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New Trends for Seismic Engineering of Steel and Composite Structures

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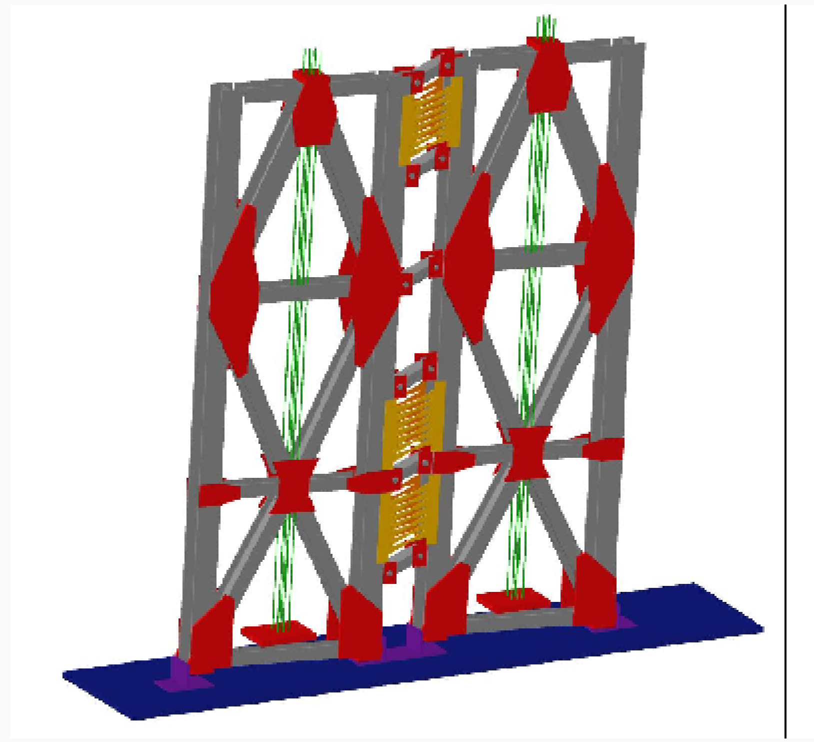
OUTLINE

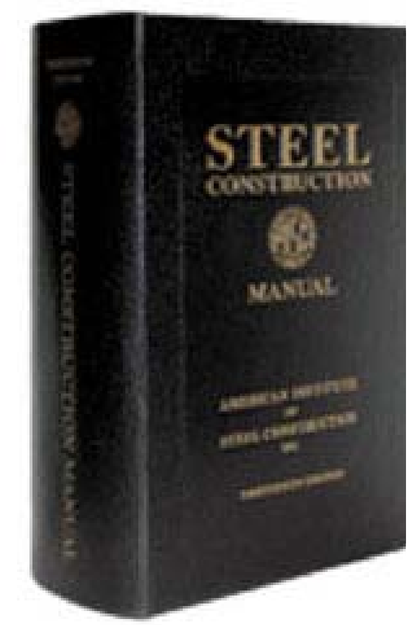
Composite Steel/Concrete Systems

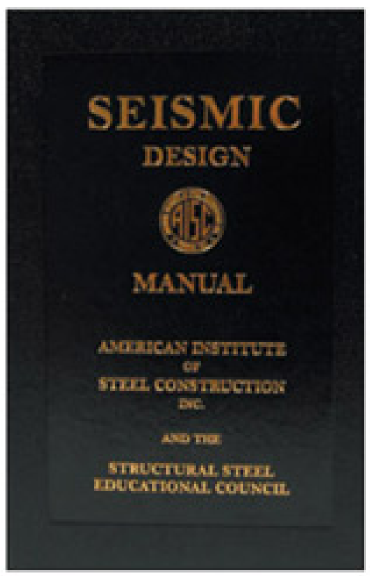


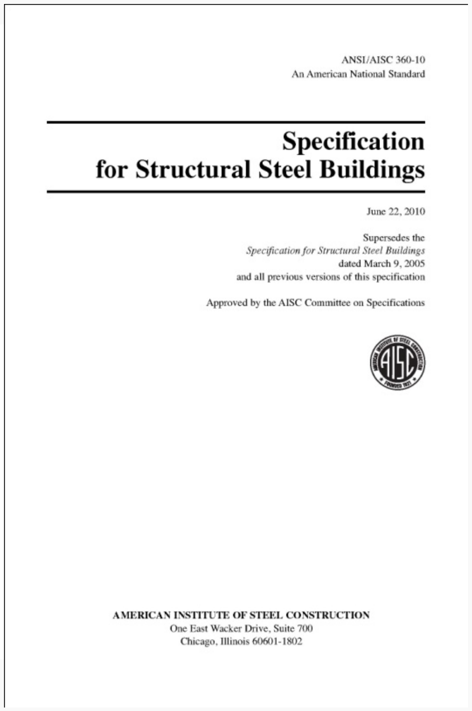
Articulated Fuse Self-Centering Systems

AISC Specification and Seismic Provisions

2005 *2010*






ANSI/AISC 360-10
An American National Standard

Specification
for Structural Steel Buildings

June 22, 2010

Supersedes the
Specification for Structural Steel Buildings
dated March 9, 2005
and all previous versions of this specification

Approved by the AISC Committee on Specifications

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802


i

ANSI/AISC 341-10
An American National Standard

Seismic Provisions for
Structural Steel Buildings

dated June 22, 2010

Supersedes the *Seismic Provisions for Structural Steel Buildings*
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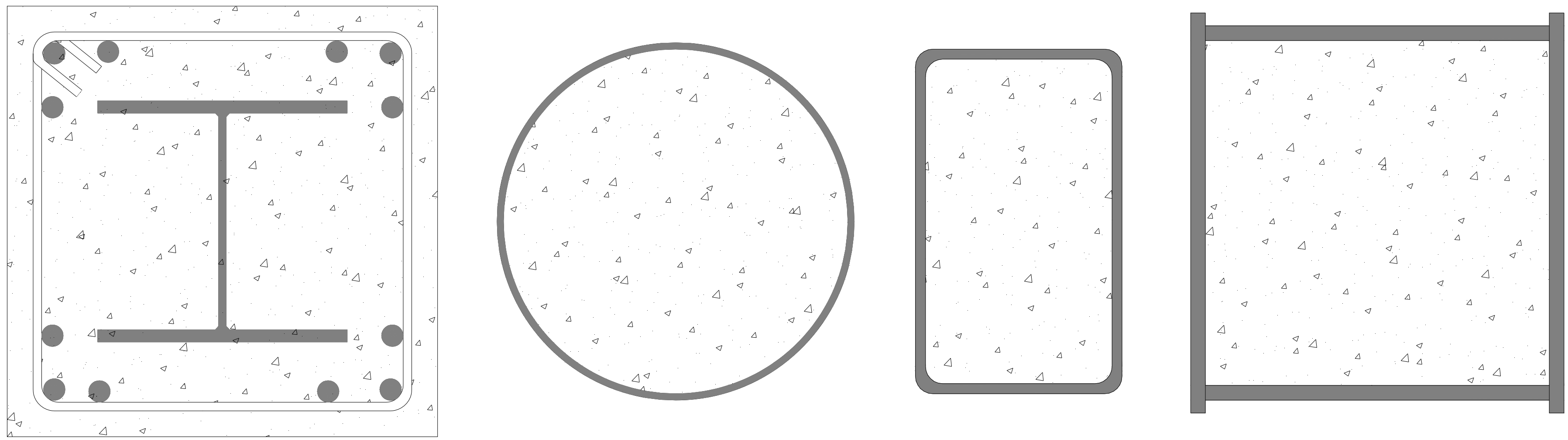
Rewritten *from scratch:*

2005 AISC Specification for Structural Steel Buildings
2010 AISC Seismic Provisions for Structural Steel Buildings

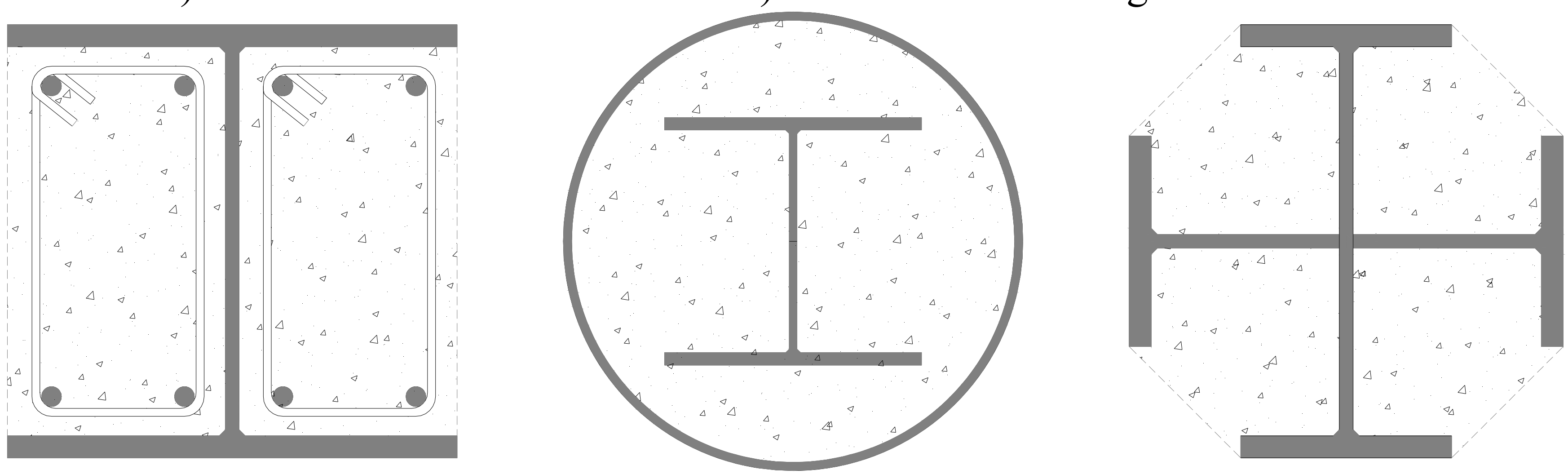
Available from http://www.aisc.org

Possible configurations in composite columns



a) SRC

b) Circular and Rectangular CFT



c) Combinations between SRC and CFT

AISC 2005 Provisions for Composite Columns

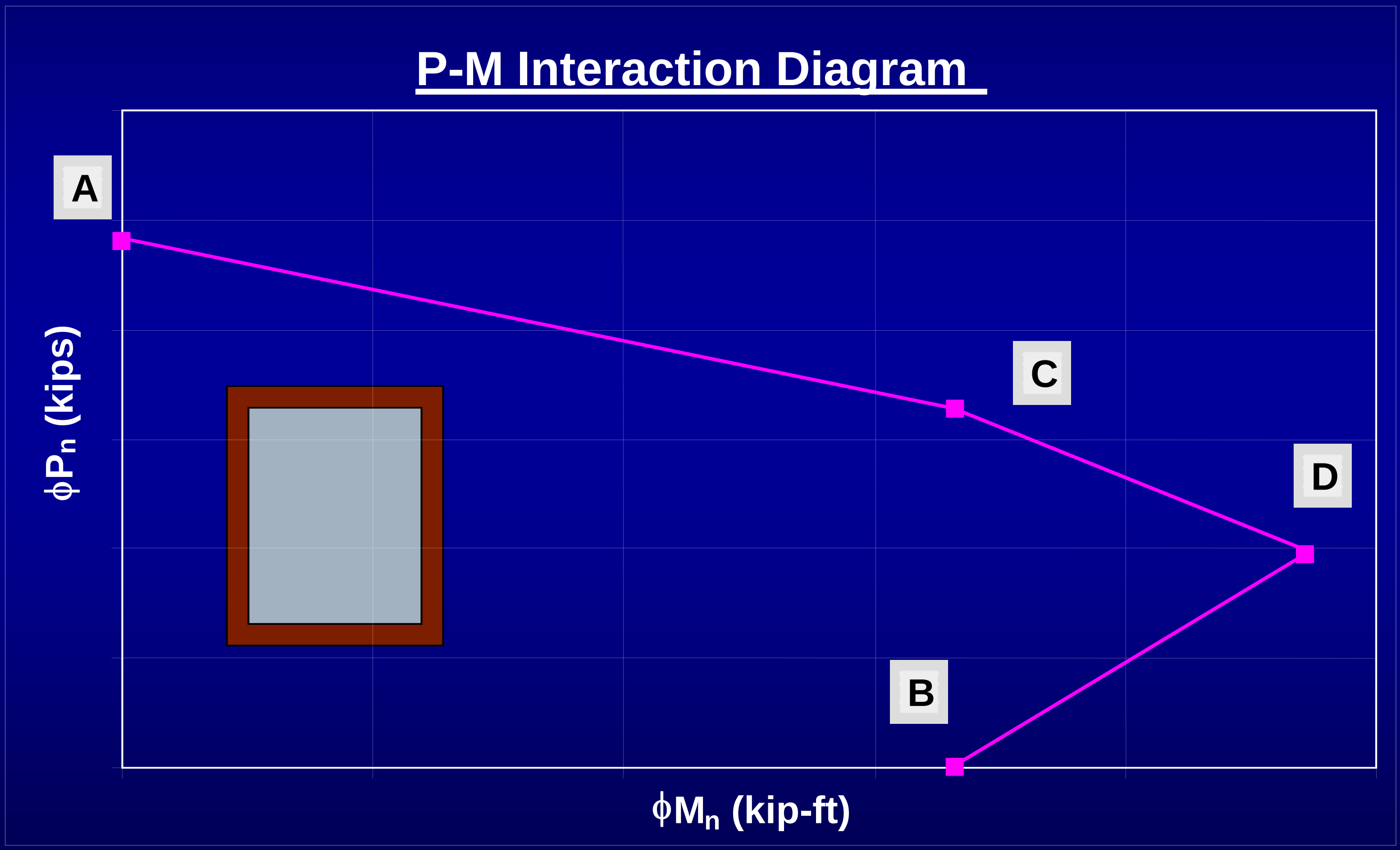
New Limitations

- *Material strengths:*
  - *Concrete 10 ksi (70 MPa)*
  - *Steel 75 ksi (525 MPa)*
- *Steel area: 0.01 A_g min*

AISC 2005 Provisions for Composite Columns

- *Typically use Plastic Stress Distribution Method*
  - *Plastic Strength Equations*
- *For axial compression:*
  - *Square, rectangular, round HSS are in tables in AISC Manual for CFTs*
  - *Tabulated versus KL (effective length)*
  - f'_c *= 4, 5 ksi concrete*

AISC 2005 Provisions for Composite Columns



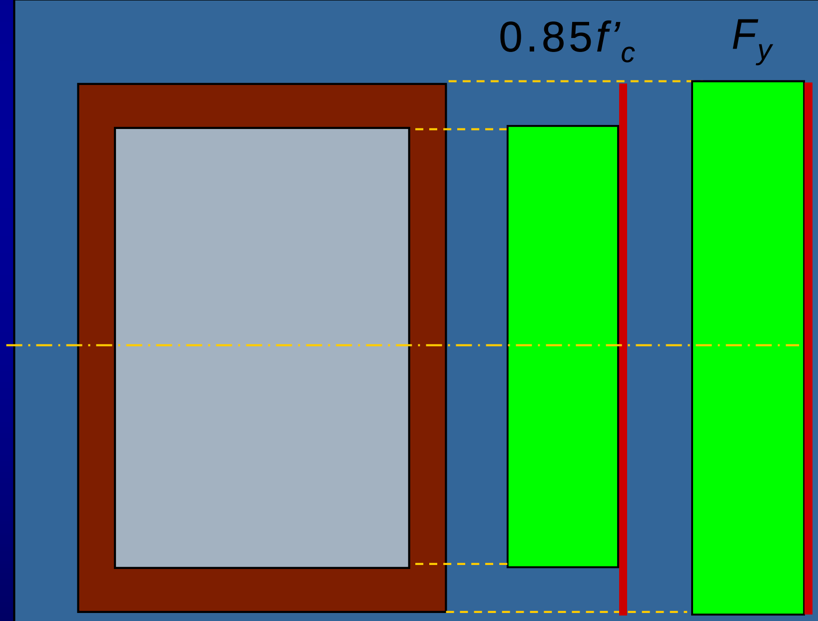
Axial force-bending moment interaction diagram

Slides from L. Griffis, Walter P. Moore & Assoc.

AISC 2005 Provisions for Composite Columns

P-M Interaction Diagram

A



$P_A = A_s F_y + 0.85 f'_c A_c$

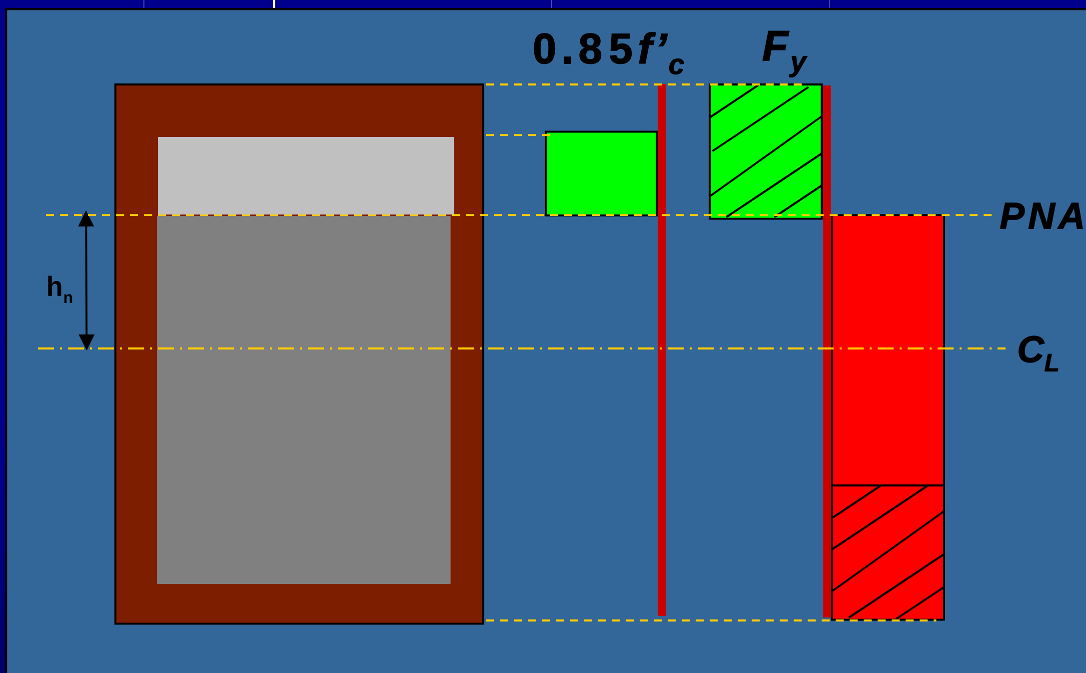
$M_A = 0$

A_s = area of steel shape

$A_c = A_g - A_s$

AISC 2005 Provisions for Composite Columns

P-M Interaction Diagram



$$P_B = 0$$

$$M_B = M_D - Z_{sn}F_y - \tfrac{1}{2} Z_{cn}(0.85f'_c)$$

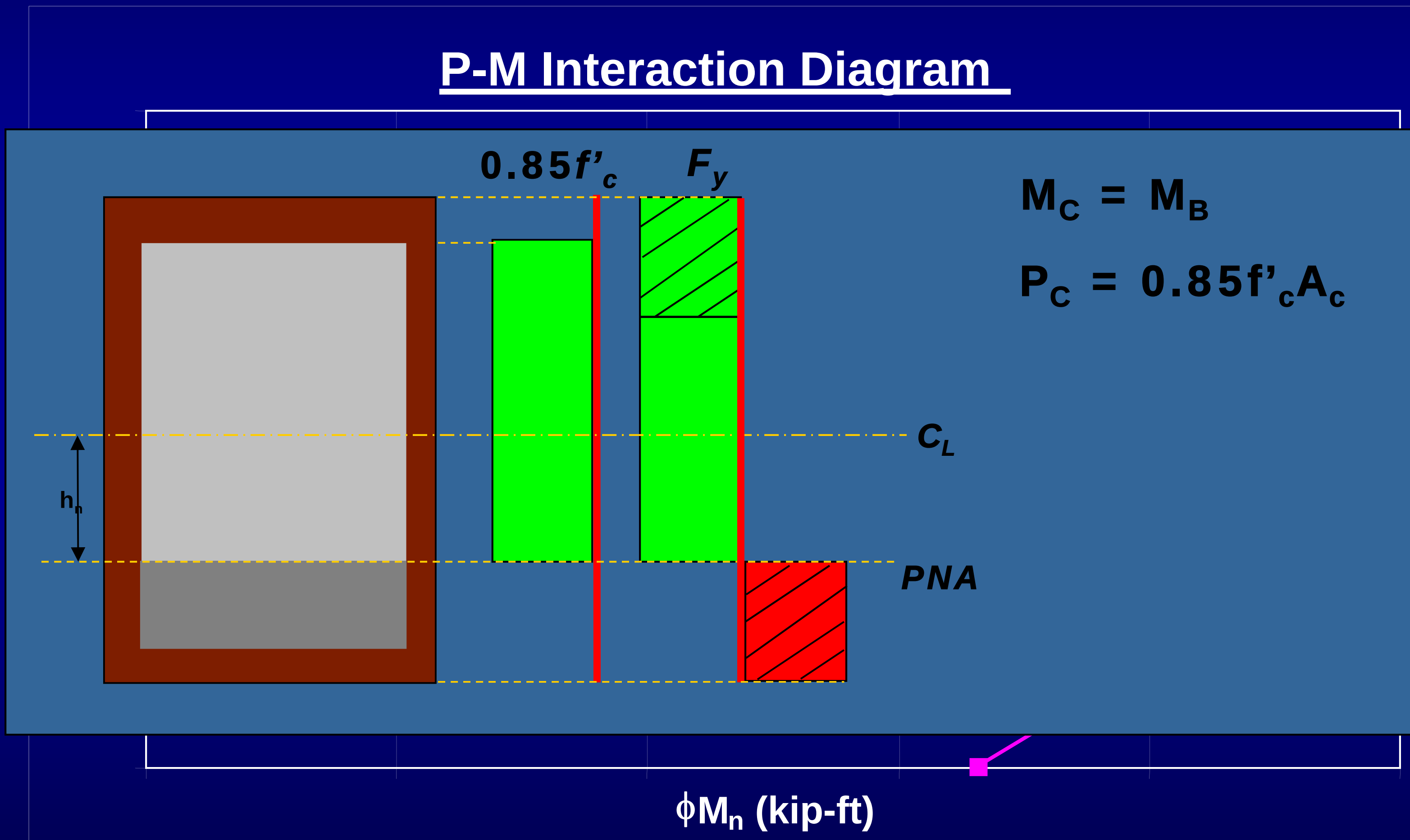
$$Z_{sn} = 2t_w h_n^2$$

$$Z_{cn} = h_1 h_n^2$$

$$h_n = \frac{0.85f'_c A_c}{2\left[0.85f'_c h_1 + 4t_w F_y\right]} \le \frac{h_2}{2}$$

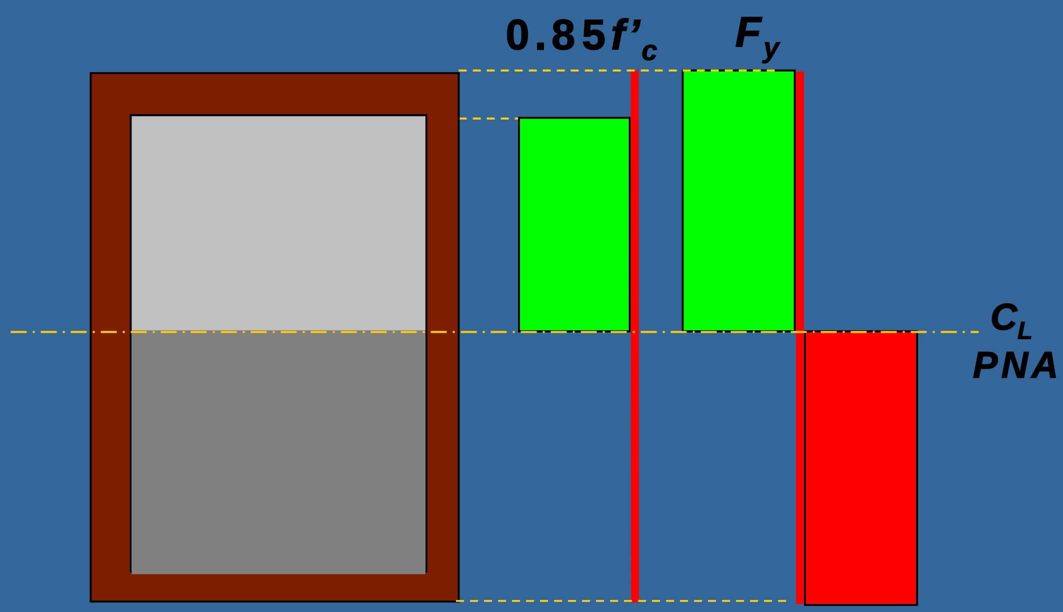
ϕM_n (kip-ft)

AISC 2005 Provisions for Composite Columns



AISC 2005 Provisions for Composite Columns

P-M Interaction Diagram



$$P_D = \frac{0.85f'_c A_c}{2}$$

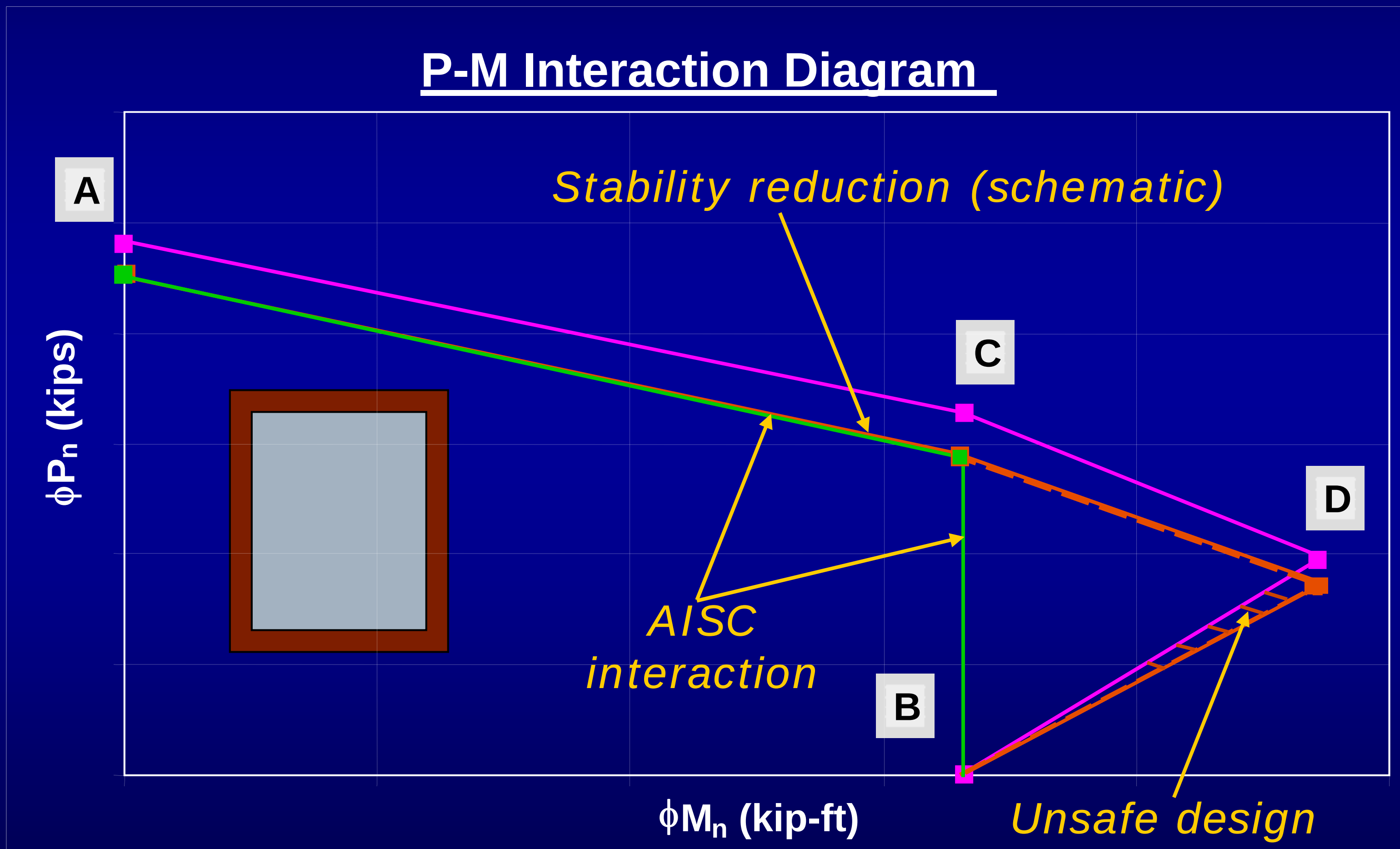
$$M_D = Z_s F_y + \tfrac{1}{2} Z_c (0.85f'_c)$$

Z_s = full y-axis plastic section modulus of steel shape

$$Z_c = \frac{h_1 h_2^2}{4} - 0.192 r_i^3$$

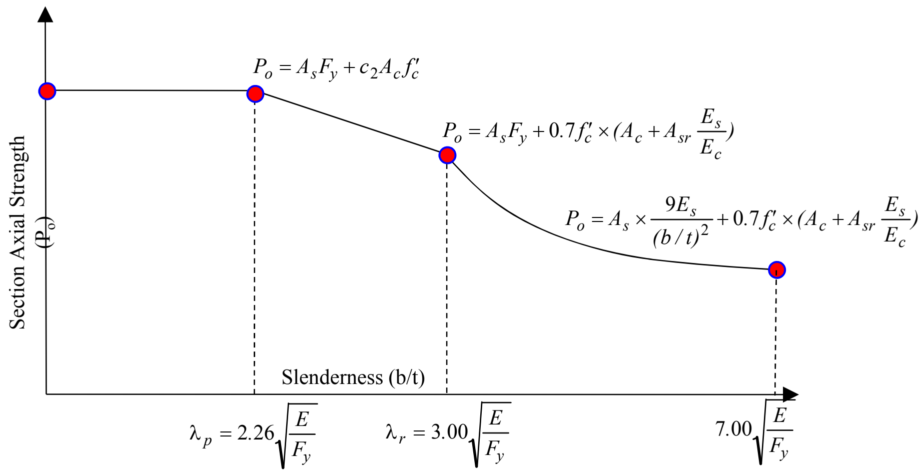
ϕM_n (kip-ft)

AISC 2005 Provisions for Composite Columns

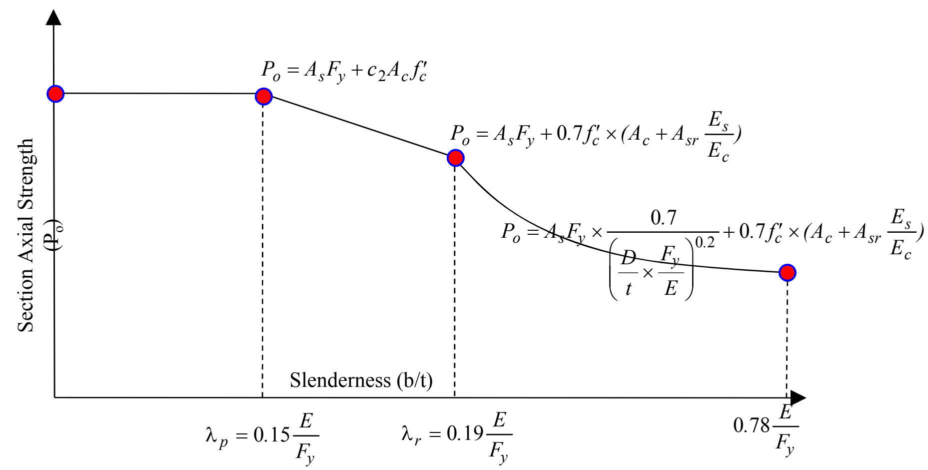


AISC 2010 Provisions for Composite Columns

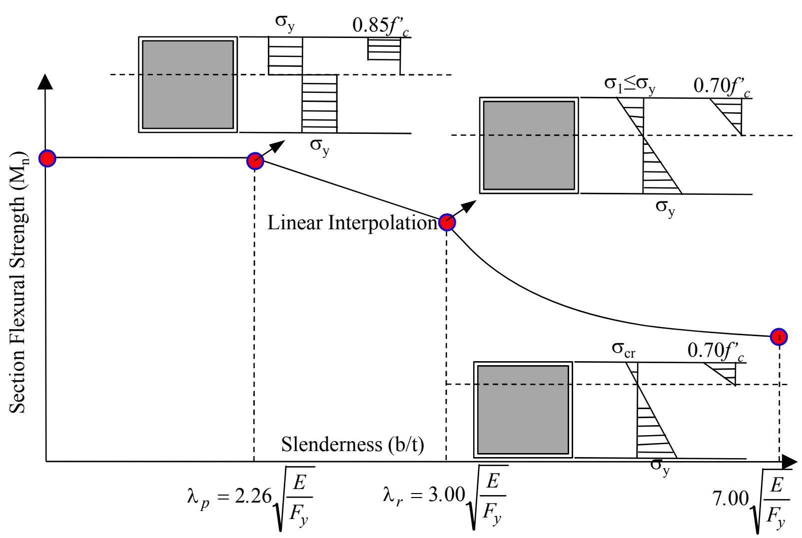
(a) Rectangular Filled Section Axial Strength as a Function of Wall Slenderness



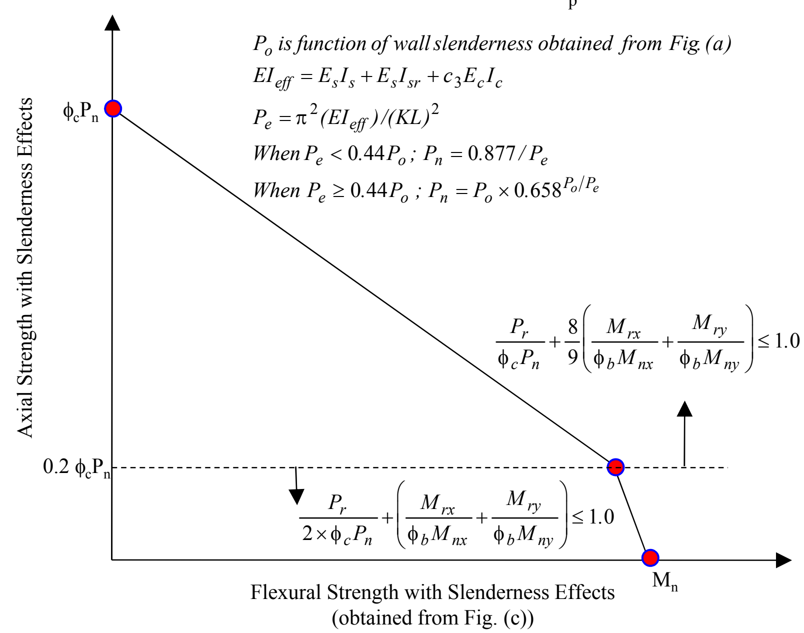
(b) Circular Filled Section Axial Strength as a Function of Wall Slenderness



(c) Rectangular Filled Section Flexural Strength as a Function of Wall Slenderness



(d) Axial Strength – Flexural Strength Interaction for Filled Columns with Wall Slenderness Greater than λ_p



AISC 2010 Load Transfer Provisions

Unified provisions for load transfer:

- Direct bearing (CFT, SRC, Composite Components)
- Bond interaction (CFT, Composite Components)
- Steel anchors (CFT, SRC, Composite Components), with adequate spacing and avoidance of concrete breakout failure

AISC 2010 Provisions: Bond Transfer

Rectangular CFTs

$$V_{in} = b^2 C_{in} F_{in}$$

Circular CFTs

$$V_{in} = 0.25\pi D^2 C_{in} F_{in}$$

V_{in} = *Nominal bond strength* < V_u/ϕ

C_{in} = *2 if CFT extend only above or below;*

4 otherwise

F_{in} = *Nominal bond stress = 0.06 ksi (0.4 MPa)*

b = *width of rectangular HSS face transferring load*

D = *diameter of circular HSS*

ϕ = *0.45 (large scatter in results)*

AISC 2010 Provisions: Steel Anchors

NON-SEISMIC PROVISIONS

- Shear: *h/d (height/depth of stud anchor) > 5* and:

$$Q_s = \phi_v C_v A_s F_u \text{ with } C_v = 0.65 \text{ and } \phi_v = 1.0 \text{ and } \beta = 4.0$$

- Tension: *h/d* >8 and:

$$Q_s = \phi_v C_v A_s F_u \text{ with } C_v = 0.75 \text{ and } \phi_v = 1.0 \text{ and } \beta = 4.0$$

- Interaction: *h/d* >8 and:

$$\left[\left(\frac{Q_t}{\phi_t Q_{nt}}\right)^{5/3} + \left(\frac{Q_v}{\phi_v Q_{nv}}\right)^{5/3}\right] \leq 1.0 \qquad \phi_t = 0.75; \phi_v = 0.65$$

- If dimensional limits are not met, use proper detailing or use ACI 318-08

SEISMIC PROVISIONS

- Use the AISC formulas but reduce the shear connector strength by 25%

New Organization in AISC 341-10: Composite integrated into provisions

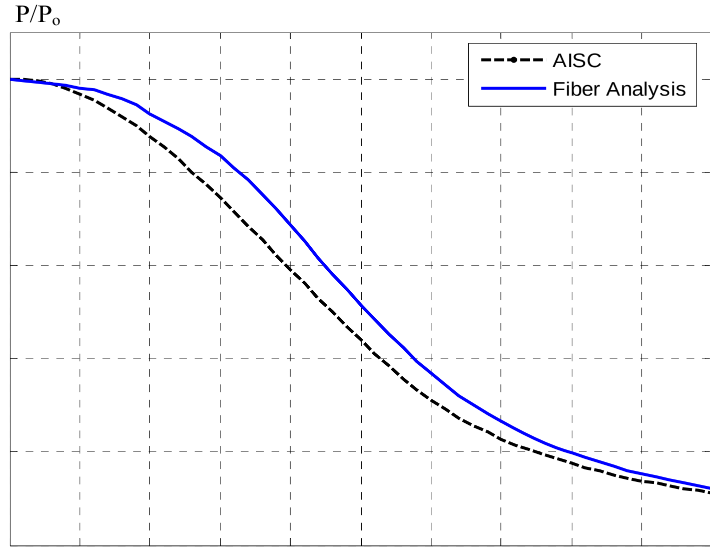
A. General Requirements
B. General Design Requirements
C. Analysis
D. General Member and Connection Requirements
E. Moment Frame Systems
F. Braced-Frame and Shear-Wall Systems
G. Composite Moment Frame Systems
H. Composite Braced-Frame and Shear-Wall Systems
I. Fabrication and Erection
J. Quality Assurance and Quality Control
K. Prequalification and Cyclic Qualification Testing

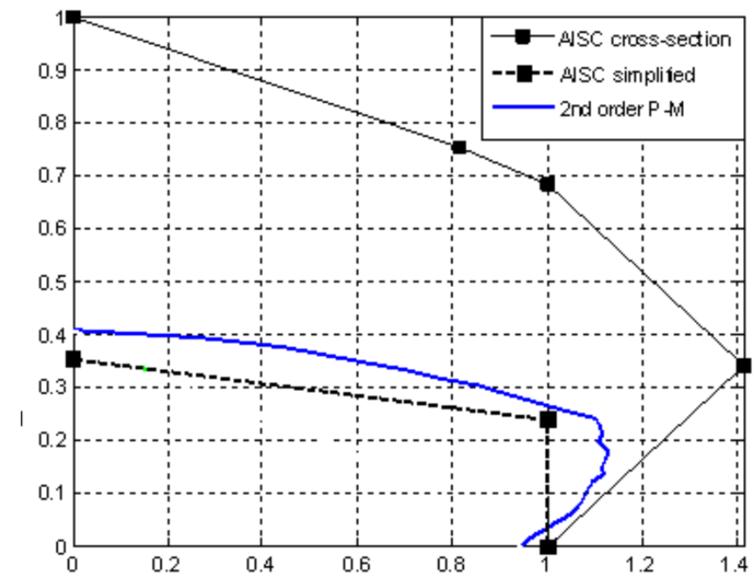
AISC 341-10 2010
Composite Seismic Systems

- Composite Moment Frames
  - Composite Ordinary Moment Frames
  - Composite Intermediate Moment Frames
  - Composite Special Moment Frames
  - Composite Partially-Restrained Moment Frames
- Composite Braced Frames
  - Composite Ordinary Braced Frames
  - Composite Special Concentrically Braced Frames
  - Composite Eccentrically Braced Frames
- Composite Walls (including coupling beams)
  - Composite Ordinary Shear Walls
  - Composite Special Shear Walls
  - Composite Plate Shear Walls

Ongoing Research
Design Recommendations

- **Design recommendations:**
  - Effective flexural (EI_{eff}) and torsional rigidity (GJ_{eff}) for 3D analysis
  - Critical load (P_n) and column curves (P_n-λ) for slender CFTs
  - *P-M* interaction for slender CFTs
  - System behavior factors for composite systems (R, C_d, Ω_o)
  - Direct analysis for composite systems





Non-Seismic and Seismic Design of Composite Beam-Columns and Composite Systems

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American Institute of Steel Construction
Georgia Institute of Technology
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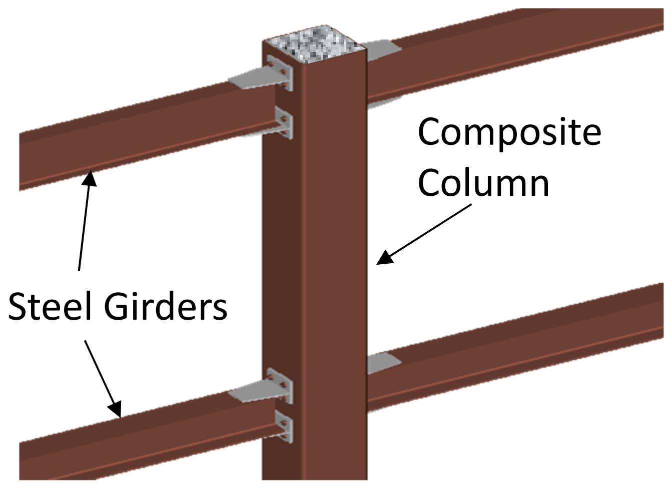
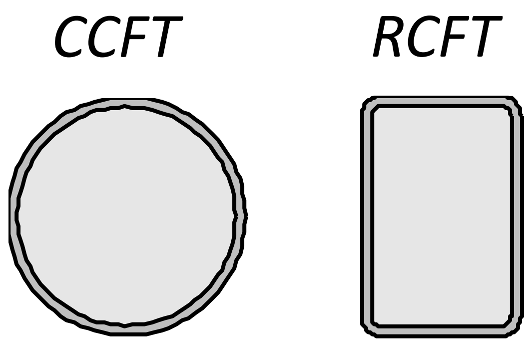
Introduction

- Experimental assessment of limit surface
  - Slender CFT beam-column tests
- Finite element formulation
  - Mixed beam-column element
  - Steel and concrete uniaxial cyclic materials
  - Localization and plastic hinge length
- Computational assessment of composite system behavior





CFT Test Matrix

Specimen name	L (ft)	Steel section HSS D x t	*Fy* (ksi)	*f_c'* (ksi)	*D/t*
1-C5-18-5	18	HSS5.563x0.134	42	5	45
2-C12-18-5	18	HSS12.75X0.25	42	5	55
3-C20-18-5	18	HSS20x0.25	42	5	86
4-Rw-18-5	18	HSS20x12x0.25	46	5	67
5-Rs-18-5	18	HSS20x12x0.25	46	5	67
6-C12-18-12	18	HSS12.75X0.25	42	12	55
7-C20-18-12	18	HSS20x0.25	42	12	86
8-Rw-18-12	18	HSS20x12x0.25	46	12	67
9-Rs-18-12	18	HSS20x12x0.25	46	12	67
10-C12-26-5	26	HSS12.75X0.25	42	5	55
11-C20-26-5	26	HSS20x0.25	42	5	86
12-Rw-26-5	26	HSS20x12x0.25	46	5	67
13-Rs-26-5	26	HSS20x12x0.25	46	5	67
14-C12-26-12	26	HSS12.75X0.25	42	12	55
15-C20-26-12	26	HSS20x0.25	42	12	86
16-Rw-26-12	26	HSS20x12x0.25	46	12	67
17-Rs-26-12	26	HSS20x12x0.25	46	12	67
18-C5-26-12	26	HSS5.563x0.134	42	12	45

Specimens designed for

Closing databases gaps in:

- *L, λ, D/t, f_c'*

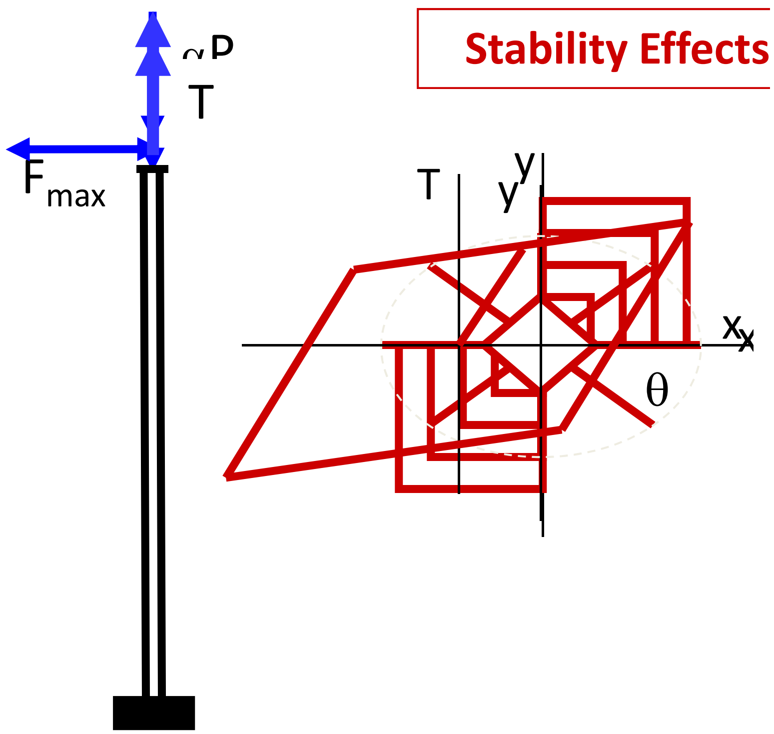
Maximize MAST capabilities:

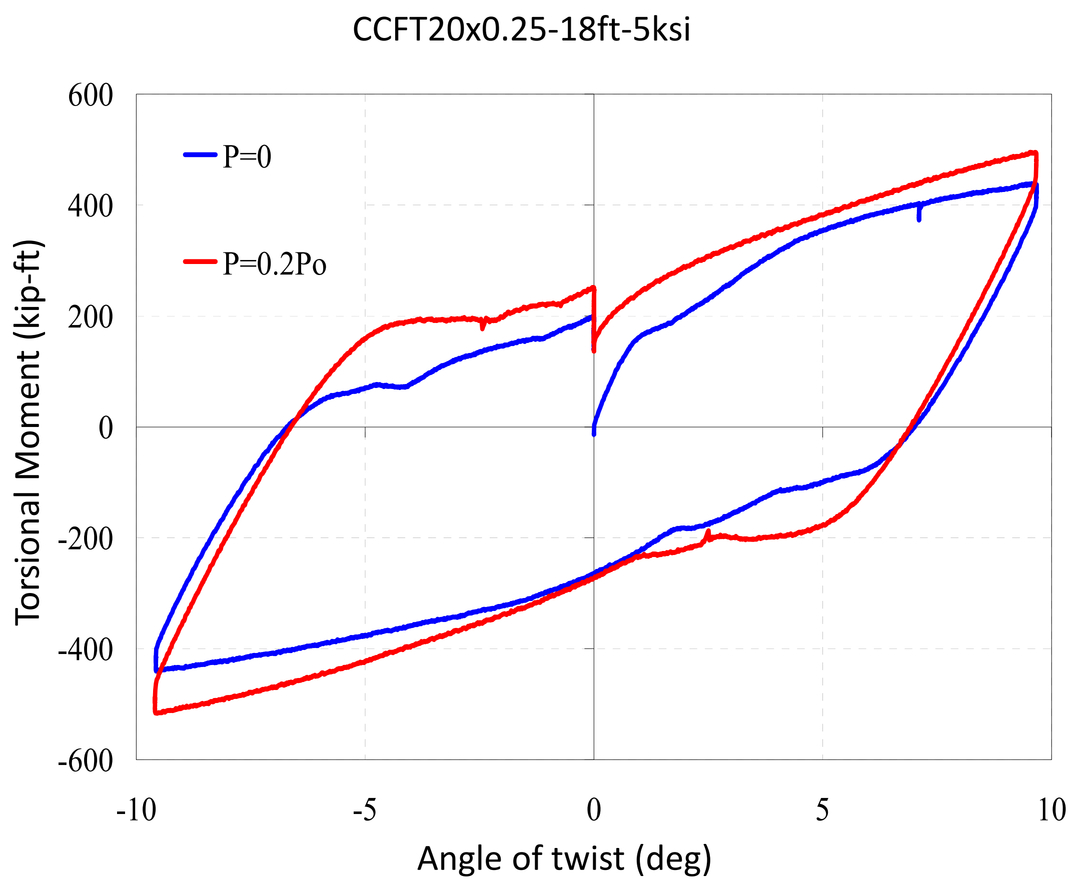
- *P_z = 1320 kip*
- *U_x=U_y=+/-16"*
- *18' < L < 26'*
- *Other constraints*

MAST Lab

Load protocol

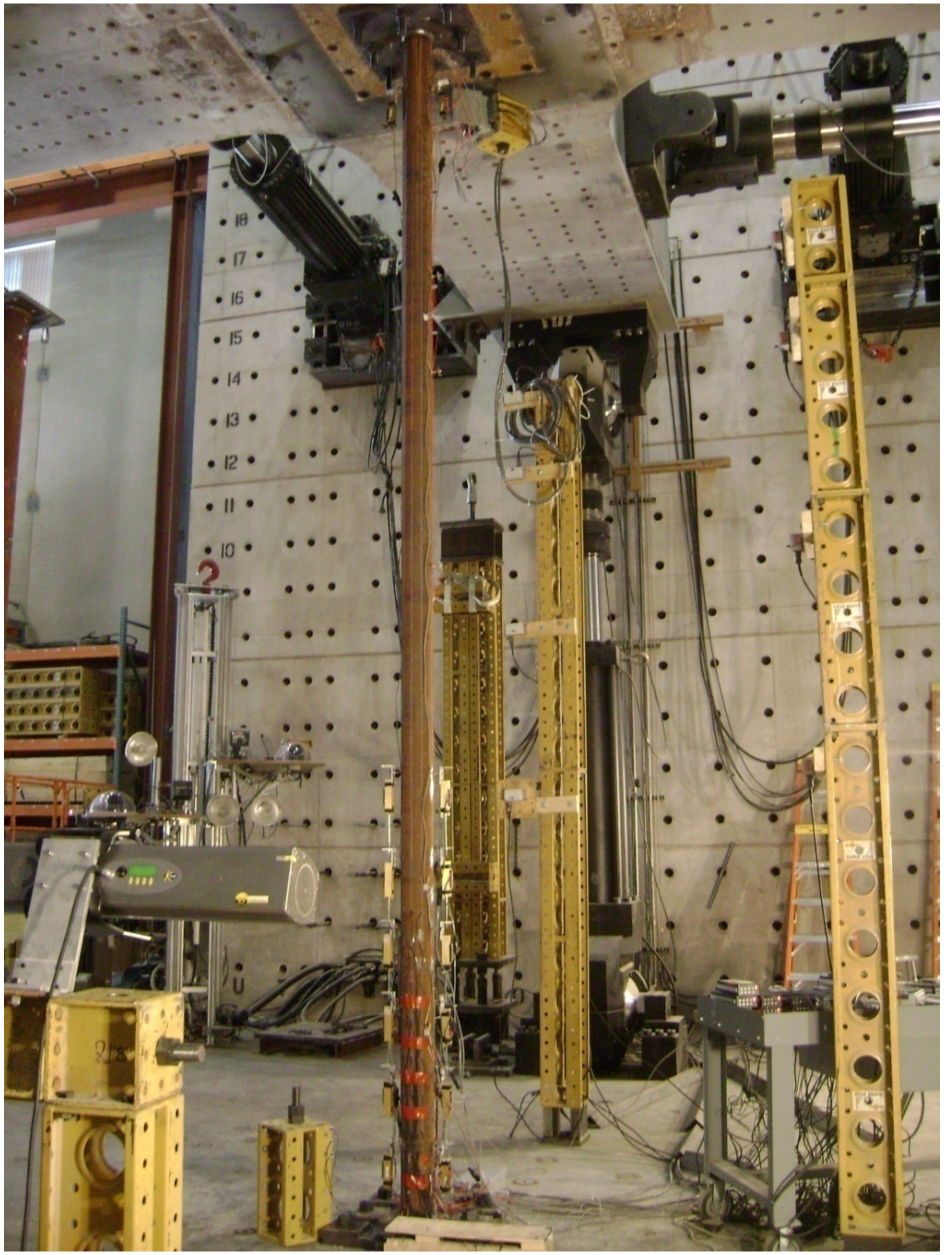
LC1 **, LC2** (a**LC4a**), **LC4b** **LC3** (a-c)

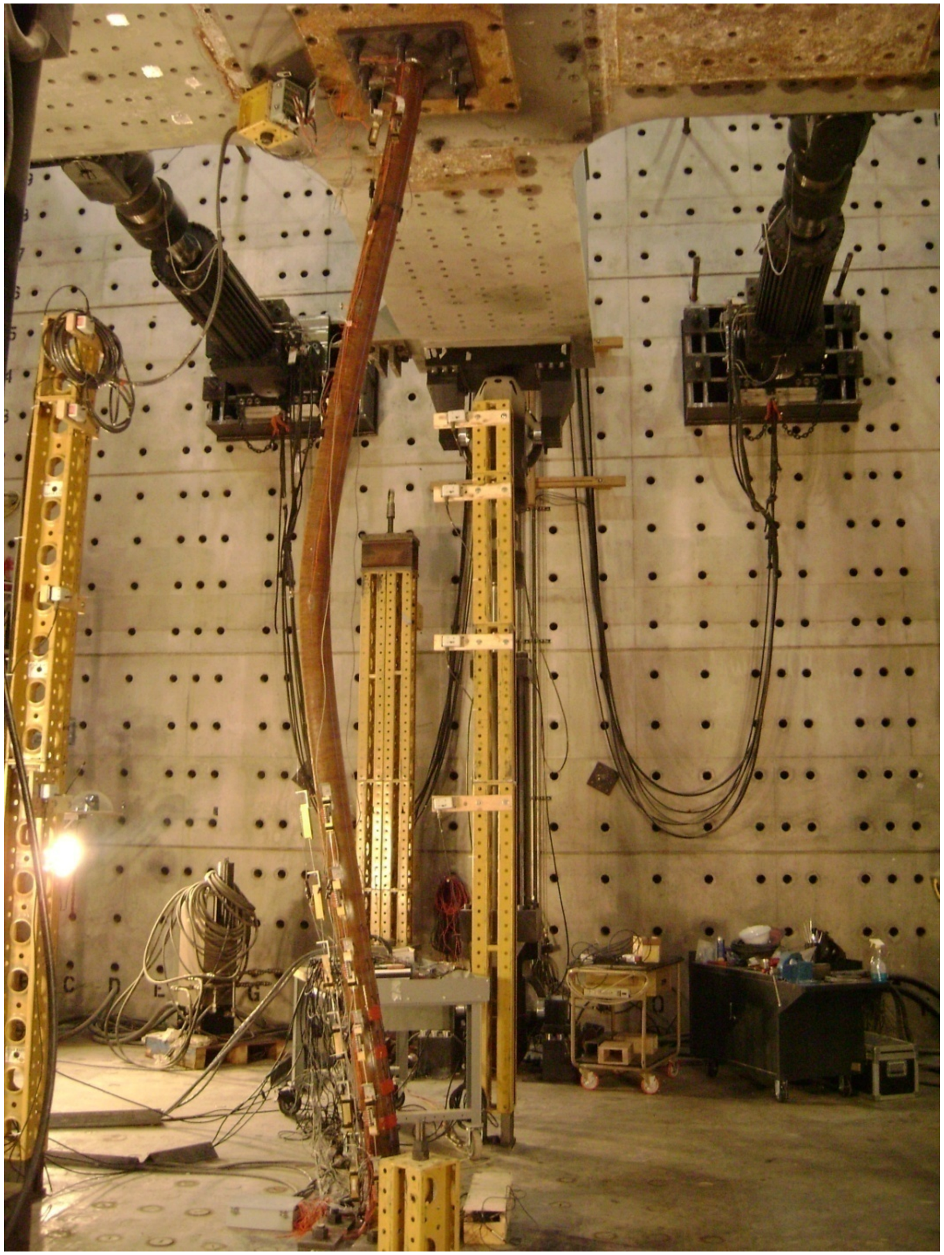


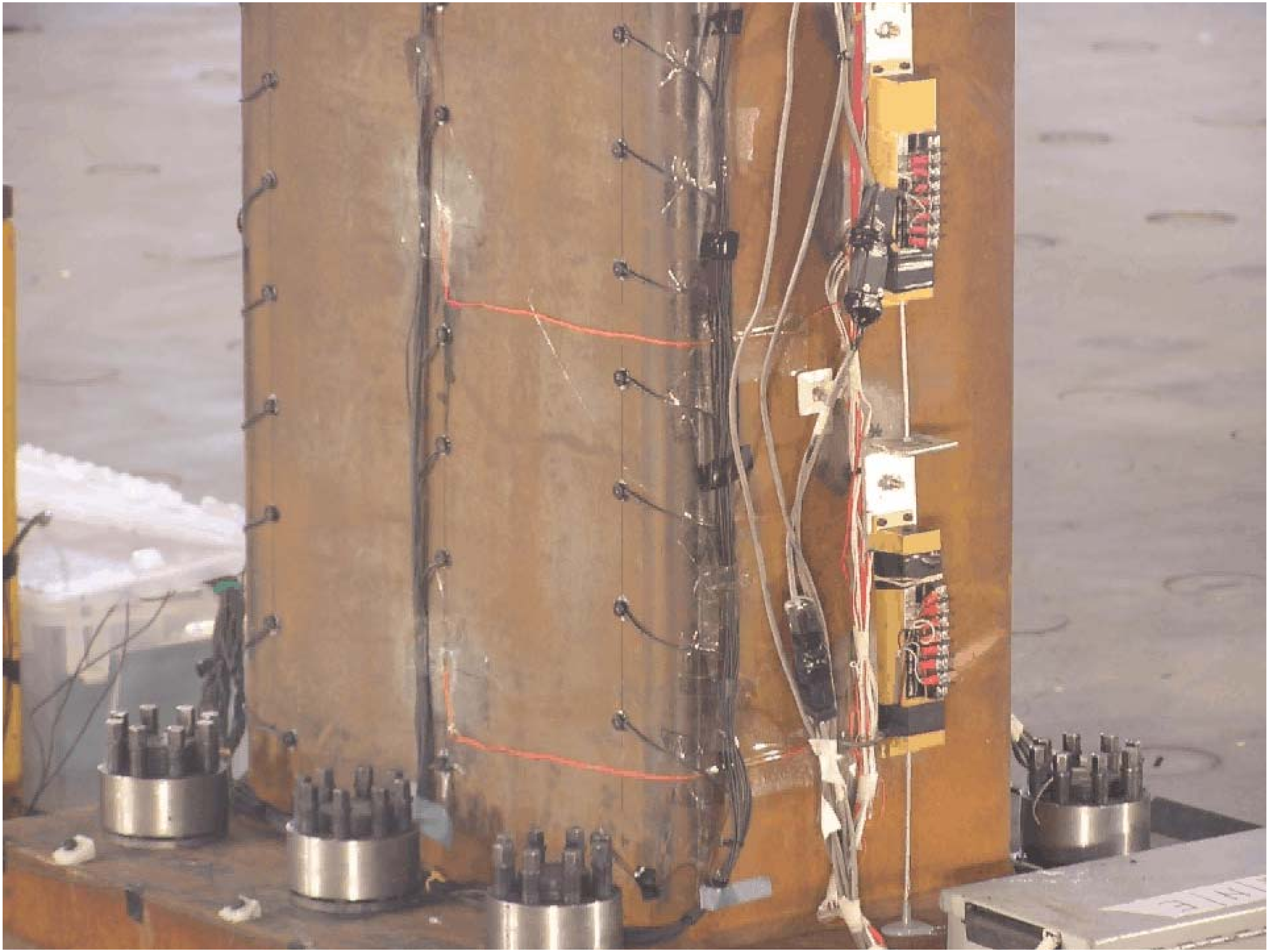


CCFT5.5x0.13-18ft-5ksi

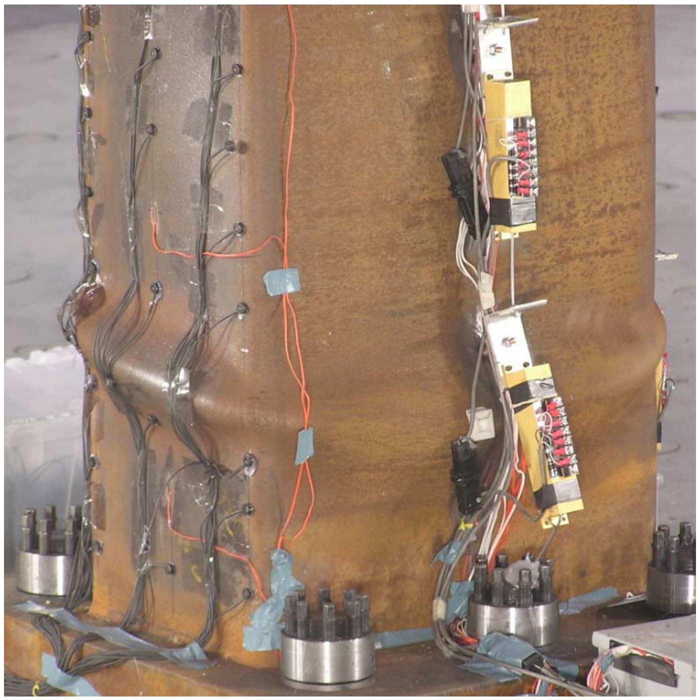
Yes, we buckled 'em!



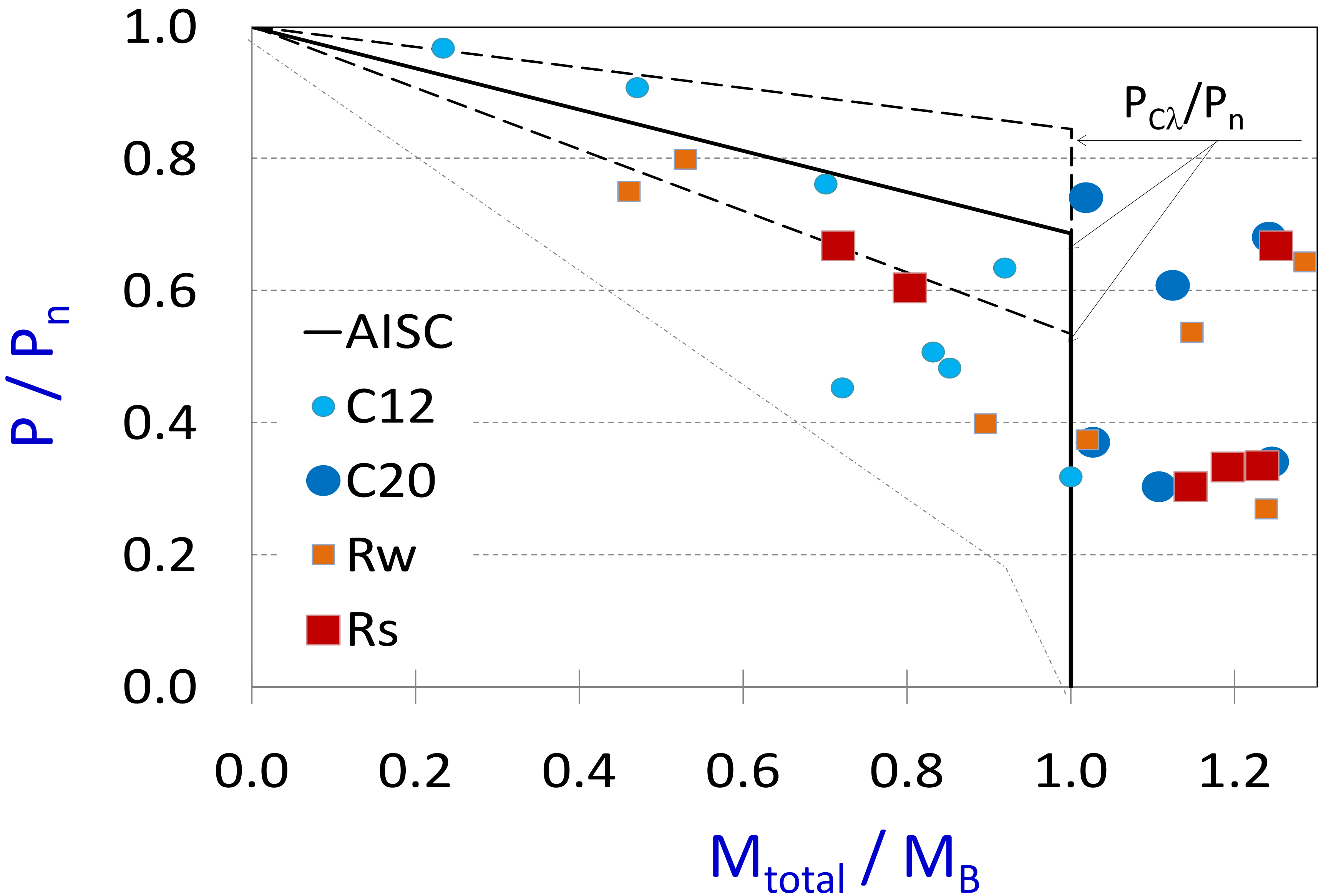




Typical Local Buckling Results



Experimental second order moments



Categories of Finite Element Formulations

Element Type

Beam

- El-Tawil and Deierlein 2001
- Alemdar and White 2005
- Tort and Hajjar 2007

Continuum

- Schneider 1998
- Johansson and Gylltoft 2002
- Varma et al. 2002
- Hu et al. 2003

Primary Unknown

Displacement

- Hajjar and Gourley 1997
- Aval et al. 2002
- Alemdar and White 2005

Force

- de Souza 2000
- El-Tawil and Deierlein 2001
- Alemdar and White 2005

Mixed

- Nukala and White 2004
- Alemdar and White 2005
- Tort and Hajjar 2007

Plasticity Type

Concentrated

- Hajjar and Gourley 1997
- El-Tawil and Deierlein 2001
- Inai et al. 2004

Distributed

- Hajjar et al. 1998
- Aval et al. 2002
- Varma et al. 2002
- Tort and Hajjar 2007

Constitutive Relation

Stress-Resultant

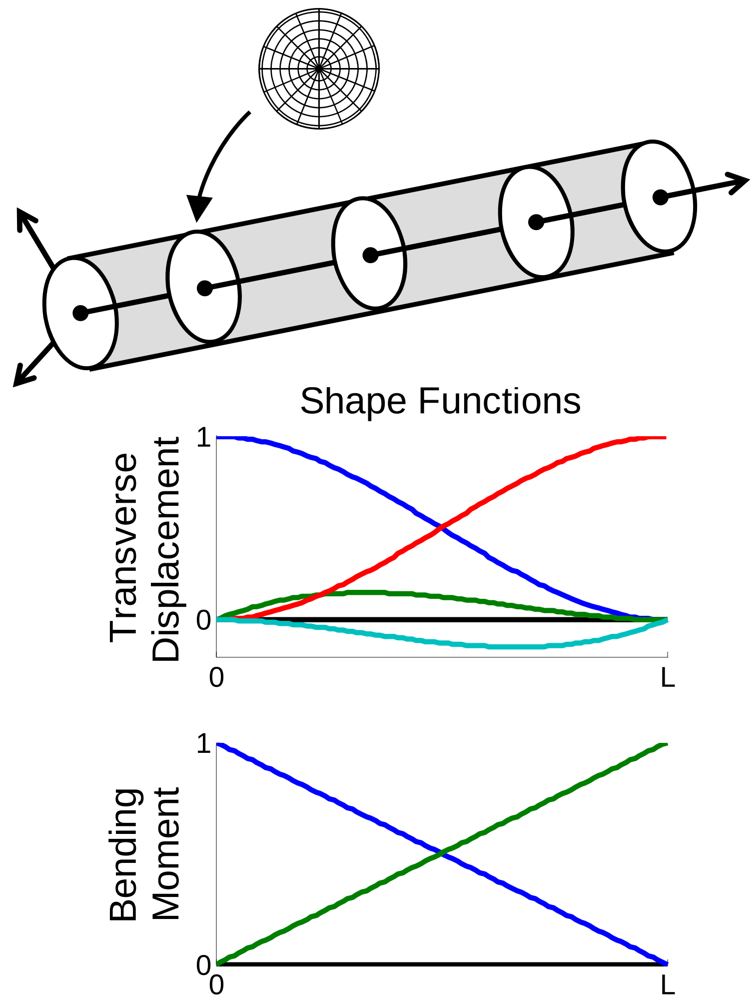
- Hajjar and Gourley 1997
- El-Tawil and Deierlein 2001

Fiber Section

- Hajjar et al. 1998
- Aval et al. 2002
- Varma et al. 2002
- Tort and Hajjar 2007

Mixed Beam-Column Element

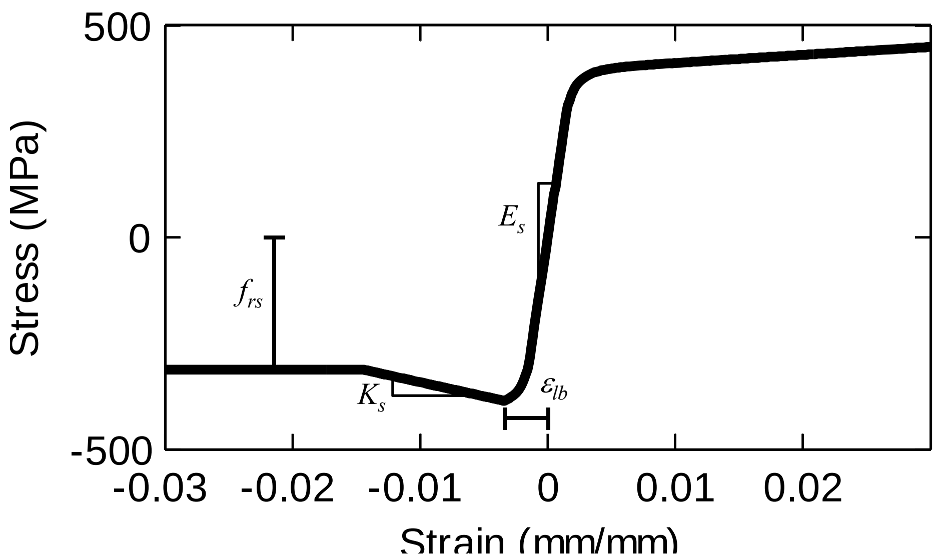
- Mixed formulation with both displacement and force shape functions
- Total-Lagrangian corotational formulation
- Distributed plasticity fiber formulation: stress and strain modeled explicitly at each fiber of cross section
- Suitable for static and dynamic analysis
- Implemented in the OpenSees framework



Uniaxial Cyclic Constitutive Relations

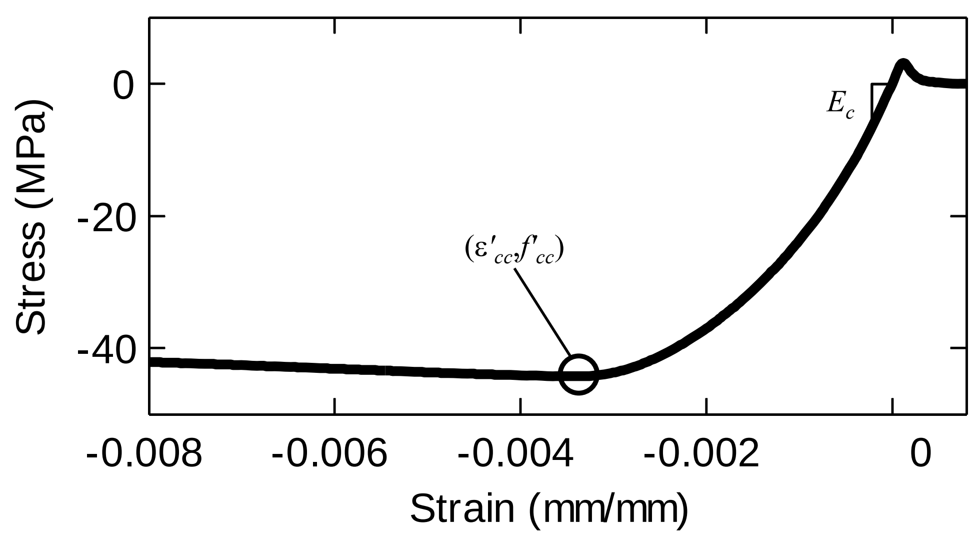
Steel

- Based on the bounding-surface plasticity model of Shen et al. (1995).
- Residual stresses modeled implicitly as an initial plastic strain
- No yield plateau, gradual transition to plasticity
- Modifications were made to model the effects of local buckling
  - Initiates with a strain limit
  - Linear degrading branch followed by constant stress branch

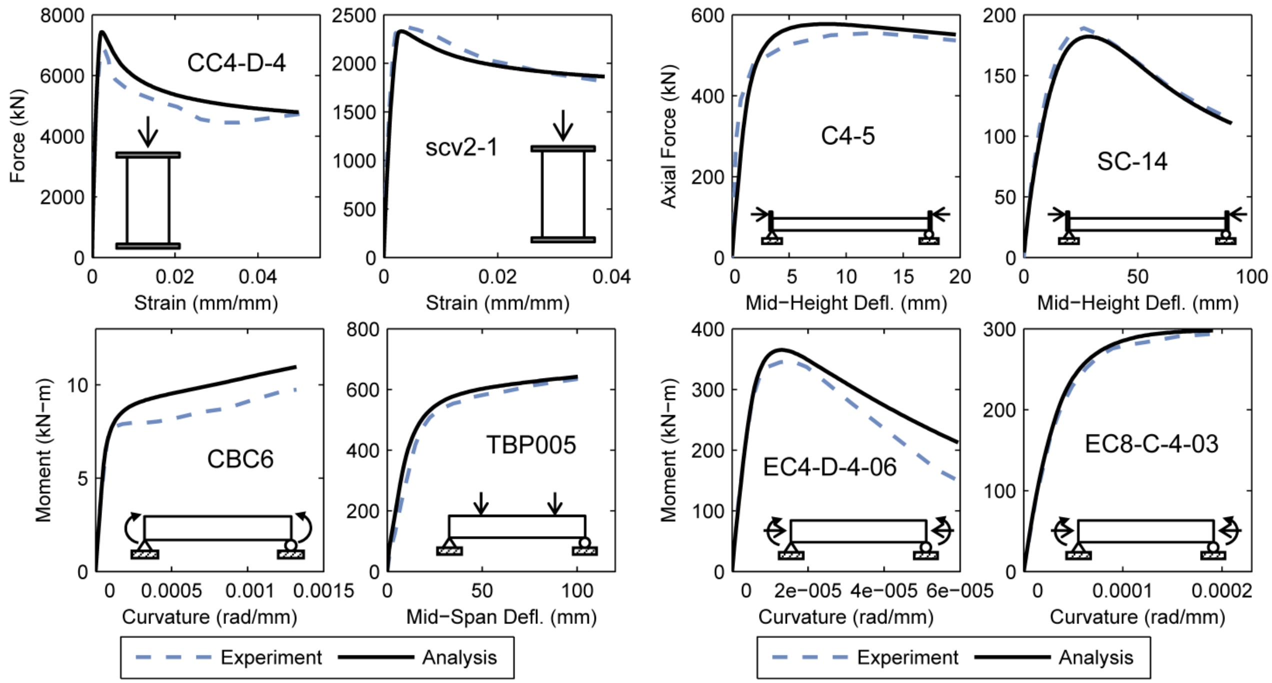


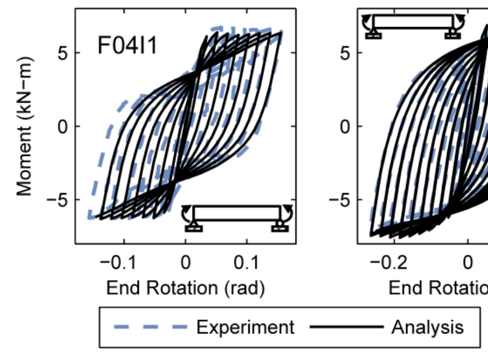
Concrete

- Based on the rule-based model of Chang and Mander (1994).
- Backbone stress-strain curve for the concrete is based on the model by Tsai, which is defined by:
  - Initial stiffness E_c
  - Peak coordinate $(\varepsilon'_{cc}, f'_{cc})$
  - r which acts as a shape factor.
- The confinement defined separately for each cross section



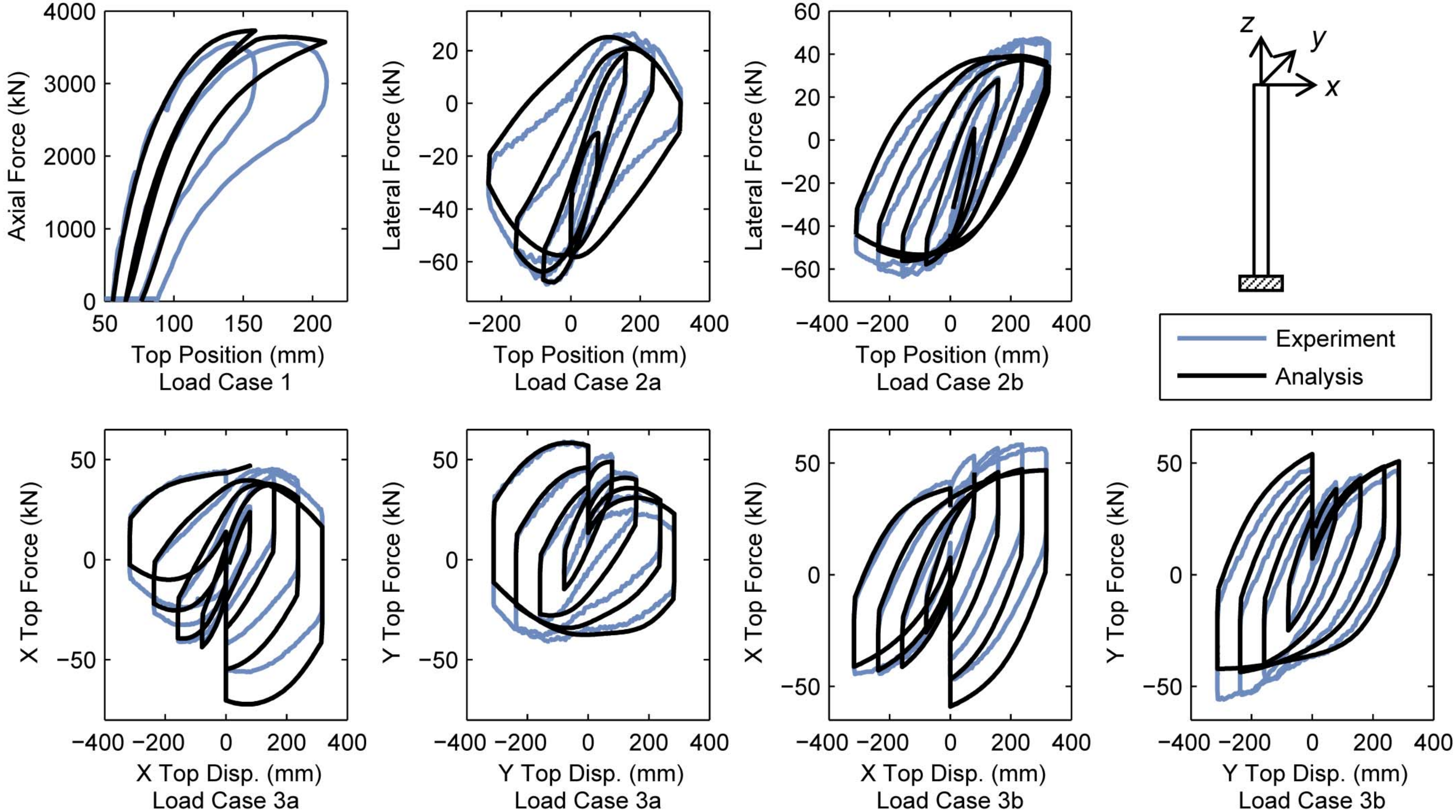
Validation of the Formulation





Specimen Name	Reference	Type	D (mm)	t (mm)	f'_c (MPa)	F_y (Mpa)	L (mm)	Other Details	# of FE
CC4-D-4	Yoshioka et al. 1995	SC	450	2.96	40.5	283	1,350	NA	**2**
scv2-1	Han and Yao 2004	SC	200	3.00	58.5	304	300	NA	**2**
CBC6	Elchalakani et al. 2001	BM	76.2	3.24	23.4	456	800	NA	**1**
TBP005	Wheeler and Bridge 2004	BM	456	6.40	48.0	351	3,800	NA	**4**
C4-5	Matsui and Tusda 1996	PBC	165	4.50	31.9	414	661	e = 103 mm	**4**
SC-14	Kilpatrick and Rangan 1999	PBC	102	2.40	58.0	410	1,947	e = 40 mm	**4**
EC4-D-4-06	Nishiyama et al. 2002	NBC	450	2.96	40.7	283	1,350	P = 4,488 kN	**1**
EC8-C-4-03	Nishiyama et al. 2002	NBC	222	6.47	40.7	834	666	P = 1,515 kN	**1**
F04I1	Elchalakani and Zhao 2008	CBM	110	1.25	23.1	430	800	NA	**2**
F14I3	Elchalakani and Zhao 2008	CBM	89.3	2.52	23.1	378	800	NA	**2**

Comparison with Analysis



Specimen: 11C20-26-5
CCFT 20x0.25

F_y = 44.3 ksi, f'_c = 8.1 ksi
L = 26 ft, KL = 52 ft

Finite Element Concentrated Plasticity "Macro" Model

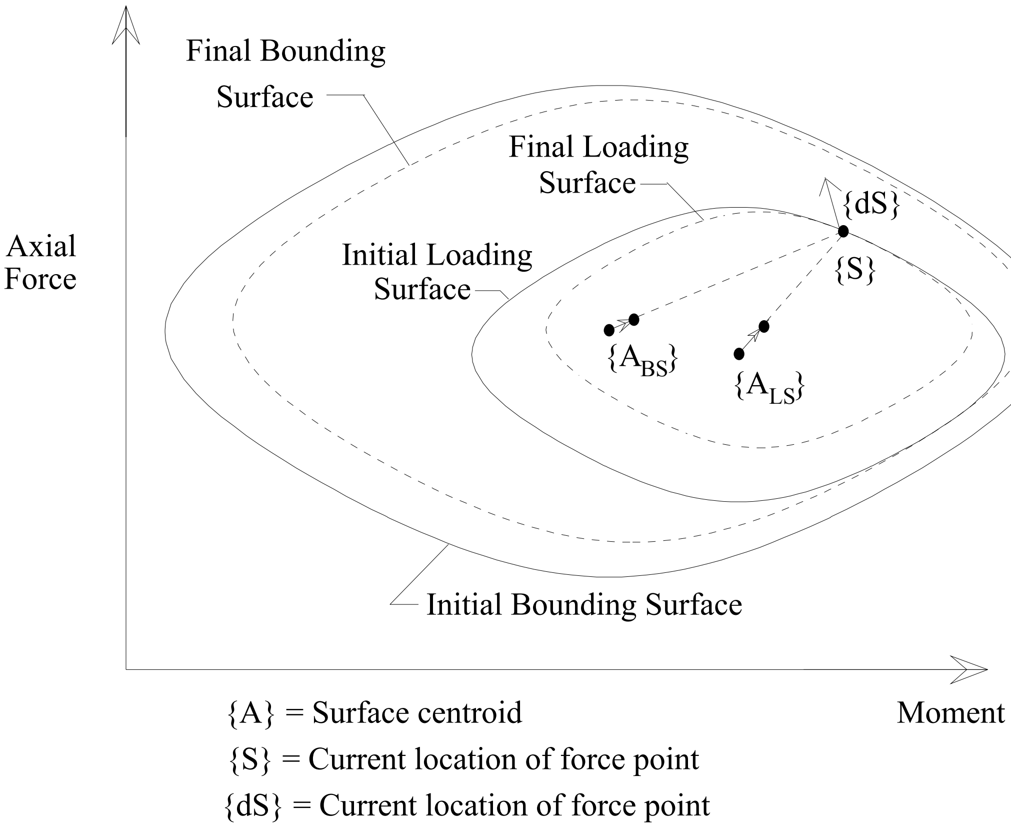
- Behavior is modeled at member ends (at the centroidal axis) by:
  - Deformations (displacements and rotations)
  - Stress-resultants (forces and bending moments)

- Standard 3D 12 degree-of-freedom beam element
- Effective elastic rigidities and updated Lagrangian geometric nonlinearity
- Concentrated plasticity constitutive formulation

Macro Model Plasticity Formulation

•Plastification is handled as a two step process:

1. Isotropic hardening
2. Kinematic hardening



Common assumption is that plasticity "yield" surfaces may change position and size but not shape

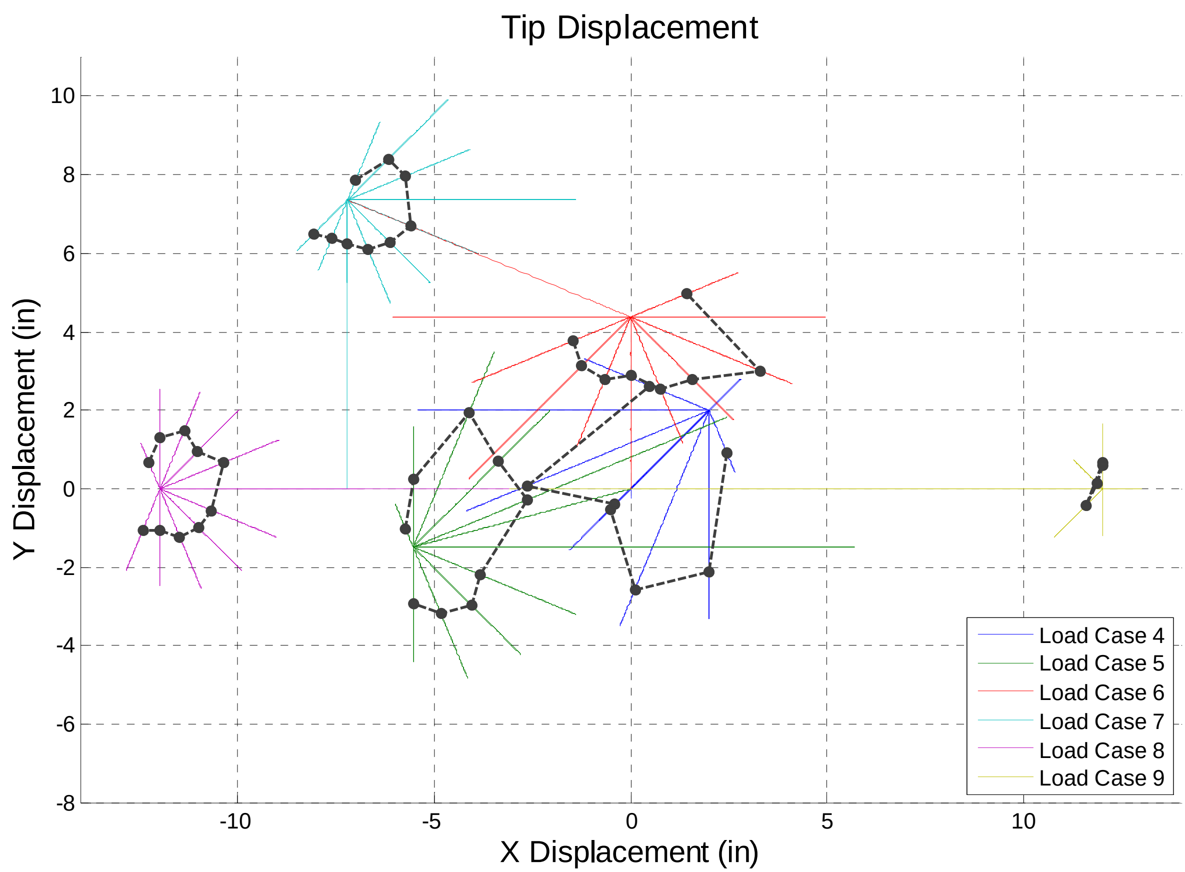
Experimental Assessment of Limit Surfaces

Evolution of Limit Surface

Specimen:
9Rs-18-12

RCFT20x12x0.3125
F_y = 53.0 ksi
f'_c = 13.3 ksi
L = 18 feet
KL = 36 feet

Axial Compression
800 kips

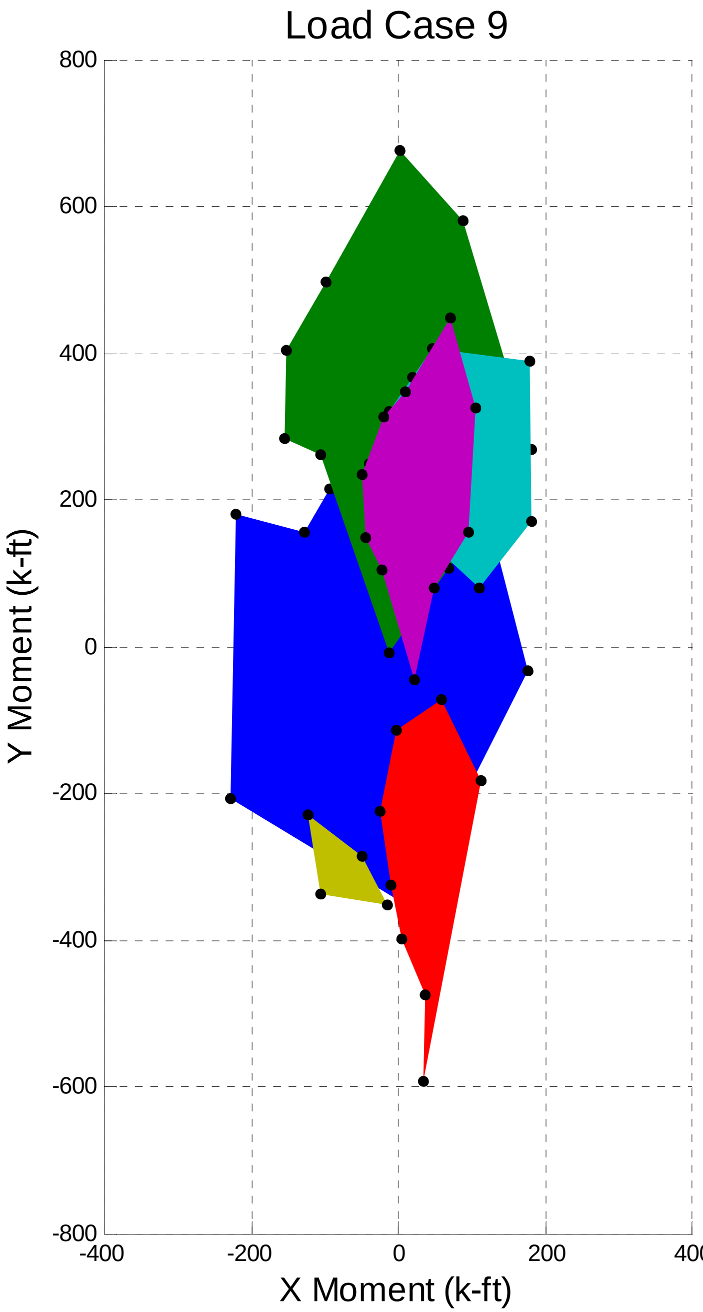


Evolution of Limit Surface

Specimen:
9Rs-18-12

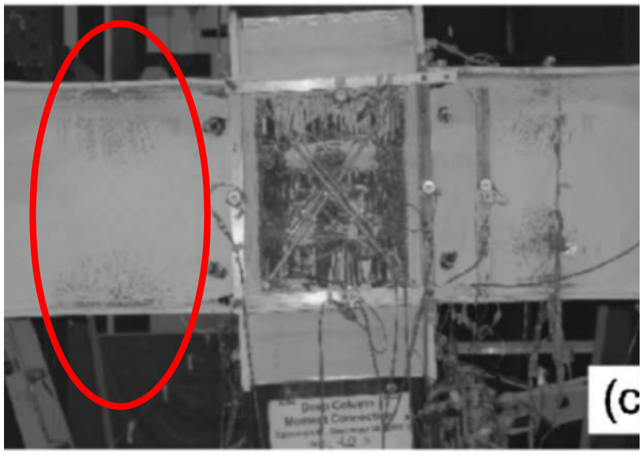
RCFT20x12x0.3125
F_y = 53.0 ksi
f'_c = 13.3 ksi
L = 18 feet
KL = 36 feet

Axial Compression
800 kips



Are we really going about this correctly?

Building codes use ductility from inelastic actions to protect structures against collapse, particularly during large earthquakes.

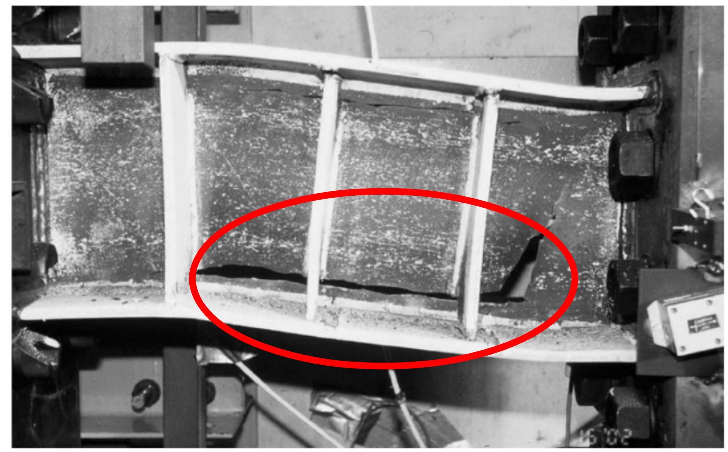


Moment-Resisting Frames

From Zhang and Ricles 2006

Concentrically Braced Frames

From Uriz and Mahin 2004

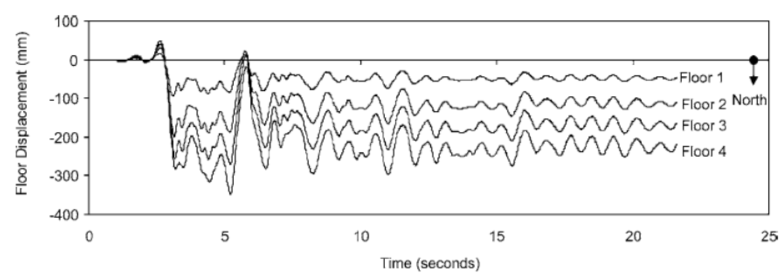


Eccentrically Braced Frames

From Okazaki et al. 2005

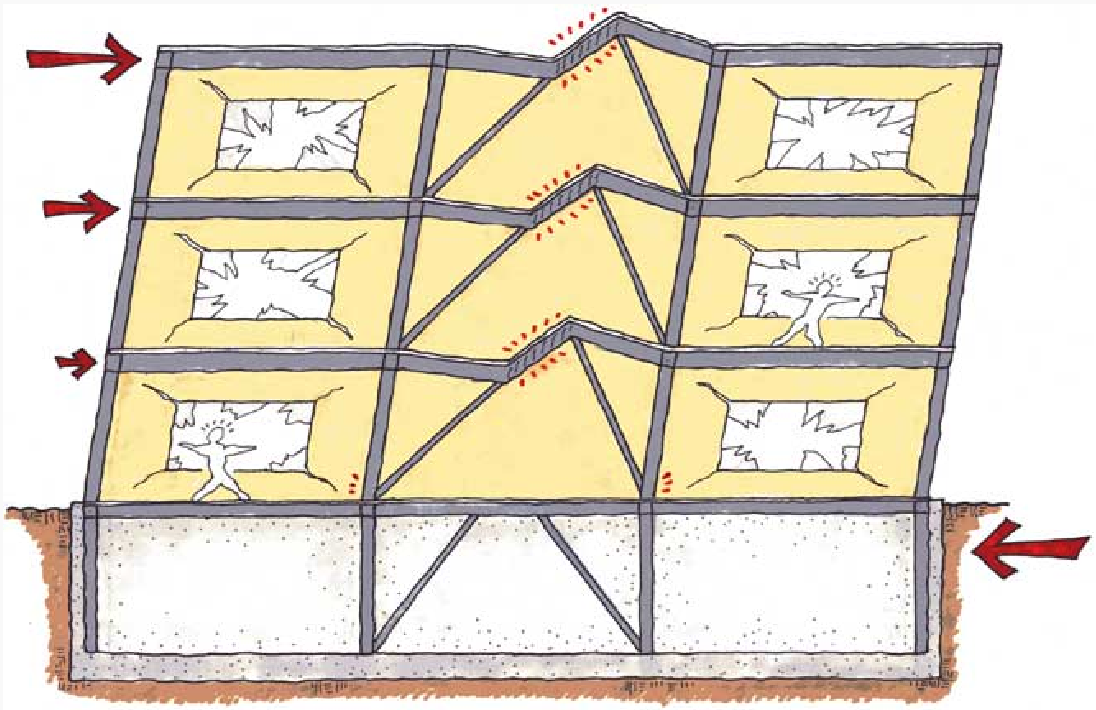
Look at the results of new BRBF systems:

- **Distributed Structural Damage**
- **Residual Drifts**



From Fahnestock et al. 2007

Unsustainable Engineering: Today's Norm



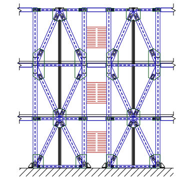
Deformed Section – Eccentric Braced Frame

Costly Permanent Damage:
Structure and Architecture absorbs energy through damage

Large Inter-story Drifts:
Result in architectural & structural damage

High Accelerations:
Result in content damage & loss of function

TIPPING · MAR + associates
structural engineers



NEESR-SG: Controlled Rocking of Steel-Framed Buildings with Replaceable Energy Dissipating Fuses

Gregory G. Deierlein, Helmut Krawinkler, Xiang Ma, *Stanford University*

Jerome F. Hajjar, *Northeastern University;* Matthew Eatherton, *Virginia Tech*

Mitsumasa Midorikawa, Tetsuhiro Asari, Ryohei Yamazaki, *Hokkaido University*

Toru Takeuchi, Kazuhiko Kasai, Shoichi Kishiki, Ryota Matsui, Masaru Oobayashi,

Yosuke Yamamoto, *Tokyo Institute of Technology*

Tsuyoshi Hikino, *Hyogo Earthquake Engineering Research Center, NIED*

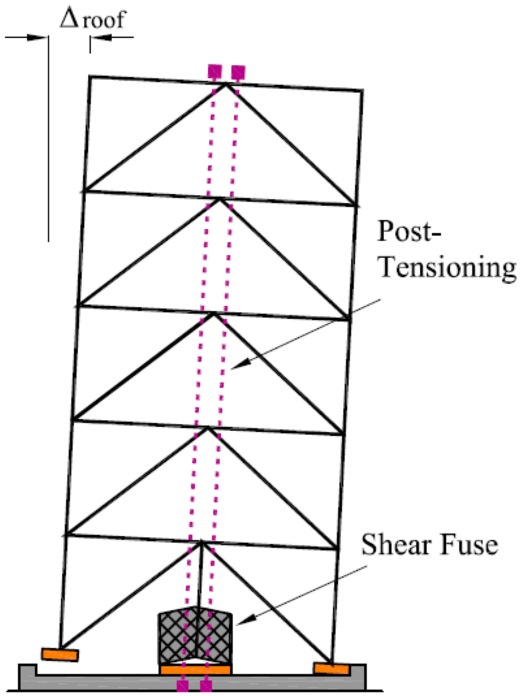
David Mar, *Tipping & Mar Associates* and Greg Luth, *GPLA*

E-Defense

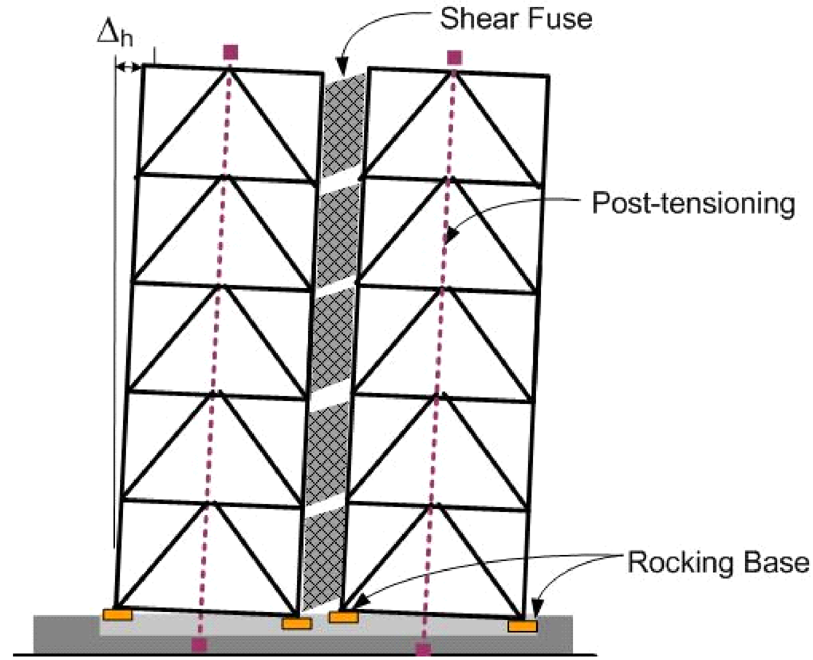
JISF The Japan Iron and Steel Federation

In-Kind Funding: ***Tefft Bridge and Iron*** *of Tefft, IN,* ***MC Detailers*** *of Merrillville, IN,* ***Munster Steel Co. Inc.*** *of Munster, IN,* ***Infra-Metals*** *of Marseilles, IN, and* ***Textron/Flexalloy Inc. Fastener Systems Division*** *of Indianapolis, IN.*

Controlled-Rocking System



single frame

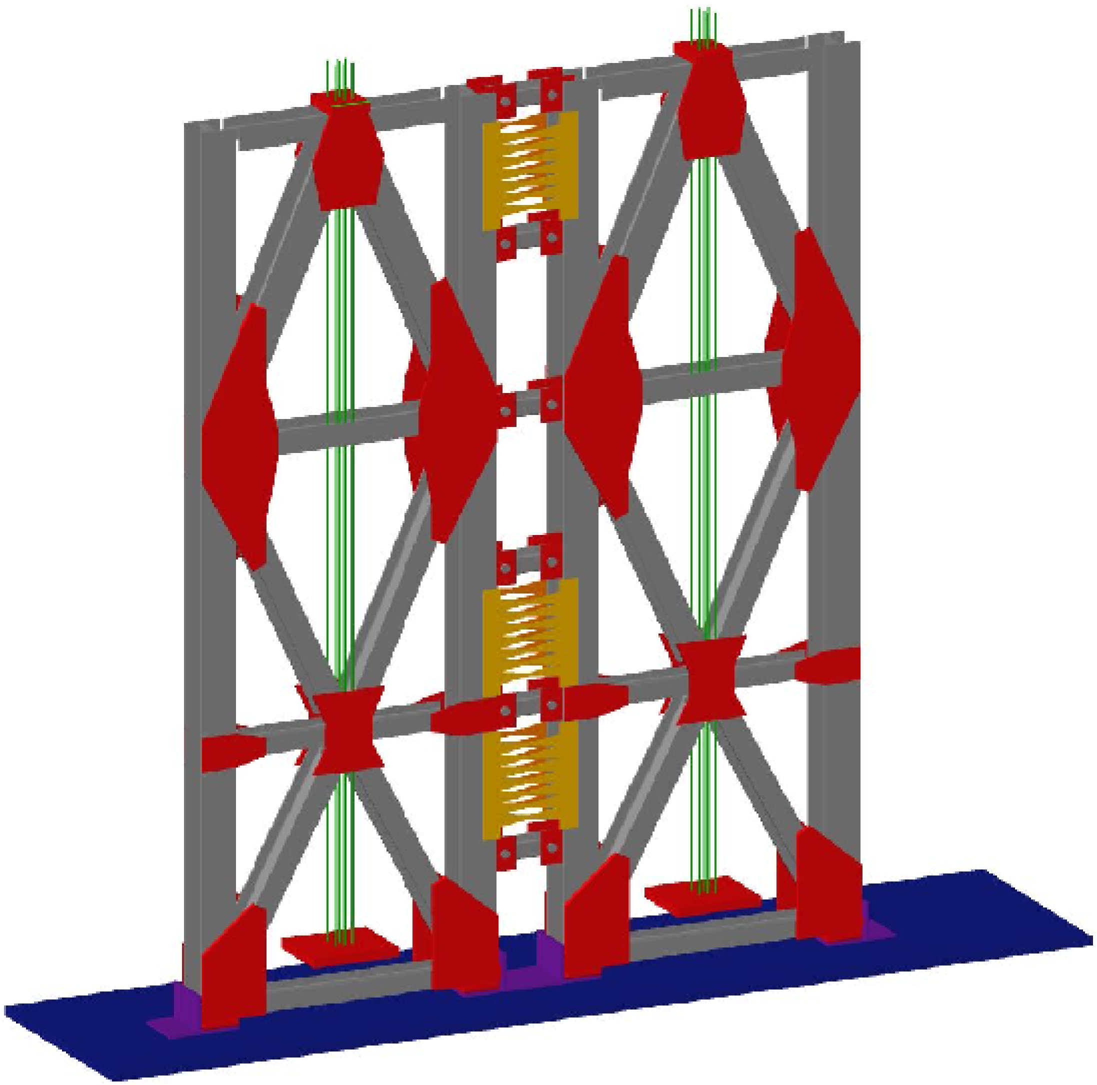


dual frames

Develop a new structural building system that employs *self-centering rocking* action and *replaceable* fuses to provide safe and cost effective earthquake resistance.

-- minimize structural damage and risk of building closure

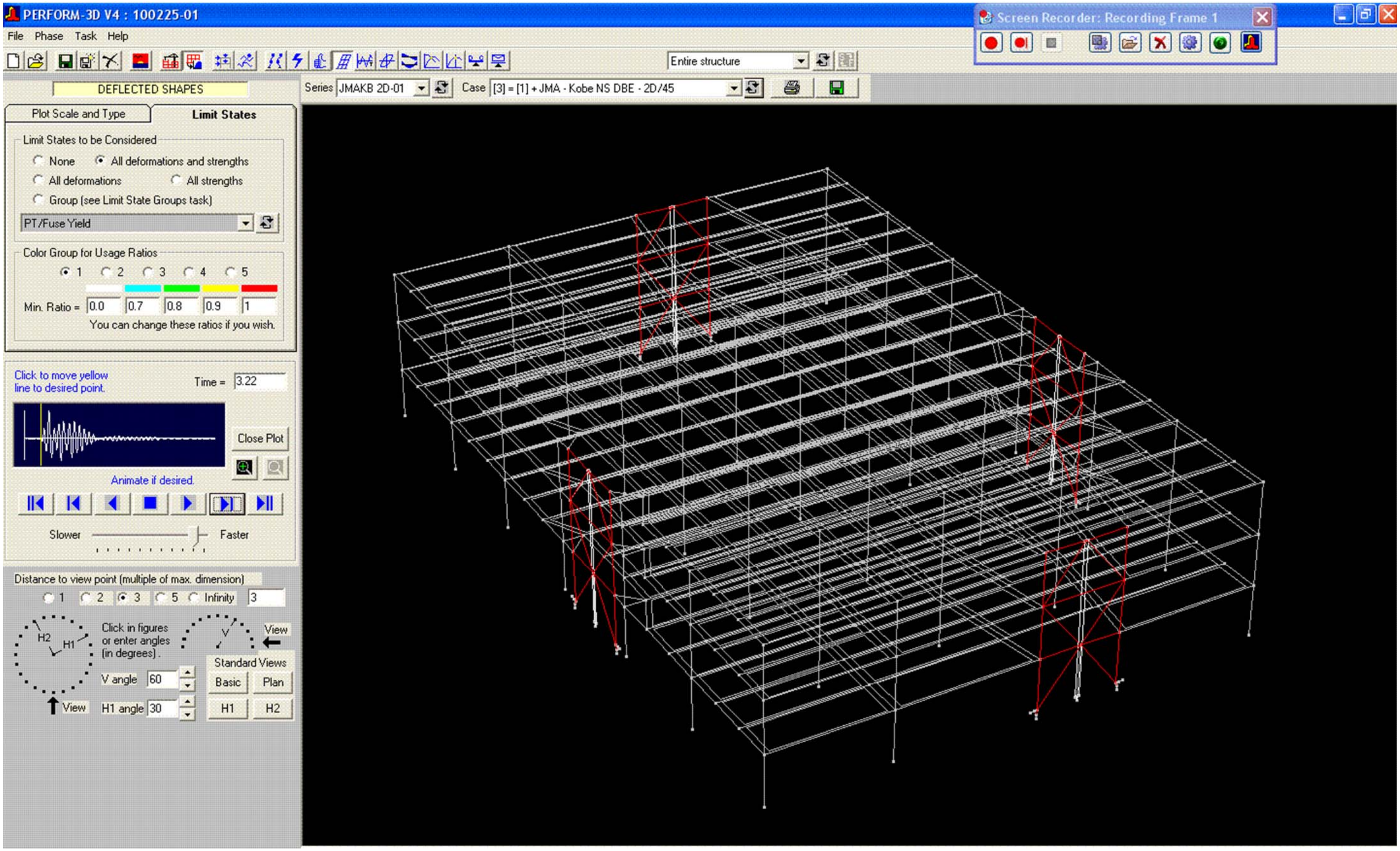
Controlled-Rocking System

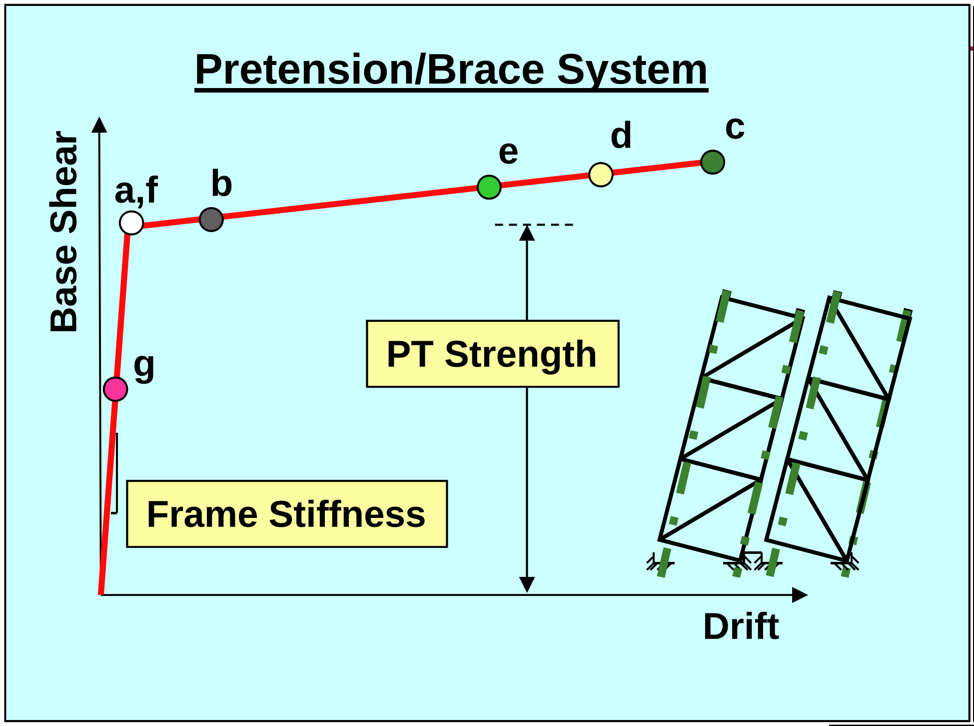


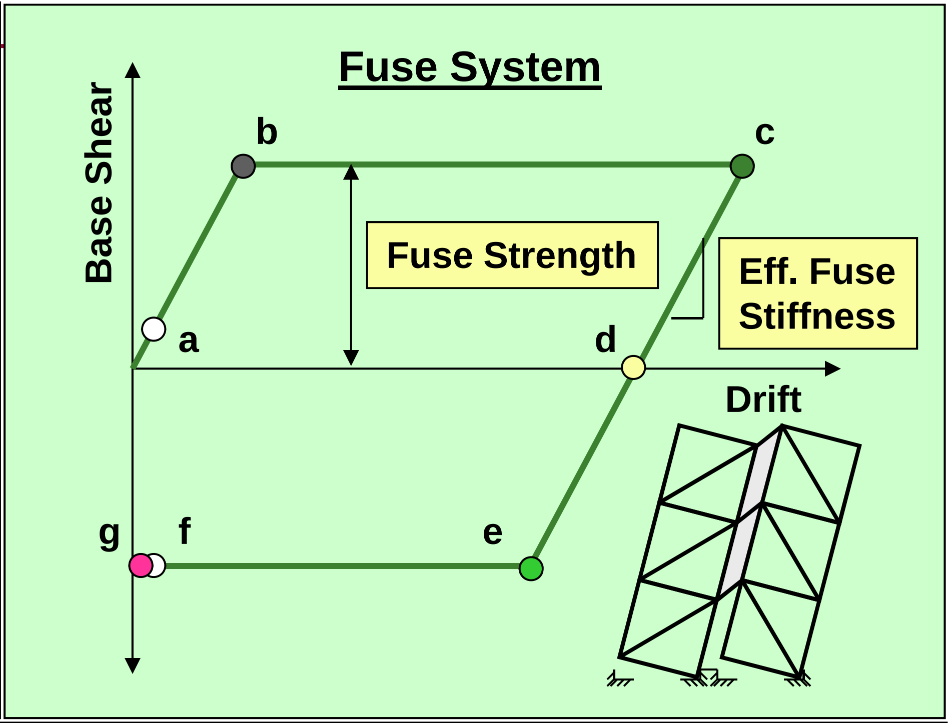
- Corner of frame is allowed to uplift.
- Fuses absorb seismic energy
- Post-tensioning brings the structure back to center.

Result is a building where the structural damage is concentrated in replaceable fuses with little or no residual drift

Controlled-Rocking in *3D*







Origin-a – *frame strain + small distortions in fuse*

a – *frame lift off, elongation of PT*

b – *fuse yield (+)*

c – *load reversal (PT yields if continued)*

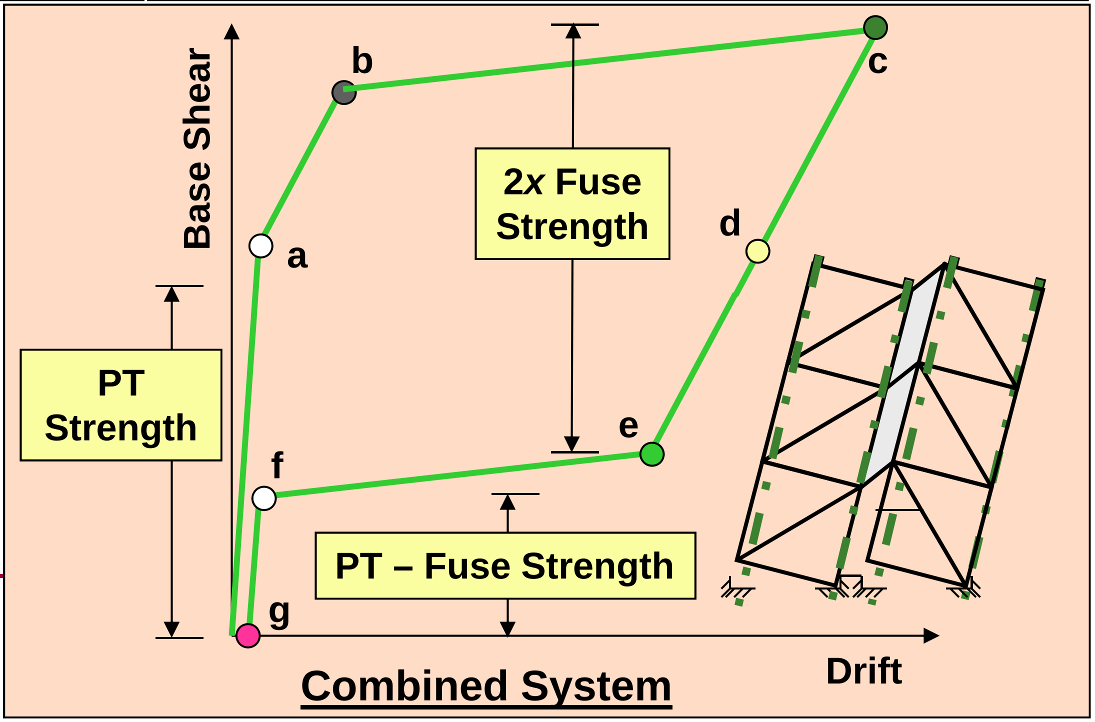
d – *zero force in fuse*

e – *fuse yield (-)*

f – *frame contact*

f-g – *frame relaxation*

g – *strain energy left in frame and fuse, small residual displacement*



Parametric Study of Prototype: Parameters Studied

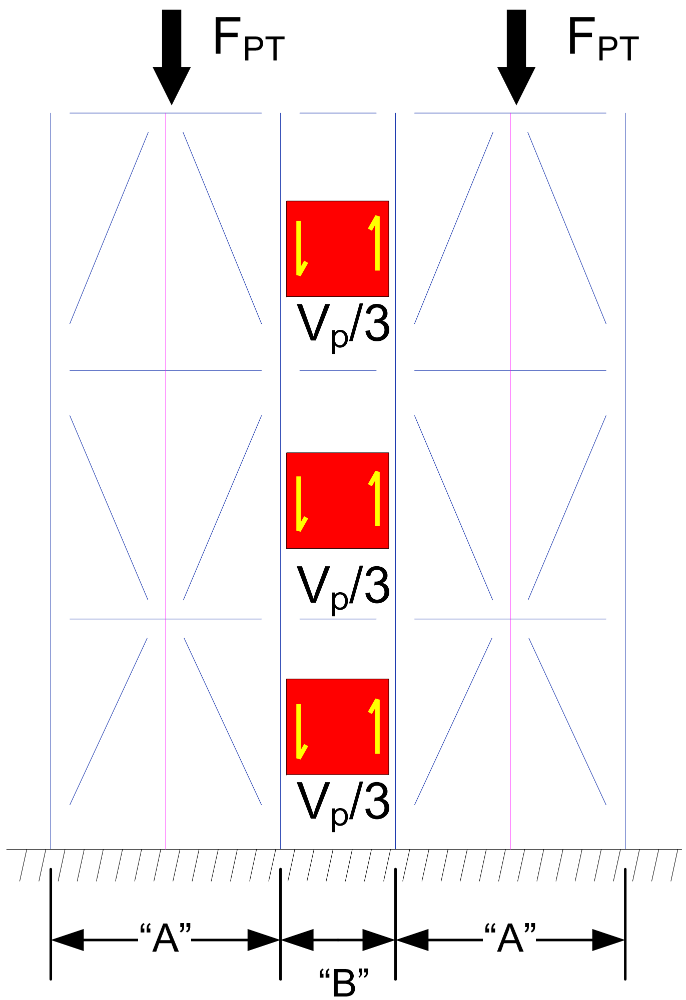
1. A/B ratio – geometry of frame
2. Overturning Ratio (OT) – ratio of resisting moment to design overturning moment. OT=1.0 corresponds to R=8.0, OT=1.5 means R=5.3

$$OT = \frac{M_{resist}}{M_{OVT}} = \frac{A\,F_{PT} + V_P(A+B)}{M_{OVT}}$$

3. Self-Centering Ratio (SC) – ratio of restoring moment to restoring resistance.

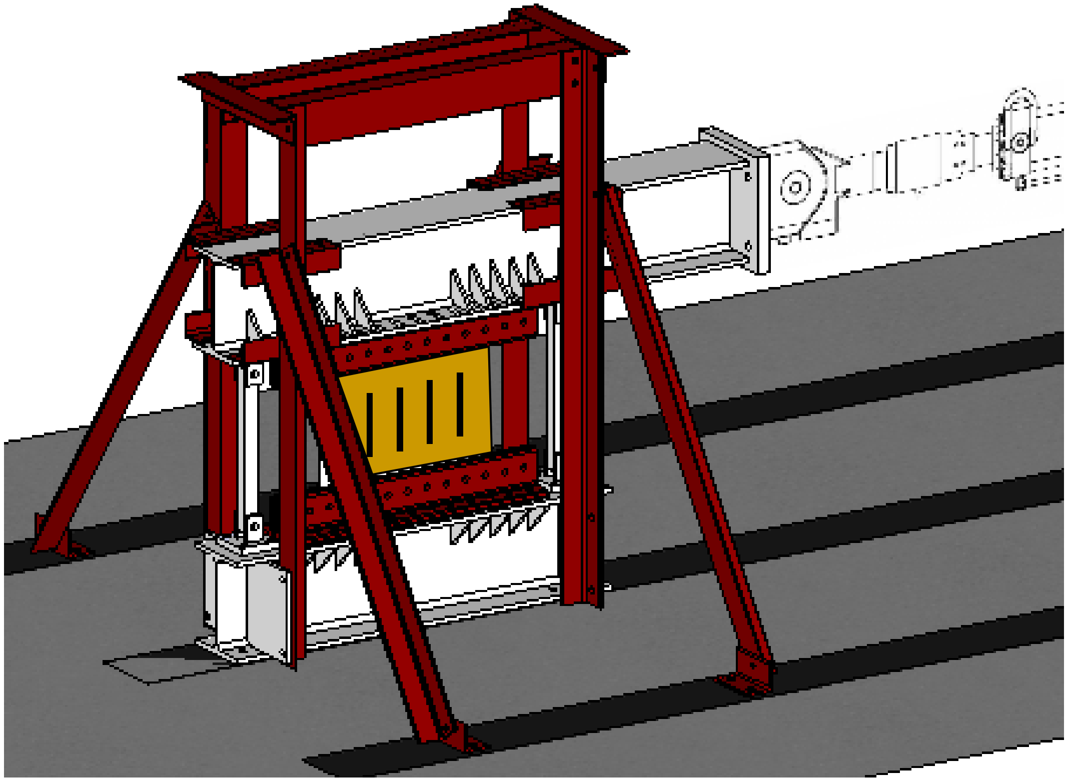
$$SC = \frac{M_{restore}}{M_{resist}} = \frac{A\,F_{PT}}{V_P(A+B)}$$

4. Initial P/T stress
5. Frame Stiffness
6. Fuse type including degradation



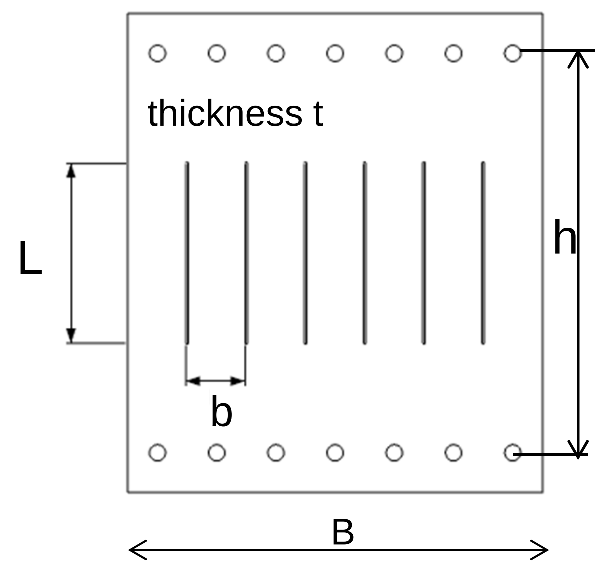
Shear Fuse Testing - Stanford

- Attributes of Fuse
  - high initial stiffness
  - large strain capacity
  - energy dissipation
- Candidate Fuse Designs
  - ductile fiber cementitious composites
  - low-yield steel plates
  - mixed sandwich panels
  - damping devices
  - steel panels with slits

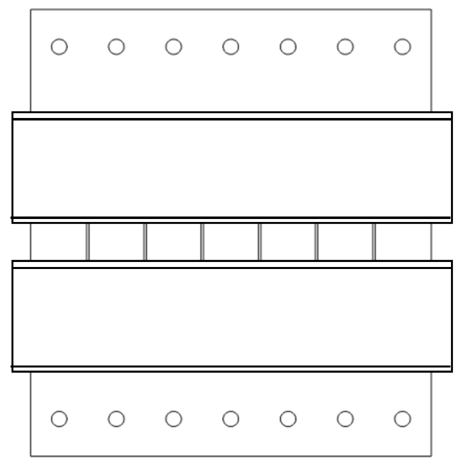


Panel Size: 400 x 900 mm

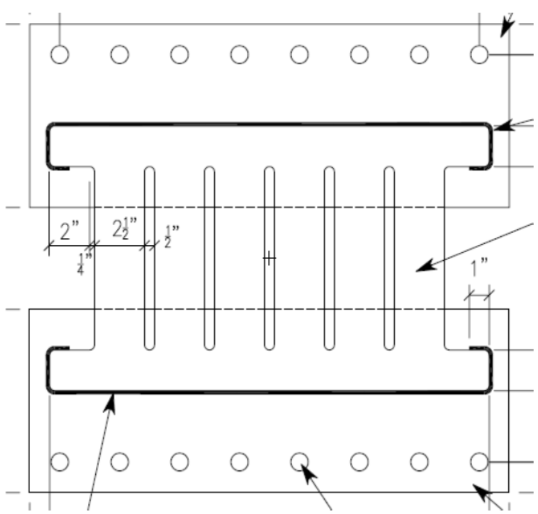
Fuse Configurations



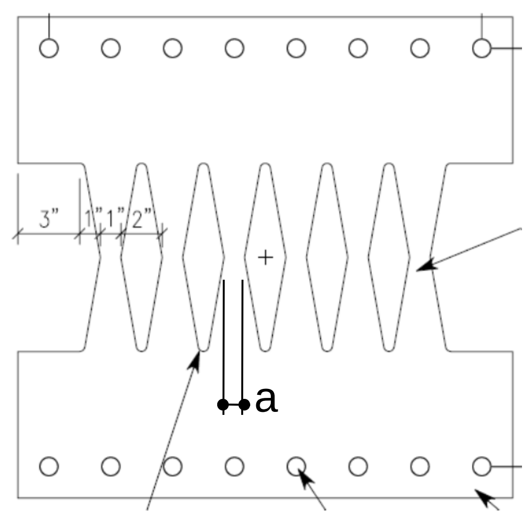
Rectangular link: *b/t* and *L/t*



Buckling-restrained



Welded end



Butterfly link: *b/a*

Notation:

R56-10BR

L/t b/t

“R”: Rectangular

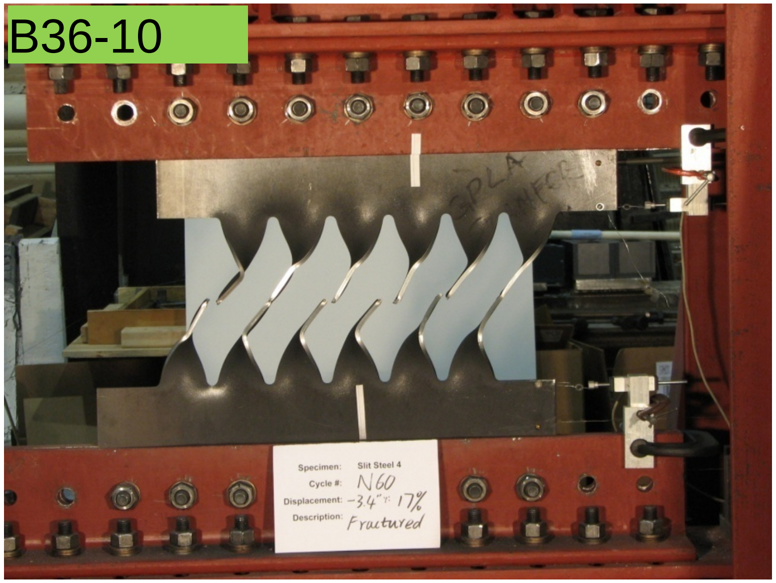
“B”: Butterfly

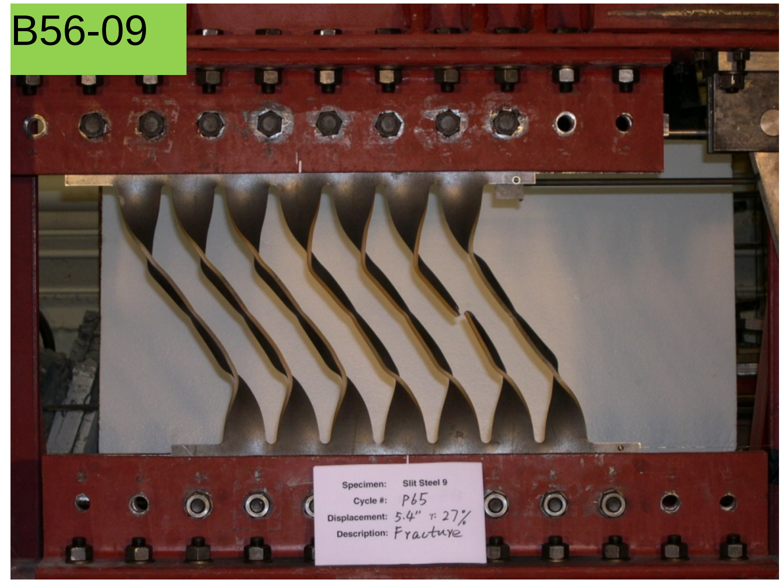
“BR”: Buckling-restrained

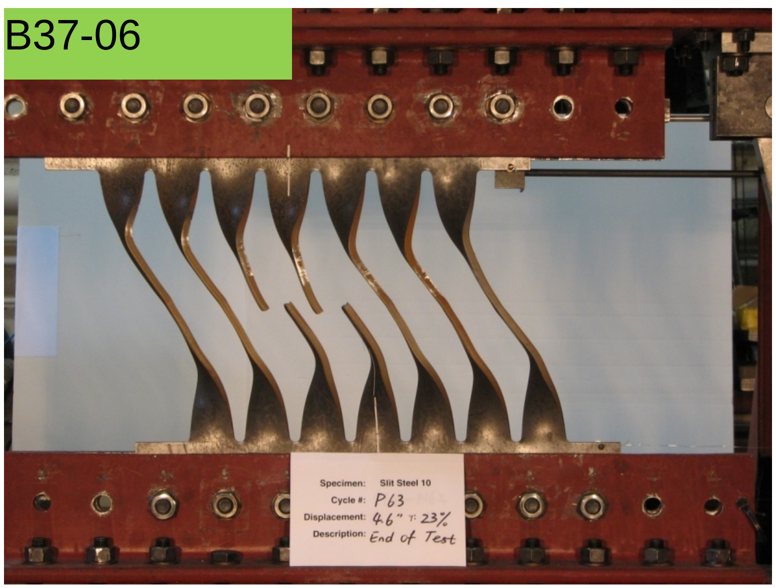
“W”: Welded

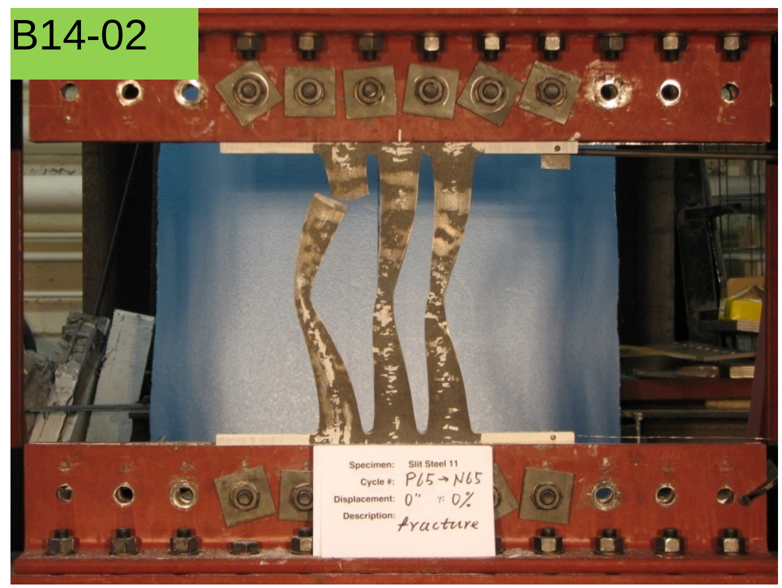
A36 steel plate varying from 1/4” to 1” thick

Testing Results: Butterfly Links

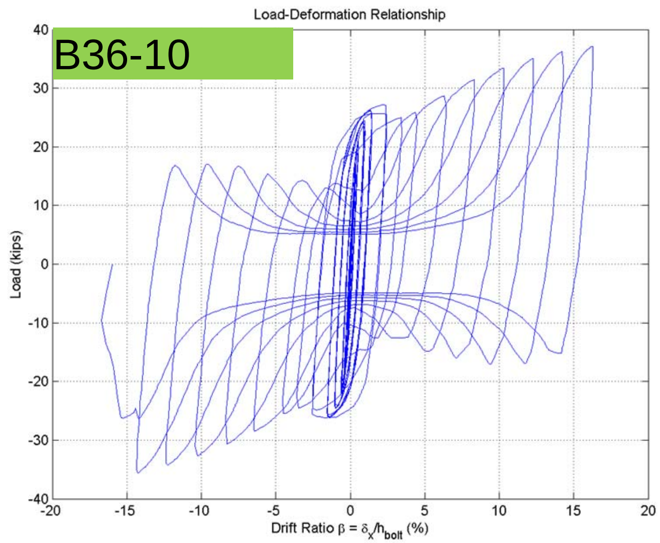


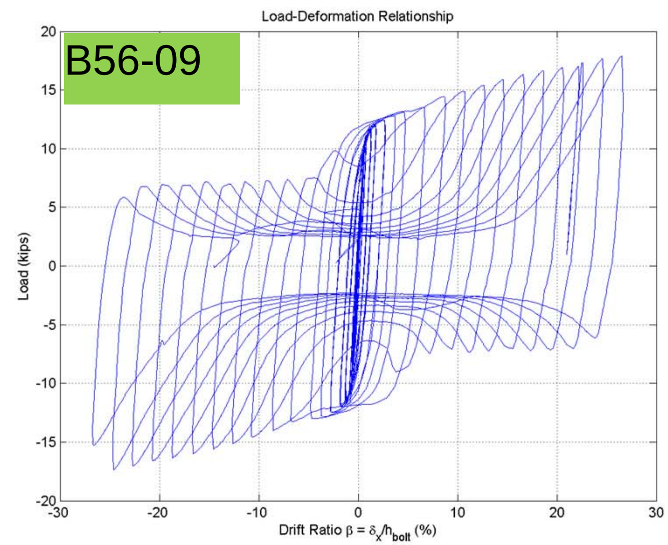


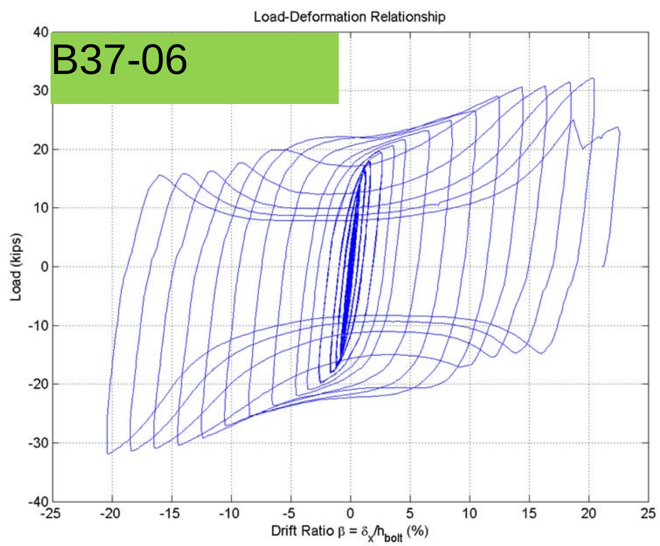


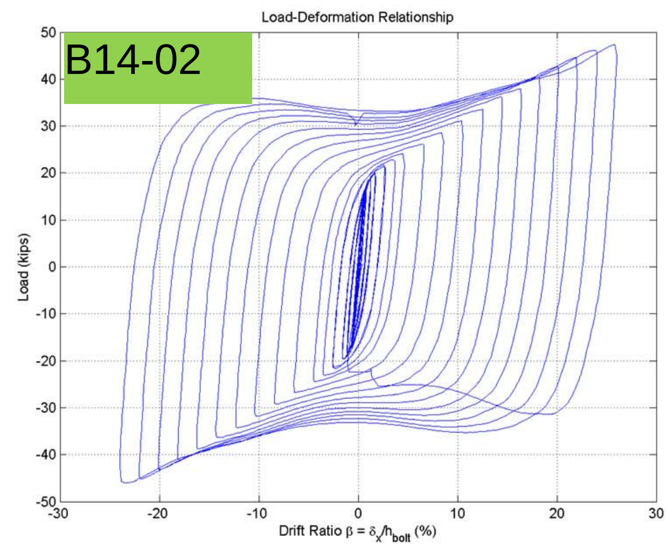


Testing Results: Shear Load-Shear Deformation

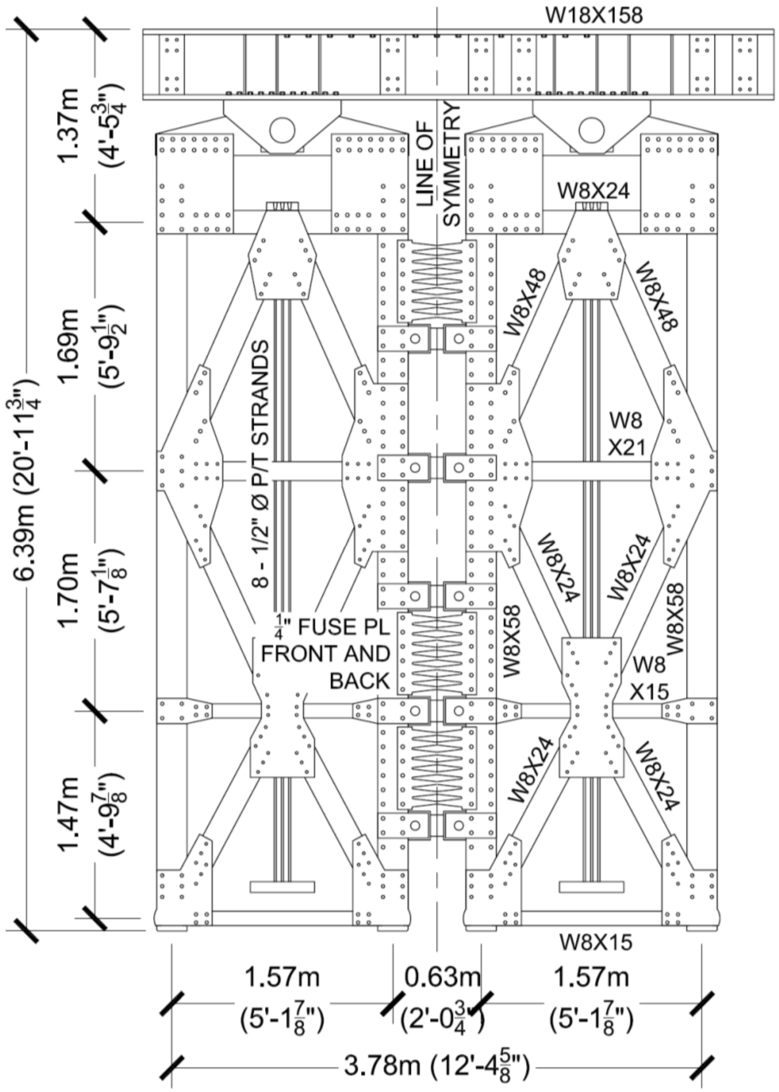


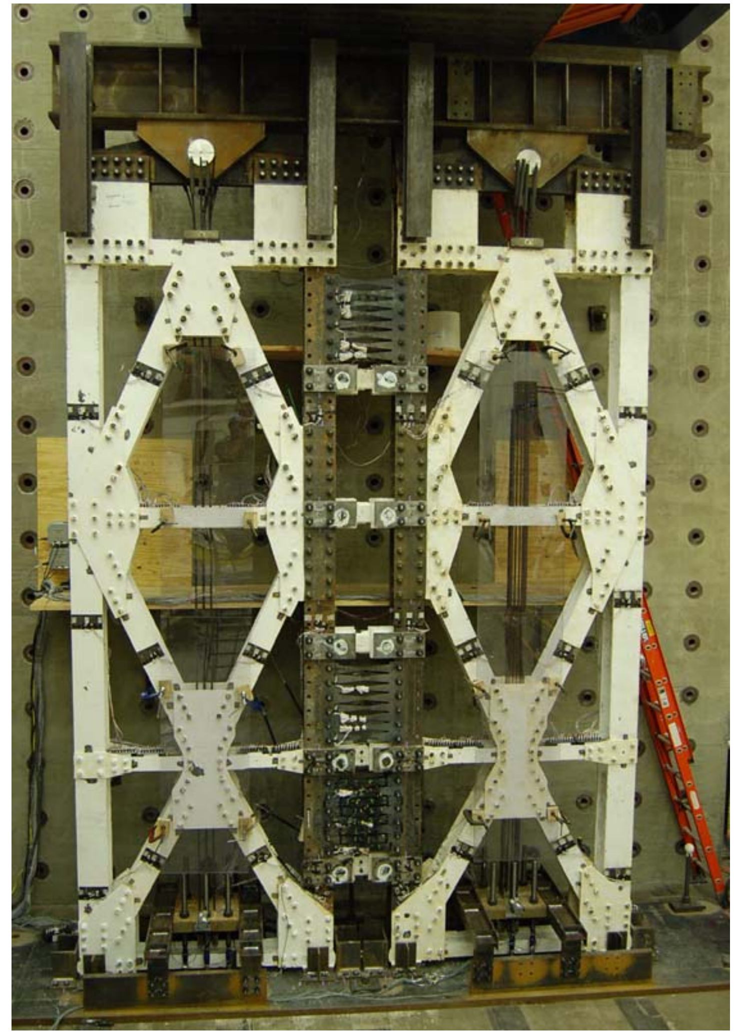






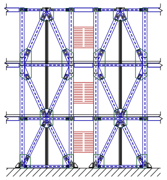
Specimen Design / Test Setup





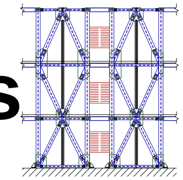
In the Rig

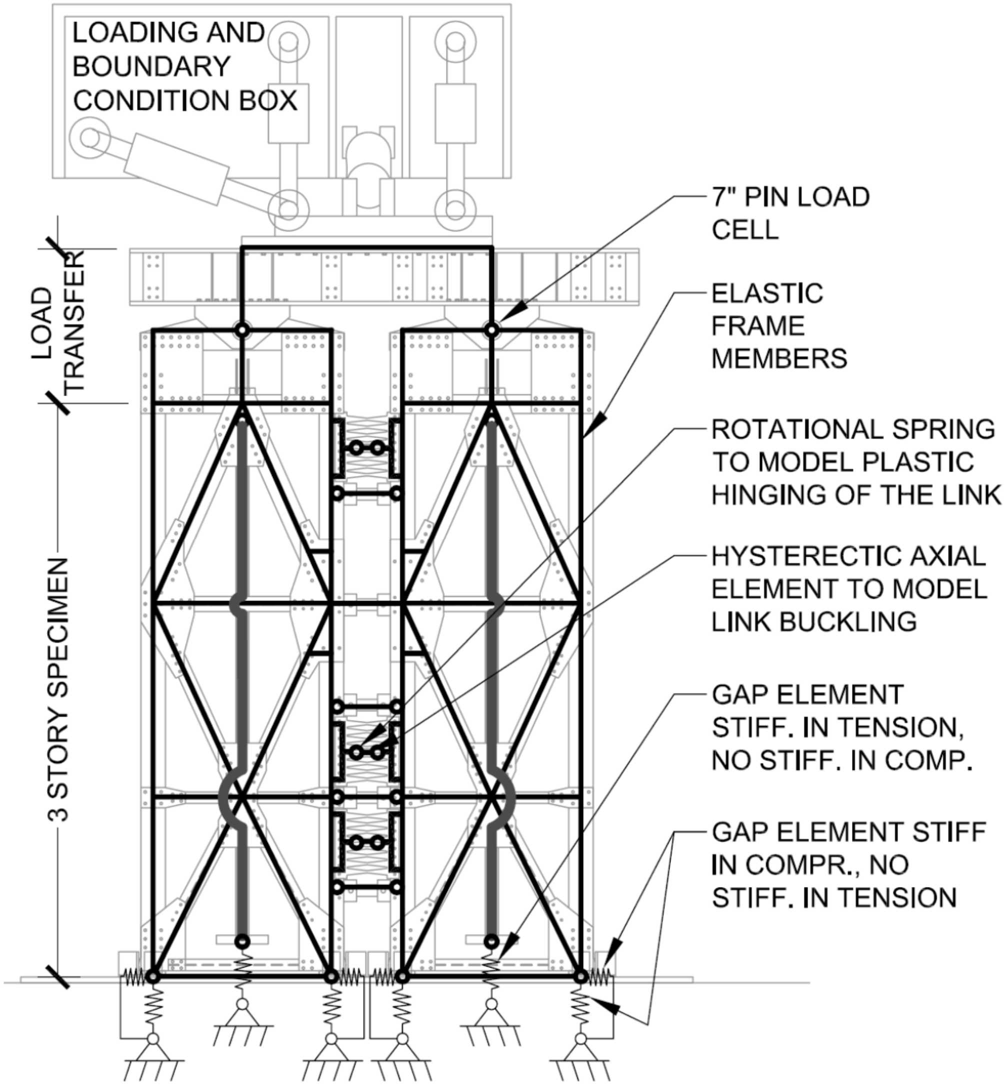
Test Matrix for System Experiments



Test ID	Dim "B"	A/B Ratio	OT Ratio	SC Ratio	Num. of 0.5" P/T Strands	Initial P/T Stress[2] and Force	Fuse Type and Fuse Strength	Fuse Configuration	Testing Protocol	Exceed P/T Yield
A1	2.06'	2.5	1.0 (R=8)	0.8	8	0.287 Fu (94.8 kips)	8 Links (84.7 kips)	Six – 1/4" thick fuses	Quasi-Static	No
A2	2.06'	2.5	1.0 (R=8)	0.8	8	0.287 Fu (94.8 kips)	10 Links (84.7 kips)	Two – 5/8" thick Fuses	Quasi-Static	No
A3	2.06'	2.5	0.85 (R=9.4)	1.1	8	0.287 Fu (94.8 kips)	7 Links (62.0 kips)	Two – 5/8" thick Fuses	Quasi-Static	No
A4	2.06'	2.5	1.4 (R= 5.7)	1.18	8	0.489 Fu (161.5 kips)	7 Links (98.0 kips)	Two – 1" thick Fuses	Quasi-Static	Yes
A5	2.06'	2.5	1.0 (R=8)	1.14	8	0.338 Fu (111.8 kips)	8 Links (70.0 kips)	Two – 5/8" thick Fuses	Pseudo-Dynamic	No
A6	2.06'	2.5	1.06 (R= 7.5)	0.97	8	0.338 Fu (111.8 kips)	8 Links (84.7 kips)	Six – 1/4" thick fuses	Hybrid Sim.	No
A7	2.06'	2.5	1.06 (R= 7.5)	0.97	8	0.338 Fu (111.8 kips)	8 Links (84.7 kips)	Six – 1/4" thick fuses	Quasi-Static	Yes
B1	Left Frame		1.0 for ten frames	1.56	4	0.454 Fu (75.0 kips)	6 Links Total (48.0 kips)	Single Fuse Thickness (3/4" thick) with bar strut across the top	Quasi-Static	Yes
B2	Right Frame'		1.0 for ten frames	1.56	4	0.454 Fu (75.0 kips)	20 Links Total (48.0 kips)	Double Fuse Thickness (3/16" thick) with Plate In Between	Quasi-Static	Yes

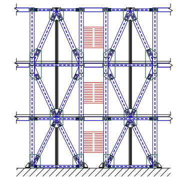
Initial Computational Modeling of Specimens



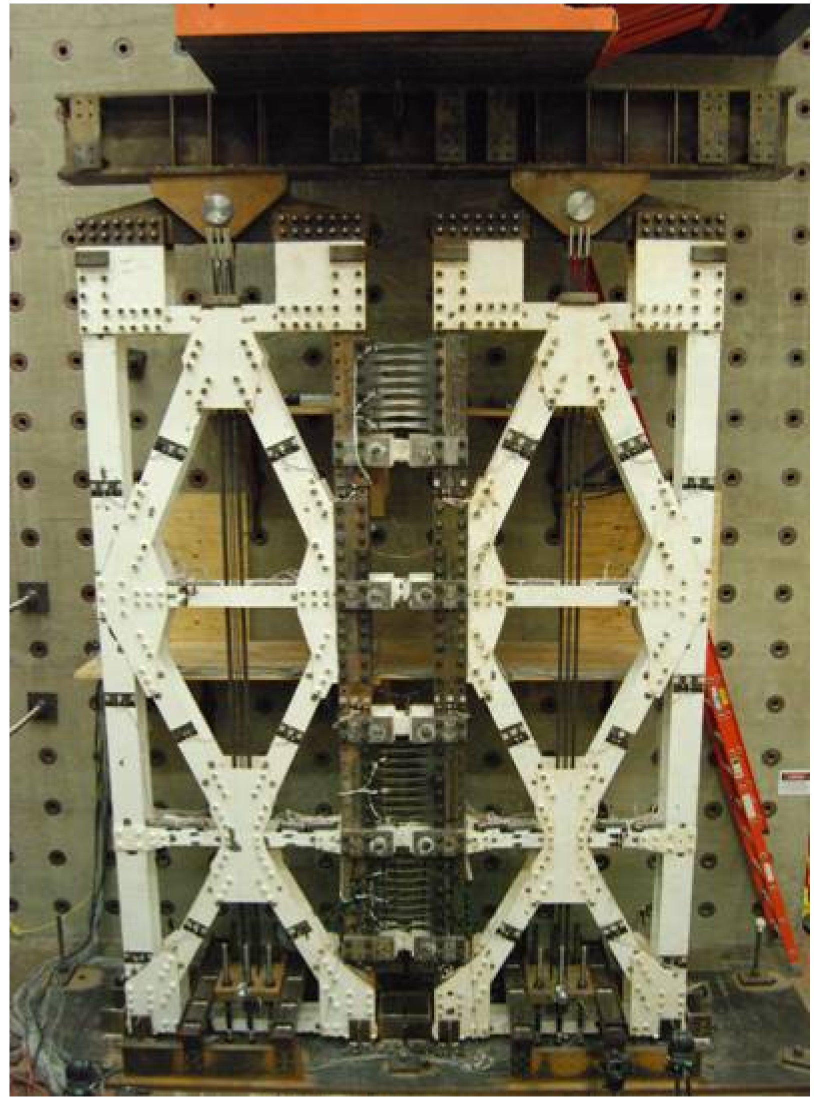


- 2D Frame Analysis in OpenSees
- Specialized Springs Model Gap Elements
- Component Model Simulates Combined Flexural and Tension Response
- Frame Elements are Elastic
- Large Displacement Formulation

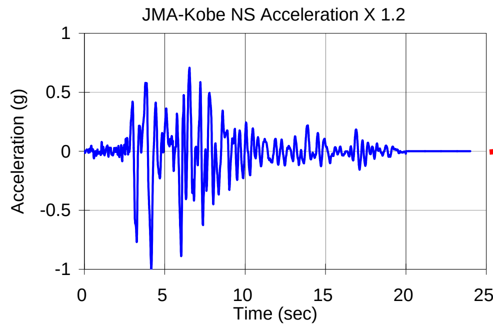
Tested Configuration: A6



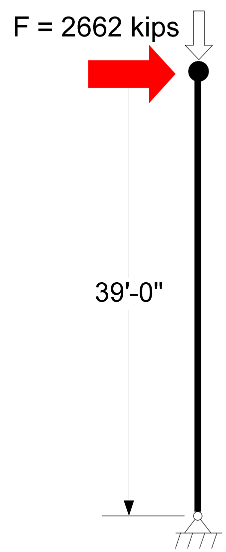
- Six fuses, each with 8 links, that are 1/4” thick
- Frame width / Fuse width (*A/B*) = 5.16’ / 2.06’ = 2.5
- Resistance is designed based code loading with *R*=7.5
- Dual frame configuration
- Initial P/T force is equal to the amount required to fully self-center
- Initial post-tensioning stress is 34% of ultimate stress
- Test was conducted as a ***Hybrid Simulation***

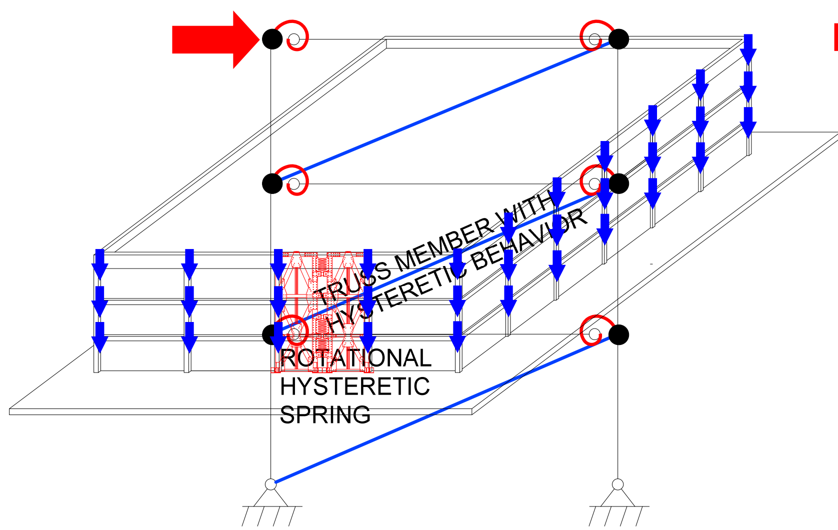


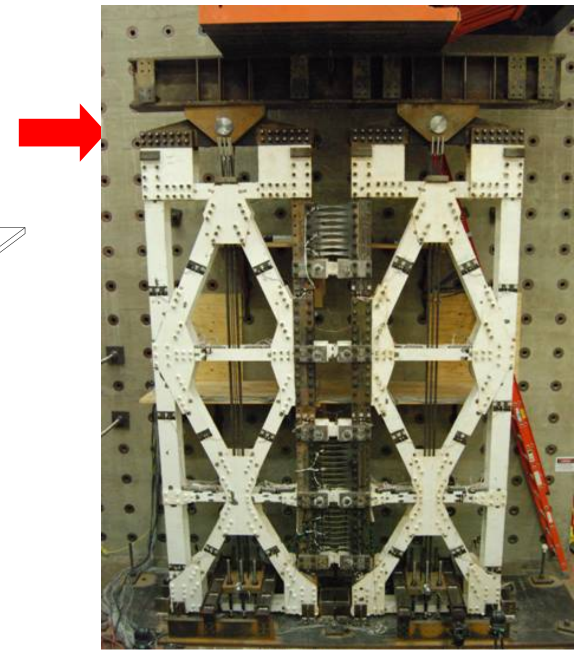
Hybrid Simulation Test Setup



UI-SIMCOR Links These DOF's and Applies the Ground Motion







LEANING COLUMN MODEL AT FULL SCALE

RESISTANCE OF PARTITIONS AND SIMPLE BEAM-TO-COLUMN CONNECTIONS AT FULL SCALE

CONTROLLED ROCKING TEST AT SPECIMEN SCALE

NEESR-SG: Controlled Rocking of Steel-Framed Buildings with Replaceable Energy Dissipating Fuses

LARGE-SCALE TESTING AT UIUC – TEST A6

Gregory G. Deierlein, Sarah Billington, Helmut Krawinkler, Xiang Ma, Alejandro Pena, Eric Borchers, *Stanford University*

Jerome F. Hajjar, Matthew Eatherton, Noel Vivar, Kerry Hall, *University of Illinois*

Toru Takeuchi, Kazuhiko Kasai, and Shoichi Kishiki, *Tokyo Institute of Technology*

Mitsumasa Midorikawa and Tetsuhiro Asari, *Hokkaido University*

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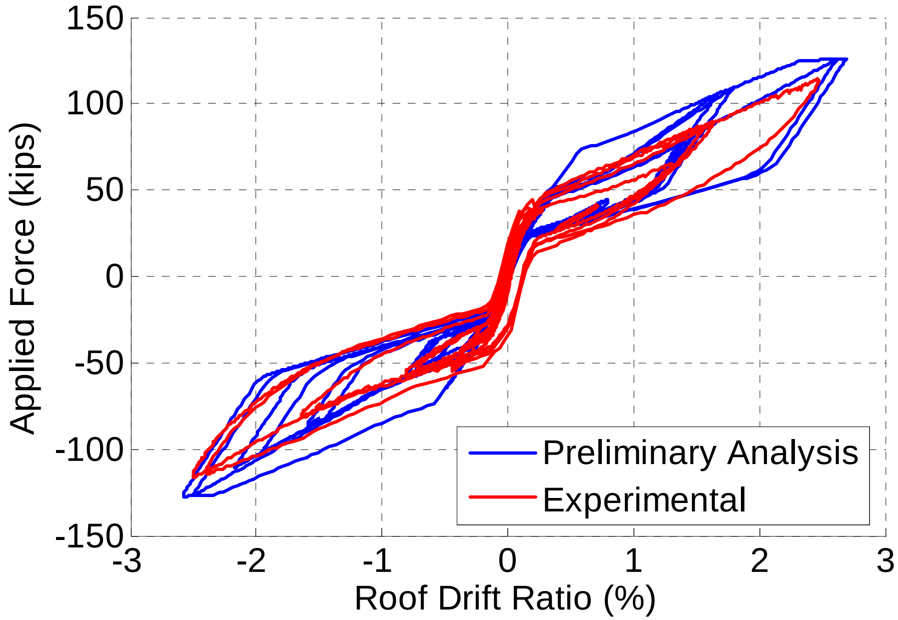
JISF
The Japan Iron and Steel Federation

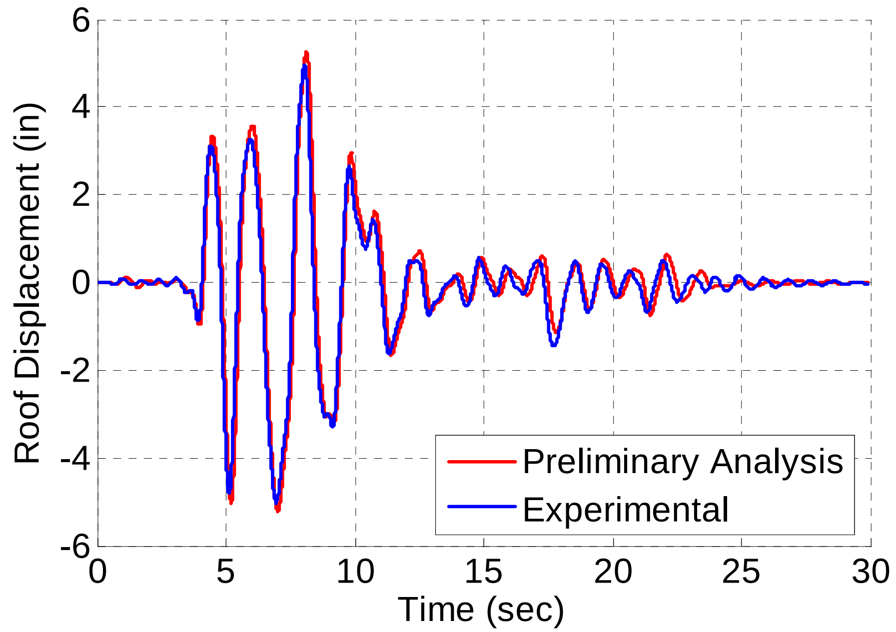
NEES

George E. Brown, Jr. Network for Earthquake Engineering Simulation

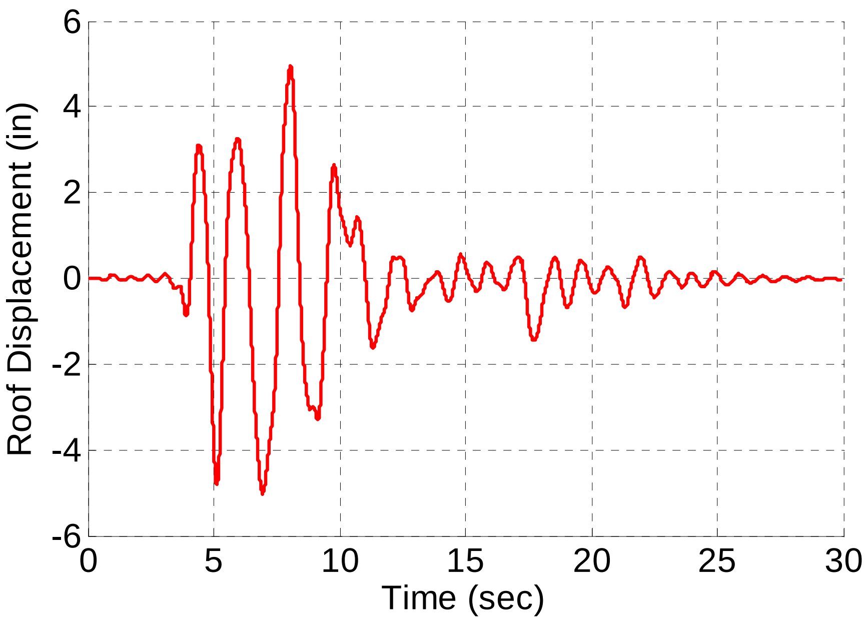
NSF

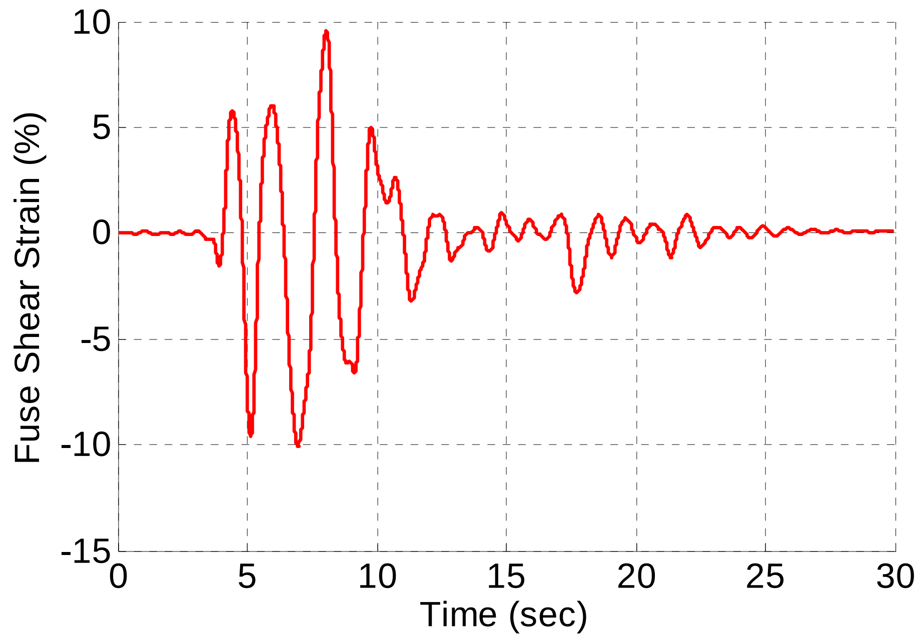
A6 Test Results – Load-Deformation and Drift



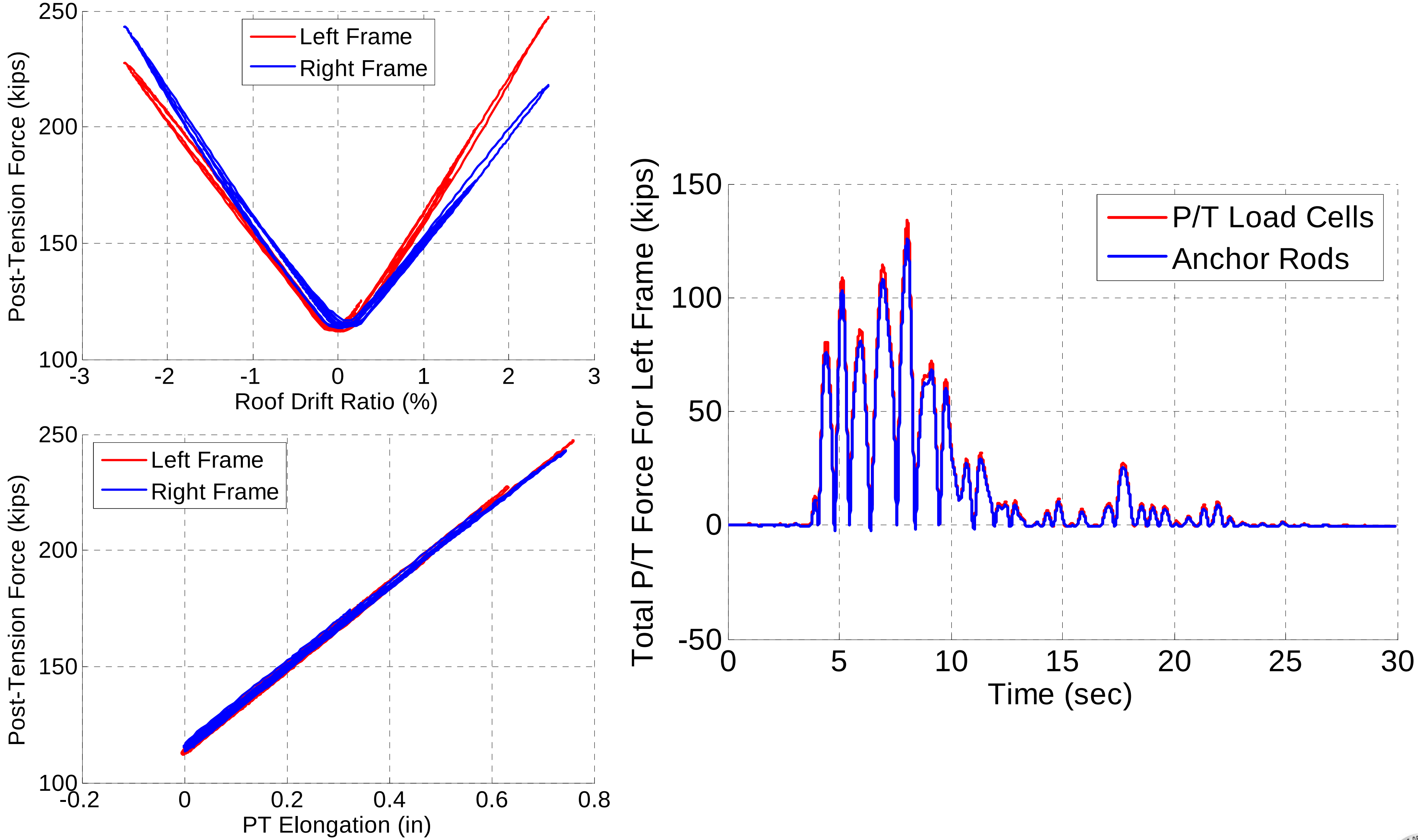


A6 Test Results – Drift and Fuse Shear Strain





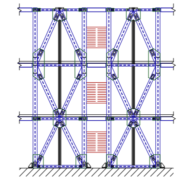
A6 Test Results – PT Force

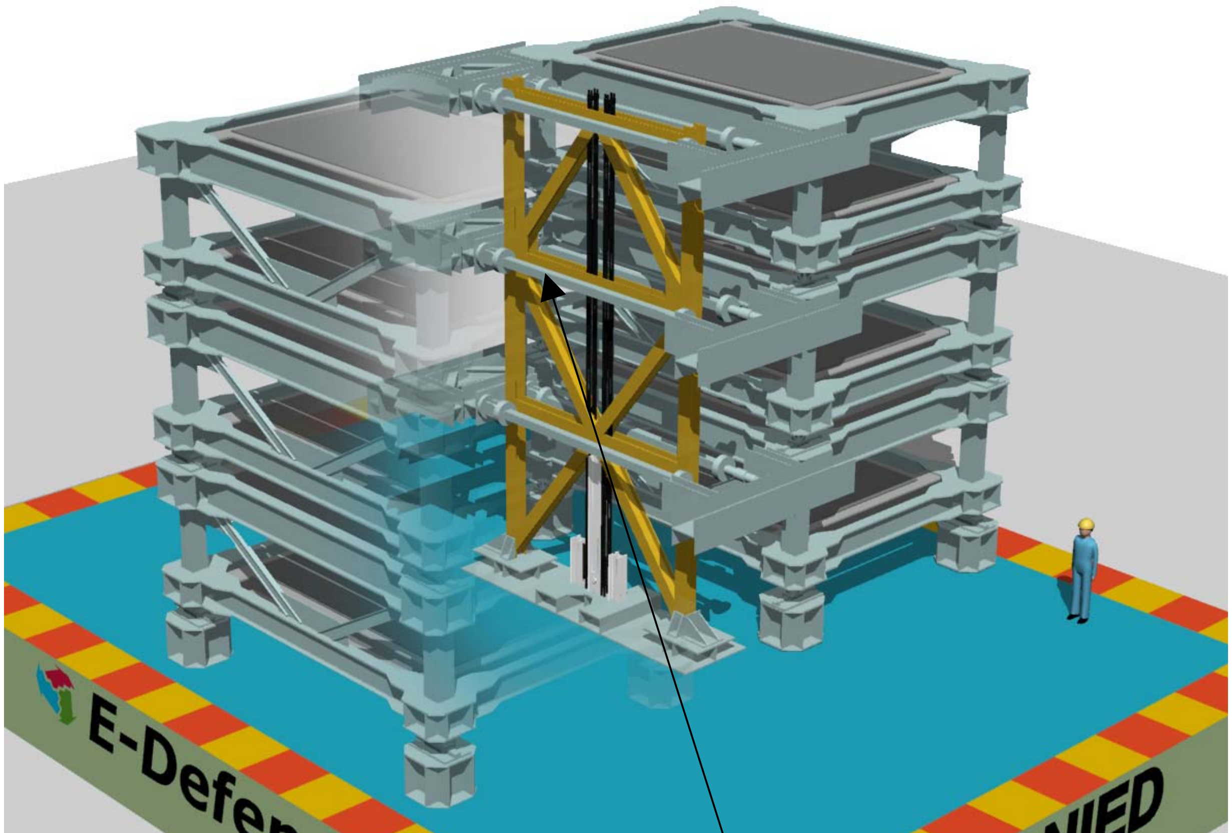


Test-bed Unit

Test Frame

E-Defense Test Setup



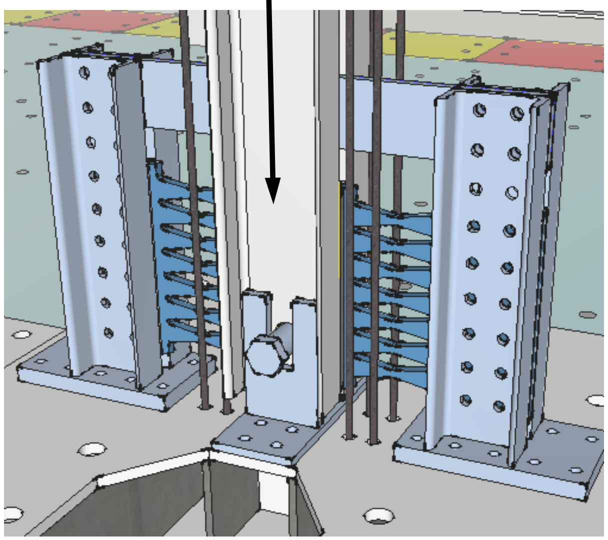


Load Cell Force
Input to Frame

- **Large-Scale Validation**
  - fuse/rocking frame interaction
  - rocking base details
  - post tensioning
  - replaceable fuses
- **Proof-of-Concept**
  - design concept & criteria
  - constructability
- **Performance Assessment**
  - nonlinear computer simulation

Controlled-Rocking System Shake-Table Test

Pin Moves Center of Fuse Up and Down



Fuse is Steel Plate with Specially Designed Cutouts

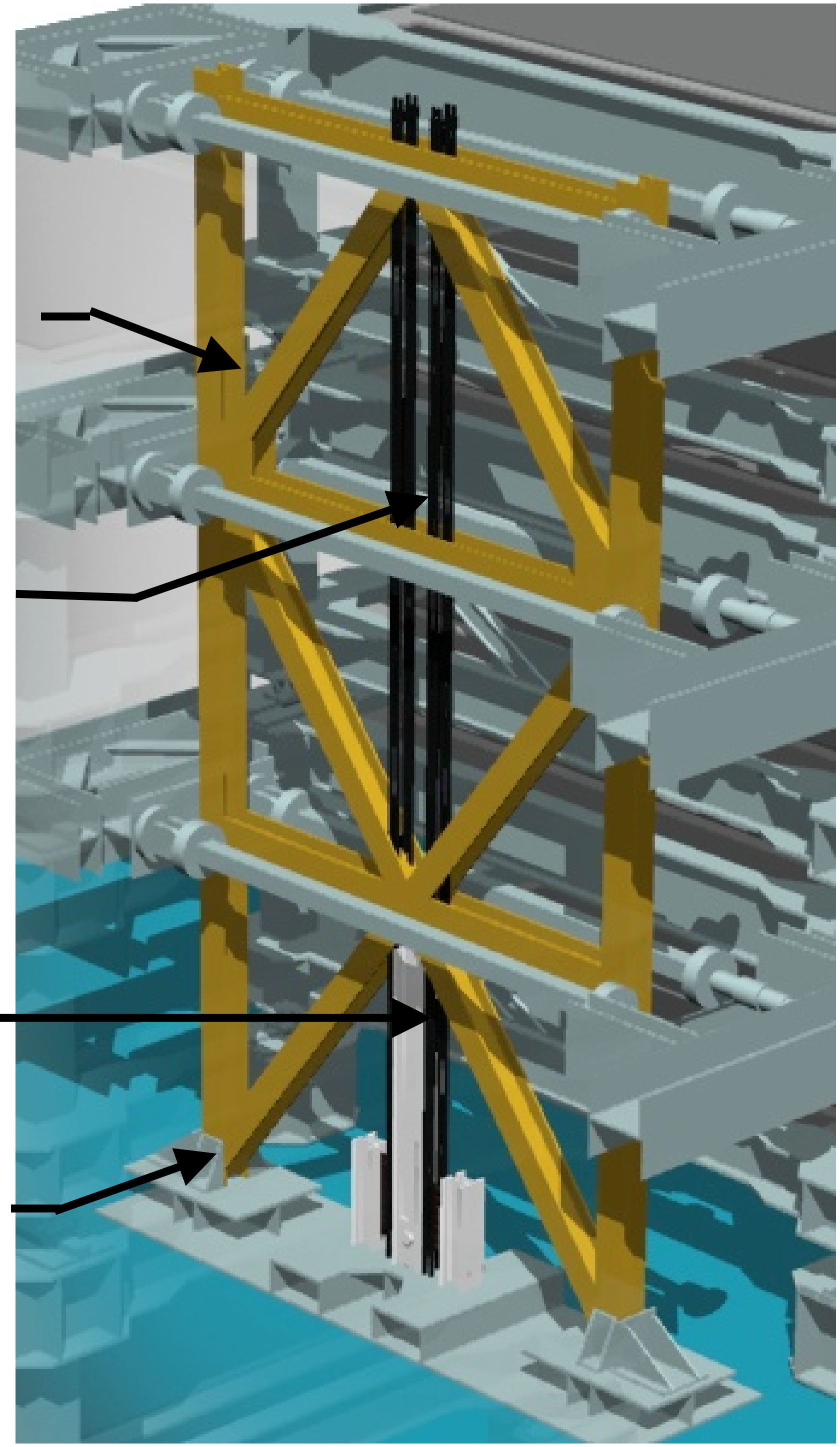
Steel Frame Remains Essentially Elastic, but is Allowed to Rock at the Base

Post-Tensioning Strands

– provide self-centering --

Center Column Connects Frame to Fuse

Base of Frame is Free to Uplift

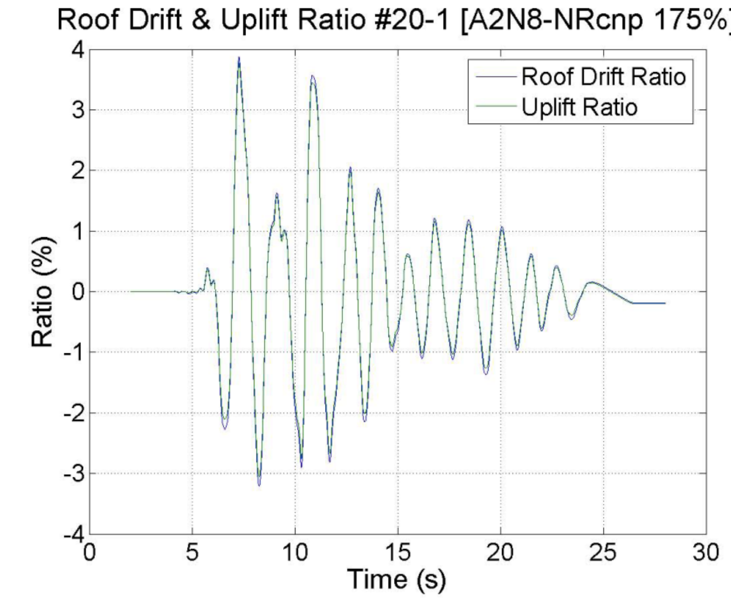


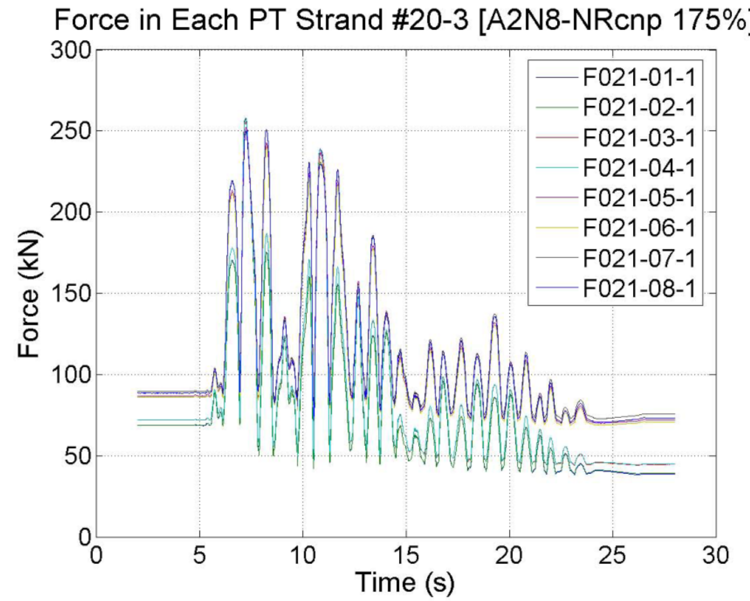
Test Matrix

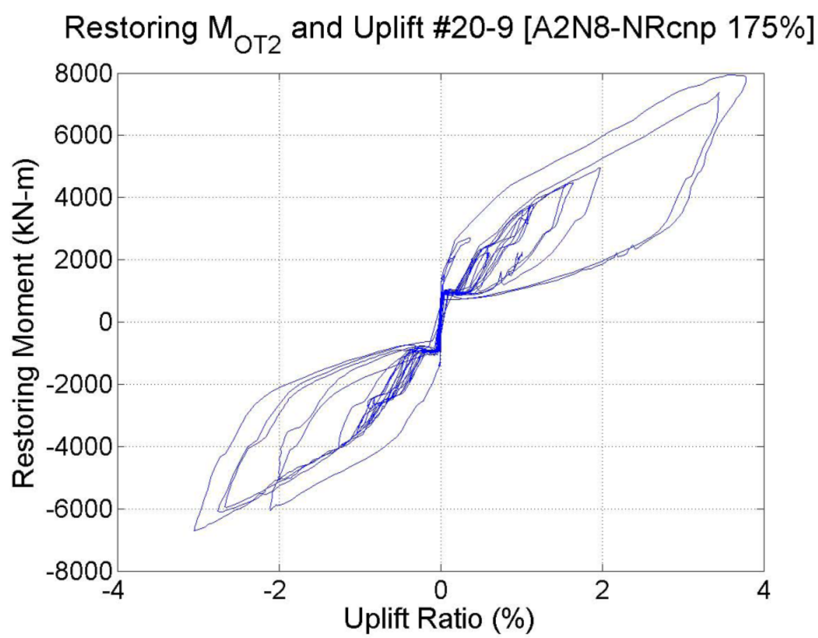
Test ID	Fuse	Ground Motions	Motion Intensity
A1	Butterfly Fuse Non-Degrading	JMA Kobe NS	30% ~ 65% (MCE)
A2	Butterfly Fuse Non-Degrading	Northridge Canoga Park	25% ~ 140% (MCE), 180%
B	Butterfly Fuse Degrading	JMA Kobe NS	10% ~ 60%
C	BRB	JMA Kobe NS	10% ~ 65% (MCE)

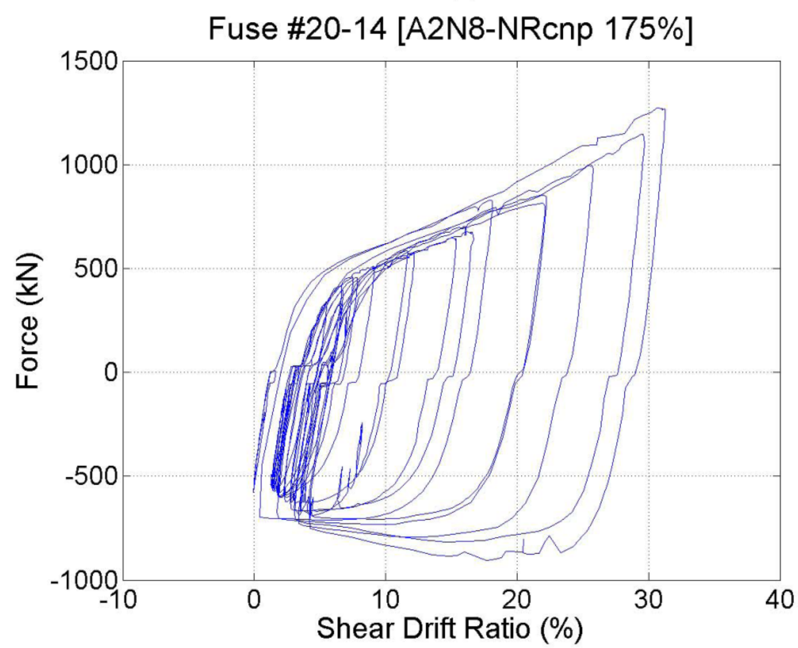
Final Test - 1.8x Northridge (1.3 MCE)

Final Test - 1.8x Northridge (1.3 MCE)

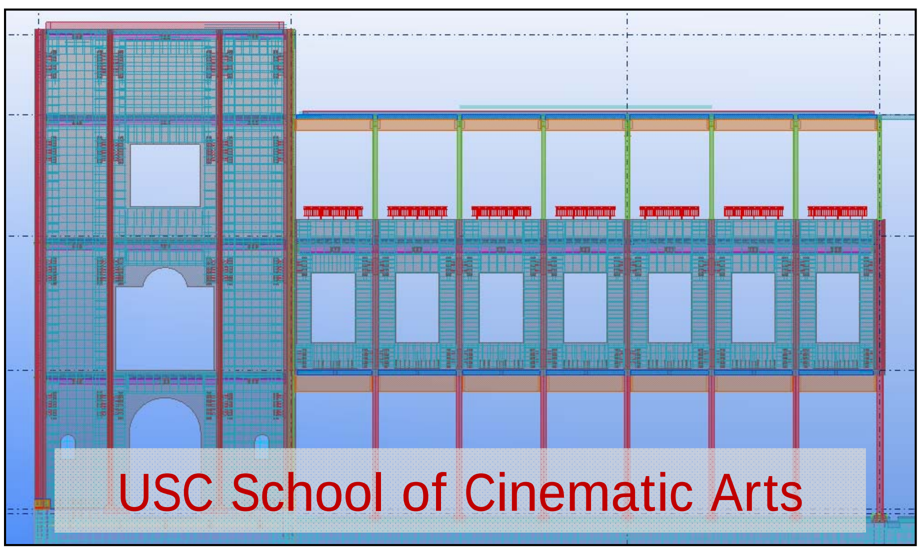








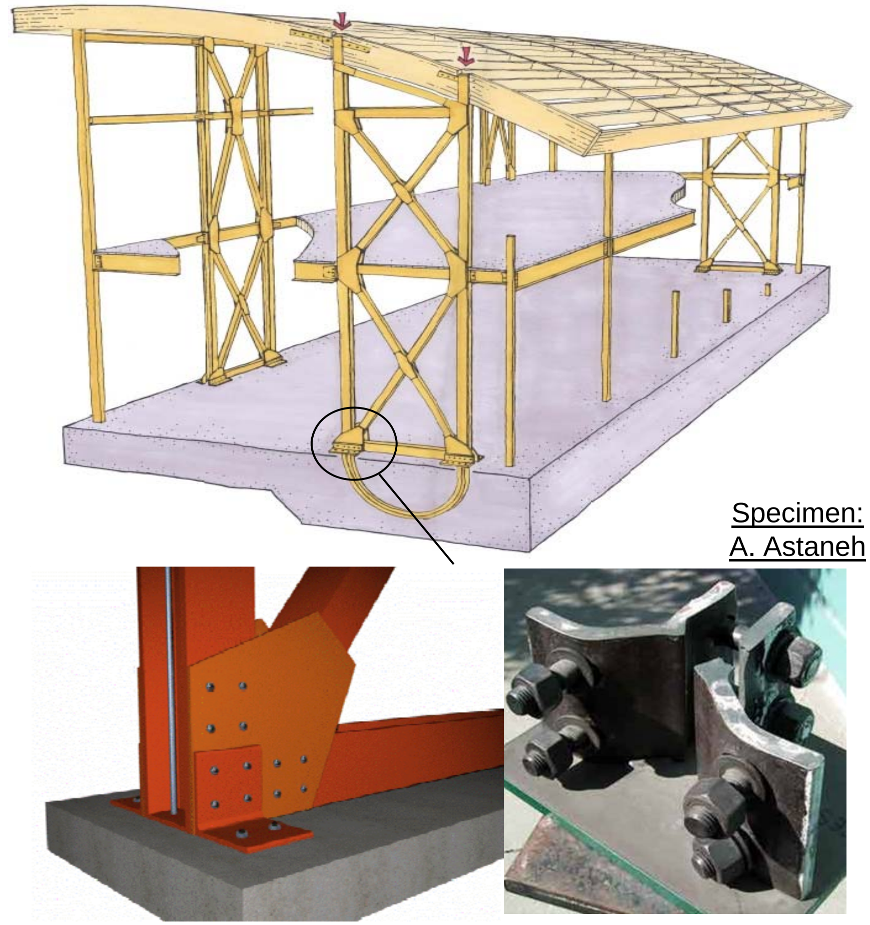
Collaboration with Industry Partners
“Early Adopters” of System Innovations



GPLA, Inc.
Engineers and Builders

Collaboration with Industry Partners
“Early Adopters” of System Innovations

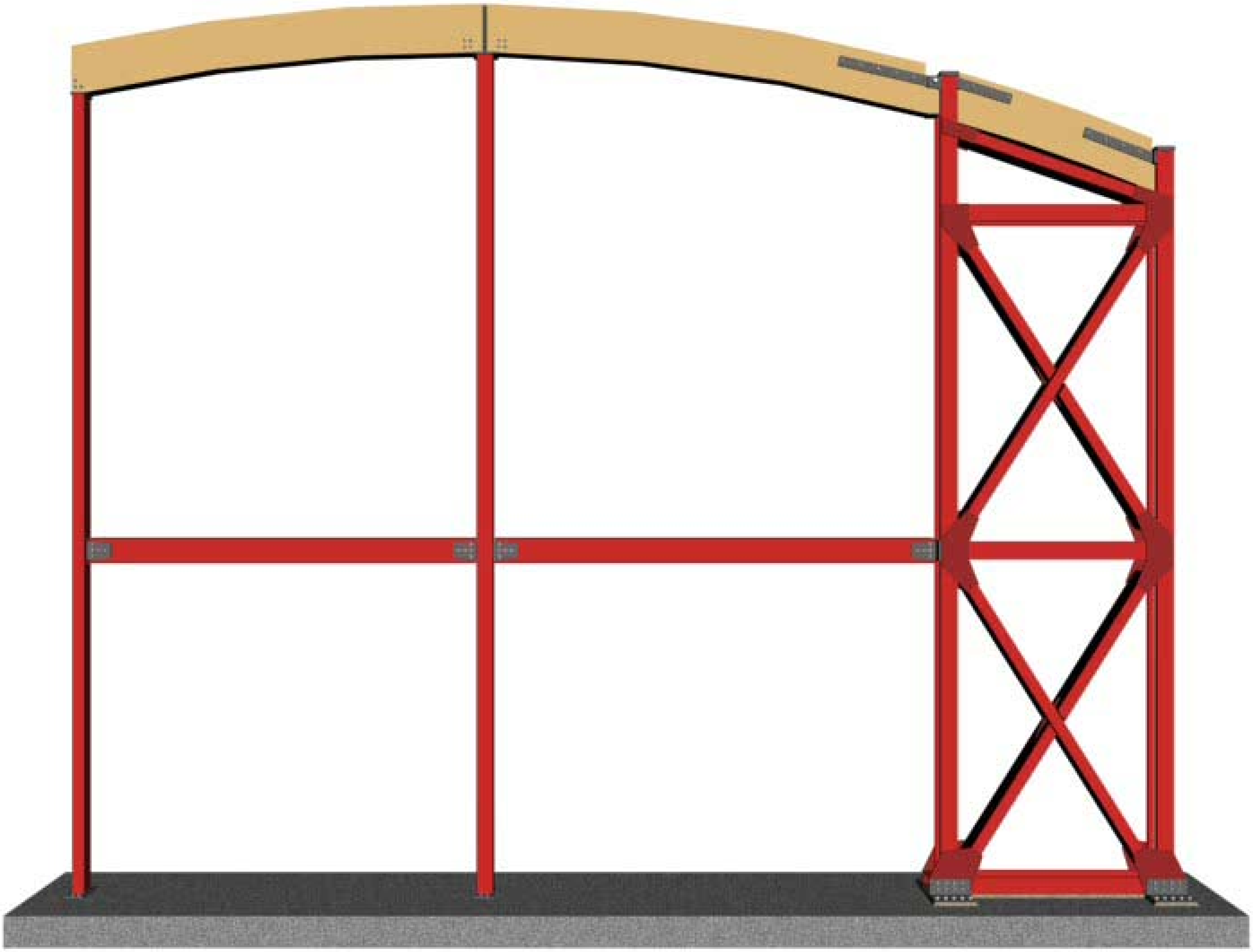
<u>LEED Innovation credits</u>
- **<u>Low-cement concrete</u>**
- **<u>Damage-resistant framing</u>**



Orinda City Offices

Architect: **Siegel and Strain Architects**

TIPPING · MAR + associates
structural engineers



Conclusions: Where can we go from here?

- **Thematic Concept**
  - life cycle design for earthquake effects
  - damage control and design for repair
- **Engineering Design Features**
  - controlled rocking and self-centering
  - energy dissipating replaceable fuses
- **Performance-Based Engineering Framework**
  - quantification of decision variables (losses, downtime)
  - integration of hazard, response, damage, loss
- **Development & Validation**
  - large scale testing and computational simulation
  - design guideline development

Thank You