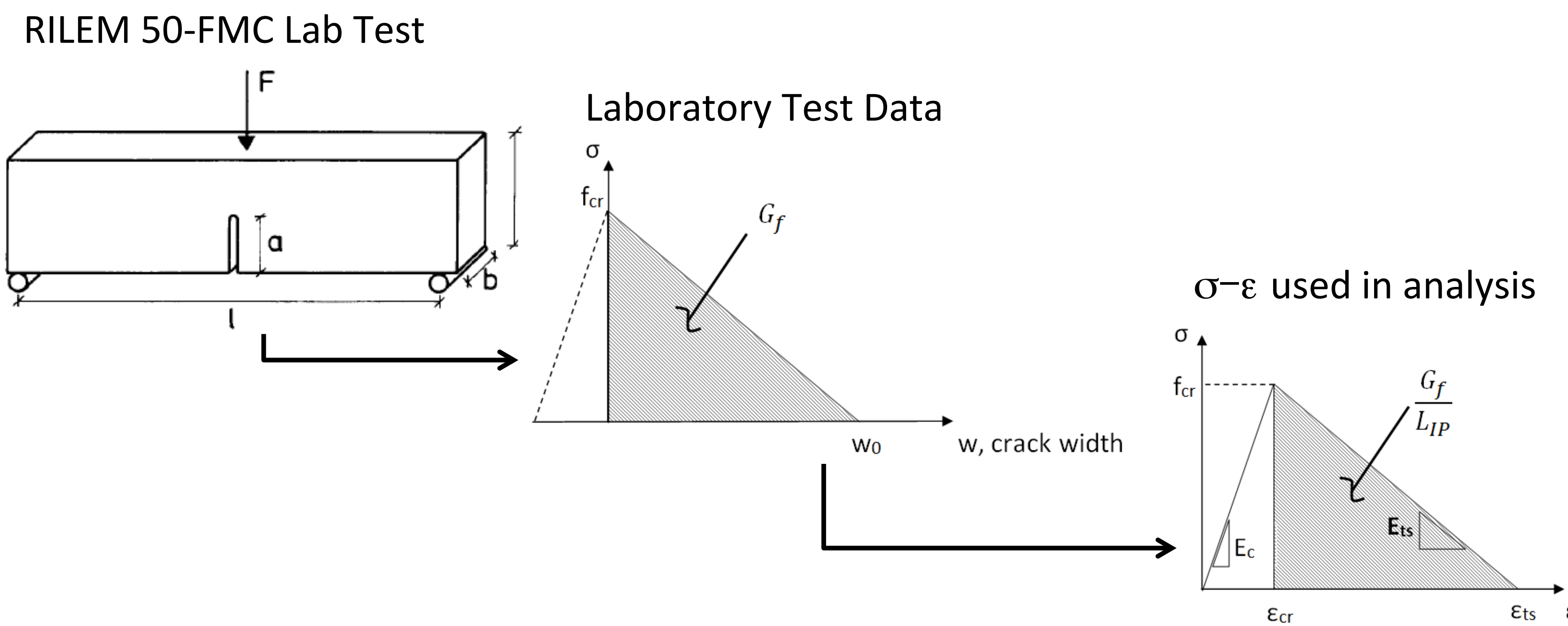
Specimen WSH
(Dazio et al. 2009)

To Achieve Mesh-Objective Results

- Regularize material response using a mesh-dependent length
- Typically done in continuum analysis
- Coleman and Spacone (2001) propose this for beam-column elements;
- To regularize
  - **Concrete:** Use experimental data to define energy under post-peak portion of the stress-deformation curve & convert stress-deformation to stress-strain using integration-point length, L_{IP}
  - **Steel:** Use experimental data to define stress-strain response and adjust post-peak strength strain response based on ratio of laboratory gage length to integration-point length, L_{IP}
- Note that regularization of steel hardening response req'd because deformation localizes to softening section

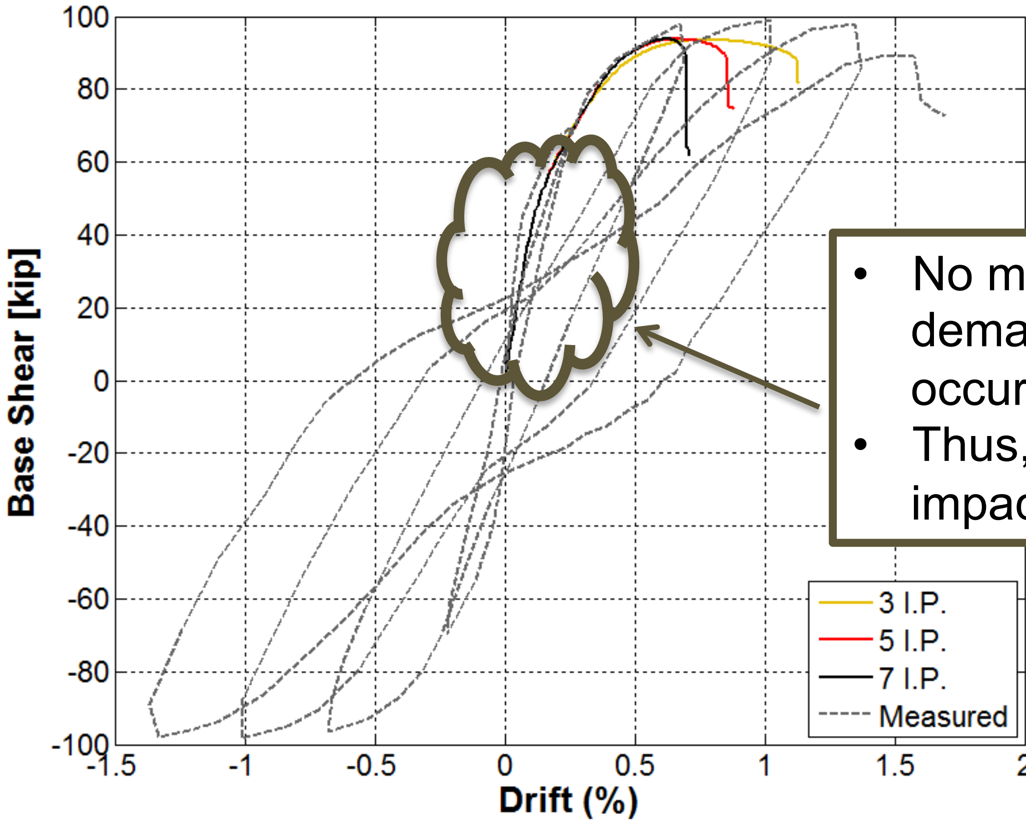
Concrete Tensile Fracture Energy

- Tensile fracture energy, G_f, commonly used to regularize material response for continuum-type finite element analysis
- Several "standard" approaches for defining G_f (e.g., RILEM 50-FMC)
- $G_f \approx$ 75-150 N/m (Wong and Vecchio, 2006)



Concrete Material Regularization Using G_f

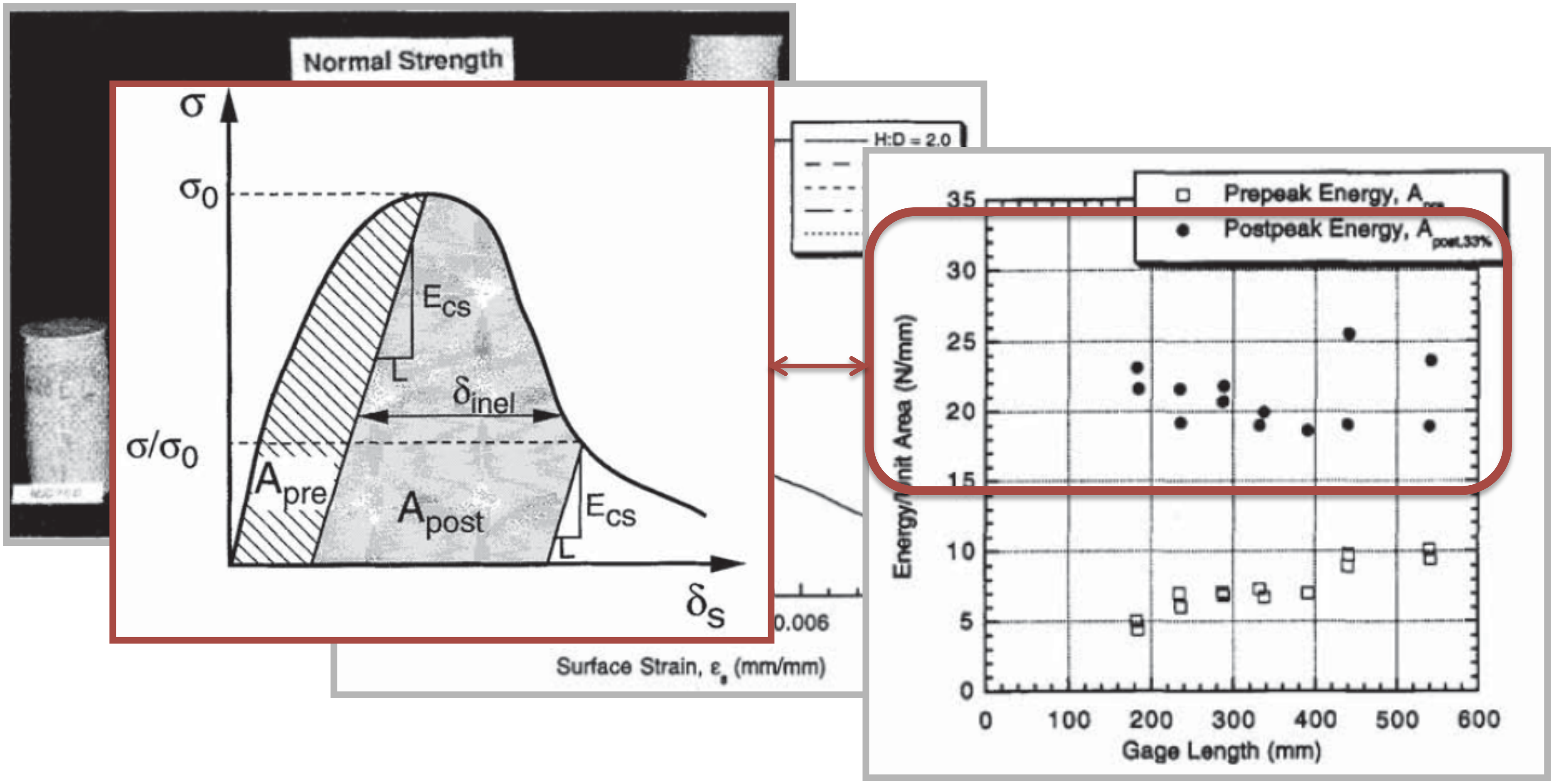
- Has essentially no impact; therefore ignore



- No mesh sensitivity in range of demands in which concrete cracking occurs
- Thus, material regularization has no impact

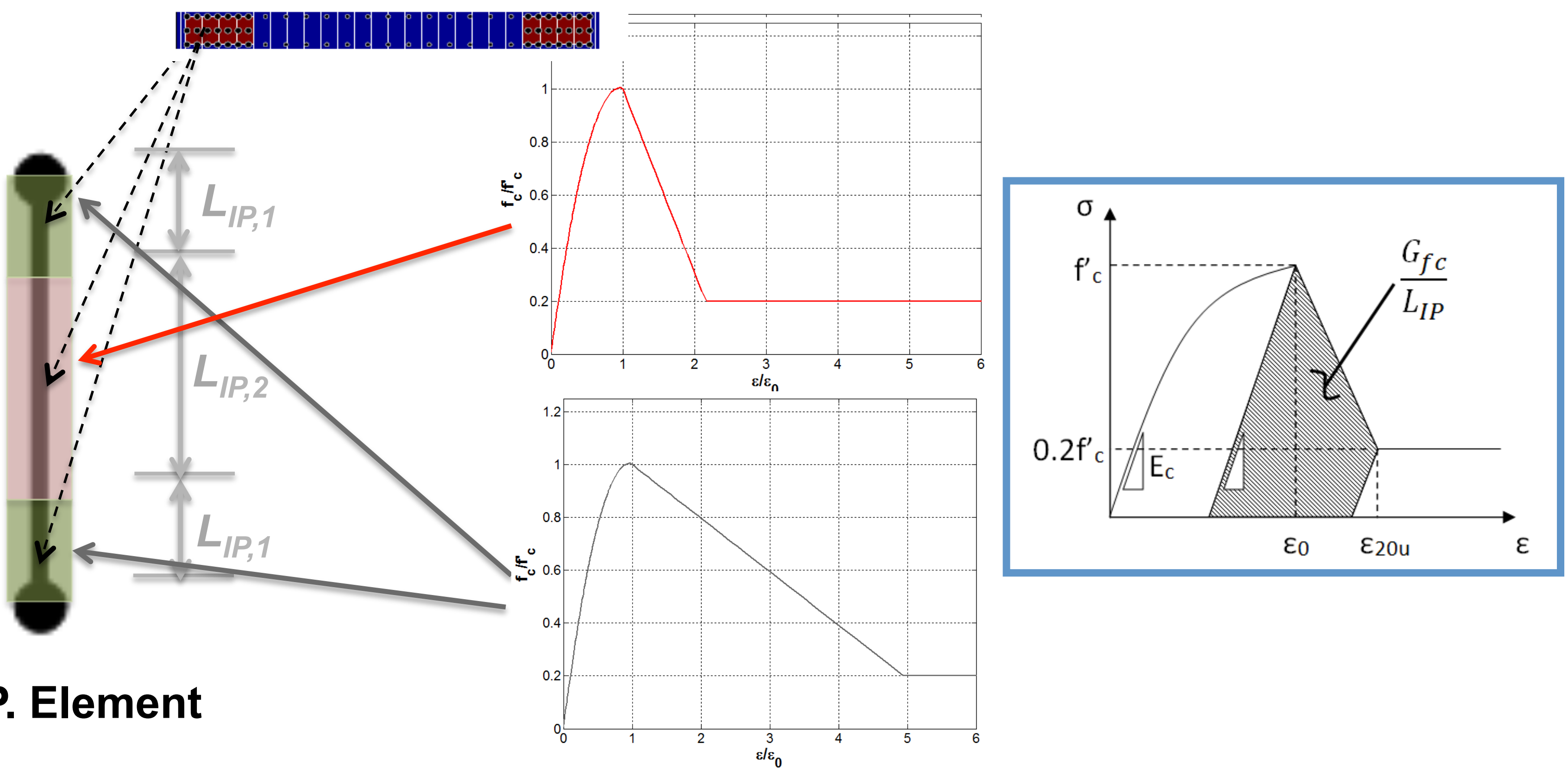
Plain Concrete Crushing Energy

- Jansen and Shah, 1997



Material Regularization: Plain Concrete

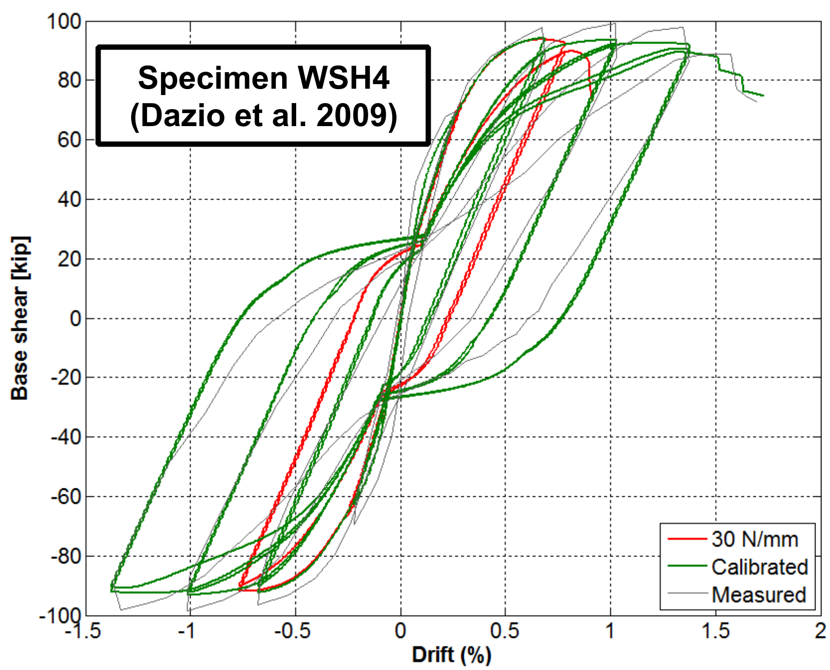
- Crushing energy, G_{fc} = ~20 N/mm per Jansen and Shah (1997)

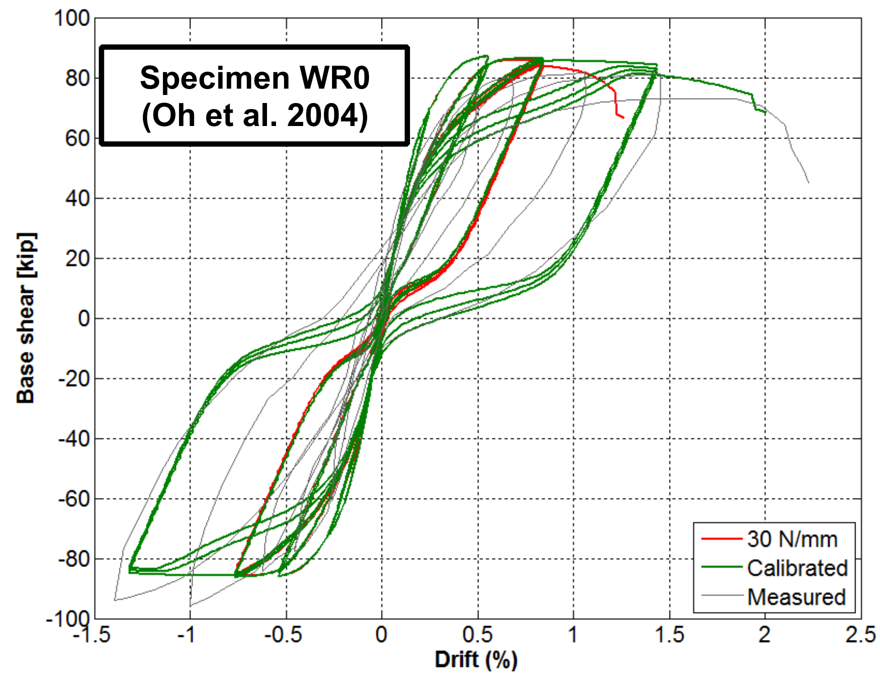


3-I.P. Element

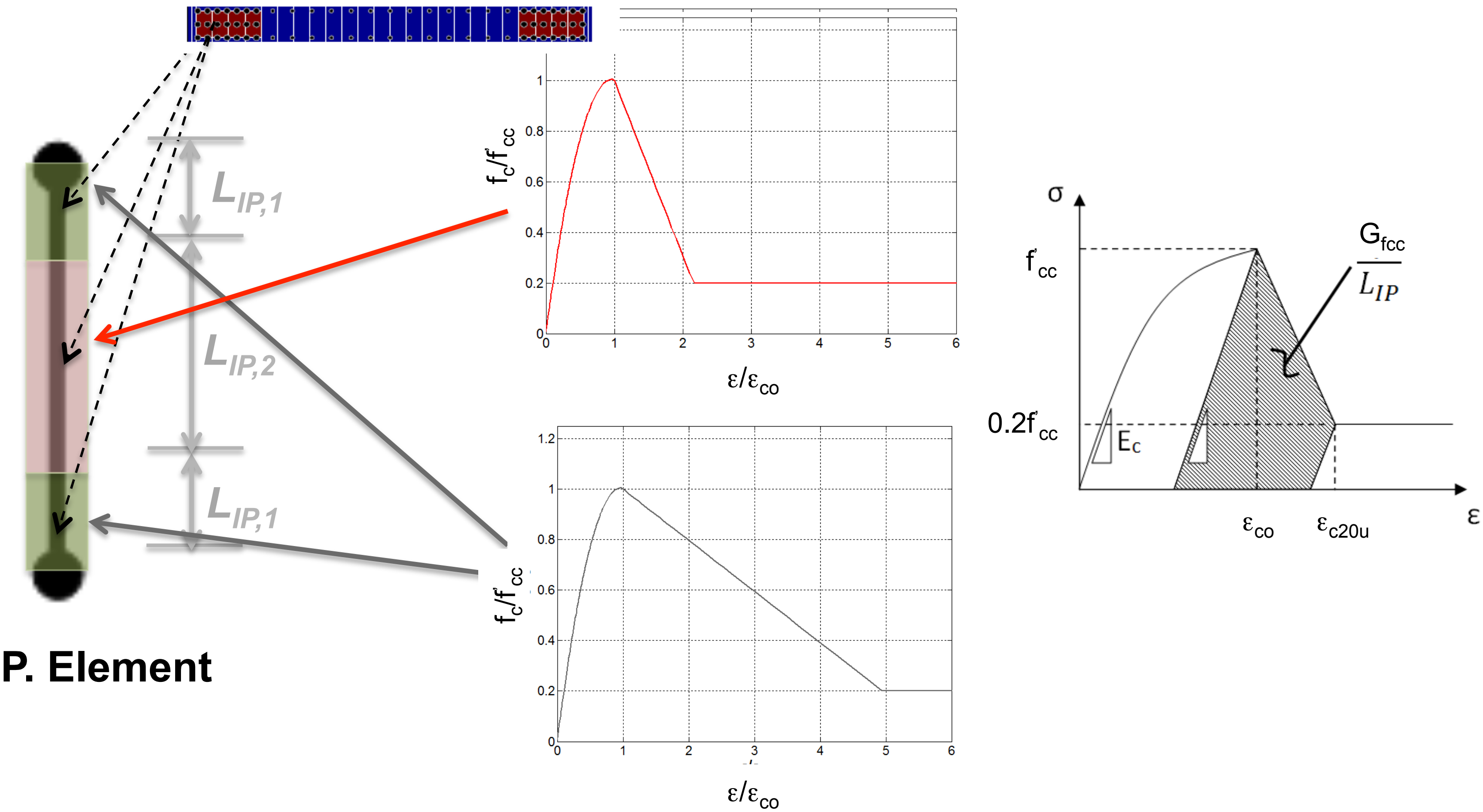
Determine Required G_{fc}

- Use experimental data for two planar walls constructed of unconfined concrete and exhibiting flexural failure due to concrete crushing
- G_{fc} = 60 – 80 N/mm = $2f_c$ with f_c in MPa
- Note that increase in G_{fc} above Jansen and Shah 20 N/mm for plain concrete cylinders is attributed to the presence of longitudinal steel





Material Regularization: Conf. Concrete



3-I.P. Element

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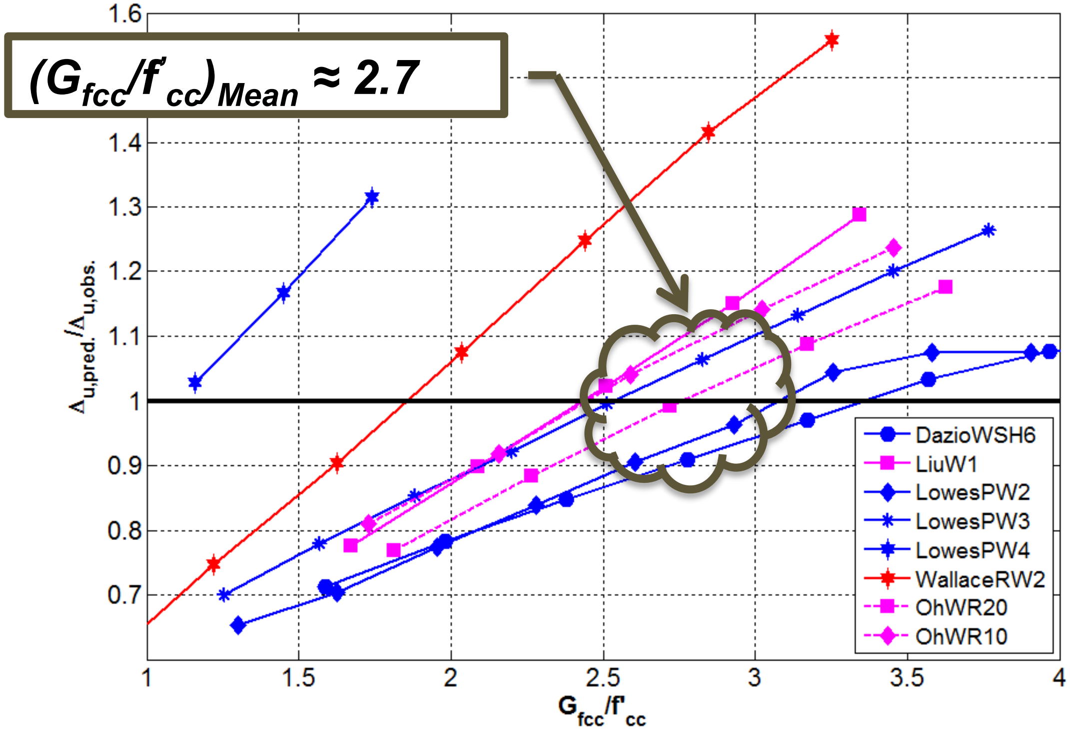
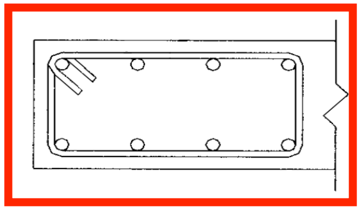
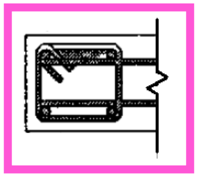
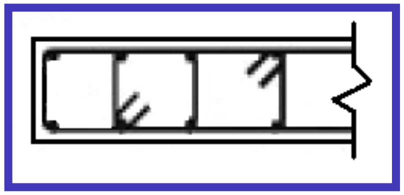
Determine Required G_{fcc}

- Use experimental data for eight planar walls w/ confined concrete exhibiting flexural failure due to concrete crushing
- G_{fcc} appears to be a function of confinement detailing, but insufficient data for model calibration



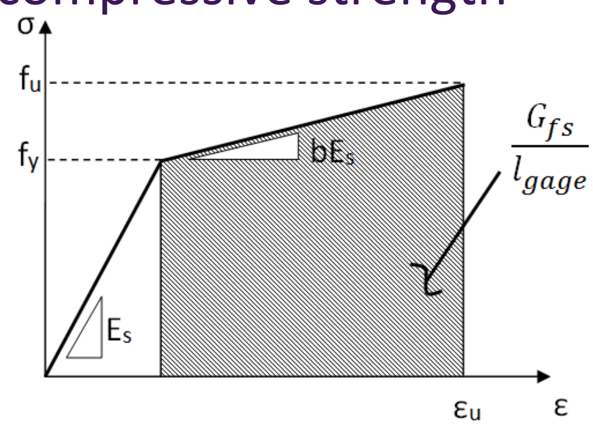




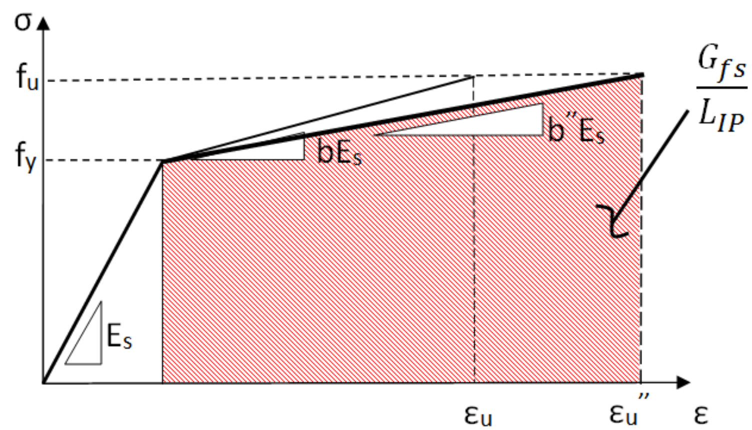


Material Regularization: Steel

- Required despite steel hardening because deformations localize to single softening section
- G_{fs} / l_{gage} determined from material tests
- Regularized steel stress-strain response determined by L_{IP}
- Regularization results in adjusted tensile rupture strain
- To simulated strength loss due to buckling, include compressive failure strain equal to strain at which concrete loses 80% of compressive strength



Response from lab data



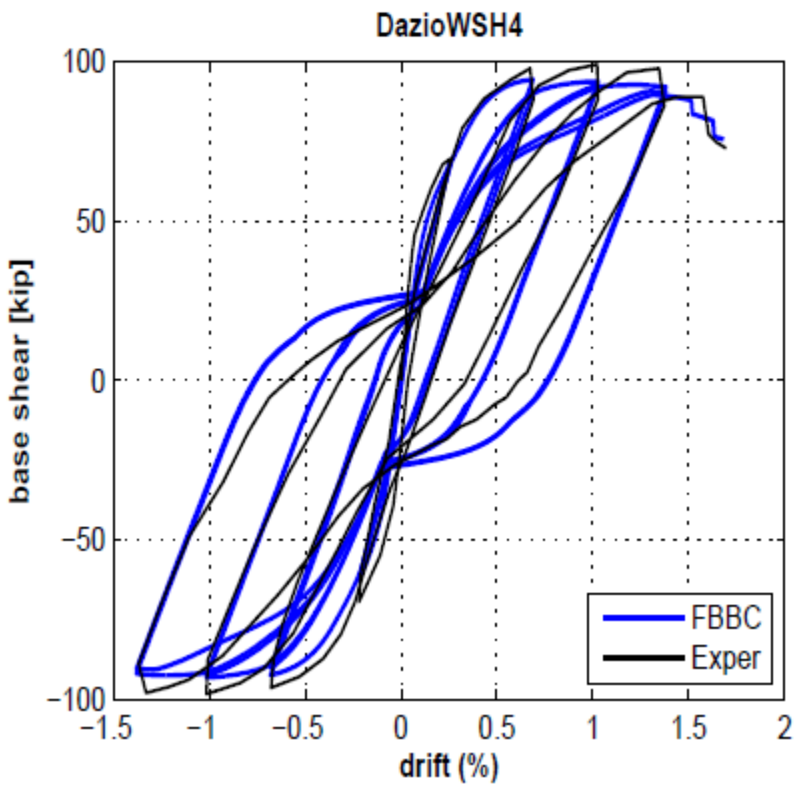
Regularized response model

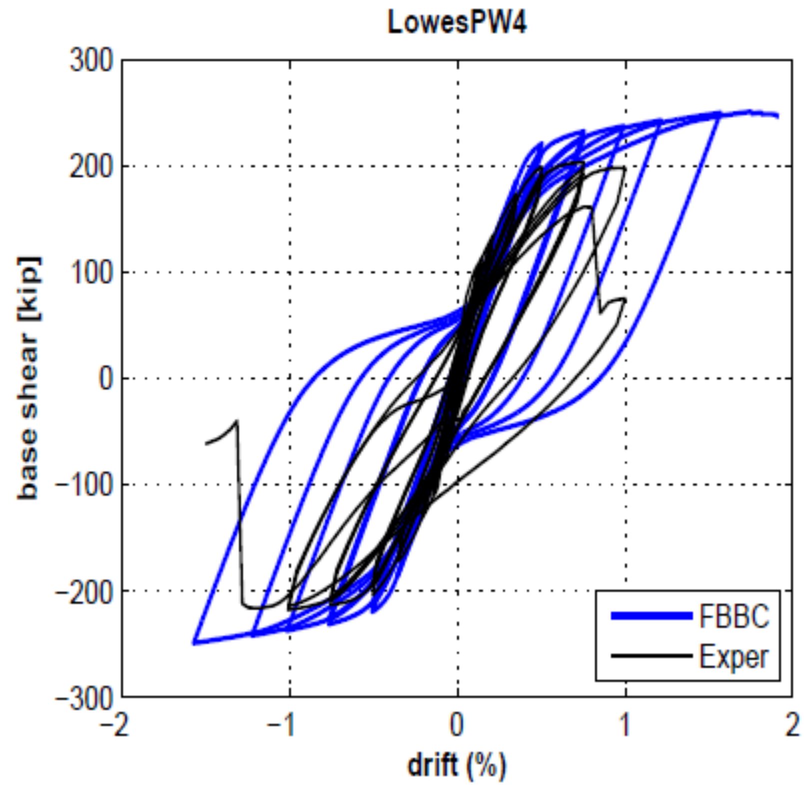
FBBC: Regularized Results for Planar Walls

	$\frac{V_{max,sim.}}{V_{max}}$		$\frac{\Delta_{yield,sim.}}{\Delta_{yield}}$		$\frac{\Delta_{u,sim.}}{\Delta_{u}}$	
Failure Mode	**Mean**	**COV**	**Mean**	**COV**	**Mean**	**COV**
Crushing (9 specimens)	**0.93**	**0.04**	**0.83**	**0.26**	**0.96**	**0.15**
Rupture/Buckling (6 specimens)	**0.95**	**0.05**	**1.01**	**0.33**	**1.12**	**0.21**
Rupture (2 specimens)	**0.98**	**0.03**	**0.94**	**0.02**	**1.08**	**0.04**
Out of Plane (2 specimens)	**0.98**	**0.03**	**0.94**	**0.28**	**1.31**	**0.08**
All Flexure	**0.95**	**0.07**	**0.90**	**0.28**	**1.06**	**0.22**

Regularized Results: Planar Walls

- Good results: WSH4 Dazio et al.
- Not so good results: PW4 Lowes et al.





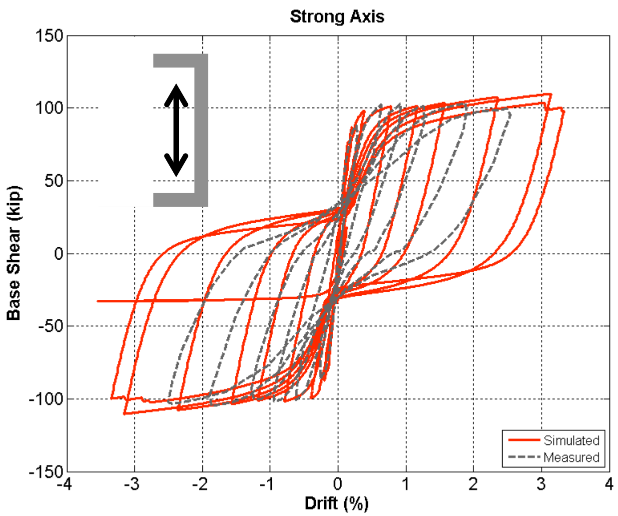
Regularized Results: C-Shaped Walls

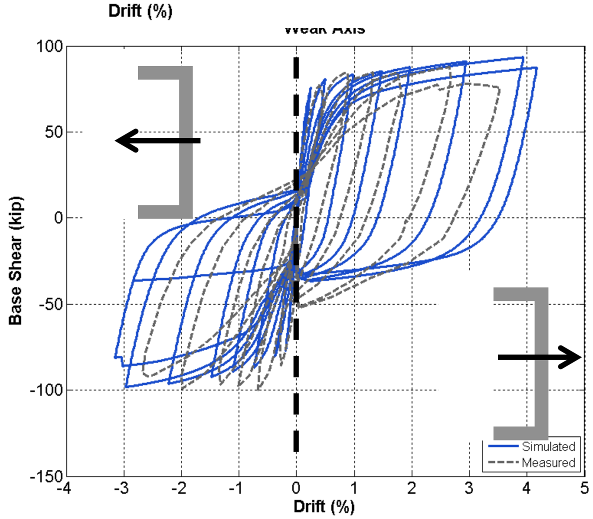
- Apply regularization method calibrated for planar walls to C-shaped walls:

Specimen	Loading	$\frac{V_{max,sim.}}{V_{max}}$	$\frac{\Delta_{yield,sim.}}{\Delta_{yield}}$	$\frac{\Delta_{u,sim.}}{\Delta_{u}}$
UW1 (Lowes et al.)	Strong Axis	1.01	1.13	1.20
W1 (Ile and Reynouard)	Strong Axis	0.90	0.85	1.00
W2 (Ile and Reynouard)	Weak Axis	0.94	0.87	0.77
W3 (Ile and Reynouard)	Bi-Directional	0.93	1.10	0.70
TUA (Beyer at al.)	Bi-Directional	1.06	0.90	1.04
TUB (Beyer et al.)	Bi-Directional	1.08	1.15	1.06
Mean (COV)		0.99 (0.08)	1.00 (0.14)	0.96 (0.20)

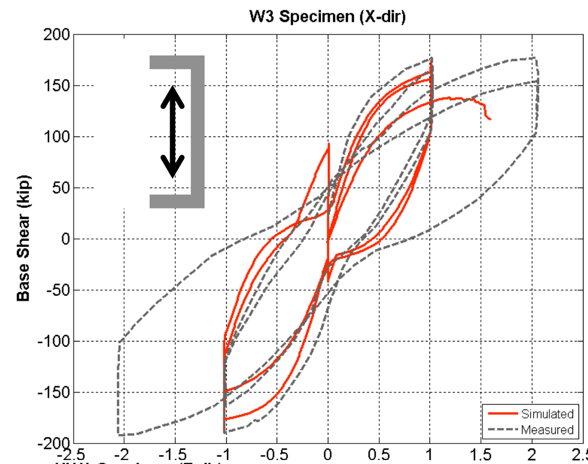
Regularized Results: C-Shaped Walls

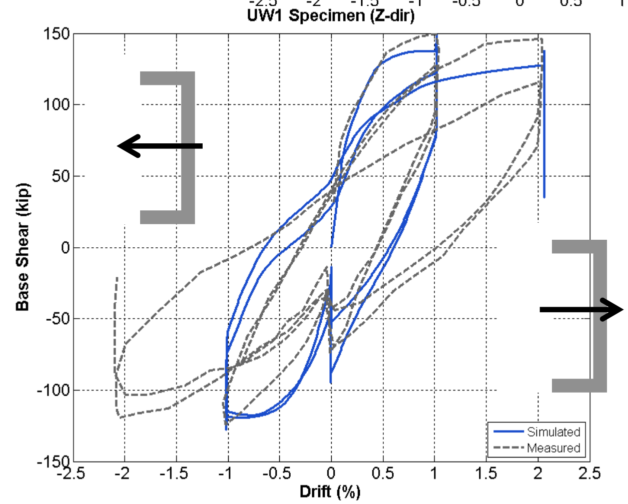
- Good: TUA Beyer et al.





- Not so good: W3 Ile and Reynouard





Regularized Results: T-Shaped Walls

- Apply regularization method calibrated for planar walls to T-shaped walls:

Specimen	Loading	$\frac{V_{max,sim.}}{V_{max}}$	$\frac{\Delta_{yield,sim.}}{\Delta_{yield}}$	$\frac{\Delta_{u,sim.}}{\Delta_u}$
TW1 (Thomsen and Wallace)	Uni-directional	1.25	2.4	0.42
TW2 (Thomsen and Wallace)	Uni-directional	1.00	1.6	0.45
NTW1 (Brueggen et al.)	Bi-Directional	1.00	1.14	0.86
NTW2 (Brueggen et al.)	Bi-Directional	0.95	1.05	0.82
Mean/COV		1.05/0.13	1.55/0.40	0.64/0.37

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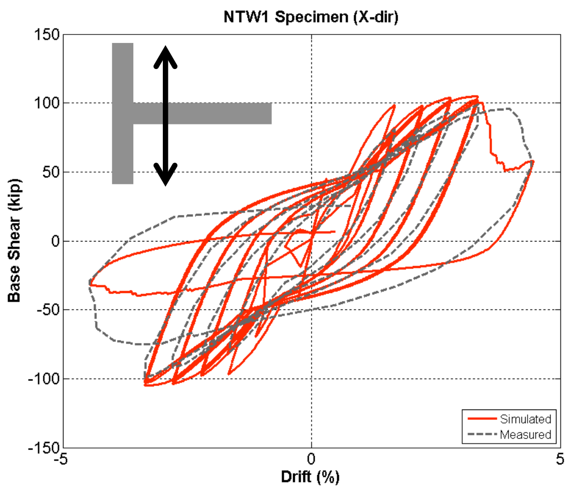
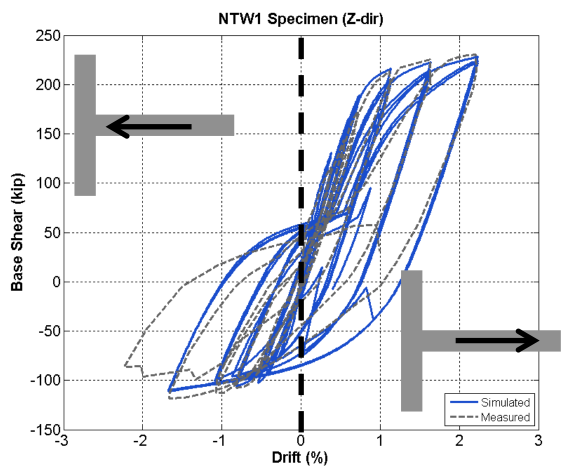
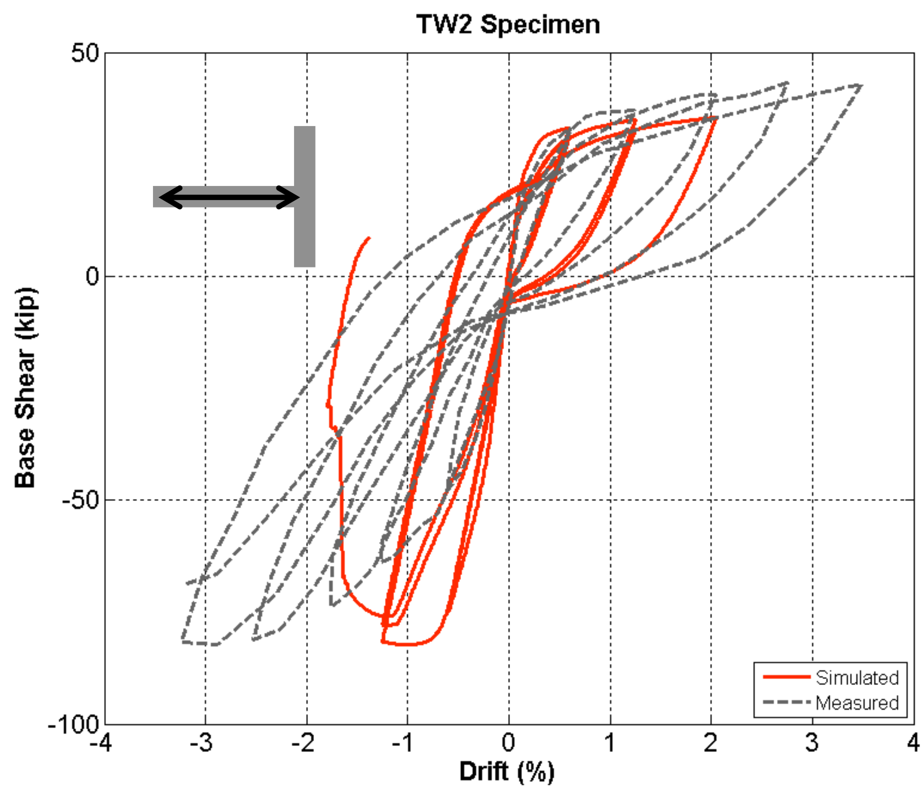
Regularized Results: T-Shaped Walls

- Good: NTW1 Brueggen et al.





- Not so good: Thomsen and Wallace
- Data show plane sections do not remain plane, so strain distribution is not correctly simulated

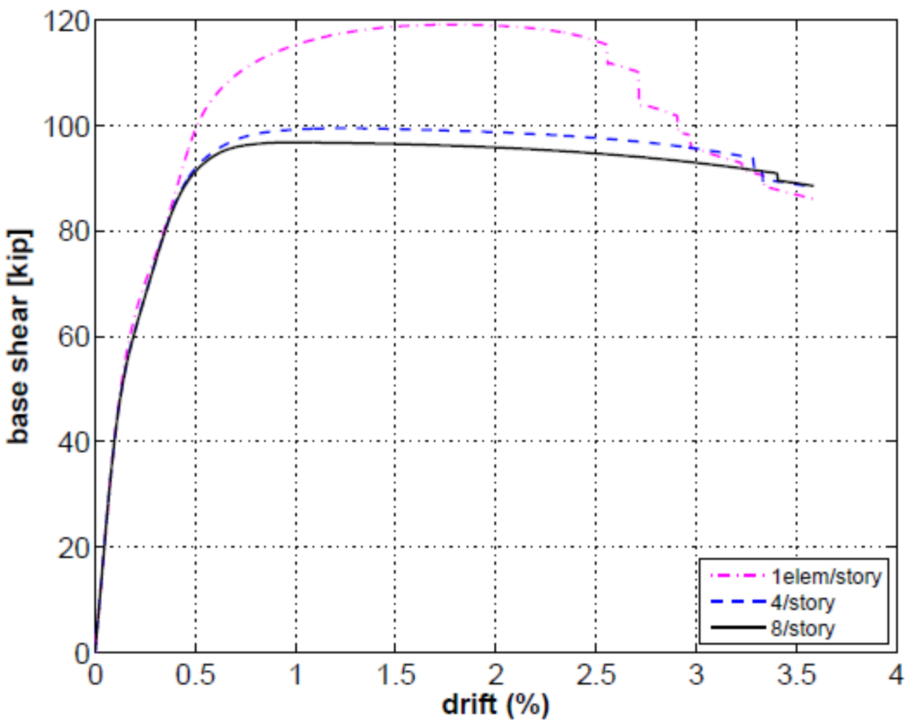


Displacement-Based Distributed-Plasticity Beam-Column Element:

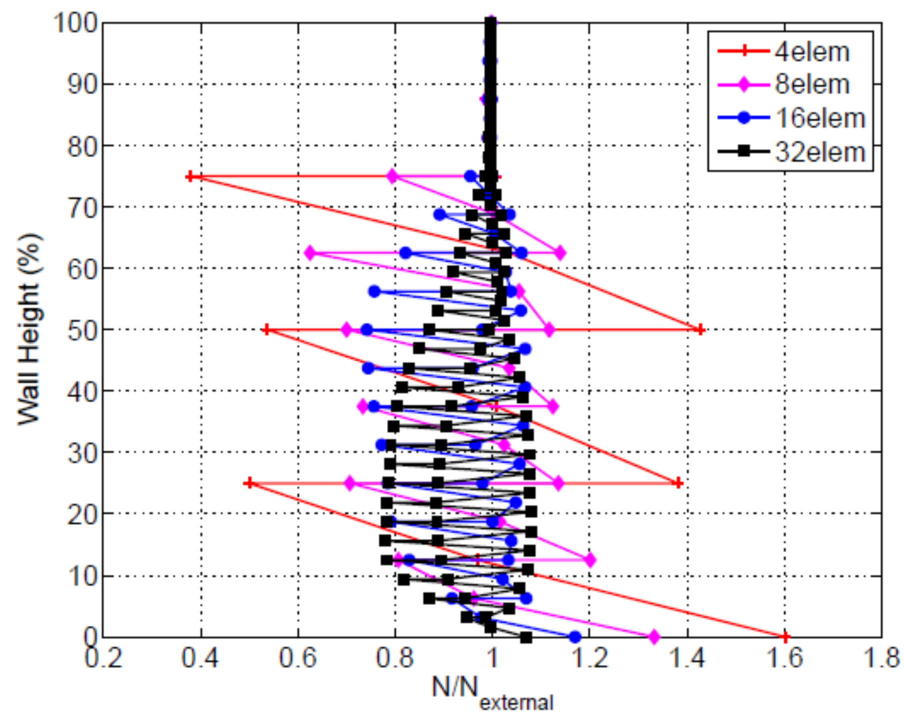
Evaluation, Calibration and Validation

Model Evaluation: Mesh Refinement Study

- Load-displacement response



- Axial load at the section (formulation assumes constant axial strain not



Impact of Axial Load Variation

- Critical (i.e. softening) section is located above the base of the wall and is not the section with highest demand:
  - Fiber section at the base of the wall has an axial load that is larger than the applied axial load; this results in increased flexural strength.
  - Fiber section above the base of the wall has an axial load that is smaller than the applied axial load; this results in reduced flexural strength.
- Accurate simulation of drift capacity requires modified G_{fc} and G_{fcc} to account for error in section axial load:
  - Unconfined: $G_{fc\_DBBE} = 0.28G_{fc\_FBBE}$
  - Confined: $G_{fcc\_DBBE} = 0.28G_{fcc\_FBBE}$
- Force-based element is preferred over disp.-based element to achieve accurate simulation of failure.

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Application of the Model to Advance Design of Walled Buildings

Application of the Model

- Use FEMA P695 Methodology to evaluate the earthquake performance of walled buildings designed using US codes.
  - **Approx. 2000 dynamic analyses:** ITHA (Incremental time-history analyses) of 8 building designs using suite of 44 ground motion records.
- To improve performance, 1) develop capacity-design procedure for shear and 2) recommend demand envelope for flexural design.
  - **Approx. 4500 dynamic analyses:** ITHA of 64 buildings using suite of 7 synthetic ground motions AND dynamic analysis of 96 building designs using suite of 14 synthetic ground motions.
- Use FEMA P695 Methodology to develop strength reduction factors (ASCE 7 R-factors) to achieve desired collapse risk.
  - **Approx. 1600 dynamic analyses:** ITHA of 6 buildings for suite of 44 ground motion records.

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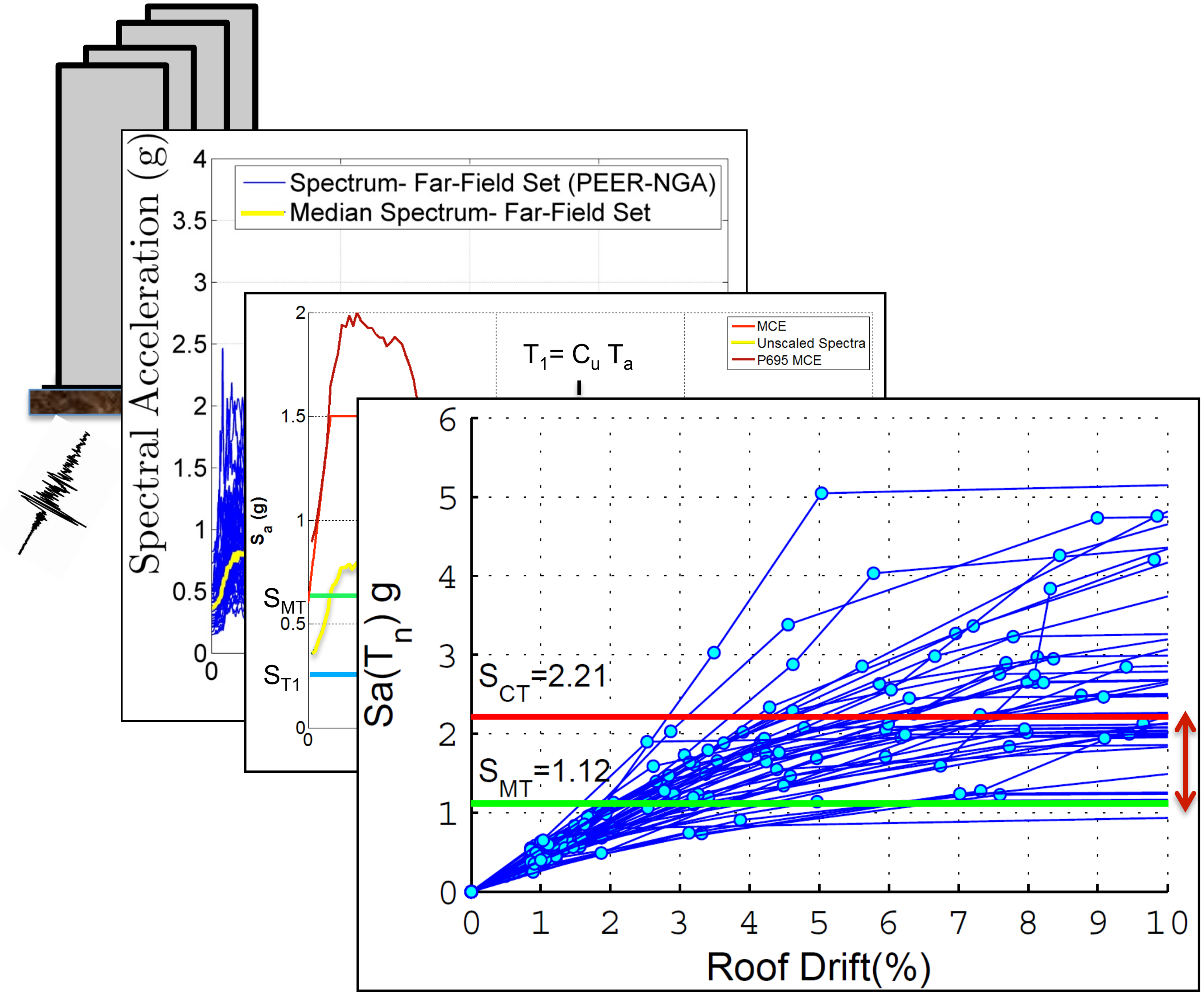
FEMA P695 Used for Evaluation



Determines

1. Probability of collapse in the MCE, and
2. If the design procedure (R-factor, etc.) is acceptable.

$$CMR = \frac{S_{CT}}{S_{MT}}$$ = Collapse Margin Ratio

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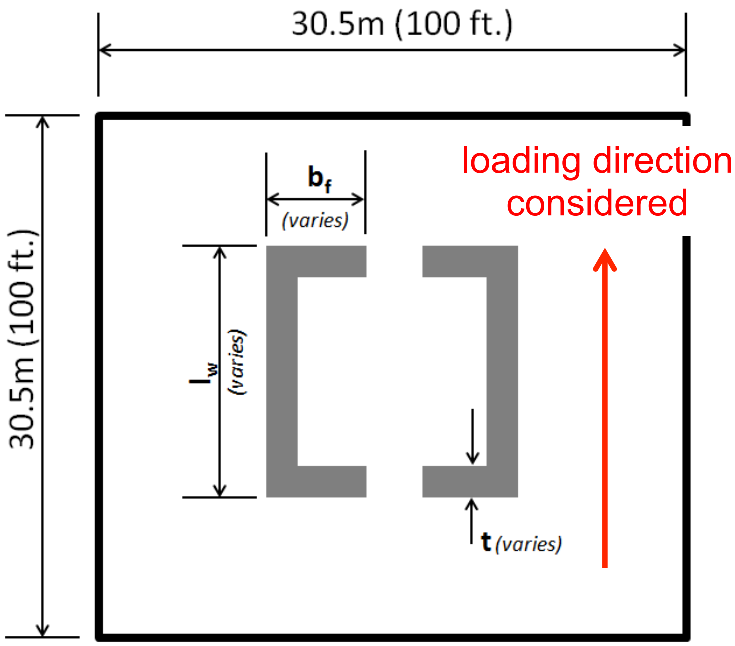
Evaluation of Current Design Procedures

Building Designs

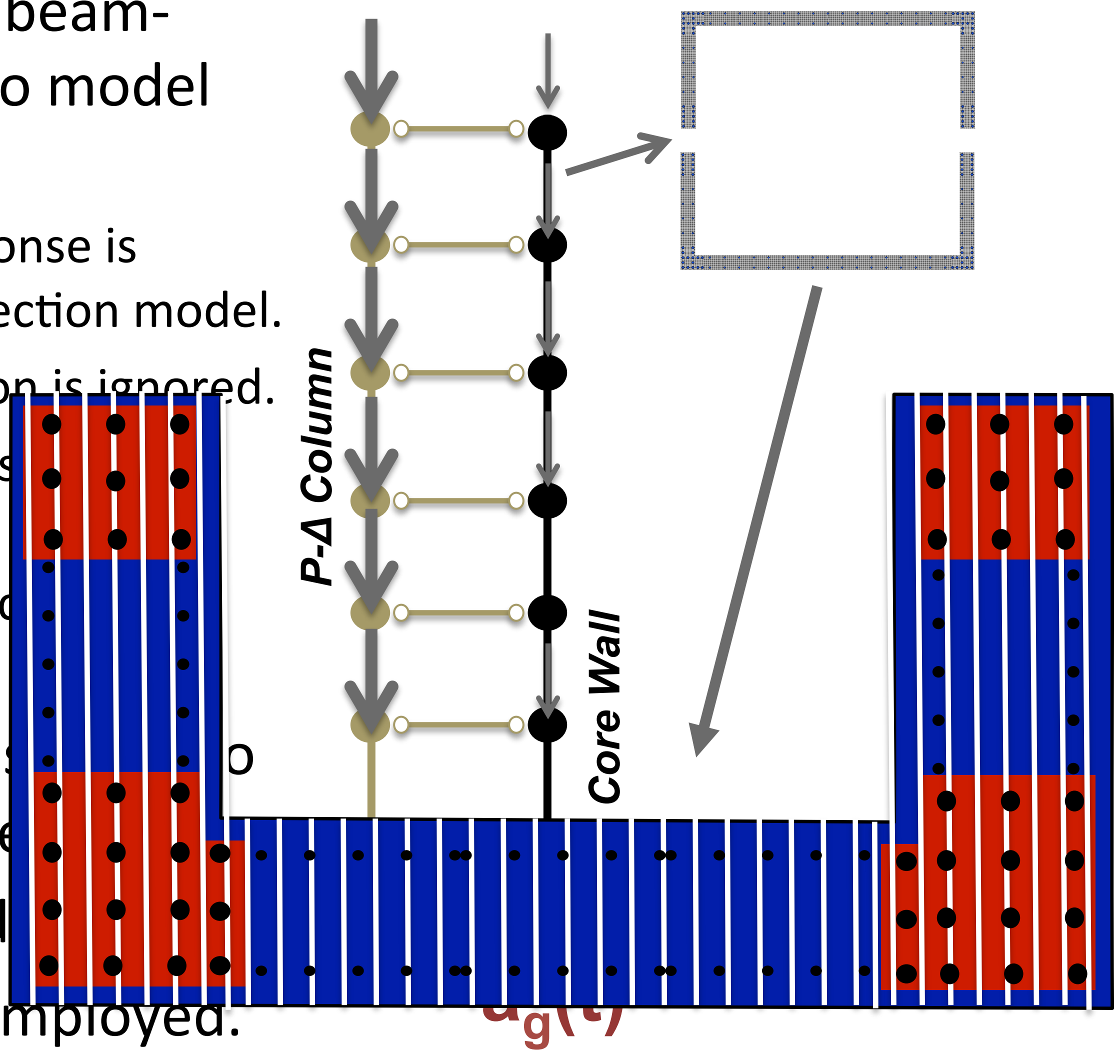


Seismic weight = 8.1 kPa (170 psf)
Gravity weight = 9.1 kPa (190 psf)
Wall axial load at base = $0.1f_cA_g$

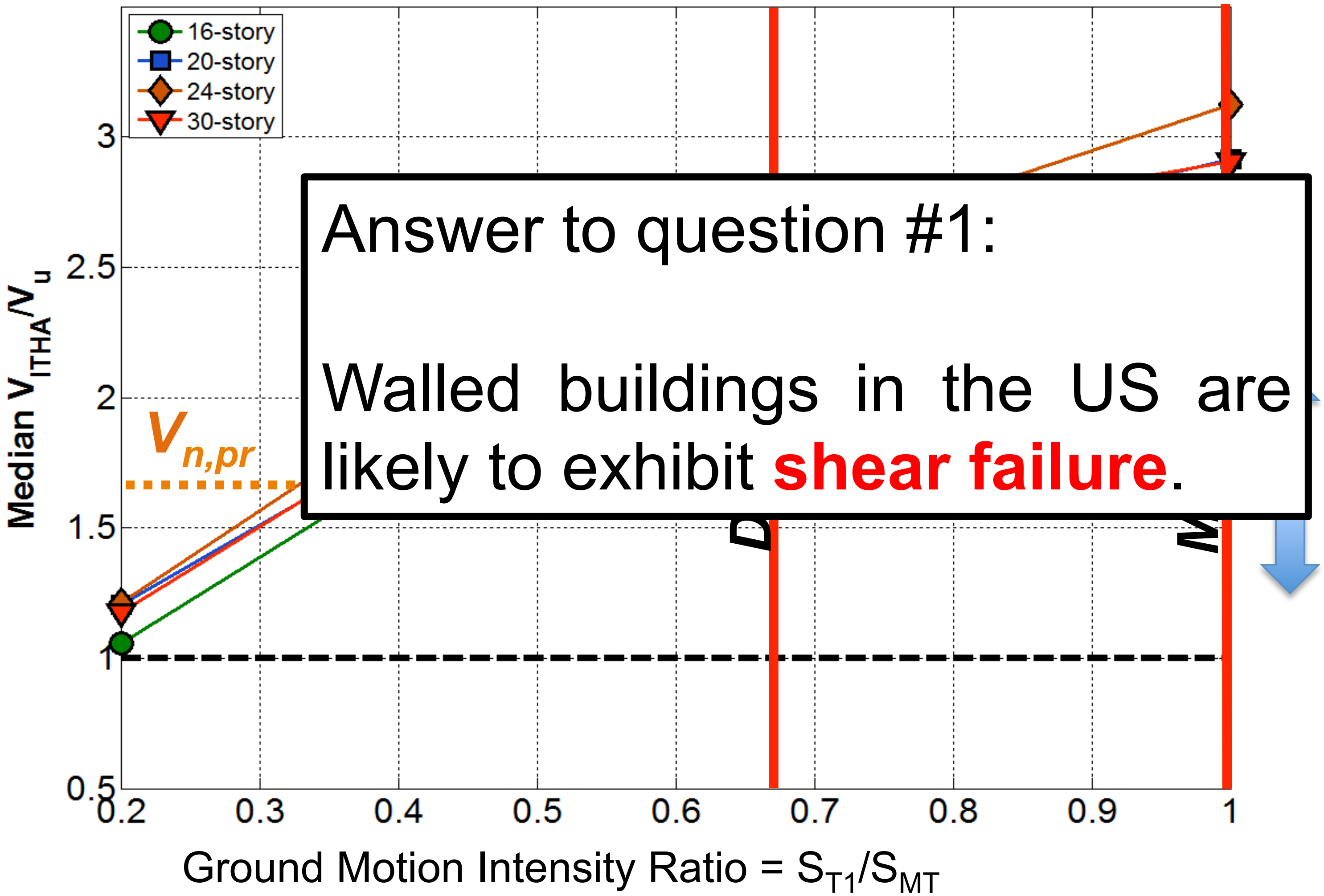
- Design 8 walled bldgs.
- 16, 20, 24 and 30 stories
  - Core-wall buildings
  - Only uncoupled loading dir. considered
- EQ demands per ASCE 7 (2010)
  - SDC D (highest eq. demand category)
  - Strength reduction factor, R = 6
  - Both ELF procedure and MRSA used; MRSA demands scaled up to meet ELF base shear.
- Walls sized to achieve
  - $l_w/t \cong 16$, per building inventory review
  - Size for shear per NIST (2011):
    $V_u = 0.2-0.3\sqrt{f_c}\,A_{cv}$ (MPa)
    $= 2-4\sqrt{f_c}\,A_{cv}$ (psi)
- Wall capacity and detailing per ACI 318 (2011)

Numerical Model Used for Evaluation

- Force-based fiber-type beam-column element used to model walls:
  - Nonlinear flexural response is simulated using fiber-section model.
  - Flexure-shear interaction is ignored.
  - Elastic shear response is (shear stiffness = GA_{cv}).
  - 1 element w/ 5 fiber sec floor.
- Contribution of gravity s o lateral stiffness is ignore
- P-delta effects included
- 2% Rayleigh damping employed.

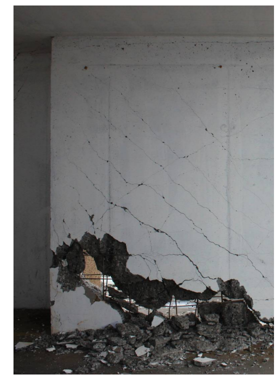


Nonlinear Analysis Results for ITHA Using FEMA P695 Far-Field Motions



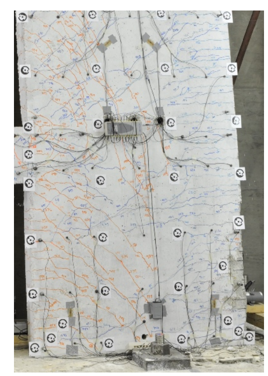
Answer to question #1:

Walled buildings in the US are likely to exhibit **shear failure**.



shear failure

flexural failure



Capacity Design for Shear in Walls

Capacity Design for Shear

- Shear demand used for design must account for
  - Flexural over-strength
  - Dynamic amplification
- Current US design method (ASCE 7 and ACI 318) does not account for either:
  - $\phi V_n \geq V_u$ with $\phi = 0.6$ and V_u from elastic analysis
- Capacity design
  - $\phi V_n \geq V_u'$ with $V_u' = \omega_v \Omega_o V_u$

Dyn. Amplification

Flex. Overstrength

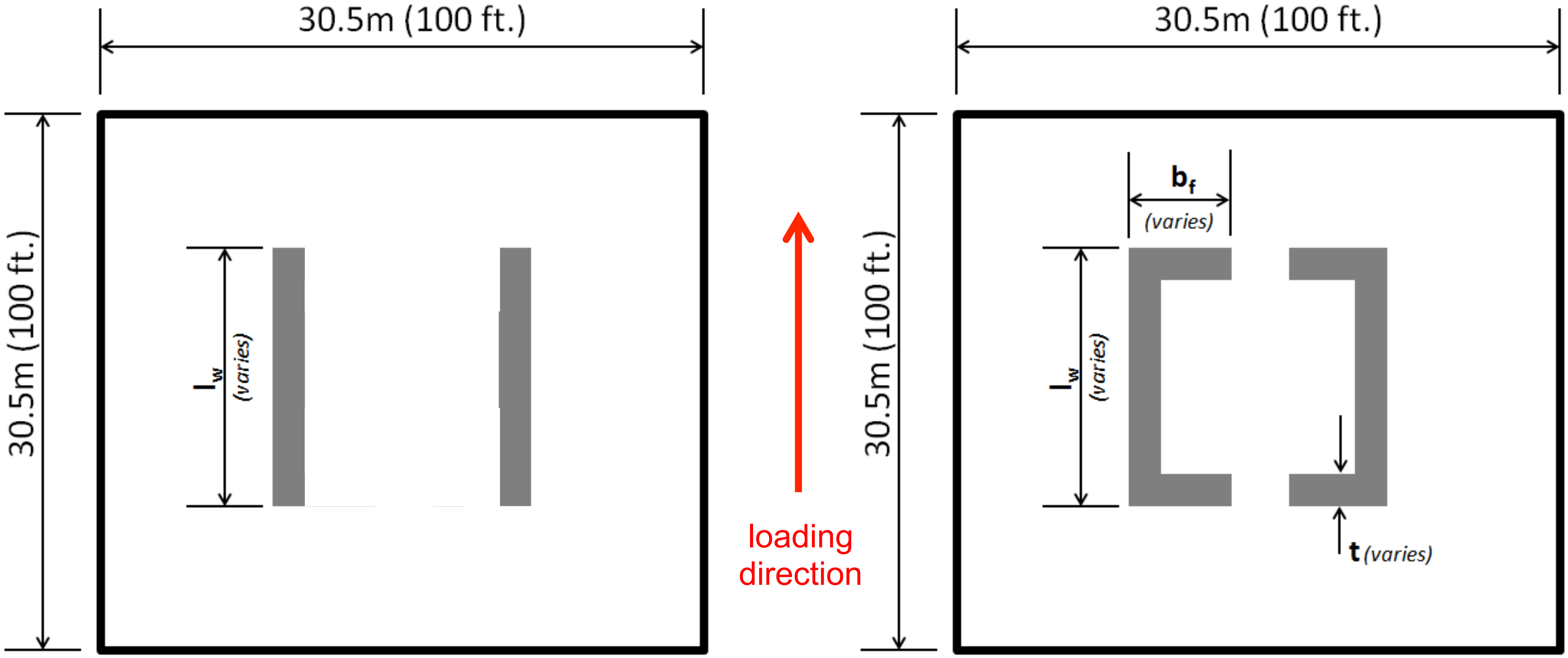
Capacity design approach for shear has been adopted by

- New Zealand Standard NZS-3101 (1995)
- Canadian Standard CSA-A23.3 (2004)
- Eurocode 8 (2004)
- SEAOC (2008)

To Determine a Capacity-Design Method for Shear

- Design and analyze a set of prototype buildings
- Compare maximum shear demand from ITHA (V_{ITHA}) with design shear (V_u) using suite of synthetic motions
- Building designs represent larger design space:
  – 64 Buildings
  – Building heights: N = 6 – 24 stories
  – Fundamental building periods: T_1 = 0.08N – 0.20N
  – ASCE 7 force reduction factors: R = 2,3,4

Idealized Buildings

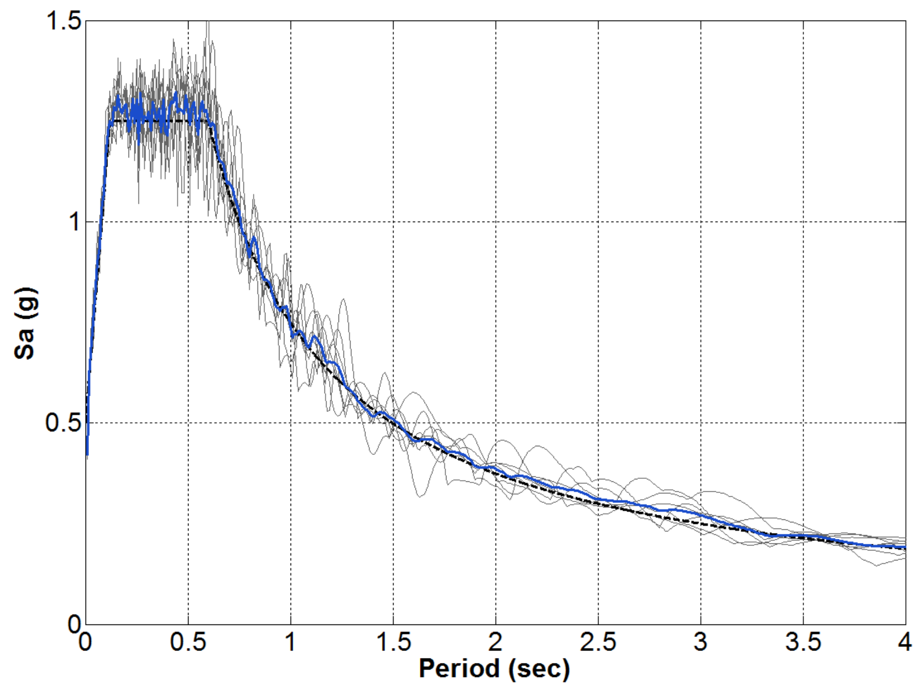


N = 6, 8, 12 stories ***N = 16, 20, 24 stories***

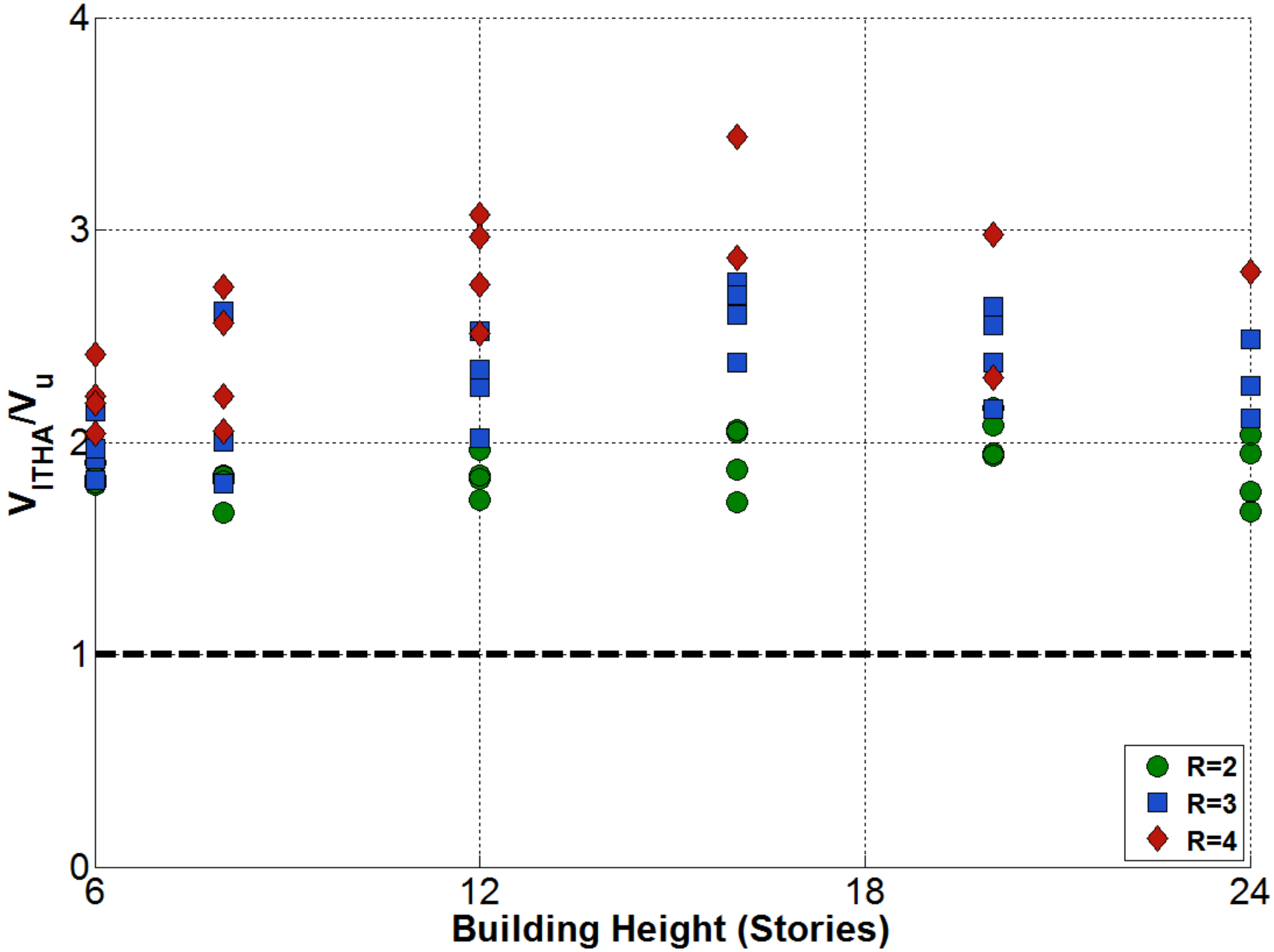
- Seismic weight = 8.1 kPa (170 psf)
- Gravity weight = 9.1 kPa (190 psf)
- Wall axial load at base = $0.1f_cA_g$

Suite of Synthetic Motions Used for ITHA

- 7 synthetic ground motion records
- Motions provide demands that are consistent with the *design* spectrum used for design



Ratio of Maximum Simulated Shear to Design Shear



Recommended Capacity Design Procedure for Shear

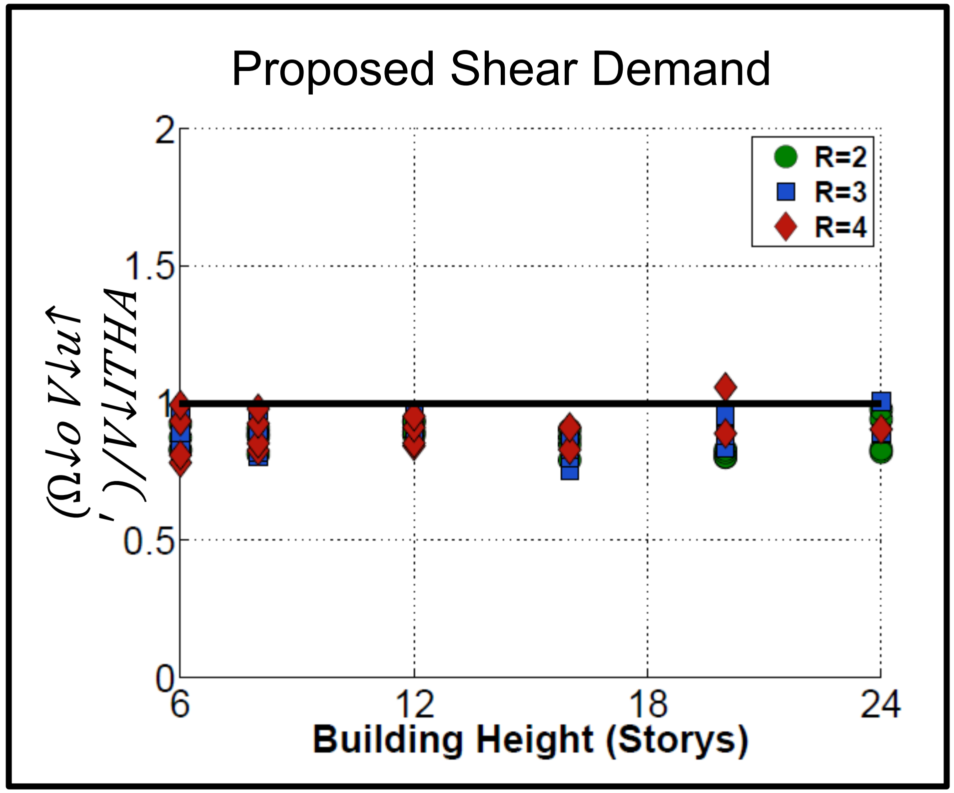
- $\phi V_n \geq \Omega_o V_u'$
  - Ω_o = 1.4 to account for flexural over-strength
  - V_u' determined using Modified MRSA method:

$$V_u' = \sqrt{(V_1/R)^2 + V_2^2 + V_3^2 + \ldots} \quad for\ V_1 > V_2$$

$$V_u' = \sqrt{V_1^2 + (V_2/R)^2 + V_3^2 + \ldots}$$

contributions control.

$$for\ V_2 > V_1$$



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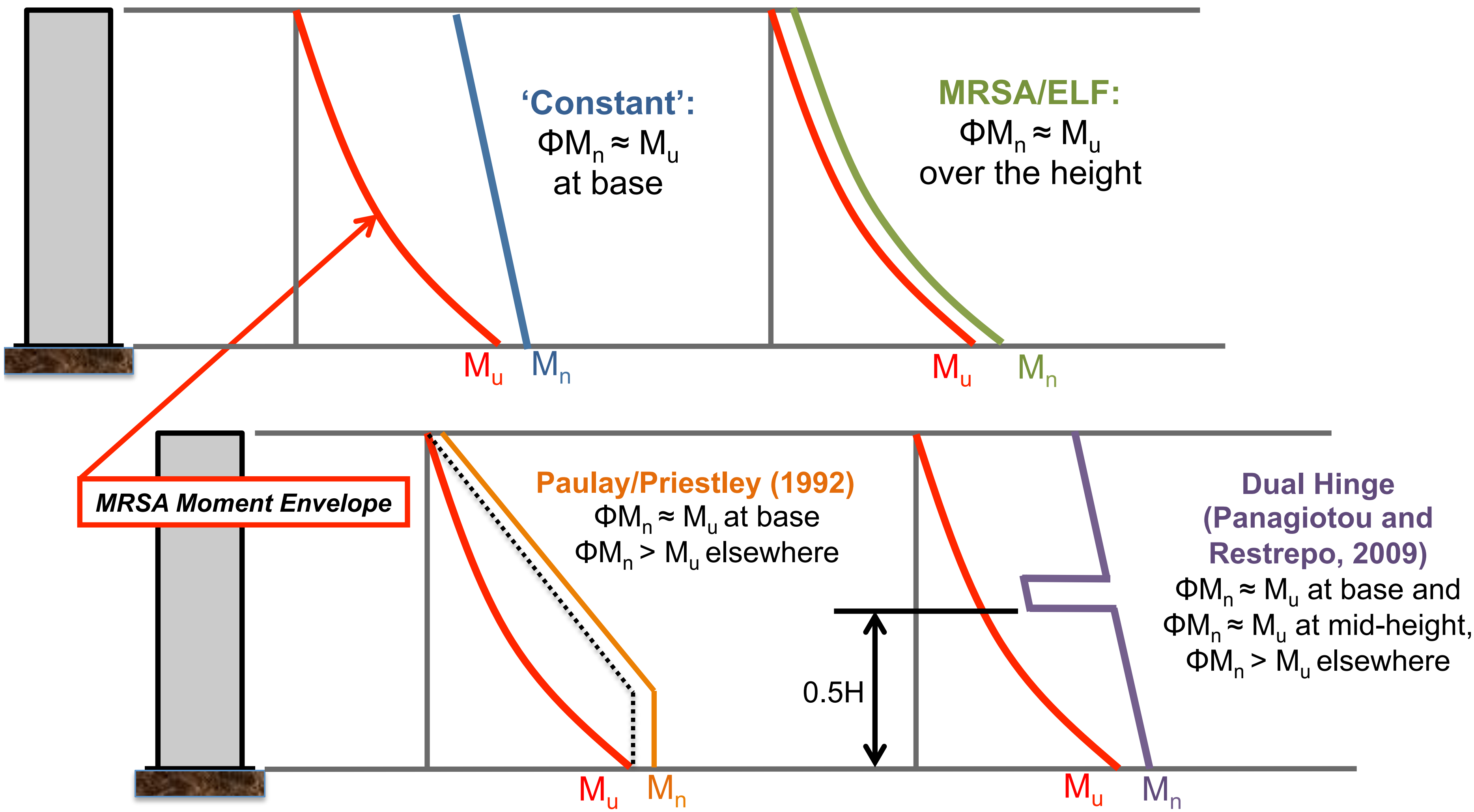
Design for Flexure

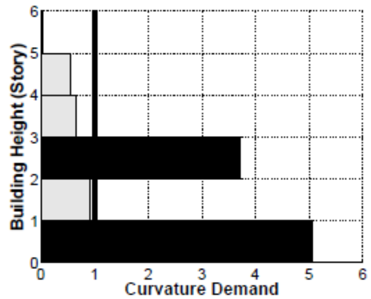
Flexural Design Envelopes



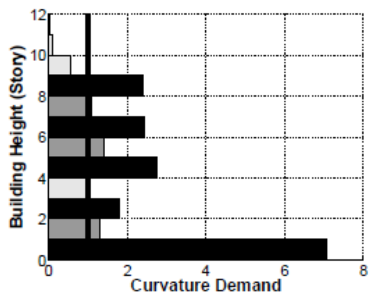
Impact of Flexural Design Envelopes

- Designs employ R=3
- Analyses are done for MCE intensity level
- Curvature Demand =

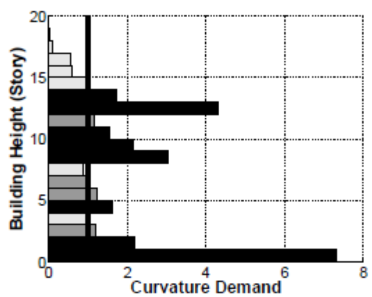
$$\mu_{\phi} = \phi_{max} / \phi_{yield}$$



(a) N = 6 (MRSA)



(d) N = 12 (MRSA)



(g) N = 20 (MRSA)

Recommendations for Wall Design

- Shear Design
  - $\phi V_n \geq \Omega_o V_u'$
  - $\Omega_o = 1.4$ to account for flexural over-strength
  - V_u' defined by Modified MRSA Method to account for dynamic amplification:

$$V_u' = \sqrt{(V_1/R)^2 + V_2^2 + V_3^2 + \dots} \quad for \; V_1 > V_2$$

$$= \sqrt{V_1^2 + (V_2/R)^2 + V_3^2 + \dots} \quad for \; V_2 > V_1$$

- Flexural Design Envelope
  - Paulay/Priestley or Dual Hinge
  - Yielding is limited to expected locations, where ductile detailing is provided
- Flexural Strength Reduction Factor (per FEMA P695)
  - Planar walls: R ≈ 2.5 $(R_{ASCE} \approx 4.0)$
  - Core walls: R ≈ 3.5 $(R_{ASCE} \approx 5.0)$

$\boxed{\text{ASCE 7-10: } R_{ASCE} = 5.6}$

Design to Achieve Acceptable Collapse Risk

1. Elastic analysis to determine demands:
   - Modal response spectrum analysis (MRSA) with wall flexural stiffness equal to 50% of gross-section stiffness.
2. Shear design:
   - Capacity design approach to prevent shear failure: $\phi V_n \geq \Omega_o \omega_v V_u$
   - ϕV_n = factored shear capacity per ACI 318
   - Ω_0 = flexural over-strength factor = 1.4
   - $\omega_v V_u$ = shear demand with dynamic amplification determined using the *Modified MRSA Method* in which only the elastic response mode that contributes most to base shear is reduced by the ASCE 7 R-factor
3. Flexural design:
   - Flexural demands determined using envelope by
     - Paulay & Priestley (1992), for which demands are increased above the base to ensure yielding only at the base, OR
     - Dual-hinge method (Panagiotou and Restrepo 2009), for which demands are increased everywhere except the base and approx. mid-height to achieve two regions of yielding.
   - Planar walls: $R \approx 2.5$ (this is approx. a 50% increase is demand per ASCE 7)
   - Core walls: $R \approx 3.5$ (this is approx. a 20% increase is demand per ASCE 7)

Design to Achieve Acceptable Collapse Risk

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   - Modal response spectrum analysis (MRSA) with wall flexural stiffness equal to 50% of gross-section stiffness.
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   - Core walls: $R \approx 3.5$ (this is approx. a 20% increase is demand per ASCE 7)

Conclusions

- Modeling Slender Concrete Walls
  - Regularization of material response is required for prediction of drift capacity because response is compression controlled with localized softening.
  - OpenSees fiber-type force-based beam-column elements with regularized material models provide accurate and precise simulation of stiffness, strength and drift capacity for planar and c-shaped walls.
- Design of Slender Concrete Walls
  - Current US code design underestimates shear demand in walls.
  - An over-strength factor, Ω_0 = 1.4 and the Modified MRSA method, in which only the 1st or 2nd mode contribution to base shear is reduce, must used to estimate shear demand in walls.
  - Force reduction factors, R-factors, that are smaller than currently specified in ASCE 7 are required to limit flexural damage at MCE.