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Design of Coupled Composite Plate Shear Walls - Concrete Filled (Coupled C-PSW/CF)

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Design Requirements / Criteria

1.1 Scope

- Coupled C-PSW/CF shall consist of composite plate shear walls / concrete filled and filled composite coupling beams
- Composite walls shall be planar, C-shaped, or I-shaped
- Flange plate at the open ends of the wall elements and no additional boundary elements
- Coupling beam consisting of concrete-filled built-up box sections or rectangular HSS with:
  - Section aspect (height-to-width) ratios less than or equal to 2 or greater than or equal to 0.5
  - Beam length-to-depth ratios greater than or equal to 3, and less than or equal to 5
- Coupling beams shall be connecting the composite shear walls at least 90% of the stories.
- Doubly symmetric in plan at each story level. Plate thickness is permitted to be reduced at higher stories

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Design Requirements / Criteria

1.2 Basis of design

- Coupled C-PSW/CF as the main lateral force resisting system
- Expected to provide significant inelastic deformation capacity through flexural plastic hinging at the ends of the composite coupling beams and flexural yielding at the base of the composite wall elements.
- Preferred mechanism consists of flexural hinging in the coupling beams over a majority of the height of the structure followed by flexural hinging at the base of the individual composite walls.
- Weak coupling beam-strong wall design

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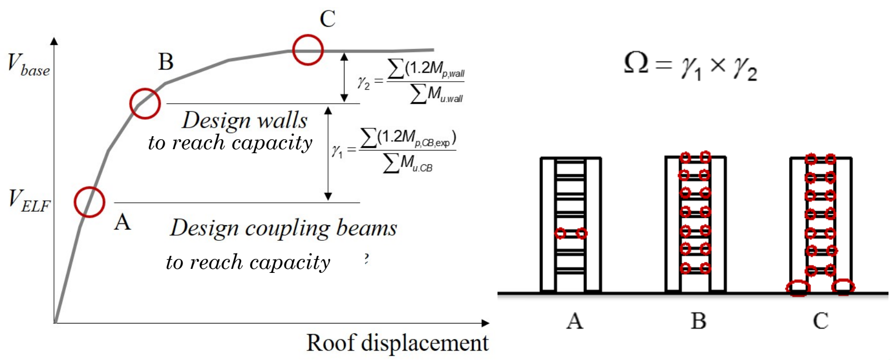
Design Requirements / Criteria

1.2 Basis of design



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Design Requirements / Criteria

- 1.3 Analysis Requirements
  - Effective flexural stiffness of planar composite walls and the coupling beams shall be calculated per Specification I2-12 with C_3 taken equal to 0.40

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \qquad \text{(Spec. I2-12)}$$

  - Effective flexural stiffness of C-shaped and I-shaped walls shall be calculated using cracked transformed section properties corresponding to 60% of the nominal flexural capacity calculated while accounting for the effects of axial force.
  - Shear stiffness of the composite wall and coupling beams shall be calculated using the shear stiffness of the composite cross section.

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Design Requirements / Criteria

- 1.3 Analysis Requirements
  - An analysis in conformance with the applicable building code shall be performed to calculate the required strengths for the coupling beams.
  - The required strength of the composite walls and the coupling beam-to-composite wall connections shall be determined using the capacity-limited seismic load effect.
  - The capacity-limited horizonal seismic load effect, Ecl, shall be determined from an analysis in which all the coupling beams are assumed to develop plastic hinges at both ends with the expected flexural capacity of $1.2M_{p,exp}$.
  - The required axial and flexural strengths of the composite walls shall be determined directly from this analysis.

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DESIGN REQUIREMENTS / CRITERIA

1.3 Analysis Requirements

- The required shear strength of the composite walls shall be determined as the shear force obtained from this analysis amplified by a factor of four.
- The required strengths of the composite wall-to-foundation connections shall be determined using a capacity-limited seismic load effect, where the composite walls are assumed to develop plastic hinges at the base with the expected flexural capacity of $1.2M_{p,exp}$, while accounting for the effects of simultaneous axial force.
- The required shear strength of the composite wall-to-foundation connections shall be equal to the required shear strength of the composite walls.

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DESIGN REQUIREMENTS / CRITERIA

1.4 System requirements

- Slenderness requirements for composite walls

$$\frac{b}{t_p} \leq 1.0\sqrt{\frac{E_s}{F_y}} \qquad \text{Eq. 1.4-1}$$

where,

b = largest unsupported length of the faceplate between rows of steel anchors or ties, in. (mm)

t_p = thickness of faceplate, in. (mm)

$$\frac{S}{t_p} \leq 1.0\sqrt{\frac{E_s}{2\alpha + 1}} \qquad \text{Eq. 1.4-2}$$

$$\alpha = 1.7\left[\frac{t_{sc}}{t_p} - 2\right]\left[\frac{t_p}{d_{tie}}\right]^4 \qquad \text{Eq. 1.4-3}$$

where,

S = largest clear spacing of the ties

t_{sc} = thickness of the composite wall

d_{tie} = effective diameter of the tie

Design Requirements / Criteria

1.4 System requirements

- Slenderness requirements for coupling beams

$$\frac{b_c}{tf} \leq 2.26\sqrt{\frac{E_s}{F_y}}$$ Eq. 1.4-4

$$\frac{h_c}{t_w} \leq 2.54\sqrt{\frac{E_s}{F_y}}$$ Eq. 1.4-5

where,

b_c = clear unsupported length of the coupling beam flange plate

h_c = clear unsupported length of the coupling beam web plate

t_f = thickness of the coupling beam flange plate

t_w = thickness of the coupling beam web plates

- Connection between tie bars and steel faceplates shall be able to develop the full yield strength of the tie bar

Design Requirements / Criteria

1.4 System requirements

- The coupling beams shall be proportioned to be flexure critical with shear strength as follows:

$$V_n \geq \frac{2.6\, M_{p,exp}}{L_{cb}}$$ Eq. 1.4-6

where,

V_n = shear strength of coupling beam calculated using Equation 1.4-10

$M_{p,exp}$ = flexural capacity of the coupling beam calculated according to Section 1.5.2.2, while using the expected yield strength, R_yF_y, for the steel in tension or compression and the expected strength $R_cf'_c$ for the concrete in compression

L_{cb} = length of the coupling beam

- Strong wall – weak coupling beam. The system shall be proportioned such that the coupling beam develop plastic hinges at both ends before the composite walls reach their flexural capacity.

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Design Requirements / Criteria

1.5 Members

- Composite plate shear walls (C-PSW/CF)
  - Limitations
    - The cross-sectional area of the steel section shall comprise at least 1% of the total composite cross-section
    - Composite walls shall satisfy the slenderness requirements
  - Compressive strength

$$P_{no} = F_y A_s + 0.85 f_c' A_c$$

  - Tensile strength

$$P_n = A_s F_y$$

$$\phi_t = 0.90 \text{ (LRFD)}$$

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Design Requirements / Criteria

1.5 Members - CPSW

- The available flexural strength of filled composite plate shear walls shall be determined as the moment, M_p, corresponding to plastic stress distribution over the composite cross section. ϕ_b = 0.90 (LRFD)
- Combined flexure and axial force. The interaction between axial force and flexure shall be based on the plastic stress distribution method of the Specification Section I1.2a or the effective stress-strain method of Section I1.2d.
- Shear strength

$$V_n = F_y A_{sw}$$

$$\phi_v = 0.90 \text{ (LRFD)}$$

where,

A_{sw} = area of steel plates parallel to the in-plane shear force being considered

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Design Requirements / Criteria

1.5 Members – Coupling beams

Limitations

- The cross-sectional area of the steel section shall comprise at least 1% of the total composite cross-section
- Composite beams shall satisfy the slenderness requirements
- Flexural strength: Moment, M_p, corresponding to plastic stress distribution over the composite cross section. $\phi_b = 0.90$ (LRFD)
- Shear strength

The shear strength, $\phi_v V_n$, shall be determined as:

$$V_n = 0.60\ F_y A_w + 0.06\,(\sqrt{f'_c} A_c)$$

$$\phi_v = 0.90 \text{ (LRFD)}$$

where,

A_w = area of coupling beam steel webs

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Design Requirements / Criteria

1.6 CONNECTIONS: BEAM-TO-WALL CONNECTIONS

The required flexural strength, M_u, for the coupling beam-to-wall connection shall be 120% of the expected flexural capacity of the coupling beam ($1.2M_{p,exp}$).

The required shear strength, V_u, for the coupling beam-to-wall connection shall be determined using capacity-limited seismic load effect, which shall be taken as:

$$V_u = 2\,(1.2\ M_{p,exp}) / L_{cb} \qquad \text{Eq. 1.6-1}$$

where, $M_{p,exp}$ is the expected flexural capacity of the coupling beam calculated using expected steel ($R_y F_y$) and concrete ($R_c f'_c$) material properties and plastic stress distribution method given in the Specification, Chapter I

L_{cb} is the clear length of the coupling beam

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Design of Coupled-CPSW/CF

- Design procedure can be generalized in 6 steps:
  1) Obtain predefined floor and core wall dimensions from architect (in this case industry recommendations);
  2) Perform Equivalent Lateral Force analysis using ASCE 7 defined loads;
  3) Perform structural analysis
  4) Choose preliminary dimensions for walls and coupling beams;
  5) Perform design checks including for strength, drift, slenderness, and tie reinforcement;
  6) Redesign as necessary.

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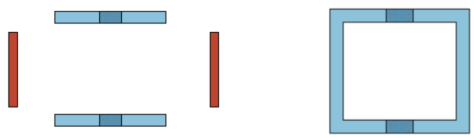
Table 1. Archetype performance groups summary table.

Performance Group Summary				
Group No.	Grouping Criterial			Number of Archetypes
	Basic Configuration	Design Load Level		
		Gravity	Seismic	
PG-1	Type I	Typical	SDC D_{max}	6 (8 & 12 Story)
PG-2			SDC D_{min}	2 (8 & 12 Story)
PG-3	Type II	Typical	SDC D_{max}	6 (18 & 22 story)
PG-4			SDC D_{min}	2 (18 & 22 story)

Table 1. Spectral Acceleration for seismic design category D.

Seismic Design Category (SDC)	Spectral Acceleration (g)
D_{max}	S_{DS} = 1.0
	S_{D1} = 0.6
D_{min}	S_{DS} = 0.5
	S_{D1} = 0.2



Type I
Coupled and uncoupled Walls

Type II
C-shaped coupled walls

Figure 1. Basic configuration Type I and II

Table 1. Archetype Structure Initial Parameters.

Parameter	Value	Reasoning
Coupling beam aspect ratio (*L*/*d*)	3, 4, or 5	ACI318-14 allows ratios of 2.0 to 5.0. In practice, most work uses an L/d of 3 as the lower bound ratio
Story Height	First story: 17ft Typical story: 14ft	Review panel recommendation for typical story heights
Seismic Weight	Floor load of 120psf	Estimated from components: Steel framing (12 psf) 2.5” Normal Weight Concrete on 3” Steel Deck (50 psf) Curtain Wall (15 psf on facade area) Superimposed Dead Load (15 psf) Partitions (15 psf)
Coupled wall length	30ft	Typical bay is 30ft long, the wall would typically run the length of the bay with the wall thickness being the parameter adjusted in design
Floor Dimensions	120ft x 200ft	Review panel recommendation for typical floor geometry
Base Shear Amplification Factor	4	Review panel recommendation

Table 4. 8 story archetype structures

Case	No. Stories	L/d	Cs	Coupled Wall Length, in	Wall Thickness, tsc, in	Plate Thickness, tp, in	CB Length, in	CB Section, in	Uncoupled Wall Length, in	Performance Group
PG-1A		3		144	20	9/16	72	20x24x 3/8(f), 3/8(w)	252	1
PG-1B	8	4	0.076	132	24	9/16	96	24x24x 1/2(f), 3/8(w)	240	1
PG-1C		5		120	24	5/8	120	24x24x 1/2(f), 3/8(w)	240	1
PG-2B	8	4	0.024	144	10	3/16	72	10x18x 3/16(f), 1/4(w)	240	2

Table 5. 12 story archetype structures

Case	No. Stories	L/d	Cs	Coupled Wall Length, in	Wall Thickness, tsc, in	Plate Thickness, tp, in	CB Length, in	CB Section, in	Uncoupled Wall Length, in	Performance Group
PG-1D		3		204	18	9/16	72	18x24x 5/16(f), 3/8(w)	348	1
PG-1E	12	4	0.057	192	22	9/16	96	22x24x 7/16(f), 3/8(w)	336	1
PG-1F		5		180	24	9/16	120	24x24x 1/2(f), 3/8(w)	324	1
PG-2E	12	4	0.017	204	8	3/16	72	8x18x 3/16(f), 1/4(w)	336	2

Table 6. 18 story archetype structures

Case	No. Stories	L/d	Cs	C wall depth, in (c-c)	C wall width, in (c-e)	$t_{sc,f}$, in	$t_{sc,w}$ in	$t_{p,bot}$, in	$t_{p,top}$, in	CB Length, in	CB Section, in	Performance Group
PG-3A		3		360	180	18	14	1/2	5/16	72	18x24x 5/16(f), 3/8(w)	3
PG-3B	18	4	0.042	360	168	24	14	1/2	5/16	96	24x24x 7/16(f), 3/8(w)	3
PG-3C		5		360	156	26	16	9/16	5/16	120	26x24x 1/2(f), 3/8(w)	3
PG-4B	18	4	0.014	360	162	12	12	3/16	3/16	72	12x18x 1/4(f), 1/4(w)	4

Table 7. 22 story archetype structures

Case	No. Stories	L/d	Cs	C wall depth, in (c-c)	C wall width, in (c-e)	$t_{sc,f}$, in	$t_{sc,w}$ in	$t_{p,bot}$, in	$t_{p,top}$, in	CB Length, in	CB Section, in	Performance Group
PG-3D		3		360	204	20	14	1/2	3/8	72	20x24x 3/8(f), 3/8(w)	3
PG-3E	22	4	0.036	360	192	24	14	1/2	3/8	96	24x24x 7/16(f), 3/8(w)	3
PG-3F		5		360	180	28	16	9/16	3/8	120	28x24x 9/16(f), 3/8(w)	3
PG-4E	22	4	0.012	360	162	14	10	3/16	3/16	72	14x18x 1/4(f), 1/4(w)	4

Table 8. Archetype structures – coupling ratio, strength, and inter-story drift ratio

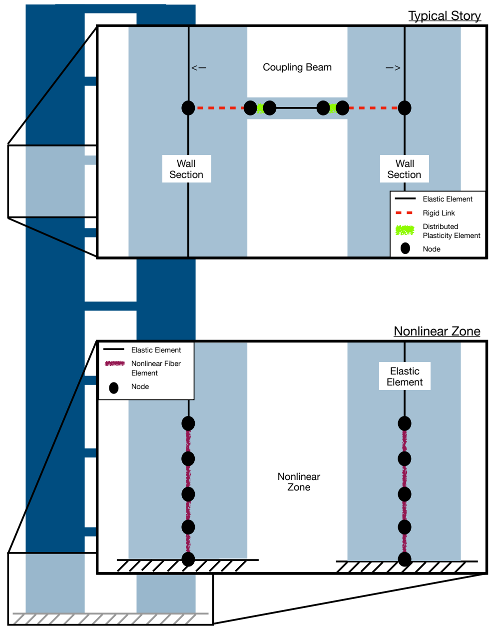
Case	No. Stories	CR_s (%)	Wall Strength Margin		Coupling Beam Strength Margin		Max. Inter-story Drift Ratio (%)
			$\phi M_{n,wall}/M_{u,wall}$	$\phi V_{n,wall}/V_{u,wall}$	$\phi M_{n,CB}/M_{u,CB}$	$\phi V_{n,CB}/V_{u,CB}$	
PG-1A	8	20.8	0.98	6.30	1.42	1.19	1.35
PG-1B		28.1	1.17	6.00	1.30	1.39	1.52
PG-1C		35.6	1.88	5.93	1.03	1.74	1.92
PG-2B	8	18.5	1.43	7.26	1.29	1.31	1.39
PG-1D	12	15.0	1.09	7.81	1.44	1.15	1.32
PG-1E		20.3	1.14	7.70	1.42	1.45	1.39
PG-1F		25.7	1.33	7.34	1.28	1.74	1.65
PG-2E	12	13.9	1.35	8.91	1.40	1.10	1.24
PG-3A	18	17.2	1.02	2.84	1.12	1.14	1.77
PG-3B		25.1	1.10	2.79	1.09	1.04	1.90
PG-3C		29.8	1.29	2.90	1.05	1.09	2.10
PG-4B	18	18.6	1.07	3.14	1.20	1.40	1.96
PG-3D	22	17.5	0.95	3.12	1.21	1.06	1.89
PG-3E		24.1	1.21	3.03	1.07	1.02	2.01
PG-3F		29.7	1.25	3.22	1.12	1.03	1.99
PG-4E	22	21.6	0.90	2.98	1.06	1.14	1.92

BRIEF MODEL COMPARISON

Differences between Purdue and SUNY Buffalo OpenSees modeling approaches

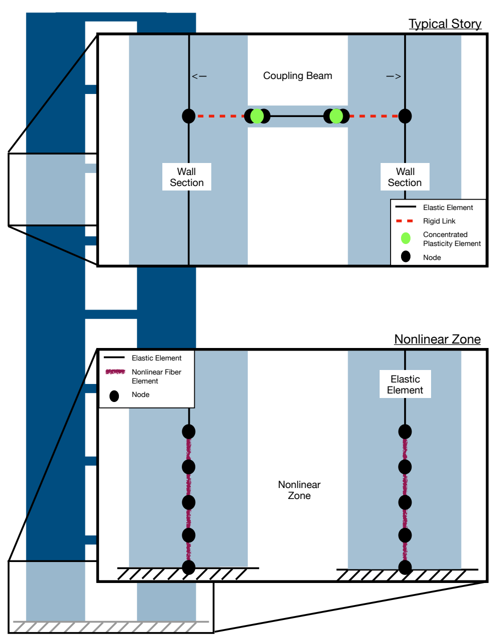
SUNY Buffalo Model

- Wall behavior captured with nonlinear fiber elements
- Coupling beam behavior represented by **fiber elements over fixed length**
- Model runs relatively slowly
- Coupling beam behavior directly tied to section geometry
- Material behavior defined using established models shown to fit experimental data



Purdue Model

- Wall behavior captured with nonlinear fiber elements
- Coupling beam behavior represented by **zero-length rotational spring**
- Model runs relatively quickly
- Coupling beam behavior not directly tied to section geometry
- Material behavior defined using effective stress-strain curves determined from 3D finite element models of experimental tests



Purdue Model Results

Concentrated Plasticity IDA Curves

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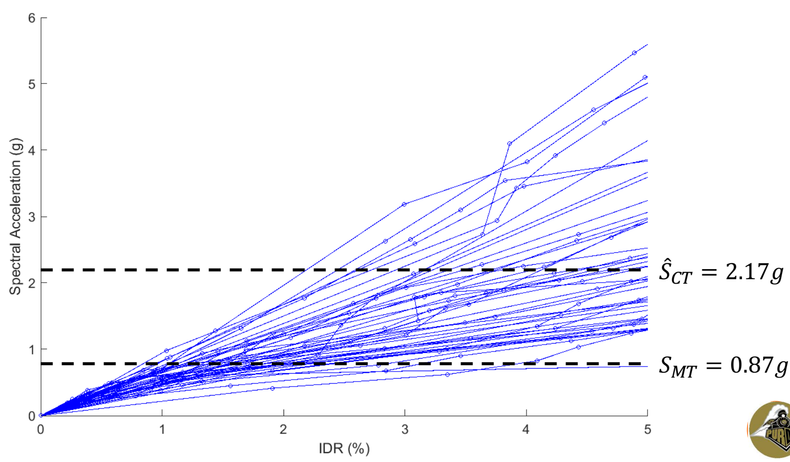
PG-1A: IDA Curve

CMR = 2.51
ACMR = 3.15



8-Story; Dmax; L/d = 3; μ_T = 3 assumed

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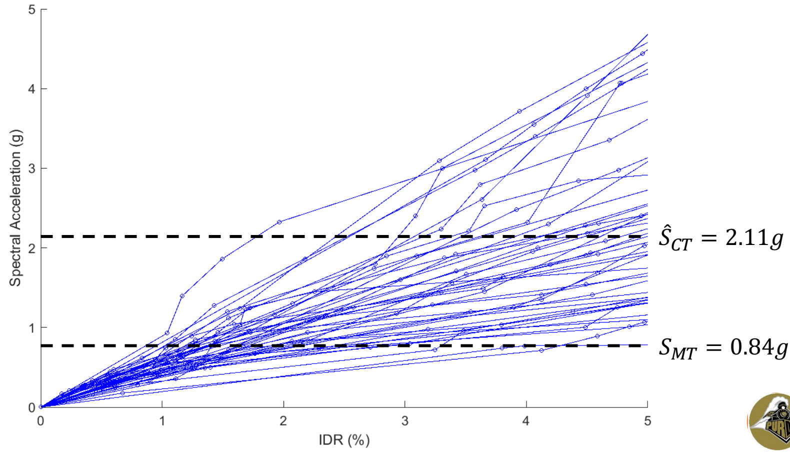
PG-1B: IDA CURVE

CMR = 2.52
ACMR = 3.19



8-Story; Dmax; L/d = 4; $\mu_T = 3$ assumed

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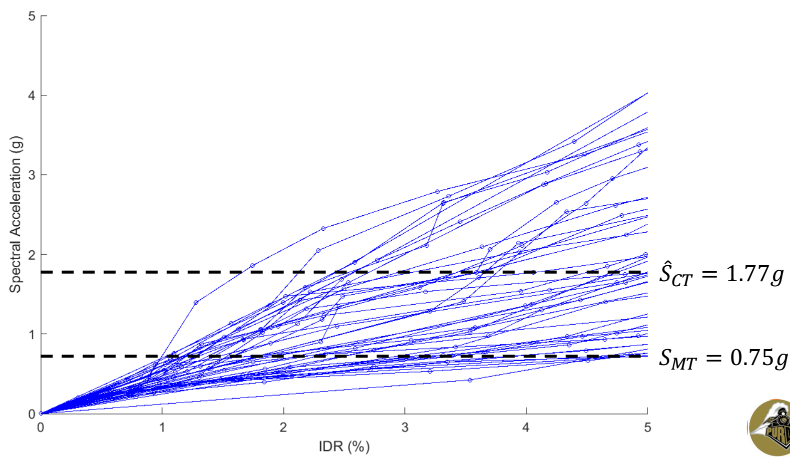
PG-1C: IDA CURVE

CMR = 2.38
ACMR = 3.04



8-Story; Dmax; L/d = 5; $\mu_T = 3$ assumed

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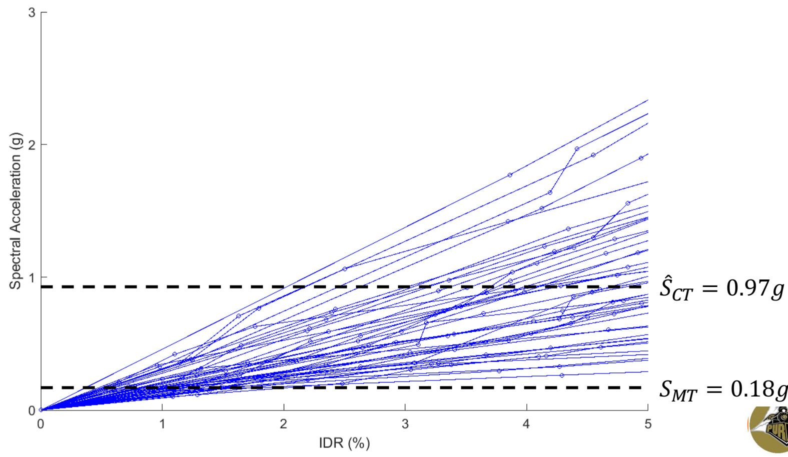
PG-2B: IDA CURVE

CMR = 5.35
ACMR = 6.51



8-Story; Dmin; L/d = 4; μ_T = 3 assumed

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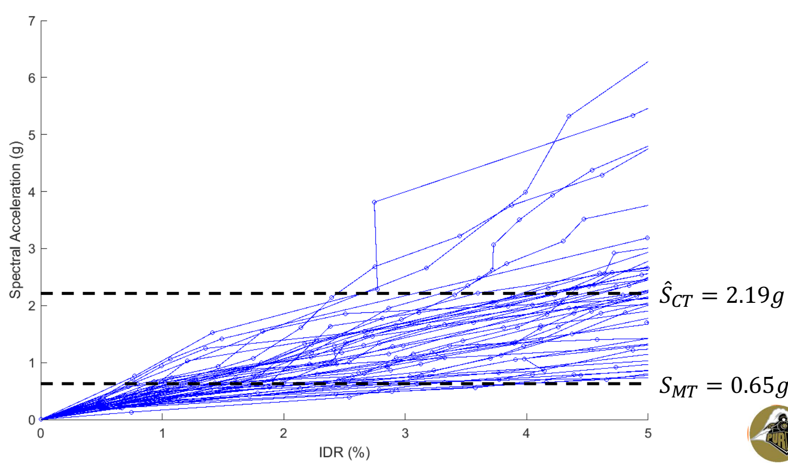
PG-1D: IDA CURVE

CMR = 3.39
ACMR = 4.43



12-Story; Dmax; L/d = 3; μ_T = 3 assumed

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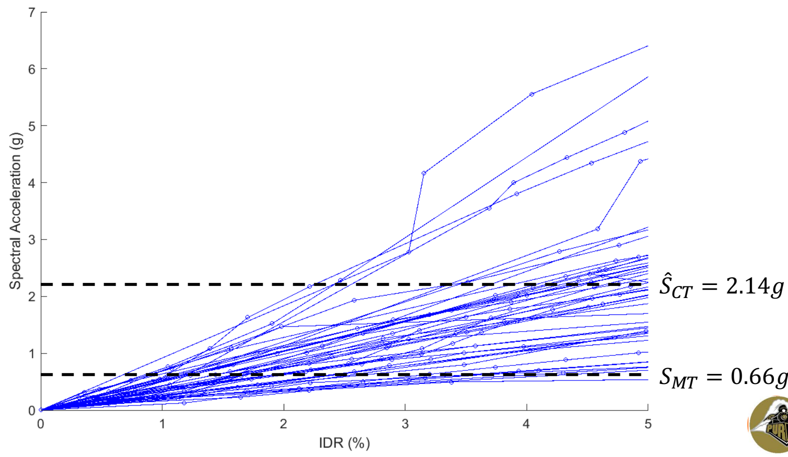
PG-1E: IDA Curve

CMR = 3.27
ACMR = 4.28



12-Story; Dmax; L/d = 4; μ_T = 3 assumed

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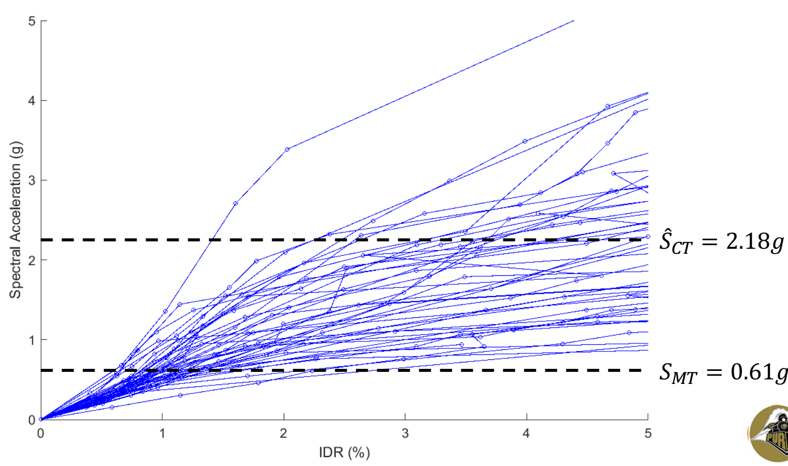
PG-1F: IDA Curve

CMR = 3.57
ACMR = 4.71



12-Story; Dmax; L/d = 5; μ_T = 3 assumed

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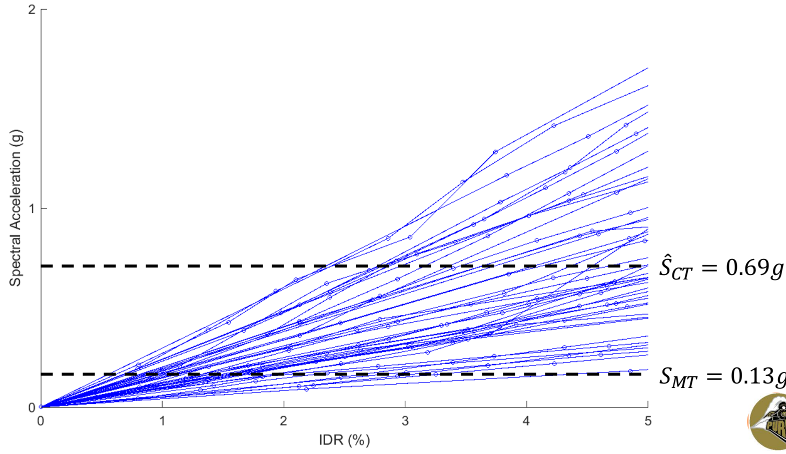
PG-2E: IDA CURVE

CMR = 5.32
ACMR = 6.44



12-Story; Dmin; L/d = 4; μ_T = 3 assumed

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COMPARISON OF ACMR VALUES

	SUNY Buffalo ACMR	Purdue ACMR	FEMA ACMR Threshold Criteria*
PG-1A	3.55	3.15	2.22
PG-1B	3.82	3.19	2.22
PG-1C	3.65	3.04	2.22
PG-1D	4.13	4.42	2.22
PG-1E	5.31	4.28	2.22
PG-1F	5.07	4.71	2.22
Average	4.26	3.80	3.38

<u>All Pass</u>

(ACMR > Threshold)

*Looking at worst case scenario where all β factors are 'poor'

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TIME HISTORY ANALYSIS USING ABAQUS

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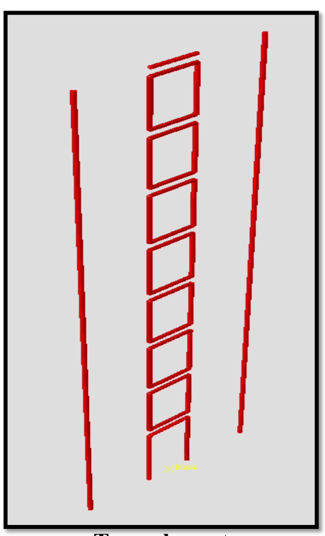
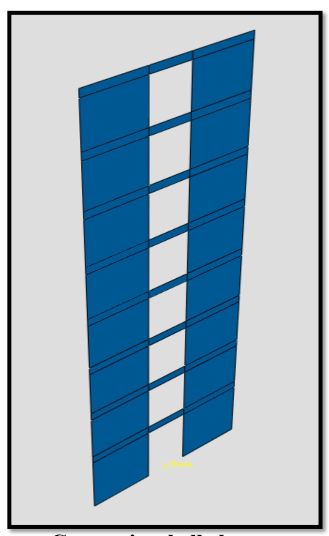
ABAQUS MODEL



Truss elements
T3D2

+



Composite shell elements
S4R

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Model Comparison

ABAQUS	OpenSees
Composite shell elements and truss elements	Fiber, concentrated plasticity, and elastic elements
Captures: Buckling using effective stress-strain material behavior, fracture criteria (initiated at 18% plastic strain), shear failure	Captures: Buckling using effective stress-strain curve, some fracture initiation, no shear
Run time ranges from 2 hours to 3 days	Run time approximately 30 minutes

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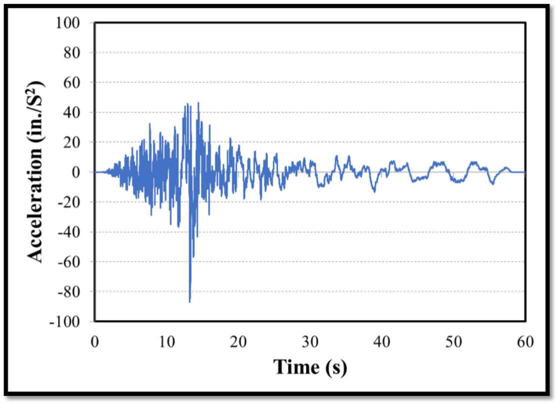
Progression of Collapse of 12-Story CF-CPSW structures (PG-1E)

TIME-HISTORY ANALYSIS

Normalized Records:
Superstition Hills, (El Centro Imp. Co.)

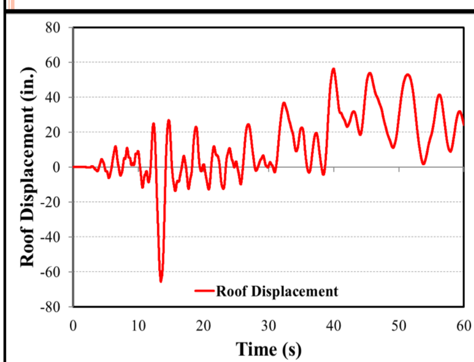
(Acceleration vs. Time)

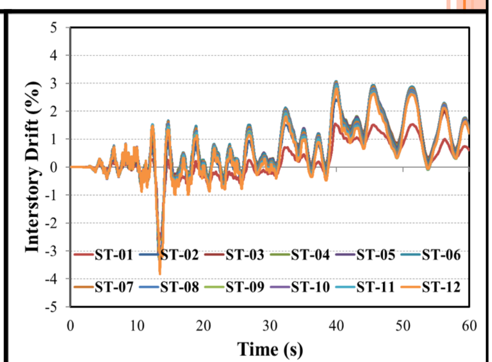


BICC090-16-2

TIME-HISTORY ANALYSIS

PG-1E-BICC090:
Response at scale factor = 7
Maximum interstory drift = 3.84%





Roof Displacement **Inter-story Drift**

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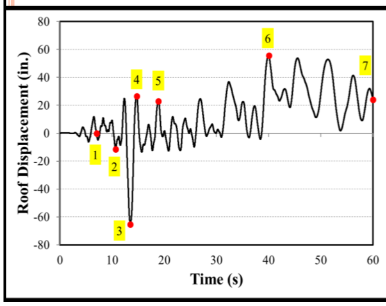
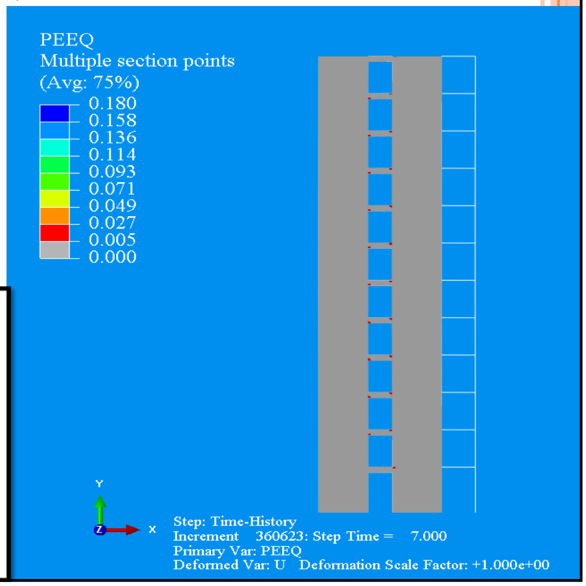
TIME-HISTORY ANALYSIS

PG-1E-BICC090:
Response at scale factor = 7

1- Yielding of CBs
2- Yielding of CF-CPSWs
3- Propagation of yielding of CB connections over CF-CPSW and yielding at the bottom of CF-CPSW
4- Fracture initiation of CBs
5- Fracture initiation of CF-CPSWs
6- Total fracture of CBs
7- End of earthquake record





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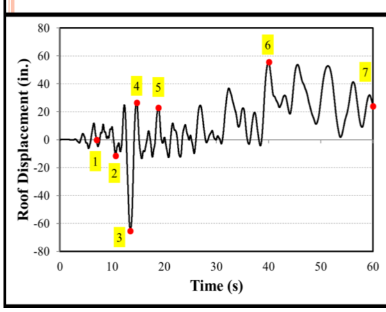
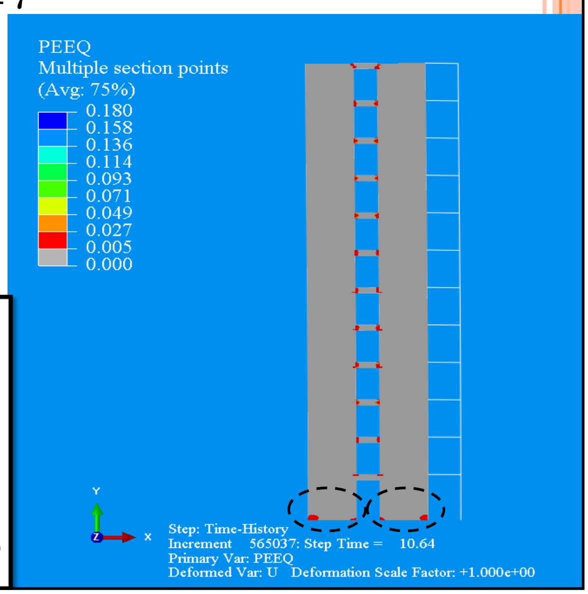
TIME-HISTORY ANALYSIS

PG-1E-BICC090:
Response at scale factor = 7

1- Yielding of CBs
2- Yielding of CF-CPSWs
3- Propagation of yielding of CB connections over CF-CPSW and yielding at the bottom of CF-CPSW
4- Fracture initiation of CBs
5- Fracture initiation of CF-CPSWs
6- Total fracture of CBs
7- End of earthquake record





TIME-HISTORY ANALYSIS

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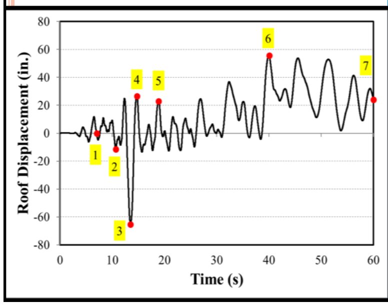

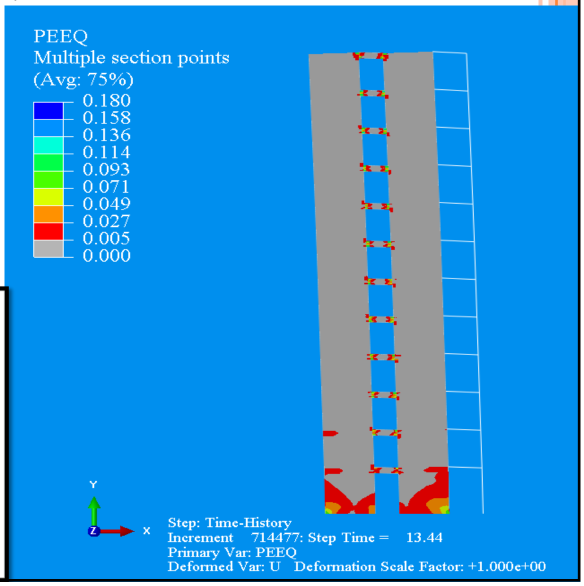

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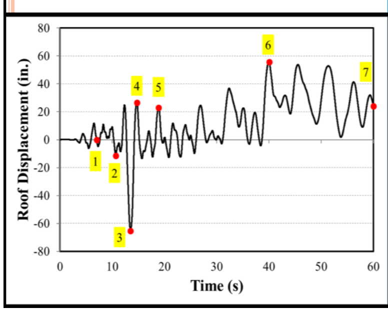

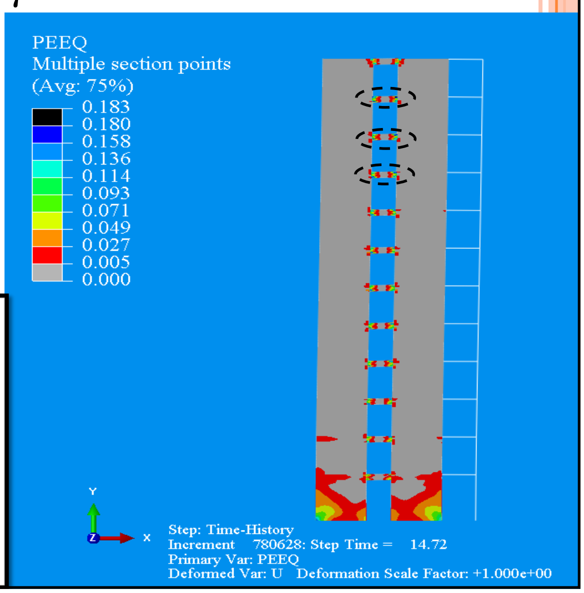

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TIME-HISTORY ANALYSIS

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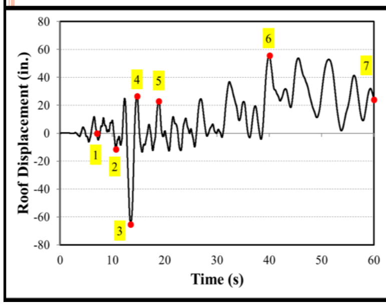

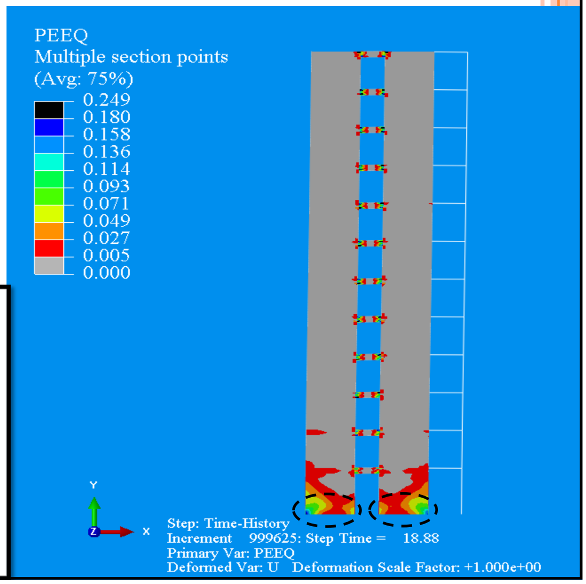

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TIME-HISTORY ANALYSIS

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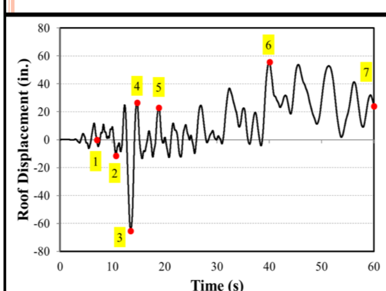

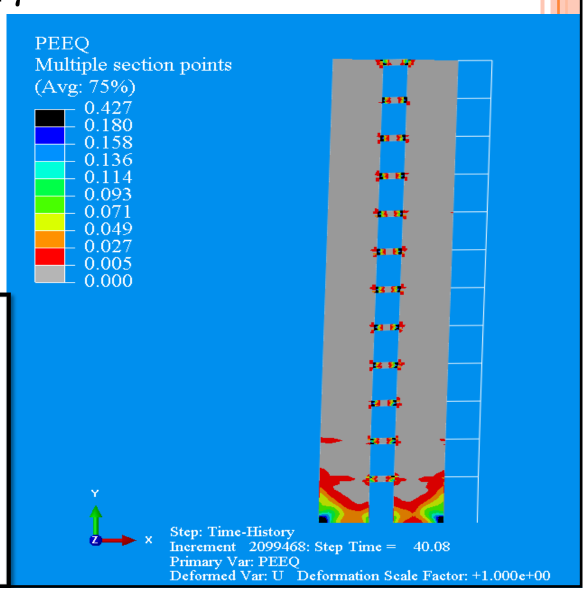

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TIME-HISTORY ANALYSIS

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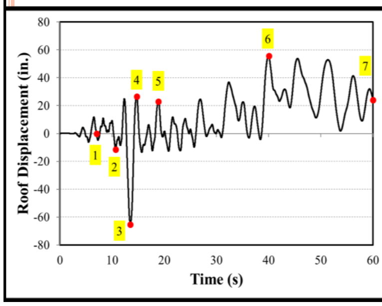

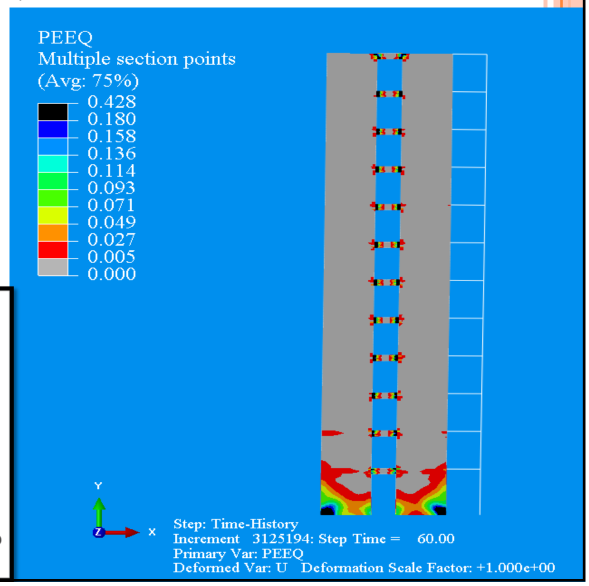

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TIME-HISTORY ANALYSIS

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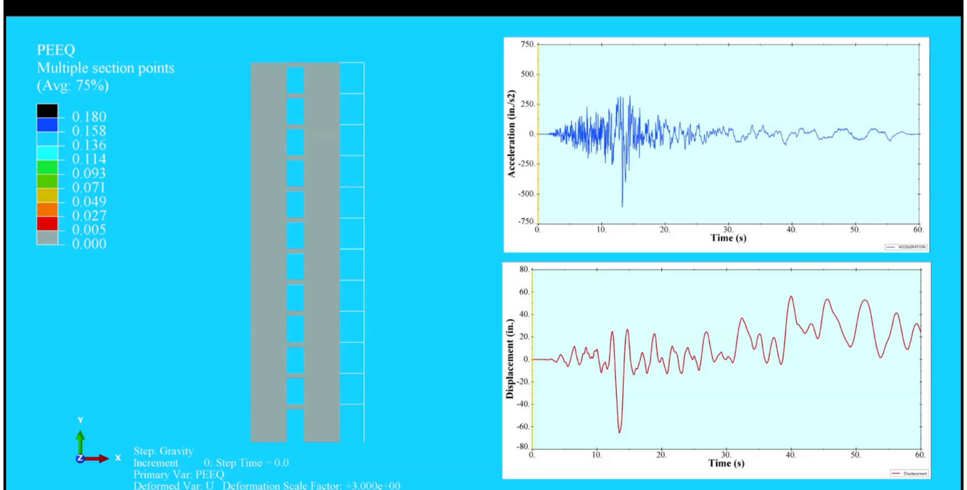

ABAQUS-OpenSees Comparison

PG-1E

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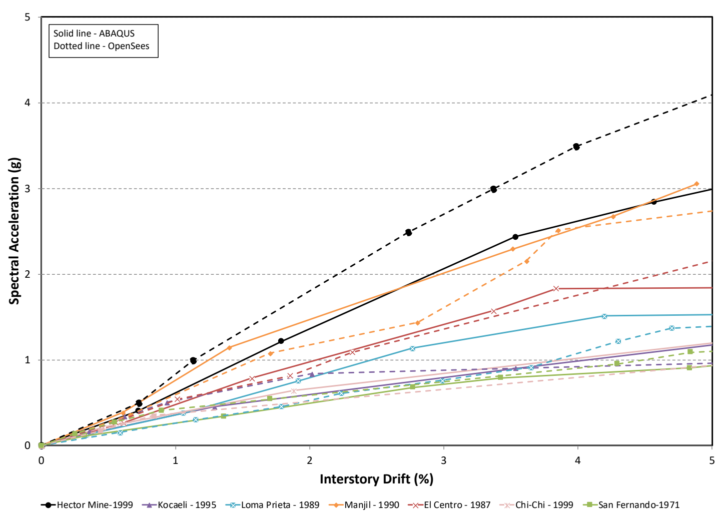
PG-1E: ABAQUS-OpenSees Comparison



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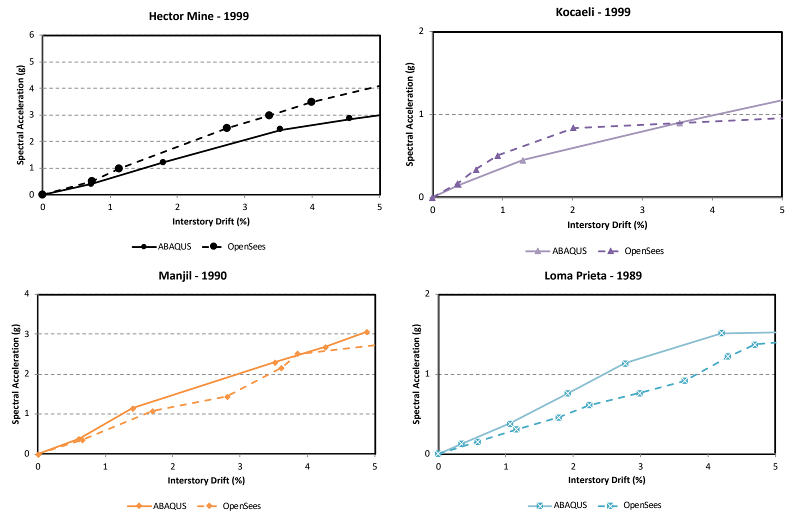
PG-1E: ABAQUS-OPENSEES COMPARISON PLOTS



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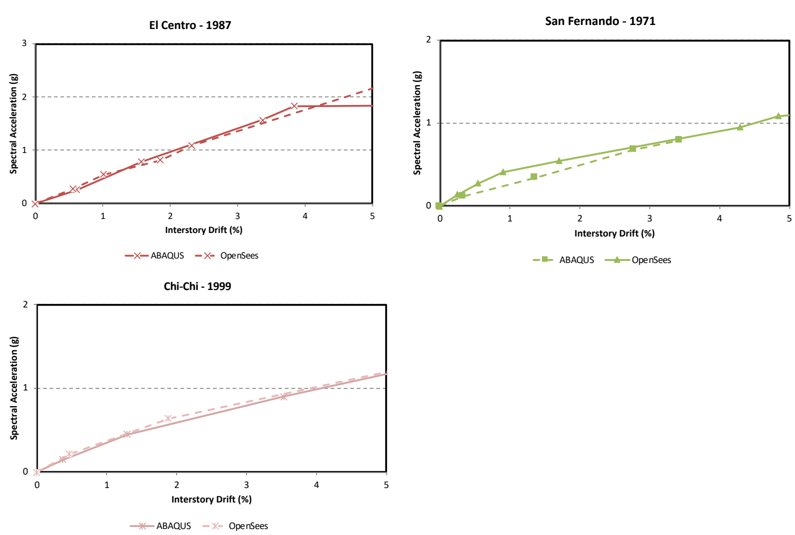
PG-1E: ABAQUS-OPENSEES COMPARISON PLOTS



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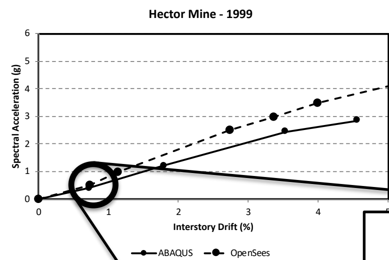
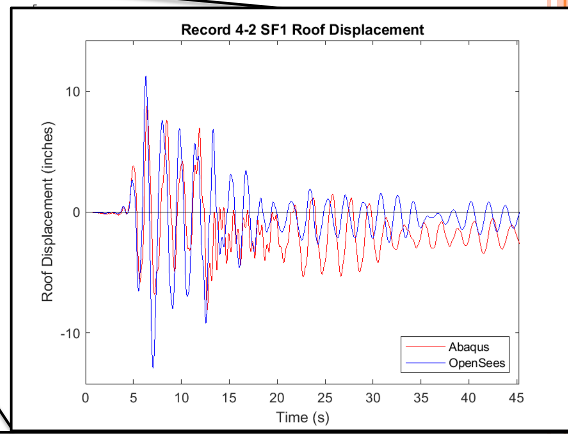
PG-1E: Roof Displacement Comparison





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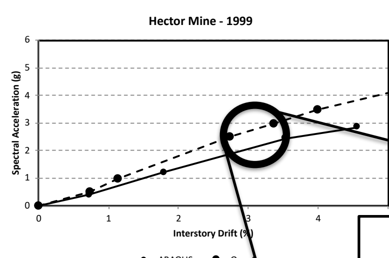

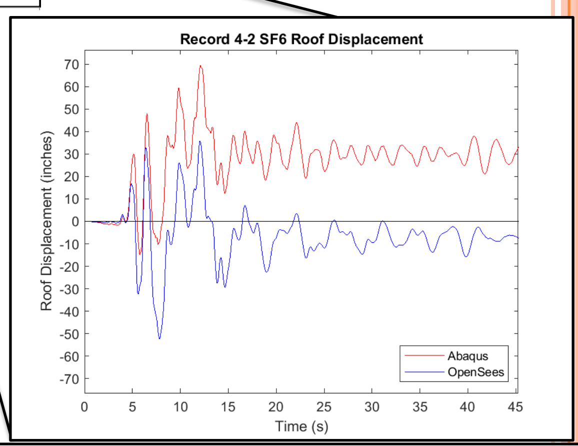

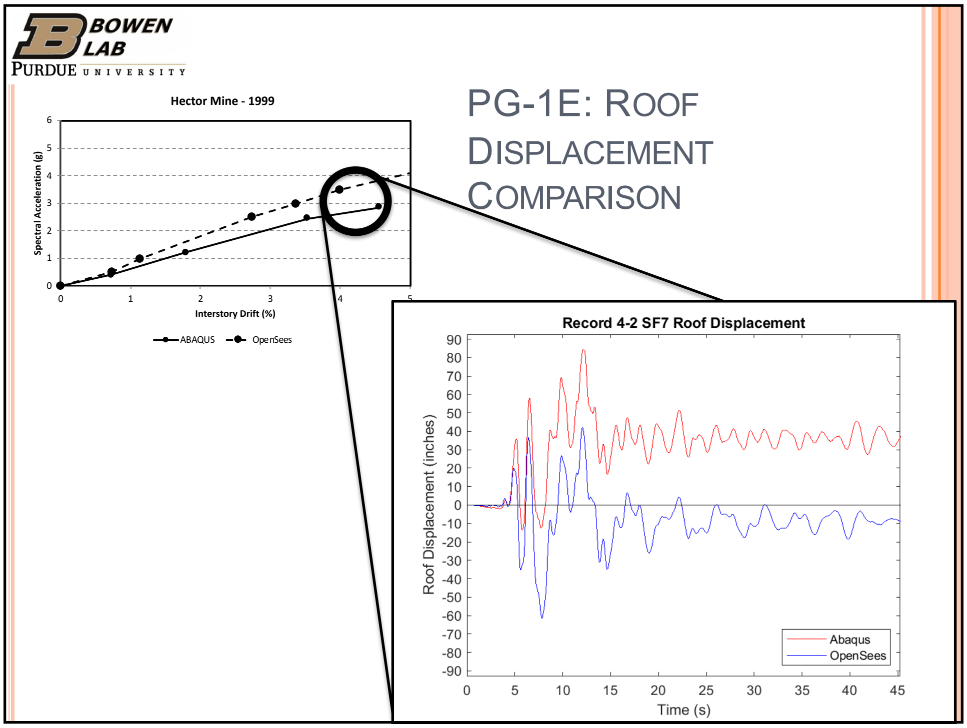

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PG-1E: ROOF DISPLACEMENT COMPARISON
Hector Mine - 1999
Spectral Acceleration (g)
Interstory Drift (%)
ABAQUS
OpenSees
Record 4-2 SF7 Roof Displacement
Roof Displacement (inches)
Time (s)
Abaqus
OpenSees