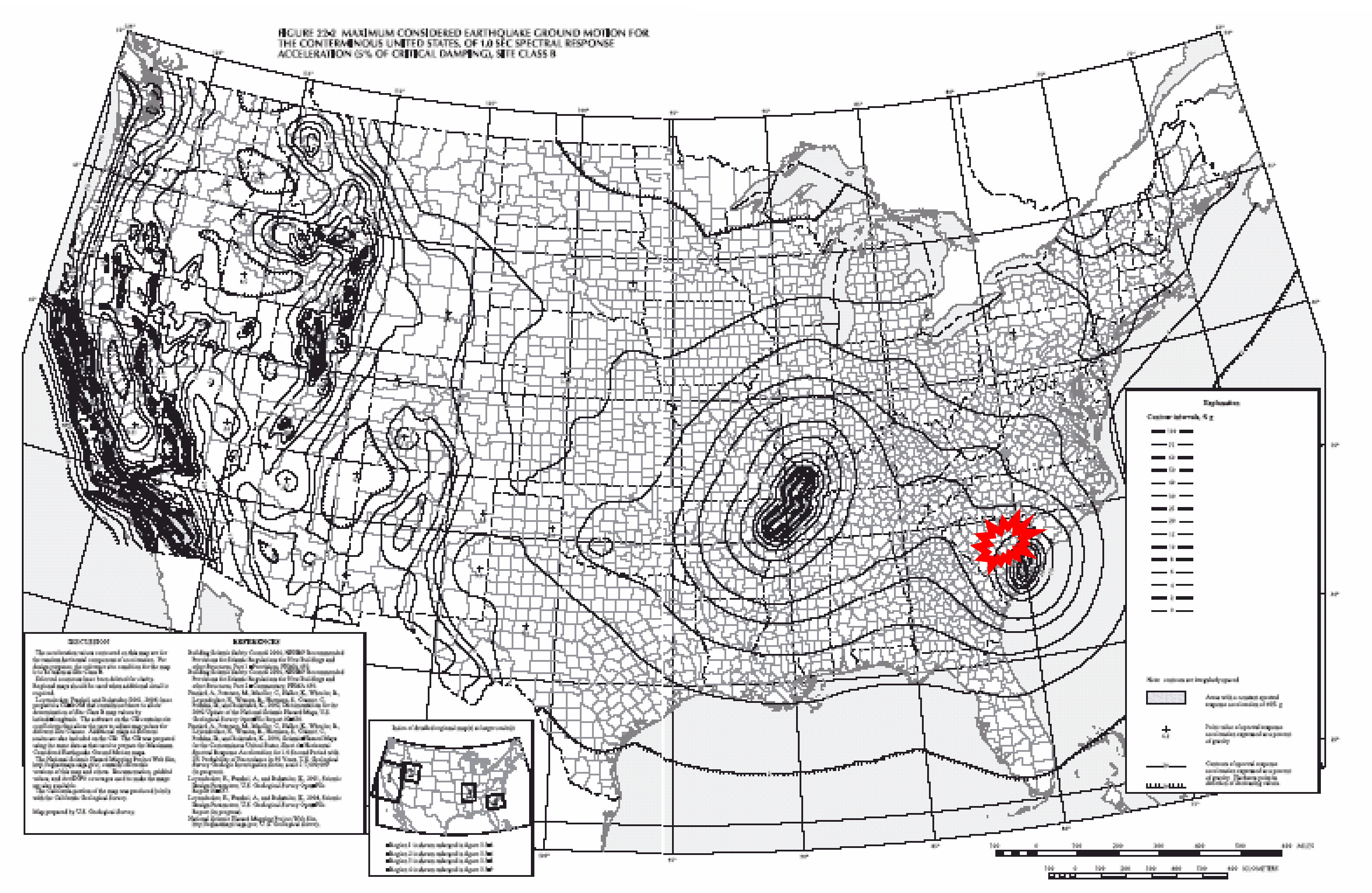
SEISMIC LOAD ANALYSIS



Topic Objectives

- Selection of method of analysis
- Description of analysis techniques
- Modeling considerations
- System regularity
- Load combinations
- Other considerations
- Drift computation and acceptance criteria
- P-delta effects

Load Analysis Procedure
(ASCE 7, *NEHRP Recommended Provisions*)

1. Determine building occupancy category (I-IV)
2. Determine basic ground motion parameters (S_S, S_1)
3. Determine site classification (A-F)
4. Determine site coefficient adjustment factors (F_a, F_v)
5. Determine design ground motion parameters (S_{dS}, S_{d1})
6. Determine seismic design category (A-F)
7. Determine importance factor
8. Select structural system and system parameters (R, C_d, Ω_o)

Load Analysis Procedure (Continued)

9. Examine system for configuration irregularities
10. Determine diaphragm flexibility (flexible, semi-rigid, rigid)
11. Determine redundancy factor (ρ)
12. Determine lateral force analysis procedure
13. Compute lateral loads
14. Add torsional loads, as applicable
15. Add orthogonal loads, as applicable
16. Perform analysis
17. Combine results
18. Check strength, deflection, stability

Occupancy Category (ASCE 7)

I) Low risk occupancy

Agricultural facilities
Temporary facilities
Minor storage facilities

II) Normal hazard occupancy

Any occupancy not described as I, III, IV

III) High hazard occupancy

High occupancy (more than 300 people in one room)
Schools and universities (various occupancy)
Health care facilities with < 50 resident patients
Power stations
Water treatment facilities
Telecommunication centers
Other….

Occupancy Category (ASCE 7, continued)

IV) Essential facilities

Hospitals or emergency facilities with surgery
Fire, rescue, ambulance, police stations
Designated emergency shelters
Aviation control towers
Critical national defense facilities
Other….

Note: *NEHRP Recommended Provisions* has Occupancy Categories I-III;
ASCE 7 I+II = NEHRP I, ASCE 7 III = NEHRP II, ASCE 7 IV = NEHRP III

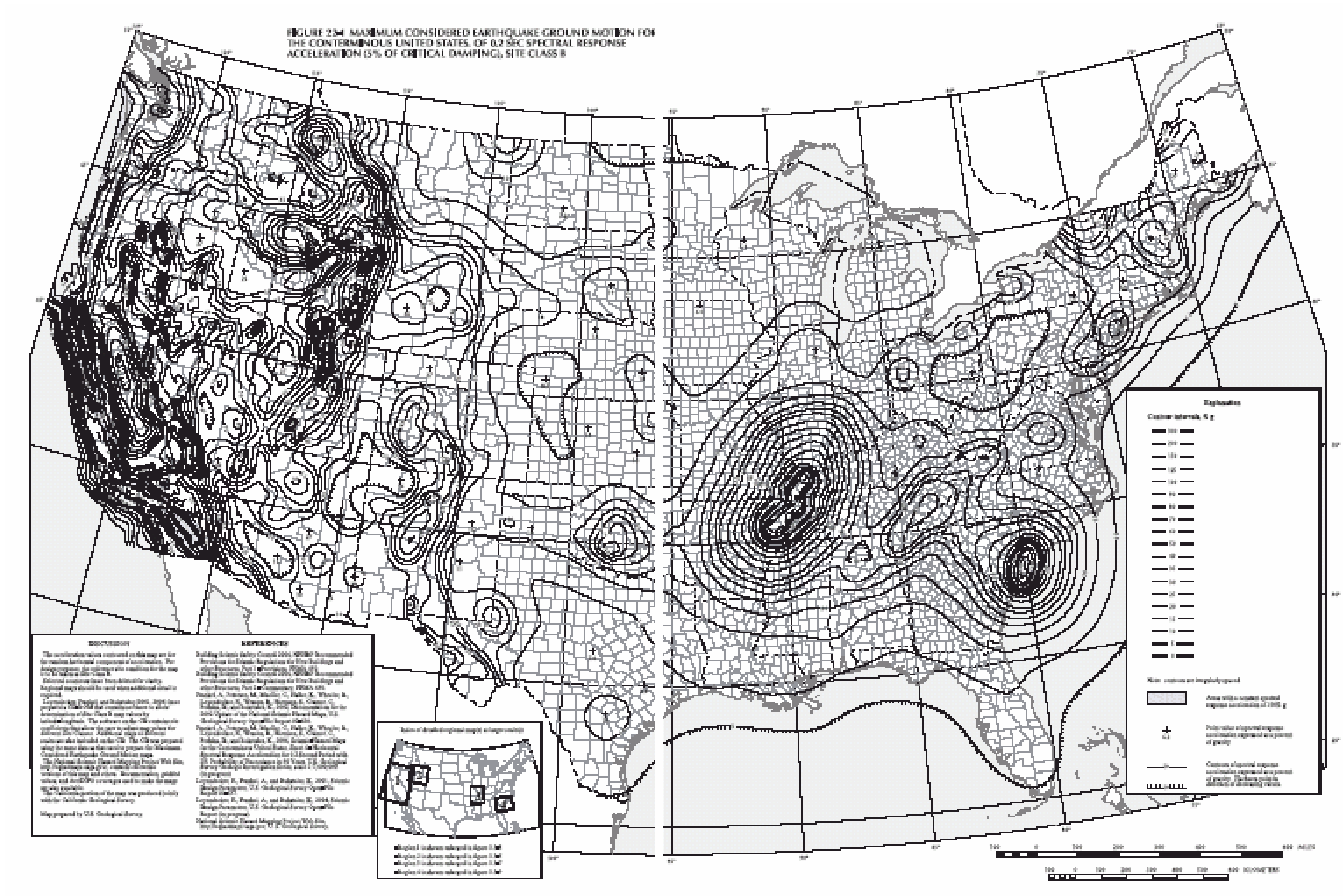
Hazard Maps → Design Ground Motions

- Provide 5% damped firm rock (Site Class B) spectral accelerations S_s and S_1 or 2% in 50 year probability or 1.5 times deterministic peak in areas of western US

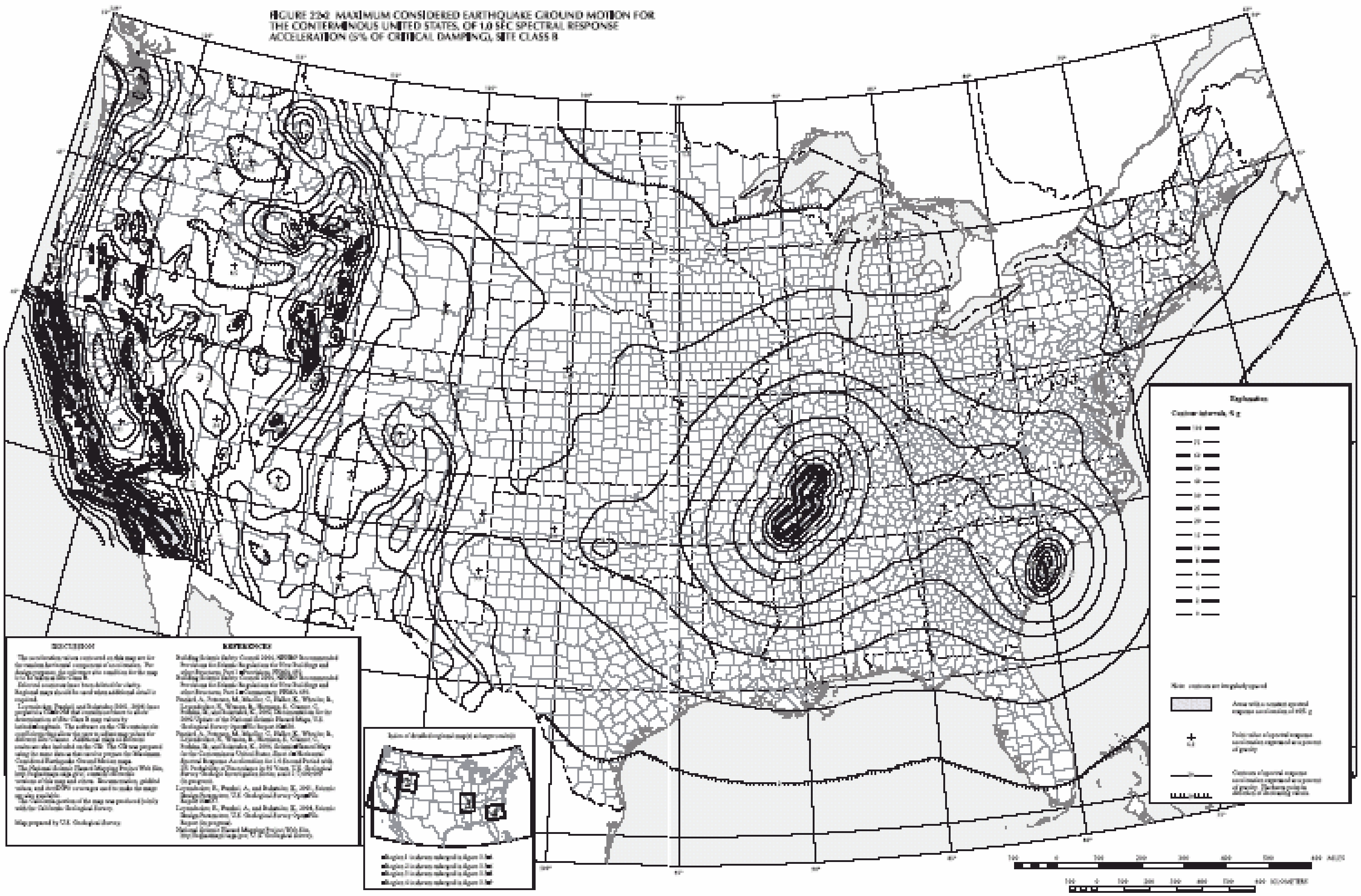
- Modified for other site conditions by coefficients F_v and F_a to determine spectral coefficients S_{MS} and S_{M1}

- Divided by 1.5 to account for expected good performance. This provides the design spectral coordinates S_{DS} and S_{D1}.

T = 0.2 Spectral Accelerations (S_s) for Conterminous US
(2% in 50 year, 5% damped, Site Class B)



T = 1 Spectral Accelerations (S_1) for Conterminous US
(2% in 50 year, 5% damped, Site Class B)



SITE CLASSES

A Hard rock $v_s > 5000$ ft/sec

B Rock: $2500 < v_s < 5000$ ft/sec

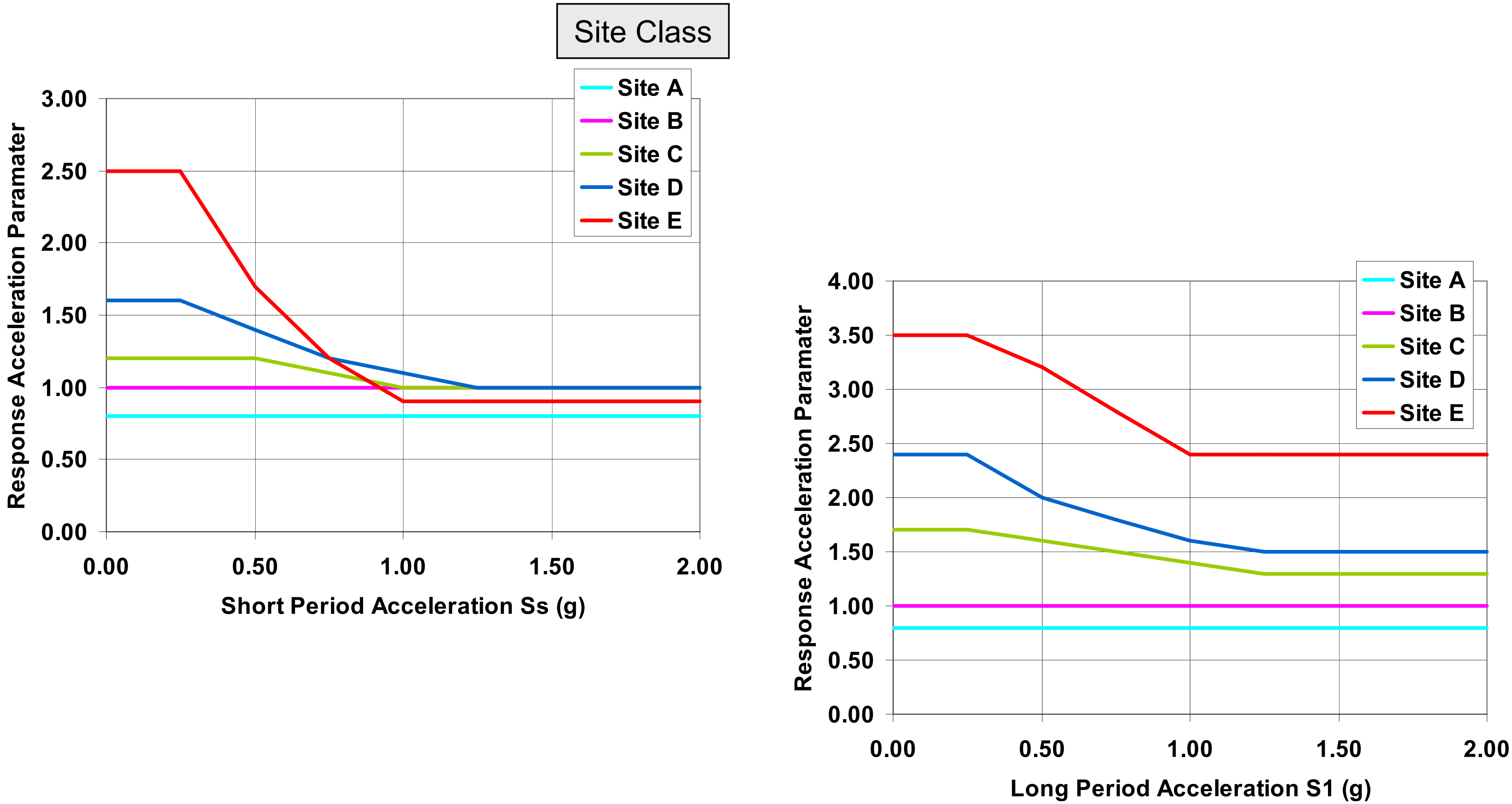
C Very dense soil or soft rock: $1200 < v_s < 2500$ ft/sec

D Stiff soil : $600 < v_s < 1200$ ft/sec

E $V_s < 600$ ft/sec

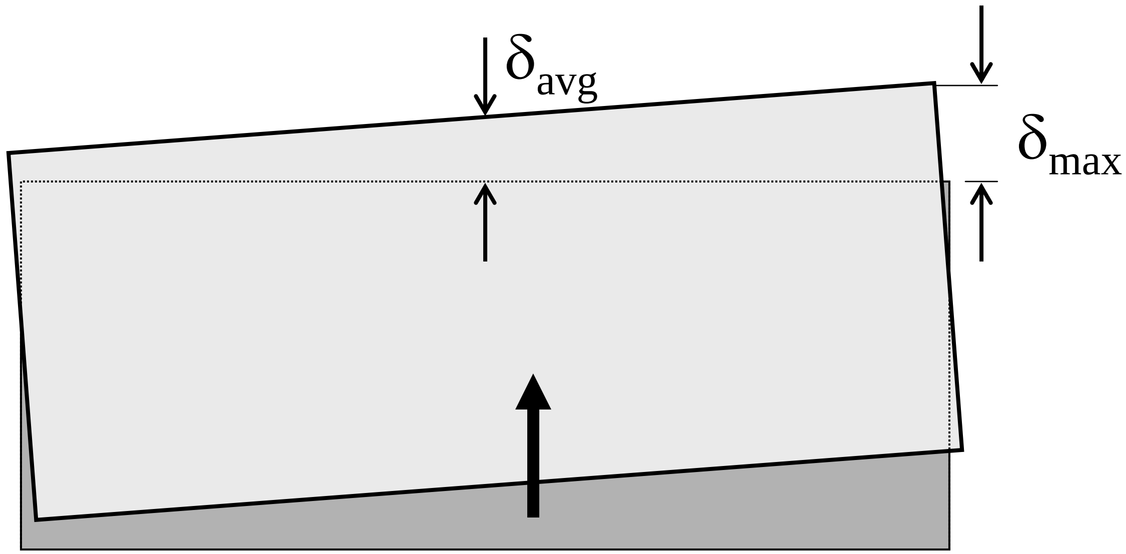
F Site-specific requirements

NEHRP Site Amplification for Site Classes A through E



Horizontal Structural Irregularities

1a) and 1b) Torsional Irregularity



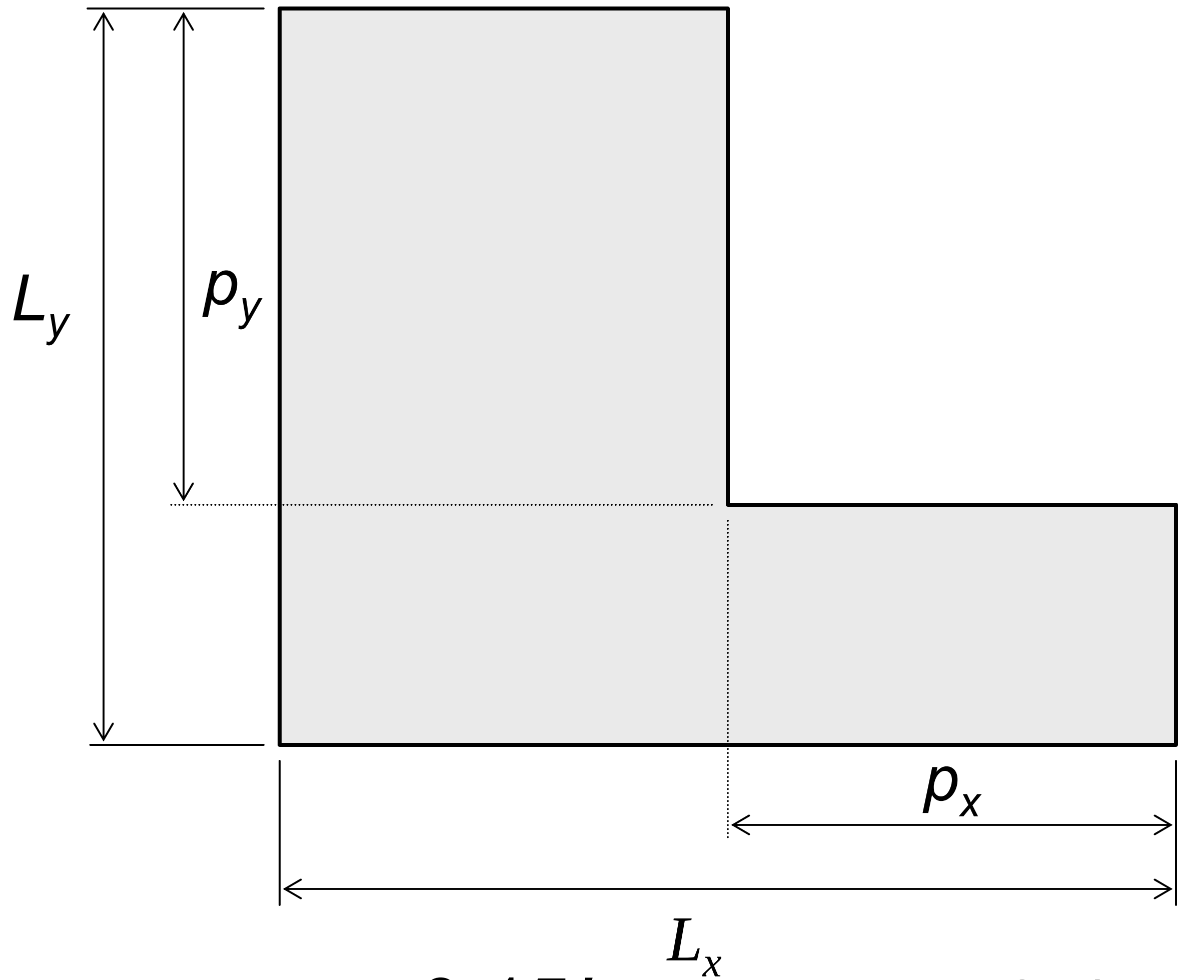
$$\delta_{max} < 1.2\delta_{avg} \quad \text{No irregularity}$$

$$1.2\delta_{avg} \le \delta_{max} \le 1.4\delta_{avg} \quad \text{Irregularity}$$

$$\delta_{max} > 1.4\delta_{avg} \quad \text{Extreme irregularity}$$

Irregularity 1b is NOT PERMITTED in SDC E or F.

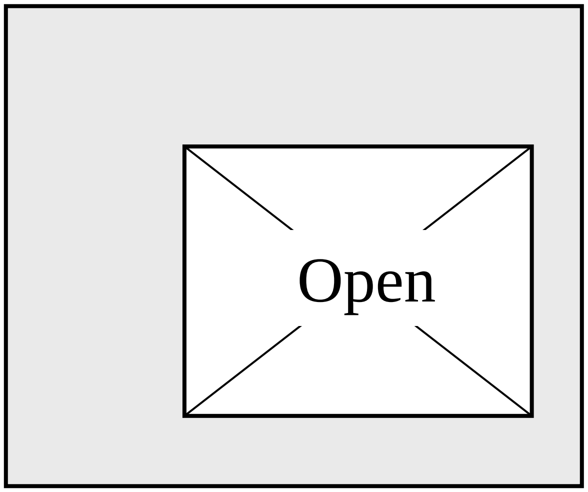
Horizontal Structural Irregularities
2) Re-entrant Corner Irregularity

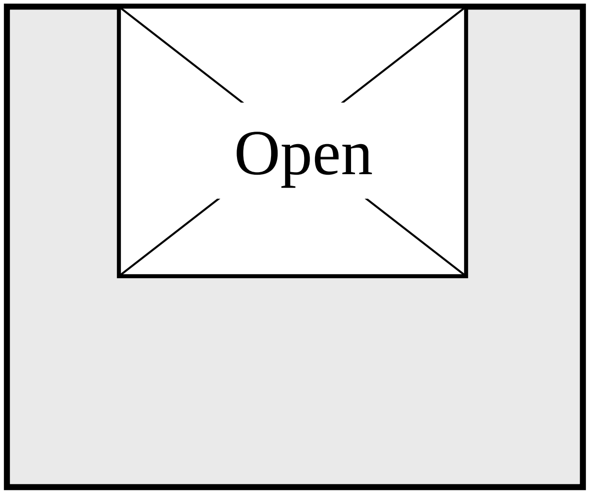


Irregularity exists if $p_y > 0.15L_y$ and $p_x > 0.15L_x$

Horizontal Structural Irregularities

3) Diaphragm Discontinuity Irregularity

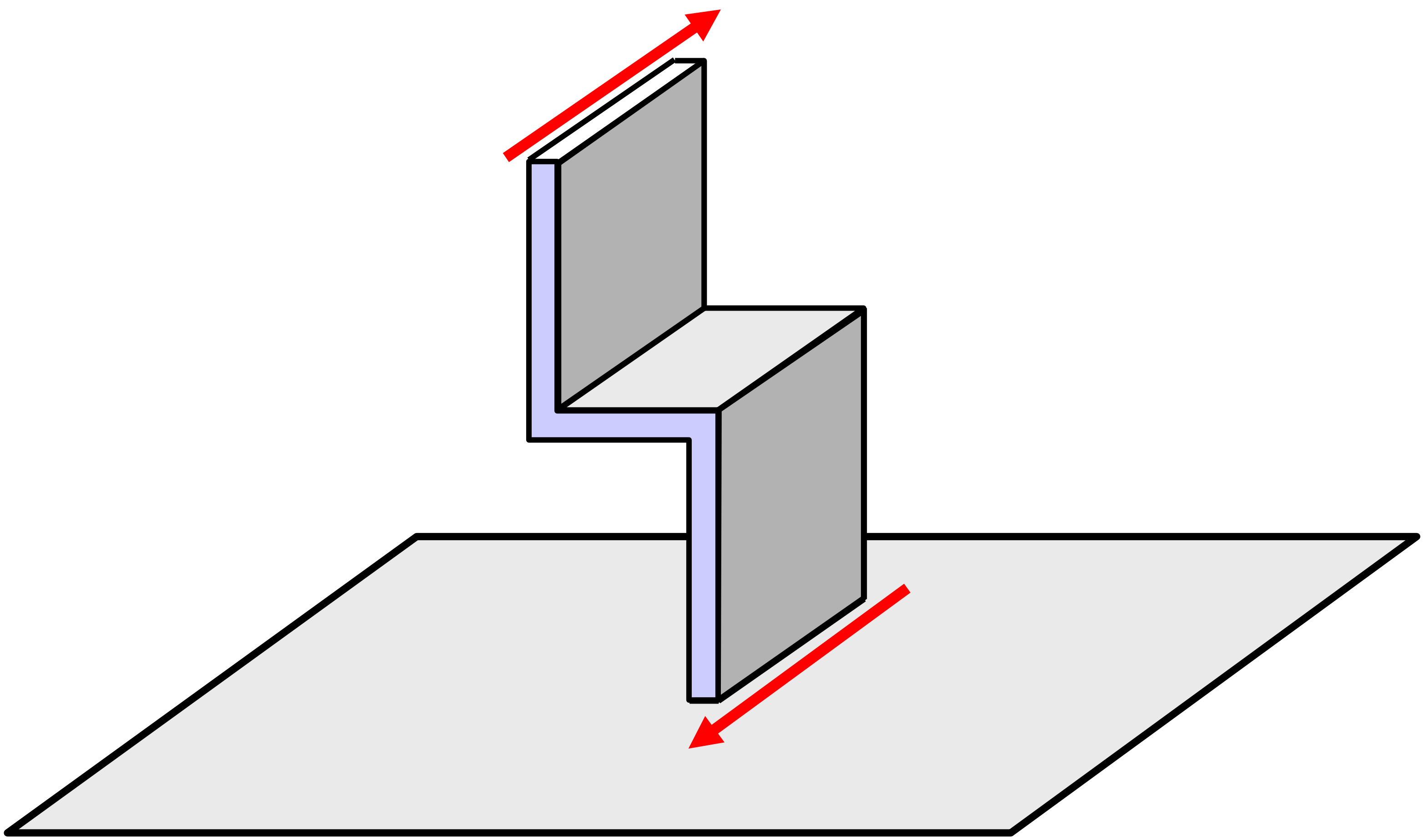




Irregularity exists if open area > 0.5 times floor area OR if effective diaphragm stiffness varies by more than 50% from one story to the next.

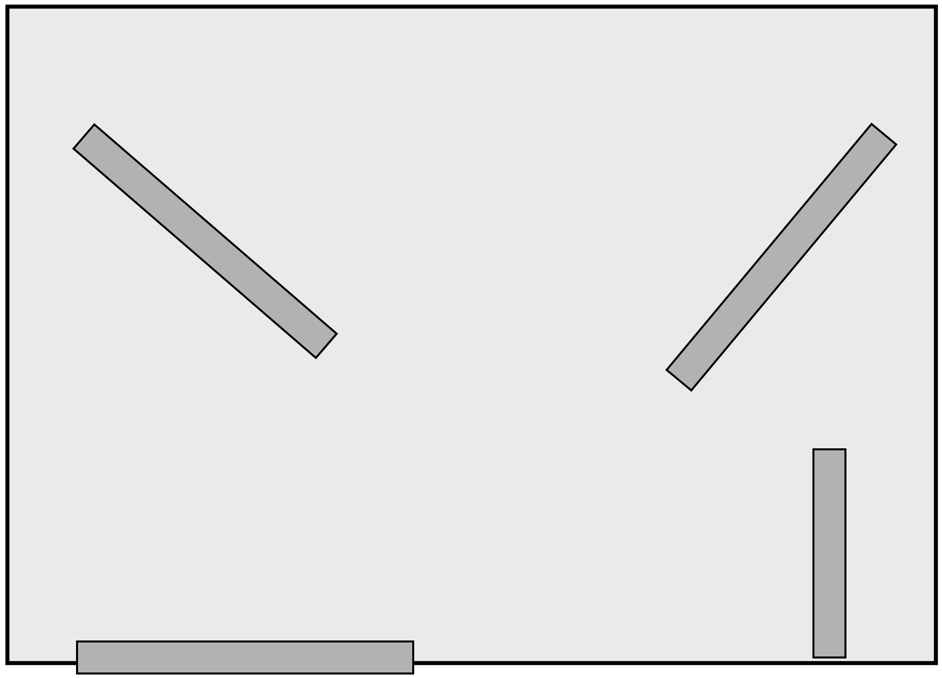
Horizontal Structural Irregularities

4) Out of Plane Offsets



Horizontal Structural Irregularities

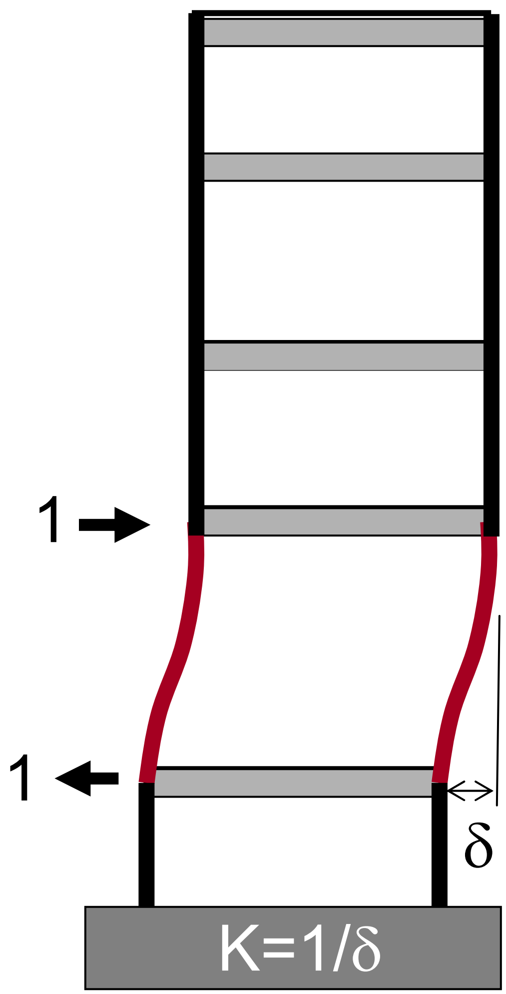
5) Nonparallel Systems Irregularity



Nonparallel system Irregularity exists when the vertical lateral force resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force resisting system.

Vertical Structural Irregularities

1a, 1b) Stiffness (Soft Story) Irregularity



Irregularity (1a) exists if stiffness of any story is less than 70% of the stiffness of the story above or less than 80% of the average stiffness of the three stories above.

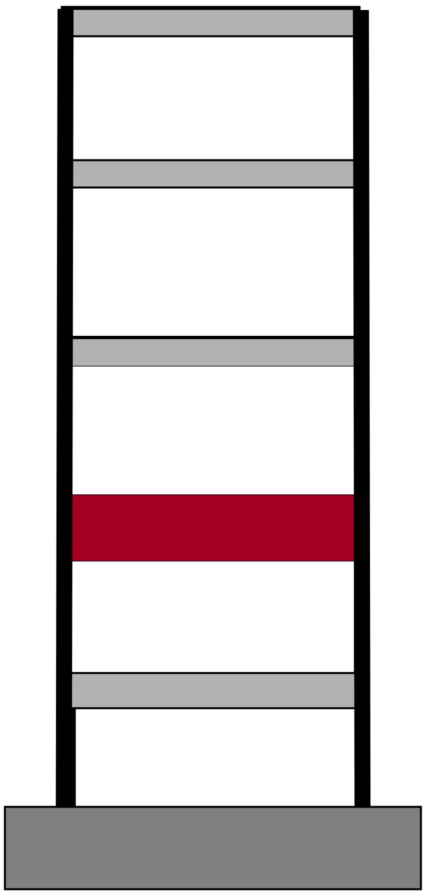
An extreme irregularity (1b) exists if stiffness of any story is less than 60% of the stiffness of the story above or less than 70% of the average stiffness of the three stories above.

Exception: Irregularity does not exist if no story drift ratio is greater than 1.3 times drift ratio of story above.

Irregularity 1b is NOT PERMITTED in SDC E or F.

Vertical Structural Irregularities

2) Weight (Mass) Irregularity

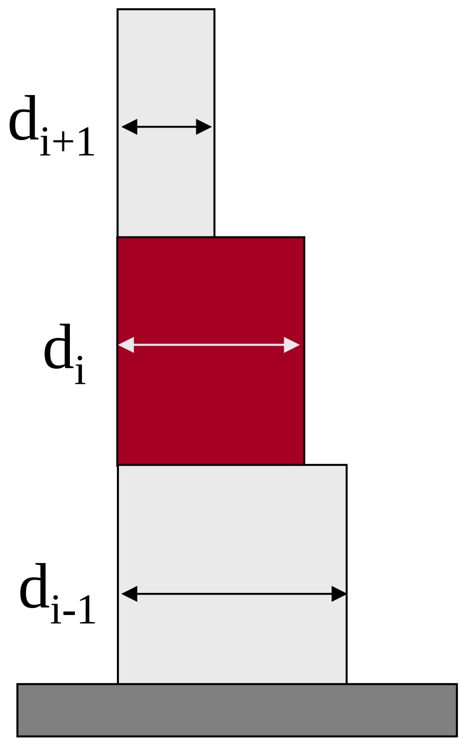


Irregularity exists if the effective mass of any story is more than 150% of the effective mass of an adjacent story.

Exception: Irregularity does not exist if no story drift ratio is greater than 1.3 times drift ratio of story above.

Vertical Structural Irregularities

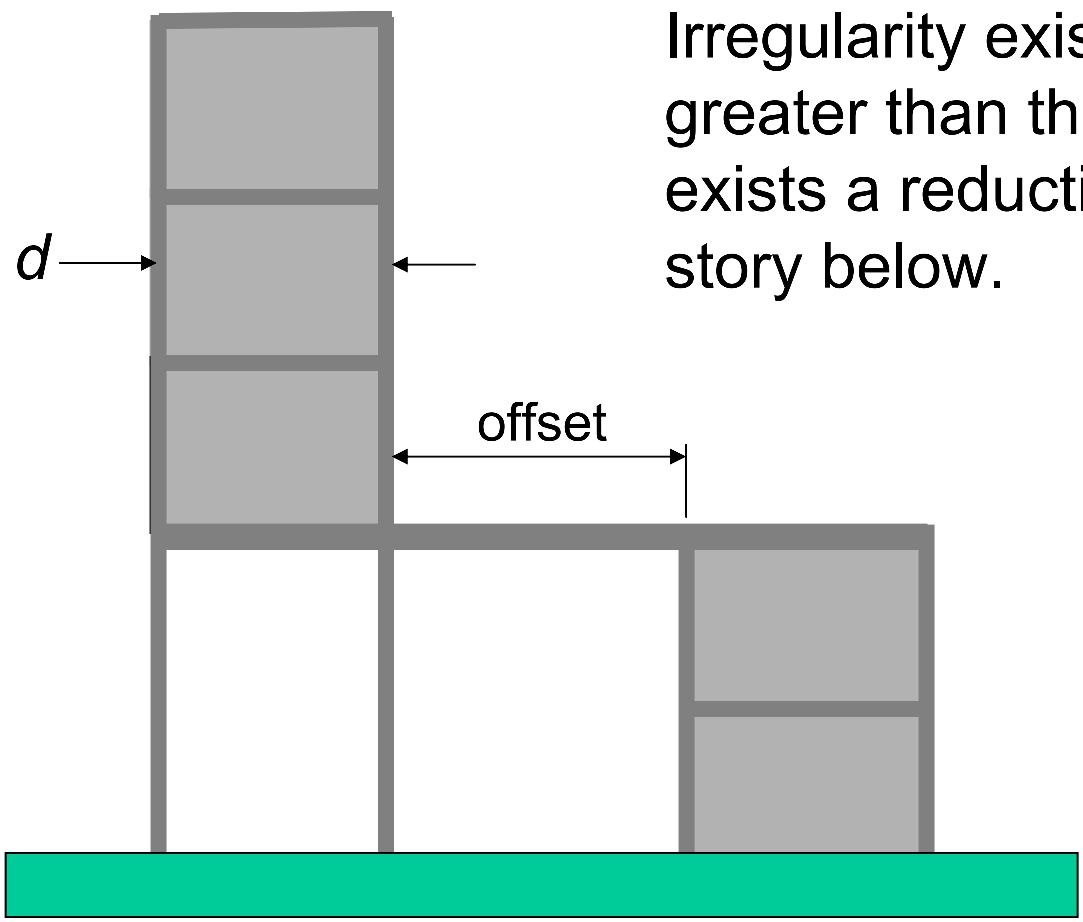
3) Vertical Geometric Irregularity



Irregularity exists if the dimension of the lateral force resisting system at any story is more than 130% of that for any adjacent story

Vertical Structural Irregularities

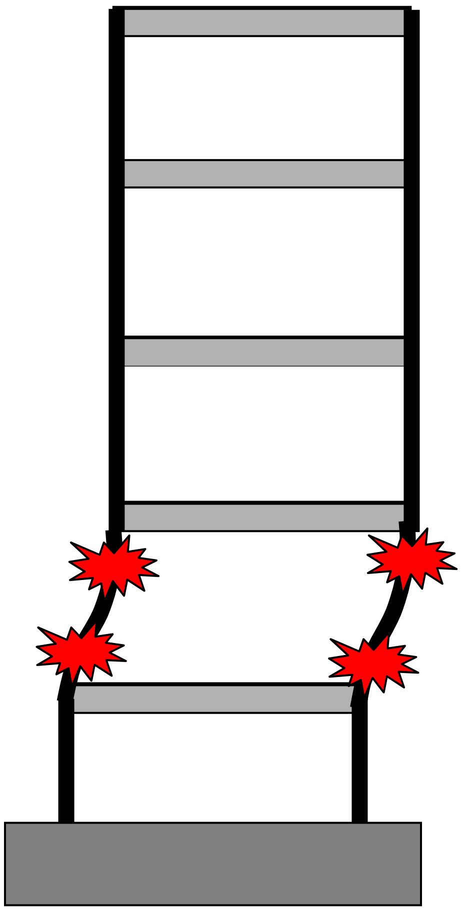
4) In-Plane Discontinuity Irregularity



Irregularity exists if the offset is greater than the width (*d*) or there exists a reduction in stiffness of the story below.

Vertical Structural Irregularities

5a, 5b) Strength (Weak Story) Irregularity



Irregularity (5a) exists if the lateral strength of any story is less than 80% of the strength of the story above.

An extreme irregularity (5b) exists If the lateral strength of any story is less than 65% of the strength of the story above.

Irregularities 5a and 5b are NOT PERMITTED in SDC E or F.
Irregularity 5b not permitted in SDC D.

Structural Systems

A. Bearing wall systems
B. Building frame systems
C. Moment resisting frame systems
D. Dual systems with SMRF
E. Dual systems with IMRF
F. Ordinary shear-wall frame interactive systems
G. Cantilever column systems
H. Steel systems not detailed for seismic

System Parameters:

Response modification coefficient = $\boldsymbol{R}$

System overstrength parameter = $\boldsymbol{\Omega_o}$

Deflection amplification factor = $\boldsymbol{C_d}$

Height limitation = by SDC

Structural Systems

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	2½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	6½	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	2½	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.1.4.2, and 14.5	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100

Bearing Wall

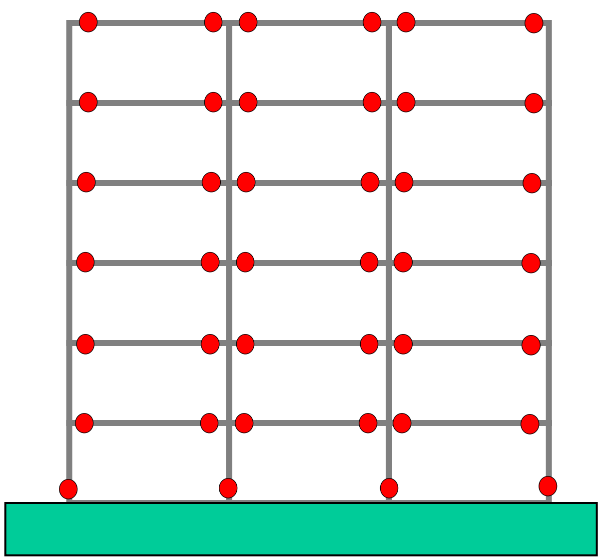
- Any metal or wood stud wall that supports more than 100 lbs/ft of vertical load in addition to its own weight
- Any concrete or masonry wall that supports more than 200 lbs/ft of vertical load in addition to its own weight

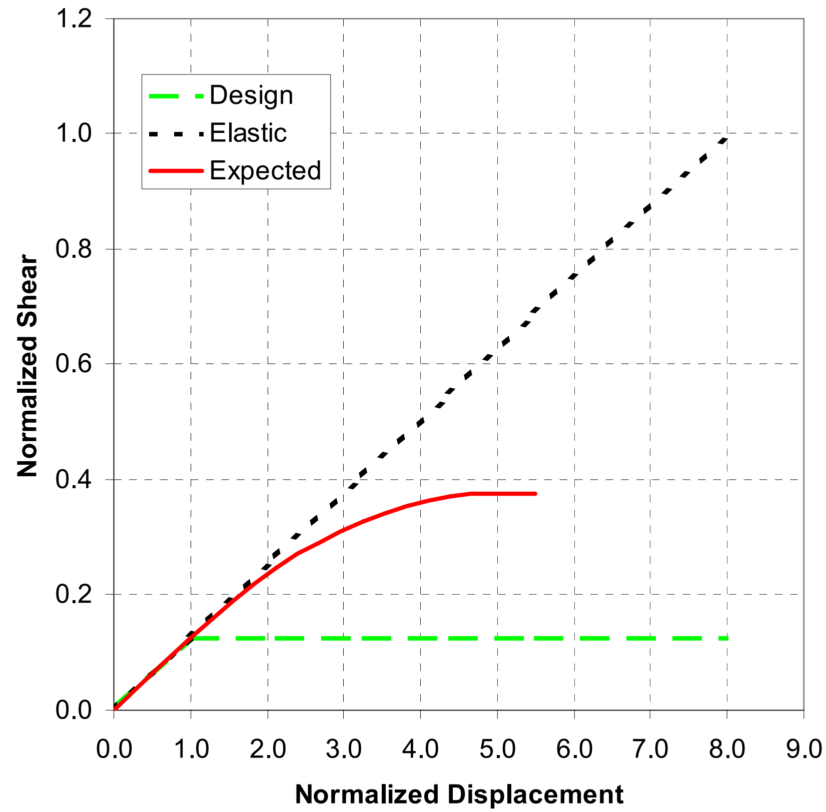
It appears that almost ANY concrete or masonry wall would be classified as a bearing wall!

Special Steel Moment Frame

R	8
C_d	5.5
Ω_o	3

A	B	C	D	E	F
NL	NL	NL	NL	NL	NL



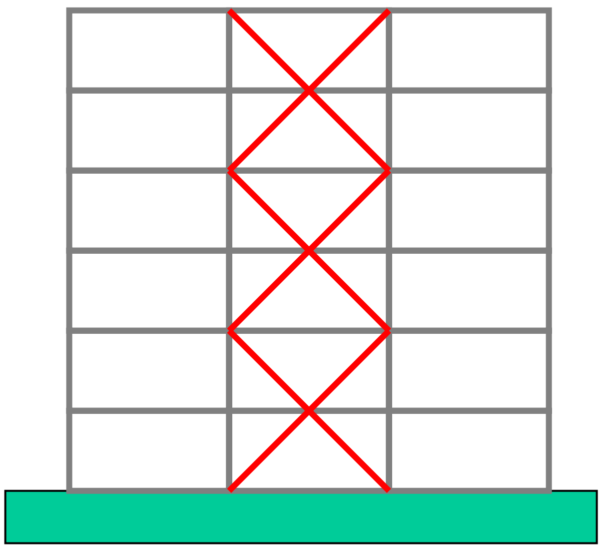


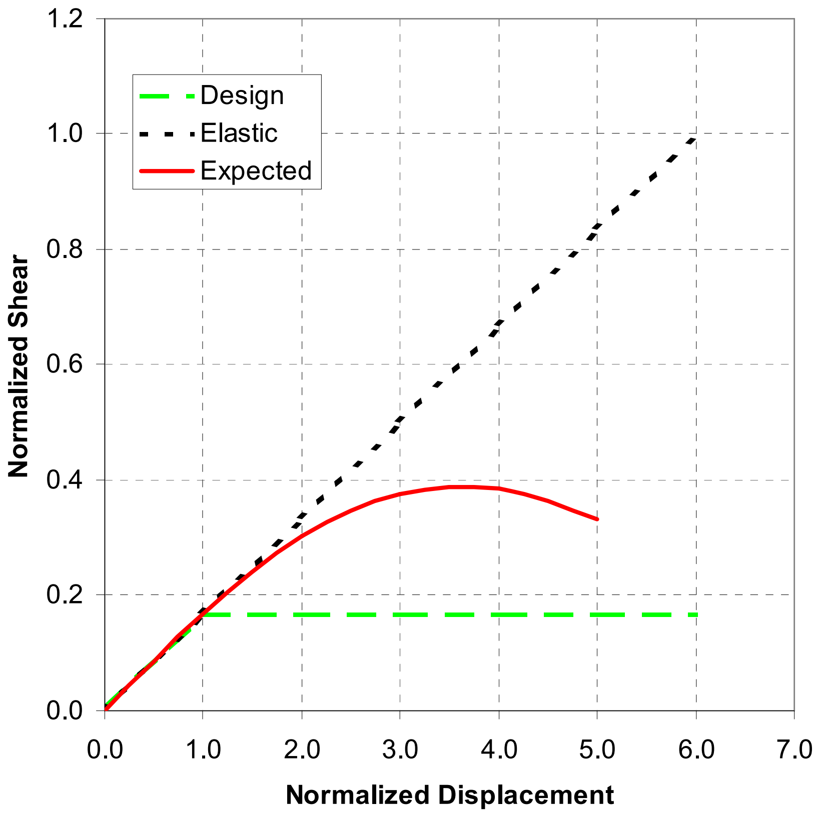
Advantages:
Architectural simplicity, relatively low base shear
Disadvantages:
Drift control, connection cost, connection testing

Special Steel Concentrically Braced Frame

R	6
C_d	5
Ω_o	2

A	B	C	D	E	F
NL	NL	NL	160	160	100



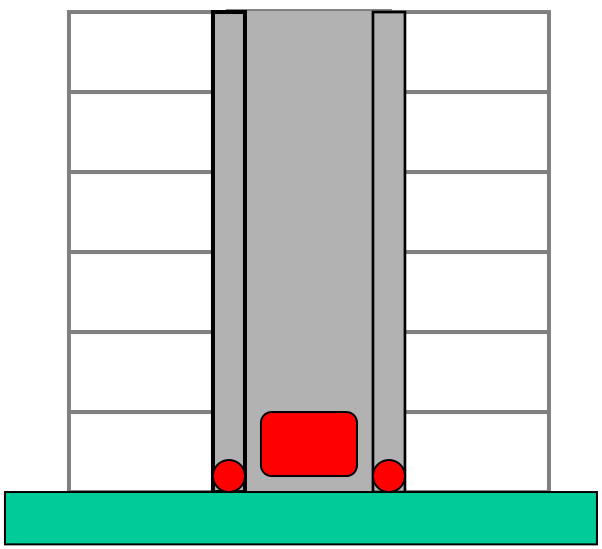


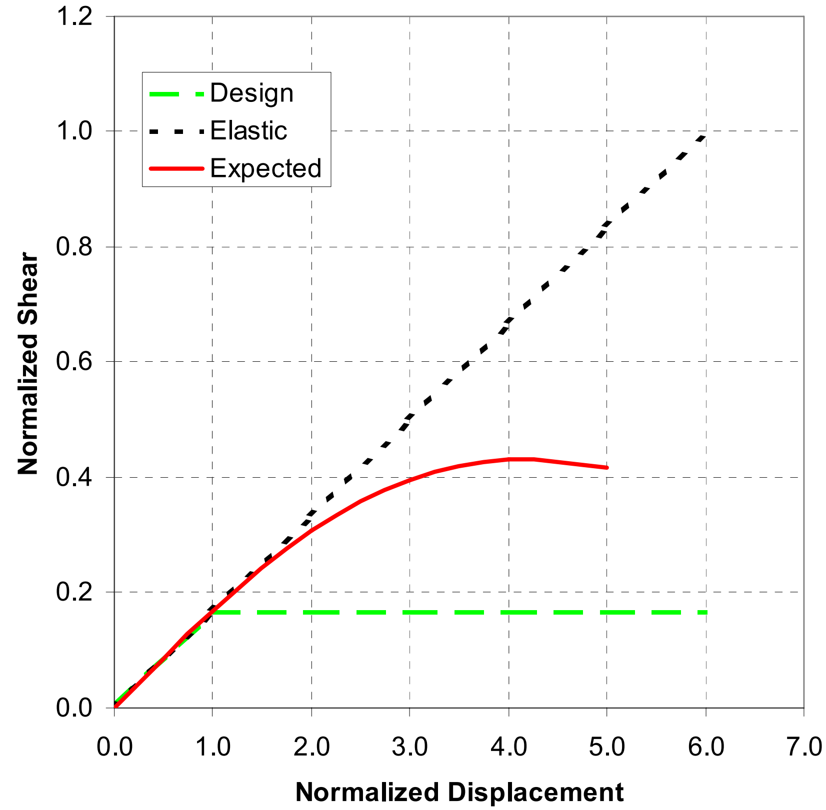
Advantages:
Lower drift, simple field connections
Disadvantages:
Higher base shear, high foundation forces, height limitations, architectural limitations

Special Reinforced Concrete Shear Wall

R 6
C_d 5
Ω_o 2.5

A	B	C	D	E	F
NL	NL	NL	160	160	100





Advantages:
Drift control
Disadvantages:
Lower redundancy (for too few walls)

Response Modification Factor *R*

Accounts for:

- Ductility
- Overstrength
- Redundancy
- Damping
- Past behavior

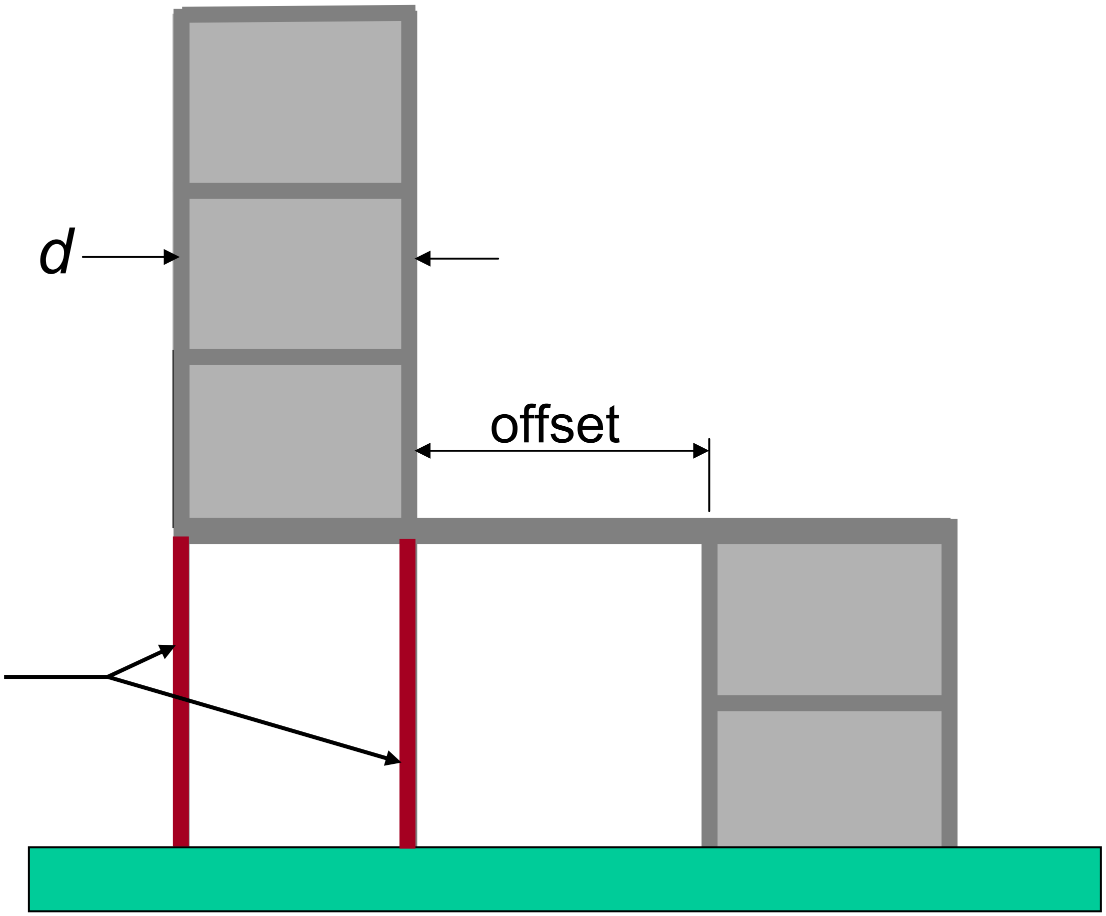
Maximum = 8

Eccentrically braced frame with welded connections
Buckling restrained brace with welded connections
Special moment frame in steel or concrete

Minimum = 1.5 (exclusive of cantilever systems)

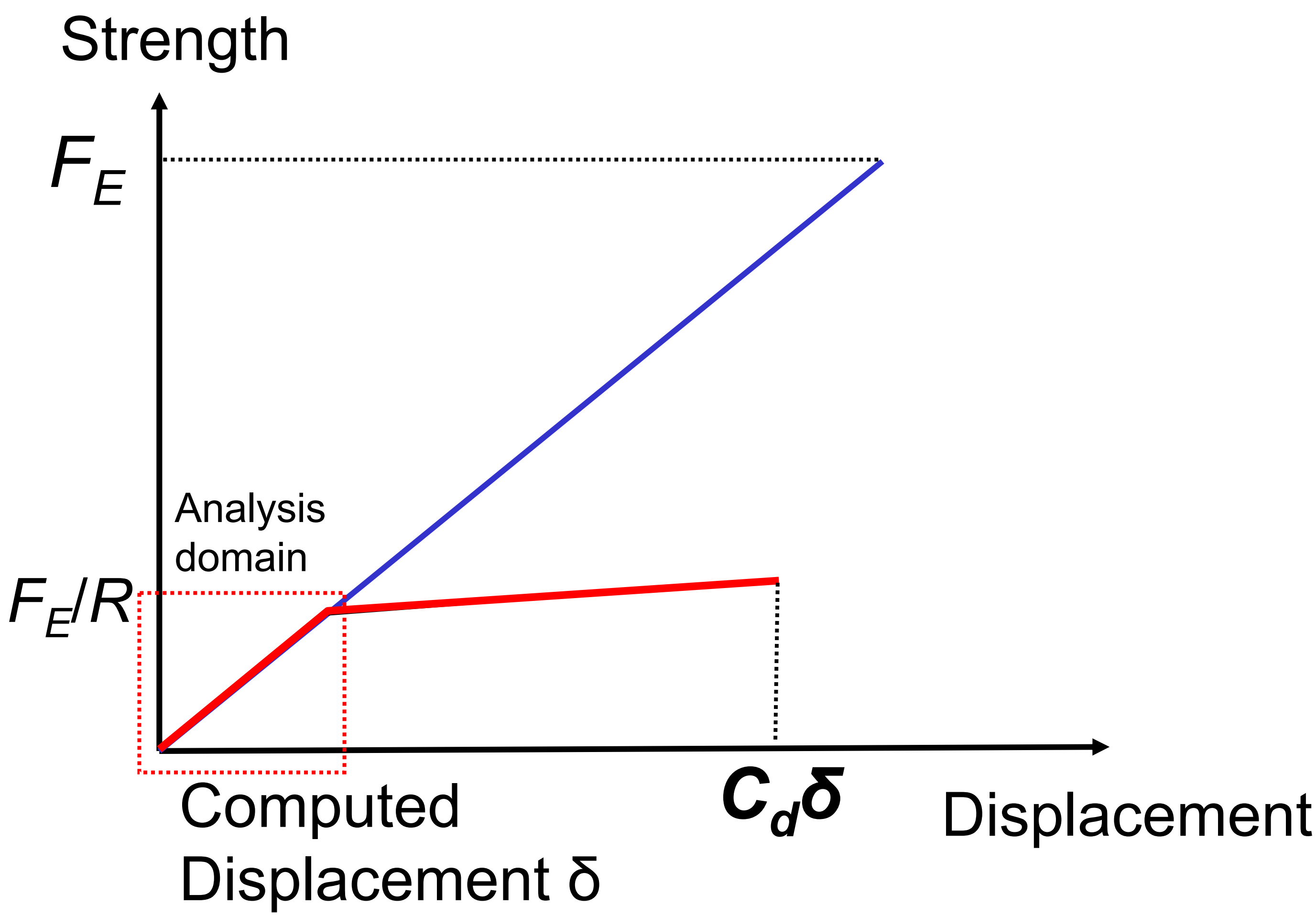
Ordinary plain masonry shear walls

Overstrength Factor Ω_o



Elements must be designed using load combination with factor Ω_o

Deflection Amplification Factor C_d

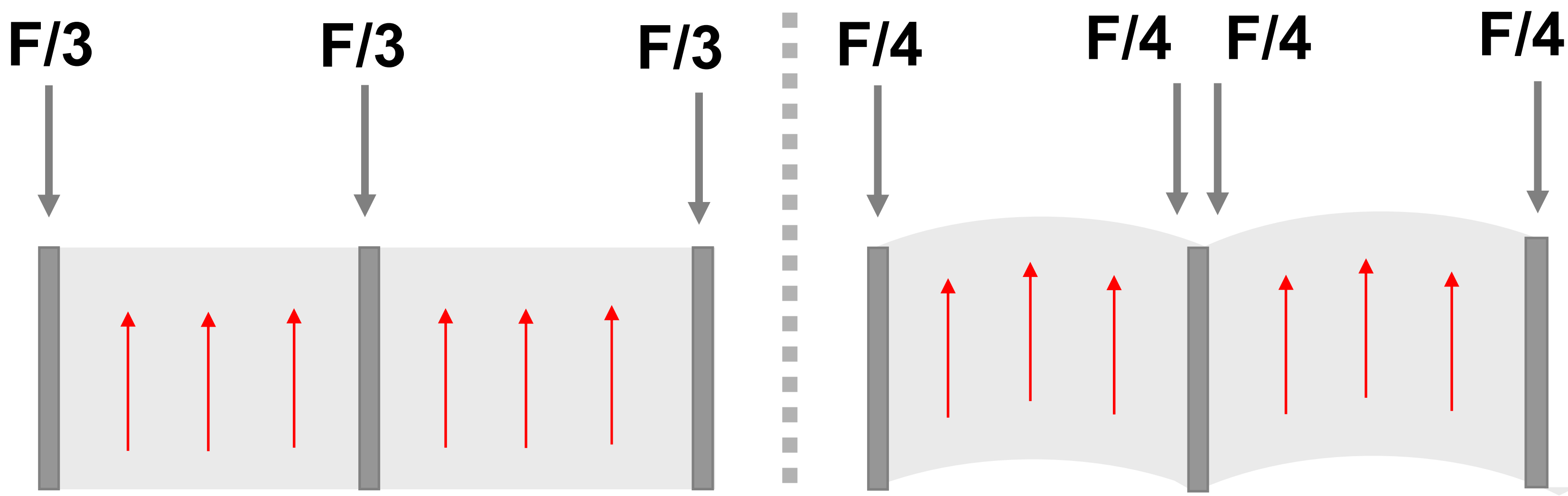


Diaphragm Flexibility

Diaphragms must be considered as semi-rigid unless they can be classified as **FLEXIBLE** or **RIGID**.

- Untopped steel decking and untopped wood structural panels are considered **FLEXIBLE** if the vertical seismic force resisting systems are steel or composite braced frames or are shear walls.
- Diaphragms in one- and two-family residential buildings may be considered **FLEXIBLE**.
- Concrete slab or concrete filled metal deck diaphragms are considered **RIGID** if the width to depth ratio of the diaphragm is less than 3 and if no horizontal irregularities exist.

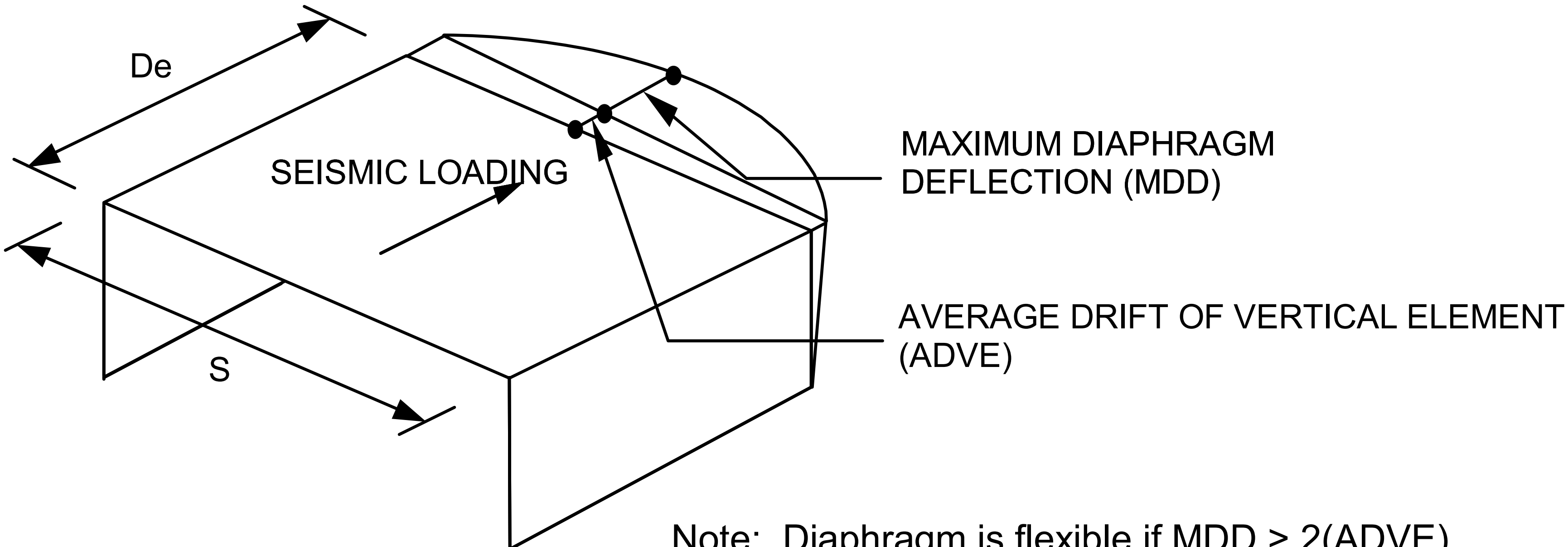
Rigid vs Flexible Diaphragms



RIGID
Center Wall Shear = F/3

FLEXIBLE
Center Wall Shear = F/2

Diaphragm Flexibility



Note: Diaphragm is flexible if MDD > 2(ADVE).

Diagram taken from ASCE 7-05

Importance Factors

SUG	Importance Factor
IV	1.50
III	1.25
I, II	1.00

Using ASCE 7-05 Use Groups

Seismic Design Category = Seismic Use Group + Design Ground Motion

Based on SHORT PERIOD acceleration

Value of S_{DS}	Seismic Use Group*		
	I, II	**III**	**IV**
S_{DS} < 0.167g	**A**	**A**	**A**
0.167g < S_{DS} < 0.33g	**B**	**B**	**C**
0.33g < S_{DS} < 0.50g	**C**	**C**	**D**
0.50g < S_{DS}	**D**	**D**	**D**

*Using ASCE 7-05 Use Groups

Seismic Design Category

Based on LONG PERIOD acceleration

Value of S_{D1}	Seismic Use Group*		
	I, II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g < S_{D1} < 0.133g$	B	B	C
$0.133g < S_{D1} < 0.20g$	C	C	D
$0.20g < S_{D1}$	D	D	D

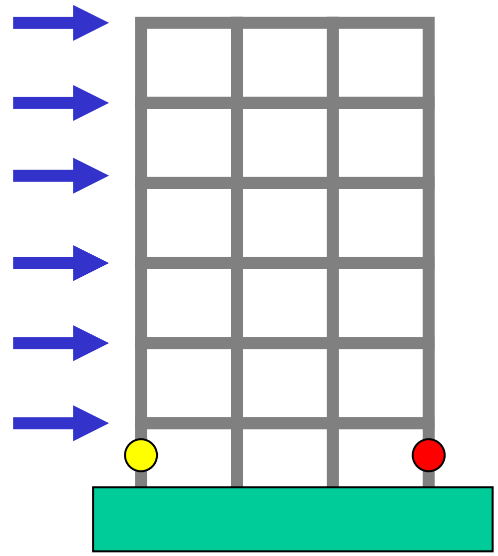
Value of S_1	Seismic Use Group*		
	I, II	III	IV
$S_1 > 0.75g$	E	E	F

*Using ASCE 7-05 Use Groups

Basic Load Combinations (involving earthquake)

$$1.2D + 1.0E + L + 0.2S$$

$$0.9D + 1.0E$$



Note: 0.5L may be used when L_o < 100 psf (except garages and public assembly)

Combination of Load Effects

Use ASCE 7 basic load combinations but substitute the following for the earthquake effect E:

$$E = E_h \pm E_v$$

$$E_h = \rho Q_E \qquad\qquad E_v = 0.2 S_{DS} D$$

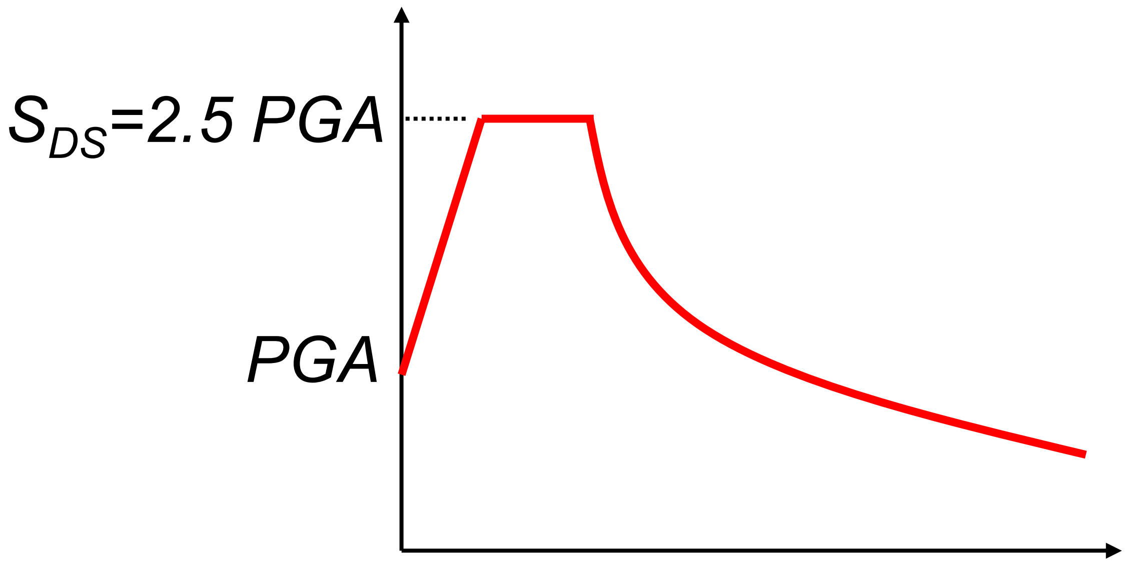
Resulting load combinations (from this and previous slide)

$$(1.2 + 0.2 S_{DS})D + \rho Q_E + L + 0.2S$$

$$(0.9 - 0.2 S_{DS})D + \rho Q_E$$

Note: See ASCE 7 for combinations including hydrostatic load

Vertical Accelerations are Included in the Load Combinations



Vertical acceleration = 0.2(2.5) = 0.5 PGA

Combination of Load Effects (including overstrength factor)

$$E = E_{mh} \pm E_v$$

$$E_{mh} = \Omega_o Q_E \qquad E_v = 0.2 S_{DS} D$$

Resulting load combinations (from this and previous slide)

$$(1.2 + 0.2 S_{DS})D + \Omega_o Q_E + L + 0.2S$$

$$(0.9 - 0.2 S_{DS})D + \Omega_o Q_E$$

Note: See ASCE 7 for combinations including hydrostatic load

Redundancy Factor ρ

Cases where $\rho = 1.0$

- Structures assigned to SDC B and C
- Drift and P-delta calculations
- Design of nonstructural components
- When overstrength (Ω_o) is required in design
- Diaphragm loads
- Systems with passive energy devices

Redundancy Factor ρ

Cases where ρ = 1.0 for SDC D, E, and F buildings

When each story resisting more than 35% of the base shear in the direction of interest complies with requirements of Table 12.3-3 (next slide)

OR

Structures that are regular in plan at all levels and have at least two bays of perimeter framing on each side of the building in each orthogonal direction for each story that resists more than 35% of the total base shear.

Otherwise ρ = 1.3

Redundancy Factor ρ

Requirements for ρ = 1 in SDC D, E, and F buildings

Braced Frames

Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

Moment Frames

Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

Redundancy Factor ρ

Requirements for $\rho = 1$ in SDC D, E, and F buildings

Shear Walls

Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

Cantilever Column

Loss of moment resistance at the base Connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

Required Methods of Analysis

The equivalent lateral force method is allowed for all buildings in SDC B and C. It is allowed in all SDC D, E, and F buildings EXCEPT:

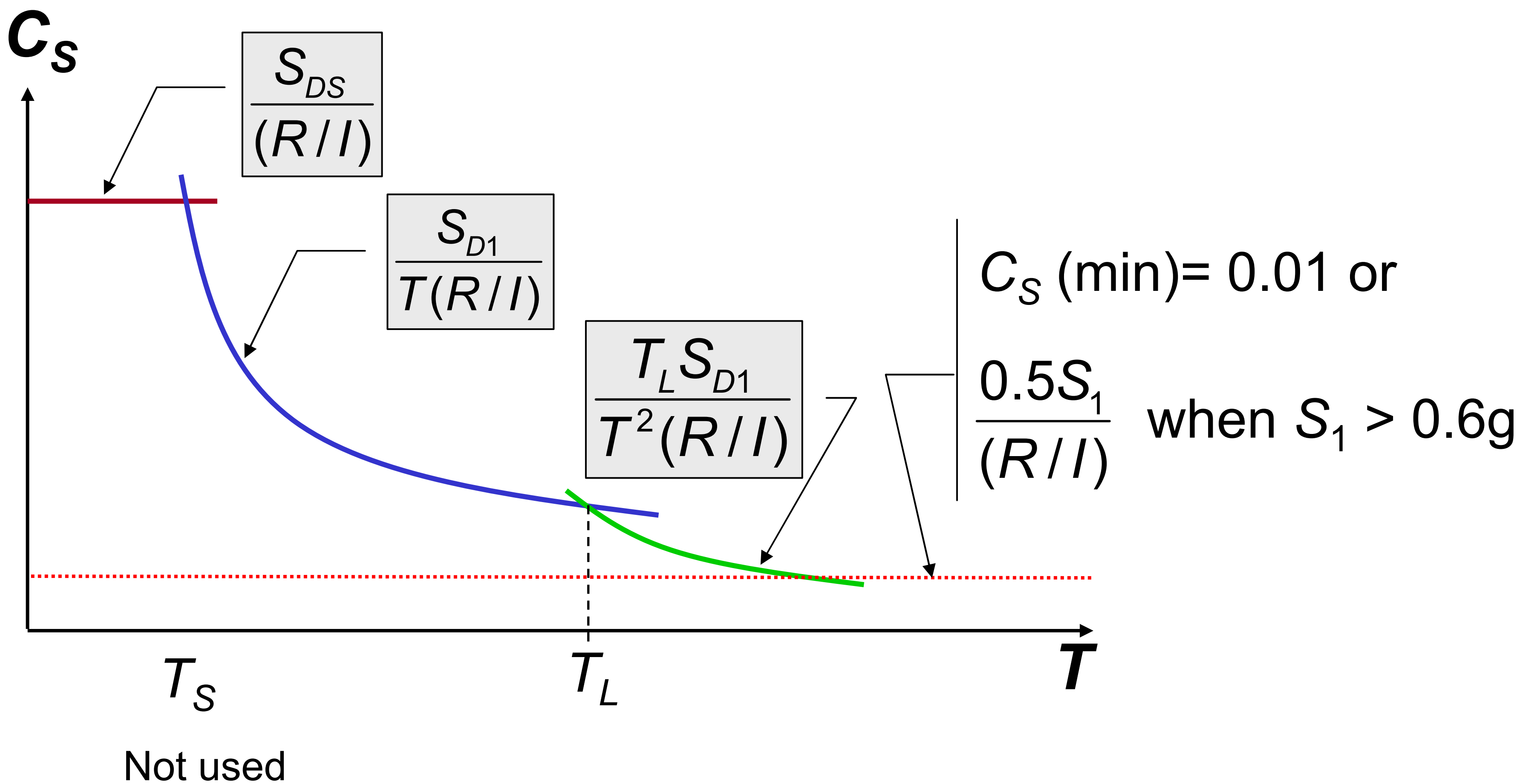
Any structure with $T > 3.5\ T_s$

Structures with $T < 3.5\ T_s$ and with Plan Irregularity 1a or 1b or Vertical Irregularity 1, 2 or 3.

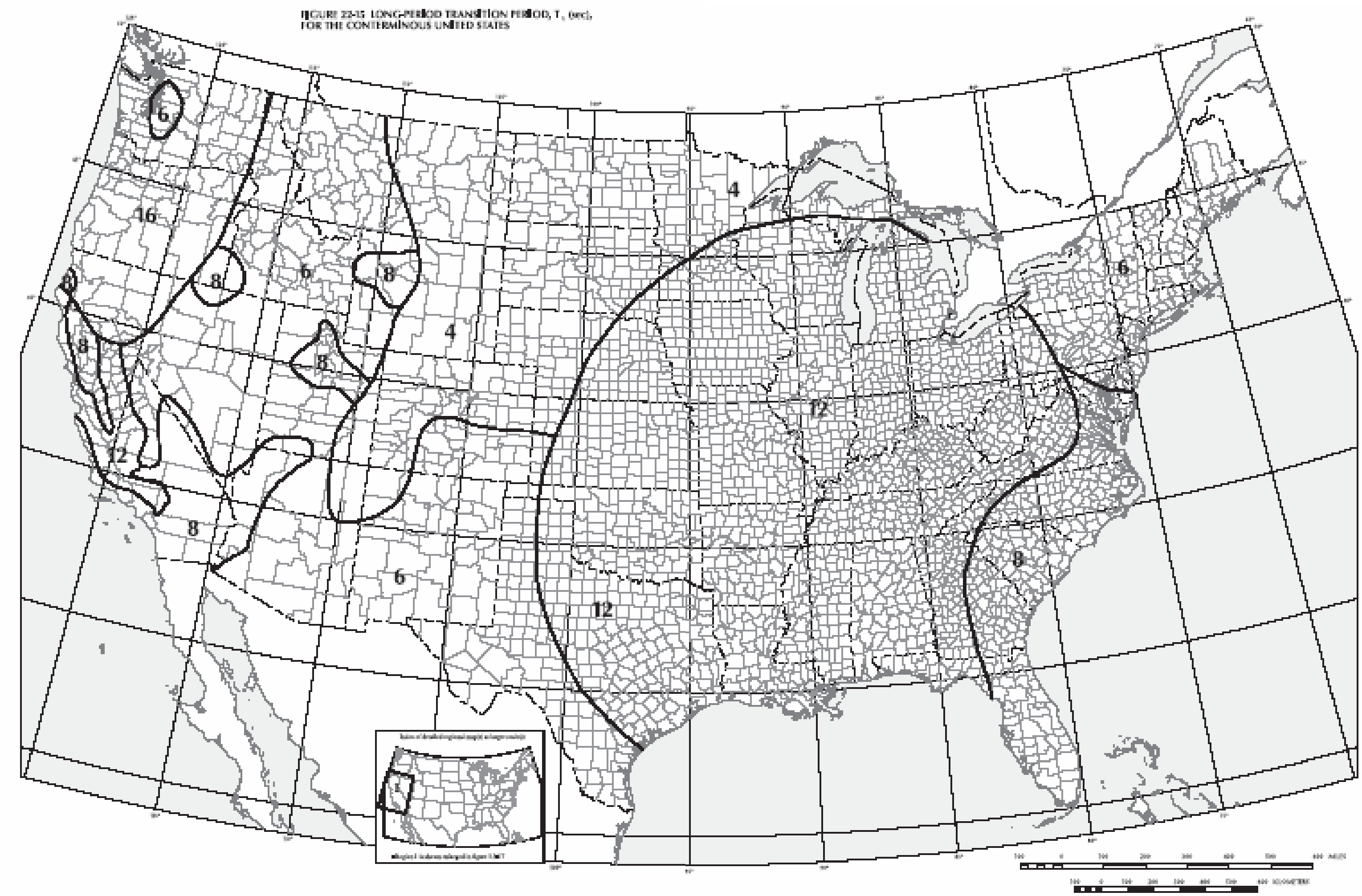
When the ELF procedure is not allowed, analysis must be performed by the response spectrum analysis procedure or by the linear (or nonlinear) response history analysis procedure.

Equivalent Lateral Force Procedure

Determine Base Shear: $V = C_S W$



Transition Periods for Conterminous United States



Effective Seismic Weight *W*

- All structural and nonstructural elements
- 10 psf <u>minimum</u> partition allowance
- 25% of storage live load
- Total weight of operating equipment
- 20% of snow load when “flat roof” snow load exceeds 30psf

Approximate Periods of Vibration

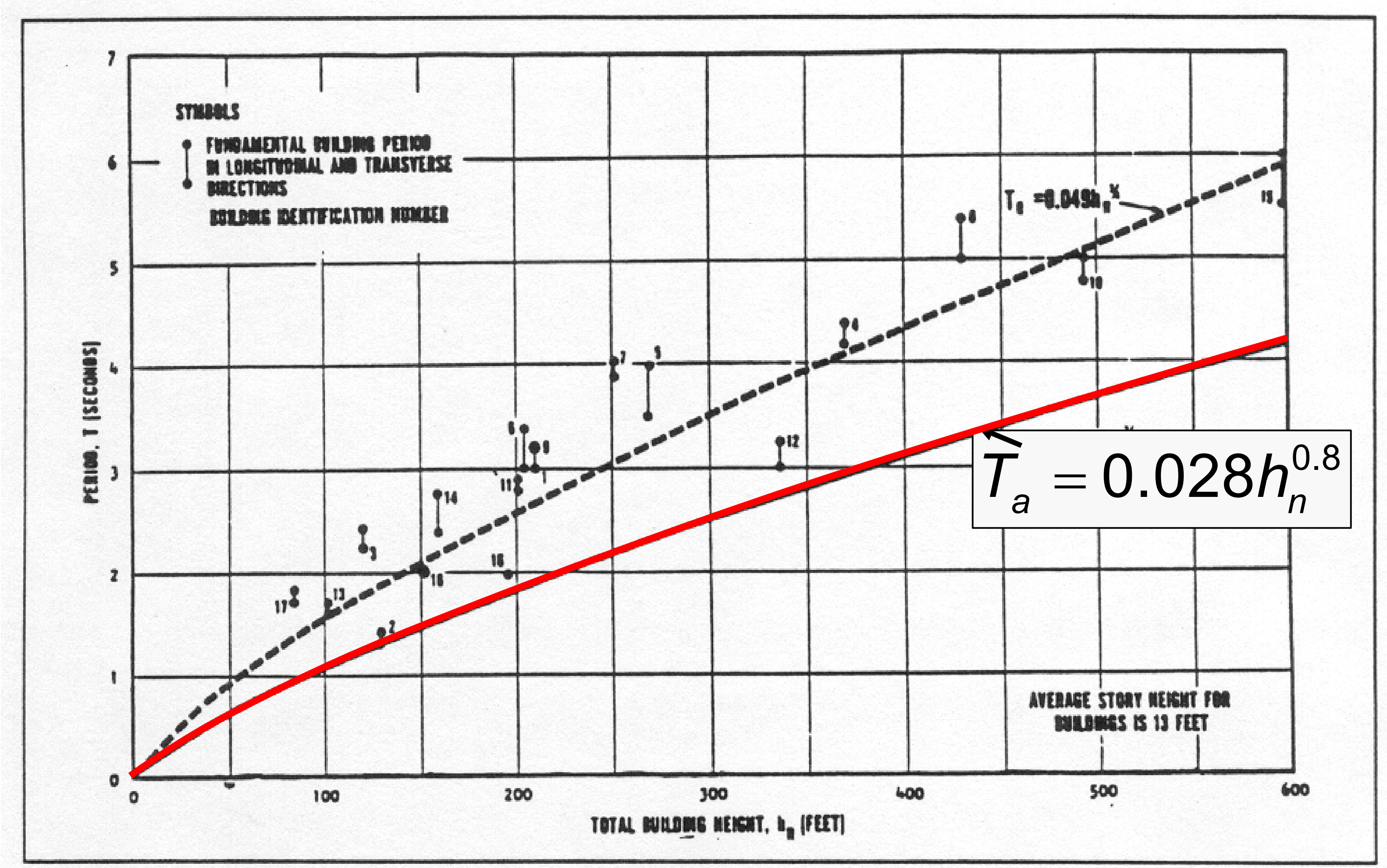
$$\boxed{T_a = C_t h_n^x}$$

$C_t = 0.028$, $x = 0.8$ for steel moment frames
$c_t = 0.016$, $x = 0.9$ for concrete moment frames
$c_t = 0.030$, $x = 0.75$ for eccentrically braced frames
$c_t = 0.020$, $x = 0.75$ for all other systems
Note: Buildings ONLY!

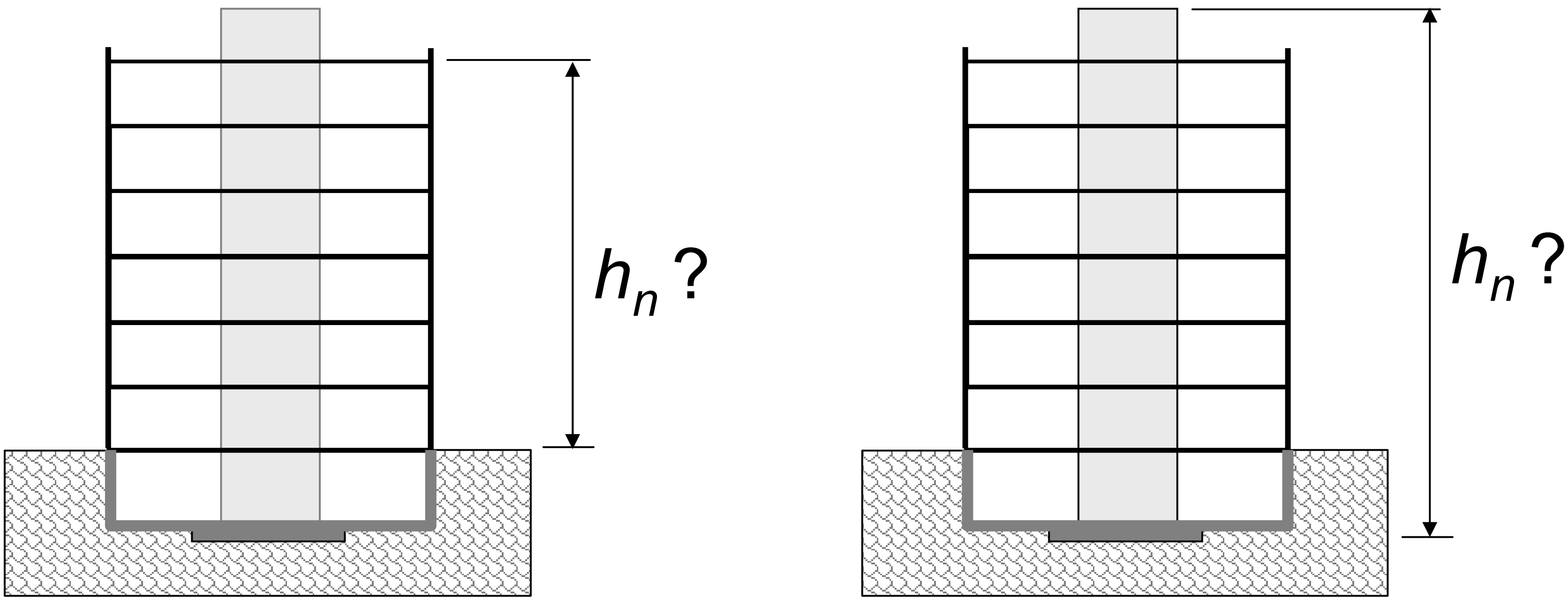
$$\boxed{T_a = 0.1N}$$

For moment frames < 12 stories in height, minimum story height of 10 feet. N = number of stories.

Empirical Data for Determination of Approximate Period for Steel Moment Frames



What to use as the “height above the base of the building?



When in doubt use the lower (reasonable) value of h_n

Adjustment Factor on Approximate Period

$$T = T_a C_u \leq T_{computed}$$

S_{D1}	$\boldsymbol{C_u}$
> 0.40g	1.4
0.30g	1.4
0.20g	1.5
0.15g	1.6
< 0.10g	1.7

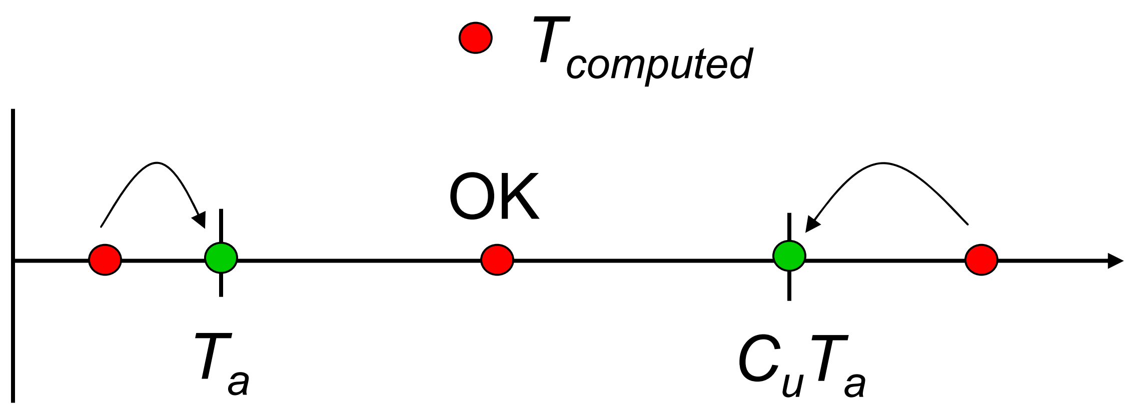
Applicable **ONLY** if $T_{computed}$ comes from a "properly substantiated analysis."

Decisions Regarding Appropriate Period to Use

if $T_{computed}$ is > $C_u T_a$ use $C_u T_a$

if $T_a < T_{computed} < C_u T_a$ use $T_{computed}$

if $T_{computed} < T_a$ use T_a

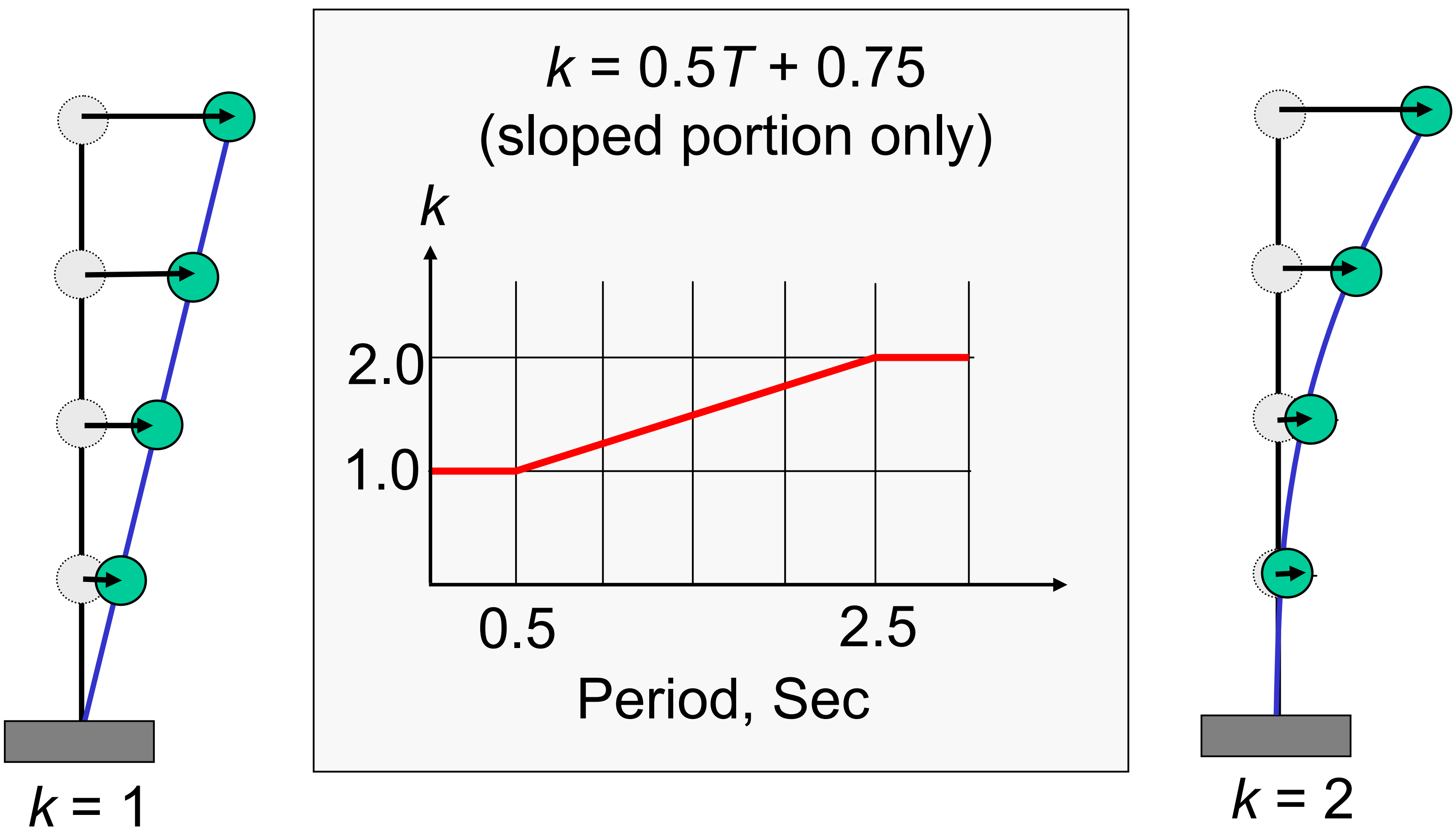


Distribution of Forces along Height

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

k accounts for Higher Mode Effects



Overturning

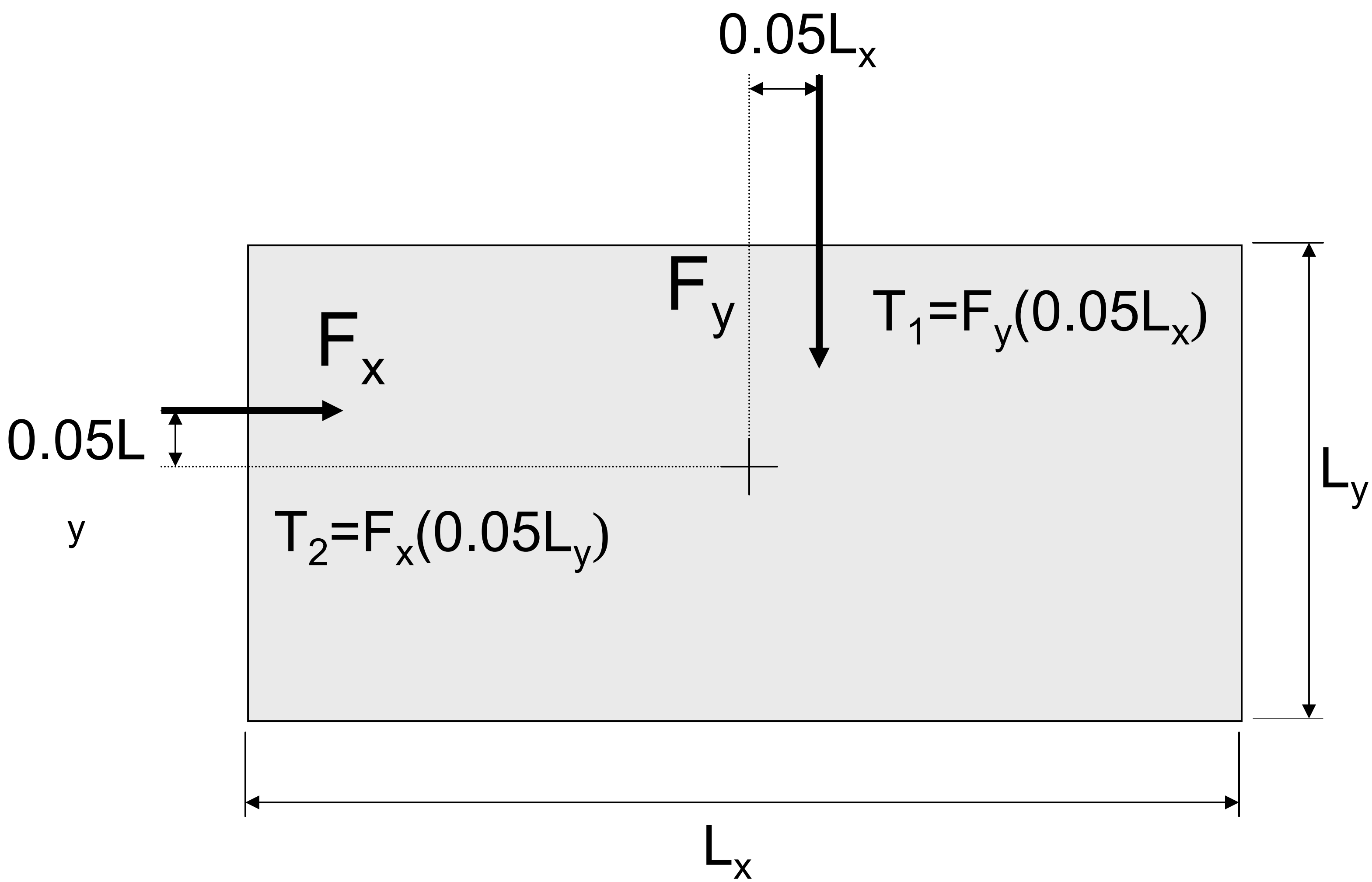
The 2003 *NEHRP Recommended Provisions* and ASCE 7-05 allow a 25% reduction at the foundation only.

No overturning reduction is allowed in the above grade portion of the structure.

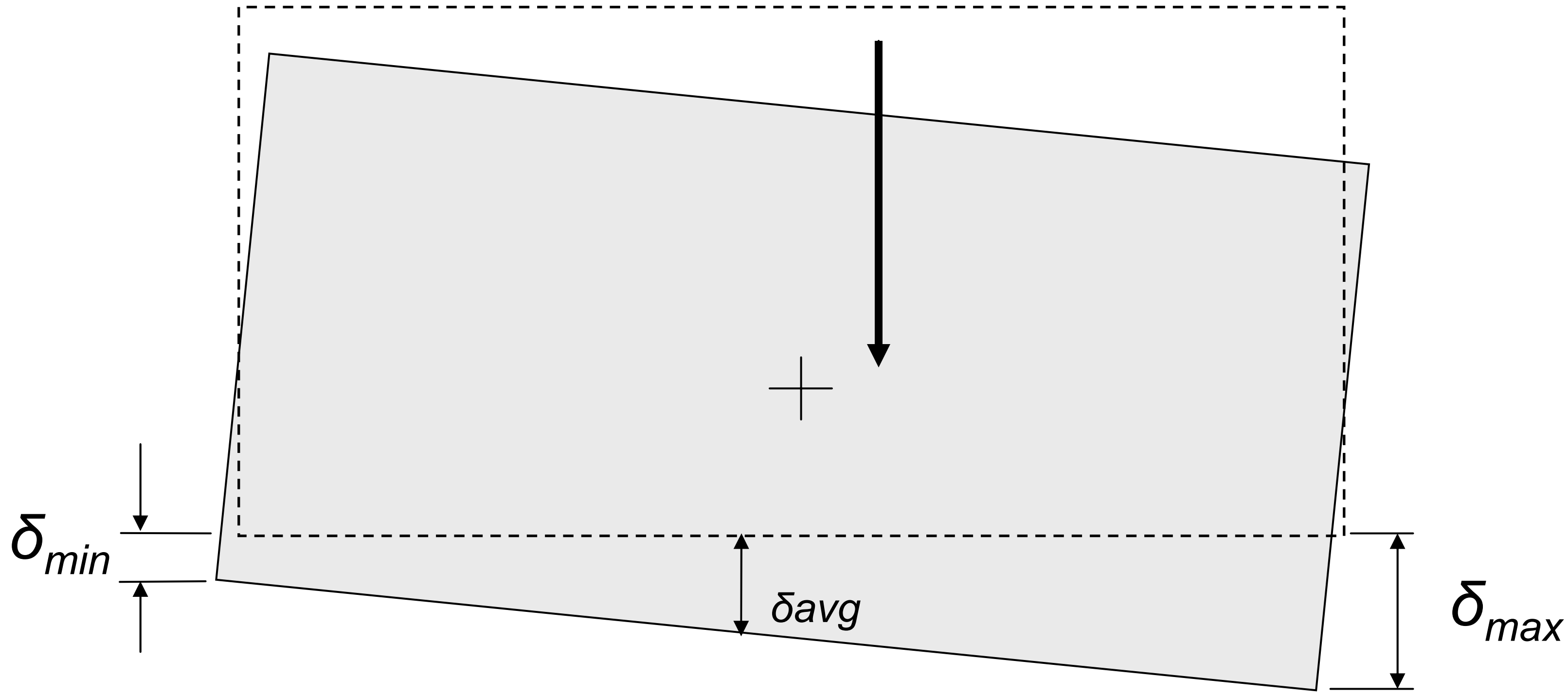
Torsional Effects

ALL	Include inherent and accidental torsion
B	Ignore torsional amplification
C, D, E, F	Include torsional amplification where Type 1a or 1b irregularity exists

Accidental Torsion

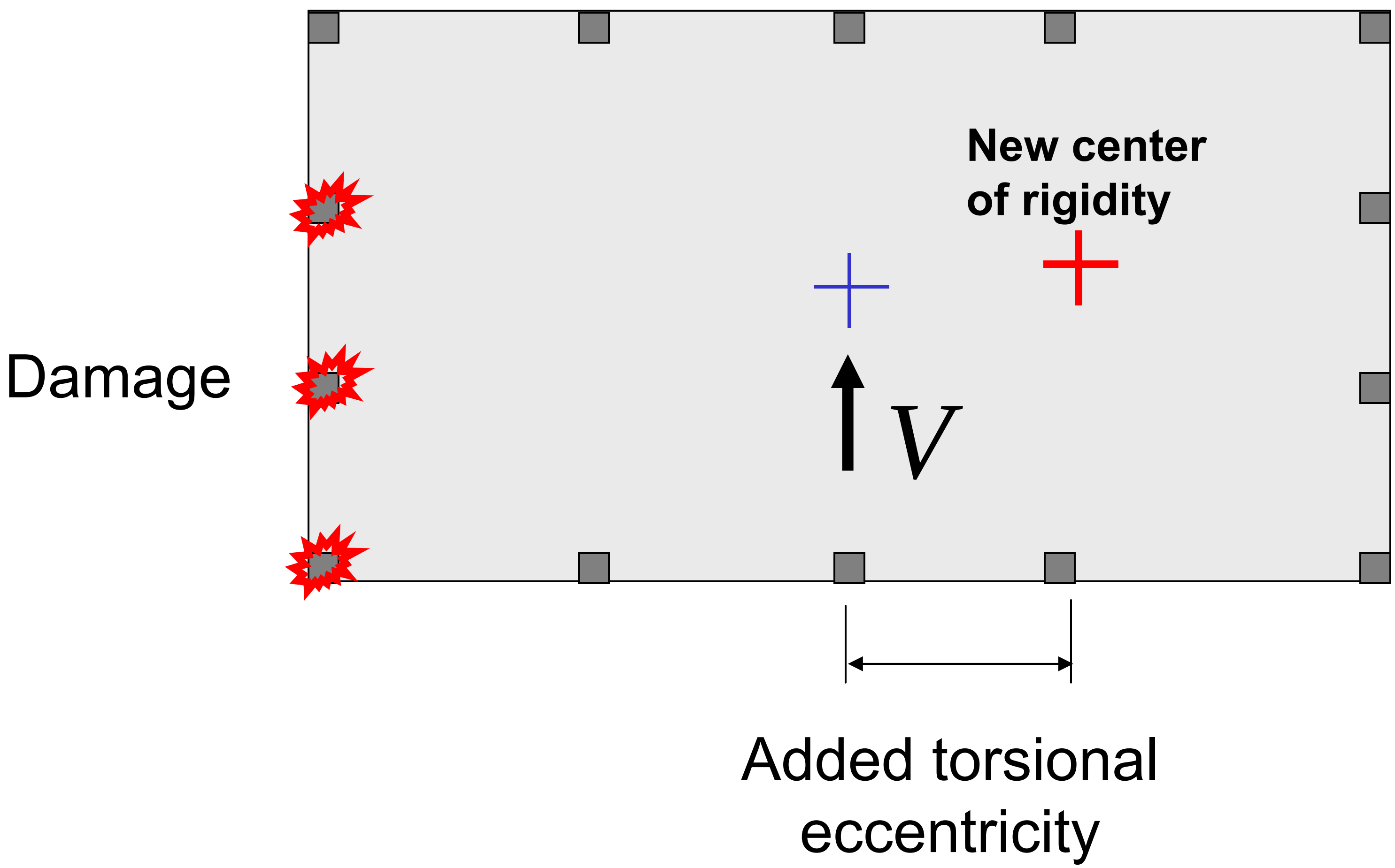


Amplification of Accidental Torsion

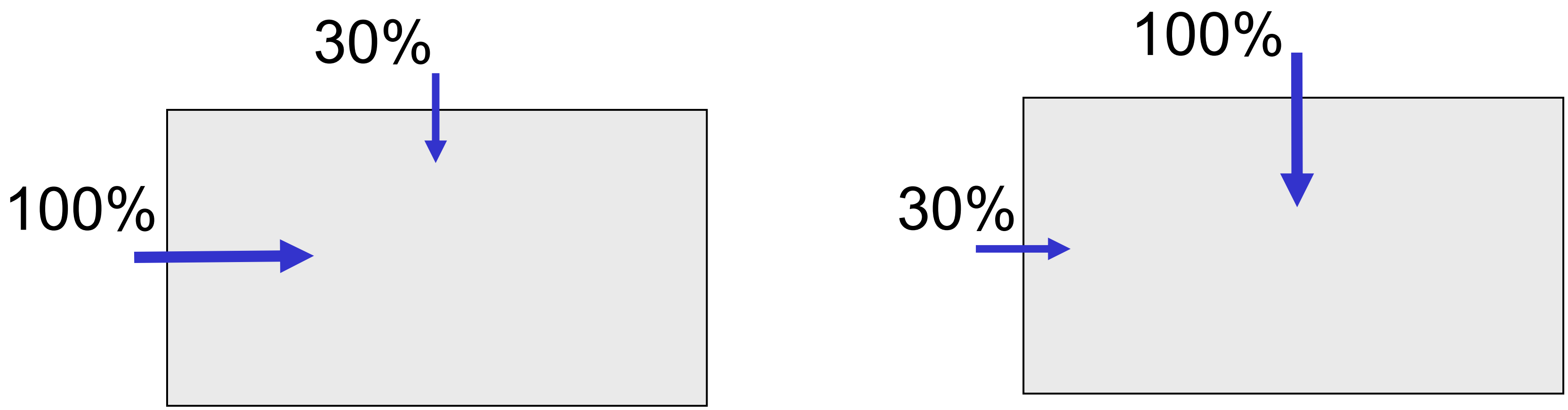


$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{avg}} \right)^2$$

Reason for Amplifying Accidental Torsion

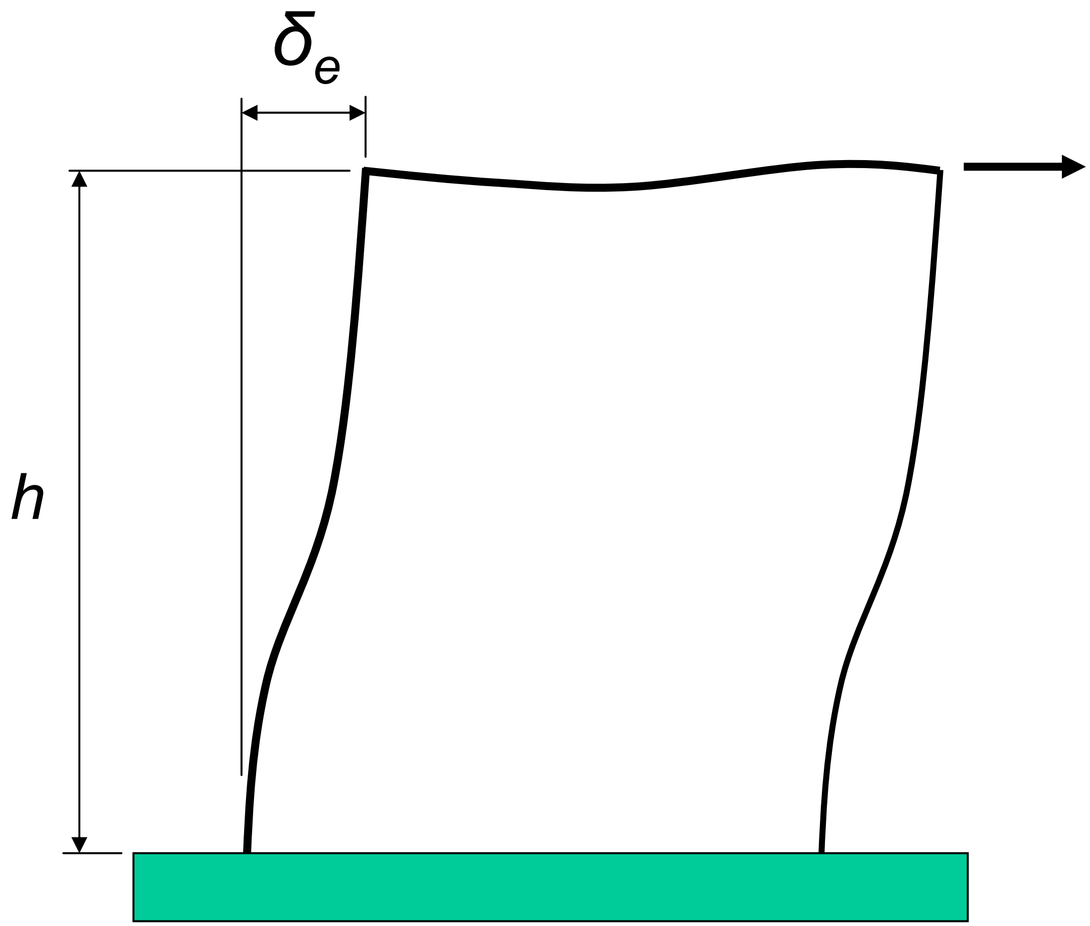


Orthogonal Load Effects



- Applicable to S.D.C. **C, D, E, and F**
- Affects primarily columns, particularly corner columns

Story Drift



Strength level forces modified by *R* and *I*

Drift reported by analysis with strength level forces:

$$\Delta_e = \frac{\delta_e / I}{h}$$

Amplified drift:

$$\Delta = C_d \Delta_e$$

Note: Drift computed at center of mass of story

Drift Limits

	Occupancy		
	I or II	III	IV
Structures other than masonry 4 stories or less with system Designed to accommodate drift	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures*	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

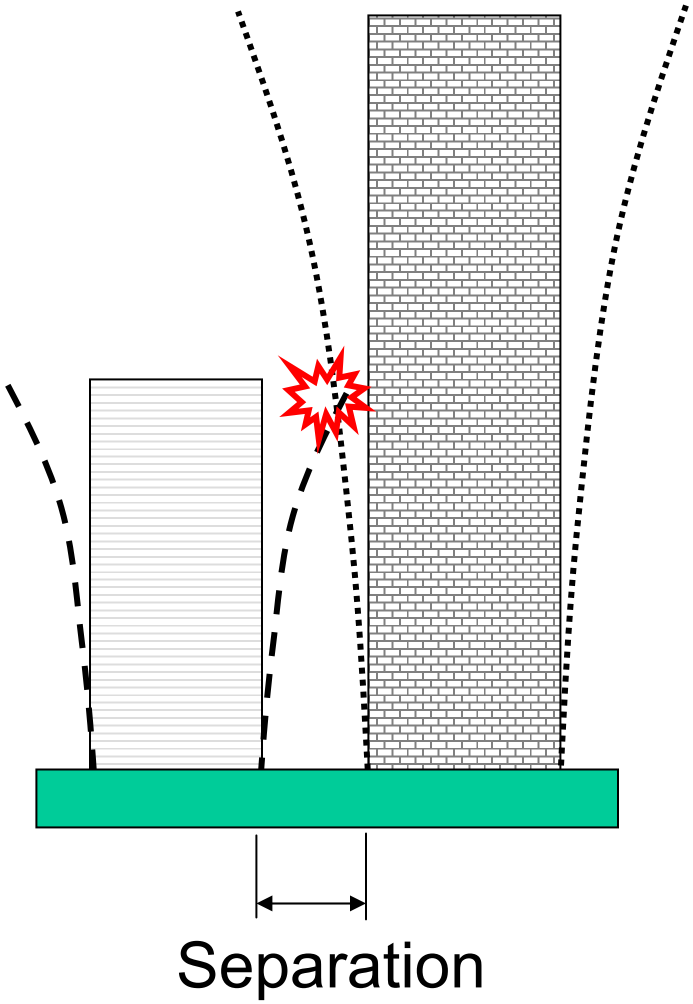
* For moment frames in SDC D, E, and F drift shall not exceed tabulated values divided by ρ.

Story Drift (continued)

For purposes of computing drift, seismic forces may be based on computed building period without upper limit $C_u T_a$.

For SDC C,D,E, and F buildings with torsional irregularities, drift must be checked at building edges.

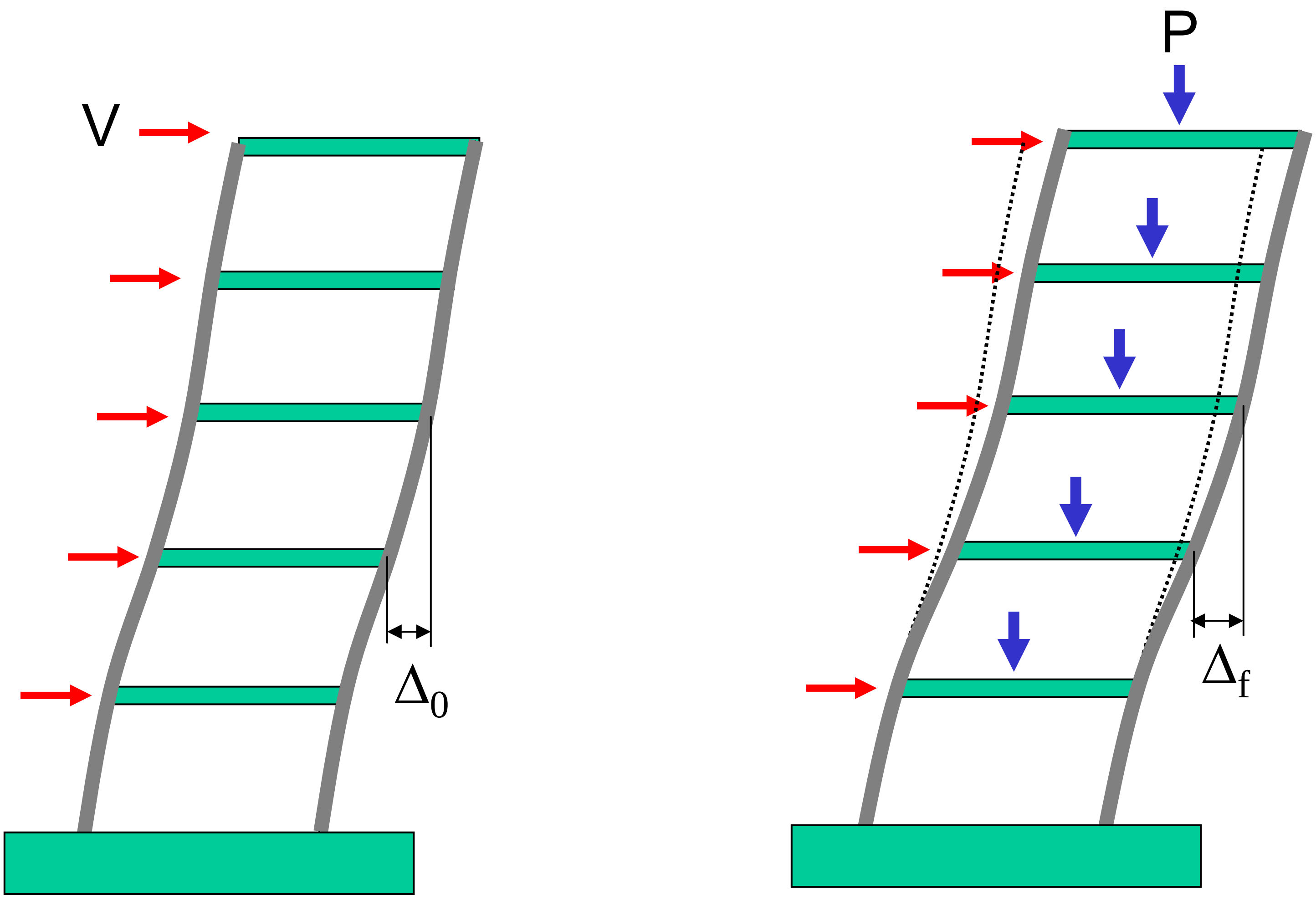
Building Separation to Avoid Pounding



Exterior damage to the back (north side) of Oviatt Library during Northridge Earthquake (attributed to pounding).

Source: http://library.csun.edu/mfinley/eqexdam1.html

P-Delta Effects



For elastic systems:

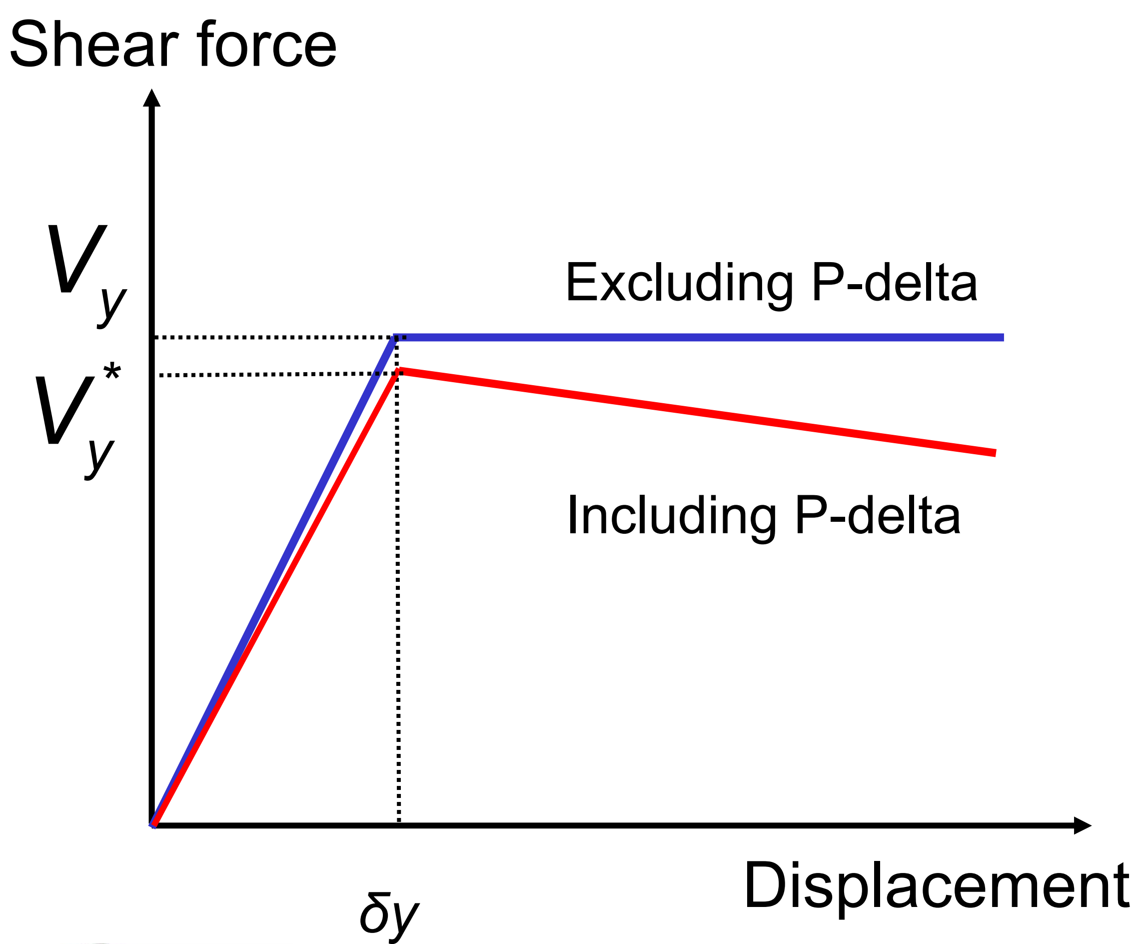
$$\Delta_f = \frac{\Delta_o}{1-\frac{P\Delta_o}{Vh}} = \frac{\Delta_o}{1-\theta}$$

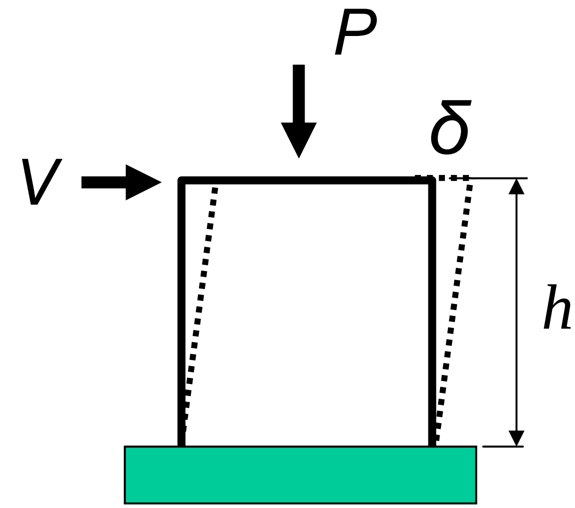
Δ_o = story drift in absence of gravity loads (excluding P-Δ)
Δ_f = story drift including gravity loads (including P-D)
P = total gravity load in story
V = total shear in story
h = story height

Θ is defined as the "story stability ratio"

For inelastic systems:

Reduced stiffness and increased displacements





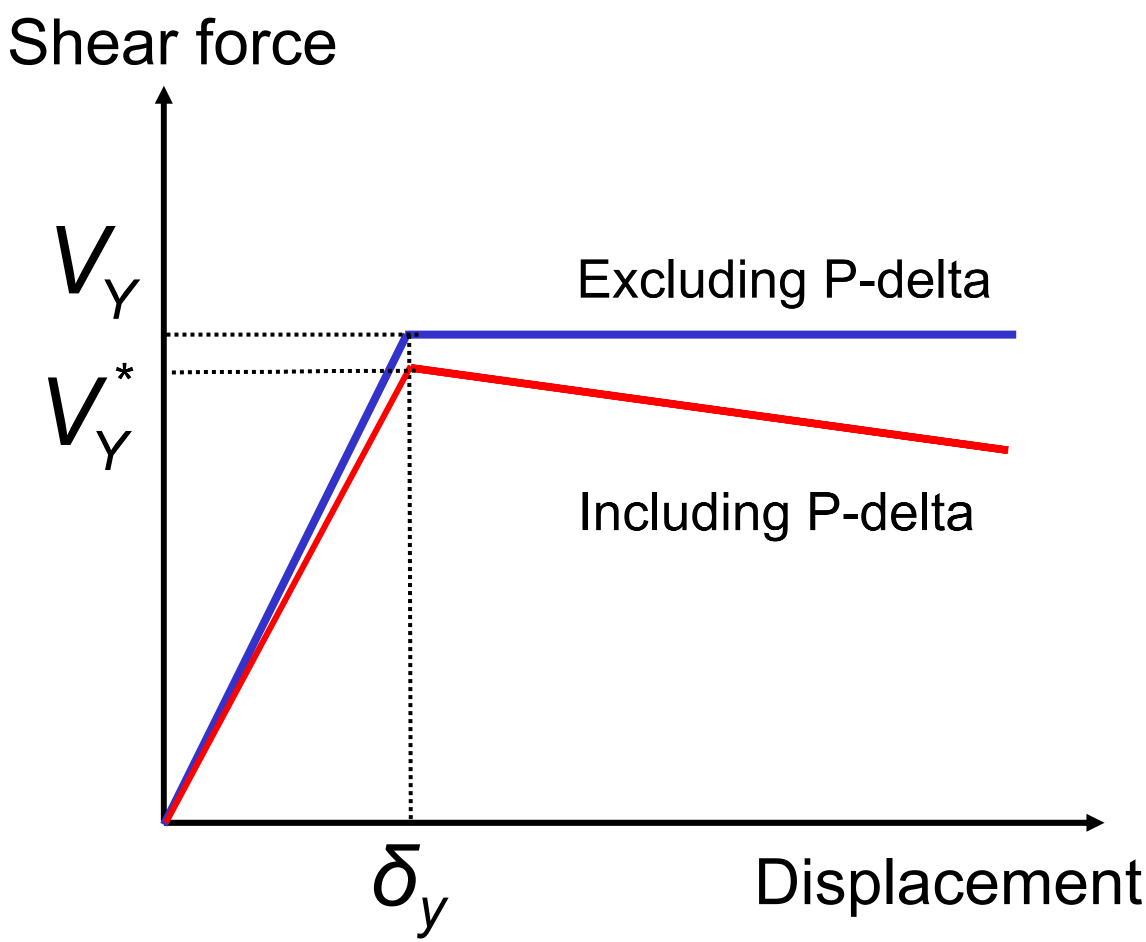
$$K_G = \frac{P}{h}$$

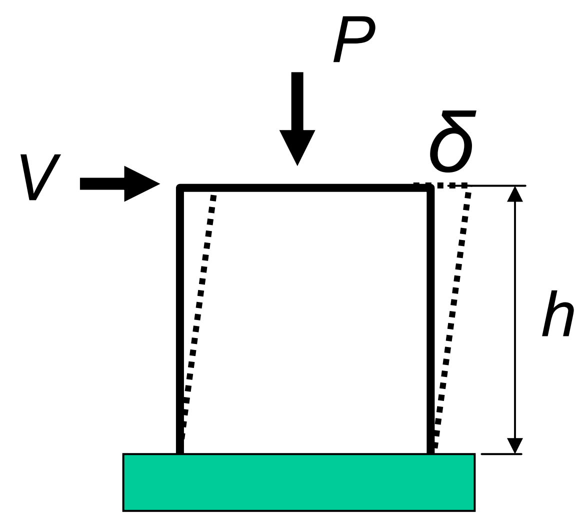
$$K_E = \frac{V_y}{\delta_y}$$

$$K = K_E - K_G$$

For inelastic systems:

Reduced strength

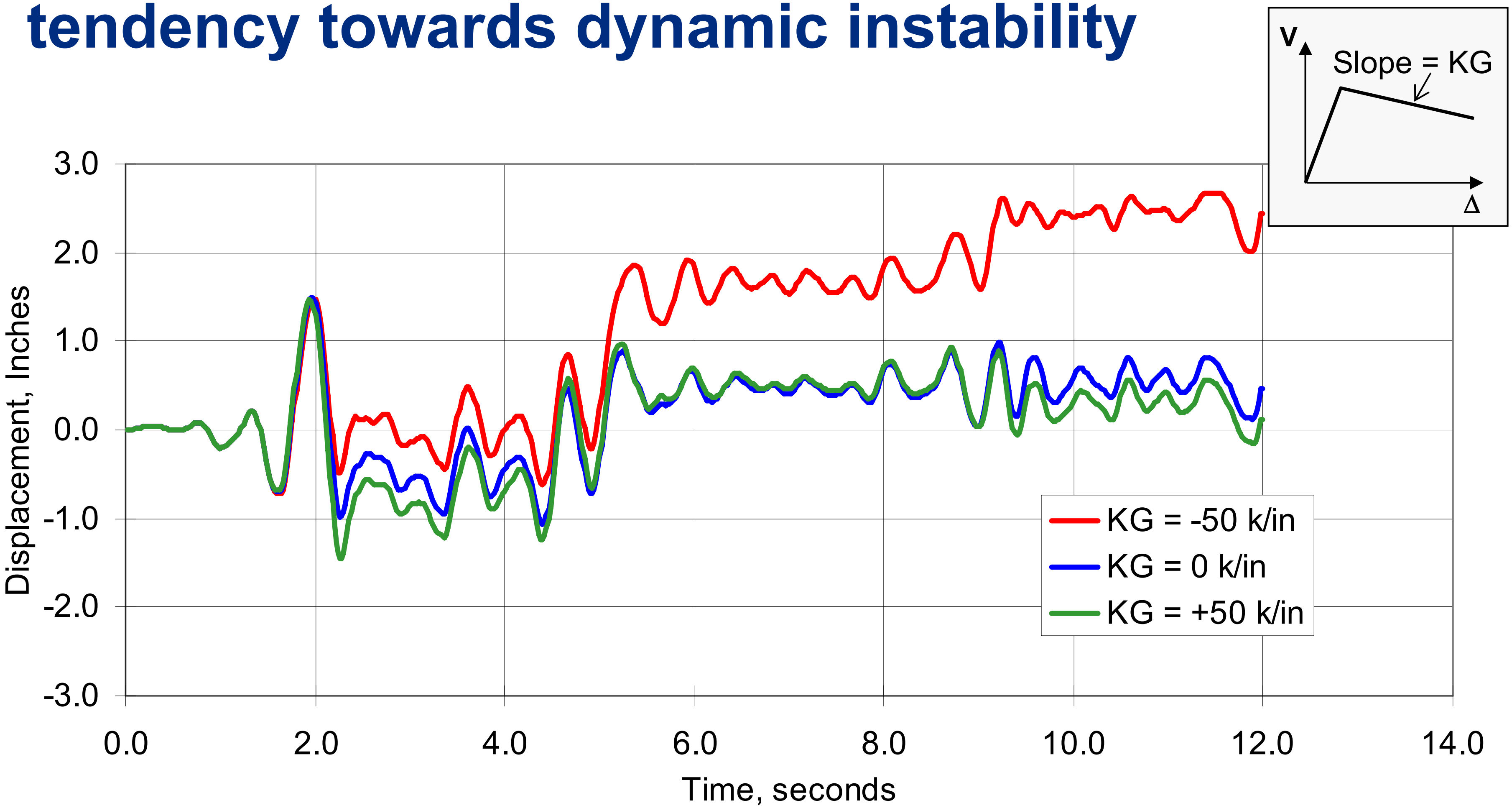




$$\theta = \frac{P\delta_y}{V_y h}$$

$$V_y^* = V_y(1-\theta)$$

For Inelastic Systems:
Larger residual deformations and increased tendency towards dynamic instability



P-Delta Effects

For each story compute:

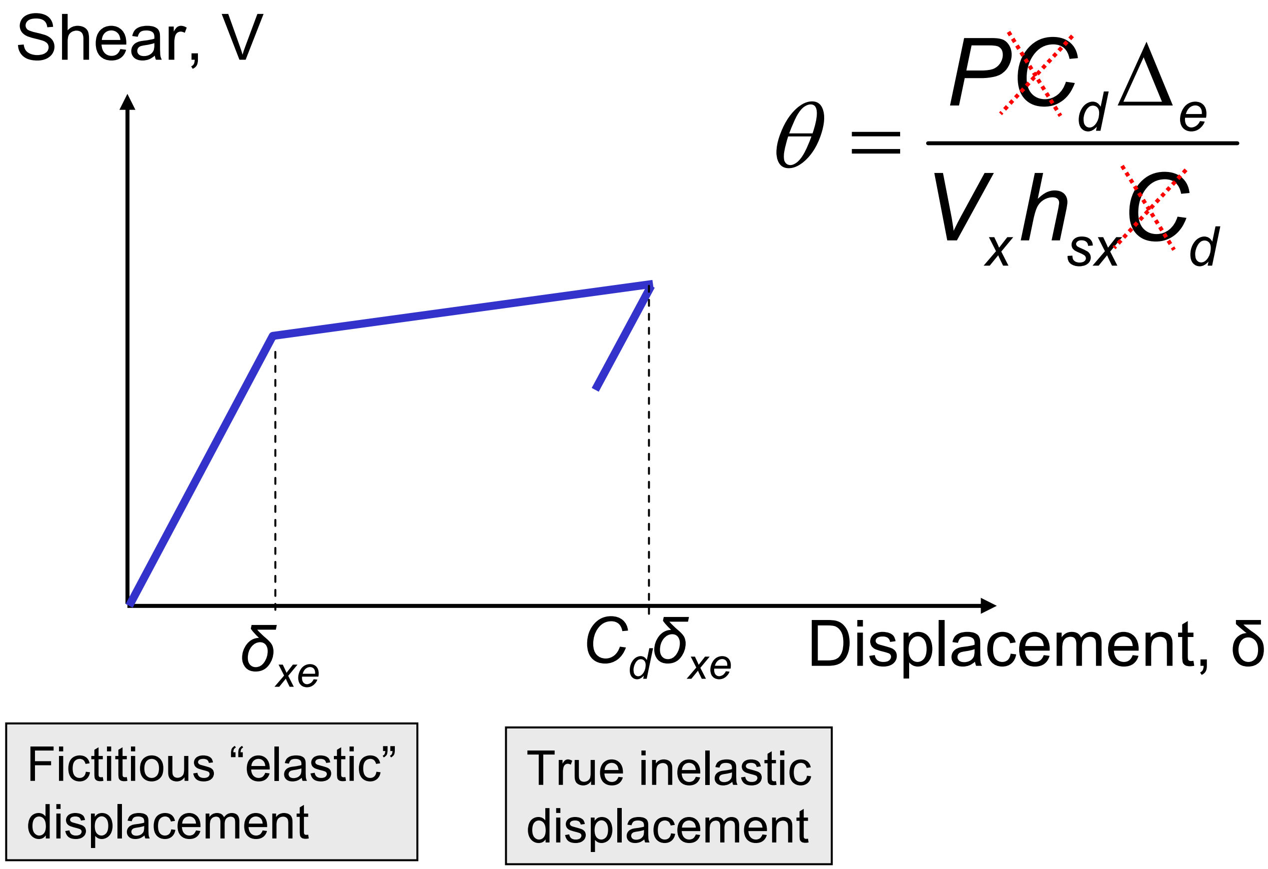
$$\theta = \frac{P\Delta}{V_x h_{sx} C_d}$$

P_x = total vertical design load at story above level x
Δ = computed story design level drift (including C_d)
V_x = total shear in story
h = story height

If $\Theta < 0.1$, ignore P-delta effects

P-Delta effects are based on the *Fictitious Elastic Displacements*

$$\theta = \frac{P\cancel{C}_d\Delta_e}{V_x h_{sx}\cancel{C}_d}$$



P-Delta Effects: ASCE 7-05 approach

If $\theta > 0.1$ then check

$$\theta_{\max} = \frac{0.5}{\beta C_d} < 0.25$$

where β is the ratio of the shear demand to the shear capacity of the story in question (effectively the inverse of the story overstrength). β may conservatively be taken as 1.0 [which gives, for example, $\Theta_{max} = 0.125$ when $C_d = 4$].

P-Delta Effects: ASCE 7-02 approach

If $\theta > 0.1$ and less than $\theta_{\max}$:

Multiply all computed element forces and displacements by:

$$a = \frac{1}{1-\theta}$$

- Check drift limits using amplified drift
- Design for amplified forces

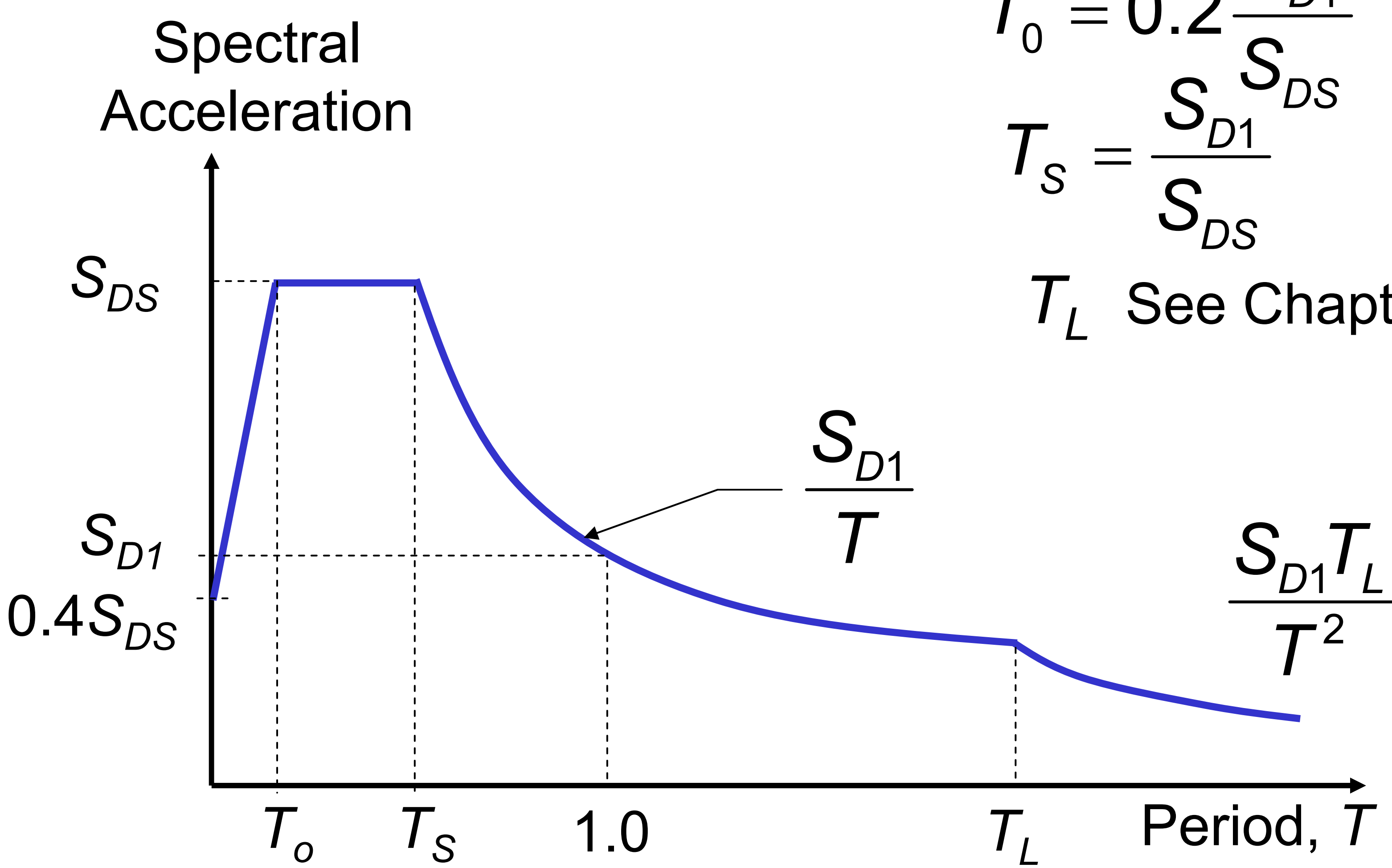
Note: P-delta effects may also be automatically included in the structural analysis. However, limit on θ still applies.

Modal Response Spectrum Analysis

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}}$$

$$T_S = \frac{S_{D1}}{S_{DS}}$$

T_L See Chapter 22



Note: Spectrum includes 5% damping

Basic Steps in
Modal Response Spectrum (RS) Analysis

1. Compute modal properties for each mode
   - Frequency (period)
   - Shape
   - Modal participation factor
   - Effective modal mass

2. Determine number of modes to use in analysis.
Use a sufficient number of modes to capture at least 90% of total mass in each direction

3. Using general spectrum (or compatible ground motion spectrum) compute spectral accelerations for each contributing mode.

Basic Steps in Modal RS Analysis (continued)

4. Multiply spectral accelerations by modal participation factor and by (*I*/*R*)

5. Compute modal displacements for each mode

6. Compute element forces in each mode

7. Statistically combine (SRSS or CQC) modal displacements to determine system displacements

8. Statistically combine (SRSS or CQC) component forces to determine design forces

Basic Steps in Modal RS Analysis (continued)

9. If the design base shear based on modal analysis is less than 85% of the base shear computed using ELF (and $T = T_a C_u$), the member forces resulting from the modal analysis and combination of modes must be scaled such that the base shear equals 0.85 times the ELF base shear.

10. Add accidental torsion as a *static loading* and amplify if necessary.

11. For determining drift, multiply the results of the modal analysis (including the I/R scaling but not the 85% scaling) by C_d/I.

Analytical Modeling for Modal Response Spectrum Analysis

- Use three-dimensional analysis
- **For concrete structures, include effect of cracking [req'd]**
- **For steel structures, include panel zone deformations [req'd]**
- Include flexibility of foundation if well enough defined
- Include actual flexibility of diaphragm if well enough defined
- Include P-delta effects in analysis if program has the capability
- Do not try to include accidental torsion by movement of center of mass
- Include orthogonal load effects by running the fill 100% spectrum in each direction, and then SRSSing the results.

Modal Response History Analysis:

uses the natural mode shapes to transform the coupled MDOF equations (with the nodal displacements as the unknowns) into several SDOF equations (with modal amplitudes as the unknowns). Once the modal amplitudes are determined, they are transformed back to nodal displacements, again using the natural mode shapes.

Coupled equations: $$\boxed{M\ddot{u} + C\dot{u} + Ku = -MR\ddot{u}_g}$$

Transformation: $$\boxed{u = \Phi y}$$

Uncoupled equations: $$\boxed{m_i^* \ddot{y}_i + c_i^* \dot{y}_i + k_i^* y_i = -\phi_i^T MR\ddot{u}_g}$$

Linear Response History Analysis:

Solves the coupled equations of motion directly, without use of natural mode shapes. Coupled equations are numerically integrated using one of several available techniques (e.g., Newmark linear acceleration). Requires explicit formation of system damping matrix *C*.

Coupled equations: $\boxed{M\ddot{u} + C\dot{u} + Ku = -MR\ddot{u}_g}$

Advantages of Modal Response History Analysis:

- Each SDOF equation may be solved exactly
- Explicit damping matrix *C* is not required (see below)
- Very good (approximate) solutions may be obtained using only a small subset of the natural modes

$$\ddot{y}_i + 2\xi_i\omega_i\dot{y}_i + \omega_i^2 y_i = -P_i\ddot{u}_g$$

Modal damping ratio

Modal frequency

Modal participation factor

Modal and Linear Response History Structural Modeling Procedures

- Follow procedures given in previous slides for modeling structure. When using modal response history analysis, use enough modes to capture 90% of the mass of the structure in each of the two orthogonal directions.

- Include accidental torsion (and amplification, if necessary) as additional static load conditions.

- Perform orthogonal loading by applying the full recorded orthogonal horizontal ground motion simultaneous with the principal direction motion.

ASCE 7-05 Ground Motion Selection

- Ground motions must have magnitude, fault mechanism, and fault distance consistent with the site and must be representative of the *maximum considered ground motion*

- Where the required number of motions are not available simulated motions (or modified motions) may be used

(Parenthesis by F. Charney)

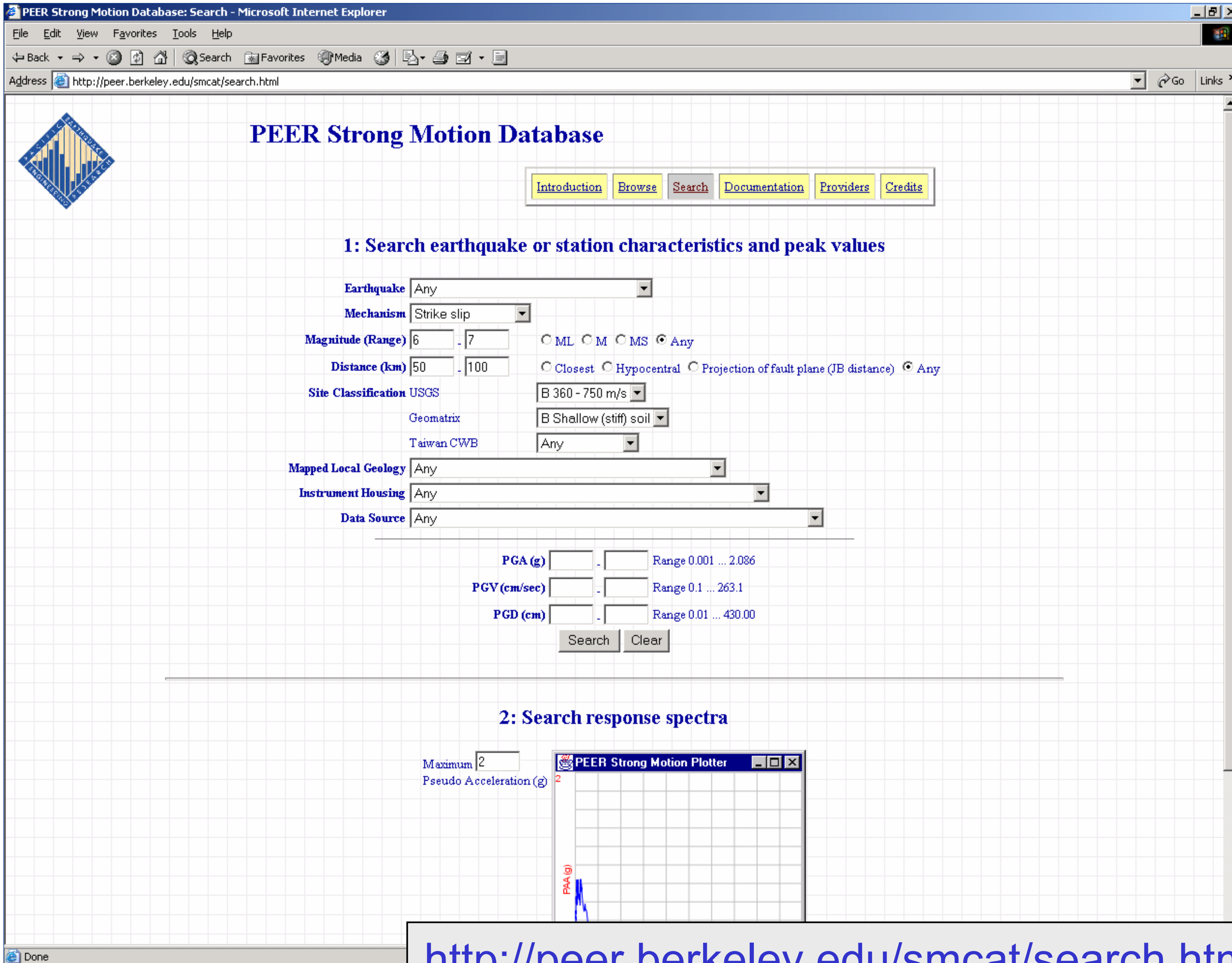
How many records should be used?
Where does one get the records?
How are ground motions scaled?

How Many Records to Use?

2003 *NEHRP Recommended Provisions* and ASCE 7-05:

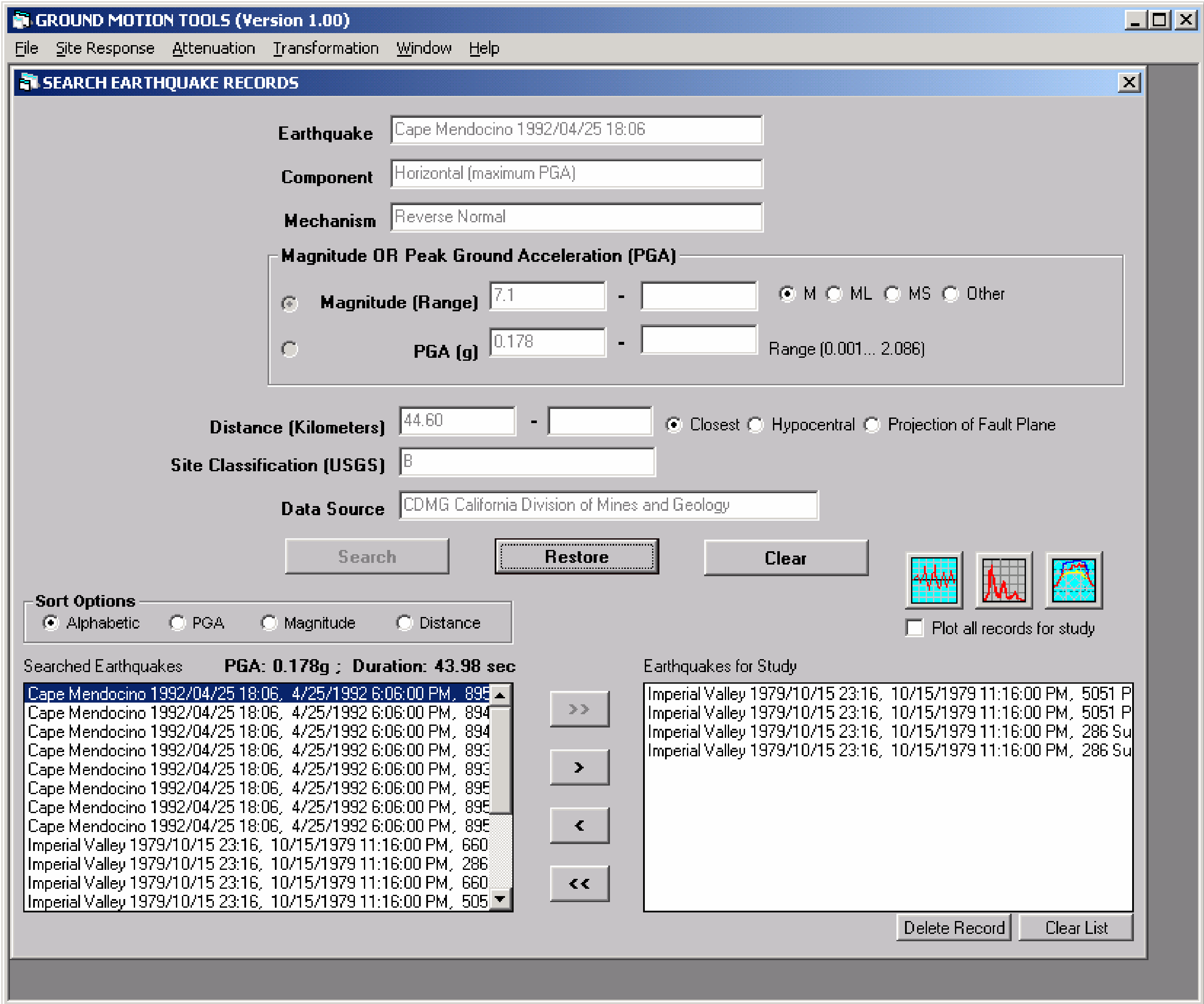
A suite of not less than three motions shall be used.

Ground Motion Sources: PEER



http://peer.berkeley.edu/smcat/search.html

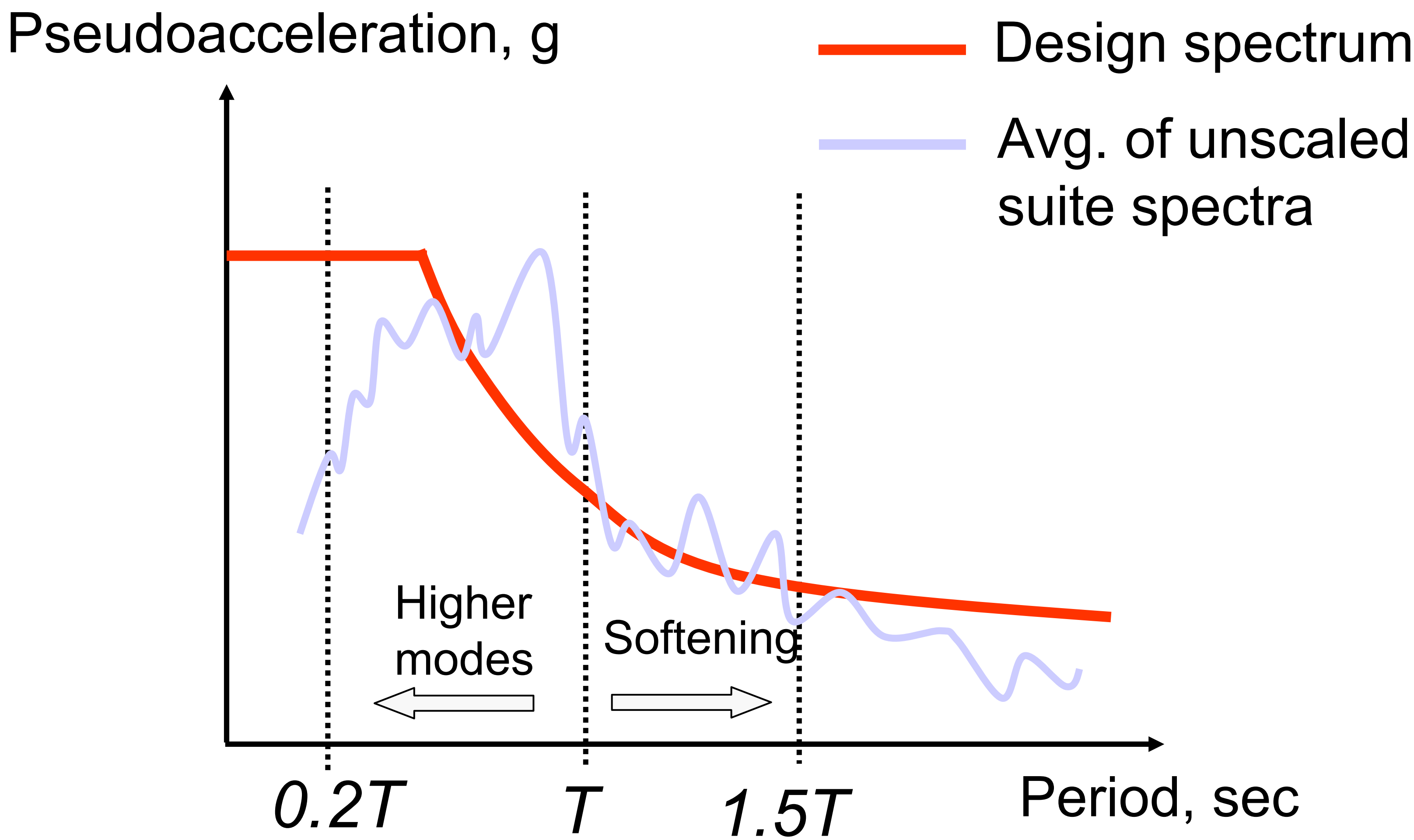
Ground Motion Sources: EQTools



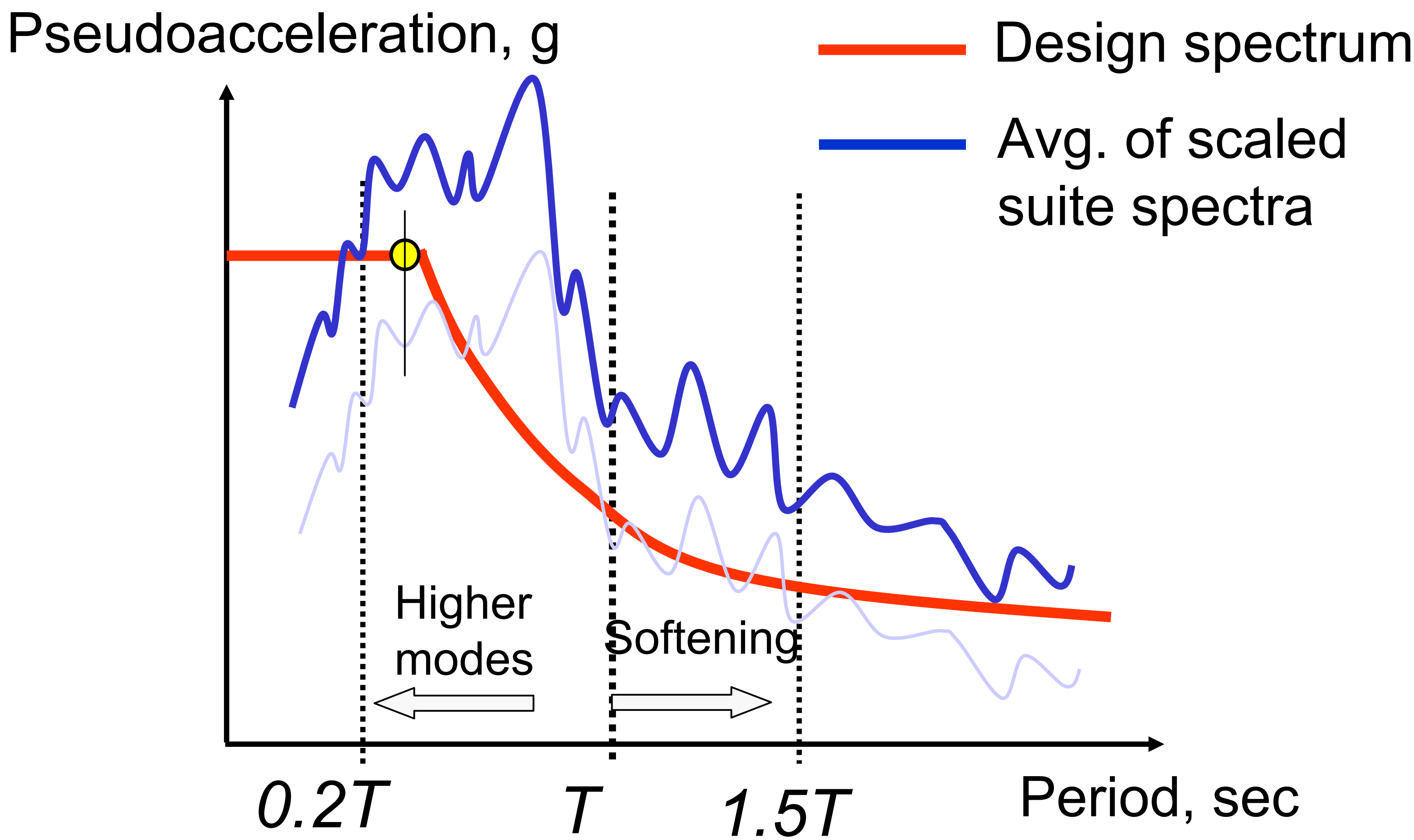
Ground Motion Scaling

Ground motions must be scaled such that the average value of the 5% damped response spectra of the suite of motions is not less than the design response spectrum in the period range $0.2T$ to $1.5T$, where T is the fundamental period of the structure.

Scaling for 2-D Analysis



Scaling for 2-D Analysis



Ground Motion Selection and Scaling

1. The square root of the sum of the squares of the 5% damped spectra of each motion pair (N-S and E-W components) is constructed.

2. Each pair of motions should be scaled such that the average of the SRSS spectra of all component pairs is not less than 1.3 times the the 5% damped design spectrum in the period range 0.2 to 1.5 T.

Potential Problems with Scaling

- A degree of freedom exists in selection of individual motion scale factors, thus different analysts may scale the same suite differently.
- The scaling approach seems overly weighted towards higher modes.
- The scaling approach seems to be excessively conservative when compared to other recommendations (e.g., Shome and Cornell)

Recommendations:

- Use a minimum of seven ground motions
- If near-field effects are possible for the site a separate set of analyses should be performed using only near field motions
- Try to use motions that are magnitude compatible with the design earthquake
- Scale the earthquakes such that they match the target spectrum at the structure's initial (undamaged) natural frequency and at a damping of at least 5% critical.

Response Parameters for Linear Response History Analysis

For each (scaled) ground motion analyzed, all computed response parameters must be multiplied by the appropriate ratio (I/R). Based on these results, the maximum base shear is computed.

The ratio of the maximum base shear to total weight for the structure must not be less than the following:

$$V/W = 0.01$$ for SDC A through D

$$V/W = \frac{0.5S_1}{R/I}$$ for SDC E and F when S_1 > 0.g

ASCE 7-02 Response Parameters for Linear Response History Analysis (continued)

If at least seven ground motions are used, response quantities for component design and story drift may be based on the *average* quantity computed for all ground motions.

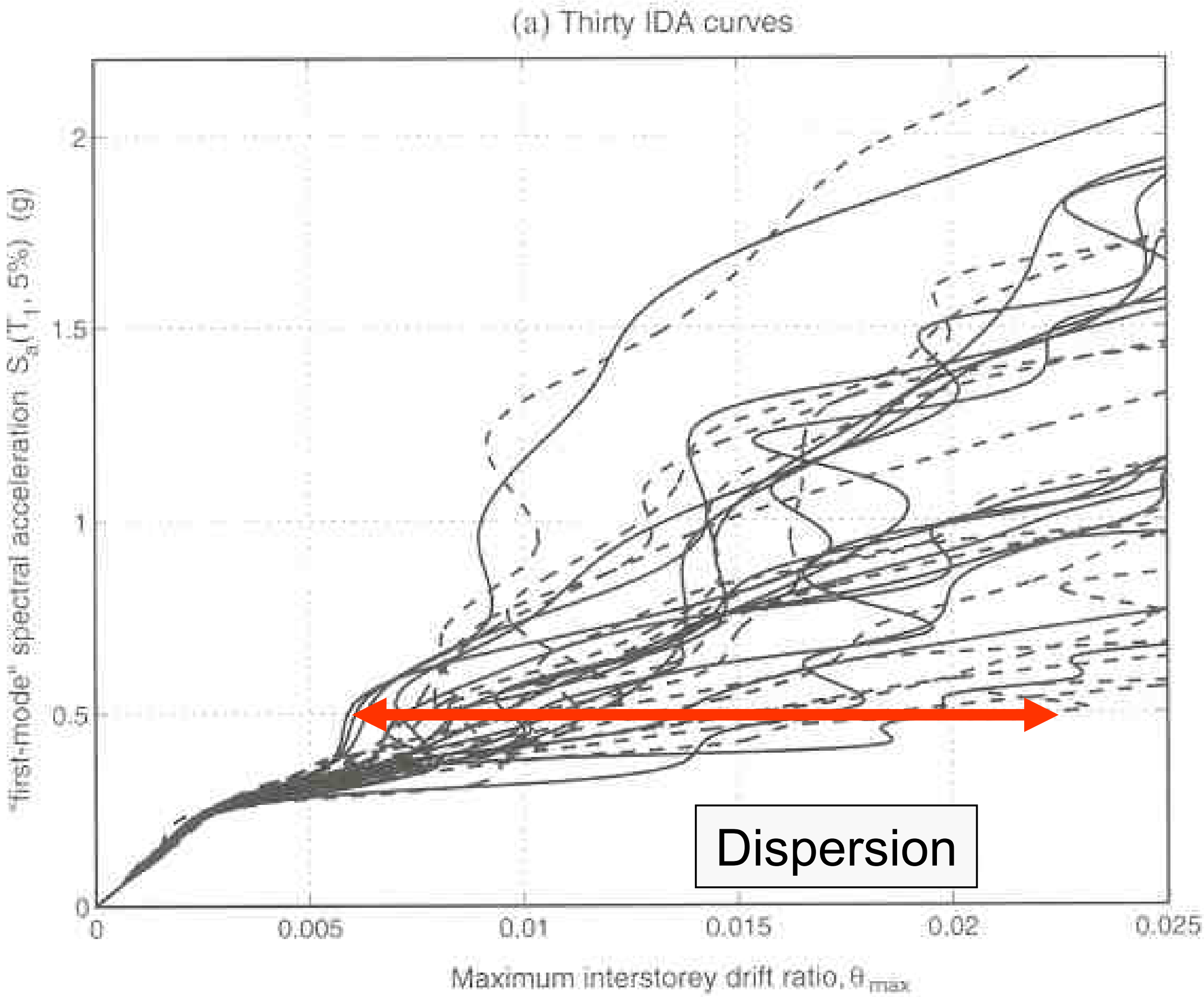
If less than seven ground motions are used, response quantities for component design and story drift must be based on the *maximum* quantity computed among all ground motions.

Nonlinear Response History Analysis is an Advanced Topic and in not covered herein.

Due to effort required, it will typically not be used except for very critical structures, or for structures which incorporate seismic isolation or passive, semi-active, or active control devices.

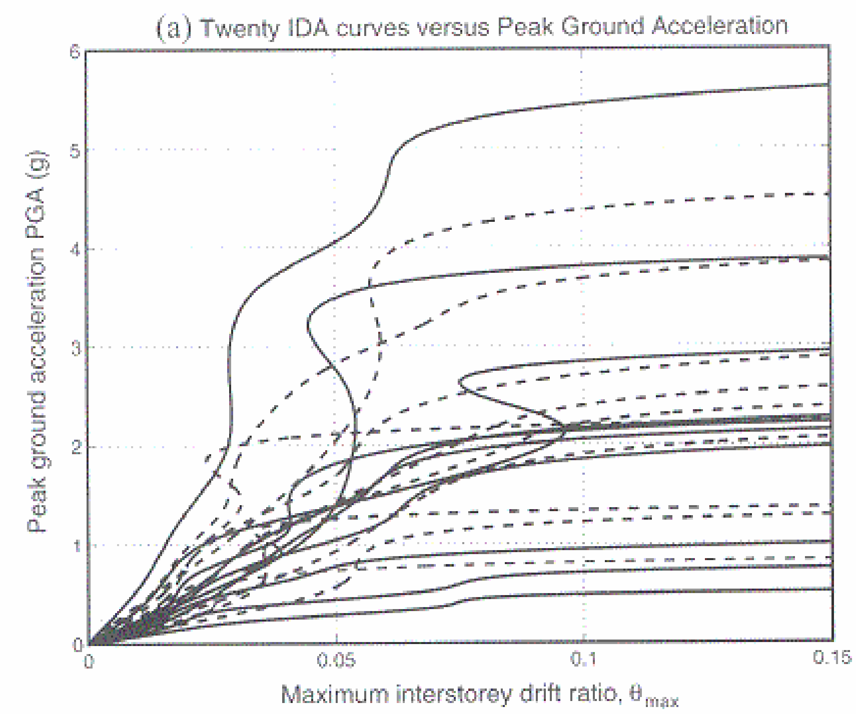
The principal difficulty with nonlinear response history analysis (aside from the effort required) are the sensitivities of the computed response due to a host of uncertainties.
Such sensitivities are exposed by a systematic analysis approach called incremental dynamic analysis.

A Family of IDA Curves of the Same Building Subjected to 30 Earthquakes [exposing effect of ground motion uncertainty]

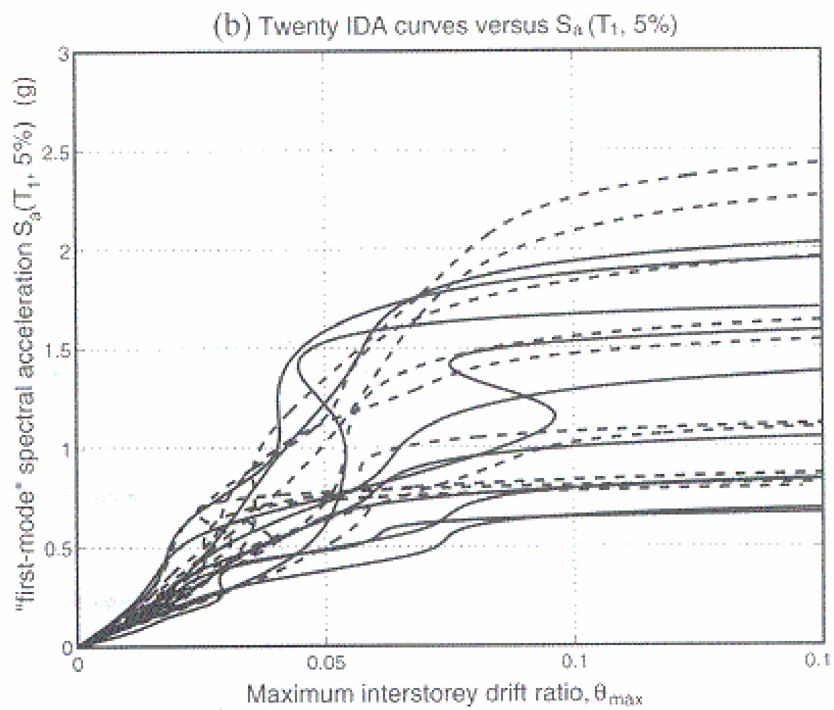


IDA Curves of the Same Building Subjected to Suite of Earthquakes Where Different Scaling Methods Have Been Used

NORMALIZED to PGA



NORMALIZED to S_a



Methods of Analysis Described in ASCE 7-05

Nonlinear static pushover analysis