MIDAS SQUARE

MIDAS SQUARE 공학 기술강연

Performance-Based Earthquake Design of Building Structure in Korea
건축물의 성능기반 내진설계

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CONTENTS

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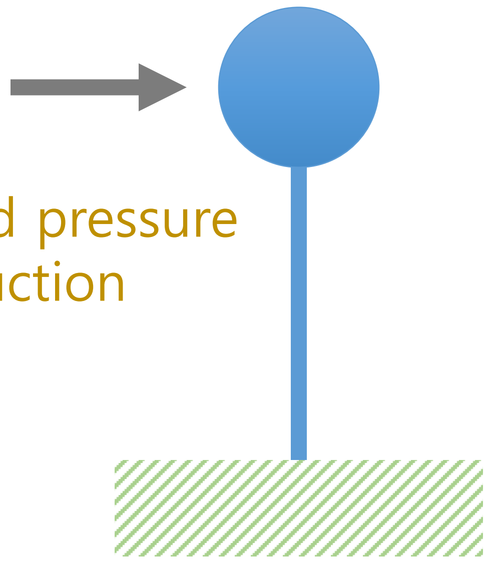
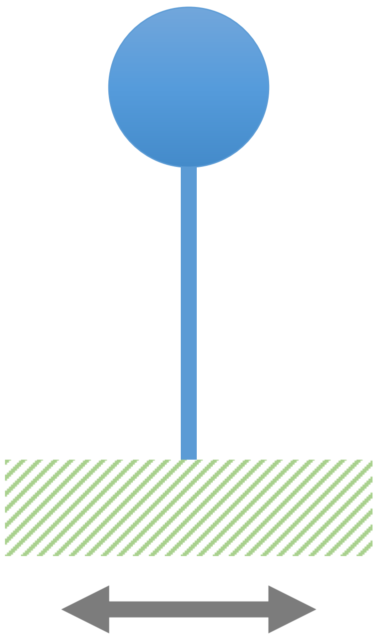
Background

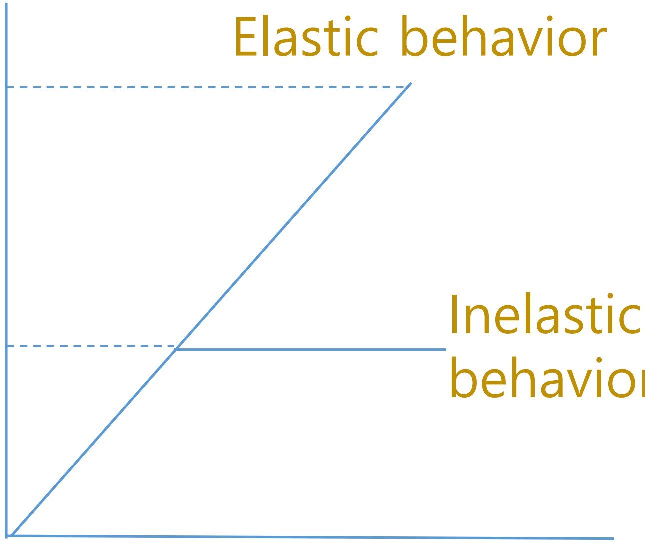
- Earthquake design of buildings (ductility design)

- Wind design



- EQ design





Background

- Earthquake design of buildings (ductility design)

Seismic force-resisting system[1]	Design factors			Limitations of system and height (m)		
	Response modification factor R	System over strength factor Ω_0	Displacement amplification factor C_d	Seismic Design Category A or B	Seismic Design Category C	Seismic Design Category D
1. Bearing wall systems						
1-a. Special reinforced concrete shear walls	5	2.5	5	-	-	-
1-b. Ordinary reinforced concrete shear walls	4	2.5	4	-	-	60
1-c. Reinforced masonry shear walls	2.5	2.5	1.5	-	60	NP
1-d. Unreinforced masonry shear walls	1.5	2.5	1.5	-	NP	NP
1 - e. Light-frame (wood) walls sheathed with wood structural panels	6	3	4	-	20	20
1 - f. Light-frame (cold-formed steel) walls sheathed with wood structural panels or steel sheets	6	3	4	-	20	20

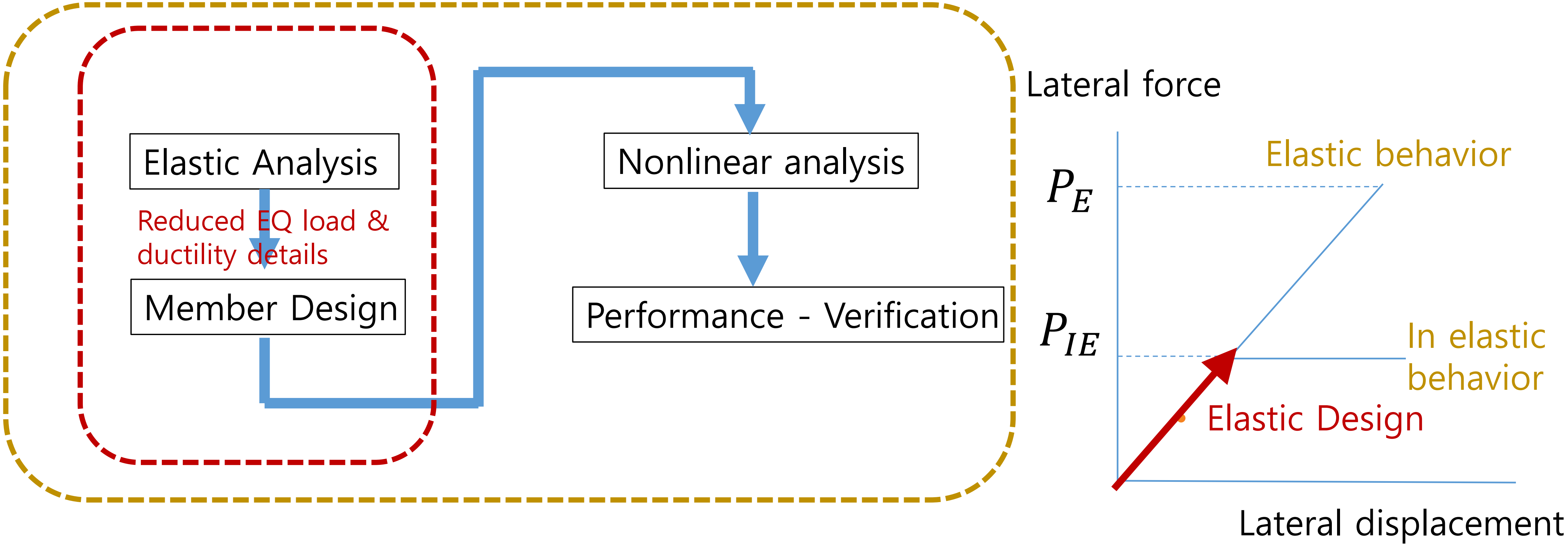
- Uniform value of R factors according to structure type regardless of design parameters (building height, strength, etc)
- Verification of performance is required.

Background

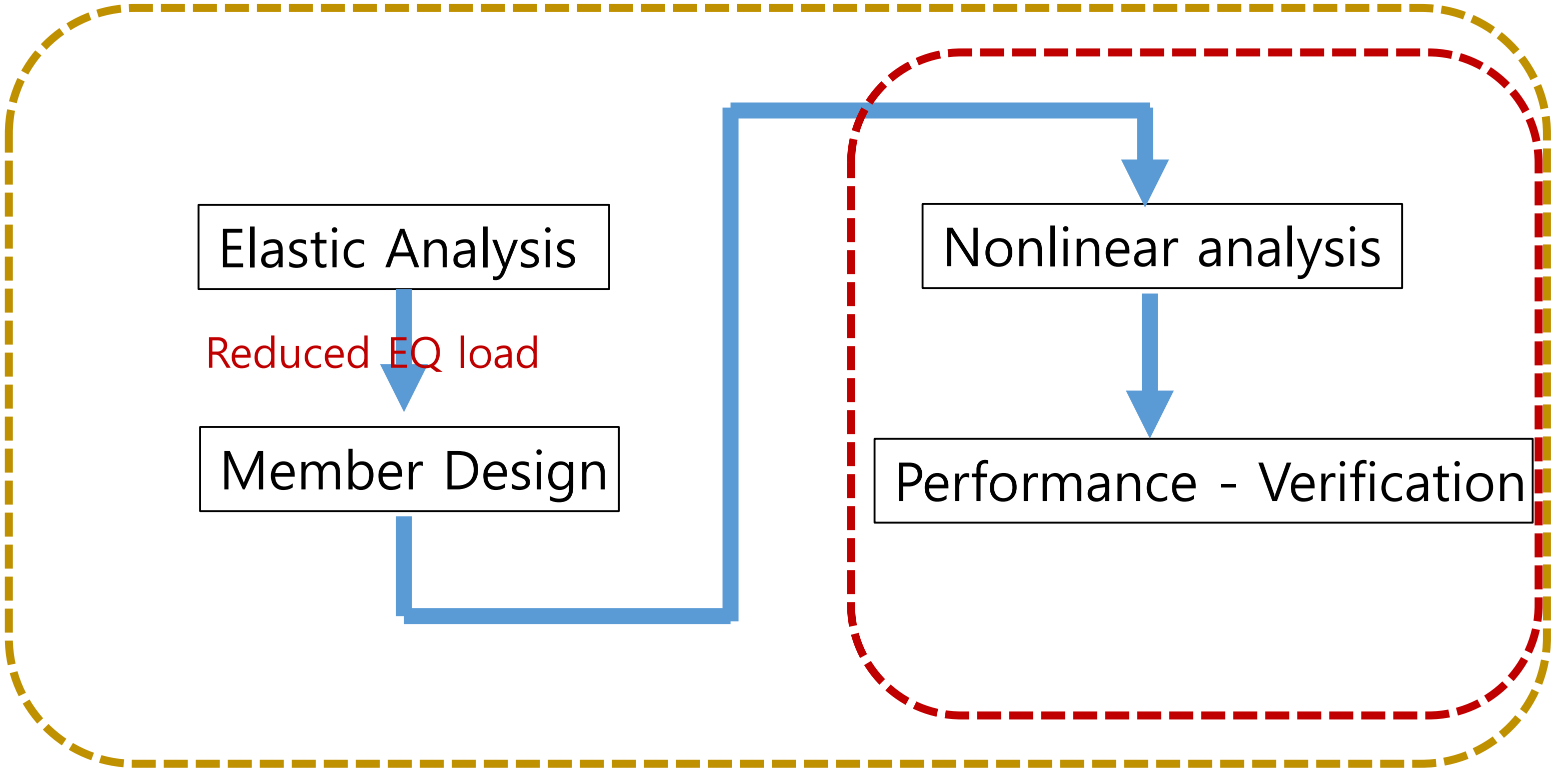
- Earthquake design of buildings (ductility design)
  - Uniform value of R factors according to structure type
  - Regardless of design parameters (building height, strength, etc)

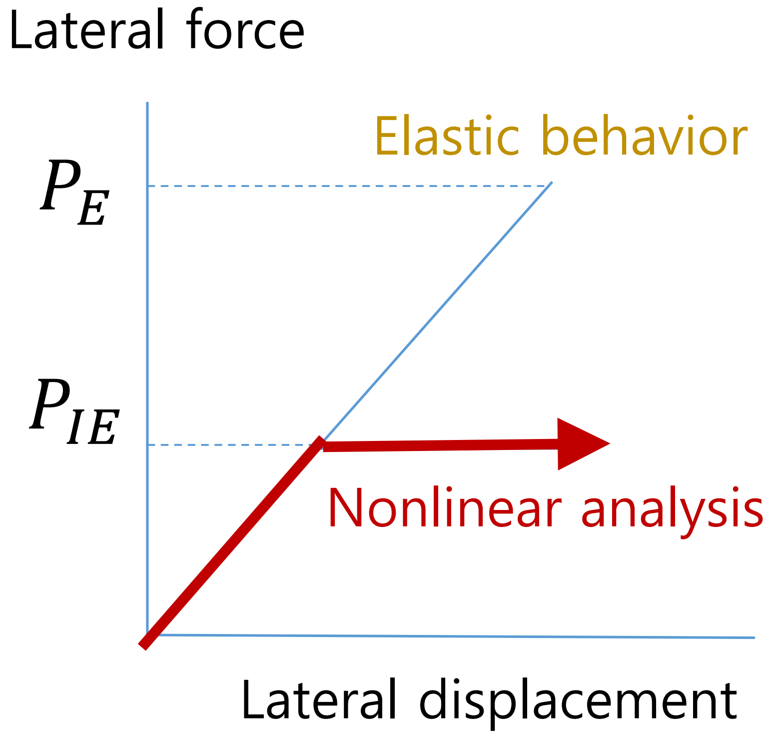
2. Building frame systems						
2-a. Steel eccentrically braced frames (moment resisting column-link connections)	8	2	4	-	-	-
2-b. Steel eccentrically braced frames (non-moment resisting column-link connections)	7	2	4	-	-	-
2-c. Steel special concentrically braced frame	6	2	5	-	-	-
2-d. Steel ordinary concentrically braced frame	3.25	2	3.25	-	-	-
2-e. Composite eccentrically braced frames.	8	2	4	-	-	-
2-f. Composite special concentrically braced frames	5	2	4.5	-	-	-
2-g. Composite ordinary concentrically braced frames	3	2	3	-	-	-
2-h. Steel composite plate shear walls	6.5	2.5	5.5	-	-	-
2-i. Composite special shear walls	6	2.5	5	-	-	-
2-j. Composite ordinary shear walls	5	2.5	4.5	-	-	60
2-k. Steel special plate shear walls	7	2	6	-	-	-
2-l. Buckling-restrained braced frames (moment-resisting beam-column connections)	8	2.5	5	-	-	-
2-m. Buckling-restrained braced frames (non-moment-resisting beam-column connections)	7	2	5.5	-	-	-
2-n. Special reinforced concrete shear walls	6	2.5	5	-	-	-
2-o. Ordinary reinforced concrete shear walls	5	2.5	4.5	-	-	60

Background • Equivalent Elastic Design vs Performance-based design



Background • Equivalent Elastic Design vs Performance-based design





- Performance-based design is important for EQ design in which verification of inelastic deformation capacity is necessary

Background

- Performance-based seismic design of building structures
  - 1st introduced in 2016 in Building Structure Design Code (current KDS 41 17)
  - For structures that are difficult to apply conventional elastic design based on Response modification factors (R-factor or ductility factor)
  - For important structures that require multiple performance goals

- Guidelines for Performance-Based Seismic Design of Reinforced Concrete Buildings (2021)
  » detailed procedure of PBSD

- Guidelines for Nonlinear Analysis Modeling for PBSD of Reinforced Concrete Buildings (2021)
  » detailed modeling technique for nonlinear analysis

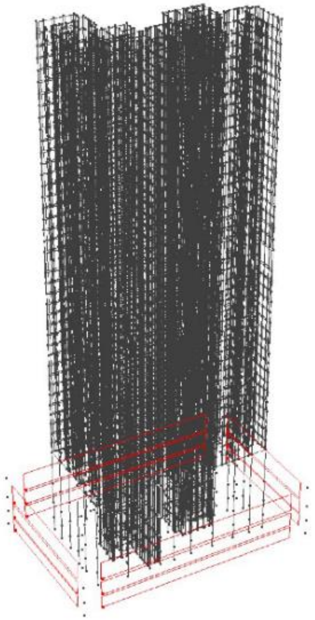
Background

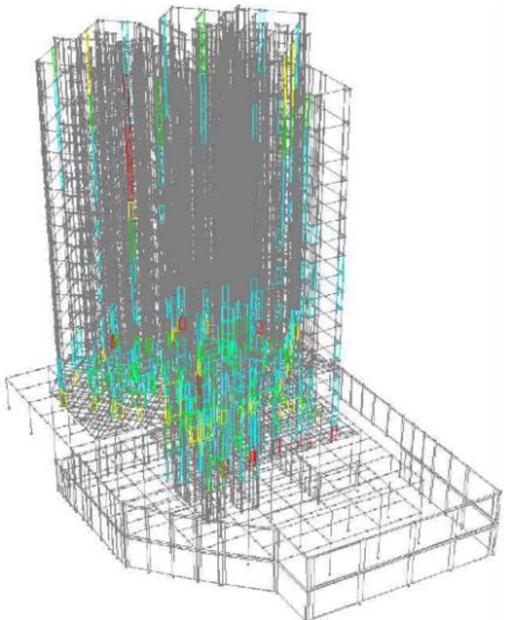
- Contents of PBSD Guidelines
  1. General
  2. Design Procedure
  3. Basic Design
  4. Nonlinear analysis model
  5. Fiber model
  6. Plastic hinge model
  7. Nonlinear Static Analysis
  8. Nonlinear Dynamic Analysis
  9. Verification of Performances

  Appendix : modeling examples

Procedure of PBSD Guidelines

- Basic Design
  - elastic analysis-based conventional design
  - linear dynamic analysis (response spectrum analysis)
  - response modification factor (*R*) corresponding to the seismic load resisting system
  - Design strength according to current design codes
- Nonlinear static and dynamic analysis
  - behavior model (deformation-control, force-control)
  - material stress-strain relationship and member force-deformation relationship curve
  - Nominal strength based on actual structural mechanism
- Verification of performance
  - interstory drift ratio for overall structure behavior
  - deformation and material strain for deformation-controlled behavior, strength of force-controlled behavior





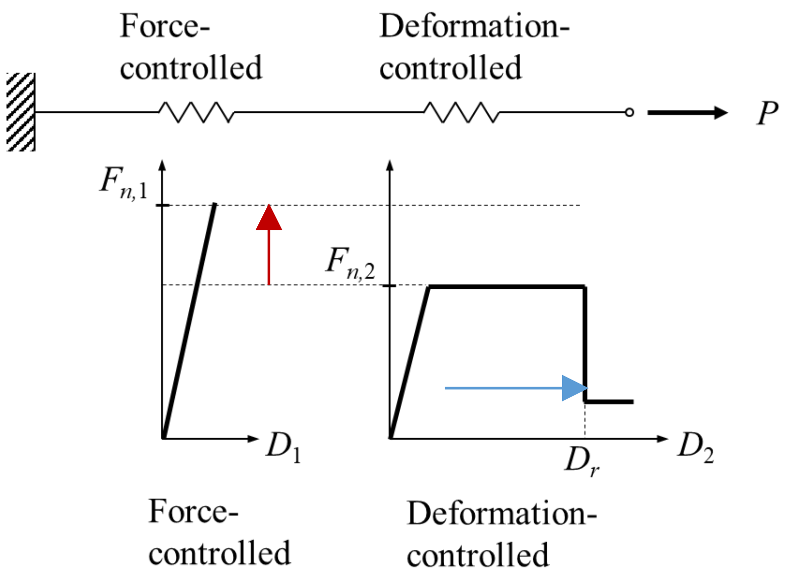
Nonlinear Analysis Modeling

- Behavior Model

  Deformation-controlled behavior : Nonlinear model to estimate deformation demand

  Force-controlled behavior : Linear model to prevent brittle failure

Member type	Deformation-controlled behavior	force-controlled behavior	
beam	flexure	shear	
column	flexure	compression, shear	
wall	flexure	compression, shear	
coupling beam	flexure	shear	



- Requirement of behavior models to guarantee ductile behavior

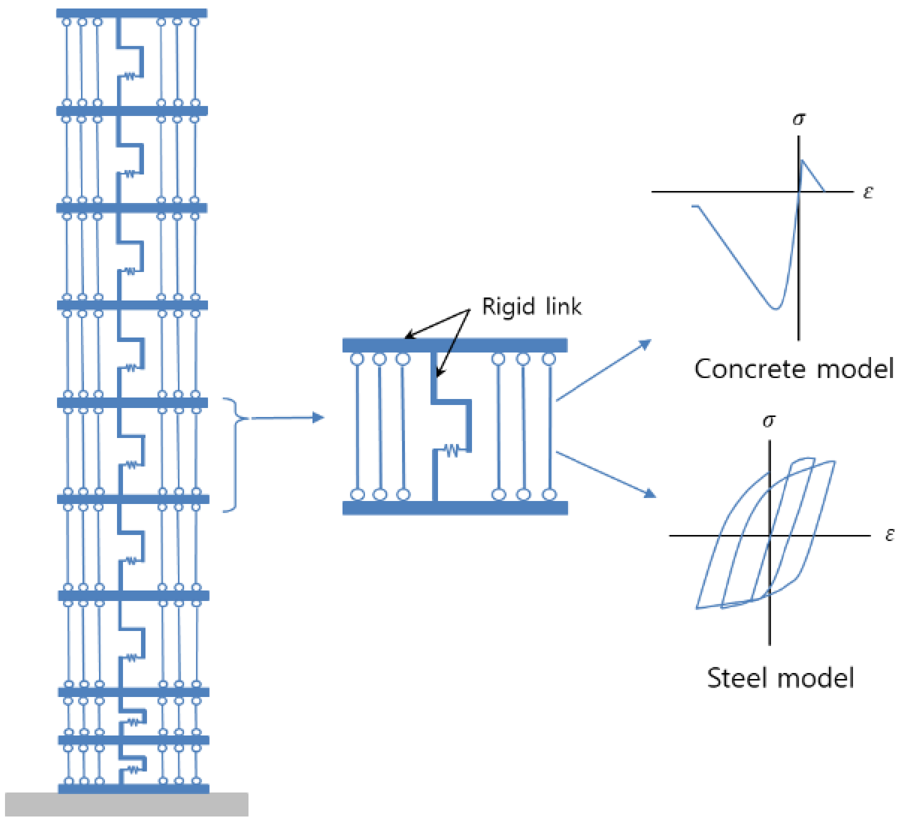
  Deformation-controlled behavior : high deformation (ductility)

  Force-controlled behavior : high strength with a safety factor

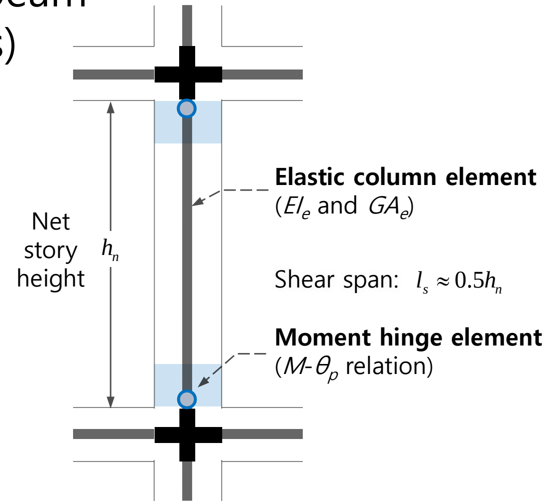
Nonlinear Analysis Modeling

- Model Type : Macro models for efficiency of analysis

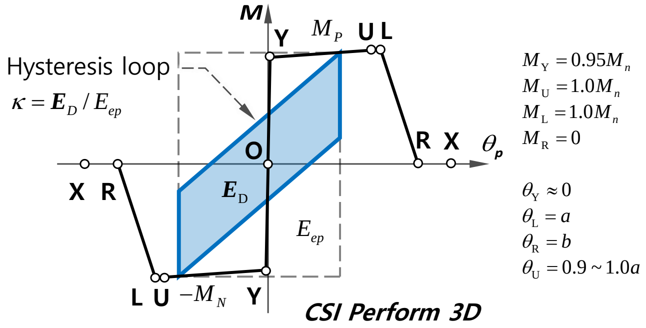
Fiber model (inelastic distributed model) :
wall and column



Plastic hinge model :
column and beam
(line elements)



(a) Modeling elements

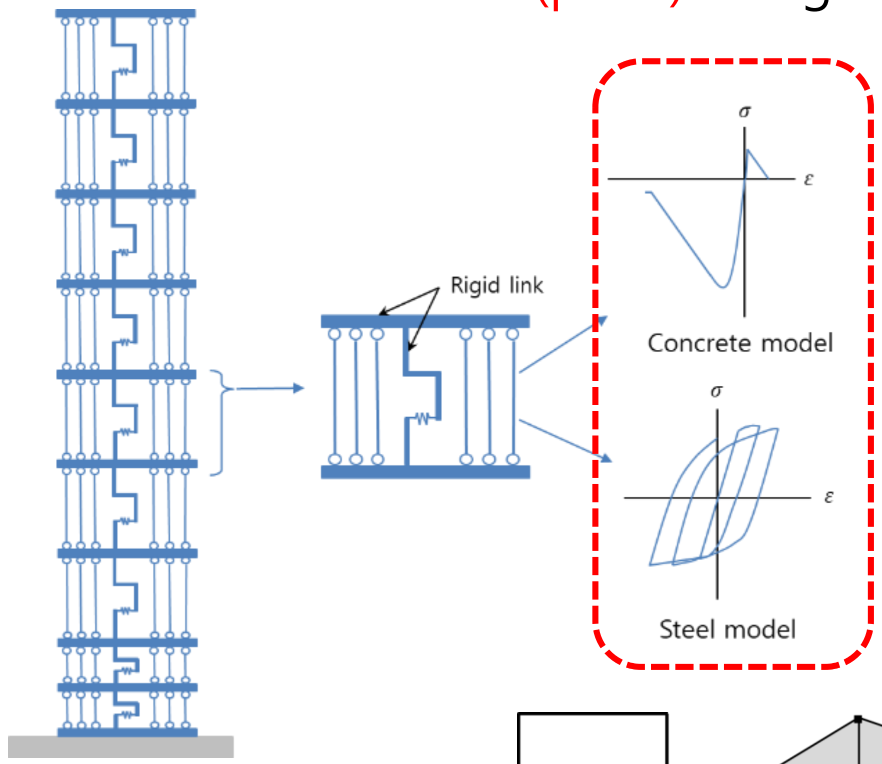


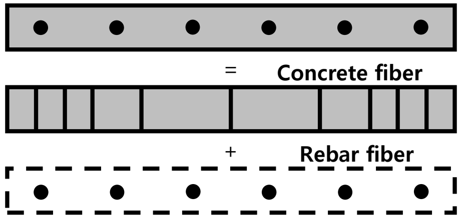
(b) Hysteresis model for moment hinge element

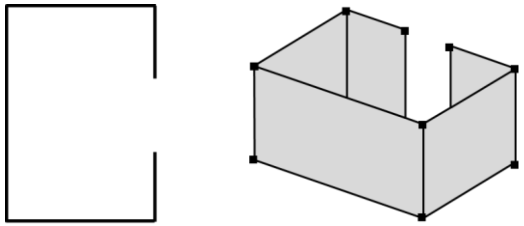
Fiber Model

Wall (plate) : single curvature

Column : double curvature

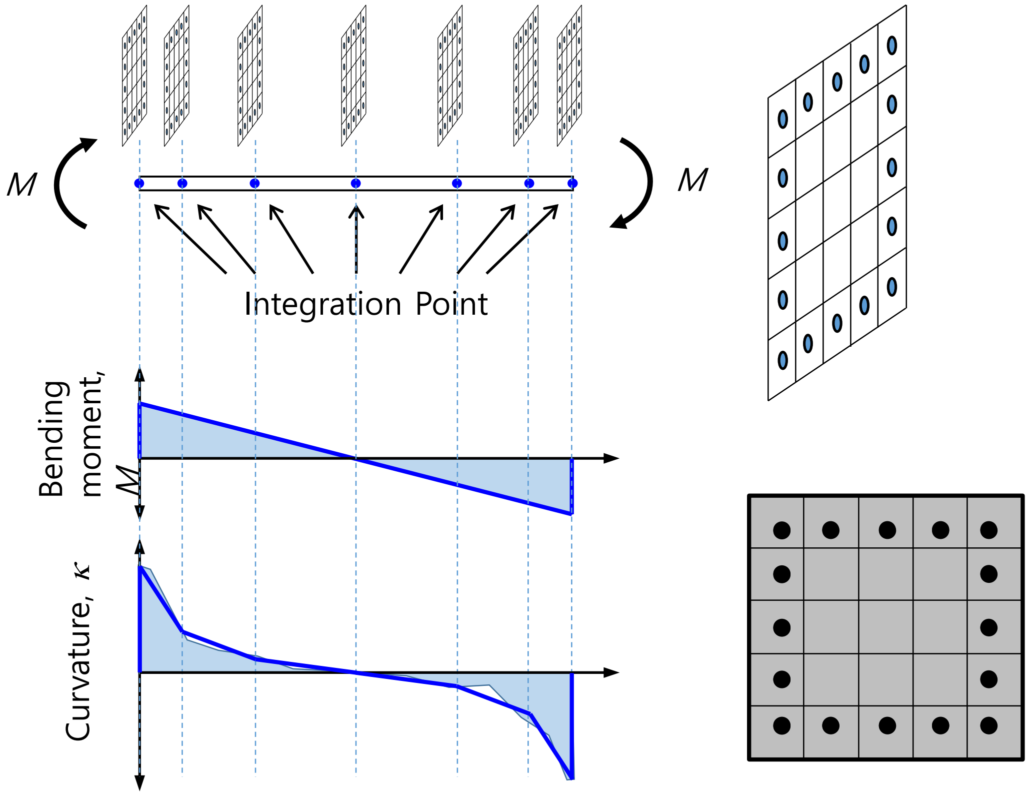






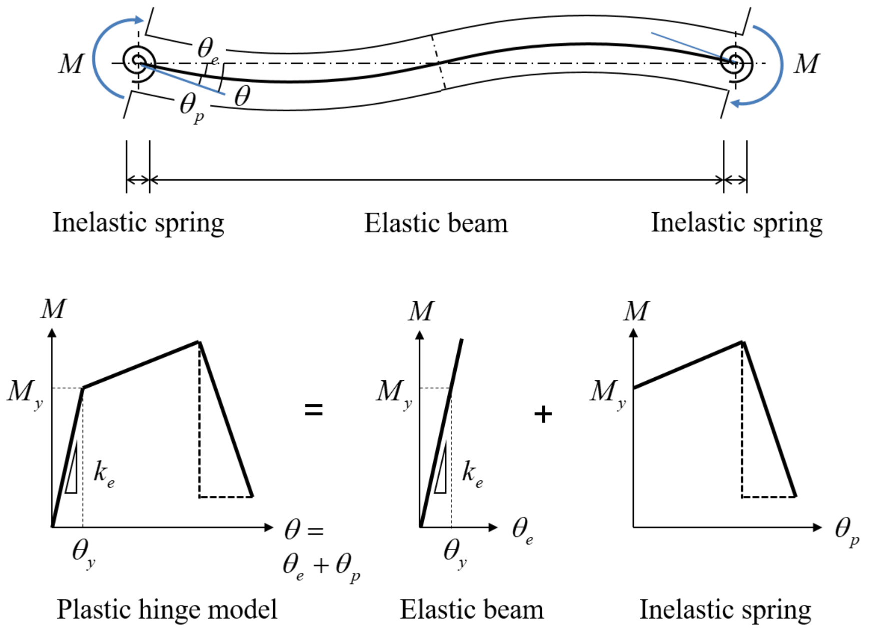
(a) Plan

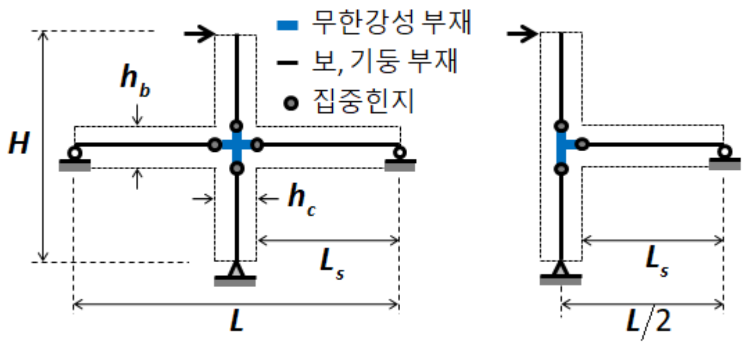
(b) Elements

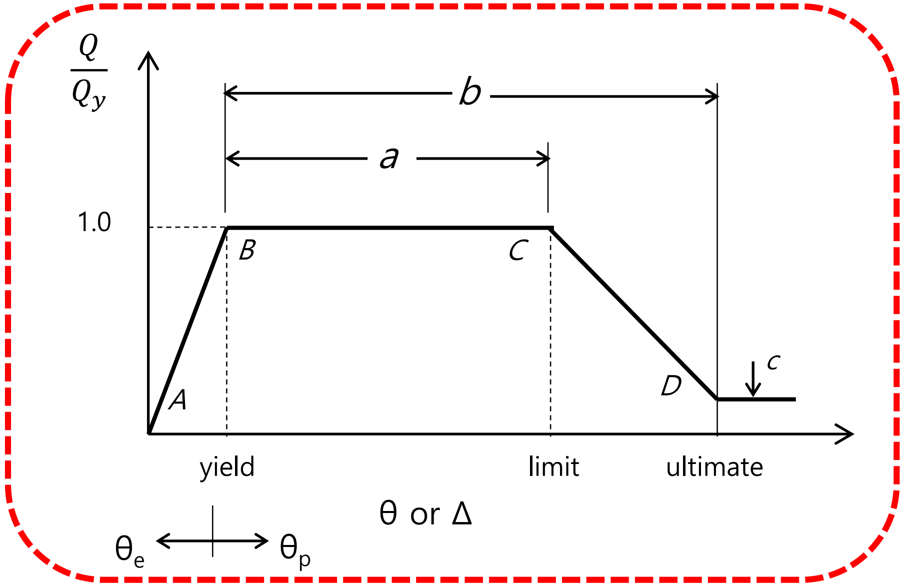


Plastic Hinge Model

- Plastic Hinge Model : beams, columns, beam-columns, walls







Plastic Hinge Model

- Effective Stiffness for elastic region

Considering the effect of concrete cracking.

	Flexural stiffness	Shear stiffness	Axial stiffness
Beams[1)]	$0.3E_cI_g$	GA_W	-
columns[2)]	$0.3E_cI_g \le (\eta + 0.2)(\frac{l_s}{4h})E_cI_g \le 0.7E_cI_g$	GA_W	E_cA_g
Walls without cracking	$0.70E_cI_g$	GA_W	E_cA_g
Walls with cracking	$0.35E_cI_g$	$0.5GA_W$	E_cA_g
Coupling beams[3)]	$0.3E_cI_g$	$0.04(\frac{l}{h})GA_W \le GA_W$	-
diaphragm[4)]	-	$0.25GA_W$	$0.25E_cA_g$

1) Non-prestressed beams

2) η = axial compression ratio, l_s = shear span = 1/2 column height, h = depth of cross section

3) : l = beam length h = beam depth

	Flexural stiffness	Shear stiffness	Axial stiffness
Beams[1)]	$0.3E_cI_g$	GA_W	-
columns[2)]	$0.3E_cI_g \le (\eta + 0.2)(\frac{l_s}{4h})E_cI_g \le 0.7E_cI_g$	GA_W	E_cA_g
Walls without cracking	$0.70E_cI_g$	GA_W	E_cA_g
Walls with cracking	$0.35E_cI_g$	$0.5GA_W$	E_cA_g
Coupling beams[3)]	$0.3E_cI_g$	$0.04(\frac{l}{h})GA_W \le GA_W$	-
diaphragm[4)]	-	$0.25GA_W$	$0.25E_cA_g$

1) Non-prestressed beams

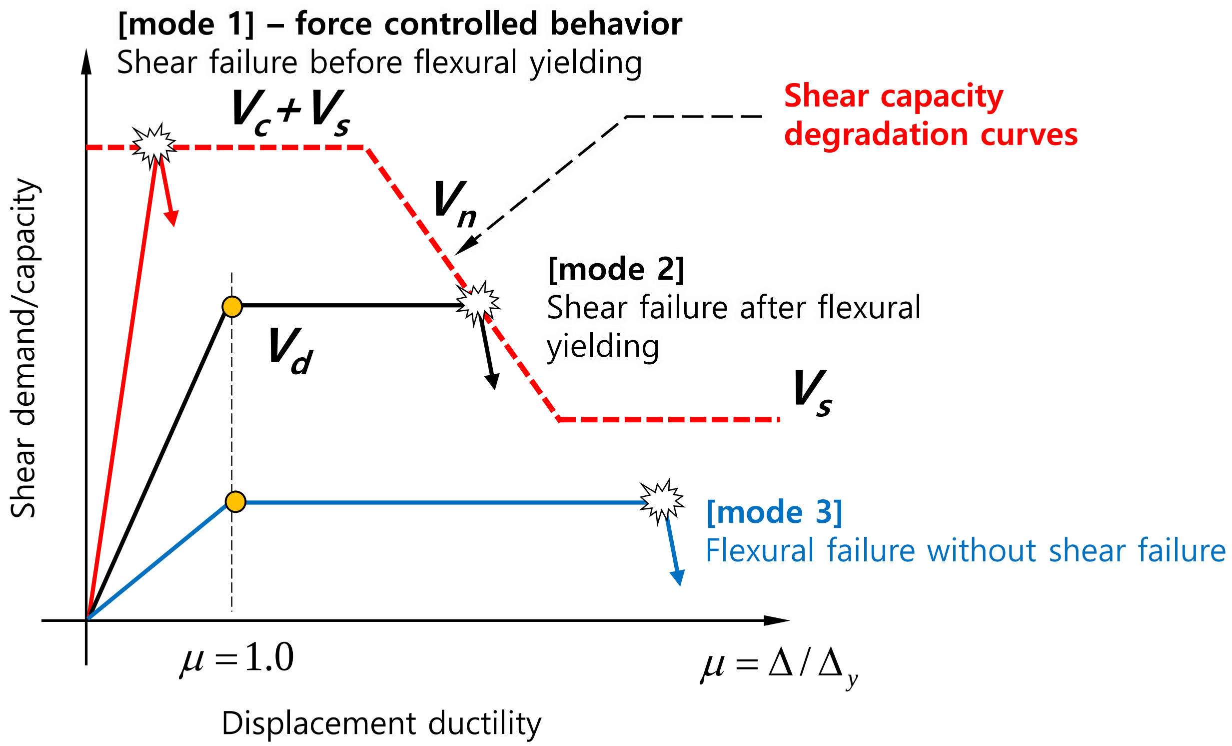
2) η = axial compression ratio, l_s = shear span = 1/2 column height, h = depth of cross section

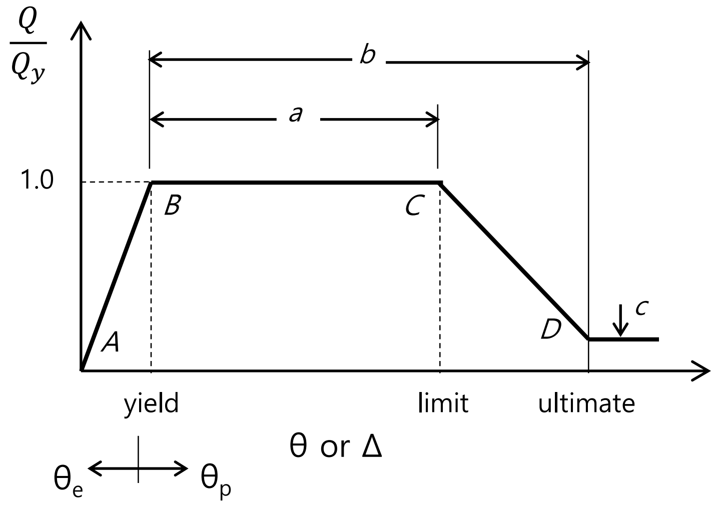
3) : l = beam length h = beam depth

Plastic Hinge Model : Beam

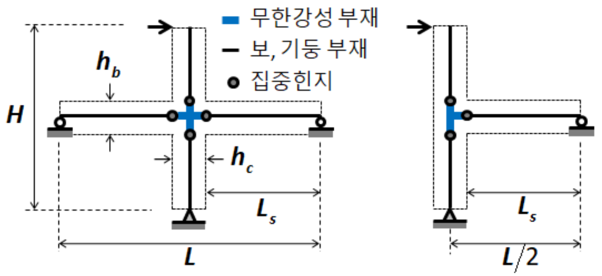
- Definition of Deformation Capacity :

  intersection between shear demand and shear capacity $V_n = V_d$



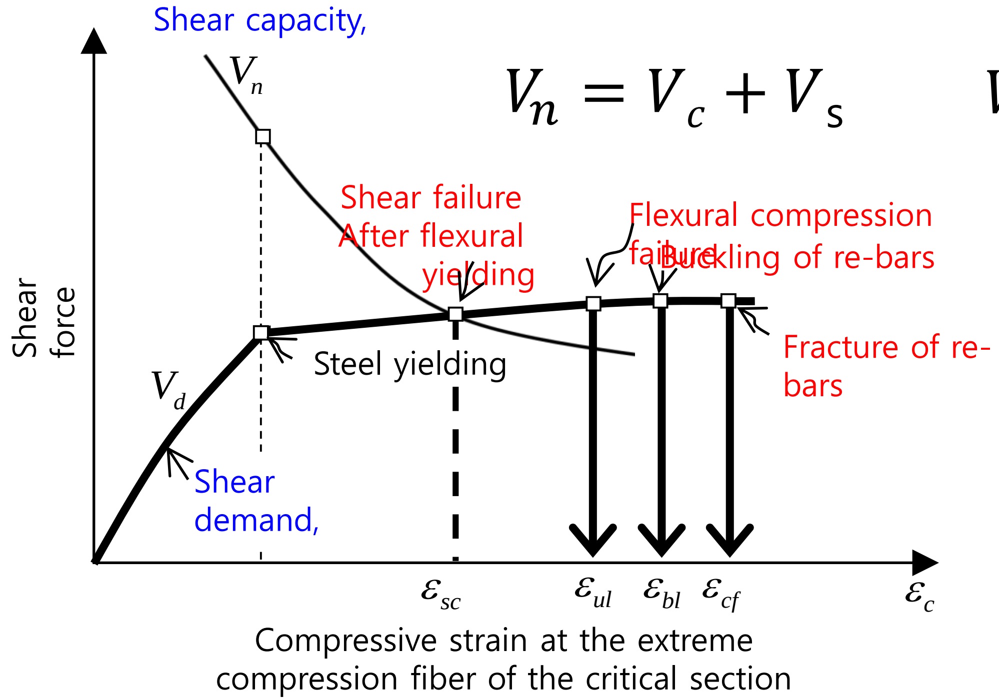


Plastic Hinge Model : Beam



- Definition of Deformation Capacity :

  intersection between shear demand and shear capacity $V_n = V_d$

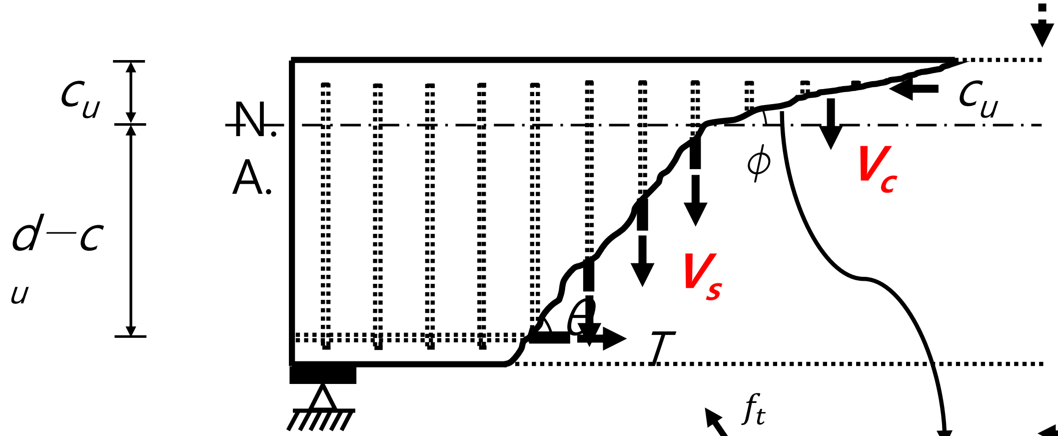


$$V_n = V_c + V_s$$

$$V_c = k_s f_t b_w c_u \cot\varphi \left(\frac{\varepsilon_o}{\varepsilon_{sc}}\right)$$

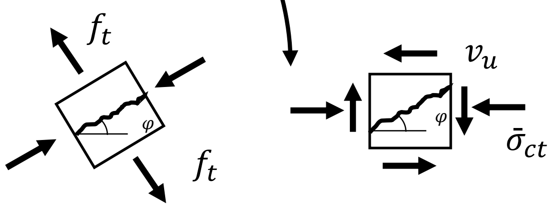
$$V_s = \frac{A_v f_{yt} d}{s}$$

Compression zone failure mechanism under shear



KDS, ACI 318-13 $V_c = 1/6\sqrt{f_c} b_w d$

$$f_t \cot\varphi = \sqrt{f_t(f_t + \bar{\sigma}_{ct})}$$

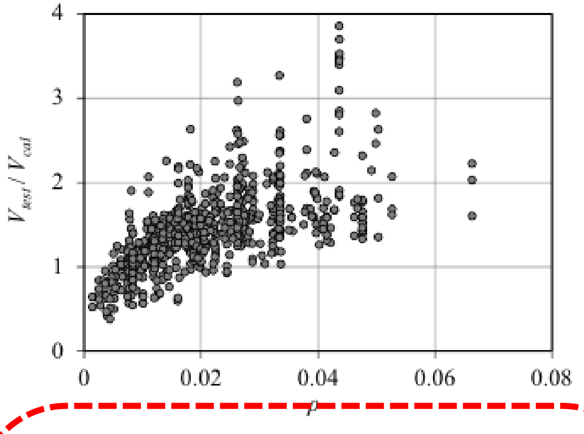


Plastic Hinge Model : Beam

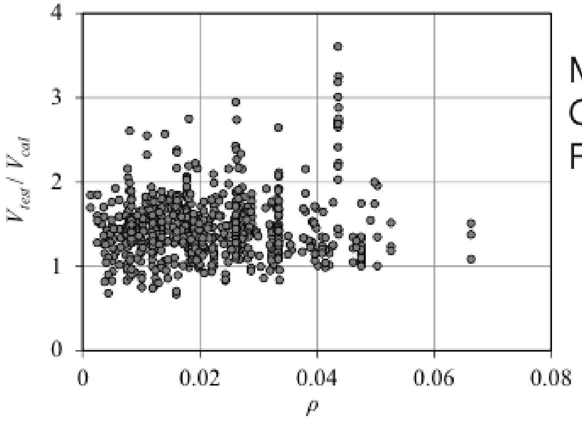
- Verification of shear strength model before flexural yielding:

KDS 14: current code

Mean: 1.46
COV: 0.341
FP: 4.97%



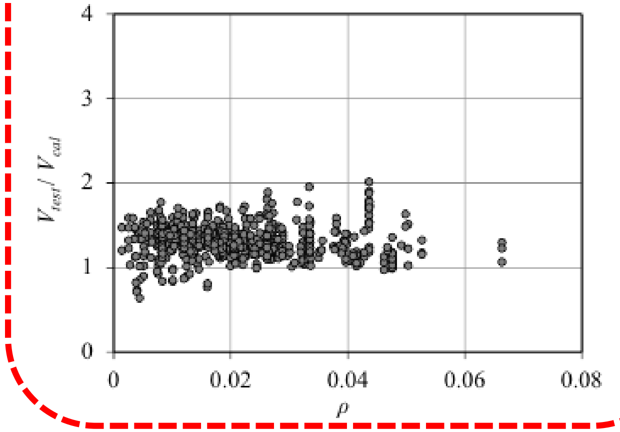
ACI 318-19



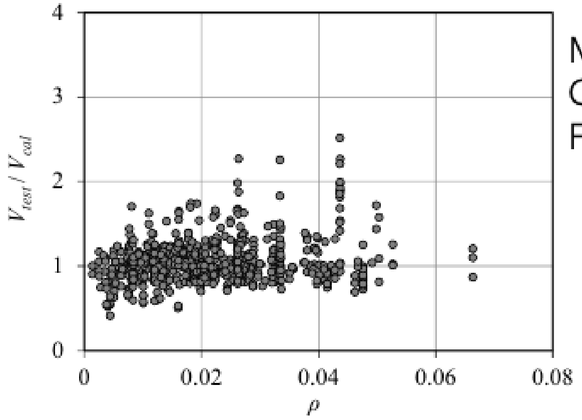
Mean: 1.45
COV: 0.246
FP: 0.51%

PBSD (KDS 14: next version)

Mean: 1.30
COV: 0.134
FP: 0.38%



EC-2



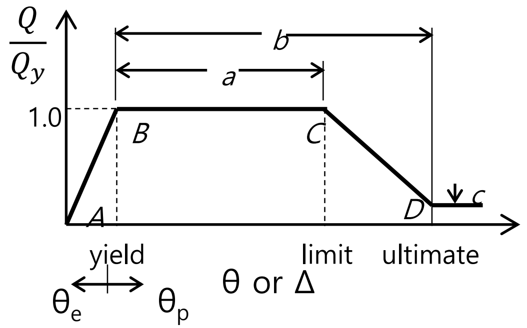
Mean: 1.04
COV: 0.231
FP: 4.59%

FP indicates failure probability; each test data is assumed to the failure case when its strength ratio is less the 0.75.

Plastic Hinge Model : Beam

	Modeling parameter			Allowable limit		
	Plastic rotation (rad.)		Residual strength	occupancy	Life safety	Collapse prevention
	a	b	c			
1. Shear failure after flexural yielding (non seismic details)						
Determined by ε_{cf}	$[\frac{\min(\varepsilon_{ul,NC}, \varepsilon_{cf}, \varepsilon_{bl}, \varepsilon_{sc})}{c_u} - \varphi_y]l_h \le 0.02$	a ≤ 0.03	0	a/3≥ 0.0017	a	b
otherwise		$(\frac{\varepsilon_{cf}}{c_u} - \varphi_y)l_h \le (2a, 0.03)$				
2. Shear failure after flexural yielding (seismic details)						
Determined by ε_{cf}	$[\frac{\min(\varepsilon_{ul}, \varepsilon_{cf}, \varepsilon_{bl}, \varepsilon_{sc})}{c_u} - \varphi_y]l_h$	a	0	a/3≥ 0.0035	a	b
otherwise		$(\frac{\varepsilon_{cf}}{c_u} - \varphi_y)l_h \le 2a$				

$\varepsilon_{ul,NC}$, = flexural compression failure
ε_{cf}, = rebar fracture
ε_{bl}, = rebar buckling
ε_{sc} = shear strength degradation



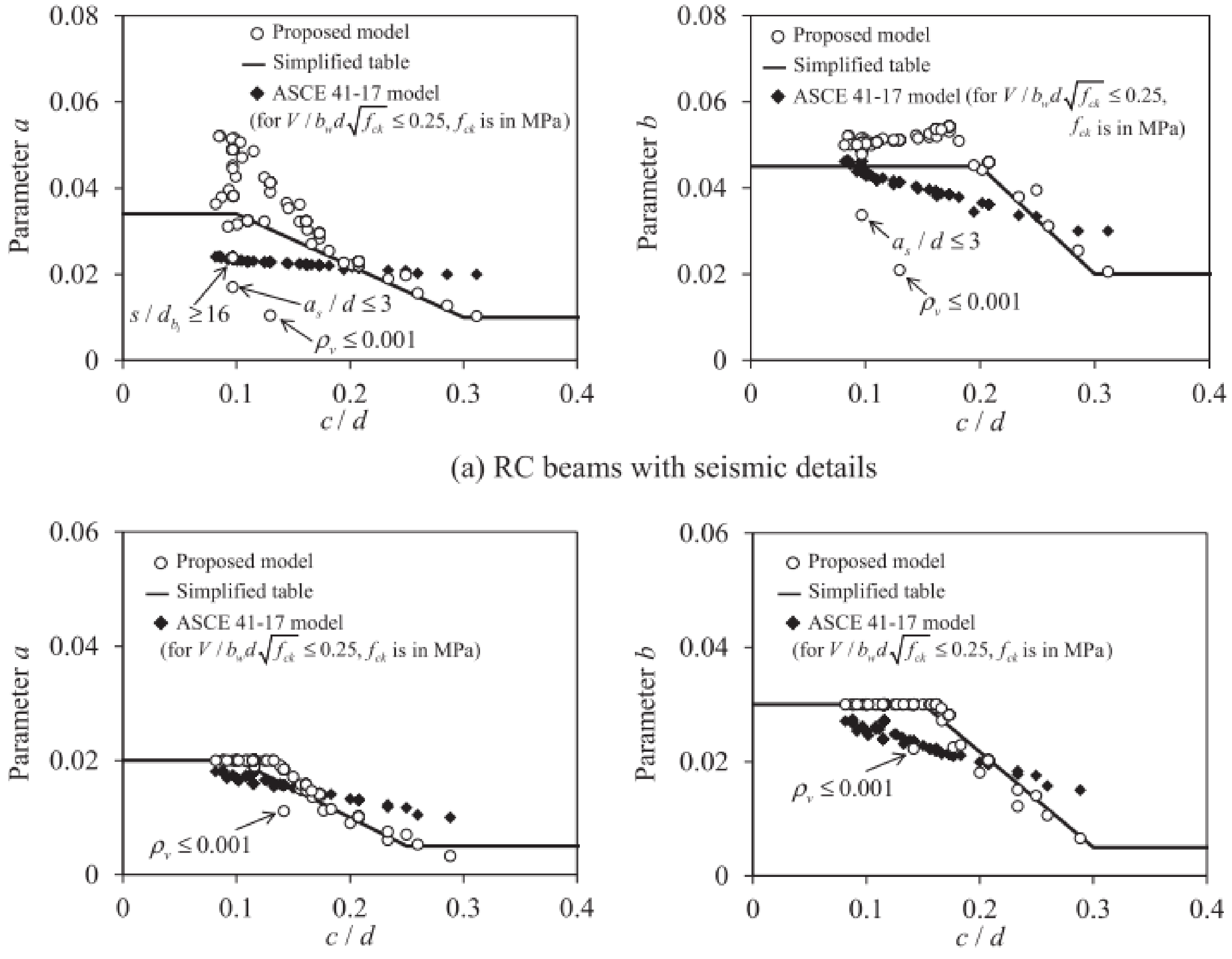
Plastic Hinge Model : Beam

Simplified modeling parameters available for $\rho_v \geq 0.0015, s/db \leq 16$, and $L/d \geq 6$

	Modeling parameter			Allowable limit		
	Plastic rotation (rad.)		Residual strength	occupancy	Life safety	Collapse prevention
	a	b	c			
1. Shear failure after flexural yielding (non seismic details)						
c_u/d				0.007	0.02	0.03
≤ 0.1	0.02	0.03		0.005	0.015	0.03
0.15	0.015	0.03	0	0.0017	0.005	0.013
0.25	0.005	0.013		0.0017	0.005	0.005
≥ 0.3	0.005	0.005				
2. Shear failure after flexural yielding (seismic details)						
c_u/d						
≤ 0.1	0.034	0.045		0.012	0.034	0.045
0.2	0.022	0.045	0	0.008	0.022	0.045
≥ 0.3	0.01	0.02		0.0035	0.01	0.02

Plastic Hinge Model : Beam

Simplified modeling parameters available for $\rho_v \geq 0.0015, s/db \leq 16$, and $L/d \geq 6$

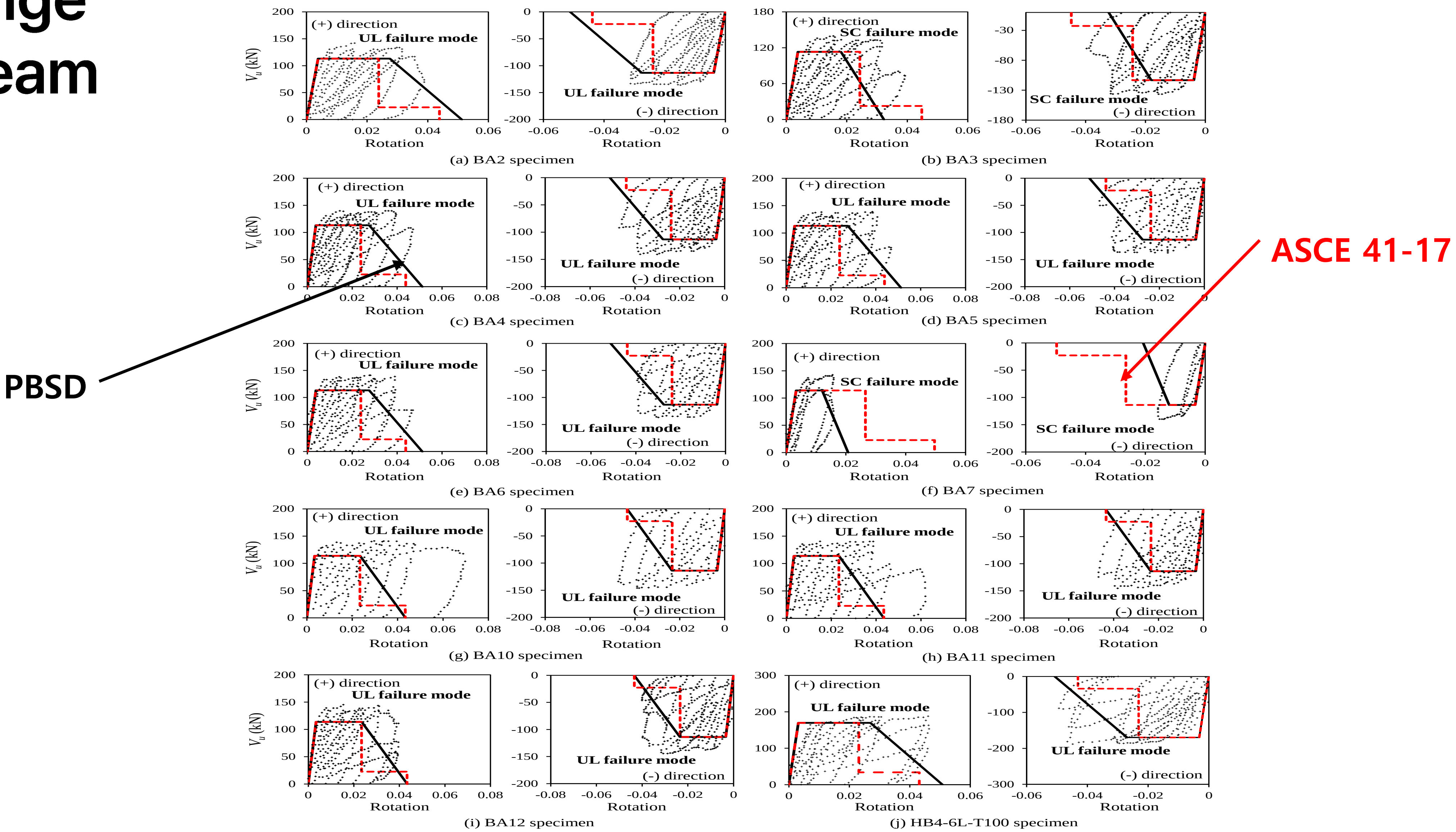


(a) RC beams with seismic details

(b) RC beams without seismic details

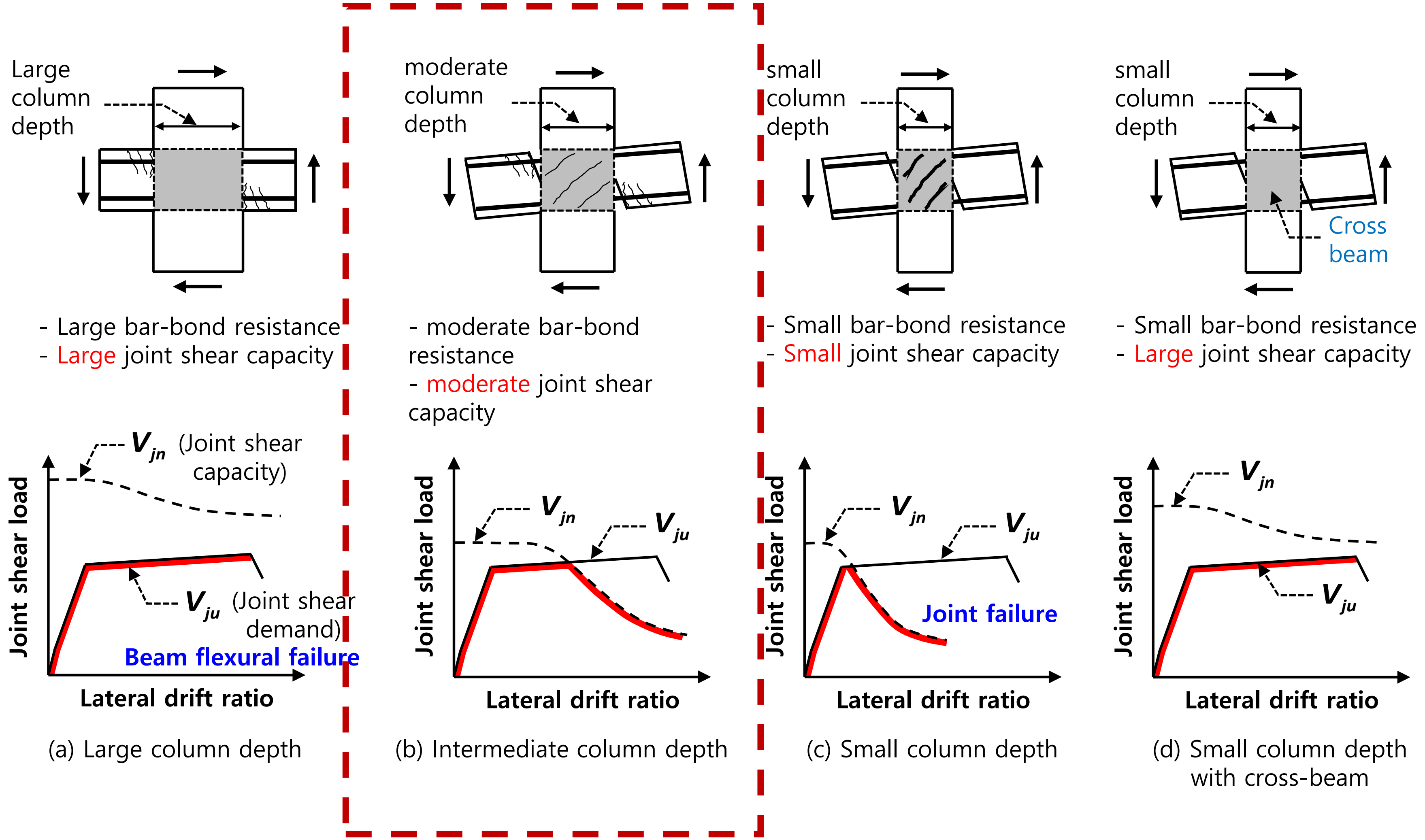
Plastic Hinge Model : Beam

- verification



Plastic Hinge Model :

Beam-Column Joint



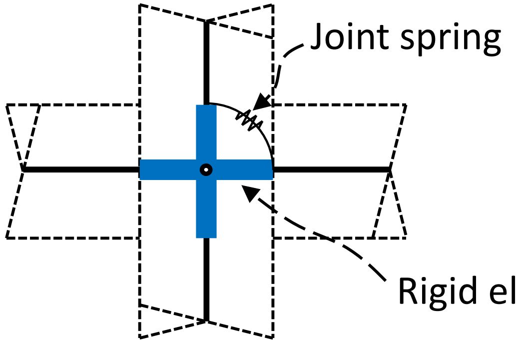
Focusing on joints at building perimeters with both joint shear damage and beam damage

In most cases, plastic beam deformation is accompanied by joint shear damage.

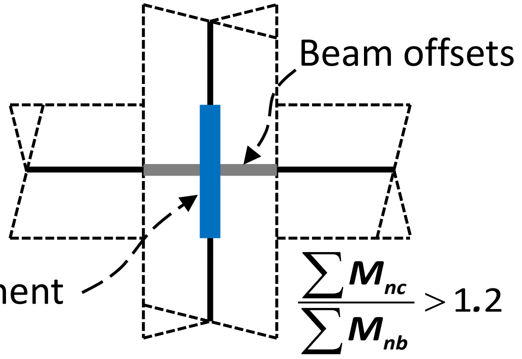
Plastic Hinge Model :

Beam-Column Joint

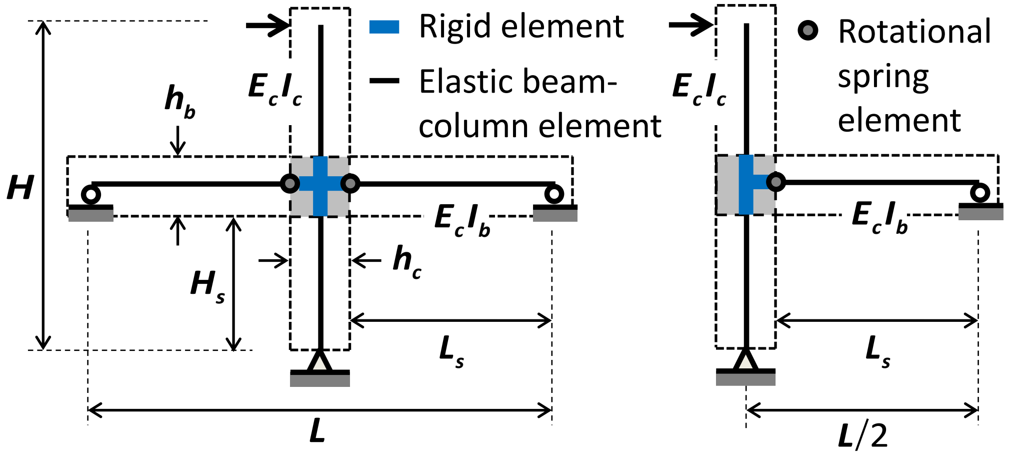
In most cases, plastic beam deformation is accompanied by joint shear damage.



Explicit joint model
(for inelastic analysis)



Offsets for implicit joint model
(for elastic analysis)



ASCE 41 model (Separated model)

Independent model of joint shear deformation separated from beam plastic deformation

PBSD model (Unified model)

Combined behavior of joint shear deformation and beam plastic deformation

Plastic Hinge Model :

Beam-Column Joint

- Shear strength model of ACI 318

$$V_{jn}\left(=0.083\gamma\sqrt{f_{ck}}\,b_j h_c\right)$$

No specific contribution of shear reinforcement
No strength degradation

- Proposed model

$$V_{jn}=V_C+V_T=\alpha_c\alpha_s f_{ck} b_s h_c+\min\left(A_h f_{yh}, 0.65A_s f_y\right)$$

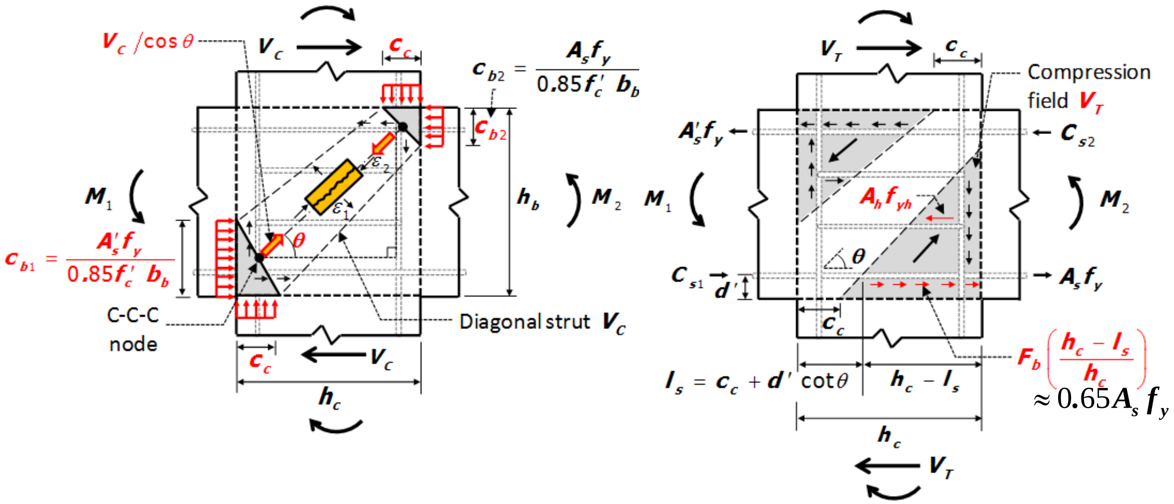
Diagonal strut mechanism

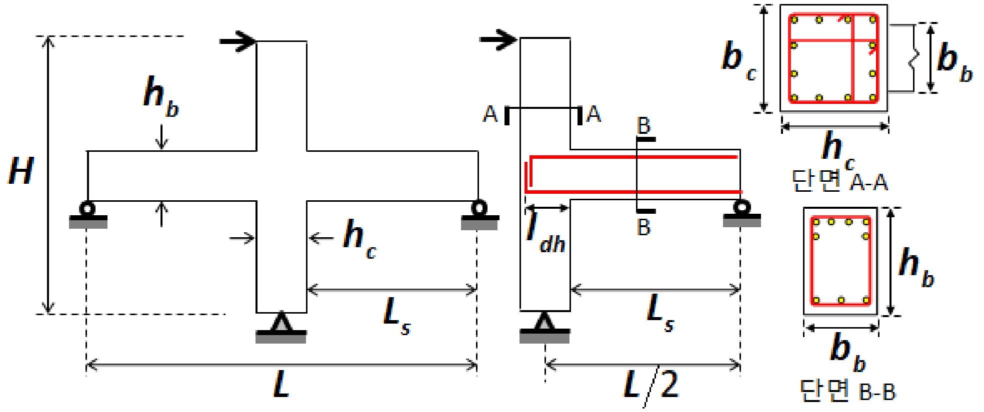
Truss mechanism

α_c = lateral confinement factor (1.0, 2.0, 2.5)

α_s = deformation degradation factor

b_s = effective width = $\min\left[\dfrac{b_b+b_c}{2}, b_b+\dfrac{h_c}{2}\right]$





Plastic Hinge Model :

Beam-Column Joint

- Joint shear capacity degradation

$$V_{jn} = V_C + V_T = \alpha_c \alpha_s f_{ck} b_s h_c + \min\left(A_h f_{yh}, 0.65 A_s f_y\right)$$

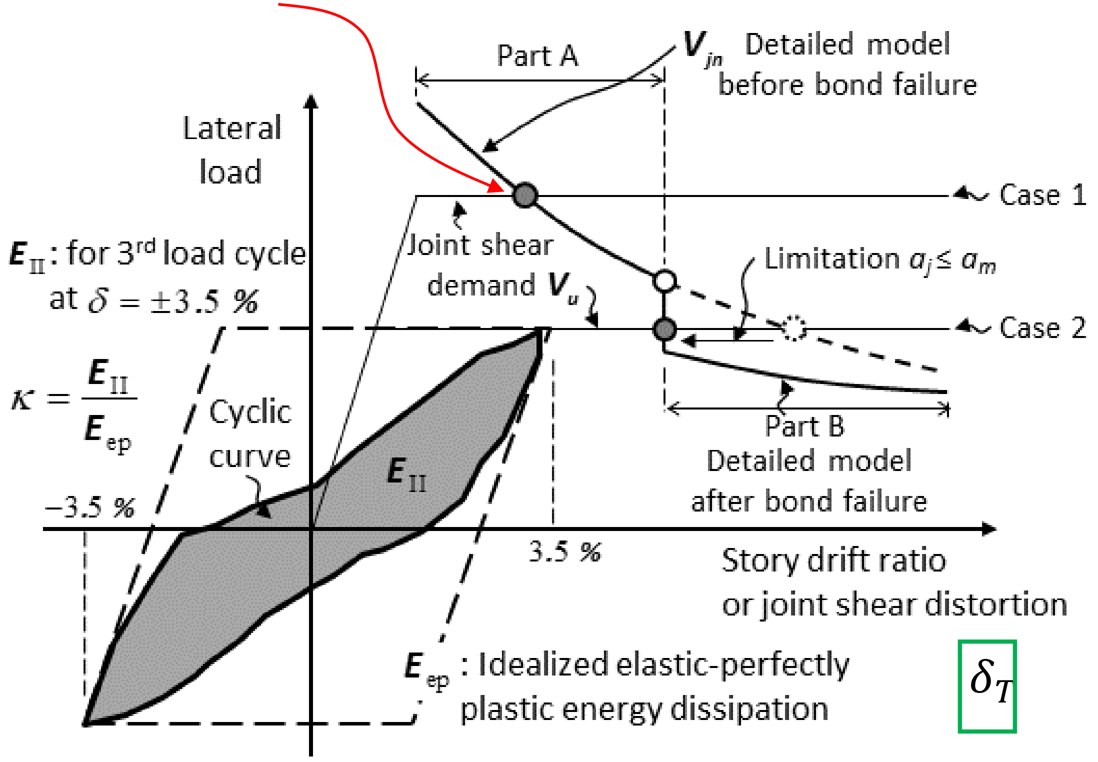
Strength degradation factor for interior joint

$$\alpha_s = \frac{0.11(6-\beta)(\beta_c + 0.2)}{100\boxed{\gamma_T} + 1.6}$$

Joint shear deformation (a part of δ_T)

$$\boxed{\gamma_T} = \left(\boxed{\delta_T} - \frac{\varepsilon_y L}{2h_b}\right) \frac{3-5\kappa}{6-3h_c/L-3h_b/H}$$

Joint shear deformation at joint shear capacity = joint shear demand



Joint shear strength degradation

Plastic Hinge Model :

Beam-Column Joint

- Joint Deformation

			Modeling parameter	Allowable limit		
				occupancy	Life safety	Collapse prevention
Joint shear deformation at joint shear capacity = joint shear demand	Interior joint (rad.)	a_j	$0 \le \frac{1}{1050}[\frac{1.1\alpha_c(6-\beta)(\beta_c+0.2)f_{ck}b_s h_c}{V_u - V_T} - 16] \le a_m$	0.0	$0.5\ b_j$ $< a_j$	$0.7\ b_j$ $\ge a_j$
		b_j	$a_j + 0.01$			
	Exterior joint (rad.)	a_j	$0 \le \frac{1}{1050}[\frac{2.8\alpha_c(3-\beta)(\beta_c+0.1)f_{ck}b_s h_c}{V_u - V_T} - 8] \le a_m$			
		b_j	$a_j + 0.01$			
Joint shear deformation at bond failure	Max. a (rad.)	a_m	$0.03(1 - 0.1\frac{a_j}{a_{pb}}) \ge 0.01$			
	a at Bond failure	a_{pb}	$a_t f_y(\kappa - 0.13)(3 - 5\kappa)^{\frac{1-0.45(h_b/L_s)}{3400}}$ (interior)	$a_t f_y(\kappa - 0.12)(3 - 5\kappa)\frac{1-0.45(h_b/L_s)}{3000}$ (exterior)		

Plastic Hinge Model :

Beam-Column Joint

- Deformation of beam affected by joint damage (penetration of yielding of beam rebars)

  overall deformation = joint shear + beam end

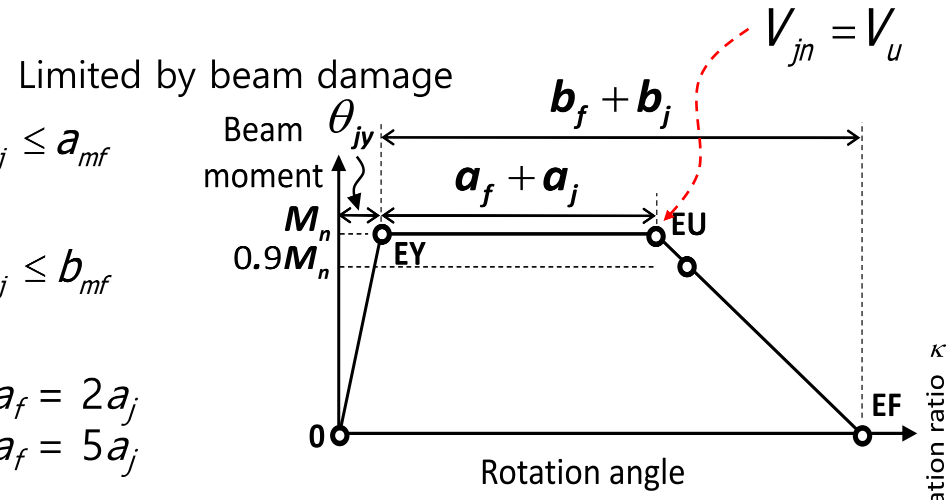
$$a = a_j + a_f$$

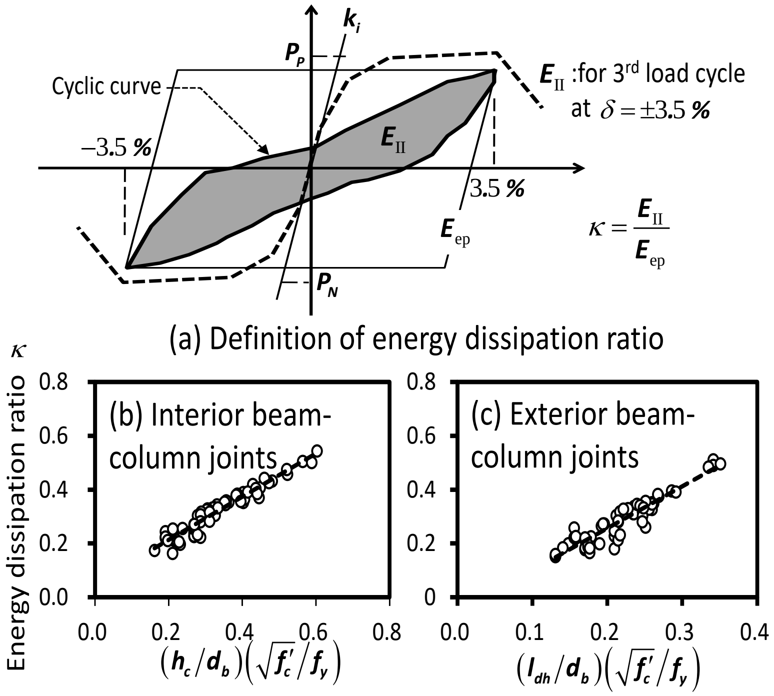
$$b = b_j + b_f$$

$$a_f = \frac{3+5\kappa}{3-5\kappa} a_j \le a_{mf}$$

$$b_f = \frac{3+5\kappa}{3-5\kappa} b_j \le b_{mf}$$

For κ= 0.2, $a_f = 2a_j$
For κ= 0.4, $a_f = 5a_j$





(a) Definition of energy dissipation ratio

Energy dissipation factor

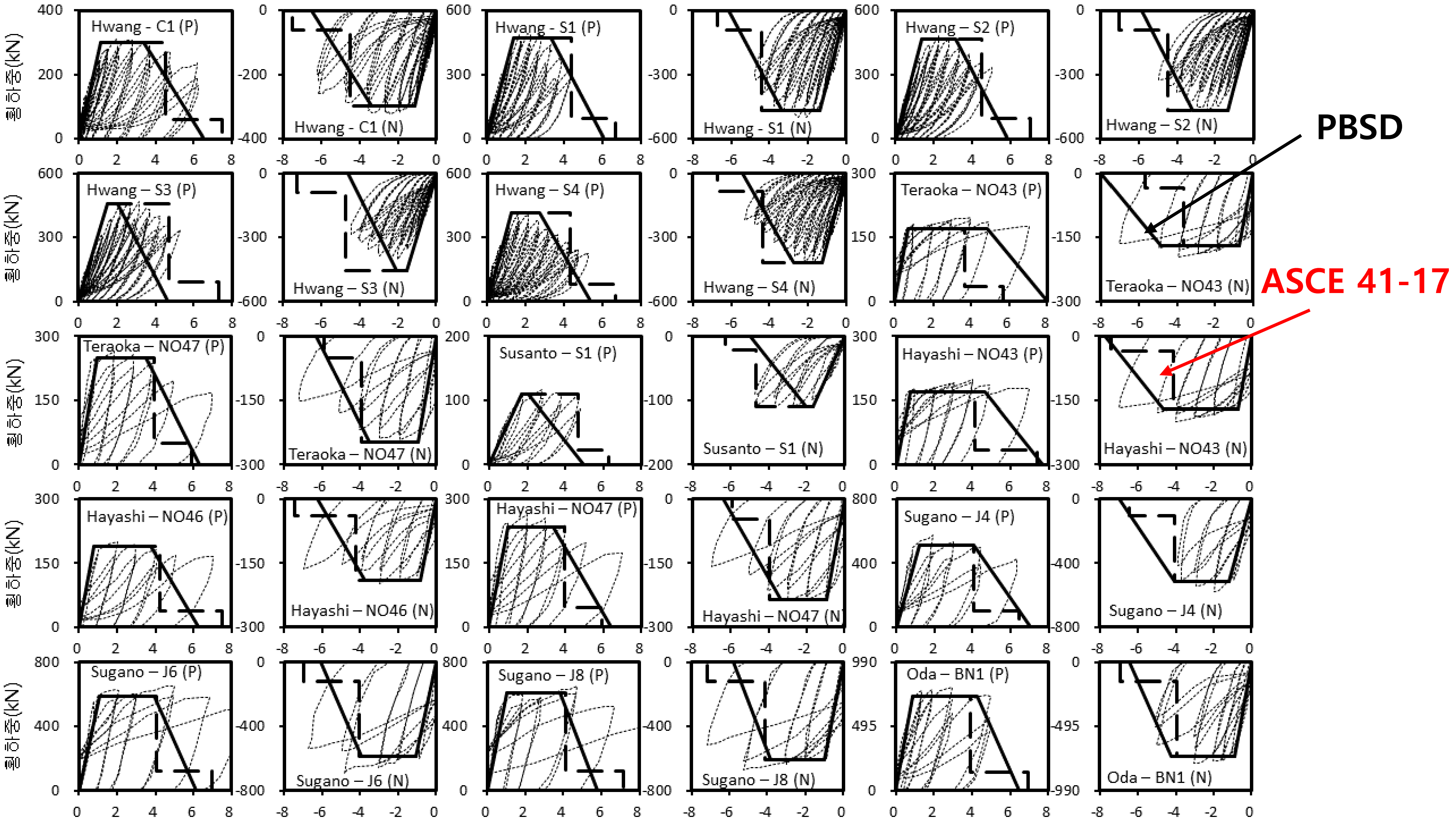
$$\kappa = 0.15 \le 0.8 \boxed{\frac{h_c}{d_b}\frac{\sqrt{f_{ck}}}{f_y}} + 0.05 \le 0.5 \qquad \kappa = 0.15 \le 1.56 \boxed{\frac{l_{dh}}{d_b}\frac{\sqrt{f_{ck}}}{f_y}} - 0.06 \le 0.5$$

Bond-resistance parameter

Plastic Hinge Model :

Beam-Column Joint

- Verification for Interior joints



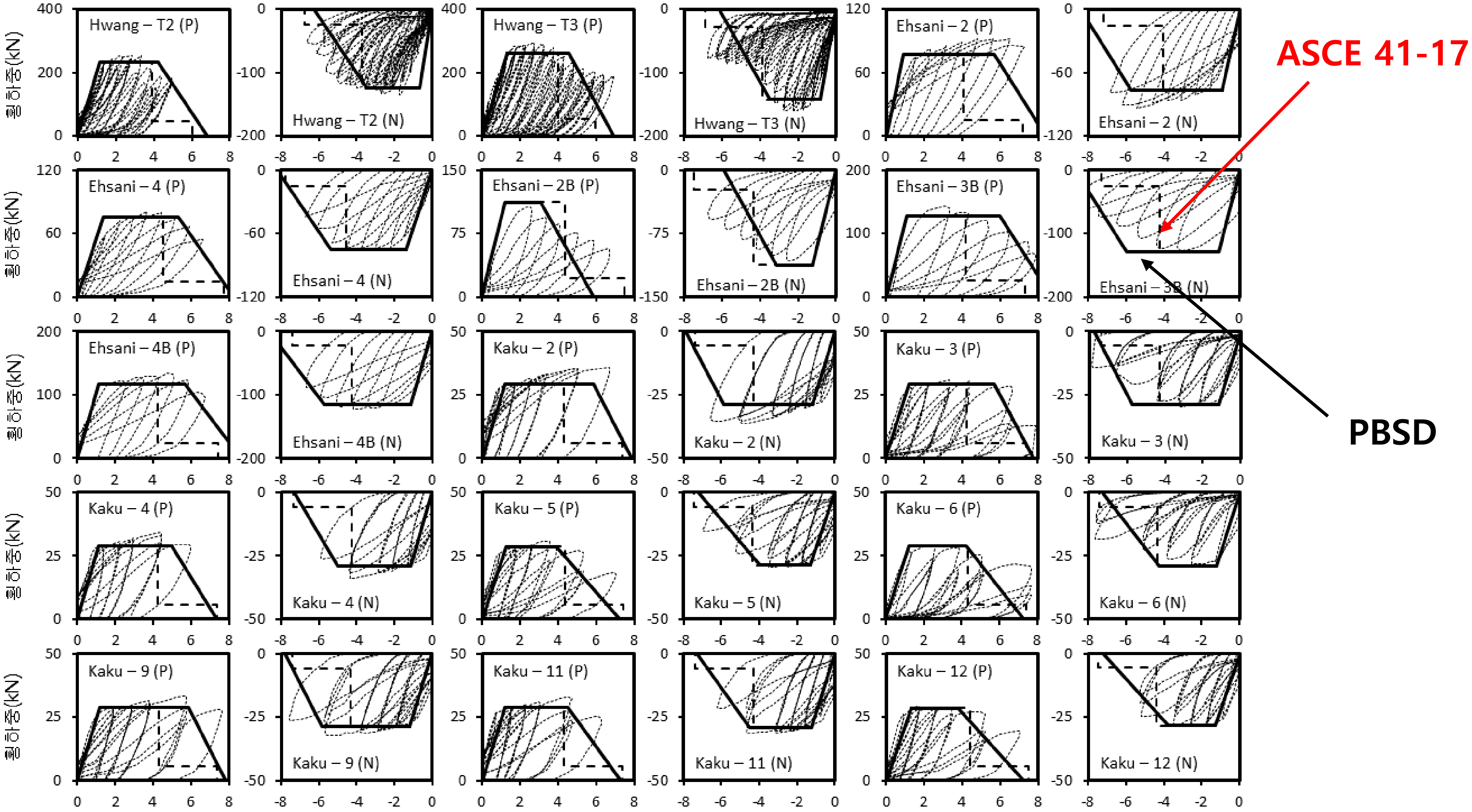
ASCE 41 overestimates the deformation capacity of interior joints showing large bond-slip due to use of high-strength bars and large diameter bars

Plastic Hinge Model :

Beam-Column Joint

- Verification for Exterior joints

ASCE 41 underestimates the deformation capacity of exterior joints



Plastic Hinge Model : Column

- Shear strength

$$V_c = 0.17\left(1+\frac{N_u}{14A_g}\right)\sqrt{f_{ck}}bd \quad \text{and} \quad V_s = f_{yt}A_v\frac{d}{s}$$

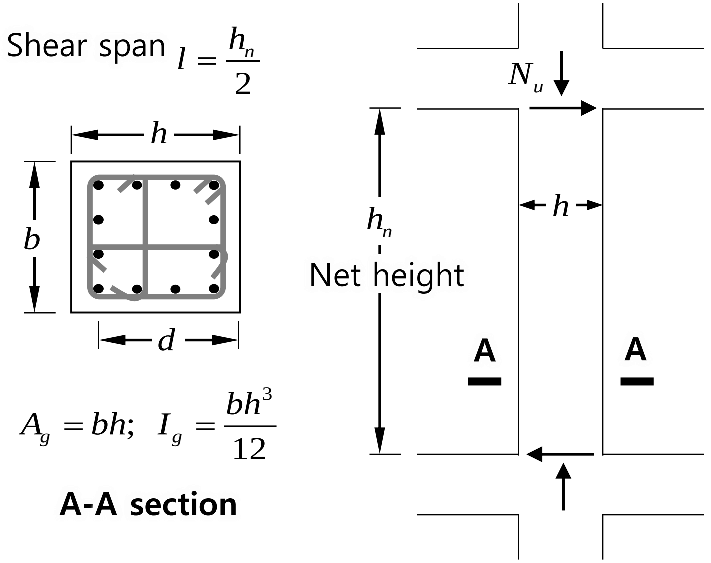
- Compression ratio and shear span

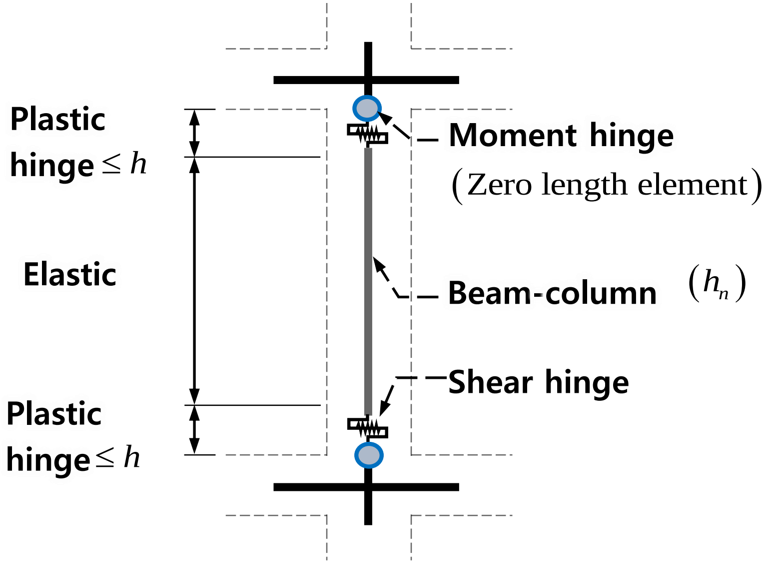
$$\eta = \frac{N_u}{f_{ck}A_g} \qquad l_s = 0.5h_n$$

- Effective stiffness, yield deformation

$$EI_e = \alpha EI_g \quad \text{where} \quad \alpha = (\eta+0.2)\left(\frac{l_s}{4h}\right) \quad (0.3 \le \alpha \le 0.7)$$

$$\theta_y = \frac{\Delta_y}{l} = \frac{V_y}{3EI_e/l^2} \quad \text{or} \quad \frac{V_y}{K_e}$$

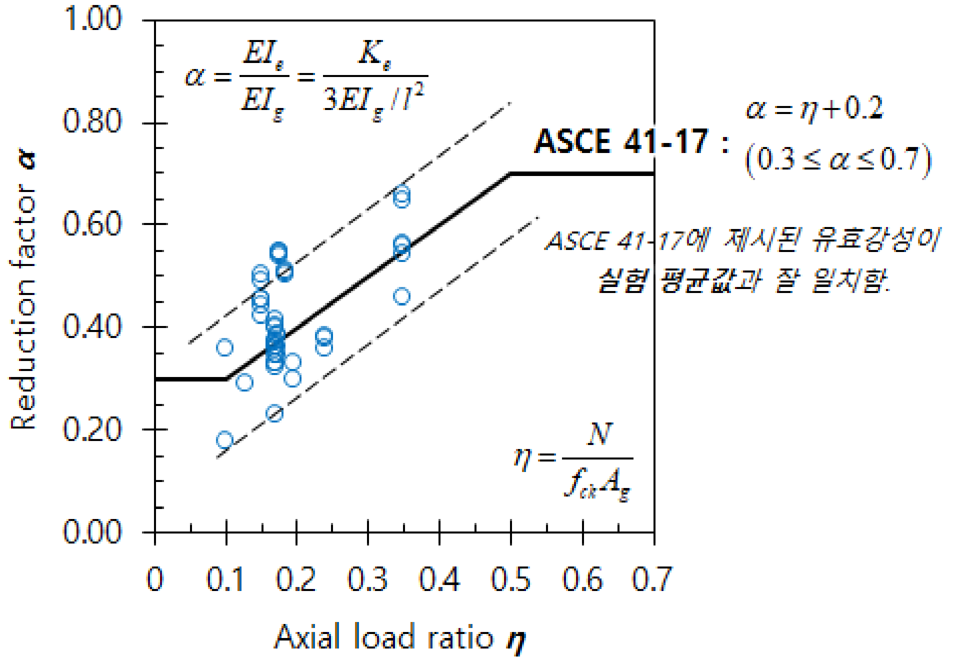


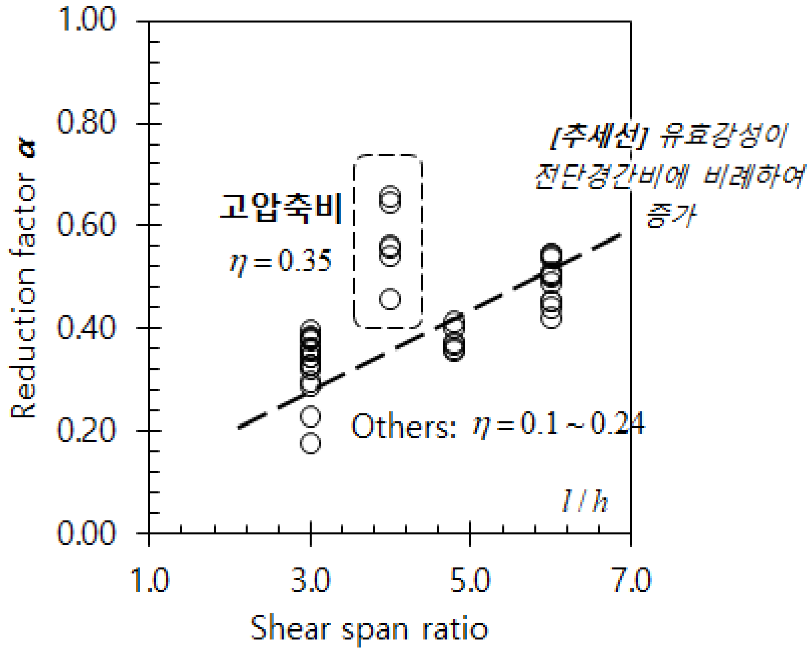


Plastic Hinge Model : Column

- Effective stiffness
  - Effective stiffness of column depends on shear span ratio as well as compression force ratio.

$$0.3E_cI_g \leq (\eta + 0.2)(\frac{l_s}{4h})E_cI_g \leq 0.7E_cI_g$$





Plastic Hinge Model : Column

- Deformation capacity according to failure mode

① **Failure mode 1 : shear failure before yielding**

$$V_c + V_s < V_y \qquad V_y = M_n / l$$

② **Failure mode 2 : shear failure after flexural yielding**

$V_c + V_s \geq V_y$ and $V_s \leq V_y$ » Shear resistance index

$$\mu = \begin{cases} 5 - 3\left(\frac{V_y - V_s}{V_c}\right) & \text{for seismic details} \\ 4 - 2\left(\frac{V_y - V_s}{V_c}\right) & \text{for non-seismic details} \end{cases} \quad \text{and} \quad \theta_a = \frac{\Delta_a}{l} = \mu\theta_y$$

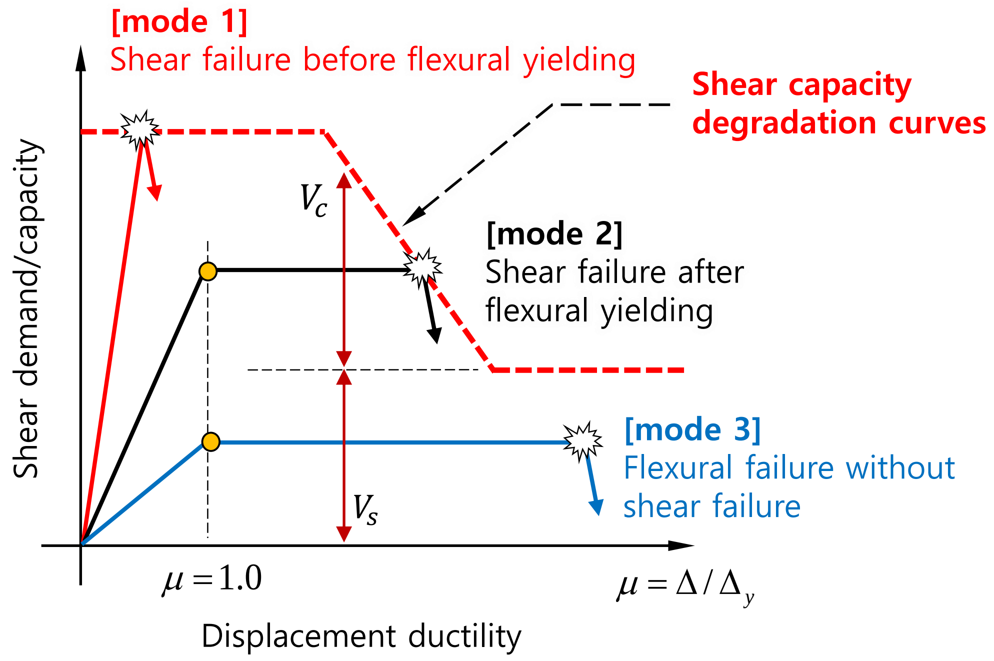
③ **Failure mode 3 : flexural failure without shear failure**

$V_c + V_s \geq V_y$ and $V_s > V_y$

$$\mu = \begin{cases} 5 & \text{for seismic details} \\ 4 & \text{for non-seismic details} \end{cases} \quad \text{and} \quad \theta_a = \frac{\Delta_a}{l} = \mu\theta_y$$

④ **Failure mode 4 : compression sliding failure**

$$\theta_b = \frac{\Delta_u}{l} = \frac{0.48}{4.6 + \frac{N_u}{V_s}}$$



$$a = \theta_a - \theta_y \qquad V_c + V_s \geq V_y$$

$$b = \theta_b - \theta_y$$

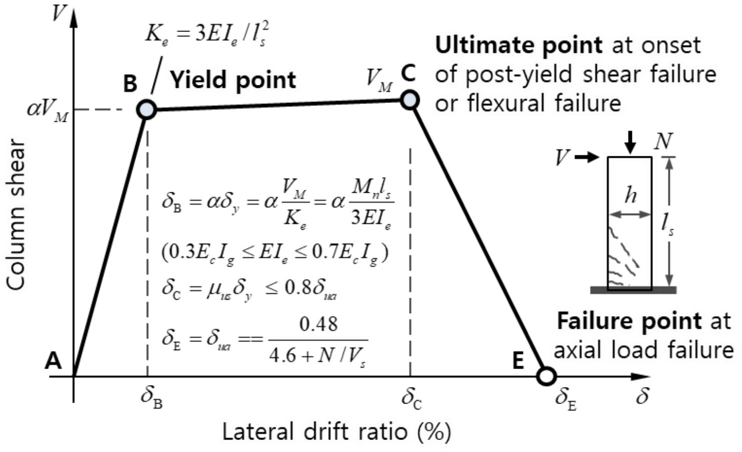
- Modeling parameters a and b are determined by yield rotation and ductility

Plastic Hinge Model : Column

- Deformation capacity according to failure mode

PBSD

Function of ductility



(a) Proposed method

구분	a	b	c
$V_c + V_s < V_y$	-	-	-
$V_c + V_s \ge V_y$	$\theta_u - \theta_y$	$\theta_a - \theta_y$	-

$$\delta_B = \alpha\delta_y = \alpha\frac{V_M}{K_e} = \alpha\frac{M_n l_s}{3EI_e}$$

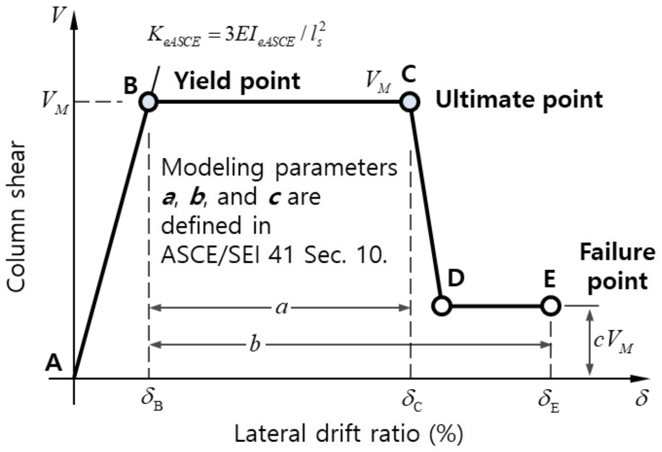
$$(0.3E_c I_g \le EI_e \le 0.7E_c I_g)$$

$$\delta_C = \mu_{us}\delta_y \le 0.8\delta_{ua}$$

$$\delta_E = \delta_{ua} == \frac{0.48}{4.6 + N/V_s}$$

$$\mu_{us} = \begin{cases} 5 - 3\left(\frac{V_y - V_s}{V_c}\right) \le 5 & \text{for seismic details} \\ 4 - 2\left(\frac{V_y - V_s}{V_c}\right) \le 4 & \text{for non-seismic details} \end{cases}$$

ASCE



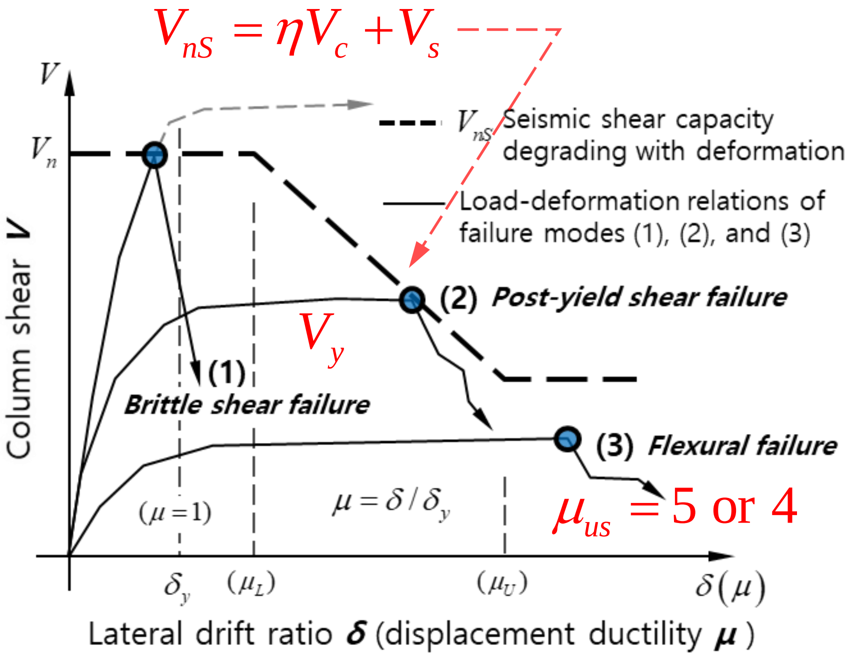
(b) ASCE/SEI 41

Table 10-8. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns Other Than Circular with Spiral Reinforcement or Seismic Hoops as Defined in ACI 318

Modeling Parameters	Acceptance Criteria		
	Plastic Rotation Angle (radians)		
	Performance Level		
Plastic Rotation Angles, a and b (radians) Residual Strength Ratio, c	IO	LS	CP
Columns not controlled by inadequate development or splicing along the clear height[a]			
$a = \left(0.042 - 0.043\frac{N_{UD}}{A_g f'_{cE}} + 0.63\rho_t - 0.023\frac{V_{yE}}{V_{ColOE}}\right) \ge 0.0$	$0.15\ a$ ≤ 0.005	$0.5\ b$[b]	$0.7\ b$[b]
For $\frac{N_{UD}}{A_g f'_{cE}} \le 0.5 \left\{ b = \frac{0.5}{5 + \frac{N_{UD}}{0.8A_g f'_{cE}}\frac{1}{\rho_t}\frac{f'_{cE}}{f_{ytE}}} - 0.01 \ge a^{a}\right.$			
$c = 0.24 - 0.4\frac{N_{UD}}{A_g f'_{cE}} \ge 0.0$			
Columns controlled by inadequate development or splicing along the clear height[c]			
$a = \left(\frac{1}{8}\frac{\rho_t f_{ytE}}{\rho_l f_{ylE}}\right) \begin{matrix} \ge 0.0 \\ \le 0.025^{d} \end{matrix}$	0.0	$0.5\ b$	$0.7\ b$
$b = \left(0.012 - 0.085\frac{N_{UD}}{A_g f'_{cE}} + 12\rho_t^{e}\right) \begin{matrix} \ge 0.0 \\ \ge a \\ \le 0.06 \end{matrix}$			
$c = 0.15 + 36\rho_t \le 0.4$			

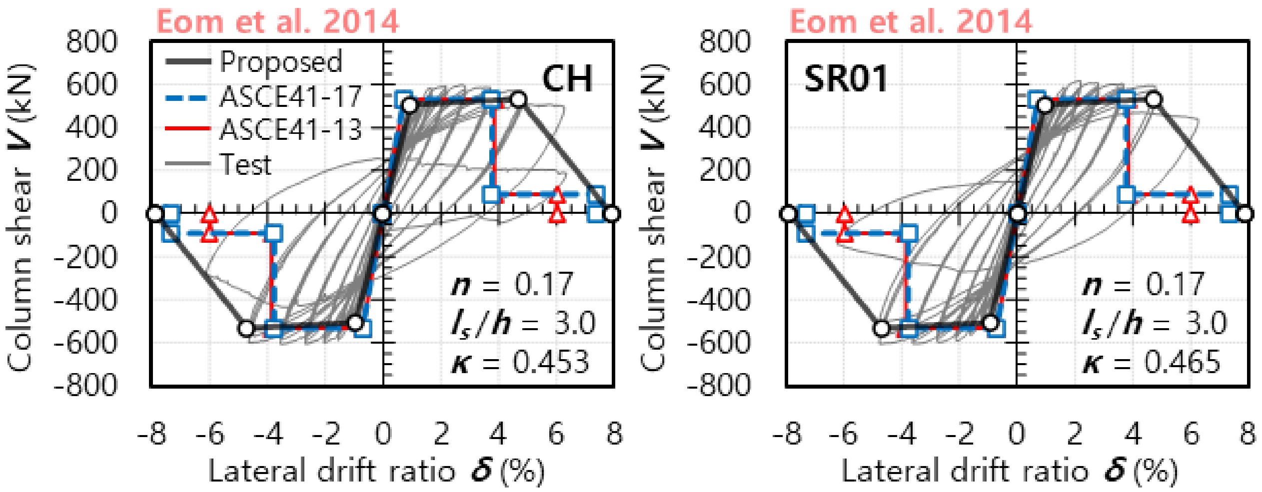
Plastic Hinge Model : Column

- verification



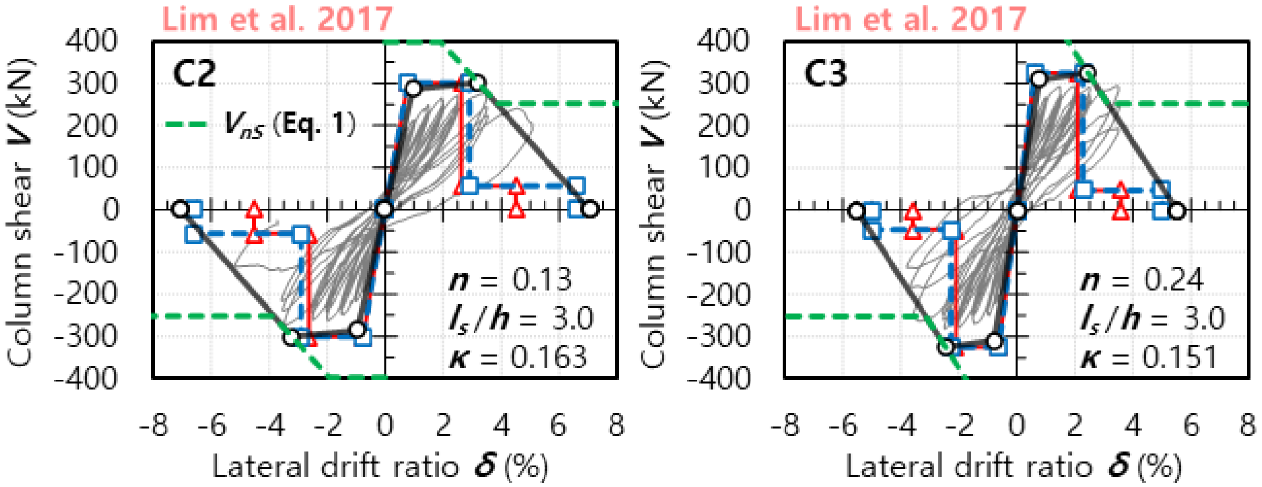
Flexural failure without shear failure

Failure mode ③



Shear failure after flexural yielding

Failure mode ②



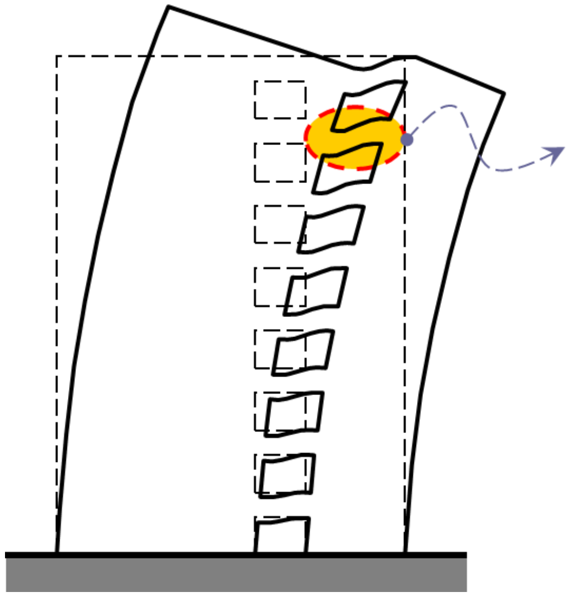
Plastic Hinge Model :

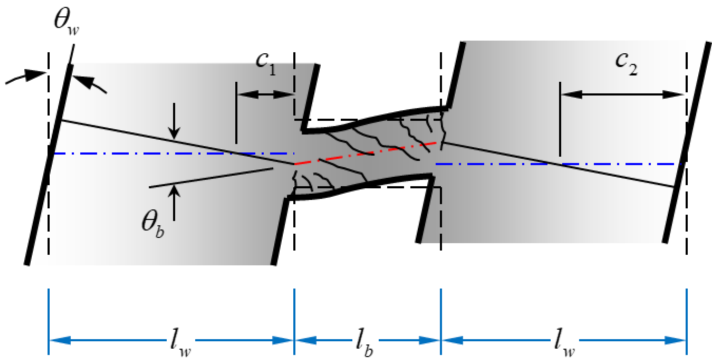
Coupling Beam

- Deformation of coupling beam

Deformation amplification

$$\theta_b = \frac{c_1\theta_w + (l_w - c_2)\theta_w}{l_b} \approx \boxed{\frac{l_w}{l_b}}\theta_w$$



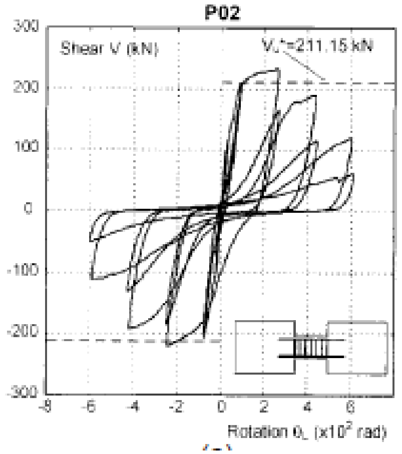


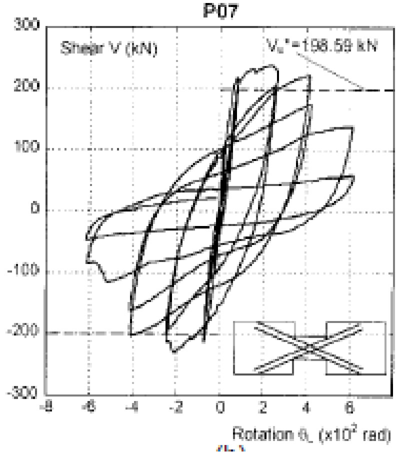
θ_b, θ_w :Rotations of coupling beam and wall

l_b, l_w :Lengths of coupling beam and wall

$$\theta_b = \frac{c_1\theta_w + (l_w - c_2)\theta_w}{l_b} \approx \frac{l_w}{l_b}\theta_w$$

Deformation angle of coupling beam



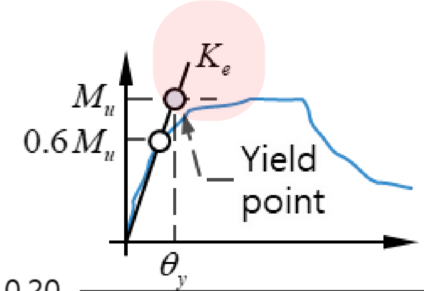


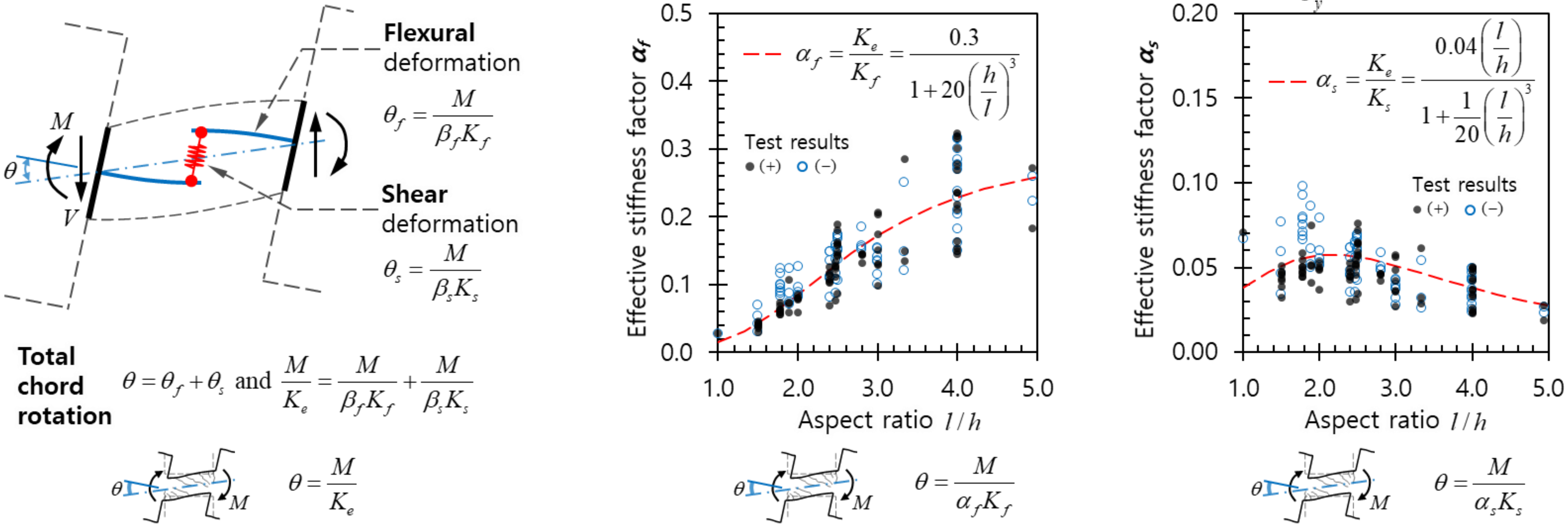
Plastic Hinge Model :
Coupling Beam

- Effective Stiffness

PBSD $EI_e = 0.3EI_g$ and $GA_e = 0.04\left(\dfrac{l}{h}\right)GA_w$

ASCE 41 $EI_e = 0.3EI_g$ and $GA_e = GA_w$



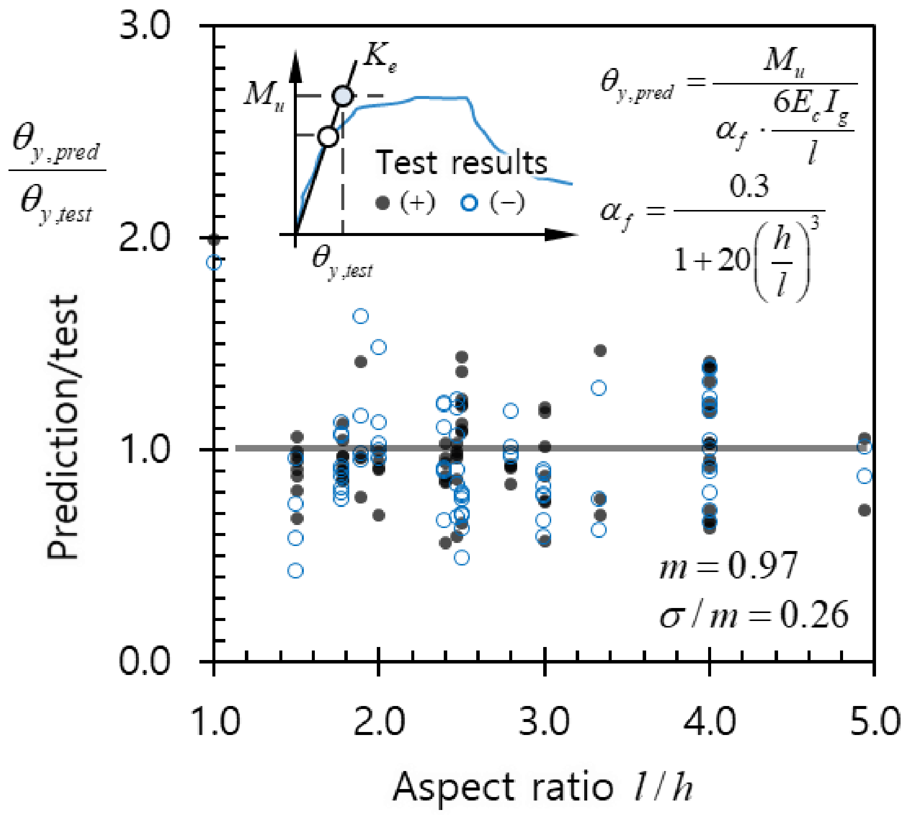


(a) Combined behavior (b) Flexural rigidity only (c) Shear rigidity only

Plastic Hinge Model :
Coupling Beam

- Yield deformation

$$\frac{\Delta_y}{l} \text{ or } \theta_y = \frac{M_n}{\alpha_f K_f} = \frac{M_n}{\dfrac{0.3}{1+20(h/l)^3}\left(\dfrac{6E_c I_g}{l}\right)} = \frac{1+20(h/l)^3}{1.8}\frac{M_n l}{E_c I_g}$$



Plastic Hinge Model :

Coupling Beam

- Plastic deformation (based existing test results)

Design parameter			Modeling parameter			Allowable limit		
Reinforcement detail		Lateral reinforcement ratio	a (rad)	b (rad)	c	Occupancy	Life safety	Collapse prevention
Diagonal	seismic†	NA	$0.030-\theta_y$	$0.050-\theta_y$	0.80	θ_y	a	b
	Non-seismic†	$\rho_t \geq 0.003$	$0.030-\theta_y$	$0.050-\theta_y$	0.80	θ_y	a	b
		$\rho_t < 0.003$	$0.020-\theta_y$	$0.035-\theta_y$	0.50	θ_y	a	b
conventional: $V_y \leq V_n$	seismic†	NA	$0.025-\theta_y$	$0.050-\theta_y$	0.75	θ_y	a	b
	Non-seismic†	$\rho_t \geq 0.003$	$0.020-\theta_y$	$0.035-\theta_y$	0.50	θ_y	a	b
		$\rho_t < 0.003$	$0.015-\theta_y$	$0.035-\theta_y$	0.25	θ_y	a	b
conventional: $V_y > V_n$	All cases	All cases	$0.008-\theta_y$††	$0.014-\theta_y$	0.20	NA	a	b

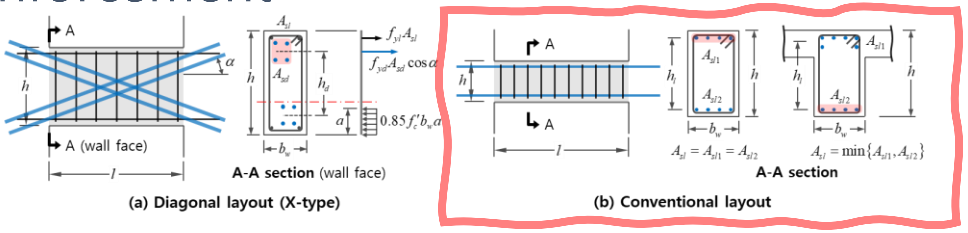
V_y = shear force corresponding to flexural yielding ($= \frac{2M_n}{l}$) M_n = flexural strength of coupling beam l = net beam length

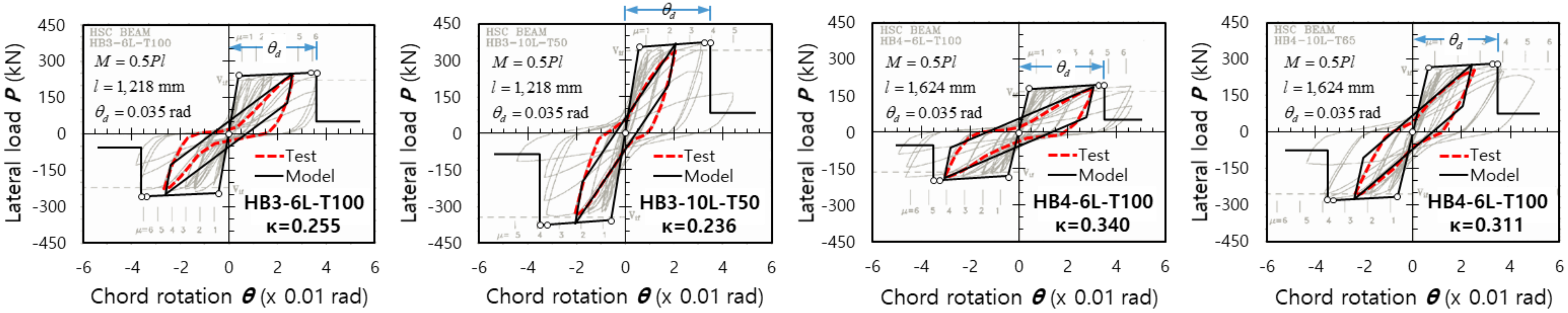
V_n = shear strength of coupling beam ρ_t = lateral reinforcement ratio

Plastic Hinge Model :

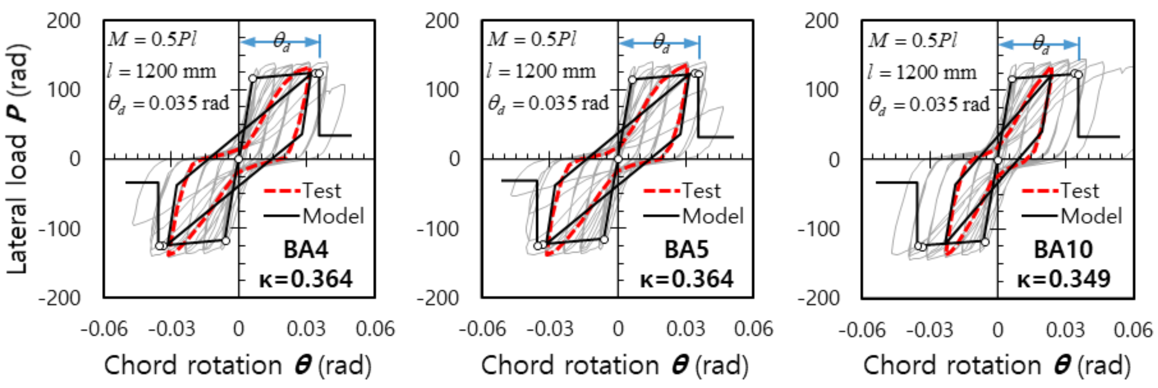
Coupling Beam

- Verification : conventional reinforcement

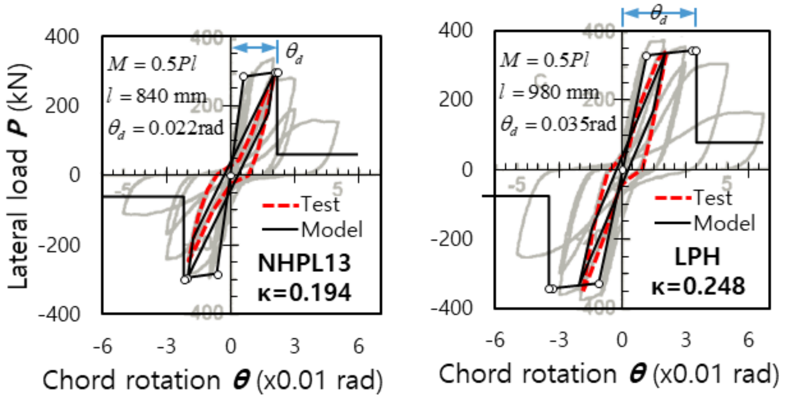




(a) Xiao et al. 1999



(b) Muguruma and Kinugasa 1990

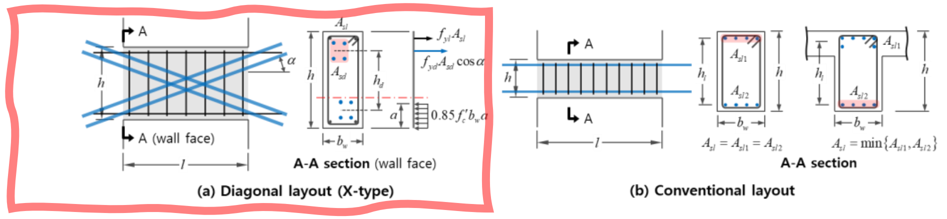


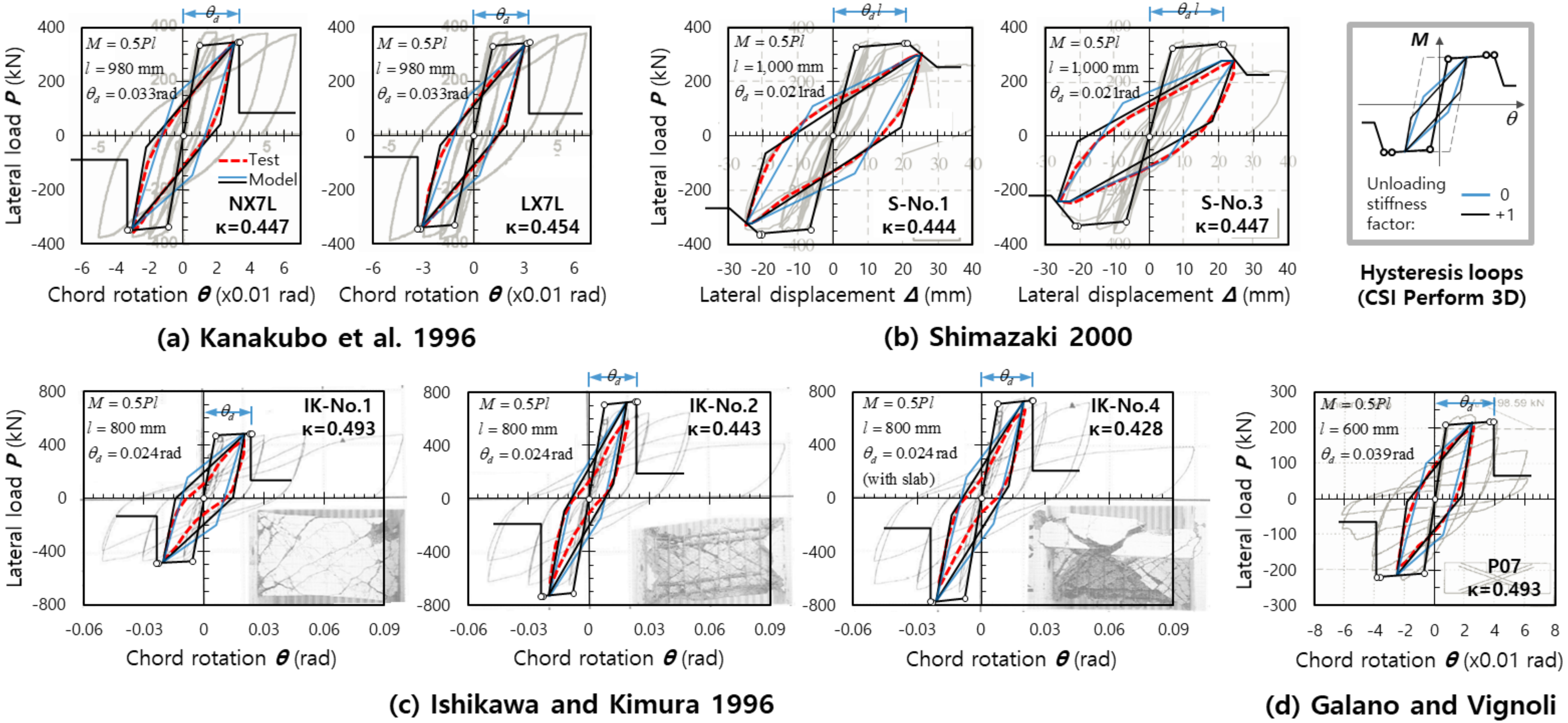
(c) KanaKubo et al. 1996

Plastic Hinge Model :

Coupling Beam

- Verification : diagonal reinforcement

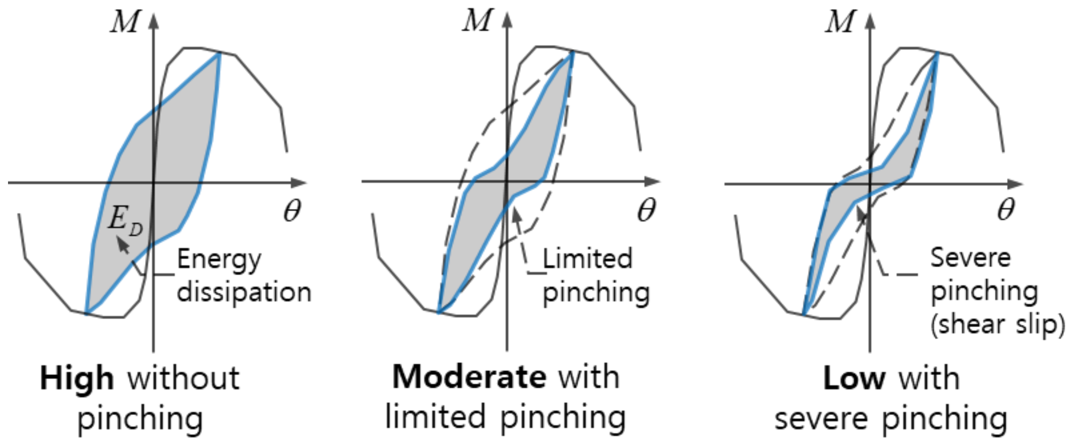




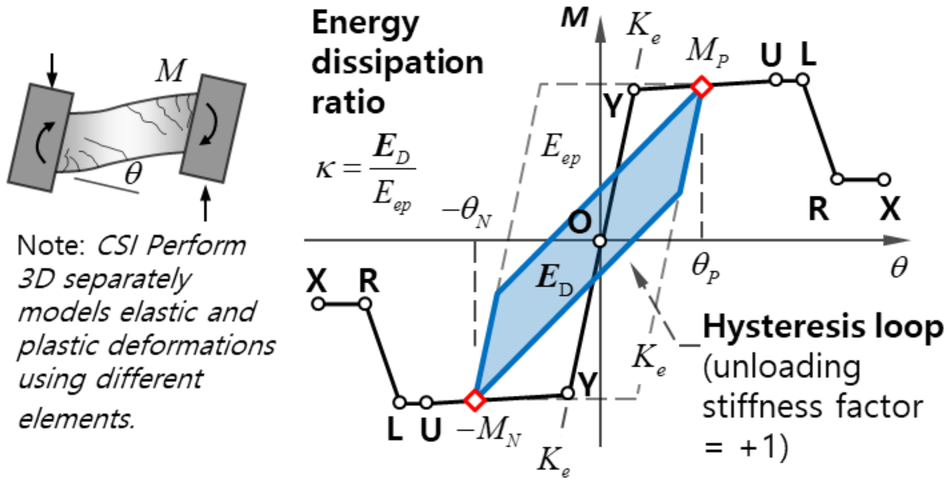
Plastic Hinge Model : Cyclic Curve for Time History Analysis

- Energy dissipation based cyclic model

◆ By controlling energy dissipation (i.e., area of hysteresis loops), the overall shape of cyclic curves and unloading/reloading stiffnesses can be described with reasonable accuracy.



(a) Classifications of hysteretic energy dissipation

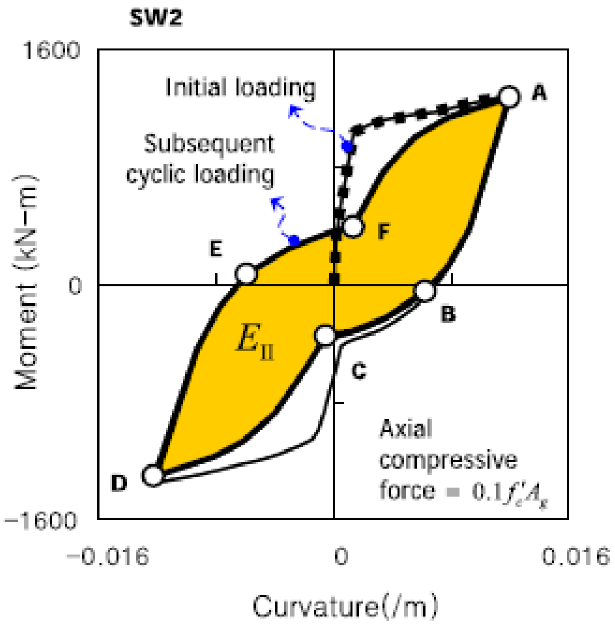


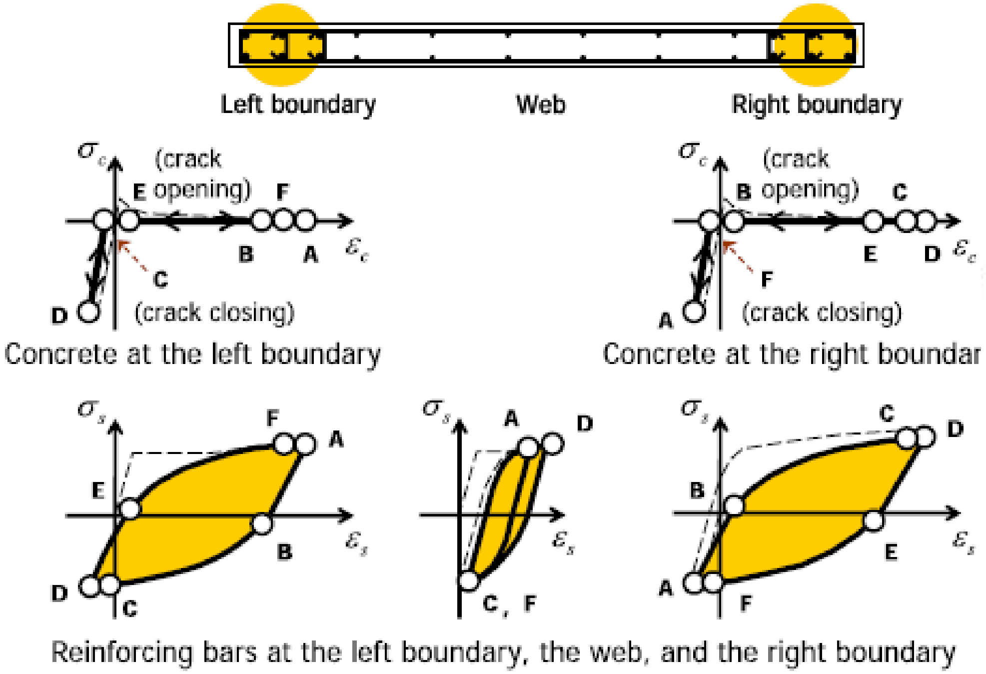
(b) Energy-based model in CSI Perform 3D

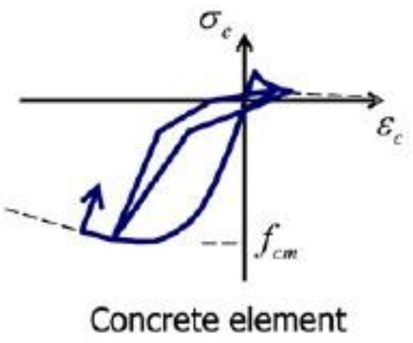
Plastic Hinge Model : Cyclic Curve for Time History Analysis

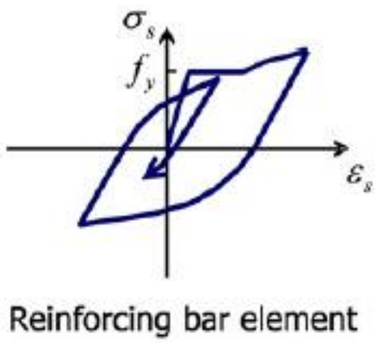
- Energy dissipation mechanism

- Most of energy during cyclic loading is dissipated by steel reinforcement (plastic material) rather than concrete (brittle material)



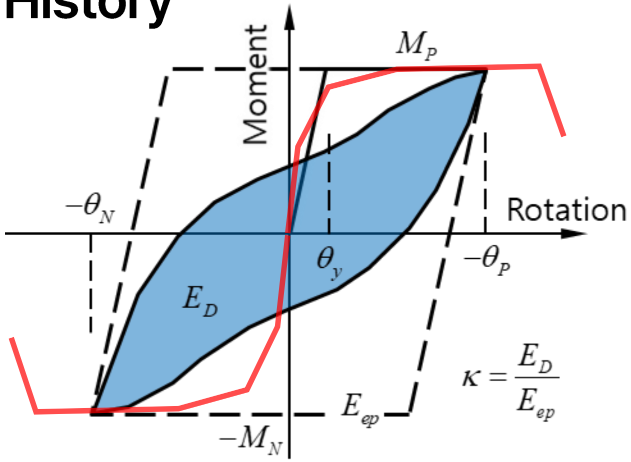






Plastic Hinge Model : Cyclic Curve for Time History Analysis

- Calculation of Energy dissipation density

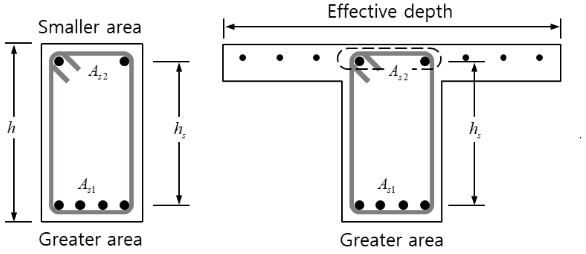


$$E_{ep} = (M_P + M_N)(\theta_P + \theta_N - 2\theta_y)$$

where

$$\begin{aligned}\theta_y &= 1.70\varepsilon_y l_p / h \text{ for beams,}\\ &\quad 2.12\varepsilon_y l_p / h \text{ for rectangular columns,}\\ &\quad 2.35\varepsilon_y l_p / D \text{ for circular columns, and}\\ &\quad 2.00\varepsilon_y l_p / h \text{ for walls}\end{aligned}$$

Priestley (2000). Performance based seismic design. (WCEE)

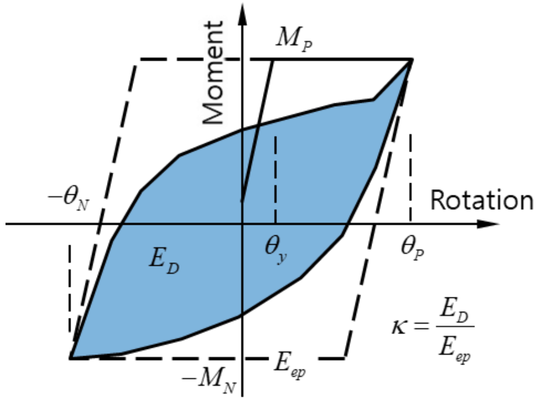


Beam with Rectangular cross-section

$$E_D = 4R_B f_y A_{s2} \frac{h_s}{2}\left(\theta_P + \theta_N - \left[3 + \frac{A_{s2}}{A_{s1}}\right]\varepsilon_y \frac{l_p}{h_s}\right)$$

$$\kappa = \frac{E_D}{E_{ep}} = \frac{4R_B f_y A_{s2} \frac{h_s}{2}\left(\theta_P + \theta_N - \left[3 + \frac{A_{s2}}{A_{s1}}\right]\varepsilon_y \frac{l_p}{h_s}\right)}{(M_P + M_N)\left(\theta_P + \theta_N - 3.4\varepsilon_y \frac{l_p}{h}\right)} \approx \frac{2R_B f_y A_{s2} h_s}{(M_P + M_N)} = \frac{3}{2}\frac{f_y A_{s2} h_s}{(M_P + M_N)}$$

Smaller area A_{s2} is used.



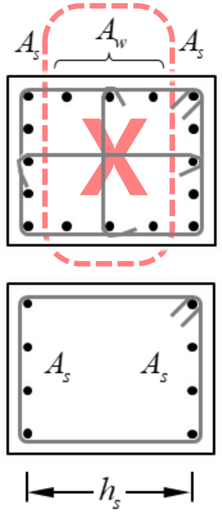
Plastic Hinge Model : Cyclic Curve for Time History Analysis

- Calculation of Energy dissipation density

Column with Rectangular cross-section

$$E_D = 4R_B f_y A_s \frac{h_s}{2}\left(\theta_P + \theta_N - 4\varepsilon_y \frac{l_p}{h_s}\right)$$

$$\kappa = \frac{E_D}{E_{ep}} = \frac{4R_B f_y A_s \frac{h_s}{2}\left(\theta_P + \theta_N - 4\varepsilon_y \frac{l_p}{h_s}\right)}{\left(M_P + M_N\right)\left(\theta_P + \theta_N - 4.24\varepsilon_y \frac{l_p}{h}\right)} \approx \frac{2R_B f_y A_s h_s}{\left(M_P + M_N\right)} = \boxed{\frac{3}{2}\frac{f_y A_s h_s}{\left(M_P + M_N\right)}}$$



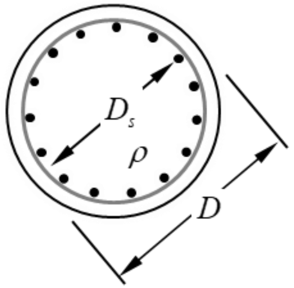
Neglect web reinforcement with small contribution

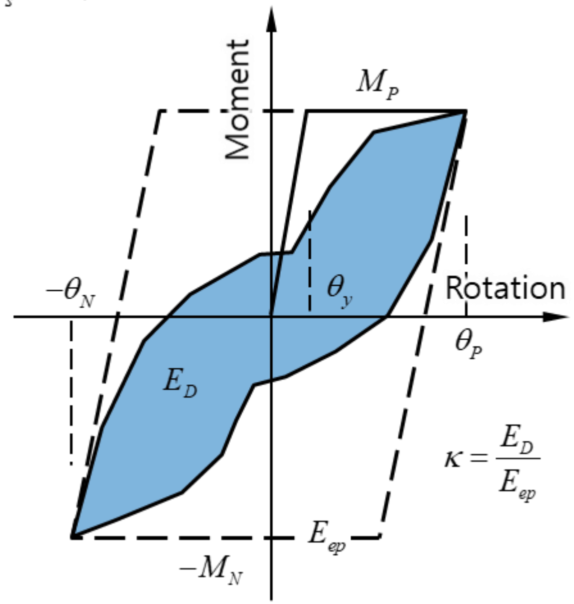
Column with Circular section

$$E_D = 4R_B f_y \frac{\rho\pi D^2}{4}\frac{D_s}{2\pi}\left(\theta_P + \theta_N - 2\pi\varepsilon_y \frac{l_p}{D_s}\right)$$

$$\kappa = \frac{E_D}{E_{ep}} = \frac{4R_B f_y \frac{\rho\pi D^2}{4}\frac{D_s}{2\pi}\left(\theta_P + \theta_N - 2\pi\varepsilon_y \frac{l_p}{D_s}\right)}{\left(M_P + M_N\right)\left(\theta_P + \theta_N - 4.7\varepsilon_y \frac{l_p}{h}\right)}$$

$$\approx \frac{1}{2}\frac{R_B f_y \rho D^2 D_s}{\left(M_P + M_N\right)} = \boxed{\frac{3}{8}\frac{\rho f_y D^2 D_s}{\left(M_P + M_N\right)}}$$





Plastic Hinge Model : Cyclic Curve for Time History Analysis

- Calculation of Energy dissipation density

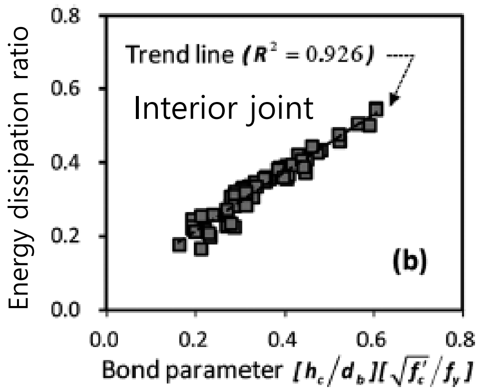
Beam-Column Joint

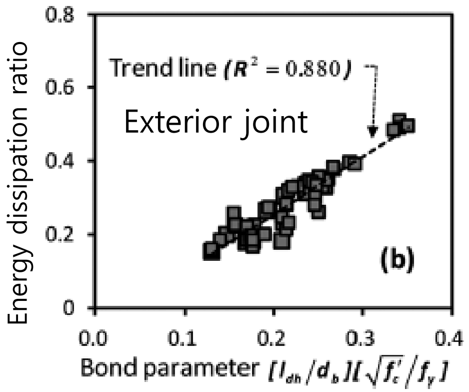
For interior joints,

$$\kappa = 0.8\frac{h_c}{d_b}\frac{\sqrt{f_{ck}}}{f_y} + 0.05 \quad (0.15 \le \kappa \le 0.5)$$

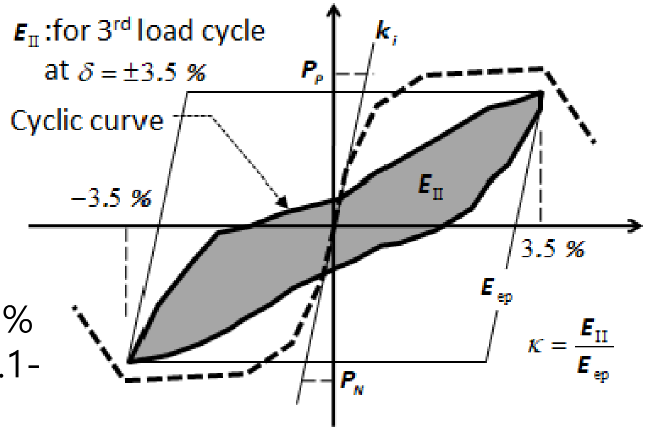
For exterior joints,

$$\kappa = 1.56\frac{l_{dh}}{d_b}\frac{\sqrt{f_{ck}}}{f_y} - 0.06 \quad (0.15 \le \kappa \le 0.5)$$

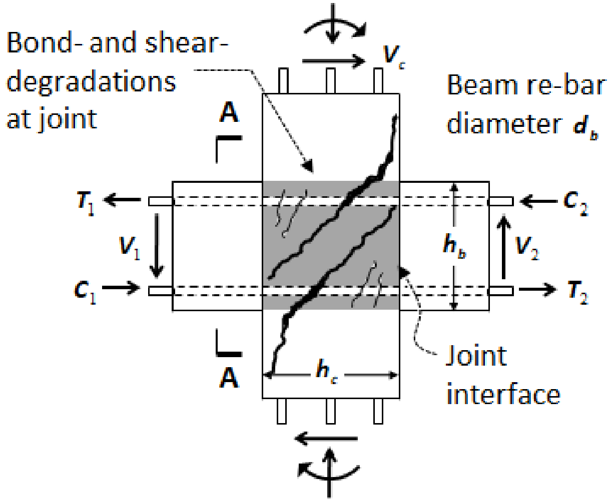




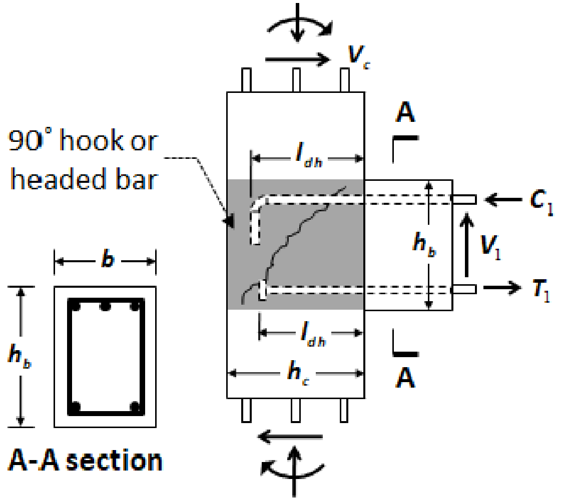
κ : defined at δ = 3.5% according to ACI 374.1-05



(a) Cyclic behavior and energy dissipation of typical beam-column joints



(b) Interior joint

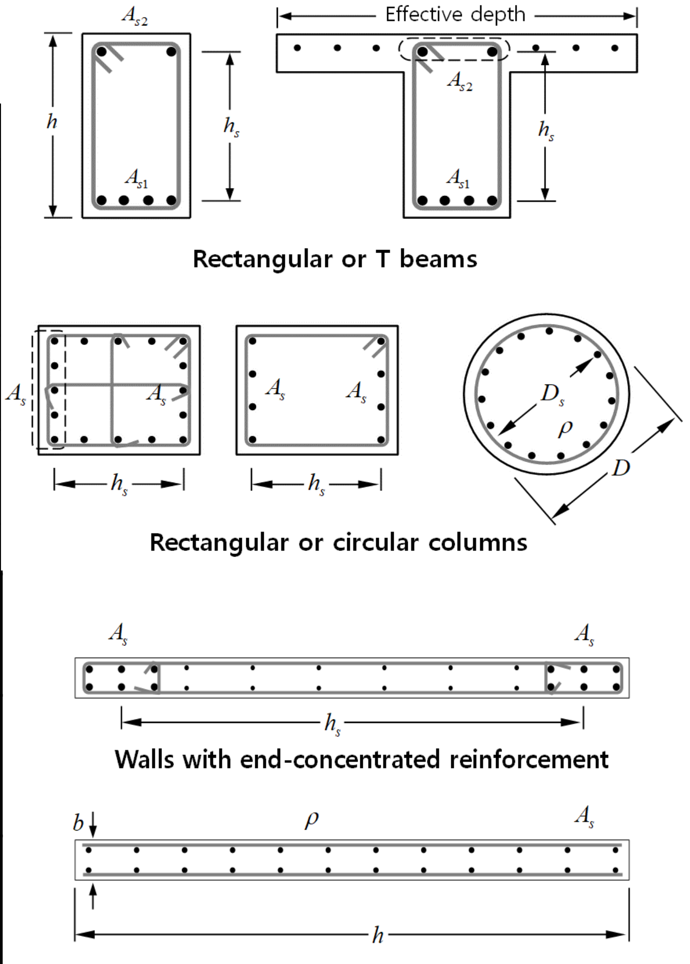


(c) Exterior joint

Plastic Hinge Model : Cyclic Curve for Time History Analysis

- Energy dissipation density

Member type	Reinforcement detail	Energy dissipation density κ
Beam (T-beam)	All reinforcement details	$\frac{3}{2}\frac{f_y A_{s2}\, h_s}{M_P+M_N}\lambda \geq 0.15$ and $\lambda = \frac{l_s}{5h}$ (≤ 1)
Column and wall†	End concentrated reinforcement	$\frac{3}{2}\frac{f_y A_s\, h_s}{M_P+M_N}\lambda \geq 0.15$ and $\lambda = \frac{l_s}{3h}$ (≤ 1)
	Uniformly distributed reinforcement	$\frac{3}{8}\frac{f_y\, \rho\, b\, h^2}{M_P+M_N}\lambda \geq 0.15$ and $\lambda = \frac{l_s}{3h}$ (≤ 1)
	Circular cross section	$\frac{3}{8}\frac{f_y\, \rho\, D_s\, D^2}{M_P+M_N}\lambda \geq 0.15$ and $\lambda = \frac{l_s}{3h}$ (≤ 1)
Coupling beam	Conventional reinforcement	0.15
	X – shaped reinforcement	$3\frac{f_y A_{Ds}}{V_P+V_N}\sin\alpha_D \geq 0.15$
Beam-column joint††	Continuous joint (+ joint)	$0.15 \leq 0.8\frac{h_c}{d_b}\frac{\sqrt{f_{ck}}}{f_y} + 0.05 \leq 0.5$
	discontinuous joint(T joint)	$0.15 \leq 1.56\frac{l_{dh}}{d_b}\frac{\sqrt{f_{ck}}}{f_y} - 0.06 \leq 0.5$



Rectangular or T beams

Rectangular or circular columns

Walls with end-concentrated reinforcement

Walls with uniformly distributed reinforcement

Plastic Hinge Model : Cyclic Curve for Time History Analysis

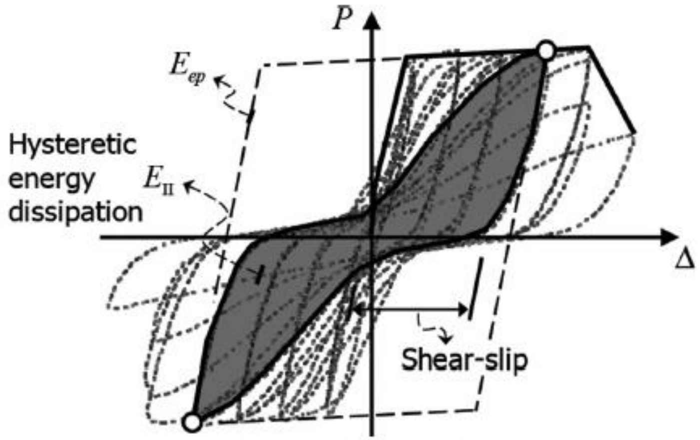
- Energy dissipation density

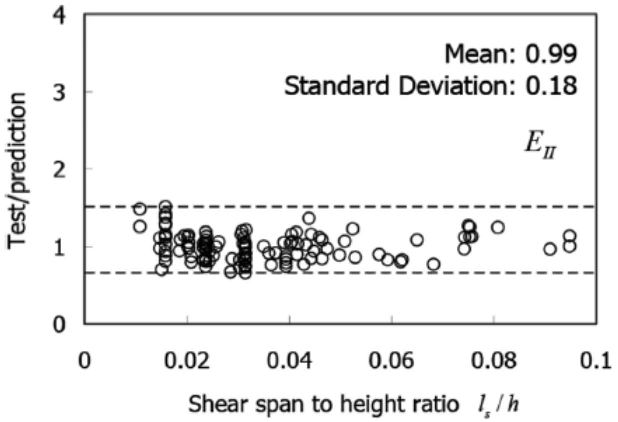
◆ Reduction factor λ for short column (l_s/h <3) and short beam (l_s/h<5)

➢ Pinching due to shear deformation and bond-slip

The energy dissipation of RC members should be evaluated using flexural deformation excluding shear deformation

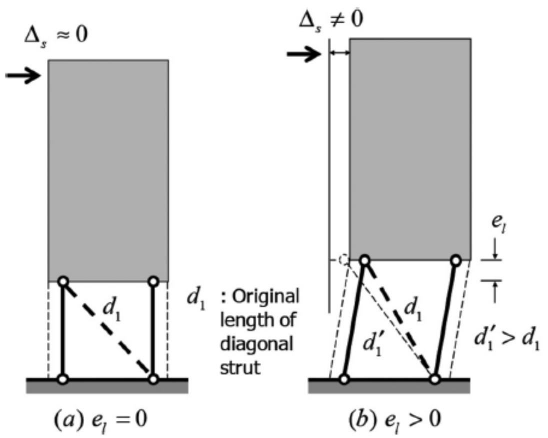
$$E_{II} = 2R_B A_{s2} f_y \left[\left(\phi^+ + \phi^-\right) l_p h_s - \left(3 + \frac{A_{s2}}{A_{s1}}\right) l_p \varepsilon_y \right] = 2R_B A_{s2} f_y \left[\left(\theta^+ + \theta^-\right) h_s - \left(3 + \frac{A_{s2}}{A_{s1}}\right) l_p \varepsilon_y \right]$$





Pinching due to shear deformation
➔ Reduction in hysteretic energy dissipation

Longitudinal elongation



Flexural (rotation) $\theta = \phi l_p = \dfrac{\Delta_t - \Delta_{ef} - \Delta_s}{(l_s - 0.5 l_p)}$

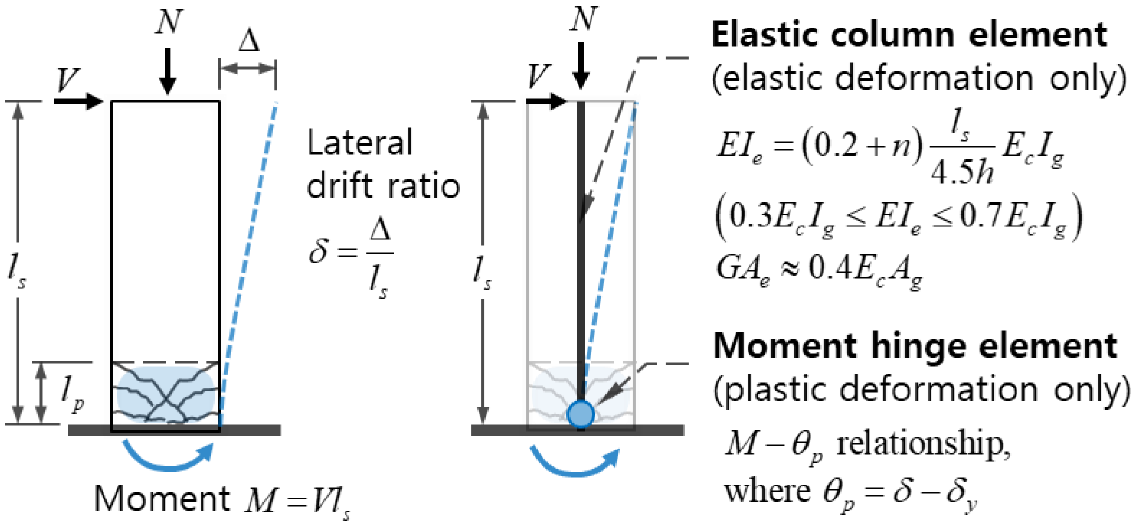
Shear $\Delta_s = \gamma l_p = e_l \dfrac{l_p}{h_s} + \left[2\varepsilon_d + \dfrac{\varepsilon_v}{2}\right] l_p$

Longitudinal elongation can cause shear deformation. As the shear span becomes longer, the contribution of the shear deformation to the total deformation decreases.

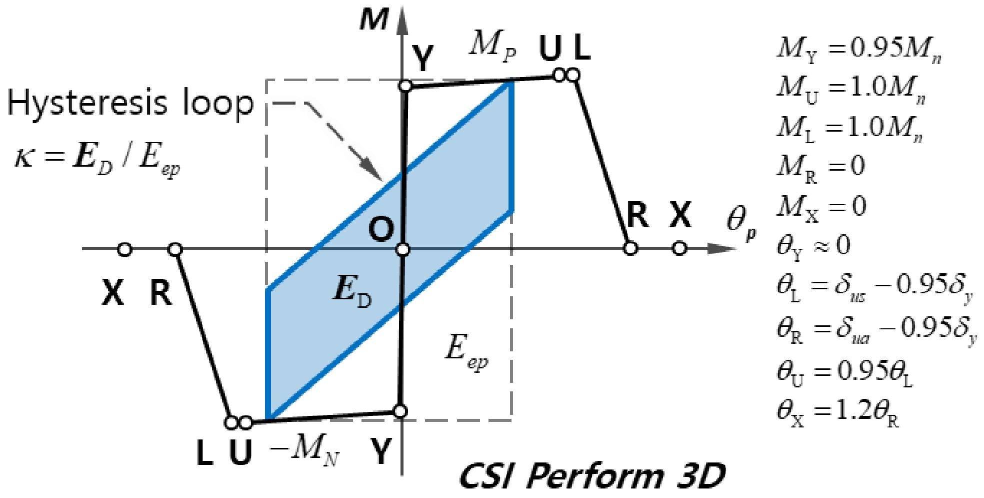
Plastic Hinge Model : Cyclic Curve for Time History Analysis

- Modeling of cyclic curve

- In the case of column



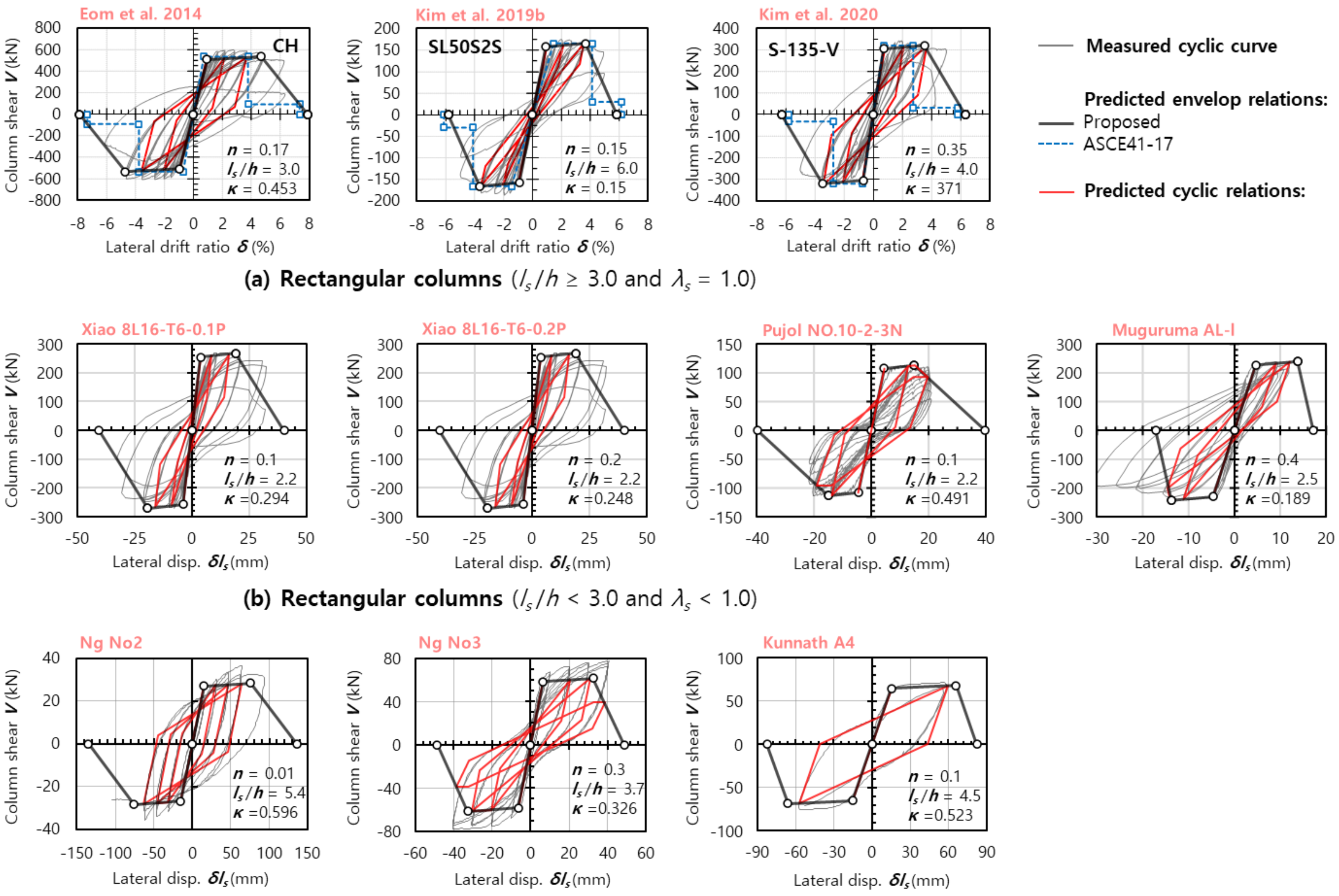
(a) Modeling components



(b) Hysteresis model for moment hinge element

Plastic Hinge Model : Cyclic Curve for Time History Analysis

- Verification
- For columns



(a) Rectangular columns ($l_s/h \geq 3.0$ and $\lambda_s = 1.0$)

(b) Rectangular columns ($l_s/h < 3.0$ and $\lambda_s < 1.0$)

(c) Circular columns ($l_s/h \geq 3.0$ and $\lambda_s = 1.0$)

Verification of Structural Performance

D_u = member plastic rotation
D_{ne} = allowable plastic rotation

F_{ns} = gravity load effect
F_s = seismic load effect
ϕF_n = design strength

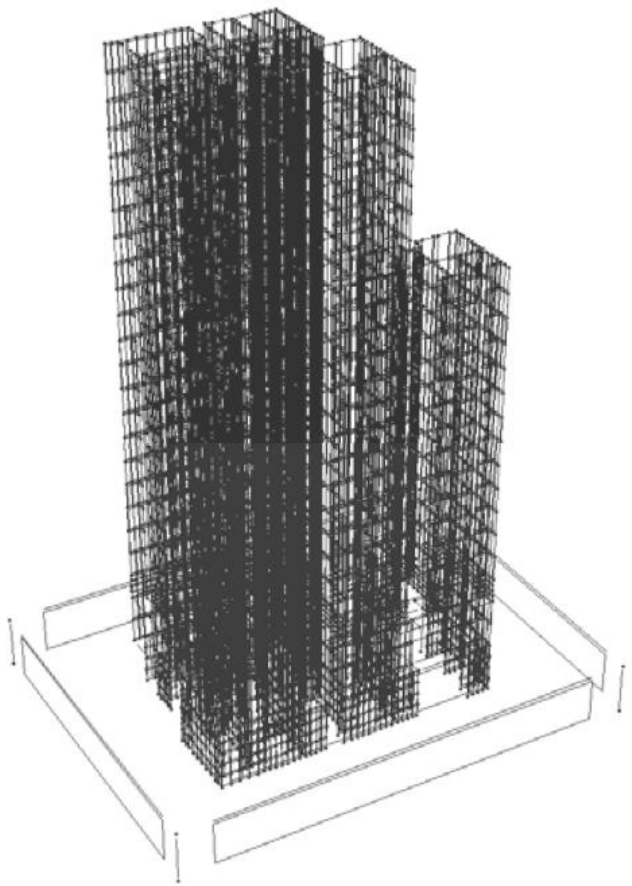
- Evaluation criteria

criteria		Life safety	Collapse prevention
Overall structure	Story drift ratio	1.5%	2.0%
Fiber model	Compression strain of concrete	-	0.2%
	Tensile strain of rebars	-	4.0%
	Shear (Force-controlled)	-	$F_{ns} + 1.2\ (F_s - F_{ns}) \leq \phi F_n$
Plastic hinge model	Plastic rotation (Deformation-controlled)	$D_u \leq D_{ne}$	$D_u \leq D_{ne}$
	Shear (Force-controlled)	-	$F_{ns} + 1.2\ (F_s - F_{ns}) \leq \phi F_n$

Application Example

- Summary of structure

- 24 story above ground, 1 story in basement
- Apartment building
- Seismic importance class 1 (KDS 41)
- Bearing wall structure (wall –slab structure)
- Transfer slab in 3rd story (moment frame below and bearing wall system above)
- Pile foundation
- Software for basic elastic design : Midas
- Software for nonlinear analysis : Perform 3D



Application Example

• Procedure

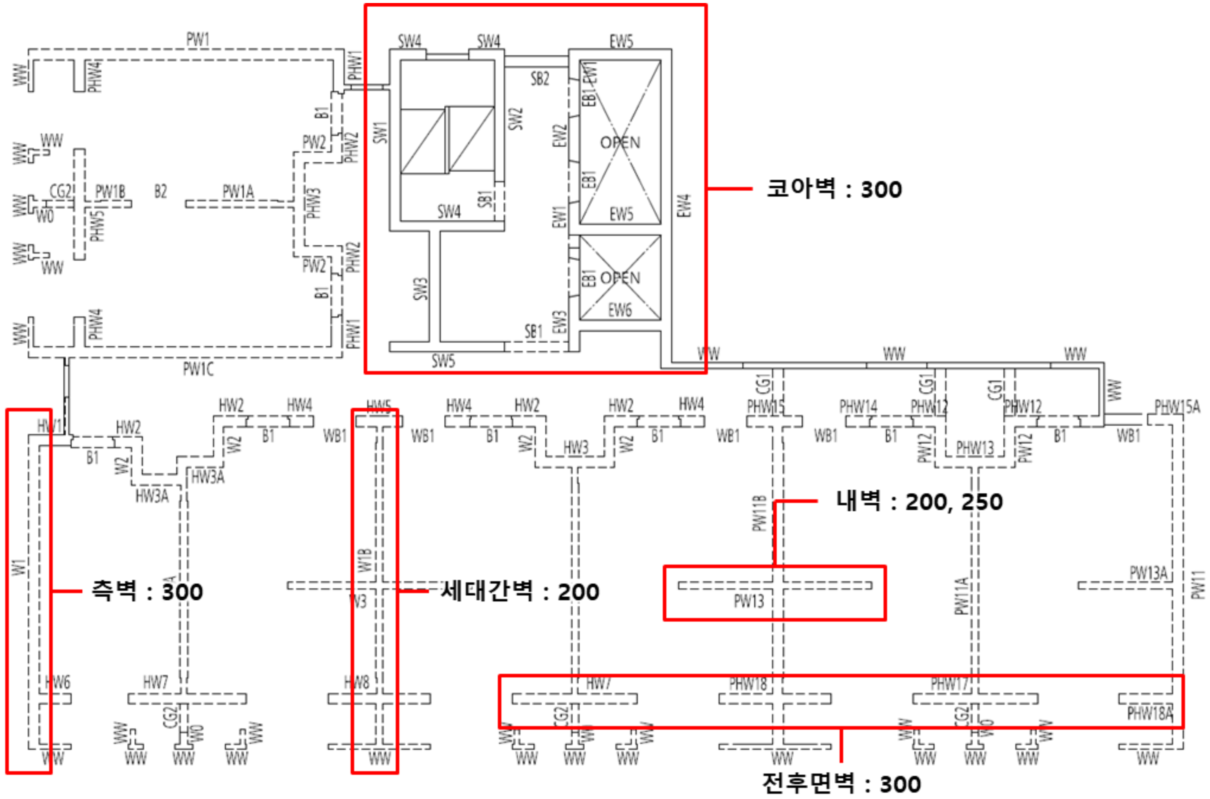
- Basic design : KDS 41 (building structural design code)
- Nonlinear analysis modeling : Guidelines for PBSD
- Selection of ground motions
- Nonlinear time history analysis
- Performance evaluation / interim report
- Peer review
- Final report

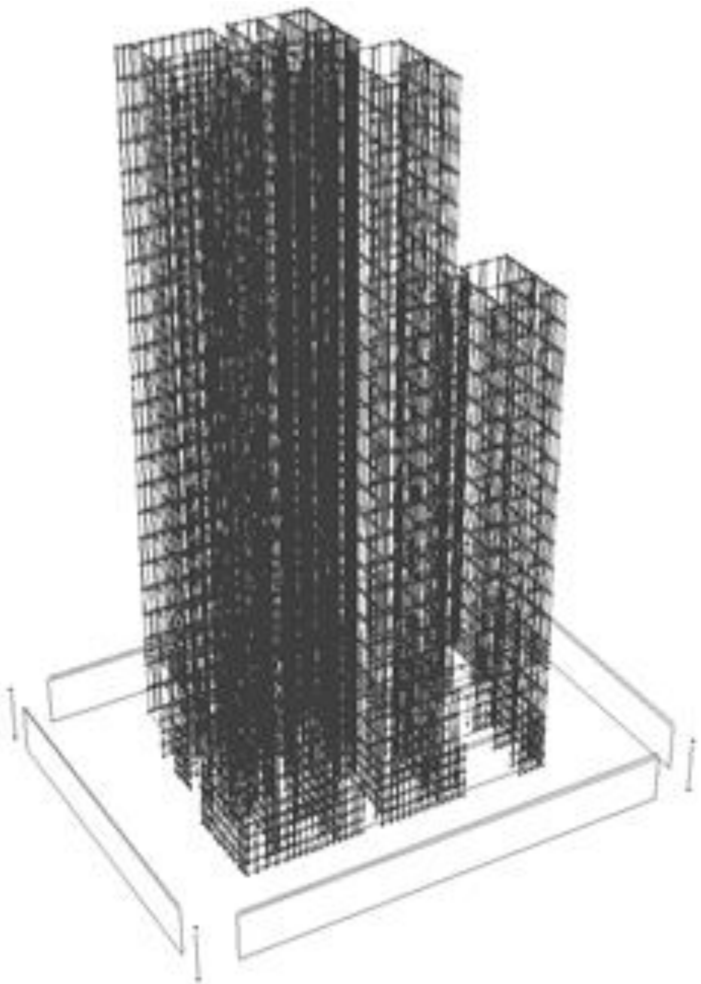
• Target performance

» Collapse prevention limit state : 2400 year return period EQ (0.22 g) - mandatory
» Life safety : 1400 year return period EQ (0.17g)

Application Example

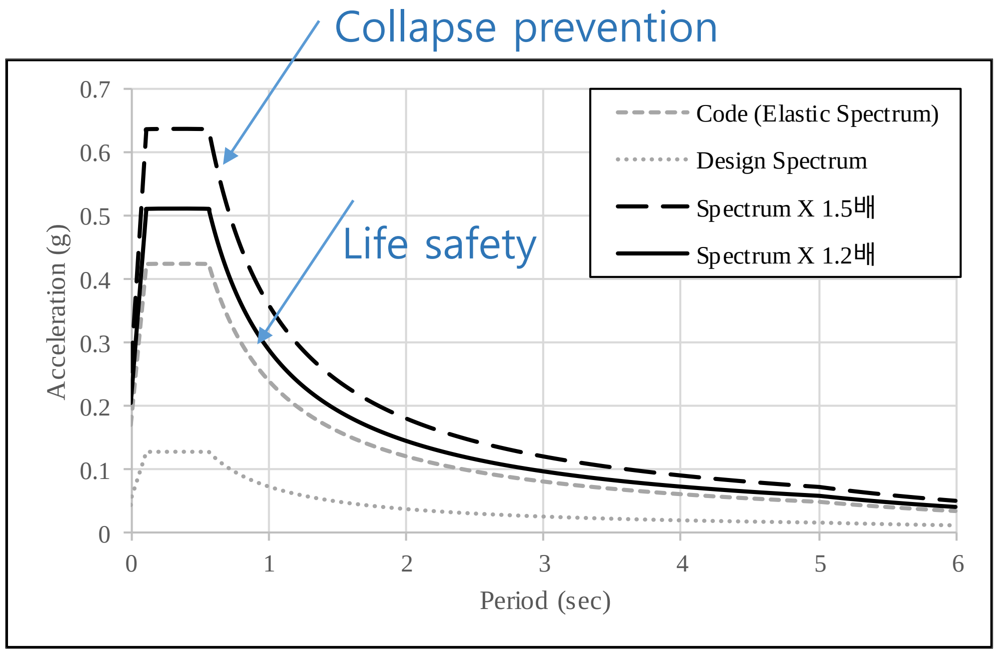
- Structural plan





Application Example

- Materials
  - Concrete strength : 24 – 27 MPa
  - Steel reinforcement : 500 Mpa D10 and D13
    600 Mpa D16 and greater



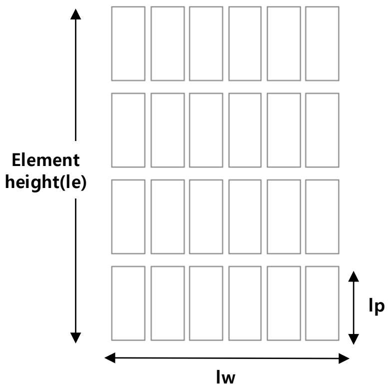
- Design information
  - » Regional factor = 0.176 g
  - » Importance factor = 1.2
  - » Site class = S4
  - » Response modification factor = 4 (bearing wall structure)
  - » Dynamic period = 2.0 sec (X direction), 1.64 sec (Y direction)

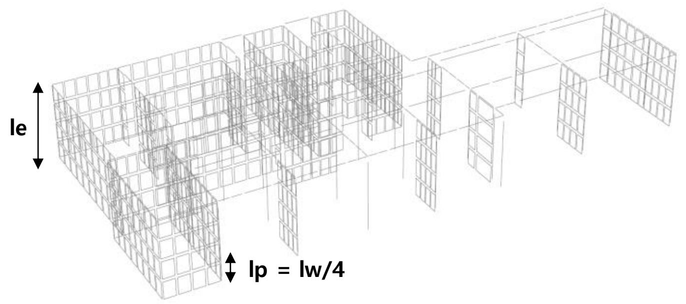
Application Example

- Fiber models for walls

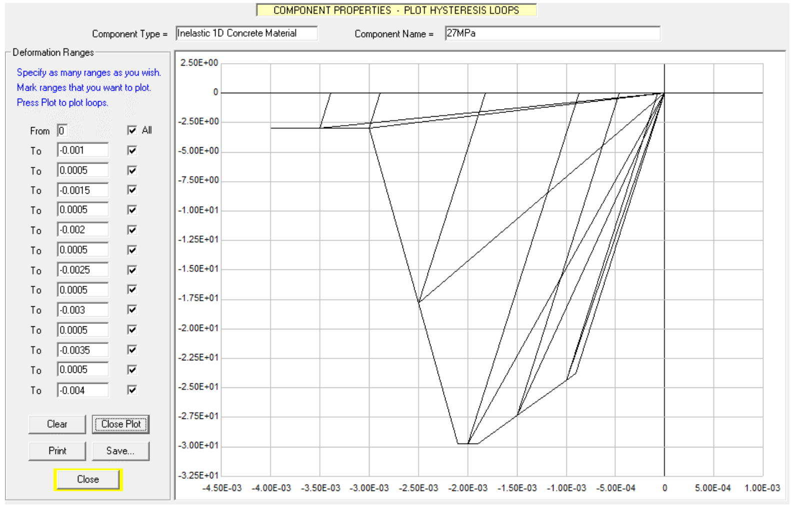
» Multiple fiber model layers for plastic hinge zone(1st story)

» Single fiber model layer for other stories

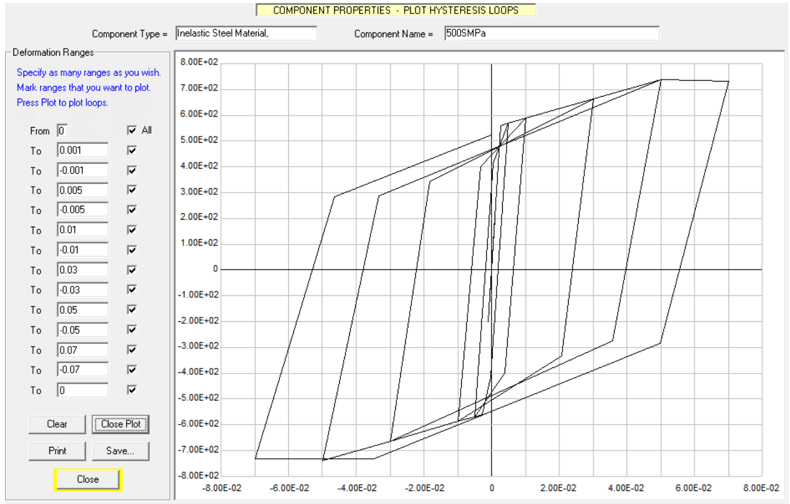




» Concrete strength : 27 MPa



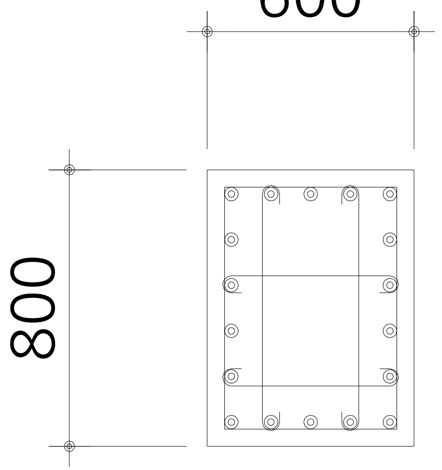
» Steel reinforcement : 500 Mpa



Application Example

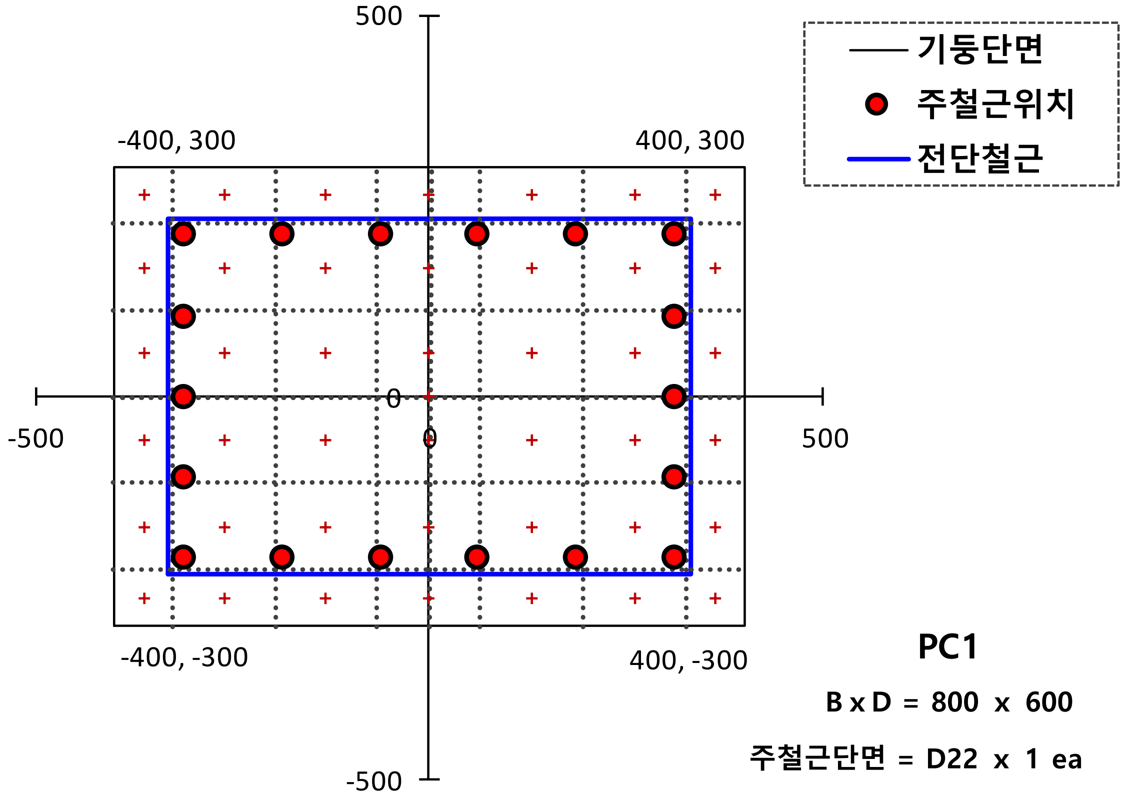
- Fiber model for columns

- Fiber model



18-D22

D13@150



PC1

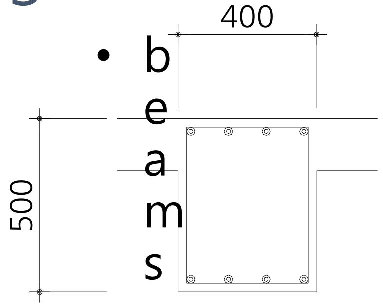
B x D = 800 x 600

주철근단면 = D22 x 1 ea

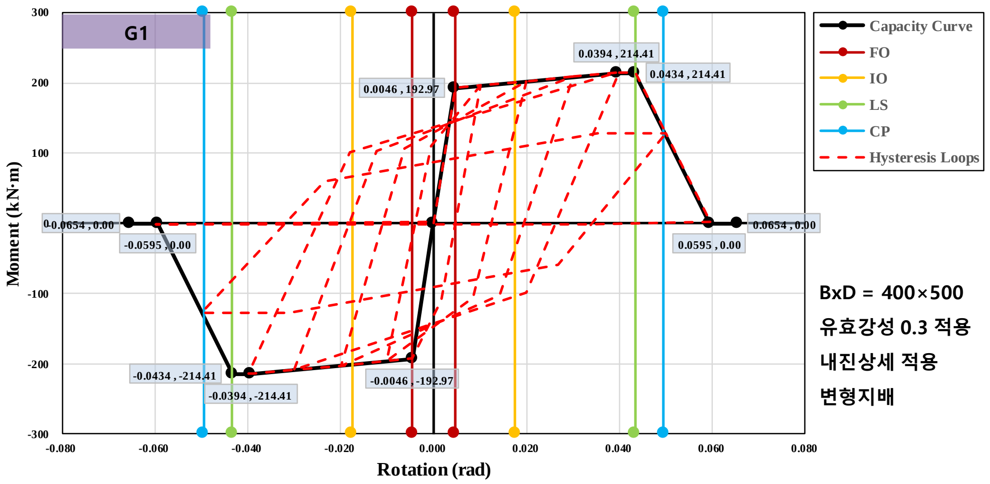
Application Example

- Plastic hinge models
  - beams

symmetric section

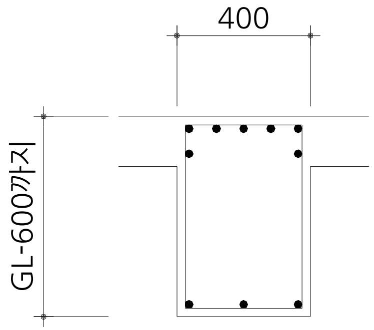


4-D16
4-D16
2-D10@100

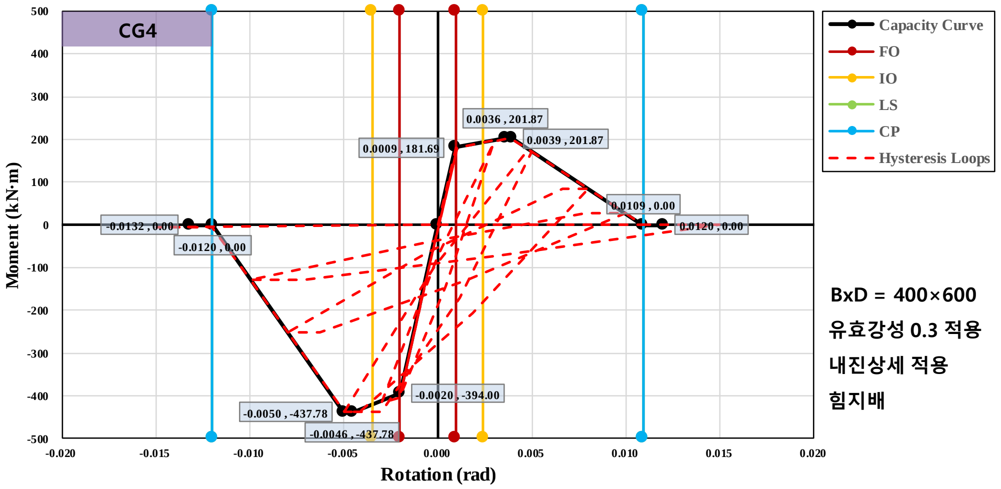


BxD = 400×500
유효강성 0.3 적용
내진상세 적용
변형지배

asymmetric section



7-D16
3-D16
2-D10@100

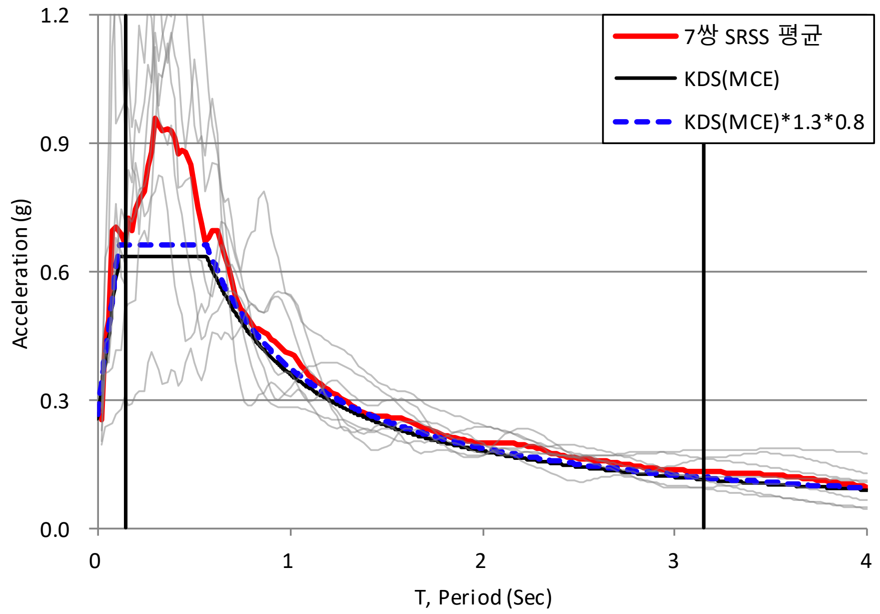


BxD = 400×600
유효강성 0.3 적용
내진상세 적용
힘지배

Application Example

- Ground motions

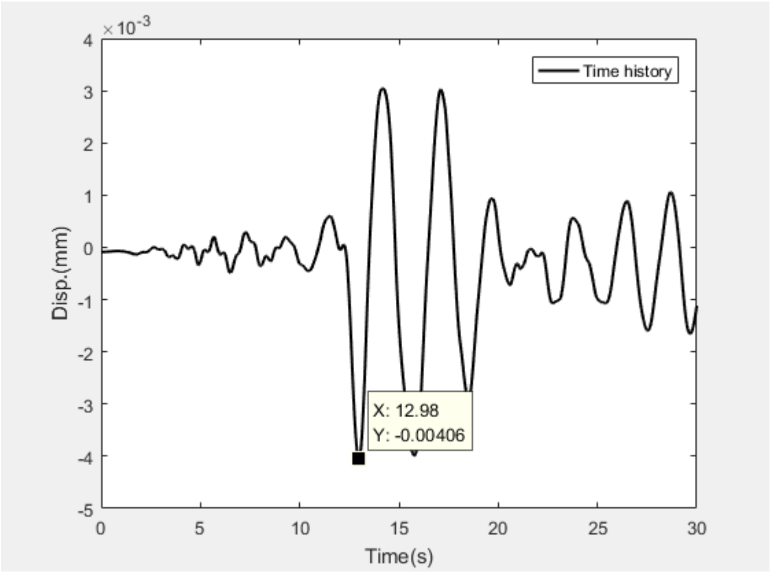
Number	Name	Year	Station	Peak Acc.		Total Time/ Interval
				X-Dir.	Y-Dir.	
1	Chi-Chi-06	1999	HWA003	0.189 g	-0.136 g	30 sec/ 0.02 sec
2	Tottori	2000	OKYH07	-0.295 g	-0.393 g	40 sec/ 0.02 sec
3	Campano Lucano	1980	Auletta	0.299 g	0.284 g	60 sec/ 0.02 sec
4	IWATE	2008	IWT010	0.215 g	-0.223 g	30 sec/ 0.02 sec
5	Landers	1992	Lucerne	-0.338 g	0.367 g	30 sec/ 0.02 sec
6	Chi-Chi	1999	TAP046	-0.308 g	0.184 g	50 sec/ 0.02 sec
7	Chi-Chi	1999	CHY102	-0.227 g	0.247 g	70 sec/ 0.02 sec



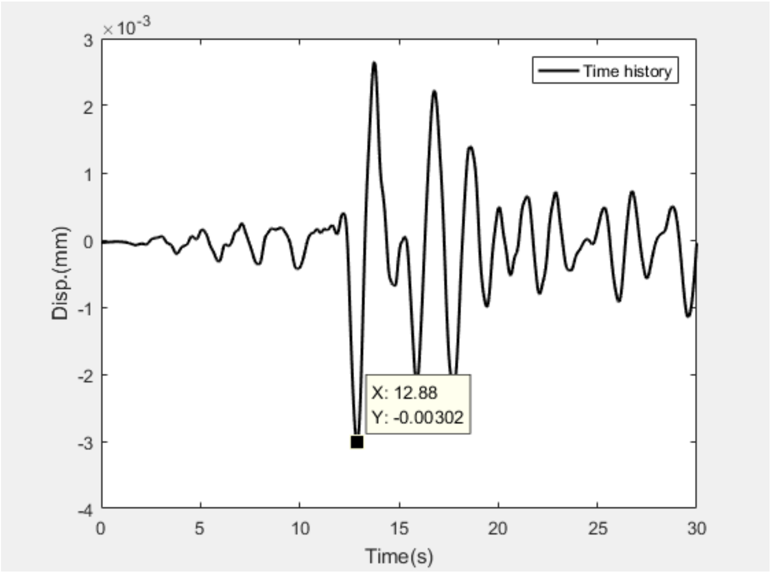
Application Example

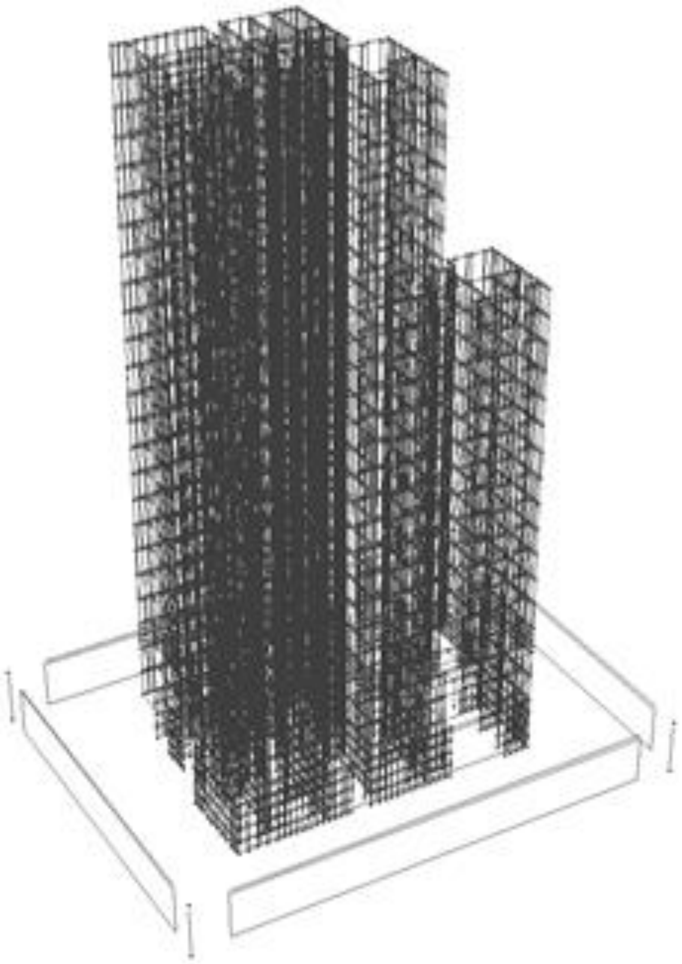
- Time history displacements at top floor mass center

» EQ 1 – x direction



» EQ 1 – y direction





Application Example

D_u = member plastic rotation
D_{ne} = allowable plastic rotation

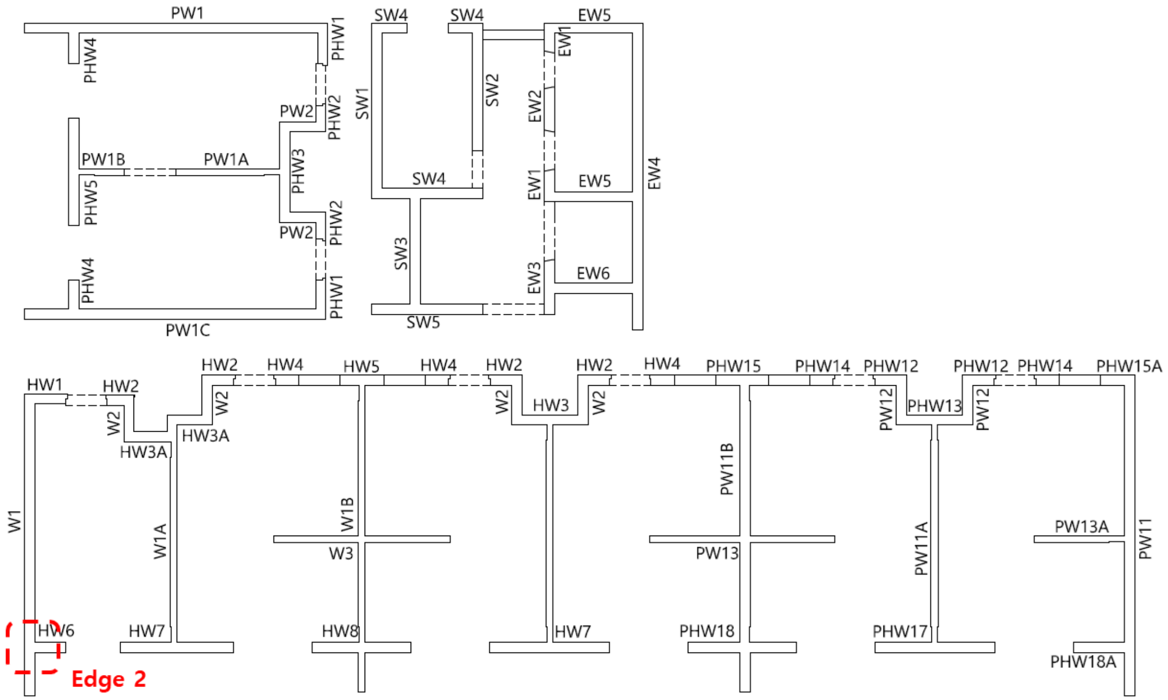
F_{ns} = gravity load effect
F_s = seismic load effect
ϕF_n = design strength

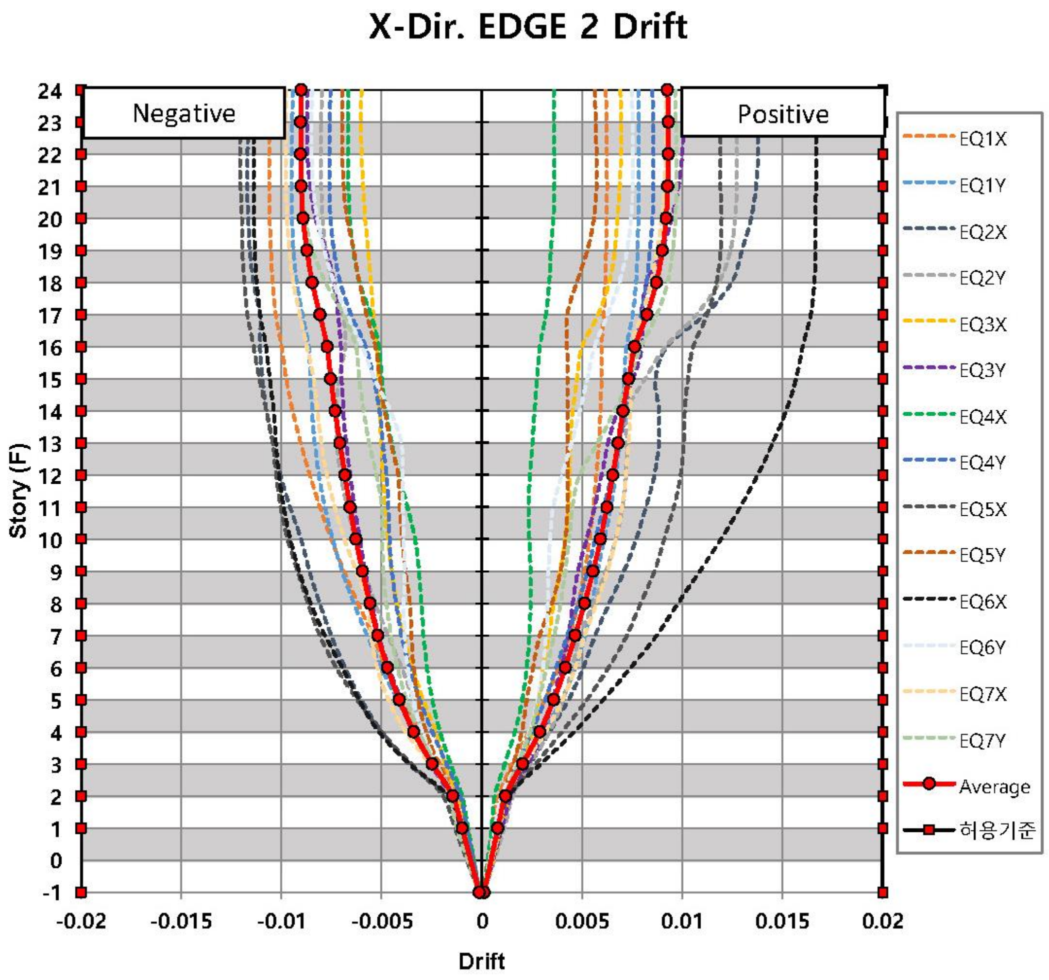
- Evaluation criteria

criteria		Life safety	Collapse prevention
Overall structure	Story drift ratio	1.5%	2.0%
Fiber model	Compression strain of concrete	-	0.2%
	Tensile strain of rebars	-	4.0%
	Shear (Force-controlled)	-	$F_{ns} + 1.2\,(F_s - F_{ns}) \le \phi F_n$
Plastic hinge model	Plastic rotation (Deformation-controlled)	$D_u \le D_{ne}$	$D_u \le D_{ne}$
	Shear (Force-controlled)	-	$F_{ns} + 1.2\,(F_s - F_{ns}) \le \phi F_n$

Application Example

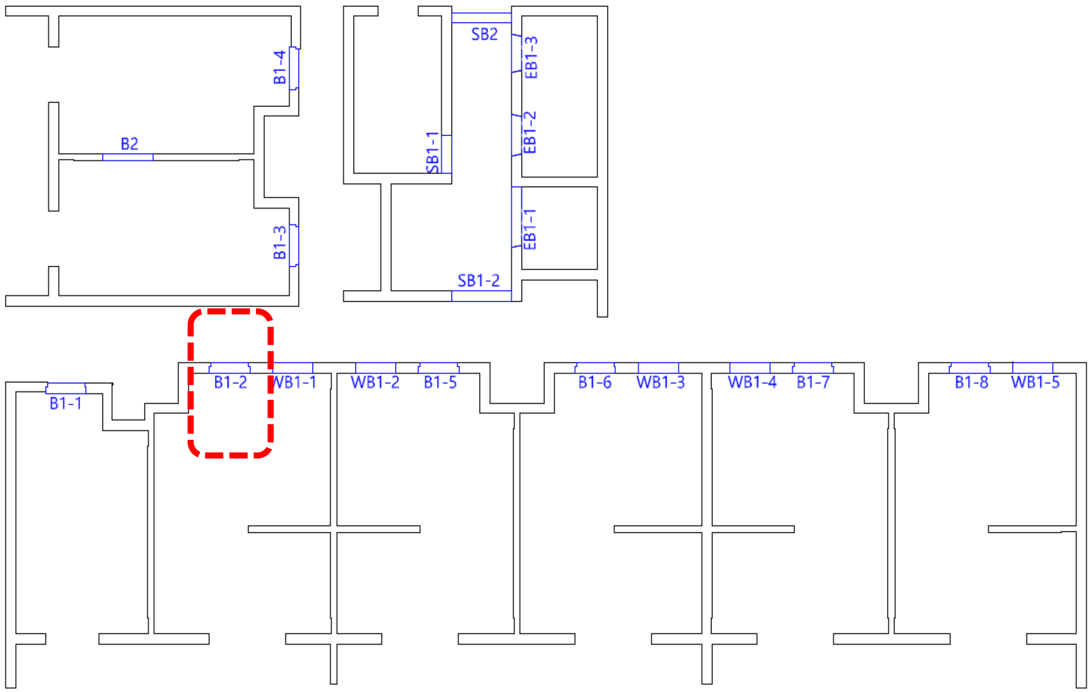
- Story drift ratio

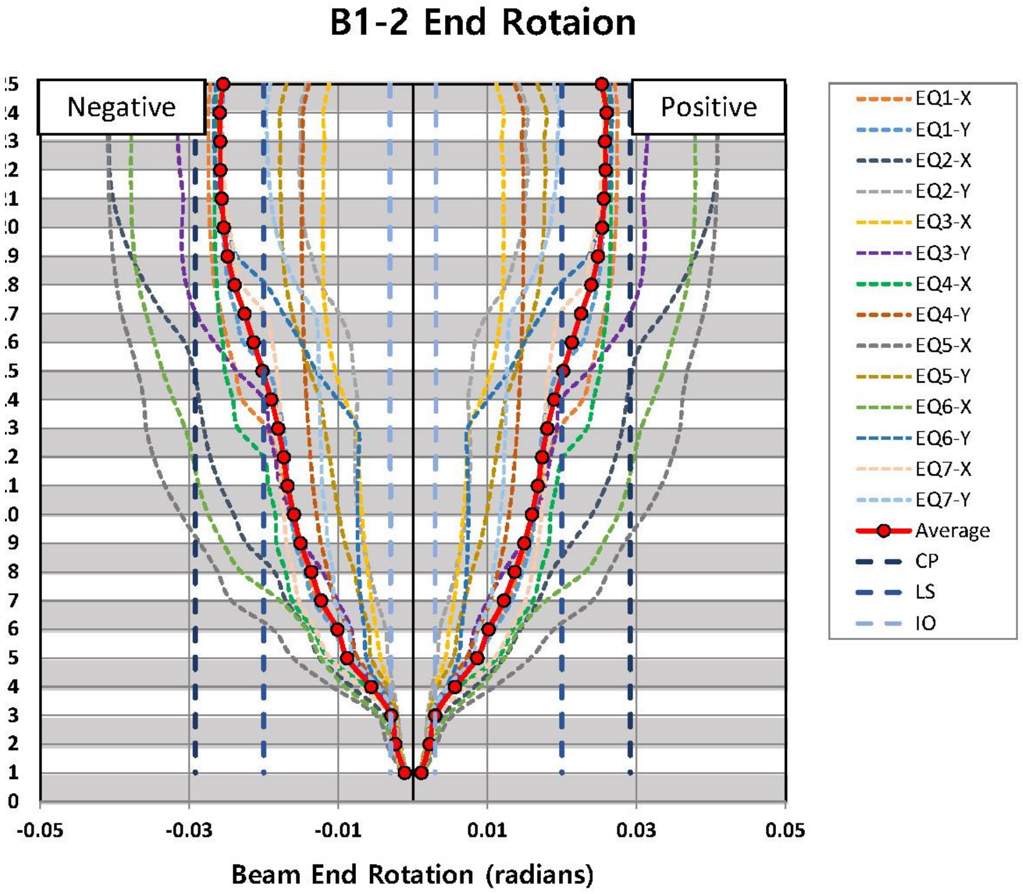




Application Example

- Plastic rotation of beam (Deformation-controlled behavior)





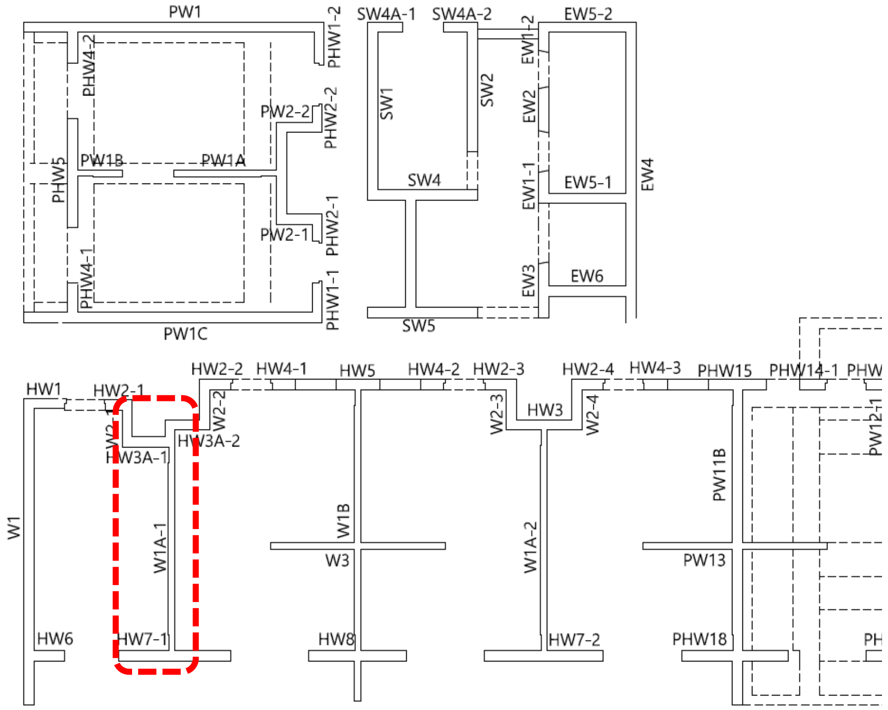
Application Example

- Shear of Walls (Force-controlled behavior)

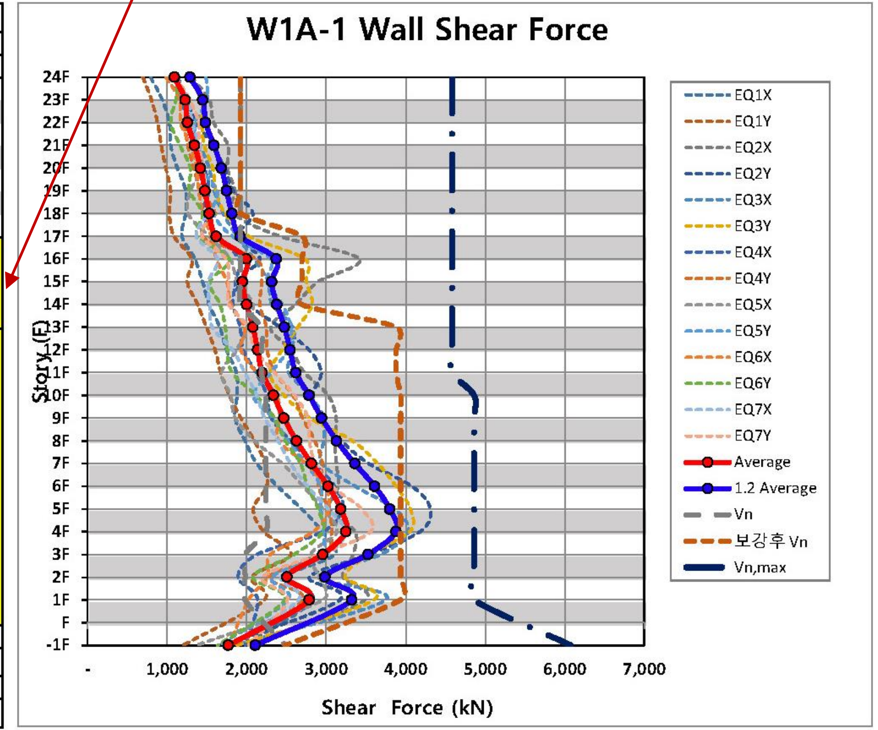
- Walls are vulnerable to shear due to dynamic amplification after flexural yielding

$$F_{ns} + 1.2\,(F_s - F_{ns}) \le \phi F_n$$

Increase of shear reinforcement



Story	Height	W1A-1								
		Elastic Design Result					After reinforcing			
		Length	fck	thk.	fy	Hor.	thk.	fck	fy	Hor.
24F	64.4	6690	24	200	500	D10@440	200	24	500	D10@440
23F	61.6									
22F	58.8									
21F	56									
20F	53.2									
19F	50.4									
18F	47.6									
17F	44.8								500	D10@250
16F	42									
15F	39.2									
14F	36.4									
13F	33.6			200	500	D10@350	200		500	D10@150
12F	30.8									
11F	28									
10F	25.2		27					27		
9F	22.4									
8F	19.6									
7F	16.8									
6F	14									
5F	11.2									
4F	8.4									
3F	5.6				500	D10@440				
2F	2.8									
1F	0									
-1F	-5.8			250	500	D10@350	250		500	D10@350
Story	Height	Length	fck	thk.	fy	Hor.	thk.	fck	fy	Hor.
		Elastic Design Result					After reinforcing			
		W1A-1								



Application Example

- Summary of Evaluation

- Most of the basic design is OK except for walls.
- Strengthening is required for shear of walls
- Boundary confinement is r equired for B/E of walls

Evaluation item			criteria	results
Story drift ratio	LS		Less than 1.5%	OK
	CP		Less than 2.0%	OK
beam	shear		F-controlled	OK
	Plastic rotation	LS	Allowable limit	OK
		CP	Allowable limit	OK
columns	shear		F-controlled	OK
	Plastic rotation		Allowable limit	OK
Transfer story members	beams	shear	F-controlled	OK
		flexural strength	F-controlled	OK
	columns	shear	F-controlled	OK
		P-M strength	F-controlled	OK
B-C joints	Inelastic rotation	LS	Allowable limit	OK
		CP	Allowable limit	OK
walls	shear		F-controlled	Shear-strengthening
	Vertical strain	LS	-	-
		CP	compression 0.2%, tension 4.0%	B/E- strengthening
	Inelastic rotation	LS	Allowable limit	OK
		CP	Allowable limit	OK

Conclusion

- RC members fail due to concrete (not due to steel)
- Shear strength of concrete is not constant but a function of deformation
- Deformation-controlled shear strength model

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