Backgrounds and Elements of Performance Based Seismic Design

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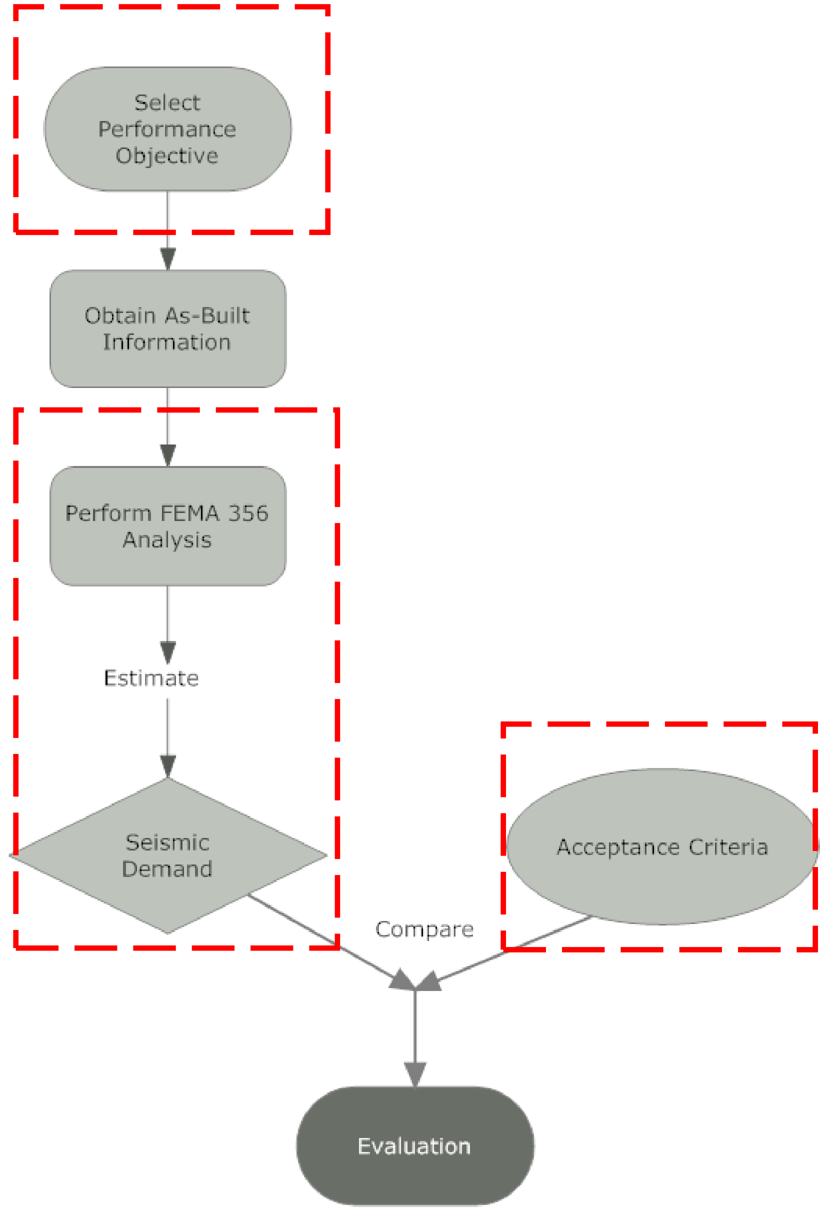
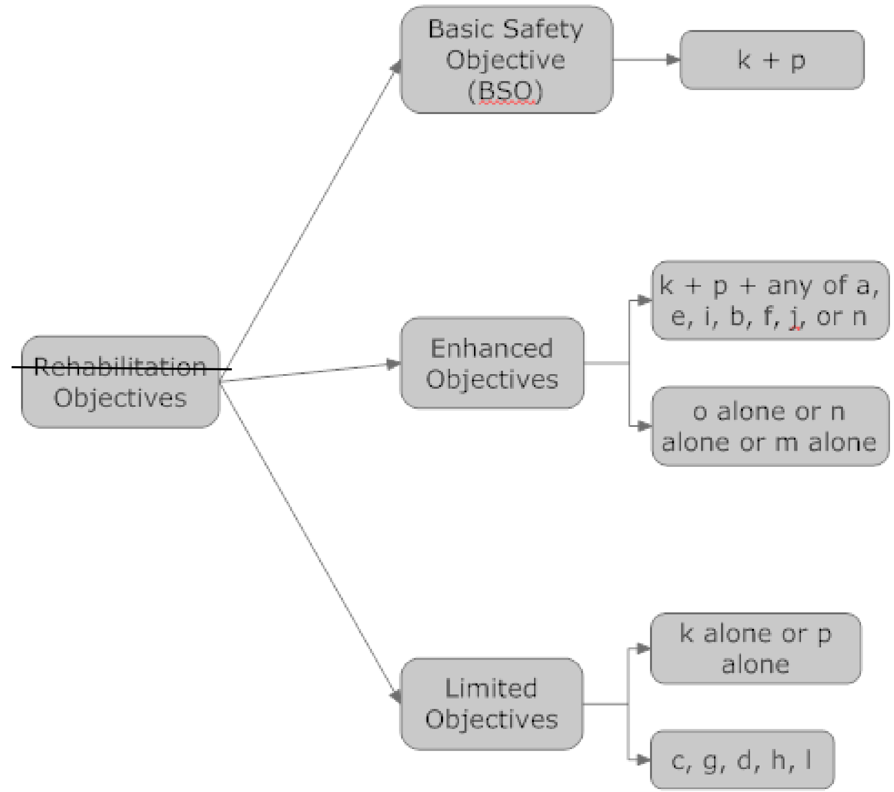
PROCEDURE OF EVALUATION OF PERFORMANCE



I. Selection of Performance Objective

Table C1-1 ~~Rehabilitation~~ Objectives

		Target Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~10%/50 year)	i	j	k	l
	BSE-2 (~2%/50 year)	m	n	o	p



Basic Safey Objective (BSO)

		Target Building Performance Levels*			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level (ground motions having a specified probability of being exceeded in a 50-year period)	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (10%/50 year)	i	j	k	l
	BSE-2 (2%/50 year)	m	n	o	p

*Alpha-numeric identifiers in parentheses defined in Table 4-2

Notes:

1. Each cell in the above matrix represents a discrete Rehabilitation Objective
2. Three specific Rehabilitation Objectives are defined in FEMA 356:

Basic Safety Objective	= cells k + p
Enhanced Objectives	= cells k + p + any of a, e, i, b, f, j, or n
Limited Objectives	= cell k alone, or cell p alone
Limited Objectives	= cells c, g, d, h, l

Enhanced Objectives

		Target Building Performance Levels*			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level (ground motions having a specified probability of being exceeded in a 50-year period)	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
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Limited Objectives

		Target Building Performance Levels*			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level (ground motions having a specified probability of being exceeded in a 50-year period)	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
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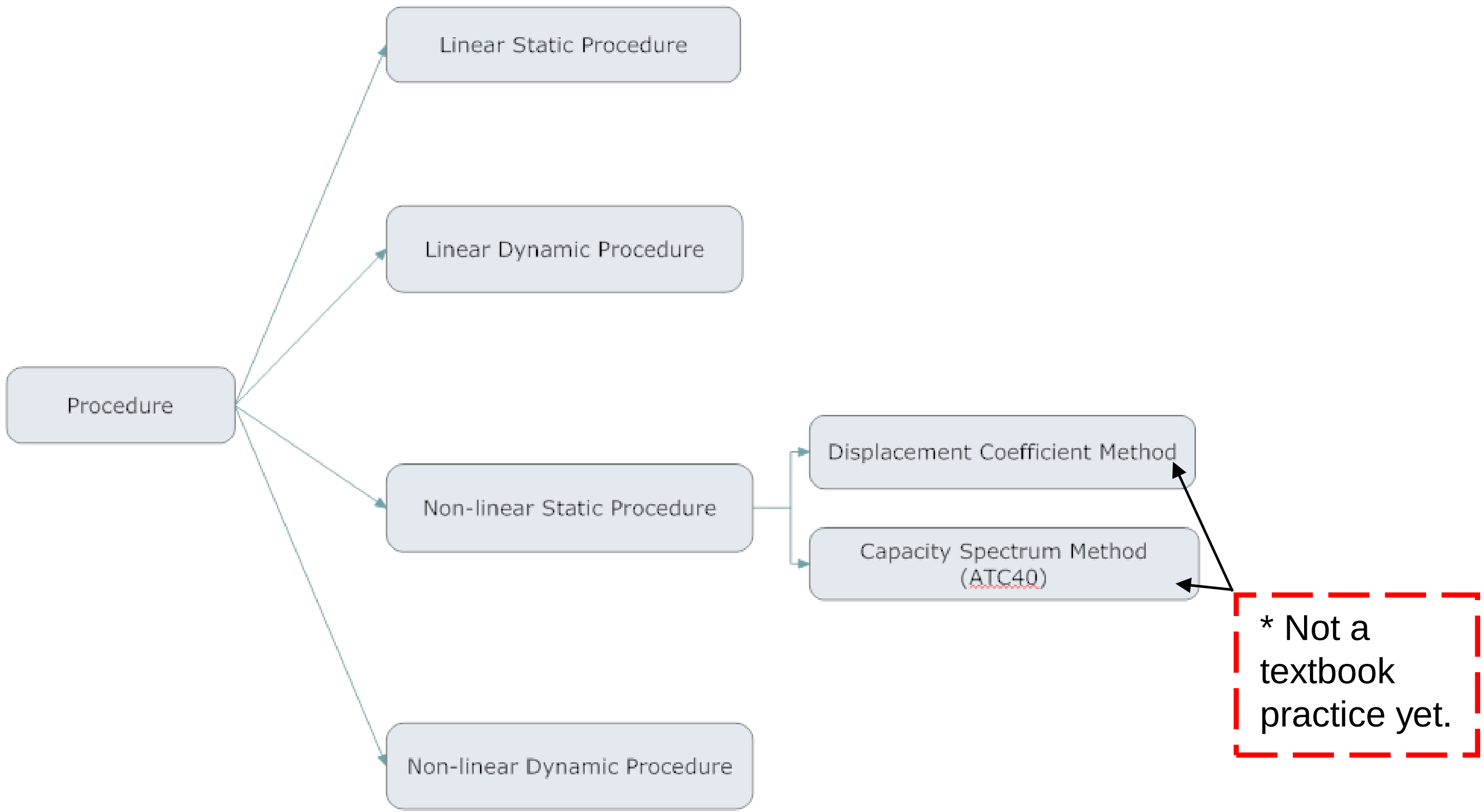
II. Analysis Procedures

1. 각 방법의 장단점 및 특기사항



	Tall, irregular buildings	Modeling	Seismic input	Advantage	Disadvantage
Linear Static Analysis (LSP)	No	Equivalent SDOF structural models	Response spectra	Very simple to analyze	Conservative/ Limitation of applicablity
Linear Dynamic Analysis (LDP)	yes	MDOF model	Response spectra / Ground-motion record	Compared to linear static procedures, higher modes can be considered	Applicability decreases with increasing nonlinear behaviour
Nonlinear Static Analysis (NSP)	Not very accurate	Equivalent SDOF structural model	Response spectra	Accounts for the non-linear behavior/ The ductility of the structure can be evaluated	Never be as accurate as Nonlinear Dynamic Analysis
Nonlinear Dynamic Analysis (NDP)	Yes	Detailed structural model	Ground-motion record	The most accurate method (이론상)	Very complicated and time consuming / Calculated response can be very sensitive to the characteristics of specific ground motion

“중약진 초고층, CH Lee”

- **Mathematical Structural Modeling**

- **Seismic Input**

1. Response Spectrum Input Required: LS/LDP_mode superpositon/NSP

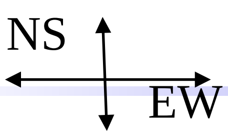
2. Acceleration Time Histories Required
   : LDP/NDP_Time History Analysis

Selection Acceleration Time Histories (1.6.6.2)

1. Time history analysis shall be performed with no fewer than three data sets (each containing two horizontal components) of ground motion time histories that shall be selected and scaled from no fewer than "three recorded events".

2. Time histories shall have magnitude, fault distances, and source mechanisms that are equivalent to those that control the design earthquake ground motion. Where three recorded ground-motion time history data sets having these characteristics are not available, simulated time history data sets having equivalent duration and spectral content shall be used to make up the total number required.

3. For each data set, the square root of the sum of the squares (SRSS) of the 5%-damped spectrum of the scaled horizontal components shall be constructed. The data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5%- damped spectrum for the design earthquake for periods between 0.2*T* seconds and 1.5*T* seconds (where *T* is the fundamental period of the building).



"SRSS= MEAN EXTREME (random vibration theory)"

4. Where three time history data sets are used in the analysis of a structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time history data sets are employed, the average value of each response parameter shall be permitted to determine design acceptability.

3.2.7 Multidirectional Seismic Effects

Buildings shall be designed for seismic motion in any horizontal direction. Multidirectional seismic effects shall be considered to act concurrently as specified in Section 3.2.7.1 for buildings meeting the following criteria:

1. The building has plan irregularities as defined in Section 2.4.1.1; or

2. The building has one or more primary columns which form a part of two or more intersecting frame or braced frame elements: 주로 corner column

3. All other buildings shall be permitted to be designed for seismic motions acting nonconcurrently in the direction of each principal axis of the building.

3.2.7.1 Concurrent Seismic Effects

When concurrent multidirectional seismic effects must be considered, horizontally oriented orthogonal X and Y axes shall be established. Elements and components of the building shall be designed for combinations of forces and deformations from separate analyses performed for ground motions in X and Y directions as follows:

1. **Where the LSP or LDP are used as the basis for design, elements and components shall be designed for:**
   (a) forces and deformations associated with 100% of the design forces in the X direction plus the forces and deformations associated with 30% of the design forces in the perpendicular horizontal Y direction, and for (b) forces and deformations associated with 100% of the design forces in the Y direction plus the forces and deformations associated with 30% of the design forces in the X direction.

2. **Where the NSP or NDP are used as the basis for design, elements and components of the building shall be designed for (a) forces and deformations associated with 100% of the design displacement in the X direction plus the forces (not deformations) associated with 30% of the design displacements in the perpendicular horizontal Y direction, and for (b) forces and deformations associated with 100% of the design displacements in the Y direction plus the forces (not deformations) associated with 30% of the design displacements in the X direction.**

3.2.7.2 Vertical Seismic Effects

For components in which Section 2.6.11 requires consideration of vertical seismic effects (cantilever, pre-stressed elements), the vertical response of a structure to earthquake ground motion need not be combined with the effects of the horizontal response.

3.2.8 Component Gravity Loads for Load Combinations

The following component gravity forces shall be considered for combination with seismic loads.

When the effects of gravity and seismic loads are additive, the gravity loads shall be obtained in accordance with Equation (3-3).

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad (3\text{-}3)$$

When the effects of gravity and seismic loads are counteracting, the gravity loads shall be obtained in accordance with Equation (3-4).

$$Q_G = 0.9 Q_D \qquad (3\text{-}4)$$

where:

Q_D = Dead-load (action).

Q_L = Effective live load (action), equal to 25% of the unreduced design live load, but not less than the actual live load.

Q_S = Effective snow load (action) contribution to W, specified in Section 3.3.1.3.1.

“FEMA 356의 구성이 다소 혼란스러움”

The analysis procedure shall comply with one of the following:

1) Linear analysis subject to limitations specified in Section 2.4.1, and complying with the Linear Static Procedure (LSP) in accordance with Section 3.3.1, or the Linear Dynamic Procedure (LDP) in accordance with Section 3.3.2.

“중점사항”

2) Nonlinear analysis subject to limitations specified in Section 2.4.2, and complying with the Nonlinear Static Procedure (NSP) in accordance with Section 3.3.3, or the Nonlinear Dynamic Procedure (NDP) in accordance with Section 3.3.4.

3) The analysis results shall comply with the applicable acceptance criteria selected in accordance with Section 2.4.4.

* **The linear procedures maintain the traditional use of a linear stress-strain relationship, but incorporate adjustments to overall building deformations and material acceptance criteria to permit better consideration of the probable nonlinear characteristics of seismic response.**

- **The Nonlinear Static Procedure, often called "pushover analysis," uses simplified nonlinear techniques to estimate seismic structural deformations.**

- **The Nonlinear Dynamic Procedure, commonly known as nonlinear time history analysis, requires considerable judgment and experience to perform, and may be used only within the limitations described in Section 2.4.2.2 of this standard.**

2. Nonlinear Analysis Procedure Study “First”
: DCM (FEMA 356) and CSM (ATC 40)

Note: “Pushover” (or nonlinear static lateral load) analysis-based method, all.

4.2 모델골조의 푸쉬오버 해석

“1998년 이철호 과학재단보고서”

4.2.1 해석 및 실험결과의 비교 방안

구조물에 적정의 내진성능이 확보되어 있는가의 여부를 확인하기 위한 이상적 방안은, 설계용지진동과 상응하는 다수의 지진가속도을 입력으로 하여 비선형 동적 시각력 해석을 수행하여 변형능력과 강도 등의 보유내진성능이 요구치 이상임을 확인하는 것이다. 그러나 이 방법은 실행에 따르는 여러가지 어려움으로 인해 특별한 경우를 제외하곤 거의 실용성이 없다. 대안으로 널리 사용되는 것이 소위 푸쉬오버 해석법이다 (Lawson-Vance-Krawinkler 1994). 즉 지진하중 작용시의 관성력 분포에 대응되는 횡하중 패턴 (통상 기본진동 모드의 관성력 패턴)을 사용하여 설계용 지진하중이 유발할 것으로 기대되는 횡변위 만큼 횡으로 정적 가력하여 근사적으로 동적 응답을 추정하는 것이다. 이러한 푸쉬오버의 해석의 제어에 사용되는 변위는 통상적으로 지붕층 횡변위가 사용된다. 적어도 푸쉬오버해석은 적어도 기본진동 모드가 유발하는 최대변형요구치를 비교적 정확하게 (가령 층간변위와 같은 global response 의 경우) 산정할 수 있다는 장점이 인정되고 있다.

본 연구에서는 실험을 통해서 얻어진 접합부의 가용 총소성회전각 (available total plastic rotation)은 곧 가용 총층간변위비 (available total plastic drift ratio)와 등가의 물리량임을 이미 주목한 바가 있다 (FIG. 9 및 2.7 절 참고). 따라서 푸쉬오버 해석의 결과에서 얻어지는 소성층간변위비의 요구치는 곧 실험에서 얻어진 가용 총소성회전각과 비교될 수 있는 응답치임을 알 수 있다.

4.2.2 목표변위의 산정

푸쉬오버의 해석의 제어에 사용되는 목표변위 (target displacement)는 일반적 관례를 따라 지붕층 횡변위를 사용한다. 지붕층의 횡변위는 탄성상태의 기본진동 모드를 사용하여서도 충분히 정확히 예측 가능하다 (Lawson et. al. 1994, Uang-Maarouf 1992). 이런 사실은 비선형 스펙트럼의 ‘속도영역”에서의 최대변위일정설과도 상통한다. 이 경우 내진규준의 설계용 스펙트럼이 유발할 지붕층 변위는 (곧 목표변위 X_R 는) 다음과 같이 계산된다:

$$X_R = \phi_{1R}(PF)_1\left(\frac{T_1^2}{4\pi^2}\right)S_a(T_1) \quad (3)$$

단 T_1 = 탄성해석에 의한 고유진동주기

$(PF)_1$ = 모드참여계수

$$= \frac{\{\phi\}_1^T[M]\{1\}}{\{\phi\}_1^T[M]\{\phi\}_1}$$

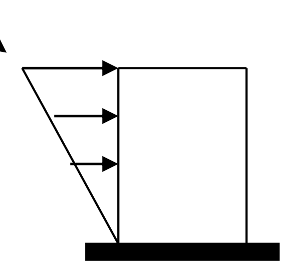
S_a = 내진규준의 탄성가속도스펙트럼

= (AICS)g

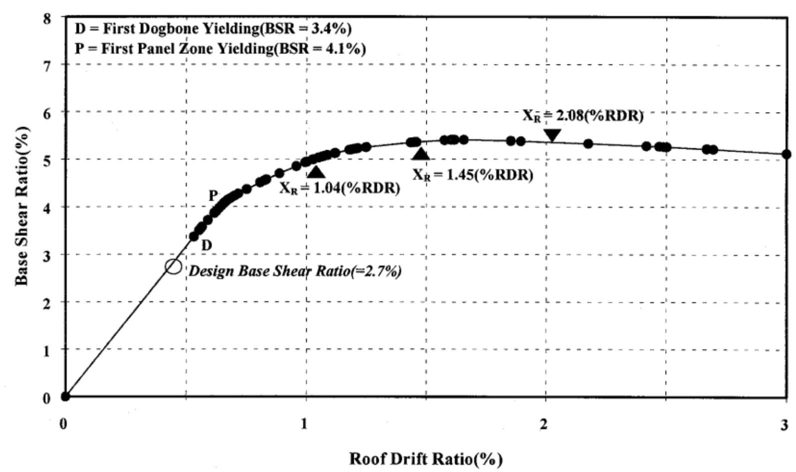
$\{\phi\}_1$ = 기본진동 모드벡터

“PLUS FINE-TUNING FACTORS”

“Essentially 1st mode inertia force-based lateral load pattern”



“Pushover” by modern Atlas (test= actuator, *analysis= computer program*)



2.1 Conceptual Basis

(1) Modal expansion of excitation vector $P(t)= \underline{S} \times p(t)$

$$\underline{m}\ddot{\underline{u}} + \underline{c}\dot{\underline{u}} + \underline{k}\underline{u} = \underline{S}p(t) \qquad \underline{P(t)} = \underline{S} \times p(t)$$

외력의 공간적 분포를 규정 **(constant vector)**

$$ex)\ P(t) = -\underline{m}\{1\}\ddot{u}_g(t) \quad \leftarrow EQ\ loading$$

$$\underline{S} = -\underline{m}\{1\},\ p(t) = \ddot{u}_g(t)$$

$$\underline{S} = \sum_{r=1}^{N} \underline{S}_r = \sum_{r=1}^{N} \Gamma_r \times \underline{m}\underline{\phi}_r$$

?

r차모드로 진동시 관성력의 공간적 분포패턴에 상당

$$\left(\because \underline{u}_n = \underline{\varphi}_n \ddot{q}_n(t),\ \underline{f}_{I_n} = -\underline{m}\underline{\varphi}_n \ddot{q}_n(t)\right)$$

orthogonality

$$\underline{\phi}_n^T \underline{S} = \phi_n^T \times \sum_{r=1}^{N} \Gamma_r \underline{m}\underline{\phi}_r = \Gamma_n\left(\underline{\phi}_n^T \underline{m}\underline{\phi}_n\right) = \Gamma_n M_n$$

$$\therefore \Gamma_n = \frac{\underline{\phi}_n^T \underline{S}}{M_n} = \frac{\underline{\phi}_n^T \underline{S}}{\underline{\phi}_n^T \underline{m}\underline{\phi}_n} \quad and \quad \underline{S}_n = \Gamma_n \times \underline{m}\underline{\phi}_n$$

“S_n 은 단지 n 차 모드의 응답만 유발한다”:

$$\underline{m}\,\underline{\ddot{u}} + \underline{c}\,? + \underline{k}\,\underline{u} = \underline{S}p(t) = \left[\underline{S}_1 + \underline{S}_2 + \dots + \underline{S}_N\right] \times p(t)$$

$$= \left[\Gamma_1\,\underline{m}\,\underline{\phi}_1 + \dots + \Gamma_N\,\underline{m}\,\underline{\phi}_N\right] \times p(t)$$

$$\underline{\phi}_n^T\left[\underline{m}\,\underline{\ddot{u}} + \underline{c}\,? + \underline{k}\,\underline{u}\right] = \underline{\phi}_n^T\left[\Gamma_1\,\underline{m}\,\underline{\phi}_1 + \dots + \Gamma_N\,\underline{m}\,\underline{\phi}_N\right] \times p(t)$$

$$= \underline{\phi}_n^T\,\Gamma_n\,\underline{m}\,\underline{\phi}_n \times p(t)$$

$$M_n\ddot{q}_n + C_n\dot{q}_n + K_n q_n = \Gamma_n M_n p(t)$$

n 차모드의 동적응답을 partial force vector Sn에만 상관됨을 알 수 있음

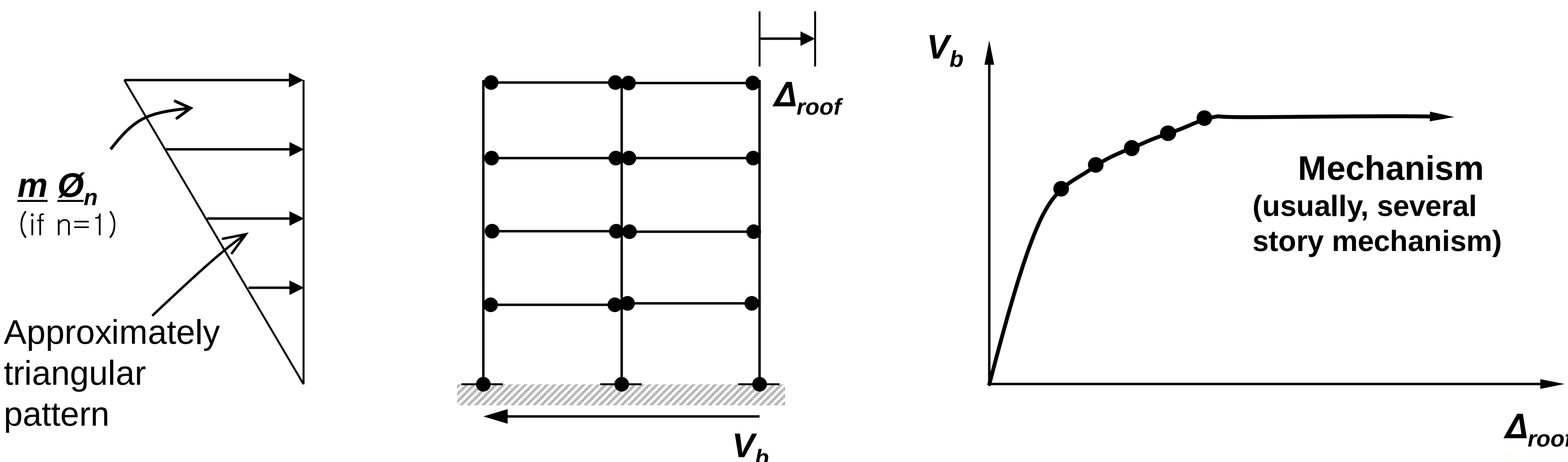
(2) The most appropriate of EQ loading (for static procedure, given a response spectrum)

$$\underline{f}_n = \Gamma_n \underline{m}\,\underline{\phi}_n \times S_a(\zeta_n, T_n) \cdots\cdots\cdots (*)$$

Equivalent static
force distribution

the most general form of the lateral force vector
to be used in a pushover analysis.

If n=1, only the first mode contributions are considered.



2.2 Summary of Disp. Coef. Method (FEMA 356)

$$\ddot{q}_n + 2\xi_n\omega_n\dot{q}_n + \omega_n^2 = -\Gamma_n\ddot{u}_g(t)$$

"Decoupled modal eq. using normal coordinate"

$$\boldsymbol{u_{n,max}} = \underline{\boldsymbol{\phi}}_{\boldsymbol{n}} \times \boldsymbol{q_{n,max}} = \underline{\boldsymbol{\phi}}_{\boldsymbol{n}} \times \boldsymbol{\Gamma_n S_d(\xi_n, \omega_n)} = \begin{bmatrix} \phi_{1n} \\ \phi_{2n} \\ \vdots \\ \phi_{Nn} \end{bmatrix} \times \boldsymbol{\Gamma_n S_d(\xi_n, \omega_n)}$$

1차모드가 지배적이고(n= 1) N= Roof level로 고려하면,

$$u_{1,ROOF,max} = \phi_{N1} \times \Gamma_1 S_d(\xi_1, \omega_1) = \phi_{N1} \times \Gamma_1 S_a(\xi_1 = 5\%, \omega_1)/\omega_1^2$$
$$= \phi_{N1} \times \Gamma_1 S_a(\xi_1 = 5\%, \omega_1)/(2\pi/T_1)^2$$

C_0

"Basic" roof(흔히 control level) target disp. for pushover analysis

Roof Target Displacement for DCM

"FINE-TUNING FACTORS"

$$\delta_t = C_0 \boxed{C_1 \ C_2 \ C_3} S_a \frac{T_e^2}{4\pi^2} g$$

T_e =Effective fundamental period of the building

S_a = Response spectrum acceleration

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$

g = acceleration of gravity

C_0 = Modification factor to relate spectral displacement

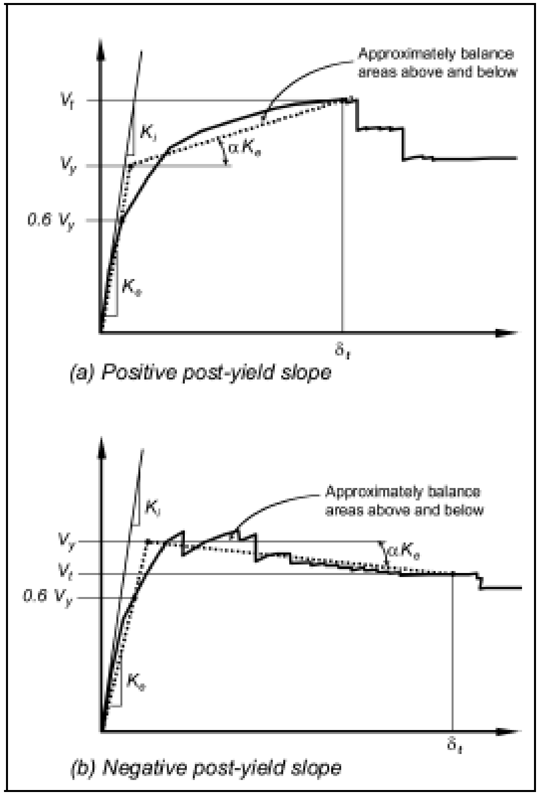


Figure 3-1 Idealized Force-Displacement Curves

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response

C_2 = Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects

$$\boxed{C_0} = \frac{\sum_{i=1}^{N}(w_i \Phi_{i,1})/g}{\sum_{i=1}^{N}(w_i \Phi_{i,1}^2)/g} \times \Phi_{con,1}$$

$$= \Gamma_1 \times \Phi_{con,1}$$

Γ1 = modal participation factor

$$C_1 = 1.0 \text{ for } T_e \geq T_S$$
$$= [1.0 + (R-1)T_S/T_e]/R \text{ for } T_e < T_S$$

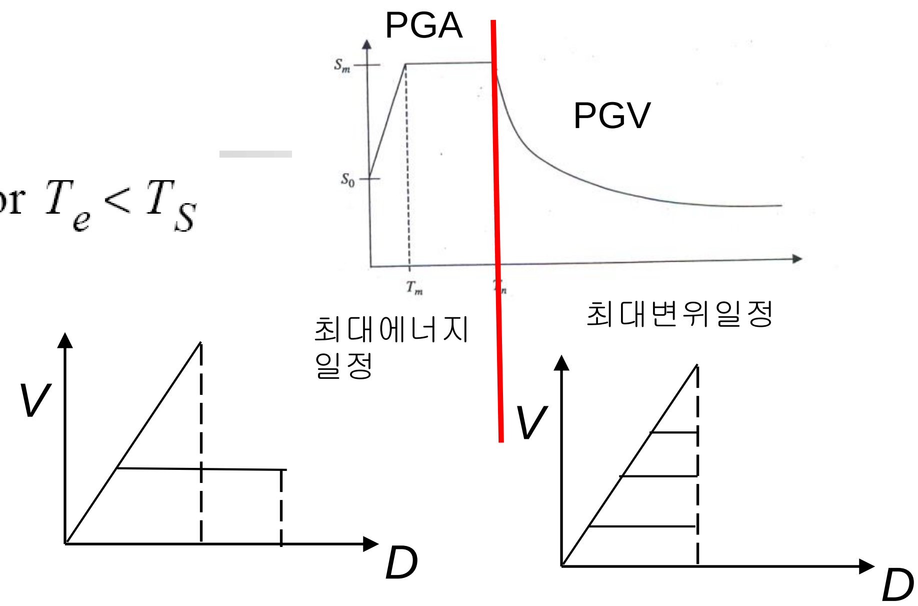


Table 3-3 Values for Modification Factor C_2				
	$T \leq 0.1$ second[3]		$T \geq T_S$ second[3]	
Structural Performance Level	Framing Type 1[1]	Framing Type 2[2]	Framing Type 1[1]	Framing Type 2[2]
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

1. Structures in which more than 30% of the story shear at any level is resisted by any combination of the following components, elements, or frames: ordinary moment-resisting frames, concentrically-braced frames, frames with partially-restrained connections, tension-only braces, unreinforced masonry walls, shear-critical, piers, and spandrels of reinforced concrete or masonry.
2. All frames not assigned to Framing Type 1.
3. Linear interpolation shall be used for intermediate values of T.

$$\boxed{C_3} = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e}$$

$$R = \frac{S_a}{V_y/W} \cdot C_m$$ = seismic force reduction factor

- ✓ **α = Ratio of post-yield stiffness to effective elastic stiffness.**

"그러나 대부분이 C1, C2, C3 등의 계수가 1.0으로 평가될 것임"

Lateral Load Pattern for Pushover

Lateral loads shall be applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load shall be applied. One pattern shall be selected from each of the following two groups:

1. A modal pattern selected from one of the following:

   1.1. A vertical distribution proportional to the values of C_{vx} given in Equation (3-12). Use of this distribution shall be permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is also used.

   1.2. A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution shall be permitted only when more than 75% of the total mass participates in this mode.

   1.3. A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum. This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.

초고층건물

2. A second pattern selected from one of the following:

   2.1. A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.

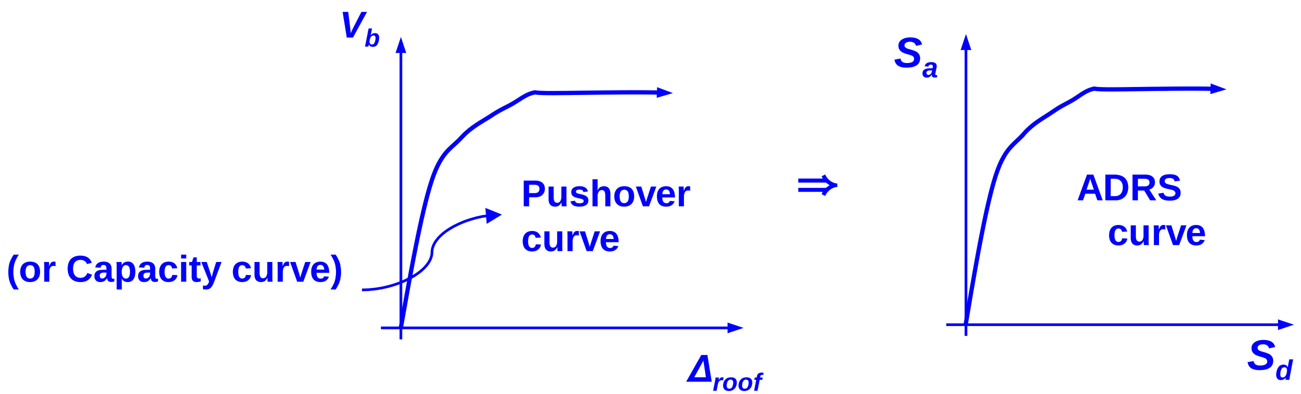
   2.2. An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution will depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure. Use of more than one lateral load pattern is intended to bound the range of design actions that may occur during actual dynamic response.

2.3 Capacity Spectrum Method (ATC 40)

*** The concept of <u>capacity</u> spectrum and <u>ADRS</u> conversion**

Acceleration
Displacement
Response Spectrum



$$\ddot{q}_n + 2\xi_n\omega_n\dot{q}_n + \omega_n^2 = -\Gamma_n\ddot{u}_g(t)$$

$$u_{n,max} = \underline{\phi}_n \times q_{n,max} = \underline{\phi}_n \times \Gamma_n S_d(\xi_n, \omega_n) = \begin{bmatrix} \phi_{1n} \\ \phi_{2n} \\ \vdots \\ \phi_{Nn} \end{bmatrix} \times \Gamma_n S_d(\xi_n, \omega_n)$$

If DOF N represents the roof level, and only the first-mode contribution is considered,

$$u_{n,\max}(=u_{roof,\max}) = \phi_{N1}\Gamma_1 S_d(\zeta_1,\omega_1) \qquad \cdots\cdots(\bigstar)$$

Eq.(★) is used to convert the roof displacement from a pushover analysis to the First-modal spectral displacement in the capacity spectrum procedure.

From pushover analysis

Or $$S_d = \frac{u_{N,\max}}{\Gamma_1\phi_{N1}} = \frac{\Delta roof}{\Gamma_1\phi_{N1}}$$

To establish the equivalent first-mode spectral acceleration from the base shear.

$$\underset{n\times 1}{\underline{f_1}} = \Gamma_1 \underset{n\times n}{\underline{m}} \underset{n\times 1}{\overline{\phi_1}} \times S_a(\zeta_1, T_1)$$

← 앞 장의 식(*) 참조 (소위 **equivalent static force**)

$$V_{b1} = \{1\}^T \underline{f_1} = \Gamma_1 \{1\}^T m\phi_1 \times S_a(\zeta_1, T_1)$$ Or $$S_a(\zeta_1, T_1) = \frac{V_b}{\Gamma_1\{1\}^T m\phi}$$

ATC 40 advocates the use of the CSM to evaluate the overall adequacy of design of a structure system. The term "capacity spectrum" refers to an altered form of the pushover curve for the building.

Provides a representation of both displacement and the force capacity of a building in terms of roof drift and base shear, respectively.

The CSM involves a simple graphical procedure wherein the reformatted capacity curve is compared to the seismic demand curve, which is also expressed in a similar format. The objective is to determine the "performance point" of the structure that identifies the demand corresponding to the hazard at the site specified in terms of a response spectrum.

- Determining Capacity

  The preferred method of choice = nonlinear static or pushover analysis

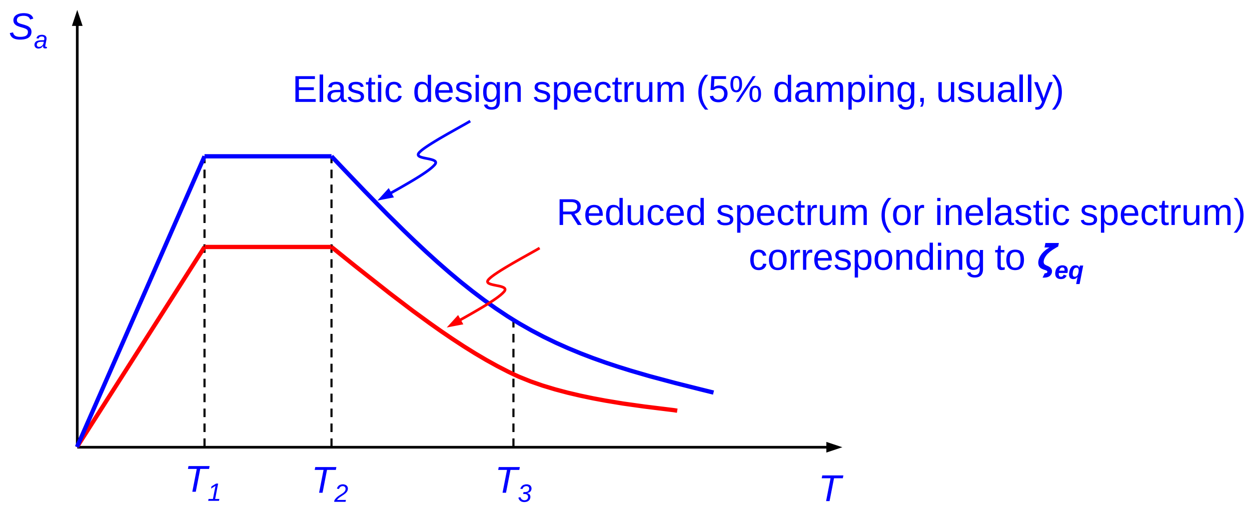
- Conversion to ADRS Format

$$S_d = \frac{\Delta_{roof}}{\Gamma_1 \phi_{N1}};\ S_a = \frac{V_b}{\Gamma_1 \{1\} \underline{m} \underline{\phi}_1}$$

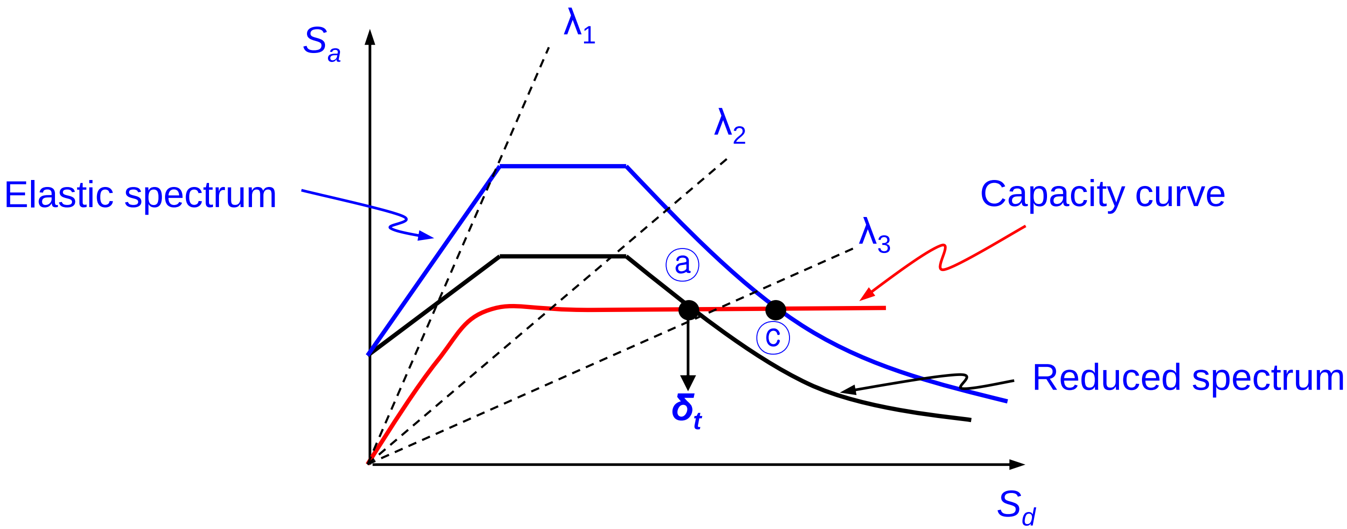
Determining demand

Performance point

Given : Design spectrum (or demand spectrum)



$$S_d = \frac{S_a}{\omega_n^2} = \left(\frac{T}{2\pi}\right)^2 \times S_a \quad ; \quad \frac{S_a}{S_d} = \left(\frac{2\pi}{T}\right)^2 = \lambda \qquad \cdots\cdots(\bigstar)$$



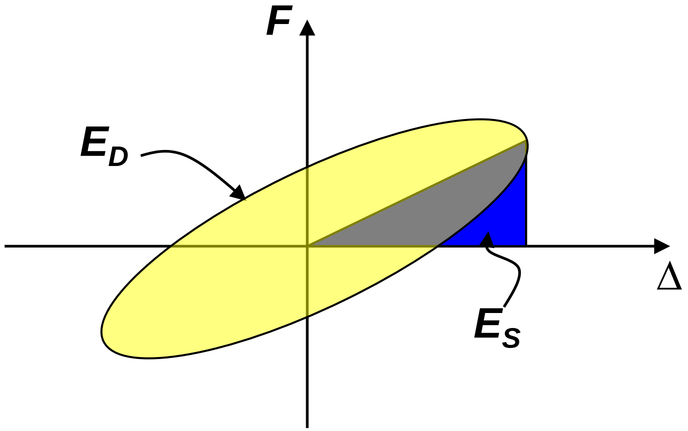
: overlaying demand and capacity spectrum

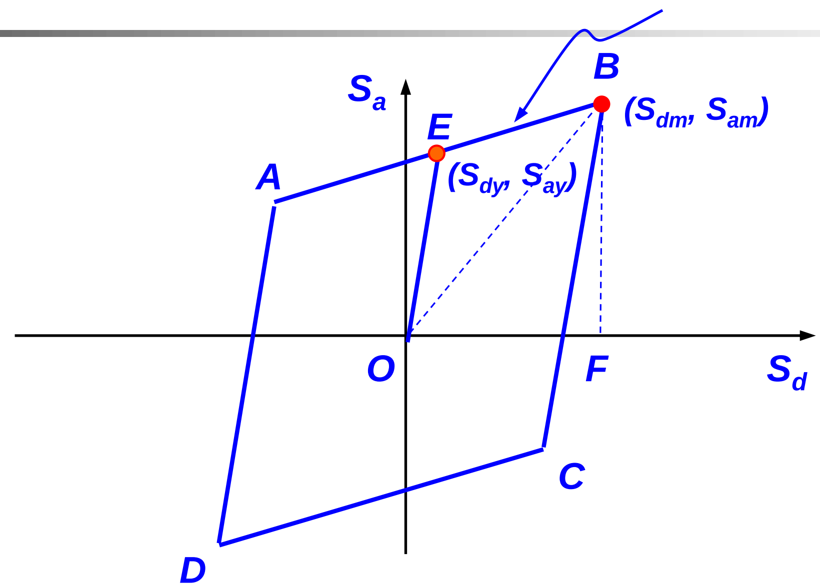
If the elastic design spectrum is used to create the demand spectrum, the overlaying is valid only if the structural response is also elastic

The concept of equivalent viscous damping is used to reduce the elastic spectrum to an inelastic spectrum in the CSM.

$$\zeta_d = \frac{1}{(\Omega/\omega)} \frac{1}{4\pi} \frac{E_D}{E_S} = \frac{1}{4\pi} \frac{E_D}{E_S}$$

Forcing to natural
Frequency ratio





: Bilinear representation of capacity curve and peak demand parameters

$$\zeta_d = \frac{1}{4\pi}\frac{(\square ABCD\text{면적})}{(\triangle OBF\text{면적})} = 0.637 \times \frac{(S_{ay} \cdot S_{dm} - S_{dy} \cdot S_{ay})}{S_{am} S_{dm}} \qquad \cdots\cdots (\bigstar)$$

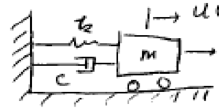
$$\zeta_{eq} = k \cdot \zeta_d + 0.05 \qquad \cdots\cdots (\bigstar\bigstar)$$

가령 pinched hysteresis loop
other than bilinear (k= 0.67, etc)
←highly empicrical

Default damping

Hysteretic damping (plastic)

3.8절 점성감쇠에 의한 에너지 손실 보충



$u(t) = u_0 \sin(\omega t - \phi)$ ← 정상상태 응답

Eqs. (3.2.10) ~ (3.2.12)

$\rightarrow p_0 \sin \omega t$

$$m\ddot{u} + c\dot{u} + ku = p_0 \sin \omega t$$

$$(m\ddot{u} + c\dot{u} + ku) \cdot du = p_0 \sin \omega t \cdot du \quad \leftarrow du = \dot{u}\,dt$$

"정상상태의 1cycle 동안에 대해 적분하면" $[\,0,\ T = \frac{2\pi}{\omega}\,]$

$$\underbrace{\int_0^{2\pi/\omega} m(\ddot{u}\dot{u})\,dt}_{E_k = 0 \text{ (orthogonal)}} + \int_0^{2\pi/\omega} c(\dot{u}^2)\,dt + \underbrace{\int_0^{2\pi/\omega} k\,u \cdot \dot{u}\,dt}_{E_s = 0 \text{ (orthogonal)}} = \int_0^{2\pi/\omega} p_0 \sin \omega t \cdot \dot{u}\,dt$$

$$E_D \text{ (per cycle)} = c \int_0^{2\pi/\omega} \{u_0 \omega \cos(\omega t - \phi)\}^2\,dt$$

$$= \pi c \omega u_0^2 = \pi \cdot (\zeta \cdot 2 m \omega_n)\,\omega\, u_0^2$$

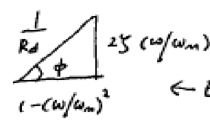
$$= \pi \left(\zeta\, 2 \cdot \frac{k}{\omega_n^2} \cdot \omega_n\right) \omega\, u_0^2$$

$$\therefore E_D = 2\pi\zeta\left(\frac{\omega}{\omega_n}\right) k\, u_0^2$$

← 가진주파수에 직선 비례,
진폭의 제곱에 비례

$$E_I \text{ (per cycle)} = \int_0^{2\pi/\omega} \{p_0 \sin \omega t \cdot u_0 \omega \cos(\omega t - \phi)\}\,dt$$

$$= \pi p_0 u_0 \sin\phi \ ;$$



← Eq (3.2.11)

$$= \pi p_0 u_0 \times \frac{2\zeta(\omega/\omega_n)}{\frac{1}{R_d}}$$

$$= \pi p_0 u_0 \times \left(2\zeta \cdot \frac{\omega}{\omega_n}\right) R_d$$

$$= \pi p_0 u_0 \times (2\zeta)\left(\frac{\omega}{\omega_n}\right) \cdot \frac{u_0}{(u_{st})_0}$$

$$= \cancel{\pi} p_0 u_0 \times (2\zeta)\left(\frac{\omega}{\omega_n}\right) \cdot \frac{u_0}{p_0/k}$$

$$\therefore E_I = (2\pi\zeta)\left(\frac{\omega}{\omega_n}\right) k\, u_0^2$$

Note:

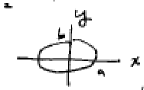
$$\underset{\text{에너지 input}}{E_I} = \underset{\text{에너지 dissipation}}{E_D}$$

A Graphical Interpretation of the energy in viscous damping ③

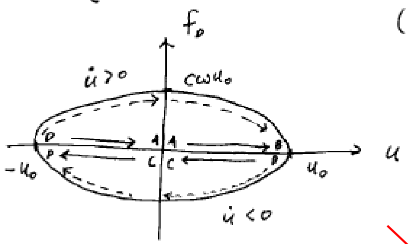
$$f_D = c\dot{u}(t) = c\omega \cdot u_0 \cos(\omega t - \phi)$$
$$= c\omega\sqrt{u_0^2 - u_0^2 \sin^2(\omega t - \phi)}$$
$$= c\omega\sqrt{u_0^2 - [u(t)]^2} = c\omega u_0 \sqrt{1 - \left\{\frac{u(t)}{u_0}\right\}^2}$$

or $\left[\frac{u(t)}{u_0}\right]^2 + \left[\frac{f_D}{c\omega u_0}\right]^2 = 1$ ← 타원방정식

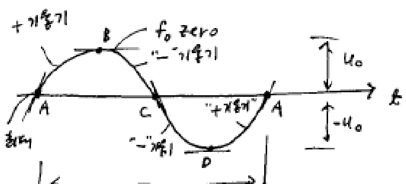
($\frac{x^2}{a^2} + \frac{y^2}{b^2} = 1$



area = πab)



" 감쇠력은 기울기에 비례,
($f_D = c\dot{u}$)

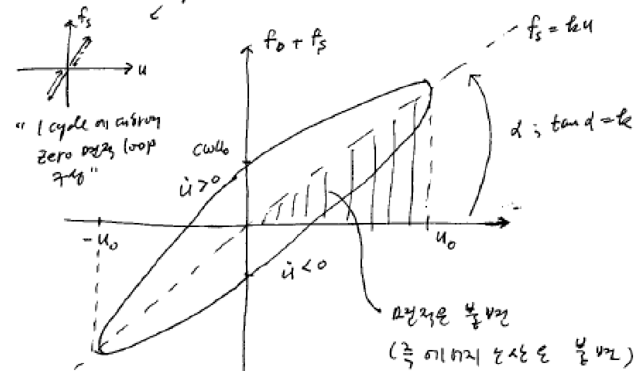


" 1 cycle 동안의 $u - f_D$ 관계를 추적해서 보면 하나의 loop, 즉 hysteresis loop 이 구성됨을 알 수 있다 "

Loop area = 에너지 손실량
$= \pi(u_0 \times c\omega u_0) = \pi c\omega u_0^2$ ← 이전의 결과와 동일!

총저항력 ($f_D + f_S$, damping + elastic force) 과 변위의 관계 ④

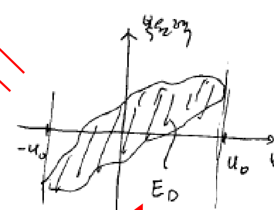
$$f_D + f_S = \underbrace{ku}_{\text{single valued function}} + c\omega\sqrt{u_0^2 - u^2}$$



* specific damping capacity

$$\frac{E_D}{E_{S0}} \equiv \frac{E_D}{\left(\frac{k}{2}u_0^2\right)}$$

* Equivalent Viscous Damping



(가령 실험에서 얻어진 one cycle 이력곡선)

"hysteretic damping"

$$E_D = 2\pi\zeta\frac{\omega}{\omega_n}ku_0^2$$
$$= 4\pi\zeta_{eq}\left(\frac{\omega}{\omega_n}\right)\underbrace{\left(\frac{1}{2}ku_0^2\right)}_{E_{S0}}$$

" 1 cycle 의 조화진동시의 감쇠에 의한 에너지 손실량 " (theory)

or $\zeta_{eq} = \frac{1}{4\pi}\left(\frac{1}{\omega/\omega_n}\right)\left(\frac{E_D}{E_{S0}}\right)$

공진주파수에서 실험하여 E_D 를 측정한다면 ($\omega = \omega_n$)

$$\therefore \zeta_{eq} = \frac{1}{4\pi}\left(\frac{E_D}{E_{S0}}\right)$$

- **New mark –Hall spectrum for <u>general damping values (including high damping): EPP</u>**

(★★★)

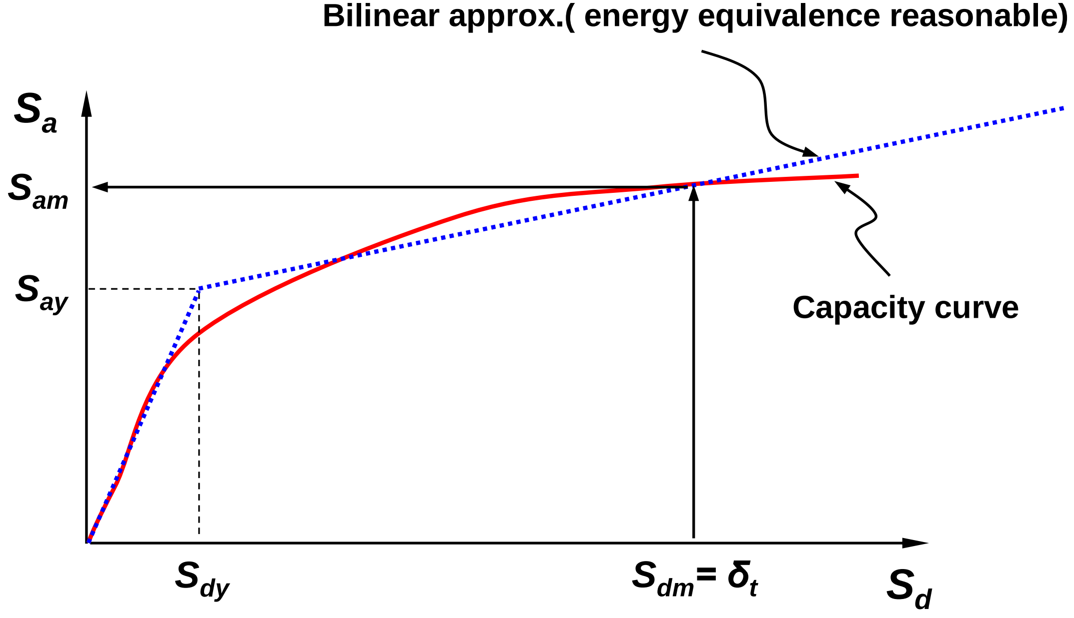
$$SR_A = \frac{3.21 - 0.68\ln(100\zeta_{eq})}{2.12}$$

$$SR_V = \frac{2.31 - 0.41\ln(100\zeta_{eq})}{1.65}$$

Performance point
(Iteration procedure, graphical method)

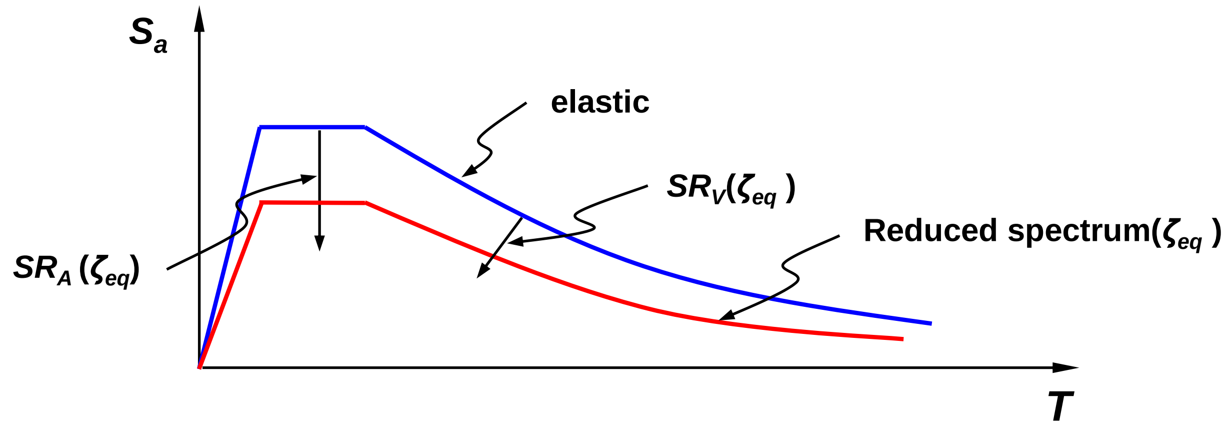
(1) Trial displacement (δ_t) : $S_{dm} = \delta_t$

(2) S_a on capacity curve

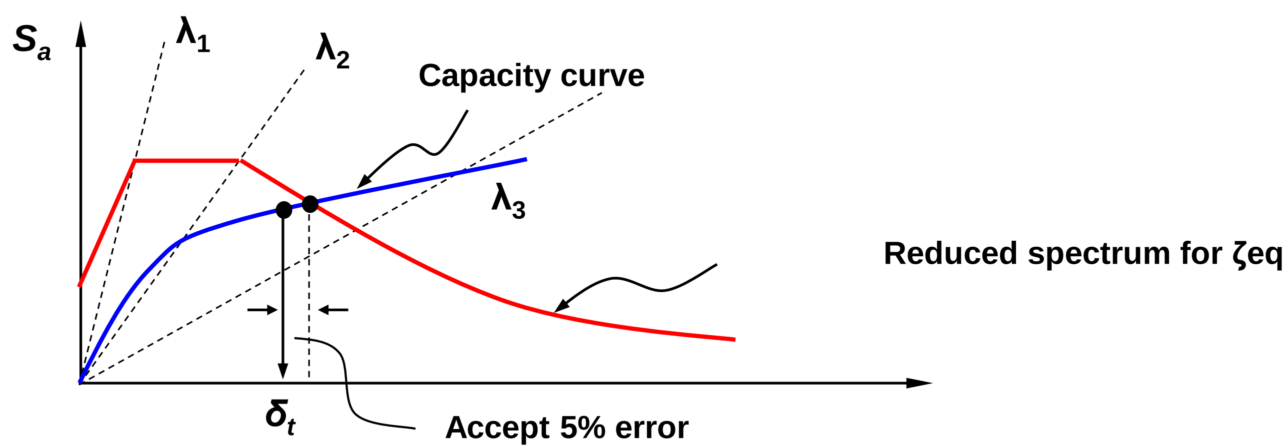


(3) Use (★) and (★★) to get ζ_{eq}

(4) Reduce elastic spectrum using (★★★)



(5) Superpose reduced spectrum and ADRS-formatted capacity spectrum



3. Linear Procedure

• Method to Determine Limitations on Use of Linear Procedures (2.4.1.1)

The results of the DCR (Demand to Capacity Ratio) analysis shall be used to identify the magnitude and uniformity of distribution of inelastic demands on the primary elements and components of the lateral-force resisting system:

$$DCR = \frac{Q_{UD}}{Q_{CE}} \qquad (2-1)$$

Q_{UD} = Force demand due to the gravity and earthquake loads calculated in accordance with Section 3.4.2.
Q_{CE} = Expected strength capacity of the component or element, calculated as specified in Chapters 5 through 8.

• DCRs shall be calculated for each action (such as axial force, moment, shear) of each primary component: 다소 **heuristic!**

• The critical action for the component shall be the one with the largest DCR. The DCR for this action shall be termed the critical component DCR.

•The largest DCR for any element at a particular story is termed the critical element DCR at that story.

The applicability of linear procedures shall be determined as follows:
1) If all component DCRs < 2.0, then linear procedures are applicable (가령 중약진대 초고층).

2) If one or more component DCRs exceed 2.0, and no irregularities described in Sections 2.4.1.1.1 through 2.4.1.1.4 are present, then linear procedures are applicable (가령 중약진대 초고층).

3) If one or more component DCRs exceed 2.0 and any irregularity described in Section 2.4.1.1.1 through Section 2.4.1.1.4 is present, then linear procedures are not applicable, and shall not be used.

Sections 2.4.1.1.1 through 2.4.1.1.4: 입면불연속/수직셋백 /severe weak story/비틀림비정형에 대한 간단한 판단근거를 제시

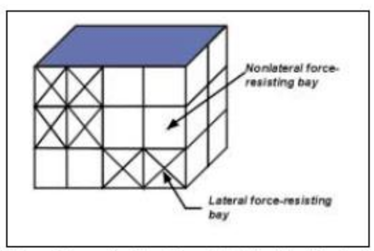


Figure 2-1 In-Plane Discontinuity in Lateral System

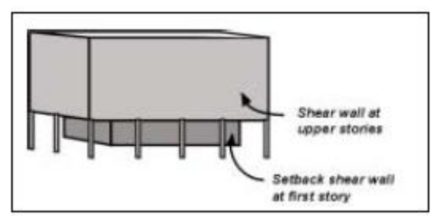


Figure 2-2 Typical Building with Out-of-Plane Offset Irregularity

• NSP Supplemented by LDP

• Nonlinear procedures shall be used for analysis of buildings when linear procedures are not permitted.

• The NSP shall be permitted for structures in which higher mode effects are not significant: Higher mode effects shall be considered significant if the shear in any story resulting from the modal analysis considering modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear considering only the first mode response: 즉 고차모드 효과가 30% 이상이면 NSP 불가.

* **If higher mode effects are significant, the NSP shall be permitted if an LDP analysis is also performed to supplement the NSP.** Buildings with significant higher mode effects must meet the acceptance criteria of this standard for both analysis procedures, except that an increase by a factor of 1.33 shall be permitted in the LDP acceptance criteria for deformation-controlled actions (*m*-factors) provided in Chapters 5 through 9.

*** When the NSP is utilized on a structure that has significant higher mode response, the LDP is also employed to verify the adequacy of the design. When this approach is taken, less restrictive criteria are permitted for the LDP, recognizing the significantly improved knowledge that is obtained by performing both analysis procedures.**

Nonlinear Dynamic Procedure

• The NDP shall be permitted for all structures. An analysis performed using the NDP shall be reviewed and approved by an independent third-party engineer with experience in seismic design and nonlinear procedures.

III. Acceptance Criteria

The current thinking in FEMA 356 is that the performance of a component in the system is critical to the overall seismic performance of the building. Consequently, acceptance criteria are specified <u>at the element level</u>.

- **Relation of Structural (global) performance levels to the limiting damaging states as reportyed in FEMA 356:**

Table C1-3 Structural Performance Levels and Damage[1, 2, 3]—Vertical Elements

Elements	Type	Structural Performance Levels: Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Many braces yield or buckle but do not totally fail. Many connections may fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary.	Same as primary.	Same as primary.
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent

Table C1-3 Structural Performance Levels and Damage[1, 2, 3]—Vertical Elements (continued)

Elements	Type	Structural Performance Levels: Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element stress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, <1/16" wide. Coupling beams experience cracking <1/8" width.
	Secondary	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at joints. Extensive crushing. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks <1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent
Unreinforced Masonry Infill Walls	Primary	Extensive cracking and crushing; portions of face course shed.	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings.	Minor (<1/8" width) cracking of masonry infills and veneers. Minor spalling in veneers at a few corner openings.
	Secondary	Extensive crushing and shattering; some walls dislodge.	Same as primary.	Same as primary.
	Drift	0.6% transient or permanent	0.5% transient; 0.3% permanent	0.1% transient; negligible permanent
Unreinforced Masonry (Noninfill) Walls	Primary	Extensive cracking; face course and veneer may peel off. Noticeable in-plane and out-of-plane offsets.	Extensive cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets.	Minor (<1/8" width) cracking of veneers. Minor spalling in veneers at a few corner openings. No observable out-of-plane offsets.
	Secondary	Nonbearing panels dislodge.	Same as primary.	Same as primary.
	Drift	1% transient or permanent	0.6% transient; 0.6% permanent	0.3% transient; 0.3% permanent

Table C1-3 Structural Performance Levels and Damage[1, 2, 3]—Vertical Elements (continued)

Elements	Type	Structural Performance Levels: Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Reinforced Masonry Walls	Primary	Crushing; extensive cracking. Damage around openings and at corners. Some fallen units.	Extensive cracking (<1/4") distributed throughout wall. Some isolated crushing.	Minor (<1/8" width) cracking. No out-of-plane offsets.
	Secondary	Panels shattered and virtually disintegrated.	Crushing; extensive cracking; damage around openings and at corners; some fallen units.	Same as primary.
	Drift	1.5% transient or permanent	0.6% transient; 0.6% permanent	0.2% transient; 0.2% permanent
Wood Stud Walls	Primary	Connections loose. Nails partially withdrawn. Some splitting of members and panels. Veneers dislodged.	Moderate loosening of connections and minor splitting of members.	Distributed minor hairline cracking of gypsum and plaster veneers.
	Secondary	Sheathing sheared off. Let-in braces fractured and buckled. Framing split and fractured.	Connections loose. Nails partially withdrawn. Some splitting of members and panels.	Same as primary.
	Drift	3% transient or permanent	2% transient; 1% permanent	1% transient; 0.25% permanent
Precast Concrete Connections	Primary	Some connection failures but no elements dislodged.	Local crushing and spalling at connections, but no gross failure of connections.	Minor working at connections; cracks <1/16" width at connections.
	Secondary	Same as primary.	Some connection failures but no elements dislodged.	Minor crushing and spalling at connections.
Foundations	General	Major settlement and tilting.	Total settlements <6" and differential settlements <1/2" in 30 ft.	Minor settlement and negligible tilting.

1. Damage states indicated in this table are provided to allow an understanding of the severity of damage that may be sustained by various structural elements when present in structures meeting the definitions of the Structural Performance Levels. These damage states are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure following an earthquake.

2. Drift values, differential settlements, crack widths, and similar quantities indicated in these tables are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design in accordance with the analysis procedures provided in this standard; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various Structural Performance Levels. Drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components. Acceptable levels of foundation settlement or movement are highly dependent on the construction of the superstructure. The values indicated are intended to be qualitative descriptions of the approximate behavior of structures meeting the indicated levels.

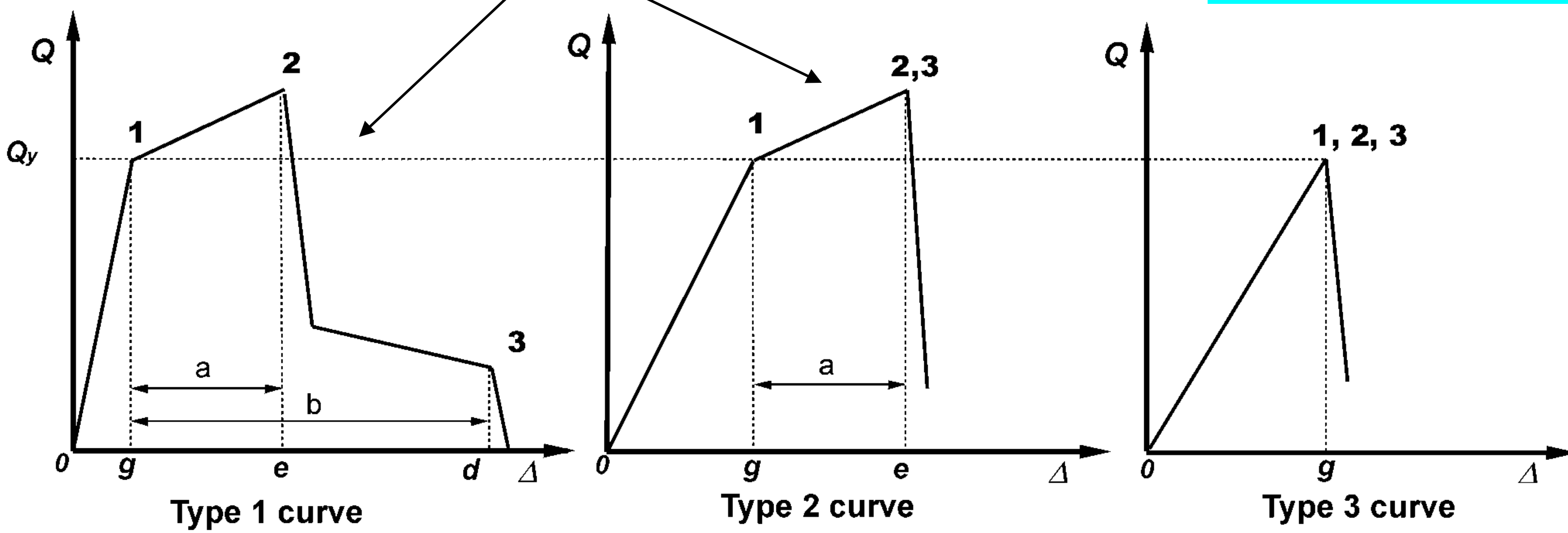
3. For limiting damage to frame elements of infilled frames, refer to the rows for concrete or steel frames.

Deformation- and Force-Controlled Actions

All actions shall be classified as either deformation controlled or force-controlled using the component force versus deformation curves shown in Figure 2-3.

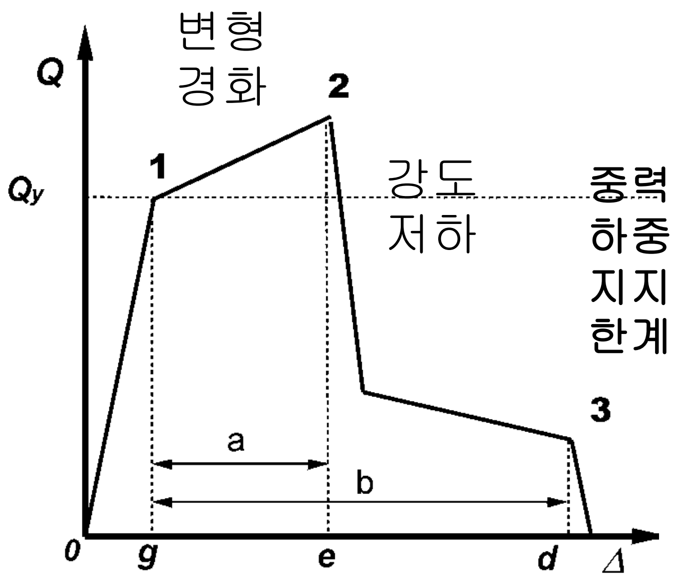
"Deformation-controlled if $e > 2g$"

"Force-controlled"



: Figure 2-3 Component Force Versus Deformation Curves

"흔히 **pushover**해석에서 이 **3**가지 가운데 하나를 택함"

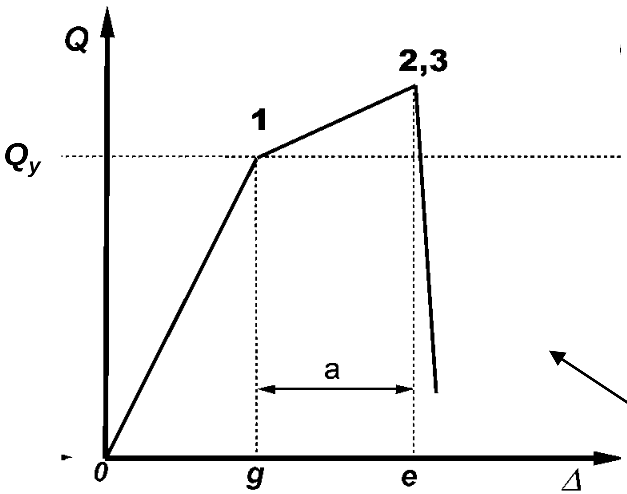


Type 1 curve

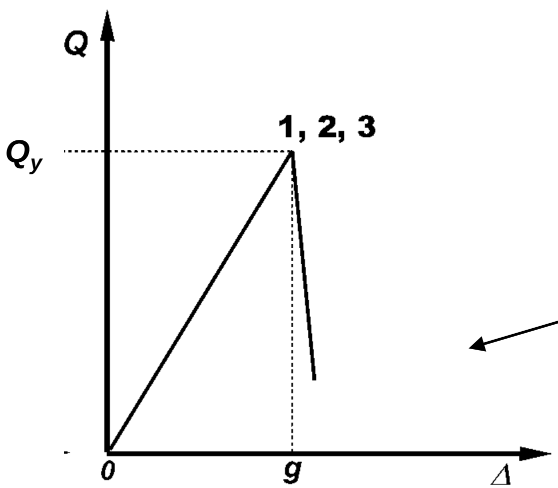
* The Type 1 curve depicted in Figure 2-3 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by a plastic range (points 1 to 3) with non-negligible residual strength and ability to support gravity loads at point 3.

The plastic range includes a strain hardening or softening range (points 1 to 2) and a strength-degraded range (points 2 to 3).

Primary component actions exhibiting this behavior shall be classified as deformation-controlled if the strain-hardening or strain softening range is such that $e > 2g$; otherwise, they shall be classified as force-controlled.



Type 2 curve



Type 3 curve

The Type 2 curve depicted in Figure 2-3 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) and a plastic range (points 1 to 2) followed by loss of strength and loss of ability to support gravity loads beyond point 2. Primary component actions exhibiting this type of behavior shall be classified as deformation-controlled if the plastic range is such that *e*> 2*g*; otherwise, they shall be classified as force controlled.

The Type 3 curve depicted in Figure 2-3 is representative of a brittle or nonductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by loss of strength and loss of ability to support gravity loads beyond point 1. Primary component actions displaying Type 3 they shall be classified as force-controlled.

1) <u>Acceptance criteria</u> for primary components that exhibit Type 1 and 2 behavior are typically within the elastic or plastic ranges between points 0 and 2, depending on the performance level.

2) <u>Acceptance criteria</u> for primary components exhibiting Type 3 behavior will <u>always be within the elastic range</u>.

*** Classification of force- or deformation-controlled actions are specified for framing components in Chapters 5 through 8:**
Classification as a deformation-controlled action is not up to the discretion of the user:
사용자의 입장에서 고민할 필요 없음

*** A given component may have a combination of both force- and deformation-controlled actions.**

•Deformation-controlled actions have been defined in this standard by the designation of *m*-factors or nonlinear deformation capacities in Chapters 5 through 8.

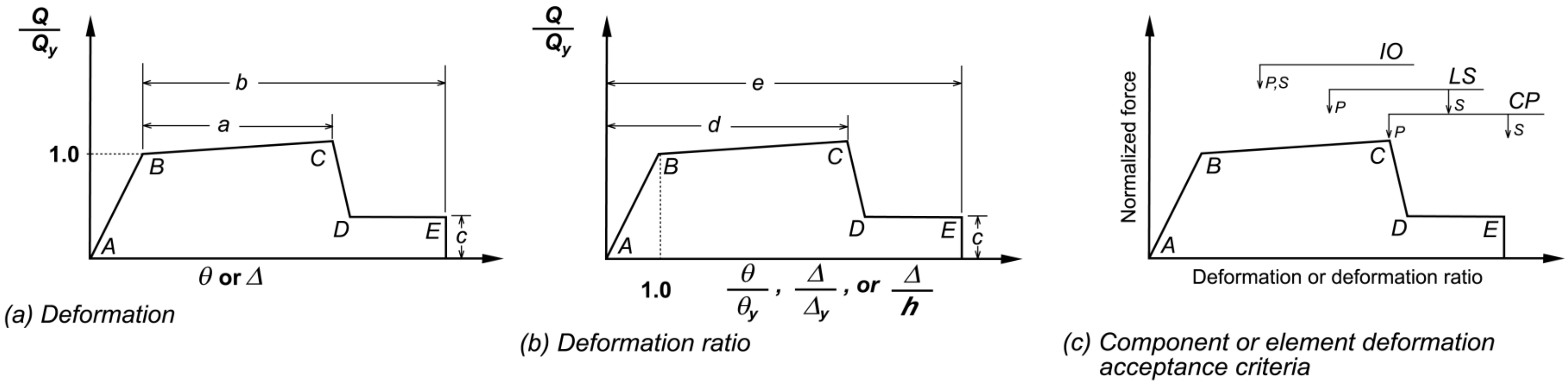
• In the absence of component testing justifying Type 1 or Type 2 behavior, all other actions are to be taken as force controlled.

Table C2-1 Examples of Possible Deformation-Controlled and Force-Controlled Actions

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V^1
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M^3	P, V, M
Diaphragms	M, V^2	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.
2. If the diaphragm carries lateral loads from vertical seismic resisting elements above the diaphragm level, then M and V shall be considered force-controlled actions.
3. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

Figure C2-1 shows the generalized force versus deformation curves used throughout this standard to specify component modeling and acceptance criteria for deformation-controlled actions in any of the four basic material types.



: Figure C2-1 Generalized Deformation-Controlled Component Force-Deformation Relations for Depicting Modeling and Acceptance Criteria

- Elastic stiffnesses and values for **the parameters *a, b, c, d,* and *e*** that can be used for modeling components are given in **Chapters 5 through 8**.

- **Acceptance criteria for deformation or deformation ratios** corresponding to the target Building Performance Levels of Collapse Prevention (CP), Life Safety (LS), and Immediate Occupancy (IO) as shown in Figure 2-1(c) **are given in Chapters 5 through 8**.

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components

Component/Action	Modeling Parameters: Plastic Rotation Angle, Radians		Modeling Parameters: Residual Strength Ratio	Acceptance Criteria: Plastic Rotation Angle, Radians				
					Primary		Secondary	
	a	b	c	IO	LS	CP	LS	CP
Beams—flexure								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							

"내진 검팩트 조건을 만족하는 강재보의 경우 (Deform. Controlled member로 분류됨)"

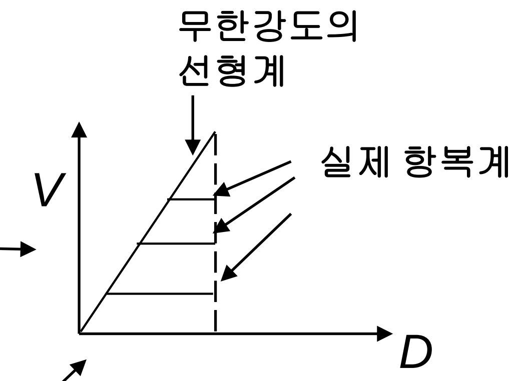
IV. Performance Evaluation of LDP (Linear Dynamic Procedure)

1. Basis of Analysis Procedure

Modal spectral analysis is carried out using linearly-elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacements that approximate maximum displacements expected during the design earthquake, but will produce internal forces that exceed those that would be obtained in a yielding building.

Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements. These design forces are evaluated through the acceptance criteria of Section 3.4.2, which include modification factors and alternative analysis procedures to account for anticipated inelastic response demands and capacities.

“R = 1”, 아예 R을 생각할 필요가 없음



Response Spectrum vs. Time History Method

3.3.2.2.3 Response Spectrum Method

Dynamic analysis using the response spectrum method shall calculate peak modal responses for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building. Modal damping ratios shall reflect the damping in the building at deformation levels less than the yield deformation.

Peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule.

Multidirectional seismic effects shall be considered in accordance with the requirements of Section 3.2.7.

3.3.2.2.4 Time History Method

Dynamic analysis using the time history method shall calculate building response at discrete time steps using discretized recorded or synthetic time histories as base motion. The damping matrix associated with the mathematical model shall reflect the damping in the building at deformation levels near the yield deformation.

Response parameters shall be calculated for each time history analysis. If three or more time history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more consistent pairs of horizontal ground motion records are used for time history analysis, use of the average of all responses of the parameter of interest shall be permitted for design.

Multidirectional seismic effects shall be considered in accordance with the requirements of Section 3.2.7. Alternatively, an analysis of a three-dimensional mathematical model using simultaneously imposed consistent pairs of earthquake ground motion records along each of the horizontal axes of the building shall be permitted.

Determination of Forces and Doformations

3.3.2.3.1 Modification of Demands

All forces and deformations calculated using either the Response Spectrum or Time History Analysis Methods shall be multiplied by the product of the modification factors C_1, C_2, and C_3 defined in Section 3.3.1.3, and further modified to consider the effects of torsion in accordance with Section 3.2.2.2.

"대부분 1.0일 것임"

2. Acceptance Criteria for LDP

Prior to selecting component acceptance criteria, actions shall be classified as deformation-controlled or force-controlled.

Computation of Action Demands

3.4.2.1.1 Deformation-Controlled Actions

Deformation-controlled design actions Q_{UD} shall be calculated in accordance with Equation (3-18):

$$Q_{UD} = Q_G \pm Q_E \qquad (3\text{-}18)$$

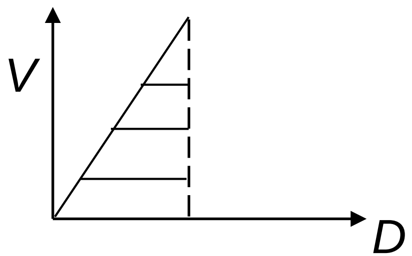
where:

Q_E = Action due to design earthquake loads calculated using forces and analysis models described in either Section 3.3.1 or Section 3.3.2

Q_G = Action due to design gravity loads as defined in Section 3.2.8

Q_{UD} = Deformation-controlled design action due to gravity loads and earthquake loads

Because of possible anticipated nonlinear response of the structure, the design actions as represented by Equation (3-18) may exceed the actual strength of the component or element to resist these actions. The acceptance criteria of Section 3.4.2.2.1 take this overload into account through use of a factor, *m*, which is an indirect measure of the nonlinear deformation capacity of the component or element.



"The concept of equivalent linear strength"

3.4.2.1.2 Force-Controlled Actions

Force-controlled design actions, Q_{UF}, shall be calculated using one of the following methods:

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering load to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.

2. Alternatively, Q_{UF} shall be calculated in accordance with Equation (3-19).

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \qquad (3\text{-}19)$$

The basic approach for calculating force-controlled actions for design differs from that used for deformation-controlled actions because nonlinear deformations associated with forced-controlled actions are not permitted. Therefore, force demands for force-controlled actions should not exceed the force capacity (strength).

Ideally, an inelastic mechanism for the structure will be identified, and the force-controlled actions, Q_{UF}, for design will be determined by limit analysis using that mechanism. This approach will always produce a conservative estimate of the design actions, even if an incorrect mechanism is selected. Where it is not possible to use limit (or plastic) analysis, or in cases where design forces do not produce significant nonlinear response in the building, it is acceptable to determine the force-controlled actions for design using Equation (3-19).

$$Q_{UF} = Q_G \pm \frac{Q_E}{\boxed{C_1 C_2 C_3} J} \qquad (3\text{-}19)$$

where:

Q_{UF} = Force-controlled design action due to gravity loads in combination with earthquake loads

J = Force-delivery reduction factor, greater than or equal to 1.0, taken as the smallest DCR of the components in the load path delivering force to the component in question, calculated in accordance with Equation (2-1).

Alternatively, values of J equal to 2.0 in Zones of High Seismicity, 1.5 in Zones of Moderate Seismicity, and 1.0 in Zones of Low Seismicity shall be permitted when not based on calculated DCRs. J shall be taken as 1.0 for the Immediate Occupancy Structural Performance Level. In any case where the forces contributing to Q_{UF} are delivered by components of the lateral force resisting system that remain elastic, J shall be taken as 1.0.

Coefficients C_1, C_2, and C_3 were introduced in Equation (3-10) to amplify the design base shear to achieve a better estimate of the maximum displacements expected for buildings responding in the inelastic range. Displacement amplifiers, C_1, C_2, and C_3 are divided out of Equation (3-19) when seeking an estimate of the force level present in a component when the building is responding inelastically.

~~Since J is included for force-controlled actions, it may appear to be more advantageous to treat an action as force-controlled when m-factors are less than J. However, proper application of force-controlled criteria requires a limit state analysis of demand and lower bound calculation of capacity that will yield a safe result whether an action is treated as force- or deformation-controlled.~~

“매우 정성적인 계수임!

Take J= 1”로 택하면 보수적이고 충분할 듯

Acceptance Criteria for "Deformation-Controlled" Actions

"Equivalent linear strength"

$$m\kappa Q_{CE} \geq Q_{UD} \qquad (3\text{-}20)$$

"내진 검팩트조건
을 만족하는 강재보
의 경우"

where:

m = Component or element demand modifier (factor) to account for expected ductility associated with this action at the selected Structural Performance Level. m-factors are specified in Chapters 4 through 8

Q_{CE} = Expected strength of the component or element at the deformation level under consideration for deformation-controlled actions

κ = Kappa= 1.0 for new construction

Q_{CE}, the expected strength, shall be determined considering all coexisting actions on the component under the design loading condition by procedures specified in Chapters 4 through 8.

ble 5-5 *Acceptance Criteria for Linear Procedures—Structural Steel Components*

mponent/Action	m-factors for Linear Procedures[1]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
ams – flexure					
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				

"가령 **P-M interaction**이 고려되어야 하는 경우 조합효과를 고려해야; 각 구조종별 산정법이 제시되어 있음 (통계자료가 있으면 **expected yield strength,** 재료의 초과강도 반영가능)

Acceptance Criteria for “Force-Controlled” Actions: 기본적으로 “탄성설계 할 것”

3.4.2.2.2 Force-Controlled Actions

Force-controlled actions in primary and secondary components and elements shall satisfy Equation (3-21):

$$\boxed{\kappa Q_{CL}} \geq Q_{UF} \qquad (3\text{-}21)$$

where: Kappa= 1.0 for new construction

Q_{CL} = Lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions

Q_{CL}, the lower-bound strength, shall be determined considering all coexisting actions on the component under the design loading condition by procedures specified in Chapters 5 through 8.

“가령, P-M interaction이 고려되어야 하는 조합 응력효과를 반영해야 **(**통계치가 없으면 **nominal yield strength** 사용하면 무난할 것임**)**

Note: Acceptance Criteria for "Nonlinear" Procedure: more straightforward!, *m factor not needed*.

3.4.3.2.1 Deformation-Controlled Actions
Primary components shall have expected deformation capacities not less than maximum deformation demands calculated at the target displacement.

3.4.3.2.3 Force-Controlled Actions
Primary components shall have lower-bound strengths not less than the maximum design forces. Lower-bound strengths shall be determined considering all coexisting forces and deformations by procedures specified in Chapters 4 through 8.

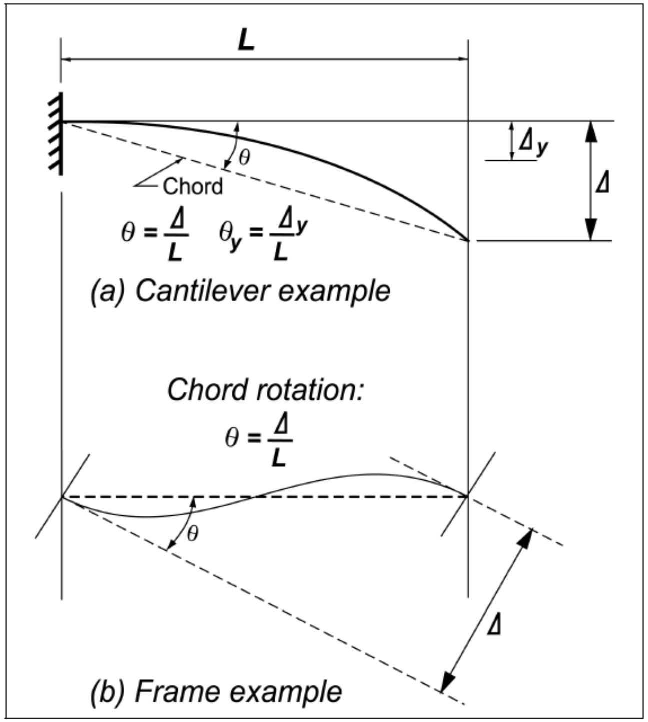


Figure 5-2 Definition of Chord Rotation

$$\text{Beams:} \quad \theta_y = \frac{ZF_{ye}l_b}{6EI_b} \qquad (5\text{-}1)$$

$$\text{Columns:} \quad \theta_y = \frac{ZF_{ye}l_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \qquad (5\text{-}2)$$

Table 5-6	*Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components*							
	Modeling Parameters			**Acceptance Criteria**				
				Plastic Rotation Angle, Radians				
	Plastic Rotation Angle, Radians		**Residual Strength Ratio**		**Primary**		**Secondary**	
Component/Action	**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
Beams—flexure								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							

"내진 검팩트 조건을 만족하는 강재보의 경우 (Deform. Controlled member로 분류됨)"

V. Acceptance Criteria for Steel Moment-Resisting Frames When Using LDP

1. Design Strengths

General

Classification of steel component actions as deformation- or force-controlled, and calculation of design strengths, shall be as specified in Sections 5.5:

Deformation-Controlled Actions

• Design strengths for deformation-controlled actions shall be taken as expected strengths obtained experimentally or calculated using accepted principles of mechanics.

• Expected strength shall be defined as the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected.

• When calculations are used to determine mean expected strength, expected material properties (including strain hardening) shall be used:

• Unless other procedures are specified in this standard, procedures contained in AISC-*LRFD Specifications* to calculate design strength shall be permitted, except that the strength reduction factor, φ, shall be taken as unity.

Force-Controlled Actions

• Design strengths for force-controlled actions shall be taken as lower-bound strengths obtained experimentally or calculated using established principles of mechanics.

• Lower-bound strength shall be defined as mean strength minus one standard deviation. When calculations are used to determine lower-bound strength, lower bound material properties shall be used (자료부재시는 공칭항복강도로서 충분할 듯)

• Unless other procedures are specified in this standard, procedures contained in AISC (1993) *LRFD Specifications* to calculate design strength shall be permitted, except that the strength reduction factor, φ, shall be taken as unity.

1. Beams: The strength of elements of structural steel under flexural actions with negligible axial load present shall be calculated in accordance with this section. These actions shall be considered deformation-controlled.

The expected flexural strength, Q_{CE}, of beam components shall be determined using equations for design strength, M_p, given in AISC (1997) *Seismic Provisions*, except that ϕ shall be taken as 1.0 and F_{ye} shall be substituted for F_y. The component expected strength, Q_{CE}, of beams and other flexural deformation-controlled members shall be the lowest value obtained for the limit states of yielding, lateral-torsional buckling, local flange buckling, or shear yielding of the web.

"Flexural Limit States for beam member: "PH, FLB, WLB, LTB"

For fully concrete-encased beams where confining reinforcement is provided to allow the concrete to remain in place during the earthquake, the values of $b_f = 0$ and $L_p = 0$ shall be permitted to be used. For bare beams bent about their major axes and symmetric about both axes, satisfying the requirements of compact sections, and $L_b < L_p$, Q_{CE} shall be computed in accordance with Equation (5-6):

$$Q_{CE} = M_{CE} = M_{pCE} = ZF_{ye} \qquad (5\text{-}6)$$

where:

b_f	=	Width of the compression flange
L_b	=	Length of beam
L_p	=	Limiting lateral unbraced length for full plastic bending capacity for uniform bending from AISC (1993) *LRFD Specifications*
M_{pCE}	=	Expected plastic moment capacity
F_{ye}	=	Expected yield strength determined in accordance with Section 5.3.2.3

If the beam strength is governed by the shear strength of the unstiffened web and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_y}}$, then V_{CE} shall be calculated in accordance with Equation (5-7):

$$Q_{CE} = V_{CE} = 0.6 F_{ye} A_w \qquad (5\text{-}7)$$

where:

V_{CE} = Expected shear strength

A_w = Nominal area of the web = $d_b t_w$

t_w = Web thickness

h = Distance from inside of compression flange to inside of tension flange

F_y = Yield strength; must be in ksi when used in Equation (5-7)

If $\frac{h}{t_w} > \frac{418}{\sqrt{F_y}}$, then the value of V_{CE} shall be calculated from AISC (1997) *Seismic Provisions*.

“보-기둥 부재”

2. Columns: This section shall be used to evaluate flexural and axial strengths of structural steel elements with non-negligible axial load present. These actions shall be considered force-controlled.

The lower-bound strength, Q_{CL}, of steel columns under axial compression shall be the lowest value obtained for the limit states of column buckling, local flange buckling, or local web buckling. The effective design strength or the lower-bound axial compressive strength, P_{CL}, shall be calculated in accordance with AISC (1997) *Seismic Provisions*, taking ϕ=1.0 and using the lower-bound strength, F_{yLB}, for yield strength.

The expected axial strength of a column in tension, Q_{CE}, shall be computed in accordance with Equation (5-8):

$$Q_{CE} = T_{CE} = A_c F_{ye} \qquad (5\text{-}8)$$

where:

A_c = Area of column

F_{ye} = Expected yield strength of column

T_{CE} = Expected tensile strength of column

3. Panel Zone: The strength of the panel zone shall be calculated using Equation (5-5).

$$Q_{CE} = V_{CE} = 0.55F_{ye}d_c t_p \qquad (5\text{-}5)$$

4. Connections ..

2. Acceptance Criteria

1. Beams: The acceptance criteria of this section shall apply to flexural actions of elements of structural steel with negligible axial load. Beam flexure and shear shall be considered deformation-controlled.

$$m\kappa Q_{CE} \geq Q_{UD} \qquad (3\text{-}20)$$

$$m_e = C_b\left[m - (m-1)\frac{L_b - L_p}{L_r - L_p}\right] \qquad \textbf{(5.9)}$$

Values for the *m*-factor used in Equation (3-20) shall be as specified in Table 5-5. *If $Q_{CE} < M_{pCE}$* due to lateral torsional buckling, then *m* in Equation (3-20) shall be replaced by *me*, calculated in accordance with Equation (5-9)

Table 5-5 Acceptance Criteria for Linear Procedures—Structural Steel Components

Component/Action	m-factors for Linear Procedures[1]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Beams – flexure					
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				

2. Columns (beam-column): For steel columns under combined axial compression and bending stress, where the axial column load is less than 50% of the lower-bound axial column strength, *PCL*, the column shall be **considered *deformation-controlled for flexural behavior* and *force controlled for compressive behavior*** and the combined strength shall be evaluated by Equation (5-10) or (5-11).

$$For\ 0.2 \le \frac{P_{UF}}{P_{CL}} \le 0.5$$

$$\frac{P_{UF}}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] \le 1.0 \qquad (5-10)$$

$$For\ \frac{P_{UF}}{P_{CL}} < 0.2$$

$$\frac{P_{UF}}{2P_{CL}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \le 1.0 \qquad (5-11)$$

* Steel columns with axial compressive forces exceeding 50% of the lower-bound axial compressive strength, *PCL*, shall be ***considered force-controlled for both axial loads and flexure*** and shall be evaluated using Equation (5-12):

$$\frac{P_{UF}}{P_{CL}}+\frac{Mu_{Fx}}{M_{CLx}}+\frac{Mu_{Fy}}{M_{CLy}}\leq 1 \qquad (5-12)$$

* Steel columns under axial tension shall be considered deformation-controlled and shall be evaluated using Equation (3-20).

* Steel columns under ***combined axial tension and bending stress*** shall be ***considered deformation-controlled*** and shall be evaluated using Equation (5-13):

$$\frac{T}{m_t T_{CE}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \leq 1 \qquad (5-13)$$

Steel Braced Frames (CBF/EBF), (Table 5-5, FEMA 356)

Table 5-5 Acceptance Criteria for Linear Procedures—Structural Steel Components (continued)

Component/Action	*m*-factors for Linear Procedures[1]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Braces in Compression (except EBF braces)					
a. Double angles buckling in-plane	1.25	6	8	7	9
b. Double angles buckling out-of-plane	1.25	5	7	6	8
c. W or I shape	1.25	6	8	6	8
d. Double channels buckling in-plane	1.25	6	8	7	9
e. Double channels buckling out-of-plane	1.25	5	7	6	8
f. Concrete-filled tubes	1.25	5	7	5	7
g. Rectangular cold-formed tubes					

Brief Review of the State of the Art US PBSD Practice

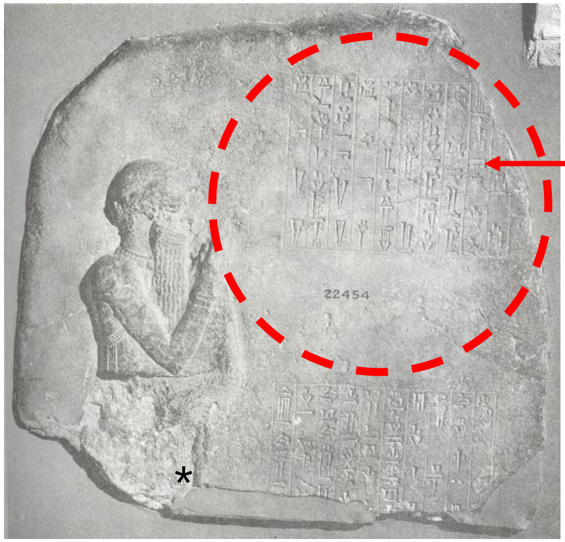
CHEOL HO LEE
Dept. of Arch and Arch Engrg, SNU

Outline

- ✓ 1. Introduction
- ✓ 2. Elements of PBSD
- ✓ 3. Some Notes on US PBSD procedure
- ✓ 4. Closing Remarks

✓1. Introduction

"The Code of Hammurabi : Bad Code versus 100% ideal PB Code? "



If a builder build a house for a man and do not make its construction firm and the house which he has built collapse and cause the death of the owner of the house – that builder shall be put to death;

"an eye for an eye and a son for a son (Lex Talionis)"

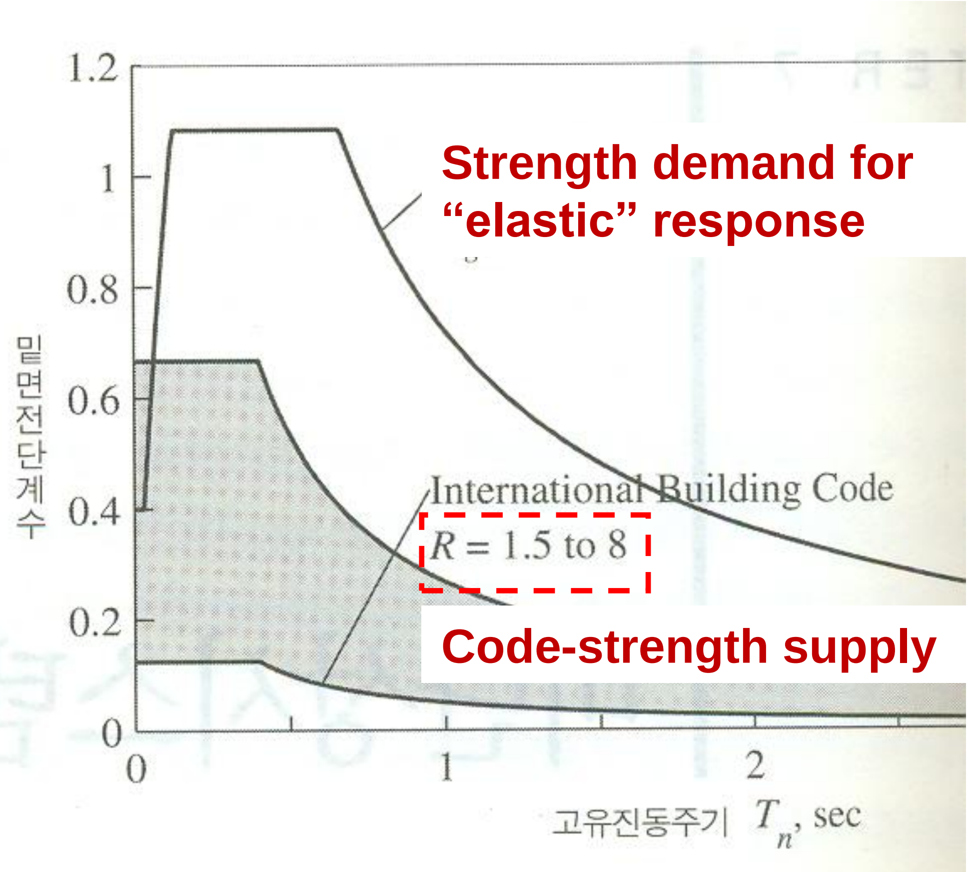
R Factor Approach: Pros and Cons

Europe: q factor
Japan: D factor

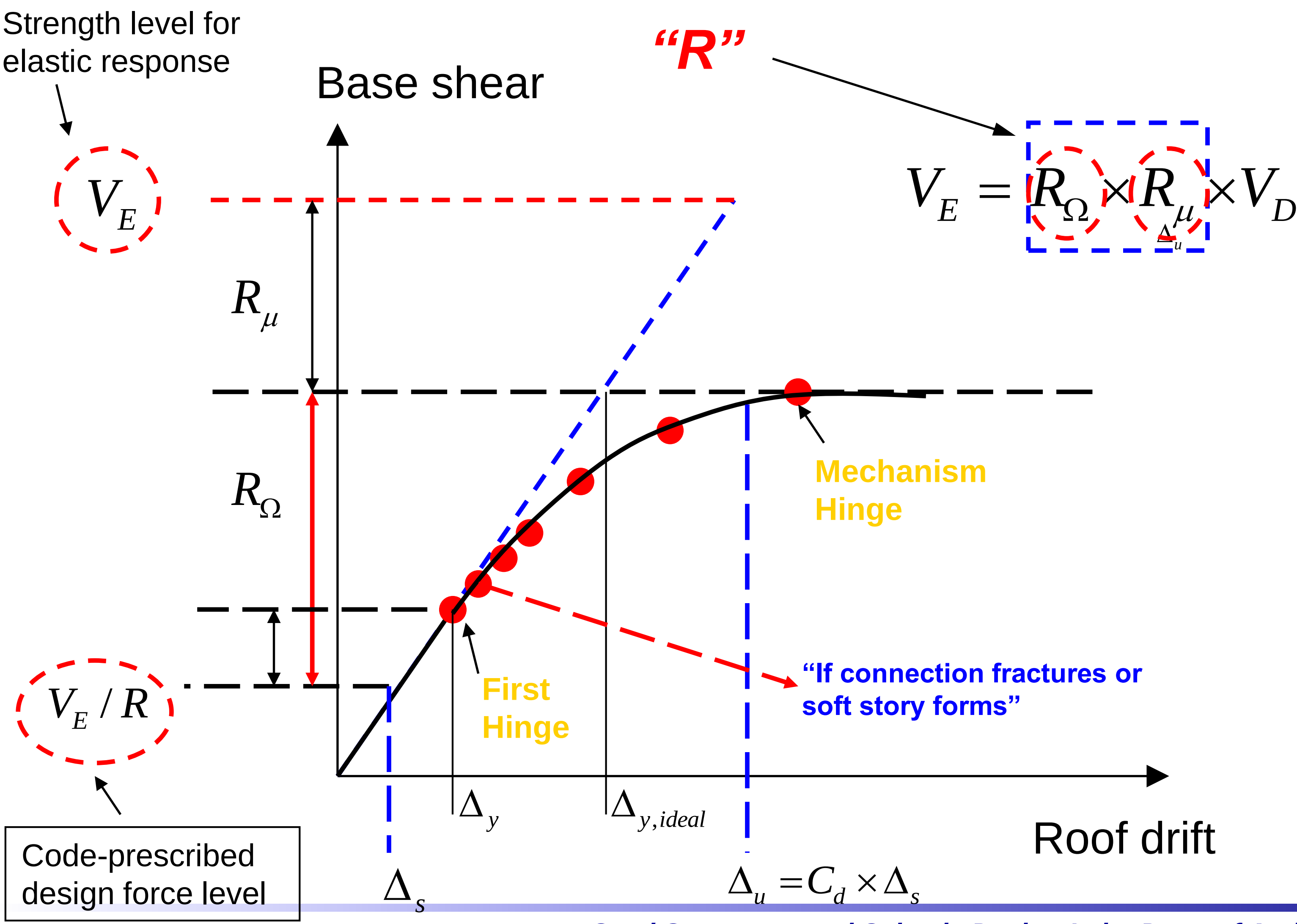
$$V\,(base\ shear) = \frac{IS \times "ACW"}{R}$$

R factor:

- Converted from former UBC empirical K factor,
- Committee consensus factor,
- Socio-economic factor,
- To specify design force level simplistically,
- Function of system ductility and overstrength.



R Factor Formula: Uang's Formula (1991)



Damage or Performance Control in Current Codes (e.g., ASCE 7, KBC 2016)

*** Indirect/Implicit Approach: Code tends to become more unclear and prescriptive**

건물용도/중요도
(Seismic Use Group)

부지의 지진위험도
(Seismic Risk)

Seismic Design Category (SDC)
A, B, C, D, E, F

Indirect damage/performance control

Year 2010_ after the 1994 Northridge EQ

EQE INTERNATIONAL
An ABS Group Company

ORMATION SERVICES

Performance-based Design

Building Codes May Not Provide Enough Protection

Building codes are not intended to protect property or business continuity.

Building codes are intended only to protect the public safety, by avoiding structural collapse. They are not intended to avoid damage to buildings or equipment, or provide for continuity of business operations. The U.S. Borax Corporate Headquarters complex in Santa Clarita, California, was constructed just two years before the Northridge earthquake and fully conformed to the applicable codes. Damage to the buildings was so severe that

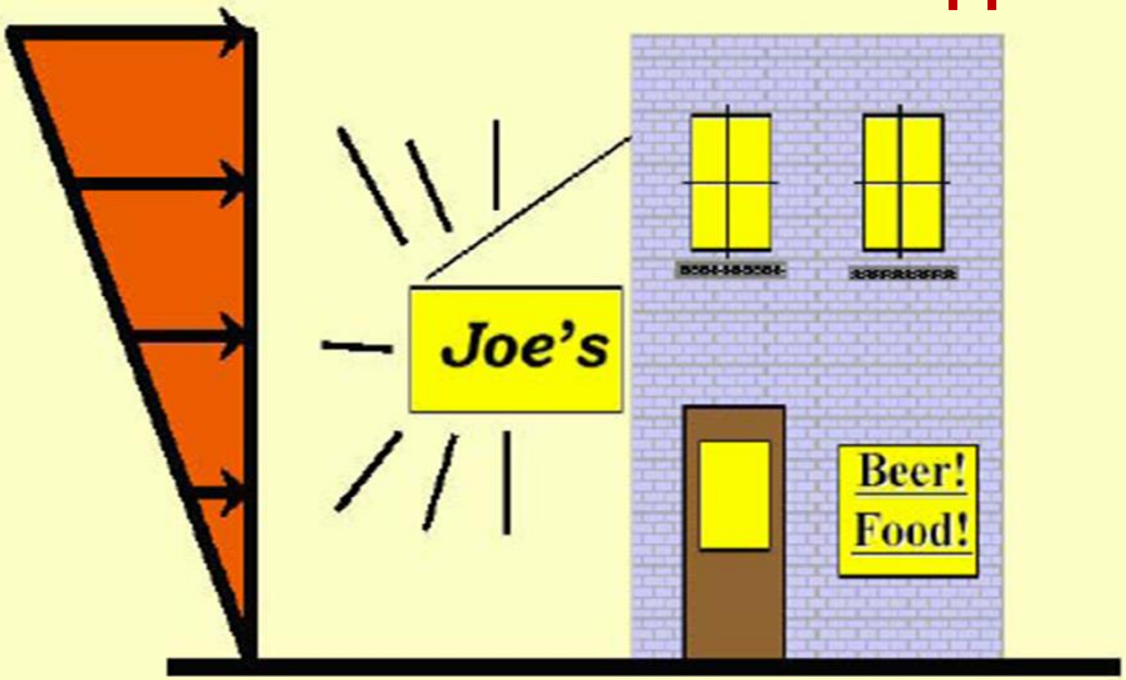
The U.S. Borax Building following the Northridge earthquake

"Highrise buildings=> often Undefined Seismic Load Resisting System *R*= ?

San Francisco's 57-story One Rincon Hill (right) and the two towers of The Infinity (left, by crane), **designed using a performance-based approach**

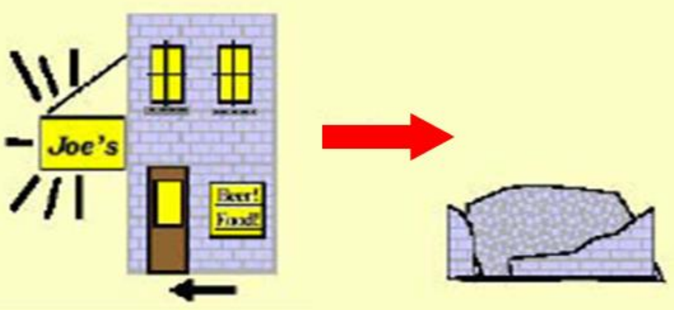
Traditional Approach

"R factor approach"



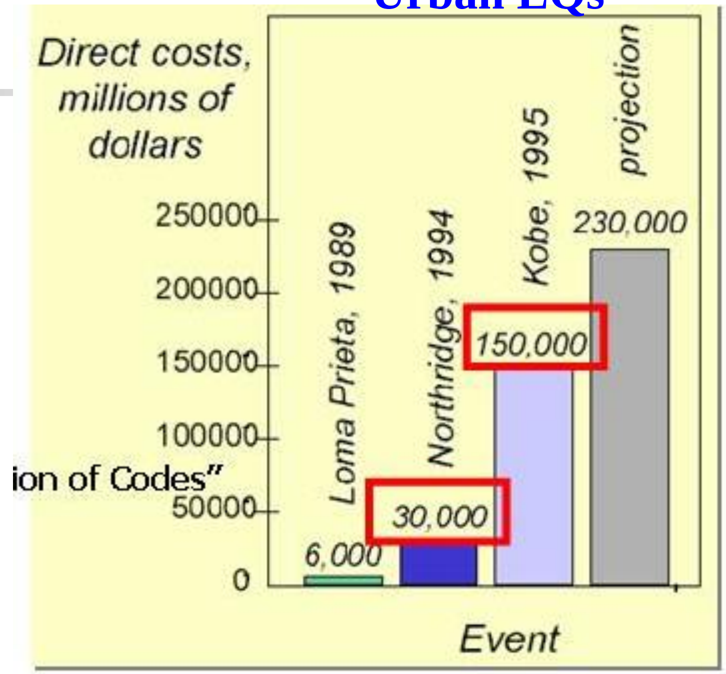
- Linear analysis model
- Simplified design base shear
- Owners informed of code conformance, but not building performance

$$V = \frac{ZICS}{R}W$$

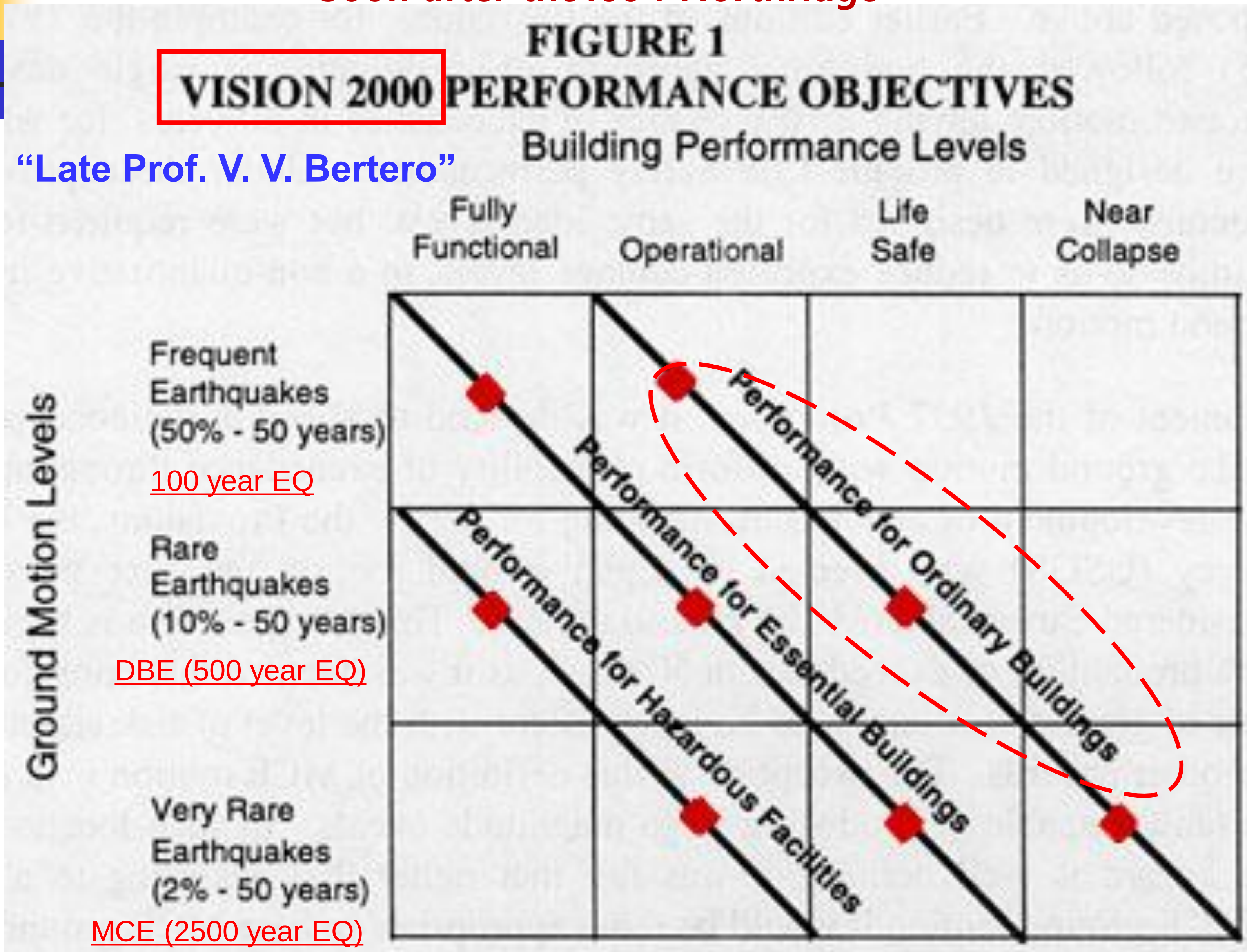


* PBSD: Appealed after the1994 Northridge/1995 Kobe earthquakes

* Strl engineers satisfied/ other stakeholders unsatisfied after the1994 Northridge
- Reply from Calif. EQ engrg. community



Soon after the1994 Northridge

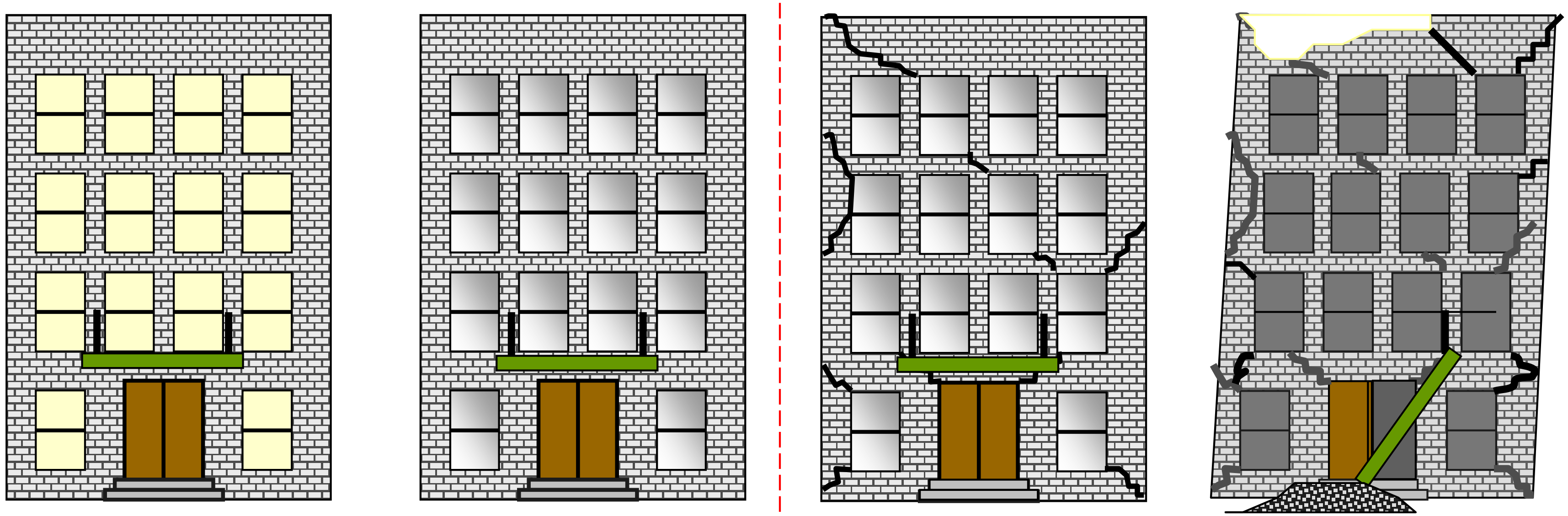


SEAOC Performance Objectives in 1960

Note: Very well-known seismic performance level implied in SEAOC Blue Book (1960)

- ✓ **Qualitative performance statements**
- ✓ **Resist**
  - ✓ minor levels of earthquake shaking without damage_ SLE
  - ✓ moderate levels of earthquake shaking without structural damage
  - ✓ major levels of earthquake shaking with structural and nonstructural damage but protect life safety_ DBE
  - ✓ the most severe levels of earthquake shaking ever anticipated without collapse_ MCE

Standard Performance Level



기능수행

Operational

70-year EQ
(minor)

즉시입주

Immediate Occupancy

200-year EQ
(moderate)

인명안전

Life Safety

DBE
(설계지진)
500~1000-year EQ
(major)

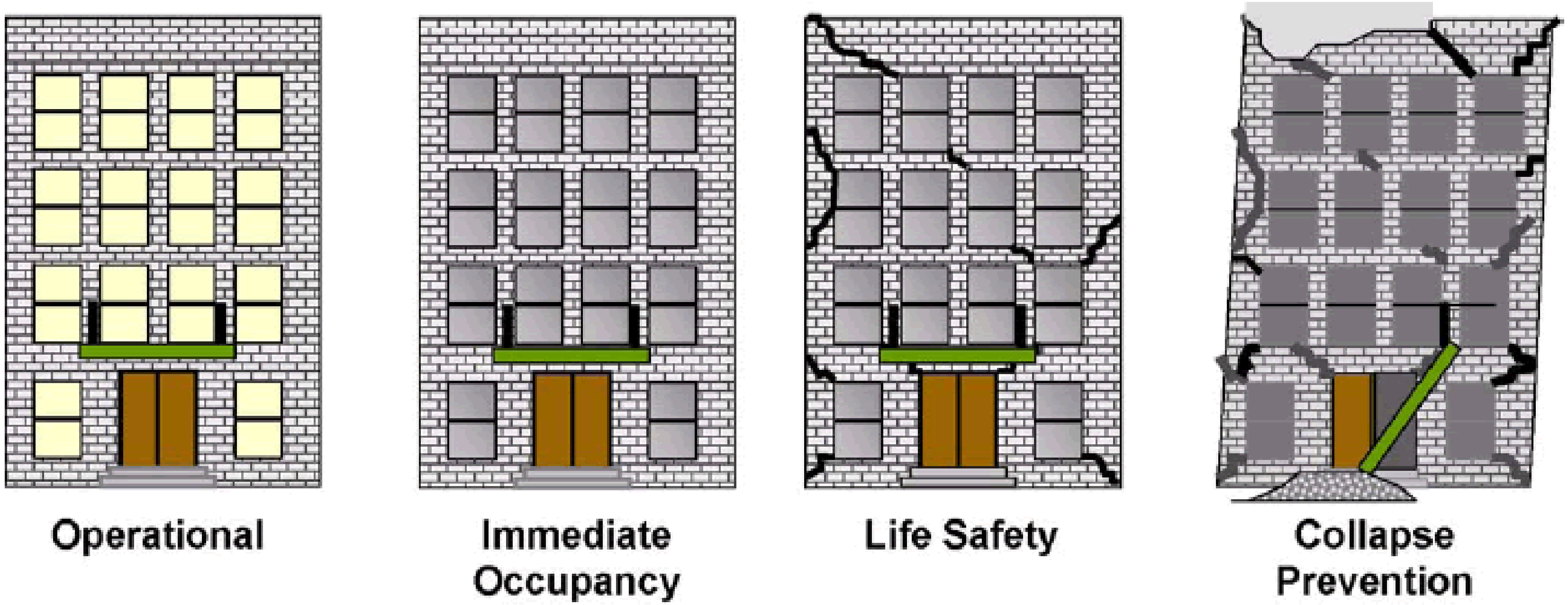
붕괴방지

Collapse Prevention

MCE
(최대한도지진)
2500 year EQ
(the most severe)

일반건물의 **BSO (Basic Safety Objective)** 로 희망하고 있으나 현행 설계코드의 방법으론 확인이 불가 **(확인하지 않아도 됨)**

More Detailed Damage Description



Operational

The lowest level of overall damage to the building.
The structure will retain nearly all of its pre-earthquake strength and stiffness.

Immediate Occupancy

Overall damage to the building is light.
Damage to the structural systems is similar to the Operational Performance Level;
However, repair and cleanup may be needed.
Many building owners may wish to achieve this level of performance when the building is subjected to moderate levels of earthquake.

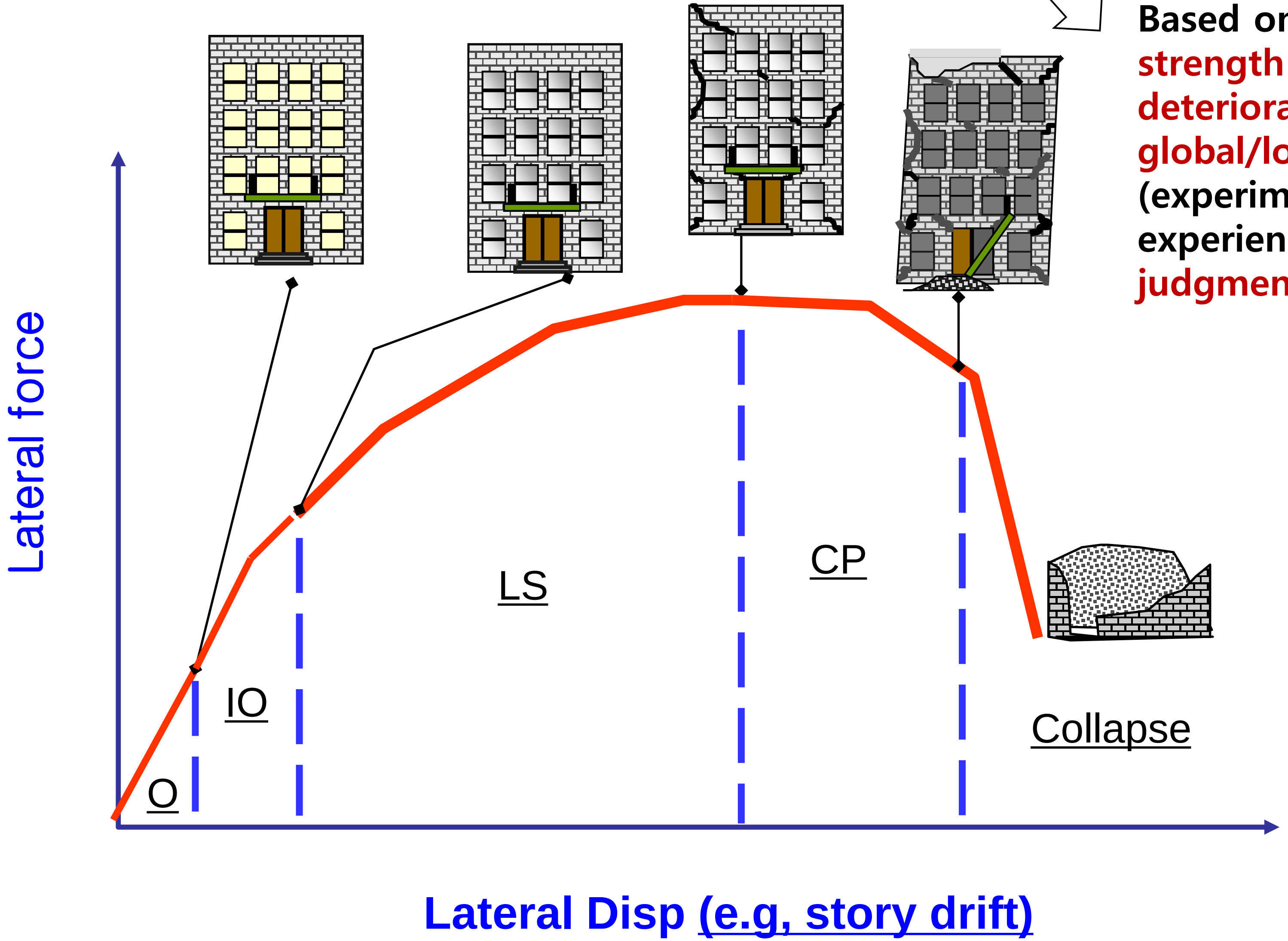
Life Safety

Structural and nonstructural damage is significant.
Buildings designed to meet the life safety performance level may not be safe for continued occupancy until repairs are done.

Collapse Prevention

The structure sustains severe damage. The lateral-force resisting system loses most of its pre-earthquake strength and stiffness.
Load-bearing columns and walls function, but the building is near collapse.

Graphical Illustration of "Acceptance Limits"

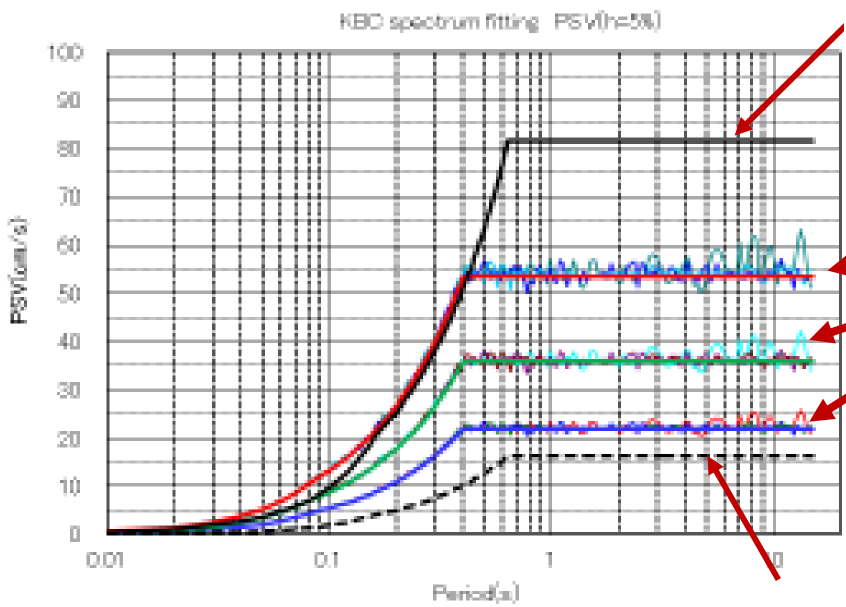


Based on cyclic strength and stiffness deterioration at global/local level (experimental/past experience; often judgmental)

Japanese Dual Spectrum Approach_ instrumental

미국식 성능기반설계의 선구?

PSV spectrum_ soft rock site



Japanese 500yr EQ (for ultimate strength)

Korea:
13,000yr EQ
5,000yr EQ
1500yr EQ

Japanese 50yr EQ (for serviceability)

2. Elements of PBSD Procedure

(1) Step 1: Establish multiple seismic hazards-multiple seismic performance objectives

(2) Step 2: Predict seismic demands through structural analysis

(3) Step 3: Evaluate performance based on acceptance criteria and iterate until satisfaction

Much of the framework for PBSD in the USA…

"Late Prof. V. V. Bertero"

1. **SEAOC (1995). *Vision 2000*: Performance Based Seismic Engineering of Buildings, SEAOC:** 최초로 다양한 포맷의 성능기반설계개념을 포괄적으로 취급

2. **ATC 40 (1996). Seismic Evaluation and Retrofit of Concrete Buildings, Report SSC 96-01, CSSC, ATC:** 기존 콘크리트 구조물의 내진성능평가 및 보수/보강법 취급

3. **FEMA 356 (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, FEMA:** 다양한 구조형식의 내진성능평가 및 보수/보강법을 포괄적으로 취급

4. **ASCE/SEI 41-13 (2013): Seismic Evaluation and Retrofit of Existing buildings.**

Much of the framework for performance-based design in the USA can be traced to Vision 2000 (SEAOC, 1995), ATC 40 (ATC, 1996) and FEMA 356 (FEMA, 2000).

More recently, guidelines for performance base seismic design of high-rise buildings have been issued by regulatory bodies in Los Angeles (LATBSDC, 2015) and San Francisco (SEAONC, 20??).

고층건물 수요 많은 도시_"Tall Buildings-Specific PBSD"

SOM Design Reference for GBC

- PEER/ATC 72-1 (2010). Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings

- PEER (2017). Tall Buildings Initiative: "Guidelines for Based Seismic Design of Tall Building, Version 2.01"

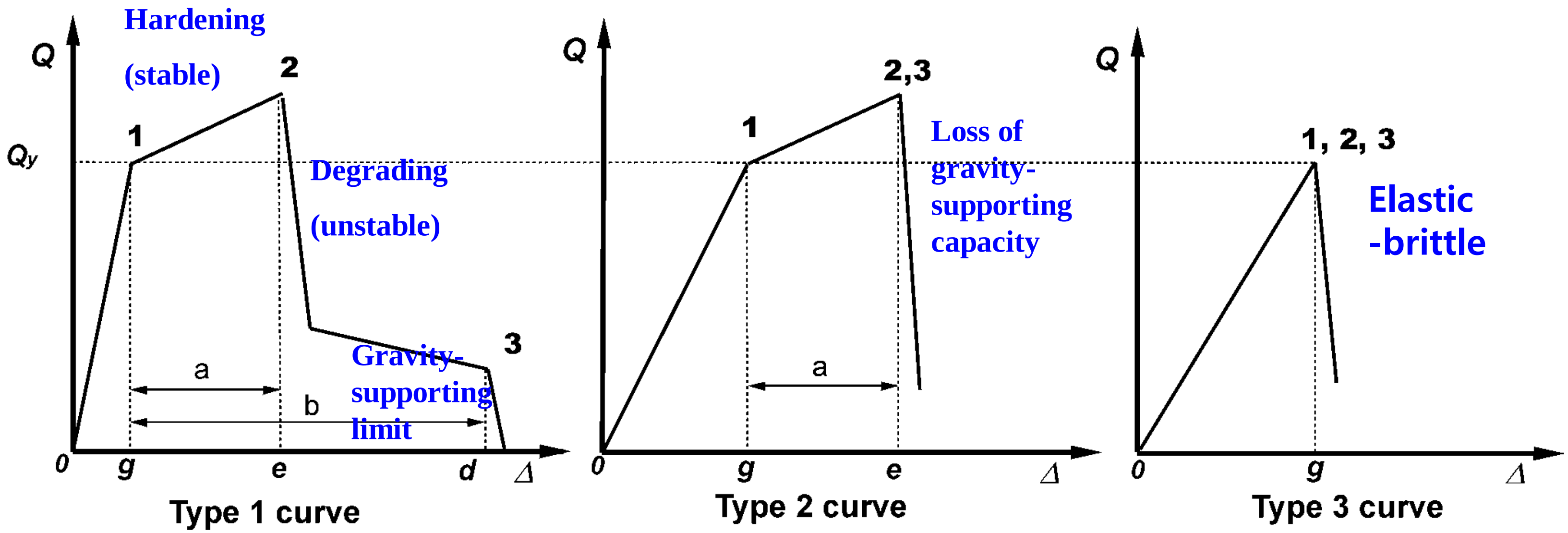
Recent Tall Buildings-Specific PBSD Documents

Deformation-Controlled vs. Force-Controlled Actions (per ASCE 41-13)

Often referred by PEER (2017)

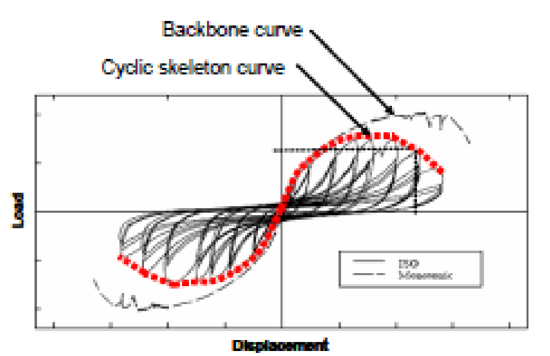
"Deformation-controlled if $e > 2g$"

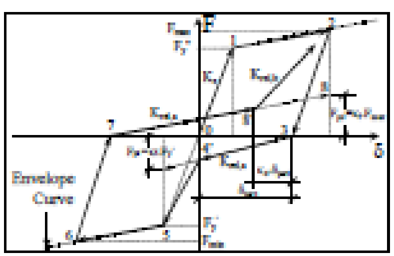
"Force-controlled"



: Figure 2-3 Component Force Versus Deformation Curves

"Should be based on cyclic envelope curve"





- **Classification of force- or deformation-controlled actions are specified for framing components in Chapters 5 through 8.**

- ***Classification as a deformation-controlled action is not up to the discretion of the user:*** 사용자의 입장에서 고민할 필요 없음

- **A given component may have a combination of both force- and deformation-controlled actions.**

- **Deformation-controlled actions have been defined in this standard by the designation of *m*-factors or nonlinear deformation capacities in Chapters 5 through 8.**

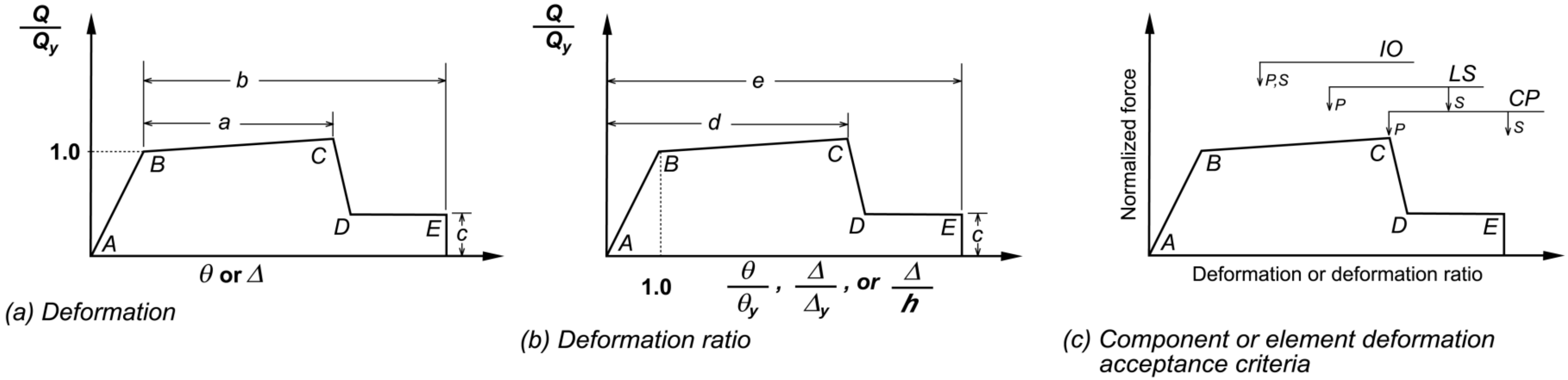
Table C2-1 Examples of Possible Deformation-Controlled and Force-Controlled Actions

Component	Deformation-Controlled Action	Force-Controlled Action
Moment Frames • Beams • Columns • Joints	 Moment (M) M --	 Shear (V) Axial load (P), V V^1
Shear Walls	M, V	P
Braced Frames • Braces • Beams • Columns • Shear Link	 P -- -- V	 -- P P P, M
Connections	P, V, M^3	P, V, M
Diaphragms	M, V^2	P, V, M

1. Shear may be a deformation-controlled action in steel moment frame construction.
2. If the diaphragm carries lateral loads from vertical seismic resisting elements above the diaphragm level, then M and V shall be considered force-controlled actions.
3. Axial, shear, and moment may be deformation-controlled actions for certain steel and wood connections.

- **Limits for each performance level_ largely judgmental, not a matured science**

- **Example_ *LS= 0.70*CP, IO= 0.50*LS***



: Figure C2-1 Generalized Deformation-Controlled Component Force-Deformation Relations for Depicting Modeling and Acceptance Criteria

PEER (2017). Tall Buildings Initiative: "Guidelines for Based Seismic Design of Tall Building, Version 2.01"

"Developed in Southern Calif. and should be adjusted if needed for GBC project with low to moderate seismicity'

Action – A strain, displacement, rotation or other deformation resulting from the application of design loads.

Deformation-controlled action – An action expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated for its ability to sustain such behavior.

Force-controlled action – An action that is not expected to undergo nonlinear behavior in response to earthquake shaking, and which is evaluated on the basis of its available strength.

Critical action – A force-controlled action, the failure of which is likely to lead to partial or total structural collapse.

Ordinary action – A force-controlled action, the failure of which might lead to local collapse comprising not more than one bay in a single story.

Noncritical action – A force-controlled action, the failure of which is unlikely to lead to structural collapse.

Capacity Design – A design approach wherein the structure is configured and proportioned to restrict yielding and inelastic behavior to specific deformation-controlled actions for which structural detailing enables reliable inelastic response without critical strength decay, and which, through their plastic response, limit the demands on other portions of the structure such that those other parts can be designed with sufficient strength to reliably remain essentially elastic.

2.2.5 Conceptual Design

Select the structural systems and materials; their approximate configuration, proportions and strengths; and the intended primary mechanisms of inelastic behavior. **Apply capacity design principles to establish the target inelastic mechanisms**.

For all members of the structural system, **define deformation-controlled actions and force-controlled actions. Categorize each forced-controlled action as being Critical, Ordinary, or Noncritical.**

Commentary: **The Engineer of Record** is to identify deformation-controlled actions and force-controlled actions, and is to categorize force-controlled actions as being Critical, Ordinary, or Noncritical, **subject to approval by the peer review.**

Appendix E provides a list of typical force-controlled actions and recommended categories. Individual design and peer review teams should consider this list when formulating the categorization of component actions for specific projects and supplement and **modify as is appropriate to those projects.**

Seems more reasonable than "automatic or mechanical" ASCE 41-13

Appendix E Typical Force-Controlled Actions and Categories

In these Guidelines, member actions are classified as either deformation-controlled or force-controlled. In addition, Chapter 6 requires force-controlled actions to be further classified in different categories of criticality. Table E-1 identifies typical force-controlled actions and categories.

Seems more reasonable than "automatic or mechanical" ASCE 41-13

Table E-1 Force-controlled actions and categories

	Action	Category: Critical	Ordinary	Non-critical
Structural Steel	Connections of braces to beams, columns and walls	X		
	Axial demand on braces in Eccentric Braced Frames	X		
	Column splice forces	X		
	Axial loads on column	X		
	Moments and shears on moment connections	X		
	Compression on vertical boundary elements of steel plate shear walls	X		
	Compression on horizontal boundary elements of steel plate shear walls			
	Forces in members of transfer trusses	X		
	All other force-controlled actions[2]	X		
Reinforced Concrete	Shear in beams, columns, and beam-column joints of special moment frames	X		
	Shear in columns not part of special moment frames	X		
	Axial load in columns of intentional outrigger systems, or in columns supporting discontinuous vertical elements	X		
	Combined moment and axial load in gravity columns[3]	X		
	Shear and moment in transfer girders	X		
	Shear in structural walls that are part of the primary lateral-force-resisting system	X		
	Shear and moment in basement walls			
	Shear in coupling beams without special diagonal reinforcing[4]	X		
	Compression on struts in strut and tie formulations	X		
	Tension on struts in strut and tie formulations		X	
	In-plane shear in transfer diaphragms[5]	X		
	In-plane shear in other diaphragms		X	

Action	Category: Critical	Ordinary	Non-critical
Force transfer from diaphragms to vertical elements of the seismic-force-resisting system, including collector forces and shear-friction between diaphragms and vertical elements	X		
In-plane normal forces in diaphragms other than collectors[6]		X	
Shear in shallow foundation elements, including spread footings and mat foundations	X		
Moment in shallow foundation elements, including spread footings and mat foundations		X	
All other force-controlled actions[2]	X		

[1]Structural steel elements designed and detailed to conform to the prescriptive requirements of AISC 341 and AISC 358 need not be evaluated in accordance with the criteria for force-controlled elements.
[2]Other force-controlled items should be categorized considering the criticality of the action to the overall building performance. The default category is shown as Critical.
[3]As an alternative, column flexure combined with axial force can be modeled as a deformation-controlled action if appropriately detailed.
[4]Coupling beam shear may be considered an ordinary action only if the consequence of element failure is minimal.
[5]Where walls beneath transfer diaphragms are adequate to provide required lateral force resistance in the event of diaphragm failure, transfer diaphragms may be treated as ordinary force-controlled actions.
[6]Diaphragm chord forces fall into this category.

PEER (2017). Tall Buildings Initiative (2017): "Guidelines for Based Seismic Design of Tall Building, Version 2.01"

"Developed in Southern Calif. and should be adjusted if needed for GBC project with low to moderate seismicity'

ASCE 7-16: loading code

ASCE 7-16: loading code

Do not use component models that do not account for post-peak strength deterioration or for cyclic deterioration for nonlinear analysis.

ASCE 41 may be referred for nonlinear modeling

Table 4-3 Reinforced concrete effective stiffness values.						
Component	Service-Level Linear Models			MCE_R-Level Nonlinear Models		
	Axial	Flexural	Shear	Axial	Flexural	Shear
Structural walls[1] (in-plane)	$1.0E_cA_g$	$0.75E_cI_g$	$0.4E_cA_g$	$1.0E_cA_g$	$0.35E_cI_g$	$0.2E_cA_g$
Structural walls (out-of-plane)	--	$0.25E_cI_g$	--	--	$0.25E_cI_g$	--

Central tendency analysis (MLE)

Refer to ASCE 41/ AISC 341 and..

Refer to ATC 72 (2010), ASCE 41…

"Very brief"

Calculated story drift shall not exceed 0.5% of story height in any story

LINEAR ANAL:
When response spectrum or linear response history analysis is used for the SLE evaluation, calculated demand-to-capacity ratios for deformation-controlled actions shall not exceed 1.5.

Calculated demand-to-capacity ratios for force-controlled actions shall not exceed 1.0.

NONLINEAR ANAL:
* Deformation-Controlled Actions: Immediate Occupancy performance as contained in ASCE 41
* Force-Controlled Actions: calculated force-controlled actions shall not exceed expected strengths

Only NDP permitted

At Global Level

- No unacceptable response
- Peak Transient Story Drift: In each story, the mean of the absolute values of the peak transient story drift ratios from each suite or set of analyses shall not exceed 0.03.
- Residual Story Drift: n each story, the mean of the absolute values of residual drift ratios from the suite of analyses shall not exceed 0.01.

At Element Level

- Strength demands on force-controlled actions or elements are sufficiently smaller than the expected strength capacities such that the probability of failure is acceptably small;
- Deformation demands on deformation-controlled (ductile) actions or elements are within deformation limits that have been verified by testing as being sustainable without critical strength loss.

3. Some Notes on US PBSD procedure

Selection of Performance Objective

Basic Safey Objective (BSO)

		Target Building Performance Levels*			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level (ground motions having a specified probability of being exceeded in a 50-year period)	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (10%/50 year)	i	j	k	l
	BSE-2 (2%/50 year)	m	n	o	p

*Alpha-numeric identifiers in parentheses defined in Table 4-2

Notes:

1. Each cell in the above matrix represents a discrete Rehabilitation Objective
2. Three specific Rehabilitation Objectives are defined in FEMA 356:

Basic Safety Objective	= cells k + p
Enhanced Objectives	= cells k + p + any of a, e, i, b, f, j, or n
Limited Objectives	= cell k alone, or cell p alone
Limited Objectives	= cells c, g, d, h, l

Enhanced Objectives

		Target Building Performance Levels*			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level (ground motions having a specified probability of being exceeded in a 50-year period)	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (10%/50 year)	i	j	k	l
	BSE-2 (2%/50 year)	m	n	o	p

*Alpha-numeric identifiers in parentheses defined in Table 4-2

Notes:

1. Each cell in the above matrix represents a discrete Rehabilitation Objective
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   Basic Safety Objective = cells k + p
   Enhanced Objectives = cells k + p + any of a, e, i, b, f, j, or n
   Limited Objectives = cell k alone, or cell p alone
   Limited Objectives = cells c, g, d, h, l

Analysis Procedures

	Tall, irregular buildings	Modeling	Seismic input	Advantage	Disadvantage
Linear Static Analysis (LSP)	No	Equivalent SDOF structural model	Response spectra	Very simple to analyze	Conservative/ Limitation of applicablity
Linear Dynamic Analysis (LDP)	yes	MDOF model	Response spectra / Ground-motion record	Compared to linear static procedures, higher modes can be considered	Applicability decreases with increasing nonlinear behavour
Nonlinear Static Analysis (NSP)	Not very accurate	Equivalent SDOF structural model	Response spectra	Accounts for the non-linear behavior/ The ductility of the structure can be evaluated	Never be as accurate as Nonlinear Dynamic Analysis
Nonlinear Dynamic Analysis (NDP)	Yes	Detailed structural model	Ground-motion record	The most accurate method	Very complicated and time consuming / Calculated response can be very sensitive to the characteristics of specific ground motion

Acceptance Criteria

The current thinking is that the performance of a component in the system is critical to the overall seismic performance of the building. Consequently, acceptance criteria are specified at the element level.

But global level criteria is useful and should be satisfied as well.

SERVICE-LEVEL EVALUATION .

5.6 Global Acceptance Criteria

5.6.1 Story Drift Limit

5.7 Component Acceptance Criteria–Linear Analysis

5.7.1 Deformation-Controlled Actions

5.7.2 Force-Controlled Actions

5.8 Component Acceptance Criteria– Nonlinear Analysis

5.8.1 Deformation-Controlled Actions

5.8.2 Force-Controlled Actions

MCE$_R$ EVALUATION .

6.7 Global Acceptance Evaluation

6.7.1 Unacceptable Response

6.7.2 Peak Transient Story Drift

6.7.3 Residual Story Drift

6.8 Component Acceptance Criteria

6.8.1 General

6.8.2 Deformation-Controlled Actions

6.8.3 Force-Controlled Actions

- **Relation of Structural (global) performance levels to the limiting damaging states as reported in FEMA 356.**

Table C1-3 Structural Performance Levels and Damage[1, 2, 3]—Vertical Elements

Elements	Type	Structural Performance Levels: Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks <1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced Steel Frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Many braces yield or buckle but do not totally fail. Many connections may fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary.	Same as primary.	Same as primary.
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent

"Rough guide"

Performance Evaluation by LDP (Linear Dynamic Procedure) permitted for SLE/ESE

3.3.2.2.3 Response Spectrum Method

Dynamic analysis using the response spectrum method shall calculate peak modal responses for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building. Modal damping ratios shall reflect the damping in the building at deformation levels less than the yield deformation.

Peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule.

Multidirectional seismic effects shall be considered in accordance with the requirements of Section 3.2.7.

"Developed in Southern Calif. and should be adjusted if needed for GBC project with low to moderate seismicity'

3.3.2.2.4 Time History Method

Dynamic analysis using the time history method shall calculate building response at discrete time steps using discretized recorded or synthetic time histories as base motion. The damping matrix associated with the mathematical model shall reflect the damping in the building at deformation levels near the yield deformation.

The signs of response quantities preserved !!

Response parameters shall be calculated for each time history analysis. If three or more time history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more consistent pairs of horizontal ground motion records are used for time history analysis, use of the average of all responses of the parameter of interest shall be permitted for design.

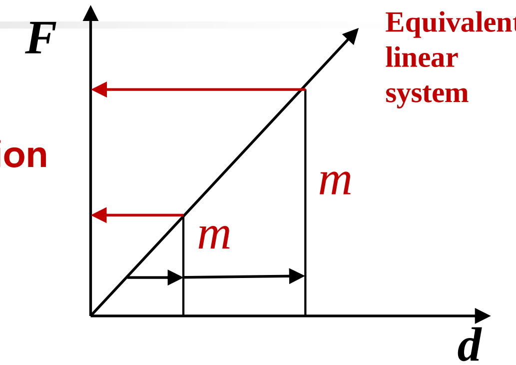
Multidirectional seismic effects shall be considered in accordance with the requirements of Section 3.2.7. Alternatively, an analysis of a three-dimensional mathematical model using simultaneously imposed consistent pairs of earthquake ground motion records along each of the horizontal axes of the building shall be permitted.

LDP Acceptance Criteria for "Deformation-Controlled" Actions

"Equivalent linear strength"

$$m\kappa Q_{CE} \geq Q_{UD} \qquad (3\text{-}20)$$

Recall "equal displacement assumption extended to member level"



where:

m = Component or element demand modifier (factor) to account for expected ductility associated with this action at the selected Structural Performance Level. m-factors are specified in Chapters 4 through 8

Q_{CE} = Expected strength of the component or element at the deformation level under consideration for deformation-controlled actions

κ = Kappa= 1.0 for new construction

Q_{CE}, the expected strength, shall be determined considering all coexisting actions on the component under the design loading condition by procedures specified in Chapters 4 through 8.

le 5-5 *Acceptance Criteria for Linear Procedures—Structural Steel Components*

nponent/Action	m-factors for Linear Procedures[1]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
ms – flexure					
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	2	6	8	"내진 컴팩트조건을 만족하는 강재보의 경우"	
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				

"가령 **P-M interaction**이 고려되어야 하는 경우 조합효과를 고려해야; 각 구조종별 산정법이 제시되어 있음 (통계자료가 있으면 **expected yield strength**, 재료의 초과강도 반영가능)"

LDP Acceptance Criteria for "Force-Controlled" Actions: 기본적으로 "탄성설계할 것", *m= 1*

3.4.2.2.2 Force-Controlled Actions

Force-controlled actions in primary and secondary components and elements shall satisfy Equation (3-21):

$$\boxed{\kappa Q_{CL}} \geq Q_{UF} \qquad (3\text{-}21)$$

where: Kappa= 1.0 for new construction

Q_{CL} = Lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions

Q_{CL}, the lower-bound strength, shall be determined considering all coexisting actions on the component under the design loading condition by procedures specified in Chapters 5 through 8.

"가령, **P-M interaction**이 고려되어야 하는 조합 응력효과를 반영해야 **(**통계치가 없으면 **nominal yield strength** 사용하면 무난할 것임**)**

LDP Acceptance Criteria for "Beam-Column" Member (mega brace?)

Beam-column member: For steel columns under combined axial compression and bending stress, where the axial column load is **less than 50% of the lower-bound axial column strength**, P_{CL}, the column shall be **considered** ***deformation-controlled for flexural behavior*** **and** ***force controlled for compressive behavior*** and the combined strength shall be evaluated by Equation (5-10) or (5-11).

$$For\ 0.2 \le \frac{P_{UF}}{P_{CL}} \le 0.5$$

$$\frac{P_{UF}}{P_{CL}} + \frac{8}{9}\left[\frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}}\right] \le 1.0 \qquad (5-10)$$

$$For\ \frac{P_{UF}}{P_{CL}} < 0.2$$

$$\frac{P_{UF}}{2P_{CL}} + \frac{M_x}{m_x M_{CEx}} + \frac{M_y}{m_y M_{CEy}} \le 1.0 \qquad (5-11)$$

* Steel columns with axial compressive forces exceeding 50% of the lower-bound axial compressive strength, P_{CL}, shall be *considered* ***force-controlled for "both" axial loads and flexure*** and shall be evaluated using Equation (5-12):

$$\frac{P_{UF}}{P_{CL}} + \frac{Mu_{Fx}}{M_{CLx}} + \frac{Mu_{Fy}}{M_{CLy}} \leq 1 \qquad (5-12)$$

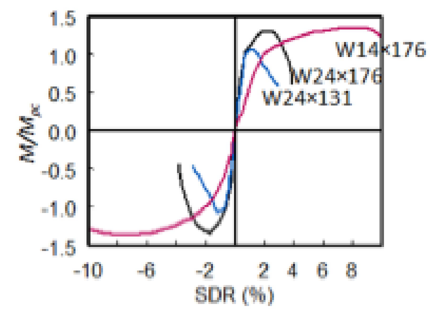
Figure 1. Shallow vs. Deep Wide-flange Sections

Classifying Cyclic Buckling Modes of Steel Wide-Flange Columns under Cyclic Loading

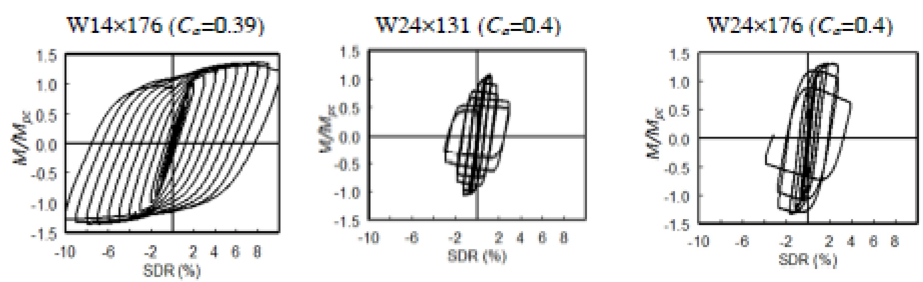
Gulen Ozkula[1], John Harris[2], Chia-Ming Uang[3]

[1] Graduate Student Researcher, University of California, San Diego; email: gozkula@ucsd.edu
[2] Research Structural Engineer, National Institute of Standards and Technology; email: john.harris@nist.gov
[3] Professor, University of California, San Diego, email; cmu@ucsd.edu

2017 ASCE Structural Congress Paper



(c) Backbone Comparison

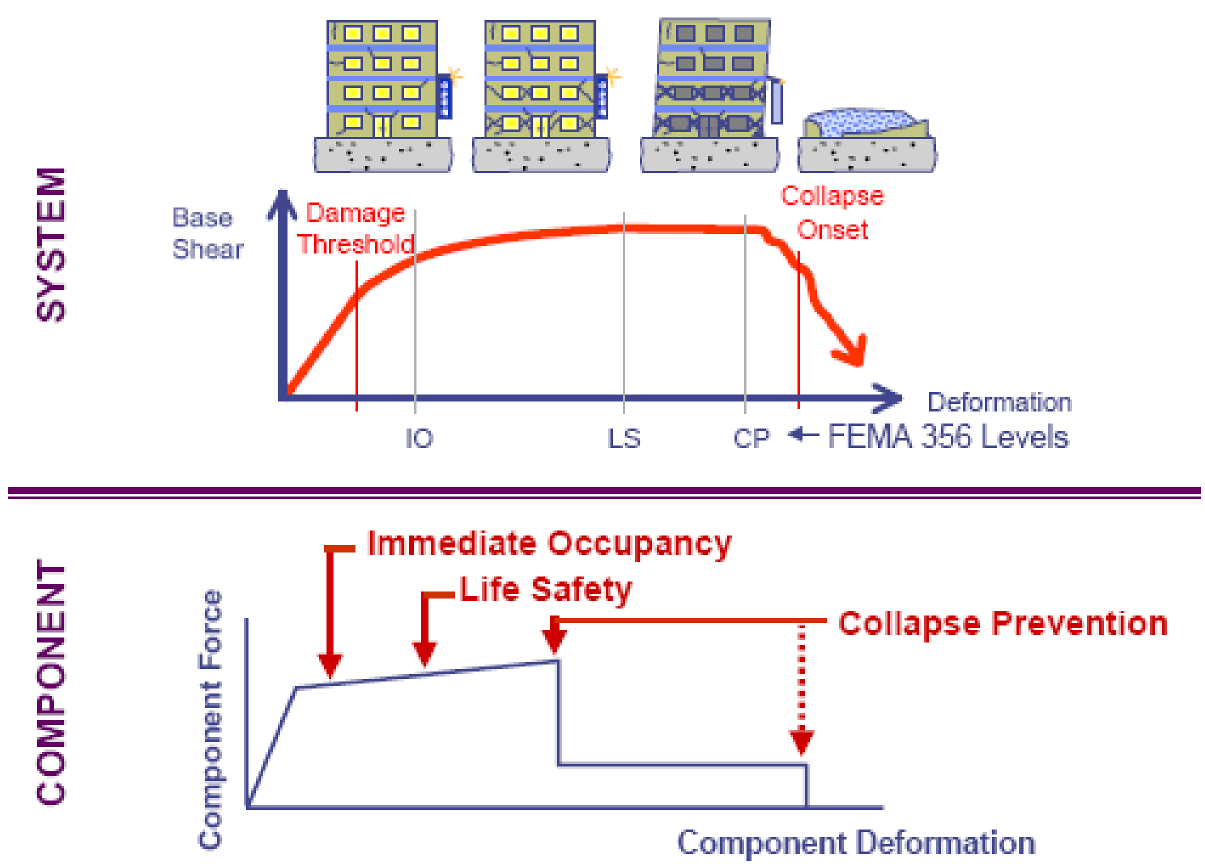


(a) Normalized Moment vs Story Drift Ratio

Han, K.-H. and Lee, C.-H. (2016). "Elastic Flange Local Buckling of I-Shaped Beams Considering Effect of Web Restraint" *Thin-walled Structures*, Vol. 105, pp.101-111, Elsevier.

Acceptance Criteria for "Nonlinear" Procedure

More straightforward, *m factor not needed*



PEER (2017). Tall Buildings Initiative (2017): "Guidelines for Based Seismic Design of Tall Building, Version 2.01"

6.8.2 Deformation-Controlled Actions

If the ultimate deformation capacity (δ_u) associated with any mode of deformation in a component is exceeded in any of the response history analyses, it is permitted either to:

1. Assume the strength associated with this mode of deformation is negligible for the remainder of that analysis and evaluate the stability of the structure and the effects on related strength quantities, or,
2. Consider the analysis to have unacceptable response.

For this purpose, δ_u shall be taken as the valid range of modeling as demonstrated by comparison of the hysteretic model with suitable laboratory test data or as described in Chapter 4.

6.8.3 Force-Controlled Actions

Categorize all force-controlled actions as being either Critical, Ordinary, or Noncritical, in accordance with Section 2.2.5.

Commentary: The Engineer of Record should identify force-controlled actions, and force-controlled actions should be categorized as being Critical, Ordinary, or Noncritical, subject to approval by the peer review. Appendix E provides recommended typical force-controlled actions and their categories.

Where force-controlled actions are limited by a well-defined yield mechanism, evaluate adequacy of force-controlled actions in accordance with Equations (6-1) and (6-2).

$$(1.2 + 0.2S_{MS})D + 1.0L + E_M \le \phi_s R_n \tag{6-1}$$

$$(0.9 - 0.2S_{MS})D + E_M \le \phi_s R_n \tag{6-2}$$

Conduct rigorous capacity design!

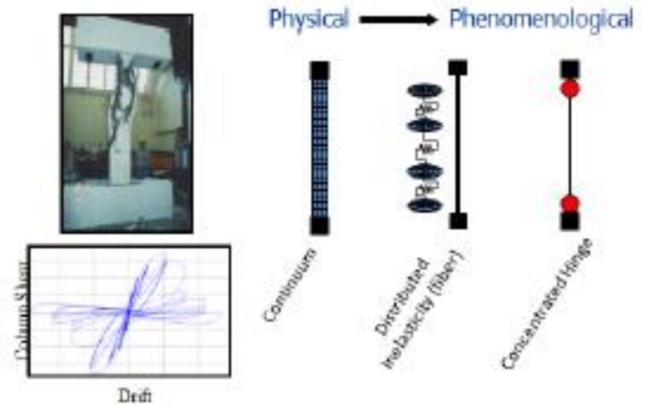
Unacceptable Response

PEER (2017). Tall Buildings Initiative (2017): "Guidelines for Based Seismic Design of Tall Building, Version 2.01"

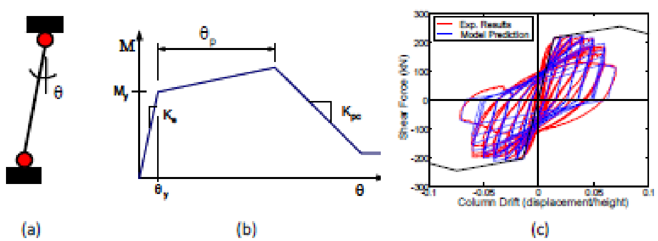
Unacceptable response to ground motion shall consist of any of the following:

1. Analytical solution fails to converge;
2. Demands on deformation-controlled elements exceed the valid range of modeling;
3. Demands on critical or ordinary force-controlled elements exceed the element capacity;
4. Deformation demands on elements not explicitly modeled exceed the deformation limits at which the members are no longer able to carry their gravity loads;
5. Peak transient story drift ratio in any story exceeds 0.045; and
6. Residual story drift ratio in any story exceeds 0.015.

State of the Art Practice and Limitations of Cyclic Deterioration Modeling for PBSD



Comparison of nonlinear component model types.



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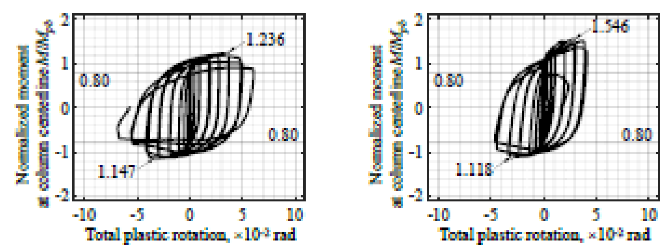
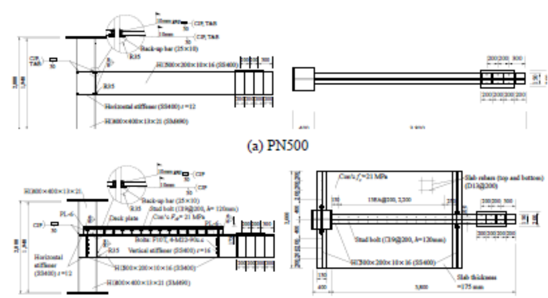
Seismic Retrofit of Welded Steel Moment Connections with Highly Composite Floor Slabs **C.H. Lee (2017)**

(a) PN500 (b) PN500C



(a) PN500 (b) PN500C



(a) PN500

(b) PN500C

Figure 1 Side view and top view of PN500 and PN500C

- PEER/ATC 72-1 (2010): Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings

3.2.1.5 Slab Effect

Steel beams are often part of a composite slab system. The presence of a composite slab will move the neutral axis, change the moment-rotation relationship, and affect the bending strength in both the positive and negative directions (Ricles et al., 2004). This effect is not captured in tests of bare steel connection subassemblies. In the positive moment direction (top flange in compression), the presence of a slab will delay local instabilities but will cause higher tensile strain demands in the bottom flange and welds. In the negative moment direction (bottom flange in compression), the presence of a slab can accelerate the occurrence of lateral-torsional buckling.

If the slab is thick, or the beam depth is small, this increase in strength can be a dominant factor. In the example shown in Figure 3-1, the capping rotation is unsymmetric in the two loading directions (about 3% in positive bending versus 1.2% in negative bending).

Unfortunately, the majority of currently available experimental test data come from tests that do not include a composite slab. Because of the scarcity of data on slab effects, recommended modeling parameters are based on bare steel beam tests without the presence of a composite slab.

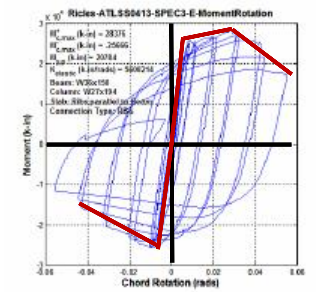
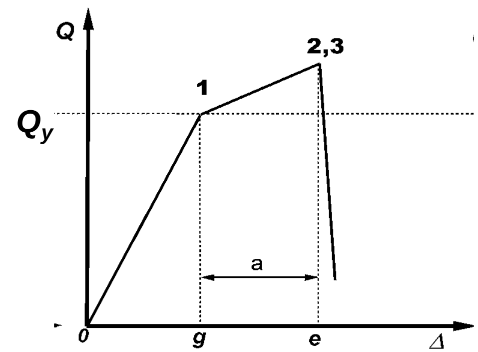


Figure 3-1 Hysteretic response of a steel beam with composite slab (data from Ricles et al., 2004).



Type 2 curve

Since **modeling of component behavior beyond the onset of significant degradation is an immature science**, it is prudent to set conservative limits on deformations associated with this limit state. These limits will typically be deformation values that are beyond the capping point, but prior to the ultimate deformation capacity in the load-deformation response of the component.

Damping issue

Damping in seismic design especially when nonlinear behavior is involved_ a kind of “chicken rib”; linear viscous damping model, originated mainly from mathematical convenience, is far from being reality; the damping constant itself is dependent upon response (stress) level…

Fortunately, very long-period structures like GBC tower seem less sensitive to the damping value assumed.

An example of effect of damping on very long-period structures (say, T1= 10 sec. like GBC tower)

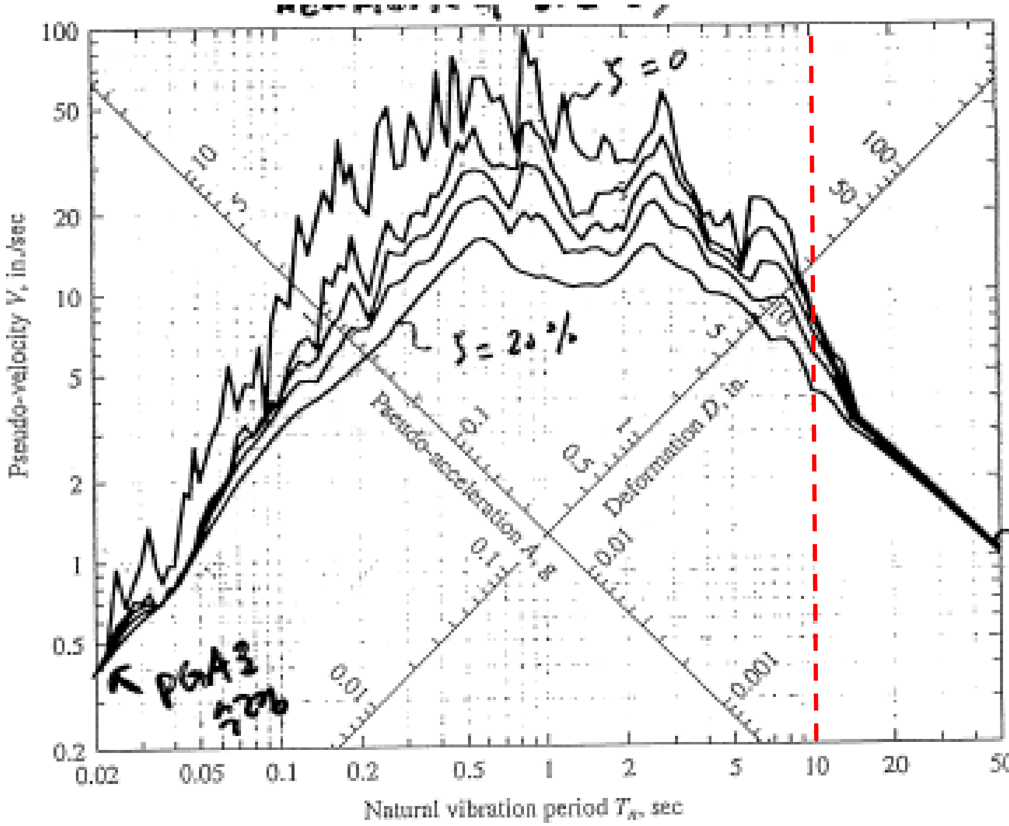
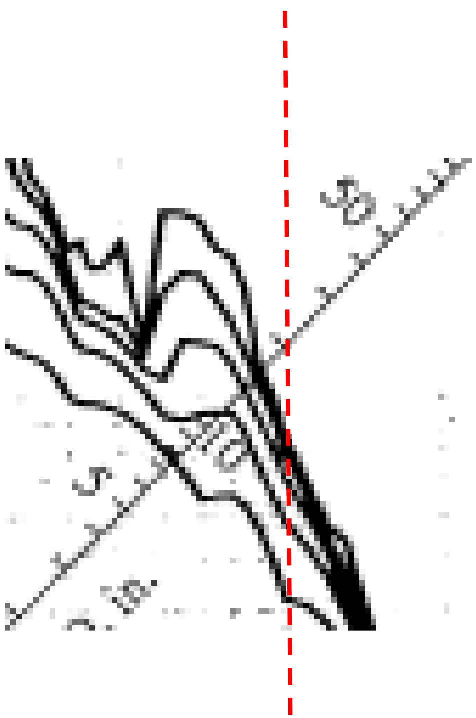


Figure 6.6.4 Combined D–V–A response spectrum for El Centro ground motion; ξ = 0, 2, 5, 10, and 20%.

$\log(\frac{T_n}{2\pi}A) = \log V$, $\log V = \log(\frac{T_n}{2\pi}) + \log A$, etc.



"El Centro input/**linear behavior assumed**"

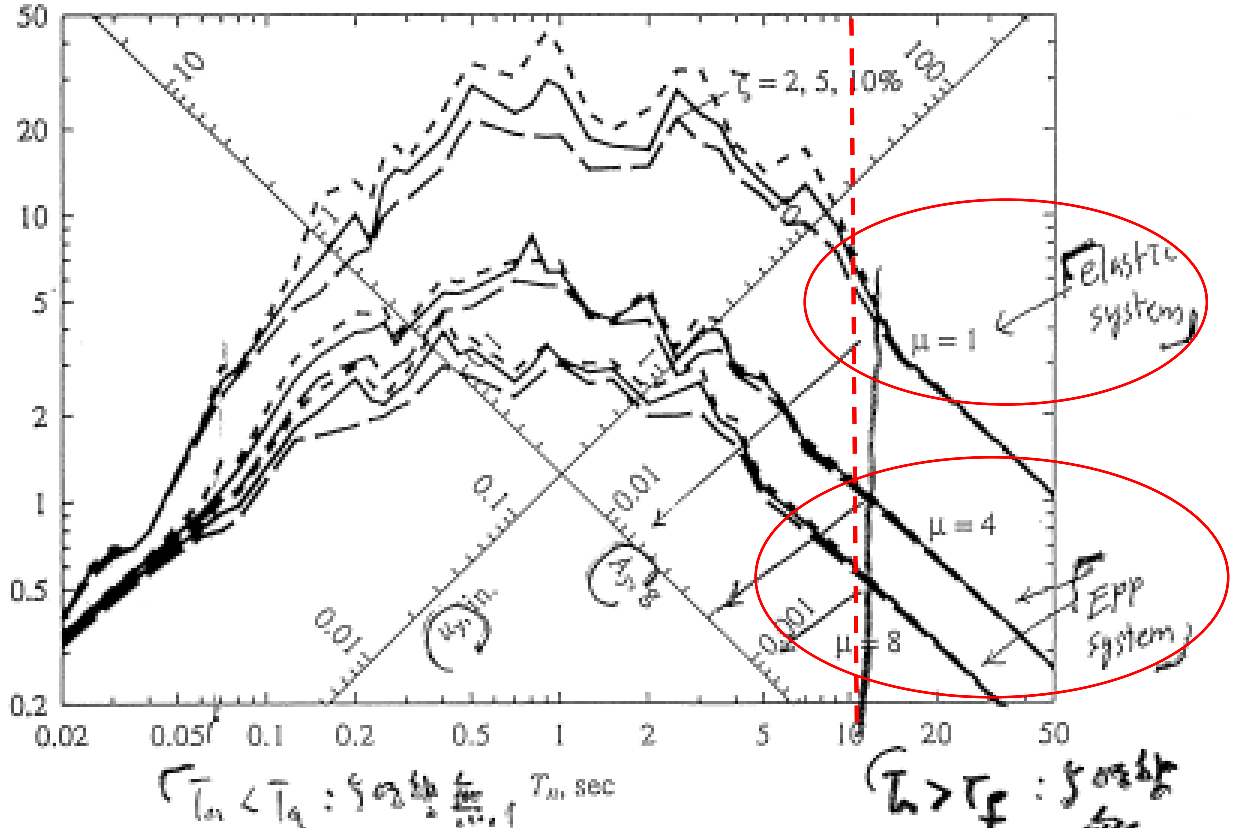


Figure 7.8.1 Response spectra for elastoplastic systems and El Centro ground motion; $\zeta = 2, 5,$ and 10% and $\mu = 1, 4,$ and 8.

4. Closing Remarks

- ✓ **Surely, next generation seismic code to circumvent or replace irrational R factor approach**

- ✓ **There still exist a lot of, often inevitable, prescriptive provisions in the overall evaluation procedure; not the level of the Code of Hammurabi yet.**

- ✓ **Many miles to go…especially test-backed/reliable cyclic deterioration modeling should be developed and implemented with acceptable accuracy in user-friendly commercial software; prudence needed.**

End of presentation