Collapse Assessment of the ACC Building Part II:
Advanced Analysis Formulation and Observations

David P. Welch, PhD
Haselton Baker Risk Group

SEAOC STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA
San Diego SEAOC 2025 CONVENTION

CONTRIBUTORS
Truly A Team Effort…

Simpson Gumpertz & Heger

Kevin S. Moore, SE
Keith D. Palmer, PhD, SE
Ali Roufegarinejad, PhD, PE
Vamshi Krishna Gudipati, PhD, PE
Ayush Singhania, PE
Aaron Yung, SE
Alvin Wong

Haselton Baker Risk Group

David P. Welch, PhD
Ed Almeter, MSc
D. Jared DeBock, PhD, PE
Curt B. Haselton, PhD, PE
Jack W. Baker, PhD

University of Colorado, Boulder

Maria Jose Echeverria, PhD
Abbie B. Liel, PhD, PE

Nat'l Institute of Standards & Technology

Dustin T. Cook, PhD, PE

Rutherford + Chekene – Peer Review

Brett Lizundia, SE
Kwasi Bempong, PE

UCSF SRC Committee

Afsaneh Ahmadi, PE, CBO
Curt Haselton, PhD
Brett Lizundia, SE
Joe Maffei, SE
Kevin Moore, SE
Maryann Phipps, SE
Ray Pugliesi, SE

Recap from Part I (2019 study)

- SPR V is untenable (requires closure/evacuation in 2030)
- Seismic retrofit costs too much and is too disruptive
- Building is incredibly well designed
- Kevin bet his career that the team could show SPR IV
- UCSF commissioned most advanced NLRHA/ASCE 41-23 analysis
  - Perform 3D and OpenSees models (full and complete)
  - Consider overall building collapse probabilities
  - FEMA P695 statistical considerations
  - Element calibration uses applicable test data

Overview

How can we take a closer look at the ACC Building?

Material Study 1: RC core walls

- Shear strength
- Shear-flexure response
- Hysteretic properties

Material Study 2: Circular tower columns

- Shear strength
- Backbone curve
- Hysteretic properties

Nonlinear Modeling in *OpenSees*

- Modeling progression
- Sub-grade modeling
- Model variations

Nonlinear Analysis

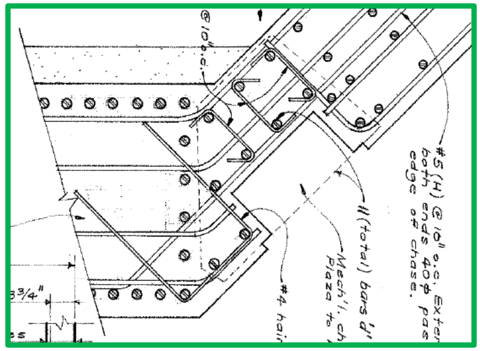
- Acceptance criteria
- Ground motions
- FEMA P695 parameter studies
- Primary results
- Additional studies and analysis

Conclusions

RC Core Wall Study: Brief Detail Check

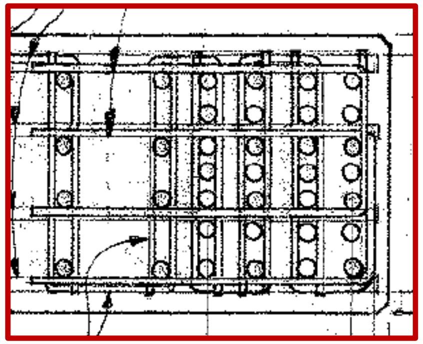
Small East/West Mech. Chase

- Continuous horizontal bars
- 90 ° - 135° hooked transverse bars



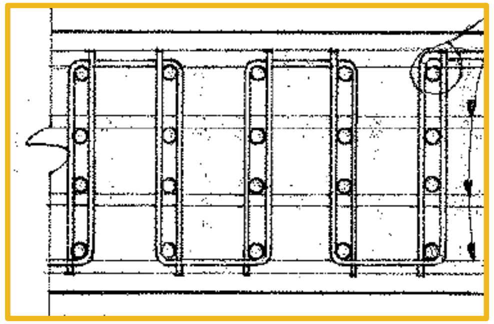
Opening Boundary Elements

- Transverse hairpin dowels
- Horizontal hairpin dowels (staggered with main horiz. bars)



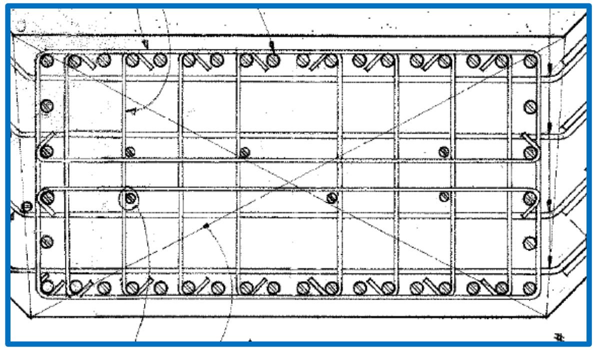
Typical “Web” Detail

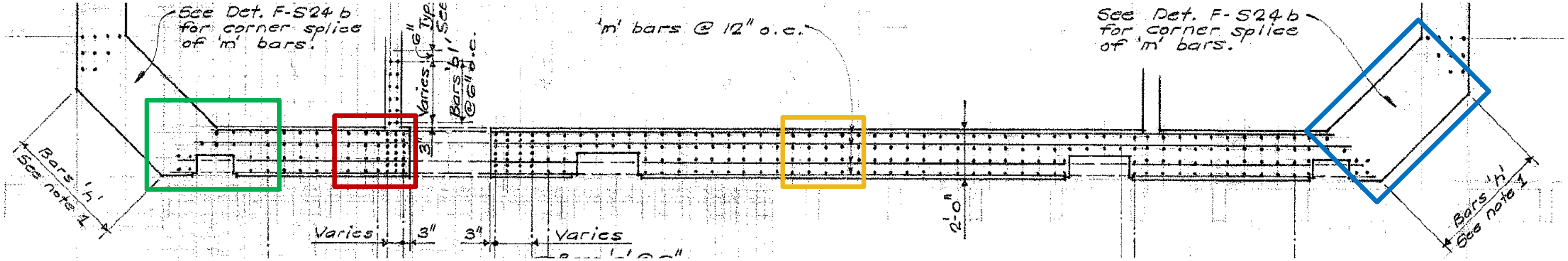
- 4 layers of horiz. rein.
- Hairpin dowels at each outside long. bar



Corner Boundary Elements

- Hooked perimeter bar
- Distributed 135° hooked bars





*Typical East/West tower wall

RC Core Wall Study: Shear Strength

Yield Shear Strength (Point B):

- Based on ACI 318 shear strength, modified based on how shear critical the wall is.

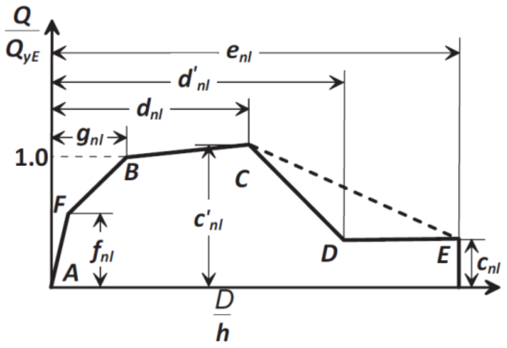
$$V_{CydWallE} = \left(2.0 - \frac{V_{CWall318E}}{\omega_v V_{MCultE}}\right) V_{CWall318E} \quad \leq 1.8 V_{CWall318E}$$
$$\geq 0.8 V_{CWall318E} \qquad (7.2.2c)$$

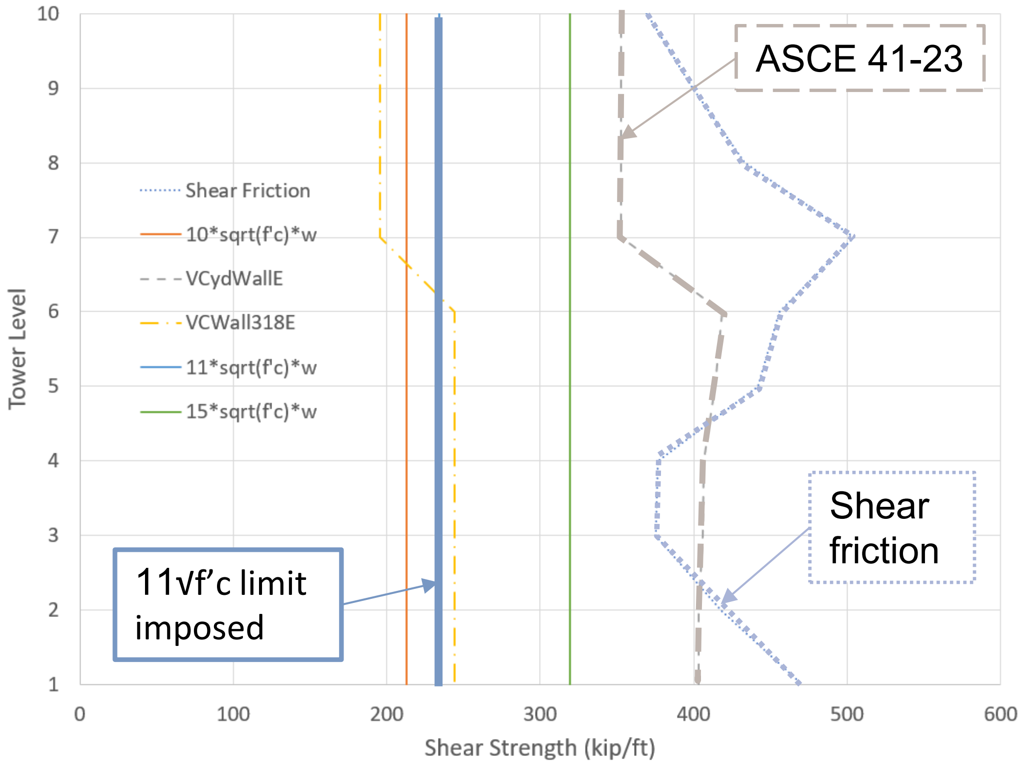
- **In ASCE 41-23, no "Xsqrt(f'c)" concrete strength cap is applied**

ASCE 41 $\quad V_{CWall318E} = (\alpha_c \lambda \sqrt{f'_{cE}} + \rho_t f_{ytE}) A_{cv} \qquad (7.2.2d)$

FEMA P-2208 $\quad V_{CWall318E} = A_{cv}\left(\alpha_c \sqrt{f'_{cE}} + \rho_{web,h} f_{ytE}\right) \underline{\leq 10 A_{cv} \sqrt{f'_{cE}}} \qquad (2\text{-}1)$

- Critical ACC walls not controlled by shear friction strength
- Feedback from Dr. Wassim Ghannoum:
  Shear strength should be limited for compression-shear failure:
  - $10A_{cv}\sqrt{f'_c}$ when compression toe does not have a flange
  - $12A_{cv}\sqrt{f'_c}$ with a flange
- **We use an average limit of $11A_{cv}\sqrt{f'_c}$**





RC Core Wall Study: Hysteretic properties

Scope of Experimental Calibration

- Two experimental databases reviewed (UoA-UW and SERIES)

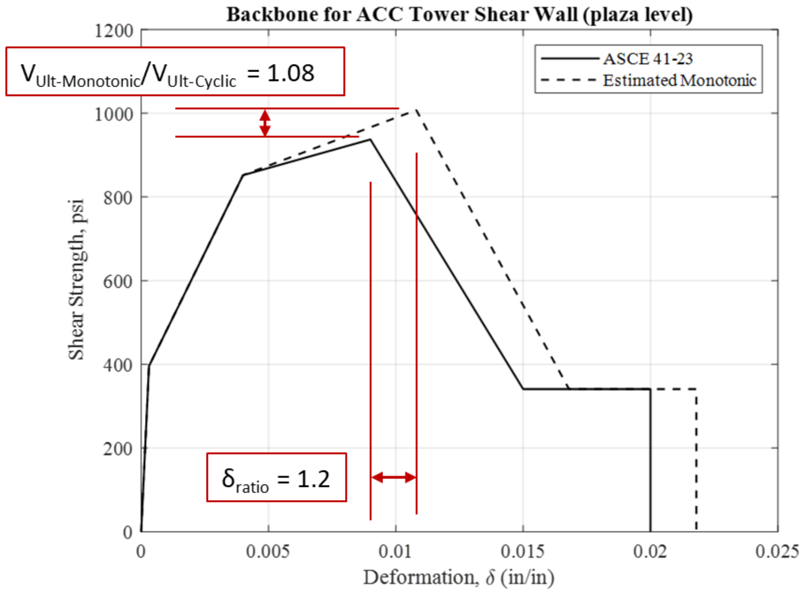
Database search criteria:

1. Specimen **failure mode in shear or flexure-shear**.
2. Presence of **two or more layers of reinforcement steel**.
3. Aspect ratio (height to length of the wall) ≤ 2.5.
4. Slenderness ratio (length to thickness of the wall) between 9 and 19.
5. Transverse reinforcement ratio between 0.0035 and 0.012.
6. Concrete compressive strength ranging from 3,000 to 7,800 psi.
7. Normalized axial load ≥ 0.07.
8. **Cyclic loading.**

RC Core Wall Study: Hysteretic properties

Scope of Experimental Calibration

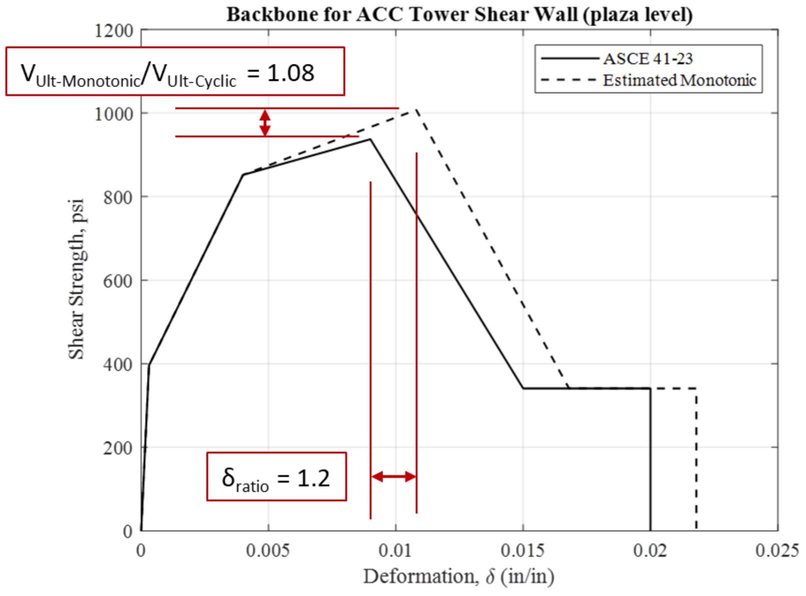
- Two experimental databases reviewed (UoA-UW and SERIES)
- Pairs of shear/shear-flexure monotonic and cyclic tests used to inform pre-degraded backbone adjustments

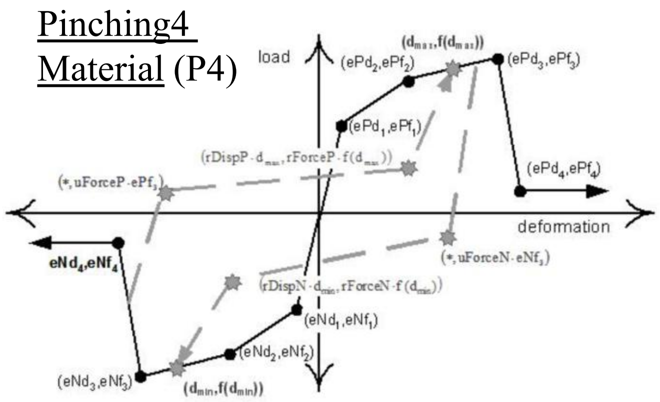


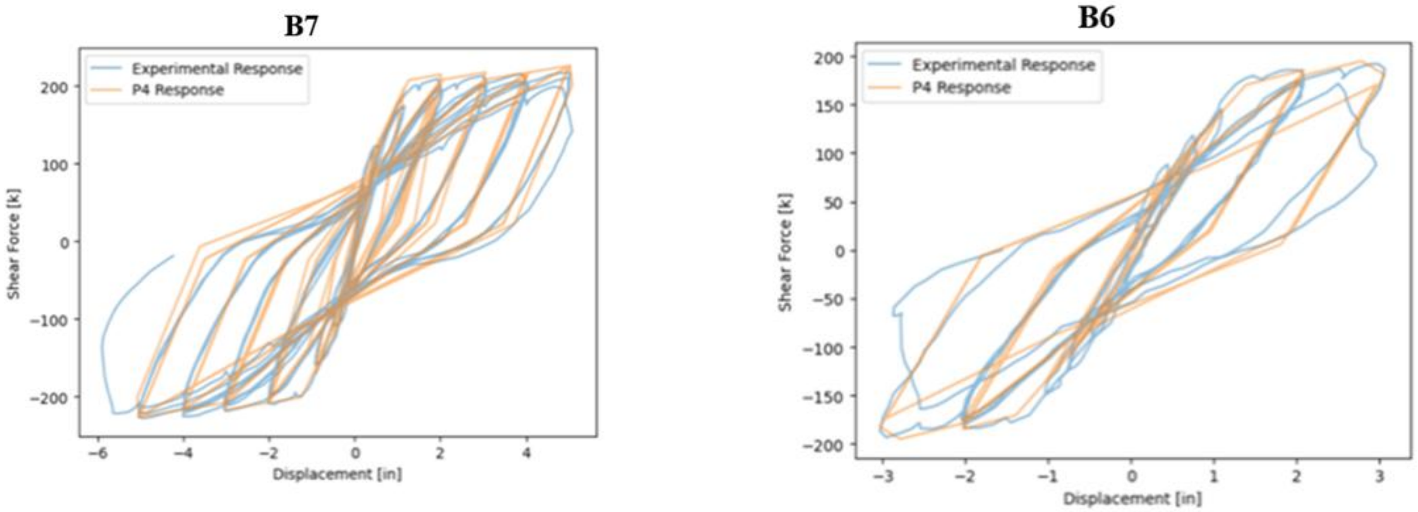
RC Core Wall Study: Hysteretic properties

Scope of Experimental Calibration

- Two experimental databases reviewed (UoA-UW and SERIES)
- Pairs of shear/shear-flexure monotonic and cyclic tests used to inform pre-degraded backbone adjustments
- Representative tests used for cyclic calibration with the *Pinching4* material (Lowes et al. 2003)

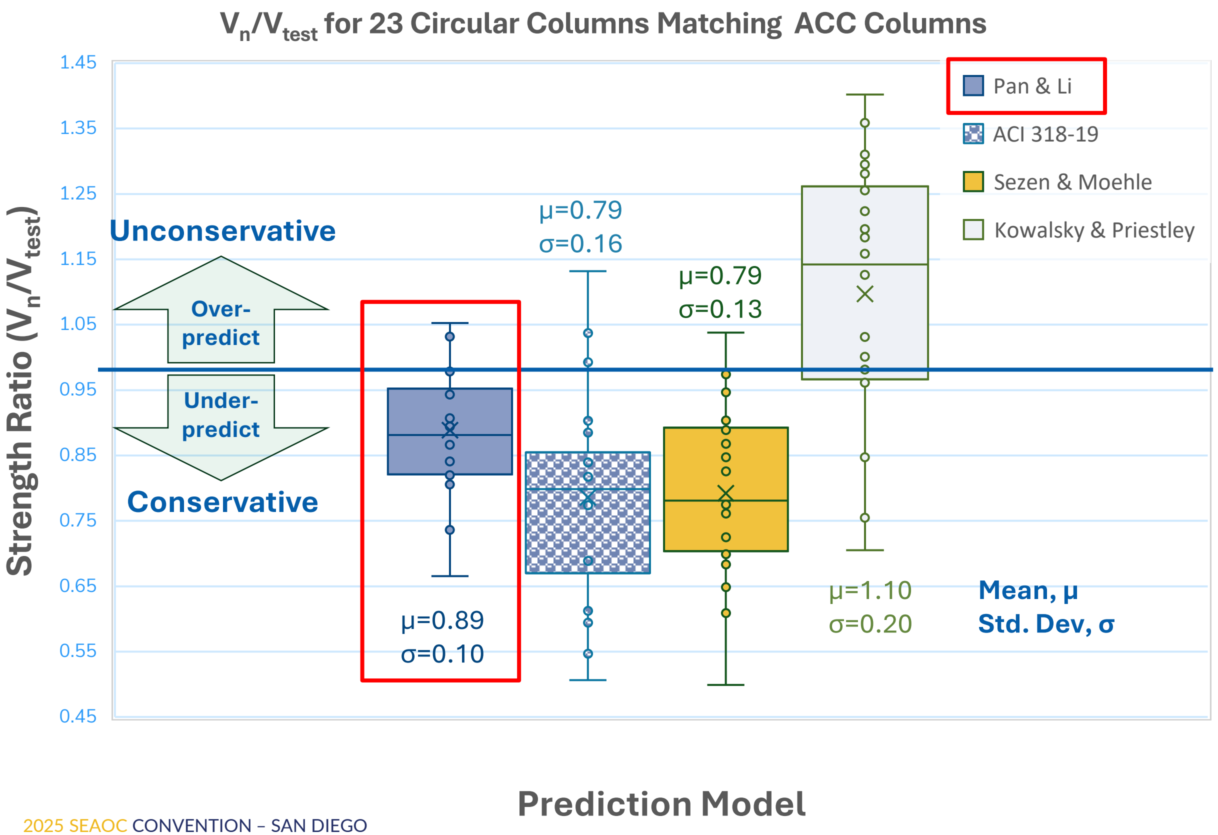


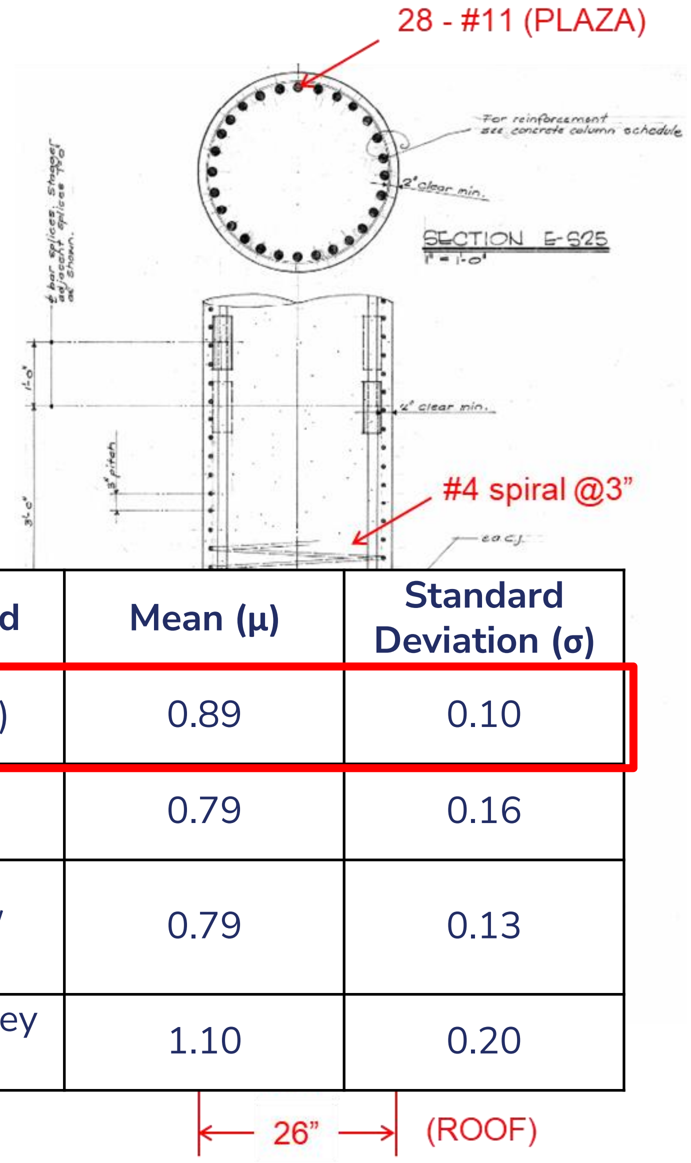




Circular Tower Column Study: Shear Strength

Property	NEES ACI 369 Circular Column Database	ACC Building Columns
a/d (Shear Span-to-Depth ratio)	1.36 - 2.49	1.21 - 1.75
ρ_l (longitudinal reinf. ratio)	2.4% - 3.9%	3.58% - 4.27%
ρ_v (trans. reinf. ratio)	0.2% - 0.6%	0.39% - 0.66%
$P/A_g f'_c$ (Axial load ratio)	0 – 38%	4% - 17%
# of test data or columns	23	

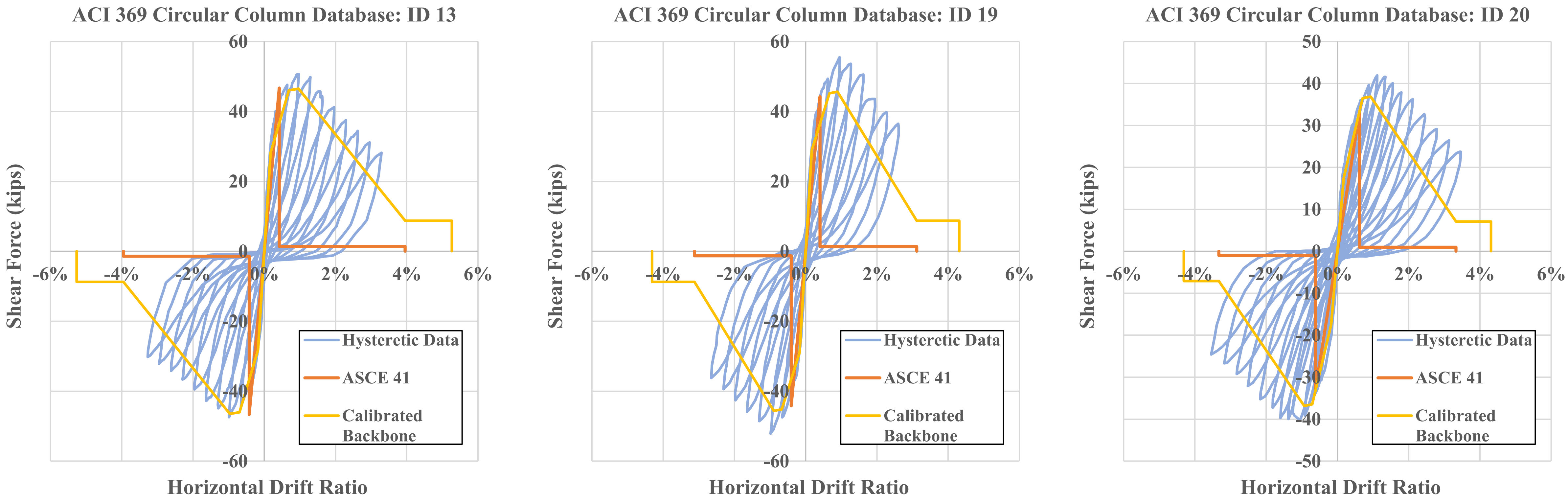




Prediction method	Mean (μ)	Standard Deviation (σ)
Pan and Li (2013)	0.89	0.10
ACI 318-19	0.79	0.16
Sezen & Moehle (2004)/ ACI 369/ ASCE 41	0.79	0.13
Kowalsky & Priestley (2000)	1.10	0.20

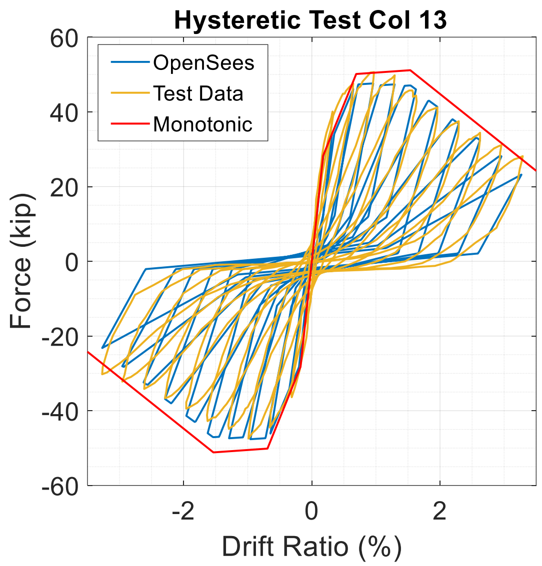
Circular Tower Column Study: Backbone

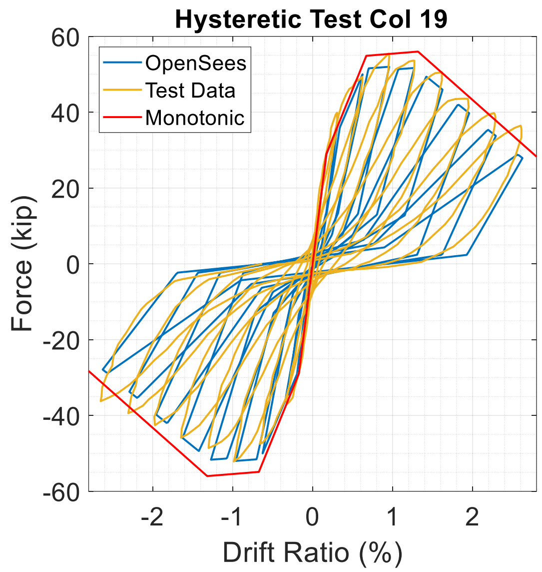
- Numerous backbone models considered
- Calibrated backbones compared with most representative circular column tests
- Much better agreement than Type 3 curve in ASCE 41-23



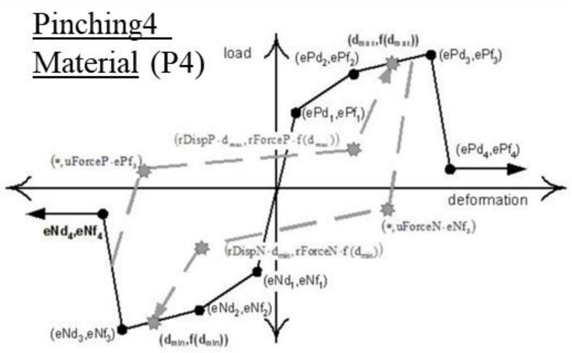
Circular Tower Column Study: Cyclic properties

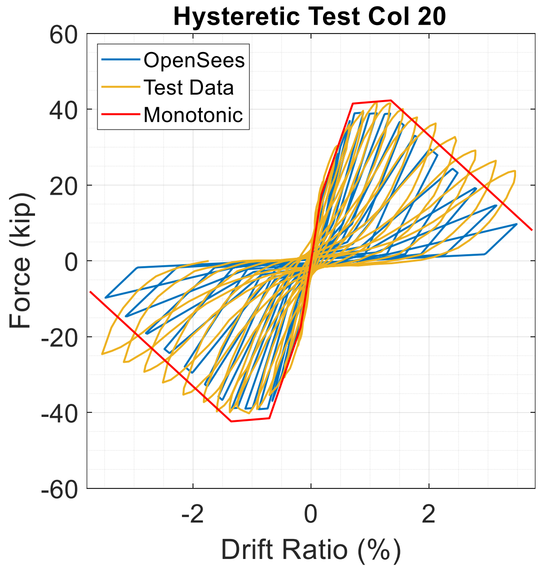
- Cyclic properties were estimated using representative column tests
- *Pinching4* material model was used for calibration in *OpenSees*





Column Properties	Test Data			ACC Columns		
	Col 13	Col 19	Col 20	Story 1	Story 2	Story 3
Shear span to depth ratio, a/d	1.39	1.39	2.02	1.17	1.56	1.29
Transverse reinforcement ratio by area, ρ_{ta}	0.0021	0.0014	0.0014	0.0017	0.0018	0.0019
Transverse reinforcement ratio by volume, ρ_{tv}	0.0097	0.0065	0.0065	0.0077	0.0082	0.0087
Longitudinal reinforcement ratio, ρ_l	0.041	0.041	0.041	0.039	0.040	0.041
Axial load ratio, $P/(f'c*A_g)$	0.126	0.115	0.115	0.114	0.113	0.110





Overview

How can we take a closer look at the ACC Building?

Material Study 1: RC core walls

- Shear strength
- Shear-flexure response
- Hysteretic properties

Material Study 2: Circular tower columns

- Shear strength
- Backbone curve
- Hysteretic properties

Nonlinear Modeling in *OpenSees*

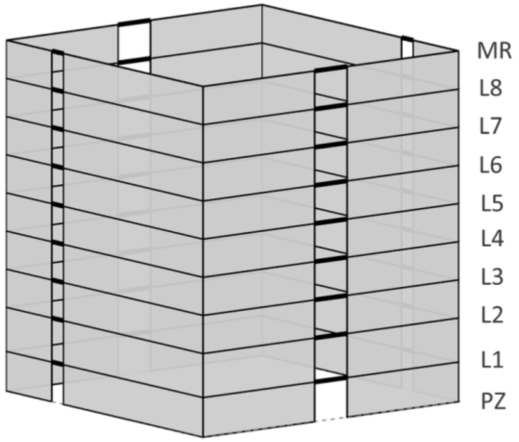
- Modeling progression
- Sub-grade modeling
- Model variations

Nonlinear Analysis

- Acceptance criteria
- Ground motions
- FEMA P695 parameter studies
- Primary results
- Additional studies and analysis

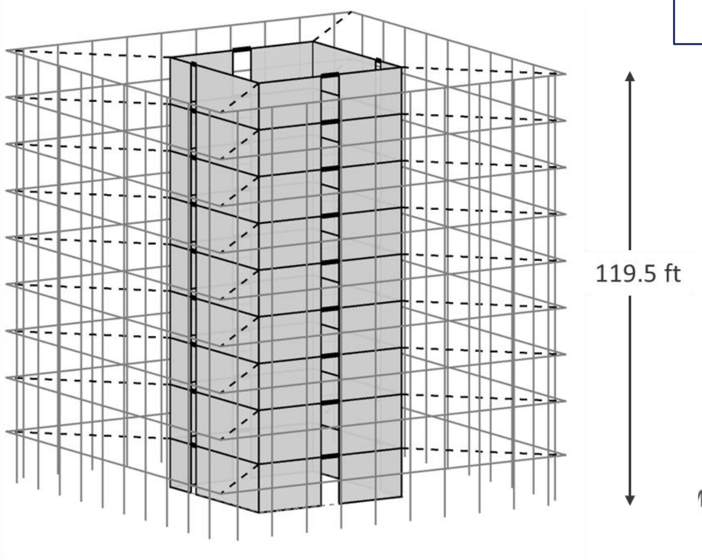
Conclusions

OpenSees Modeling: Overview

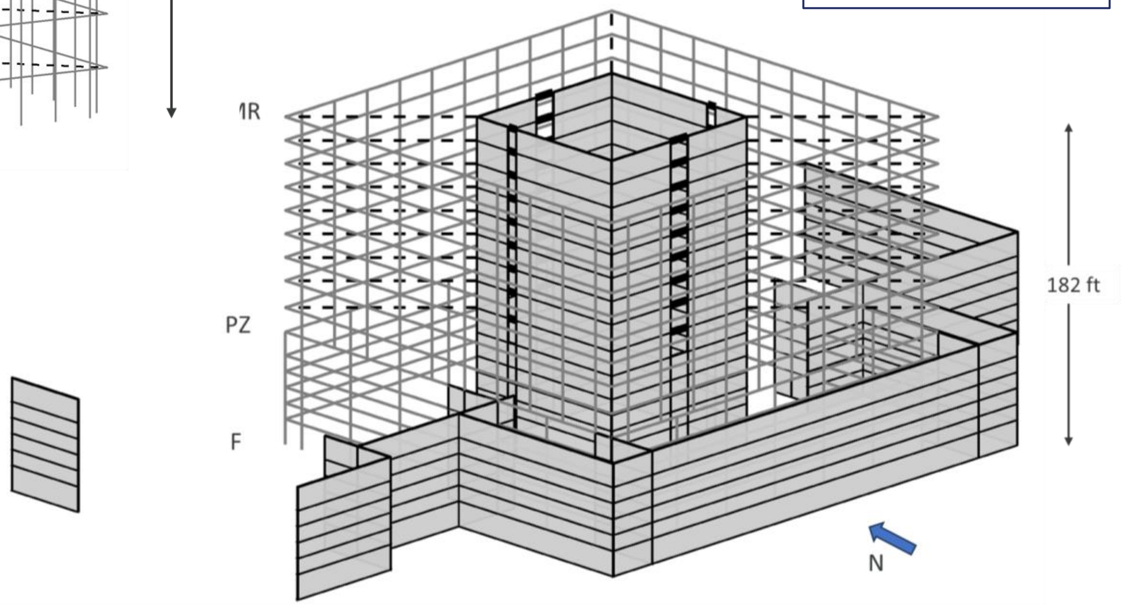


Core Only

Tower Only



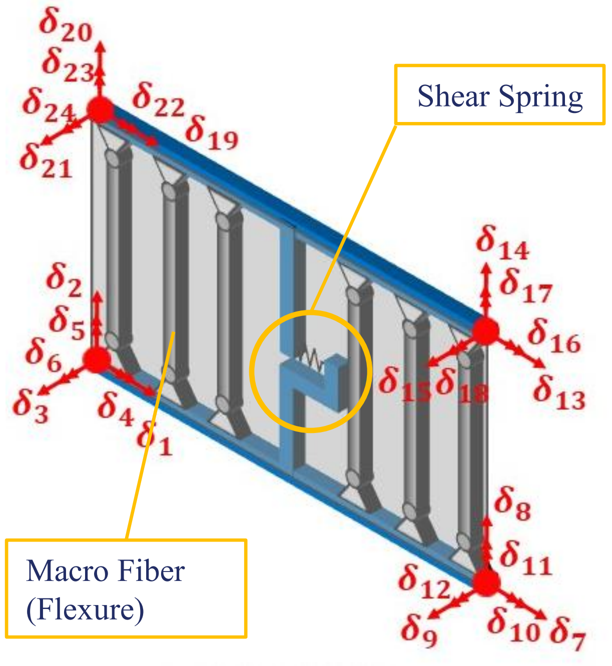
Tower plus Subgrade



*Progressive modeling and analysis for interpretation and validation (and keeping sanity)

OpenSees Modeling: Core Walls

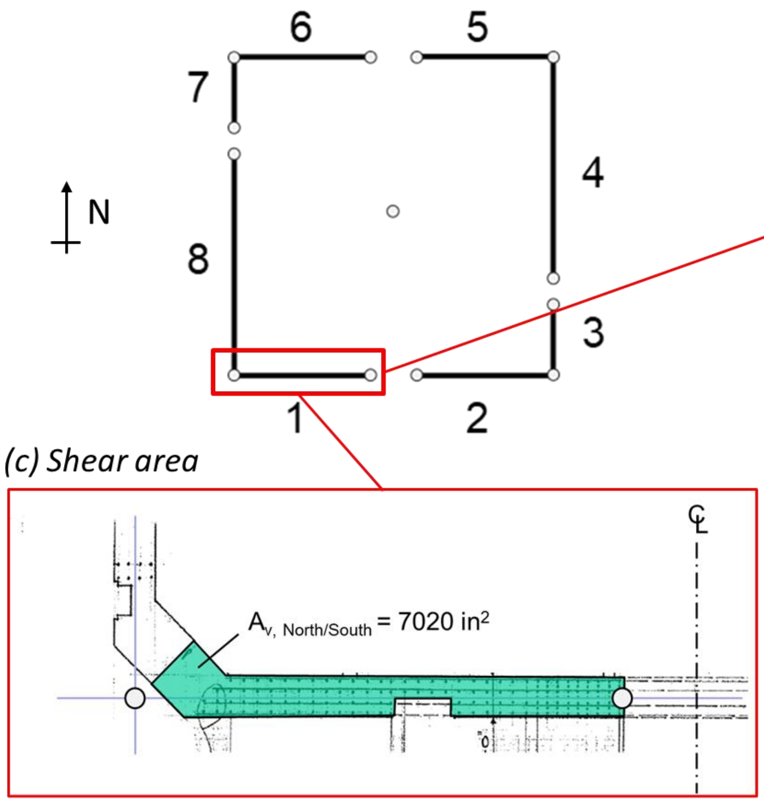
- Core walls modeled using MVLEM-3D elements (Kolozvari et al. 2021)
- Out-of-plane resistance not relied upon ($0.025EI_g$)



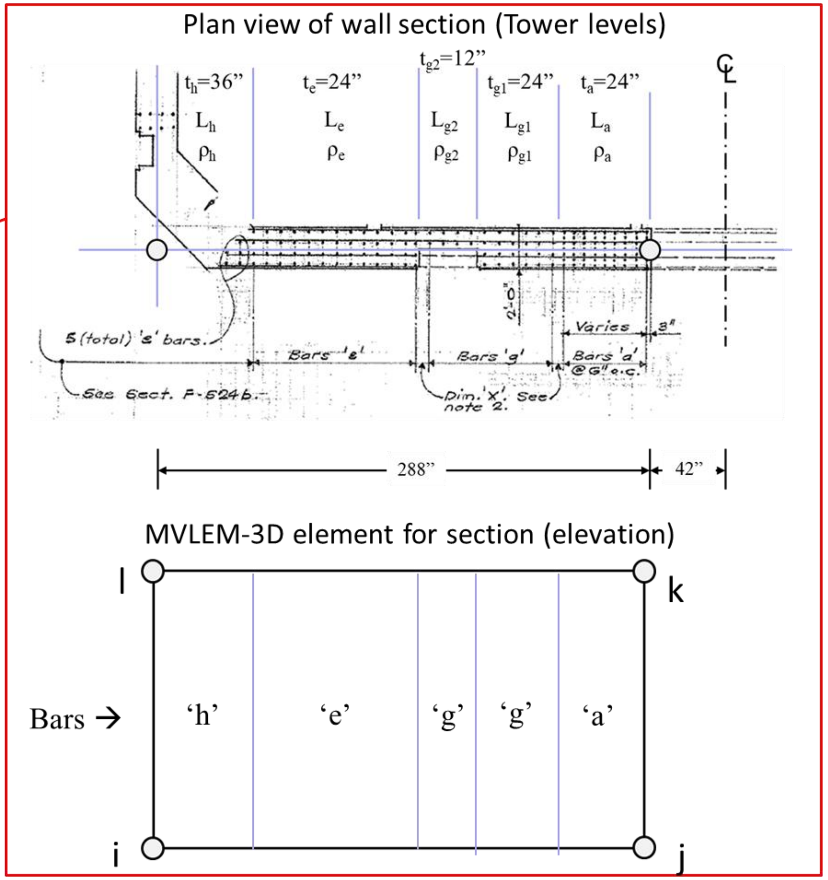
MVLEM-3D element
(Kolozvari et al. 2021)

(a) MVLEM-3D elements in plan (Tower levels)

(c) Shear area

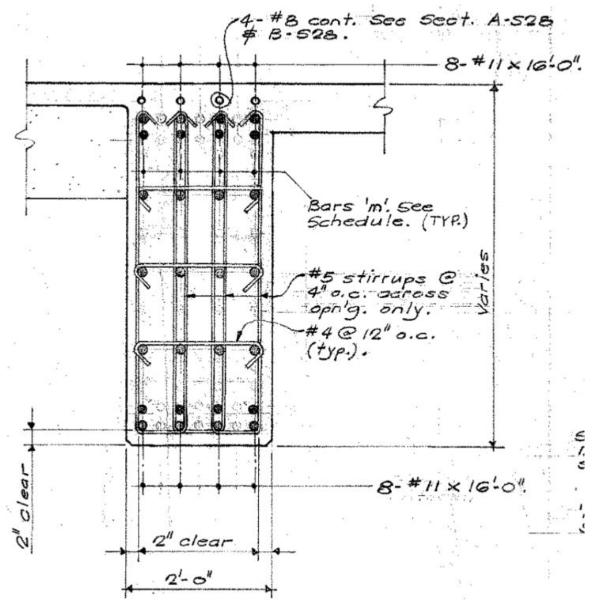


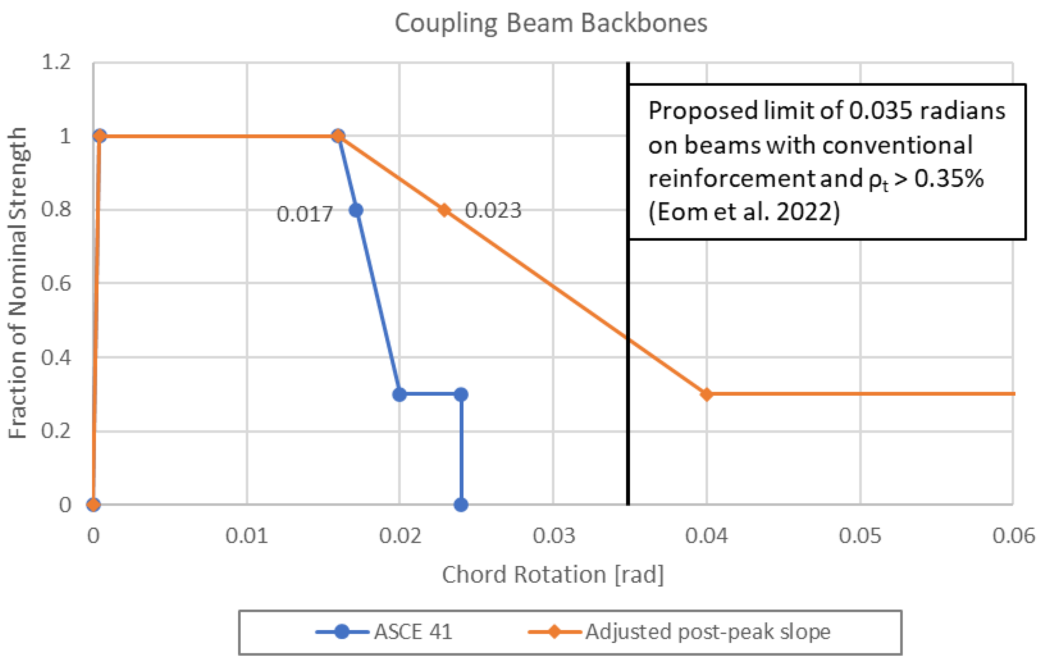
(b) Varying reinforcement and thickness for flexural fibers

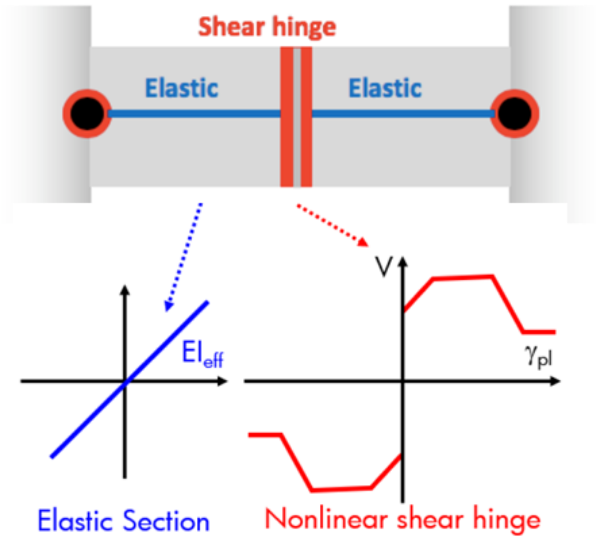


OpenSees Modeling: Coupling Beams

- ACC coupling beams are **low aspect ratio** (0.6 to 1.5) with **significant transverse reinforcement** (ρ_t ~ 1.9%)
- Nonlinear shear behavior follows ASCE 41-23 up to rotation at peak strength.
- Deformation at residual strength increased based on test data synthesis by Eom et al. (2022)
- Coupling beams use modeling approach by Terzic and Kolozvari (2020) and Tauberg et al. (2019)
- Hysteretic properties based on testing by Naish et al. (2013) and cyclic calibration by Tauberg et al. (2019)



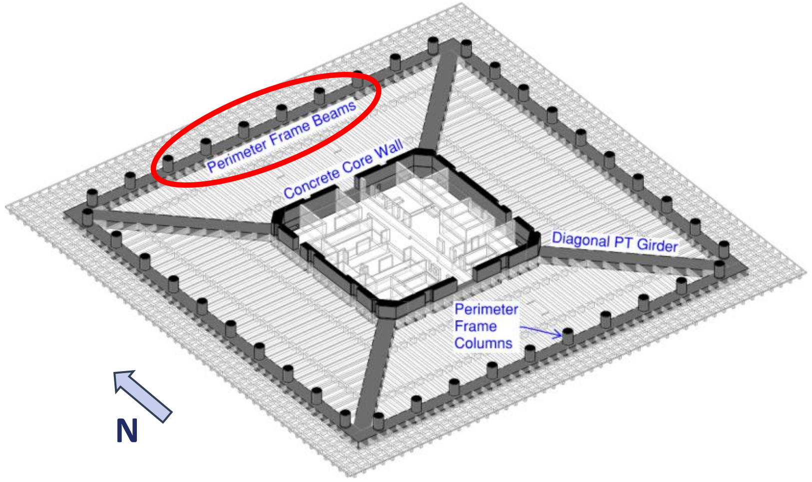


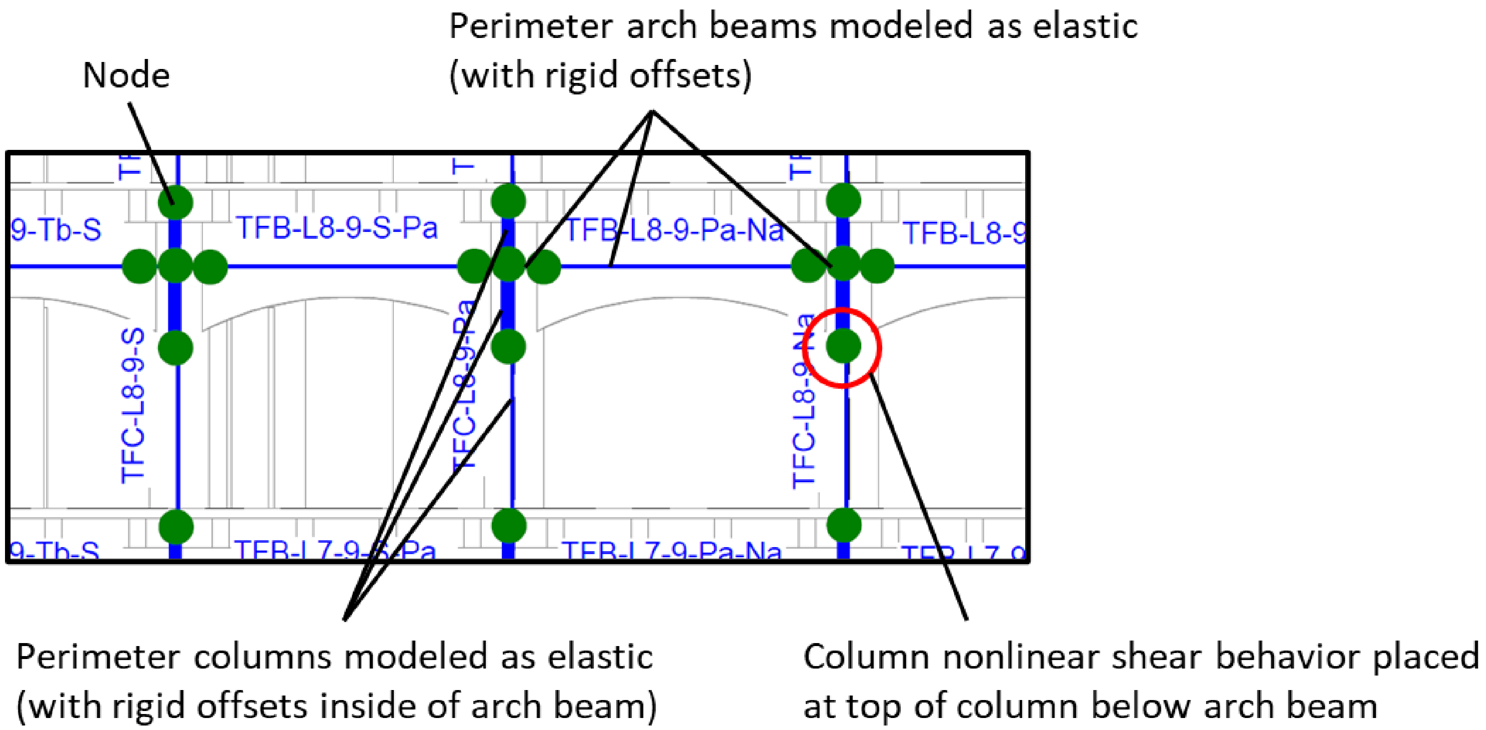


Terzic and Kolozvari (2019)

OpenSees Modeling: Tower Framing

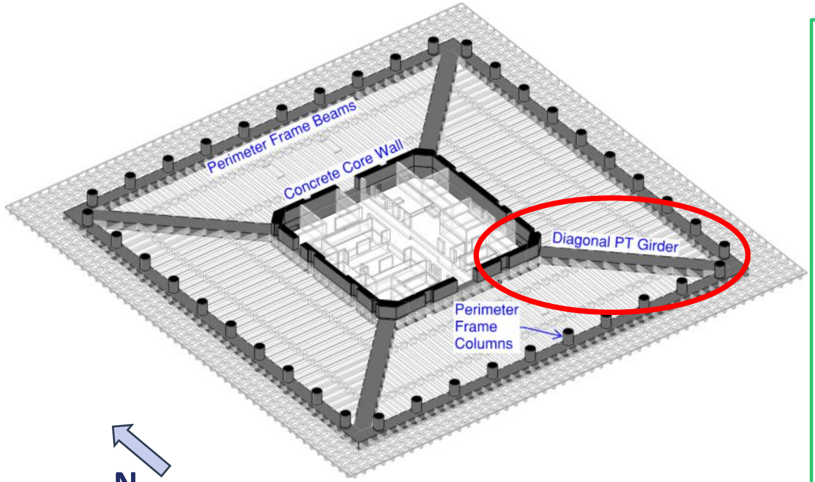
- Perimeter tower framing modeled using rigid offset to capture large arch beam depths
- Nonlinear shear behavior based on Circular Column Study

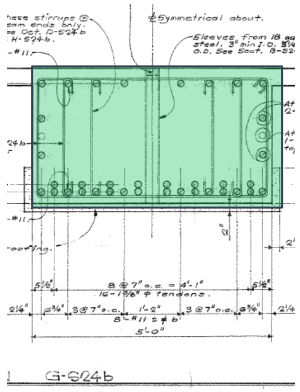




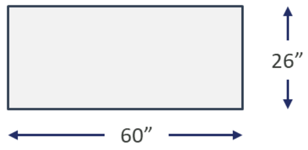
OpenSees Modeling: PT Girders

- Large diagonal PT girders act as outriggers from core to perimeter frame (tower only)

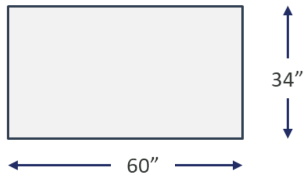


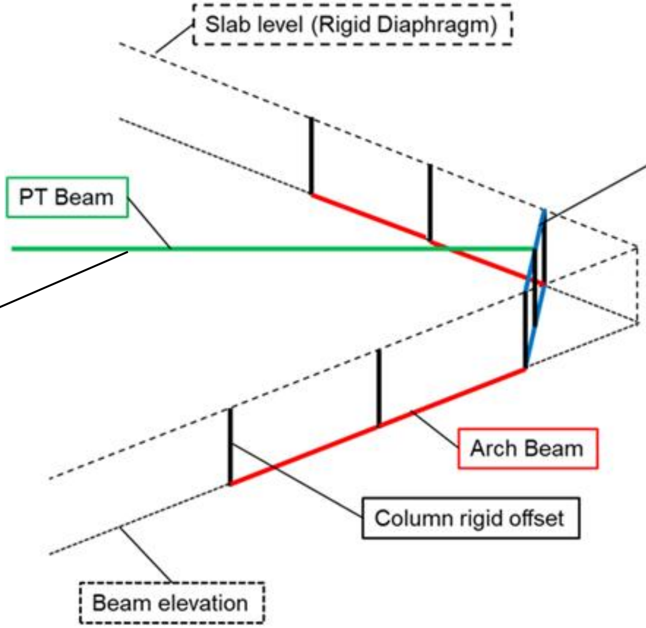


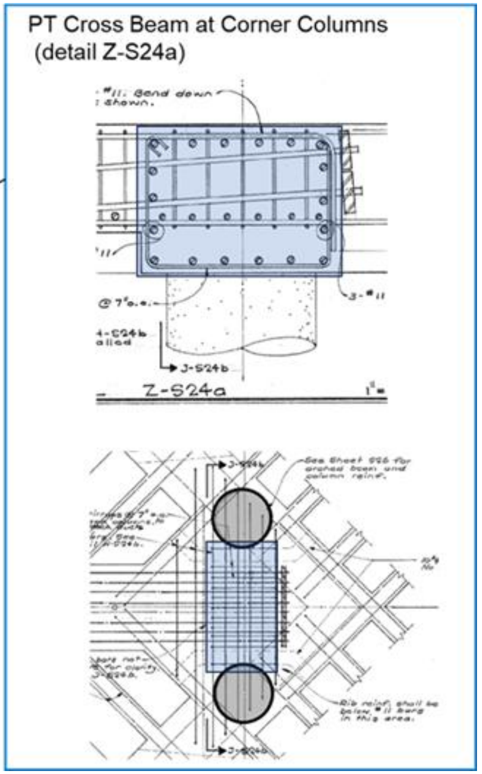
Typical Tower Level



Level 3 and 7

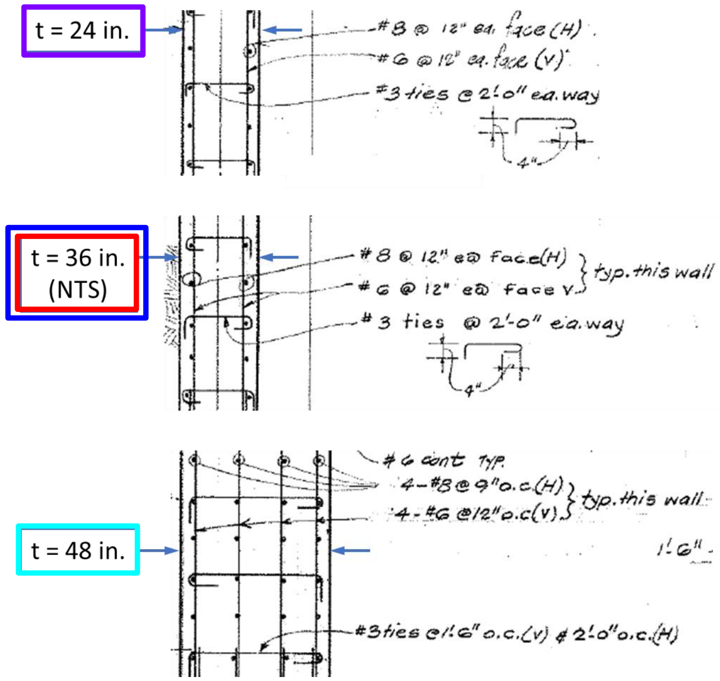


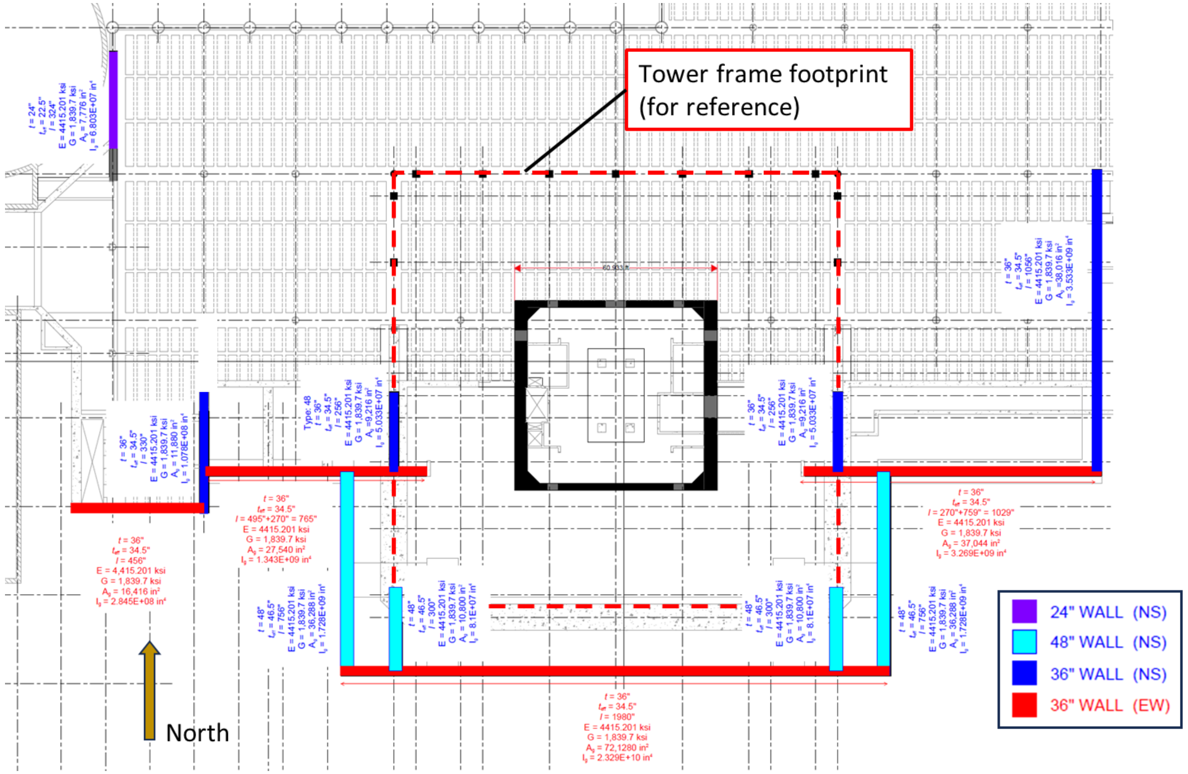




OpenSees Modeling: Subgrade Walls

- Subgrade basement walls modeled as elastic MVLEM-3D elements
- Walls range from 24" to 48 " thick
- Elastic behavior verified at BSE-C

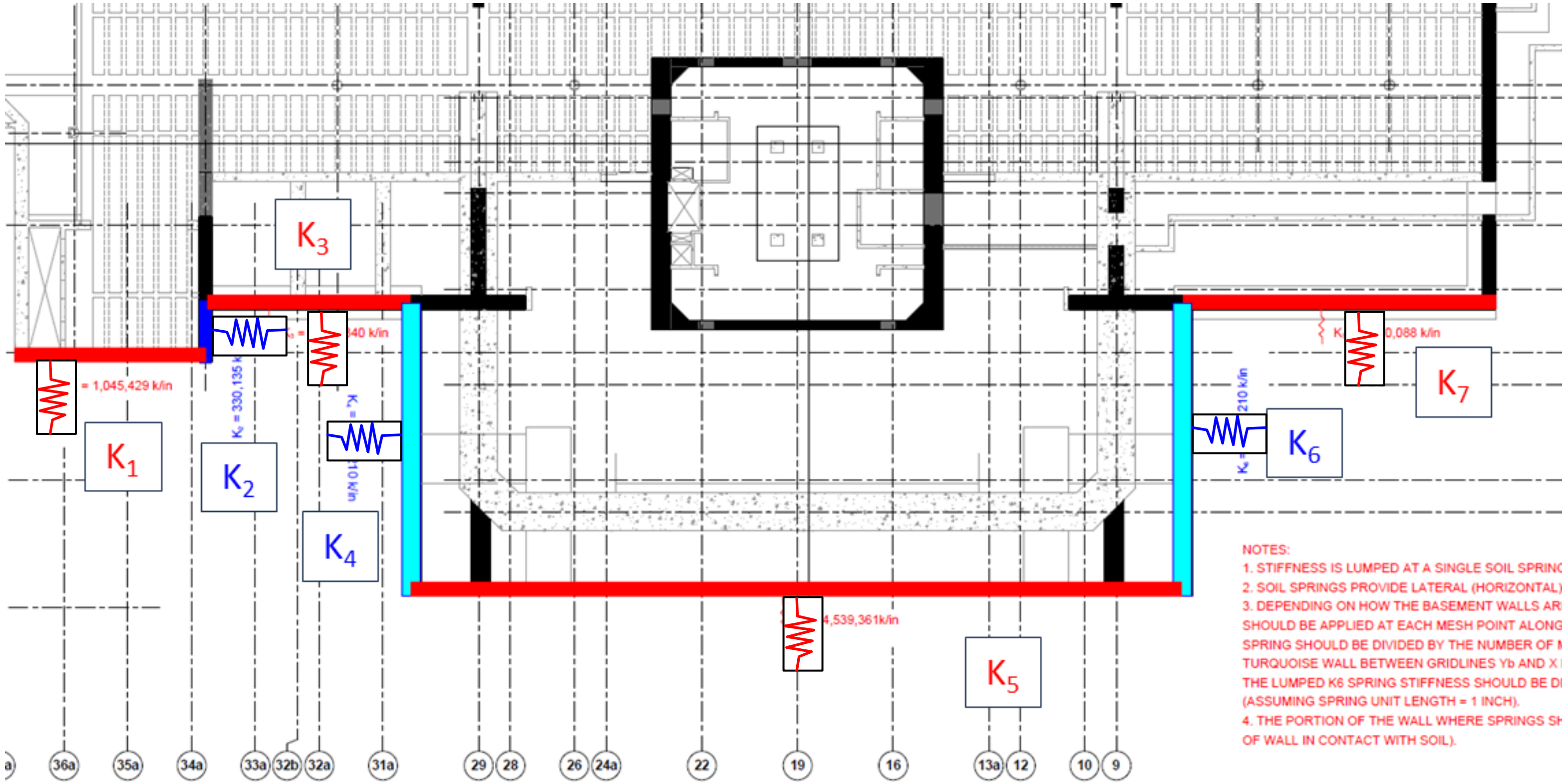




OpenSees Modeling: Subgrade Soil Springs

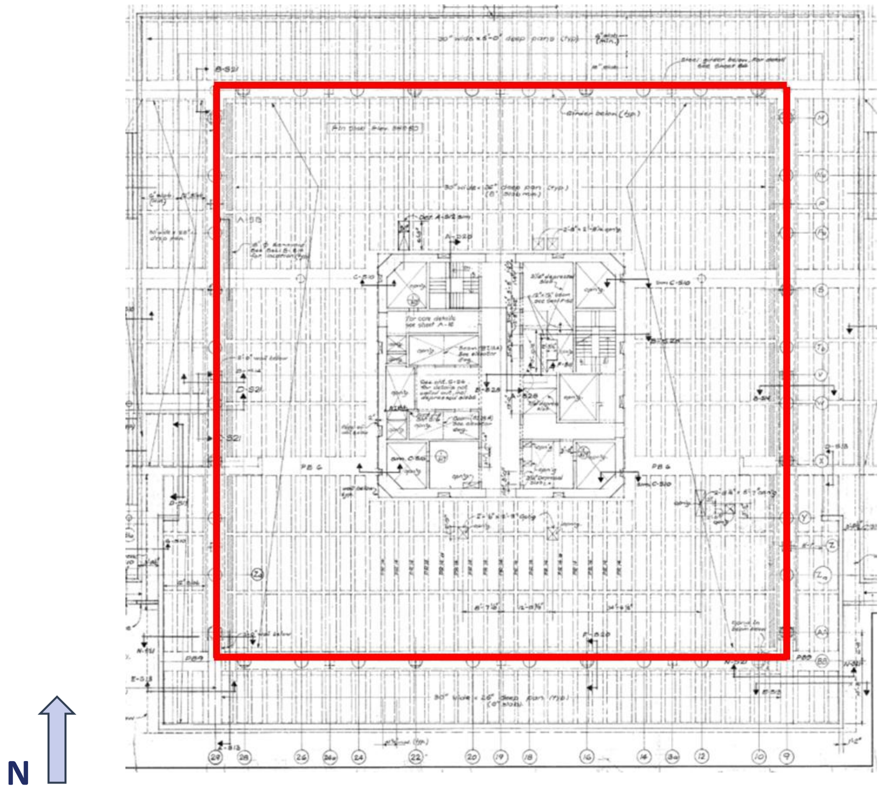
- Soil springs included on the perimeter of uphill walls (Based on previous SSI work in 2019)
- Elastic (Baseline) and nonlinear spring cases analyzed
- Total stiffness/strength discretized into tributary wall node locations (not shown)

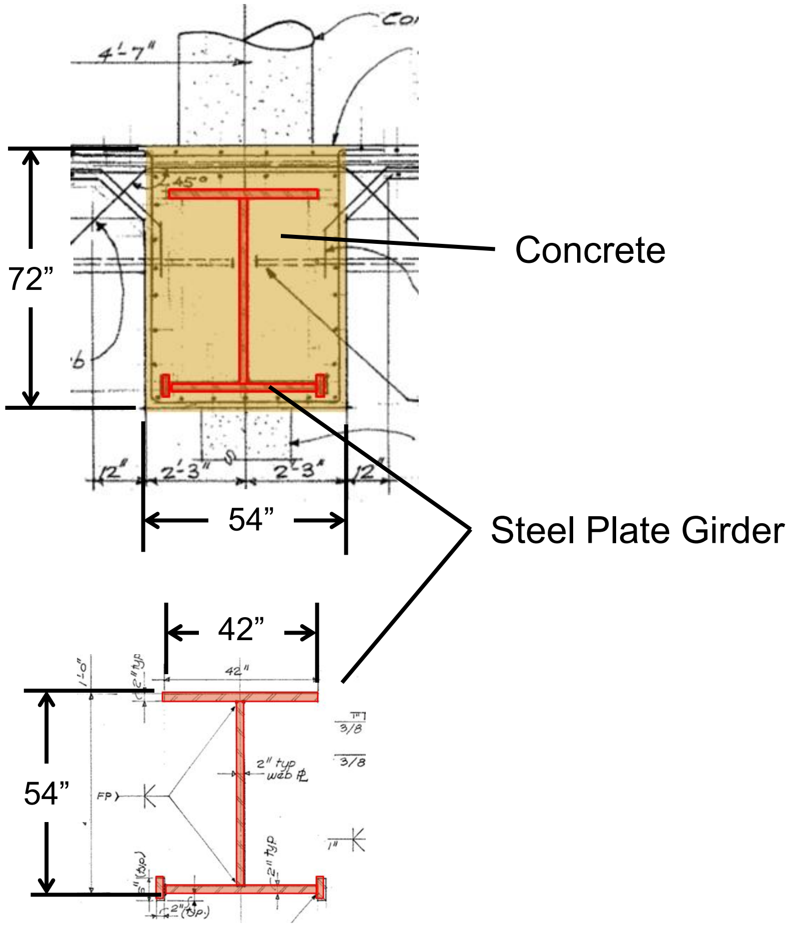
	K (kip/in)	F_Y (kip)
K_1	1,045,429	27,585
K_2	330,135	8,711
K_3	1,134,840	29,945
K_4	1,733,210	45,734
K_5	4,539,361	119,779
K_6	1,733,210	45,734
K_7	1,740,088	45,915



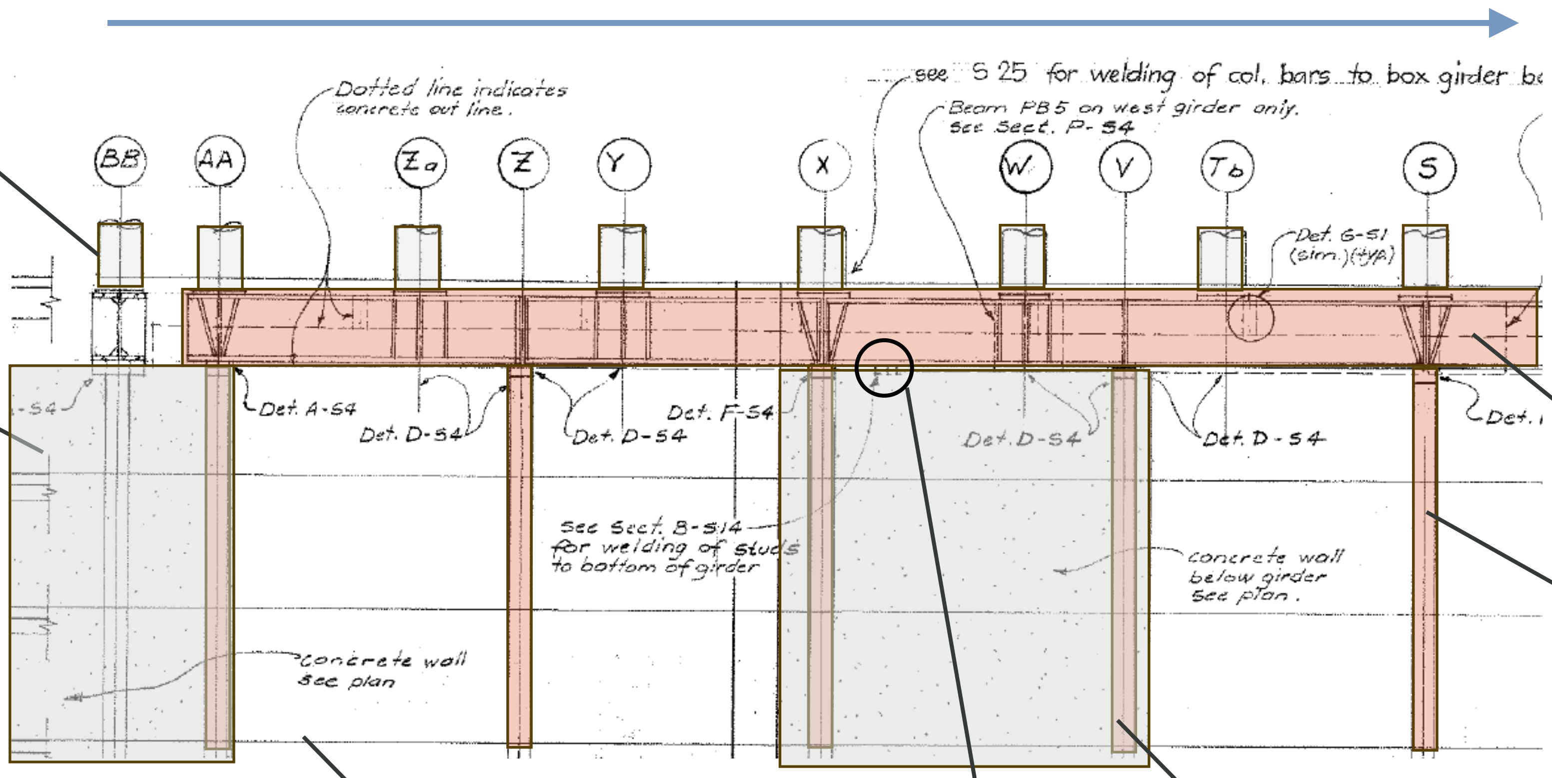
OpenSees Modeling: Transfer Girder

- Large built-up steel section embedded in lightly reinforced concrete beam (Plaza level)
- Transfers loads from circular tower columns to steel basement column layout
- Modeled as elastic with steel girder properties only





OpenSees Modeling: Transfer Girder



Circular tower RC column

Basement wall

(Only Plaza Level to Level C shown)

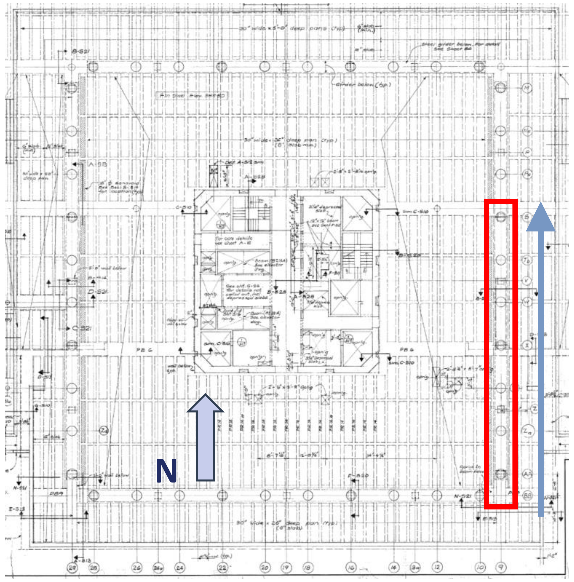
Transfer girder

Steel column

Sub-grade slab beam (not shown in drawing)

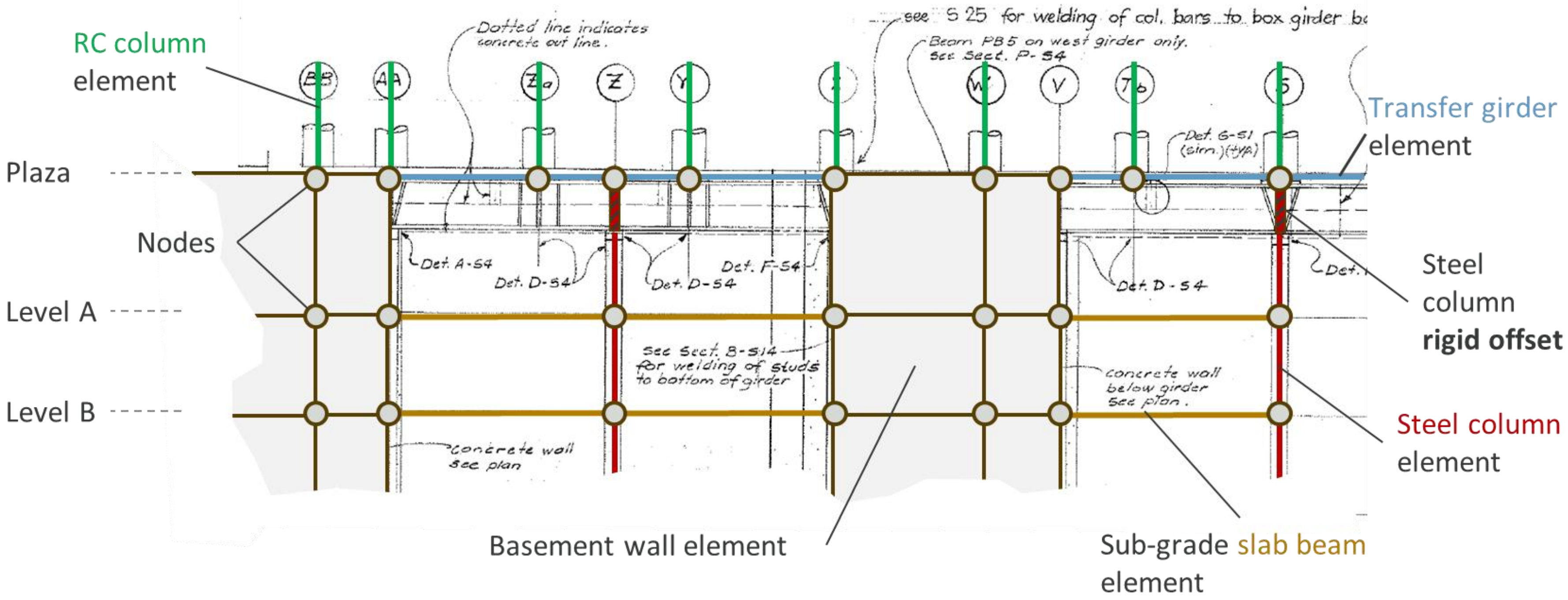
Walls connected to girder via shear studs

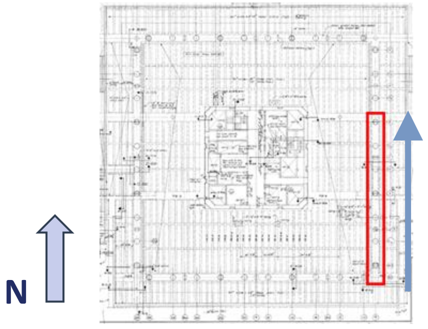
Steel column embedded in basement wall



OpenSees Modeling: Transfer Girder

- "Simple" transfer girder assumes continuous section around perimeter (conservative for tower demands)

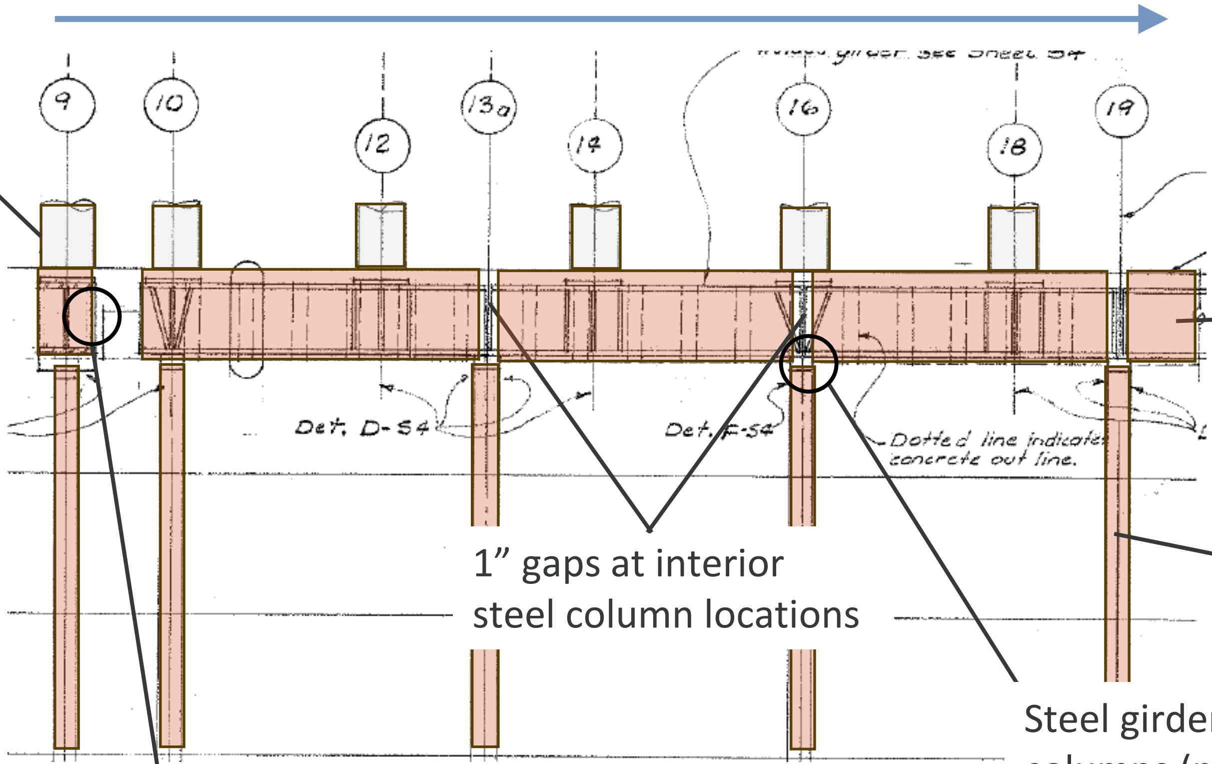




- Transfer girder elements are elastic and consider the steel plate section only
- Sub-grade steel columns have rigid offsets at plaza level to represent transfer girder depth
- Lower levels (Level A and below) model sub-grade frames as centerline elements

OpenSees Modeling: Transfer Girder

Circular tower RC column



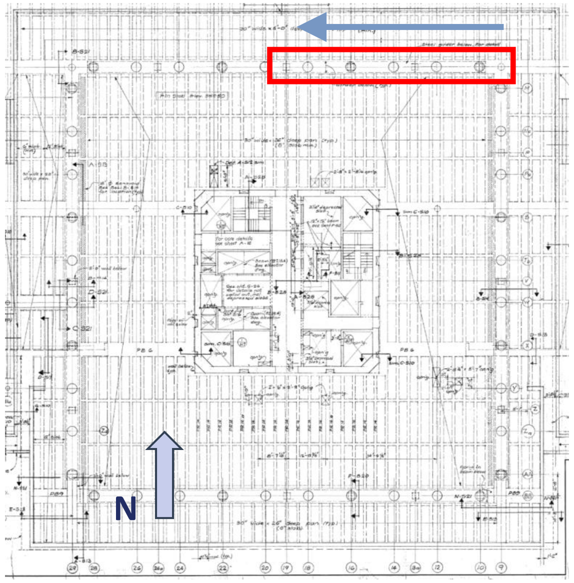
Transfer girder section

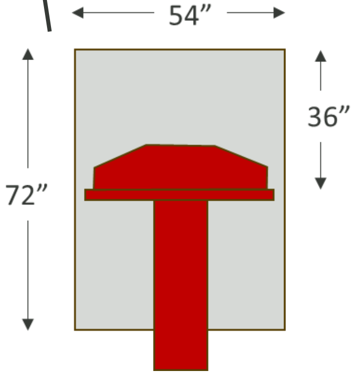
1” gaps at interior steel column locations

Steel column

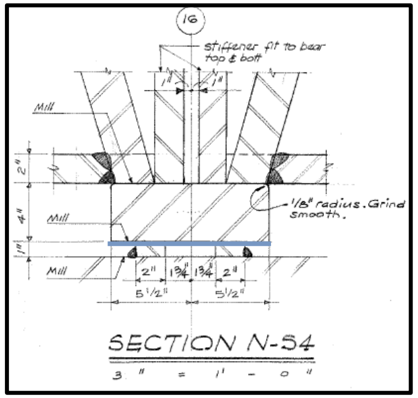
Steel girders are only bearing on columns (no welds)

(Only Plaza Level to Level C shown)



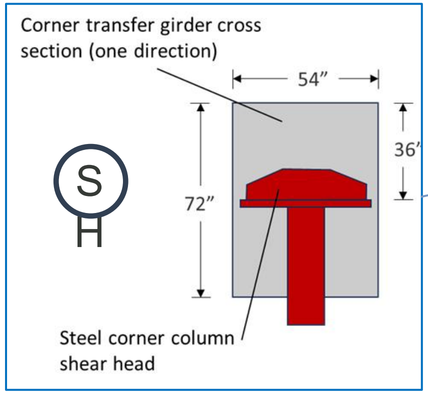


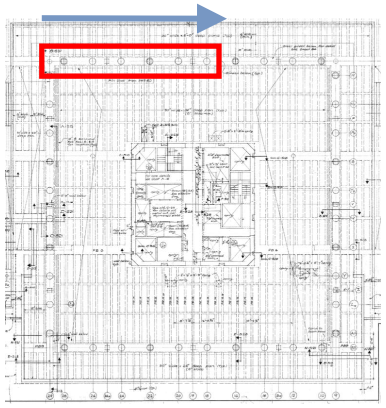
Shear heads embedded in North corners to provide uplift resistance



OpenSees Modeling (Verification Example)

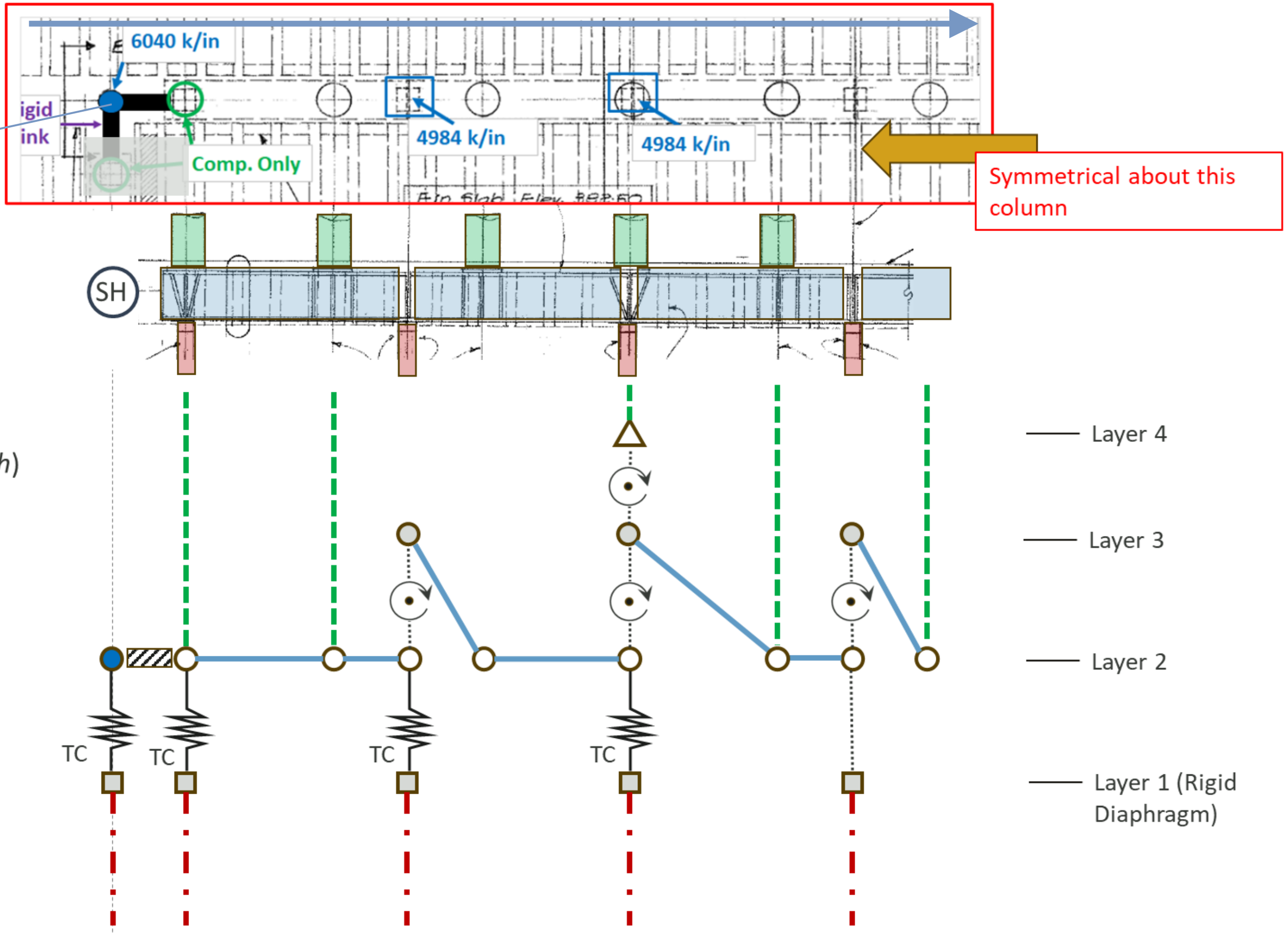
- "Explicit" version of transfer girder used to verify uplift demands on north tower corners
- Uplift springs and moment releases added to model corner shear heads and transfer girder gaps





** Note: Vertical direction of schematic is to show connectivity (coincidental in plan)*

= Rigid element
= Trans. girder element
= RC column element
= Steel column element
= Moment release (*zeroLength*)
TC = Tension/Comp. spring (*zeroLength*)
= Rigid dia. node (ID = 1)
= Layer 2 node (ID = 2)
= Layer 3 node (ID = 3)
= Layer 4 node (ID = 4)



Overview

How can we take a closer look at the ACC Building?

Material Study 1: RC core walls

- Shear strength
- Shear-flexure response
- Hysteretic properties

Material Study 2: Circular tower columns

- Shear strength
- Backbone curve
- Hysteretic properties

Nonlinear Modeling in *OpenSees*

- Modeling progression
- Sub-grade modeling
- Model variations

Nonlinear Analysis

- Acceptance criteria
- Ground motions
- FEMA P695 parameter studies
- Primary results
- Additional studies and analysis

Conclusions

Nonlinear Analysis: Acceptance Criteria

Primary Objective: Estimate the probability of collapse at the BSE-C (975-yr intensity)

Collapse Definition (Unacceptable Response):

- Sidesway instability
- Loss of vertical load carrying capacity (LVLCC)
- Analysis fails to converge
- Loads on force-controlled element(s) exceeds capacity

Conservatively taken as a 2% drift ratio threshold

Verified via post-processing (OpenSees, 2019 Perform3D)

Method: "Modified FEMA P695 Analysis"

- P695 represents accepted concepts for IDA and collapse assessment (uncertainty, spectral shape)
- Original acceptance criteria developed for an entire design space broke into performance groups
- **Best Estimate Models: P[C|975yr] ≤ 10%** (similar to the average of a performance group)
- **Conservative Models: P[C|975yr] ≤ 20%** (similar to the worst performer in a performance group)

Additional Analysis Methods:

- Nonlinear Static (Pushover) and Dynamic (NLRHA) at the BSE-C

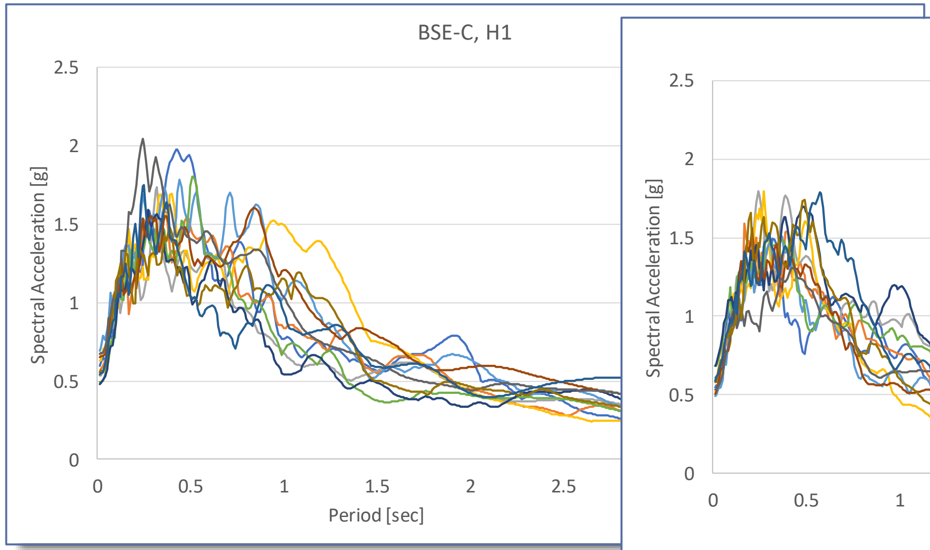
Nonlinear Analysis: Ground Motions

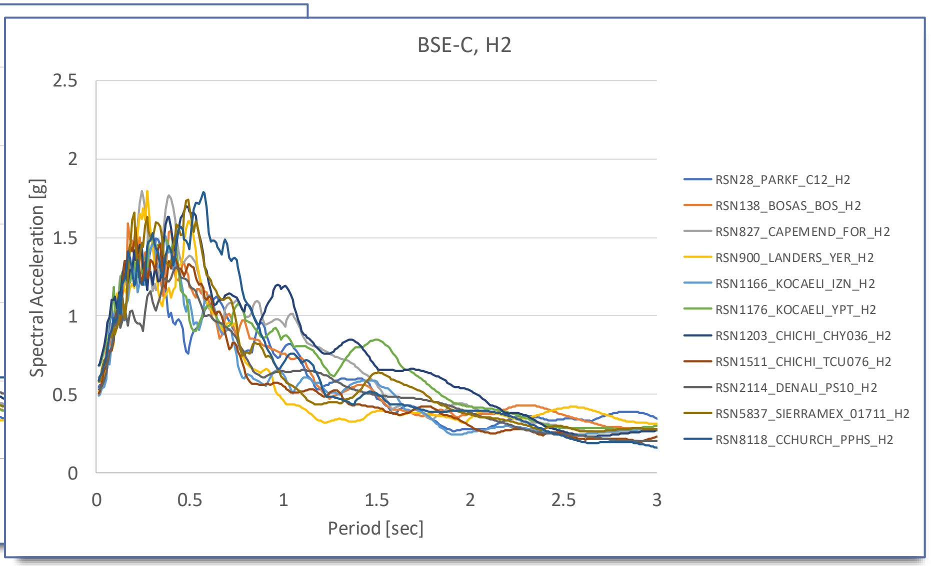
Intensity Measure at BSE-C:

- Sa(T=0.75s) = 1.09g

Site-Specific Set:

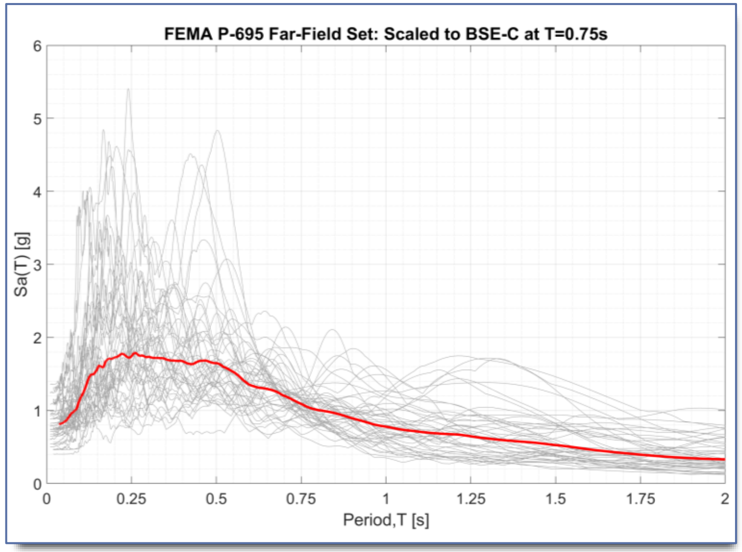
- From 2019 study (Part I)
- Primary GMs for collapse assessment
- Run as 22 horizontal pairs (each orientation)
- Spectrally matched to UHS

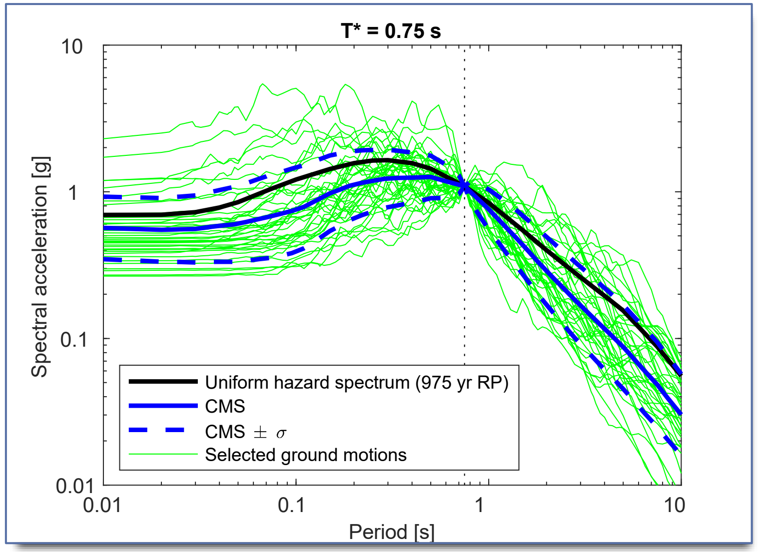




P695 Parametric Sets:

- P695 Far-Field Set (22 pairs)
- Conditional Mean Spectrum set (T* = 0.75s, ε = 1.08, M = 7.6, R = 12 km)
- CMS set has 40 GMs total

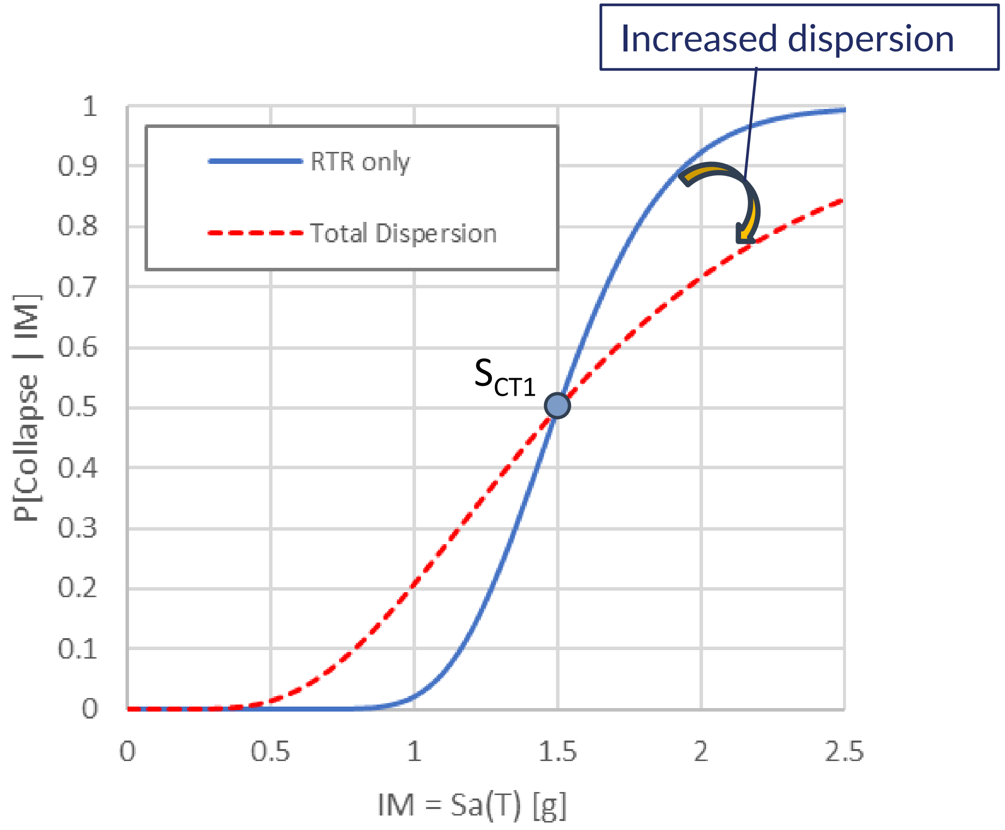




Nonlinear Analysis: P-695 Parameter Studies

Key Parameters for P695 Fragility Adjustment

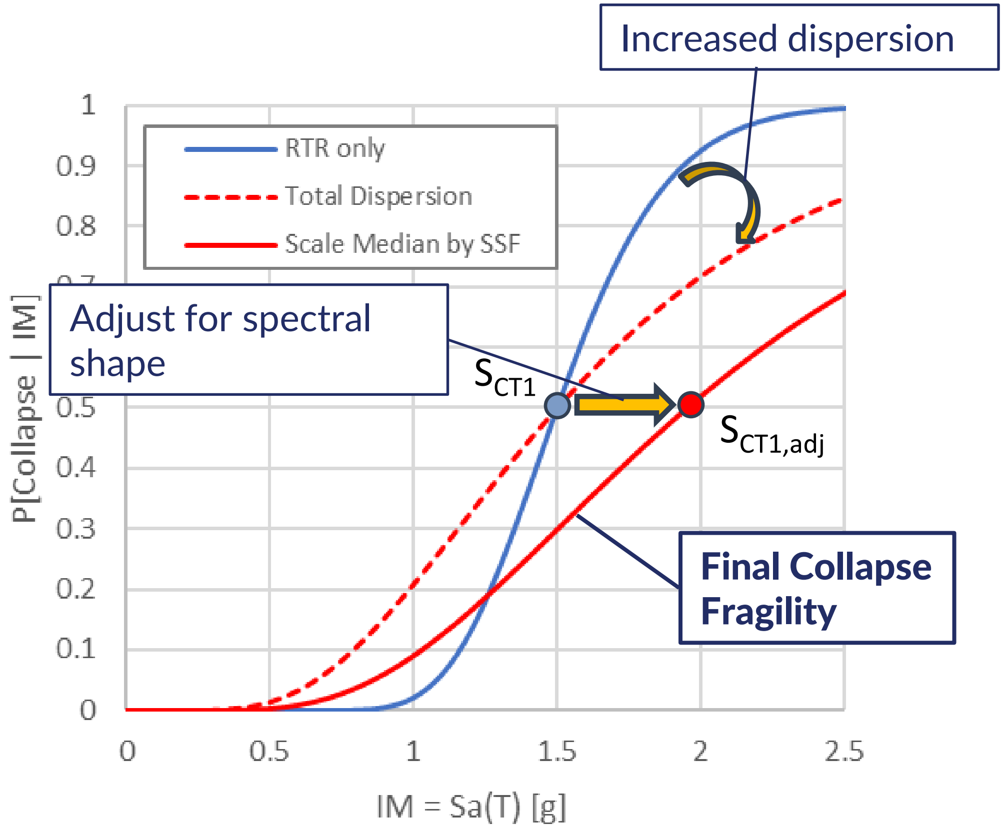
- Median collapse intensity (S_{CT1})
- Total system dispersion (β_{TOT})
- Spectral Shape Factor (SSF)
- Adjusted median capacity ($S_{CT1,adj}$)



Nonlinear Analysis: P-695 Parameter Studies

Key Parameters for P695 Fragility Adjustment

- Median collapse intensity (S_{CT1})
- Total system dispersion (β_{TOT})
- Spectral Shape Factor (SSF)
- Adjusted median capacity ($S_{CT1,adj}$)



Nonlinear Analysis: P-695 Parameter Studies

Record to Record Uncertainty (β_{RTR})

- P695 limits: $0.2 \leq \beta_{RTR} \leq 0.4$ (based on ductility)
- Spectrally matched BSE-C records → < 0.20
- P695 Far-Field Set → 0.25 to 0.30
- CMS ground motions → 0.25
- **We select β_{RTR} = 0.3**

* Each result is from a full IDA on a tower only model

Total System Dispersion (β_{TOT})

- Additional uncertainty sources consider single building analysis (as opposed to a design space)
- **We select β_{TOT} = 0.4**

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} = 0.39 \approx \mathbf{0.4}$$

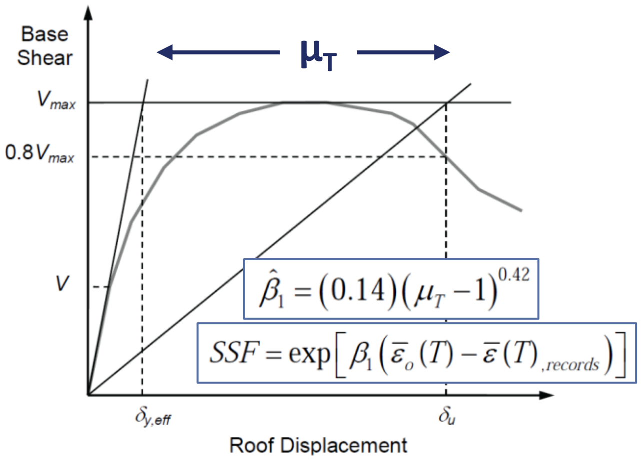
(0.3) (0.1) (0.2) (0.1)

Nonlinear Analysis: P-695 Parameter Studies

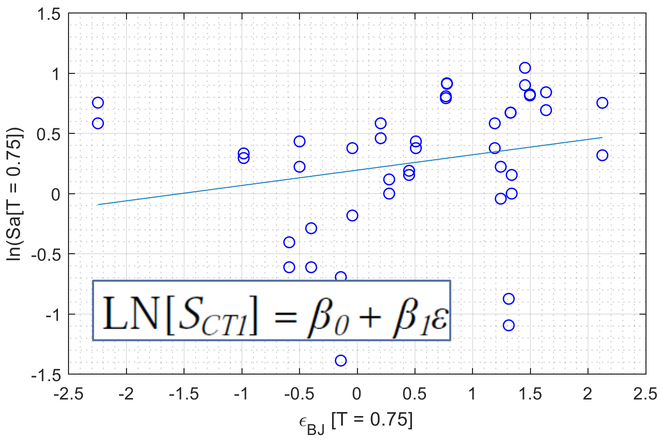
Spectral Shape Factor (SSF)

- Three methods considered using underlying techniques behind P695
- Ductility (pushover) Method → 1.12 to 1.30
- Epsilon Regression (P695 Set) → 1.08 to 1.27
- CMS versus UHS Ground Motions → 1.20 to 1.30
- **We select SSF = 1.15**

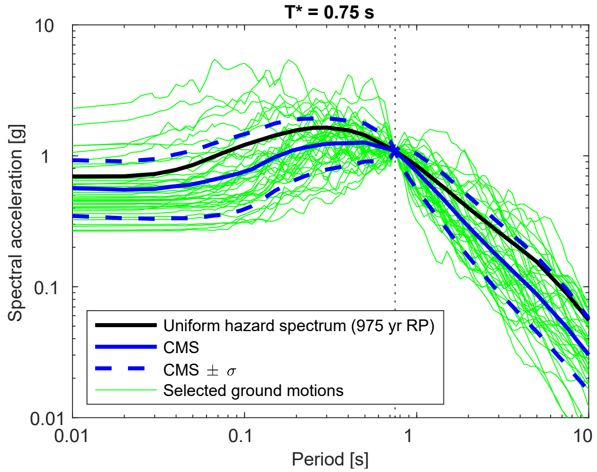
Ductility Method



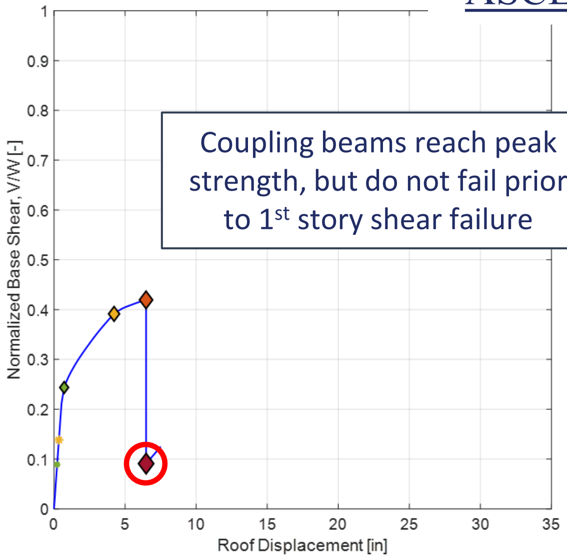
Epsilon Regression



CMS vs. UHS GMs

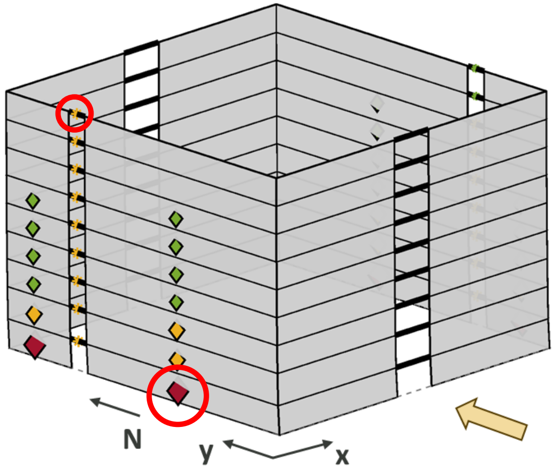


Nonlinear Analysis Results

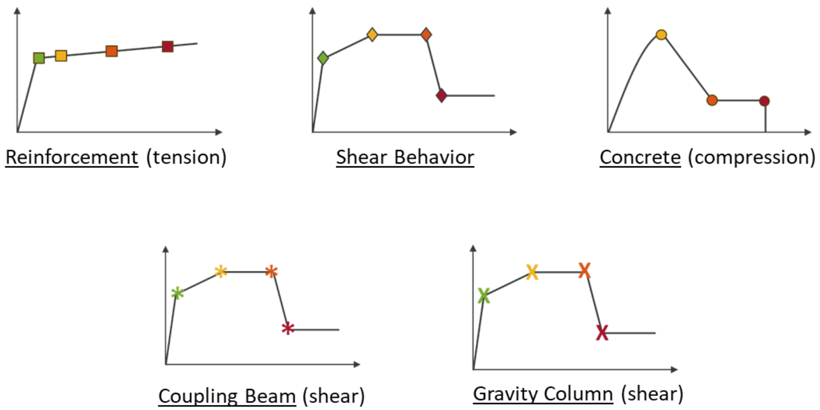


Coupling beams reach peak strength, but do not fail prior to 1st story shear failure

ASCE 41-17 Core Only (2019 inputs)

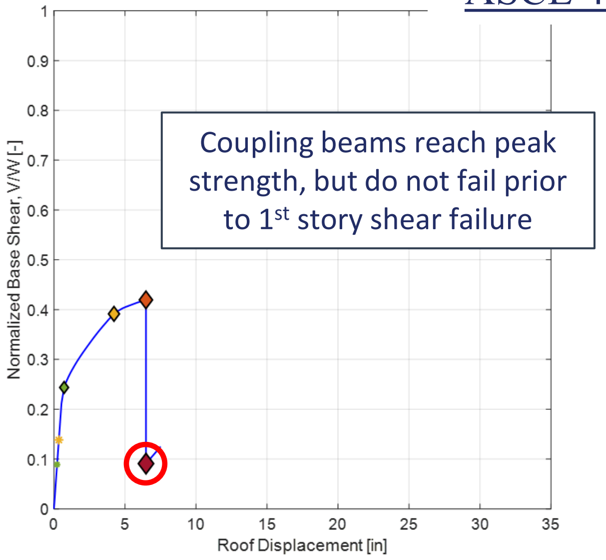


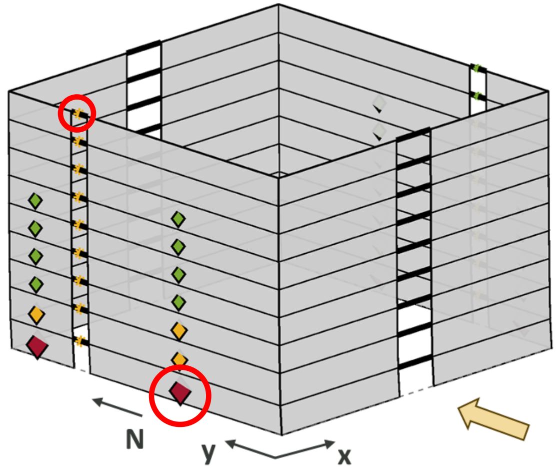
*1st Mode Pushover in North-South Direction at last point of pushover



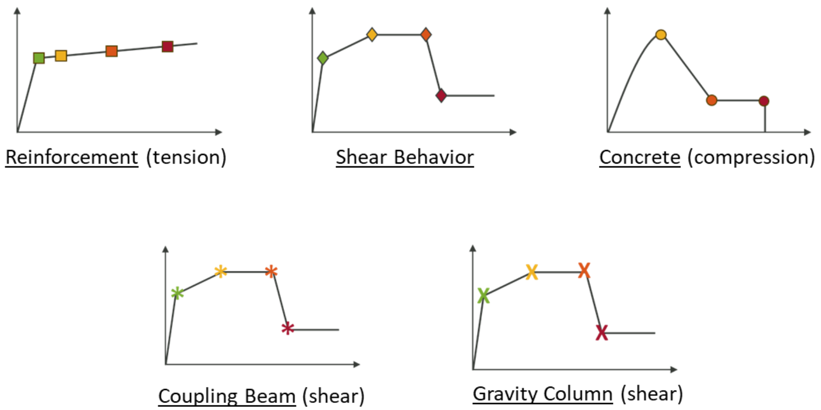
Nonlinear Analysis Results

ASCE 41-17 Core Only (2019 inputs)

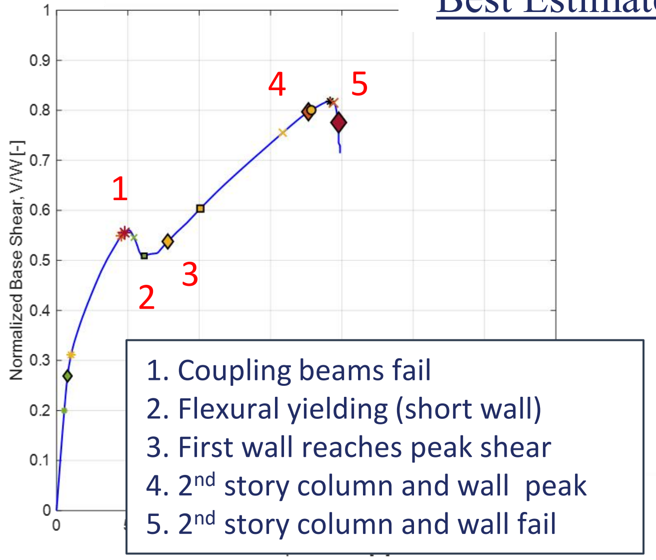


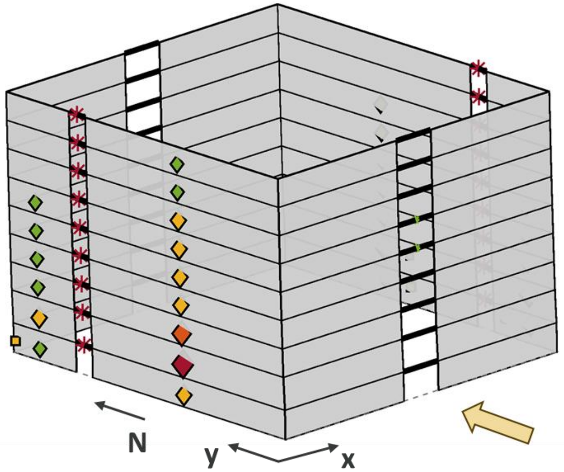


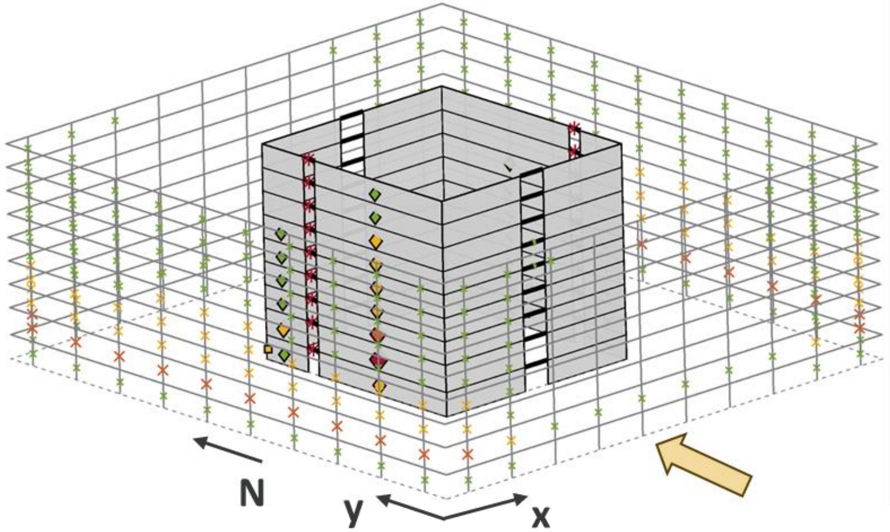
*1st Mode Pushover in North-South Direction at last point of pushover



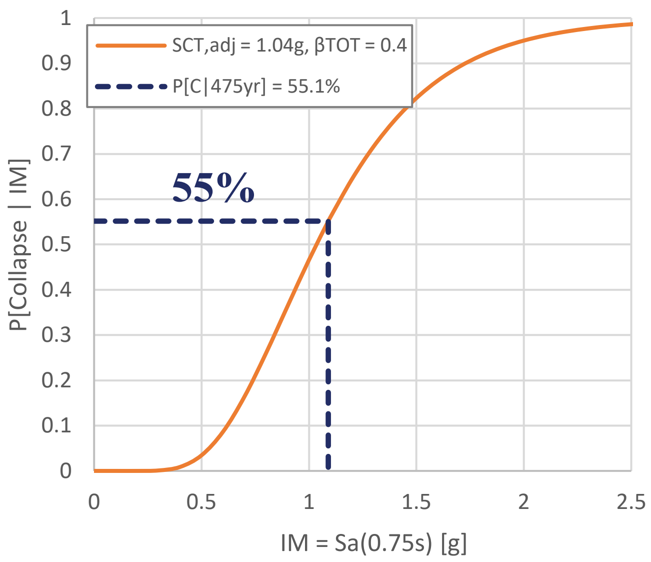
Best Estimate Tower Only Model

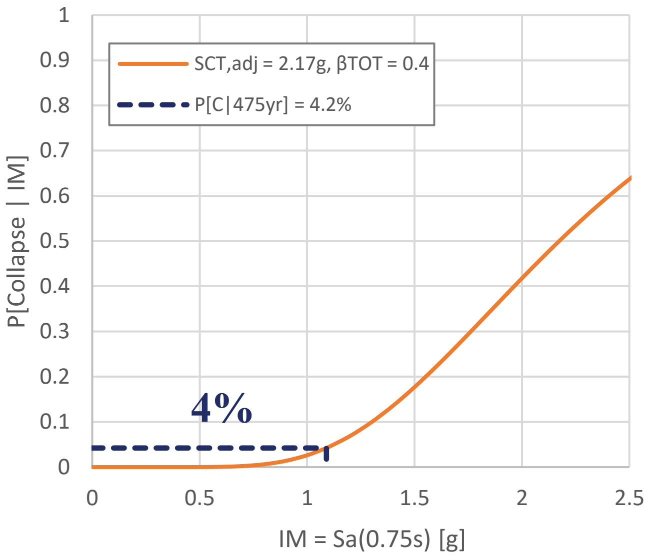


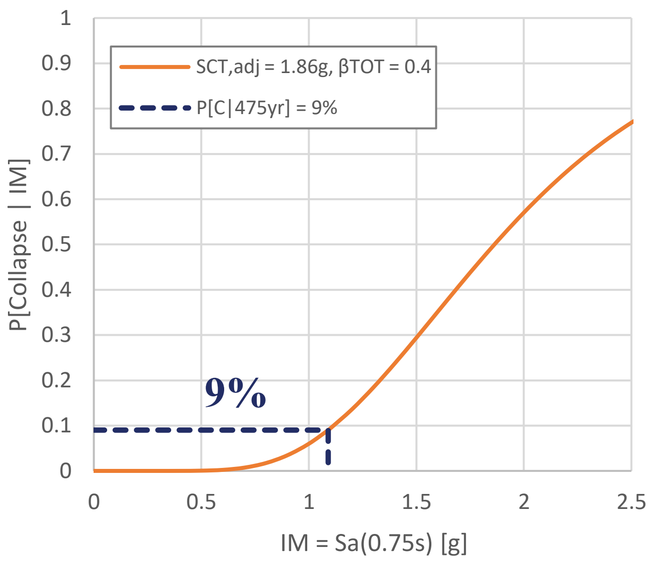




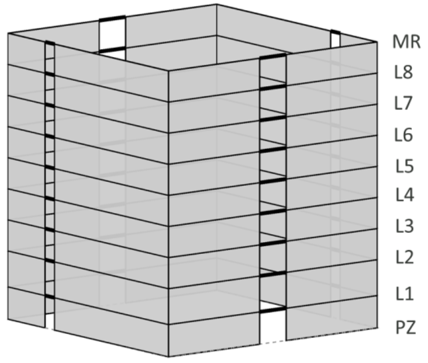
Nonlinear Analysis: Results



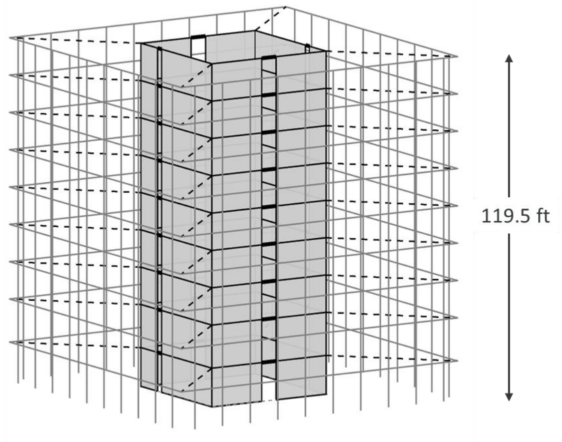




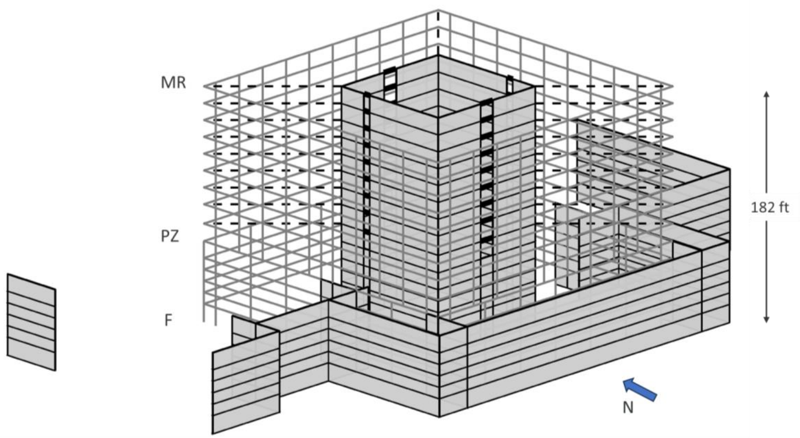
ASCE 41-17 Core Only (2019 inputs)



Best Estimate Tower Only



Best Estimate Tower plus Subgrade



Nonlinear Analysis Results
Tower Only Models

Model Variation	P[C|975yr]	NL RHA	Pushover
Nonlinear versus Elastic Flexure:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Worst-case elastic flexure (v.2.7.2)	11%	Pass	Pass
Fixed versus Pinned Gravity Column Bases:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Pinned-base gravity columns (v.2.7.5)	8%	Pass	Pass
Spectral Shape for Baseline (v.2.7.1, NL flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	9%	Pass	Pass
Spectral Shape Factor = 1.1	5%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	4%	Pass	Pass
Spectral Shape Factor = 1.2	3%	Pass	Pass
Spectral Shape Factor = 1.3	2%	Pass	Pass
Spectral Shape for Worst-Case (v.2.7.2, worst-case elastic flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	14%	Pass	Pass
Spectral Shape Factor = 1.05	11%	Pass	Pass
Spectral Shape Factor = 1.1	9%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	7%	Pass	Pass
Effects of Uncertainties for Baseline (v.2.7.1, NL flexure):	--	--	--
$\beta_{Total} = 0.30$	1%	Pass	Pass
$\beta_{Total} = 0.40$ (baseline)	4%	Pass	Pass
$\beta_{Total} = 0.50$	9%	Pass	Pass
$\beta_{Total} = 0.60$	13%	Pass	Pass
$\beta_{Total} = 0.70$	16%	Pass	Pass
$\beta_{Total} = 0.80$	20%	Pass	Pass

- **Baseline model returned P[C|975yr] = 4%**

Nonlinear Analysis Results
Tower Only Models

Model Variation	P[C|975yr]	NL RHA	Pushover
Nonlinear versus Elastic Flexure:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Worst-case elastic flexure (v.2.7.2)	11%	Pass	Pass
Fixed versus Pinned Gravity Column Bases:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Pinned-base gravity columns (v.2.7.5)	8%	Pass	Pass
Spectral Shape for Baseline (v.2.7.1, NL flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	9%	Pass	Pass
Spectral Shape Factor = 1.1	5%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	4%	Pass	Pass
Spectral Shape Factor = 1.2	3%	Pass	Pass
Spectral Shape Factor = 1.3	2%	Pass	Pass
Spectral Shape for Worst-Case (v.2.7.2, worst-case elastic flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	14%	Pass	Pass
Spectral Shape Factor = 1.05	11%	Pass	Pass
Spectral Shape Factor = 1.1	9%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	7%	Pass	Pass
Effects of Uncertainties for Baseline (v.2.7.1, NL flexure):	--	--	--
β_{Total} = 0.30	1%	Pass	Pass
β_{Total} = 0.40 (baseline)	4%	Pass	Pass
β_{Total} = 0.50	9%	Pass	Pass
β_{Total} = 0.60	13%	Pass	Pass
β_{Total} = 0.70	16%	Pass	Pass
β_{Total} = 0.80	20%	Pass	Pass

- **Baseline model returned P[C|975yr] = 4%**
- Enforcing elastic flexure resulted in the worst tower only performance with SSF = 1.15 and β_{TOT} = 0.4 (P[C|975yr = 11%)

Nonlinear Analysis Results
Tower Only Models

Model Variation	P[C|975yr]	NL RHA	Pushover
Nonlinear versus Elastic Flexure:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Worst-case elastic flexure (v.2.7.2)	11%	Pass	Pass
Fixed versus Pinned Gravity Column Bases:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Pinned-base gravity columns (v.2.7.5)	8%	Pass	Pass
Spectral Shape for Baseline (v.2.7.1, NL flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	9%	Pass	Pass
Spectral Shape Factor = 1.1	5%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	4%	Pass	Pass
Spectral Shape Factor = 1.2	3%	Pass	Pass
Spectral Shape Factor = 1.3	2%	Pass	Pass
Spectral Shape for Worst-Case (v.2.7.2, worst-case elastic flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	14%	Pass	Pass
Spectral Shape Factor = 1.05	11%	Pass	Pass
Spectral Shape Factor = 1.1	9%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	7%	Pass	Pass
Effects of Uncertainties for Baseline (v.2.7.1, NL flexure):	--	--	--
β_{Total} = 0.30	1%	Pass	Pass
β_{Total} = 0.40 (baseline)	4%	Pass	Pass
β_{Total} = 0.50	9%	Pass	Pass
β_{Total} = 0.60	13%	Pass	Pass
β_{Total} = 0.70	16%	Pass	Pass
β_{Total} = 0.80	20%	Pass	Pass

- **Baseline model returned P[C|975yr] = 4%**
- Enforcing elastic flexure resulted in the worst tower only performance with SSF = 1.15 and β_{TOT} = 0.4 (P[C|975yr = 11%)
- Pinned column bases increased P[C|975yr] to 8%

Nonlinear Analysis Results
Tower Only Models

Model Variation	P[C|975yr]	NL RHA	Pushover
Nonlinear versus Elastic Flexure:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Worst-case elastic flexure (v.2.7.2)	11%	Pass	Pass
Fixed versus Pinned Gravity Column Bases:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Pinned-base gravity columns (v.2.7.5)	8%	Pass	Pass
Spectral Shape for Baseline (v.2.7.1, NL flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	9%	Pass	Pass
Spectral Shape Factor = 1.1	5%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	4%	Pass	Pass
Spectral Shape Factor = 1.2	3%	Pass	Pass
Spectral Shape Factor = 1.3	2%	Pass	Pass
Spectral Shape for Worst-Case (v.2.7.2, worst-case elastic flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	14%	Pass	Pass
Spectral Shape Factor = 1.05	11%	Pass	Pass
Spectral Shape Factor = 1.1	9%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	7%	Pass	Pass
Effects of Uncertainties for Baseline (v.2.7.1, NL flexure):	--	--	--
β_{Total} = 0.30	1%	Pass	Pass
β_{Total} = 0.40 (baseline)	4%	Pass	Pass
β_{Total} = 0.50	9%	Pass	Pass
β_{Total} = 0.60	13%	Pass	Pass
β_{Total} = 0.70	16%	Pass	Pass
β_{Total} = 0.80	20%	Pass	Pass

- **Baseline model returned P[C|975yr] = 4%**
- Enforcing elastic flexure resulted in the worst tower only performance with SSF = 1.15 and β_{TOT} = 0.4 (P[C|975yr = 11%)
- Pinned column bases increased P[C|975yr] to 8%
- Removing all benefit for spectral shape adjustment (SSF = 1.0) increased P[C|975yr] to 9%

Nonlinear Analysis Results
Tower Only Models

Model Variation	P[C|975yr]	NL RHA	Pushover
Nonlinear versus Elastic Flexure:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Worst-case elastic flexure (v.2.7.2)	11%	Pass	Pass
Fixed versus Pinned Gravity Column Bases:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Pinned-base gravity columns (v.2.7.5)	8%	Pass	Pass
Spectral Shape for Baseline (v.2.7.1, NL flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	9%	Pass	Pass
Spectral Shape Factor = 1.1	5%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	4%	Pass	Pass
Spectral Shape Factor = 1.2	3%	Pass	Pass
Spectral Shape Factor = 1.3	2%	Pass	Pass
Spectral Shape for Worst-Case (v.2.7.2, worst-case elastic flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	14%	Pass	Pass
Spectral Shape Factor = 1.05	11%	Pass	Pass
Spectral Shape Factor = 1.1	9%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	7%	Pass	Pass
Effects of Uncertainties for Baseline (v.2.7.1, NL flexure):	--	--	--
$\beta_{Total} = 0.30$	1%	Pass	Pass
$\beta_{Total} = 0.40$ (baseline)	4%	Pass	Pass
$\beta_{Total} = 0.50$	9%	Pass	Pass
$\beta_{Total} = 0.60$	13%	Pass	Pass
$\beta_{Total} = 0.70$	16%	Pass	Pass
$\beta_{Total} = 0.80$	20%	Pass	Pass

- **Baseline model returned P[C|975yr] = 4%**
- Enforcing elastic flexure resulted in the worst tower only performance with SSF = 1.15 and $\beta_{TOT} = 0.4$ (P[C|975yr = 11%)
- Pinned column bases increased P[C|975yr] to 8%
- Removing all benefit for spectral shape adjustment (SSF = 1.0) increased P[C|975yr] to 9%
- Combining elastic flexure and SSF = 1.0 returned P[C|975yr] = 14%

Nonlinear Analysis Results
Tower Only Models

Model Variation	P[C|975yr]	NL RHA	Pushover
Nonlinear versus Elastic Flexure:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Worst-case elastic flexure (v.2.7.2)	11%	Pass	Pass
Fixed versus Pinned Gravity Column Bases:	--	--	--
Baseline (v.2.7.1)	4%	Pass	Pass
Pinned-base gravity columns (v.2.7.5)	8%	Pass	Pass
Spectral Shape for Baseline (v.2.7.1, NL flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	9%	Pass	Pass
Spectral Shape Factor = 1.1	5%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	4%	Pass	Pass
Spectral Shape Factor = 1.2	3%	Pass	Pass
Spectral Shape Factor = 1.3	2%	Pass	Pass
Spectral Shape for Worst-Case (v.2.7.2, worst-case elastic flexure):	--	--	--
Spectral Shape Factor = 1.0 (no benefit)	14%	Pass	Pass
Spectral Shape Factor = 1.05	11%	Pass	Pass
Spectral Shape Factor = 1.1	9%	Pass	Pass
Spectral Shape Factor = 1.15 (baseline)	7%	Pass	Pass
Effects of Uncertainties for Baseline (v.2.7.1, NL flexure):	--	--	--
β_{Total} = 0.30	1%	Pass	Pass
β_{Total} = 0.40 (baseline)	4%	Pass	Pass
β_{Total} = 0.50	9%	Pass	Pass
β_{Total} = 0.60	13%	Pass	Pass
β_{Total} = 0.70	16%	Pass	Pass
β_{Total} = 0.80	20%	Pass	Pass

- **Baseline model returned P[C|975yr] = 4%**
- Enforcing elastic flexure resulted in the worst tower only performance with SSF = 1.15 and β_{TOT} = 0.4 (P[C|975yr = 11%)
- Pinned column bases increased P[C|975yr] to 8%
- Removing all benefit for spectral shape adjustment (SSF = 1.0) increased P[C|975yr] to 9%
- Combining elastic flexure and SSF = 1.0 returned P[C|975yr] = 14%
- Baseline model returns P[C|975yr] > 10% when β_{Total} > 0.5

Nonlinear Analysis Results
Tower plus Subgrade Models

Model Variation	P[C|975yr]	NL RHA	Pushover
With Sub-Grade, elastic and nonlinear soil springs:	--	--	--
Baseline with Sub-Grade (v.3.1.6)	9%	Pass	Pass
With Nonlinear Soil Springs (~same, but more convergence)	10.2%	Pass	Pass
Nonlinear versus elastic flexure:	--	--	--
Baseline with Sub-Grade (v.3.1.6)	9%	Pass	Pass
Worst-case elastic flexure	19.4%	Pass	Pass

- **Baseline model returned P[C|975yr] = 9%** (Increase from tower only)
- Nonlinear soil springs had minor effect on collapse performance (mostly due to non-convergence) (P[C|975yr = 10.2%)
- Elastic flexure case was very close to 20% limit
- Adding explicit transfer girder detailing slightly improved performance from baseline (P[C|975yr = 8%) [Not shown in table]

Model Variants and Verification Studies

OpenSees

- Effect of pushover loading pattern on tower only models
- Effect of pinned column bases in tower only models
- Effect of elastic flexural response in core walls
- Effect of nonlinear (vs. elastic) soil springs in sub-grade models
- **Verification of transfer girder force and uplift demands**
- **Verification of deformation demands on North frame in garage**
- Evaluation of wall and column overturning axial force effects
- **Evaluation of tower post-tensioned girders**
- Influence of coupling beam post-peak stiffness on response

Perform3D (and Plaxis)

- Garage (sub-grade) behavior effects on tower
- Hillside effects
- Base slab averaging (SSI)
- Foundation damping (SSI)
- Embedment (SSI)
- **Verification of tower diaphragms**
- **Verification of garage diaphragms**

Conclusions

We used FEMA P695 analysis modified for a single building to estimate the collapse performance of the ACC building at the 975-year return period

- All best estimate models returned P[C|975yr] ≤ 10%
- All conservative models returned P[C|975yr] ≤ 20%
- Additionally, all models were shown to pass ASCE 41-23 criteria for static pushover and NRHA at the 975-year return period

Tower gravity framing and PT girders significantly improved performance from ASCE 41-17 core only

The ACC building is distinguished among other buildings of the same vintage because of:

- Very large, and thick shear walls, with well detailed reinforcement (e.g., 4 layers of longitudinal and considerable shear reinforcement for a 1960's design)
- Substantial gravity system with good confinement tied to core through large PT girders

Based on results and recommendations of the study, the SRC revised the SPR from V to IV

San Diego SEAOC
2025 CONVENTION

Q & A

PDH Code
9NNC5

SEAOC
STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA