Nonlinear Structural Analysis

Designing for Safer Infrastructure
Innovative design that goes beyond the codes
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AIT Asian Institute of Technology
AIT Solutions
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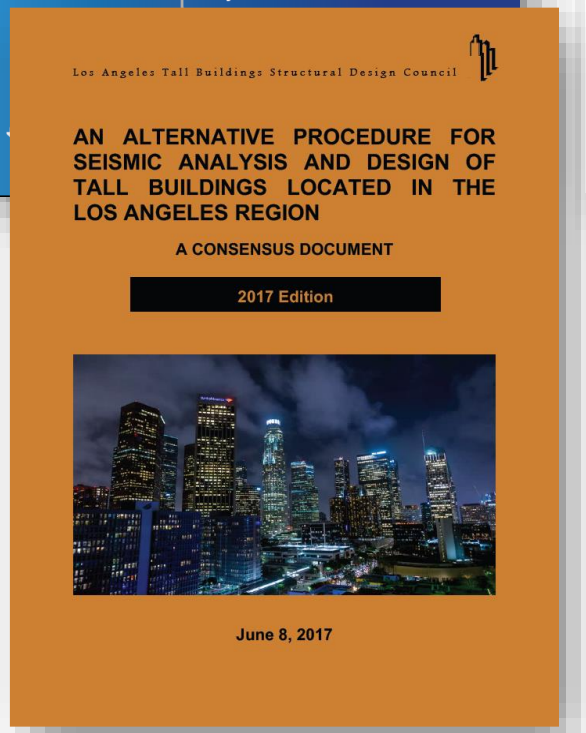
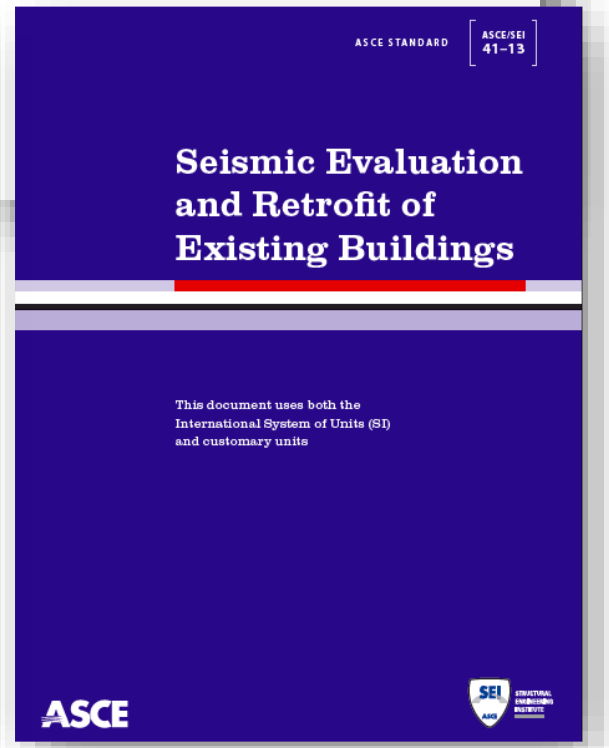
PBD Guidelines

- PEER 2017/06, “Tall Building Initiative, Guidelines for Performance Based Seismic Design of Tall Buildings”
- PEER/ATC 72-1, “Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings”
- ASCE/SEI 41-13, “Seismic Evaluation and Retrofit of Existing Buildings”
- LATBSDC 2017, “An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region”





Classification of Structural Actions

- All actions must be classified as either
  - Deformation-controlled, or
  - Force-controlled

Deformation-controlled action – An action expected to undergo nonlinear behavior in response to earthquake shaking.

Force-controlled action – An action that is not expected to undergo nonlinear behavior in response to earthquake shaking.

Examples of Deformation-controlled Actions

Table C.3.3.2 Zones and actions commonly designated for nonlinear behavior

Structural System	*Zones and Actions*
Special Moment Resisting Frames (steel , concrete, or composite)	• *Flexural yielding of Beam ends (except for transfer girders)* • *Shear in Beam-Column Panel Zones* • *P-M-M* yielding at the base of columns (top of foundation or basement podiums)*
Special Concentric Braced Frames	• *Braces (yielding in tension and buckling in compression)* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Eccentric Braced Frames	• *Shear Link portion of the beams (shear yielding preferred but combined shear and flexural yielding permitted).* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Unbonded Braced Frames	• *Unbonded brace cores (yielding in tension and compression)* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Special Steel-Plate Shear Walls	• *Shear yielding of web plates* • *Flexural yielding of Beam ends*
R/C Shear Walls	• *P-M-M yielding at the base of the walls (top of foundation or basement podiums) and other clearly defined locations throughout the height of the wall.* • *Flexural yielding and/or shear yielding of link beams*
Foundations	• *Controlled rocking* • *Controlled settlement*

* yielding caused by combined axial force and uniaxial or biaxial flexure

Source: 2017 LATBSDC

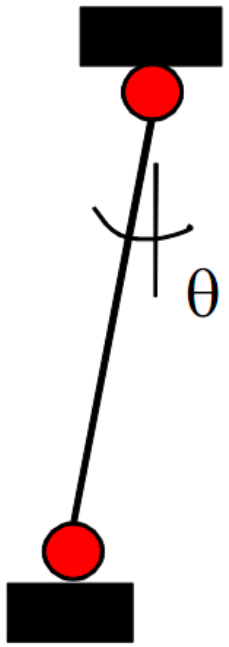
Types of Nonlinear Component Models

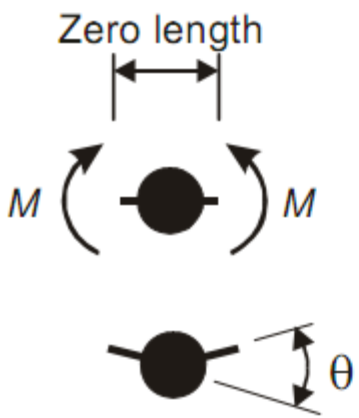
- Typically there are two types of nonlinear component models used in commercial software.

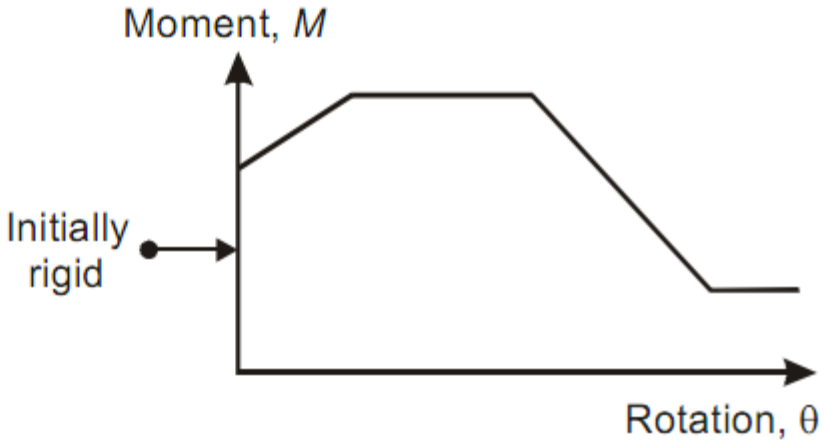
- Concentrated Hinge Component Model

- Fiber-Type Component Model

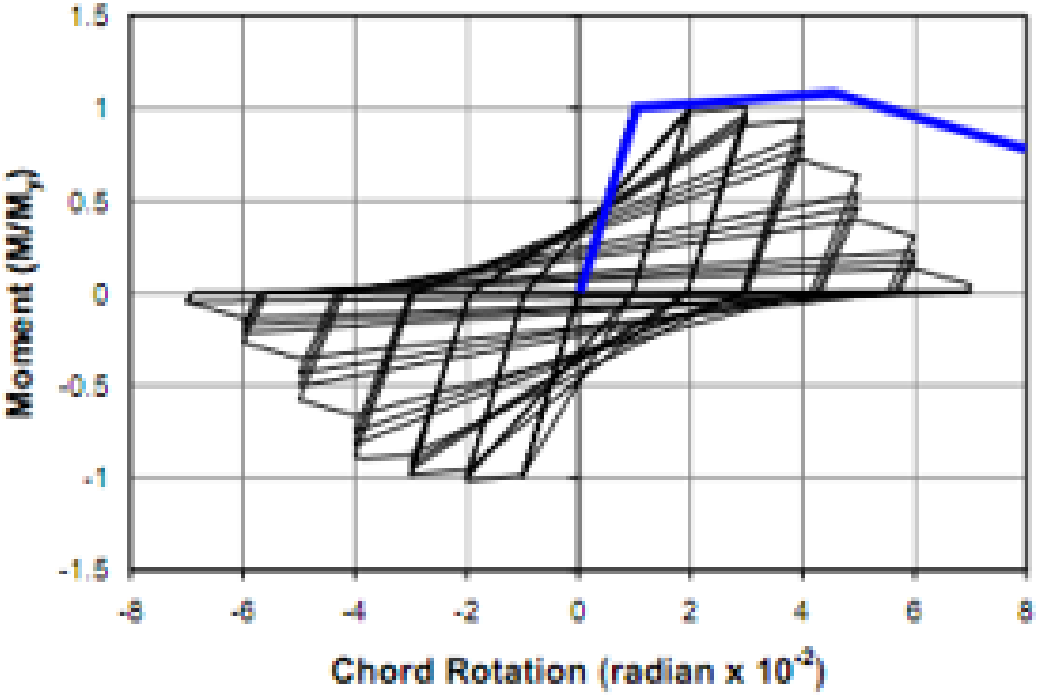
Concentrated Hinge Model

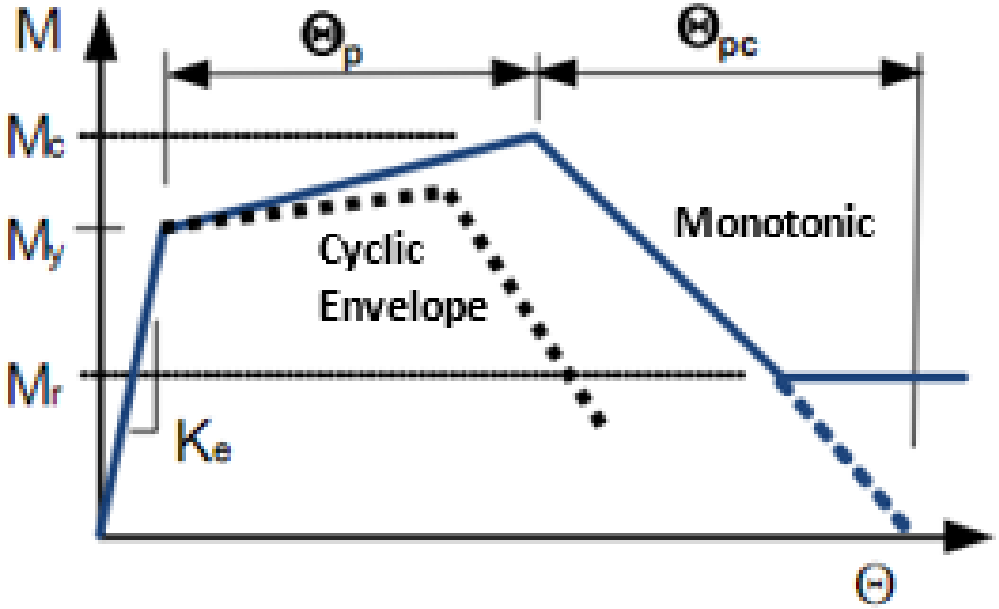






Monotonic versus Cyclic Envelope

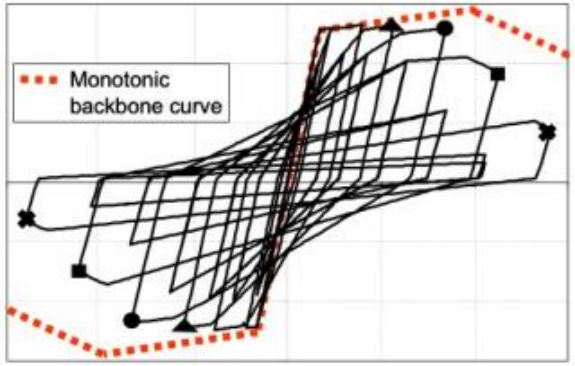




Source: ATC 72-1

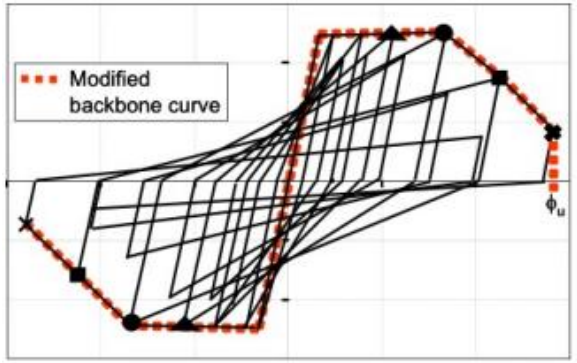
Backbone Curves

Backbone Curves Model Types (PEER TBI & ATC 72-1)



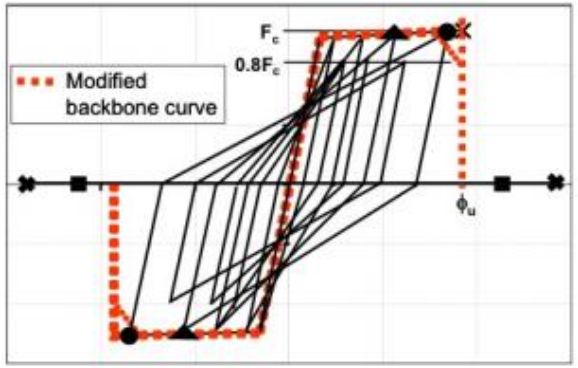
Option 1

Cyclic and in-cycle degradation explicitly modeled during analysis; backbone curve hardens/softens as a function of damage



Option 2

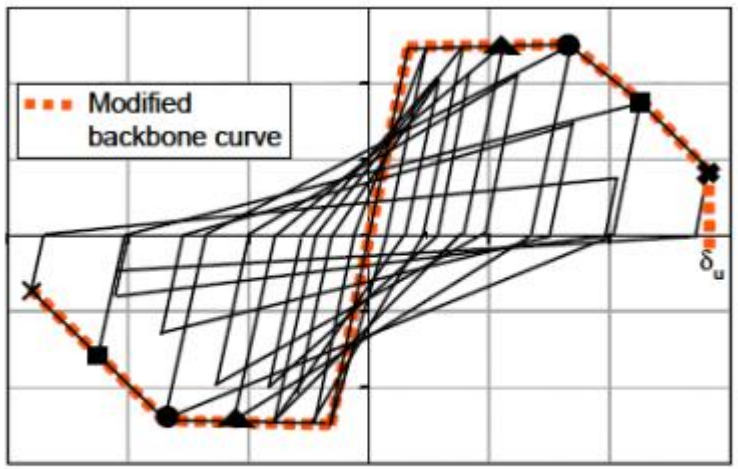
Post-peak capping and cyclic degradation modeled with fixed backbone curve that remains fixed during analysis; backbone curve is defined based on measured(data) or assumed cyclic softening.



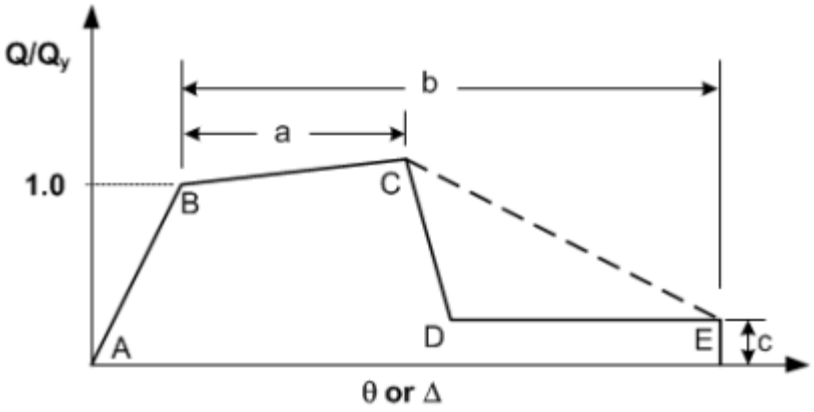
Option 3

Model captures cyclic degradation, but post-peak softening is not modeled; and ultimate limit state is imposed to avoid unconservative analysis in post-peak realm

Option 2



(b) Option 2 – modified backbone curve = envelope curve



ASCE 41 (and related models)

Generalized Component Response

- Response curves in ASCE 41 are essentially the same as "Option 2"

– cyclic envelope fit to cyclic test data

– ASCE 41 (FEMA 273) originally envisioned for static pushover analysis without any cyclic deformation in the analysis

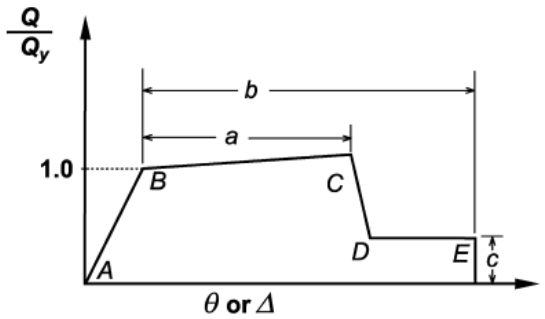
- Option 2/ASCE 41: reasonable for most Commercial analysis programs that cannot simulate cyclic degradation of the backbone curve

- Post-Peak Response: dashed line connecting points C-E in ASCE 41 response curve is more reasonable representation of post-peak (softening) response

ASCE 41 Beam Modeling Parameters and Acceptance Criteria

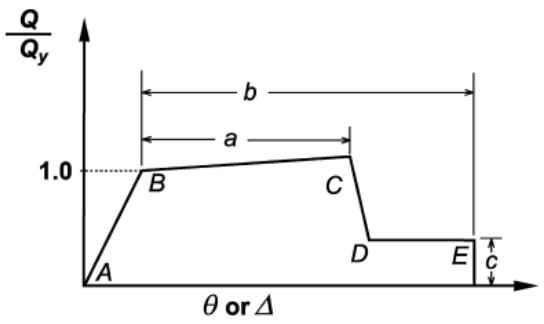
Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[a]			Acceptance Criteria[a]		
						Plastic Rotation Angle (radians)		
			Plastic Rotation Angle (radians)		Residual Strength Ratio	Performance Level		
Conditions			*a*	*b*	*c*	IO	LS	CP
Condition i. Beams controlled by flexure[b]								
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V^{d}}{b_w d\sqrt{f'_{cE}}}$						
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3 (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6 (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ *d*/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > *d*/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[b]								
Stirrup spacing ≤ *d*/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > *d*/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]								
			0.015	0.03	0.2	0.01	0.02	0.03

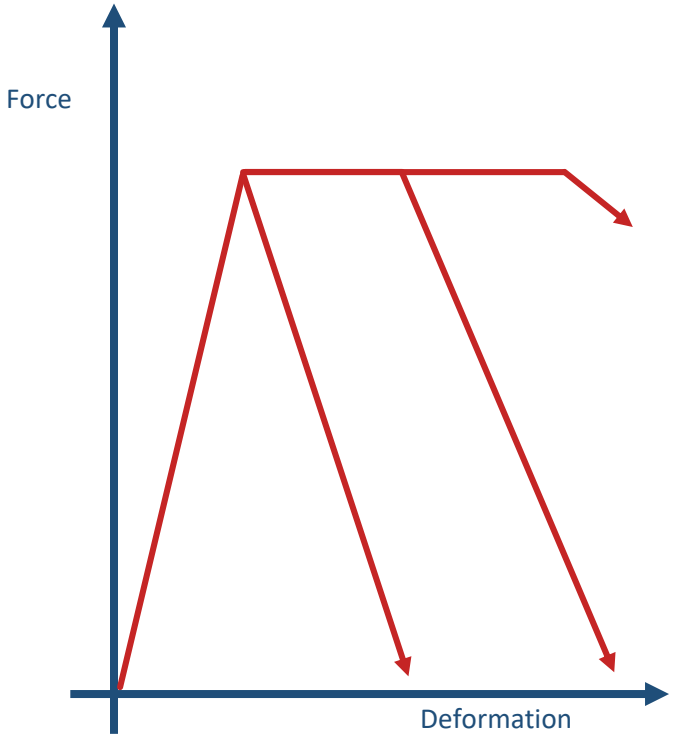


ASCE 41 Column Modeling Parameters and Acceptance Criteria

Conditions			Modeling Parameters[a]: Plastic Rotations Angle (radians)		Modeling Parameters[a]: Residual Strength Ratio	Acceptance Criteria[a]: Plastic Rotations Angle (radians), Performance Level		
			a	*b*	*c*	IO	LS	CP
Condition i.[b]								
$\frac{P}{A_g f_c'}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.035	0.060	0.2	0.005	0.045	0.060
≥0.6	≥0.006		0.010	0.010	0.0	0.003	0.009	0.010
≤0.1	=0.002		0.027	0.034	0.2	0.005	0.027	0.034
≥0.6	=0.002		0.005	0.005	0.0	0.002	0.004	0.005
Condition ii.[b]								
$\frac{P}{A_g f_c'}$ [c]	$\rho = \frac{A_v}{b_w s}$	$\frac{V}{b_w d\sqrt{f_c'}}$ [d]						
≤0.1	≥0.006	≤3 (0.25)	0.032	0.060	0.2	0.005	0.045	0.060
≤0.1	≥0.006	≥6 (0.5)	0.025	0.060	0.2	0.005	0.045	0.060
≥0.6	≥0.006	≤3 (0.25)	0.010	0.010	0.0	0.003	0.009	0.010
≥0.6	≥0.006	≥6 (0.5)	0.008	0.008	0.0	0.003	0.007	0.008
≤0.1	≤0.0005	≤3 (0.25)	0.012	0.012	0.2	0.005	0.010	0.012
≤0.1	≤0.0005	≥6 (0.5)	0.006	0.006	0.2	0.004	0.005	0.006
≥0.6	≤0.0005	≤3 (0.25)	0.004	0.004	0.0	0.002	0.003	0.004
≥0.6	≤0.0005	≥6 (0.5)	0.0	0.0	0.0	0.0	0.0	0.0
Condition iii.[b]								
$\frac{P}{A_g f_c'}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.0	0.060	0.0	0.0	0.045	0.060
≥0.6	≥0.006		0.0	0.008	0.0	0.0	0.007	0.008
≤0.1	≤0.0005		0.0	0.006	0.0	0.0	0.005	0.006
≥0.6	≤0.0005		0.0	0.0	0.0	0.0	0.0	0.0



Column Rotation



$V_p/V_o > 1.0$

$0.6 \leq V_p/V_o \leq 1.0$

$V_p/V_o \leq 0.6$

ATC 72 Recommended Modeling Parameters

$$\theta_p = 0.12(1+0.55a_{sl})(0.16)^{\upsilon}(0.02+40\rho_{sh})^{0.43}(0.54)^{0.01c_{units}f_c'}(0.66)^{0.1s_n}(2.27)^{10.0\rho}$$

$$\theta_{pc} = (0.76)(0.031)^{\nu}(0.02+40\rho_{sh})^{1.02} \leq 0.10$$

Key Design/Detailing Variables:
ρ_{sh} – amount of steel stirrups
ρ – amount of Longitudinal steel
ν – axial load ratio(P/Agfc')
a_{sl} – joint bond slip
s_n – tie spacing

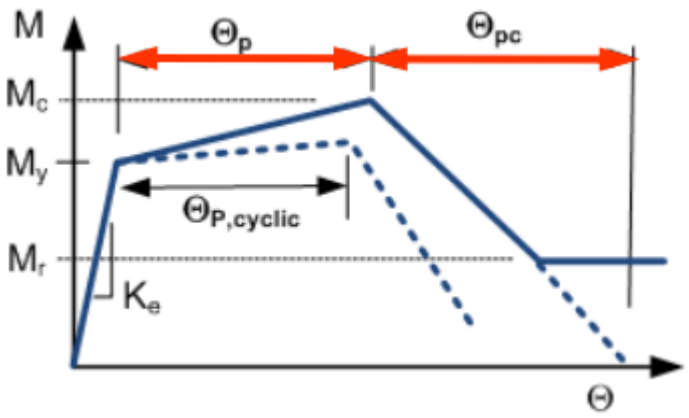


Table 3-3 Empirical Plastic Rotation Values, θ_p and θ_{pc}, for a Representative Column Section (Haselton et al., 2008)

$\nu = P / A_g f_c'$	ρ_{sh}	θ_p	θ_{pc}
0.1	0.002	0.031	0.052
	0.006	0.047	0.100
	0.020	0.077	0.100
0.6	0.002	0.012	0.009
	0.006	0.019	0.024
	0.020	0.031	0.077

- Values given by the equations are initial monotonic backbone curve.
- To convert monotonic backbone curve to cyclic envelope curve reduction factors are recommended

$$\Theta_{p,cyclic} = 0.7\Theta_p$$
$$\Theta_{pc,cyclic} = 0.7\Theta p_c$$

Acceptance Criteria

- For evaluation of designs under MCE ground motions, the Section 16.4.2.2 of ASCE/SEI 7-16 specifies that deformation criteria should be determined either from the collapse prevention (CP) limits of ASCE/SEI 41, or alternatively, by mean values determined from tests
- Collapse prevention acceptance criteria for a component is the limit when loss in vertical-load-carrying-capacity would occur.

$$Q_u < \phi_s Q_{ne} \text{ or } Q_{CP}$$

where:

Q_u = the mean deformation demand from analysis

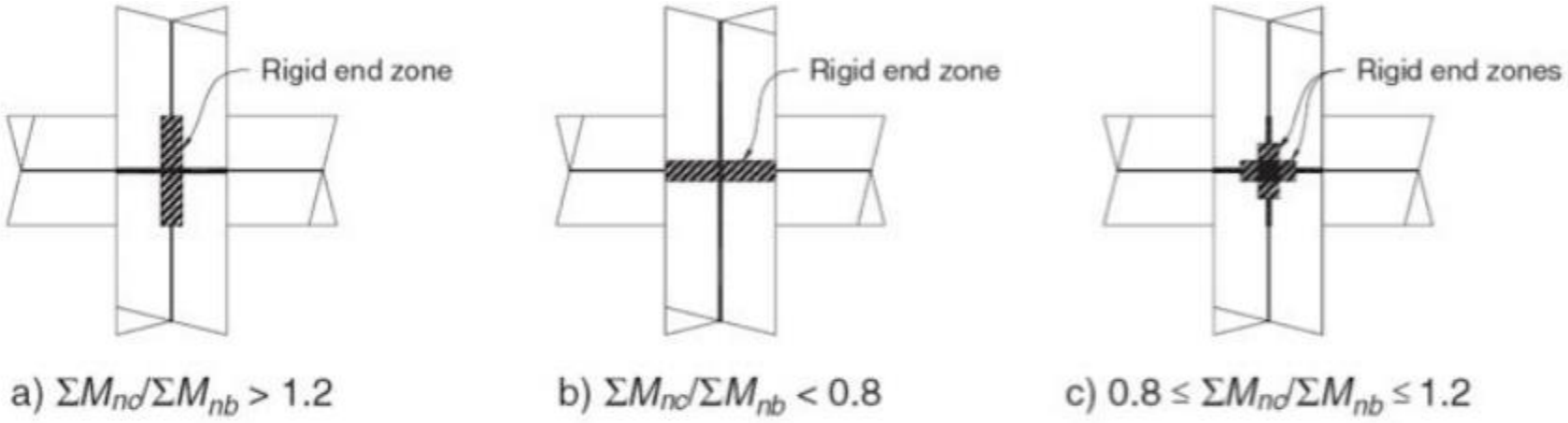
Q_{ne} = the mean deformation capacity from tests

Q_{CP} = the CP deformation limit from ASCE/SEI 41

ϕ_s = $(0.3/I_e)$ for critical components, $(0.5/I_e)$ for ordinary components, and $(1/I_e)$ for non-critical components

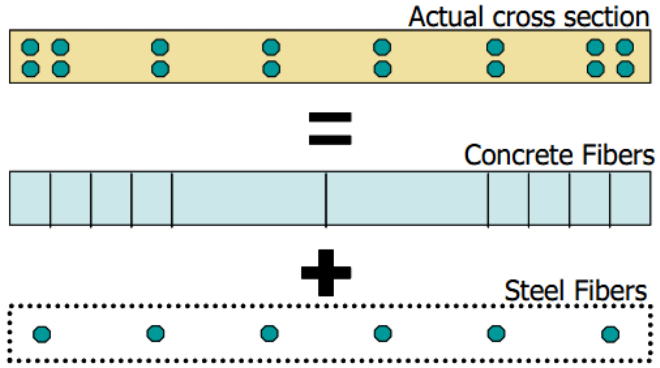
I_e = the importance factor, depending on the ASCE/SEI 7 Risk Category

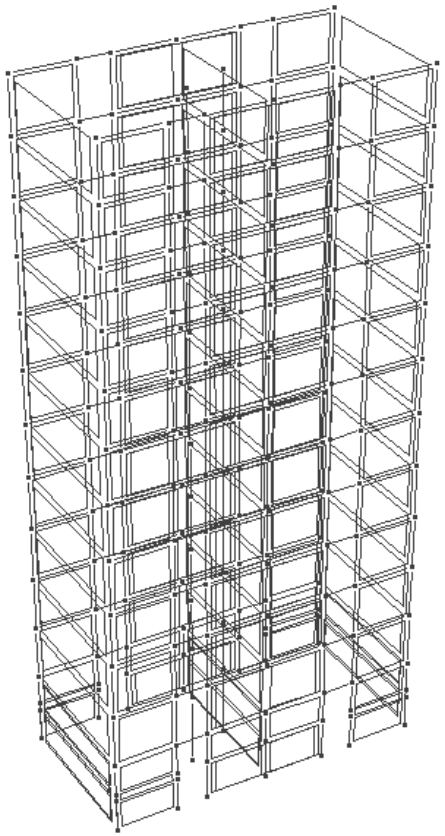
Joint Modeling



Recommended rigid end zone offsets for RC joints, based on relative column and beam strengths (Elwood et al., 2007; ASCE 41-13).

Modeling of Structural Components: RC Shear Walls

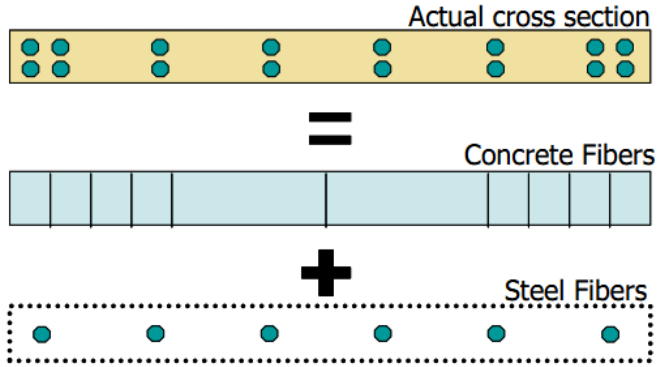


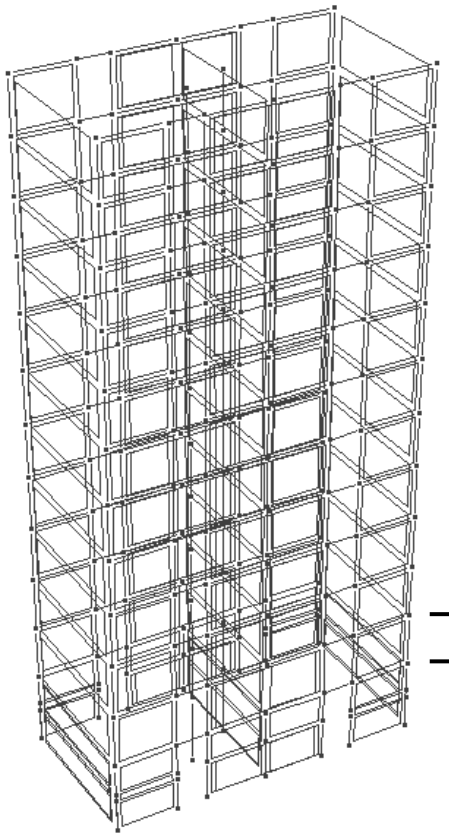


RC Shear Walls

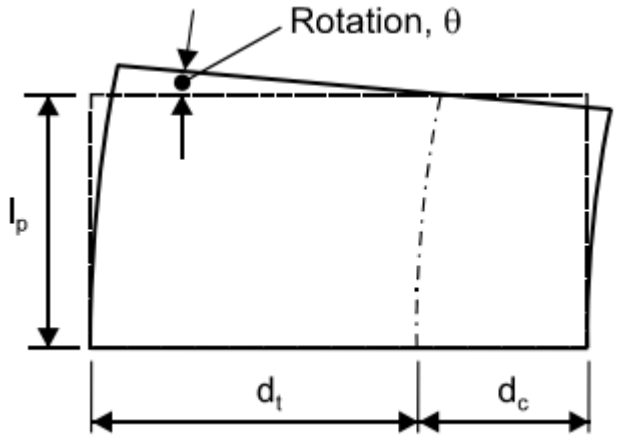
- **Preferred Model:** Fiber wall panels
- **Thing to keep in mind**
  - Fiber models are only nonlinear in axial flexure(P-M3)
  - Flexural stiffness is derived from specified material stress stains
  - Shear in fiber models is generally modelled as elastic
  - Fiber predict the flexural response reasonably well
  - Fiber models may not predict strength degradation (and ultimately, failure) well
  - Strain predicted by fiber model are sensitive to material stress strains and fiber element discretization

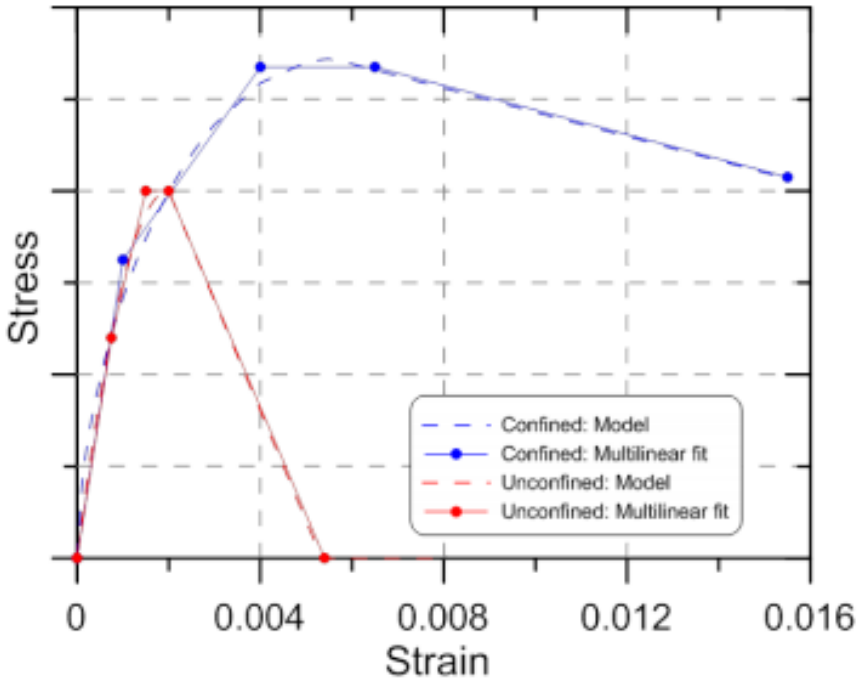
Modeling of Structural Components: RC Shear Walls

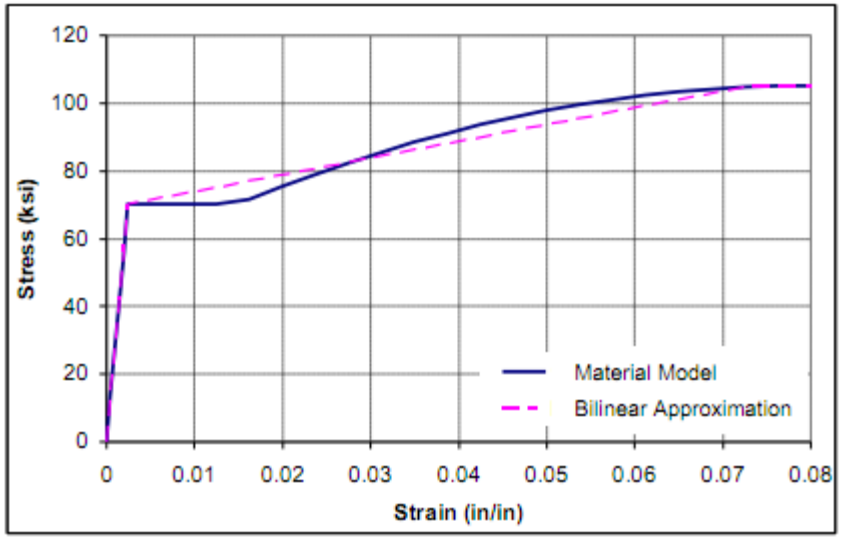




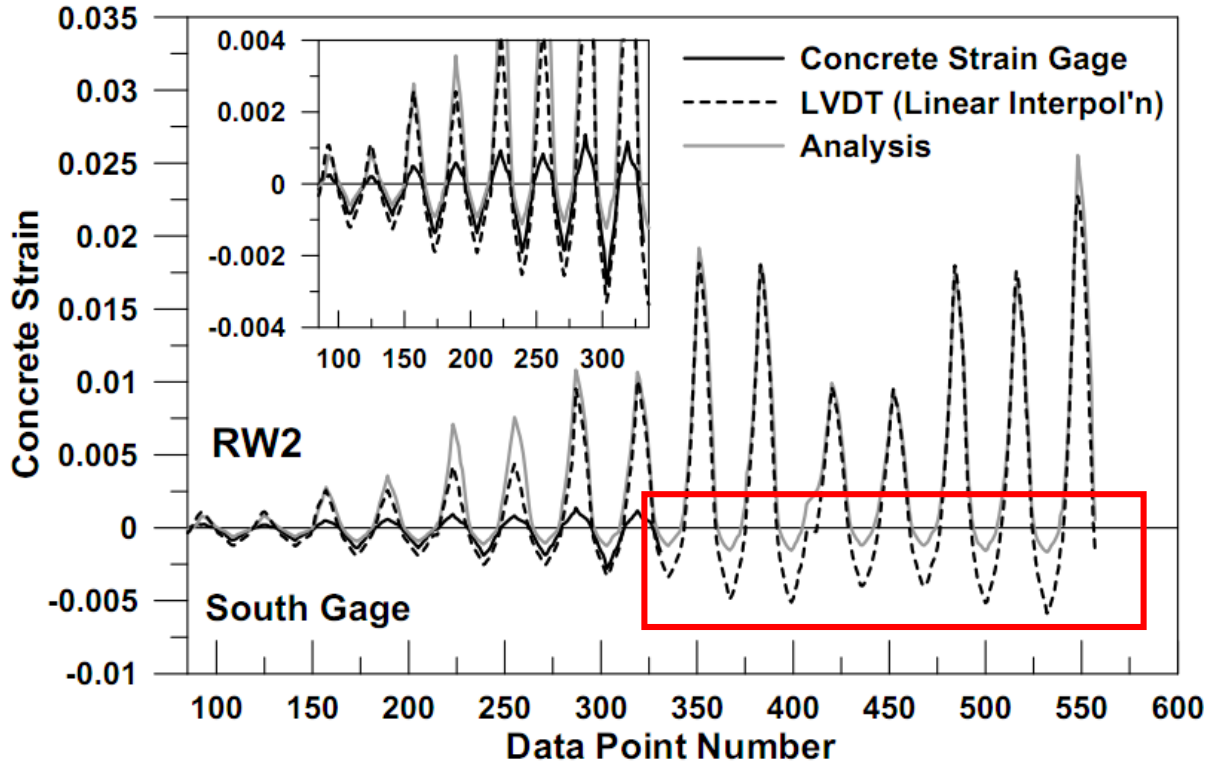
Strain ε=δ/L

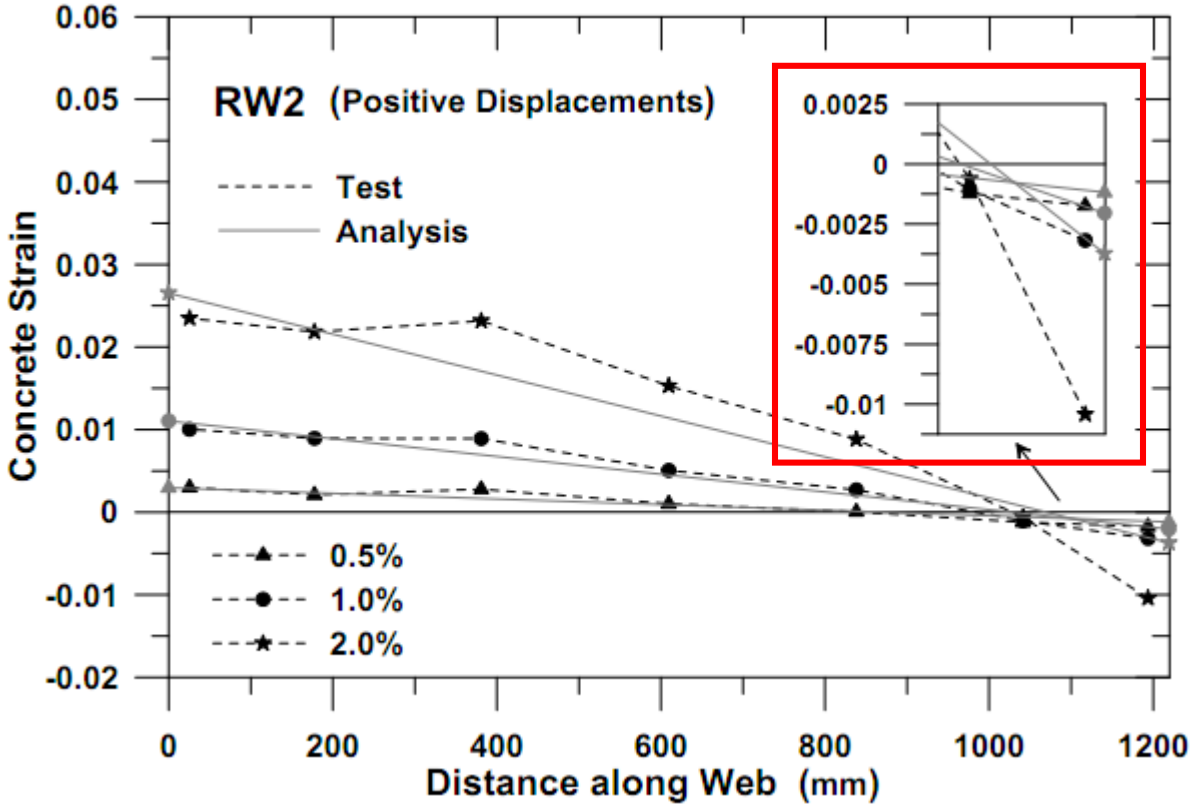






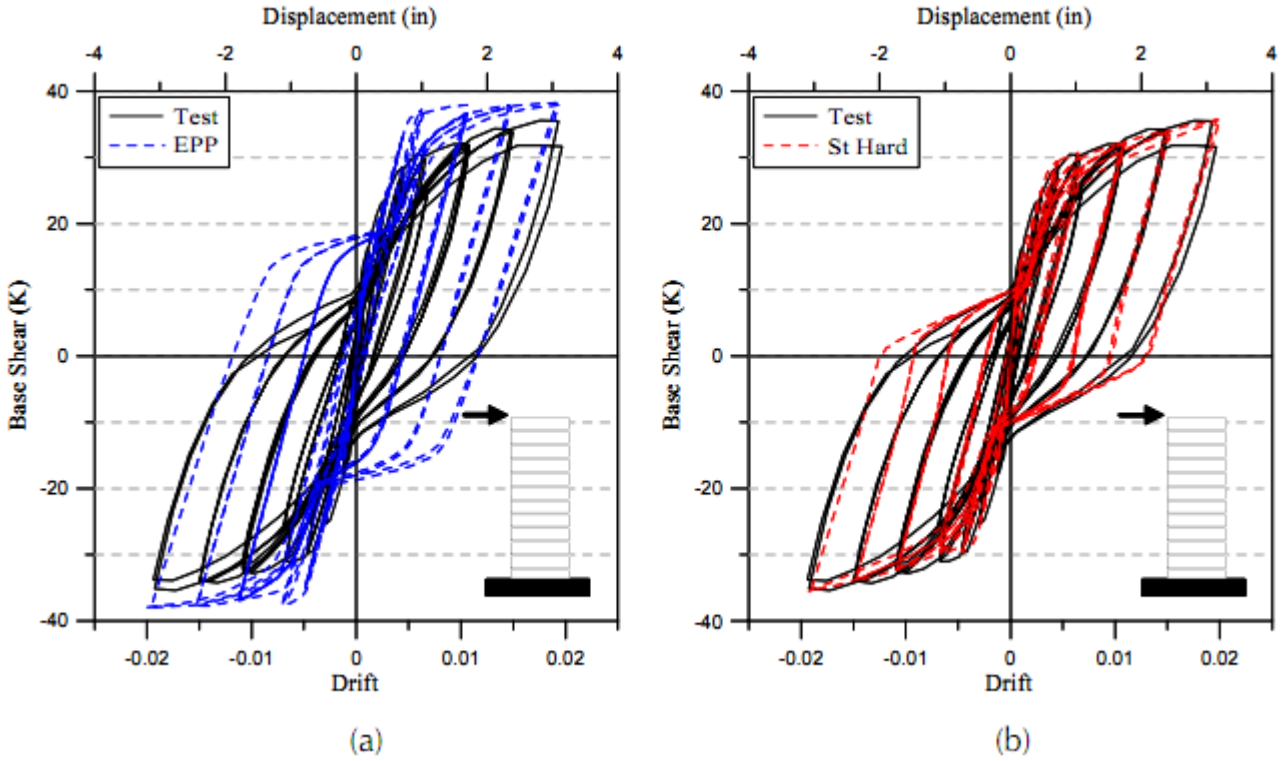
Fiber model Strain Prediction





Source: ATC 72-1

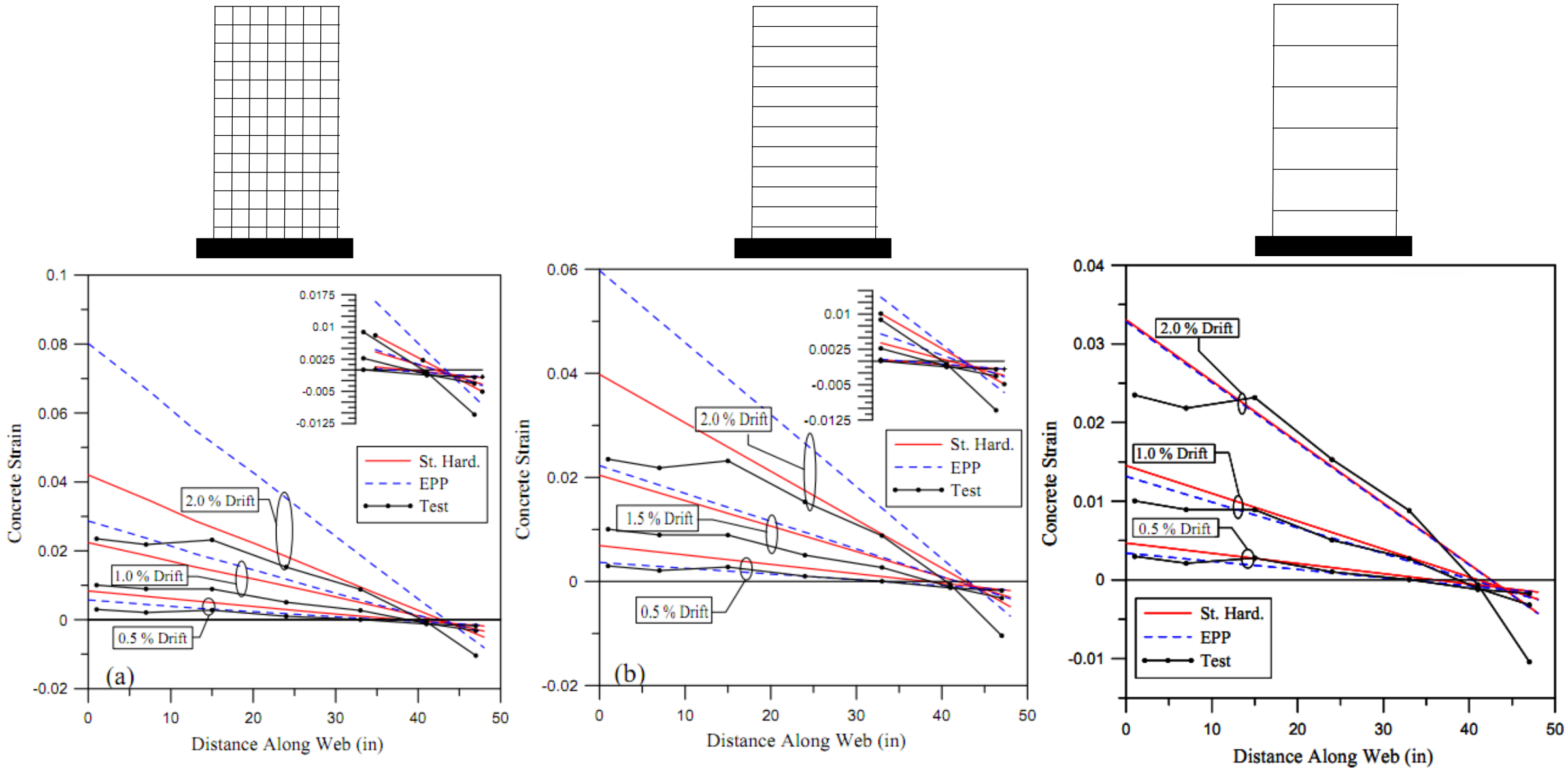
Influence of Reinforcing Steel Stress-Strain Relation



Source: ATC 72-1

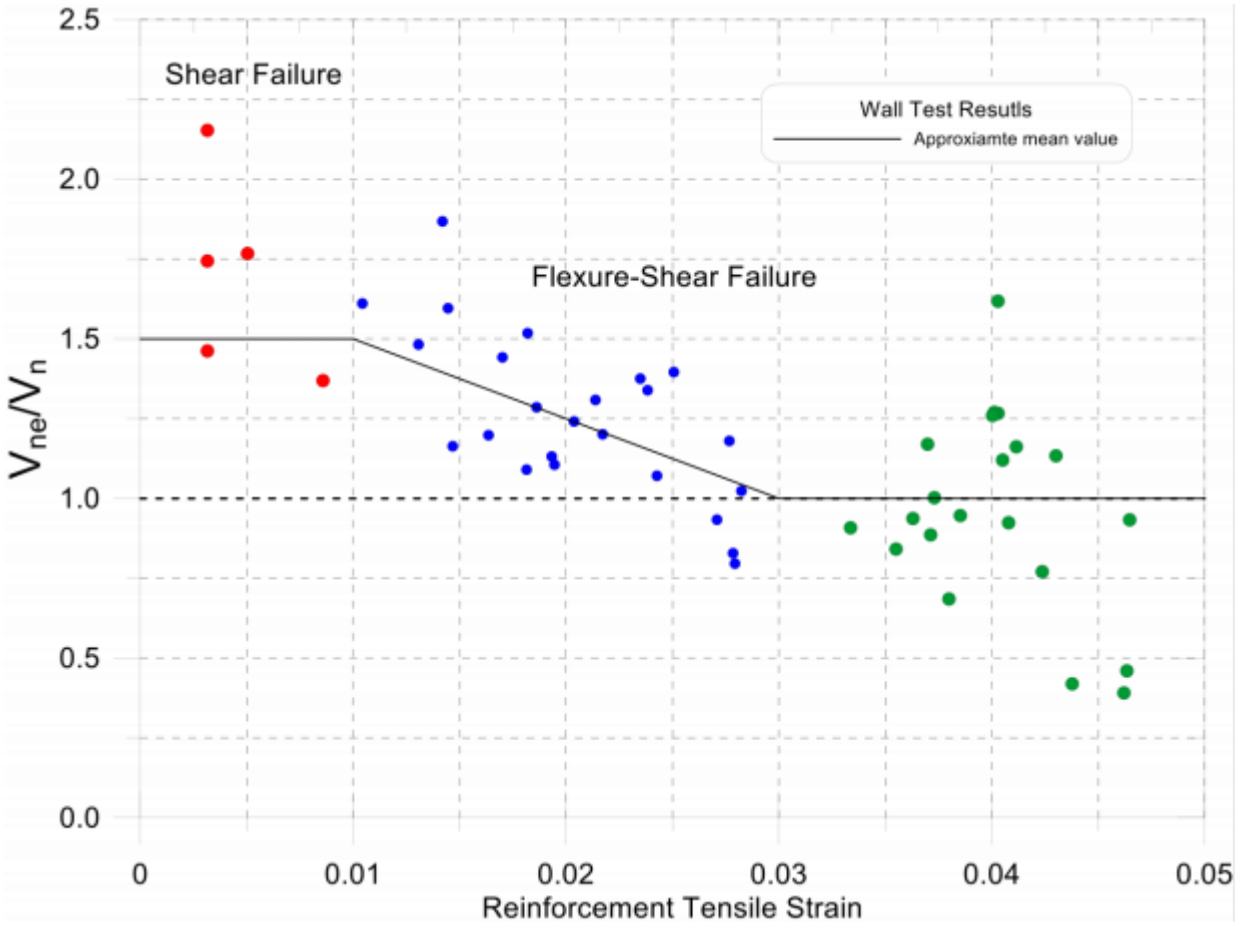
Fiber Model Wall Meshing Sensitivity

Source: ATC 72-1



(a) 91 elements (b) 12 elements (c) 6 elements

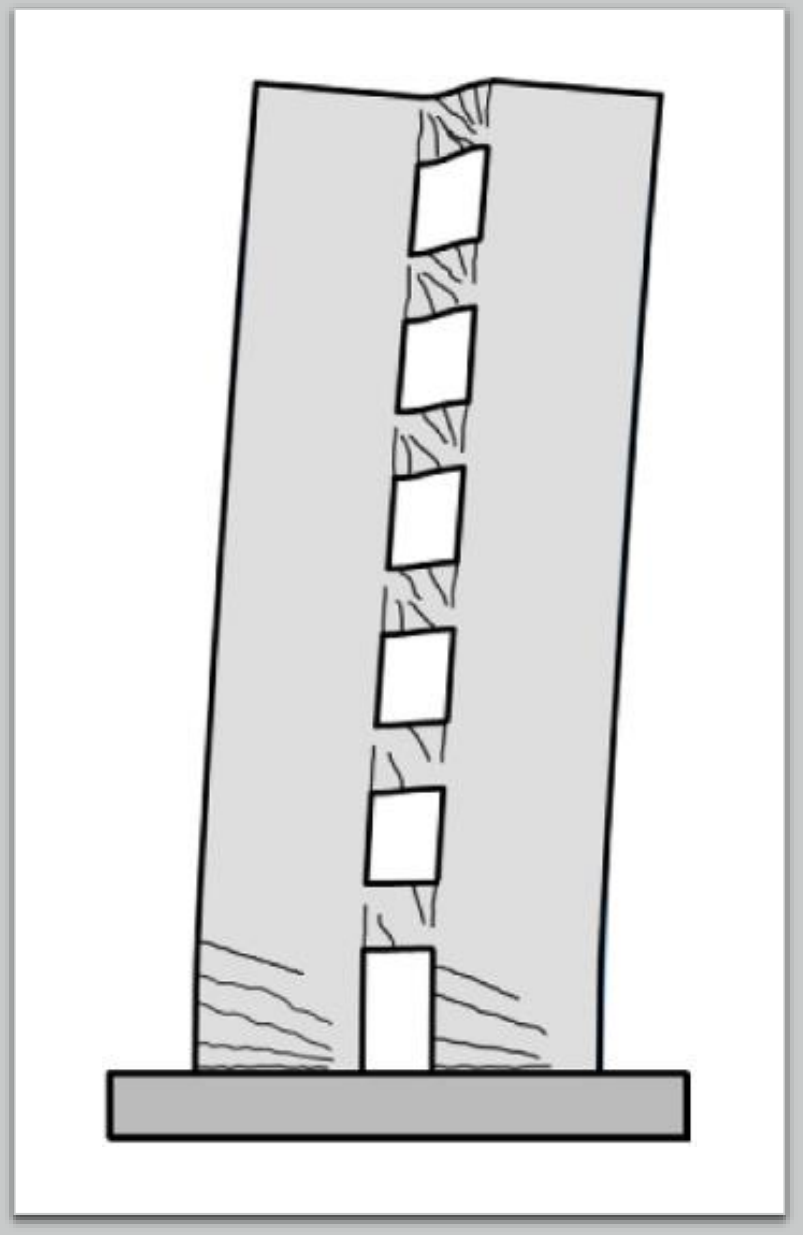
Reinforcement Tensile Strain vs Shear Capacity



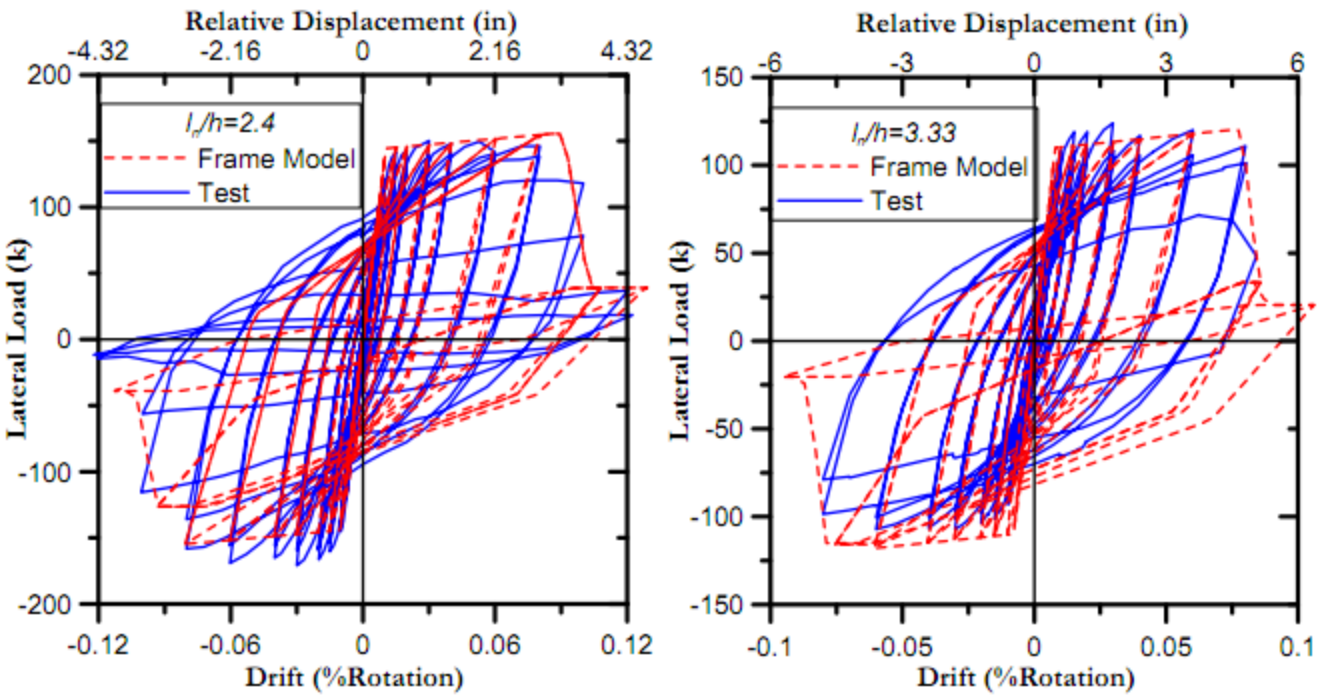
Source: 2017 LATBSDC

Coupling Beam

- Bending of shear wall can induce a lot of deformations in coupling beam.
- A consistent finding from coupling beam studies is that the use of diagonal reinforcement improves the cyclic performance of beams with clear span-to-depth ratios r less than four.
- For ratios greater than four, use of diagonal reinforcement is not practical given the shallow angle of the bars
- For modeling of coupling beams lumped plasticity models i.e. moment hinge or shear hinge are capable of producing test results with reasonable accuracy
- UCLA-SGEL Report 2009/06 and ASCE 41-13 are good resources



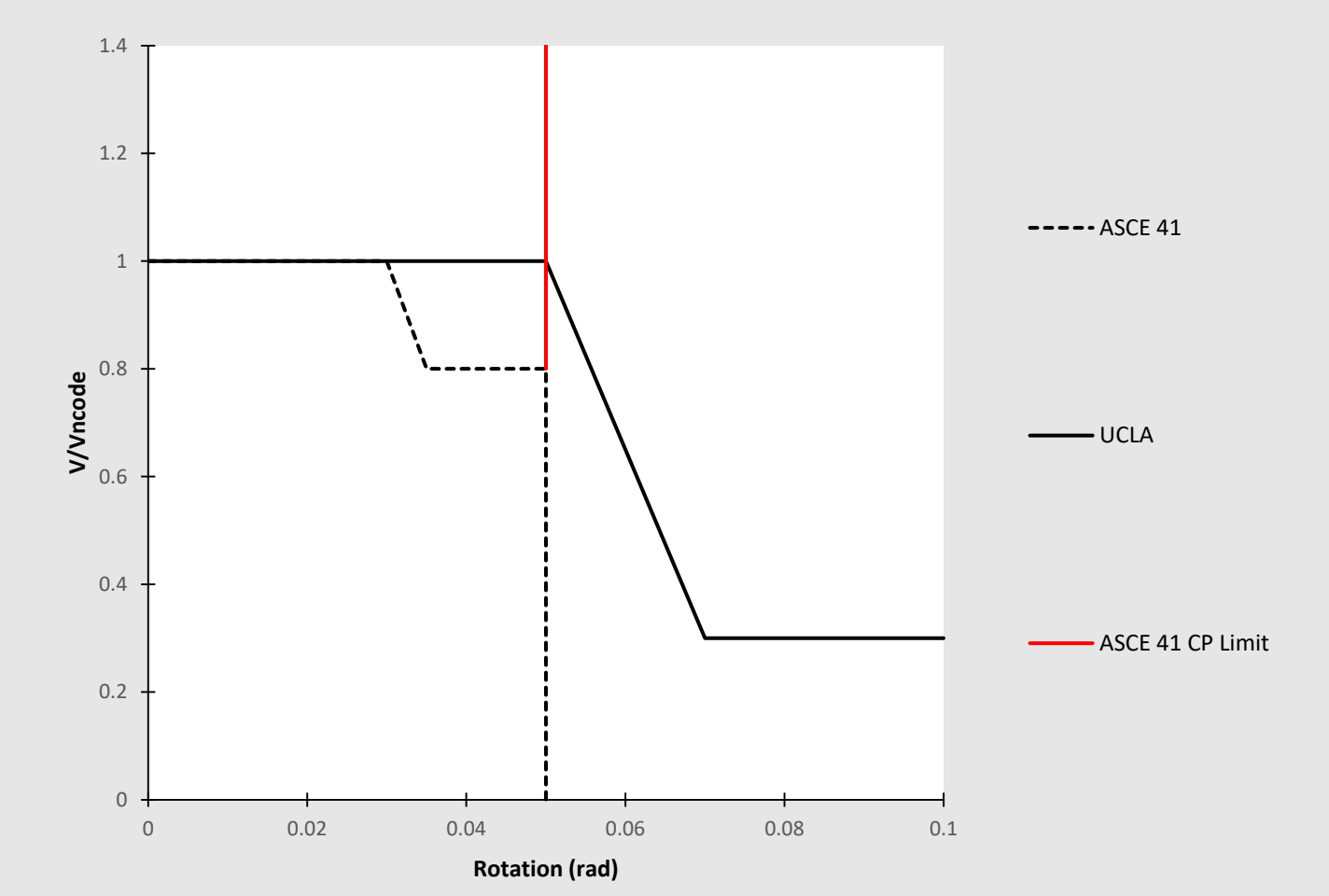
Coupling Beam Model Calibration



(a) Moment-hinge model (l_n/h = 2.4) (b) Moment-hinge model (l_n/h = 3.33)

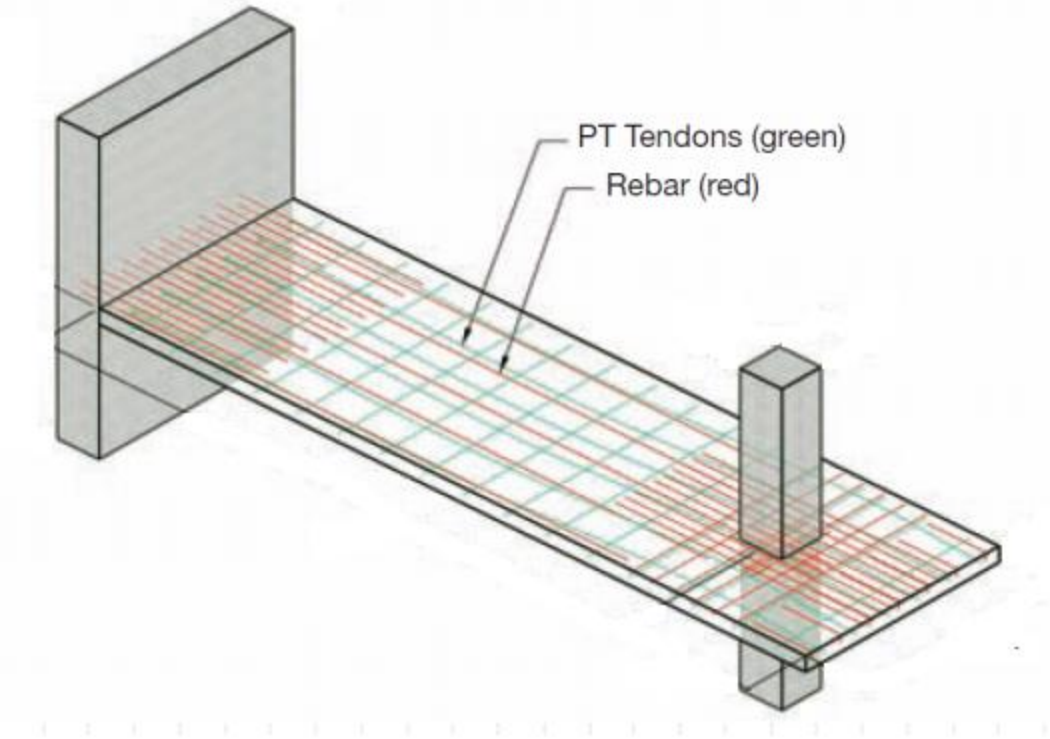
Source: ATC 72-1

Coupling Beam Back Bone Comparision

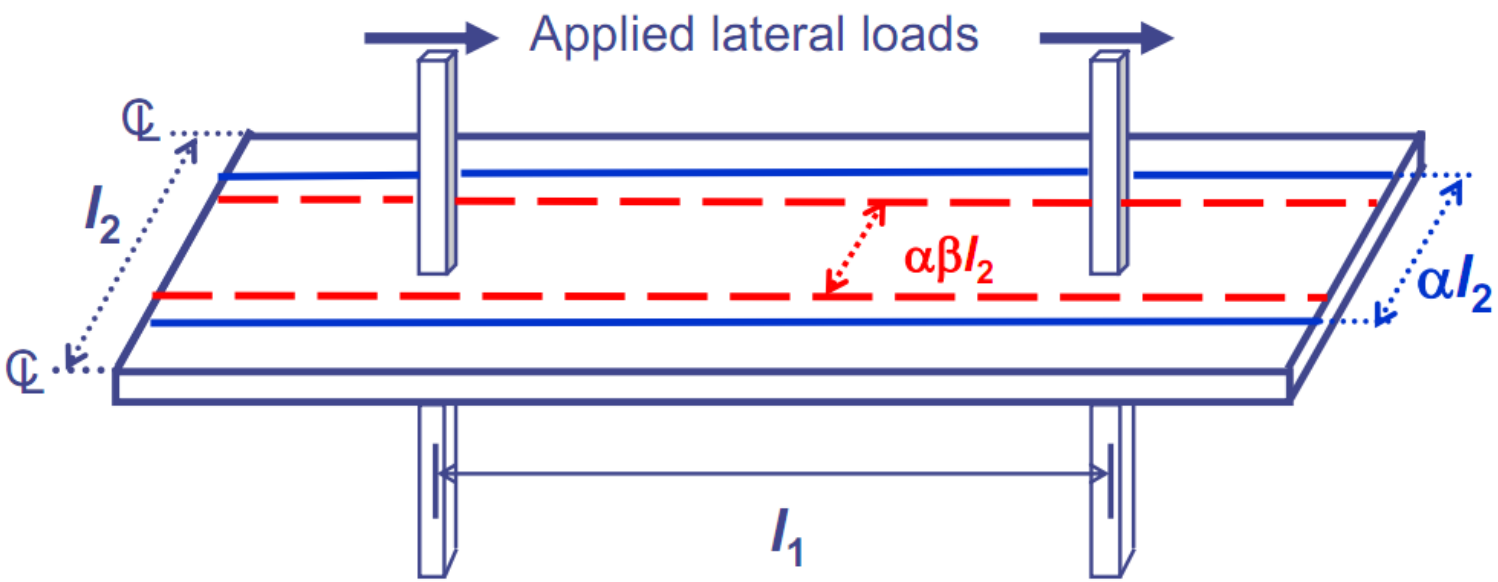


Slab-column Frame Components

- Flat slabs are very popular in tall buildings these days.
- Effective slab beam model can be used to capture coupling between slab-column frame
- Modeling of slab-column frames involves assigning appropriate stiffness and strength
- Nonlinear hinges can be used in the effective slab beams following the recommendations ASCE 41



Effective Slab Beam Model



α: Effective Beam Width Factor
β: Coefficient accounting for Cracking

$\alpha l_2 = 2c_1 + l_1/3$ for Interior Frames

$\beta = 4c_1 / l_1 \geq 1/3$ for RC Slabs
$\beta = 1/2$ for PT

Nonlinear Modeling Parameters

ASCE 41-13 Table 10-15

Conditions		Plastic Rotation Angle (radians)		Residual Strength Ratio	Performance Level		
						Secondary	
		a	*b*	*c*	IO	LS	CP
Condition i. Reinforced concrete slab–column connections[b]							
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]						
0	Yes	0.035	0.05	0.2	0.01	0.035	0.05
0.2	Yes	0.03	0.04	0.2	0.01	0.03	0.04
0.4	Yes	0.02	0.03	0.2	0	0.02	0.03
≥0.6	Yes	0	0.02	0	0	0	0.02
0	No	0.025	0.025	0	0.01	0.02	0.025
0.2	No	0.02	0.02	0	0.01	0.015	0.02
0.4	No	0.01	0.01	0	0	0.008	0.01
0.6	No	0	0	0	0	0	0
>0.6	No	0	0	0	—[e]	—[e]	—[e]
Condition ii. Posttensioned slab–column connections[b]							
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]						
0	Yes	0.035	0.05	0.4	0.01	0.035	0.05
0.6	Yes	0.005	0.03	0.2	0	0.025	0.03
>0.6	Yes	0	0.02	0.2	0	0.015	0.02
0	No	0.025	0.025	0	0.01	0.02	0.025
0.6	No	0	0	0	0	0	0
>0.6	No	0	0	0	—[e]	—[e]	—[e]

Salient Points on Slab-Column Frame Modeling

- Vg in ASCE41-13 Table has to be calculated from load combination 1.2DL + 0.5 LL
- Slab Beam Moment Capacity = M_{nCS} - M_{gCS}

M_{nCS} = the design flexural strength of the column strip

M_{gCS} = the column strip moment caused by gravity loads

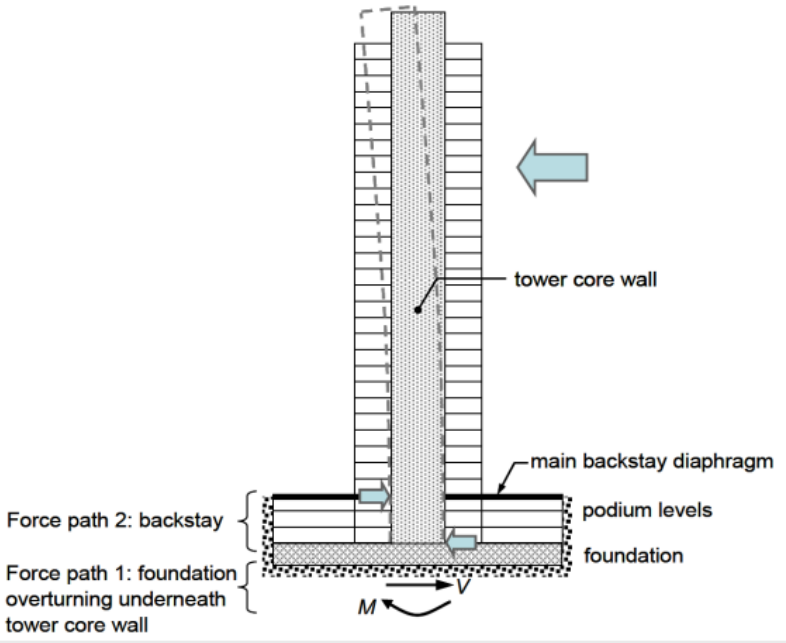
- If Slab-column frame are not modeled

– the potential adverse impacts of additional axial load induced on the gravity system columns should be considered

– Punching shear failure possibilities can still be checked following the requirments of ACI 318-14 18.14.5.1

Back Stay Effect

- Backstay effects are the transfer of lateral forces from the seismic-force resisting elements in the tower into additional elements that exist within the podium
- The lateral force resistance in the podium levels, and force transfer through floor diaphragms at these levels, helps a tall building resist seismic overturning forces
- Force transfer is sensitive to podium diaphragm stiffness assumption.
- The UB analysis provides an upper-bound estimate of forces in the backstay load path and a lower bound estimate of forces in the foundation below the tower
- The LB analysis provides a lower-bound estimate of forces in the backstay load path and an upper-bound estimate of forces in the foundation below the tower.



(Illustration from ATC-72)

Table 5. Stiffness parameters for Upper Bound and Lower Bound Models

Stiffness Parameters	UB	LB
Diaphragms at the podium and below		
$E_c I_g$	0.35	0.10
$E_c A$	0.14	0.04

Remaining Information

- 2.5% Viscous damping is generally used in the nonlinear model to account for unmodeled energy dissipation
- Model the effects of expected gravity loads in the nonlinear analysis using the load combination D + 0.5L, where L is 80% of unreduced live loads that exceed 100 pounds-per-square-foot (4.79 kN/m2) and 40% of other unreduced live loads.
- Include the seismic mass of the entire building in the model, including both the superstructure and below-grade structure

Global Acceptance Criteria

- Analytical solution fails to converge.
- Peak transient story drift ratio in any story exceeds 0.045;
- Residual story drift ratio in any story exceeds 0.015
- Demands on deformation-controlled elements exceed the valid range of modeling

Thank you