Stability Analysis and Design of Steel-Concrete Composite Columns

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Direct Analysis Method

Calculation of Required Strengths

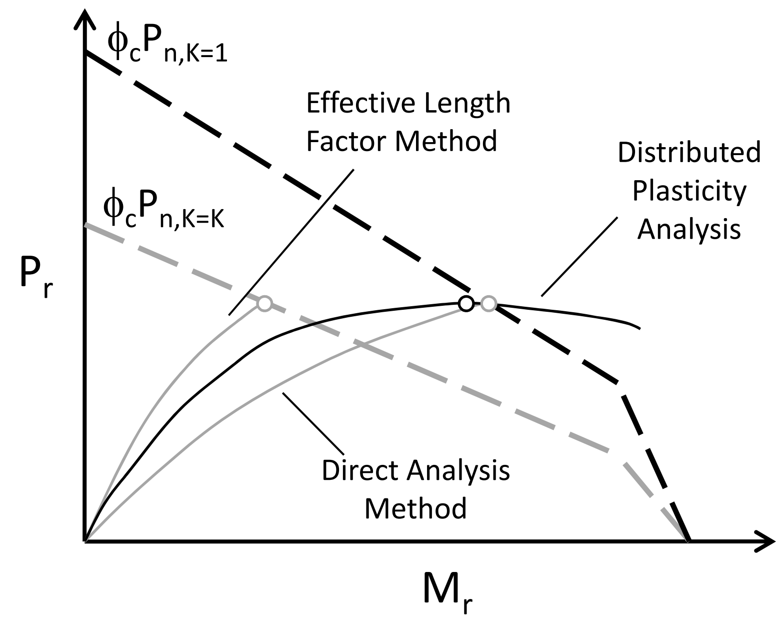
- Analysis Requirements
  Second-Order Elastic Analysis
- Consideration of Initial Imperfections

$$N_i = 0.002Y_i$$

- Adjustments to Stiffness

$$EI_{DA} = 0.8\tau_b EI_{elastic}$$

$$EA_{DA} = 0.8EA_{elastic}$$



Calculation of Available Strengths

- Chapters D though K without further consideration of overall structure stability

$$K = 1$$

Elastic Flexural Rigidity

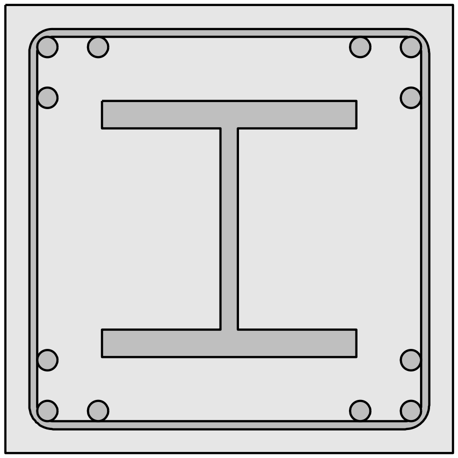
- ***EI_{eff}*** – used to determine the axial compressive strength of columns
- ***$EI_{elastic}$*** – used in a 1st or 2nd order static, dynamic, or eigenvalue analysis
  - in conjunction with Direct Analyses stiffness reductions to perform strength checks
  - to compute story drifts used in interstory drift checks
  - to compute fundamental periods and mode shapes (including for response spectrum analysis)
  - as the elastic component of a concentrated plasticity beam-column element
- ***EI_{DA}*** – used in the Direct Analysis method

For Structural Steel: $EI_{eff} = EI_{elastic} = E_s I_s$

EI_{eff} for Composite Column

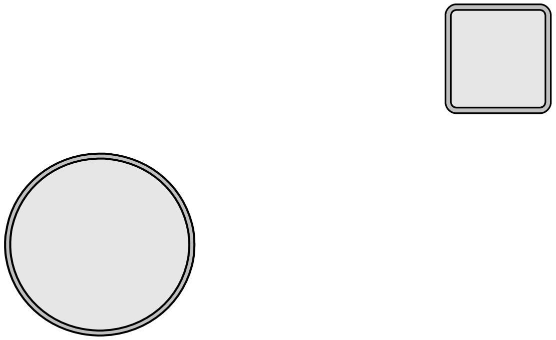
$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \qquad \text{(SRC)}$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3$$

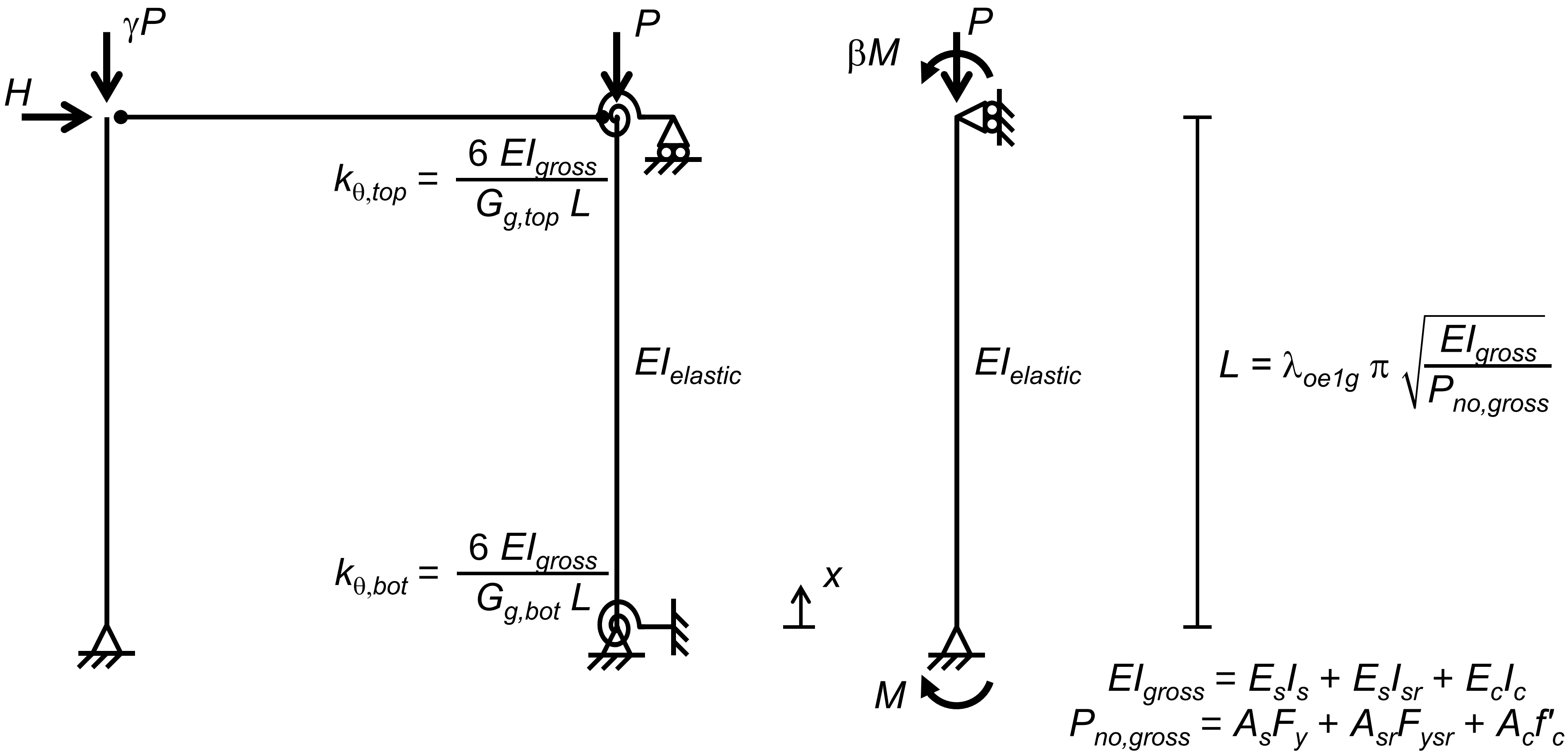


$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \qquad \text{(CFT)}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9$$



Benchmark Frame Schematic



Initial Imperfections:

Out-of-plumbness $\Delta_o = L/500$

Out-of-straightness $\delta_o = L/1000$ (sinusoidal)

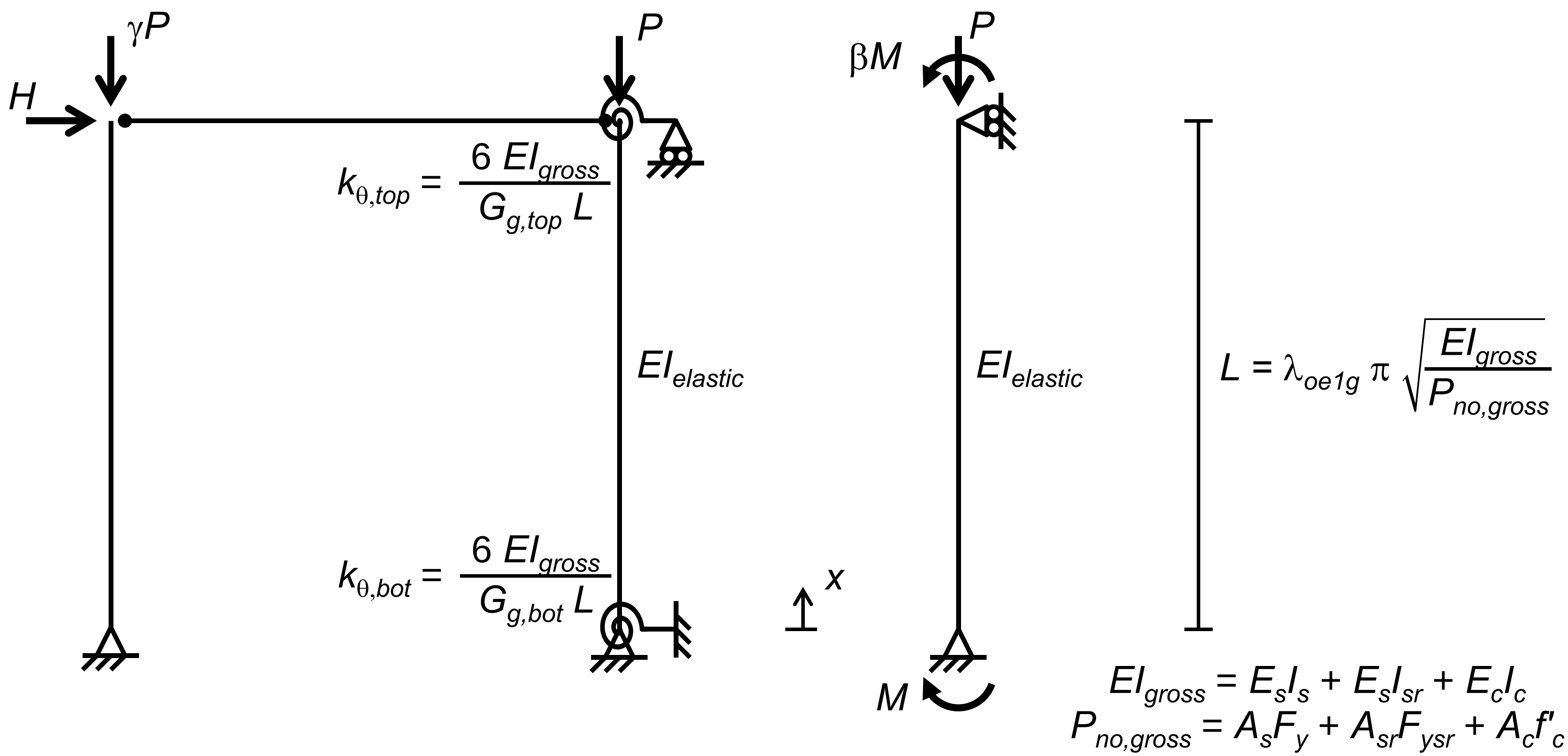
Parametric Variations of the Benchmark Frames

Frame	Slenderness	End Restraint	Leaning Column Load Ratio	End Moment Ratio	Number of Frames
Sidesway Uninhibited	4 values λ_{oe1g} = {0.22, 0.45, 0.67, 0.90}	4 value pairs (Table Below)	4 values γ = {0, 1, 2, 3}	n/a	64 (= 4 × 4 × 4)
Sidesway Inhibited	4 values λ_{oe1g} = {0.45, 0.90, 1.35, 1.90}	n/a	n/a	5 values β = {–1.0, –0.5, 0.0, 0.5, 1.0}	20 (= 4 × 5)

Pair	$G_{g,top}$	$G_{g,bot}$
A	0	0
B	1 or 3*	1 or 3*
C	0	∞
D	1 or 3*	∞

*3 when γ = 0; 1 otherwise

Benchmark Frame Schematic



Initial Imperfections:

Out-of-plumbness $\Delta_o = L/500$

Out-of-straightness $\delta_o = L/1000$ (sinusoidal)

Selected Sections

CCFT

Index	D	t	ρ_s
A	7	0.500	24.82%
B	10	0.500	17.70%
C	12.75	0.375	10.65%
D	16	0.250	5.72%
E	24	0.125	1.93%

F_y = 42 ksi; f'_c = 4, 8, 16 ksi

RCFT

Index	H	B	t	ρ_s
A	6	6	1/2	27.63%
B	9	9	1/2	19.06%
C	8	8	1/4	11.13%
D	9	9	1/8	5.05%
E	14	14	1/8	3.27%

F_y = 46 ksi; f'_c = 4, 8, 16 ksi

SRC

Index	Steel Shape	ρ_s
A	W14x311	11.66%
B	W14x233	8.74%
C	W12x120	4.49%
D	W8x31	1.16%

Index	Rebar	ρ_{sr}
A	20 #11	3.98%
B	12 #10	1.94%
C	4 #8	0.40%

Gross dimensions of all SRC sections = 28″ x 28″

F_y = 50 ksi; F_{yr} = 60 ksi; ; f'_c = 4, 8, 16 ksi

Analysis of the Benchmark Frames

- Second-Order Elastic Analysis
  - Closed form solution of the governing differential equation using the appropriate boundary conditions

$$v''''(x)+\sqrt{\frac{P}{EI_{elastic}}}v''(x)=0$$

- Fully Nonlinear Analysis
  - Distributed Plasticity Mixed Beam Finite Element Formulation
  - Implemented in the OpenSees framework
  - Extensive validation to experimental results

Steel Material Models for SRC and CFT Members

- For SRC: Steel Shape (WF)
  - Elastic-perfectly plastic
  - Lehigh residual stress pattern ($f_c = 0.3F_y$)
- For SRC: Steel Reinforcement
  - Elastic-perfectly plastic
  - No residual stress
- For CFT: Steel Shape (HSS)
  - Multi-linear model by Abdel-Rahman and Sivakumaran (1997)
  - Residual stress accounted for in the shape of the curve
  - Sections with high D/t ratios are included in the study, but local buckling is not modeled, noting that the AISC strength reductions for local buckling will still apply.

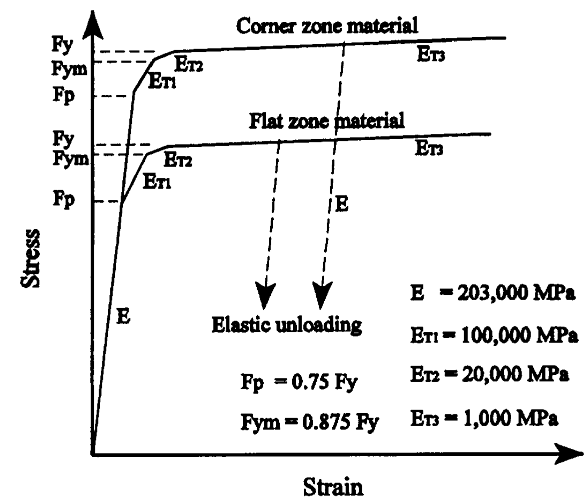


FIG. 5. Idealized Stress-Strain Relationship for Cold-Formed Steel

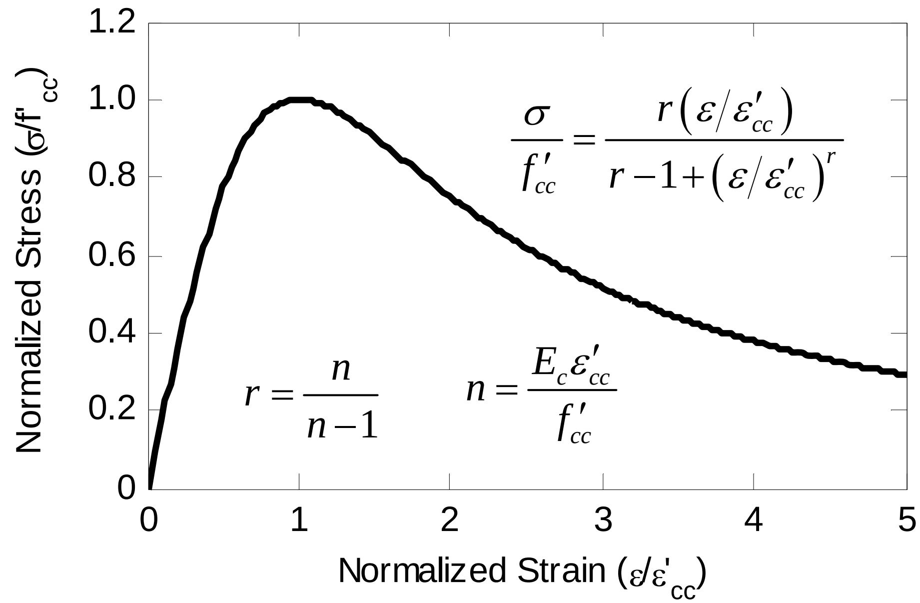
from: Abdel-Rahman and Sivakumaran 1997

Concrete Material Models for SRC and CFT Members

- Popovics Concrete Model
  - Three parameters define the stress-strain response
- Peak stress
  - Taken as f'_c or greater to account for confinement
- Strain at peak stress
- Initial stiffness

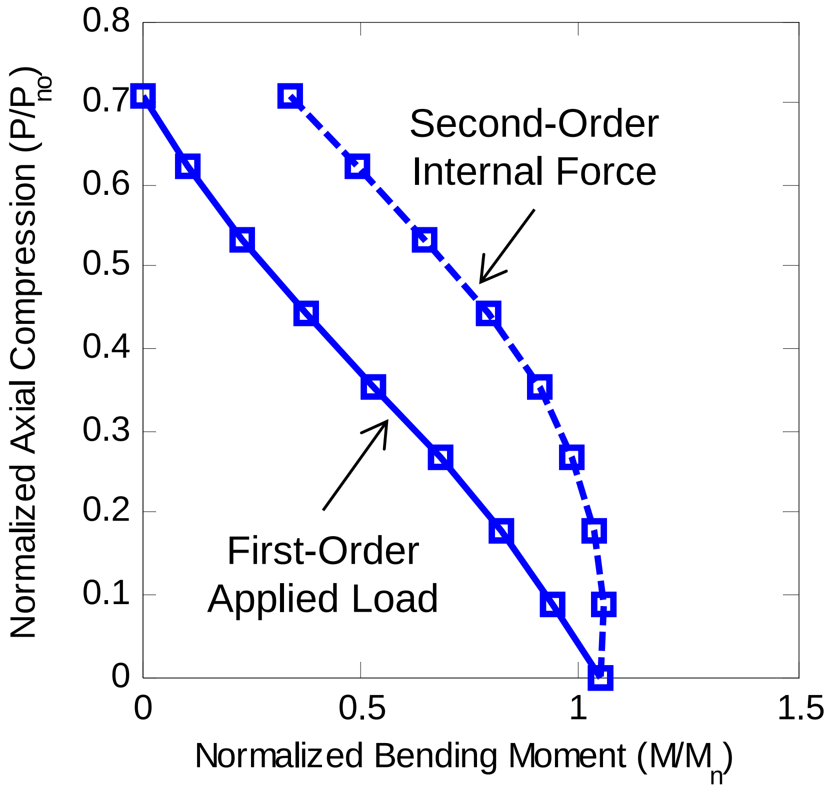
$$E_c\left[\text{ksi}\right] = 1802\sqrt{f'_c\left[\text{ksi}\right]}$$

(corresponds to w_c = 148.1 pcf using the AISC formula)

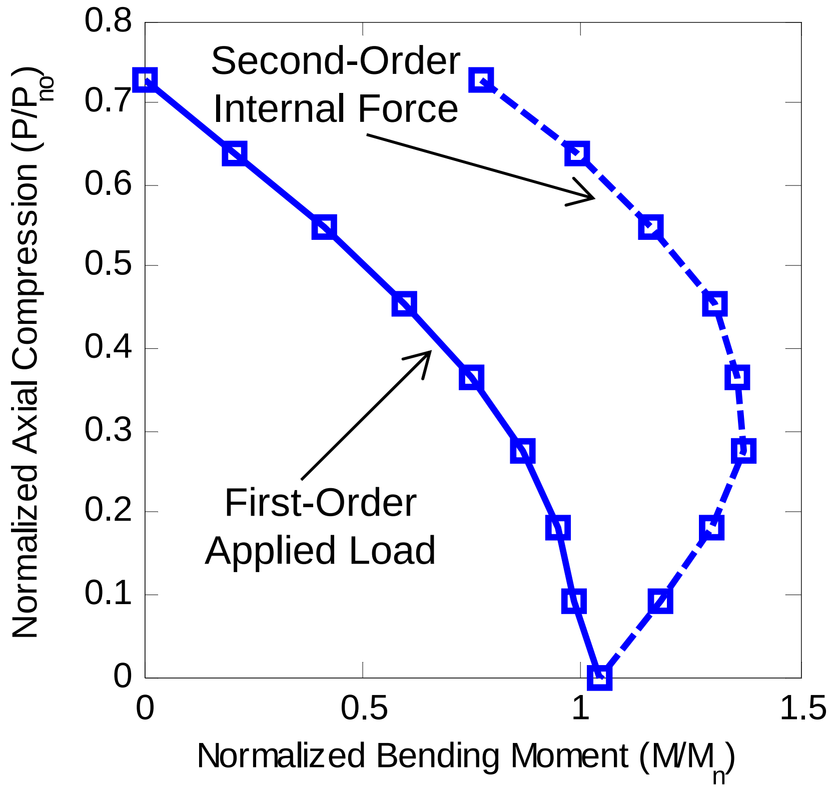


$$\varepsilon_c = \frac{f'_c\left[\text{ksi}\right]^{1/4}}{710} \qquad \varepsilon_{cc} = \varepsilon_c\left(1+5\left(\frac{f'_{cc}}{f'_c}-1\right)\right)$$

Example Results: Fully Nonlinear Applied Load and Internal Force Interaction Diagrams

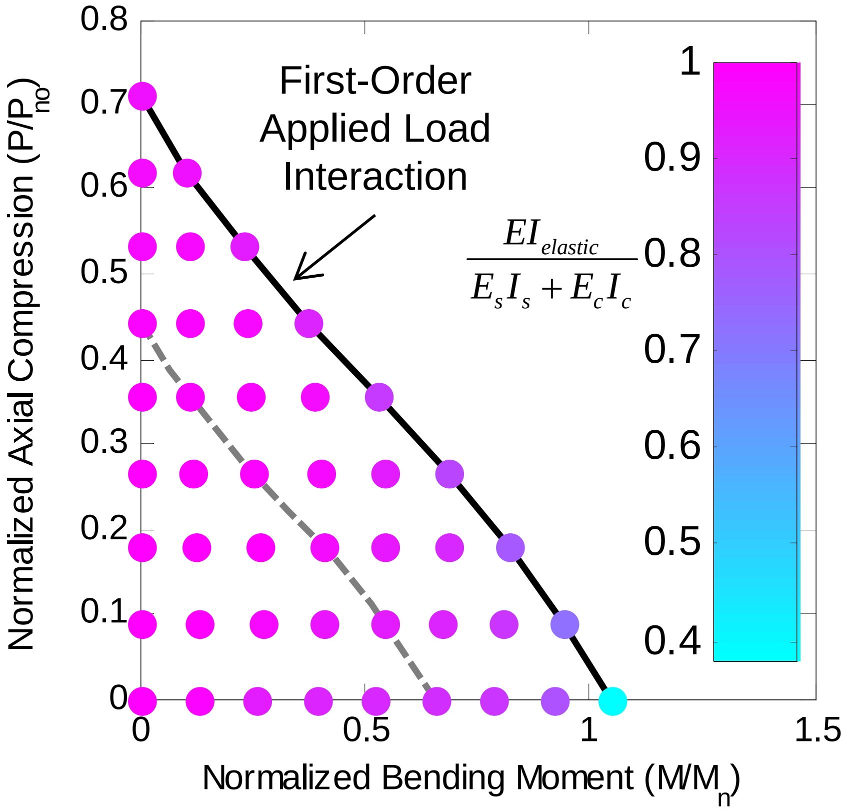


Section 4: RCFT-B-4, Frame 37: UA-67-g1

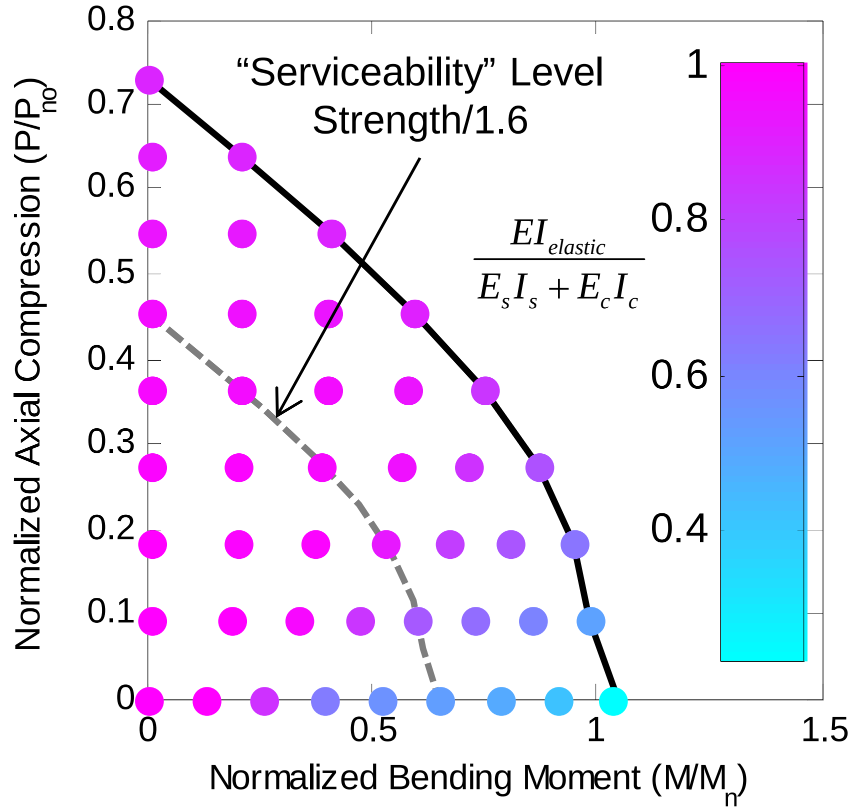


Section 13: RCFT-E-4, Frame 37: UA-67-g1

Example Results: $EI_{elastic}$



Section 4: RCFT-B-4, Frame 37: UA-67-g1



Section 13: RCFT-E-4, Frame 37: UA-67-g1

Expression for $EI_{elastic}$

- Linear regression of data obtained in $EI_{elastic}$ study

$$EI_{elastic} = E_s I_s + C_4 E_c I_c \qquad \text{(RCFT)}$$

$$C_4 = 1.01 - 0.90 \frac{M}{M_n}\left(1 - 3.39 \frac{P}{P_{no}}\right) \leq 1.00$$

10.10.4.1 — It shall be permitted to use the following properties for the members in the structure:

(a) Modulus of elasticity	E_c from 8.5.1
(b) Moments of inertia, I	
Compression members:	
Columns	$0.70I_g$
Walls—Uncracked	$0.70I_g$
—Cracked	$0.35I_g$
Flexural members:	
Beams	$0.35I_g$
Flat plates and flat slabs	$0.25I_g$
(c) Area	$1.0A_g$

Alternatively, the moments of inertia of compression and flexural members, I, shall be permitted to be computed as follows:

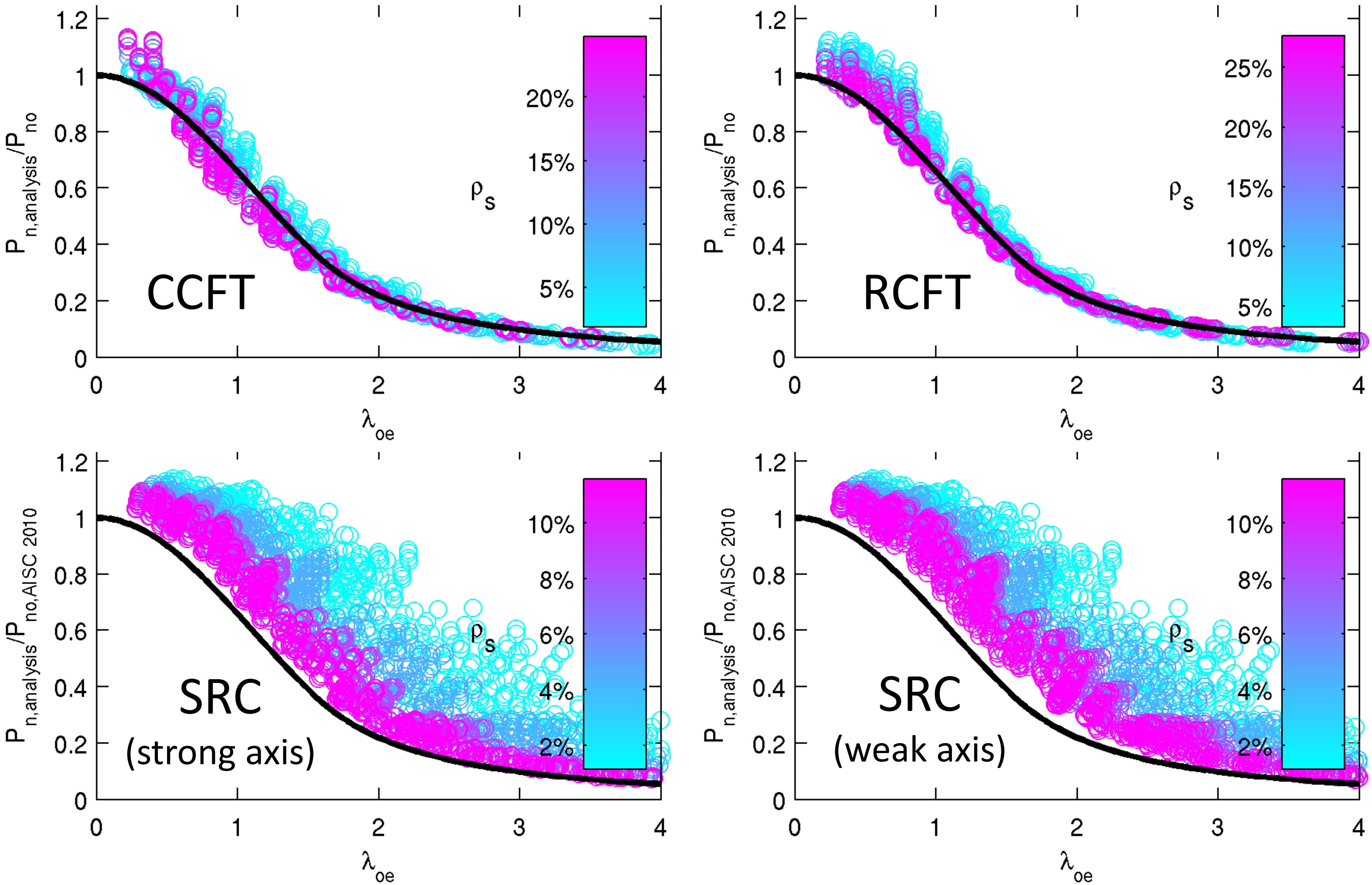
Compression members:

$$I = \left(0.80 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{P_o}\right)I_g \leq 0.875I_g \qquad (10\text{-}8)$$

where P_u and M_u shall be determined from the particular load combination under consideration, or the combination of P_u and M_u determined in the smallest value of I. I need not be taken less than $0.35I_g$.

from: ACI 318-08

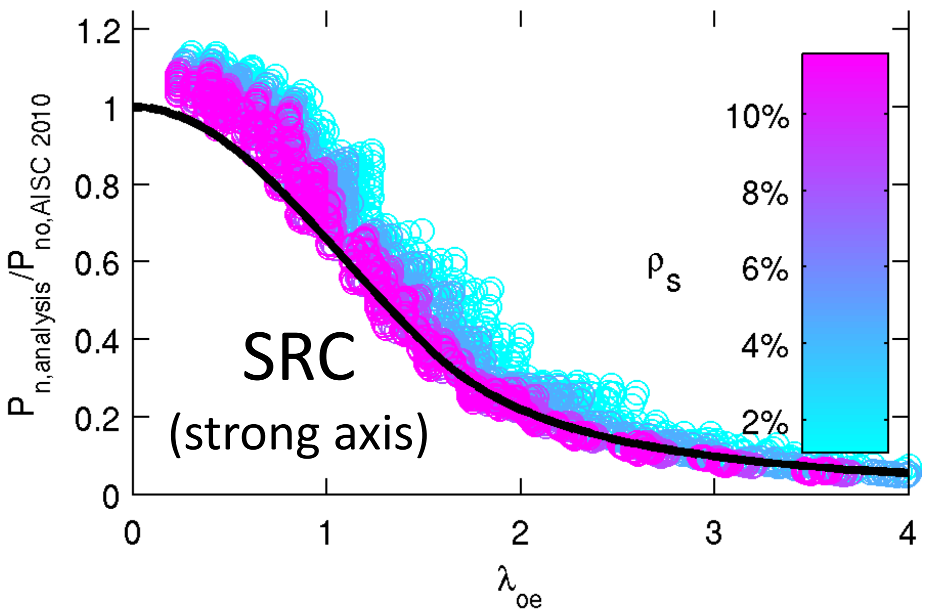
Axial Strength

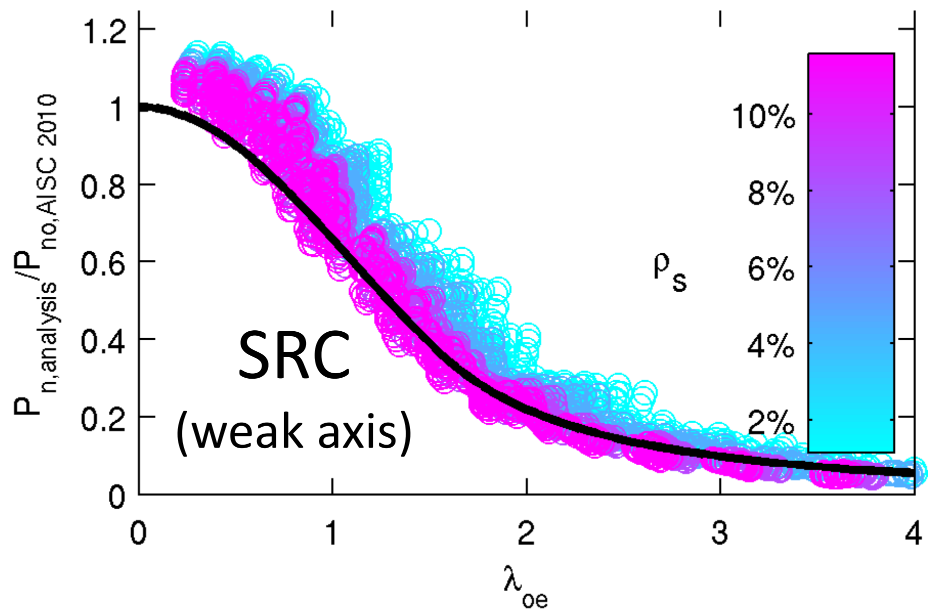


Axial Strength of SRC Columns

$$EI_{eff,proposed} = E_s I_s + E_s I_{sr} + C_{1,proposed} E_c I_c \qquad \text{(SRC)}$$

$$C_{1,proposed} = 0.60 + \frac{2A_s}{A_g} \leq 0.75$$





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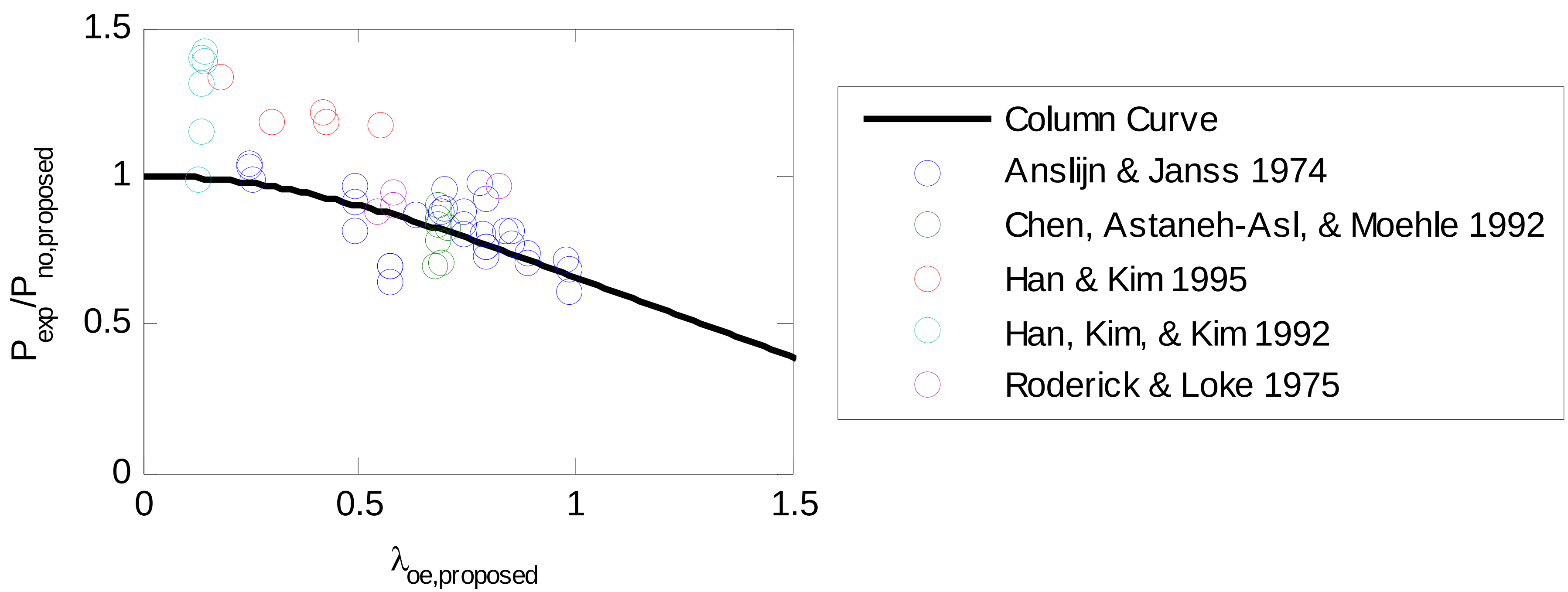


Direct Analysis

- From a practical standpoint it is best to maintain a stiffness reduction of $0.8\tau_b$

$$EI_{DA} = 0.8\tau_b EI_{elastic}$$

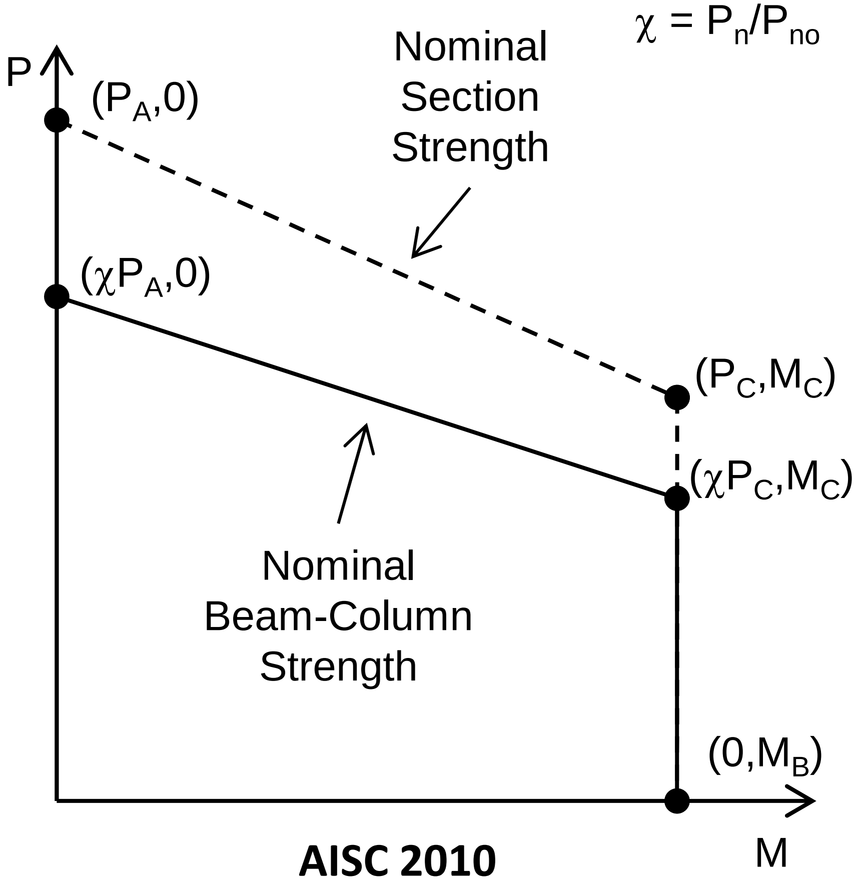
$$\tau_b = \begin{cases} 1.0 & \text{for } P_r/P_{no} \le 0.5 \\ 4\left(P_r/P_{no}\right)\left(1 - P_r/P_{no}\right) & \text{for } P_r/P_{no} > 0.5 \end{cases}$$

- Thus, differences between composite and steel are embodied in $EI_{elastic}$

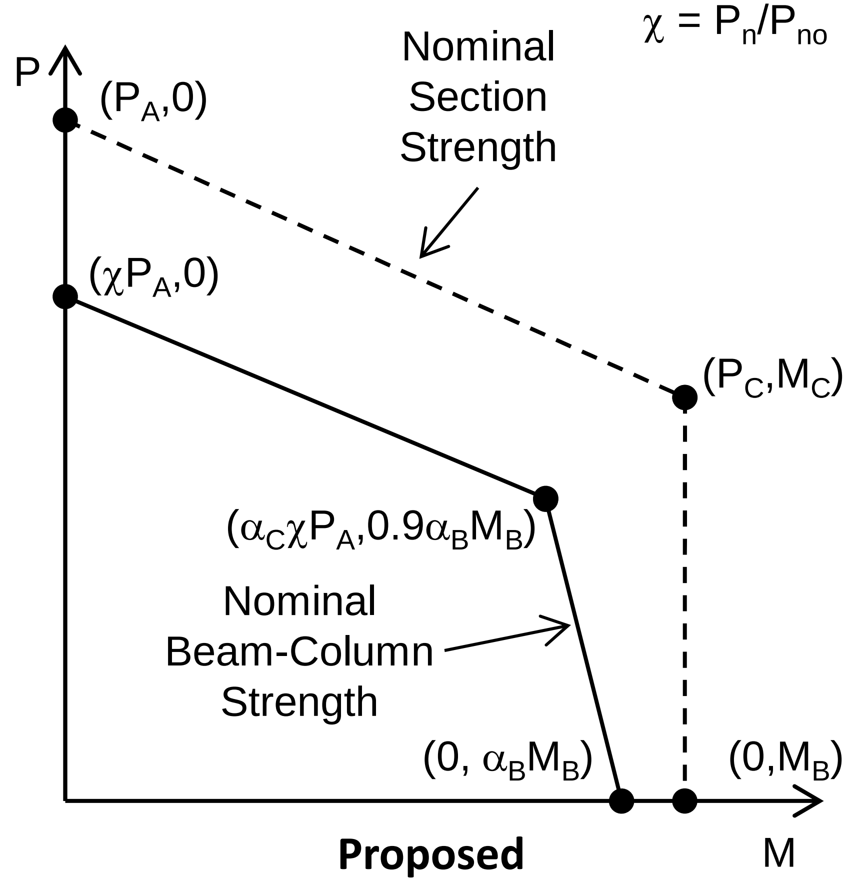
$$EI_{elastic} = E_s I_s + E_s I_{sr} + 0.75 C_1 E_c I_c \qquad \text{(SRC)}$$

$$EI_{elastic} = E_s I_s + 0.75 C_3 E_c I_c \qquad \text{(CFT)}$$

Interaction Strength



AISC 2010

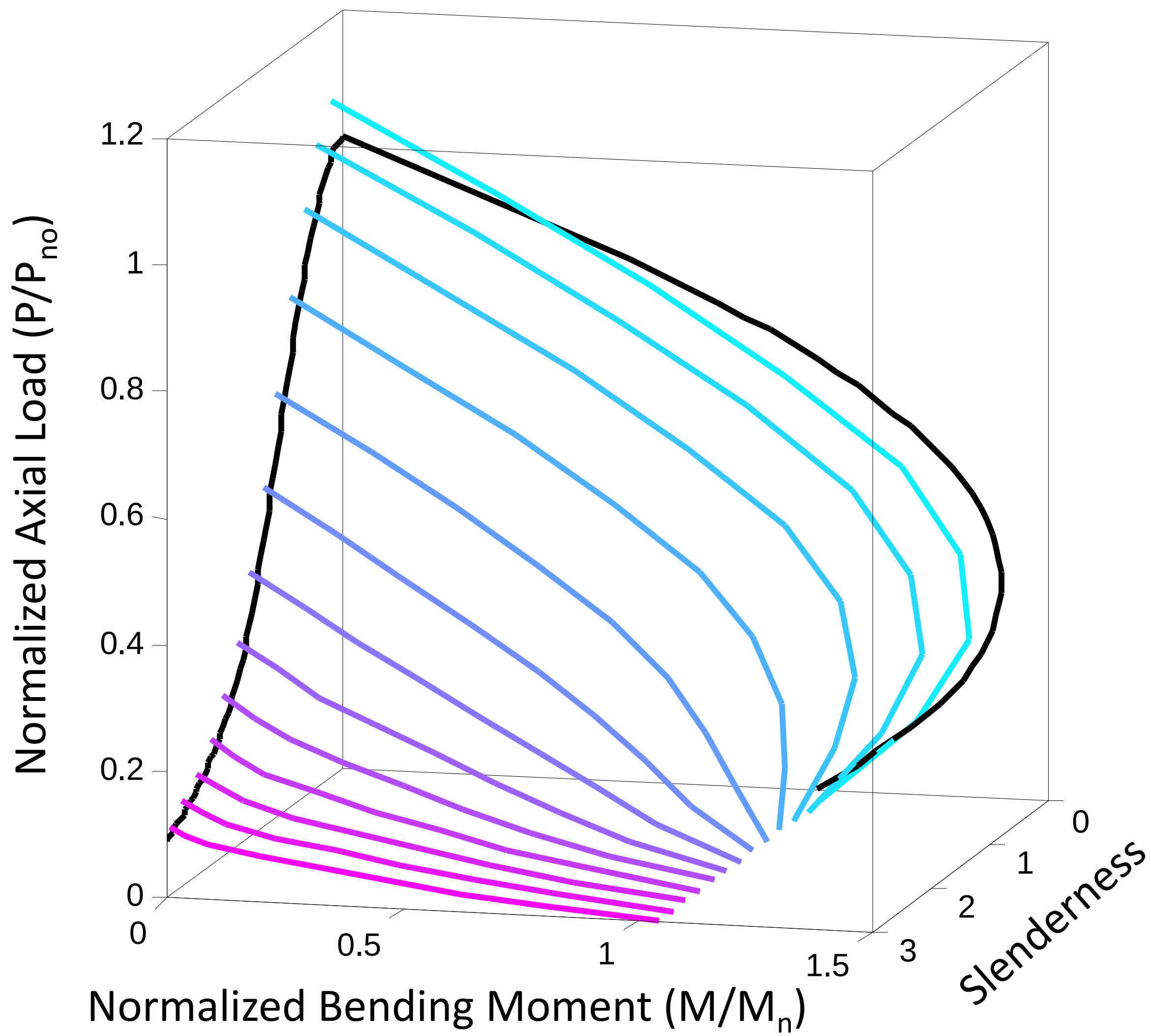


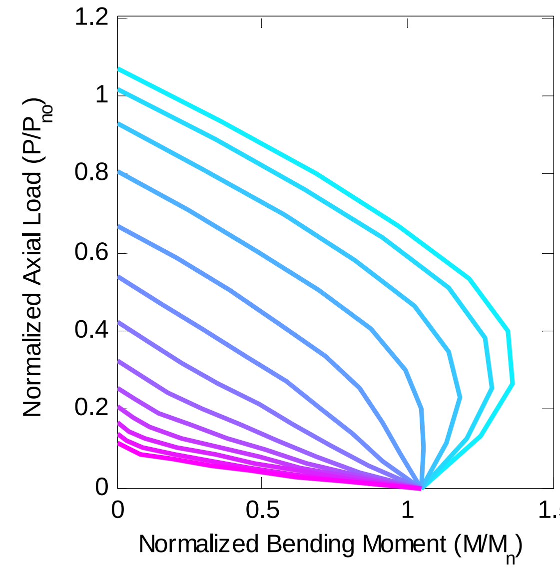
Proposed

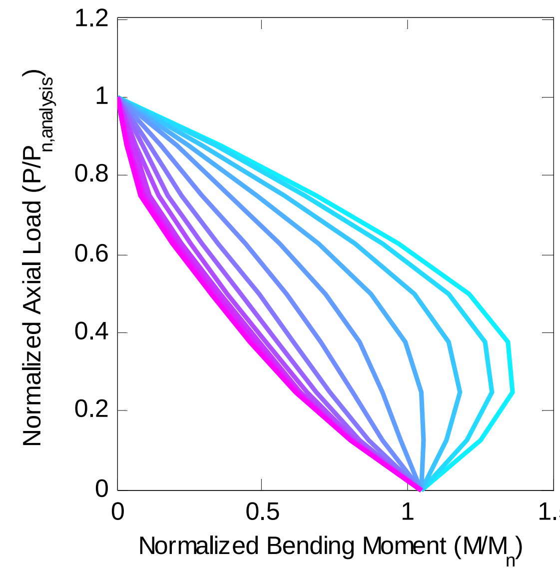
$$\alpha_B = \begin{cases} 1 & \text{for } \lambda_{oe} \le 1 \\ 1 - 0.2\left(\lambda_{oe} - 1\right) & \text{for } 1 < \lambda_{oe} \le 2 \\ 0.8 & \text{for } \lambda_{oe} > 2 \end{cases}$$

$$\alpha_C = \begin{cases} P_C / P_A & \text{for } \lambda_{oe} \le 0.5 \\ P_C / P_A - \left(P_C / P_A - 0.2\right)\left(\lambda_{oe} - 0.5\right) & \text{for } 0.5 < \lambda_{oe} \le 1.5 \\ 0.2 & \text{for } \lambda_{oe} > 1.5 \end{cases}$$

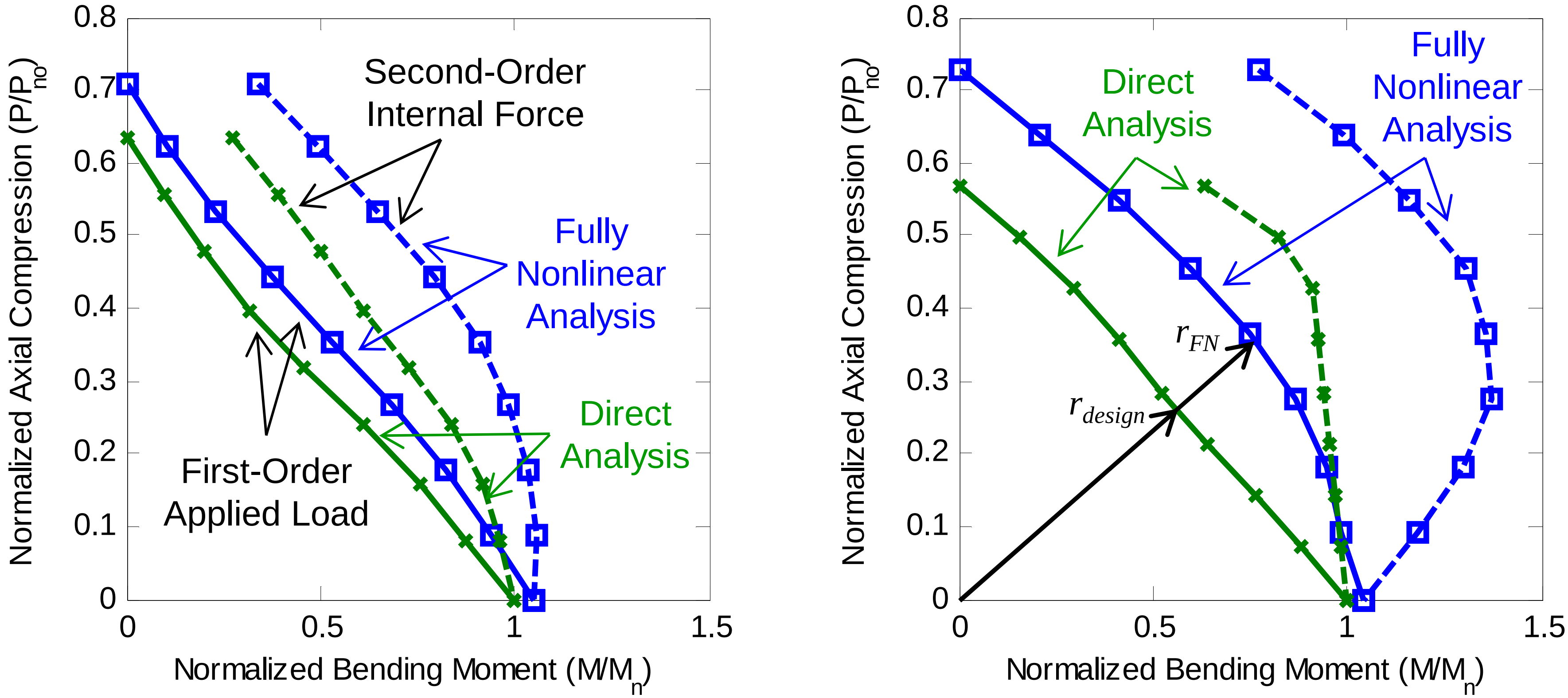
Variation of the Interaction Diagram with Slenderness







Example Results: Fully Nonlinear and Design Interaction Diagrams

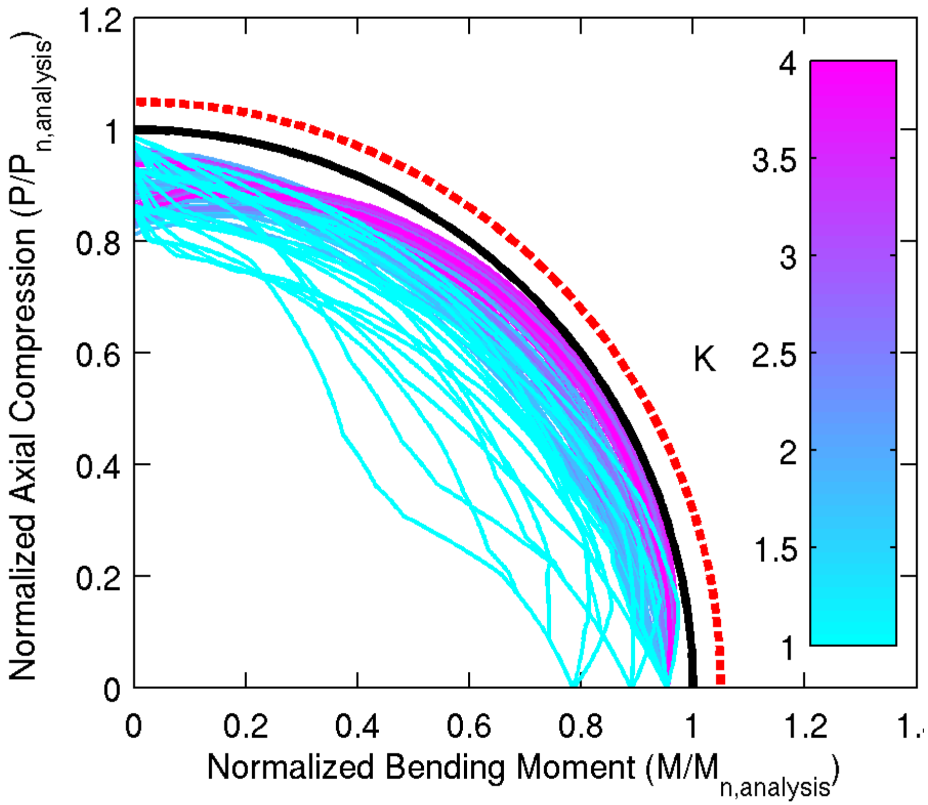


Section 4: RCFT-B-4, Frame 37: UA-67-g1

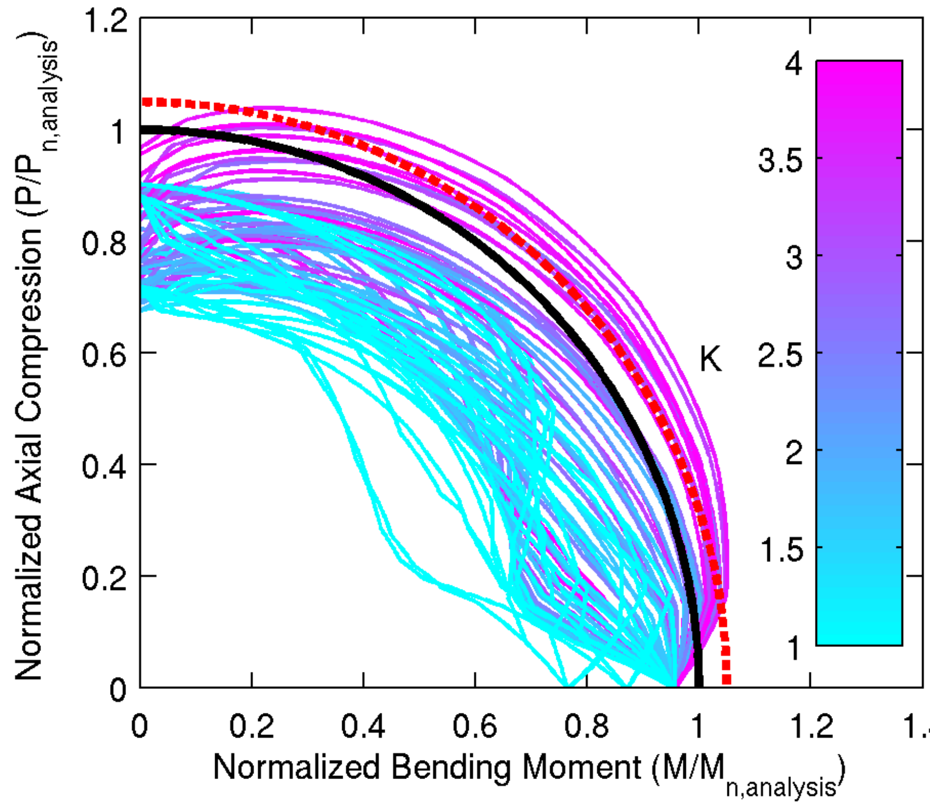
Section 13: RCFT-E-4, Frame 37: UA-67-g1

Radial Error Measure: $\varepsilon = \dfrac{r_{FN} - r_{design}}{r_{FN}}$

Example Results: Normalized Interaction Diagrams



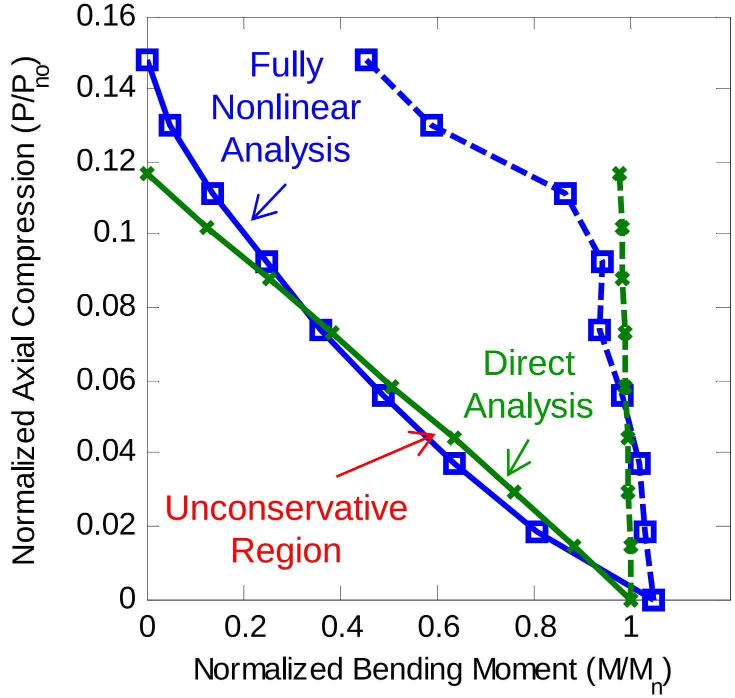
Section 4: RCFT-B-4



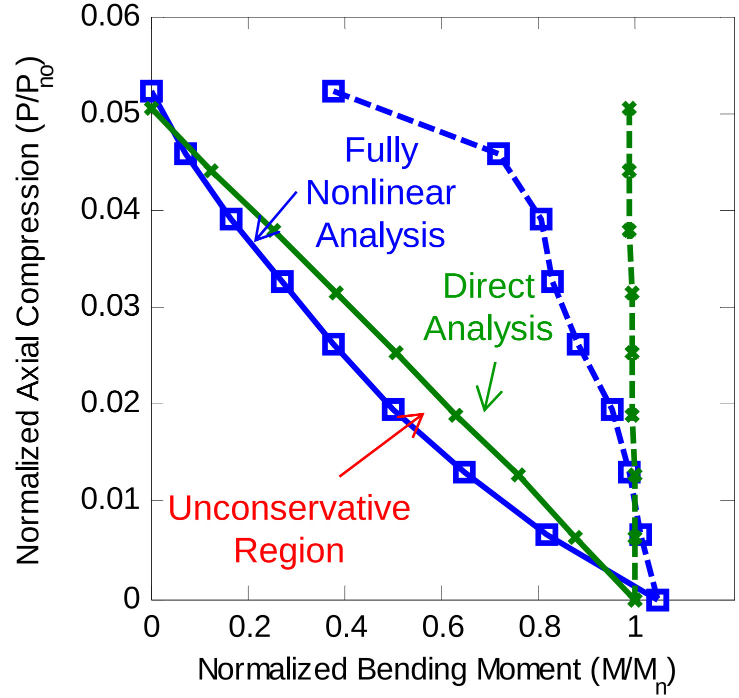
Section 13: RCFT-E-4

Exceptions to the 5% unconservative limit

1. Members with high effective length factors (e.g., an effective length factor, *K*, greater than approximately 3)



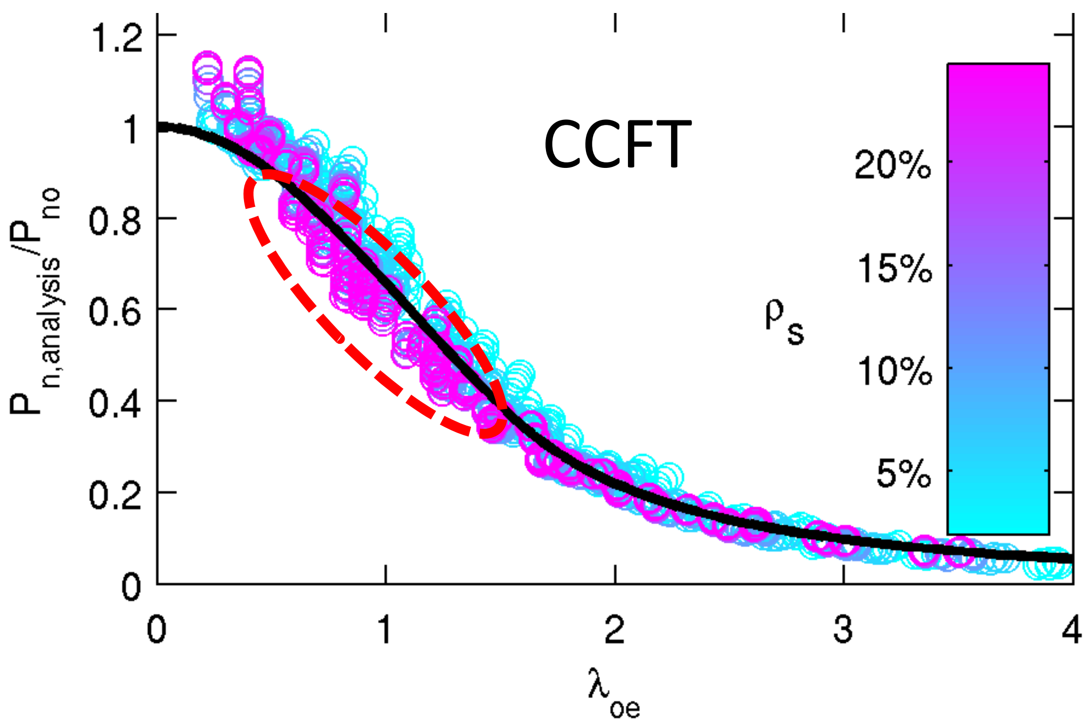
Section 13: RCFT-E-4, Frame 43: UC-67-g2



Section 13: RCFT-E-4, Frame 63: UC-90-g3

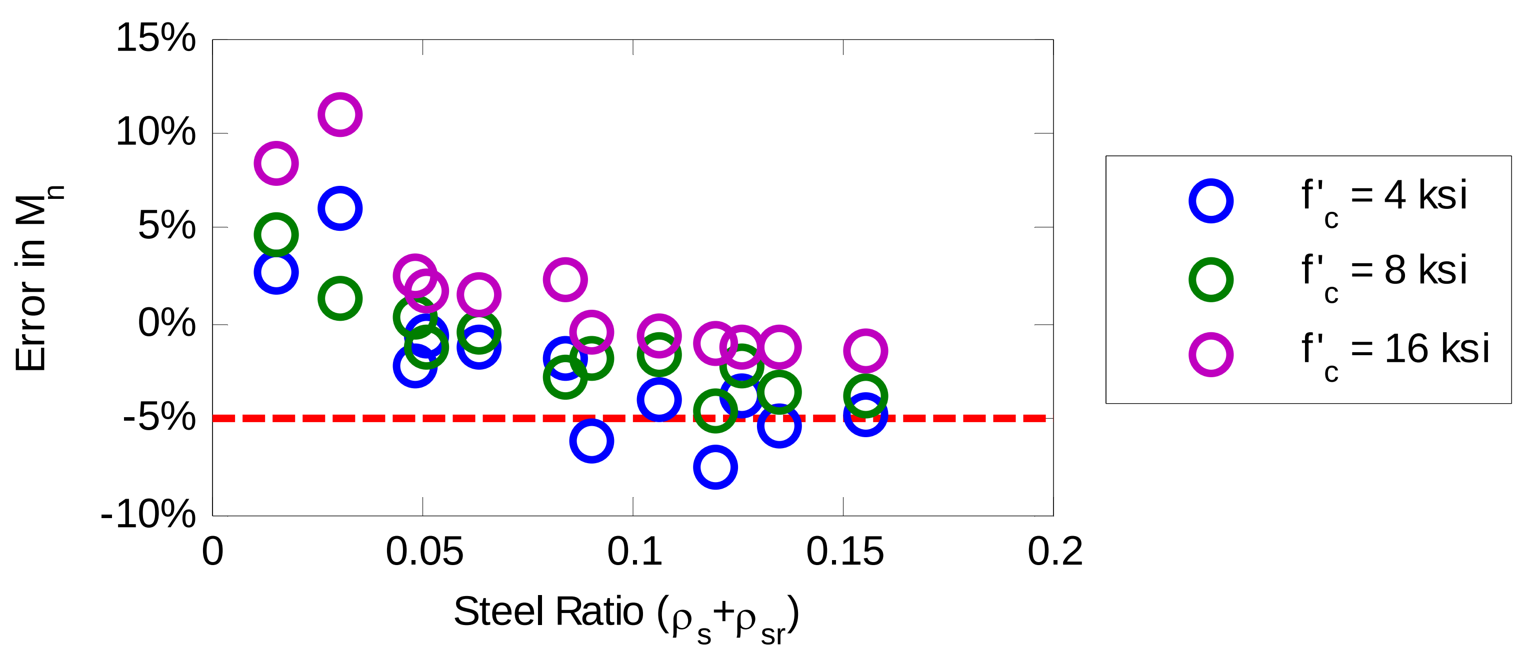
Exceptions to the 5% unconservative limit

2. Steel dominant CCFT members where the axial compressive strength, P_n, is overpredicted by the design equations

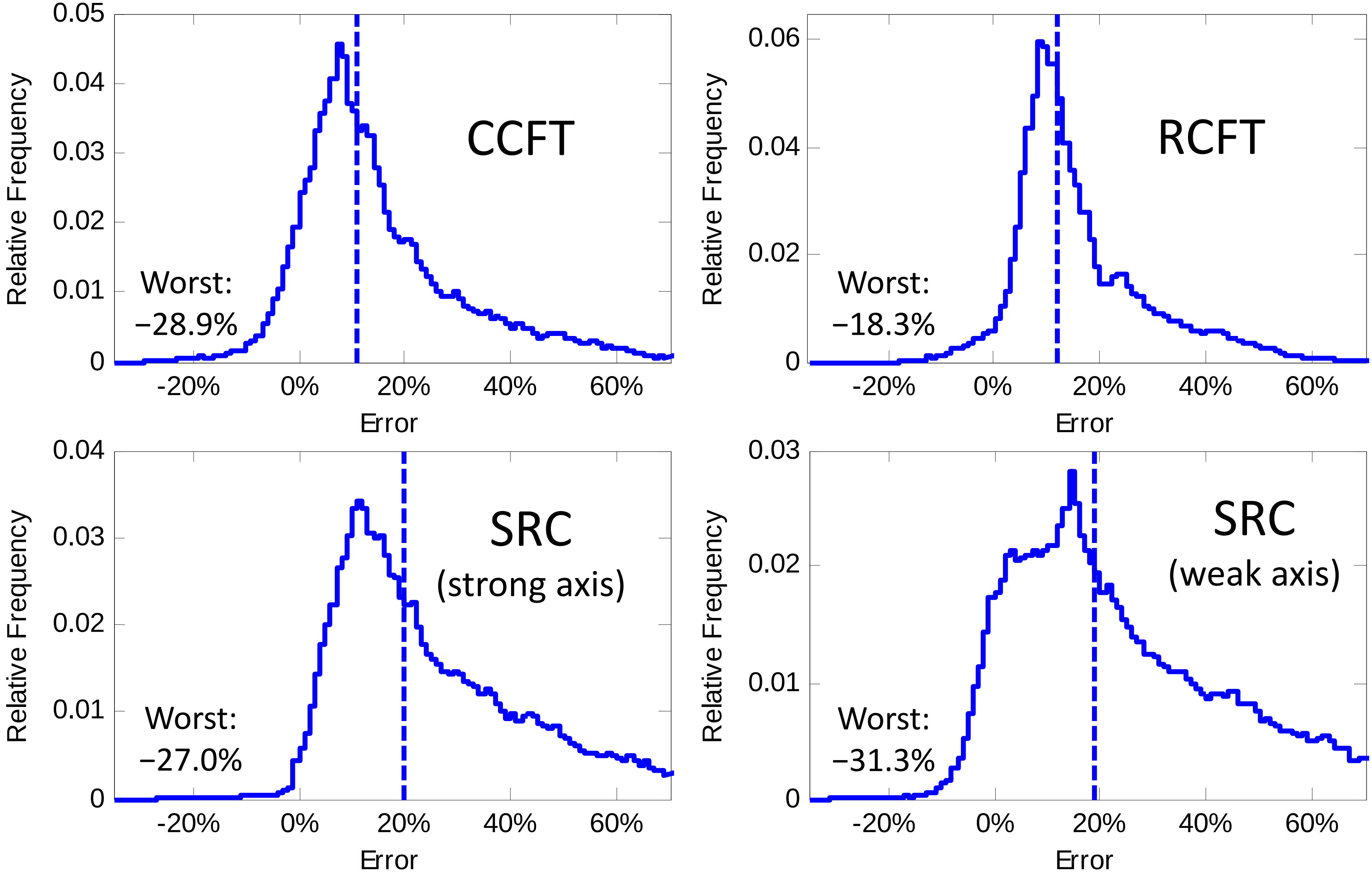


Exceptions to the 5% unconservative limit

3. Steel dominant weak axis SRC members where the flexural strength, M_n, is overpredicted by the design equations



Summary Error Statistics



Conclusions

- Large parametric study undertaken to assess the in-plane stability behavior of steel-concrete composite members
- This research has developed:
  - New elastic flexural rigidities for elastic analysis of composite members
  - New effective flexural rigidities for calculating the axial compressive strength of SRC members
  - New recommendations for the construction of the interaction diagram for composite members.
- The proposed beam-column design methodology is safe and accurate for the vast majority of common cases of composite member behavior

Extra Slides

Confinement Model for SRC and CFT Members

CCFT

- Confinement pressure computed as a hoop stress based on the tube slenderness
- Peak stress computed using Mander's model for symmetric states of confining pressure

$$\alpha_\theta = 0.138 - 0.00174\left(D/t\right) \geq 0 \qquad f_l = \frac{2\alpha_\theta F_y}{D/t - 2}$$

$$f'_{cc} = f'_c\left(-1.254 + 2.254\sqrt{1 + 7.94\left(f_l / f'_c\right)} - 2.0\left(f_l / f'_c\right)\right)$$

RCFT

- Confinement assumed to have no effect on the peak stress

SRC

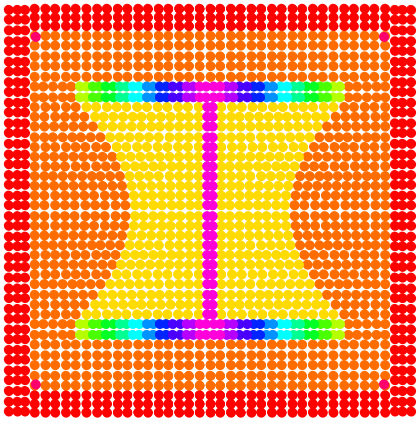
- Concrete divided into three regions:
  - Cover
  - Medium Confined (within the lateral reinforcement)
  - Highly Confined (within the flanges of the steel shape)
- Confinement pressure computed in two orthogonal directions for each region
- Peak stress computed using Mander's model for a general triaxial state of stress

$$f_{ly,medium} = K_e \rho_y F_{yr}$$

$$f_{lz,medium} = K_e \rho_z F_{yr}$$

$$f_{ly,high} = f_{ly,medium} + \frac{t_f^2 F_{ys}}{0.75\left(b_f - t_w\right)^2}$$

$$K = \frac{f'_{cc}}{f'_c} = 1 + A\overline{x}\left(0.1 + \frac{0.9}{1 + B\overline{x}}\right) \qquad \overline{x} = \frac{f_{l1} + f_l}{2 f_c}$$



Axial Strength

$$\frac{P_n}{P_{no}} = \begin{cases} 0.658^{\lambda_{oe}^2} & \text{for } \lambda_{oe} \le 1.5 \\ 0.877/\lambda_{oe}^2 & \text{for } \lambda_{oe} > 1.5 \end{cases}$$

$$\lambda_{oe} = \frac{KL}{\pi}\sqrt{\frac{P_{no}}{EI_{eff}}}$$

$$P_{no} = F_y A_s + F_{ysr} A_{sr} + 0.85 f_c' A_c \qquad \text{(SRC)}$$

$$P_{no} = F_y A_s + C_2 f_c' A_c \qquad \text{(CFT)}$$

Variation of the Second-Order Internal Force Interaction Diagram with Slenderness