Challenges in Collapse Prediction for Steel Moment Frame Structures

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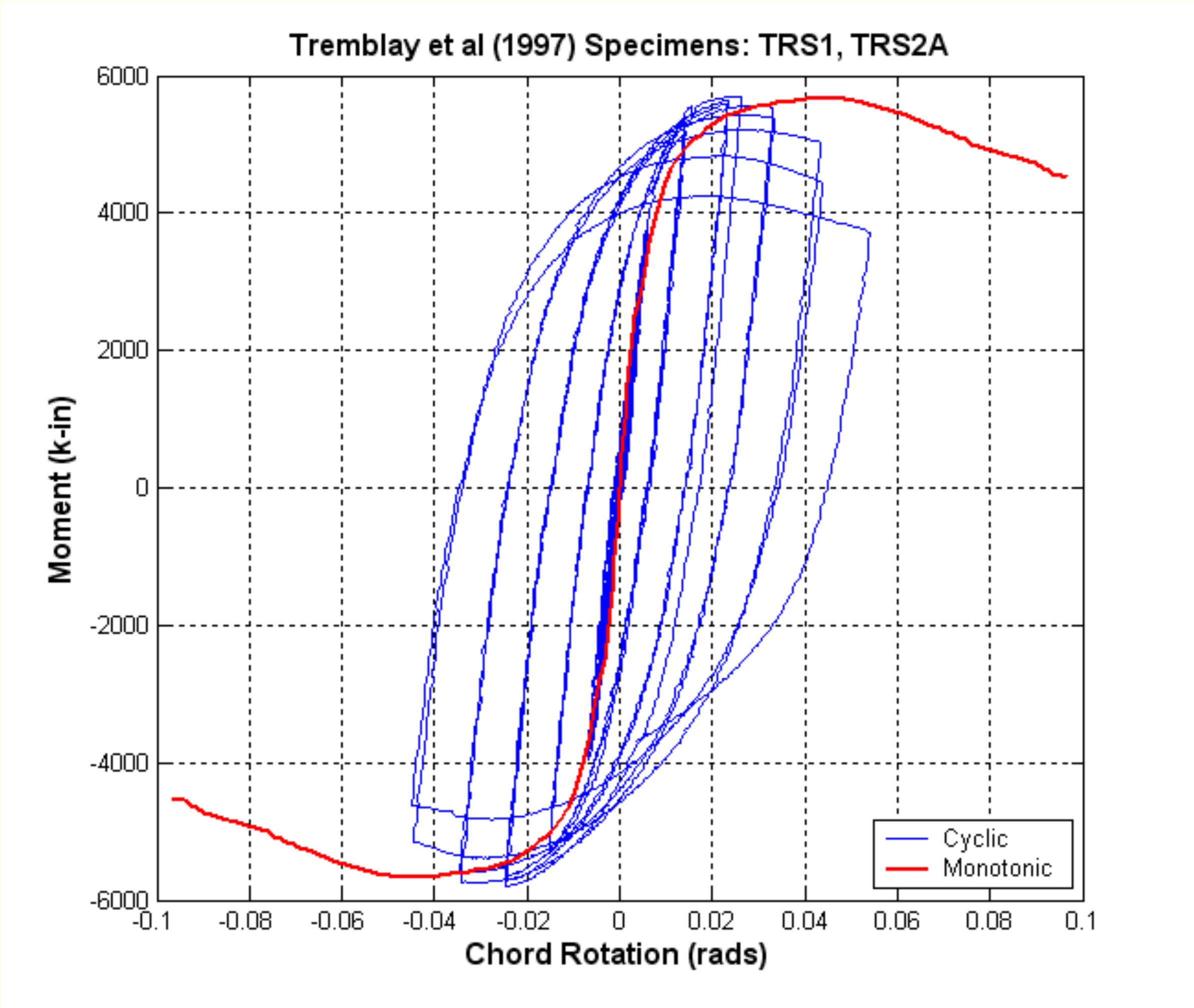
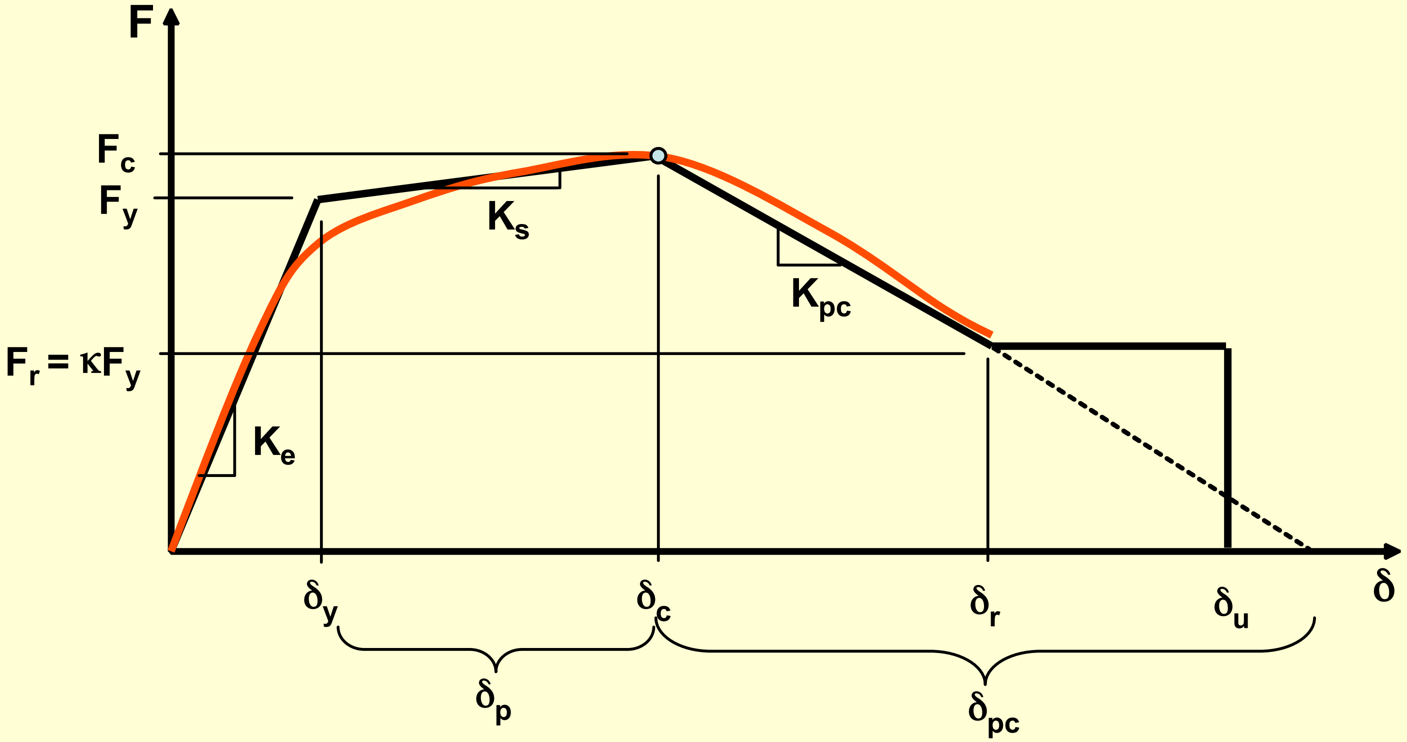
Loading History Effects
Identical Steel Beams



Component Models with Deterioration

1. Backbone Curve:



2. Cyclic Deterioration Parameter

Cyclic Deterioration Parameter

A single deterioration parameter:

$$\boxed{\beta_i = \left(\frac{E_i}{E_t - \sum_{j=1}^{i} E_j} \right)^c}$$

in which β_i = **parameter defining the deterioration in excursion *i***

E_i = **hysteretic energy dissipated in excursion *i***

E_t = **hysteretic energy dissipation capacity = $\lambda\theta_p M_y = \Lambda M_y$**

$\sum E_j$ = **hysteretic energy dissipated in all previous excursions**

c = **exponent defining the rate of deterioration**

Steel Component Modeling

- Deterioration parameters (θ_p, θ_{pc}, Λ) based on database of 300 beam specimens
  - Tests fully documented
  - Separate calibrations for RBS and “other-than RBS” connections
  - Deterioration due to local and lateral torsional buckling
  - Sensitivity to geometric parameters
    - d, L/d, L_b/r_y, d/t_w, b/t
  - Inadequate data for columns (heavy sections, axial load effect)

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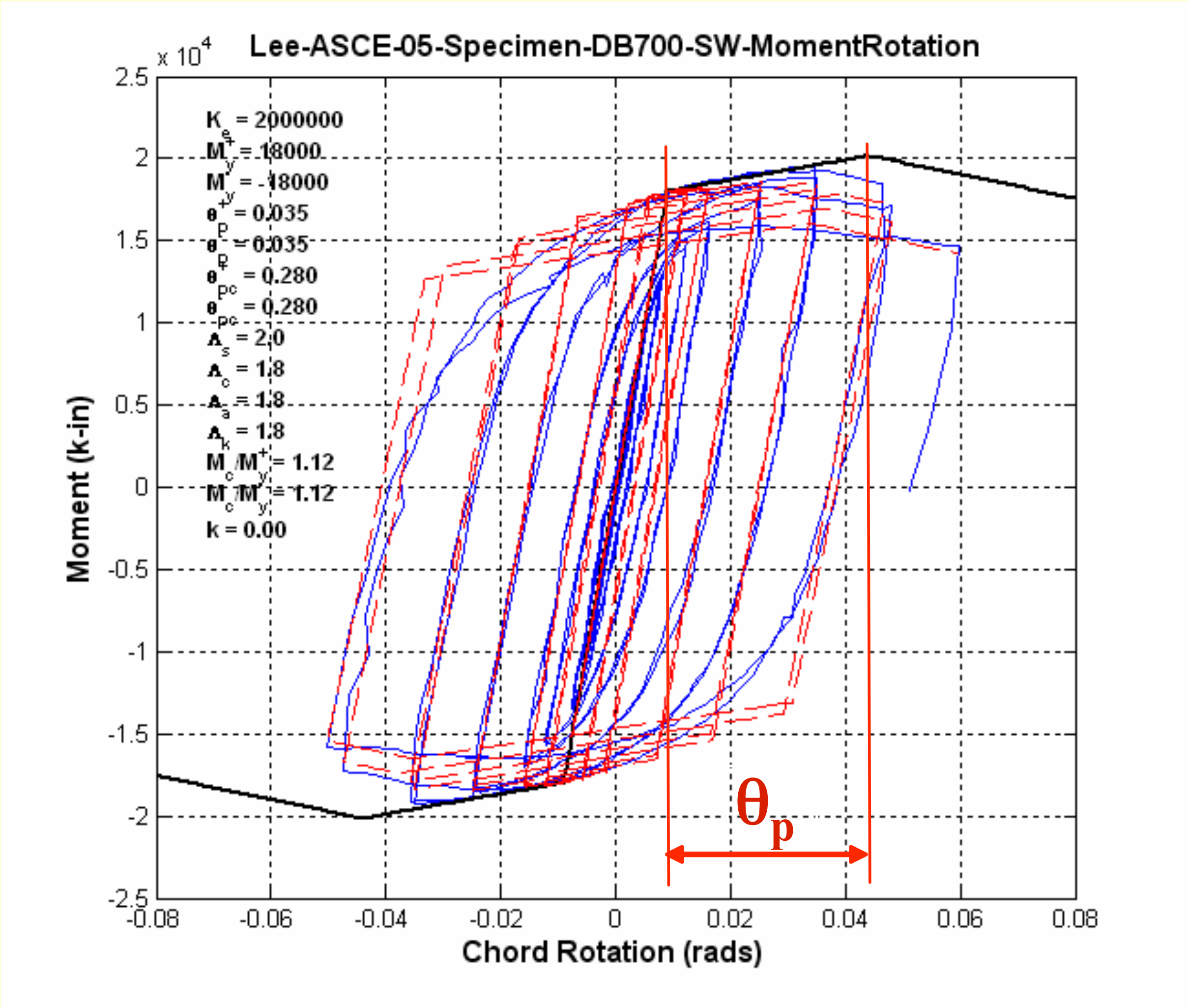
Calibration Process



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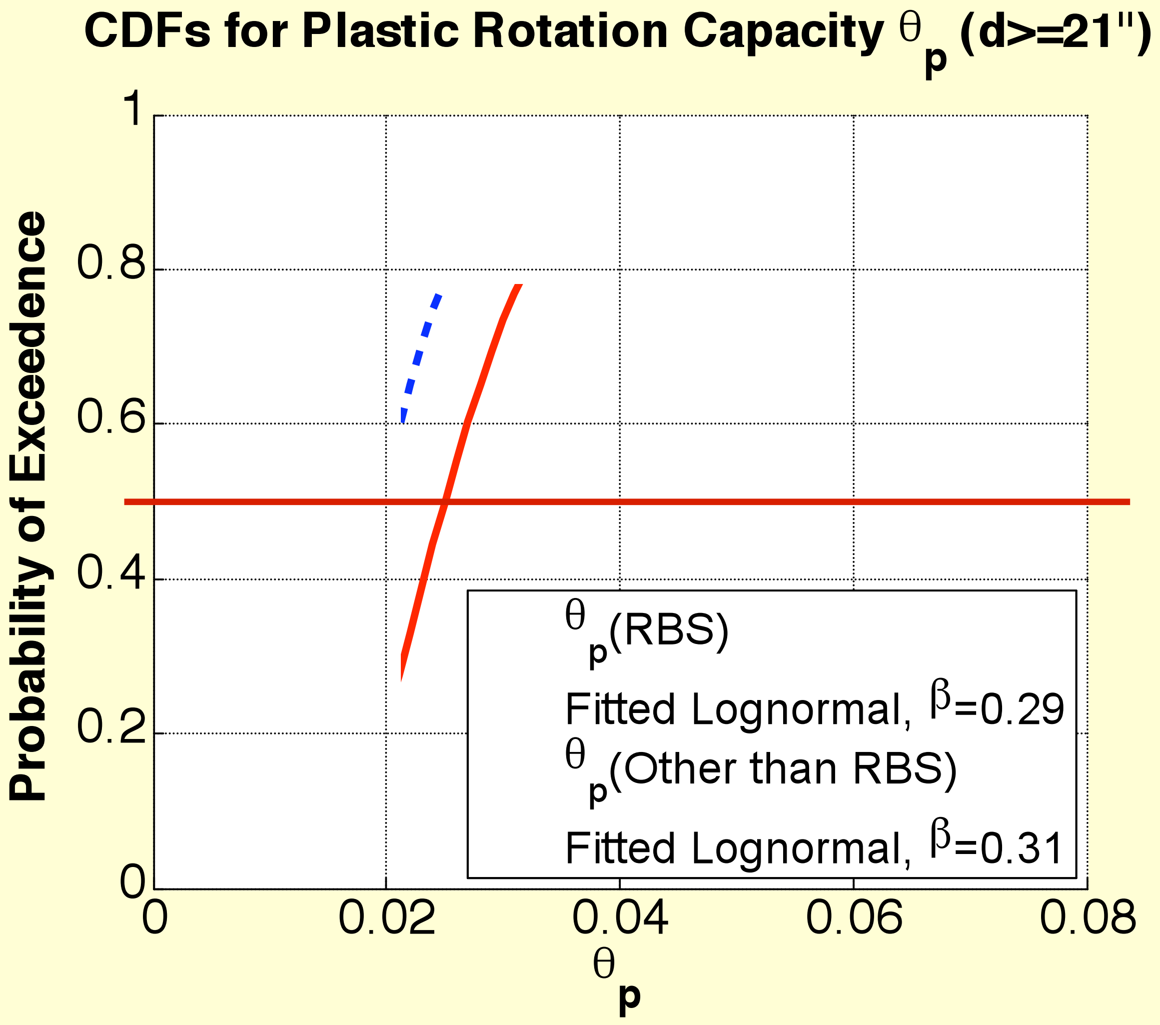
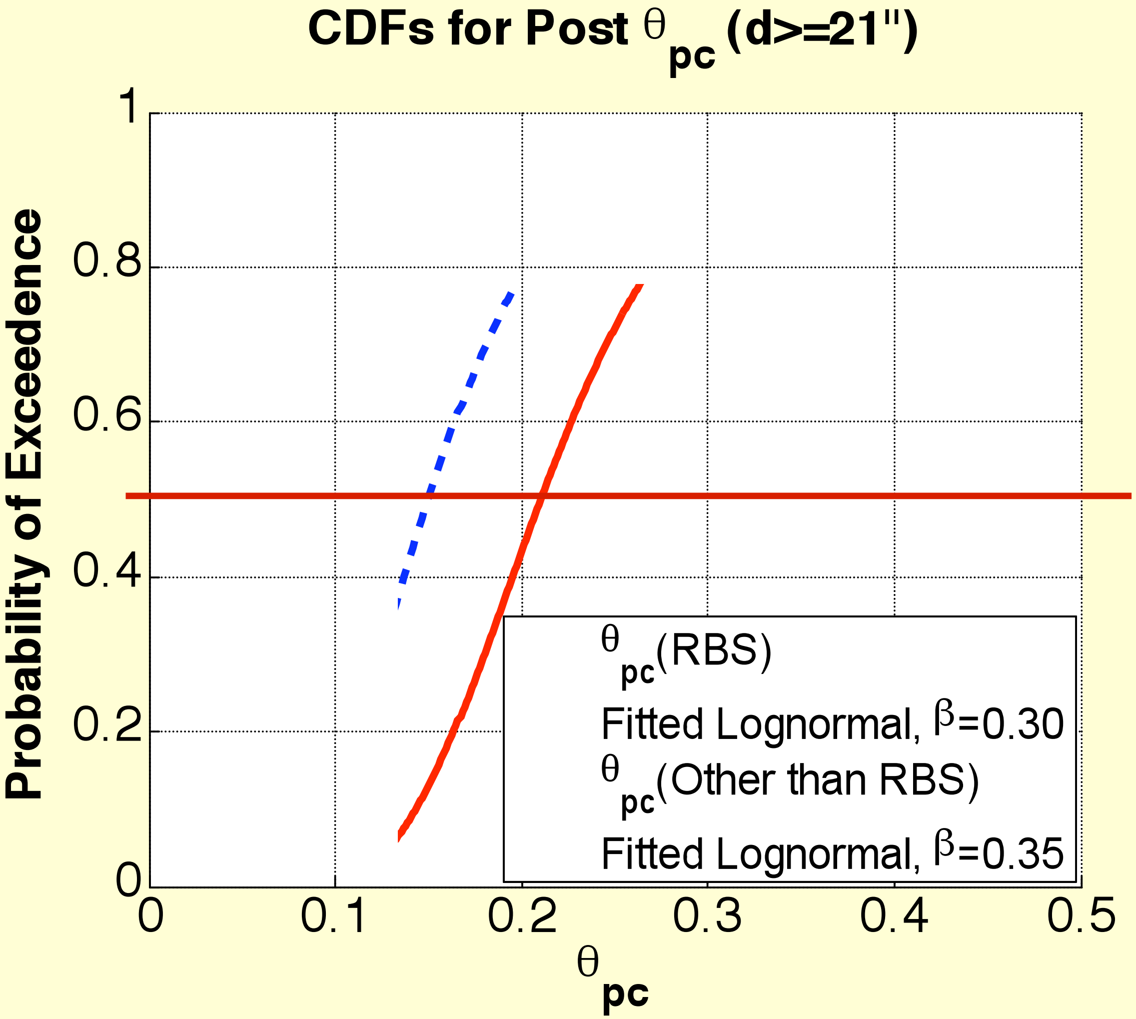
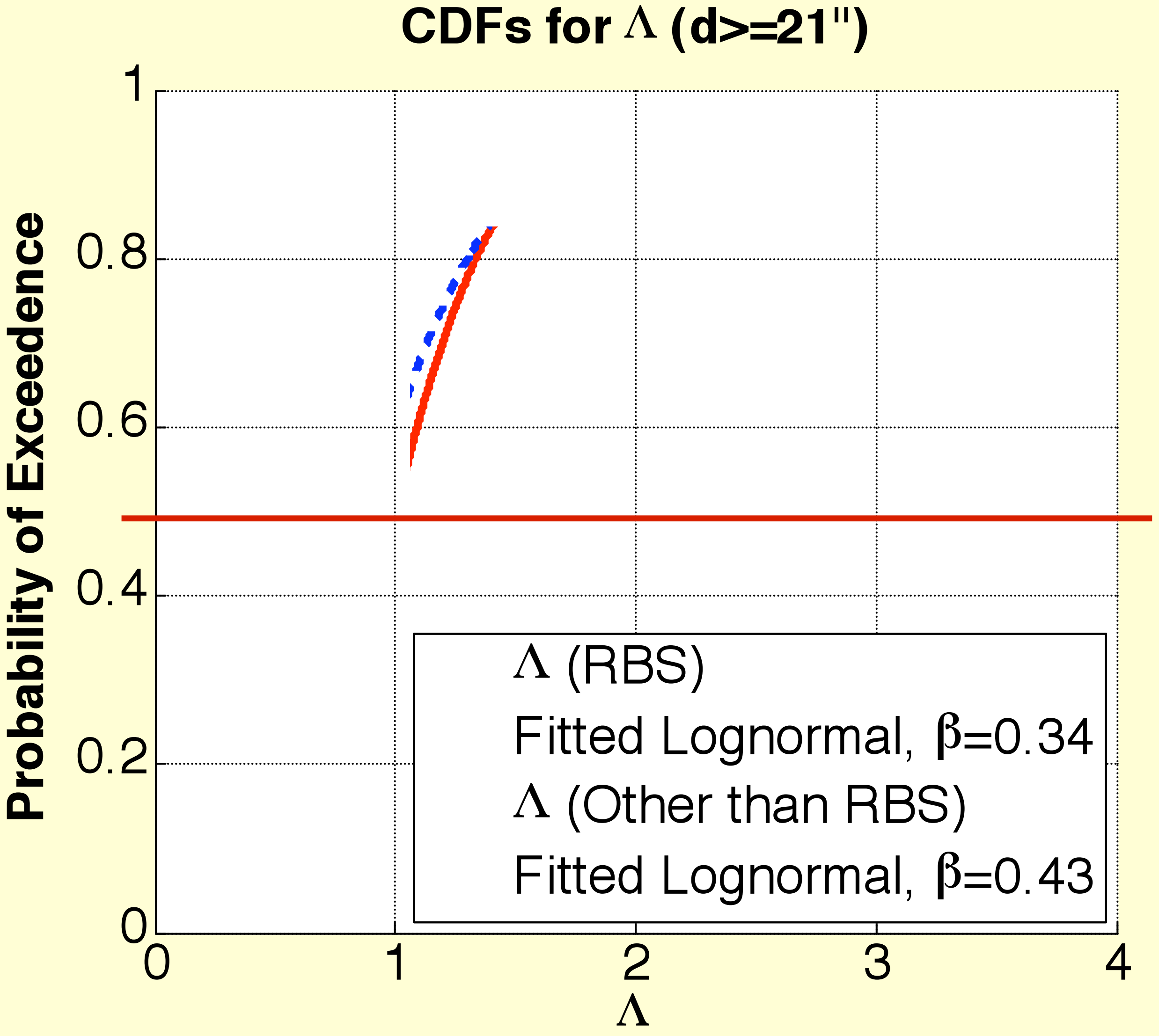
Statistics for θ_p



Statistics for θ_{pc}



Statistics for Λ



Component Modeling

Results from database:

Regression eqs. for deterioration parameters, e.g.,

$$\theta_p = 0.087 \cdot \left(\frac{h}{t_w}\right)^{-0.365} \cdot \left(\frac{b_f}{2 \cdot t_f}\right)^{-0.14} \cdot \left(\frac{L}{d}\right)^{0.34} \cdot \left(\frac{d}{c_{unit}^{1} \cdot 21''}\right)^{-0.721} \cdot \left(\frac{c_{unit}^{2} \cdot F_y}{50}\right)^{-0.23}$$

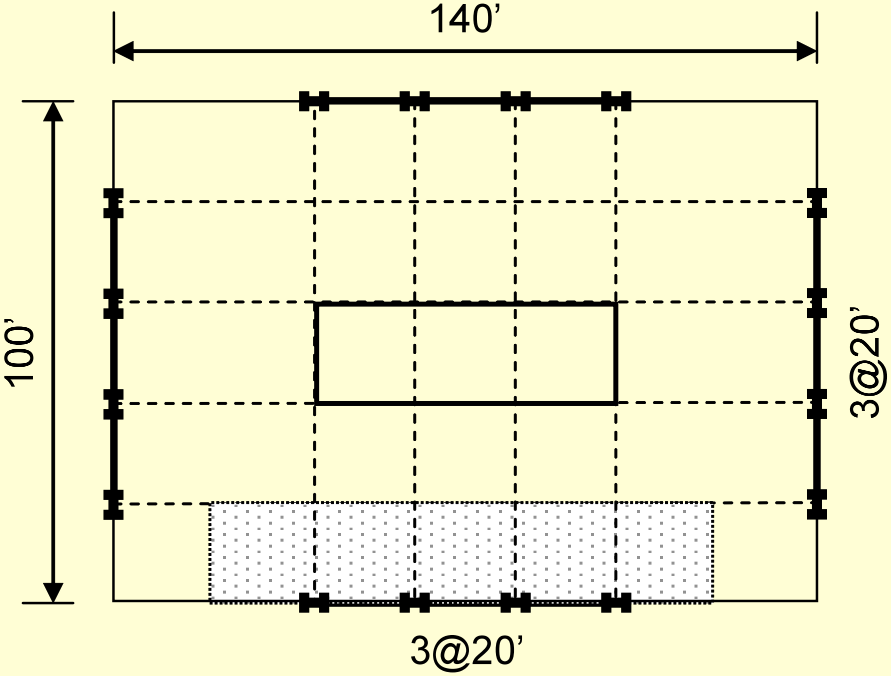
Section	θ_p (rad)	θ_{pc} (rad)	Λ
W14x257	0.078	0.53	10.2
W24x207	0.034	0.28	2.8
W36x150	0.017	0.12	0.8

Implementation: Collapse Prediction for Archetype Steel Moment Frame Structures

- Part of ATC 76-1 project
- Testing of FEMA P695 (ATC-63) methodology for collapse prediction

Steel SMRF Archetypes

- Occupancy - Office
- Number of stories:
  - 1, 2, 4, 8, 12, 20
- Story heights:
  - First story = 15’
  - Other stories, h = 13’
  - 3 bays @20’ MRF only
- Perimeter frames only
- RBS connections



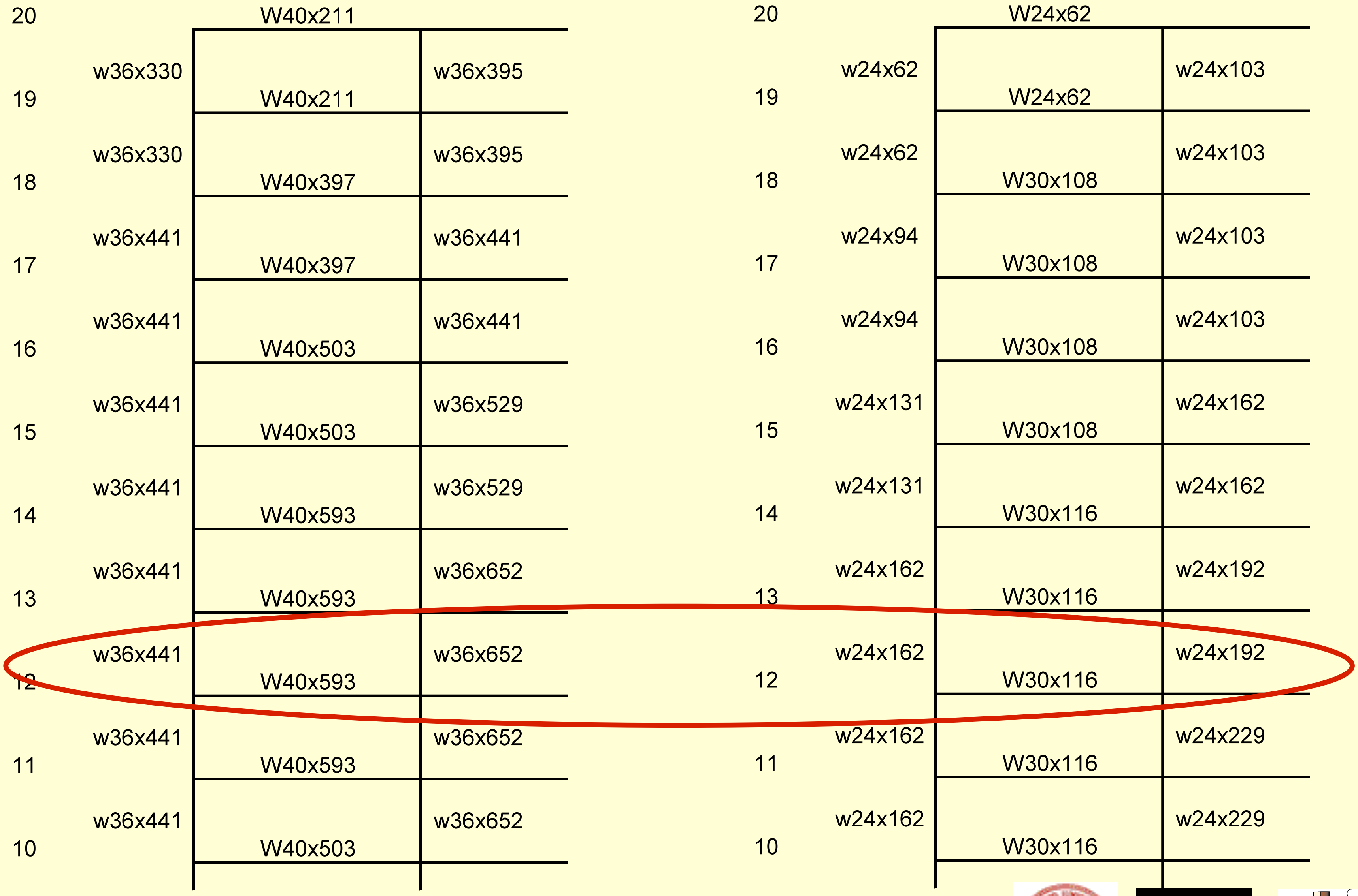
Performance Groups

- Seismic Design Categories:
  - D – maximum:
    - Ss = 1.5g, S1 = 0.6g
    - S_{DS} = (2/3)(1.0*1.5) = 1.0g
    - S_{D1} = (2/3)(1.5*0.6) = 0.6g
  - D – minimum:
    - Ss = 0.55g, S1 = 0132g
    - S_{DS} = (2/3)(1.36*0.55) = 0.50g
    - S_{D1} = (2/3)(2.28*0.13) = 0.20g
- Design Procedures
  - ELF design
  - RSA design

What Controls Design of Steel SMF?

- Code strength design is mostly irrelevant
- Most member sizes are controlled by drift limitations or P-Delta design requirements
- For taller frame structures largest beam sizes are required around midheight in order to control drift
- Because of strong-column / weak girder design requirements largest columns are also around midheight for taller frame structures
- This leads to “excessive” column sizes close to bottom of structure
- For taller frames, ELF designs become “irrational”

20-story, D_{max} -- ELF & RSA



20-story ELF

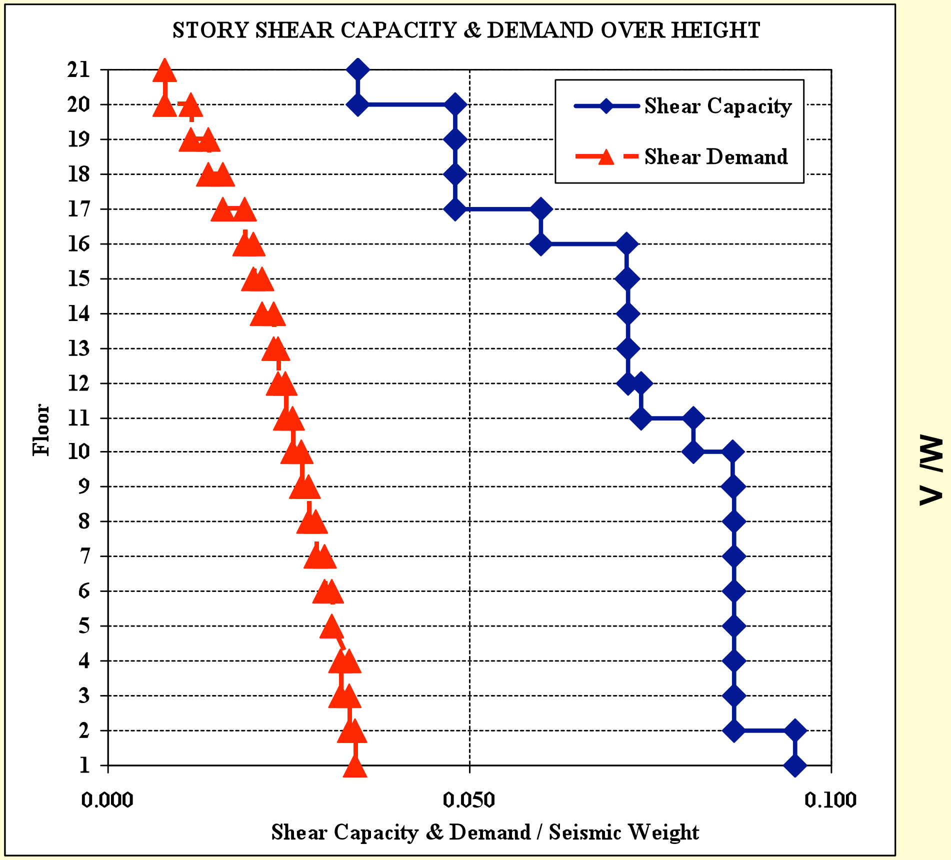
20	W40x211	w36x330	w36x395
19	W40x211	w36x330	w36x395
18	W40x397	w36x441	w36x441
17	W40x397	w36x441	w36x441
16	W40x503	w36x441	w36x529
15	W40x503	w36x441	w36x529
14	W40x593	w36x441	w36x652
13	W40x593	w36x441	w36x652
12	W40x593	w36x441	w36x652
11	W40x593	w36x441	w36x652
10	W40x503		

20-story RSA

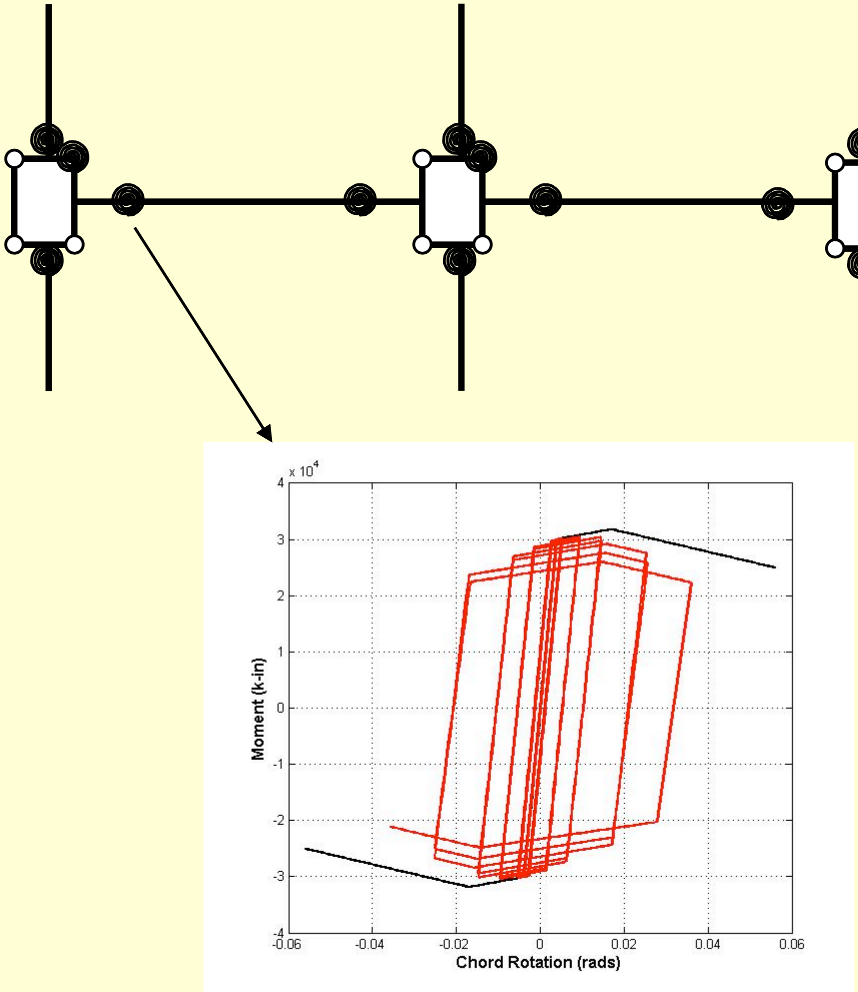
20	W24x62	w24x62	w24x103
19	W24x62	w24x62	w24x103
18	W30x108	w24x94	w24x103
17	W30x108	w24x94	w24x103
16	W30x108	w24x131	w24x162
15	W30x108	w24x131	w24x162
14	W30x116	w24x162	w24x192
13	W30x116	w24x162	w24x192
12	W30x116	w24x162	w24x229
11	W30x116	w24x162	w24x229
10	W30x116		

Overstrength

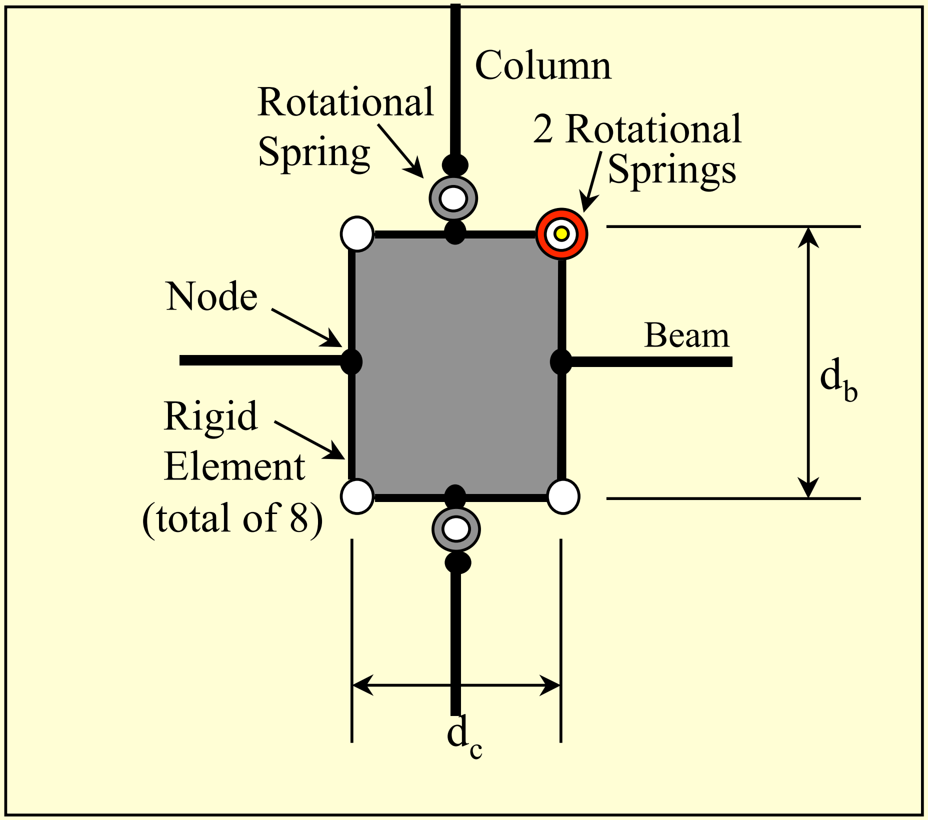
Large overstrength for most buildings is direct result of drift or P-Delta controlling member sizes

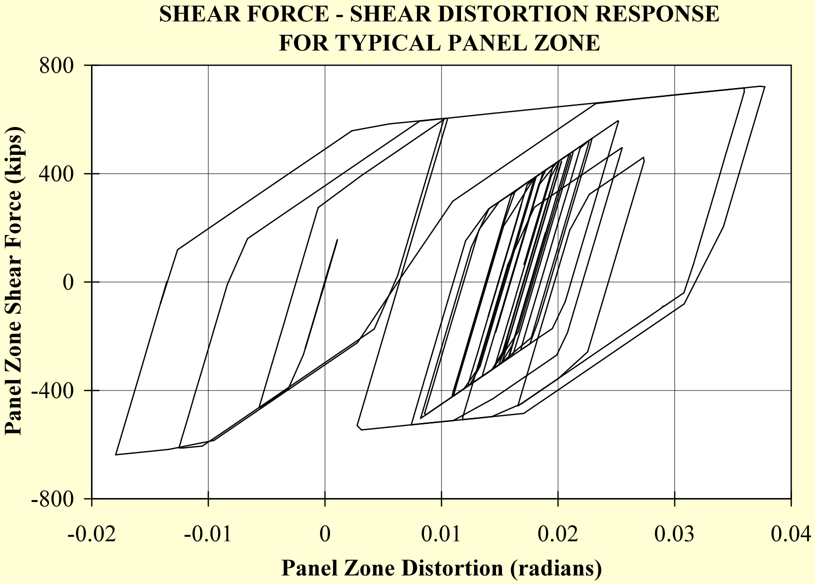


Analytical Modeling of Structure



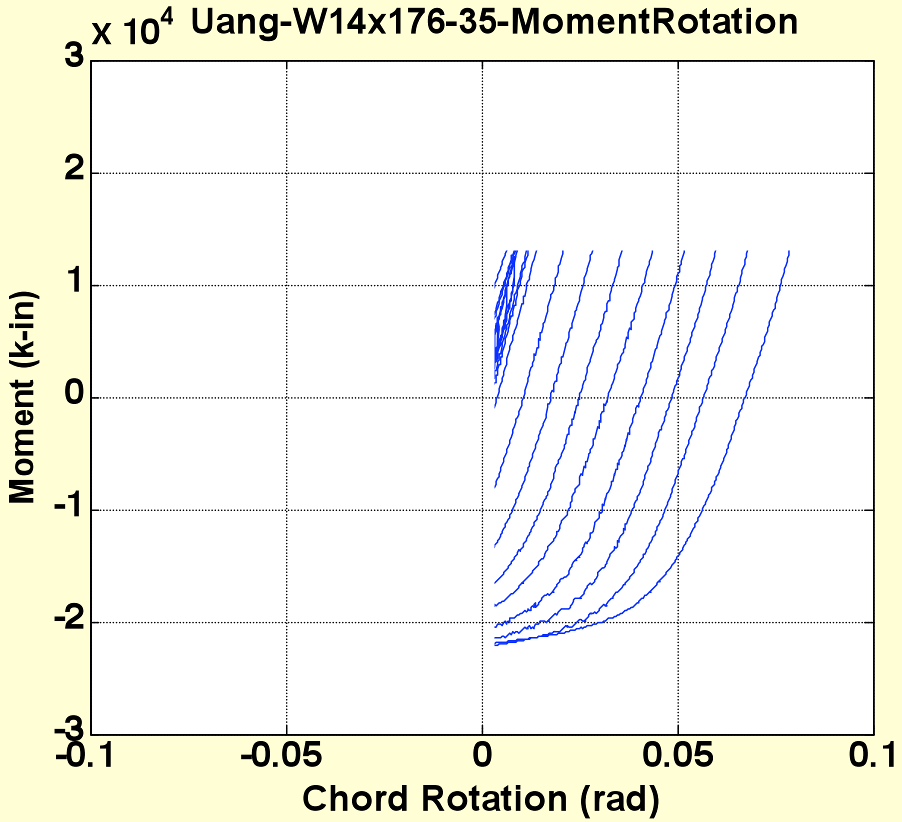
Panel Zone Modeling





Modeling of Plastic Hinging in Columns (Axial Load Effects)

- Heavy W14 Columns are very ductile, even if large axial loads are present



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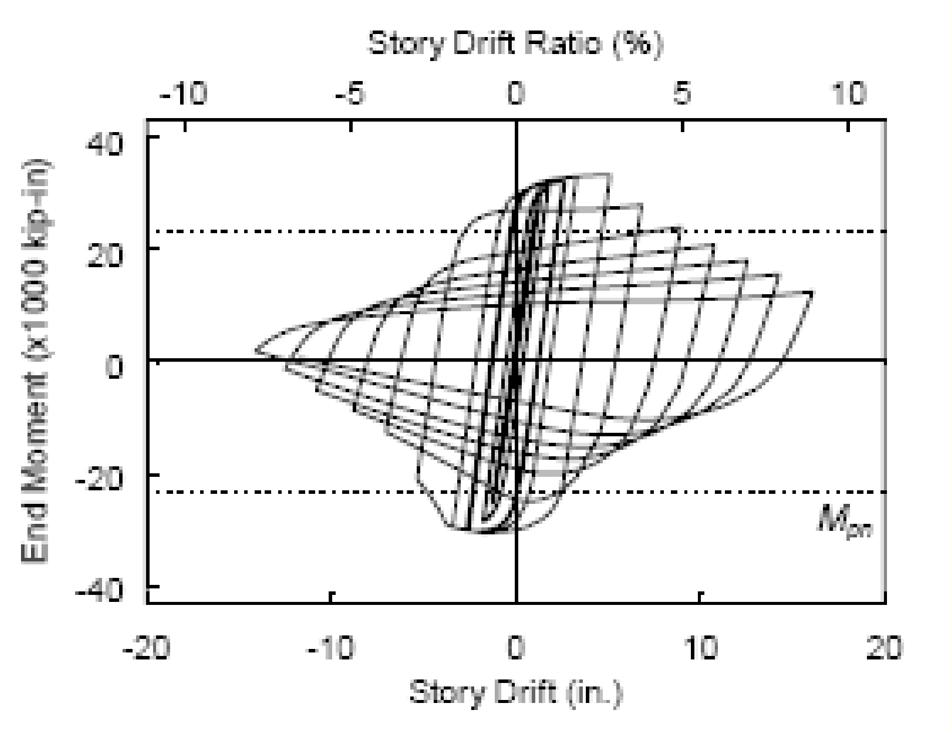
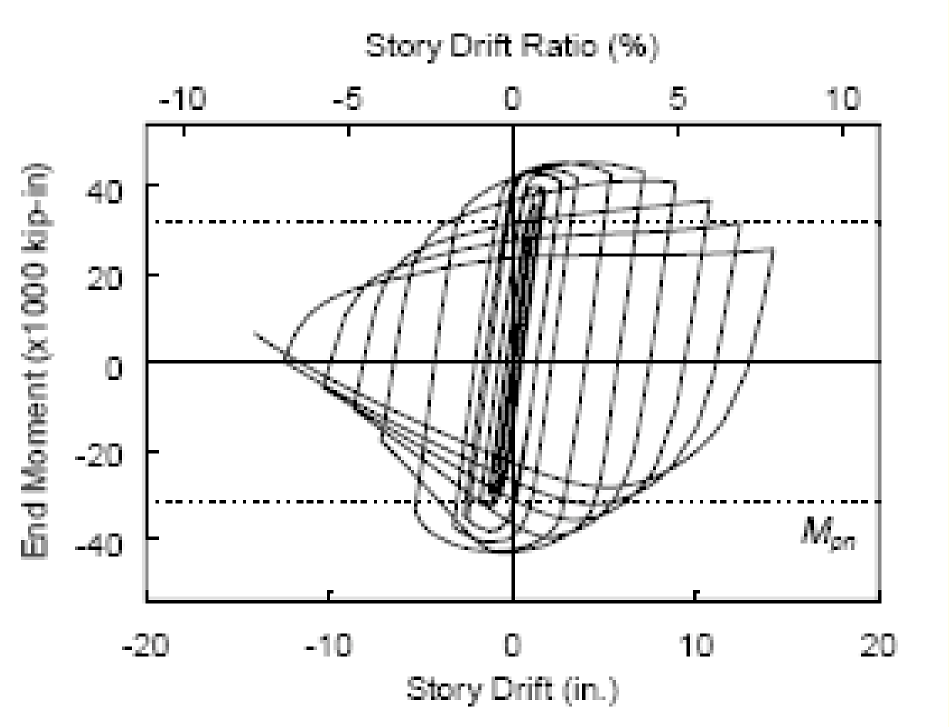
Modeling of Plastic Hinging in Columns (Axial Load Effects)

- Deep Column are not very ductile



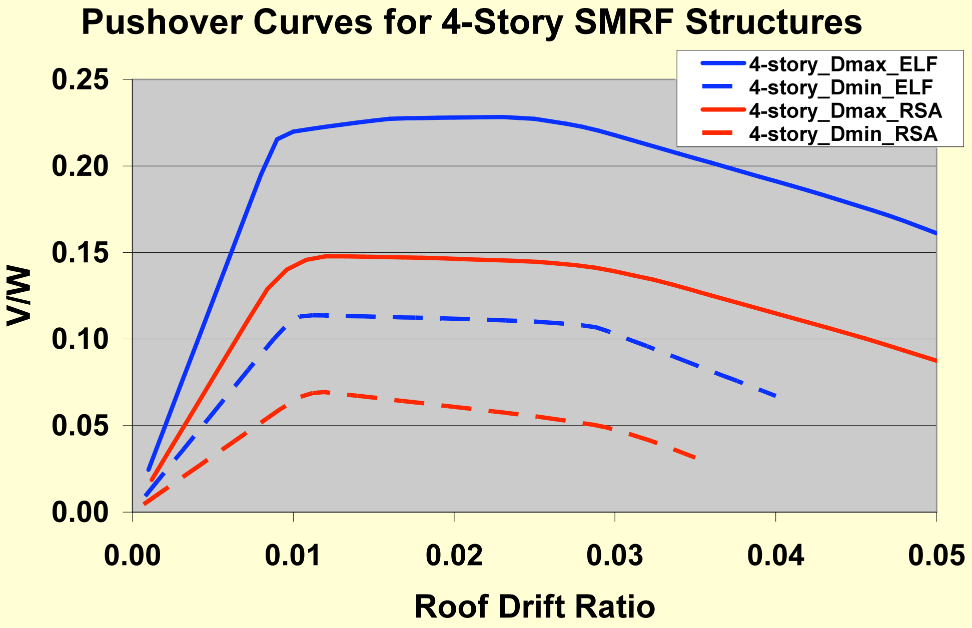
W27x146, P/Py = 0.35

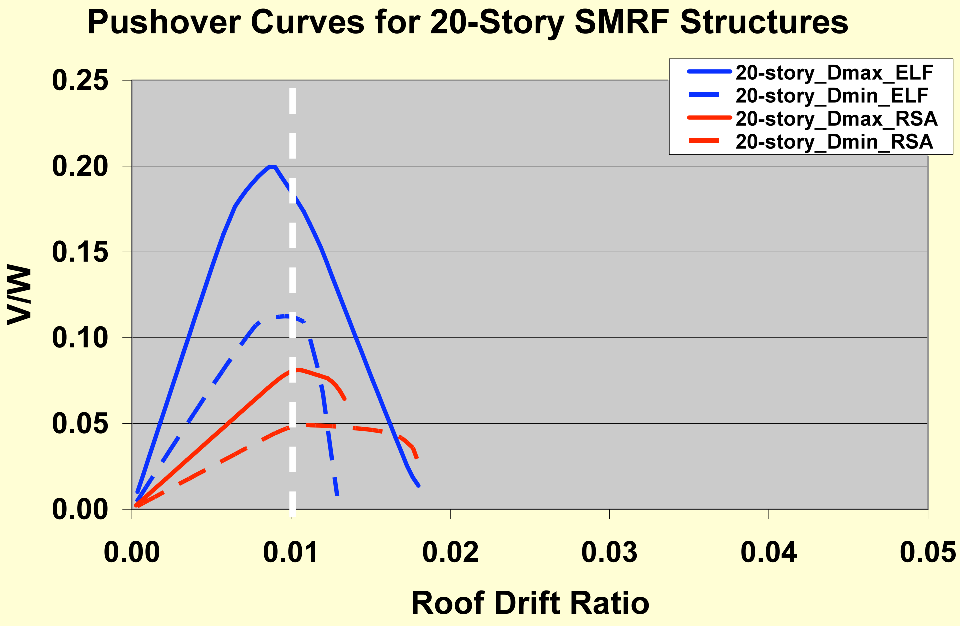


W27x194, P/Py = 0.55

From C.M. Uang

Pushovers





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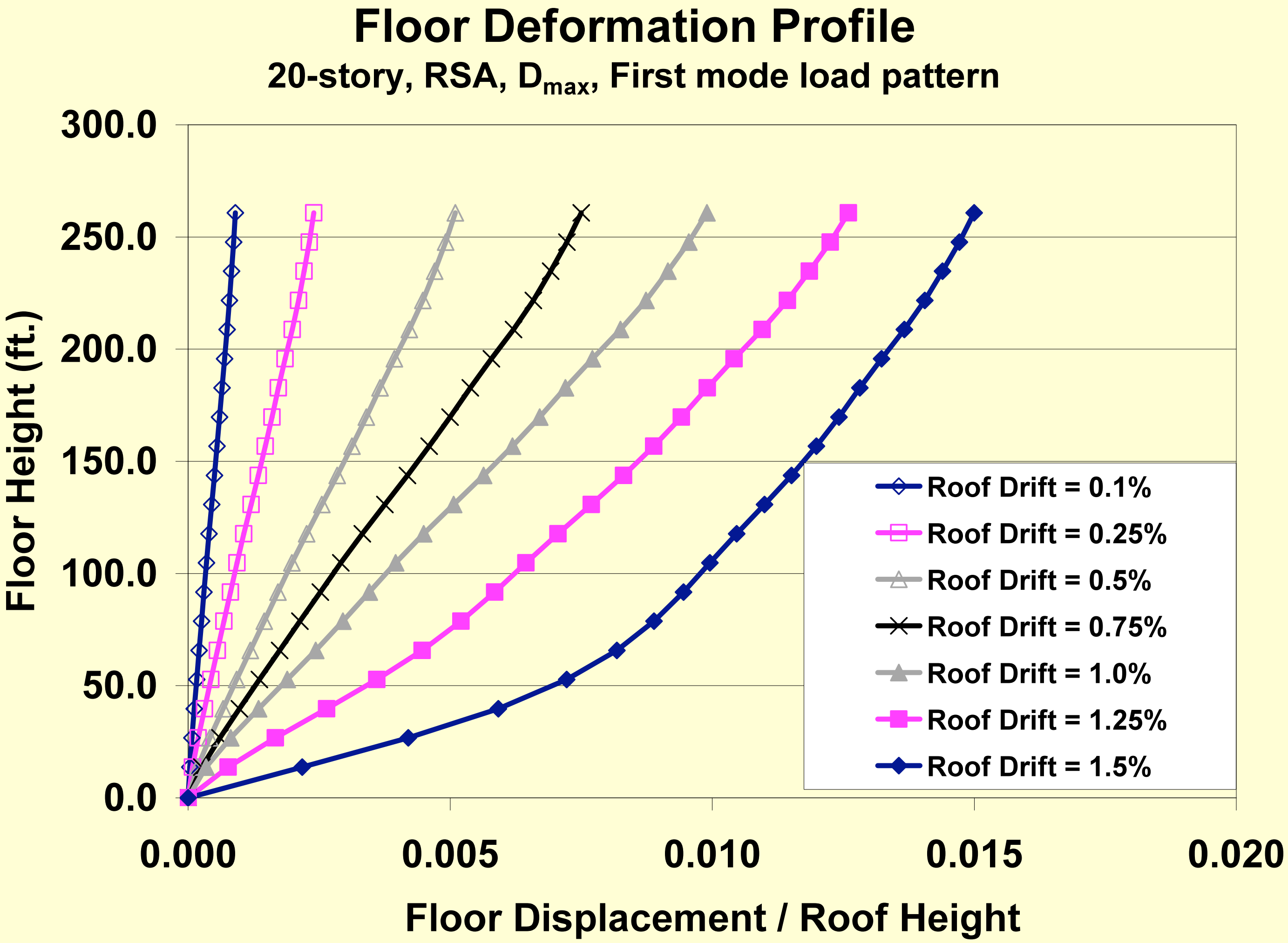
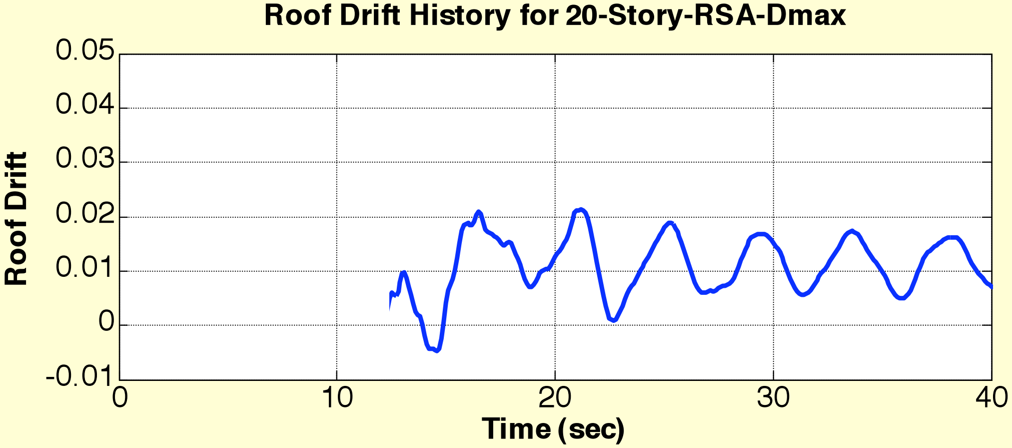
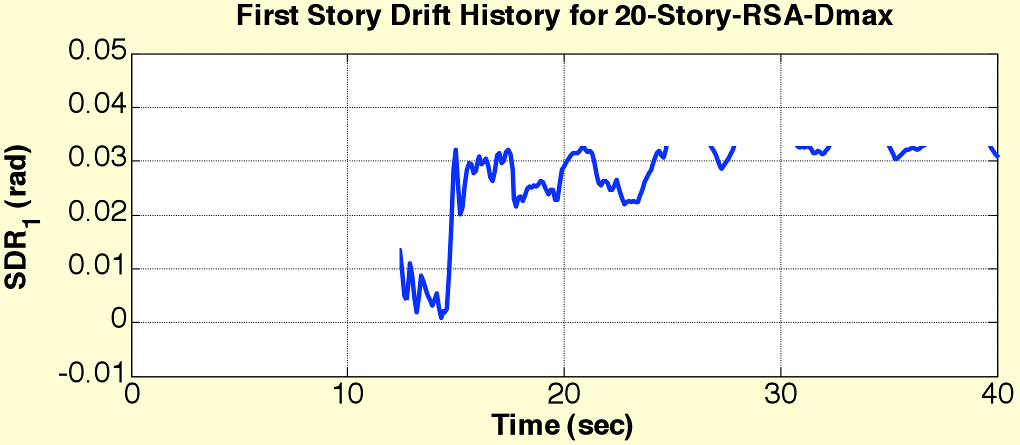
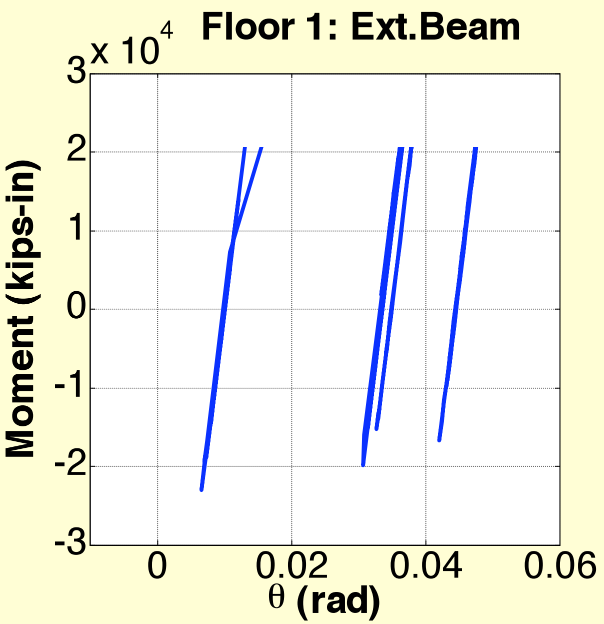
P-Delta Effect

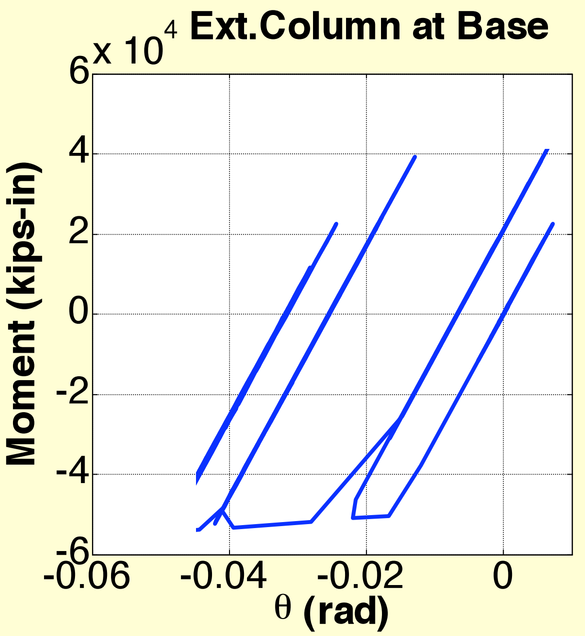


Nonlinear Response History Analysis


Roof Drift History for 20-Story-RSA-Dmax


First Story Drift History for 20-Story-RSA-Dmax


Floor 1: Ext.Beam


Ext.Column at Base

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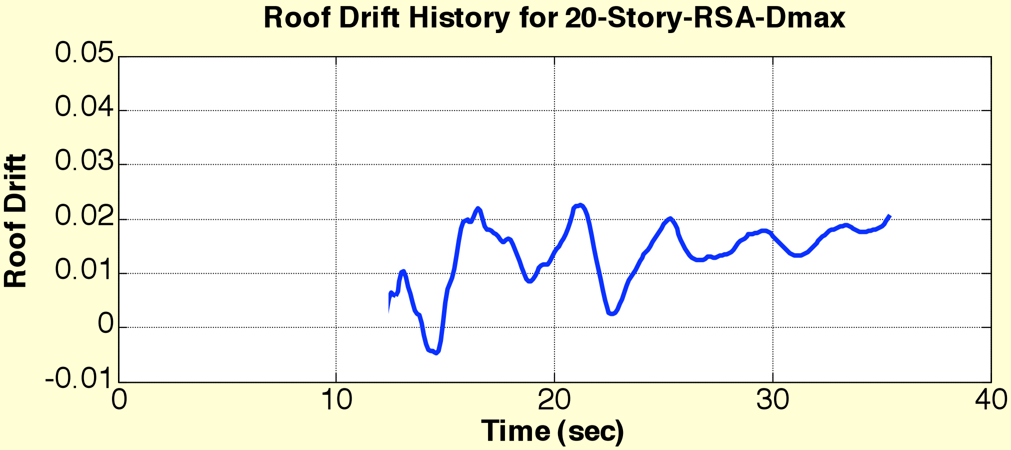
Nonlinear Response History Analysis



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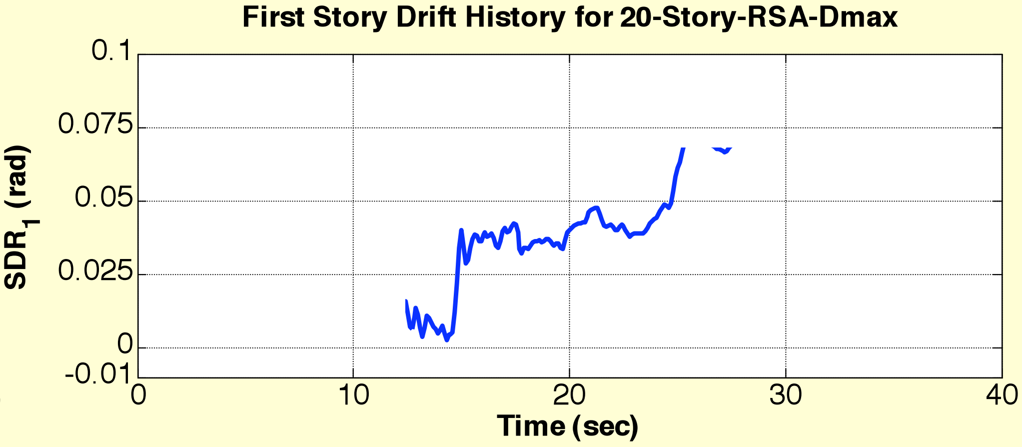
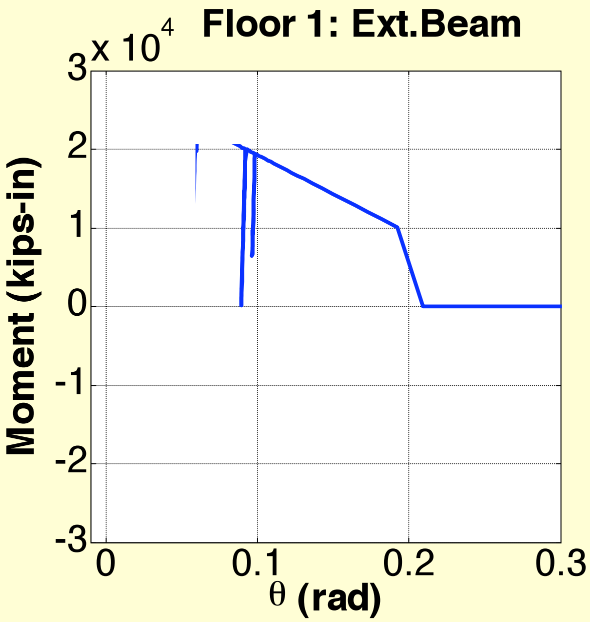
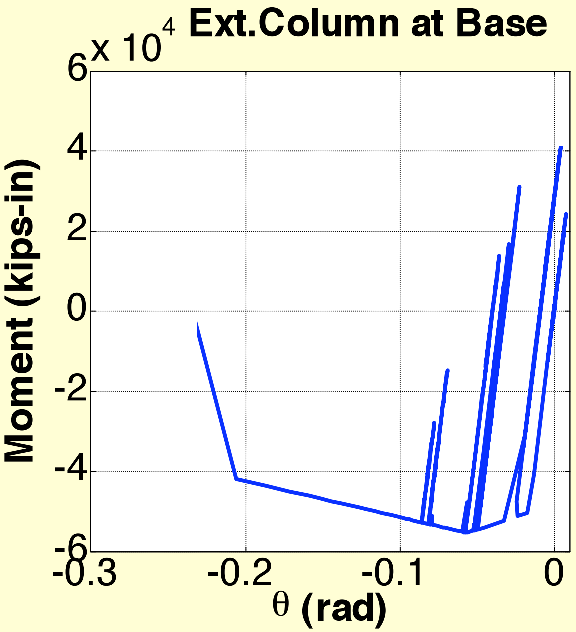


First Story Drift History for 20-Story-RSA-Dmax



Floor 1: Ext.Beam



Ext.Column at Base

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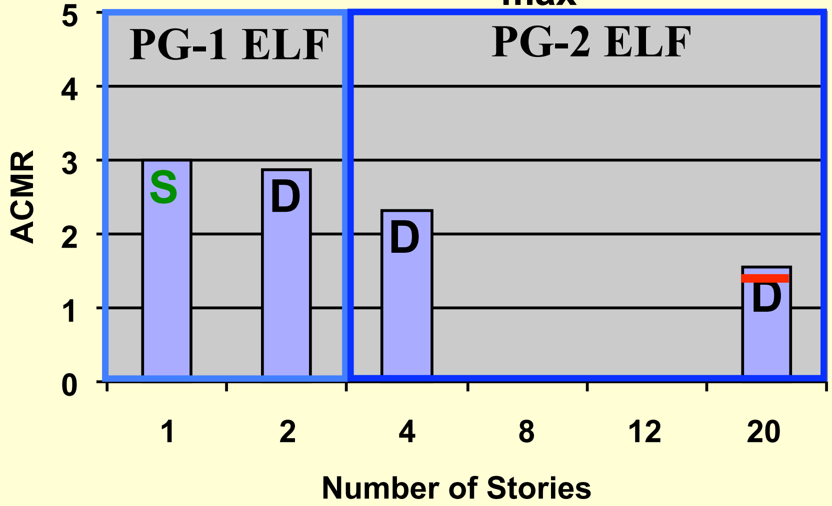
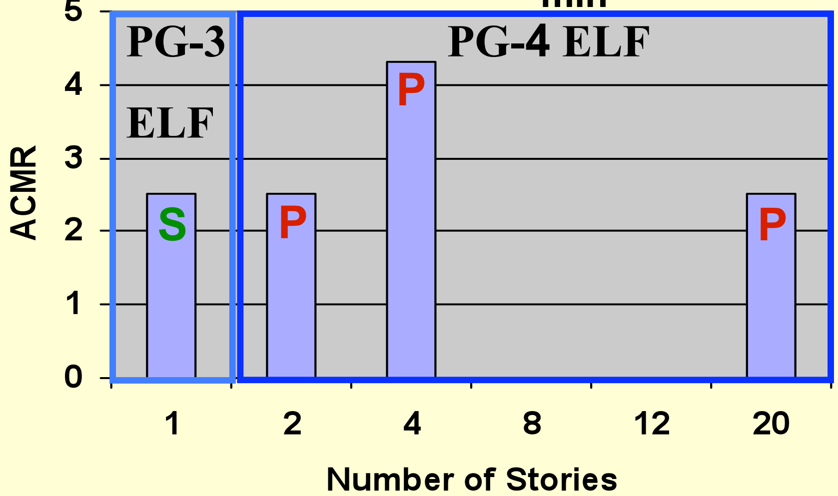
ACMR – From Nonlinear Response History Analysis – 44 Records

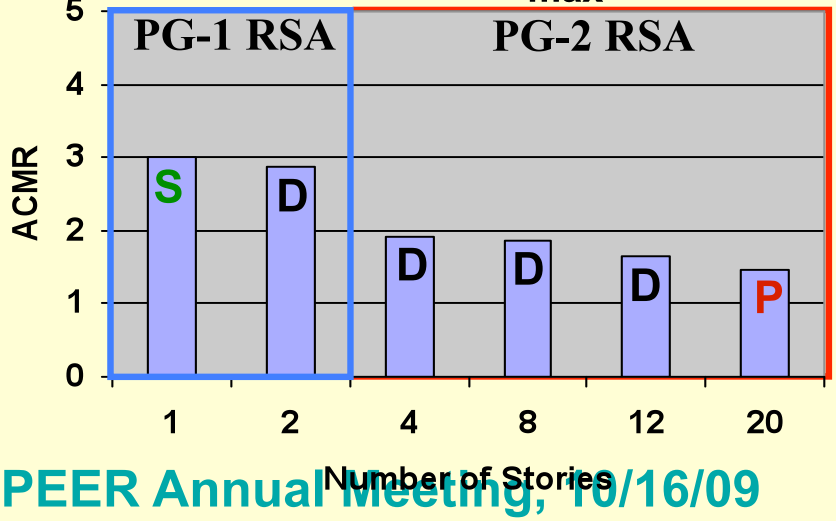
ELF - D_{max}



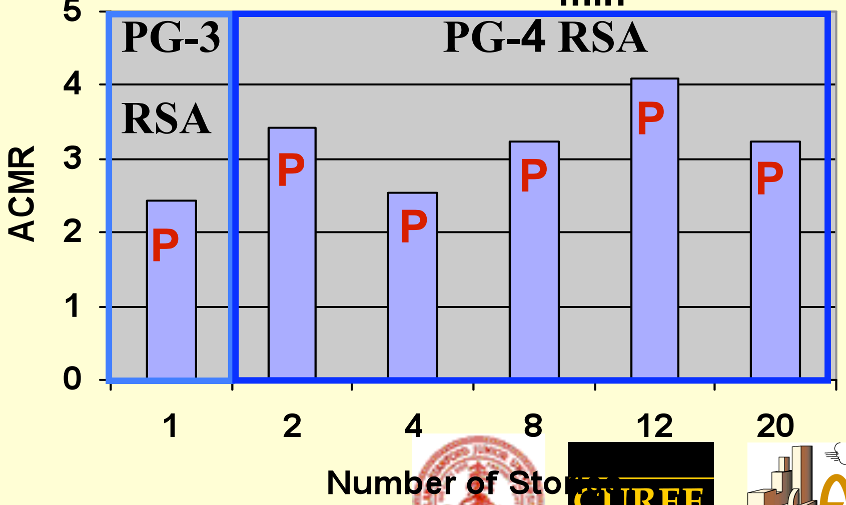
ELF - D_{min}



RSA - D_{max}



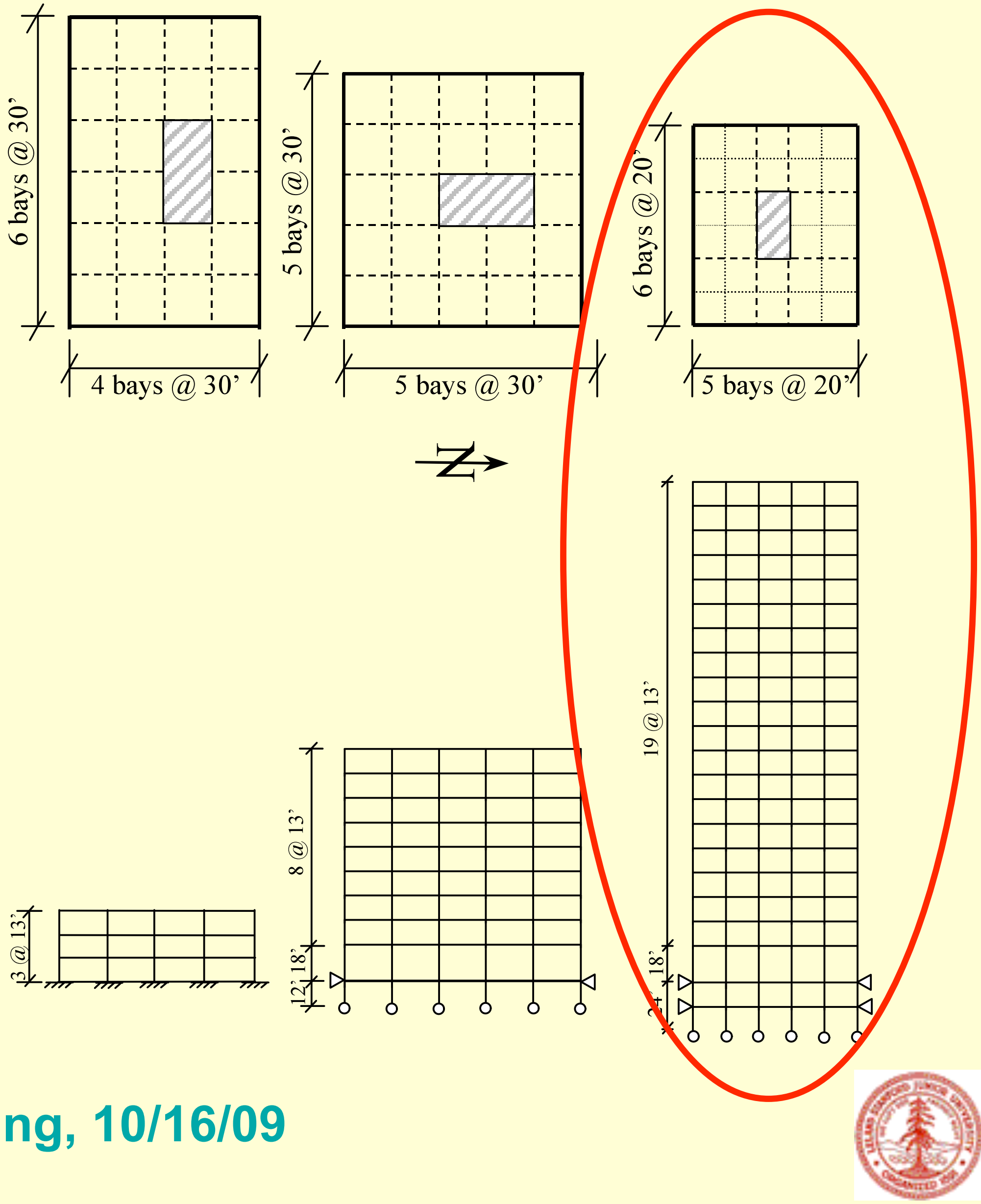
RSA - D_{min}



Are Response Predictions for Tall Frame Structures Realistic?

- Probably not because contribution of gravity system to lateral strength and stiffness has been ignored

Remembering SAC 20-story Response Prediction



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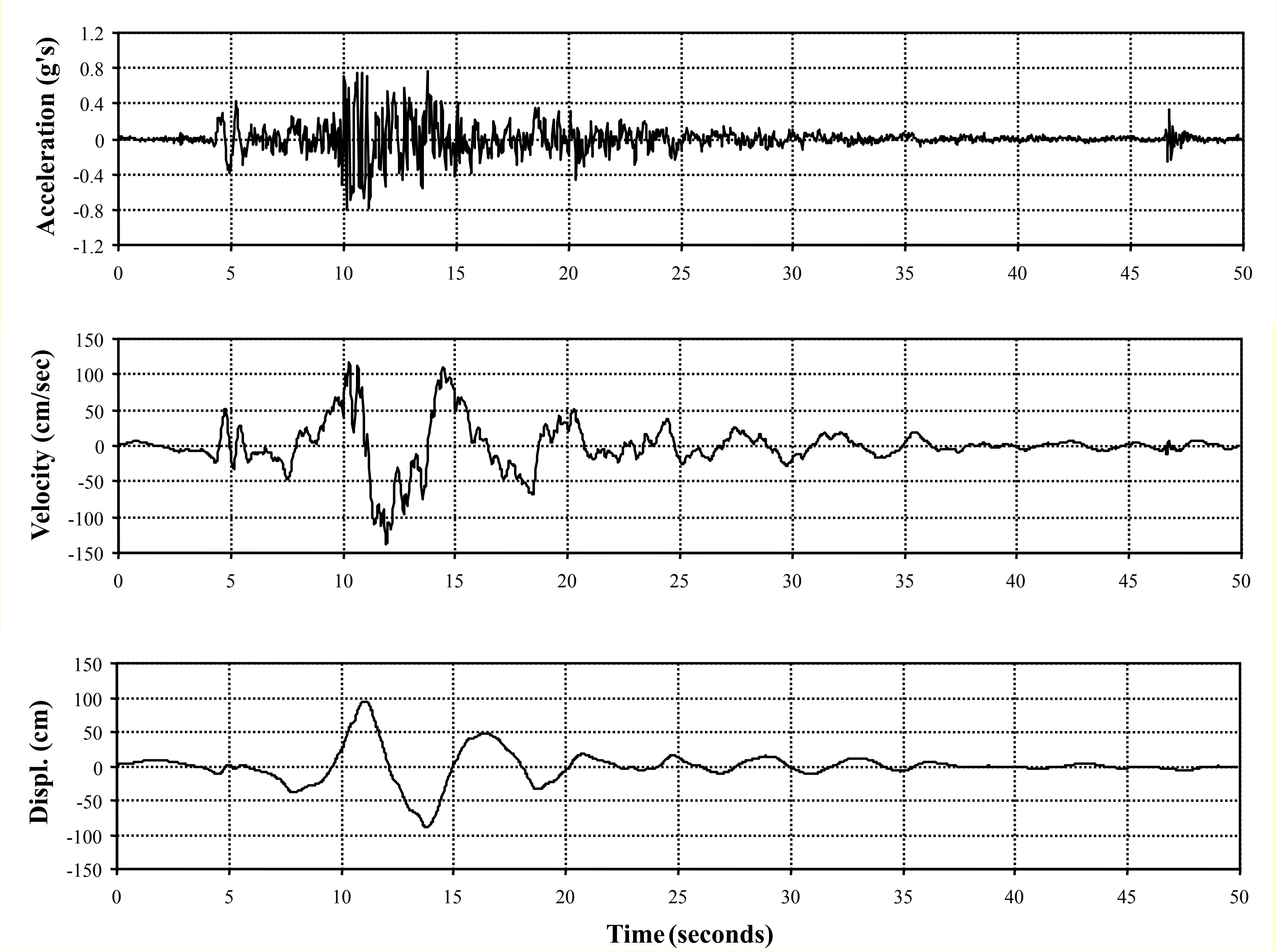
Tabas Record (Record LA30)



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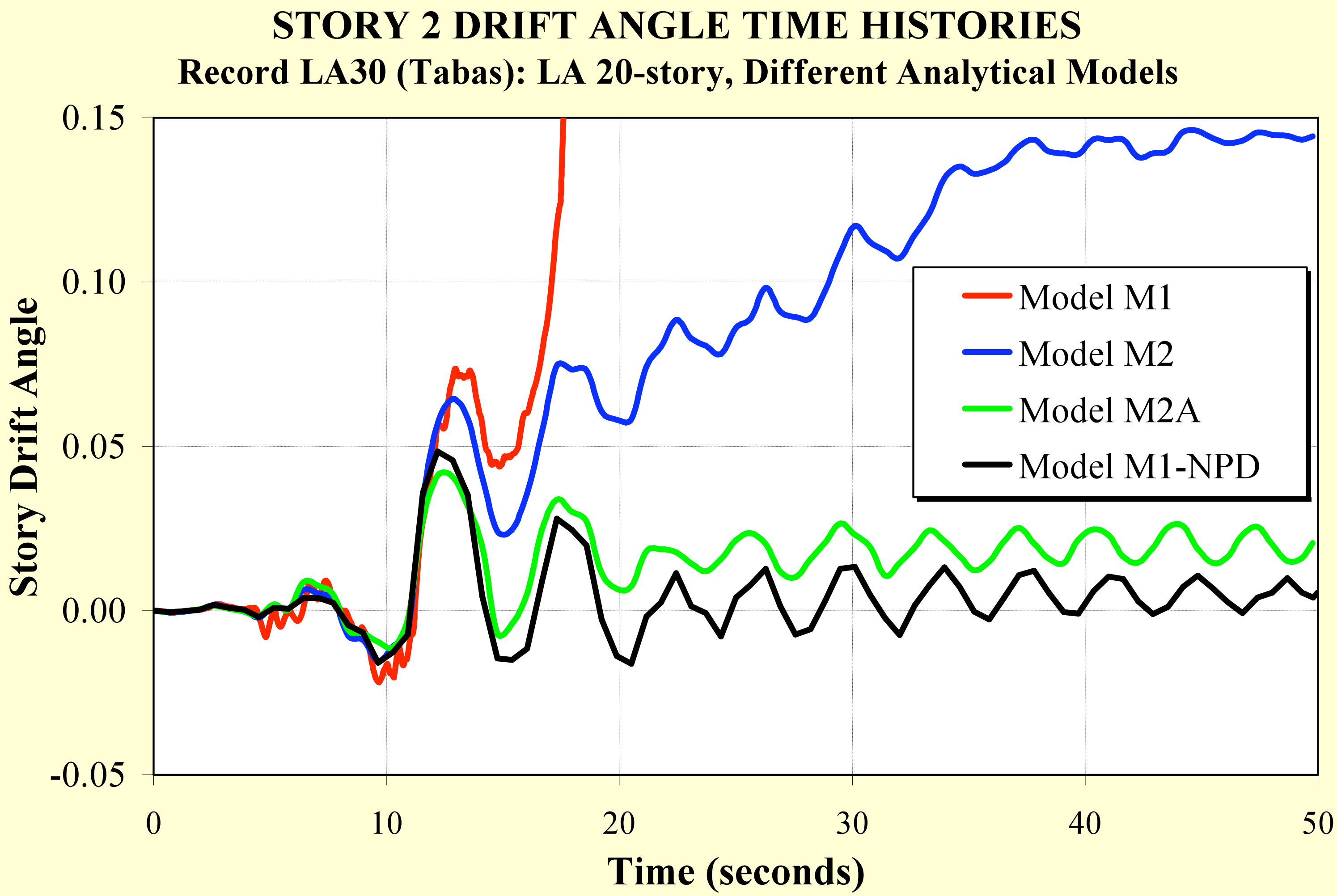
Story 2 Drift Response, LA-20, Various Models



M1 = centerline model

M2 = model with panel zones

M2A = including gravity system

Challenges in Collapse Prediction for Steel Moment Frame Structures

- More data is needed to model plastic hinging in columns in the presence of a large axial load
- There are several missing contributions to lateral strength and stiffness that might improve collapse performance
  - Gravity system (columns and “simple” connections”)
  - Composite slab action
  - Non-tangible contributions (stair cases, exterior cladding)