Seismic Safety and United States Design Practice for Steel-Concrete Composite Frame Structures

Mark D. Denavit

Stanley D. Lindsey and Associates, Ltd.
Atlanta, Georgia

Jerome F. Hajjar

Northeastern University
Boston, Massachusetts

Tiziano Perea

Universidad Autónoma Metropolitana,
Azcapotzalco, D. F., Mexico

Roberto T. Leon

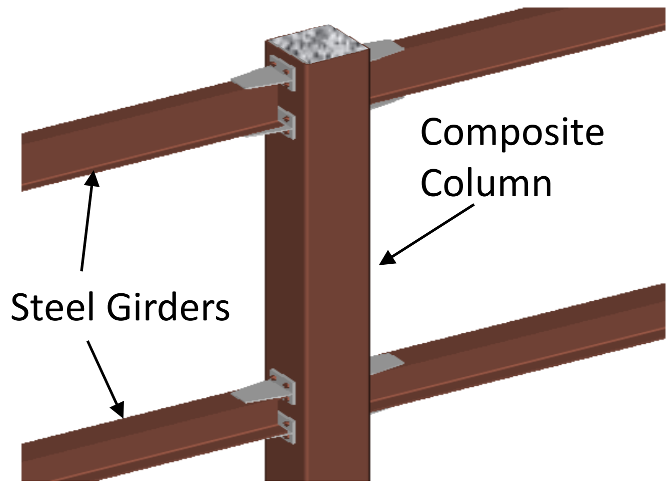
Virginia Polytechnic Institute and State University
Blacksburg, Virginia

Sponsors: National Science Foundation
American Institute of Steel Construction
Georgia Institute of Technology
University of Illinois at Urbana-Champaign

Proceedings of the 10th International Conference
on Urban Earthquake Engineering
Tokyo, Japan, March 1-2, 2013

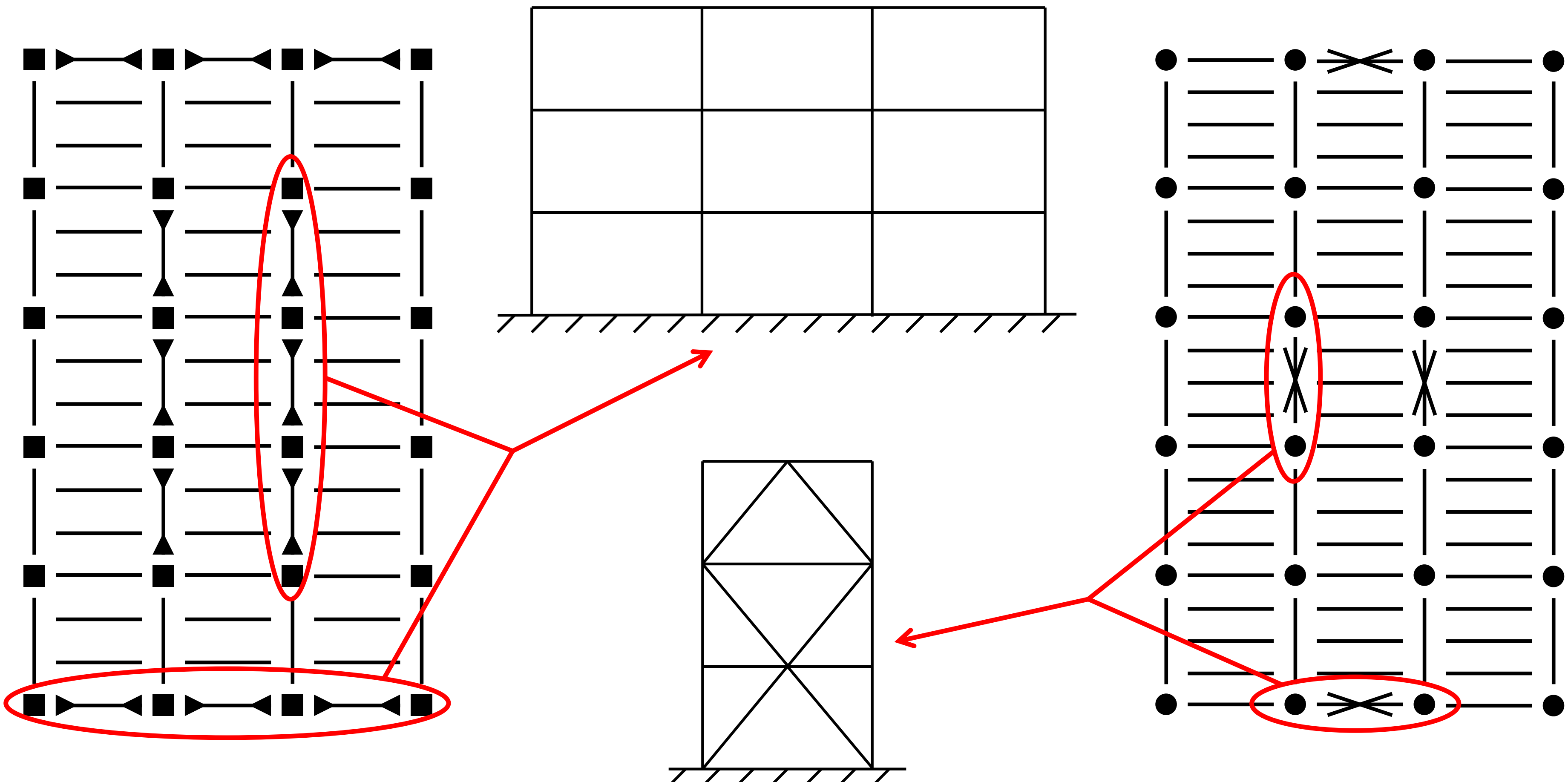
Seismic Performance Factors for Composite Frames

- NEESR-II: System Behavior Factors for Composite and Mixed Structural Systems
- FEMA P695 - Quantification of Building Seismic Performance Factors
- Seismic Performance Factors:
  - Ω_0 = *Overstrength factor*
  - *R = Seismic Response Factor*
  - C_d = *Deflection Amplification Factor*
- Two seismic force resisting systems as defined in the AISC *Seismic Specification*
  - Composite Special Moment Frames (C-SMF) using RCFT or SRC columns and steel beams
  - Composite Special Concentrically Braced Frames (C-SCBF) using CCFT column and steel beams and braces



System	Ω_o	R	C_d
C-SMF	3.0	8.0	5.5
C-SCBF	2.0	5.0	4.5

Selection and Design of Archetype Frames



Moment Frames

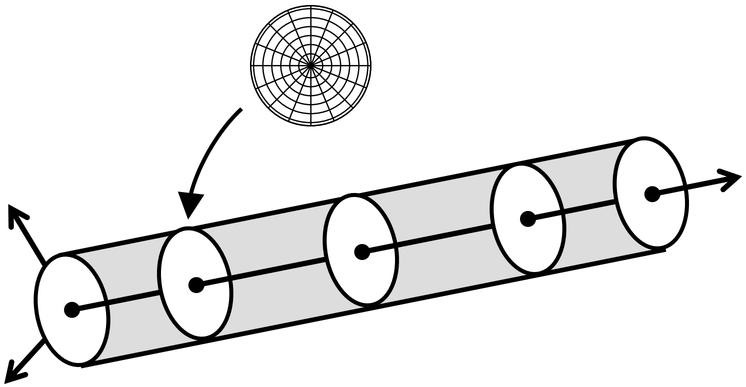
Braced Frames

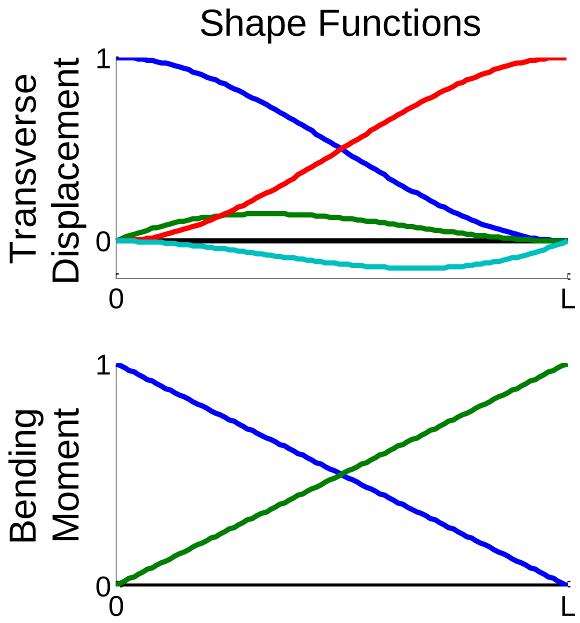
Selected Frames

Design Gravity Load	Bay Width	Design Seismic Load	Conc. Strength (f'_c)	Index	Moment Frames				Braced Frames	
					RCFT	RCFT	SRC	RCFT-Cd	CCFT	CCFT
					3 Stories	9 Stories	3 Stories	3 Stories	3 Stories	9 Stories
High	20’	D_{max}	4 ksi	1	✔	✔	✔	✔	✔	✔
High	20’	D_{max}	12 ksi	2	✔		✔		✔	
High	20’	D_{min}	4 ksi	3	✔	✔	✔	✔	✔	✔
High	20’	D_{min}	12 ksi	4	✔		✔		✔	
High	30’	D_{max}	4 ksi	5	✔	✔			✔	✔
High	30’	D_{max}	12 ksi	6	✔				✔	
High	30’	D_{min}	4 ksi	7	✔	✔			✔	✔
High	30’	D_{min}	12 ksi	8	✔				✔	
Low	20’	D_{max}	4 ksi	9	✔	✔	✔	✔	✔	✔
Low	20’	D_{max}	12 ksi	10	✔		✔		✔	
Low	20’	D_{min}	4 ksi	11	✔	✔	✔	✔	✔	✔
Low	20’	D_{min}	12 ksi	12	✔		✔		✔	
Low	30’	D_{max}	4 ksi	13	✔	✔			✔	✔
Low	30’	D_{max}	12 ksi	14	✔				✔	
Low	30’	D_{min}	4 ksi	15	✔	✔			✔	✔
Low	30’	D_{min}	12 ksi	16	✔				✔	

Mixed Beam-Column Element

- Mixed formulation with both displacement and force shape functions
- Total-Lagrangian corotational formulation
- Distributed plasticity fiber formulation: stress and strain modeled explicitly at each fiber of cross section
- Perfect composite action assumed (i.e., slip neglected)
- Implemented in the OpenSees framework





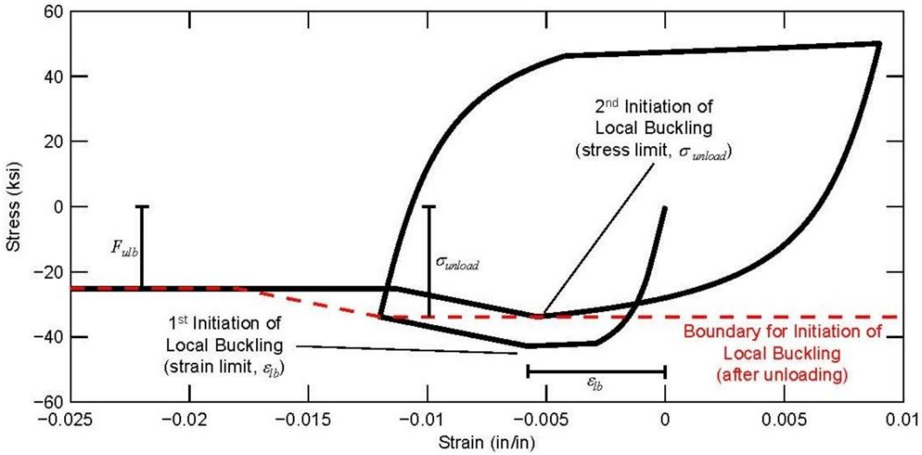
Uniaxial Cyclic Constitutive Relations

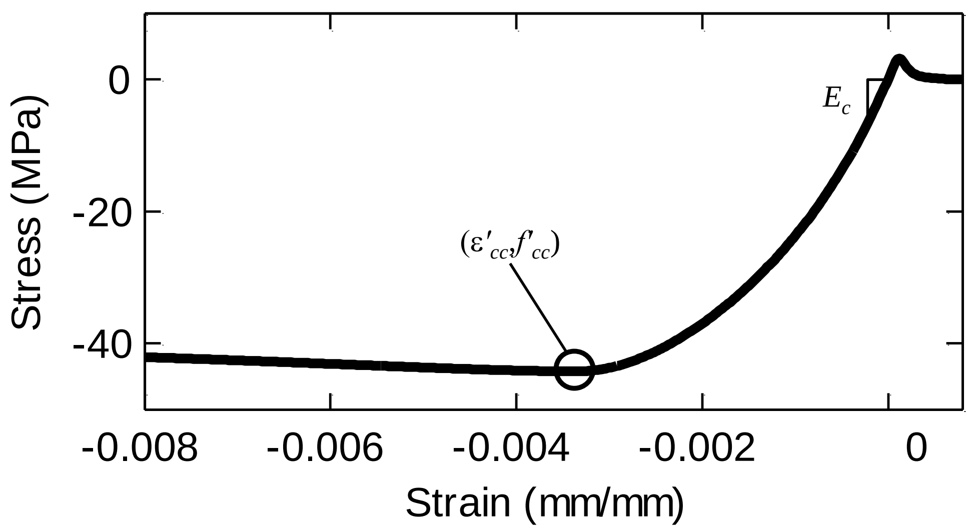
Steel

- Based on the bounding-surface plasticity model of Shen et al. (1995)
- Modifications were made to model the effects of local buckling and cold-forming process

Concrete

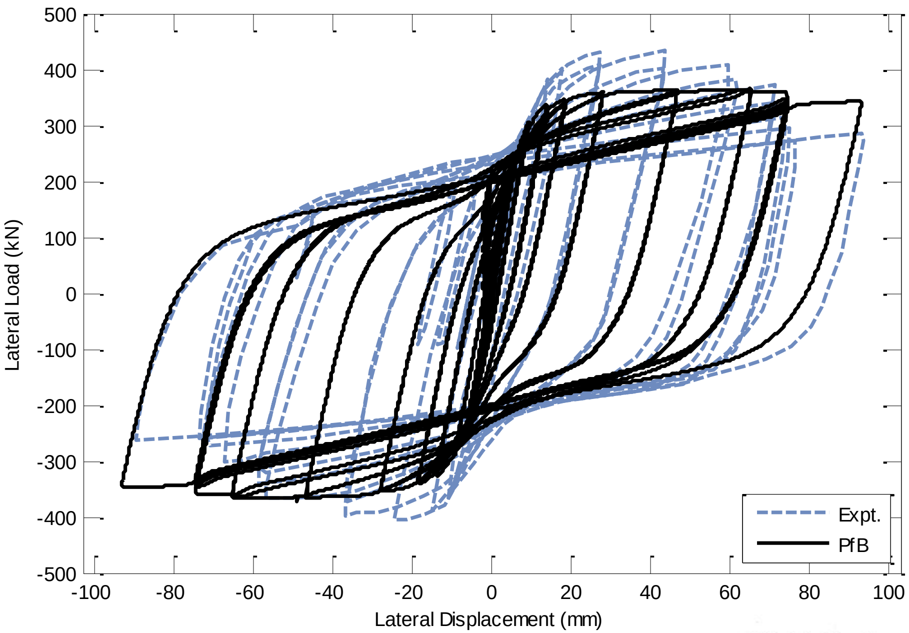
- Based on the rule-based model of Chang and Mander (1994)
- Tsai's equation used for the monotonic backbone curve
- The confinement defined separately for each cross section





RCFT Beam-Column Validation

Varma 2000



Test #5: CBC-32-46-10 (Varma 2000)

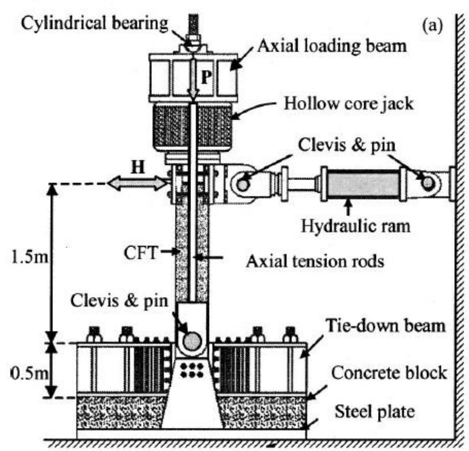
$H/t = B/t = 35$

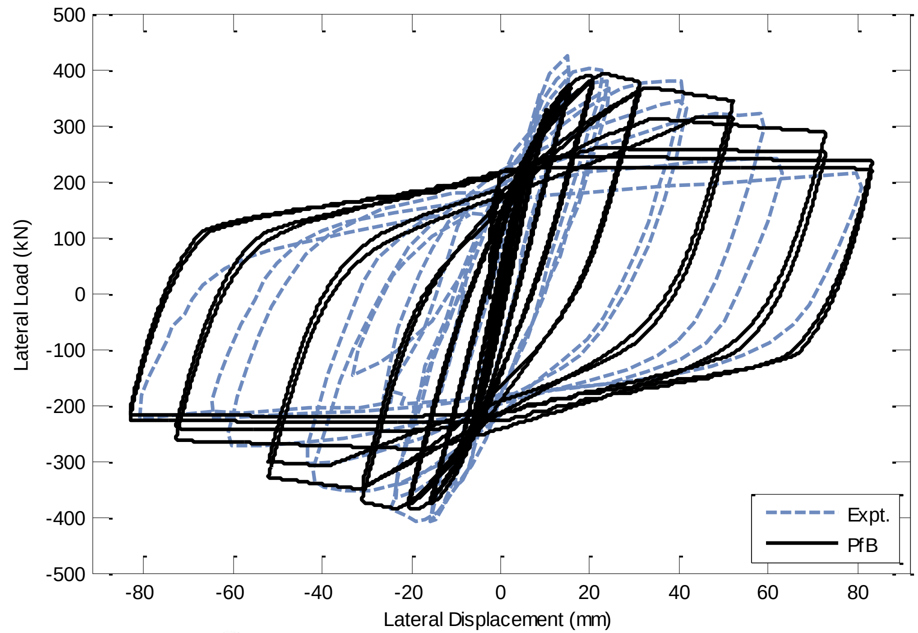
$F_y = 269$ MPa

$f'_c = 110$ MPa

$P/P_{no} = 0.11$

$L/H = 4.9$





Test #8: CBC-48-46-20 (Varma 2000)

$H/t = B/t = 53$

$F_y = 471$ MPa

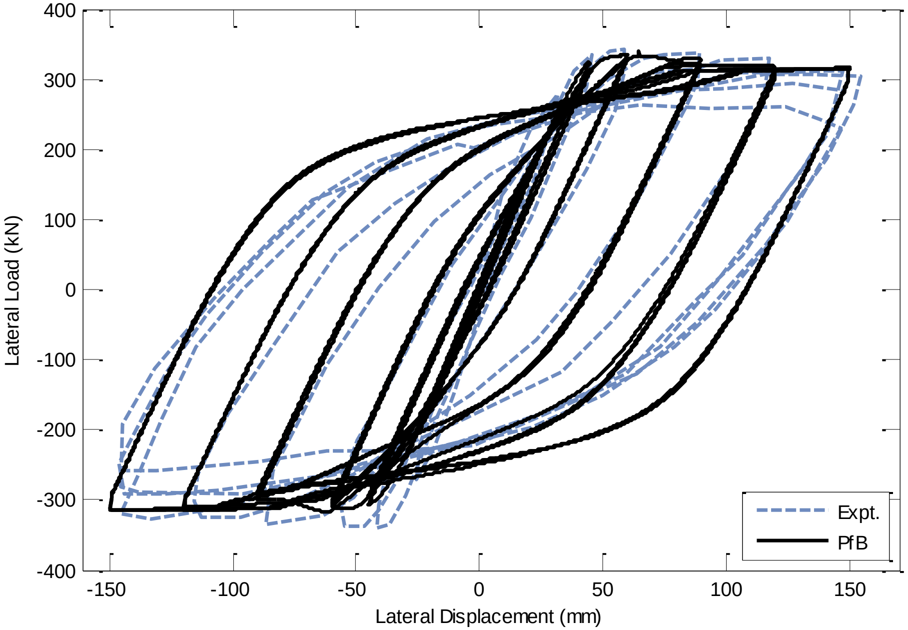
$f'_c = 110$ MPa

$P/P_{no} = 0.18$

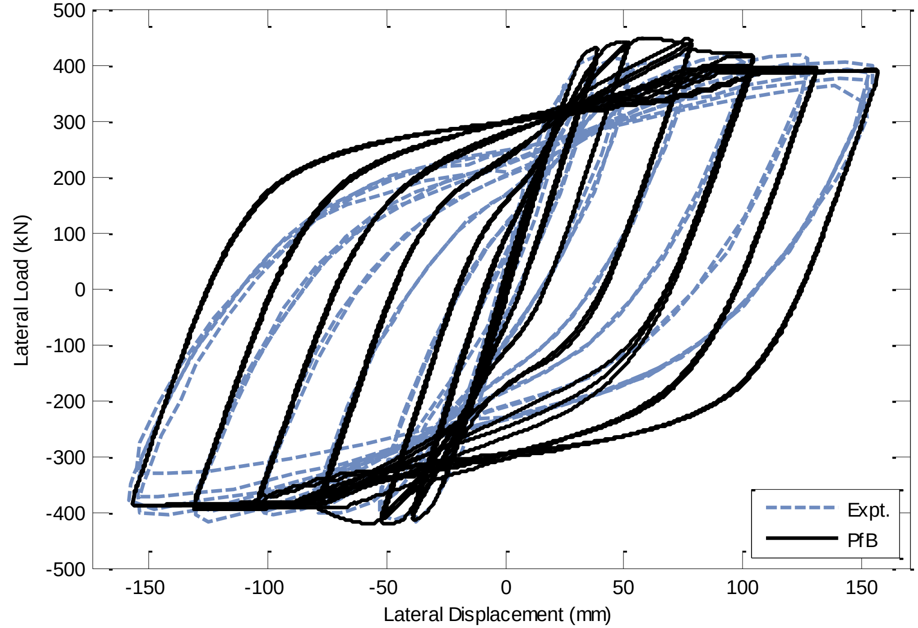
$L/H = 4.9$

SRC Beam-Column Validation

Ricles and Paboojian 1994

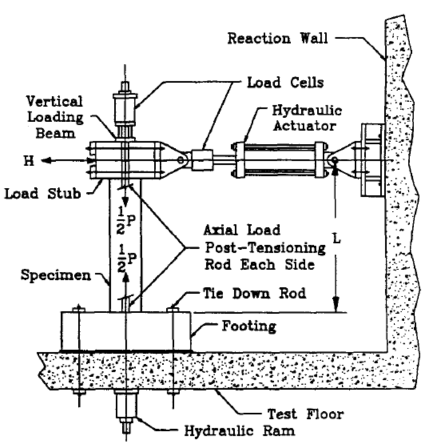


Test #4: 4 (Ricles and Paboojian 1994)



Test #8: 8 (Ricles and Paboojian 1994)

H = 406 mm; B = 406 mm
W8x40
F_y = 372 MPa
4 #9; F_{yr} = 448 MPa
f'_c = 31 MPa
P/P_{no} = 0.19
L/H = 4.8

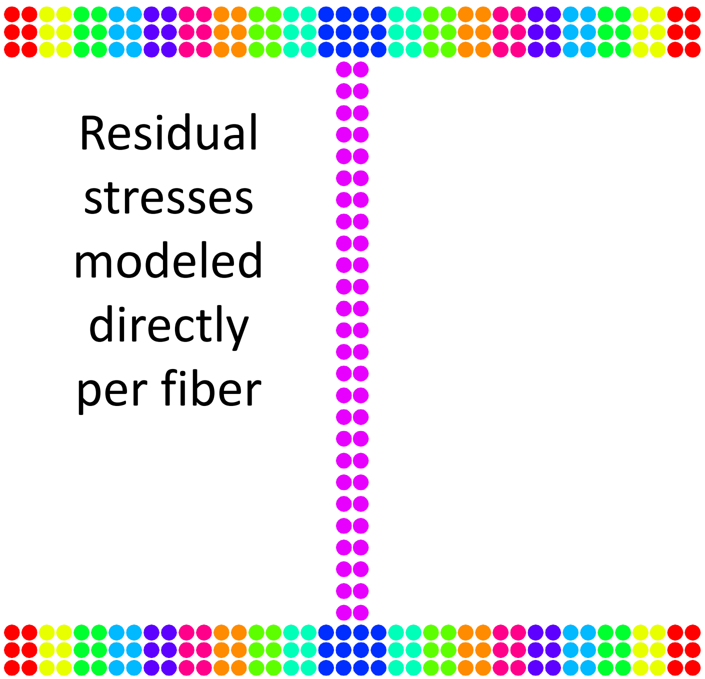


H = 406 mm; B = 406 mm
W8x40
F_y = 372 MP
12 #7; F_{yr} = 434 MPa
f'_c = 63 MPa
P/P_{no} = 0.11
L/H = 4.8

Wide Flange Steel Beam Formulation

Local buckling strain based on plastic hinge length from regression analysis to mitigate localization

$$\frac{L_p}{L_i} = 1 - \frac{M_p}{M_{\max}} = 0.405 - 0.0033\frac{h}{t_w} - 0.0268\frac{b_f}{2t_f} + 0.184\left(\frac{F_u}{F_y} - 1\right)$$

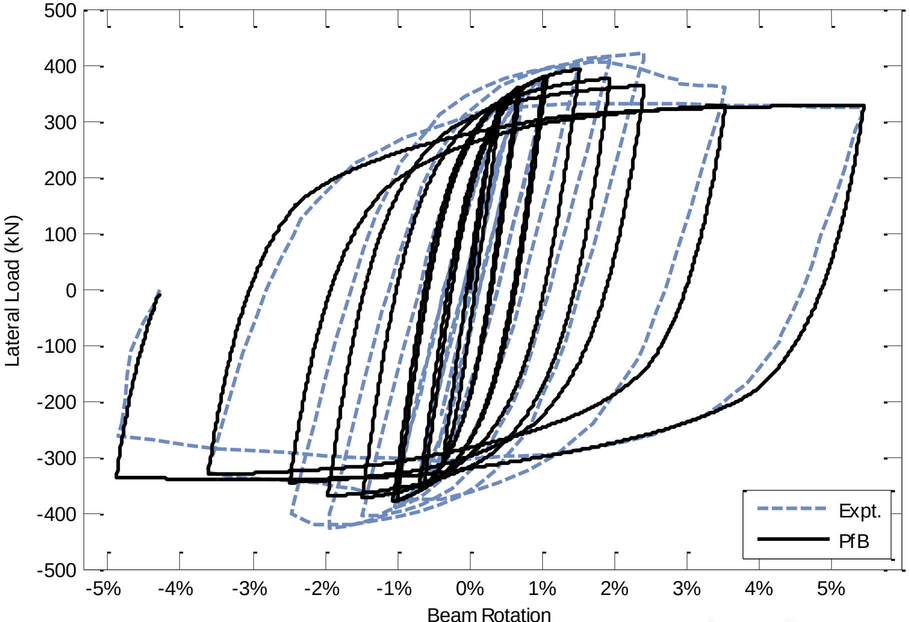


Residual stresses modeled directly per fiber

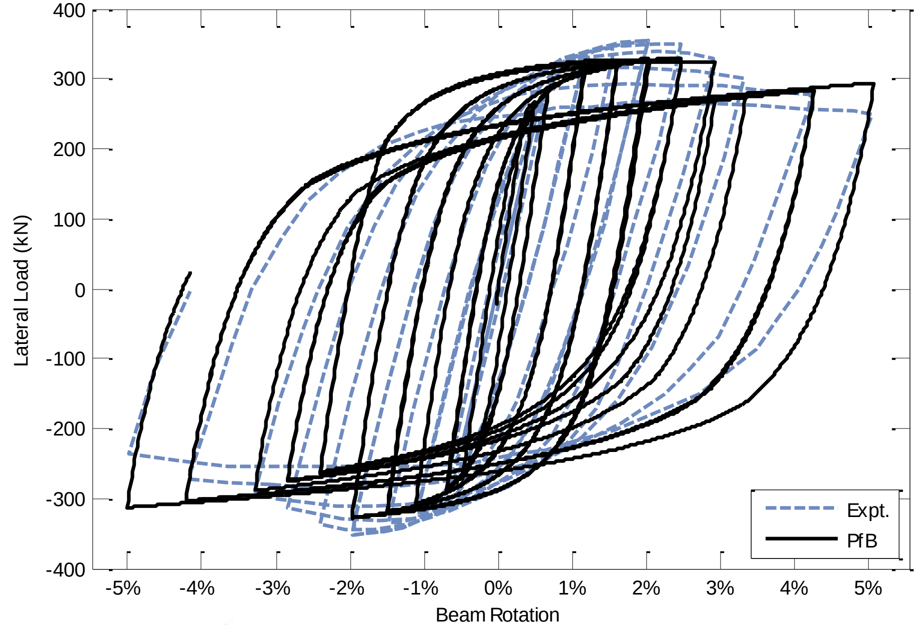
Parameter	Expression
Strain at Local Buckling	$\frac{\varepsilon_{lb}}{\varepsilon_y} = 1 + \frac{E_s}{E_h}\frac{L_p}{L_i - L_p}$
Local Buckling Softening Slope	$K_{lb} = -\frac{E_s}{200}$
Local Buckling Ultimate Residual Stress	$F_{ulb} = 0.2F_y$
Degradation of Plastic Modulus	$\gamma_{E^p} = \left(1 - 2.0\sqrt{\frac{W^p}{F_y}}\right) \geq 0.05$
Degradation of the Size of the Elastic Zone	$\gamma_\kappa = \left(1 - 2.0\sqrt{\frac{W^p}{F_y}}\right) \geq 0.05$

WF Cyclic Local Buckling Calibration

Tsai and Popov 1988



Test #2: 8 (Tsai & Popov 1988)



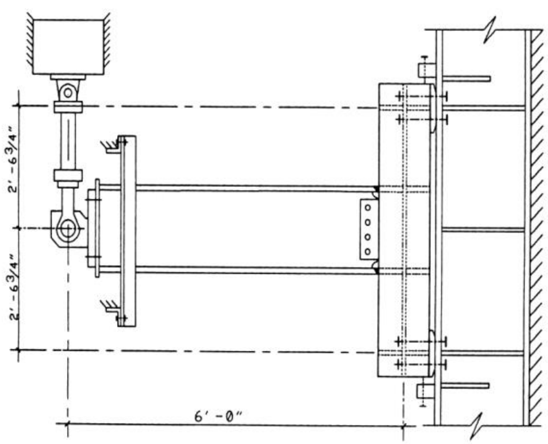
Test #4: 10R (Tsai & Popov 1988)

W21x44

$F_y = 333$ Mpa

$h/t_w = 56.3$

$b_f/2t_f = 7.22$



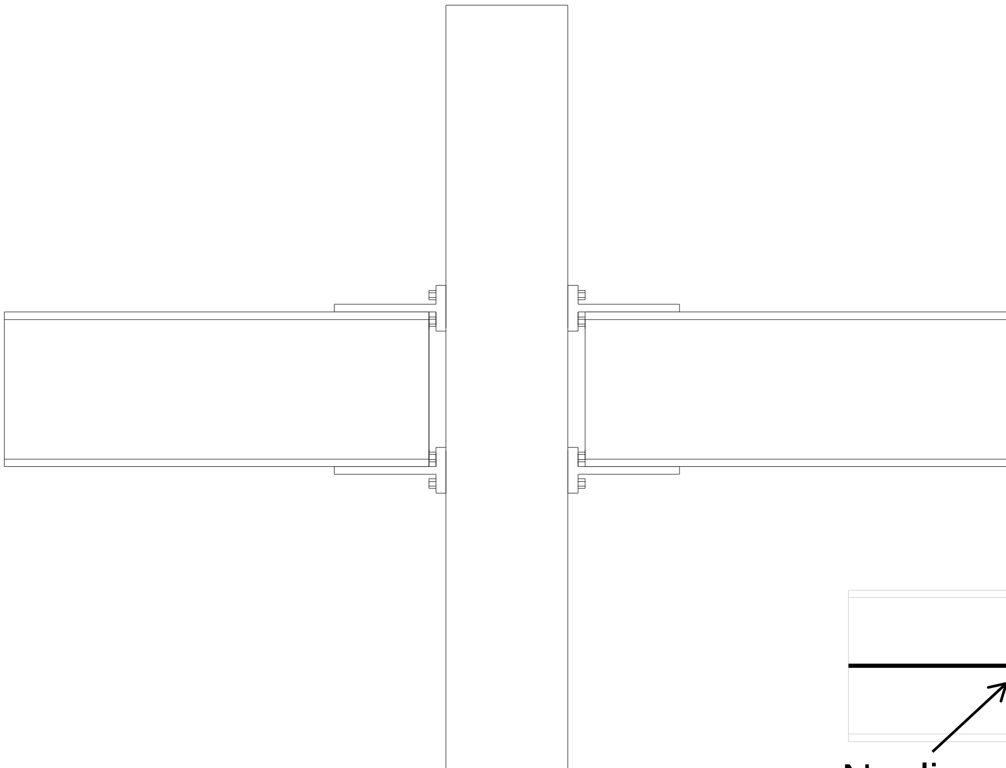
W18x40

$F_y = 310$ MPa

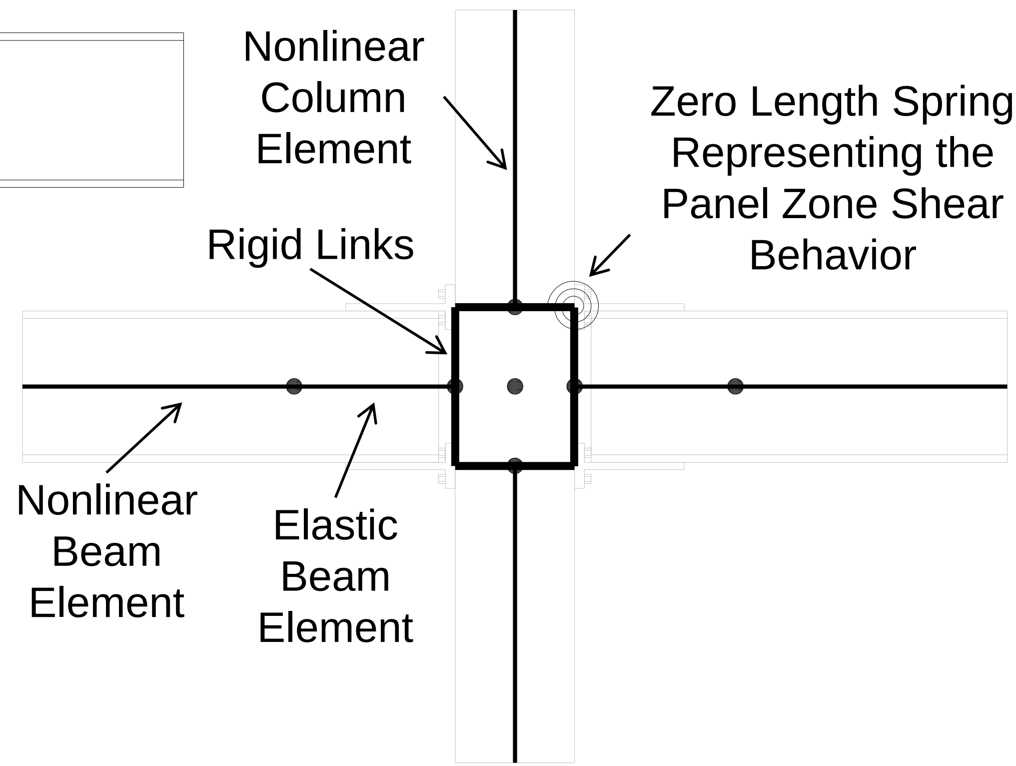
$h/t_w = 50.9$

$b_f/2t_f = 5.73$

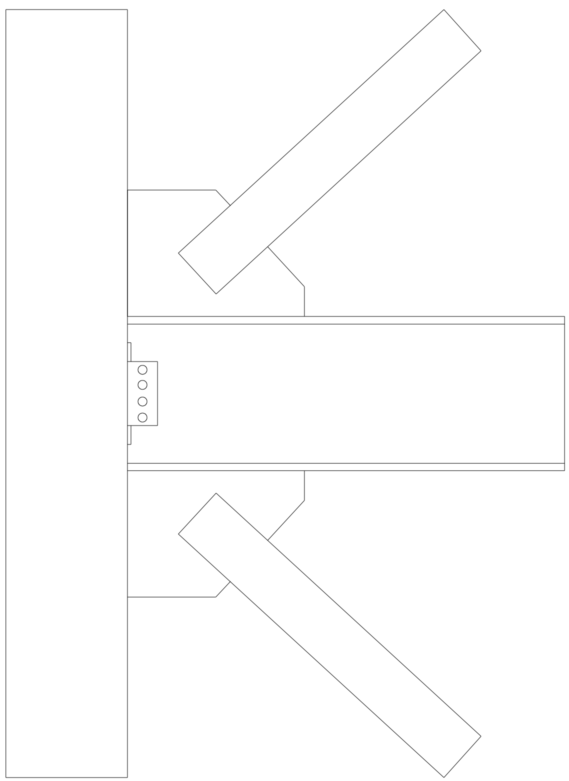
Connection Regions in Special Moment Frames

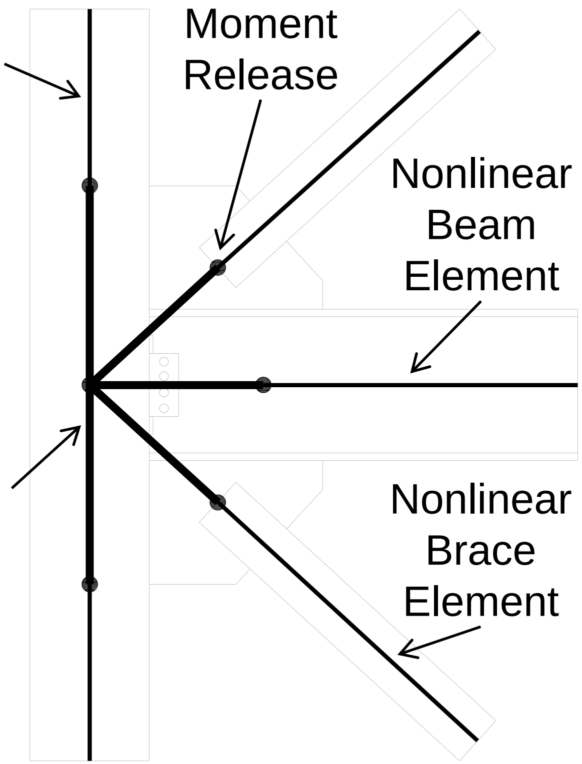


Nonlinear stress-resultant-space multi-surface kinematic hardening model used for rotational spring formulation (after Muhummud 2003)



Connection Regions in Special Concentrically Braced Frames

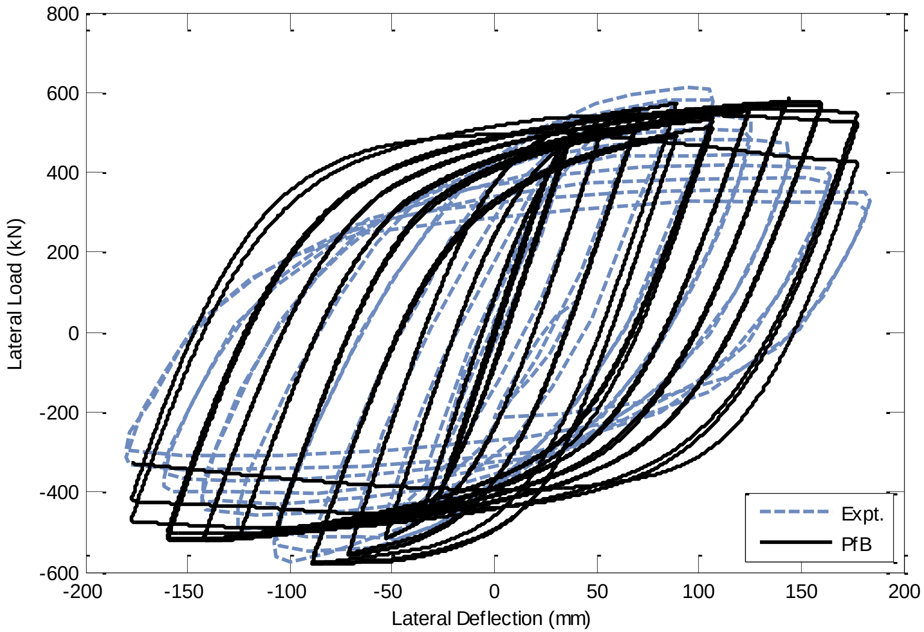




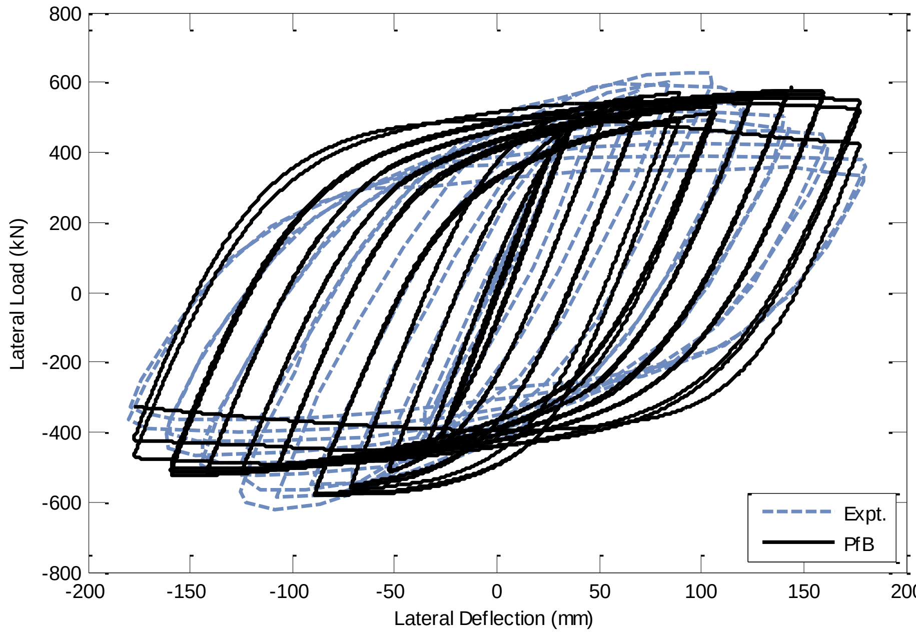
Modeling assumptions established by Hsiao et al. (2012)

Subassemblage Validation

Ricles, Peng, and Lu 2004



Test #2: 6 (Ricles et al. 2004)



Test #3: 7 (Ricles et al. 2004)

Column:

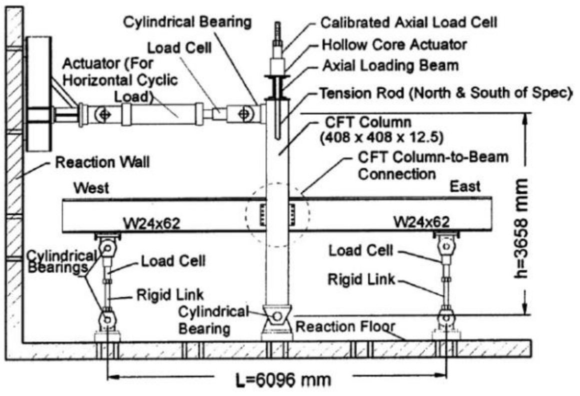
H = 406 mm; B = 406 mm; t = 12.5 mm;

F_y = 352 MPa; f'_c = 58 MPa; P/P_{no} = 0.18;

Beam:

W24x62; F_y = 230 MPa;

$h/t_w = 50.1$; $b_f/2t_f = 5.97$



These specimens are strong column, strong panel zone, weak beam

Evaluation of Seismic Performance Factors

Archetype frames are categorized into performance groups based on basic structural characteristics

Group Number	Design Gravity Load Level	Design Seismic Load Level	Period Domain	Number of C-SMFs	Number of C-SCBFs
PG-1	High	D_{max}	Short	6	4
PG-2	High	D_{max}	Long	2	2
PG-3	High	D_{min}	Short	6	4
PG-4	High	D_{min}	Long	2	2
PG-5	Low	D_{max}	Short	6	4
PG-6	Low	D_{max}	Long	2	2
PG-7	Low	D_{min}	Short	6	4
PG-8	Low	D_{min}	Long	2	2

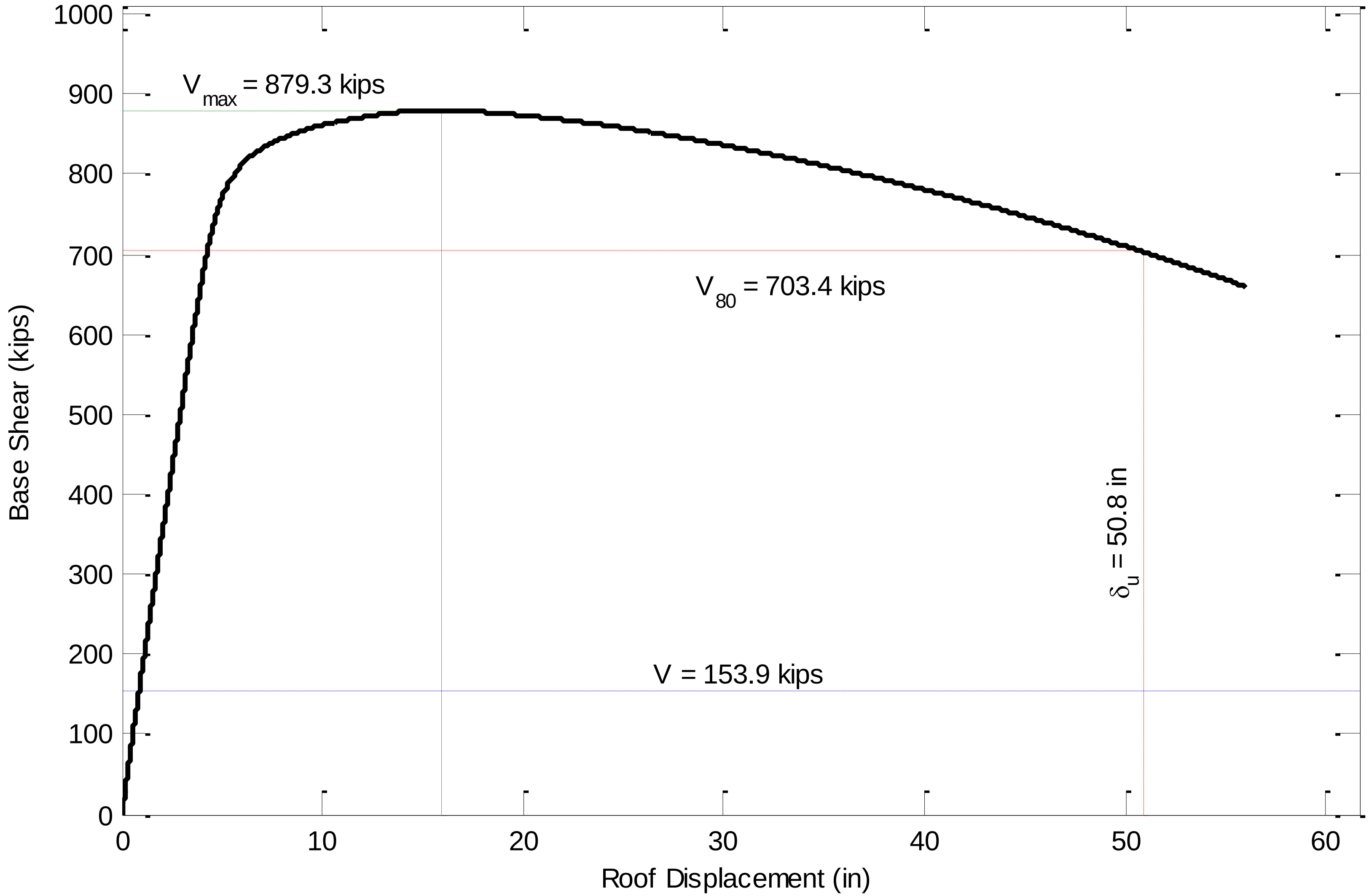
Evaluation of Seismic Performance Factors

Gravity Load, Mass, Damping

	Design	**Analysis**
Gravity Load	1.4 D 1.2 D + 1.6 L + 0.5 L_r 1.2 D + 0.5 L + 1.6 L_r etc., including live load reduction (Section 2.3, ASCE 7-10)	1.05 D + 0.25 L + 0.25 L_r (FEMA P695)
Mass	D + 25% storage live load + 10 psf for partitions (Section 12.7.2, ASCE 7-10)	Same as for design

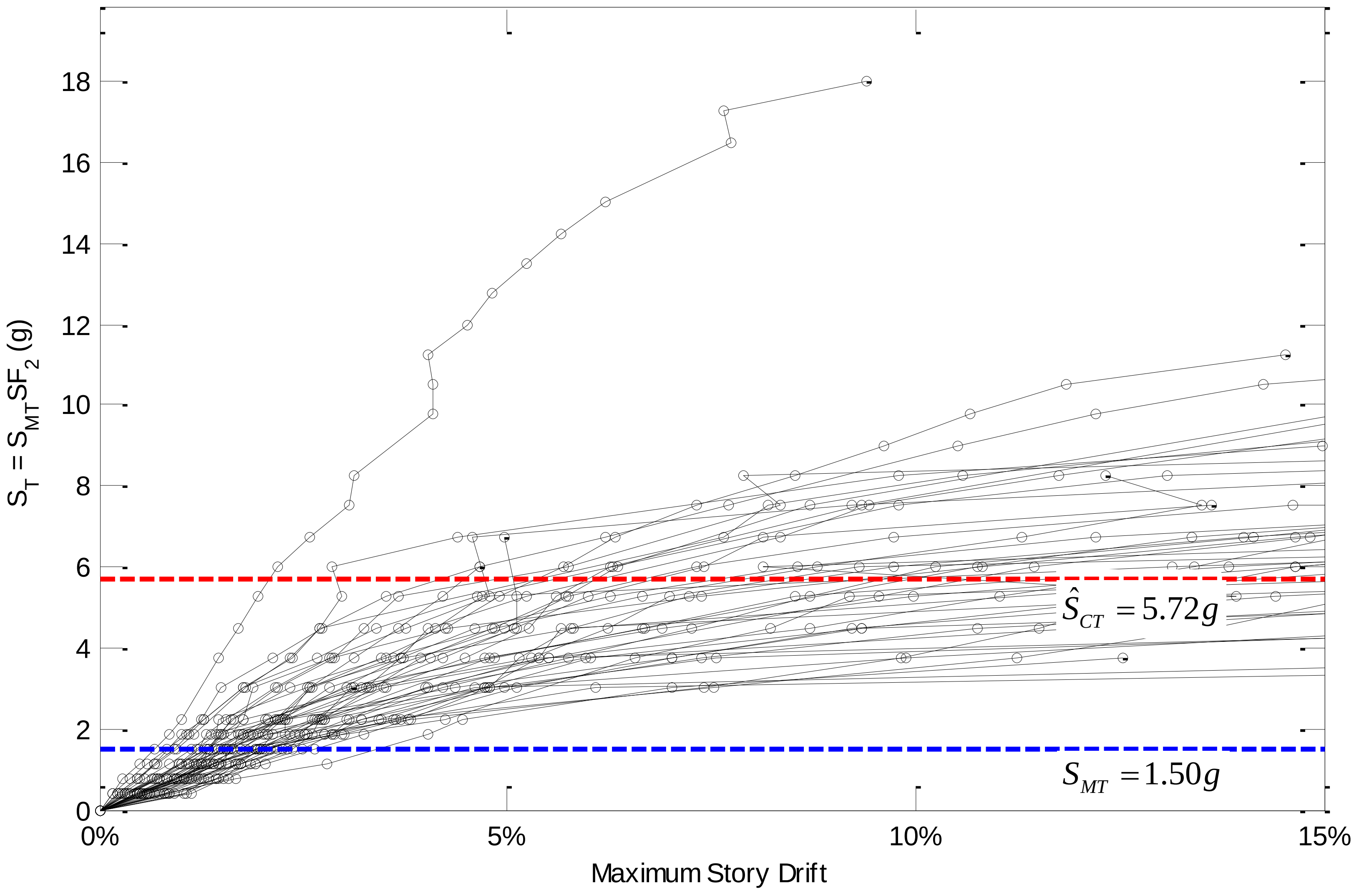
- Rayleigh damping defined equal to 2.5% of critical in the 1st and 3rd mode
- Modeling does not include:
  - Fracture
  - Connection degradation
  - Lateral torsional buckling

Typical Static Pushover Analysis



SFRS: C-SMF, Frame: RCFT-3-1

Typical Dynamic Time History Analyses: Incremental Dynamic Analysis



SFRS: C-SMF, Frame: RCFT-3-1

System Overstrength Factor, Ω_o

- By the FEMA P695 methodology, Ω_o should be taken as the largest average value of Ω from any performance group
  - Rounded to nearest 0.5
  - Upper limits of 1.5R and 3.0
- High overstrength for C-SMFs
  - Displacement controlled design
  - Current value (Ω_o = 3.0) is upper limit and is acceptable
- Overstrength for C-SCBFs near current value (Ω_o = 2.0)
  - Higher for PG-3 and PG-4 (High gravity load, SDC D_{min})

Group Number	Average Ω	
	C-SMF	C-SCBF
PG-1	5.9	2.1
PG-2	5.3	1.9
PG-3	7.6	2.8
PG-4	9.9	2.7
PG-5	6.2	1.8
PG-6	5.5	1.7
PG-7	7.5	2.3
PG-8	6.5	2.2

Response Modification Factor, R

By the FEMA P695 methodology, the R factor assumed in the design of the frames is acceptable if:

- the probability of collapse for maximum considered earthquake ground motions is less than 20% for each frame

$$ACMR_i \geq ACMR_{20\%}$$

- and less than 10% on average across a performance group.

$$\text{mean}\left(ACMR_i\right) \geq ACMR_{10\%}$$

Parameter	**Expression**
Collapse margin ratio	$CMR = \hat{S}_{CT} / S_{MT}$
Spectral shape factor	$SSF = f\left(T, SDC, \mu_T\right)$
Adjusted collapse margin ratio	$ACMR = SSF\ CMR$
Total system collapse uncertainty	$\beta_{total} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2}$
Acceptable value of ACMR	$ACMR_{X\%} = f\left(X, \beta_{total}\right)$

System	**Quality of Design Requirements**	**Quality of Test Data**	**Quality of Nonlinear Modeling**	**Total System Collapse Uncertainty for $\mu_T \geq 3$**
C-SMF	B (Good) $\beta_{DR} = 0.2$	B (Good) $\beta_{TD} = 0.2$	B (Good) $\beta_{MDL} = 0.2$	$\beta_{total} = 0.525$
C-SCBF	B (Good) $\beta_{DR} = 0.2$	B (Good) $\beta_{TD} = 0.2$	B (Good) $\beta_{MDL} = 0.2$	$\beta_{total} = 0.525$

Response Modification Factor, R

- $ACMR_{10\%}$ = Acceptable value of the Adjusted Collapse Margin Ratio for 10% collapse probability
- $ACMR_{10\%}$ = 1.96 for both C-SMF and C-SCBF and are less than the ACMR shown for each performance group in the table
- *ACMR* values show correlation with the overstrength
- C-SMFs
  - Current value (R = 8.0) is acceptable
- C-SCBFs
  - Current value (R = 5.0) is acceptable

Group Number	ACMR	
	C-SMF	C-SCBF
PG-1	4.8	3.3
PG-2	3.7	2.3
PG-3	7.5	5.1
PG-4	8.5	5.4
PG-5	4.9	2.6
PG-6	3.9	2.9
PG-7	7.1	3.8
PG-8	6.9	3.7

Deflection Amplification Factor, C_d

- By the FEMA P695 methodology, C_d = R for these systems
- Would represent a minor change for C-SCBF
  - Current values: C_d = 4.5, R = 5.0
  - Typically strength controlled design
- Would represent a significant change for C-SMF
  - Current values: C_d = 5.5, R = 8.0
  - Typically already displacement controlled design
- Four C-SMF archetype frames designed with the current C_d value
  - Lower overstrength with current C_d (average 4.9 vs. 6.4 with C_d = R)
  - Acceptable performance with current C_d

Conclusions

- Steel-concrete composite frames shown to exhibit consistently excellent seismic behavior, with significant ductility and generally good distribution of deformation demands over the building height
- Current seismic performance factors for C-SMF and C-SCBF found to be acceptable
  - Significant overstrength in C-SMFs (stiffness-controlled)
- Further investigation of the need for and effects of setting C_d equal to R with current deformation limits is warranted for C-SMF