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Performance-based Seismic Design of Structures

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Dr. Naveed Anwar

Dr. Pennung Warnitchai

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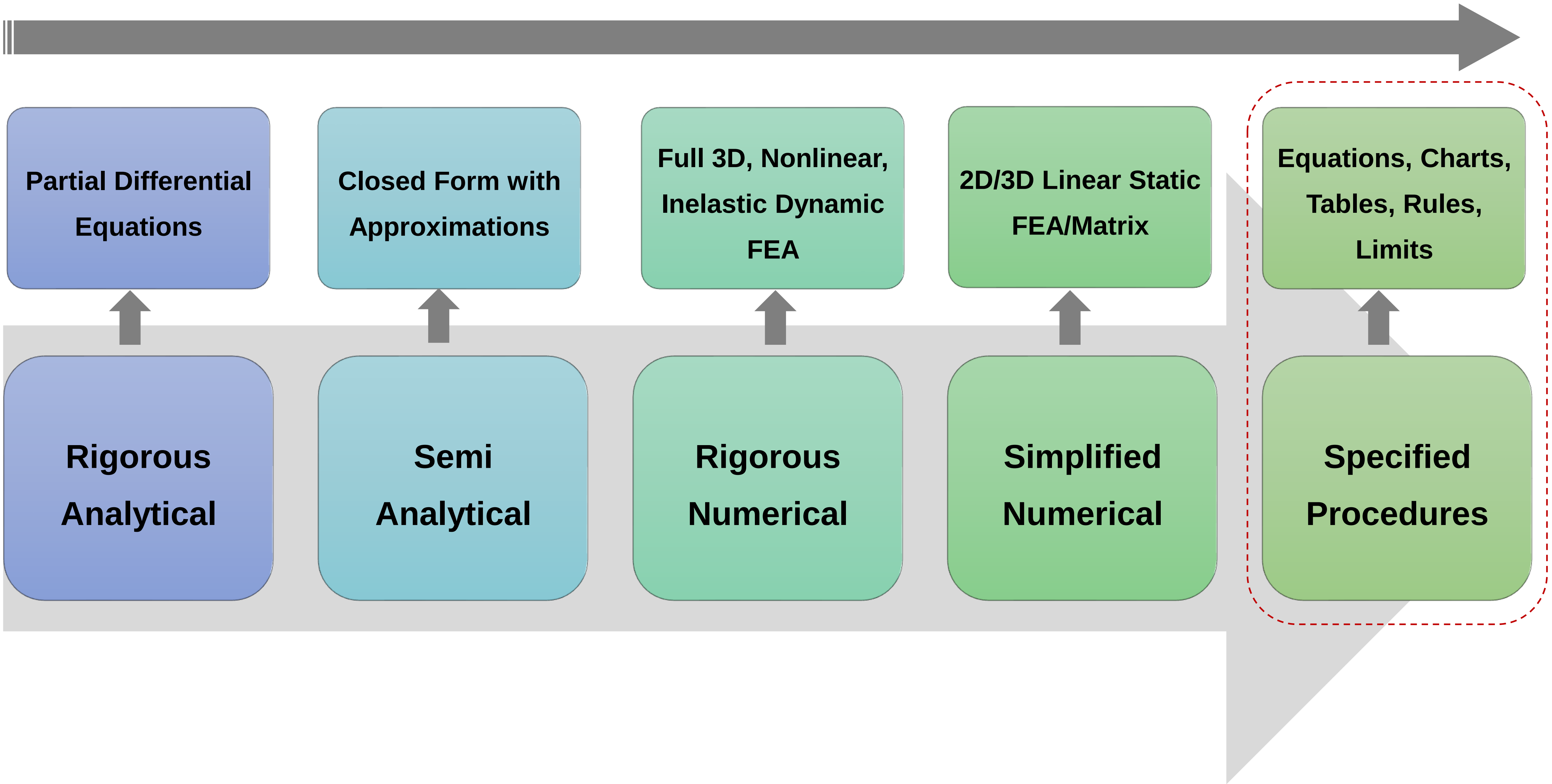
  - Training material developed by Mr. Thaung Htut Aung at AIT Solutions, AIT, Thailand
  - Various International Codes and Guidelines
  - Online Training Material from US Geological Survey (USGS)
  - Lectures of Dr. Punchet Thammarak at Asian Institute of Technology (AIT), Thailand
  - Online Educational Resources from IRIS (www.iris.edu)
  - Class Notes of Prof. Dr. Worsak Kanok-Nukulchai at Asian Institute of Technology (AIT), Thailand

- The material is taken solely for educational purposes. **All sources are duly acknowledged.**

Lecture 4(a): Code-based Seismic Design of Structures

- The Development of Seismic Design Codes and the Equivalent Static Analysis Procedure
  - The ELF Procedure in UBC 97
  - The ELF Procedure in IBC 2000
  - The Seismic Design Procedure in BCP (2007)
- The Concept of Vibration Modes of a Structure
- The Response Spectrum Analysis Procedure
  - The Concept of Response Spectrum
  - The Elastic Response Spectrum Analysis (RSA) Procedure
  - The Concept of Inelastic Response Spectra and Design Spectra
  - The Code-based Response Spectrum Analysis (RSA) Procedure
- The Linear Time History Analysis Procedure
- Combining Responses for Member Design
- The ELF and RSA Procedures as prescribed in ASCE 7-16 – Software Demonstration


Partial Differential Equations
Closed Form with Approximations
Full 3D, Nonlinear, Inelastic Dynamic FEA
2D/3D Linear Static FEA/Matrix
Equations, Charts, Tables, Rules, Limits
Rigorous Analytical
Semi Analytical
Rigorous Numerical
Simplified Numerical
Specified Procedures

Seismic Analysis Procedures

Next Lecture 4(b)

Structural Model / Seismic Loading	**Linear** E, A, I, L, G etc. = Constant, K= Constant	**Nonlinear** E ≠ Constant, EI ≠ Constant, K≠ Constant
Static	**1.** Equivalent Lateral Force (ELF) Procedure **2.** Response Spectrum Analysis (RSA) Procedure	**4.** NSPs (Several Pushover Analysis Methods)
Dynamic	**3.** Linear Response History Analysis Procedure (LTHA, DI, LDP)	**5.** Nonlinear Response History Analysis Procedure (NLRHA) NLTHA, NDP, DI

This Lecture 4(a)

SEISMIC DESIGN CODES

There are several design methods suggested in seismic codes:

- **Equivalent static analysis (ESF) method (or ELF method)**
- **Response spectrum analysis (RSA) method**
- **Time history analysis method**

Equivalent static analysis method is commonly used for the seismic design of ordinary and "regular" buildings and structures.

SEISMIC DESIGN CODES

Dynamic analysis methods (Response spectrum and Time history) are required for "irregular" buildings and structures, very important structures, and structures that seismic response is not dominated by the fundamental vibration mode. ... more tedious, more difficult.

Irregularity = Irregular geometry, non-uniform distribution of mass or stiffness, structural discontinuity, etc.

- Vertical Structural Irregularities
- Plan Structural Irregularities

The Development of Seismic Design Codes and the Equivalent Static Analysis Procedure

DEVELOPMENT OF SEISMIC DESIGN CODES

1906 ***The great San Francisco Earthquake***

Intensive destruction of buildings by the earthquake and fire (that was induced by the earthquake).

The city was rebuilt.

The only **lateral force requirement** placed on structures designed and constructed in San Francisco was a $\boldsymbol{30\, lbf/ft^2}$ **wind loading**. This was intended to safeguard structures from the effect of both wind and earthquakes.

Lateral force is proportional to the surface **area** of the building.

DEVELOPMENT OF SEISMIC DESIGN CODES

1927 Introduction of lateral earthquake force proportional to **mass**.

$$\boldsymbol{V_B} \cong \mathbf{0.1}\, \boldsymbol{W} \text{ (an empirical formula)}$$

where, V_B is the base-shear and W is the weight of the building.

The Basic Notion

Convert the Seismic Excitation to an "Equivalent Static Force" applied at the base of the building, called the Base Shear. Then Distribute the Base Shear to various parts of the Building

1940 The El Centro Earthquake

The first set of strong motion records (acceleration time histories) ... the first set of elastic response spectra ... Period-dependent elastic base shear demand.

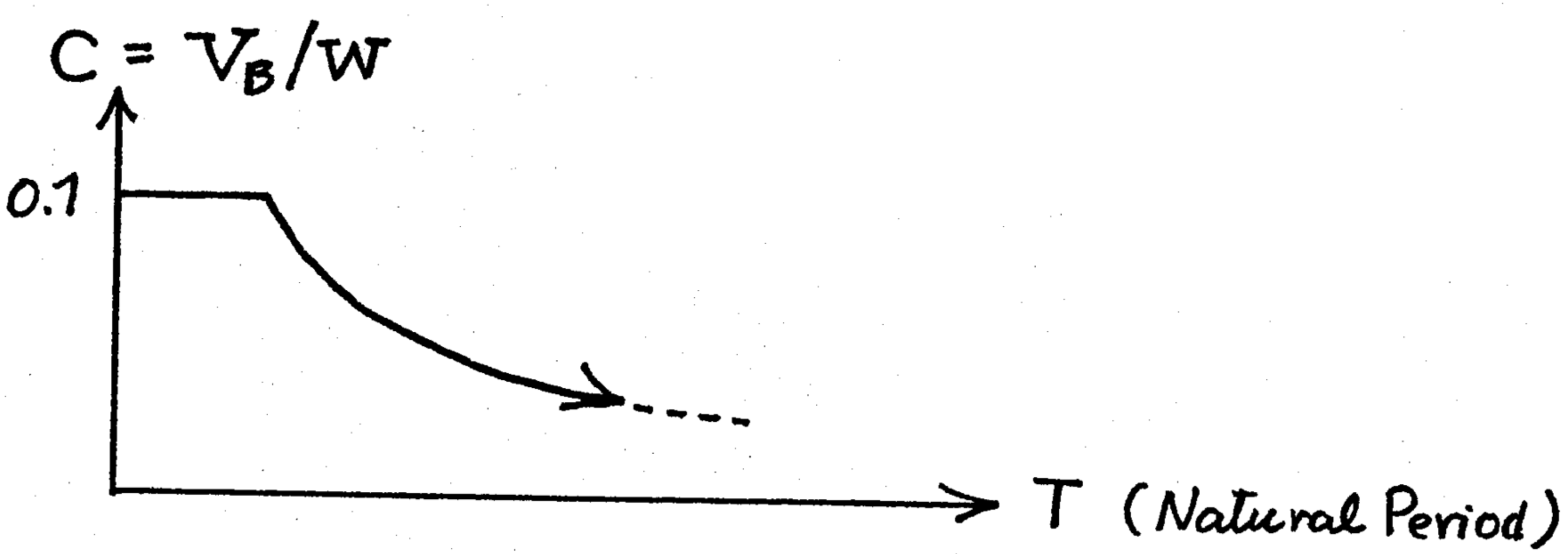
DEVELOPMENT OF SEISMIC DESIGN CODES

1940-50s Introduction of period-dependent design spectra.

$$\boldsymbol{V_B = CW}$$

where, C is the period-dependent base shear coefficient.

For long-natural-period structures, the lateral seismic force is inversely proportional to the period of structure.



DEVELOPMENT OF SEISMIC DESIGN CODES

1960s $V_B = Z\ I\ K\ S\ C\ W$ (an empirical design formula).

$Z =$ Zone factor = $1, {}^{3}/_{4}\,, {}^{1}/_{2}\,, {}^{1}/_{4}\,, \ldots..$ (adjustment for seismic hazard level of the zone relative to that of California)

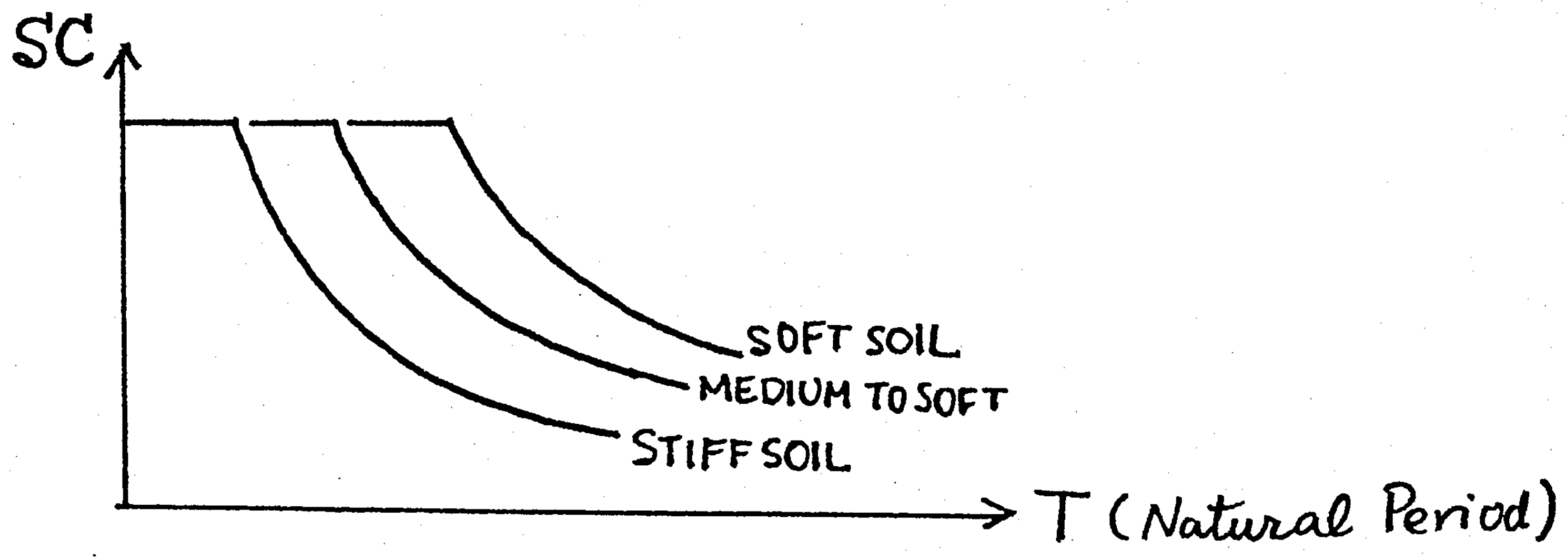
I = Important factor = 1 (ordinary) ~ 1.5 (very important building)

K = Ductility related factor = 0.67 (ductile system), 1.0 (normal), 1.3 (less ductile)

S = Soil factor = 1, 1.2, 1.5 (see diagram below)

C = Period-dependent coefficient (see diagram below)

DEVELOPMENT OF SEISMIC DESIGN CODES



DEVELOPMENT OF SEISMIC DESIGN CODES

1970s Revolution of Seismic Design Codes

Introduction of a modern design code based on several new scientific concepts—structural dynamics, soil dynamics, probabilistic seismic hazard analysis, inelastic response spectra, etc.

$$\boldsymbol{V_B} = {}^{(\boldsymbol{Z\ I\ C\ W})}\big/_{\boldsymbol{R_W}}$$

Z= Zone factor = Normalized Effective Peak Ground Acceleration (EPA/g) = PGA/g that has a 10% chance of being exceeded in a 50-year exposure period. The new probabilistic seismic hazard map constructed by Dr. Algermission in USGS was employed for the development of seismic zone map.

$Z = 0.075, 0.15, 0.2, 0.3, 0.4$ for seismic zones 1, 2A, 2B, 3, and 4, respectively.

DEVELOPMENT OF SEISMIC DESIGN CODES

I = Important factor = 1 (ordinary), 1.25 (important).

C = Normalized (and adjusted) Elastic Design Response Spectra = A/PGA, where A is spectral acceleration (pseudo acceleration).

$$C = 1.25\ {}^{S}\!/_{T^{2/3}} \text{ and } C \leq 2.75$$

S = Site coefficient, S = 1 (rock), 1.2 (stiff), 1.5 (soft to medium), 2.0 (soft clay).

Note: see the comparison between CS spectra and the average acceleration spectra for different site conditions (constructed by Prof. H. Bolton Seed at UC Berkeley).

DEVELOPMENT OF SEISMIC DESIGN CODES

$$\boldsymbol{Z\ I\ C\ W} = \left(\frac{PGA}{g}\right)\ 1\left(\frac{A}{PGA}\right)\ W\ \ = \left(\frac{A}{g}\right)\ W$$

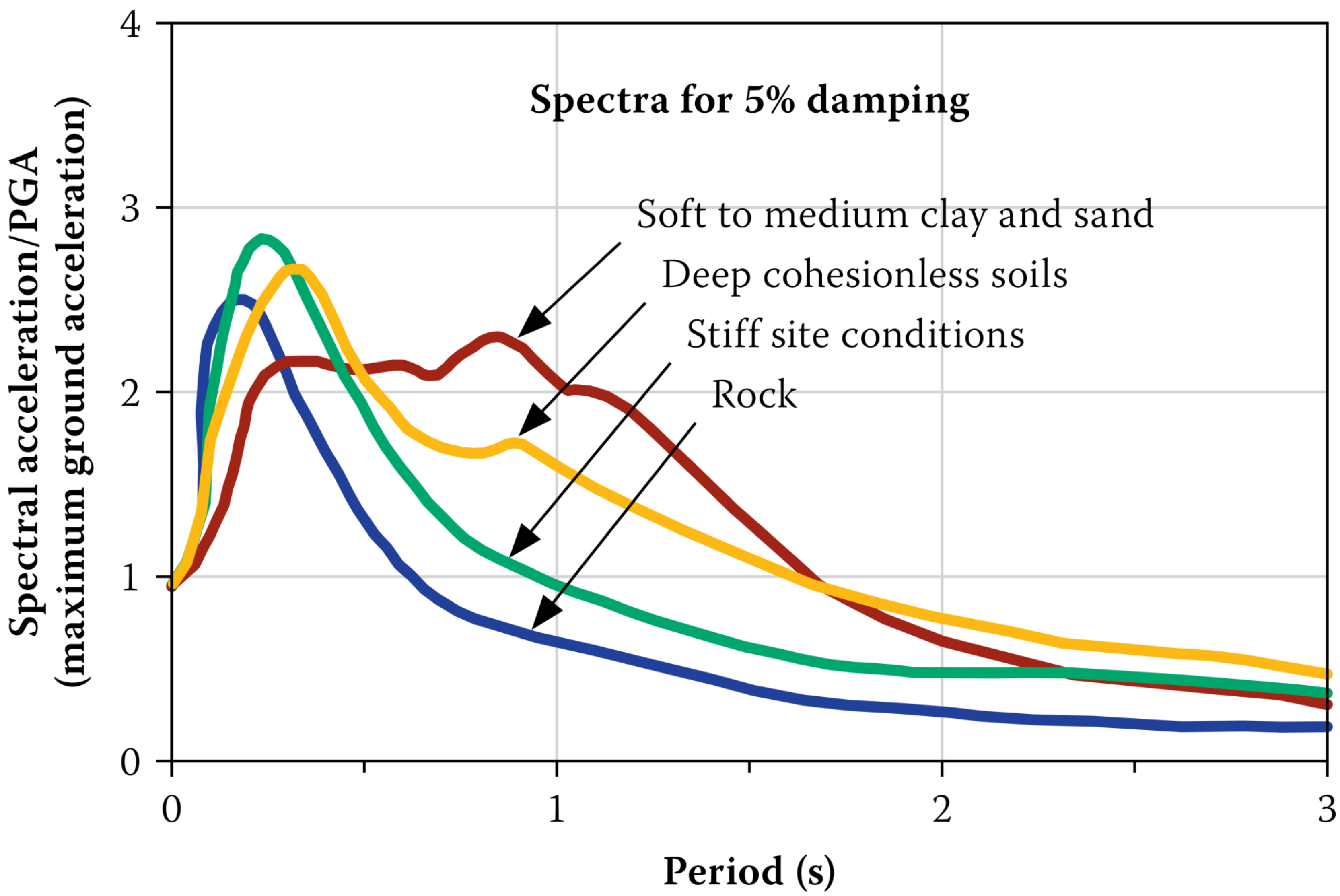
$= V_B =$ **Elastic Base Shear Demand**

= The base shear that the structure will have if the response is in its linearly elastic range during the earthquake.

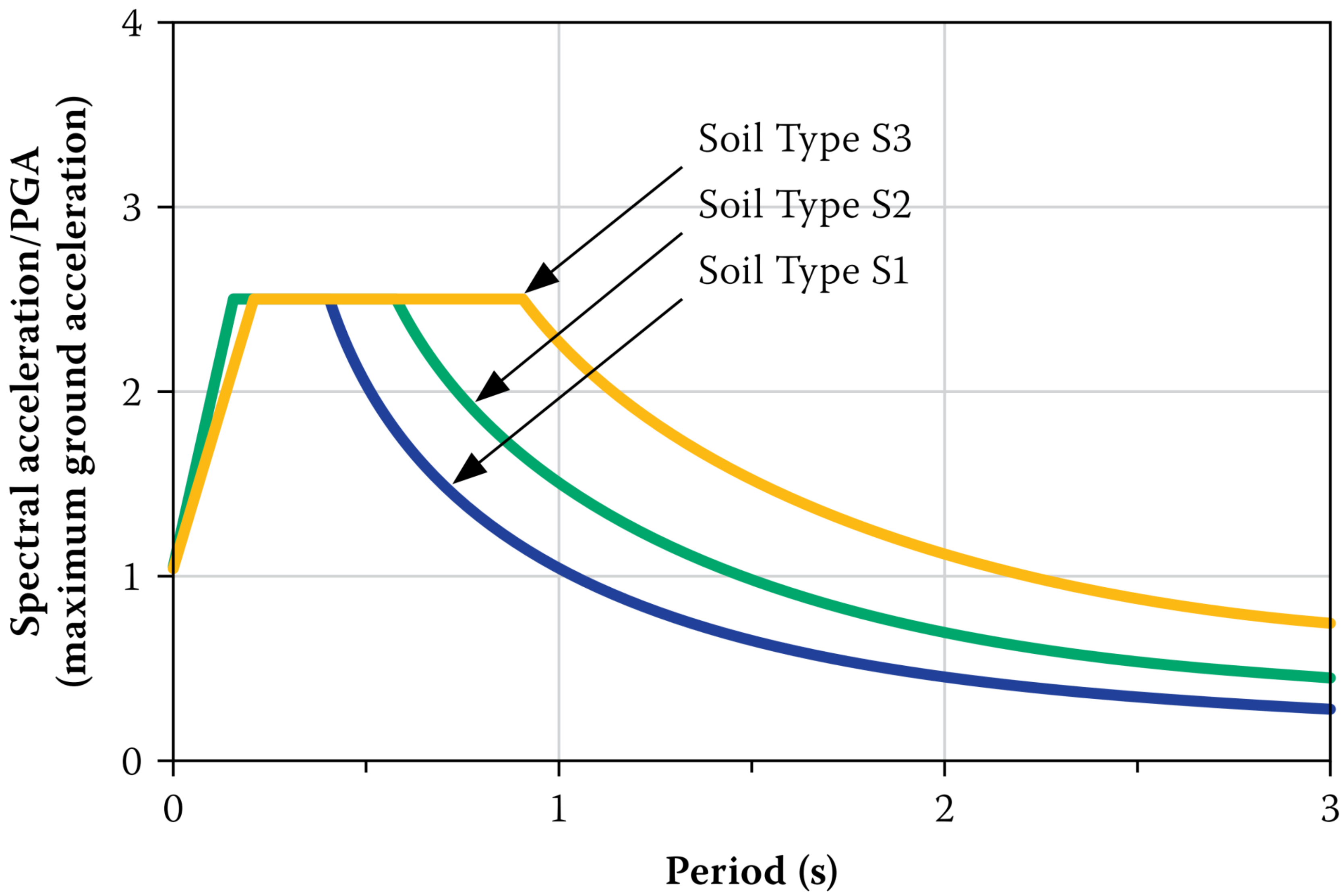
R_W = Reduction factor $= 4 \sim 12$

The factor takes into account of two important effects: **"ductility"** and **"overstrength"**.

The concept of inelastic response spectra (inelastic yield strength demand for various levels of target ductility), which was introduced by Prof. Newmark, was employed.



Average acceleration spectra for different site conditions (After Seed, et al., 1976; NEHRP, 1988)



Site dependent design spectra modified from Seed et al. (1976) and specified by ATC-3 (Dorby et al. 2000)

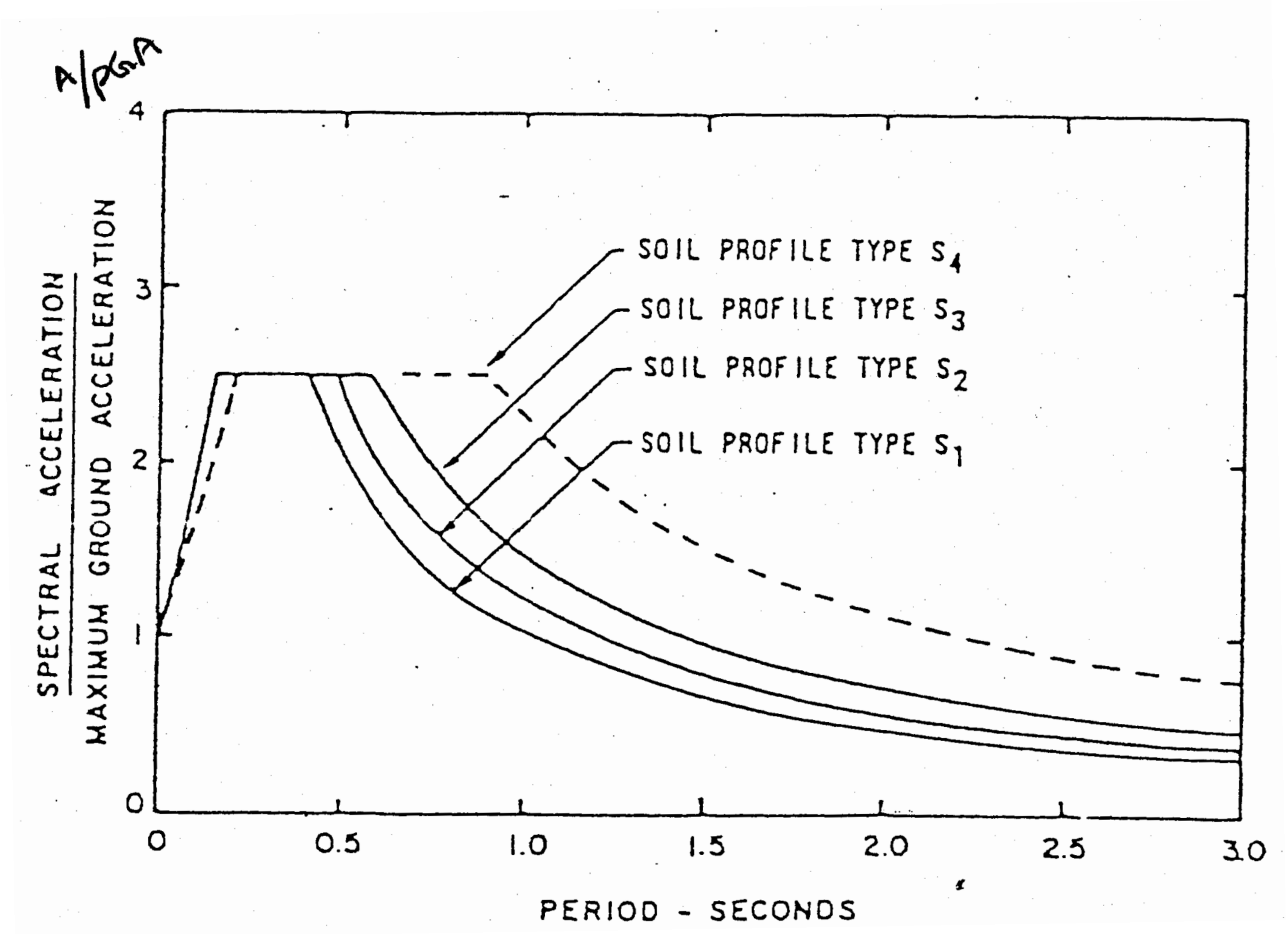


Fig. 10. Normalized response spectra recommended by 1988 NEHRP for use in building codes (NEHRP, 1988)

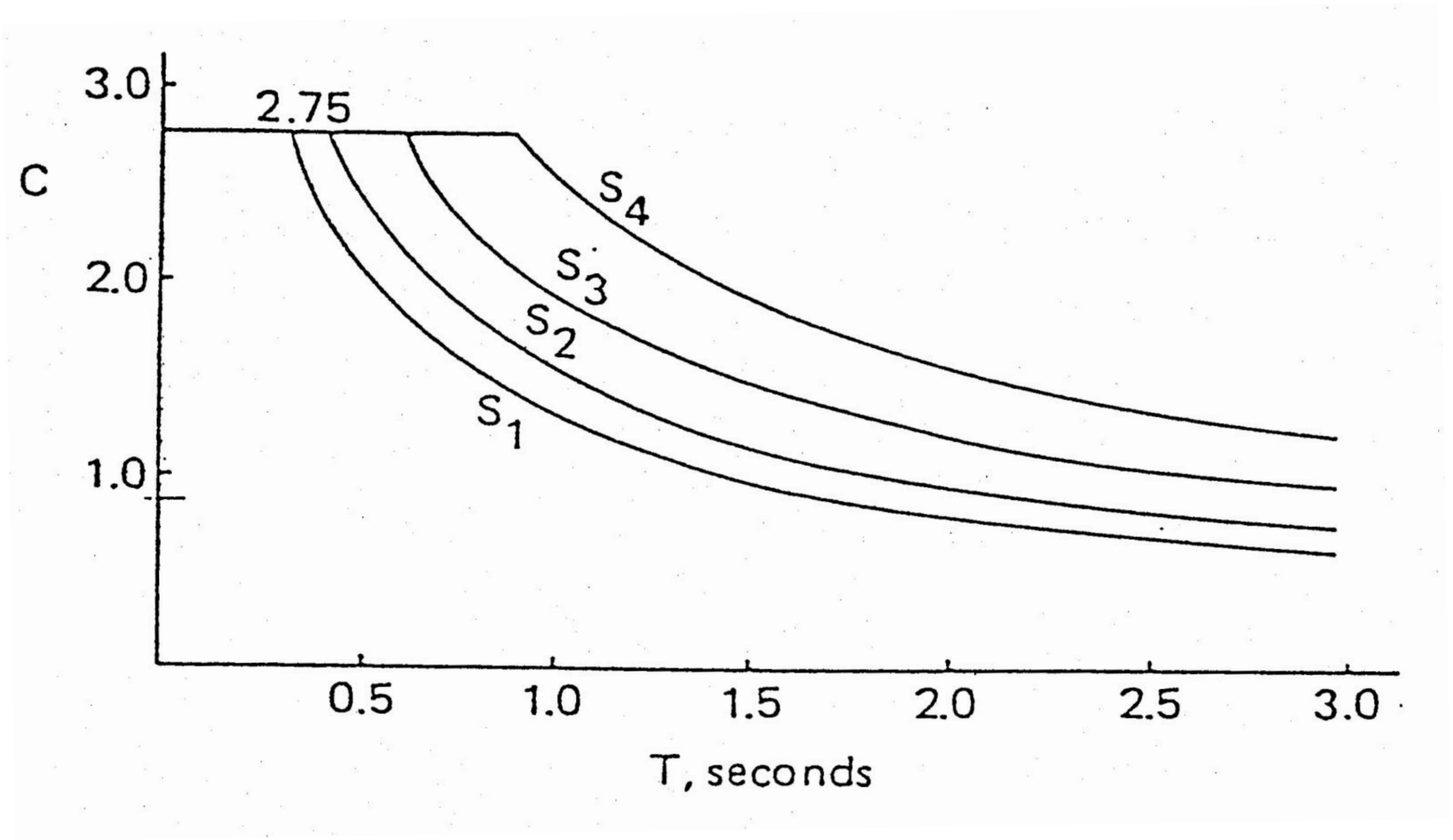


Figure 30. *C* coefficient Versus Building Period and Soil Type

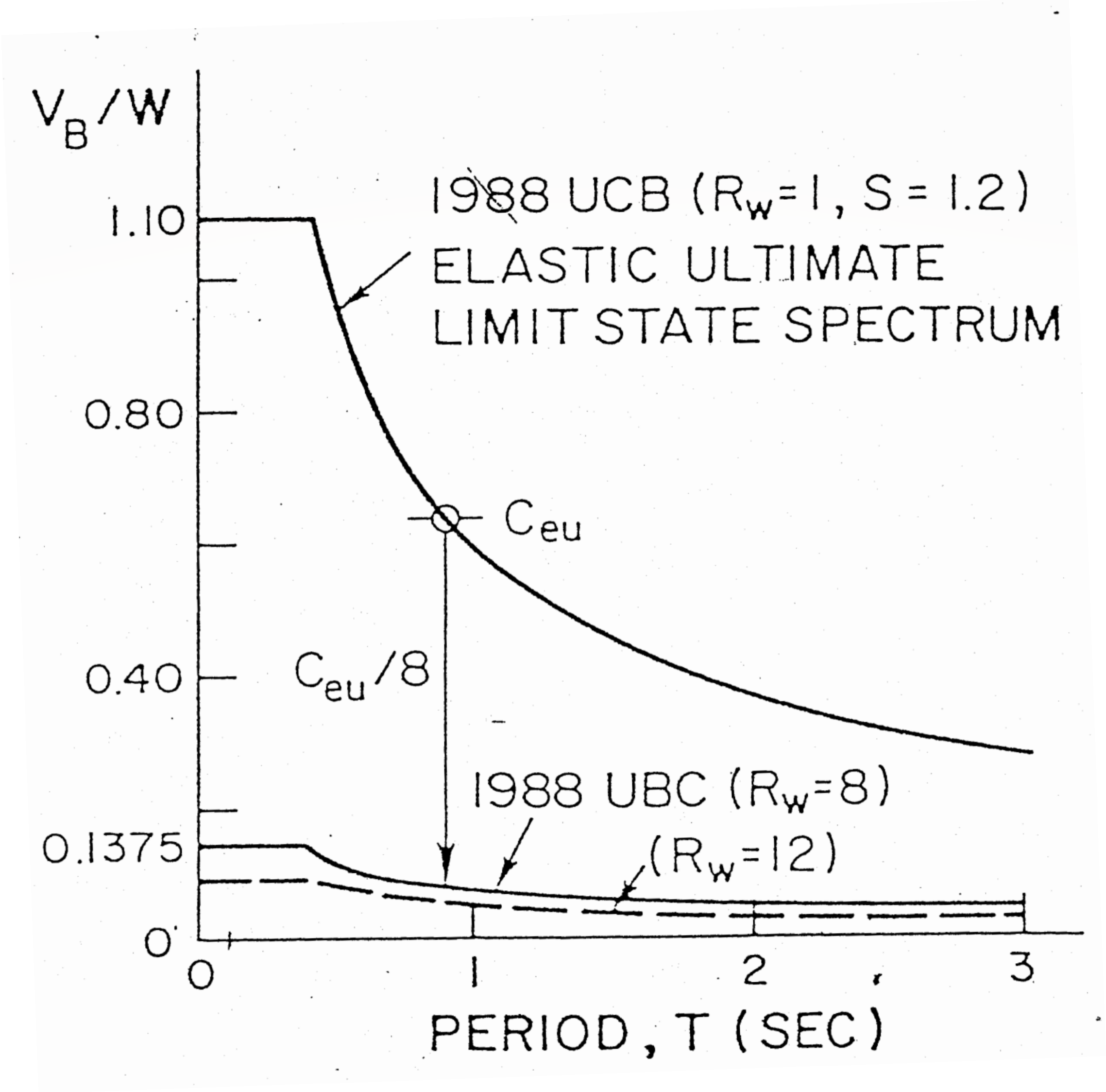


Figure 13. Empirical seismic response spectra

Equivalent Static Force Procedure in UBC (1997)

The UBC-97 Form of Equation

- The new equation:

$$V = C_s W$$

$$C_s = \frac{C_v I}{RT}$$

  - I = Importance factor, for a specific occupancy category, from UBC Table 16-K
  - C_v = Velocity based ground response coefficient, for a specific seismic zone and soil profile, from UBC Table 16-R
  - R = response modification factor, for a specific structural system, from UBC Table 16-N
  - T = Fundamental; period of vibration, from UBC Formula (30-8) or (30-10)

1630.2 Static Force Procedure.

1630.2.1 Design base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v \; I}{R \; T} \; W \qquad (30\text{-}4)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 \; C_a \; I}{R} \; W \qquad (30\text{-}5)$$

The total design base shear shall not be less than the following:

$$V = 0.11 \; C_a \; I \; W \qquad (30\text{-}6)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 \; Z N_v \; I}{R} \; W \qquad (30\text{-}7)$$

1630.2.2 Structure period. The value of T shall be determined from one of the following methods:

1. **Method A:** For all buildings, the value T may be approximated from the following formula:

$$T = C_t (h_n)^{3/4} \tag{30-8}$$

WHERE:

C_t = 0.035 (0.0853) for steel moment-resisting frames.

C_t = 0.030 (0.0731) for reinforced concrete moment-resisting frames and eccentrically braced frames.

C_t = 0.020 (0.0488) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$ (For **SI:** $0.0743/\sqrt{A_c}$ for A_c in m^2).

The value of A_c shall be determined from the following formula:

$$A_c = \Sigma A_e \left[0.2 + (D_e/h_n)^2\right] \tag{30-9}$$

The value of D_e/h_n used in Formula (30-9) shall not exceed 0.9.

2. **Method B:** The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 1630.1.2. The value of T from Method B shall not exceed a value 30 percent greater than the value of T obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period T may be computed by using the following formula:

$$T = 2\pi \sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^{\,2}\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)} \tag{30-10}$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (30-13), (30-14) and (30-15) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

The Overall Equivalent Static Analysis Procedure in UBC 97

Step 1: Determine Seismic Zone Factor Z

Step 2: Determine Seismic Source Type

Step 3: Determine Near Source Factor

Step 4: Determine Soil Profile Type

Step 5: Determine Ground Response Coefficients C_a and C_v

Step 6: Determine Fundamental period T

Step7: Classify the Structural System and determine the Response Modification Factor R

Step 8: Determine the Occupancy Categories and Importance Factor I

Step 9: Determine the Seismic Response Coefficient C_s

Step 10: Determine the Base Shear

Step 11: Vertical Distribution of Base Shear into Lateral Forces

The Overall Procedure

- Overstrength Factor (Table 16-N)
  - Global ductility capacity of lateral force-resisting system
- Soil Profile Types
  - SA, SB, SC, SD, SE
  - Ranging from hard rock to soft soil
  - Ground vibration tends to be greater on soft soil than hard rock
- Seismic Zone Factor (Table 16-I)
  - Zone 1, 2A, 2B, 3, 4
  - Effective peak ground acceleration as a function of g
  - Recurrence interval of 475 years which gives a 10% probability exceeded in a fifty years period
- Seismic Source Type (Table 16-U)
  - Depends on maximum moment magnitude potential of a fault and slip rate
- Distance to Source (Table 16-T)
  - Ground acceleration increases near the fault in Zone 4
- Importance Factor (Table 16-K)
  - Depends on occupancy categories

Inelastic Displacement and Drift (UBC 97)

- Maximum Inelastic Displacement (Eq. 30-17)

$$\Delta_M = 0.7\, R\, \Delta_s$$

Δ_M = maximum inelastic displacement

R = Overstrength factor

Δ_s = design level displacement by design seismic forces

- Drift Limit:

Story drifts shall be computed using the Maximum Inelastic Response Displacement, Δ_M.

For structures having time period < 0.7s, Drift limit = 0.025 x story height

For structures having time period ≥ 0.7s, Drift limit = 0.02 x story height

1630.7 Horizontal Torsional Moments. Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical-resisting elements in that story plus an accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 1630.6.

Where torsional irregularity exists, as defined in Table 16-M, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following formula:

$$A_x = \left[\frac{\delta_{max}}{1.2\,\delta_{avg}} \right]^2 \qquad (30\text{-}16)$$

WHERE:

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x.

δ_{max} = the maximum displacement at Level x.

The value of A_x need not exceed 3.0.

The Equivalent Static Analysis Procedure in UBC 97

TABLE 16-I—SEISMIC ZONE FACTOR Z

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

NOTE: The zone shall be determined from the seismic zone map in Figure 16-2.

TABLE 16-J—SOIL PROFILE TYPES

SOIL PROFILE TYPE	SOIL PROFILE NAME/GENERIC DESCRIPTION	AVERAGE SOIL PROPERTIES FOR TOP 100 FEET (30 480 mm) OF SOIL PROFILE		
		Shear Wave Velocity, $\bar{v}_s$ feet/second (m/s)	Standard Penetration Test, $\bar{N}$ [or $\bar{N}_{CH}$ for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, $\bar{s}_u$ psf (kPa)
S_A	Hard Rock	> 5,000 (1,500)	—	—
S_B	Rock	2,500 to 5,000 (760 to 1,500)		
S_C	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E[1]	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation. See Section 1629.3.1.			

[1]Soil Profile Type S_E also includes any soil profile with more than 10 feet (3048 mm) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 500$ psf (24 kPa). The Plasticity Index, PI, and the moisture content, w_{mc}, shall be determined in accordance with approved national standards.

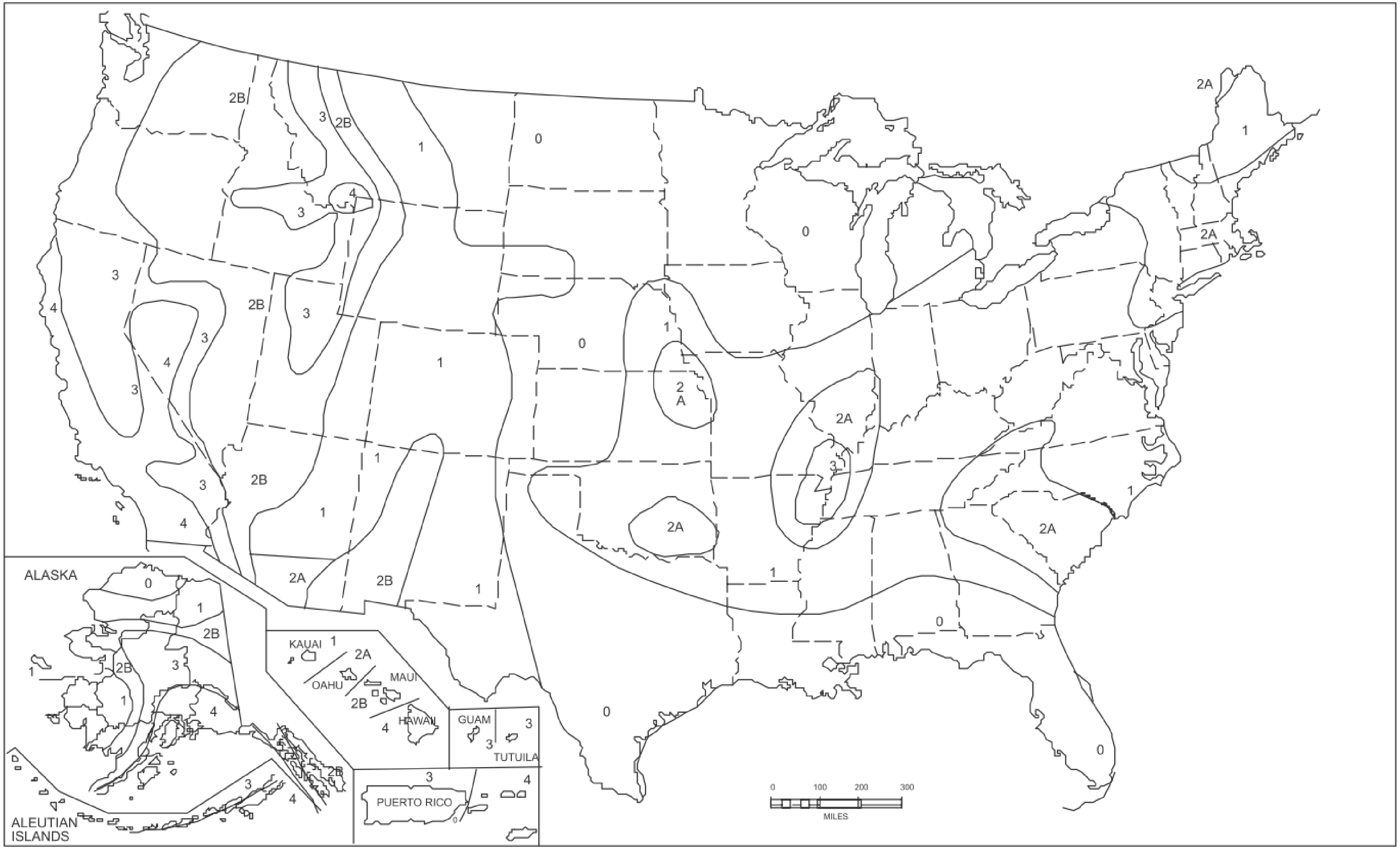


FIGURE 16-2—SEISMIC ZONE MAP OF THE UNITED STATES
For areas outside of the United States, see Appendix Chapter 16.

TABLE 16-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	1.00	1.00	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

TABLE 16-L—VERTICAL STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Stiffness irregularity—soft story** A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1629.8.4, Item 2
2. **Weight (mass) irregularity** Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	1629.8.4, Item 2
3. **Vertical geometric irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	1629.8.4, Item 2
4. **In-plane discontinuity in vertical lateral-force-resisting element** An in-plane offset of the lateral-load-resisting elements greater than the length of those elements.	1630.8.2
5. **Discontinuity in capacity—weak story** A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	1629.9.1

TABLE 16-M—PLAN STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Torsional irregularity—to be considered when diaphragms are not flexible** Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	1633.1, 1633.2.9, Item 6
2. **Re-entrant corners** Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1633.2.9, Items 6 and 7
3. **Diaphragm discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1633.2.9, Item 6
4. **Out-of-plane offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	1630.8.2; 1633.2.9, Item 6; 2213.9.1
5. **Nonparallel systems** The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	1633.1

TABLE 16-N—STRUCTURAL SYSTEMS[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel	5.6	2.2	160
	b. Concrete[3]	5.6	2.2	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[6]	4.5	2.8	160
	b. Concrete[7]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240

4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF	4.2	2.8	160
	c. Concrete with concrete IMRF[5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF	4.2	2.8	160
	f. Masonry with concrete IMRF[3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.—no limit

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.

[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2211.6 may use a *R* value of 8.

[7] Total height of the building including cantilevered columns.

[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

TABLE 16-Q—SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16-R—SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16-S—NEAR-SOURCE FACTOR N_a [1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]		
	≤ 2 km	5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

[1] The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2] The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3] The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

TABLE 16-T—NEAR-SOURCE FACTOR N_v [1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

[1] The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2] The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3] The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

TABLE 16-U—SEISMIC SOURCE TYPE[1]

SEISMIC SOURCE TYPE	SEISMIC SOURCE DESCRIPTION	SEISMIC SOURCE DEFINITION[2]	
		Maximum Moment Magnitude, *M*	Slip Rate, *SR* (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

[1]Subduction sources shall be evaluated on a site-specific basis.

[2]Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining the seismic source type.

1630.5 Vertical Distribution of Force. The total force shall be distributed over the height of the structure in conformance with Formulas (30-13), (30-14) and (30-15) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (30\text{-}13)$$

The concentrated force F_t at the top, which is in addition to F_n, shall be determined from the formula:

$$F_t = 0.07\ T\ V \qquad (30\text{-}14)$$

The value of T used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Formula (30-4). F_t need not exceed $0.25V$ and may be considered as zero where T is 0.7 second or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n, according to the following formula:

$$F_x = \frac{(V - F_t)\, w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (30\text{-}15)$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

Equivalent Static Force Procedure in IBC (2000)

IBC-2000: Equivalent Lateral Force Procedure

- A building is considered to be fixed at the base
- Seismic base shear,

$$V = C_s W$$

where

C_s = The seismic response coefficient

W = The effective seismic weight of the structure including the total dead load and other loads

W includes:

- In areas use for storage , a minimum of 25 % of the reduced floor live load (floor live load in public garages and open parking structures need not be included.
- Where an allowance for partition weight or a minimum weight of 50 kg/m2 of floor area , whichever is greater.
- Total operating weight of permanent equipment.
- 20 % of flat roof snow load where the flat roof snow load exceeds 150 kg/m2

Equivalent Lateral Load

$$V = C_s W$$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)}$$

$$S_{DS} = \frac{2}{3} S_{MS} \longrightarrow S_{MS} = F_a S_s$$

F_a = Site coefficient, short period

S_s = Spectral accelerations for short periods, Maps

R = The response modification factor

I_E = The occupancy importance factor

S_{DS} = Design spectral response acceleration parameter for short period

Cs need not be greater than

For $T \leq T_L$, $C_s = \dfrac{S_{DI}}{\left(\frac{R}{I_E}\right) T}$

For $T > T_L$, $C_s = \dfrac{S_{D1} T_L}{T^2 \left(\frac{R}{I}\right)}$ for $T > T_L$

$$S_{D1} = \frac{2}{3} S_{M1} \longrightarrow S_{M1} = F_V S_1$$

F_V = Site coefficient, 1 sec period

T = Fundamental period (in seconds) of the structure

S_1 = Spectral accelerations for a 1-second period, Maps

S_{D1} = Design spectral response acceleration parameter for long period

T_L = Long-period transition period, map

Cs must be greater than $C_s = \dfrac{0.5 S_1}{\left[\frac{R}{I_E}\right]}$

$C_s = 0.01$

Buildings and structure for which the 1-second spectral response S1 is equal to or greater than 0.6 g

Equivalent Lateral Load Procedure

- R is dependent on structural system and ranges from 3 to 8 (bad to good). It reduces the design loads to account for the damping and ductility of the structural system.
- F_a is site modification for short period spectrum and ranges from 0.8 to 2.5 (good to bad)
- F_v is a site modification for 1 sec period spectrum and ranges from 0.8 to 3.5 (good to bad)
- I_E Ranges from 1.0 to 1.5 (Normal to important)

- **Total variation can be up to 16 times for buildings in the same Seismic Zone**

1617.4 Equivalent lateral force procedure for seismic design of buildings. See Section 1616.6 for limitations on the use of this procedure. For purposes of this analysis procedure, a building is considered to be fixed at the base.

1617.4.1 Seismic base shear. The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \qquad \textbf{(Equation 16-34)}$$

where:

C_s = The seismic response coefficient determined in accordance with Section 1617.4.1.1.

W = The effective seismic weight of the structure, including the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking structures need not be included).
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 pounds per square foot (500 Pa/m²) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Twenty percent of flat roof snow load where the flat roof snow load exceeds 30 pounds per square foot (1.44 kN/m²).

1617.4.1.1 Calculation of seismic response coefficient. The seismic response coefficient, C_s, shall be determined in accordance with the following equation:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} \qquad \textbf{(Equation 16-35)}$$

where:

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

R = The response modification factor from Table 1617.6.

S_{DS} = The design spectral response acceleration at short period as determined in Section 1615.1.3.

The value of the seismic response coefficient, C_s, computed in accordance with Equation 16-35 need not exceed the following:

$$C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} \qquad \textbf{(Equation 16-36)}$$

but shall not be taken less than:

$$C_s = 0.044 S_{DS} I_E \qquad \textbf{(Equation 16-37)}$$

For buildings and structures in Seismic Design Category E or F, and those buildings and structures for which the 1-second spectral response, S_1, is equal to or greater than 0.6g, the value of the seismic response coefficient, C_s, shall not be taken as less than:

$$C_s = \frac{0.5 S_1}{R/I_E} \qquad \textbf{(Equation 16-38)}$$

where I and R are as defined above and

S_{D1} = The design spectral response acceleration at 1-second period as determined from Section 1615.1.3.

S_1 = The mapped maximum considered earthquake spectral response acceleration at 1-second period determined in accordance with Section 1615.1.

T = The fundamental period of the building (seconds) determined in Section 1617.4.2.

IBC-2000: General Procedure

- Maximum Considered Earthquake (MCE) based on 1996 USGS probabilistic hazard maps
- Deterministic limits used in high seismicity areas where the hazard can be driven by tails of distributions
- Hazards maps provide spectral accelerations for
  - T = 0.2 Sec called S_s
  - T= 1.0 Sec called S_1
- Local soil conditions considered using site coefficients
  - F_a for short duration
  - F_v for longer duration

S_s

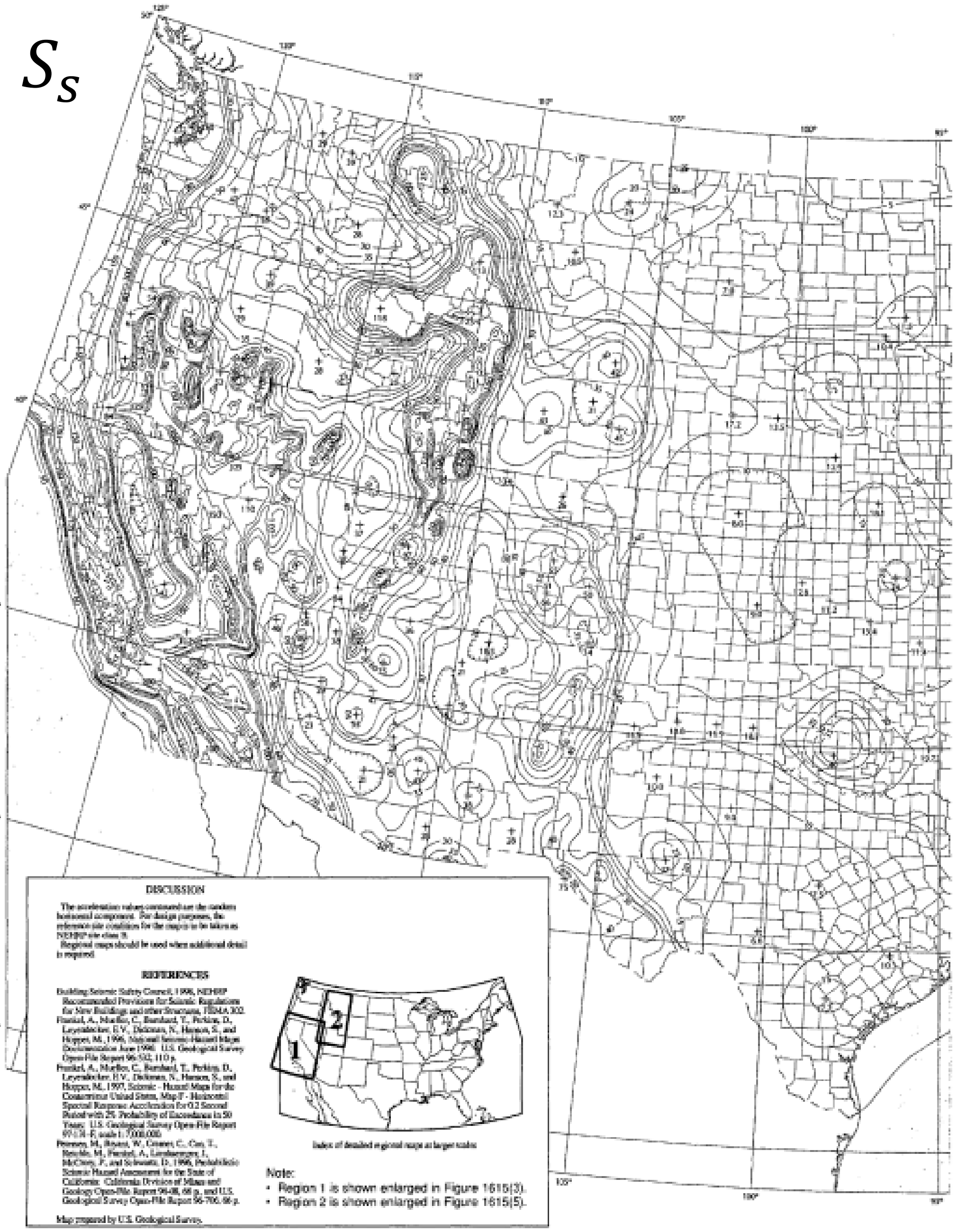


FIGURE 1615(1)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5 PERCENT OF CRITICAL DAMPING), SITE CLASS B

S_1

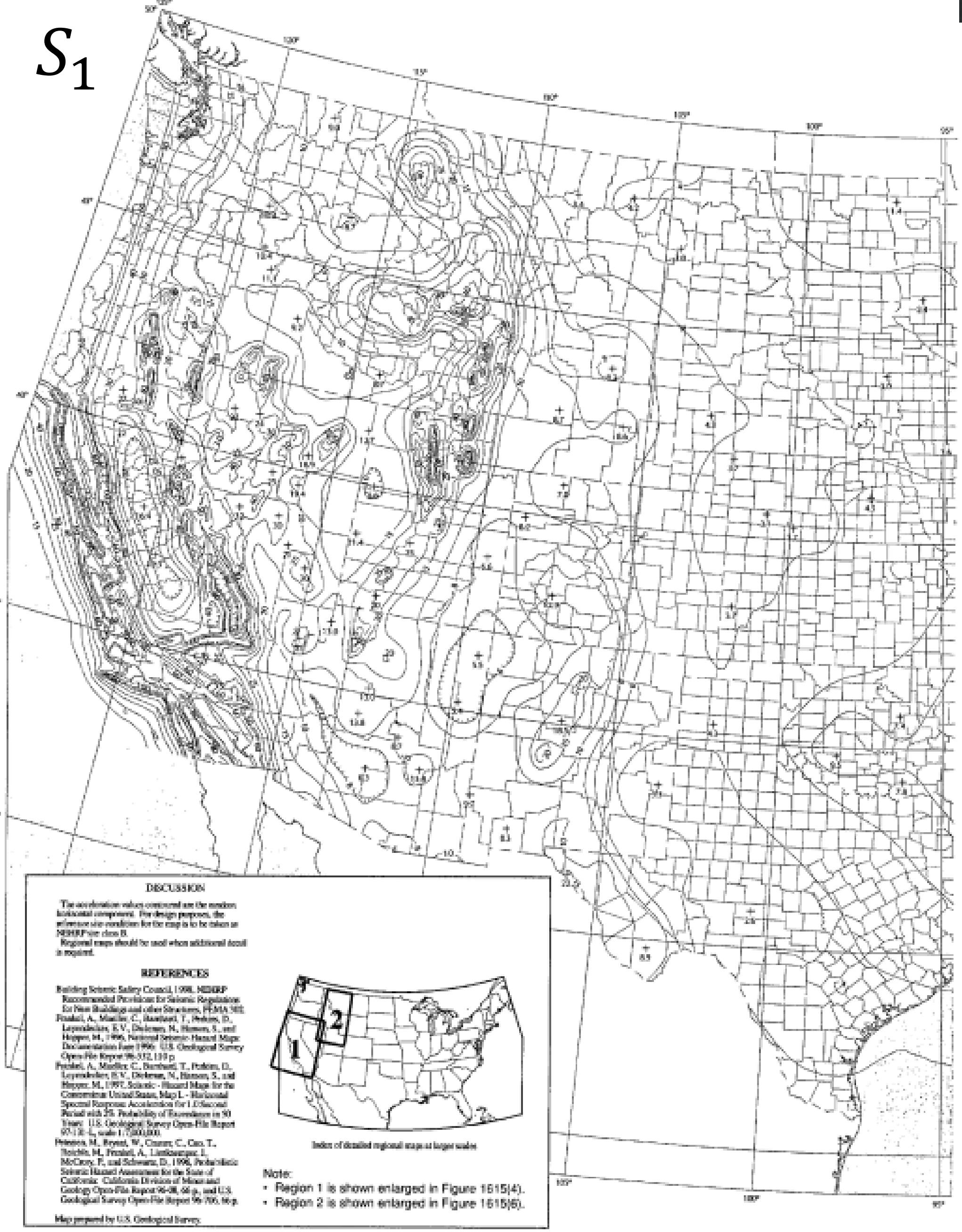


FIGURE 1615(2)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5 PERCENT OF CRITICAL DAMPING), SITE CLASS B

1615.1.2 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for site class effects, shall be determined by Equations 16-16 and 16-17, respectively:

$$S_{MS} = F_a S_s \qquad \textbf{(Equation 16-16)}$$

$$S_{M1} = F_v S_1 \qquad \textbf{(Equation 16-17)}$$

where:

F_a = Site coefficient defined in Table 1615.1.2(1).

F_v = Site coefficient defined in Table 1615.1.2(2).

S_S = The mapped spectral accelerations for short periods as determined in Section 1615.1.

S_1 = The mapped spectral accelerations for a 1-second period as determined in Section 1615.1.

1615.1.3 Design spectral response acceleration parameters. Five-percent damped design spectral response acceleration at short periods, S_{DS}, and at 1 second period, S_{D1}, shall be determined from Equations 16-18 and 16-19, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \textbf{(Equation 16-18)}$$

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \textbf{(Equation 16-19)}$$

where:

S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1615.1.2.

S_{M1} = The maximum considered earthquake spectral response accelerations for 1 second period as determined in Section 1615.1.2.

F_a and F_v: To adjust maximum considered earthquake spectral response acceleration according to soil conditions

TABLE 1615.1.1
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\overline{v}_s$, (ft/s)	Standard penetration resistance, $\overline{N}$	Soil undrained shear strength, $\overline{s}_u$, (psf)
A	Hard rock	$\overline{v}_s > 5,000$	Not applicable	Not applicable
B	Rock	$2,500 < \overline{v}_s \leq 5,000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1,200 < \overline{v}_s \leq 2,500$	$\overline{N} > 50$	$\overline{s}_u \geq 2,000$
D	Stiff soil profile	$600 \leq \overline{v}_s \leq 1,200$	$15 \leq \overline{N} \leq 50$	$1,000 \leq \overline{s}_u \leq 2,000$
E	Soft soil profile	$\overline{v}_s < 600$	$\overline{N} < 15$	$\overline{s}_u < 1,000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$; 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\overline{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ ft)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m^2, 1 pound per square foot = 0.0479 kPa.

Site Coefficient F_a, Modification for short period Response

TABLE 1615.1.2(1)
VALUES OF SITE COEFFICIENT F_a AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS (S_S)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at short period, S_S.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

Site Coefficient F_v, Modification for 1 Sec period Response

TABLE 1615.1.2(2)
VALUES OF SITE COEFFICIENT F_V AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD (S_1)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	Note b
F	Note b	Note b	Note b	Note b	Note b

a. Use straight line interpolation for intermediate values of mapped spectral acceleration at 1-second period, S_1.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

Seismic Design Categories based on Short Period Response Accelerations

VALUE OF S_{DS}	SEISMIC USE GROUP		
	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.5g$	C	C	D
$0.50g \le S_{DS}$	D[a]	D[a]	D[a]

Note a: Seismic Use Groups I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1-second period, equal to or greater than 0.75g, shall be assigned to Seismic Design Category E, and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

Seismic Design Categories based on 1 Second Period Response Accelerations

VALUE OF S_{DS}	SEISMIC USE GROUP		
	I	**II**	**III**
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 1.33g$	B	B	C
$1.33g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D[a]	D[a]	D[a]

Note a: Seismic Use Groups I and II structures located on sites with mapped maximum considered earthquake spectral response acceleration at 1-second period, equal to or greater than 0.75g, shall be assigned to Seismic Design Category E, and Seismic Use Group III structures located on such sites shall be assigned to Seismic Design Category F.

Seismic design category

- The structure must be assigned a seismic design category.
  - determines the permissible structural systems.
  - determines limitations on height and irregularity.
  - determines those components of the structure that must be designed for seismic loads, and the types of analysis required.
- The seismic design categories, designated A through F.
- They depend on the seismic use group and the design spectral acceleration coefficients, S_{DS} and S_{D1}. The structure is assigned the more severe of the two values taken from tables.

Why Seismic Design Categories?

- Seismic Design Category are used to select:
  - Type of analysis
    - Very Simplified
    - Equivalent Lateral Load Procedure
    - Response Spectrum
    - Time-history
  - Type of design and detailing
    - Special Detailing
    - Intermediate Detailing
    - Ordinary Detailing
  - Many other checks/requirements

Example of Use of Seismic Design Categories

Analysis Procedures for Seismic Design Categories D, E or F

STRUCTURE DESCRIPTION	MINIMUM ALLOWABLE ANALYSIS PROCEDURE FOR SEISMIC DESIGN
Regular structures, other than those in Item 1 above, up to 240 feet in height.	**Equivalent lateral-force procedure**
Structures that have vertical irregularities of Type 1a, 1b, 2 or 3 in Table 1616.5.2, or plan irregularities of Type 1a or 1b of Table 1616.5.1 and have a height exceeding five stories or 65 feet and structures exceeding 240 feet in height.	**Modal analysis procedure**
Other structures designated as having plan or vertical irregularities.	**Equivalent lateral-force procedure with dynamic characteristics included in the analytical model.**

TABLE 1616.6.3
ANALYSIS PROCEDURES FOR SEISMIC DESIGN CATEGORIES D, E OR F

STRUCTURE DESCRIPTION	MINIMUM ALLOWABLE ANALYSIS PROCEDURE FOR SEISMIC DESIGN
1. Seismic Use Group I buildings of light-framed construction three stories or less in height and of other construction, two stories or less in height with flexible diaphragms at every level.	Simplified procedure of Section 1617.5.
2. Regular structures, other than those in Item 1 above, up to 240 feet in height.	Equivalent lateral-force procedure (Section 1617.4).
3. Structures that have vertical irregularities of Type 1a, 1b, 2 or 3 in Table 1616.5.2, or plan irregularities of Type 1a or 1b of Table 1616.5.1, and have a height exceeding five stories or 65 feet and structures exceeding 240 feet in height.	Modal analysis procedure (Section 1618).
4. Other structures designated as having plan or vertical irregularities.	Equivalent lateral-force procedure (Section 1617.4) with dynamic characteristics included in the analytical model.
5. Structures with all of the following characteristics: - located in an area with S_{D1} of 0.2 or greater, as determined in Section 1615.1.3; - located in an area assigned to Site Class E or F, in accordance with Section 1615.1.1 and; - with a natural period T of 0.7 second or greater, as determined in Section 1617.4.2.	Modal analysis procedure (Section 1618). A site- specific response spectrum shall be used but the design base shear shall not be less than that determined from Section 1617.4.1.

For SI: 1 foot = 304.8 mm.

Seismic Design Category

Seismic Design Category	Seismic Use Group	Value of S_{DS}	Value of S_{D1}
A	I, II, III	$S_{DS} < 0.167g$	$S_{D1} < 0.067g$
B	I, II	$0.167g \leq S_{DS} \leq 0.33g$	$0.067g \leq S_{D1} \leq 0.133g$
C	III	$0.167g \leq S_{DS} \leq 0.33g$	$0.067g \leq S_{D1} \leq 0.133g$
	I, II	$0.33g \leq S_{DS} \leq 0.50g$	$0.133g \leq S_{D1} \leq 0.20g$
D	III	$0.33g \leq S_{DS} \leq 0.50g$	$0.133g \leq S_{D1} \leq 0.20g$
	I, II	$0.50g \leq S_{DS}$	$0.20g \leq S_{D1}$
E	I, II	-	$0.75g \leq S_{D1}$
F	III	-	$0.75g \leq S_{D1}$

- **I = Normal Buildings , (I_E=1.0)**
- **II = Important Structures (I_E=1.25)**
- **III = Essential Structures (I_E=1.5)**

Analysis Procedures

Analysis Procedures	Seismic Design Category	Limit
Minimum Lateral Forces (1616.4.1)	A	-
Simplified Analysis (1617.5)	A, B, C, D, E, F	Seismic Use Group I buildings of light-framed construction three stories or less in height with flexible diaphragms at every level
Equivalent Lateral Forces (1617.4)	D, E, F	Regular structures other than those in Item I above, up to 240 feet in height
Equivalent Lateral Forces (1617.4) with dynamic characteristics included in the analysis model	D, E, F	Other structures designated as having plan or vertical irregularities
Modal Analysis Procedure (1618)	D, E, F	Structures that have vertical irregularities of Type 1a, 1b, 2 or 3 Plan irregularities of Type 1a or 1b Height exceeding five stories or 65 feet Structures exceeding 240 feet in height
Modal Analysis Procedure (1618) With A site-specific response spectrum	D, E, F	$S_{D1} \geq 0.2$, Site Class E or F and $T \geq 0.7$

TABLE 1616.5.1
PLAN STRUCTURAL IRREGULARITIES

	IRREGULARITY TYPE AND DESCRIPTION	REFERENCE SECTION	SEISMIC DESIGN CATEGORY[a] APPLICATION
1a	Torsional Irregularity—to be considered when diaphragms are not flexible as determined in Section 1602.1. Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.	1617.4.4.5 1620.3.1 1616.6.3 Table 1616.6.3 1617.4.6.1	C, D, E and F D, E and F D, E and F D, E and F C, D, E and F
1b	Extreme Torsional Irregularity—to be considered when diaphragms are not flexible as determined in Section 1602.1. Extreme torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure.	1617.4.4.5 1620.3.1 1620.4.1 1616.6.3 Table 1616.6.3 1617.4.6.1	C, D, E and F D E and F D, E and F D, E and F C, D, E and F
2	Re-entrant Corners Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1620.3.1	D, E and F
3	Diaphragm Discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1620.3.1	D, E and F
4	Out-of-Plane Offsets Discontinuities in a lateral- force-resistance path, such as out-of-plane offsets of the vertical elements.	1620.3.1 1616.6.3 1620.1.9	D, E and F D, E and F B, C, D, E and F
5	Nonparallel Systems The vertical lateral-force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	1620.2.2	C, D, E and F

a. Seismic Design Category is determined in accordance with Section 1616.

TABLE 1616.5.2
VERTICAL STRUCTURAL IRREGULARITIES

	IRREGULARITY TYPE AND DESCRIPTION	REFERENCE SECTION	SEISMIC DESIGN CATEGORY[a] APPLICATION
1a	Stiffness Irregularity—Soft Story A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1616.6.3 Table 1616.6.3	D, E and F D, E and F
1b	Stiffness Irregularity—Extreme Soft Story An extreme soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of the three stories above.	1620.4.1 1616.6.3 Table 1616.6.3	E and F D, E and F D, E and F
2	Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 1616.6.3	D, E and F
3	Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story.	Table 1616.6.3	D, E and F
4	In-plane Discontinuity in Vertical Lateral-Force-Resisting Elements An in-plane offset of the lateral-force-resisting elements greater than the length of those elements or a reduction in stiffness of the resisting element in the story below.	1620.3.1 1616.6.3 1620.1.9	D, E and F D, E and F B, C, D, E and F
5	Discontinuity in Capacity—Weak Story A weak story is one in which the story lateral strength is less than 80 percent of that in the story above. The story strength is the total strength of seismic-resisting elements sharing the story shear for the direction under consideration.	1620.1.3 1616.6.3 1620.4.1	B, C, D, E and F D, E and F E and F

a. Seismic Design Category is determined in accordance with Section 1616.

Seismic Load Effect (E)

- Seismic Load Effect (E)
  - Where the effects of gravity and the seismic ground motion are additive

$$E = \rho Q_E + 0.2\, S_{DS}\, D$$

  - Where the effects of gravity and seismic ground motion counteract

$$E = \rho Q_E - 0.2\, S_{DS}\, D$$

where D = The effect of dead load.

E = The combined effect of horizontal and vertical earthquake-induced forces

ρ = A reliability factor based on system redundancy obtained in accordance with Section 1617.2

Q_E = The effect of horizontal seismic forces

S_{DS}= The design spectral response acceleration at short periods

- Design shall use the load combinations prescribed in Section 1605.2 for strength or load and resistance factor design methodologies, or Section 1605.3 for allowable stress design methods.

Maximum Seismic Load Effect (E_m)

- Maximum Seismic Load Effect (Em)
  - Where the effects of gravity and the seismic ground motion are additive

$$E = \Omega_0 Q_E + 0.2\, S_{DS}\, D$$

  - Where the effects of gravity and seismic ground motion counteract

$$E = \Omega_0 Q_E - 0.2\, S_{DS}\, D$$

Ω_0 is the system overstrength factor as given in Table 1617.6

- Redundancy Factor (ρ)
  - For Seismic Design Category A, B or C, ρ = 1.0
  - For Seismic Design Category D, E or F

$$\rho = 2 - \frac{20}{r_{\max}\sqrt{A_i}} \quad (for\, US)$$

$$\rho = 2 - \frac{6.1}{r_{\max}\sqrt{A_i}} \quad (for\, SI)$$

r_{max} = The ratio of the design story shear resisted by the most heavily loaded single element in the story to the total story shear, for a given direction of loading

A_i = The floor area in square feet of the diaphragm level immediately above the story.

The value, p, shall not be less than 1.0, and need not exceed 1.5.

Vertical distribution of seismic forces

- The lateral force, (kip or kN) , induced at any level:

$$F_x = C_{vx} \cdot V$$

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

where

- C_{vx} =Vertical distribution factor.
- k = A distribution exponent related to the buildings period as follows:
  - For buildings having a period of 0.5 second or less, k = 1.
  - For building having a period of 2.5 seconds or more, k = 2.
  - For building having a period between 0.5 and 2.5 seconds, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

h_i and h_x = The height (feet or m) from the base to Level i or x.

V = Total design lateral force or shear at the base of the building (kip or kN).

w_i and w_x = The portion of the total gravity load of the building, W, located or assigned to Level i or x.

1617.4.4.3 Torsion. Where diaphragms are not flexible, the design shall include the torsional moment, M_t (kip·ft or kN·m), resulting from the difference in locations of the center of mass and the center of stiffness.

1617.4.4.4 Accidental torsion. Where diaphragms are not flexible, in addition to the torsional moment, the design also shall include accidental torsional moments, M_{ta} (kip·ft or kN·m), caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the building perpendicular to the direction of the applied forces.

1617.4.4.5 Dynamic amplification of torsion. For structures in Seismic Design Category C, D, E or F (Section 1616), where Type 1a or 1b plan torsional irregularity exists as defined in Table 1616.5.1, effects of torsional irregularity shall be accounted for by multiplying the sum of M_t plus M_{ta} (as determined in the preceding sections) at each level by a torsional amplification factor, A_x, determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad \textbf{(Equation 16-44)}$$

where:

δ_{max} = The maximum displacement at Level x (inches or mm).

δ_{avg} = The average of the displacements at the extreme points of the structure at Level x (inches or mm).

The torsional amplification factor, A_x, is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

1617.4.6.1 Story drift determination. The design story drift, Δ, shall be computed as the difference of the deflections at the center of mass at the top and bottom of the story under consideration. Where allowable stress design is used, Δ shall be computed using earthquake forces without dividing by 1.4. For structures assigned to Seismic Design Category C, D, E or F (see Section 1616) having plan irregularity Types 1a or 1b of Table 1616.5.1, the design story drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

The deflections of Level x, δ_x (inches or mm), shall be determined in accordance with following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I_E} \quad \textbf{(Equation 16-46)}$$

where:

C_d = The deflection amplification factor in Table 1617.6.

δ_{xe} = The deflections (inches or mm) determined by an elastic analysis of the seismic-force-resisting system.

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

Deflection and Story Drift

- Deflection

$$\delta_x = \frac{C_d \delta_{xe}}{I_E}$$

where

δ_{xe} = deflection determined by elastic analysis

C_d = deflection amplification factor

I_E = importance factor

Allowable Story Drift

TABLE 1617.3

ALLOWABLE STORY DRIFT, Δ_a (inches)

BUILDING	SEISMIC USE GROUP		
	I	II	III
Buildings, other than masonry shear wall or masonry wall frame buildings, four stories or less in height with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025 h_{sx}$[b]	$0.020 h_{sx}$	$0.015 h_{sx}$
Masonry cantilever shear wall buildings[c]	$0.010 h_{sx}$	$0.010 h_{sx}$	$0.010 h_{sx}$
Other masonry shear wall buildings	$0.007 h_{sx}$	$0.007 h_{sx}$	$0.007 h_{sx}$
Masonry wall frame buildings	$0.013 h_{sx}$	$0.013 h_{sx}$	$0.010 h_{sx}$
All other buildings	$0.020 h_{sx}$	$0.015 h_{sx}$	$0.010 h_{sx}$

For SI: 1 inch = 25.4 mm.

a. There shall be no drift limit for single-story buildings with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.

b. h_{sx} is the story height below Level x.

c. Buildings in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

TABLE 1617.6
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $\Omega_0{}^g$	DEFLECTION AMPLIFICATION FACTOR, $C_d{}^b$	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.3				
					A or B	C	D[d]	E[e]	F[e]
1. Bearing Wall Systems									
A. Ordinary steel braced frames	(14)[j] 2211	4	2	3 1/2	NL	NL	160	160	160
B. Special reinforced concrete shear walls	1910.2.4	5 1/2	2 1/2	5	NL	NL	160	160	100
C. Ordinary reinforced concrete shear walls	1910.2.3	4 1/2	2 1/2	4	NL	NL	NP	NP	NP
D. Detailed plain concrete shear walls	1910.2.2	2 1/2	2 1/2	2	NL	NP	NP	NP	NP
E. Ordinary plain concrete shear walls	1910.2.1	1 1/2	2 1/2	1 1/2	NL	NP	NP	NP	NP
F. Special reinforced masonry shear walls	2106.1.1.5	5	2 1/2	3 1/2	NL	NL	160	160	100
G. Intermediate reinforced masonry shear walls	2106.1.1.4	3 1/2	2 1/2	2 1/4	NL	NL	NP	NP	NP
H. Ordinary reinforced masonry shear walls	2106.1.1.2	2 1/2	2 1/2	1 3/4	NL	160	NP	NP	NP
I. Detailed plain masonry shear walls	2106.1.1.3	2	2 1/2	1 3/4	NL	NP	NP	NP	NP
J. Ordinary plain masonry shear walls	2106.1.1.1	1 1/2	2 1/2	1 1/4	NL	NP	NP	NP	NP
K. Light frame walls with shear panels—wood structural panels/sheet steel panels	2306.4.1/ 2211	6	3	4	NL	NL	65	65	65
L. Light frame walls with shear panels—all other materials	2306.4.5	2	2 1/2	2	NL	NL	35	NP	NP
2. Building Frame Systems									
A. Steel eccentrically braced frames, moment-resisting, connections at columns away from links	(15)[j]	8	2	4	NL	NL	160	160	100
B. Steel eccentrically braced frames, nonmoment resisting, connections at columns away from links	(15)[j]	7	2	4	NL	NL	160	160	100
C. Special steel concentrically braced frames	(13)[j]	6	2	5	NL	NL	160	160	100
D. Ordinary steel concentrically braced frames	(14)[j]	5	2	4 1/2	NL	NL	160	100	100
E. Special reinforced concrete shear walls	1910.2.4	6	2 1/2	5	NL	NL	160	160	100
F. Ordinary reinforced concrete shear walls	1910.2.3	5	2 1/2	4 1/2	NL	NL	NP	NP	NP
G. Detailed plain concrete shear walls	1910.2.2	3	2 1/2	2 1/2	NL	NP	NP	NP	NP
H. Ordinary plain concrete shear walls	1910.2.1	2	2 1/2	2	NL	NP	NP	NP	NP
I. Composite eccentrically braced frames	(14)[k]	8	2	4	NL	NL	160	160	100

(continued)

TABLE 1617.6—continued

DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R[a]	SYSTEM OVER-STRENGTH FACTOR, Ω_0[g]	DEFLECTION AMPLIFICATION FACTOR, C_d[b]	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.3				
					A or B	C	D[d]	E[e]	F[e]
J. Composite concentrically braced frames	(13)[k]	5	2	4½	NL	NL	160	160	100
K. Ordinary composite braced frames	(12)[k]	3	2	3	NL	NL	NP	NP	NP
L. Composite steel plate shear walls	(17)[k]	6½	2½	5½	NL	NL	160	160	100
M. Special composite reinforced concrete shear walls with steel elements	(16)[k]	6	2½	5	NL	NL	160	160	100
N. Ordinary composite reinforced concrete shear walls with steel elements	(15)[k]	5	2½	4½	NL	NL	NP	NP	NP
O. Special reinforced masonry shear walls	2106.1.1.5	5½	2½	4	NL	NL	160	160	100
P. Intermediate reinforced masonry shear walls	2106.1.1.4	4	2½	2½	NL	NL	NP	NP	NP
Q. Ordinary reinforced masonry shear walls	2106.1.1.2	3	2½	2¼	NL	160	NP	NP	NP
R. Detailed plain masonry shear walls	2106.1.1.3	2½	2½	2¼	NL	NP	NP	NP	NP
S. Ordinary plain masonry shear walls	2106.1.1.1	1½	2½	1¼	NL	NP	NP	NP	NP
T. Light frame walls with shear panels—wood structural panels/sheet steel panels	2306.4.1/ 2211	6½	2½	4½	NL	NL	65	65	65
U. Light frame walls with shear panels—all other materials	2306.4.5	2½	2½	2½	NL	NL	35	NP	NP
3. Moment-resisting Frame Systems									
A. Special steel moment frames	(9)[j]	8	3	5½	NL	NL	NL	NL	NL
B. Special steel truss moment frames	(12)[j]	7	3	5½	NL	NL	160	100	NP
C. Intermediate steel moment frames	(10)[j]	6	3	5	NL	NL	160	100	NP[h]
D. Ordinary steel moment frames	(11)[j]	4	3	3½	NL	NL	35[h]	NP[h,i]	NP[h,i]
E. Special reinforced concrete moment frames	(21.1)[l]	8	3	5½	NL	NL	NL	NL	NL
F. Intermediate reinforced concrete moment frames	(21.1)[l]	5	3	4½	NL	NL	NP	NP	NP
G. Ordinary reinforced concrete moment frames	(21.1)[l]	3	3	2½	NL	NP	NP	NP	NP
H. Special composite moment frames	(9)[k]	8	3	5½	NL	NL	NL	NL	NL
I. Intermediate composite moment frames	(10)[k]	5	3	4½	NL	NL	NP	NP	NP
J. Composite partially restrained moment frames	(8)[k]	6	3	5½	160	160	100	NP	NP
K. Ordinary composite moment frames	(11)[k]	3	3	2½	NL	NP	NP	NP	NP
L. Masonry wall frames	2108.9.6 2106.1.1.6	5½	3	5	NL	NL	160	160	100

(continued)

TABLE 1617.6—continued

DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $\Omega_0{}^g$	DEFLECTION AMPLIFICATION FACTOR, $C_d{}^b$	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.3				
					A or B	C	D[d]	E[e]	F[e]
4. Dual Systems with Special Moment Frames									
A. Steel eccentrically braced frames, moment-resisting connections, at columns away from links	(15)[j]	8	2$2^1/_2$	4	NL	NL	NL	NL	NL
B. Steel eccentrically braced frames, nonmoment-resisting connections, at columns away from links	(15)[j]	7	$2^1/_2$	4	NL	NL	NL	NL	NL
C. Special steel concentrically braced frames	(13)[j]	8	$2^1/_2$	$6^1/_2$	NL	NL	NL	NL	NL
D. Ordinary steel concentrically braced frames	(14)[j]	6	$2^1/_2$	5	NL	NL	NL	NL	NL
E. Special reinforced concrete shear walls	1910.2.4	8	$2^1/_2$	$6^1/_2$	NL	NL	NL	NL	NL
F. Ordinary reinforced concrete shear walls	1910.2.3	7	$2^1/_2$	6	NL	NL	NP	NP	NP
G. Composite eccentrically braced frames	(14)[k]	8	$2^1/_2$	4	NL	NL	NL	NL	NL
H. Composite concentrically braced frames	(13)[k]	6	$2^1/_2$	5	NL	NL	NL	NL	NL
I. Composite steel plate shear walls	(17)[k]	8	$2^1/_2$	$6^1/_2$	NL	NL	NL	NL	NL
J. Special composite reinforced concrete shear walls with steel elements	(16)[k]	8	$2^1/_2$	$6^1/_2$	NL	NL	NL	NL	NL
K. Ordinary composite reinforced concrete shear walls with steel elements	(15)[k]	7	$2^1/_2$	6	NL	NL	NP	NP	NP
L. Special reinforced masonry shear walls	2106.1.1.5	7	3	$6^1/_2$	NL	NL	NL	NL	NL
M. Intermediate reinforced masonry shear walls	2106.1.1.4	$6^1/_2$	3	$5^1/_2$	NL	NL	NP	NP	NP
5. Dual Systems with Intermediate Moment Frames									
A. Special steel concentrically braced frames[f]	(13)[j]	6	$2^1/_2$	5	NL	NL	160	100	NP
B. Ordinary steel concentrically braced frames[f]	(14)[j]	5	$2^1/_2$	$4^1/_2$	NL	NL	160	100	NP
C. Special reinforced concrete shear walls	1910.2.4	6	$2^1/_2$	5	NL	NL	160	100	100
D. Ordinary reinforced concrete shear walls	1910.2.3	$5^1/_2$	$2^1/_2$	$4^1/_2$	NL	NL	NP	NP	NP
E. Ordinary reinforced masonry shear walls	2106.1.1.2	3	3	$2^1/_2$	NL	160	NP	NP	NP
F. Intermediate reinforced masonry shear walls	2106.1.1.4	5	3	$4^1/_2$	NL	NL	NP	NP	NP
G. Composite concentrically braced frames	(13)[k]	5	$2^1/_2$	$4^1/_2$	NL	NL	160	100	NP
H. Ordinary composite braced frames	(12)[k]	4	$2^1/_2$	3	NL	NL	NP	NP	NP
I. Ordinary composite reinforced concrete shear walls with steel elements	(15)[k]	$5^1/_2$	$2^1/_2$	$4^1/_2$	NL	NL	NP	NP	NP

(continued)

TABLE 1617.6—continued
DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC-FORCE-RESISTING SYSTEMS

BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, $\Omega_0{}^g$	DEFLECTION AMPLIFICATION FACTOR, $C_d{}^b$	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.3				
					A or B	C	D[d]	E[e]	F[e]
J. Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	21.1[l] 1910.2.3	5 1/2	2 1/2	5	NL	NP	NP	NP	NP
6. Inverted Pendulum Systems									
A. Cantilevered column systems		2 1/2	2	2 1/2	NL	NL	35	35	35
B. Special steel moment frames	(9)[j]	2 1/2	2	2 1/2	NL	NL	NL	NL	NL
C. Ordinary steel moment frames	(11)[j]	1 1/4	2	2 1/2	NL	NL	NP	NP	NP
D. Special reinforced concrete moment frames	21.1[l]	2 1/2	2	1 1/4	NL	NL	NL	NL	NL
7. Structural steel systems not specifically detailed for seismic resistance	AISC—ASD AISC—LRFD AISI AISC—HSS	3	3	3	NL	NL	NP	NP	NP

For SI: 1 foot = 304.8 mm, 1 pound per square foot = 0.0479 KN/m^2.

a. Response modification coefficient, R, for use throughout.

b. Deflection amplification factor, C_d.

c. NL = not limited and NP = not permitted.

d. See Section 1617.6.4.1 for a description of building systems limited to buildings with a height of 240 feet or less.

e. See Section 1617.6.4.1 for building systems limited to buildings with a height of 160 feet or less.

f. Ordinary moment frame is permitted to be used in lieu of intermediate moment frame in Seismic Design Categories B and C.

g. The tabulated value of the overstrength factor, Ω_0, may be reduced by subtracting 1/2 for structures with flexible diaphragms but shall not be taken as less than 2.0 for any structure.

h. Steel ordinary moment frames and intermediate moment frames are permitted in single story buildings up to a height of 60 feet, when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 15 pounds per square foot. The dead weight of the portion of walls more than 35 feet above the base shall not exceed 15 pounds per square foot.

i. Steel ordinary moment frames are permitted in buildings up to a height of 35 feet, where the dead load of the walls, floors and roof does not exceed 15 pounds per square foot.

j. AISC Seismic Part I or Part III, Section number.

k. AISC Seismic Part II, Section number.

l. ACI 318, Section number.

Response Modification Factor (R)

- Intended to account for inelastic deformations.
- Represent a measure of the amount of ductility allowed in various types of building
- Typical Values

R = 8.0	for ductile steel frames
7.0	for ductile concrete frames
4.5	for ordinary steel frames
2.0	for ordinary concrete frames
5.5	for RC shear walls
4.5	for reinforced masonry shear walls
1.5	for unreinforced masonry shear walls

Over Strength Factor (Ω_0)

- Typical Values

3.0	for ductile steel frames (Special Case)
2.5	for ductile concrete frames
2.5	for ordinary concrete frames
2.5	for RC shear walls
2.5	for reinforced masonry shear walls
2.5	for unreinforced masonry shear walls
2.0	for ordinary steel frames

UBC-97 vs. IBC2000

- **UBC-97**

$$V = C_s W$$

$$C_s = \frac{C_v I}{RT}$$

C_v = 0.05 to 0.5
I = 1.0 to 1.5
R= 2.2 to 8

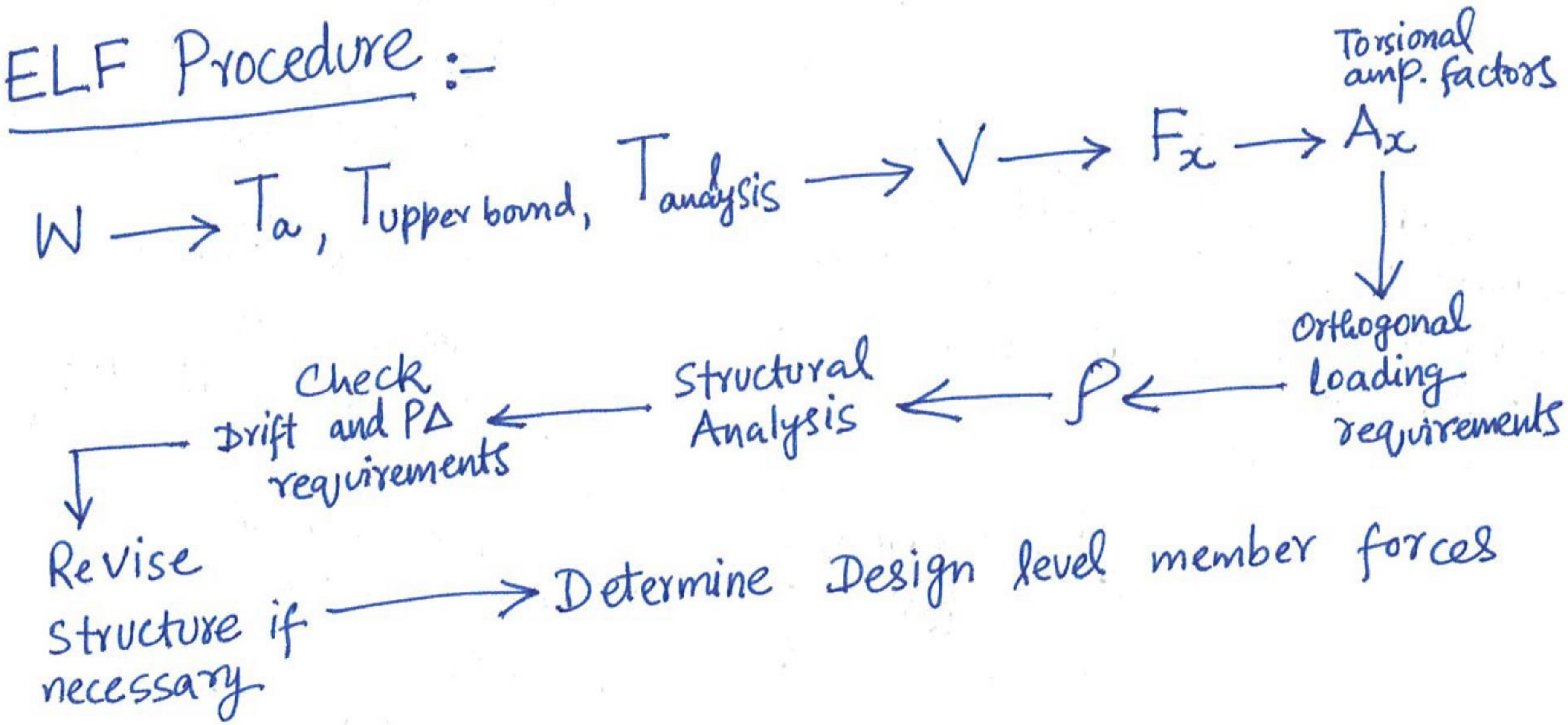
- **IBC-2000**

$$V = CsW$$

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I_E}\right)} = \frac{S_{DS} I_E}{R} \qquad C_S = \frac{S_{DI}}{\left(\frac{R}{I_E}\right)T} = \frac{S_{DI} I}{RT}$$

S_{DS} = 0.13 to nearly 1.0
I_E = 1 to 1.5
R= 4 to 8
S_{D1}= 0.05 to nearly 0.5

The ELF Procedure in ASCE 7-10



The Equivalent Lateral Force (ELF) Procedure

- The ELF Procedure in IBC 2003
  - *Check yourself*
- The ELF Procedure in ASCE 7-05
  - *Check yourself*
- The ELF Procedure in IBC 2006
  - *Check yourself*
- The ELF Procedure in IBC 2009
  - *Check yourself*
- The ELF Procedure in ASCE 7-10
  - *Check yourself*
- The ELF Procedure in IBC 2012
  - *Check yourself*
- The ELF Procedure in ASCE 7-16
  - *Check yourself*

…

- The ELF Procedure in EC 2003/2008
  - *Check yourself*
- The ELF Procedure in AS Codes
  - *Check yourself*
- The ELF Procedure in BS 8110
  - *Check yourself*
- The ELF Procedure in CSA Codes
  - *Check yourself*
- The ELF Procedure in IS Codes
  - *Check yourself*
- The ELF Procedure in MNBC
  - *Check yourself*
- The ELF Procedure in NZS
  - *Check yourself*

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The Seismic Design Procedure in BCP (2007)

GOVERNMENT OF ISLAMIC REPUBLIC OF PAKISTAN
MINISTRY OF HOUSING & WORKS, ISLAMABAD

Building Code of Pakistan

(Seismic Provisions - 2007)

Table 2.1-Seismic Zones

Seismic Zone	Peak Horizontal Ground Acceleration
1	0.05 to 0.08g
2A	0.08 to 0.16g
2B	0.16 to 0.24g
3	0.24 to 0.32g
4	> 0.32g

Where “g” is the acceleration due to gravity.

Table 4.1-Soil Profile Types

<table>
<tr><th rowspan="2">Soil Profile Type</th><th rowspan="2">Soil Profile Name/ Generic Description</th><th colspan="3">Average Properties for Top 30 M (100 ft) of Soil Profile</th></tr>
<tr><th>Shear Wave Velocity, v_s m/sec (ft/sec)</th><th>Standard Penetration Tests, N [or N_{CH} for cohesionless soil layers] (blows/foot)</th><th>Undrained Shear Strength, s_u kPa (psf)</th></tr>
<tr><td>S_A</td><td>Hard Rock</td><td>>1,500 (>4,920)</td><td rowspan="2">–</td><td rowspan="2">–</td></tr>
<tr><td>S_B</td><td>Rock</td><td>750 to 1,500 (2,460 to 4,920)</td></tr>
<tr><td>S_C</td><td>Very Dense Soil and Soft Rock</td><td>350 to 750 (1,150 to 2,460)</td><td>>50</td><td>>100 (>2,088)</td></tr>
<tr><td>S_D</td><td>Stiff Soil Profile</td><td>175 to 350 (575 to 1,150)</td><td>15 to 50</td><td>50 to 100 (1,044 to 2,088)</td></tr>
<tr><td>S_E[1]</td><td>Soft Soil Profile</td><td><175 (<575)</td><td><15</td><td><50 (<1,044)</td></tr>
<tr><td>S_F</td><td colspan="4">Soil requiring Site-specific Evaluation. See 4.4.2</td></tr>
</table>

1 Soil Profile Type S_E also includes any soil profile with more than 3 m (10 ft) of soft clay defined as a soil with a plasticity index, PI > 20, $w_{mc} \geq 40$ percent and $s_u < 25$ kPa (522 psf). The Plasticity Index, *PI*, and the moisture content, w_{mc}, shall be determined in accordance with the latest ASTM procedures.

5.30.2 Static Force Procedure

5.30.2.1 Design base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v I}{RT} W \tag{5.30-4}$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \tag{5.30-5}$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a I W \tag{5.30-6}$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 Z N_v I}{R} W \tag{5.30-7}$$

5.30.2.2 *Structure period.* The value of T shall be determined from one of the following methods:

1. *Method A*: For all buildings, the value T may be approximated from the following formula:

$$T = C_t (h_n)^{\frac{3}{4}} \tag{5.30-8}$$

Where:

C_t = 0.0853 (0.035) for steel moment-resisting frames.
C_t = 0.0731 (0.030) for reinforced concrete moment-resisting frames and eccentrically braced frames.
C_t = 0.0488 (0.020) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $\dfrac{0.0743}{\sqrt{A_c}}$ (For FPS: $\dfrac{0.1}{\sqrt{A_c}}$ for A_c in ft^2). The value of A_c shall be determined from the following formula:

$$A = \sum A_e \left[0.2 + \left(\frac{D_e}{h_n} \right)^2 \right] \tag{5.30-9}$$

The value of D_e / h_n used in Formula (5.30-9) shall not exceed 0.9.

2. *Method B*: The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 5.30.1.2. The value of T from Method B shall not exceed a value 30 percent greater than the value of T obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period T may be computed by using the following formula:

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^{\,2}\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)} \qquad (5.30\text{-}10)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (5.30-13), (5.30-14) and (5.30-15) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

5.30.3 *Determination of Seismic Factors*

5.30.3.1 *Determination of Ω_o.* For specific elements of the structure, as specifically identified in this code, the minimum design strength shall be the product of the seismic force overstrength factor Ω_o and the design seismic forces set forth in Section 5.30. For both Allowable Stress Design and Strength Design, the Seismic Force Overstrength Factor, Ω_o, shall be taken from Table 5.13.

5.30.3.2 *Determination of R.* The notation R shall be taken from Table 5.13.

5.30.5 *Vertical Distribution of Force*

The total force shall be distributed over the height of the structure in conformance with Formulas (5.30-13), (5.30-14) and (5.30-15) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \tag{5.30-13}$$

The concentrated force F_t at the top, which is in addition to F_n, shall be determined from the formula:

$$F_t = 0.07TV \tag{5.30-14}$$

The value of T used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Formula (5.30-4). F_t need not exceed $0.25V$ and may be considered as zero where T is 0.7 second or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n, according to the following formula:

$$F_x = (V - F_t)\frac{w_x h_x}{\sum_{i=1}^{n}(w_i h_i)} \tag{5.30-15}$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

5.31.5 *Response Spectrum Analysis.*

5.31.5.1 *Response spectrum representation and interpretation of results.* The ground motion representation shall be in accordance with Section 5.31.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 5.31.5.4.

5.31.5.2 *Number of modes.* The requirement of Section 5.31.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

5.31.5.3 *Combining modes.* The peak member forces, displacements, storey forces, storey shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

5.31.5.4 *Reduction of Elastic Response Parameters for design*
Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of R.

Design Response Spectrum (UBC 1997, p 2-38)

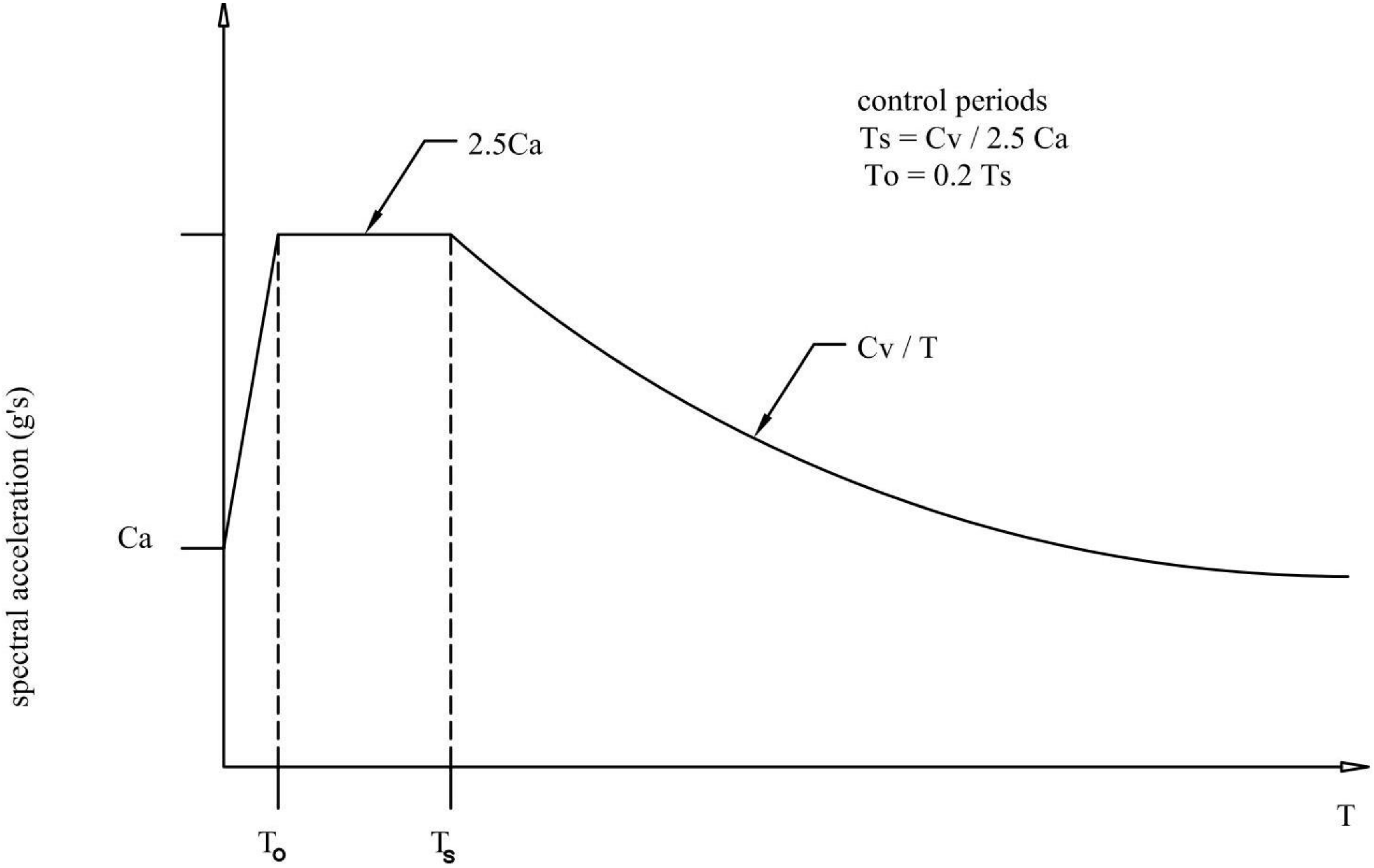


Table 5.9 – Seismic Zone Factor Z

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

Note: The zone shall be determined from the seismic zone map in Figure 2-1 or from Table 2.2.

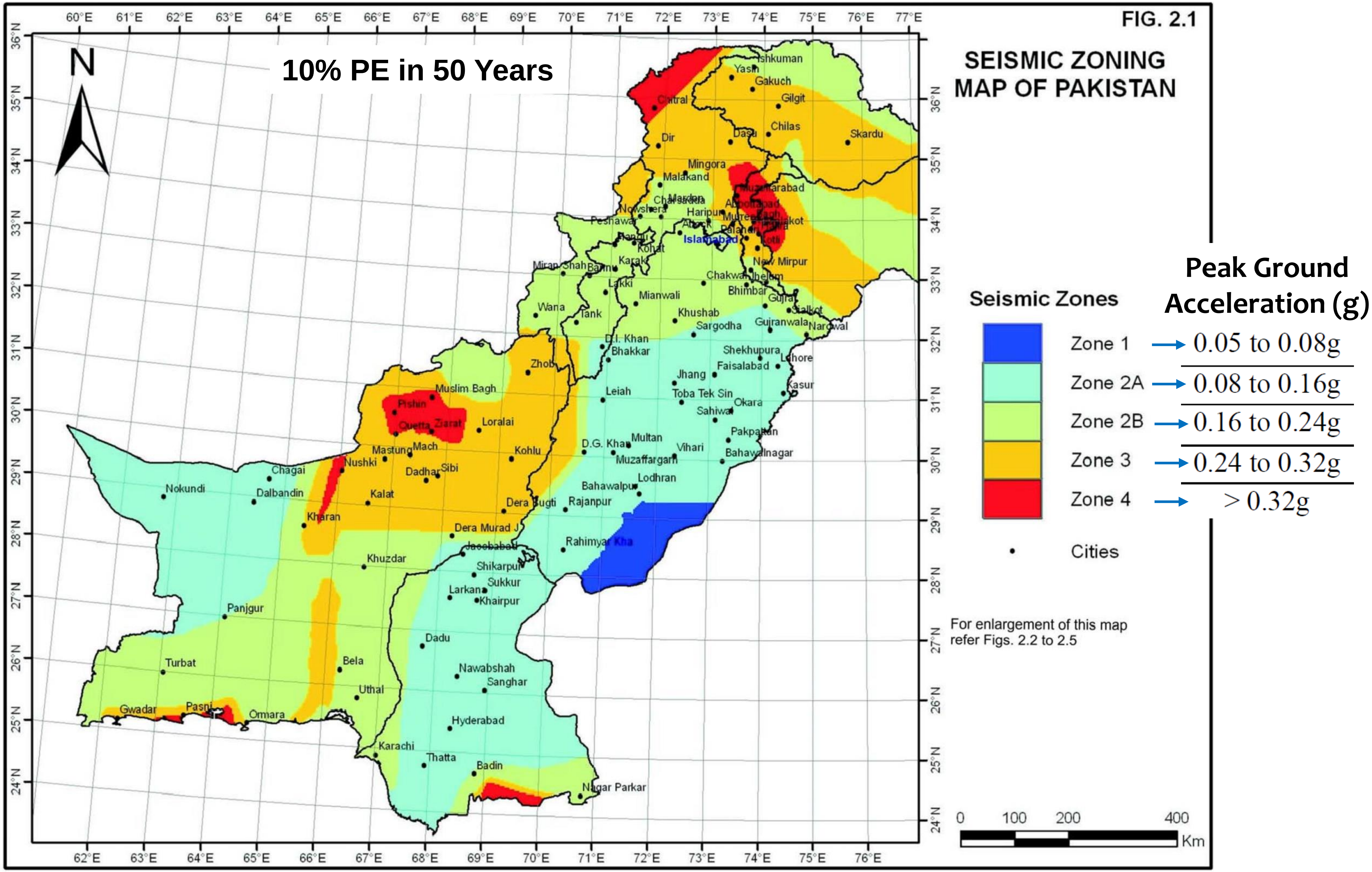


FIG. 2.1
SEISMIC ZONING MAP OF PAKISTAN
10% PE in 50 Years
N
Seismic Zones
Peak Ground Acceleration (g)
Zone 1 → 0.05 to 0.08g
Zone 2A → 0.08 to 0.16g
Zone 2B → 0.16 to 0.24g
Zone 3 → 0.24 to 0.32g
Zone 4 → > 0.32g
Cities
For enlargement of this map refer Figs. 2.2 to 2.5
0 100 200 400
Km
Ishkuman
Yasin
Gakuch
Gilgit
Chitral
Chilas
Dasu
Skardu
Dir
Mingora
Malakand
Muzaffarabad
Charsadda
Mardan
Abbottabad
Nowshera
Haripur
Murree
Bagh
Peshawar
Attock
Islamabad
Kotli
Kohat
Hangu
Karak
New Mirpur
Miran Shah
Bannu
Chakwal
Jhelum
Lakki
Bhimbar
Gujrat
Mianwali
Sialkot
Wana
Tank
Khushab
Sargodha
Gujranwala
Narowal
D.I. Khan
Bhakkar
Shekhupura
Lahore
Faisalabad
Zhob
Jhang
Kasur
Muslim Bagh
Leiah
Toba Tek Sin
Okara
Pishin
Sahiwal
Quetta
Ziarat
Loralai
Pakpattan
Mastung
Mach
D.G. Khan
Multan
Vihari
Kohlu
Muzaffargarh
Bahawalnagar
Nushki
Chagai
Dadhar
Sibi
Lodhran
Nokundi
Dalbandin
Kalat
Bahawalpur
Dera Bugti
Rajanpur
Kharan
Dera Murad J
Jacobabad
Rahimyar Kha
Khuzdar
Shikarpur
Sukkur
Larkana
Khairpur
Panjgur
Dadu
Bela
Nawabshah
Turbat
Sanghar
Uthal
Gwadar
Pasni
Ormara
Hyderabad
Karachi
Thatta
Badin
Nagar Parkar
60°E 61°E 62°E 63°E 64°E 65°E 66°E 67°E 68°E 69°E 70°E 71°E 72°E 73°E 74°E 75°E 76°E 77°E
24°N 25°N 26°N 27°N 28°N 29°N 30°N 31°N 32°N 33°N 34°N 35°N 36°N

Table 5.10 – Occupancy Category[4]

Occupancy Category	Occupancy Or Functions of Structure	Seismic Importance Factor, I	Seismic Importance[1] Factor, I_p	Wind Importance Factor, I_w
1. Essential facilties[2]	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1,2,6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1,2 or 7 Occupancy.	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1,2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation.	1.00	1.00	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 5.33.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 6.2.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.
[4]See Table 5.21

Table 5.13 – Structural Systems[1]

Basic Structural System[2]	Lateral-Force-Resisting System Description	R	Ω_O	Height Limit for Seismic Zones 3 And 4 (m)	Height Limit for Seismic Zones 3 And 4 (ft)
1. Bearing wall system	1. Light-framed walls with shear panels				
	a. Wood structural panel walls for structures three storeys or less	5.5	2.8	20	65
	b. All other light-framed walls	4.5	2.8	20	65
	2. Shear walls				
	a. Concrete	4.5	2.8	50	160
	b. Masonry	4.5	2.8	50	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	20	65
	4. Braced frames where bracing carries gravity load				
	a. Steel	4.4	2.2	50	160
	b. Concrete[3]	2.8	2.2	-	-
	c. Heavy timber	2.8	2.2	20	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	75	240
	2. Light-framed walls with shear panels				
	a. Wood structural panel walls for structures three storeys or less	6.5	2.8	20	65
	b. All other light-framed walls	5.0	2.8	20	65
	3. Shear walls				
	a. Concrete	5.5	2.8	75	240
	b. Masonry	5.5	2.8	50	160
	4. Ordinary braced frames				
	a. Steel	5.6	2.2	50	160
	b. Concrete[3]	5.6	2.2	-	-
	c. Heavy timber	5.6	2.2	20	65
	5. Special concentrically braced frames				
	a. Steel	6.4	2.2	75	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)				
	a. Steel	8.5	2.8	N.L	N.L.
	b. Concrete[4]	8.5	2.8	N.L	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	50	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	-	-
	4. Ordinary moment-resisting frame (OMRF)				
	a. Steel[6]	4.5	2.8	50	160
	b. Concrete[7]	3.5	2.8	-	-
	5. Special truss moment frames of steel (STMF)	6.5	2.8	75	240
4. Dual systems	1. Shear walls				
	a. Concrete with SMRF	8.5	2.8	N.L	N.L.
	b. Concrete with steel OMRF	4.2	2.8	50	160
	c. Concrete with concrete IMRF[5]	6.5	2.8	50	160
	d. Masonry with SMRF	5.5	2.8	50	160
	e. Masonry with steel OMRF	4.2	2.8	50	160
	f. Masonry with concrete IMRF[3]	4.2	2.8	-	-
	g. Masonry with masonry MMRWF	6.0	2.8	50	160
	2. Steel EBF				
	a. With steel SMRF	8.5	2.8	N.L	N.L.
	b. With steel OMRF	4.2	2.8	50	160
	3. Ordinary braced frames				
	a. Steel with steel SMRF	6.5	2.8	N.L	N.L.
	b. Steel with Steel OMRF	4.2	2.8	50	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	-	-
	d. Concrete with concrete IMRF[3]	4.2	2.8	-	-
	4. Special concentrically braced frames				
	a. Steel with steel SMRF	7.5	2.8	N.L	N.L.
	b. Steel with steel OMRF	4.2	2.8	50	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	11[7]	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	50	160
7. Undefined systems	See Sections 5.29.6.7 and 5.29.9.2	-	-	-	-

Table 5.16 – Seismic Coefficients C_a

Soil Profile Type[2]	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1] Site Specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

[2] For soil profile types, See Table 4.1.

Table 5.17 – Seismic Coefficient C_v

Soil Profile Type[2]	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1] Site Specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

[2] For soil profile types, See Table 4.1.

Table 5.18 – Near Source Factor N_a[1]

Seismic Source Type	Closest Distance To Known Seismic Source[2,3]		
	≤ 2 km	5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

[1] The Near Source Factor may be based on the linear interpolation of values for distance other than those shown in the table.
[2] The location and type of seismic sources to be used for design shall be established based on approved geotechnical data.
[3] The closet distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all source shall be used for design.

Table 5.19 – Near Source Factor N_v[1]

Seismic Source Type	Closest Distance To Known Seismic Source[2,3]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

[1] The Near Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.
[2] The location and type of seismic sources to be used for design shall be established based on approved geotechnical data.
[3] The closet distance to seismic source shall be taken as the minimum distance between the site and the areas described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

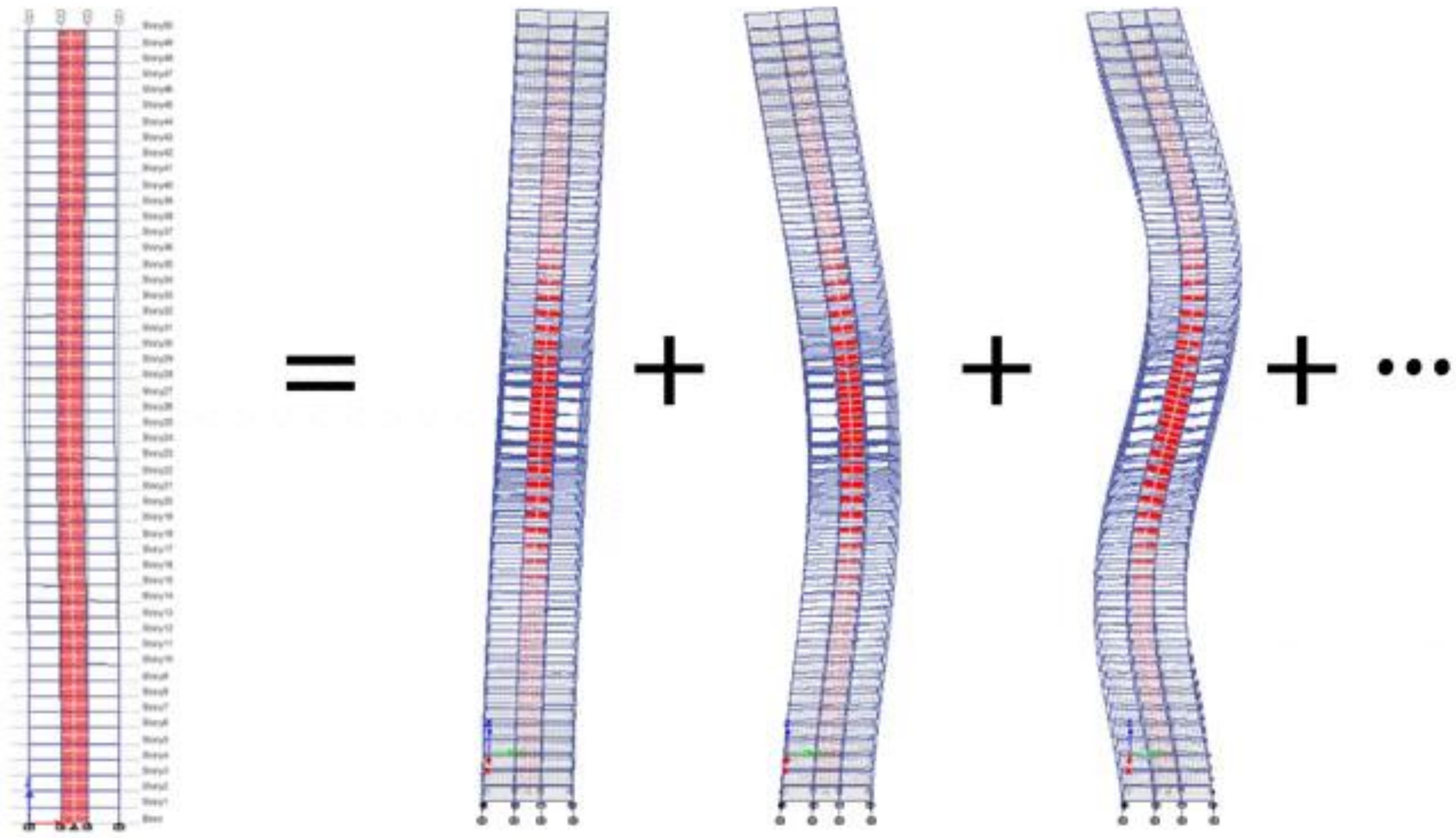
Table 5.20 – Seismic Source Type[1]

Seismic Source Type	Seismic Source Description	Seismic Source Definition[2]	
		Maximum Moment Magnitude, M	**Slip Rate, SR (mm/year)**
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

[1] Subduction sources shall be evaluated on a site-specific basis.
[2] Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining the seismic source type.

The Concept of Vibration Modes of a Structure

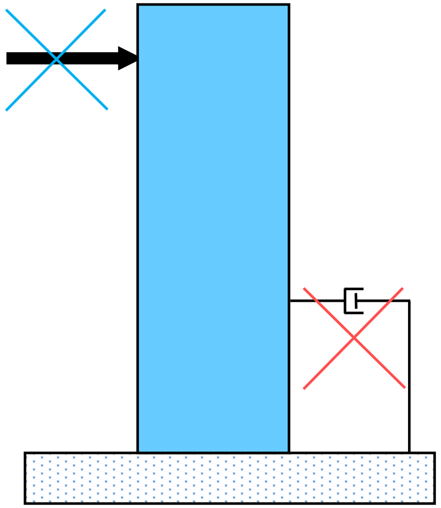
The Basic Concept



Free Vibration Analysis

- Definition
  - Natural vibration of a structure released from initial condition and subjected to no external load or damping
- Main governing equation -Eigenvalue Problem

$$[M]\{\ddot{u}\}_t + \cancel{[c]\{\dot{u}\}_t} + [K]\{u\}_t = \cancel{\{P\}_t}$$



- Solution gives
  - Natural Frequencies
  - Associated mode shapes
  - An insight into the dynamic behavior and response of the structure

Natural Periods or Frequency

- The **heartbeat** of the structure
- Indicates the **“stiffness” and “mass” relationship**
- Basis for **damping, resonance and amplification effects**
- Many relationships for tall buildings (0.1 N, with Height etc.)

Mode Shapes

- A mode shape is a set of **relative (not absolute) nodal displacements** for a particular mode of free vibration for a specific natural frequency
- There are as many modes as there are DOF in the system
- **Not** all of the modes are significant
- **Local modes** may disrupt the modal mass participation

Modal Analysis

- The modal analysis determines the **inherent** natural frequencies of vibration
- Each natural frequency is related to a time period and a mode shape
- Time Period is the time it takes to complete one cycle of vibration
- The **Mode Shape** is **normalized deformation pattern**
- The number of Modes is typically equal to the number of Degrees of Freedom
- The Time Period and Mode Shapes are inherent properties of the structure and **do not depend on the applied loads**

Modal Analysis

- The Modal Analysis should be run **before applying loads** any other analysis to check the model and to understand the response of the structure.
- Modal analysis is **precursor** to most types of analysis including Gravity Load Analysis, Response Spectrum Analysis, Time History Analysis, Push-over Analysis, etc.
- Modal analysis is a useful tool even if full Dynamic Analysis is not performed.
- Modal analysis is easy to run and is **fun to watch** when animated.

Application of Modal Analysis

- **The Time Period and Mode Shapes, together with animation immediately exhibit the strengths and weaknesses of the structure.**

- **Modal analysis can be used to check the accuracy of the structural model**
  - The Time Period should be within reasonable range,
  - The disconnected members are identified
  - Local modes are identified that may need suppression

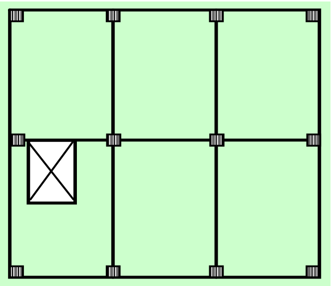
- **The symmetry of the structure can be determined**
  - For doubly symmetrical buildings, generally the first two modes are translational and the third mode is rotational
  - If the first mode is rotational, the structural is un-symmetrical

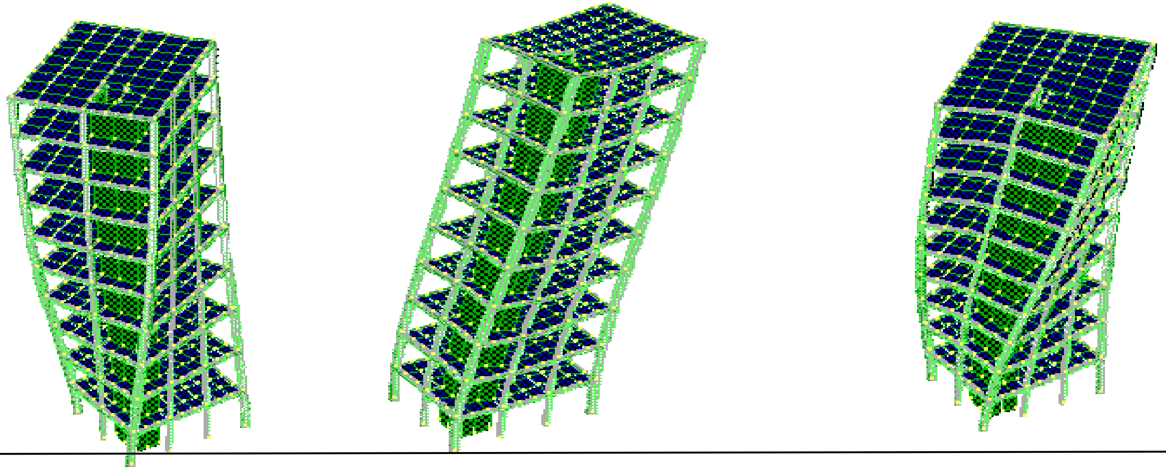
- **The resonance with the applied loads or excitation can be avoided**
  - The natural frequency of the structure should not be close to excitation frequency

Eccentric and Concentric Response

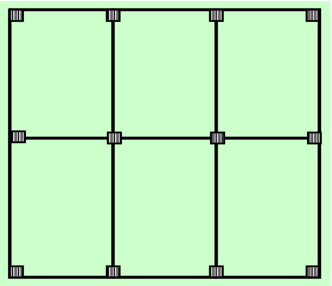
Unsymmetrical Mass and Stiffness

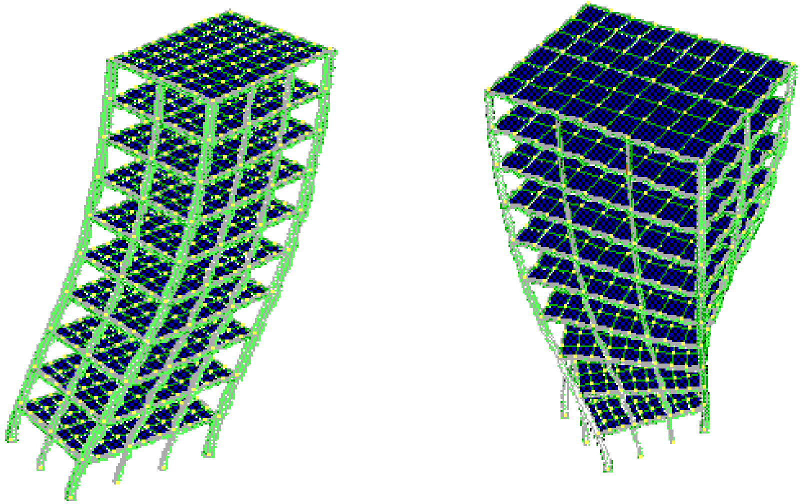
Mode-1 Mode-2 Mode-3



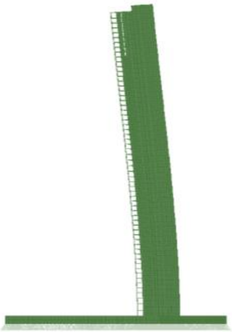
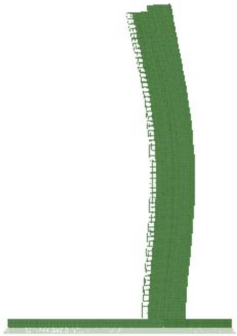
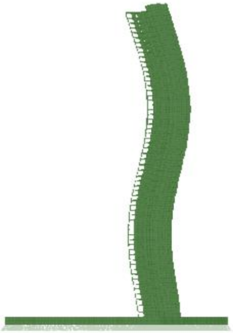
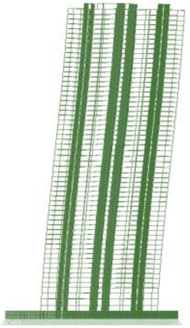
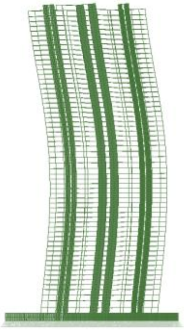
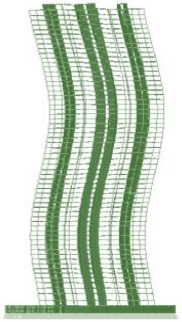
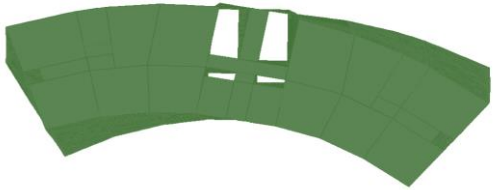
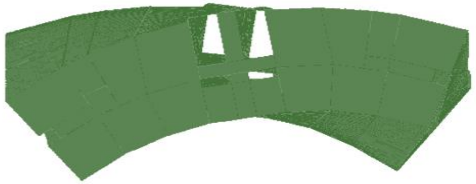
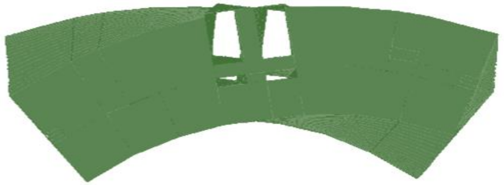


Symmetrical Mass and Stiffness





Modal Analysis Results

Translation in Minor	 • T_1 = 5.32 sec • 60% in Minor direction	 • T_6 = 1.28 sec • 18% in Minor direction	 • T_9 = 0.75 sec • 6.5% in Minor direction
Translation in Major	 • T_2 = 4.96 sec • 66% in Major direction	 • T_4 = 1.56 sec • 15% in Major direction	 • T_7 = 0.81 sec • 5.2% in Major direction
Torsional	T_3 = 4.12 sec 	T_5 = 1.30 sec 	T_8 = 0.65sec 

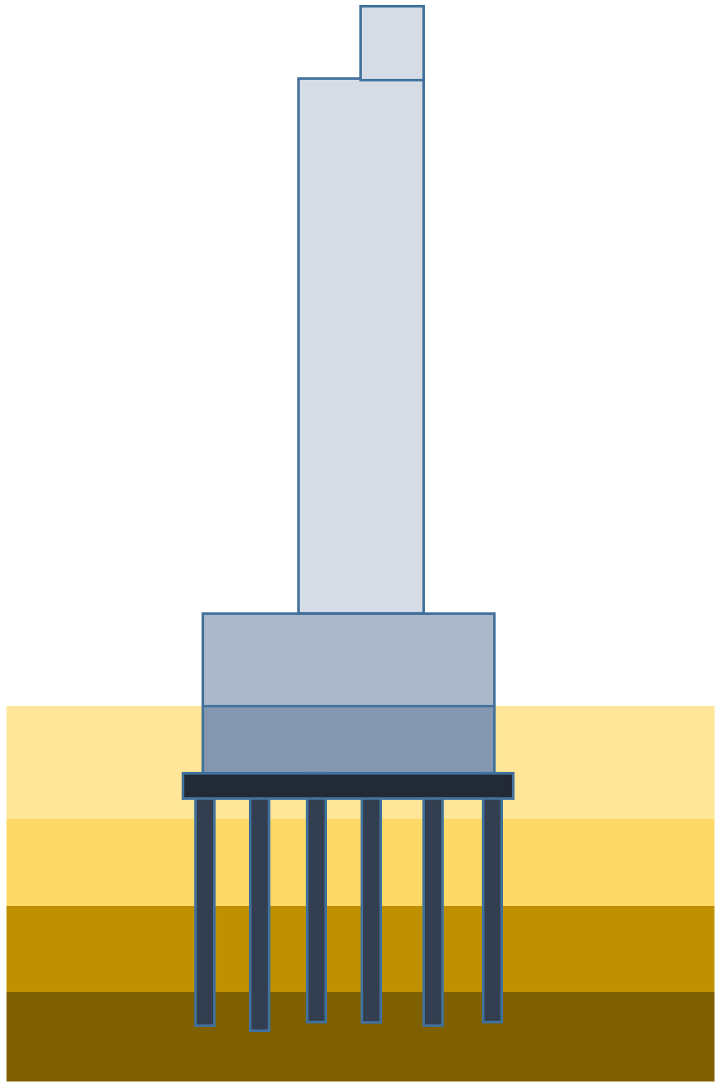
Modal Response Influenced by

Mass

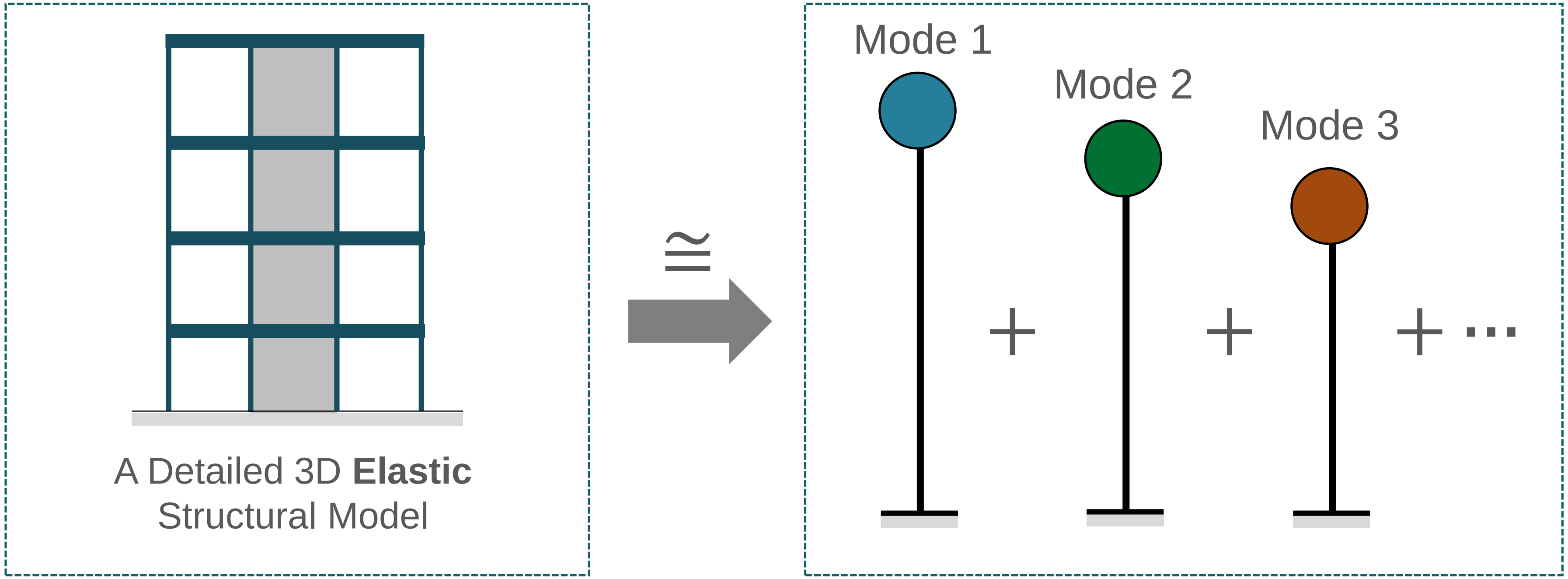
- Structure
- Attachments
- Occupants

Stiffness

- Tower
- Podium
- Basement
- Foundation
- Soil



The Classical Modal Analysis Procedure for Forced Vibrations



Modal Analysis

- To determine vibration modes of building
- To understand behaviour of building in schematic design stage
  - Adequacy of lateral stiffness
  - Minimize torsional response under earthquake
  - Tune to structure to be dynamically regular
  - Determine the principal directions of building
- Mass source

1.0 DL + 1.0 LL MEP + 0.25 LL STO

- DL = Dead load
- LL MEP = MEP and other permanent equipment live load
- LL STO = Storage live load

The Response Spectrum Analysis (RSA) Procedure

The Concept of Response Spectrum

The governing equation of motion of an SDF system subjected to a ground motion $\ddot{u}_g(t)$ can be written as follows.

$$m\,\ddot{u}(t) + c\,\dot{u}(t) + k\,u(t) = -m\,\ddot{u}_g(t)$$

$$c = 2\,m\,\xi\,\omega \quad , \quad \omega^2 = \frac{k}{m}$$

$$\ddot{u}(t) + 2\,\xi\,\omega\,\dot{u}(t) + \omega^2 u(t) = -\ddot{u}_g(t)$$

Solution

$$u(t, T, \xi)$$
$$\dot{u}(t, T, \xi)$$
$$\ddot{u}(t, T, \xi)$$

Output of the above equation (u, $\dot{u}$, $\ddot{u}$) are the dynamic response to the ground motion for a structure considered as a single DOF

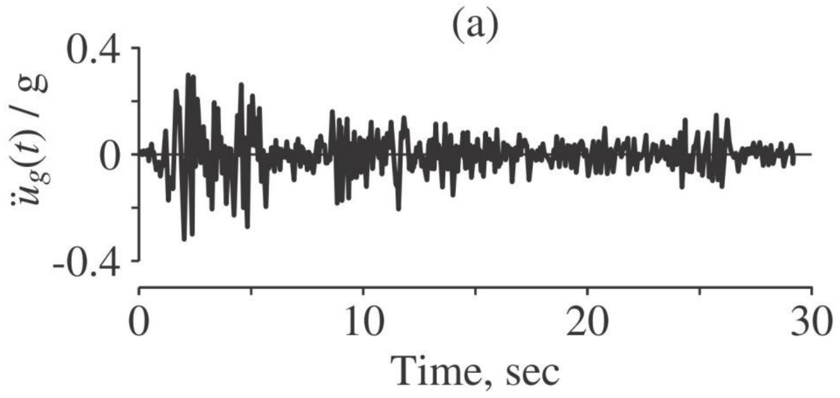
A plot of the "**maximum**" response for different ground motion history, different time period and damping ratio give the "**Spectrum of Response**"

$$u_o(T,\xi) = max|u(t,T,\xi)|$$

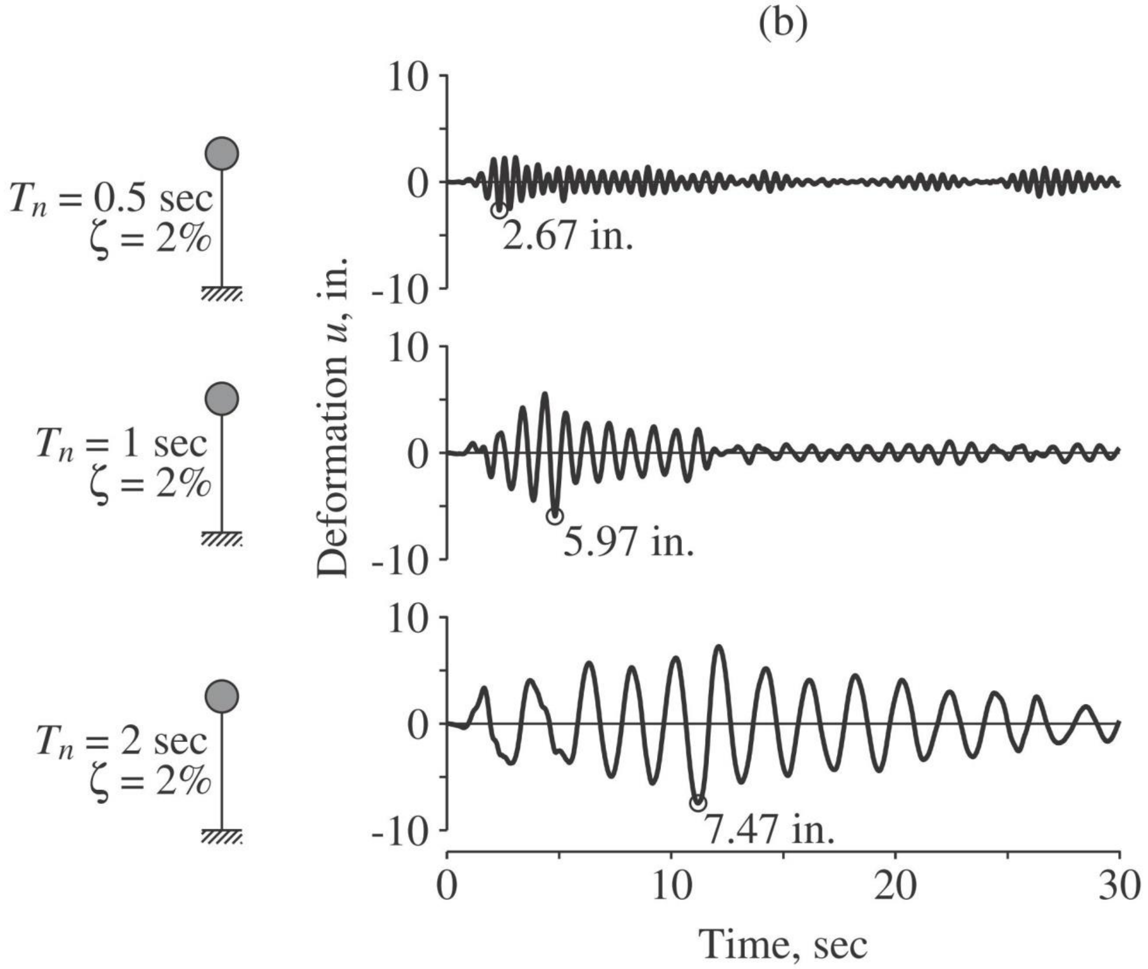
$$\dot{u}_o(T,\xi) = max|\dot{u}(t,T,\xi)|$$

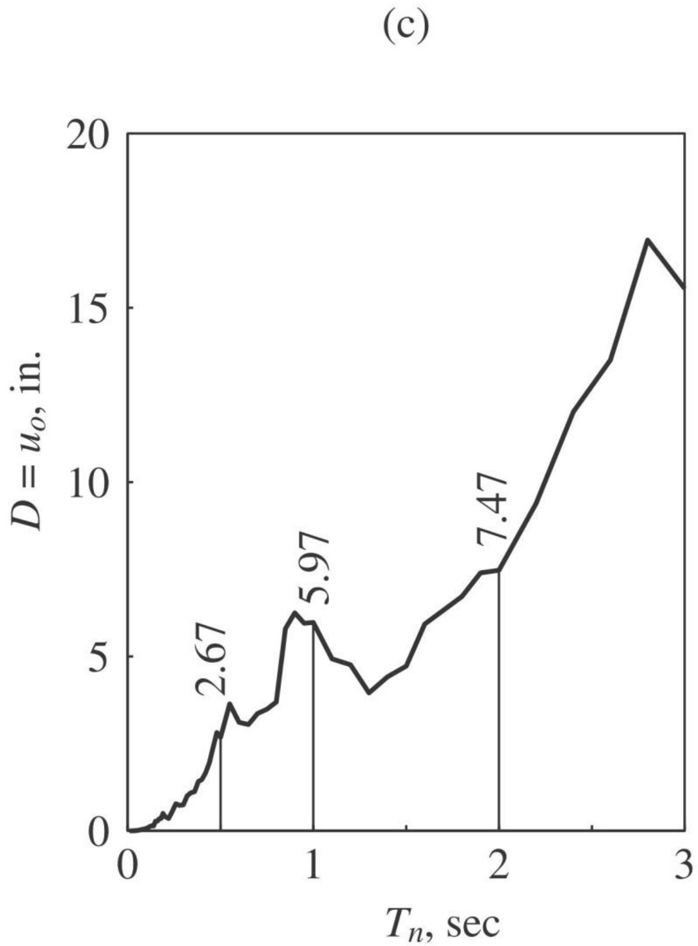
$$\ddot{u}_o(T,\xi) = max|\ddot{u}(t,T,\xi)|$$

The deformation response spectrum is a plot of u_o against T for fixed ξ. A similar plot for $\dot{u}_o$ is the velocity response spectrum, and for $\ddot{u}_o$ is the acceleration response spectrum.



(a) Ground acceleration; (b) deformation response of three SDF systems with ξ = 2% and T = 0.5, 1, and 2 sec; (c) deformation response spectrum for ξ = 2%.





Elastic Response Spectra

If a record of ground acceleration $\ddot{u}_g(t)$ is known, then the deformation response of a linearly elastic SDOF system can be computed by the convolution integral (See Eq.(27)), and internal forces of interest to structural engineers such as bending moments, shears can be subsequently determined.

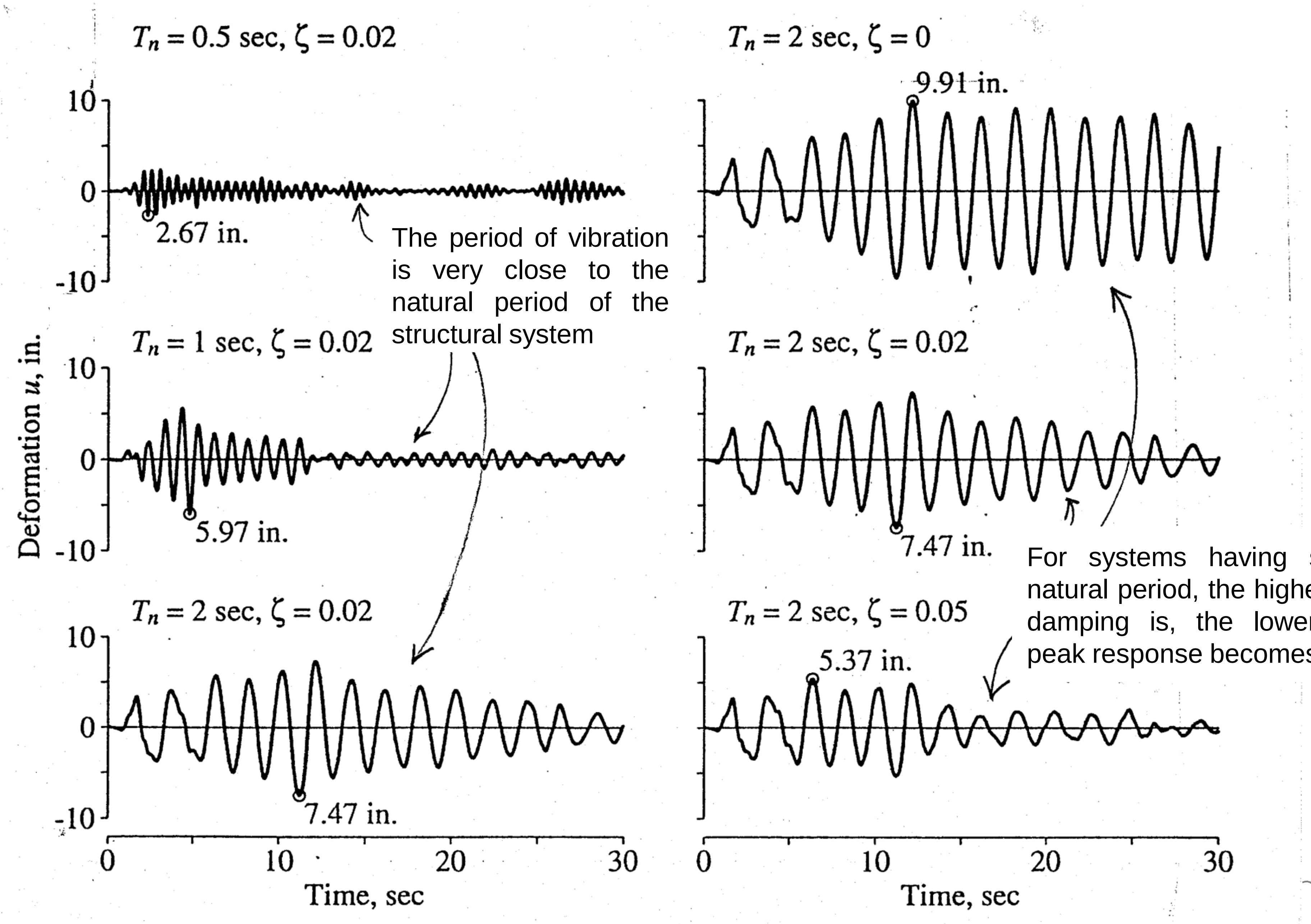
Equation of motion: $$m\,\ddot{u} + c\,\dot{u} + k\,u = -m\,\ddot{u}_g(t)$$

The equation can also be written in the form of

$$\ddot{u} + 2\,\xi\omega_n\,\dot{u} + \omega_n^2\,u = -\ddot{u}_g(t) \qquad (29)$$

It turns out that, for a given ground acceleration $\ddot{u}_g(t)$, the deformation response depends only on ω_n (or T_n) and ξ of the SDOF system.

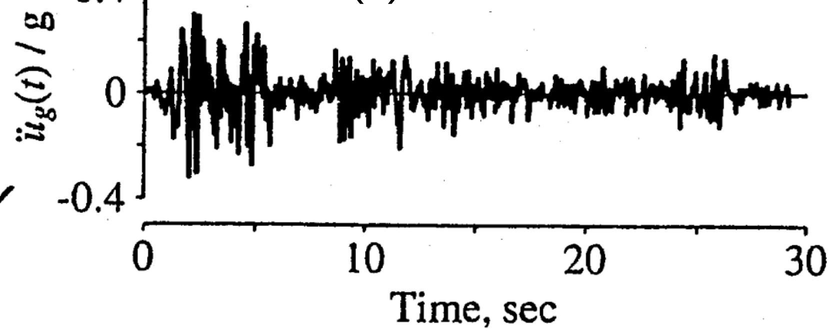
Deformation Response of SDOF systems to the El Centro Ground Motion



The period of vibration is very close to the natural period of the structural system

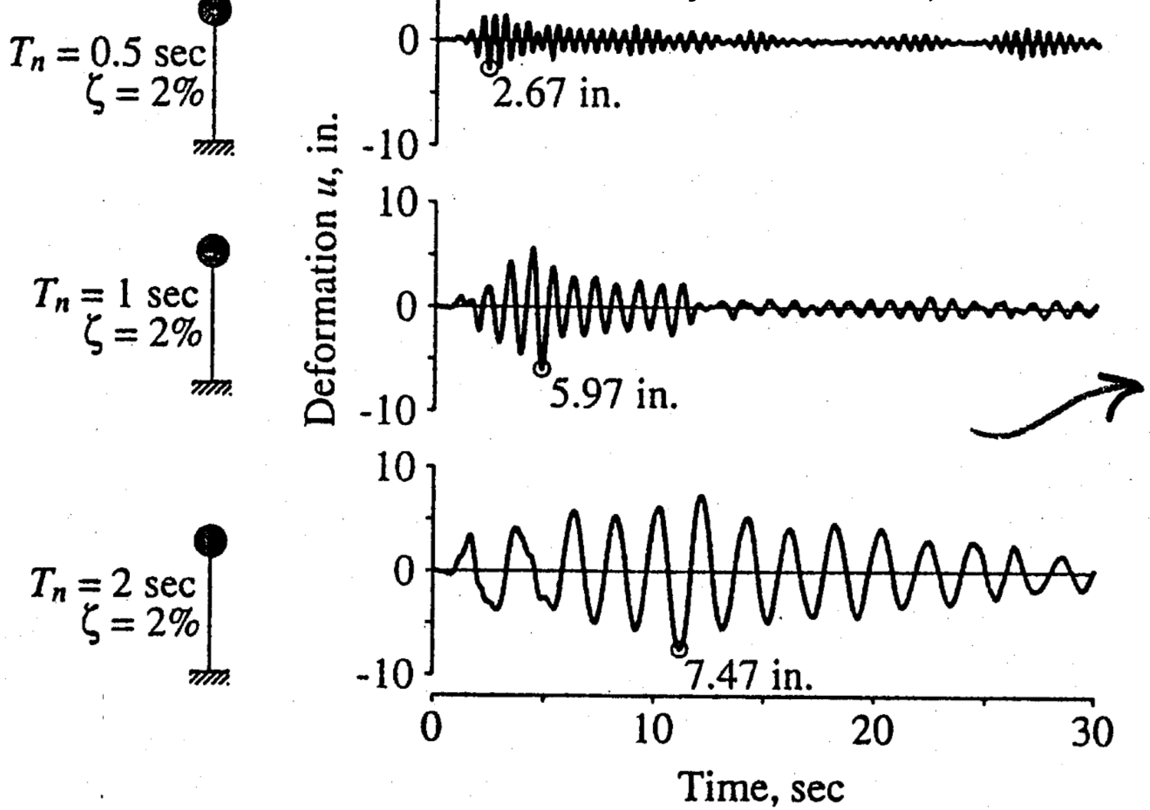
For systems having same natural period, the higher the damping is, the lower the peak response becomes.

(a) Ground acceleration



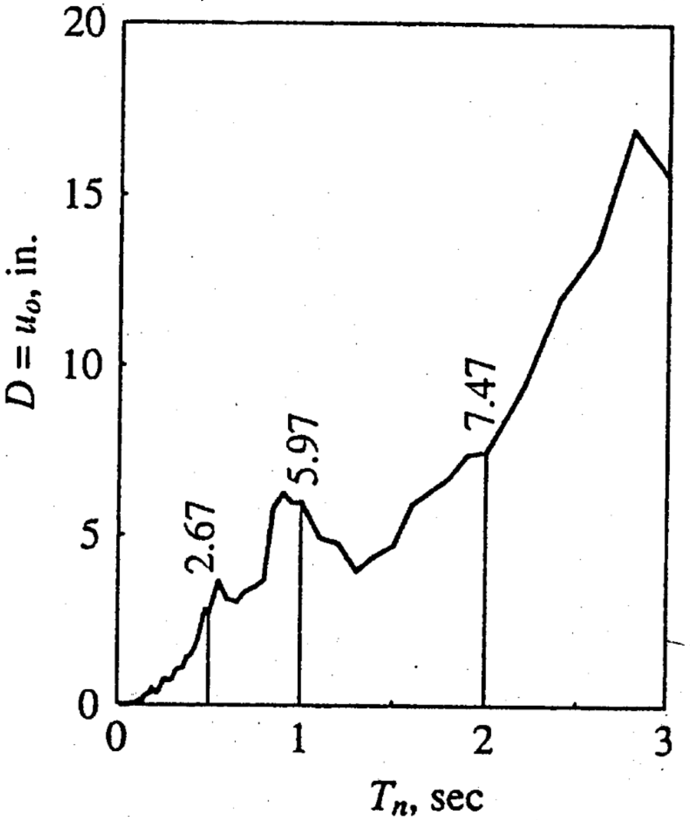
(b)

Deformation response of three SDOF systems with ξ = 0.02

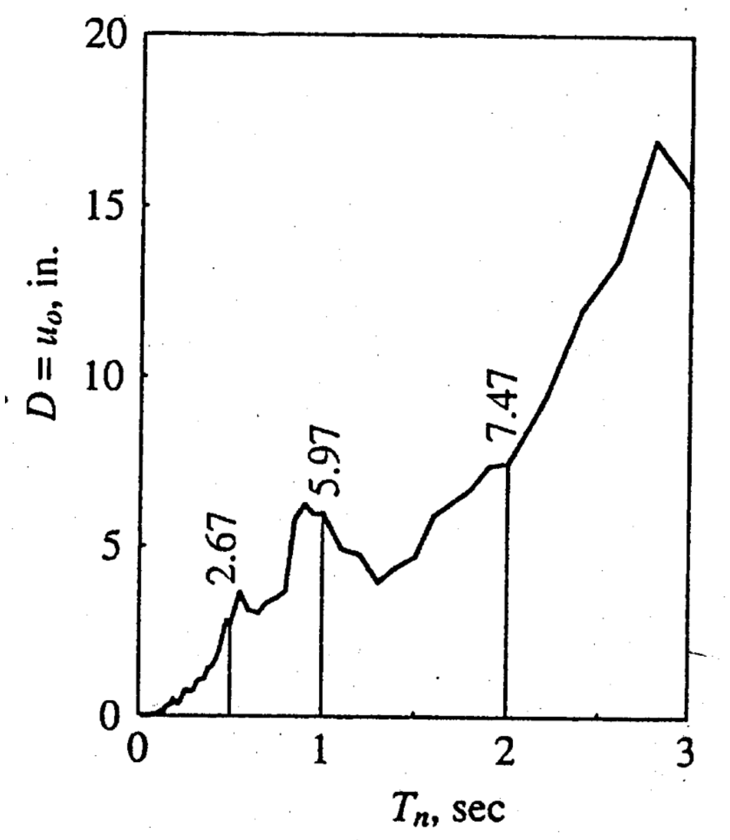


(c)

Deformation response spectrum for ξ = 0.02



Deformation response spectrum for ξ = 0.02



Response Spectrum: A plot of the peak value of a response quantity as a function of the natural vibration period T_n of the system, or related parameter, is called the response spectrum for the quantity.

The response spectrum provides a convenient mean to summarize the peak response of all possible linear SDOF systems to a particular component of ground motion.

It also provides **a practical approach to apply the knowledge of structural dynamics to the design of structures** and development of lateral force requirements in building codes.

Let u_0 be the peak displacement of SDOF system,

Once u_0 is obtained from the deformation response spectrum, the corresponding peak internal forces f_{s0} can be determined by:

$$f_{s0} = k\,u_0$$

or

$$f_{s0} = m\,\omega_n^2\,u_0 = m\,A \qquad (30)$$

where

$$A = \omega_n^2\,u_0 \qquad (31)$$

Note that f_{s0} is $m \times A$ not $m \times the\ peak\ value\ of\ acceleration\ (\ddot{u}_t)_0$

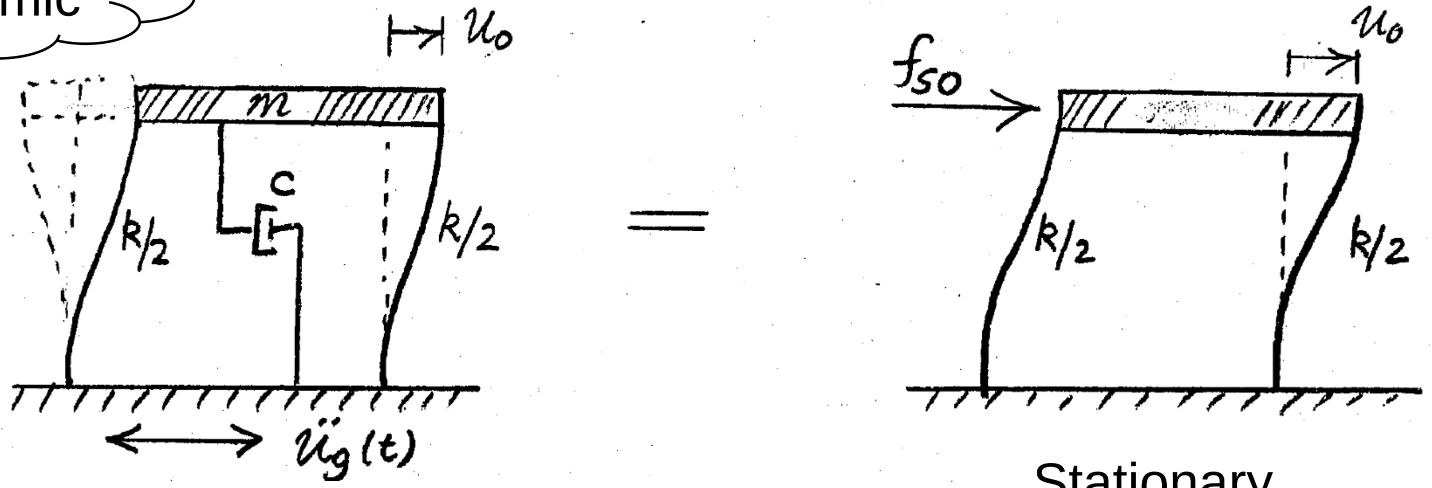
A is not the real peak acceleration response but it has units of acceleration.

A is called “Peak Pseudo-acceleration” or “Spectral Acceleration”.

f_{s0} can also be considered as an "**equivalent static force**" because if the force f_{s0} is applied to the structure statically it will produce the equivalent amount of peak deformation response u_0.

Dynamic

Static



at the instant where $u(t) = u_0$

Stationary

Let V_{b0} be the peak value of base shear

$$V_{b0} = f_{s0} = m\,A \qquad (32)$$

It can be written in the form

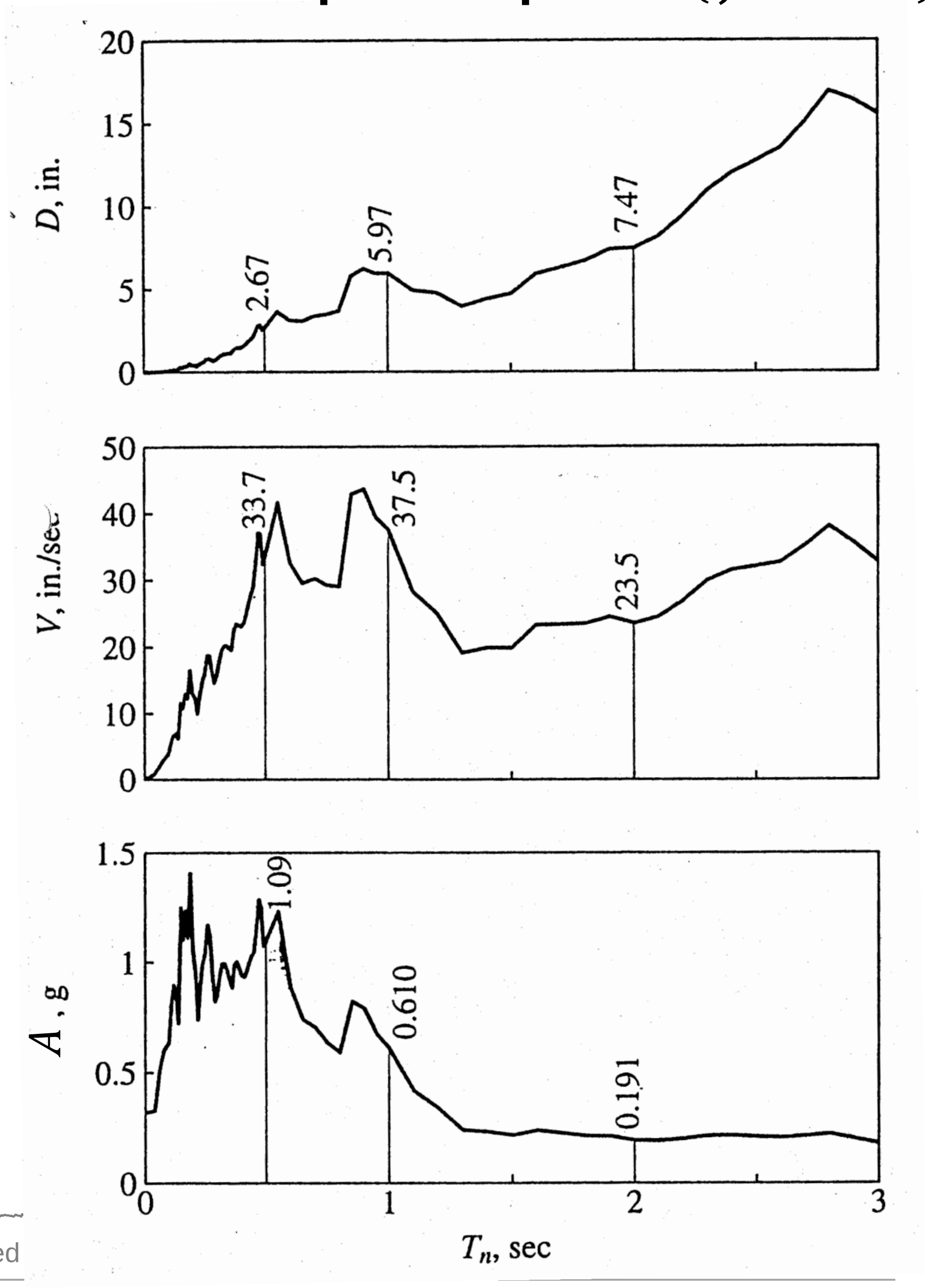
$$V_{b0} = \frac{A}{g}\,w \qquad (33)$$

Where w is the weight of the structure and g is the gravitational acceleration.

A/g may be interpreted as the base shear coefficient or lateral force coefficient*.

*It is used in the building codes to represent the coefficient by which the <u>structural weight is multiplied to obtain the base shear</u>.

Response Spectra $(\xi = 0.02)$ for El Centro Ground Motion



Deformation Response Spectrum

$$D = u_0$$

Pseudo-Velocity Response Spectrum

$$V = \omega_n D$$

Pseudo-Acceleration Response Spectrum

$$A = \omega_n^2 D$$

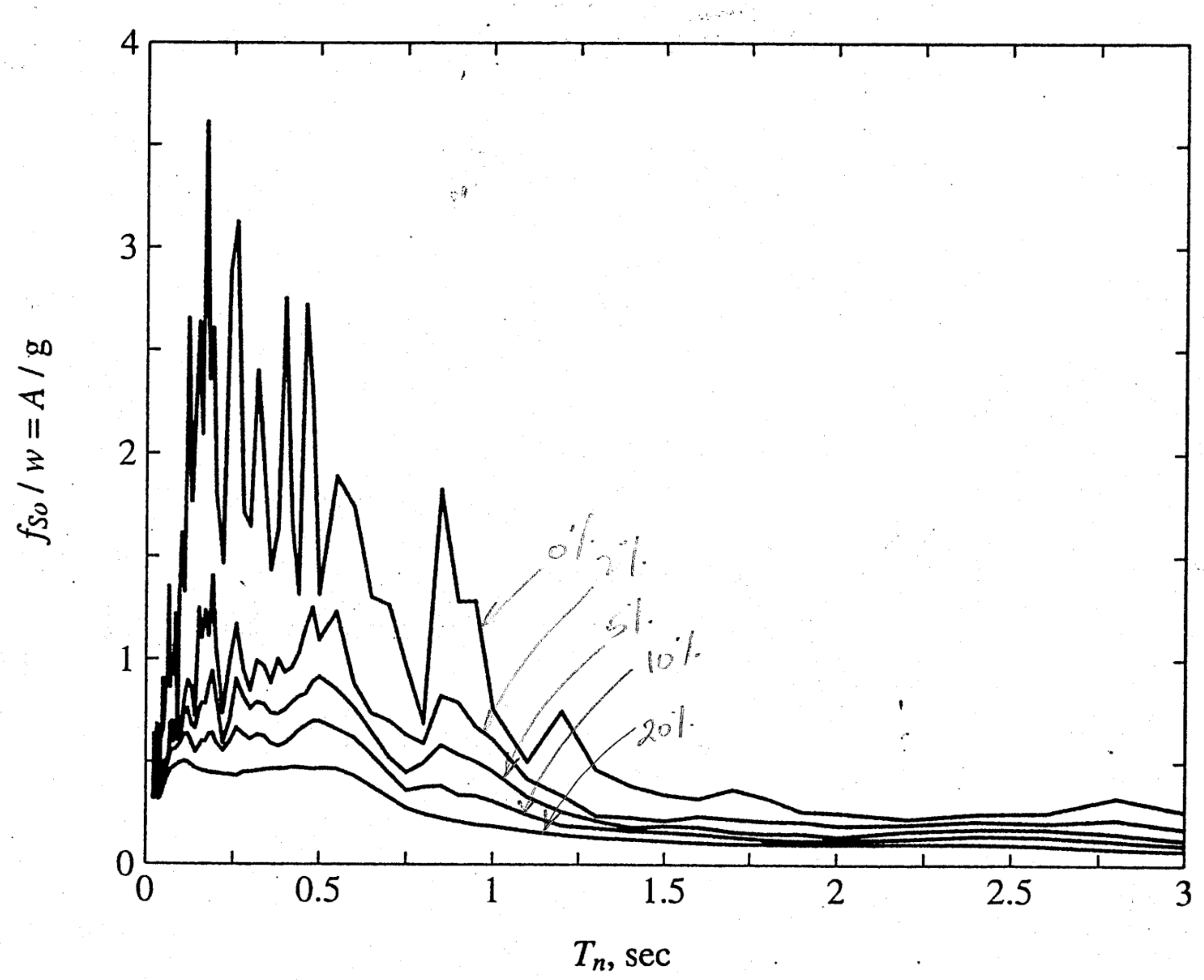
(This graph shows A/g)

The Pseudo velocity $\mathcal{V}$ is related to the peak values of the strain energy, E_{s0}, stored in the system:

$$E_{s0} = \frac{1}{2}ku^2{}_0 = \frac{1}{2}kD^2 = \frac{1}{2}m\omega^2{}_nD^2 = \frac{1}{2}m\mathcal{V}^2 \qquad (34)$$

The kinetic energy of the structural mass m with velocity $\mathcal{V}$.

Normalized pseudo-acceleration, or base shear coefficient, response spectrum for El Centro ground motion; $\xi = 0, 0.02, 0.05, 0.1, 0.2$



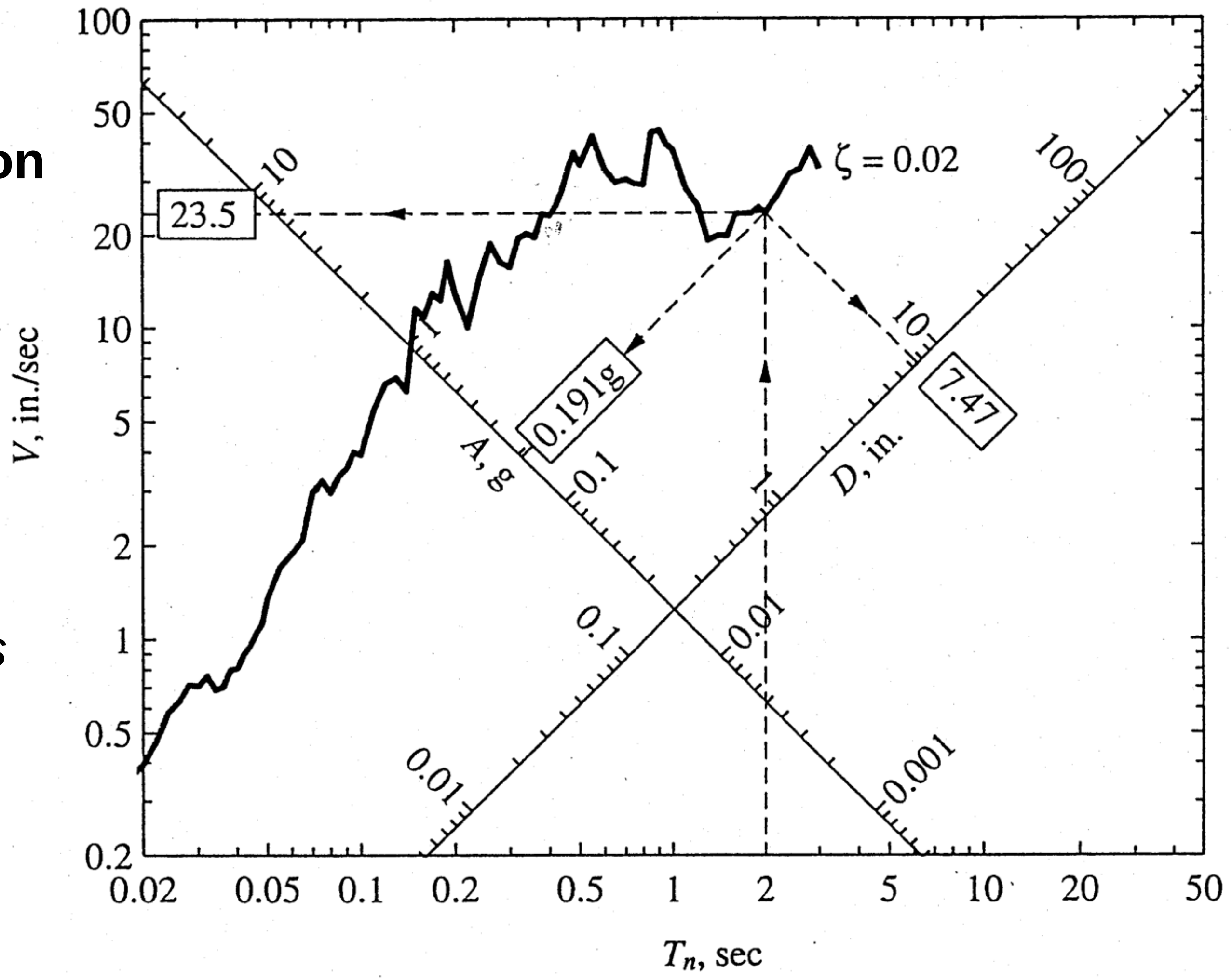
Combined D-V-A Spectrum

Each of the deformation, pseudo-velocity, and pseudo-acceleration response spectra for a given ground motion continue the same information-they are simply different ways of presenting the same information on structural response.

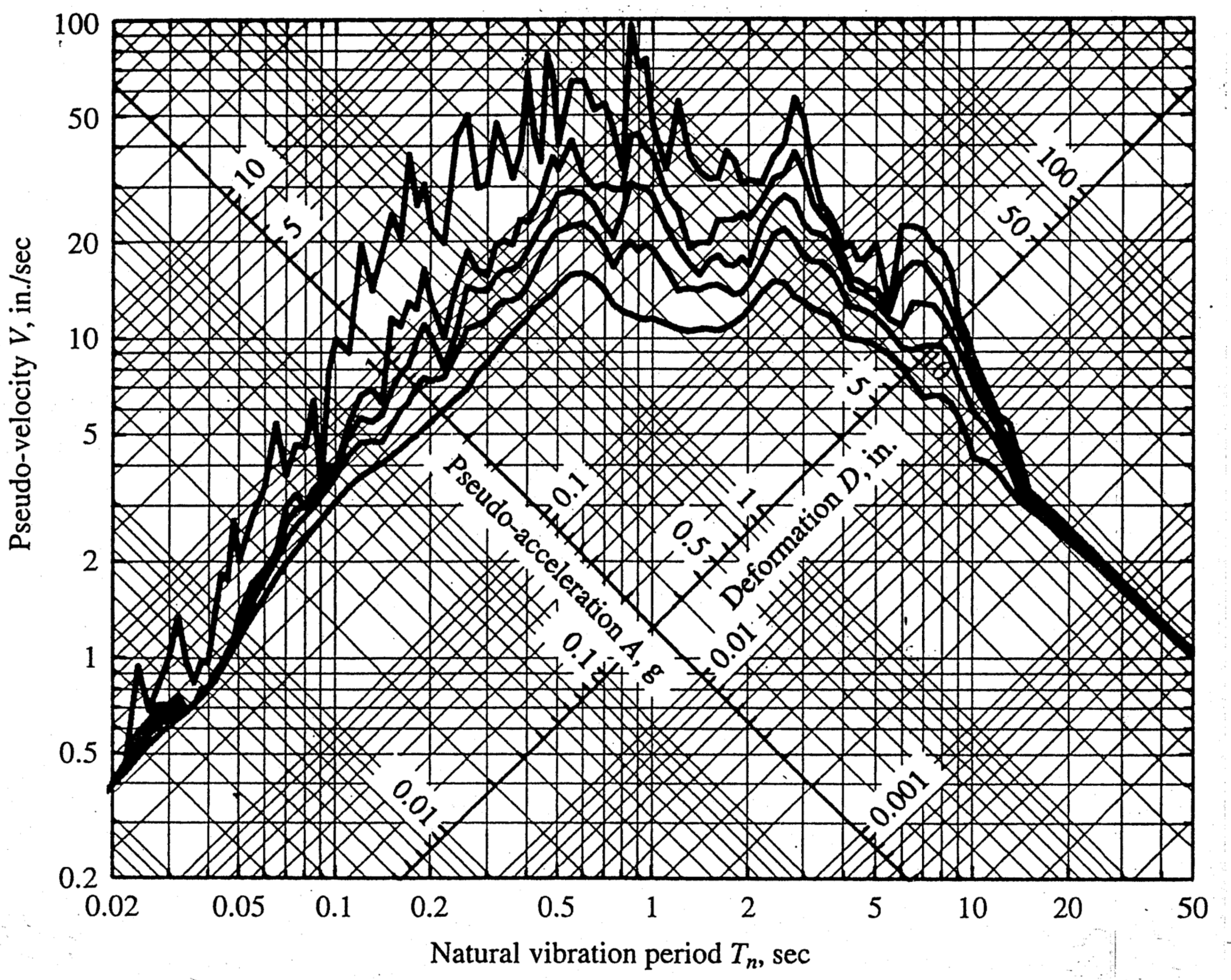
Combined D-V-A response spectrum $(\xi = 0.02)$ for El Centro ground motion

A single curve can simultaneously show three different quantities.

- *The peak deformation*
- *The peak pseudo-velocity which is related to the peak strain energy*
- *The peak pseudo-acceleration which is related to the peak value of equivalent static force (and base shear).*



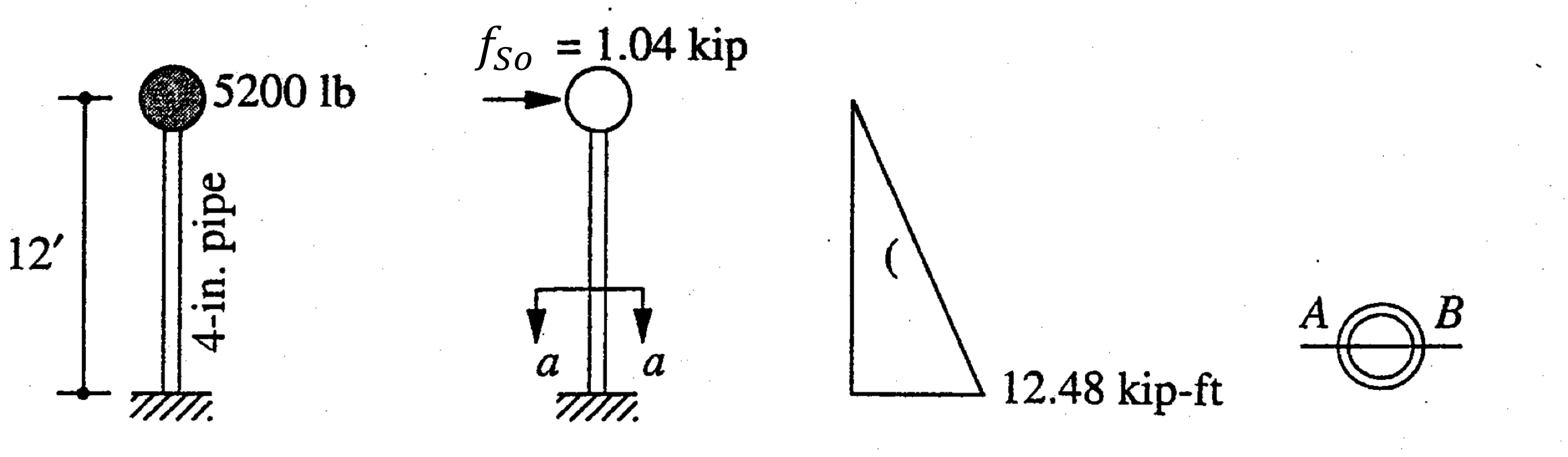
Combined D-V-A response spectrum for El Centro ground motion; $\xi = 0, 0.02, 0.05, 0.10\ and\ 0.20$



Example

A 12-ft-long vertical cantilever, a 4-in.-nominal-diameter standard steel pipe, supports a 5200-lb weight attached at the tip as shown in Fig. E6.2. The properties of the pipe are: outside diameter, d_o= 4.500 in., inside diameter d_i= 4.026 in., thickness t = 0.237 in., and second moment of cross-sectional area, I = 7.23 in^4, elastic modulus E = 29,000 ksi, and weight = 10.79 lb/foot length. Determine the peak deformation and bending stress in the cantilever due to the El Centro ground motion. Assume that ξ = 2%.

Solution



(a) (c) (d) Moment (e) Section a-a

Note: The unit of **force** is $\boldsymbol{kip}$: 1 kip = 1000 lb

The unit of **mass** is therefore the unit of **force** divided by the unit of **acceleration**.

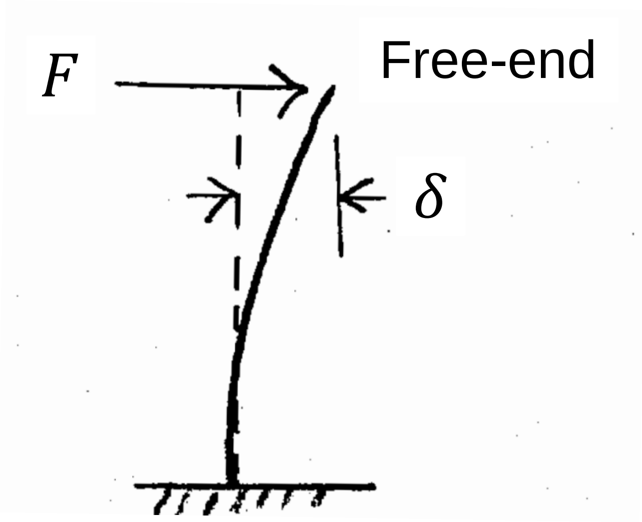
The unit of **acceleration** is $\boldsymbol{in/sec^2}$

$$1g = 9.81\, m/\sec^2 = 32.2\, ft/\sec^2 = 386\, in/\sec^2$$

The unit of $\boldsymbol{E}$ is the unit of **force** divided by the unit of **area**.

The lateral stiffness $\boldsymbol{K}$ in this case is determined from

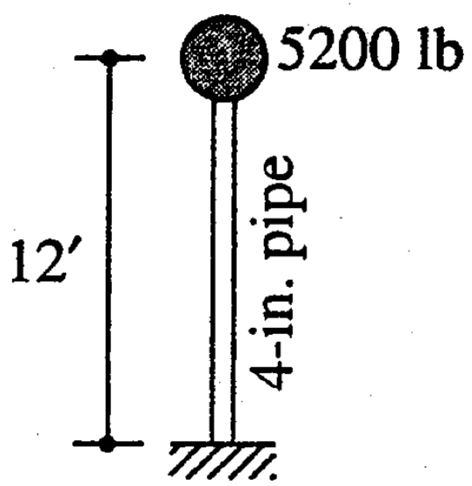
$$K = \frac{F}{\delta}$$



Fixed-End

The lateral stiffness of this SDF system is

$$k = \frac{3EI}{L^3} = \frac{3(29 \times 10^3)7.23}{(12 \times 12)^3} = 0.211\, kip/in.$$

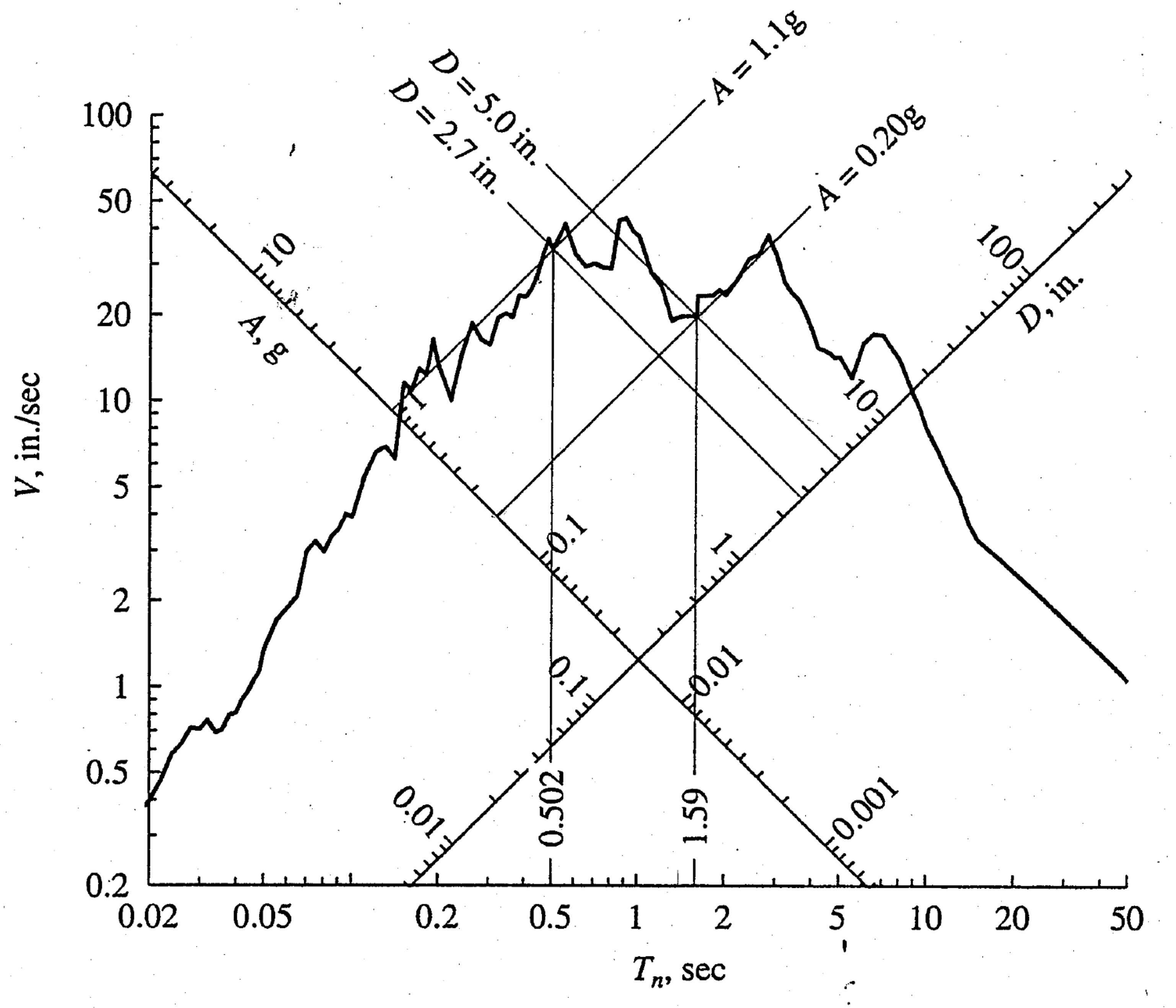


The total weight of the pipe is $10.79 \times 12 = 129.5\ lb$, which may be neglected relative to the lumped weight of $5200\ lb$. Thus

$$m = \frac{w}{g} = \frac{5.20}{386} = 0.01347\ kip - sec^2/in.$$

The natural vibration frequency and period of the system are

$$\omega_n = \sqrt{\frac{k}{m}} = \sqrt{\frac{0.211}{0.01374}} = 3.958\ rad/sec \quad \Rightarrow \quad T_n = 1.59\, sec$$



From the response spectrum curve for ξ = 2%, for T_n = 1.59 sec,

$$D = 5.0\ in. = u_o$$

$$A = 0.20g$$

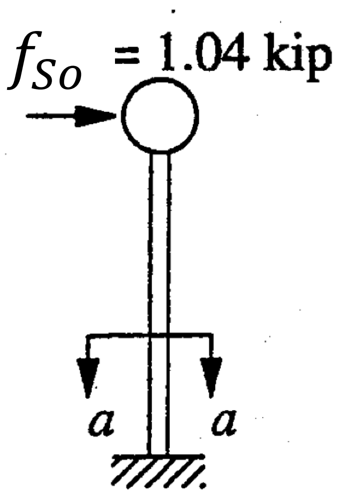
The equivalent static force is

$$f_{So} = \frac{A}{g}w = 0.20 \times 5.2 = 1.04\ kips$$

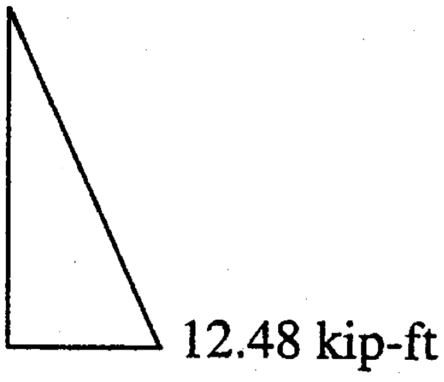
The **bending moment diagram** is shown in Fig. (d) with the maximum moment at the base = 12.48 kip-ft. Points *A* and *B* shown in Fig. (e) are the locations of maximum bending stress

$$\sigma_{max} = \frac{Mc}{I} = \frac{(12.48 \times 12)(4.5/2)}{7.23} = 46.5\ ksi$$

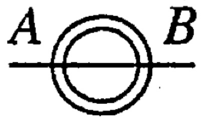
As shown, σ = +46.5 ksi at A and σ = −46.5 ksi at B, where + denotes tension. **The algebraic signs of these stresses are irrelevant** because the direction of the peak force is not known, as the pseudo-acceleration spectrum is, by definition, positive.



(c)



(d) Moment



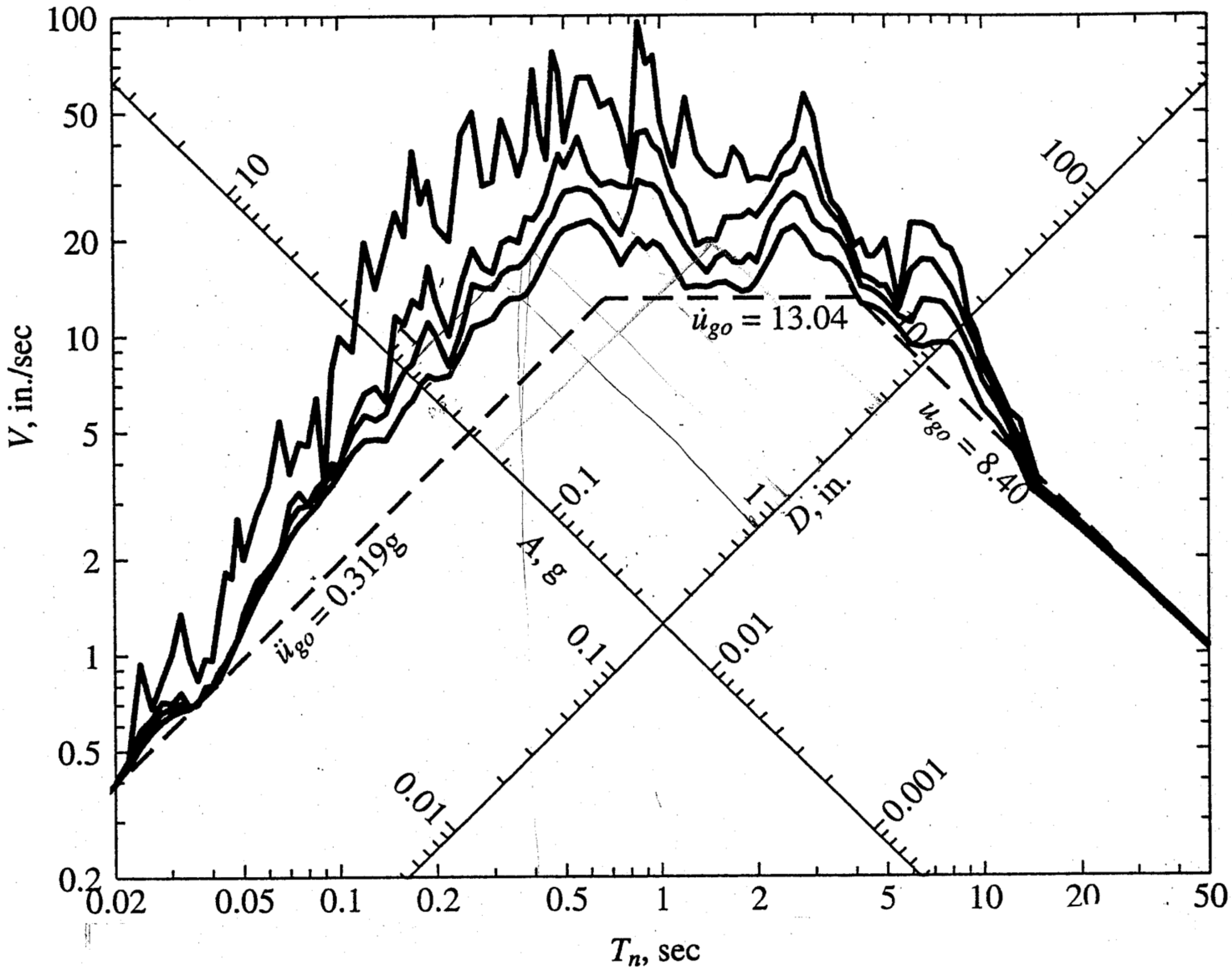
(e) Section *a-a*

Response Spectrum Characteristics

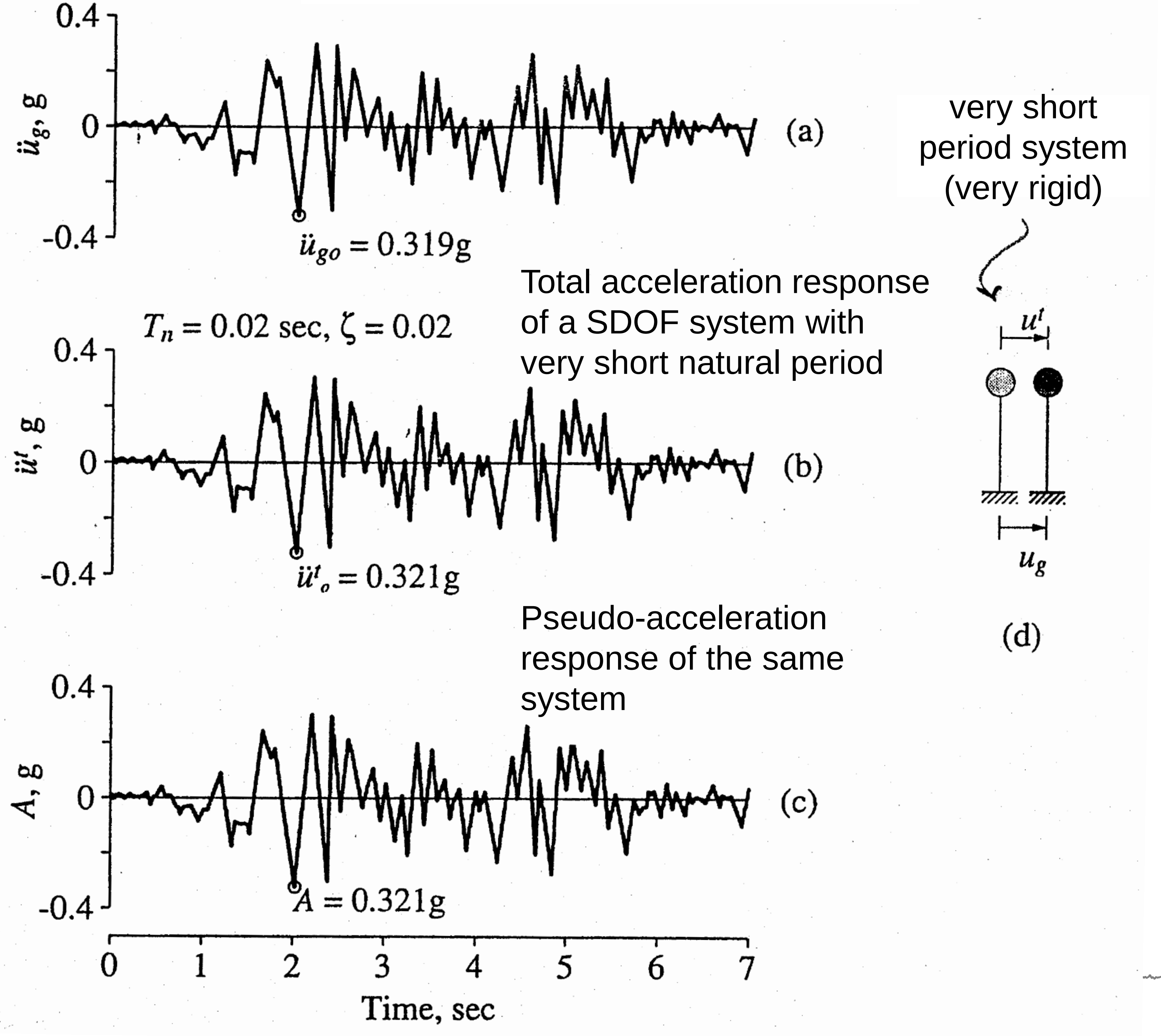
Combined D-V-A response spectrum ($\xi = 0$, 0.02, 0.05, 0.1) and peak values of ground acceleration, ground velocity, and ground displacement for El Centro ground motion

For systems with very short period, the pseudo-acceleration A for all damping values approach $\ddot{u}_{g0}$ and D is very small.

For systems with very long period, D for all the damping values approach u_{g0} and A is very small; thus the forces in the structures, which are related to the mA, would be very small.



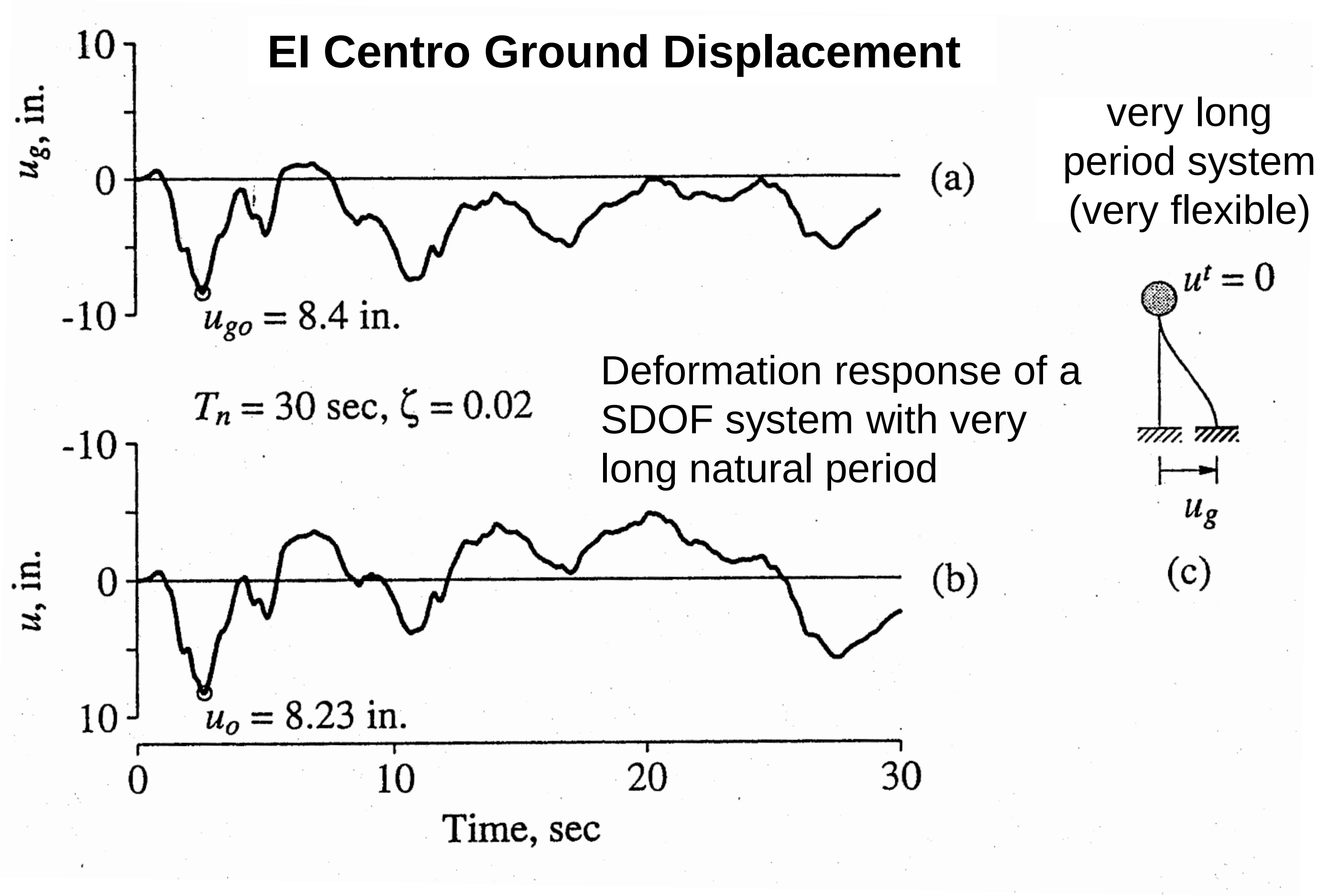
El Centro Ground Acceleration



A very short period system is extremely stiff and rigid. Its deformation response to the ground motion is very small. So its mass move rigidly with the ground and its peak structural acceleration should be approximately $\ddot{u}_{g0}$.

To drive the structural mass to move with acceleration of $\ddot{u}_{g0}$, it is necessary to have $f_{s0} \approx m\,\ddot{u}_{g0}$,

therefore, $A \approx \ddot{u}_{g0}$



A very long period system is extremely flexible. The mass would be expected to remain essentially stationary while the ground below moves.

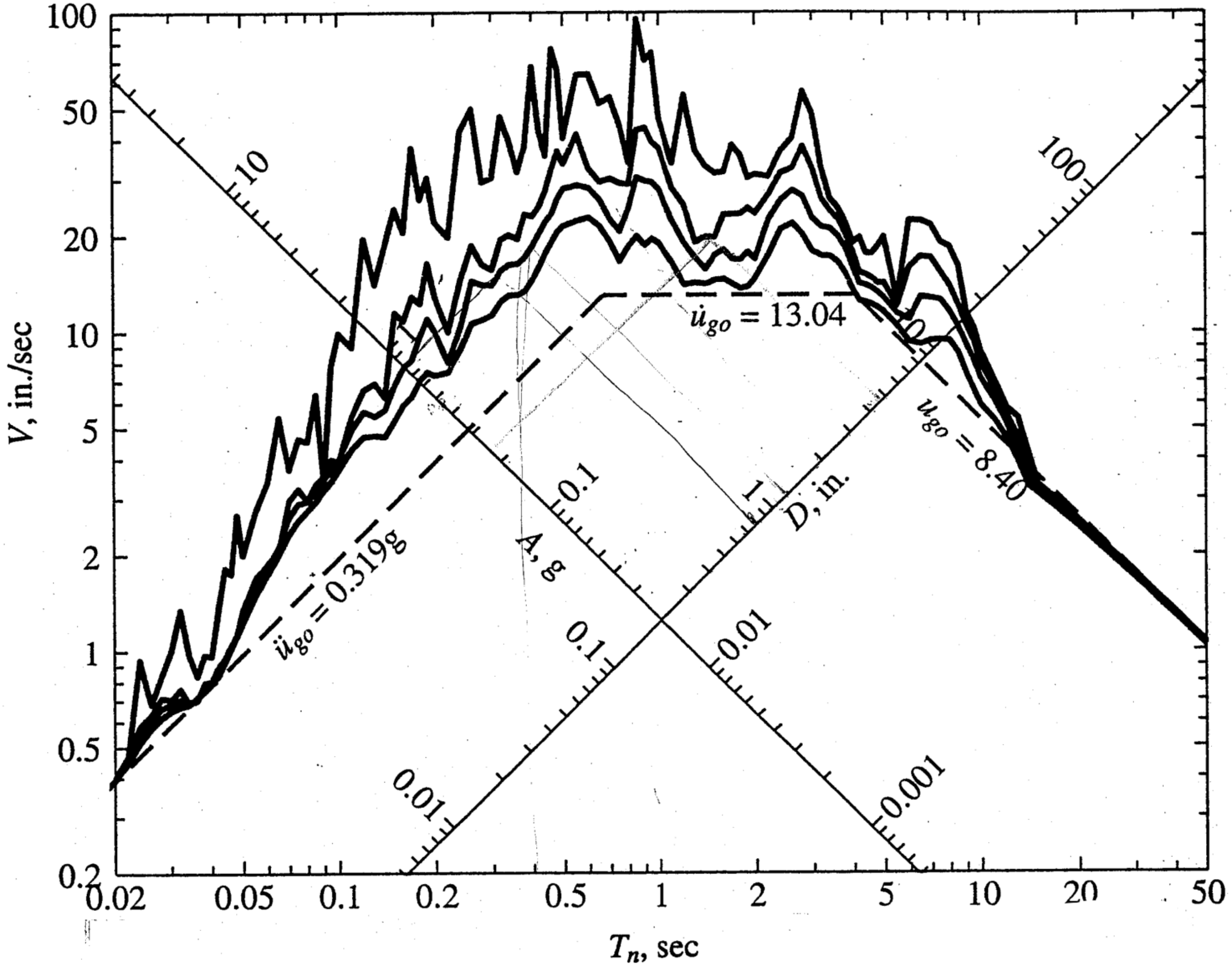
Thus $u(t) \cong -u_g(t)$ that is $D \cong u_{g0}$

Response Spectrum Characteristics

Combined D-V-A response spectrum ($\xi = 0$, 0.02, 0.05, 0.1) and peak values of ground acceleration, ground velocity, and ground displacement for El Centro ground motion

The reduction of response due to additional damping is different for the different spectral regions- greatest in the velocity-sensitive region.

The effectiveness of damping in reducing the structural response also depends on the ground motion characteristics.



- For Mexico city 85 earthquake where ground motion is nearly harmonic over many cycles, the effect of damping would be large for a system near “resonance”.

- For Park Filed 66 earthquake where ground motion is very short and shock like, the effect of damping would be small, as in the case of half cycle sine pulse excitation.

Response Spectra

- The construction of response spectra plots requires the **solution of single degree of freedom systems for a sequence of natural frequency and of the damping ratio** in the range of interest.

- Every solution provides **only one point** (the maximum value) of the response spectrum.

- Since a large number of systems must be analyzed in order to fully plot each response spectrum, the task is lengthy and time consuming even with the use of computer.

- Once these curves are constructed and are available for the excitation of interests, the **analysis for the design of structures subjected to dynamic loading is reduced to a simple calculation of natural frequency of the system and the use of response spectra.**

Response Spectra

- Dynamic analysis of a system with n **degree of freedom** can be transformed to the problem of solving **n systems** in which each one is a **single degree of freedom system**

- The understanding and mastery of the concepts and methods of solutions for a single degree of freedom system is quit important.

Pseudo-velocity response spectrum

- Consider a quantity V for an SDF system with natural frequency ω related to its peak deformation $D \equiv u_o$ due to earthquake ground motion:

$$V = \omega D = \frac{2\pi}{T} D$$

- The quantity V has units of velocity. This is pseudo-velocity.

- Using the above expression, the displacement response spectrum can be converted to the Pseudo-velocity response spectrum.

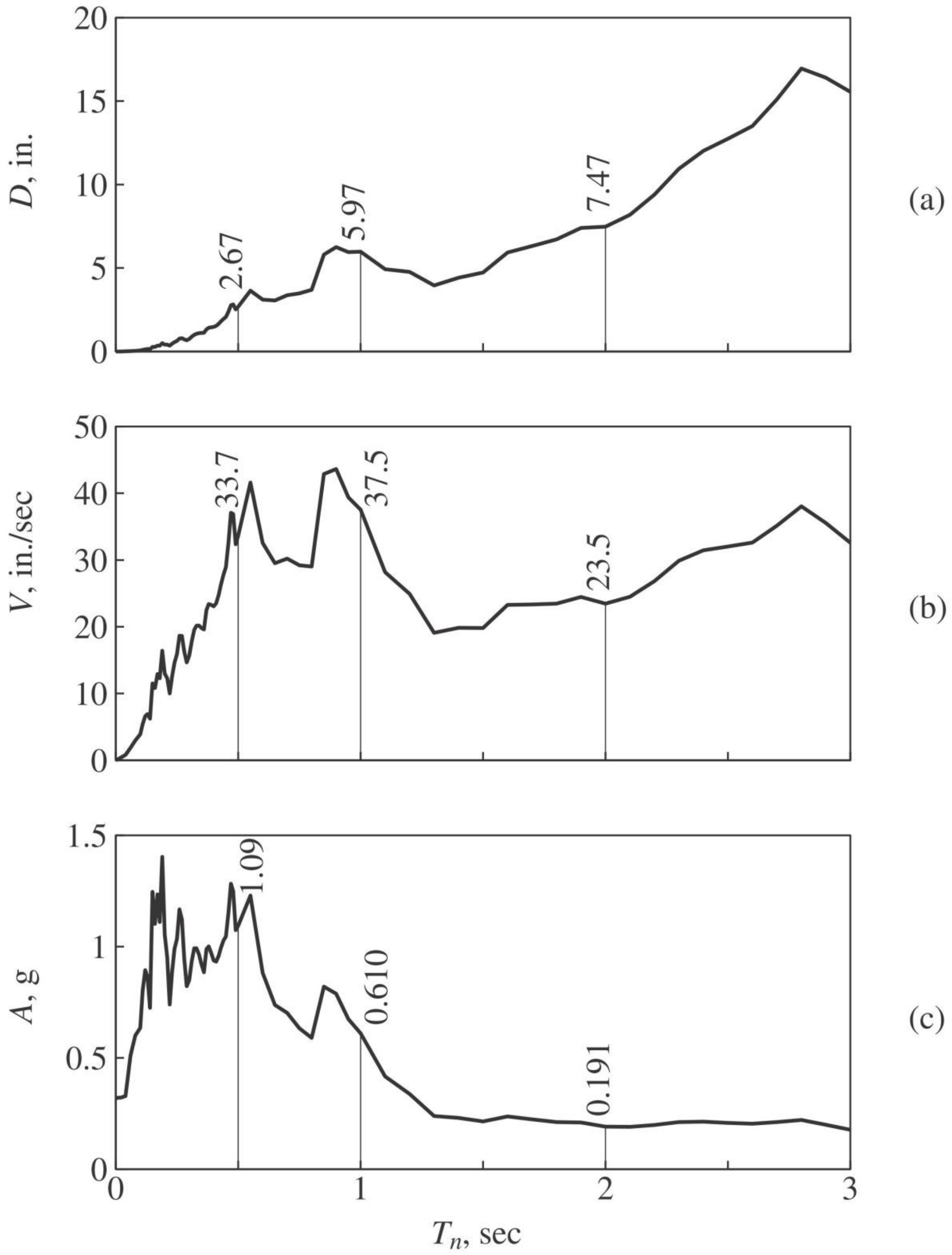
Pseudo-acceleration response spectrum

- Consider a quantity A for an SDF system with natural frequency ω related to its peak deformation $D \equiv u_o$ due to earthquake ground motion:

$$A = \omega^2 D = \left(\frac{2\pi}{T}\right)^2 D$$

- The quantity A has units of acceleration. This is pseudo-acceleration.

- Using the above expression, the displacement response spectrum can be converted to the Pseudo-acceleration response spectrum.



Response spectra ($\xi = 0.02$) for El Centro ground motion: (a) deformation response spectrum; (b) pseudo-velocity response spectrum; (c) pseudo-acceleration response spectrum.

Pseudo-acceleration response spectrum

- The quantity A is related to the peak value of base shear V_{bo} or the peak value of the equivalent static force f_{so}.

$$V_{bo} = f_{so} = mA$$

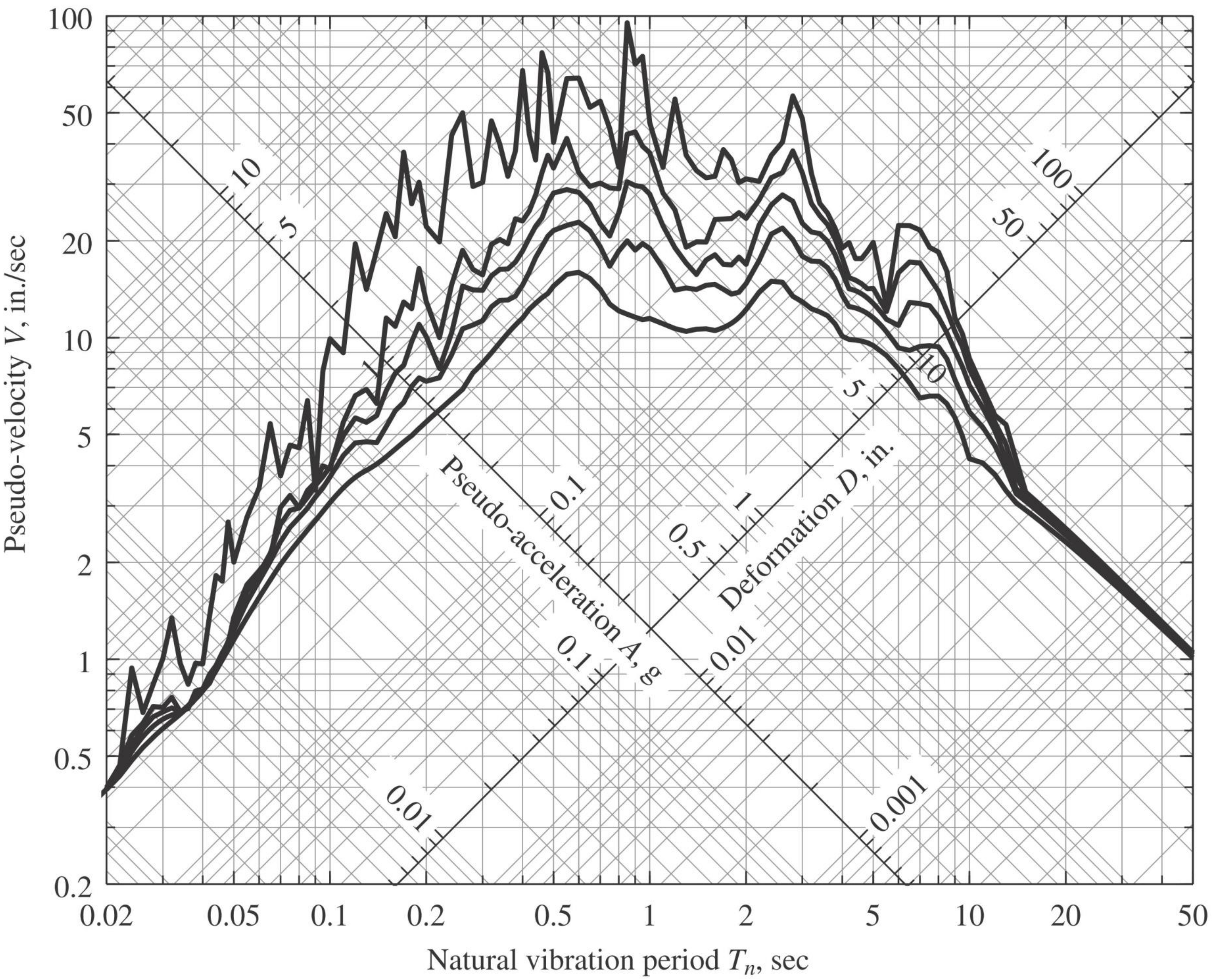
- The peak base shear can be written in the form

$$V_{bo} = \frac{A}{g}w \quad , \quad f_{so} = \frac{A}{g}w$$

Combined D-V-A response spectrum

- The three spectra (deformation, pseudo-velocity, and pseudo-acceleration) are simply different ways of presenting the same information on structural response for a given ground motion.

- Knowing one of the spectra, the other two can be obtained by algebraic operations mentioned earlier.



Combined D–V –A response spectrum for El Centro ground motion

ξ = 0, 2, 5, 10, and 20%.

Construction of Response Spectrum

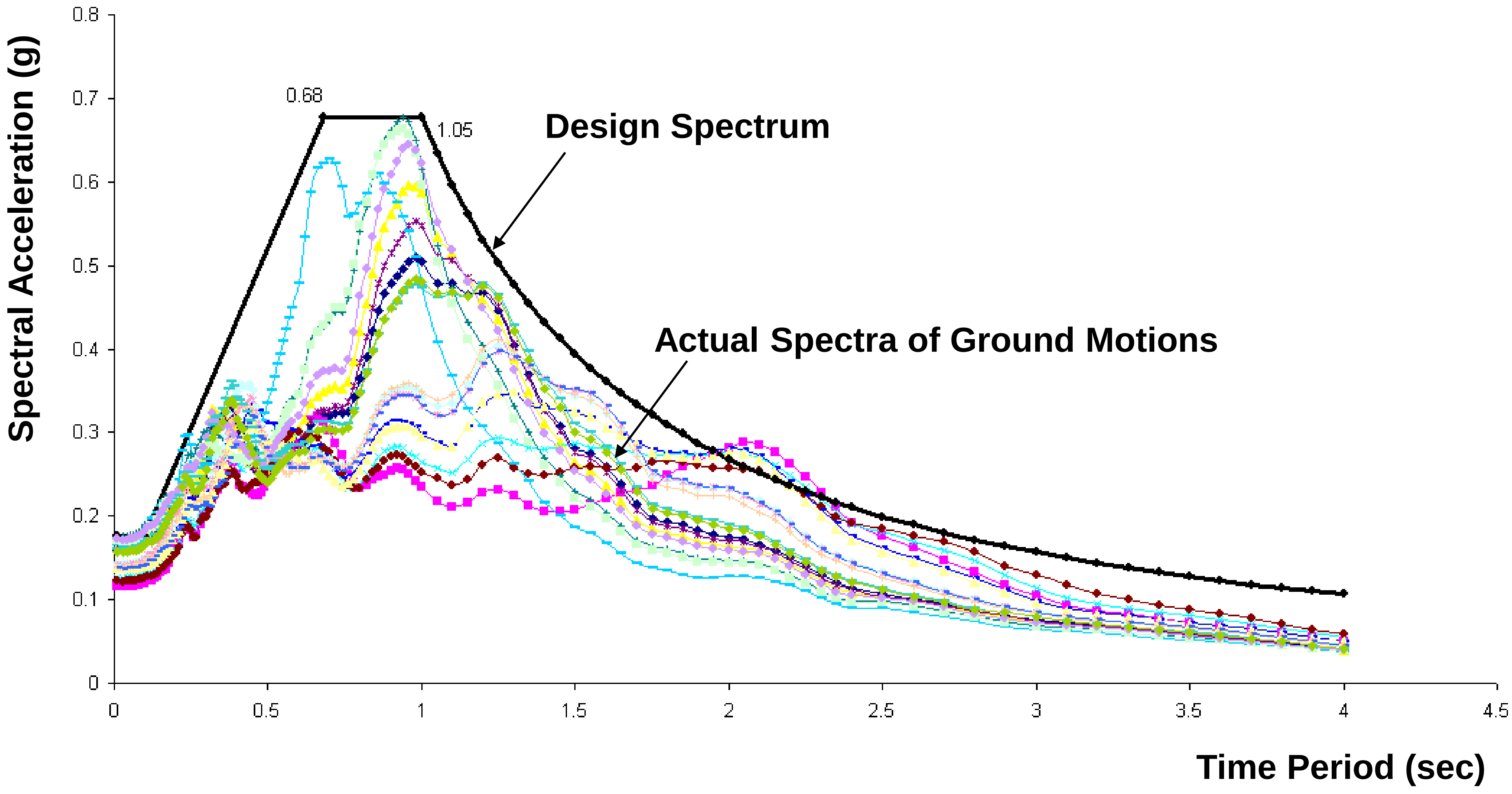
The response spectrum for a given ground motion component $\ddot{u}_g(t)$ can be developed by implementation of the following steps:

1) Numerically define the ground acceleration $\ddot{u}_g(t)$; typically, the ground motion ordinates are defined every 0.02 sec.
2) Select the natural vibration period T and damping ratio ξ of an SDF system.
3) Compute the deformation response $u(t)$ of this SDF system due to the ground motion $\ddot{u}_g(t)$ by any of the numerical methods.
4) Determine u_o, the peak value of $u(t)$.

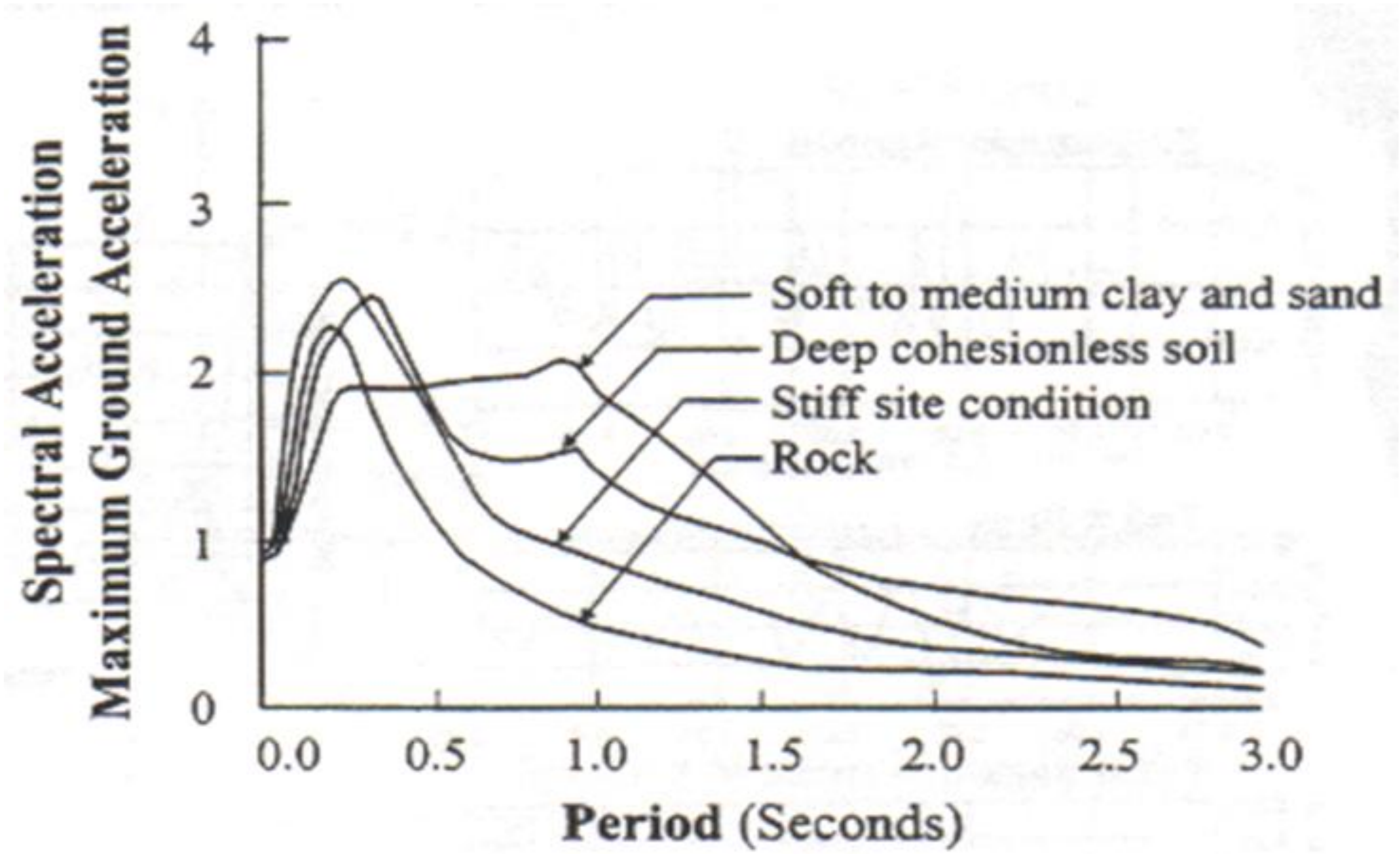
Construction of Response Spectrum

5) The spectral ordinates are $D = u_o$, $V = (2\pi/T)D$, and $A = (2\pi/T)^2 D$.
6) Repeat steps 2 to 5 for a range of T and ξ values covering all possible systems of engineering interest.
7) Present the results of steps 2 to 6 graphically to produce three separate spectra or a combined spectrum.

Sample Response Spectra



Spectra For Different Soils



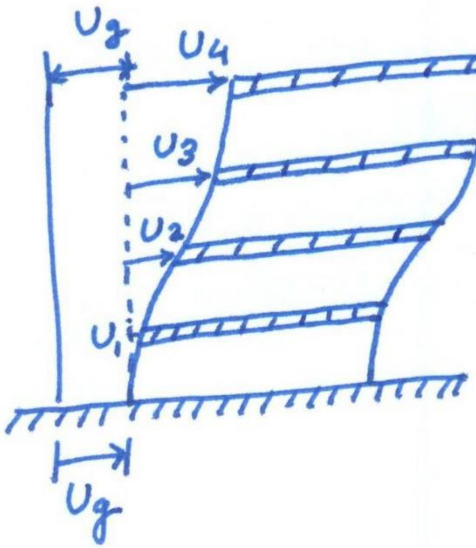
The Elastic Response Spectrum Analysis (RSA) Procedure

The Elastic Response Spectrum Analysis (RSA) Procedure

Arturo Danusso (1880 - 1968) Theodore von Karman (1881 - 1963) Maurice Anthony Biot (1905 - 1985)

Earthquake Response of MDF Structures

A shear building subjected to ground displacement $u_g(t)$:



u_g : Ground displacement

u : structural (relative) displacement

$$\mathbf{u}^t(t) = \begin{Bmatrix} u_g+u_4\\ u_g+u_3\\ u_g+u_2\\ u_g+u_1\end{Bmatrix} = \begin{Bmatrix}u_g\\u_g\\u_g\\u_g\end{Bmatrix}+\begin{Bmatrix}u_4\\u_3\\u_2\\u_1\end{Bmatrix}$$

$$= \begin{Bmatrix}1\\1\\1\\1\end{Bmatrix} u_g + \mathbf{u}(t)$$

$$\mathbf{u}^t(t) = \mathbb{1}\, u_g(t) + \mathbf{u}(t)$$

$$\ddot{\mathbf{u}}^t(t) = \mathbb{1}\, \ddot{u}_g(t) + \ddot{\mathbf{u}}(t)$$

Earthquake Response of MDF Structures

Mathematical Formulation:

$$\mathbb{M}\ddot{\mathbb{u}}^t(t) + \mathbb{C}\dot{\mathbb{u}}(t) + \mathbb{K}\mathbb{u}(t) = \mathbb{0}$$

(Eq. of motion for MDF Systems Subjected to ground motion)

$$\ddot{\mathbb{u}}^t(t) = \mathbb{1}\ddot{u}_g(t) + \ddot{\mathbb{u}}(t)$$

$$\mathbb{M}\ddot{\mathbb{u}}(t) + \mathbb{C}\dot{\mathbb{u}}(t) + \mathbb{K}\mathbb{u}(t) = -\mathbb{M}\mathbb{1}\ddot{u}_g(t)$$

Transforming the above eq. into principal (modal) coordinates,

$$\mathbb{u}(t) = \sum_{i=1}^{4} \phi_i q_i(t) = \Phi \mathbb{q}$$

where

$$\Phi = [\phi_1 \ \ \phi_2 \ \ \phi_3 \ \ \phi_4], \quad \mathbb{q} = \begin{Bmatrix} q_1(t) \\ q_2(t) \\ q_3(t) \\ q_4(t) \end{Bmatrix}$$

ϕ_i = ith-mode vector

Using Modal Orthogonality:

$$\mathbb{M}\Phi\ddot{q}(t) + \mathbb{C}\Phi\dot{q}(t) + \mathbb{K}\Phi q(t) = -\mathbb{M}\mathbb{1}\ddot{u}_g(t)$$

multiplying with Φ^T, (and using modal orthogonality)

$$\Phi^T\mathbb{M}\Phi\ddot{q}(t) + \Phi^T\mathbb{C}\Phi\dot{q}(t) + \Phi^T\mathbb{K}\Phi q(t) = -\Phi^T\mathbb{M}\mathbb{1}\ddot{u}_g(t)$$

$$\begin{bmatrix} \mu_1 & 0 & 0 & 0 \\ 0 & \mu_2 & 0 & 0 \\ 0 & 0 & \mu_3 & 0 \\ 0 & 0 & 0 & \mu_4 \end{bmatrix}\ddot{q}(t) + \begin{bmatrix} 2\xi_1\mu_1\omega_1 & & 0 \\ & \ddots & \\ 0 & & 2\xi_4\mu_4\omega_4 \end{bmatrix}\dot{q}(t) + \begin{bmatrix} \mu_1\omega_1^2 & & 0 \\ & \ddots & \\ 0 & & \mu_4\omega_4^2 \end{bmatrix}q(t)$$

$$= -\mathbb{L}\ddot{u}_g(t)$$

where $\mathbb{L} = \begin{bmatrix} L_1 \\ L_2 \\ L_3 \\ L_4 \end{bmatrix}$, $L_i = \phi_i^T . \mathbb{M} . \mathbb{1}$

$$= \begin{bmatrix} \phi_{i4} & \phi_{i3} & \phi_{i2} & \phi_{i1} \end{bmatrix} \begin{bmatrix} M_4 & & & 0 \\ & M_3 & & \\ & & M_2 & \\ 0 & & & M_1 \end{bmatrix} \times \begin{bmatrix} 1 \\ 1 \\ 1 \\ 1 \end{bmatrix}$$

$$L_i = \sum_{j=1}^{4} M_j \phi_{ij}$$

Uncoupled Equations of Motion:

The "ith-mode" equation is

$$\mu_i \ddot{q}_i(t) + 2\xi_i \mu_i \omega_i \dot{q}_i(t) + \mu_i \omega_i^2 q_i(t) = -L_i \ddot{u}_g(t)$$

$$\mu_i = \Phi_i^T \mathbb{M} \Phi_i = \begin{bmatrix} \phi_{i4} & \phi_{i3} & \phi_{i2} & \phi_{i1} \end{bmatrix} \begin{bmatrix} M_4 & & & 0 \\ & M_3 & & \\ & & M_2 & \\ 0 & & & M_1 \end{bmatrix} \begin{bmatrix} \phi_{i4} \\ \phi_{i3} \\ \phi_{i2} \\ \phi_{i1} \end{bmatrix}$$

$$\mu_i = \sum_{j=1}^{4} M_j \phi_{ij}^2$$

Modal Participation Factor:

$$\Gamma_i = \frac{L_i}{\mu_i} = \frac{\Phi_i^T \cdot \mathbb{M} \cdot \mathbb{1}}{\Phi_i^T \cdot \mathbb{M} \cdot \Phi_i}$$

(How a particular ith mode responds to ground vibration)

Uncoupled Equations of Motion

Elastic Modal Response History Analysis (RHA) Procedure

Solve the uncoupled equation of motion for each significant mode (represented by an SDF system) and sum the dynamic responses to get the dynamic response of MDF system

Response Spectrum Analysis (RSA) Procedure

Directly pick the peak response of each significant mode (represented by an SDF system) and approximately combine those peak responses to get the peak response of MDF system

The "Elastic" Response

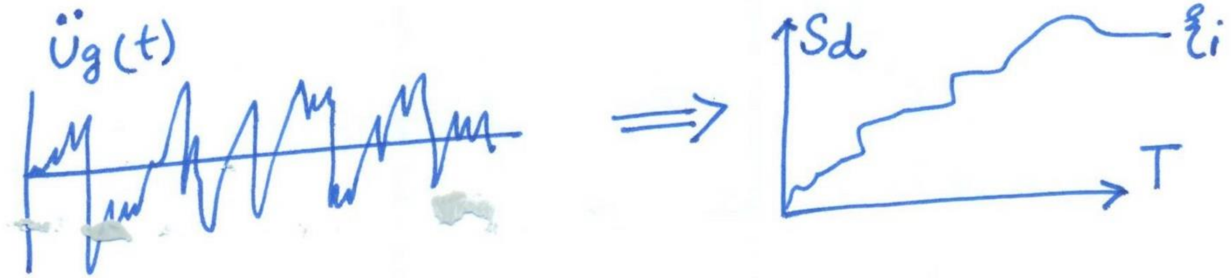
Spectrum Analysis Procedure

So the ith-mode equation becomes,

$$\ddot{q}_i(t) + 2\xi_i \omega_i \dot{q}_i(t) + \omega_i^2 q_i(t) = -\Gamma_i \ddot{u}_g(t)$$

The maximum deformation in this ith mode is

$$q_{i,max} = \Gamma_i S_{di}$$



$$U_{i,max} = \Gamma_i \Phi_i S_{di}$$

$S_{di} \rightarrow$ determined from elastic response spectrum at T_i and ξ_i.

$$IDR_{i,max} = \Gamma_i \left(\phi_{ji} - \phi_{j-1,i}\right) S_{di}$$

(at any jth floor)

The “Elastic” Response

Spectrum Analysis Procedure

The equivalent static forces corresponding to ith-mode: (i.e. the forces which will deform structure in $u_{i,max}$ Shape and magnitude):

$$f_{si} = \mathbb{K}\, u_{i,max}$$

From eigen-value equation of ith-mode:

$$\mathbb{K}\Phi_i = \omega_i^2 \mathbb{M}\Phi_i$$

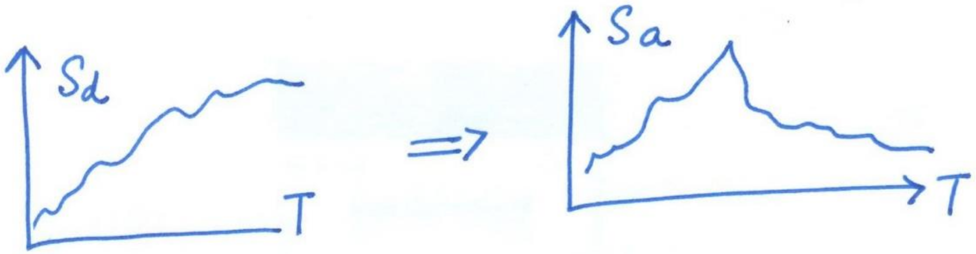
$$= \Gamma_i \mathbb{K} \Phi_i S_{di}$$

$$= \Gamma_i \omega_i^2 \mathbb{M} \Phi_i S_{di}$$

$$= \Gamma_i \cdot \omega_i^2 \cdot S_{di} \begin{bmatrix} M_4 & & 0 \\ & \ddots & \\ 0 & & M_1 \end{bmatrix} \begin{bmatrix} \phi_{i4} \\ \vdots \\ \phi_{i1} \end{bmatrix}$$

The “Equivalent Static Forces” for each Mode

$$S_a = \omega^2 S_d$$



Therefore,

$$\boxed{f_{si} = \Gamma_i \mathbb{M} \Phi_i S_{ai}}$$

$S_{ai} \rightarrow$ determined from elastic response spectrum at T_i and ξ_i.

The "Elastic" Response Spectrum Analysis Procedure

Modal Combination:

Modal responses attain peaks at different time intervals → So how to combine them? approximate rules

a) SRSS : $r_{max} \approx \sqrt{\sum_{i=1}^{N} r_{i,max}^2}$ (most widely used)

$r_{i,max}$ → Peak modal response

N → No. of modes considered in analysis

b) Absolute Sum : $r_{max} \simeq \sum_{i=1}^{N} r_{i,max}$

(upper-bound, too conservative not popular)

SRSS combination developed → Rosenblueth's PhD Thesis (1951)

→ good for structures with well-separated natural frequencies. (closely spaced → not good)

c) Complete Quadratic Combination (CQC) :

(Chopra, Chapter 13)

The “Elastic” Response Spectrum Analysis Procedure

Summary:

1) Determine $\mathbb{M}$, $\mathbb{K}$ matrices and ξ_i
2) T_n, Φ_n (eigen-value analysis)
3) Compute peak modal responses of each significant mode. For each ith-mode,
   (i) For T_i and $\xi_i \rightarrow$ Pick S_{ai} and S_{di} from spectrum
   (ii) Compute story disp and story IDRS
   $$u_{i,max} = \Gamma_i \Phi_i S_{di}$$
   $$IDR_{i,max} = \Gamma_i \left(\phi_{j,i} - \phi_{j-1,i}\right) S_{di}$$
   at jth floor
   (iii) Compute equivalent static forces
   $$f_{si} = \Gamma_i \mathbb{M} \Phi_i S_{ai}$$
   (iv) Compute all responses by static analysis of structure under f_{si}
4) Estimate Peak respose by combining peaks of individual modal Responses. Usually lower modes contribute significantly to overall response so may be only first few modes can be included in analysis.

The "Elastic" Response

Spectrum Analysis Procedure

Notes:

a) RSA — reduces the dynamic problem into a static problem, infact a series of static analysis under f_{si} (equivalent static forces for few modes).

RSA still retains the features of dynamic analysis — T_n, ξ_i, ω_i, ϕ_i

The "dynamic part of the problem" is already done while developing spectra (S_{di} vs. T and S_{ai} vs. T).

The "Elastic" Response Spectrum Analysis Procedure

Notes:

b) RSA vs. RHA

(i) $U_{i,max} = \Gamma_i \Phi_i S_{di}$ = Same as peak disp from RHA because the spectra is already based on RHA.

but

U_{max} from modal combination rule $\neq$ Peak disp extracted from RHA results (no trend)

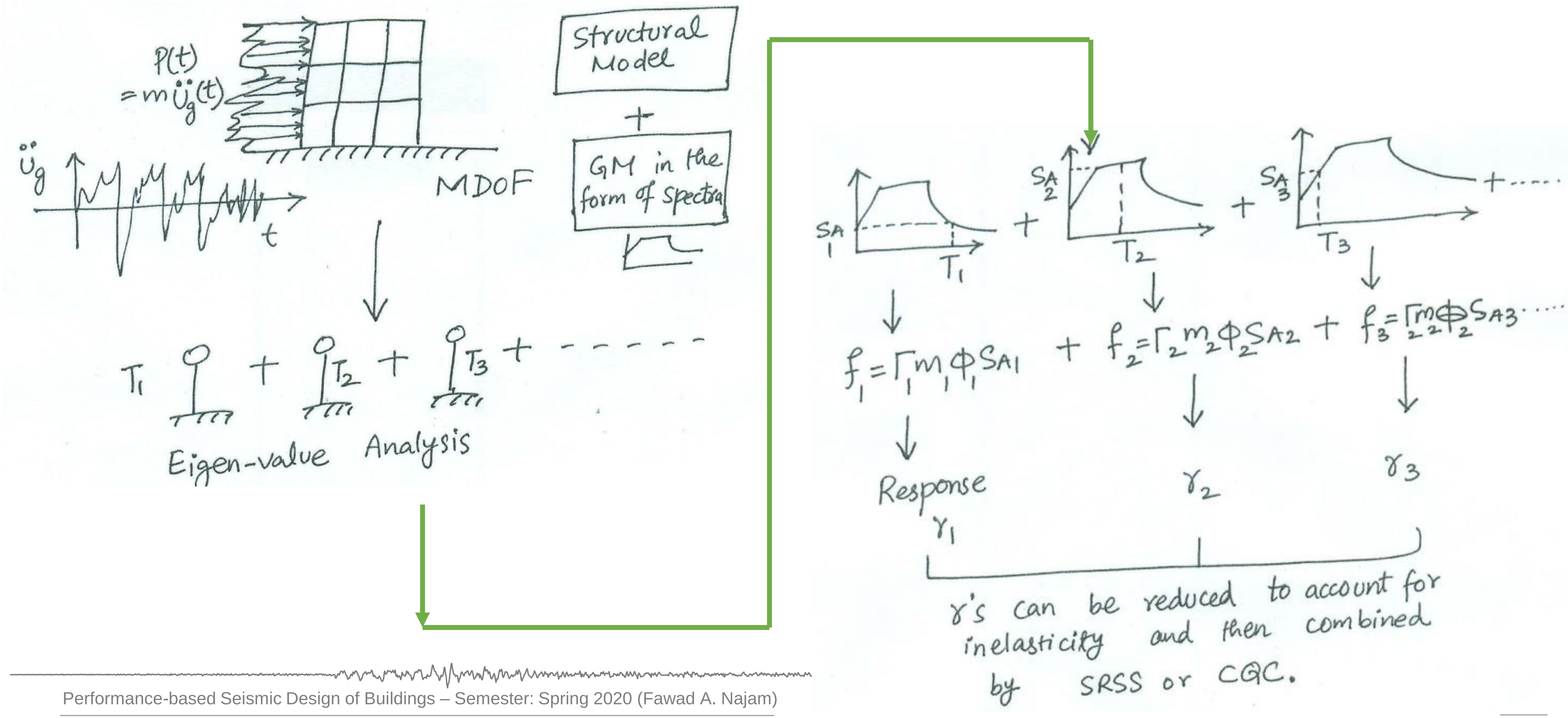
c) The direction of f_{si} is controlled by algebraic sign of Φ_i.  , , , -----

d) Avoid a pitfall: In RSA, calculating one response from combined peak of other response is WRONG. e.g

IDR_{max} at any floor $\neq U_{max\,upper} - U_{max\,lower}$ (U_{max} is combined from all modes)

or Story Shear at any floor $\neq \sum_{i=1}^{N} f_{si}$ $\Rightarrow$ combined equivalent forces for all modes)

The “Elastic” Response Spectrum Analysis Procedure



How to Use Response Spectra?

1) For each mode of free vibration, corresponding **Time Period** is obtained.
2) For each Time Period and specified damping ratio, the specified Response Spectrum is read to obtain the corresponding **Acceleration**.
3) For each Spectral Acceleration, corresponding **velocity and displacements** response for the particular degree of freedom is obtained.
4) The displacement response is then used to obtain the corresponding **stress resultants**.
5) The stress resultants for each mode are then **added using some combination rule** to obtain the final response envelop.

ELF vs. Response Spectrum Analysis

- Static methods specified in building codes are based on single mode response and appropriate for simple and regular structures
- Dynamic analysis should be used for complex buildings to determine significant response characteristics
  - Effects of structure's dynamic characteristics on vertical distribution of lateral forces
  - Increase in dynamic loads due to torsional motions
  - Influence of higher modes, resulting in an increase in story shear and deformations

The Concept of Inelastic Response Spectra and Design Spectra

(Evolution of Seismic Design Factors (R, Ω and C_d) in Building Codes)

(Evolution of the Concept of Ductility in Building Codes)

Basic Design Objective of Seismic Resistant Design:

- *To protect the life safety of the building occupants and the general public.*
- *To control the severity of damage in small or moderate earthquakes.*

Expected Seismic Performance of Buildings:

- *Resist a **minor level of earthquake** ground motion **without damage***
- *Resist a **moderate level of earthquake** ground motion without structural damage, but possibly experience **some non-structural damage***

❑ *Elastic or lightly inelastic response*
❑ *Drift control*

Need **Elastic Response Spectrum**

- *Acceleration spectra for strength design*
- *Displacement spectra for stiffness (Drift) design*

- *Resist a **major level of earthquake** ground motion having an intensity equal to the strongest either experienced or forecast for the building site, **without collapse**, but possibly with some structural as well as nonstructural damage*

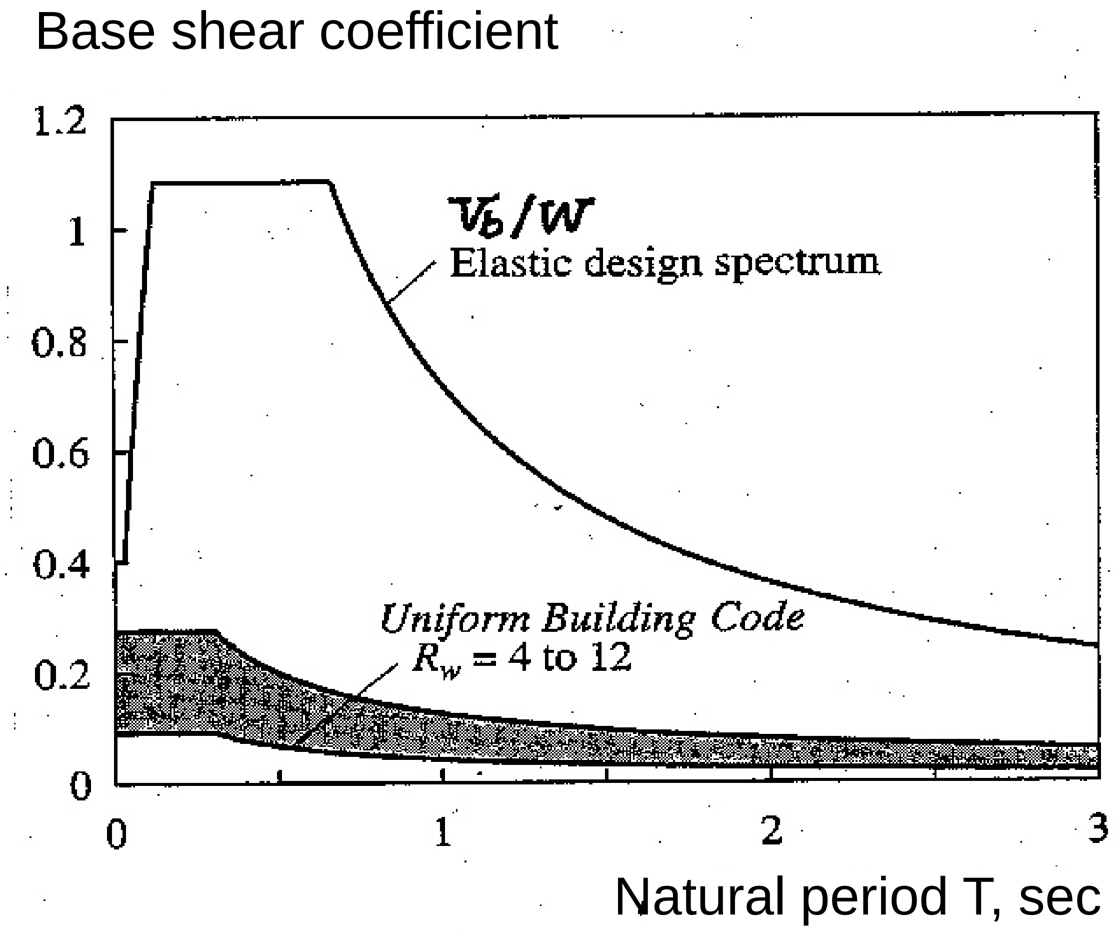
❑ *Inelastic response*
❑ *Control of inelastic deformation*

Need **Inelastic Response Spectrum**

- *Inelastic strength Demand spectra for strength design*

UNIFORM BUILDING CODE

Comparison of base shear coefficient from elastic design spectrum and Uniform Building Code (a US. code) :



V_b: *Peak base shear induced in a linear elastic system by strong ground motion (PGA=0.4g)*

W: *The weight of the system*

Most buildings are designed for base shear much smaller than the elastic base shear *associated with the strongest shaking that can occur at the site.*

Buildings design by the code forces will be deformed ***beyond the limit of linearly elastic*** *behavior when subjected to ground motions represented by the 0.4g design spectrum. Thus it should not be surprising that buildings suffer damage during intense ground shaking.*

The design should be based on Inelastic Response Spectrum.

The challenge to the engineer is to design the structure so that the damage is controlled to an acceptable degree.

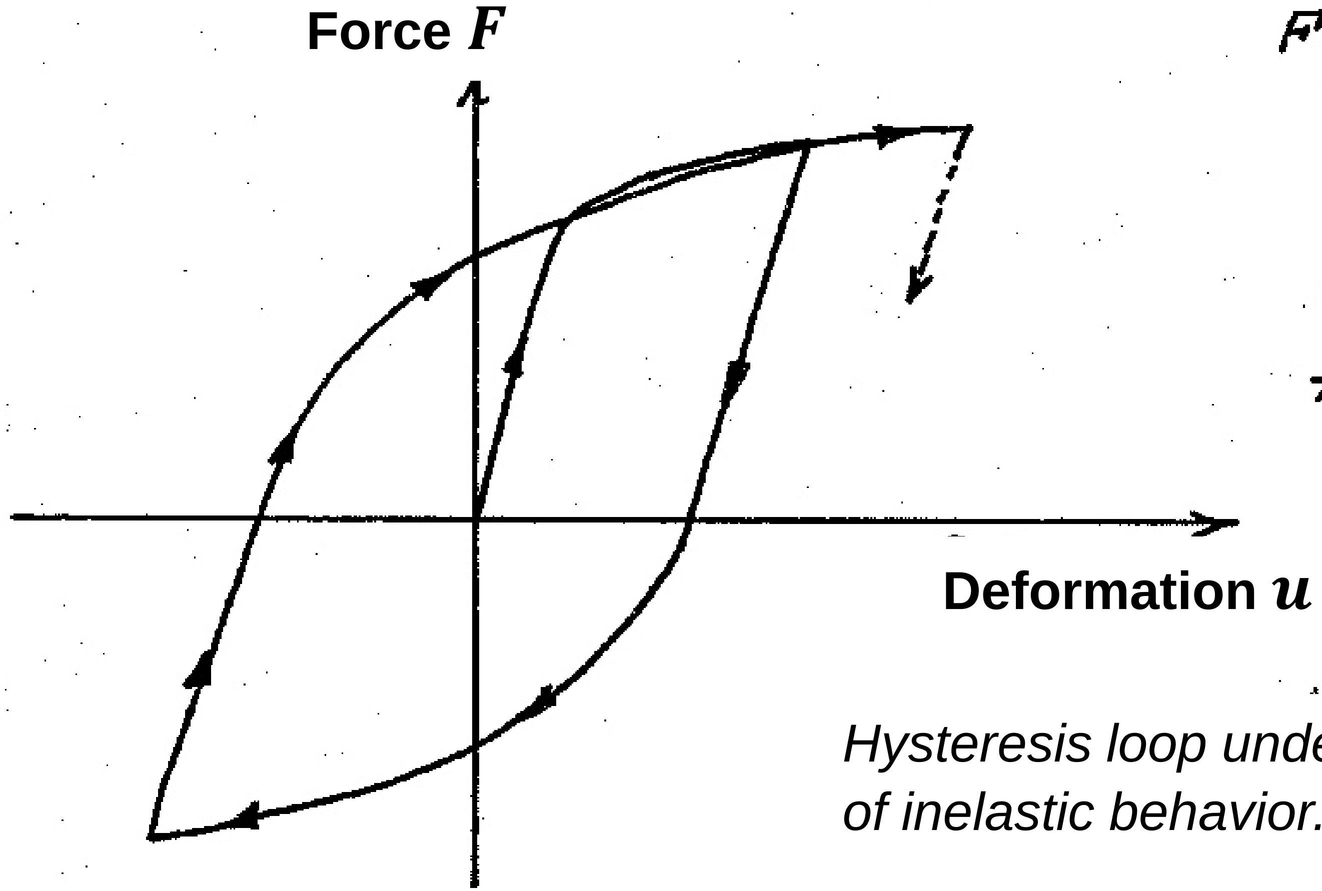
FORCE-DEFORMATION RELATIONS

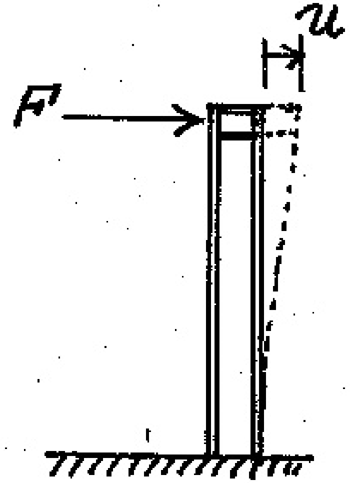
During an earthquake, structures undergo oscillatory motion with reversal of deformation.

The experimental results from cyclic loading conditions indicate that the cyclic force-deformation behavior of a structure depends on

- *the structural material (concrete, steel)*
- *the type of structural members (beam, shear member, axial member)*
- *how members are assembled into the structural system.*

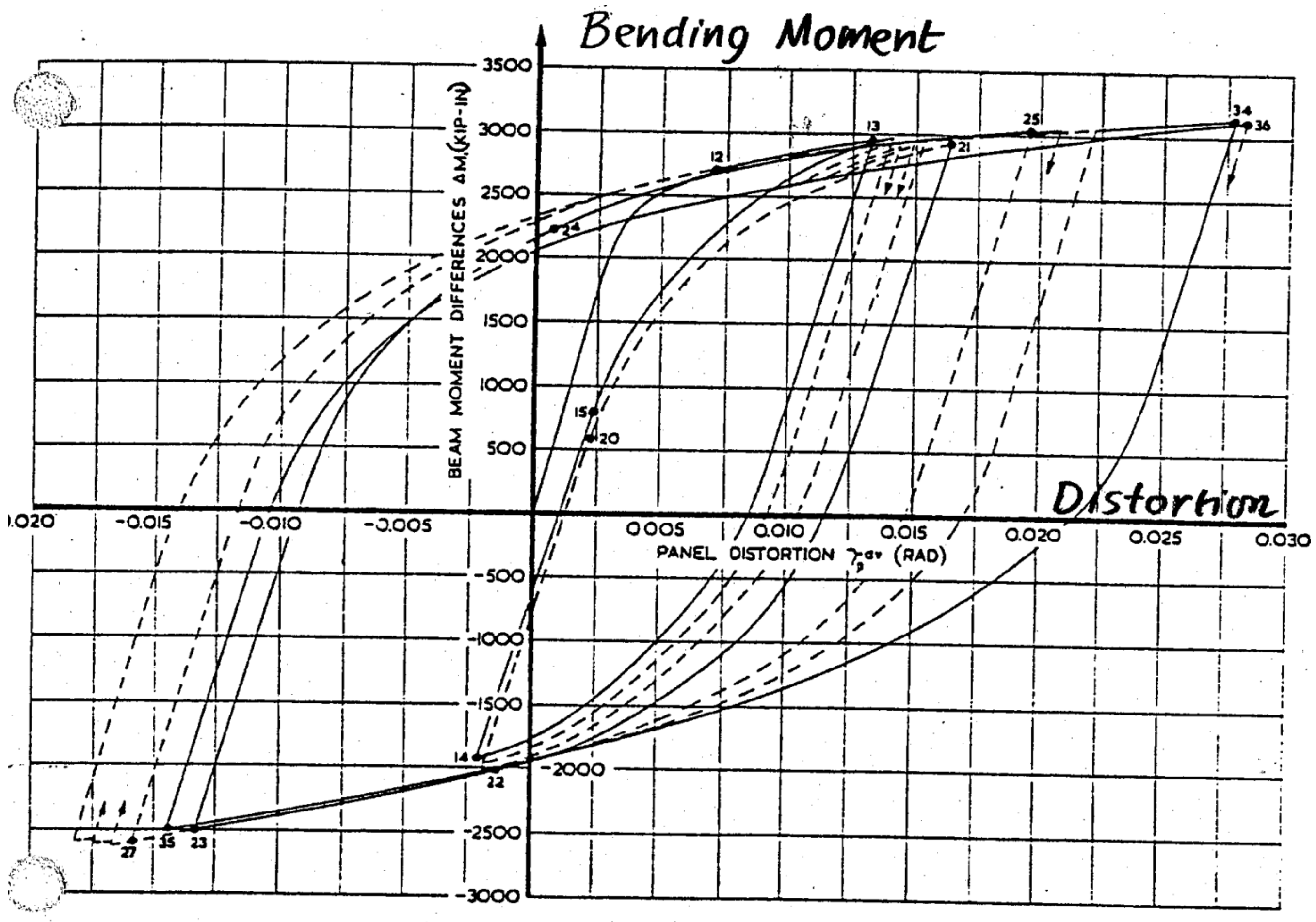
Force-deformation relation of a steel beam

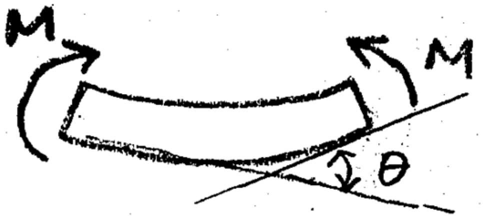




Hysteresis loop under cyclic deformation because of inelastic behavior.

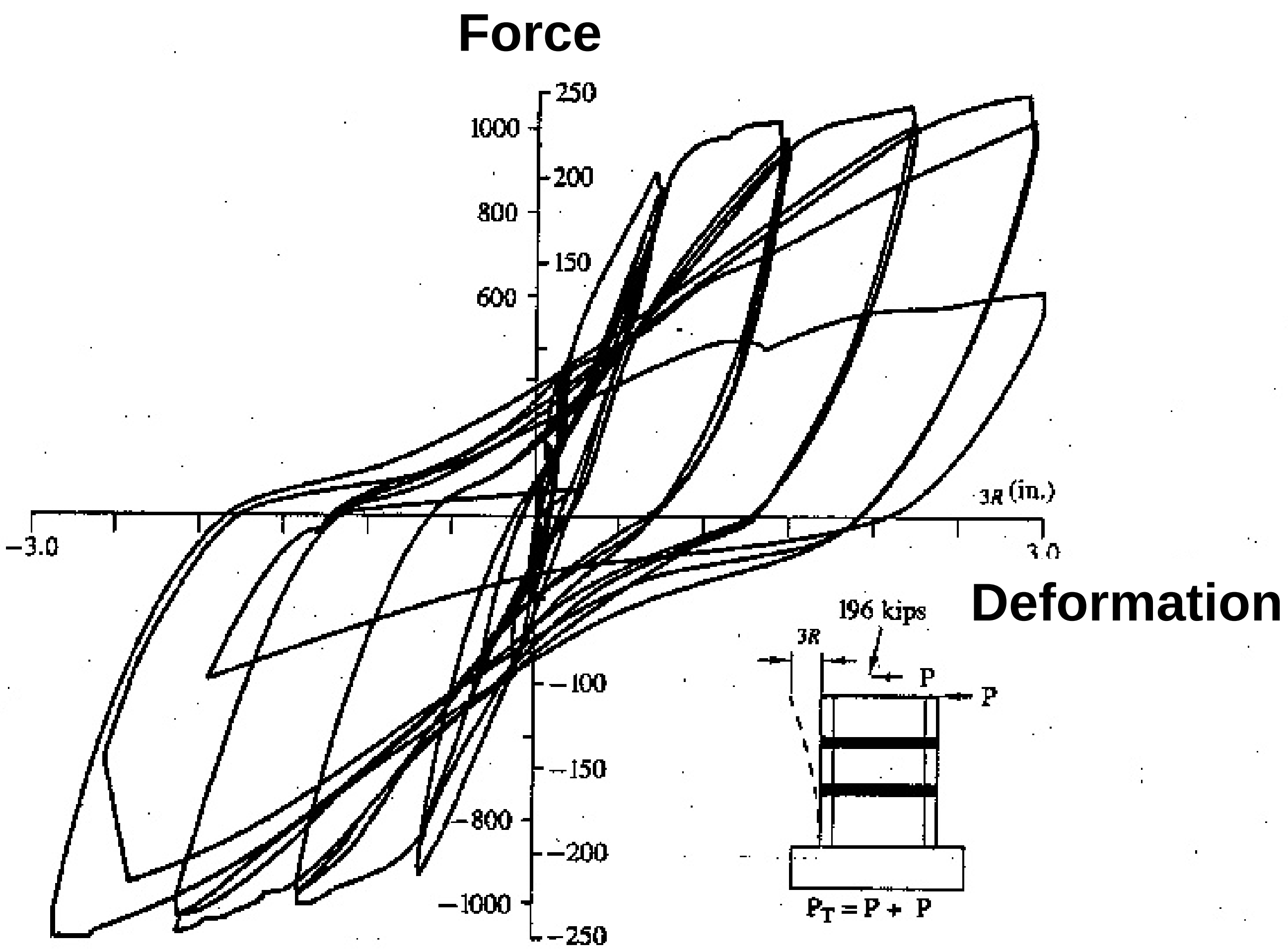
Force-deformation relations of a structural steel component



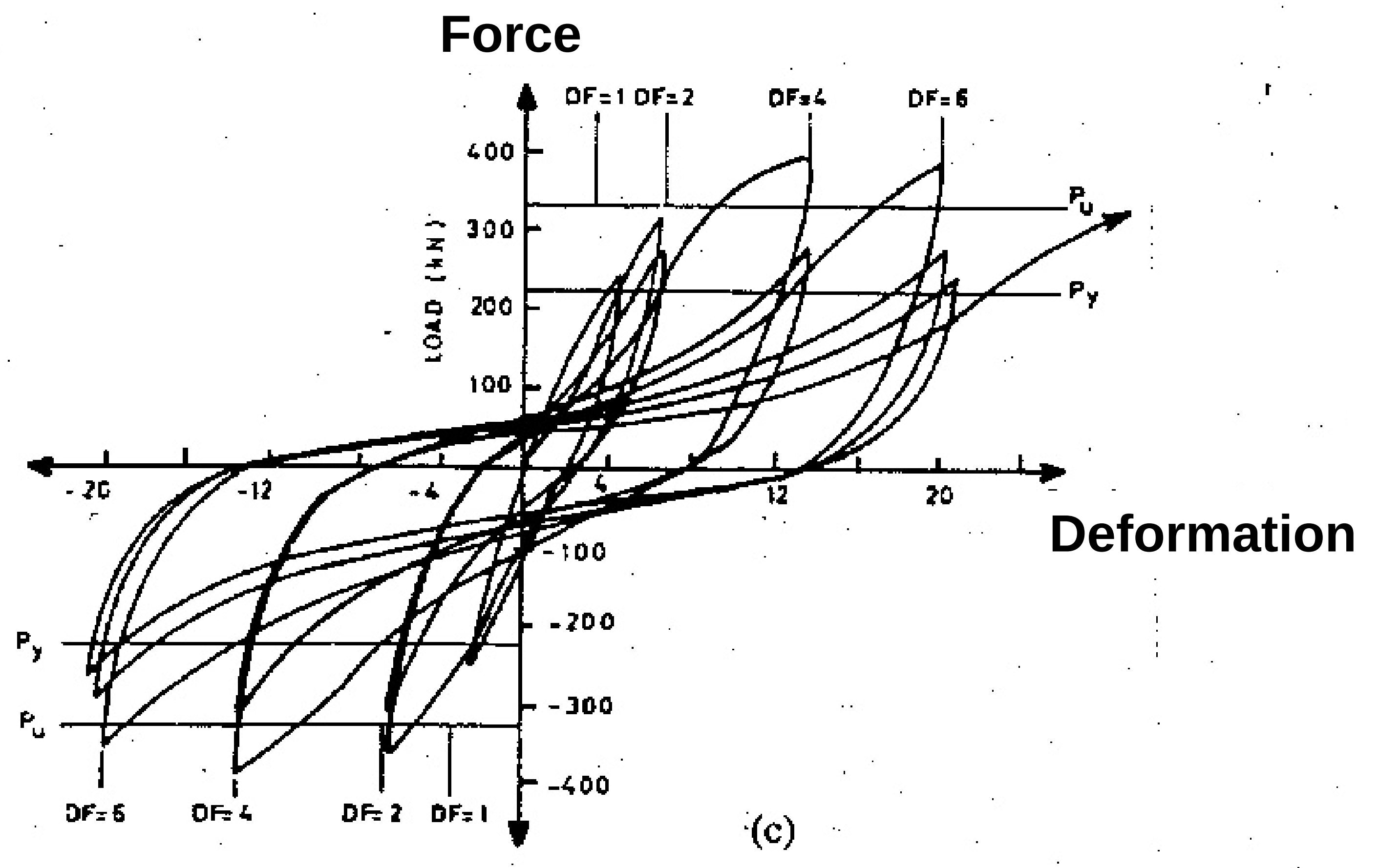


Hysteresis Loop

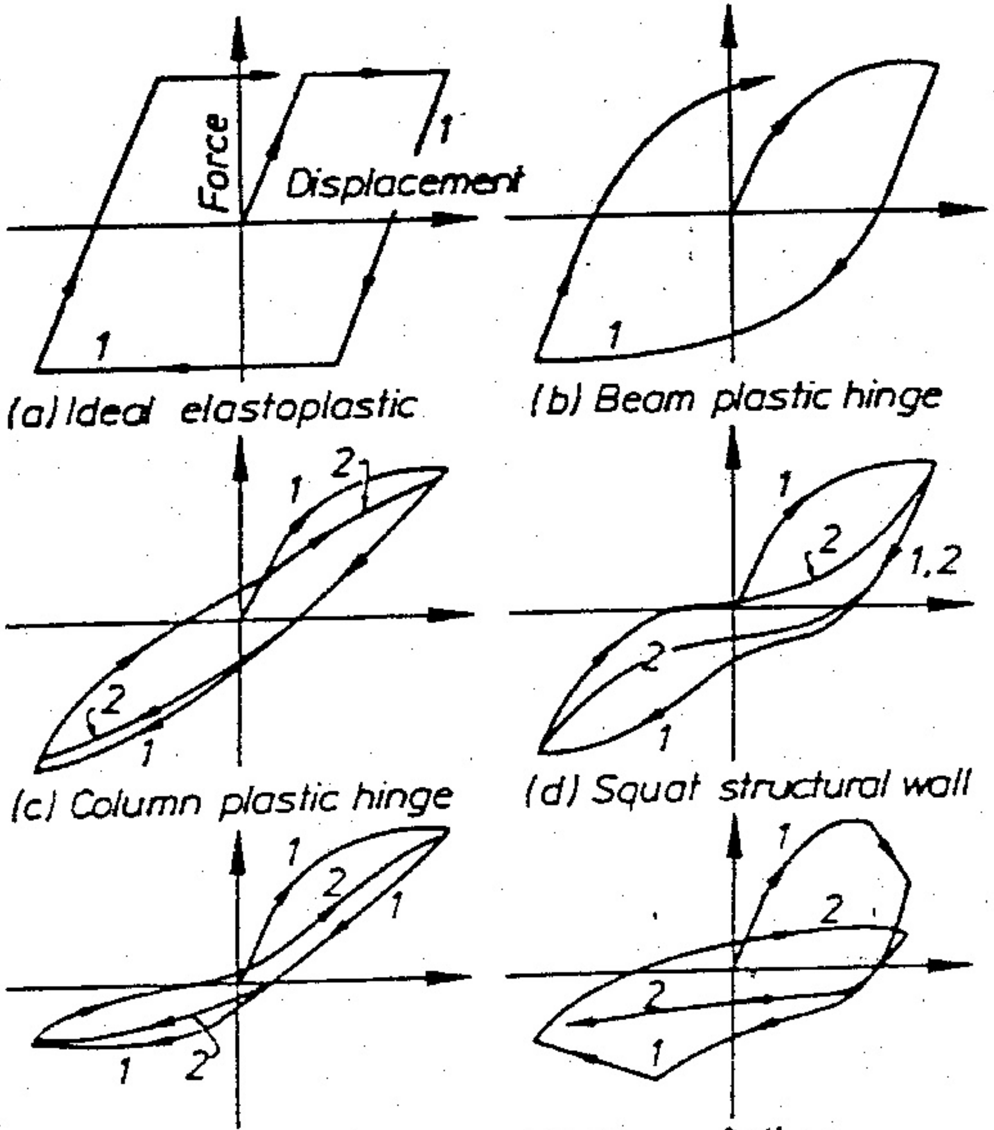
Force-deformation relation of a reinforced concrete structure



Force-deformation relation of a masonry structure



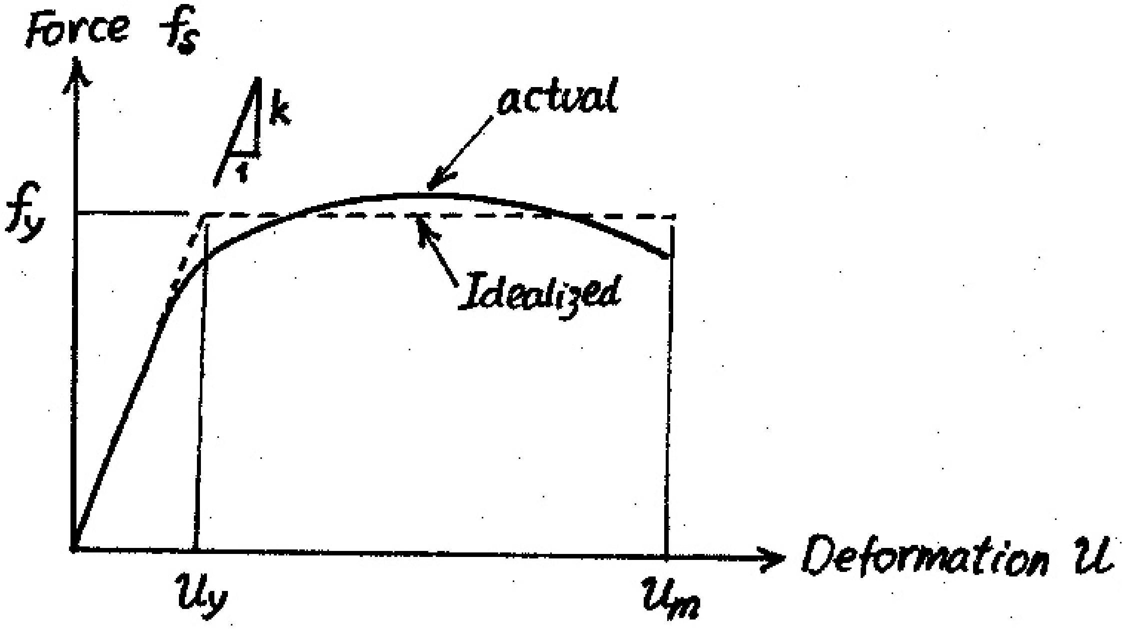
Typical force-deformation hysteresis loop shapes for concrete and masonry structural elements

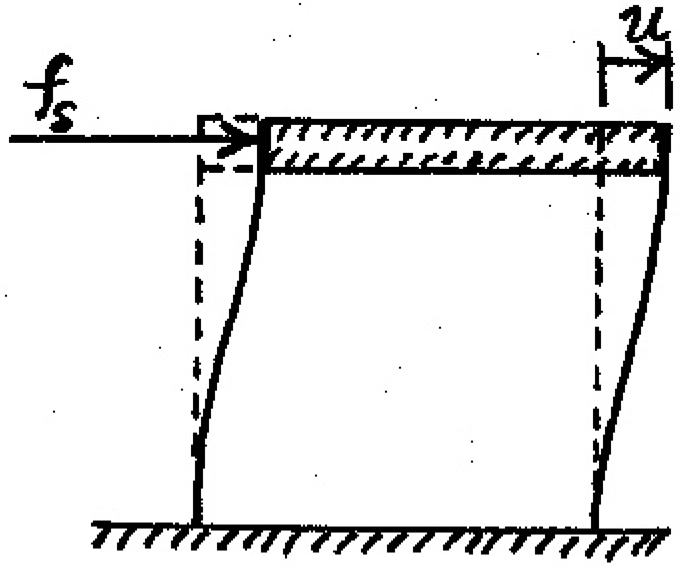


(a) Ideal elastoplastic
(b) Beam plastic hinge
(c) Column plastic hinge
(d) Squat structural wall
(e) T-section wall
(f) Shear failure

ELASTOPLASTIC IDEALIZATION

Force-deformation curve during initial loading: Actual and elastoplastic idealization





u_m: *Maximum deformation*

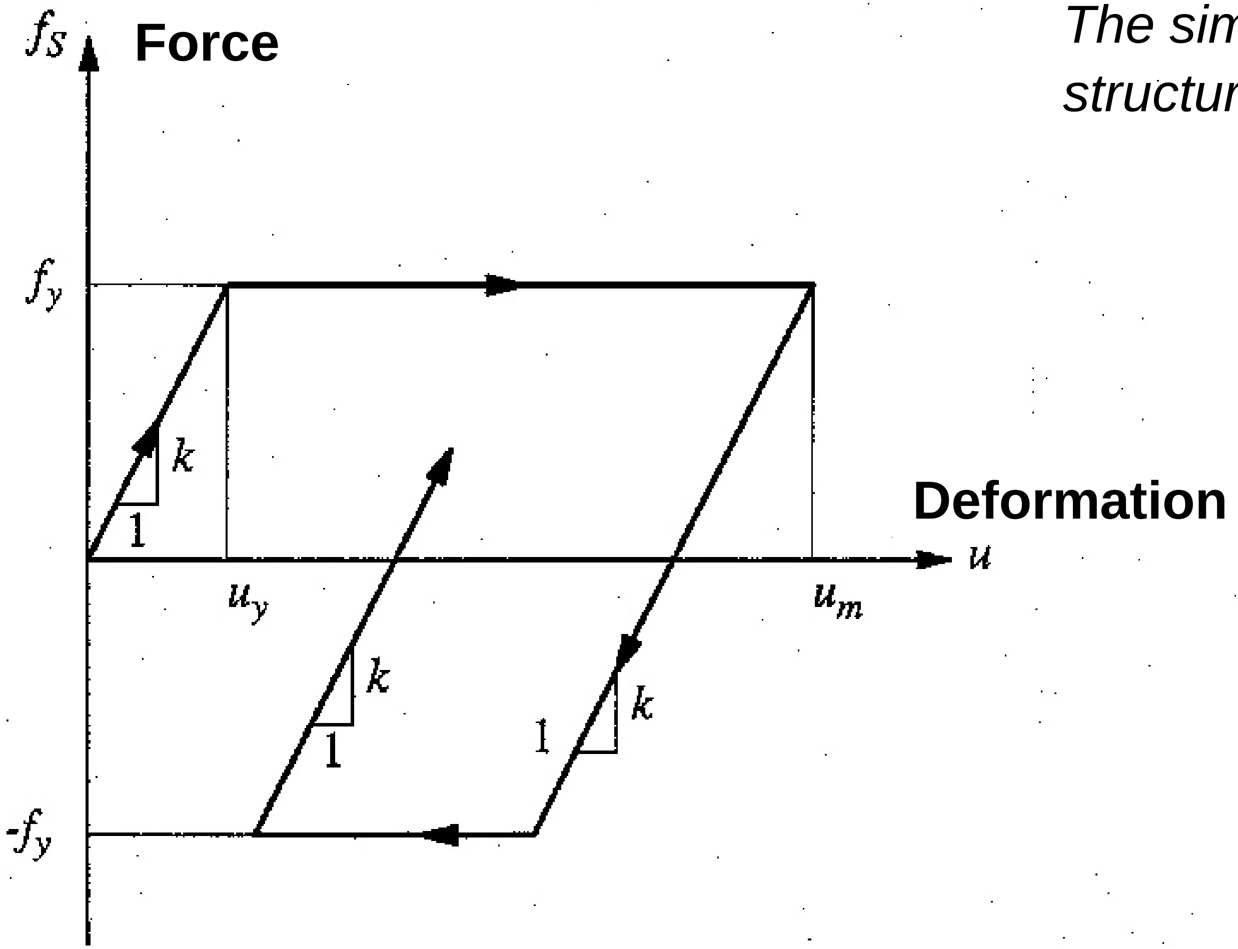
u_y: *Yield deformation*

f_y: *Yield strength*

K: *Stiffness in elastic range*

ELASTOPLASTIC IDEALIZATION

Force-deformation curve of an elastoplastic system for a typical cycle of loading, unloading, and reloading:



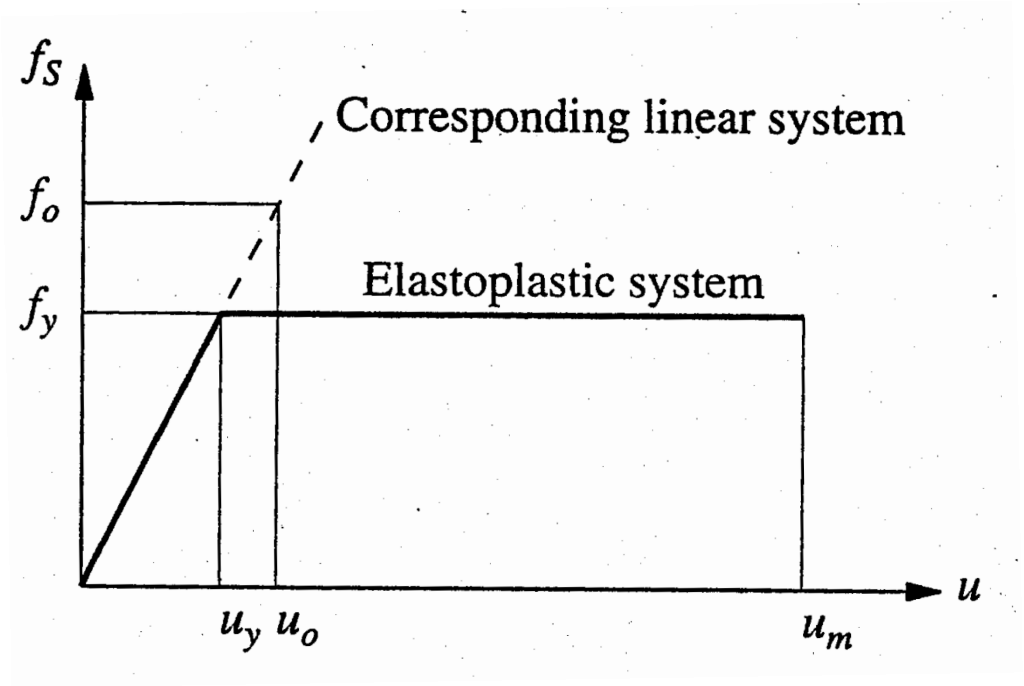
The simplest model of inelastic behavior of simple structure under cyclic loading

$$\boxed{\mu = ductility\ factor = u_m/u_y}$$

*The ductility factor is commonly used as **an index of seismic-induced inelastic damage**.*

ELASTOPLASTIC IDEALIZATION

Elastoplastic system and its corresponding linear system (having the same k in their initial loading phases)



In order to understand the effects of the inelastic force-deformation relation on the earthquake response, it is necessary to evaluate the peak deformation of an elastroplastic system and compare this deformation to the peak deformation of the corresponding linear system.

RESPONSE OF ELASTOPLATIC SYSTEM TO EARTHQUAKE GROUND MOTION

Equation of motion: $$m\frac{d^2u}{dt^2} + c\frac{du}{dt} + f_S(u) = -m\frac{d^2u_g}{dt^2}$$

where $f_S(u)$ is **a nonlinear function of $\boldsymbol{u}$** (as shown earlier).

The response $u(t)$ to an earthquake ground motion $\frac{d^2u_g}{dt^2}$ can be computed from the above equation by a **step-by-step direct integration**.

The nonlinear resisting force f_S can also be obtained from **the elastoplastic force-deformation relation.**

For a low-level ground motion, the system may behave like a linear elastic system with

$$f = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \qquad \left(T = {}^{1}/_{f}\right),$$

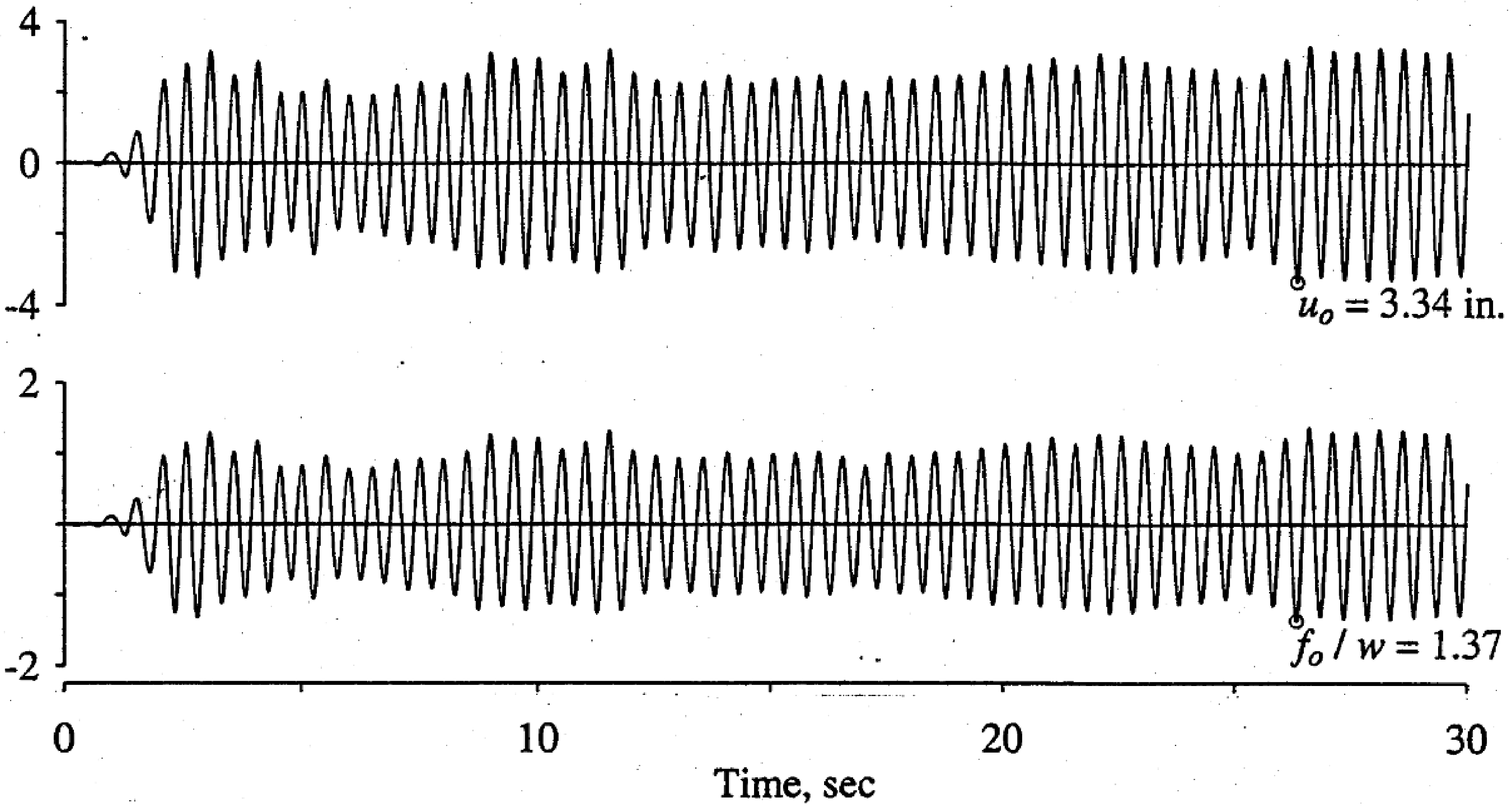
where k= initial stiffness in the elastic range.

EFFECTS OF INELASTICITY ON EARTHQUAKE RESPONSES

Response of a linearly elastic system with T=0.5 sec and $\xi = 0$ to El Centro ground motion:

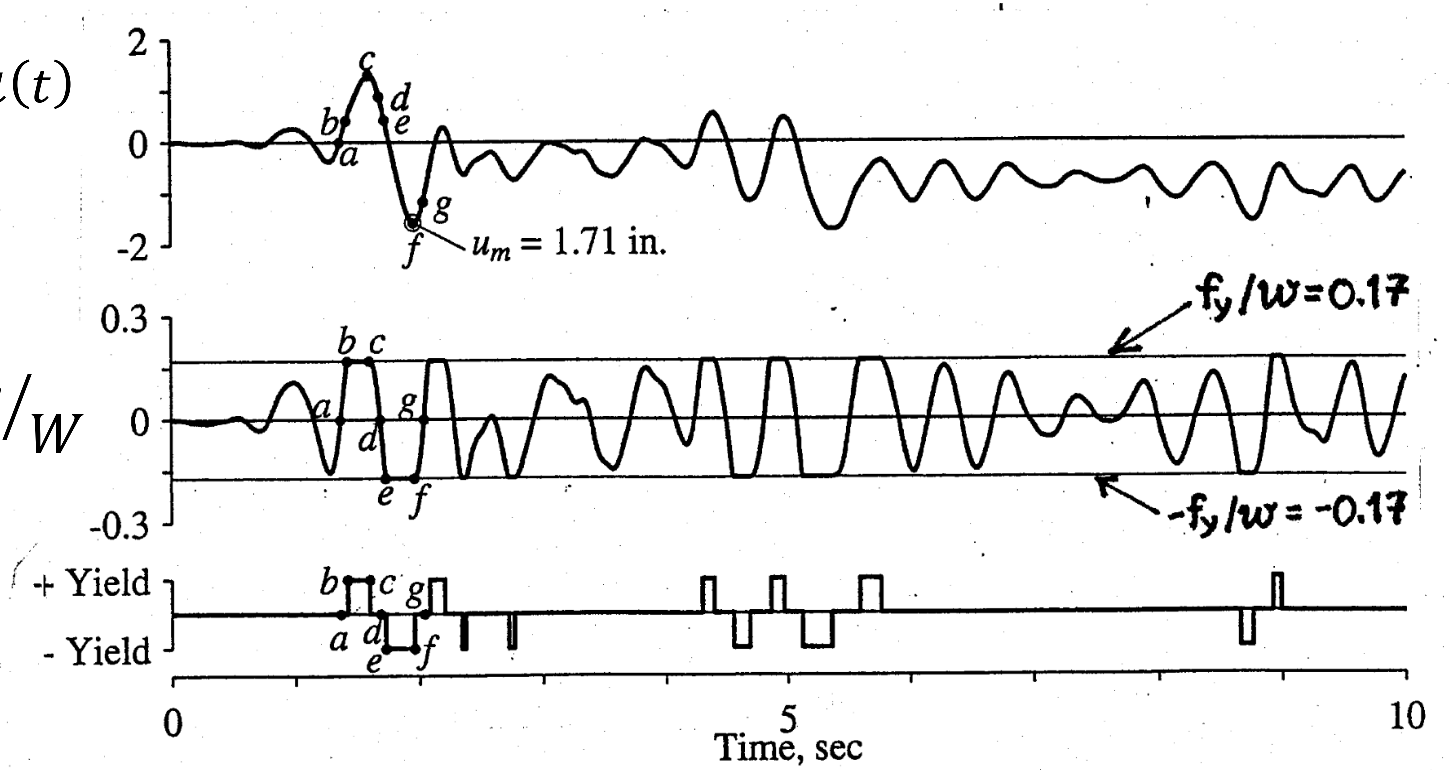
Deformation $u(t)$

Force $^{f_s}/_{W}$



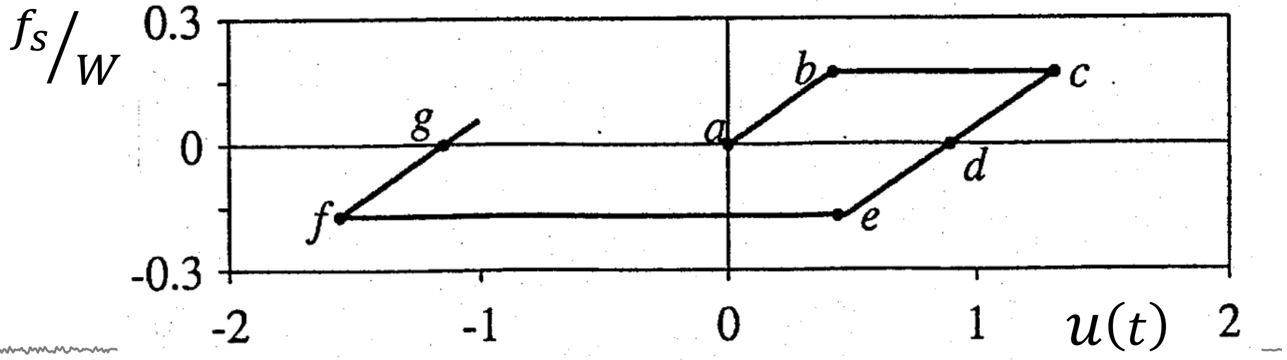
EFFECTS OF INELASTICITY ON EARTHQUAKE RESPONSES

Response of an elastoplastic system having the same mass and initial stiffness as the linear system ($T = 0.5\ and\ \xi = 0$) to El Centro ground motion.



Time intervals of yielding

The yield strength f_y of the system is set to $0.125\ f_0$, that is $f_y = 0.125\ f_0 = 0.125 \times 1.37\ W = 0.17W$



EFFECTS OF INELASTICITY ON EARTHQUAKE RESPONSES

Deformation response and yielding of four elastoplastic systems due to El Centro ground motion;

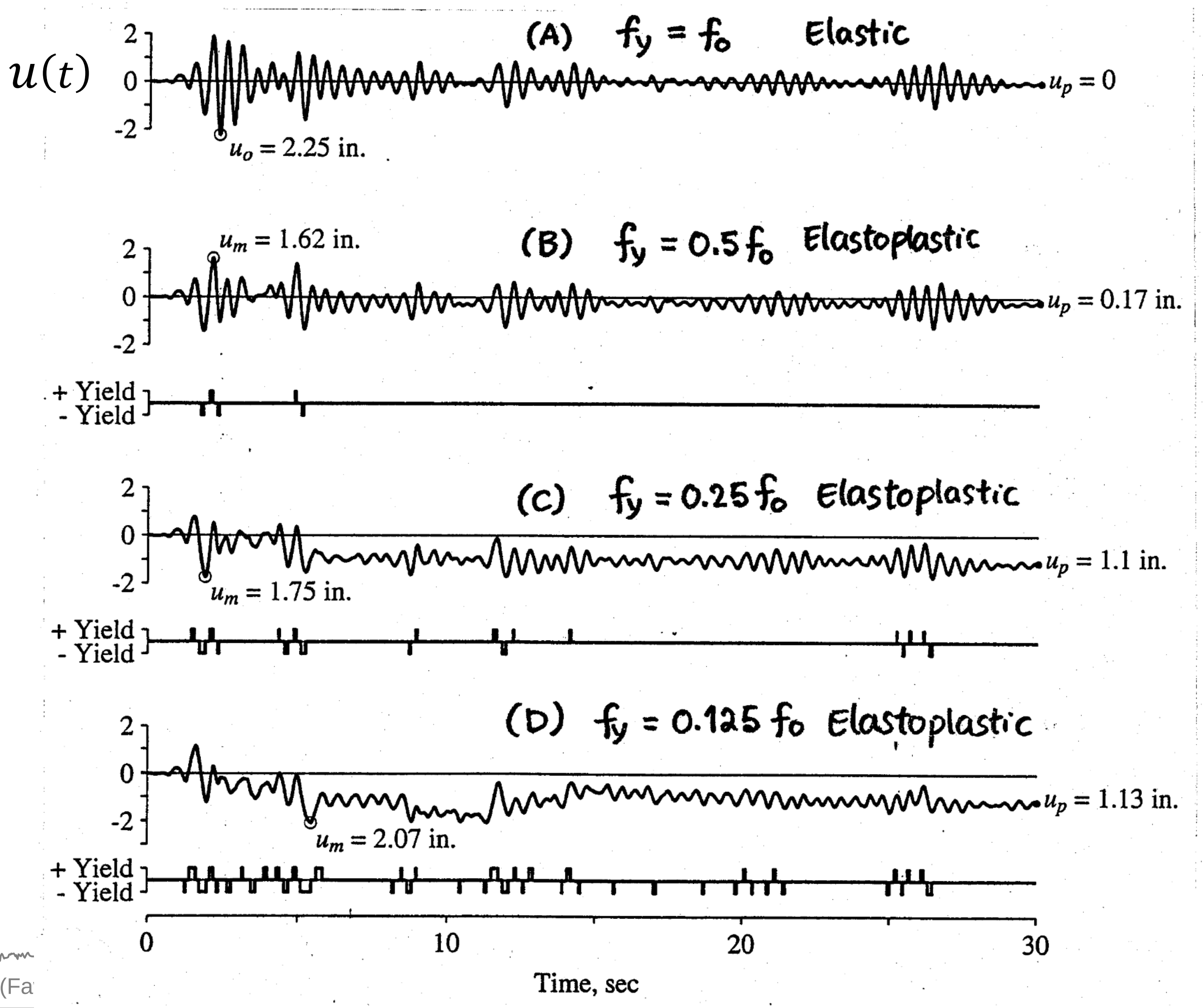
$$T = 0.5\ sec, \xi = 0.05$$

$$and\ f_y = f_0, 0.5 f_0, 0.25\ f_0\ and\ 0.125\ f_0$$

All four systems have identical properties in their linearly elastic range, but they differ in their yield strength.

Systems with lower yield strength yield more frequently and for longer intervals.

With more yielding, the permanent deformation u_p of the structure after the ground stops shaking tends to increase.



DUCTILITY FACTOR

Ductility factor for the four elastroplastic systems:

System	Yield strength $\boldsymbol{f_y}$	Yield deform. $\boldsymbol{u_y\ (in)}$	Max. Deform. $\boldsymbol{u_m(in)}$	Ductility factor $\boldsymbol{\mu}$
A	f_0	*2.25*	*2.25*	*1.00*
B	$0.5f_0$	*1.125*	*1.62*	*1.44*
C	$0.25f_0$	*0.562*	*1.75*	*3.11*
D	$0.125f_0$	*0.281*	*2.07*	*7.36*

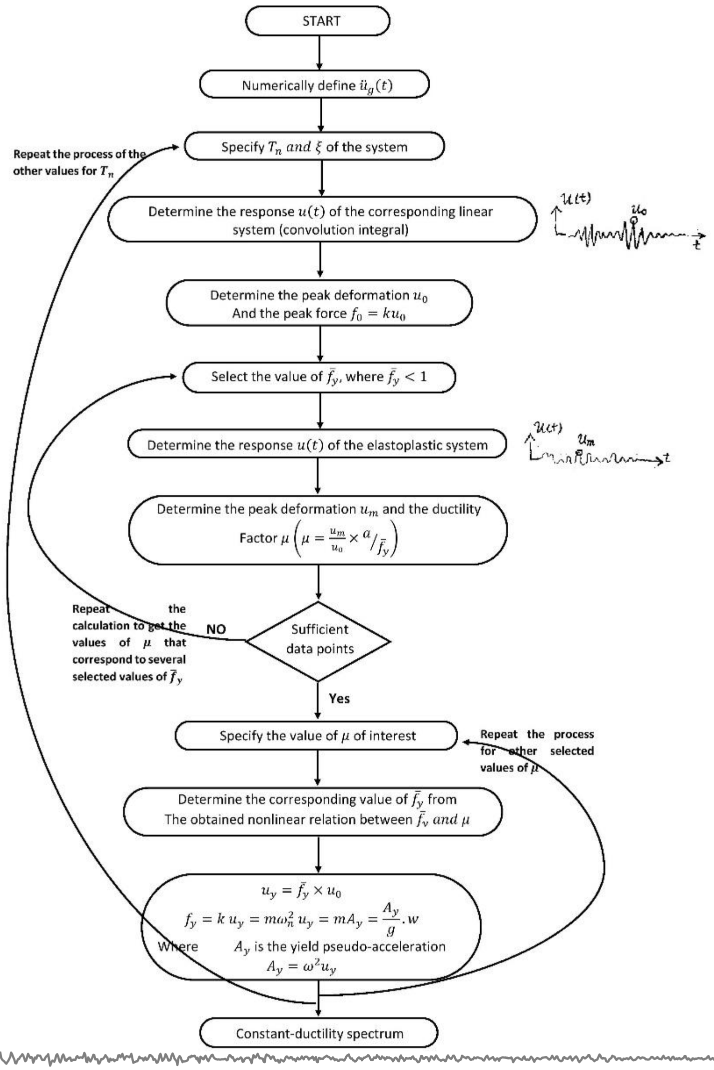
- *For each system, the computer ductility factor μ is the "**ductility demand**" imposed on elastoplastic system by the ground motion.*

- *The system should be designed such that its "**ductility capacity**" (i.e. the ability to deform beyond the elastic limit) **exceeds the "ductility demand".***

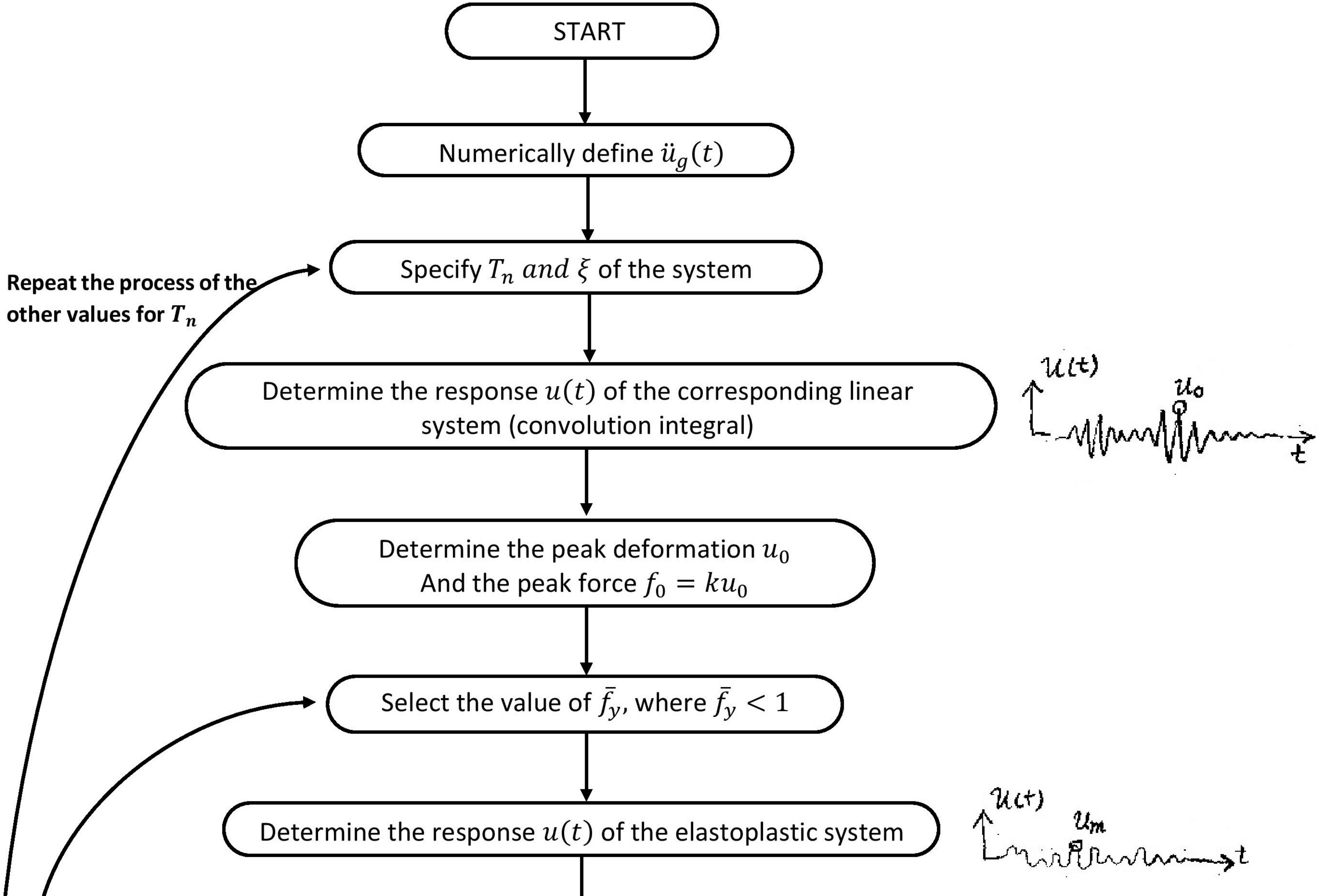
Decreasing in Yield Strength → Increasing in "Ductility Demand".

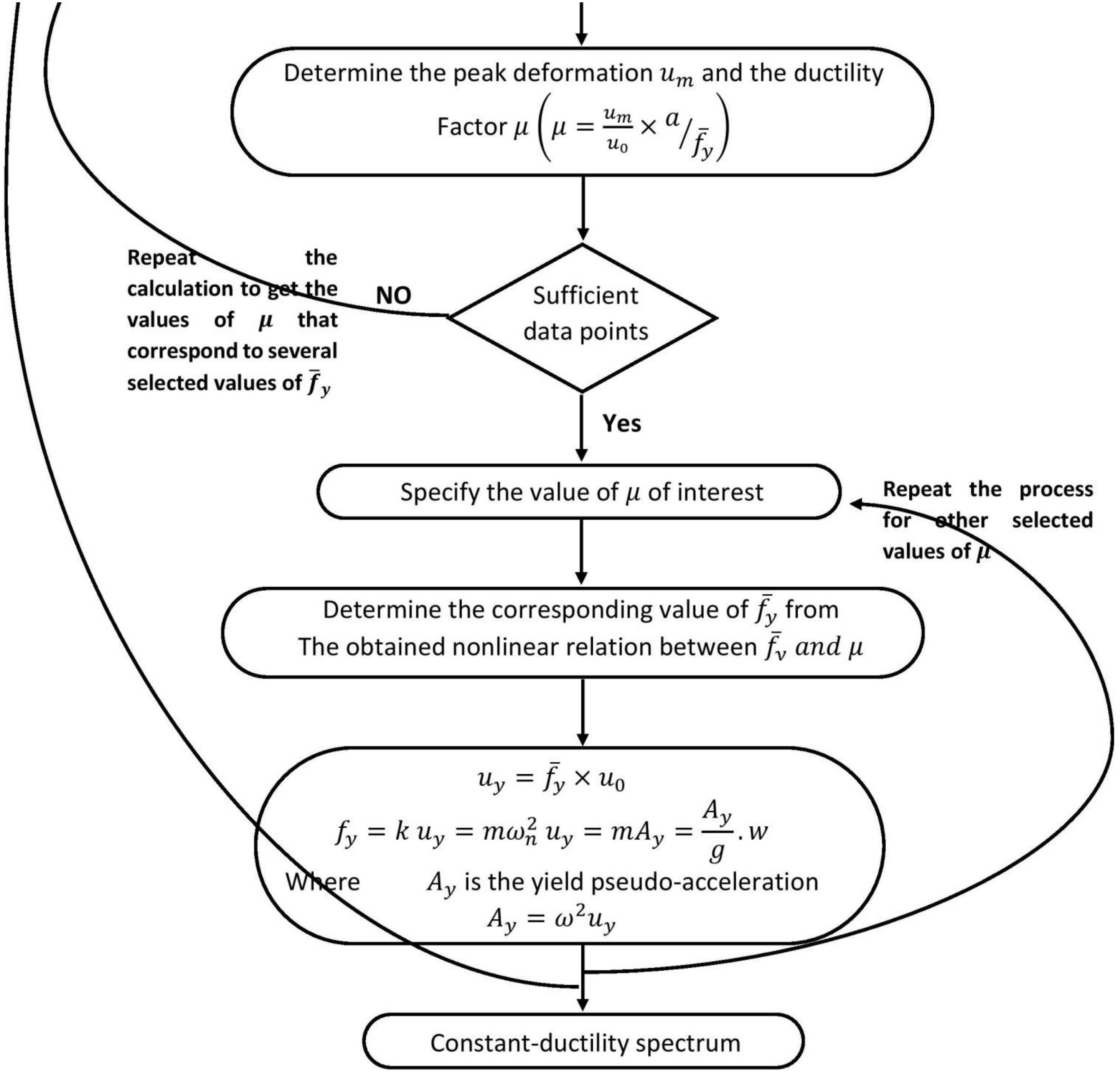
CONSTANT-DUCTILITY SPECTRUM

The procedure to construct the response spectrum for elastoplastic systems corresponding to specified levels of ductility factor is shown by a flow chart below.



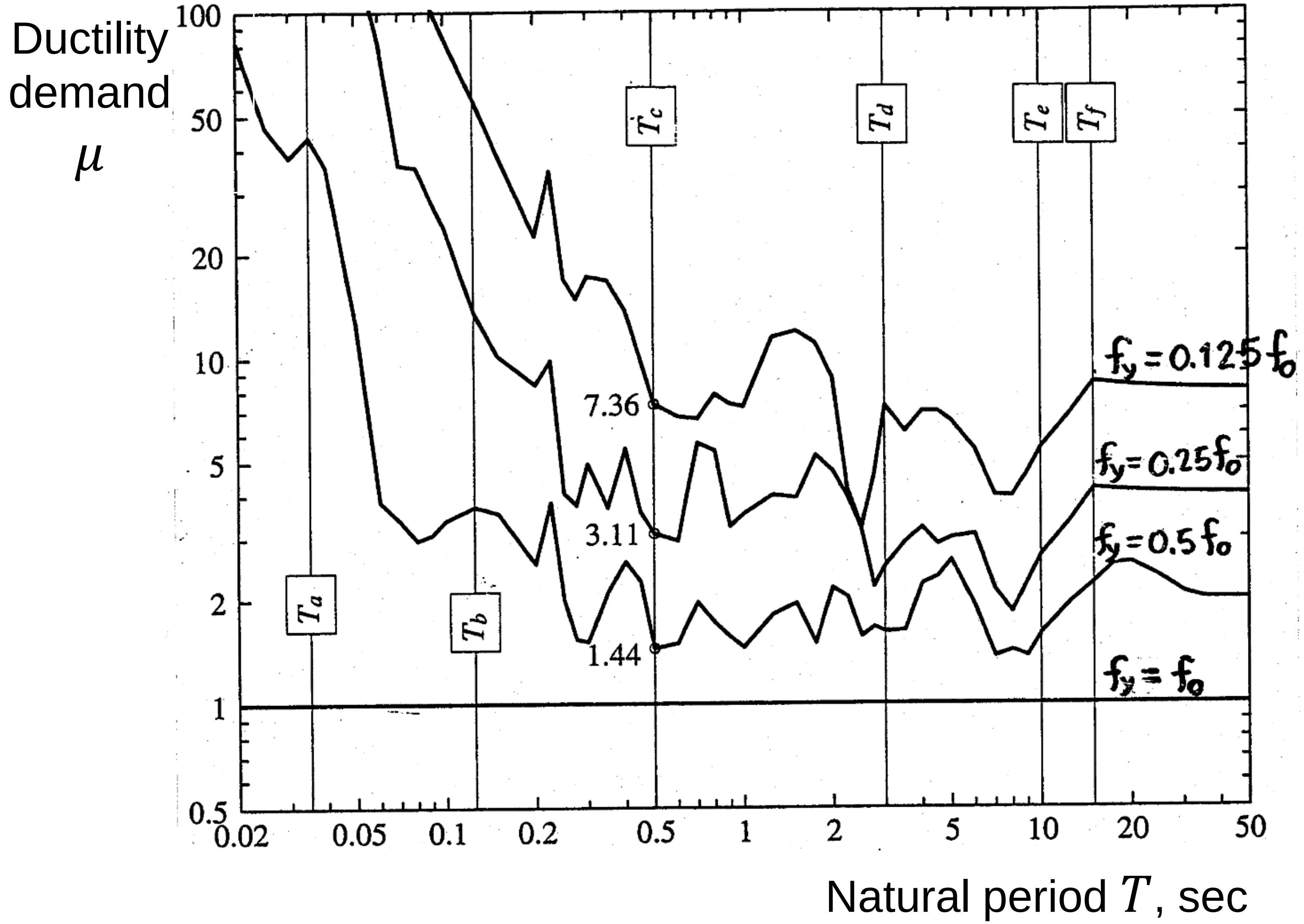
Please see next slides for enlarged flow chart


START
Numerically define $\ddot{u}_g(t)$
Specify T_n and ξ of the system
Repeat the process of the other values for T_n
Determine the response $u(t)$ of the corresponding linear system (convolution integral)
$u(t)$
u_o
t
Determine the peak deformation u_0
And the peak force $f_0 = ku_0$
Select the value of $\bar{f}_y$, where $\bar{f}_y < 1$
Determine the response $u(t)$ of the elastoplastic system
$u(t)$
u_m
t


Determine the peak deformation u_m and the ductility Factor μ $\left(\mu = \frac{u_m}{u_0} \times {}^{a}/_{\bar{f}_y}\right)$
Repeat the calculation to get the values of μ that correspond to several selected values of $\bar{f}_y$
NO
Sufficient data points
Yes
Specify the value of μ of interest
Repeat the process for other selected values of μ
Determine the corresponding value of $\bar{f}_y$ from The obtained nonlinear relation between $\bar{f}_y$ and μ
$u_y = \bar{f}_y \times u_0$
$f_y = k\,u_y = m\omega_n^2\,u_y = mA_y = \frac{A_y}{g}.w$
Where A_y is the yield pseudo-acceleration
$A_y = \omega^2 u_y$
Constant-ductility spectrum

DUCTILITY DEMAND

Ductility demand for elastroplastic system due to El Centro ground motion:
$\xi = 0.05 \; and \;\; f_y = f_0, 0.5f_0, 0.25f_0 \; and \; 0.125f_0.$



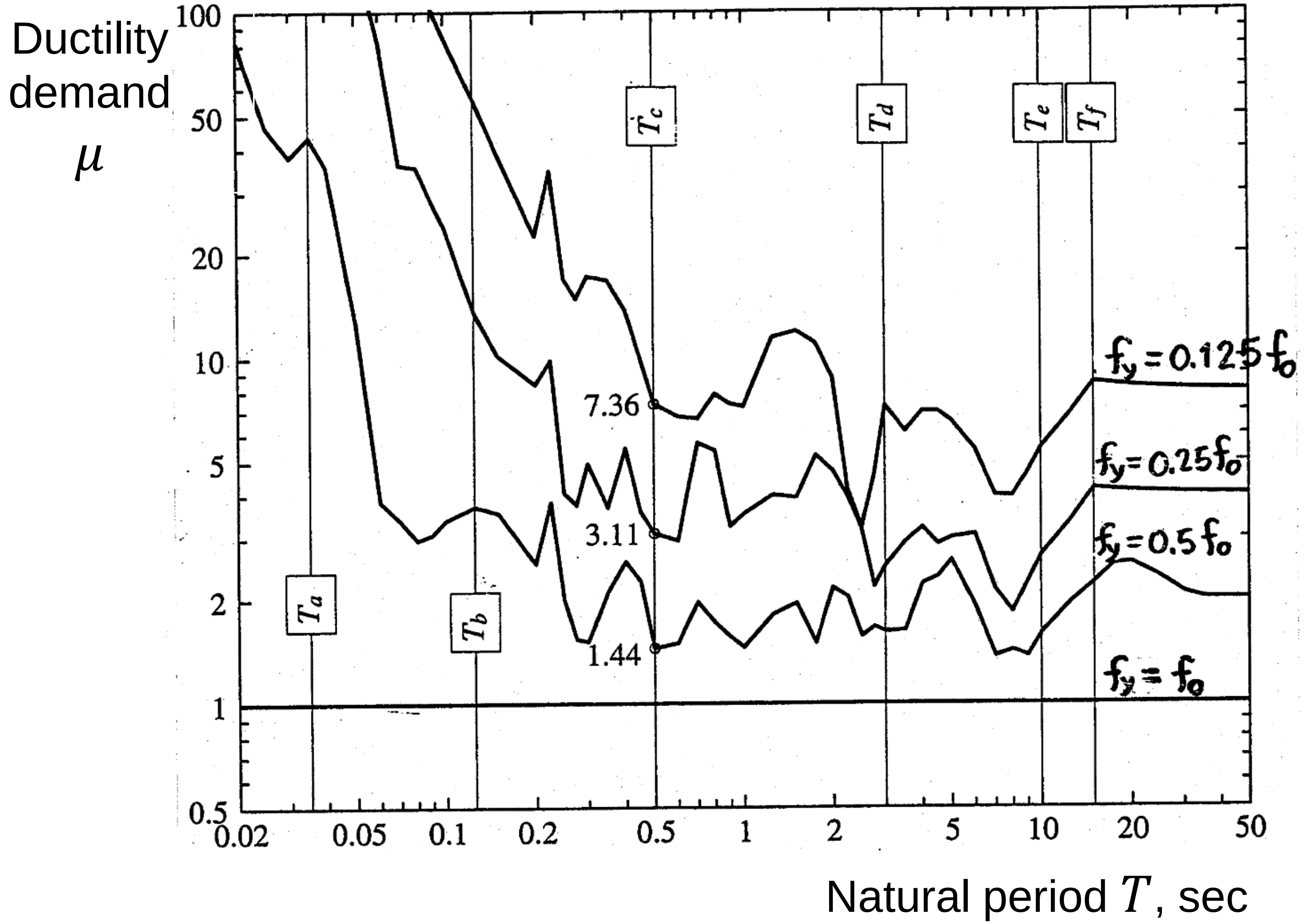
For systems with long natural periods, The ductility demand $\mu \approx f_0/f_y$

For systems with short natural period, The ductility demand μ can be longer than f_0/f_y.

It may be more appropriate to design these short-period systems to remain <u>elastic:</u> otherwise, the inelastic deformation and ductility demand may be excessive.

DUCTILITY DEMAND Ductility demand for elastroplastic system due to El Centro ground motion:

$$\xi = 0.05 \; and \;\; f_y = f_0, 0.5f_0, 0.25f_0 \; and \; 0.125f_0.$$

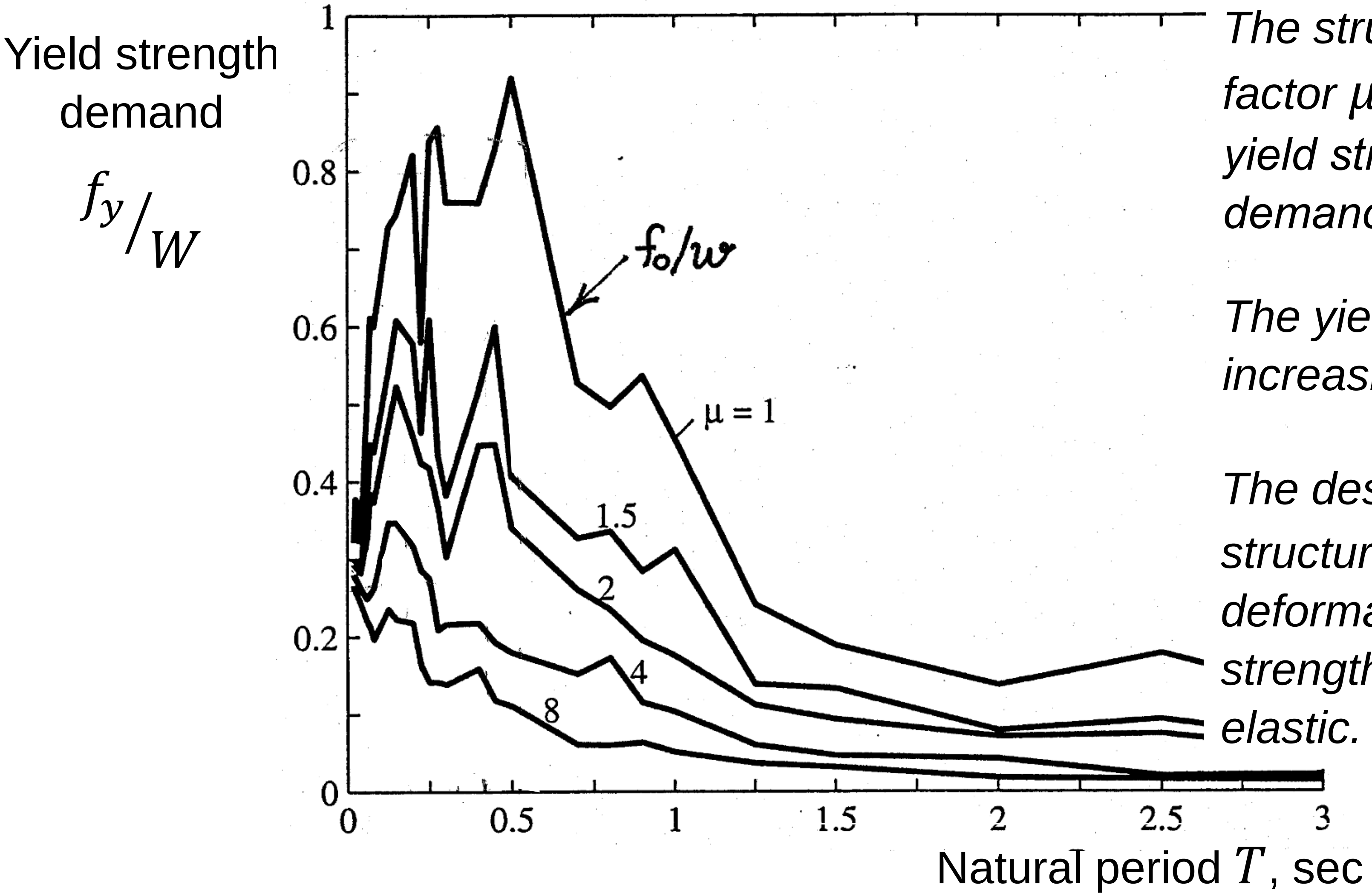


If the acceptable value of ductility factor μ is specified, it is possible to determine the corresponding yield strength f_y by an interpolative procedure.

This "f_y" may be considered as the <u>*"yield strength demand"*</u> *for the desired ductility factor μ.*

YIELD STRENGTH DEMAND

Yield strength demand for elastroplastic systems ($\xi = 0.05$) for specified ductility $\mu = 1, 1.5, 2, 4, and\ 8$; El Centro ground motion.

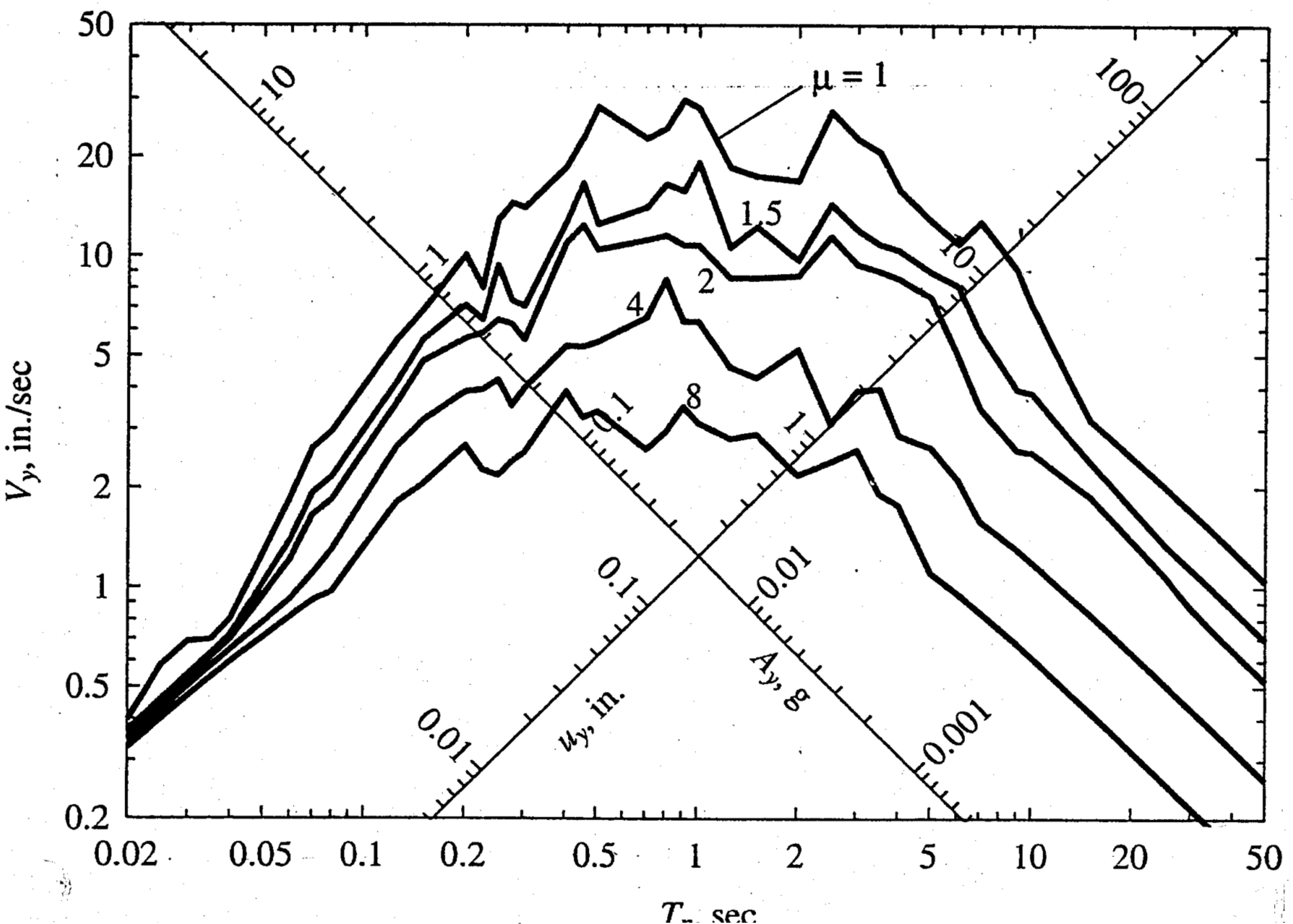


The structure having the specified ductility factor μ should be designed such that its yield strength exceeds the "yield strength demand".

The yield strength demand is reduced with increasing values of the ductility factor.

The design yield strength f_y for a simple structure permitted to undergo inelastic deformation can be much lower than the strength required for the structure to remain elastic.

Combined $D_y - V_y - A_y$ plot of the above constant ductility response spectrum



Note:

$D_y = u_y$

the yield deformation

$V_y = \omega_n u_y$

the yield pseudo-velocity

$A_y = \omega_n^2 u_y$

the yield pseudo-acceleration

DESIGN OPTIONS

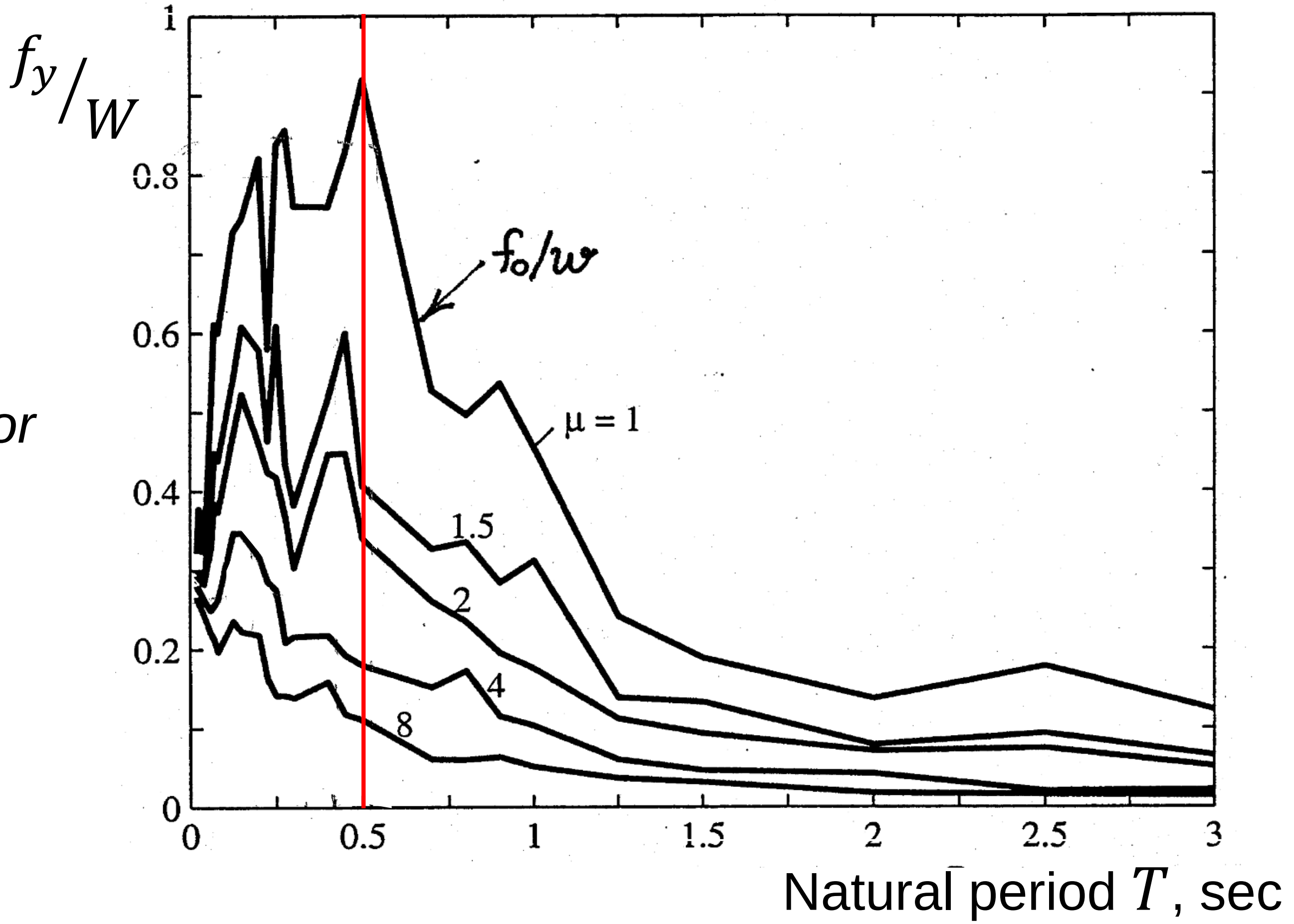
A structure may be designed for earthquake resistance by making it strong, by making it ductile, or by designing it for **economic combinations of both properties.**

Consider a simple structure with $T = 0.5\,\sec\ and\ \xi = 0.05$

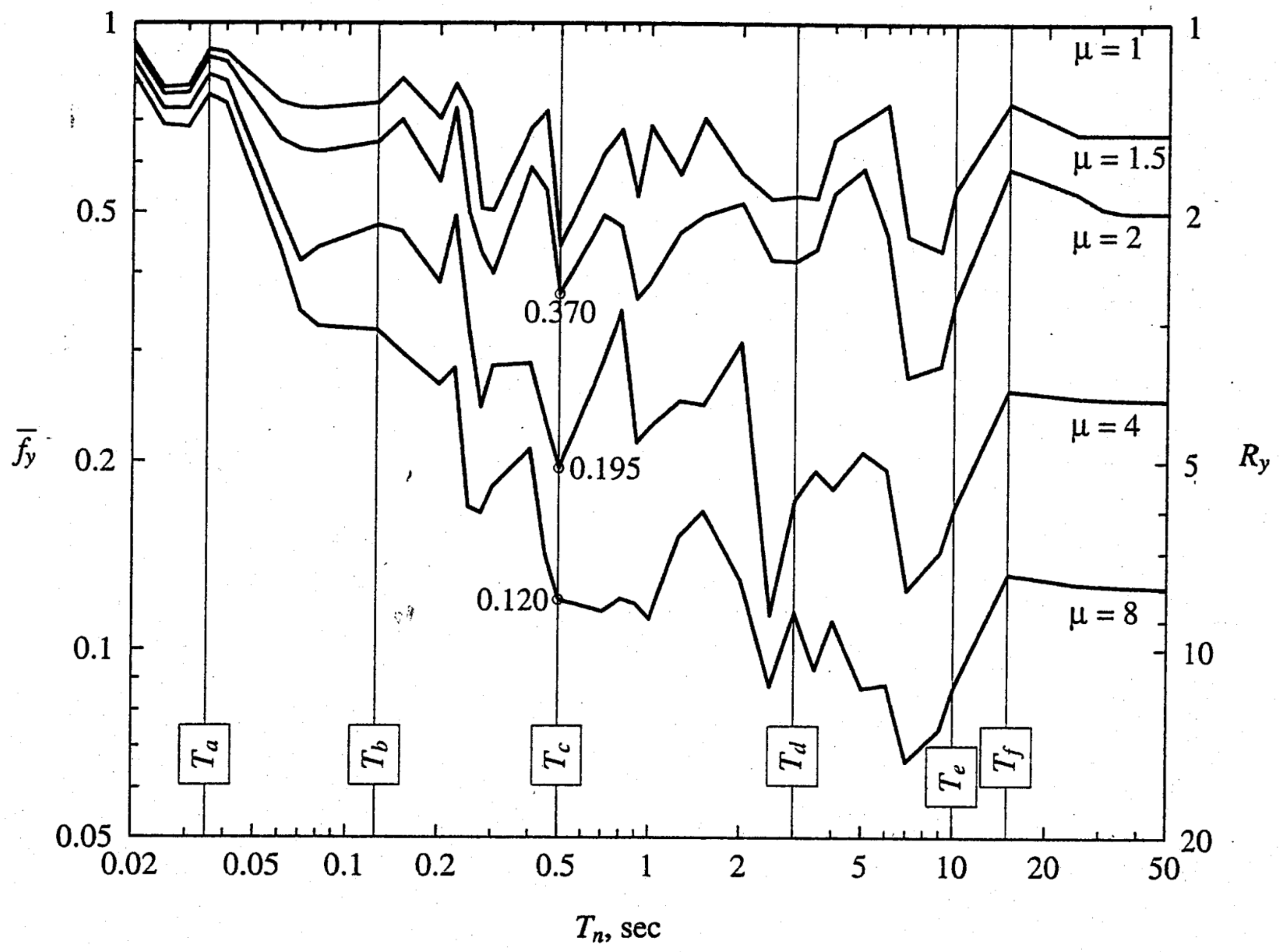
If the structure is designed for strength $f_0 = 0.919\ W$ or larger, it will remain within its linearly elastic range during the El Centro ground motion; therefore, it needs not to be ductile.

On the other hand, if it can develop a ductility factor of 8, it needs to be designed only 12% of the strength f_0 (that is only $0.11W$!)

Alternatively, it may be designed for strength equal to 37% of f_0 and a ductility capacity of 2.



Normalized strength $\bar{f}_y$ of elastoplastic systems as a function of natural vibration period T_n for $\mu = 1, 1.5,\ 2,\ 4$ and 8; $\xi = 5\%$; El Centro ground motion.



DESIGN OPTIONS

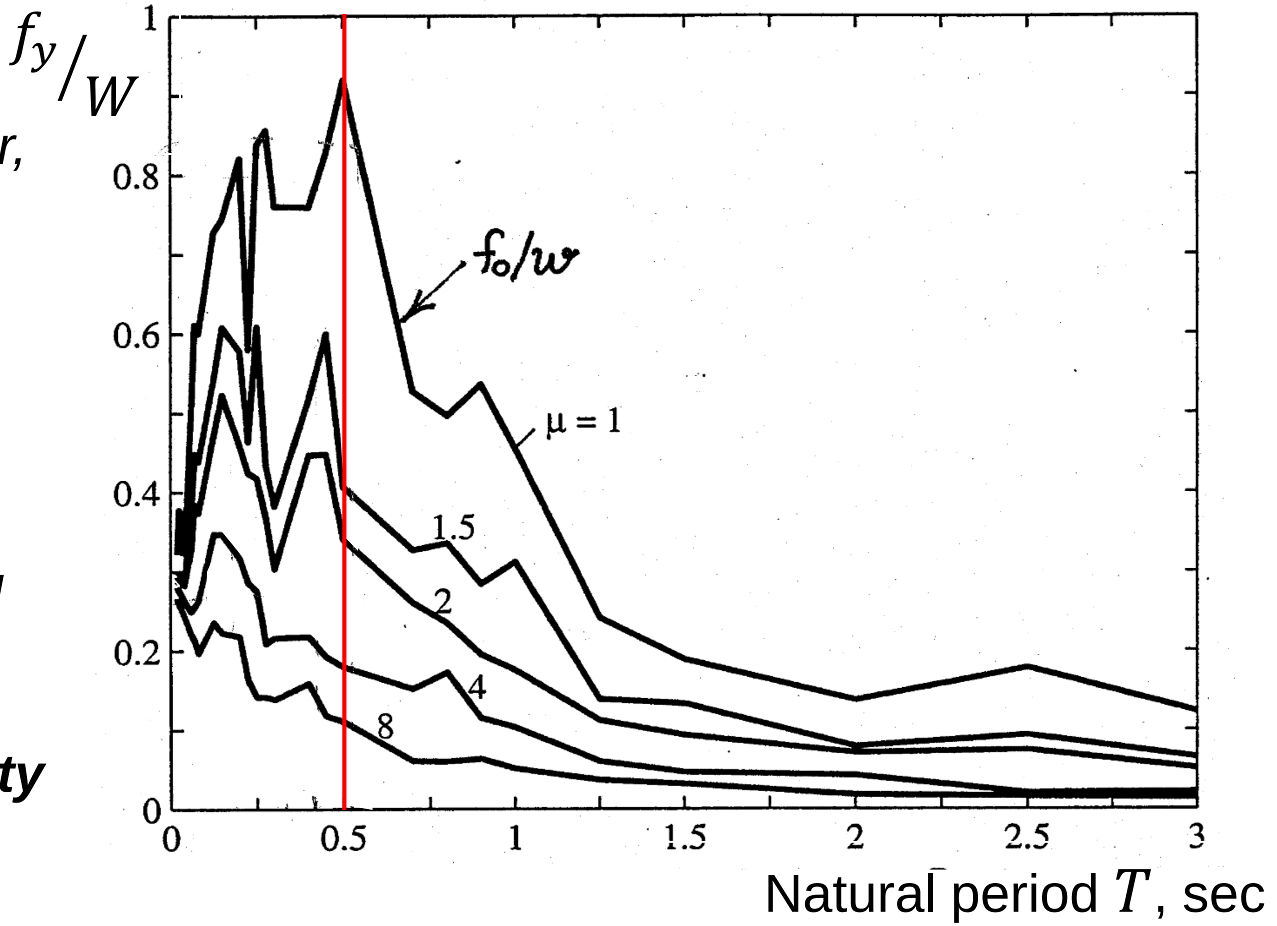
A structure may be designed for earthquake resistance by making it strong, by making it ductile, or by designing it for **economic combinations of both properties.**

Consider a simple structure with $T = 0.5\,\sec\, and\ \xi = 0.05$

For some types of materials and structural member, ductility is difficult to achieve. In such cases, the structure should be designed to have a high yield strength and low ductility.

For others, providing ductility is much easier than providing lateral strength. So, the structure in this case should be designed for low yield strength and high ductility.

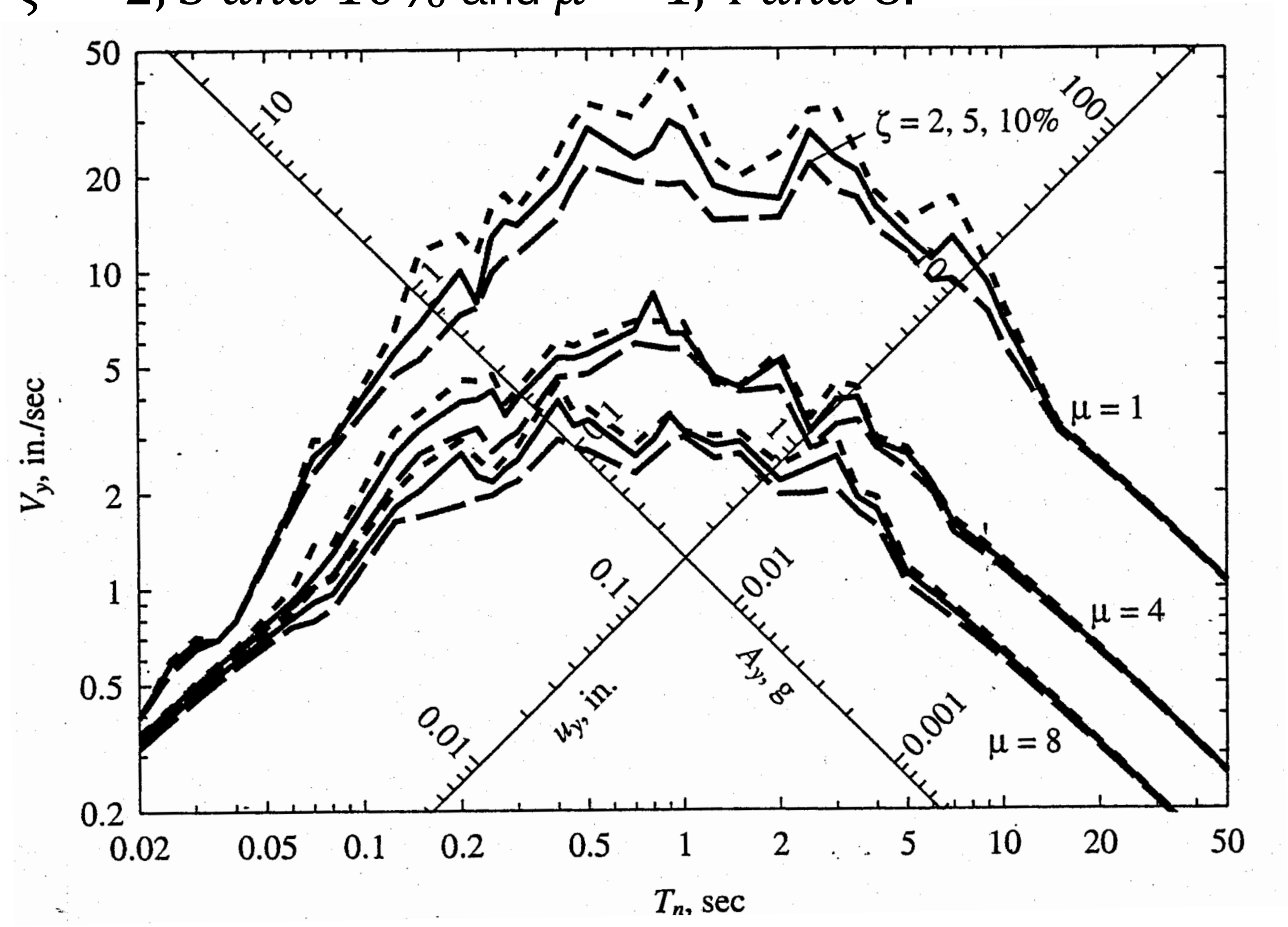
Economic combinations of strength and ductility properties.



RELATIVE EFFECTS OF YIELDING AND DAMPING

Response spectra for elastoplastic systems and EL Centro ground motion;

$\xi = 2, 5\ and\ 10\%$ and $\mu = 1, 4\ and\ 8$.

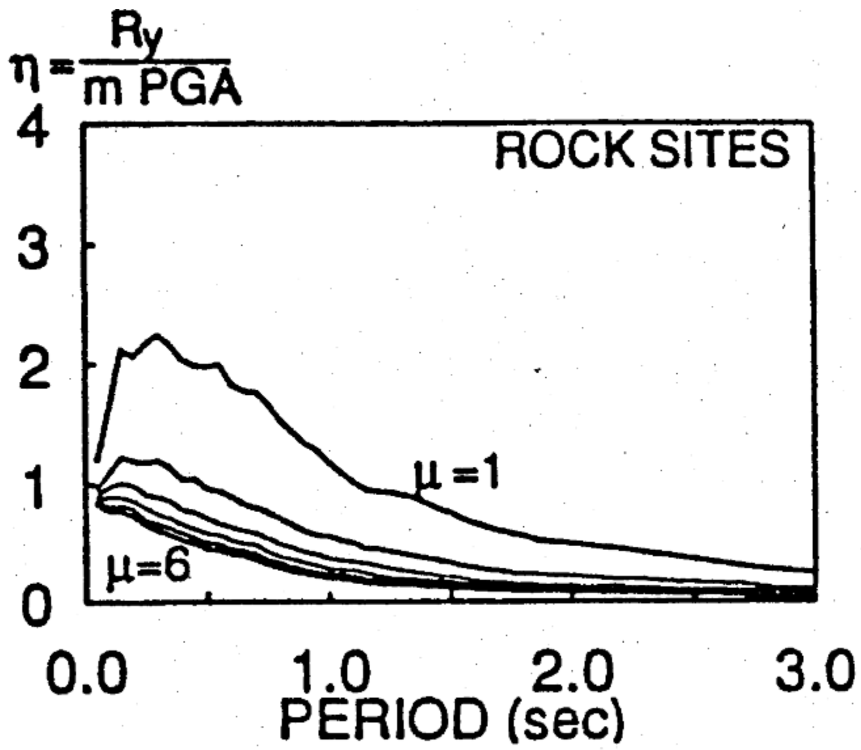


- Both yielding and damping reduce the pseudo-acceleration A_y and hence the peak value of the lateral force for which the system should be designed.
- But the damping is, in general, not as effective as yielding

INELASTIC DESIGN RESPONSE SPECTRA

Obtained from a comprehensive statistical study of yield strength demands on simple structure when subjected to many different ground motions recorded in various earthquakes.

Mean Normalized Strength Demand Spectra for $\boldsymbol{\mu = 1, 2, 3, 4, 5, 6}$



$R_y = the\ system's\ yield\ strength = f_y$

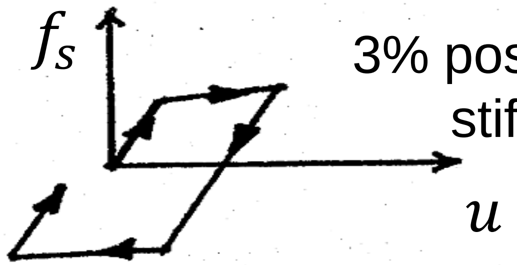
$m\ =\ the\ mass\ of\ the\ system$

$PGA = Peak\ ground\ accelaration$

The computed results are based on:

- *124 ground motions*
- *systems with bilinear hysteretic behavior.*

(Miranda, ASCE J.Struc. Eng., 119, No5, May, 1993)

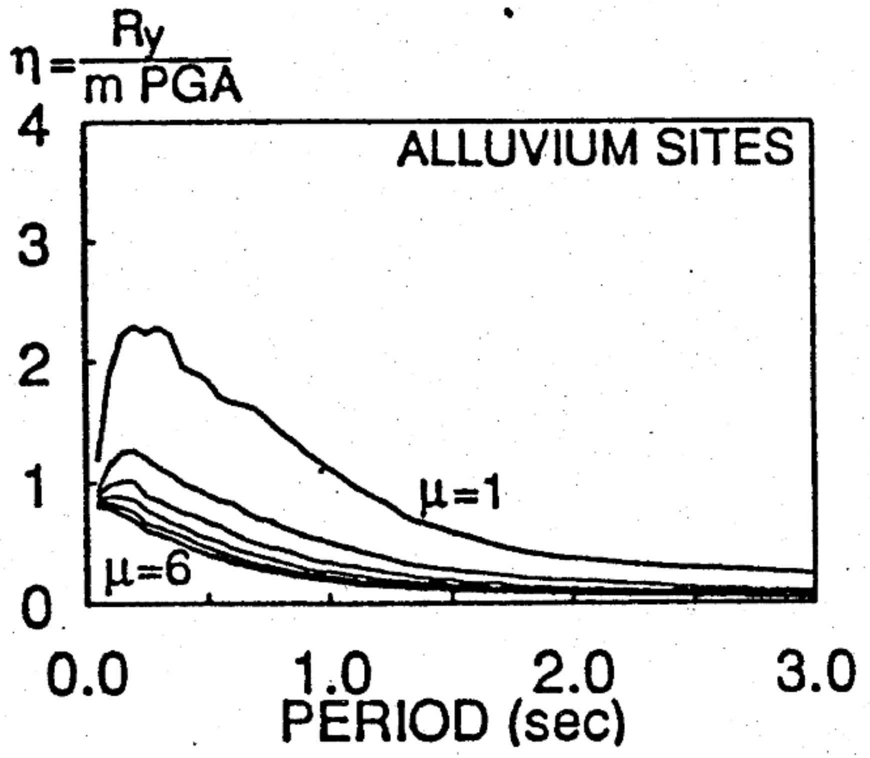


3% post-yielding stiffness

INELASTIC DESIGN RESPONSE SPECTRA

Obtained from a comprehensive statistical study of yield strength demands on simple structure when subjected to many different ground motions recorded in various earthquakes.

Mean Normalized Strength Demand Spectra for $\boldsymbol{\mu = 1, 2, 3, 4, 5, 6}$



$R_y = the\ system's\ yield\ strength = f_y$

$m\ =\ the\ mass\ of\ the\ system$

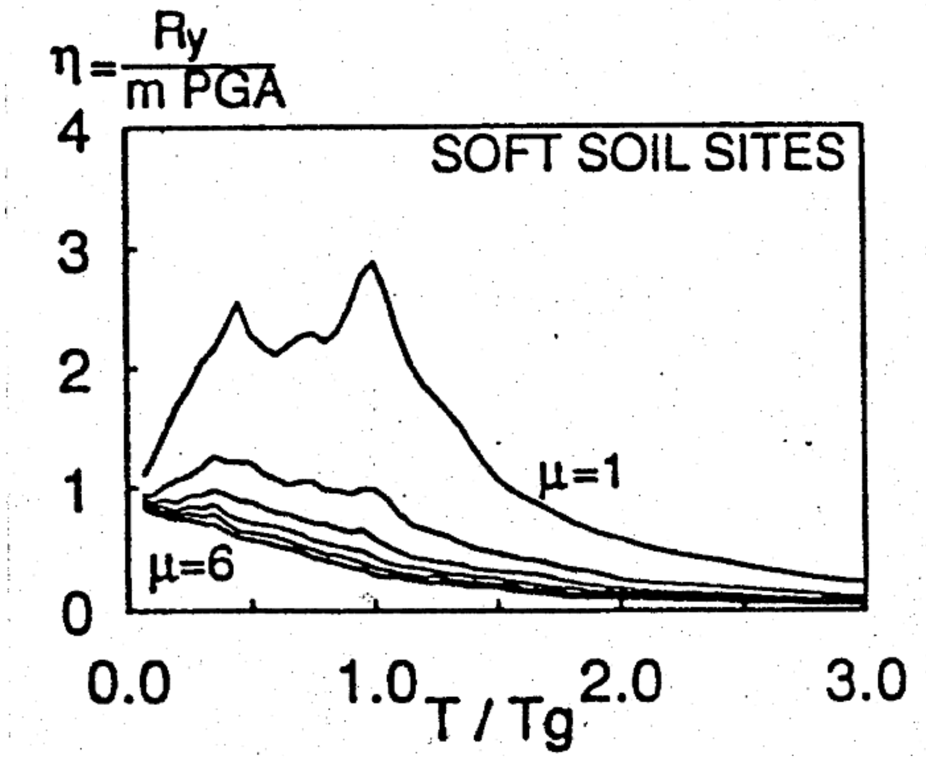
$PGA = Peak\ ground\ accelaration$

(Miranda, ASCE J.Struc. Eng., 119, No5, May, 1993)

INELASTIC DESIGN RESPONSE SPECTRA

Obtained from a comprehensive statistical study of yield strength demands on simple structure when subjected to many different ground motions recorded in various earthquakes.

Mean Normalized Strength Demand Spectra for $\boldsymbol{\mu = 1, 2, 3, 4, 5, 6}$



$R_y = \textit{the system's yield strength} = f_y$

$m = \textit{the mass of the system}$

$PGA = \textit{Peak ground accelaration}$

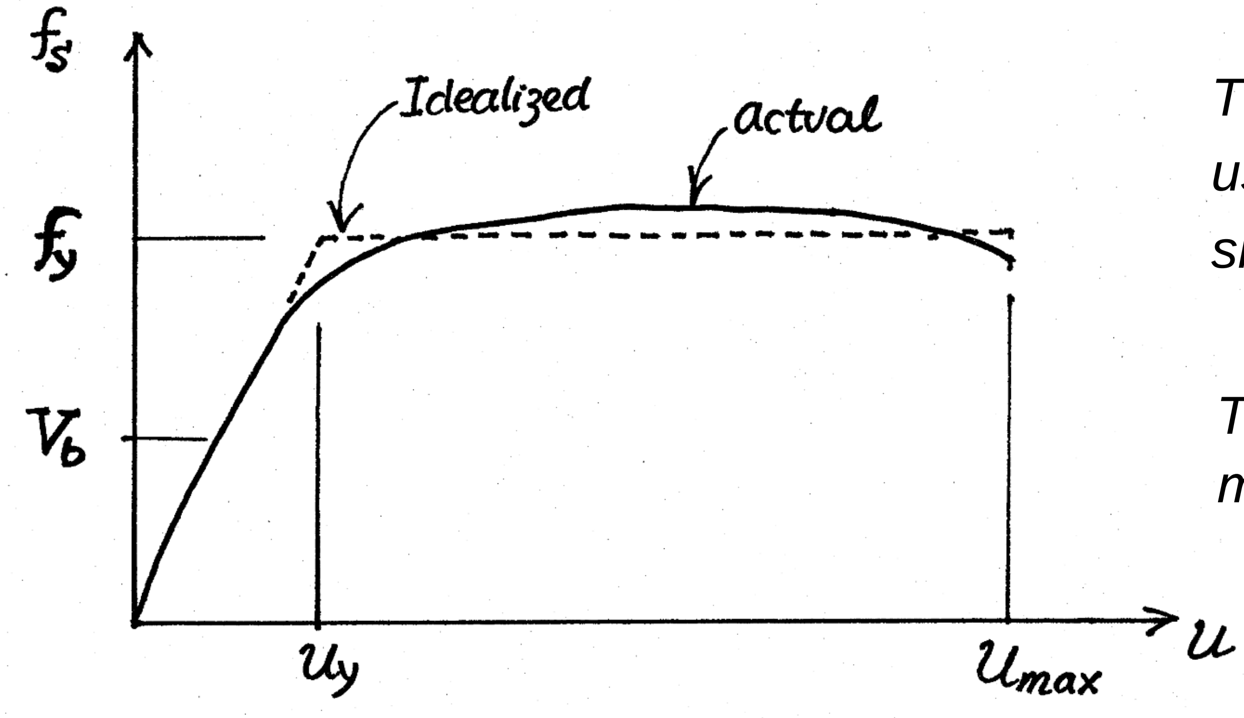
$T_g = \textit{the predominant period of the ground motion}$

(Miranda, ASCE J.Struc. Eng., 119, No5, May, 1993)

OVERSTRENGTH

Suppose that a structure is designed to resist a code-prescribed design shear V_b *by "Allowable Stress Design" method.*

The behavior of the structure under a monotonically increasing load would be:



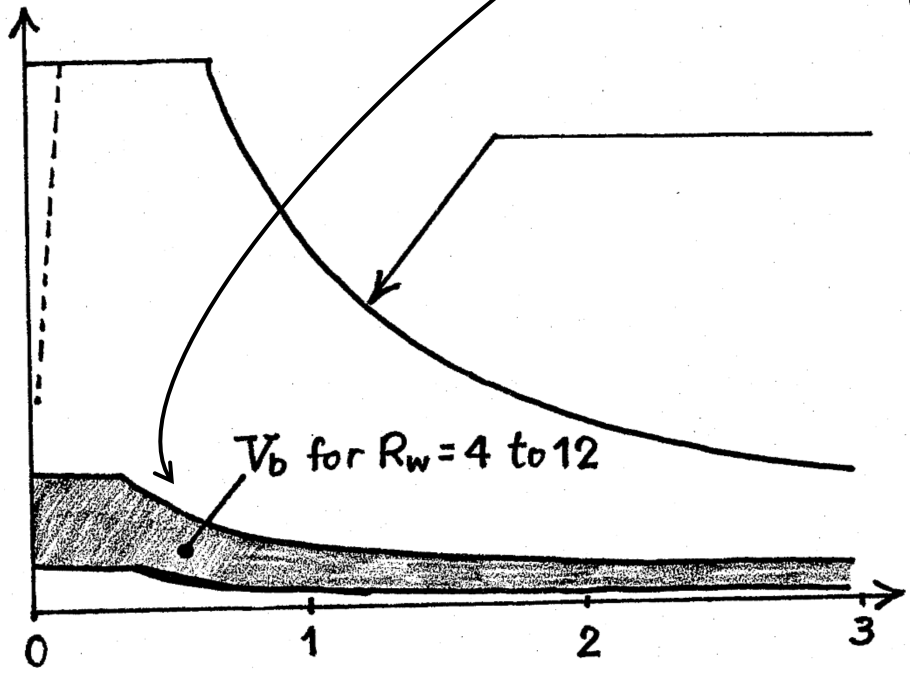
The yield strength of the structural system f_y *is usually considerably higher than the design base shear* V_b*, (the effect of "overstrength")*

Therefore, the design base shear V_b *can be set much lower than the "yield strength demand".*

UNIFORM BUILDING CODE (1994)

Base Shear

Design Base Shear



Natural period, sec

$$V_b = \frac{ZIC}{R_w}\,W$$

$ZICW =$ Elastic Strength Demand (f_0)

$R_w =$ Reduction Factor $= R_\Omega\, R_y$

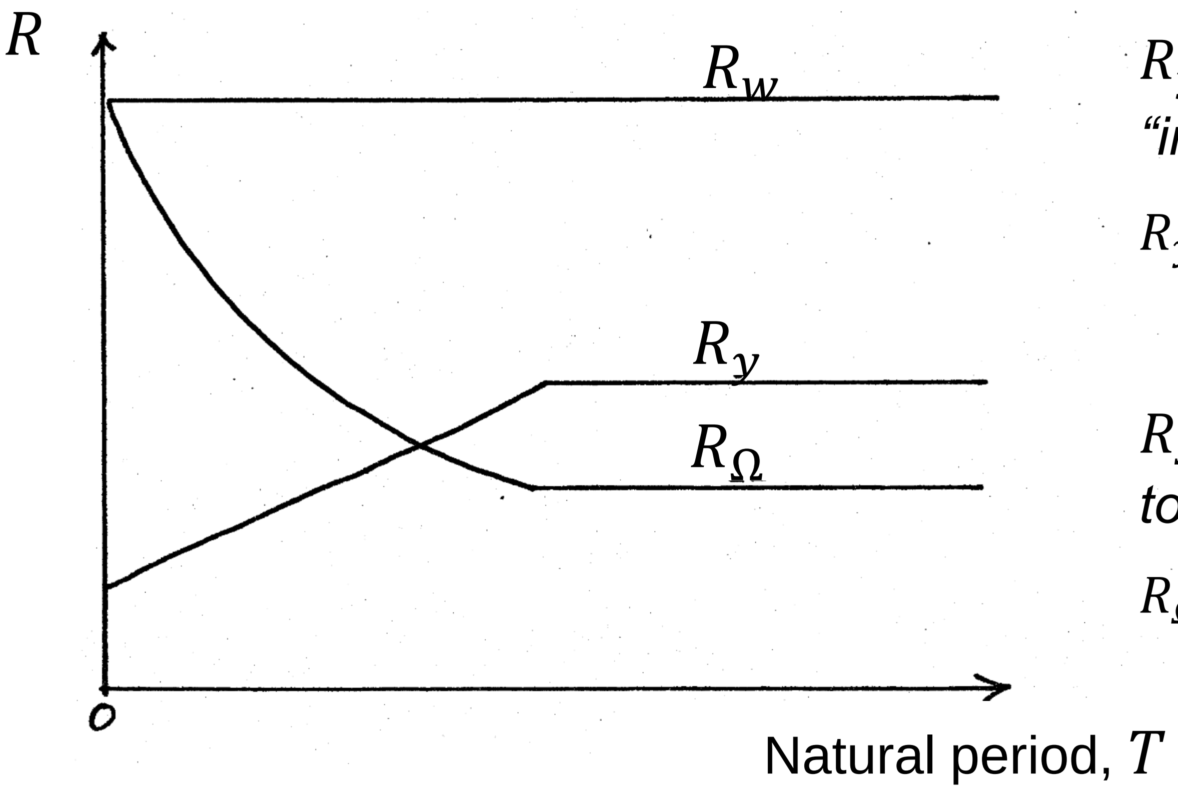
R_y *= factor for reducing "elastic strength demand" (f_0) to "inelastic yield strength demand" (f_y)*

R_y *is associated with the ductility capacity of the system.*

R_Ω *= factor for reducing "yield strength demand" (f_y) to "design base shear" (V_b).*

R_Ω *is associated with the overstrength of the system.*

REDUCTION FACTOR



R_y = factor for reducing "elastic strength demand" (f_0) to "inelastic yield strength demand" (f_y)

R_y is associated with the ductility capacity of the system.

R_Ω = factor for reducing "yield strength demand" (f_y) to "design base shear" (V_b).

R_Ω is associated with the overstrength of the system.

REDUCTION FACTOR R_w

1994 UNIFORM BUILDING CODE

TABLE 16N-STRUCTURAL SYSTEMS

BASIC STRUCTURAL SYSTEM[1]	LATERAL-FORCE-RESISTING SYSTEM—DESCRIPTION	R_w	H × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels		
	a. Wood structural panel walls for structures three stories or less	8	65
	b. All other light-framed walls	6	65
	2. Shear walls		
	a. Concrete	6	160
	b. Masonry	6	160
	3. Light steel-framed bearing walls with tension-only bracing	4	65
	4. Braced frames where bracing carries gravity loads		
	a. Steel	6	160
	b. Concrete[4]	4	—
	c. Heavy timber	4	65

$Note$: $\boldsymbol{H}$ $is\ height\ limit\ applicable\ to\ seismic\ zones\ 3\ and\ 4$

REDUCTION FACTOR R_w

TABLE 16N-STRUCTURAL SYSTEMS

		R_w	H
2. Building frame system	1. Steel eccentrically braced frame (EBF)	10	240
	2. Light-framed walls with shear panels		
	a. Wood structural panel walls for structures three stories or less	9	65
	b. All other light-framed walls	7	65
	3. Shear walls		
	a. Concrete	8	240
	b. Masonry	8	160
	4. Ordinary braced frames		
	a. Steel	8	160
	b. Concrete[4]	8	—
	c. Heavy timber	8	65
	5. Special concentrically braced frames		
	a. Steel	9	240
3. Moment-resisting frame system	1. Special moment-resisting frames (SMRF)		
	a. Steel	12	N.L.
	b. Concrete	12	N.L.
	2. Masonry moment-resisting wall frame	9	160
	3. Concrete intermediate moment-resisting frames (IMRF)[5]	8	—
	4. Ordinary moment-resisting frames (OMRF)		
	a. Steel[6]	6	160
	b. Concrete[7]	5	—

$Note: \boldsymbol{N.L.} = No\ limit$

REDUCTION FACTOR R_w

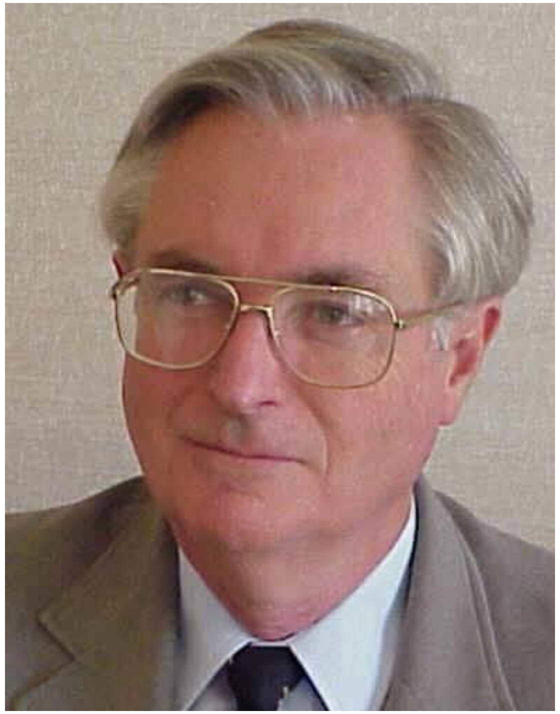
TABLE 16N-STRUCTURAL SYSTEMS

		R_w	H
4. Dual systems	1. Shear walls		
	a. Concrete with SMRF	12	N.L.
	b. Concrete with steel OMRF	6	160
	c. Concrete with concrete IMRF[5]	9	160
	d. Masonry with SMRF	8	160
	e. Masonry with steel OMRF	6	160
	f. Masonry with concrete IMRF[4]	7	—
	2. Steel EBF		
	a. With steel SMRF	12	N.L.
	b. With steel OMRF	6	160
	3. Ordinary braced frames		
	a. Steel with steel SMRF	10	N.L.
	b. Steel with steel OMRF	6	160
	c. Concrete with concrete SMRF[4]	9	—
	d. Concrete with concrete IMRF[4]	6	—
	4. Special concentrically braced frames		
	a. Steel with steel SMRF	11	N.L.
	b. Steel with steel OMRF	6	160
5. Undefined systems	See Sections 1627.8.3 and 1627.9.2	—	—

The Code-based Response Spectrum Analysis (RSA) Procedure

The Code-based Response Spectrum Analysis (RSA) Procedure

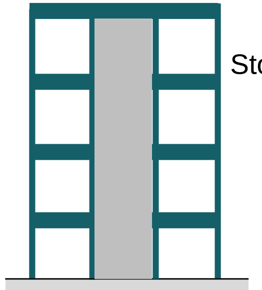
Ray W. Clough (1920 - 2016)



Edward L. Wilson (Born 1931)

The Standard RSA Procedure (ASCE 7-10, IBS 2012, EC 8)

Linear Elastic Model



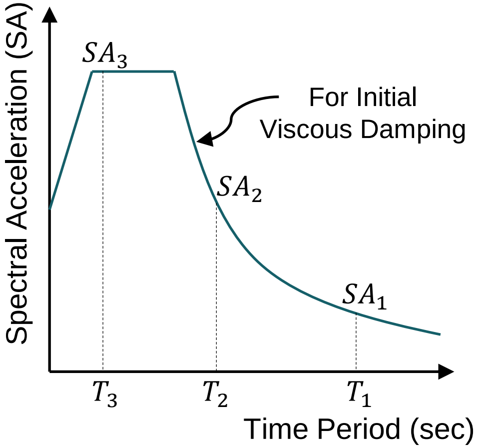
Eigen-value Analysis

$[\mathbb{K} - \omega^2 \mathbb{M}]\Phi = \mathbb{O}$

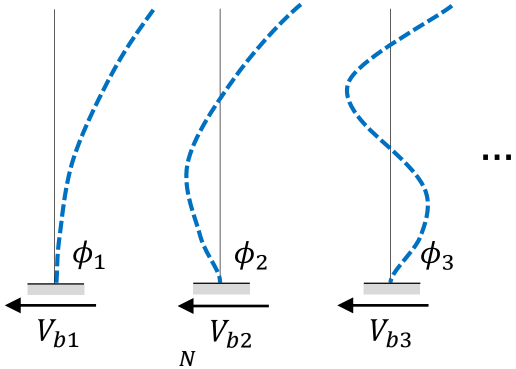
Determine Modal Properties

T_i, ϕ_i, Γ_i

Determine Spectral Acceleration for each Significant Mode



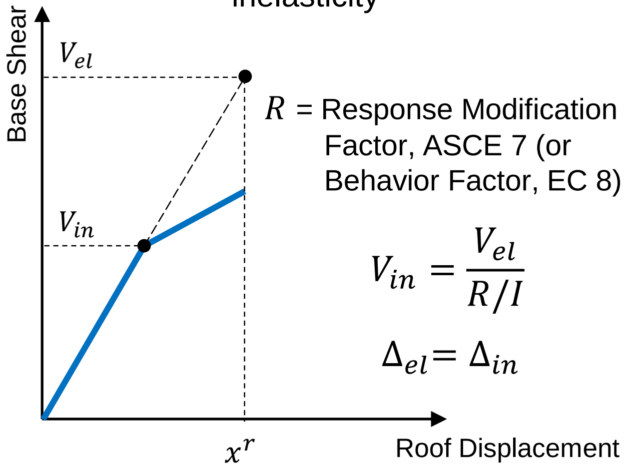
Determine Elastic Base Shears



$$V_{bn} = \sum_{i=1}^{N} \Gamma_n \, . m_i \, . \phi_{i,n} \, . SA_n$$

$$V_{el} = \sqrt{(V_{b1})^2 + (V_{b2})^2 + (V_{b3})^2 + \ \ldots}$$

Reduce Elastic Base Shear to account for inelasticity



R = Response Modification Factor, ASCE 7 (or Behavior Factor, EC 8)

$$V_{in} = \frac{V_{el}}{R/I}$$

$$\Delta_{el} = \Delta_{in}$$

$$V_{Design} = \frac{\sqrt{(V_{b1})^2 + (V_{b2})^2 + (V_{b3})^2 + \ \ldots}}{R/I}$$

$$M_{Design} = \frac{\sqrt{(M_{b1})^2 + (M_{b2})^2 + (M_{b3})^2 + \ \ldots}}{R}$$

$$\Delta = \frac{C_d\sqrt{(\Delta_{el1})^2 + (\Delta_{el2})^2 + (\Delta_{el3})^2 + \ \ldots}}{R/I}$$

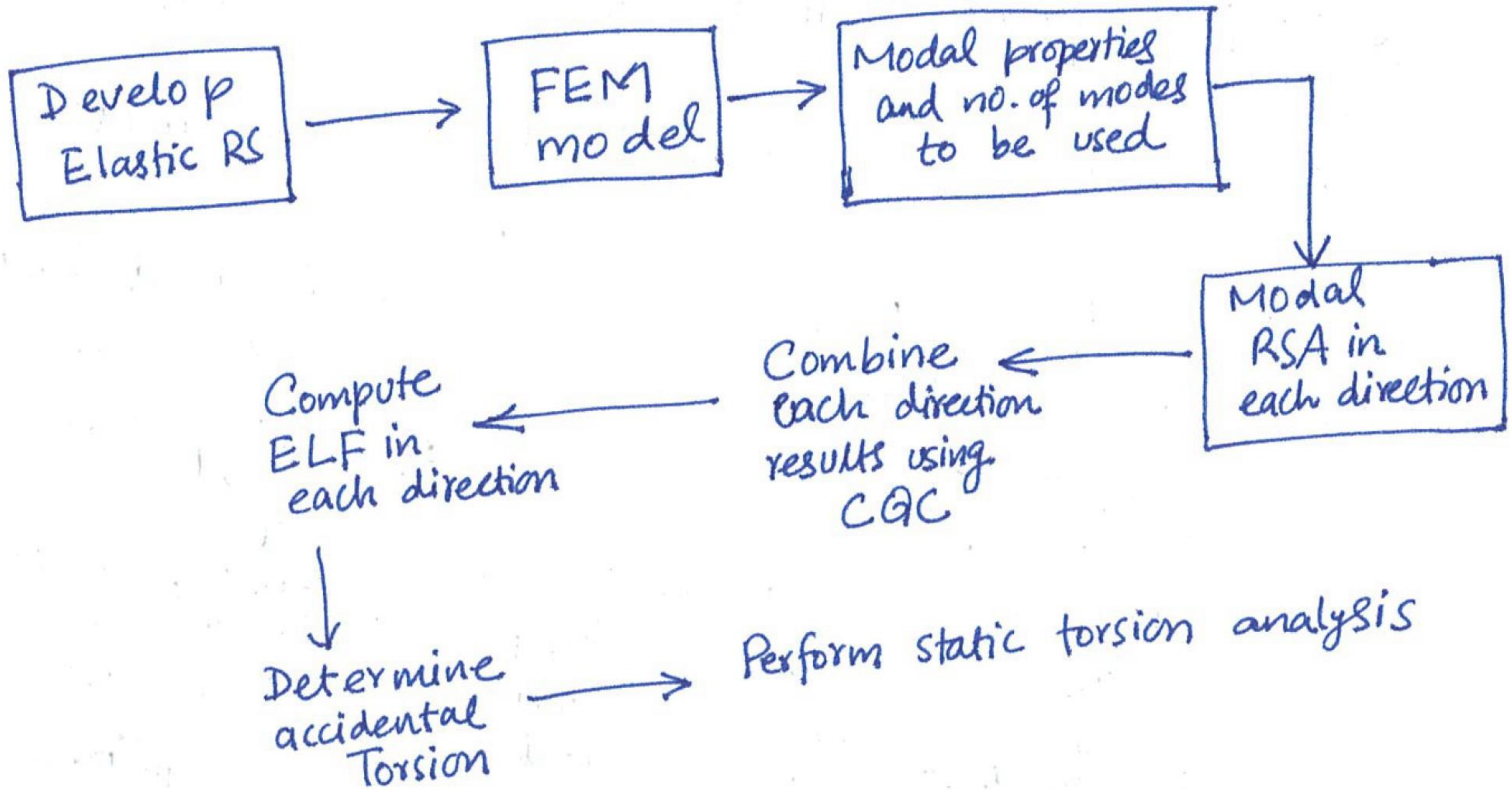
For members not desired to yield during a design earthquake $V_{Design} = \Omega \dfrac{V_{el}}{R}$

Ω = Structural Over-strength Factor

Maximum Displacement during a design earthquake $x^r_{max} = \dfrac{C_d x^r}{I}$

C_d = Displacement Amplification Factor

The Standard RSA Procedure (ASCE 7-10, IBS 2012, EC 8)



Modal Combination Rules

- ABSSUM Rule
  - Add the absolute maximum value from each mode. Not so popular and not used in practice

$$r_o \le \sum_{n=1}^{N} \left| r_{n0} \right|$$

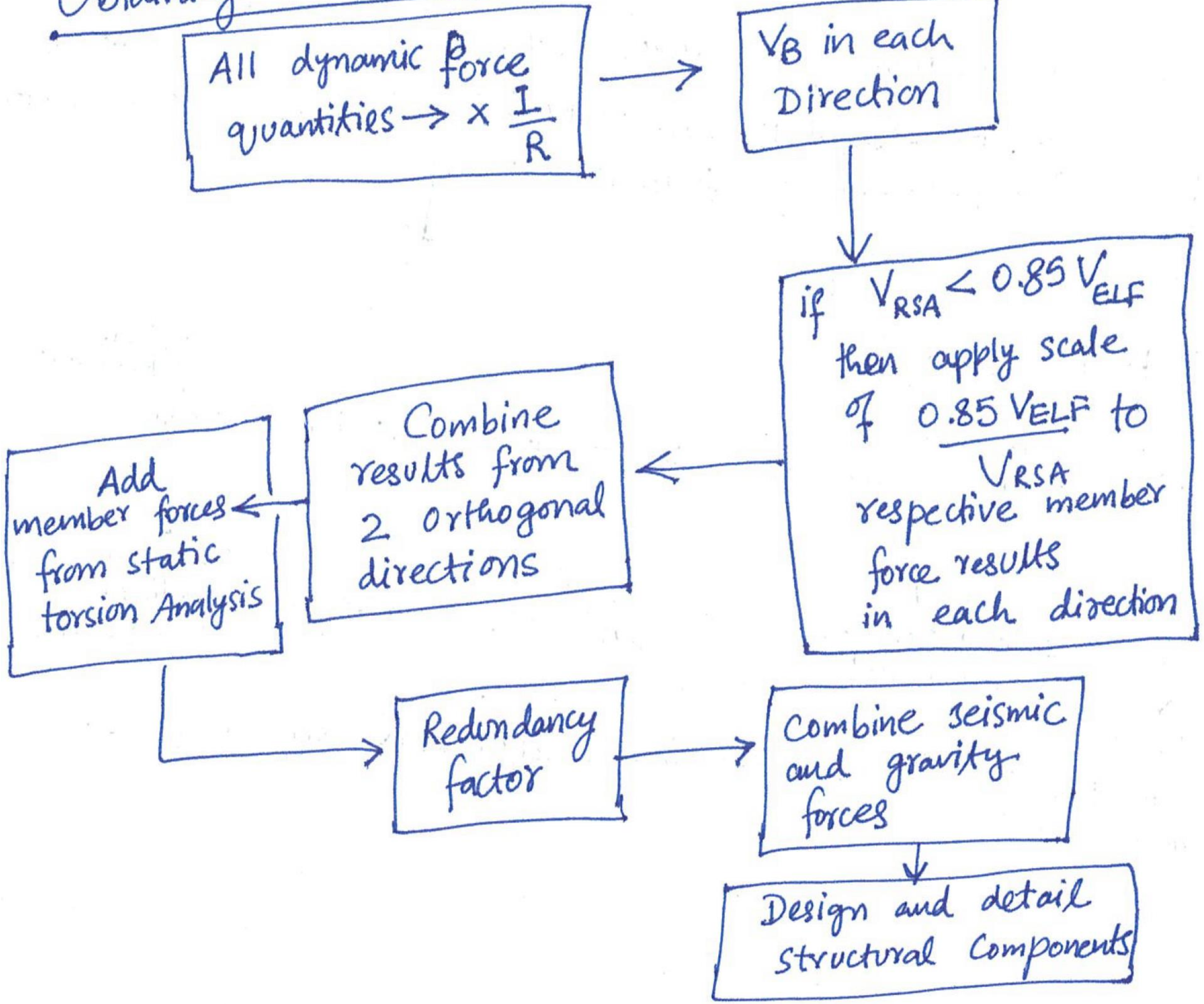
- SRSS
  - Square Root of Sum of Squares of the peak response from each mode. Suitable for well separated natural frequencies.

$$r_o \cong \sqrt{\sum_{n=1}^{N} r_{n0}^2}$$

- CQC
  - Complete Quadric Combination is applicable to large range of structural response and gives better results than SRSS.

$$r_o \cong \sqrt{\sum_{i=1}^{N} \sum_{n=1}^{N} \rho_{in} r_{i0} r_{n0}}$$

Obtaining Member Design Forces :-



The Input – Output Summary

- Input needed for Response Spectrum Analysis
  - Mass and stiffness distribution
  - A Specified Response Spectrum Curve
  - The Response Input Direction
  - The Response Scaling Factors
  - The number of modes to be included
- Output From Response Spectrum Analysis
  - Unsigned displacements, stress resultants and stresses etc.

The RSA Procedure in UBC 97

1631.5 Response Spectrum Analysis.

1631.5.1 Response spectrum representation and interpretation of results. The ground motion representation shall be in accordance with Section 1631.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 1631.5.4.

1631.5.2 Number of modes. The requirement of Section 1631.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

1631.5.3 Combining modes. The peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

1631.5.4 Reduction of Elastic Response Parameters for design. Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of *R*.

1. For all regular structures where the ground motion representation complies with Section 1631.2, Item 1, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with Section 1630.2.

2. For all regular structures where the ground motion representation complies with Section 1631.2, Item 2, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with Section 1630.2.

3. For all irregular structures, regardless of the ground motion representation, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 1630.2.

The corresponding reduced design seismic forces shall be used for design in accordance with Section 1612.

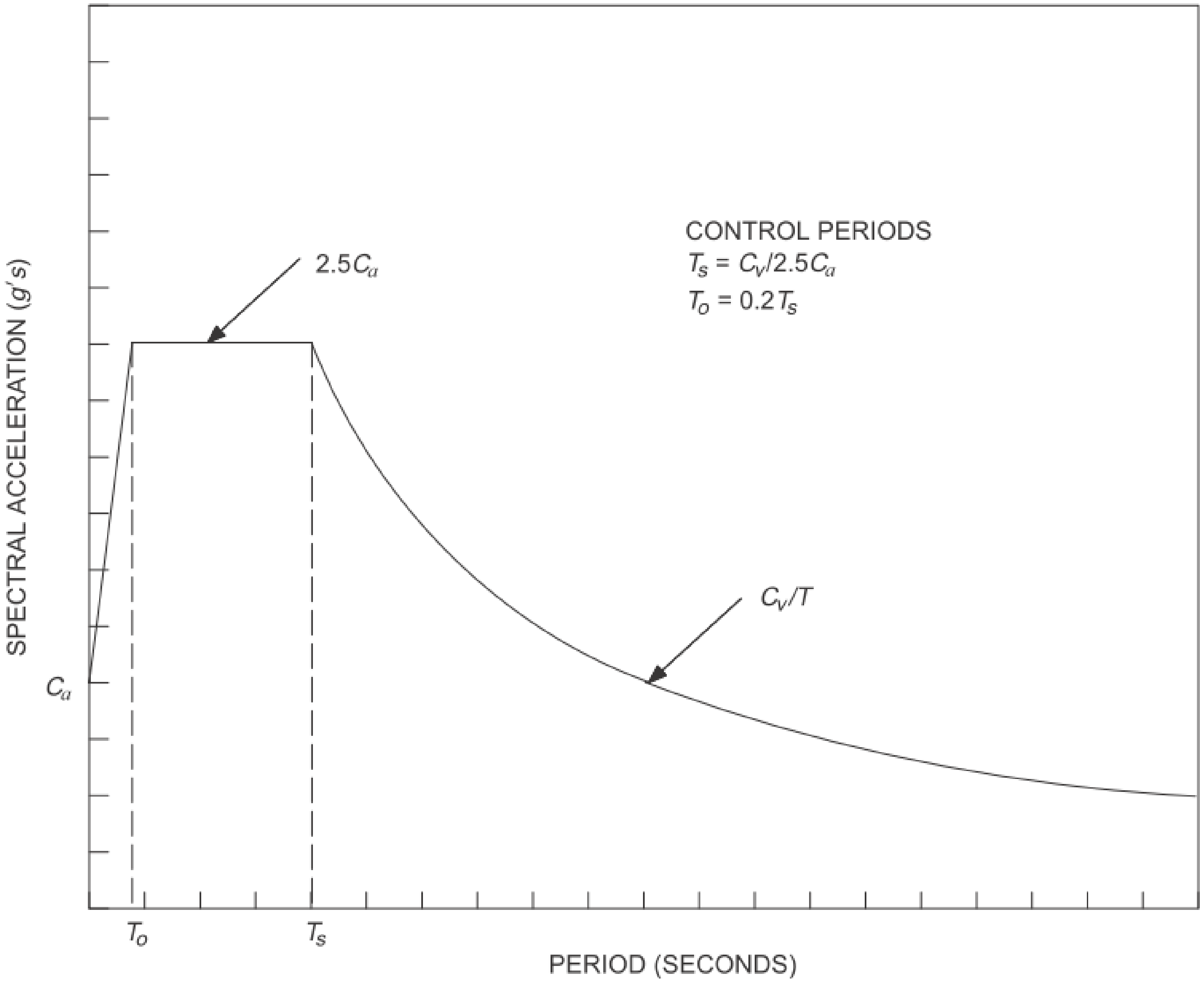


FIGURE 16-3—DESIGN RESPONSE SPECTRA

Dynamic Analysis Procedure in IBC 2000

SECTION 1618
DYNAMIC ANALYSIS PROCEDURE FOR THE SEISMIC DESIGN OF BUILDINGS

1618.1 Dynamic anaylsis procedures. The following dynamic analysis procedures performed in accordance with the requirements of this section may be used in lieu of equivalent lateral force procedure of Section 1617.4:

1. Modal Response Spectra Analysis.
2. Linear Time-history Analysis.
3. Nonlinear Time-history Analysis.

1618.1.1 Modeling. A mathematical model of the building shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models may be constructed to represent each system. For irregular structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the building. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model shall include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall include the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

1615.1.4 General procedure response spectrum. The general design response spectrum curve shall be developed as indicated in Figure 1615.1.4 and as follows:

1. For periods less than or equal to T_0, the design spectral response acceleration, S_a, shall be given by Equation 16-20.
2. For periods greater than or equal to T_0 and less than or equal to the T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.
3. For periods greater than T_S, the design spectral response acceleration, S_a, shall be given by Equation 16-21.

$$S_a = 0.6\frac{S_{DS}}{T_O}T + 0.4S_{DS} \qquad \textbf{(Equation 16-20)}$$

$$S_a = \frac{S_{D1}}{T} \qquad \textbf{(Equation 16-21)}$$

The RSA Procedure in IBC 2000

Design Spectral Values

- Adjust Maximum Considered Earthquake (MCE) values of Ss and $S1$ for local site effects

$$S_{MS} = F_a \times S_s$$
$$S_{M1} = F_v \times S_1$$

- Calculate the spectral design values

$$S_{DS} = 2/3 \times S_{MS}$$
$$S_{D1} = 2/3 \times S_{M1}$$

where:

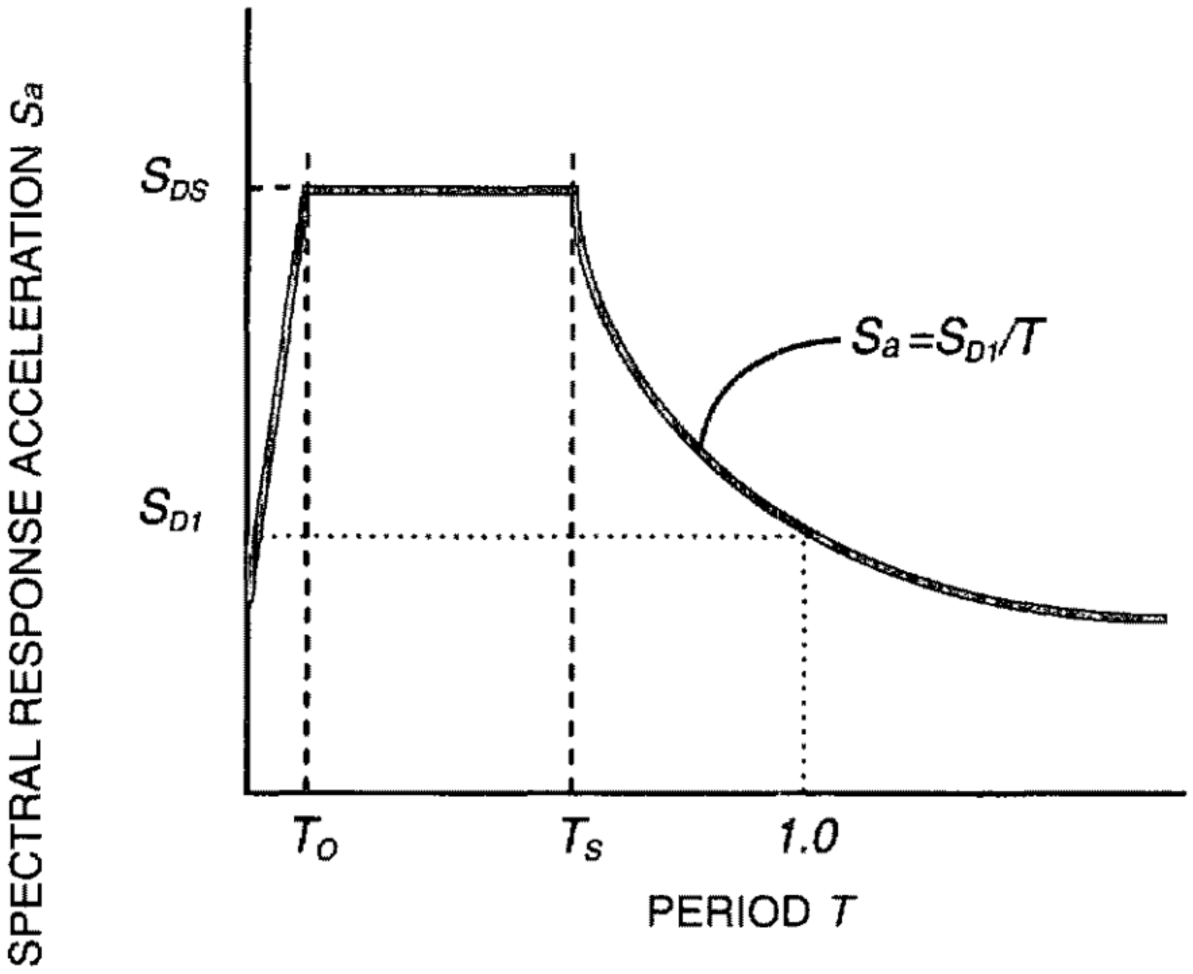
S_{DS} = The design spectral response acceleration at short periods as determined in Section 1615.1.3.

S_{D1} = The design spectral response acceleration at 1 second period as determined in Section 1615.1.3.

T = Fundamental period (in seconds) of the structure (Section 1617.4.2).

$T_O = 0.2\, S_{D1}/S_{DS}$

$T_S = S_{D1}/S_{DS}$


SPECTRAL RESPONSE ACCELERATION S_a
S_{DS}
S_{D1}
$S_a = S_{D1}/T$
T_O
T_S
1.0
PERIOD T

1618.2 Modes. An analysis shall be conducted to determine the natural modes of vibration for the building including the period of each mode, the modal shape vector, ϕ, the mass participation factor, and the modal mass. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual building mass in each of two orthogonal directions.

1618.3 Modal properties. The required periods, mode shapes, and participation factors of the building shall be calculated by established methods of structural analysis for the fixed base condition using the masses and elastic stiffnesses of the seismic-force-resisting system.

1618.4 Modal base shear. The portion of the base shear contributed by the m^{th} mode, V_m, shall be determined from the following equations:

$$V_m = C_{sm}\overline{W}_m \quad \textbf{(Equation 16-51)}$$

$$\overline{W}_m = \frac{\left(\sum_{i=1}^{n} w_i\phi_{im}\right)^2}{\sum_{i=1}^{n} w_i\phi_{im}^2} \quad \textbf{(Equation 16-52)}$$

where:

C_{sm} = The modal seismic response coefficient determined in Equation 16-53.

$\overline{W}_m$ = The effective modal gravity load.

w_i = The portion of the total gravity load, W, of the building at Level i, where W = the total dead load and other loads listed below:

1. In areas used for storage, a minimum of 25 percent of the reduced floor live load (floor live load in public garages and open parking structures need not be included).
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 pounds per square foot (0.479 kN/m²) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Twenty percent of flat roof snow load where the flat roof snow load exceeds 30 pounds per square foot (1.44 kN/m²).

ϕ_{im} = The displacement amplitude at the i^{th} level of the building when vibrating in its m^{th} mode.

The modal seismic response coefficient, C_{sm}, shall be determined by the following equation:

$$C_{sm} = \frac{S_{am}}{\left(\frac{R}{I_E}\right)} \qquad \textbf{(Equation 16-53)}$$

where:

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

S_{am} = The modal design spectral response acceleration at period T_m determined from either the general design response spectrum of Section 1615.1 or a site-specific response spectrum per Section 1615.2.

R = The response modification factor determined from Table 1617.6.

1618.5 Modal forces, deflections and drifts. The modal force, F_{xm}, at each level shall be determined by the following equations:

$$F_{XM} = C_{vxm} V_m \qquad \textbf{(Equation 16-55)}$$

$$C_{vxm} = \frac{w_x \phi_{xm}}{\sum_{i=1}^{n} w_i \phi_{im}} \qquad \textbf{(Equation 16-56)}$$

where:

C_{vxm} = The vertical distribution factor in the m^{th} mode.

V_m = The total design lateral force or shear at the base in the m^{th} mode.

w_i, w_x = The portion of the total gravity load of the building, W, located or assigned to Level i or x.

ϕ_{im} = The displacement amplitude at the i^{th} level of the building when vibrating in its m^{th} mode.

ϕ_{xm} = The displacement amplitude at the x^{th} level of the building when vibrating in its m^{th} mode.

The modal deflection at each level, δ_{xm}, shall be determined by the following equations:

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I_E} \qquad \textbf{(Equation 16-57)}$$

$$\delta_{xem} = \left(\frac{g}{4\pi^2}\right)\left(\frac{T_m^2 F_{xm}}{W_x}\right) \qquad \textbf{(Equation 16-58)}$$

where:

C_d = The deflection amplification factor determined from Table 1617.6.

F_{xm} = The portion of the seismic base shear in the m^{th} mode, induced at Level x.

g = The acceleration due to gravity (ft/s² or m/s²).

I_E = The occupancy importance factor determined in accordance with Section 1616.2.

T_m = The modal period of vibration, in seconds, of the m^{th} mode of the building.

w_x = The portion of the total gravity load of the building, W, located or assigned to Level x.

δ_{xem} = The deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis.

1618.7 Design values. The design value for the modal base shear, V_t; each of the story shear, moment and drift quantities; and the deflection at each level shall be determined by combining their modal values as obtained from Sections 1618.5 and 1618.6. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or by the complete quadratic combination (CQC) technique.

The base shear, V, using the equivalent lateral force procedure in Section 1617.4 shall be calculated using a fundamental period of the building, T, in seconds, of 1.2 times the coefficient for upper limit on the calculated period, C_u, taken from Table 1617.4.2, times the approximate fundamental period of the building, T_a, calculated in accordance with Section 1617.4.2.1. Where the thus calculated base shear, V, is greater than the modal base shear, V_t, the design story shears, moments, drifts and floor deflections shall be multiplied by C_m, the modification factor:

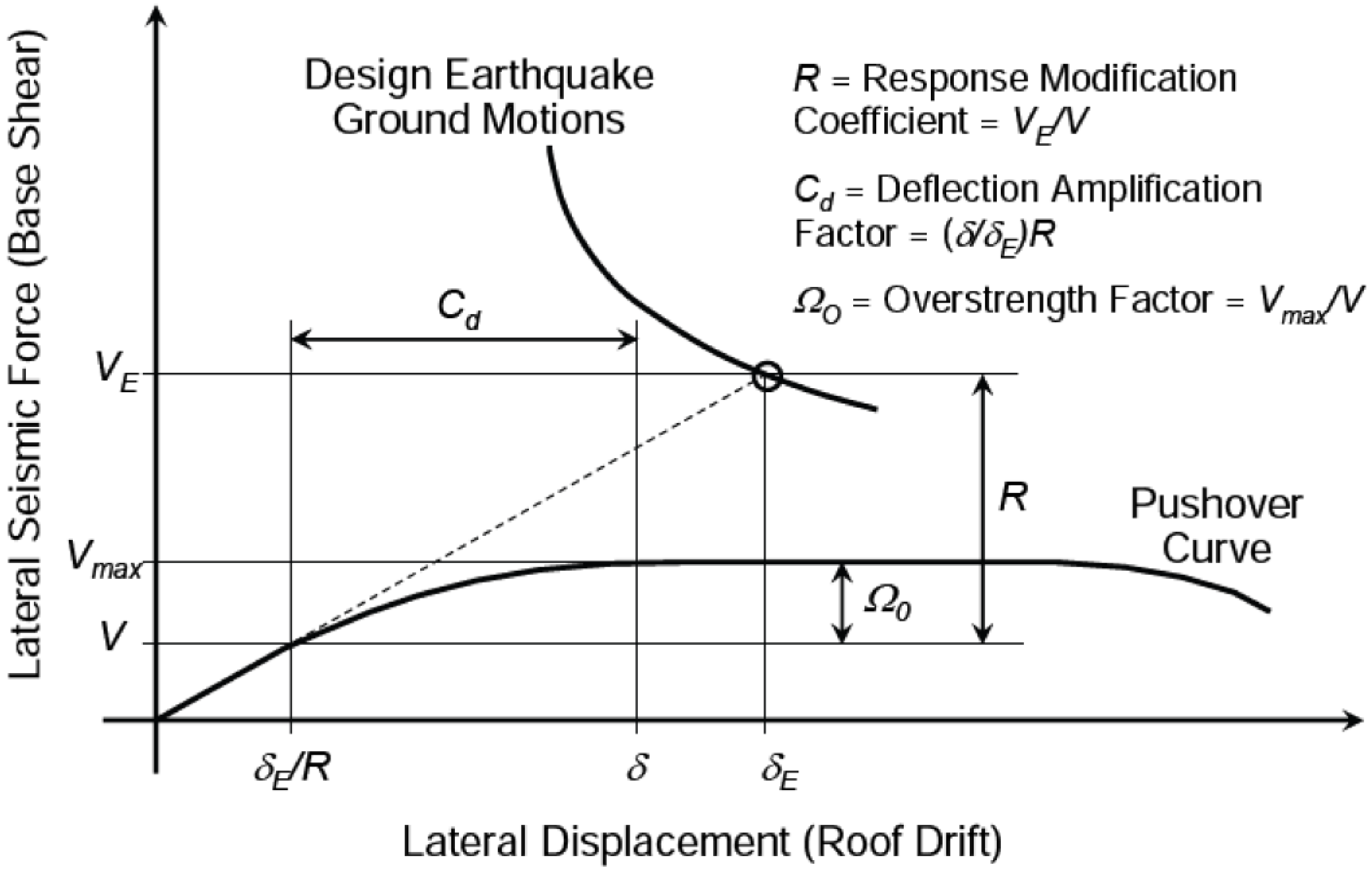
$$C_m = \frac{V}{V_t} \qquad \textbf{(Equation 16-59)}$$

where

V = The equivalent lateral force procedure base shear, calculated in accordance with this section and Section 1617.4.

V_t = The modal base shear, calculated in accordance with this section.

The modal base shear, V_t, need not exceed the base shear calculated from the equivalent lateral force procedure in Section 1617.4. However, for buildings with a value of the design spectral response acceleration at 1 second period, S_{D1}, of 0.2 or greater, as determined in Section 1615.1.3, with a period T, as determined in Section 1617.4.2, of 0.7 second or greater, and located on Site Class E or F sites (Section 1615.1.1), the design base shear shall not be less than that determined using the equivalent lateral force procedure in Section 1617.4.


Design Earthquake
Ground Motions
R = Response Modification
Coefficient = V_E/V
C_d = Deflection Amplification
Factor = $(\delta/\delta_E)R$
Ω_O = Overstrength Factor = V_{max}/V
C_d
V_E
R
Pushover
Curve
V_{max}
Ω_0
V
δ_E/R
δ
δ_E
Lateral Displacement (Roof Drift)
Lateral Seismic Force (Base Shear)

Routine Design Office Practice

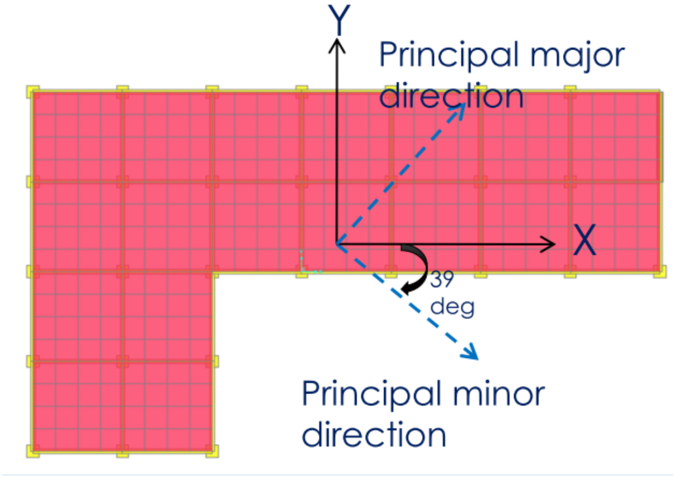
- Run equivalent static analysis according to code
- Find principal directions from modal base shear
- Run response spectrum analysis along principal directions with scale factor of 9.81 (m/s^2)
- Find the scale factor for scaling elastic response spectrum results to equivalent static base shear
- Run response spectrum analysis with calculated scale factor in principal directions

Scaling the Results

- Reduce the elastic response for design purpose, but design base shear is not less than elastic base shear divided by R.
- Design base shear shall not be less than 85% of static lateral force base shear according to ASCE 7-05.
- In ASCE 7-16, base shear shall be scaled to 100% of static lateral force base shear.

Principal Directions

- Lack of definitions of the principal directions in code
- The direction of the base reaction of the mode shape associated with the fundamental frequency of the system is used to define the principal direction.
  - Run modal analysis
  - Extract base shear of mode 1
  - Find angle between X and Y components of base shear
  - Another direction is 90 degrees apart



Fx = -1,041 kN
Fy = 846 kN
Angle = tan-1 (Fy/Fx)
= -39 deg.

Courtesy: Mr. Aung (AIT Solutions)

Directional and Orthogonal Effects

- Seismic forces act in both principal directions of the building simultaneously
- But seismic effects in two directions are unlikely to reach their maxima simultaneously
- 100% of seismic forces in one principal direction combined with 30% of seismic forces in the orthogonal direction

Accidental Torsion

- Arise from several factors
  - Rotational components of ground motions
  - Effects of non-structural elements
  - Actual distribution of dead and live loads
  - Uncertainties in defining building's material properties for dynamic analysis
- Generally 5% of eccentricity from center of mass is considered

- Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists, accidental torsional moment needs to be amplified.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = the maximum displacement at Level x (in. or mm) computed assuming $A_x = 1$

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

EXCEPTION: The accidental torsional moment need not be amplified for structures of light-frame construction.

The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

Table 12.3-1 Horizontal Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.3.4.2 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	E and F D D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 16.3.4	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel System Irregularity:** Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.	12.5.3 12.7.3 Table 12.6-1 16.3.4	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Seismic Design as per

ASCE 7-10

All members should be designed for these effects.

Seismic Load Effect (E)
(for load Combinations)

$$E = E_h \pm E_v$$

$E_h = \rho Q_E$

ρ → redundancy factor (1 or 1.3)

Q_E → effects from $\frac{V_e}{R}$

$E_v = 0.2 S_{DS} D$

D → effect of dead load

So combinations for strength design,

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

$$(0.9 - 0.2 S_{DS}) D + \rho Q_E + 1.6H$$

Seismic load Effect Including Ω_o

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

E_{mh} → effect of seismic forces including Ω

$E_{mh} = \Omega_o Q_E$

Q_E → effects from $\frac{V_e}{R}$

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

$$(0.9 - 0.2 S_{DS}) D + \Omega_o Q + 1.6H$$

The RSA Procedure (ASCE 7-10)

Table 1.5–1 → Risk Catagory of Buildings
I, II, III, IV

Table 1.5-2 → Importance Factors of Risk category of buildings. $I_e = 1$ (for I and II)
$= 1.25$ for III
$= 1.5$ for IV.

SDC → based on S_{DS} or S_{D1} and Risk Category
A, B, C, D, E, F

E → $S_1 > 0.75g$ for I, II, III

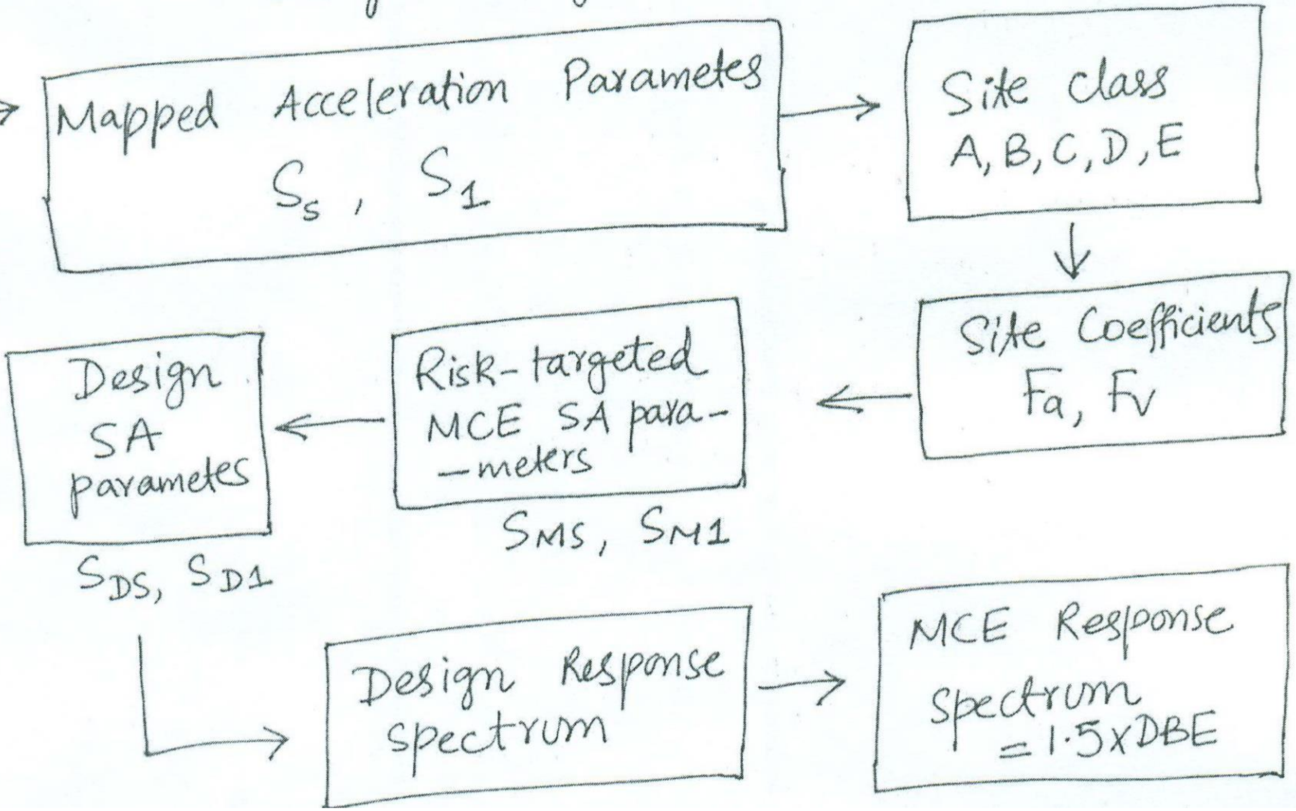
F → $S_1 > 0.75g$ for IV

→ Seismic Design Catagory (SDC) → assigned to a structure based on its risk category, and the severity of design EQ motio.

→ Nominal strength → without any reduction factor.

Design strength = Nominal strength × ϕ

→



The RSA Procedure (ASCE 7-10)

→ For SDC D, E, F → PGA shall be determined based on (a) A site-specific study taking into account soil amplification effects or (b) $PGA_M = F_{PGA}\, PGA$

PGA_M: adjusted for site class effects

F_{PGA}: site coefficient Table 11.8-1

→ R, Ω_o and C_d (Table 12.2-1)

R: base shear

Ω_o: element design forces

C_d: design story drift

→ Combining factored loads using strength design.

Basic combinations

1) $1.4D$
2) $1.2D + 1.6L + 0.5(L_r, S \text{ or } R)$
3) $1.2D + 1.6(L_r, S \text{ or } R) + (L \text{ or } 0.5W)$
4) $1.2D + 1.0W + L + 0.5(L_r, S \text{ or } R)$
5) $1.2D + 1.0E + L + 0.2S$
6) $0.9D + 1.0W$
7) $0.9D + 1.0E$

The RSA Procedure (ASCE 7-10)

→ Redundancy → Sometimes synonym of "Alternative loading path". The ability of structure to redistribute among its members/connections the loads which can no longer be carried by some other damaged portions. Non-redundant structures → fail immediately under local damage.

A "ρ" redundancy factor is assigned to seismic force resisting system.

$\rho = 1$ (SDC B, C and .....)

$\rho = 1.3$ (SDC D, E, F except ....)

The RSA Procedure (ASCE 7-10)

→ The "E" in load combination 5 is

$$E = E_h + E_V$$

For load combination 7, $E = E_h - E_V$

$E_h = \rho Q_E$ → from V (application of horizontal forces simultaneously in 2 directions). at right angles.

$E_V = 0.2 S_{DS} D$ └ dead load

↓

(Some exceptions)

→ So basic combinations for strength design

5) $(1.2 + 0.2 S_{DS}) D + \rho Q_E + L + 0.2S$

6) $(0.9 \pm 0.2 S_{DS}) D + \rho Q_E + 1.6H$ └ lateral earth pressure

→ Where specifically required, conditions requiring overstrength factor →

For Load Combination 5) $E_m = E_{mh} + E_V$

6) $E_m = E_{mh} - E_V$

$$E_{mh} = \Omega_0 Q_E$$

↳ need not exceed the max force that can develop in the element as determined by a rational, plastic mechanism analysis or NL analysis.

So

$$(1.2 + 0.2 S_{DS}) D + \Omega_0 Q_E + L + 0.2S$$

$$(1.2 - 0.2 S_{DS}) D + \Omega_0 Q_E + 1.6H$$

→ Direction of loading :-

SDC B → permitted to be applied independently in each of the two orthogonal directions (Interaction effects neglected).

The RSA Procedure (ASCE 7-10)

SDC C → Minimum as SDC B.
If irregularity type 5 — Orthogonal combination procedure
— Simultaneous application of orthogonal ground motions.

SDC D, E, F → Minimum as SDC
+ .....

→ Table 12.6-1 for "Permitted Analytical Procedures"

Structural Characteristics	ELF	RSA	RHA
⋮	P	P	P
⋮	⋮	⋮	⋮
⋮	NP	P	P

→ Modeling Criteria
- for determining seismic loads → fixed base permitted
- Effective seismic weight = D + 0.25 L + Partitions + Operating + Snow + Landscaping
- Spatial distribution of mass and stiffness
- Concrete → cracked sections
- Steel frame → the contribution of panel zone deformations to overall story drift shall be included.
- For 3D, a minimum of 3 dynamic DOF (2 Trans 1 rot) shall be included at each level.

Strength Design Load Combinations (ASCE 7-16)

Gravity Load Combinations

1) $1.4DL$

2) $1.2DL + 1.6LL$

Seismic Load Combinations

1) $1.2DL + 1.0EQX + 0.3EQY + 0.5LL$

2) $1.2DL + 0.3EQX + 1.0EQY + 0.5LL$

3) $0.9DL + 1.0EQX + 0.3EQY$

4) $0.9DL + 0.3EQX + 1.0EQY$

$EQ = \rho\, E_h \pm E_v$

$E_v = 0.2\, S_{DS}\, DL$

e.g. If $S_{DS} = 0.7$ and $\rho = 1.3$,

$E_v = 0.2 \times 0.7 \times DL = 0.14\, DL$

1) $(1.2 + 0.14)DL + 1.3EQX + 0.39EQY + 0.5LL$

or $1.34DL + 1.3EQX + 0.39EQY + 0.5LL$

2) $1.34DL + 0.39EQX + 1.3EQY + 0.5LL$

3) $(0.9 - 0.14)DL + 1.3EQX + 0.39EQY$

or $0.76DL + 1.3EQX + 0.39EQY$

4) $0.76DL + 0.39EQX + 1.3EQY$

The Response Spectrum Analysis (RSA) Procedure

- The RSA Procedure in IBC 2003
  - ***Check yourself***
- The RSA Procedure in ASCE 7-05
  - ***Check yourself***
- The RSA Procedure in IBC 2006
  - ***Check yourself***
- The RSA Procedure in IBC 2009
  - ***Check yourself***
- The RSA Procedure in ASCE 7-10
  - ***Check yourself***
- The RSA Procedure in IBC 2012
  - ***Check yourself***
- The RSA Procedure in ASCE 7-16
  - ***Check yourself***

…

- The RSA Procedure in EC 2003/2008
  - ***Check yourself***
- The RSA Procedure in AS Codes
  - ***Check yourself***
- The RSA Procedure in BS 8110
  - ***Check yourself***
- The RSA Procedure in CSA Codes
  - ***Check yourself***
- The RSA Procedure in IS Codes
  - ***Check yourself***
- The RSA Procedure in MNBC
  - ***Check yourself***
- The RSA Procedure in NZS
  - ***Check yourself***

…

The Linear Time History Analysis (LTHA) Procedure

The Linear Time History Analysis (LTHA) Procedure

- Direct Integration (DI) Analysis Procedure
- LDP, LTHA, LRHA
- NDP, NDA, NLTHA, NLRHA

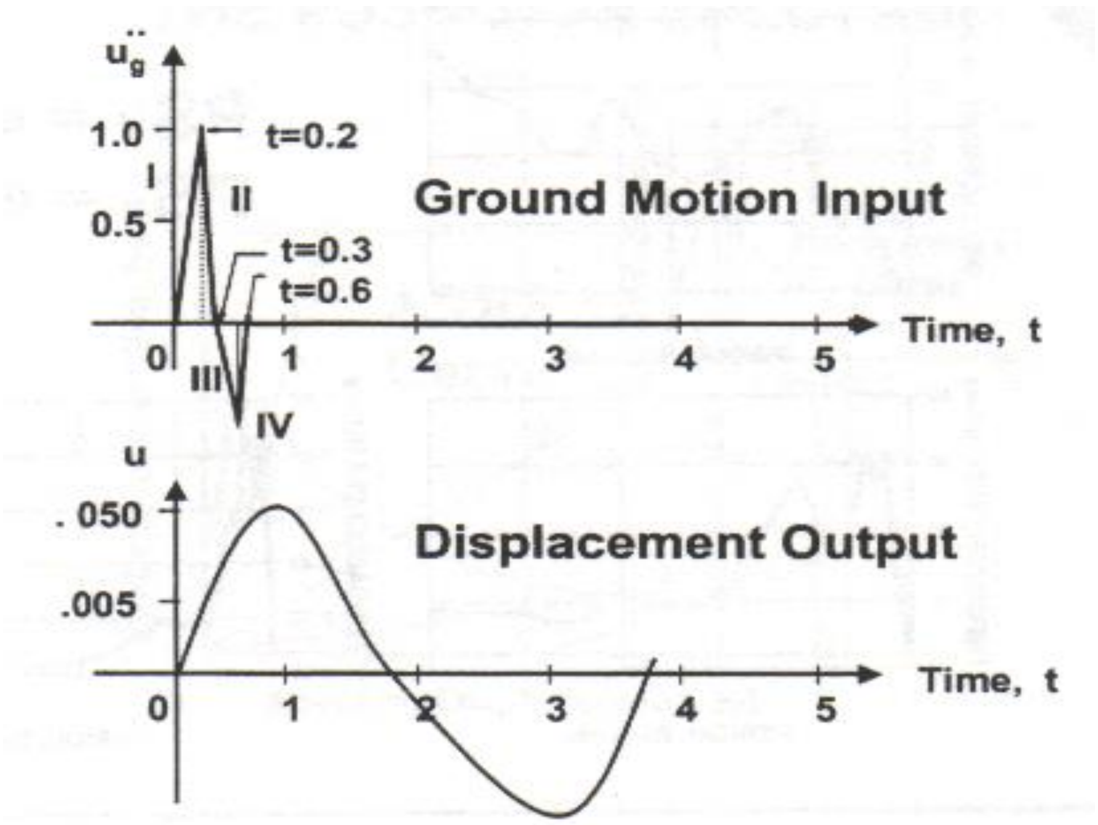
$$M\ddot{u} + C\dot{u} + Ku = -M1\ddot{u}_g$$

- The full dynamic equilibrium equation is solved for each time step on the acceleration-time curve.
- The history of the deformations resulting from previous time step calculation is considered in computing the response for the current time step.
- The time-history analysis is in-fact a piece-wise solution of the entire force histogram.

Ground Motion

$$\ddot{u} + 2\xi\omega\dot{u} + \omega^2 u = -\ddot{u}_g$$

- The input Variables are ground acceleration, damping ratio and circular frequency
- The final unknown is displacement (and its derivatives)



Input-Output for Time History Analysis

- Input
  - Mass and stiffness distribution
  - The acceleration-time record
  - The scaling factors
  - Directional factors
  - Analysis Time step etc.
- Output
  - Displacements, stress resultants and stresses are each time step
  - The envelop values of response

Time History Analysis in UBC 97

1631.6 Time-history Analysis.

1631.6.1 Time history. Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time-history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground-motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from $0.2T$ second to $1.5T$ seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

Time History Analysis in UBC 97

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

1631.6.2 Elastic time-history analysis. Elastic time history shall conform to Sections 1631.1, 1631.2, 1631.3, 1631.5.2, 1631.5.4, 1631.5.5, 1631.5.6, 1631.5.7 and 1631.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 1631.5.4.

1631.6.3 Nonlinear time-history analysis.

1631.6.3.1 Nonlinear time history. Nonlinear time-history analysis shall meet the requirements of Section 1629.10, and time histories shall be developed and results determined in accordance with the requirements of Section 1631.6.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with Section 1630.10.

1631.6.3.2 Design review. When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral-force-resisting system shall be performed by an independent engineering team, including persons licensed in the appropriate disciplines and experienced in seismic analysis methods. The lateral-force-resisting system design review shall include, but not be limited to, the following:

1. Reviewing the development of site-specific spectra and ground-motion time histories.

2. Reviewing the preliminary design of the lateral-force-resisting system.

3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

The engineer of record shall submit with the plans and calculations a statement by all members of the engineering team doing the review stating that the above review has been performed.

Dynamic Analysis Procedure in IBC 2000

SECTION 1618
DYNAMIC ANALYSIS PROCEDURE FOR THE SEISMIC DESIGN OF BUILDINGS

1618.1 Dynamic anaylsis procedures. The following dynamic analysis procedures performed in accordance with the requirements of this section may be used in lieu of equivalent lateral force procedure of Section 1617.4:

1. Modal Response Spectra Analysis.
2. Linear Time-history Analysis.
3. Nonlinear Time-history Analysis.

1618.1.1 Modeling. A mathematical model of the building shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models may be constructed to represent each system. For irregular structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the building. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model shall include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. In addition, the model shall comply with the following:

1. Stiffness properties of concrete and masonry elements shall include the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

1618.10.1 Time history. Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time-history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distance and source mechanisms that are consistent with those that control the maximum considered earthquake. Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs shall be used to make up the total number required. For each pair of horizontal ground-motion components, the square root of the sum of the squares (SRSS) of the 5 percent damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra is not less than 1.4 times the 5 percent damped spectrum of two-thirds the maximum considered earthquake for periods from 0.2 T second to 1.5 T seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

1618.10.2 Elastic time-history analysis. Elastic time-history analysis shall conform to Sections 1616.6, 1618.1, 1618.9 and the base shear scaled in accordance with Section 1618.7. Strength design shall be used to determine member capacities.

1618.10.3 Nonlinear time-history analysis.

1618.10.3.1 Nonlinear time history. Time histories shall be developed and results determined in accordance with the requirements of Section 1618.10.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the importance factor. The responses shall not be reduced by R/I_E. The maximum inelastic response displacement shall comply with Table 1617.3. Strength design shall be used to determine member capacities.

1618.10.3.2 Design review. When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral-force-resisting system shall be performed by an independent team of registered design professionals in the appropriate disciplines and others experienced in seismic analysis methods. The seismic-force-resisting system design review shall include, but not be limited to, the following:

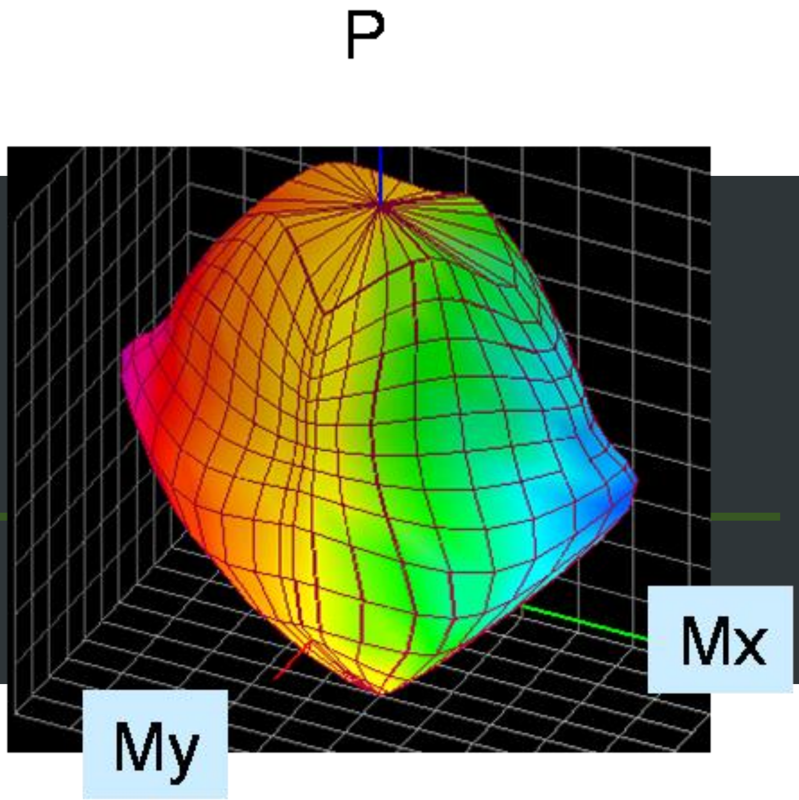
Nonlinear Time History Analysis Procedure in IBC 2000

1. Reviewing the development of site-specific spectra and ground-motion time histories.
2. Reviewing the preliminary design of the lateral-force-resisting system.
3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

Selection and Modification of Ground Motions for the LRHA

Will be discussed in-detail in Lecture 6

Combining Responses for Member Design



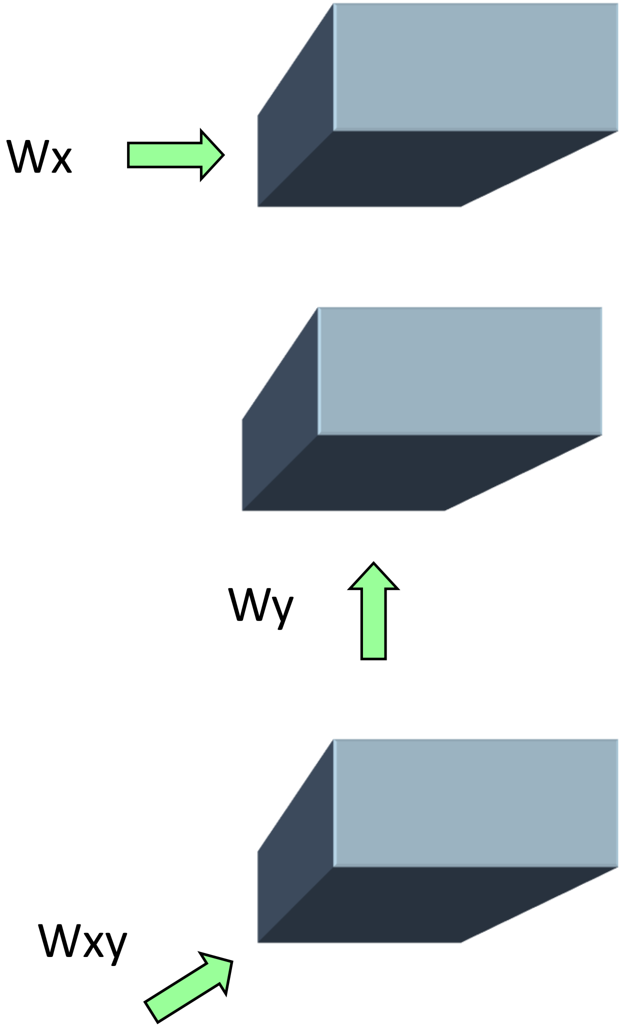
Applying Wind Loads

- At least 3 basic Wind Load Cases should be considered
  - Along X-Direction
  - Along Y Direction
  - Along Diagonal
- Each Basic Wind Load Case should be entered separately into load combinations twice, once with (+ve) and once with (-ve) sign
- Total of 6 Wind Load Cases should considered in Combinations, but only 3 Load Cases need to be defined and analyzed

Applying Wind Loads

- At least 3 Basic Load Case for Wind Load should be considered

- Diagonal wind load may be critical for special types and layouts of buildings



Wind Load Combinations

	Comb1	Comb2	Comb3	Comb4	Comb5	Comb6
Wx			0	0	0	0
Wy	0	0	+f	-f	0	0
Wxy	0	0	0	0		

Example:
Comb = 0.75(1.4D + 1.7W) will need Six Actual Combinations

Comb1= 0.75(1.4D + 1.7Wx)
Comb2 = 0.75(1.4D - 1.7Wx)
Comb3 = 0.75(1.4D + 1.7Wy)
Comb4 = 0.75(1.4D - 1.7Wy)
Comb5 = 0.75(1.4D + 1.7Wxy)
Comb6 = 0.75(1.4D - 1.7Wxy)

- (f) Is the load factor specified for Wind in the design codes
- Six Additional Load Combinations are required where ever “Wind” is mentioned in the basic Load Combinations

Obtaining Envelop Results

	Comb1	Comb2	Comb3	Comb N
Load Case -1				
Load Case - 2				
Load Case - 3				
Load Case - M				
Total	P1	P2	P3	P N

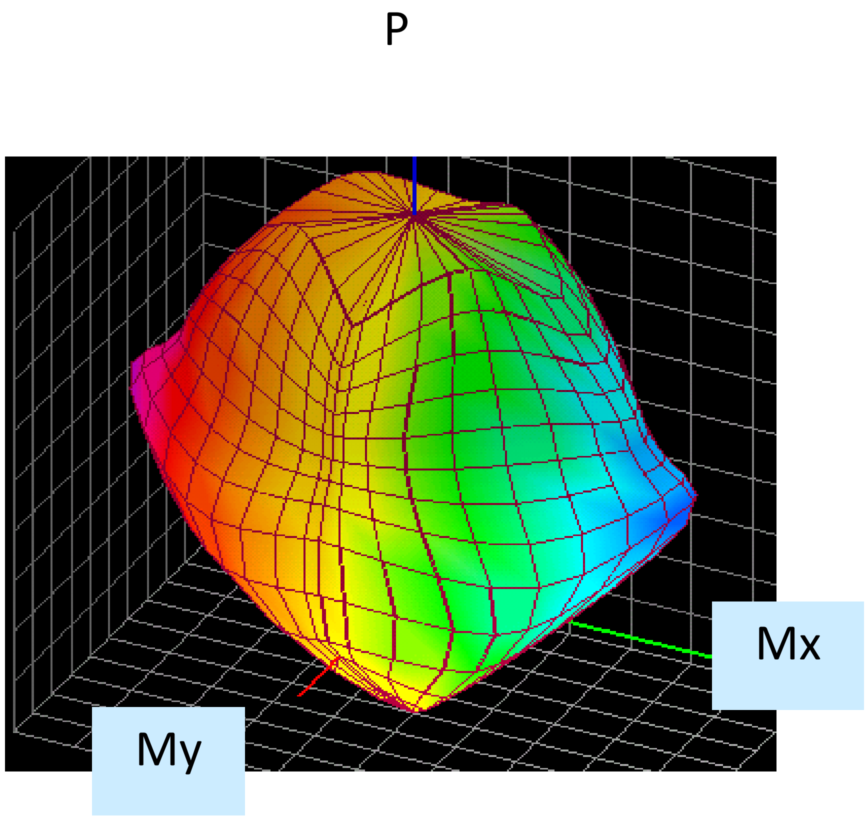
Envelope Results

Max, P
Min, P

Can Envelop Results be Used

- Actions Interact with each other, effecting the stresses

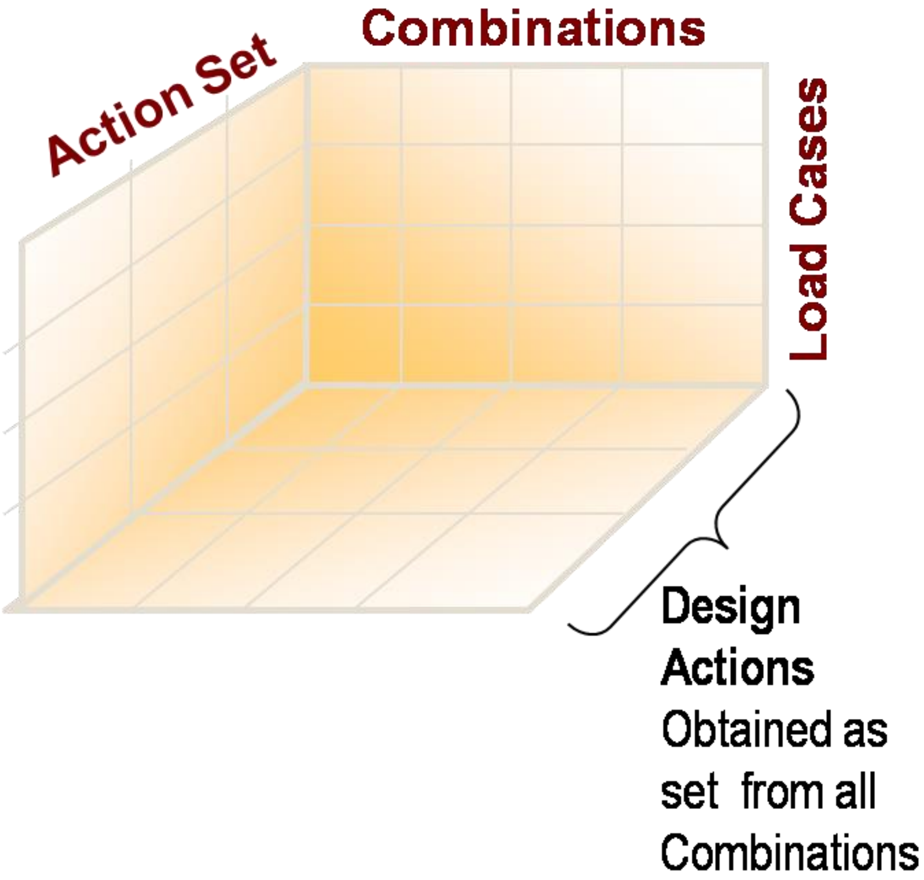
- For Column Design: P, Mx, My
- For Beam Design: Mx, Vy, Tz
- For Slabs: Mx, My, Mxy
- At least 3 Actions from each combination must be considered together as set
- Therefore, Envelop Results Can Not be Used
- Every Load Combinations must be used for design with complete “Action Set”



Design Actions For Static Loads

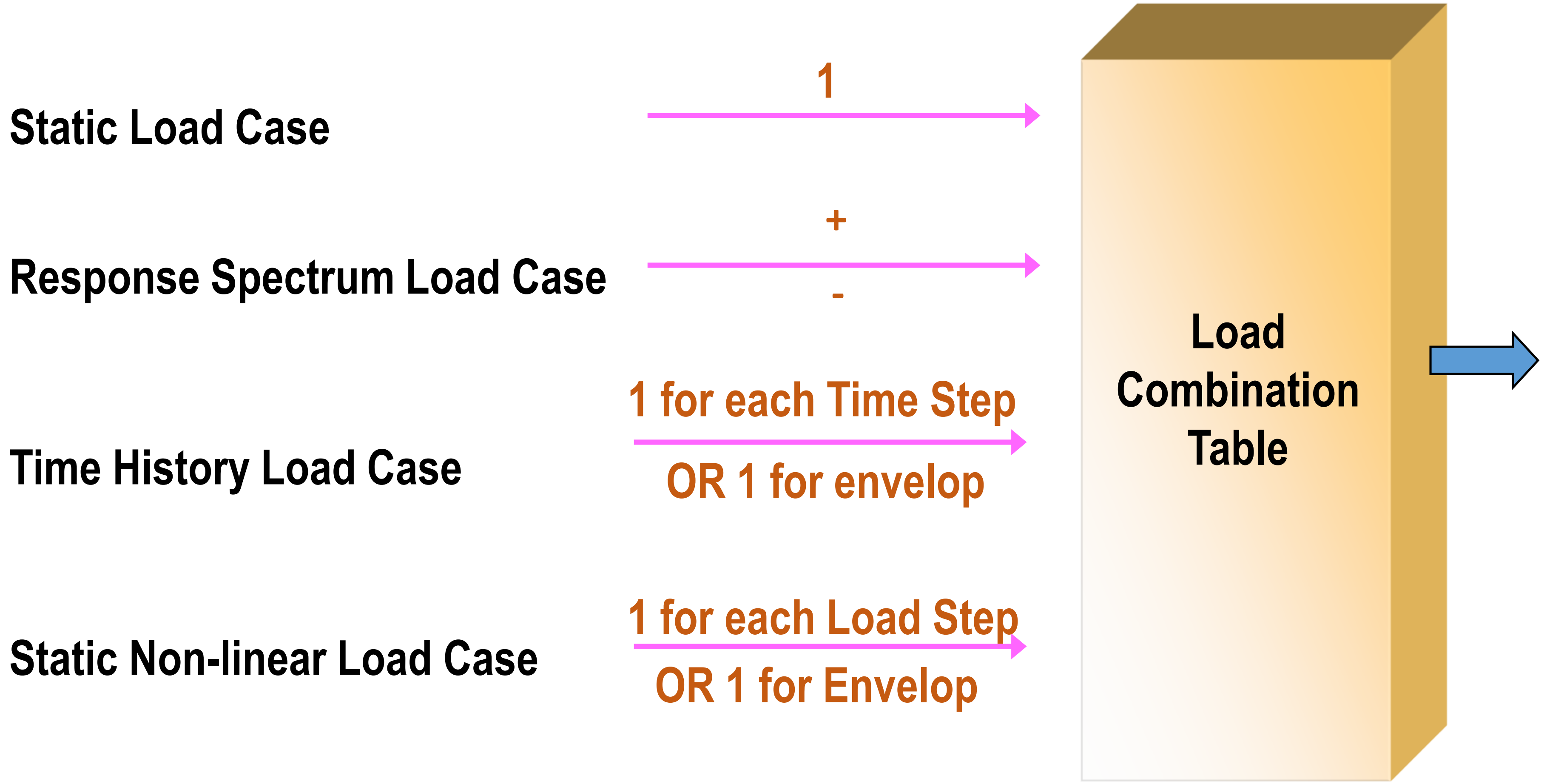
- For static loads, Design Actions are obtained as the cumulative result from each load combination, as set for all interacting actions

- The final or critical results from design of all load combinations are adopted



Static, Dynamic and Nonlinear Results

- For a Single Action:

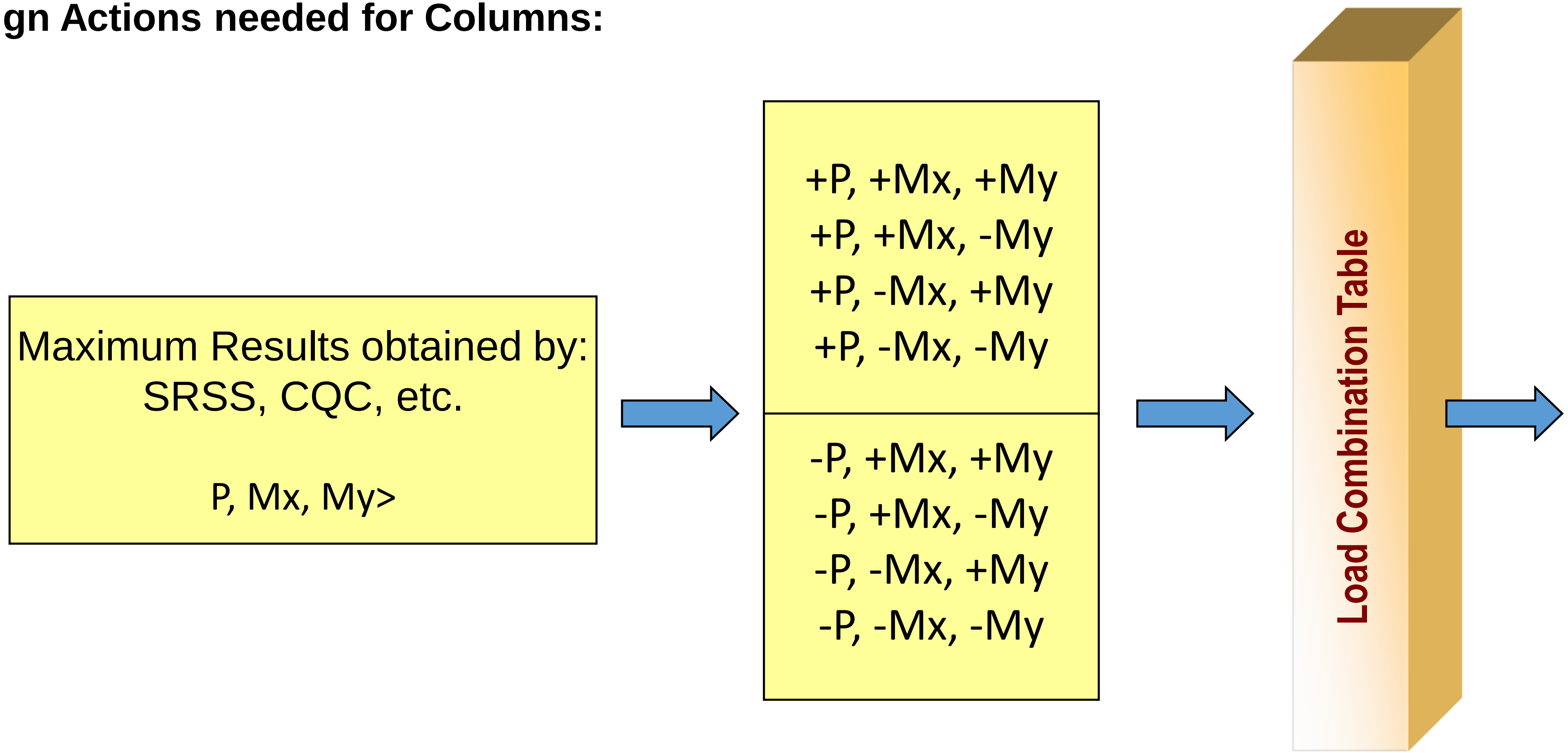


Response Spectrum Case

- All response spectrum cases are assumed to be earthquake load cases
- The output from a response spectrum is all positive.
- Design load combination that includes a response spectrum load case is checked for all possible combinations of signs (+, -) on the response spectrum values
- A 3D element will have eight possible combinations of P, M2 and M3 and eight combinations for M3, V, T

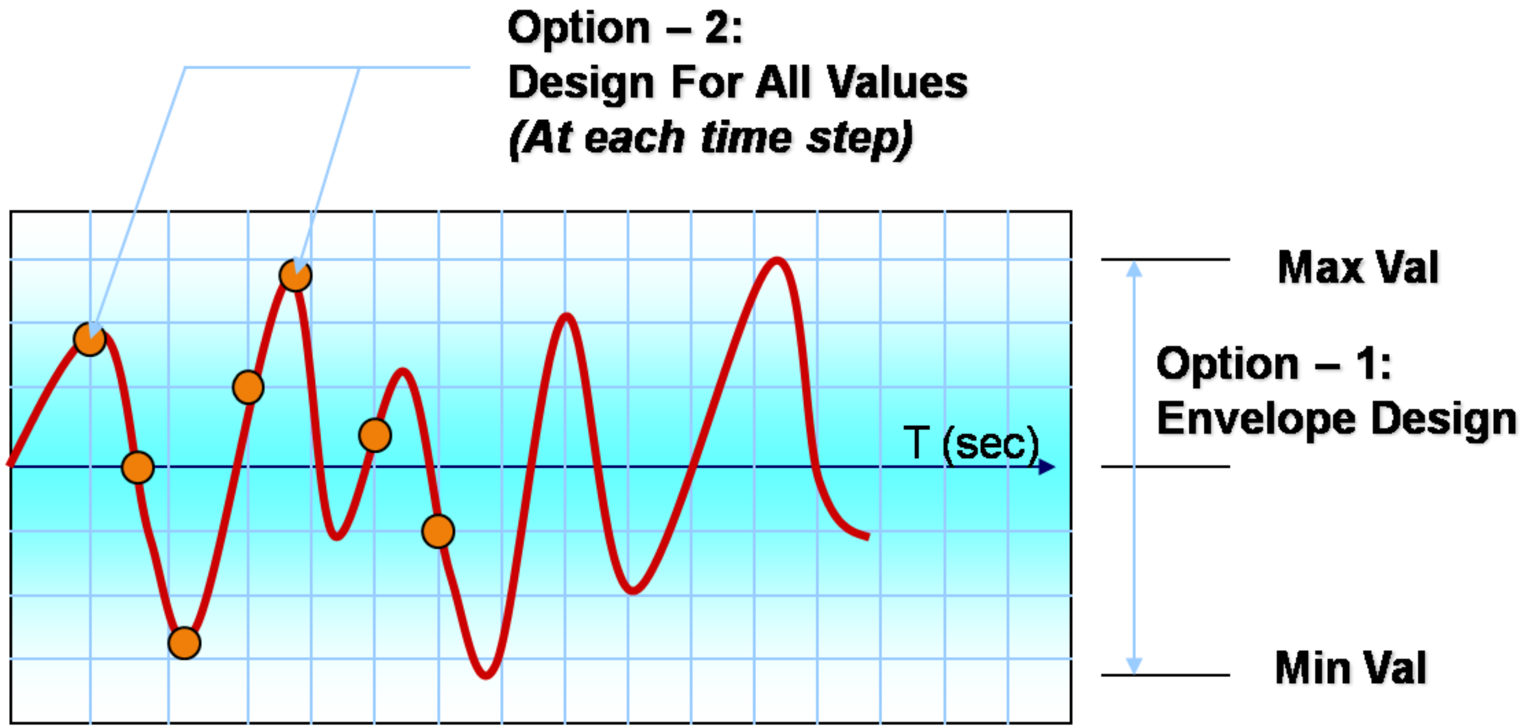
Response Spectrum Results

- **Design Actions needed for Columns:**



Time History Analysis Results

- **Response Curve for One Action**



Time-History Results

- The default design load combinations do not include any time history results
- Define the load combination, to include time history forces in a design load combination
- Can perform design for each step of Time History or design for envelops for those results
- For envelope design, the design is for the maximum of each response quantity (axial load, moment, etc.) as if they occurred simultaneously.
- Designing for each step of a time history gives correct correspondence between different response quantities

Time History Results

- The program gets a maximum and a minimum value for each response quantity from the envelope results for a time history
- For a design load combination any load combination that includes a time history load case in it is checked for all possible combinations of maximum and minimum time history design values.
- If a single design load combination has more than one time history case in it, that design load combination is designed for the envelopes of the time histories, regardless of what is specified for the Time History Design item in the preferences.

Static Non Linear Results

- The default design load combinations do not include any Static Nonlinear results
- Define the load combination, to include Static Nonlinear Results in a design load combination
- For a single static nonlinear load case the design is performed for each step of the static nonlinear analysis.

The Result Combination

- The Result combination does not consider the interaction of Earthquake, Wind, Vibration and Fire.
- That would be really complex and probably not realistic scenario.

The ELF and RSA Procedures as prescribed in ASCE 7-16

Software Demonstration

ETABS v2016

Thank you