This Federal Emergency Management Agency (FEMA) CD contains a set of instructional materials for use with FEMA Publication 451, *NEHRP Recommended Provisions: Design Examples,* in the form of PowerPoint slides with notes. These training materials provide a means for gaining additional knowledge about earthquake engineering as presented in the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 450). Also on the CD is NONLIN, an educational program for dynamic analysis of simple linear and nonlinear structures. The instructional materials can be presented to engineers/architects by a qualified speaker with expertise in the practice of earthquake engineering, can be used by an individual who wishes to enhance his/her understanding of earthquake engineering, or can be applied by engineering academics as the basis for classroom instruction on earthquake-resistant design. The CD contains a series of topic folders. In each folder are pdf files for the PowerPoint presentation, for the notes pages, and for student handouts. Also included is a folder for NONLIN that contains zip files for this program and a file that lists items referenced in the presentation.

Any opinions, findings, conclusions, or recommendations expressed in this instructional material publication do not necessarily reflect the views of the Federal Emergency Management Agency. Additionally, neither FEMA nor any of its employees make any warranty, expressed or implied, nor assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, product, or process included in this publication. The opinions expressed herein regarding the requirements of the *NEHRP Recommended Provisions*, the referenced standards, and the building codes are not to be used for design purposes. Rather the user should consult the jurisdiction's building official who has the authority to render interpretation of the code.

NEHRP Recommended Provisions: Instructional Materials (FEMA 451B)

- These instructional materials complement FEMA 451, *NEHRP Recommended Provisions: Design Examples*
- Needed are copies of FEMA 451 and FEMA 450, the 2003 *NEHRP Recommended Provisions for New Buildings and Other Structures (*Part 1, *Provisions,* and Part 2, *Commentary)*

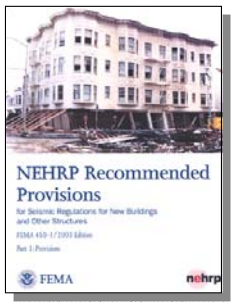
FEMA Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 2

In addition to the *Design Examples* volume, the training requires copies of FEMA Publication 450, the 2003 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*.

FEMA 450 and 451

Single copies of both publications are available at no charge from the FEMA Publications Center at **1-800-480-2520**

(order by publication number)



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Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 3

Individual copies of these publications can be obtained at no charge from the FEMA Publications Center, 1-800-480-2520 (order by FEMA Publications number). If multiple copies are needed for presentation of the training materials to a group, e-mail bssc@nibs.org.

Acknowledgments

- FEMA 451 and 451B were developed for FEMA by the Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences (NIBS).
- The BSSC also manages development and updating of the *NEHRP Recommended Provisions*.
- For information about the BSSC and its member organizations or to download FEMA 451 and 451B, see

http://bssconline.org

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Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 4

This CD was developed by the Building Seismic Safety Council under Contract EMW-1998-CO-0419 between the Federal Emergency Management Agency and the National Institute of Building Sciences. For further information on the Building Seismic Safety Council, see the Council's website – www.bssconline.org – or contact the Building Seismic Safety Council, 1090 Vermont, Avenue, N.W., Suite 700, Washington, D.C. 20005; phone 202-289-7800; fax 202-289-1092; e-mail bssc@nibs.org.

Acknowledgments

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- **Finley A. Charney**, Ph.D., P.E., Virginia Tech, Blacksburg
- **W. Samuel Easterling**, Ph.D., P.E., Virginia Tech
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- **James R. Martin**, Jr., Ph.D., Virginia Tech
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- **Carin Roberts-Wollmann**, Ph.D., P.E., Virginia Tech

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Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 5

Much of this material was originally developed for the Multihazard Building Design Summer Course offered at FEMA's Emergency Management Institute. Managing the development of that course material for the Building Seismic Safety Council (BSSC) was Advanced Structural Concepts, Inc., Blacksburg, Virginia (Finley A. Charney, PhD., PE, President). Further, the course materials were developed in association with the Center for Extreme Load Effects on Structures, Virginia Tech (Finley A. Charney, PhD, PE, Director, and James R. Martin, Jr., Associate Director)

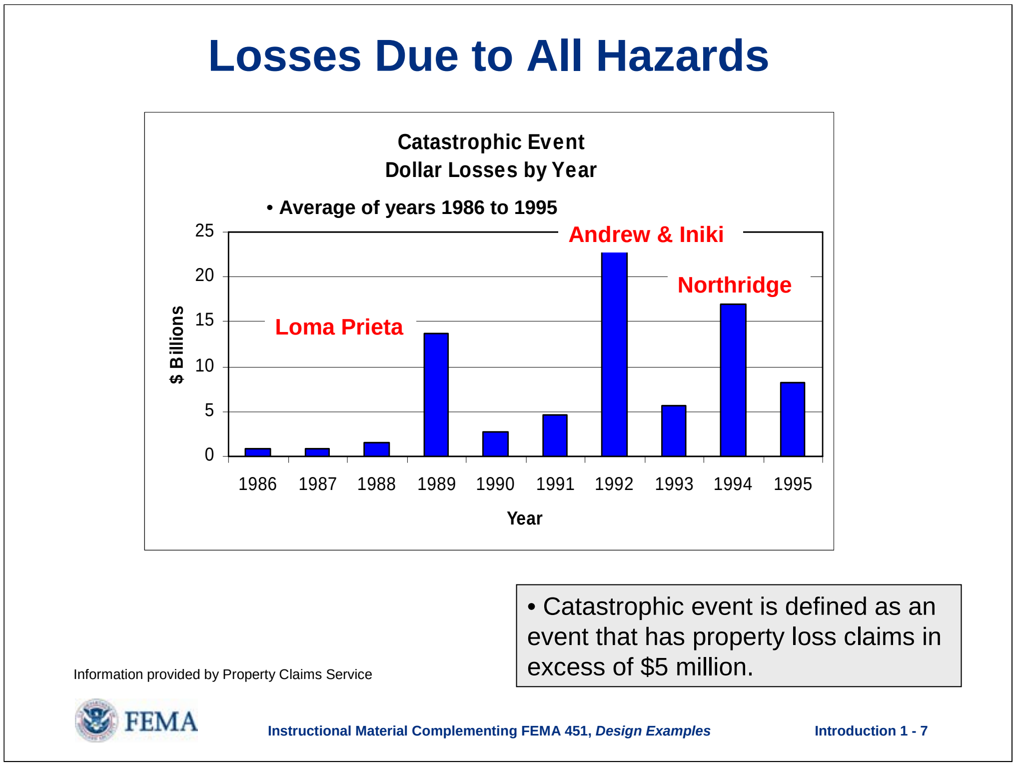
Motivation for Earthquake Engineering

- Minimize human death and injury
- Minimize economic loss
  - Direct (collapse and damage)
  - Indirect (loss of use, business interruption)
- Maintain lifelines

Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 6

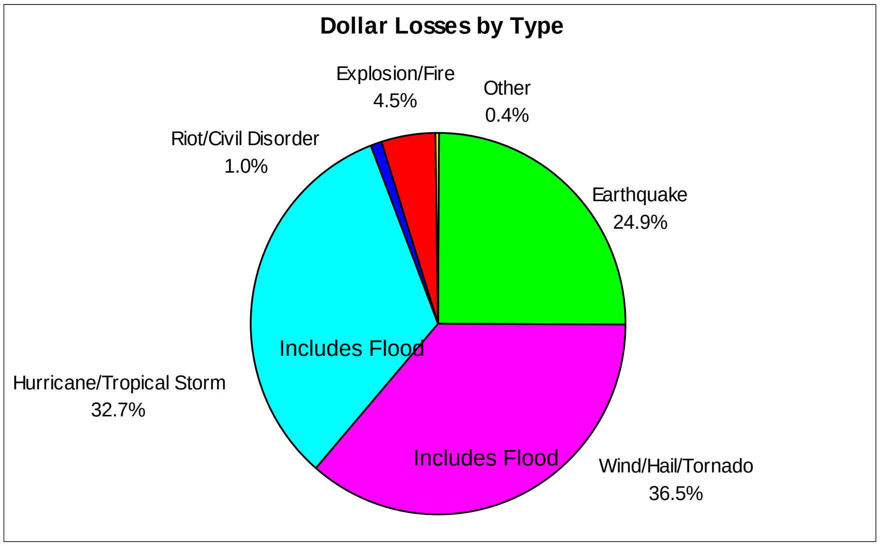
Earthquake-resistant design and construction are important in those areas of the nation at risk.

Users of this document who are also interested in residential construction are encouraged to consult FEMA Publication 232, *Homebuilders' Guide to Earthquake-Resistant Design and Construction*. This guide provides information on current best practices for earthquake-resistant home design and construction for use by builders, designers, code enforcement personnel, and potential homeowners. It incorporates lessons learned from the 1989 Loma Prieta and 1994 Northridge earthquakes as well as knowledge gained from the FEMA CUREE-Caltech Wood Frame Project. It also introduces and explains the effects of earthquake loads on one- and two-family detached houses and identifies the requirements of the 2003 *International Residential Code* (IRC) intended to resist these loads.



Natural hazards can be catastrophic to life and property. This slide indicates dollar losses for all natural hazards in the United States for the past several years. The Loma Prieta and Northridge earthquakes were matched in dollar damage by hurricanes Hugo, Andrew and Iniki and all were surpassed by the damage caused by Hurricane Katrina.

A Significant Portion of Dollar Loss Due to Earthquake



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Earthquakes are a significant hazard but generally cause less dollar damage than wind, rain, and associated flooding. This slide does not break out flood damage, however, it should be emphasized that floods are one of the largest sources of losses due to natural disasters.

Nevertheless, mitigation actions to reduce the losses from these natural hazards are cost-effective. In 2006, the National Institute of Building Sciences through its Multihazard Mitigation Council completed a report to the Congress of the United States on behalf of Federal Emergency Management Agency (FEMA) that presents the results of an independent study to assess the future savings from hazard mitigation activities. This study shows that money spent on reducing the risk of natural hazards is a sound investment. On average, a dollar spent by FEMA on hazard mitigation (actions to reduce disaster losses) provides the nation about $4 in future benefits. In addition, FEMA grants to mitigate the effects of floods, hurricanes, tornados, and earthquakes between 1993 and 2003 are expected to save more than 220 lives and prevent almost 4,700 injuries over approximately 50 years. Recent disaster events painfully demonstrate the extent to which catastrophic damage affects all Americans and the federal treasury.

Those interested in reading the report of the study should see http://nibs.org/MMC/mmcactiv5.html

Examples of US Earthquake Losses

- 1906 San Francisco
- 1933 Long Beach
- 1964 Alaska
- 1971 San Fernando Valley
- 1989 Loma Prieta
- 1994 Northridge

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These are but a few of the major earthquakes occurring in the United States during the previous century. This presentation emphasizes the Loma Prieta and Northridge earthquakes.

The Northridge earthquake, like the 1971 San Fernando Valley earthquake, was a “wakeup” call to engineers and ultimately resulted in significant changes to building codes. Much of the current emphasis on performance-based engineering is due to the greater than expected damage that occurred as a result of the Northridge earthquake.

1971 Earthquake in the San Fernando Valley of California

Earth dam located about 20 km from the epicenter. Part of the upstream face lost bearing strength and slipped beneath the water.

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This slide emphasizes the fact that damage occurs to nonbuilding structures as well as building structures.

1971 San Fernando Valley Earthquake

"Soft story" failure of the Olive View Hospital. The column failure caused a collapse that pinned the ambulances under the rubble, rendering them useless.

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Damage to the Olive View Hospital was particularly disturbing because the structure was relatively new and was designed according to the "modern" code at the time. The building was a complete loss and had to be demolished. Note that the ambulance canopy in the foreground is a separate structure, and was also a complete loss. Also significant is the fact that the ambulances were trapped in the collapsed canopy and were not available for use.

During the 1994 Northridge earthquake, the new Olive View Hospital structure fared rather well, but there were significant losses associated with nonstructural elements and components.

1989 Earthquake in Loma Prieta, California

Oakland Bay Bridge failure.

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Losses of transportation structures are very dramatic and can be among the most costly in terms of loss of life and property and indirect effects. This bridge was out of service for several weeks after the earthquake requiring major rerouting of traffic.

The collapse of the Oakland Cyprus Street Viaduct (not shown) was responsible for the loss of 42 lives. There were similar but less catastrophic failures of sections of the Embarcadero Freeway in San Francisco.

The Loma Prieta earthquake killed more than 60 people, injured 3,700, and left 12,000 homeless.

1994 Earthquake in Northridge, California

Bull Creek Canyon Channel Bridge on the Simi Valley freeway near the epicenter to the north. Shear failure of a flared column.

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Freeways in the Los Angeles area were not immune to damage during the Northridge earthquake. Ironically, many of the bridges that failed were scheduled for rehabilitation prior to the earthquake.

Approximately 60 people were killed by the quake.

1994 Northridge Earthquake

Gavin Canyon Undercrossing on I-5

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Another illustration of damage as a result of the Northridge earthquake.

Examples of Earthquake Losses Outside the United States

- 1923 Tokyo
- 1927 China
- 1985 Chile
- 1985 Mexico City
- 1988 Armenia
- 1993 Hokkaido
- 1995 Kobe
- 1999 Turkey, Taiwan
- 2001 India

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 15

Earthquakes occur all over the world and often produce unimaginable destruction. Codes and enforcement in developing countries are often decades behind those of the industrialized world.

1985 Mexico City Earthquake

Pino Suarez Towers looking north -- one of the few steel frame buildings to collapse.

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 16

The damage in Mexico City was due to an earthquake that occurred more than 350 km away from the city center. The main shock killed 10,000, left 50,000 homeless, and caused $4 billion dollars damage.

The vast destruction was attributed in large part to dynamic amplification of seismic waves through the soft soil in Mexico City. The dominant seismic waves had a period of about 2.0 seconds, wreaking havoc on buildings in the 10- to 20-story range.

1988 Leninakan, Armenia, Earthquake

Damage to a stone bearing wall building. The floor planks were not tied to the supporting bearing walls.

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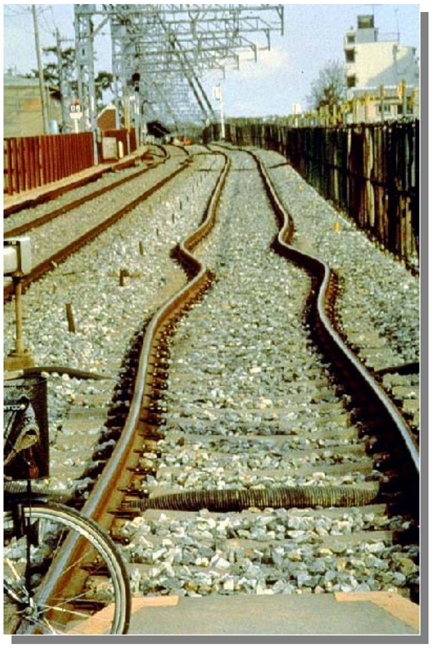
This is an example of the devastation caused by earthquakes in countries without adequate seismic design building code requirements and/or enforcement.

Many (almost complete destruction) precast concrete frame buildings collapsed because of inadequate detailing. This earthquake killed at least 25,000 people, and left 500,000 homeless. Dollar damage was estimated in excess of 13 billion. Overall, 95% of the precast frame structures (5 to 12 stories) in Leninakan collapsed or were damaged beyond repair.



1995 Kobe, Japan, Earthquake

Distorted train tracks.

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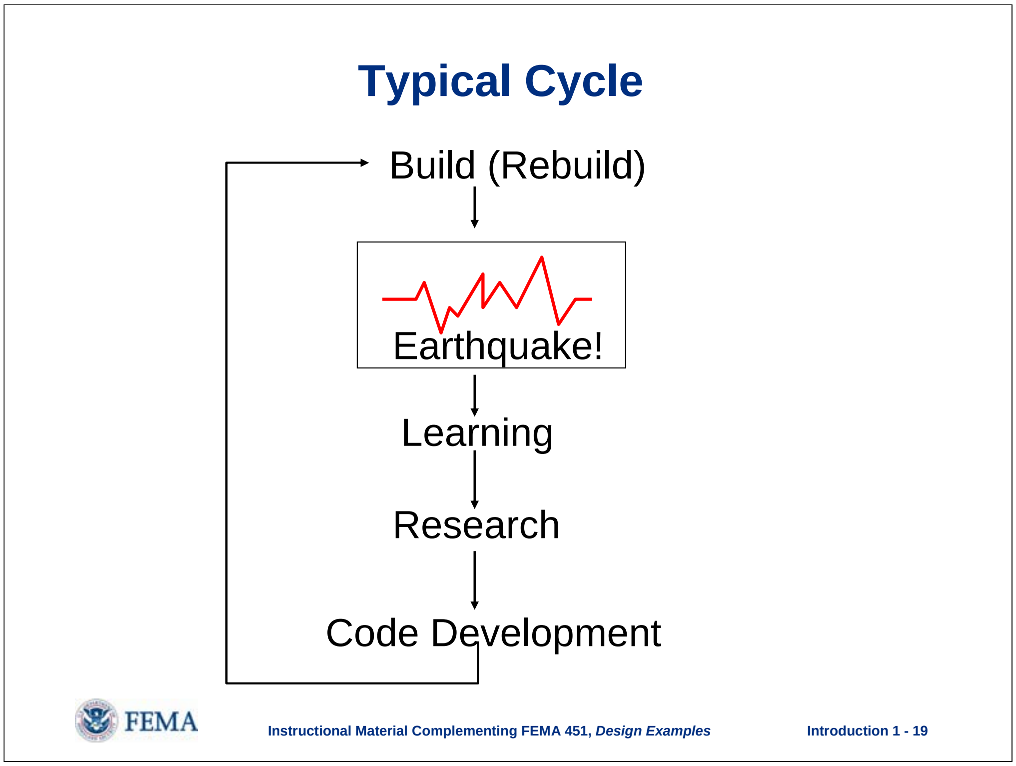
Instructional Material Complementing FEMA 451, *Design Examples*

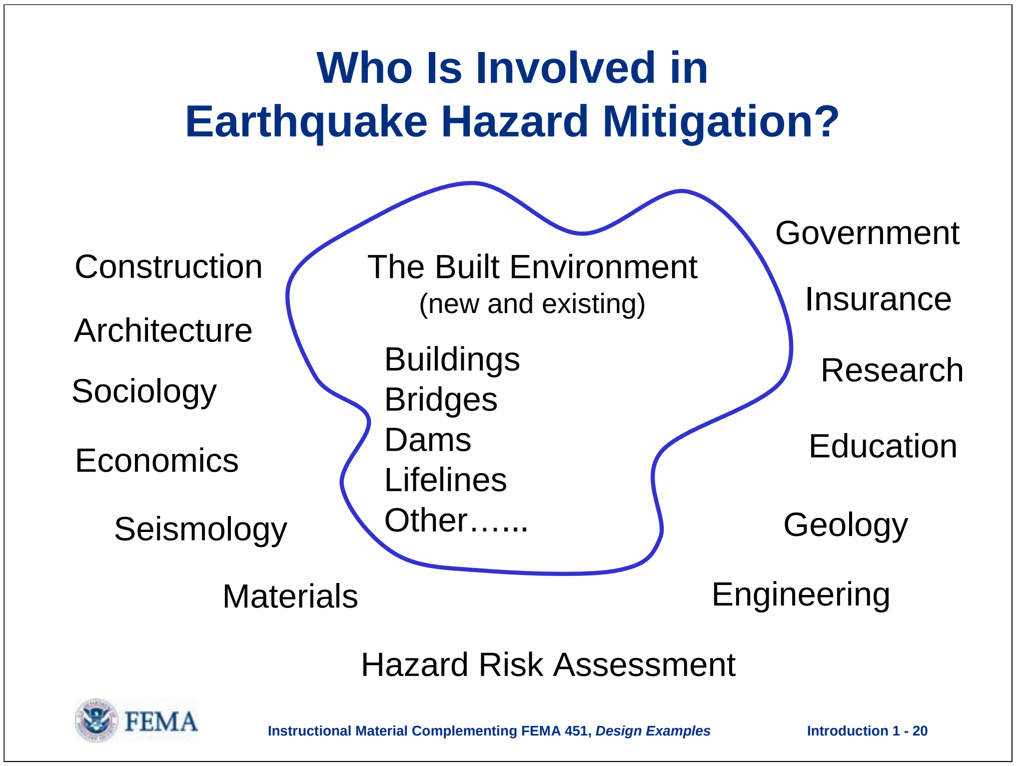
Introduction 1 - 18

The Kobe earthquake killed more than 5,000 people and injured 26,000 others. More than 56,000 buildings were destroyed. Losses were estimated at more than $2 billion. This is more than 10 times the dollar loss for the Northridge earthquake which occurred exactly one year earlier in southern California. This slide was selected to emphasize the point that loss to nonbuilding structures and lifelines can have a significant effect on society. Further, it should be noted that a considerable amount of business and industrial activities that moved from the area after the earthquake never returned.



If there is any fortunate aspect of earthquakes, it is that the built environment is an excellent proving ground. Damage occurring during earthquakes is extensively studied and research is performed, ultimately leading to the development of improved building codes.

However, it seems that no matter how diligently we react to earthquakes, we are taught new and serious lessons when the next quake strikes. The damage occurring to welded frame structures during the Northridge earthquake is an excellent example.



Many disciplines are involved in earthquake hazard mitigation. All groups must work together to provide the level of protection needed by society.

These Instructional Materials FOCUS on STRUCTURAL ENGINEERING and

- New buildings
- Hazards associated with ground shaking
- “Force-Based” approach of 2003 *NEHRP Recommended Provisions* (FEMA 450)
- Examples presented in *NEHRP Recommended Provisions: Design Examples* (FEMA 451)
- Probabilistic and deterministic based ground motions
- New concepts of performance-based engineering

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 21

These instructional materials focus almost entirely on new buildings. However, some information is provided for existing buildings, particularly as related to performance-based engineering, and on nonbuilding structures and nonstructural building components.

Further, these instructional materials concentrate on ground shaking, which is only one of the many hazards associated with earthquakes (e.g. fault rupture, liquefaction, landslides, flooding, and fire).

Published Design Documents for New Buildings

- NEHRP Recommended Provisions (FEMA 450)
- IBC and IRC
- ASCE 7

1906 San Francisco Earthquake

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Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 22

2003 *NEHRP Recommended Provisions for New Buildings and Other Structures*

- Uses seismic hazard map (2%-50years) for evaluation purposes
- Relies on "equal displacement" concept to establish design forces
- Utilizes *linear elastic* static or dynamic analysis
- Deformations checked globally

Intended result (obtained somewhat *implicitly*):

- Little or no damage for frequent earthquakes
- Minor nonstructural damage for common earthquakes
- Life-safety or collapse prevention for rare earthquakes

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Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 23

This slide emphasizes the underlying principles of the *NEHRP Recommended Provisions*. Performance is evaluated somewhat implicitly, meaning that local deformations in members are not addressed. Before the Northridge earthquake, it was thought that this methodology was sufficient. Many engineers are now moving towards performance-based concepts, particularly in the rehabilitation of existing buildings.

Other Topics in this Series

Topic 1 Introduction to Course
Topic 2 Earthquakes Mechanics and Effects
Topic 3 Structural Dynamics of SDOF Systems
Topic 4 Structural Dynamics of MDOF Systems
Topic 5a Seismic Hazard Analysis
Topic 5b Ground Motion Maps
Topic 6 Inelastic Behavior of Materials and Structures
Topic 7 Concepts of Earthquake Engineering [FEMA 451, Ch. 1]
Topic 8a Introduction to the NEHRP [FEMA 451, Ch. 2]
Topic 8b Overview of Standards used in *NEHRP Recommended Provisions*
Topic 9 Seismic Load Analysis
Topic 10 Seismic Design of Structural Steel Structures [FEMA 451, Ch. 5]
Topic 11 Seismic Design of Reinforced Concrete Structures [FEMA 451, Ch. 6]
Topic 12 Seismic Design of Masonry Structures [FEMA 451, Ch. 9]
Topic 13 Seismic Design of Wood Structures [FEMA 451, Ch. 10]
Topic 14 Foundation Design [FEMA 451, Ch. 4]
Topic 16 Nonstructural Components [FEMA 451, Ch. 13]

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 24

Topics 1 through 14 and 16 are the "basic" topics and include most of the concepts required to understand how earthquake analysis and design procedures are developed (Topics 1-7) and then how they are incorporated into the *NEHRP Recommended Provisions* and/or ASCE-7. These topics could generally be covered in a four- to five-day course with seven hours of instruction per day. If presented in such a classroom setting, instructors should consider developing a series of group exercises to help illustrate the concepts and to break up a long series of lectures. One of the exercises should use the computer program NONLIN that is included on the FEMA 451B CD.

The chapter numbers to the right of some of the topics refer to chapters in FEMA 451, *NEHRP Recommended Provisions: Design Examples*. In some cases, there is direct correlation between the examples in the slide sets and the FEMA 451 CD. For example, the topics in concrete and steel are related in this manner.

Other Topics in this Series
Part 2: Advanced Topics

Topic 15-1	**Introduction**
Topic 15-2	**Performance Based Engineering**
Topic 15-3	**Seismic Hazard Analysis**
Topic 15-4	**Geotechnical Earthquake Engineering**
Topic 15-5a	**Advanced Analysis, Part 1 of 3**
Topic 15-5b	**Advanced Analysis, Part 2 of 3**
Topic 15-5c	**Advanced Analysis, Part 3 of 3**
Topic 15-6	**Passive Energy Systems [FEMA 451, Ch. 6]**
Topic 15-7	**Seismic Isolation [FEMA 451, Ch. 11]**
Topic 15-8	**Nonbuilding Systems [FEMA 451, Ch. 12]**

FEMA   Instructional Material Complementing FEMA 451, *Design Examples*   Introduction 1 - 25

These topics are considered to be “advanced topics” and would be covered in a separate four-day course. Note that there is considerable overlap between the materials in Topics 5a and 15-3. As with the previous slide, the chapter numbers to the right of some of the topics refer to chapters in the FEMA 451 volume.

Chapters in the FEMA 451 Examples CD

Ch. 1	**Fundamentals**
Ch. 2	**Guide to the Use of the *NEHRP Recommended Provisions***
Ch. 3	**Structural Analysis (including nonlinear analysis)**
Ch. 4	**Foundation Design**
Ch. 5	**Steel Structures**
Ch. 6	**Reinforced Concrete Structures**
Ch. 7	**Precast Concrete Structures**
Ch. 8	**Composite Steel/Concrete Structures**
Ch. 9	**Masonry Structures**
Ch. 10	**Wood Structures**
Ch. 11	**Seismically Isolated Structures**
Ch. 12	**Nonbuilding Structures**
Ch. 13	**Nonstructural Components**

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Instructional Material Complementing FEMA 451, *Design Examples* Introduction 1 - 26

The FEMA 451 CD contains 13 chapters as shown in this slide. The examples are extremely detailed and should be worked into the coursework where possible. Individuals pursuing earthquake engineering knowledge using these presentations for self-study also are strongly encouraged to work through these examples after working through with the presentation information.

Structural engineering:
The art of using materials that
have properties which can only be estimated
to build real structures that
can only be approximately analyzed
to withstand forces that
are not accurately known
so that our responsibility to the public safety is satisfied.

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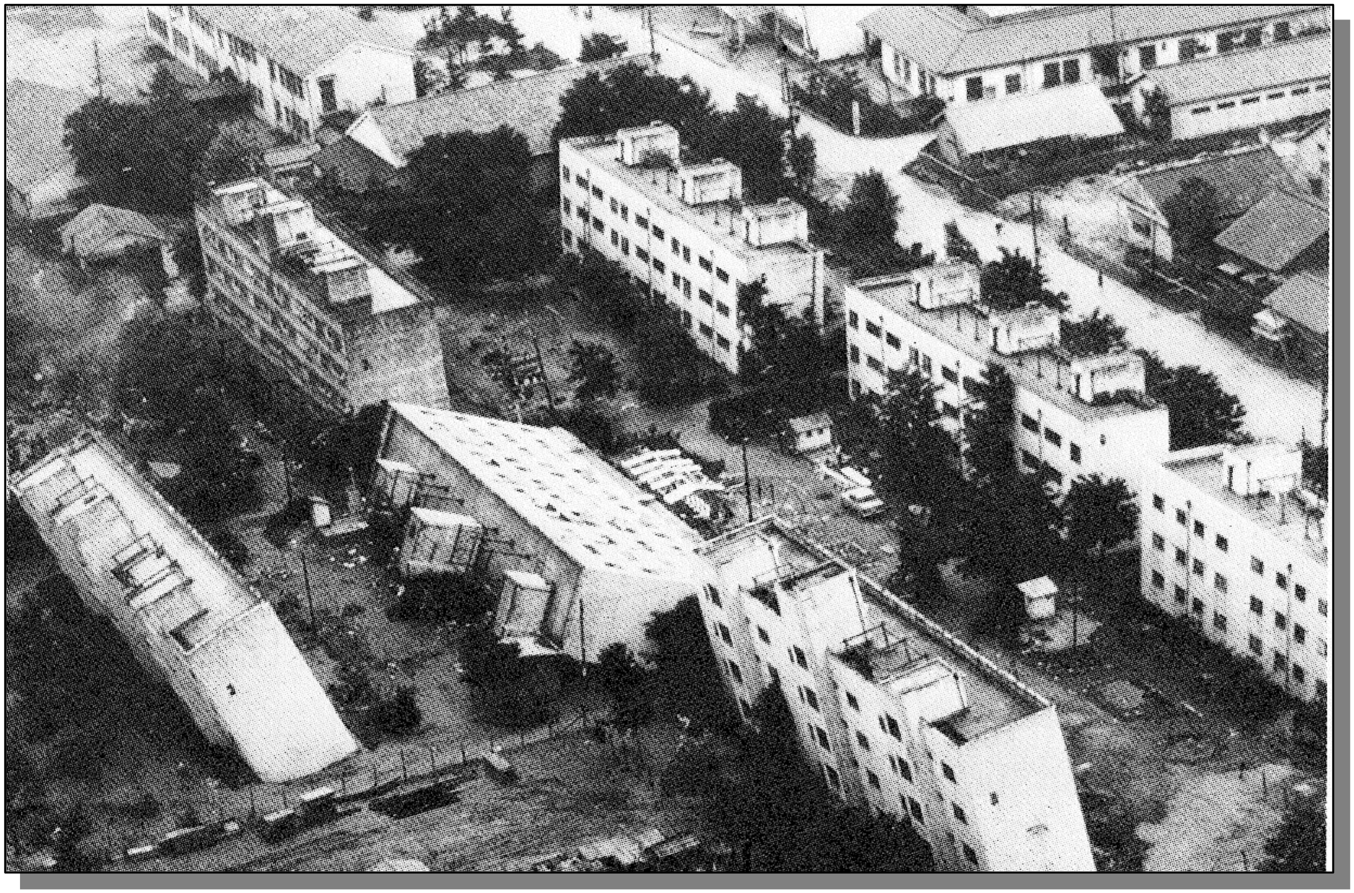
Earthquakes Mechanics and Effects



Earthquakes: Cause and Effect

- Why earthquakes occur
- How earthquakes are measured
- Earthquake effects
- Mitigation strategy
- Earthquake time histories

Seismic Activity: 1961-1967

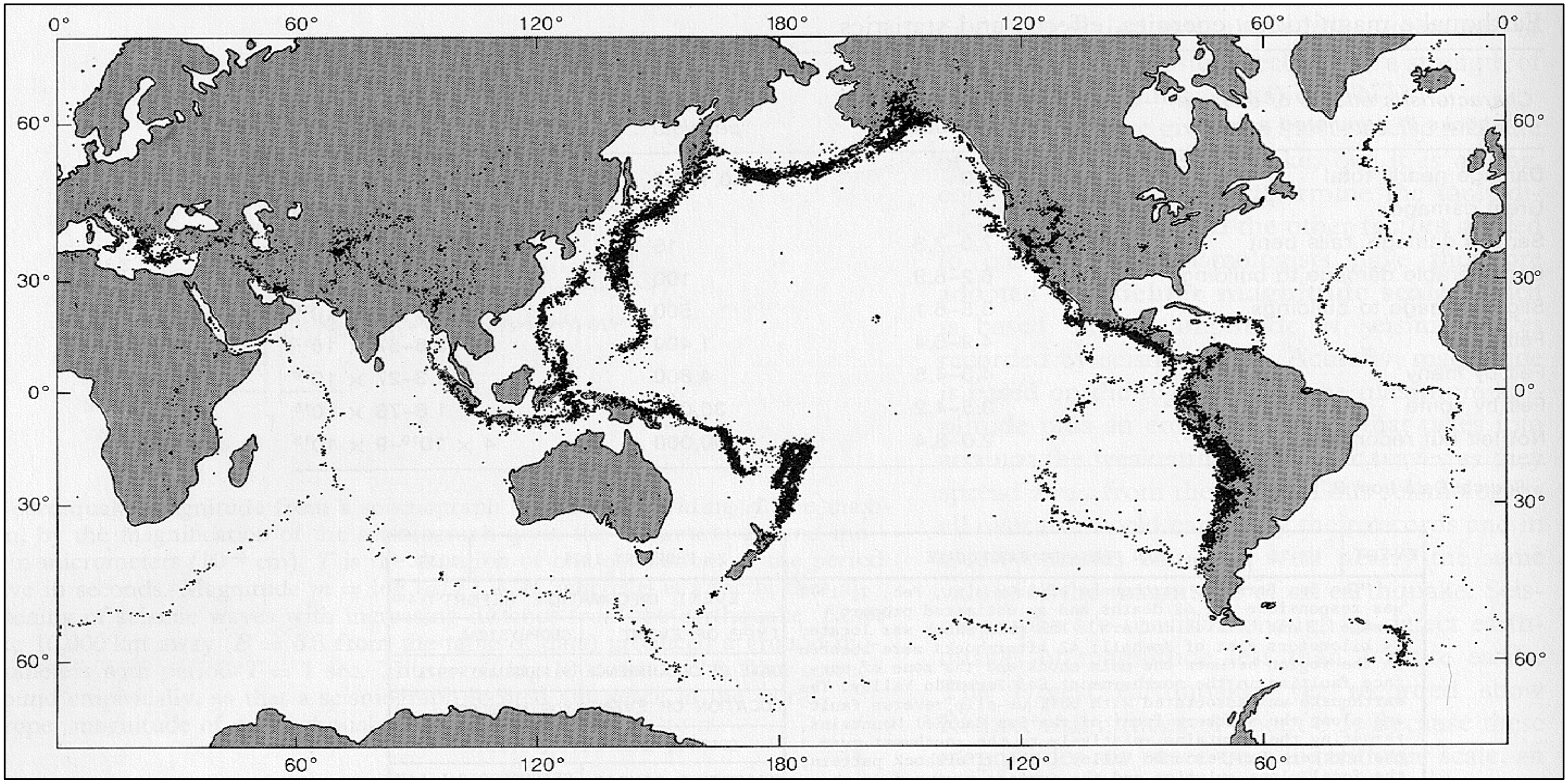


Plate Boundaries

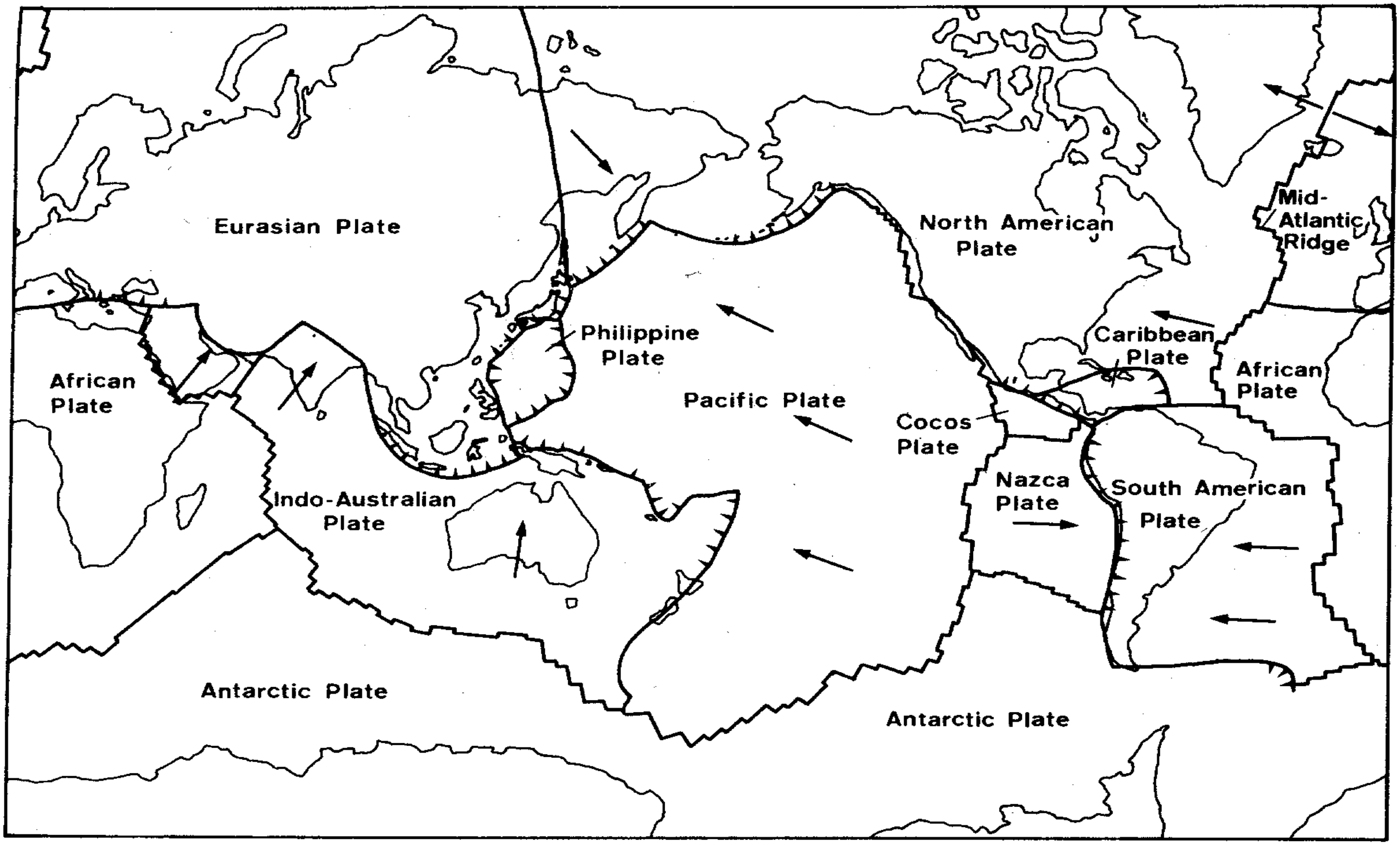


Plate Tectonics: Driving Mechanism

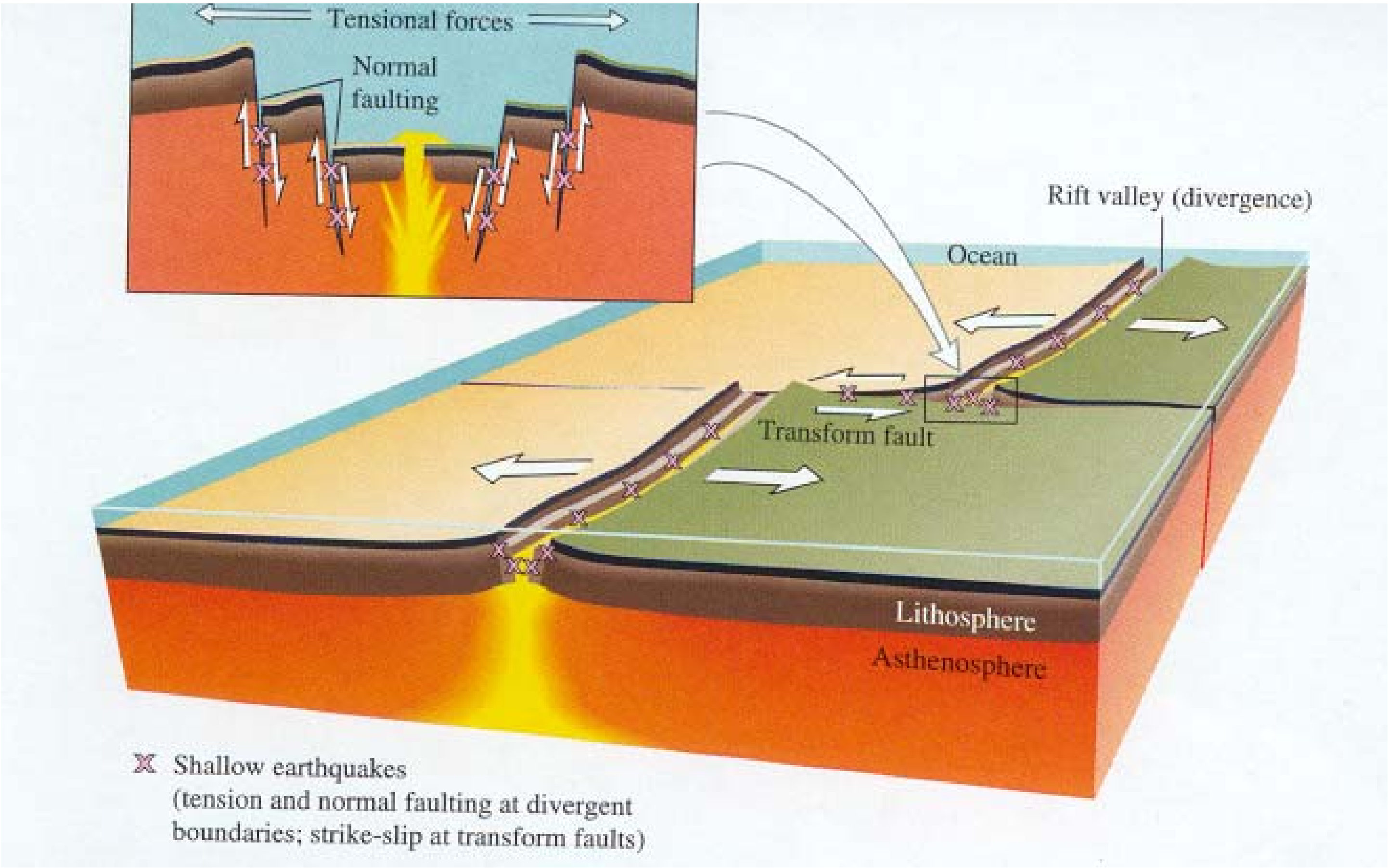
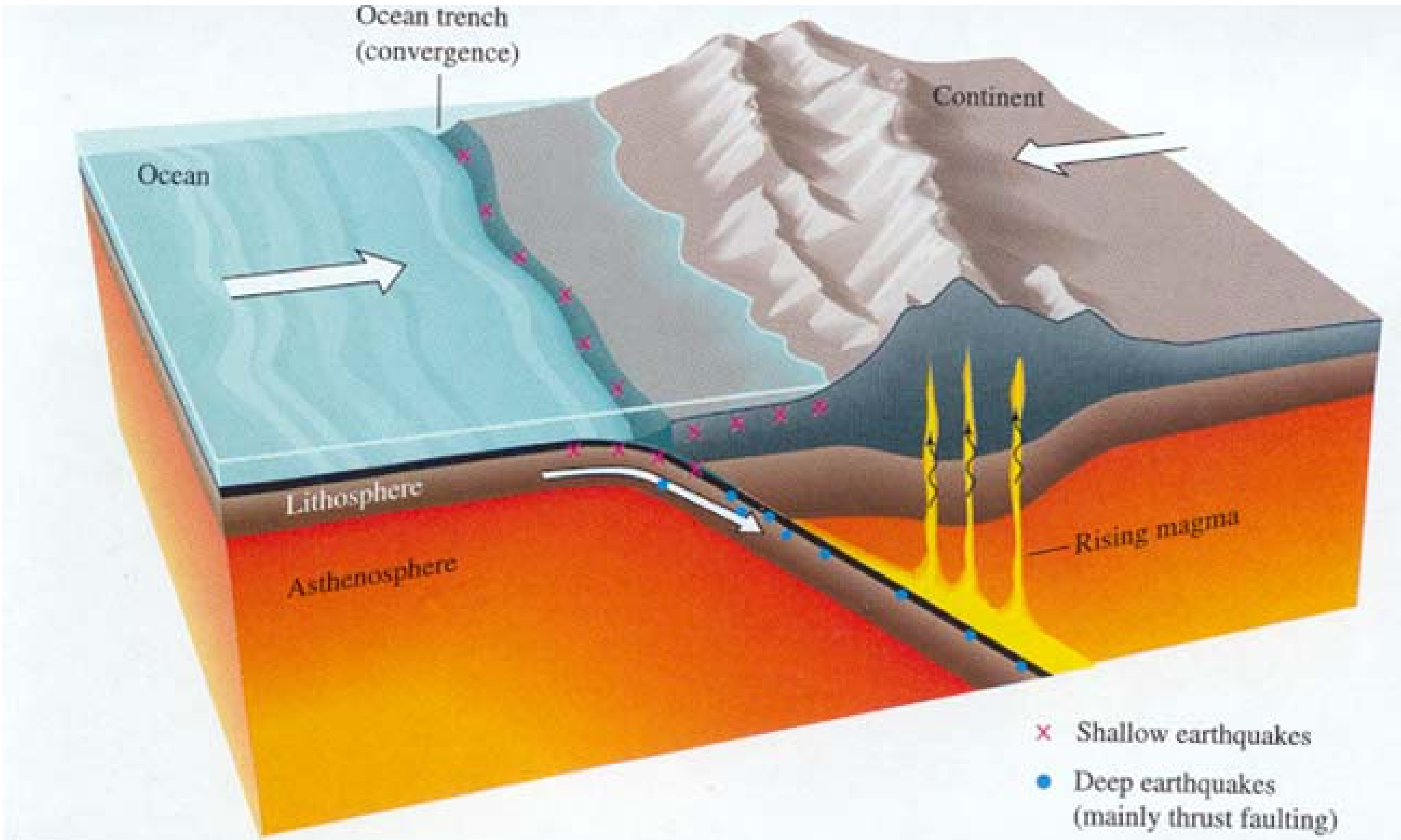
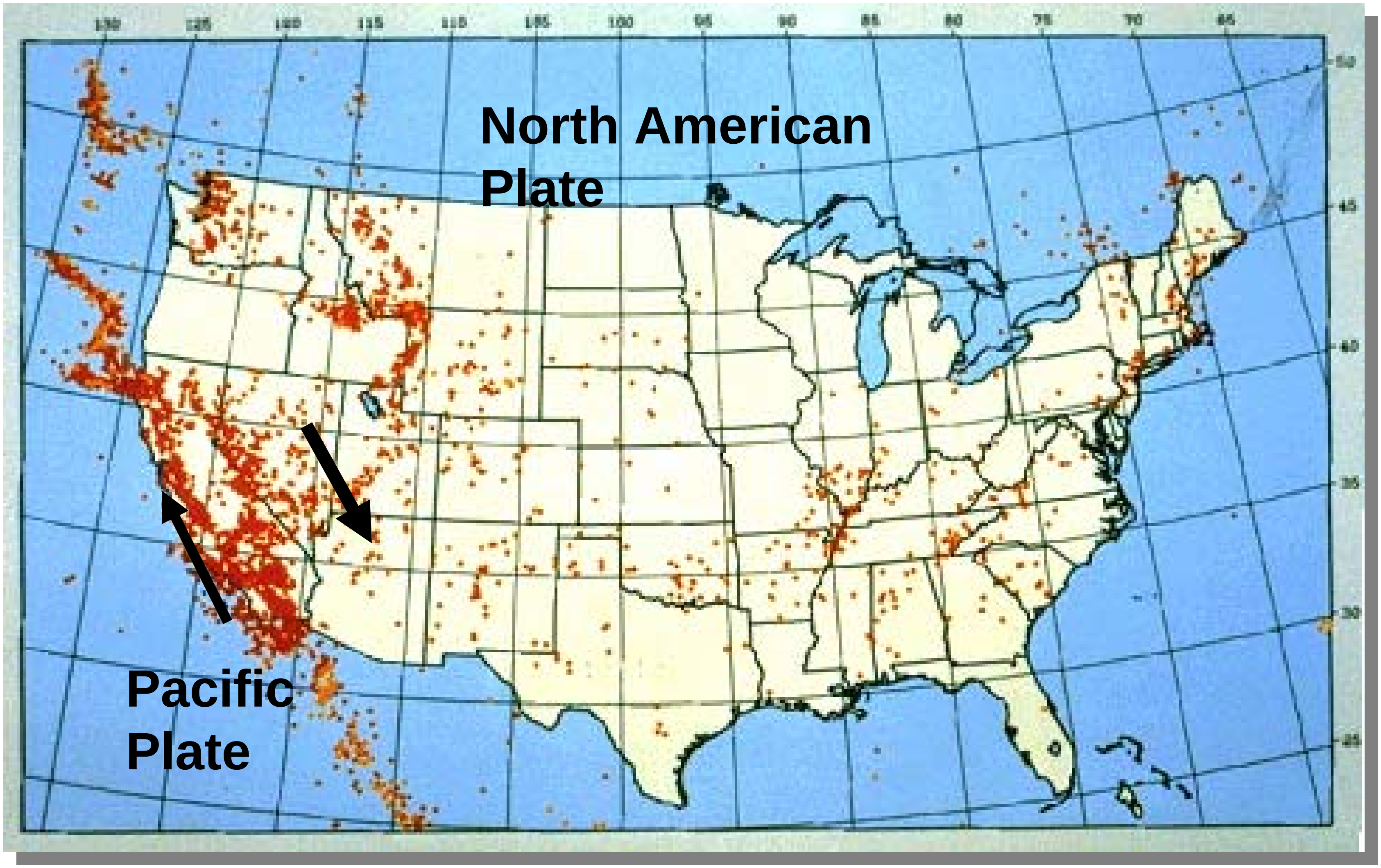


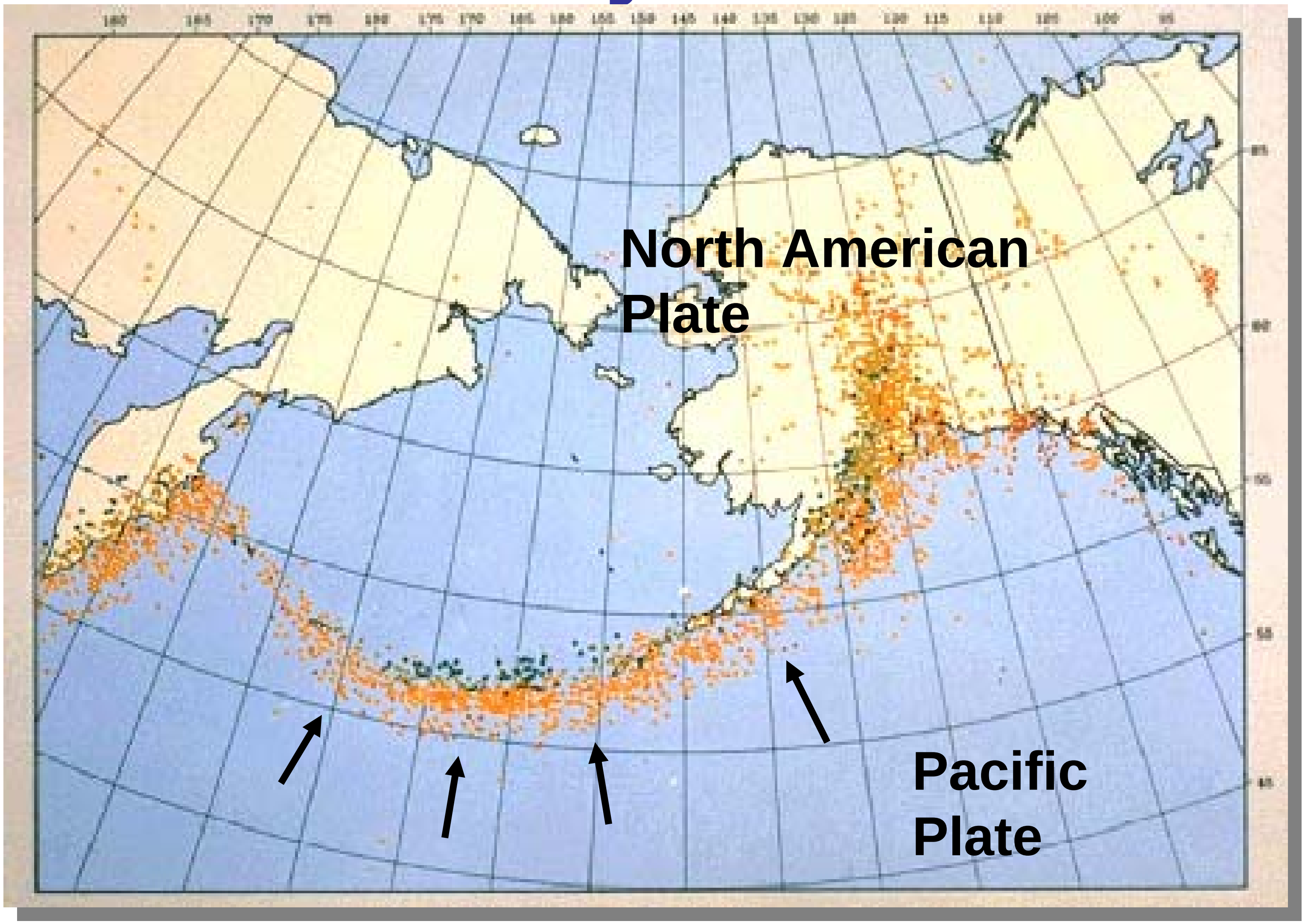
Plate Tectonics: Details in Subduction Zone



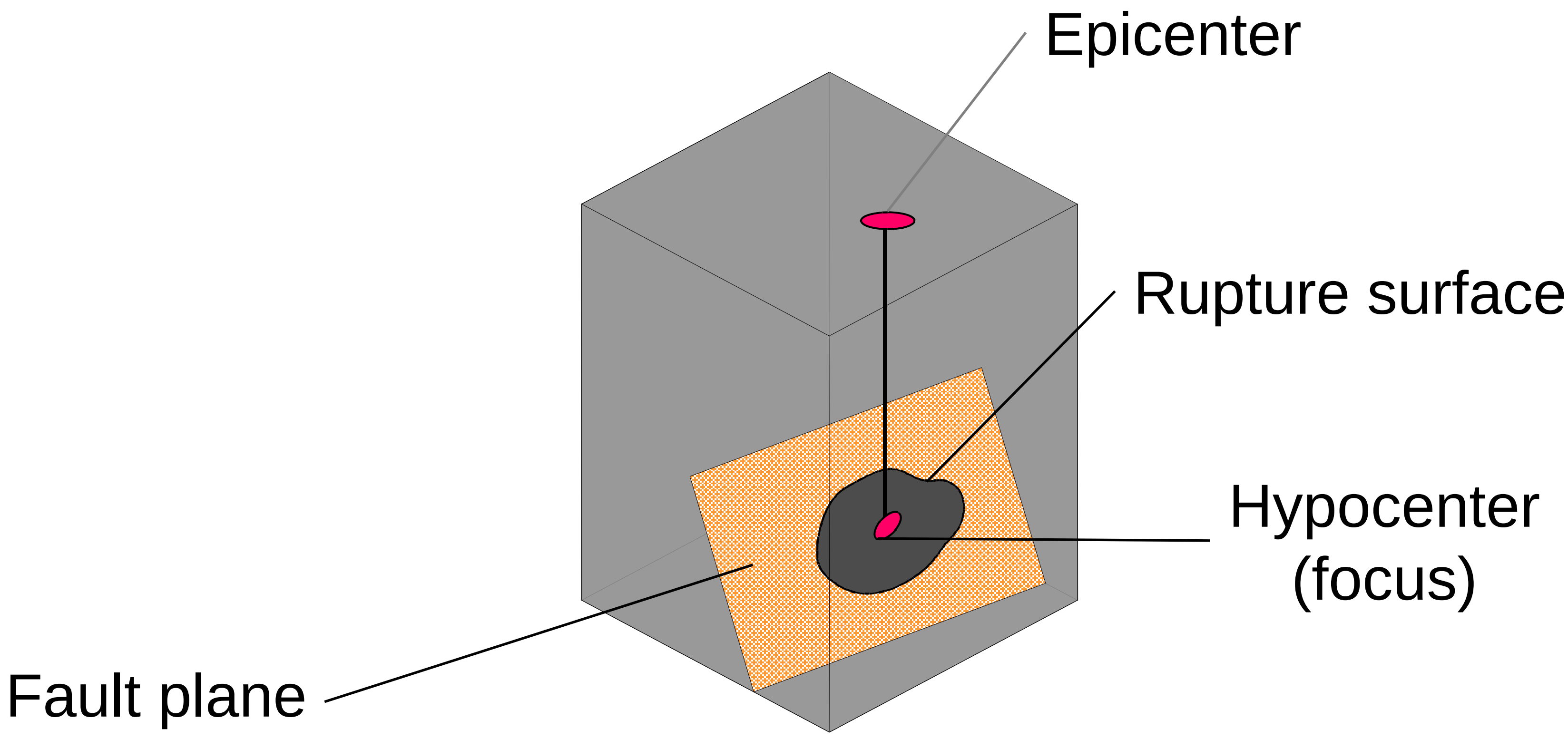
Seismicity of North America



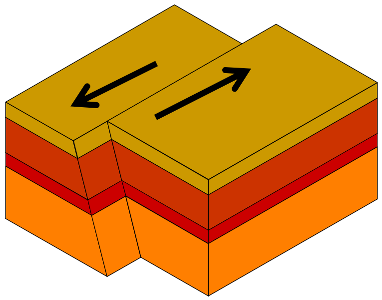
Seismicity of Alaska



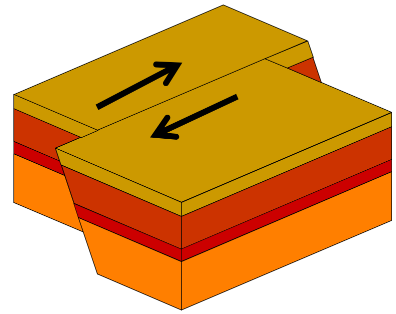
Faults and Fault Rupture



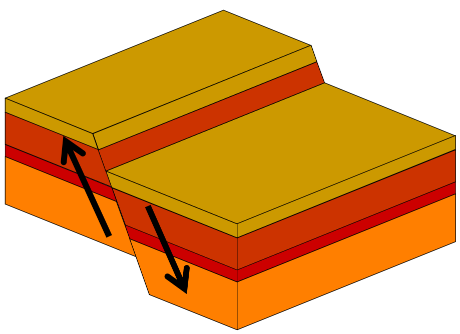
Types of Faults



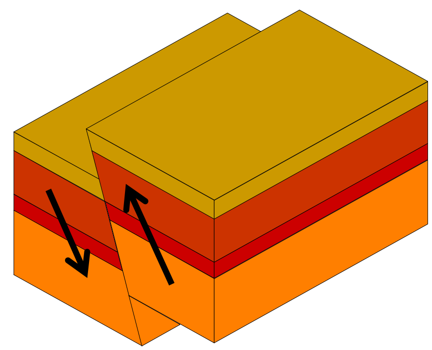
Strike slip
(left lateral)



Strike slip
(right lateral)



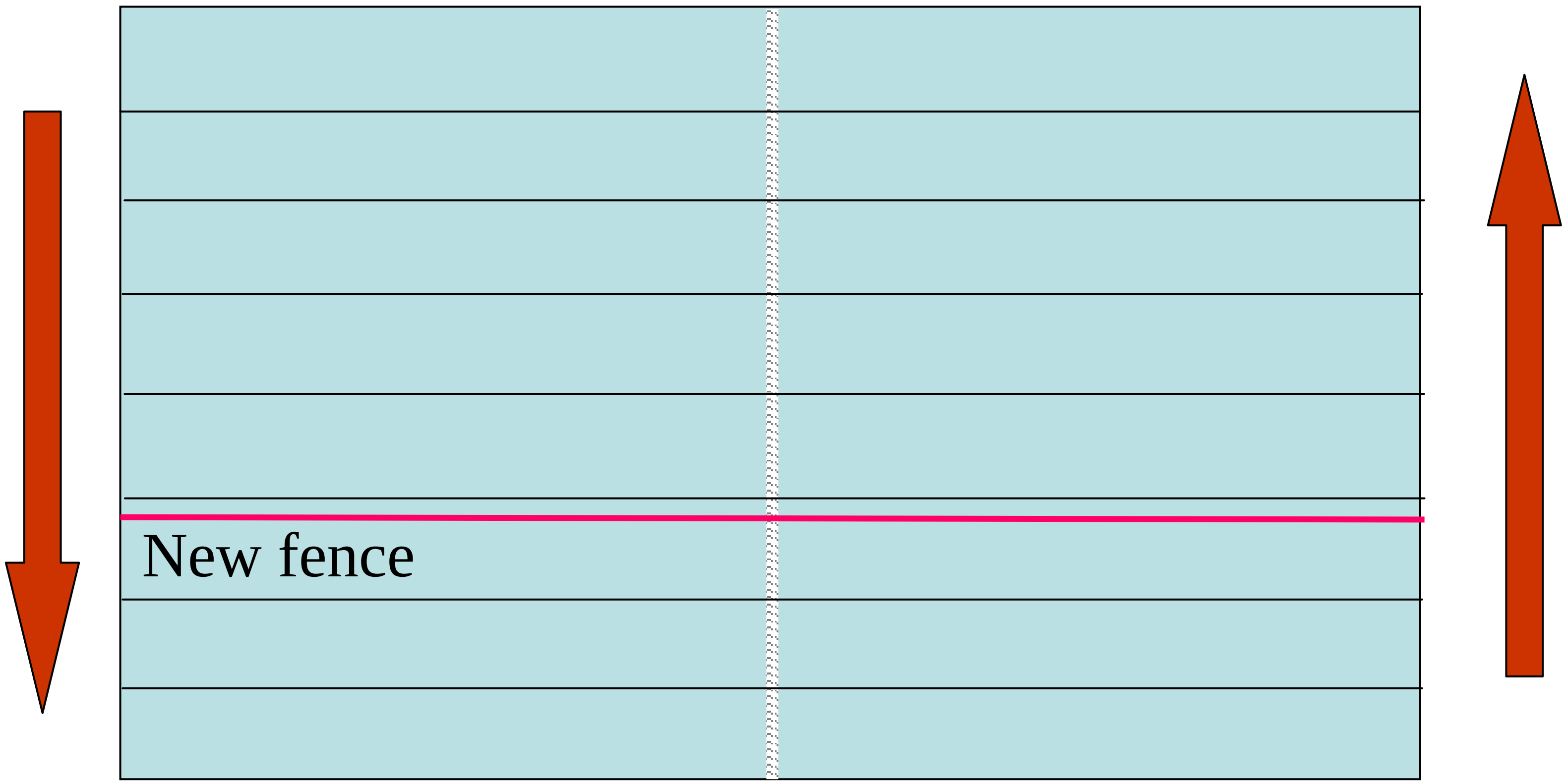
Normal



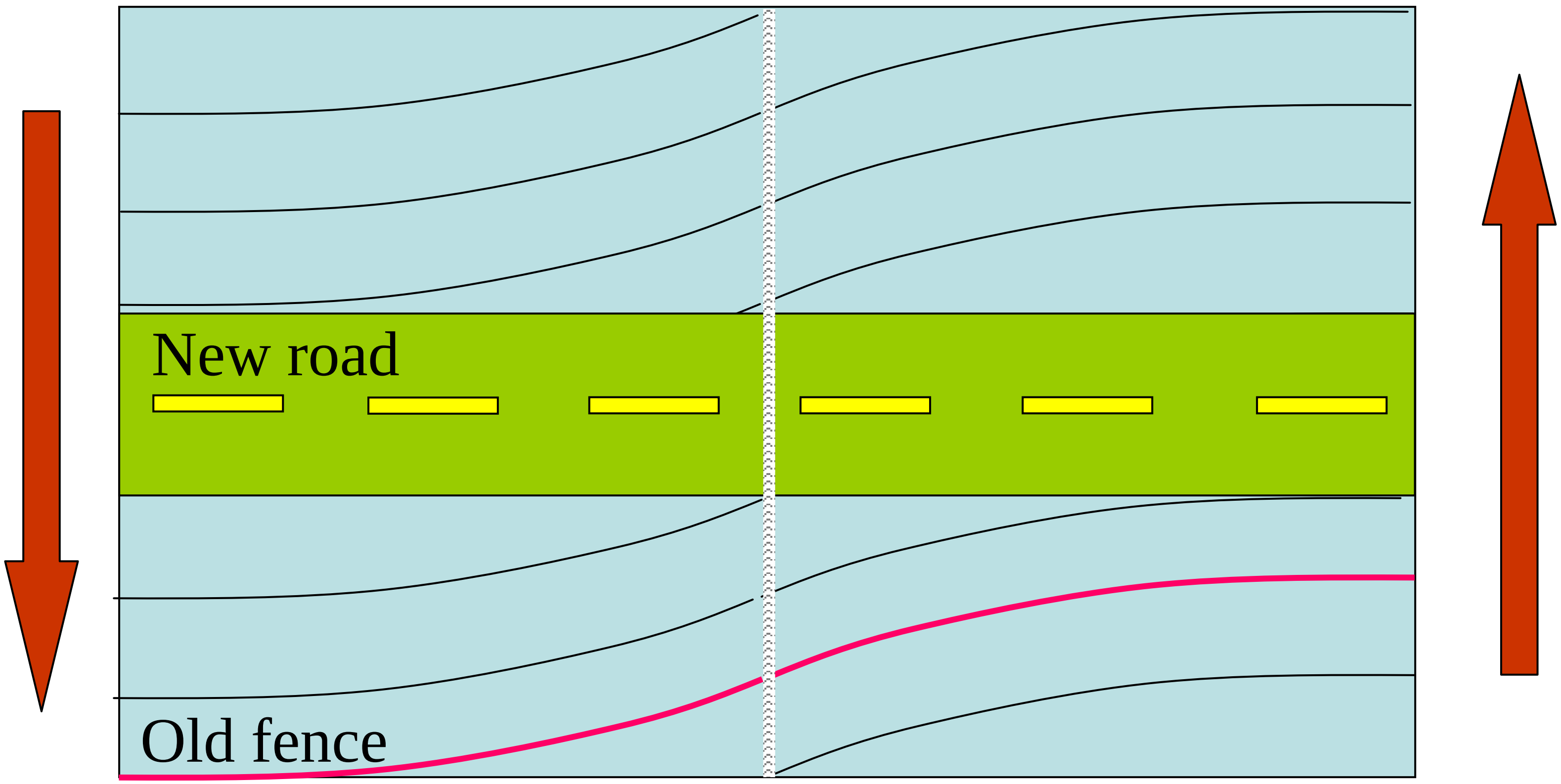
Reverse (thrust)

Elastic Rebound Theory

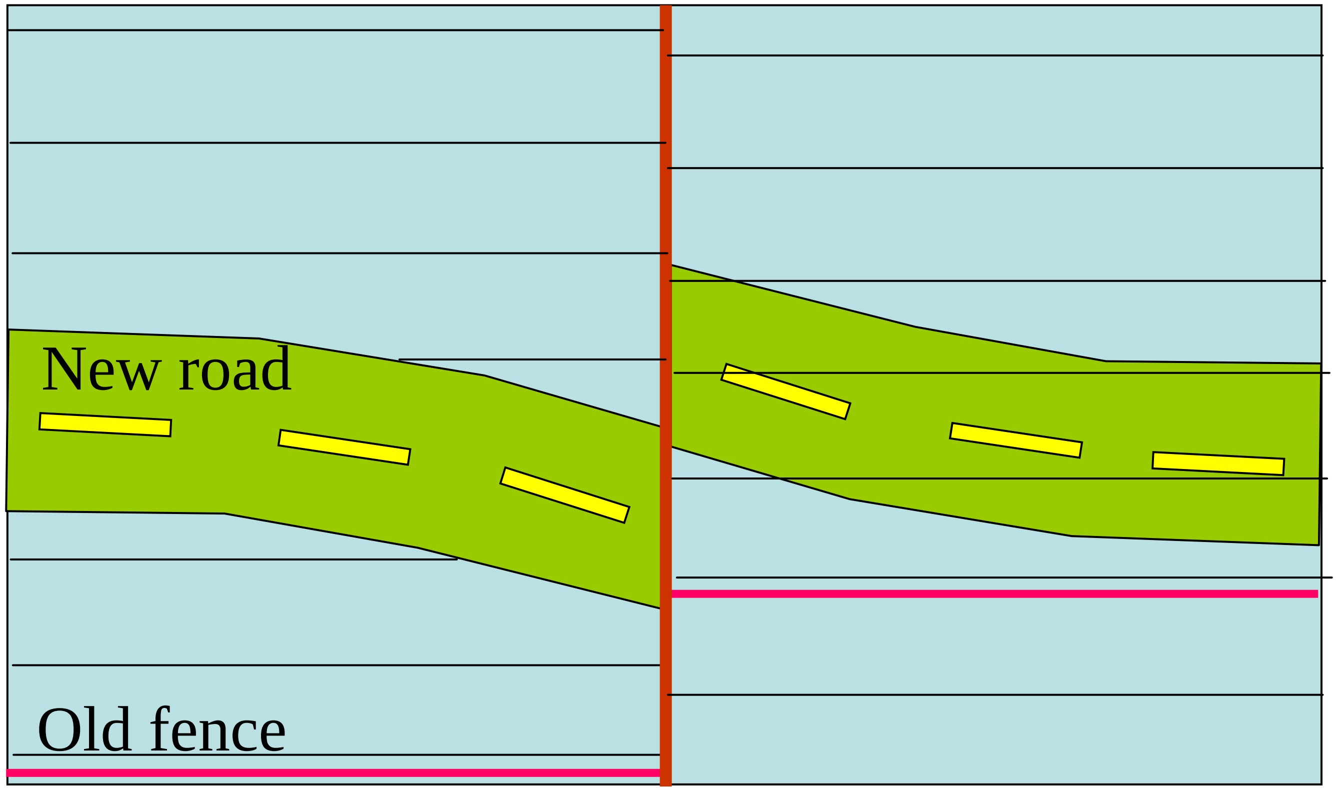
Time = 0 Years

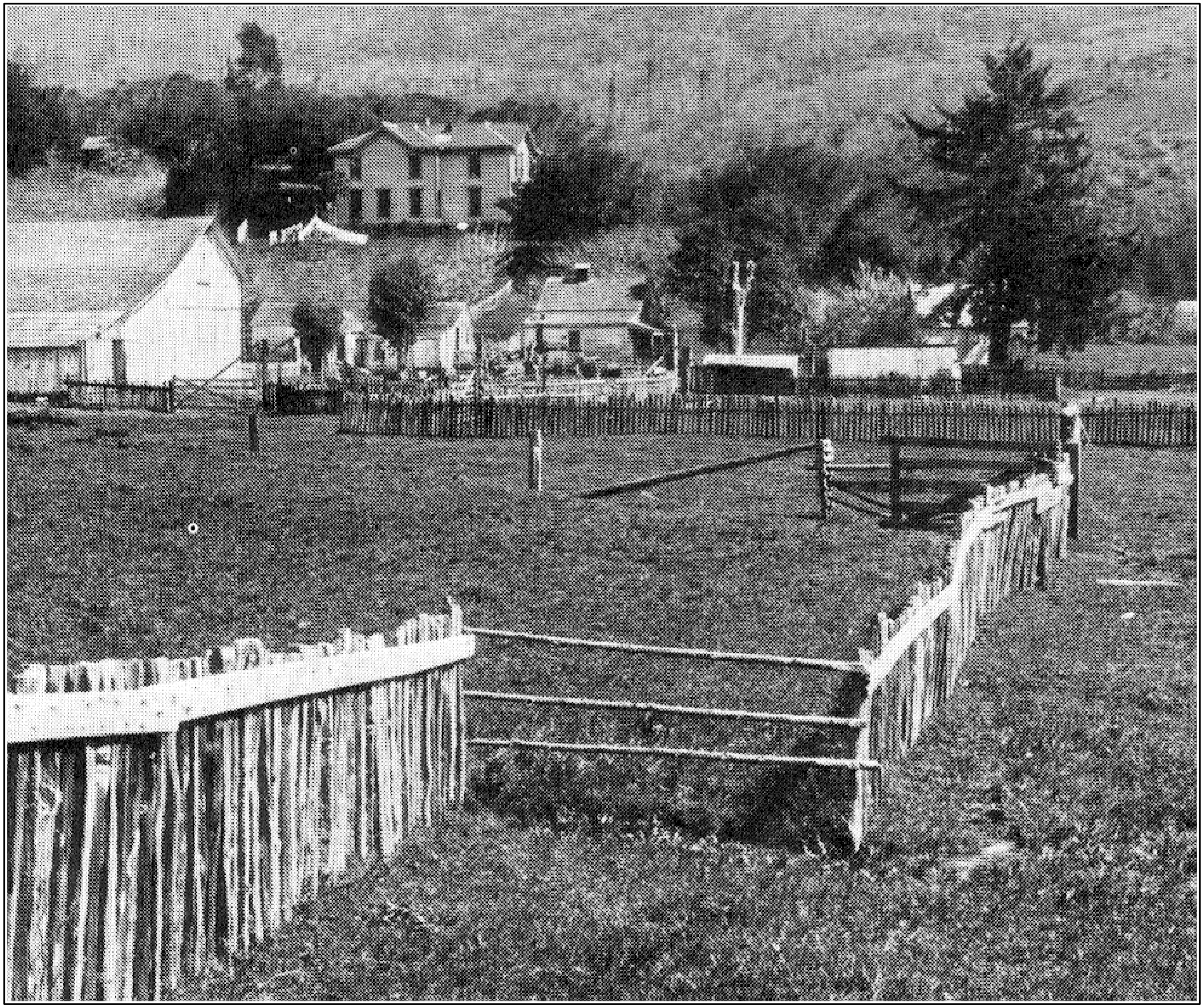


Time = 40 Years

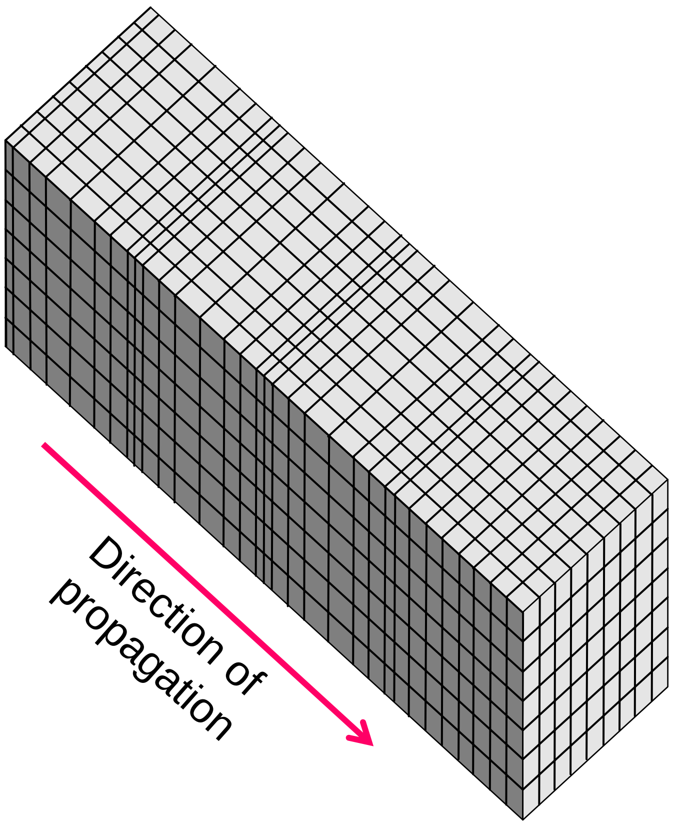


Time = 41 Years

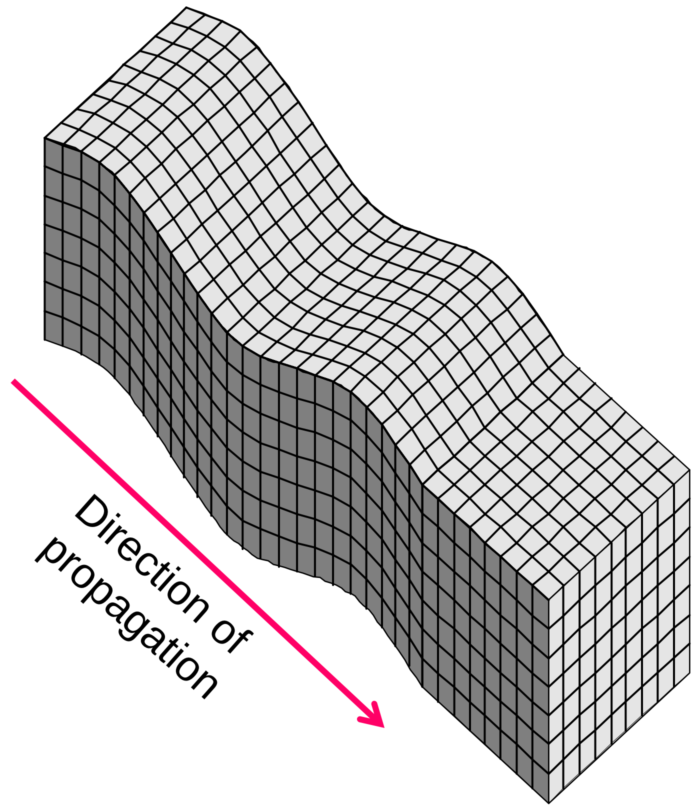




Seismic Wave Forms
(Body Waves)

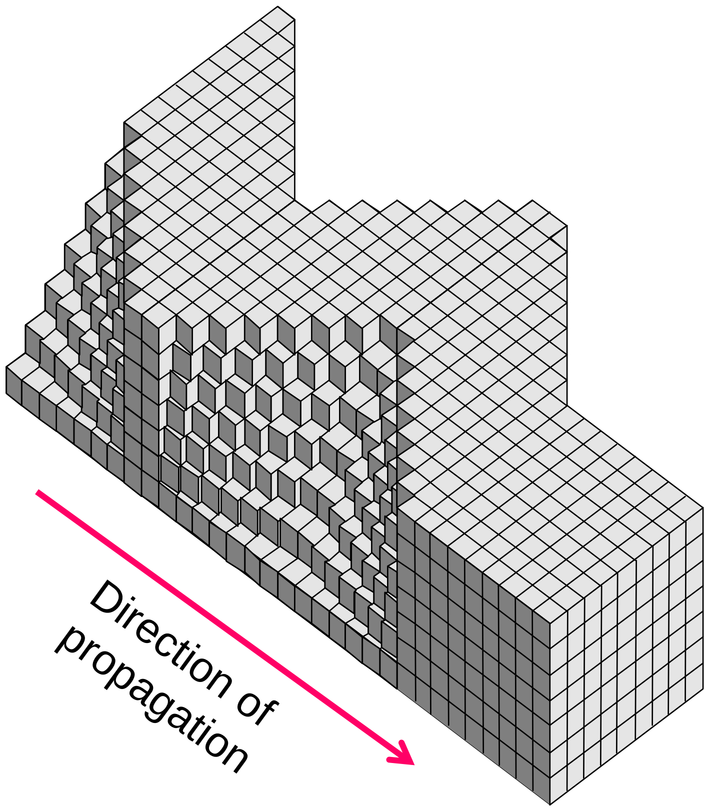


Compression wave
(P wave)

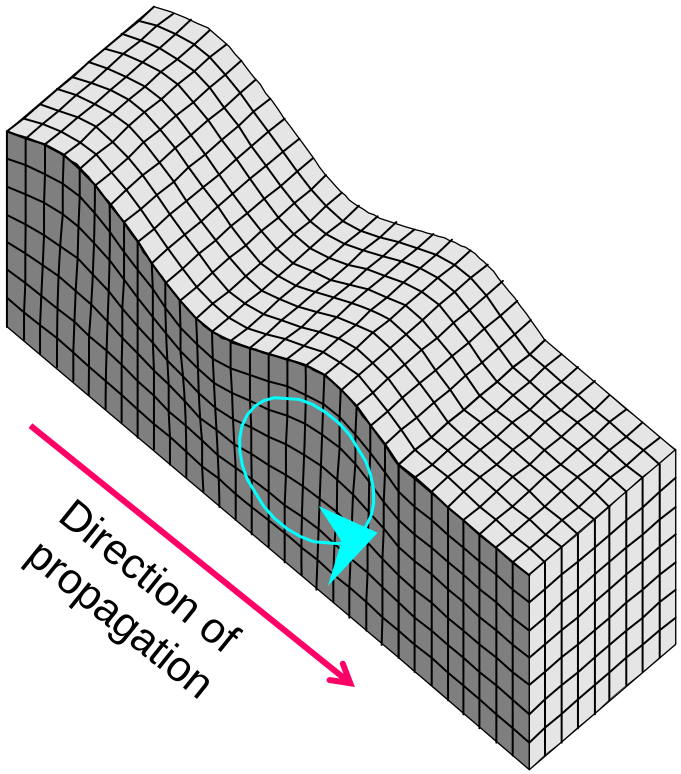


Shear wave
(S wave)

Seismic Wave Forms (Surface Waves)

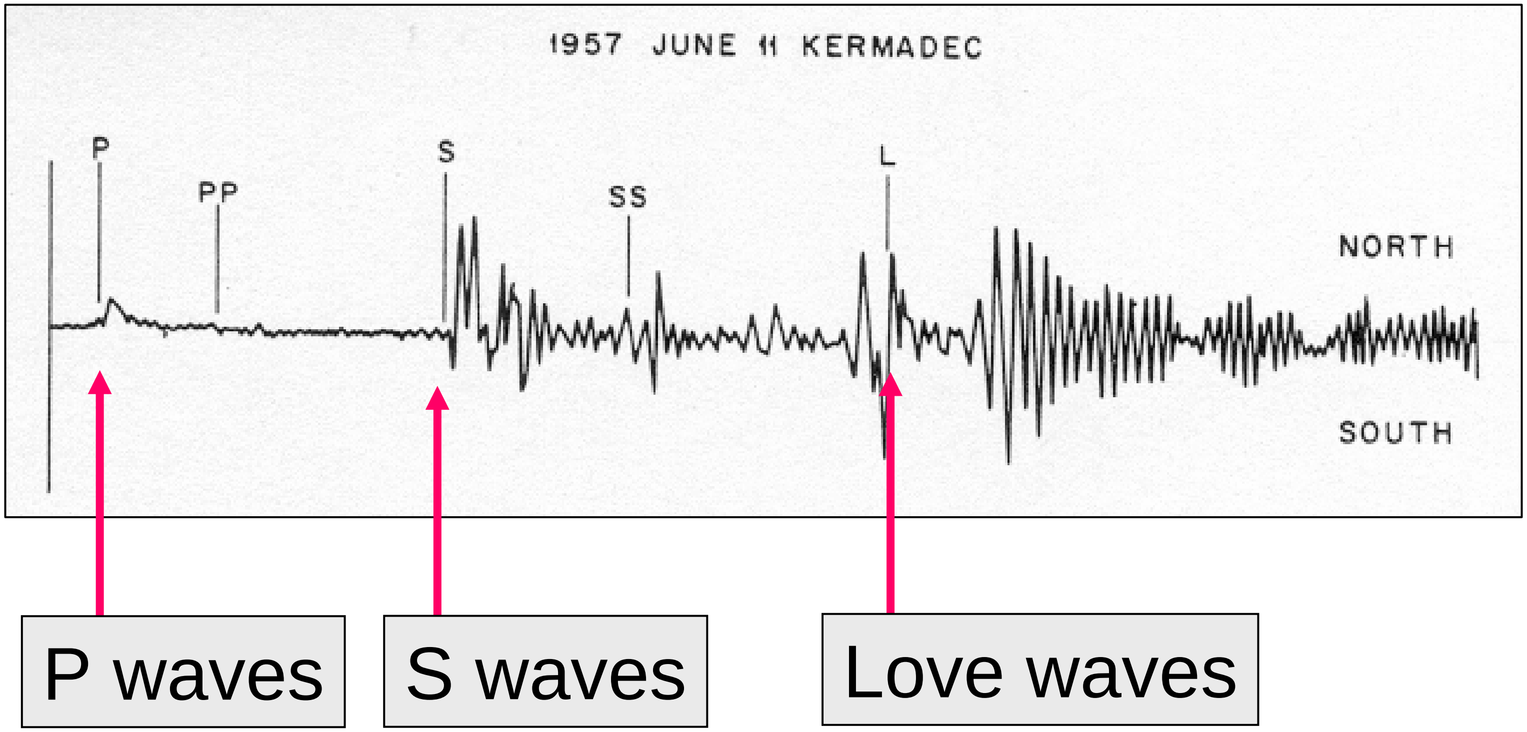


Love wave

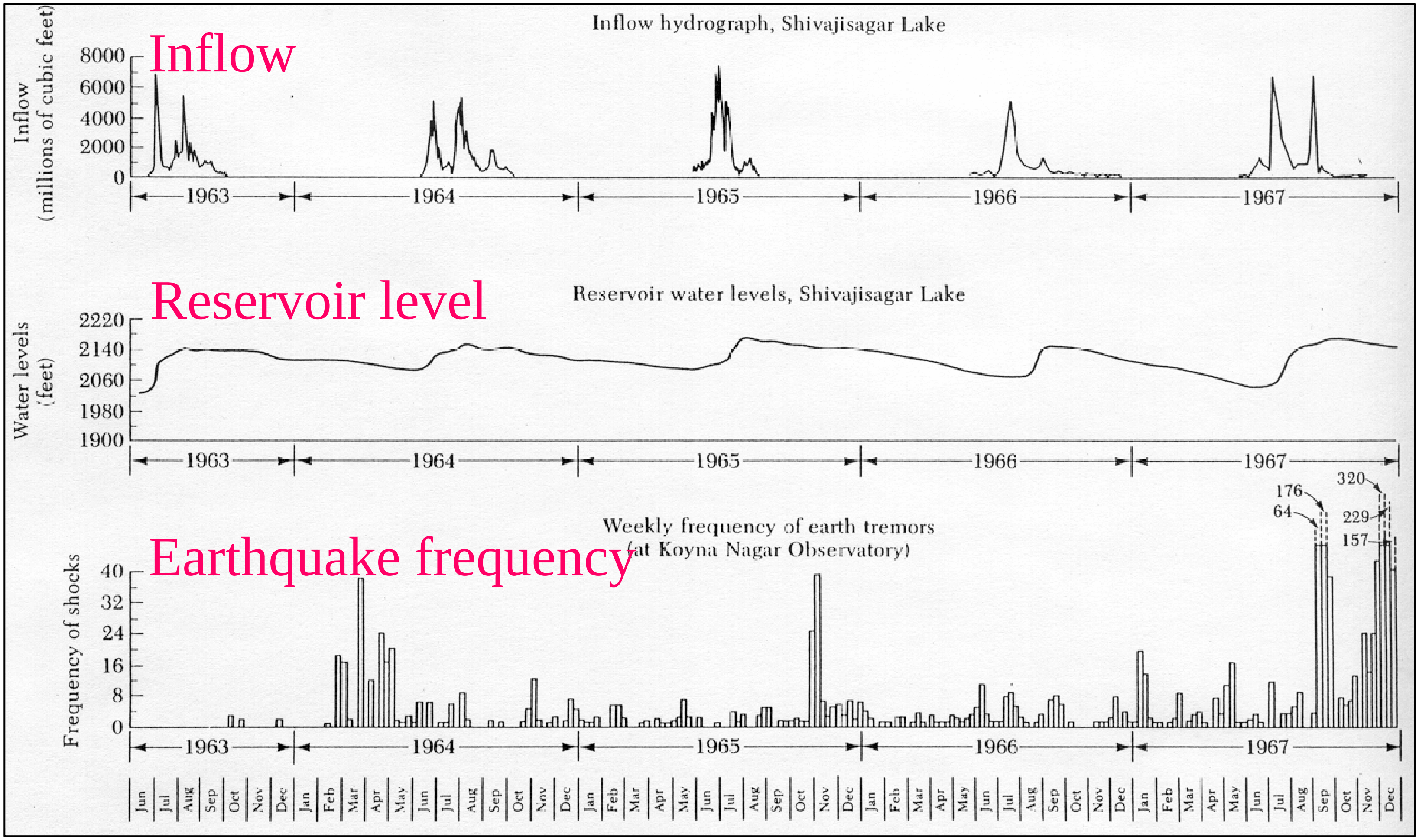


Rayleigh wave

Arrival of Seismic Waves



Relationship Between Reservoir Level and Seismic Activity at Koyna Dam, India



Effects of Seismic Waves

- Fault rupture
- Ground shaking
- Landslides
- Liquefaction
- Tsunamis
- Seiches

Surface Fault Rupture, 1971 Earthquake in San Fernando, California

Cause of Liquefaction

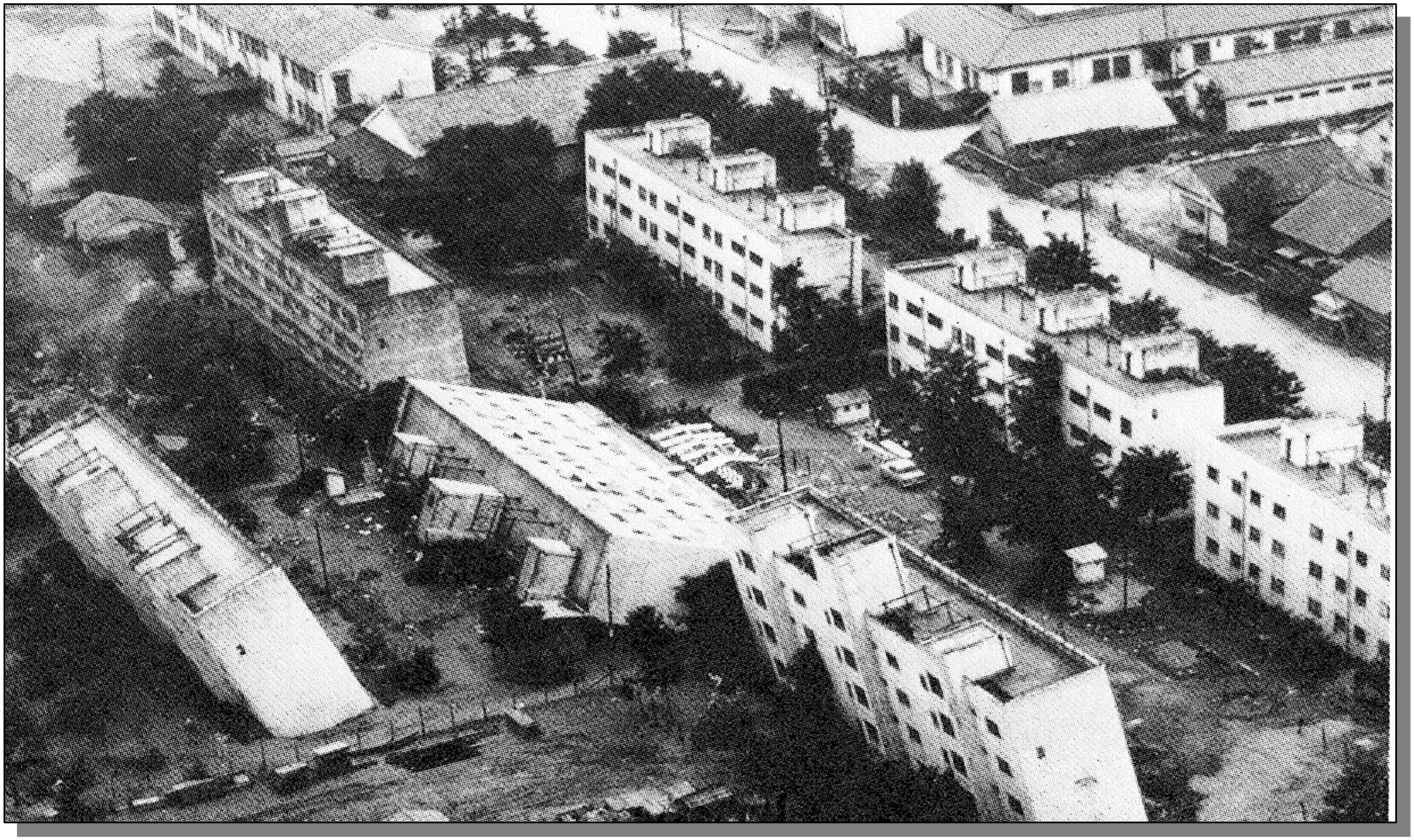
"If a saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume.

If drainage is unable to occur, the tendency to decrease in volume results in an increase in pore pressure.

If the pore water pressure builds up to the point at which it is equal to the overburden pressure, the effective stress becomes zero, the sand loses its strength completely, and liquefaction occurs."

Seed and Idriss (1971)

Liquefaction Damage, Niigata, Japan, 1964



Liquefaction and Lateral Spreading, 1993 Earthquake in Kobe, Japan

Landslide on Coastal Bluff, 1989 Earthquake in Loma Prieta, California

Cause of Tsunamis

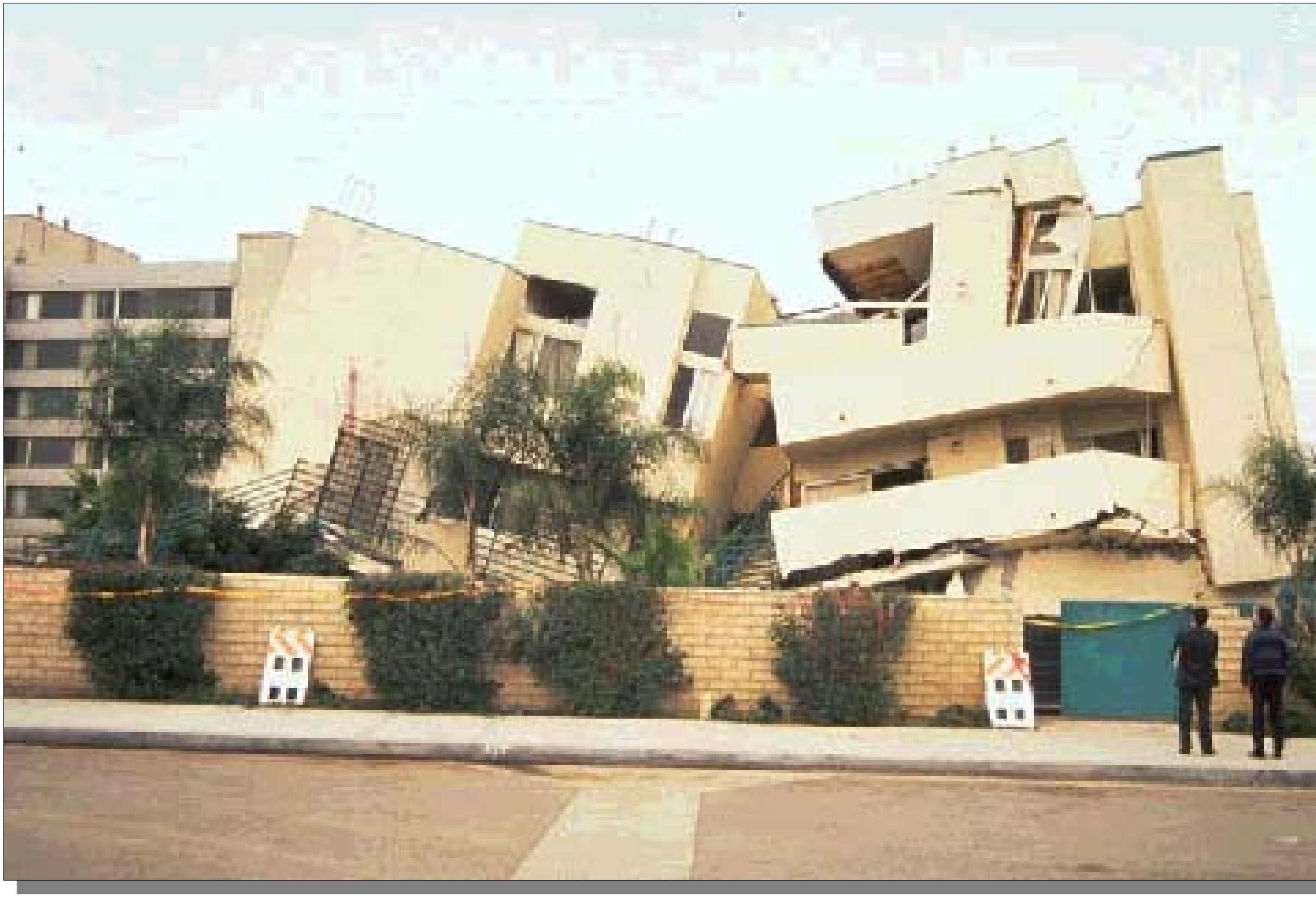
Tsunamis are created by a sudden vertical movement of the sea floor.

These movements usually occur in subduction zones.

Tsunamis move at great speeds, often 600 to 800 km/hr.

Tsunami Damage, Seward, Alaska, 1964

Result of Ground Shaking, 1994 Earthquake in Northridge, California



Mitigation Strategies

Earthquake effect	Strategy
Fault rupture	Avoid
Tsunami/seiche	Avoid
Landslide	Avoid
Liquefaction	Avoid/resist
Ground shaking	**Resist**

Measuring Earthquakes

INTENSITY

- Subjective
- Used where instruments are not available
- Very useful in historical seismicity

MAGNITUDE

- Measured with seismometers
- Direct measure of energy released
- Possible confusion due to different measures

Modified Mercalli Intensity

- Developed by G. Mercalli in 1902 (after a previous version of M. S. De Rossi in the 1880s)
- Subjective measure of human reaction and damage
- Modified by Wood and Neuman to fit California construction conditions
- Intensity range I (lowest) to XII (most severe)

Modified Mercalli Intensity

I. Not felt except by a few under especially favorable circumstances.

II. Felt only by a few persons at rest, especially on upper floors if buildings. Suspended objects may swing.

III. Felt quite noticeably indoors, especially on upper floors of buildings. Standing automobiles may rock slightly. Vibration like passing truck.

Modified Mercalli Intensity

IV. During the day, felt indoors by many, outdoors by few. At night, some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing automobiles rocked noticeably. [0.015 to 0.02g]

V. Felt by nearly everyone, many awakened. Some dishes and windows broken. Cracked plaster. Unstable objects overturned. Disturbance of trees, poles and other tall objects. [0.03 to 0.04g]

VI. Felt by all. Many frightened and run outdoors. Some heavy furniture moved. Fallen plaster and damaged chimneys. Damage slight. [0.06 to 0.07g]

Modified Mercalli Intensity

VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction, slight to moderate in well built ordinary structures, considerable in poorly built or badly designed structures. Noticed by persons driving cars. [0.10 to 0. 15g]

VIII. Damage slight in specially designed structures, considerable in ordinary construction, great in poorly built structures. Fall of chimneys, stacks, monuments. Sand and mud ejected is small amounts. Changes in well water. Persons driving cars disturbed. [0.25 to 0.30g]

Modified Mercalli Intensity

IX. Damage considerable in specially designed structures, well designed frame structures thrown out of plumb, damage great in substantial buildings with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. [0.50 to 0.55g]

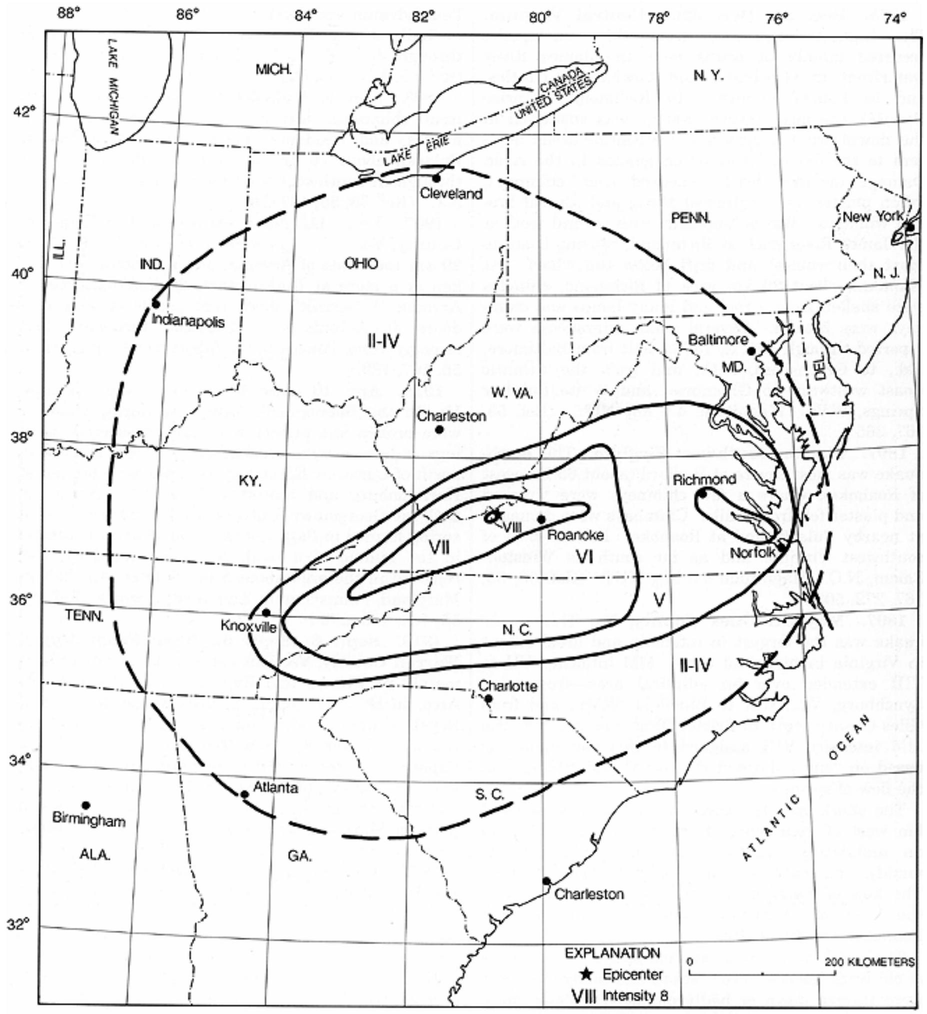
X. Some well built wooden structures destroyed. Most masonry and frame structures destroyed with foundations badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed over banks. [More than 0.60g]

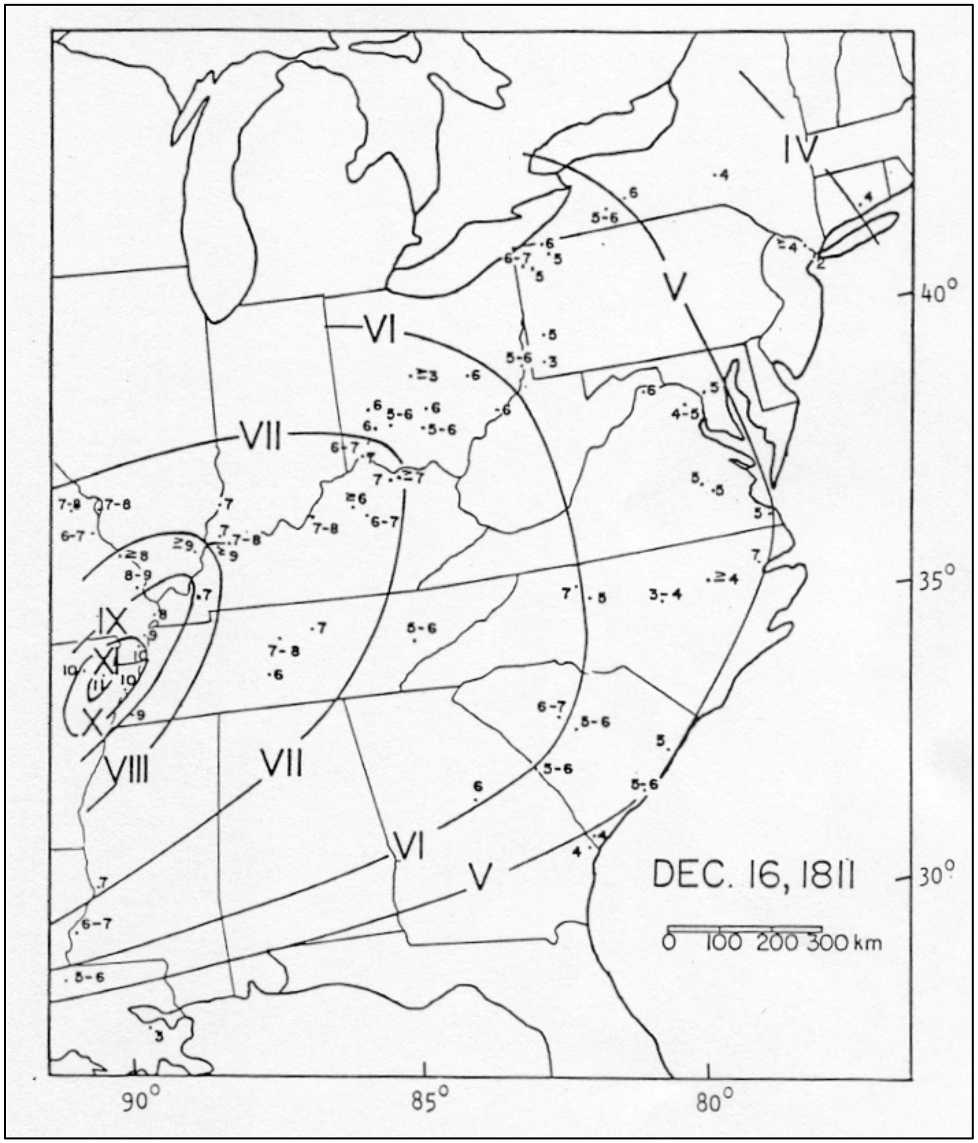
Modified Mercalli Intensity

XI. Few, if any, (masonry) structures left standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

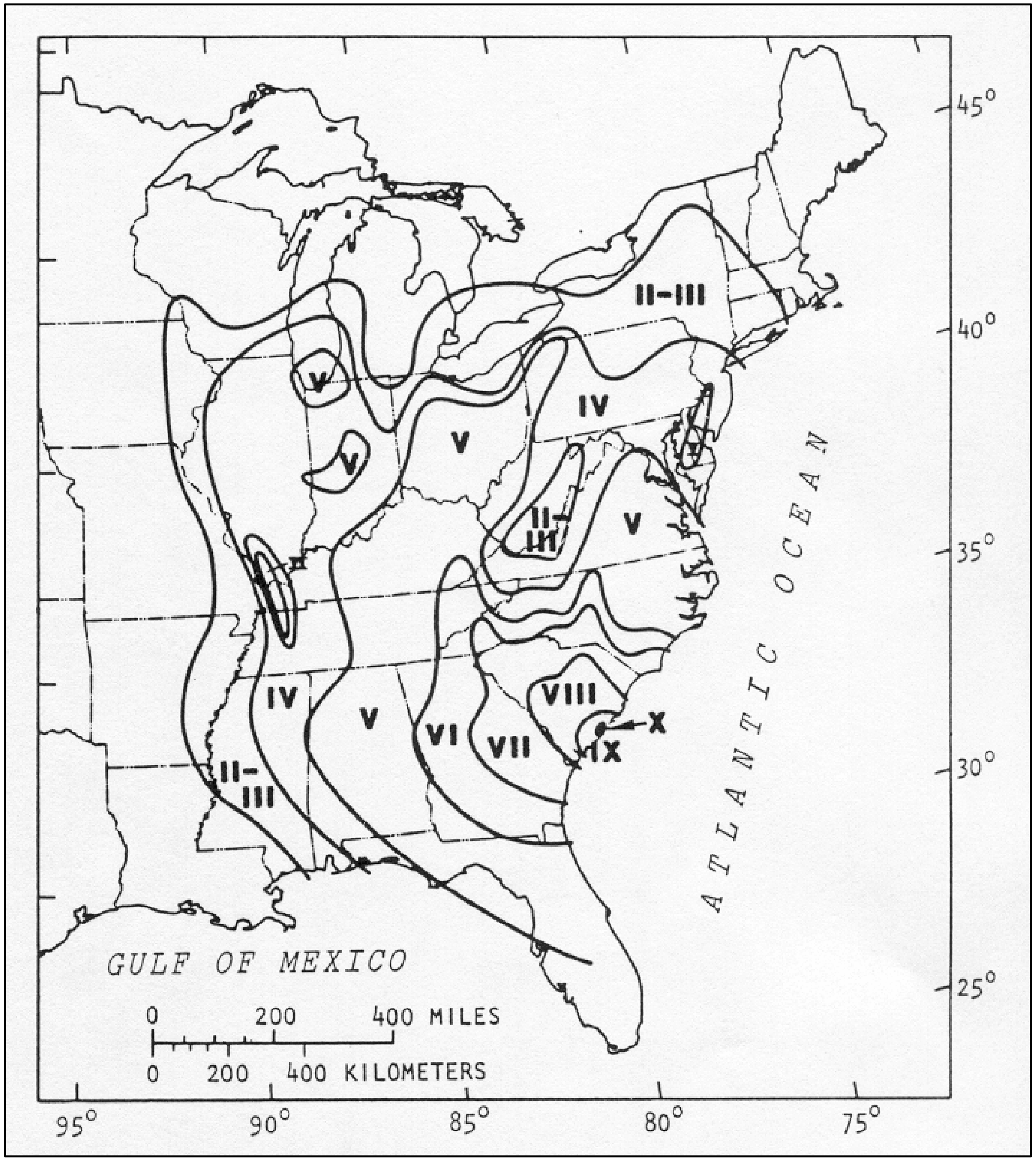
XII. Damage total. Waves seen on ground surface. Lines of sight and level distorted. Objects thrown into air.

Isoseismal Map for the Giles County, Virginia, Earthquake of May 31, 1897.



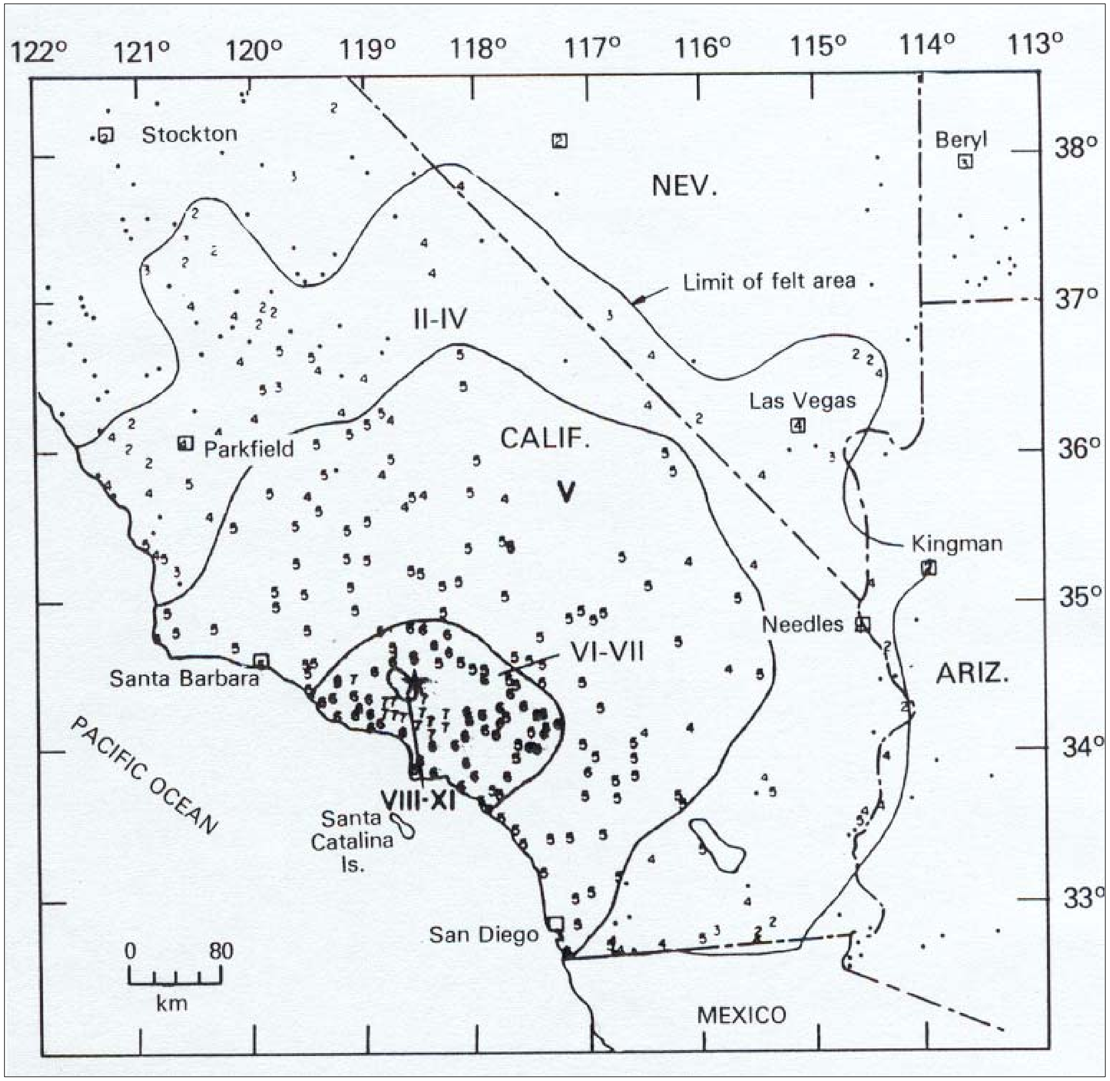


Isoseismal Map For New Madrid Earthquake of December 16, 1811

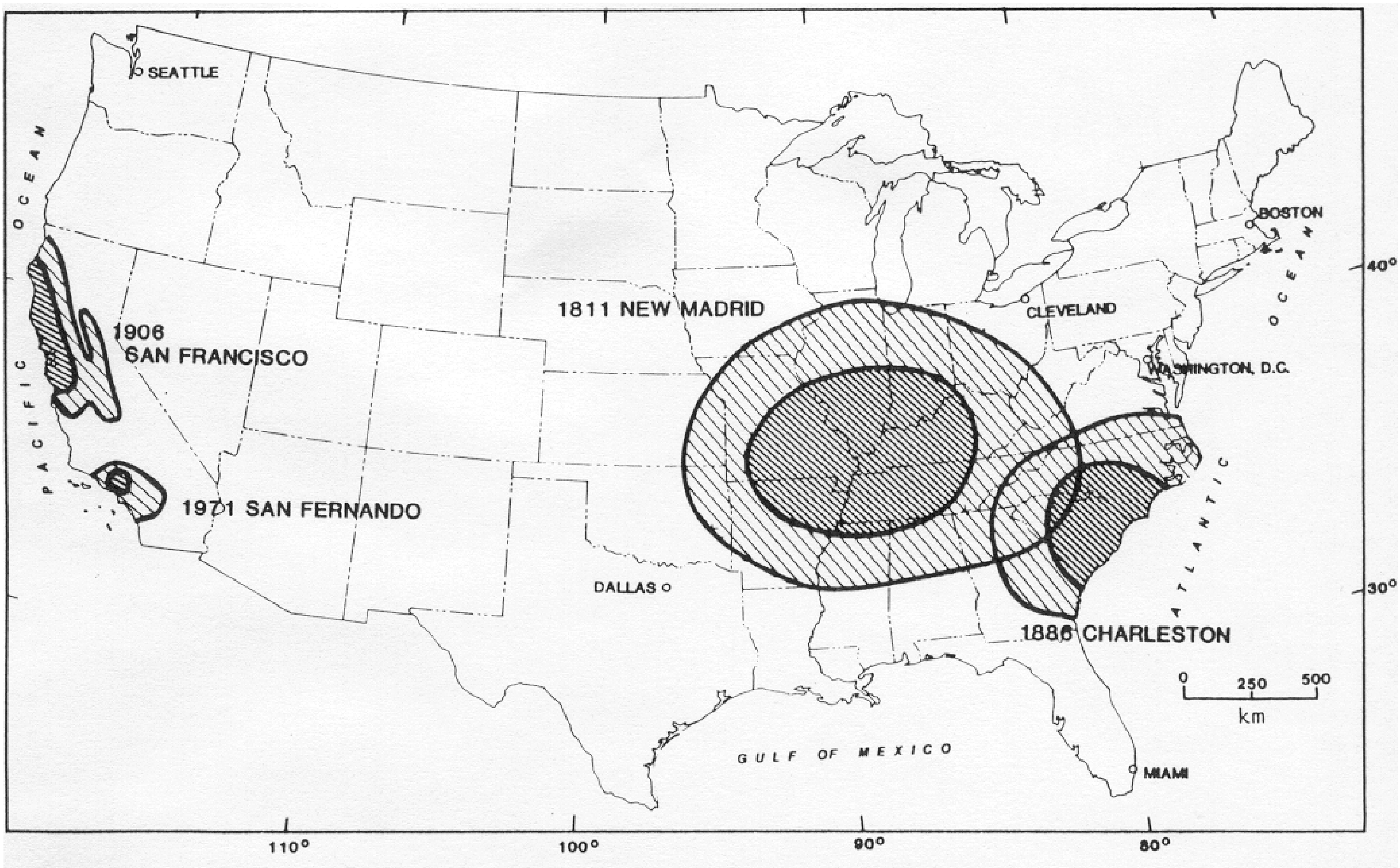


Isoseismal Map for 1886 Charleston Earthquake

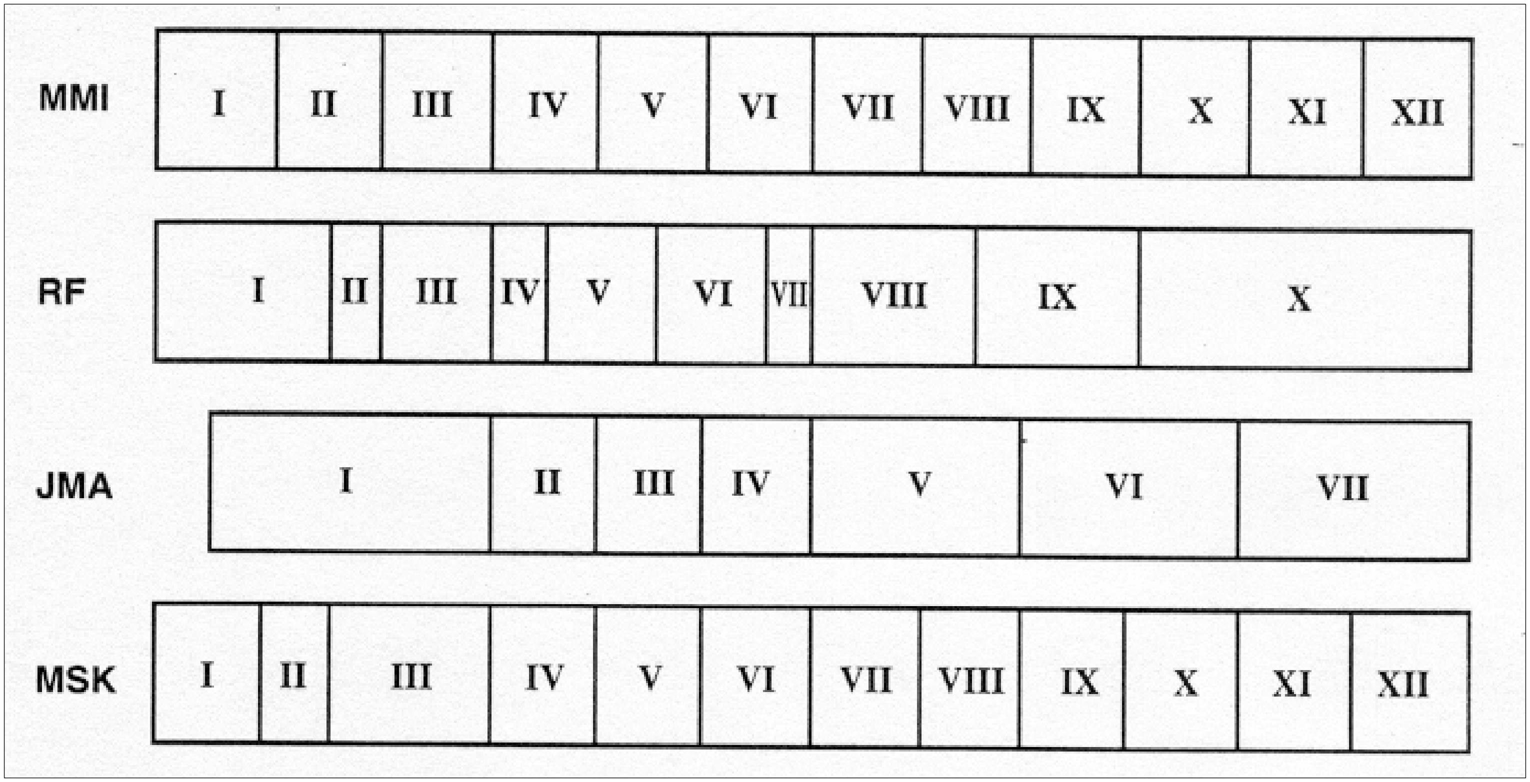
Isoseismal Map for February 9, 1971, San Fernando Earthquake



Comparison of Isosiesmal Intensity for Four Earthquakes



Comparisons of Various Intensity Scales



MMI = Modified Mercalli
RF = Rossi-Forel
JMA = Japan Meteorological Agency
MSK =Medvedez-Spoonheur-Karnik

Instrumental Seismicity

Magnitude (Richter, 1935)

Also called local magnitude

M_L = Log [Maxumum Wave Amplitude (in mm/1000)]

Recorded Wood-Anderson seismograph

100 km from epicenter

Magnitude (in general)

$$M = Log\ A + f(d,h) + C_S + C_R$$

A is wave amplitude

$F(d,h)$ accounts for focal distance and depth

C_S and C_R, are station and regional corrections

Other Wave-Based Magnitudes

M_S Surface-wave magnitude (Rayleigh waves)

m_b Body-wave magnitude (P waves)

M_B Body-wave magnitude (P and other waves)

m_{bLg} (Higher order Love and Rayleigh waves)

M_{JMA} (Japanese, long period)

Moment Magnitude

Seismic moment $= M_O = \mu AD$

[Units = force times distance]

Where:

μ = modulus of rigidity

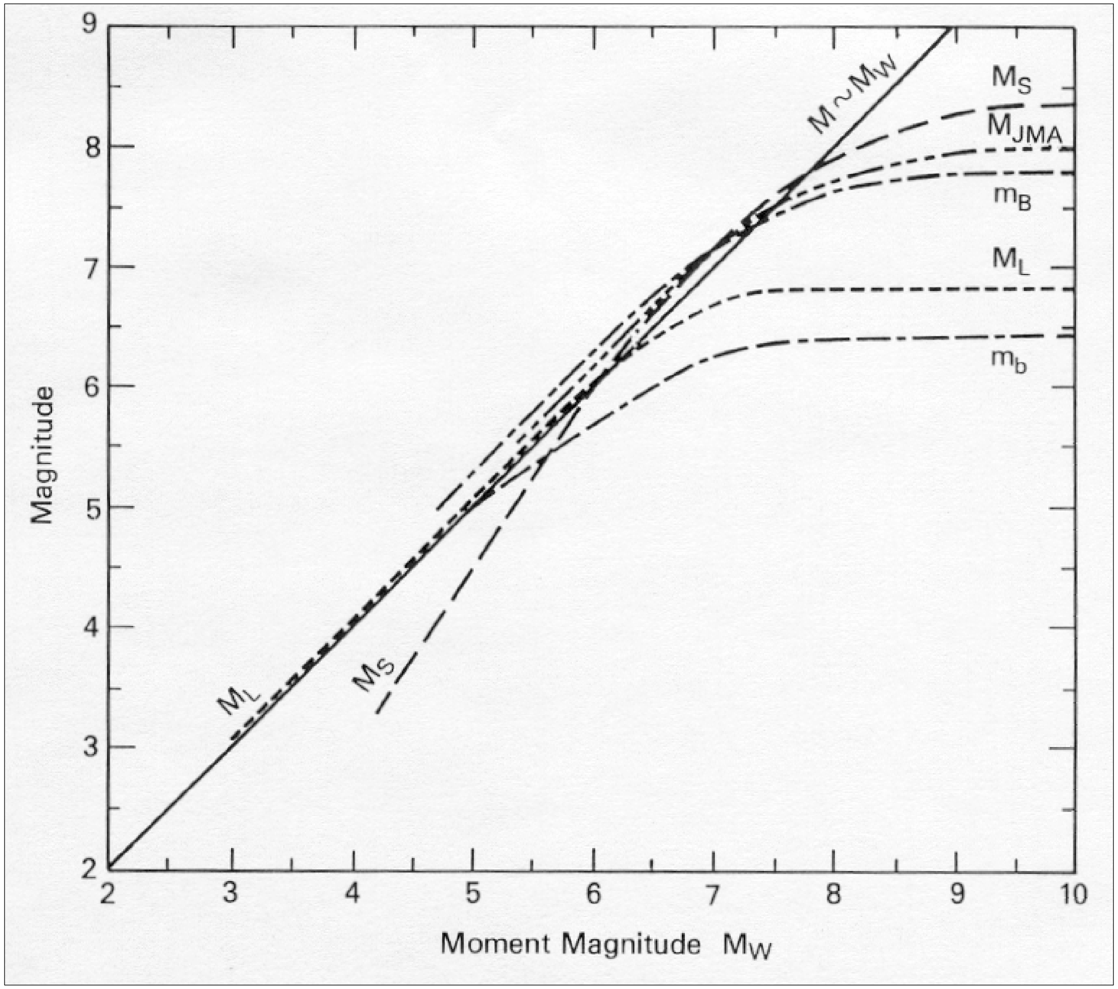
A = fault rupture area

D = fault dislocation or slip

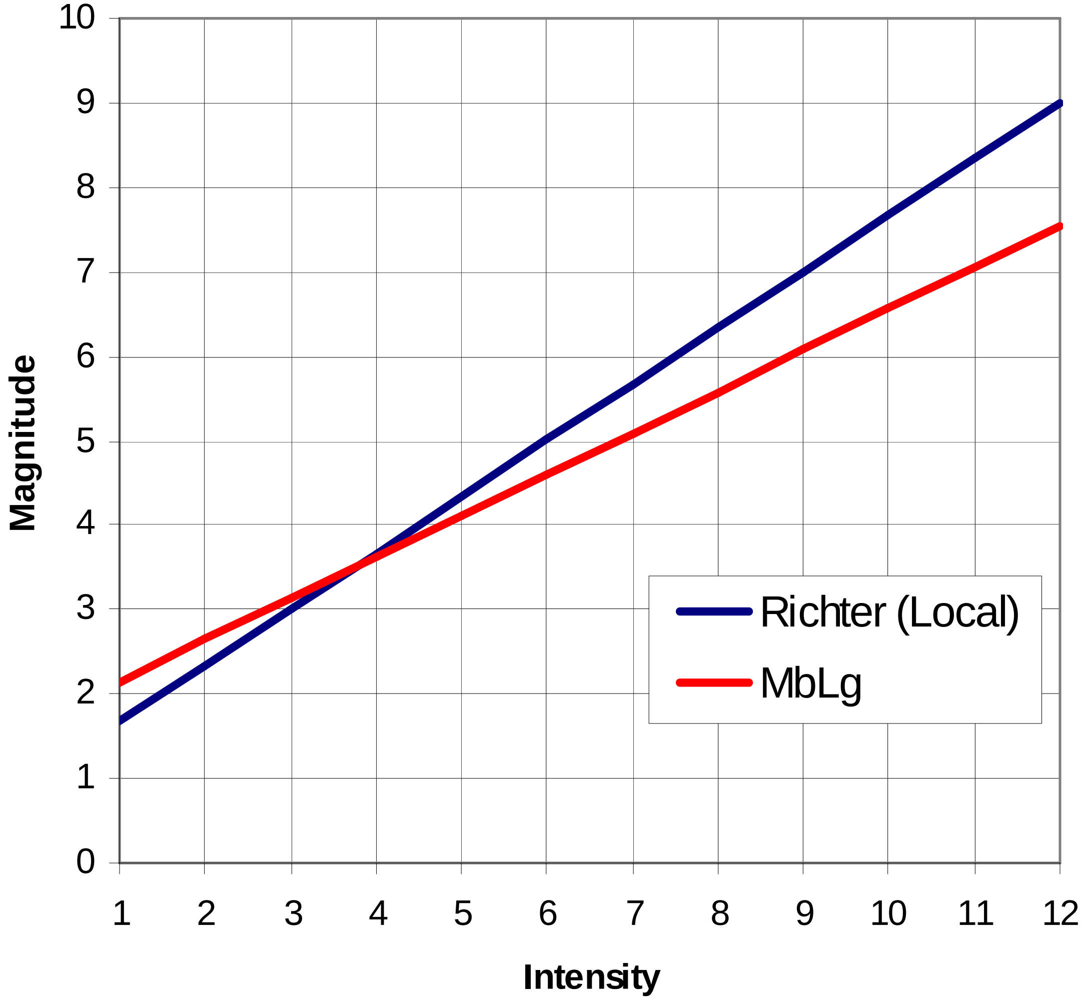
Moment magnitude = $M_W = (\mathit{Log}\ M_O - 16.05)/1.5$

(Units = dyne-cm)

Moment Magnitude vs Other Magnitude Scales (Magnitude Saturation)



Approximate Relationship Between Magnitude and Intensity

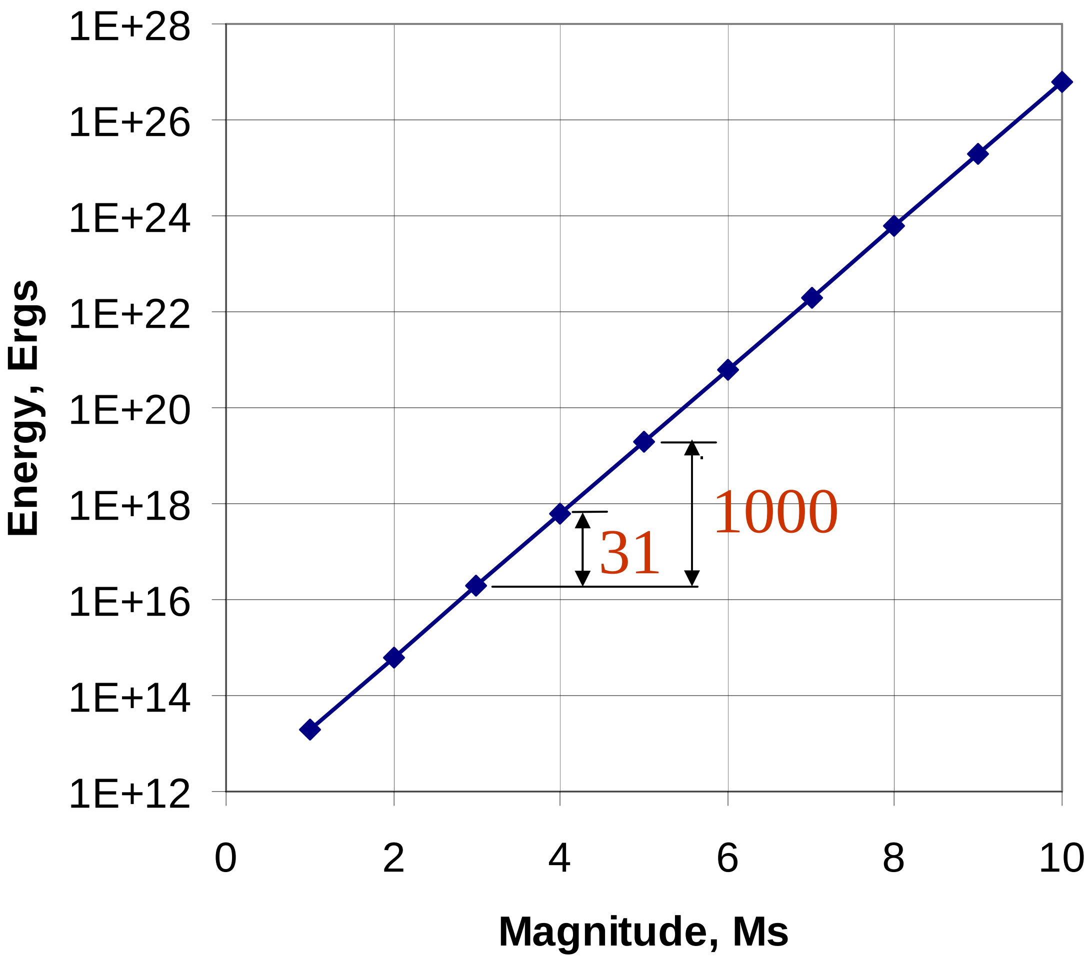


$$\hat{M}_L = 0.67 I_0 + 1$$

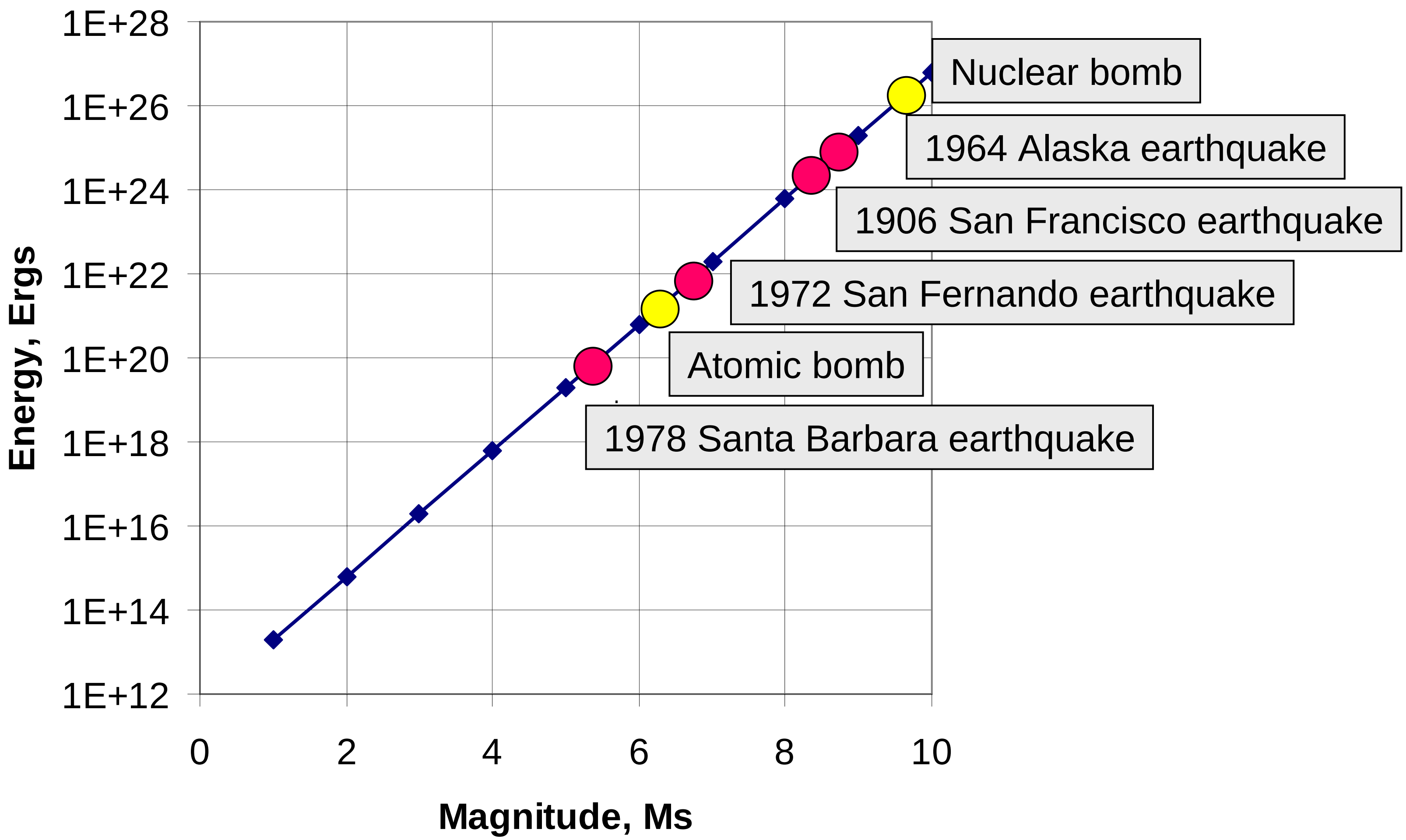
$$\hat{m}_{bLg} = 0.49 I_0 + 1.66$$

Seismic Energy Release

$$\text{Log } E = 1.5\, M_S + 11.8$$



Seismic Energy Release



Ground Motion Accelerograms

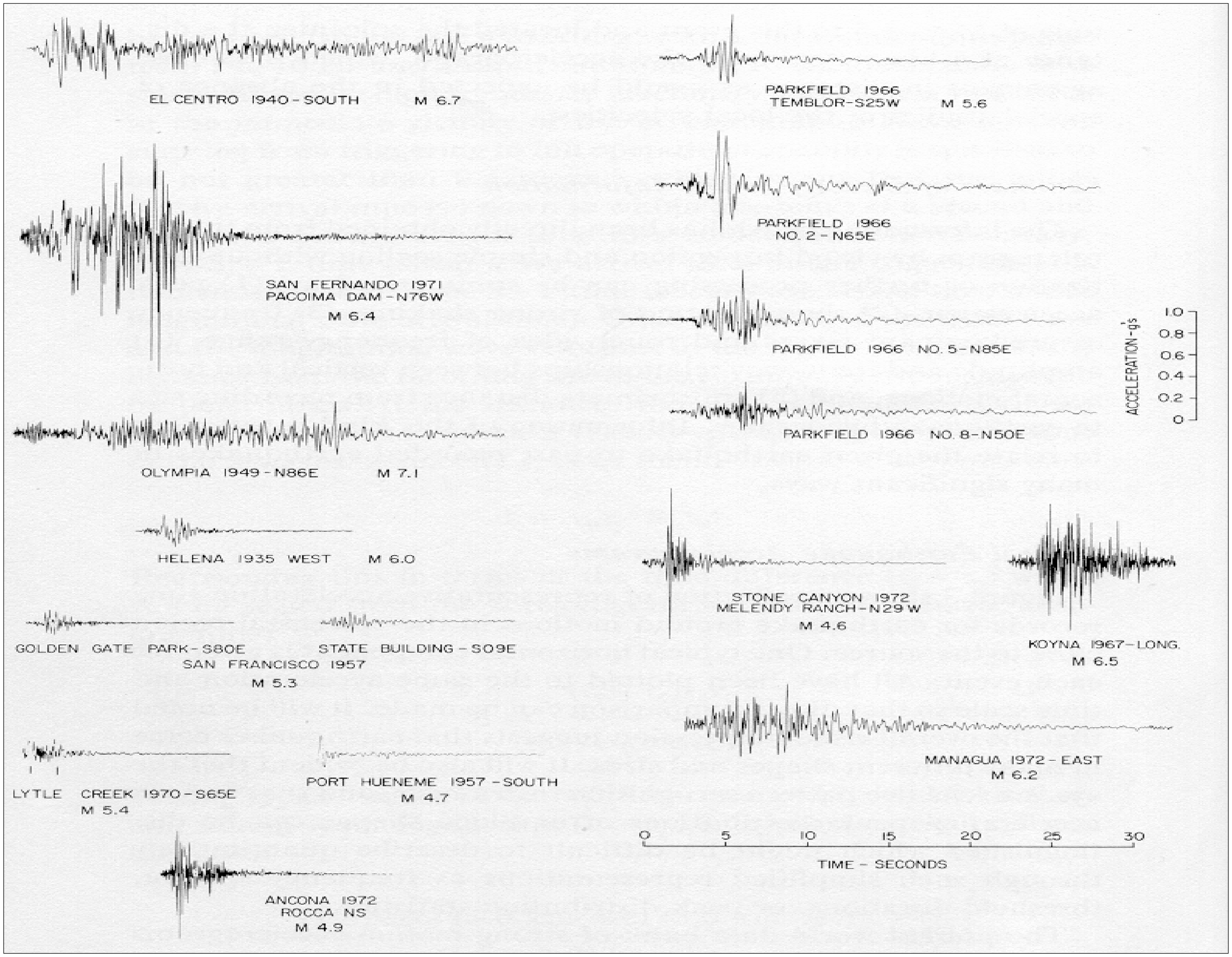
Sources:

- NONLIN (more than 100 records)
- Internet (e.g., National Strong Motion Data Center)
- USGS CD ROM

Uses:

- Evaluation of earthquake characteristics
- Development of response spectra
- Time history analysis

Sample Ground Motion Records



Ground Motion Characteristics

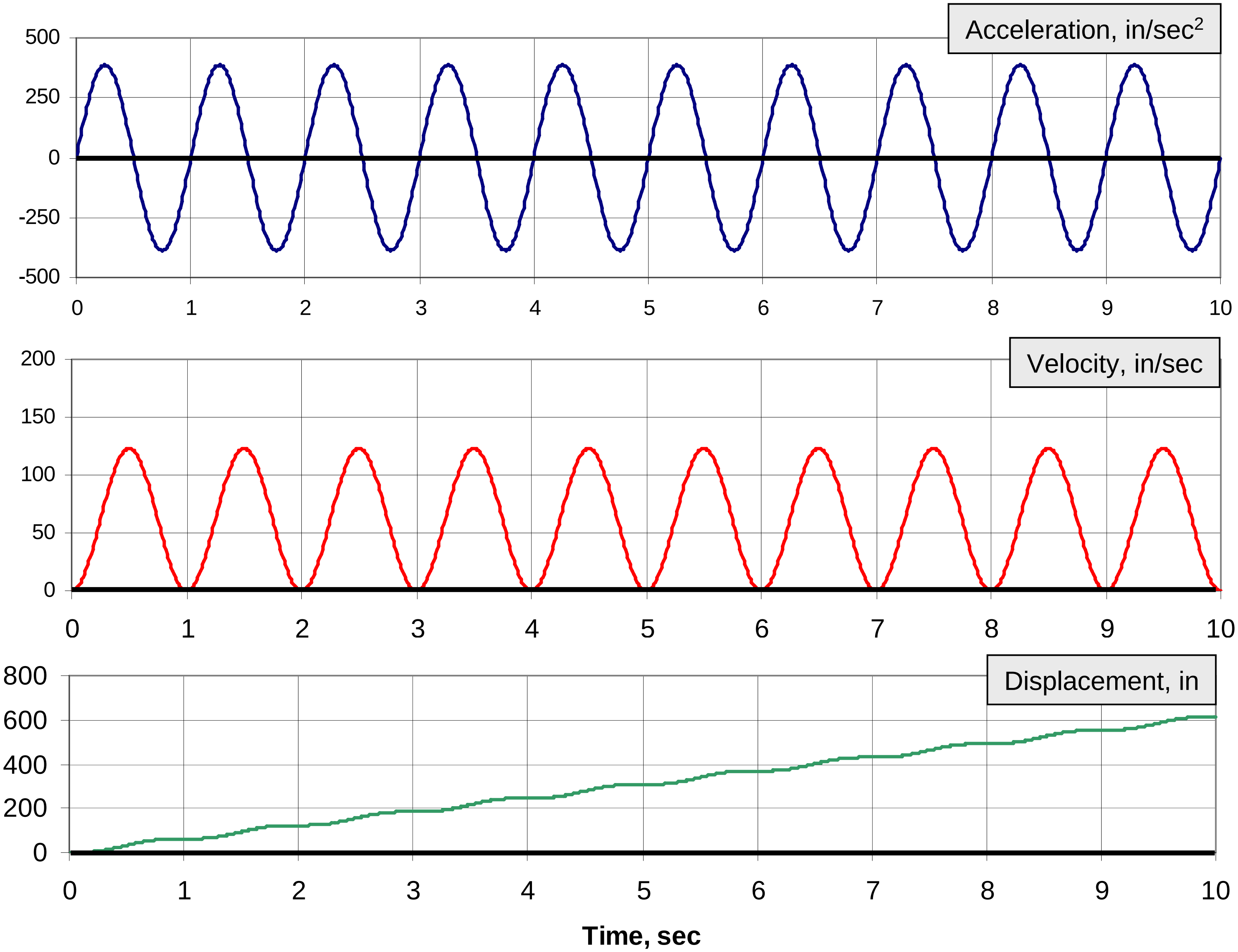
- Acceleration, velocity, displacement
- Effective peak acceleration and velocity
- Fourier amplitude spectra
- Duration (bracketed duration)
- Incremental velocity (killer pulse)
- Response spectra
- Other (see, for example, Naiem and Anderson 2002)

Corrected vs Uncorrected Motions

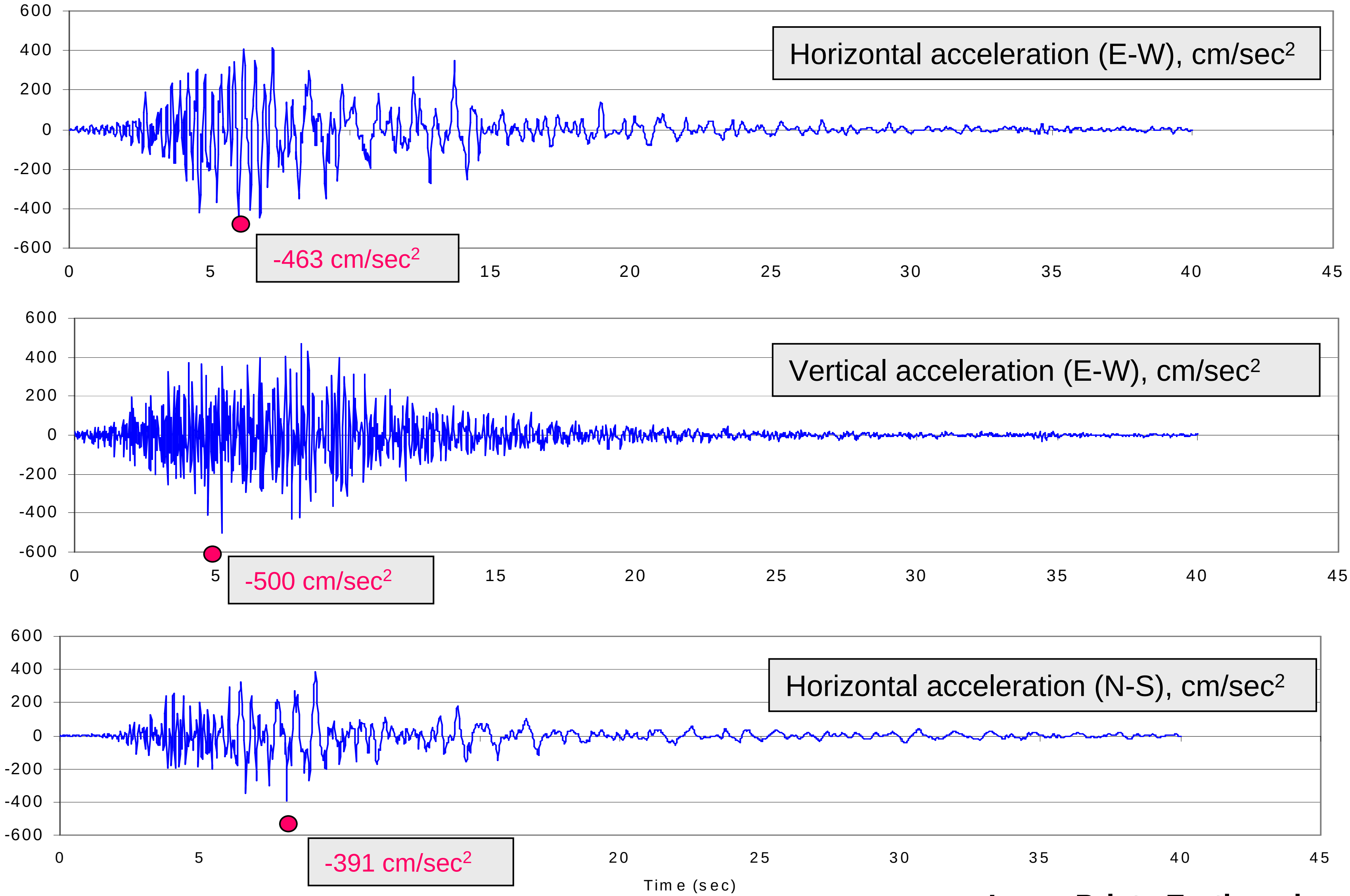
Corrections made primarily:

- To remove instrument response
- To account for base line shift

Base Line Correction for Simple Ground Motion



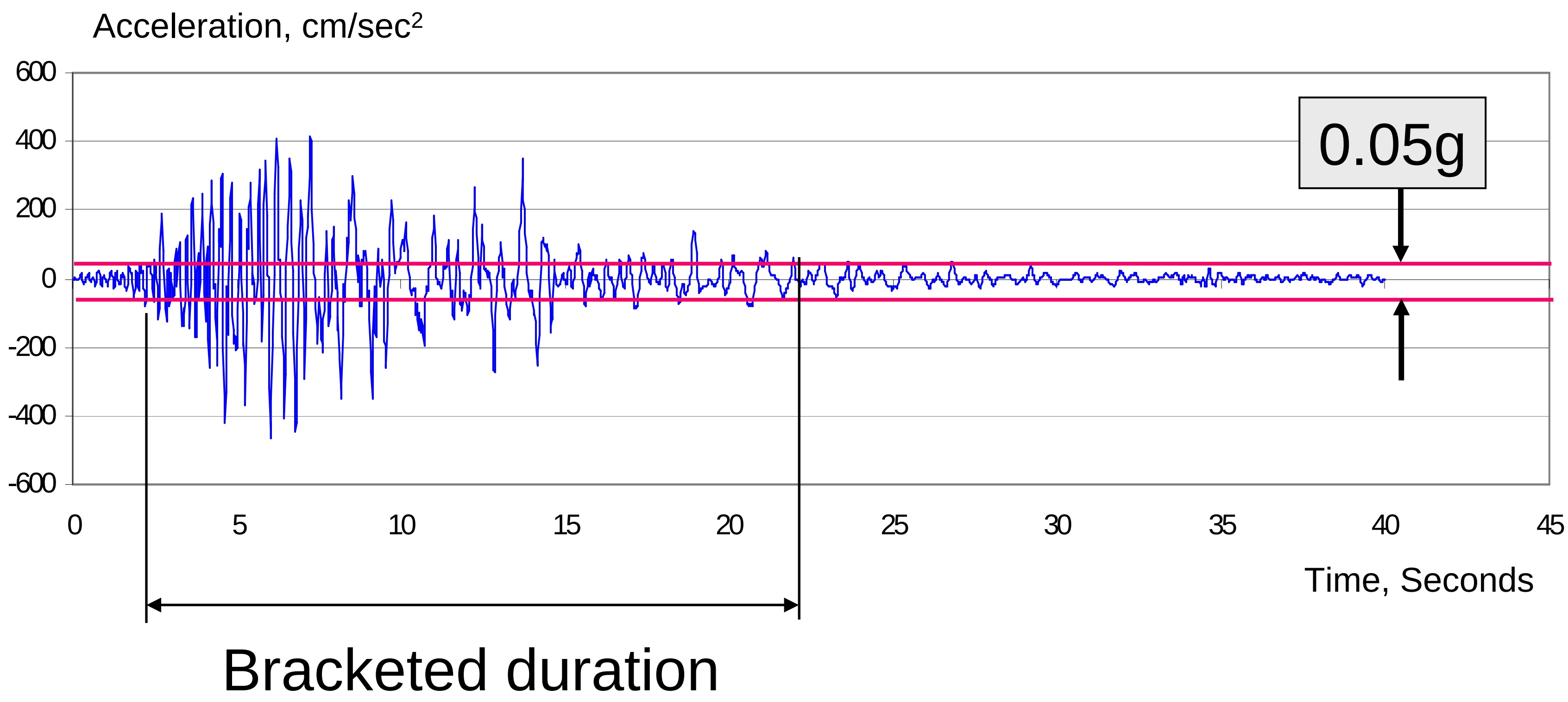
Typical Earthquake Accelerogram Set



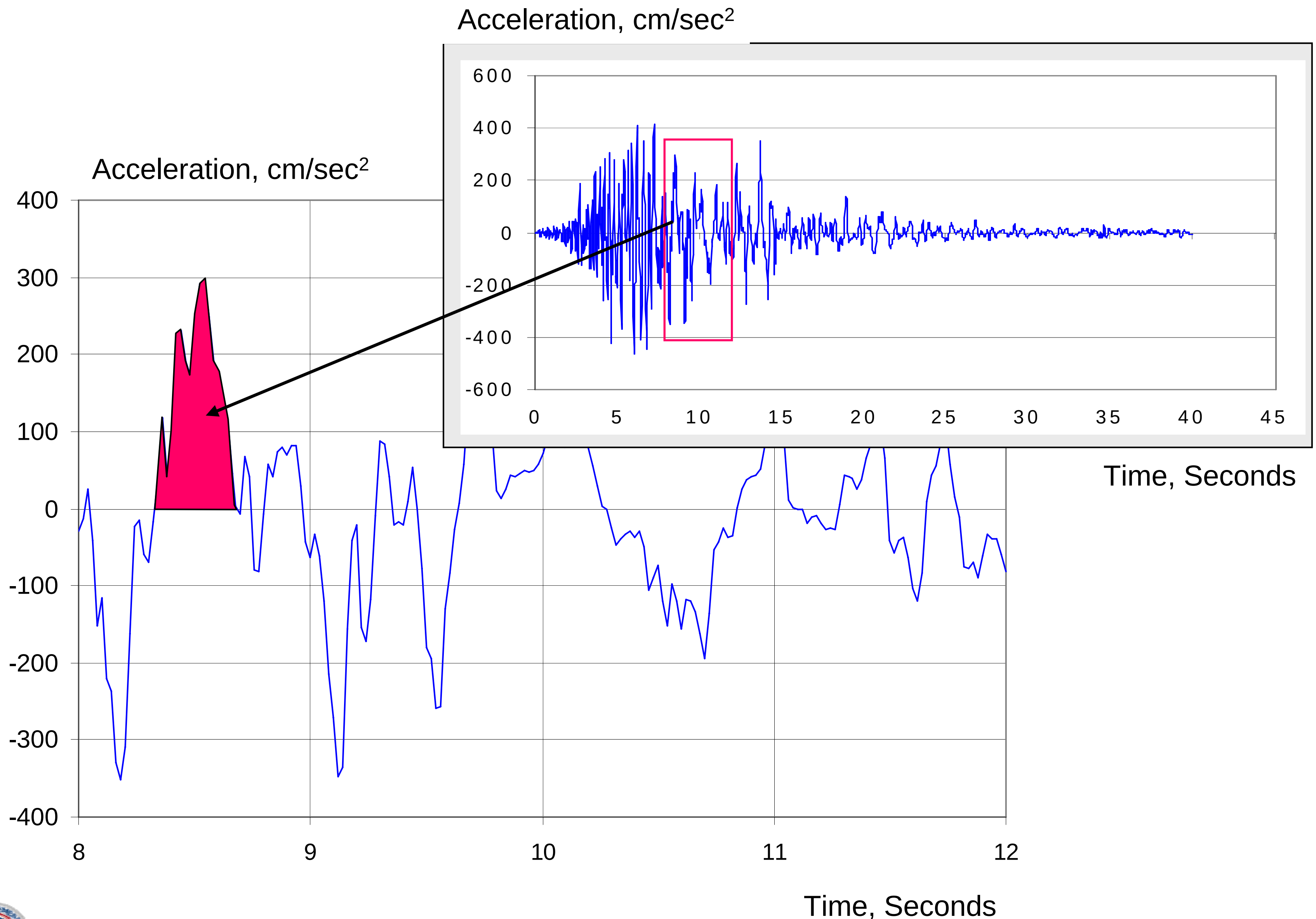
Time, Seconds

Loma Prieta Earthquake

Definition of Bracketed Duration



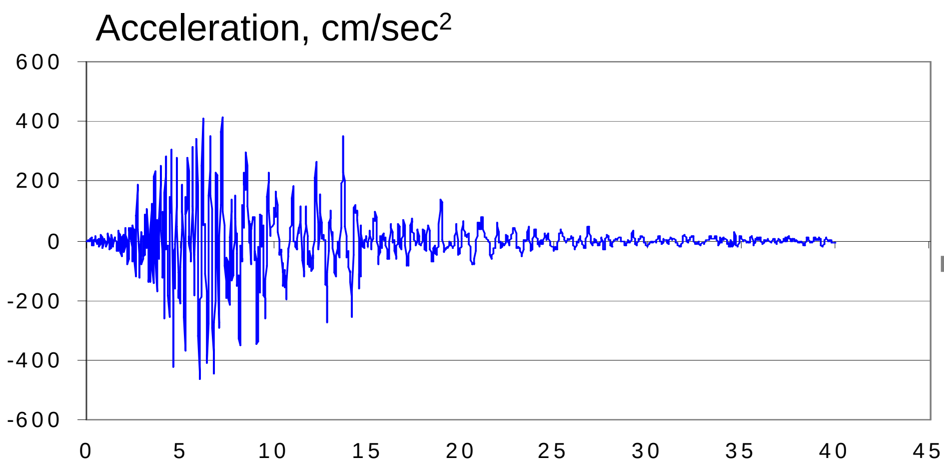
Definition of Incremental Velocity

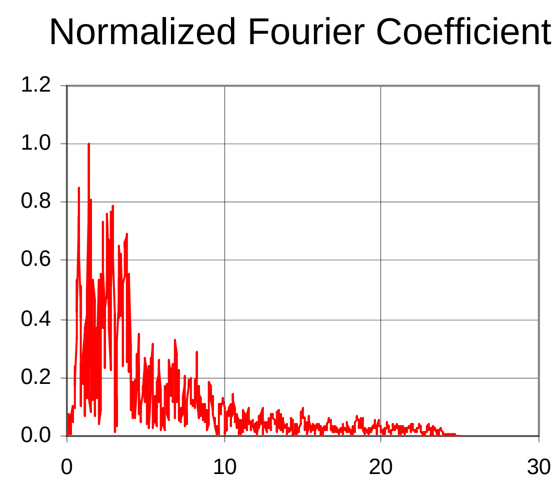


Concept of Fourier Amplitude Spectra

$$\ddot{v}_g(t) \cong a_0 + \sum_{j=1}^{N/2} a_j \cos(2\pi j f_0) + \sum_{j=1}^{N/2} b_j \sin(2\pi j f_0) = a_0 + \sum_{j=1}^{N/2} A_j \cos(2\pi j f_0 + \phi_j)$$

$$f_0 = df = 1/Ndt \qquad \phi_j = \arctan\left(-\frac{b_j}{a_j}\right) \qquad A_j = \sqrt{a_j^2 + b_j^2}$$

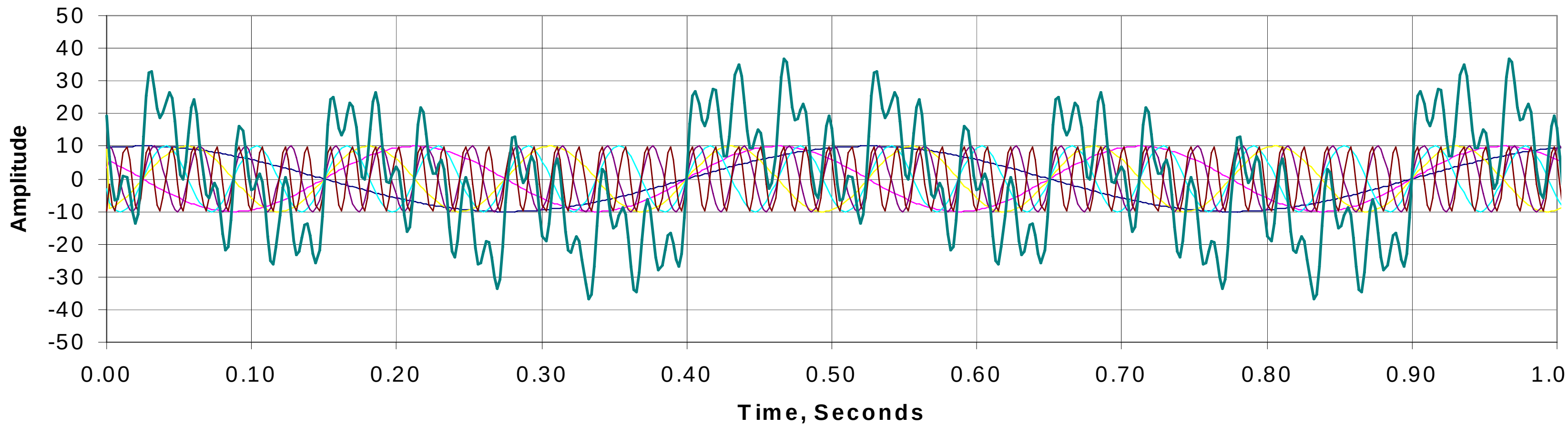


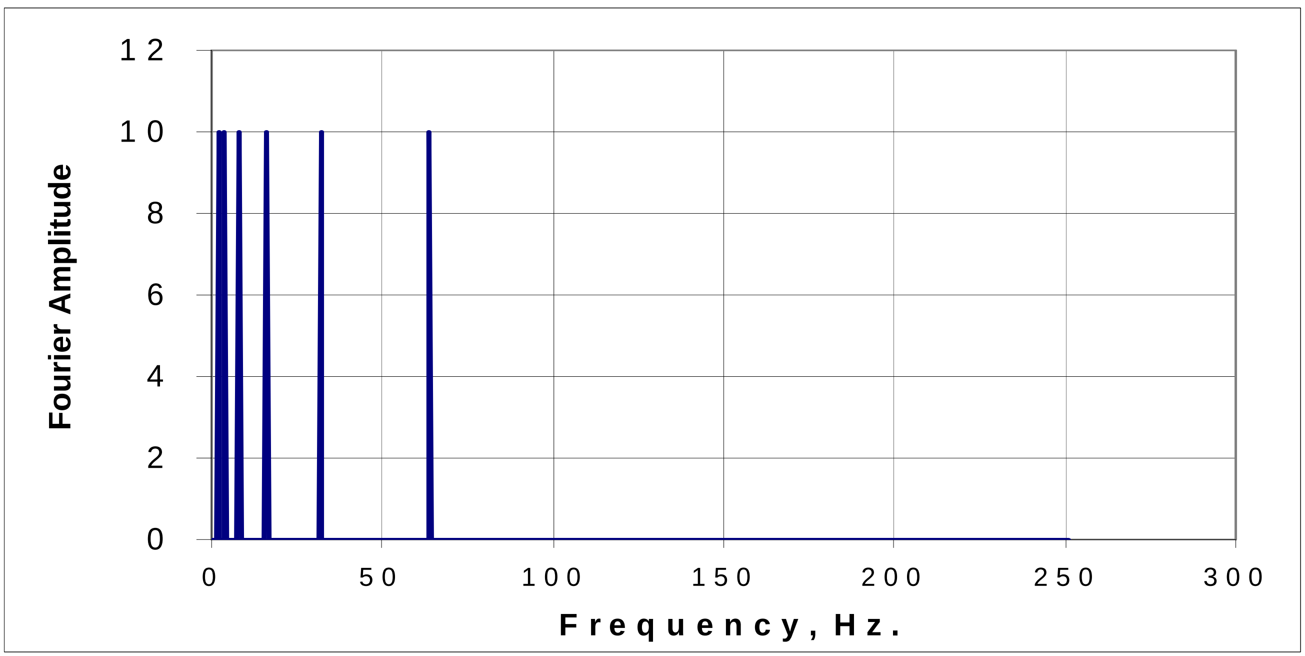


N points at timestep *dt*

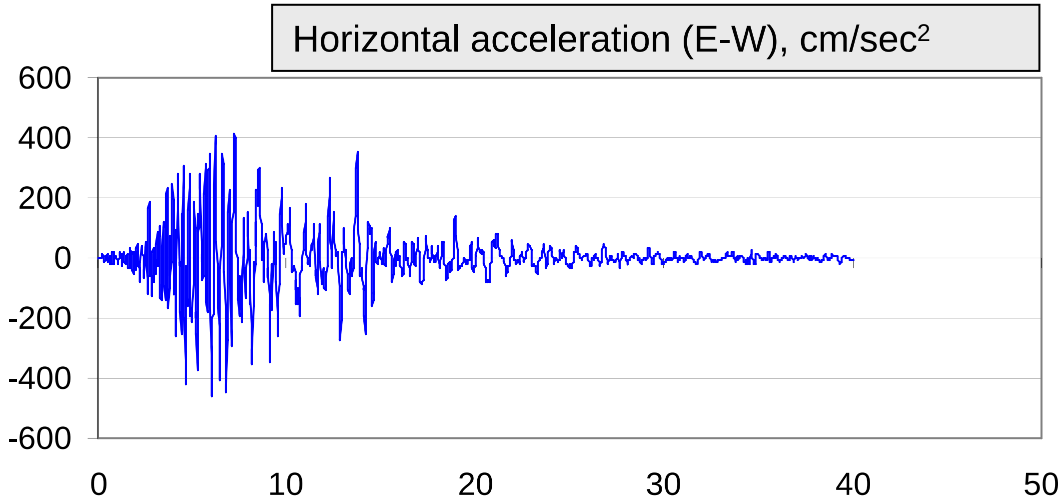
N/2 points at frequency *df*

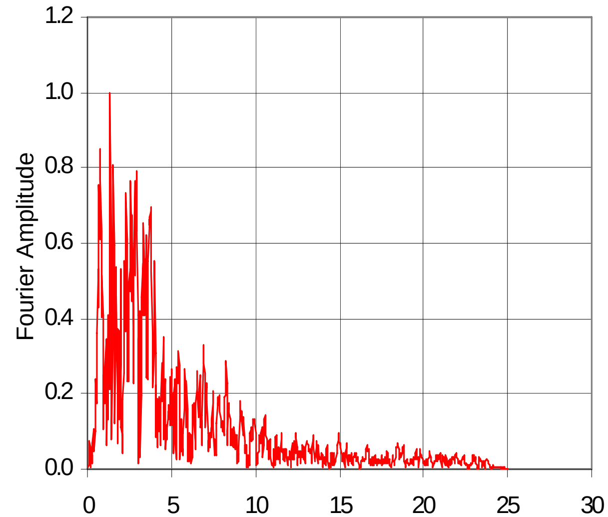
Concept of Fourier Amplitude Spectra

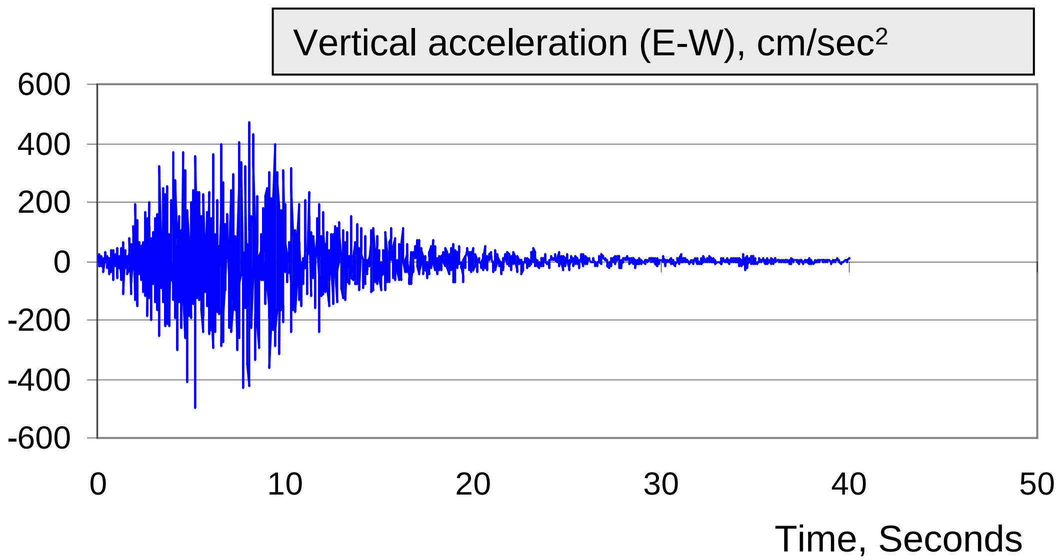


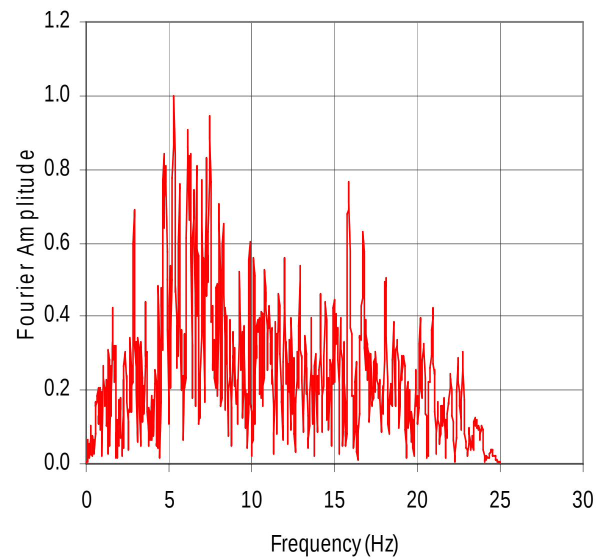


Ground Motion Frequency Content (1)

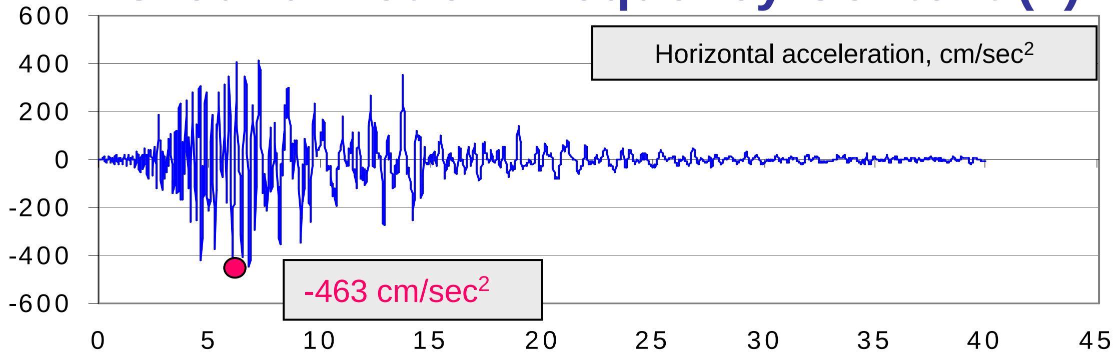


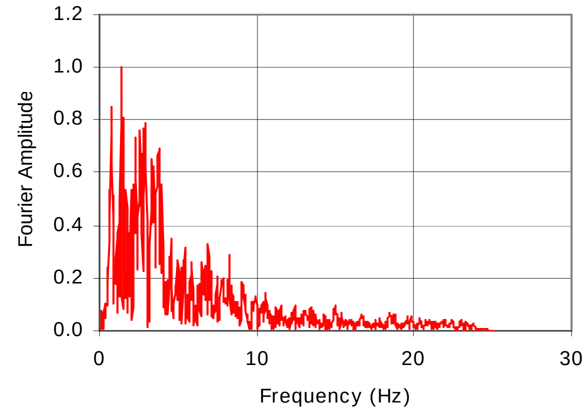


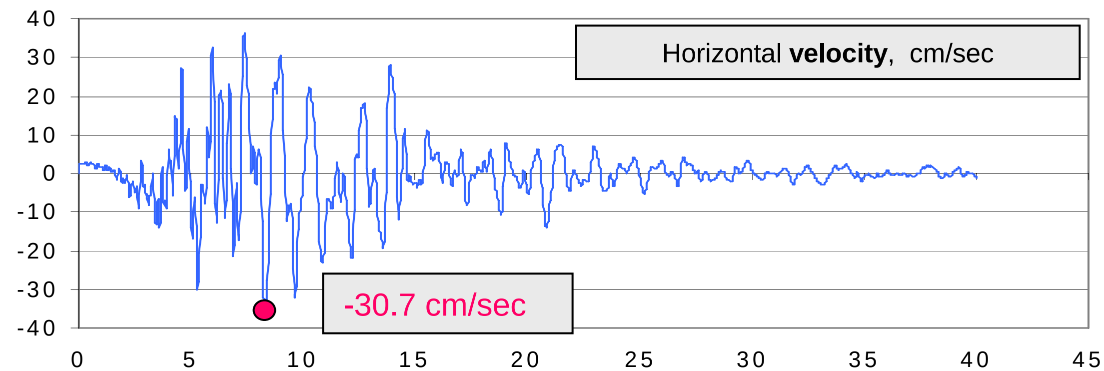


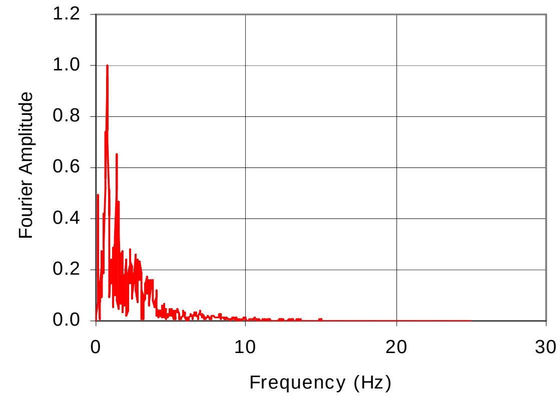


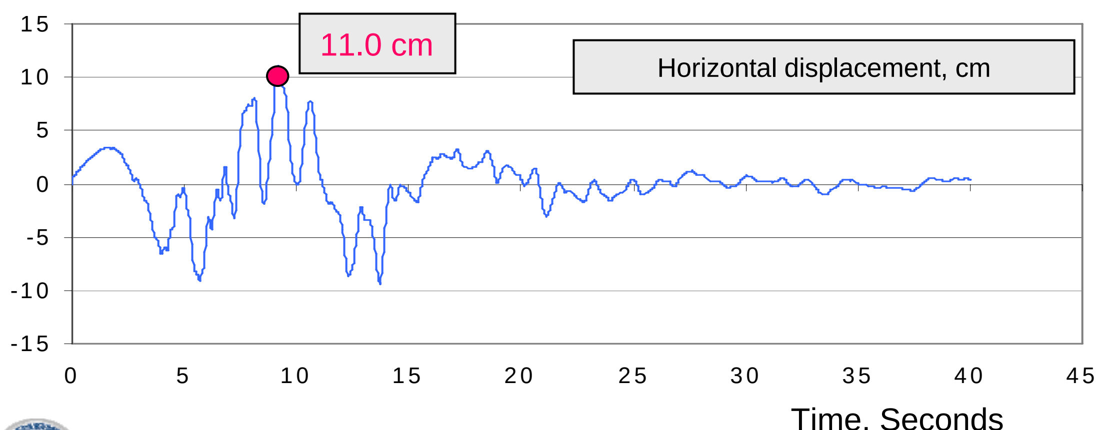
Ground Motion Frequency Content (2)

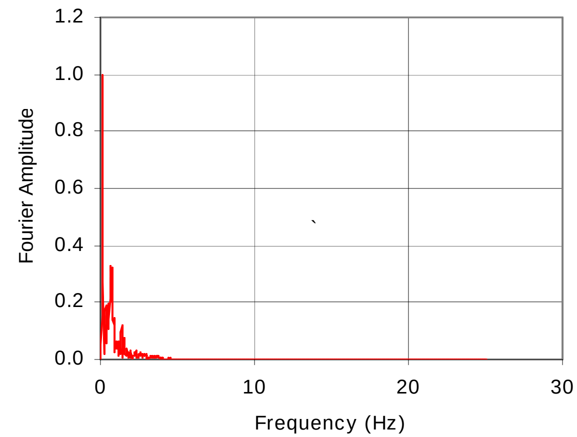




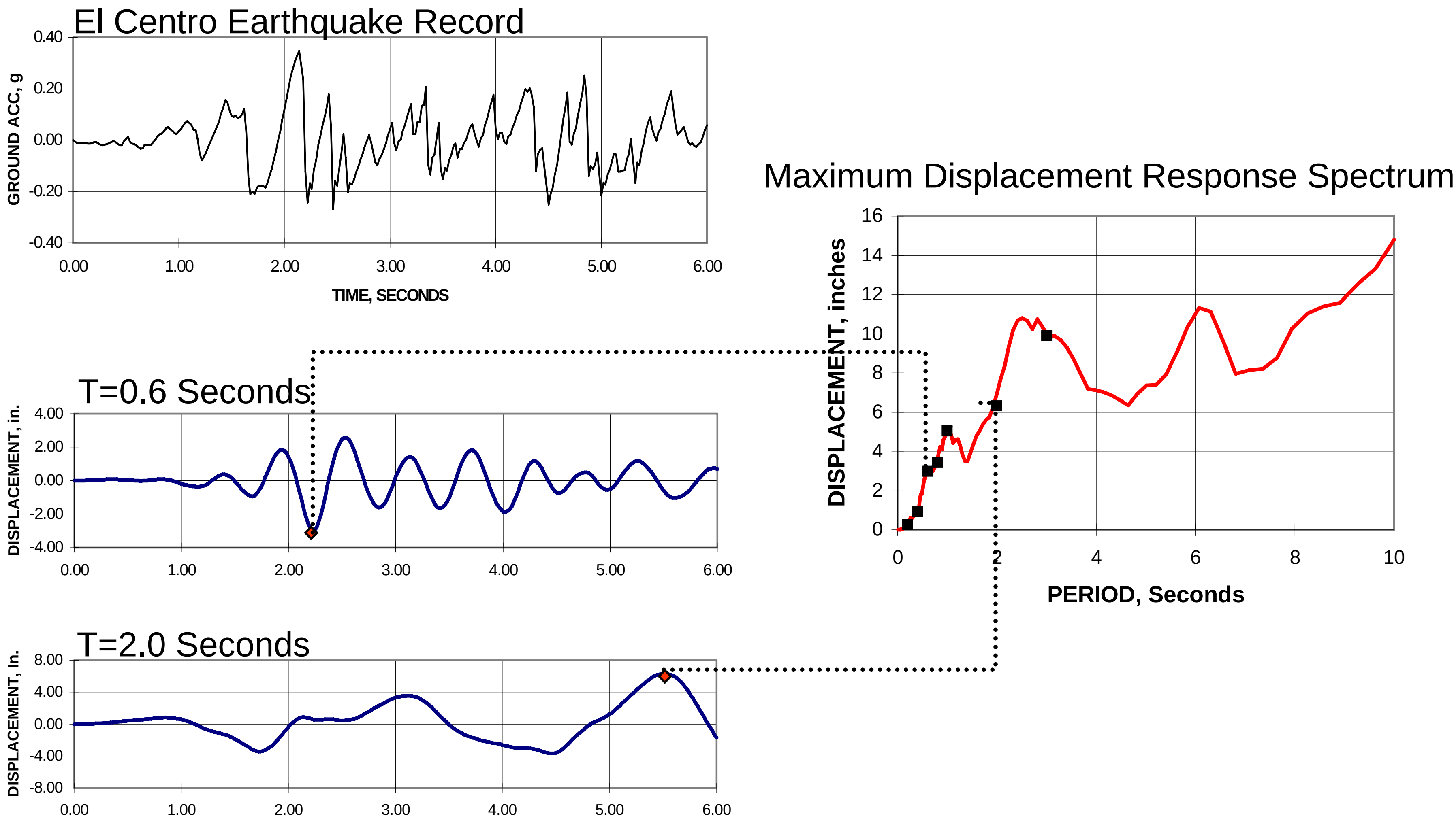




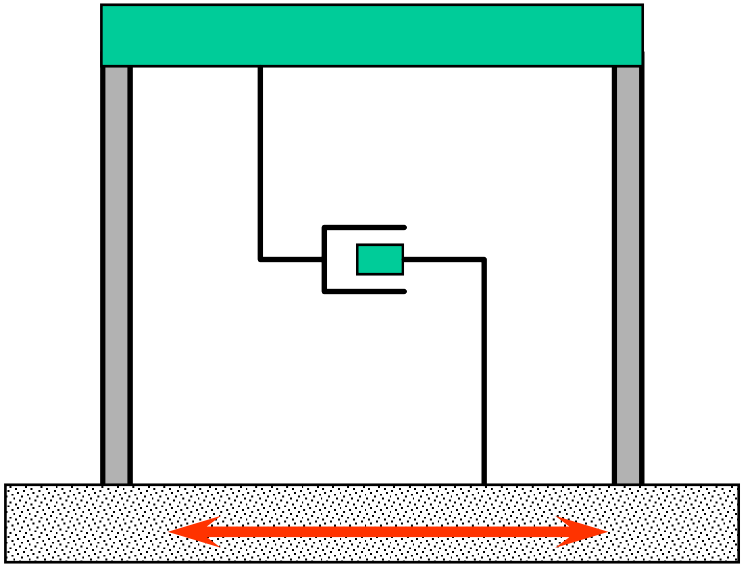




Development of an Elastic Displacement Response Spectrum



Structural Dynamics of Linear Elastic Single-Degree-of-Freedom (SDOF) Systems



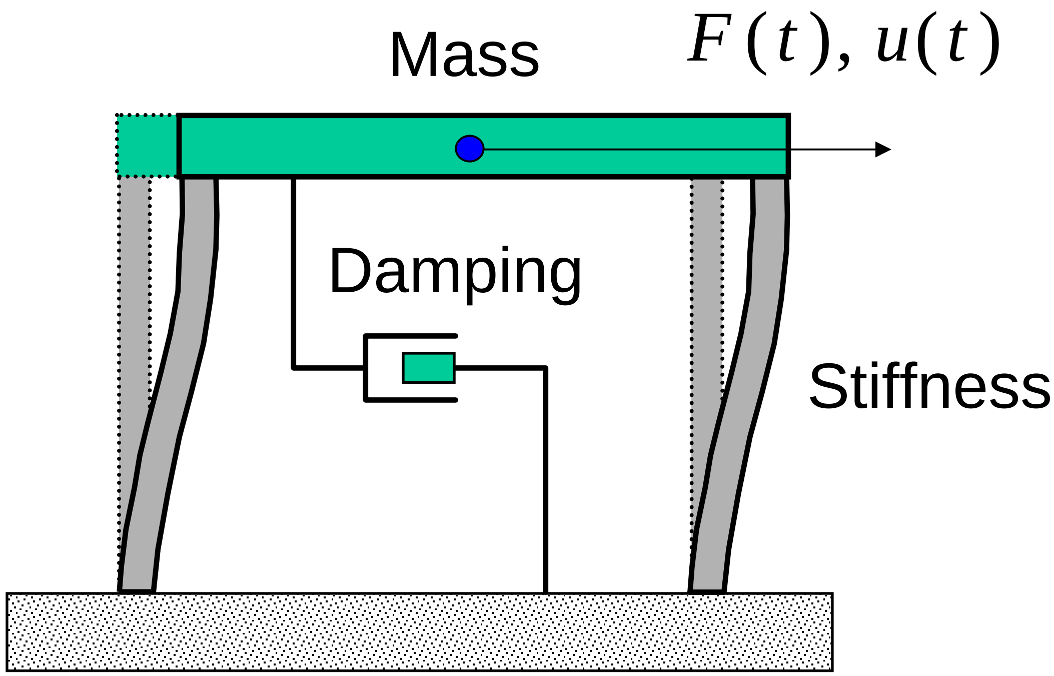
Structural Dynamics

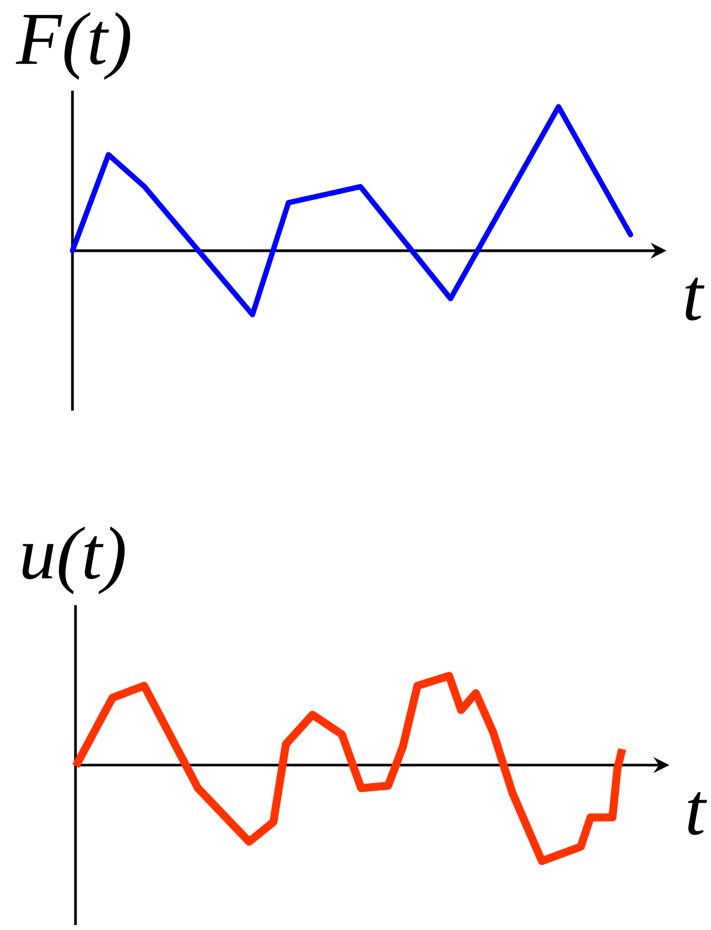
- Equations of motion for SDOF structures
- Structural frequency and period of vibration
- Behavior under dynamic load
- Dynamic magnification and resonance
- Effect of damping on behavior
- Linear elastic response spectra

Importance in Relation to ASCE 7-05

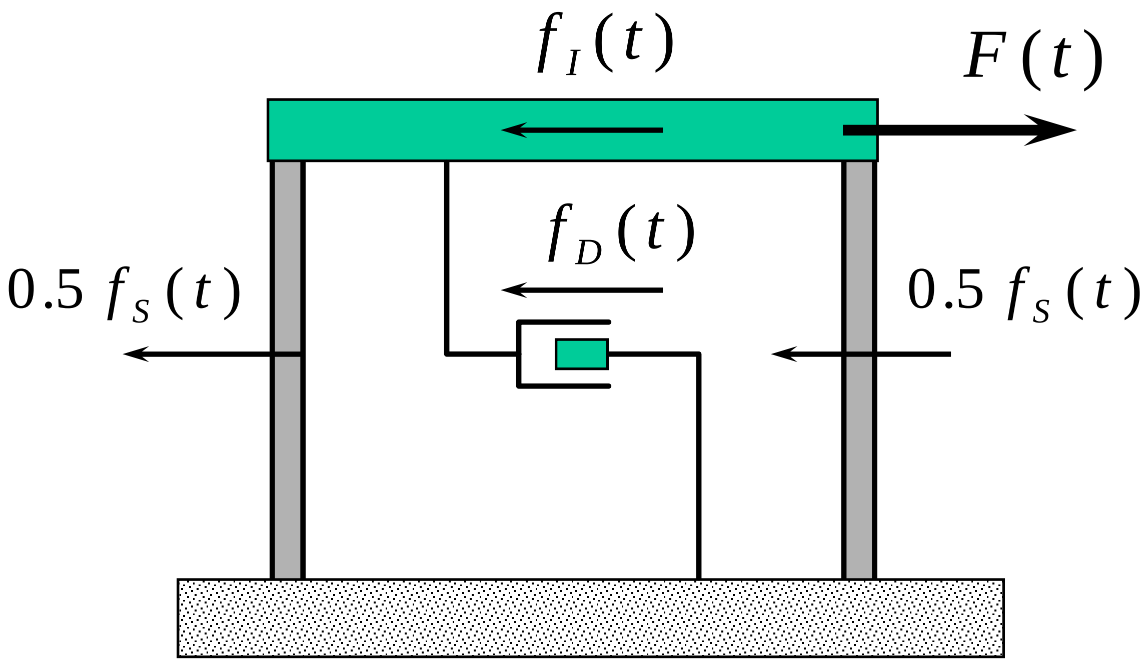
- Ground motion maps provide ground accelerations in terms of *response spectrum* coordinates.
- Equivalent lateral force procedure gives base shear in terms of *design spectrum* and *period of vibration*.
- Response spectrum is based on *5% critical damping* in system.
- Modal superposition analysis uses design *response spectrum* as basic ground motion input.

Idealized SDOF Structure





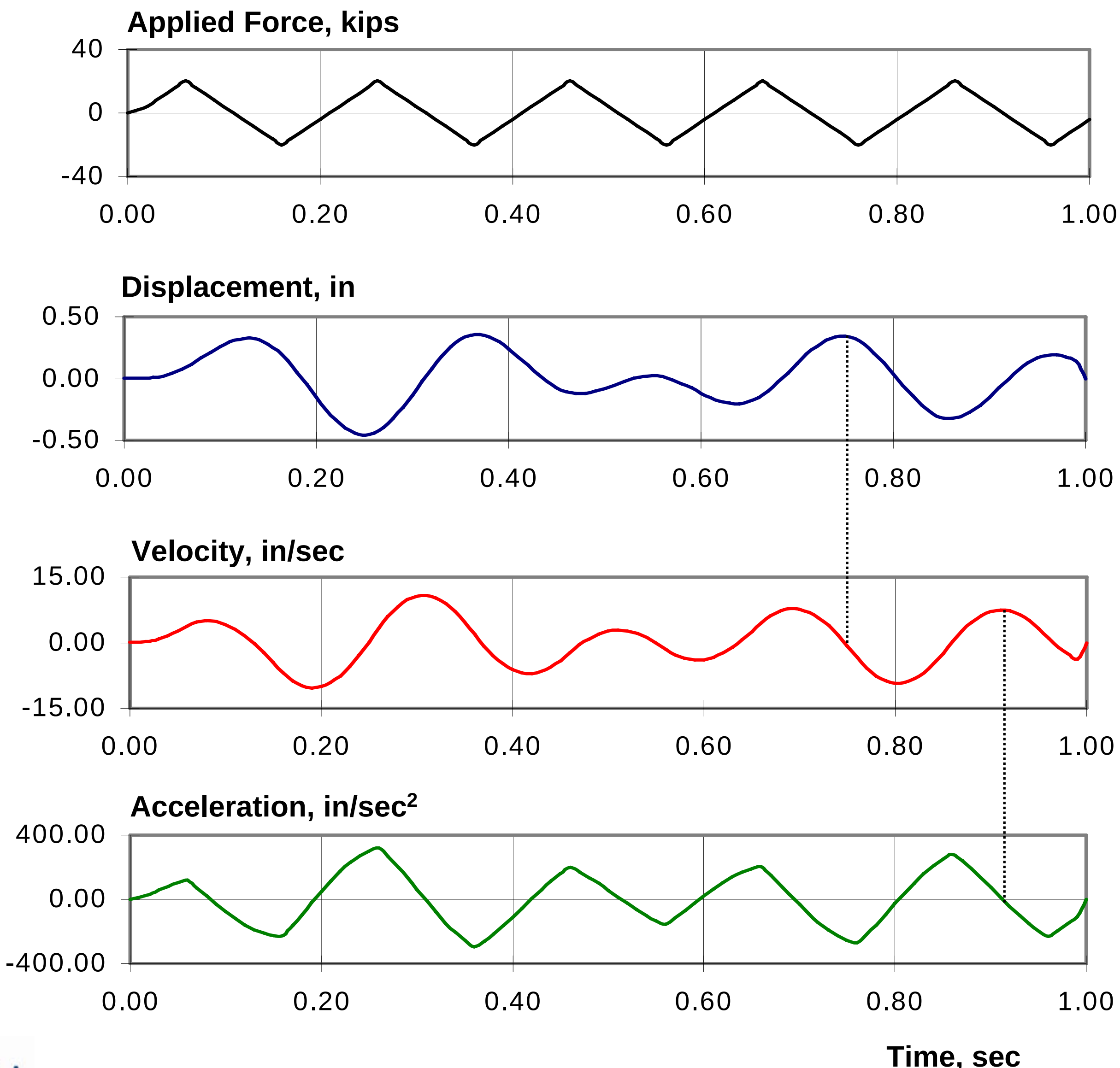
Equation of Dynamic Equilibrium



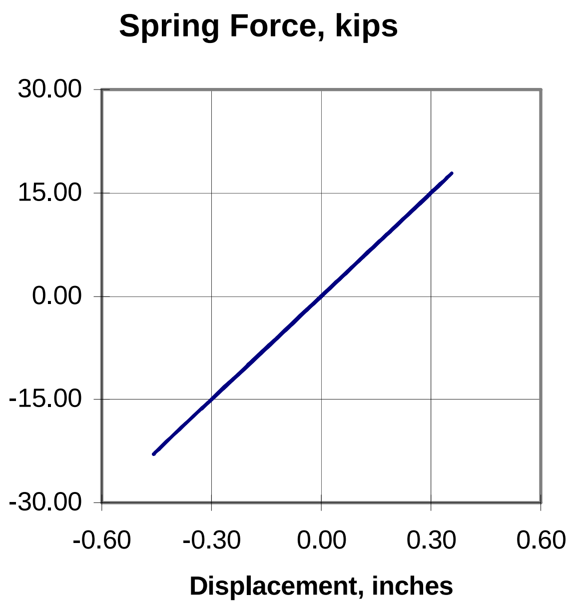
$$F(t) - f_I(t) - f_D(t) - f_S(t) = 0$$

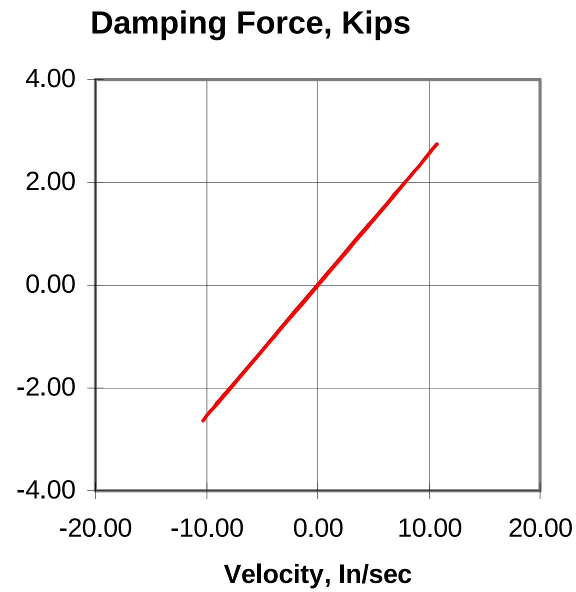
$$f_I(t) + f_D(t) + f_S(t) = F(t)$$

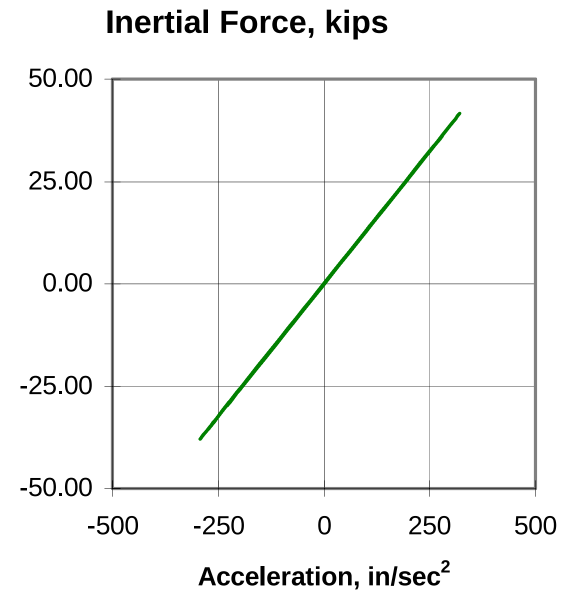
Observed Response of Linear SDOF



Observed Response of Linear SDOF
(Development of Equilibrium Equation)







Slope = *k*
= 50 kip/in

$$f_S(t) = k\ u(t)$$

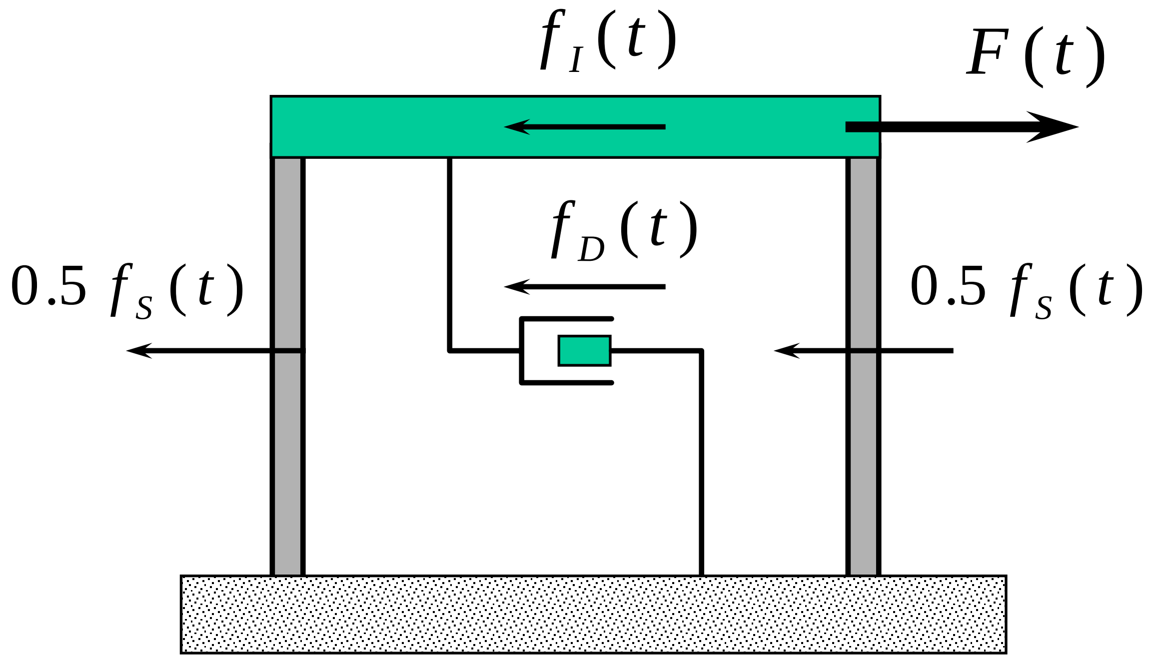
Slope = *c*
= 0.254 kip-sec/in

$$f_D(t) = c\ \dot{u}(t)$$

Slope = *m*
= 0.130 kip-sec²/in

$$f_I(t) = m\ \ddot{u}(t)$$

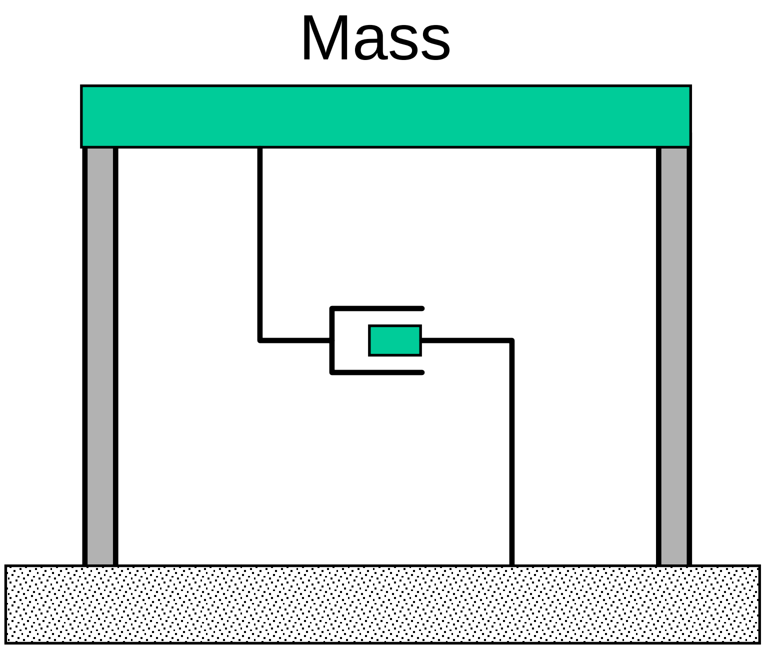
Equation of Dynamic Equilibrium

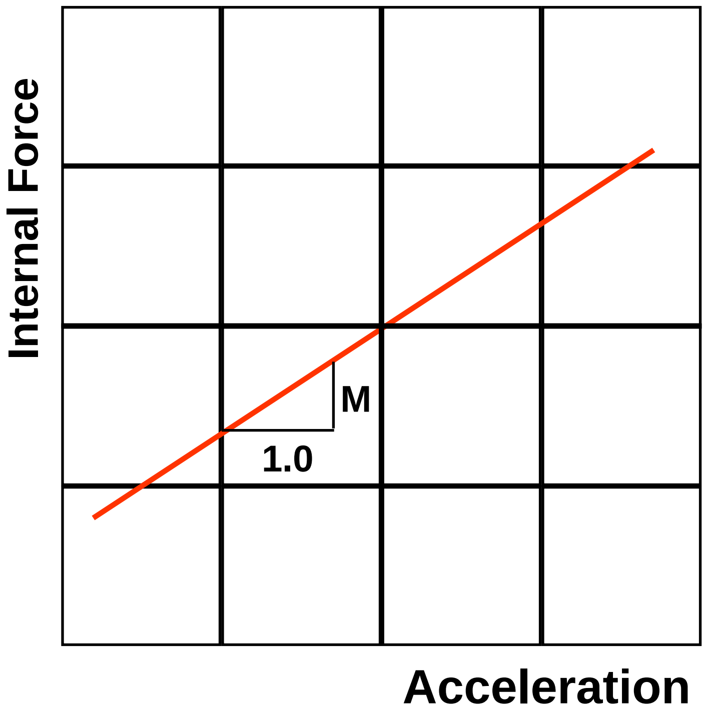


$$f_I(t) + f_D(t) + f_S(t) = F(t)$$

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

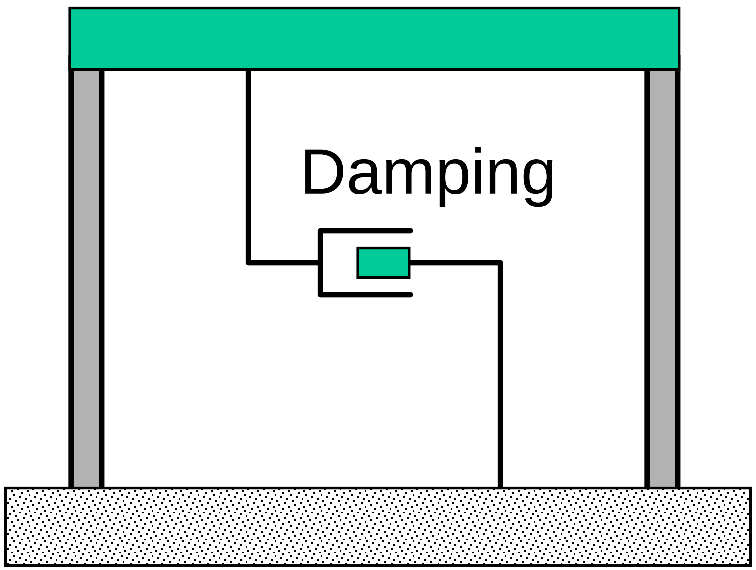
Properties of Structural Mass

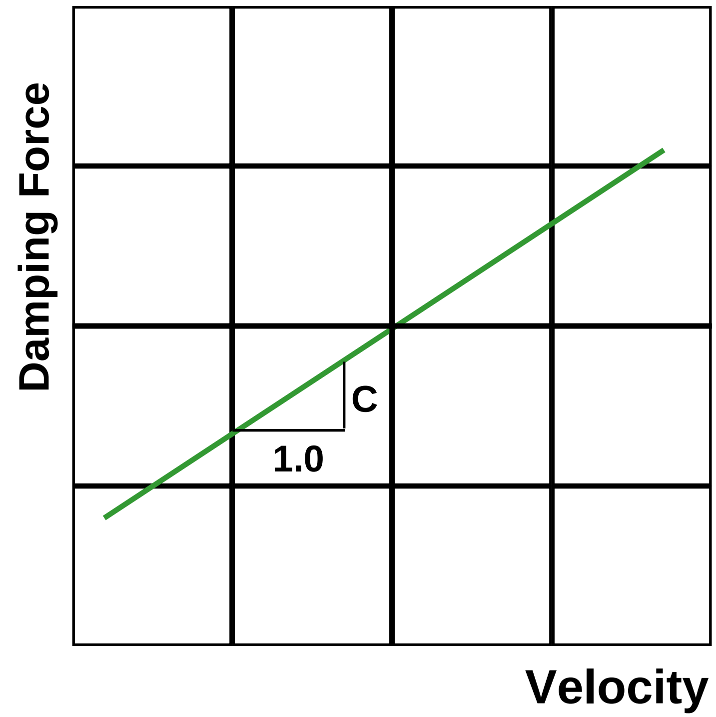




- Includes all dead weight of structure
- May include some live load
- Has units of force/acceleration

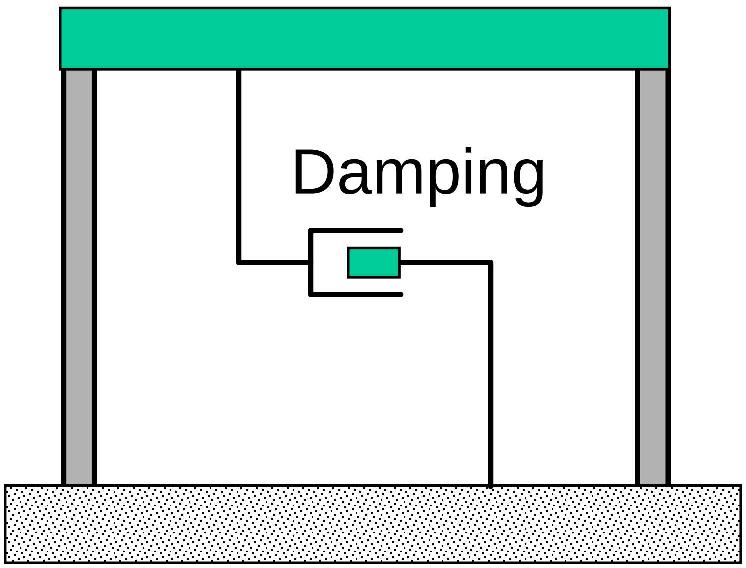
Properties of Structural Damping

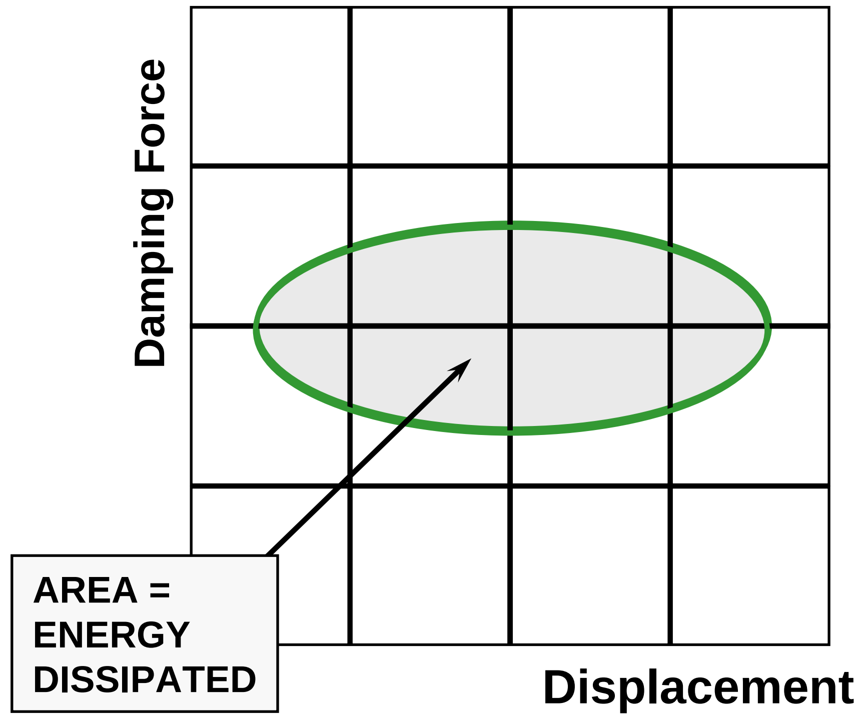




- In absence of dampers, is called *inherent damping*
- Usually represented by *linear* viscous dashpot
- Has units of force/velocity

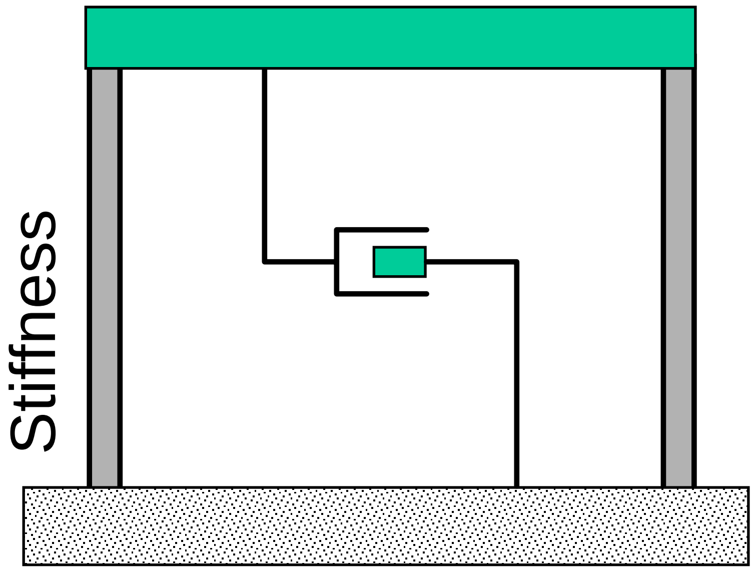
Properties of Structural Damping (2)

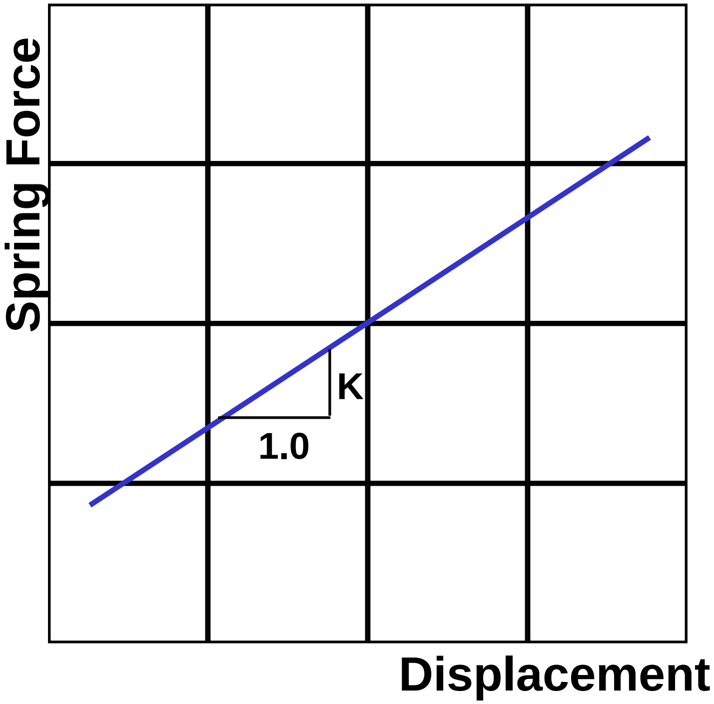




Damping vs displacement response is elliptical for linear viscous damper.

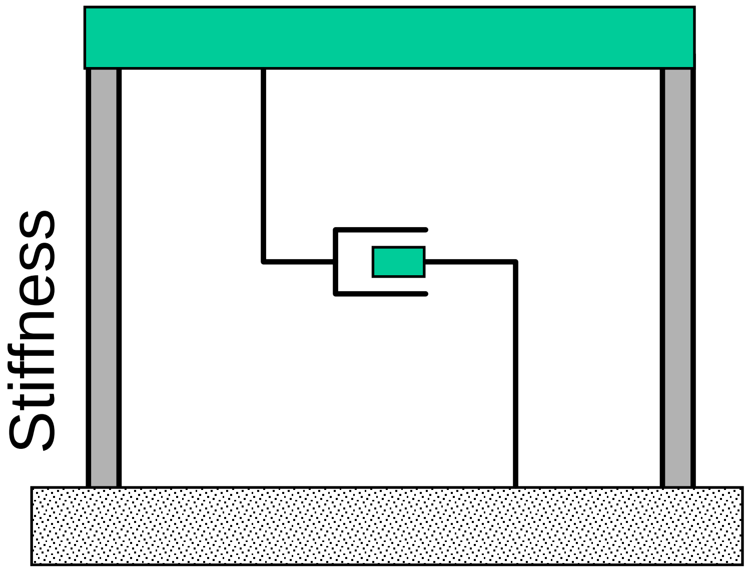
Properties of Structural Stiffness

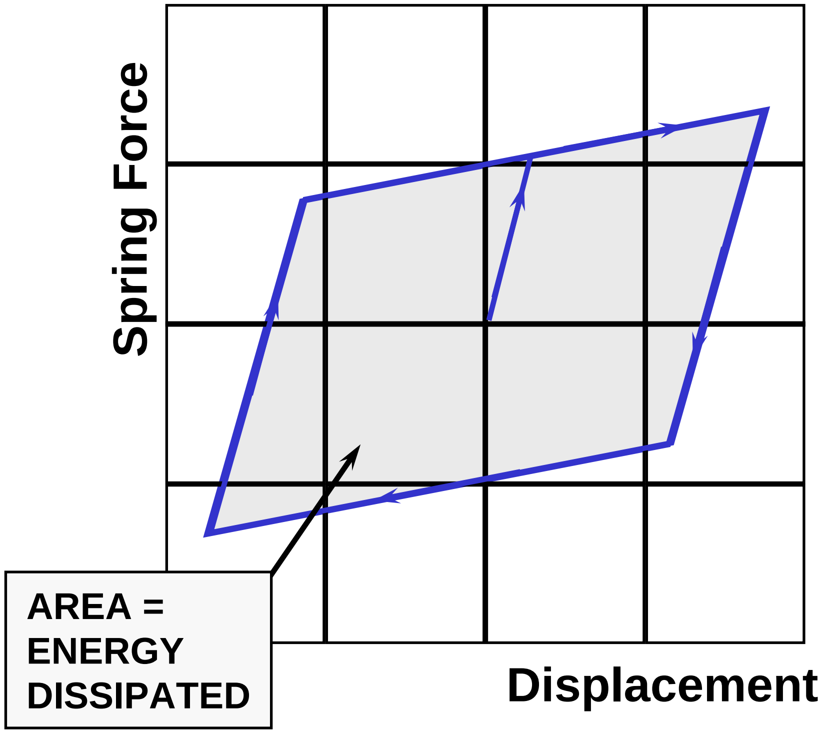




- Includes all structural members
- May include some “seismically nonstructural” members
- Requires careful mathematical modelling
- Has units of force/displacement

Properties of Structural Stiffness (2)





- Is almost always nonlinear in real seismic response
- Nonlinearity is implicitly handled by codes
- Explicit modelling of nonlinear effects is possible

Undamped Free Vibration

Equation of motion: $m\,\ddot{u}(t) + k\,u(t) = 0$

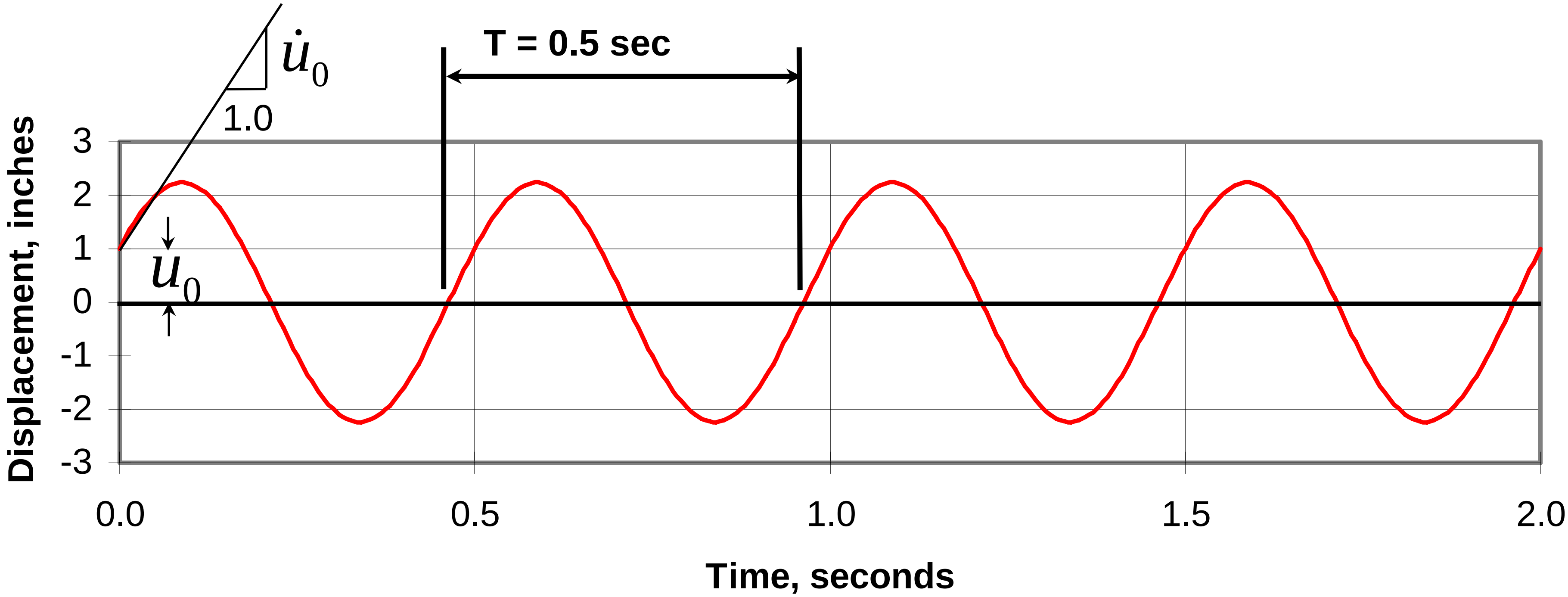
Initial conditions: $\dot{u}_0 \quad u_0$

Assume: $u(t) = A\sin(\omega t) + B\cos(\omega t)$

Solution: $A = \dfrac{\dot{u}_0}{\omega} \qquad B = u_0 \qquad \omega = \sqrt{\dfrac{k}{m}}$

$$\boxed{u(t) = \frac{\dot{u}_0}{\omega}\sin(\omega t) + u_0\cos(\omega t)}$$

Undamped Free Vibration (2)



Circular Frequency (radians/sec)

$$\omega = \sqrt{\frac{k}{m}}$$

Cyclic Frequency (cycles/sec, Hertz)

$$f = \frac{\omega}{2\pi}$$

Period of Vibration (sec/cycle)

$$T = \frac{1}{f} = \frac{2\pi}{\omega}$$

Approximate Periods of Vibration
(ASCE 7-05)

$$T_a = C_t h_n^x$$

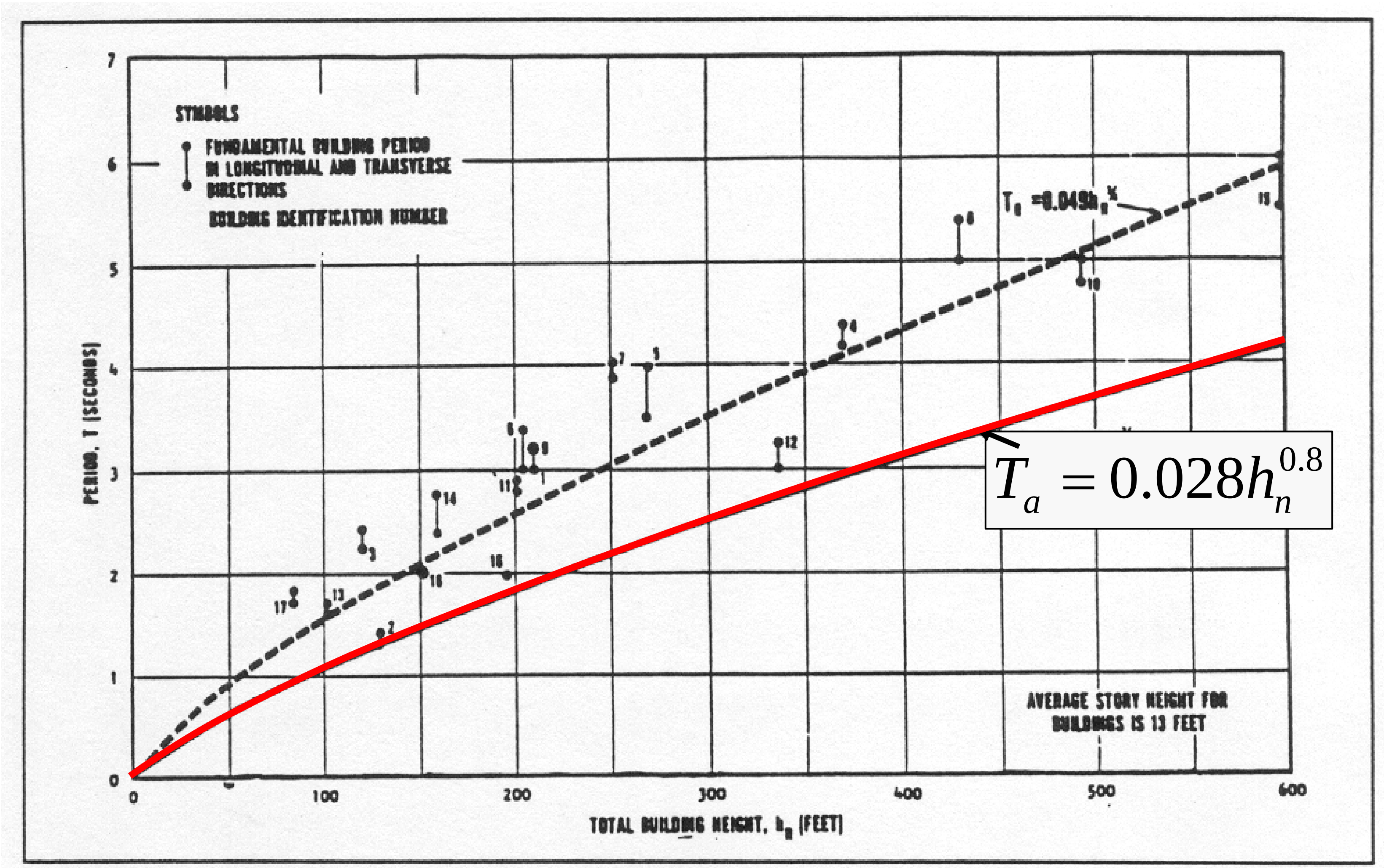
C_t = 0.028, x = 0.8 for steel moment frames
C_t = 0.016, x = 0.9 for concrete moment frames
C_t = 0.030, x = 0.75 for eccentrically braced frames
C_t = 0.020, x = 0.75 for all other systems

Note: This applies ONLY to building structures!

$$T_a = 0.1N$$

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

Empirical Data for Determination of Approximate Period for Steel Moment Frames



Periods of Vibration of Common Structures

20-story moment resisting frame	T = 1.9 sec
10-story moment resisting frame	T = 1.1 sec
1-story moment resisting frame	T = 0.15 sec
20-story braced frame	T = 1.3 sec
10-story braced frame	T = 0.8 sec
1-story braced frame	T = 0.1 sec
Gravity dam	T = 0.2 sec
Suspension bridge	T = 20 sec

Adjustment Factor on Approximate Period
(Table 12.8-1 of ASCE 7-05)

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

S_{D1}	C_u
> 0.40g	1.4
0.30g	1.4
0.20g	1.5
0.15g	1.6
< 0.1g	1.7

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

Which Period of Vibration to Use in ELF Analysis?

If you do not have a "more accurate" period (from a computer analysis), you must use $T = T_a$.

If you have a more accurate period from a computer analysis (call this T_c), then:

$$\text{if } \ T_c > C_u T_a \qquad \text{use } T = C_u T_a$$

$$\text{if } \ T_a < T_c < T_u C_a \qquad \text{use } T = T_c$$

$$\text{if } \ T_c < T_a \qquad \text{use } T = T_a$$

Damped Free Vibration

Equation of motion: $m\,\ddot{u}(t) + \boxed{c\,\dot{u}(t)} + k\,u(t) = 0$

Initial conditions: $u_0 \quad \dot{u}_0$

Assume: $u(t) = e^{st}$

Solution:

$$u(t) = e^{-\xi\omega t}\left[u_0 \cos(\omega_D t) + \frac{\dot{u}_0 + \xi\omega u_0}{\omega_D}\sin(\omega_D t)\right]$$

$$\boxed{\xi = \frac{c}{2m\omega} = \frac{c}{c_c}} \qquad \boxed{\omega_D = \omega\sqrt{1-\xi^2}}$$

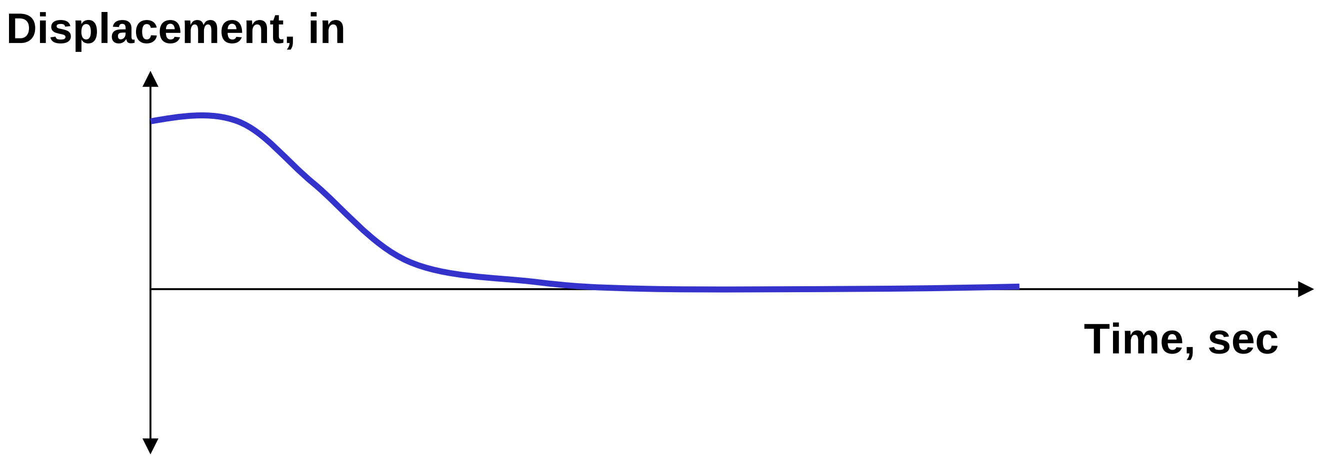
Damping in Structures

$$\xi = \frac{c}{2m\omega} = \frac{c}{c_c}$$

c_c is the c*ritical damping constant.*

ξ is expressed as a ratio ($0.0 < \xi < 1.0$) in computations.

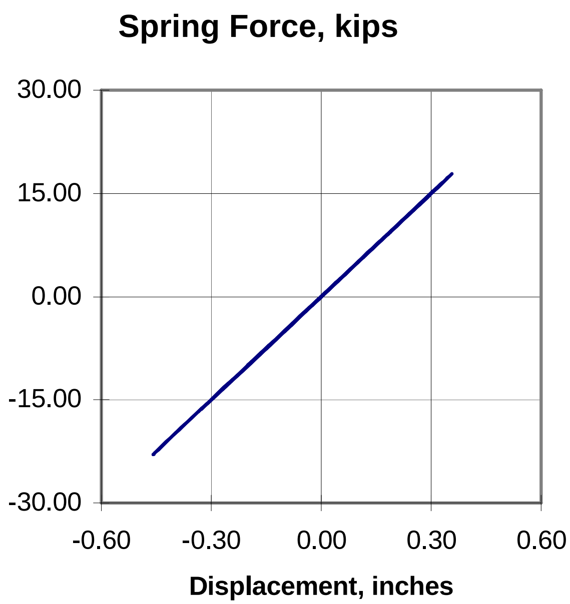
Sometimes ξ is expressed as a% ($0 < \xi < 100\%$).

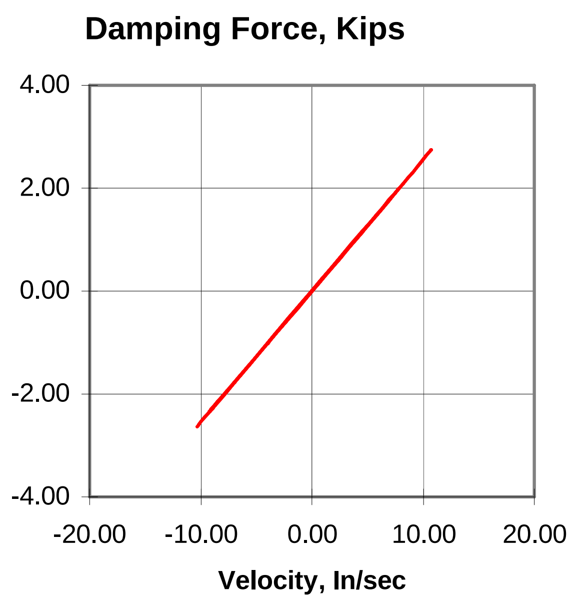


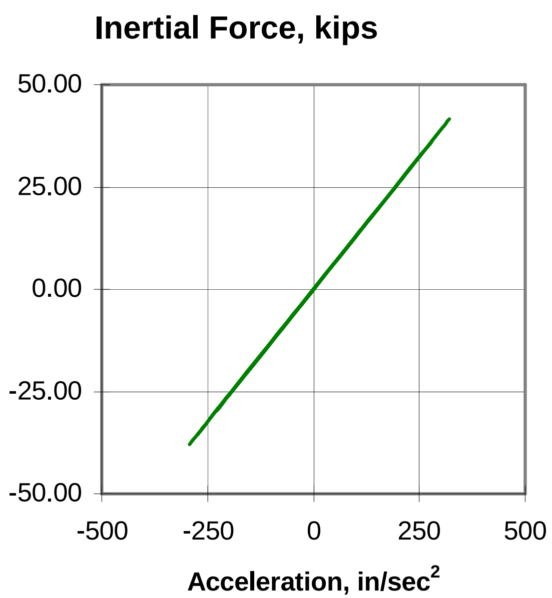
Response of Critically Damped System, ξ=1.0 or 100% critical

Damping in Structures

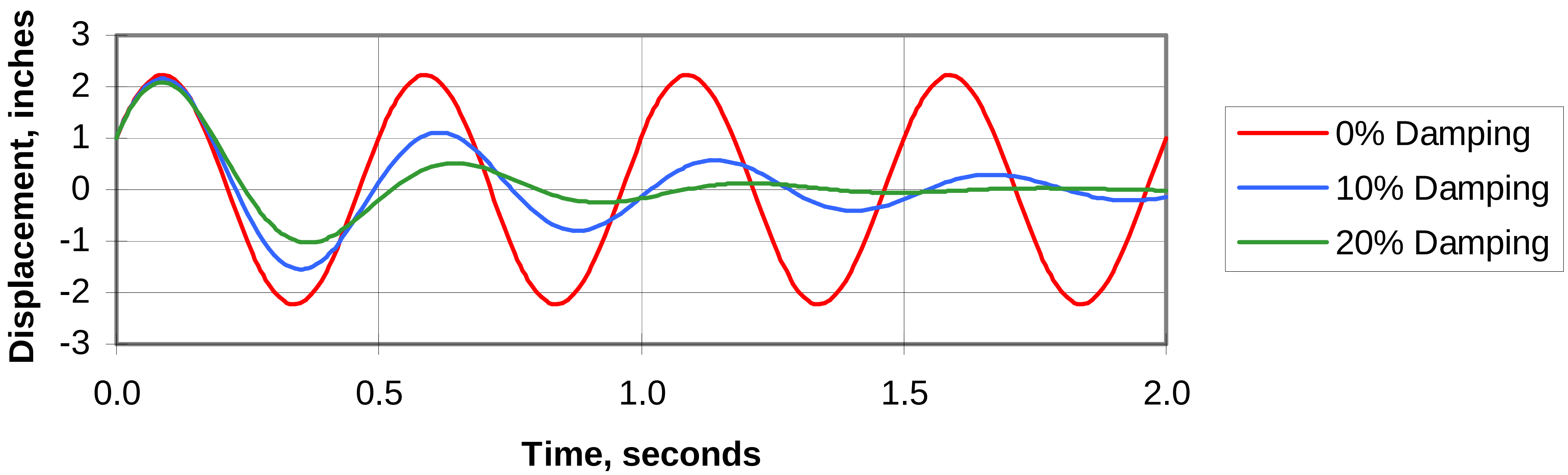
True damping in structures is NOT viscous. However, for low damping values, viscous damping allows for linear equations and vastly simplifies the solution.







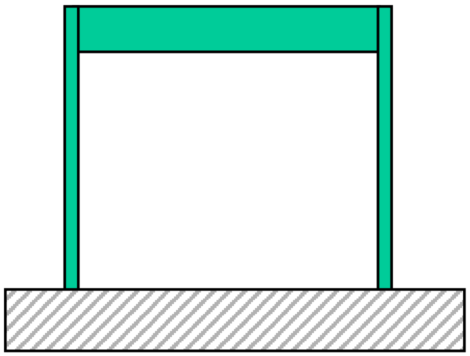
Damped Free Vibration (2)



Damping in Structures (2)

Welded steel frame	$\xi = 0.010$
Bolted steel frame	$\xi = 0.020$
Uncracked prestressed concrete	$\xi = 0.015$
Uncracked reinforced concrete	$\xi = 0.020$
Cracked reinforced concrete	$\xi = 0.035$
Glued plywood shear wall	$\xi = 0.100$
Nailed plywood shear wall	$\xi = 0.150$
Damaged steel structure	$\xi = 0.050$
Damaged concrete structure	$\xi = 0.075$
Structure with added damping	$\xi = 0.250$

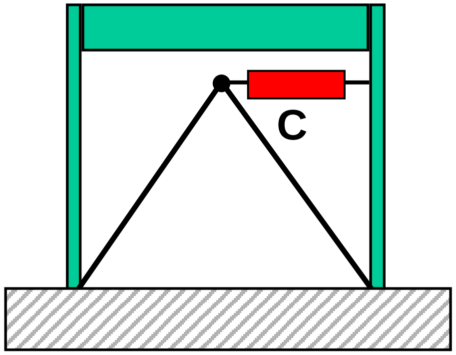
Damping in Structures (3)



Inherent damping

ξ is a structural (material) property <u>independent</u> of mass and stiffness

$$\xi_{Inherent} = 0.5 \text{ to } 7.0\% \text{ critical}$$

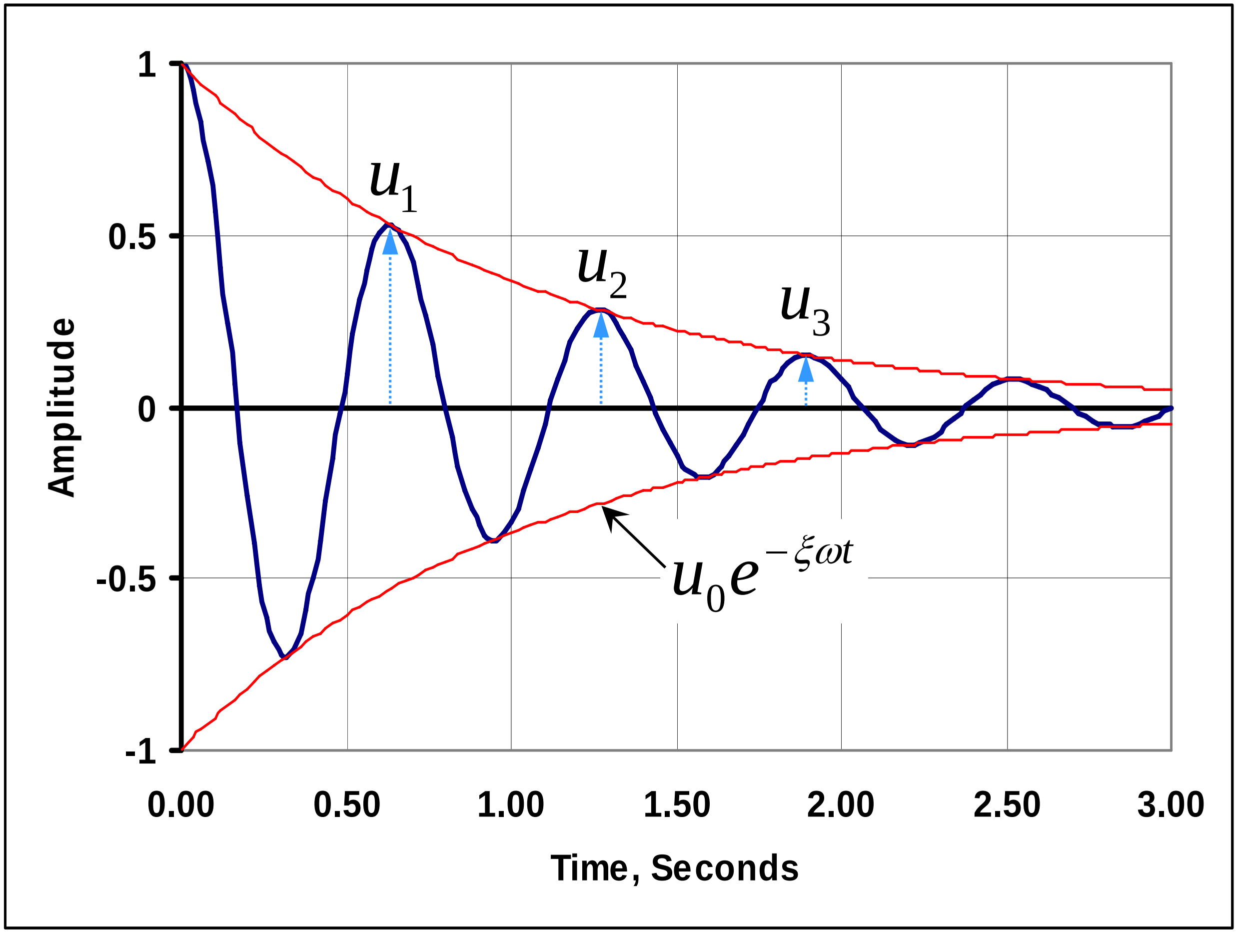


Added damping

ξ is a structural property dependent on mass and stiffness and damping constant C of device

$$\xi_{Added} = 10 \text{ to } 30\% \text{ critical}$$

Measuring Damping from Free Vibration Test



For all damping values

$$\ln \frac{u_1}{u_2} = \frac{2\pi\xi}{\sqrt{1-\xi^2}}$$

For <u>very low</u> damping values

$$\xi \cong \frac{u_1 - u_2}{2\pi u_2}$$

Undamped Harmonic Loading

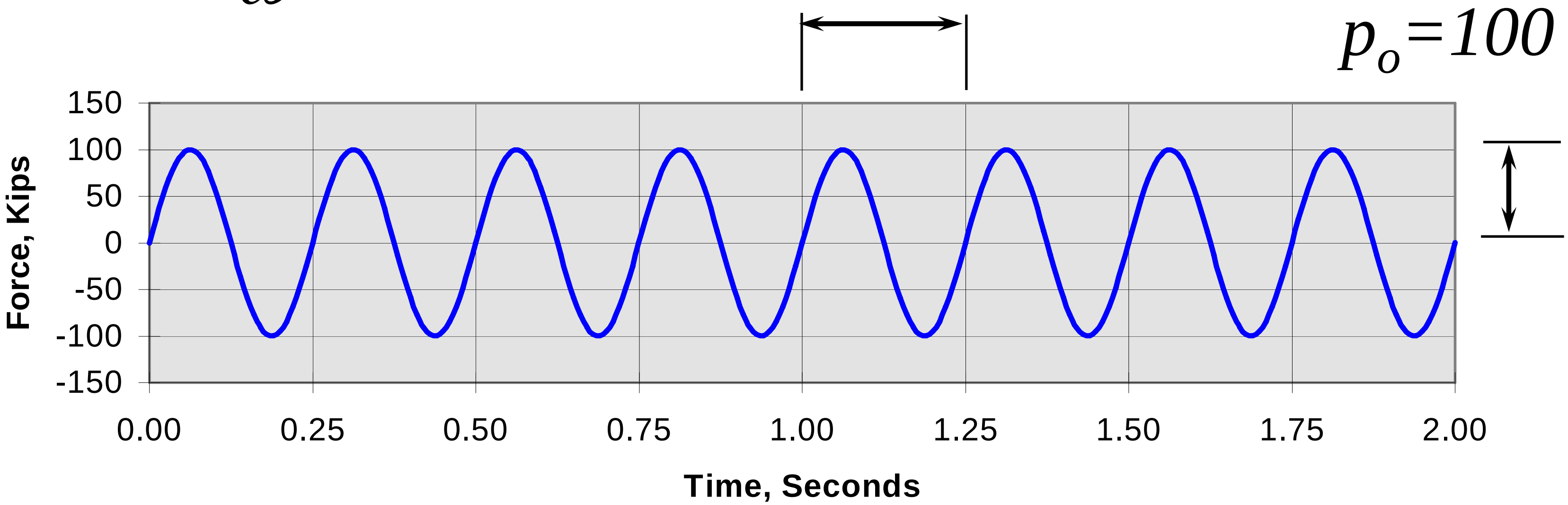
Equation of motion: $m\ddot{u}(t) + ku(t) = \boxed{p_0 \sin(\overline{\omega} t)}$

$\overline{\omega}$ = **frequency of the forcing function**

$$\overline{T} = \frac{2\pi}{\overline{\omega}}$$

$\overline{T}$ = *0.25 sec*

p_o=*100* kips



Undamped Harmonic Loading (2)

Equation of motion: $m\ddot{u}(t) + k\,u(t) = p_0 \sin(\overline{\omega} t)$

Assume system is initially at rest:

Particular solution: $u(t) = C\sin(\overline{\omega} t)$

Complimentary solution: $u(t) = A\sin(\omega t) + B\cos(\omega t)$

Solution:

$$\boxed{u(t) = \frac{p_0}{k}\frac{1}{1-(\overline{\omega}/\omega)^2}\left(\sin(\overline{\omega} t) - \frac{\overline{\omega}}{\omega}\sin(\omega t)\right)}$$

Undamped Harmonic Loading

Define $$\boxed{\beta = \frac{\overline{\omega}}{\omega}}$$

Loading frequency

Structure's natural frequency

Dynamic magnifier

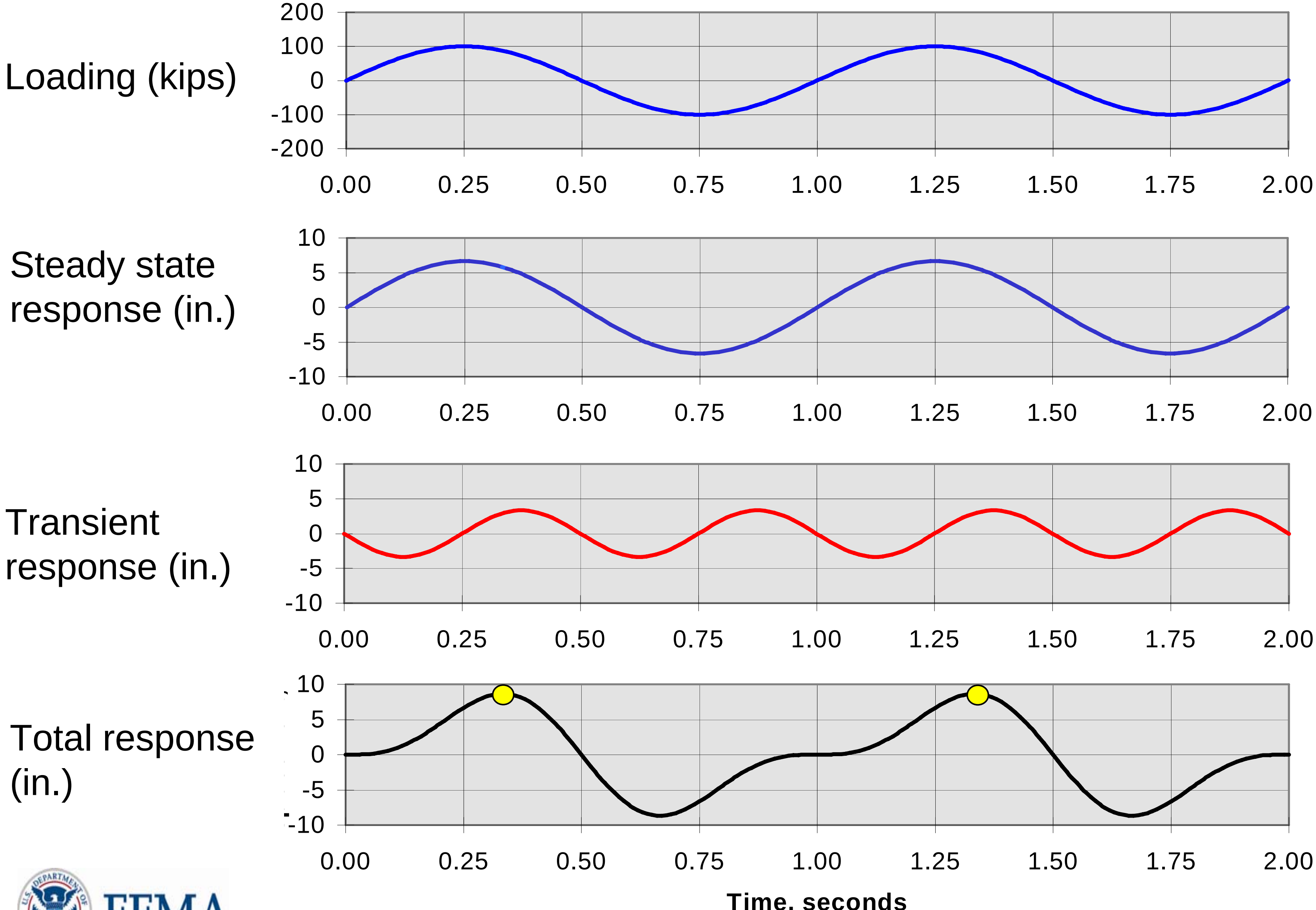
Transient response (at structure's frequency)

$$u(t) = \boxed{\frac{p_0}{k}}\ \boxed{\frac{1}{1-\beta^2}}\left(\boxed{\sin(\overline{\omega}t)} - \boxed{\beta \sin(\omega t)}\right)$$

Static displacement

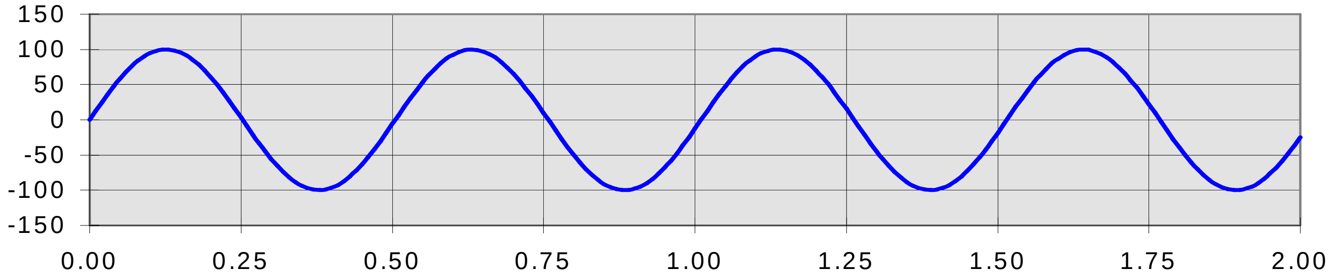
Steady state response (at loading frequency)

$$\omega = 4\pi \text{ rad / sec} \quad \overline{\omega} = 2\pi \text{ rad / sec} \quad \boxed{\beta = 0.5} \quad u_S = 5.0 \text{ in.}$$

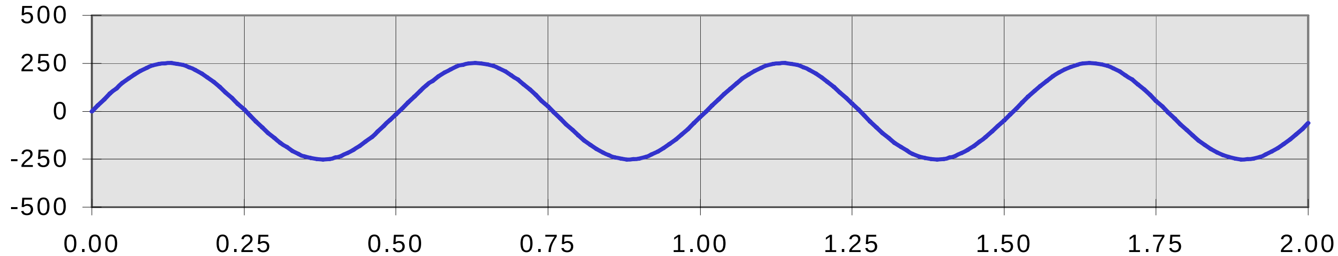


$$\omega \approx 4\pi \text{ rad / sec} \quad \overline{\omega} = 4\pi \text{ rad / sec} \quad \boxed{\beta = 0.99} \; u_S = 5.0 \text{ in.}$$

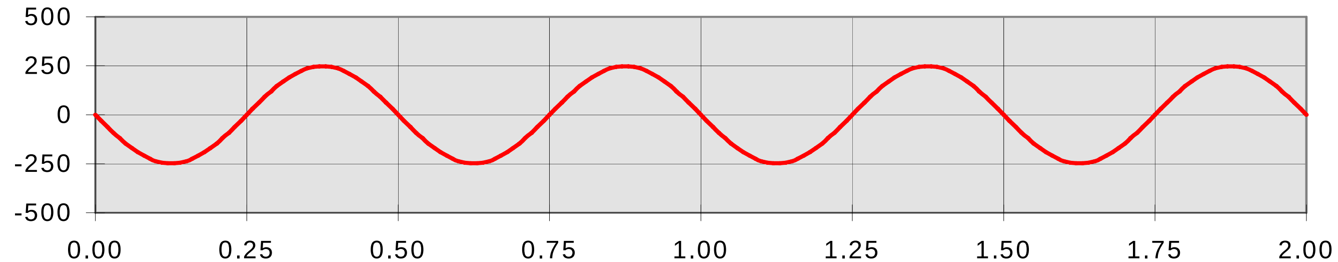
Loading (kips)



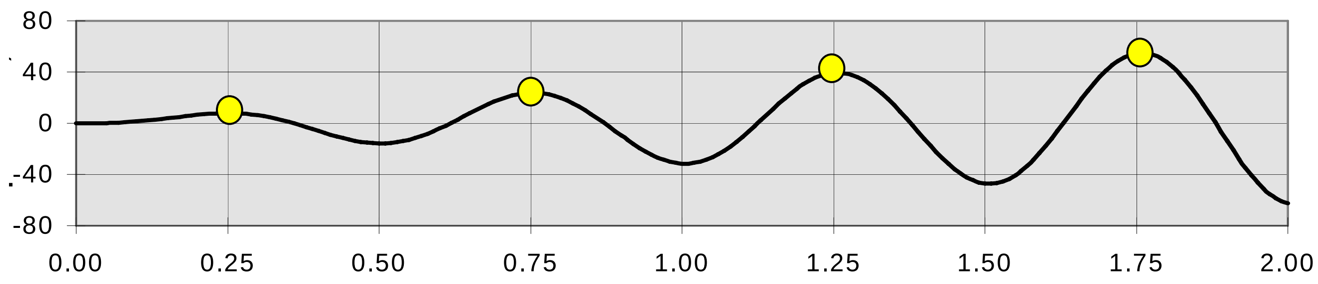
Steady state response (in.)



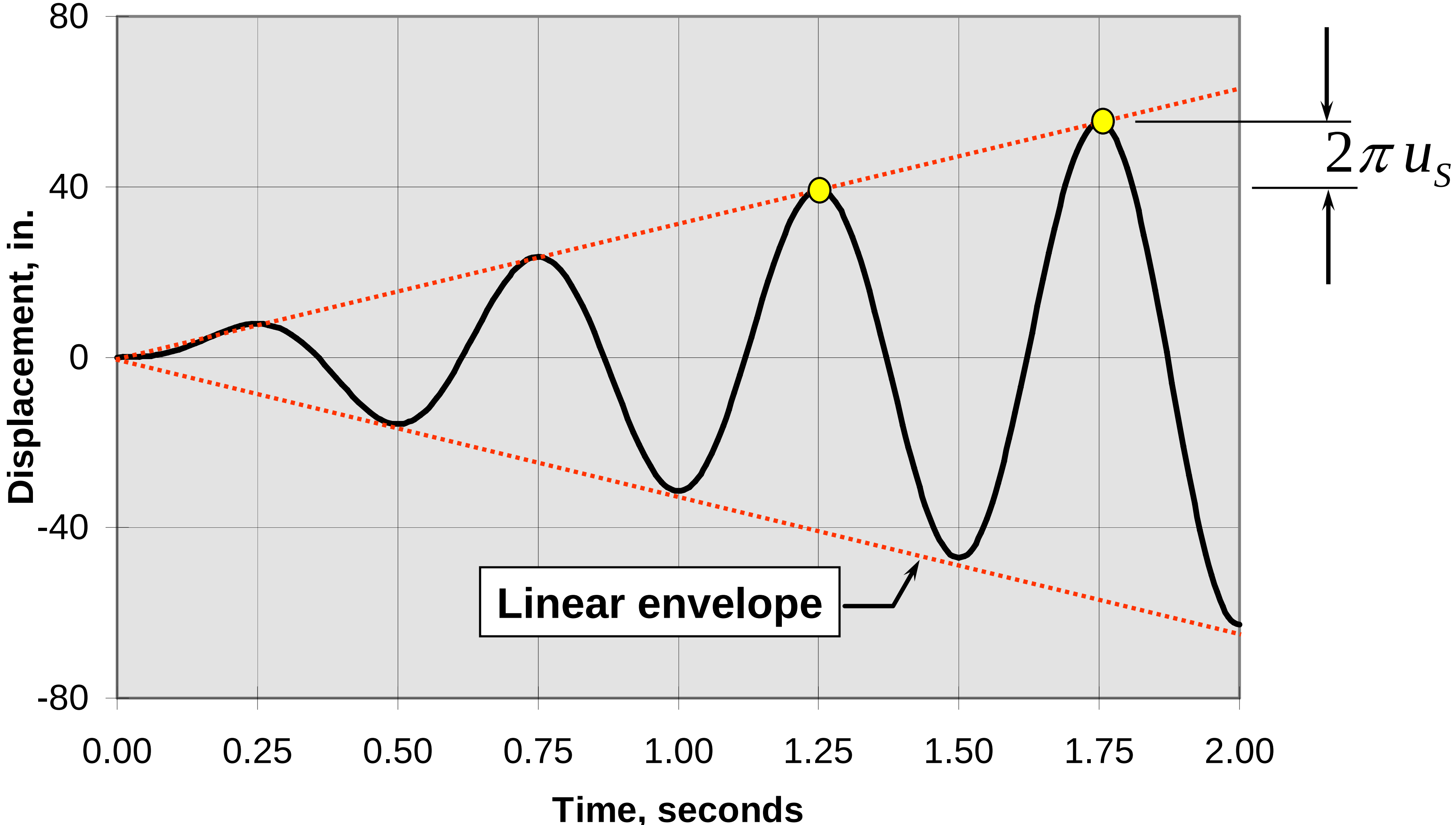
Transient response (in.)



Total response (in.)



Undamped Resonant Response Curve



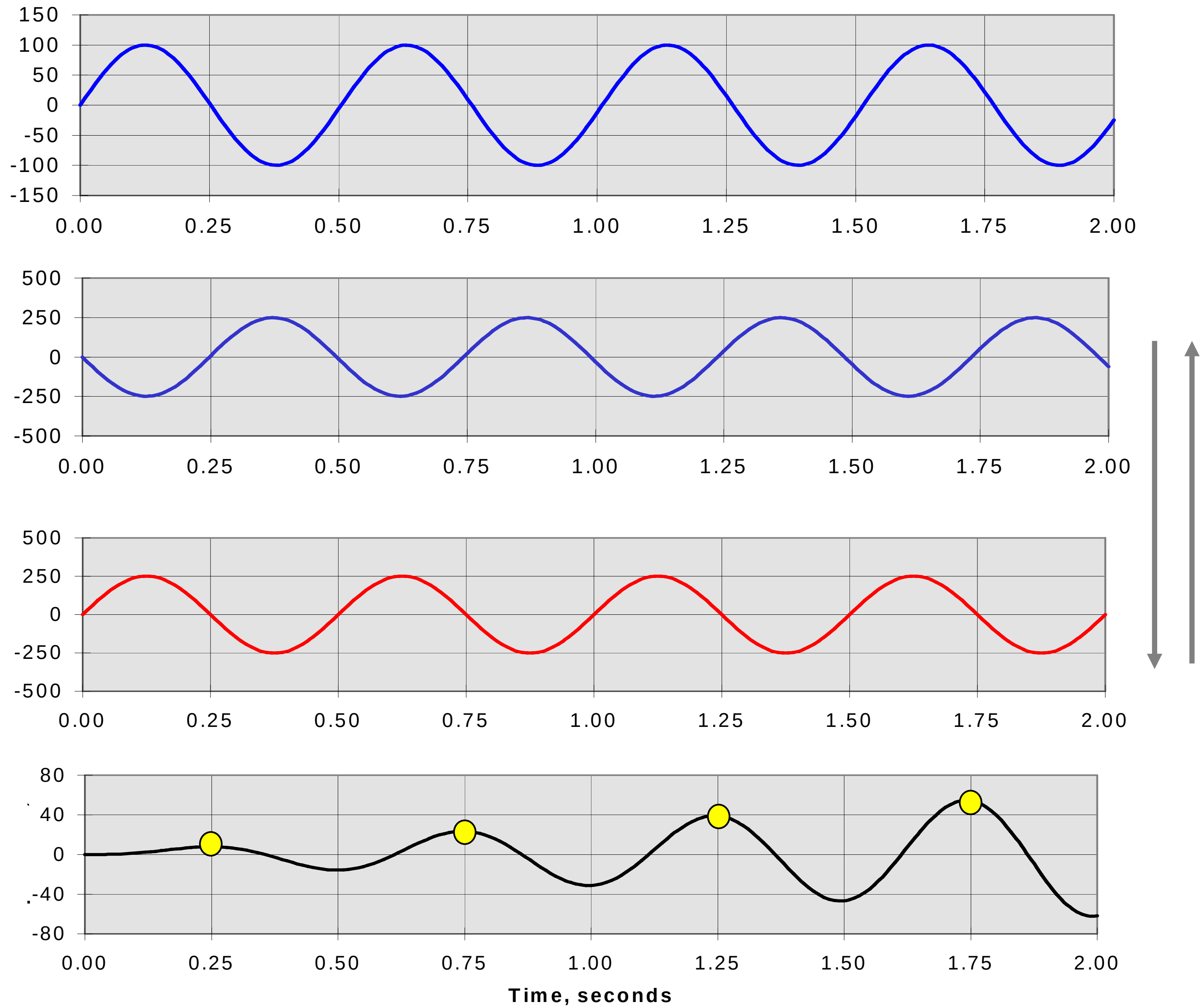
$\omega \approx 4\pi$ rad / sec $\quad \overline{\omega} = 4\pi$ rad / sec $\quad \boxed{\beta = 1.01} \quad u_S = 5.0$ in.

Loading (kips)

Steady state response (in.)

Transient response (in.)

Total response (in.)



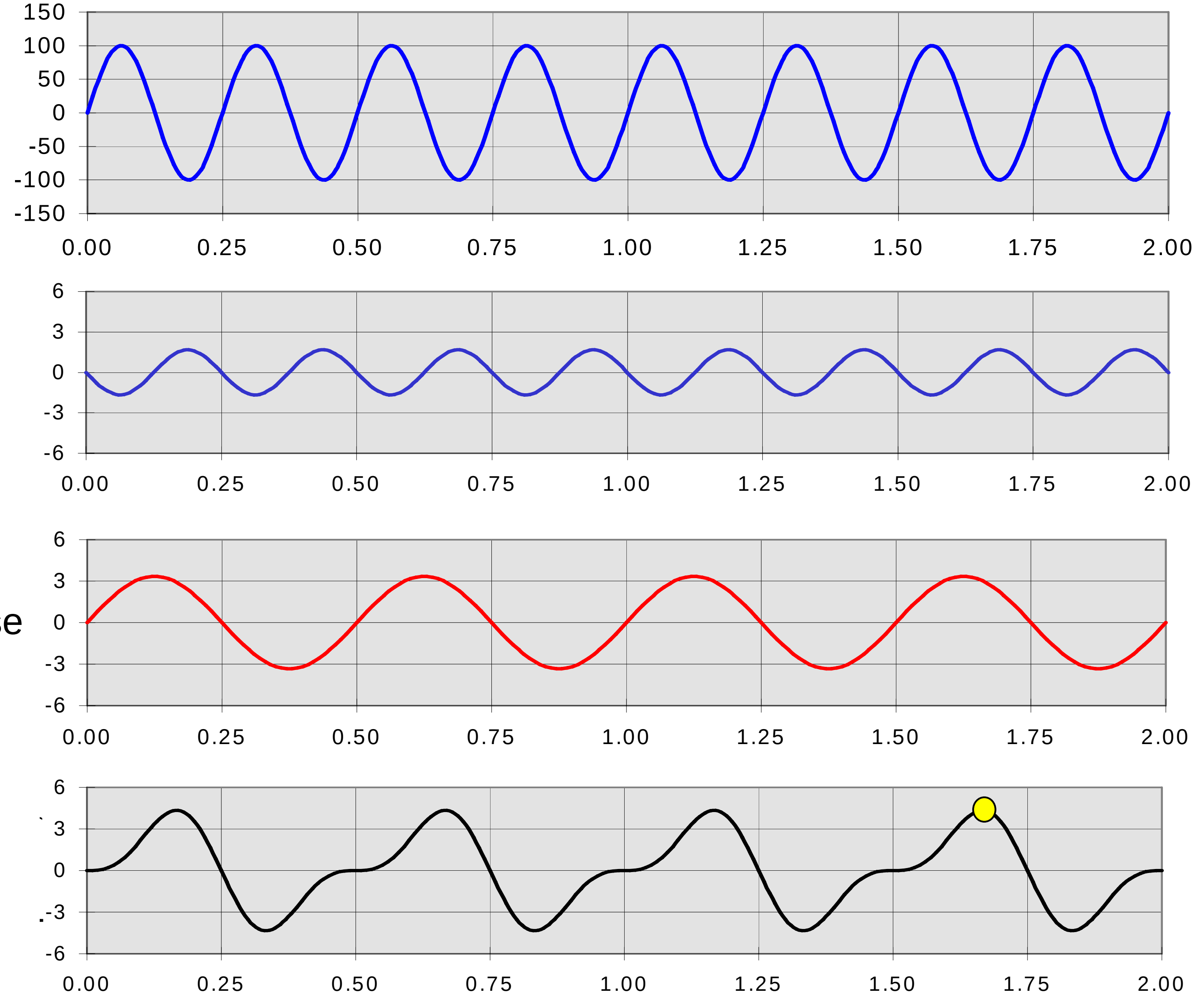
$\omega = 4\pi \text{ rad/sec} \quad \bar{\omega} = 8\pi \text{ rad/sec} \quad \boxed{\beta = 2.0} \quad u_S = 5.0 \text{ in.}$

Loading (kips)

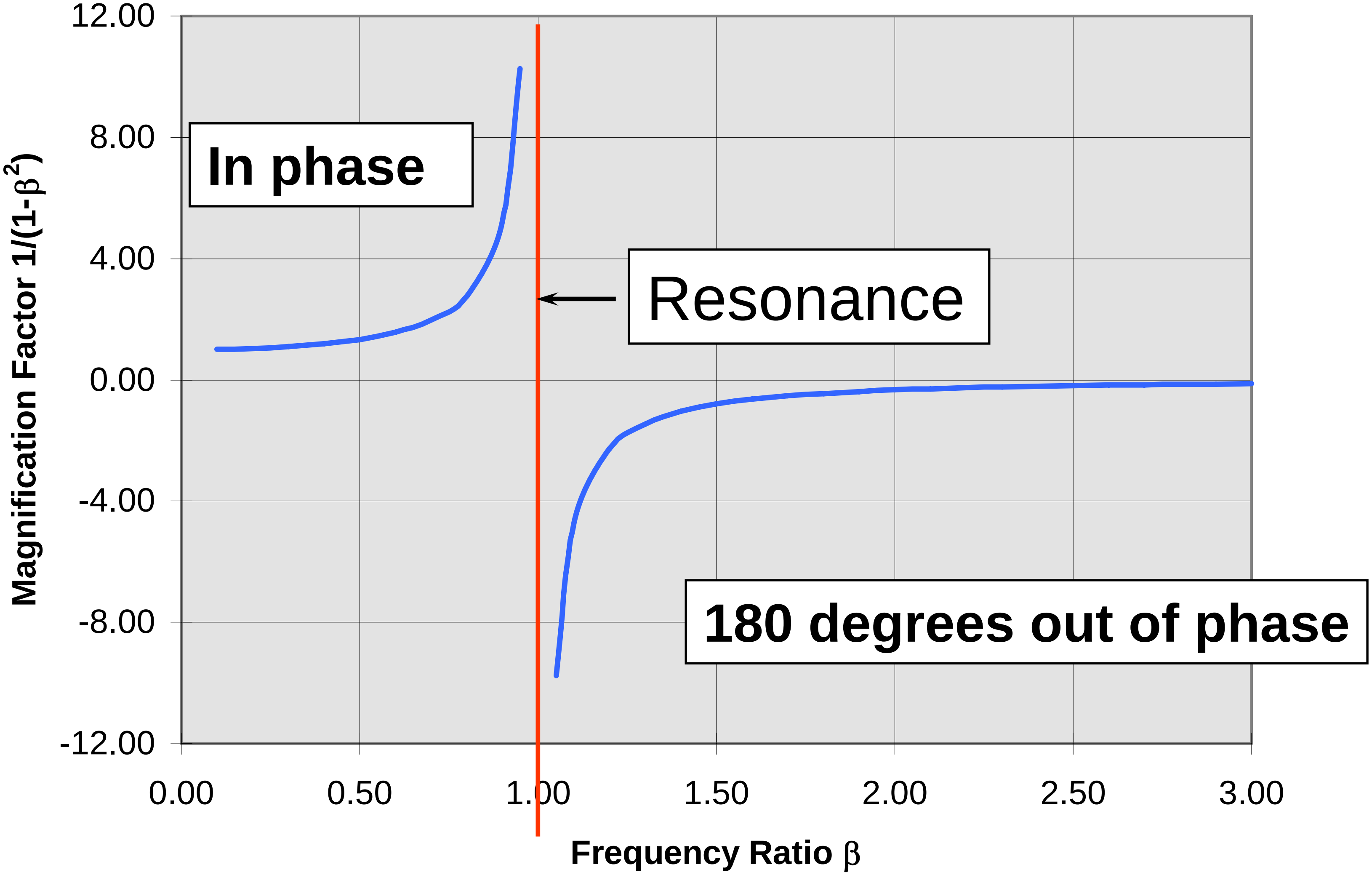
Steady state response (in.)

Transient response (in.)

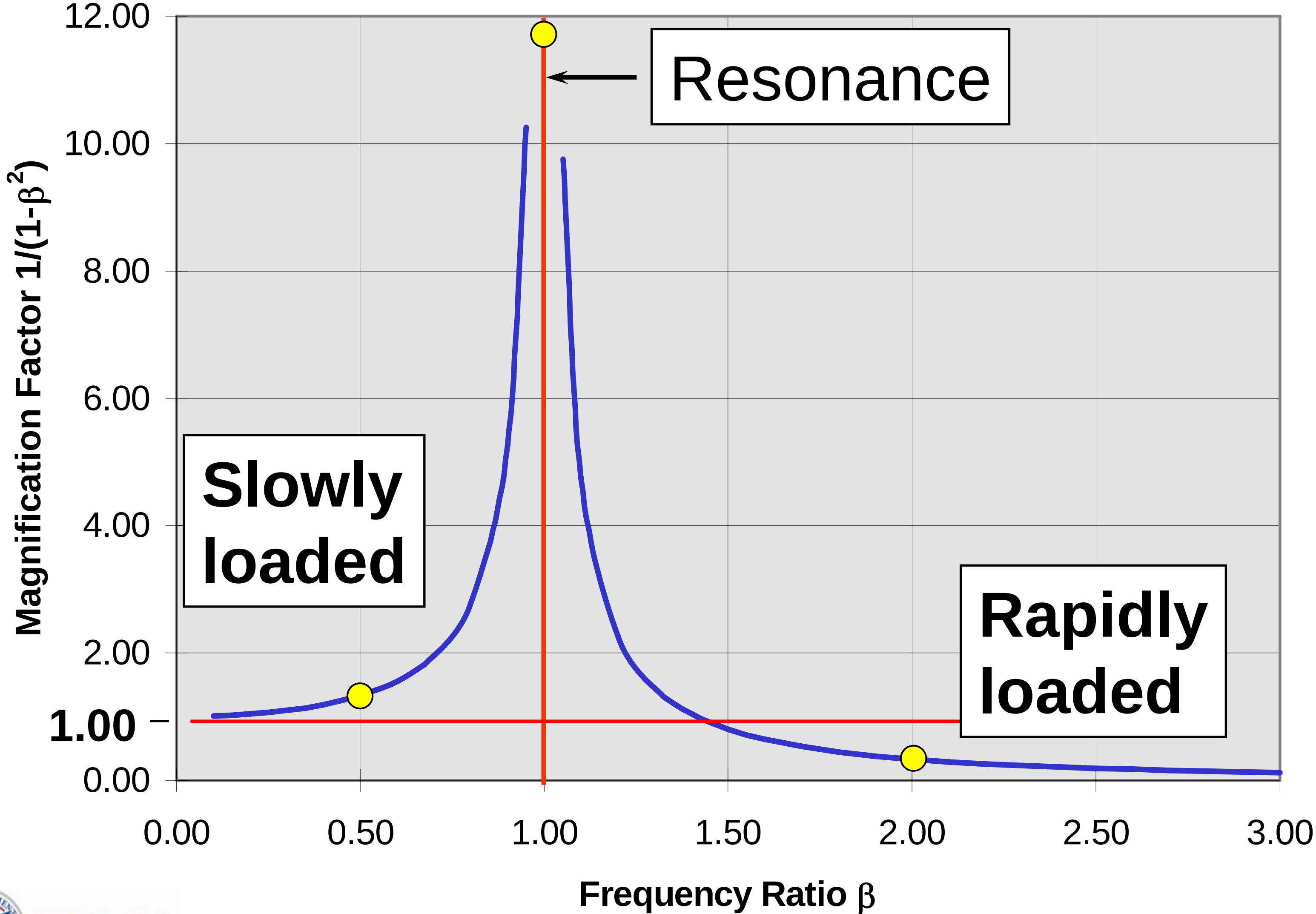
Total response (in.)



Response Ratio: Steady State to Static
(Signs Retained)



Response Ratio: Steady State to Static
(Absolute Values)

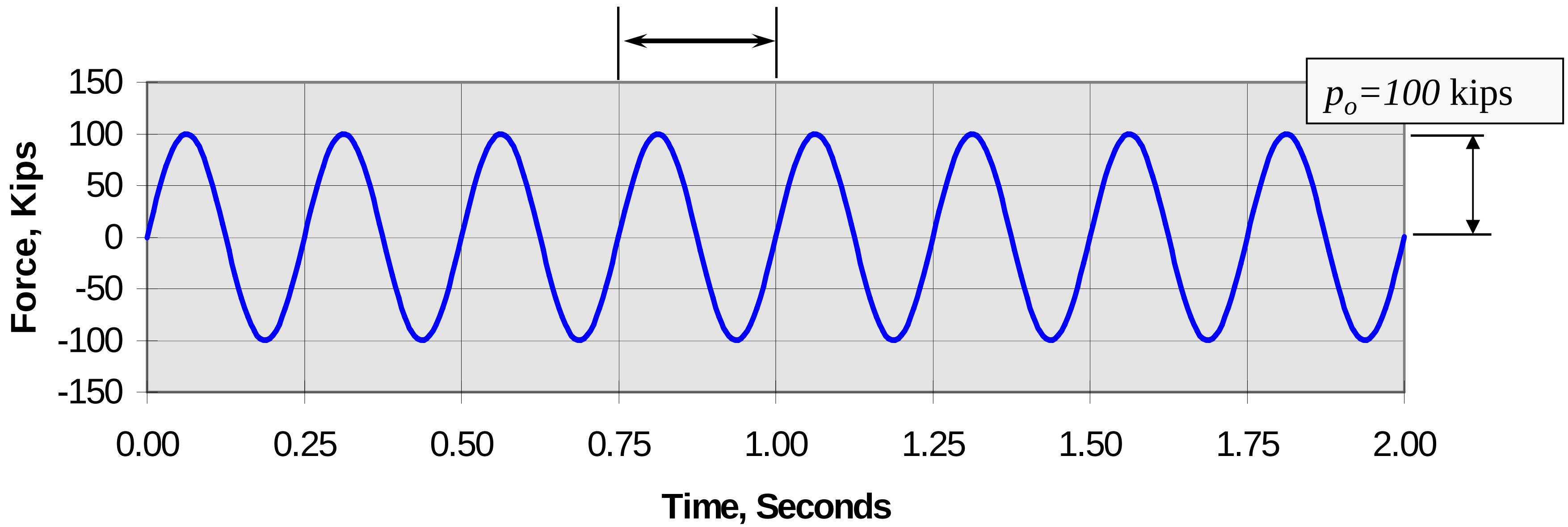


Damped Harmonic Loading

Equation of motion:

$$m\,\ddot{u}(t) + \boxed{c\dot{u}(t)} + k\,u(t) = p_0 \sin(\overline{\omega} t)$$

$$\overline{T} = \frac{2\pi}{\overline{\omega}} = 0.25\,\text{sec}$$



Damped Harmonic Loading

Equation of motion:

$$m\ddot{u}(t) + c\dot{u}(t) + k\,u(t) = p_0 \sin(\overline{\omega} t)$$

Assume system is initially at rest

Particular solution: $u(t) = C\sin(\overline{\omega} t) + D\cos(\overline{\omega} t)$

Complimentary solution:

$$u(t) = e^{-\xi\omega t}\left[A\sin(\omega_D t) + B\cos(\omega_D t)\right]$$

$$\xi = \frac{c}{2m\omega}$$

Solution:

$$u(t) = e^{-\xi\omega t}\left[A\sin(\omega_D t) + B\cos(\omega_D t)\right] + C\sin(\overline{\omega} t) + D\cos(\overline{\omega} t)$$

$$\omega_D = \omega\sqrt{1-\xi^2}$$

Damped Harmonic Loading

Transient response at structure's frequency (eventually damps out)

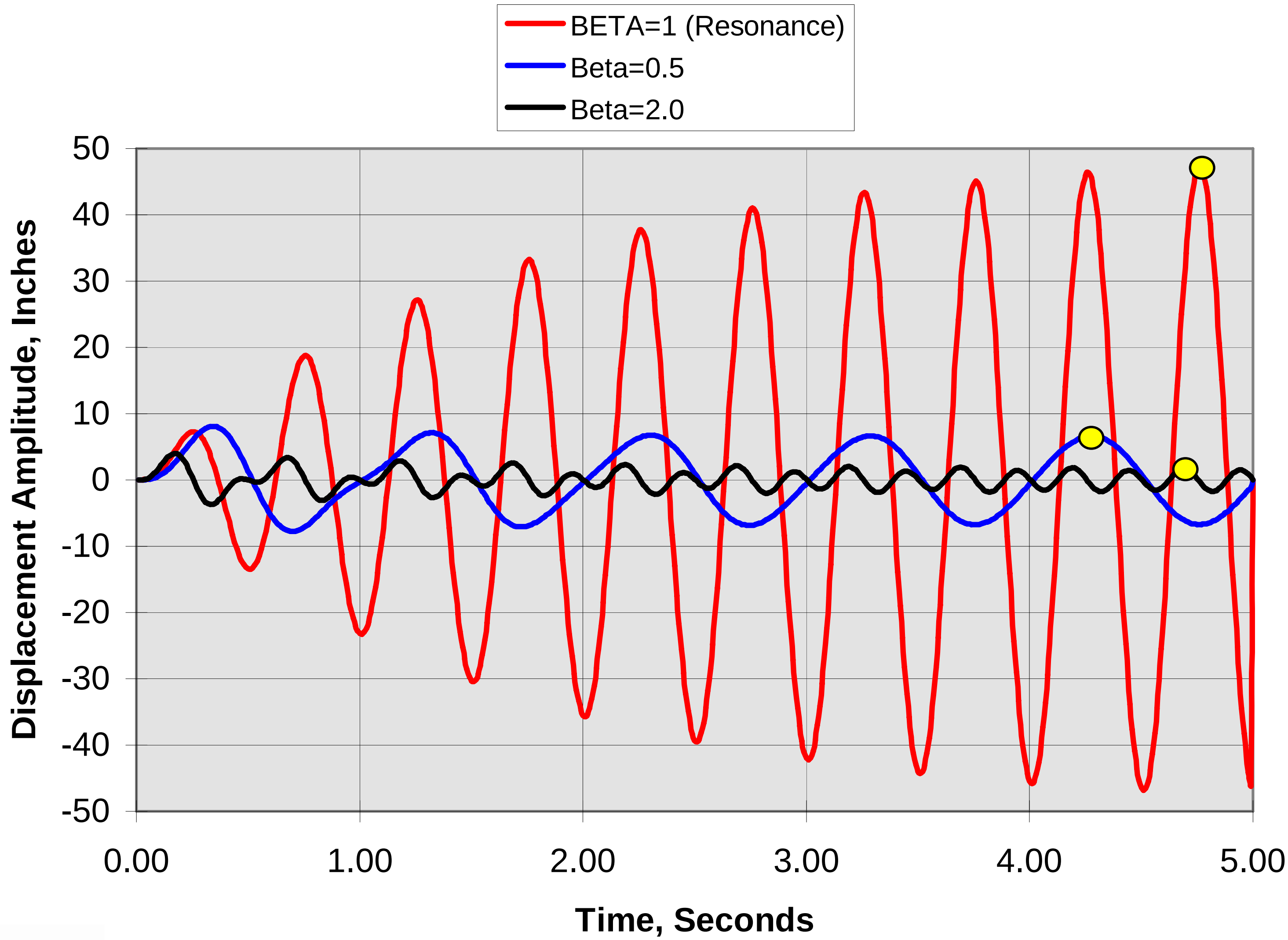
$$u(t) = \boxed{e^{-\xi\omega t}\left[A\sin(\omega_D t) + B\cos(\omega_D t)\right]} +$$

$$\boxed{C\sin(\overline{\omega}t) + D\cos(\overline{\omega}t)}$$

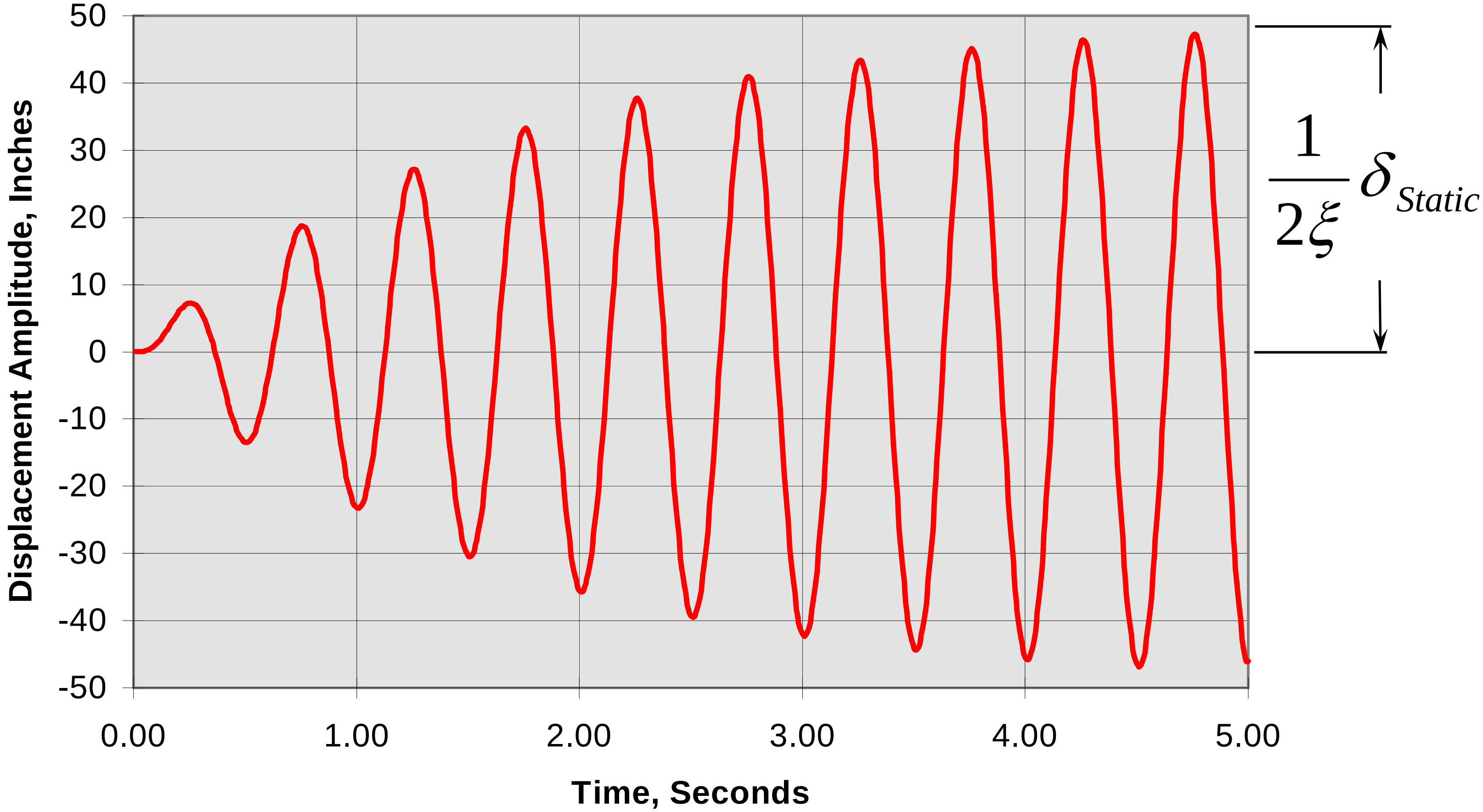
Steady state response, at loading frequency

$$C = \frac{p_o}{k}\frac{1-\beta^2}{(1-\beta^2)^2 + (2\xi\beta)^2} \qquad D = \frac{p_o}{k}\frac{-2\xi\beta}{(1-\beta^2)^2 + (2\xi\beta)^2}$$

Damped Harmonic Loading (5% Damping)

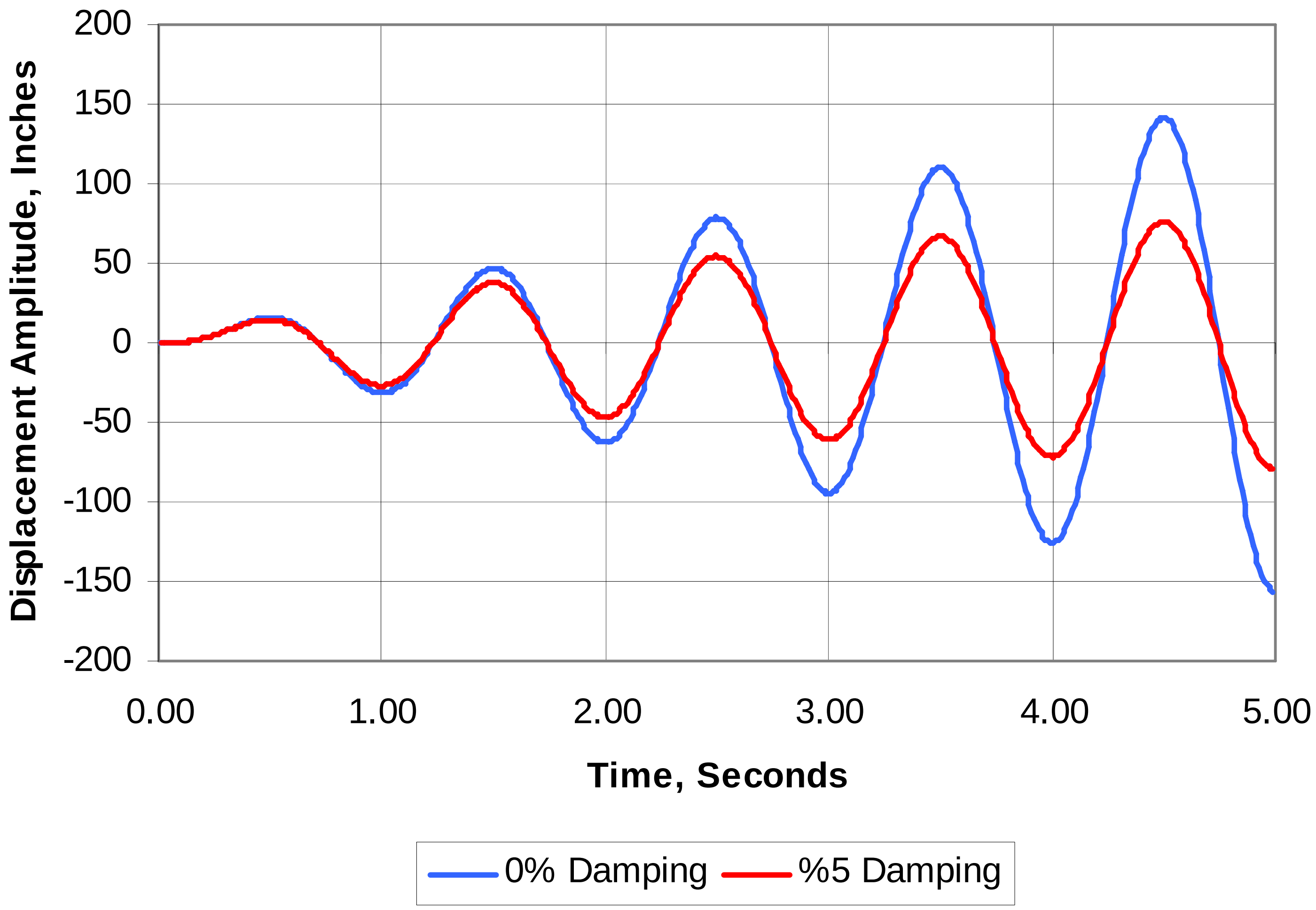


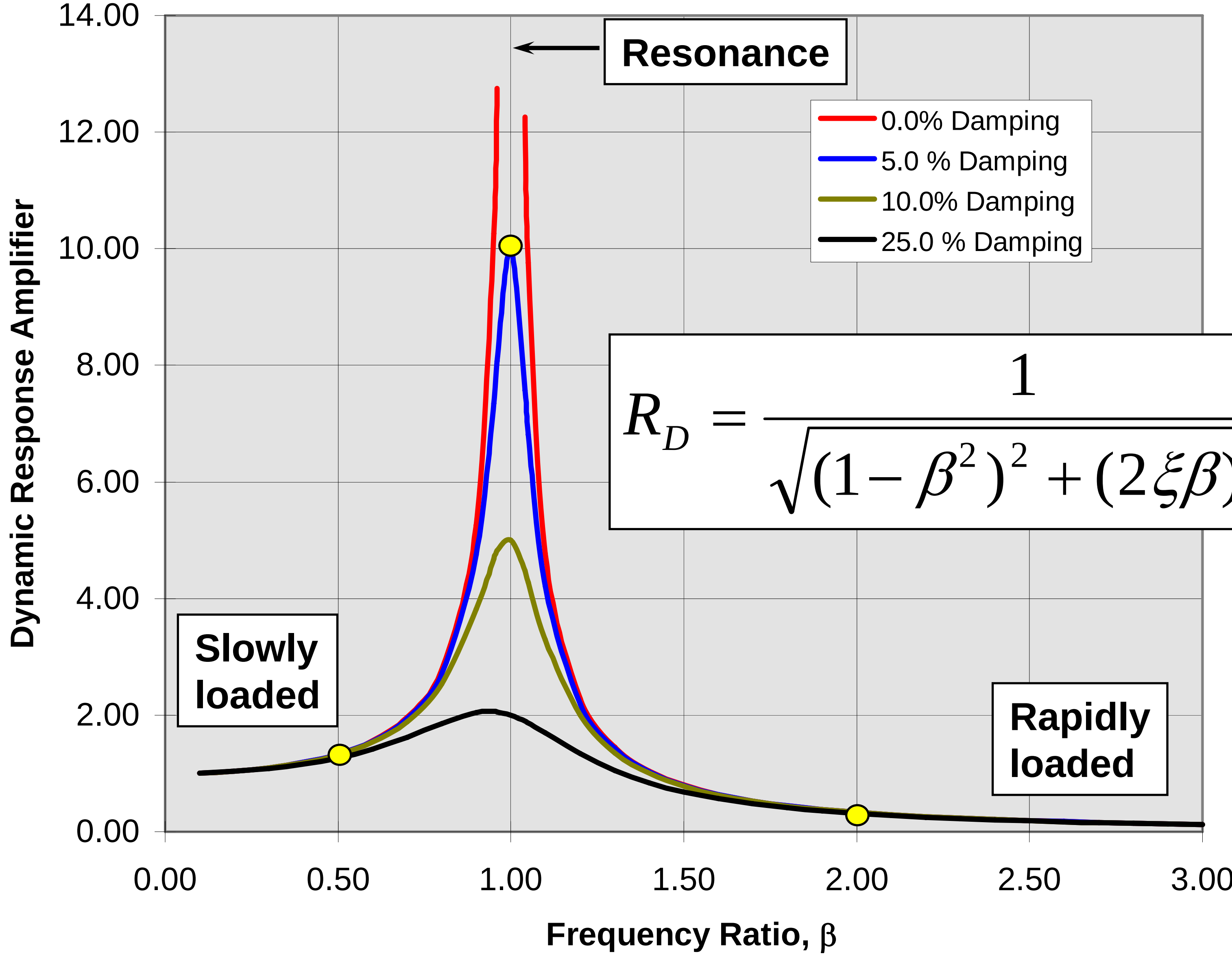
Damped Harmonic Loading (5% Damping)



Harmonic Loading at Resonance

Effects of Damping





$$R_D = \frac{1}{\sqrt{(1-\beta^2)^2 + (2\xi\beta)^2}}$$

Summary Regarding Viscous Damping in Harmonically Loaded Systems

- For systems loaded at a frequency near their natural frequency, the dynamic response exceeds the static response. This is referred to as *dynamic amplification*.

- An undamped system, loaded at resonance, will have an unbounded increase in displacement over time.

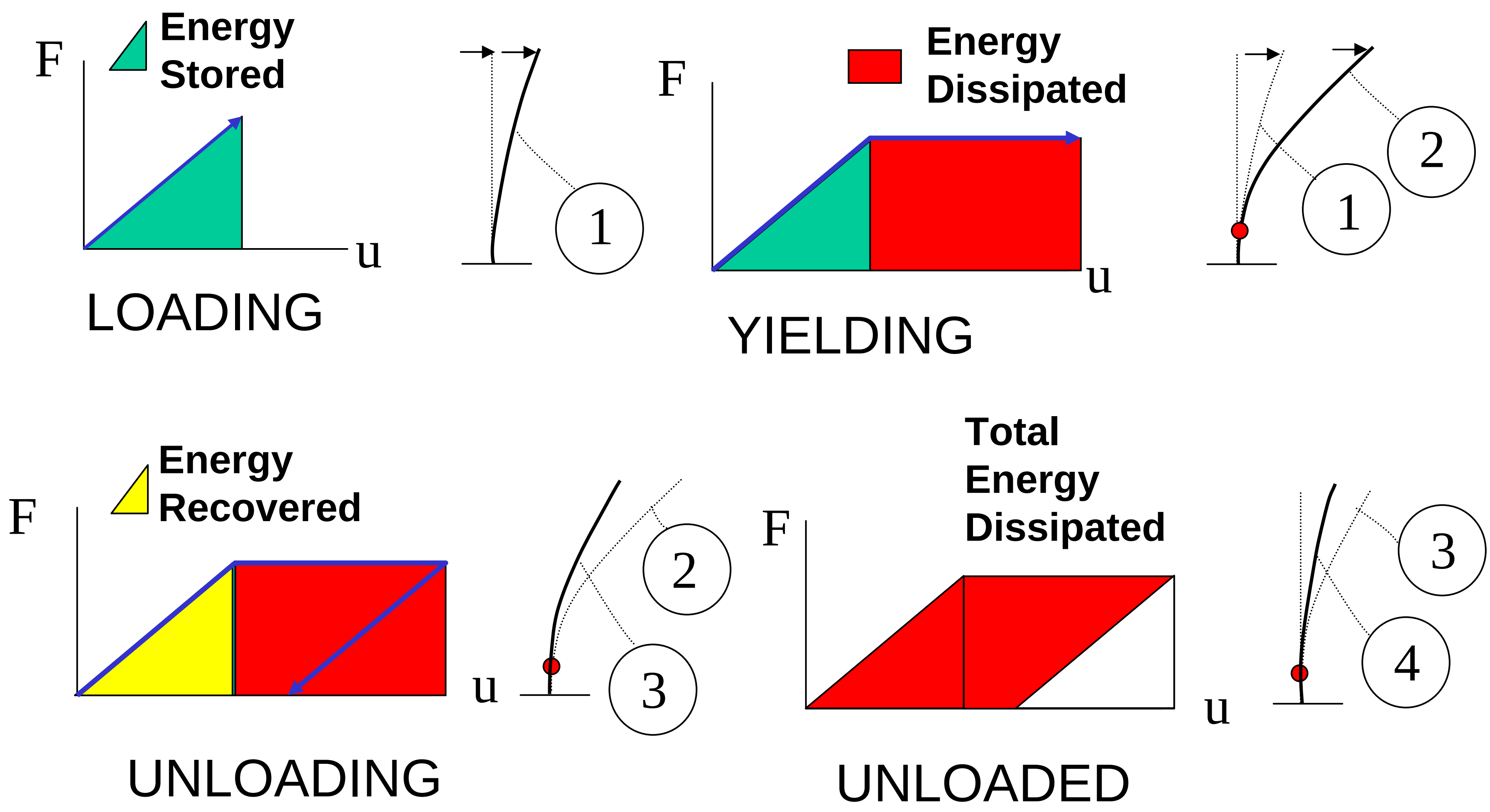
Summary Regarding Viscous Damping in Harmonically Loaded Systems

- Damping is an effective means for *dissipating energy* in the system. Unlike strain energy, which is recoverable, dissipated energy is not recoverable.

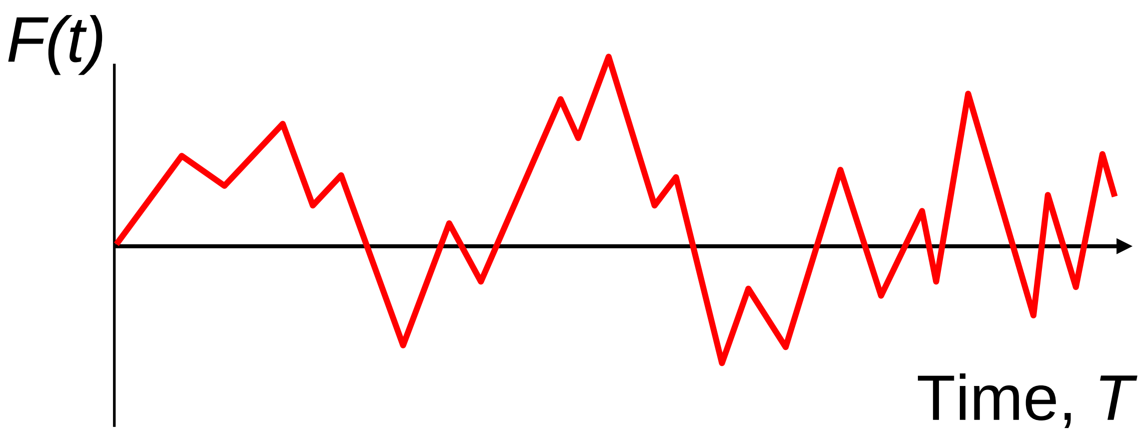
- A damped system, loaded at resonance, will have a limited displacement over time with the limit being $(1/2\xi)$ times the static displacement.

- Damping is most effective for systems loaded at or near resonance.

CONCEPT of ENERGY STORED and Energy DISSIPATED



General Dynamic Loading

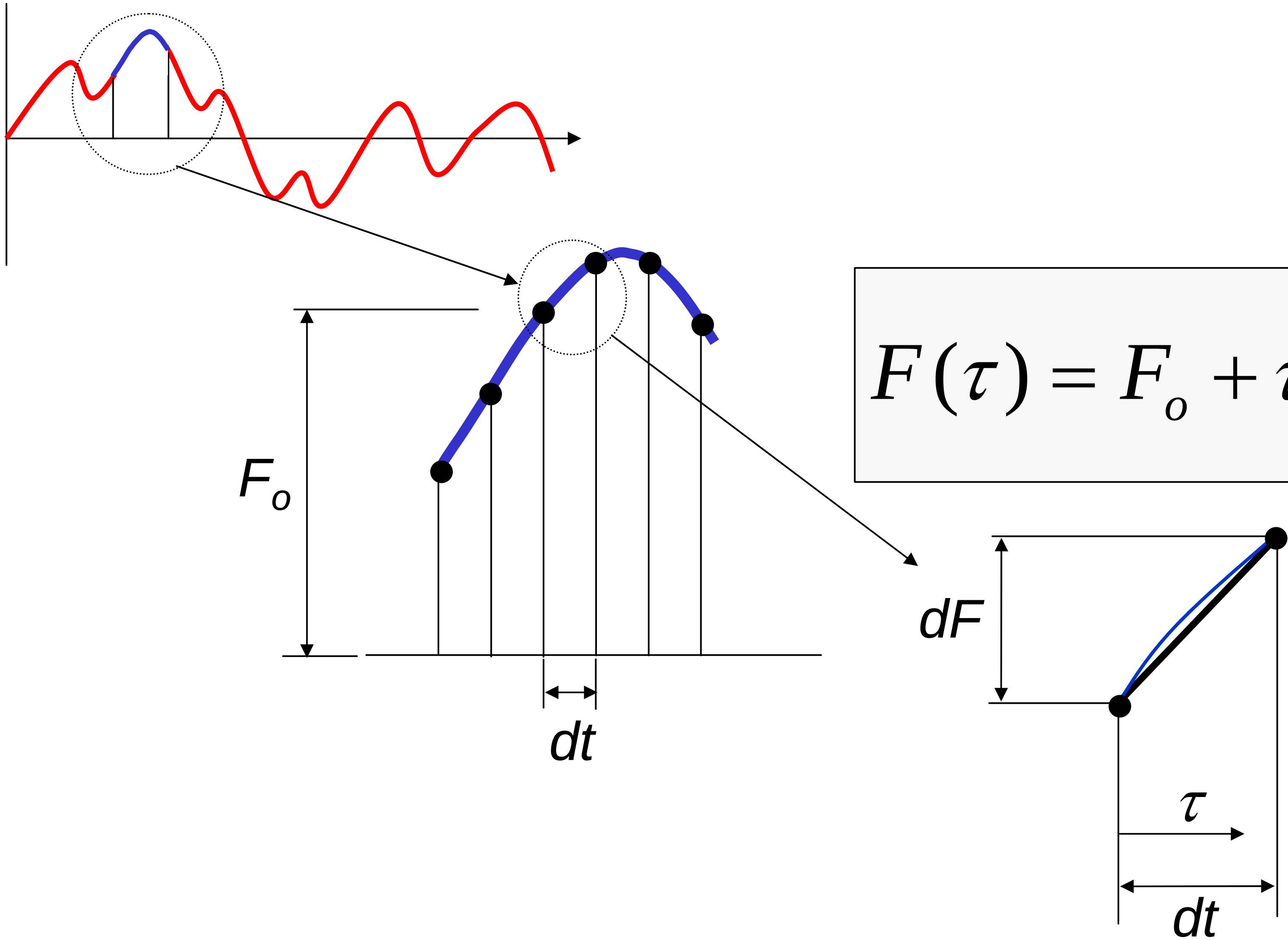


General Dynamic Loading Solution Techniques

- Fourier transform
- Duhamel integration
- Piecewise exact
- Newmark techniques

All techniques are carried out numerically.

Piecewise Exact Method



$$F(\tau) = F_o + \tau \frac{dF}{dt}$$

Piecewise Exact Method

Initial conditions $u_{o,0} = 0 \quad \dot{u}_{o,0} = 0$

Determine "exact" solution for 1st time step

$$u_1 = u(\tau) \qquad \dot{u}_1 = \dot{u}(\tau) \qquad \ddot{u}_1 = \ddot{u}(\tau)$$

Establish new initial conditions

$$u_{o,1} = u(\tau) \qquad \dot{u}_{0,1} = \dot{u}(\tau)$$

LOOP

Obtain exact solution for next time step

$$u_2 = u(\tau) \qquad \dot{u}_2 = \dot{u}(\tau) \qquad \ddot{u}_2 = \ddot{u}(\tau)$$

Piecewise Exact Method

Advantages:

- Exact if load increment is linear
- Very computationally efficient

Disadvantages:

- Not generally applicable for inelastic behavior

Note: NONLIN uses the piecewise exact method for response spectrum calculations.

Newmark Techniques

- Proposed by Nathan Newmark
- General method that encompasses a family of different integration schemes
- Derived by:
  - Development of incremental equations of motion
  - Assuming acceleration response over short time step

Newmark Method

Advantages:

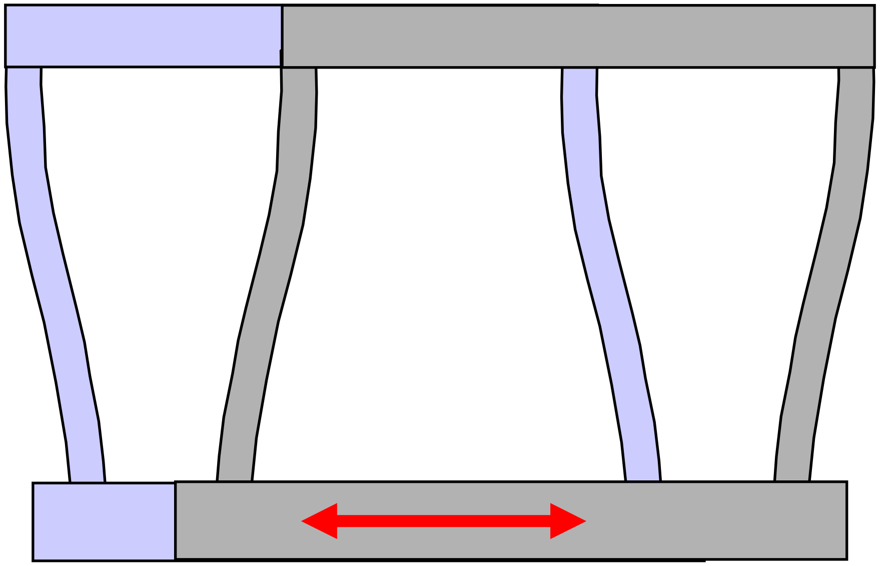
- Works for inelastic response

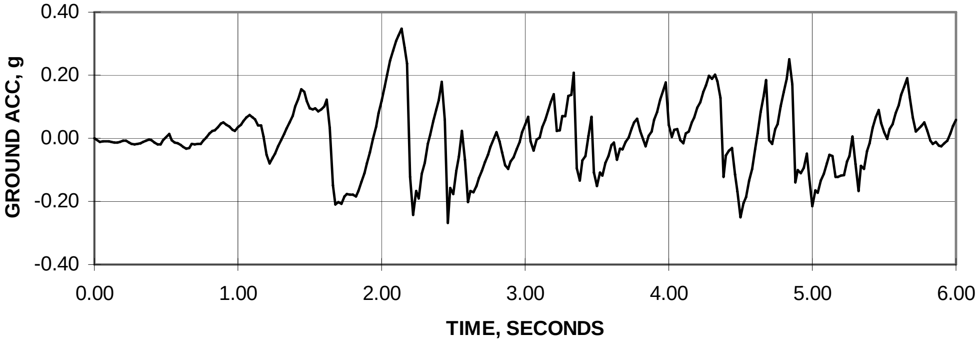
Disadvantages:

- Potential numerical error

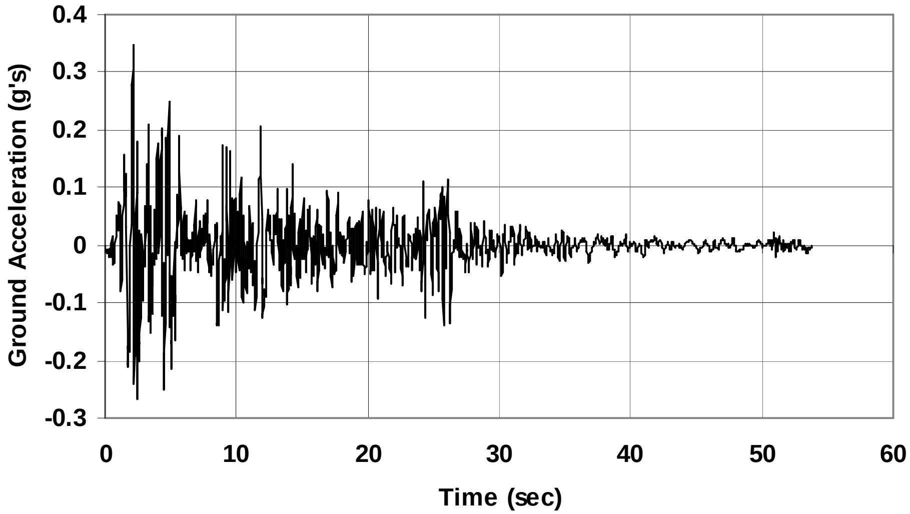
Note: NONLIN uses the Newmark method for general response history calculations

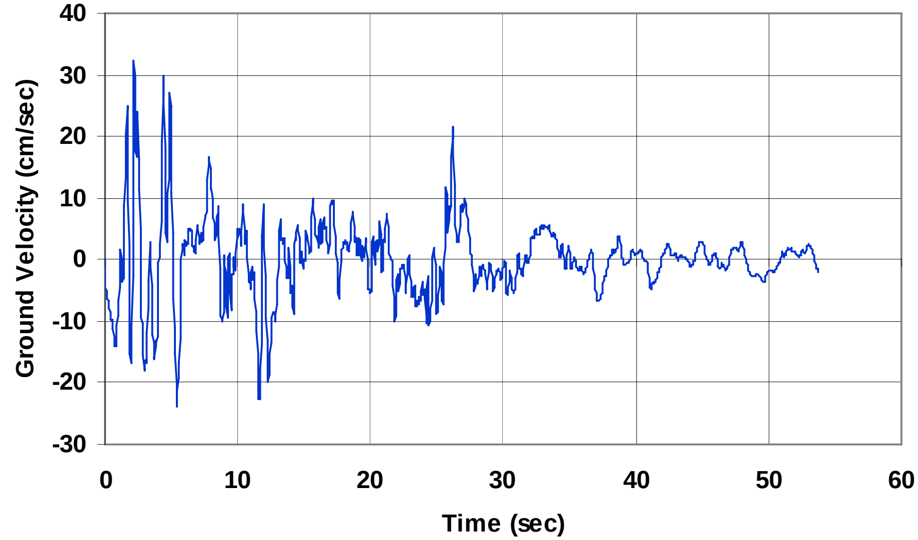
Development of Effective Earthquake Force

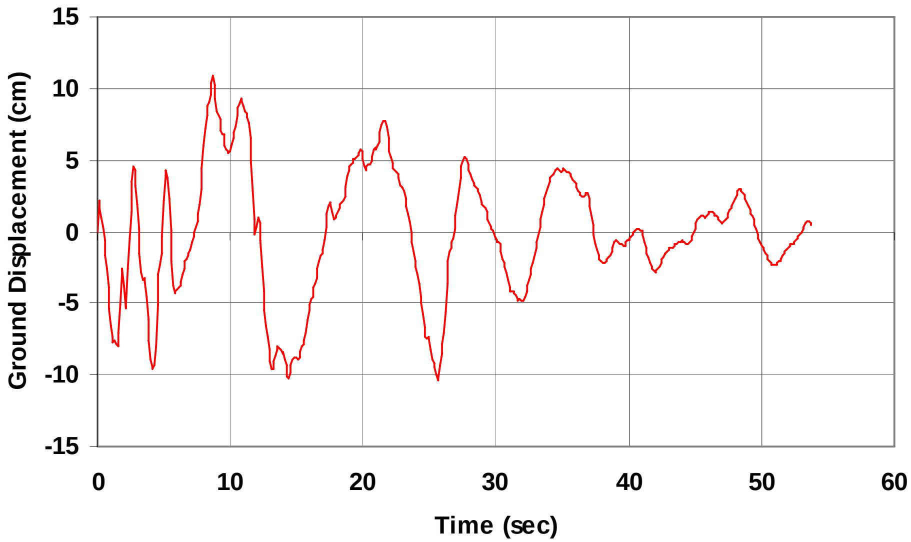




Earthquake Ground Motion, 1940 El Centro

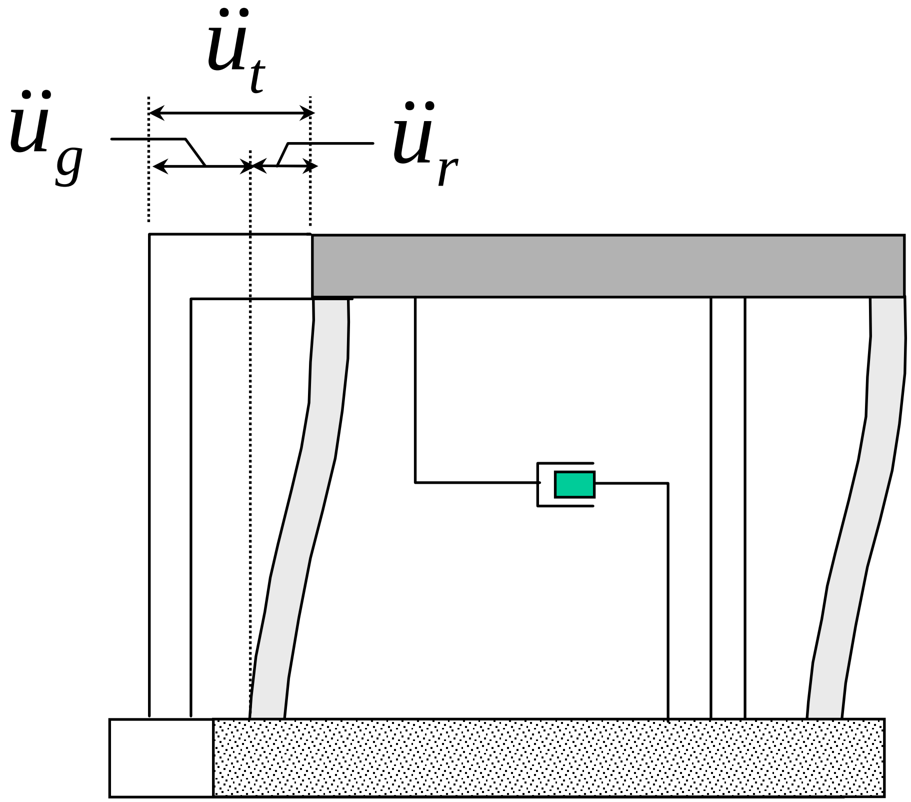


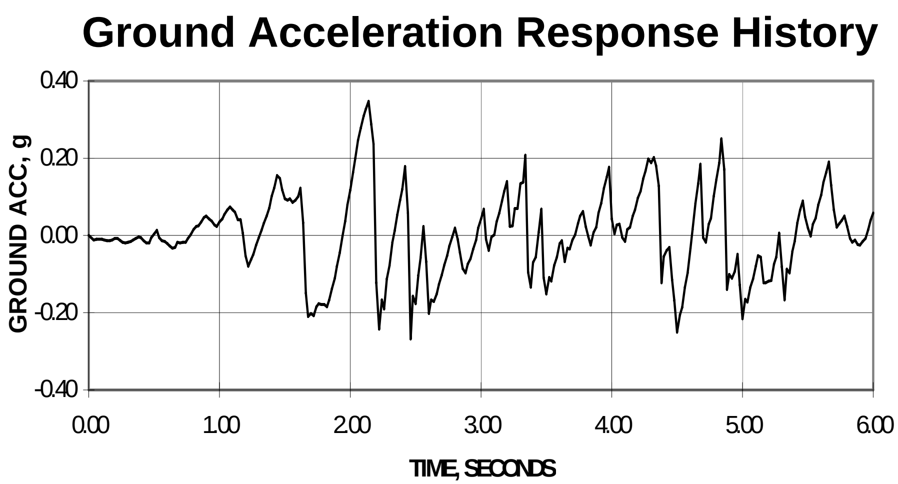




Many ground motions now are available via the Internet.

Development of Effective Earthquake Force





$$m[\ddot{u}_g(t) + \ddot{u}_r(t)] + c\,\dot{u}_r(t) + k\,u_r(t) = 0$$

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

"Simplified" form of Equation of Motion:

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

Divide through by ***m:***

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

Make substitutions:

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

Simplified form:

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

For a given ground motion, the response history $u_r(t)$ is function of the structure's frequency ω and damping ratio ξ.

Structural frequency

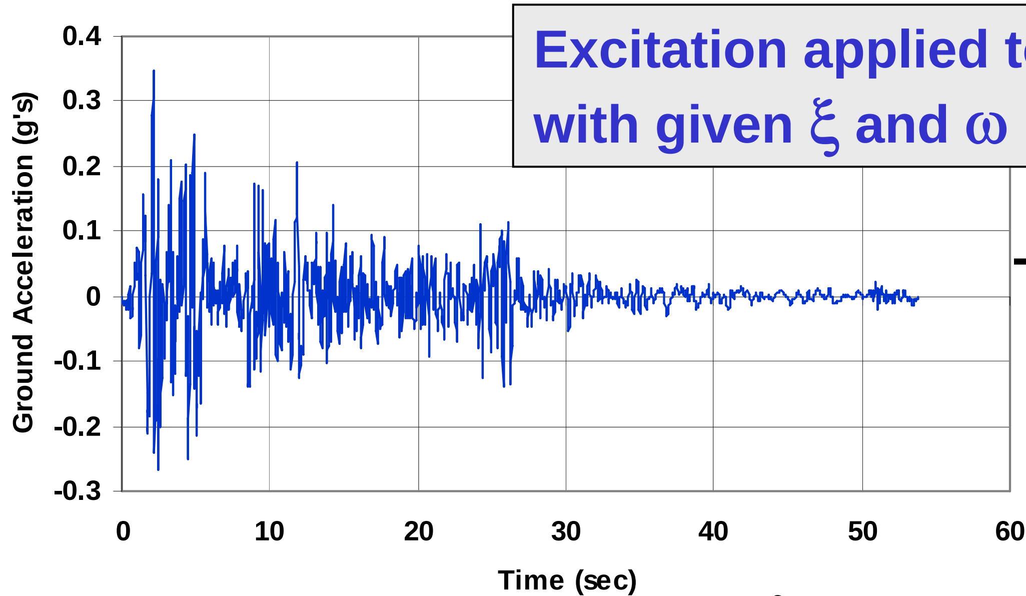
$$\ddot{u}_r(t) + 2\xi\omega\dot{u}_r(t) + \omega^2 u_r(t) = -\ddot{u}_g(t)$$

Damping ratio

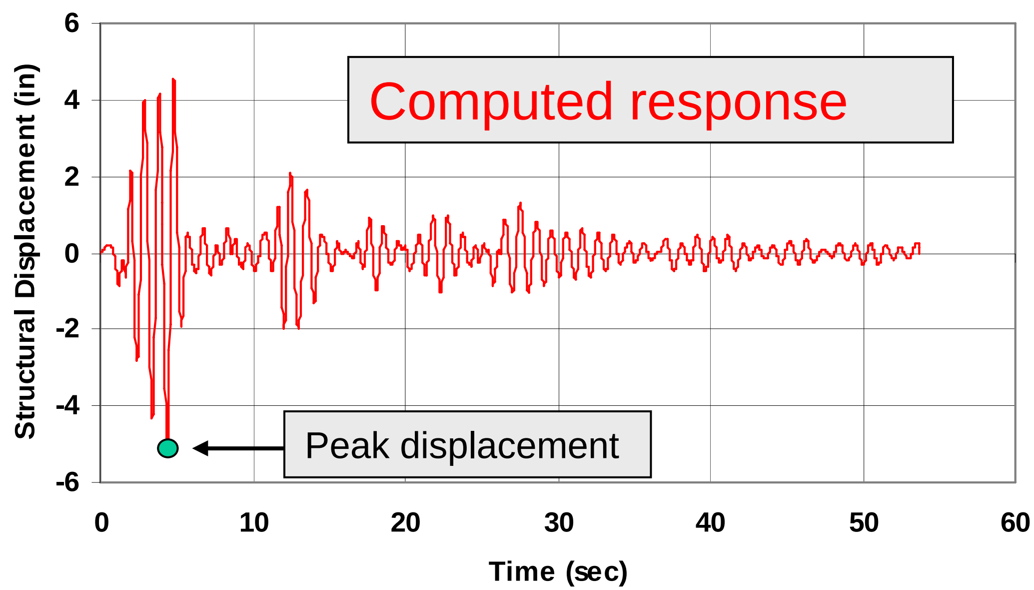
Ground motion acceleration history

Response to Ground Motion (1940 El Centro)

Excitation applied to structure with given ξ and ω



SOLVER

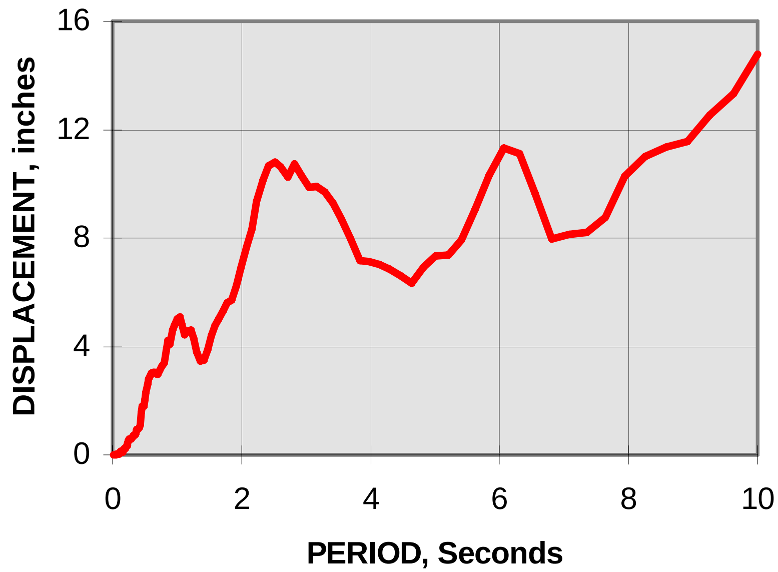


Change in ground motion or structural parameters ξ and ω requires re-calculation of structural response

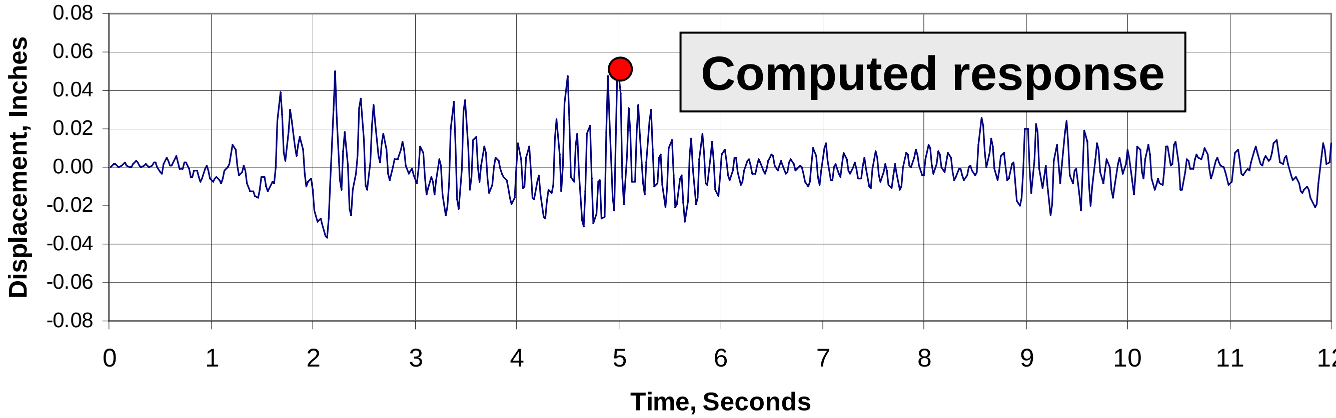
The Elastic Displacement Response Spectrum

An ***elastic displacement response spectrum*** is a plot of the peak computed relative displacement, u_r, for an elastic structure with a constant damping ξ, a varying fundamental frequency ω (or period $T = 2\pi/\omega$), responding to a given ground motion.

5% damped response spectrum for structure responding to 1940 El Centro ground motion



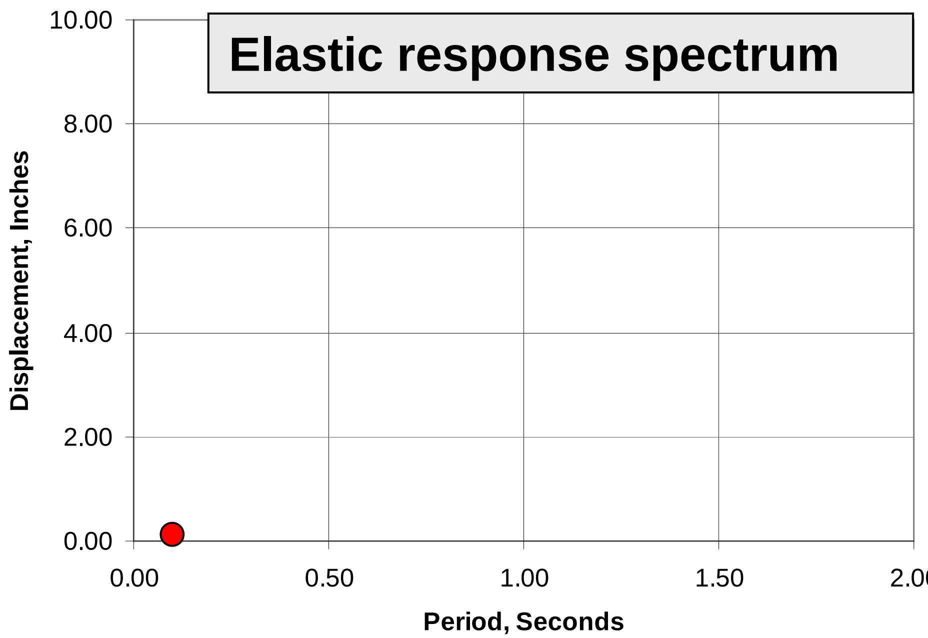
Computation of Response Spectrum for El Centro Ground Motion



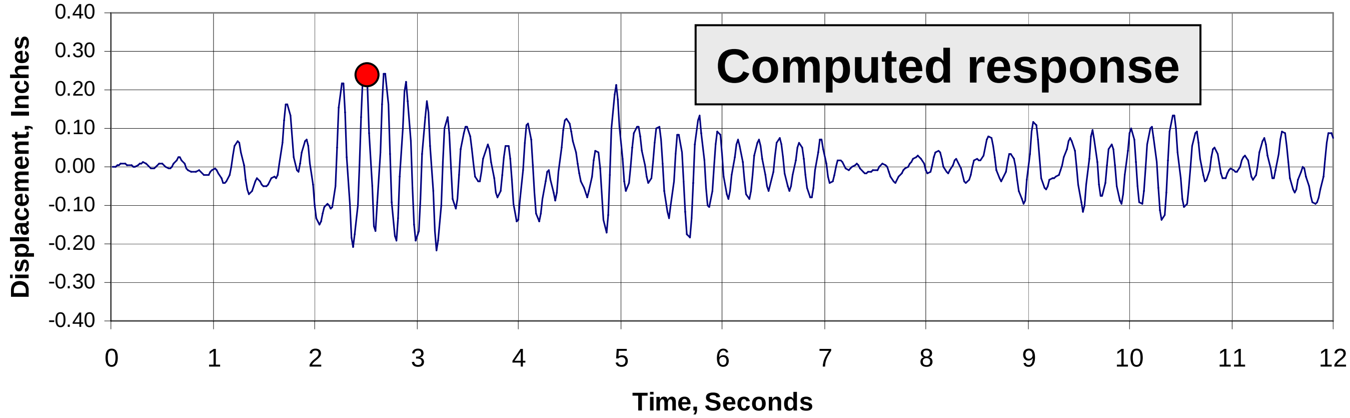
$\xi = 0.05$

$T = 0.10$ sec

$U_{max} = 0.0543$ in.



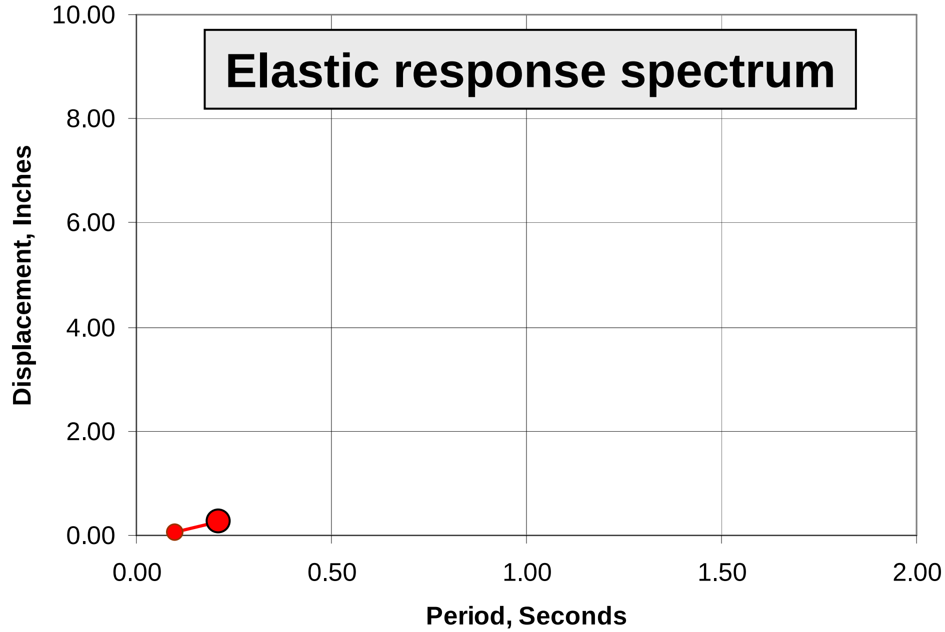
Computation of Response Spectrum for El Centro Ground Motion



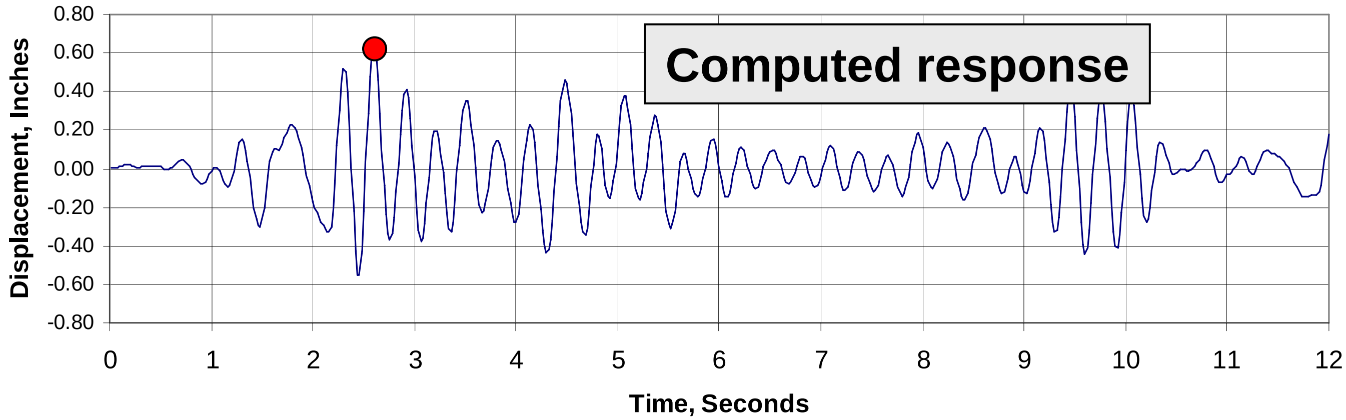
$\xi = 0.05$

$T = 0.20$ sec

$U_{max} = 0.254$ in.



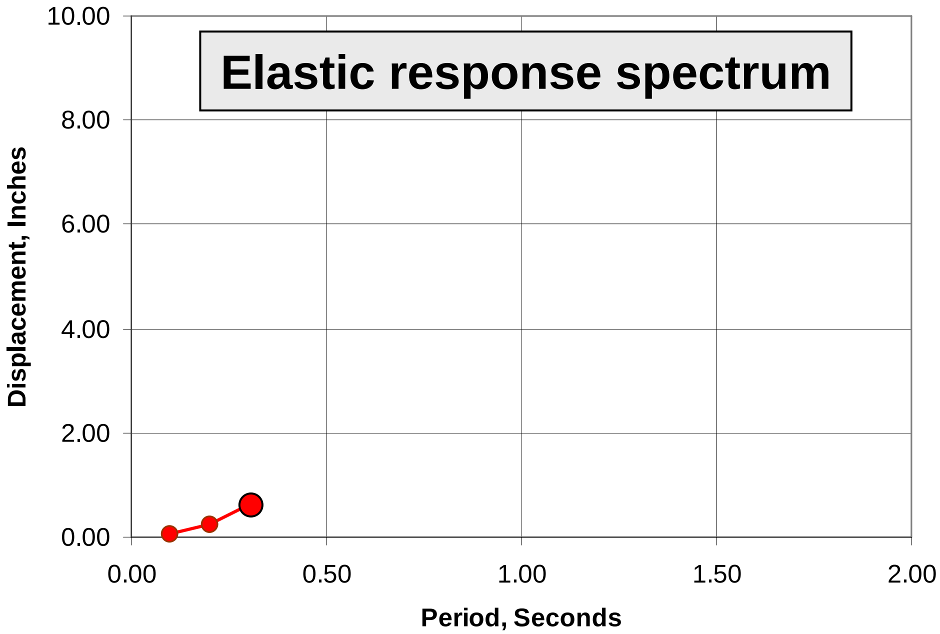
Computation of Response Spectrum for El Centro Ground Motion



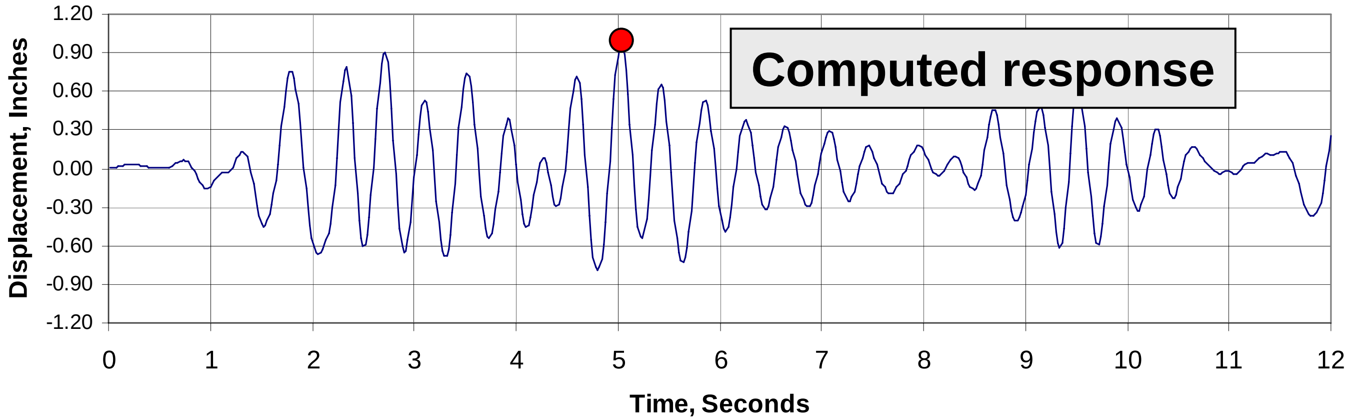
$\xi = 0.05$

$T = 0.30$ sec

$U_{max} = 0.622$ in.



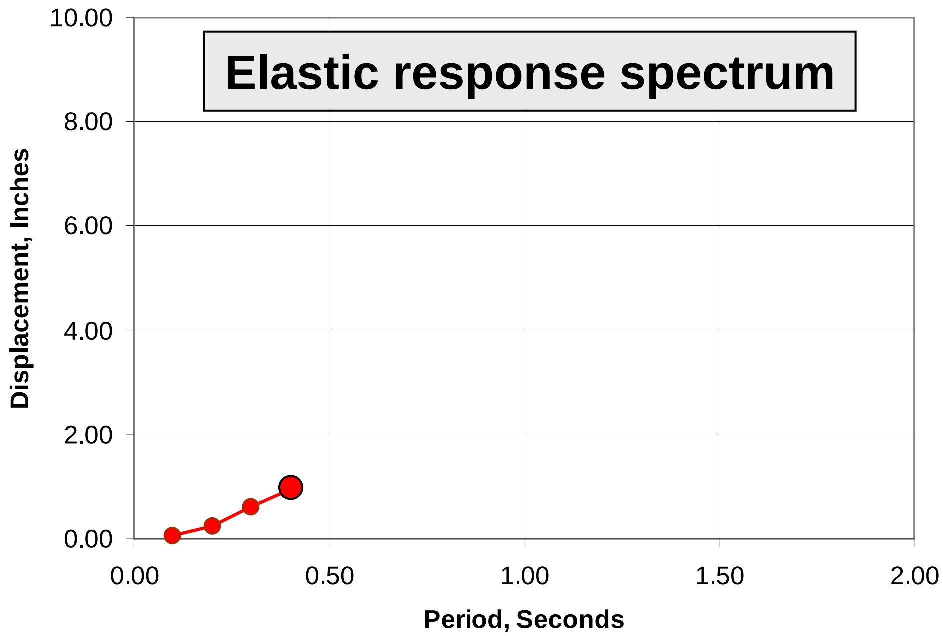
Computation of Response Spectrum for El Centro Ground Motion



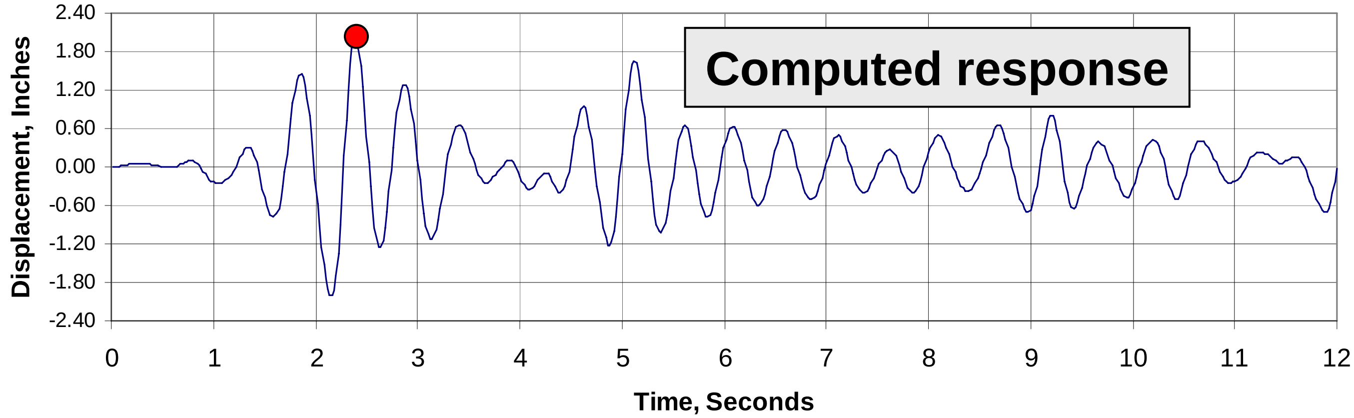
$\xi = 0.05$

$T = 0.40$ sec

$U_{max} = 0.956$ in.



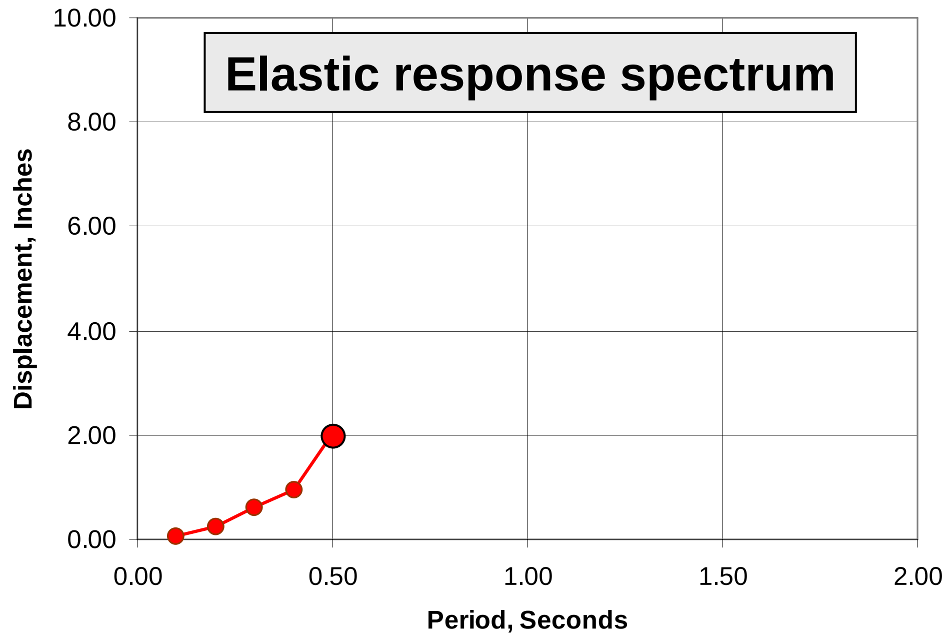
Computation of Response Spectrum for El Centro Ground Motion



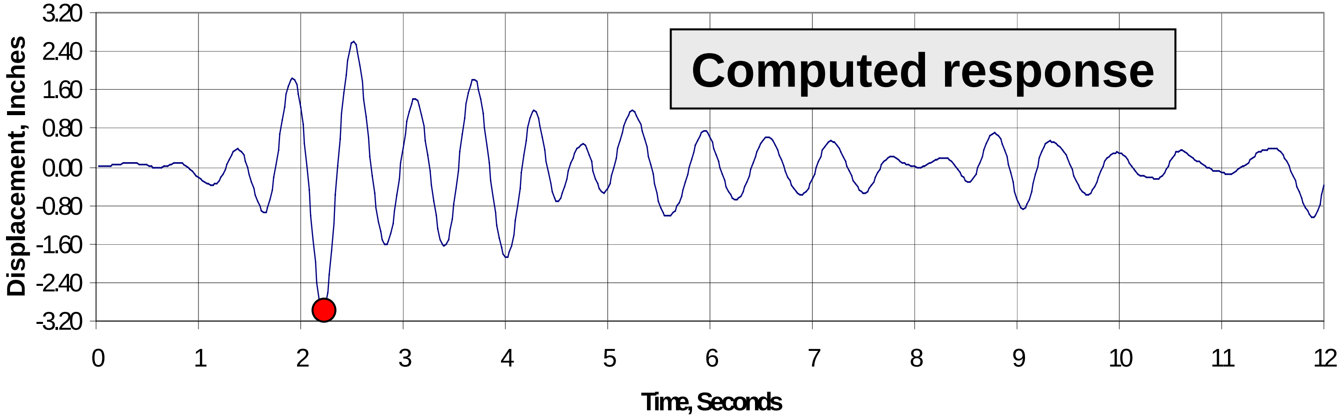
$\xi = 0.05$

$T = 0.50$ sec

$U_{max} = 2.02$ in.



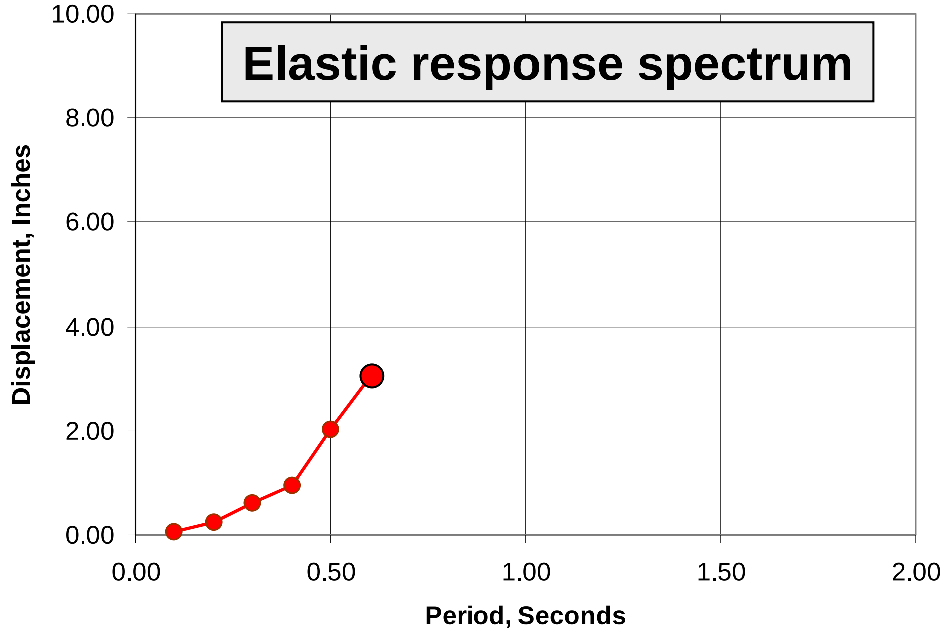
Computation of Response Spectrum for El Centro Ground Motion



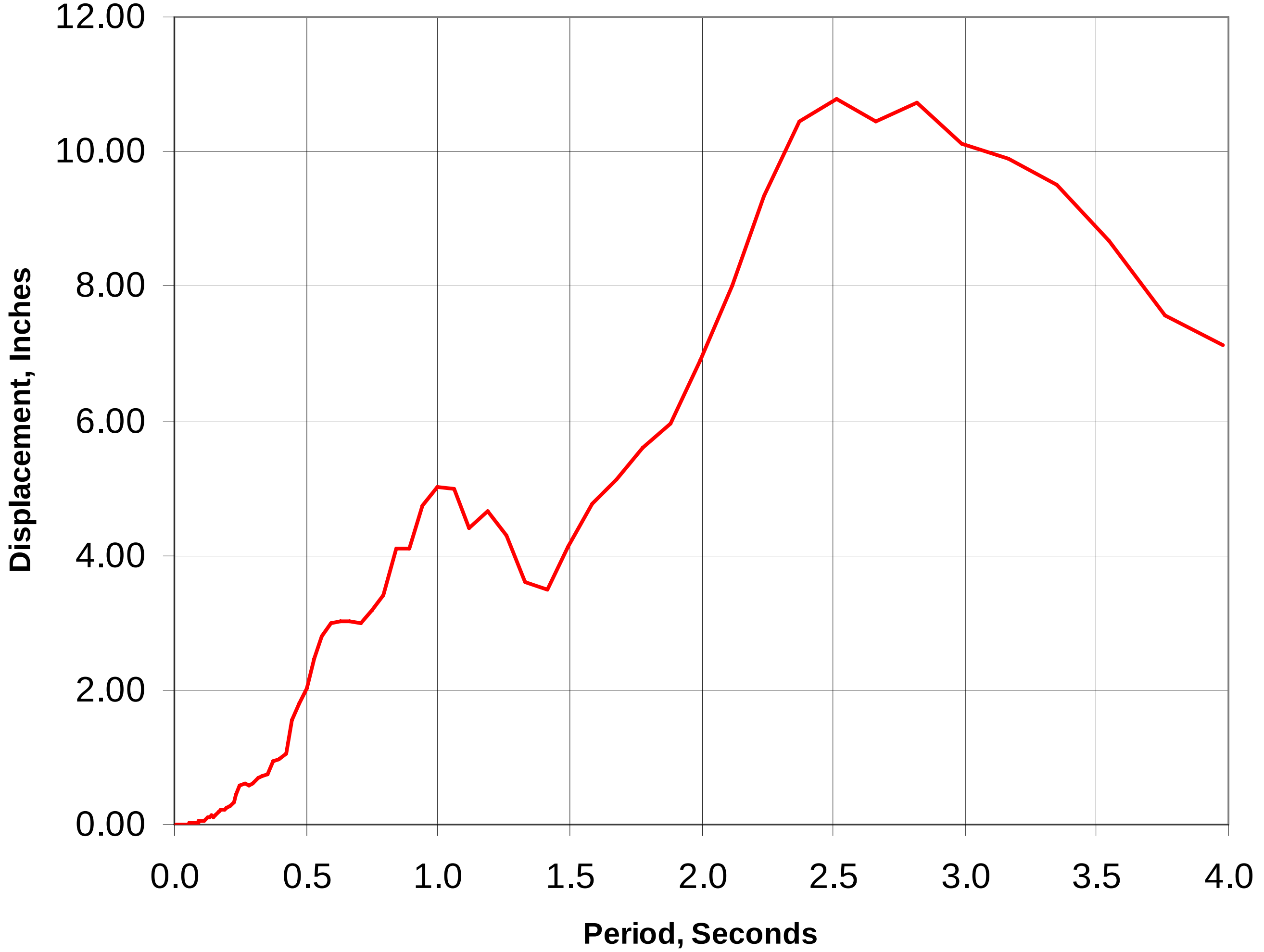
$\xi = 0.05$

$T = 0.60$ sec

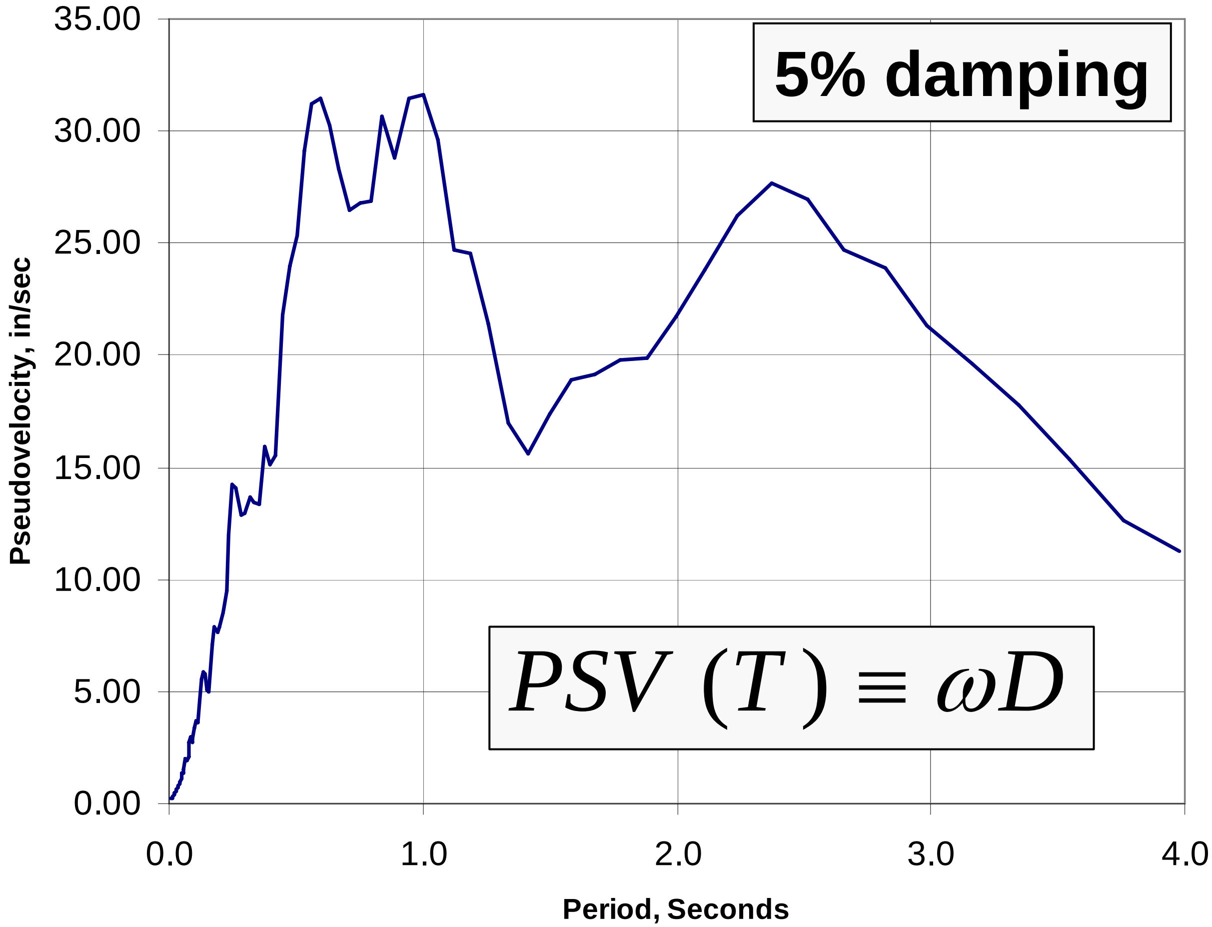
$U_{max} = -3.00$ in.



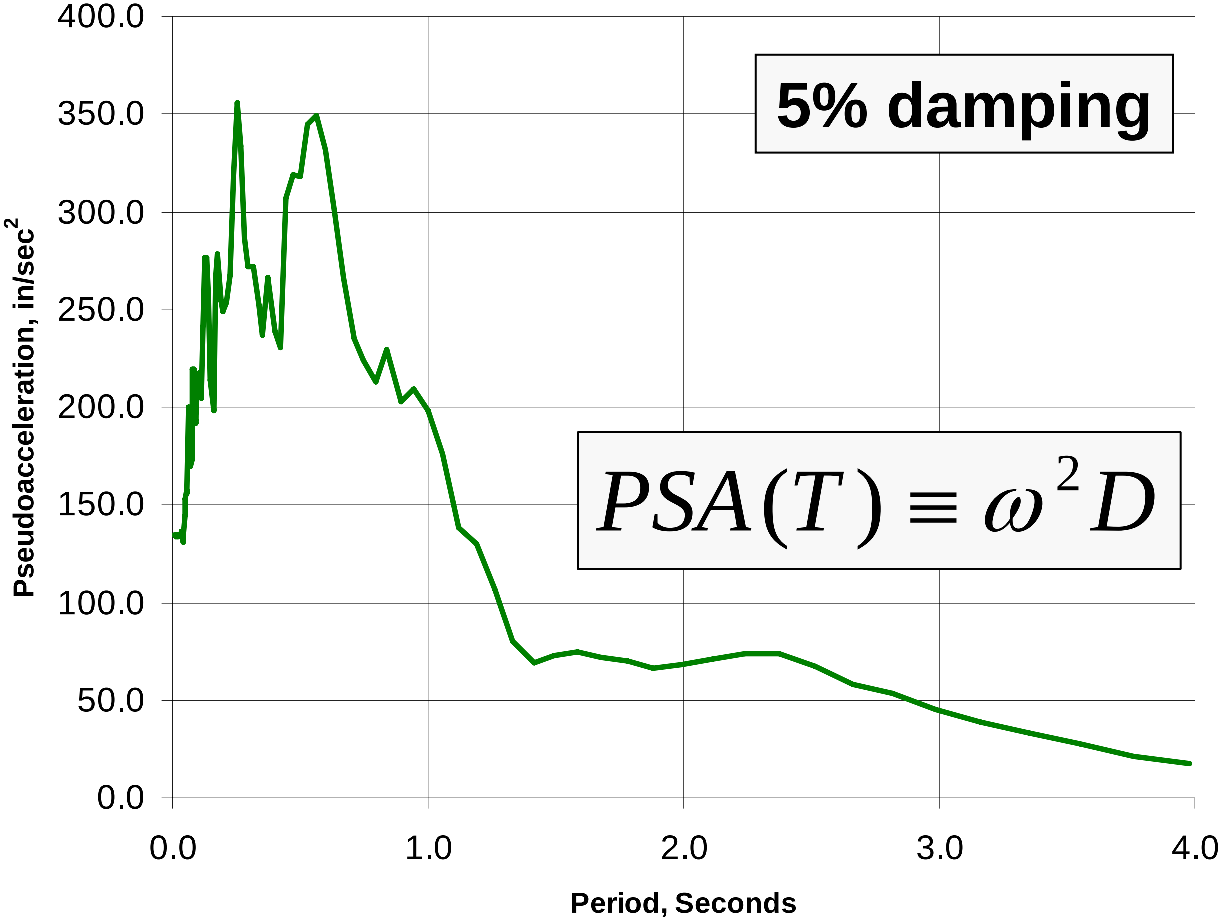
Complete 5% Damped Elastic Displacement Response Spectrum for El Centro Ground Motion



Development of *Pseudovelocity Response Spectrum*

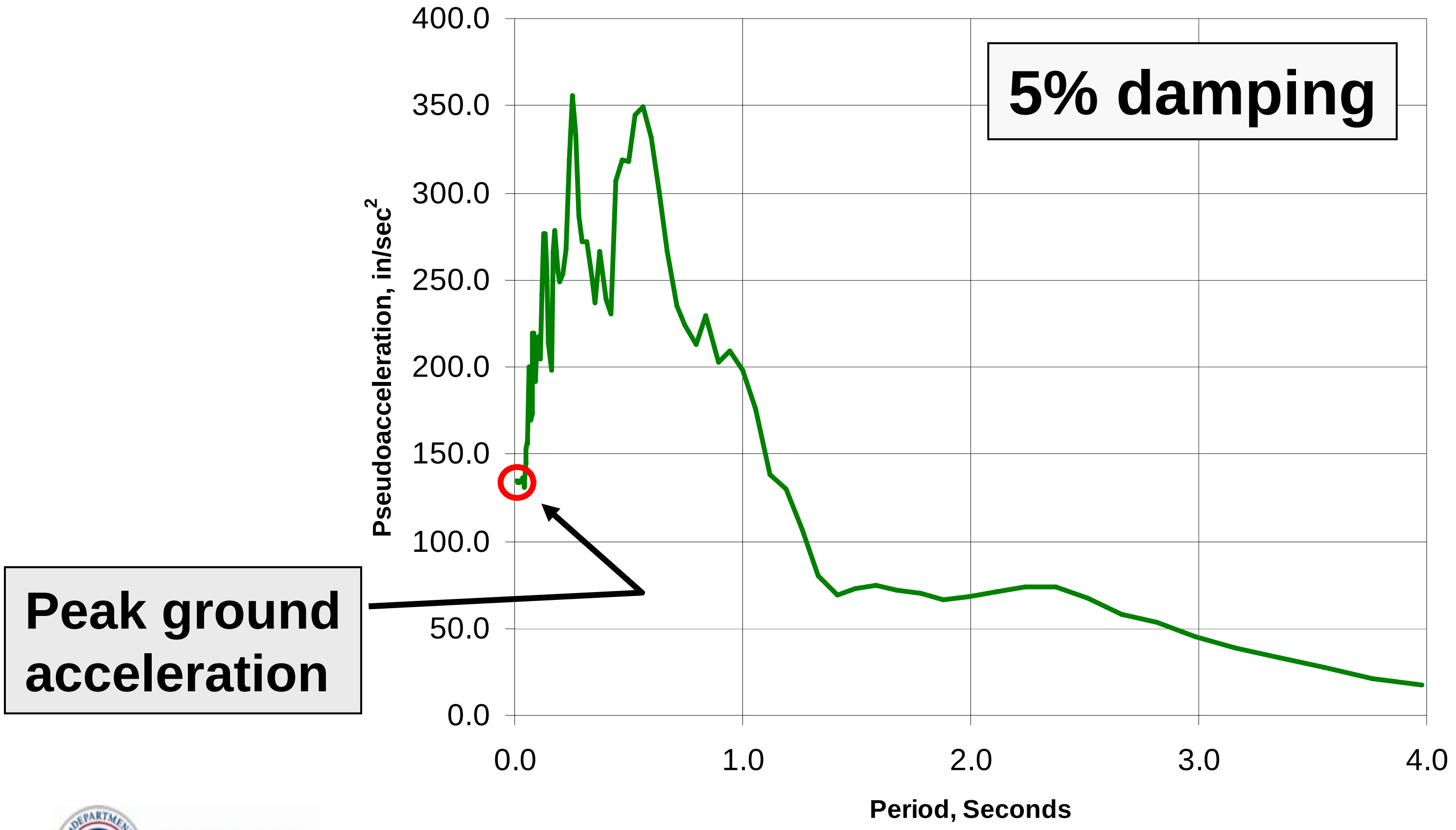


Development of *Pseudoacceleration Response Spectrum*

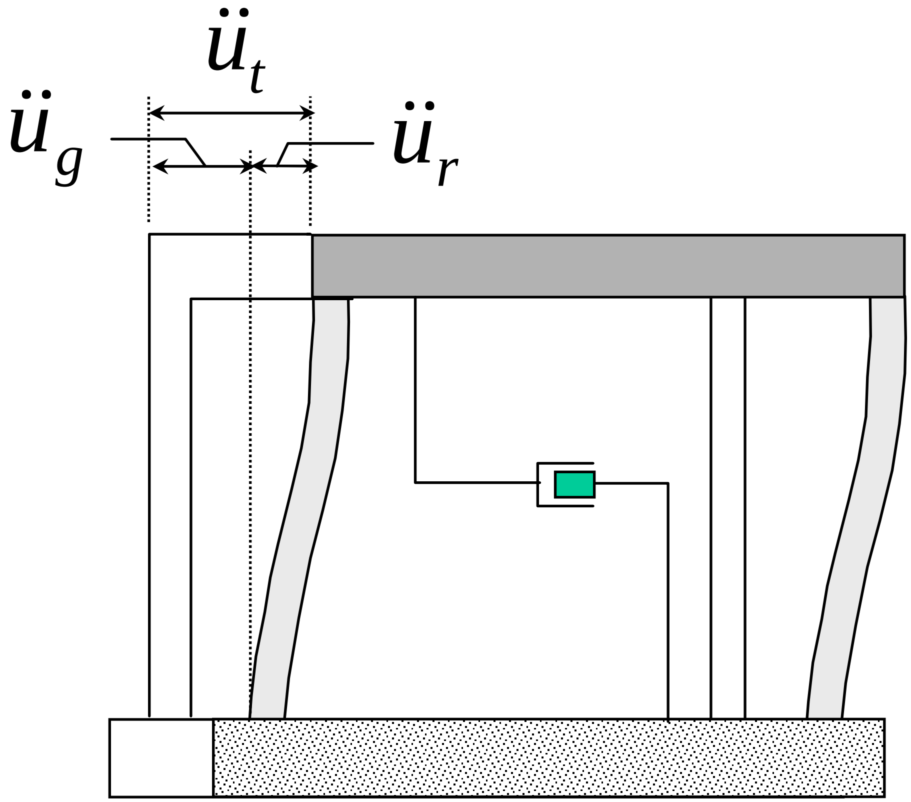


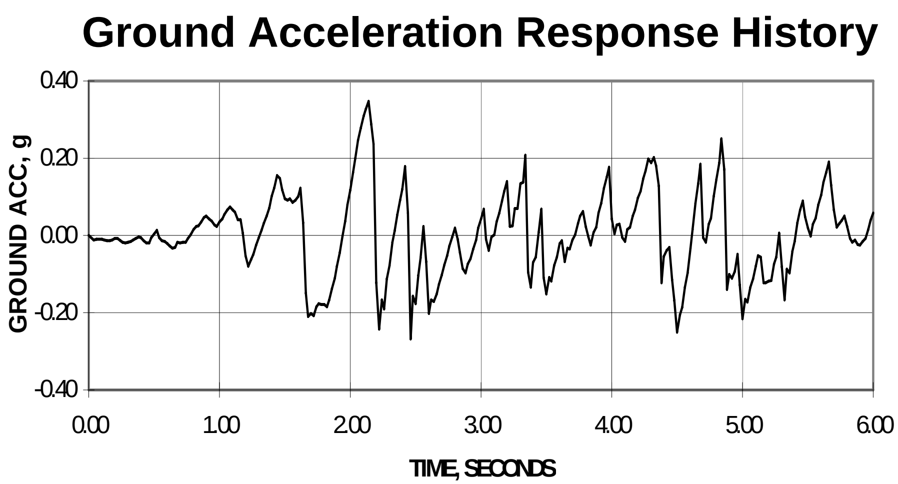
Note About the Pseudoacceleration Response Spectrum

The pseudoacceleration response spectrum represents the **total acceleration** of the system, not the relative acceleration. It is nearly identical to the true total acceleration response spectrum for lightly damped structures.



PSA is TOTAL Acceleration!



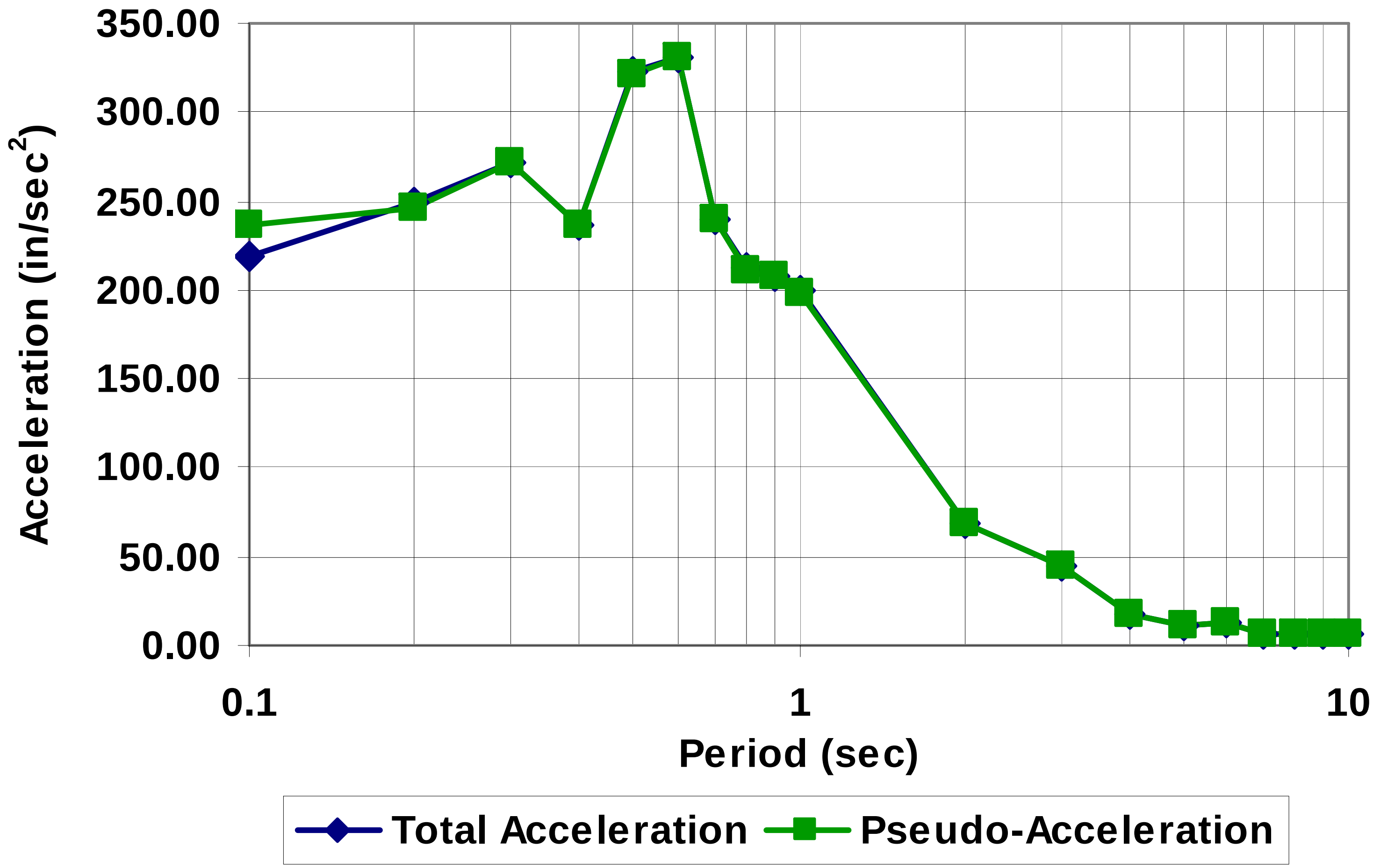


$$m[\boxed{\ddot{u}_g(t)+\ddot{u}_r(t)}]+c\,\dot{u}_r(t)+k\,u_r(t)=0$$

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

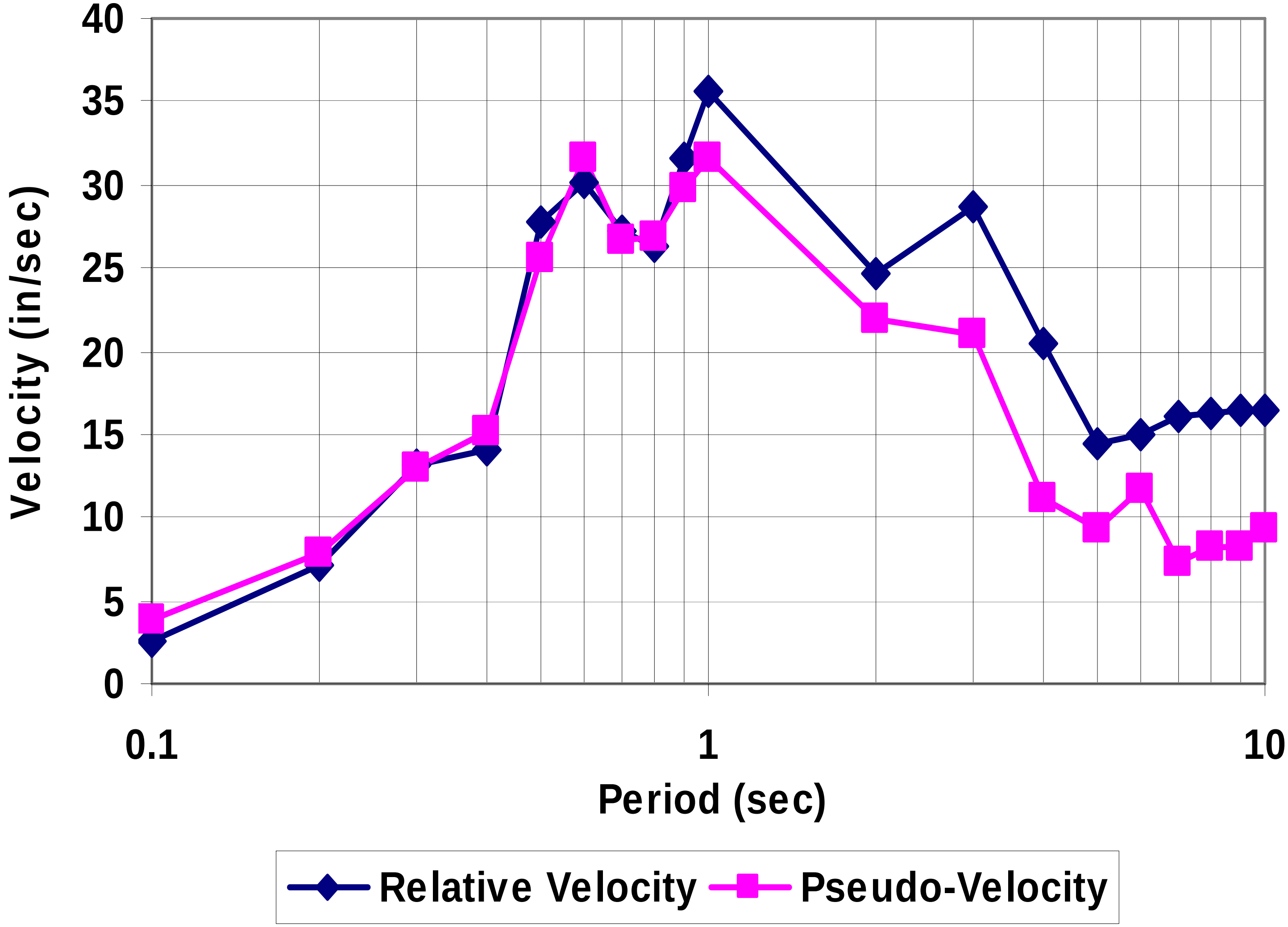
Difference Between Pseudo-Acceleration and Total Acceleration

(System with 5% Damping)

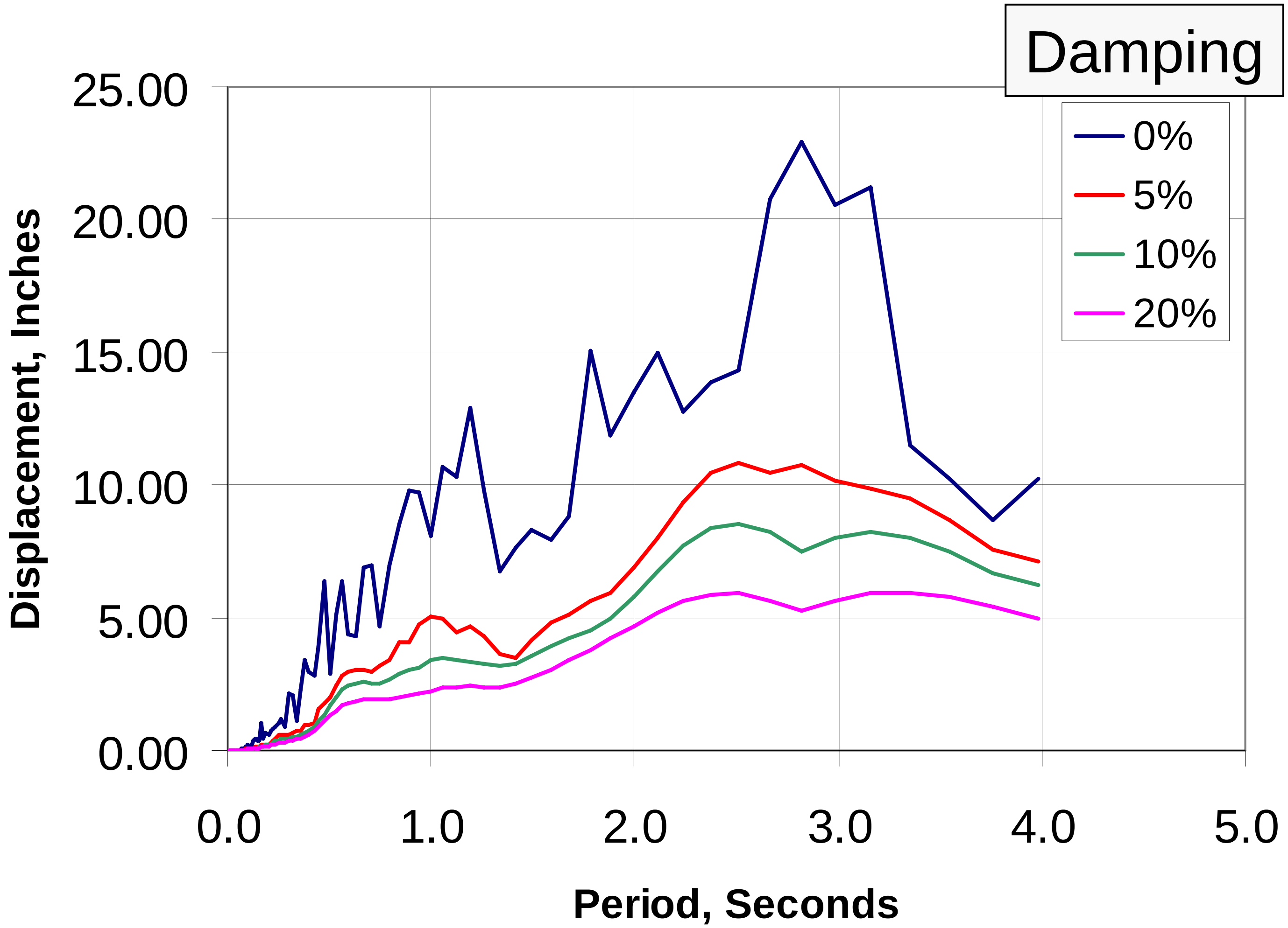


Difference Between Pseudovelocity and Relative Velocity

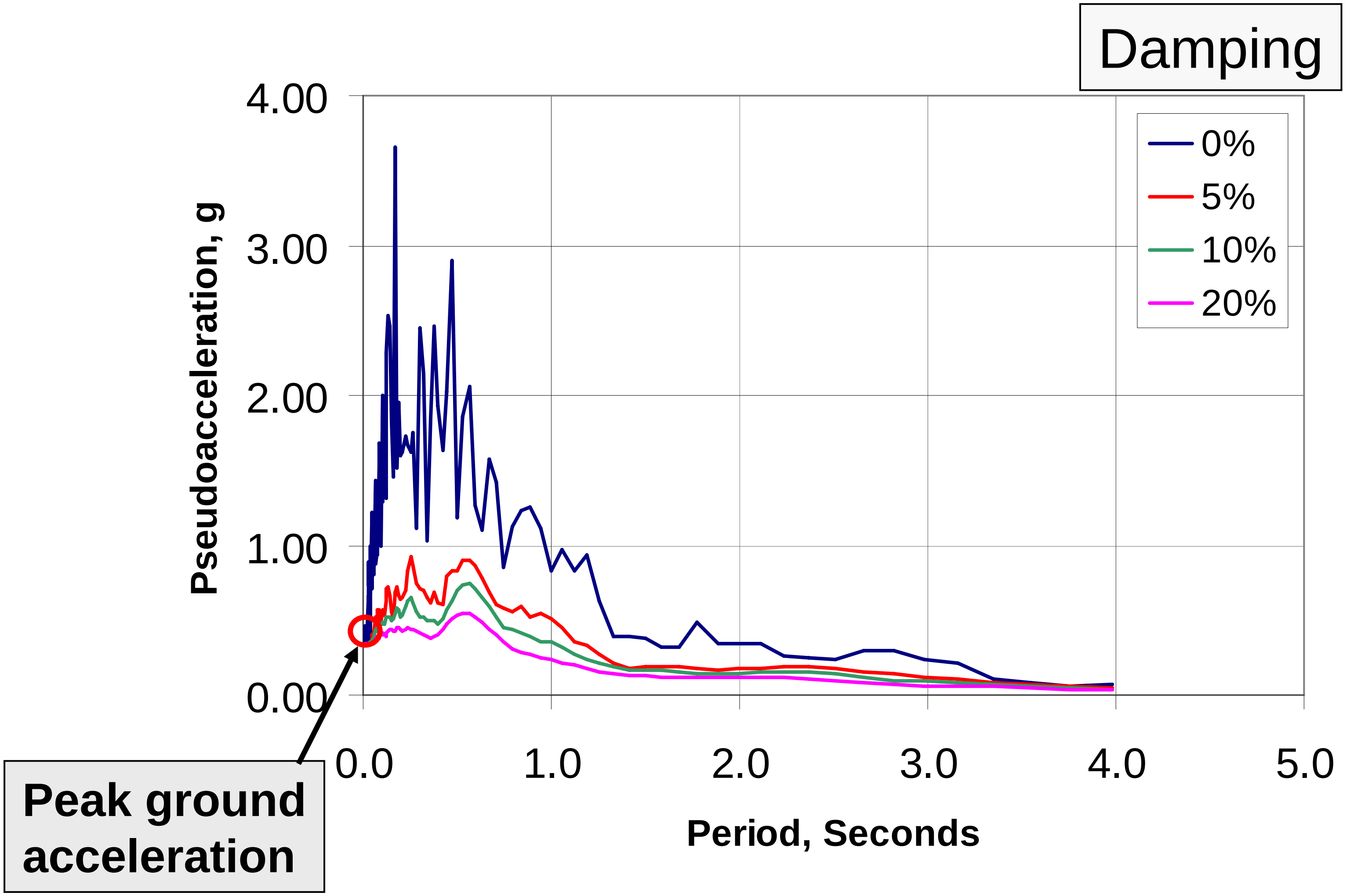
(System with 5% Damping)



Displacement Response Spectra for Different Damping Values



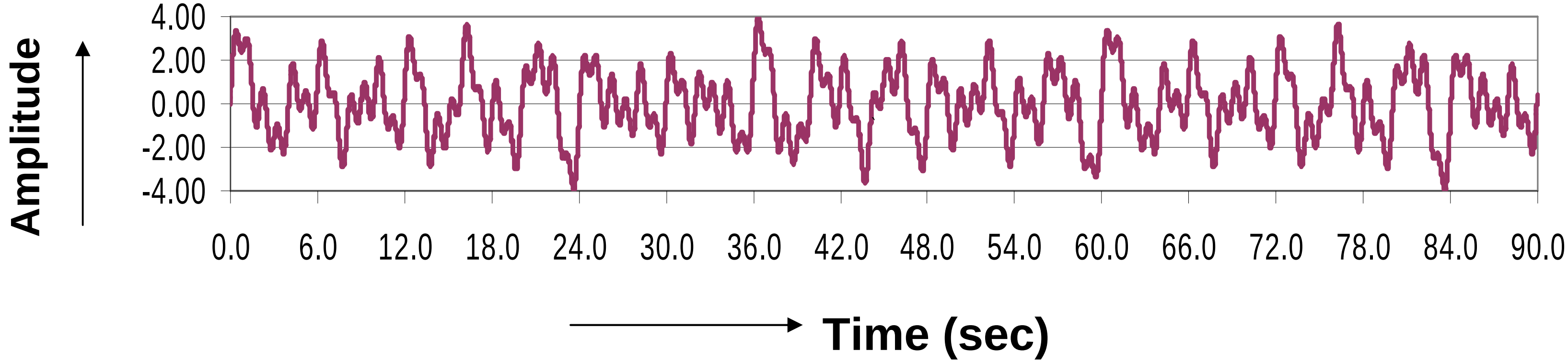
Pseudoacceleration Response Spectra for Different Damping Values



Damping Is Effective in Reducing the Response for (Almost) Any Given Period of Vibration

- An earthquake record can be considered to be the combination of a large number of harmonic components.
- Any SDOF structure will be in near resonance with one of these harmonic components.
- Damping is most effective at or near resonance.
- Hence, a response spectrum will show reductions due to damping at all period ranges (except $T = 0$).

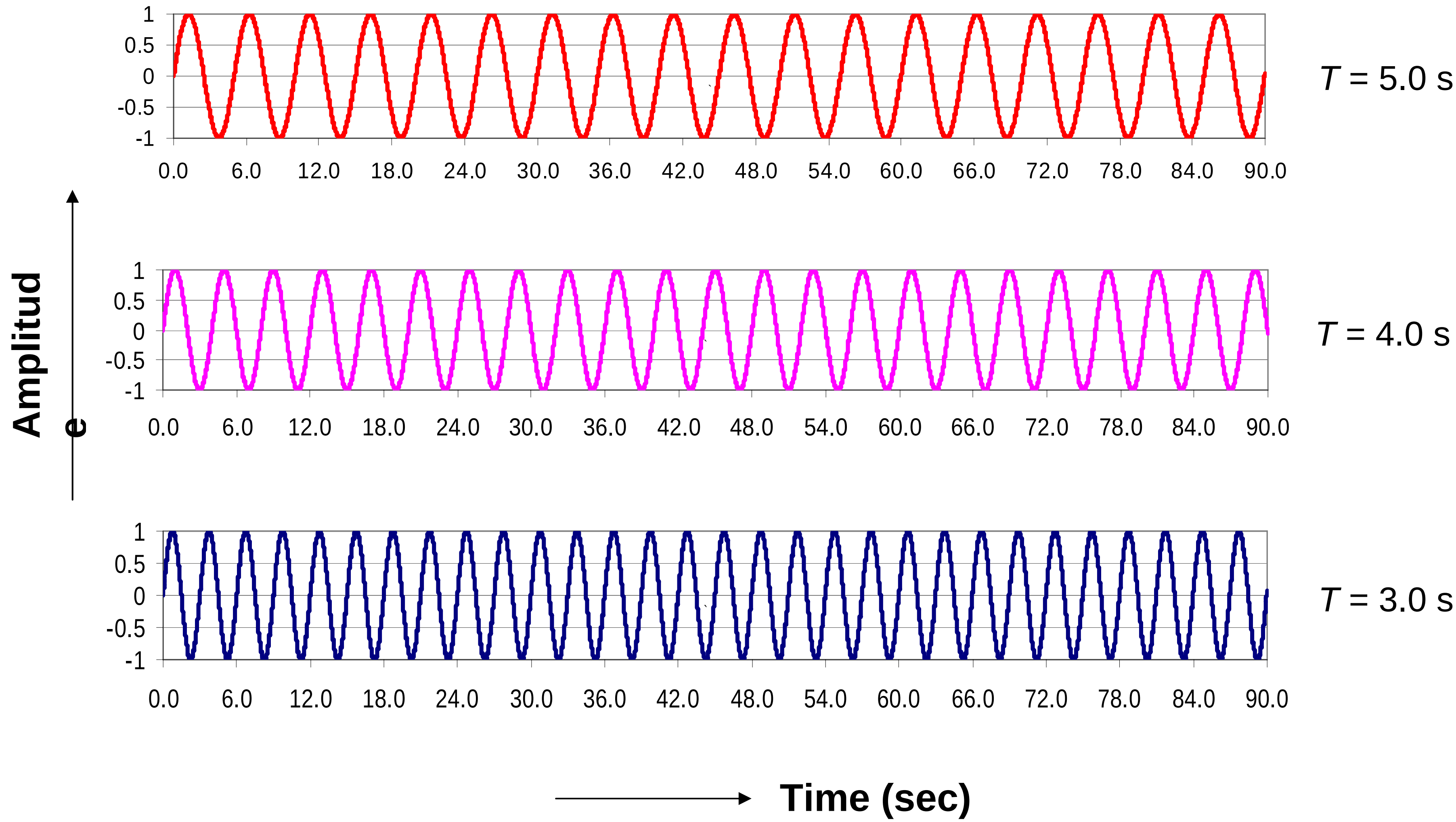
Damping Is Effective in Reducing the Response for Any Given Period of Vibration



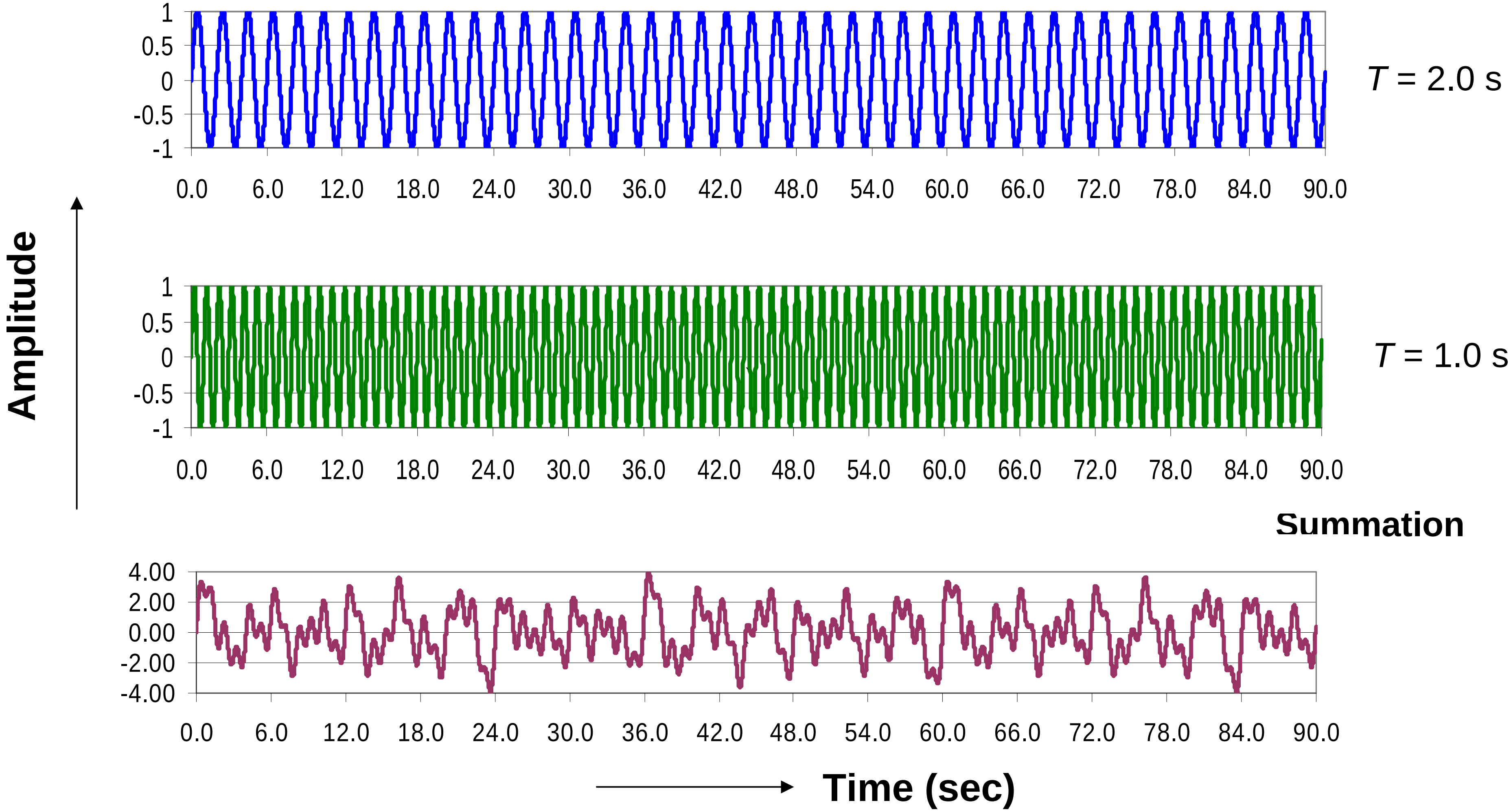
- Example of an artificially generated wave to resemble a real time ground motion accelerogram.

- Generated wave obtained by combining five different harmonic signals, each having equal amplitude of 1.0.

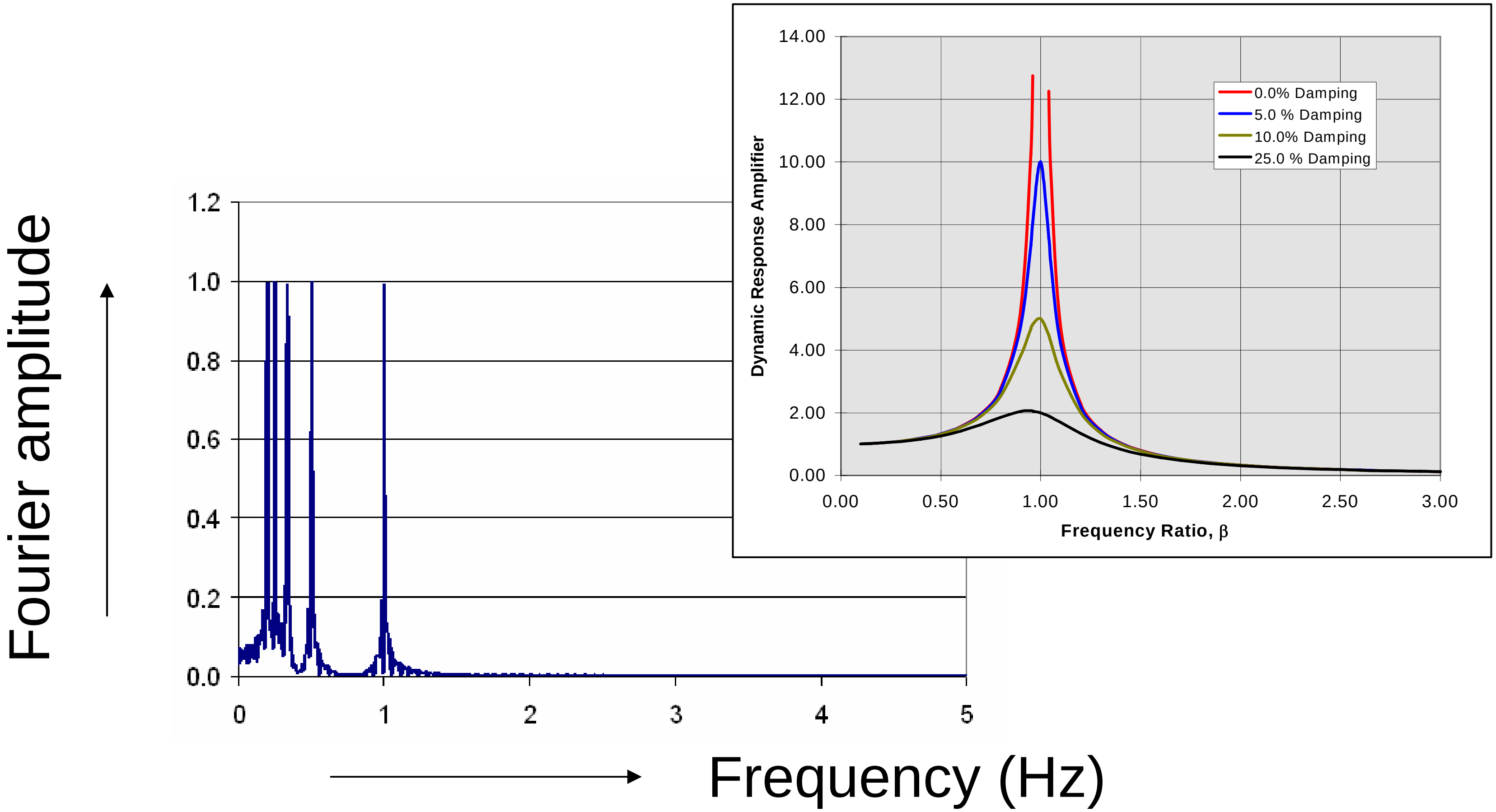
The Artificial Wave Is the Sum of Five Harmonics



The Artificial Wave Is the Sum of Five Harmonics



Damping Reduces the Response at Each Resonant Frequency



FFT curve for the combined wave

Use of an Elastic Response Spectrum

Example Structure

K = 500 k/in

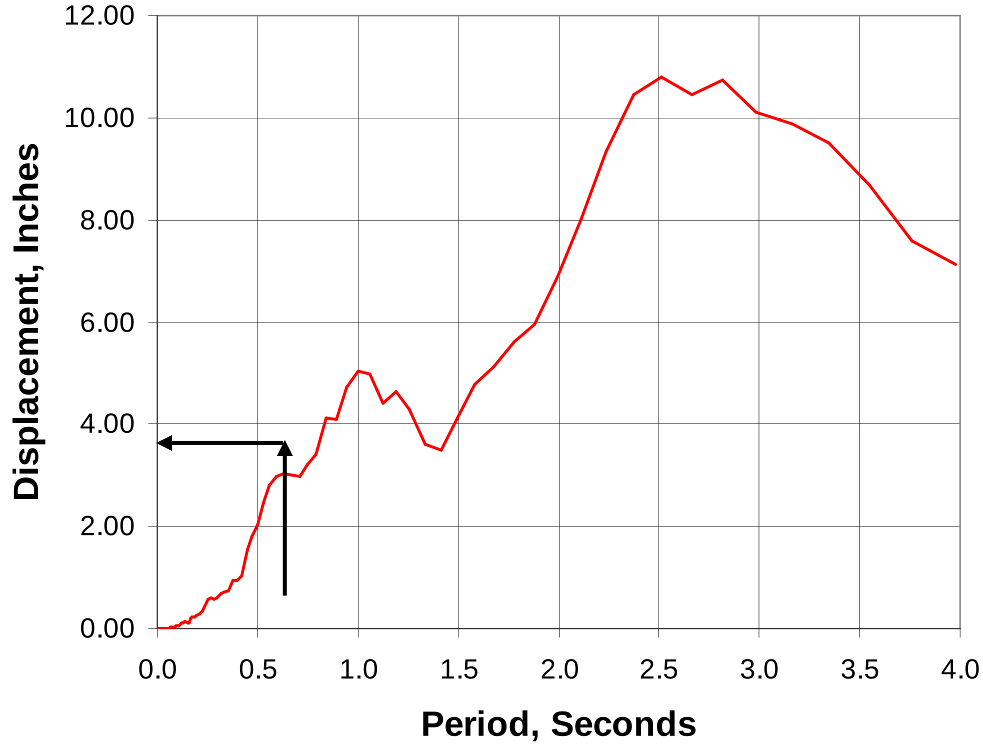
W = 2,000 k

M = 2000/386.4 = 5.18 k-sec^2/in

$\omega = (K/M)^{0.5}$ =9.82 rad/sec

$T = 2\pi/\omega$ = 0.64 sec

5% critical damping



At T = 0.64 sec, displacement = 3.03 in.

Use of an Elastic Response Spectrum

Example Structure

K = 500 k/in

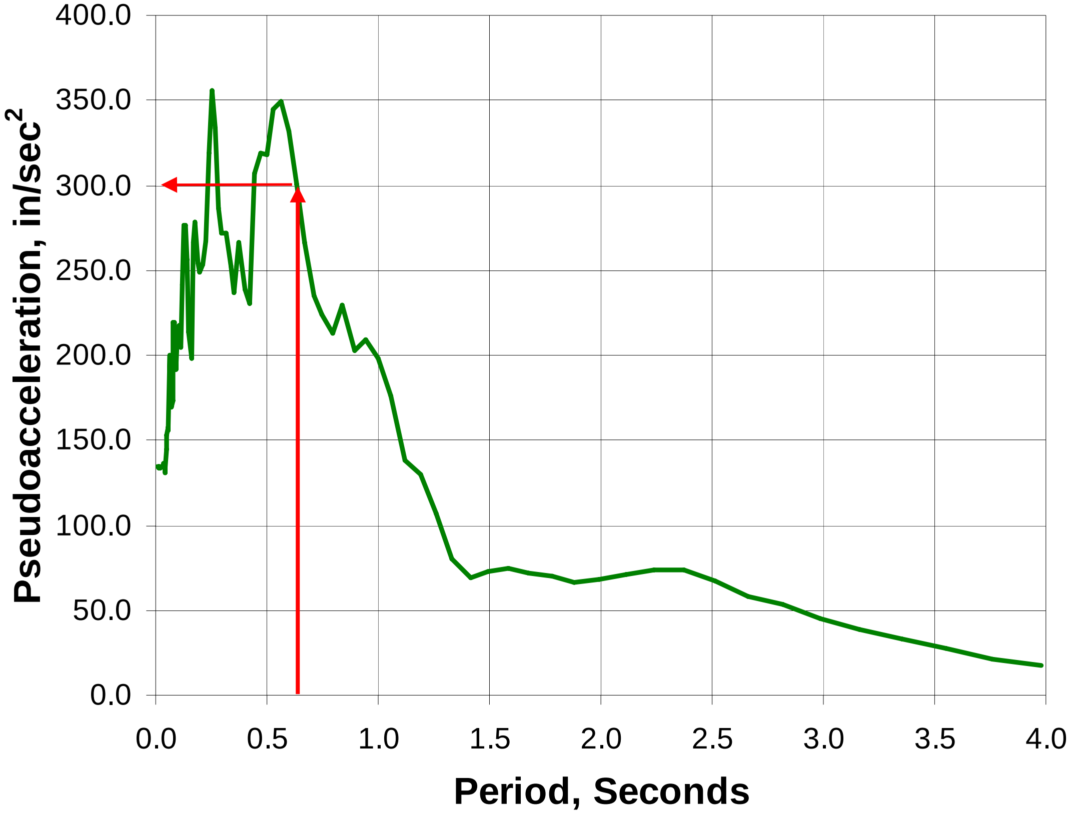
W = 2,000 k

M = 2000/386.4 = 5.18 k-sec^2/in

$\omega = (K/M)^{0.5}$ =9.82 rad/sec

$T = 2\pi/\omega$ = 0.64 sec

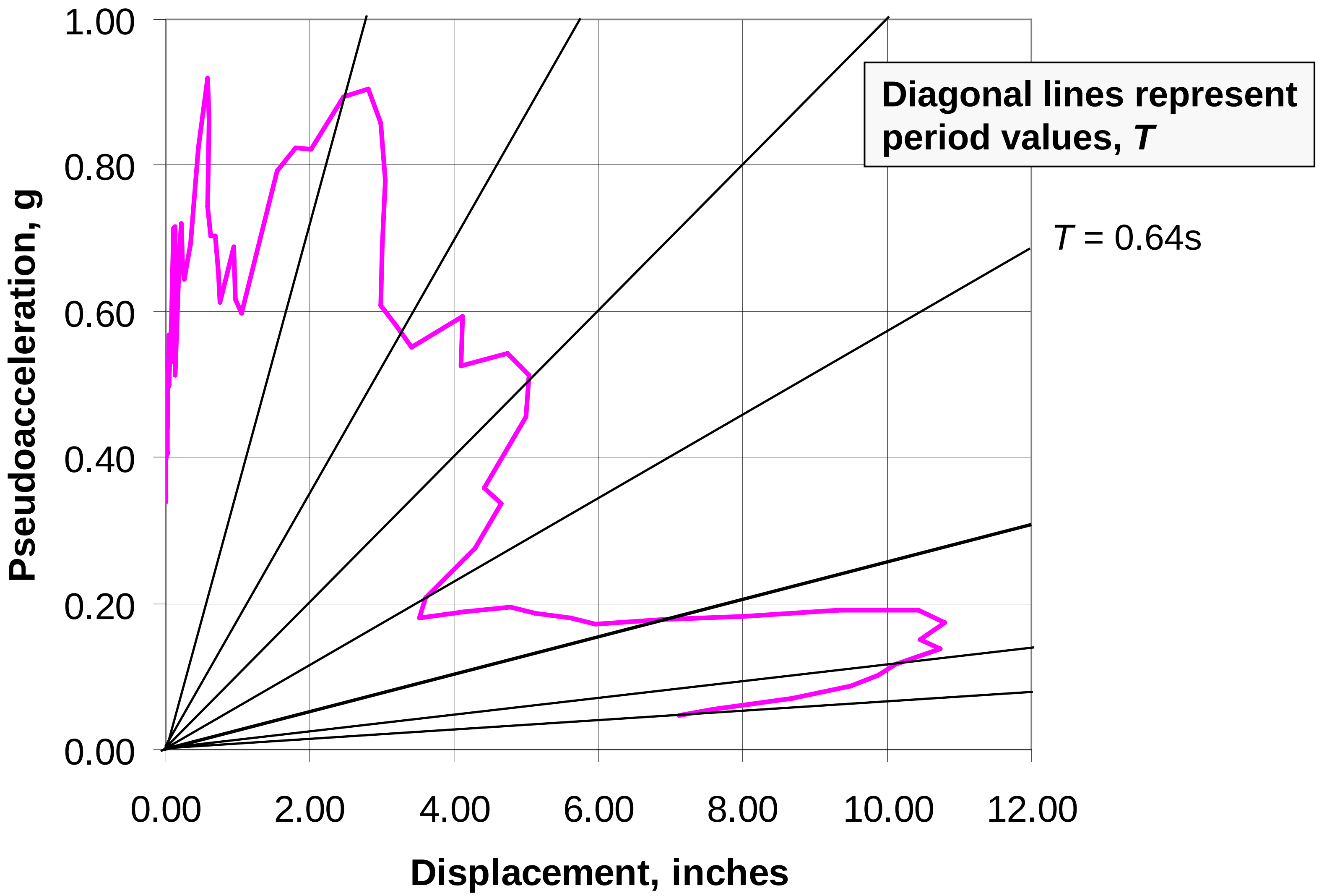
5% critical damping



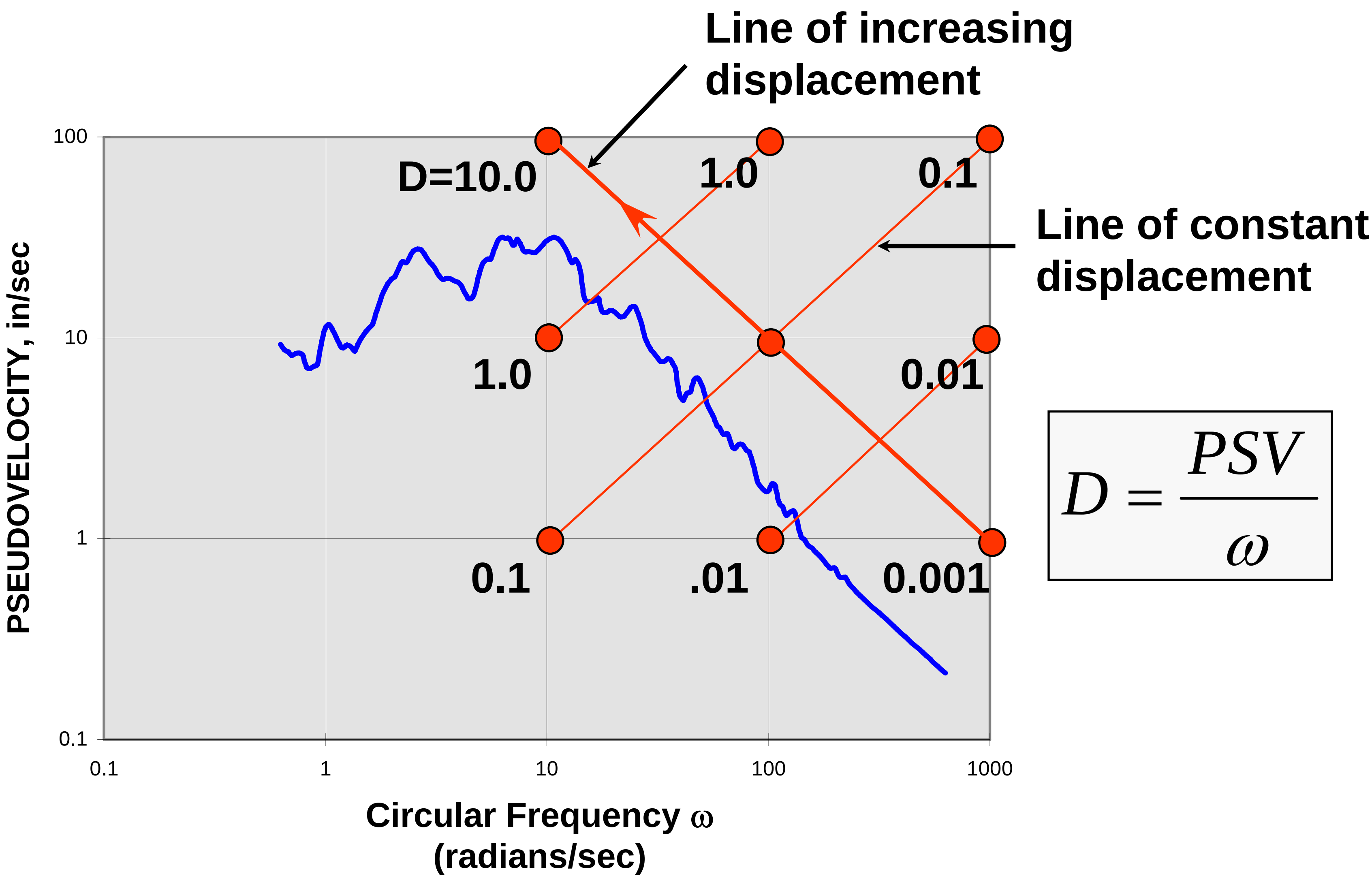
At T = 0.64 sec, pseudoacceleration = 301 in./sec^2

Base shear = M x PSA = 5.18(301) = 1559 kips

Response Spectrum, ADRS Space

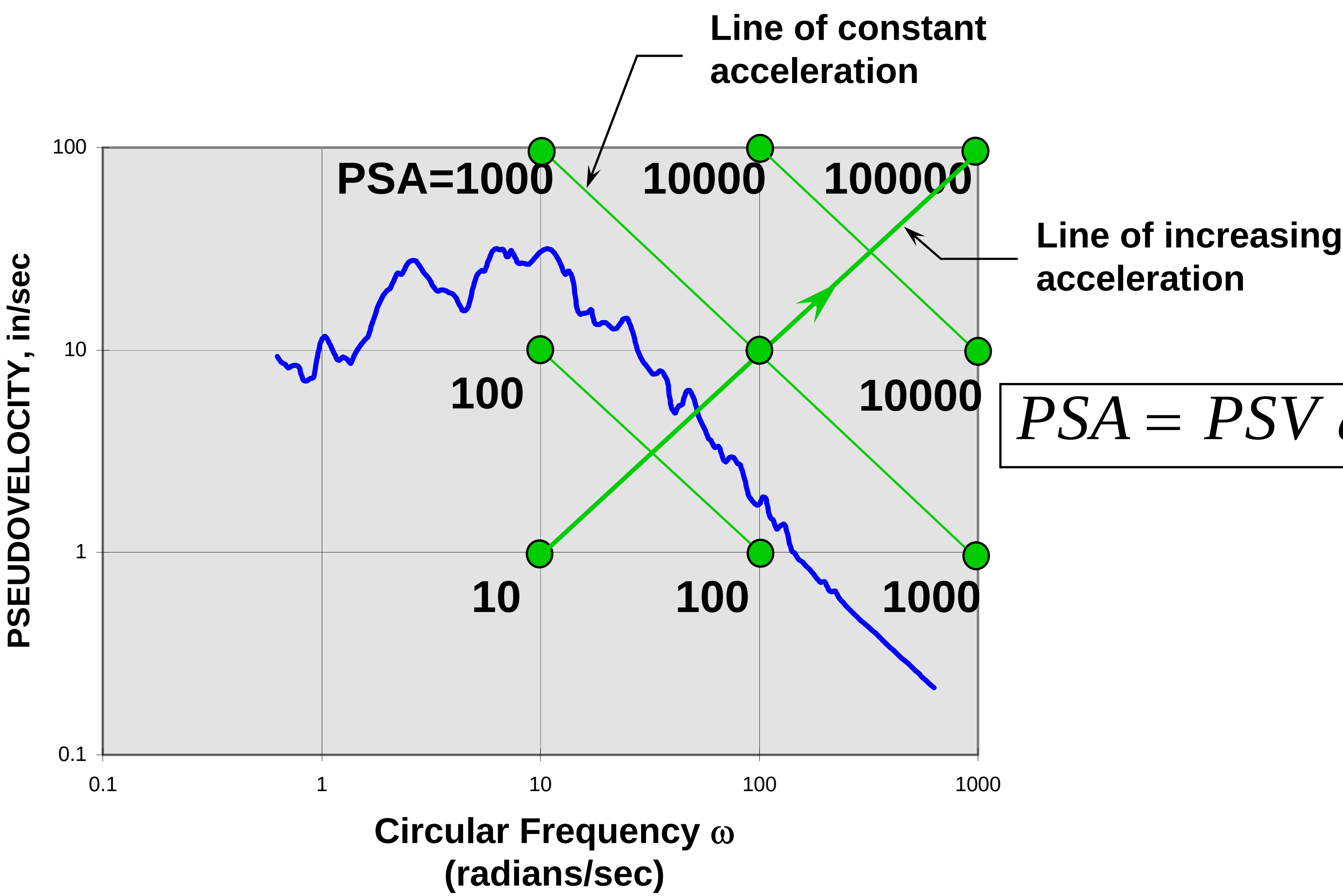


Four-Way Log Plot of Response Spectrum



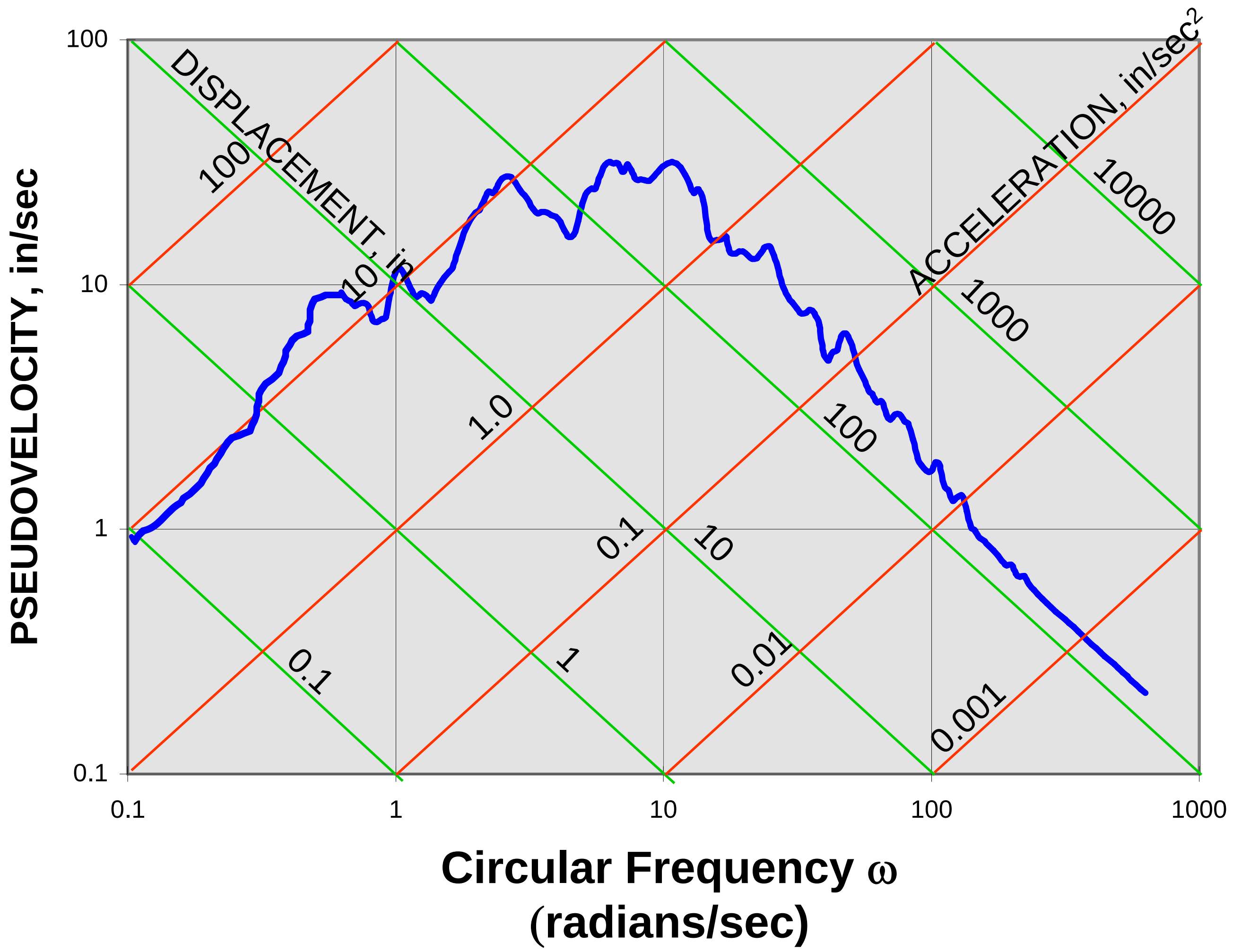
$$D = \frac{PSV}{\omega}$$

Four-Way Log Plot of Response Spectrum



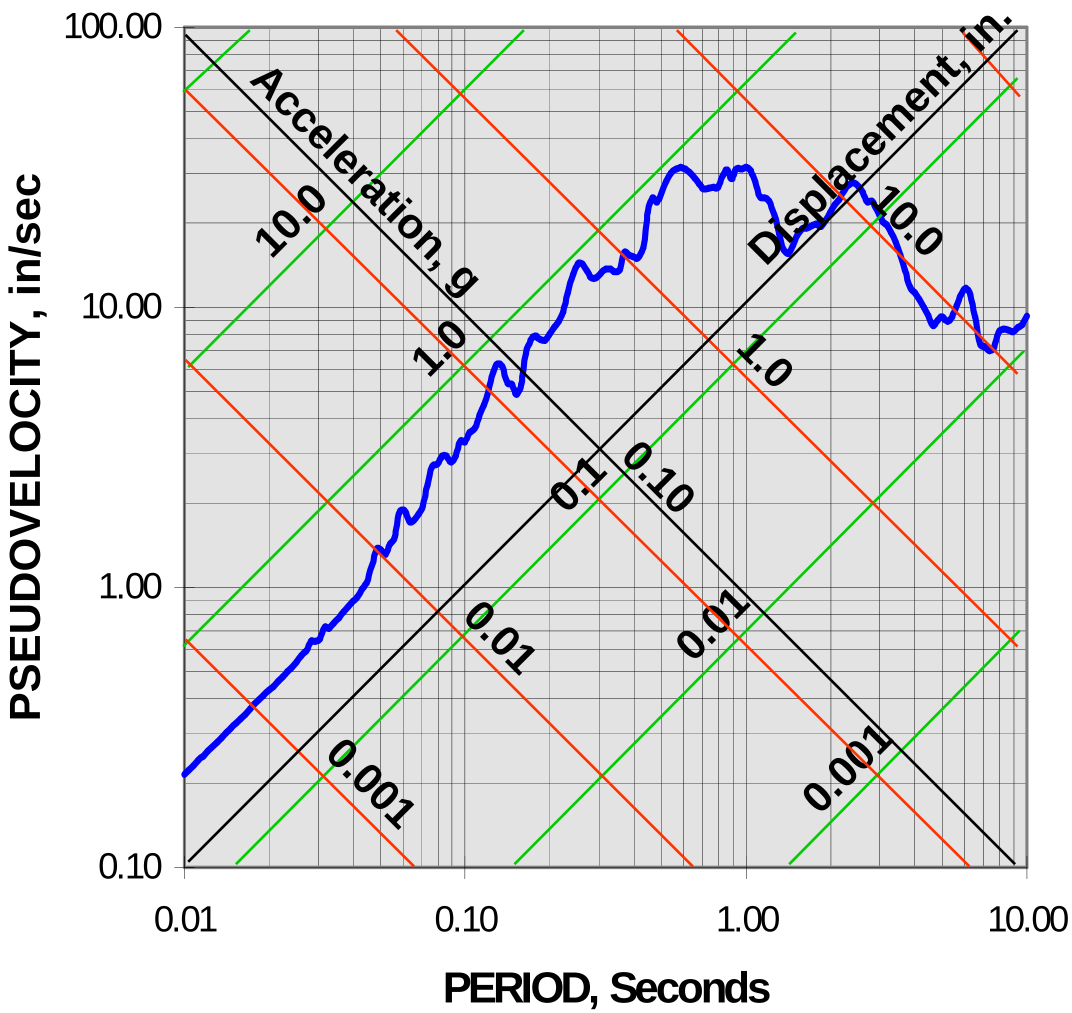
$$PSA = PSV\ \omega$$

Four-Way Log Plot of Response Spectrum

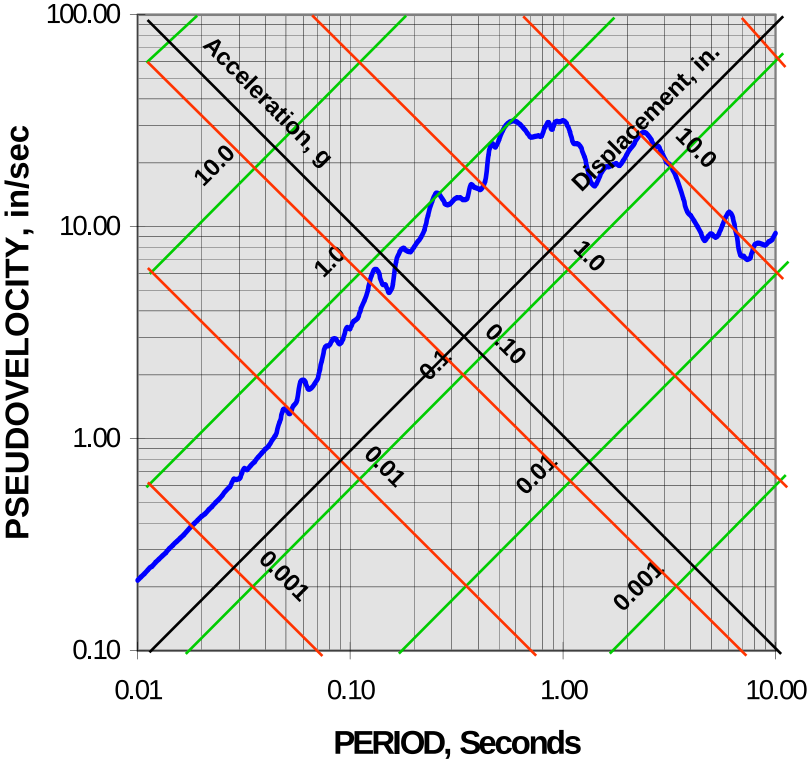


Four-Way Log Plot of Response Spectrum

Plotted vs Period



Development of an Elastic Response Spectrum

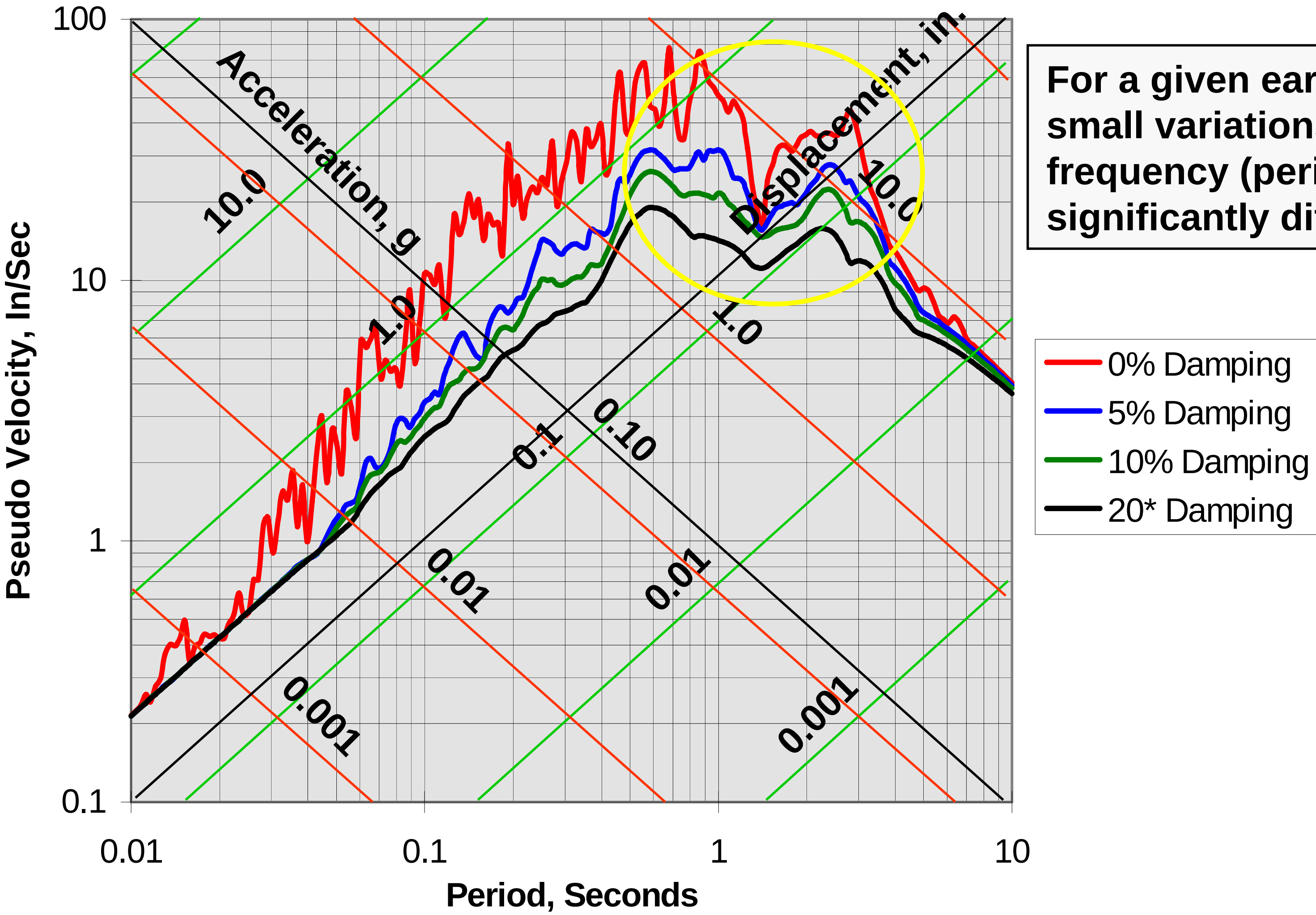


<u>Problems with Current Spectrum:</u>

For a given earthquake, small variations in structural frequency (period) can produce significantly different results.

It is for a single earthquake; other earthquakes will have different Characteristics.

1940 El Centro, 0.35 g, N-S

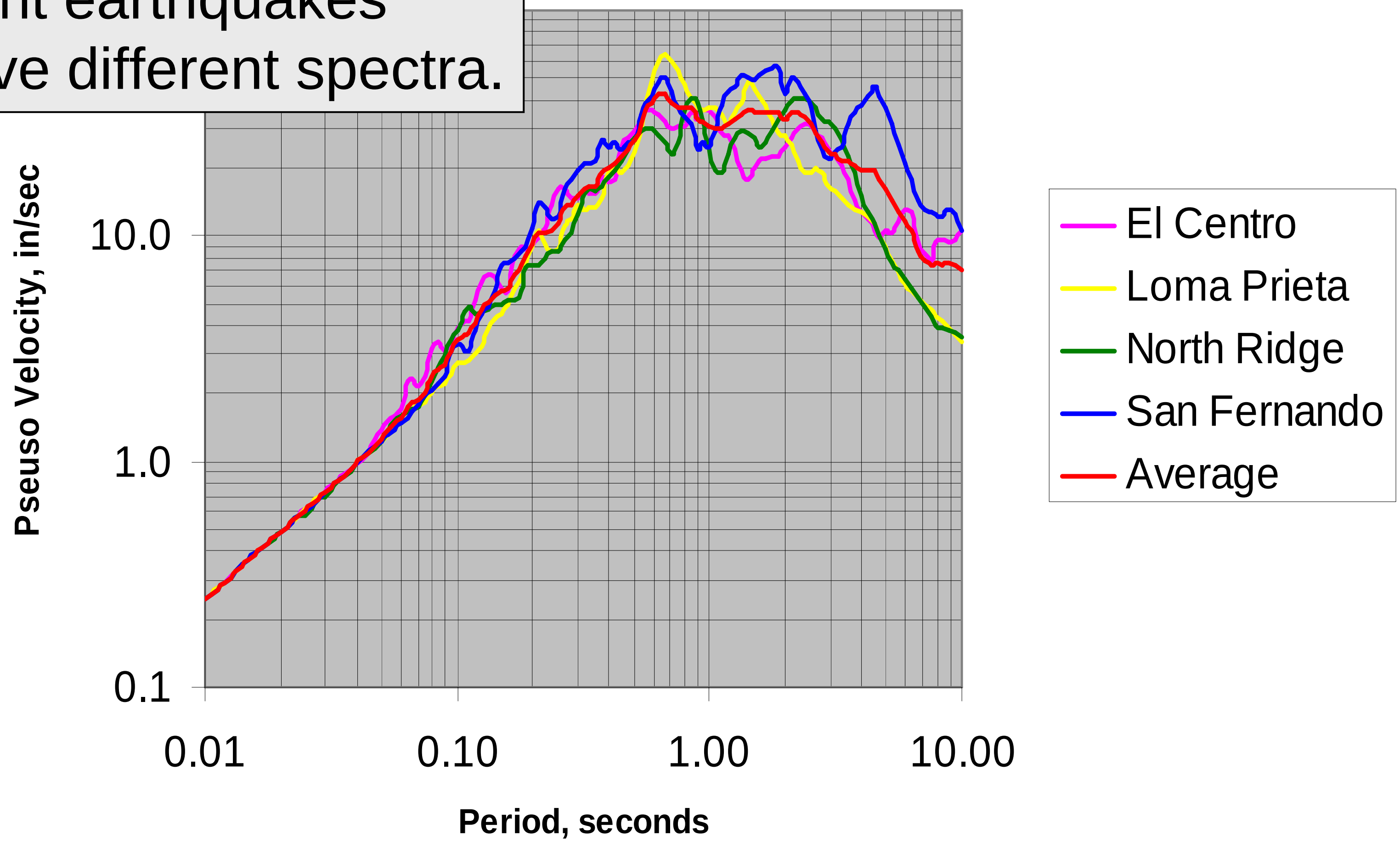


For a given earthquake, small variations in structural frequency (period) can produce significantly different results.

5% Damped Spectra for Four California Earthquakes

Scaled to 0.40 g (PGA)

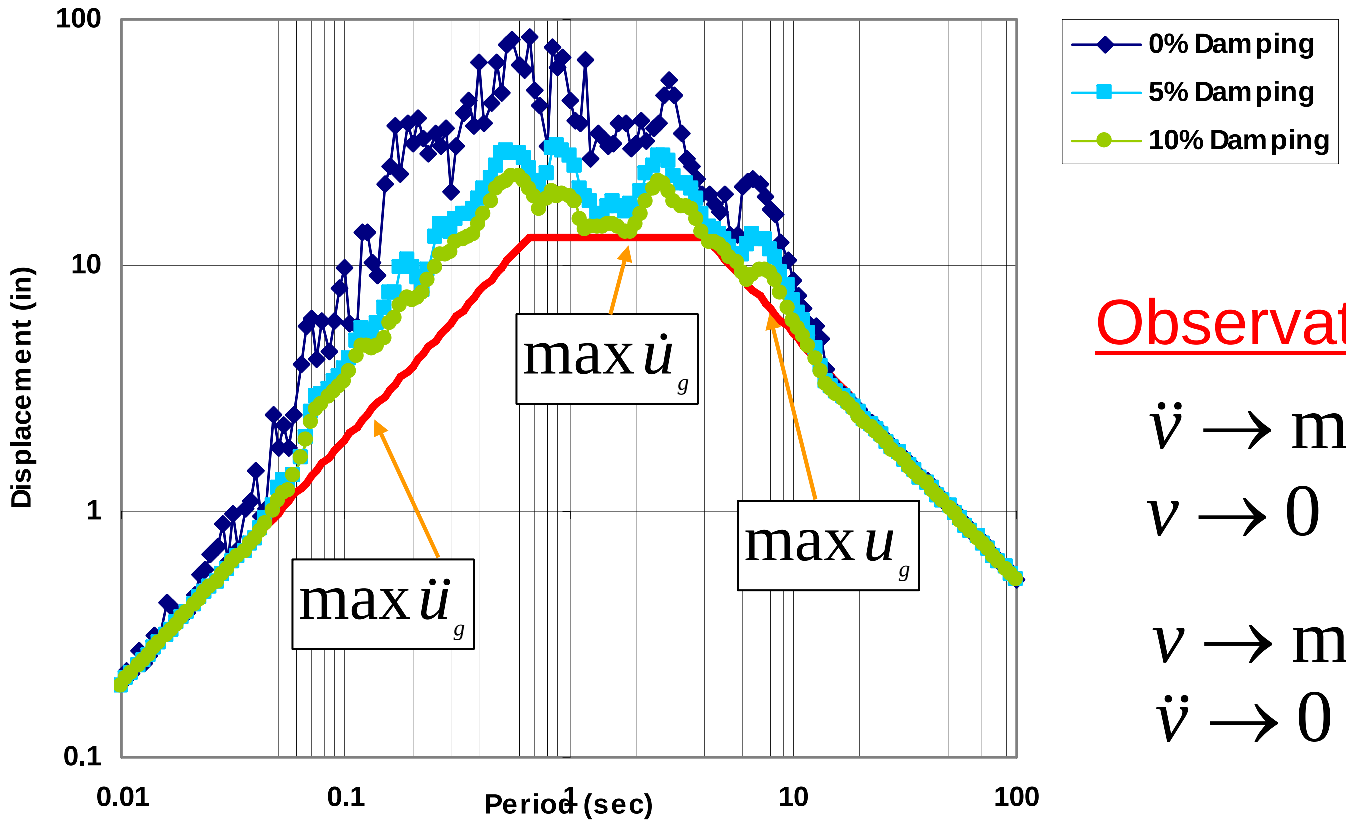
Different earthquakes will have different spectra.



Smoothed Elastic Response Spectra (Elastic DESIGN Response Spectra)

- Newmark-Hall spectrum
- ASCE 7 spectrum

Newmark-Hall Elastic Spectrum



Observations

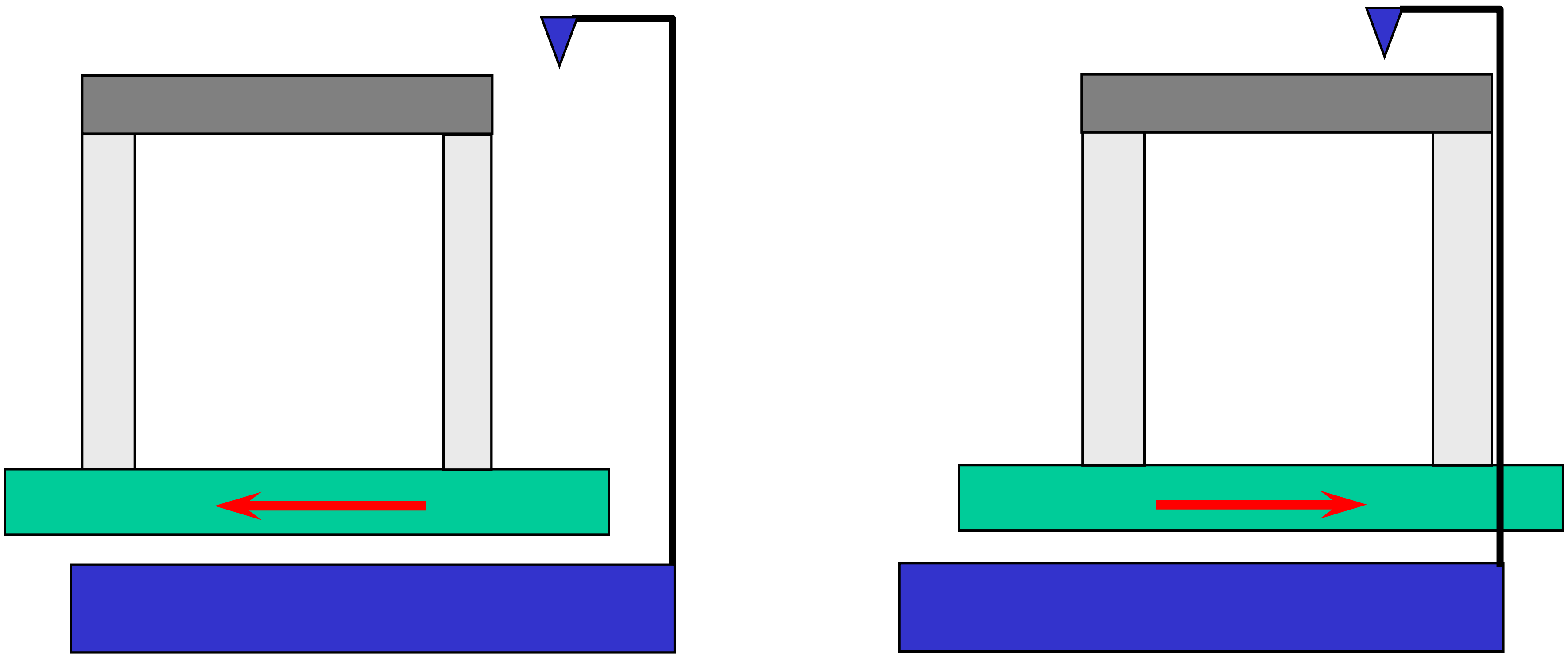
$$\ddot{v} \rightarrow \max \ddot{v}_g$$
$$v \rightarrow 0$$

at short *T*

$$v \rightarrow \max v_g$$
$$\ddot{v} \rightarrow 0$$

at long *T*

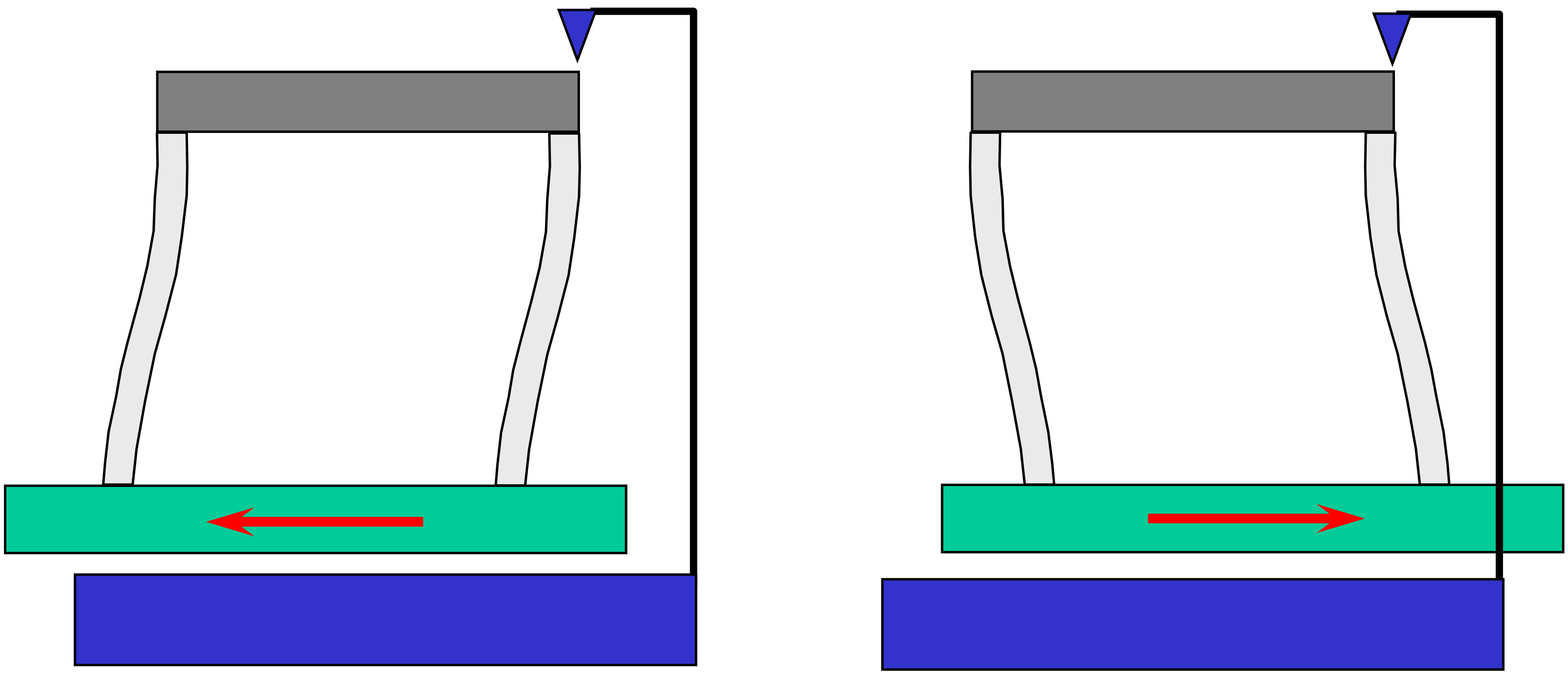
Very Stiff Structure (T < 0.01 sec)



Relative displacement ⟹ Zero

Total acceleration ⟹ Ground acceleration

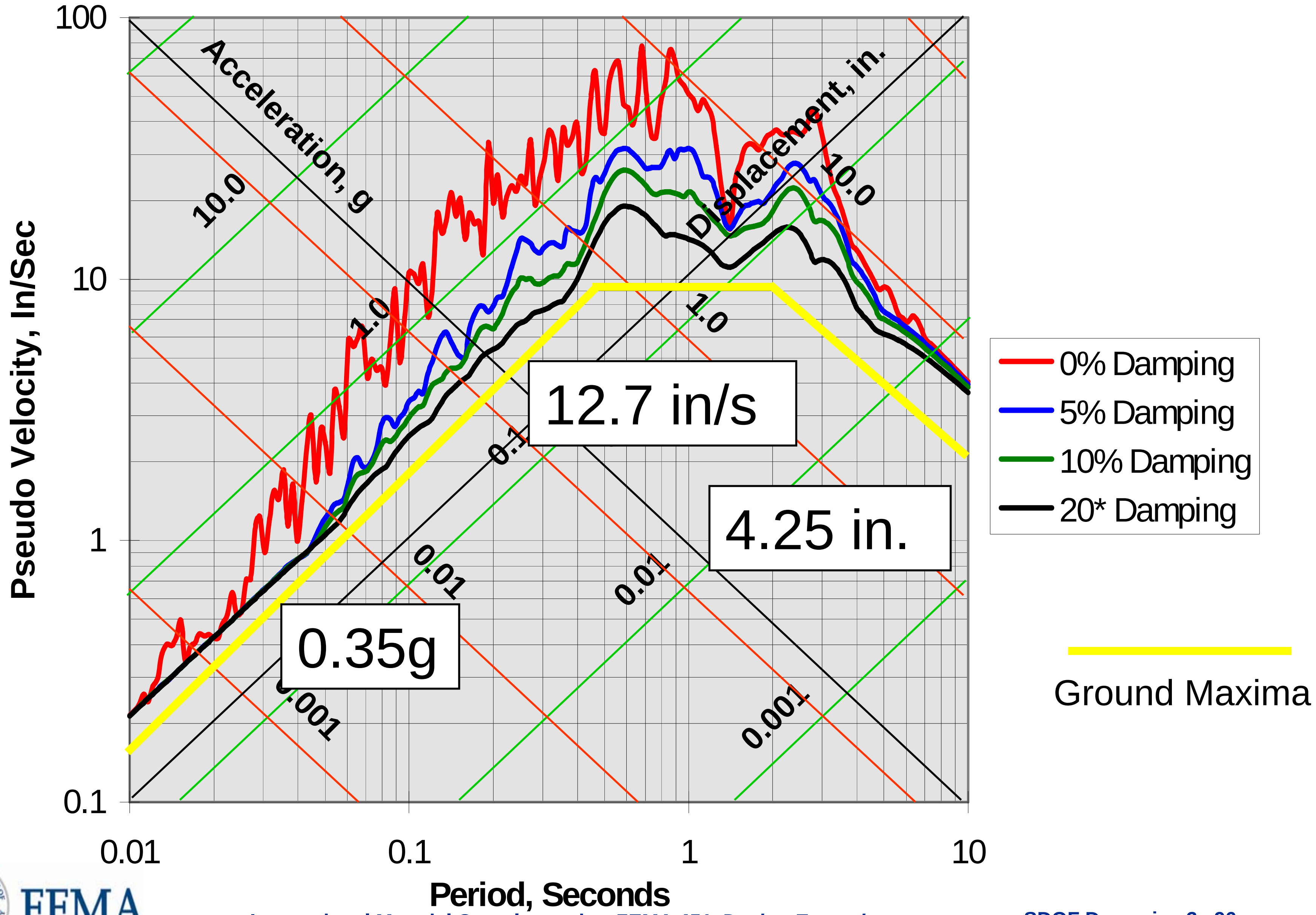
Very Flexible Structure (T > 10 sec)



Relative displacement ⟹ Ground displacement

Total acceleration ⟹ Zero

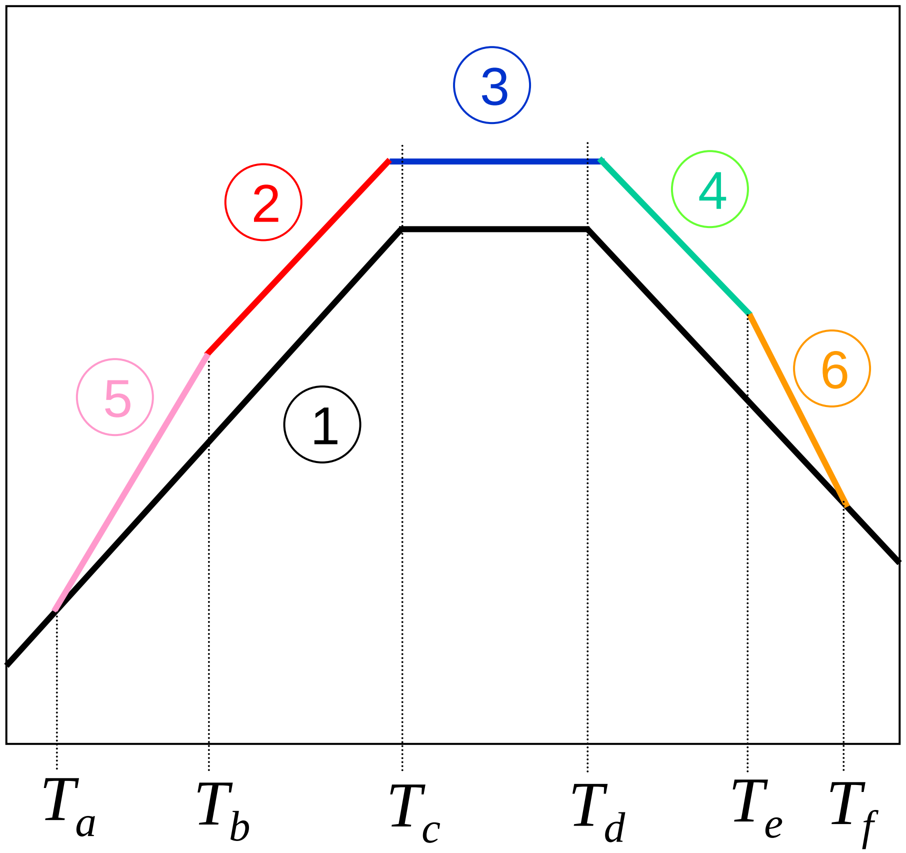
1940 El Centro, 0.35 g, N-S



Newmark's Spectrum Amplification Factors for Horizontal Elastic Response

Damping % Critical	One Sigma (84.1%) a_a	a_v	a_d	Median (50%) a_a	a_v	a_d
.05	5.10	3.84	3.04	3.68	2.59	2.01
1	4.38	3.38	2.73	3.21	2.31	1.82
2	3.66	2.92	2.42	2.74	2.03	1.63
3	3.24	2.64	2.24	2.46	1.86	1.52
5	**2.71**	**2.30**	**2.01**	2.12	1.65	1.39
7	2.36	2.08	1.85	1.89	1.51	1.29
10	1.99	1.84	1.69	1.64	1.37	1.20
20	1.26	1.37	1.38	1.17	1.08	1.01

Newmark-Hall Elastic Spectrum



1) Draw the lines corresponding to max $\ddot{v}_g, \dot{v}_g, v_g$

2) Draw line $\alpha_A \max \ddot{v}_g$ from T_b to T_c

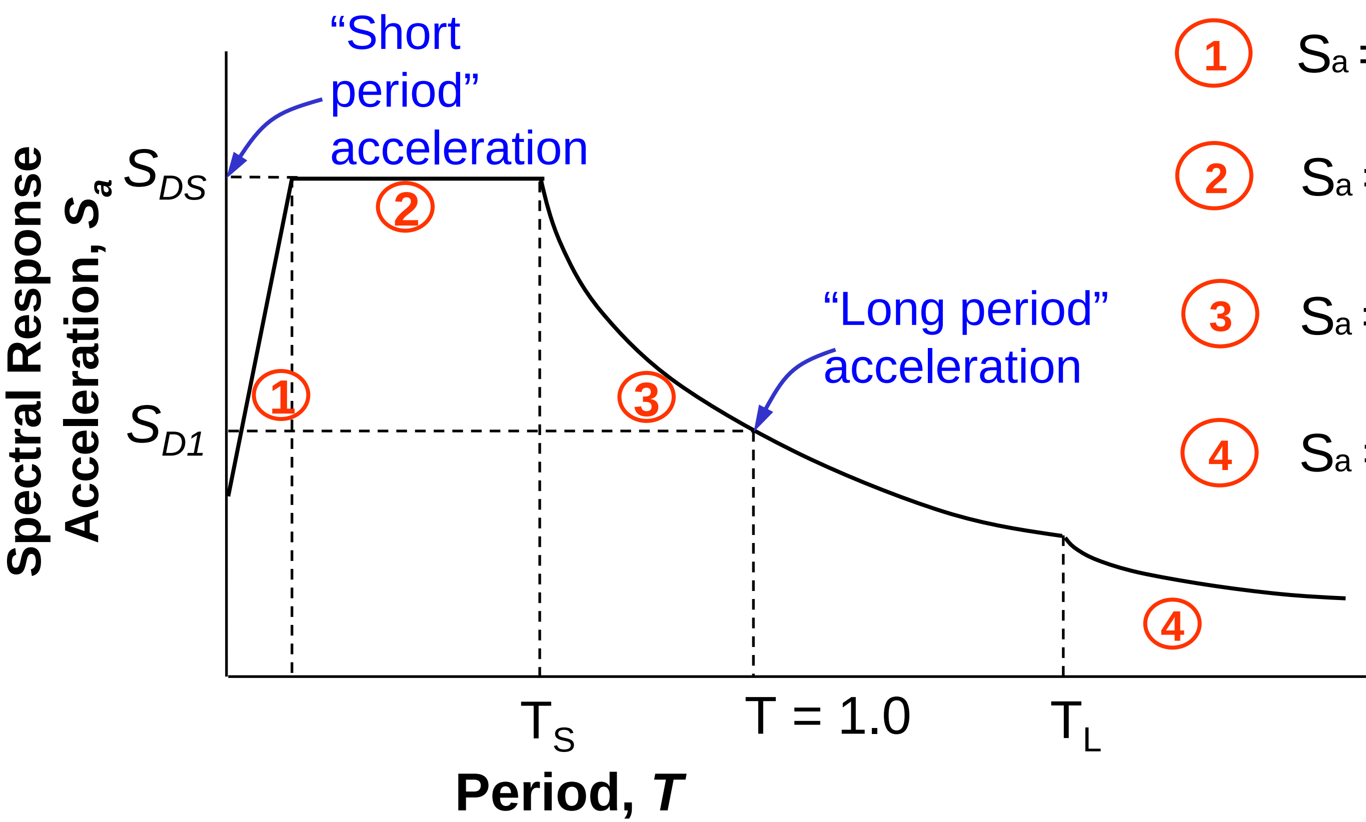
3) Draw line $\alpha_V \max \dot{v}_g$ from T_c to T_d

4) Draw line $\alpha_D \max v_g$ from T_d to T_e

5) Draw connecting line from T_a to T_b

6) Draw connecting line from T_e to T_f

ASCE 7
Uses a Smoothed Design Acceleration Spectrum



1. $$S_a = 0.6\frac{S_{DS}}{T_0}T + 0.4\,S_{DS}$$

2. $$S_a = S_{DS}$$

3. $$S_a = \frac{S_{D1}}{T}$$

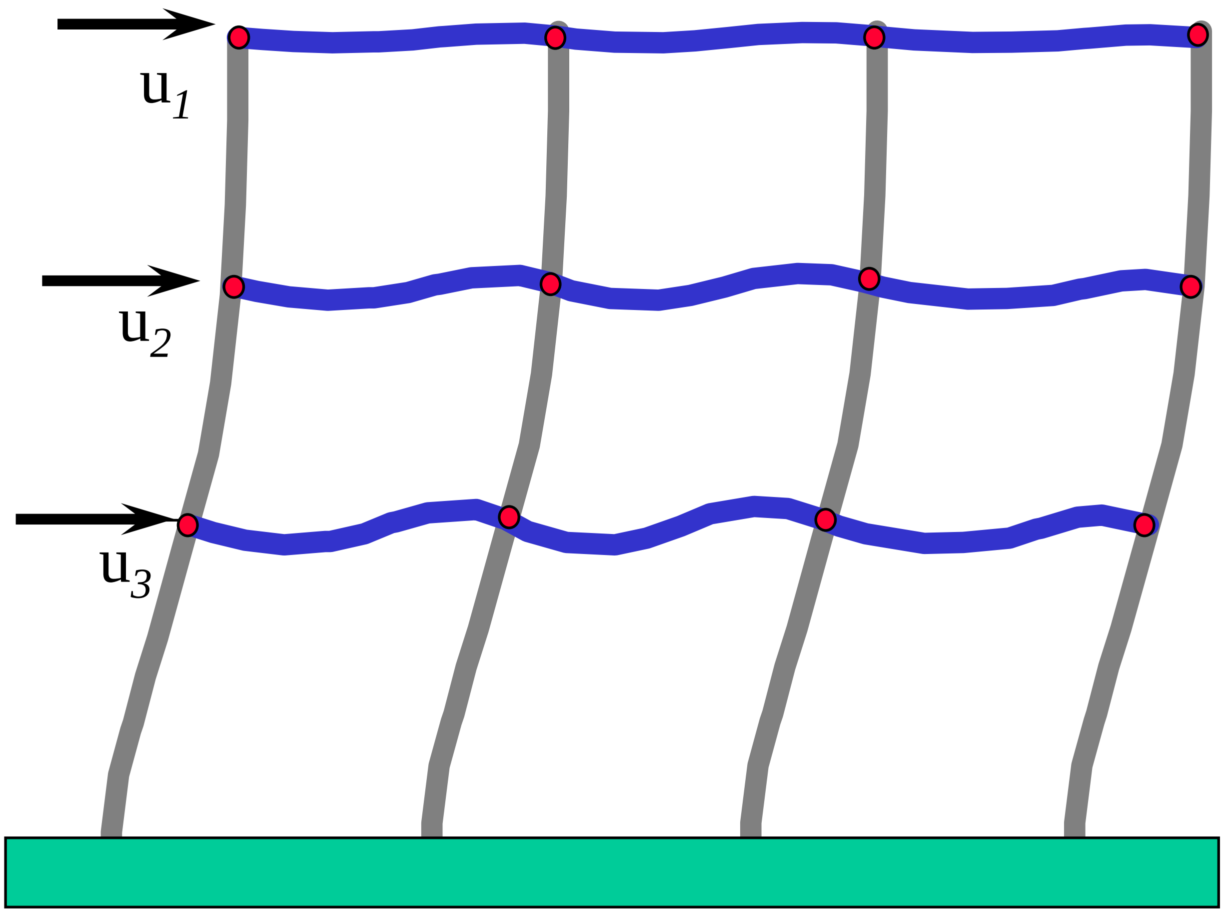
4. $$S_a = \frac{T_L S_{D1}}{T^2}$$

Note exceptions at larger periods

The ASCE 7 Response Spectrum

is a uniform hazard spectrum based on probabilistic and deterministic seismic hazard analysis.

Structural Dynamics of Linear Elastic Multiple-Degrees-of-Freedom (MDOF) Systems



Structural Dynamics of Elastic MDOF Systems

- Equations of motion for MDOF systems
- Uncoupling of equations through use of natural mode shapes
- Solution of uncoupled equations
- Recombination of computed response
- Modal response history analysis
- Modal response spectrum analysis
- Equivalent lateral force procedure

Symbol Styles Used in this Topic

$$\mathbf{M} \quad \mathbf{U}$$

Matrix or vector (column matrix)

$$\mathrm{m} \quad \mathrm{u}$$

Element of matrix or vector or set

(often shown with subscripts)

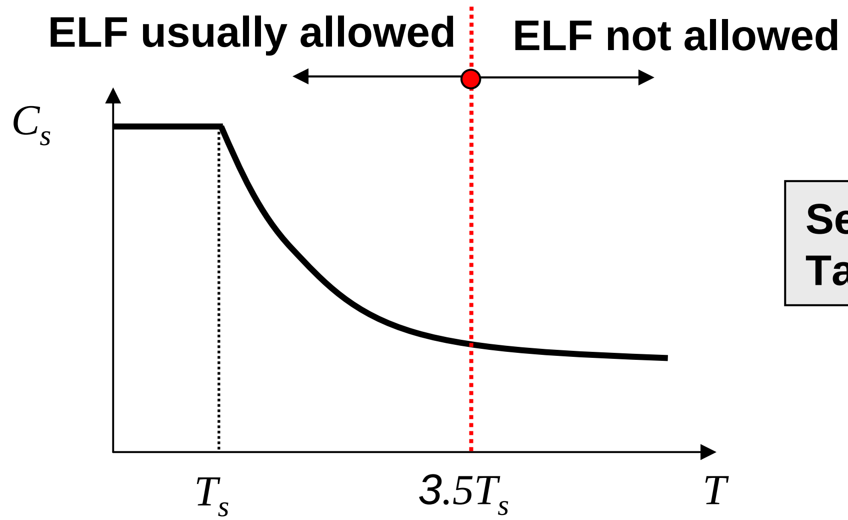
$$W \quad g$$

Scalars

Relevance to ASCE 7-05

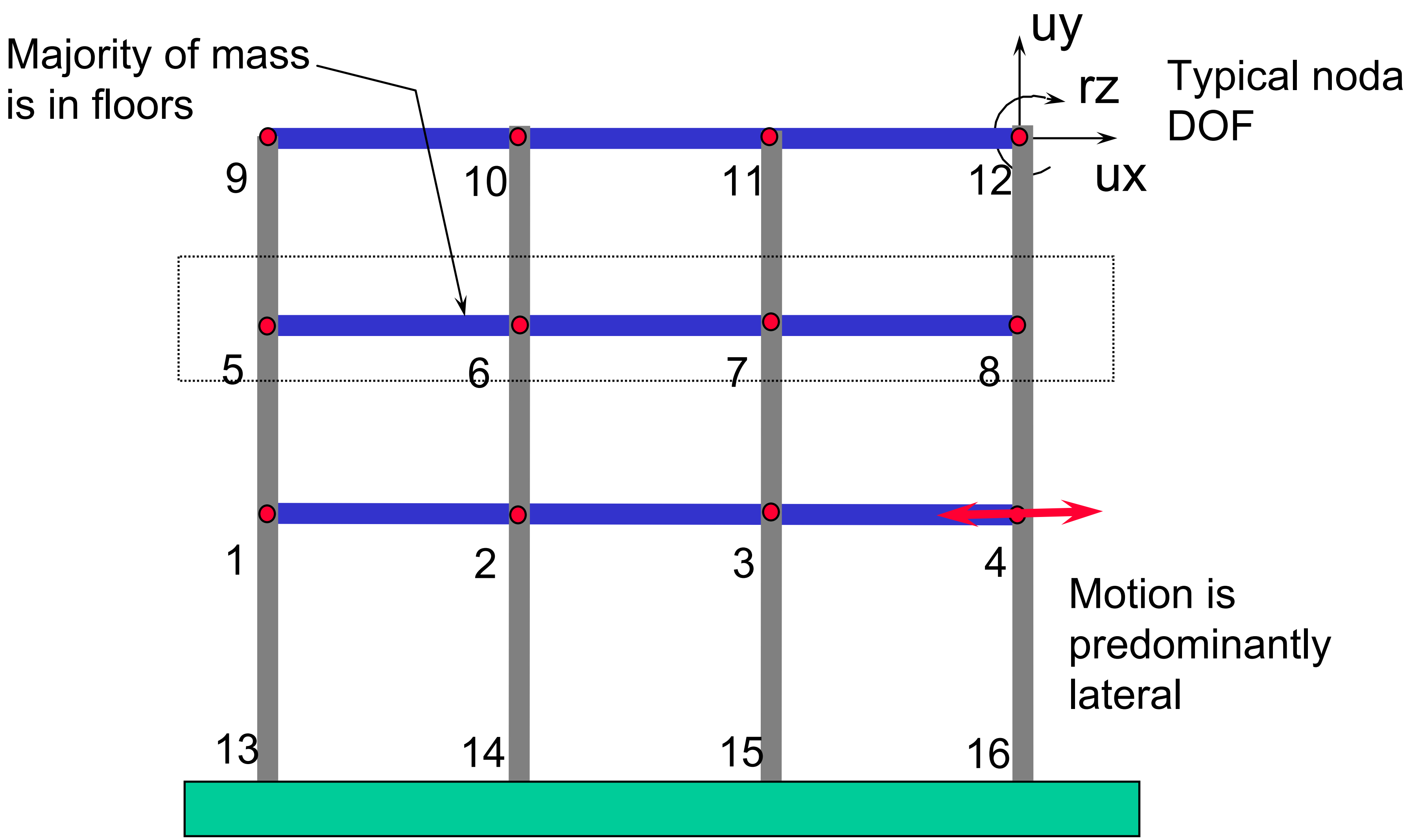
ASCE 7-05 provides guidance for three specific analysis procedures:

- Equivalent lateral force (ELF) analysis
- Modal superposition analysis (MSA)
- Response history analysis (RHA)

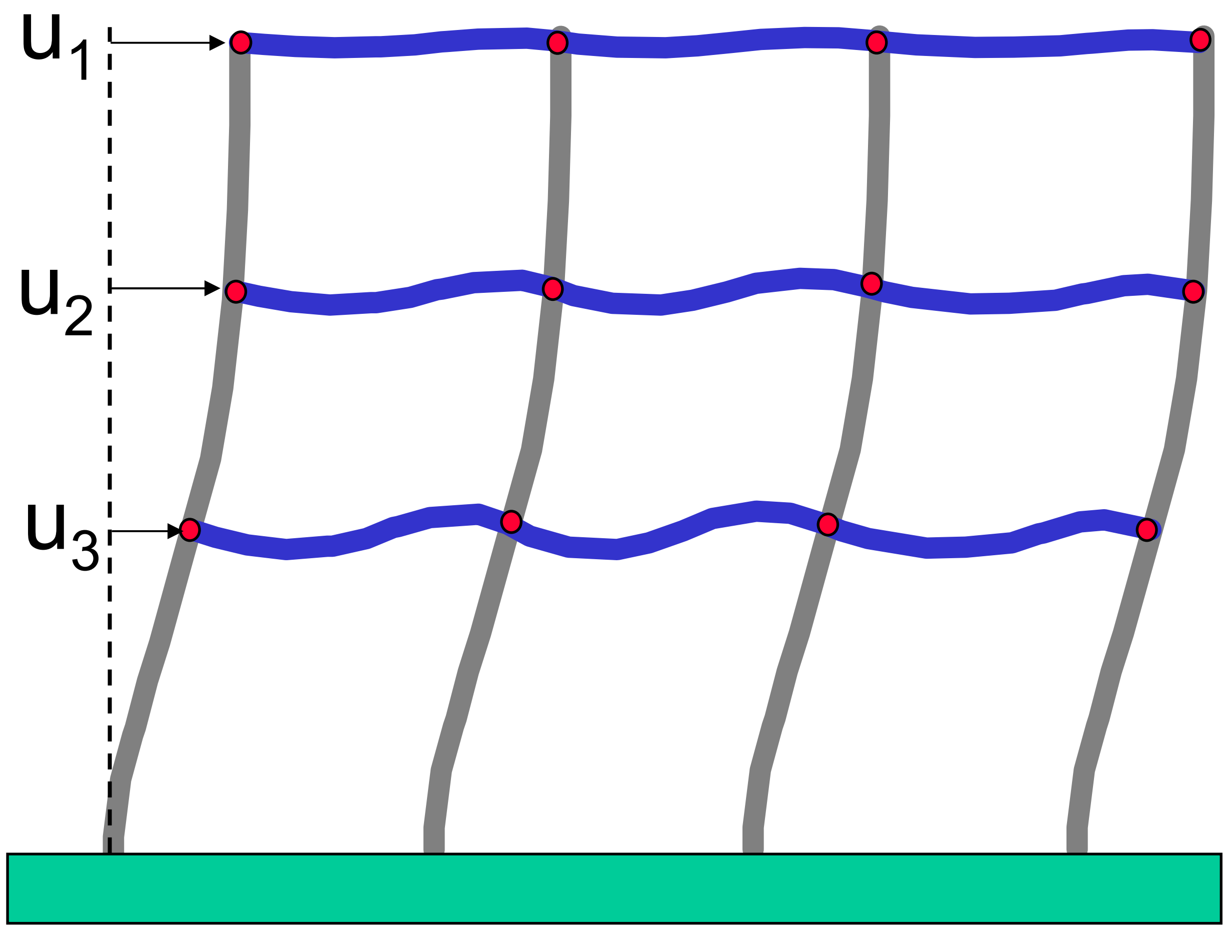


See ASCE 7-05 Table 12.6-1

Planar Frame with 36 Degrees of Freedom

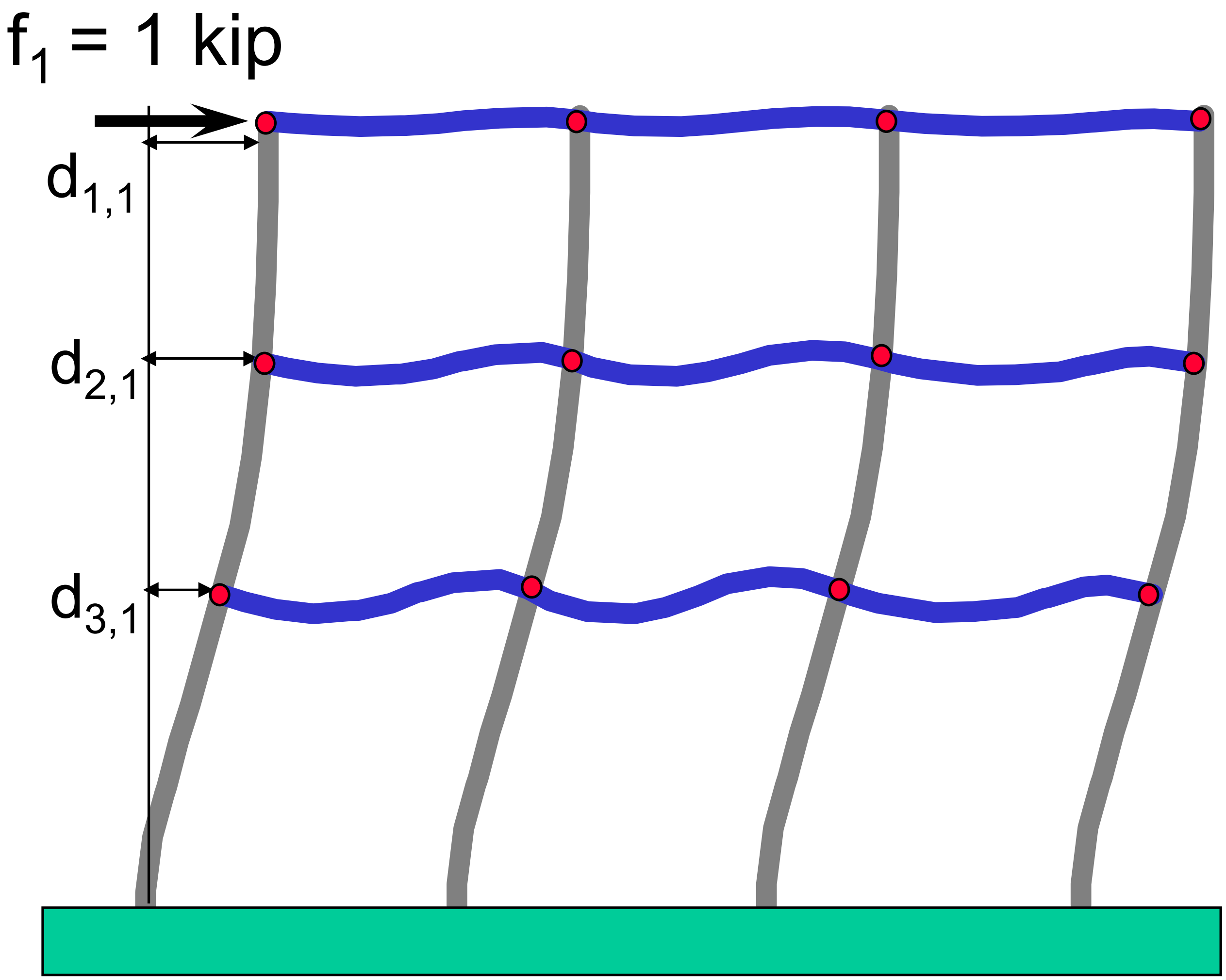


Planar Frame with 36 Static Degrees of Freedom But with Only THREE Dynamic DOF



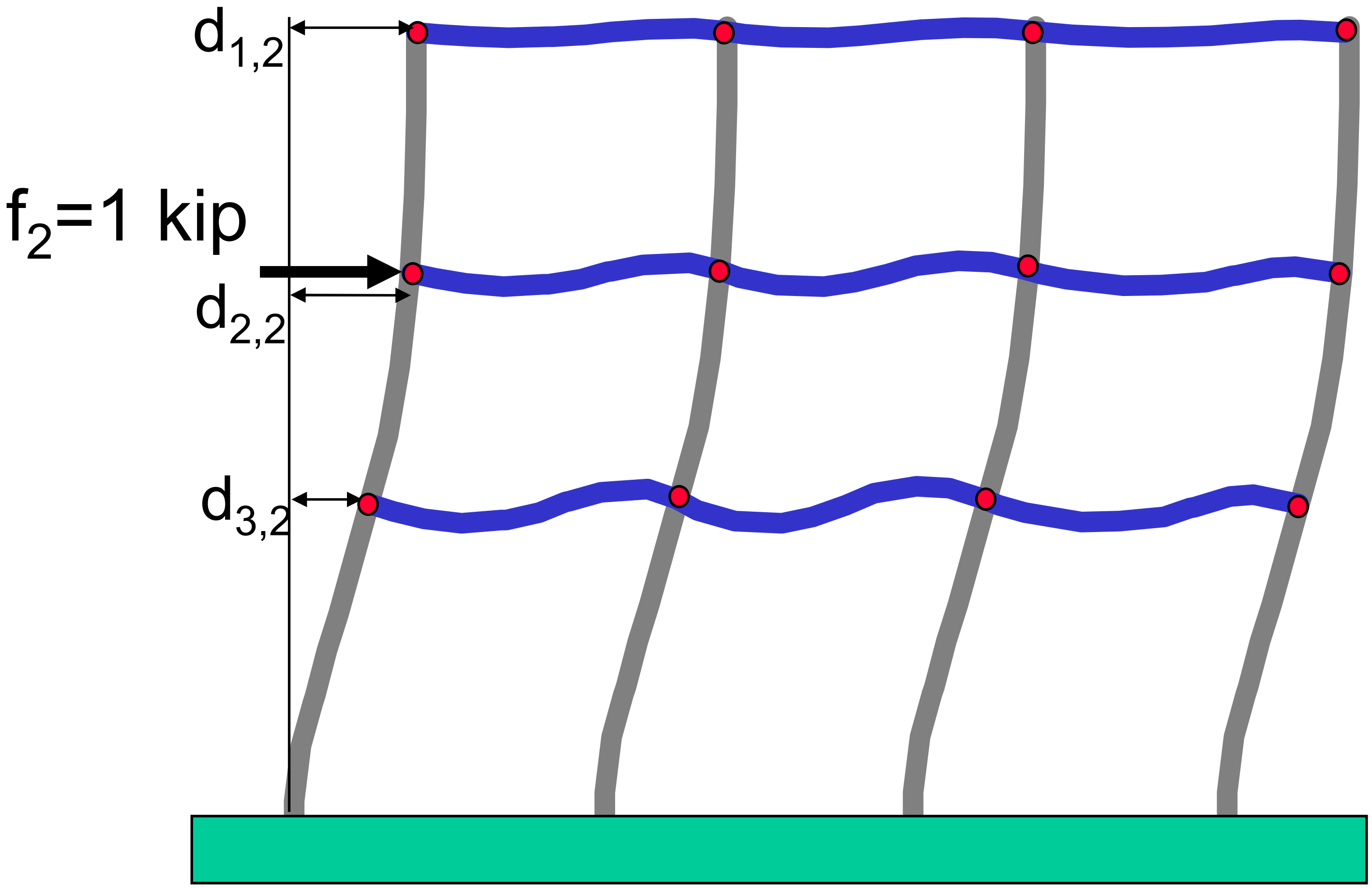
$$U = \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix}$$

Development of Flexibility Matrix



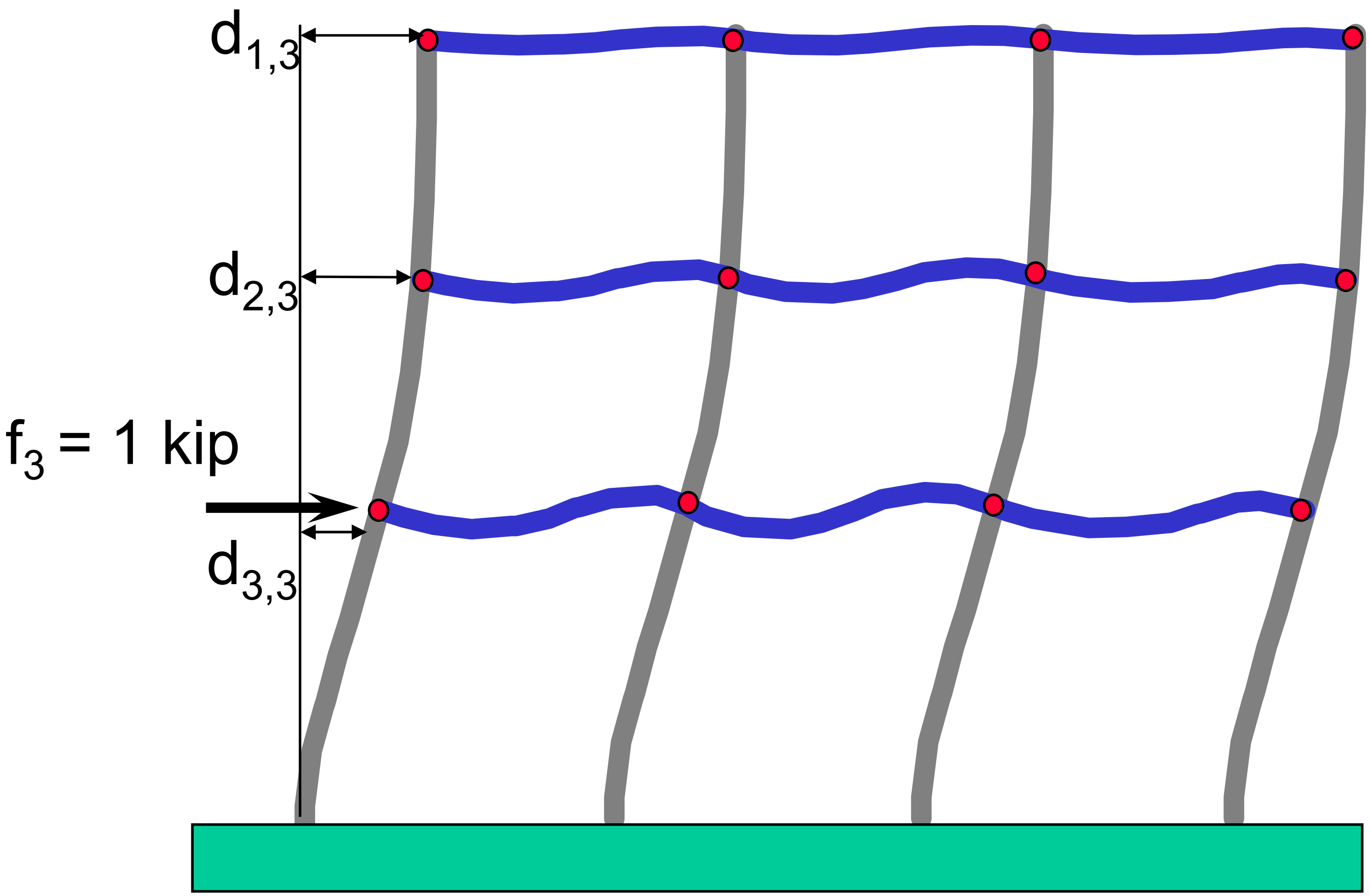
$$\begin{bmatrix} d_{1,1} & \bullet & \bullet \\ d_{2,1} & \bullet & \bullet \\ d_{3,1} & \bullet & \bullet \end{bmatrix}$$

Development of Flexibility Matrix (continued)



$$\begin{bmatrix} d_{1,1} & d_{1,2} & \bullet \\ d_{2,1} & d_{2,2} & \bullet \\ d_{3,1} & d_{3,2} & \bullet \end{bmatrix}$$

Development of Flexibility Matrix
(continued)



$$\begin{bmatrix} d_{1,1} & d_{1,2} & d_{1,3} \\ d_{2,1} & d_{2,2} & d_{2,3} \\ d_{3,1} & d_{3,2} & d_{3,3} \end{bmatrix}$$

Concept of Linear Combination of Shapes
(Flexibility)

$$U = \begin{bmatrix} d_{1,1} & d_{1,2} & d_{1,3} \\ d_{2,1} & d_{2,2} & d_{2,3} \\ d_{3,1} & d_{3,2} & d_{3,3} \end{bmatrix} \begin{Bmatrix} f_1 \\ f_2 \\ f_3 \end{Bmatrix}$$

$$U = \begin{Bmatrix} d_{1,1} \\ d_{2,1} \\ d_{3,1} \end{Bmatrix} f_1 + \begin{Bmatrix} d_{1,2} \\ d_{2,2} \\ d_{3,2} \end{Bmatrix} f_2 + \begin{Bmatrix} d_{1,3} \\ d_{2,3} \\ d_{3,3} \end{Bmatrix} f_3$$

$$D F = U \qquad K = D^{-1} \qquad K U = F$$

Static Condensation

$$\begin{bmatrix} K_{m,m} & K_{m,n} \\ K_{n,m} & K_{n,n} \end{bmatrix} \begin{Bmatrix} U_m \\ U_n \end{Bmatrix} = \begin{Bmatrix} F_m \\ \{0\} \end{Bmatrix}$$

DOF with mass

Massless DOF

1 $K_{m,m}U_m + K_{m,n}U_n = F_m$

2 $K_{n,m}U_m + K_{n,n}U_n = \{0\}$

Static Condensation
(continued)

Rearrange (2) $$U_n = -K_{n,n}^{-1} K_{n,m} U_m$$

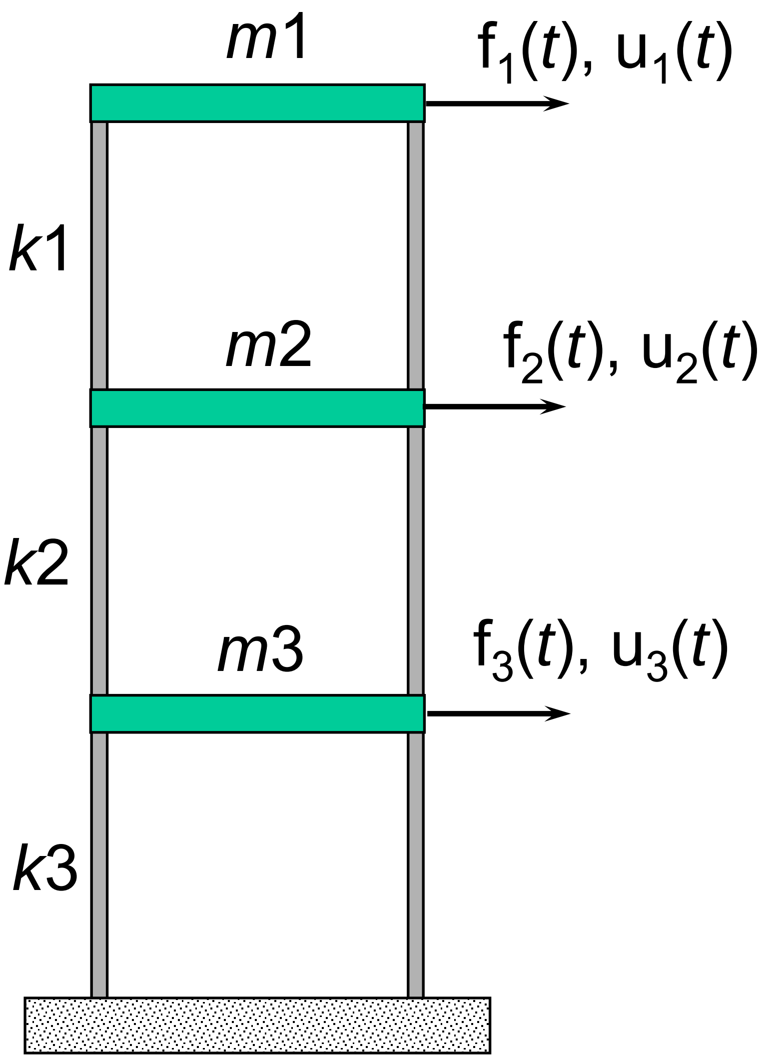
Plug into (1) $$K_{m,m} U_m - K_{m,n} K_{n,n}^{-1} K_{n,m} U_m = F_m$$

Simplify $$\left[K_{m,m} - K_{m,n} K_{n,n}^{-1} K_{n,m} \right] U_m = F_m$$

$$\boxed{\hat{K} = K_{m,m} - K_{m,n} K_{n,n}^{-1} K_{n,m}}$$

Condensed stiffness matrix

Idealized Structural Property Matrices



$$K = \begin{bmatrix} k1 & -k1 & 0 \\ -k1 & k1+k2 & -k2 \\ 0 & -k2 & k2+k3 \end{bmatrix}$$

$$M = \begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix}$$

$$F(t) = \begin{Bmatrix} f_1(t) \\ f_2(t) \\ f_3(t) \end{Bmatrix} \qquad U(t) = \begin{Bmatrix} u_1(t) \\ u_2(t) \\ u_3(t) \end{Bmatrix}$$

Note: Damping to be shown later

Coupled Equations of Motion for Undamped Forced Vibration

$$M\ddot{U}(t) + KU(t) = F(t)$$

$$\begin{bmatrix} m1 & 0 & 0 \\ 0 & m2 & 0 \\ 0 & 0 & m3 \end{bmatrix} \begin{Bmatrix} \ddot{u}_1(t) \\ \ddot{u}_2(t) \\ \ddot{u}_3(t) \end{Bmatrix} + \begin{bmatrix} k1 & -k1 & 0 \\ -k1 & k1+k2 & -k2 \\ 0 & -k3 & k2+k3 \end{bmatrix} \begin{Bmatrix} u_1(t) \\ u_2(t) \\ u_3(t) \end{Bmatrix} = \begin{Bmatrix} f_1(t) \\ f_2(t) \\ f_3(t) \end{Bmatrix}$$

DOF 1 $\quad m1\ddot{u}_1(t) + k1u_1(t) - k1u_2(t) = f_1(t)$

DOF 2 $\quad m2\ddot{u}_2(t) - k1u_1(t) + k1u_2(t) + k2u_2(t) - k2u_3(t) = f_2(t)$

DOF 3 $\quad m3\ddot{u}_3(t) - k2u_2(t) + k2u_3(t) + k3u_3(t) = f_3(t)$

Developing a Way To Solve the Equations of Motion

- This will be done by a transformation of coordinates from *normal coordinates* (displacements at the nodes) To *modal coordinates* (amplitudes of the natural Mode shapes).
- Because of the *orthogonality property* of the natural mode shapes, the equations of motion become uncoupled, allowing them to be solved as SDOF equations.
- After solving, we can transform back to the normal coordinates.

Solutions for System in Undamped Free Vibration (Natural Mode Shapes and Frequencies)

$$M\ddot{U}(t)+KU(t)=\{0\}$$

Assume $\quad U(t)=\phi\sin\omega t \quad \ddot{U}(t)=-\omega^2\phi\sin\omega t$

Then $K\phi-\omega^2 M\phi=\{0\}$ has three (n) solutions:

$$\phi_1=\begin{Bmatrix}\phi_{1,1}\\ \phi_{2,1}\\ \phi_{3,1}\end{Bmatrix},\quad \omega_1 \qquad \phi_2=\begin{Bmatrix}\phi_{1,2}\\ \phi_{2,2}\\ \phi_{3,2}\end{Bmatrix},\quad \omega_2 \qquad \phi_3=\begin{Bmatrix}\phi_{1,3}\\ \phi_{2,3}\\ \phi_{3,3}\end{Bmatrix},\quad \omega_3$$

Natural frequency

Natural mode shape

Solutions for System in Undamped Free Vibration (continued)

For a SINGLE Mode

$$K\phi = \omega^2 M\phi$$

$$K\Phi = M\Phi\Omega^2$$

For ALL Modes

Where: $\Phi = \begin{bmatrix} \phi_1 & \phi_2 & \phi_3 \end{bmatrix}$

$$\Omega^2 = \begin{bmatrix} \omega_1^2 & & \\ & \omega_2^2 & \\ & & \omega_3^2 \end{bmatrix}$$

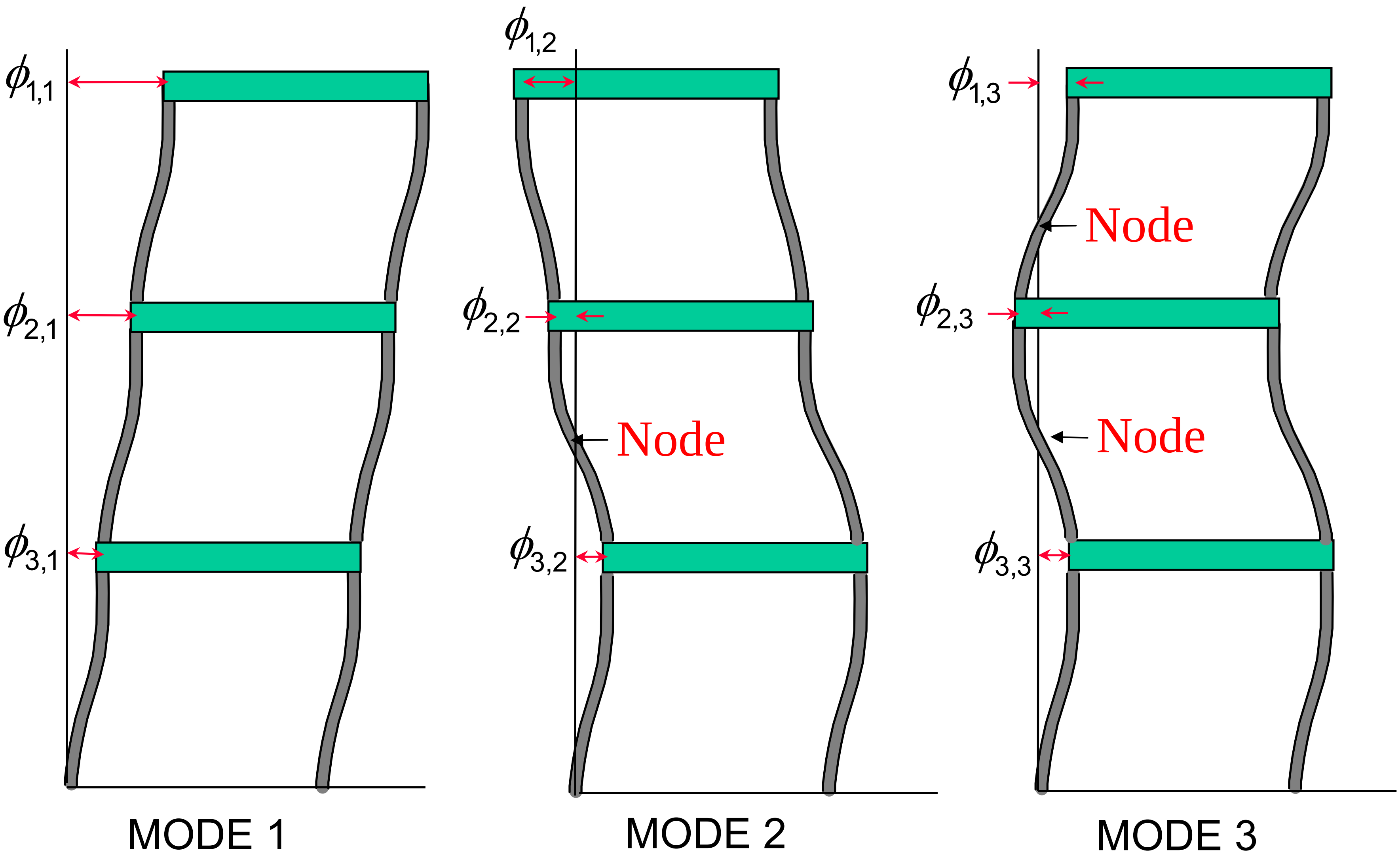
Note: Mode shape has arbitrary scale; usually

$$\phi_{1,i} = 1.0$$

or

$$\Phi^T M \Phi = I$$

Mode Shapes for Idealized 3-Story Frame



Concept of Linear Combination of Mode Shapes (Transformation of Coordinates)

$$U = \Phi Y$$

Mode shape

$$U = \begin{bmatrix} \phi_{1,1} & \phi_{1,2} & \phi_{1,3} \\ \phi_{2,1} & \phi_{2,2} & \phi_{2,3} \\ \phi_{3,1} & \phi_{3,2} & \phi_{3,3} \end{bmatrix} \begin{Bmatrix} y_1 \\ y_2 \\ y_3 \end{Bmatrix} \qquad U = \begin{Bmatrix} \phi_{1,1} \\ \phi_{2,1} \\ \phi_{3,1} \end{Bmatrix} y_1 + \begin{Bmatrix} \phi_{1,2} \\ \phi_{2,2} \\ \phi_{3,2} \end{Bmatrix} y_2 + \begin{Bmatrix} \phi_{1,3} \\ \phi_{2,3} \\ \phi_{3,3} \end{Bmatrix} y_3$$

Modal coordinate = amplitude of mode shape

Orthogonality Conditions

$$\Phi = \begin{bmatrix} \phi_1 & \phi_2 & \phi_3 \end{bmatrix}$$

Generalized mass

$$\Phi^T M \Phi = \begin{bmatrix} m_1^* & & \\ & m_2^* & \\ & & m_3^* \end{bmatrix}$$

Generalized stiffness

$$\Phi^T K \Phi = \begin{bmatrix} k_1^* & & \\ & k_2^* & \\ & & k_3^* \end{bmatrix}$$

Generalized damping

$$\Phi^T C \Phi = \begin{bmatrix} c_1^* & & \\ & c_2^* & \\ & & c_3^* \end{bmatrix}$$

Generalized force

$$\Phi^T \mathrm{F}(t) = \begin{Bmatrix} \mathrm{f}_1^*(t) \\ \mathrm{f}_2^*(t) \\ \mathrm{f}_3^*(t) \end{Bmatrix}$$

Development of Uncoupled Equations of Motion

MDOF equation of motion: $\mathrm{M}\ddot{\mathrm{U}} + \mathrm{C}\dot{\mathrm{U}} + \mathrm{KU} = \mathrm{F}(t)$

Transformation of coordinates: $\mathrm{U} = \Phi \mathrm{Y}$

Substitution: $\mathrm{M}\Phi \ddot{\mathrm{Y}} + \mathrm{C}\Phi \dot{\mathrm{Y}} + \mathrm{K}\Phi \mathrm{Y} = \mathrm{F}(t)$

Premultiply by Φ^{T}: $\Phi^T M \Phi \ddot{Y} + \Phi^T C \Phi \dot{Y} + \Phi^T K \Phi Y = \Phi^T F(t)$

Using orthogonality conditions, uncoupled equations of motion are:

$$\begin{bmatrix} m_1^* & & \\ & m_2^* & \\ & & m_3^* \end{bmatrix} \begin{Bmatrix} \ddot{y}_1 \\ \ddot{y}_2 \\ \ddot{y}_3 \end{Bmatrix} + \begin{bmatrix} c_1^* & & \\ & c_2^* & \\ & & c_3^* \end{bmatrix} \begin{Bmatrix} \dot{y}_1 \\ \dot{y}_2 \\ \dot{y}_3 \end{Bmatrix} + \begin{bmatrix} k_1^* & & \\ & k_2^* & \\ & & k_3^* \end{bmatrix} \begin{Bmatrix} y_1 \\ y_2 \\ y_3 \end{Bmatrix} = \begin{Bmatrix} f_1^*(t) \\ f_2^*(t) \\ f_3^*(t) \end{Bmatrix}$$

Development of Uncoupled Equations of Motion (Explicit Form)

Mode 1 $$m_1^*\ddot{y}_1 + c_1^*\dot{y}_1 + k_1^* y_1 = f_1^*(t)$$

Mode 2 $$m_2^*\ddot{y}_2 + c_2^*\dot{y}_2 + k_2^* y_2 = f_2^*(t)$$

Mode 3 $$m_3^*\ddot{y}_3 + c_3^*\dot{y}_3 + k_3^* y_3 = f_3^*(t)$$

Development of Uncoupled Equations of Motion (Explicit Form)

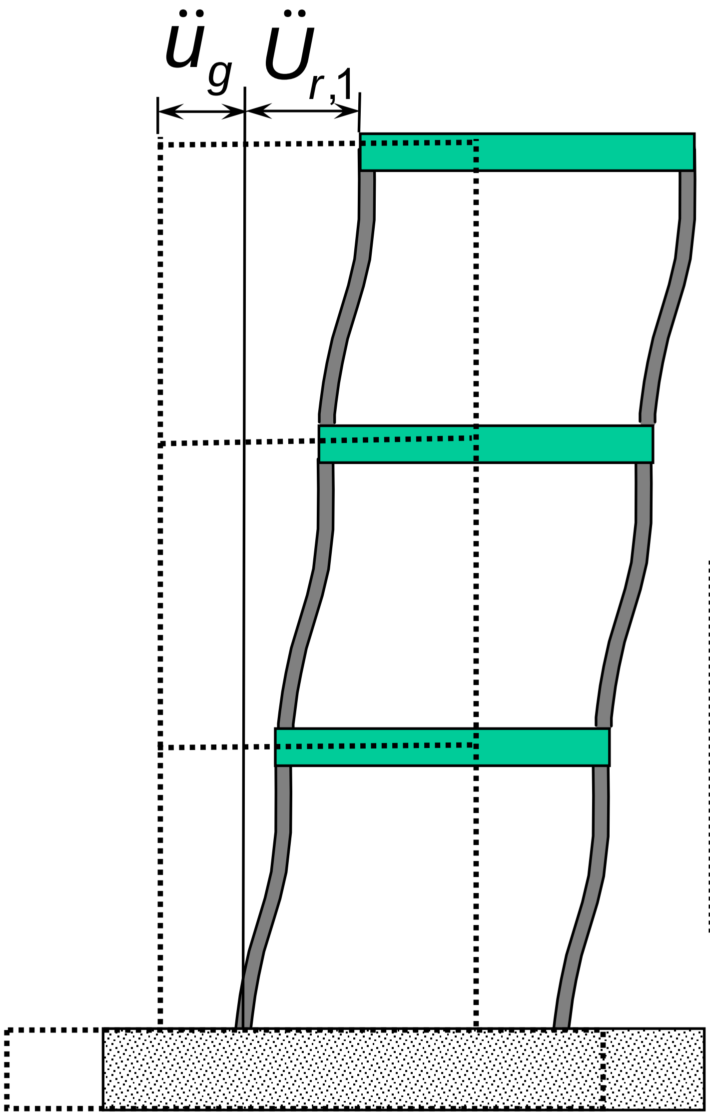
Simplify by dividing through by m* and defining $\xi_i = \dfrac{c_i^*}{2m_i^*\omega_i}$

Mode 1 $$\ddot{y}_1 + 2\xi_1\omega_1\dot{y}_1 + \omega_1^2 y_1 = f_1^*(t)/m_1^*$$

Mode 2 $$\ddot{y}_2 + 2\xi_2\omega_2\dot{y}_2 + \omega_2^2 y_2 = f_2^*(t)/m_2^*$$

Mode 3 $$\ddot{y}_3 + 2\xi_3\omega_3\dot{y}_3 + \omega_3^2 y_3 = f_3^*(t)/m_3^*$$

Earthquake "Loading" for MDOF System



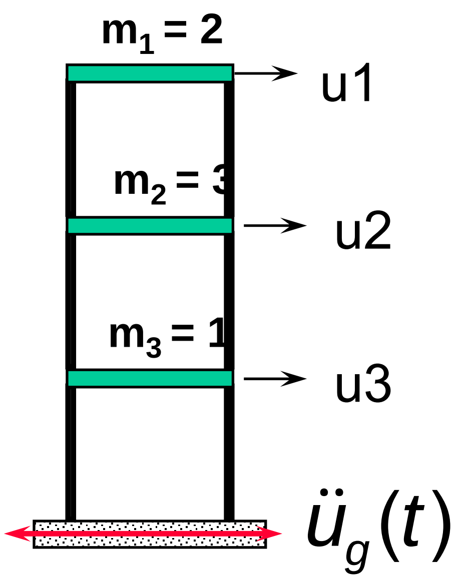
$$F_I(t) = M\begin{Bmatrix} \ddot{u}_g(t) + \ddot{u}_{r,1}(t) \\ \ddot{u}_g(t) + \ddot{u}_{r,2}(t) \\ \ddot{u}_g(t) + \ddot{u}_{r,3}(t) \end{Bmatrix} =$$

$$\boxed{M\begin{Bmatrix} 1.0 \\ 1.0 \\ 1.0 \end{Bmatrix}\ddot{u}_g(t)} + M\begin{Bmatrix} \ddot{u}_{r,1}(t) \\ \ddot{u}_{r,2}(t) \\ \ddot{u}_{r,3}(t) \end{Bmatrix}$$

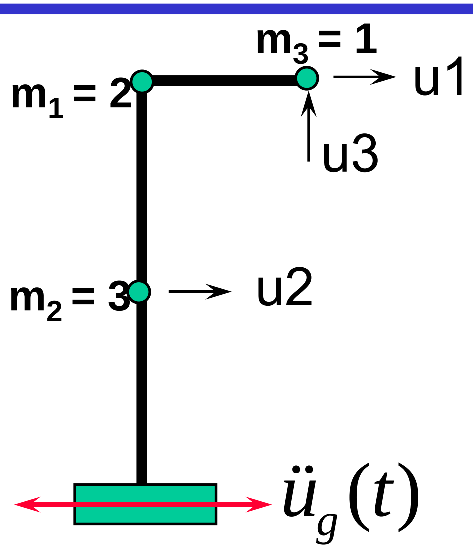
Move to RHS as $F_{EFF}(t) = -\,M\,R\ddot{u}_g(t)$

Modal Earthquake Loading

$$F^*(t) = -\Phi^T M R \ddot{u}_g(t)$$



$$M = \begin{bmatrix} 2 & & \\ & 3 & \\ & & 1 \end{bmatrix} \qquad R = \begin{Bmatrix} 1 \\ 1 \\ 1 \end{Bmatrix}$$



$$M = \begin{bmatrix} 2+1 & & \\ & 3 & \\ & & 1 \end{bmatrix} \qquad R = \begin{Bmatrix} 1 \\ 1 \\ 0 \end{Bmatrix}$$

Definition of Modal Participation Factor

For earthquakes: $f_i^*(t) = -\phi_i^T M R \ddot{u}_g(t)$

Typical modal equation:

$$\ddot{y}_i + 2\xi_i \omega_i \dot{y}_i + \omega_i^2 y_i = \frac{f_i^*(t)}{m_i^*} = -\boxed{\frac{\phi_i^T M R}{m_i^*}} \ddot{u}_g(t)$$

Modal participation factor p_i

Caution Regarding Modal Participation Factor

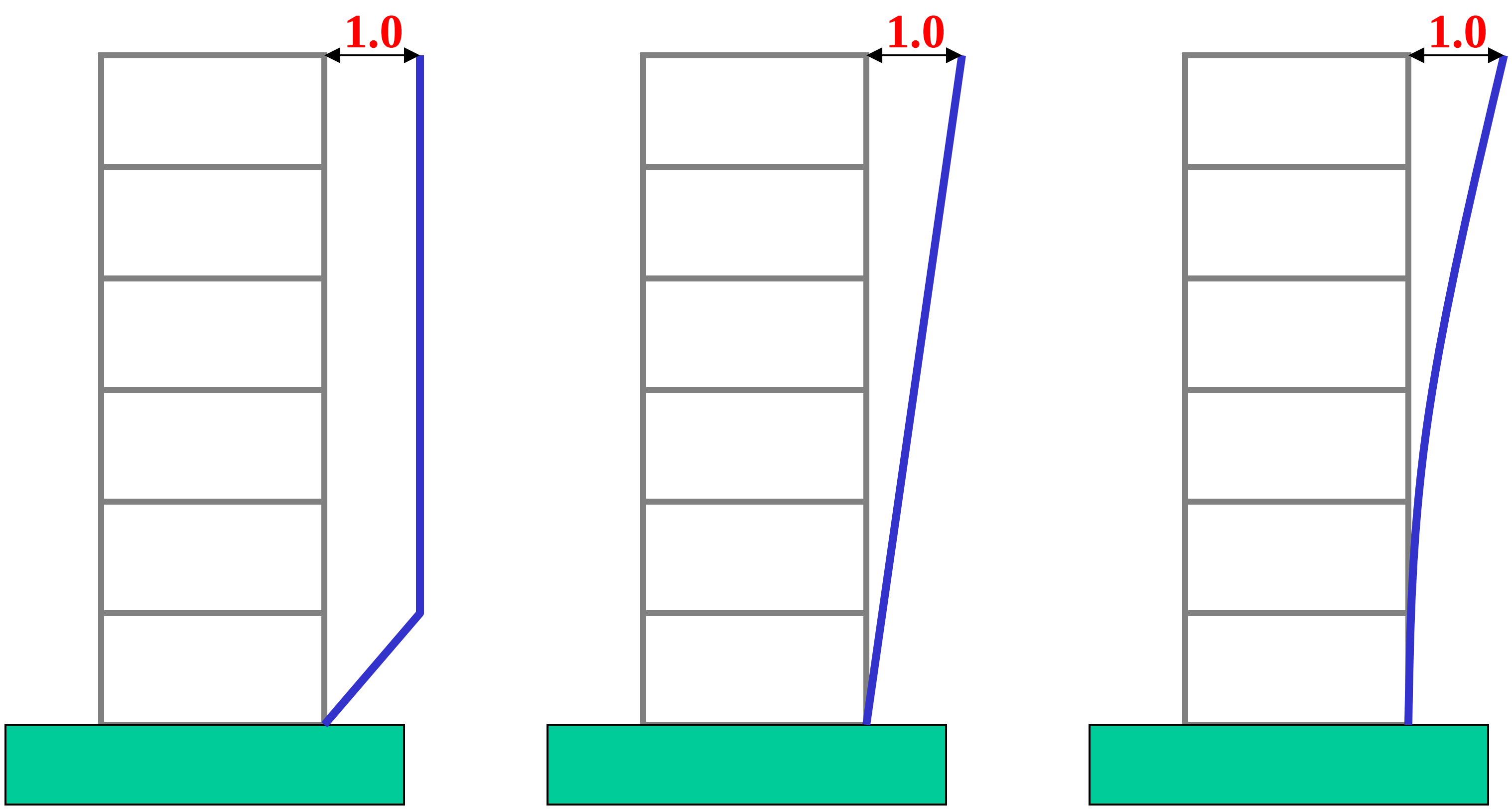
$$p_i = \frac{\phi_i^T M R}{m_i^*}$$

$$\boxed{\phi_i^T M \phi_i}$$

Its value is dependent on the (arbitrary) method used to scale the mode shapes.

Variation of First Mode Participation Factor with First Mode Shape

$p_1 = 1.0$ $p_1 = 1.4$ $p_1 = 1.6$



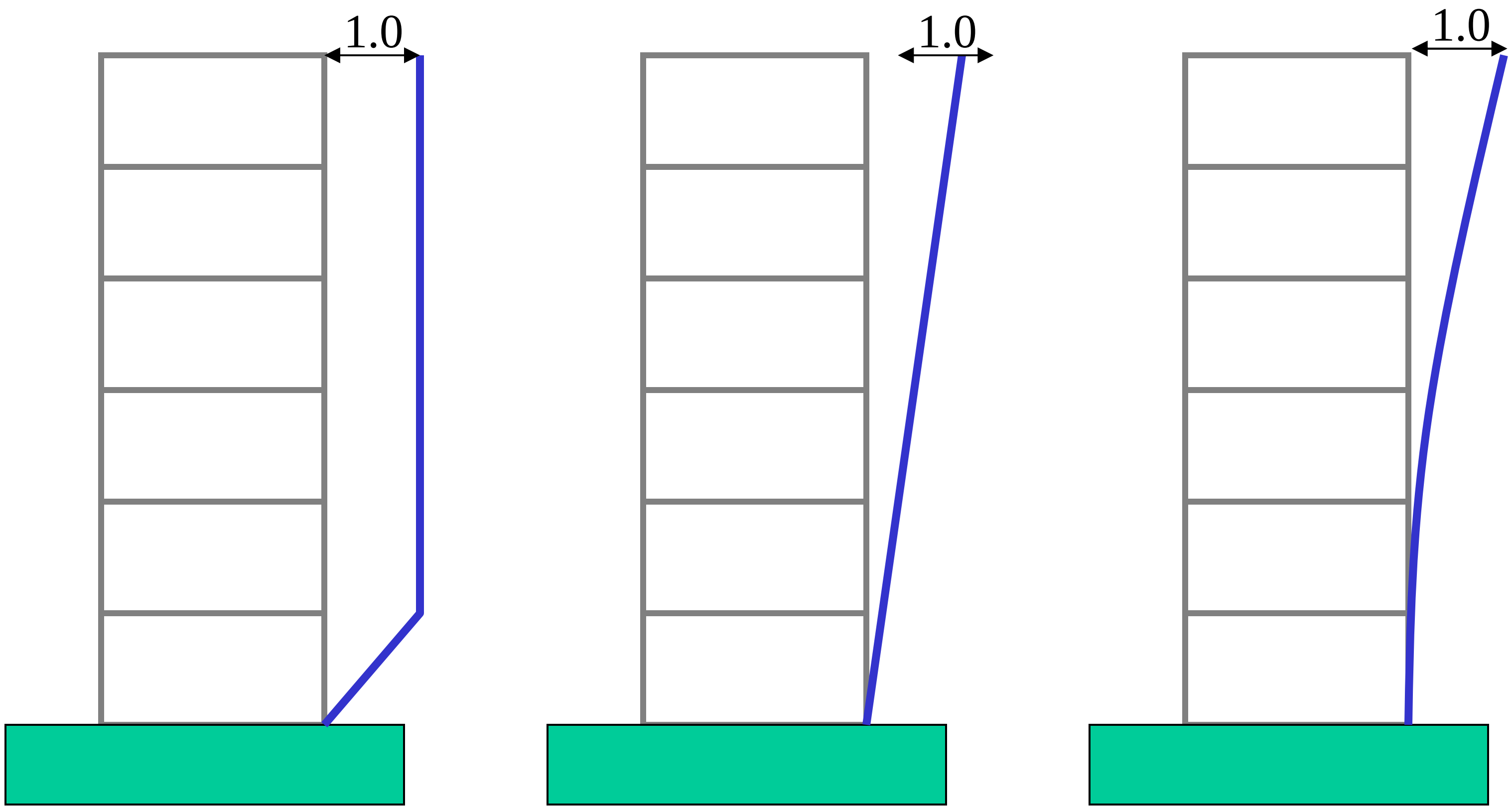
Concept of Effective Modal Mass

For each Mode *I*, $\bar{m}_i = p_i^2 m_i^*$

- The sum of the effective modal mass for all modes is
  equal to the total structural mass.
- The value of effective modal mass is *independent* of mode shape scaling.
- Use enough modes in the analysis to provide a total effective mass not less than 90% of the total structural mass.

Variation of First Mode Effective Mass with First Mode Shape

$\bar{m}_1 / M = 1.0$ $\bar{m}_1 / M = 0.9$ $\bar{m}_1 / M = 0.7$



Derivation of Effective Modal Mass

(continued)

For each mode:

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

SDOF system:

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

Modal response history, $q_i(t)$ is obtained by first solving the SDOF system.

Derivation of Effective Modal Mass

(continued)

From previous slide $$y_i(t) = p_i\, q_i(t)$$

Recall $$u_i(t) = \phi_i\, y_i(t)$$

Substitute $$u_i(t) = p_i\, \phi_i\, q_i(t)$$

Derivation of Effective Modal Mass
(continued)

Applied “static” forces required to produce $u_i(t)$:

$$V_i(t) = Ku_i(t) = P_i K \phi_i q_i(t)$$

Recall: $$K\phi_i = \omega_i^2 M \phi_i$$

Substitute:

$$V_i(t) = M\phi_i P_i \omega_i^2 q_i(t)$$

Derivation of Effective Modal Mass
(continued)

Total shear in mode: $\bar{V}_i = V_i^T R$

$$\bar{V}_i = (M\phi_i)^T RP_i\omega^2 q_i(t) = \phi_i^T MRP_i \boxed{\omega^2 q_i(t)}$$

"Acceleration" in mode

Define effective modal mass:

$$\bar{M}_i = \phi_i^T MRP_i$$

and

$$\bar{V}_i = \bar{M}_i \omega^2 q_i(t)$$

Derivation of Effective Modal Mass
(continued)

$$\bar{M}_i = \phi_i^T MRP_i = \left[\frac{\phi_i^T MR}{\phi_i^T M \phi_i} \right]_i \phi_i^T M \phi_i P_i$$

$$\bar{M}_i = P_i^2 m_i^*$$

Development of a Modal Damping Matrix

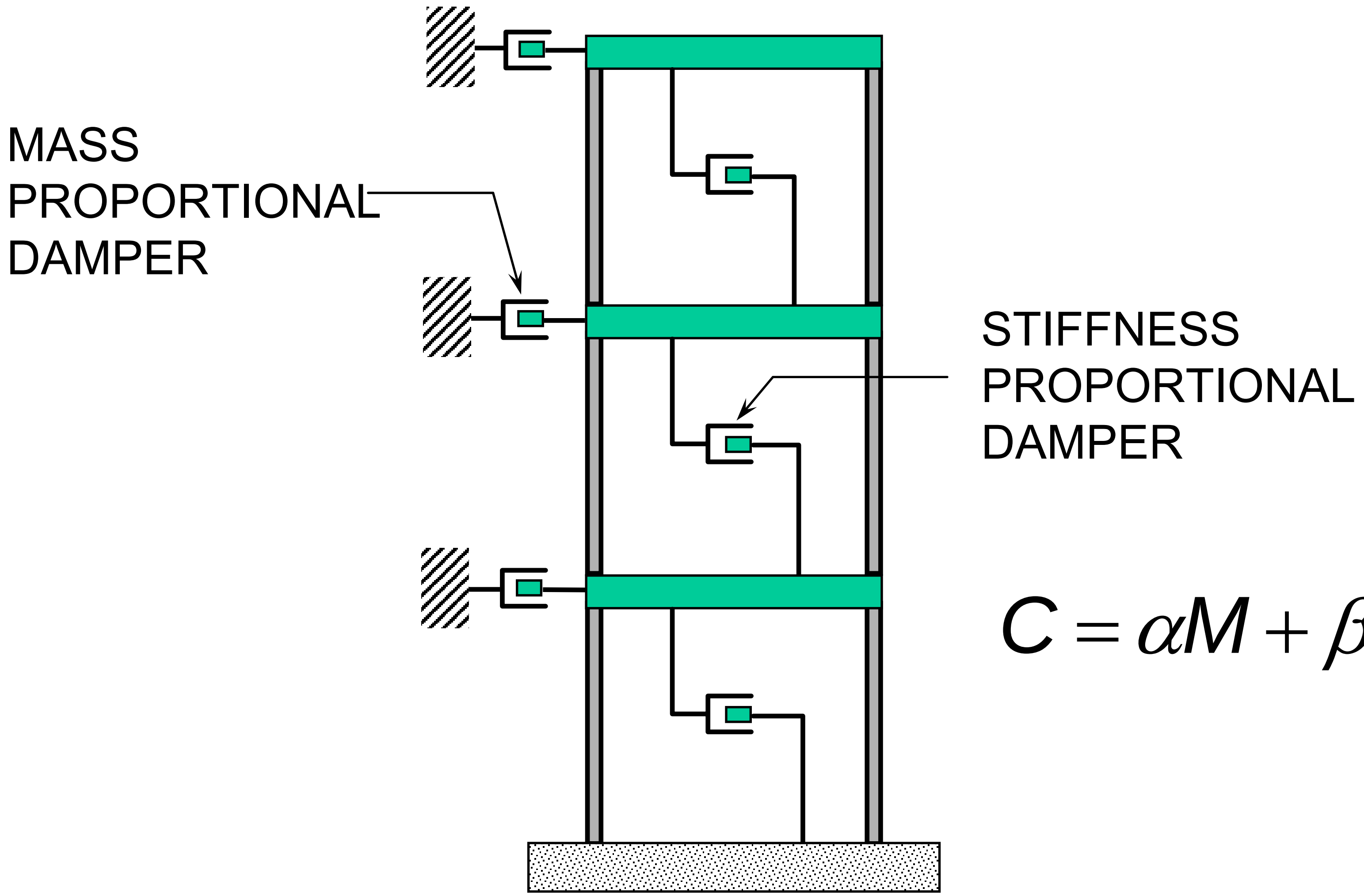
In previous development, we have assumed:

$$\Phi^T C \Phi = \begin{bmatrix} c_1^* & & \\ & c_2^* & \\ & & c_3^* \end{bmatrix}$$

Two methods described herein:

- Rayleigh "proportional damping"
- Wilson "discrete modal damping"

Rayleigh Proportional Damping
(continued)



$$C = \alpha M + \beta K$$

Rayleigh Proportional Damping
(continued)

$$C = \alpha M + \beta K$$

For modal equations to be uncoupled:

$$2\omega_n \xi_n = \phi_n^T C \phi_n$$

Assumes

$$\Phi^T M \Phi = I$$

Using orthogonality conditions:

$$2\omega_n \xi_n = \alpha + \beta \omega_n^2$$

$$\xi_n = \frac{1}{2\omega_n}\alpha + \frac{\omega_n}{2}\beta$$

Rayleigh Proportional Damping
(continued)

Select damping value in two modes, ξ_m and ξ_n

Compute coefficients α and β:

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = 2\frac{\omega_m \omega_n}{\omega_n^2 - \omega_m^2} \begin{bmatrix} \omega_n & -\omega_m \\ -1/\omega_n & 1/\omega_m \end{bmatrix} \begin{Bmatrix} \xi_m \\ \xi_n \end{Bmatrix}$$

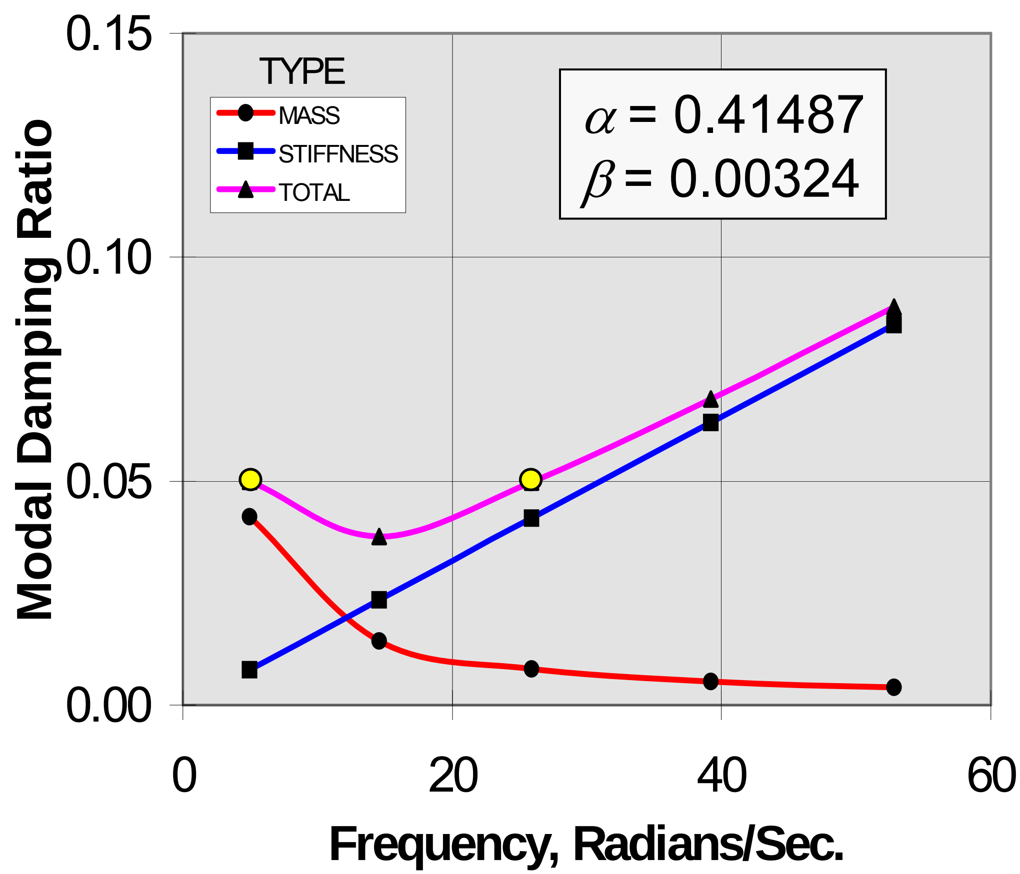
Form damping matrix $C = \alpha M + \beta K$

Rayleigh Proportional Damping (Example)

5% critical in Modes 1 and 3

Structural frequencies

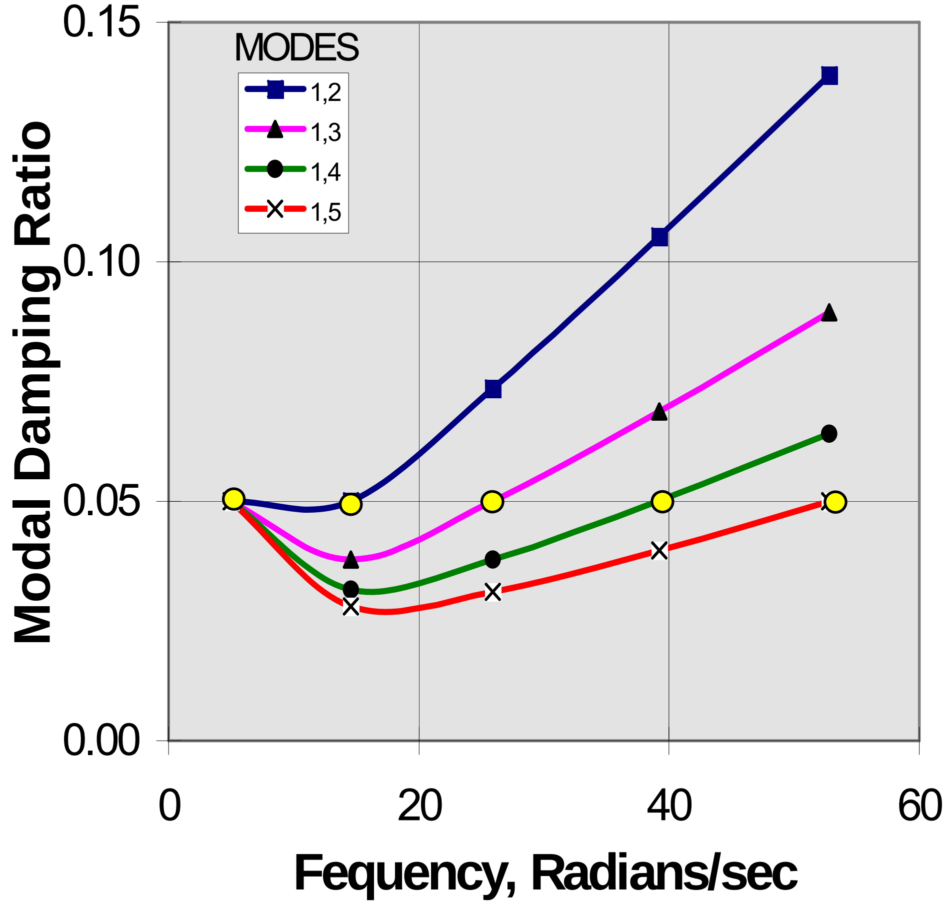
Mode	ω
1	**4.94**
2	14.6
3	**25.9**
4	39.2
5	52.8



Rayleigh Proportional Damping (Example)
5% Damping in Modes 1 & 2, 1 & 3, 1 & 4, or 1 & 5

Proportionality factors
(5% each indicated mode)

Modes	α	β
1 & 2	.36892	0.00513
1 & 3	.41487	0.00324
1 & 4	.43871	0.00227
1 & 5	.45174	0.00173



Wilson Damping

Directly specify modal damping values ξ_i^*

$$\Phi^T C \Phi = \begin{bmatrix} c_1^* & & \\ & c_2^* & \\ & & c_3^* \end{bmatrix} = \begin{bmatrix} 2m_1^*\omega_1\xi_1^* & & \\ & 2m_2^*\omega_2\boxed{\xi_2^*} & \\ & & 2m_3^*\omega_3\xi_3^* \end{bmatrix}$$

Formation of Explicit Damping Matrix From "Wilson" Modal Damping

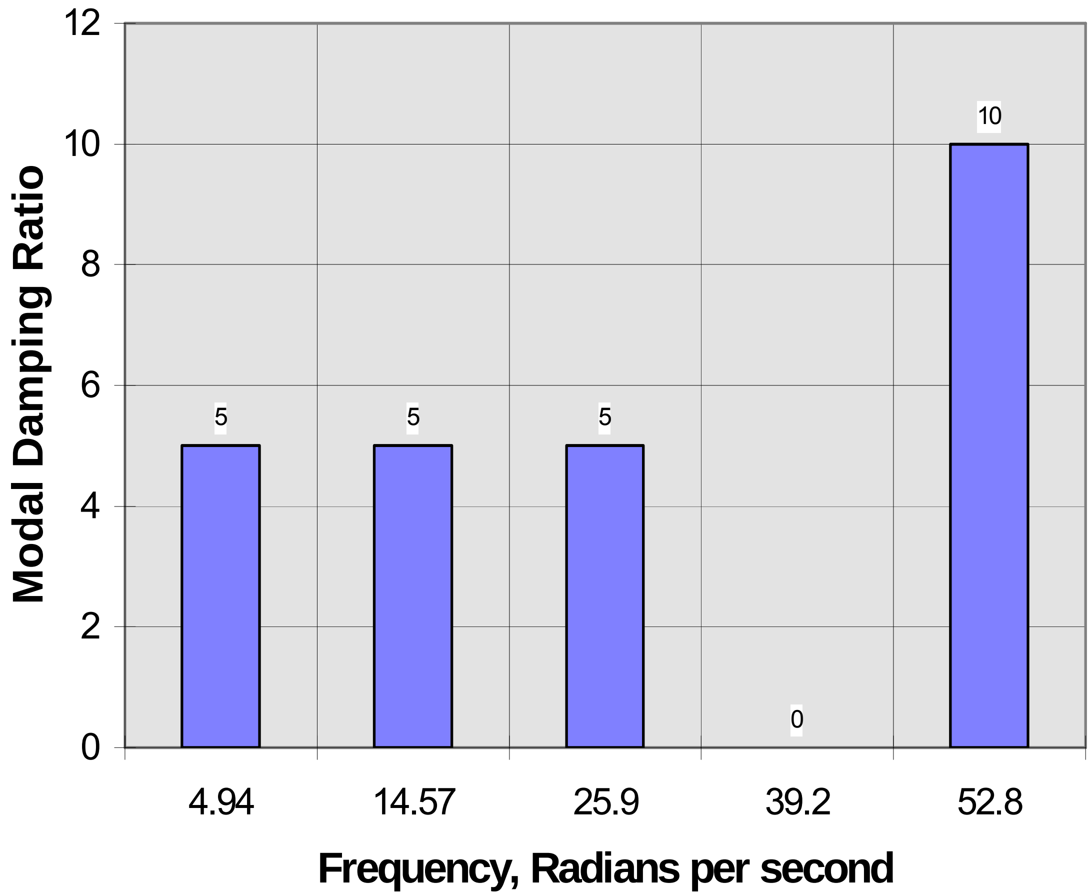
(NOT Usually Required)

$$\Phi^T C \Phi = \begin{bmatrix} 2\xi_1\omega_1 & & & & \\ & 2\xi_2\omega_2 & & & \\ & & \bullet\bullet\bullet & & \\ & & & 2\xi_{n-1}\omega_{n-1} & \\ & & & & 2\xi_n\omega_n \end{bmatrix} = c$$

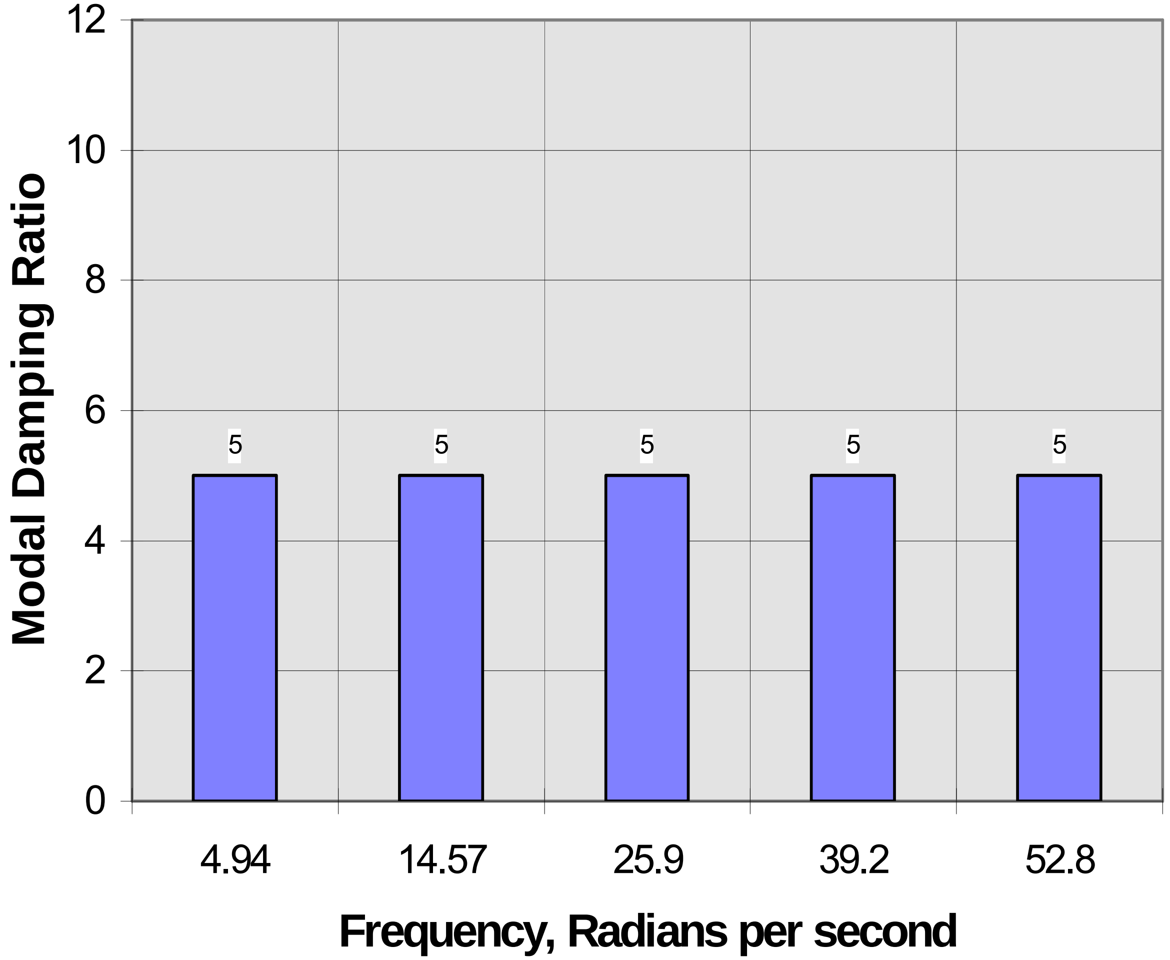
$$\left(\Phi^T\right)^{-1} c\Phi^{-1} = C$$

$$C = M\left[\sum_{i=1}^{n} 2\xi_i\omega_i\phi_i^T\phi_i\right]M$$

Wilson Damping (Example)
5% Damping in Modes 1 and 2, 3
10% in Mode 5, Zero in Mode 4



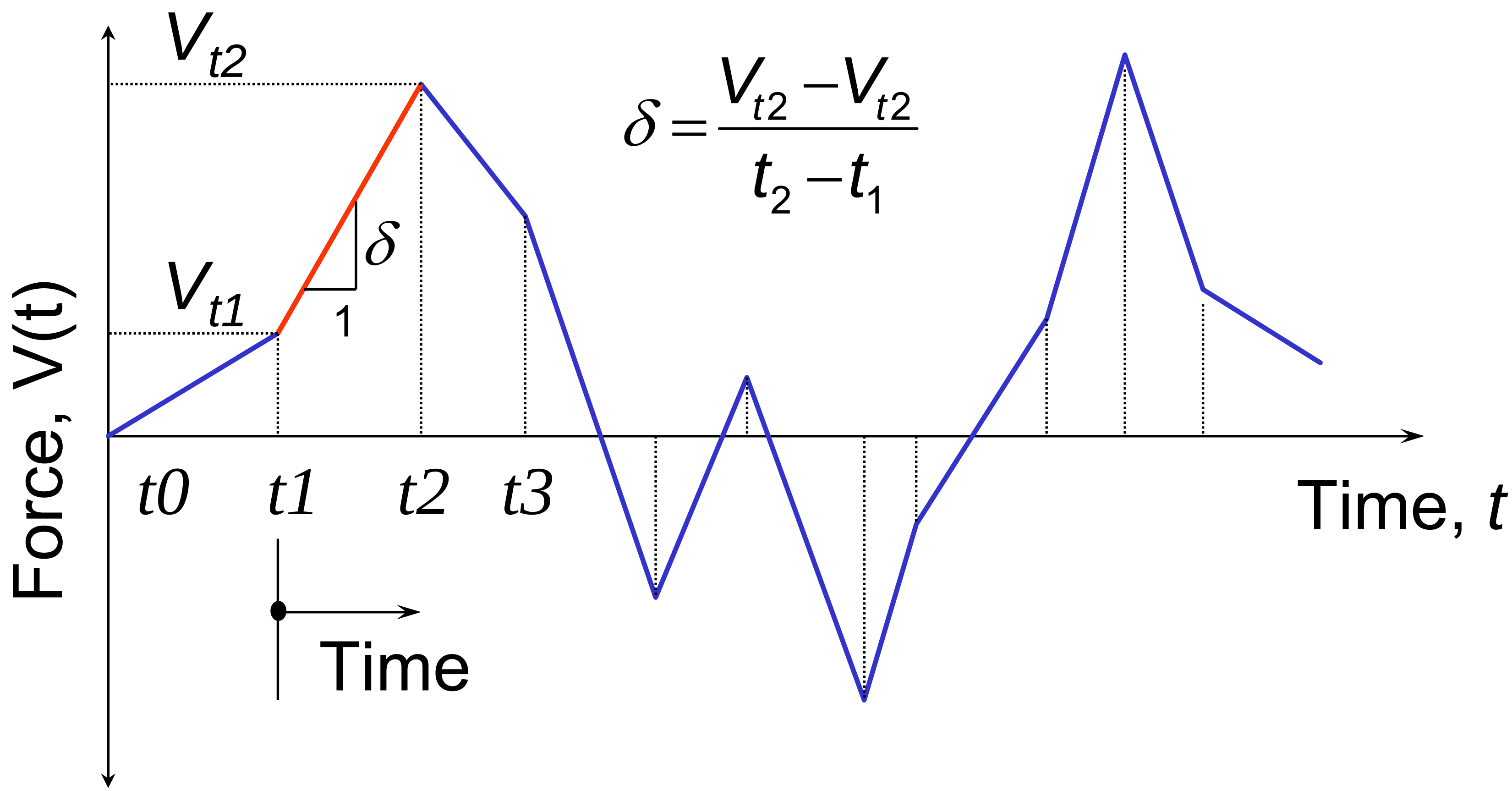
Wilson Damping (Example)
5% Damping in all Modes



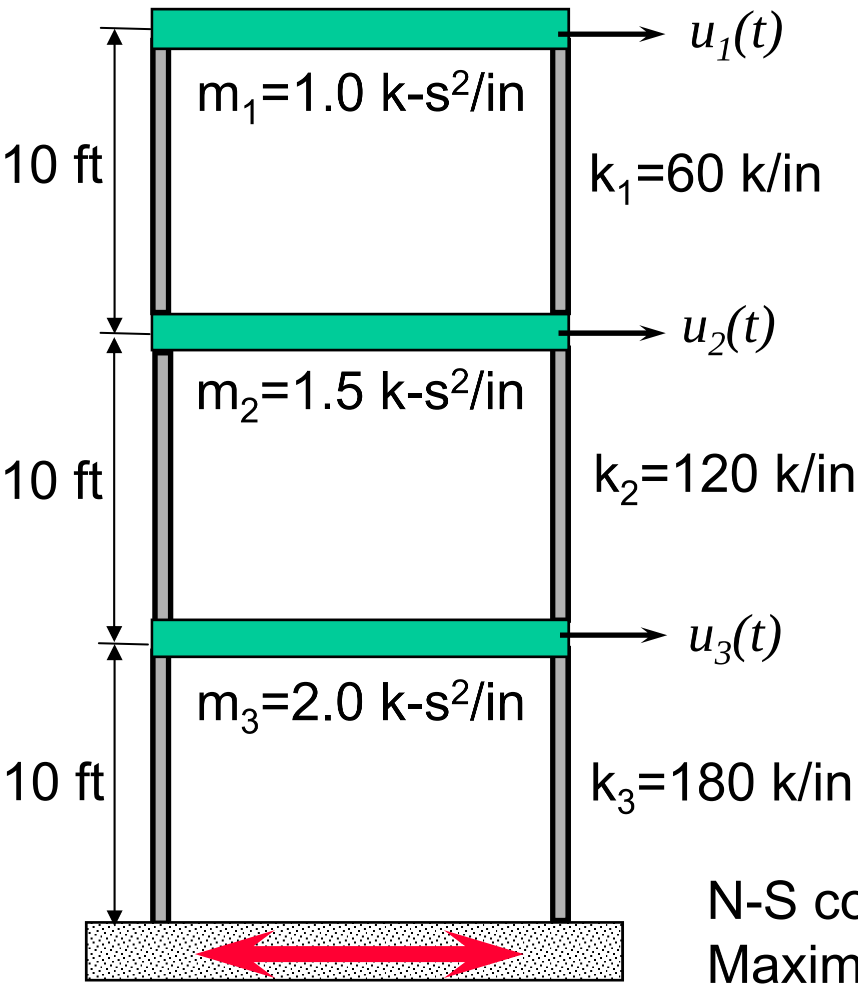
Solution of MDOF Equations of Motion

- Explicit (step by step) integration of *coupled* equations
- Explicit integration of FULL SET of *uncoupled* equations
- Explicit integration of PARTIAL SET of *uncoupled* Equations (approximate)
- Modal response spectrum analysis (approximate)

Computed Response for Piecewise Linear Loading



Example of MDOF Response of Structure Responding to 1940 El Centro Earthquake

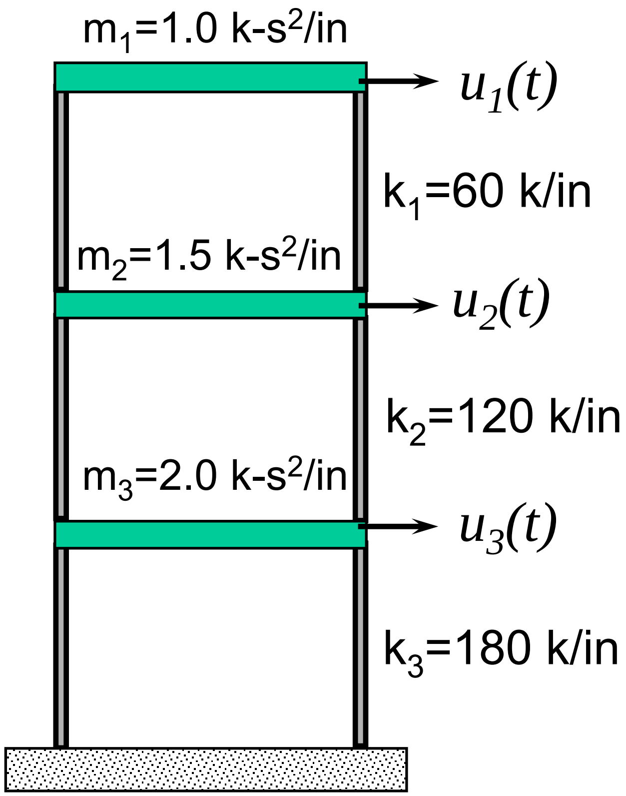


Example 1

Assume Wilson damping with 5% critical in each mode.

N-S component of 1940 El Centro earthquake
Maximum acceleration = 0.35 g

Example 1 (continued)

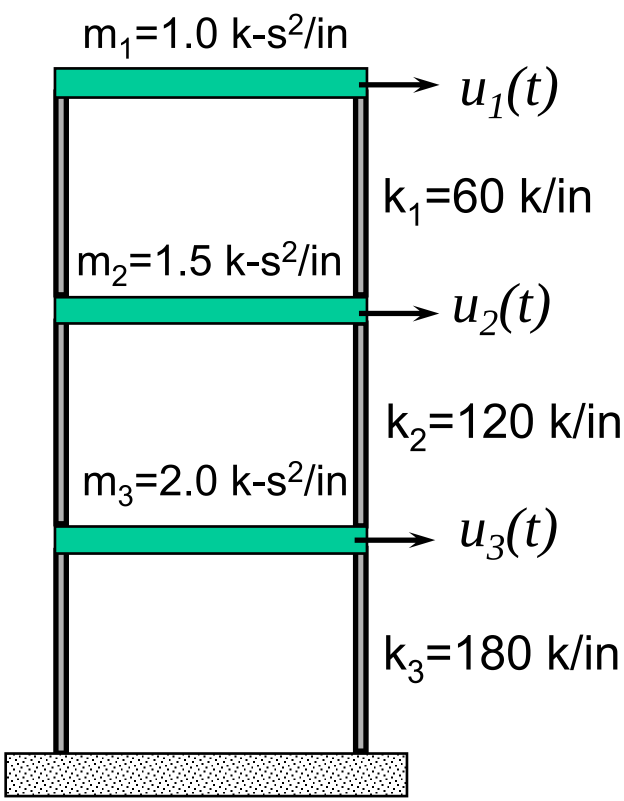


Form property matrices:

$$M = \begin{bmatrix} 1.0 & & \\ & 1.5 & \\ & & 2.0 \end{bmatrix} \text{kip} - \text{s}^2/\text{in}$$

$$K = \begin{bmatrix} 60 & -60 & 0 \\ -60 & 180 & -120 \\ 0 & -120 & 300 \end{bmatrix} \text{kip/in}$$

Example 1 (continued)



Solve eigenvalue problem:

$$K\Phi = M\Phi\Omega^2$$

$$\Omega^2 = \begin{bmatrix} 21.0 & & \\ & 96.6 & \\ & & 212.4 \end{bmatrix} \sec^{-2}$$

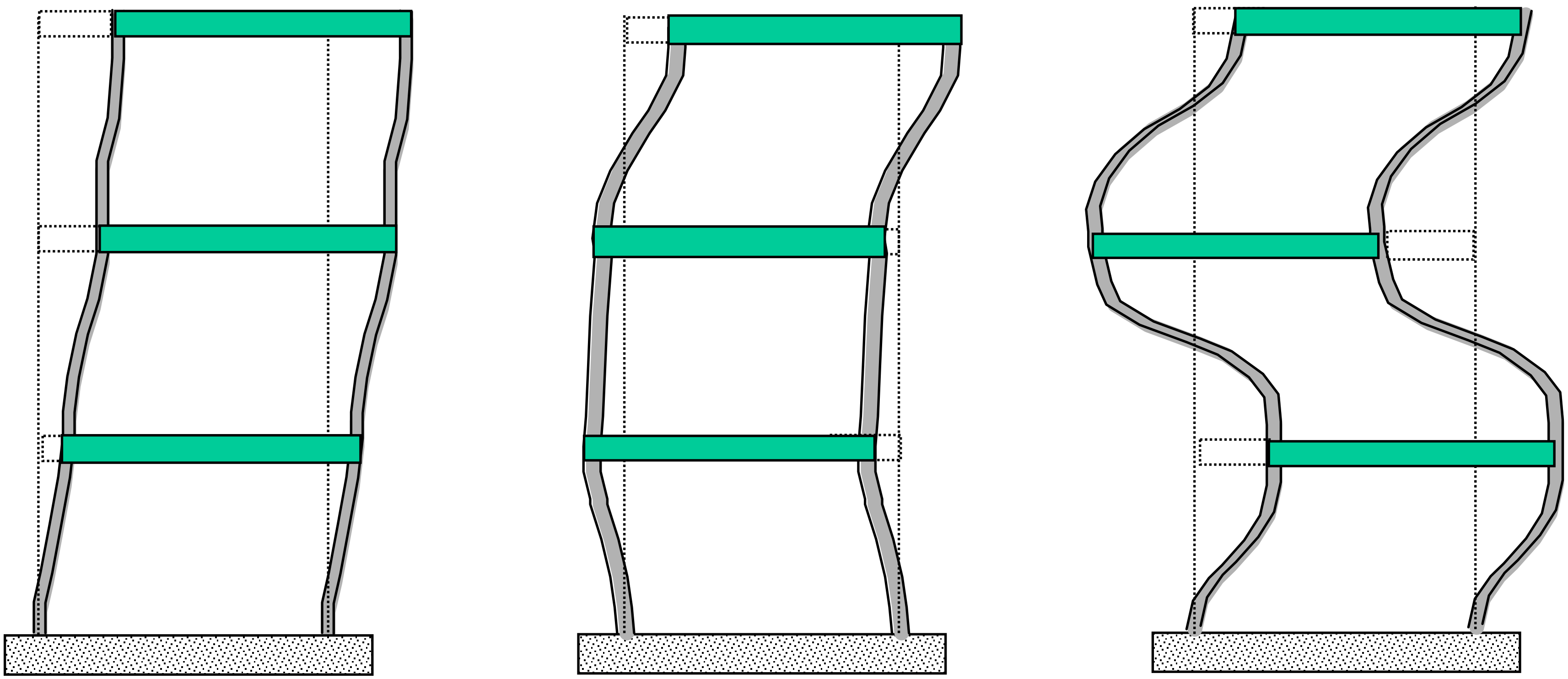
$$\Phi = \begin{bmatrix} 1.000 & 1.000 & 1.000 \\ 0.644 & -0.601 & -2.57 \\ 0.300 & -0.676 & 2.47 \end{bmatrix}$$

Normalization of Modes Using $\Phi^T M \Phi = I$

$$\Phi = \begin{bmatrix} 0.749 & 0.638 & 0.208 \\ 0.478 & -0.384 & -0.534 \\ 0.223 & -0.431 & 0.514 \end{bmatrix} \quad \text{vs} \quad \begin{bmatrix} 1.000 & 1.000 & 1.000 \\ 0.644 & -0.601 & -2.57 \\ 0.300 & -0.676 & 2.47 \end{bmatrix}$$

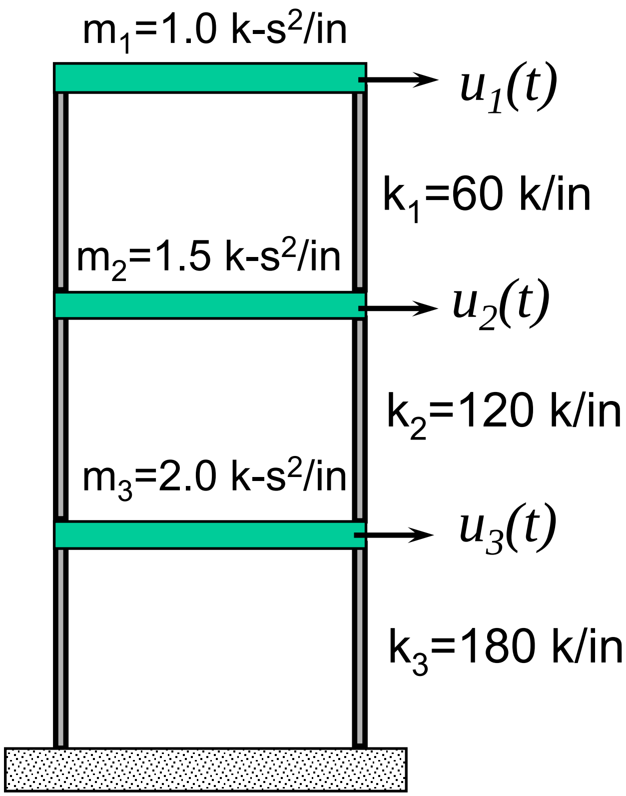
Example 1 (continued)

Mode Shapes and Periods of Vibration



MODE 1	MODE 2	MODE 3
ω = 4.58 rad/sec	ω = 9.83 rad/sec	ω = 14.57 rad/sec
T = 1.37 sec	T = 0.639 sec	T = 0.431 sec

Example 1 (continued)

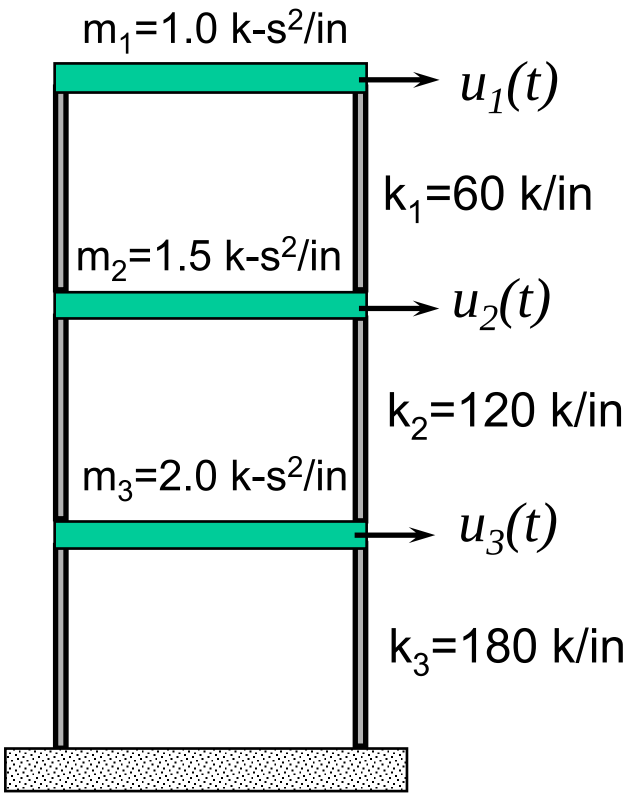


$$\omega_n = \begin{Bmatrix} 4.58 \\ 9.83 \\ 14.57 \end{Bmatrix} \text{rad/sec} \qquad T_n = \begin{Bmatrix} 1.37 \\ 0.639 \\ 0.431 \end{Bmatrix} \text{sec}$$

Compute Generalized Mass:

$$M^* = \Phi^T M \Phi = \begin{bmatrix} 1.801 & & \\ & 2.455 & \\ & & 23.10 \end{bmatrix} \text{kip} - \text{sec}^2/\text{in}$$

Example 1 (continued)



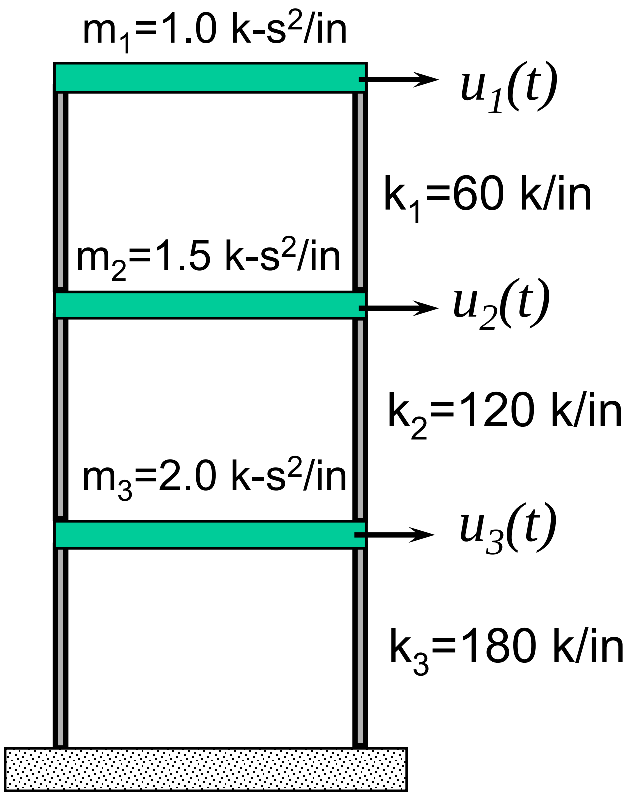
Compute generalized loading:

$$V^*(t) = -\Phi^T M R \ddot{v}_g(t)$$

$$V_n^* = -\begin{Bmatrix} 2.566 \\ -1.254 \\ 2.080 \end{Bmatrix} \ddot{v}_g(t)$$

Example 1 (continued)

Write uncoupled (modal) equations of motion:



$$\ddot{y}_1 + 2\xi_1\omega_1\dot{y}_1 + \omega_1^2 y_1 = V_1^*(t)/m_1^*$$

$$\ddot{y}_2 + 2\xi_2\omega_2\dot{y}_2 + \omega_2^2 y_2 = V_2^*(t)/m_2^*$$

$$\ddot{y}_3 + 2\xi_3\omega_3\dot{y}_3 + \omega_3^2 y_3 = V_3^*(t)/m_3^*$$

$$\ddot{y}_1 + 0.458\dot{y}_1 + 21.0 y_1 = -1.425\ddot{v}_g(t)$$

$$\ddot{y}_2 + 0.983\dot{y}_2 + 96.6 y_2 = 0.511\ddot{v}_g(t)$$

$$\ddot{y}_3 + 1.457\dot{y}_3 + 212.4 y_3 = -0.090\ddot{v}_g(t)$$

Modal Participation Factors

Mode	1	1.425	1.911
Mode	2	–0.511	–0.799
Mode	3	0.090	0.435

Modal scaling $\phi_{i,1} = 1.0$ $\phi_i^T M \phi_i = 1.0$

Modal Participation Factors (continued)

$$1.425\begin{Bmatrix}1.000\\0.644\\0.300\end{Bmatrix} = 1.911\begin{Bmatrix}0.744\\0.480\\0.223\end{Bmatrix}$$

using $\phi_{1,1} = 1$ using $\phi_1{}^T M\phi_1 = 1$

Effective Modal Mass

$$\bar{M}_n = P_n^{\,2} m_n^*$$

	$\bar{M}_n$	%	Accum%
Mode 1	3.66	81	81
Mode 2	0.64	14	95
Mode 3	0.20	5	100%
	4.50	100%	

Example 1 (continued)

Solving modal equation via NONLIN:

For Mode 1:

$$\ddot{y}_1 + 2\xi_1\omega_1\dot{y}_1 + \omega_1^2 y_1 = V_1^*(t)/m_1^*$$

$$1.00\ddot{y}_1 + 0.458\dot{y}_1 + 21.0y_1 = -1.425\ddot{v}_g(t)$$

M = 1.00 kip-sec^2/in

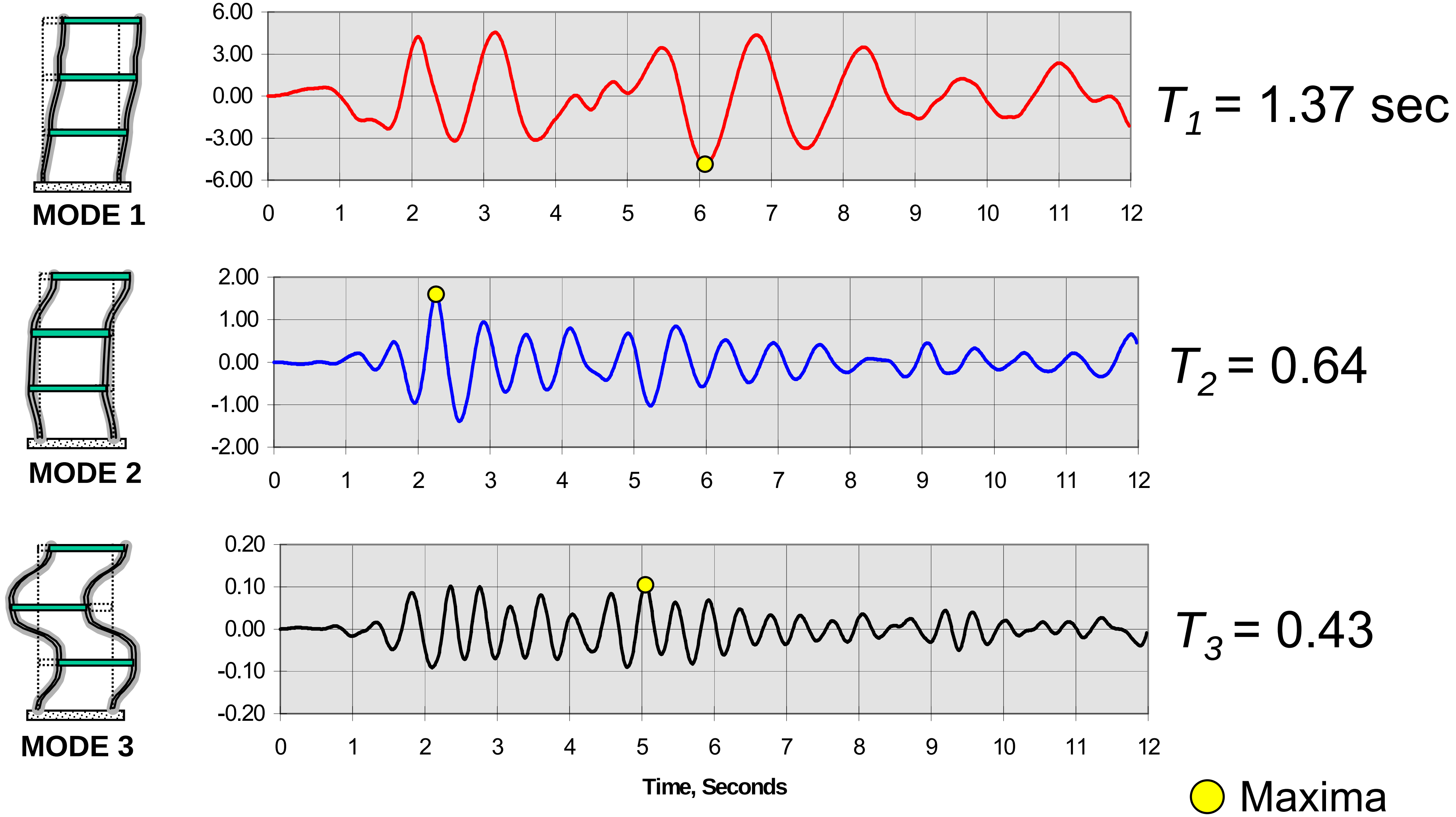
C = 0.458 kip-sec/in

K_1 = 21.0 kips/inch

Scale ground acceleration by factor 1.425

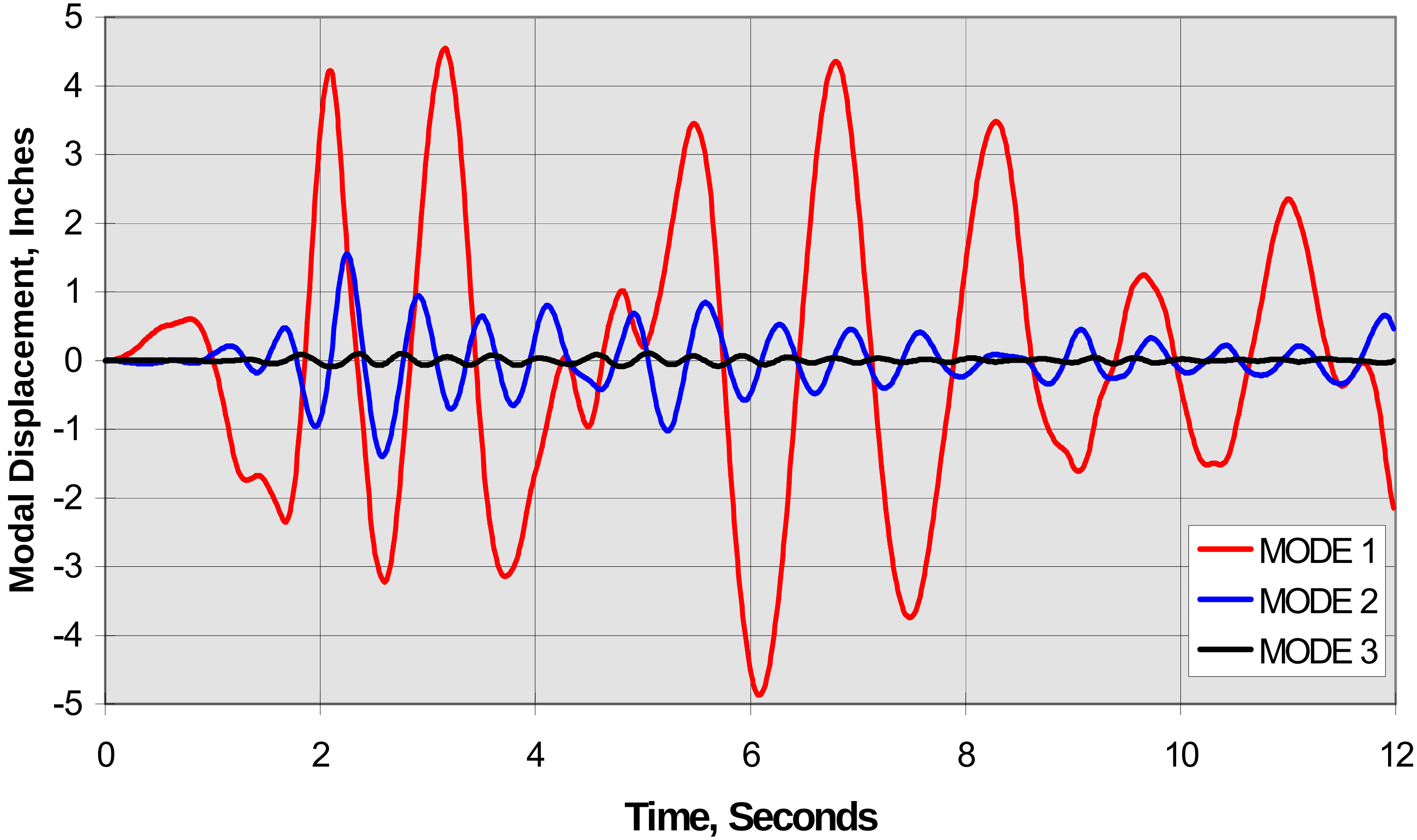
Example 1 (continued)

Modal Displacement Response Histories (from NONLIN)



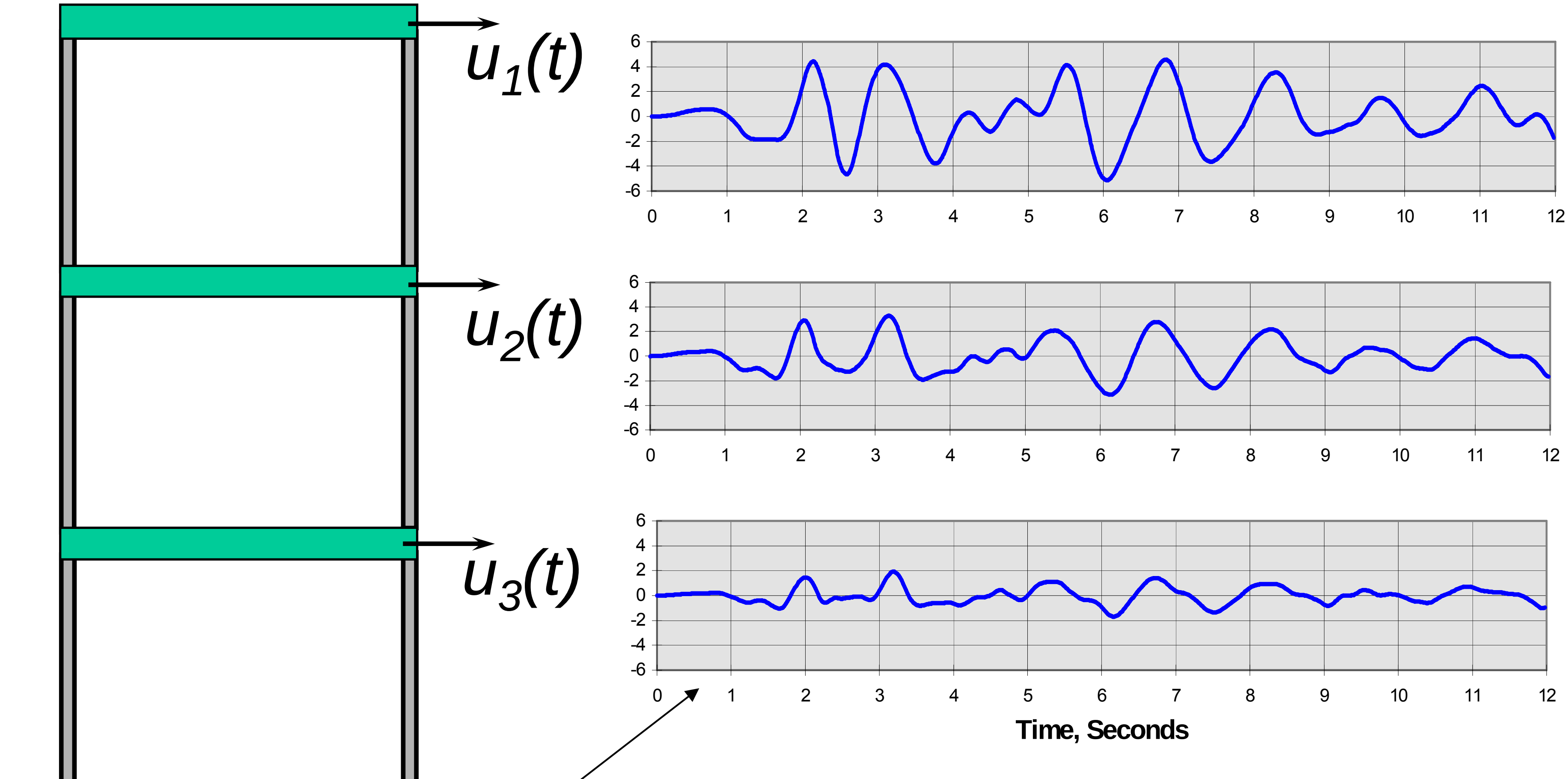
Example 1 (continued)

Modal Response Histories:



Example 1 (continued)

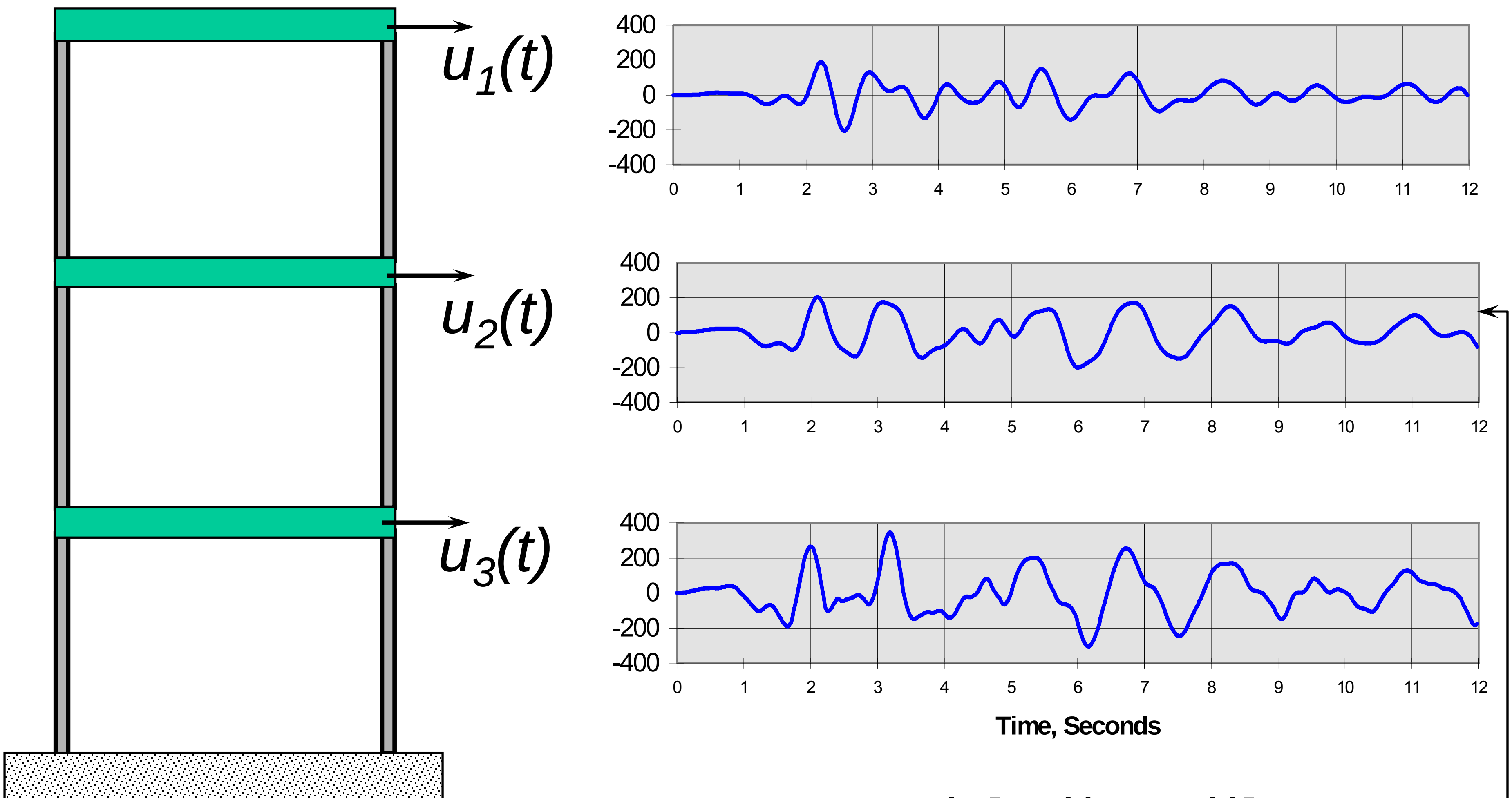
Compute story displacement response histories: $u(t) = \Phi y(t)$



= 0.300 x Mode 1 - 0.676 x Mode 2 + 2.47 x Mode 3

Example 1 (continued)

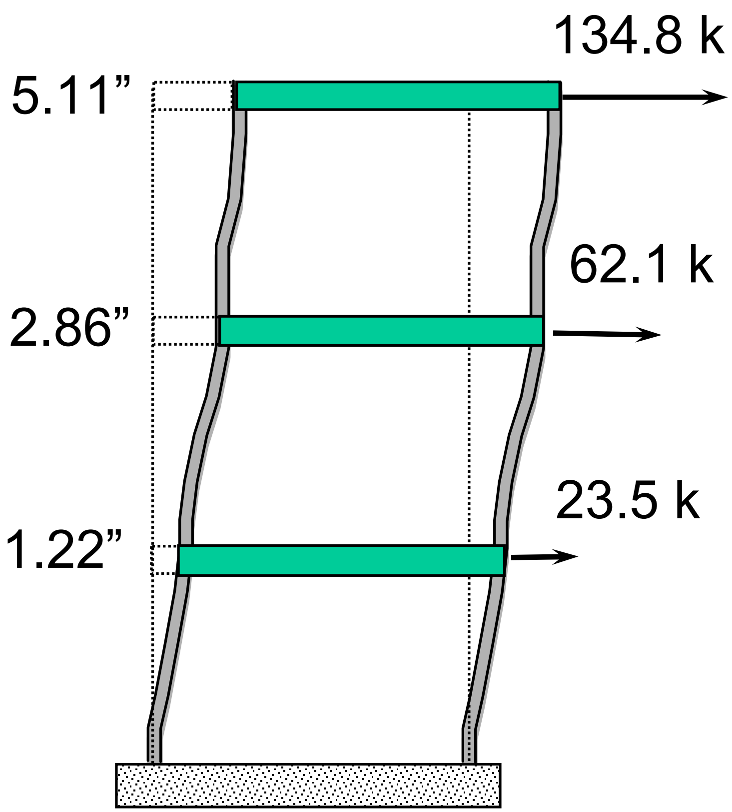
Compute story shear response histories:

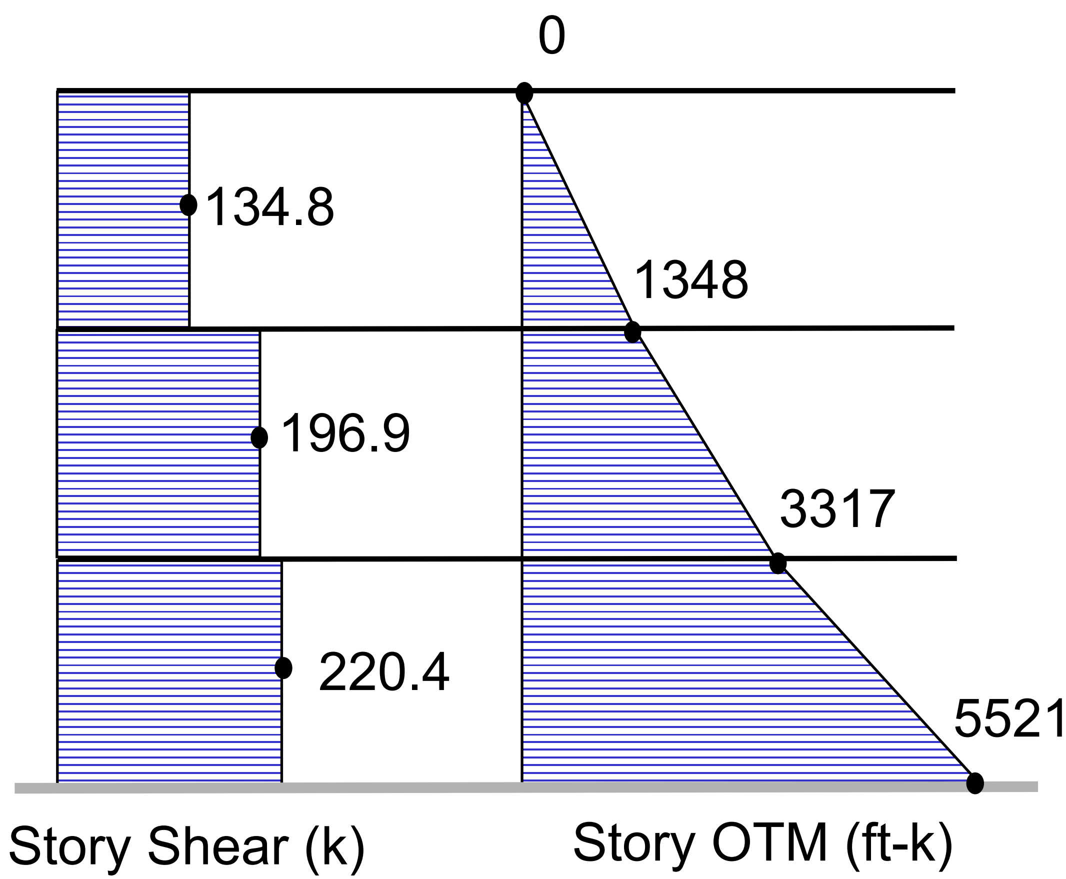


$$= k_2[u_2(t) - u_3(t)]$$

Example 1 (continued)

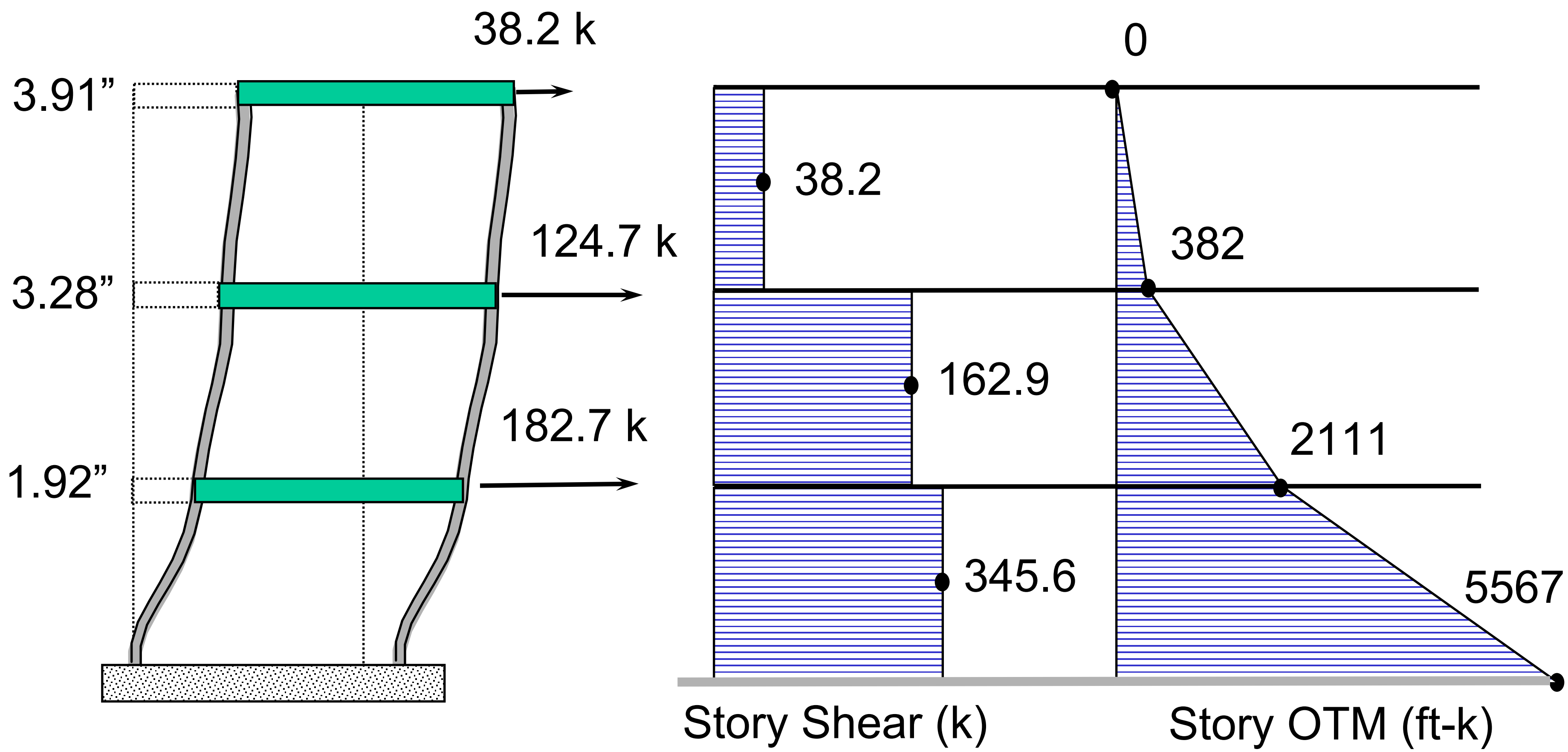
Displacements and forces at time of maximum displacements
(t = 6.04 sec)





Example 1 (continued)

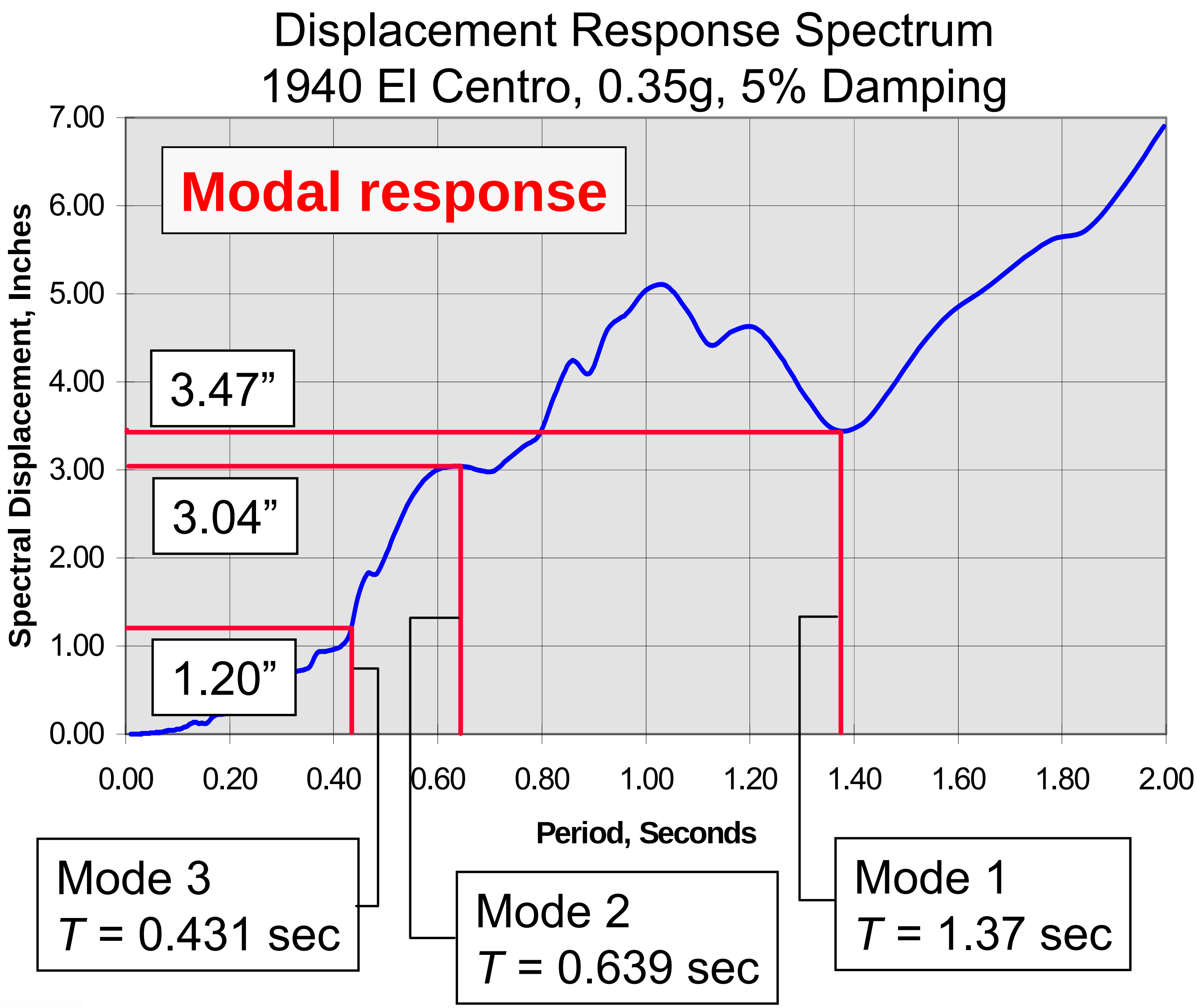
Displacements and forces at time of maximum shear
(t = 3.18 sec)



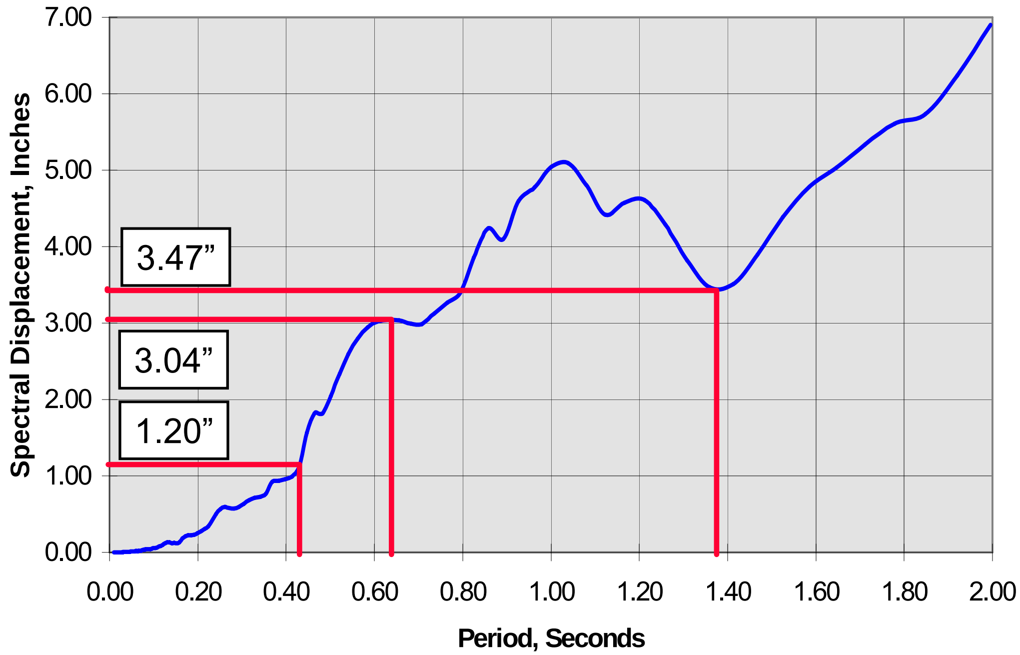
Modal Response Response Spectrum Method

- Instead of solving the time history problem for each mode, use a response spectrum to compute the **maximum** response in each mode.
- These maxima are generally **nonconcurrent**.
- Combine the maximum modal responses using some statistical technique, such as square root of the sum of the squares (SRSS) or complete quadratic combination (CQC).
- The technique is **approximate**.
- It is the basis for the equivalent lateral force (ELF) method.

Example 1 (Response Spectrum Method)



Example 1 (continued)



Modal Equations of Motion

$$\ddot{y}_1 + 0.458\dot{y}_1 + 21.0y_1 = \underline{-1.42}5\ddot{v}_g(t)$$

$$\ddot{y}_2 + 0.983\dot{y}_2 + 96.6y_2 = 0.\underline{511}\ddot{v}_g(t)$$

$$\ddot{y}_3 + 1.457\dot{y}_3 + 212.4y_3 = \underline{-0.09}0\ddot{v}_g(t)$$

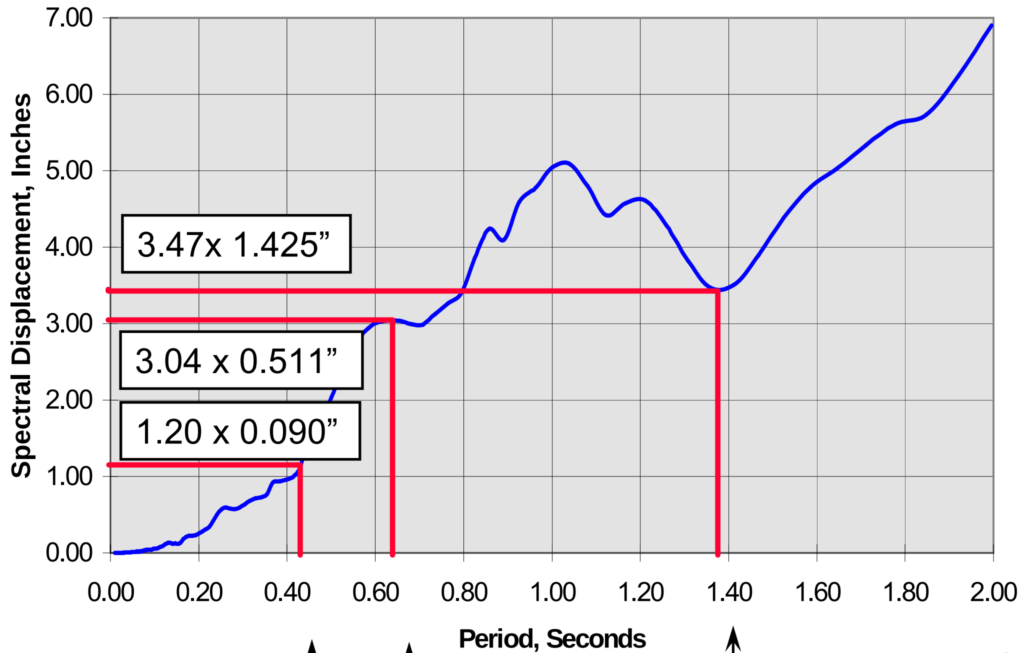
Modal Maxima

$$\bar{y}_1 = 1.\underline{425} * 3.47 = 4.94''$$

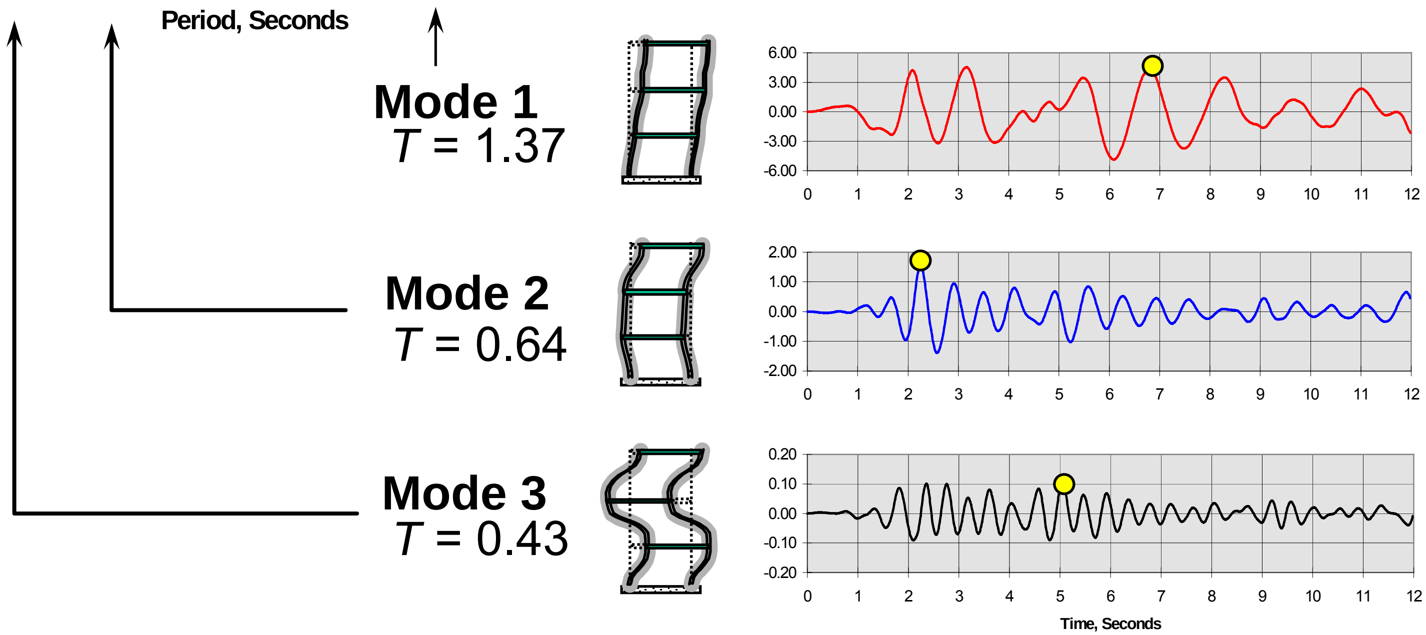
$$\bar{y}_2 = 0.\underline{511} * 3.04 = 1.55''$$

$$\bar{y}_3 = 0.\underline{090} * 1.20 = 0.108''$$

Example 1 (continued)



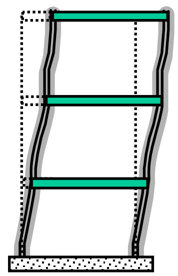
The **scaled** response spectrum values give the same **modal maxima** as the previous time Histories.



Example 1 (continued)

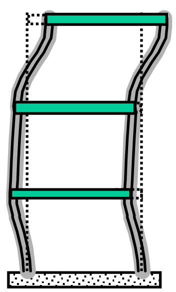
Computing **Nonconcurrent** Story Displacements

Mode 1



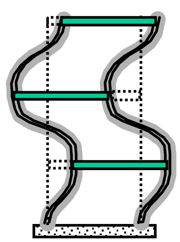
$$\begin{Bmatrix} 1.000 \\ 0.644 \\ 0.300 \end{Bmatrix} 4.940 = \begin{Bmatrix} 4.940 \\ 3.181 \\ 1.482 \end{Bmatrix}$$

Mode 2



$$\begin{Bmatrix} 1.000 \\ -0.601 \\ -0.676 \end{Bmatrix} 1.550 = \begin{Bmatrix} 1.550 \\ -0.931 \\ -1.048 \end{Bmatrix}$$

Mode 3



$$\begin{Bmatrix} 1.000 \\ -2.570 \\ 2.470 \end{Bmatrix} 0.108 = \begin{Bmatrix} 0.108 \\ -0.278 \\ 0.267 \end{Bmatrix}$$

Example 1 (continued)

Modal Combination Techniques (for Displacement)

Sum of Absolute Values:

$$\begin{Bmatrix} 4.940 + 1.550 + 0.108 \\ 3.181 + 0.931 + 0.278 \\ 1.482 + 1.048 + 0.267 \end{Bmatrix} = \begin{Bmatrix} 6.60 \\ 4.39 \\ 2.80 \end{Bmatrix}$$

At time of maximum displacement

$$\text{“Exact”} \quad \begin{Bmatrix} 5.15 \\ 2.86 \\ 1.22 \end{Bmatrix}$$

Square Root of the Sum of the Squares:

$$\begin{Bmatrix} \sqrt{4.940^2 + 1.550^2 + 0.108^2} \\ \sqrt{3.181^2 + 0.931^2 + 0.278^2} \\ \sqrt{1.482^2 + 1.048^2 + 0.267^2} \end{Bmatrix} = \begin{Bmatrix} 5.18 \\ 3.33 \\ 1.84 \end{Bmatrix}$$

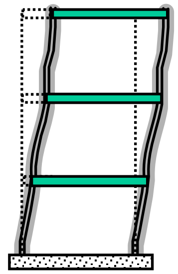
Envelope of story displacement

$$\begin{Bmatrix} 5.15 \\ 3.18 \\ 1.93 \end{Bmatrix}$$

Example 1 (continued)

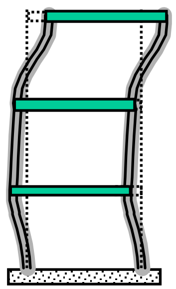
Computing Interstory Drifts

Mode 1



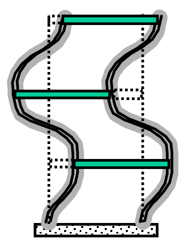
$$\begin{Bmatrix} 4.940-3.181 \\ 3.181-1.482 \\ 1.482-0 \end{Bmatrix} = \begin{Bmatrix} 1.759 \\ 1.699 \\ 1.482 \end{Bmatrix}$$

Mode 2



$$\begin{Bmatrix} 1.550-(-0.931) \\ -0.931-(-1.048) \\ -1.048-0 \end{Bmatrix} = \begin{Bmatrix} 2.481 \\ 0.117 \\ -1.048 \end{Bmatrix}$$

Mode 3

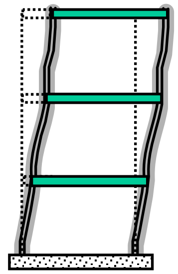


$$\begin{Bmatrix} 0.108-(-0.278) \\ -0.278-0.267 \\ 0.267-0 \end{Bmatrix} = \begin{Bmatrix} 0.386 \\ -0.545 \\ 0.267 \end{Bmatrix}$$

Example 1 (continued)

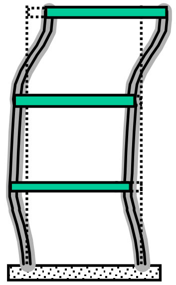
Computing Interstory Shears (Using Drift)

Mode 1



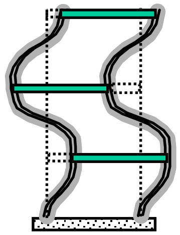
$$\begin{Bmatrix} 1.759(60) \\ 1.699(120) \\ 1.482(180) \end{Bmatrix} = \begin{Bmatrix} 105.5 \\ 203.9 \\ 266.8 \end{Bmatrix}$$

Mode 2



$$\begin{Bmatrix} 2.481(60) \\ 0.117(120) \\ -1.048(180) \end{Bmatrix} = \begin{Bmatrix} 148.9 \\ 14.0 \\ -188.6 \end{Bmatrix}$$

Mode 3



$$\begin{Bmatrix} 0.386(60) \\ -0.545(120) \\ 0.267(180) \end{Bmatrix} = \begin{Bmatrix} 23.2 \\ -65.4 \\ 48.1 \end{Bmatrix}$$

Example 1 (continued)

Computing Interstory Shears: SRSS Combination

$$\begin{Bmatrix} \sqrt{106^2 + 149^2 + 23.2^2} \\ \sqrt{204^2 + 14^2 + 65.4^2} \\ \sqrt{267^2 + 189^2 + 48.1^2} \end{Bmatrix} = \begin{Bmatrix} 220 \\ 215 \\ 331 \end{Bmatrix}$$

"Exact"	"Exact"	"Exact"
$\begin{Bmatrix} 38.2 \\ 163 \\ 346 \end{Bmatrix}$	$\begin{Bmatrix} 135 \\ 197 \\ 220 \end{Bmatrix}$	$\begin{Bmatrix} 207 \\ 203 \\ 346 \end{Bmatrix}$
At time of max. shear	At time of max. displacement	Envelope = maximum per story

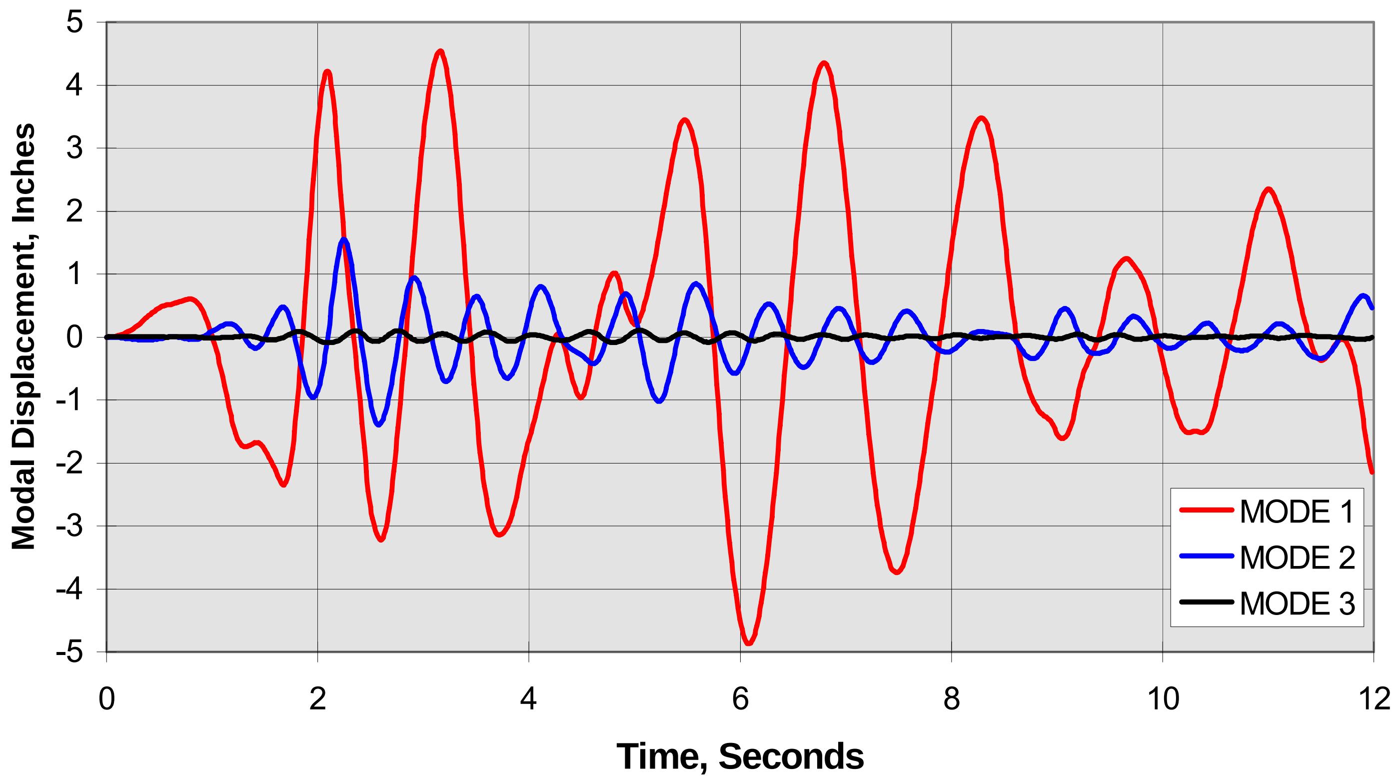
Caution:

Do NOT compute story shears from the story drifts derived from the SRSS of the story displacements.

Calculate the story shears in each mode (using modal drifts) and then SRSS the results.

Using Less than Full (Possible) Number of Natural Modes

Modal Response Histories:



Using Less than Full Number of Natural Modes

Time-History for **Mode** 1

$$y(t)=\begin{bmatrix} \boxed{y_1(t1) & y_1(t2) & y_1(t3) & y_1(t4) & y_1(t5) & y_1(t6) & y_1(t7) & y_1(t8) & .... & y_1(tn)} \\ y_2(t1) & y_2(t2) & y_2(t3) & y_2(t4) & y_2(t5) & y_2(t6) & y_2(t7) & y_2(t8) & .... & y_2(tn) \\ y_3(t1) & y_3(t2) & y_3(t3) & y_3(t4) & y_3(t5) & y_3(t6) & y_3(t7) & y_3(t8) & .... & y_3(tn) \end{bmatrix}$$

Transformation:

$$u(t)=\begin{bmatrix}\phi_1 & \phi_2 & \phi_3\end{bmatrix}y(t)$$

3 x nt | 3 x 3 | 3 x nt

3 x nt

Time History for **DOF** 1

$$u(t)=\begin{bmatrix} u_1(t1) & u_1(t2) & u_1(t3) & u_1(t4) & u_1(t5) & u_1(t6) & u_1(t7) & u_1(t8) & .... & u_1(tn) \\ u_2(t1) & u_2(t2) & u_2(t3) & u_2(t4) & u_2(t5) & u_2(t6) & u_2(t7) & u_2(t8) & .... & u_2(tn) \\ u_3(t1) & u_3(t2) & u_3(t3) & u_3(t4) & u_3(t5) & u_3(t6) & u_3(t7) & u_3(t8) & .... & u_3(tn) \end{bmatrix}$$

Using Less than Full Number of Natural Modes

Time History for **Mode** 1

$$y(t)=\begin{bmatrix} y_1(t1) & y_1(t2) & y_1(t3) & y_1(t4) & y_1(t5) & y_1(t6) & y_1(t7) & y_1(t8) & .... & y_1(tn) \\ y_2(t1) & y_2(t2) & y_2(t3) & y_2(t4) & y_2(t5) & y_2(t6) & y_2(t7) & y_2(t8) & .... & y_2(tn) \end{bmatrix}$$

NOTE: Mode 3 **NOT** Analyzed

Transformation:

$$u(t)=\begin{bmatrix}\phi_1 & \phi_2\end{bmatrix} y(t)$$

3 x nt   3 x 2   2 x nt

$$\underbrace{\quad 3 \times 2 \qquad 2 \times nt \quad}_{3 \times nt}$$

Time history for **DOF** 1

$$u(t)=\begin{bmatrix} u_1(t1) & u_1(t2) & u_1(t3) & u_1(t4) & u_1(t5) & u_1(t6) & u_1(t7) & u_1(t8) & .... & u_1(tn) \\ u_2(t1) & u_2(t2) & u_2(t3) & u_2(t4) & u_2(t5) & u_2(t6) & u_2(t7) & u_2(t8) & .... & u_2(tn) \\ u_3(t1) & u_3(t2) & u_3(t3) & u_3(t4) & u_3(t5) & u_3(t6) & u_3(t7) & u_3(t8) & .... & u_3(tn) \end{bmatrix}$$

Using Less than Full Number of Natural Modes

(Modal Response Spectrum Technique)

Sum of absolute values:

$$\begin{Bmatrix} 4.940+1.550+0.108 \\ 3.181+0.931+0.278 \\ 1.482+1.048+0.267 \end{Bmatrix} = \begin{Bmatrix} 6.60 \\ 4.39 \\ 2.80 \end{Bmatrix} \qquad \begin{Bmatrix} 6.49 \\ 4.112 \\ 2.53 \end{Bmatrix}$$

At time of maximum displacement

Square root of the sum of the squares:

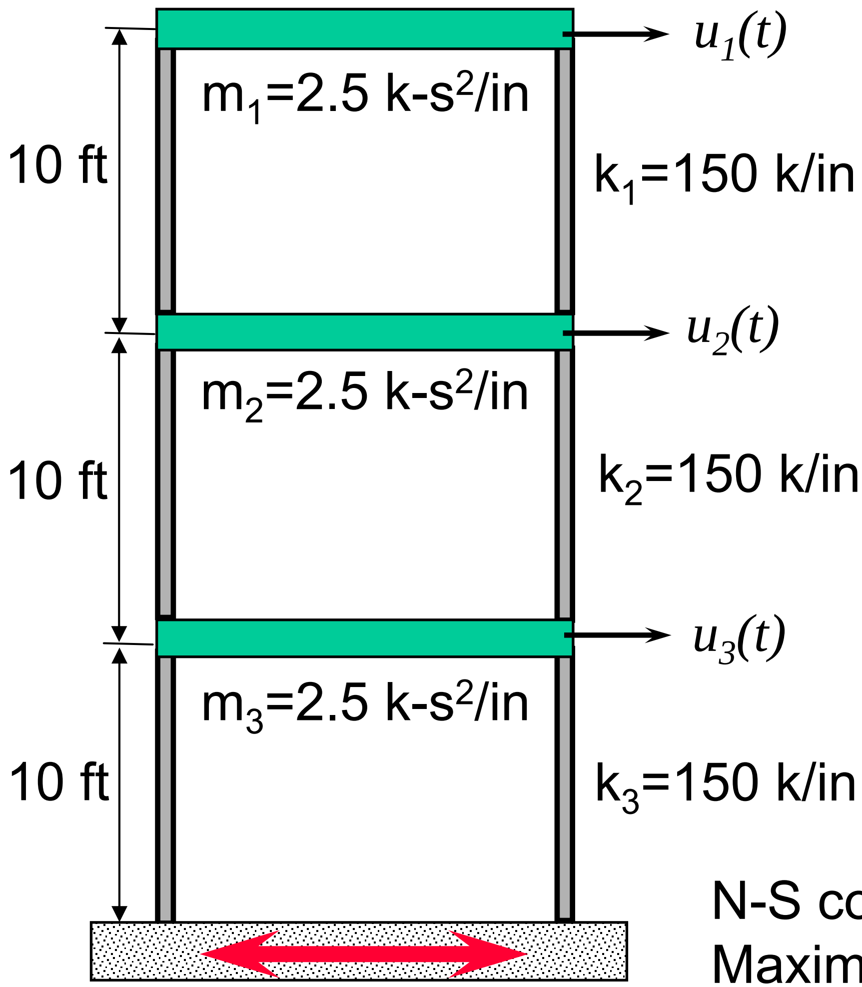
$$\begin{Bmatrix} \sqrt{4.940^2+1.550^2+0.108^2} \\ \sqrt{3.181^2+0.931^2+0.278^2} \\ \sqrt{1.482^2+1.048^2+0.267^2} \end{Bmatrix} = \begin{Bmatrix} 5.18 \\ 3.33 \\ 1.84 \end{Bmatrix} \qquad \begin{Bmatrix} 5.18 \\ 3.31 \\ 1.82 \end{Bmatrix}$$

3 modes 2 modes

“Exact”:

$$\begin{Bmatrix} 5.15 \\ 2.86 \\ 1.22 \end{Bmatrix}$$

Example of MDOF Response of Structure Responding to 1940 El Centro Earthquake

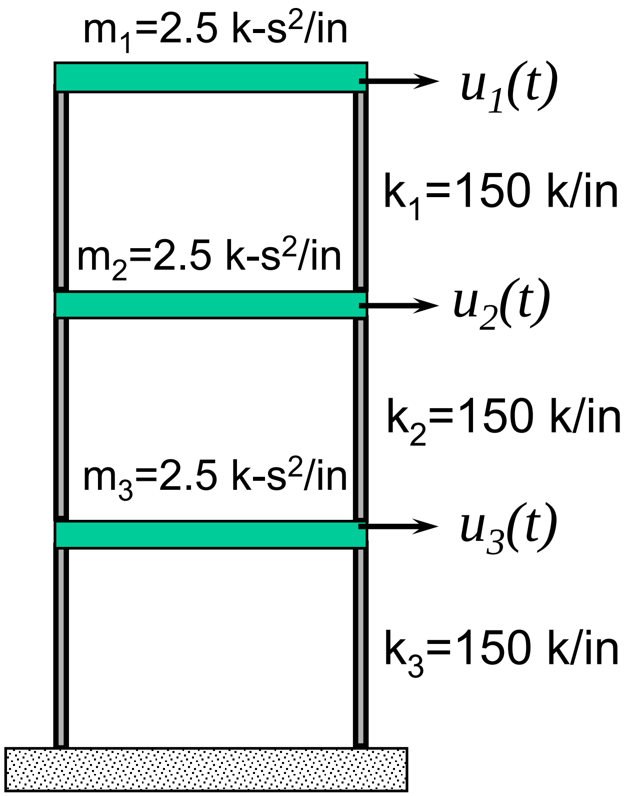


Example 2

Assume Wilson damping with 5% critical in each mode.

N-S component of 1940 El Centro earthquake
Maximum acceleration = 0.35 g

Example 2 (continued)

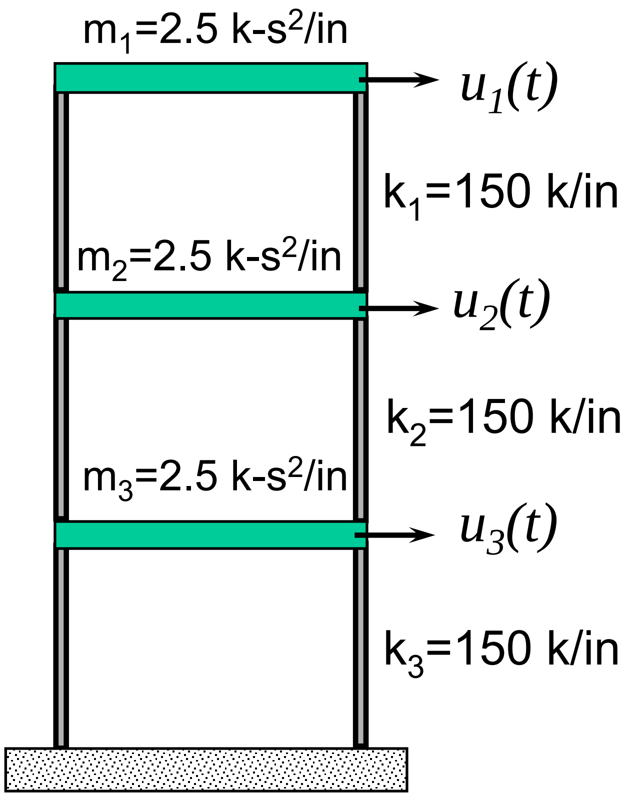


Form property matrices:

$$M = \begin{bmatrix} 2.5 & & \\ & 2.5 & \\ & & 2.5 \end{bmatrix} \text{kip} - \text{s}^2/\text{in}$$

$$K = \begin{bmatrix} 150 & -150 & 0 \\ -150 & 300 & -150 \\ 0 & -150 & 300 \end{bmatrix} \text{kip/in}$$

Example 2 (continued)



Solve = eigenvalue problem:

$$K\Phi = M\Phi\Omega^2$$

$$\Omega^2 = \begin{bmatrix} 11.9 & & \\ & 93.3 & \\ & & 194.8 \end{bmatrix} \sec^{-2}$$

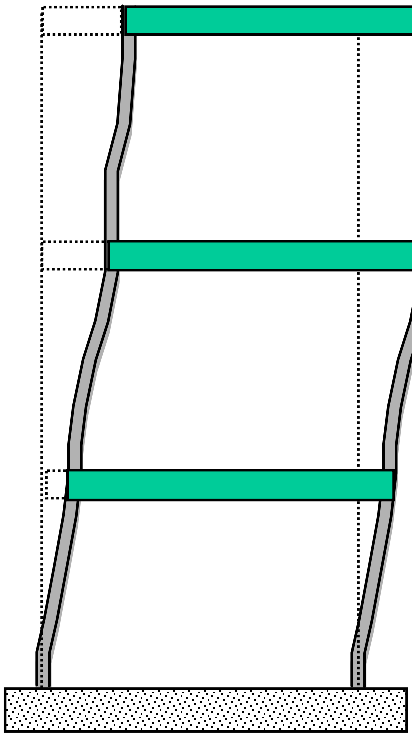
$$\Phi = \begin{bmatrix} 1.000 & 1.000 & 1.000 \\ 0.802 & -0.555 & -2.247 \\ 0.445 & -1.247 & 1.802 \end{bmatrix}$$

Normalization of Modes Using $\Phi^T M \Phi = I$

$$\Phi = \begin{bmatrix} 0.466 & 0.373 & 0.207 \\ 0.373 & -0.207 & -0.465 \\ 0.207 & -0.465 & 0.373 \end{bmatrix} \text{ vs } \begin{bmatrix} 1.000 & 1.000 & 1.000 \\ 0.802 & -0.555 & -2.247 \\ 0.445 & -1.247 & 1.802 \end{bmatrix}$$

Example 2 (continued)

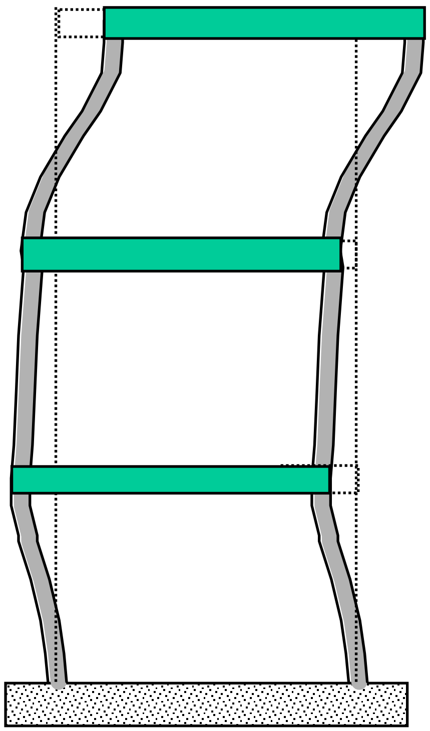
Mode Shapes and Periods of Vibration



Mode 1

ω = 3.44 rad/sec

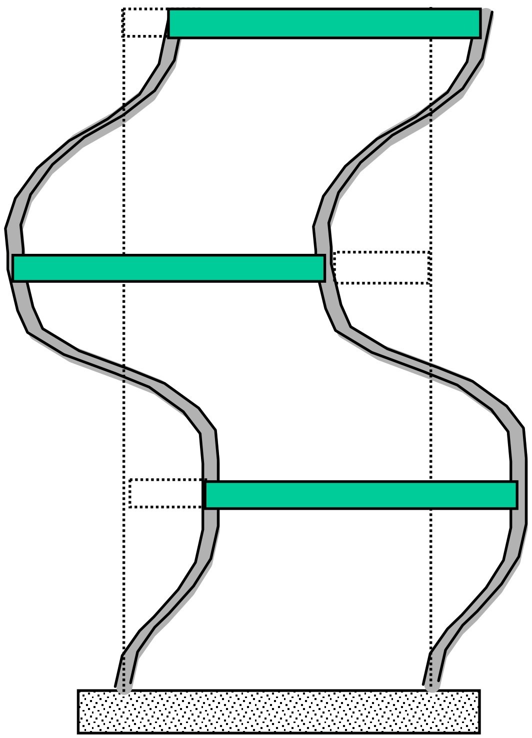
T = 1.82 sec



Mode 2

ω = 9.66 rad/sec

T = 0.65 sec

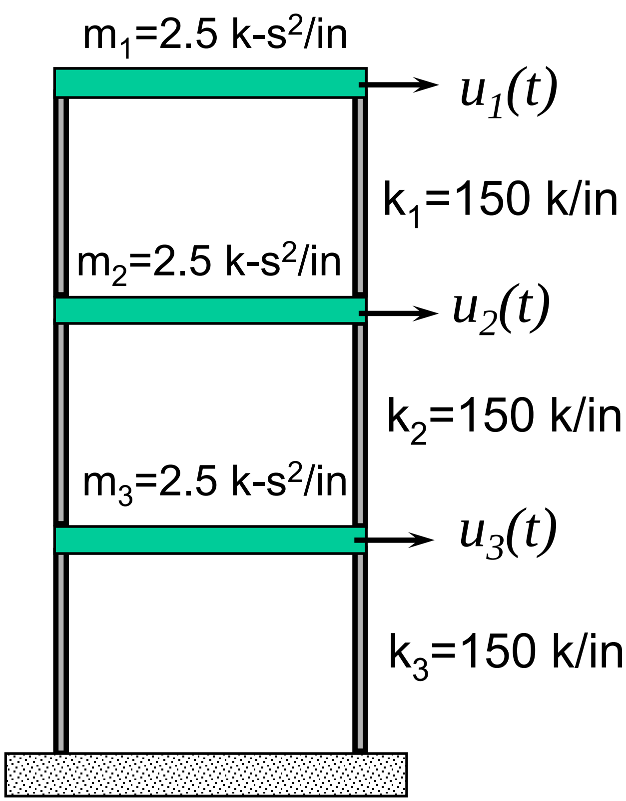


Mode 3

ω = 13.96 rad/sec

T = 0.45 sec

Example 2 (continued)

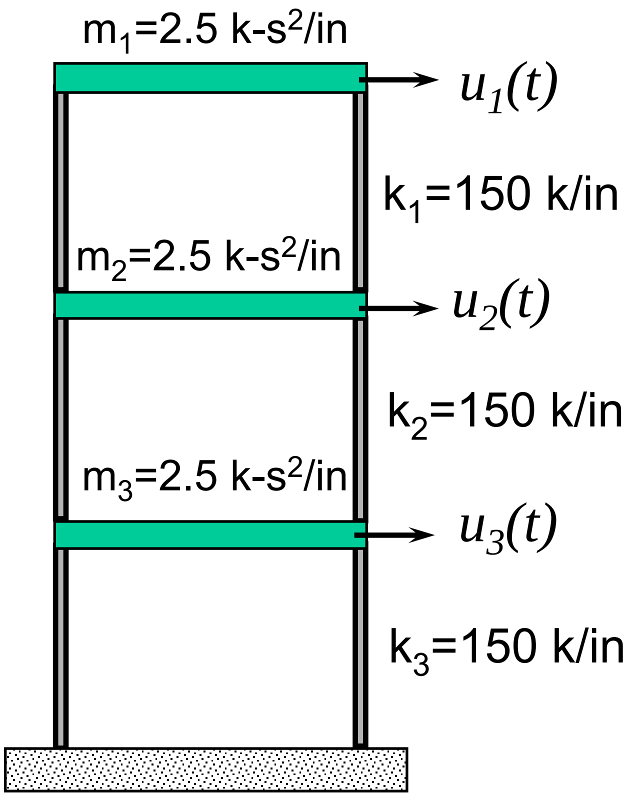


$$\omega_n = \begin{Bmatrix} 3.44 \\ 9.66 \\ 13.96 \end{Bmatrix} \text{rad/sec} \qquad T_n = \begin{Bmatrix} 1.82 \\ 0.65 \\ 0.45 \end{Bmatrix} \text{sec}$$

Compute generalized mass:

$$M^* = \Phi^T M \Phi = \begin{bmatrix} 4.603 & & \\ & 7.158 & \\ & & 23.241 \end{bmatrix} \text{kip} - \text{sec}^2/\text{in}$$

Example 2 (continued)



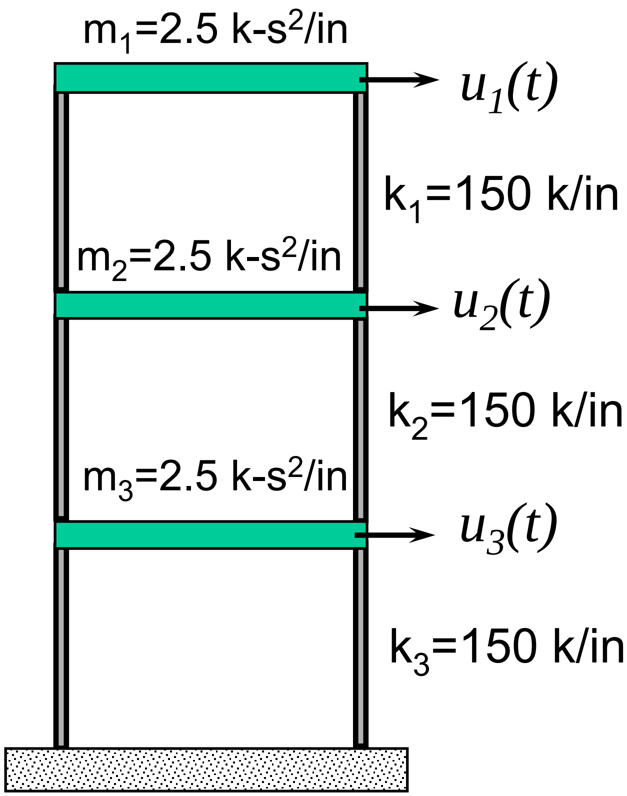
Compute generalized loading:

$$V^*(t) = -\Phi^T M R \ddot{v}_g(t)$$

$$V_n^* = \begin{Bmatrix} -5.617 \\ 2.005 \\ -1.388 \end{Bmatrix} \ddot{v}_g(t)$$

Example 2 (continued)

Write uncoupled (modal) equations of motion:



$$\ddot{y}_1 + 2\xi_1\omega_1\dot{y}_1 + \omega_1^2 y_1 = V_1^*(t)/m_1^*$$

$$\ddot{y}_2 + 2\xi_2\omega_2\dot{y}_2 + \omega_2^2 y_2 = V_2^*(t)/m_2^*$$

$$\ddot{y}_3 + 2\xi_3\omega_3\dot{y}_3 + \omega_3^2 y_3 = V_3^*(t)/m_3^*$$

$$\ddot{y}_1 + 0.345\dot{y}_1 + 11.88 y_1 = -1.22\ddot{v}_g(t)$$

$$\ddot{y}_2 + 0.966\dot{y}_2 + 93.29 y_2 = 0.280\ddot{v}_g(t)$$

$$\ddot{y}_3 + 1.395\dot{y}_3 + 194.83 y_3 = -0.06\ddot{v}_g(t)$$

Modal Participation Factors

$$\begin{array}{ll} \textit{Mode} & 1 \\ \textit{Mode} & 2 \\ \textit{Mode} & 3 \end{array} \boxed{\begin{array}{c} -1.22 \\ 0.28 \\ -0.060 \end{array}} \boxed{\begin{array}{c} -2.615 \\ 0.748 \\ -0.287 \end{array}}$$

Modal scaling $\phi_{i,1} = 1.0$ $\phi_i^T M \phi_i = 1.0$

Effective Modal Mass

$$\overline{M}_n = P_n^{\ 2} m_n$$

	$\overline{M}_n$	%	Accum%
Mode 1	6.856	91.40	91.40
Mode 2	0.562	7.50	98.90
Mode 3	0.083	1.10	100.0
	7.50	100%	

Example 2 (continued)

Solving modal equation via NONLIN:

For Mode 1:

$$\ddot{y}_1 + 2\xi_1\omega_1\dot{y}_1 + \omega_1^2 y_1 = V_1^*(t)/m_1^*$$

$$1.00\ddot{y}_1 + 0.345\dot{y}_1 + 11.88y_1 = -1.22\ddot{v}_g(t)$$

M = 1.00 kip-sec^2/in

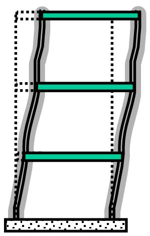
C = 0.345 kip-sec/in

K_1 = 11.88 kips/inch

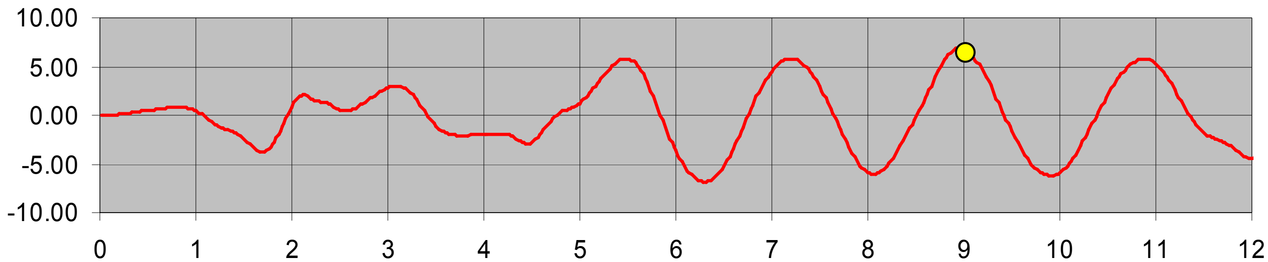
Scale ground acceleration by factor 1.22

Example 2 (continued)

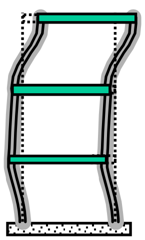
Modal Displacement Response Histories (from NONLIN)



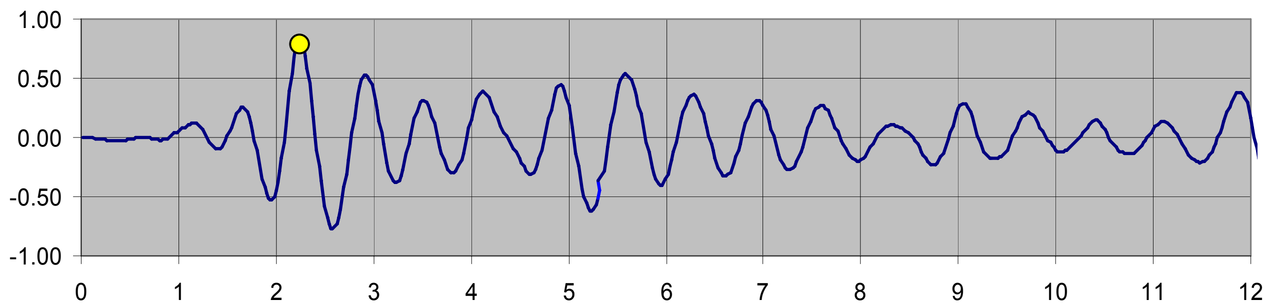
Mode 1



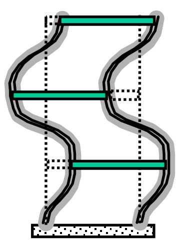
T=1.82



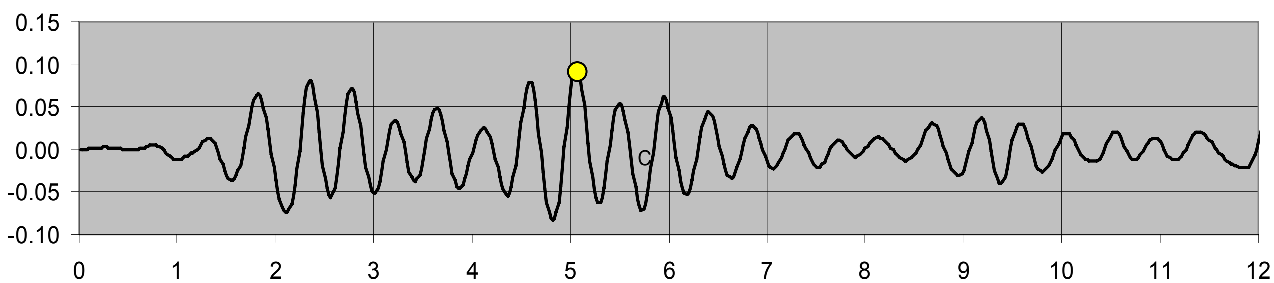
Mode 2



T=0.65



Mode 3

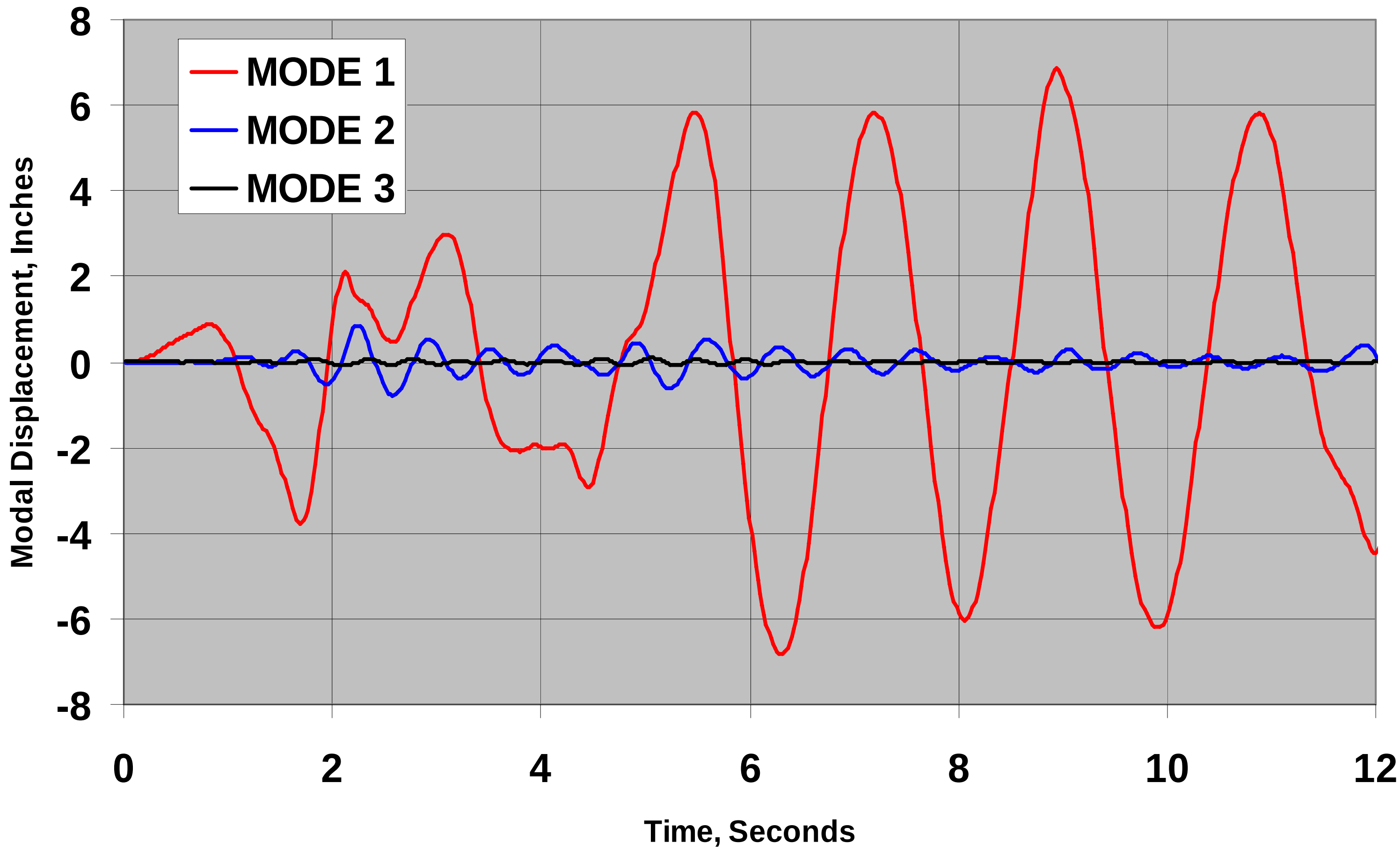


T=0.45

Maxima

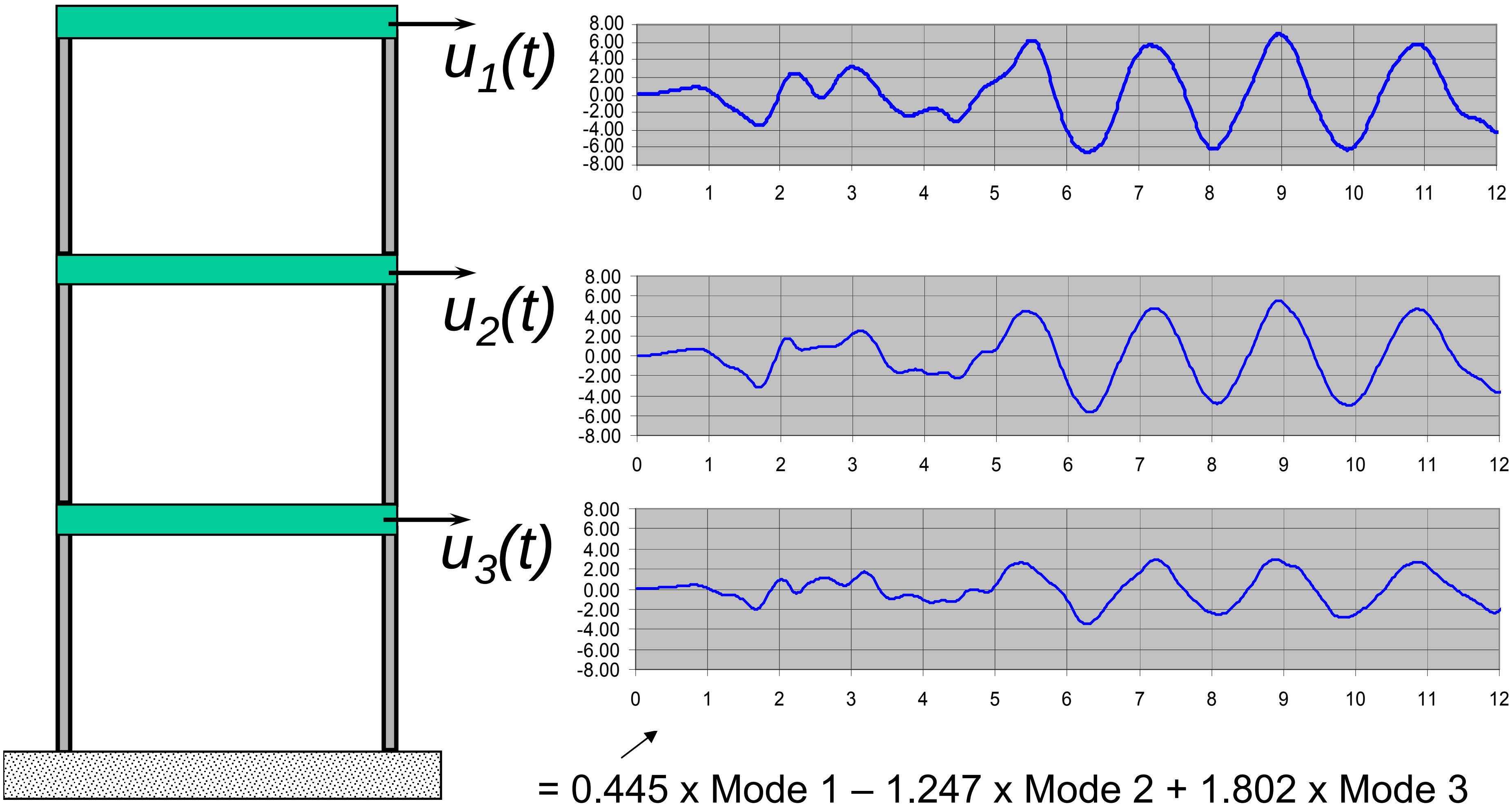
Example 2 (continued)

Modal Response Histories



Example 2 (continued)

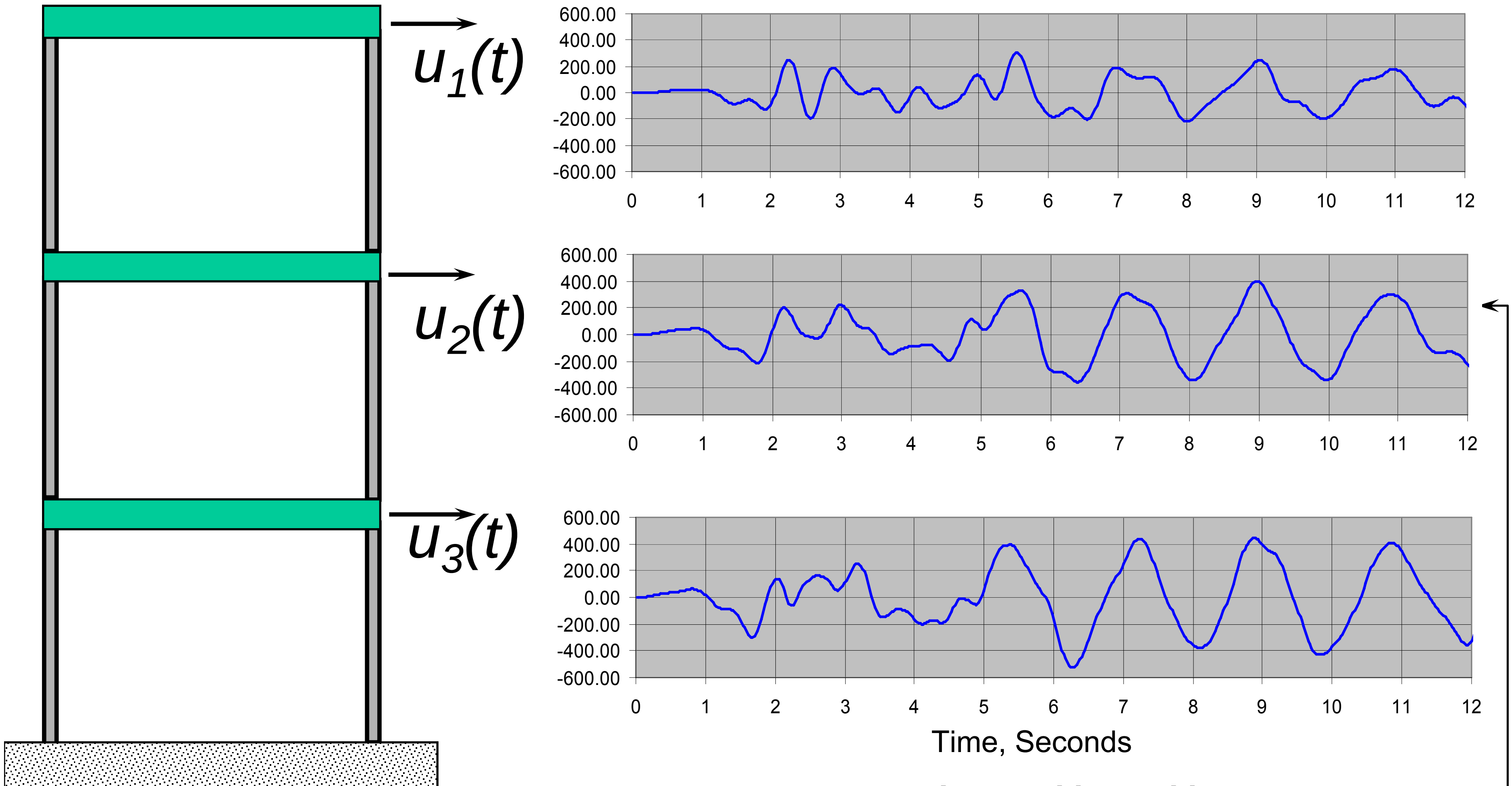
Compute story displacement response histories: $u(t) = \Phi y(t)$



= 0.445 x Mode 1 – 1.247 x Mode 2 + 1.802 x Mode 3

Example 2 (continued)

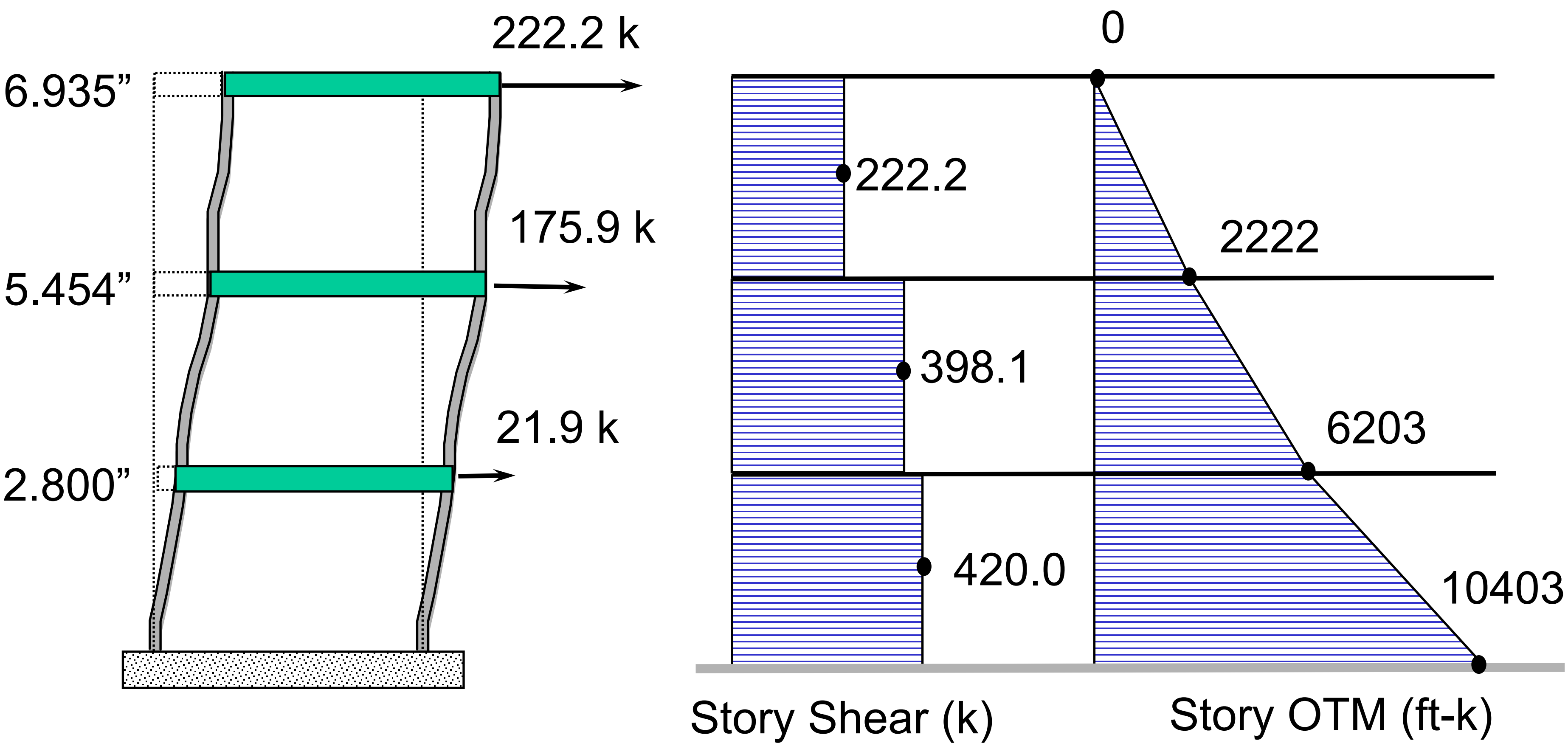
Compute story shear response histories:



$$=k_2[u_2(t)-u_3(t)]$$

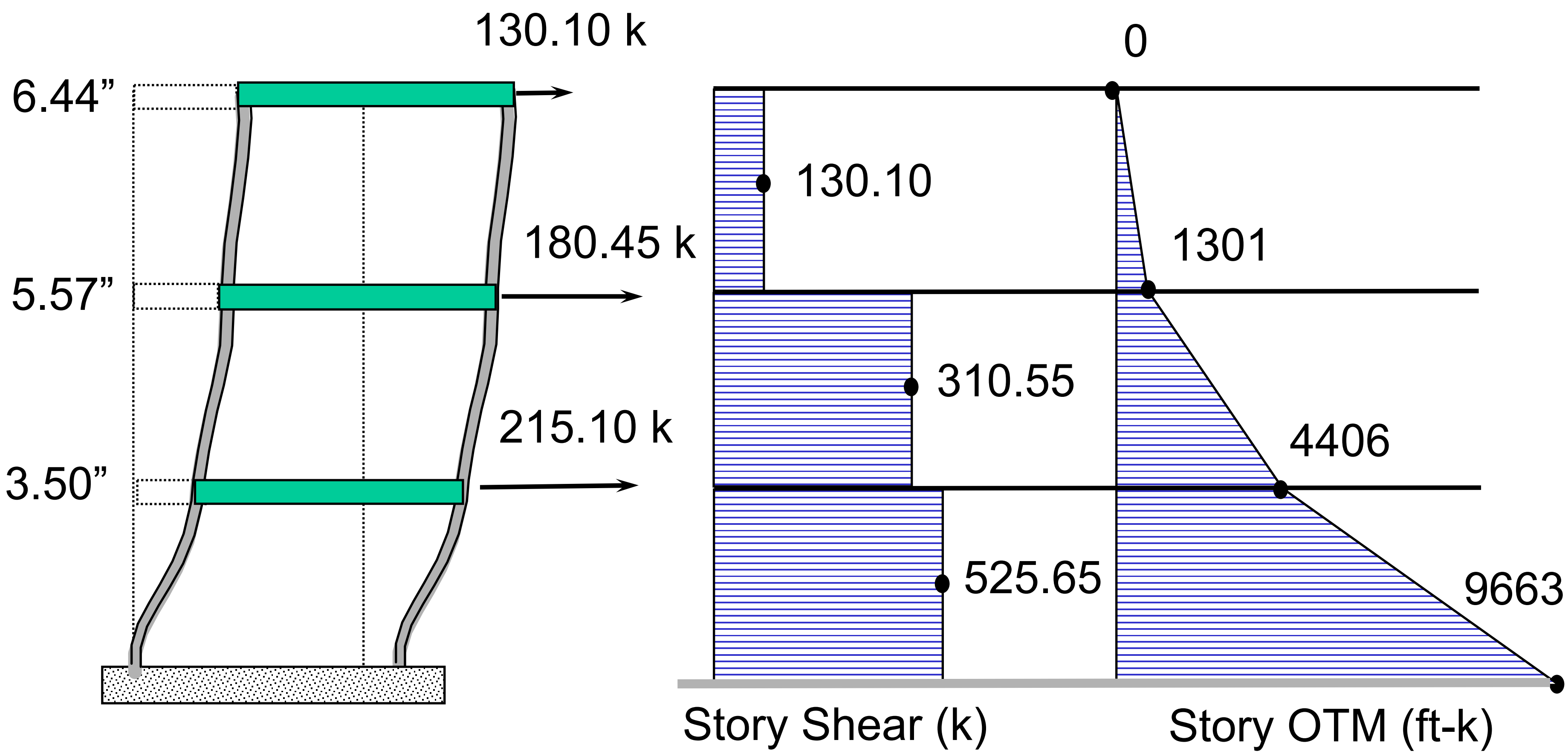
Example 2 (continued)

Displacements and Forces at time of Maximum Displacements
(t = 8.96 seconds)



Example 2 (continued)

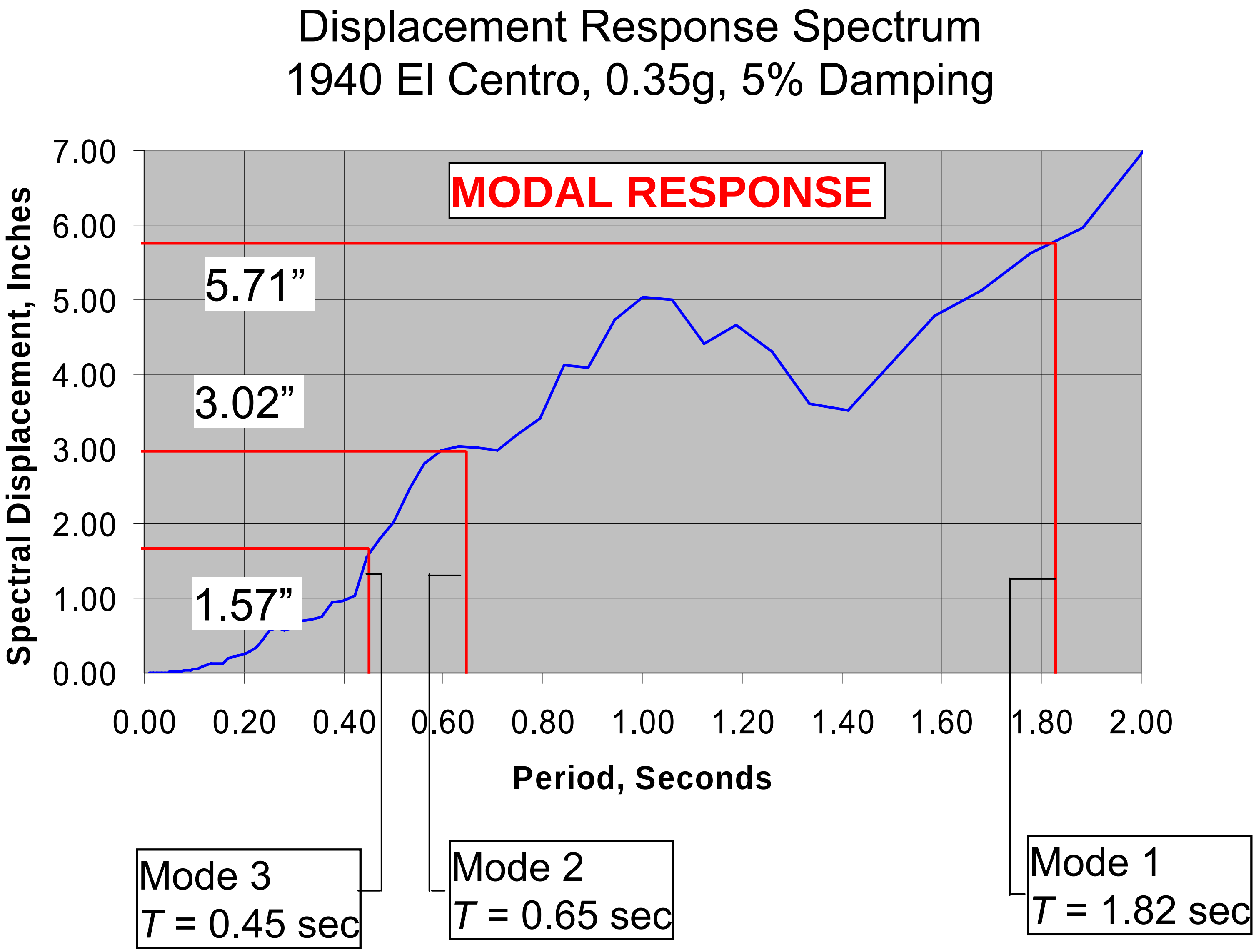
Displacements and Forces at Time of Maximum Shear
(t = 6.26 sec)



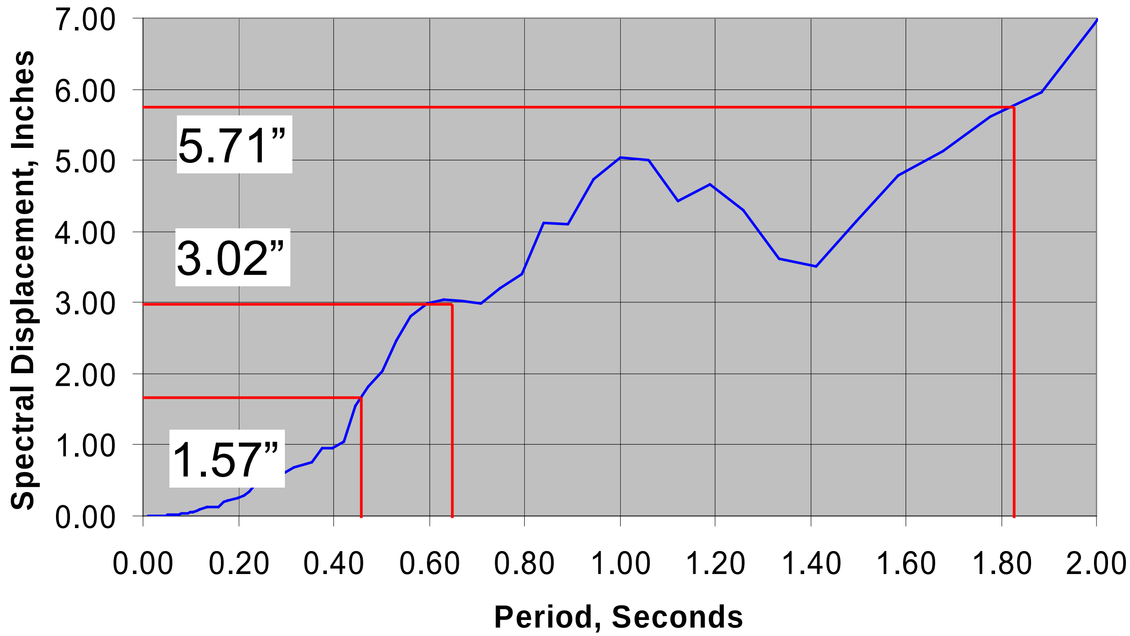
Modal Response Response Spectrum Method

- Instead of solving the time history problem for each mode, use a response spectrum to compute the **maximum** response in each mode.
- These maxima are generally **nonconcurrent**.
- Combine the maximum modal responses using some statistical technique, such as square root of the sum of the squares (SRSS) or complete quadratic combination (CQC).
- The technique is **approximate**.
- It is the basis for the equivalent lateral force (ELF) method.

Example 2 (Response Spectrum Method)



Example 2 (continued)



Modal Equations of Motion

$$\ddot{y}_1 + 0.345\dot{y}_1 + 11.88y_1 = -1.22\ddot{v}_g(t)$$

$$\ddot{y}_2 + 0.966\dot{y}_2 + 93.29y_2 = 0.280\ddot{v}_g(t)$$

$$\ddot{y}_3 + 1.395\dot{y}_3 + 194.83y_3 = -0.060\ddot{v}_g(t)$$

Modal Maxima

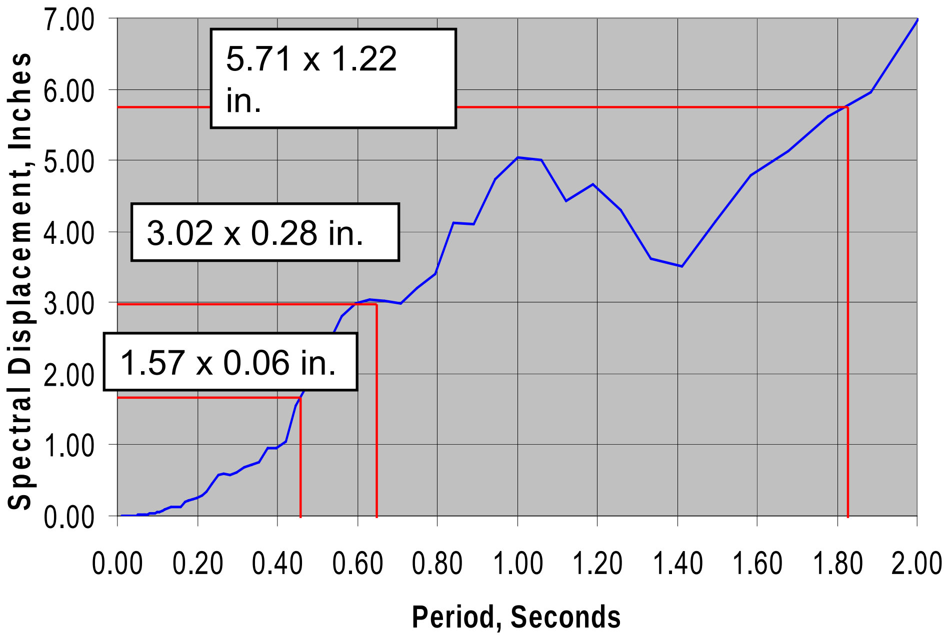
$$\bar{y}_1 = 1.22 * 5.71 = 6.966"$$

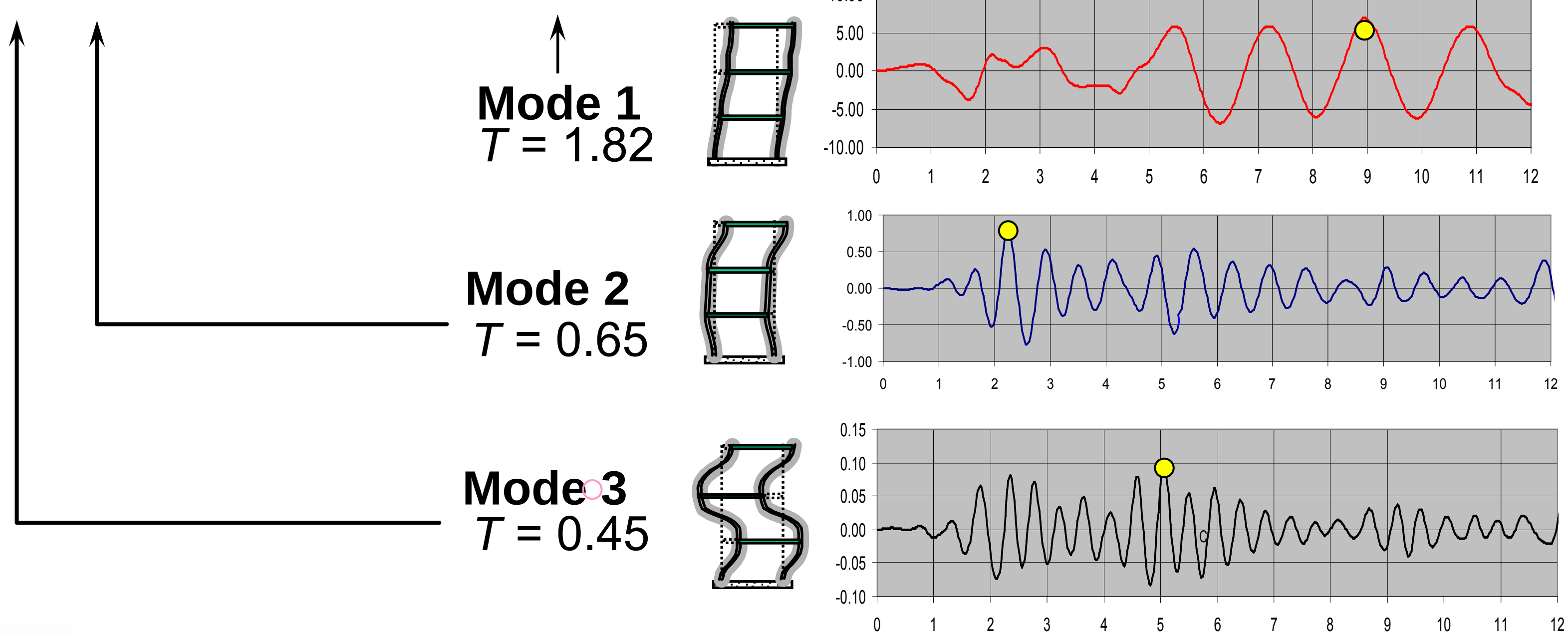
$$\bar{y}_2 = 0.28 * 3.02 = 0.845"$$

$$\bar{y}_3 = 0.060 * 1.57 = 0.094"$$

Example 2 (continued)

The **scaled** response spectrum values give the same **modal maxima** as the previous time histories.

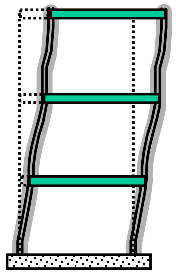




Example 2 (continued)

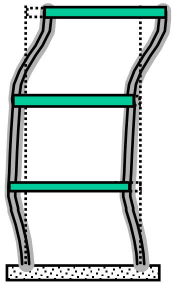
Computing **Nonconcurrent** Story Displacements

Mode 1



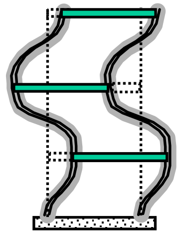
$$\begin{Bmatrix} 1.000 \\ 0.802 \\ 0.445 \end{Bmatrix} 6.966 = \begin{Bmatrix} 6.966 \\ 5.586 \\ 3.100 \end{Bmatrix}$$

Mode 2



$$\begin{Bmatrix} 1.000 \\ -0.555 \\ -1.247 \end{Bmatrix} 0.845 = \begin{Bmatrix} 0.845 \\ -0.469 \\ -1.053 \end{Bmatrix}$$

Mode 3



$$\begin{Bmatrix} 1.000 \\ -2.247 \\ 1.802 \end{Bmatrix} 0.094 = \begin{Bmatrix} 0.094 \\ -0.211 \\ 0.169 \end{Bmatrix}$$

Example 2 (continued)

Modal Combination Techniques (For Displacement)

Sum of absolute values:

$$\begin{Bmatrix} 6.966+0.845+0.108 \\ 5.586+0.469+0.211 \\ 3.100+1.053+0.169 \end{Bmatrix} = \begin{Bmatrix} 7.919 \\ 6.266 \\ 4.322 \end{Bmatrix}$$

At time of maximum displacement

$$\boxed{\begin{array}{c}\text{“Exact”} \\ \begin{Bmatrix} 6.935 \\ 5.454 \\ 2.800 \end{Bmatrix}\end{array}}$$

Square root of the sum of the squares

$$\begin{Bmatrix} \sqrt{6.966^2+0.845^2+0.108^2} \\ \sqrt{5.586^2+0.469^2+0.211^2} \\ \sqrt{3.100^2+1.053^2+0.169^2} \end{Bmatrix} = \begin{Bmatrix} 7.02 \\ 5.61 \\ 3.28 \end{Bmatrix}$$

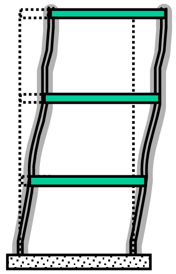
Envelope of story displacement

$$\boxed{\begin{Bmatrix} 6.935 \\ 5.675 \\ 2.965 \end{Bmatrix}}$$

Example 2 (continued)

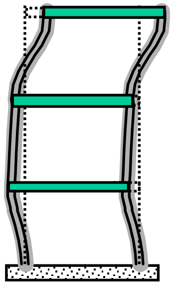
Computing Interstory Drifts

Mode 1



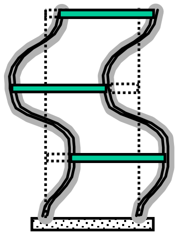
$$\begin{Bmatrix} 6.966-5.586 \\ 5.586-3.100 \\ 3.100-0 \end{Bmatrix} = \begin{Bmatrix} 1.380 \\ 2.486 \\ 3.100 \end{Bmatrix}$$

Mode 2



$$\begin{Bmatrix} 0.845-(-0.469) \\ -0.469-(-1.053) \\ -1.053-0 \end{Bmatrix} = \begin{Bmatrix} 1.314 \\ 0.584 \\ -1.053 \end{Bmatrix}$$

Mode 3

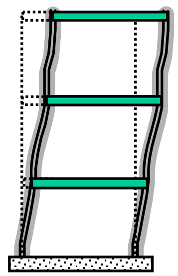


$$\begin{Bmatrix} 0.108-(-0.211) \\ -0.211-0.169 \\ 0.169-0 \end{Bmatrix} = \begin{Bmatrix} 0.319 \\ -0.380 \\ 0.169 \end{Bmatrix}$$

Example 2 (continued)

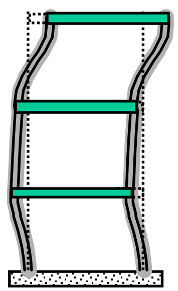
Computing Interstory Shears (Using Drift)

Mode 1



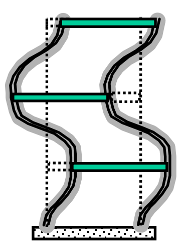
$$\begin{Bmatrix} 1.380(150) \\ 2.486(150) \\ 3.100(150) \end{Bmatrix} = \begin{Bmatrix} 207.0 \\ 372.9 \\ 465.0 \end{Bmatrix}$$

Mode 2



$$\begin{Bmatrix} 1.314(150) \\ 0.584(150) \\ -1.053(150) \end{Bmatrix} = \begin{Bmatrix} 197.1 \\ 87.6 \\ -157.9 \end{Bmatrix}$$

Mode 3



$$\begin{Bmatrix} 0.319(150) \\ -0.380(150) \\ 0.169(150) \end{Bmatrix} = \begin{Bmatrix} 47.9 \\ -57.0 \\ 25.4 \end{Bmatrix}$$

Example 2 (continued)

Computing Interstory Shears: SRSS Combination

$$\begin{Bmatrix} \sqrt{207^2 + 197.1^2 + 47.9^2} \\ \sqrt{372.9^2 + 87.6^2 + 57^2} \\ \sqrt{465^2 + 157.9^2 + 25.4^2} \end{Bmatrix} = \begin{Bmatrix} 289.81 \\ 387.27 \\ 491.73 \end{Bmatrix}$$

"Exact"

$$\begin{Bmatrix} 130.1 \\ 310.5 \\ 525.7 \end{Bmatrix}$$

At time of max. shear

"Exact"

$$\begin{Bmatrix} 222.2 \\ 398.1 \\ 420.0 \end{Bmatrix}$$

At time of max. displacement

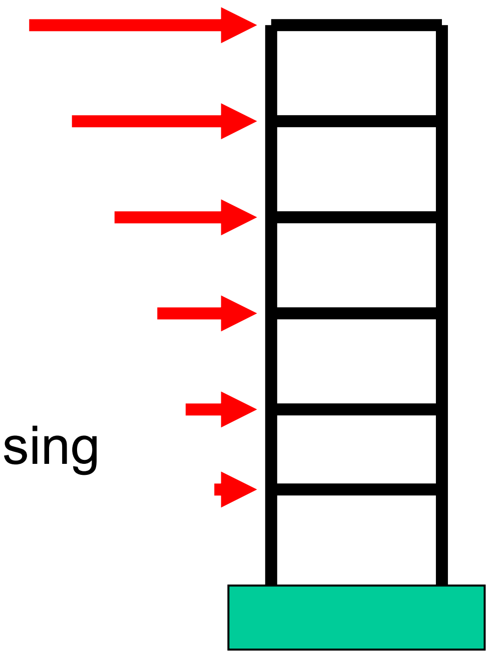
"Exact"

$$\begin{Bmatrix} 304.0 \\ 398.5 \\ 525.7 \end{Bmatrix}$$

Envelope = maximum per story

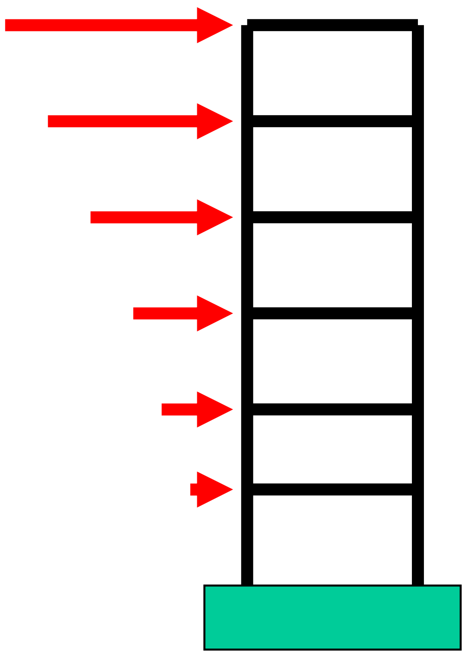
ASCE 7 Allows an Approximate Modal Analysis Technique Called the Equivalent Lateral Force Procedure

- Empirical period of vibration
- Smoothed response spectrum
- Compute total base shear, **V**, as if SDOF
- Distribute **V** along height assuming “regular” geometry
- Compute displacements and member forces using standard procedures



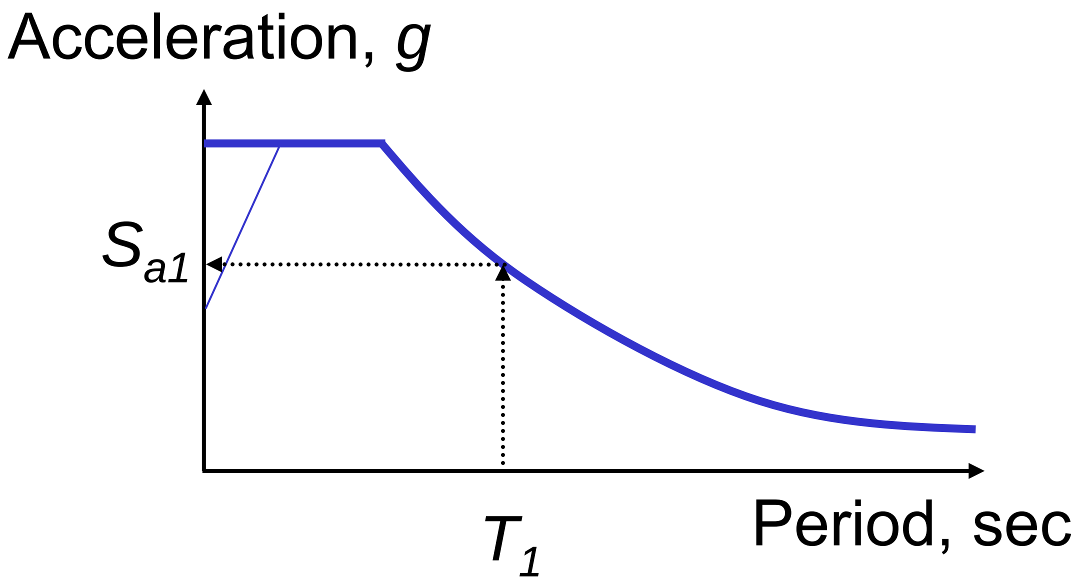
Equivalent Lateral Force Procedure

- Method is based on **first mode** response.
- Higher modes can be included empirically.
- Has been calibrated to provide a reasonable estimate of the envelope
  of story shear, NOT to provide accurate estimates of story force.
- May result in overestimate of overturning moment.



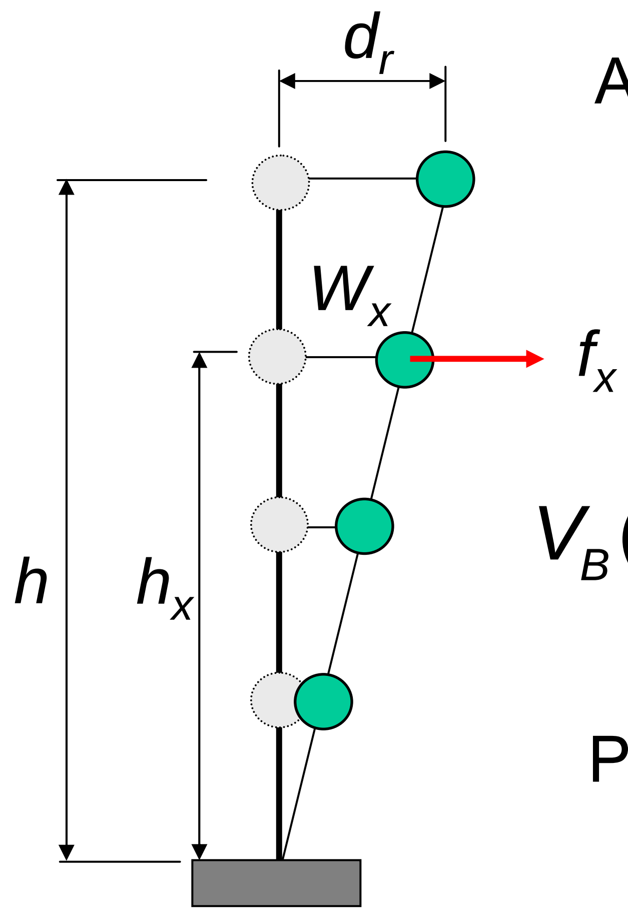
Equivalent Lateral Force Procedure

- Assume first mode effective mass = total Mass = $M = W/g$
- Use response spectrum to obtain total acceleration @ T_1



$$V_B = (S_{a1}g)M = (S_{a1}g)\frac{W}{g} = S_{a1}W$$

Equivalent Lateral Force Procedure



Assume linear first mode response

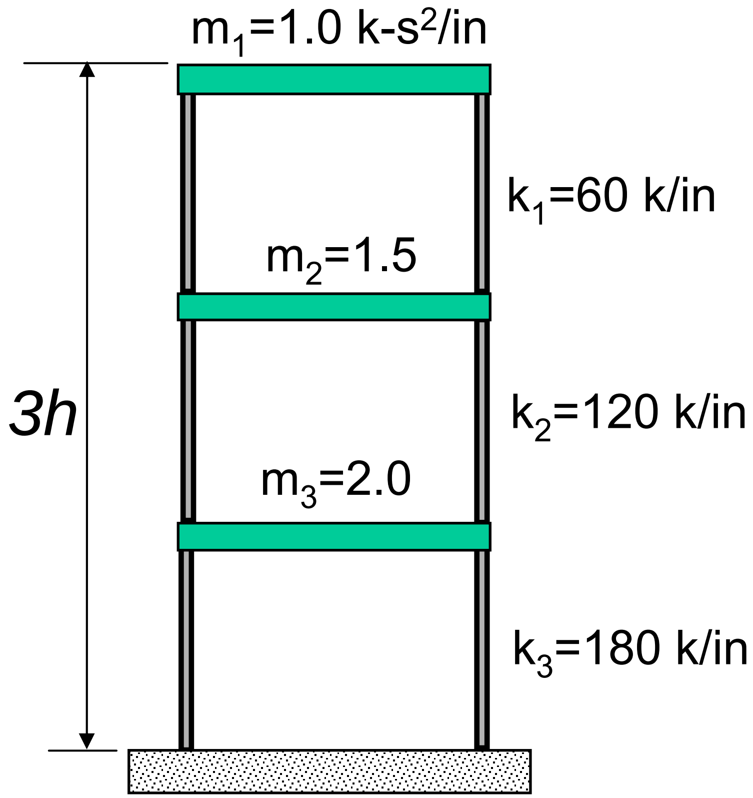
$$f_x(t) = \frac{h_x}{h} d_r(t)\omega_1^2 \frac{W_x}{g}$$

$$V_B(t) = \sum_{i=1}^{nstories} f_i(t) = \frac{d_r(t)\omega_1^2}{hg} \sum_{i=1}^{nstories} h_i W_i$$

Portion of base shear applied to story i

$$\boxed{\frac{f_x(t)}{V_B(t)} = \frac{h_x W_x}{\sum_{i=1}^{nstories} h_i W_i}}$$

ELF Procedure Example



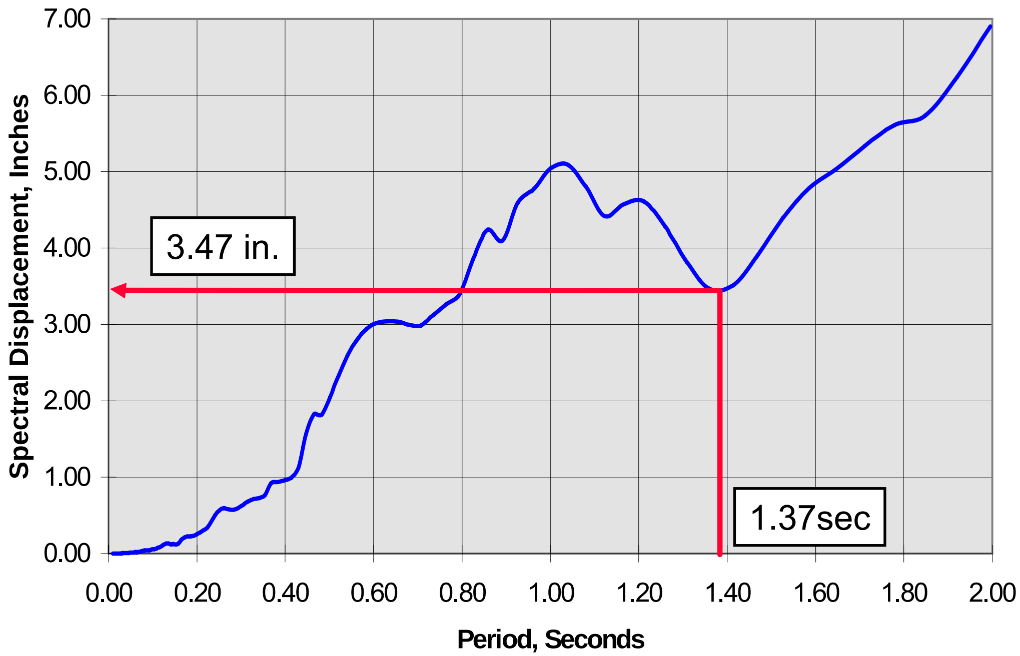
Recall

$T_1 = 1.37$ sec

ELF Procedure Example

Total weight = M x g = (1.0 + 1.5 + 2.0) 386.4 = 1738 kips

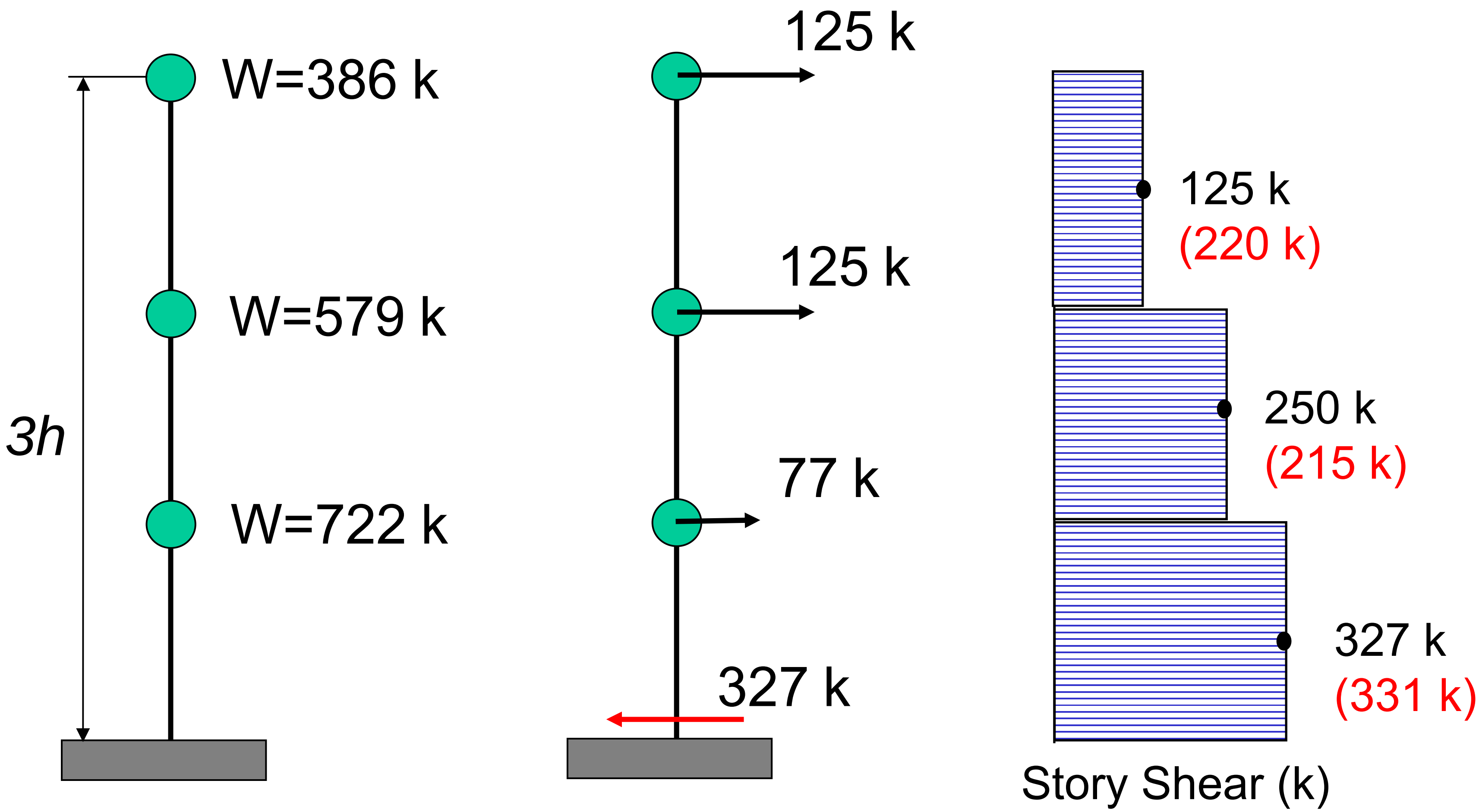
Spectral acceleration = $w^2 S_D$ = (2p/1.37)^2 x 3.47 = 72.7 in/sec^2 = 0.188g



Base shear = $S_a W$ = 0.188 x 1738 = 327 kips

ELF Procedure Example (Story Forces)

$$f_3 = \frac{386(3h)}{386(3h)+579(2h)+722(h)} = 0.381V_B = 0.375(327) = 125\ kips$$



ELF Procedure Example (Story Displacements)

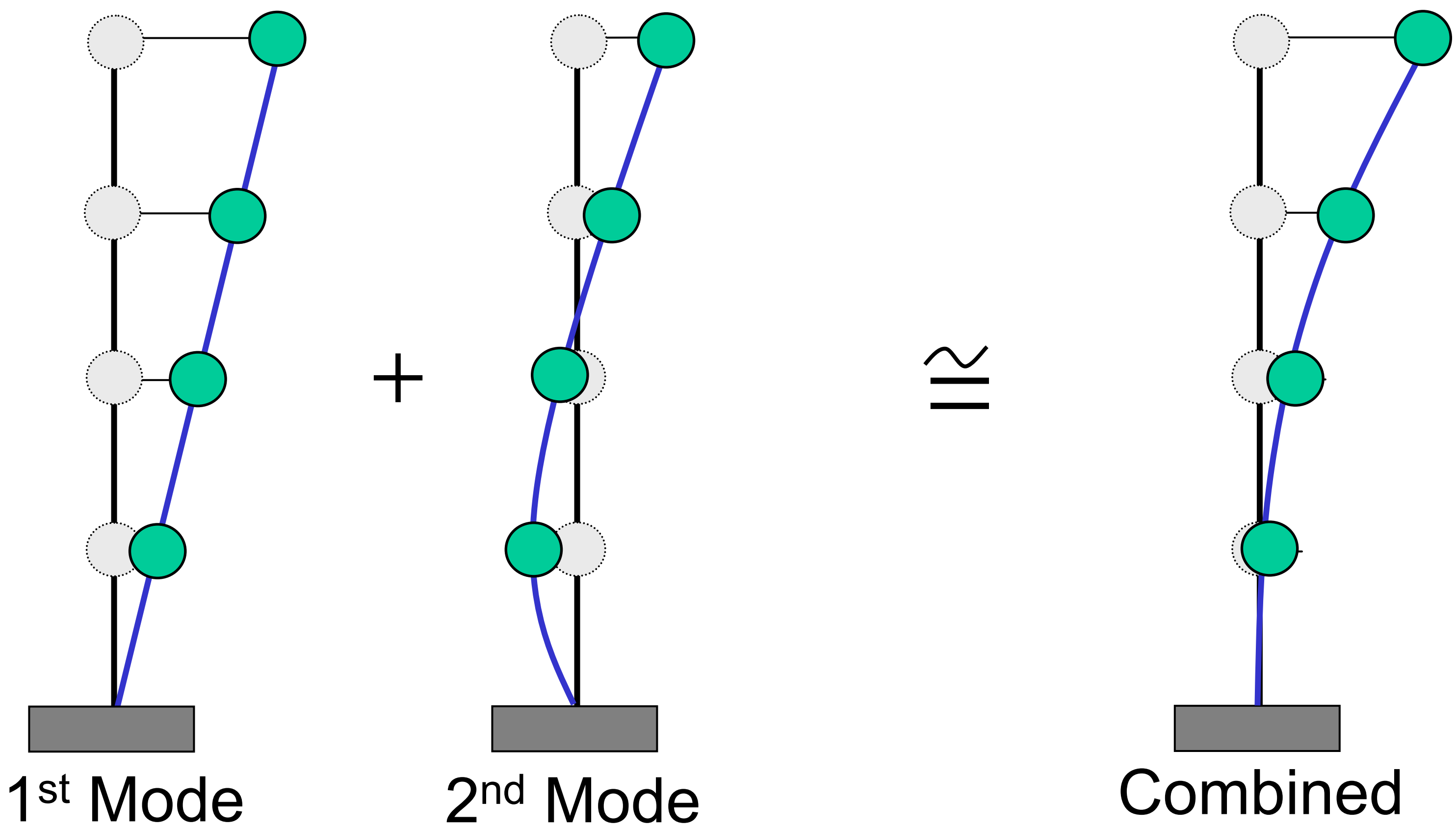
Units = inches

Time History (Envelope)	Modal Response Spectrum	**ELF**
$\begin{Bmatrix} 5.15 \\ 3.18 \\ 1.93 \end{Bmatrix}$	$\begin{Bmatrix} 5.18 \\ 3.33 \\ 1.84 \end{Bmatrix}$	$\begin{Bmatrix} 5.98 \\ 3.89 \\ 1.82 \end{Bmatrix}$

ELF Procedure Example (Summary)

- ELF procedure gives **good correlation** with base shear (327 kips ELF vs 331 kips modal response spectrum).
- ELF story force distribution is **not** as **good**. ELF underestimates shears in upper stories.
- ELF gives reasonable correlation with displacements.

Equivalent Lateral Force Procedure
Higher Mode Effects



ASCE 7-05 ELF Approach

- Uses empirical period of vibration
- Uses smoothed response spectrum
- Has correction for higher modes
- Has correction for overturning moment
- Has limitations on use

Approximate Periods of Vibration

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

C_t = 0.028, x = 0.8 for steel moment frames
C_t = 0.016, x = 0.9 for concrete moment frames
C_t = 0.030, x = 0.75 for eccentrically braced frames
C_t = 0.020, x = 0.75 for all other systems
Note: For building structures **only**!

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

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

Adjustment Factor on Approximate Period

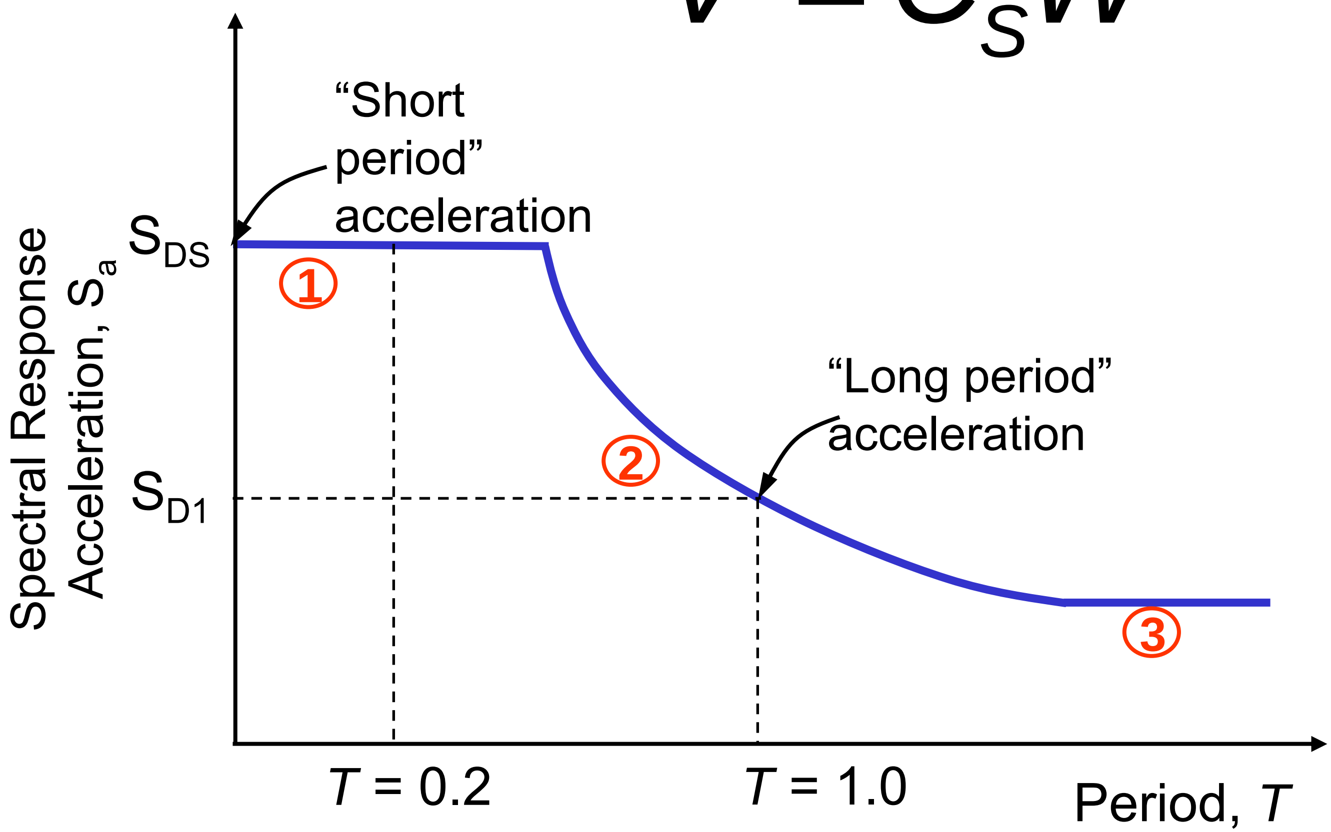
$$T = T_a C_u \leq T_{computed}$$

S_{D1}	C_u
> 0.40g	1.4
0.30g	1.4
0.20g	1.5
0.15g	1.6
< 0.1g	1.7

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

ASCE 7 Smoothed Design Acceleration Spectrum (for Use with ELF Procedure)

$$V = C_S W$$



1 $$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

2 $$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

3 **Varies**

R is the *response modification factor*, a function of system inelastic behavior. This is covered in the topic on inelastic behavior. For now, use R = 1, which implies linear elastic behavior.

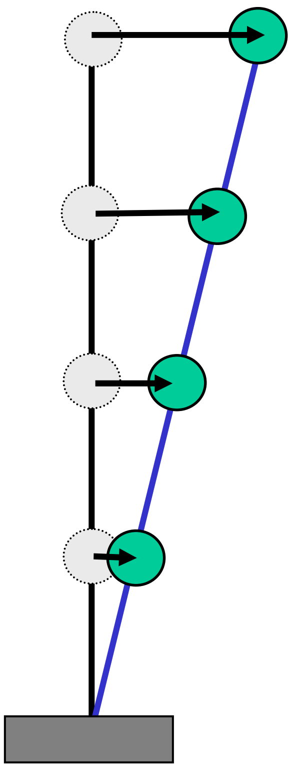
I is the *importance factor* which depends on the Seismic Use Group. I = 1.5 for essential facilities, 1.25 for important high occupancy structures, and 1.0 for normal structures. For now, use I = 1.

Distribution of Forces Along Height

$$F_x = C_{vx} V$$

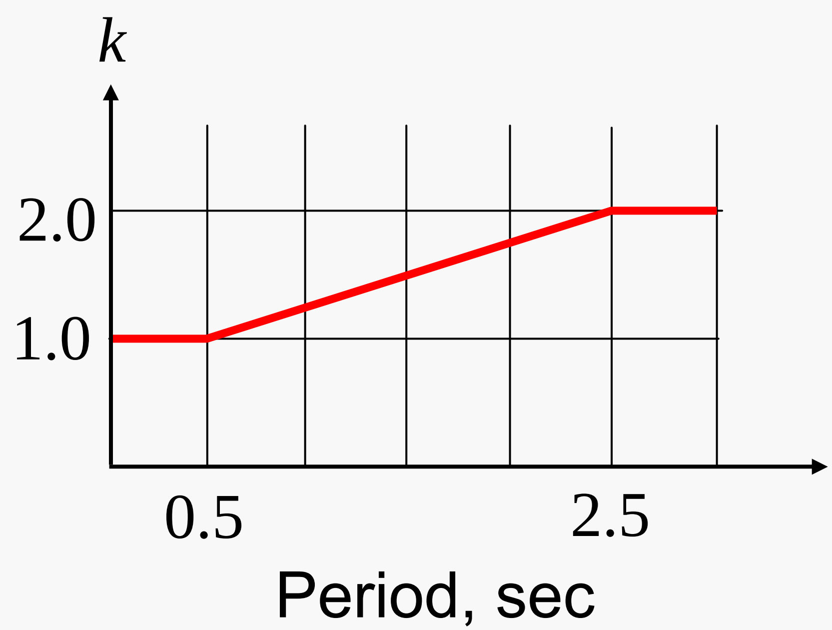
$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

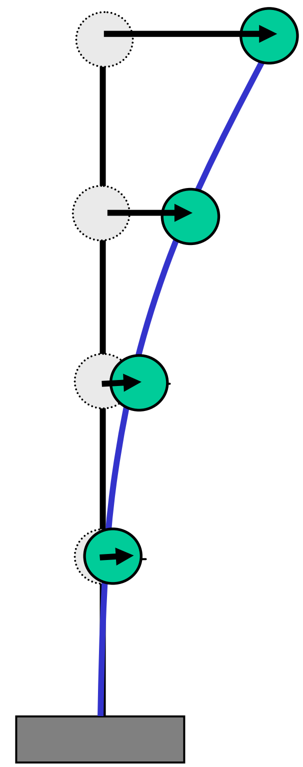
k Accounts for Higher Mode Effects



k = 1

$k = 0.5T + 0.75$
(sloped portion only)

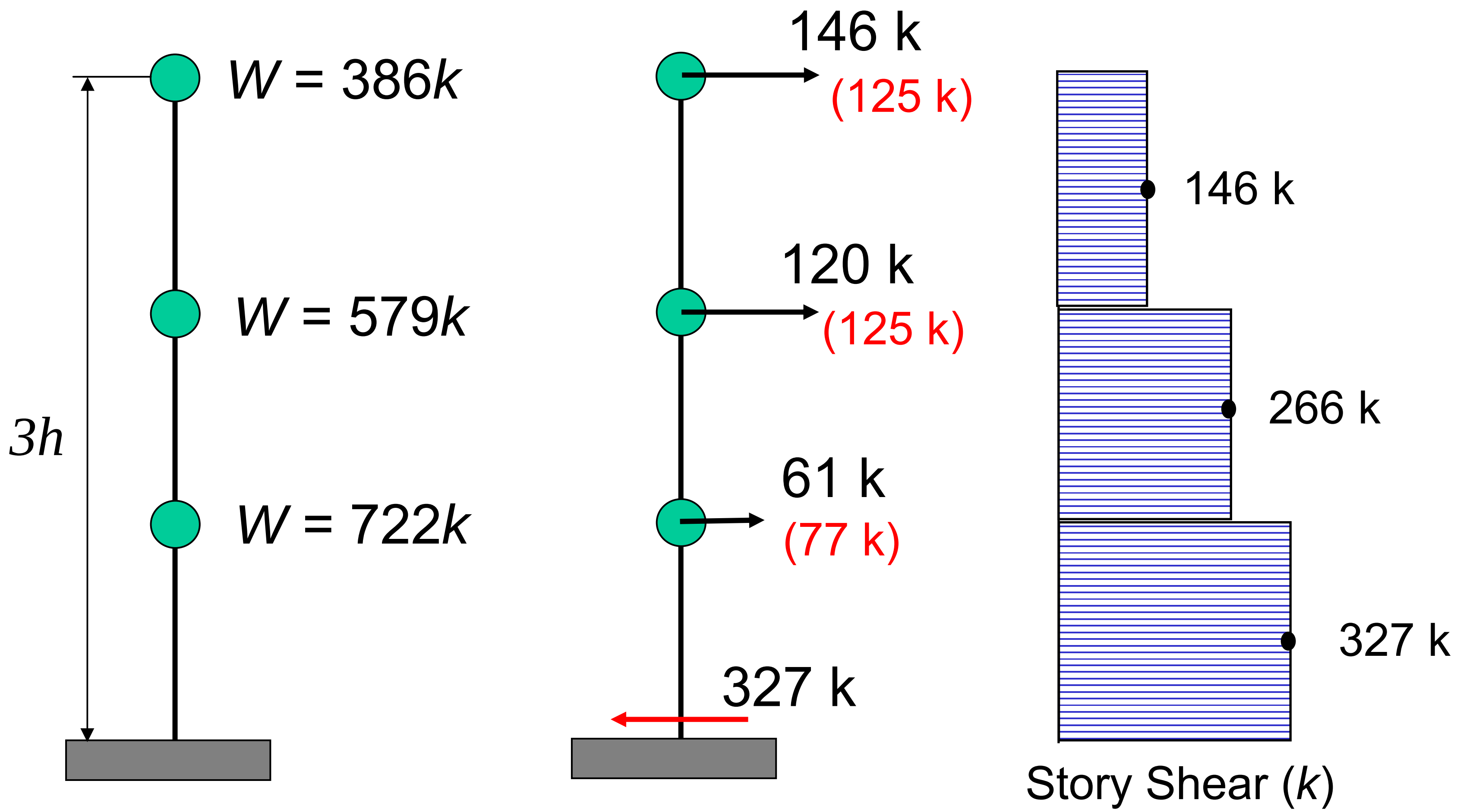




k = 2

ELF Procedure Example (Story Forces)

V = 327 kips $\quad$ T = 1.37 sec $\quad$ k = 0.5(1.37) + 0.75 = 1.435



ASCE 7 ELF Procedure Limitations

- Applicable **only** to “regular” structures with T less than $3.5T_s$. Note that $T_s = S_{D1}/S_{DS}$.

- Adjacent story stiffness does not vary more than 30%.

- Adjacent story strength does not vary more than 20%.

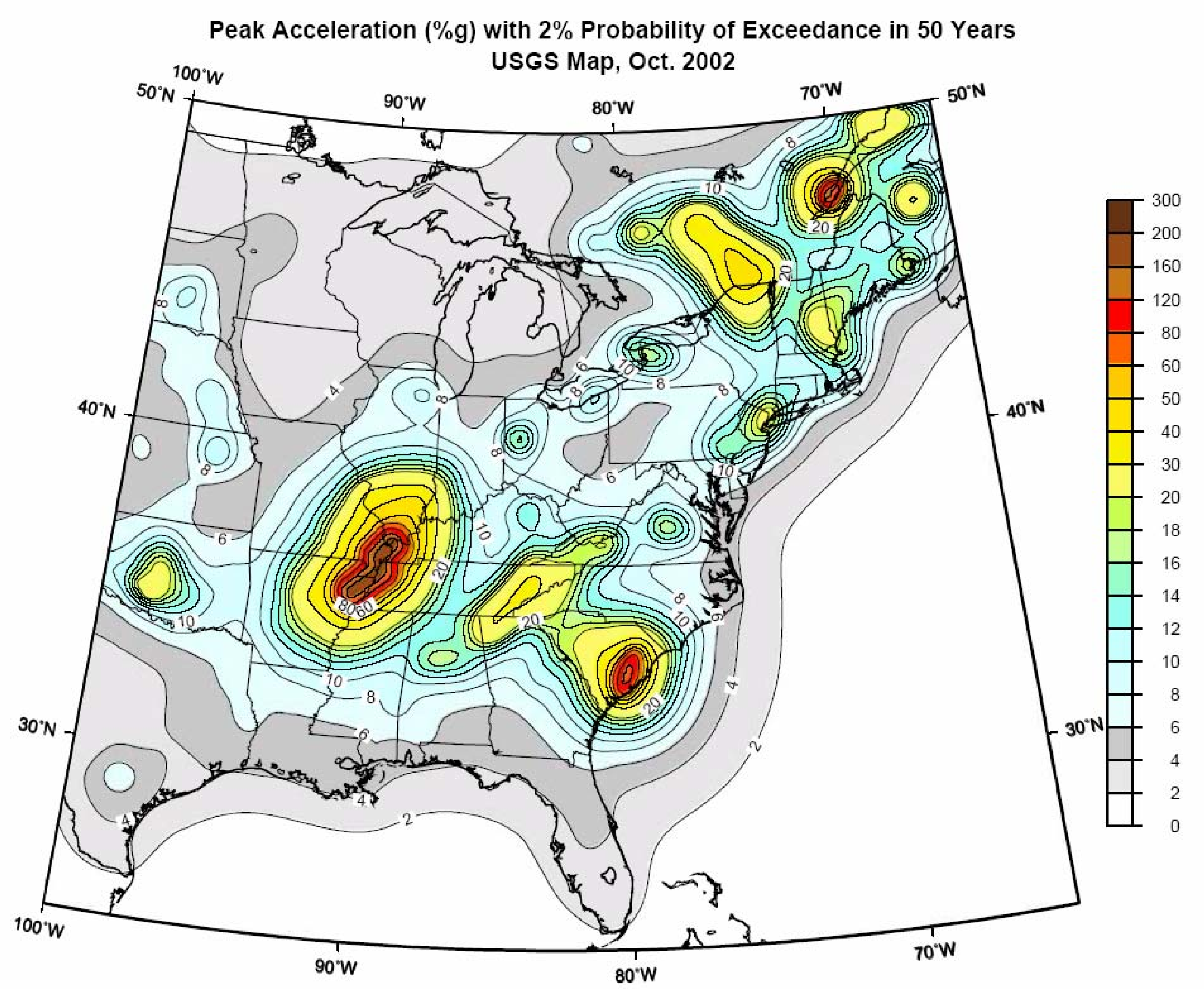
- Adjacent story masses does not vary more than 50%.

If violated, must use more advanced analysis (typically modal response spectrum analysis).

ASCE 7 ELF Procedure
Other Considerations Affecting Loading

- Orthogonal loading effects
- Redundancy
- Accidental torsion
- Torsional amplification
- P-delta effects
- Importance factor
- Ductility and overstrength

SEISMIC HAZARD ANALYSIS



Seismic Hazard Analysis

- Deterministic procedures
- Probabilistic procedures
- USGS hazard maps
- 2003 *NEHRP Provisions* design maps
- Site amplification
- *NEHRP Provisions* response spectrum
- UBC response spectrum

Hazard vs Risk

Seismic hazard analysis

describes the potential for dangerous, earthquake-related natural phenomena such as ground shaking, fault rupture, or soil liquefaction.

Seismic risk analysis

assesses the probability of occurrence of losses (human, social, economic) associated with the seismic hazards.

Approaches to Seismic Hazard Analysis

Deterministic

"The earthquake hazard for the site is a peak ground acceleration of 0.35g resulting from an earthquake of magnitude 6.0 on the Balcones Fault at a distance of 12 miles from the site. "

Probabilistic

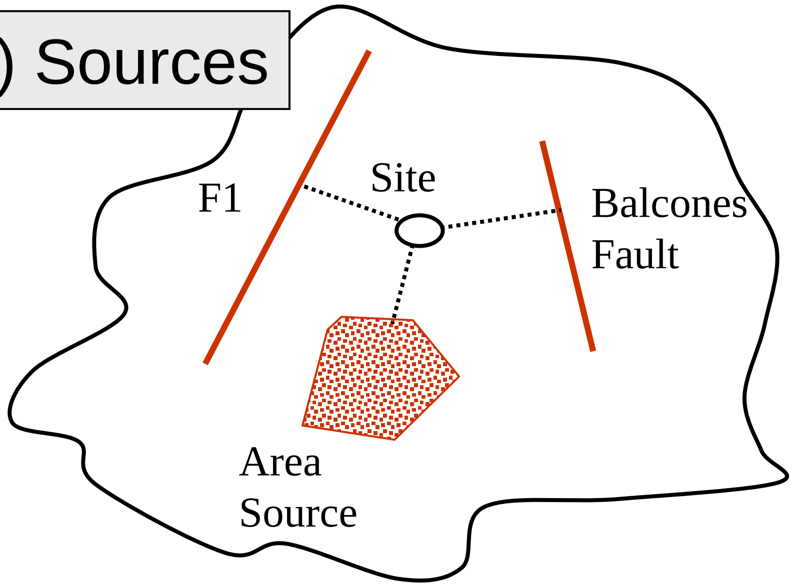
"The earthquake hazard for the site is a peak ground acceleration of 0.28g with a 2 percent probability of being exceeded in a 50-year period."

Probabilistic Seismic Hazard Analysis

First addressed in 1968 by C. Allin Cornell in "Engineering Seismic Risk Analysis," and article in the *Bulletin of the Seismological Society* (Vol. 58, No. 5, October).

Steps in Deterministic Seismic Hazard Analysis

(1) Sources

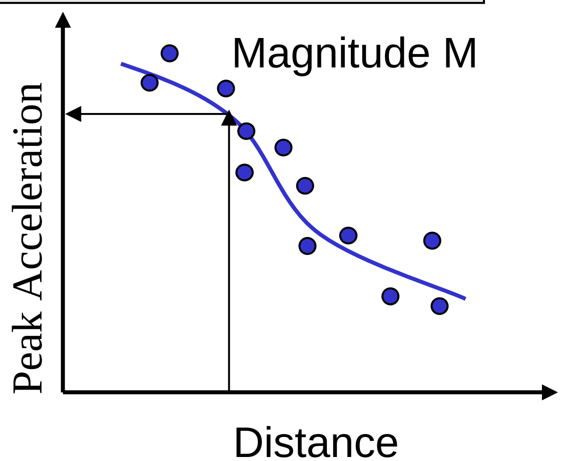


(2) Controlling Earthquake

Fixed distance R

Fixed magnitude M

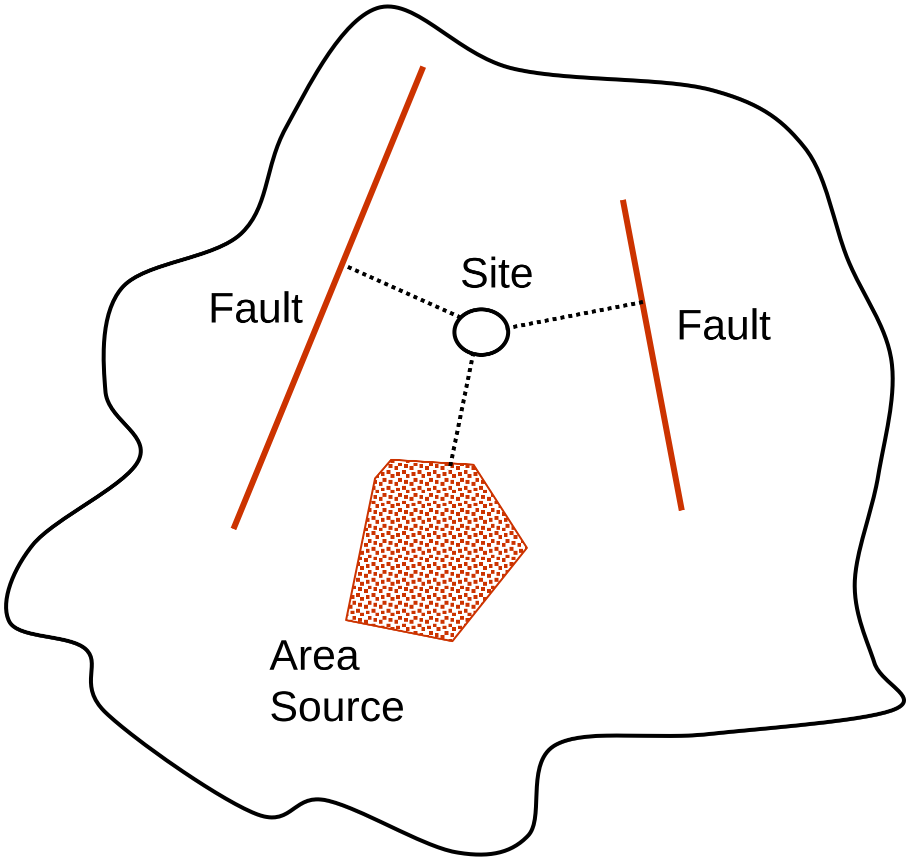
(3) Ground Motion



(4) Hazard at Site

"The earthquake hazard for the site is a peak ground acceleration of 0.35 g resulting from an earthquake of magnitude 6.0 on the Balcones Fault at a distance of 12 miles from the site. "

Source Types



- Fault
- Localizing structure
- Seismotectonic province

Source Types

Localizing structure: An identifiable geological structure that is assumed to generate or "localize" earthquakes. This is generally a concentration of known or unknown active faults.

Seismotectonic province: A region where there is a known seismic hazard but where there are no identifiable active faults or localizing structures.

Maximum Earthquake

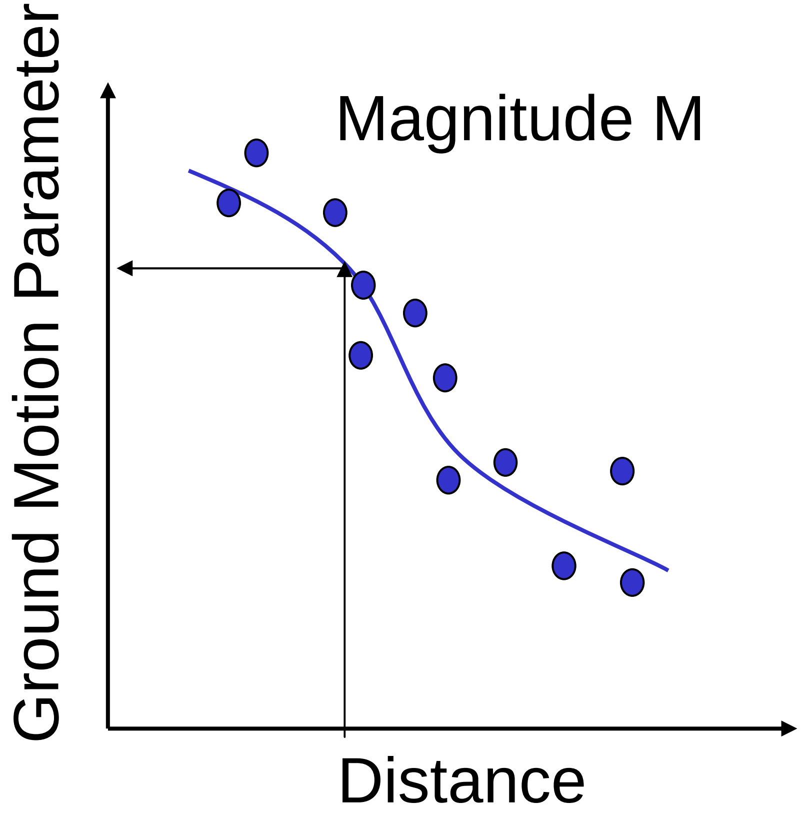
Maximum possible earthquake: An upper bound to size (however unlikely) determined by earthquake processes (e.g., maximum seismic moment).

Maximum credible earthquake: The maximum reasonable earthquake size based on earthquake processes (but does not imply likely occurrence).

Maximum historic earthquake: The maximum historic or instrumented earthquake that is often a lower bound on maximum possible or maximum credible earthquake.

Maximum considered earthquake: Described later.

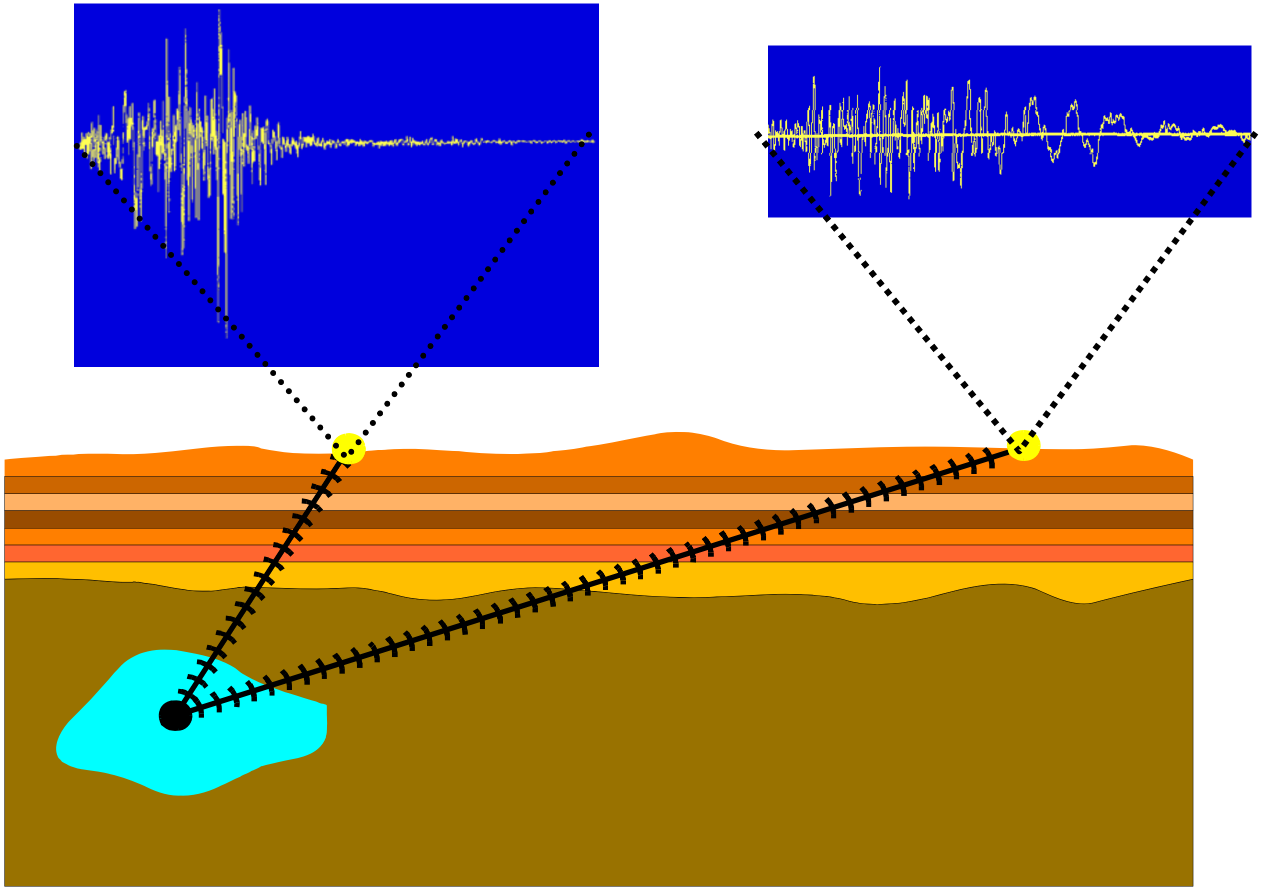
Ground Motion Attenuation



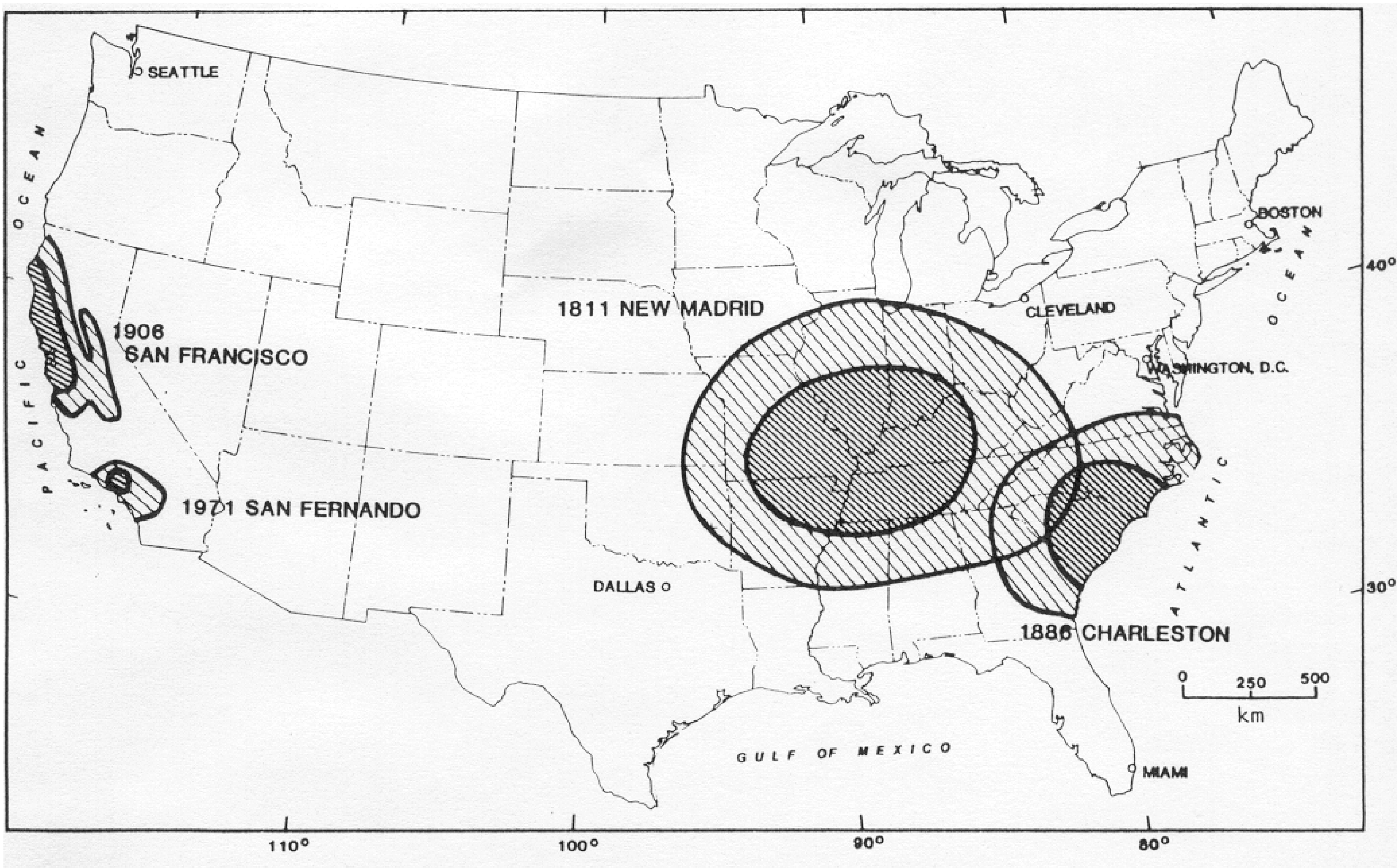
Reasons:

- Geometric spreading
- Absorption (damping)

Attenuation with Distance



Comparison of Attenuation for Four Earthquakes



Ground Motion Attenuation
Steps to Obtain Empirical Relationship

1. Obtain catalog of appropriate ground motion records
2. Correct for aftershocks, foreshocks
3. Correct for consistent magnitude measure
4. Fit data to empirical relationship of type:

$$\ln \hat{Y} = \ln b_1 + f_1(M) + \ln f_2(R) + \ln f_3(M, R) + \ln f_4(P_i) + \ln \varepsilon$$

Ground Motion Attenuation
Basic Empirical Relationships

$$\ln \hat{Y} = \ln b_1 + f_1(M) + \ln f_2(R) + \ln f_3(M,R) + \ln f_4(P_i) + \ln \varepsilon$$

$\hat{Y}$	Ground motion parameter (e.g. PGA)
b_1	Scaling factor
$f_1(M)$	Function of magnitude
$f_2(R)$	Function of distance
$f_3(M,R)$	Function of magnitude and distance
$f_4(P_i)$	Other variables
ε	Error term

Ground Motion Attenuation Relationships for Different Conditions

- Central and eastern United States
- Subduction zone earthquakes
- Shallow crustal earthquakes
- Near-source attenuation
- Extensional tectonic regions
- Many others

May be developed for any desired quantity (PGA, PGV, spectral response).

Ground Motion Attenuation Relationships

Seismological Research Letters
Volume 68, Number 1
January/February, 1997

Earthquake Catalog for Shallow Crustal Earthquakes

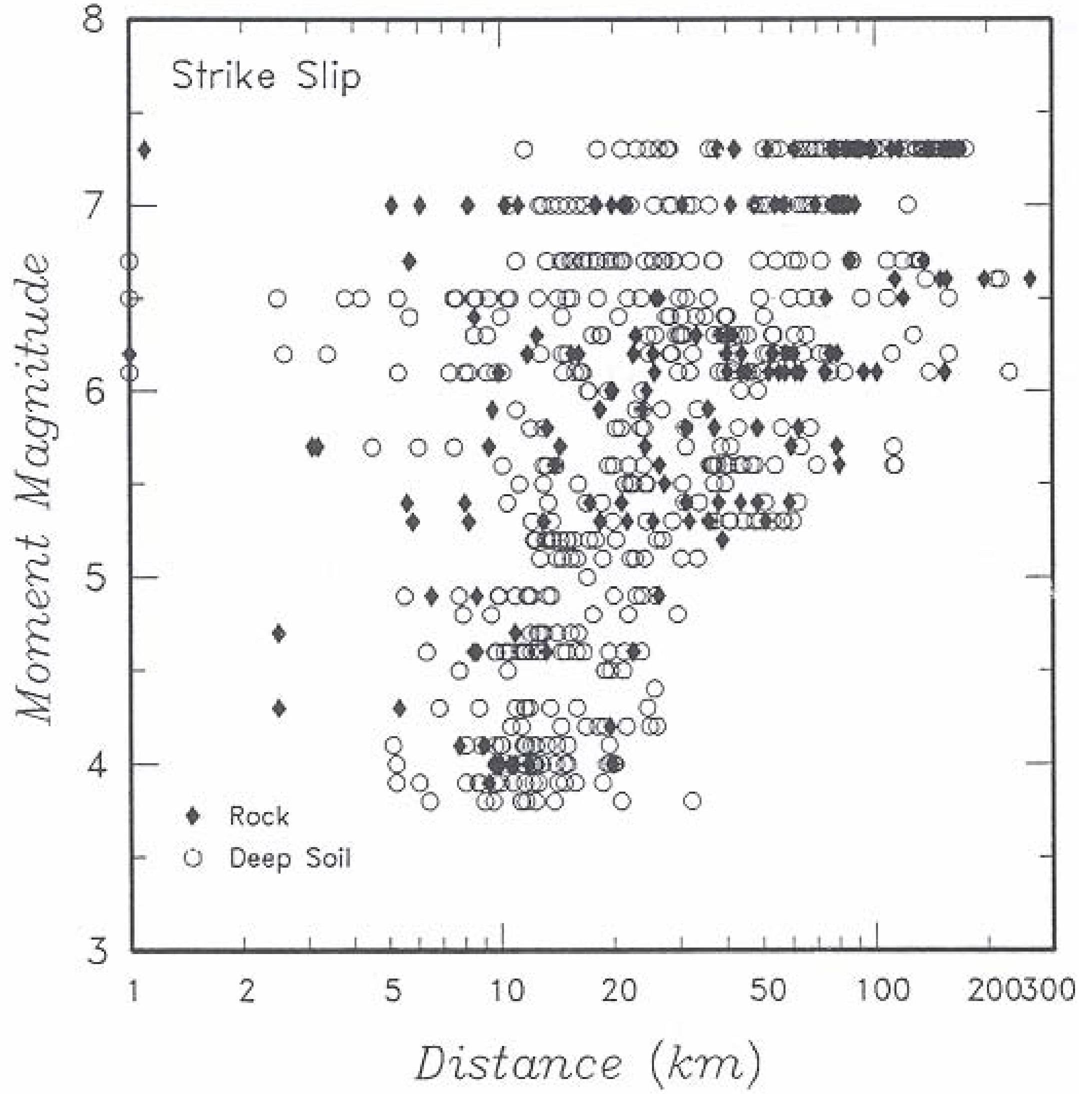
(Sadigh, Chang, Egan, Makdisi, and Youngs)

TABLE 1
List of Earthquakes Used to Develop Attenuation Relationships

Earthquake	Date	M	Fault Type[1]	Distance Range (km)	No. of Records[2]	
					R	DS
Kern County,CA	1952/07/21	7.4	RV	120.5–224.0	0	3
Port Hueneme, CA	1957/03/18	4.7	RV	14.1–14.1	0	1
Daly City, CA	1957/03/22	5.3	RV	9.5–9.5	1	0
Parkfield, CA	1966/06/27	6.1	SS	0.1–230.0	1	6
Borrego Mtn., CA	1968/04/09	6.6	SS	113.0–261.0	5	3
Santa Rosa, CA (A)	1969/10/02	5.6	SS	80.0–113.0	1	2
Santa Rosa, CA (B)	1969/10/02	5.7	SS	78.9–112.0	1	2
Lytle Creek, CA	1970/09/12	5.3	RV	19.7–76.0	5	2
San Fernando, CA	1971/02/09	6.6	RV	2.8–305.0	11	14
Lake Isabella, CA	1971/03/08	4.1	SS	8.9–8.9	1	0
Bear Valley, CA	1972/02/24	4.7	SS	2.5–2.5	1	0
Point Mugu, CA	1973/02/21	5.6	RV	25.0–25.0	0	1
Hollister, CA	1974/11/28	5.2	SS	39.0–39.0	1	0
Oroville, CA	1975/08/01	5.9	SS	9.5–35.8	2	2
Oroville, CA (R)	1975/08/02	5.1	SS	12.7–14.6	0	2
Oroville, CA (S)	1975/08/02	5.2	SS	12.4–15.0	0	2
Oroville, CA (A)	1975/08/03	4.6	SS	8.4–14.9	1	6

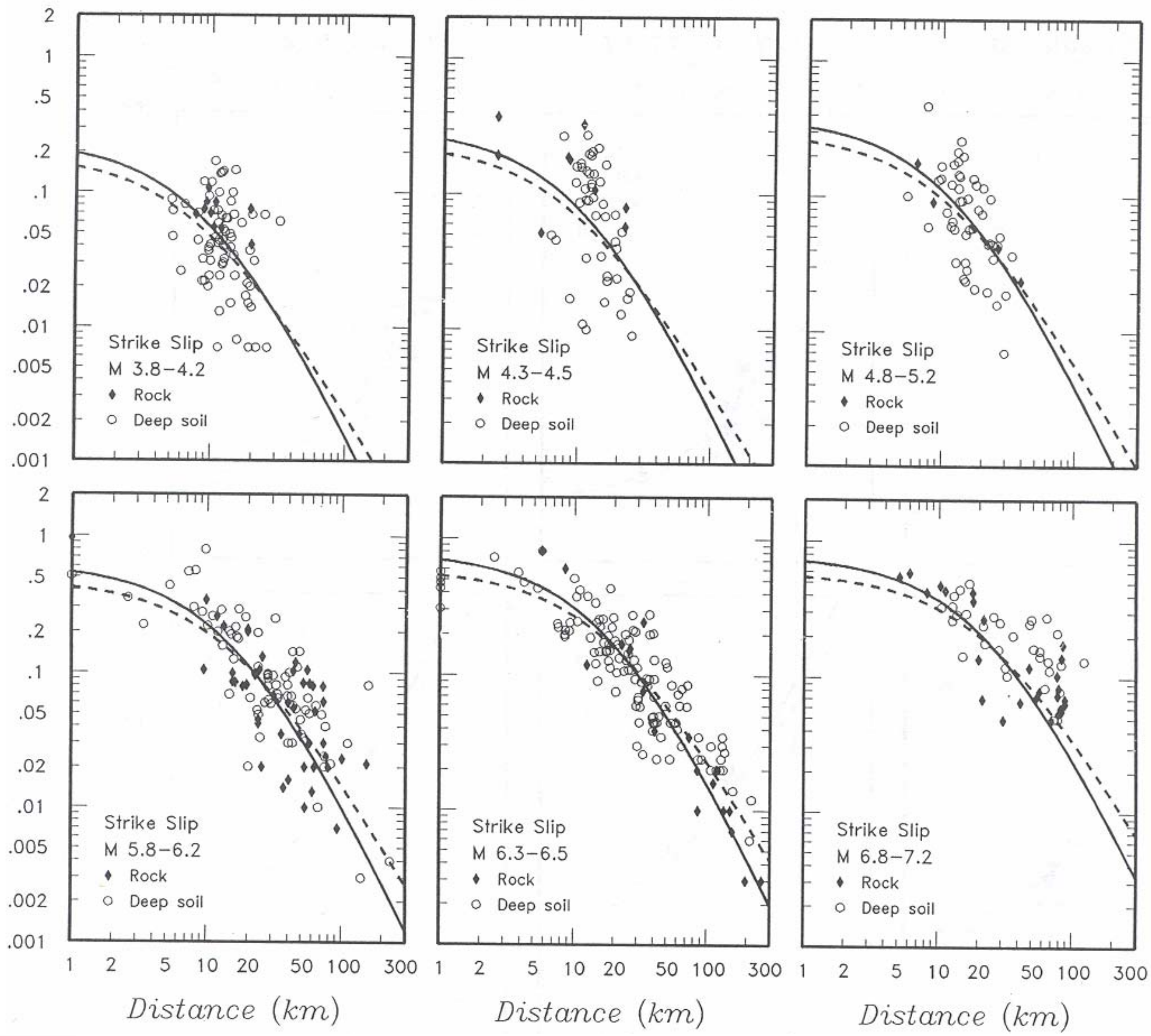
Earthquake Catalog for Shallow Crustal Earthquakes

(Sadigh, Chang, Egan, Makdisi, and Youngs)



Attenuation Relation for Shallow Crustal Earthquakes

(Sadigh, Chang, Egan, Makdisi, and Youngs)



Attenuation Relation for Shallow Crustal Earthquakes

(Sadigh, Chang, Egan, Makdisi, and Youngs)

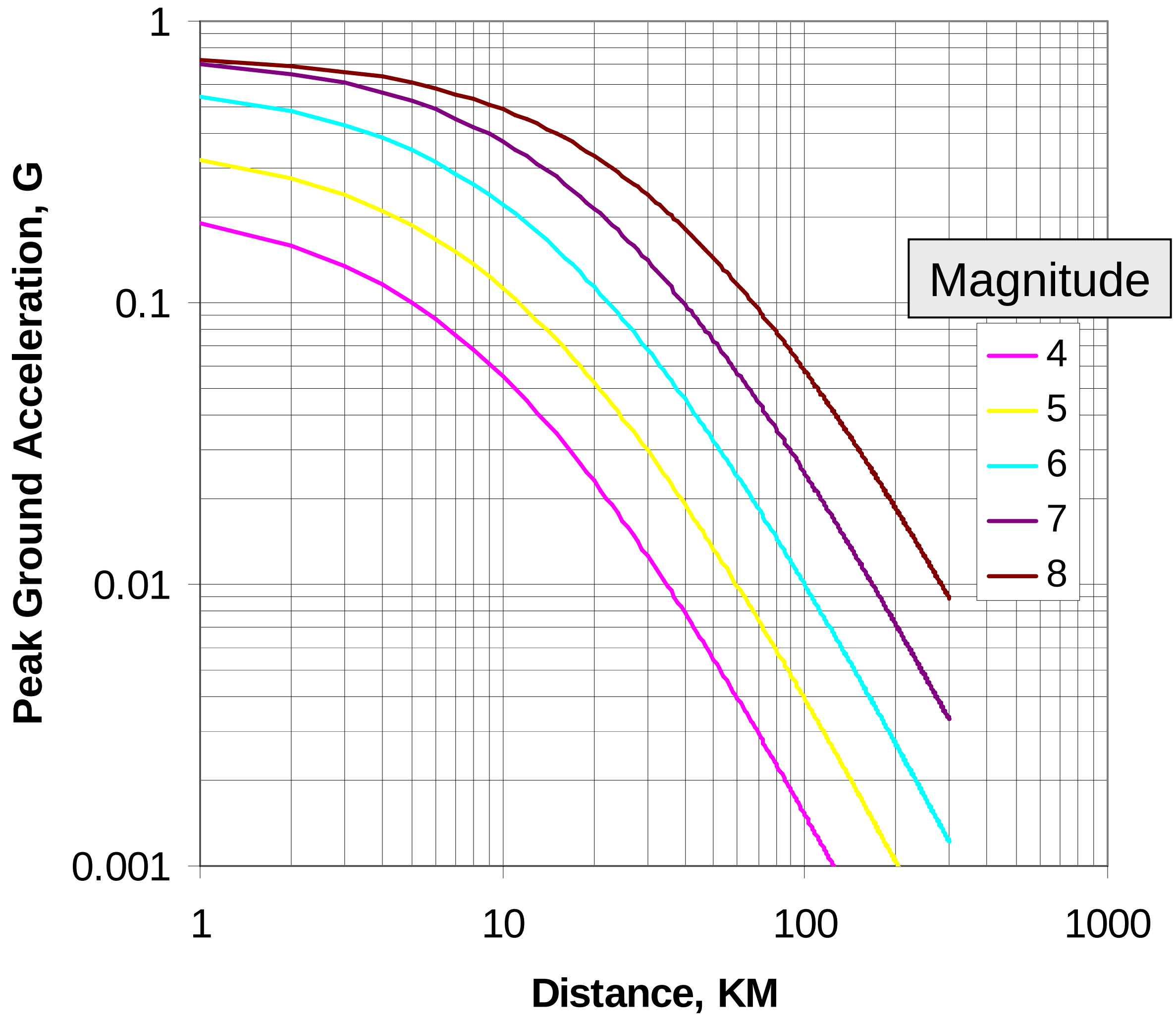
$$\ln(y) = C_1 + C_2 M + C_3(8.5 - M) + C_4 \ln(r_{rup} + \exp(C_5 + C_6 M)) + C_7(r_{rup} + 2)$$

T	C_1	C_2	C_3	C_4	C_5	C_6	C_7
PGA	-0.624	1.000	0.000	-2.100	1.296	0.250	0.000
0.07	0.110	1.000	0.006	-2.128	1.296	0.250	-0.082
0.1	0.275	1.000	0.006	-2.148	1.296	0.250	-0.041
0.2	0.153	1.000	-0.004	-2.080	1.296	0.250	0.000
0.3	-0.057	1.000	-0.017	-2.028	1.296	0.250	0.000
0.4	-0.298	1.000	-0.028	-1.990	1.296	0.250	0.000
0.5	-0.588	1.000	-0.040	-1.945	1.296	0.250	0.000
0.75	-1.208	1.000	-0.050	-1.865	1.296	0.250	0.000
1	-1.705	1.000	-0.055	-1.800	1.296	0.250	0.000
1.5	-2.407	1.000	-0.065	-1.725	1.296	0.250	0.000
2	-2.945	1.000	-0.070	-1.670	1.296	0.250	0.000
3	-3.700	1.000	-0.080	-1.610	1.296	0.250	0.000
4	-4.230	1.000	-0.100	-1.570	1.296	0.250	0.000

Table for Magnitude <= 6.5

Attenuation Relation for Shallow Crustal Earthquakes

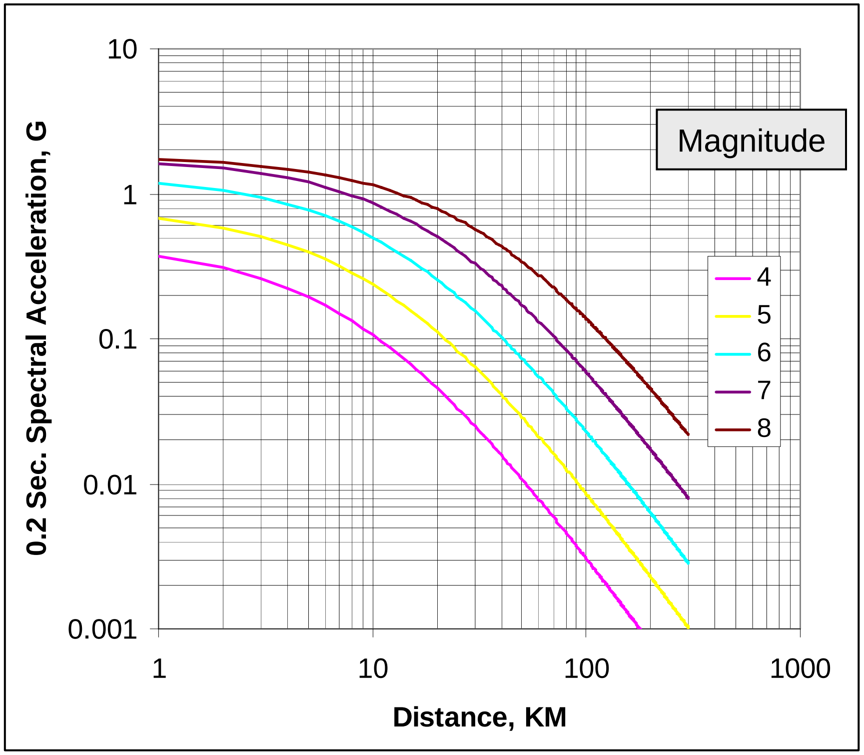
(Sadigh, Chang, Egan, Makdisi, and Youngs)



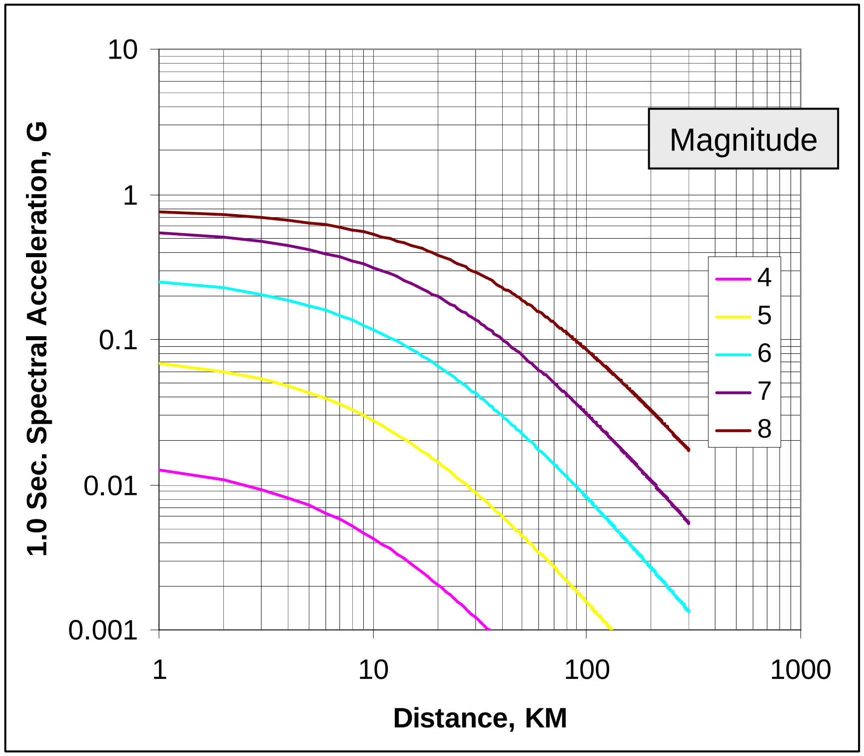
Attenuation Relation for Shallow Crustal Earthquakes

(Sadigh, Chang, Egan, Makdisi, and Youngs)

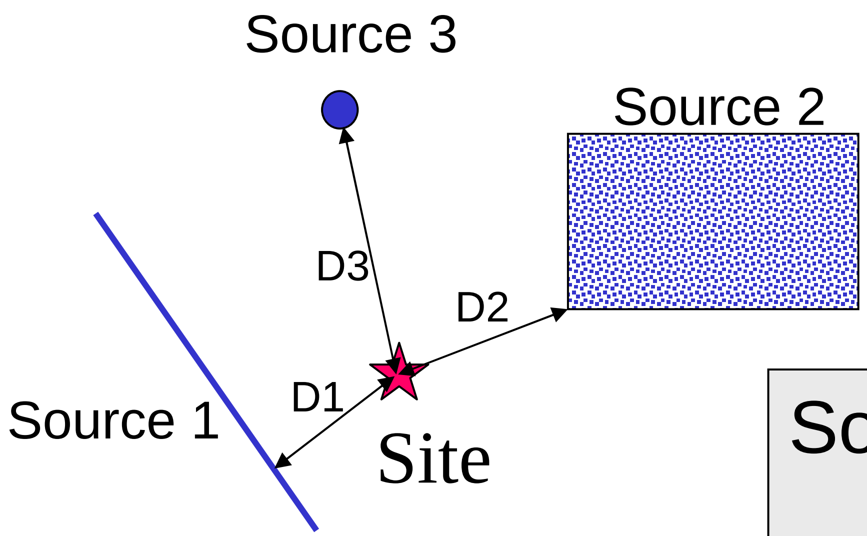
0.2 Second Acceleration



1.0 Second Acceleration



Example Deterministic Analysis (Kramer)



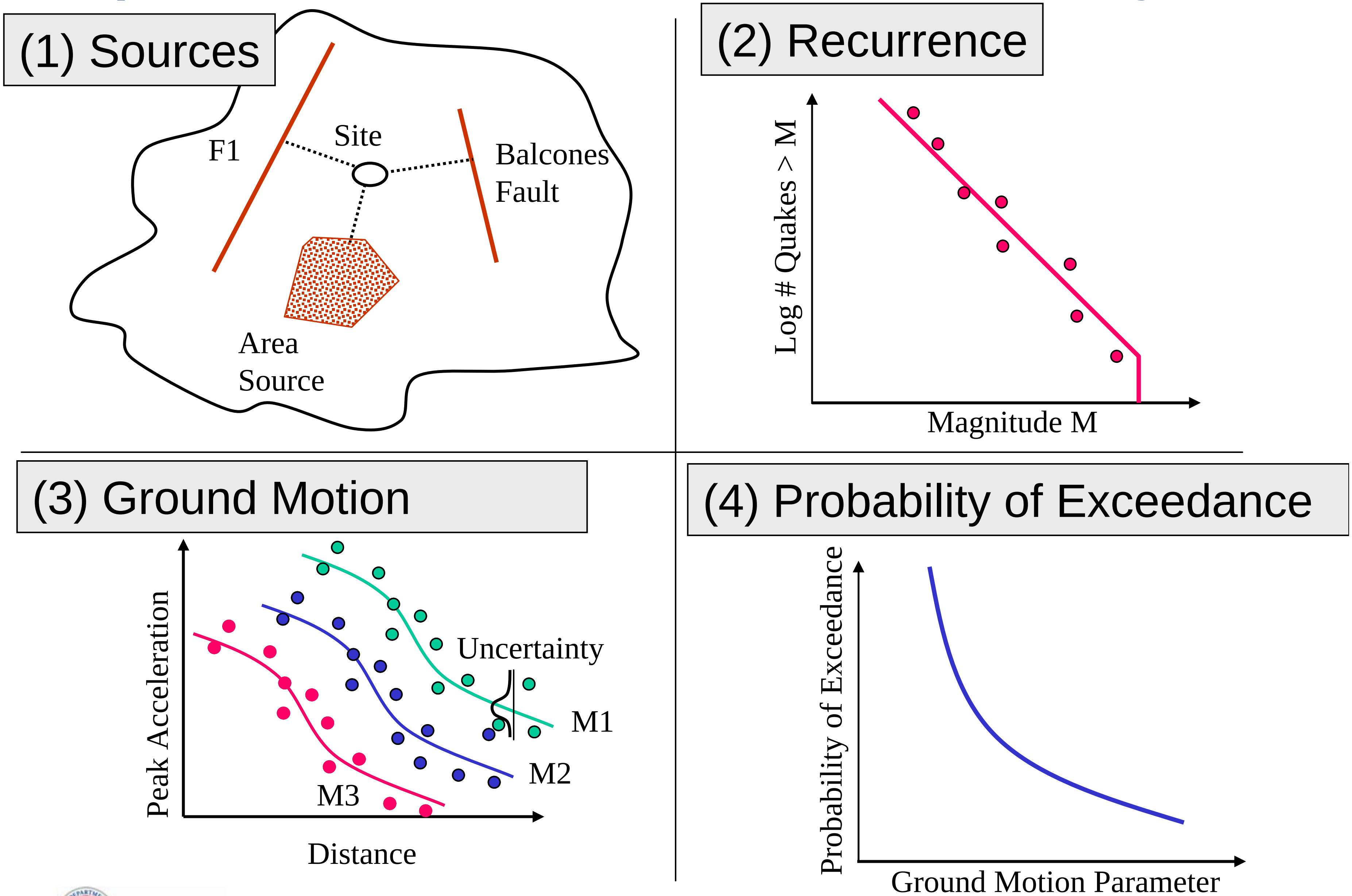
Source	M	D (km)	PGA (g)
1	7.3	23.7	0.42
2	7.7	25.0	0.57
3	5.0	60.0	0.02

Maximum on source

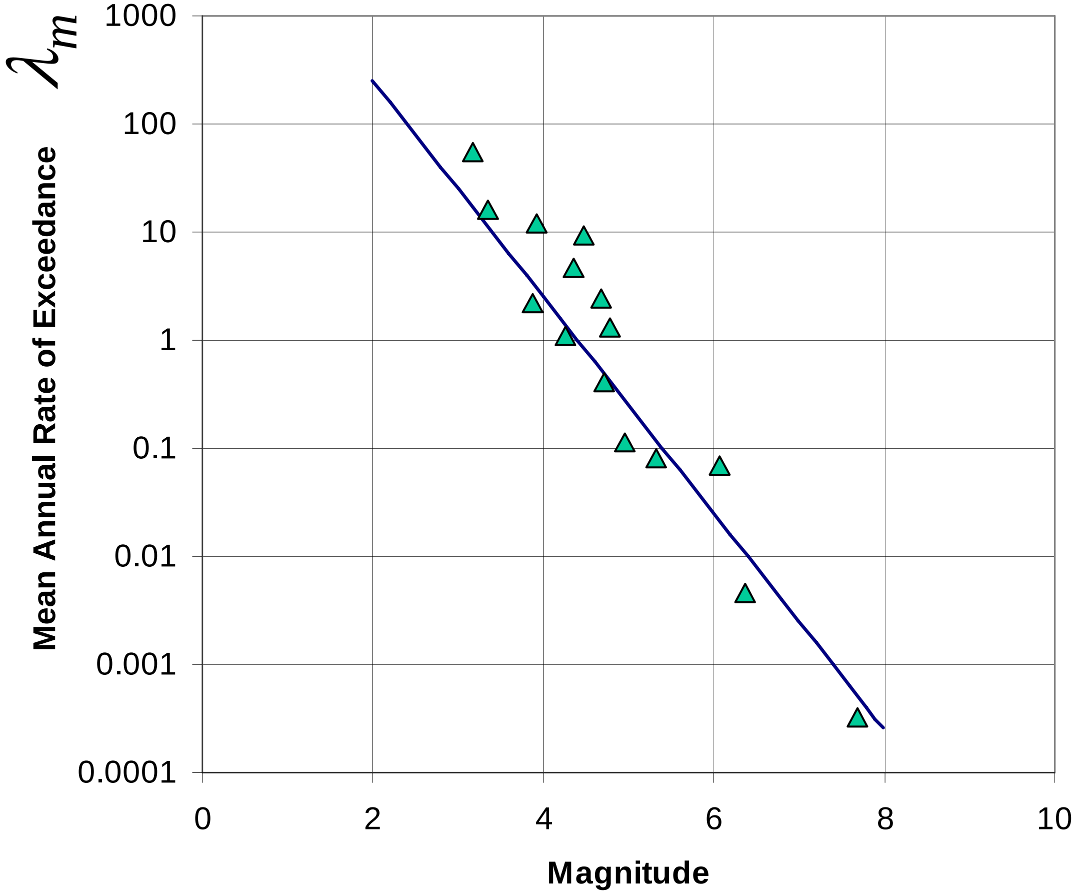
Closest distance

From attenuation relationship

Steps in Probabilistic Seismic Hazard Analysis



Empirical Gutenberg-Richter Recurrence Relationship



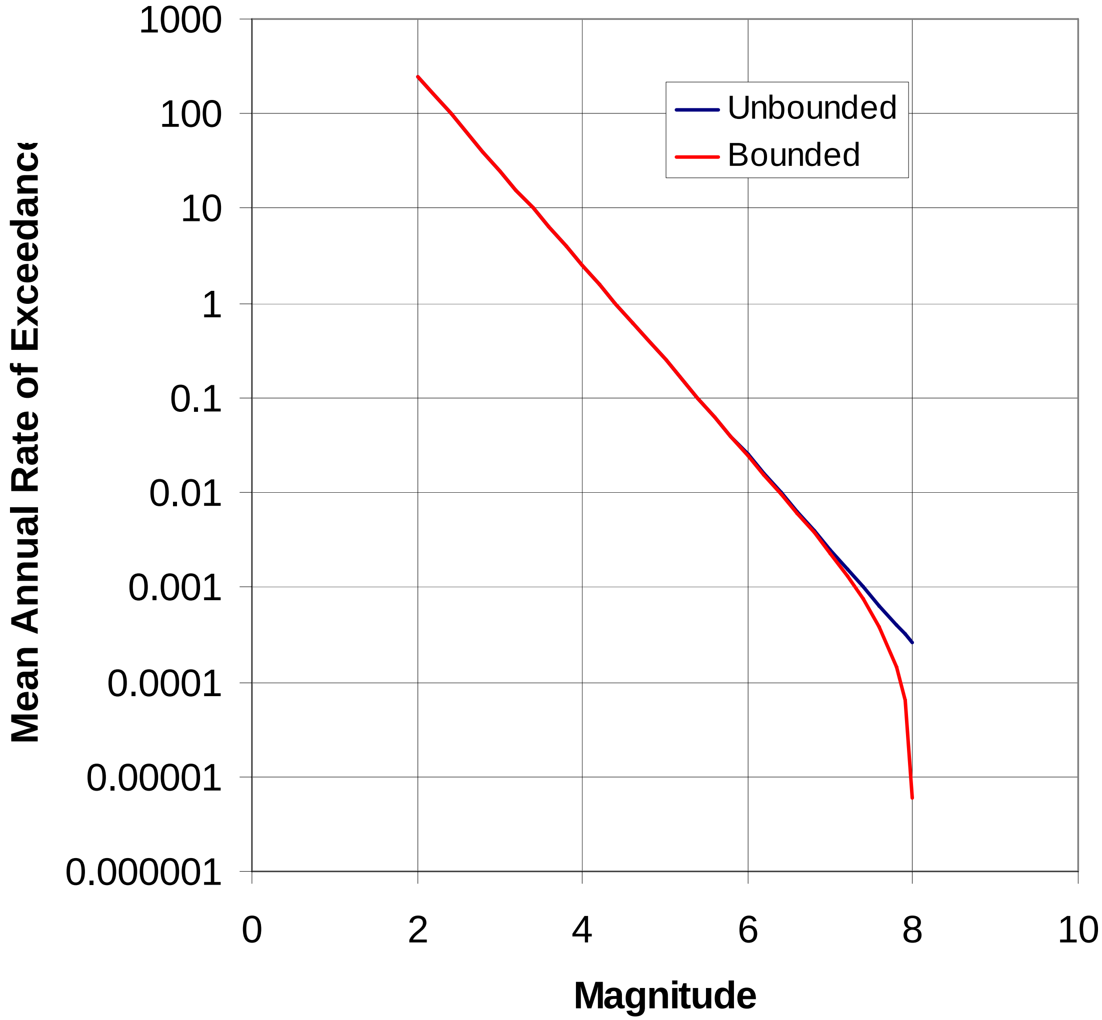
$$\boxed{\log \lambda_m = a - bm}$$

λ_m = mean rate of recurrence (events/year)

$1/\lambda_m$ = return period

a and b to be determined from data

Bounded vs Unbounded Recurrence Relationship



Uncertainties Included in Probabilistic Analysis

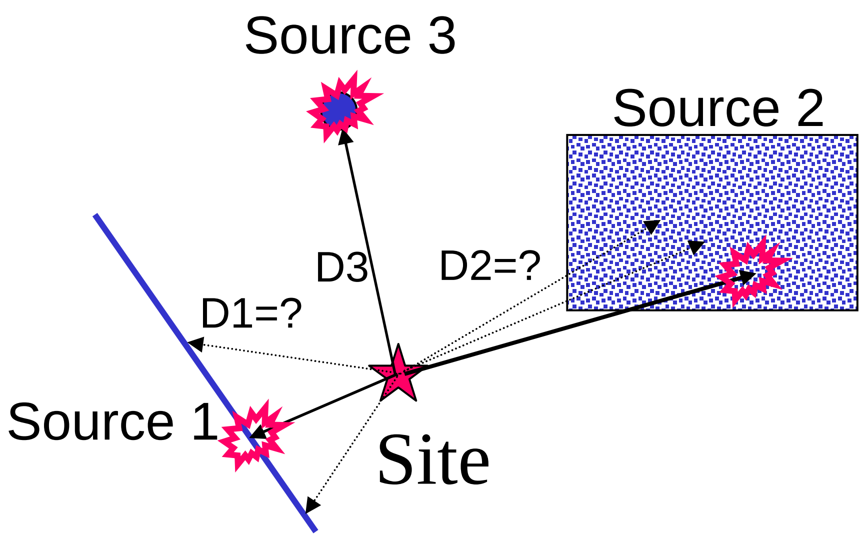
Attenuation laws

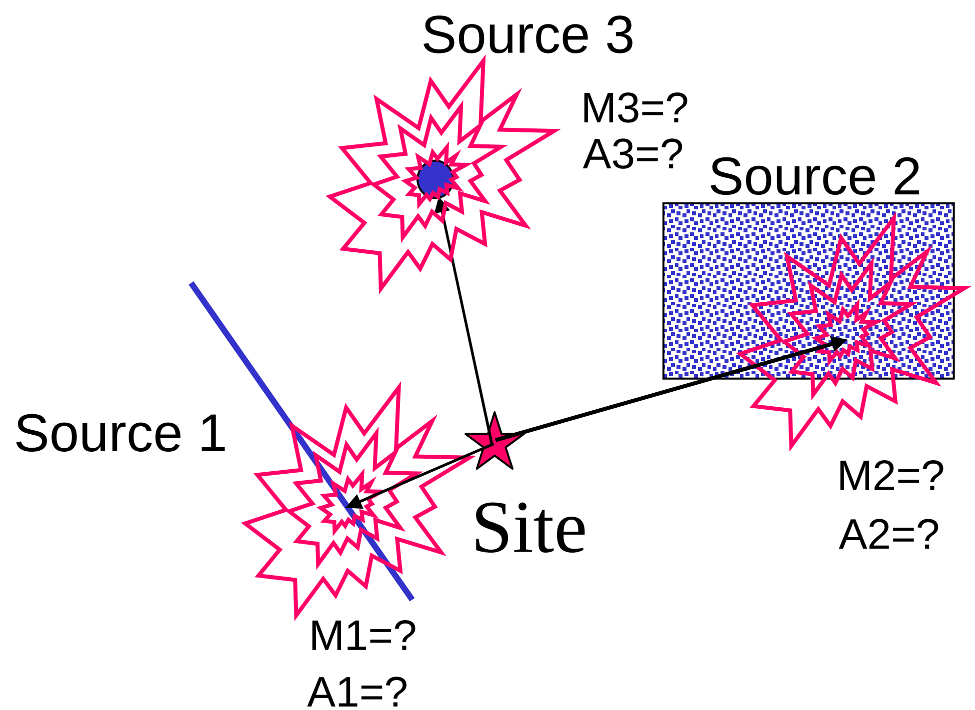
Recurrence relationship

Distance to site

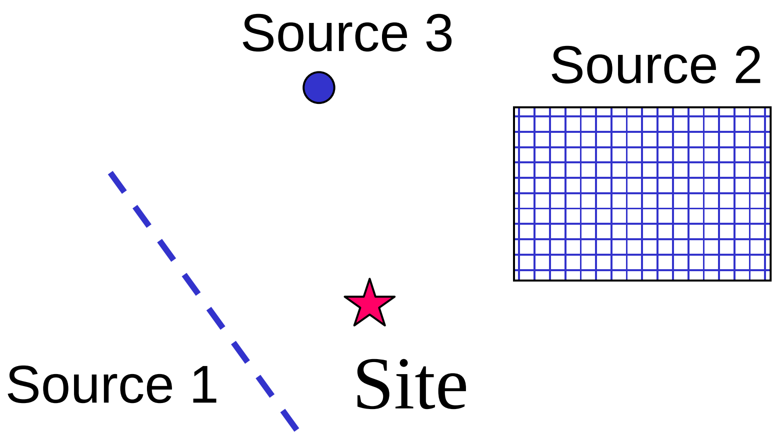
$$\lambda_{y*} = \sum_{i=1}^{N_S} \sum_{j=1}^{N_M} \sum_{k=1}^{N_R} v_i \, P[Y > y* \mid m_j, r_k] \, P[M = m_j] \, P[R = r_k]$$

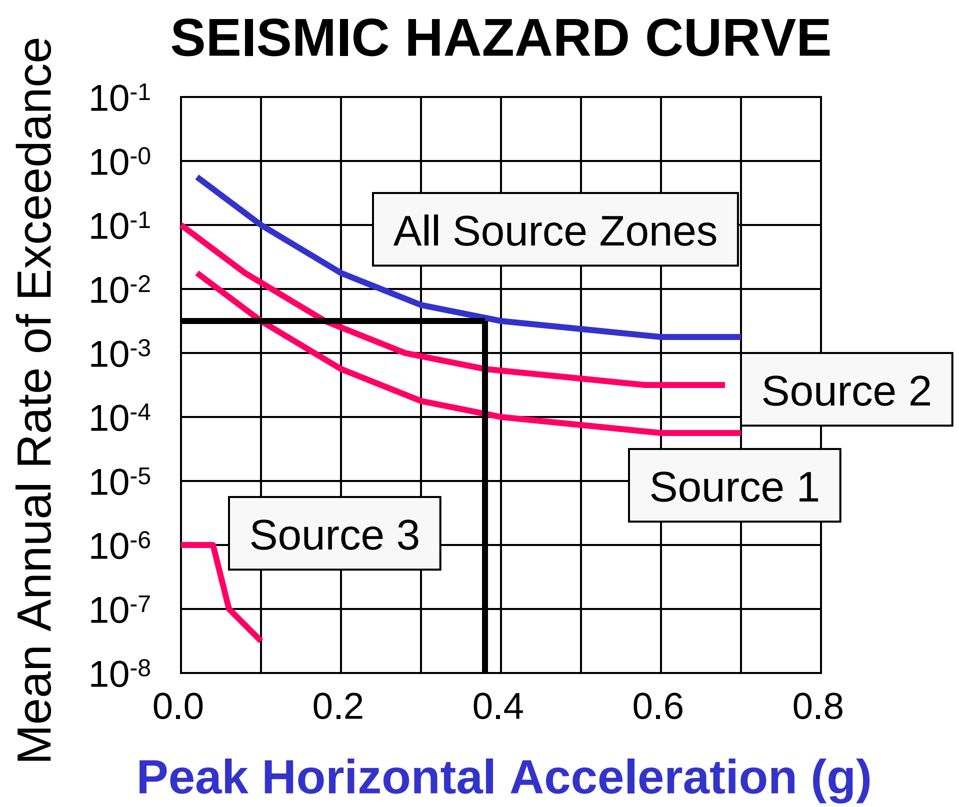
Example Probabilistic Analysis (Kramer)





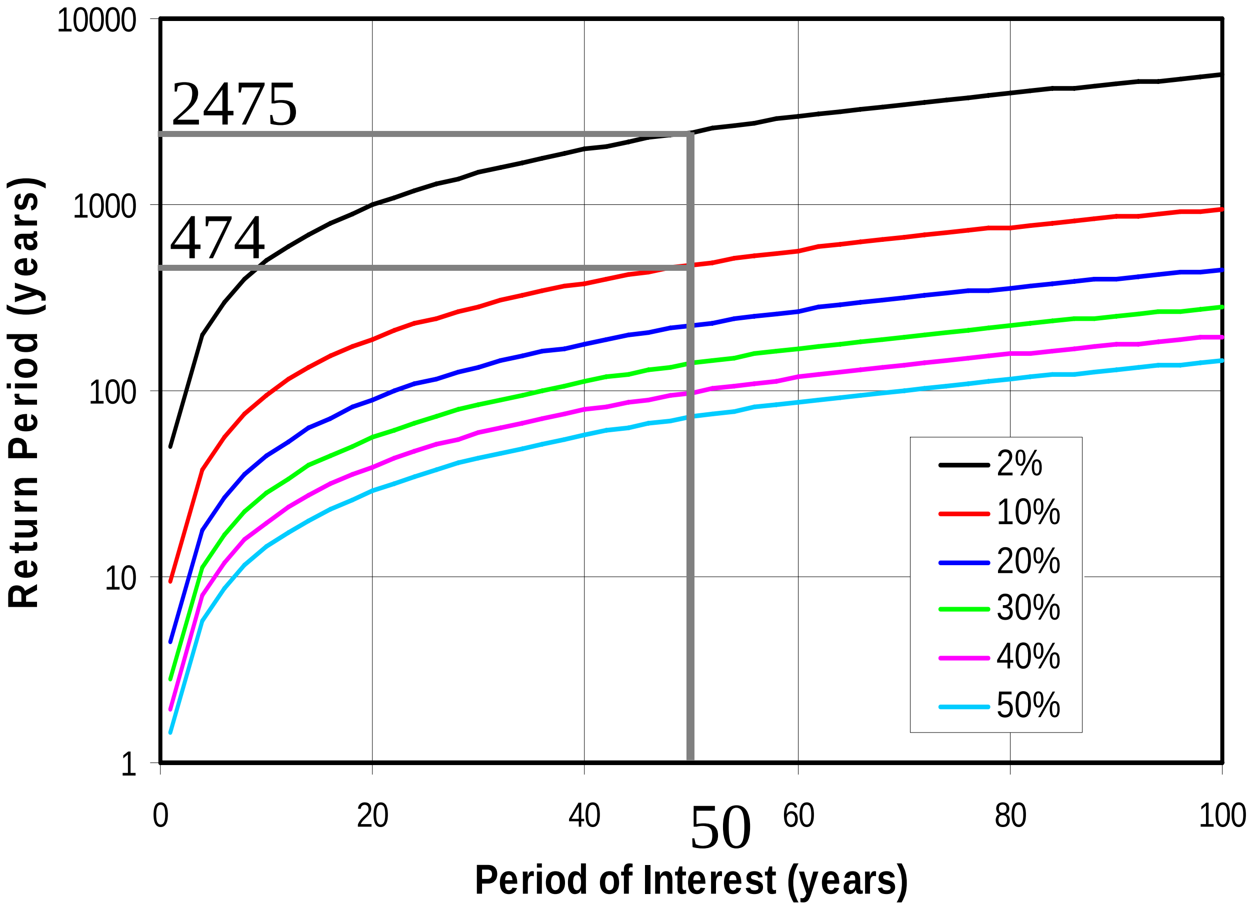
Result of Probabilistic Hazard Analysis



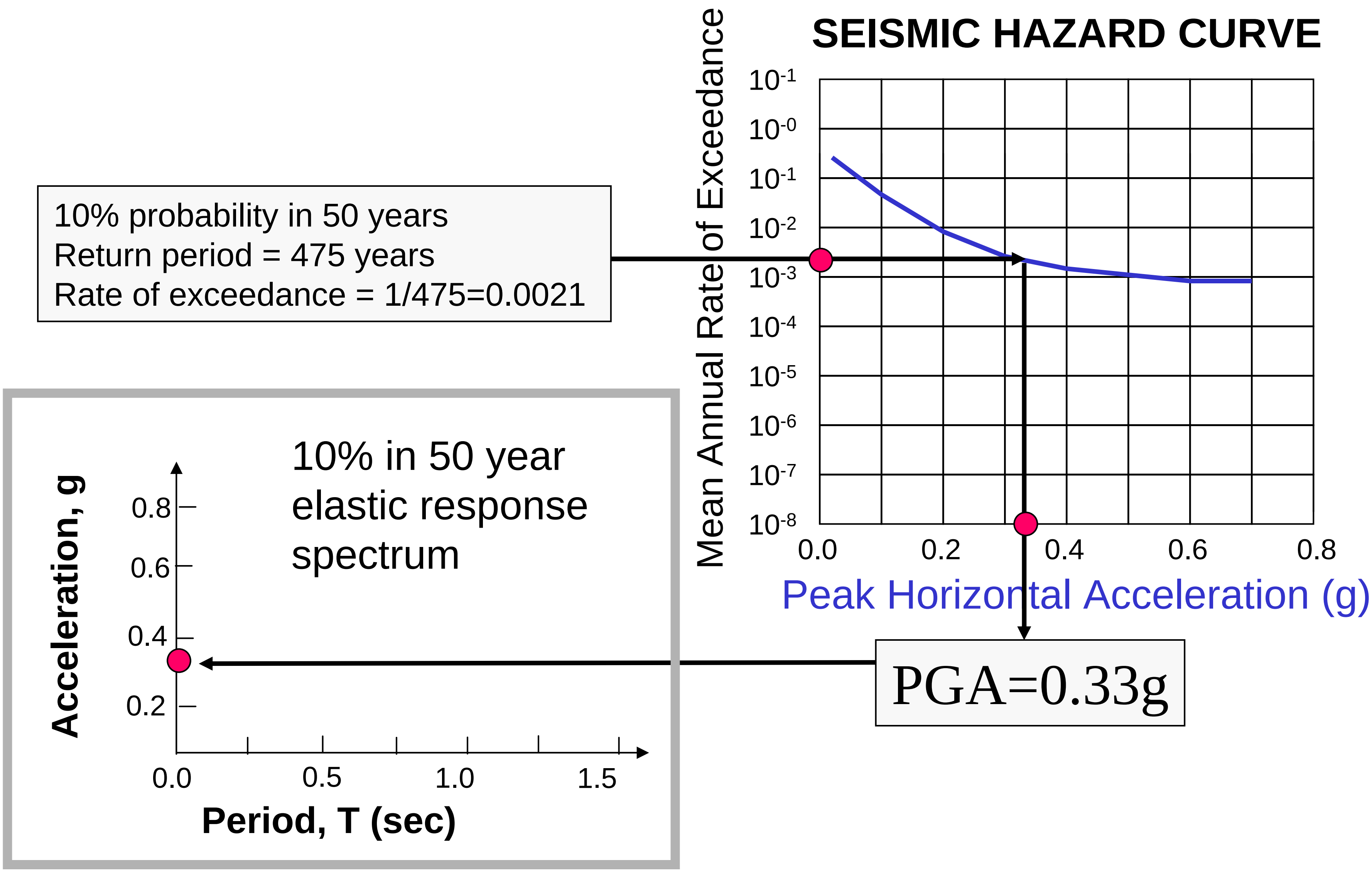


Relationship Between Return Period, Period of Interest, and Probability of Exceedance

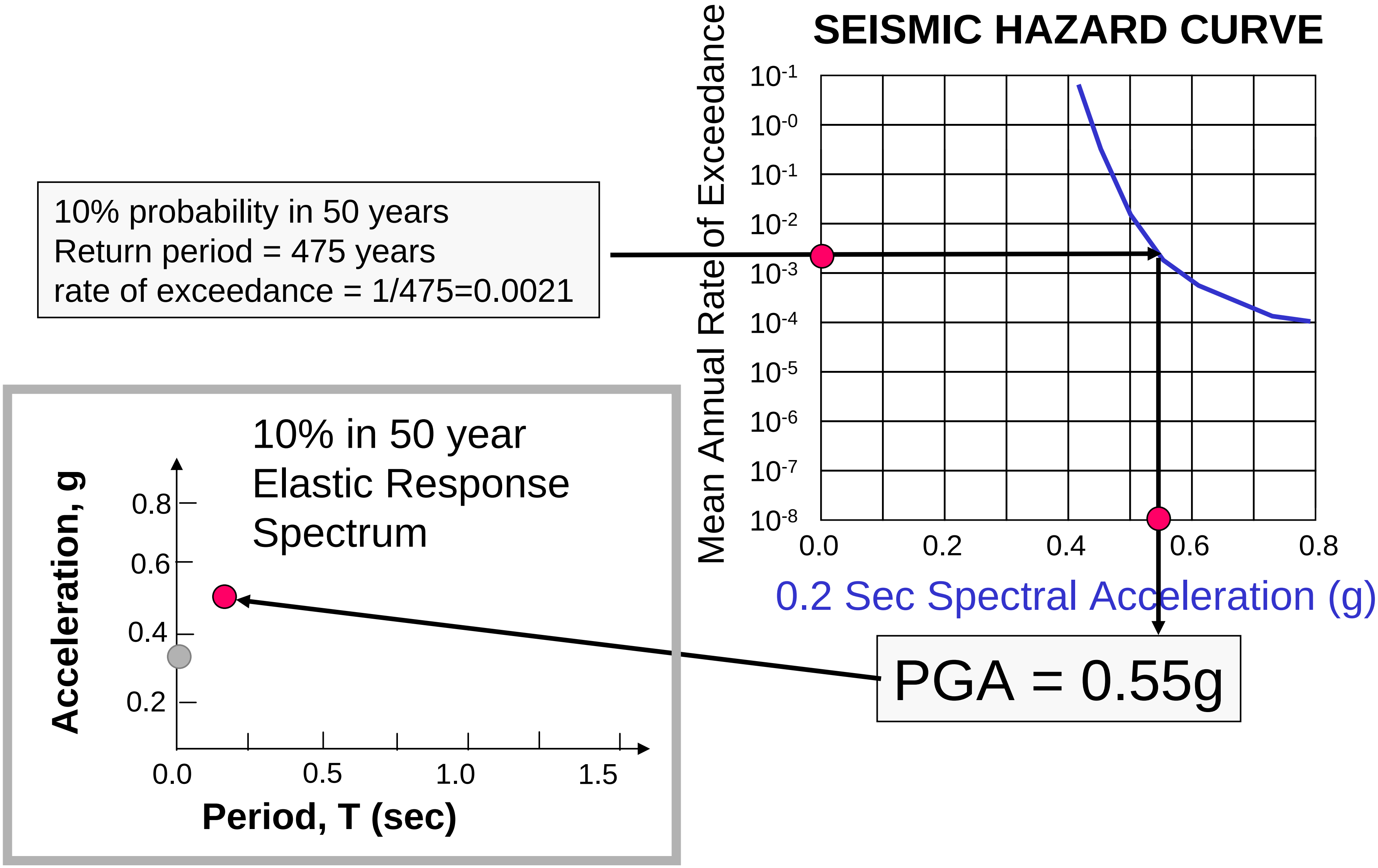
$$\text{Return period} = -T/\ln(1-P(Z>z))$$



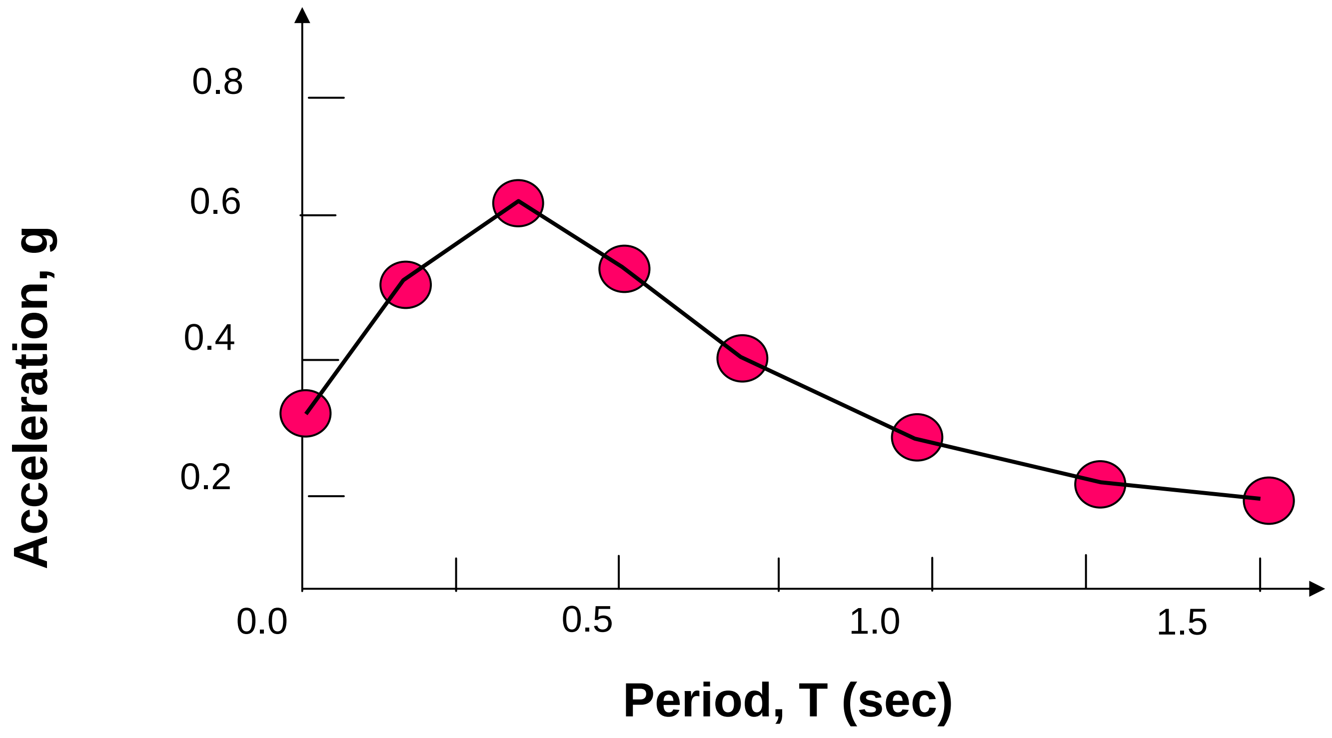
Use of PGA Seismic Hazard Curve



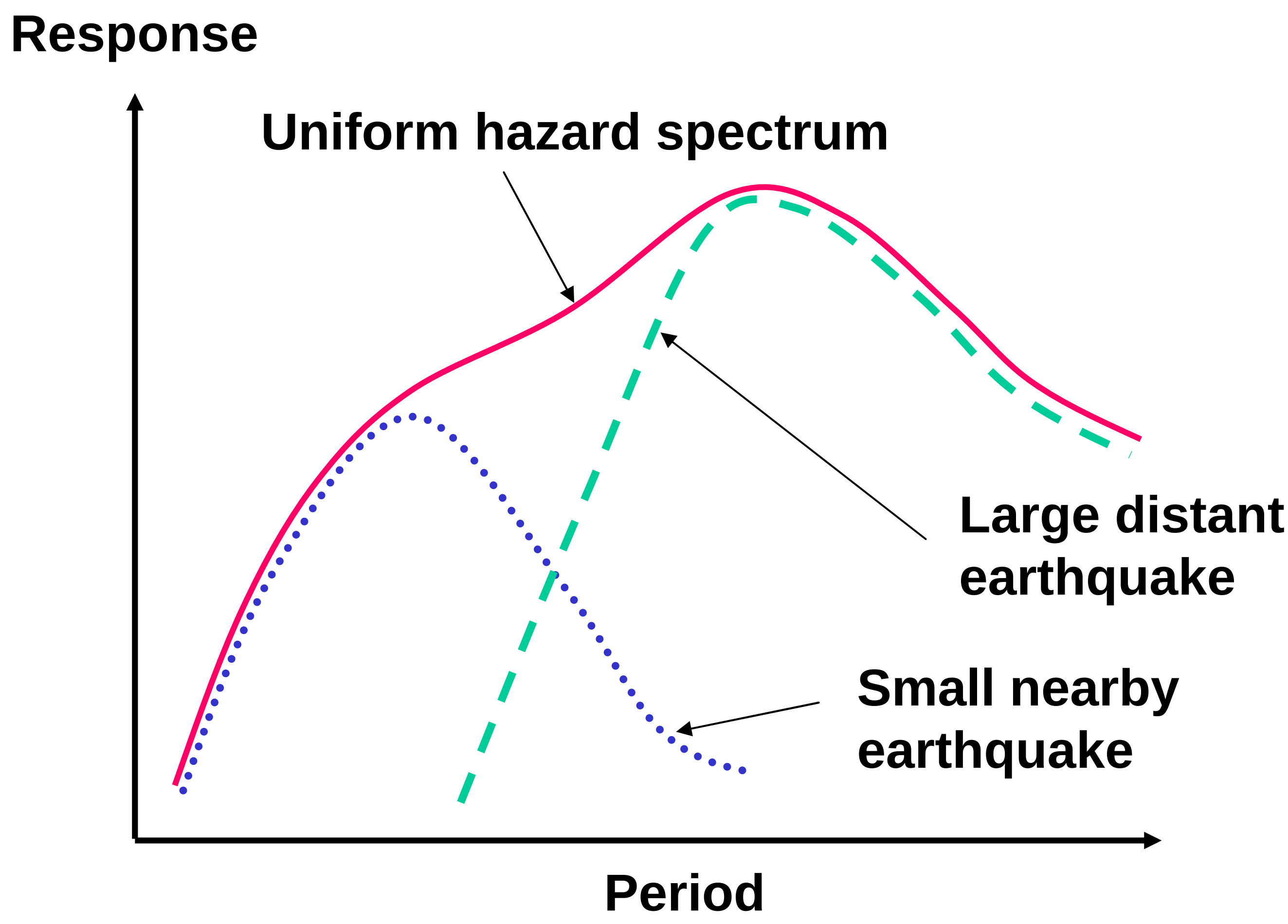
Use of 0.2 Sec. Seismic Hazard Curve



10% in 50 Year Elastic Response Spectrum



Uniform Hazard Spectrum



Uniform Hazard Spectrum

Developed from *probabilistic* analysis

All ordinates have equal probability of exceedance

Represents contributions from small local, large distant earthquakes

May be overly conservative for modal response spectrum analysis

May not be appropriate for artificial ground motion generation

Probabilistic vs Deterministic Seismic Hazard Analysis

"The *deterministic* approach provides a clear and trackable method of computing seismic hazard whose assumptions are easily discerned. It provides understandable scenarios that can be related to the problem at hand."

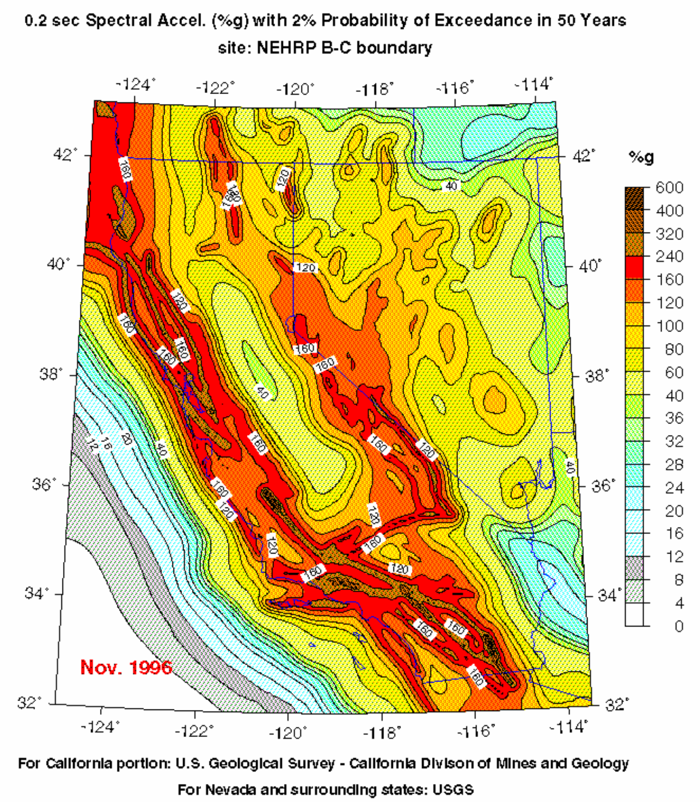
"However, it has no way for accounting for uncertainty. Conclusions based on deterministic analysis can easily be upset by the occurrence of new earthquakes."

Probabilistic vs Deterministic Seismic Hazard Analysis

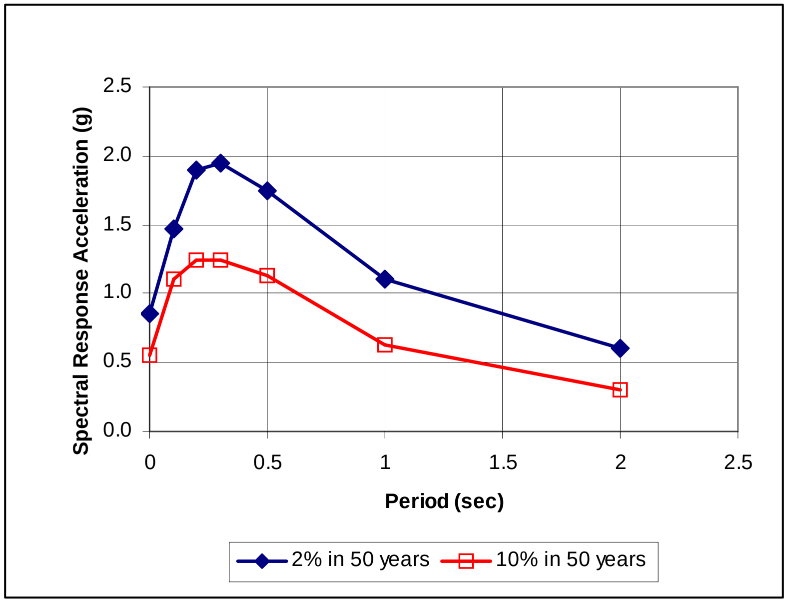
"The *probabilistic* approach is capable of integrating a wide range of information and uncertainties into a flexible framework."

"Unfortunately, its highly integrated framework can obscure those elements which drive the results, and its highly quantitative nature can lead to false impressions of accuracy."

USGS Probabilistic Hazard Maps (Project 97)



HAZARD MAP



RESPONSE SPECTRA

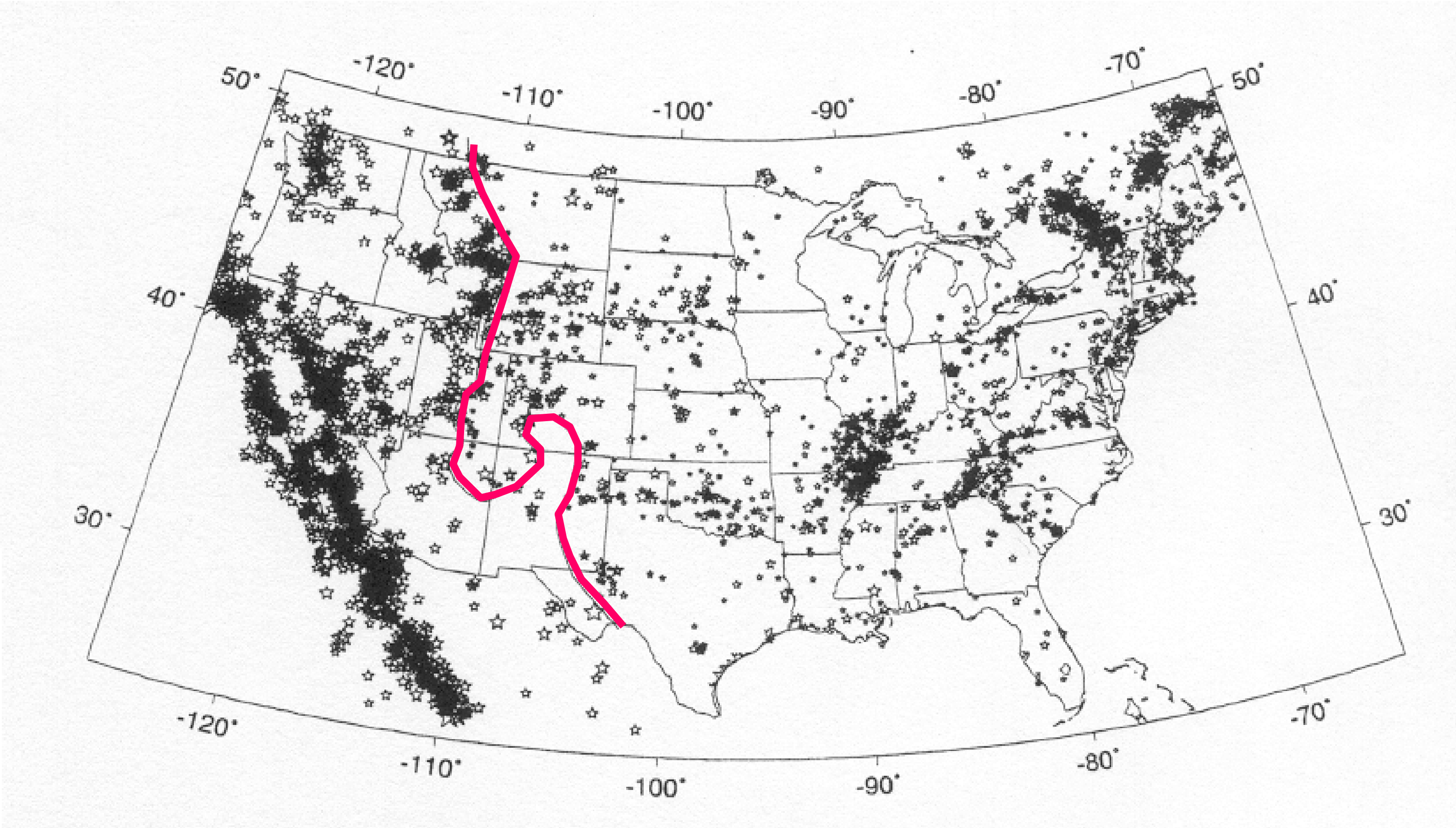
USGS Probabilistic Hazard Maps (and *NEHRP Provisions* Maps)

Earthquake Spectra, Seismic Design Provisions and Guidelines Theme Issue, Volume 16, Number 1, February 2000

Maximum Considered Earthquake (MCE)

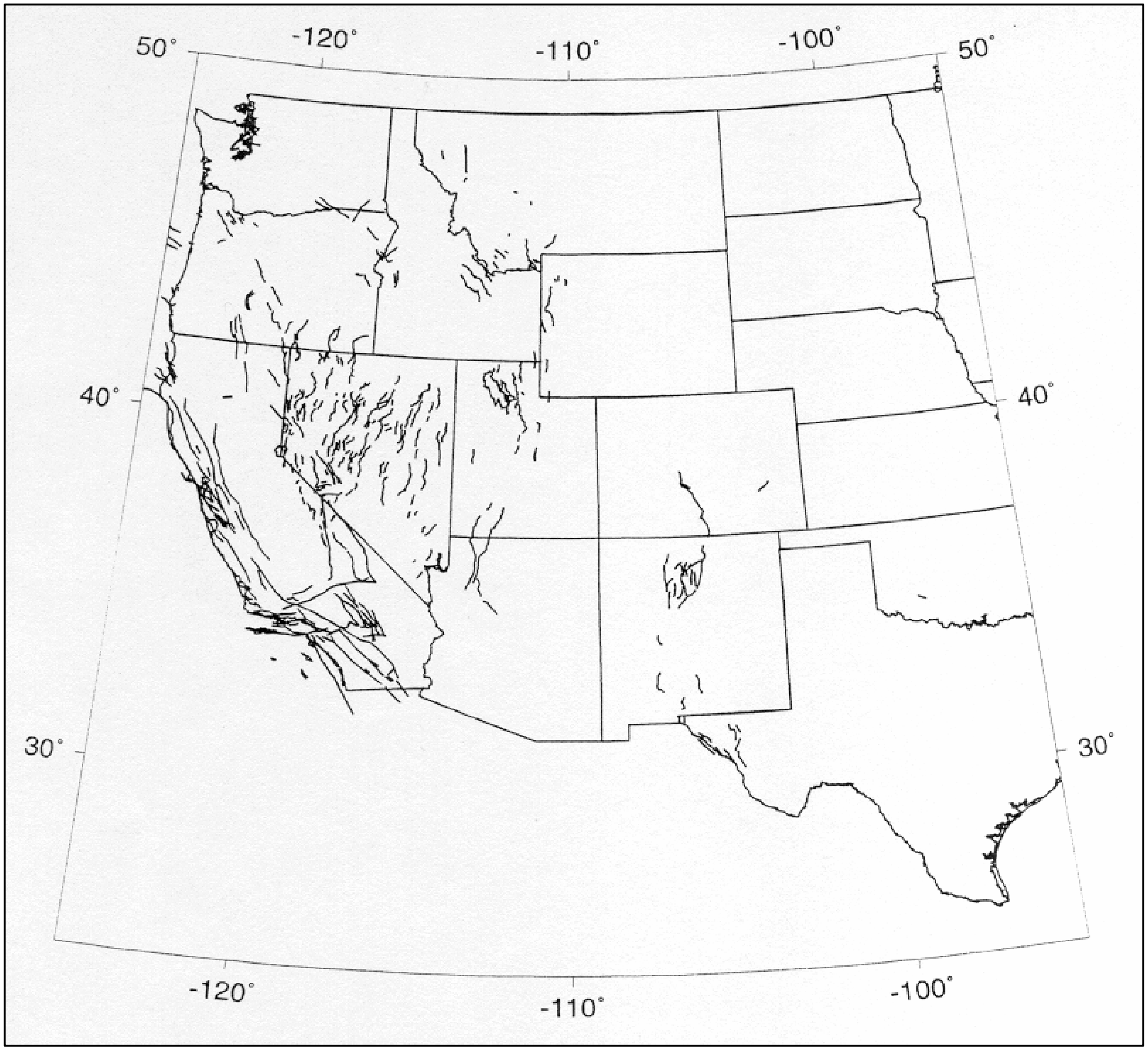
The MCE ground motions are defined as the maximum level of earthquake shaking that is considered as reasonable to design normal structures to resist.

USGS Seismic Hazard Regions

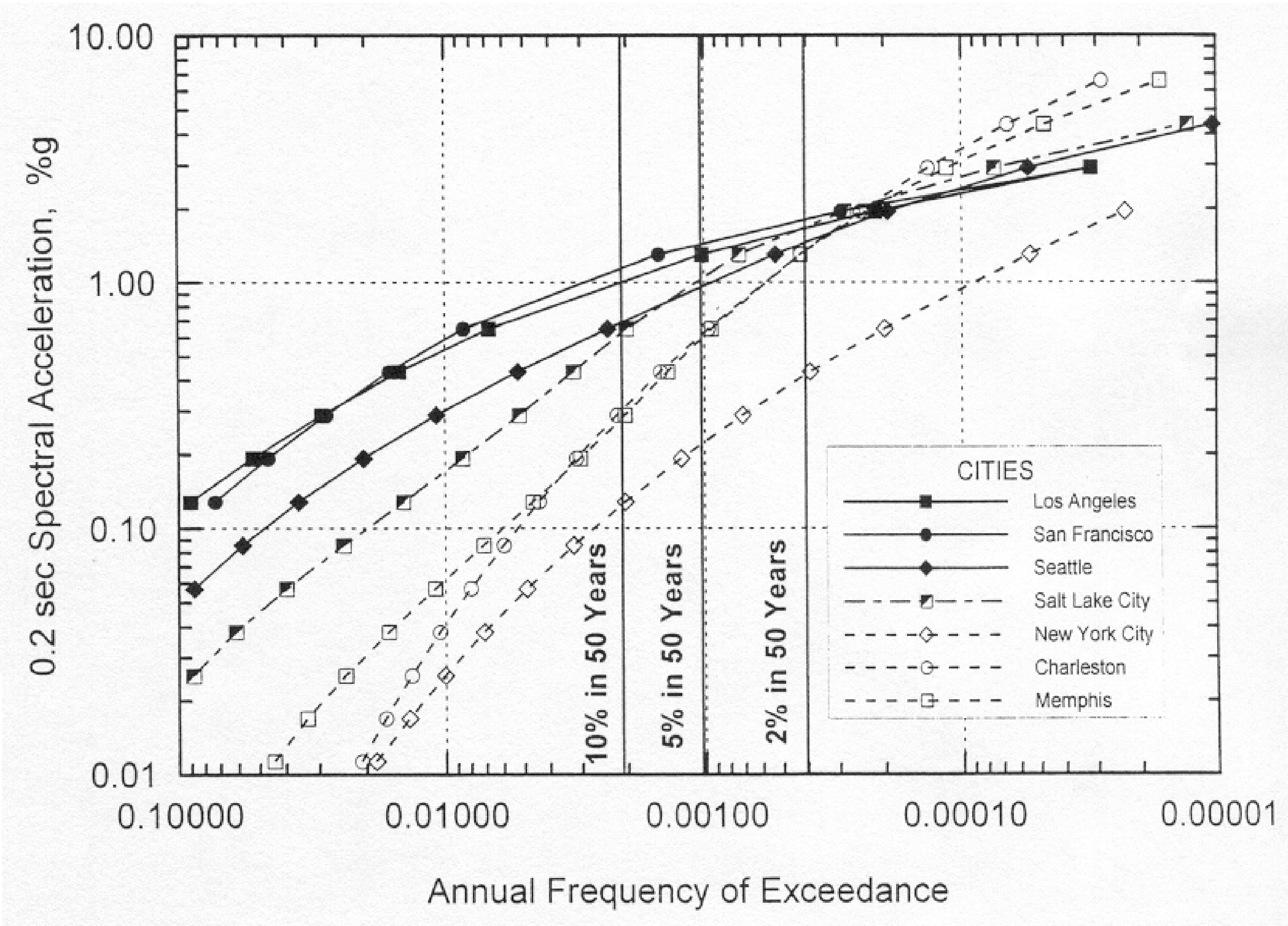


Note: Different attenuation relationships used for different regions.

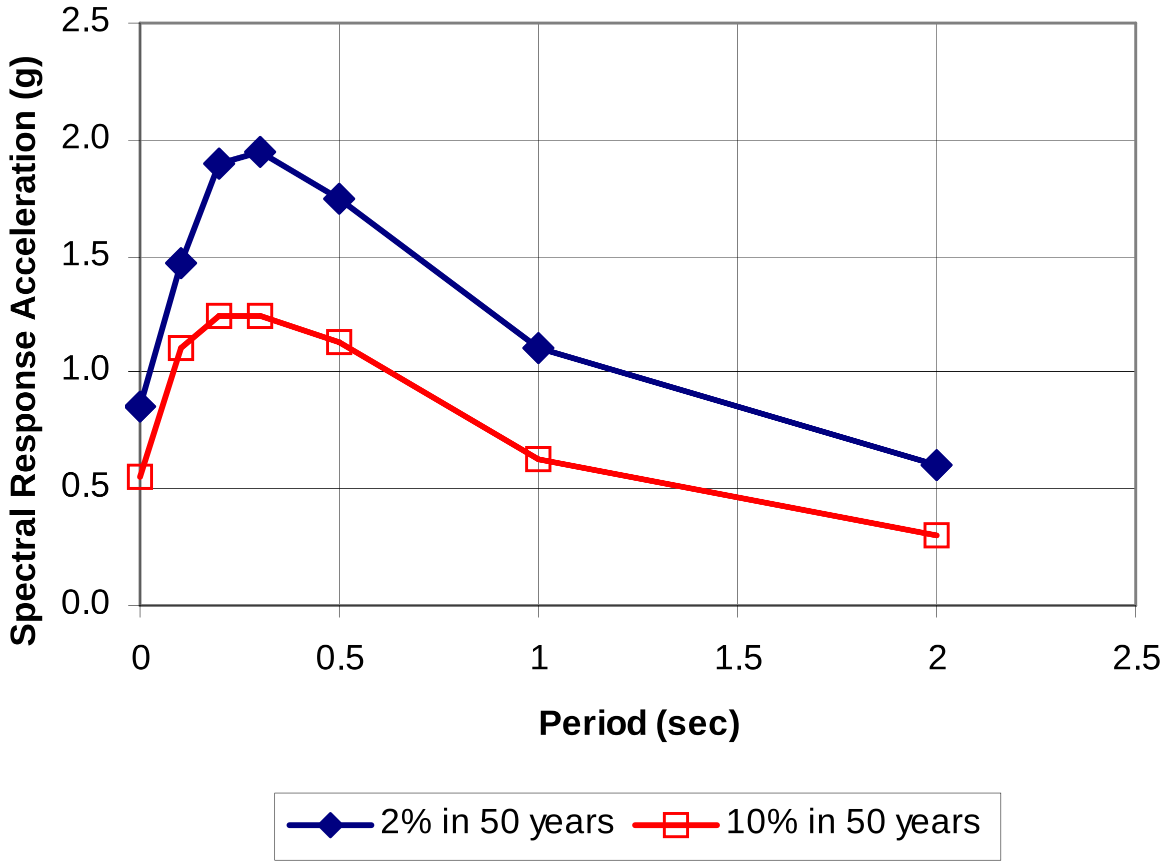
USGS Seismic Hazard WUS Faults



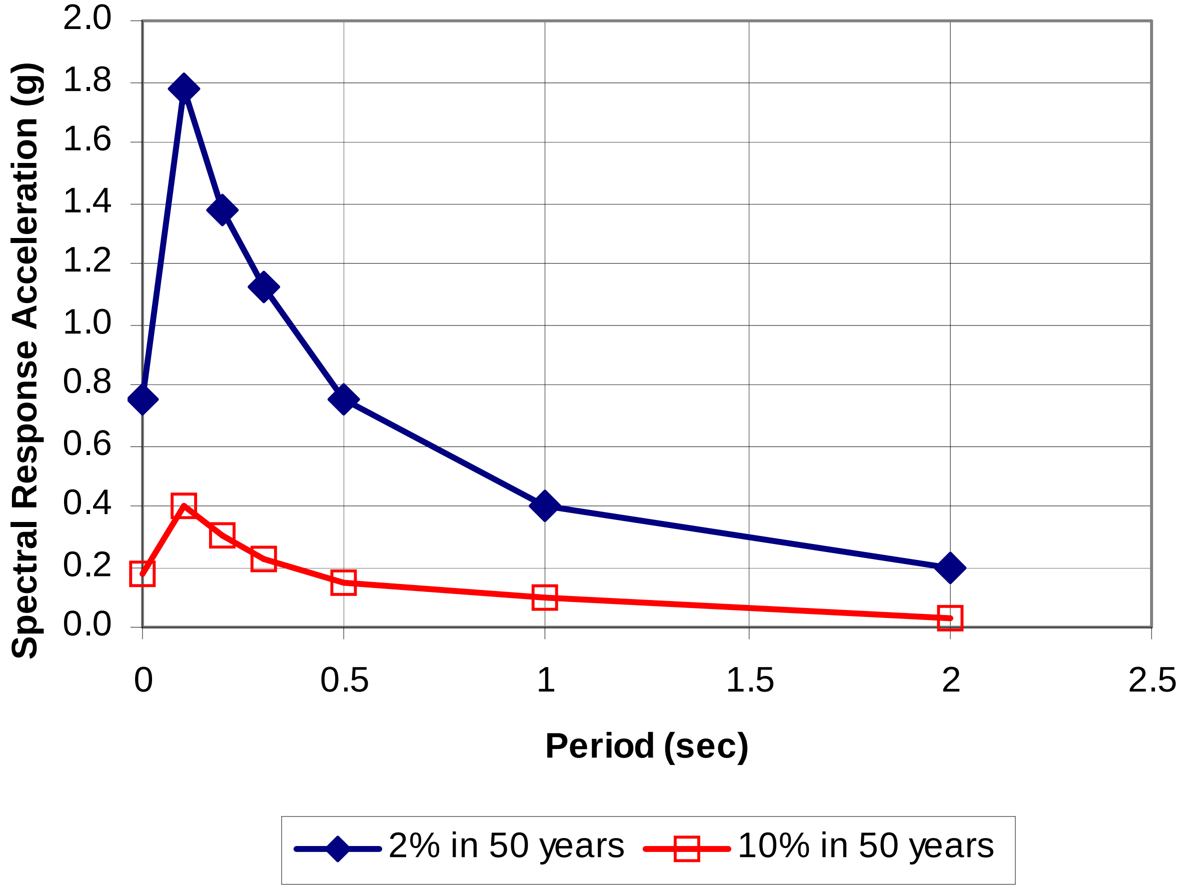
USGS Seismic Hazard Curves for Various Cities



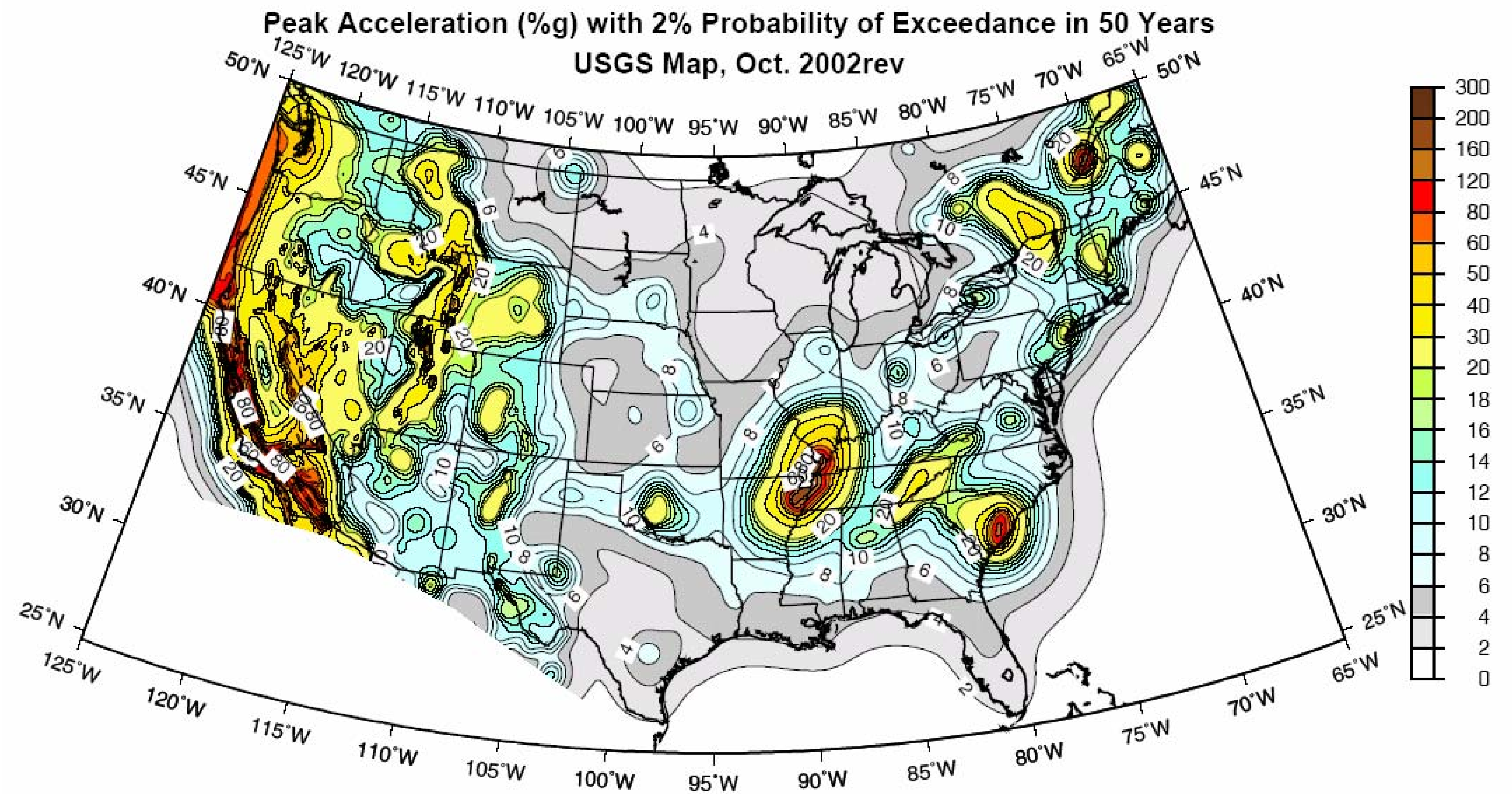
Uniform Hazard Spectra for San Francisco



Uniform Hazard Spectra for Charleston, SC

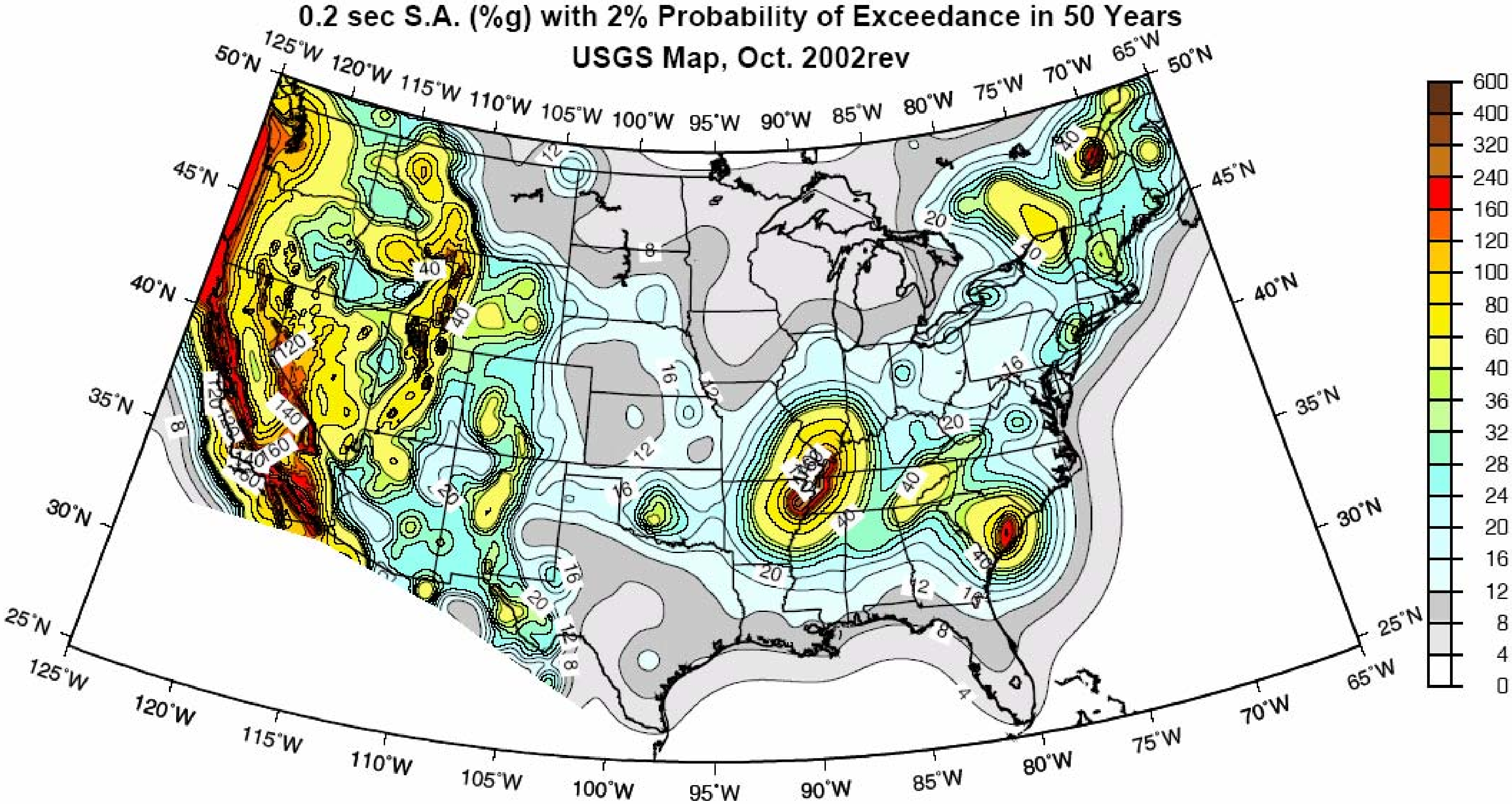


USGS Seismic Hazard Map of Coterminous United States

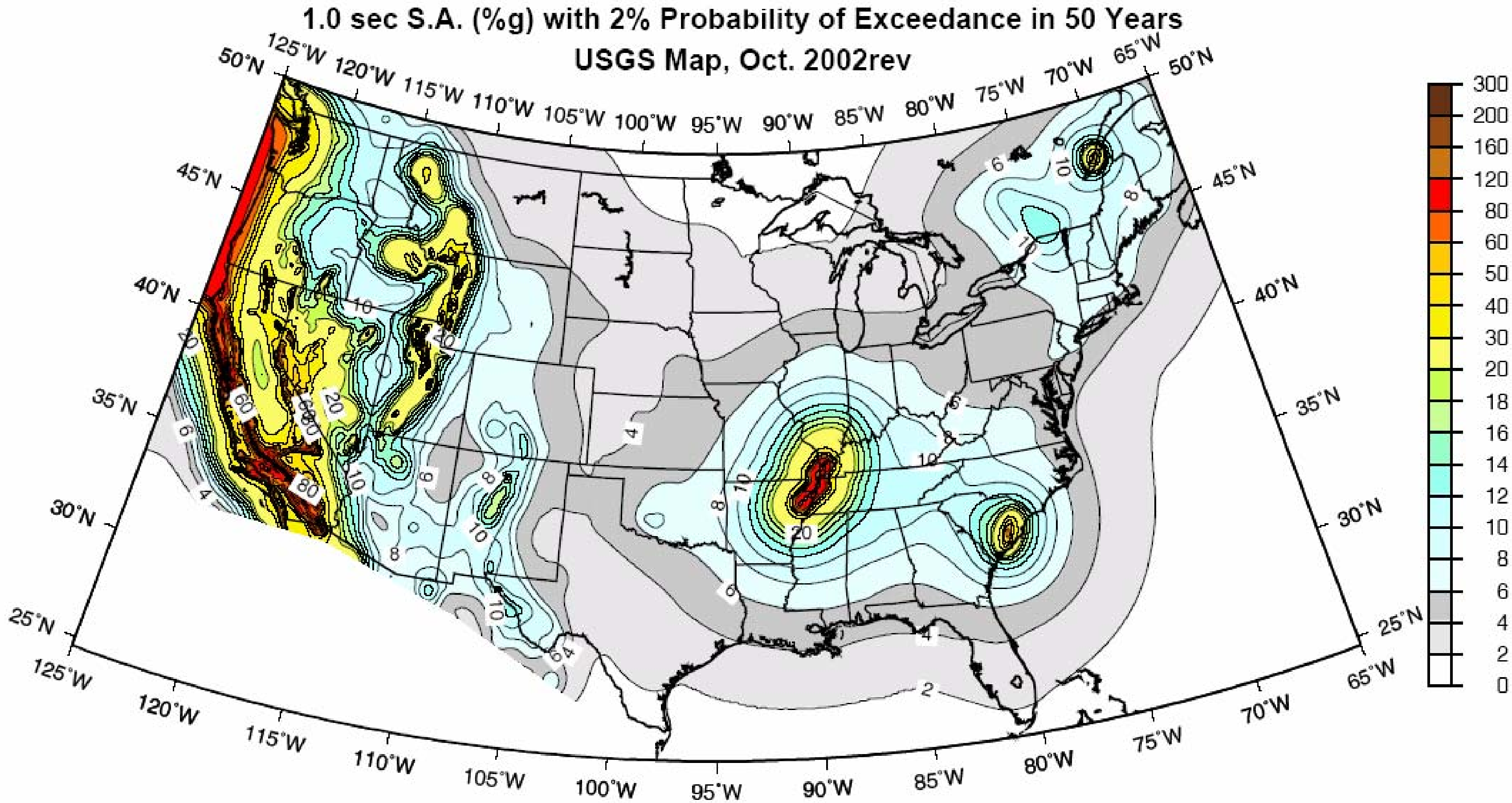


http://earthquake.usgs.gov/hazmaps/

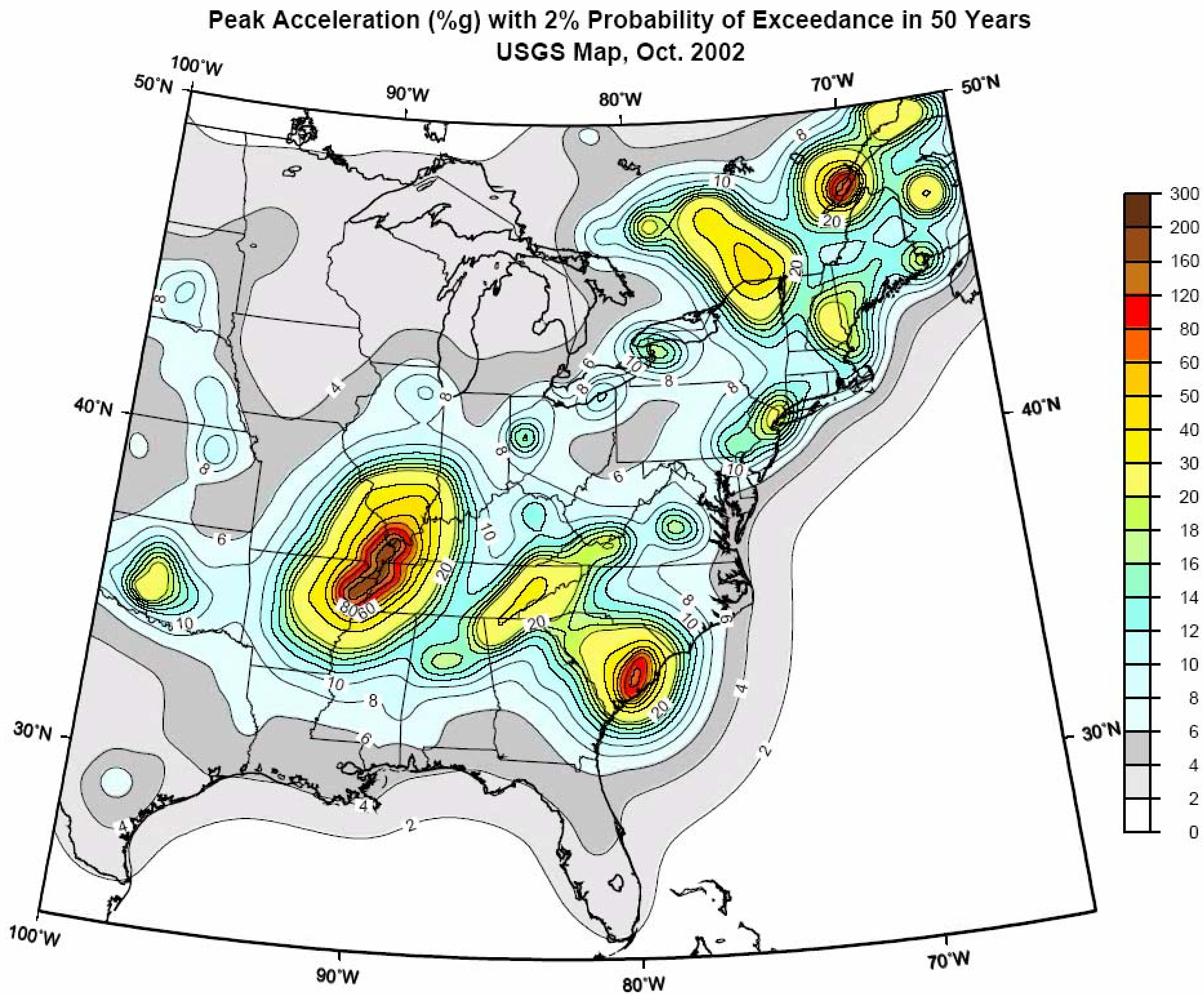
USGS Seismic Hazard Map of Coterminous United States



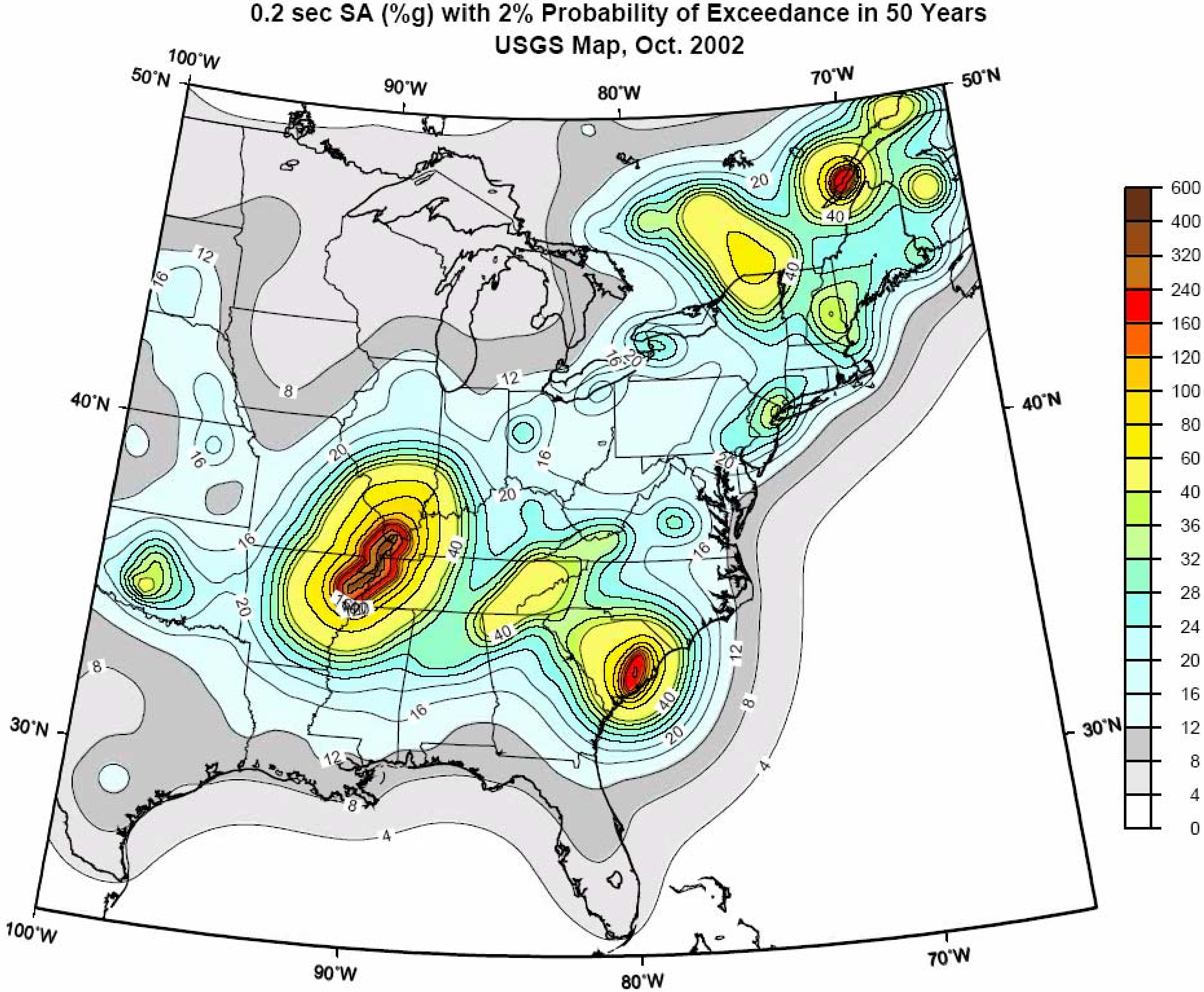
USGS Seismic Hazard Map for Coterminous United States



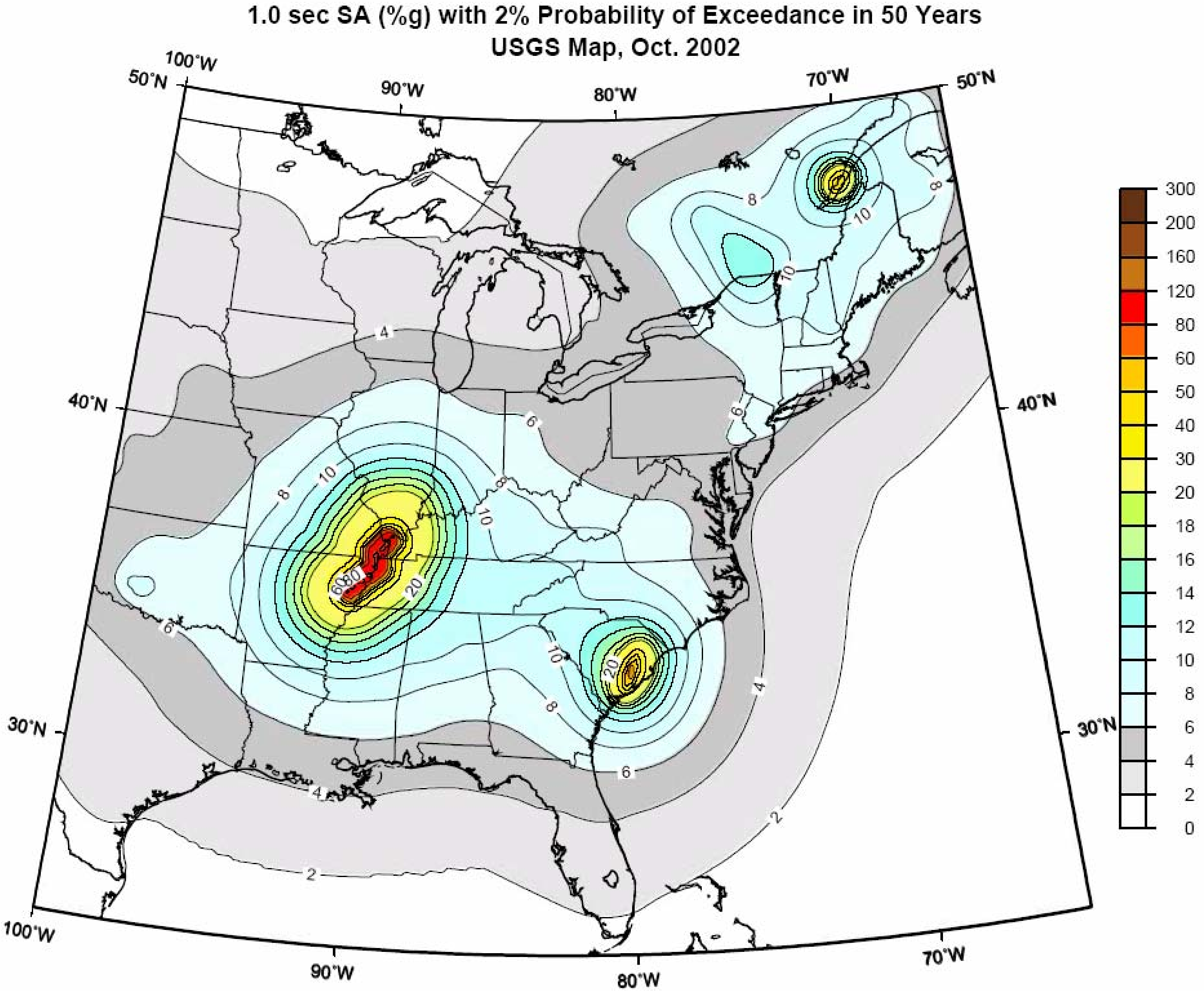
USGS Map for Central and Eastern United States



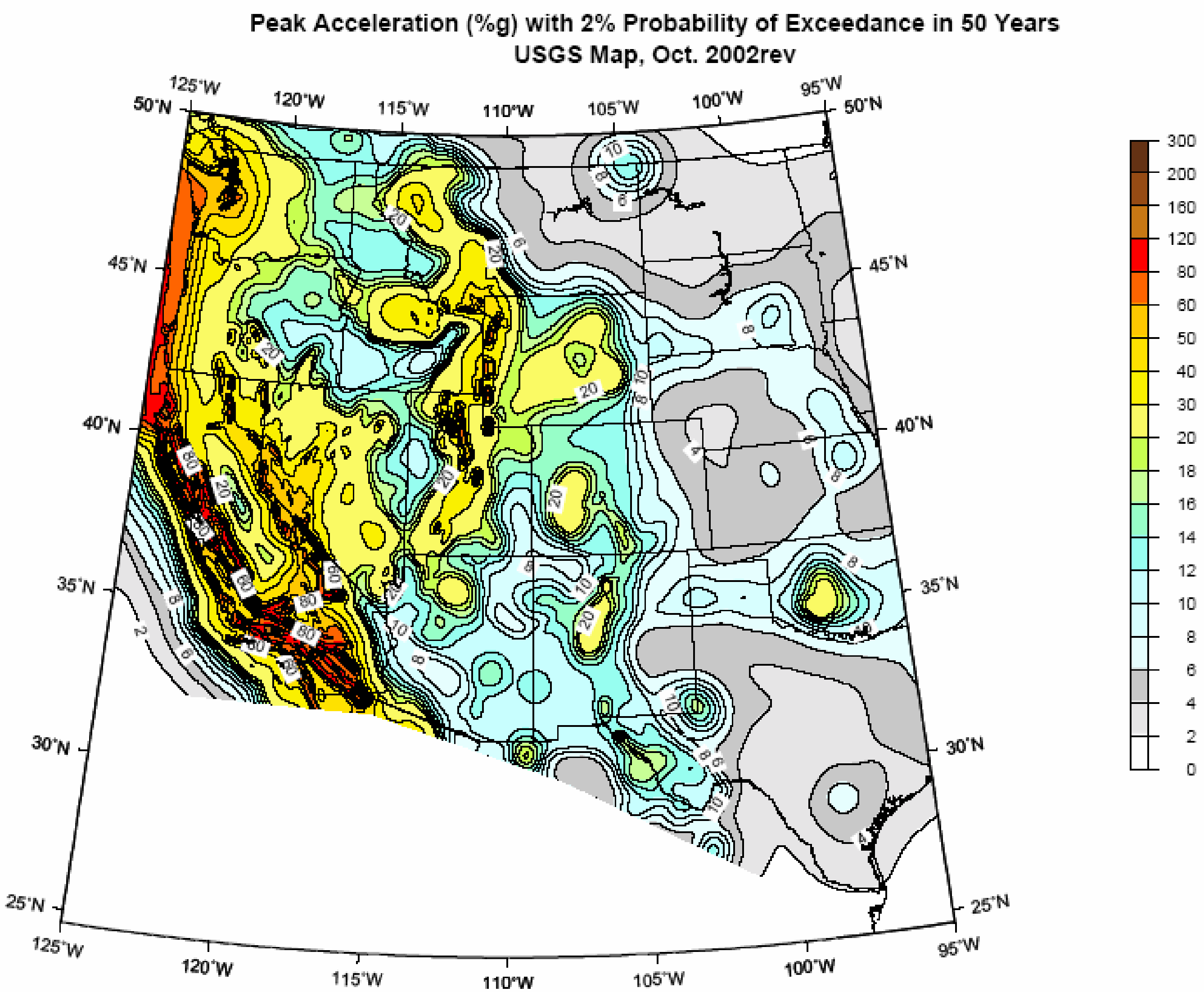
USGS Map for Central and Eastern United States



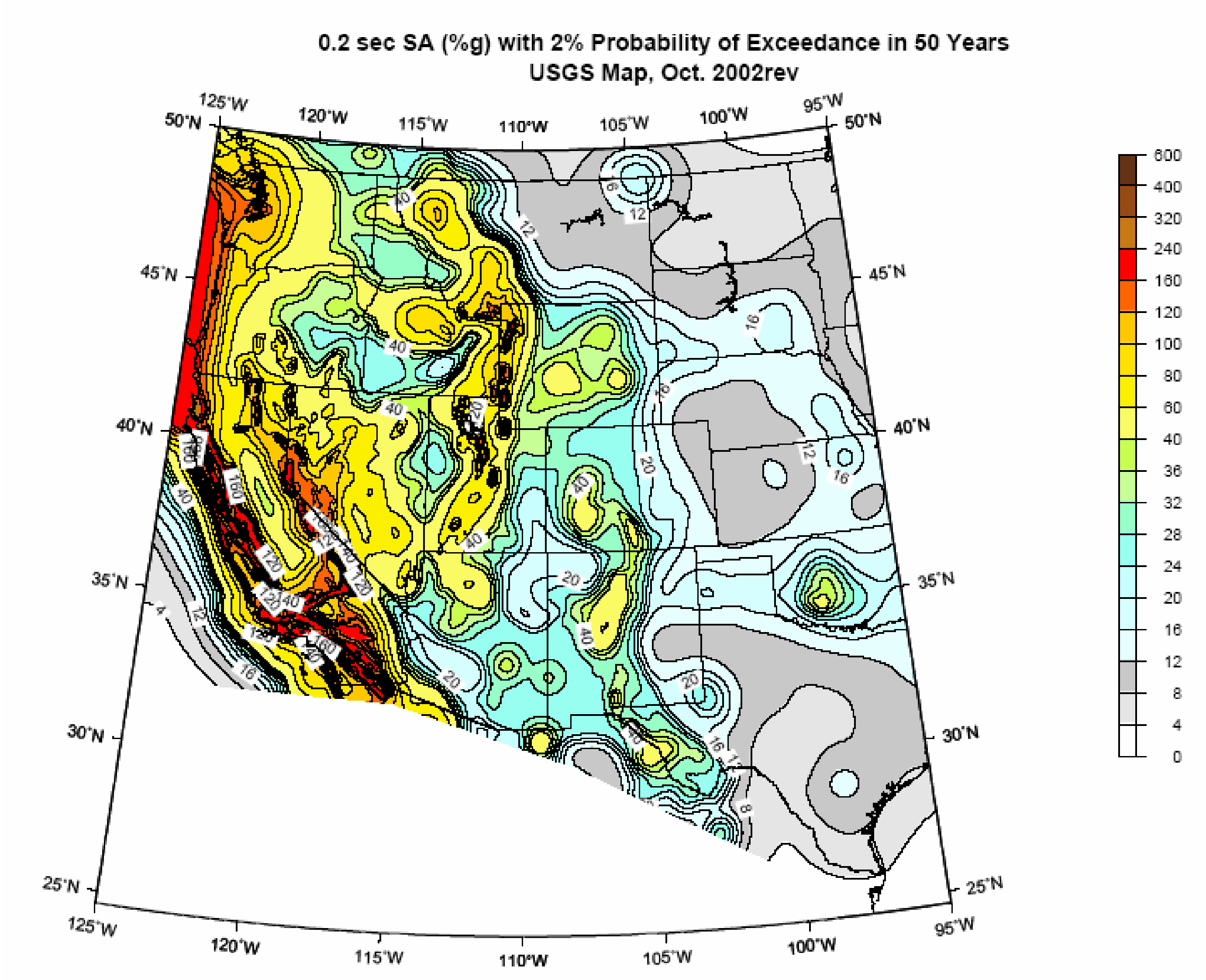
USGS Map for Central and Eastern United States



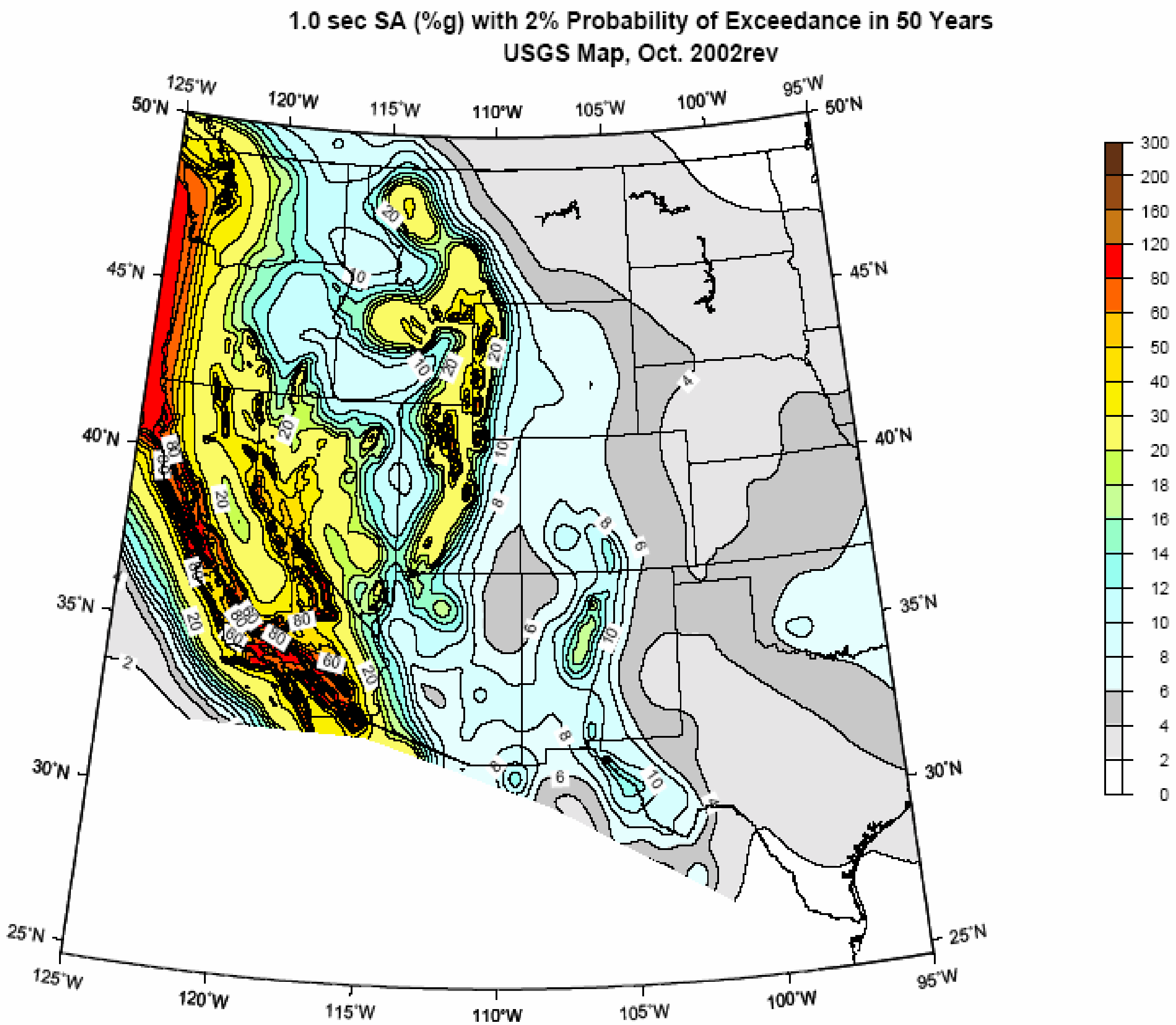
USGS Map for Western United States



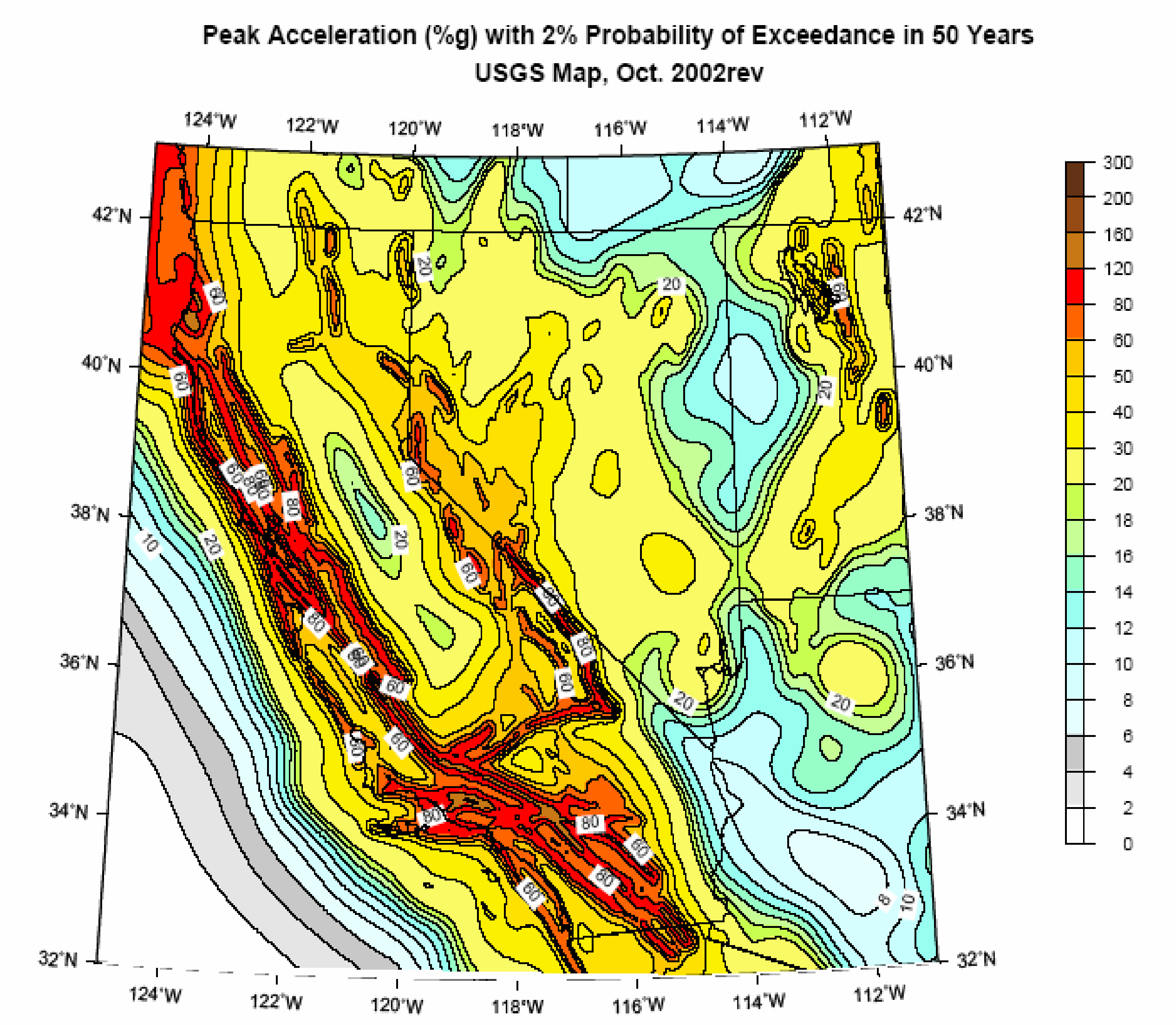
USGS Map for Western United States



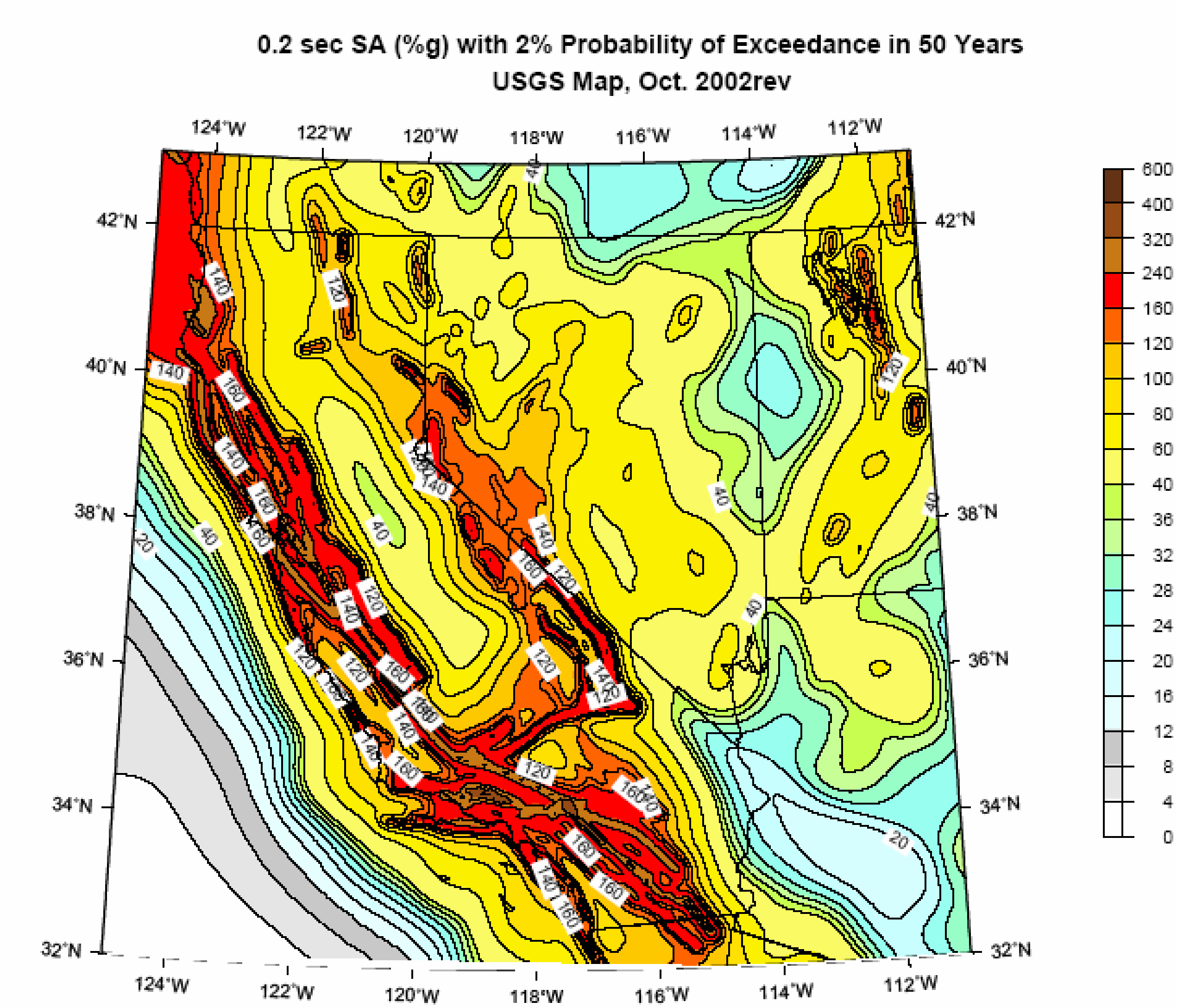
USGS Map for Western United States



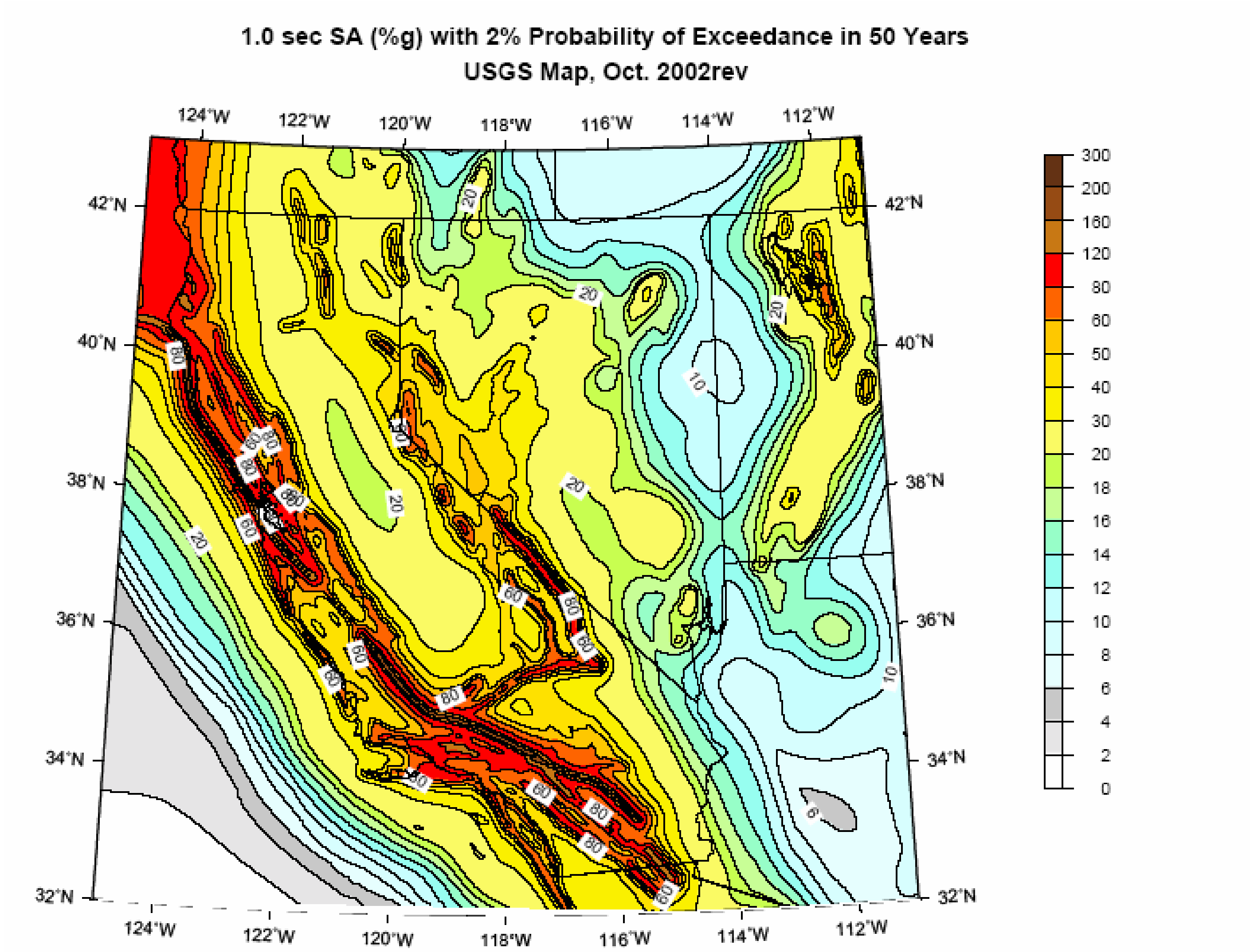
USGS Map for California



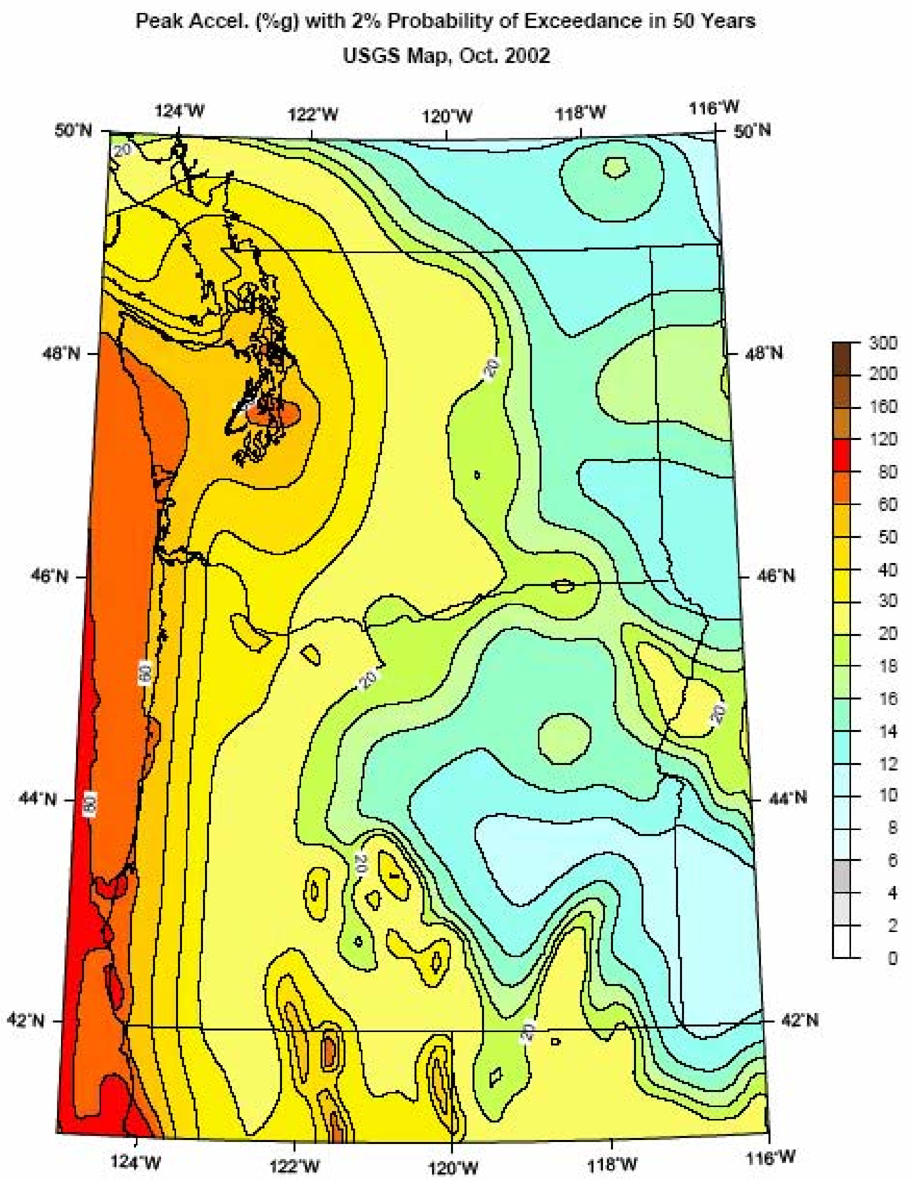
USGS Map for California



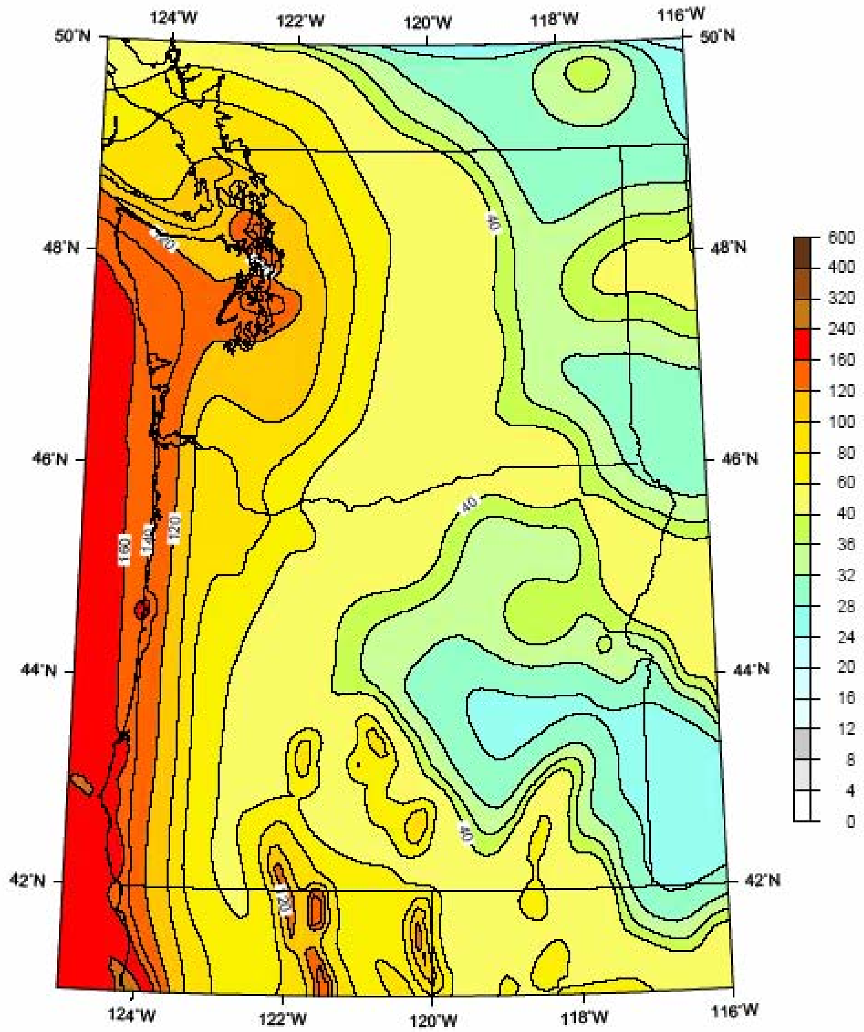
USGS Map for California



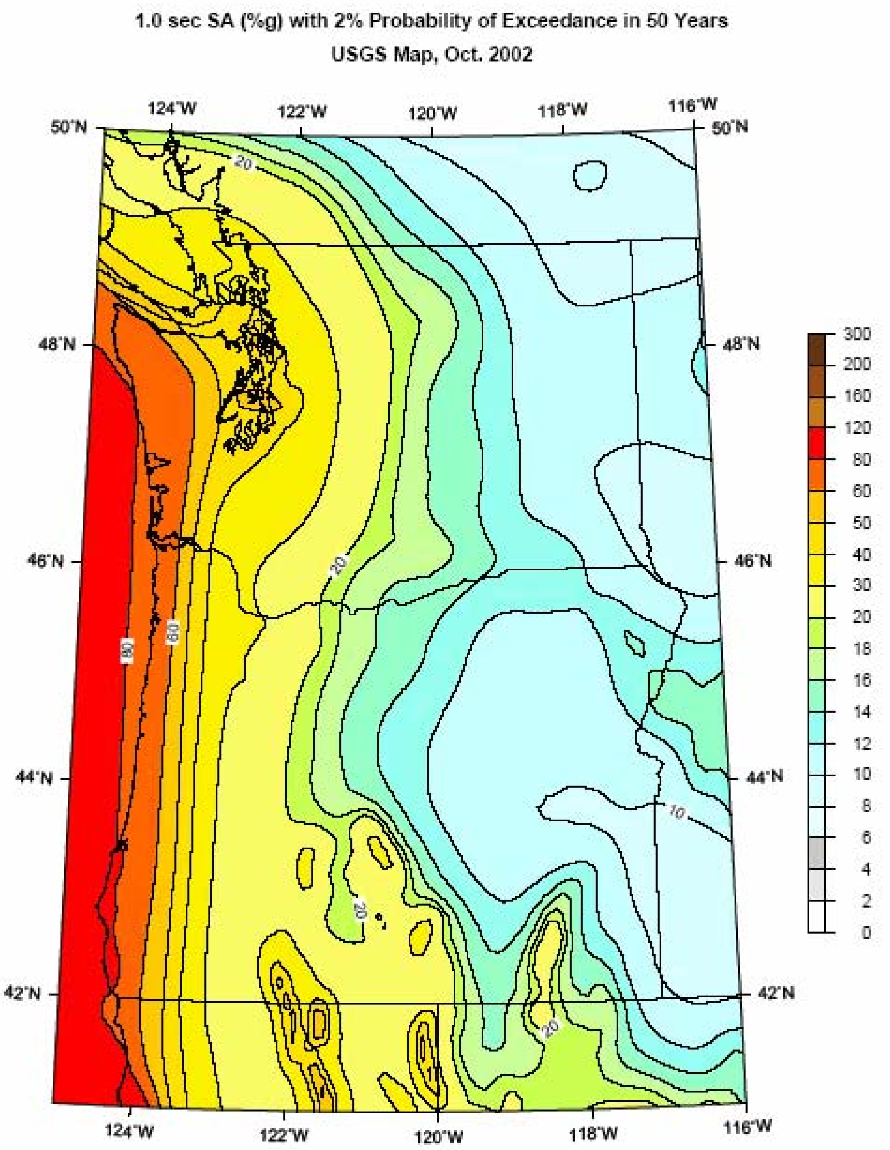
USGS Map for Pacific Northwest



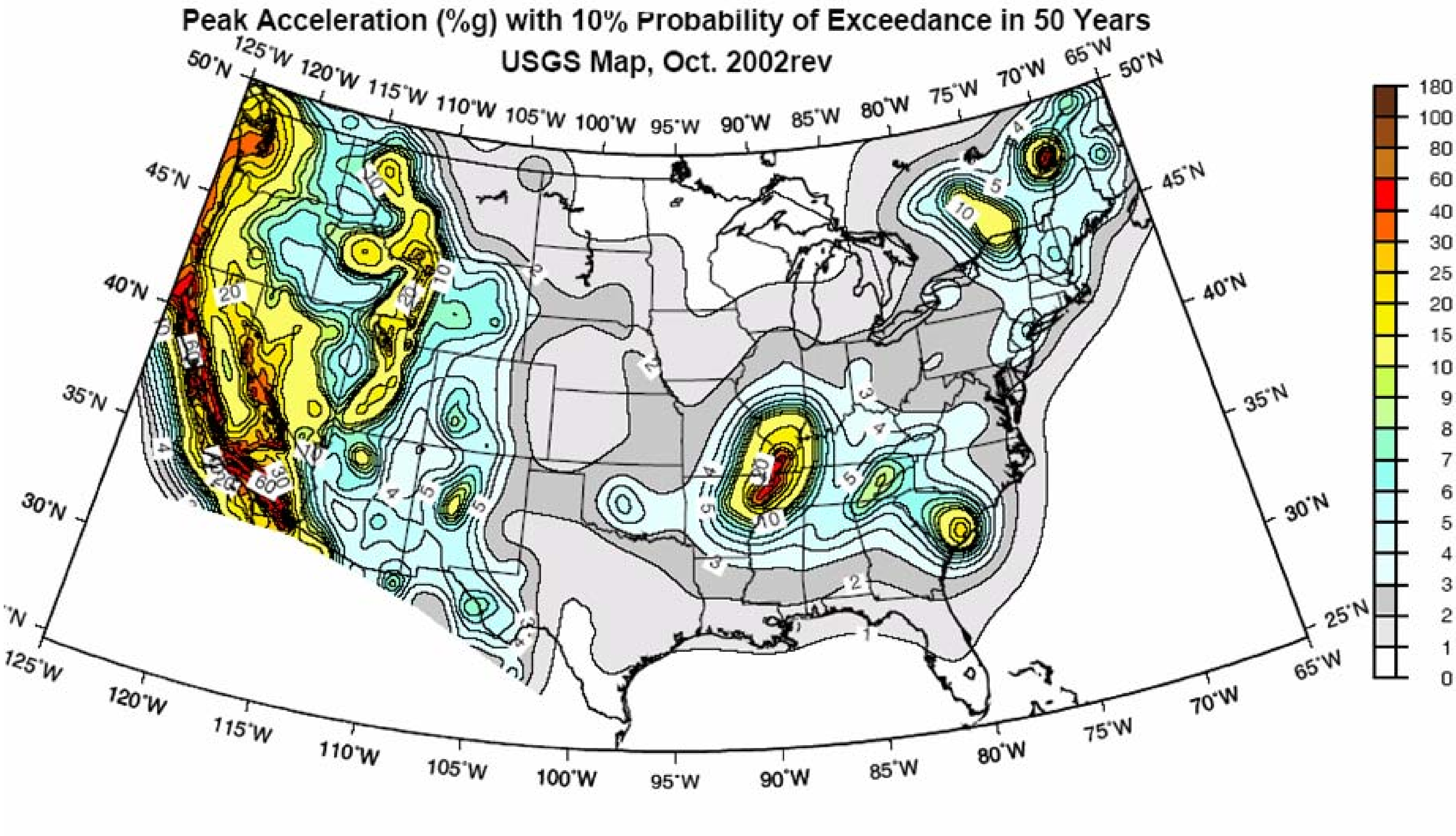
USGS Map for Pacific Northwest



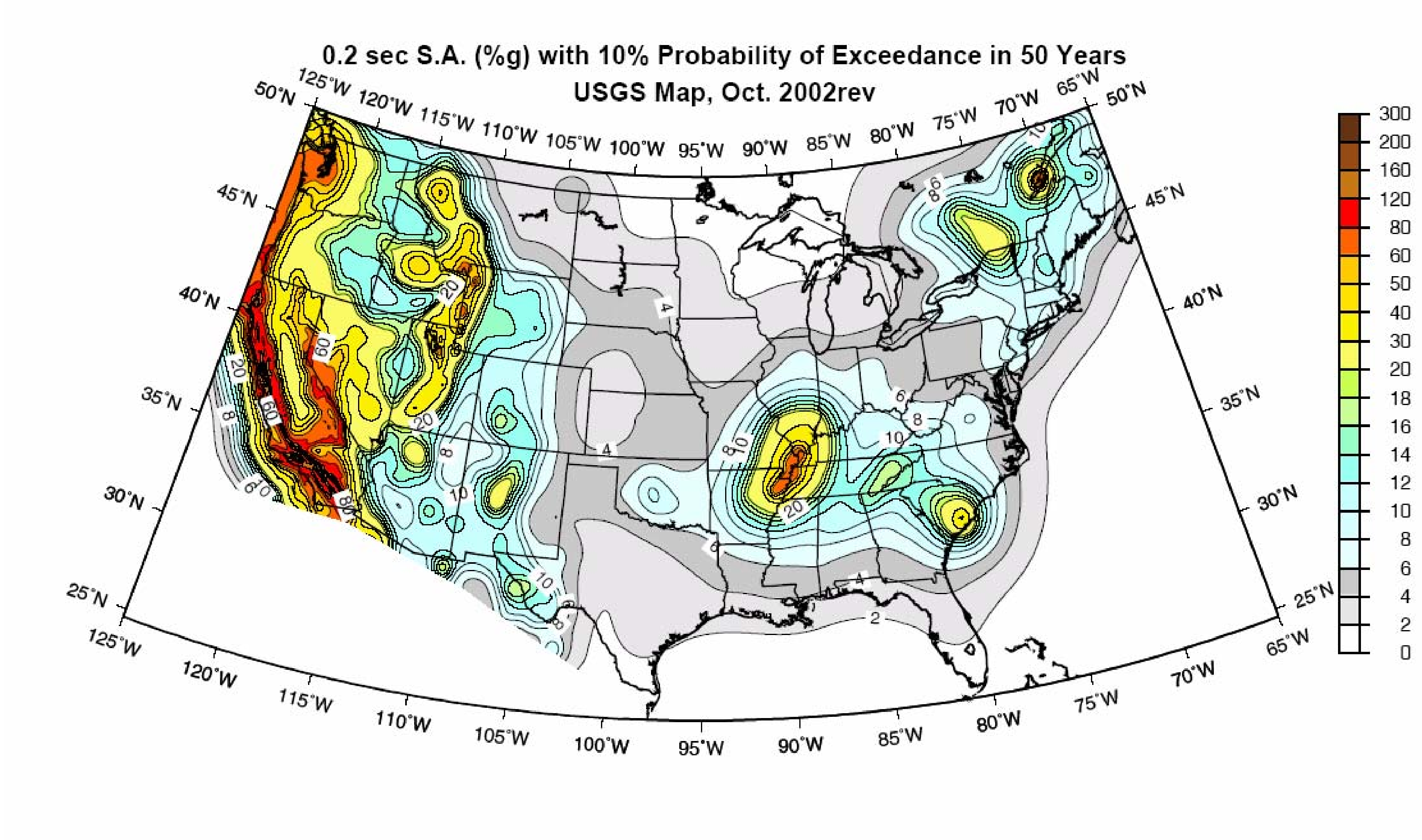
USGS Map for Pacific Northwest



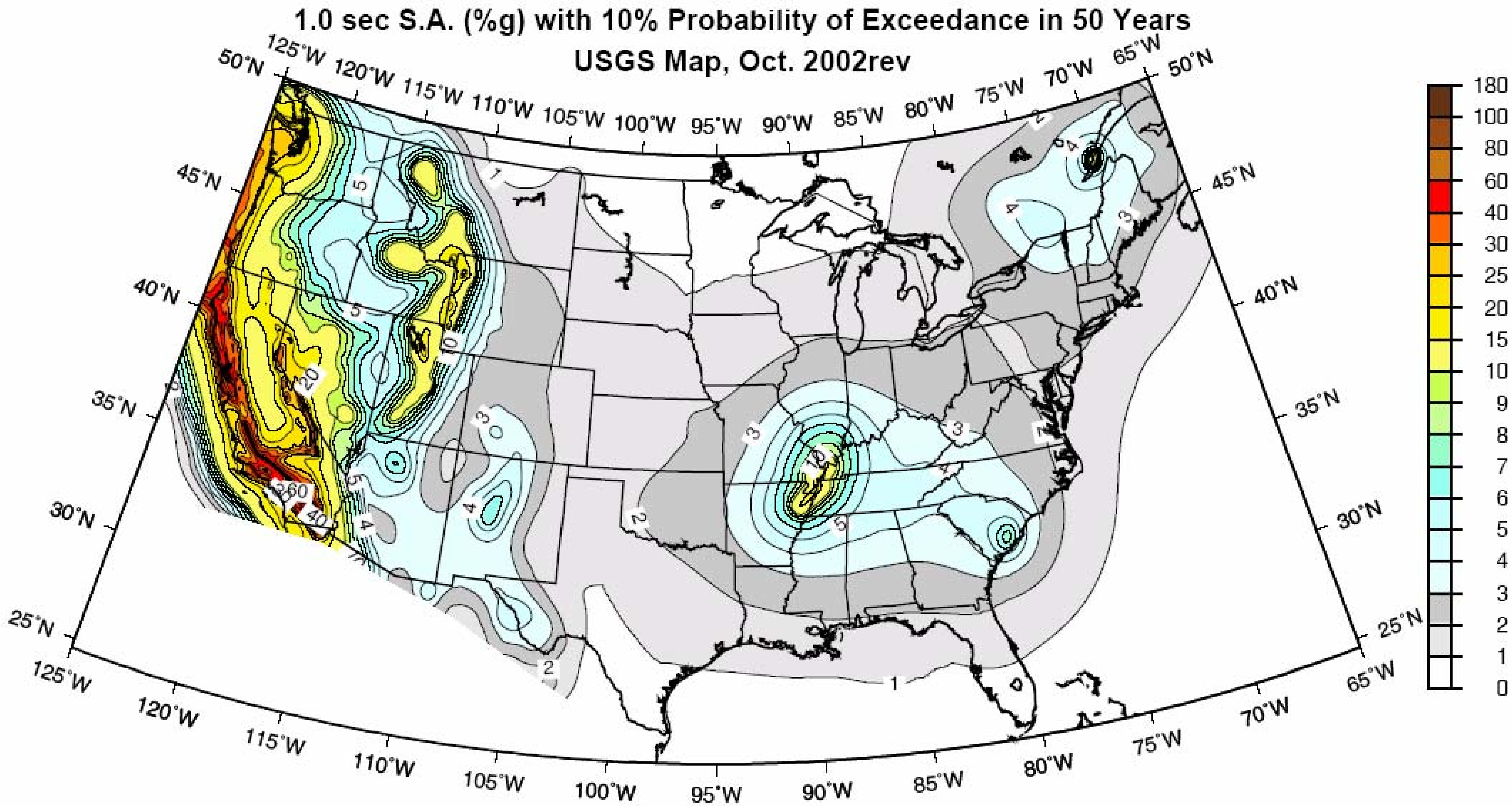
USGS Seismic Hazard Map of Coterminous United States



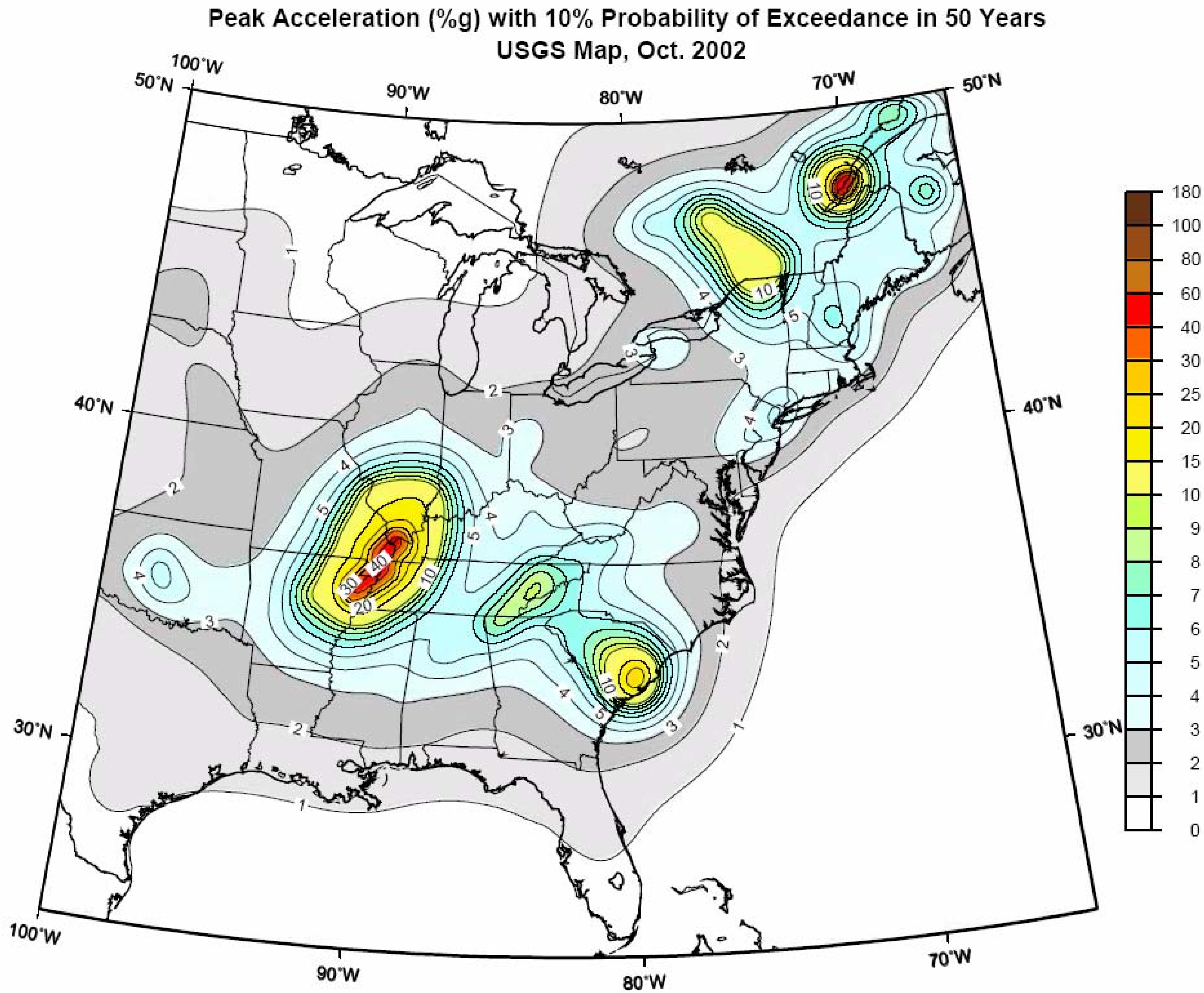
USGS Seismic Hazard Map of Coterminous United States



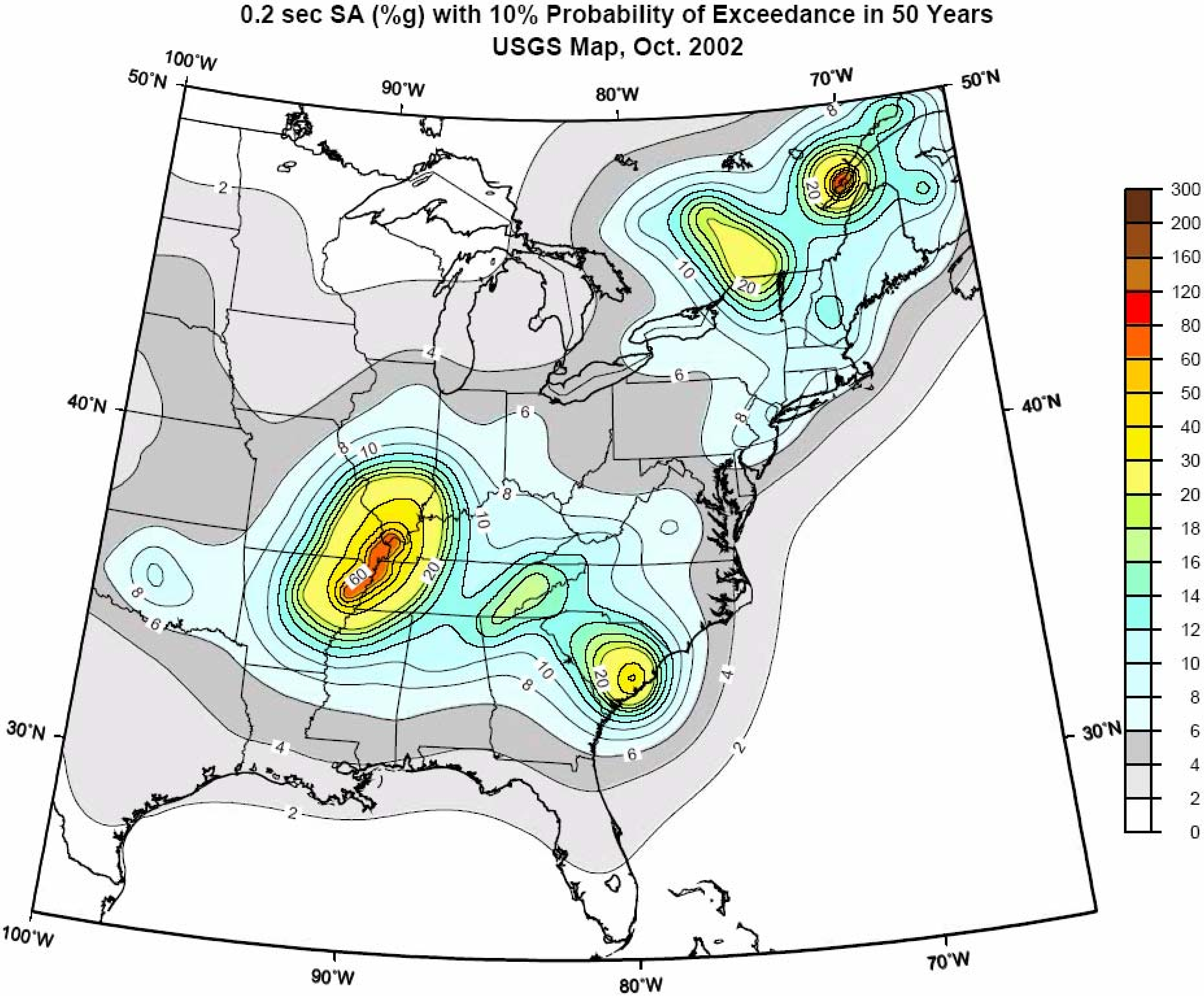
USGS Seismic Hazard Map of Coterminous United States



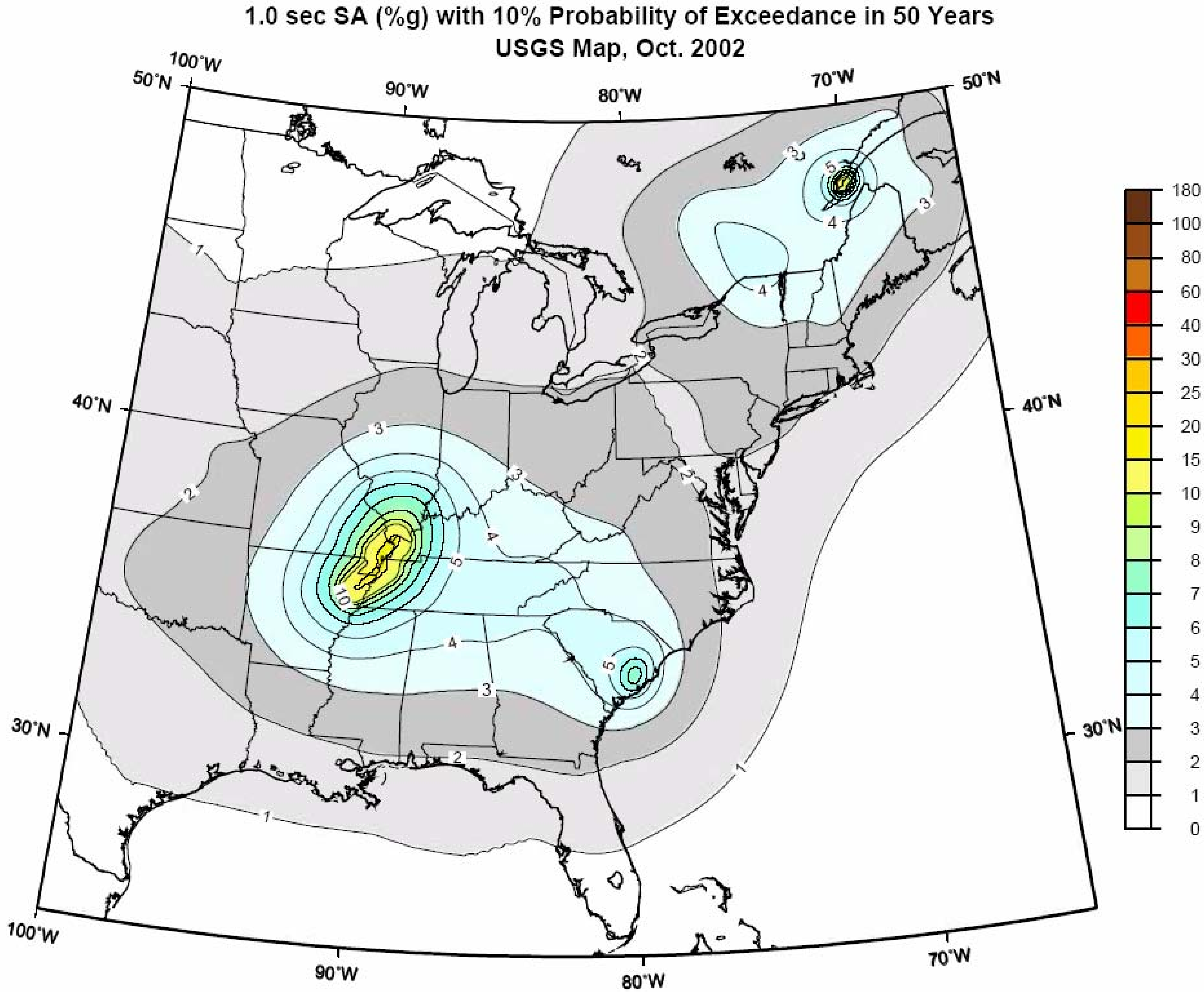
USGS Map for Central and Eastern United States



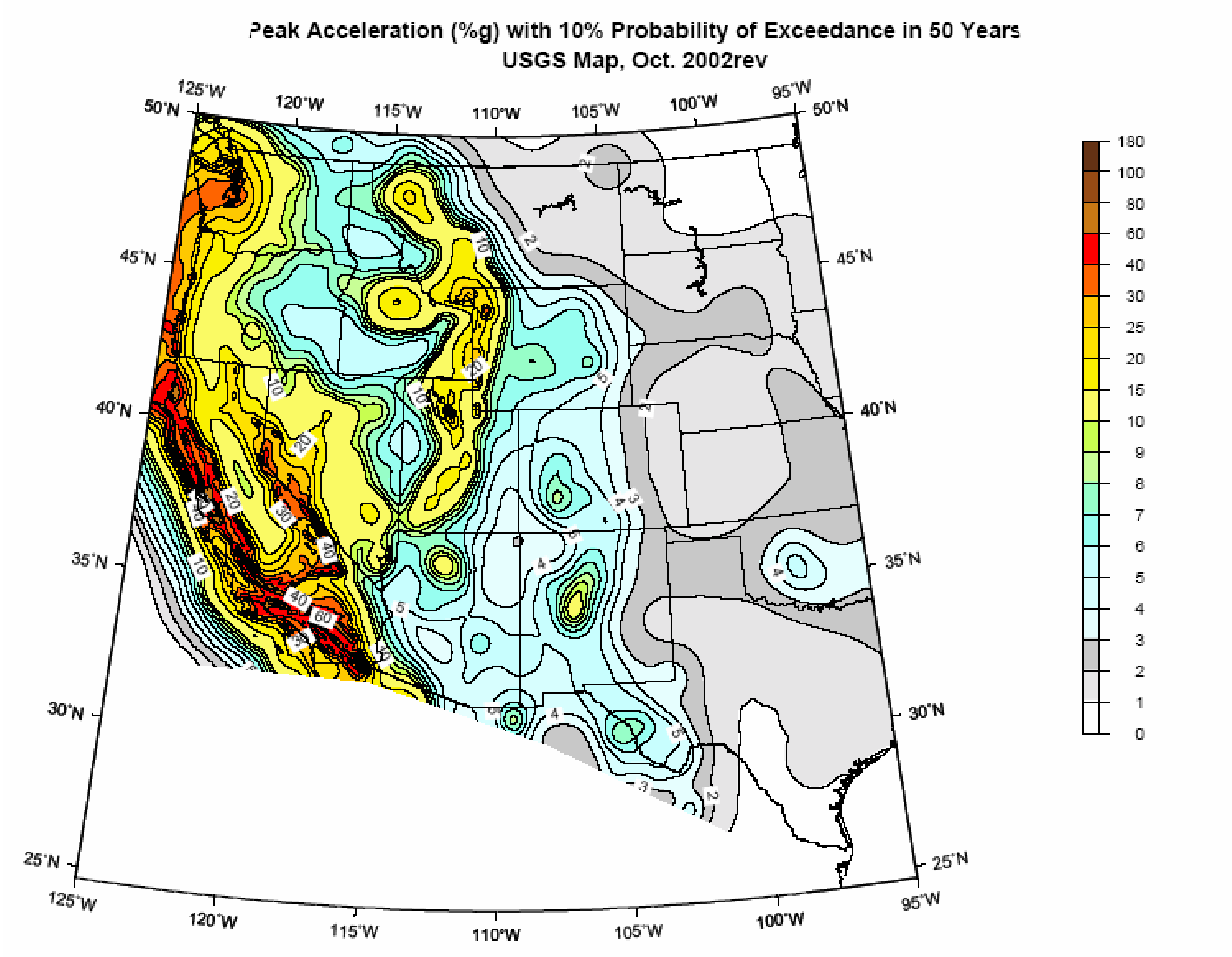
USGS Map for Central and Eastern United States



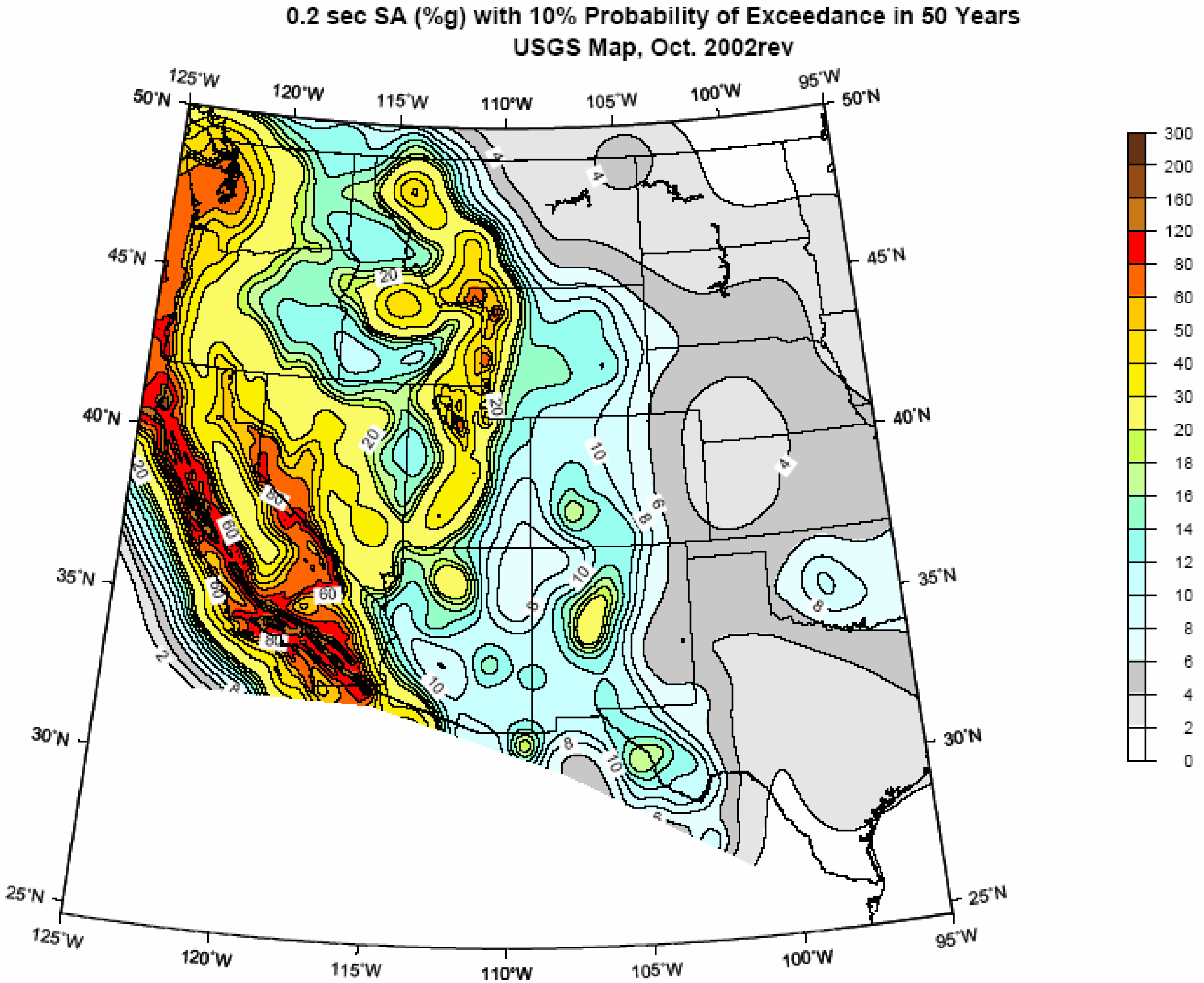
USGS Map for Central and Eastern United States



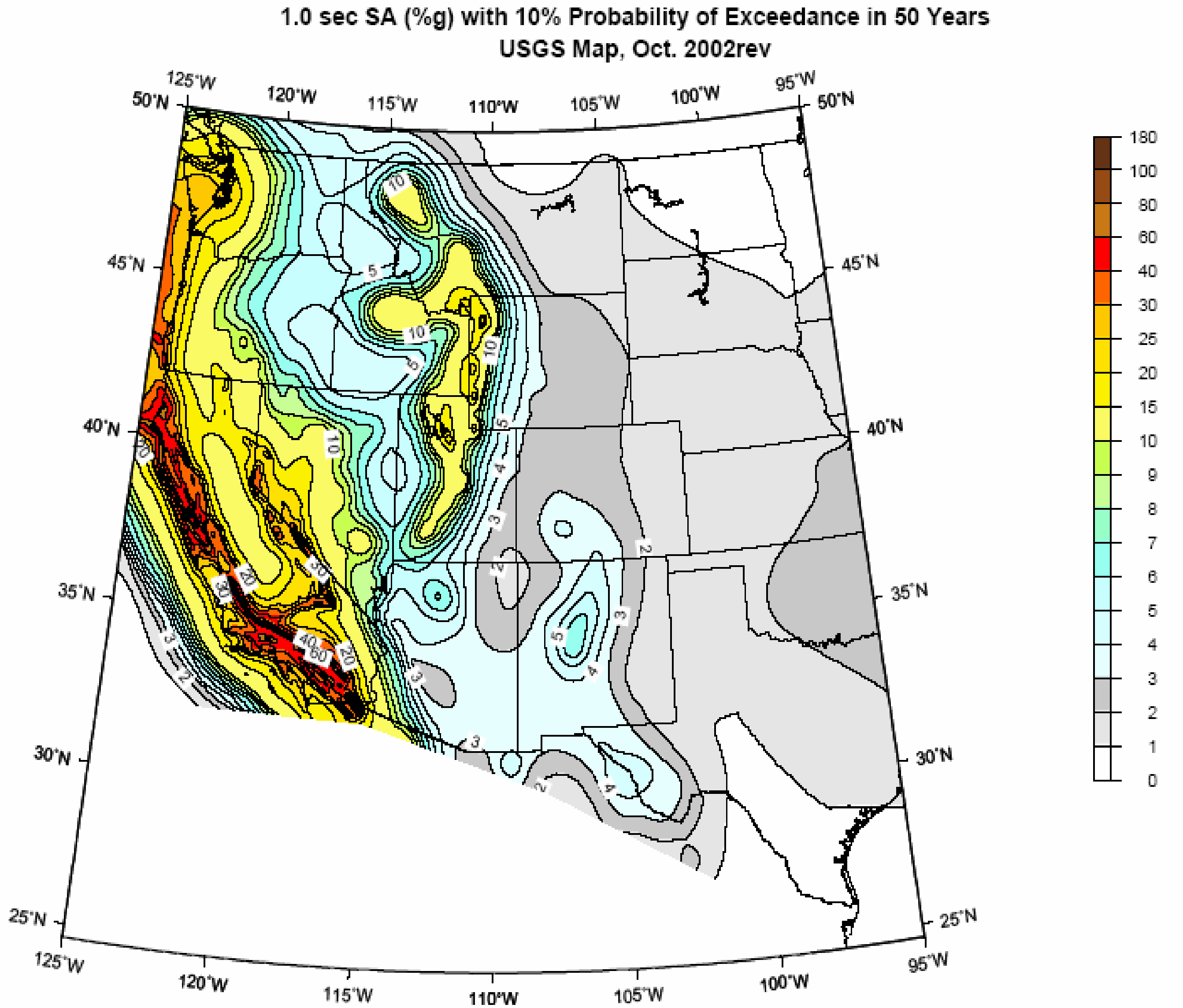
USGS Map for Western United States



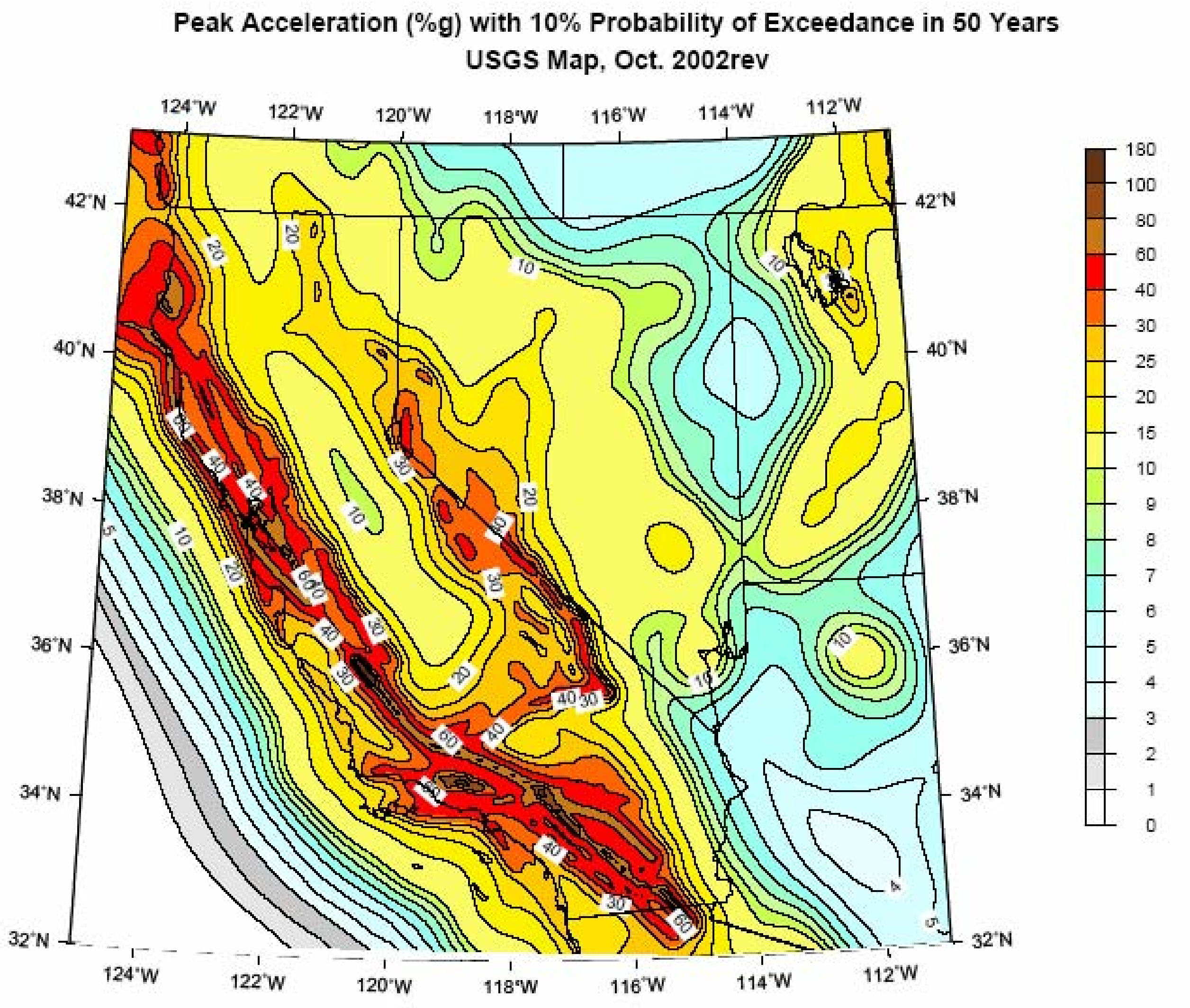
USGS Map for Western United States



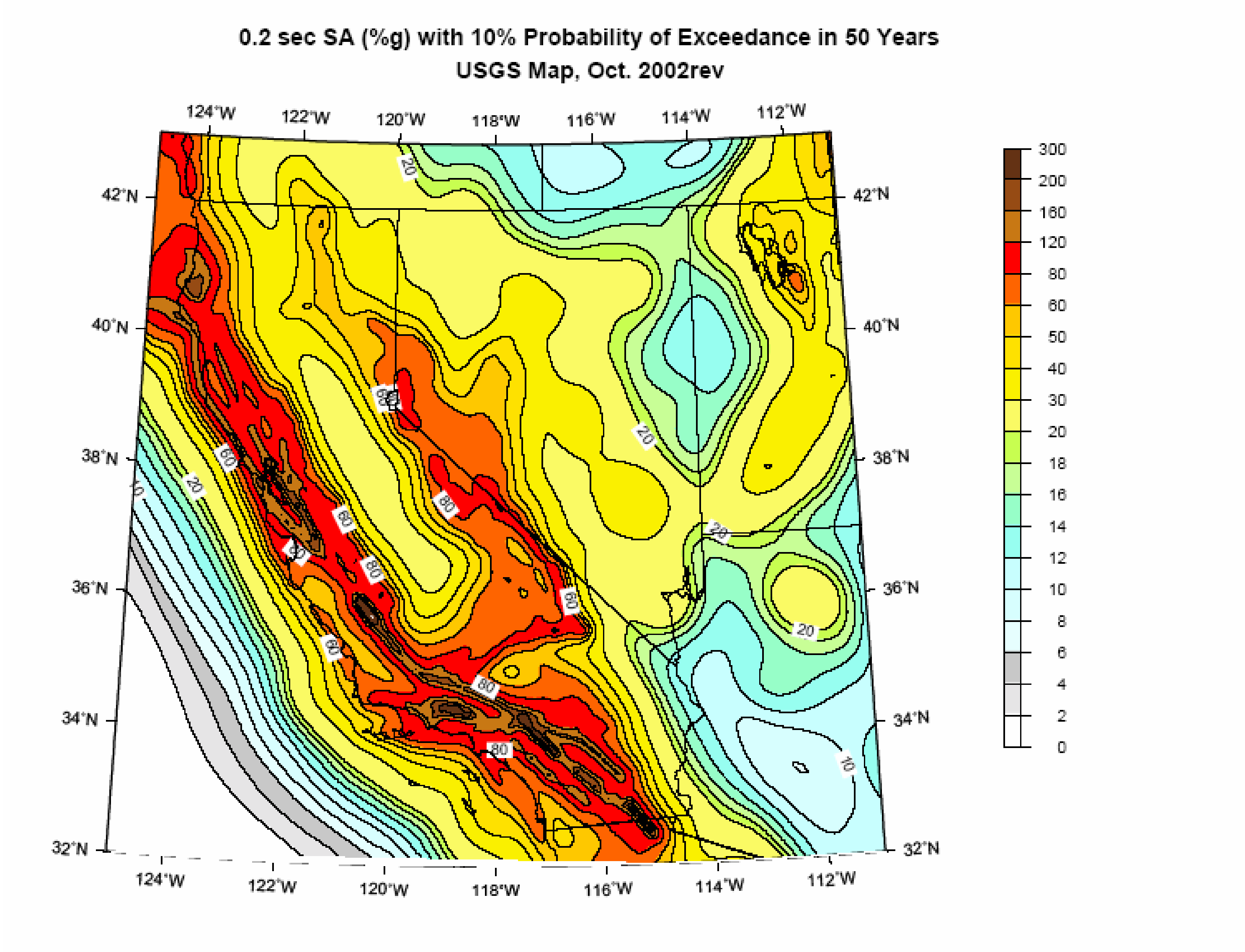
USGS Map for Western United States



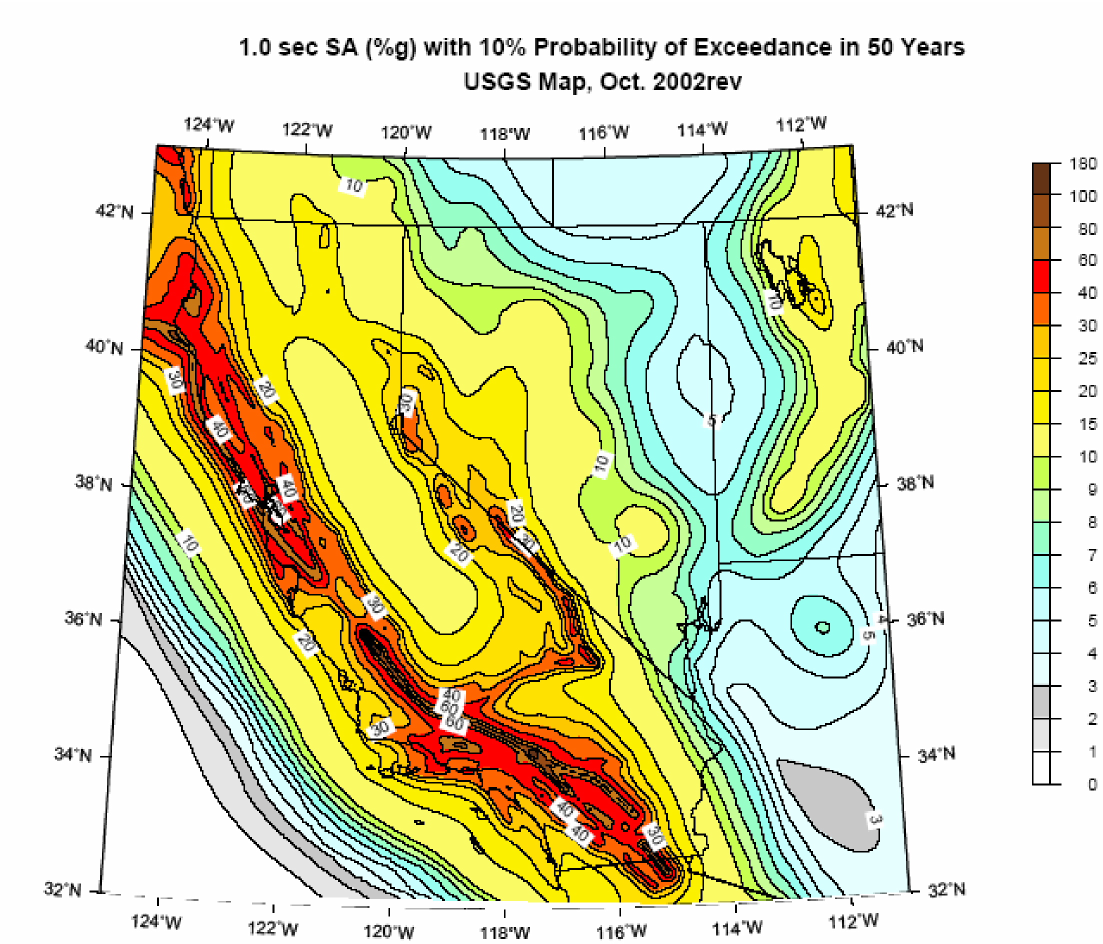
USGS Map for California



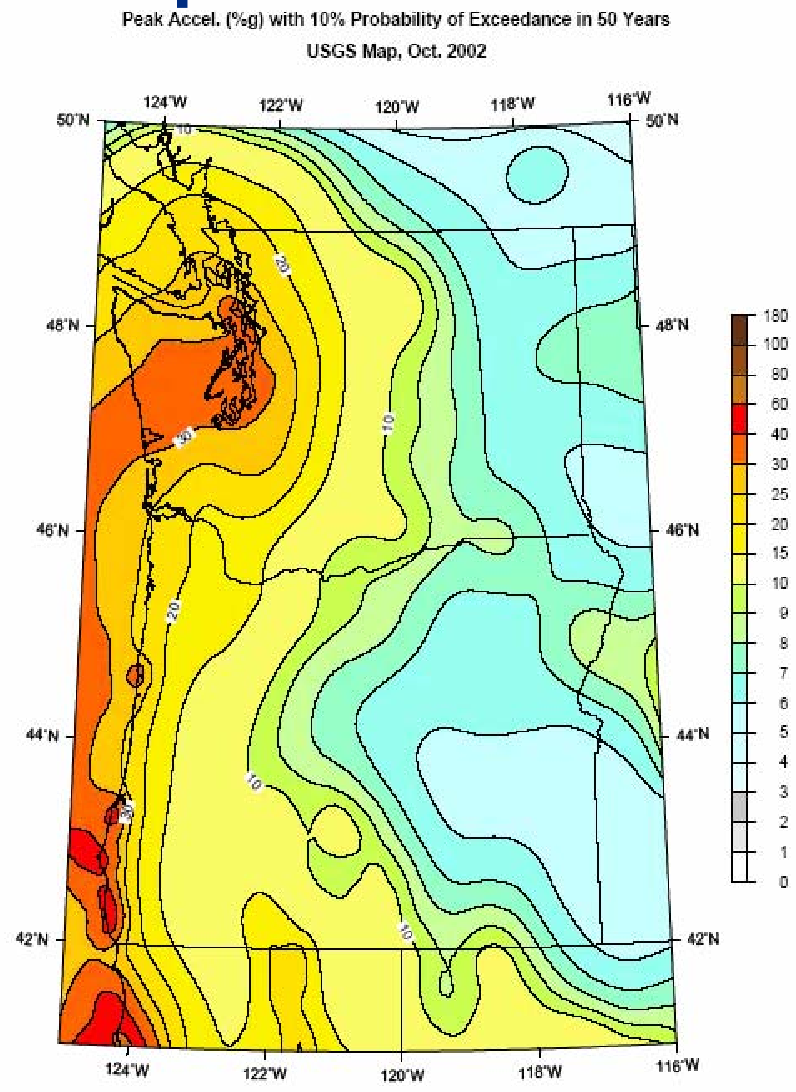
USGS Map for California



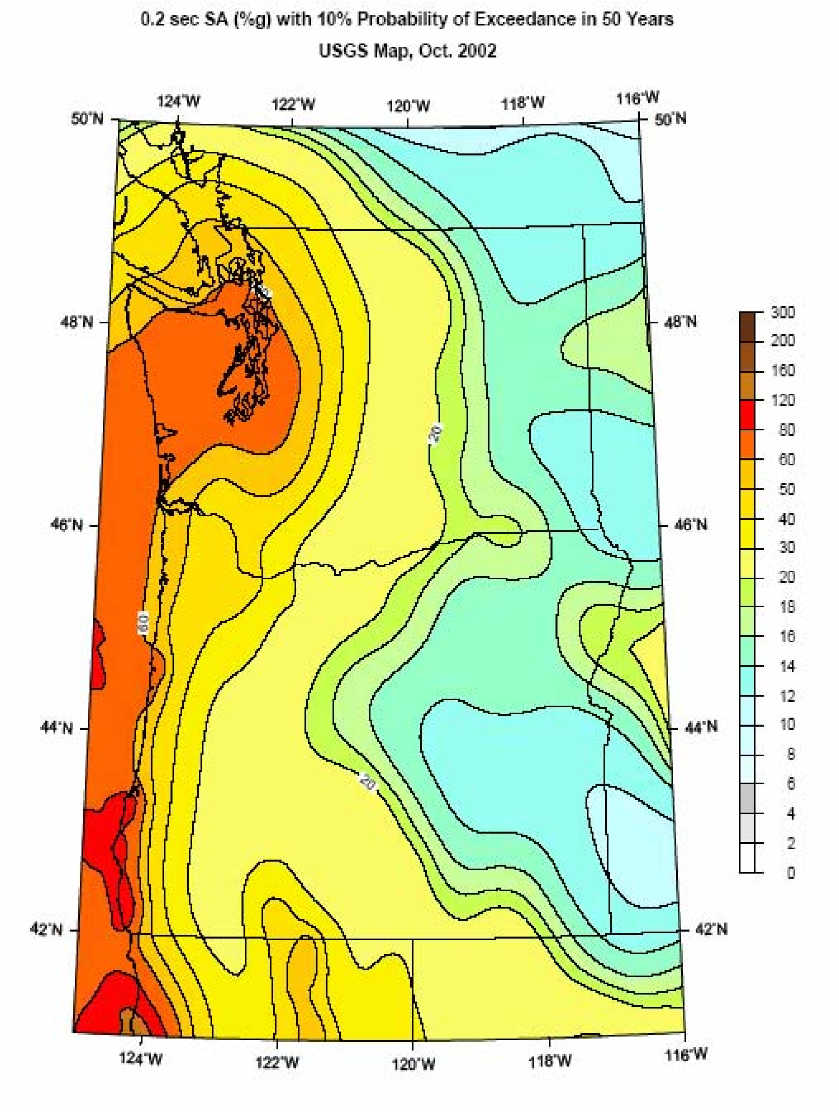
USGS Map for California



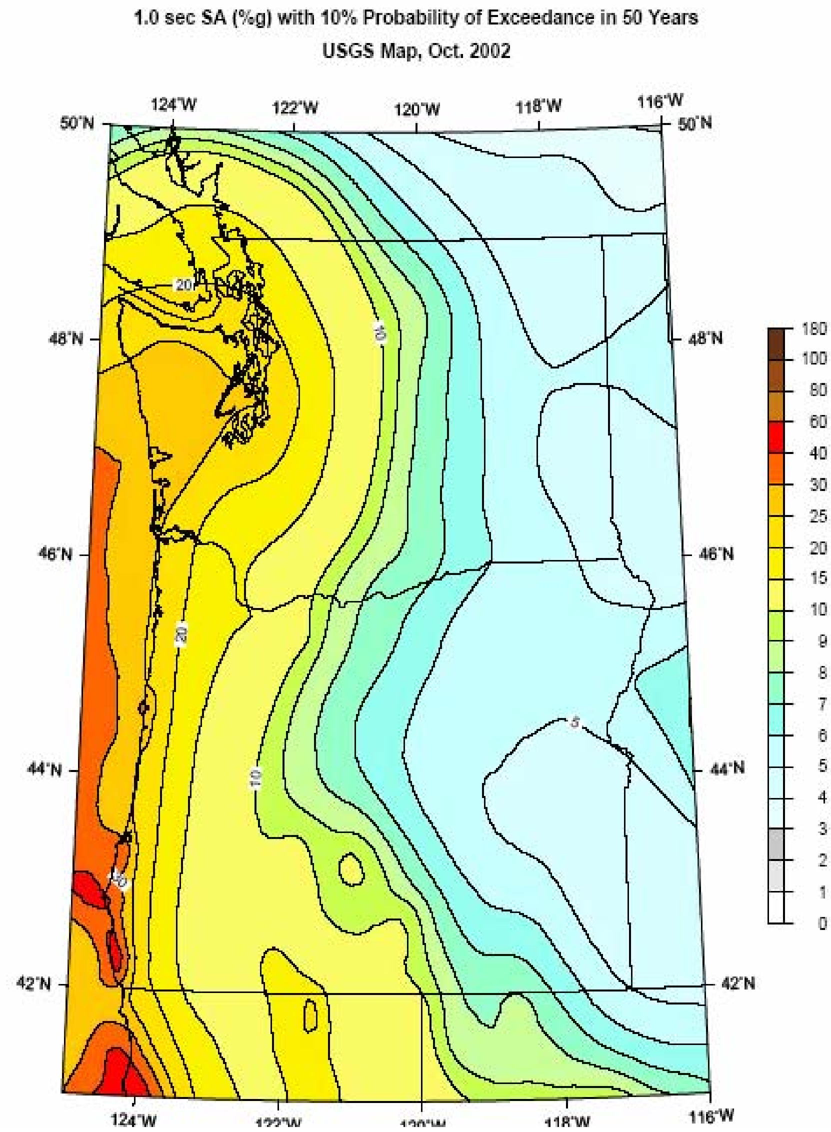
USGS Map for Pacific Northwest



USGS Map for Pacific Northwest



USGS Map for Pacific Northwest



USGS Website for Map Values

http://earthquake.usgs.gov/research/hazmaps/design/

```
   The input zipcode is 80203. (DENVER)
   ZIP CODE                        80203
   LOCATION                        39.7310 Lat. -104.9815 Long.
   DISTANCE TO NEAREST GRID POINT  3.7898 kms
   NEAREST GRID POINT              39.7 Lat. -105.0 Long.
   Probabilistic ground motion values, in %g, at the Nearest Grid
point are:

               10%PE in 50 yr   5%PE in 50 yr   2%PE in 50 yr
      PGA        3.299764         5.207589         9.642159
   0.2 sec SA    7.728900        11.917400        19.921591
   0.3 sec SA    6.178438         9.507714        16.133711
   1.0 sec SA    2.334019         3.601994         5.879917
```

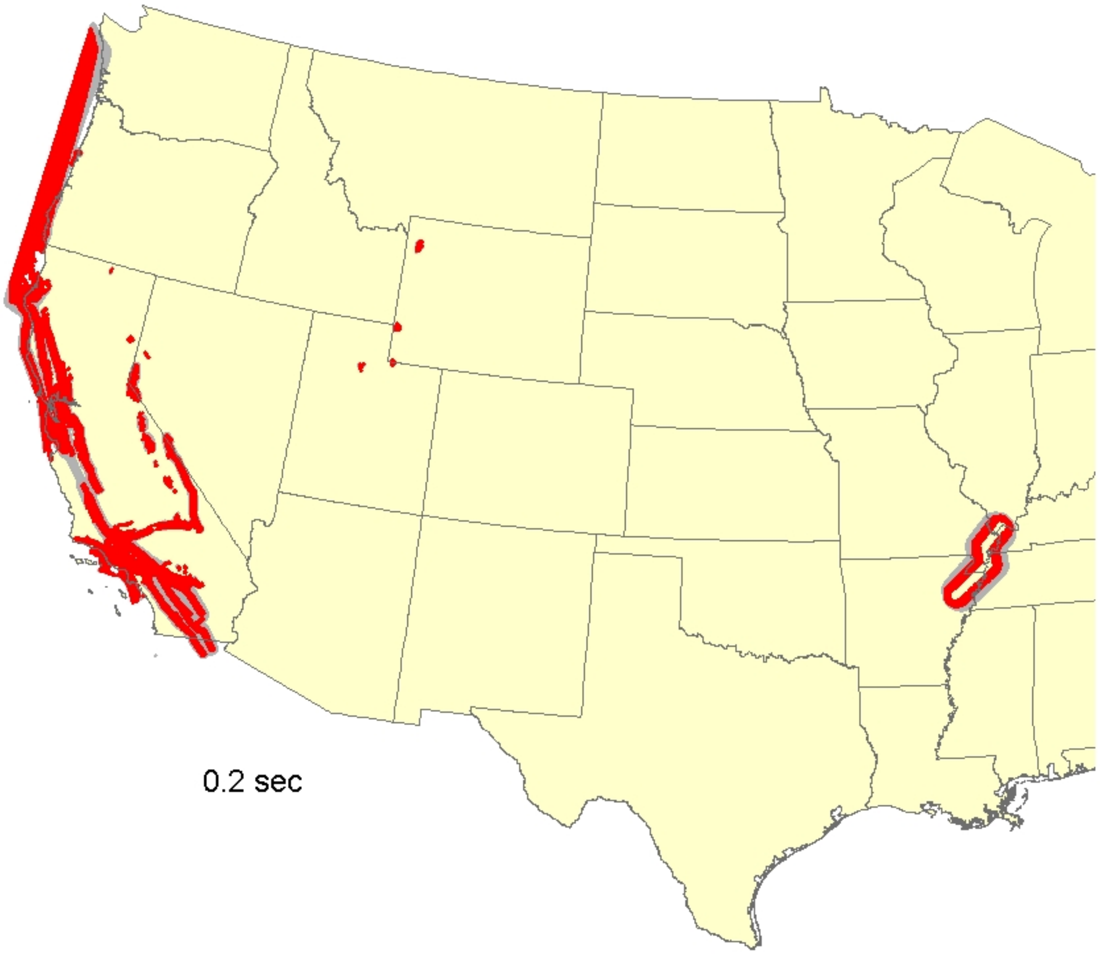
CAUTION: USE OF ZIPCODES IS DISCOURAGED; LAT-LONG VALUES WILL GIVE ACCURATE RESULTS.

Relative PGAs for the United States

2000 *NEHRP Recommended Provisions* Maps

- 5% damped, 2% in 50 years, Site Class B (firm rock)
- 0.2 second and 1.0 second spectral ordinates provided
- On certain faults in California, Alaska, Hawaii, and CUS *Provisions* values are <u>deterministic</u> cap times 1.5. Outside deterministic areas, *Provisions* maps are the same as the USGS maps.
- USGS longitude/latitude and zipcode values are probabilistic MCE. To avoid confusion, ALWAYS use *Provisions* (adopted by ASCE and IBC) maps for design purposes.

Location of Deterministic Areas



Deterministic Cap

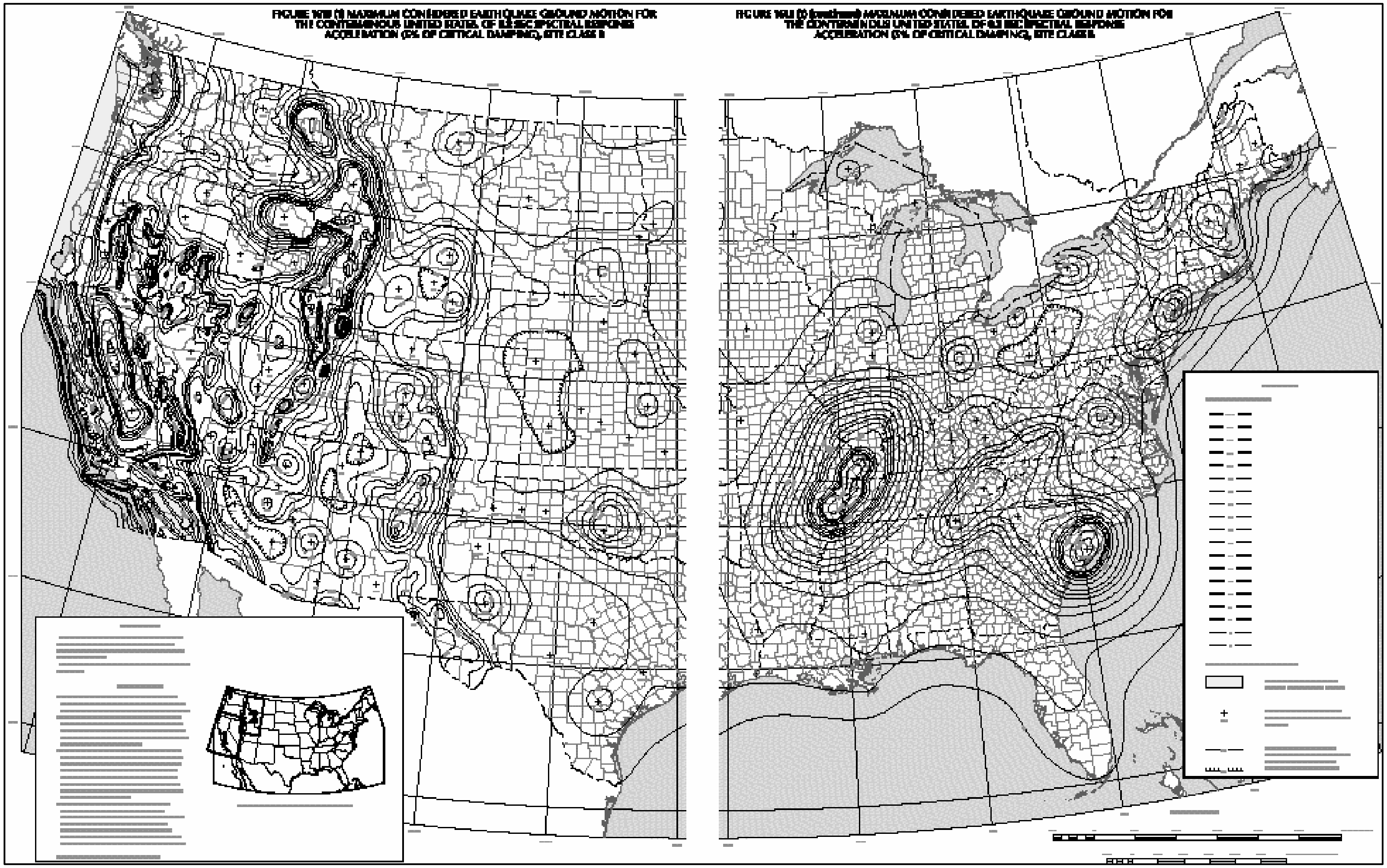
Applies only where probabilistic values exceed highest design values from old (Algermissen and Perkins) maps.

The deterministic procedure for mapping applies:

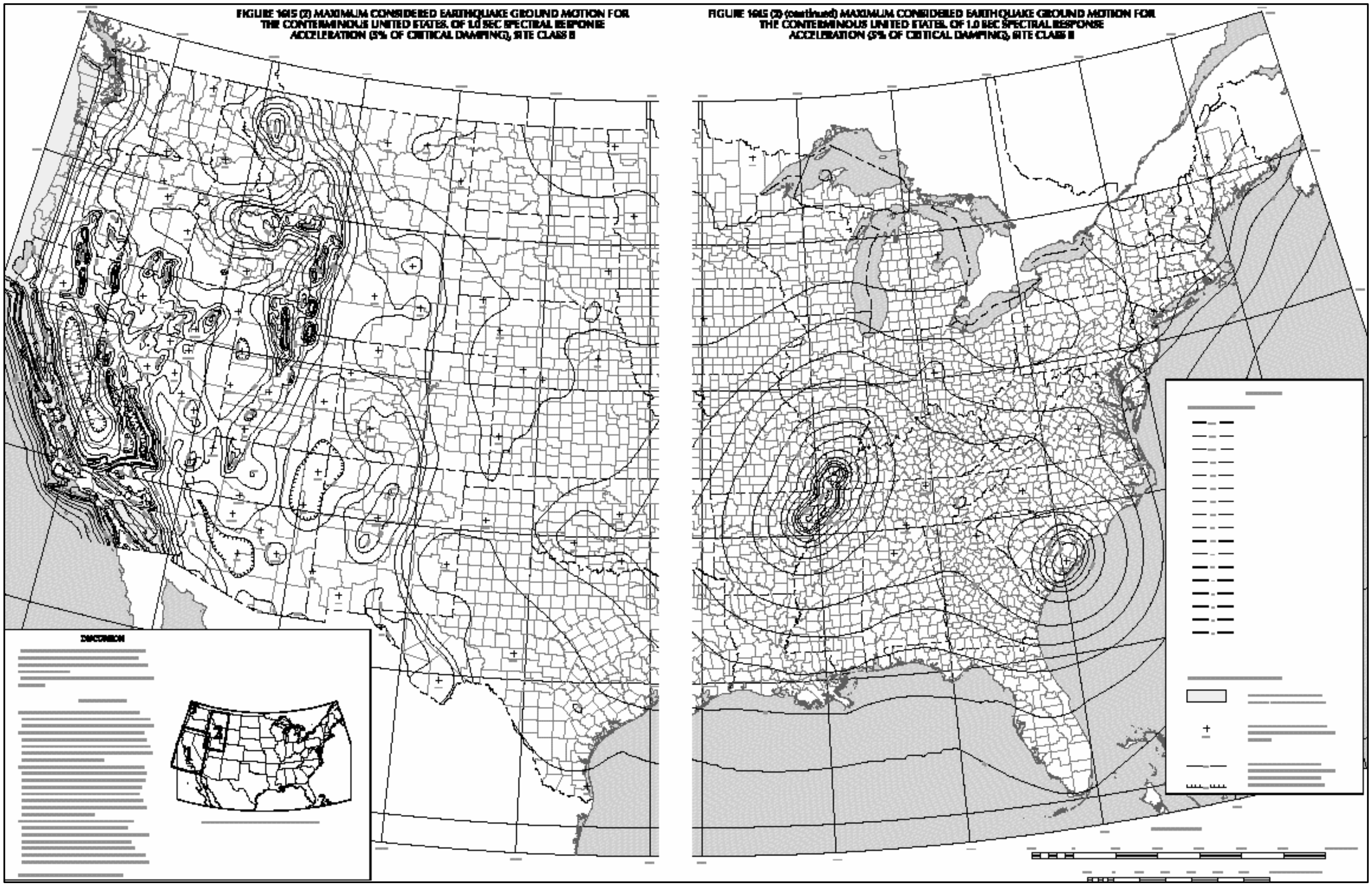
- For known “active” faults
- Uses characteristic largest earthquake on fault
- Uses 150% of value from median attenuation

Use deterministic value if lower than 2% in 50 year value

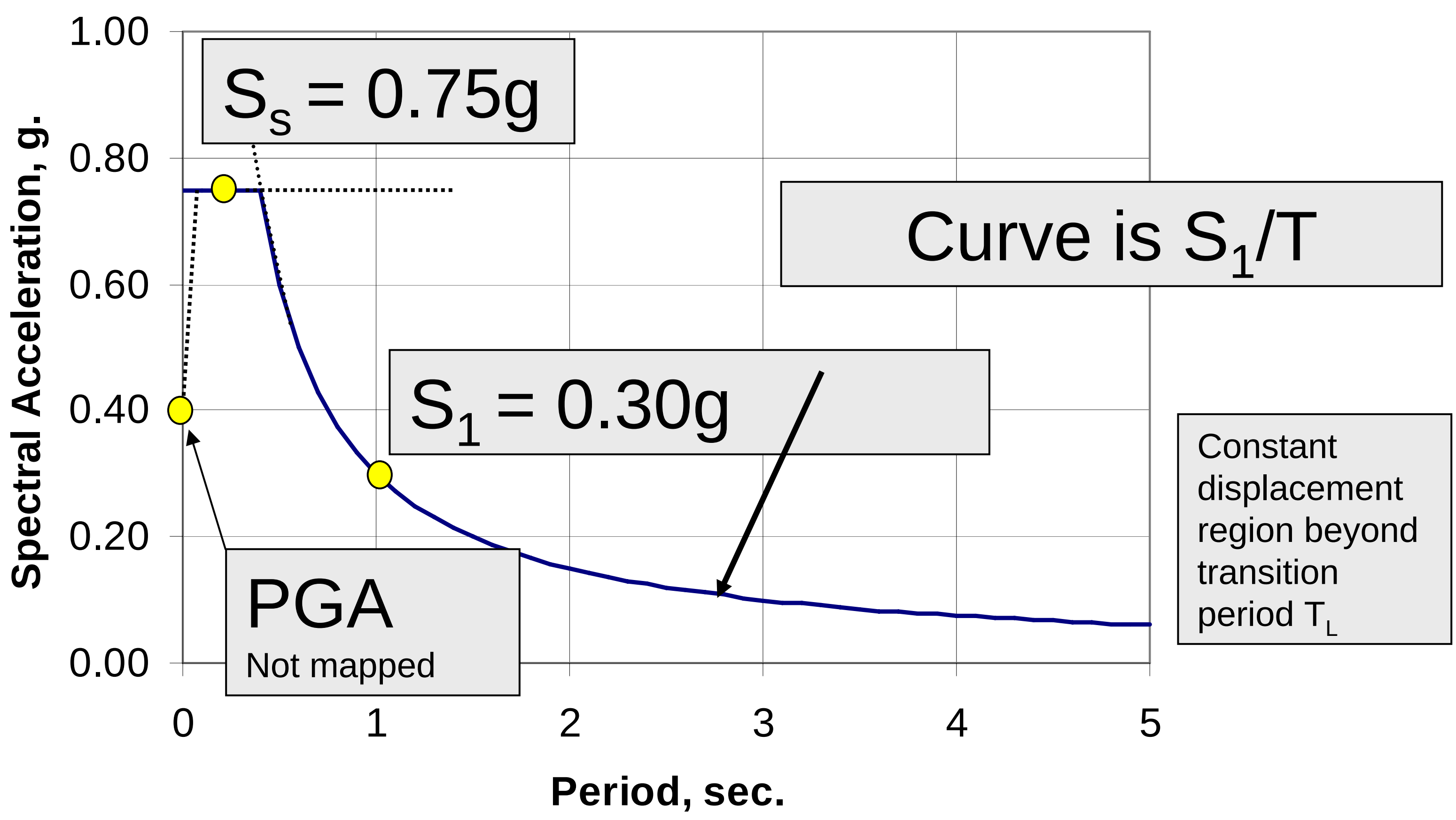
NEHRP Provisions Maps
0.2 Second Spectral Response (S_S)



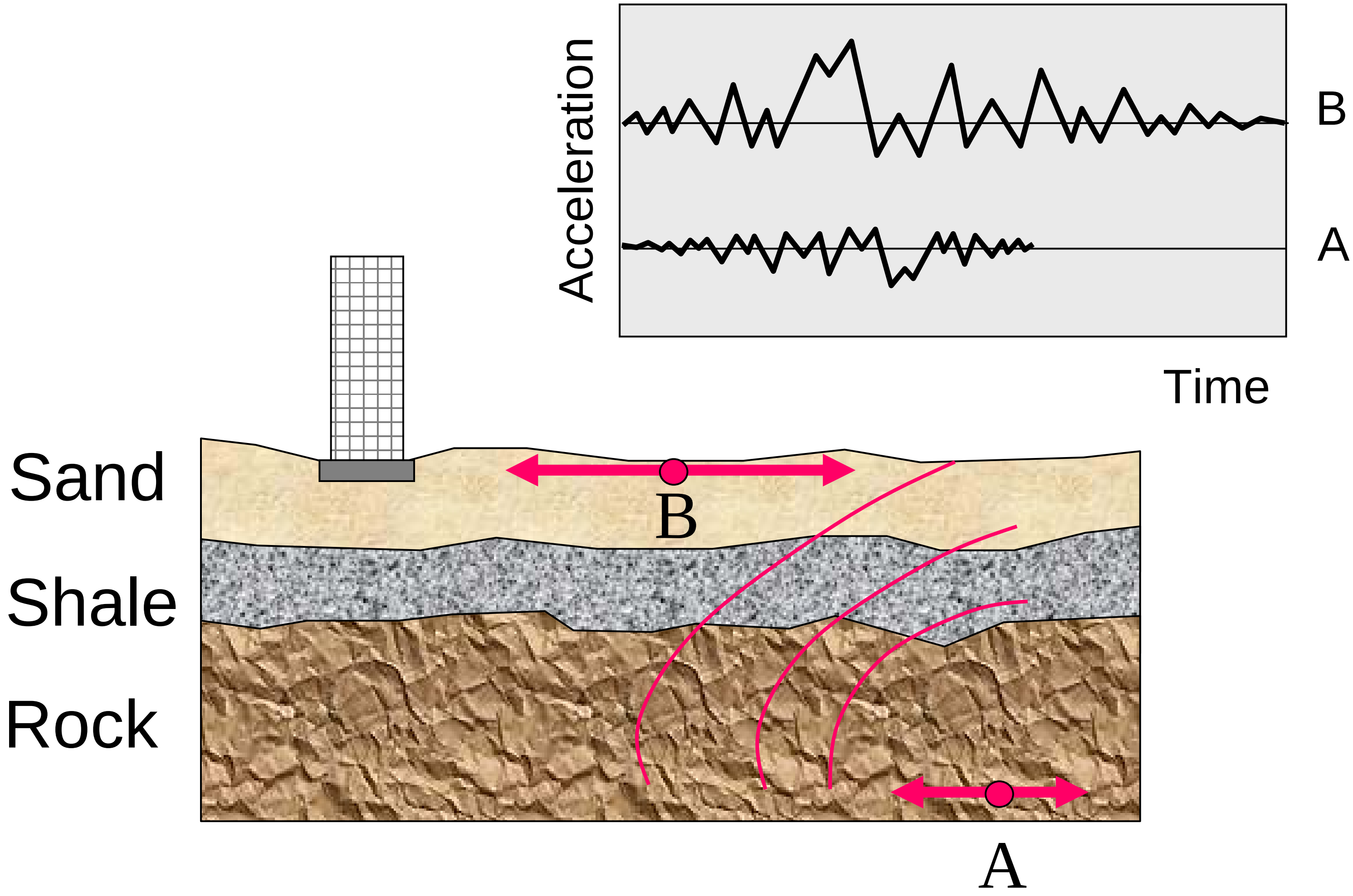
NEHRP Provisions Maps
1.0 Second Spectral Response (S_1)



2% in 50 Year 5% Damped MCE Elastic Spectra Site Class B (Firm Rock)



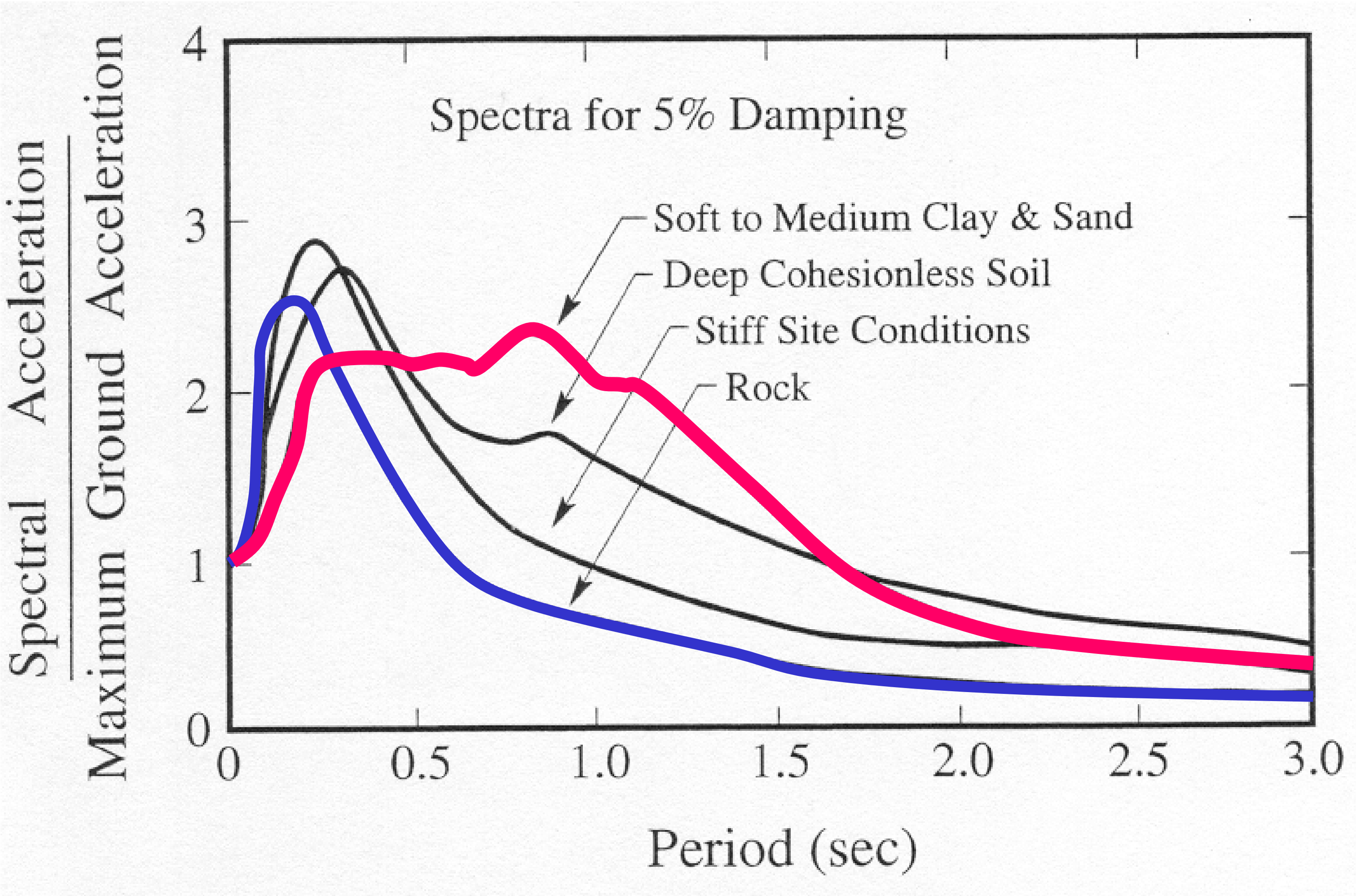
Site Amplification Effects



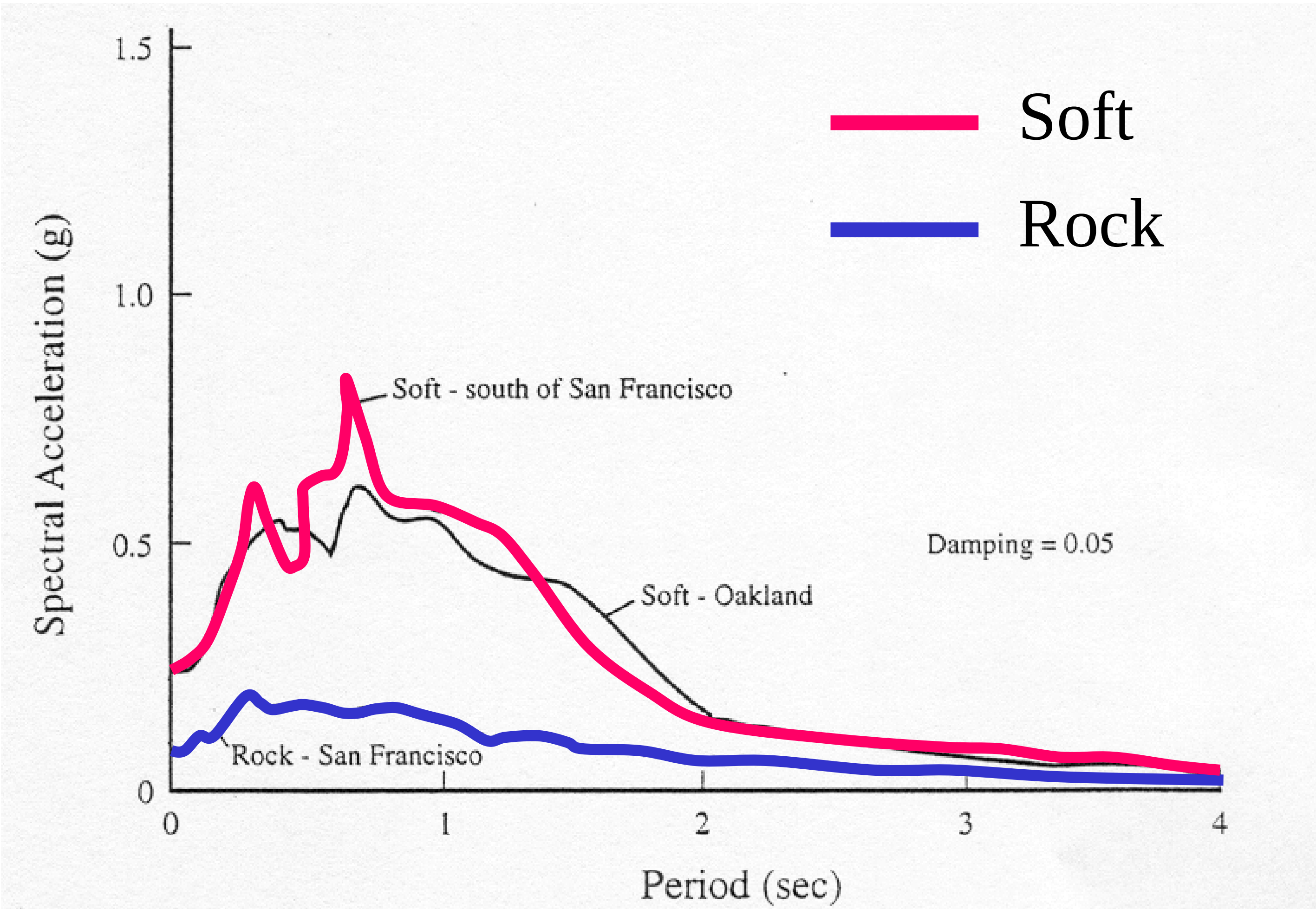
Site Amplification Effects

- Amplification of ground motion
- Longer duration of motion
- Change in frequency content of motion
- Not the same as soil-structure interaction

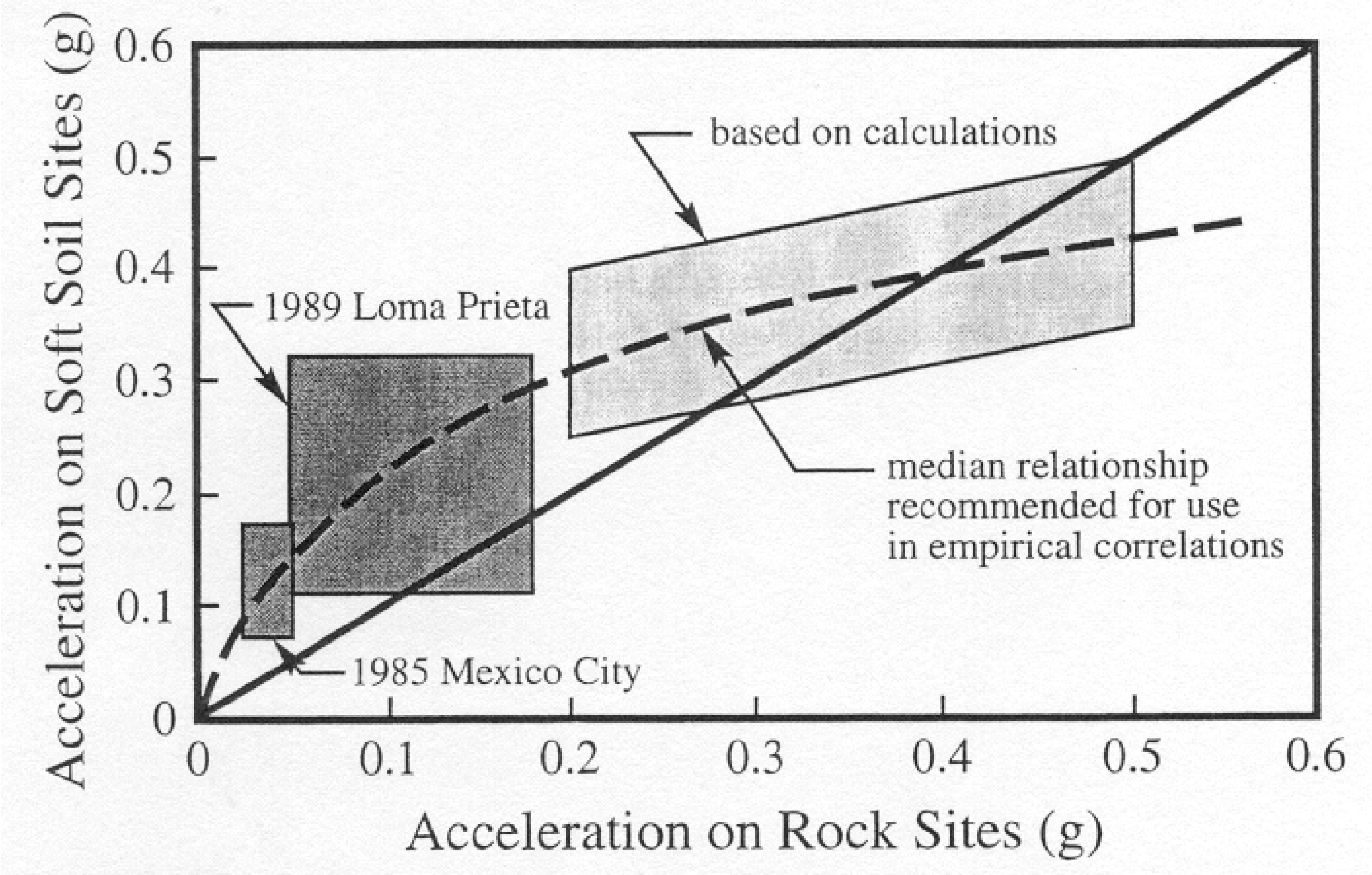
Site Amplification (Seed et al.)



Site Amplification: Loma Prieta Earthquake



Site Amplification: Loma Prieta and Mexico City Earthquakes



NEHRP Provisions Site Classes

A Hard rock v_s > 5000 ft/sec

B Rock: 2500 < v_s < 5000 ft/sec

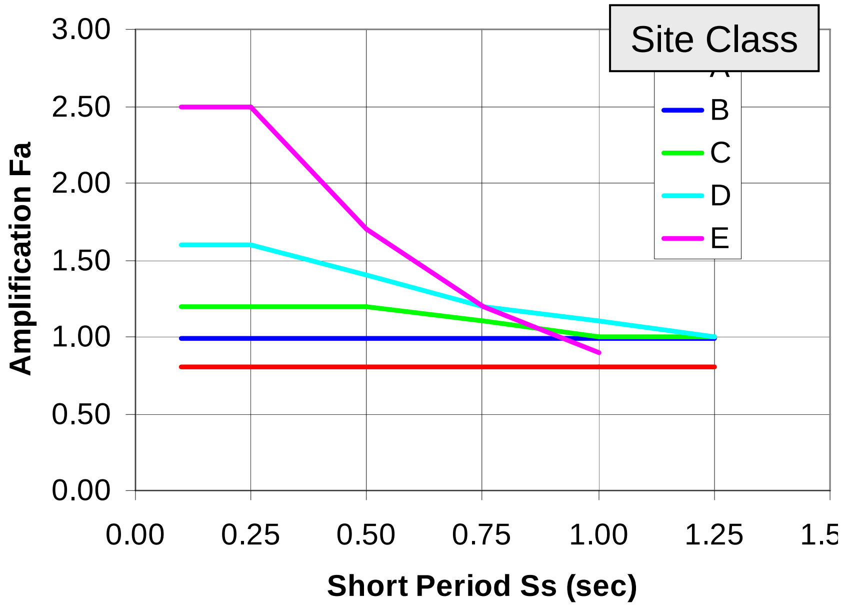
C Very dense soil or soft rock: 1200 < v_s < 2500 ft/sec

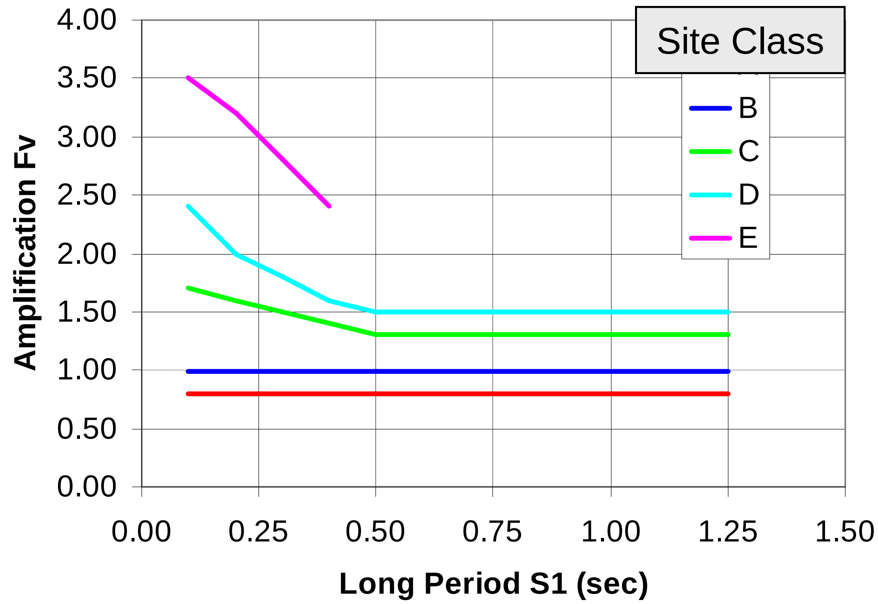
D Stiff soil : 600 < v_s < 1200 ft/sec

E V_s < 600 ft/sec

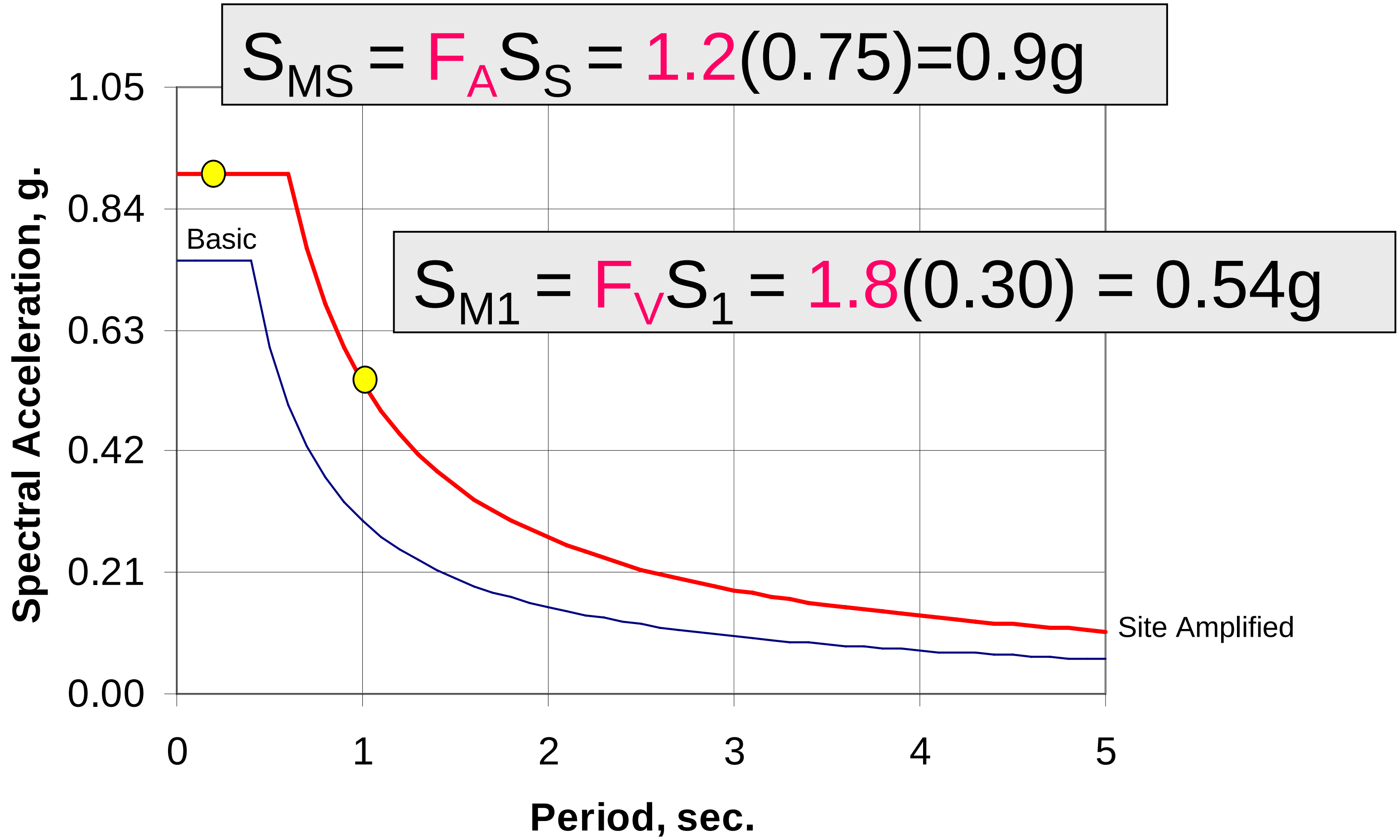
F Site-specific requirements

NEHRP Site Amplification for Site Classes A through E





2% in 50 Year 5% Damped MCE Elastic Spectra Modified for Site Class D



Scaling of *NEHRP Provisions* Spectra by 2/3 for “Margin of Performance”

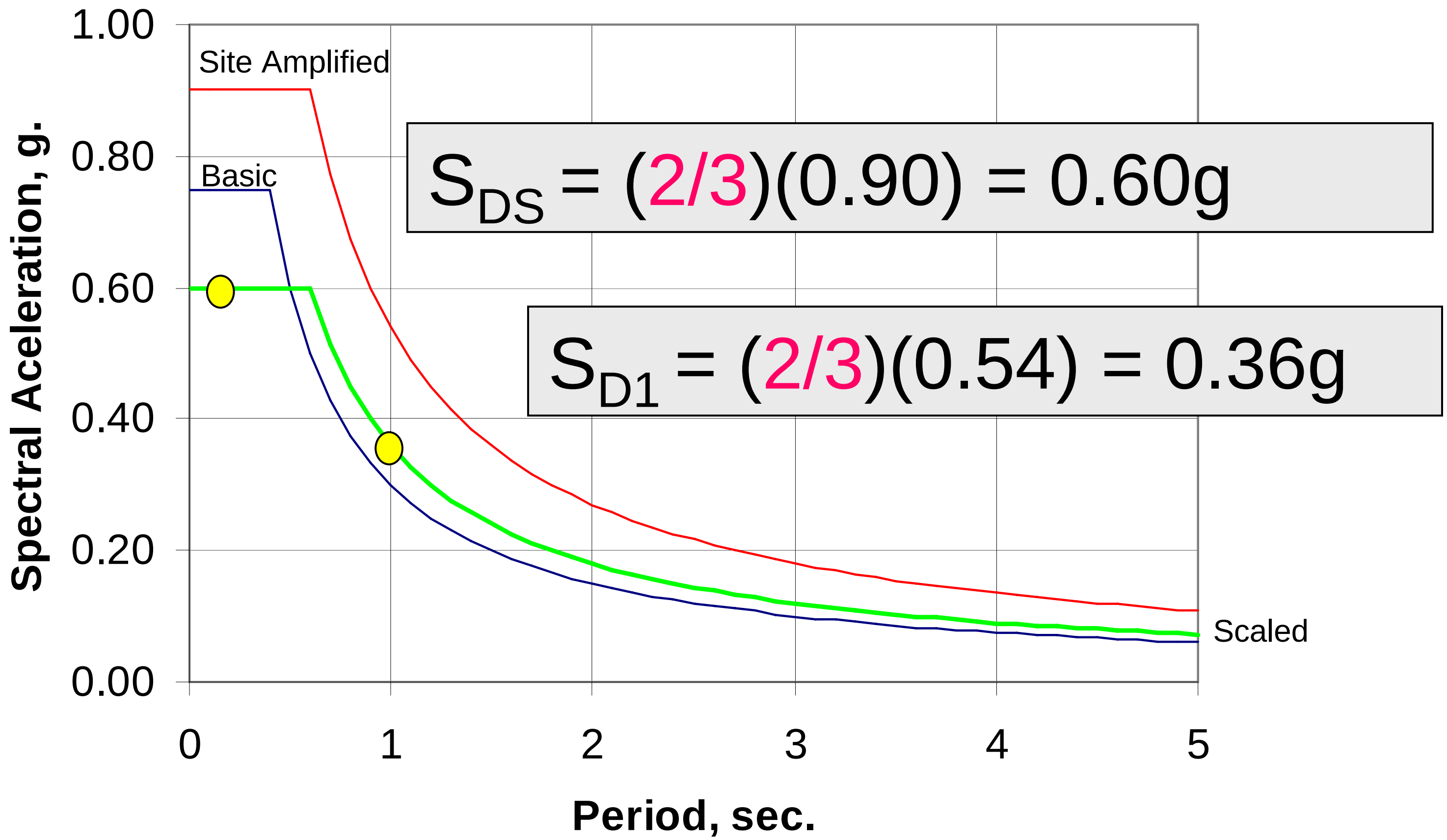
Buildings designed according to current procedures assumed to have margin of collapse of 1.5

Judgment of “lower bound” margin of collapse given by current design procedures

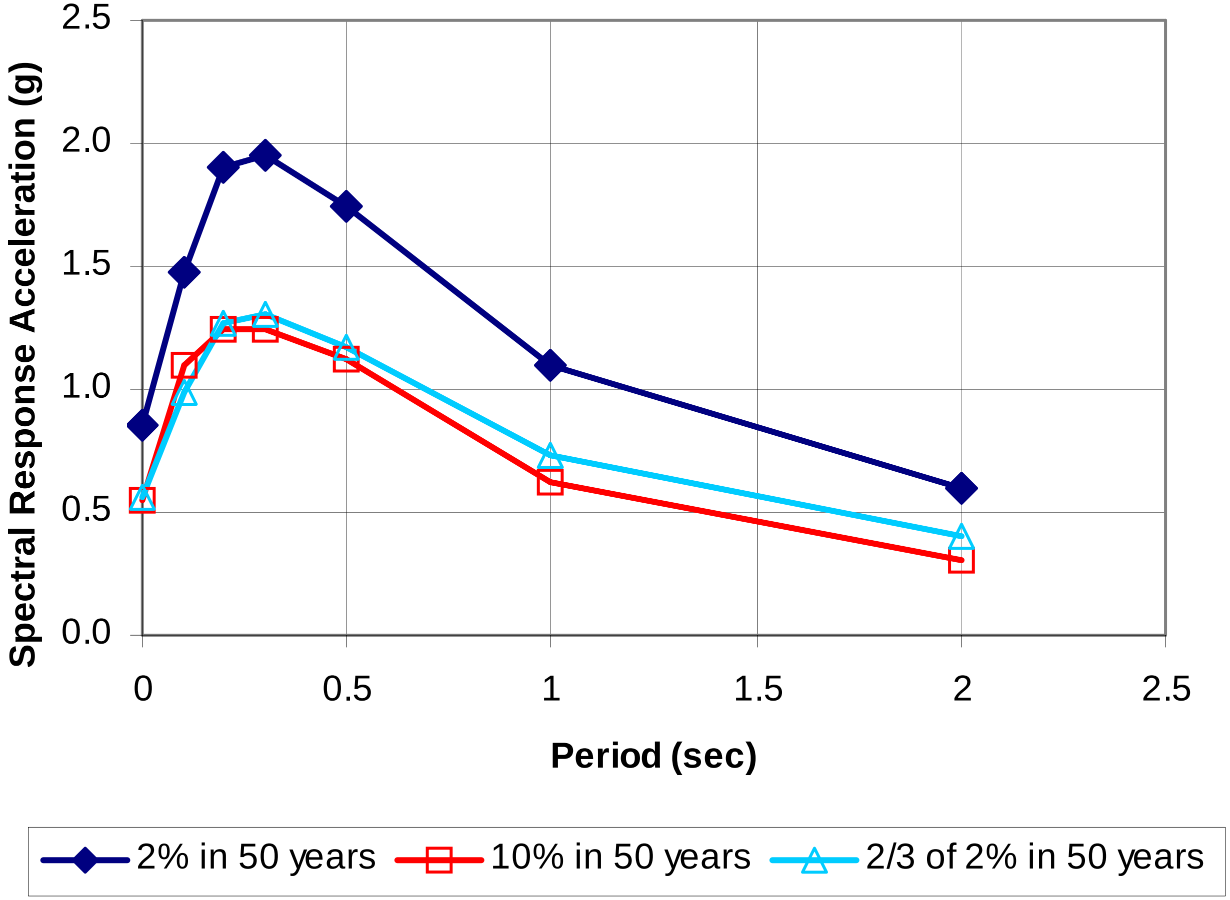
Design with current maps (2% in 50 year) but scale motions by 2/3

Results in 2/3 x 1.5 = 1.0 deterministic earthquake (where applicable)

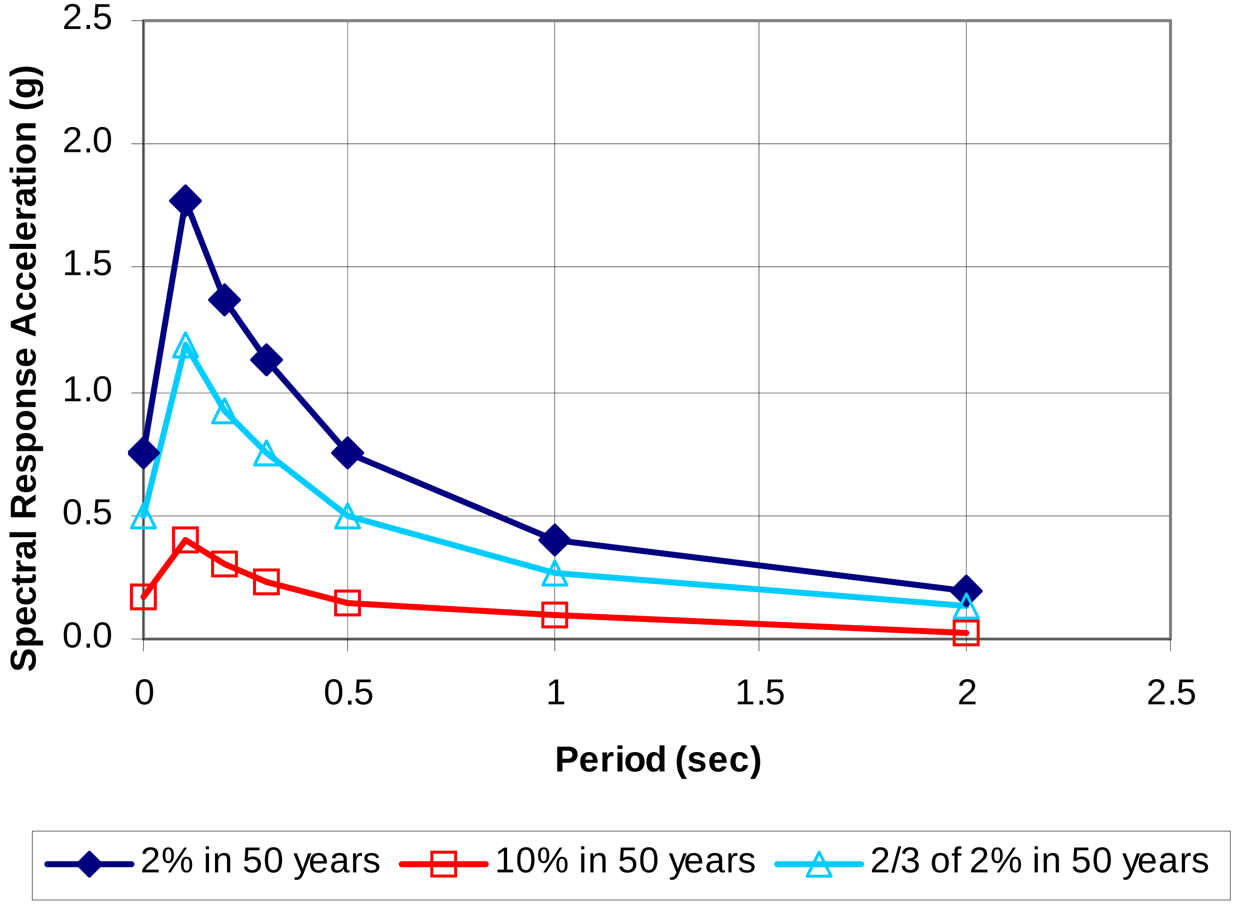
2% in 50 Year 5% Damped Elastic Design Spectra (Scaled by 2/3)



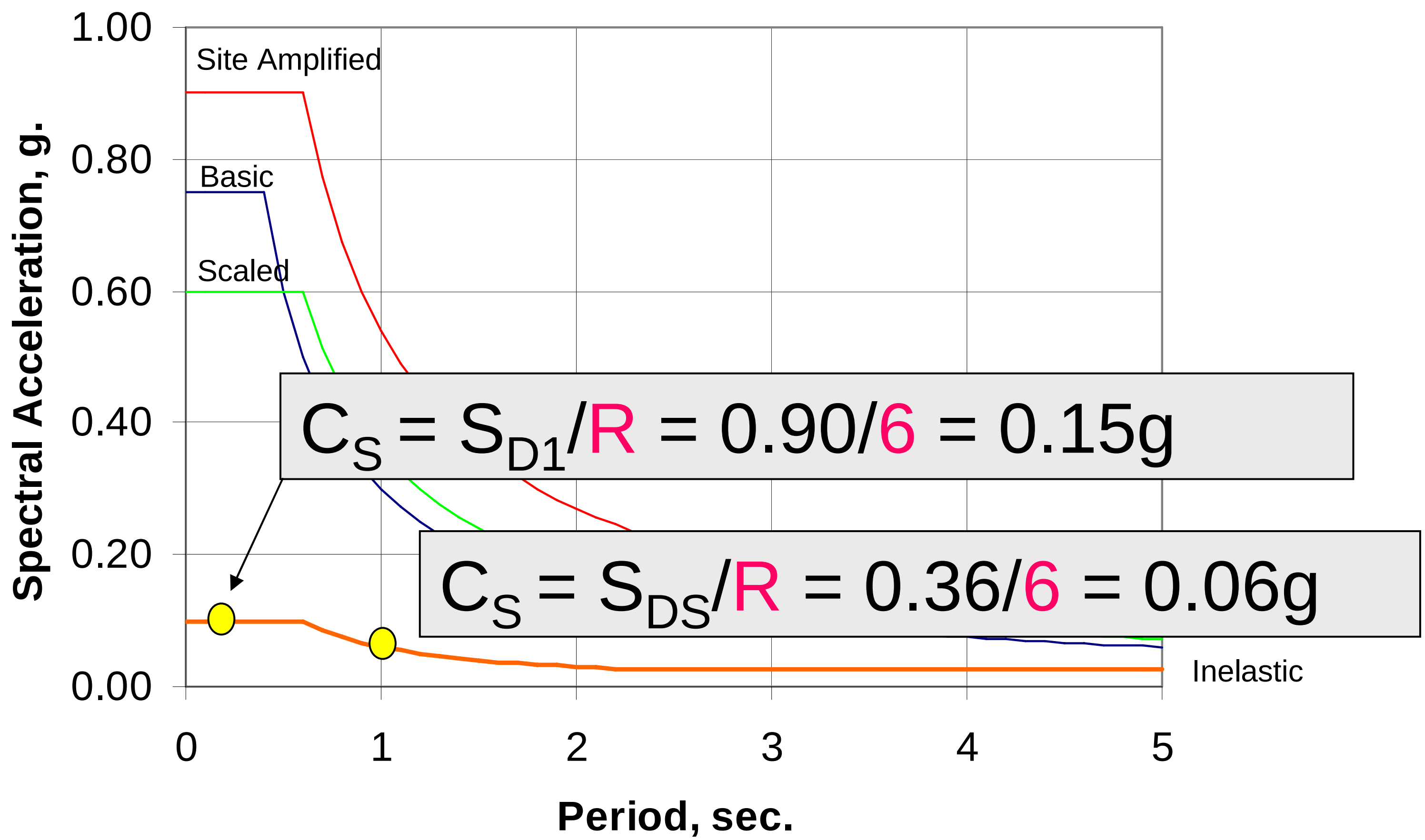
Effect of Scaling in Western United States



Effect of Scaling in Eastern United States



2% in 50 Year 5% Damped Inelastic Design Spectra (R=6, I=1) Site Class D



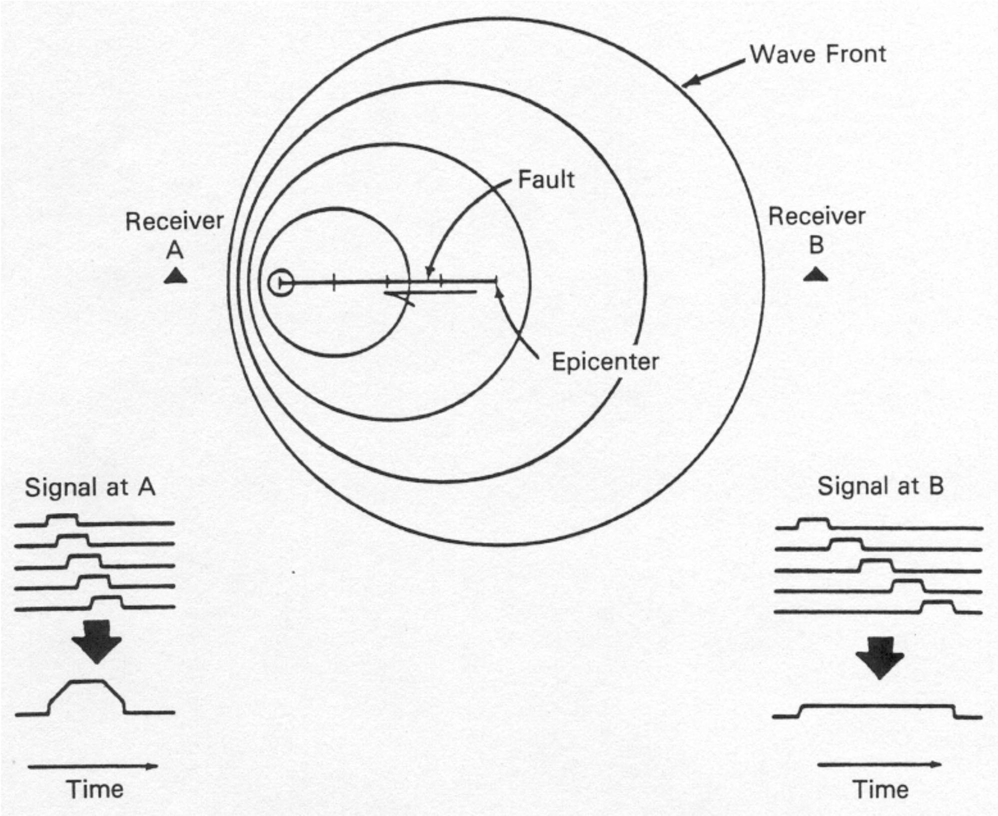
Basis for Reduction of Elastic Spectra by *R*

Inelastic behavior of structures

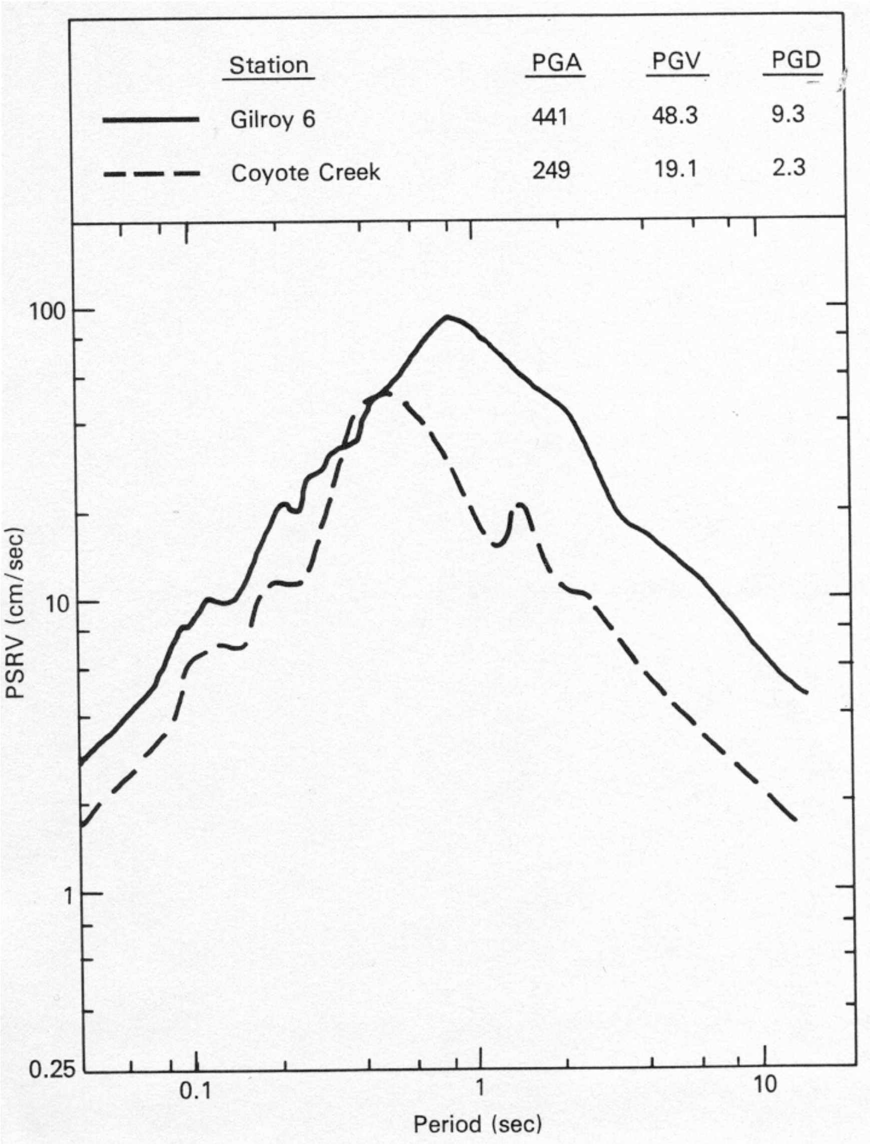
Methods for obtaining acceptable inelastic response are presented in later topics

Directionality and "Killer Pulse" Earthquakes

For sites relatively close to the fault, the direction of fault rupture can have an amplifying effect on ground motion amplitude.

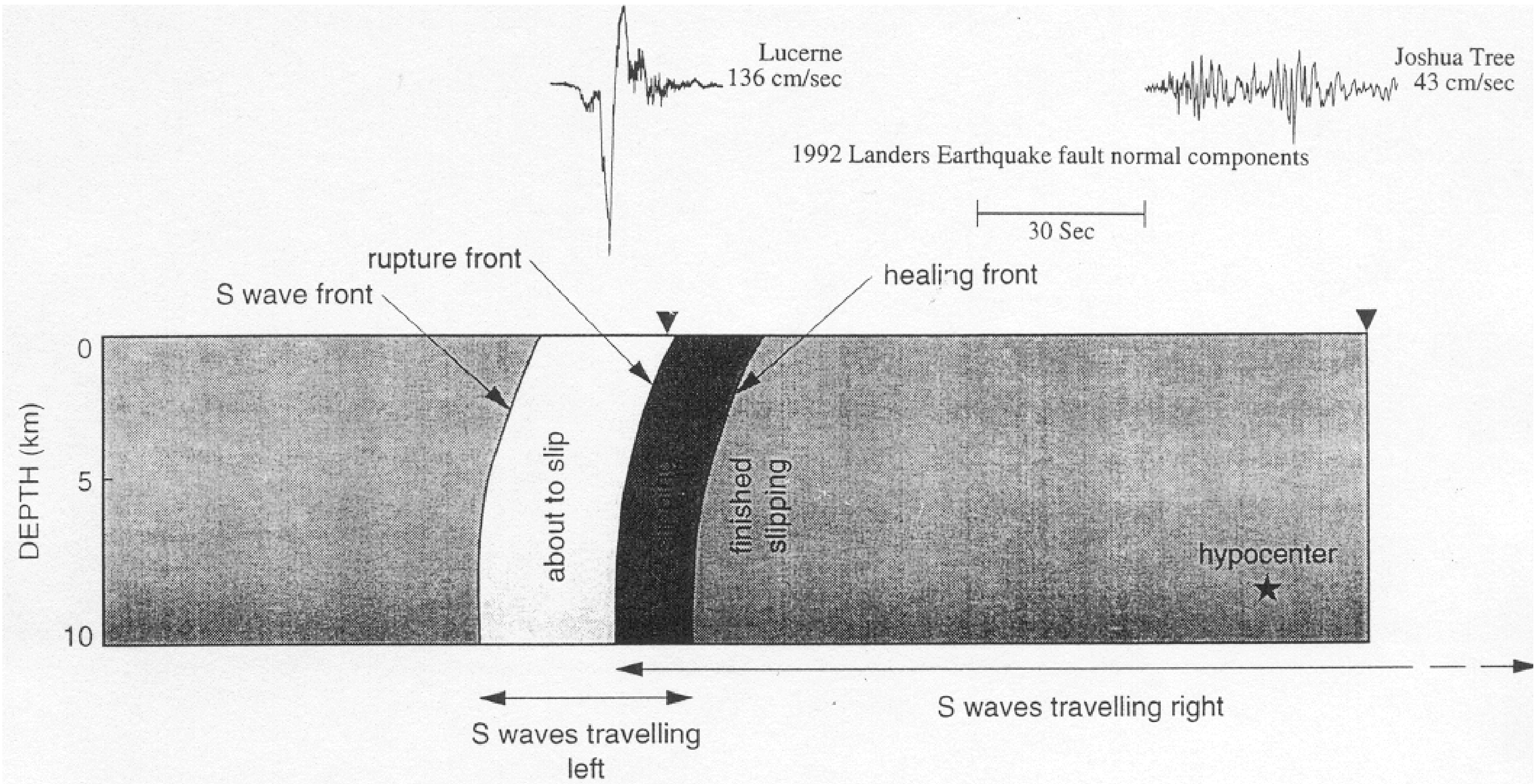


Effect of Directionality on Response Spectra



ards

Effect of Directionality on Ground Motion



GROUND MOTION MAPS

How To Obtain the Basic Values

Seismic Ground Motions

1 **Determine basic values from maps for bedrock conditions**

2, 3 Classify soil conditions at site and determine site coefficients

4 Determine site-adjusted values

4 Take two-thirds for use in design

5 Construct design response spectrum

7 Site-specific studies permitted/required

Mapped Acceleration Parameters

- Two sets of maps; acceleration parameter is in units of gravity
- S_S for spectral response acceleration at 0.2 sec
- S_1 for spectral response acceleration at 1.0 sec
- Shortcut to Seismic Design Category A:
  - $S_S < 0.15$ and $S_1 < 0.04$

Ground Motion Parameters & Seismic Hazard

Mapped Contours of S_S

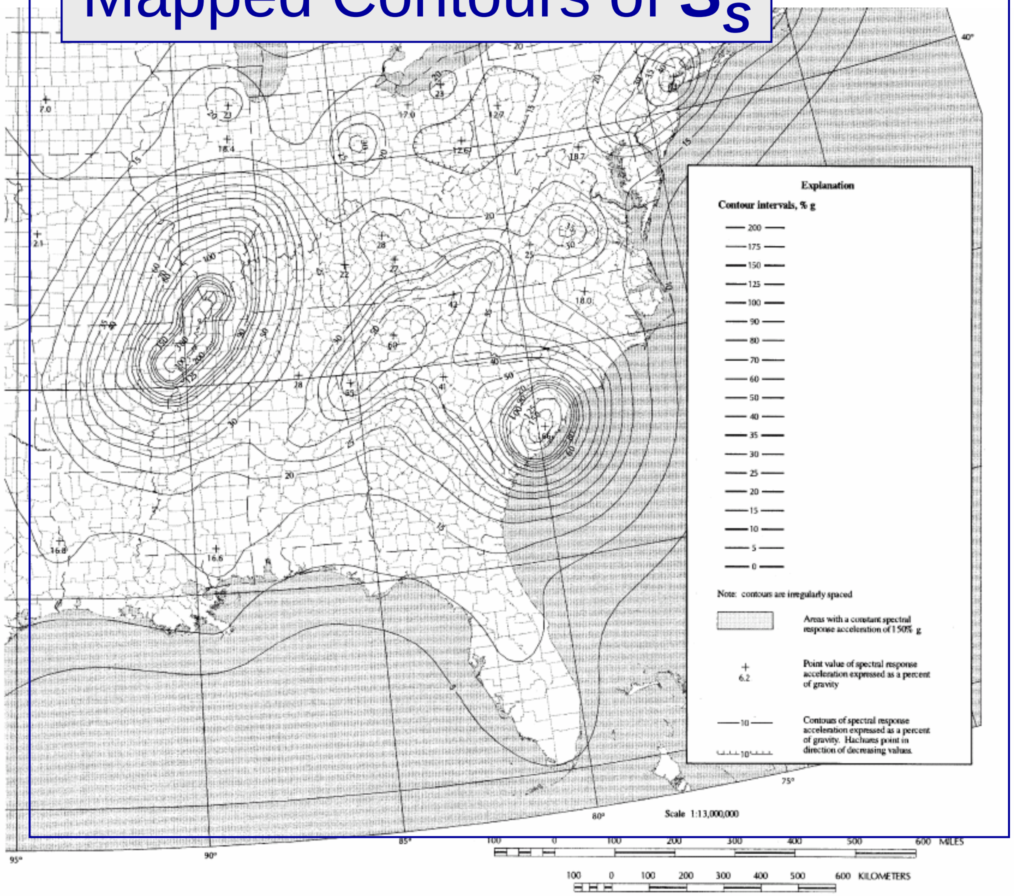


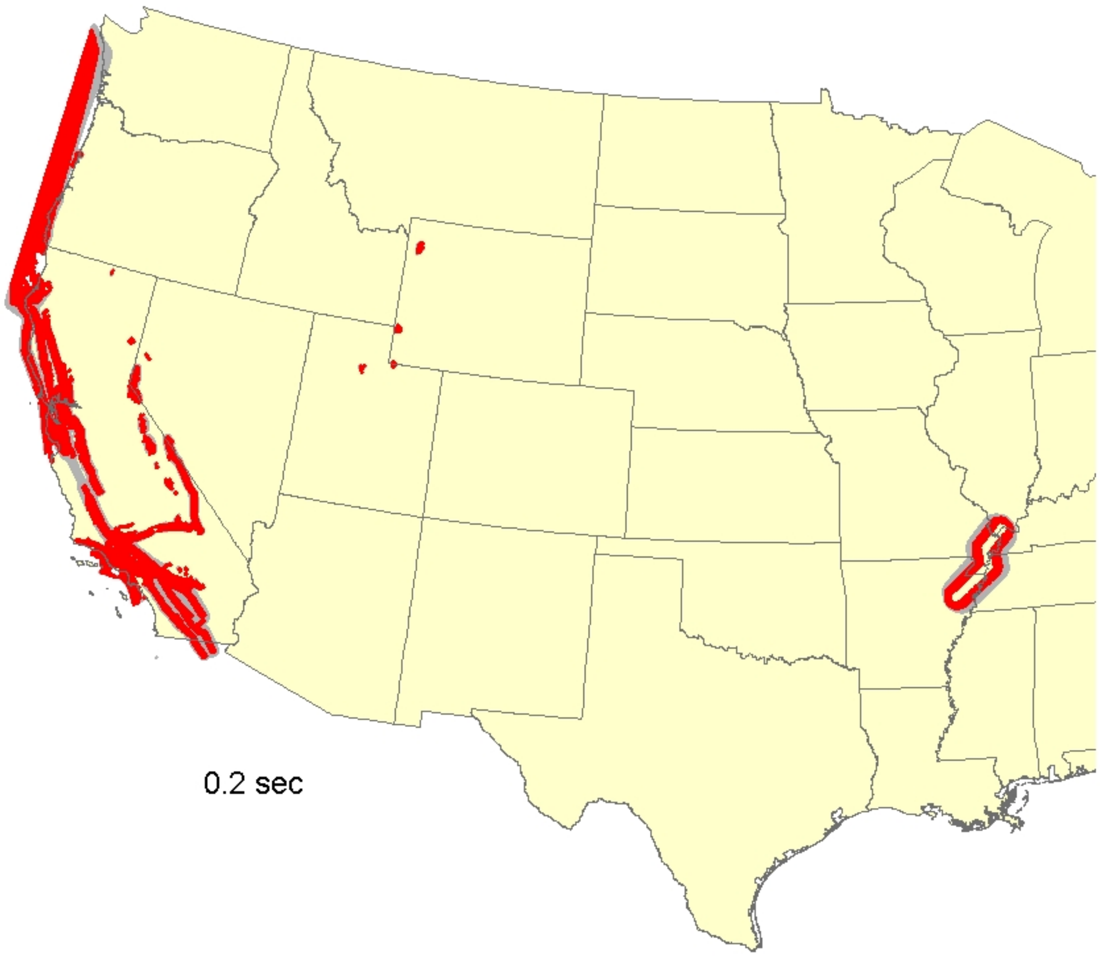
FIGURE 9.4.1.1(a) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR CONTERMINOUS UNITED STATES. OF 0.2 s SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

S_S and S_1 are the mapped 2% in 50 year spectral accelerations for firm rock

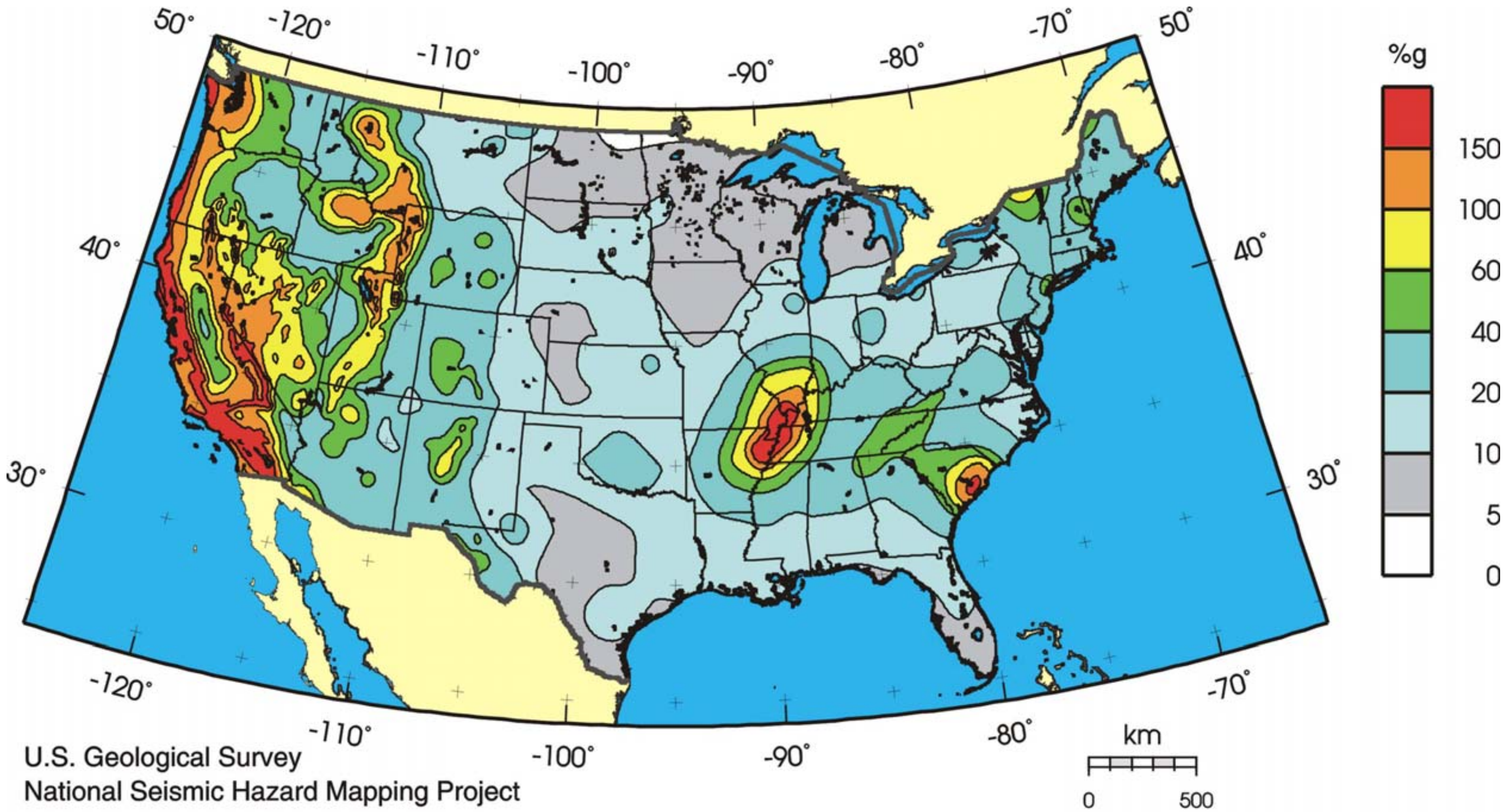
S_{DS} and S_{D1} are the design level spectral accelerations (modified for site and "expected good performance")

Long-Period Transition Maps

Location of Deterministic Areas



Typical Probabilistic Map



S_S - 0.2 Spectral Response Acceleration

CD vs Internet

- Internet
- CD
- Both sources give the same answers
- Both sources have a similar user interface
- The graphics are somewhat different

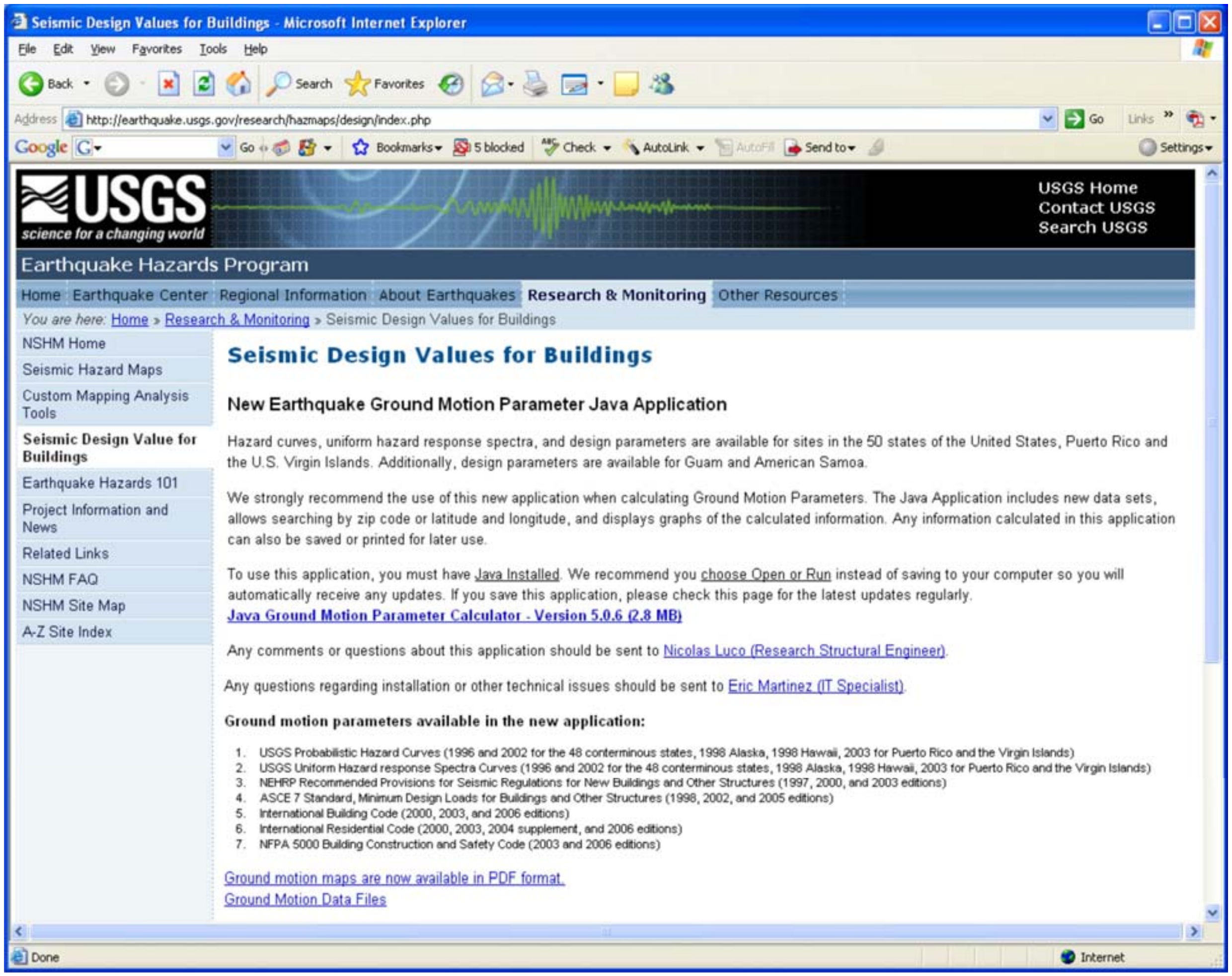
Internet Ground Motion Tool

http://earthquake.usgs.gov/research/hazmaps/

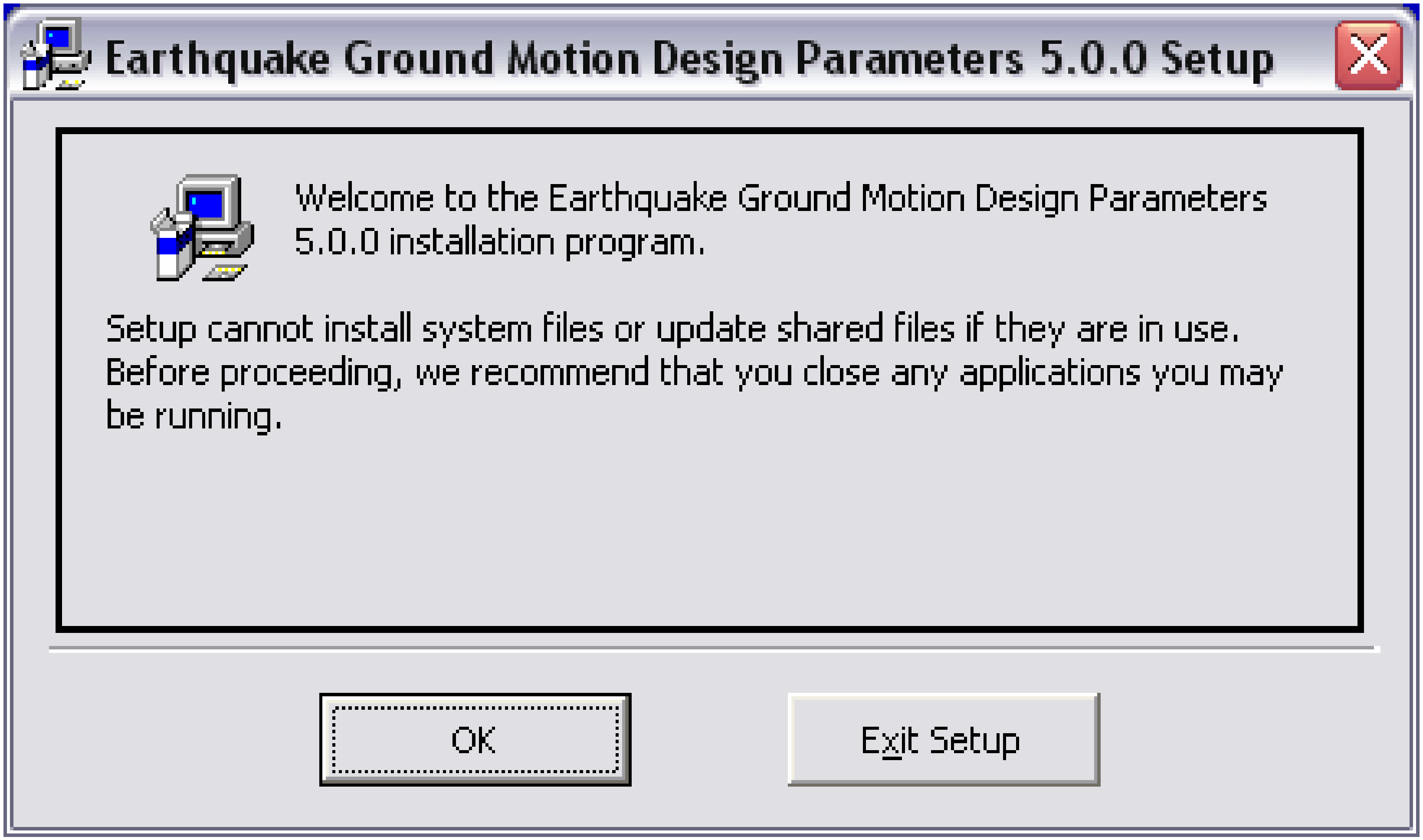
SEISMIC DESIGN VALUES FOR BUILDINGS

***S_s* and *S_1*, Hazard Curves, Uniform Hazard Spectra, and Residential Design Category**

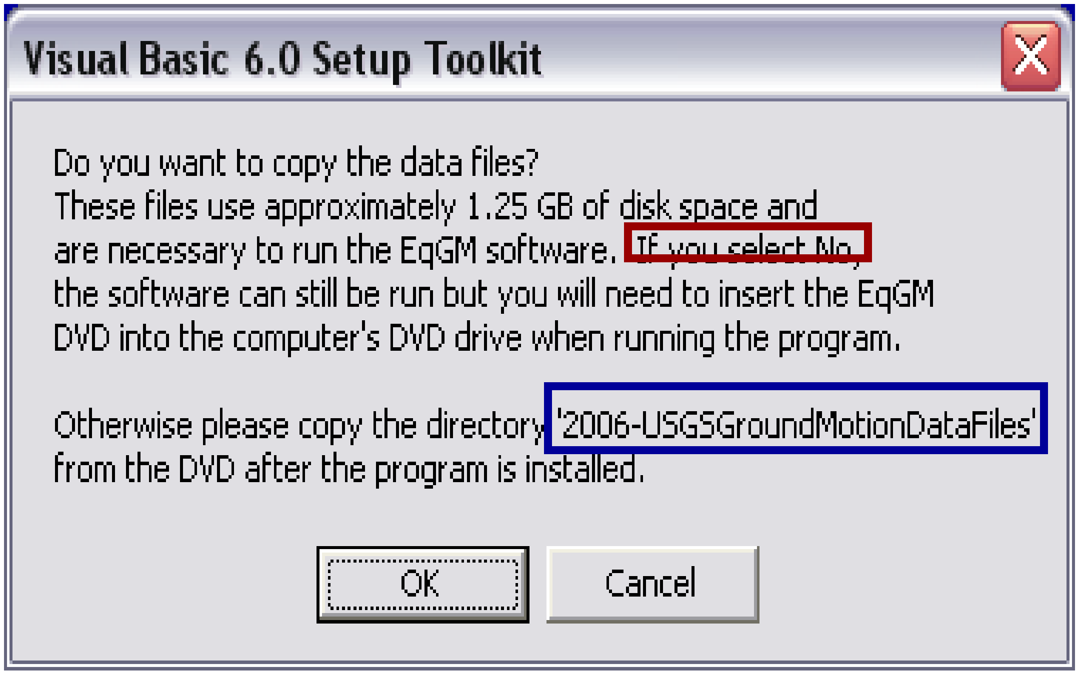
USGS Ground Motion Calculator



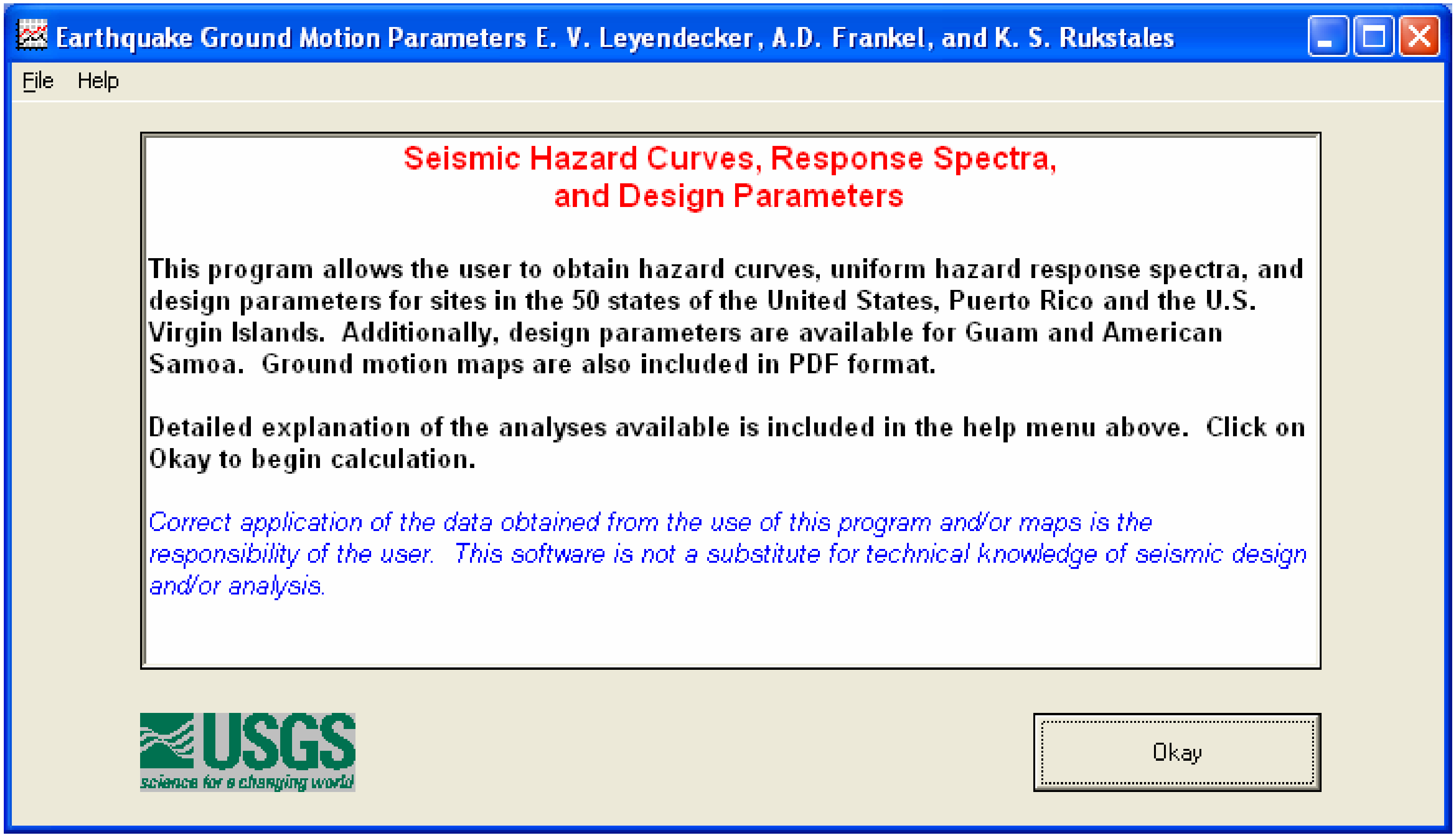
Installation



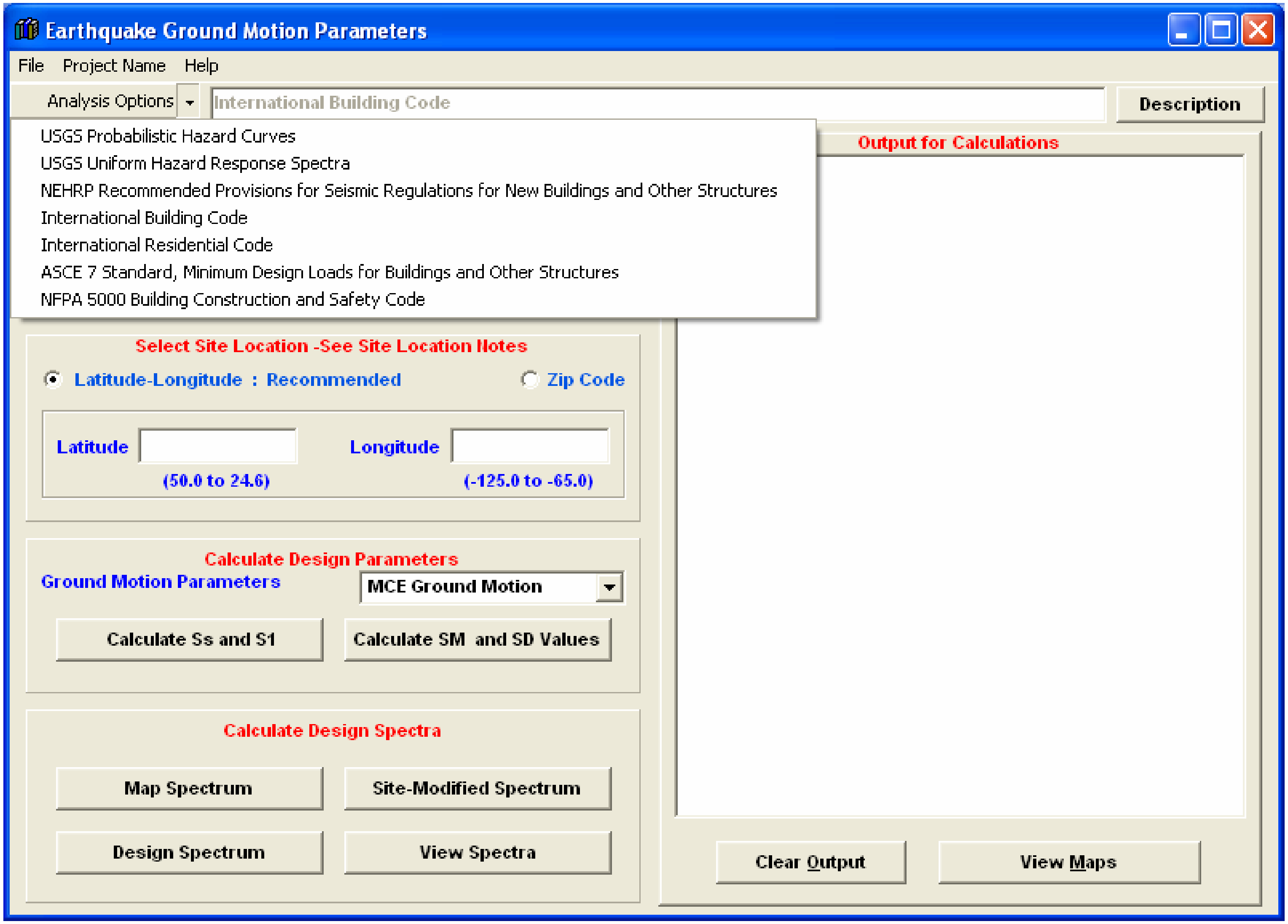
Installation Caution



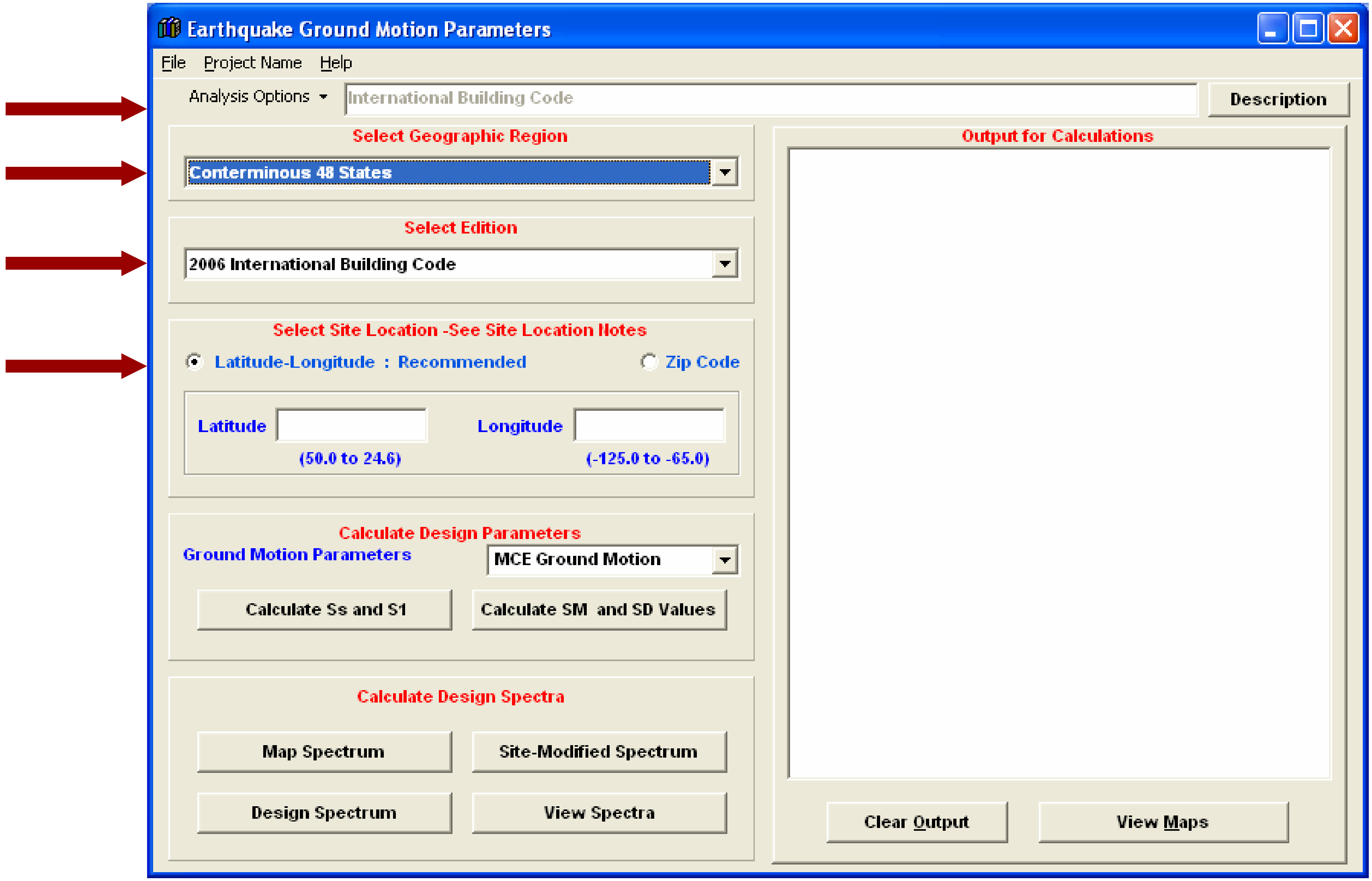
Opening Screen



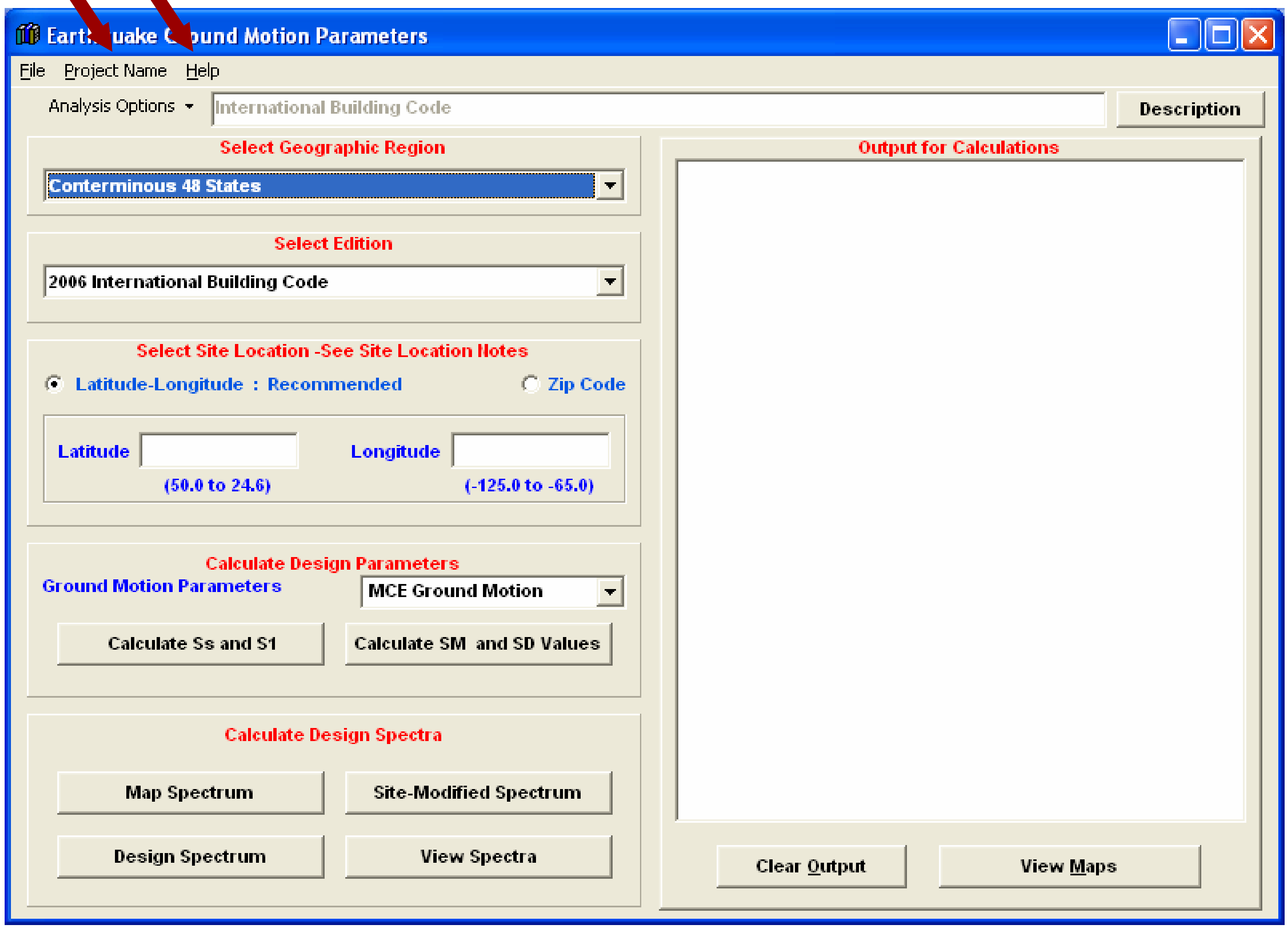
Analysis Options



IBC Option

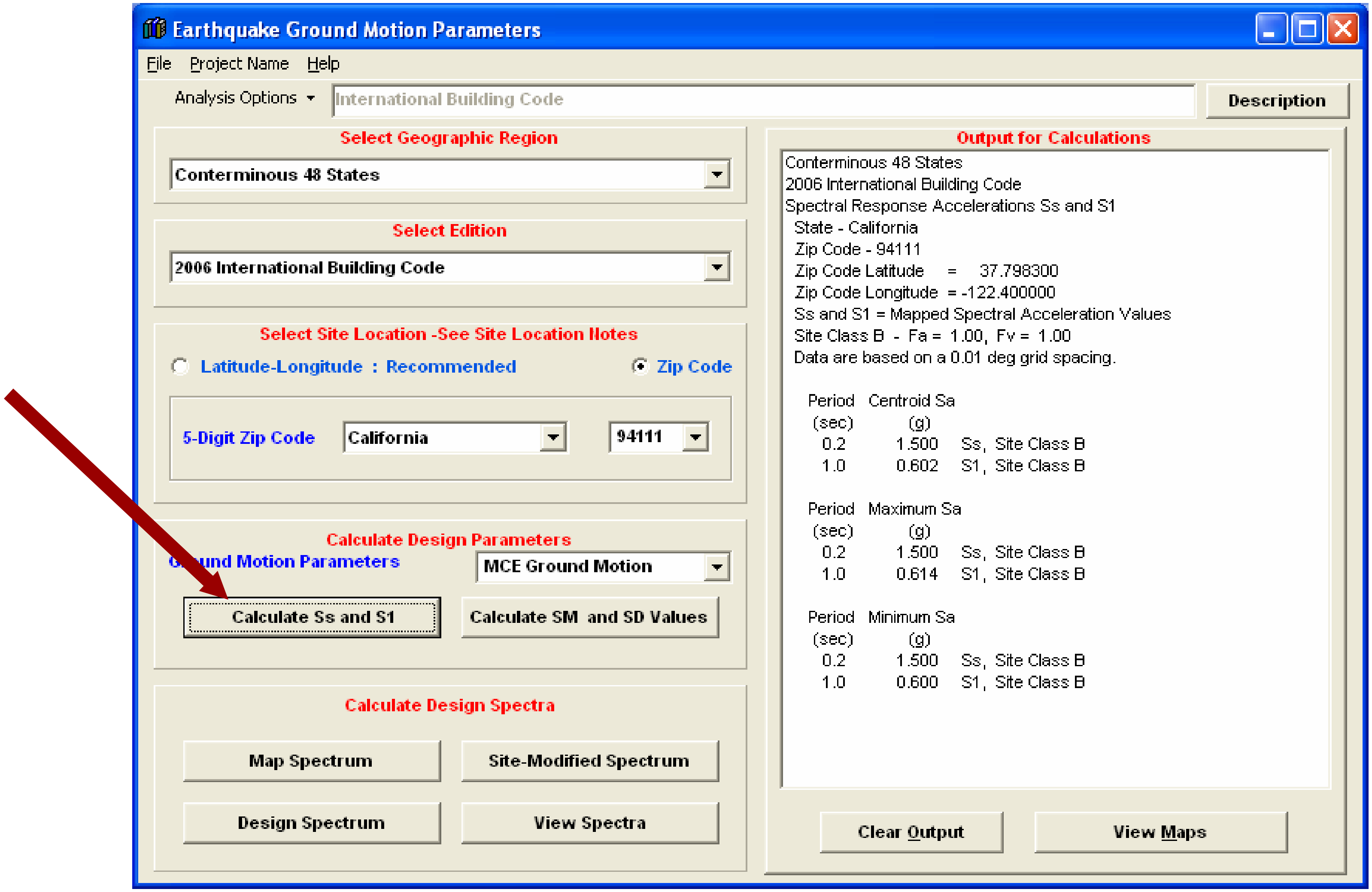

Earthquake Ground Motion Parameters
File
Project Name
Help
Analysis Options
International Building Code
Description
Select Geographic Region
Conterminous 48 States
Select Edition
2006 International Building Code
Select Site Location -See Site Location Notes
Latitude-Longitude : Recommended
Zip Code
Latitude
(50.0 to 24.6)
Longitude
(-125.0 to -65.0)
Calculate Design Parameters
Ground Motion Parameters
MCE Ground Motion
Calculate Ss and S1
Calculate SM and SD Values
Calculate Design Spectra
Map Spectrum
Site-Modified Spectrum
Design Spectrum
View Spectra
Output for Calculations
Clear Output
View Maps

User Aids

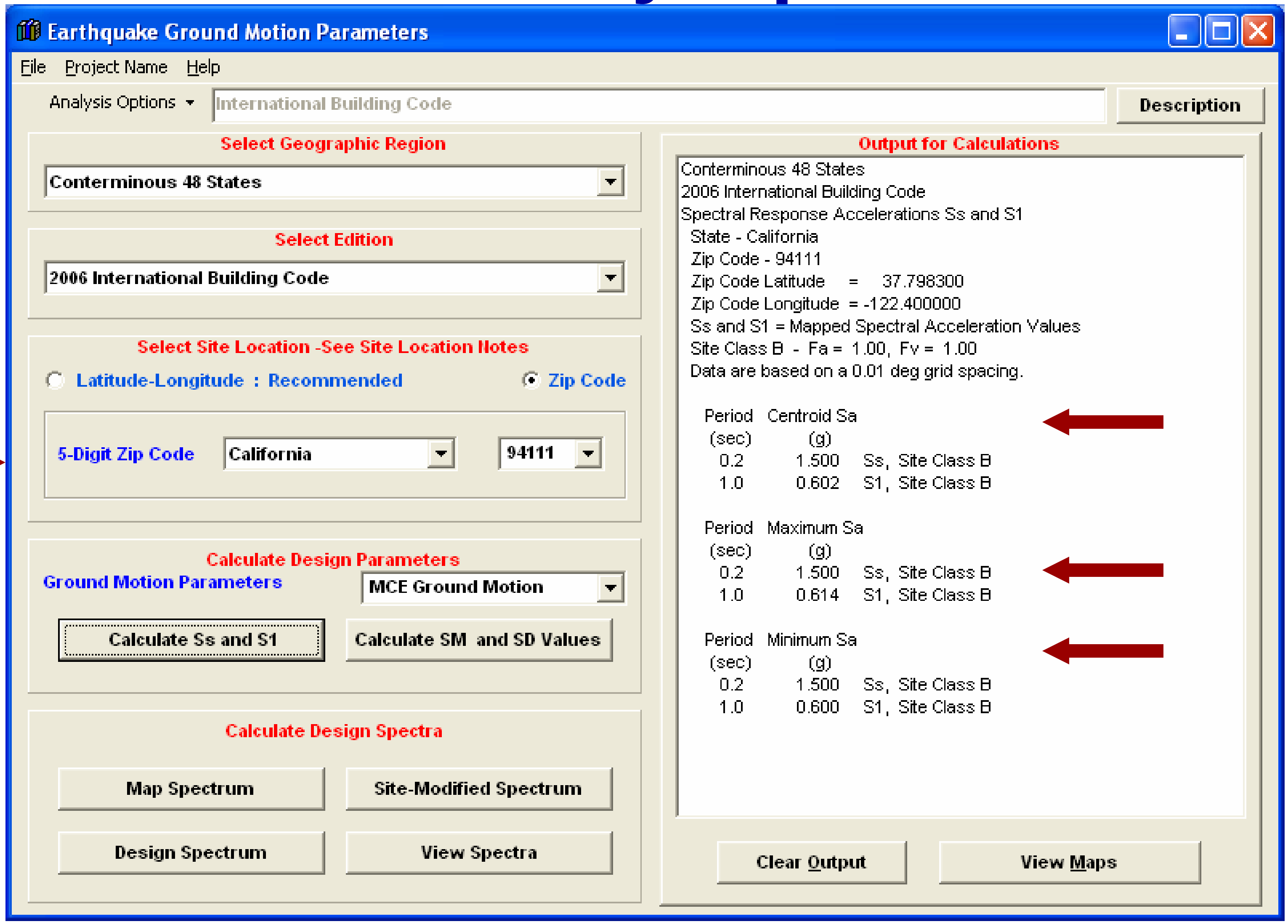


Earthquake Ground Motion Parameters
File
Project Name
Help
Analysis Options
International Building Code
Description
Select Geographic Region
Conterminous 48 States
Select Edition
2006 International Building Code
Select Site Location -See Site Location Notes
Latitude-Longitude : Recommended
Zip Code
Latitude
(50.0 to 24.6)
Longitude
(-125.0 to -65.0)
Calculate Design Parameters
Ground Motion Parameters
MCE Ground Motion
Calculate Ss and S1
Calculate SM and SD Values
Calculate Design Spectra
Map Spectrum
Site-Modified Spectrum
Design Spectrum
View Spectra
Output for Calculations
Clear Output
View Maps

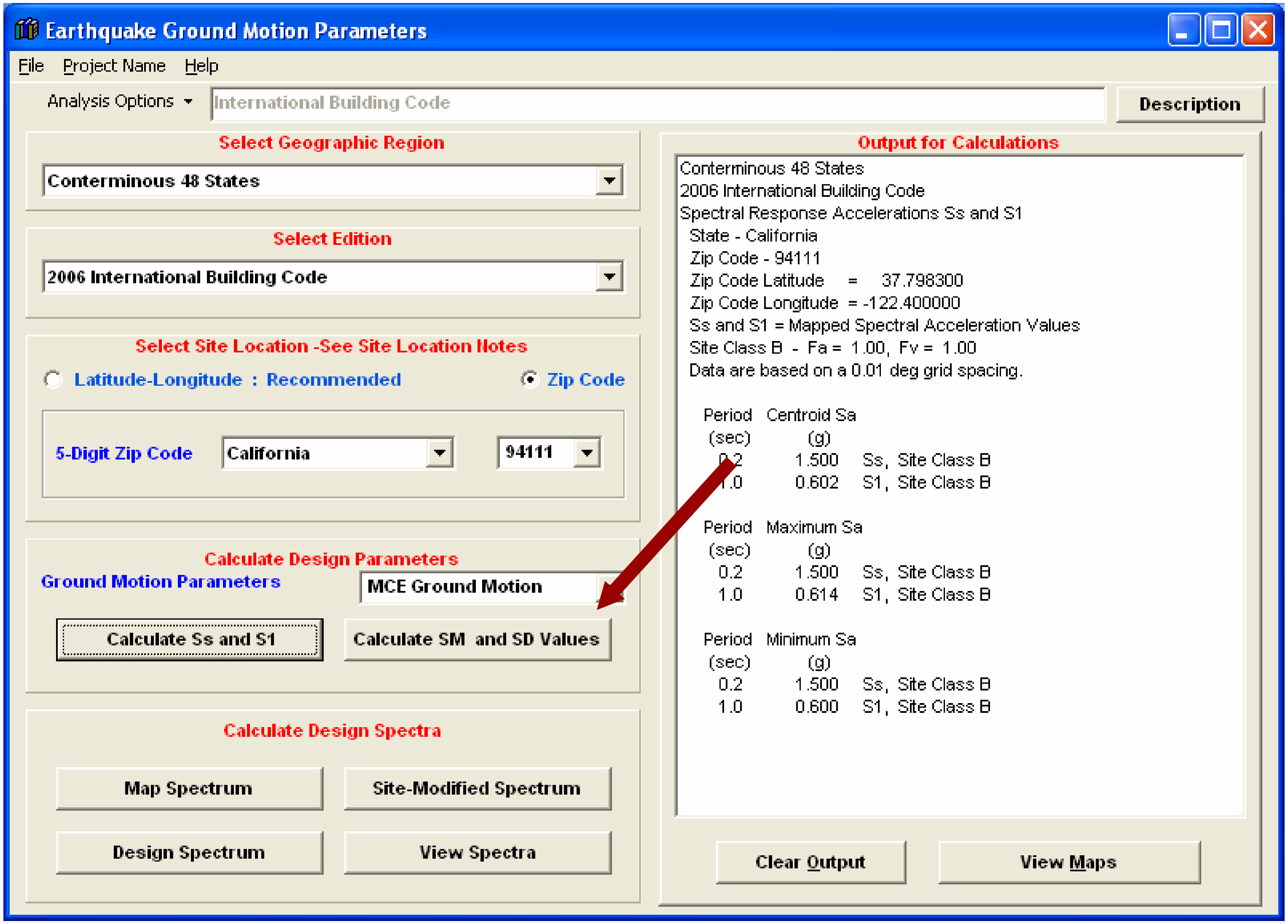
Calculate S_S AND S_1



Location By Zipcode



Calculate S_{MS}, S_{M1}, S_{DS}, S_{D1}



Calculate Site Coefficients

Site Coefficients

Soil Factors as a Function of Site Class and Spectral Accelerations

Values of Fa as a Function of Site Class and 0.2 sec MCE Spectral Acceleration

Site Class	Ss <= 0.25	Ss = 0.50	Ss = 0.75	Ss = 1.00	Ss >= 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	0.9
F	a	a	a	a	a

Values of Fv as a Function of Site Class and 1.0 sec MCE Spectral Acceleration

Site Class	S1 <= 0.10	S1 = 0.20	S1 = 0.30	S1 = 0.40	S1 >= 0.50
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	2.4
F	a	a	a	a	a

Notes:

Use straight-line interpolation for intermediate values of Sa and S1.

Note a: Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Calculate Site Coefficient

Spectral Accelerations

Ss, g: 1.500 S1, g: 0.602

Site Class

- Site Class A
- Site Class B
- Site Class C
- Site Class D
- Site Class E
- Site Class F

Discussion

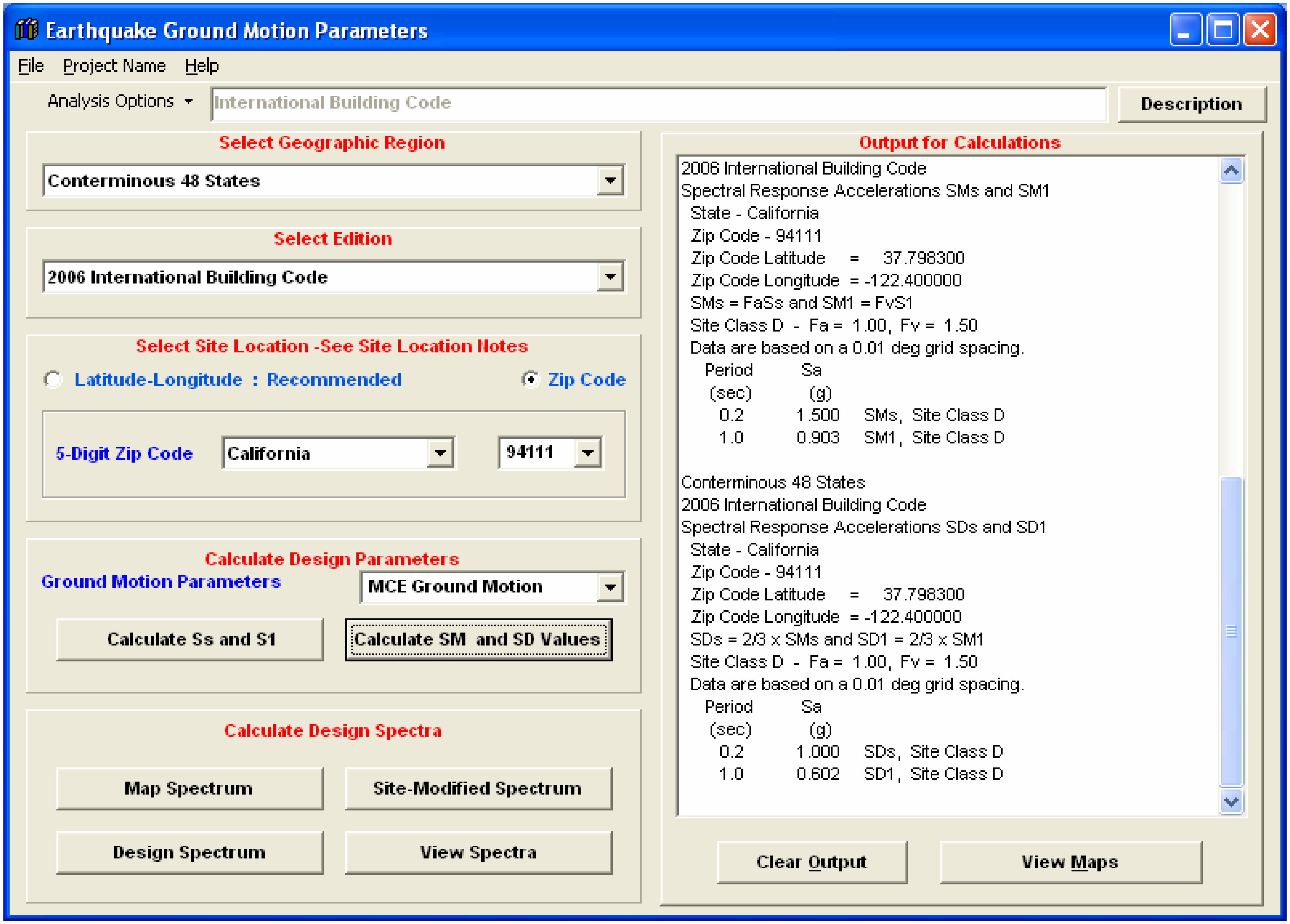
Site Coefficients

Interpolated soil factors for the conditions shown. Values may also be entered manually.

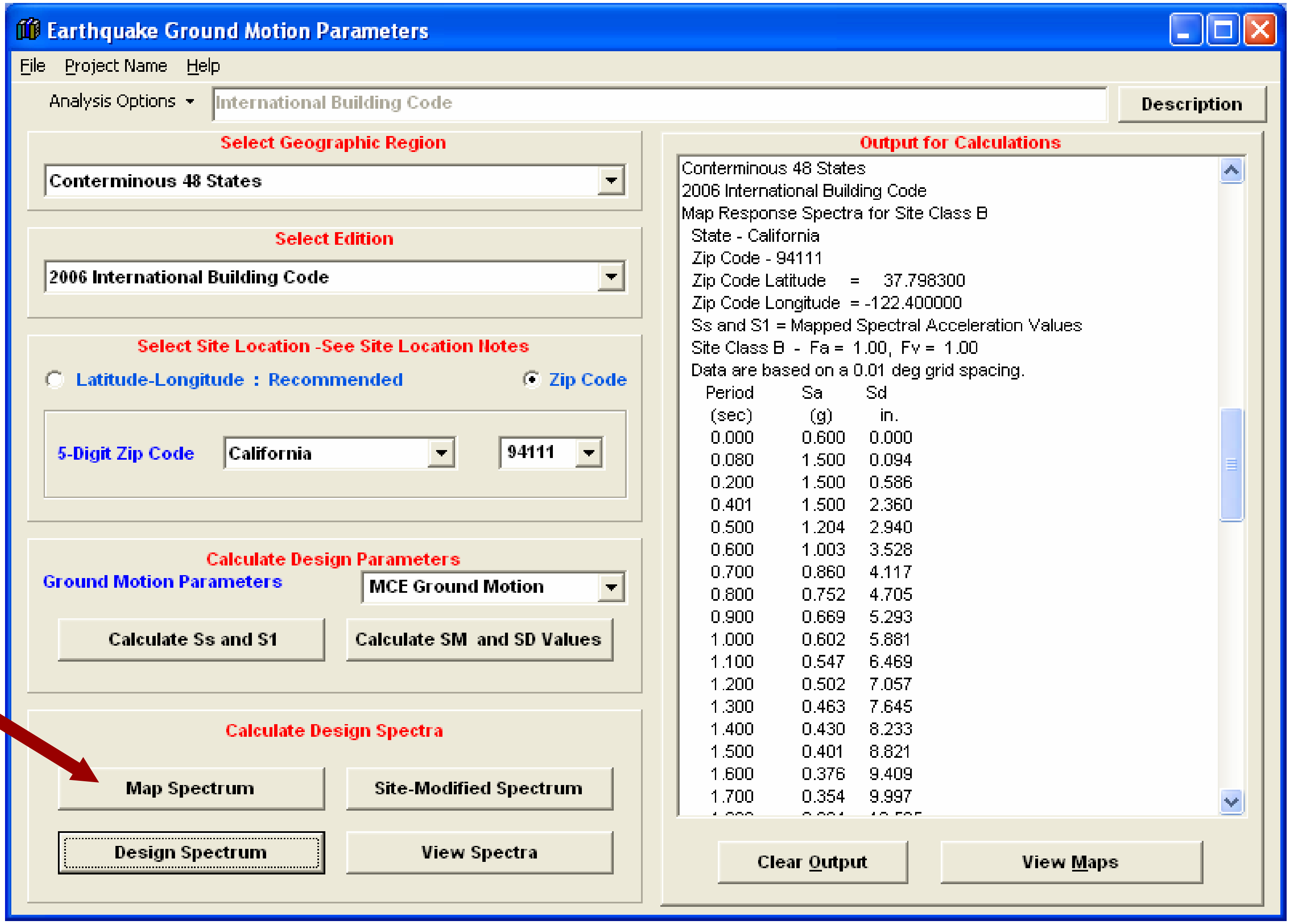
Fa 1.00 Fv 1.50

OK

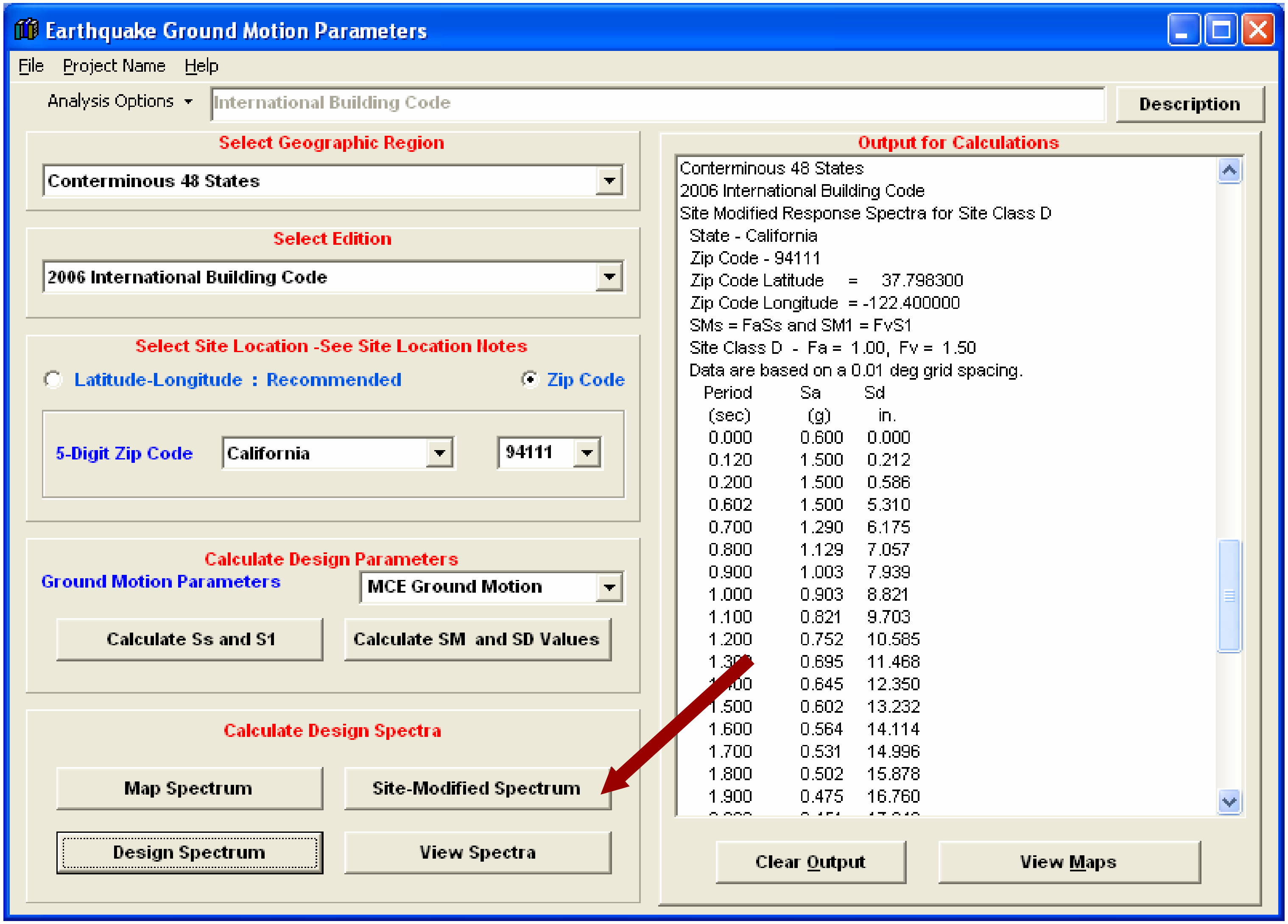
S_{MS}, S_{M1}, S_{DS}, S_{D1} Values



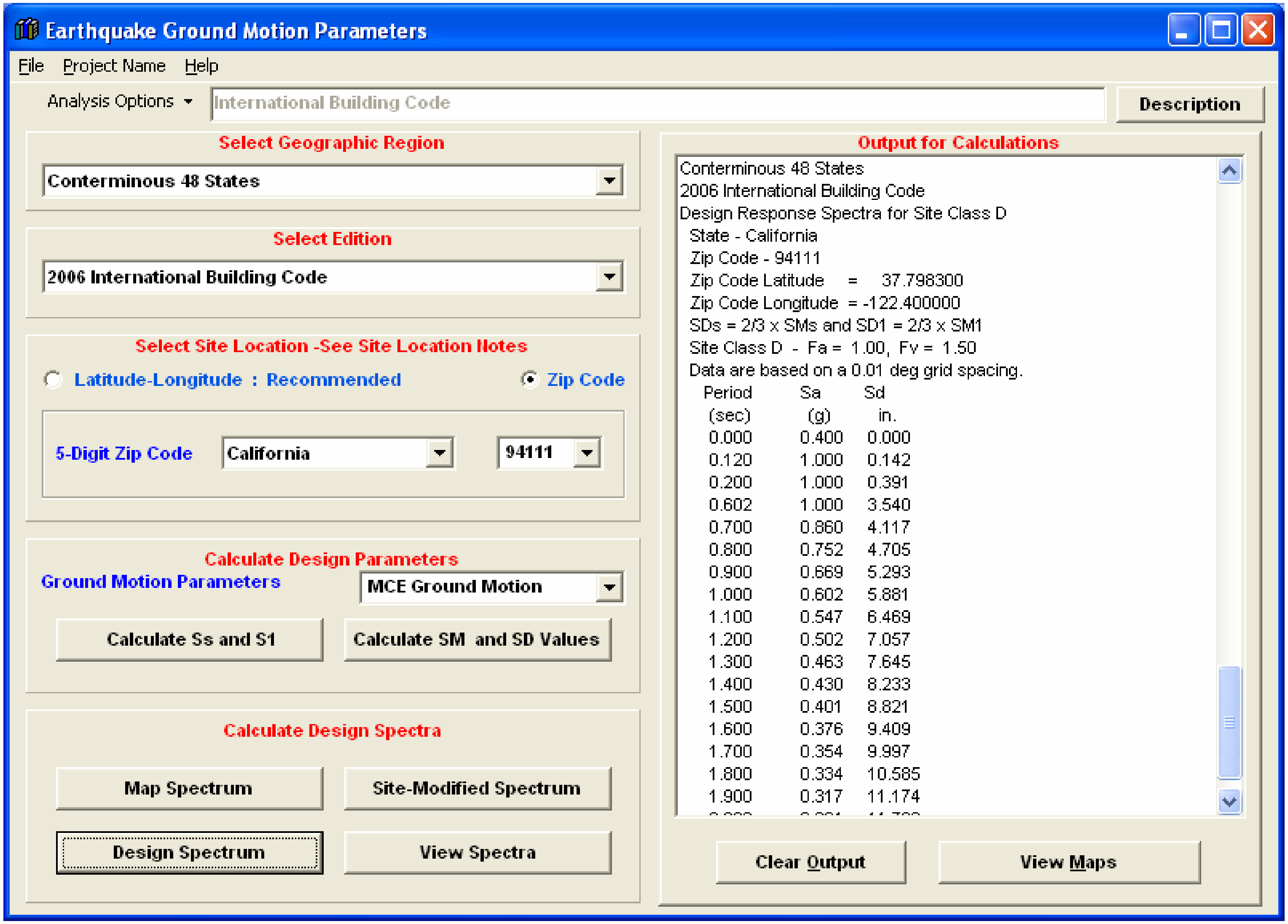
Calculate MCE Spectrum



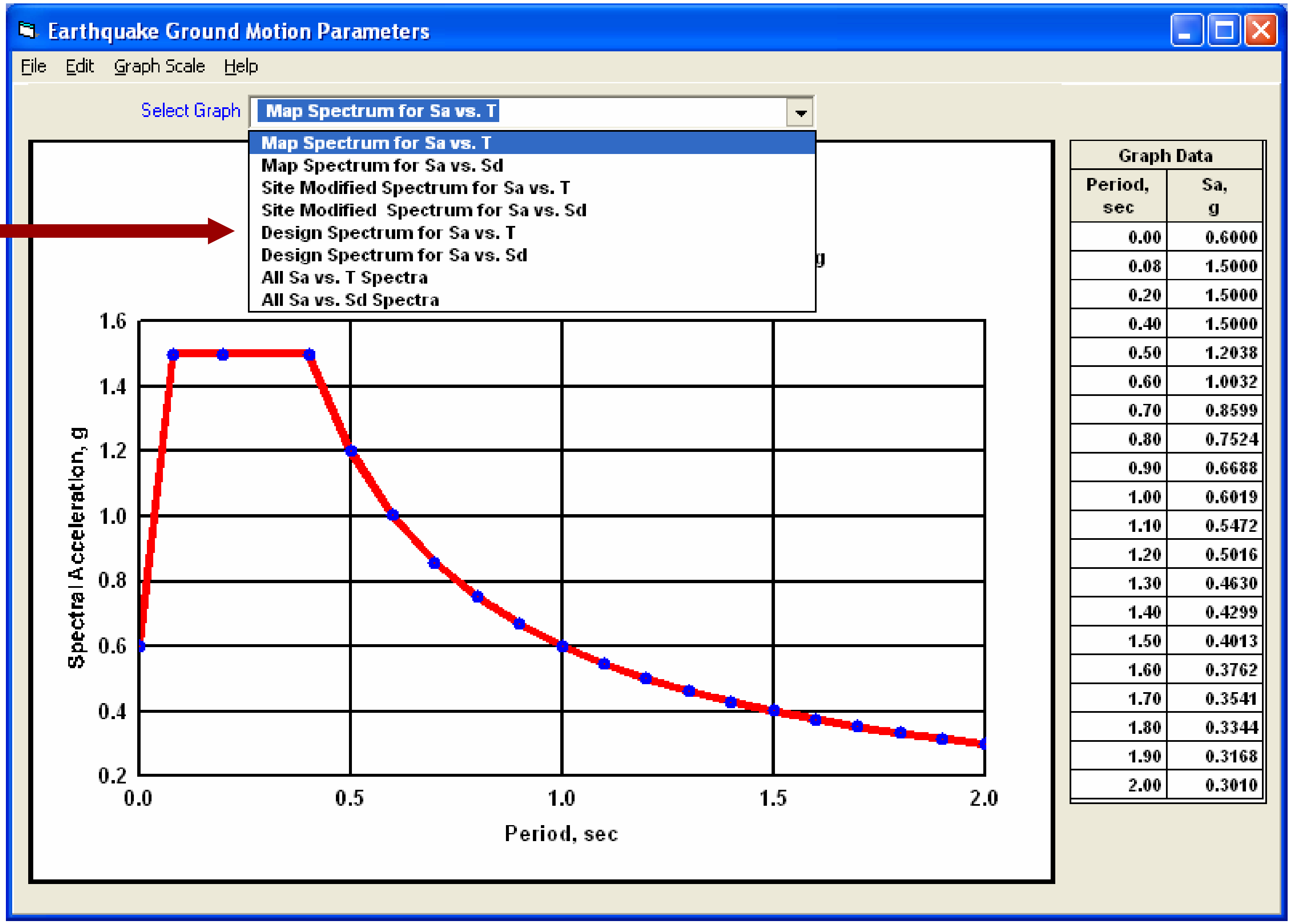
Calculate S_M Spectrum



Calculate S_D Spectrum



Graphic Options

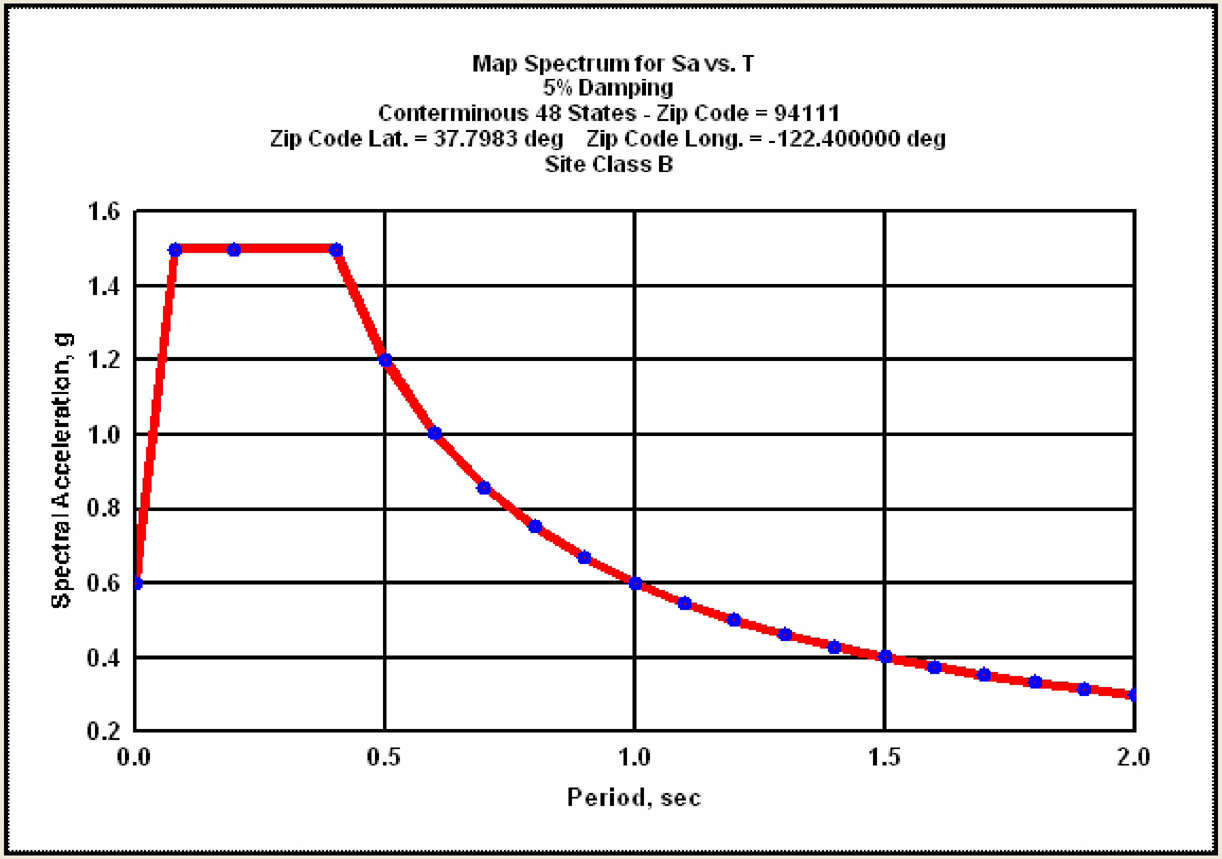


Map Spectrum: S_a - T

Earthquake Ground Motion Parameters

File Edit Graph Scale Help

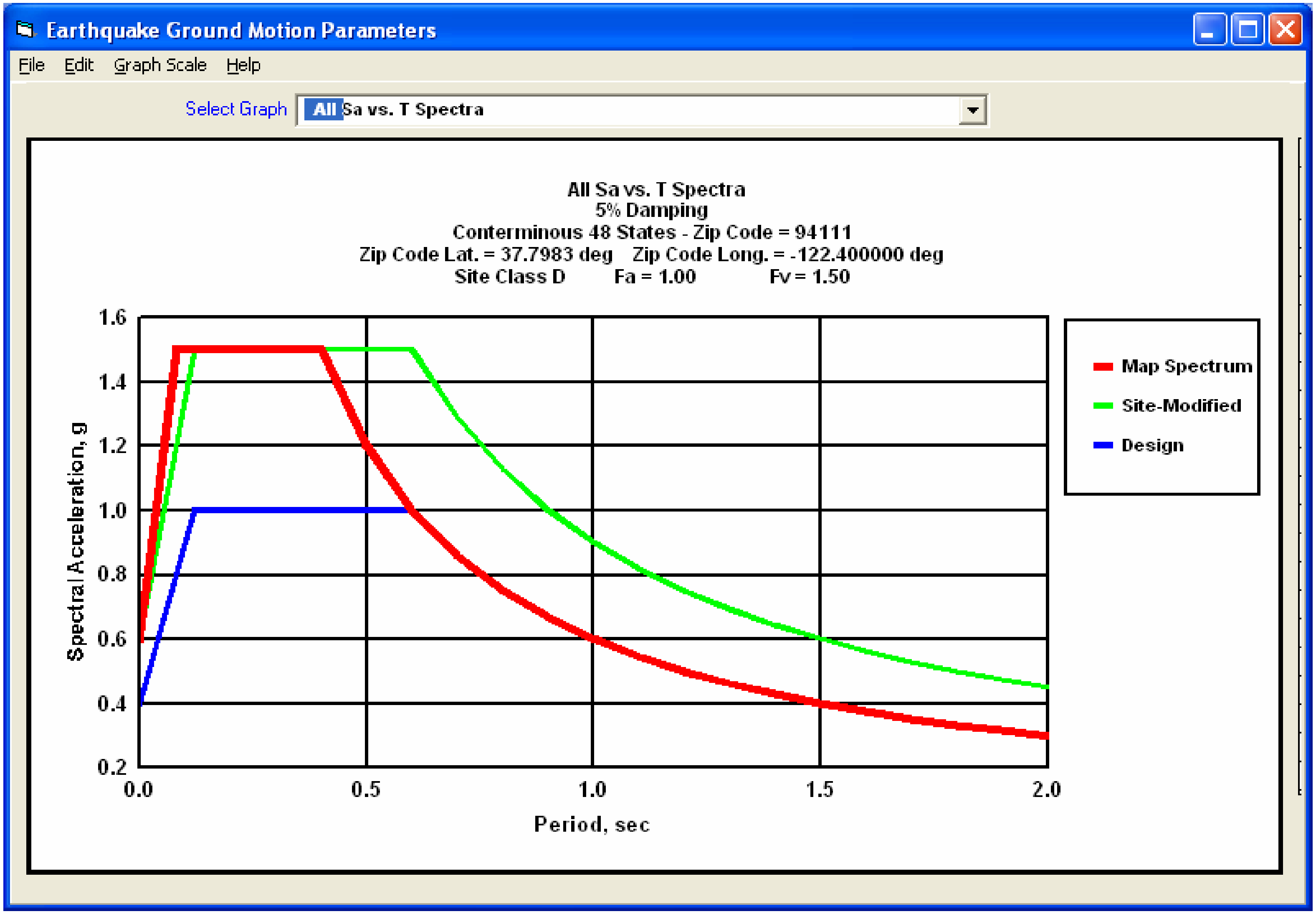
Select Graph: Map Spectrum for Sa vs. T



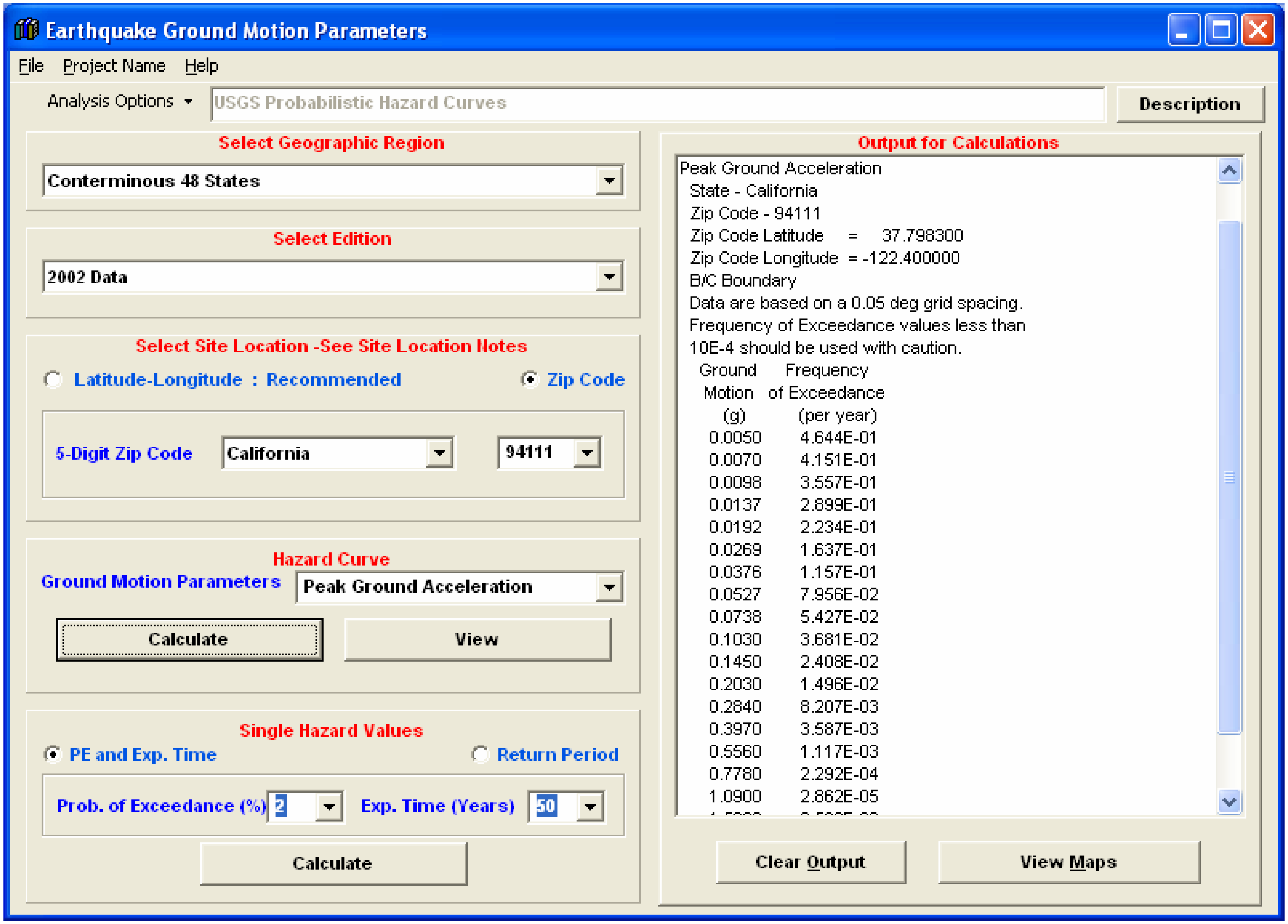
Graph Data

Period, sec	Sa, g
0.00	0.6000
0.08	1.5000
0.20	1.5000
0.40	1.5000
0.50	1.2038
0.60	1.0032
0.70	0.8599
0.80	0.7524
0.90	0.6688
1.00	0.6019
1.10	0.5472
1.20	0.5016
1.30	0.4630
1.40	0.4299
1.50	0.4013
1.60	0.3762
1.70	0.3541
1.80	0.3344
1.90	0.3168
2.00	0.3010

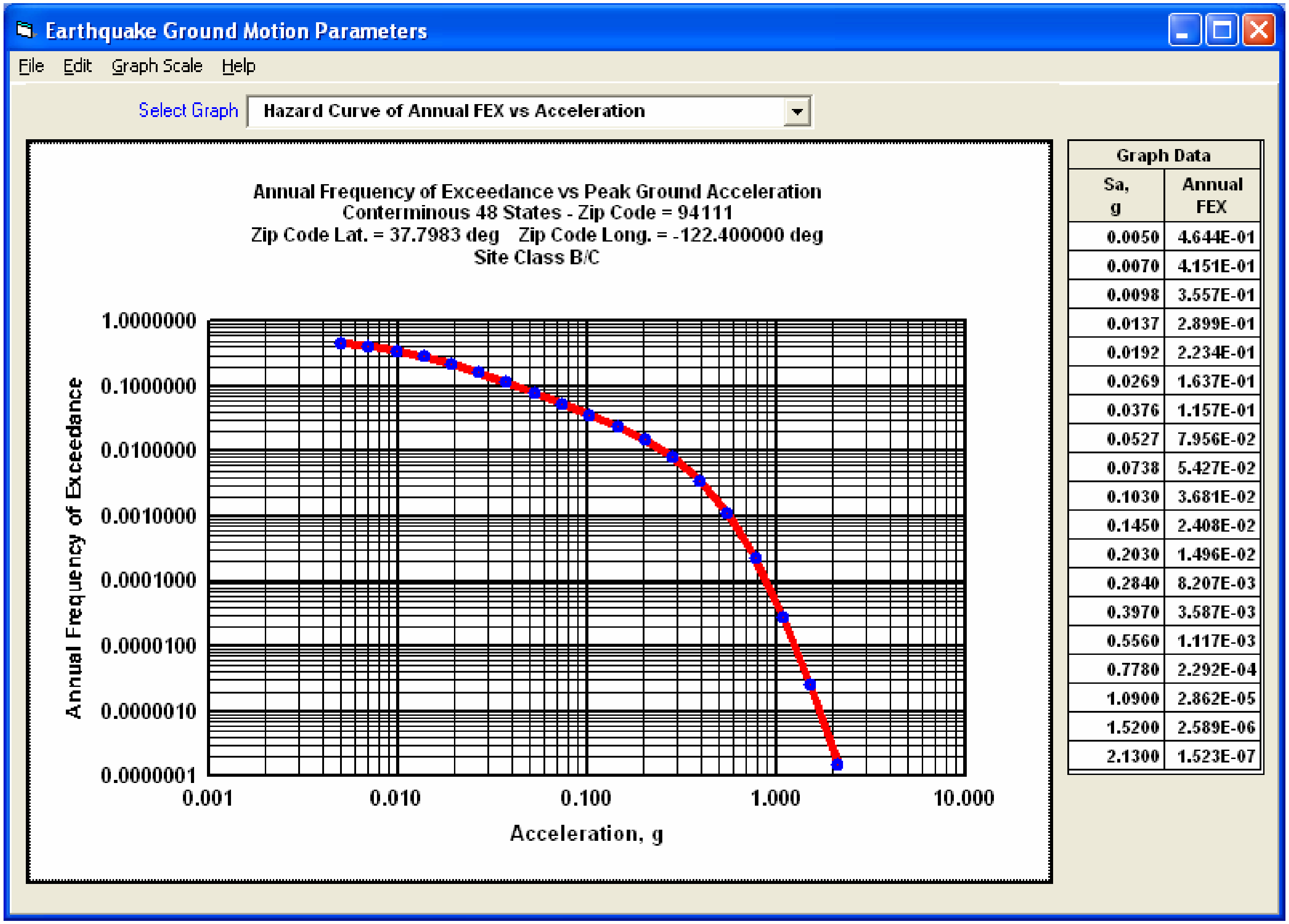
All Spectra: S_a - T



Calculate Hazard Curves



Annual Frequency of Exceedance



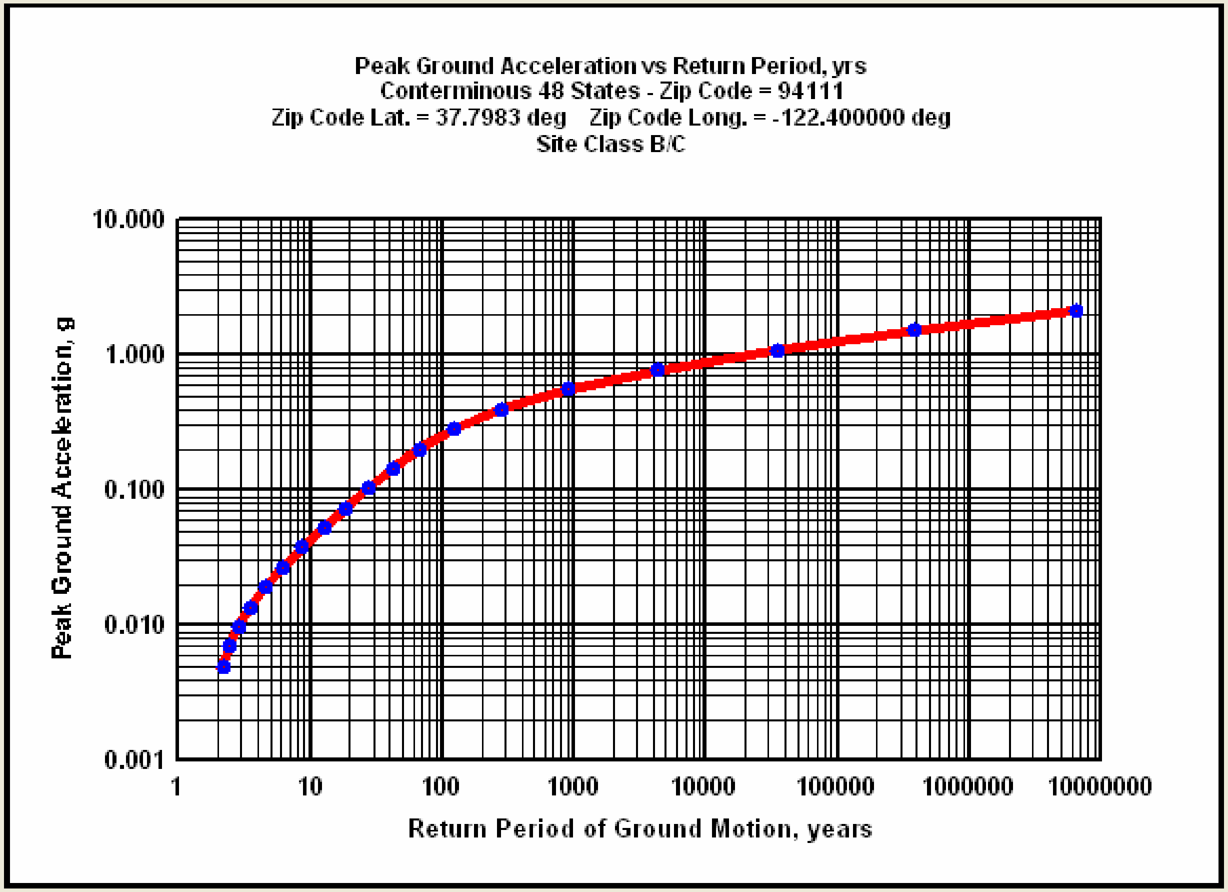
Graph Data	
Sa, g	Annual FEX
0.0050	4.644E-01
0.0070	4.151E-01
0.0098	3.557E-01
0.0137	2.899E-01
0.0192	2.234E-01
0.0269	1.637E-01
0.0376	1.157E-01
0.0527	7.956E-02
0.0738	5.427E-02
0.1030	3.681E-02
0.1450	2.408E-02
0.2030	1.496E-02
0.2840	8.207E-03
0.3970	3.587E-03
0.5560	1.117E-03
0.7780	2.292E-04
1.0900	2.862E-05
1.5200	2.589E-06
2.1300	1.523E-07

Return Period

Earthquake Ground Motion Parameters

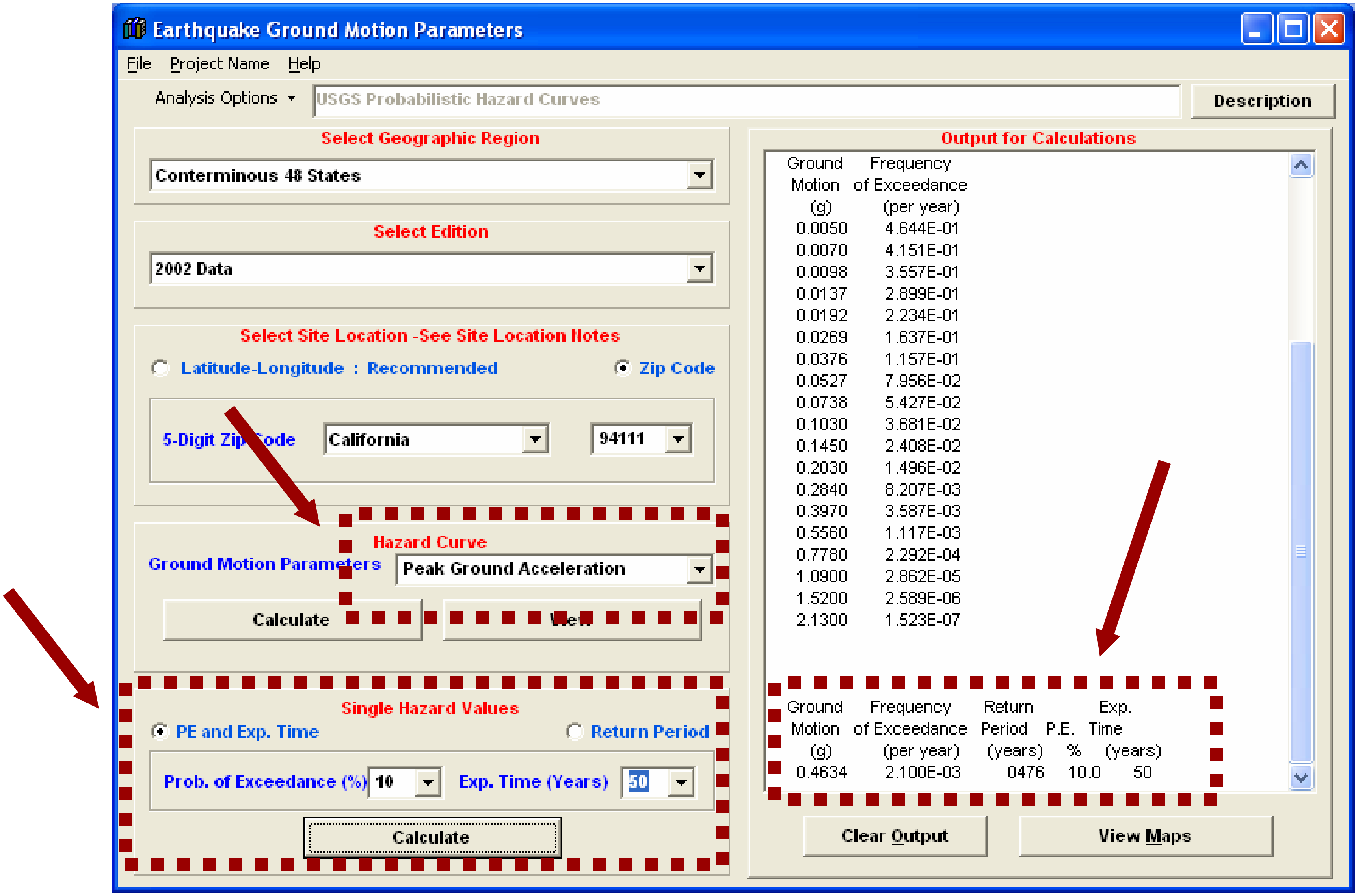
File Edit Graph Scale Help

Select Graph: Hazard Curve of Acceleration vs Return Period



Graph Data	
Return Period	Sa, g
2.153E+00	0.0050
2.409E+00	0.0070
2.812E+00	0.0098
3.449E+00	0.0137
4.477E+00	0.0192
6.107E+00	0.0269
8.643E+00	0.0376
1.257E+01	0.0527
1.843E+01	0.0738
2.717E+01	0.1030
4.154E+01	0.1450
6.686E+01	0.2030
1.218E+02	0.2840
2.788E+02	0.3970
8.952E+02	0.5560
4.363E+03	0.7780
3.494E+04	1.0900
3.863E+05	1.5200
6.564E+06	2.1300

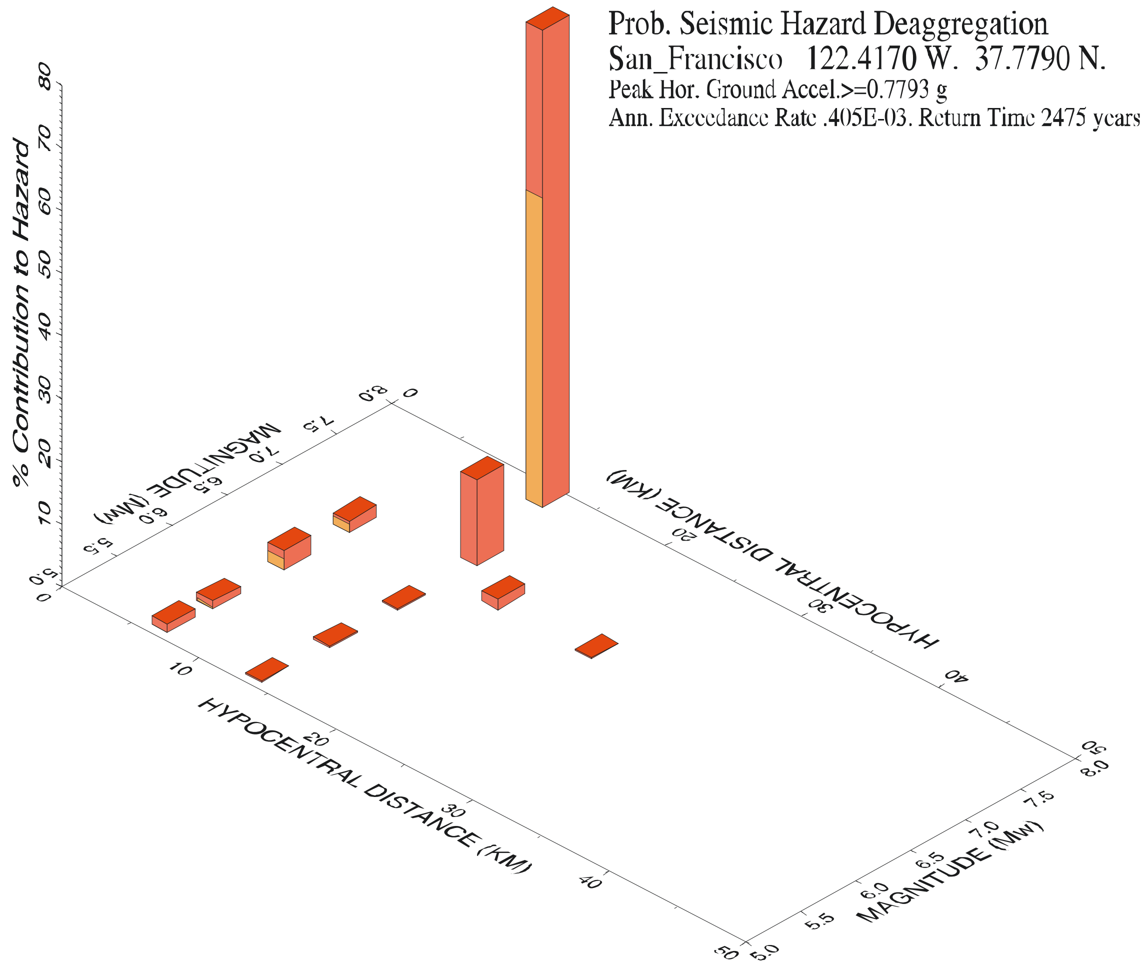
Single Values



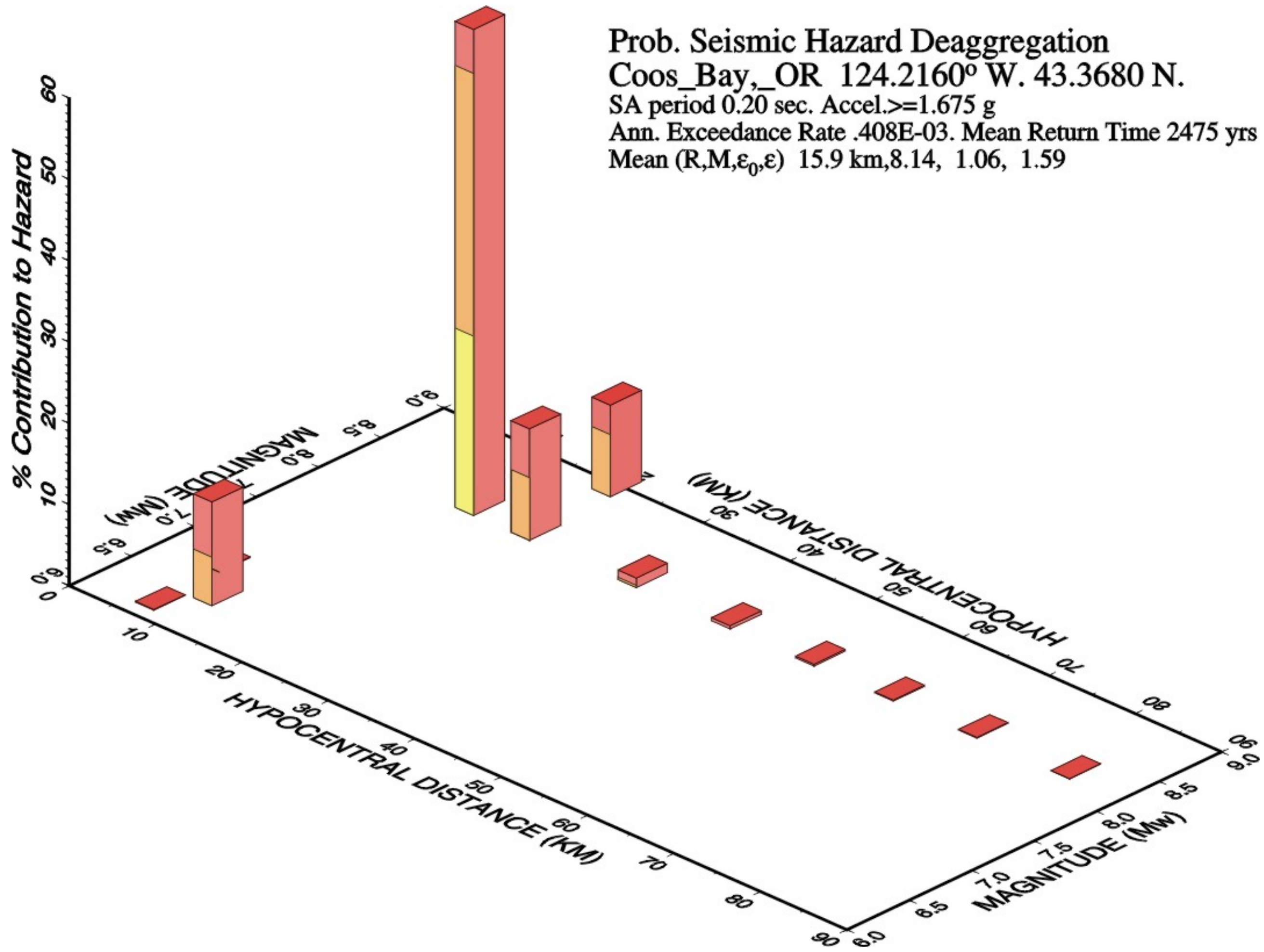
Deaggregation

- Breaking apart of the probabilistic hazard analysis
- Helps remove some of the “black box” effect
- Helps visualize the source of the hazard
- Many uses, e.g. liquefaction analysis, time history determination

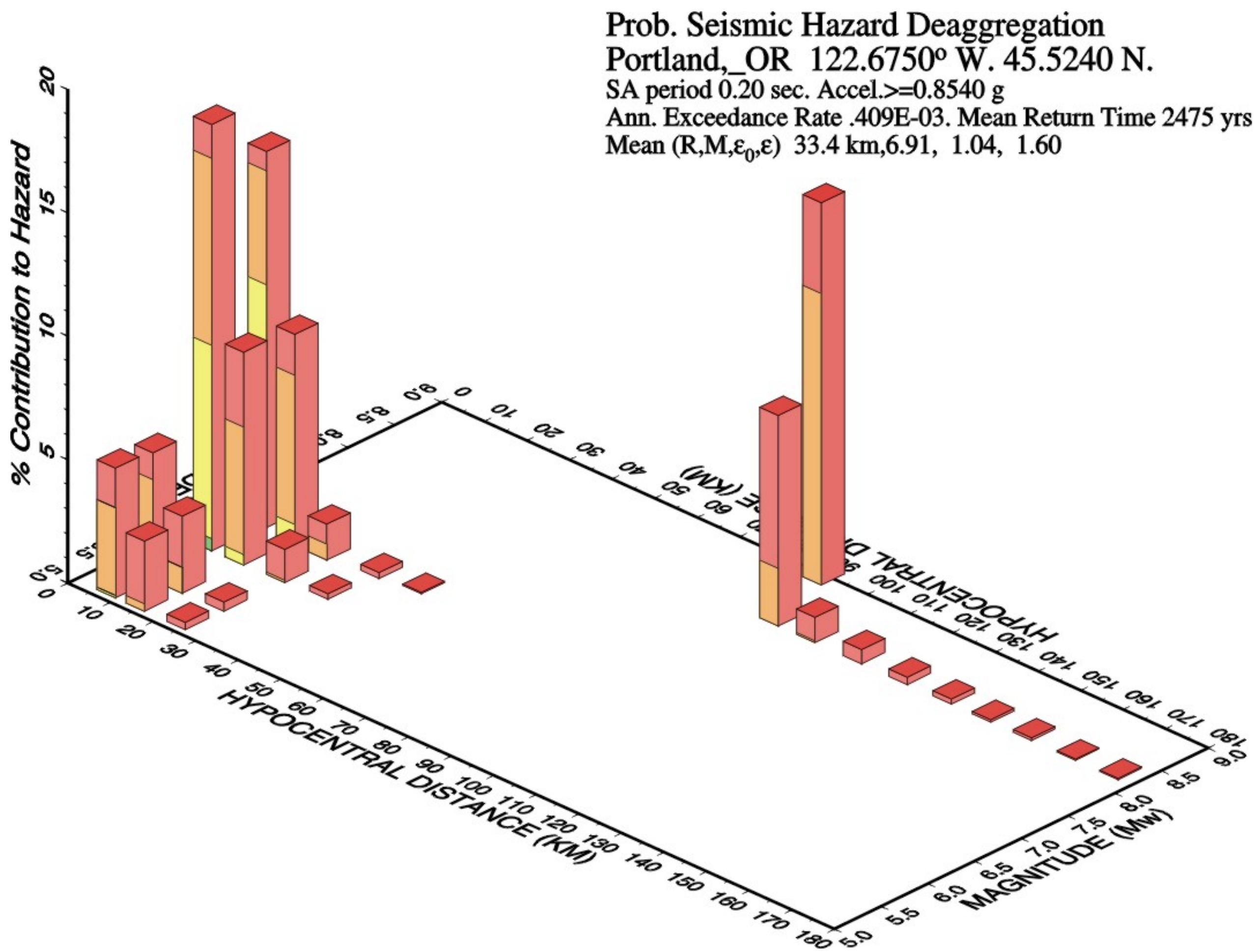
Deaggregation – San Francisco



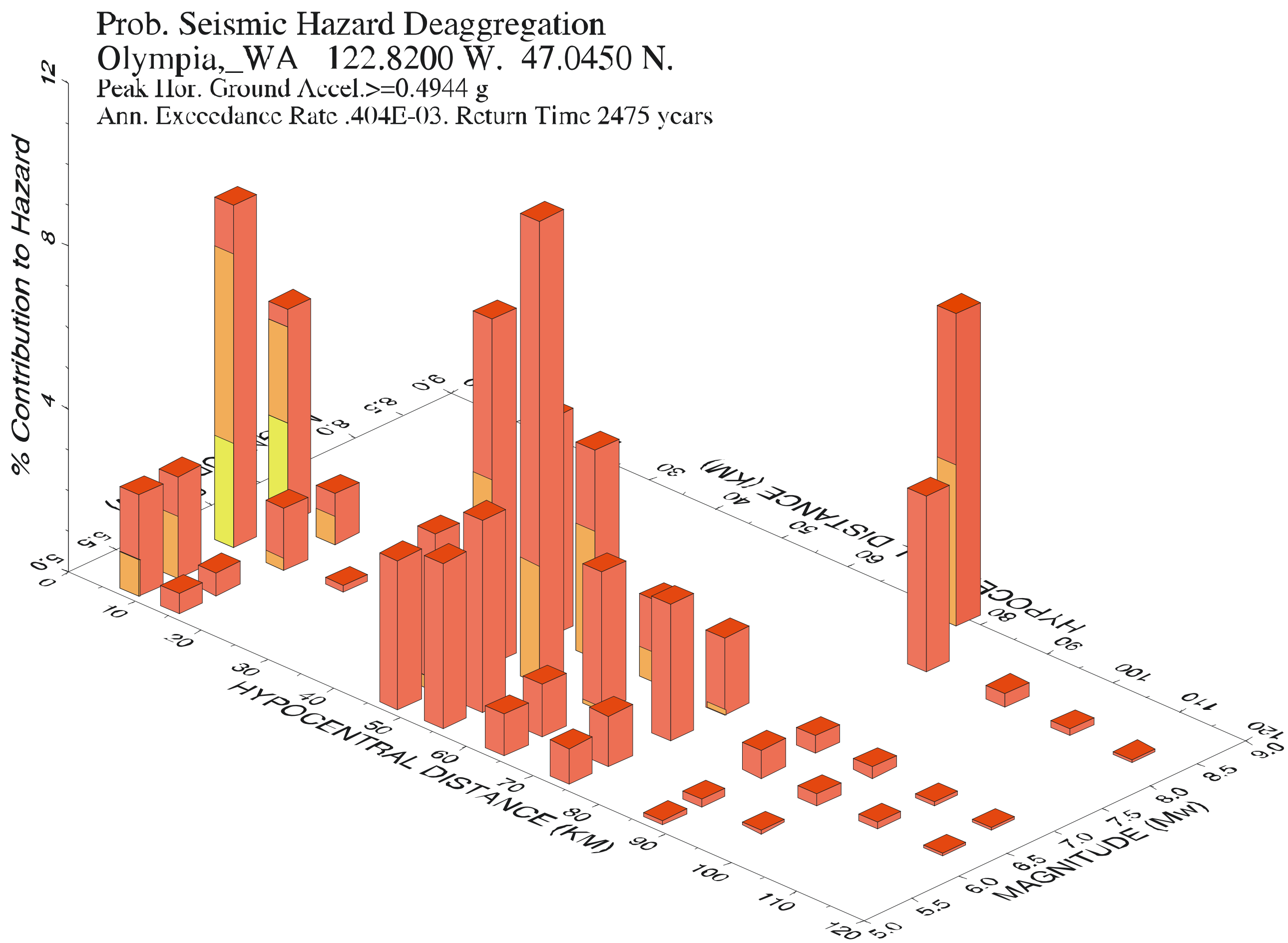
Deaggregation - Coos Bay, Oregon



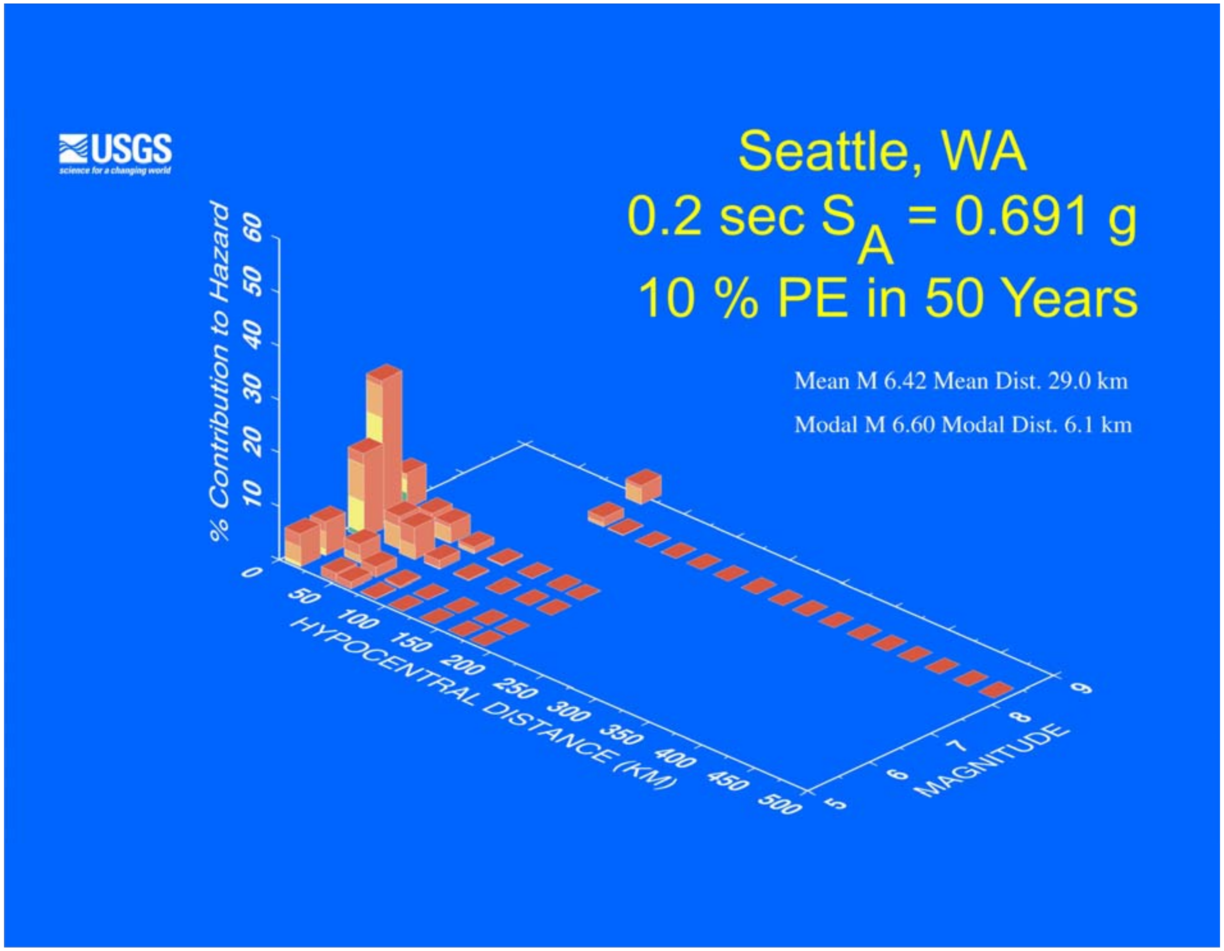
Deaggregation - Portland, Oregon



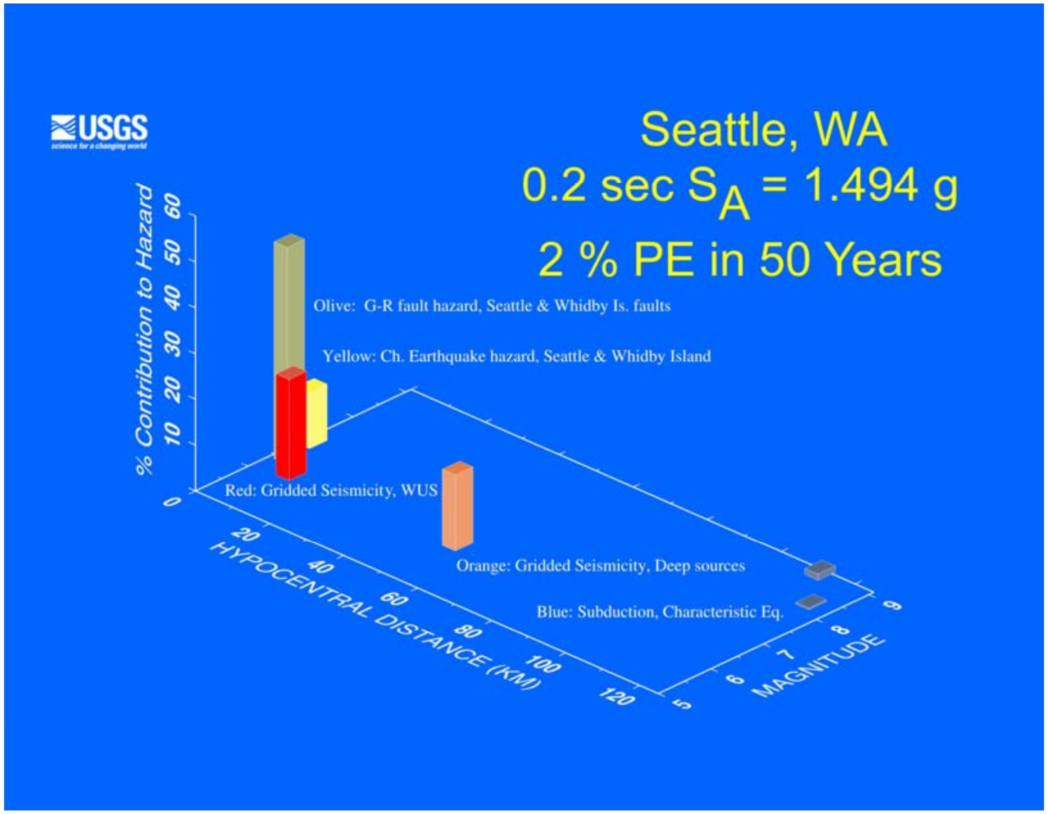
Deaggregation – Olympia, Washington



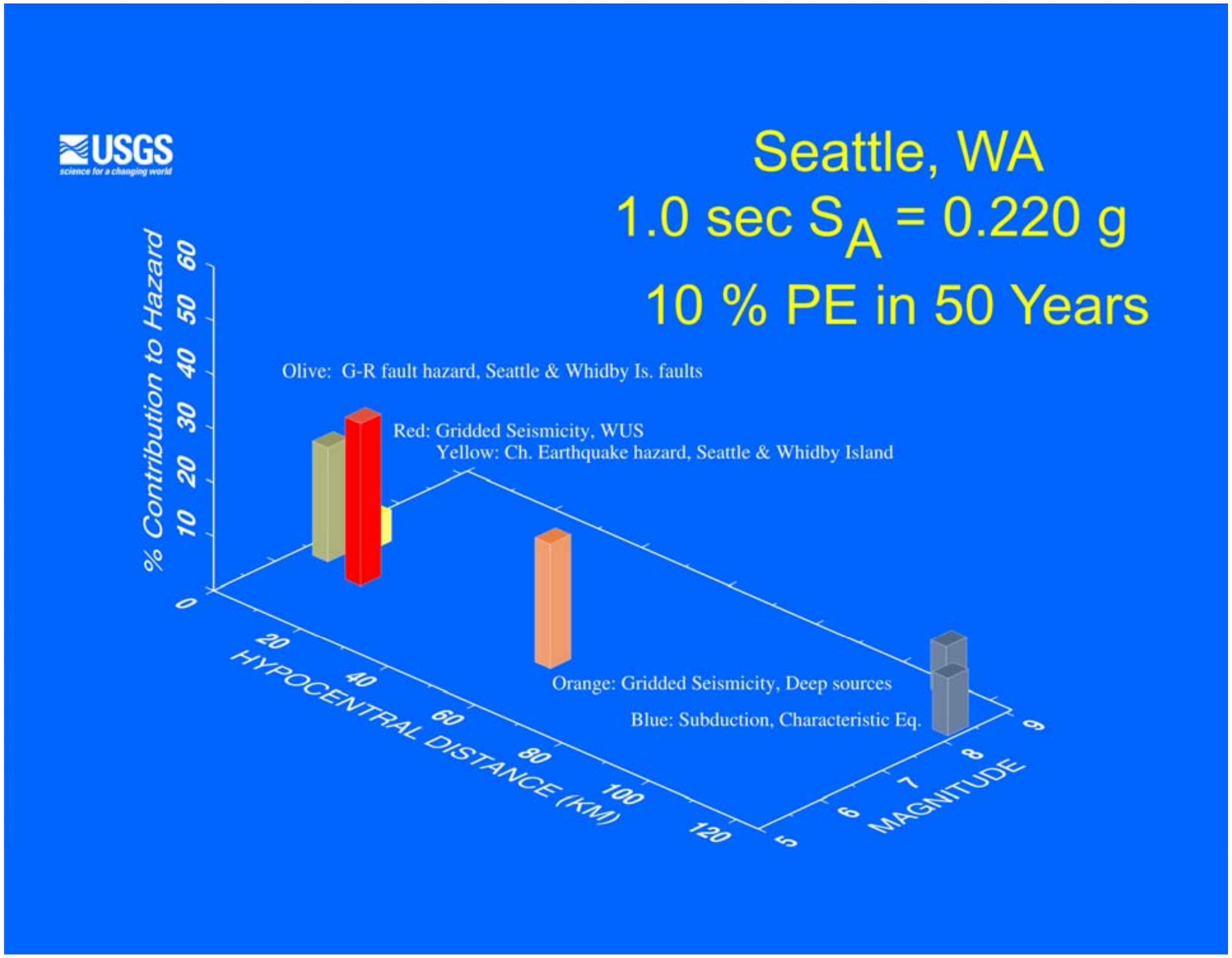
Seattle – 0.2 sec, Detailed



Seattle – 0.2 sec



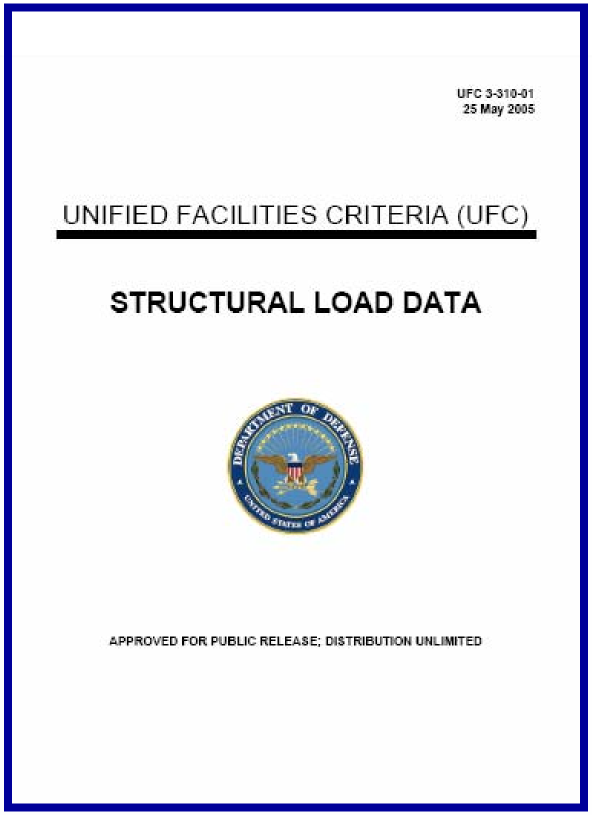
Seattle – 1.0 sec



Design Values Outside the United States

- Based on GASHAP Data
- 10% PE in 50 years
- PGA only
- Estimate 2% from 10% PE by multiplying by 2.0
- S_s = 2.5xPGA
- S_1 = PGA
- Use site-specific studies where available
- USGS studies where available

UFC 3-310-1



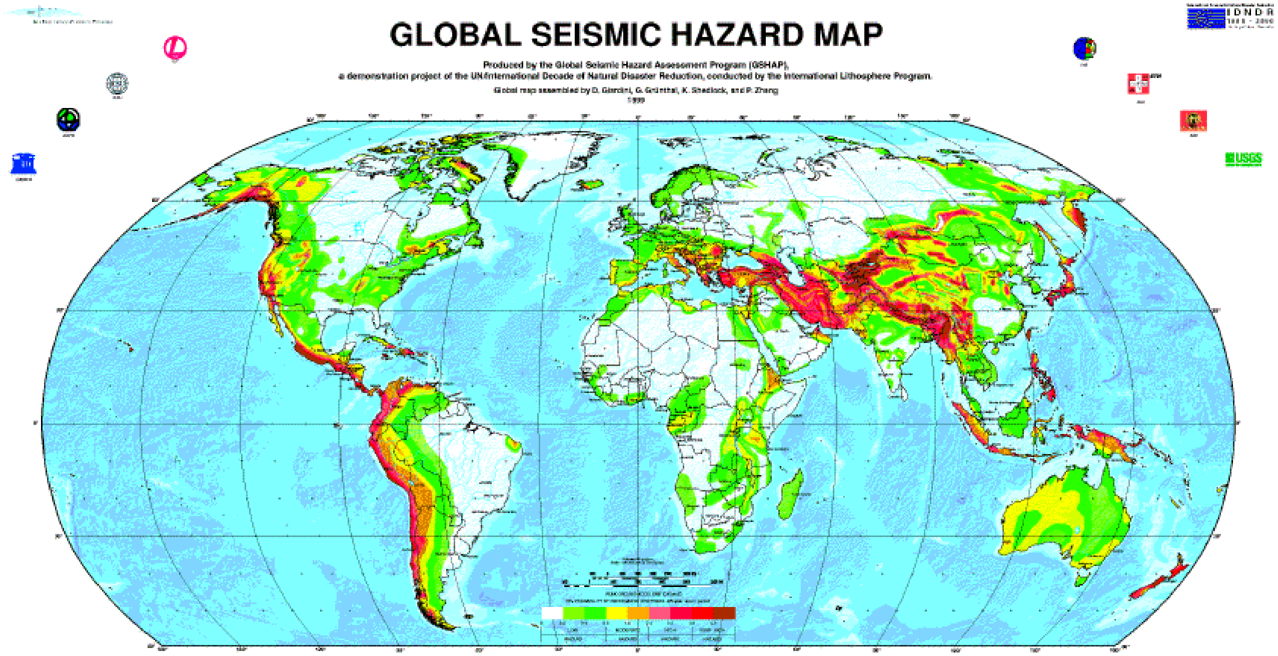
UFC 3-310-01
25 May 2005

UNIFIED FACILITIES CRITERIA (UFC)

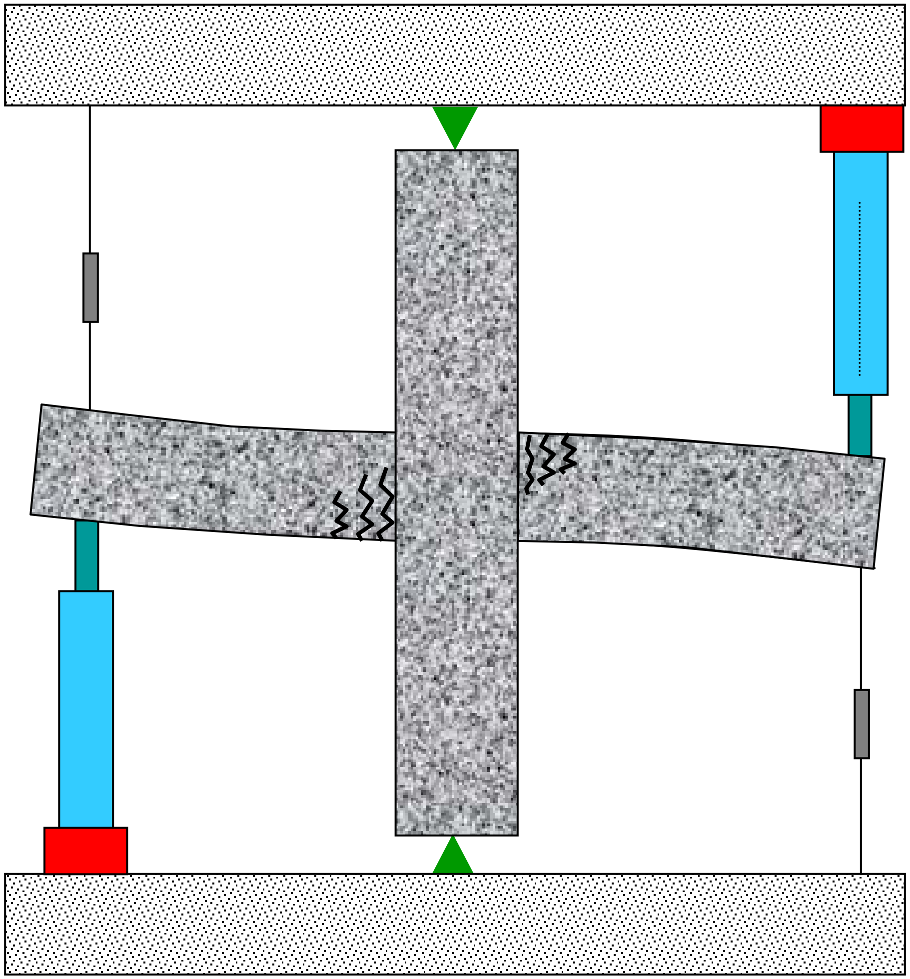
STRUCTURAL LOAD DATA

APPROVED FOR PUBLIC RELEASE; DISTRIBUTION UNLIMITED

What is GSHAP?



INELASTIC BEHAVIOR OF MATERIALS AND STRUCTURES



Inelastic Behavior of Materials and Structures

- Illustrates inelastic behavior of materials and structures
- Explains why inelastic response may be necessary
- Explains the "equal displacement " concept
- Introduces the concept of inelastic design response spectra
- Explains how inelastic behavior is built into the *NEHRP Recommended Provisions* and ASCE 7-05

Importance in Relation to ASCE 7-05

- Derivation and explanation of the *response reduction factor, R*
- Derivation and explanation of the *displacement amplification factor,* C_d
- Derivation and explanation of the *overstrength factor,* Ω_o

Inelastic Behavior of Structures

From material

$\downarrow$

to cross section

$\downarrow$

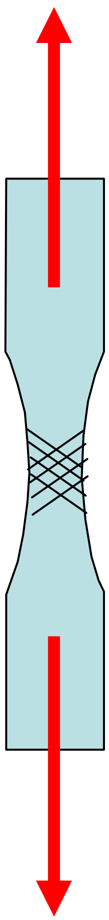
to critical region

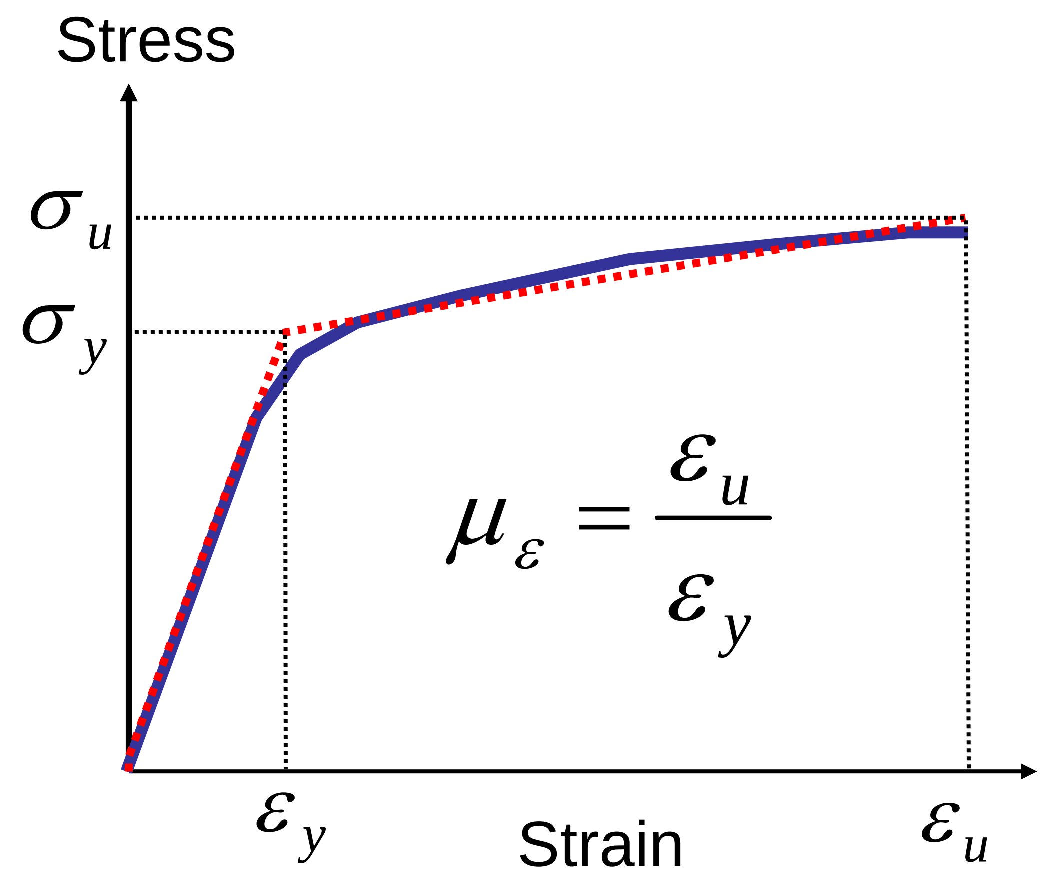
$\downarrow$

to structure

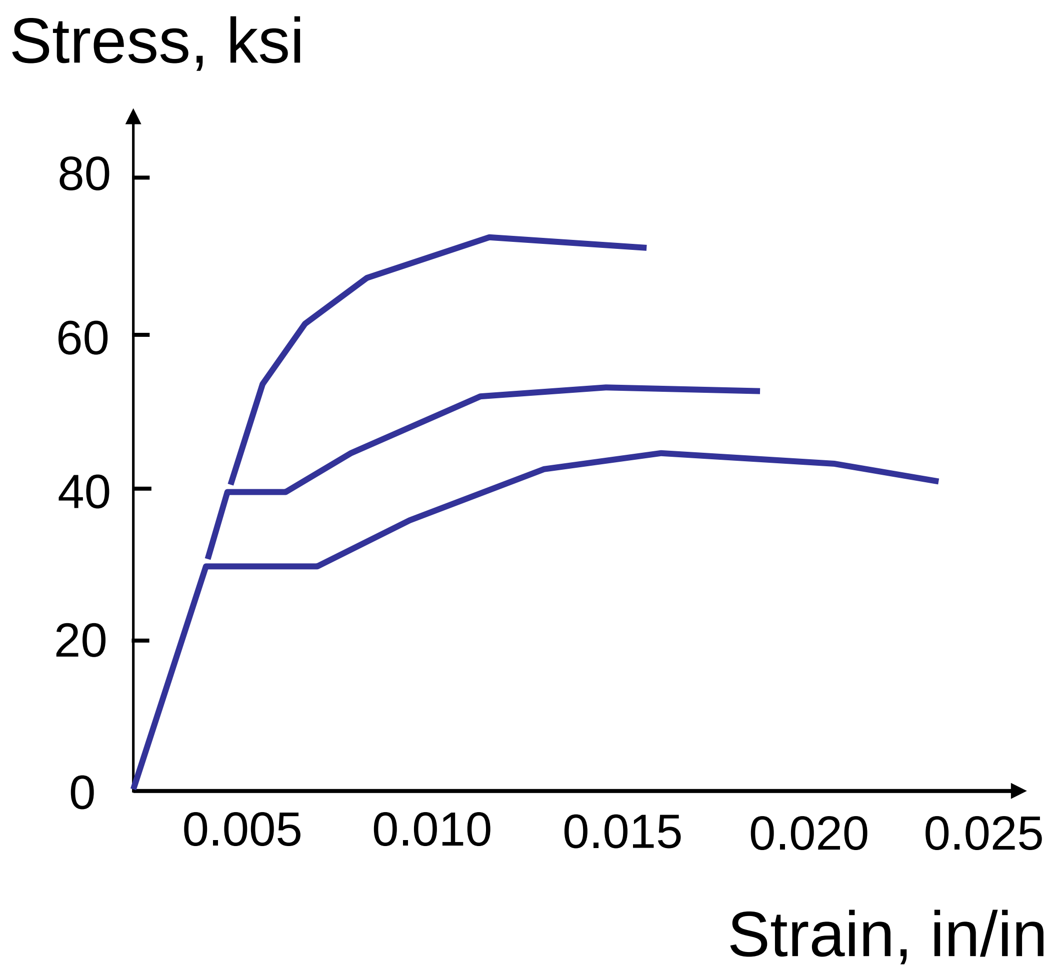
Idealized Inelastic Behavior

From Material…..

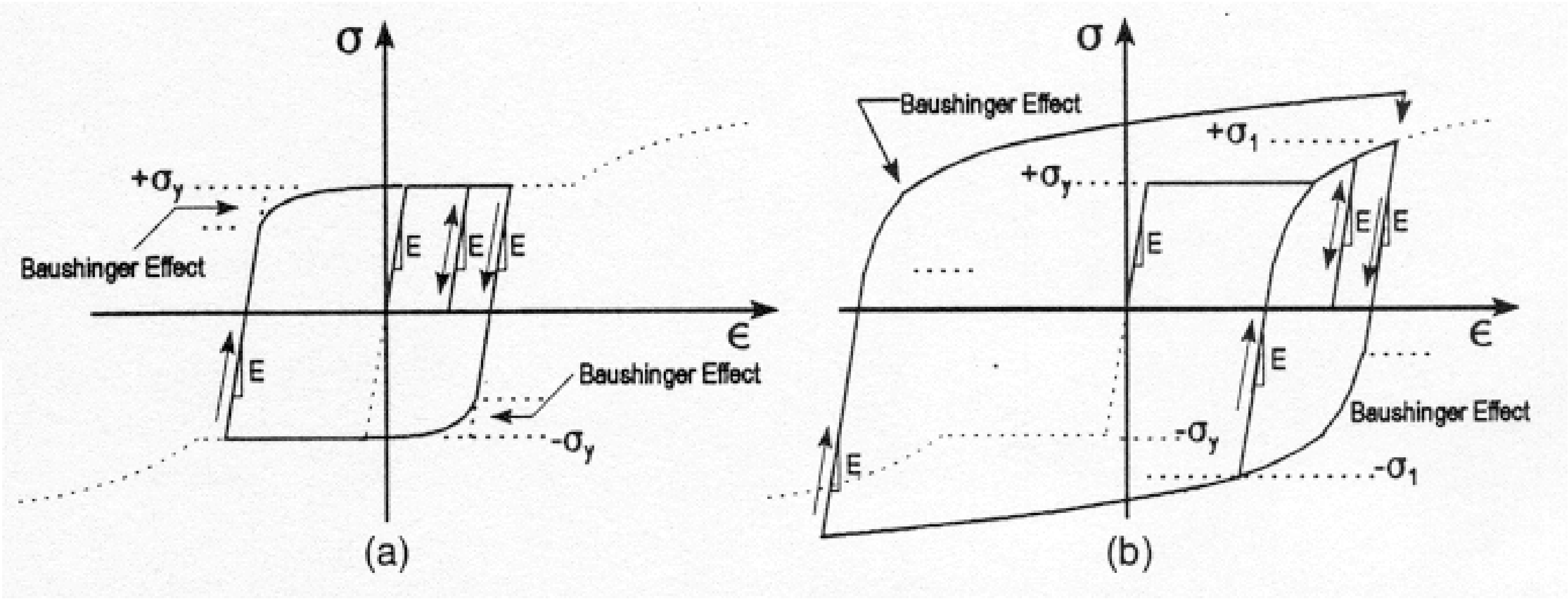




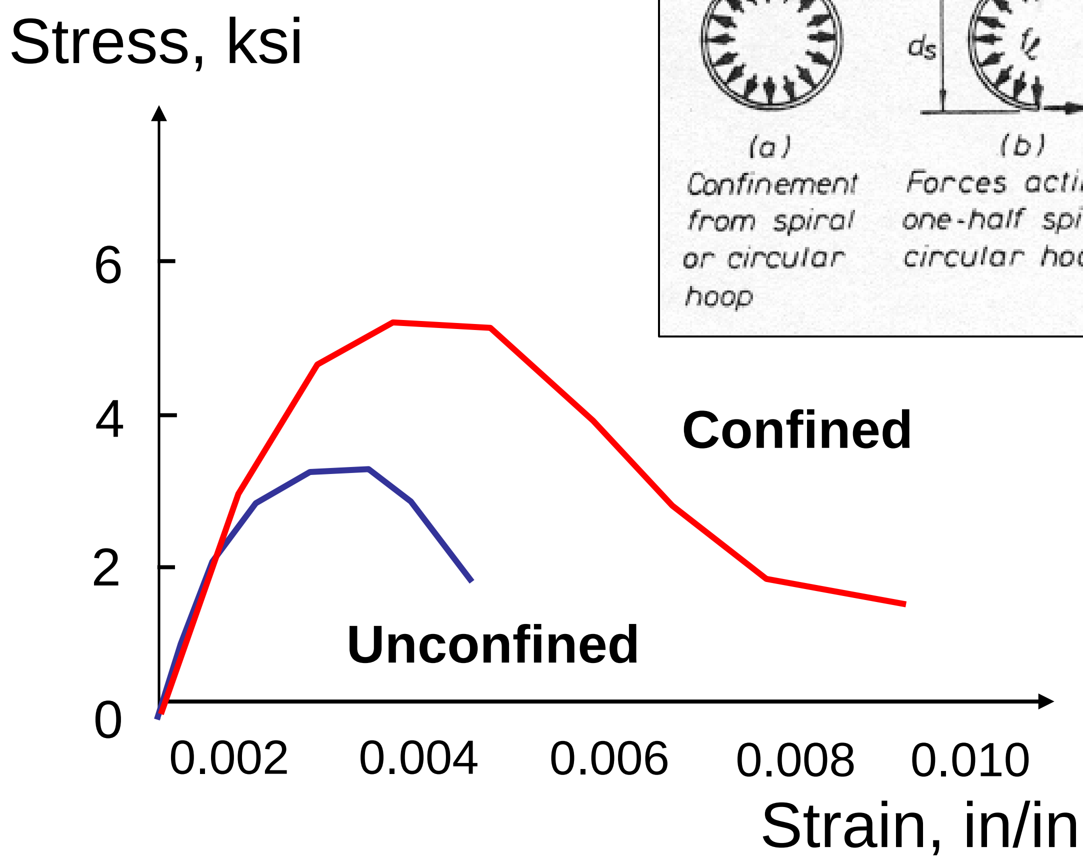
Stress-Strain Relationships for Steel

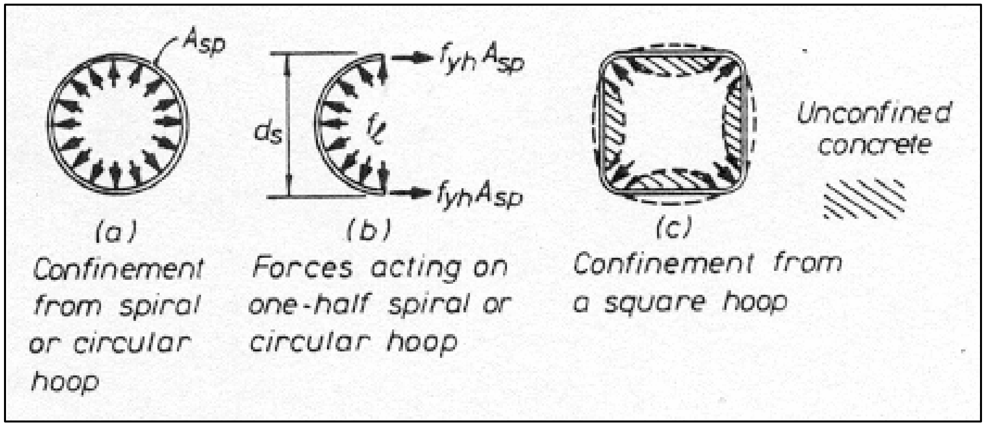


Stress-Strain Relationships for Steel

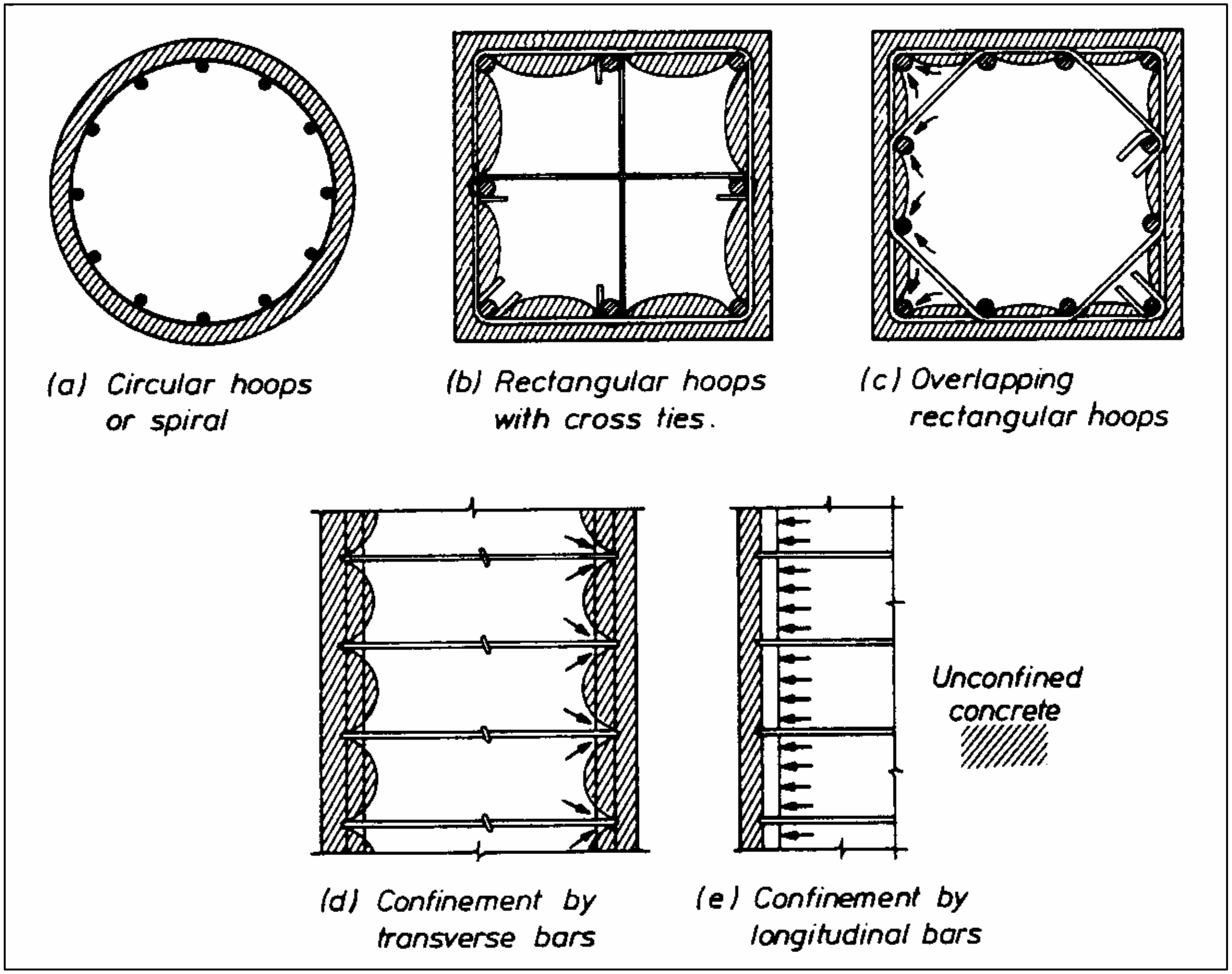


Stress-Strain Relationships for Concrete
(Unconfined and Confined)

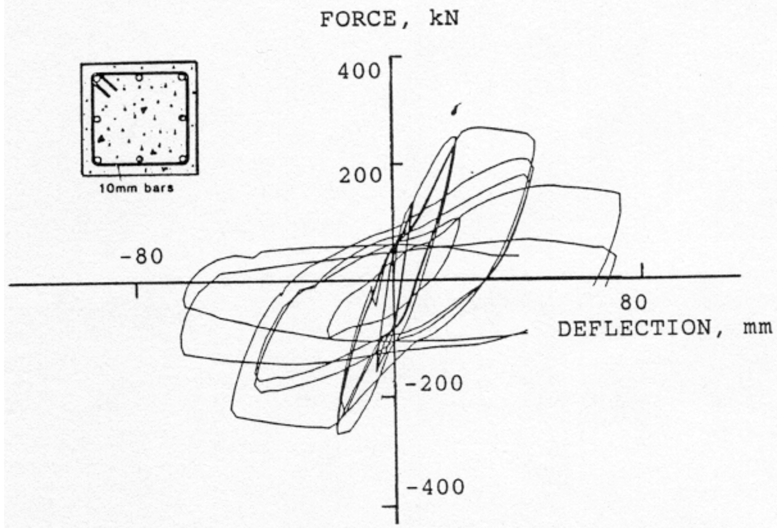




Concrete Confinement

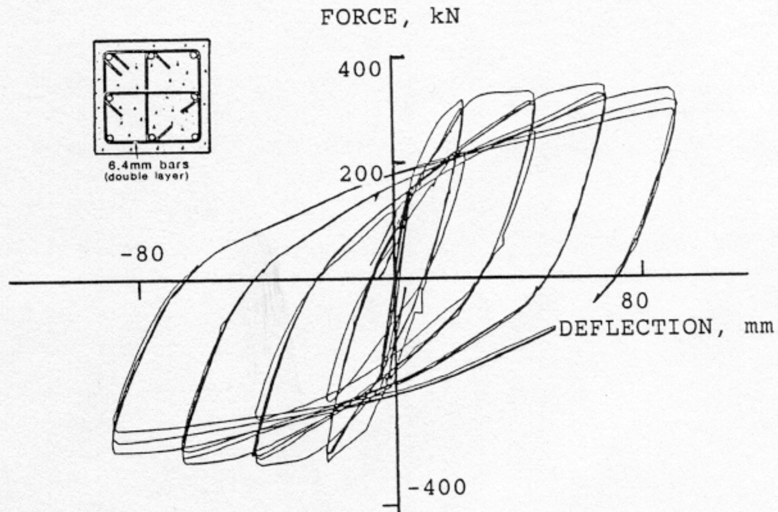


Unconfined



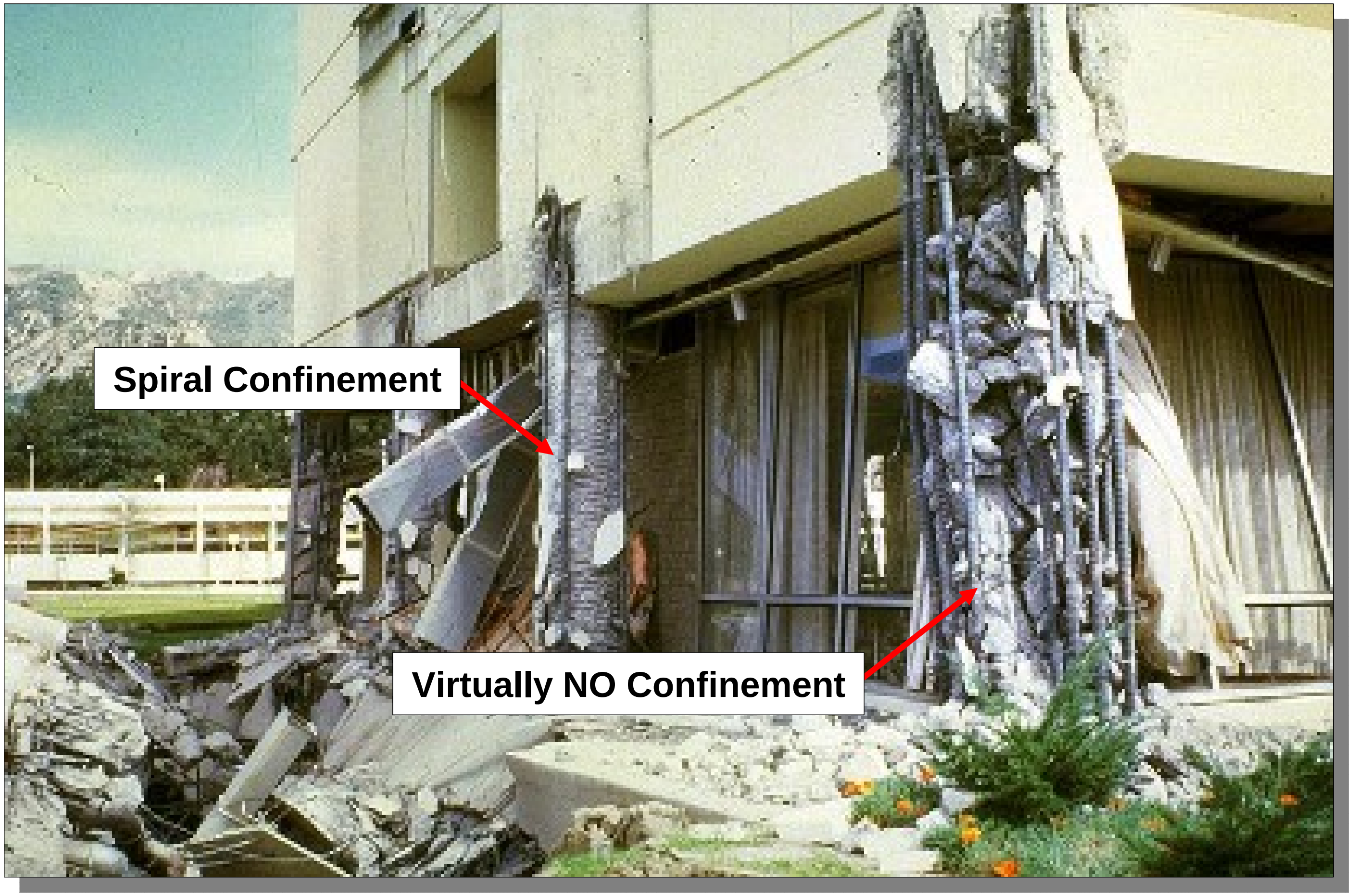
a) Poorly confined column

Confined



b) Well confined column

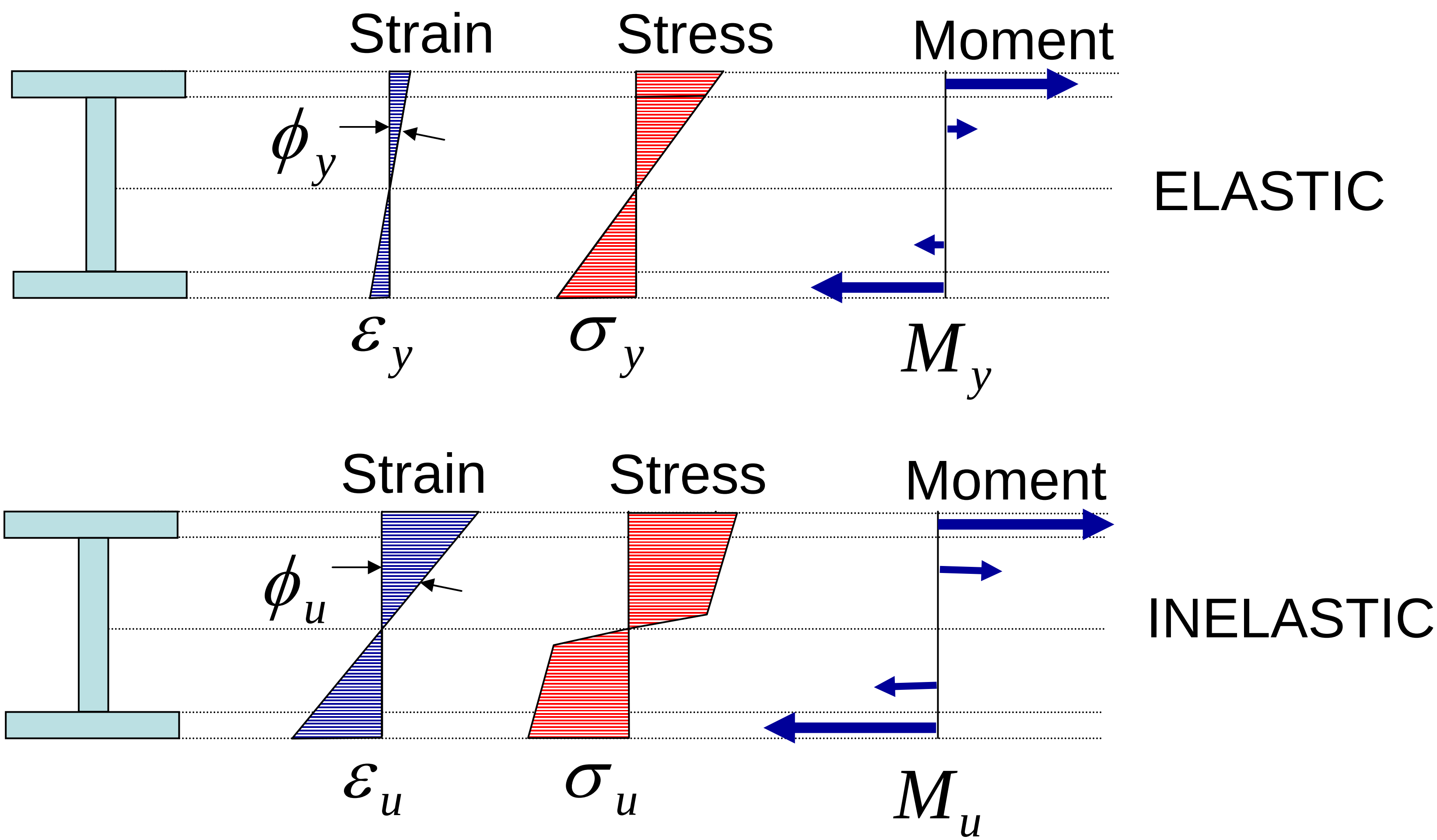
Benefits of Confinement



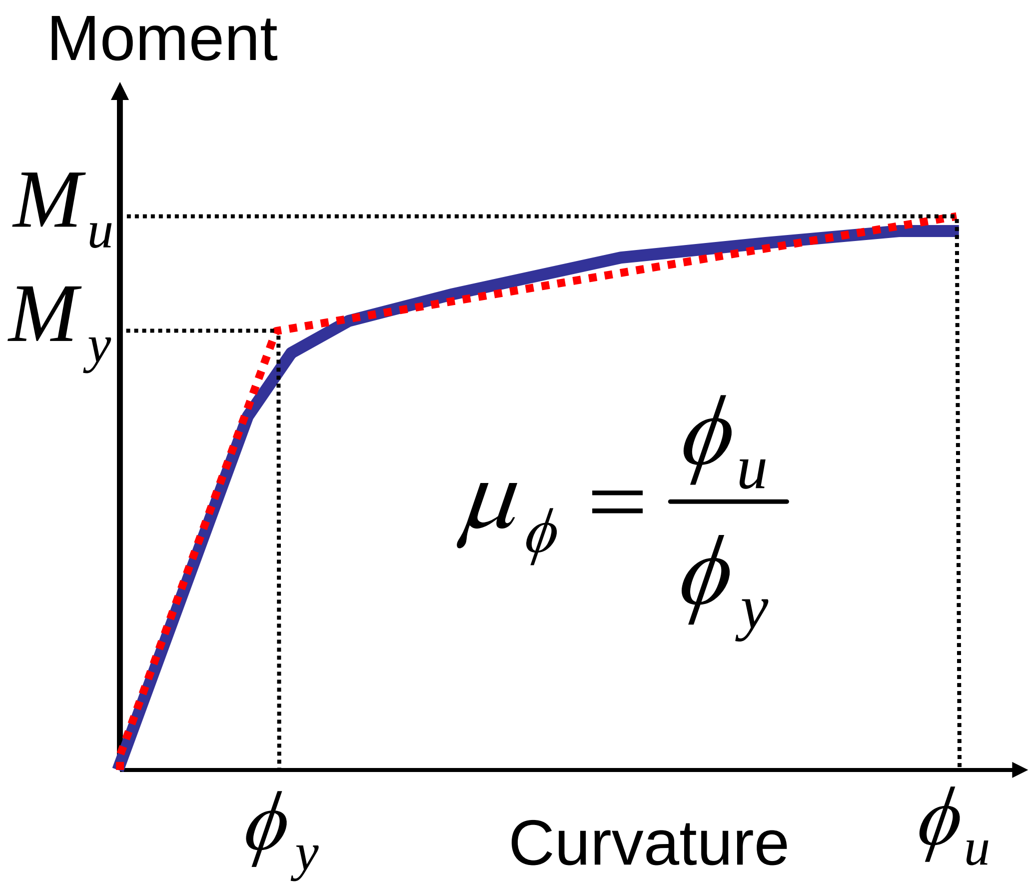
Olive View Hospital, 1971 San Fernando Valley earthquake

Idealized Inelastic Behavior

To Section.....

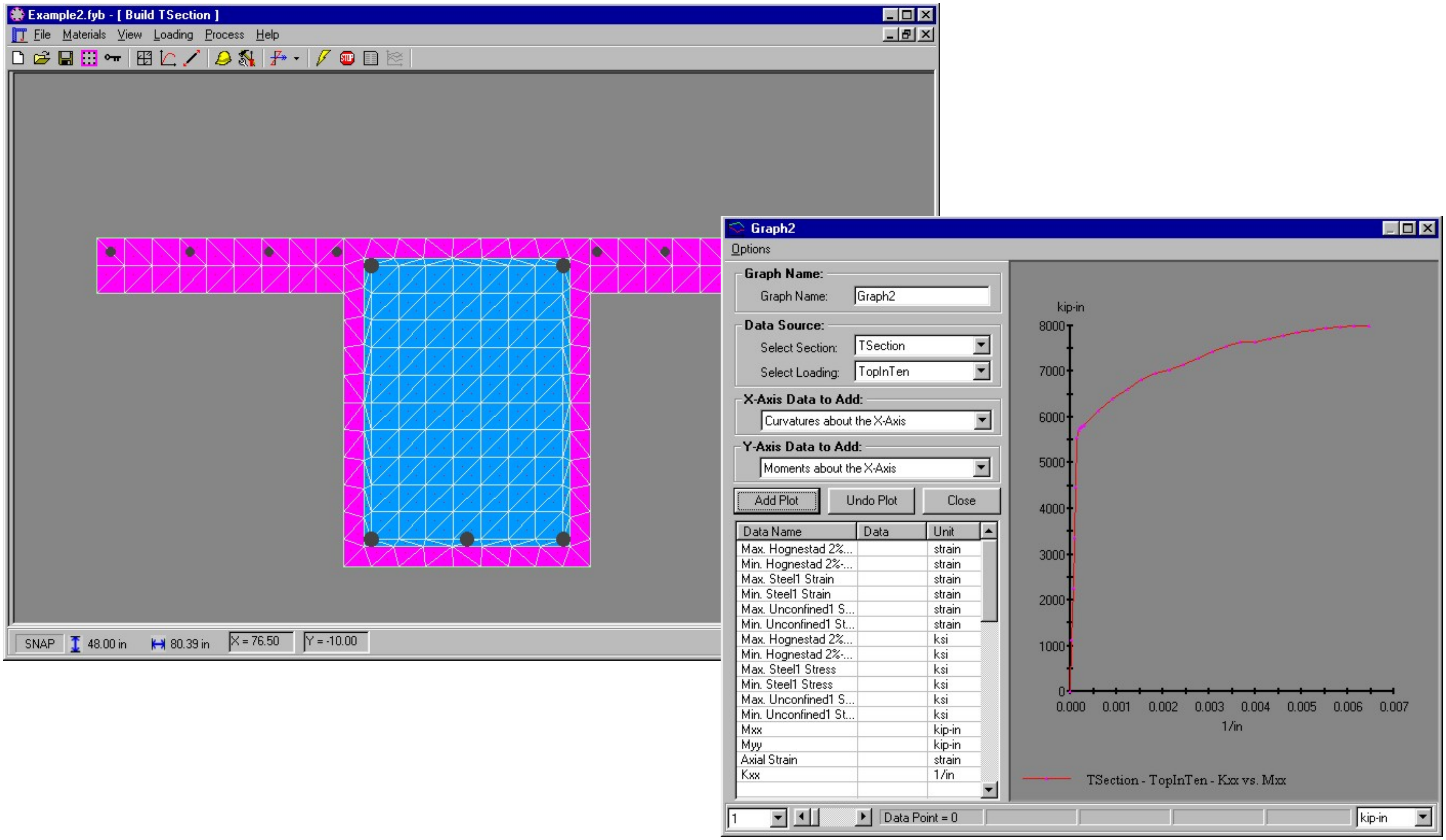


Idealized Inelastic Behavior
To Section…..



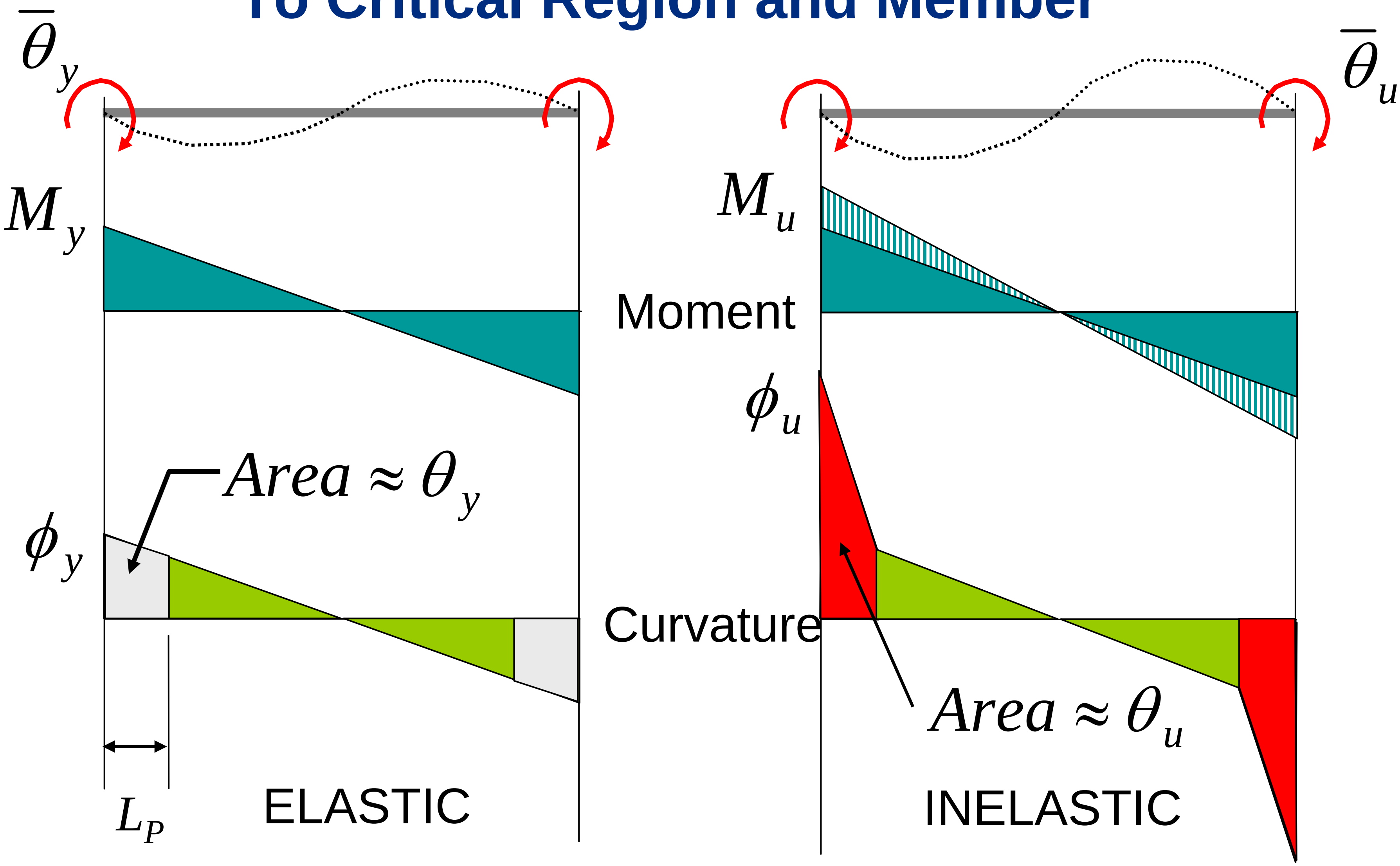
NOTE: $\mu_\phi \le \mu_\varepsilon$

Software for Moment - Curvature Analysis "XTRACT"



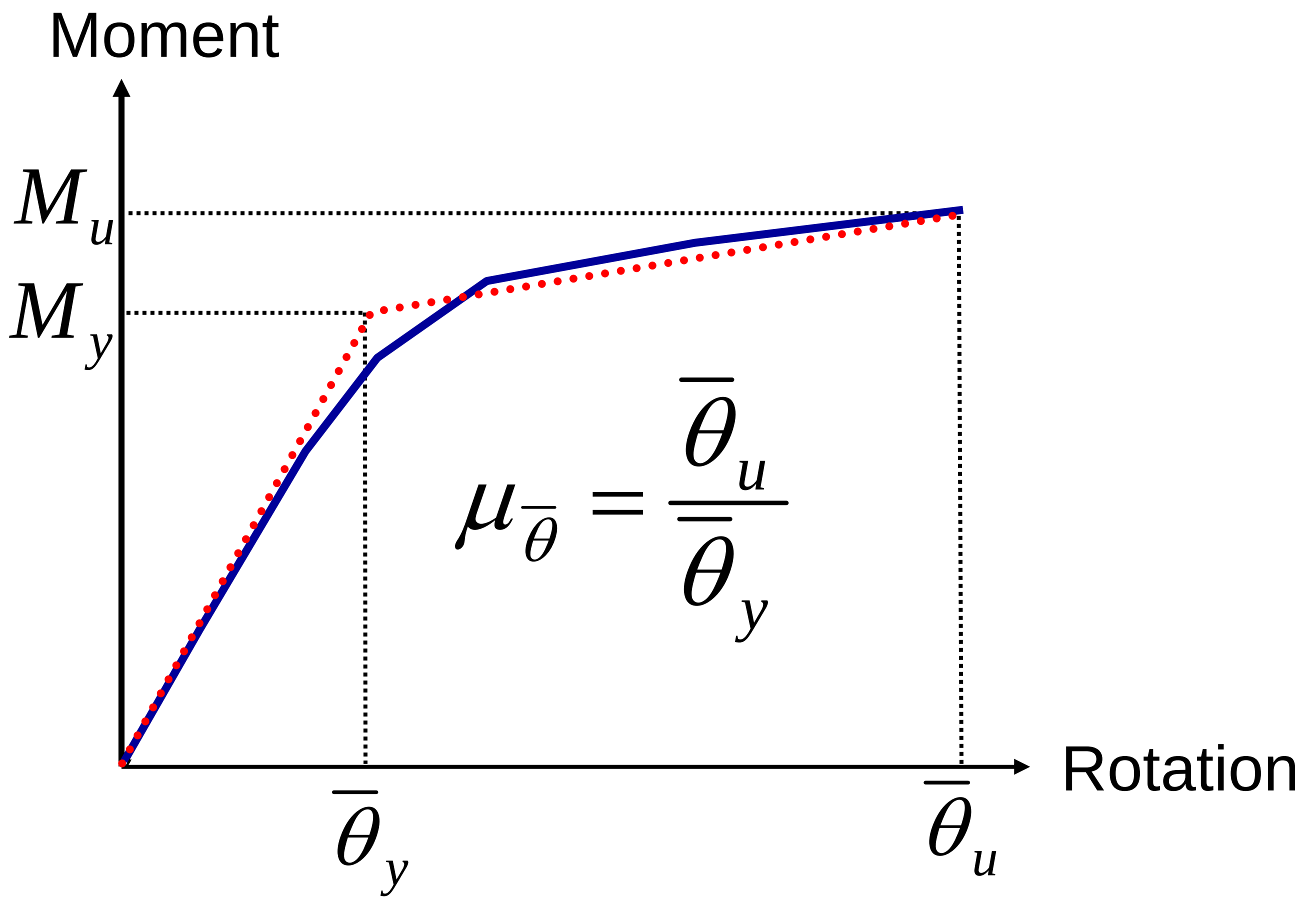
Idealized Inelastic Behavior

To Critical Region and Member



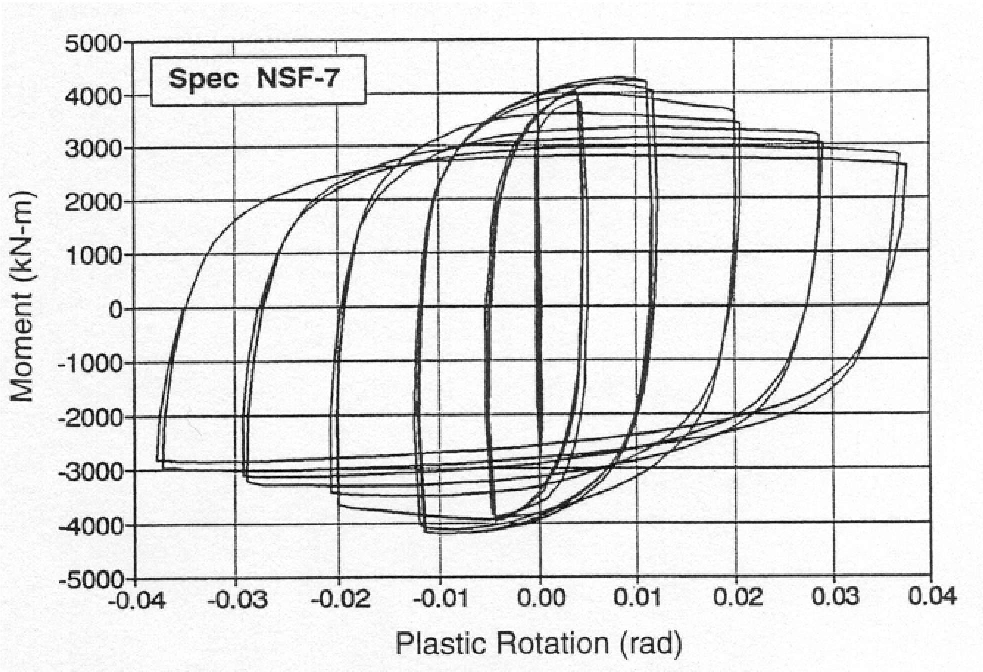
Idealized Inelastic Behavior

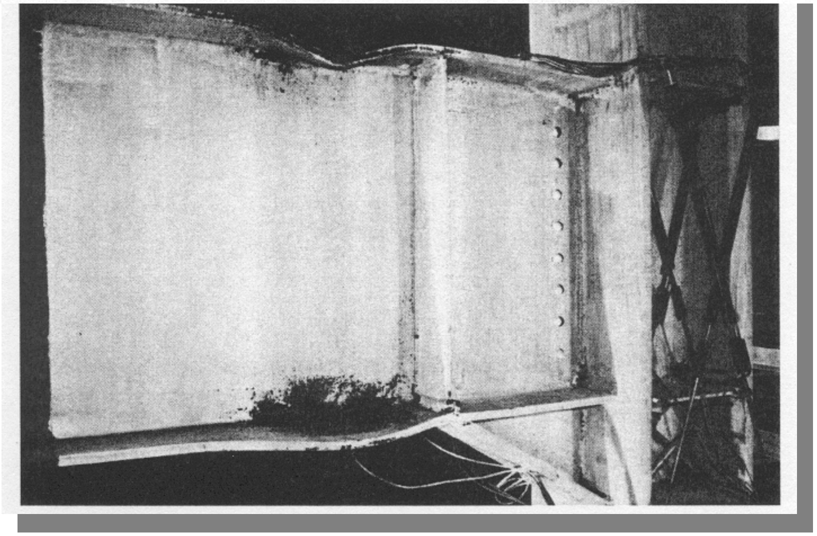
To Critical Region and Member



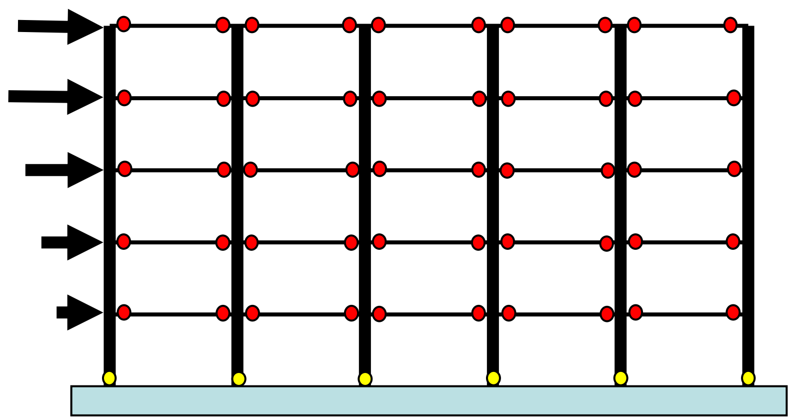
NOTE: $\mu_{\bar{\theta}} \leq \mu_{\theta} \leq \mu_{\phi}$

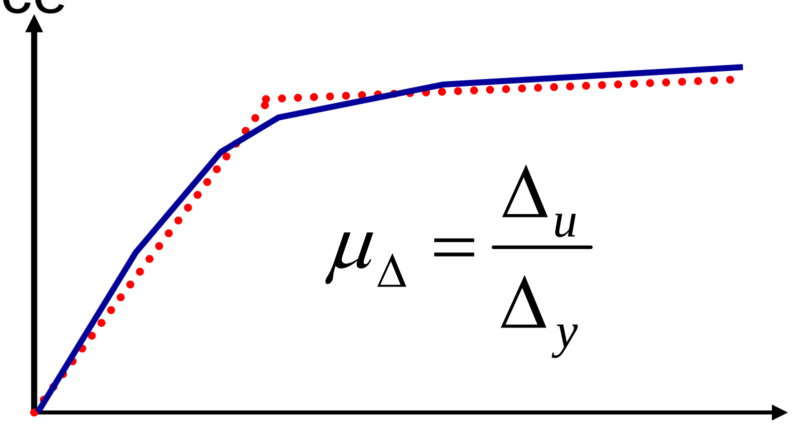
Critical Region Behavior of a Steel Girder





Idealized Inelastic Behavior
To Structure…..





$$\mu_\Delta = \frac{\Delta_u}{\Delta_y}$$

Note: $\mu_\Delta \le \mu_{\bar{\theta}}$

Loss of Ductility Through Hierarchy

Strain $\mu_\varepsilon = 100$

Curvature $\mu_\phi = 12$ to 20

Rotation $\mu_\theta = 8$ to 14

Displacement $\mu_\Delta = 4$ to 10

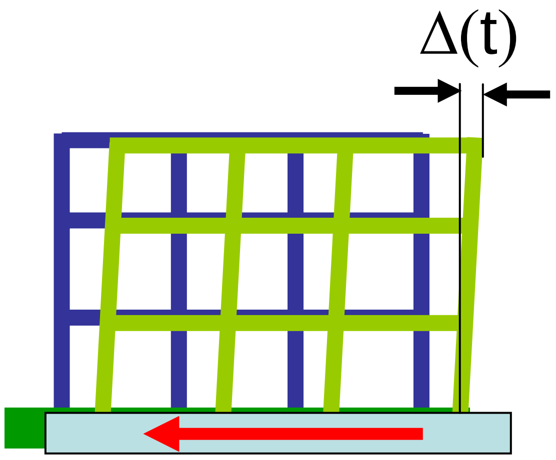
Ductility and Energy Dissipation Capacity

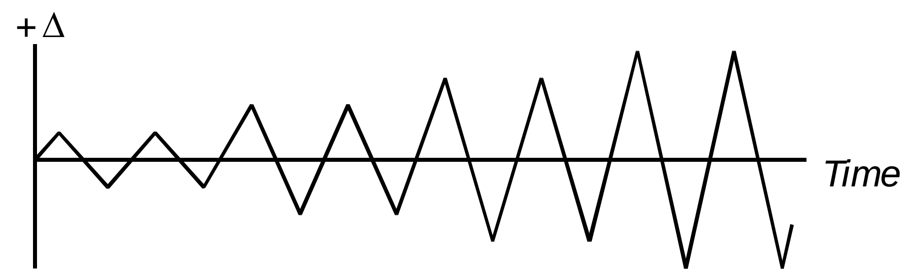
- System ductility of 4 to 6 is required for acceptable seismic behavior.

- Good hysteretic behavior requires ductile materials. However, ductility in itself is insufficient to provide acceptable seismic behavior.

- Cyclic energy dissipation capacity is a better indicator of performance.

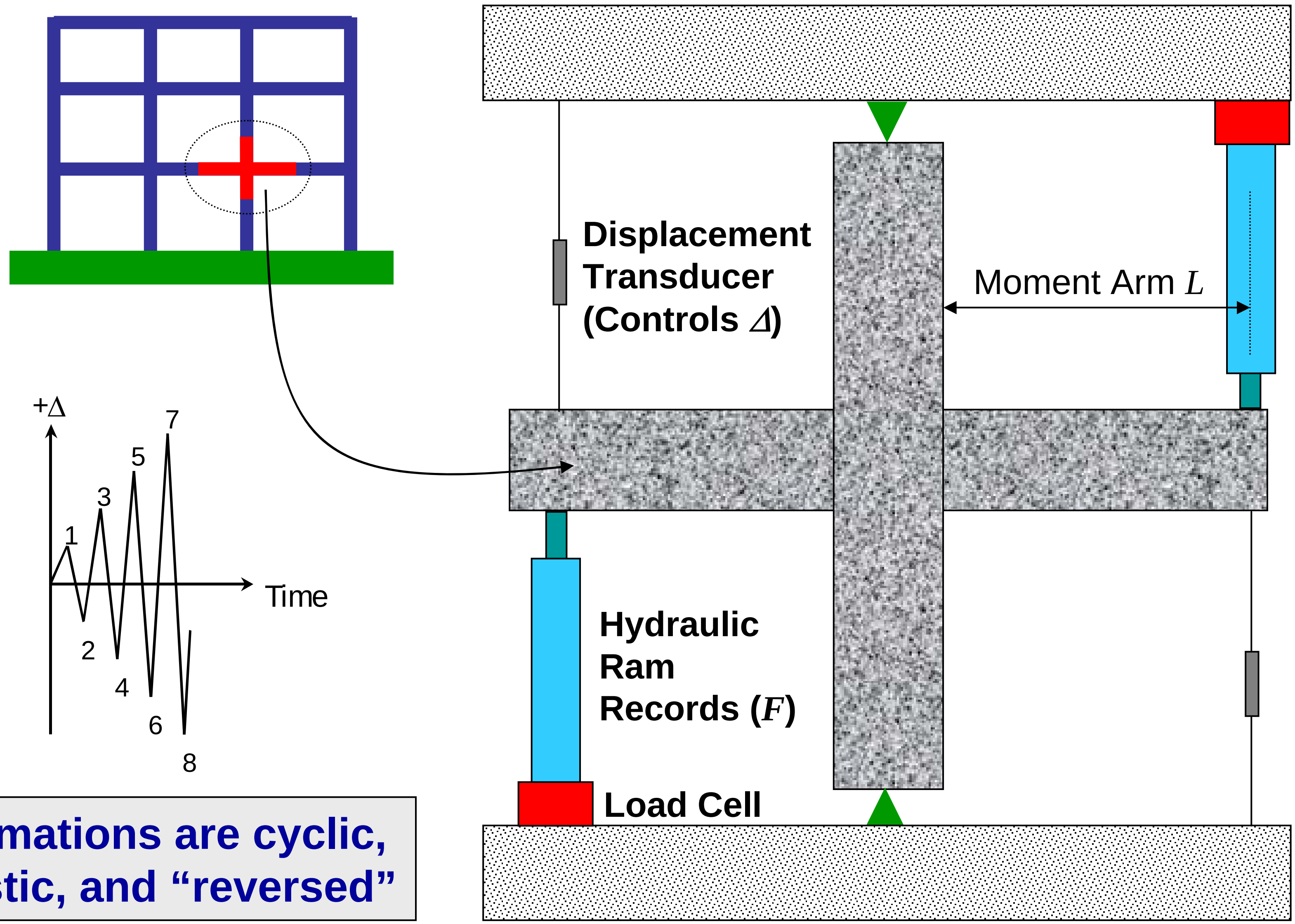
Response Under Reversed Cyclic “Loading”





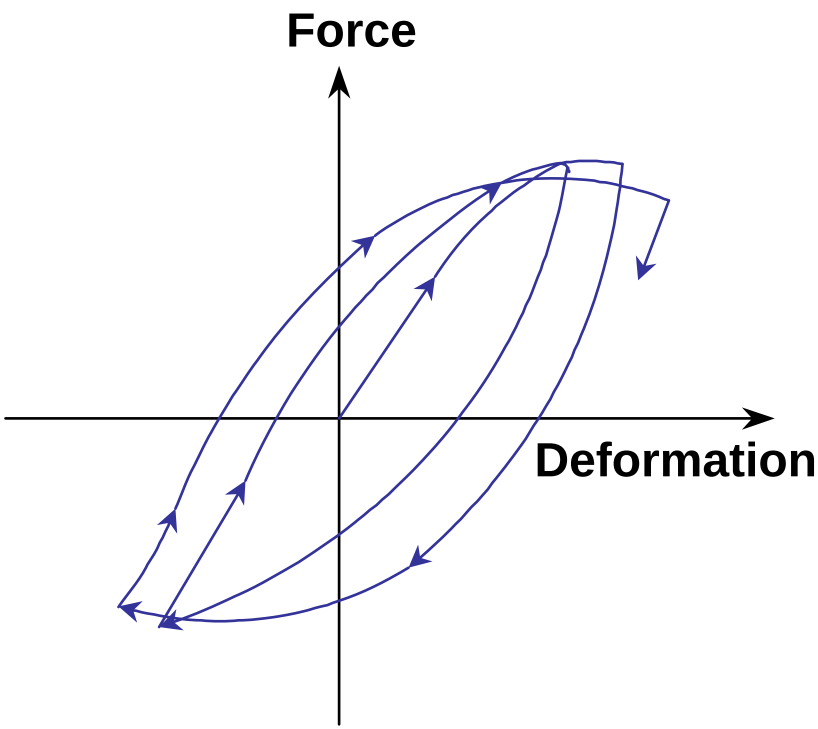
Earthquakes impose *DEFORMATIONS*. Internal forces develop as a result of the deformations.

Laboratory Specimen under Cyclic Deformation Loading

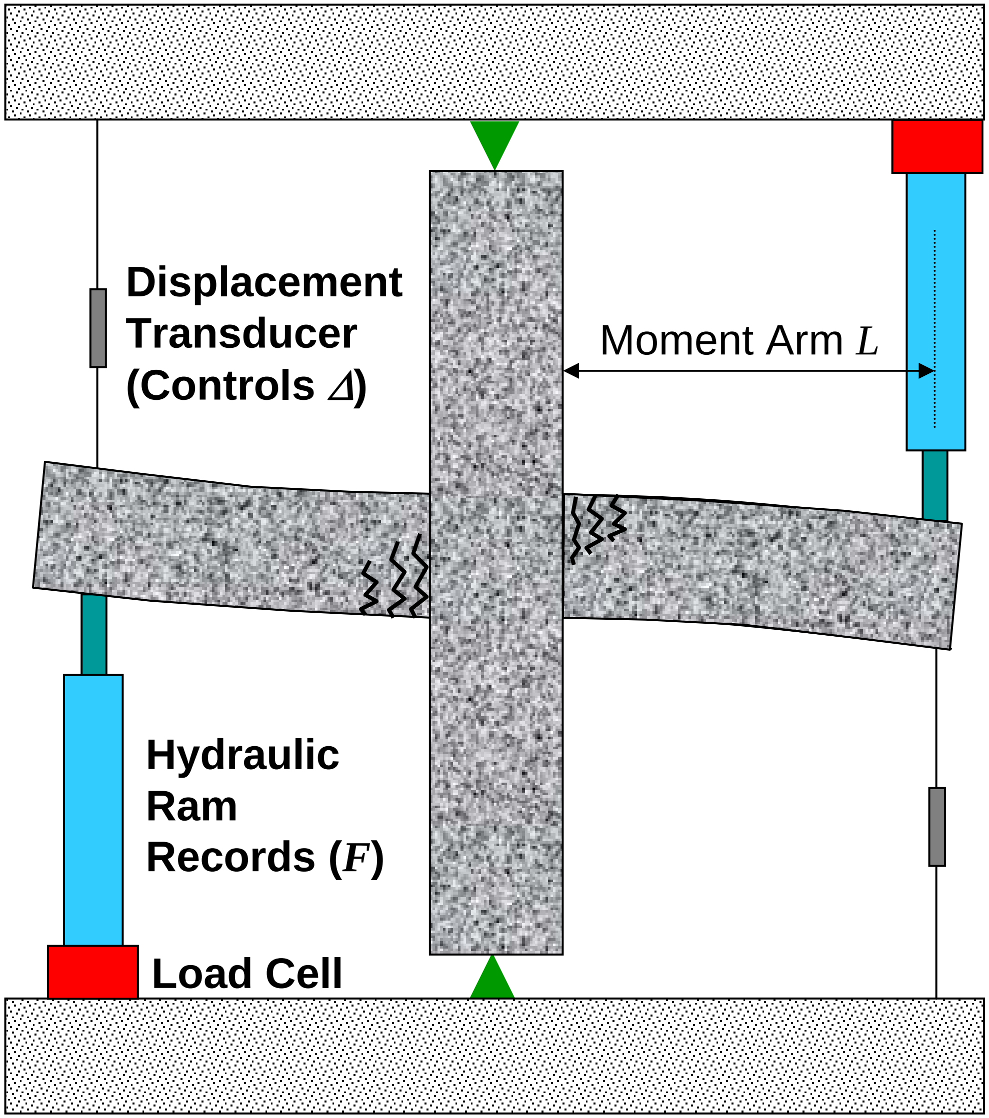


Deformations are cyclic, inelastic, and "reversed"

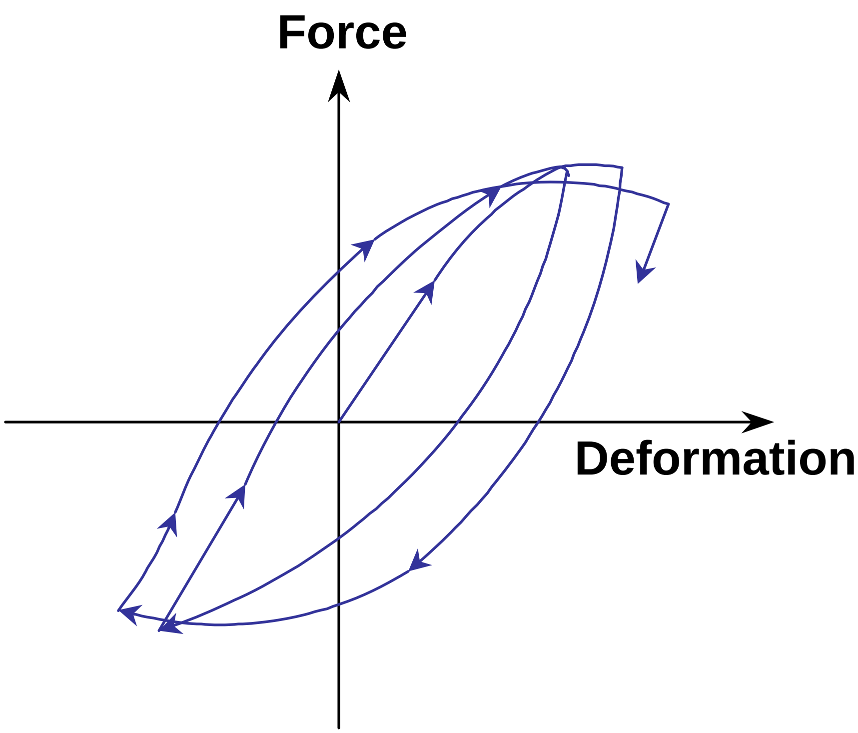
Laboratory Specimen Under Cyclic Deformation Loading



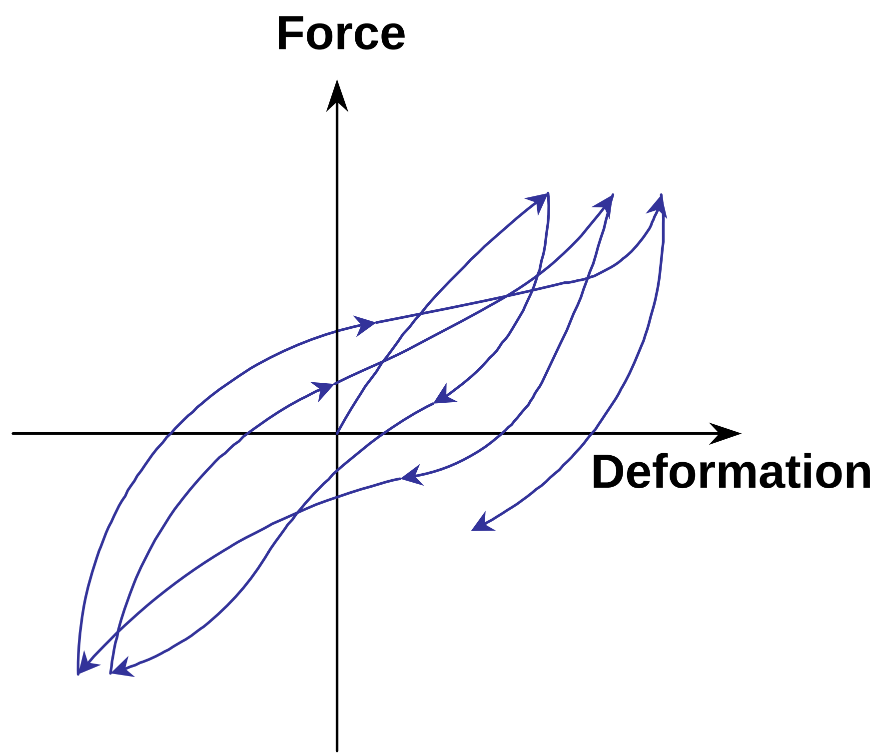
Measured Hysteretic Behavior



Hysteretic Behavior

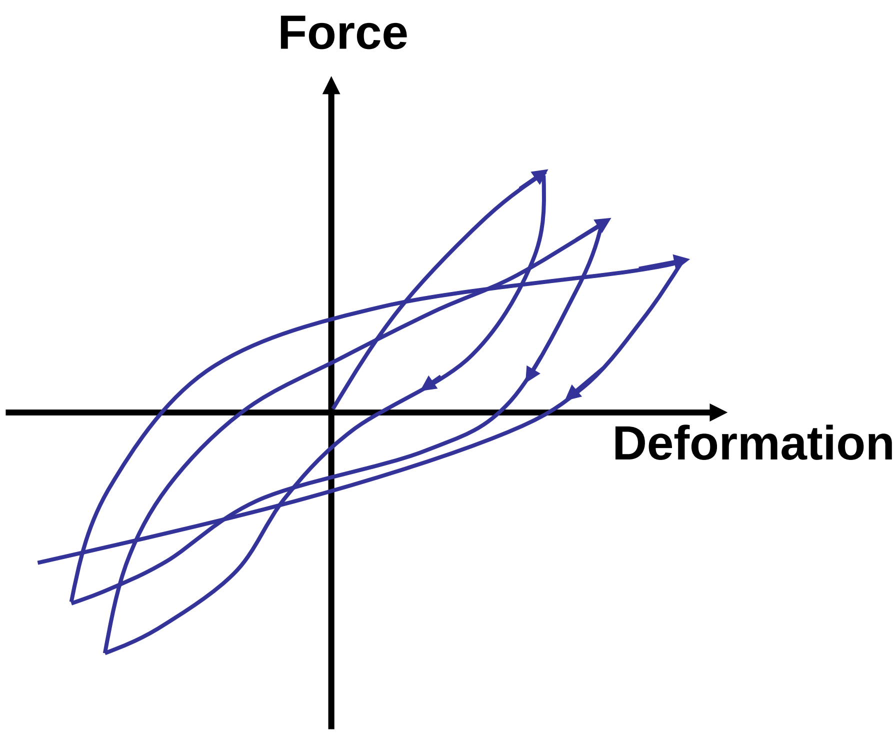


ROBUST (Excellent)

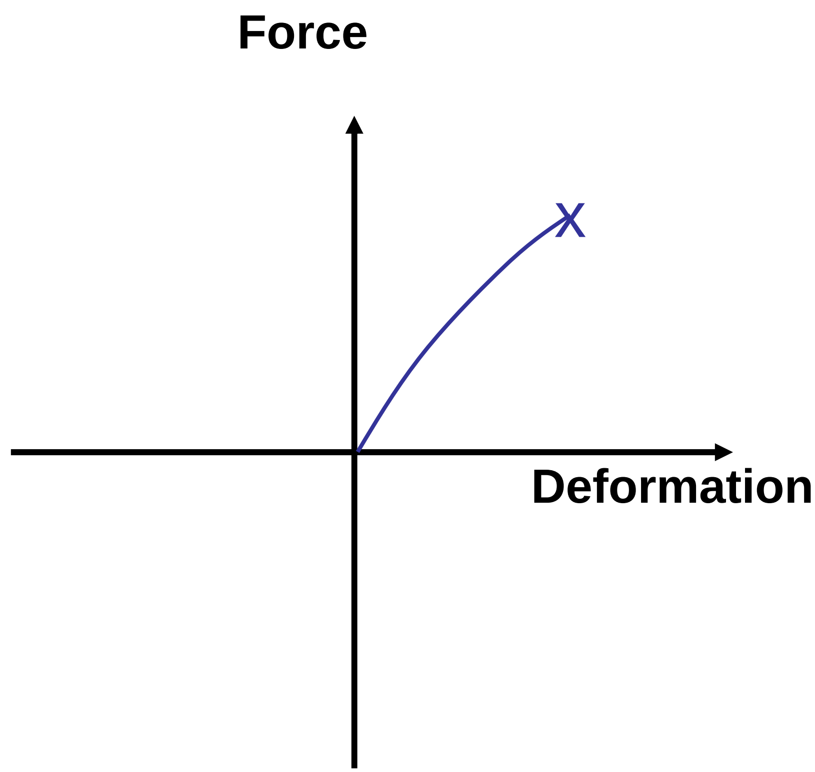


PINCHED (Good)

Hysteretic Behavior



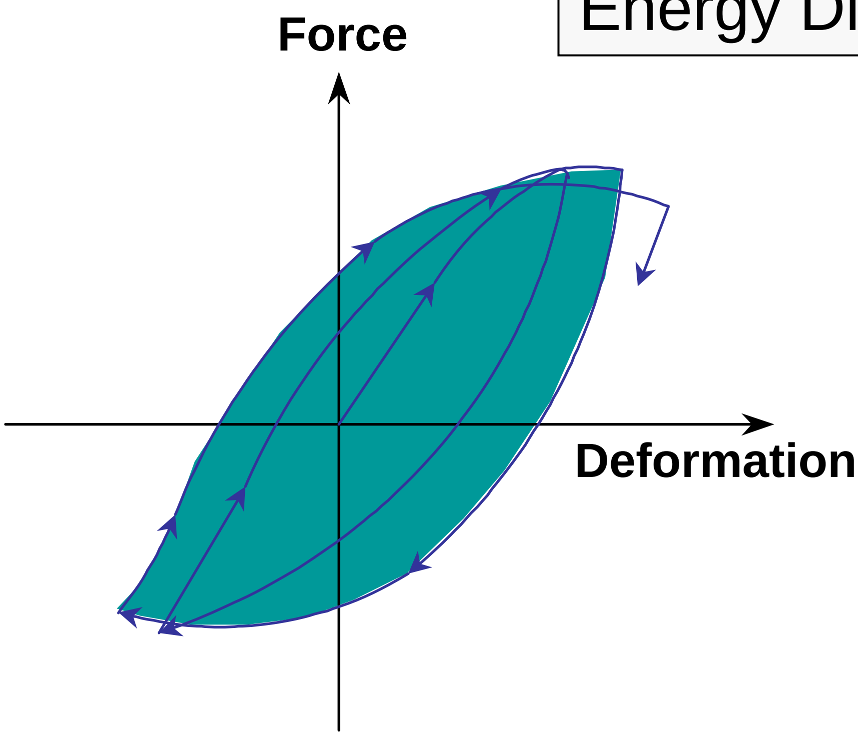
PINCHED (with strength loss)
Poor



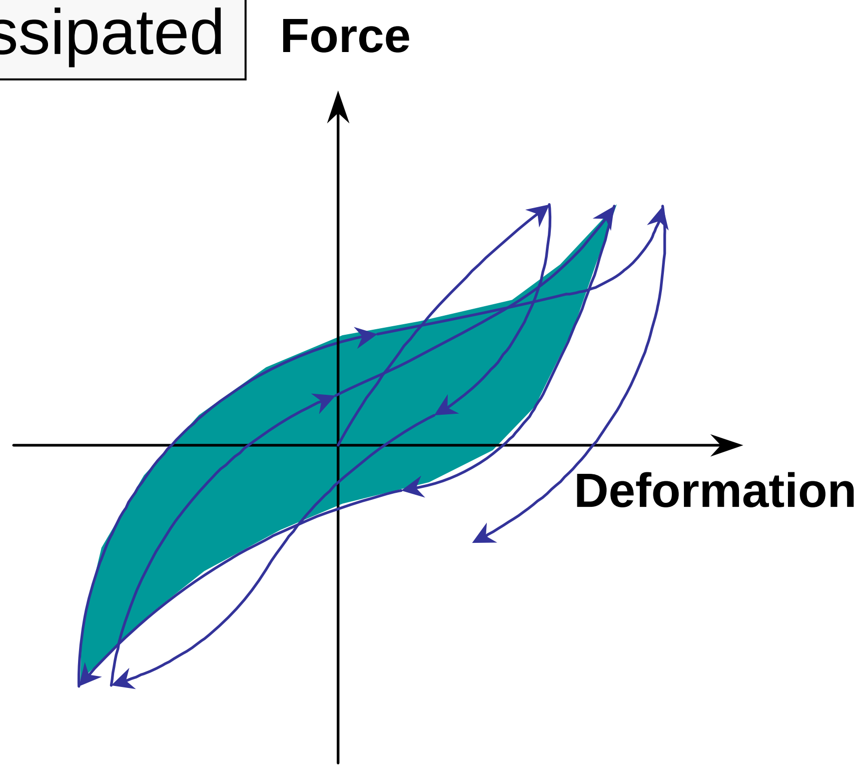
BRITTLE
Unacceptable

Hysteretic Behavior

AREA=
Energy Dissipated



ROBUST



PINCHED (No Strength Loss)

Ductility and Energy Dissipation Capacity

- The structure should be able to sustain several cycles of inelastic deformation without significant loss of strength.

- Some loss of stiffness is inevitable, but excessive stiffness loss can lead to collapse.

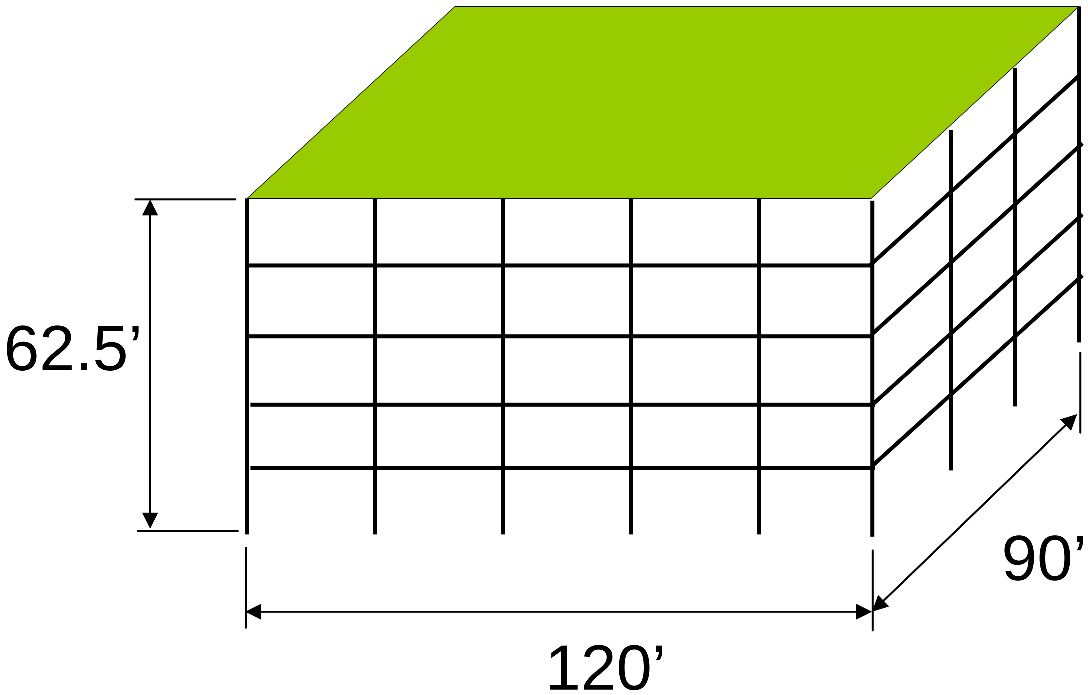
- The more energy dissipated per cycle without excessive deformation, the better the behavior of the structure.

Ductility and Energy Dissipation Capacity

- The art of seismic-resistant design is in the details.
- With good detailing, structures can be designed for force levels significantly lower than would be required for elastic response.

Why Is Inelastic Response Necessary?

Compare the Wind and Seismic Design of a Simple Building



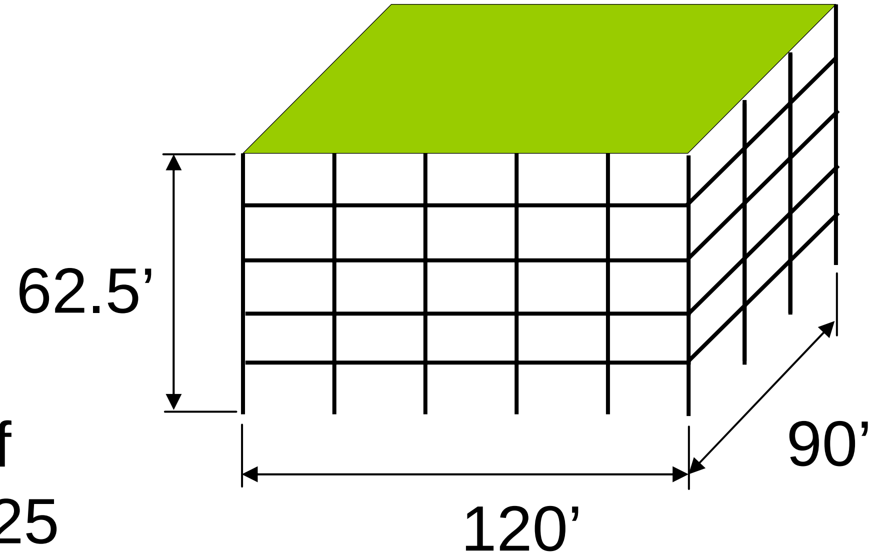
Building properties:
Moment resisting frames
Density $\rho = 8$ pcf
Period $T = 1.0$ sec
Damping $\xi = 5\%$
Soil Site Class "B"

Wind:
100 mph Exposure C

Earthquake:
Assume $S_{D1} = 0.48g$

Wind:

100 mph fastest
Exposure C



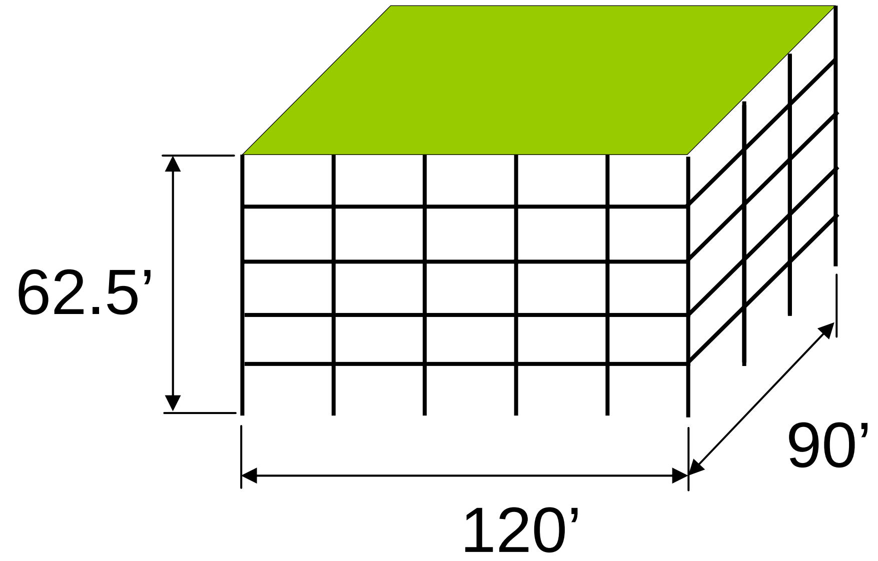
Velocity pressure q_s= 25.6 psf
Gust/exposure factor C_e = 1.25
Pressure coefficient C_q = 1.3
Load factor for wind = 1.3

Total wind force on 120-foot face:
V_{W120}= 62.5*120*25.6*1.25*1.3*1.3/1000 = **406 kips**

Total wind force on 90-foot face:
V_{W90} = 62.5*90*25.6*1.25*1.3*1.3/1000 = **304 kips**

Earthquake:

Building weight, W = 120*90*62.5*8/1000 = 5400 kips



$$V_{EQ} = C_S W$$

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.48}{1.0(1.0/1.0)} = 0.480$$

Total ***ELASTIC*** earthquake force (in each direction):
V_{EQ} = 0.480*5400 = **2592 kips**

Comparison: Earthquake vs. Wind

$$\frac{V_{EQ}}{V_{W120}} = \frac{2592}{406} = 6.4$$

$$\frac{V_{EQ}}{V_{W90}} = \frac{2592}{304} = 8.5$$

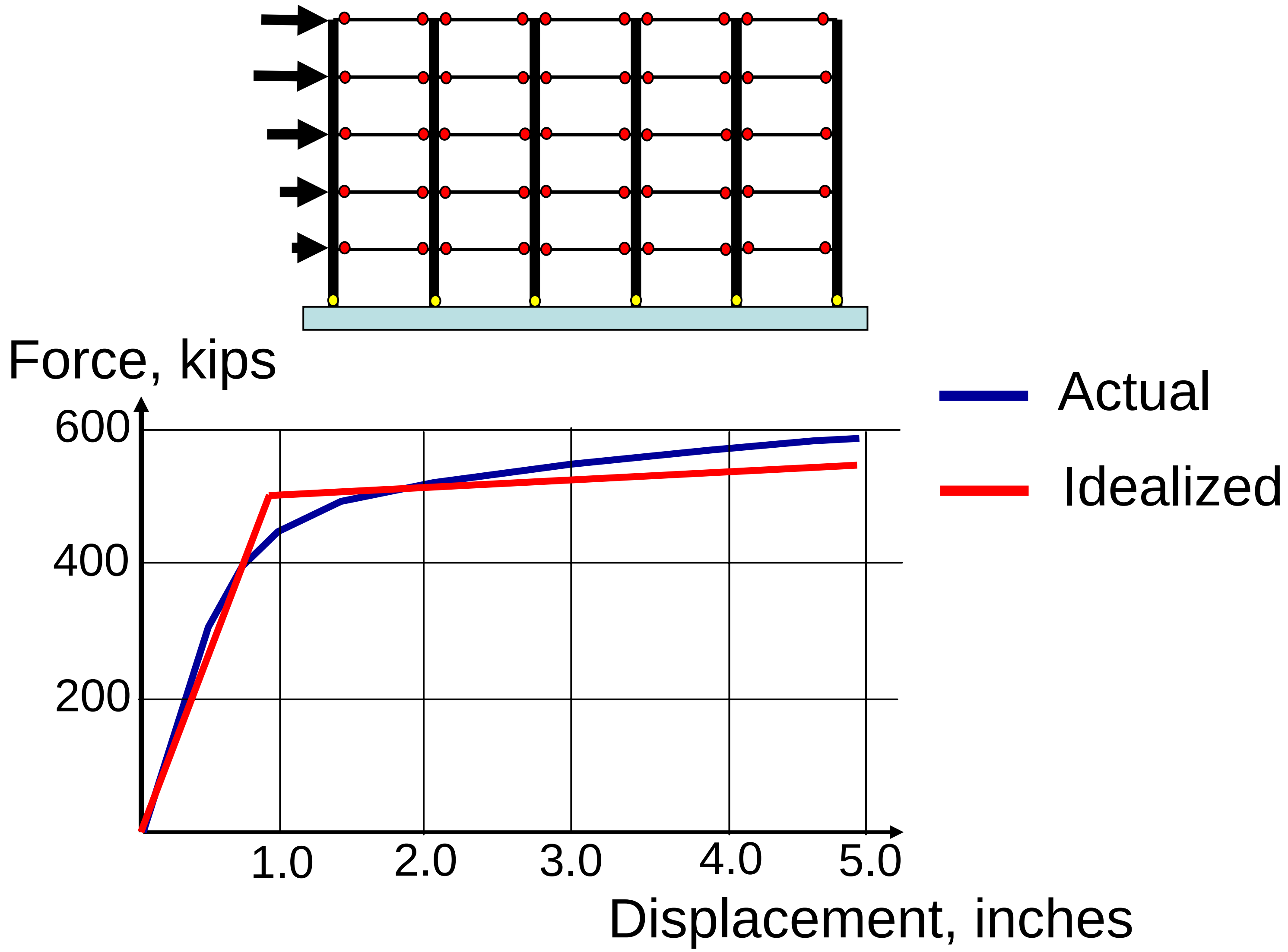
- ELASTIC earthquake forces 6 to 9 times wind!
- Virtually impossible to obtain economical design

How to Deal with Huge Earthquake Force?

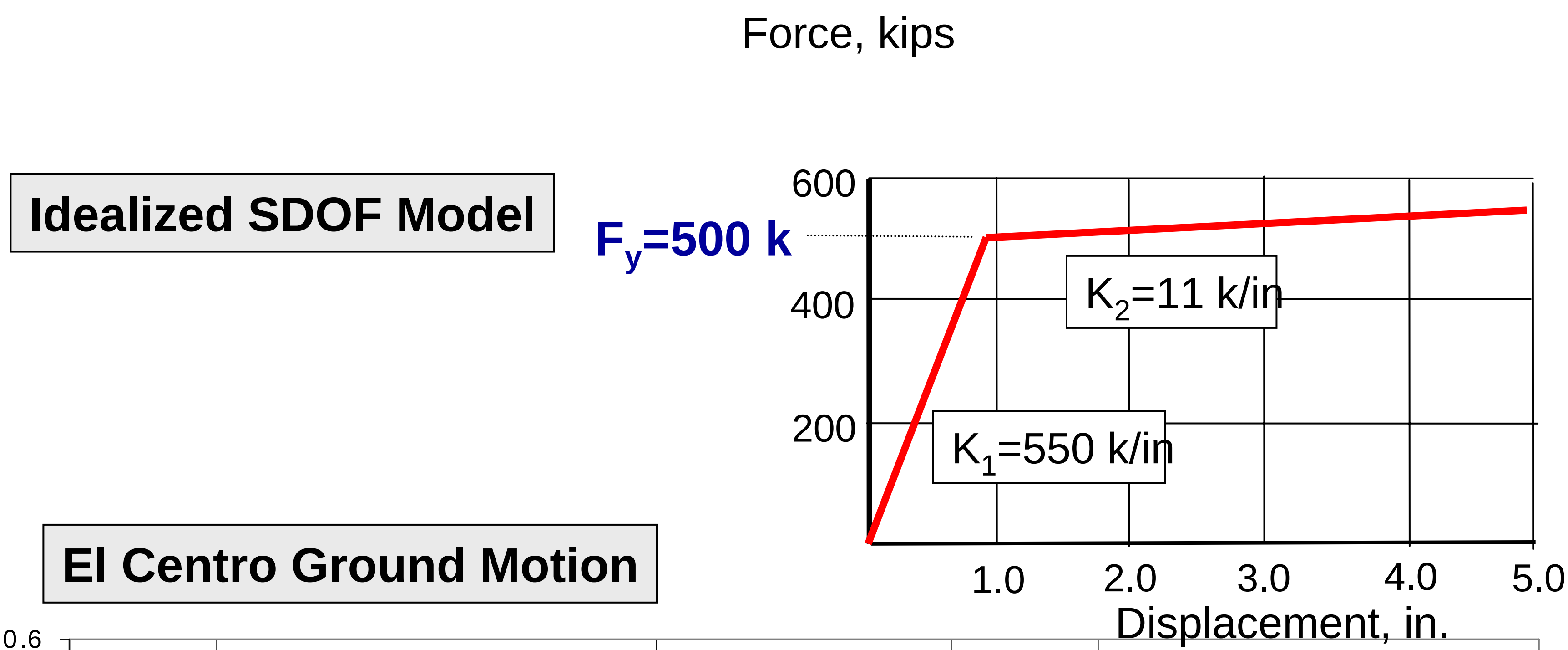
- Isolate structure from ground (base isolation)
- Increase damping (passive energy dissipation)
- Allow controlled inelastic response

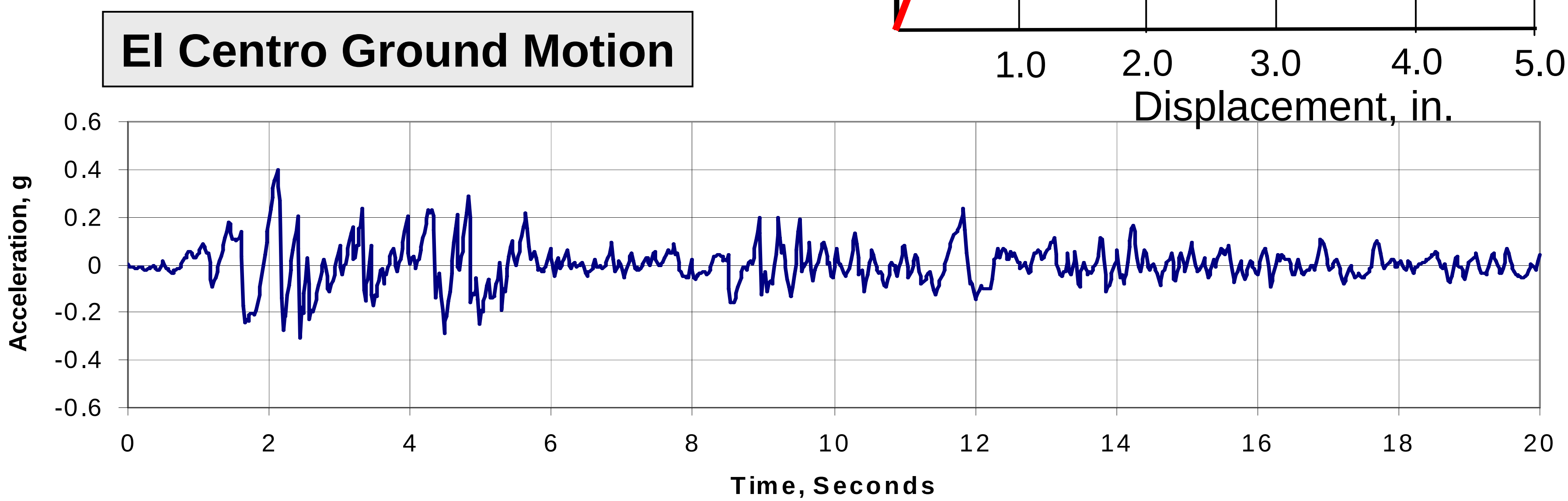
Historically, building codes use **inelastic response procedure**.
Inelastic response occurs through structural **damage** (yielding).
We must control the damage for the method to be successful.

Assume Frame Is Designed for Wind
"Pushover" Analysis Predicts Strength = 500 k



How Will Frame Respond During 0.4g El Centro Earthquake?





Response Computed by NONLIN

Maximum displacement:

4.79”

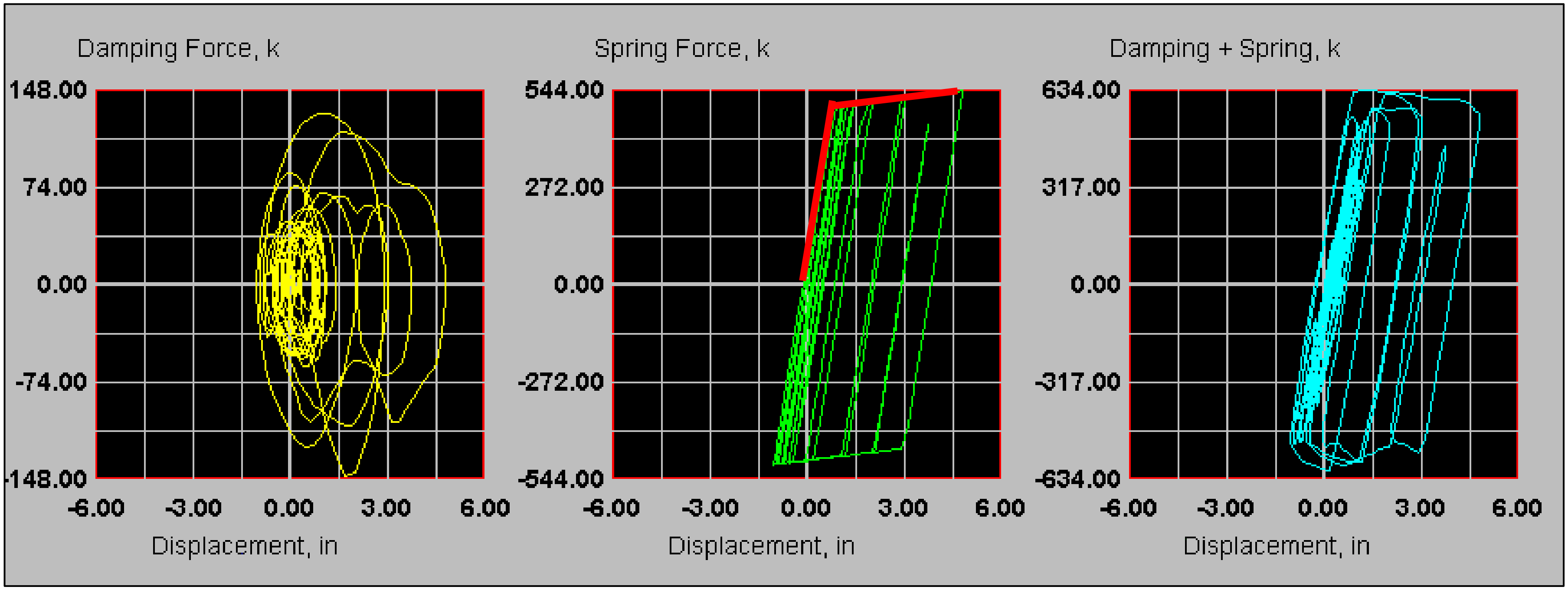
Maximum shear force:

542 k

Number of yield events:

15

Response Computed by NONLIN



Yield displacement = 500/550 = 0.91 inch

$$\textbf{\textit{Ductility Demand}} \equiv \frac{\text{Maximum Displacement}}{\text{Yield Displacement}} = \frac{4.79}{0.91} = 5.26$$

Interim Conclusion (The Good News)

The frame, designed for a wind force <u>that is 15% of the ELASTIC earthquake force</u>, can survive the earthquake if:

- It has the capability to undergo ***numerous cycles of INELASTIC deformation.***
- It has the capability to ***deform at least 5 to 6 times the yield deformation.***
- It suffers ***no appreciable loss of strength.***

REQUIRES ADEQUATE DETAILING

Interim Conclusion (The Bad News)

As a result of the large displacements associated with the inelastic deformations, the structure will suffer considerable structural and nonstructural damage.

- This damage must be controlled by adequate detailing and by limiting structural deformations (drift).

Development of “Equal Displacement” Concept of Seismic Resistant Design

Concept used by:

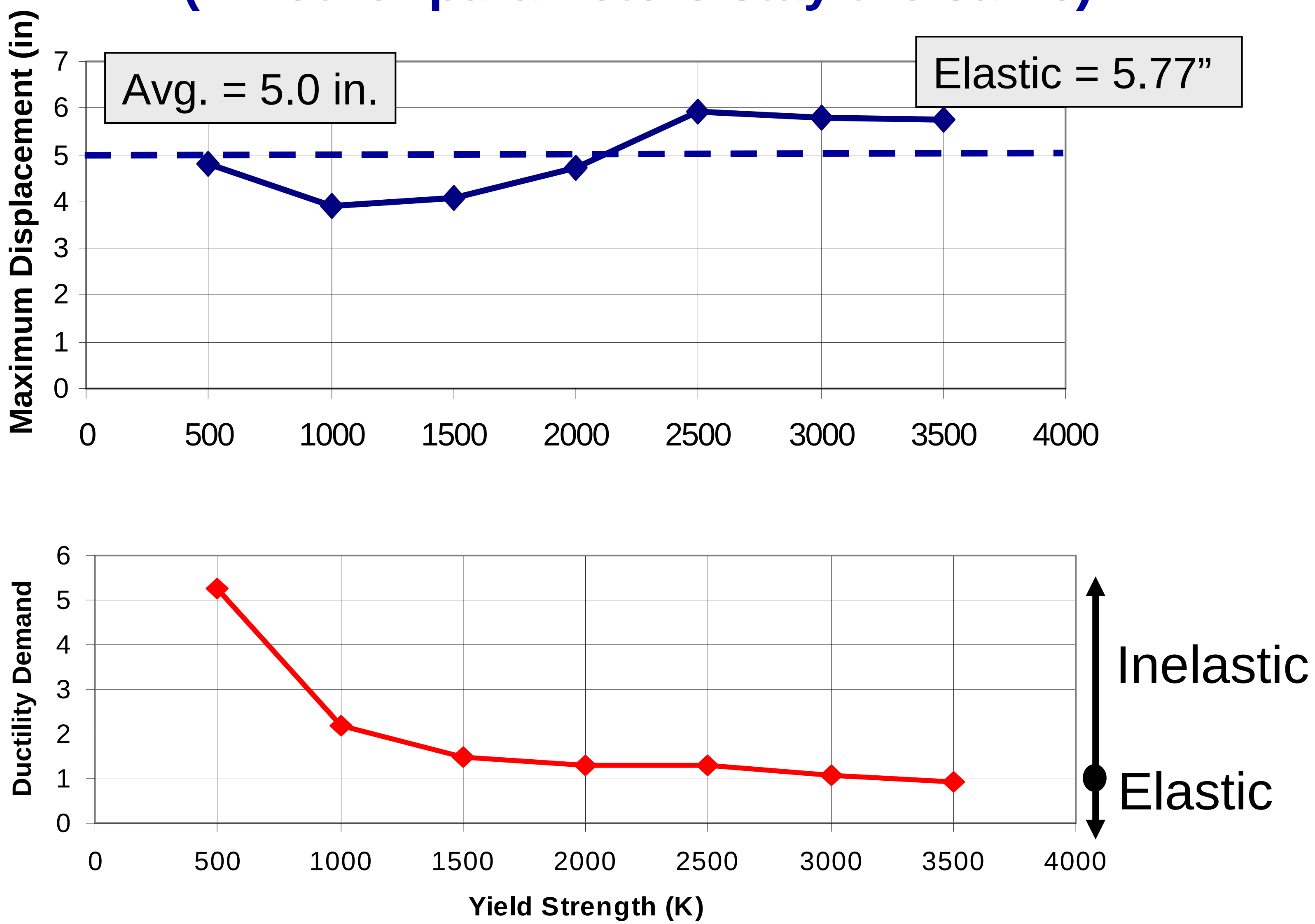
IBC NEHRP ASCE-7	In association with “force based” design concept. Used to predict design forces and displacements
FEMA 273	In association with static pushover analysis. Used to predict displacements at various performance points.

The Equal Displacement Concept

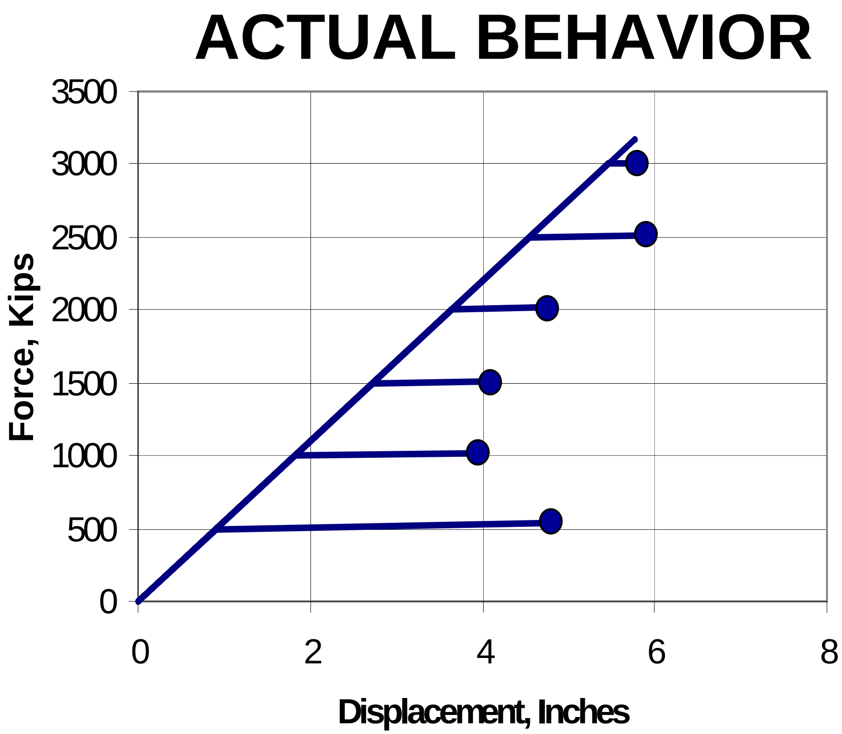
"The displacement of an inelastic system, with stiffness K and strength F_y, subjected to a particular ground motion, is approximately equal to the displacement of the same system responding elastically."

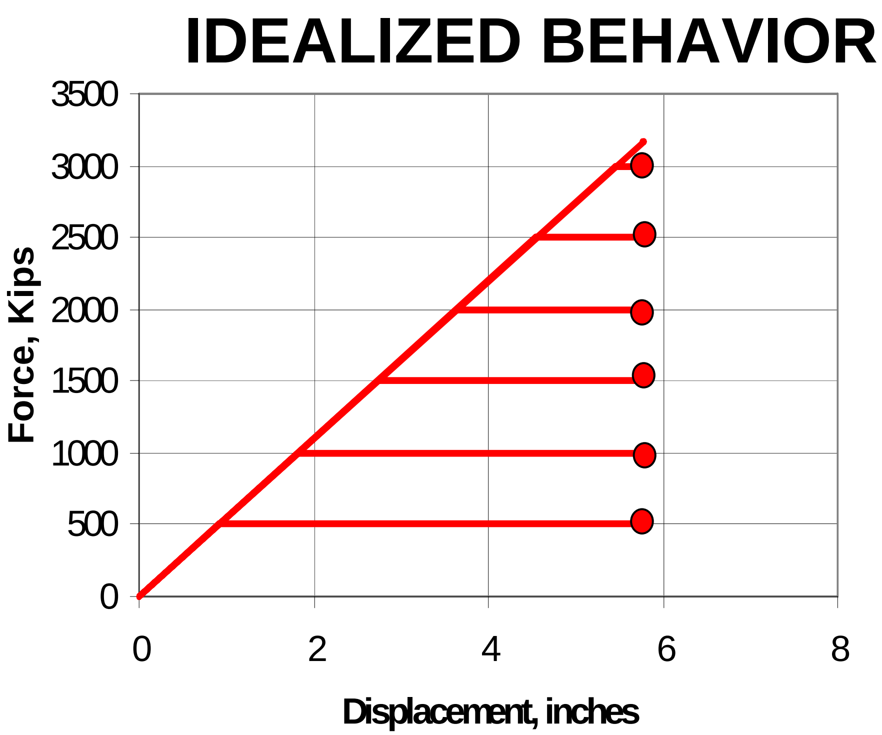
(The displacement of a system is independent of the yield strength of the system.)

Repeat Analysis for Various Yield Strengths (All other parameters stay the same)



Constant Displacement Idealization of Inelastic Response



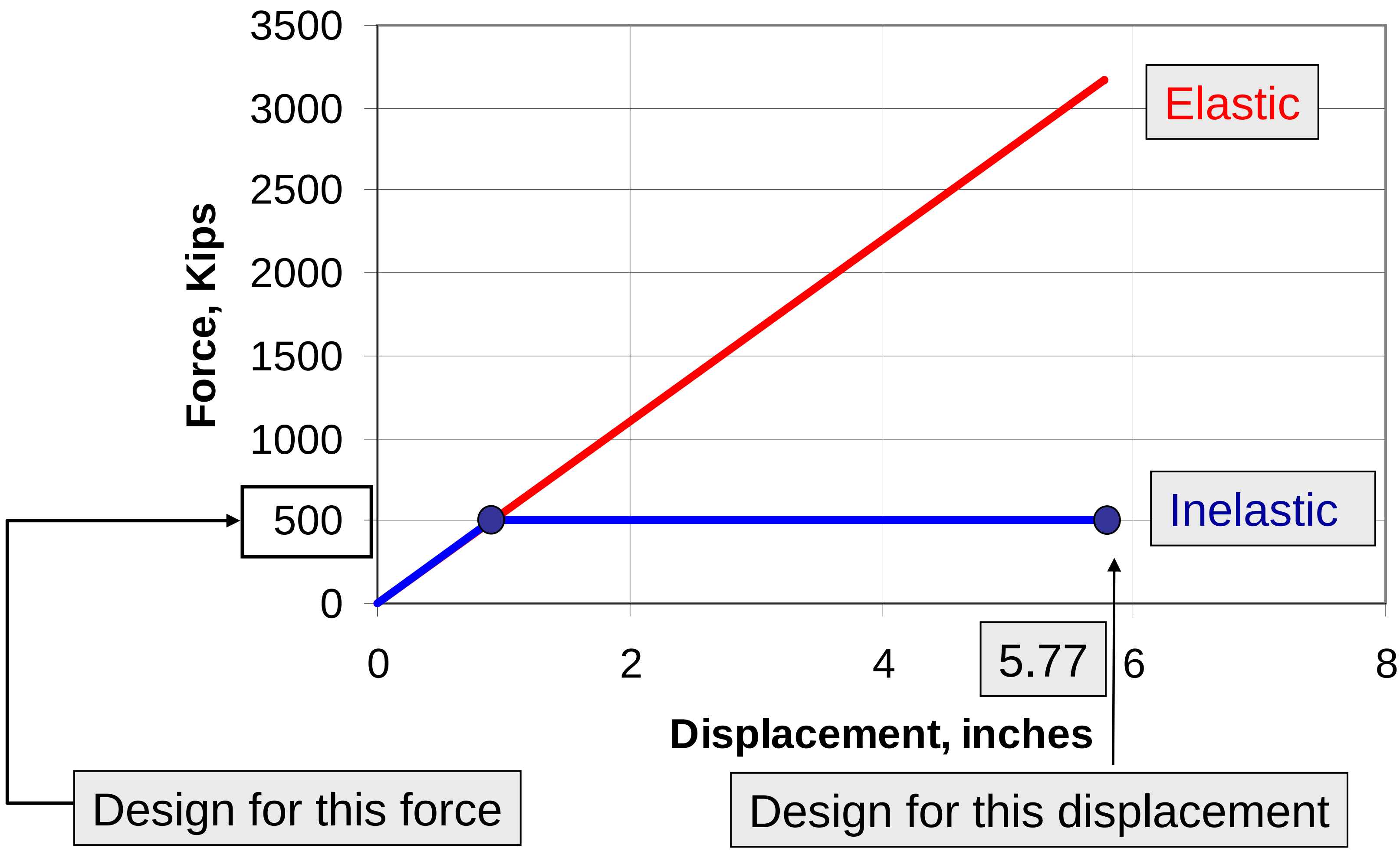


Equal Displacement Idealization of Inelastic Response

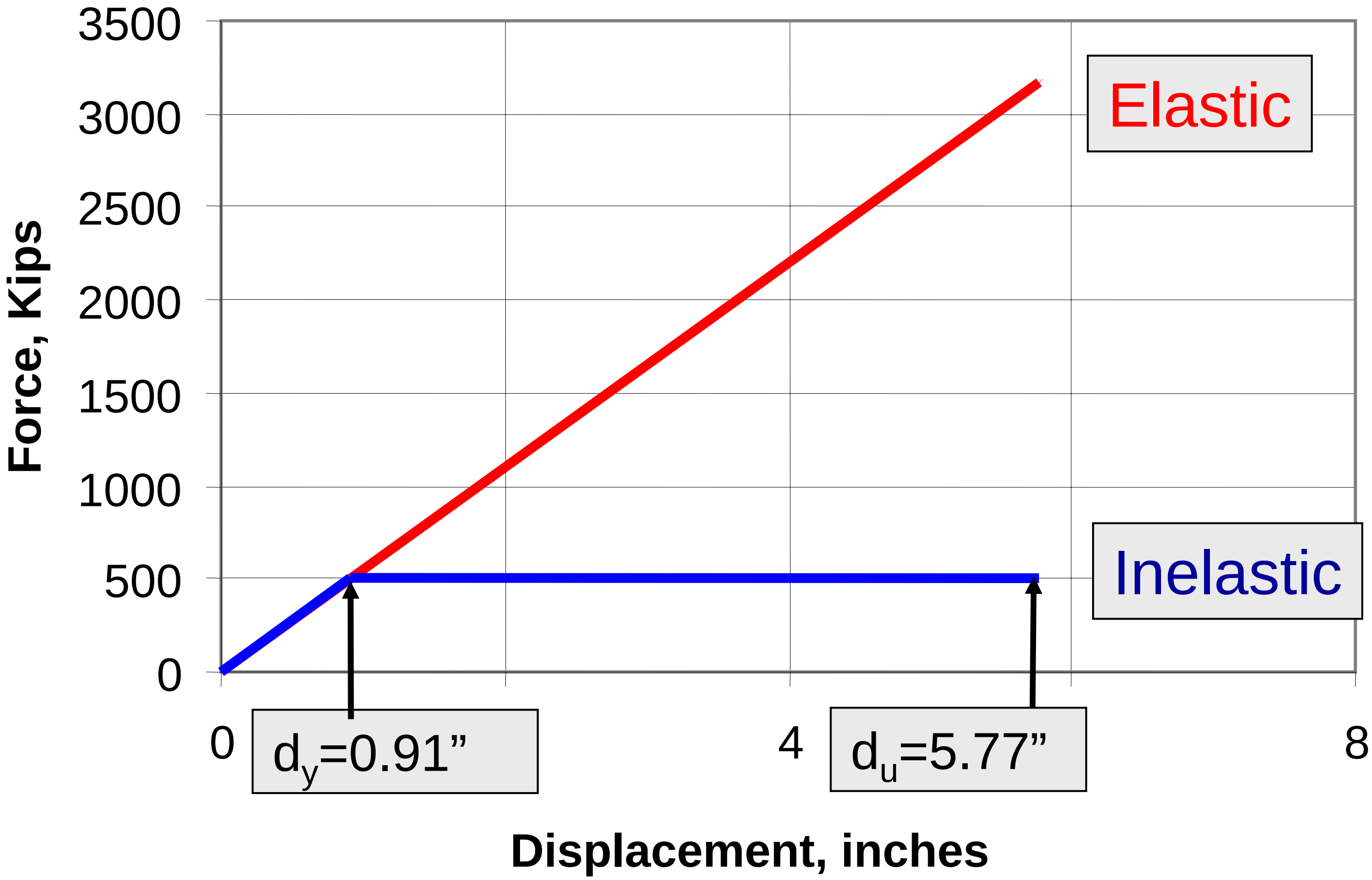
- For design purposes, it may be assumed that inelastic displacements are equal to the displacements that would occur during an elastic response.

- The required force levels under inelastic response are much less than the force levels required for elastic response.

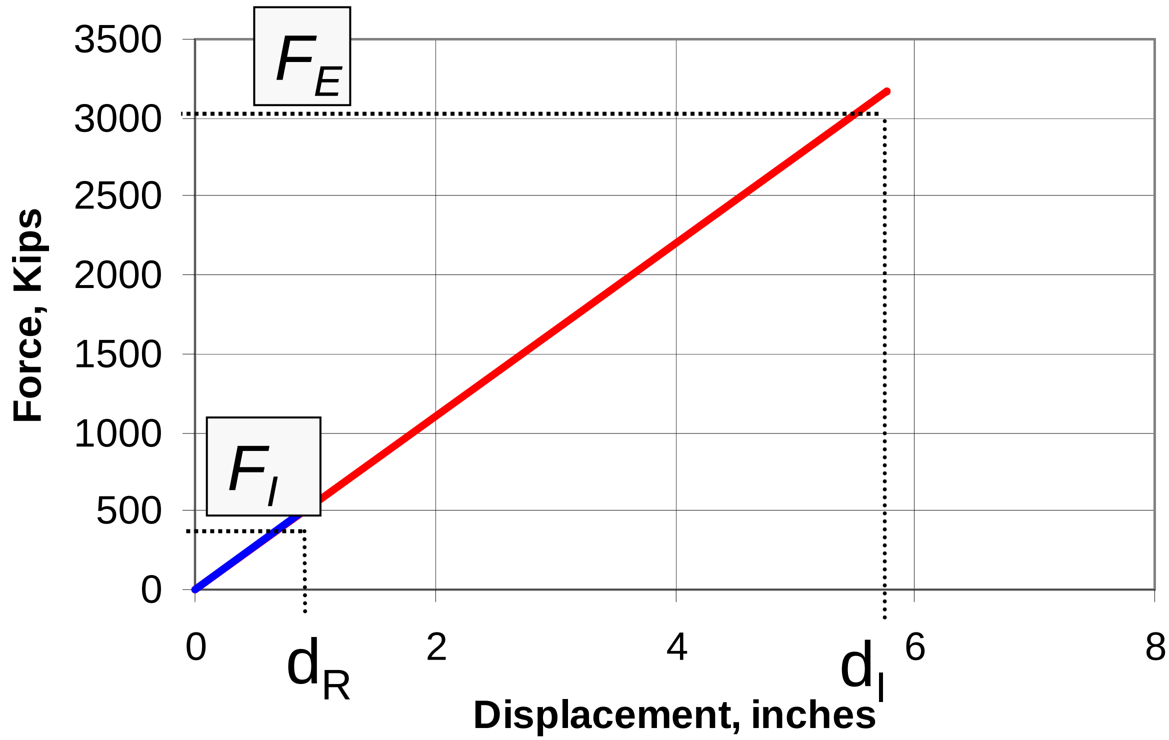
Equal Displacement Concept of Inelastic Design



Equal Displacement Concept of Inelastic Design



Ductility supply MUST BE > ductility demand = $\frac{5.77}{0.91} = 6.34$



Using response spectra, estimate **elastic** force demand $\boldsymbol{F_E}$

Estimate ductility supply, μ, and determine **inelastic** force demand $\boldsymbol{F_I = F_E / \mu}$.. **Design structure for** $\boldsymbol{F_I}$.

Compute reduced displacement. $\boldsymbol{d_R}$, and multiply by μ to obtain true inelastic eisplacement, $\boldsymbol{d_I}$. **Check drift using** $\boldsymbol{d_i}$.

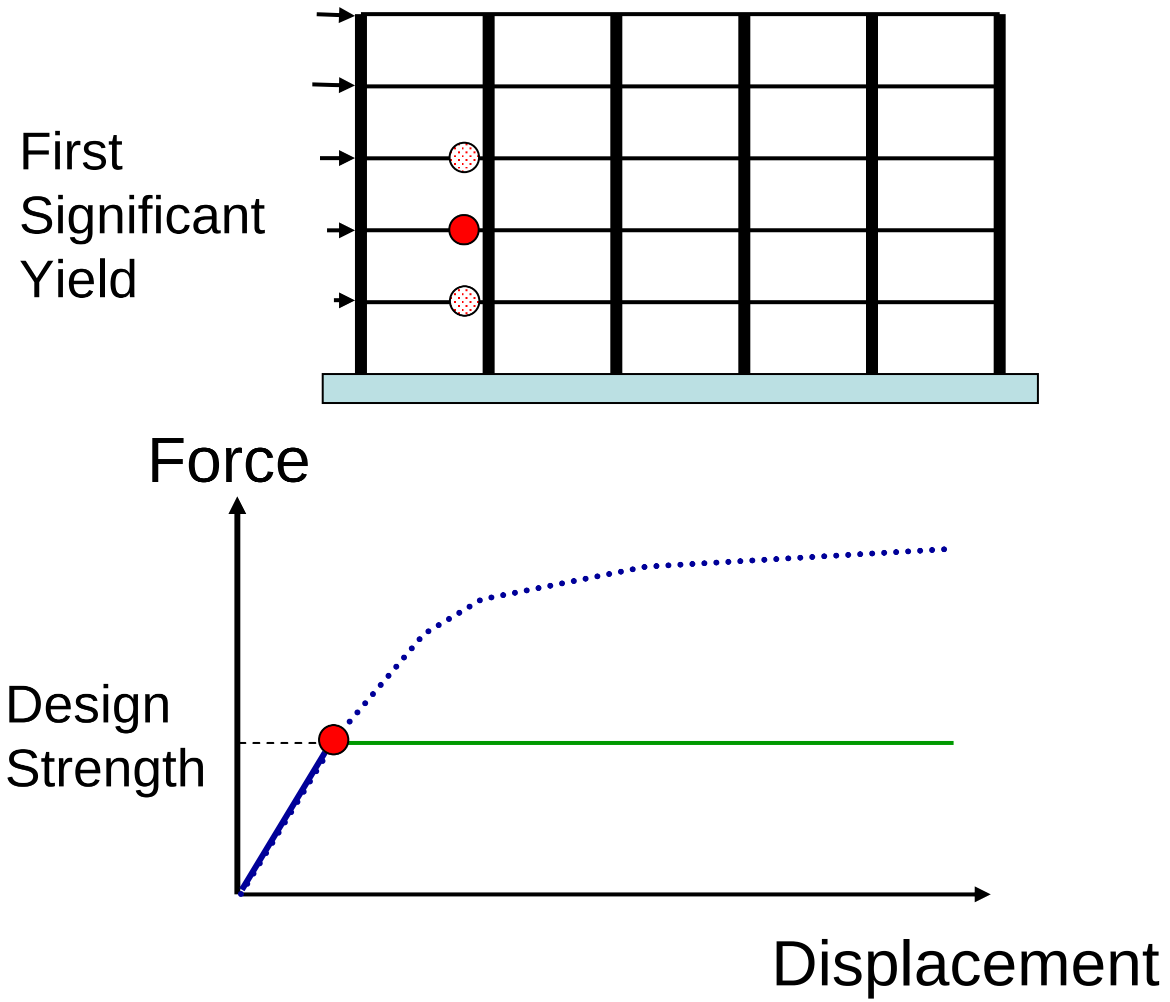
ASCE 7 Approach

Use basic elastic spectrum but, for strength, divide all pseudoacceleration values by ***R***, a response modification factor that accounts for:

- Anticipated ductility supply
- Overstrength
- Damping (if different than 5% critical)
- Past performance of similar systems
- Redundancy

Ductility/Overstrength

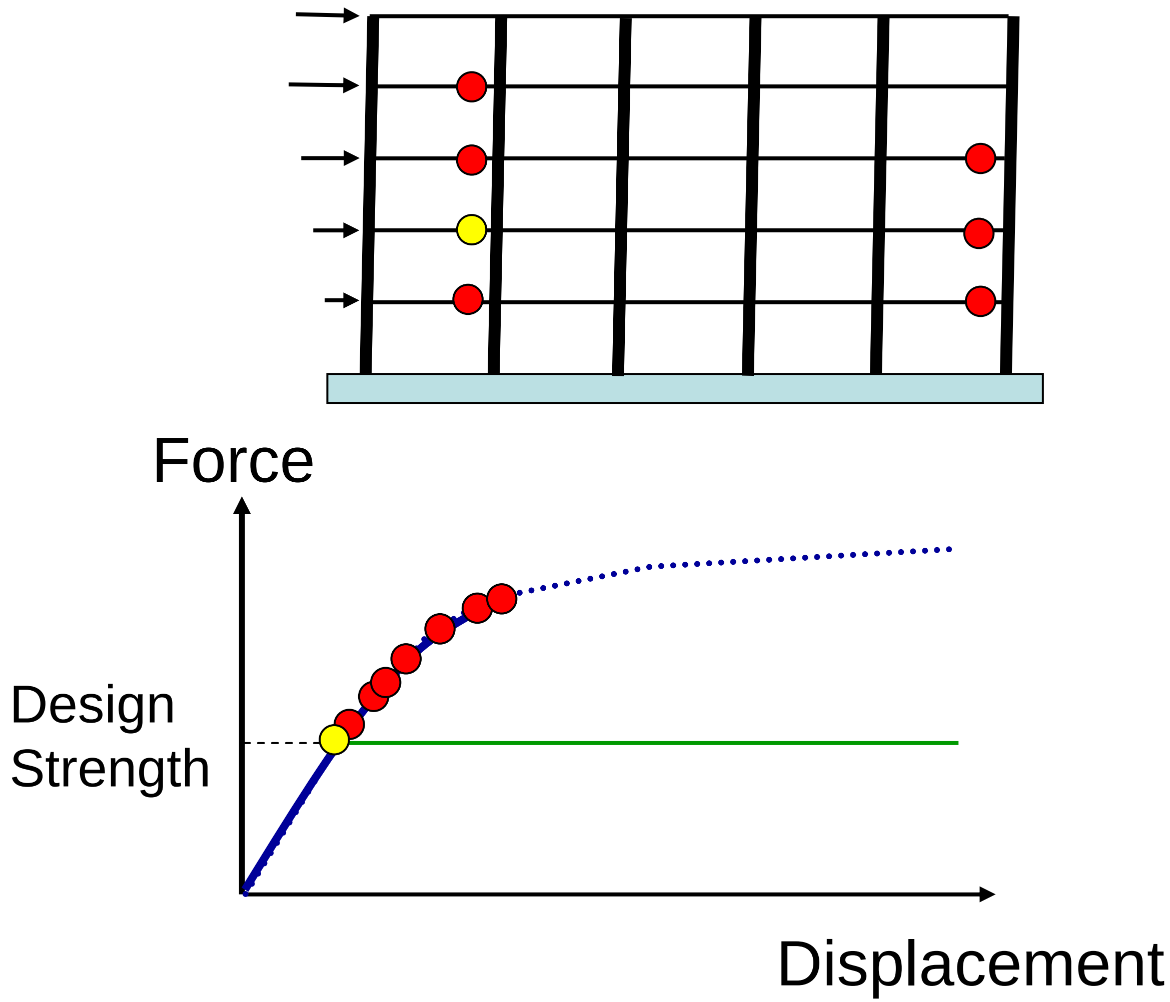
FIRST SIGNIFICANT YIELD



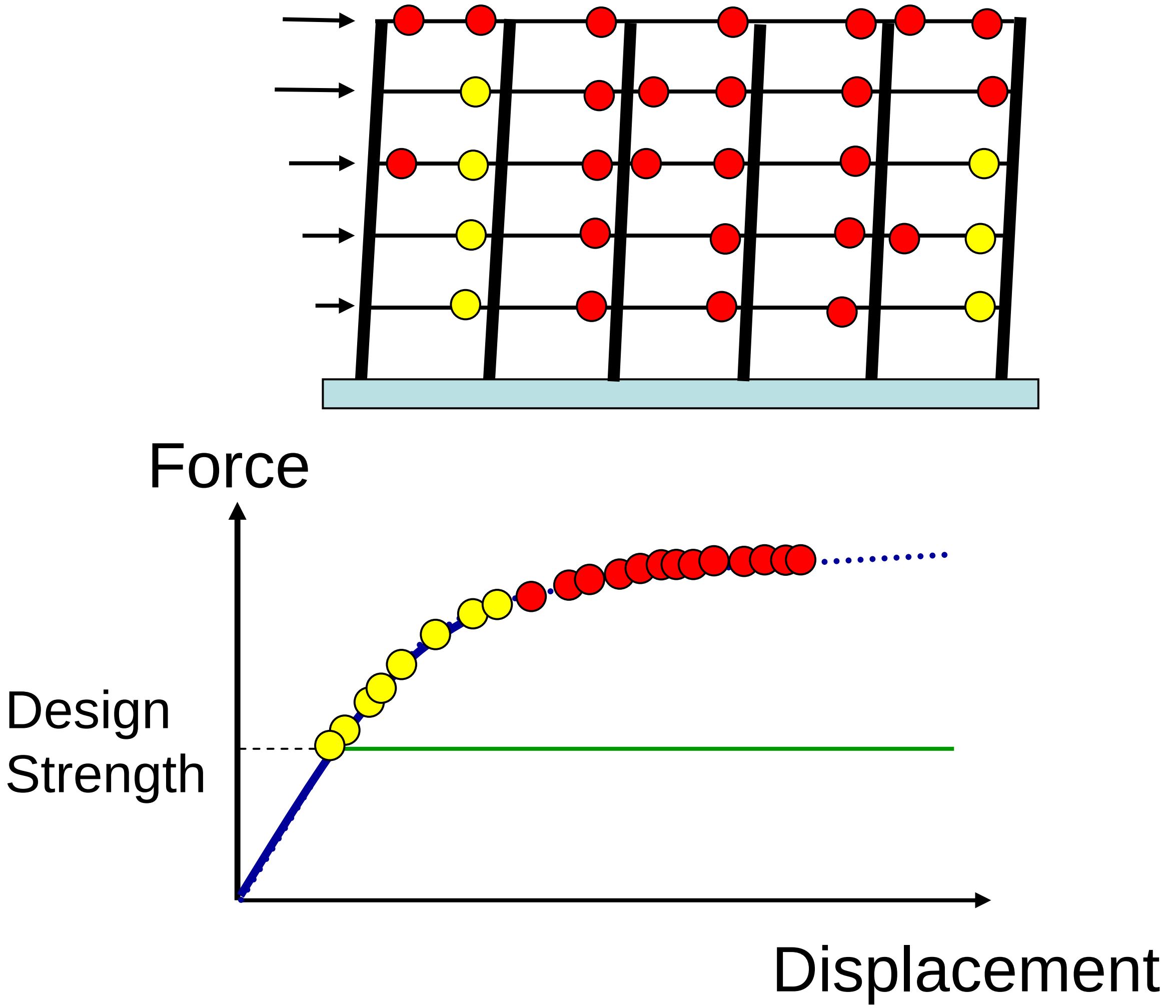
First Significant Yield is the level of force that causes complete plastification of at least the most critical region of the structure (e.g., formation of the first plastic hinge).

The **design strength** of a structure is equal to the resistance at first significant yield.

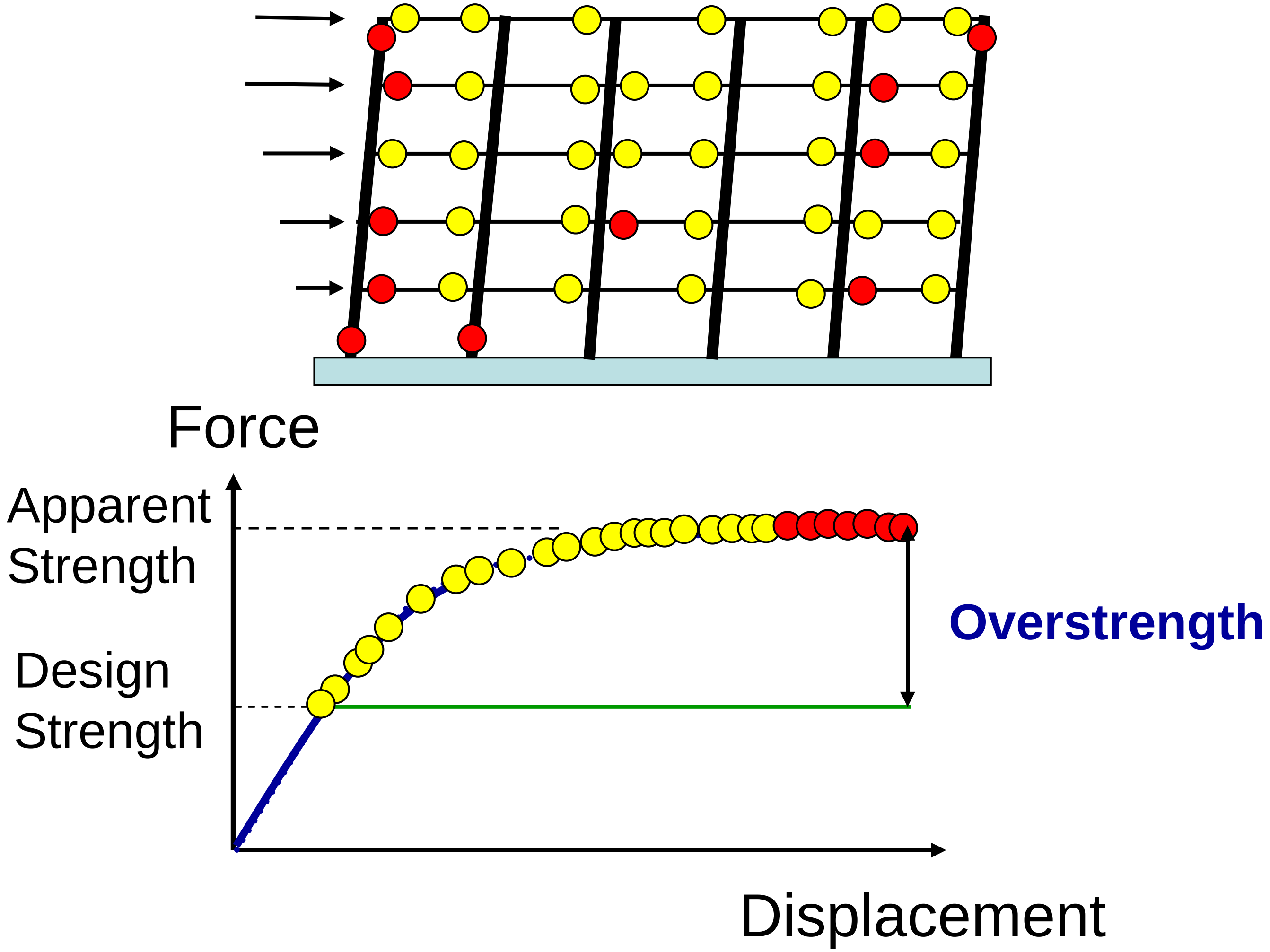
Overstrength (1)



Overstrength (2)



Overstrength (3)

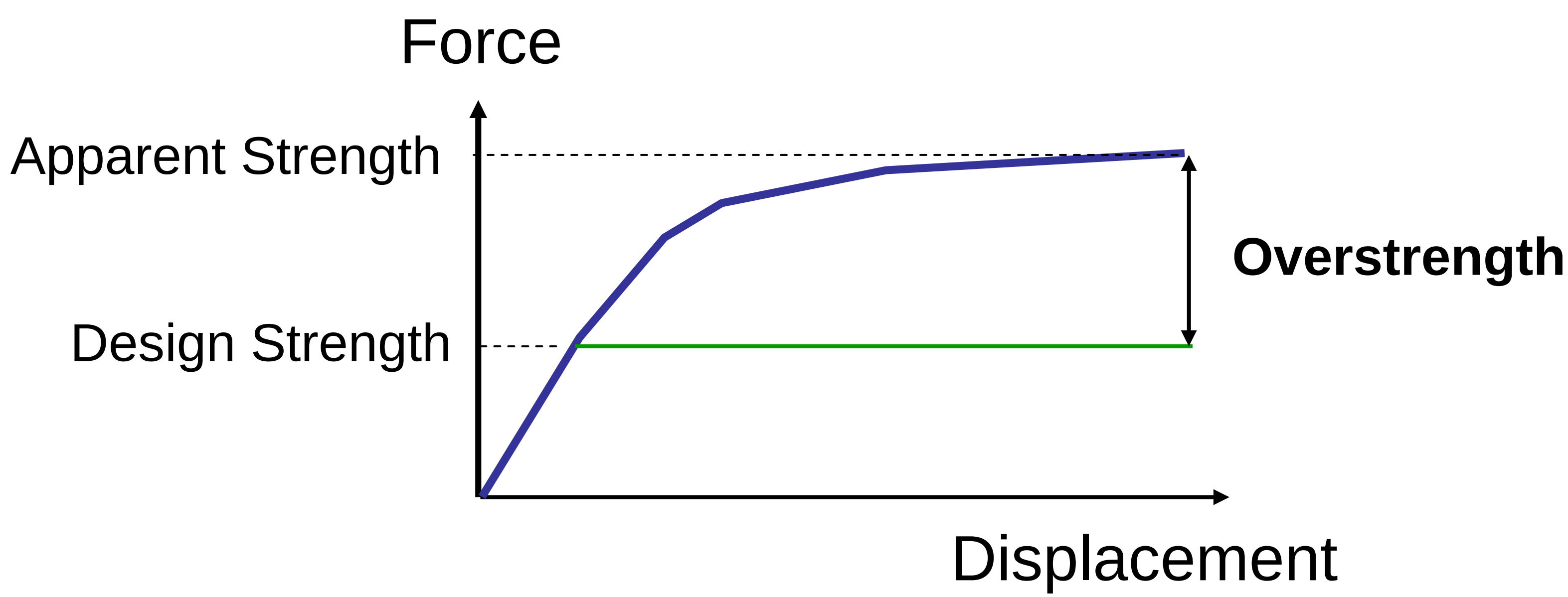


Sources of Overstrength

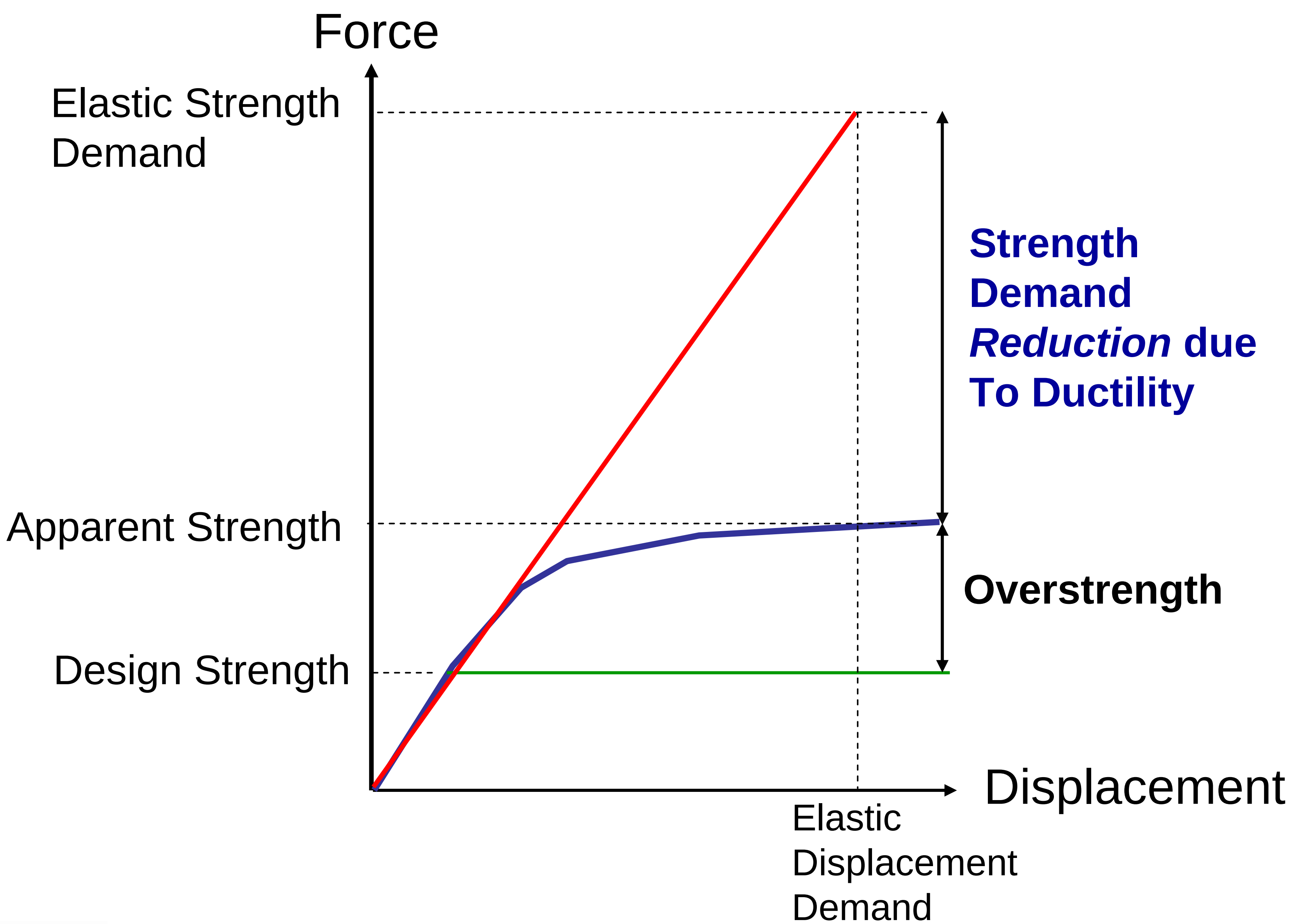
- Sequential yielding of critical regions
- Material overstrength (actual vs specified yield)
- Strain hardening
- Capacity reduction (ϕ) factors
- Member selection

Definition of Overstrength Factor Ω

$$\text{Overstrength Factor } \Omega = \frac{\text{Apparent Strength}}{\text{Design Strength}}$$

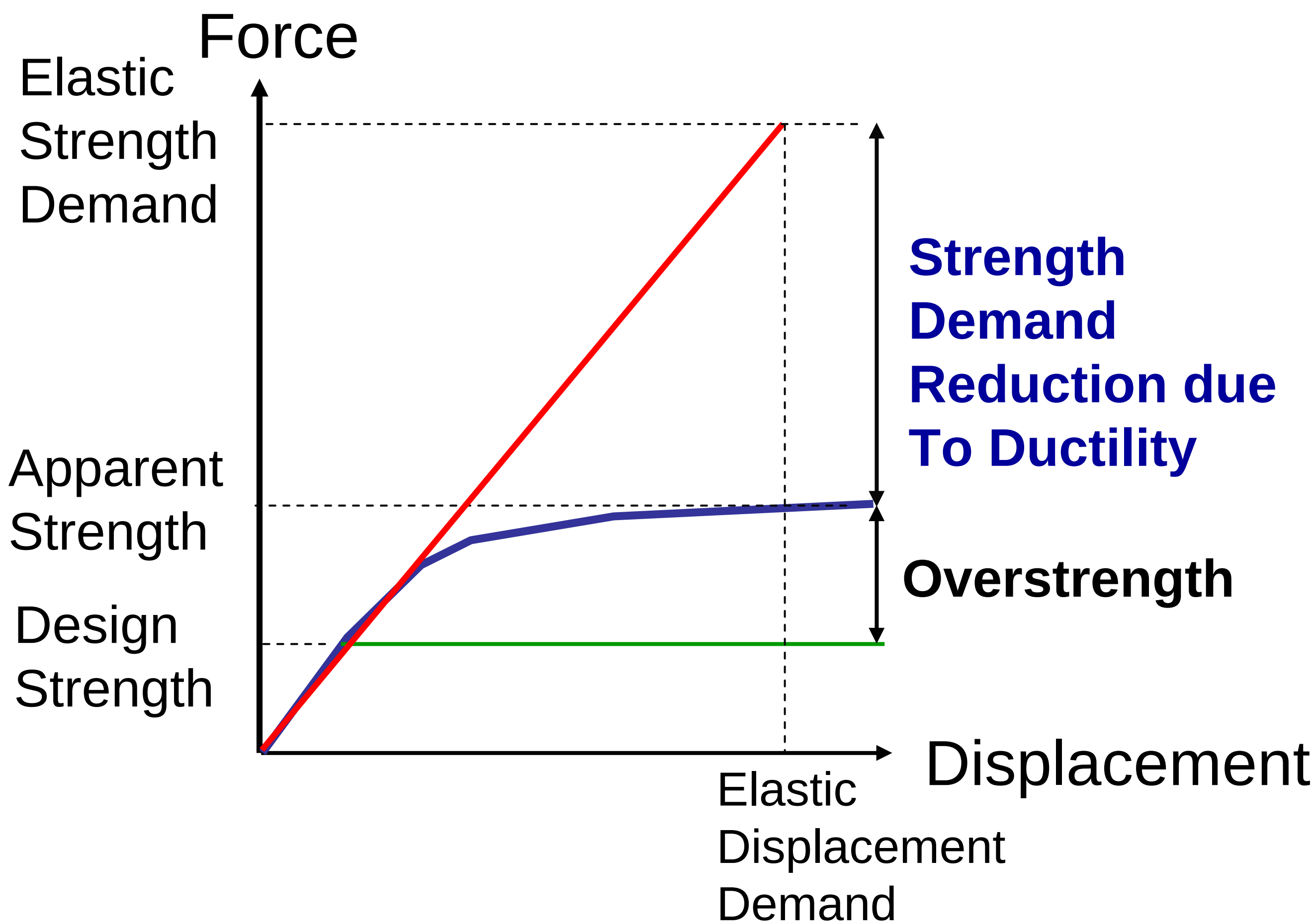


Definition of Ductility Reduction Factor R_d



Definition of Ductility Reduction Factor

$$\text{Ductility Reduction } R_d = \frac{\text{Elastic Strength Demand}}{\text{Apparent Strength}}$$



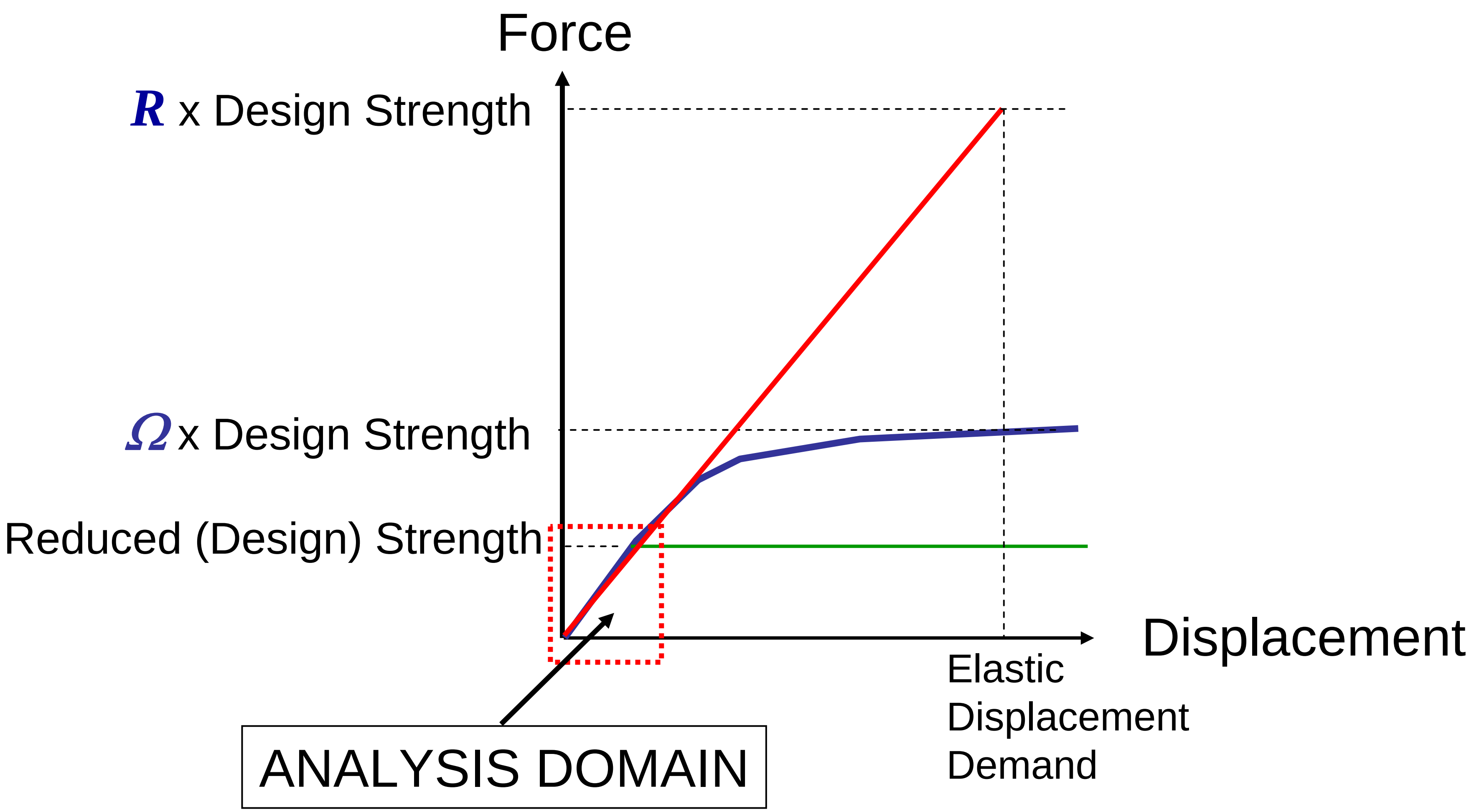
Definition of Response Modification Coefficient R

$$\text{Overstrength Factor } \Omega = \frac{\text{Apparent Strength}}{\text{Design Strength}}$$

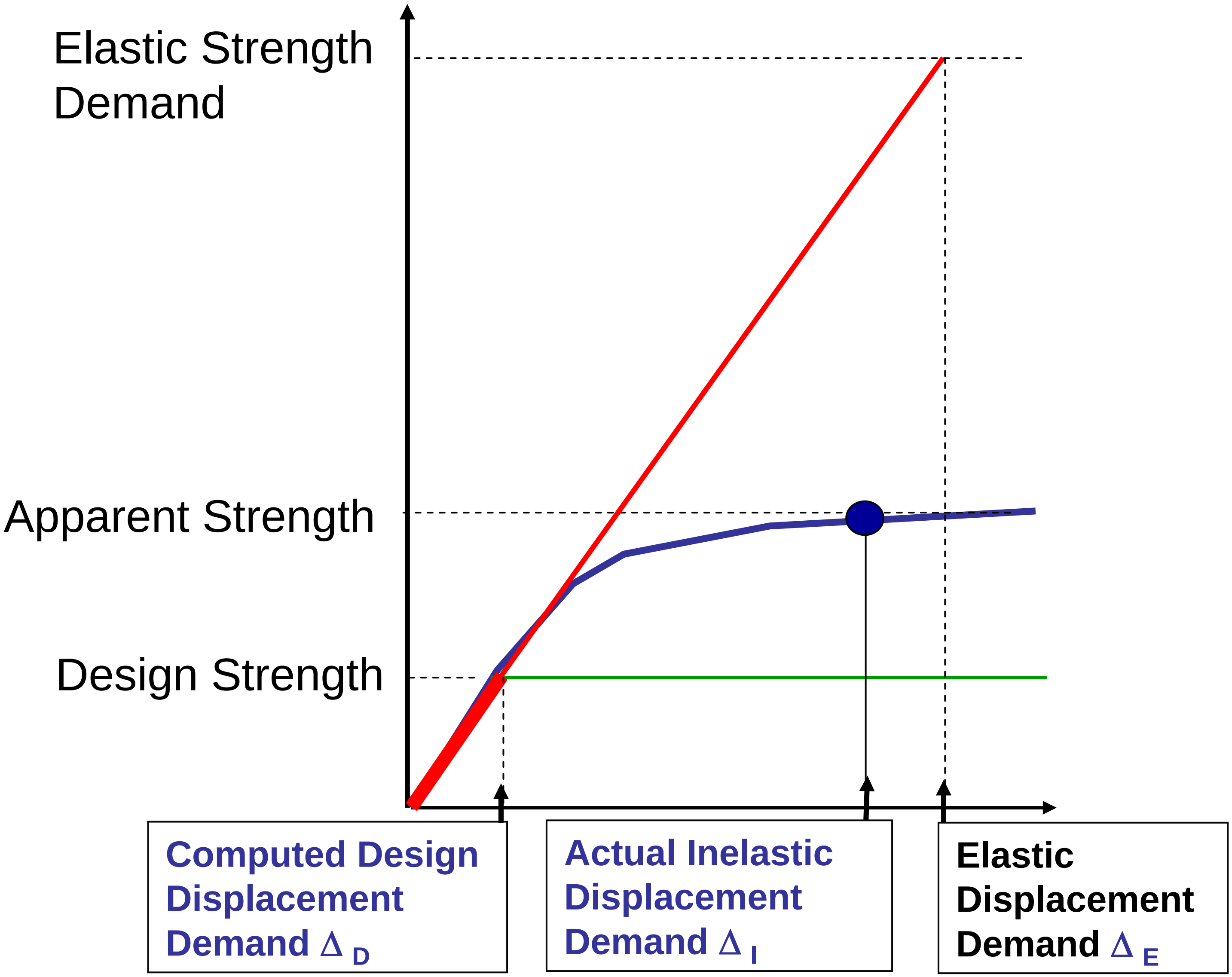
$$\text{Ductility Reduction } R_d = \frac{\text{Elastic Strength Demand}}{\text{Apparent Strength}}$$

$$R = \frac{\text{Elastic Strength Demand}}{\text{Design Strength}} = R_d\,\Omega$$

Definition of Response Modification Coefficient *R*



Definition of Deflection Amplification Factor Coefficient C_d



ASCE 7 Approach for Displacements

Determine design forces: $V = C_s W$, where C_s includes ductility/overstrength reduction factor R.

Distribute forces vertically and horizontally and compute displacements using linear elastic analysis.

Multiply computed displacements by C_d to obtain estimate of true inelastic response.

Examples of Design Factors for Steel Structures ASCE 7-05

	R	Ω_o	R_d	C_d
Special Moment Frame	8	3	2.67	5.5
Intermediate Moment Frame	4.5	3	1.50	4.0
Ordinary Moment Frame	3.5	3	1.17	3.0
Eccentric Braced Frame	8	2	4.00	4.0
Eccentric Braced Frame (Pinned)	7	2	3.50	4.0
Special Concentric Braced Frame	6	2	3.00	5.0
Ordinary Concentric Braced Frame	3.25	2	1.25	3.25
Not Detailed	3	3	1.00	3.0

Note: R_d is ductility demand ONLY IF Ω_o is achieved.

Examples of Design Factors for Reinforced Concrete Structures ASCE 7-05

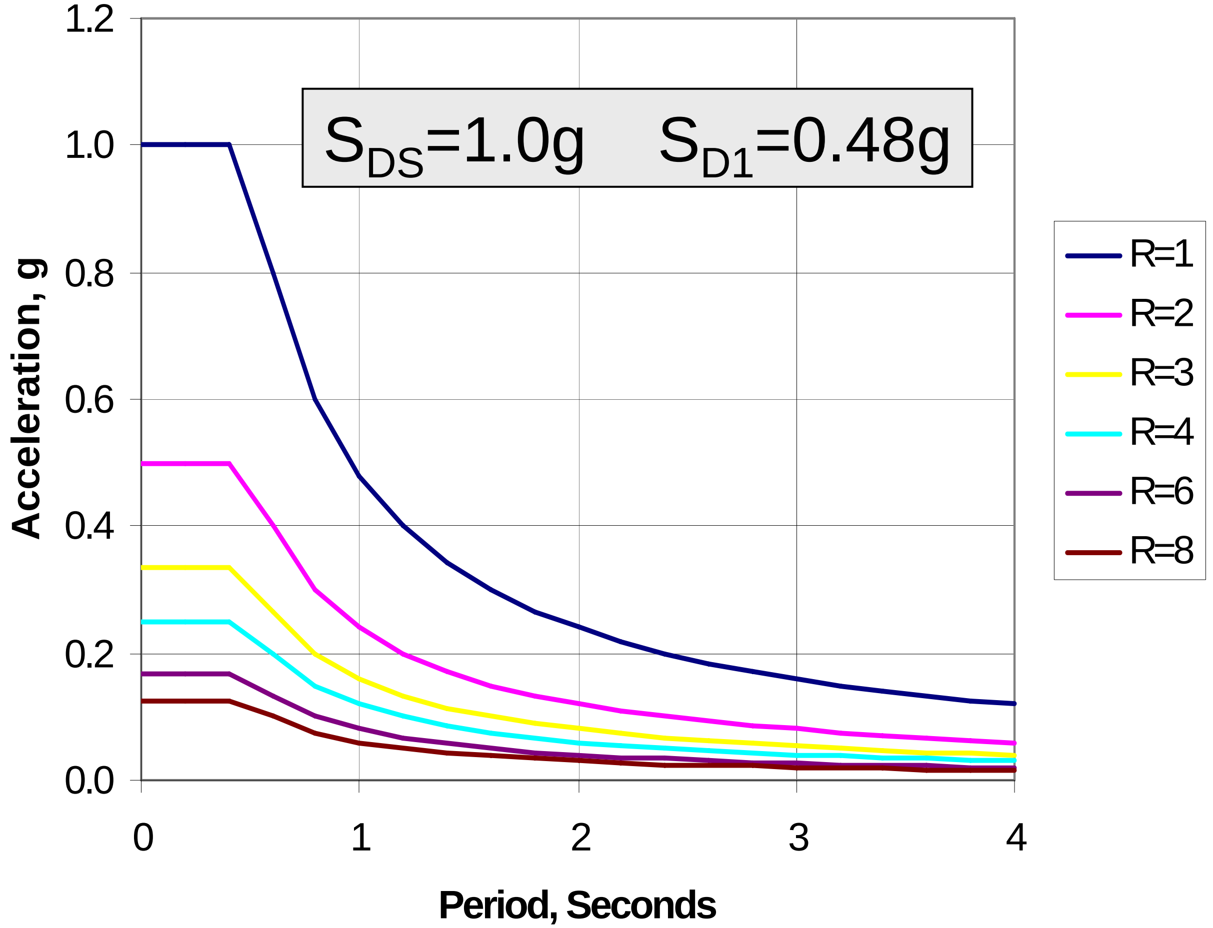
	R	Ω_o	R_d	C_d
Special Moment Frame	8	3	2.67	5.5
Intermediate Moment Frame	5	3	1.67	4.5
Ordinary Moment Frame	3	3	1.00	2.5
Special Reinforced Shear Wall	5	2.5	2.00	5.0
Ordinary Reinforced Shear Wall	4	2.5	1.60	4.0
Detailed Plain Concrete Wall	2	2.5	0.80	2.0
Ordinary Plain Concrete Wall	1.5	2.5	0.60	1.5

Note: R_d is Ductility Demand ONLY IF Ω_o is Achieved.

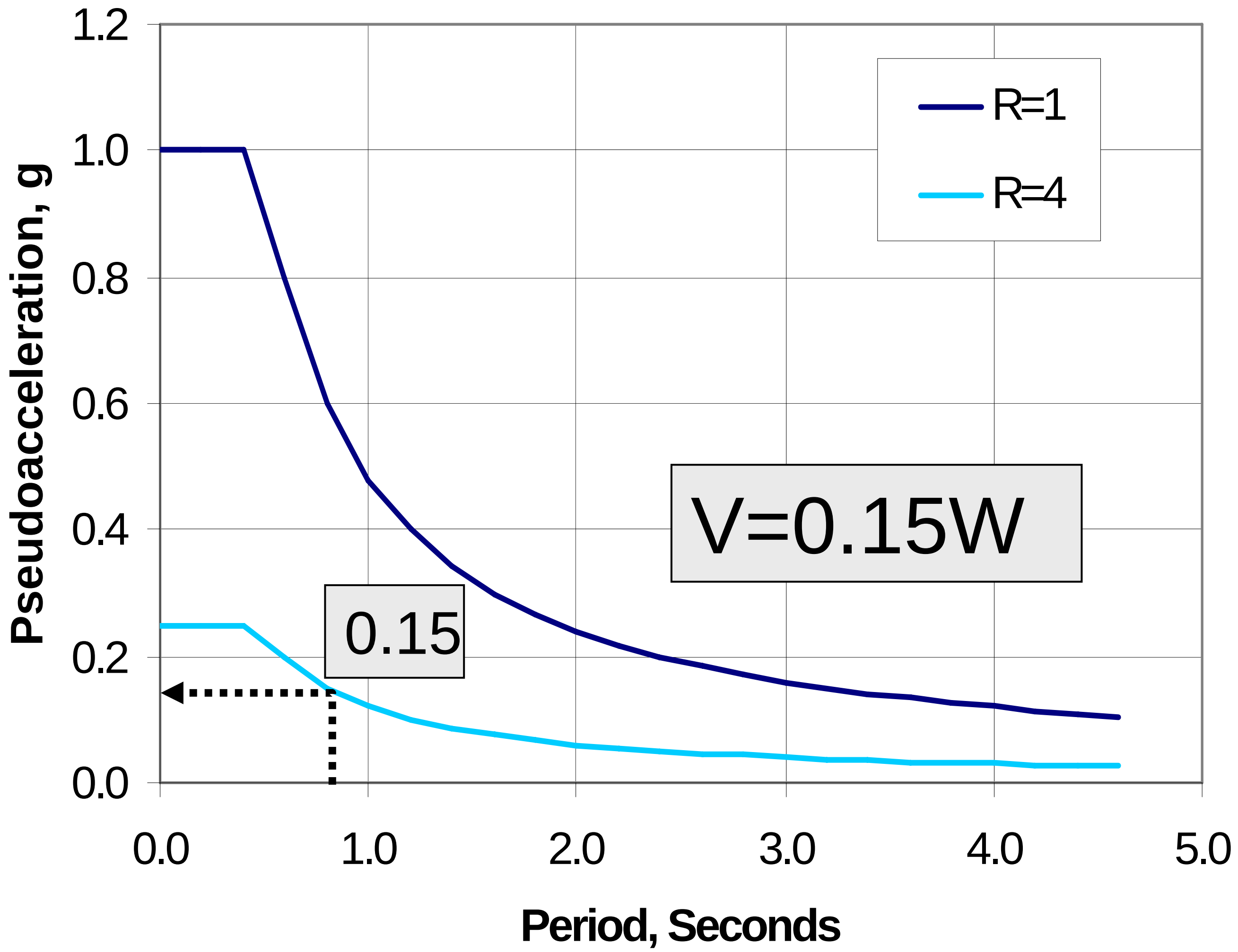
ASCE 7 Elastic Spectra as Adjusted for Ductility and Overstrength

$$C_S = \frac{S_{DS}}{R / I}$$

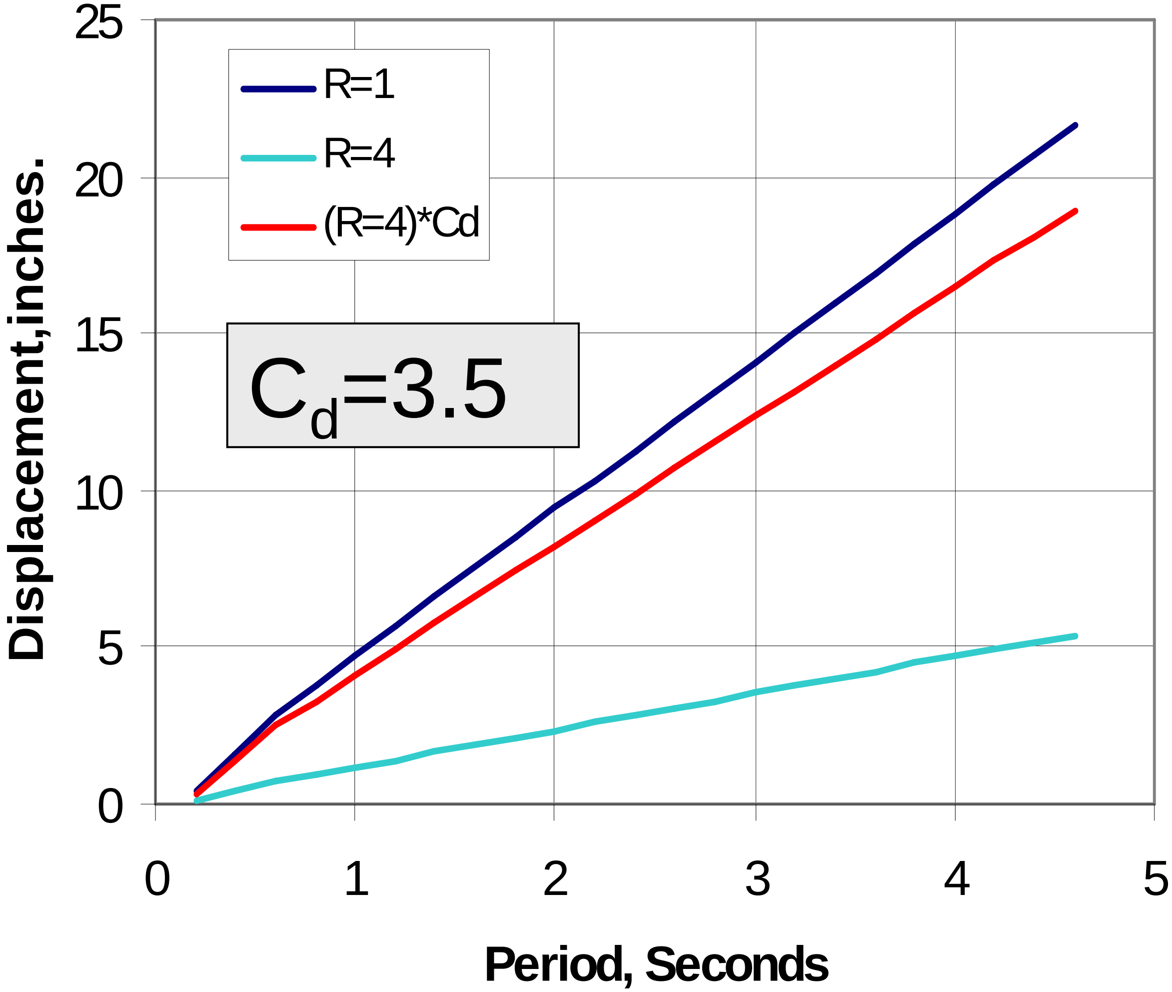
$$C_S = \frac{S_{D1}}{T(R / I)}$$



Using Modified ASCE 7 Spectrum to Determine Force Demand

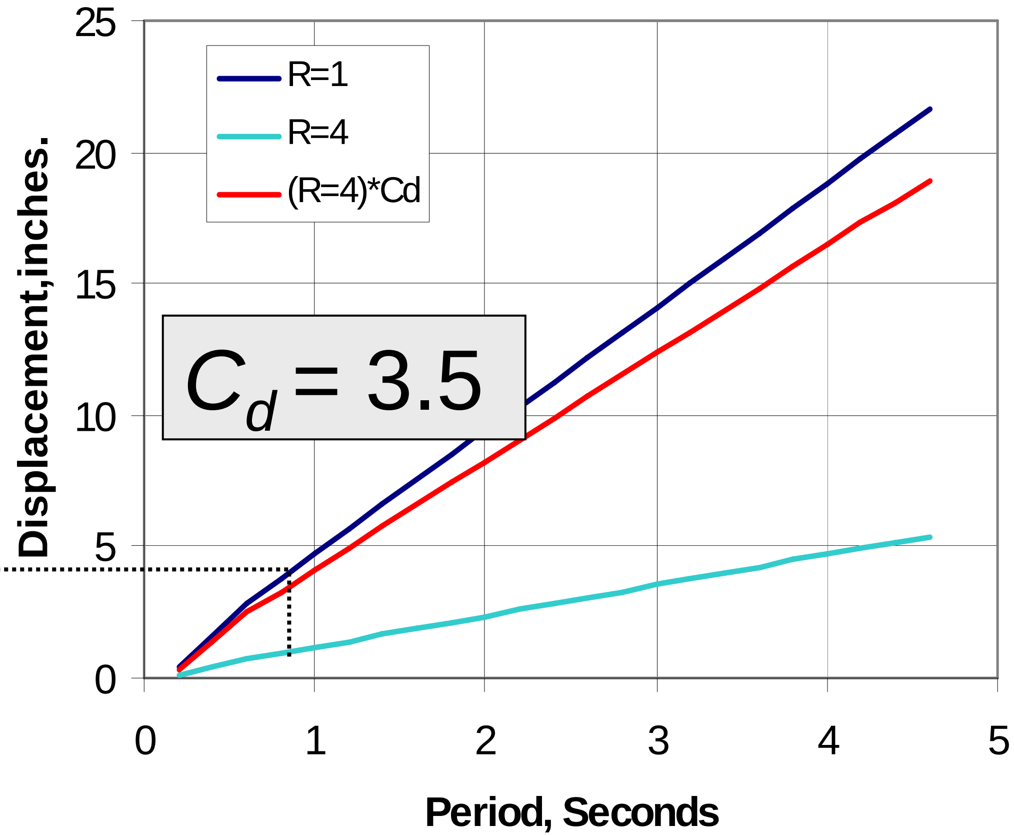


Using Modified ASCE-7 Spectrum to Determine Displacement Demand



Displacements must be multiplied by factor C_d because displacements based on reduced force would be too low

$$\Delta_{INELASTIC} = C_d \times \Delta_{REDUCEDELASTIC}$$

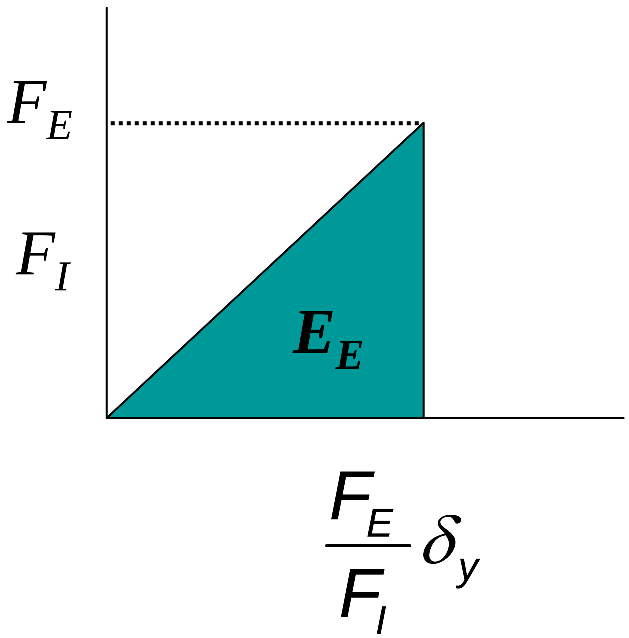


$$\Delta_{INELASTIC} = 3.65\ in.$$

“Equal displacement” approach may not be applicable at very low period values.

Equal Energy Concept
(Applicable at Low Periods)

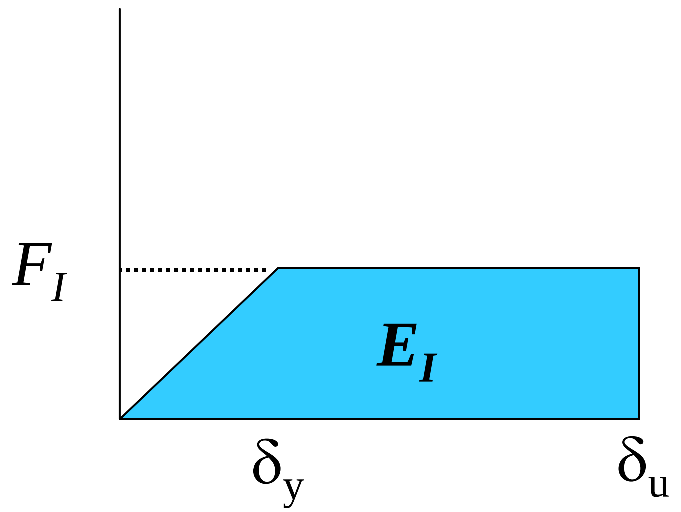
ELASTIC ENERGY



$$\boxed{E_E = 0.5F_E \frac{F_E}{F_I}\delta_y = 0.5\delta_y \frac{F_E^2}{F_I}}$$

Equal Energy Concept
(Applicable at Low Periods)

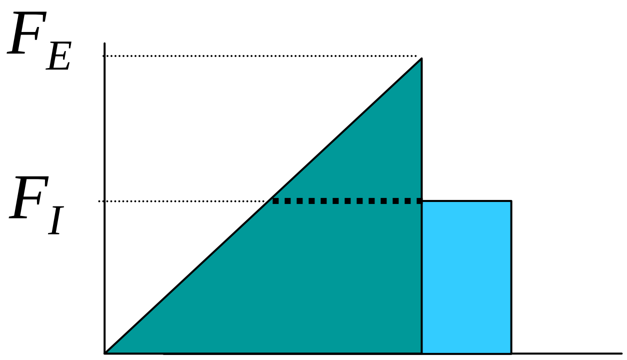
INELASTIC ENERGY



$$\boxed{E_I = F_I \delta_u - 0.5 F_I \delta_y = F_I \delta_y \left(\mu - 0.5 \right)}$$

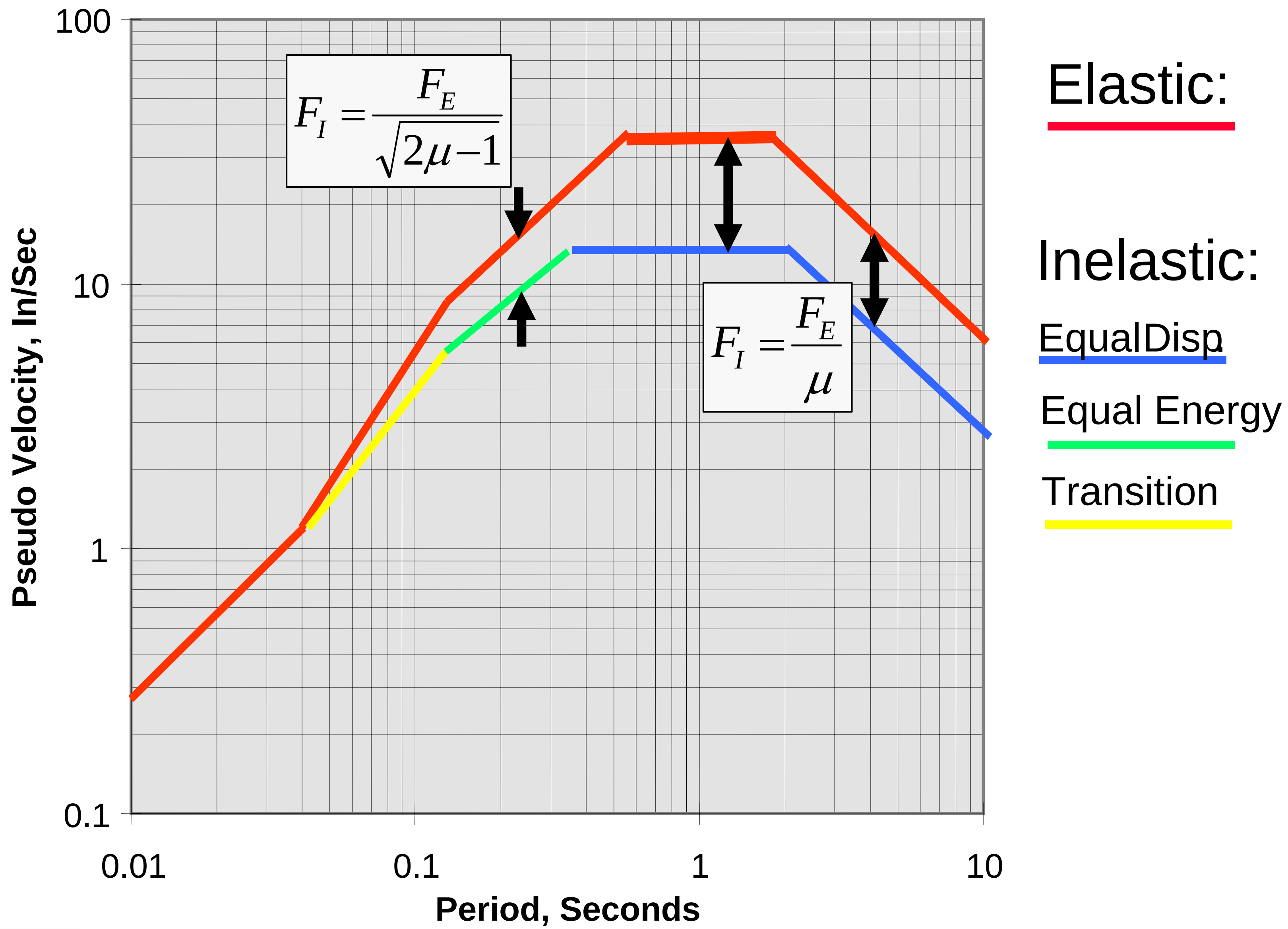
Equal Energy Concept
(Applicable at Low Periods)

Assuming $E_E = E_I$:



$$\boxed{\frac{F_E}{F_I} = \sqrt{2\mu - 1}}$$

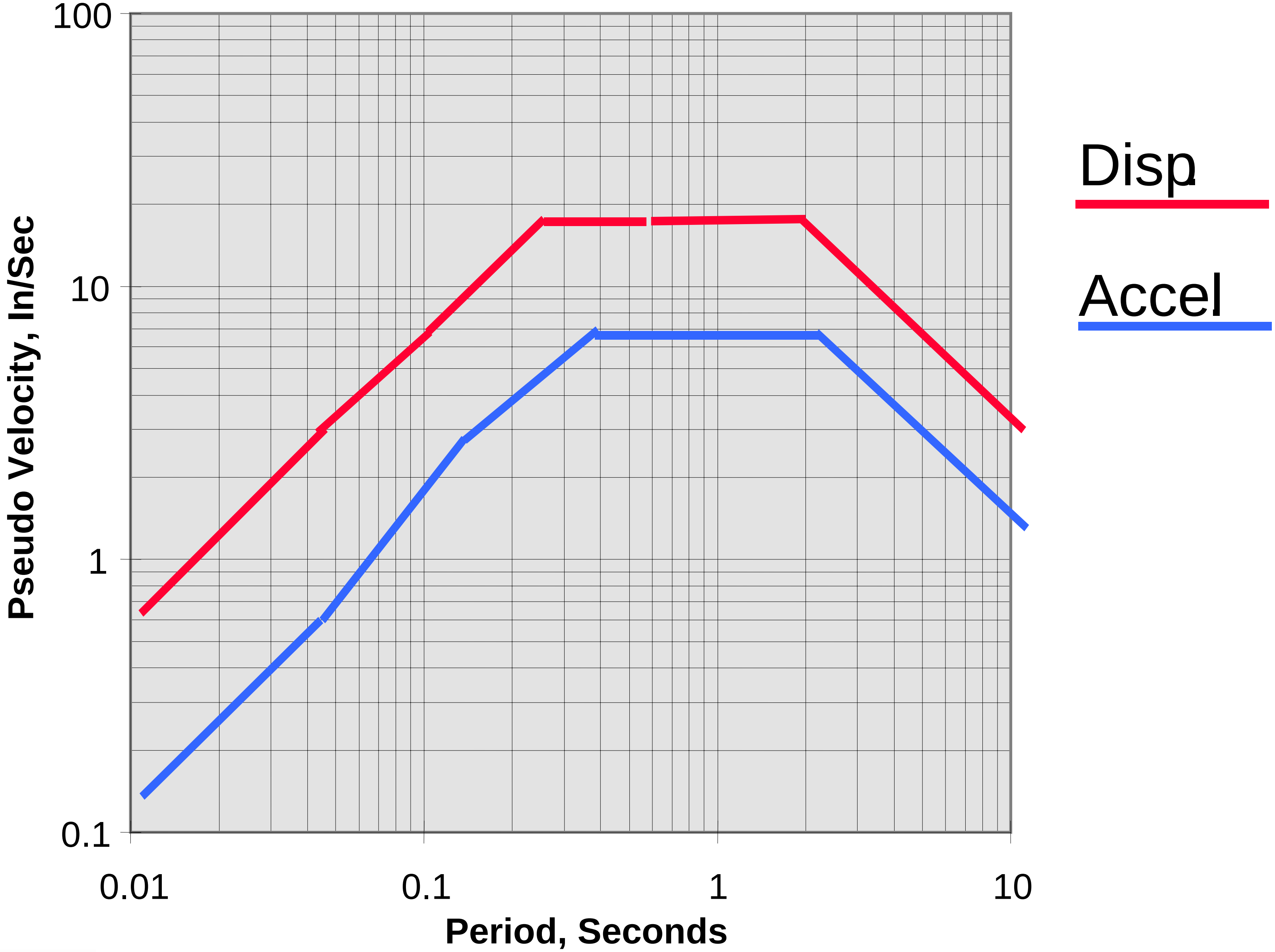
Newmark Inelastic Spectrum (for Psuedoacceleration)



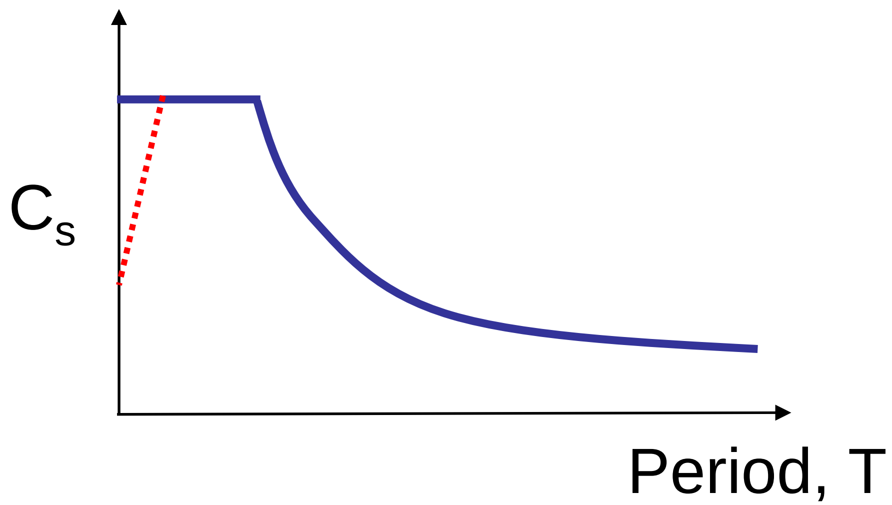
Newmark's
Inelastic Design Response Spectrum

To obtain inelastic displacement spectrum, multiply the spectrum shown in previous slide by μ (for all periods).

Inelastic Design Response Spectrum for Acceleration & Displacement



At very low periods, the ASCE 7 spectrum does not reduce to ground acceleration so this partially compensates for "error" in equal displacement assumption at low period values.



Note: FEMA 273 has explicit modifications for computing "target at low periods."

CONCEPTS OF SEISMIC-RESISTANT DESIGN

Steps in the Seismic Design of a Building

1. Develop concept (design philosophy)
2. Select structural system
3. Establish performance objectives
4. Estimate external seismic forces
5. Estimate internal seismic forces (linear analysis)
6. Proportion components
7. Evaluate performance (linear or nonlinear analysis)
8. Final detailing
9. Quality assurance

Seismic Design Practice in the United States

- Seismic requirements provide *minimum standards* for use in building design to maintain public safety in an extreme earthquake.
- Seismic requirements *safeguard against major failures and loss of life* – they DO NOT necessarily limit damage, maintain function, or provide for easy repair.
- Design forces are based on the assumption that a significant amount of *inelastic behavior* will take place in the structure during a design earthquake.

Seismic Design Practice in the United States continued

- For reasons of economy and affordability, the design forces are much lower than those that would be required if the structure were to remain elastic.
- In contrast, wind-resistant structures are designed to remain elastic under factored forces.
- Specified code requirements are intended to provide for the necessary inelastic seismic behavior.
- In nearly all buildings designed today, survival in large earthquakes depends directly on the ability of their framing systems to dissipate energy hysteretically while undergoing (relatively) large inelastic deformations.

The Difference Between Wind-Resistant Design and Earthquake-Resistant Design

For Wind:

Excitation is an applied pressure or **force** on the facade.

Loading is dynamic but response is nearly **static** for most structures.

Structure deforms due to applied force.

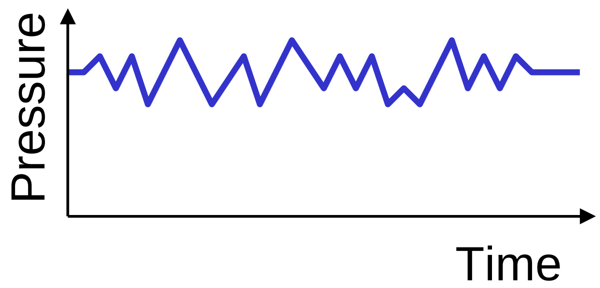
Deformations are **monotonic (unidirectional).**

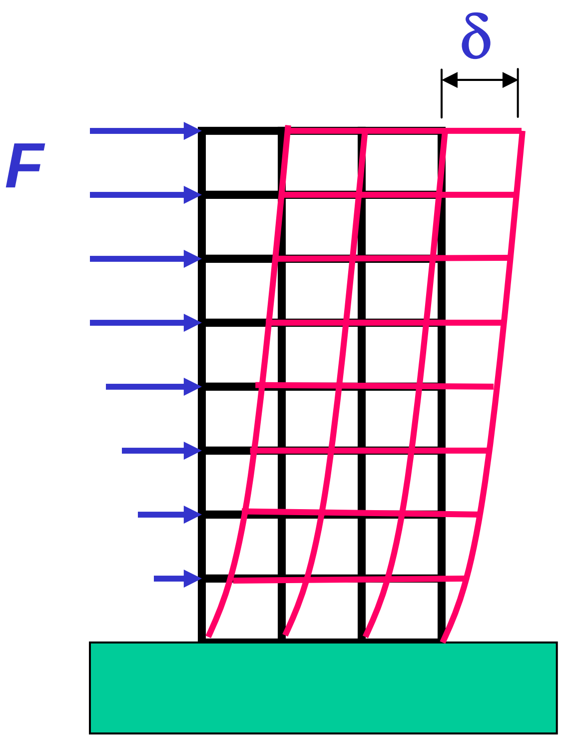
Structure is designed to respond **elastically** under factored loads.

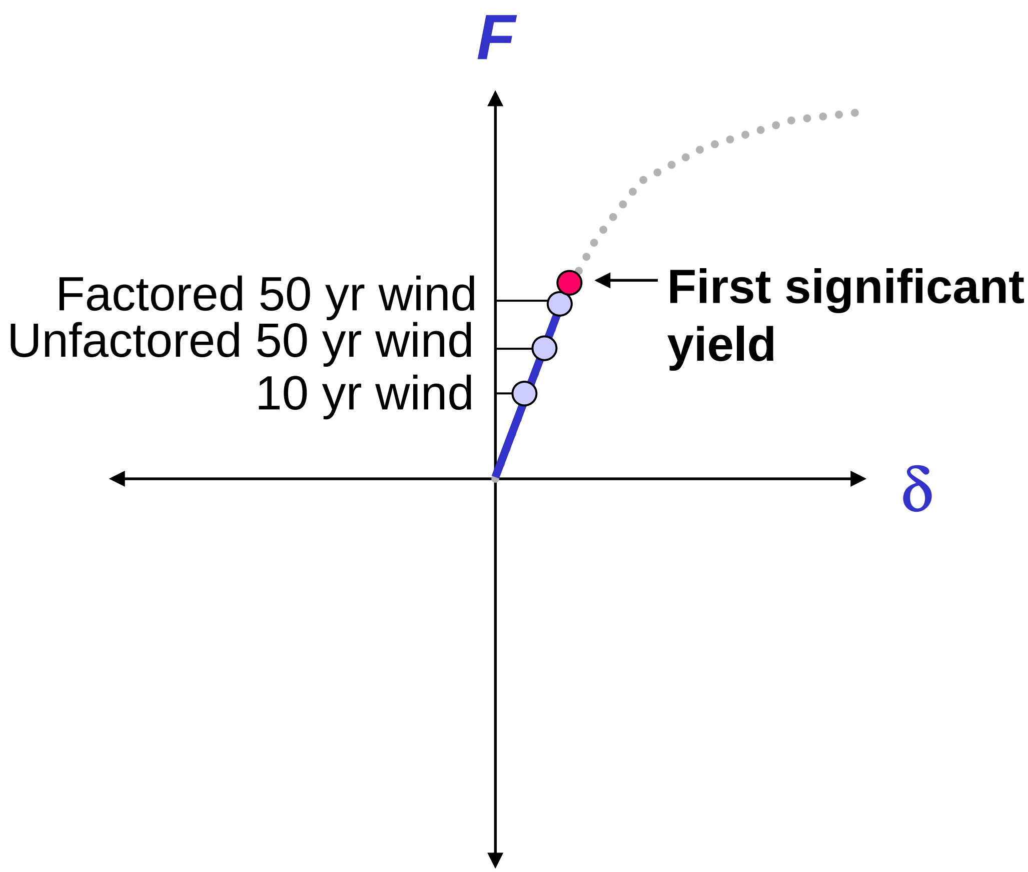
The controlling life safety limit state is **strength**.

Enough strength is provided to resist forces elastically.

Behavior Under Wind Excitation







The Difference Between Wind-Resistant Design and Earthquake-Resistant Design

For Earthquake:

Excitation is an applied **displacement** at the base.

Loading and response are truly **dynamic.**

Structural system deforms as a result of **inertial forces.**

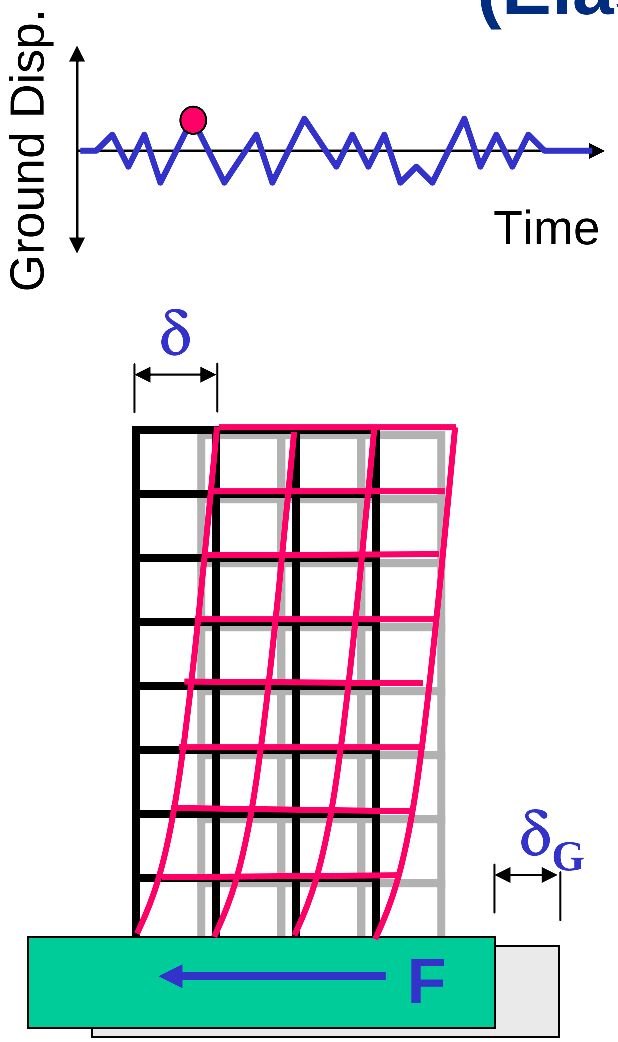
Deformations are fully **reversed.**

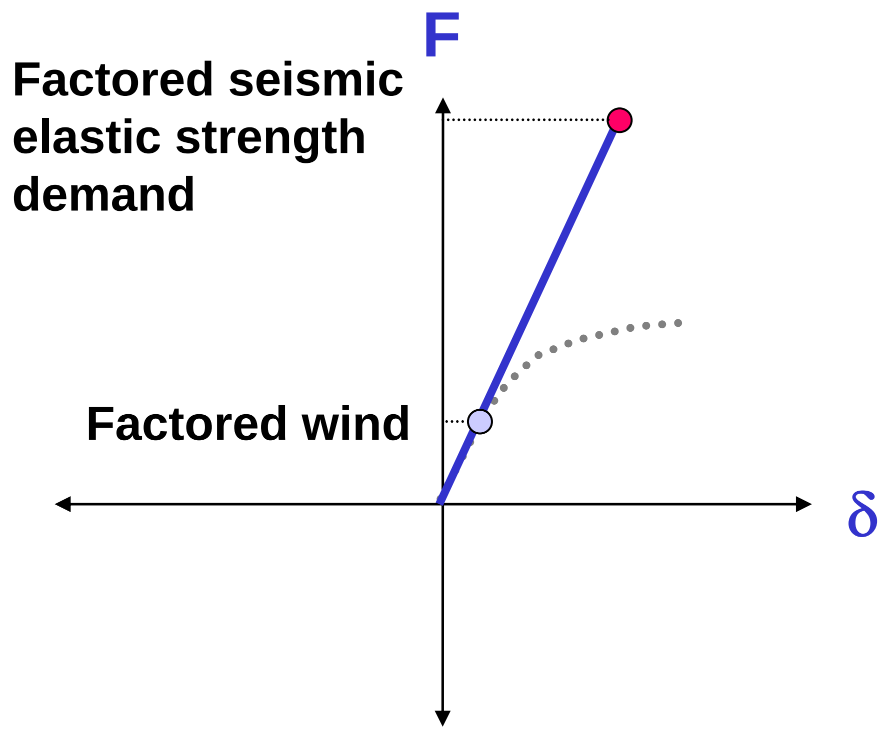
Structure is designed to respond **inelastically** under factored loads.

Controlling life safety limit state is **deformability.**

Enough strength is provided to ensure that inelastic deformation demands do not exceed deformation capacity.

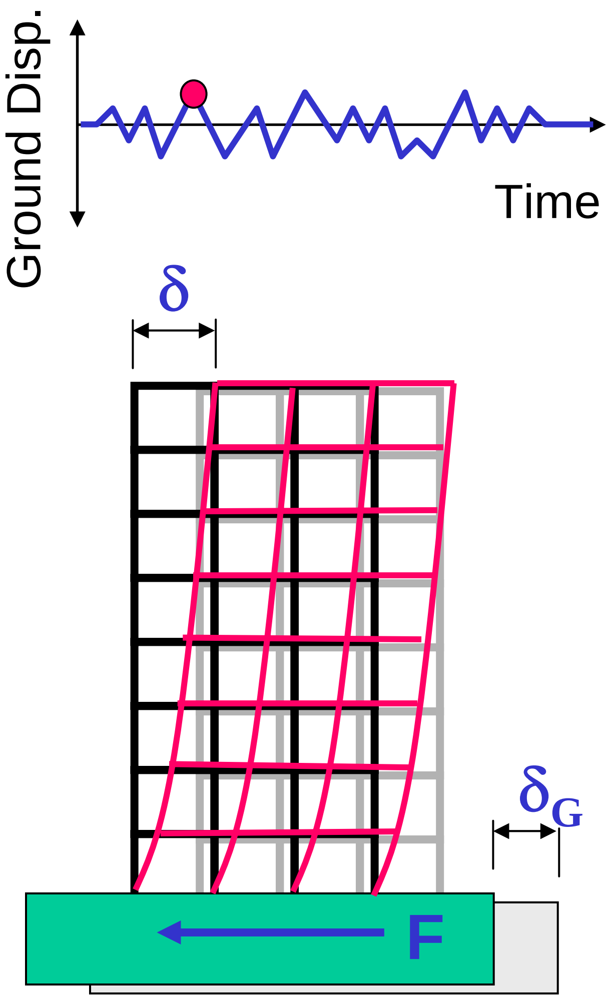
Behavior Under Seismic Excitation (Elastic Response)

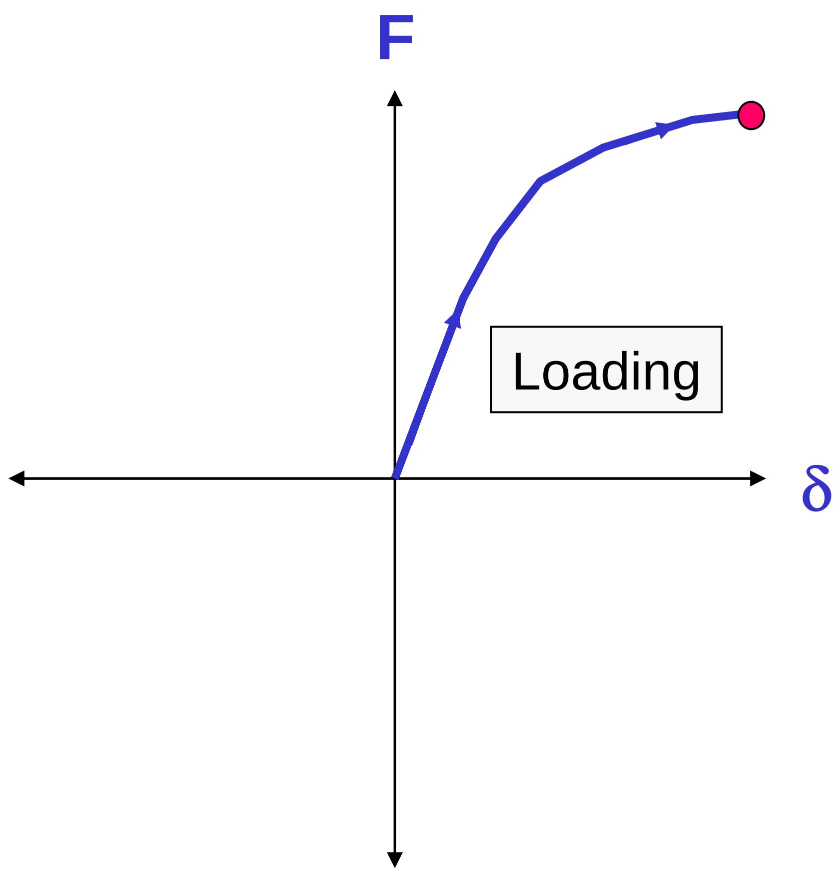




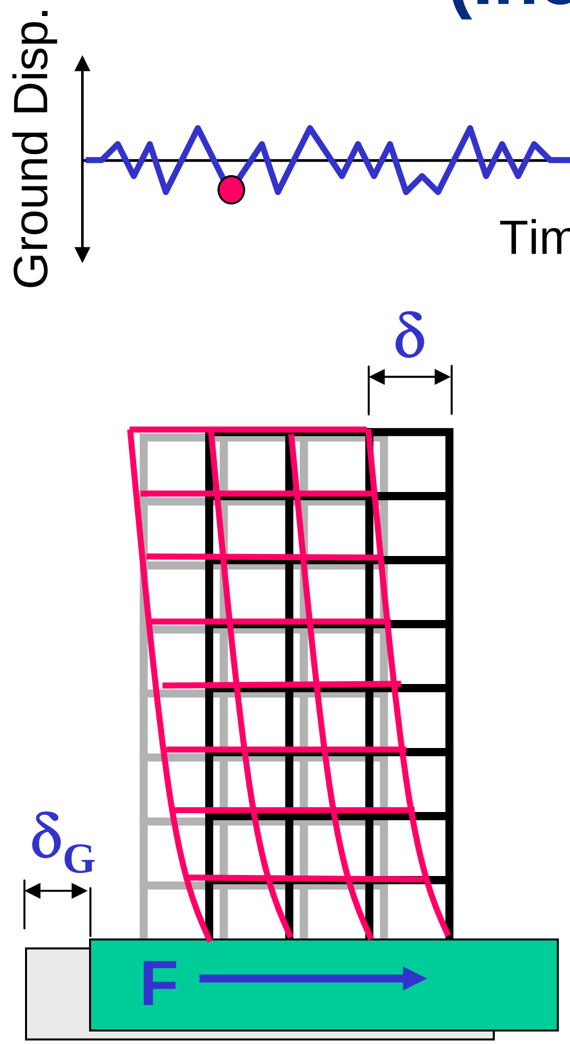
In general, it is not economically feasible to design structures to respond elastically to earthquake ground motions.

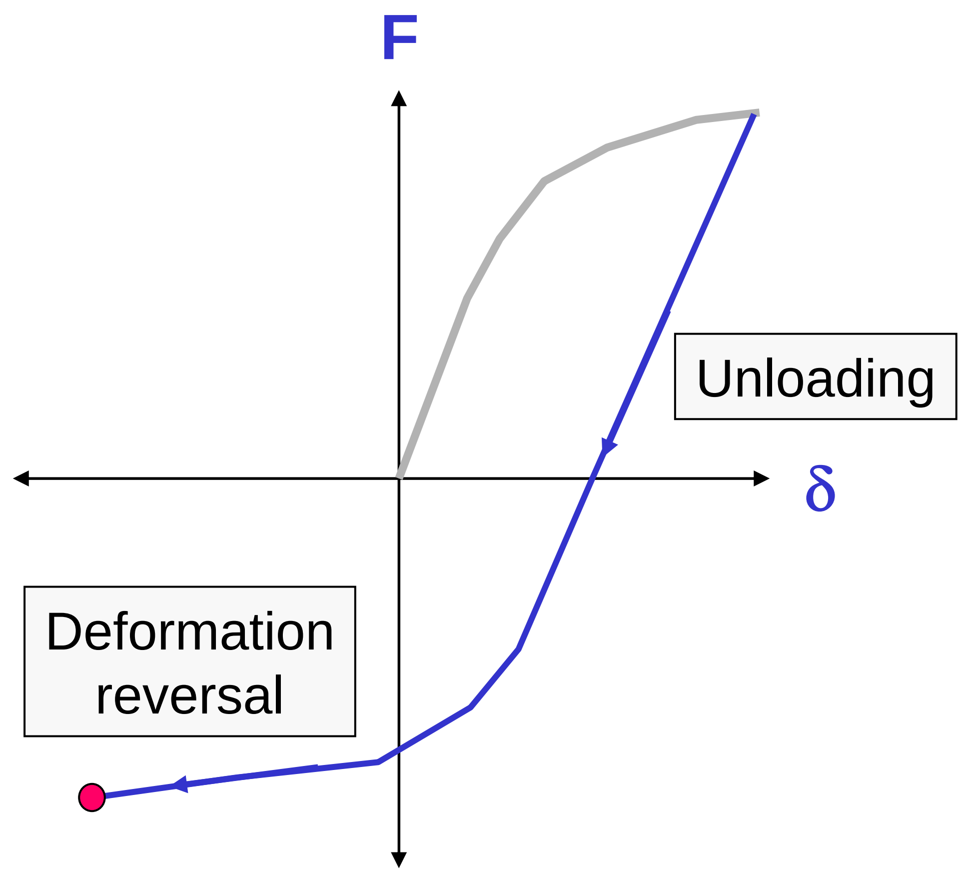
Behavior Under Seismic Excitation (Inelastic Response)



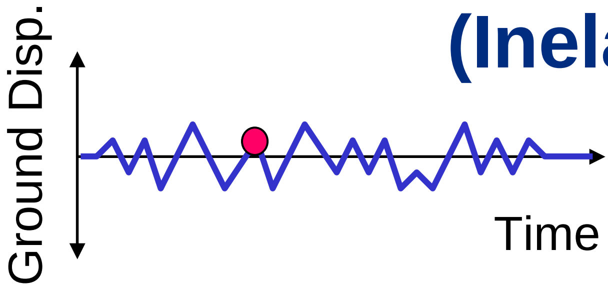


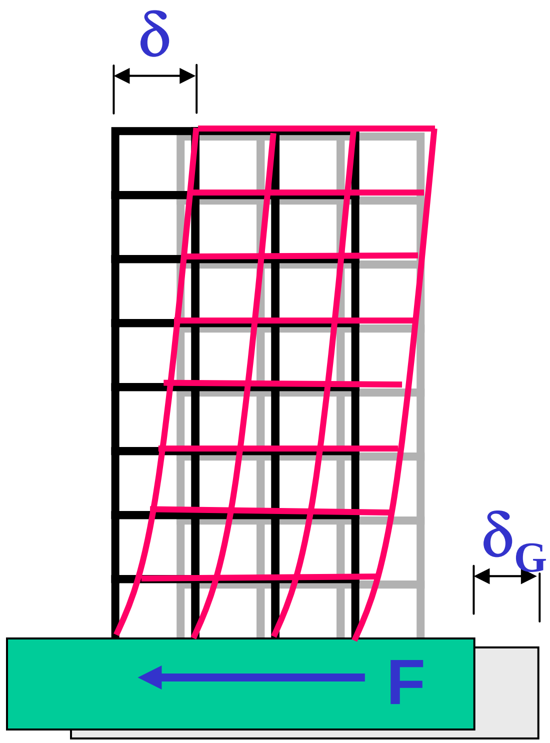
Behavior Under Seismic Excitation (Inelastic Response)

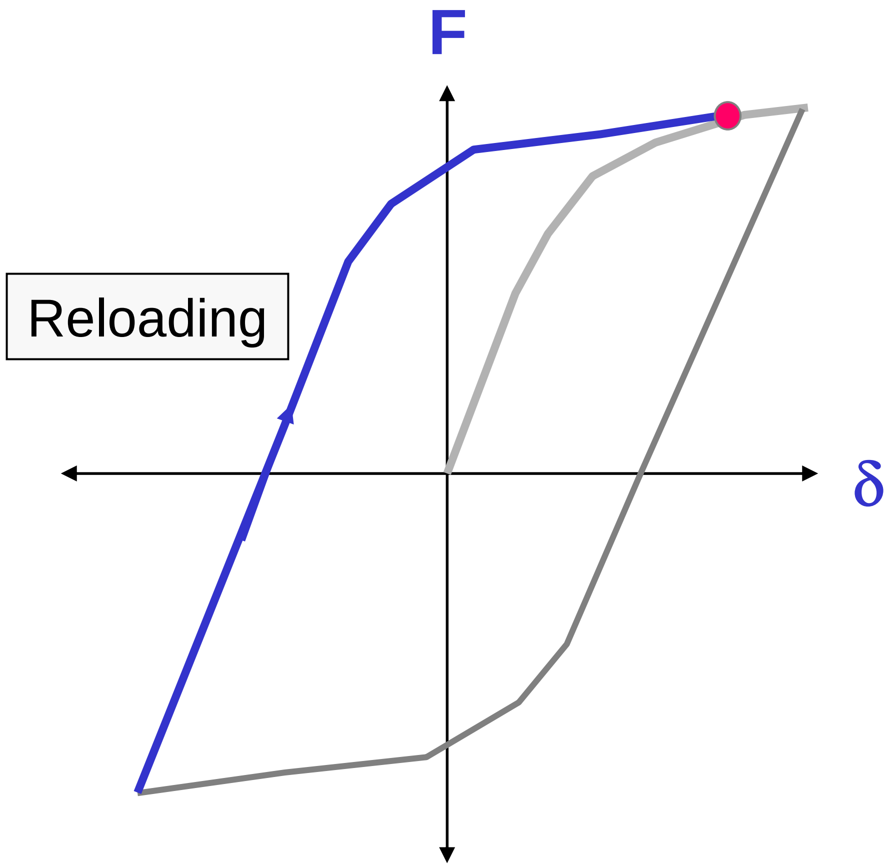




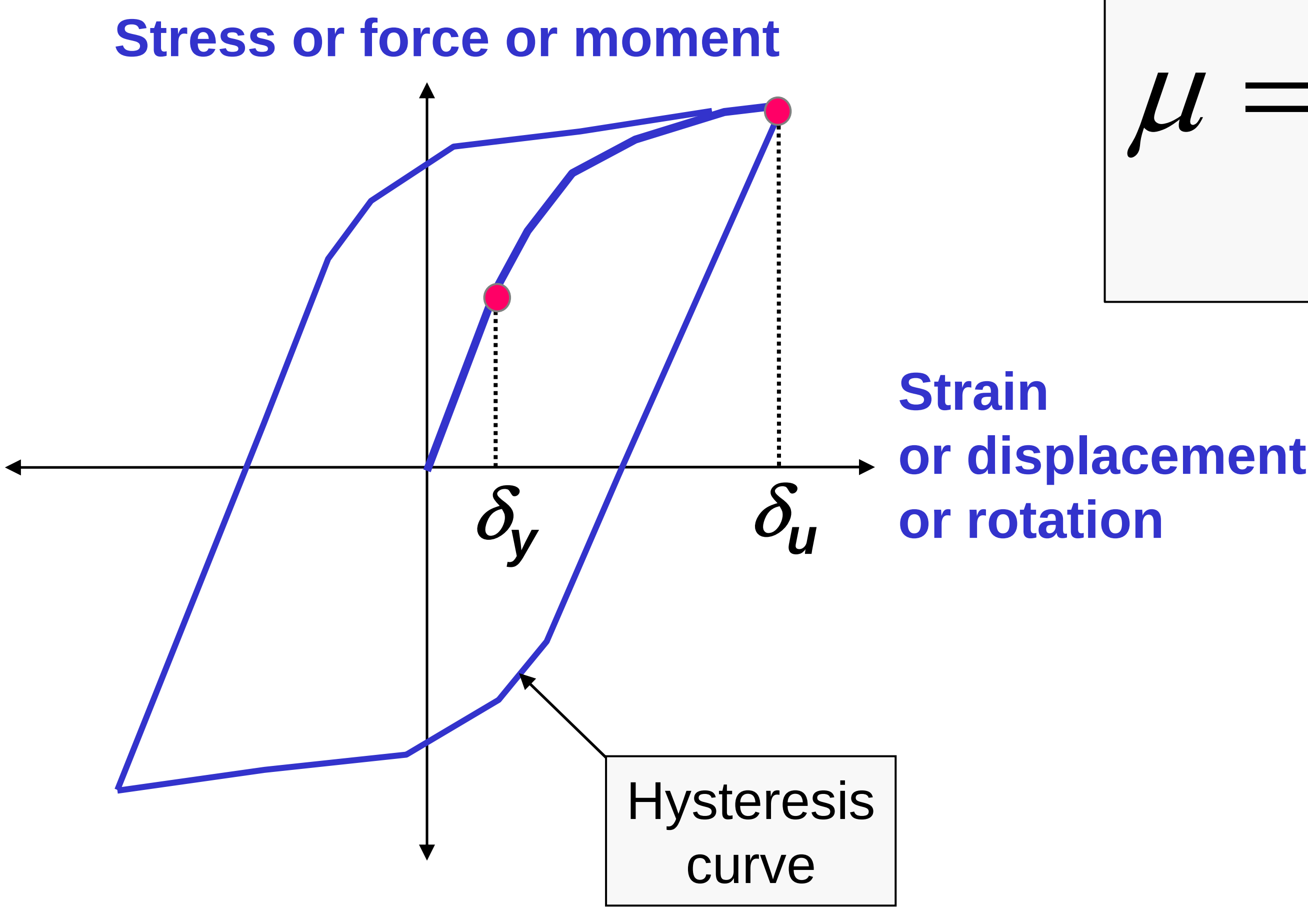
Behavior Under Seismic Excitation (Inelastic Response)







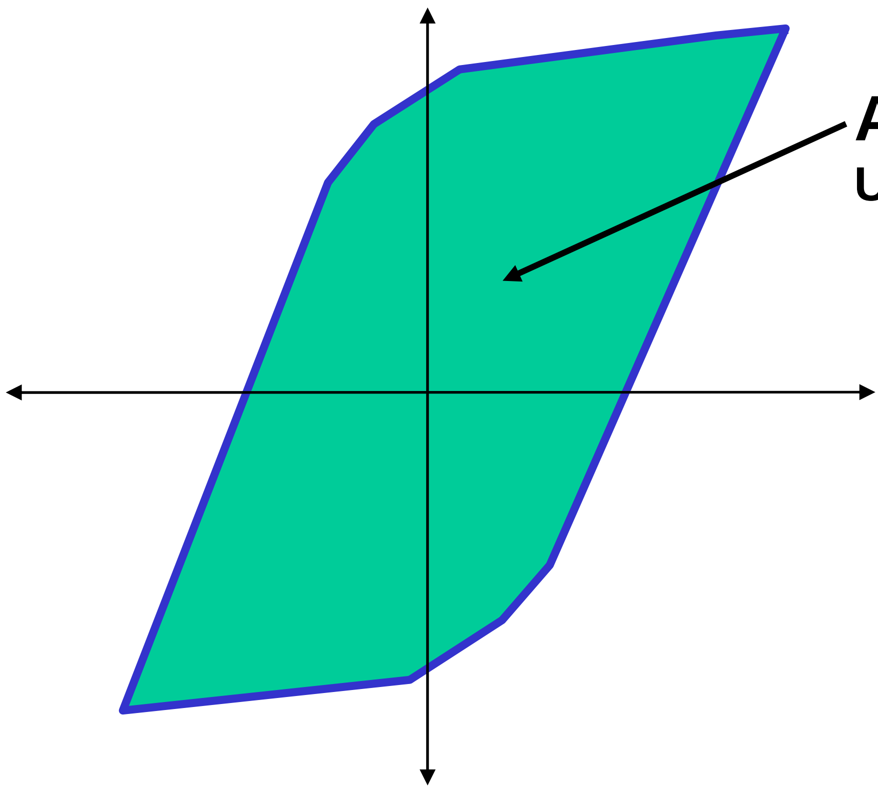
Definition of Ductility, μ



$$\mu = \frac{\delta_u}{\delta_y}$$

Definition of Energy Dissipation, Θ

Stress or force or moment



Area = Θ = energy dissipated
Units = force x displacement

Strain
or displacement
or rotation

Basic Earthquake Engineering Performance Objective (Theoretical)

An adequate design is accomplished when a structure is dimensioned and detailed in such a way that the local ductility demands (energy dissipation demands) are smaller than their corresponding capacities.

$$\boxed{\mu_{Demand} \leq \mu_{Supply}}$$

$$\boxed{\Theta_{Demand} \leq \Theta_{Supplied}}$$

Concept of Controlled Damage

$$\text{Seismic input energy} = E_S + E_K + E_D + E_H$$

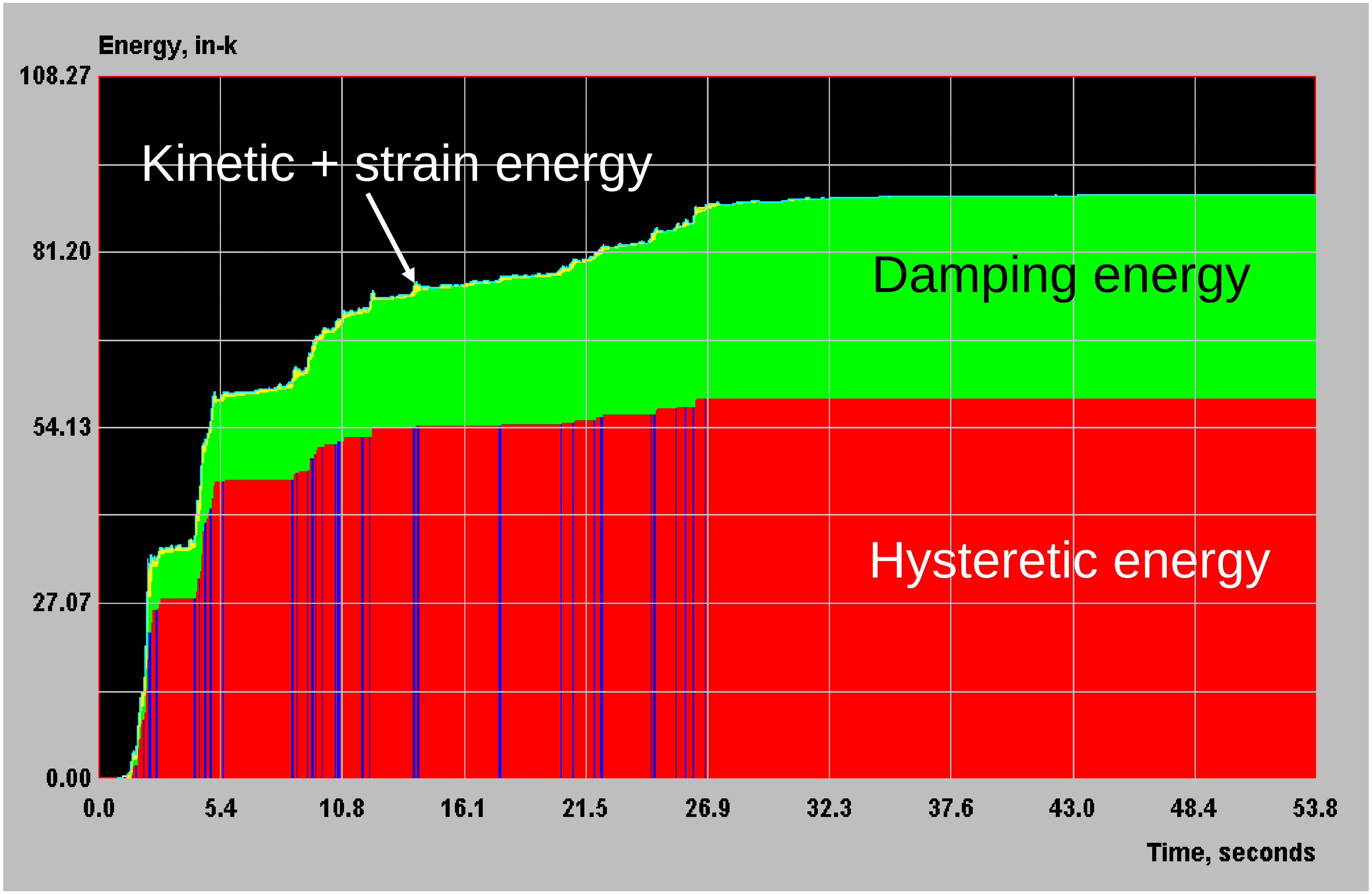
E_S = Elastic strain energy

E_K = Kinetic energy

E_D = Viscous damping energy

E_H = Hysteretic energy

Typical Energy Time History



$$Damage = \frac{\delta_{\max}}{\delta_{ult}} + 0.15\frac{E_H}{F_y \delta_{ult}}$$

- Yielding is necessary for affordable design.
- Yielding causes hysteretic energy dissipation.
- Hysteretic energy dissipation causes damage.

Therefore, **damage is necessary for affordable design**

The Role of Design

The role of "design" is to estimate the structural strength required to limit the ductility demand to the available supply *and to provide the desired engineering economy*.

Design Philosophies

New Buildings (FEMA 450, IBC 2003, ASCE 7-05)

- Force-based approach
- Single event (2/3 of 2% in 50 year earthquake)
- Single performance objective (life safety)
- Simple global acceptance criteria (drift)
- Linear analysis

Existing Buildings (ATC40, FEMA 273)

- Displacement-based approach
- Multiple events
- Multiple performance objectives
- Detailed local and global acceptance criteria
- Nonlinear analysis

Building Performance Levels and Ranges

Structural

(1) IMMEDIATE OCCUPANCY

(2) Damage Control Range

(3) LIFE SAFETY

(4) Limited Safety Range

(5) COLLAPSE PREVENTION

Nonstructural

(A) OPERATIONAL

(B) IMMEDIATE OCCUPANCY

(C) LIFE SAFETY

(D) HAZARDS REDUCED

Combined

(1-A) OPERATIONAL

(1-B) IMMEDIATE OCCUPANCY

(3-C) LIFE SAFETY

(5-D) HAZARDS REDUCED

Earthquake Hazard Levels (FEMA 273)

Probability	MRI	Frequency
50%-50 year	72 years	Frequent
20%-50 year	225 years	Occasional
10%-50 year **(BSE-1)**	474 years	Rare
2%-50 year* **(BSE-2)**	2475 years	Very rare

*2003 *NEHRP Recommended Provisions* maximum considered earthquake.

Performance Objectives (FEMA 273)

Building Performance Level + EQ Design Level = *Performance Objective*

Performance Level

		Immediate Occ.	Operational	Life Safety	Collapse Prev.
Earthquake	72 year	a	b	c	d
	225 year	e	f	g	h
	474 year	i	j	**k**	l
	2475 year	m	n	o	**p**

"Basic Safety Objective" is design for **k** and **p**.

Performance Objectives (FEMA 273)
Enhanced Safety Objectives

Performance Level

Earthquake		Immediate Occ.	Operational	Life Safety	Collapse Prev.
	72 year	a	b	c	d
	225 year	e	f	g	h
	474 year	i	j	k	l
	2475 year	m	n	o	p
	5000 year				x

"Enhanced Safety Objective" is designed for **j** , **o** , and **x**.

Steps in the Seismic Design of a Building

1. Develop Concept
2. **Select Structural System**
3. Establish Performance Objectives
4. Estimate External Seismic Forces
5. Estimate Internal Seismic Forces (Linear Analysis)
6. Proportion Components
7. Evaluate Performance (Linear or Nonlinear Analysis)
8. Final Detailing
9. Quality Assurance

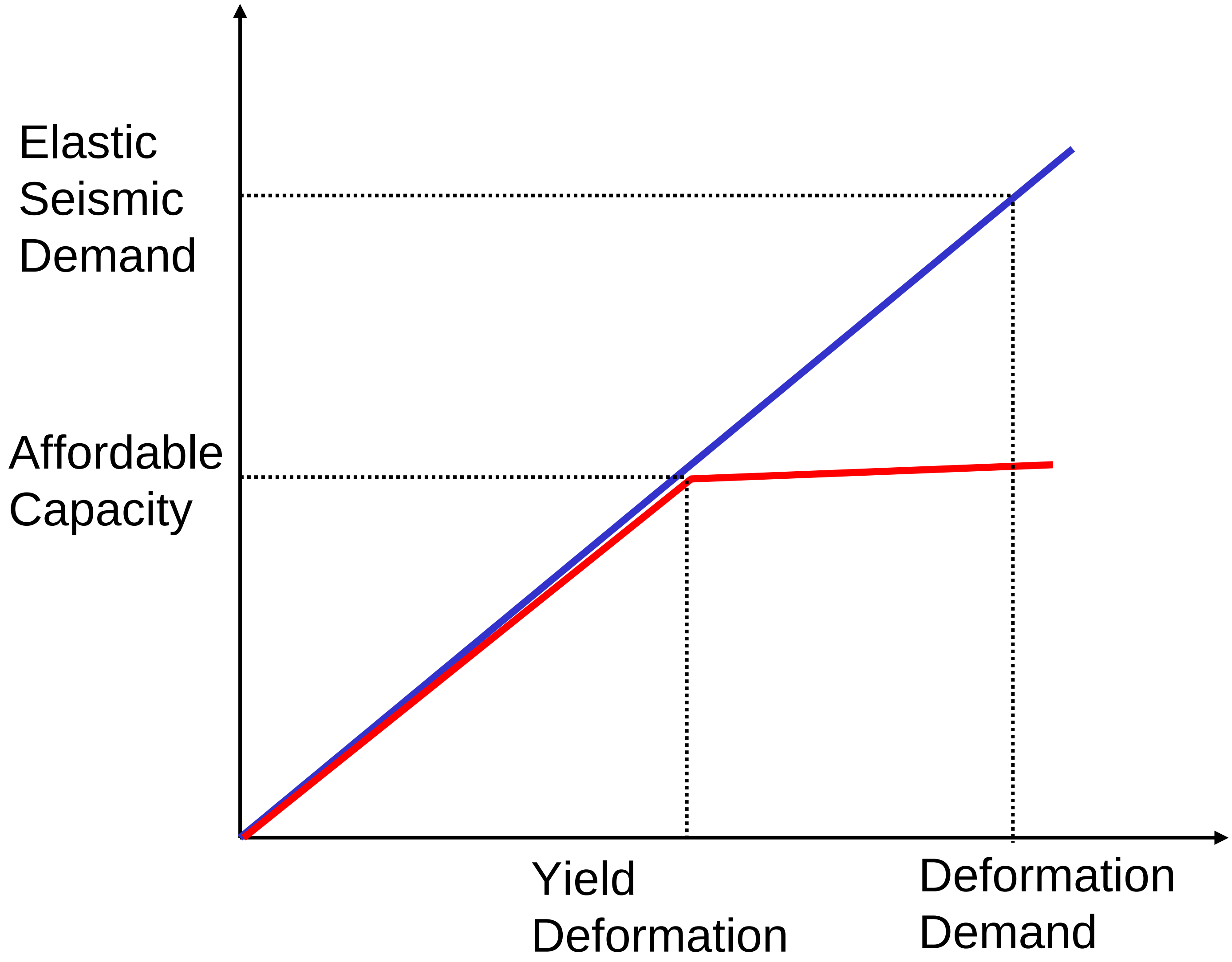
Definitions

Inherent Capacity: That capacity provided by the gravity system or by gravity plus wind.

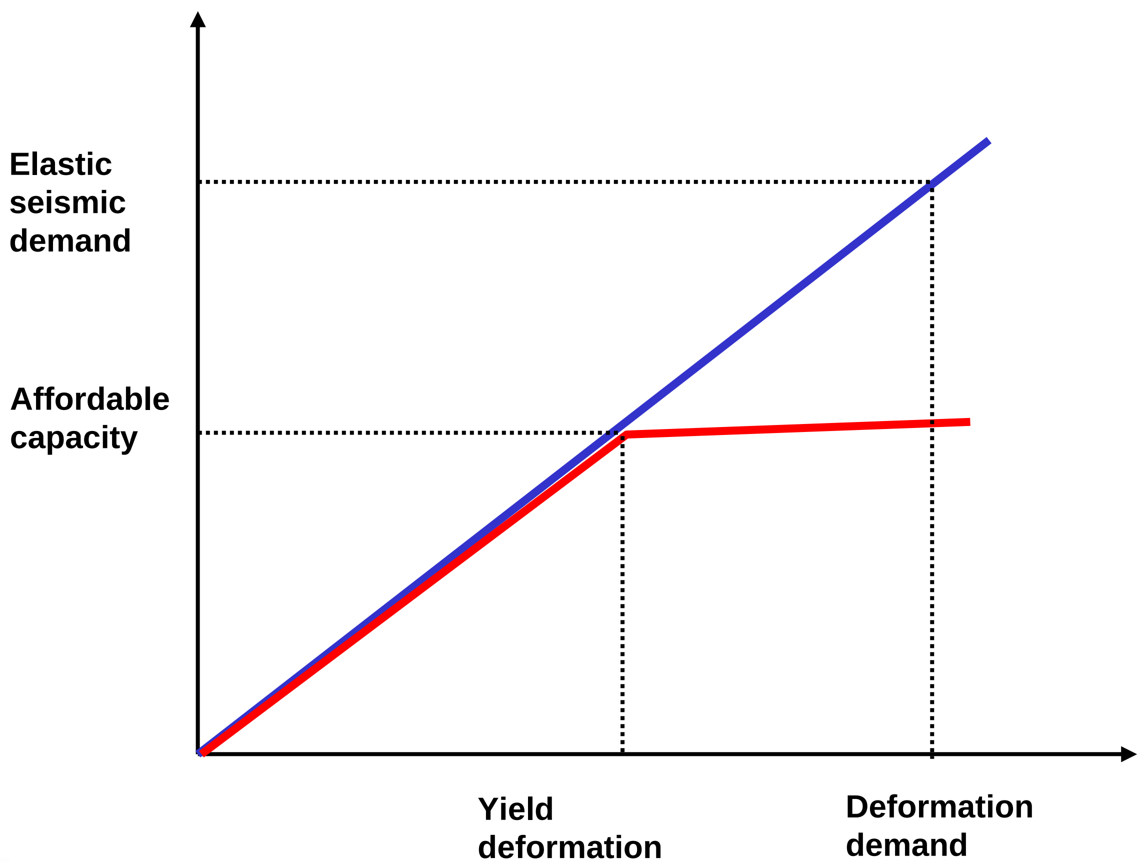
Affordable Capacity: The capacity governed by reasonable (ordinary) building costs in the geographic area of interest.

Seismic Premium: The ratio of the (reduced) seismic strength demand to the inherent capacity.

The Role of Design



$$\text{Ductility demand} = \frac{\text{Elastic seismic demand}}{\text{Affordable capacity}}$$



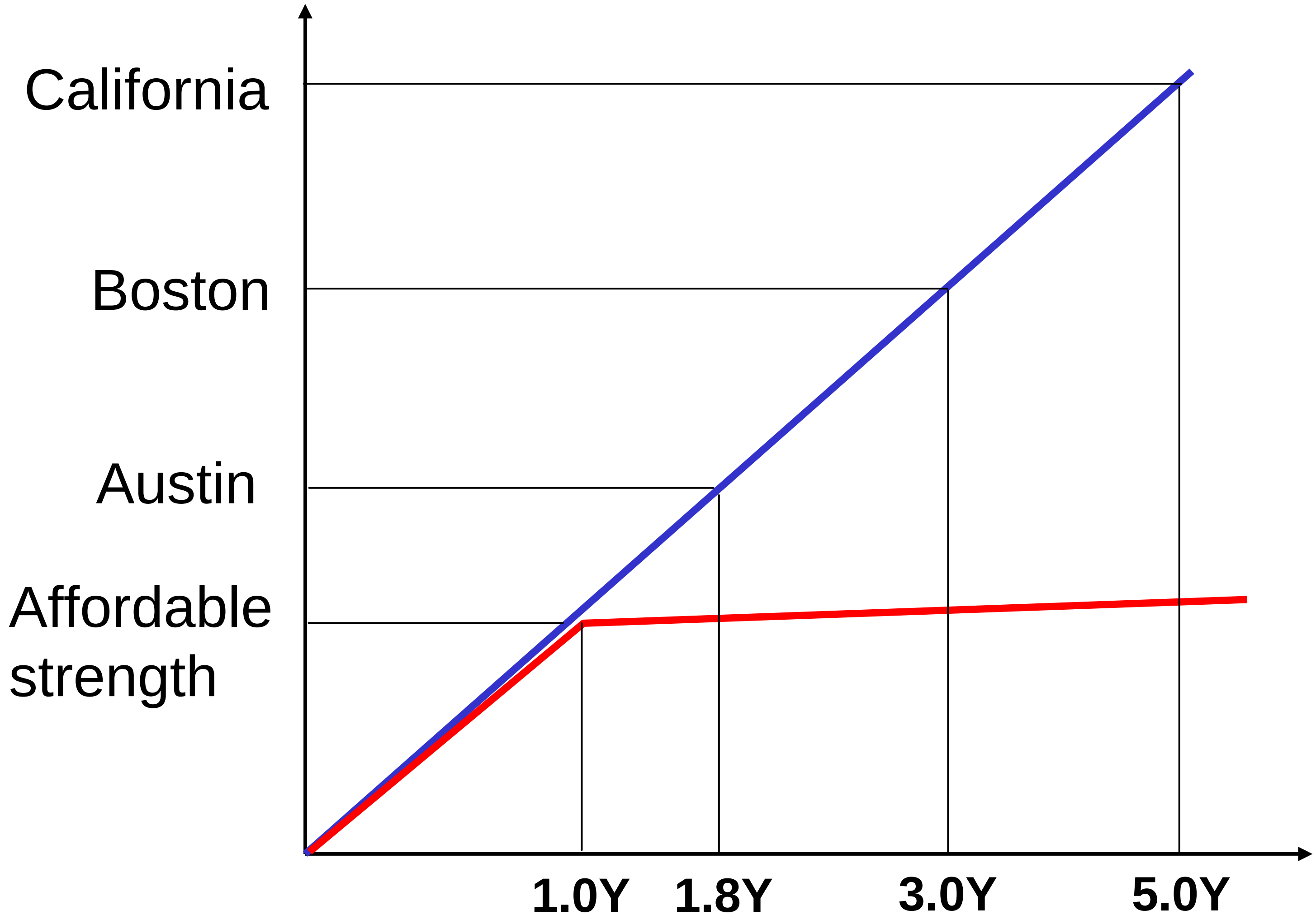
The Role of Design

If “affordable capacity” is relatively constant, then ductility demand is primarily a function of elastic seismic demand.

Because elastic seismic demand is a function of local seismicity, ductility demand is directly proportional to local seismicity.

Hence, California, which has higher seismicity than, for example, Austin, has a higher inherent ductility demand than does Austin.

Elastic demand



Limitation

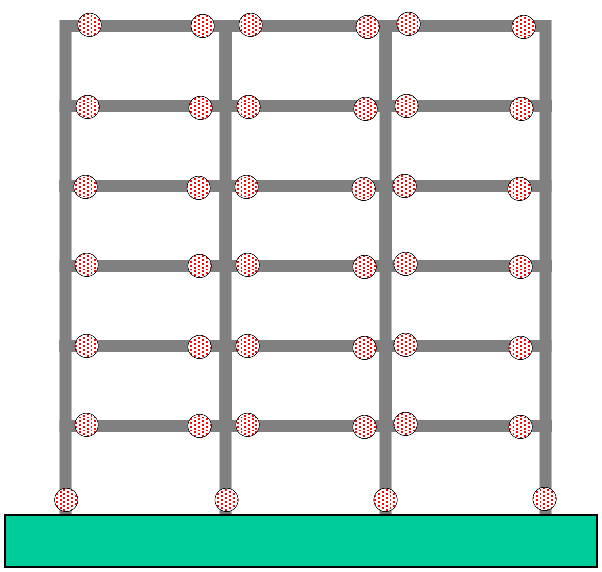
The ductility demand cannot exceed the ductility supply.

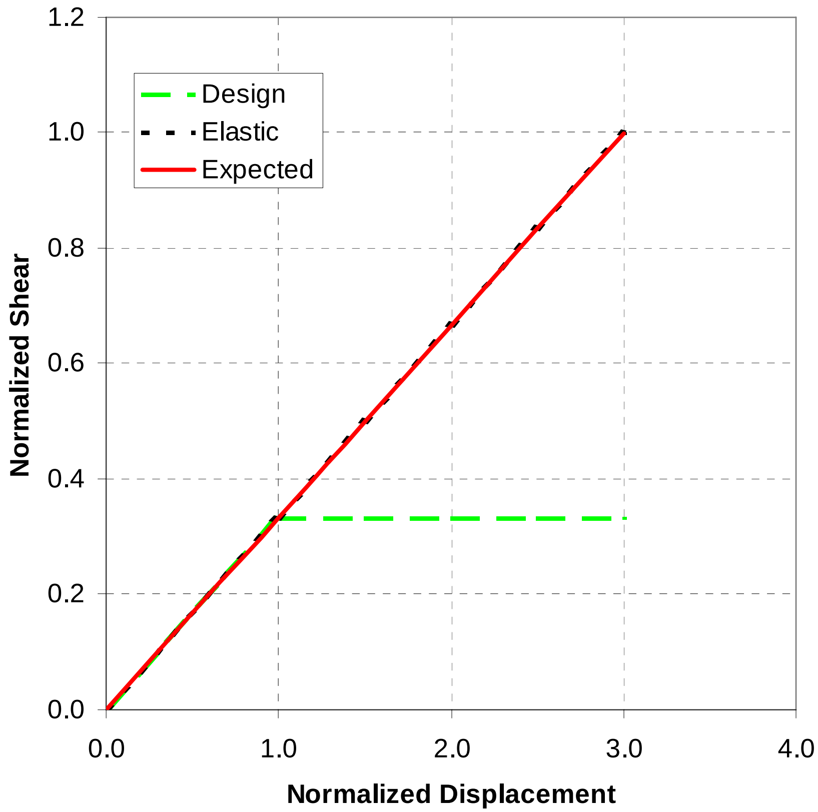
Moment Frame Ductility Supply	
Ordinary detailing	1.5
Intermediate detailing	2.5
Special detailing	5.0

In California, the high seismicity dictates a high ductility demand (typically > 3); hence, only moment frames with special detailing may be used.

Ordinary Concrete Moment Frame

No special detailing required



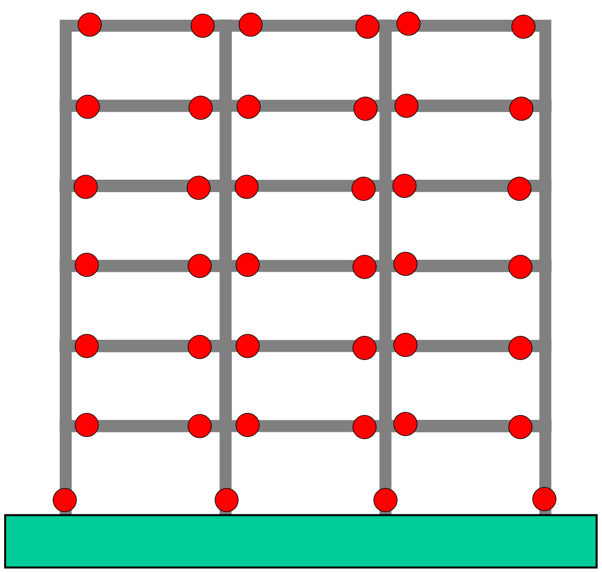


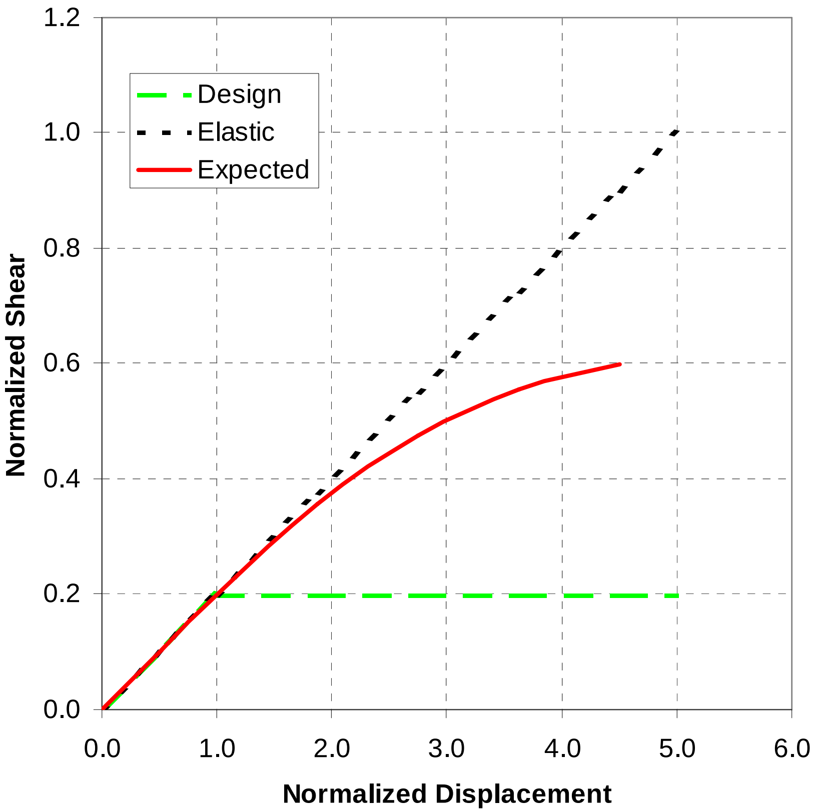
Advantages:
Architectural simplicity, low detailing cost
Disadvantages:
Higher base shear, highly restricted use

Intermediate Concrete Moment Frame

DETAILING REQUIREMENTS:

- Continuous top and bottom reinforcement
- Special requirements for shear strength
- Special detailing in critical regions



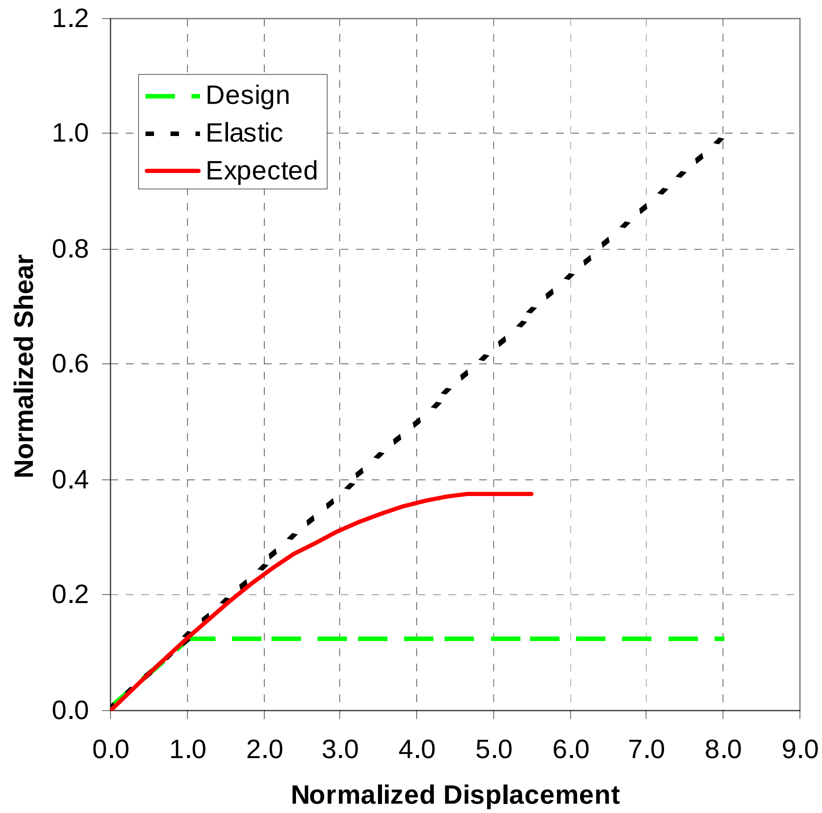


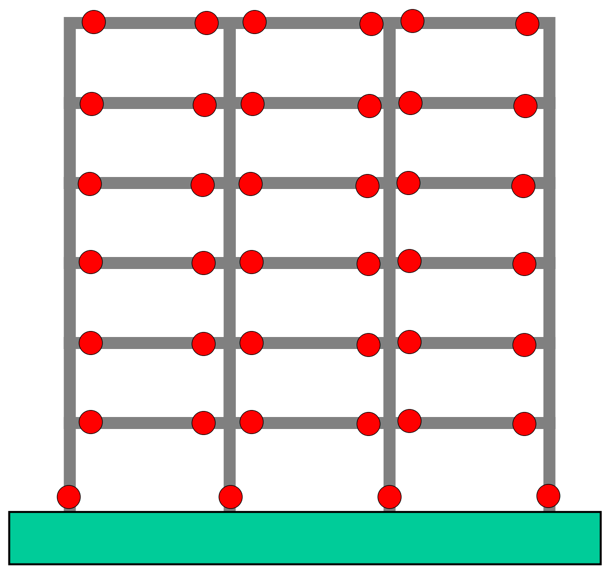
Advantages:
Architectural simplicity, relatively low base shear, less congested reinforcement
Disadvantages:
Restricted use

Special Concrete Moment Frame

DETAILING REQUIREMENTS

- Restrictions on steel grades
- Continuous top & bottom reinforcement
- Joint shear strength requirements
- Strong column - weak beam
- Use of maximum probable strength
- Closely spaced ties in critical regions





Advantages:
Architectural simplicity, relatively low base shear
Disadvantages:
Drift control, congested reinforcement

In Austin, the relatively low seismicity dictates a low ductility demand (typically < 2); hence, intermediate and special detailing may be used.

However, there is no motivation to use special detailing if the resulting design forces fall below the inherent capacity.

What if Supplied Ductility Cannot Meet the Demand?

$$\text{Ductility demand} = \frac{\text{Elastic seismic demand}}{\text{Affordable capacity}}$$

- Increase affordable capacity
  (pay a higher seismic premium)

- Reduce elastic seismic demand
  - Base isolation
  - Added damping

System Development (Summary)

Could I use an ordinary moment frame in California?

- Theoretically, YES if affordability is not an issue.
- Practically, NO as costs will be unreasonable.

Could I Use a special moment frame in Austin?

- Theoretically, YES but detailing would be governed by inherent strength requirements.
- Practically, NO as costs would be unreasonable.

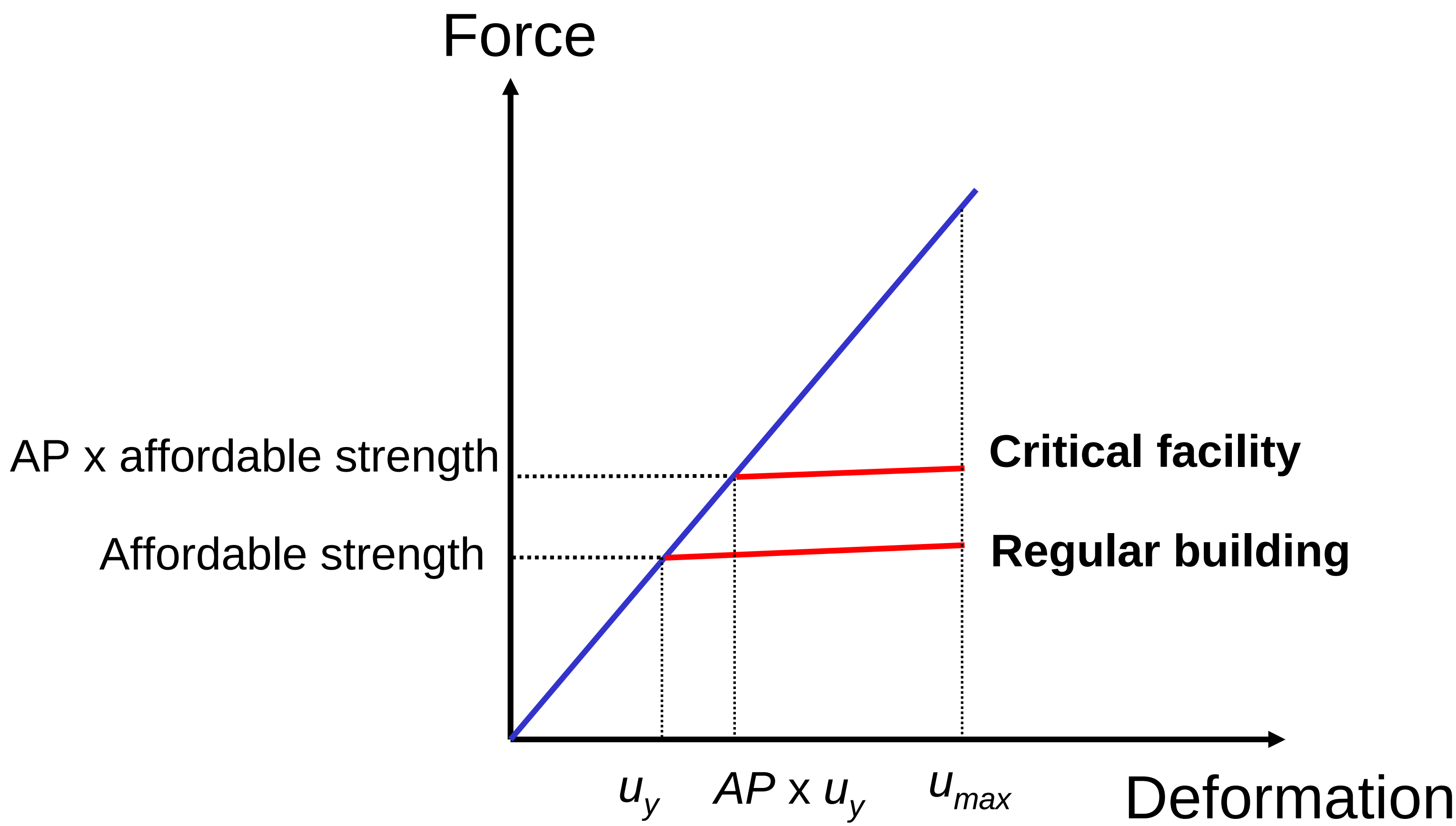
Note: Comments are without regard to building code requirements

Essential Facilities: How To Provide More Protection?

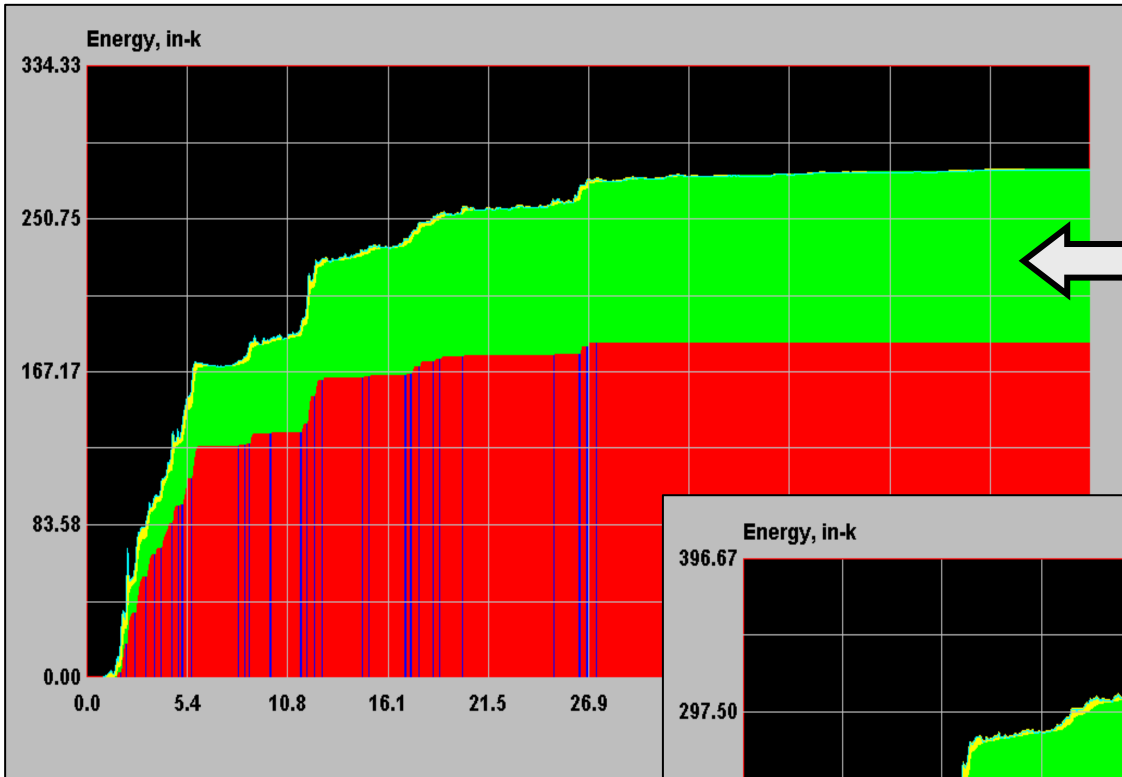
$$\text{Ductility demand} = \frac{\text{Elastic seismic demand}}{\text{Affordable capacity}}$$

Reduce ductility demand by increasing affordable capacity (make system stronger).

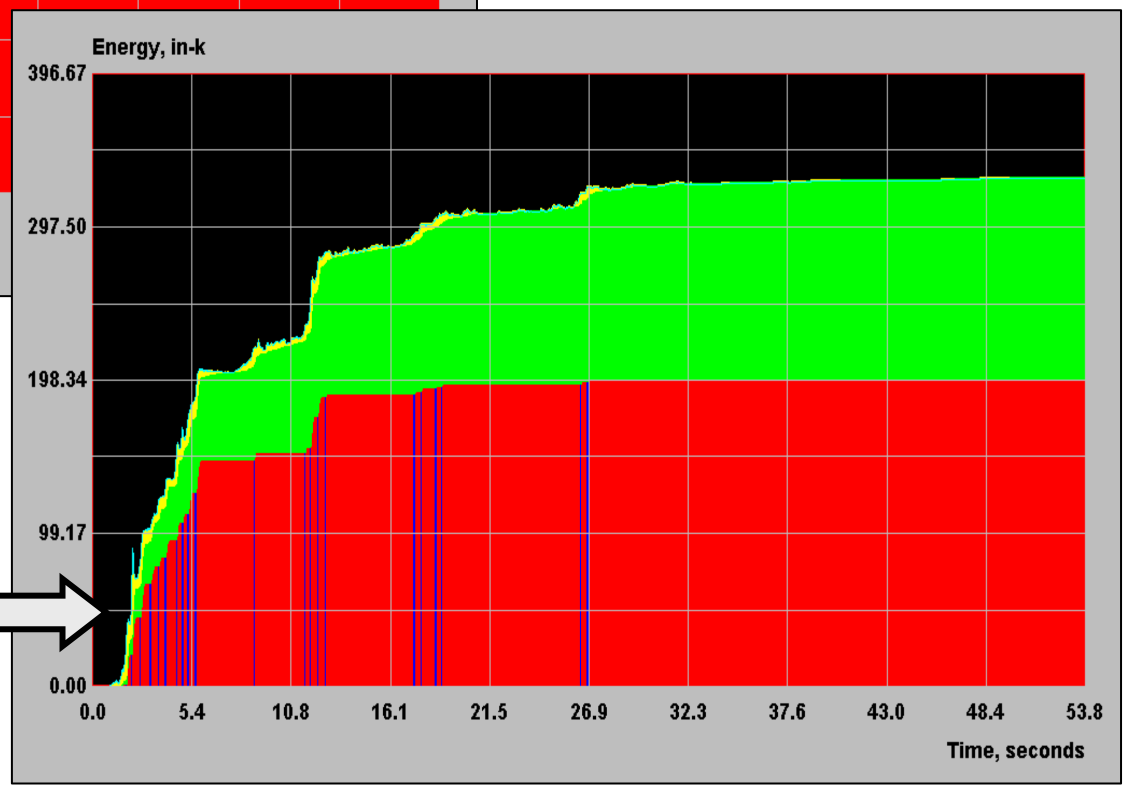
Reduction in Ductility Demand Is in Direct Proportion to Additional Premium Paid



AP = Additional premium (1 in *NEHRP Provisions*)



Strength index = 1.0
Max drift = 2.2 in.
Duct. demand = 4.4
Max E_H = 183 in-k



Strength index = 1.5
Max drift = 2.4 in.
Duct. demand = 3.1
Max E_H = 199 in-k

Damage Reduction Is Apparent in Denominator of Second Term

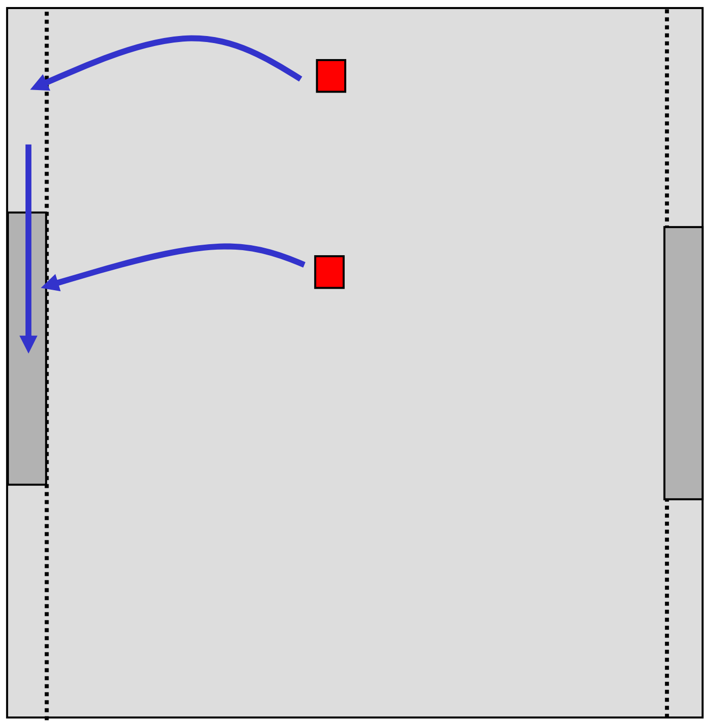
$$Damage = \frac{\delta_{\max}}{\delta_{ult}} + 0.15 \frac{E_H}{AP \times F_y \delta_{ult}}$$

System Concepts

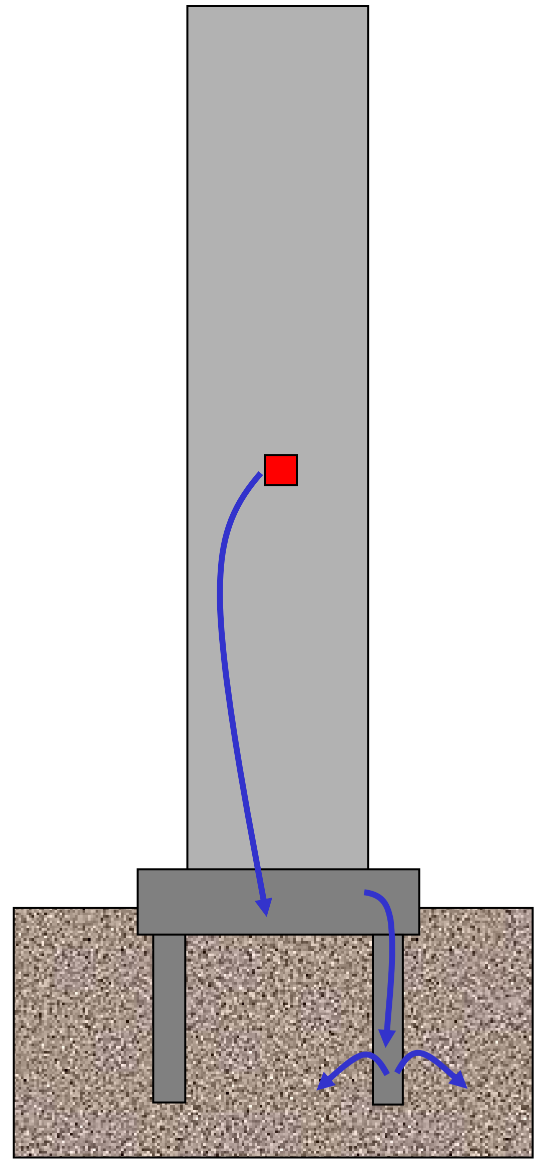
Optimal performance achieved by:

- Providing competent load path
- Providing redundancy
- Avoiding configuration irregularities
- Proper consideration of “nonstructural” elements and components
- Avoiding excessive mass
- Detailing for controlled energy dissipation
- Limiting deformation demands

Concept of Competent Load Path

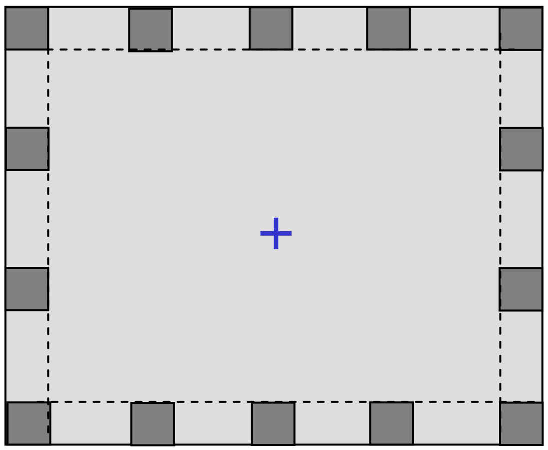


Plan

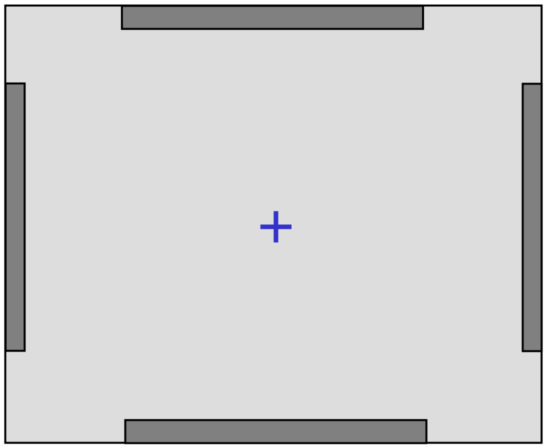


Elevation

Which System is Better?



System A

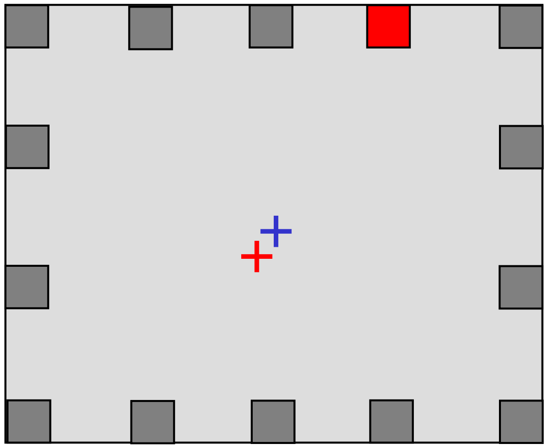


System B

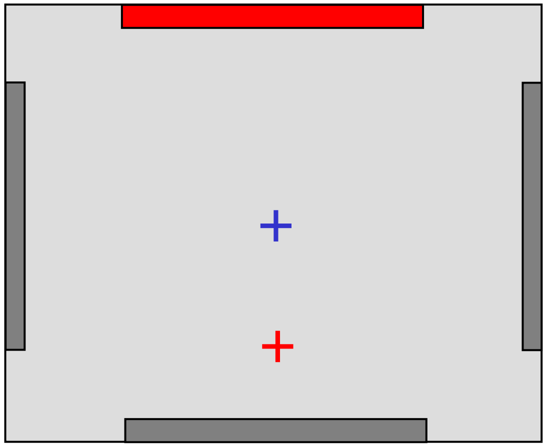
Overall strength of System A = System B

Systems have same overall deformation capacity.

Which System is Better?



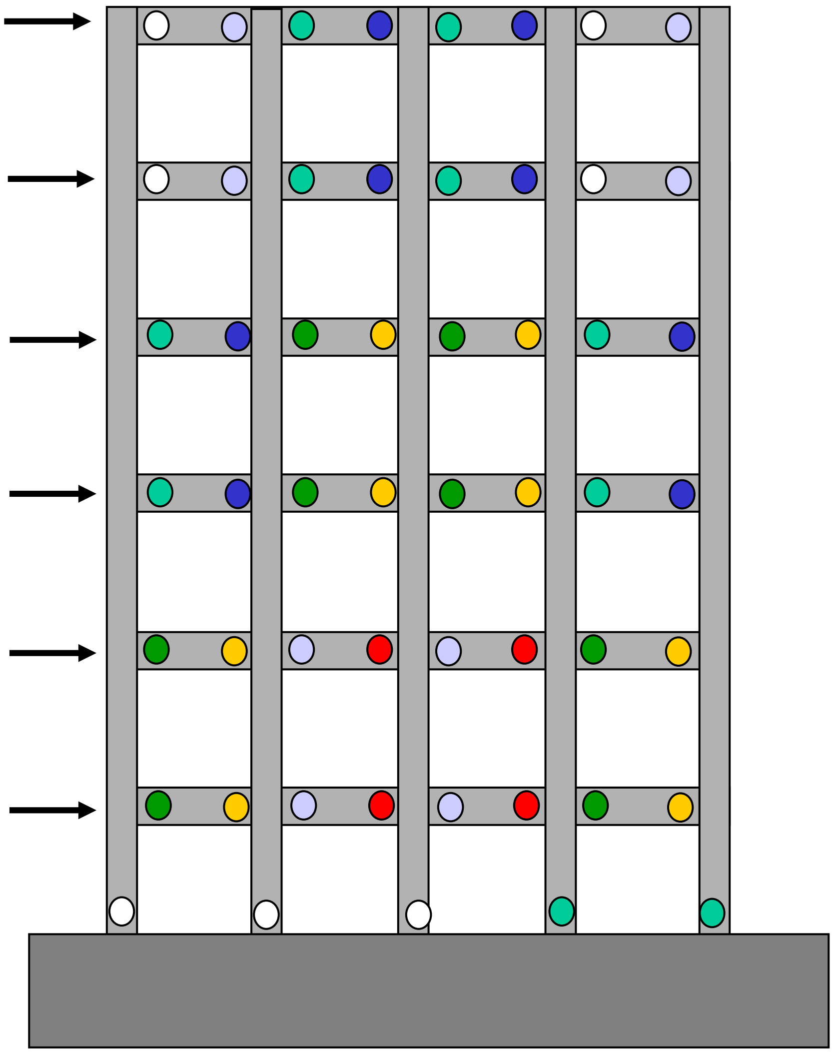
System A



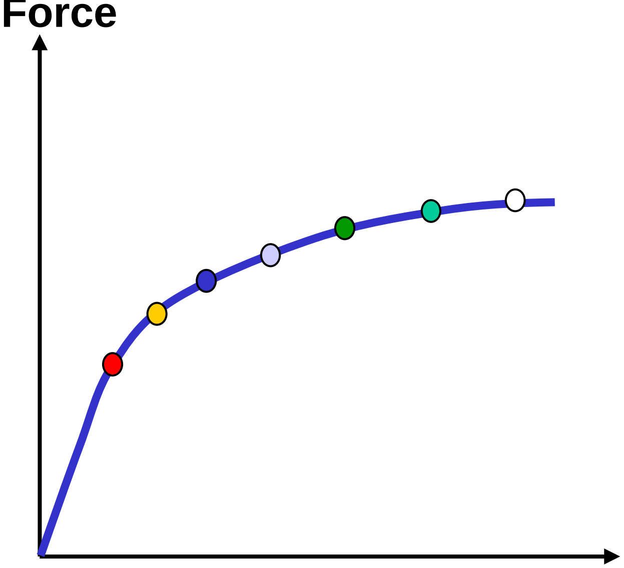
System B

What is the effect of a premature loss of one element?

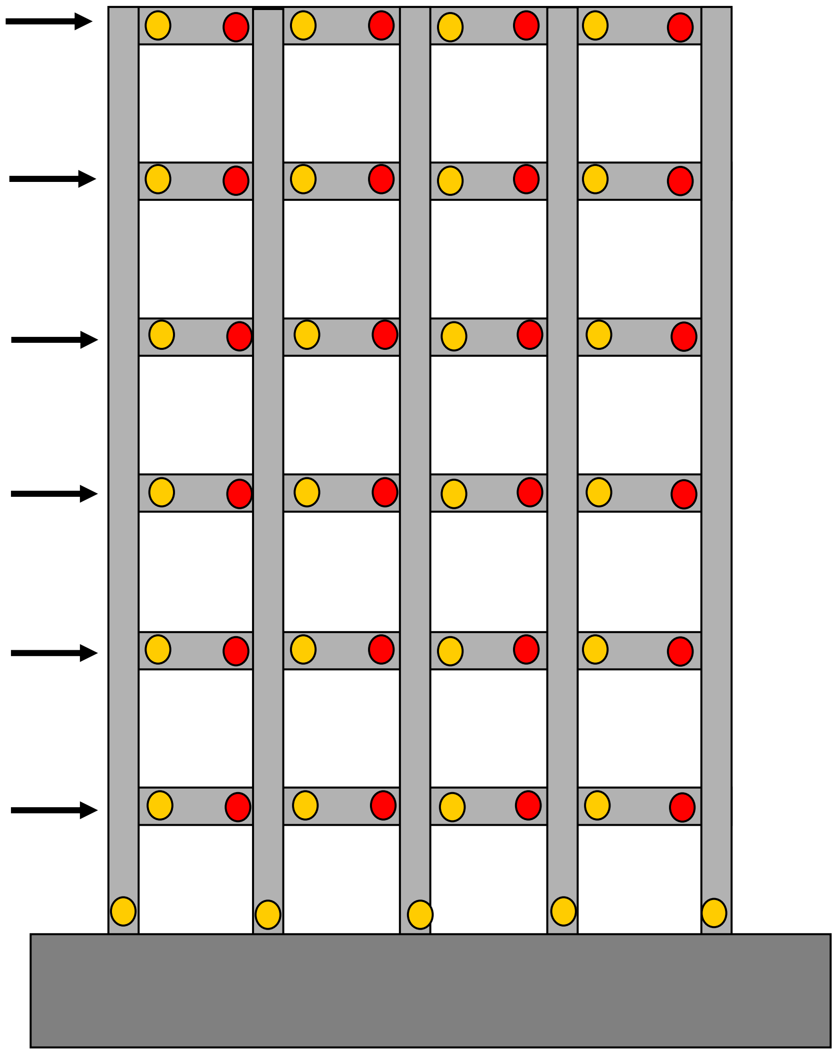
Increase Local Redundancy by Designing Hinge Sequence



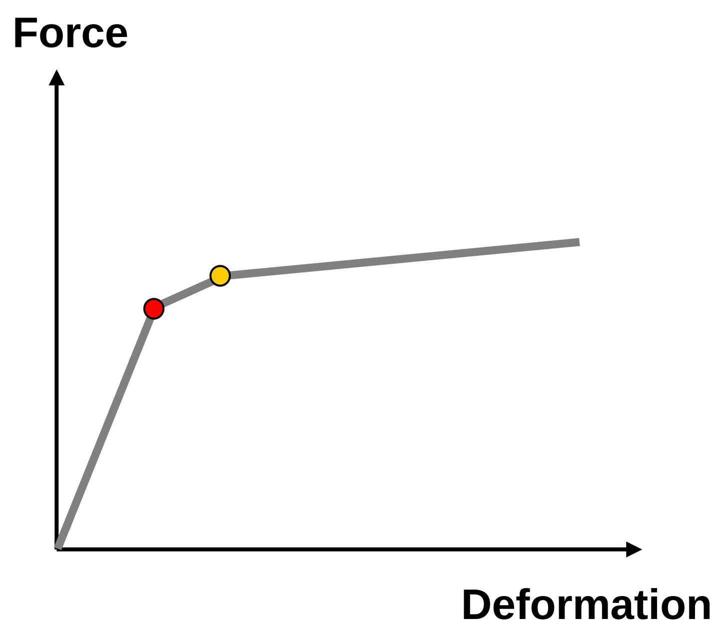
Hinge sequence



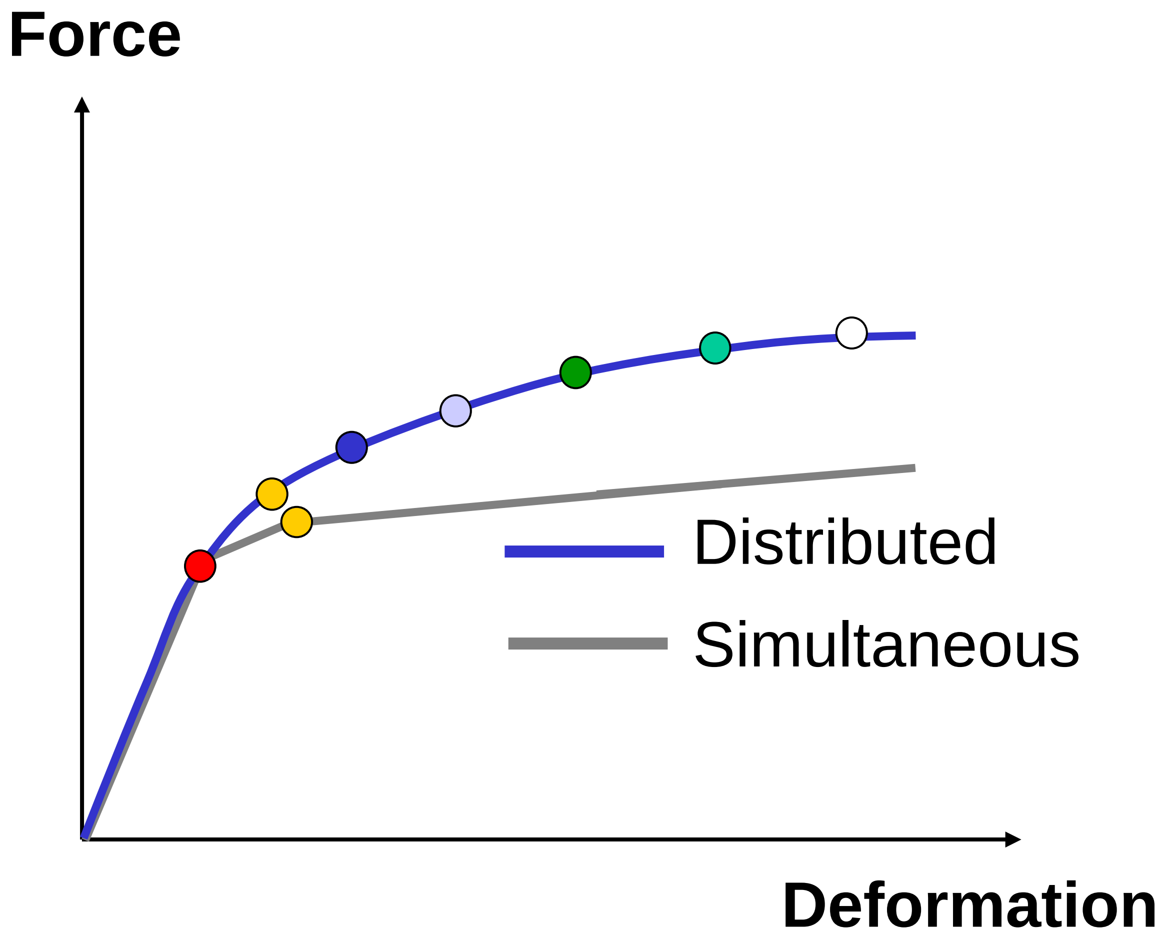
Versus Simultaneous Hinging



Hinge sequence



Distributed vs Simultaneous Hinging

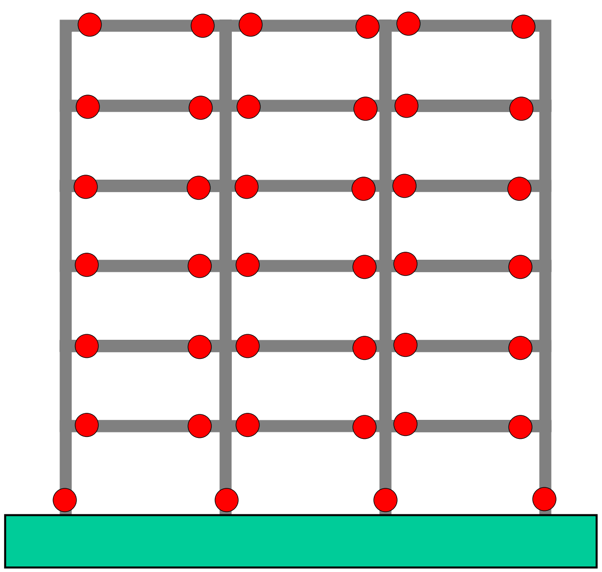


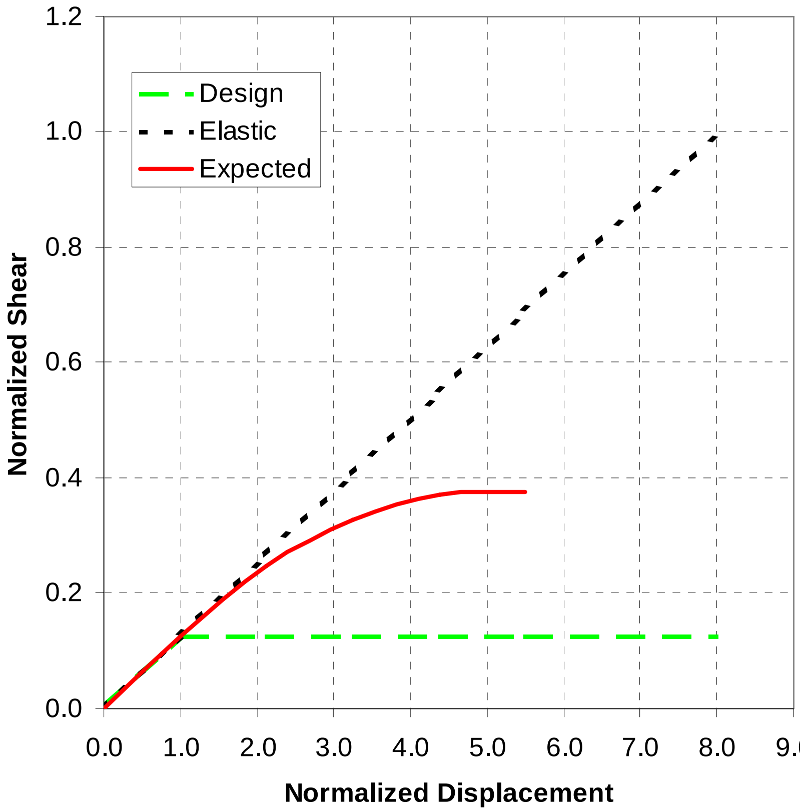
Simultaneous: Less apparent overstrength
Less post-yield stability

Special Concrete Moment Frame

DETAILING REQUIREMENTS

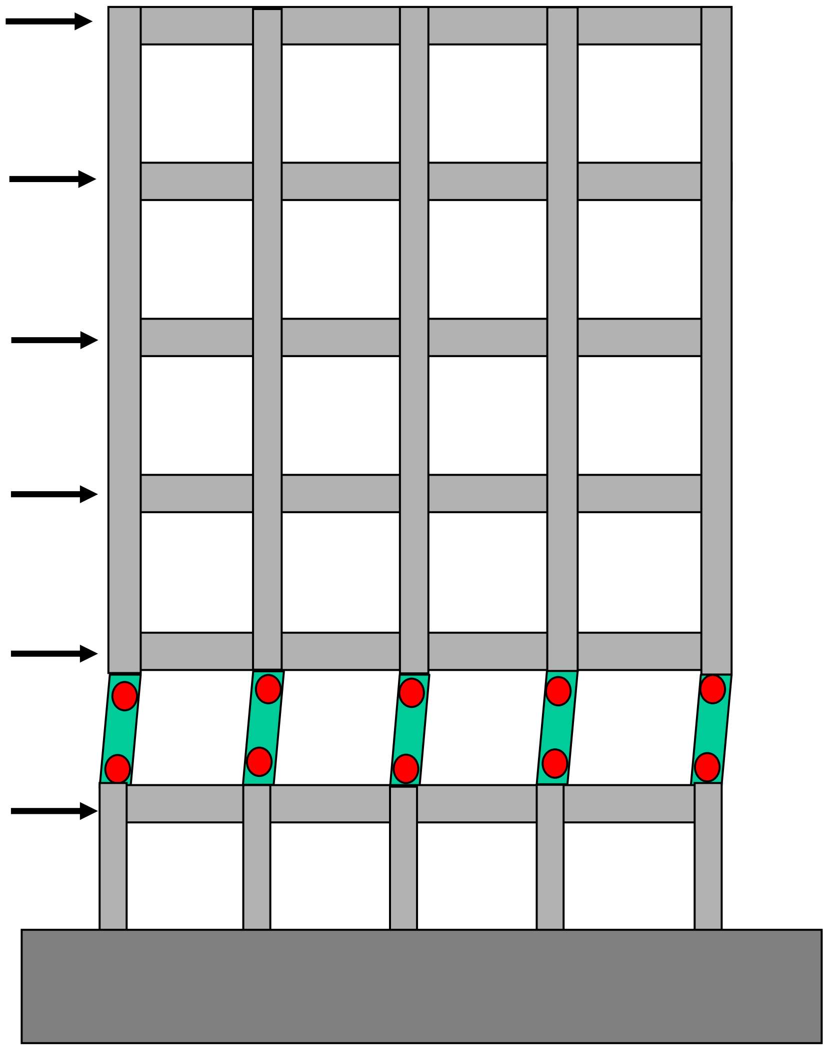
- Restrictions on steel grades
- Continuous top & bottom reinforcement
- Joint shear strength requirements
- Strong column - weak beam
- Use of maximum probable strength
- Closely spaced ties in critical regions

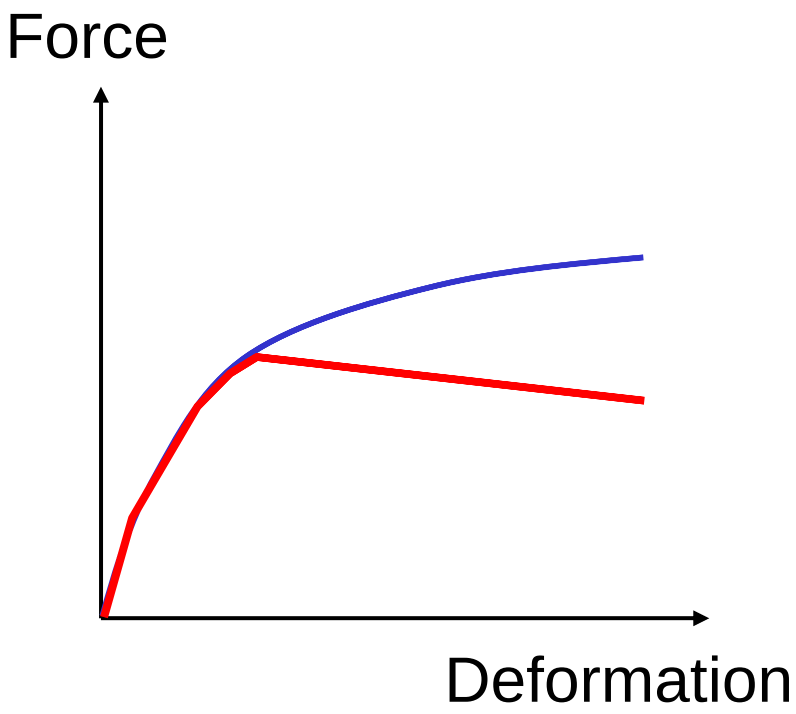




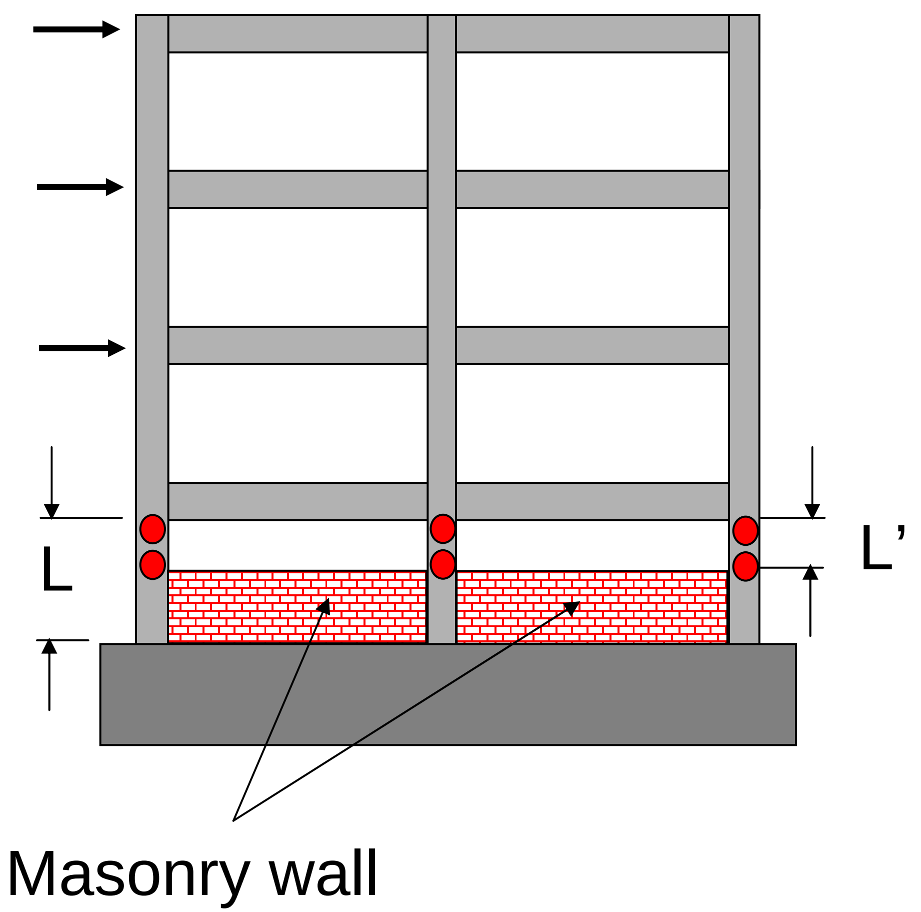
Advantages:
Architectural simplicity, relatively low base shear
Disadvantages:
Drift control, congested reinforcement

Avoid Undesirable Mechanisms





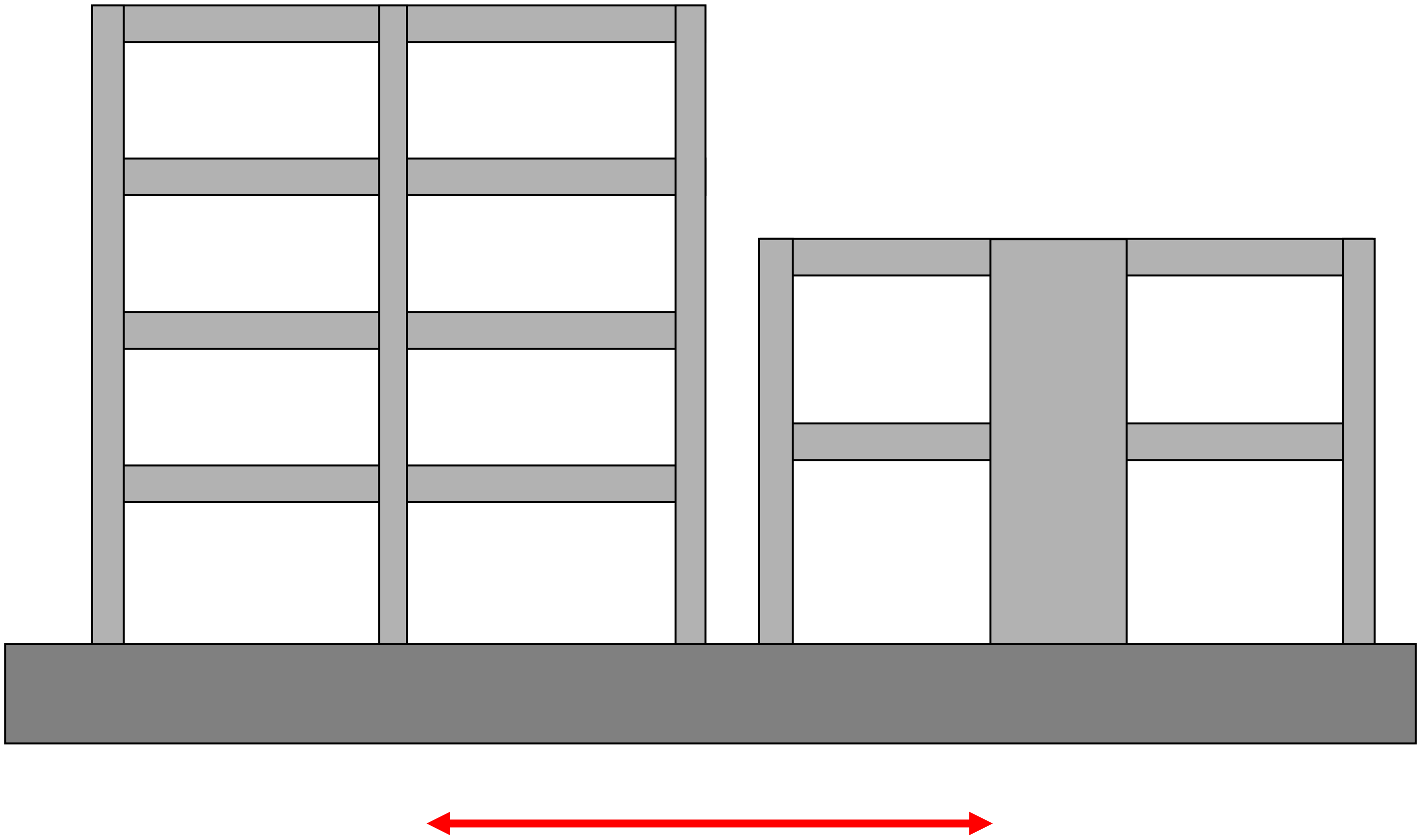
Avoid Accidental Mechanisms



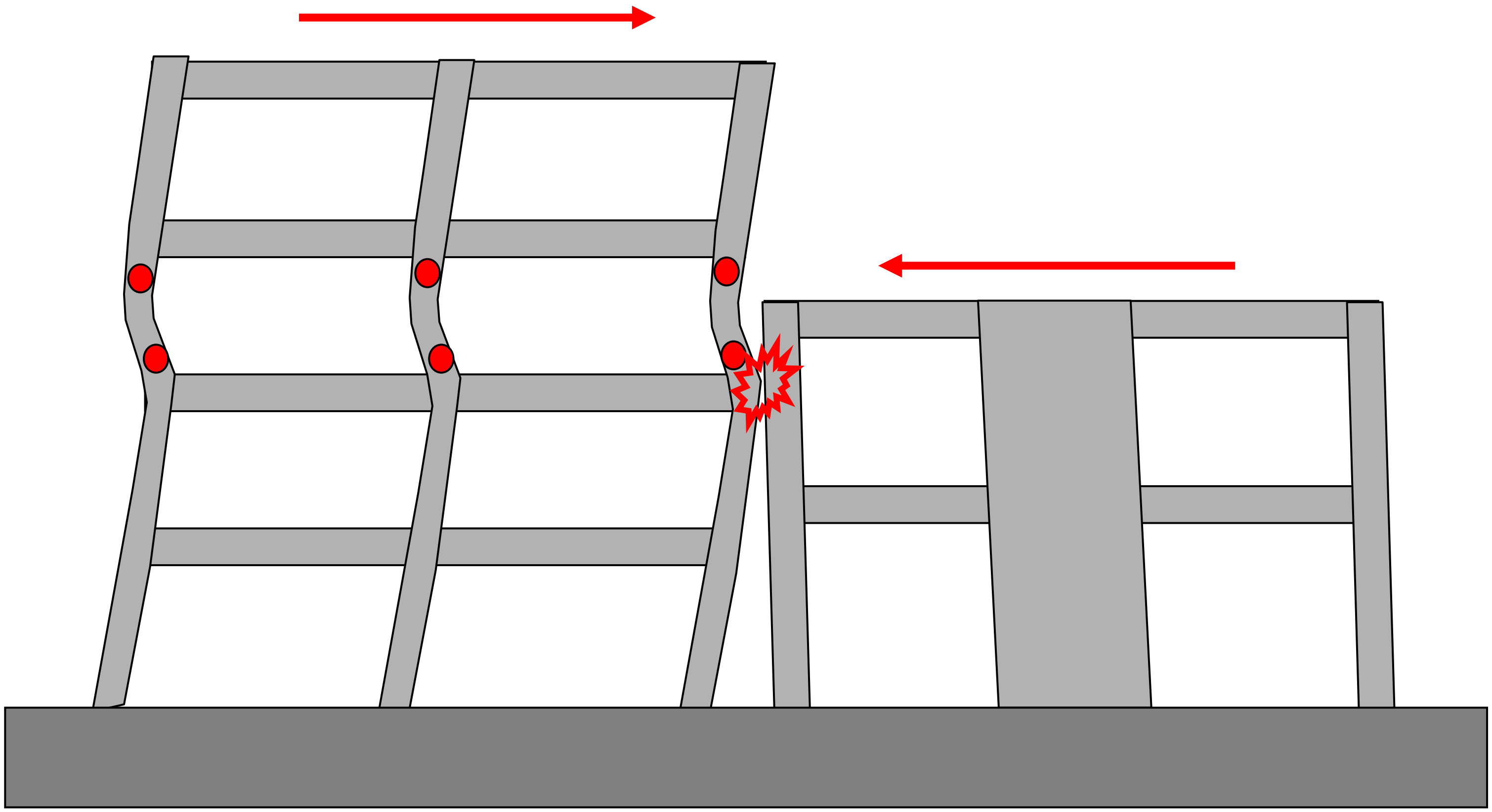
$$V_{design} = 2M_p/L$$

$$V_{actual} = 2M_p/L'$$

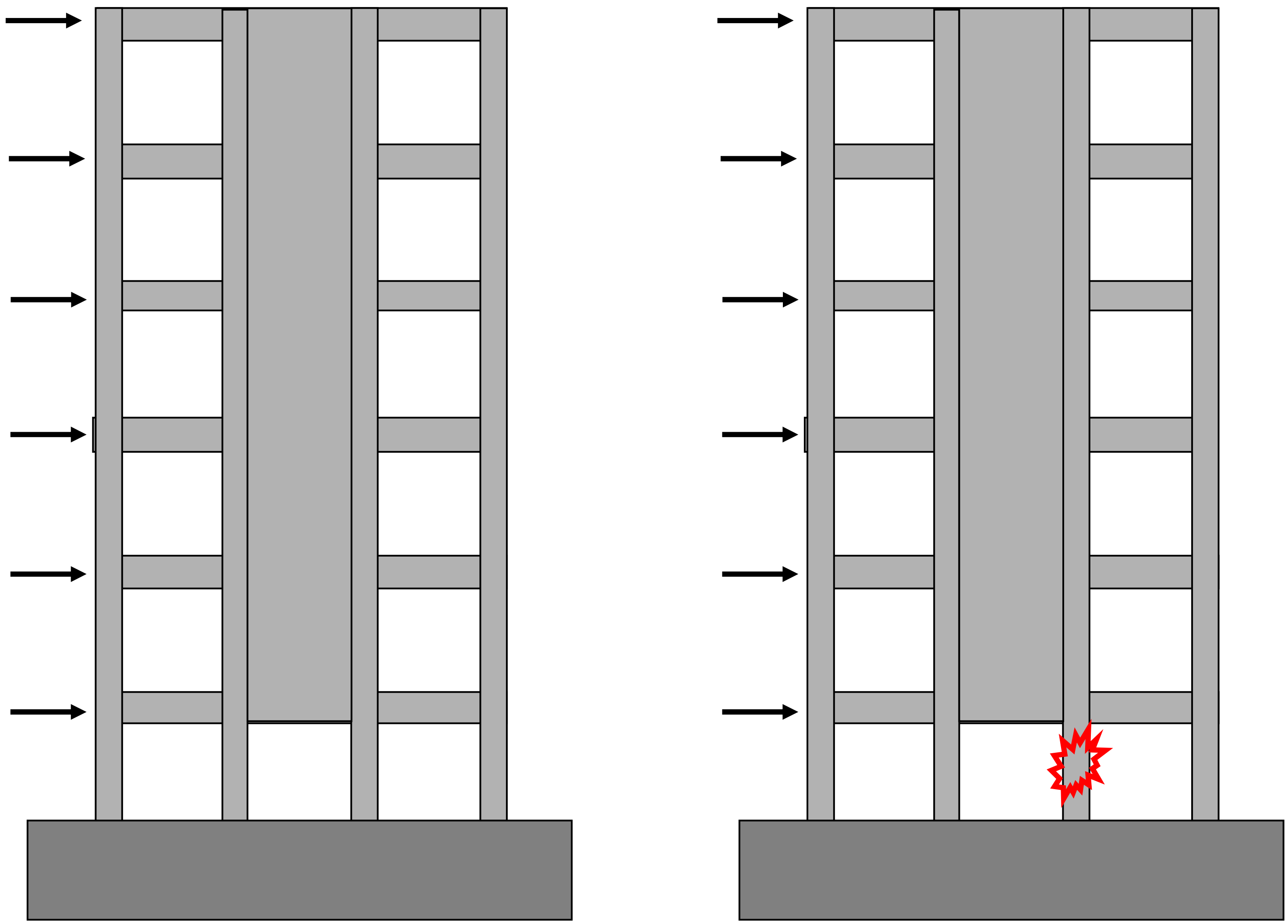
Avoid Accidental Mechanisms



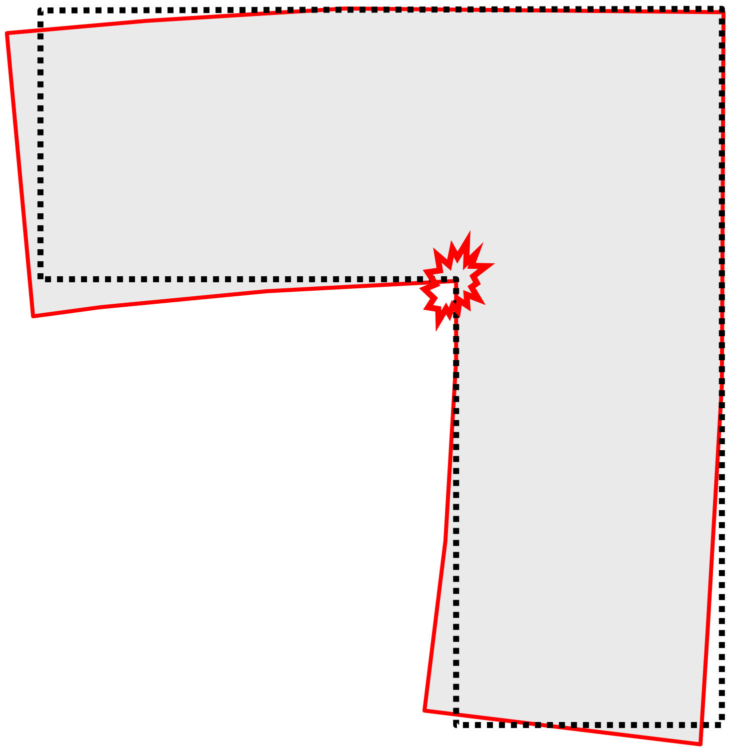
Avoid Accidental Mechanisms



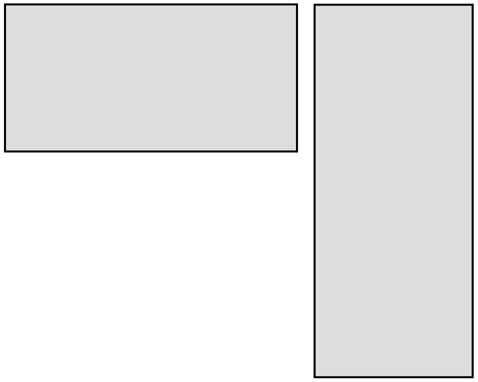
Avoid Situations Where the Loss of One Element Is Catastrophic



Avoid Re-entrant Corners (or Reinforce Accordingly)

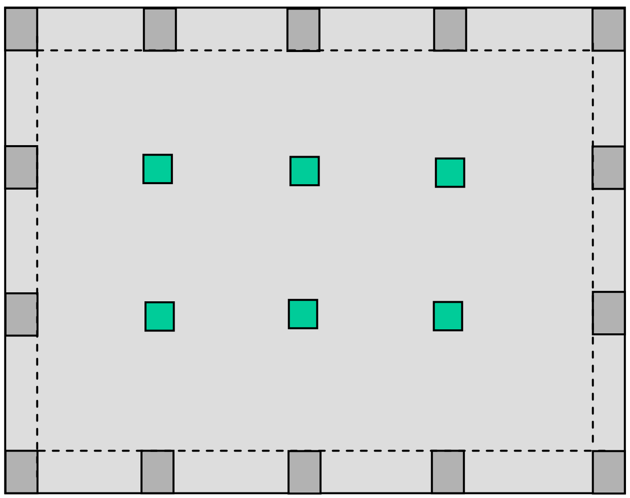


Structurally: Improved



Architecturally Dubious

Protect “Nonstructural” Elements



Steps in the Seismic Design of a Building

1. Develop Concept
2. Select Structural System
3. Establish Performance Objectives
4. **Estimate External Seismic Forces**
5. **Estimate Internal Seismic Forces (Linear Analysis)**
6. Proportion Components
7. Evaluate Performance (Linear or Nonlinear Analysis)
8. Final Detailing
9. Quality Assurance

Structural Analysis

In the context of the *NEHRP Recommended Provisions*, the purpose of structural analysis is to estimate:

1. The forces required to proportion members
2. Global deformations (e.g., story drift)

What kind of analysis to use?

- ☐ Equivalent lateral force (ELF) analysis
- ☑ **Modal response spectrum (MRS) analysis**
- ☐ Linear time history (LTH) analysis
- ☐ Nonlinear static pushover (NSP) analysis
- ☐ Nonlinear dynamic time history (NTH) analysis

Structural Analysis

The analysis must be **good enough for design**. There should be **no expectation** that the analysis can predict actual response (linear or nonlinear)

ELF: Good enough for preliminary design but not final design

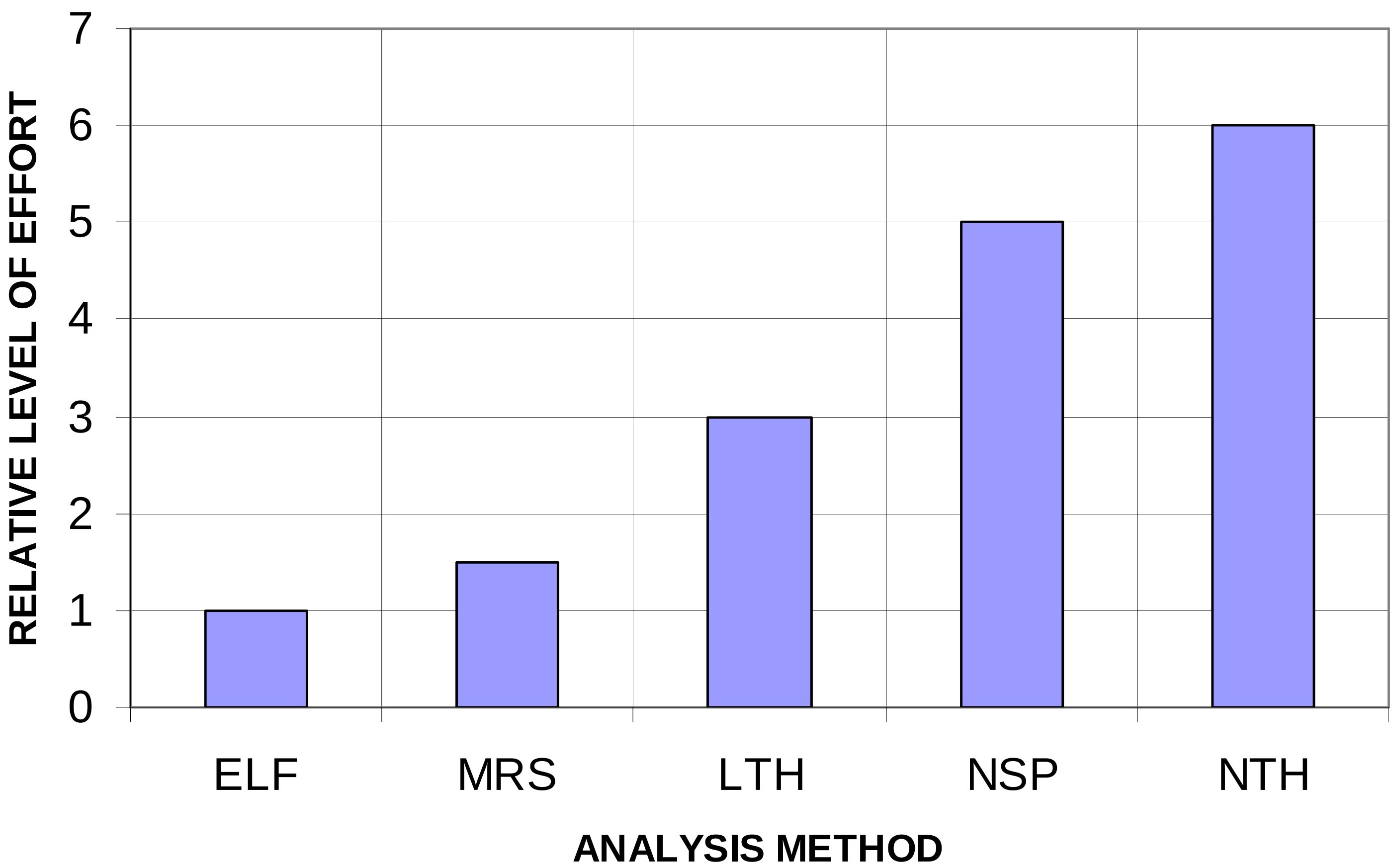
MRS: Good enough for design

LTH: Not significantly better than MRS

NSP: The Jury is deliberating

NTH: The best choice for predicting local deformation demands (Note: NTH is not required by *NEHRP Recommended Provisions* or IBC.)

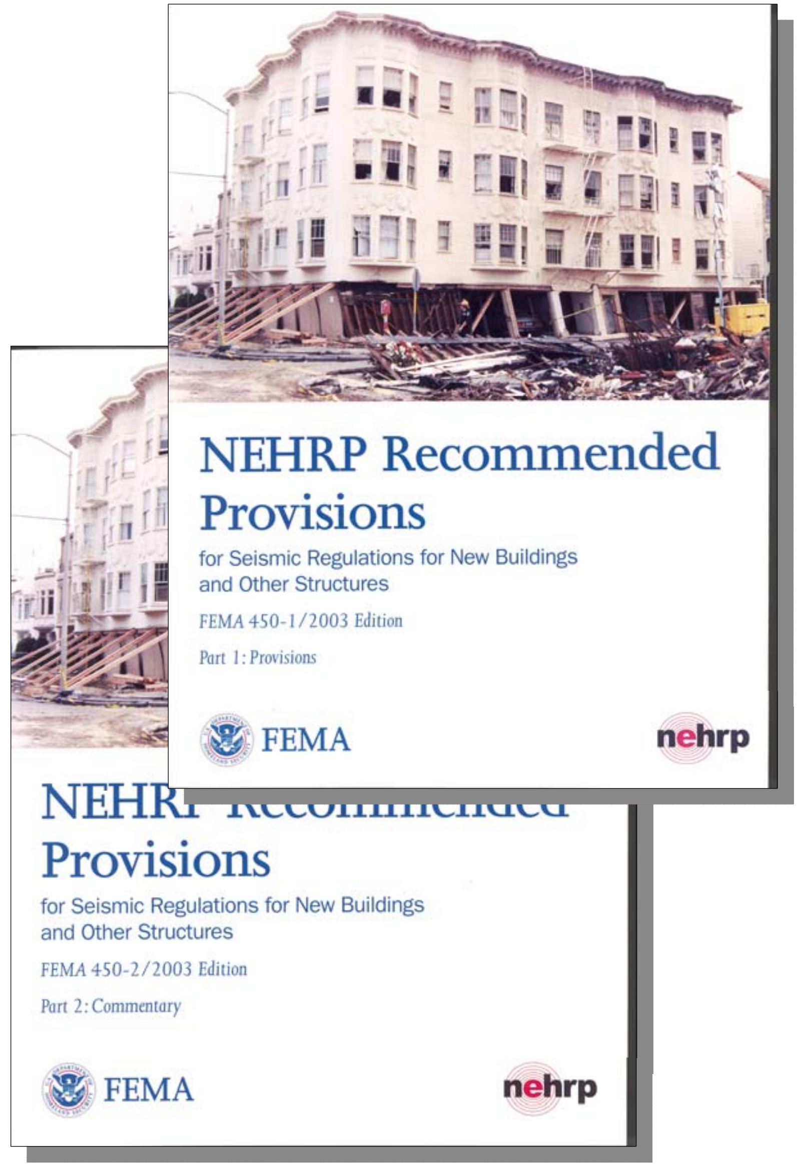
Structural Analysis: Relative Level of Effort



Seismic Design (and Analysis) Is as Much an Art as It Is a Science

INTRODUCTION

Development of the *NEHRP Recommended Provisions*



NATIONAL
EARTHQUAKE
HAZARDS
REDUCTION
PROGRAM

San Fernando, 1971

- Damage to Olive View Hospital
  - Stair tower separated from building
  - Near collapse in second story

Responses to San Fernando

- 1972 Workshop - Improve codes
- 1974 SEAOC - Quick Change
  - Higher forces
  - Soil factor
  - Importance factor
- 1974-76 ATC-3 Project
  - Fundamental changes
- 1977 Earthquake Hazard Reduction Act

Creation of NEHRP

- Public Law 95-124: EHRA of 1977
  - "to reduce risks to life and property from future earthquakes in the United States."
- Construction
- Model codes
- Plus response, recovery, and many other concerns

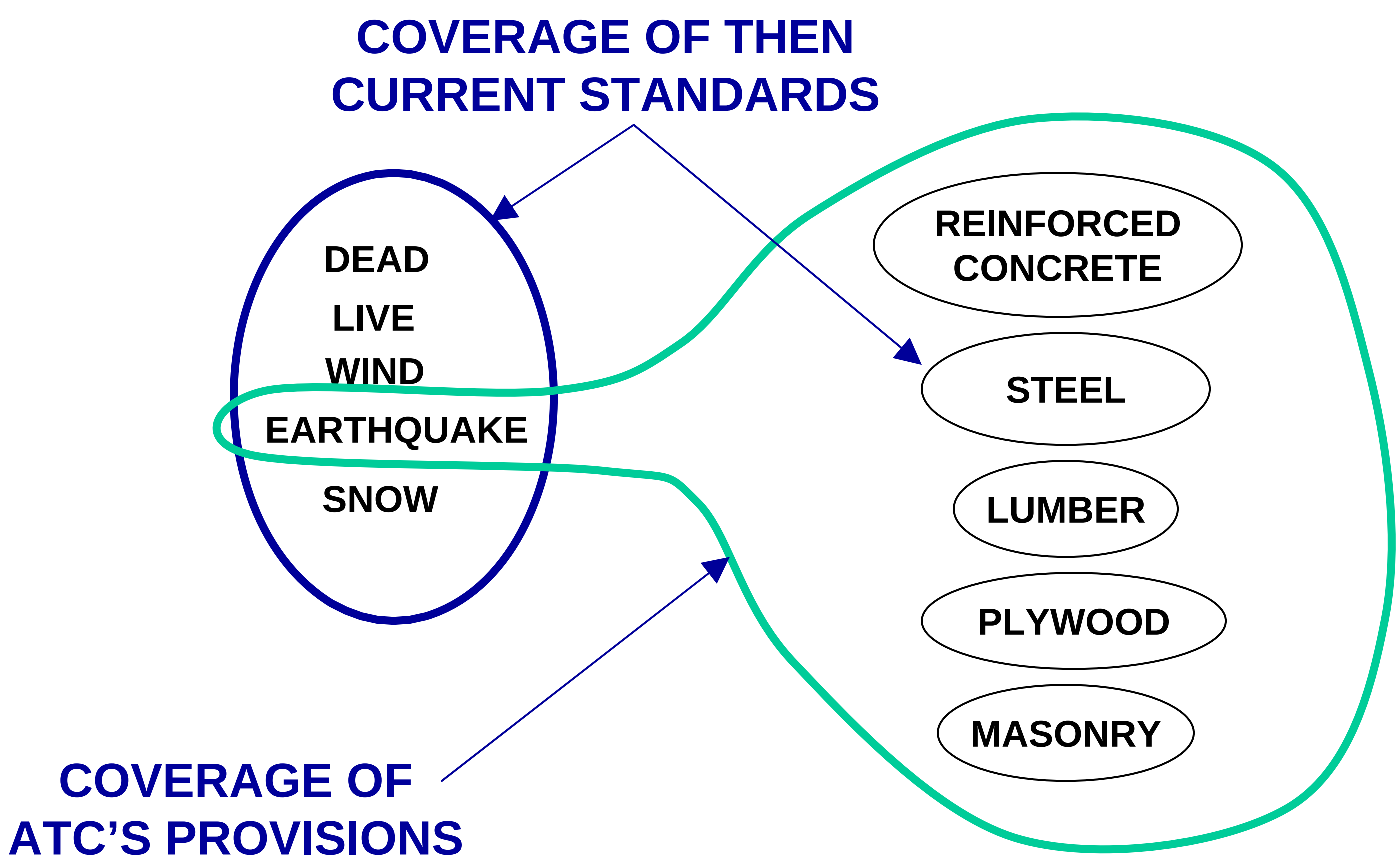
Principal NEHRP Agencies

- Federal Emergency Management Agency
- National Institute of Standards and Technology (new lead agency)
- United States Geological Survey
- National Science Foundation

ATC-3 Report

- 1978 Publication of ATC / NSF / NBS
- New approaches:
  - Nationwide
  - Probabilistic
  - Inelastic behavior
  - Strength level
  - Nonstructural
  - Existing Buildings

Standards for Structural Design vs. ATC's Provisions



Building Seismic Safety Council

BSSC

- Private, voluntary
- A council of NIBS
- National forum for issues:
  - Technical
  - Social
  - Economic
- 60+ organizational members
- Consensus process

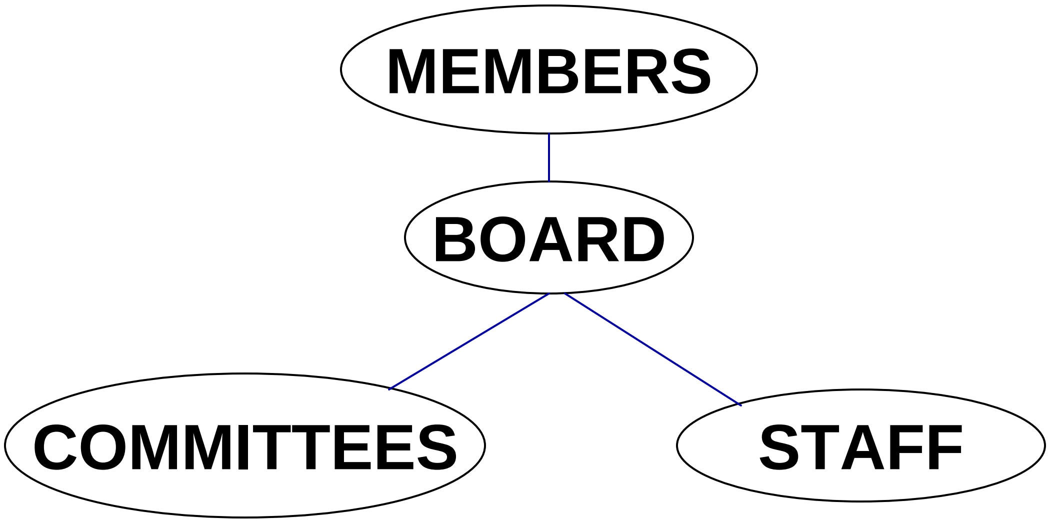
BSSC Members

Building Community Organizations

Examples:

ACI	AF&PA	AIA	AISC
AISI	AITC	APA	ASCE
ASME	BIA	EERI	ICC
NCMA	NCSEA	NFPA	PCA
SEAOC	TMS		

BSSC Organization



History of *NEHRP Recommended Provisions* for New Buildings

- 1974-78 ATC 3 Tentative Provisions
- 1979-80 Review Committees
- 1981-84 Trial Designs
- 1985 Edition (first edition to be "Recommended")
- New editions in 1988, 1991, 1994, 1997, 2000, and 2003

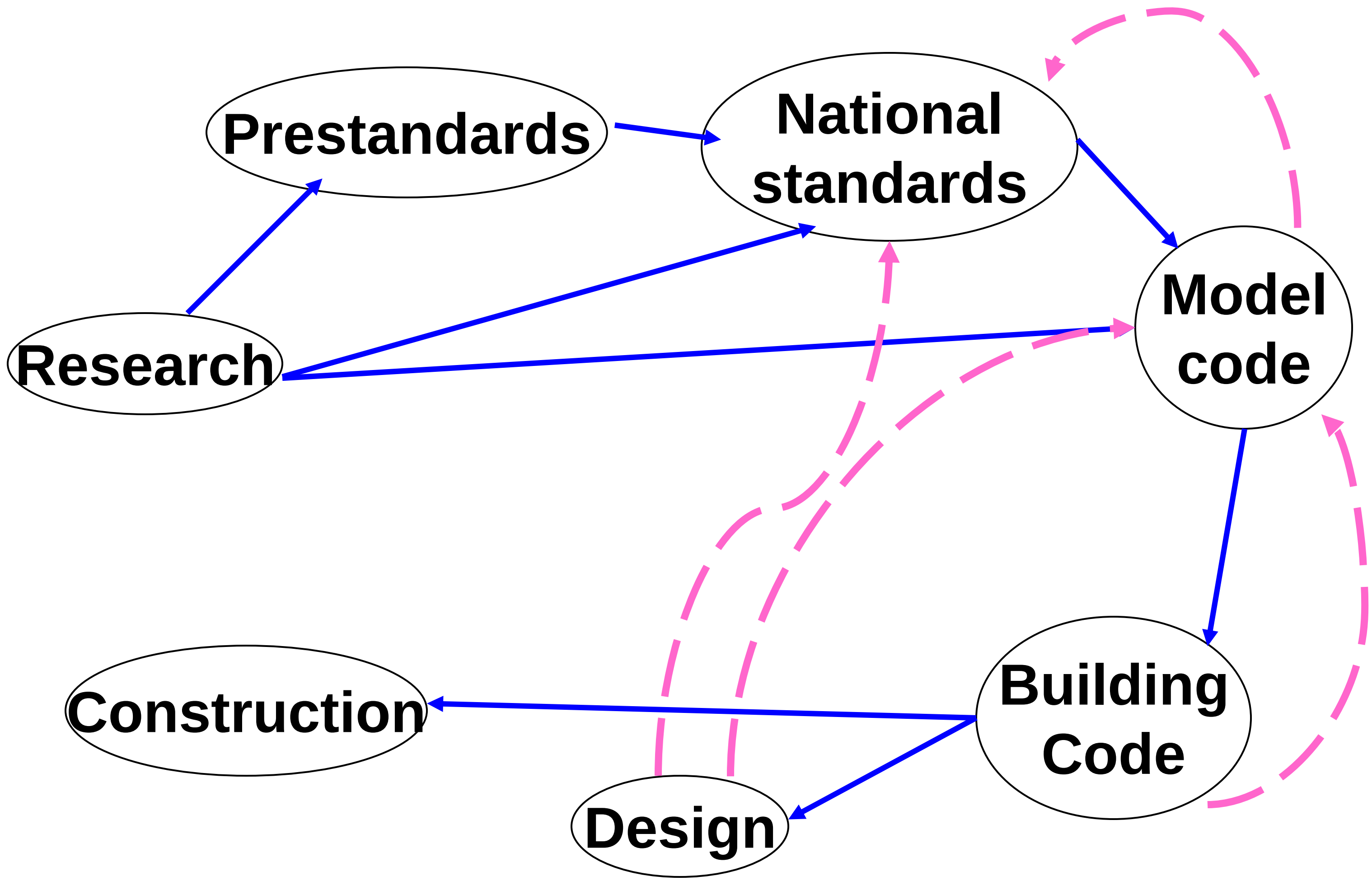
Context and Use of the *NEHRP Recommended Provisions*

- Building codes and standards

- Organizations generating standards and model codes

- Role of BSSC

Information Flow Through Building Codes



Model Code Organizations

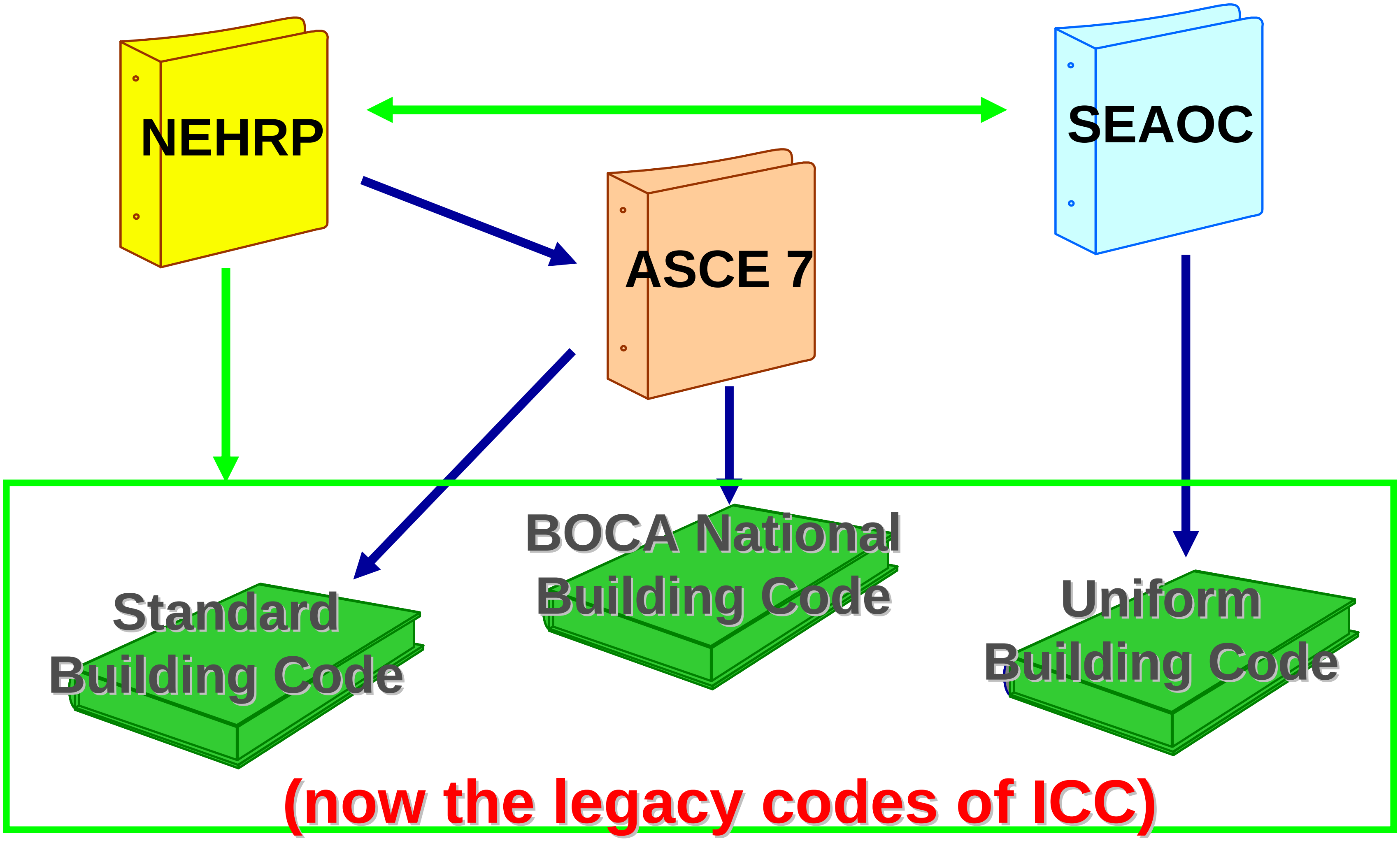
- International Conference of Building Officials
  - Uniform Building Code
- Building Officials and Code Administrators
  - National Building Code
- Southern Building Code Congress

Previous

Current

  - Standard Building Code
- International Code Council
  - International Building Code
- National Fire Protection Association
  - NFPA 5000

Prior Model of Information Flow

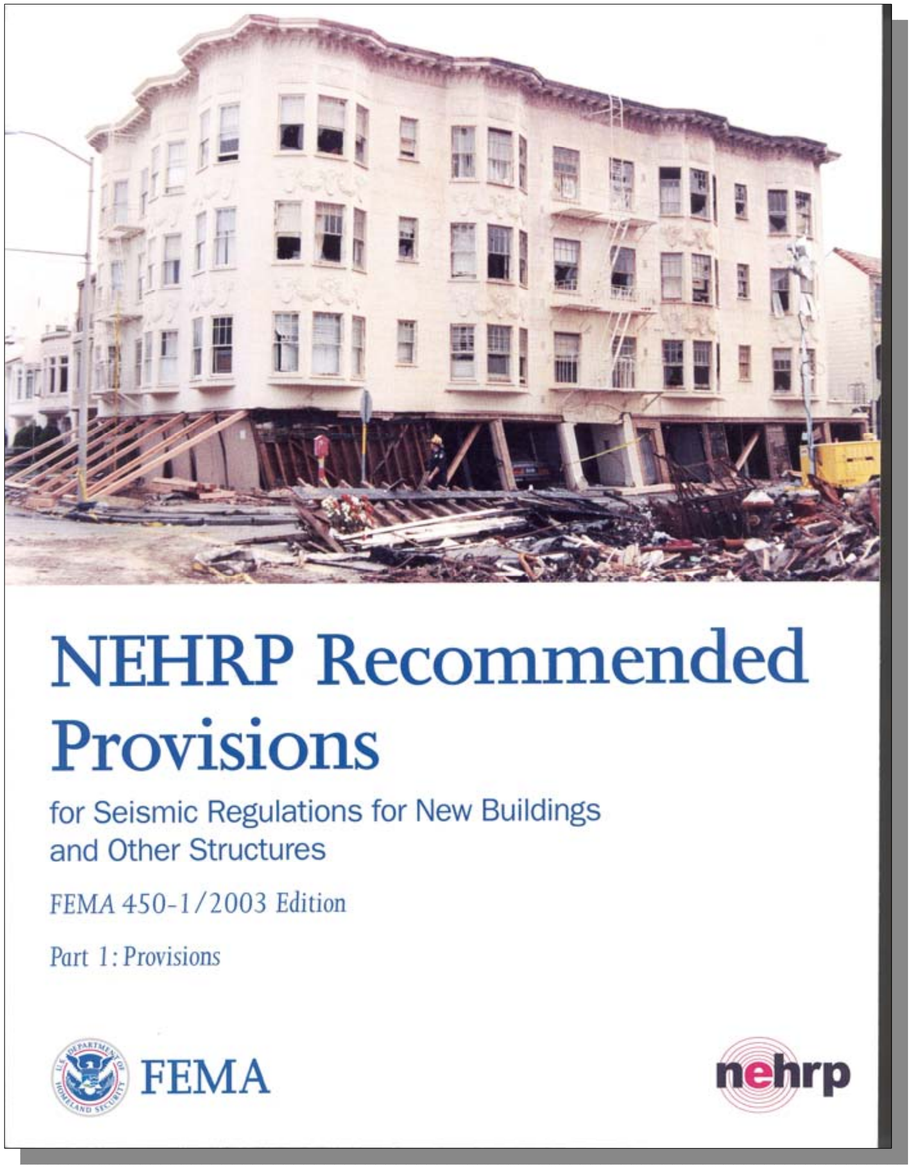


OVERVIEW OF STRUCTURAL ENGINEERING STANDARDS

The Basic Implementation of the 2003 *NEHRP Recommended Provisions*

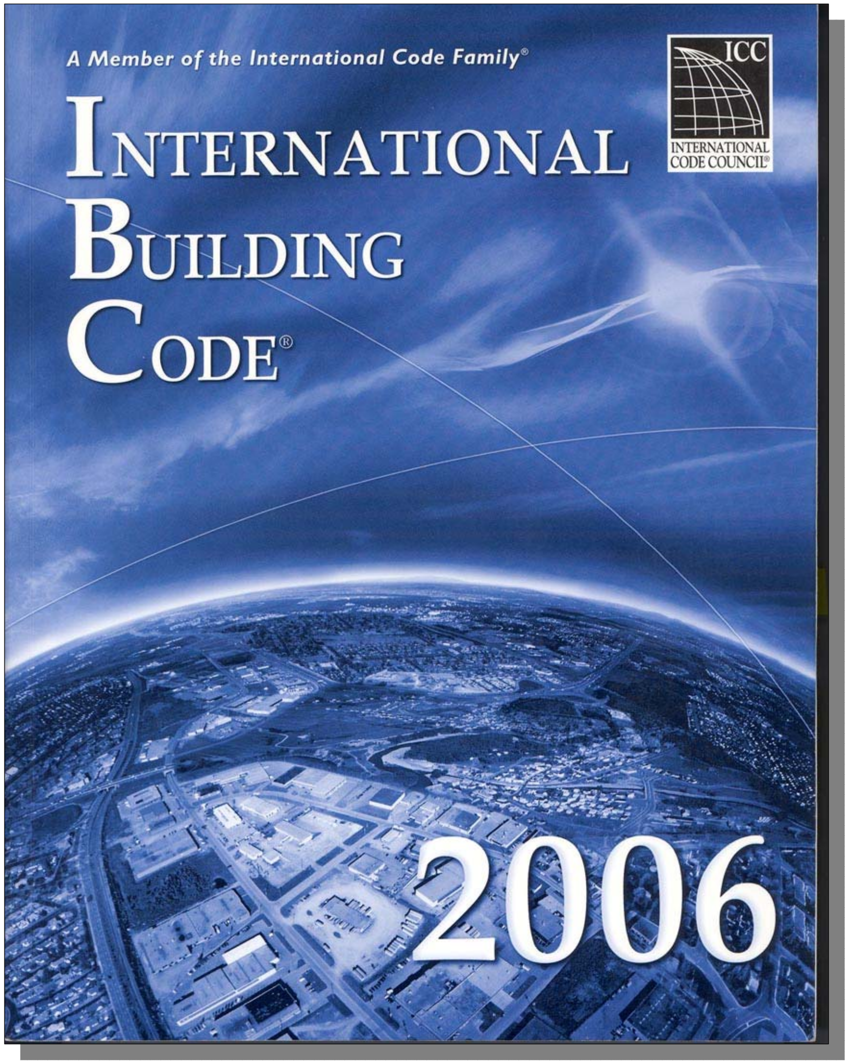
Scope

- Brief description of standards for design of basic building structures that implement the 2003 *NEHRP Recommended Provisions*
- Does not include standards referenced for design of nonstructural components and anchorages
- Does not include standards referenced for design of nonbuilding structures



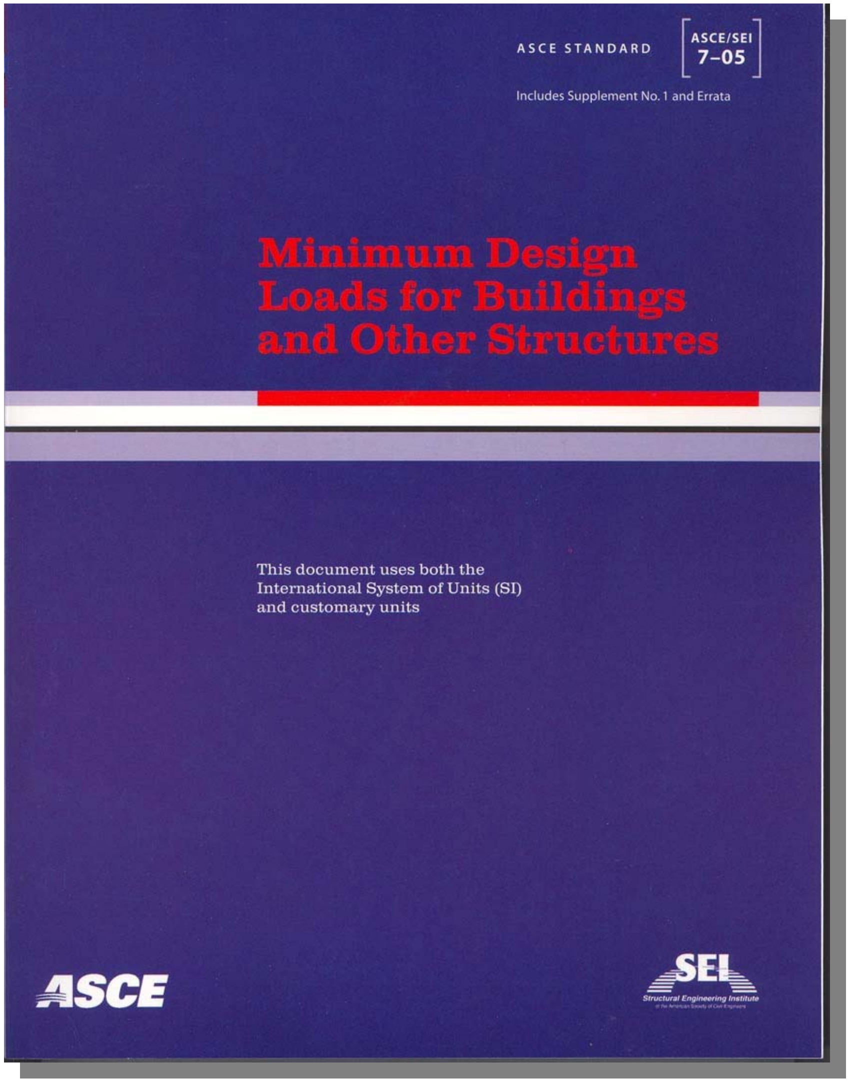
NEHRP Recommended Provisions

- Fundamentally a resource document
- Produced at the Building Seismic Safety Council
- 2003 edition influences many standards
- 3 year cycle till now



IBC 2006

- Sets some basic requirements, but mostly cites structural design standards by reference.
- A distinct change from the UBC, more like SBC and BNBC.



ASCE/SEI 7
2005 edition
with Supplement 1

- Includes the bulk of 2003 *NEHRP Provisions* for its seismic chapters
- Reorganized and strongly edited

ASCE 7

- Developed by ASCE-SEI using ANSI standard consensus process
- Publication cycle varies (1988, 1993, 1995, 1998, 2002, 2005)
- Latest Version ASCE 7-05 Including Supplement 1 includes references to latest (2005 editions) material standards
- Extensive errata – go to www.seinstitute.org & click on publications

Vision of the Future

- Code "evolution" should slow somewhat (next edition of ASCE 7 in 2010/2011)
  - Standards are more difficult to change than codes – ASCE 7-10/11 should be adopted by 2012 IBC
  - Less rapid fire adoption of major changes
- However, IBC Code Supplements will still occur every 18 months with new full editions every 3 years.

ASCE 7-05 Reorganization

Goals of seismic section reorganization:

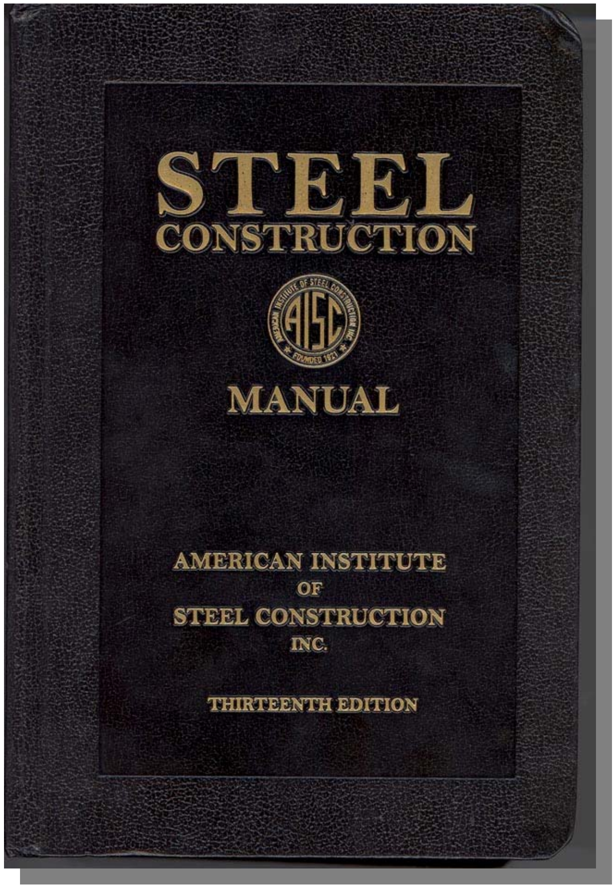
- To improve clarity and use
- Reduce depth of section numbering from 6 max typical to 4 max typical (i.e., Sec. 9.5.2.5.2.2 is now Sec. 12.5.3)
- Create logical sequence of provisions aim at the structural engineering community
- Improve headings and clarify ambiguous provisions

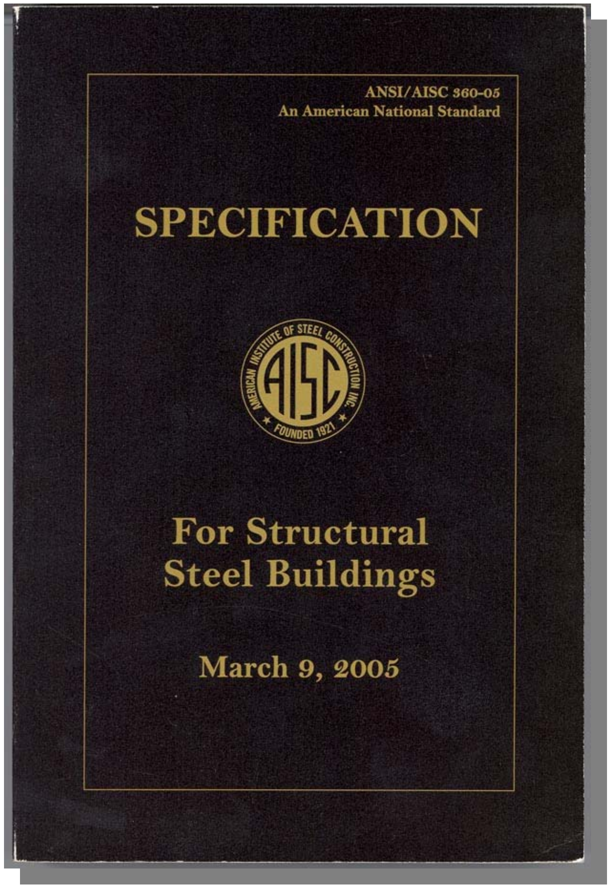
ASCE 7-05 Chapter 14: Material Specific Design and Detailing

- 1 – Steel
- 2 – Concrete
- 3 – Composite Steel and Concrete
- 4 – Masonry
- 5 – Wood

IBC 2006 does not cite Chapter 14 by reference; it includes the same information in its chapters dealing with the material of construction

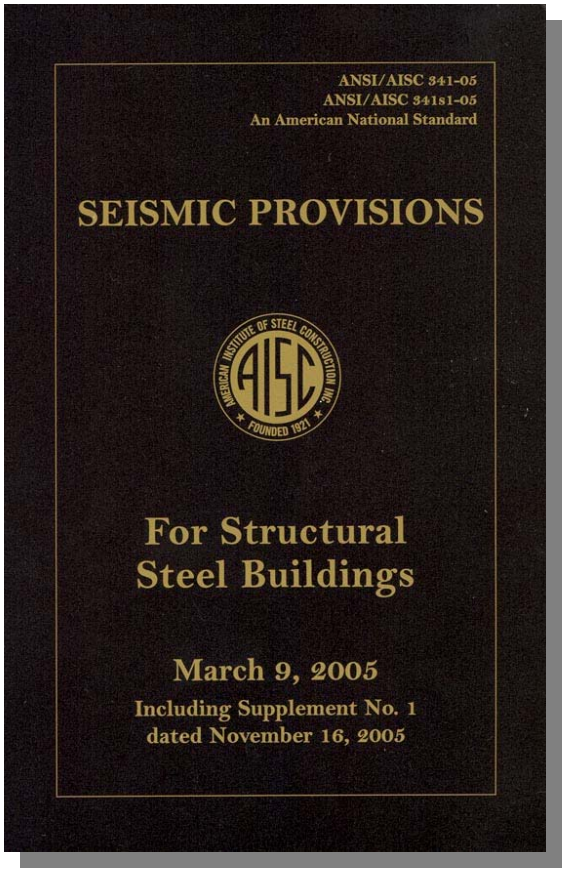
Structural Steel





AISC 360

AISC 341



Structural Steel

- Can ignore AISC 341 (seismic provisions) in Seismic Design Categories B, C if use $R = 3$
- Seismic provisions (341) required for all other situations
  - Special, intermediate, ordinary moment resisting frames
  - Special, ordinary concentrically braced frames
  - Eccentrically braced frames
  - Buckling restrained braced frames
  - Steel plate shear walls
  - Composite steel and concrete systems

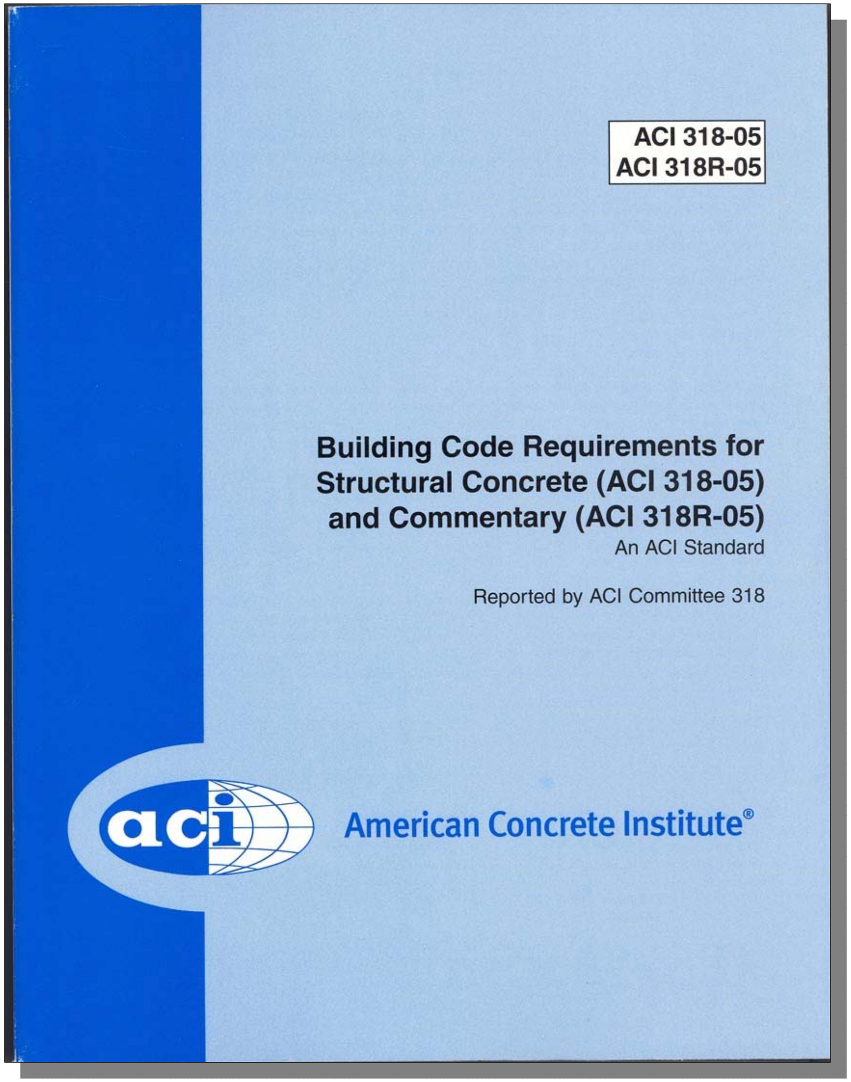
Cold Formed Steel

Cold Formed Steel

New lateral design standard covers:

- Diaphragms and walls sheathed with structural wood panels
- Walls sheathed with light gage steel sheet
- Walls braced with diagonal steel straps

No specific reference for untopped steel deck acting as a diaphragm.

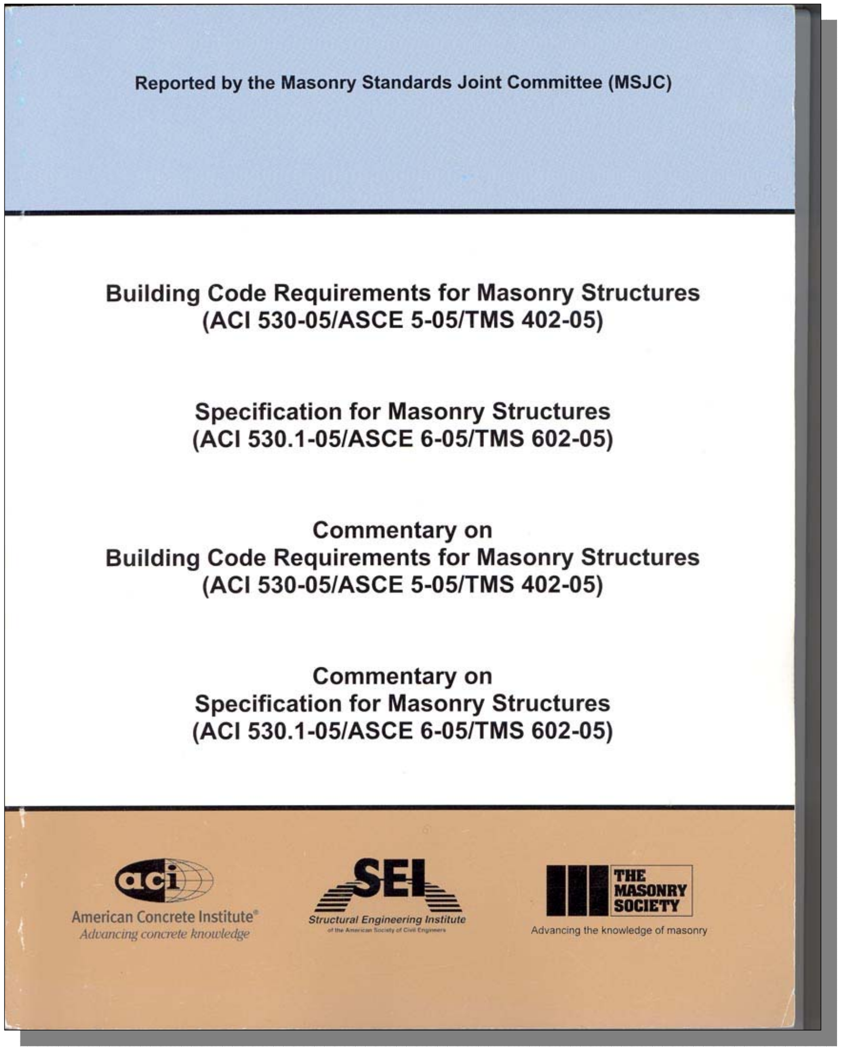


ACI 318-05

- Seismic requirements are primarily found in Chapter 21
- Composite steel and concrete is covered in AISC 341

Structural Concrete

- Special, intermediate, ordinary moment resisting frames
- Special, ordinary shear walls (structural walls)
- Special, intermediate, ordinary precast concrete shear walls
- Special precast concrete moment frames
- Provisions for concrete structure not designed as part of seismic force resisting systems



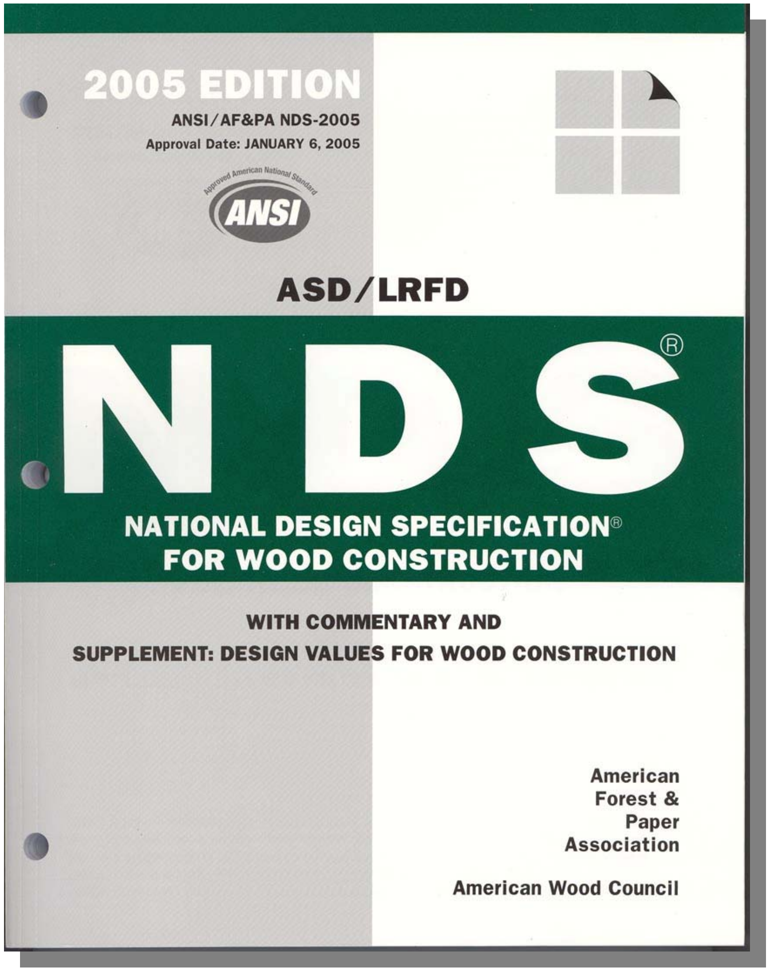
TMS 401-05 ACI 530-05 ASCE 5-05 (MSJC Code)

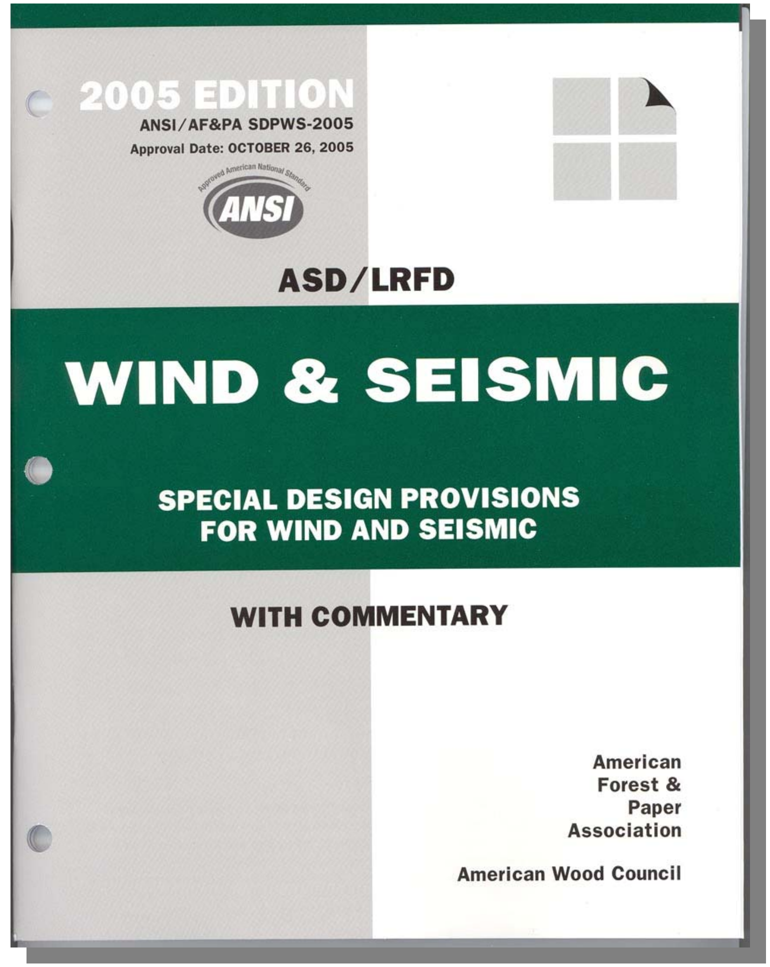
- Mostly incorporated into IBC chapter 21 by transcription as opposed to citation by reference

Masonry

- Five types of masonry shear walls
  - Special, intermediate, ordinary reinforced walls
  - Detailed, ordinary plain walls
- Seismic provisions somewhat buried and convoluted (2008 edition will be better!)
- Prestressed shear walls
- Autoclaved aerated concrete (AAC) masonry

Wood (Timber)


2005 EDITION
ANSI/AF&PA NDS-2005
Approval Date: JANUARY 6, 2005
ANSI
ASD/LRFD
NDS®
NATIONAL DESIGN SPECIFICATION®
FOR WOOD CONSTRUCTION
WITH COMMENTARY AND
SUPPLEMENT: DESIGN VALUES FOR WOOD CONSTRUCTION
American Forest & Paper Association
American Wood Council


2005 EDITION
ANSI/AF&PA SDPWS-2005
Approval Date: OCTOBER 26, 2005
ANSI
ASD/LRFD
WIND & SEISMIC
SPECIAL DESIGN PROVISIONS
FOR WIND AND SEISMIC
WITH COMMENTARY
American Forest & Paper Association
American Wood Council

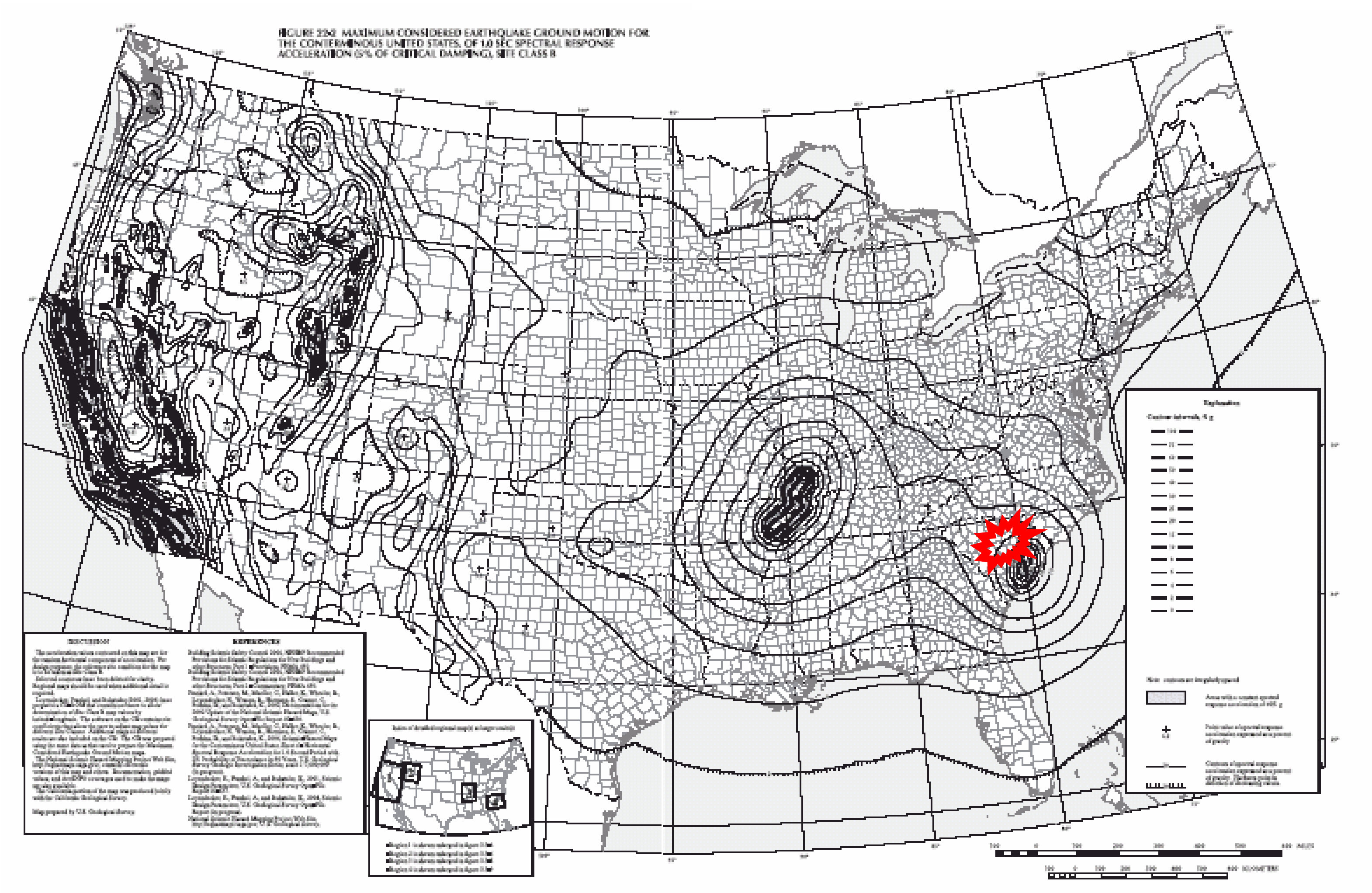
Timber Structures: Seismic Supplement

- Diaphragms and shear walls
- Various sheathing types
- Framing and configuration requirements
- Note that much of this information was formerly included directly in the model building code rather than a design standard.

Structural Standards: Summary

- IBC 2006 cites ASCE 7-05; based on 2003 *NEHRP Recommended Provisions*
- Both IBC and ASCE 7 cite and supplement the 2005 material design standards:
  - AISC for structural steel and composite steel/concrete
  - AISI for cold formed steel
  - ACI for concrete
  - TMS 402 (MSJC) for masonry
  - AF&PA NDS for timber

SEISMIC LOAD ANALYSIS



Topic Objectives

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

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

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

Load Analysis Procedure
(Continued)

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

Occupancy Category (ASCE 7)

I) Low risk occupancy

Agricultural facilities
Temporary facilities
Minor storage facilities

II) Normal hazard occupancy

Any occupancy not described as I, III, IV

III) High hazard occupancy

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

Occupancy Category (ASCE 7, continued)

IV) Essential facilities

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

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

Hazard Maps → Design Ground Motions

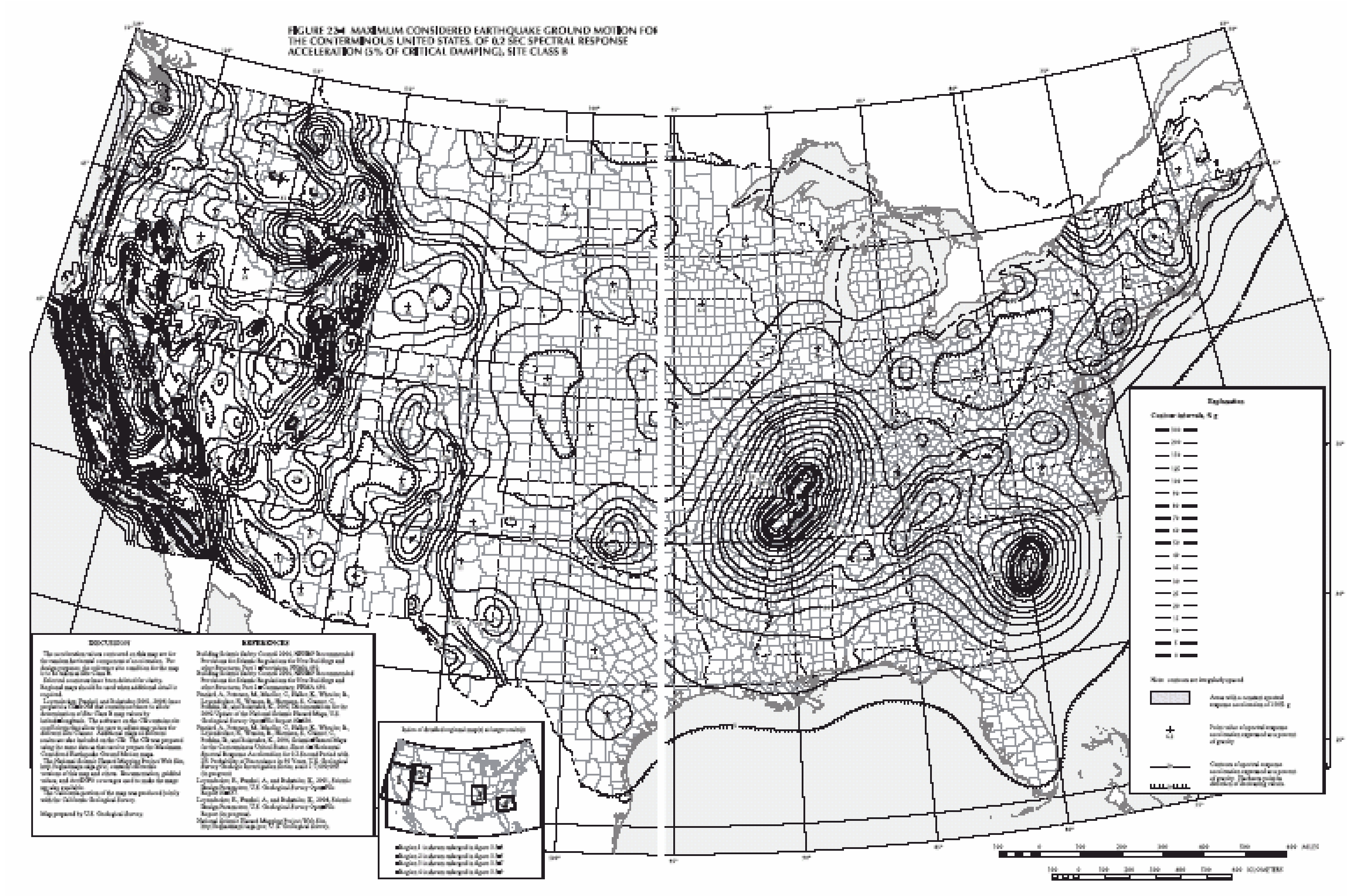
- Provide 5% damped firm rock (Site Class B) spectral accelerations S_s and S_1 or 2% in 50 year probability or 1.5 times deterministic peak in areas of western US

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

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

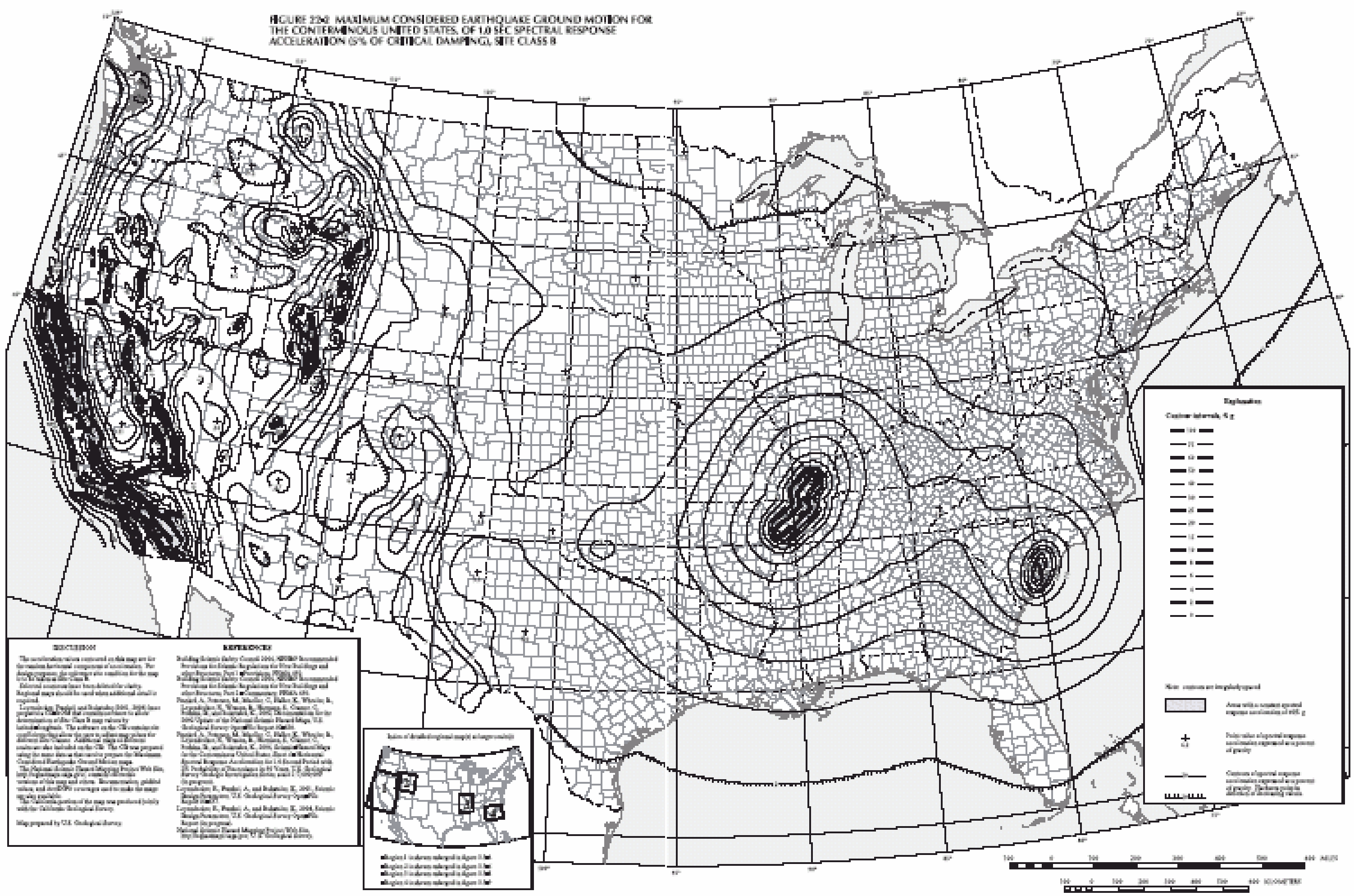
T = 0.2 Spectral Accelerations (S_s) for Conterminous US

(2% in 50 year, 5% damped, Site Class B)



T = 1 Spectral Accelerations (S_1) for Conterminous US

(2% in 50 year, 5% damped, Site Class B)



SITE CLASSES

A Hard rock $v_s > 5000$ ft/sec

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

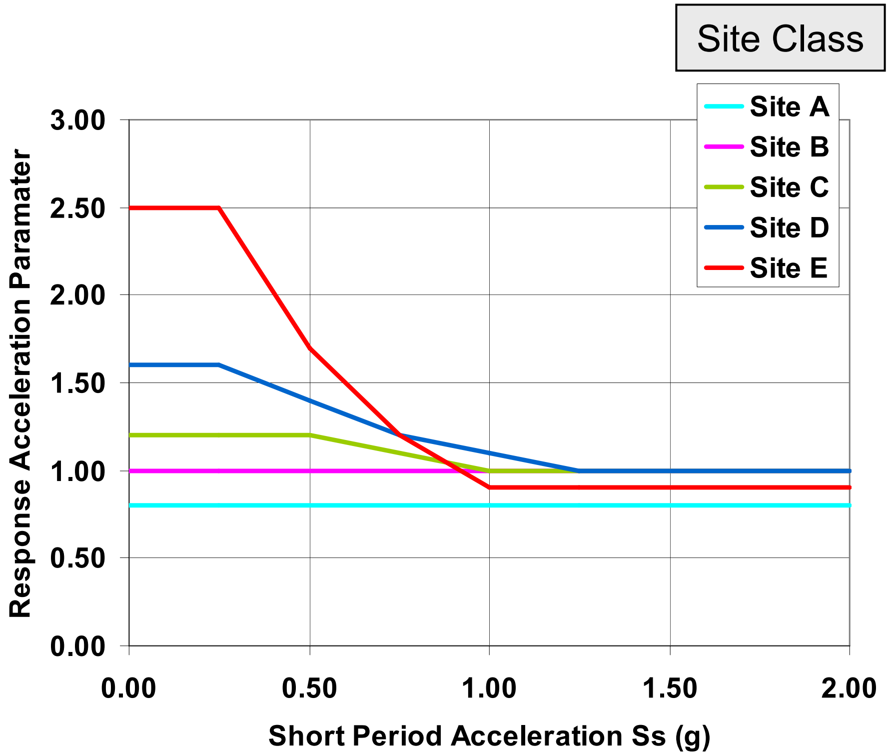
C Very dense soil or soft rock: $1200 < v_s < 2500$ ft/sec

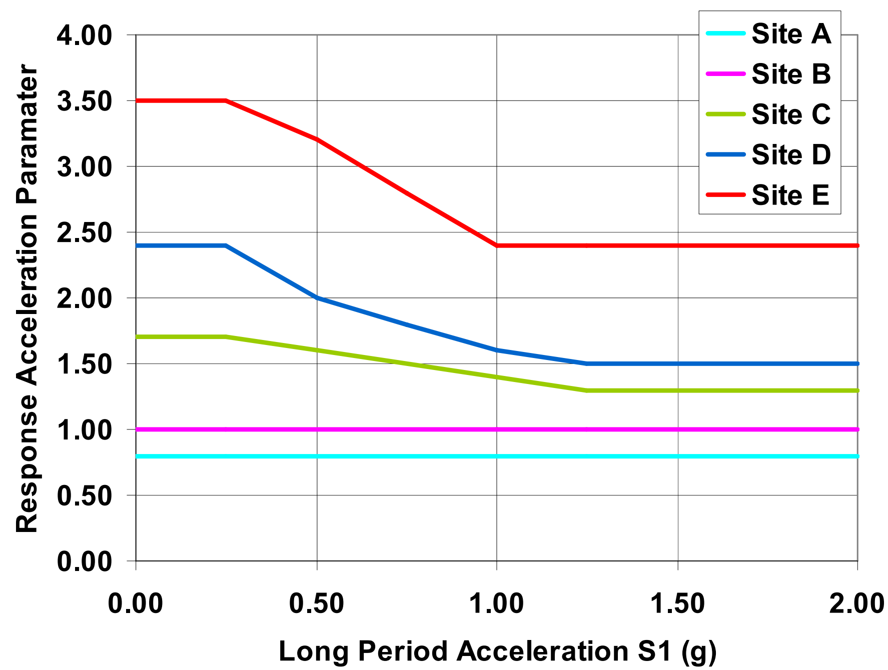
D Stiff soil : $600 < v_s < 1200$ ft/sec

E $V_s < 600$ ft/sec

F Site-specific requirements

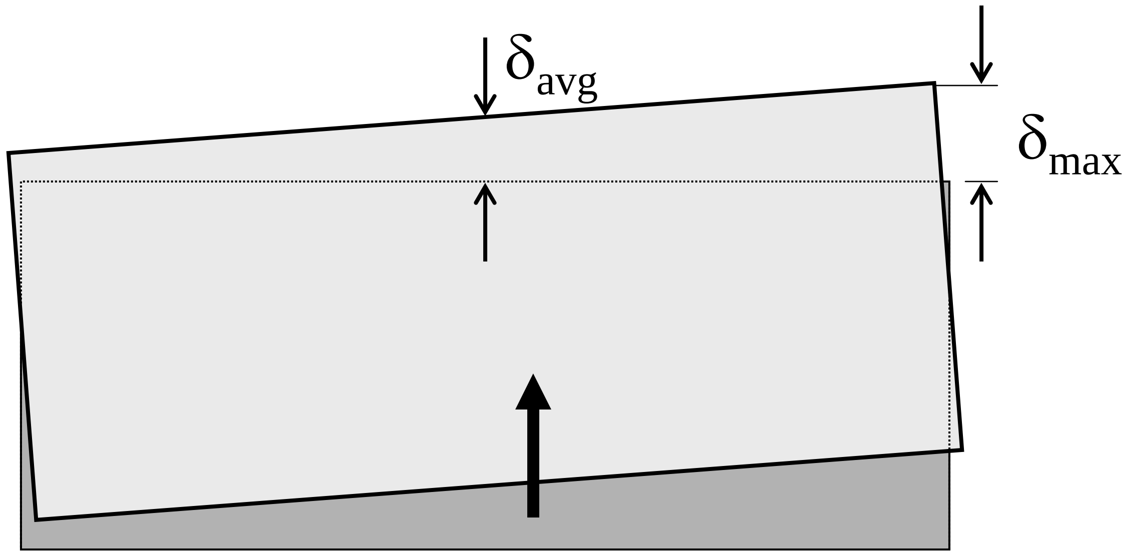
NEHRP Site Amplification for Site Classes A through E





Horizontal Structural Irregularities

1a) and 1b) Torsional Irregularity



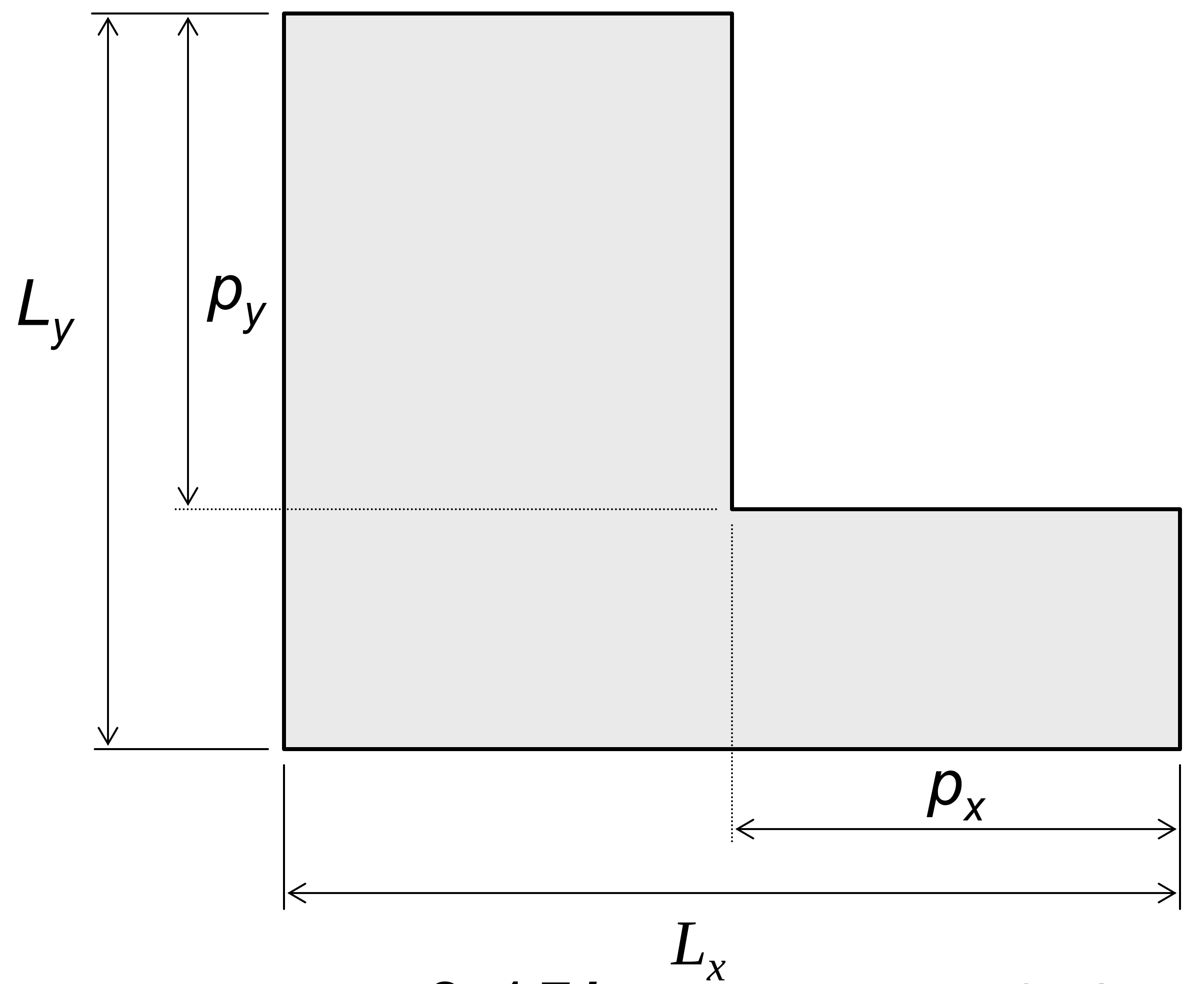
$$\delta_{max} < 1.2\delta_{avg}$$ No irregularity

$$1.2\delta_{avg} \leq \delta_{max} \leq 1.4\delta_{avg}$$ Irregularity

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

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

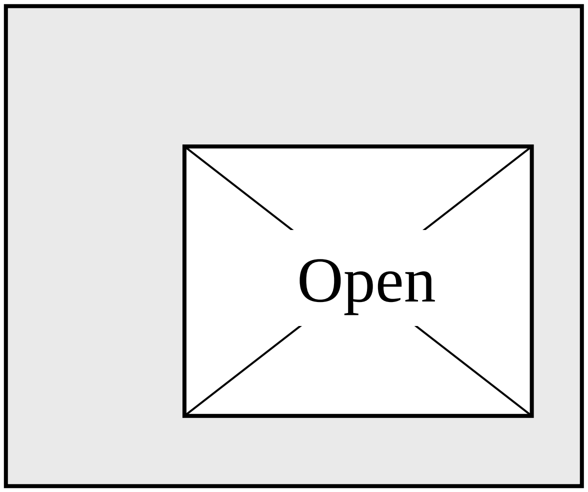
Horizontal Structural Irregularities
2) Re-entrant Corner Irregularity

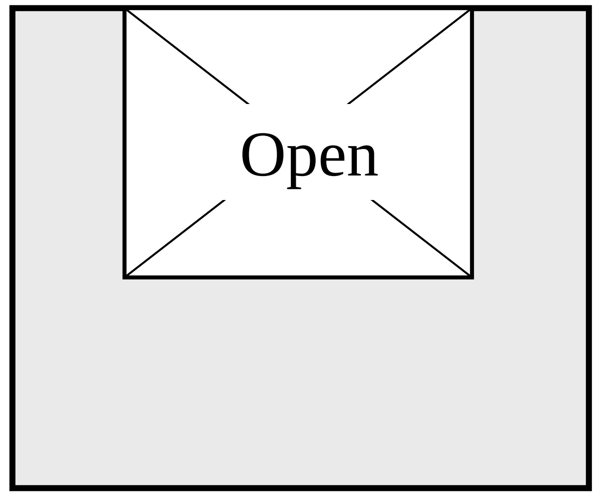


Irregularity exists if $p_y > 0.15L_y$ and $p_x > 0.15L_x$

Horizontal Structural Irregularities

3) Diaphragm Discontinuity Irregularity

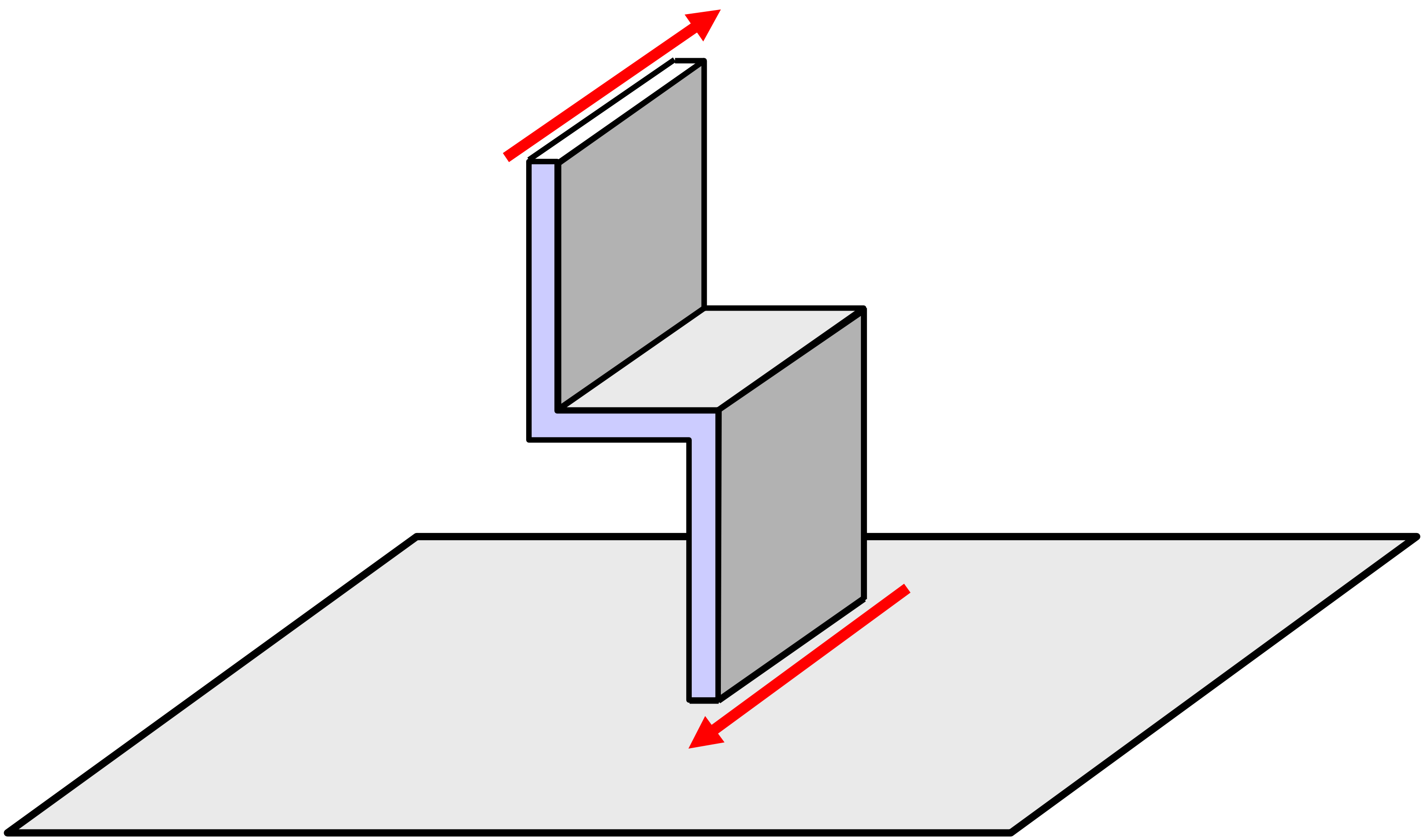




Irregularity exists if open area > 0.5 times floor area OR if effective diaphragm stiffness varies by more than 50% from one story to the next.

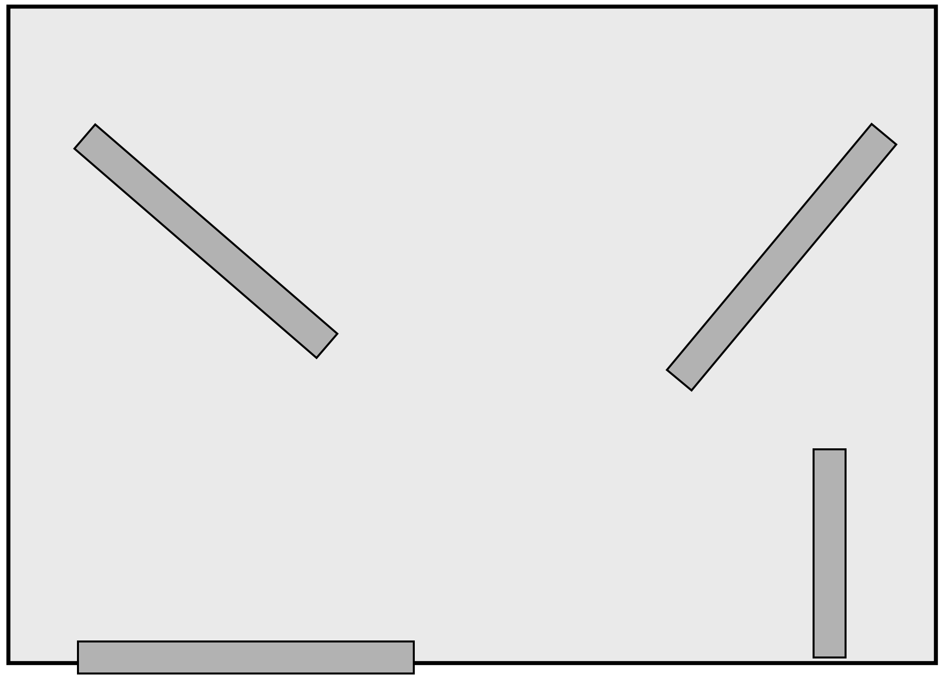
Horizontal Structural Irregularities

4) Out of Plane Offsets



Horizontal Structural Irregularities

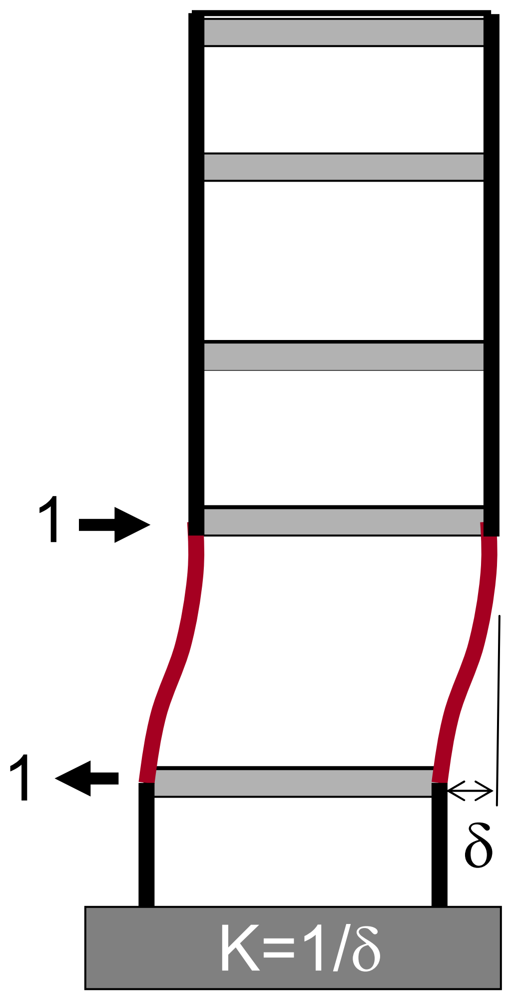
5) Nonparallel Systems Irregularity



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

Vertical Structural Irregularities

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



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

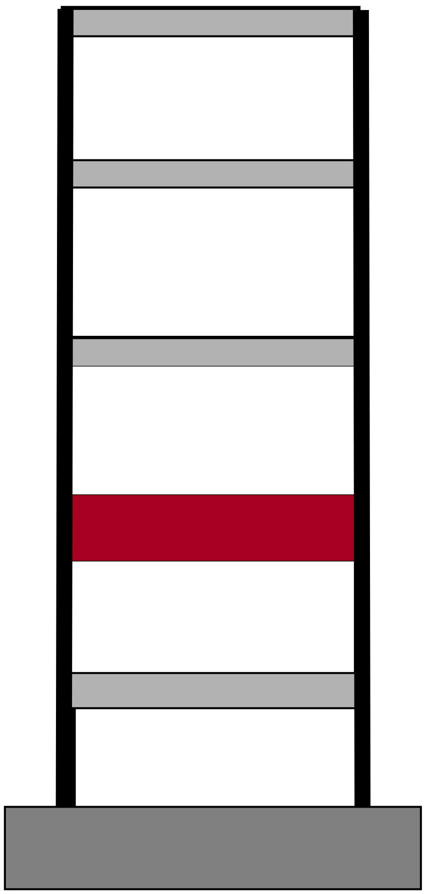
An extreme irregularity (1b) exists if stiffness of any story is less than 60% of the stiffness of the story above or less than 70% of the average stiffness of the three stories above.

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

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

Vertical Structural Irregularities

2) Weight (Mass) Irregularity

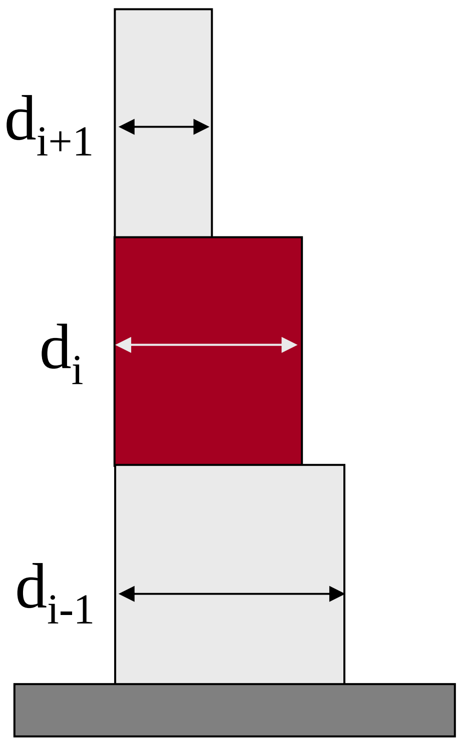


Irregularity exists if the effective mass of any story is more than 150% of the effective mass of an adjacent story.

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

Vertical Structural Irregularities

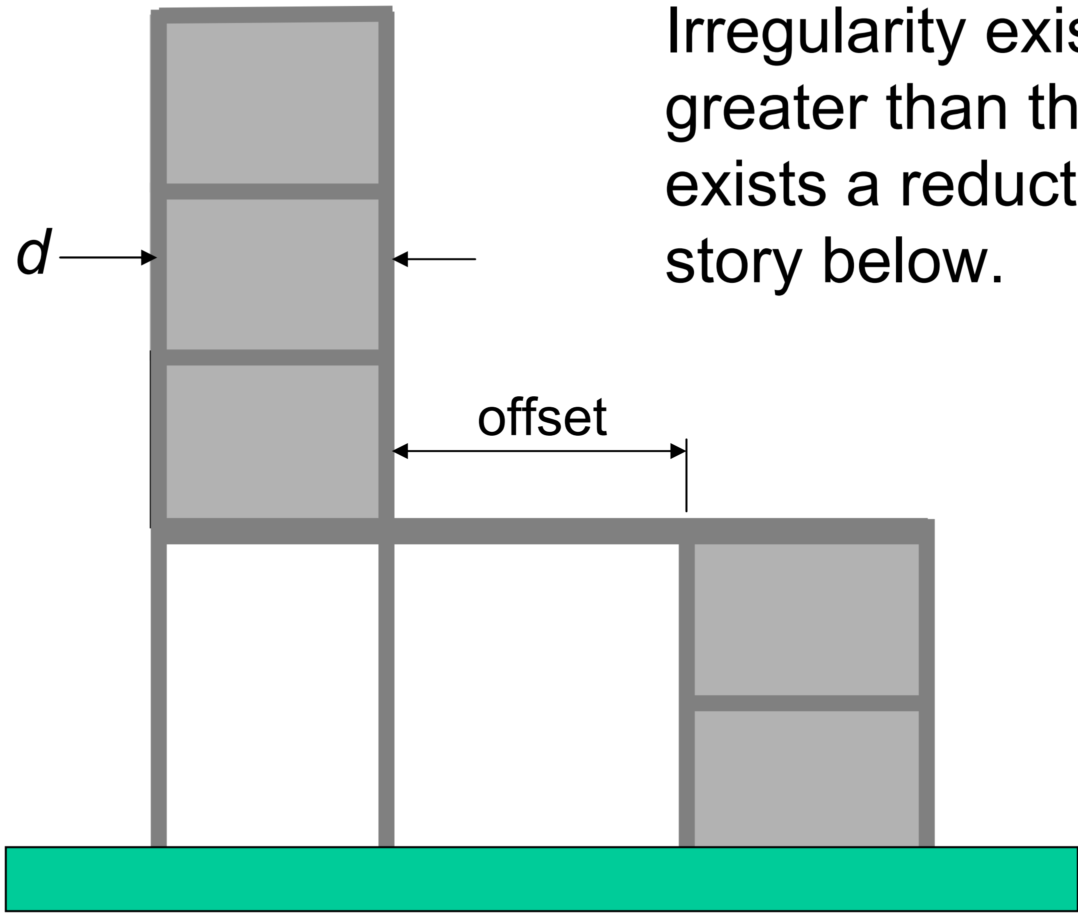
3) Vertical Geometric Irregularity



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

Vertical Structural Irregularities

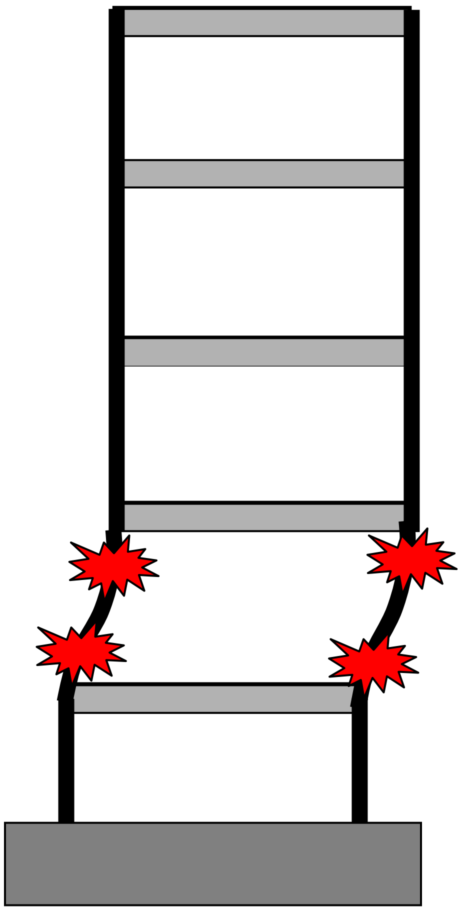
4) In-Plane Discontinuity Irregularity



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

Vertical Structural Irregularities

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



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

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

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

Structural Systems

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

System Parameters:
Response modification coefficient = R
System overstrength parameter = Ω_o
Deflection amplification factor = C_d
Height limitation = by SDC

Structural Systems

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

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D^d	E^d	F^e
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	$2\frac{1}{2}$	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	$2\frac{1}{2}$	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	$2\frac{1}{2}$	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{2}$	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	$2\frac{1}{2}$	4	NL	NL	40^k	40^k	40^k
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	$2\frac{1}{2}$	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	$2\frac{1}{2}$	$3\frac{1}{2}$	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	$3\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	$6\frac{1}{2}$	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	$2\frac{1}{2}$	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.1.4.2, and 14.5	4	2	$3\frac{1}{2}$	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100

Bearing Wall

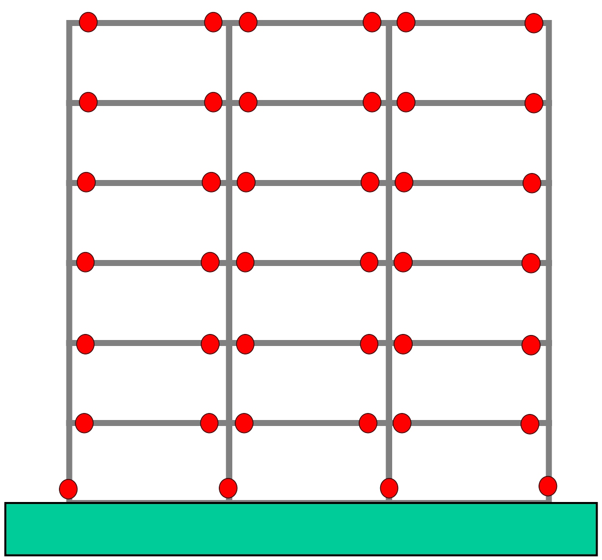
- Any metal or wood stud wall that supports more than 100 lbs/ft of vertical load in addition to its own weight
- Any concrete or masonry wall that supports more than 200 lbs/ft of vertical load in addition to its own weight

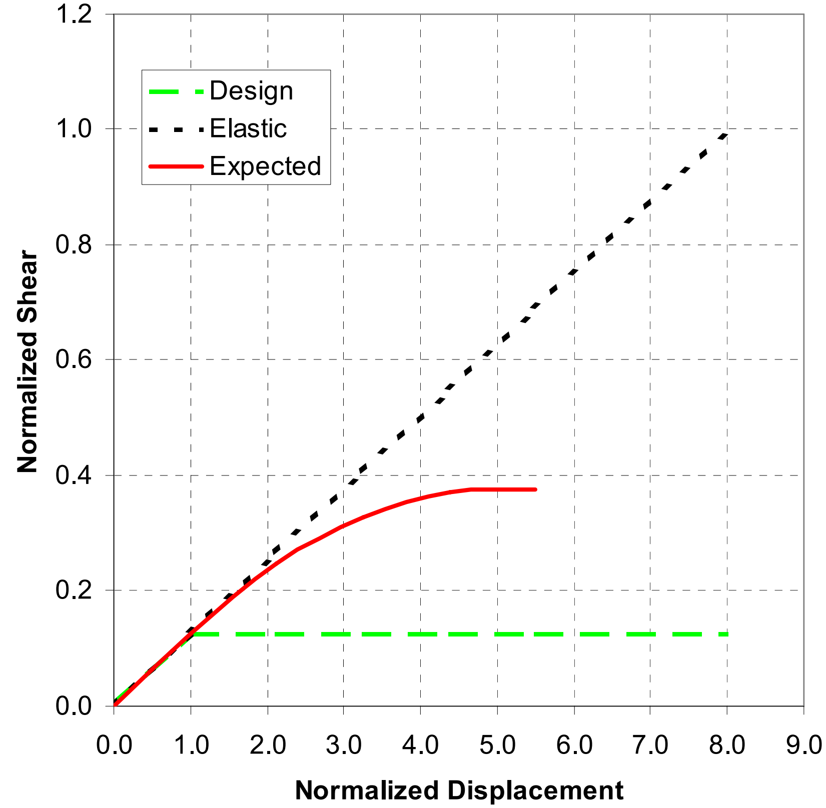
It appears that almost ANY concrete or masonry wall would be classified as a bearing wall!

Special Steel Moment Frame

R	8
C_d	5.5
Ω_o	3

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



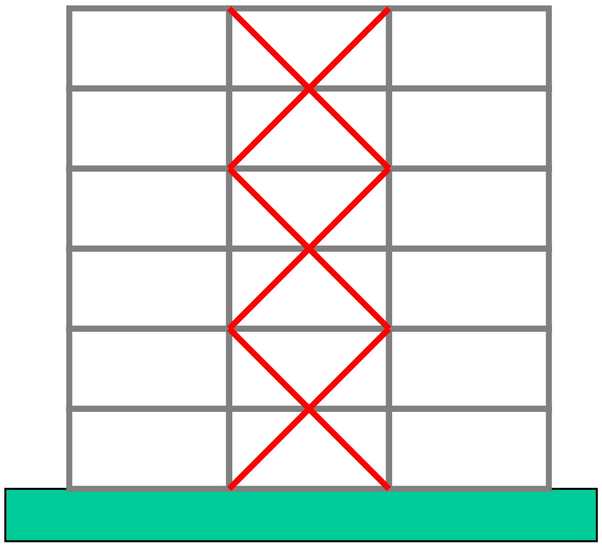


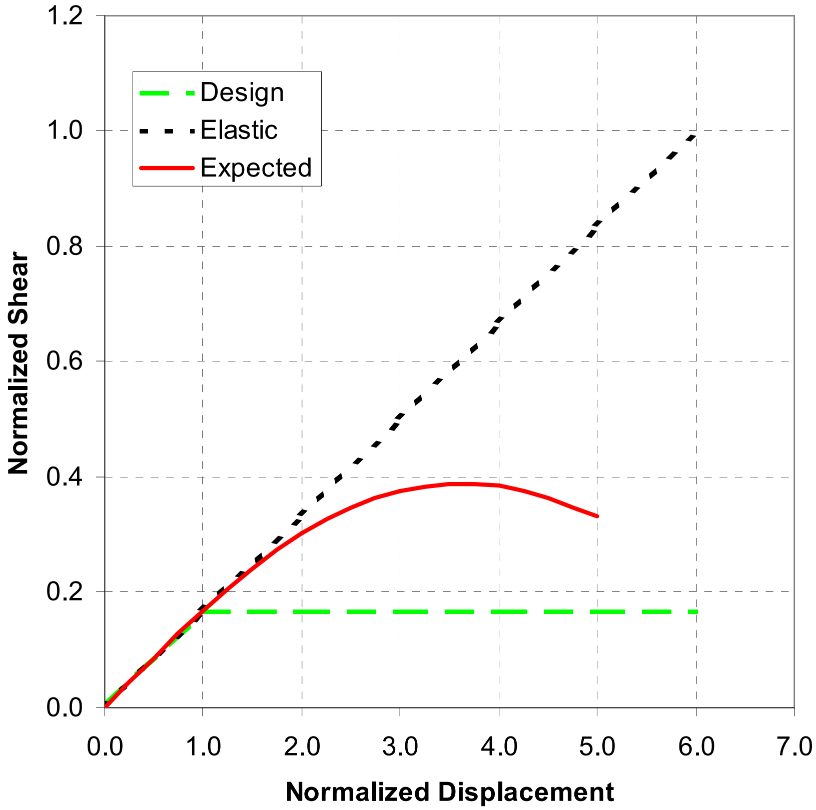
Advantages:
Architectural simplicity, relatively low base shear
Disadvantages:
Drift control, connection cost, connection testing

Special Steel Concentrically Braced Frame

R	6
C_d	5
Ω_o	2

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



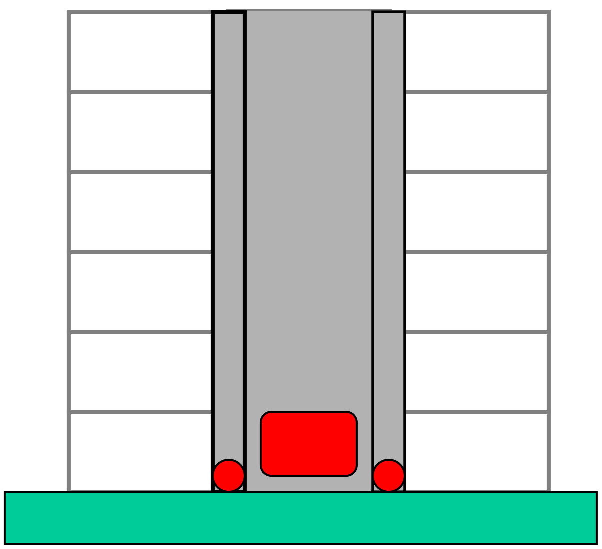


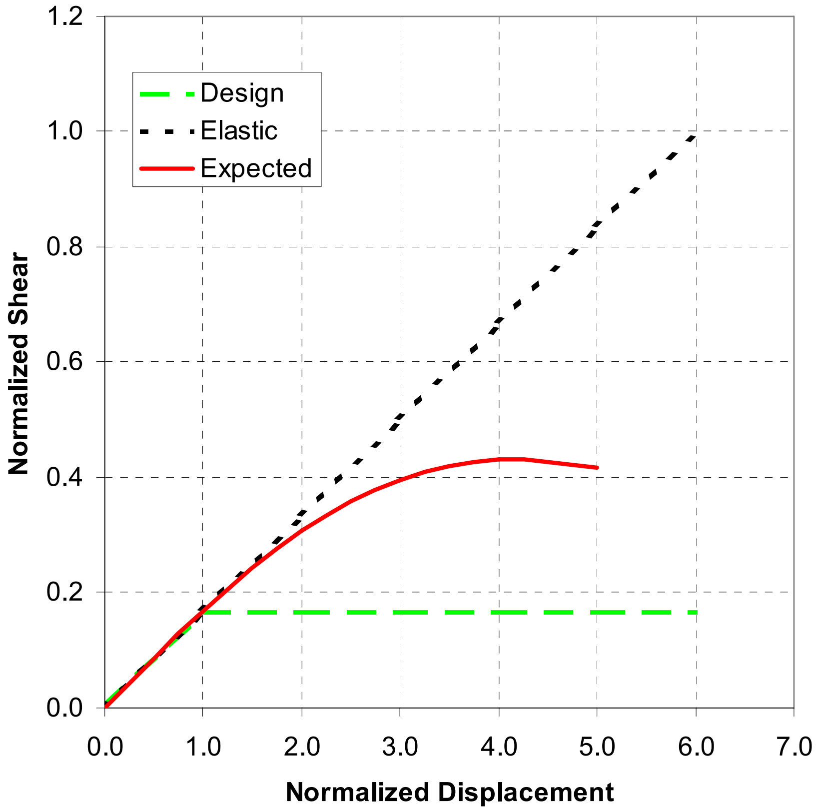
Advantages:
Lower drift, simple field connections
Disadvantages:
Higher base shear, high foundation forces, height limitations, architectural limitations

Special Reinforced Concrete Shear Wall

R	6
C_d	5
Ω_o	2.5

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





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

Response Modification Factor *R*

Accounts for:

- Ductility
- Overstrength
- Redundancy
- Damping
- Past behavior

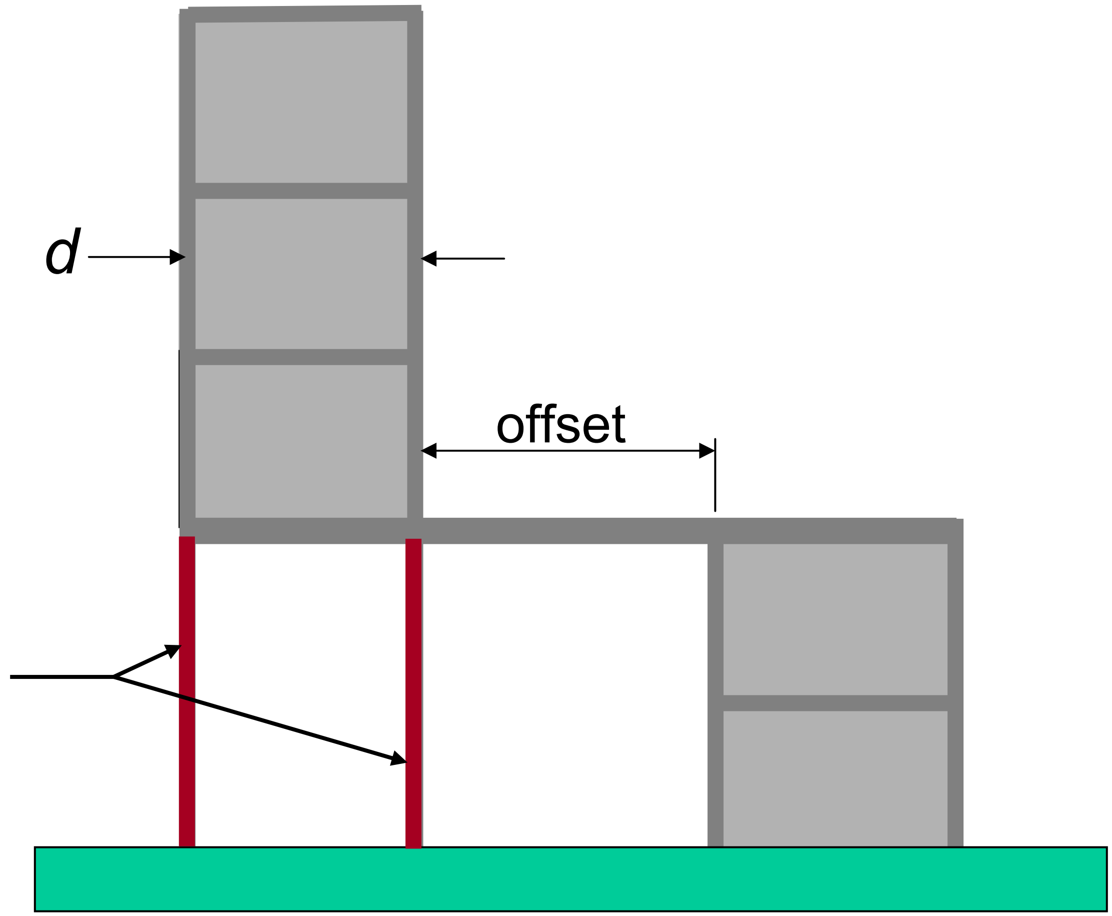
Maximum = 8

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

Minimum = 1.5 (exclusive of cantilever systems)

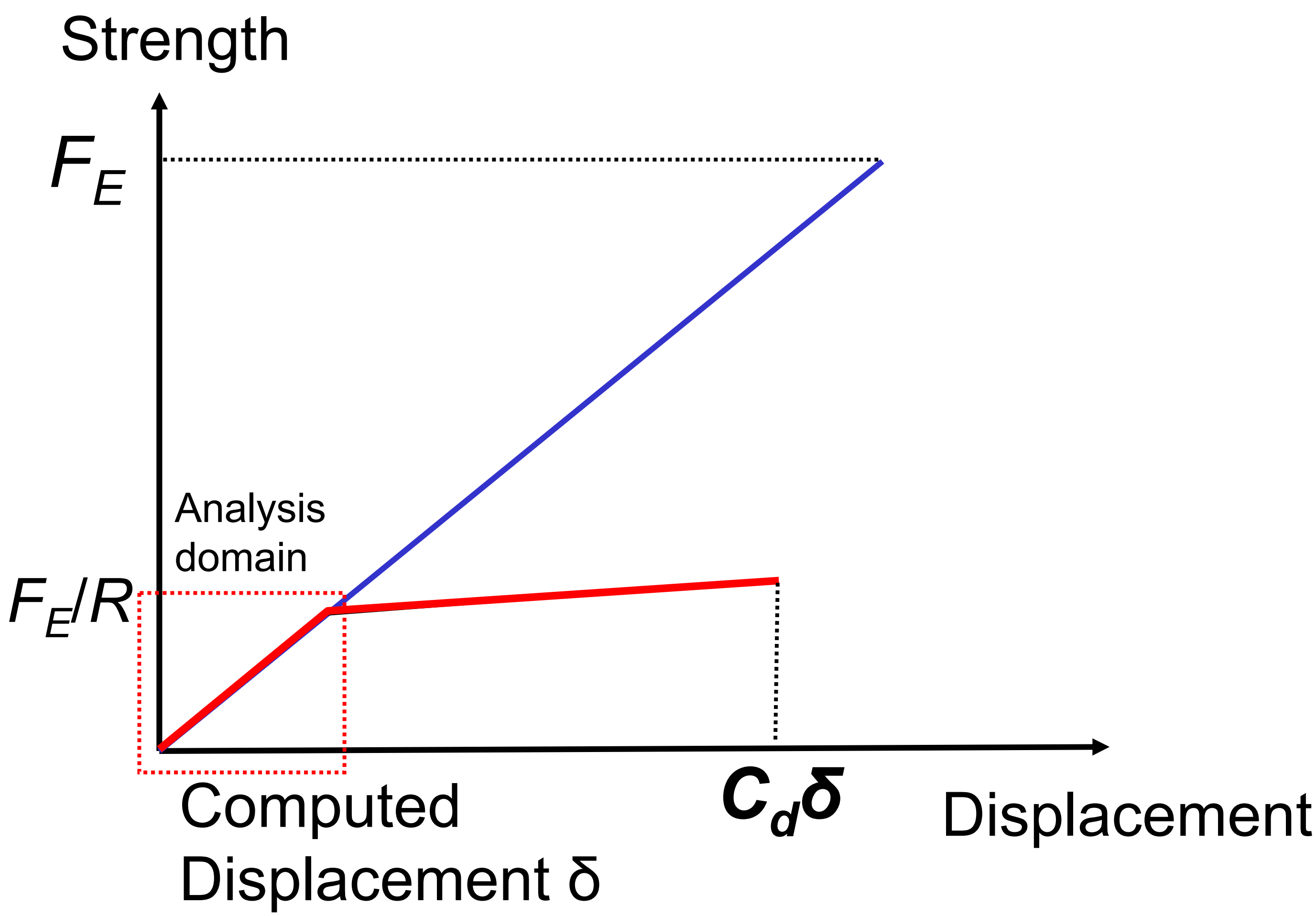
Ordinary plain masonry shear walls

Overstrength Factor Ω_o



Elements must be designed using load combination with factor Ω_o

Deflection Amplification Factor C_d



Diaphragm Flexibility

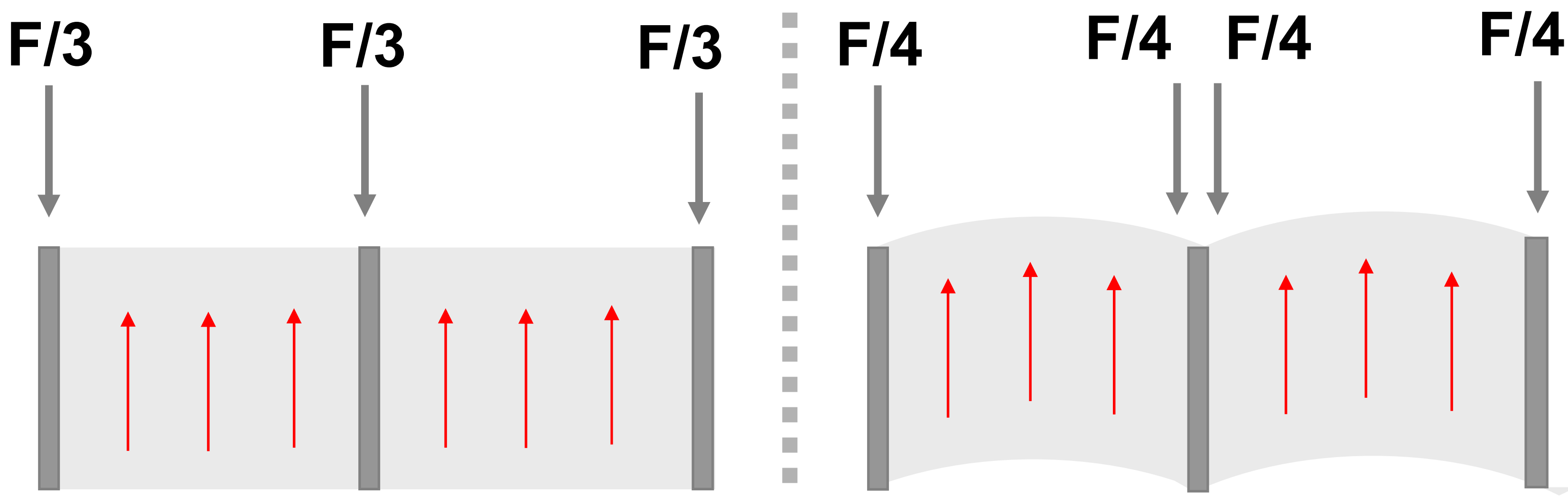
Diaphragms must be considered as semi-rigid unless they can be classified as **FLEXIBLE** or **RIGID**.

- Untopped steel decking and untopped wood structural panels are considered **FLEXIBLE** if the vertical seismic force resisting systems are steel or composite braced frames or are shear walls.

- Diaphragms in one- and two-family residential buildings may be considered **FLEXIBLE**.

- Concrete slab or concrete filled metal deck diaphragms are considered **RIGID** if the width to depth ratio of the diaphragm is less than 3 and if no horizontal irregularities exist.

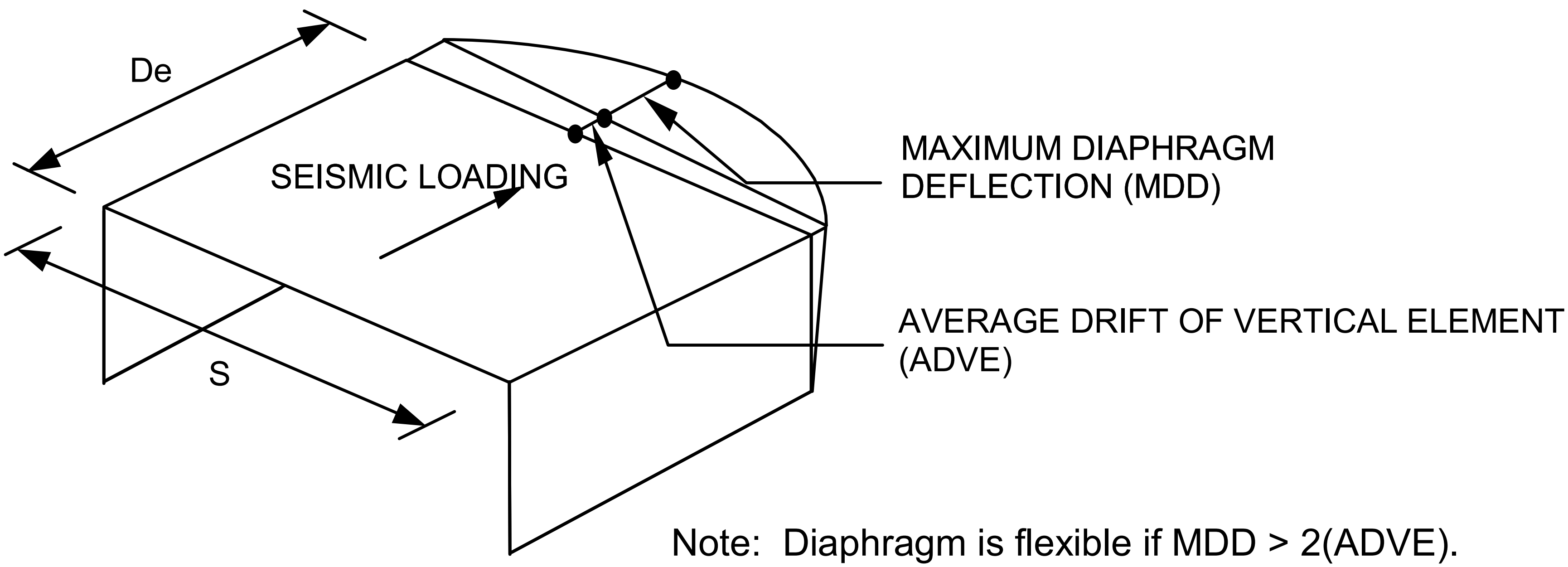
Rigid vs Flexible Diaphragms



RIGID
Center Wall Shear = F/3

FLEXIBLE
Center Wall Shear = F/2

Diaphragm Flexibility



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

Diagram taken from ASCE 7-05

Importance Factors

SUG	Importance Factor
IV	1.50
III	1.25
I, II	1.00

Using ASCE 7-05 Use Groups

Seismic Design Category = Seismic Use Group + Design Ground Motion

Based on SHORT PERIOD acceleration

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

*Using ASCE 7-05 Use Groups

Seismic Design Category

Based on LONG PERIOD acceleration

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

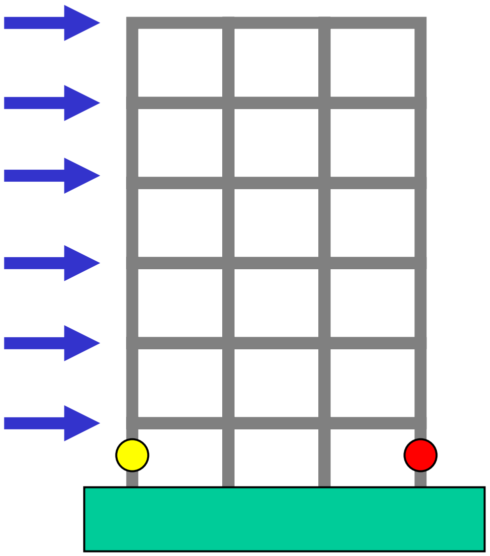
Value of S_1	Seismic Use Group*		
	I, II	III	IV
$S_1 > 0.75g$	E	E	F

*Using ASCE 7-05 Use Groups

Basic Load Combinations (involving earthquake)

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

$$0.9D + 1.0E$$



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

Combination of Load Effects

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

$$E = E_h \pm E_v$$

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

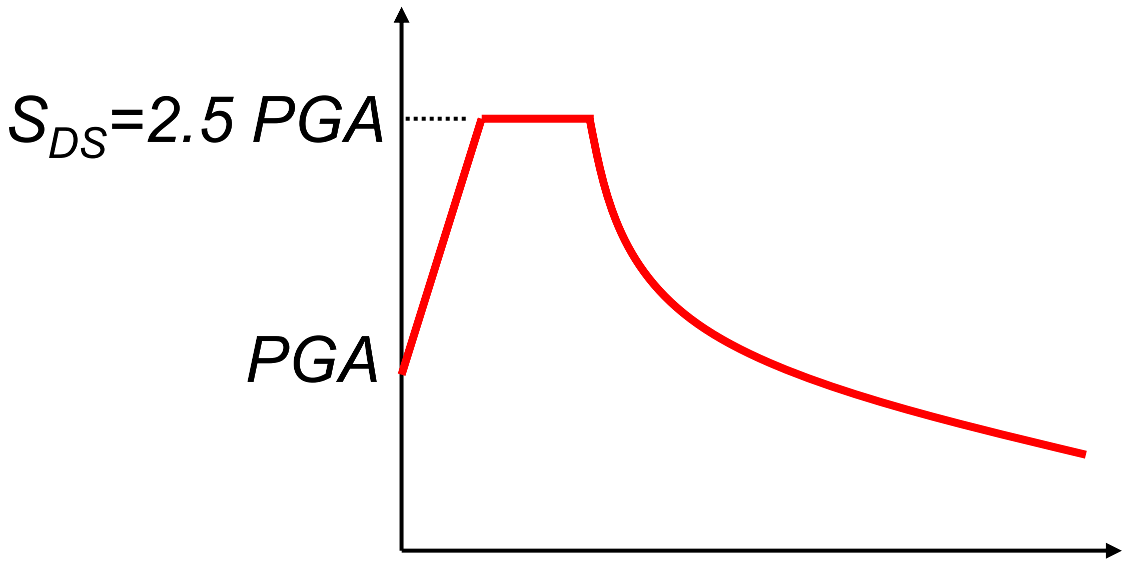
Resulting load combinations (from this and previous slide)

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

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

Note: See ASCE 7 for combinations including hydrostatic load

Vertical Accelerations are Included in the Load Combinations



Vertical acceleration = 0.2(2.5) = 0.5 PGA

Combination of Load Effects (including overstrength factor)

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

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

Resulting load combinations (from this and previous slide)

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

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

Note: See ASCE 7 for combinations including hydrostatic load

Redundancy Factor ρ

Cases where ρ = 1.0

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

Redundancy Factor ρ

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

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

OR

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

Otherwise ρ = 1.3

Redundancy Factor ρ

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

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

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

Redundancy Factor ρ

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

Shear Walls

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

Cantilever Column

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

Required Methods of Analysis

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

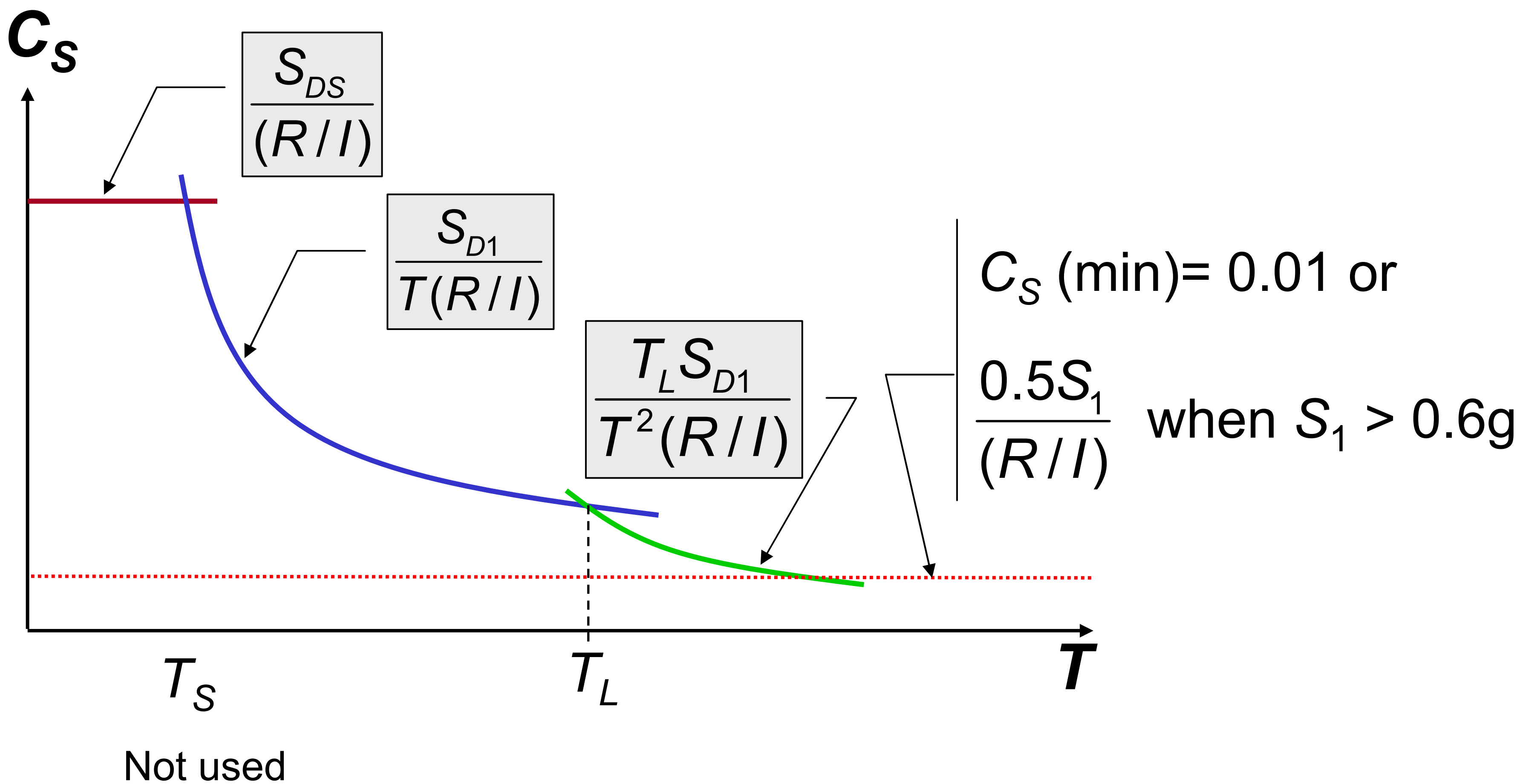
Any structure with $T > 3.5\ T_s$

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

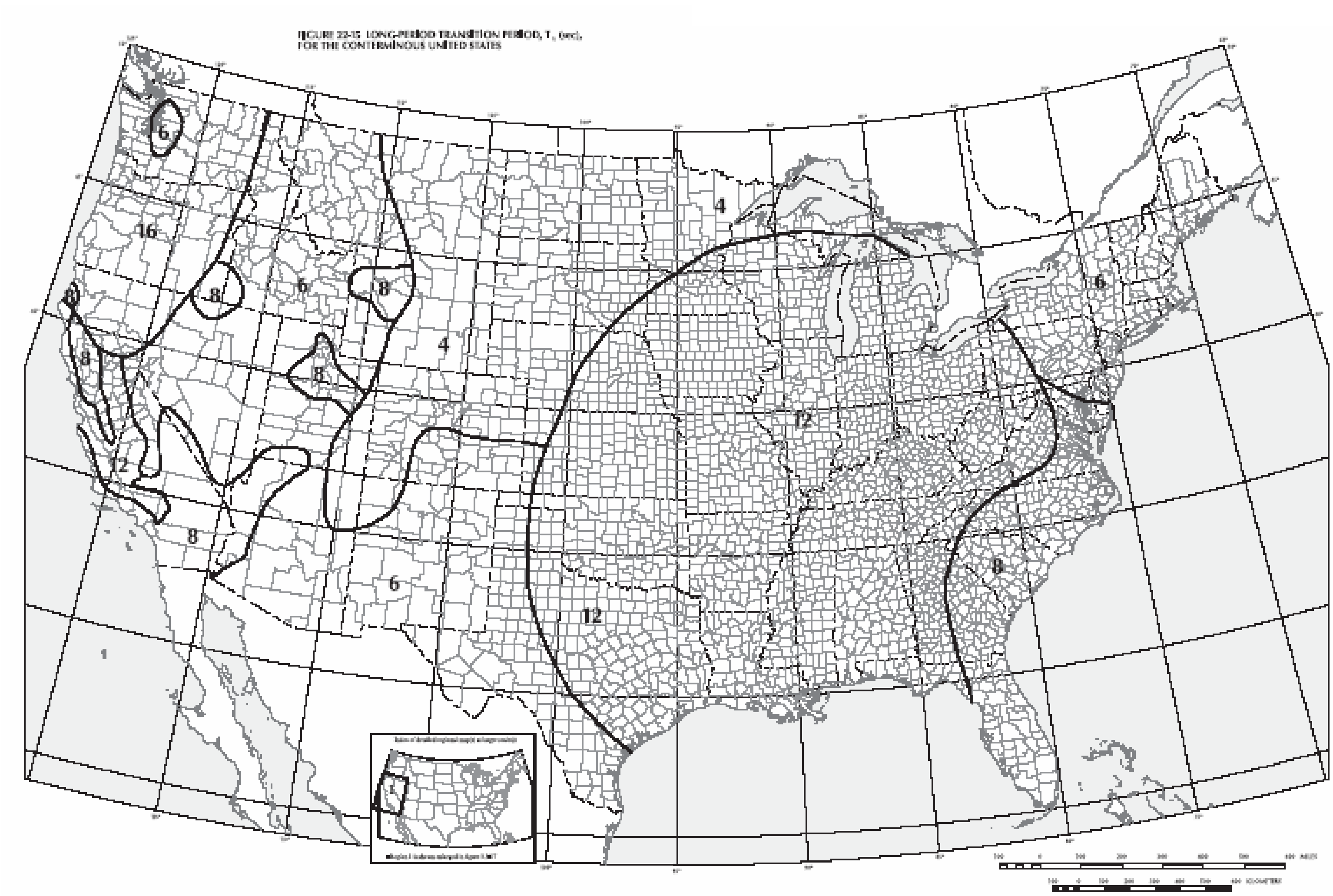
When the ELF procedure is not allowed, analysis must be performed by the response spectrum analysis procedure or by the linear (or nonlinear) response history analysis procedure.

Equivalent Lateral Force Procedure

Determine Base Shear: $V = C_S W$



Transition Periods for Conterminous United States



Effective Seismic Weight *W*

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

Approximate Periods of Vibration

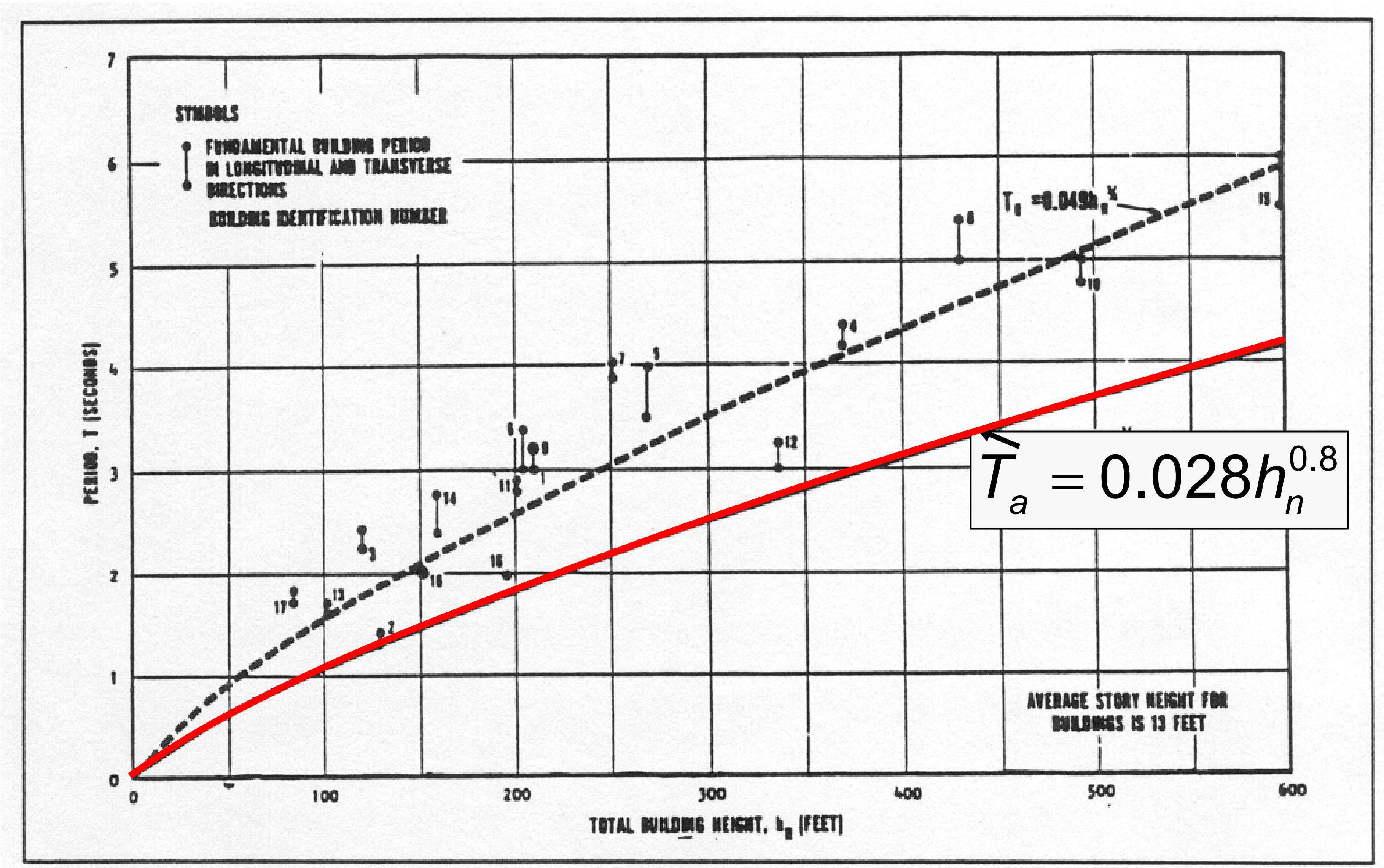
$$\boxed{T_a = C_t h_n^x}$$

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

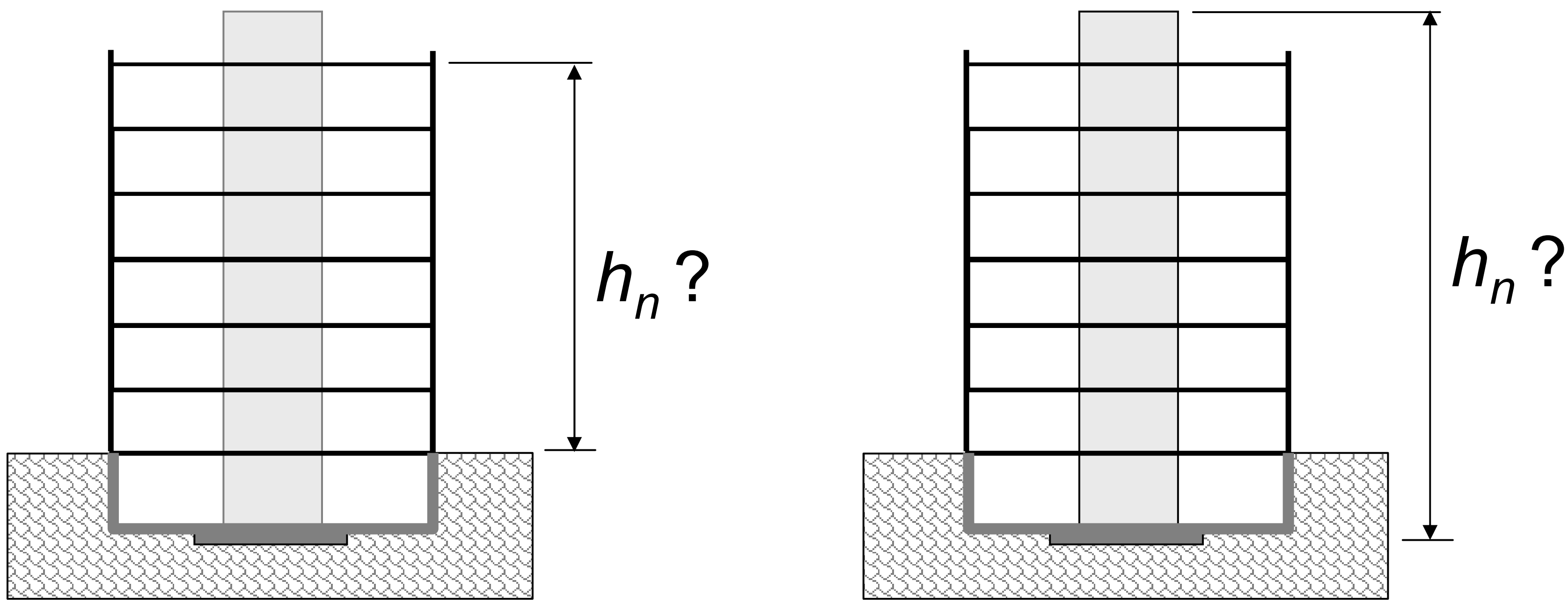
$$\boxed{T_a = 0.1N}$$

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

Empirical Data for Determination of Approximate Period for Steel Moment Frames



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



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

Adjustment Factor on Approximate Period

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

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

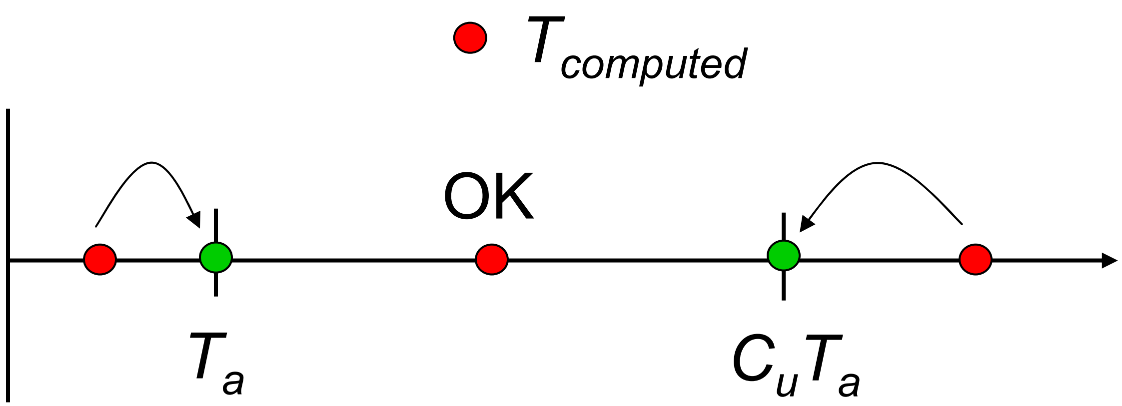
Applicable **ONLY** if $T_{computed}$ comes from a “properly substantiated analysis.”

Decisions Regarding Appropriate Period to Use

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

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

if $T_{computed} < T_a$ use T_a

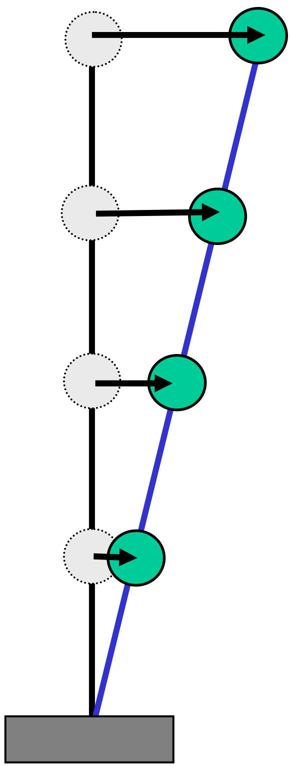


Distribution of Forces along Height

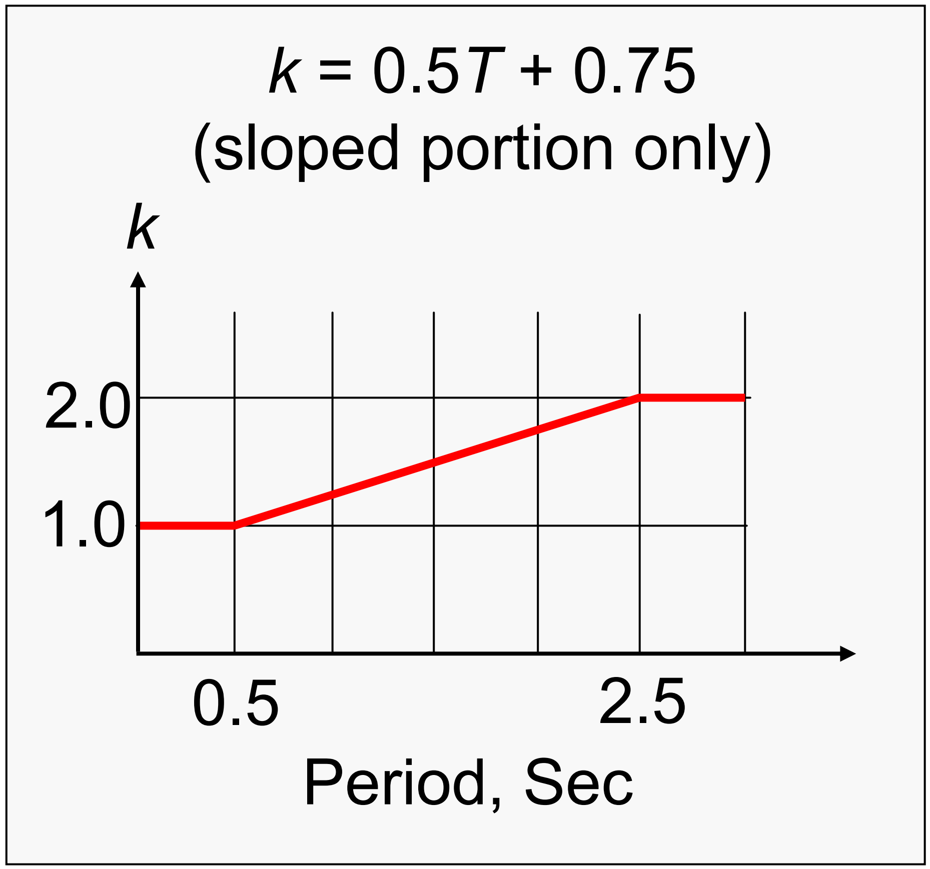
$$F_x = C_{vx} V$$

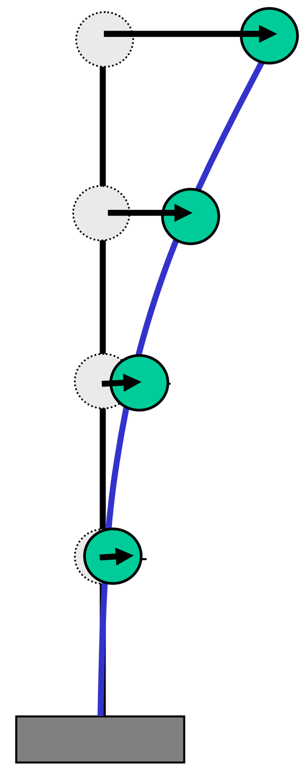
$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

k accounts for Higher Mode Effects



$k = 1$





$k = 2$

Overturning

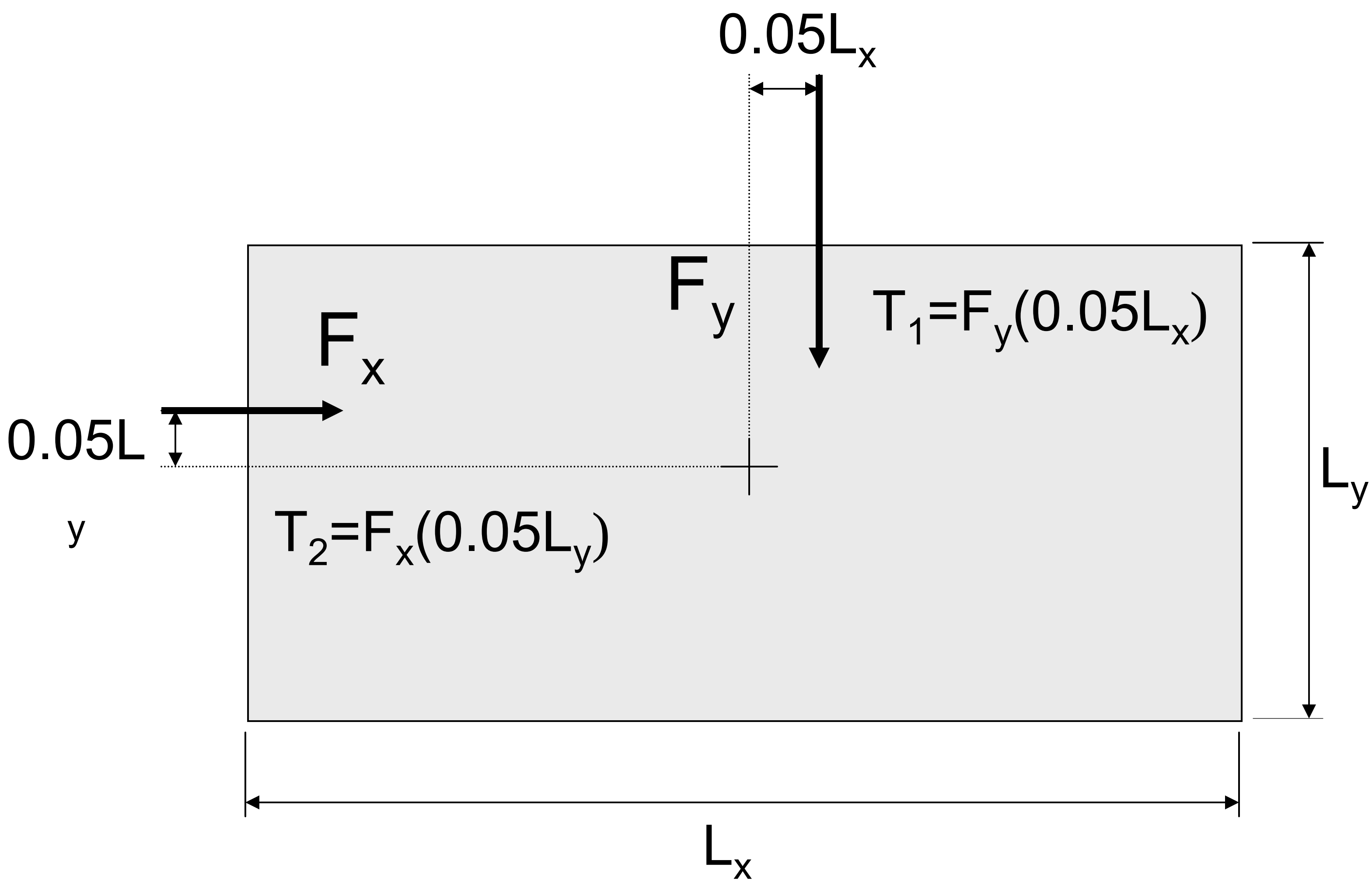
The 2003 *NEHRP Recommended Provisions* and ASCE 7-05 allow a 25% reduction at the foundation only.

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

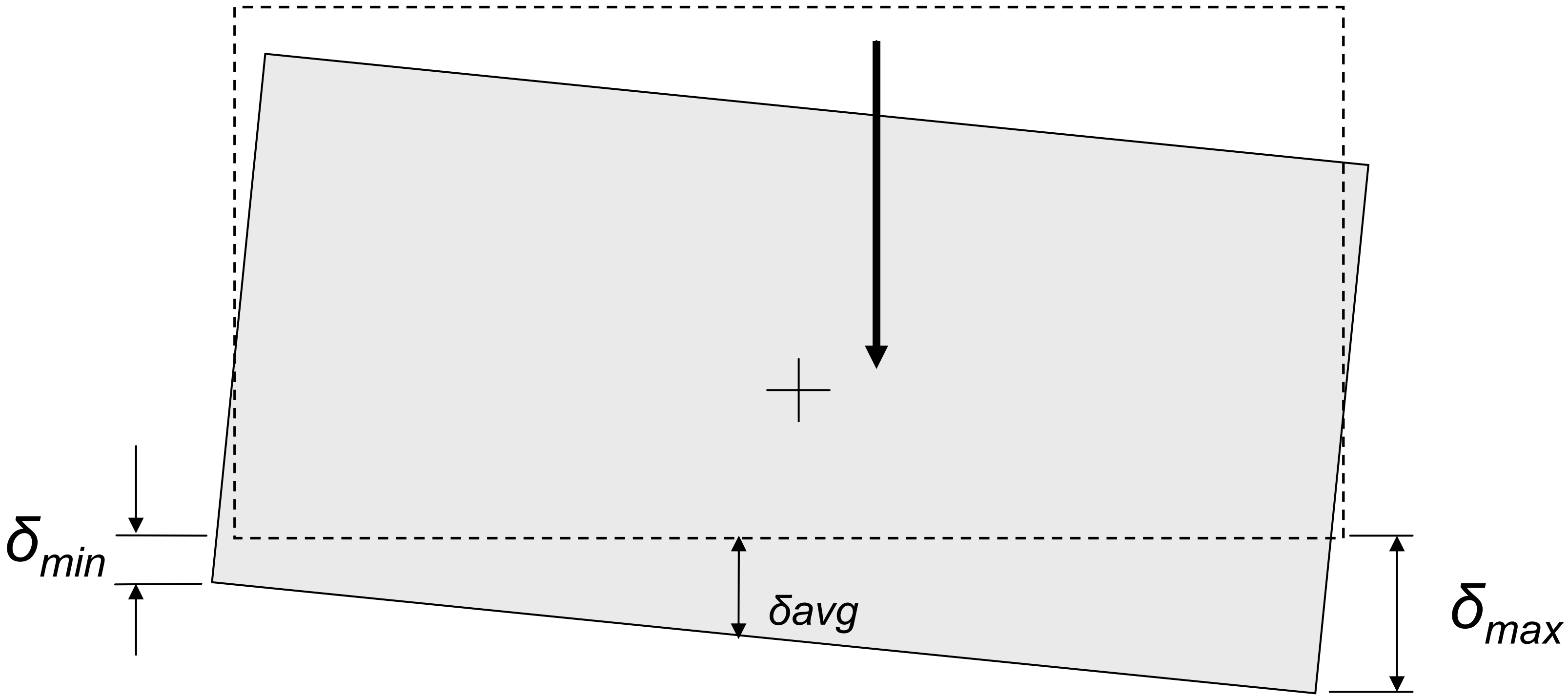
Torsional Effects

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

Accidental Torsion



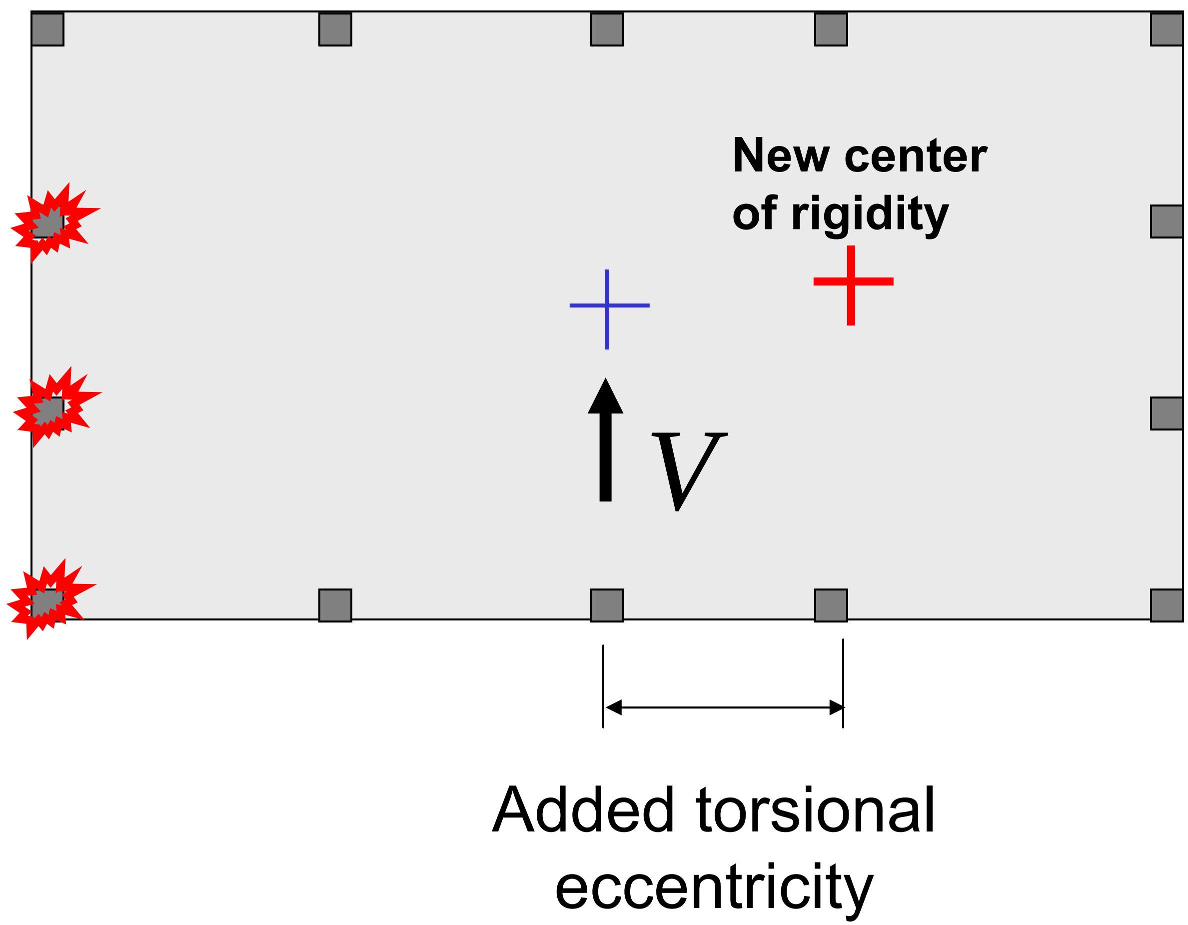
Amplification of Accidental Torsion



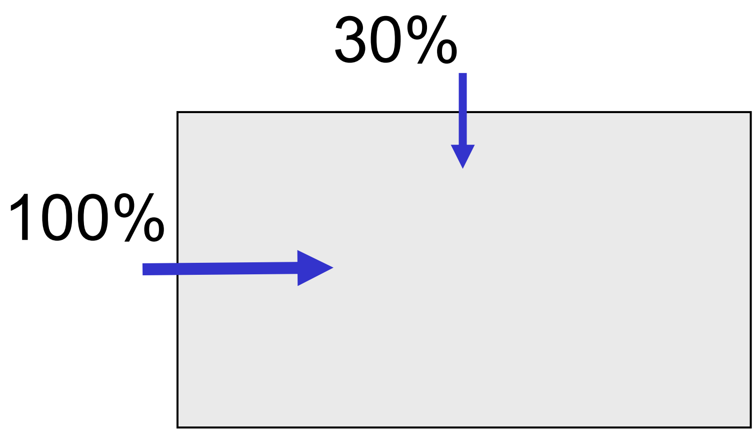
$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{avg}} \right)^2$$

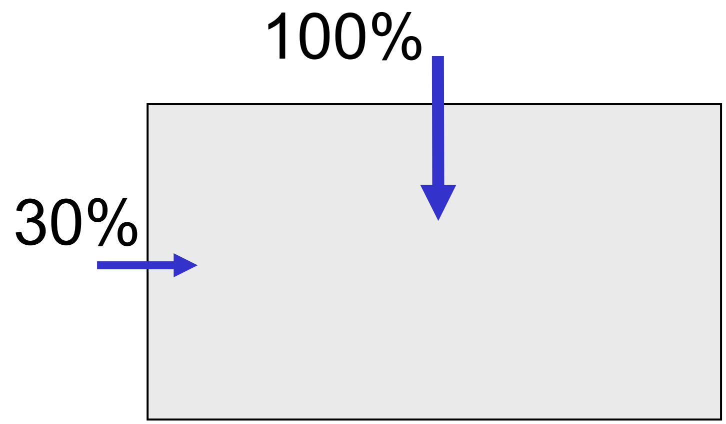
Reason for Amplifying Accidental Torsion

Damage



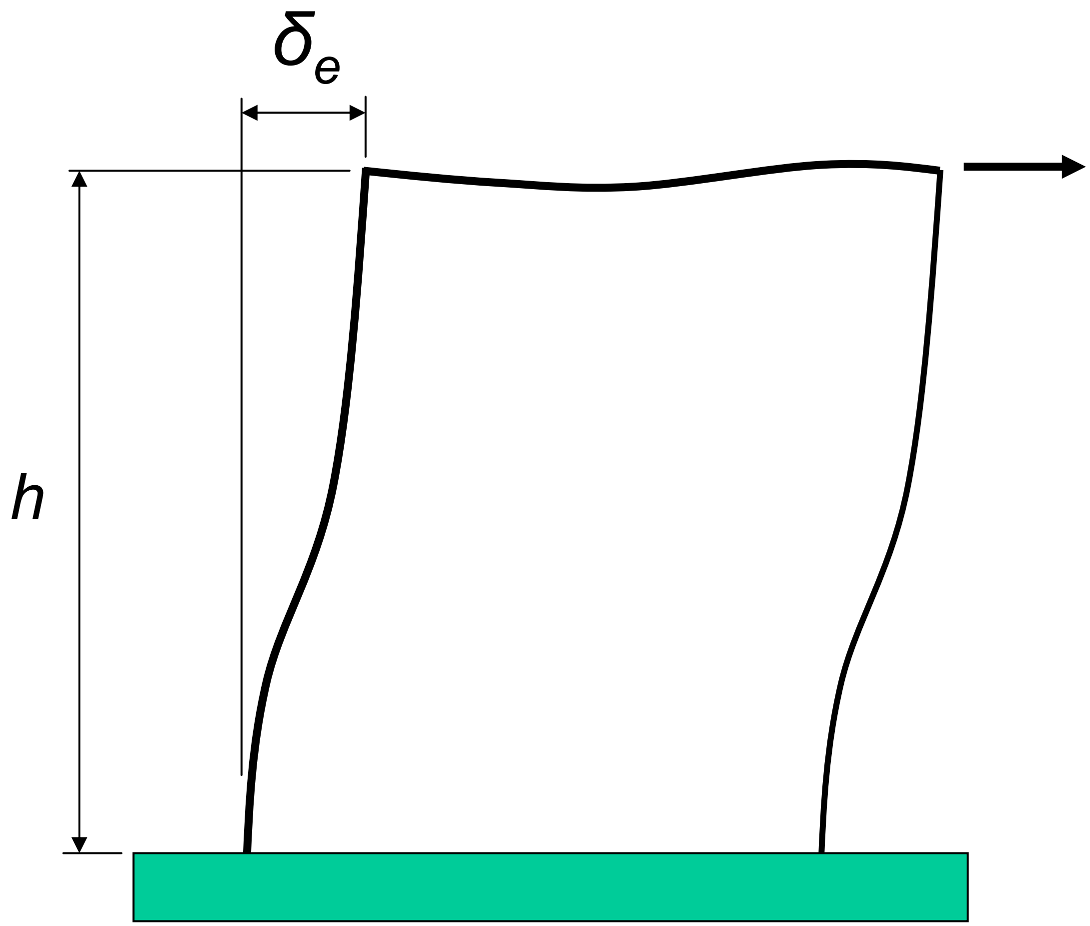
Orthogonal Load Effects





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

Story Drift



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

Drift reported by analysis with strength level forces:

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

Amplified drift:

$$\Delta = C_d \Delta_e$$

Note: Drift computed at center of mass of story

Drift Limits

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

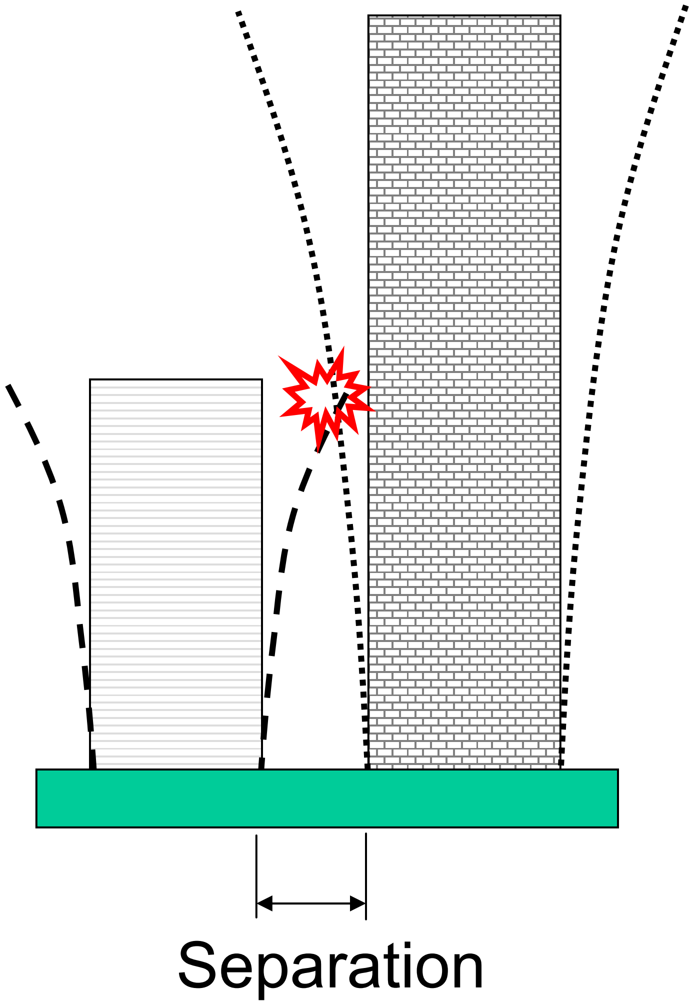
* For moment frames in SDC D, E, and F drift shall not exceed tabulated values divided by ρ.

Story Drift (continued)

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

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

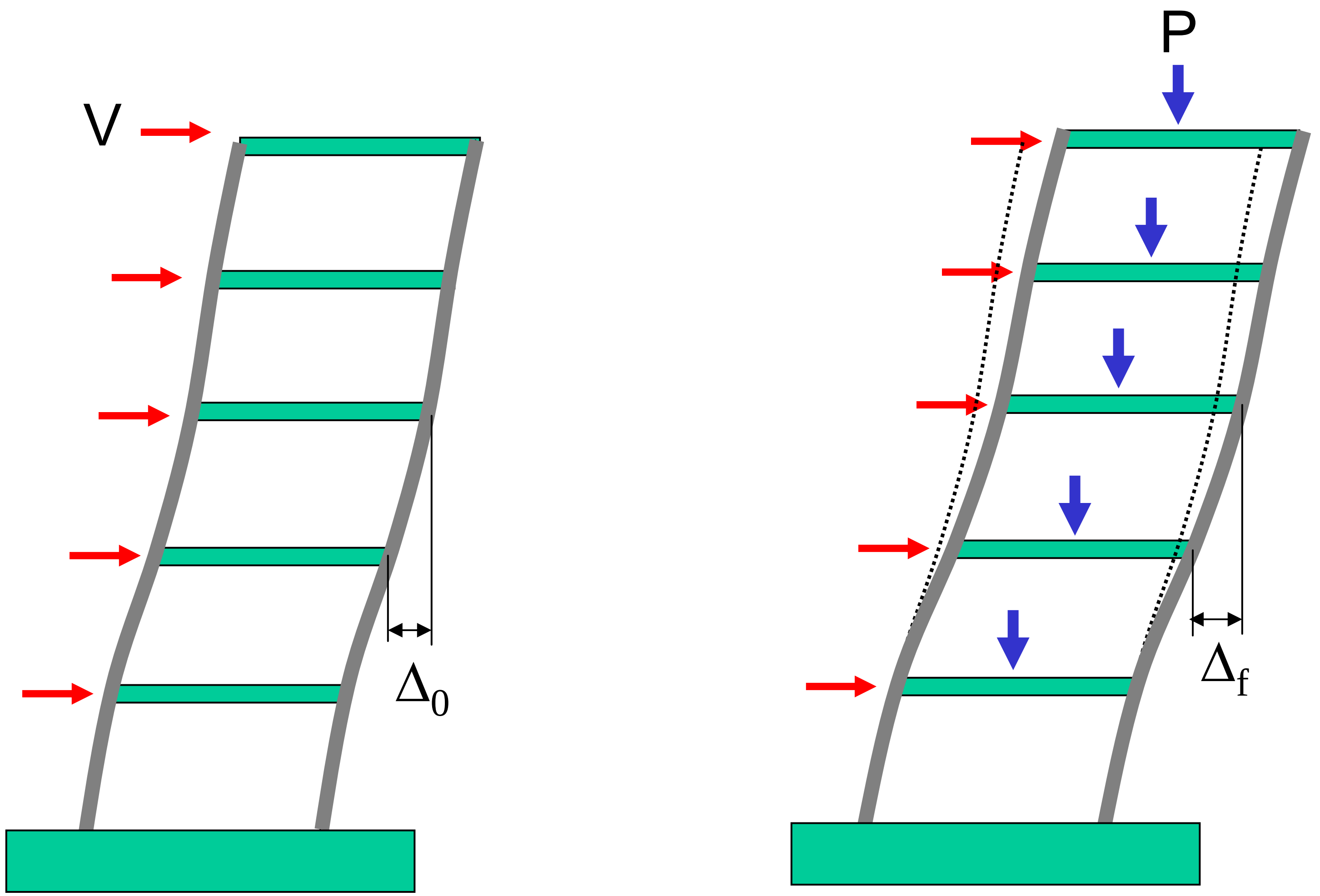
Building Separation to Avoid Pounding



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

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

P-Delta Effects



For elastic systems:

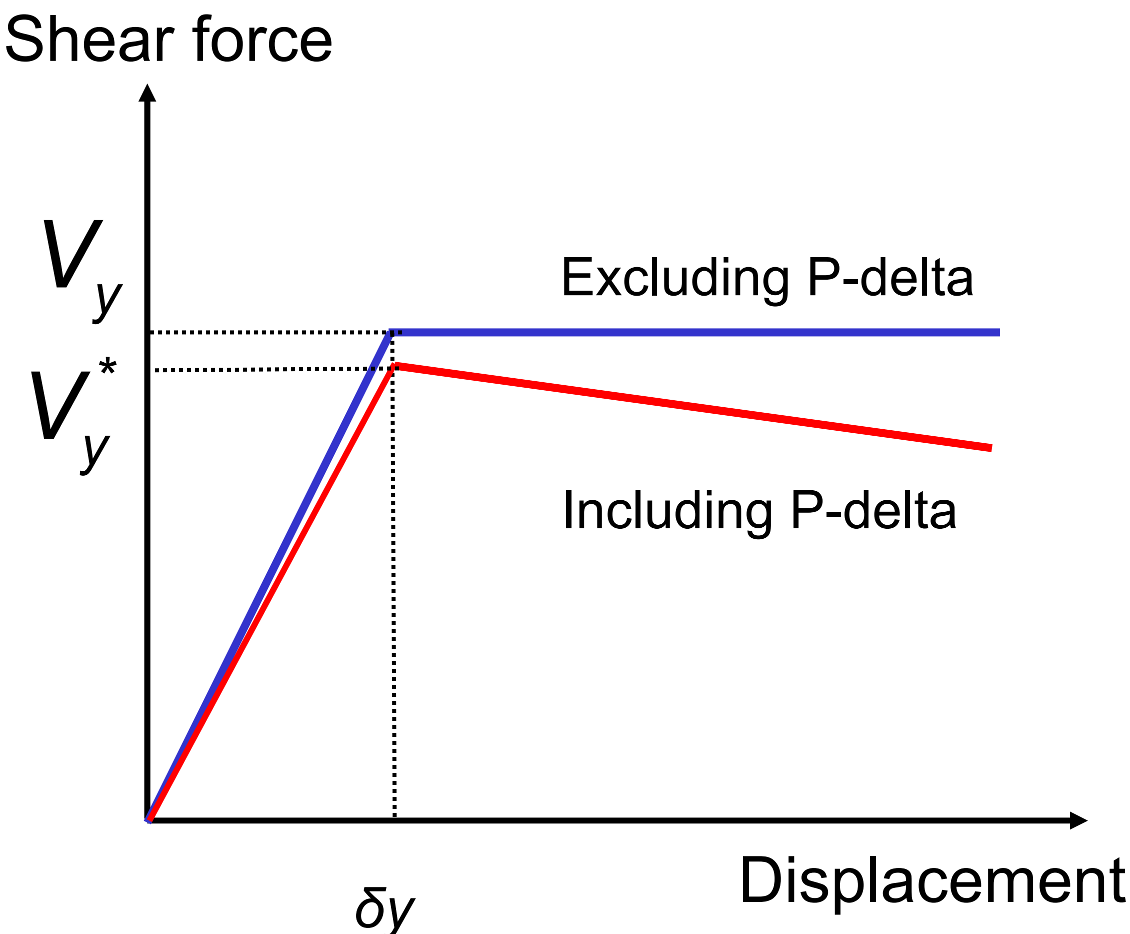
$$\Delta_f = \frac{\Delta_o}{1 - \dfrac{P\Delta_o}{Vh}} = \frac{\Delta_o}{1-\theta}$$

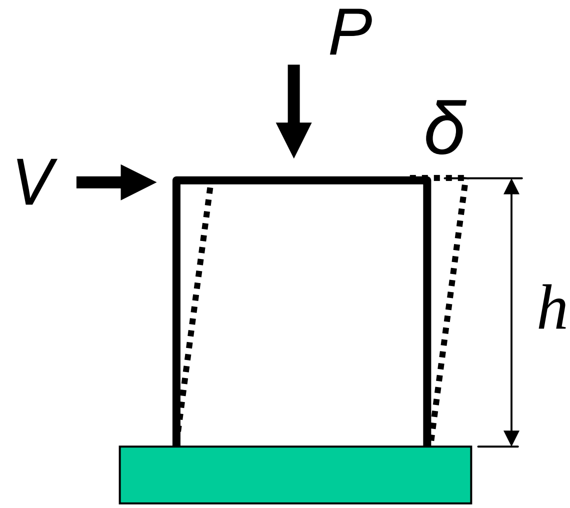
Δ_o = story drift in absence of gravity loads (excluding P-Δ)
Δ_f = story drift including gravity loads (including P-D)
P = total gravity load in story
V = total shear in story
h = story height

Θ is defined as the "story stability ratio"

For inelastic systems:

Reduced stiffness and increased displacements





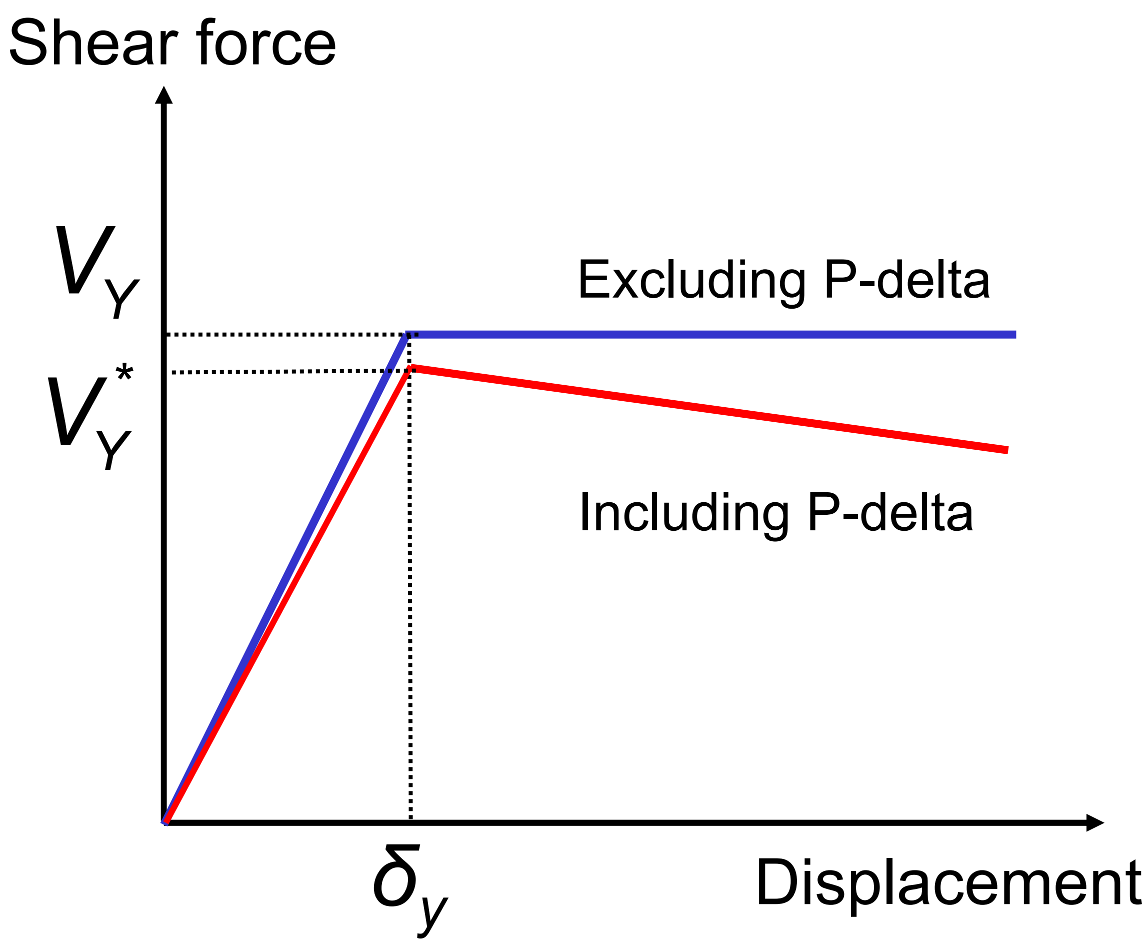
$$K_G = \frac{P}{h}$$

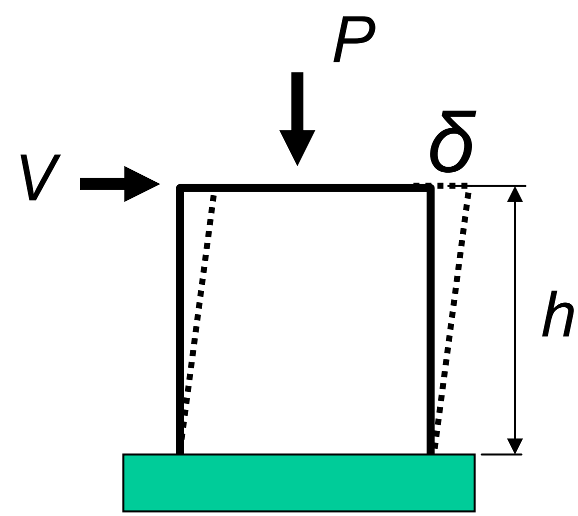
$$K_E = \frac{V_y}{\delta_y}$$

$$K = K_E - K_G$$

For inelastic systems:

Reduced strength

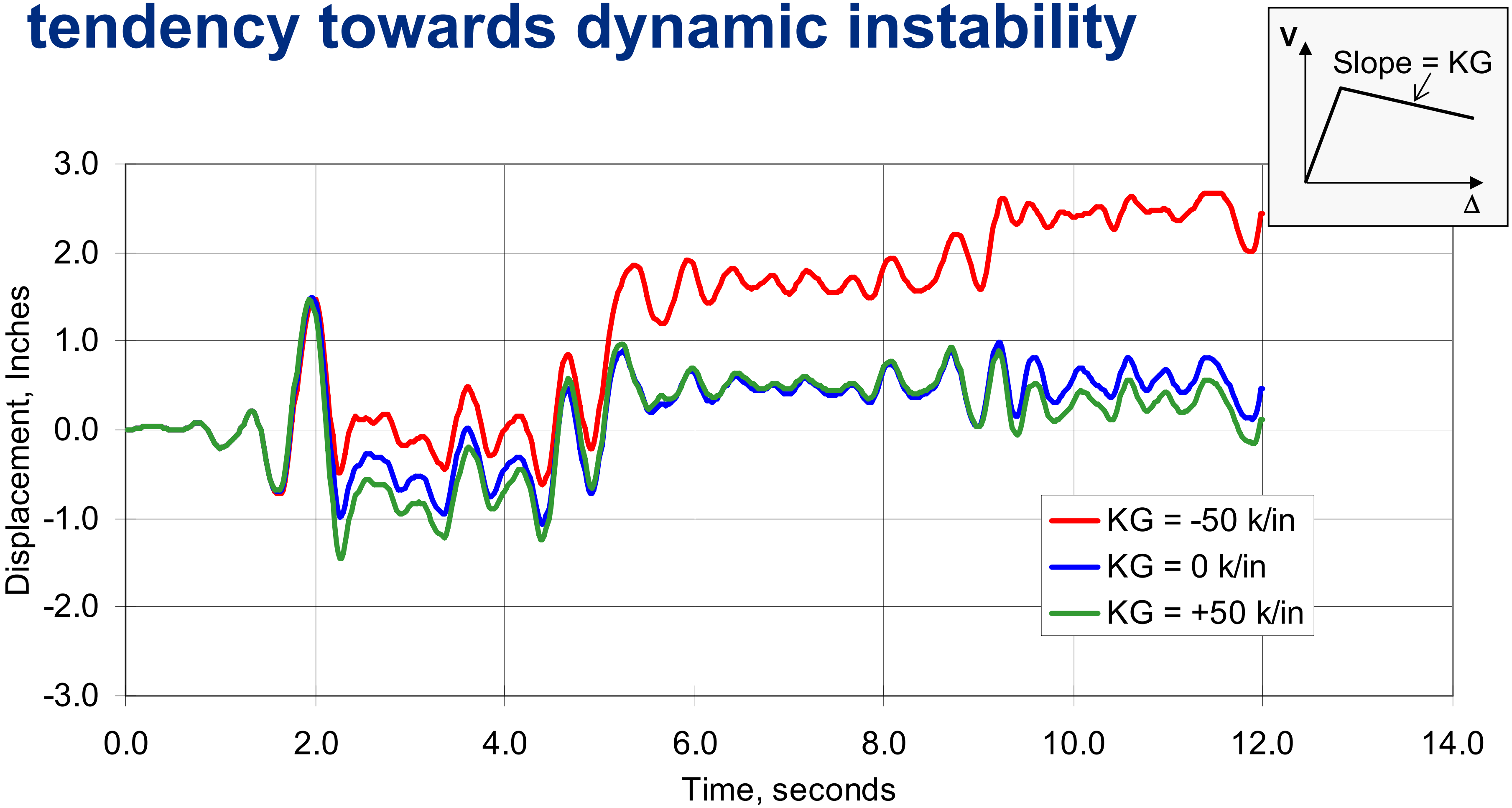




$$\theta = \frac{P\delta_y}{V_y h}$$

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

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



P-Delta Effects

For each story compute:

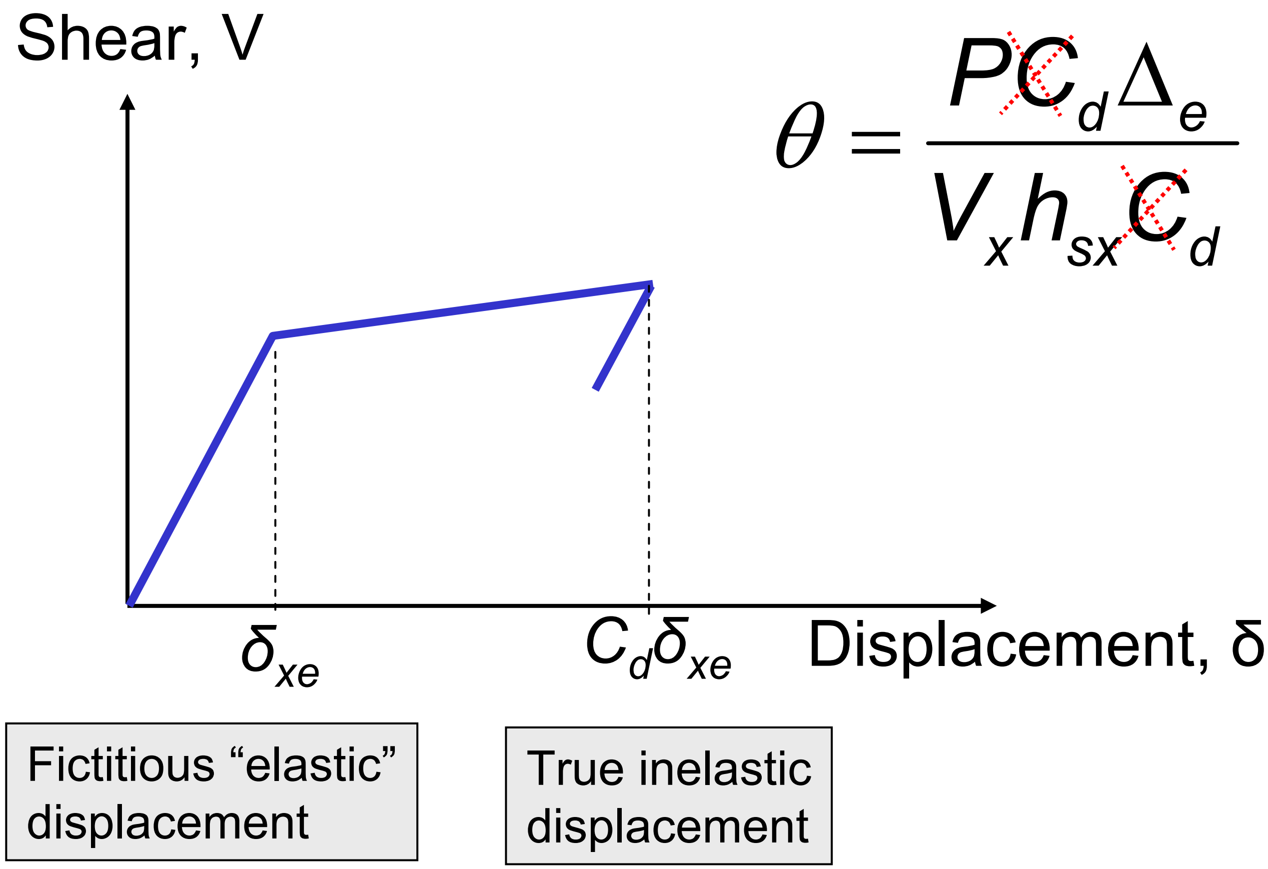
$$\theta = \frac{P\Delta}{V_x h_{sx} C_d}$$

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

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

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

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



P-Delta Effects: ASCE 7-05 approach

If $\theta > 0.1$ then check

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

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

P-Delta Effects: ASCE 7-02 approach

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

Multiply all computed element forces and displacements by:

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

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

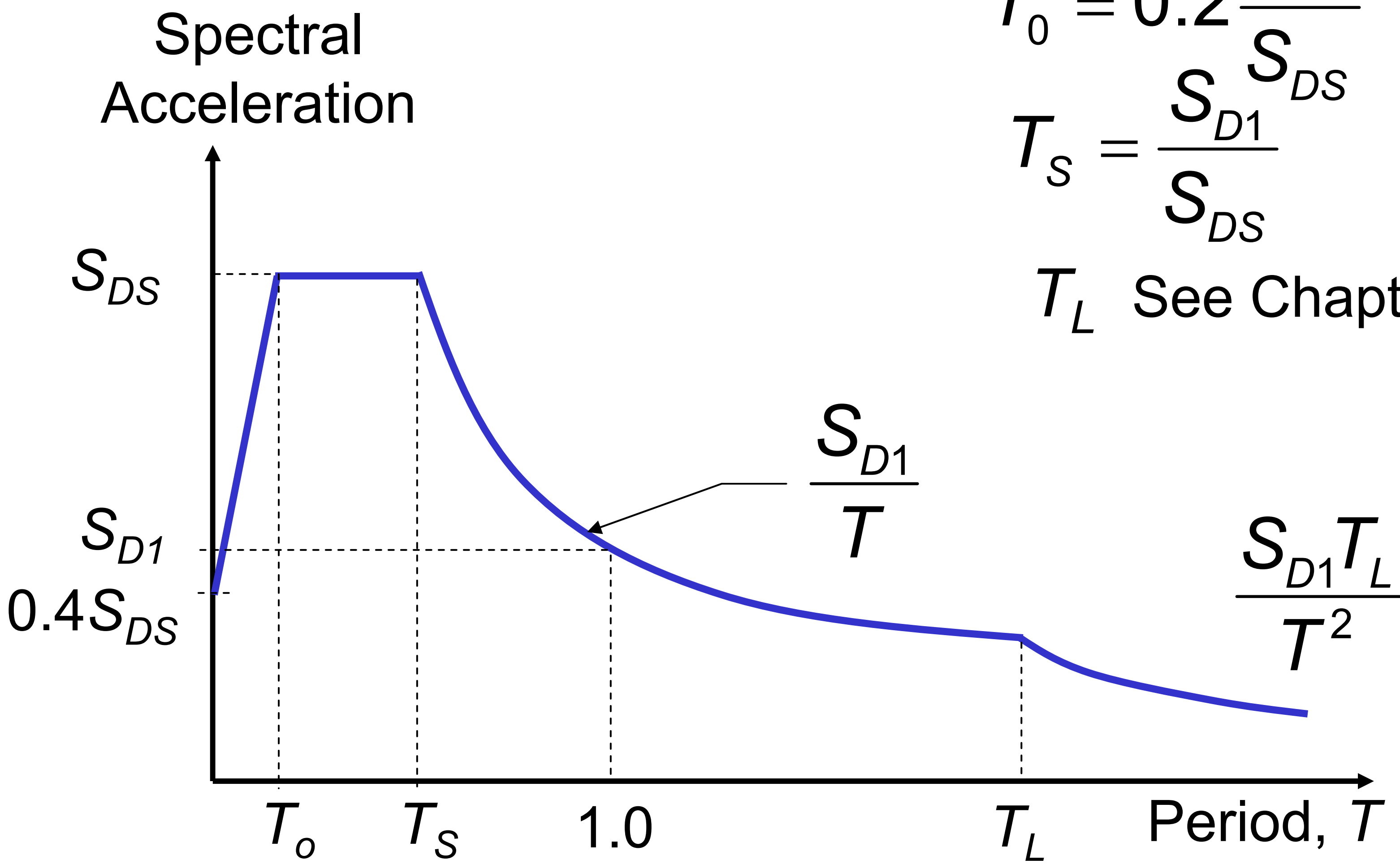
Note: P-delta effects may also be automatically included in the structural analysis. However, limit on θ still applies.

Modal Response Spectrum Analysis

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

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

T_L See Chapter 22



Note: Spectrum includes 5% damping

Basic Steps in Modal Response Spectrum (RS) Analysis

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

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

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

Basic Steps in Modal RS Analysis (continued)

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

5. Compute modal displacements for each mode

6. Compute element forces in each mode

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

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

Basic Steps in Modal RS Analysis (continued)

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

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

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

Analytical Modeling for Modal Response Spectrum Analysis

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

Modal Response History Analysis:

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

Coupled equations:

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

Transformation:

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

Uncoupled equations:

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

Linear Response History Analysis:

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

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

Advantages of Modal Response History Analysis:

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

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

Modal damping ratio

Modal frequency

Modal participation factor

Modal and Linear Response History Structural Modeling Procedures

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

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

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

ASCE 7-05 Ground Motion Selection

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

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

(Parenthesis by F. Charney)

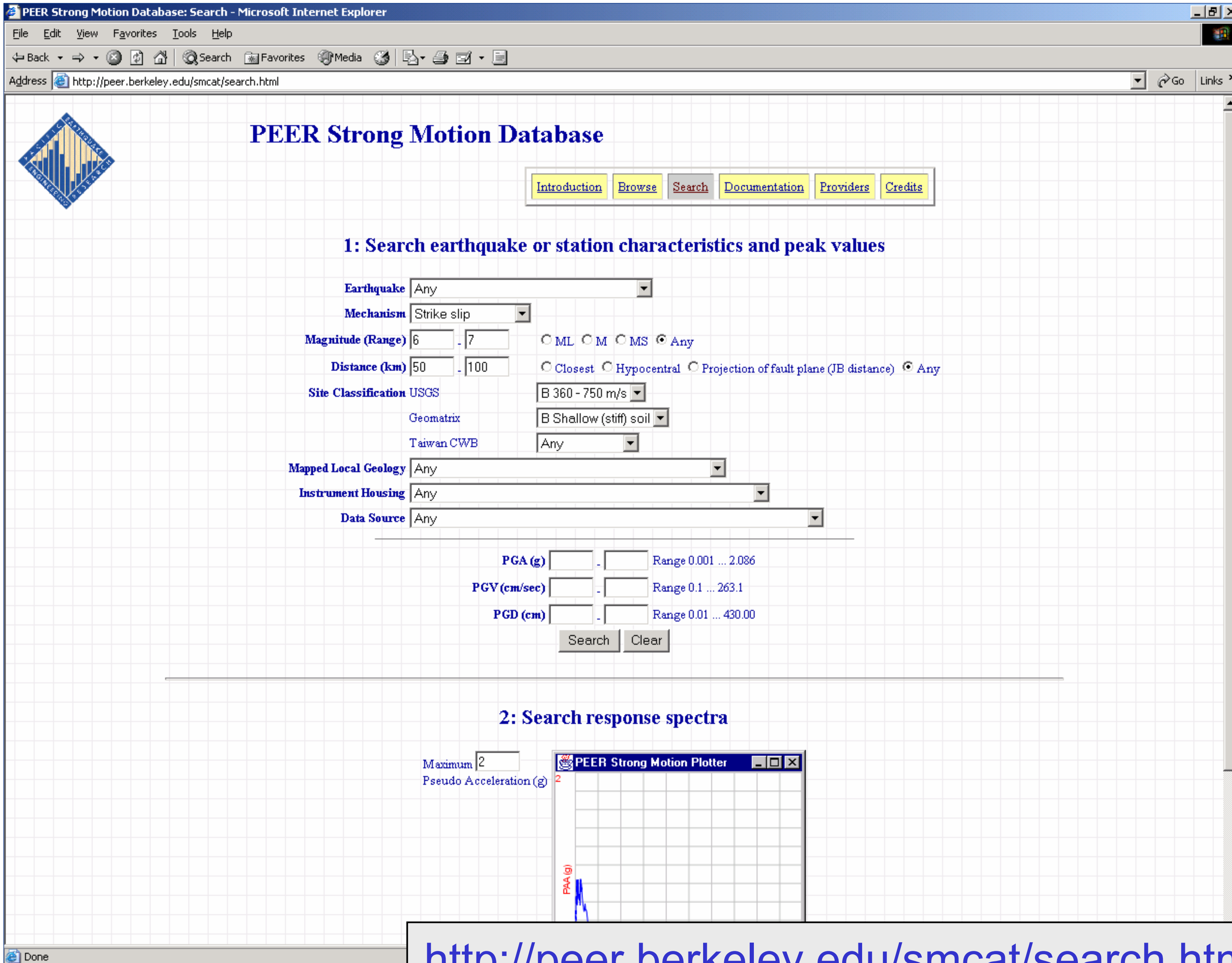
How many records should be used?
Where does one get the records?
How are ground motions scaled?

How Many Records to Use?

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

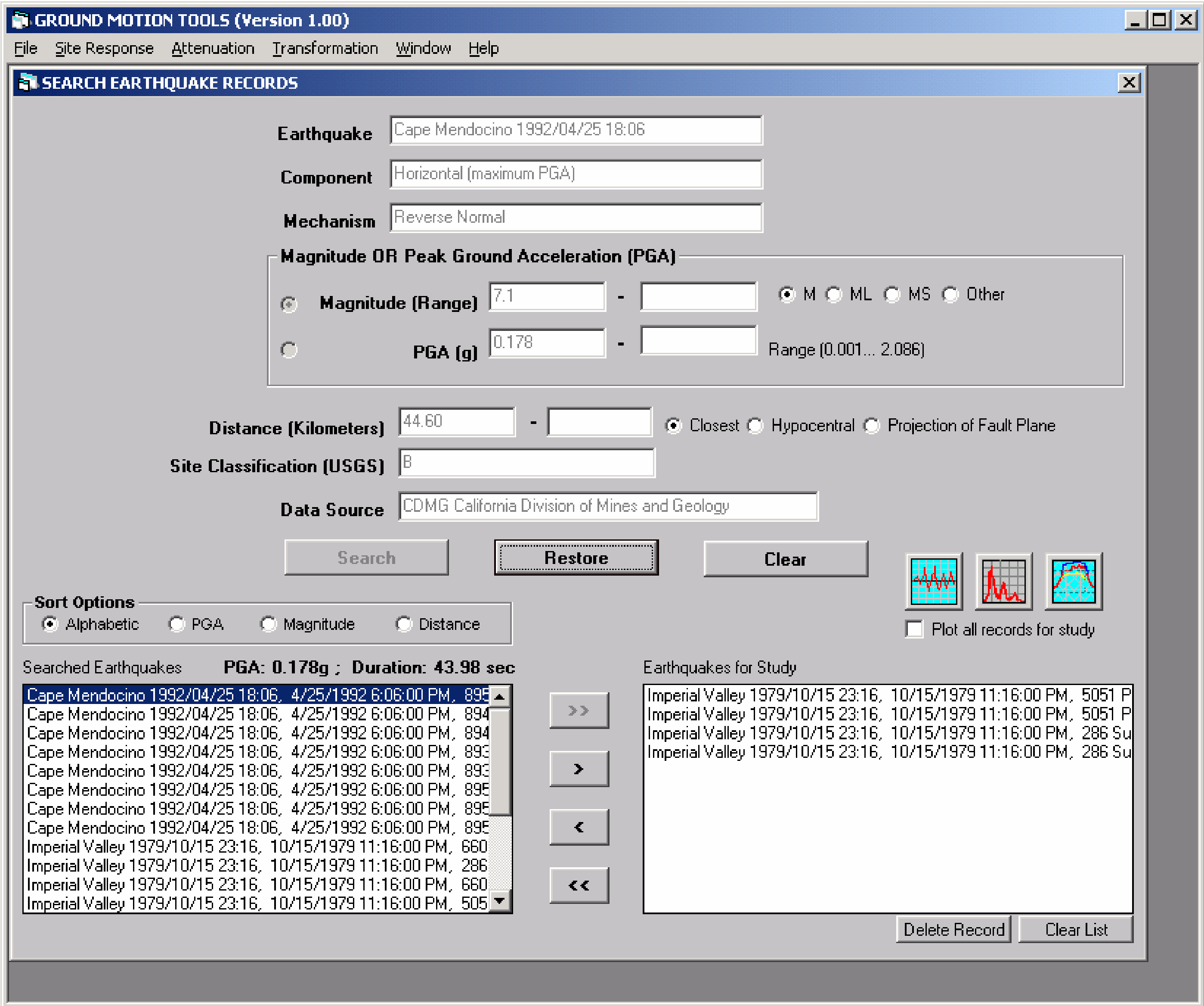
A suite of not less than three motions shall be used.

Ground Motion Sources: PEER



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

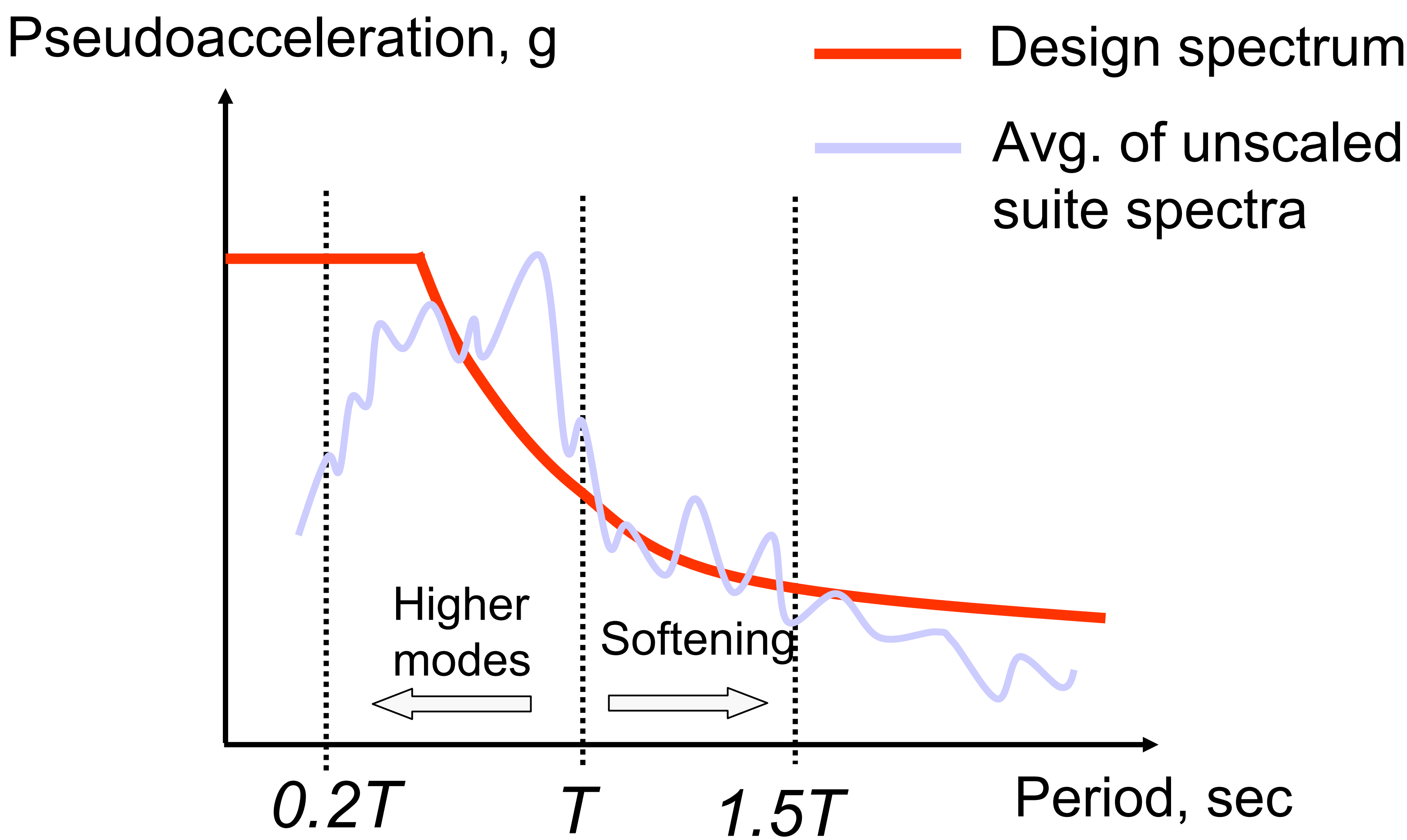
Ground Motion Sources: EQTools



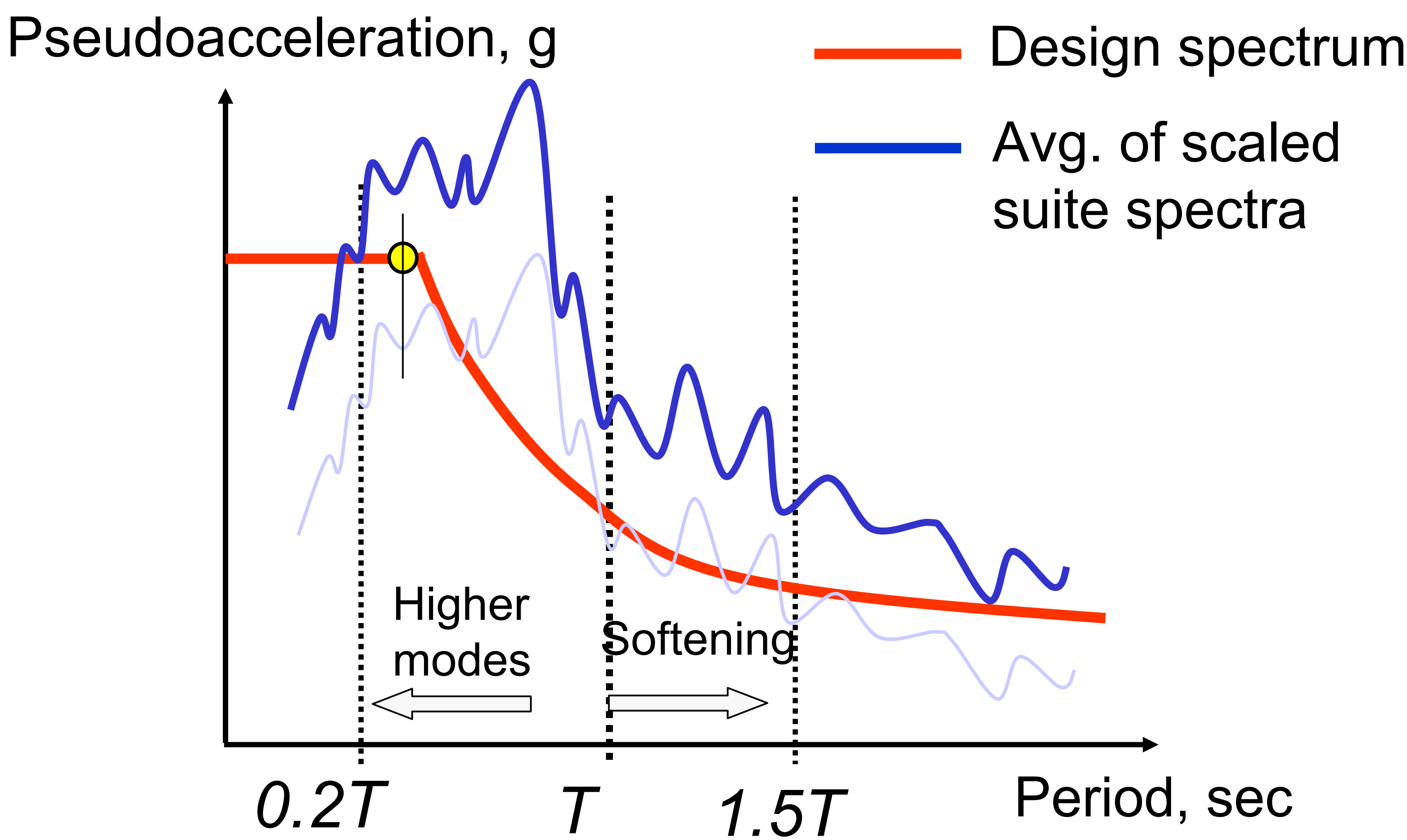
Ground Motion Scaling

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

Scaling for 2-D Analysis



Scaling for 2-D Analysis



Ground Motion Selection and Scaling

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

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

Potential Problems with Scaling

- A degree of freedom exists in selection of individual motion scale factors, thus different analysts may scale the same suite differently.

- The scaling approach seems overly weighted towards higher modes.

- The scaling approach seems to be excessively conservative when compared to other recommendations (e.g., Shome and Cornell)

Recommendations:

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

Response Parameters for Linear Response History Analysis

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

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

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

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

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

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

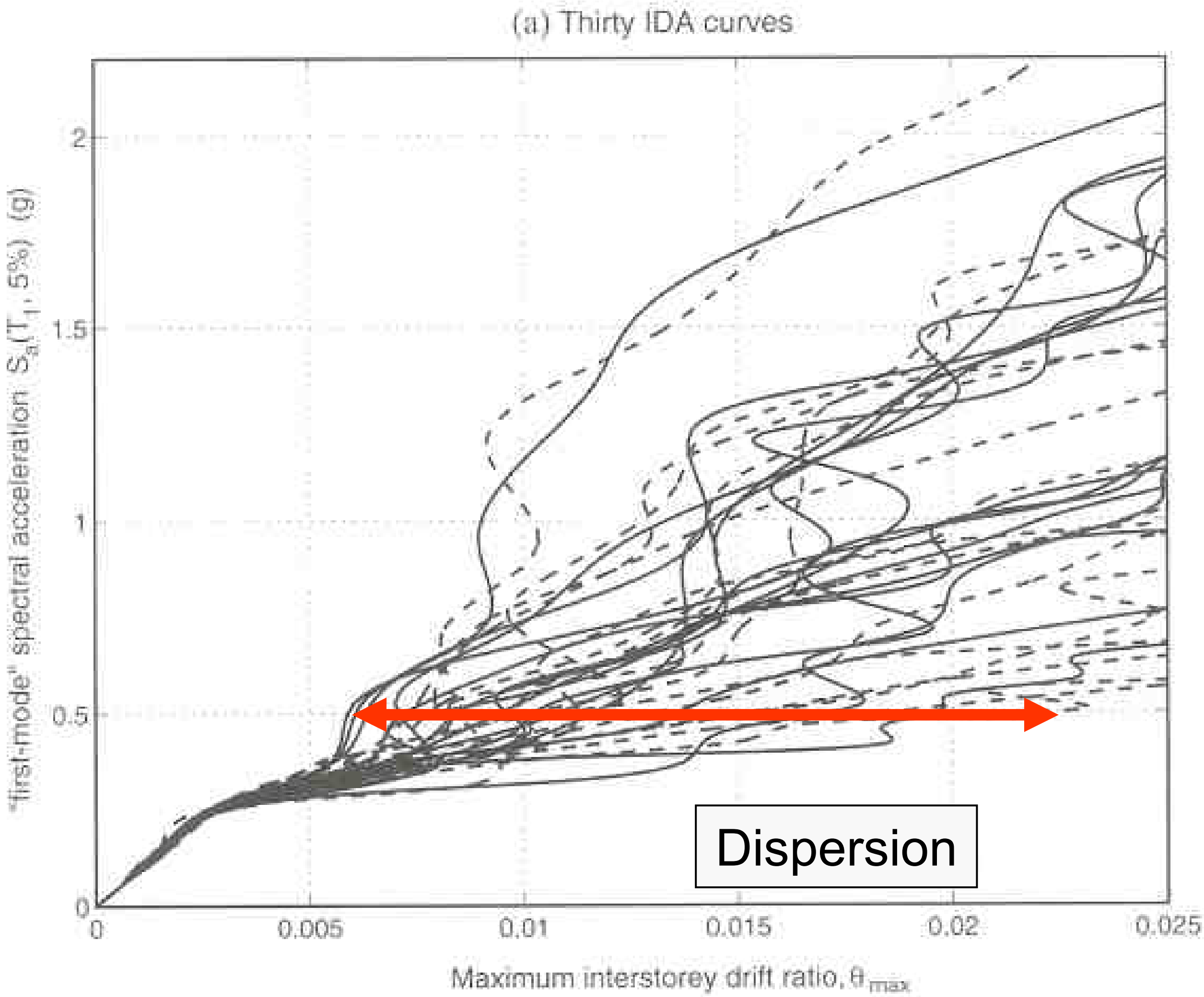
If less than seven ground motions are used, response quantities for component design and story drift must be based on the *maximum* quantity computed among all ground motions.

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

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

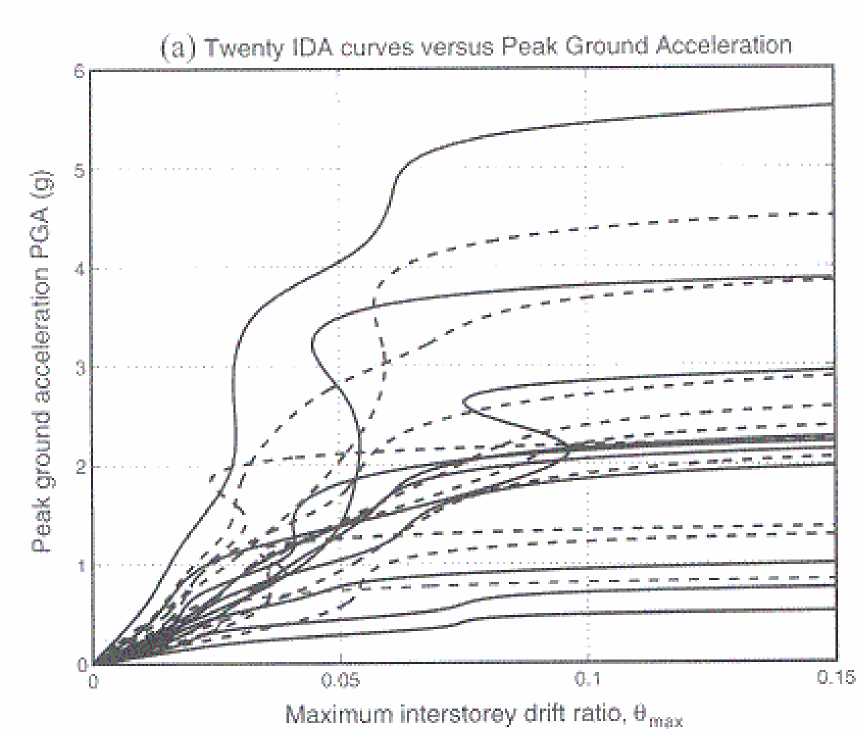
The principal difficulty with nonlinear response history analysis (aside from the effort required) are the sensitivities of the computed response due to a host of uncertainties.
Such sensitivities are exposed by a systematic analysis approach called incremental dynamic analysis.

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

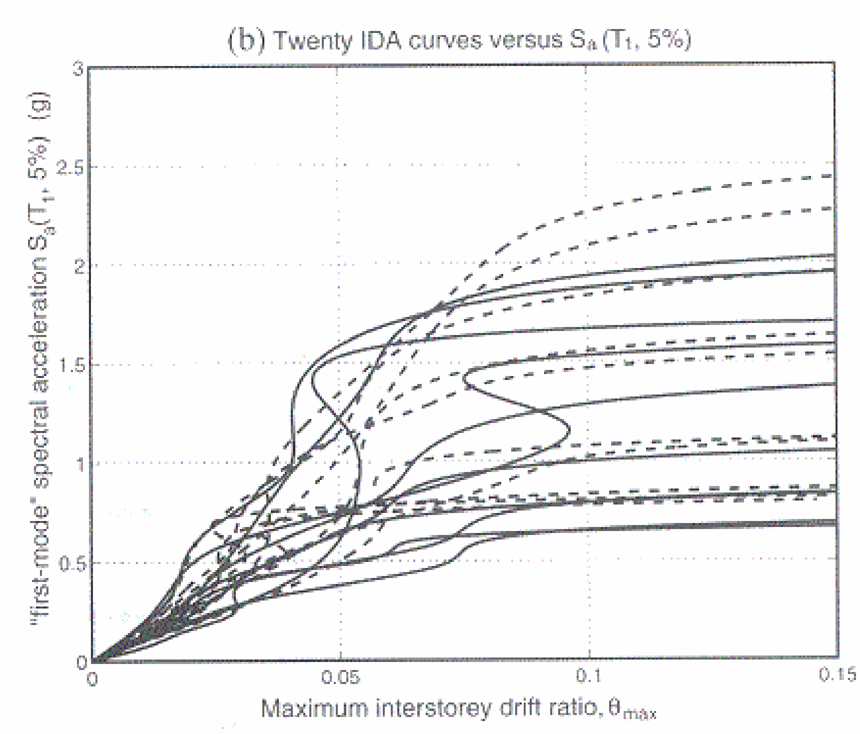


IDA Curves of the Same Building Subjected to Suite of Earthquakes Where Different Scaling Methods Have Been Used

NORMALIZED to PGA



NORMALIZED to S_a



Methods of Analysis Described in ASCE 7-05

Nonlinear static pushover analysis

NEHRP RECOMMENDED PROVISIONS SEISMIC DESIGN OF STEEL STRUCTURES

- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics
- Summary

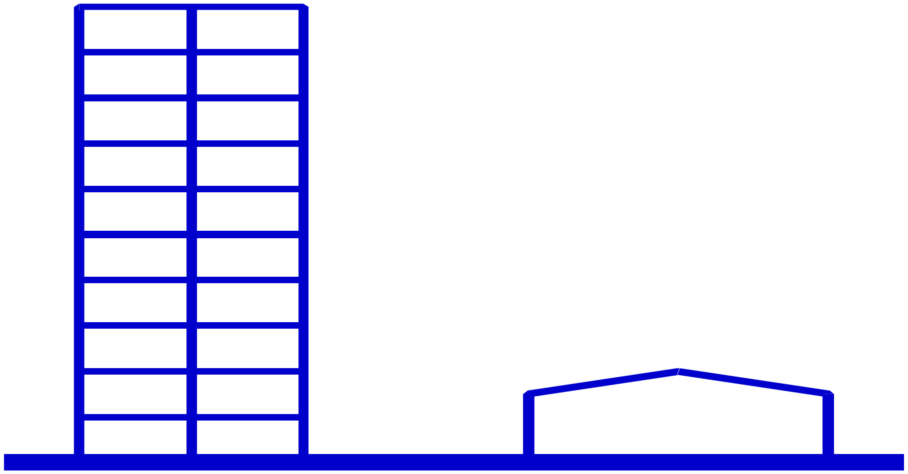
Steel Design: Context in Provisions

Design basis: Strength limit state

<u>Using the 2003 *NEHRP Recommended Provisions*:</u>

Load combination	Chap. 4
Seismic load analysis	Chap. 5
Components and attachments	Chap. 6
Design of steel structures	Chap. 8
	AISC Seismic
	and others

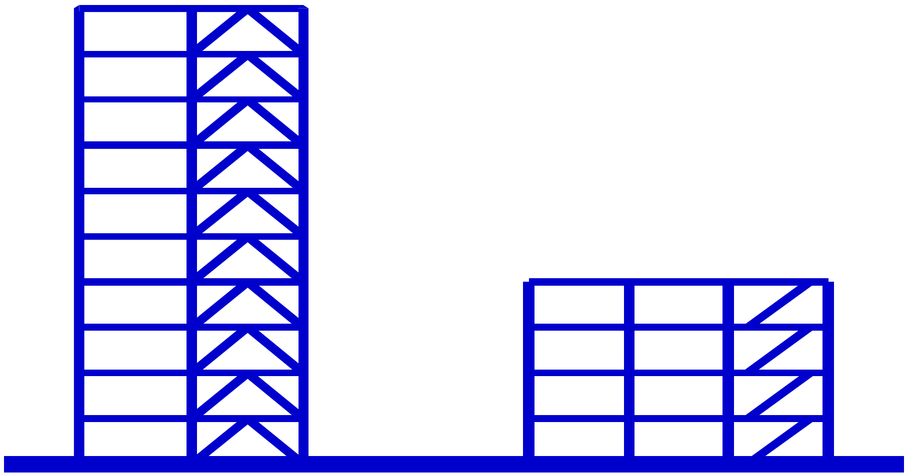
Seismic Resisting Systems



Unbraced Frames

•Joints are:
Rigid/FR/PR/
Moment-resisting

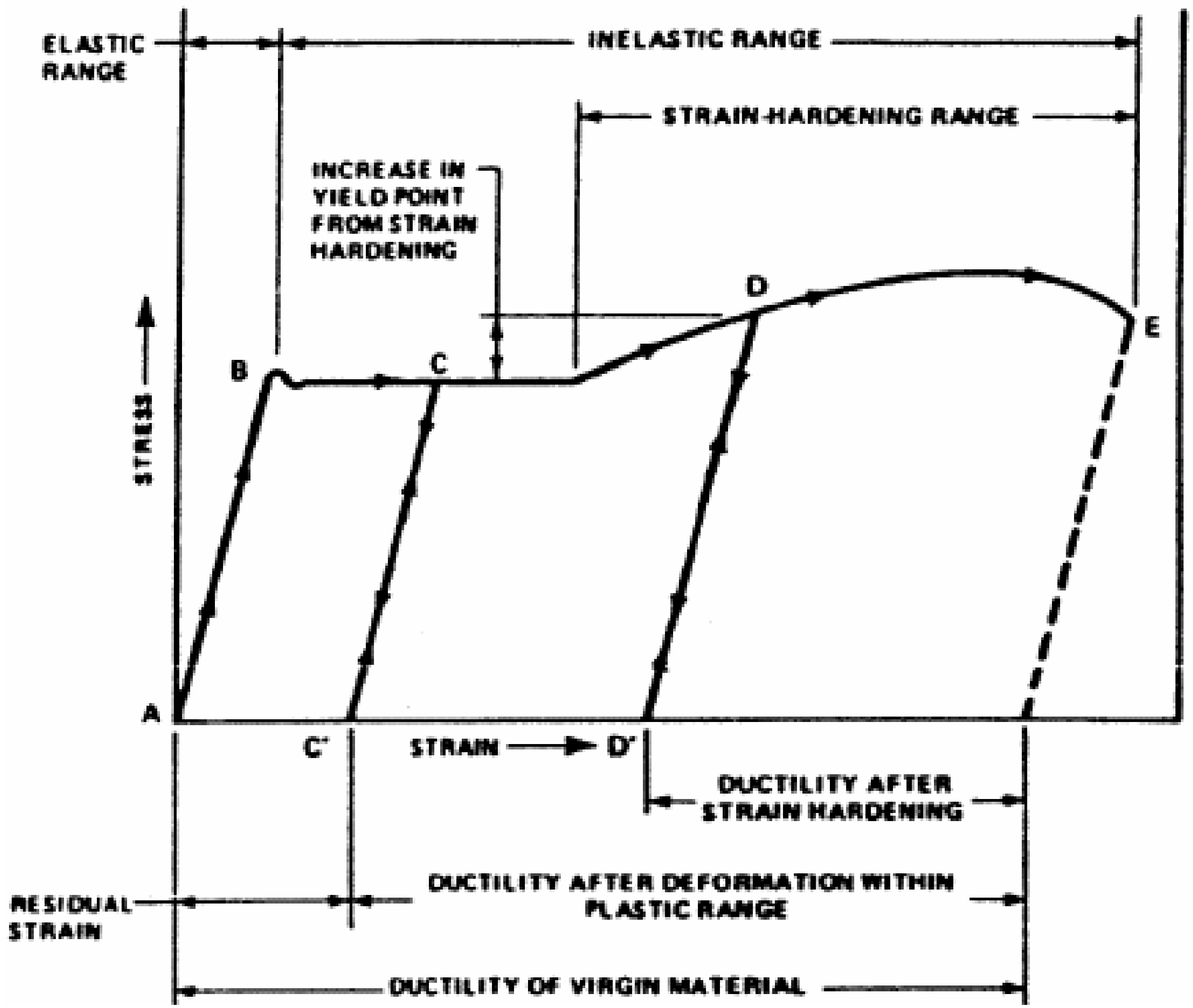
•Seismic classes are:
Special/intermediate/
Ordinary/not detailed



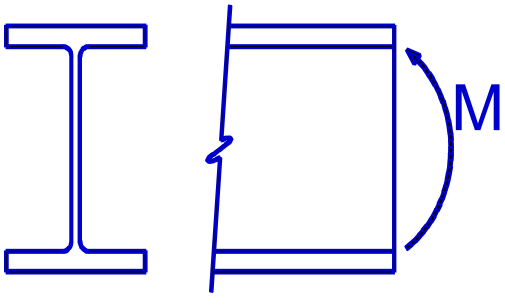
Braced Frames

- Concentric bracing
- Eccentric bracing

Monotonic Stress-Strain Behavior



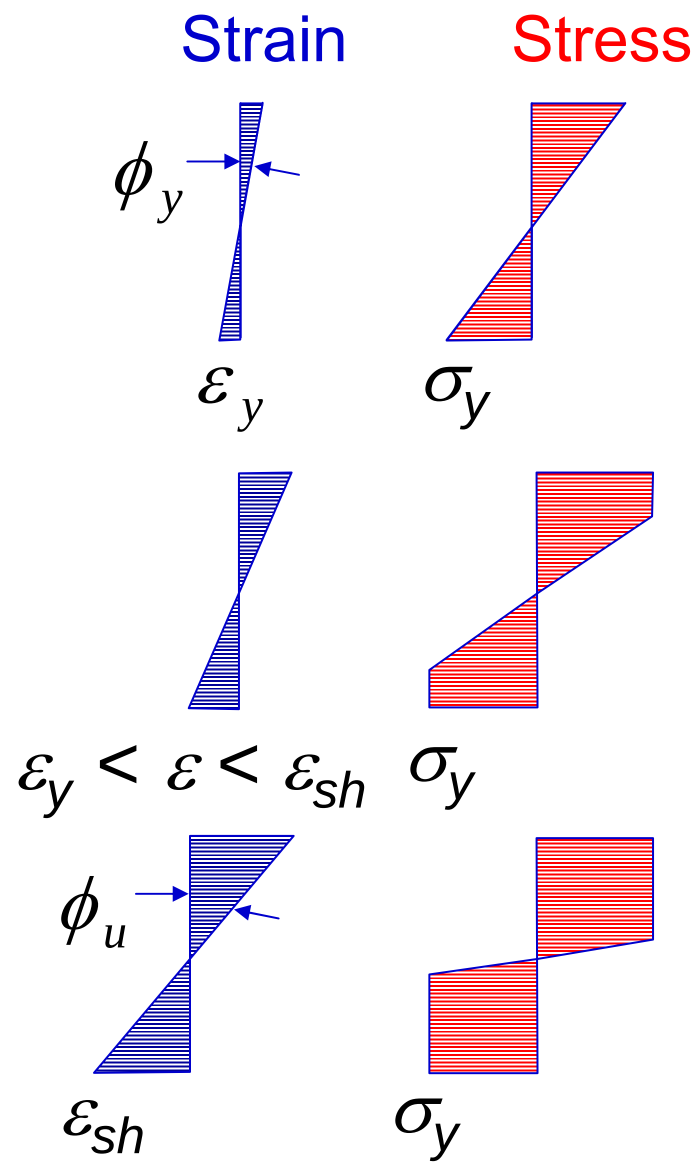
Bending of Steel Beam



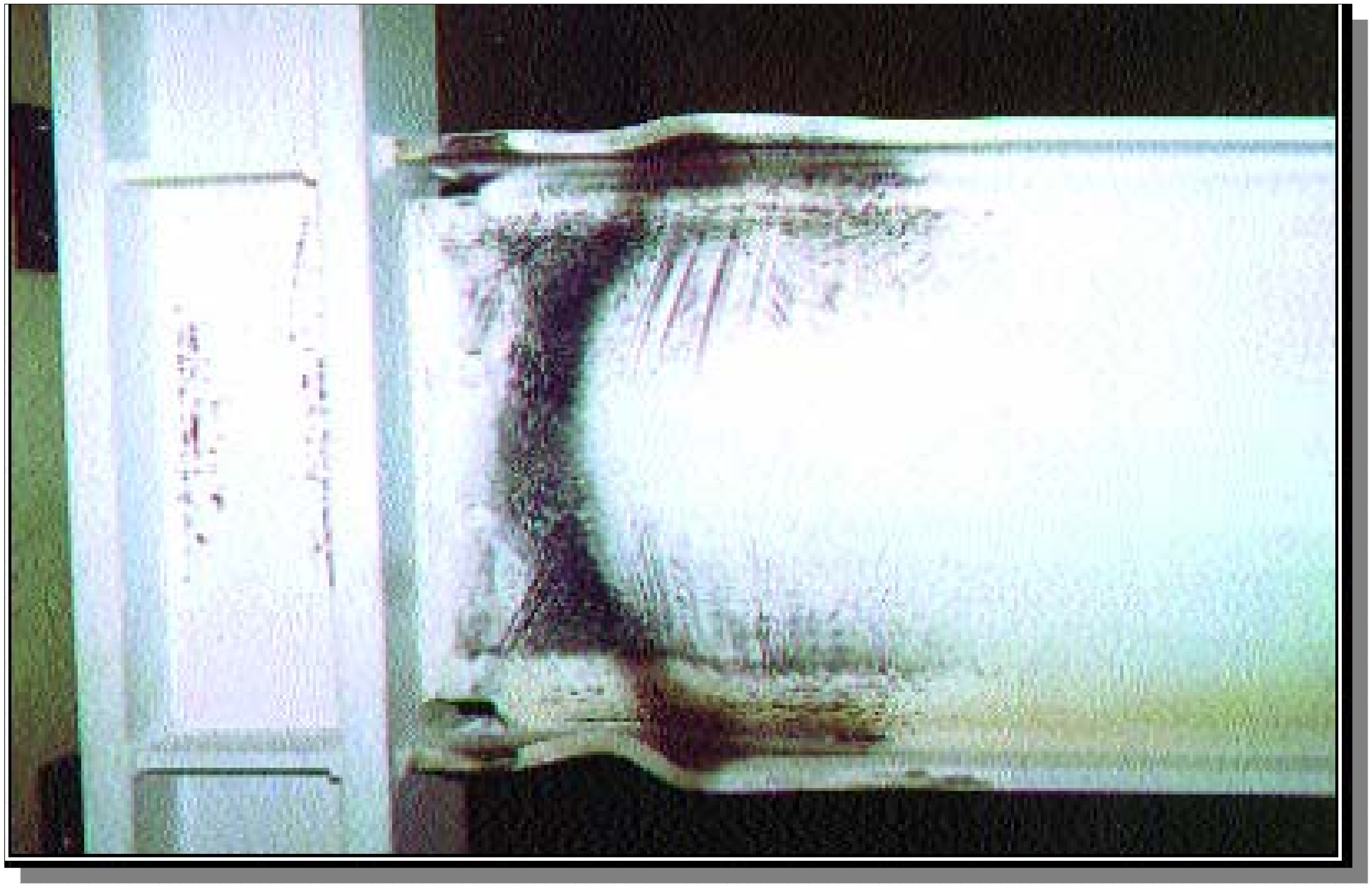
Extreme fiber reaches yield strain and stress

Strain slightly above yield strain

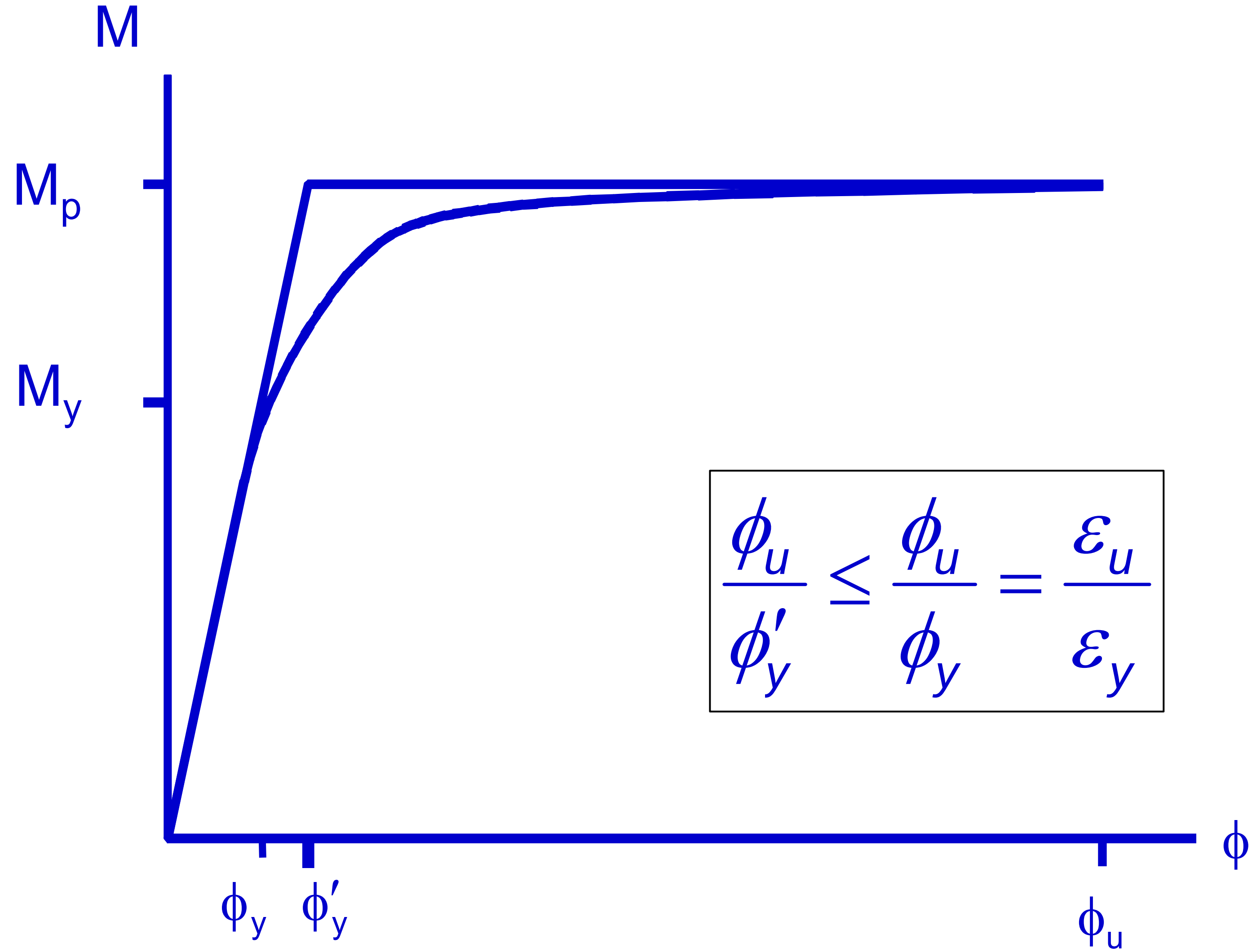
Section near "plastic"



Plastic Hinge Formation



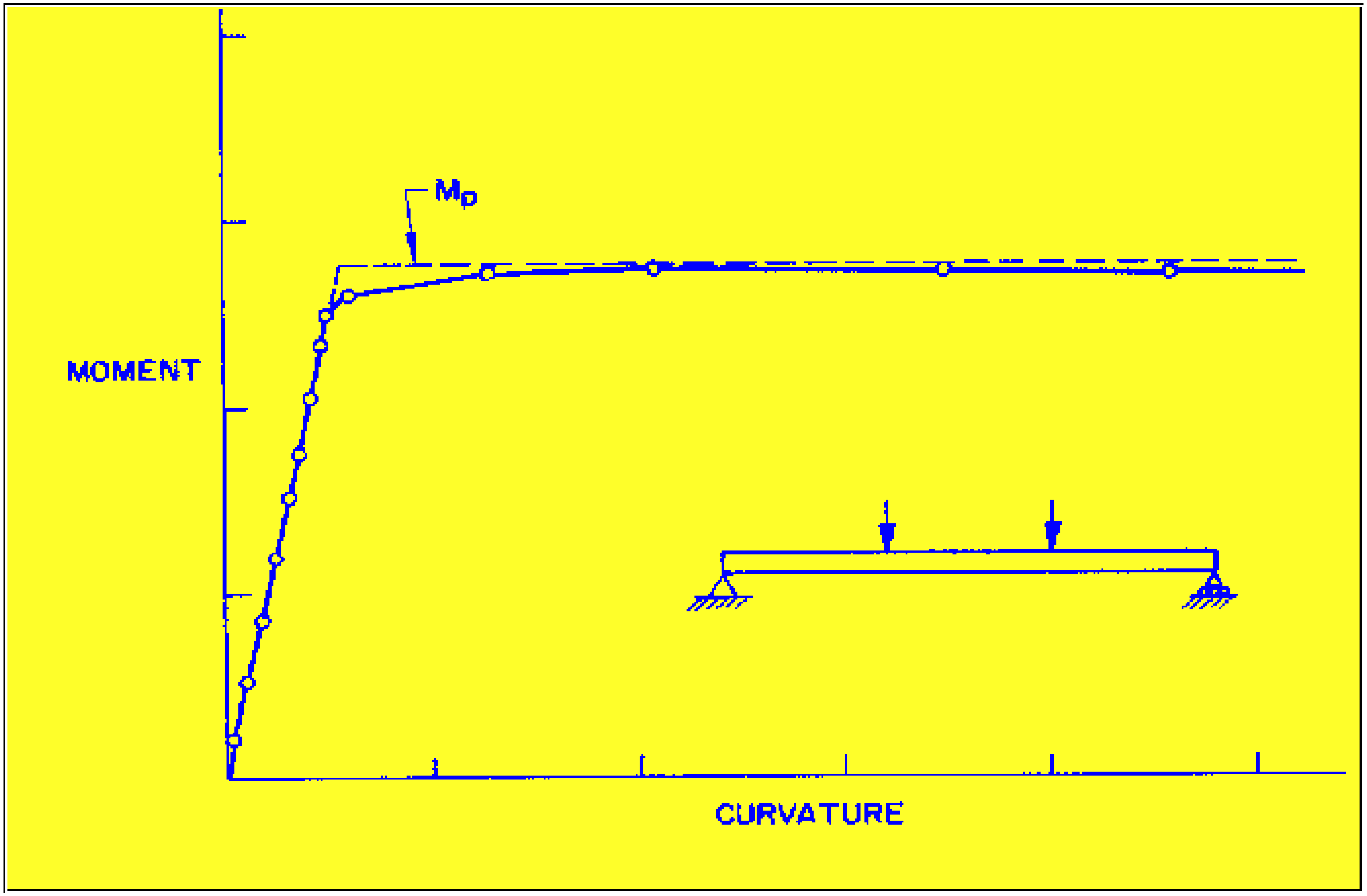
Cross - section Ductility
Conceptual moment - curvature



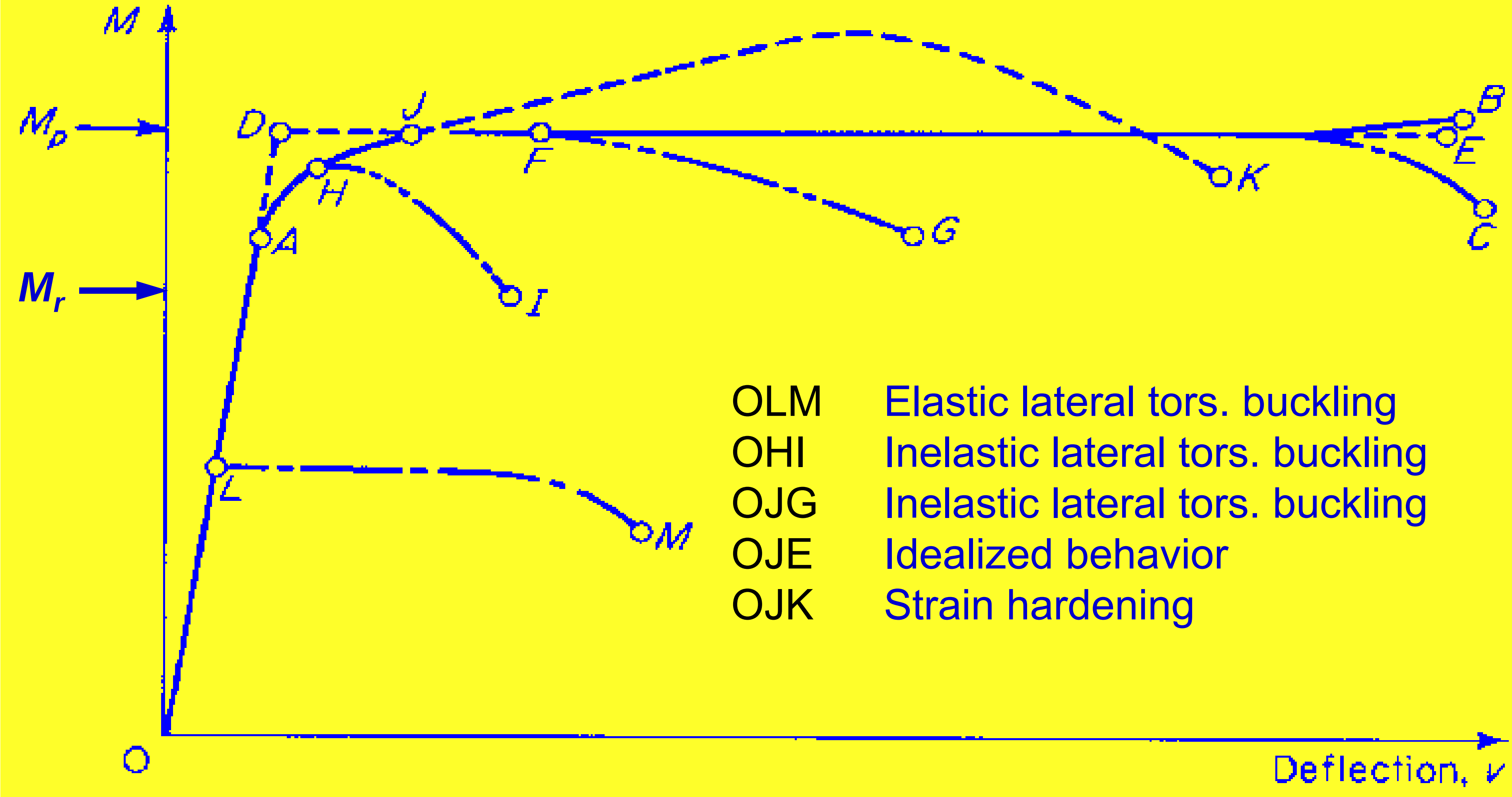
$$\frac{\phi_u}{\phi'_y} \leq \frac{\phi_u}{\phi_y} = \frac{\varepsilon_u}{\varepsilon_y}$$

Moment Curvature

Laboratory Test -- Annealed W Beam



Behavior Modes For Beams



OLM	Elastic lateral tors. buckling
OHI	Inelastic lateral tors. buckling
OJG	Inelastic lateral tors. buckling
OJE	Idealized behavior
OJK	Strain hardening

Flexural Ductility of Steel Members
Practical Limits

1 Lateral torsional buckling

Brace well

2 Local buckling

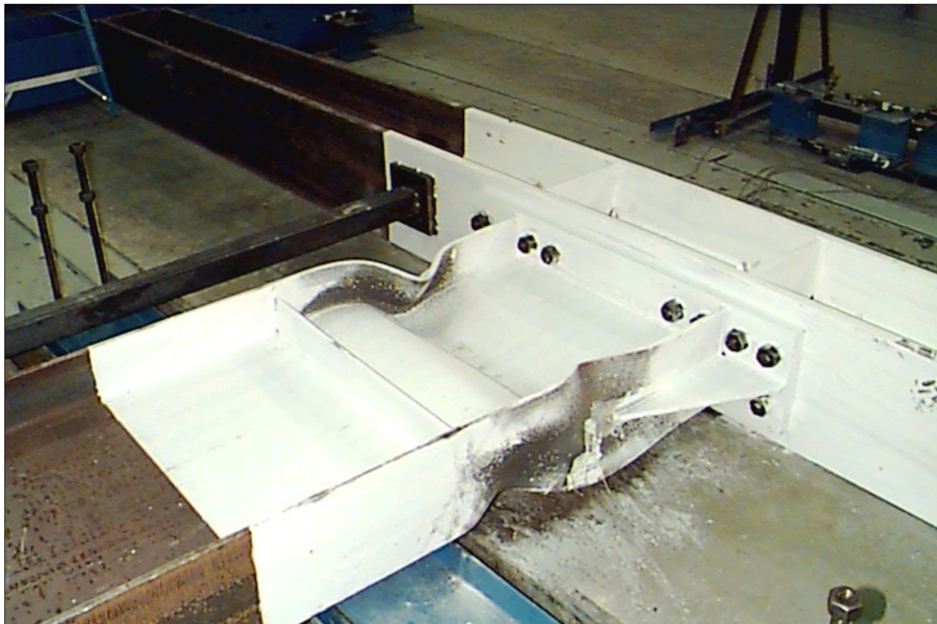
Limit width-to-thickness ratios

for compression elements

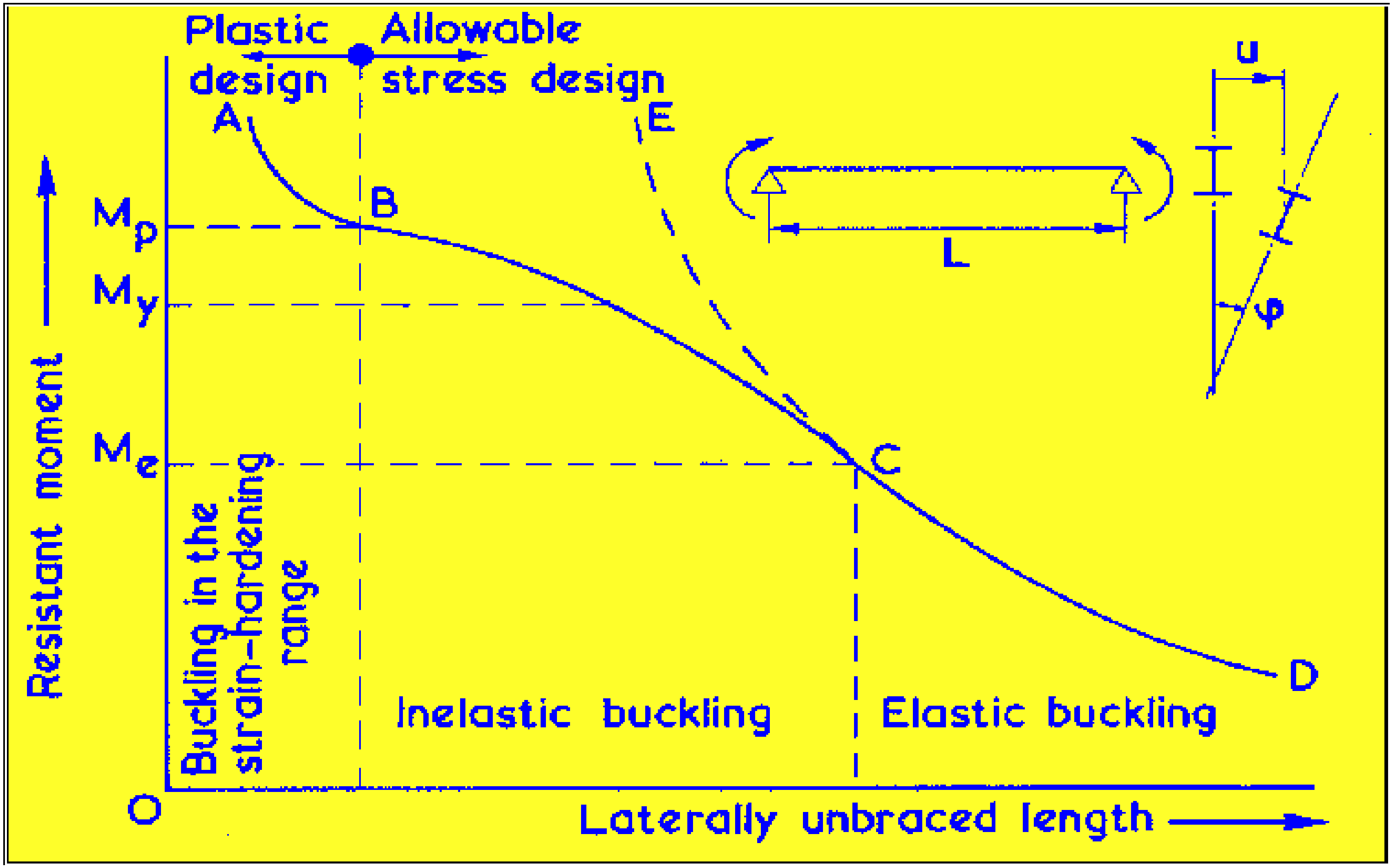
3 Fracture

Avoid by proper detailing

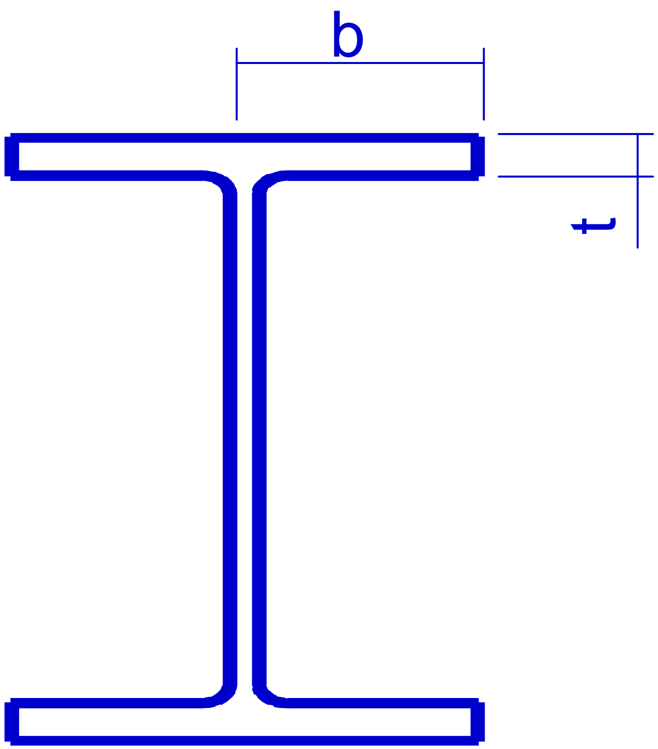
Local and Lateral Buckling



Lateral Torsional Buckling



Local Buckling



Classical plate buckling solution:

$$\sigma_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)(b/t)^2} \le \sigma_y$$

Substituting *μ = 0.3 and rearranging*:

$$\frac{b}{t} \le 0.95\sqrt{\frac{kE}{F_y}}$$

Local Buckling
continued

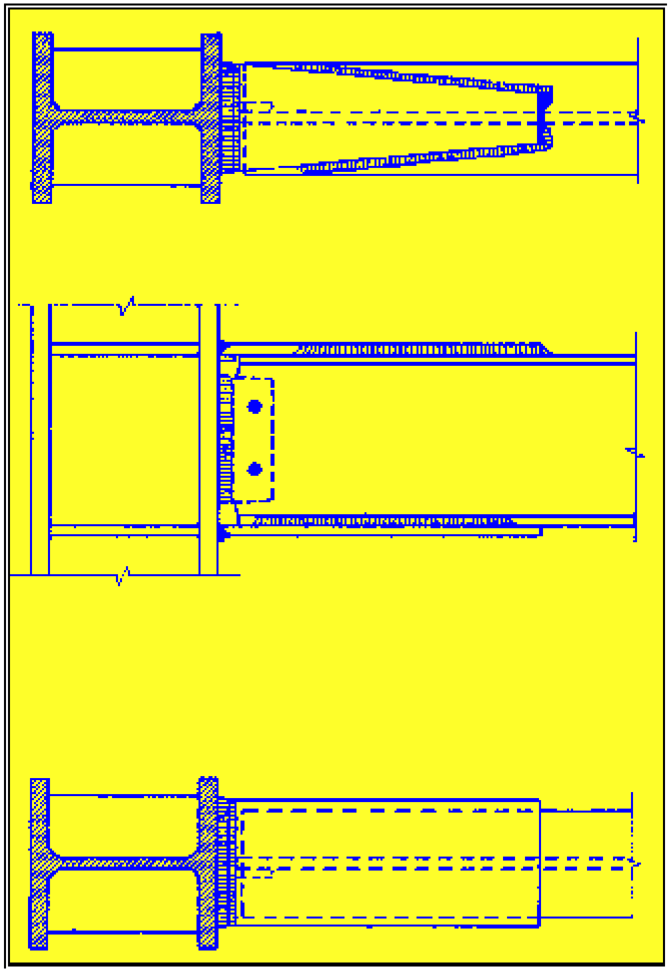
With the plate buckling coefficient taken as 0.7 and an adjustment for residual stresses, the expression for b/t becomes:

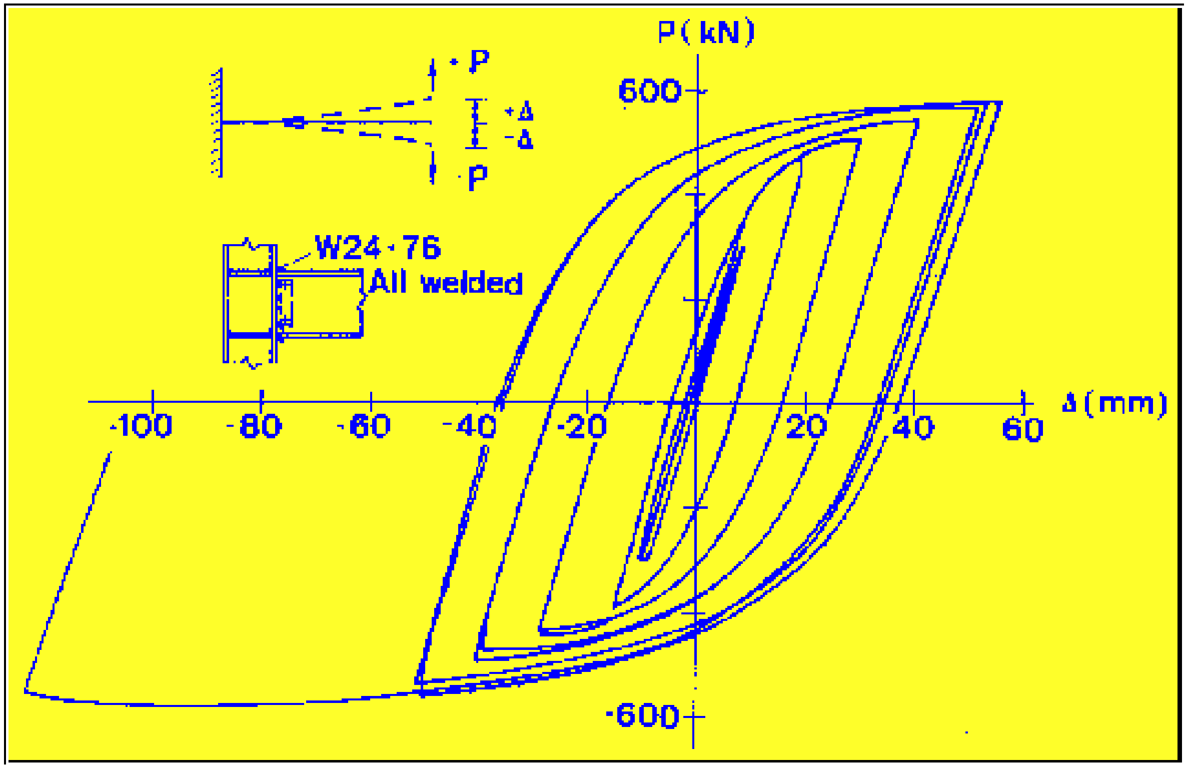
$$\frac{b}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$$

This is the slenderness requirement given in the AISC specification for compact flanges of I-shaped sections in bending. The coefficient is further reduced for sections to be used in seismic applications in the AISC Seismic specification

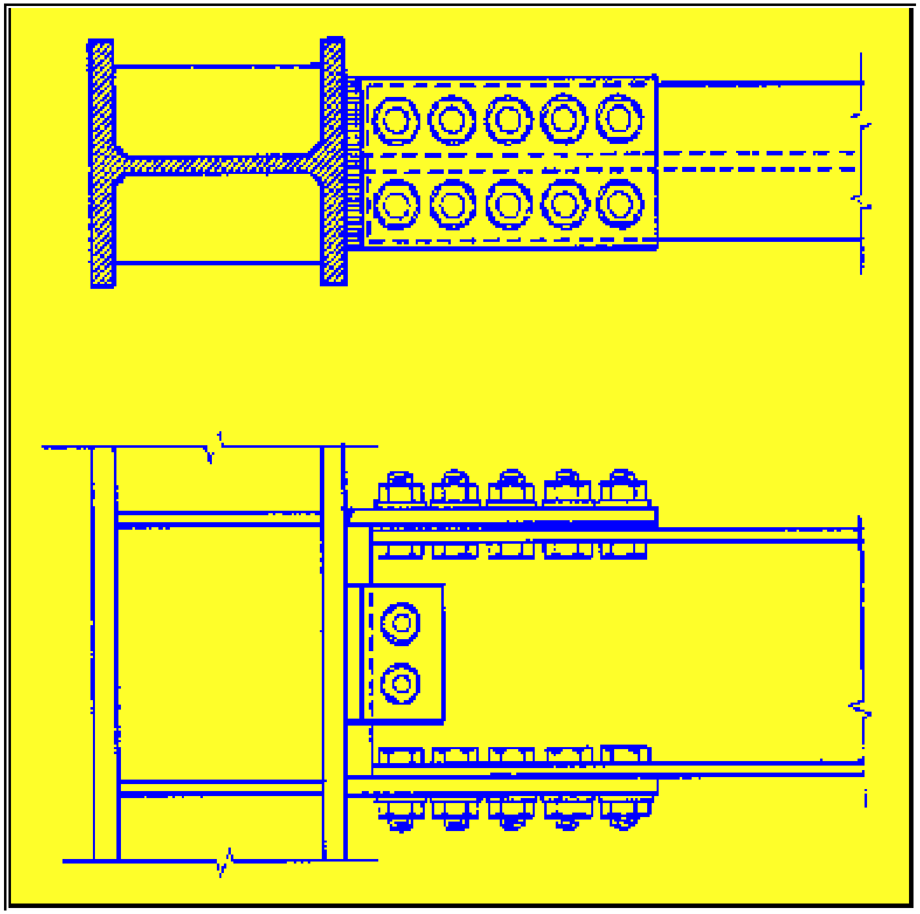
$$\frac{b}{t} \leq 0.3\sqrt{\frac{E}{F_y}}$$

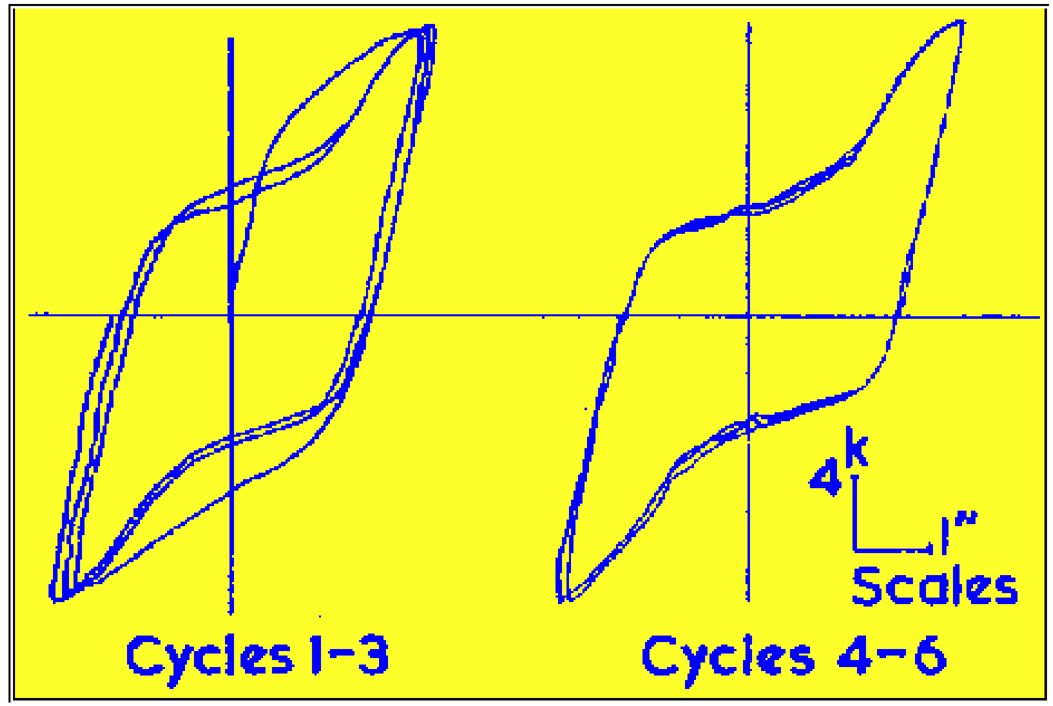
Welded Beam to Column Laboratory Test - 1960s



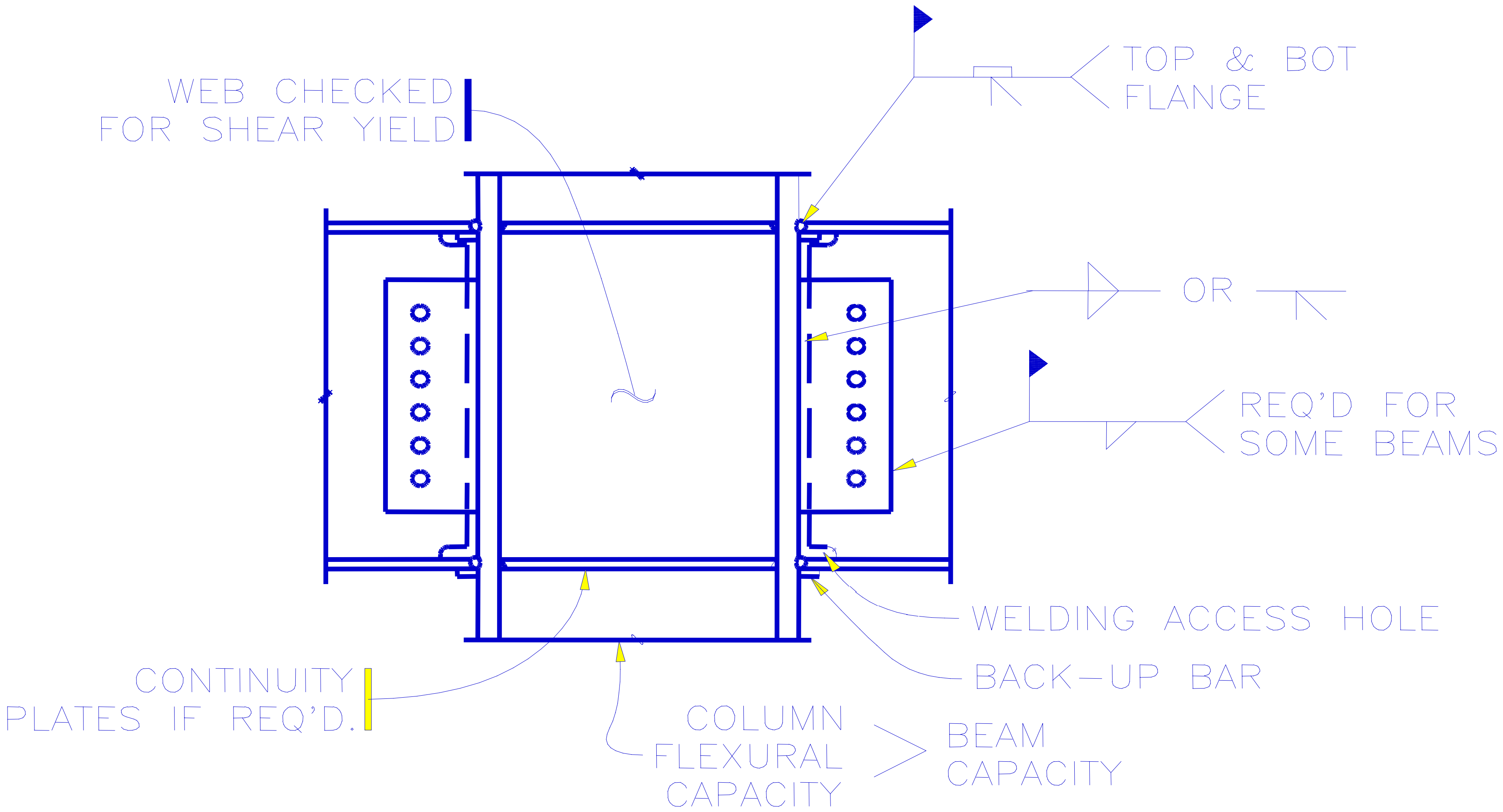


Bolted Beam to Column Laboratory Test - 1960s





Pre-Northridge Standard



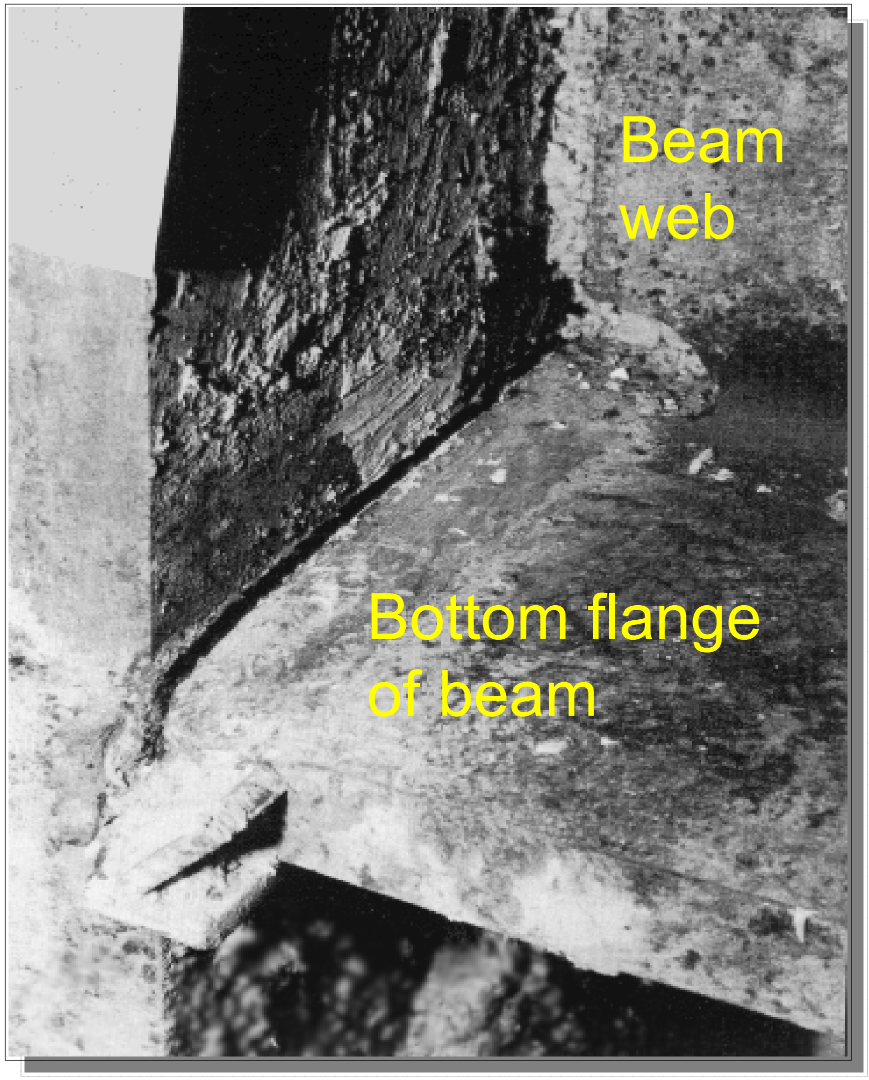
Following the 1994 Northridge earthquake, numerous failures of steel beam-to-column moment connections were identified. This led to a multiyear, multimillion dollar FEMA-funded research effort known as the SAC joint venture. The failures caused a fundamental rethinking of the design of seismic resistant steel moment connections.

Bottom Flange Weld Fracture Propagating Through Column Flange and Web

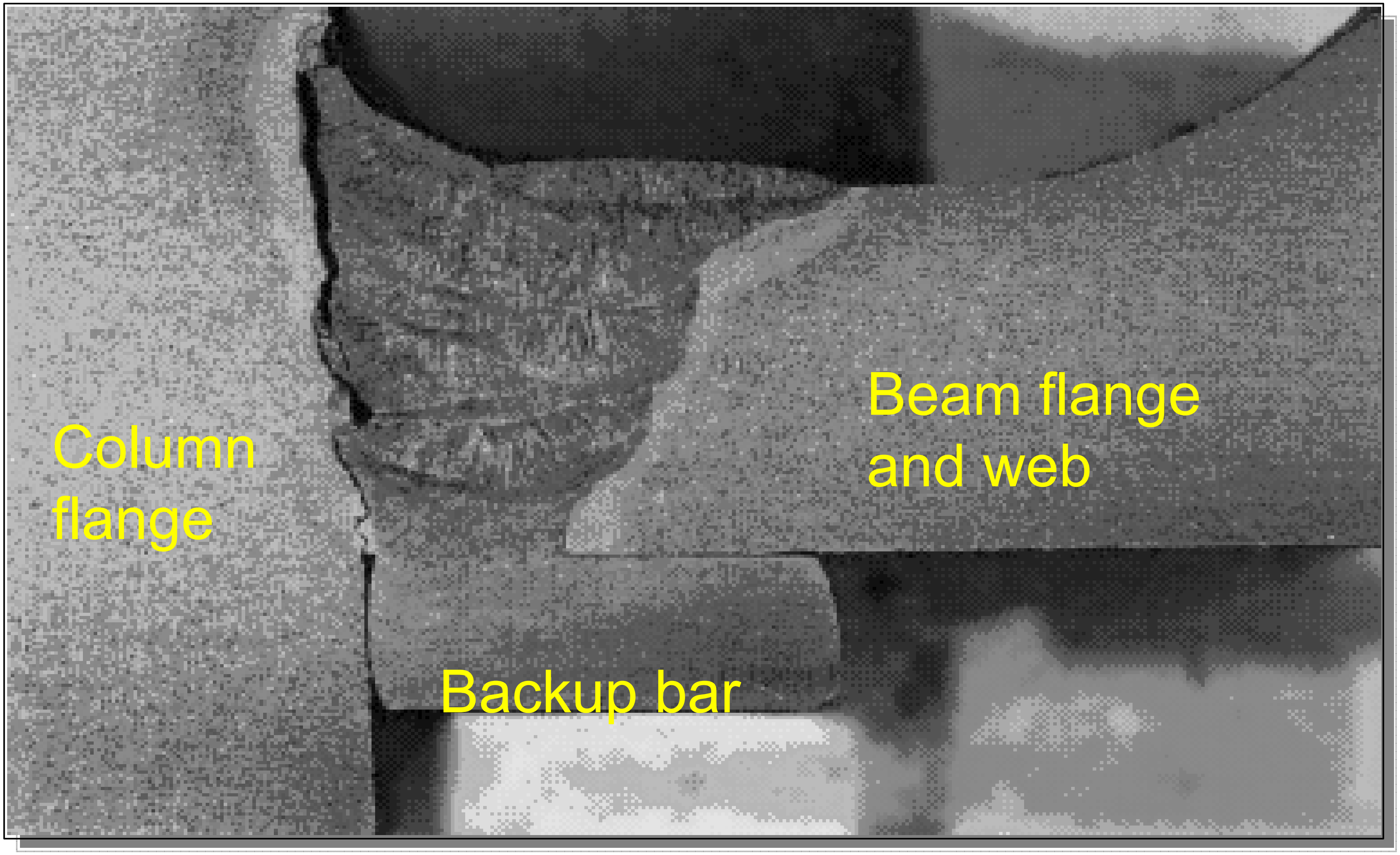
Beam Bottom Flange Weld Fracture Causing a Column Divot Fracture

Northridge Failure

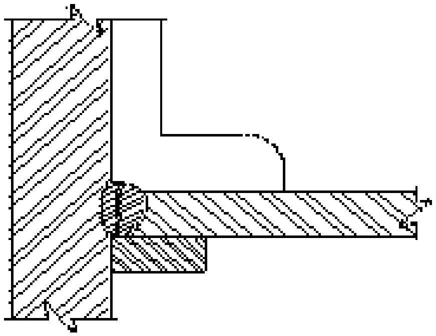
- Crack through weld
- Note backup bar
  and runoff tab



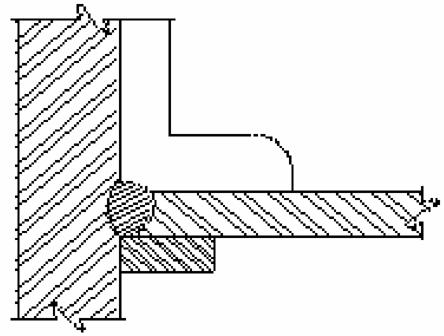
Northridge Failure



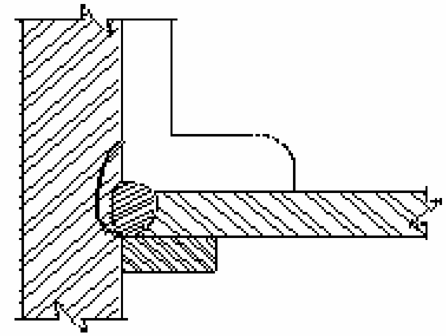
Northridge Failures



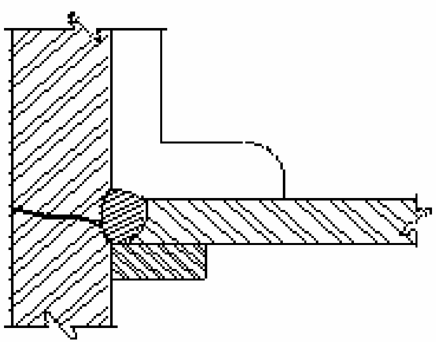
Weld



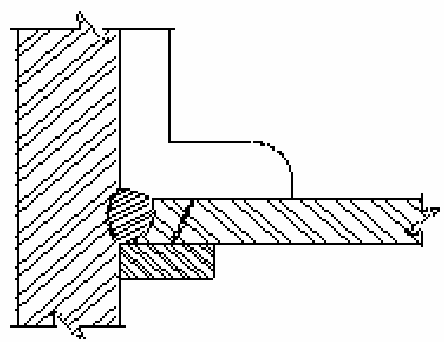
Weld Fusion



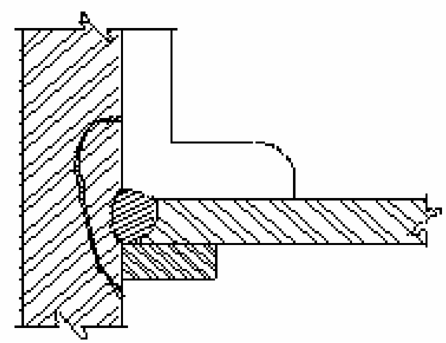
Column Divot



Column Flange

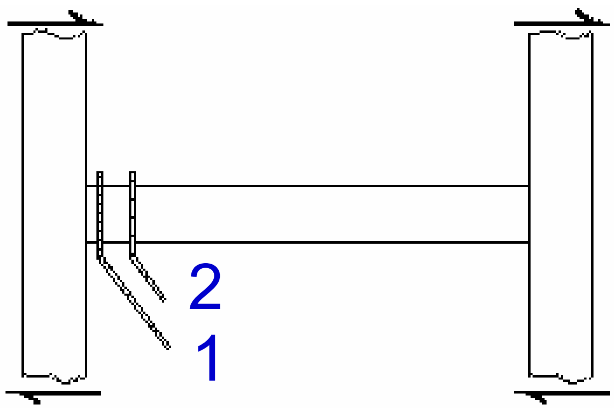


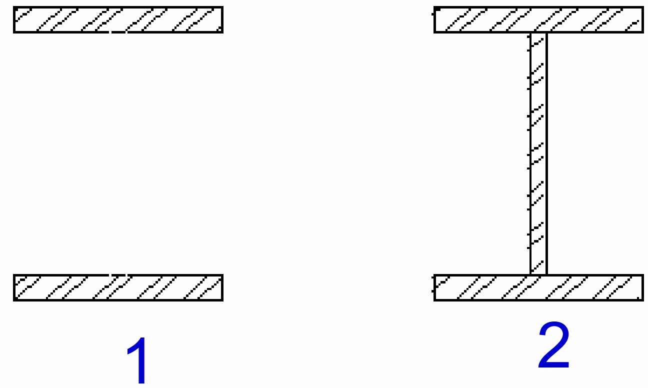
HAZ



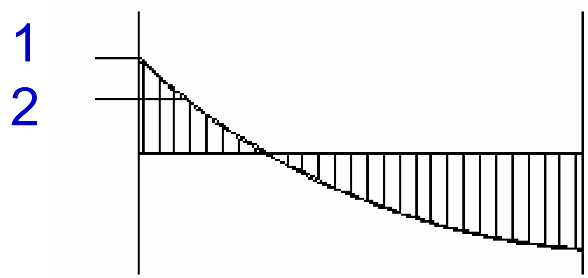
Lamellar Tear

Flexural Mechanics at a Joint

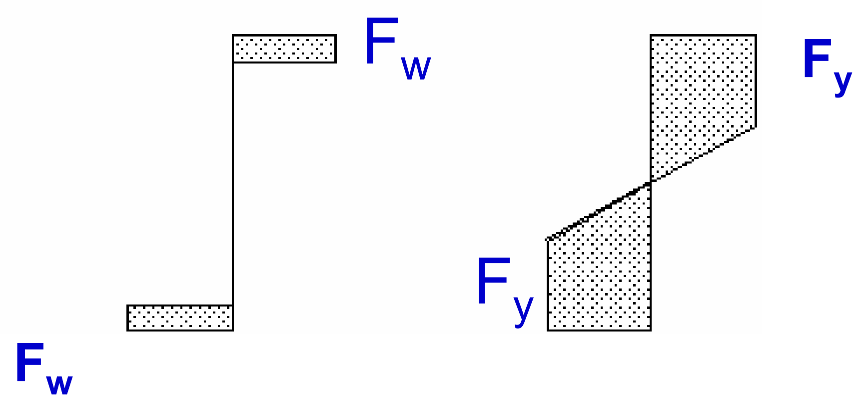




Cross Sections



Beam Moment

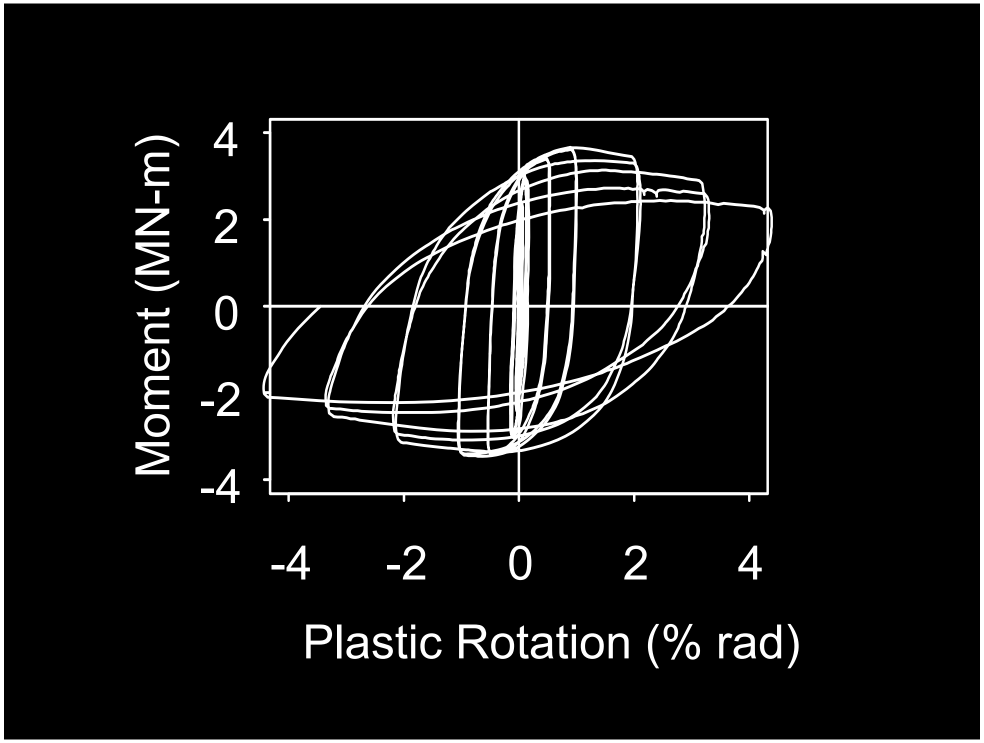


$$F_w \cdot Z_1 > F_y \cdot Z_2$$

Welded Steel Frames

- Northridge showed serious flaws. Problems correlated with:
  - Weld material, detail concept and workmanship
  - Beam yield strength and size
  - Panel zone yield
- Repairs and new design
  - Move yield away from column face (cover plates, haunches, "dog bone")
  - Verify through tests
- SAC Project: FEMA Publications 350 through 354

Reduced Beam Section (RBS) Test Specimen
SAC Joint Venture

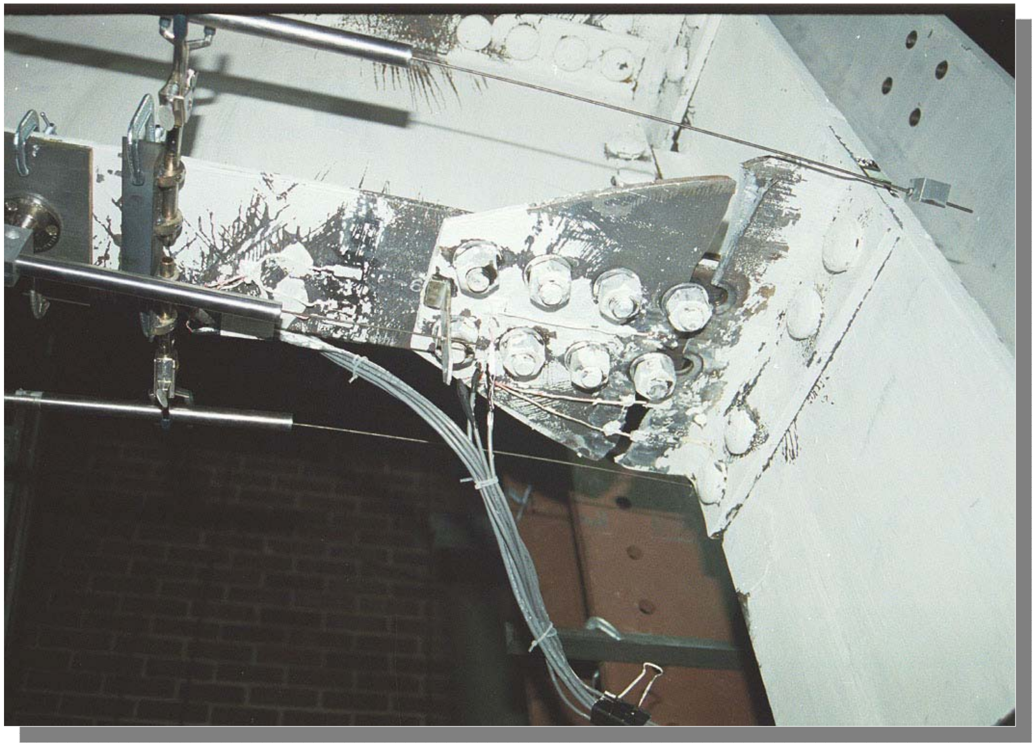


Graphics courtesy of Professor Chia-Ming Uang, University of California San Diego

T-stub Beam-Column Test
SAC Joint Venture

Photo courtesy of Professor Roberto Leon, Georgia Institute of Technology

T-Stub Failure Mechanisms

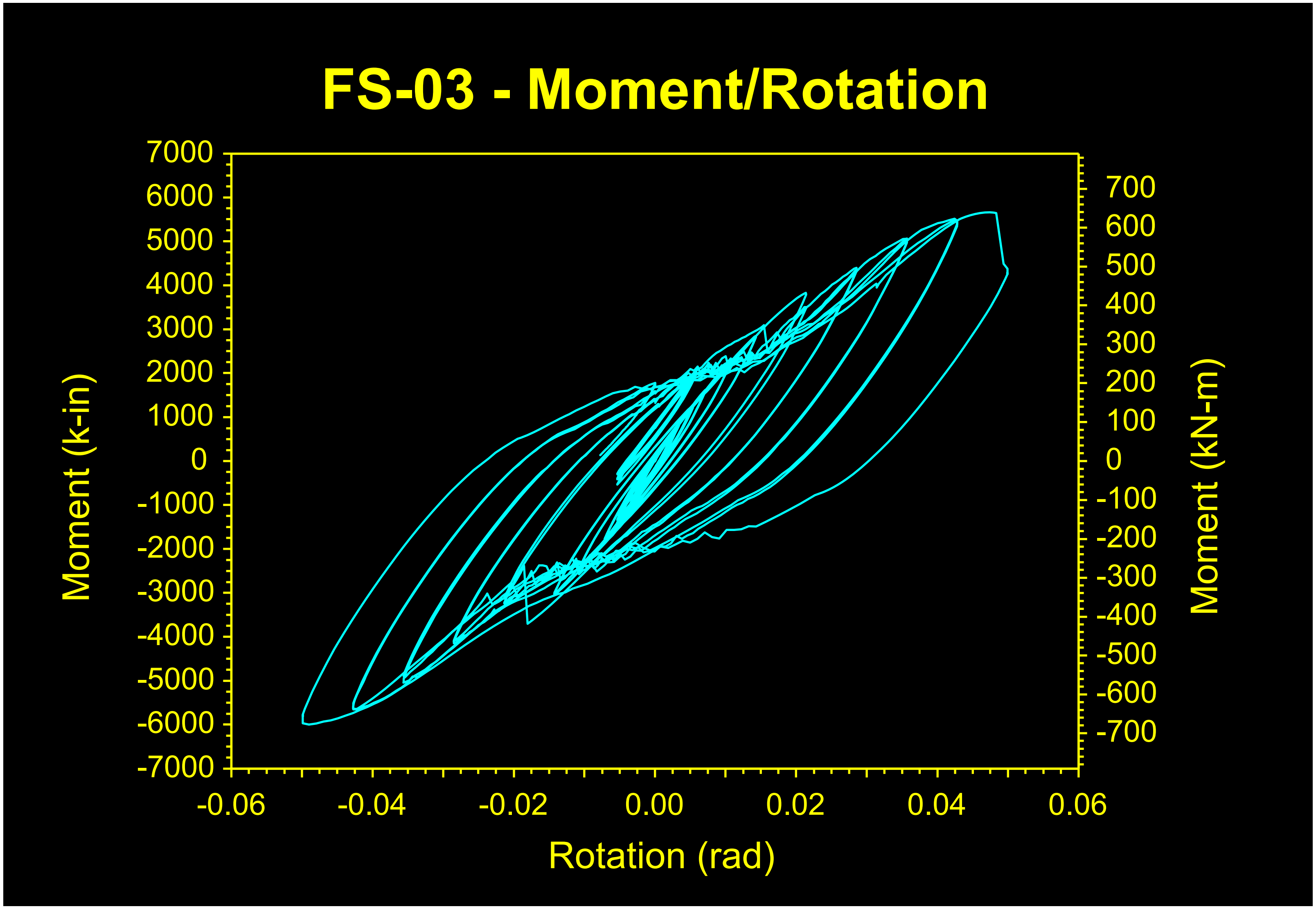


Net section fracture in stem of T-stub

Plastic hinge formation -- flange and web local buckling

Photos courtesy of Professor Roberto Leon, Georgia Institute of Technology

T-Stub Connection Moment Rotation Plot

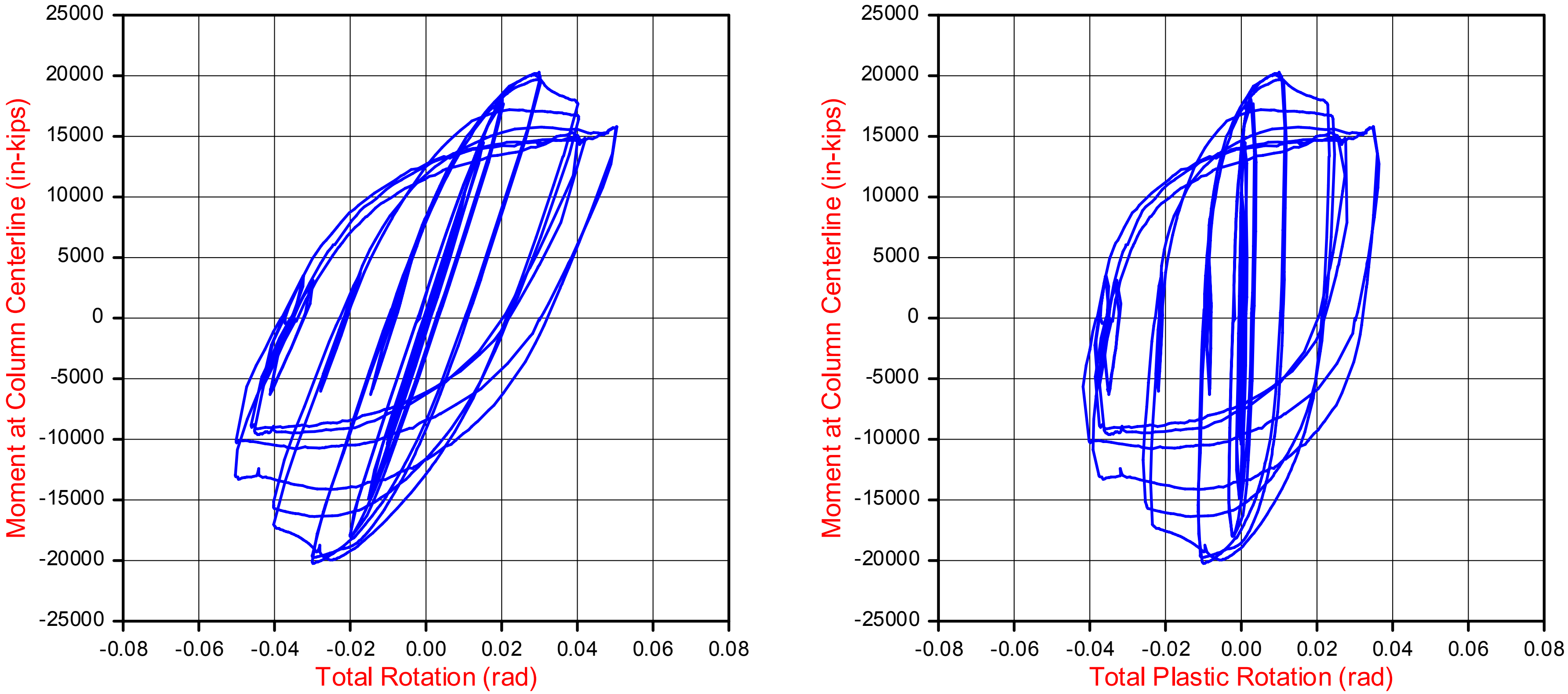


Graphic courtesy of Professor Roberto Leon, Georgia Institute of Technology

Extended Moment End-Plate Connection Results

Photo courtesy of Professor Thomas Murray, Virginia Tech

Extended Moment End-Plate Connection Results



(a) Moment vs Total Rotation **(b) Moment vs Plastic Rotation**

Graphics courtesy of Professor Thomas Murray, Virginia Tech

Ductility of Steel Frame Joints

Limits

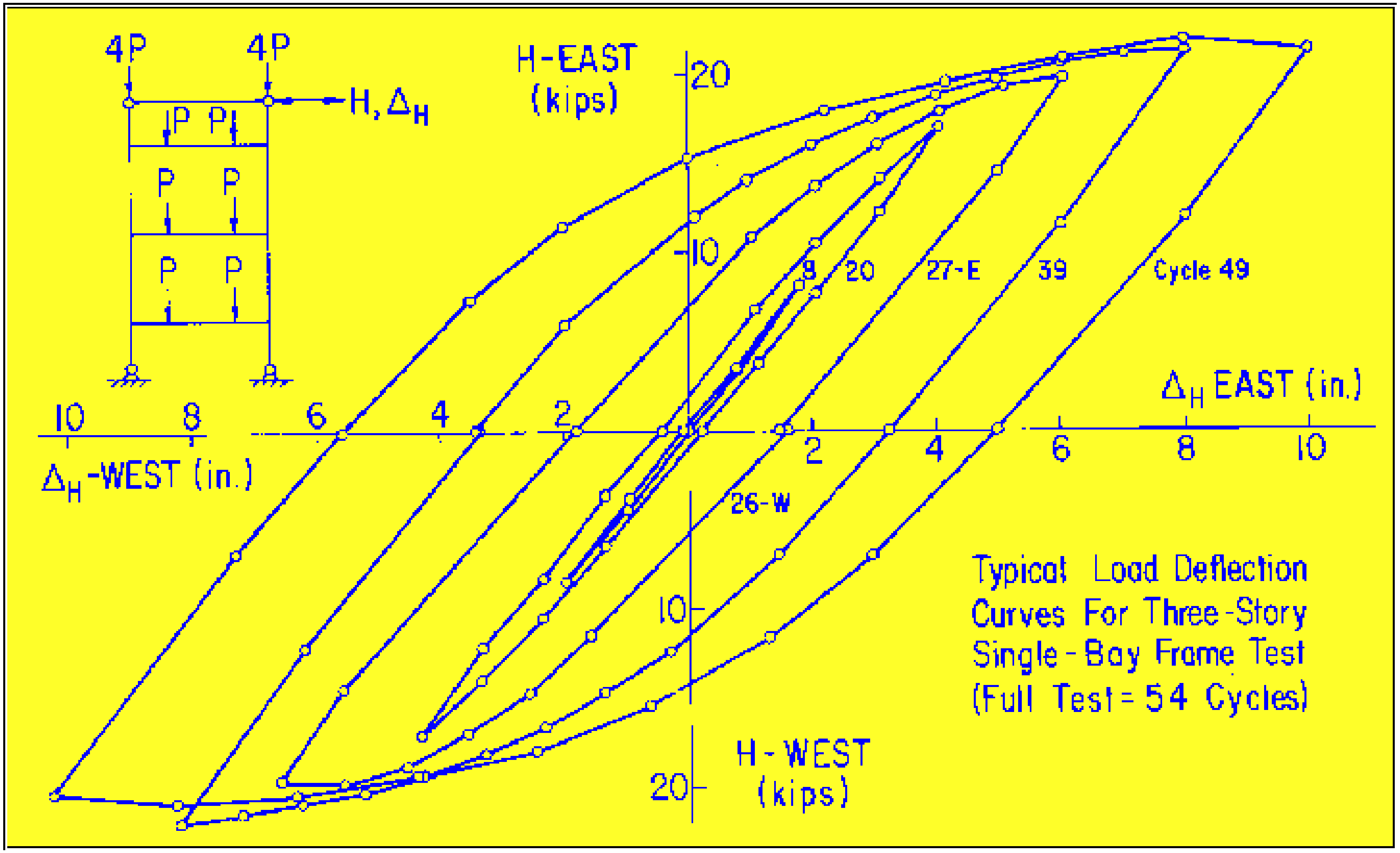
Welded Joints
- Brittle fracture of weld
- Lamellar tearing of base metal
- Joint design, testing, and inspection

Bolted Joints
- Fracture at net cross-section
- Excessive slip

Joint Too Weak For Member
- Shear in joint panel

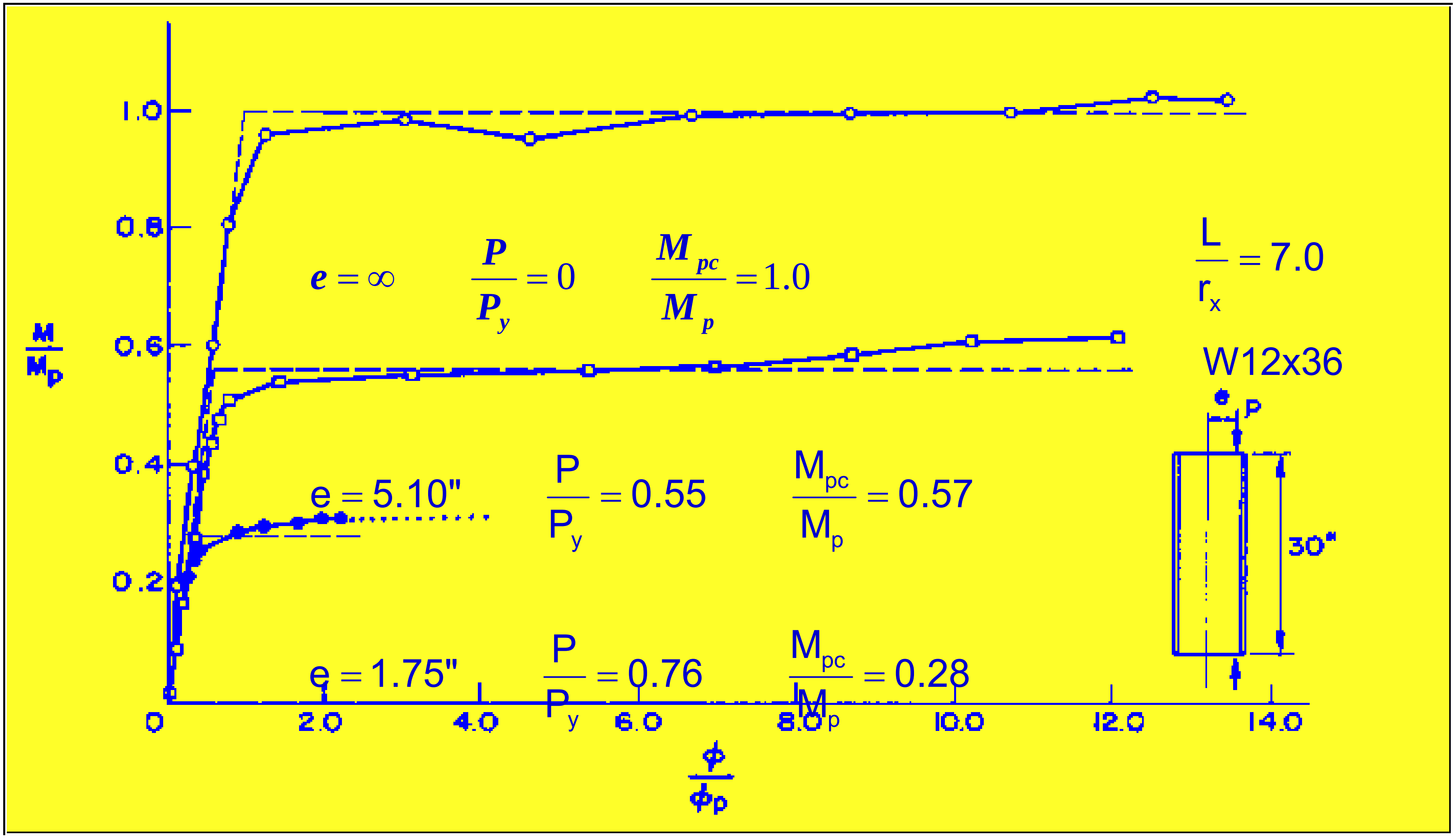
Multistory Frame

Laboratory Test



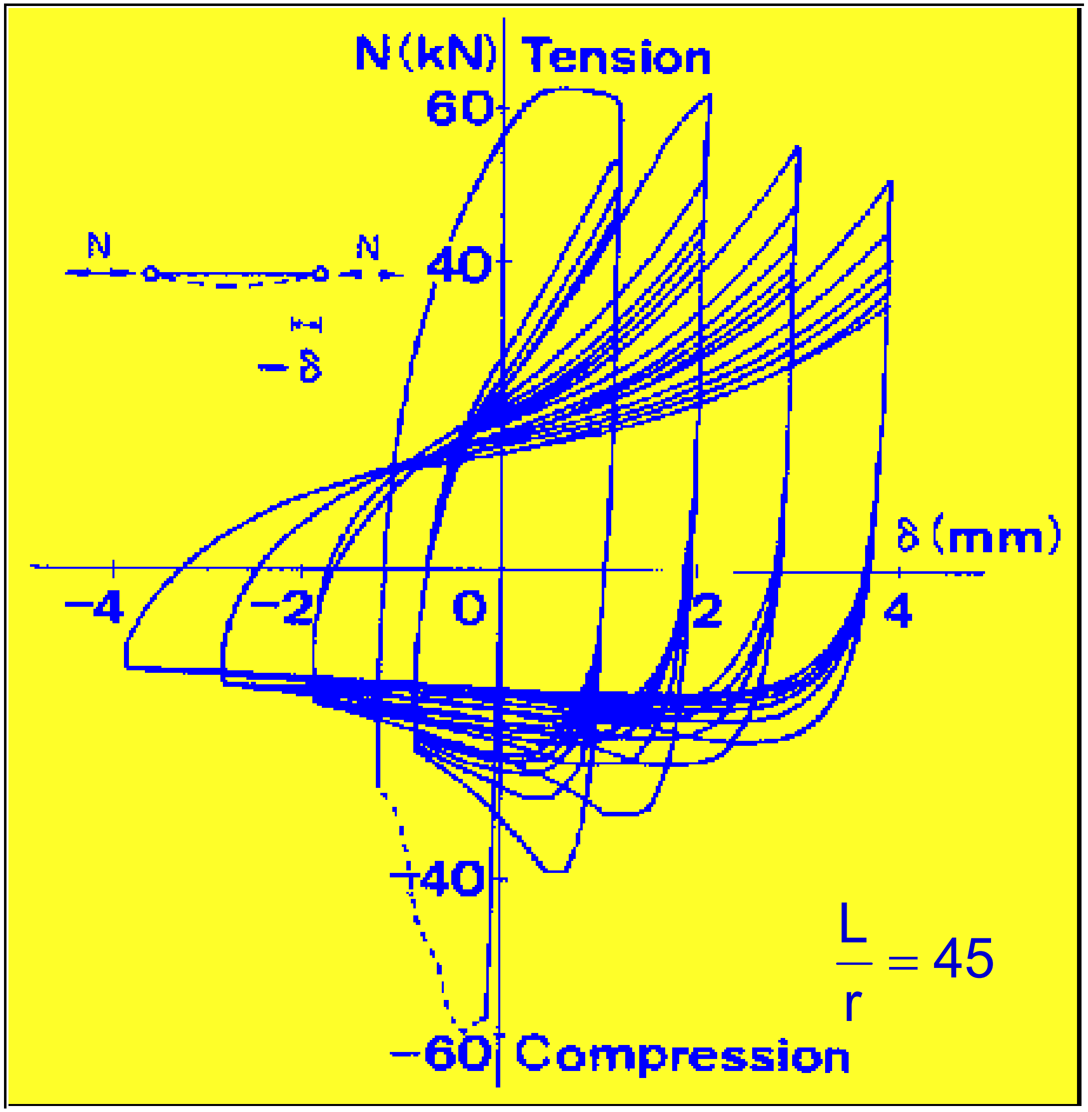
Flexural Ductility

Effect of Axial Load



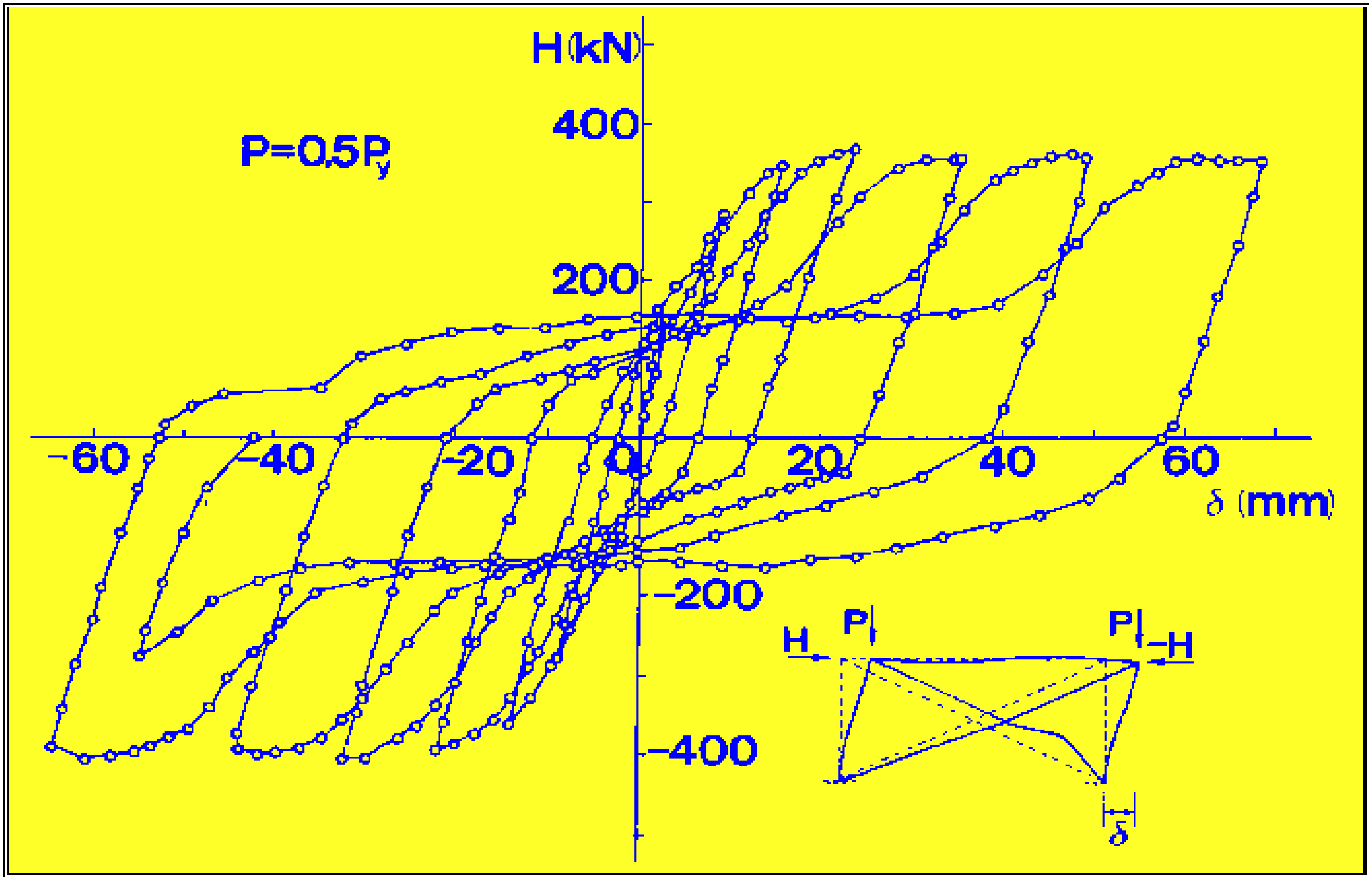
Axial Strut

Laboratory test



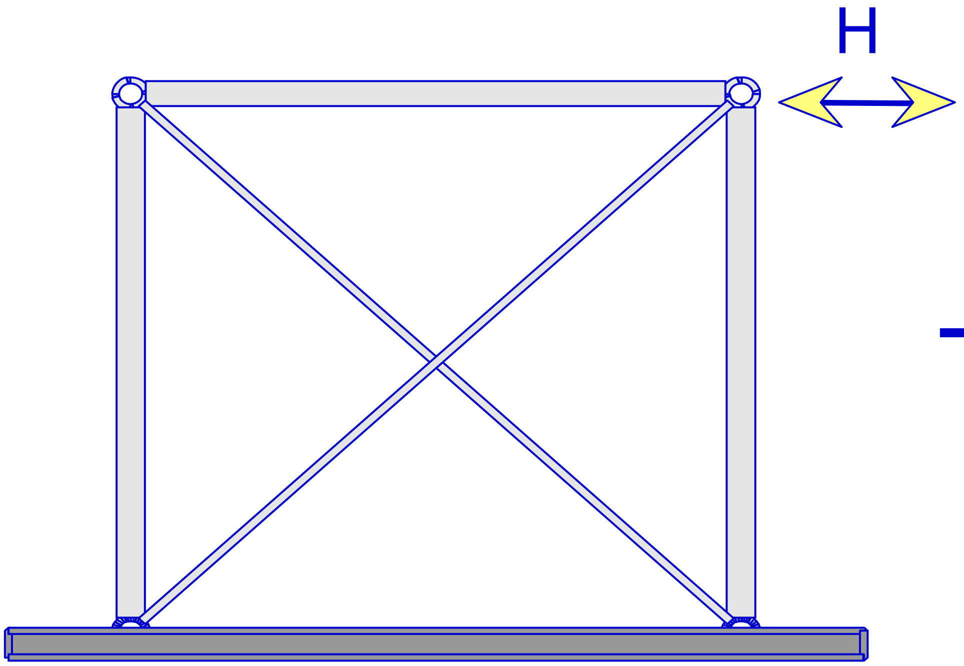
Cross Braced Frame

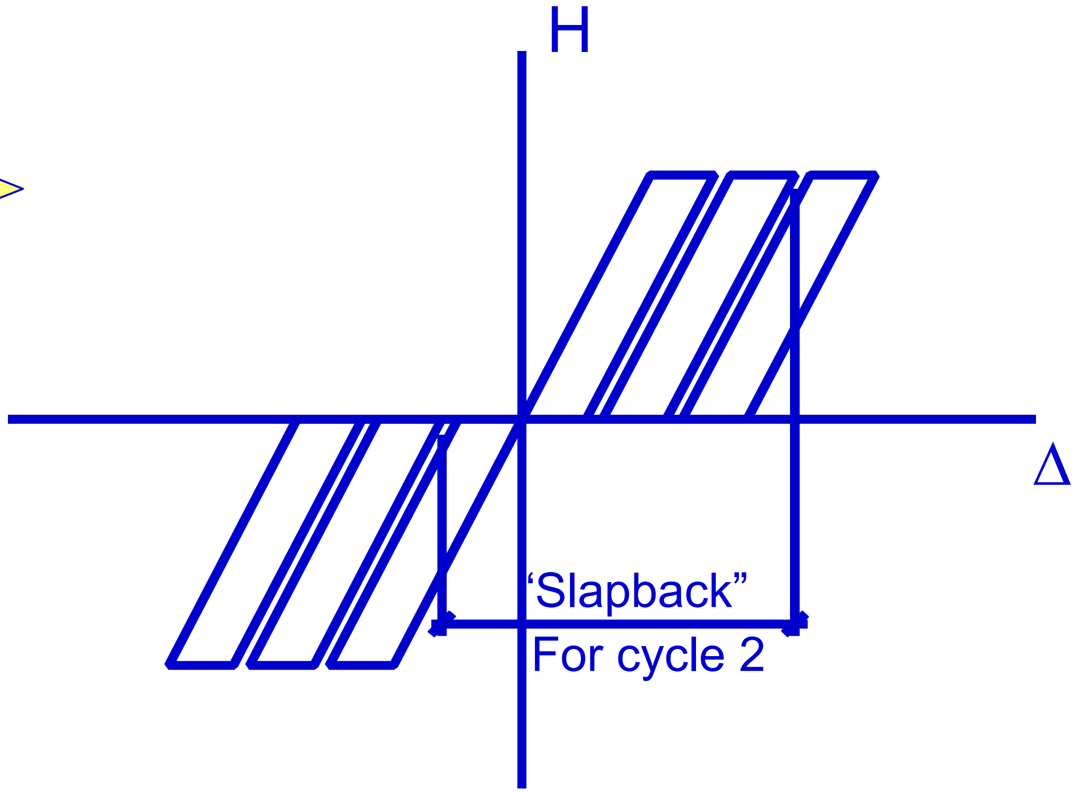
Laboratory test



Tension Rod (Counter) Bracing

Conceptual Behavior

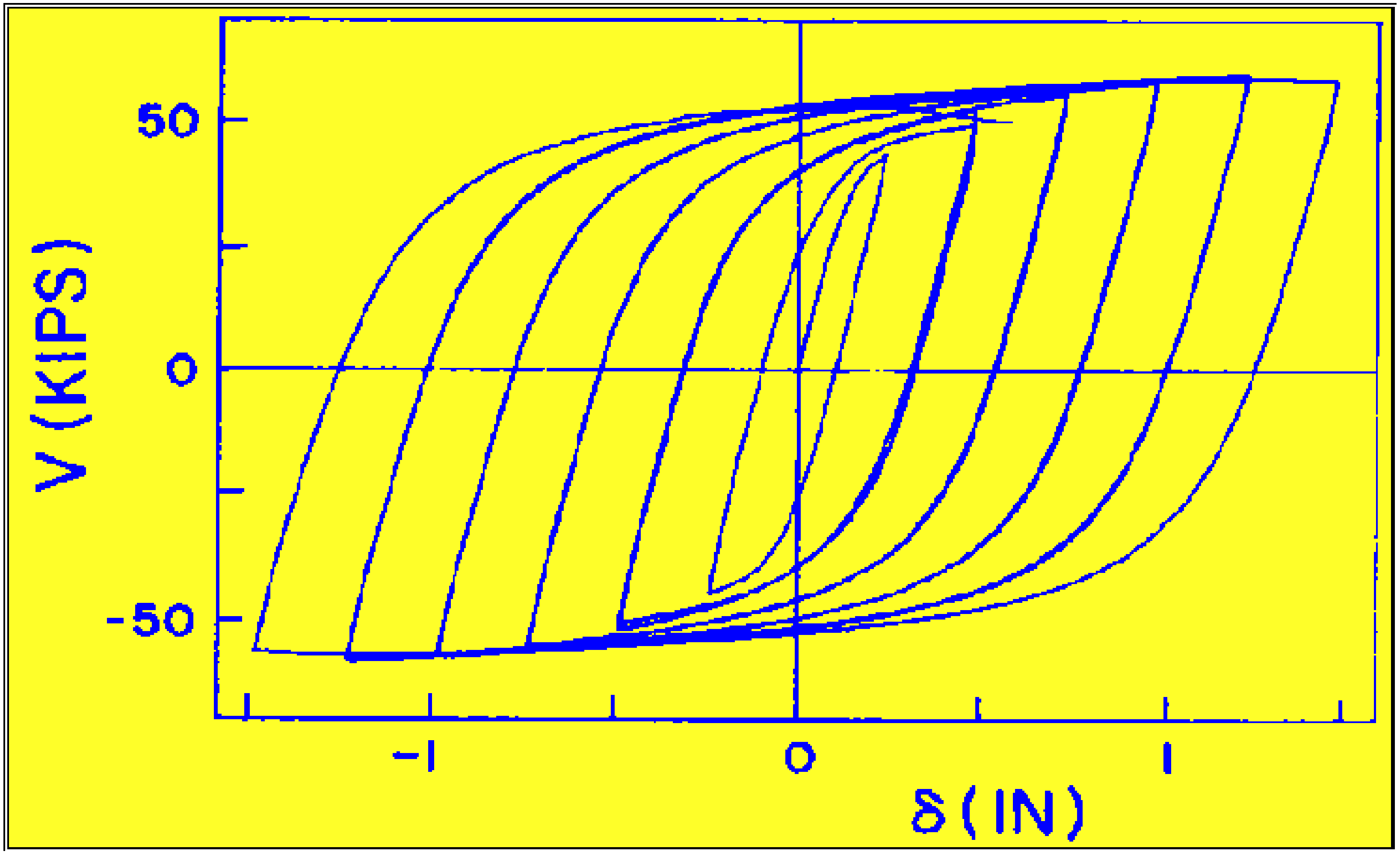




Eccentrically Braced Frame

Eccentrically Braced Frame

Lab test of link



Steel Behavior

- Ductility
  - Material inherently ductile
  - Ductility of structure < ductility of material
- Damping
  - Welded structures have low damping
  - More damping in bolted structures due to slip at connections
  - Primary energy absorption is yielding of members

Steel Behavior

- Buckling
  - Most common steel failure under earthquake loads
  - Usually not ductile
  - Local buckling of portion of member
  - Global buckling of member
  - Global buckling of structure
- Fracture
  - Nonductile failure mode under earthquake loads
  - Heavy welded connections susceptible

NEHRP Recommended Provisions
Steel Design

- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength

ANSI/AISC 360-05
An American National Standard

Specification for Structural Steel Buildings

March 9, 2005

Supersedes the *Load and Resistance Factor Design Specification for Structural Steel Buildings* dated December 27, 1999, the *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* dated June 1, 1989, including Supplement No. 1, the *Specification for Allowable Stress Design of Single-Angle Members* dated June 1, 1989, the *Load and Resistance Factor Design Specification for Single-Angle Members* dated November 10, 2000, and the *Load and Resistance Factor Design Specification for the Design of Steel Hollow Structural Sections* dated November 10, 2000, and all previous versions of these specifications.

Approved by the AISC Committee on Specifications and issued by the AISC Board of Directors

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

ANSI/AISC 341-05
ANSI/AISC 341s1-05
An American National Standard

Seismic Provisions for Structural Steel Buildings
Including Supplement No. 1

Seismic Provisions for Structural Steel Buildings dated **March 9, 2005**
and Supplement No. 1 dated xxx, 2005

Supersedes the *Seismic Provisions for Structural Steel Buildings*
dated May 21, 2002
and all previous versions

Approved by the
AISC Committee on Specifications and
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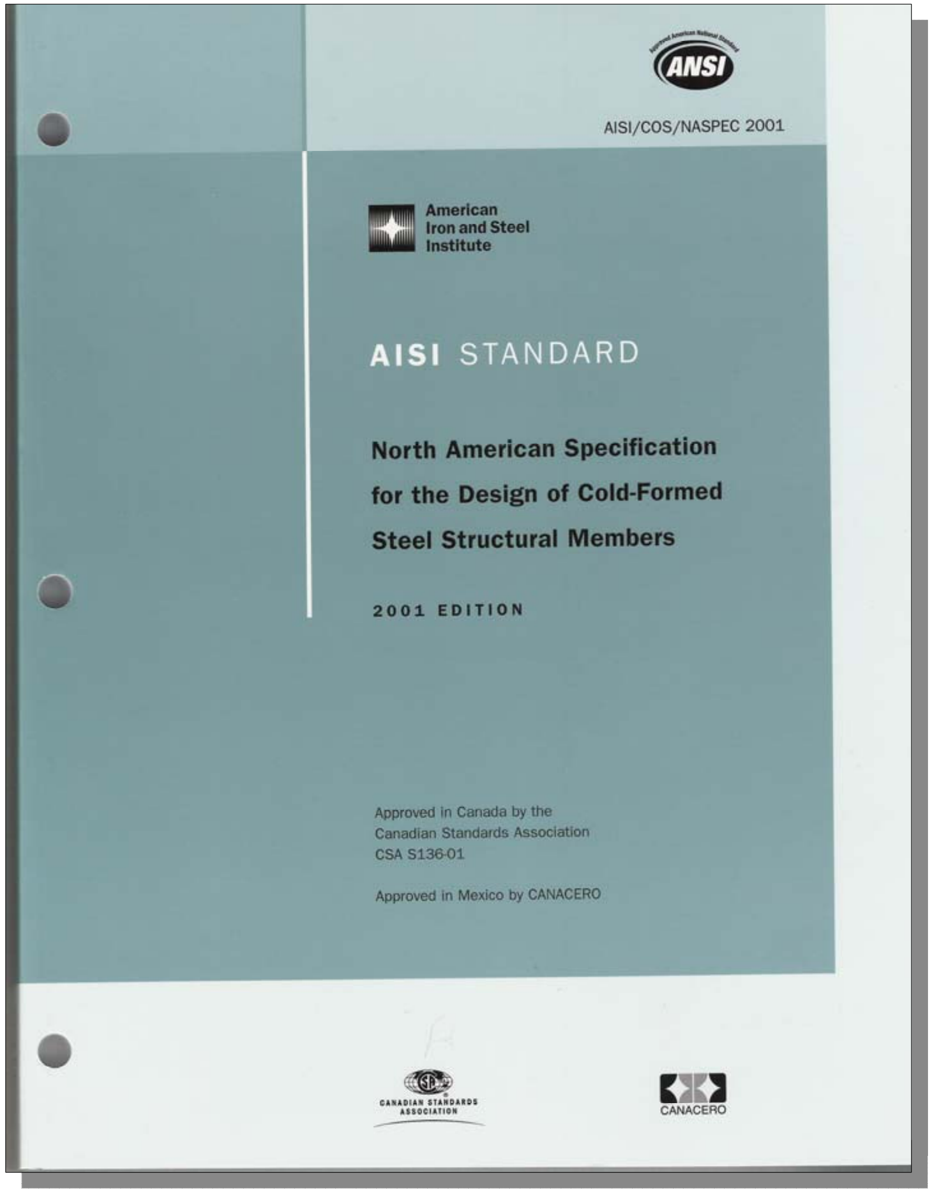
AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

Using Reference Standards
Structural Steel

Both the AISC LRFD and ASD methodologies are presented in a unified format in both the *Specification for Structural Steel Buildings* and the *Seismic Provisions for Structural Steel Buildings*.


AISI/COS/NASPEC 2001

American Iron and Steel Institute

AISI STANDARD

North American Specification for the Design of Cold-Formed Steel Structural Members

2001 EDITION

Approved in Canada by the Canadian Standards Association CSA S136-01

Approved in Mexico by CANACERO

CANADIAN STANDARDS ASSOCIATION

CANACERO

Other Steel Members

Steel Joist Institute

Standard Specifications, 2002

Steel Cables

ASCE 19-1996

Steel Deck Institute

Diaphragm Design Manual, 3rd Ed., 2005

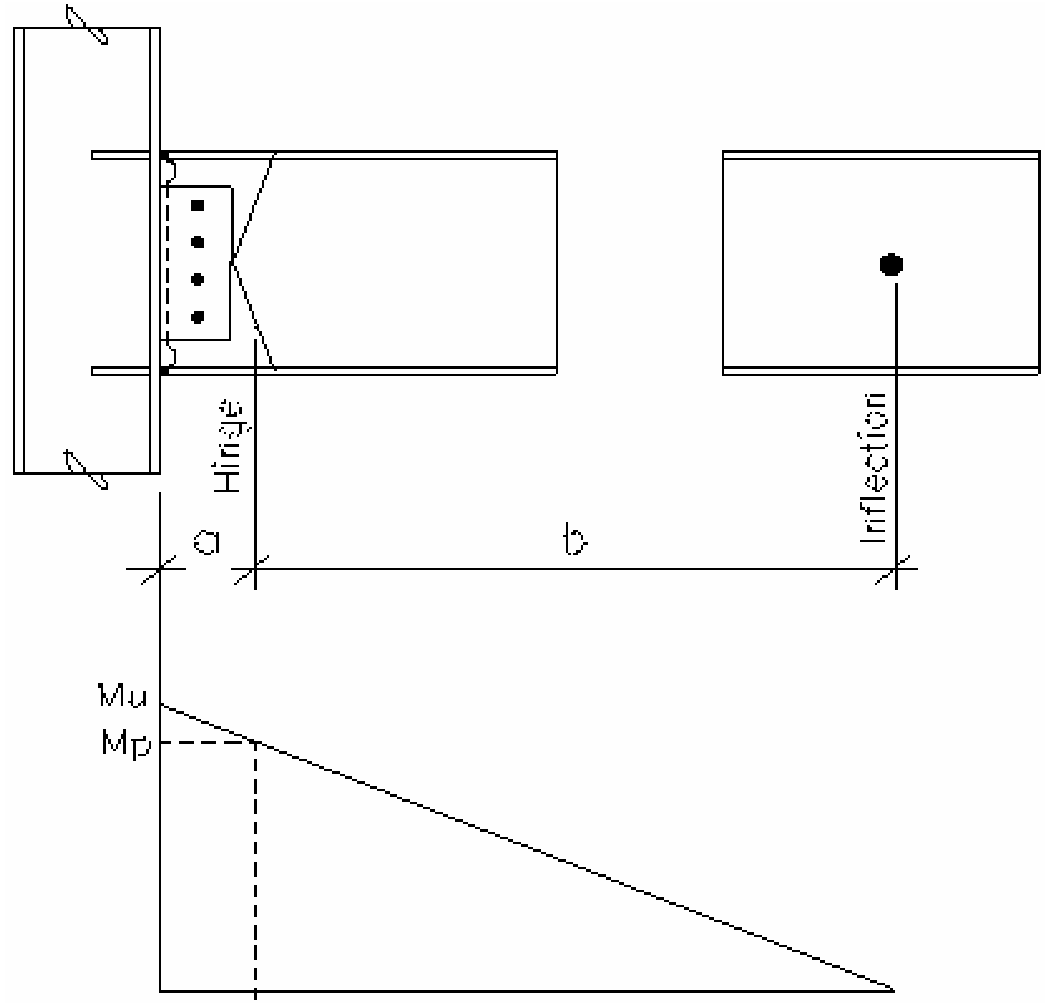
NEHRP Recommended Provisions
Steel Design

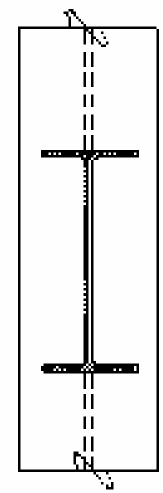
- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames

Steel Moment Frame Joints

Frame	Test	θ_i	Details
Special	Req'd	0.04	Many
Intermediate	Req'd	0.02	Moderate
Ordinary	Allowed	NA	Few

Steel Moment Frame Joints





$$M_u \approx M_p \cdot \frac{a+b}{b}$$

$$F_y^* = R_y \cdot F_y$$

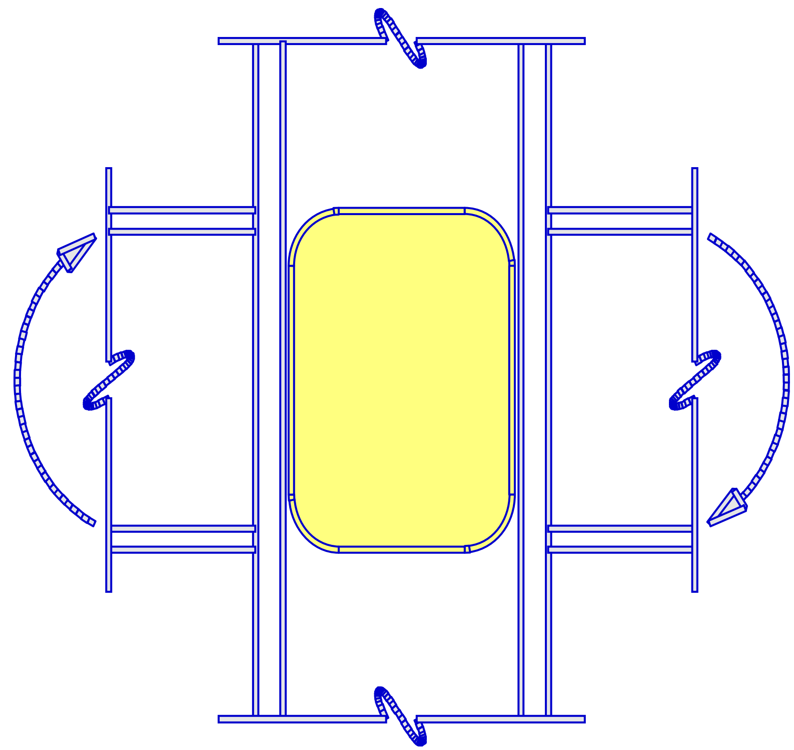
$$F_u \approx F_y^* Z \cdot \frac{a+b}{b} \cdot \frac{1}{A_f d} \approx 1.7 F_y^*$$

Panel Zones

Special and intermediate moment frame:

- Shear strength demand:

  Basic load combination or $\phi R_y M_p$ of beams
- Shear capacity equation
- Thickness (for buckling)
- Use of doubler plates



Steel Moment Frames

- Beam shear: $1.1R_yM_p$ + gravity
- Beam local buckling
  - Smaller b/t than LRFD for plastic design
- Continuity plates in joint per tests
- Strong column - weak beam rule
  - Prevent column yield except in panel zone
  - Exceptions: Low axial load, strong stories, top story,and non-SRS columns

Steel Moment Frames

- Lateral support of column flange
  - Top of beam if column elastic
  - Top and bottom of beam otherwise
  - Amplified forces for unrestrained

- Lateral support of beams
  - Both flanges
  - Spacing < $0.086 r_y E/F_y$

Prequalified Connections

See FEMA 350: *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*

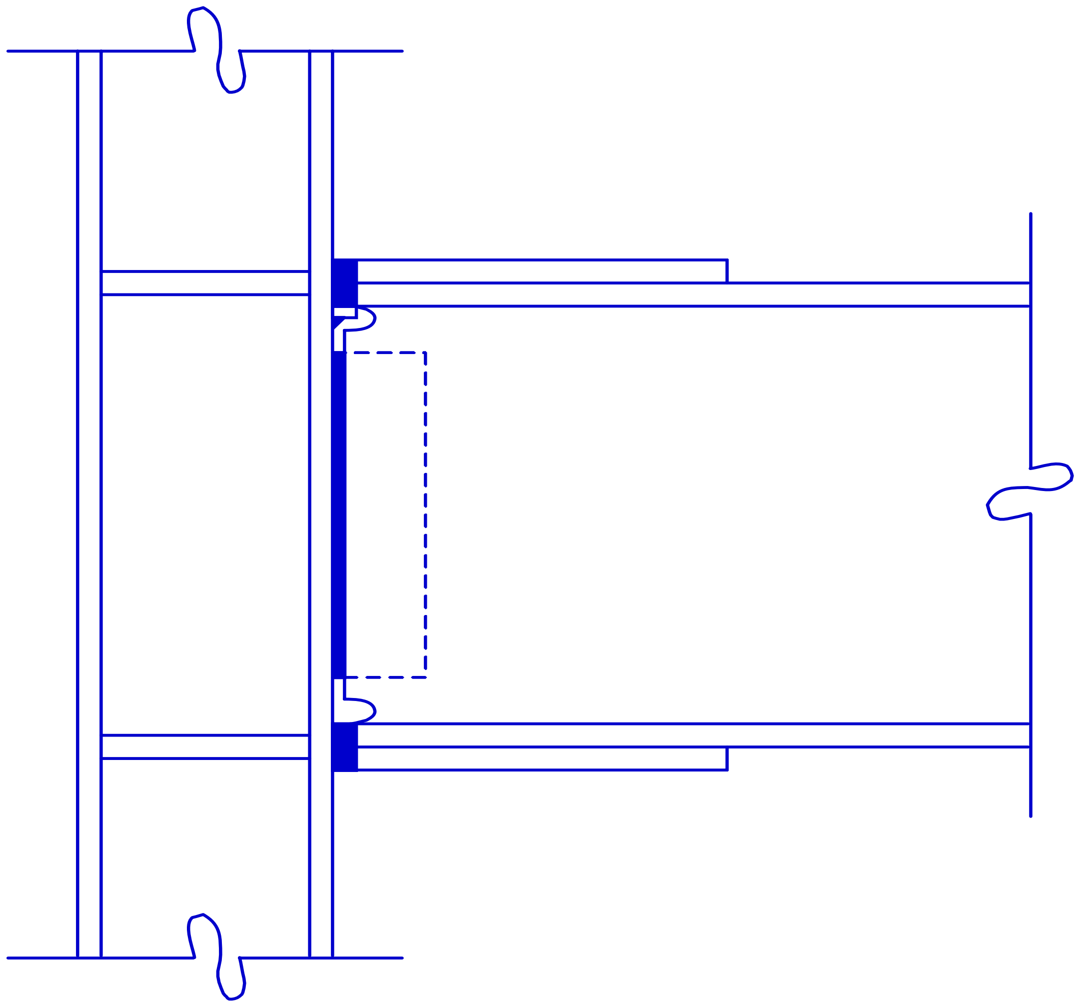
-Welded Unreinforced Flange
-Welded Free Flange Connection
-Welded Flange Plate Connection
-Reduced Beam Section Connections

-Bolted Unstiffened End Plate Connection
-Bolted Stiffened End Plate Connection
-Bolted Flange Plate Connection

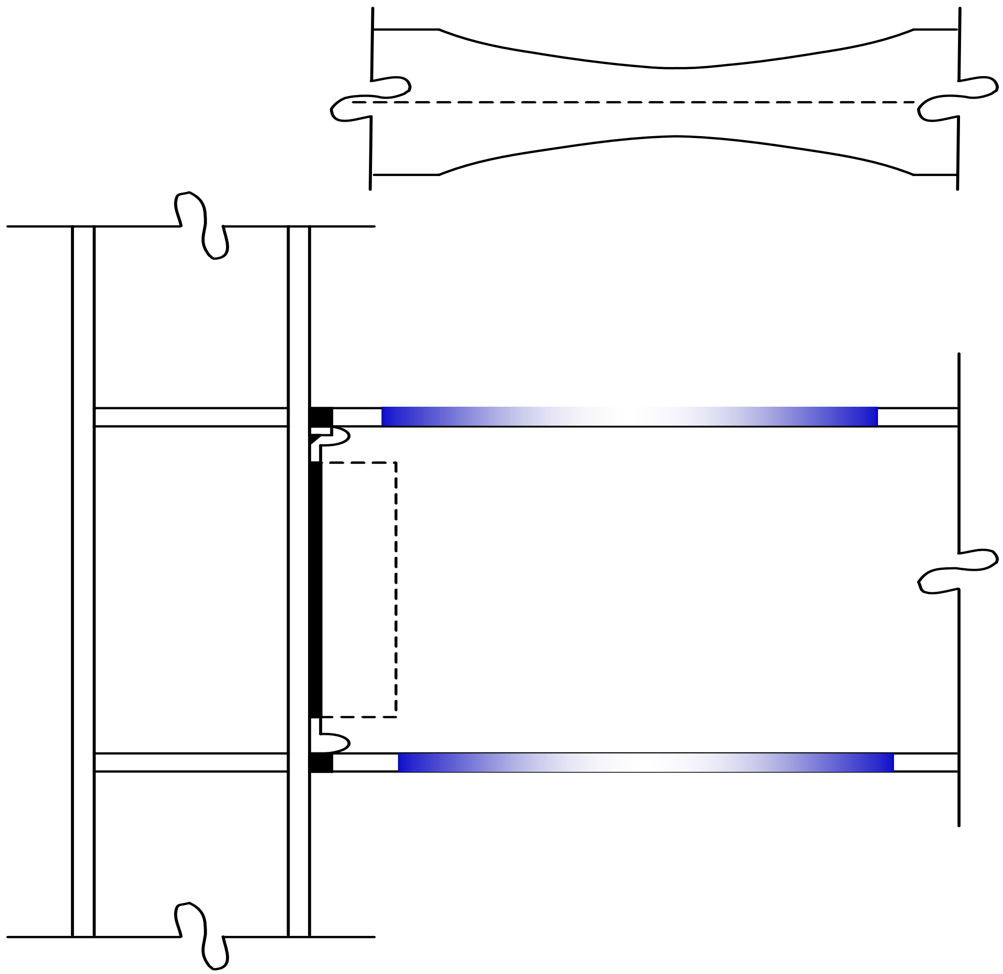
See ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*

-Reduced Beam Section Connections
-Bolted Stiffened and Unstiffened Extended Moment End Plate Connections

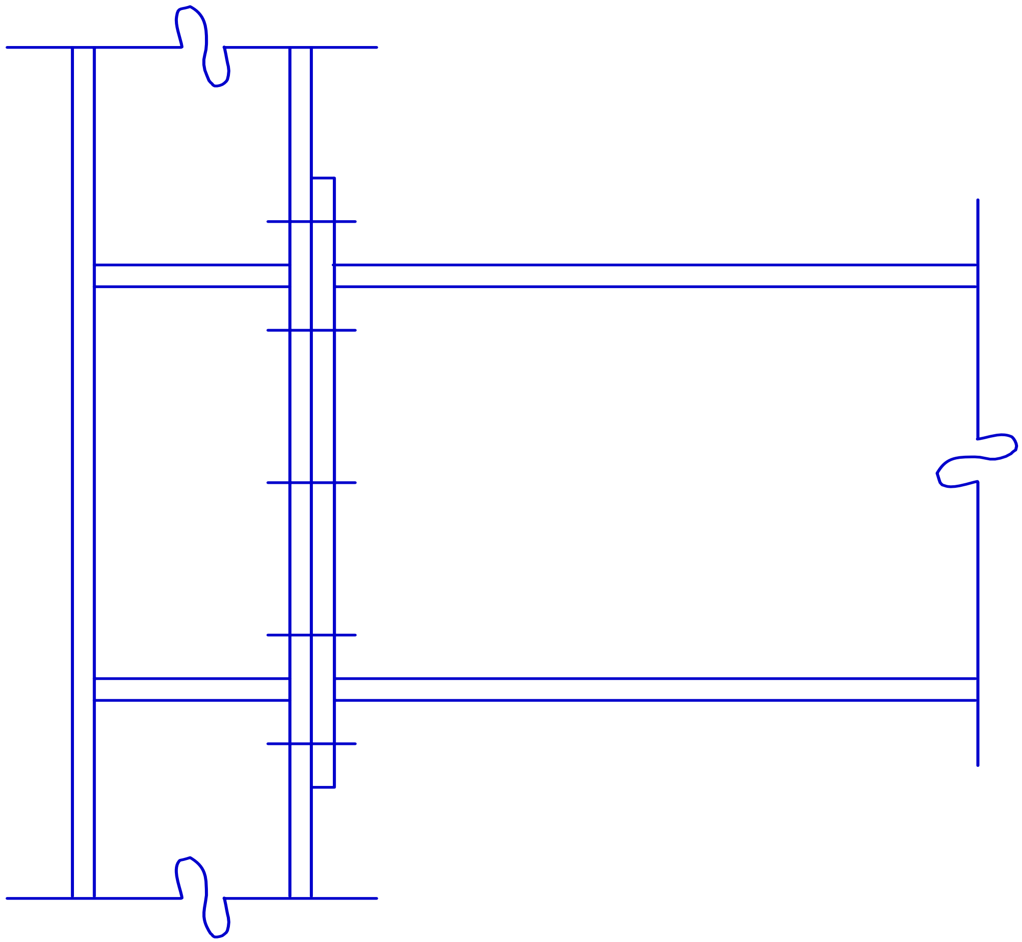
Welded Coverplates



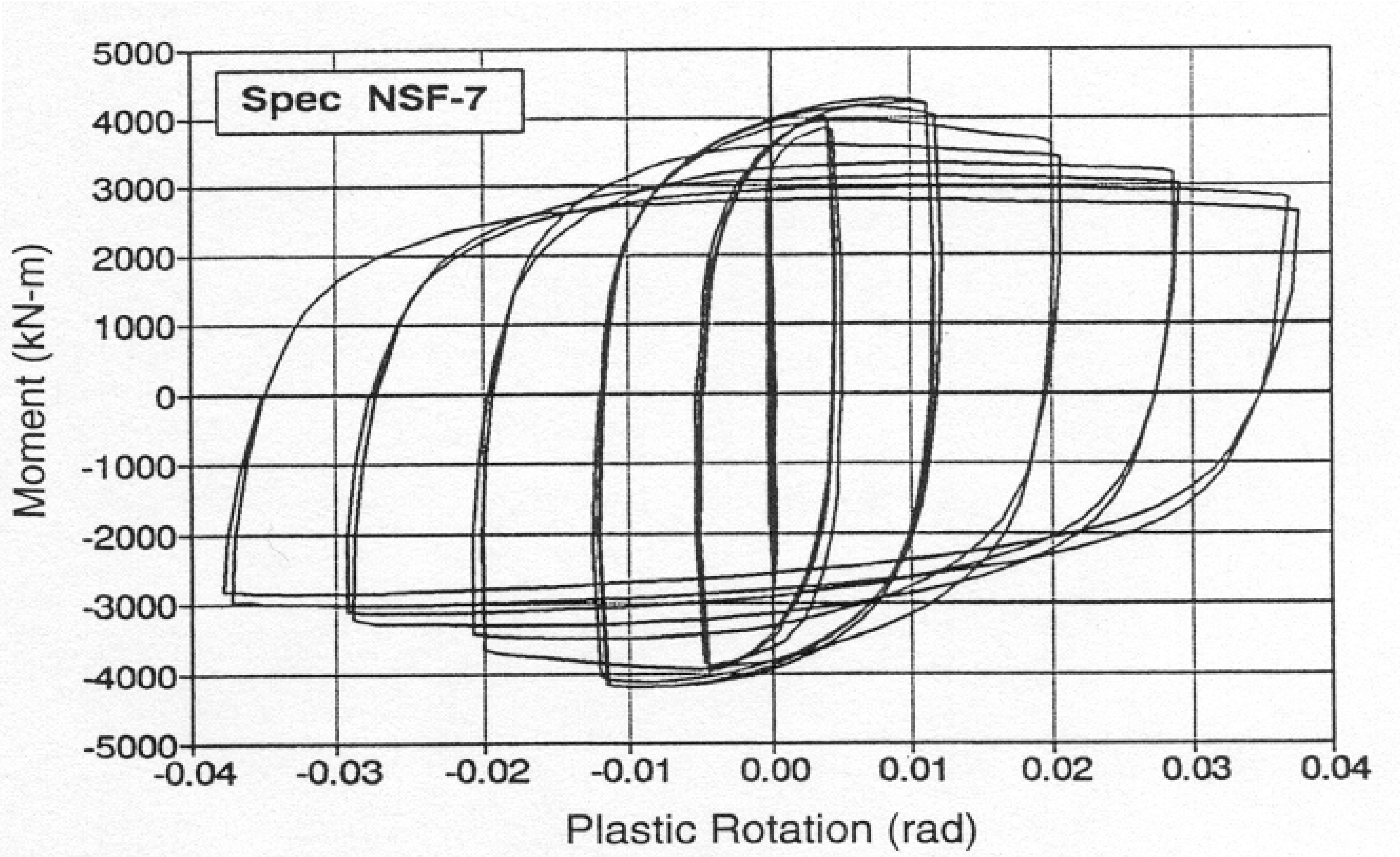
Reduced Beam Section (RBS)



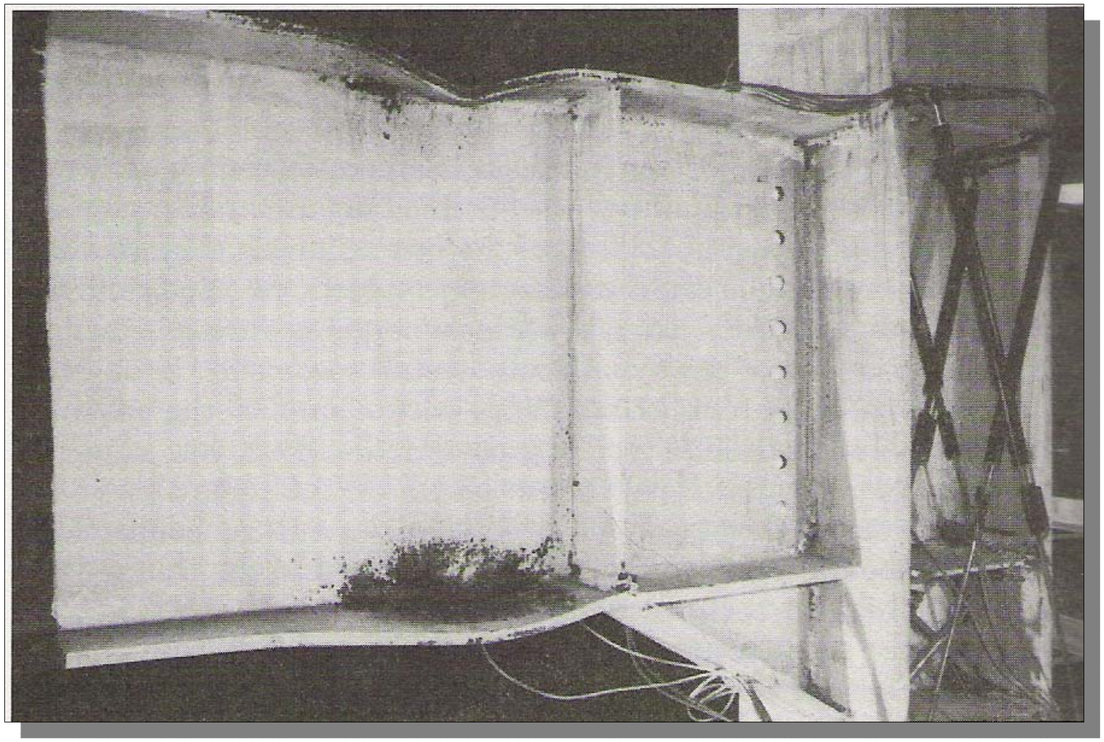
Extended End Plate

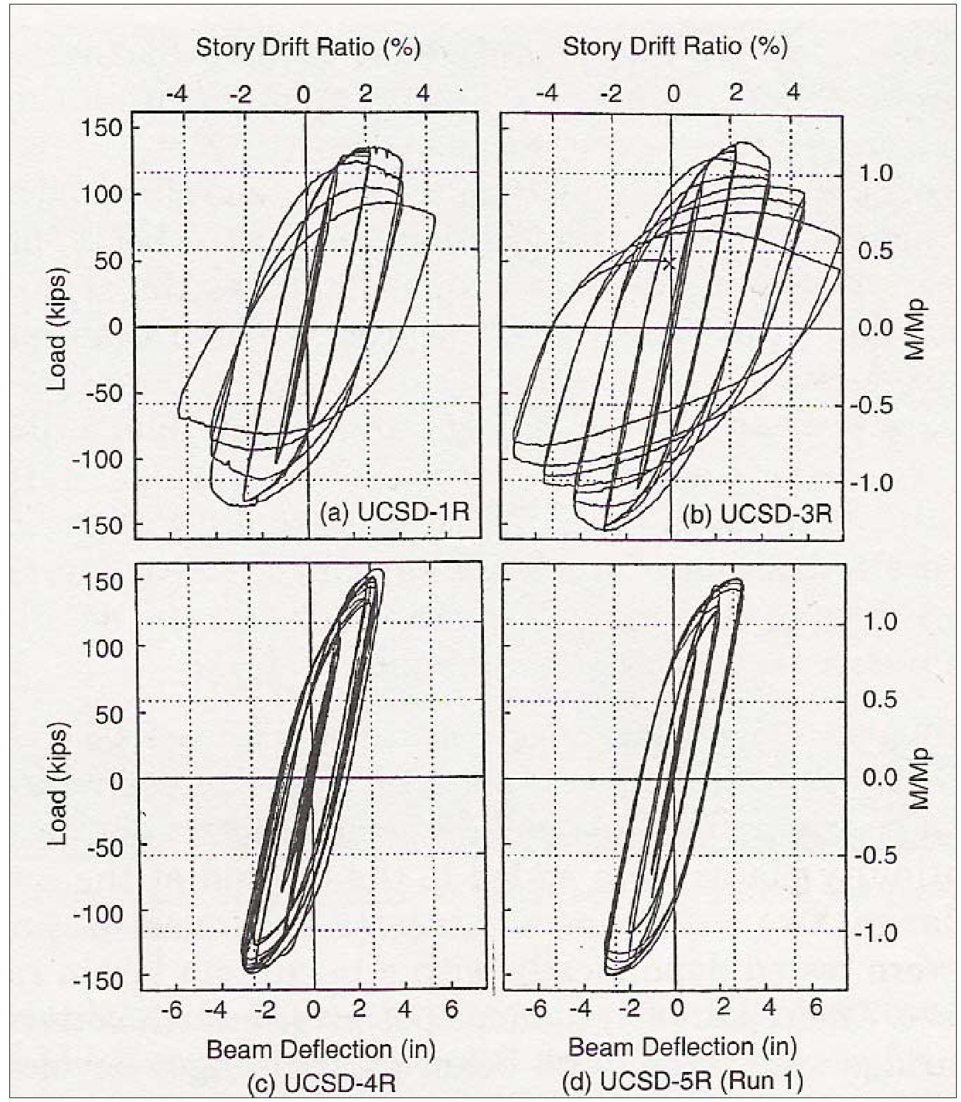


Excellent Moment Frame Behavior

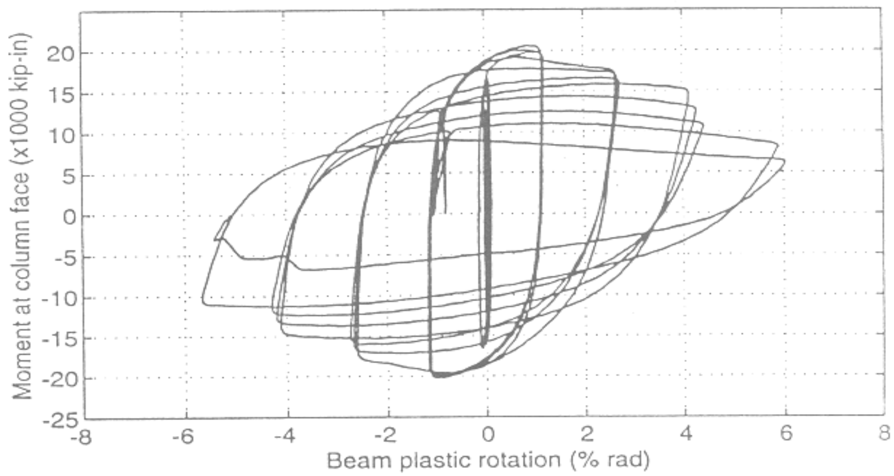


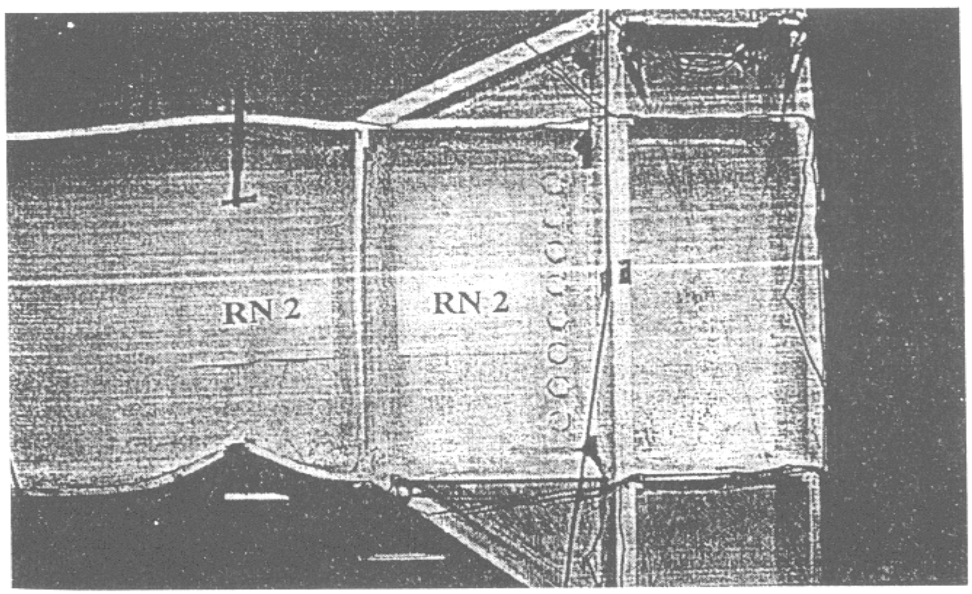
Excellent Moment Frame Behavior





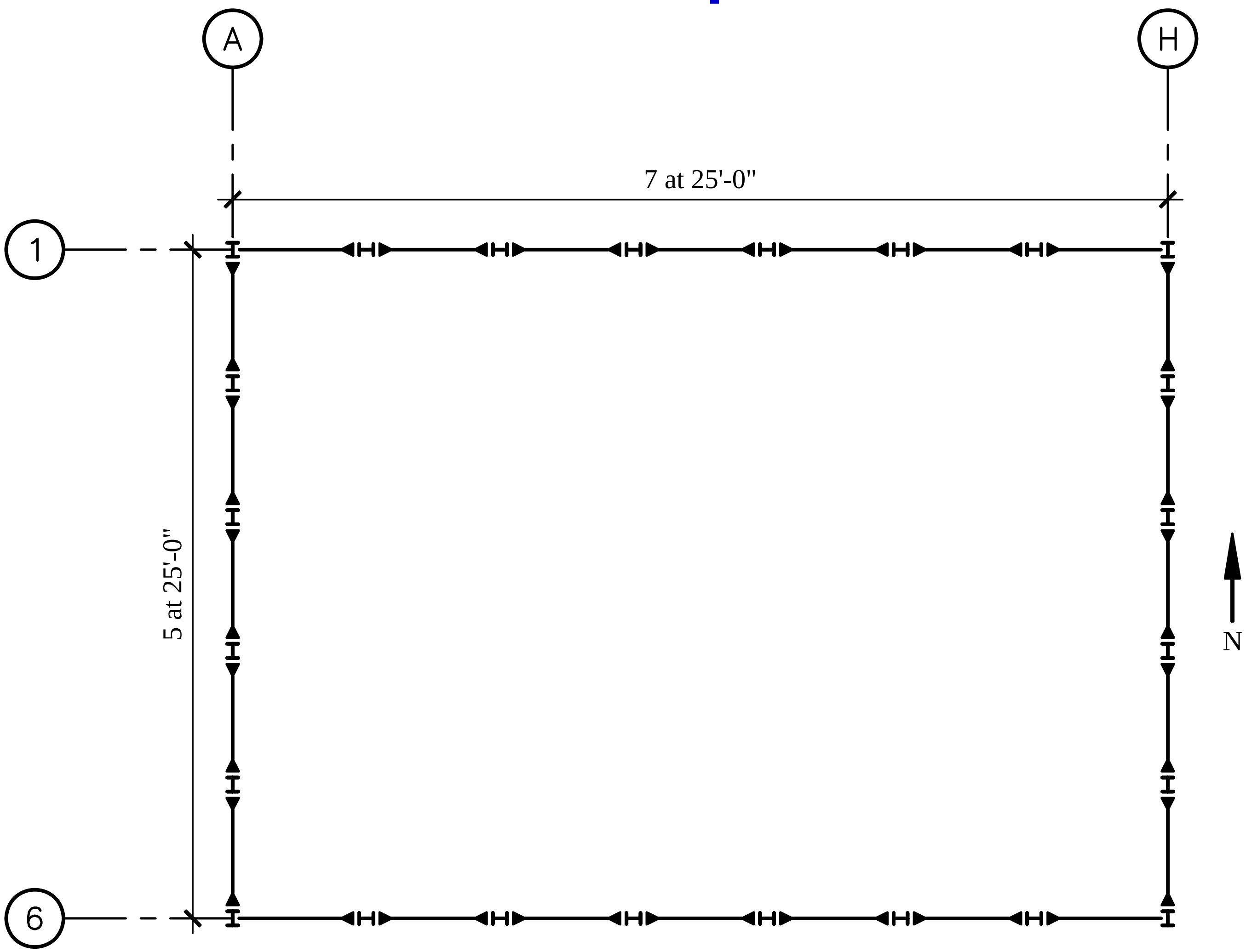
Excellent Moment Frame Behavior





Special Moment Frames

Example



Special Moment Frames

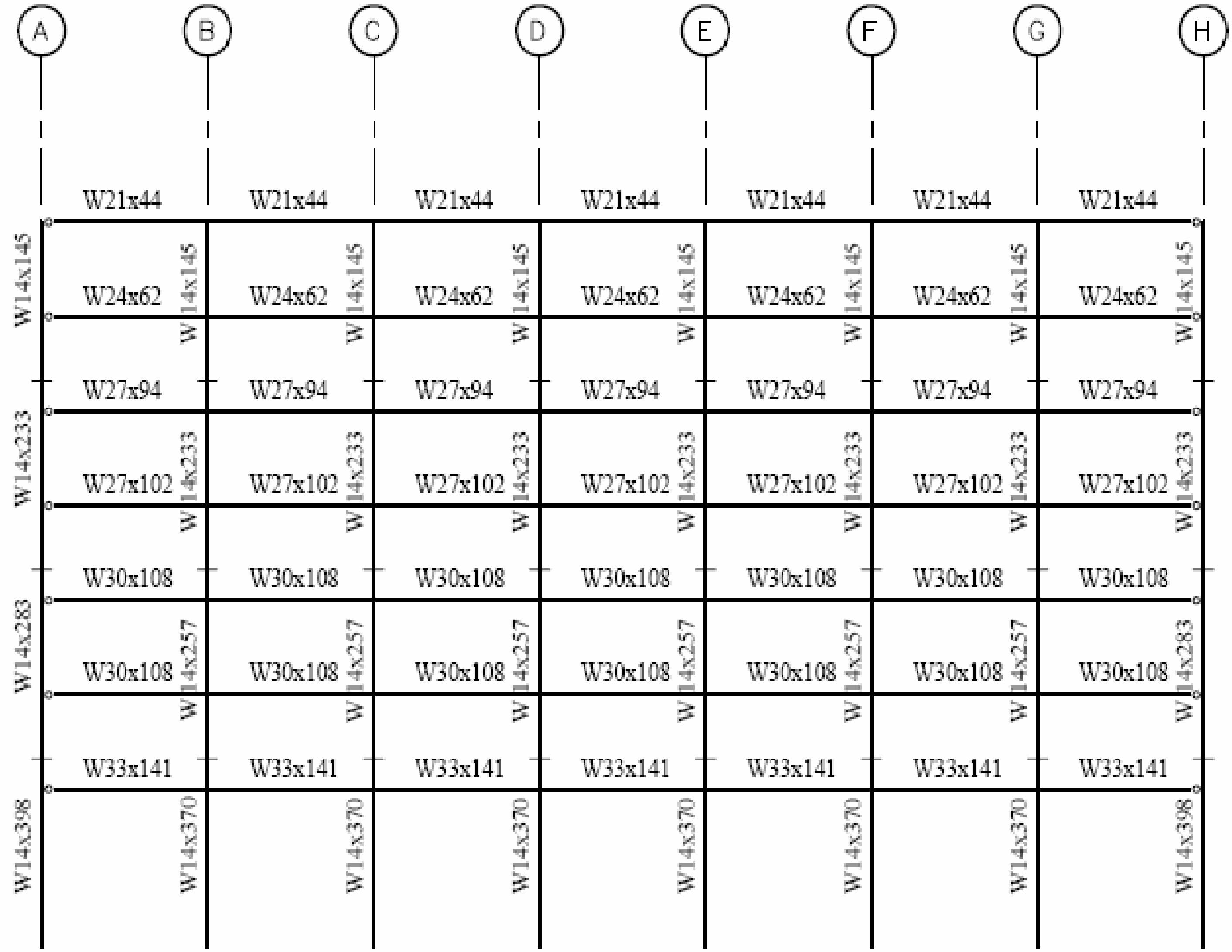
The following design steps will be reviewed:

- Select preliminary member sizes
- Check member local stability
- Check deflection and drift
- Check torsional amplification
- Check the column-beam moment ratio rule
- Check shear requirement at panel zone
- Select connection configuration

Special Moment Frames

Select preliminary member sizes – The preliminary member sizes are given in the next slide for the frame in the East-West direction. These members were selected based on the use of a 3D stiffness model in the program RAMFRAME. As will be discussed in a subsequent slide, the drift requirements controlled the design of these members.

SMF Example – Preliminary Member Sizes



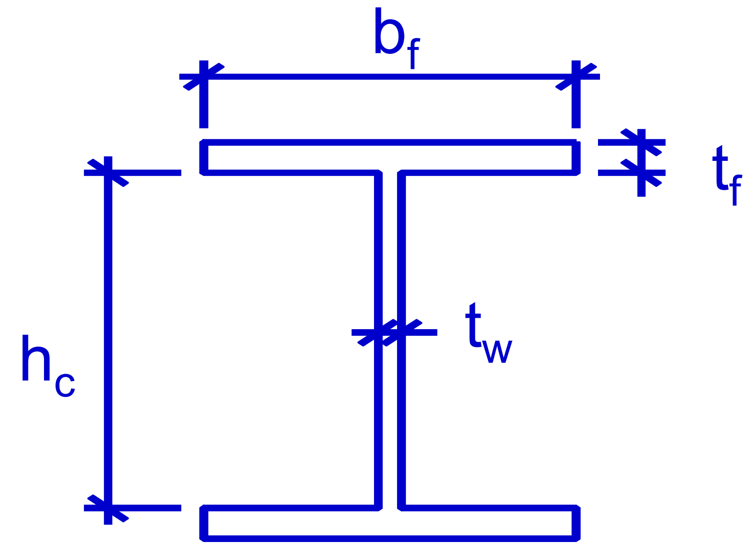
SMF Example – Check Member Local Stability

Check beam flange: $\dfrac{b_f}{2t_f} = 6.01$
(W33x141 A992)

Upper limit: $0.3\sqrt{\dfrac{E}{F_y}} = 7.22$ OK

Check beam web: $\dfrac{h_c}{t_w} = 49.6$

Upper limit: $3.76\sqrt{\dfrac{E}{F_y}} = 90.6$ OK



SMF Example – Check Deflection and Drift

The frame was checked for an allowable story drift limit of $0.020h_{sx}$. All stories in the building met the limit. Note that the *NEHRP Recommended Provisions* Sec. 4.3.2.3 requires the following check for vertical irregularity:

$$\frac{C_d \Delta_{x \ \ story\ 2}}{C_d \Delta_{x \ \ story\ 3}} = \frac{\left(\dfrac{5.17in.}{268in.}\right)}{\left(\dfrac{3.14in.}{160in.}\right)} = 0.98 < 1.3$$

Therefore, there is no vertical irregularity.

SMF Example – Check Torsional Amplification

The torsional amplification factor is given below. If Ax < 1.0 then torsional amplification is not required. From the expression it is apparent that if $\delta_{max} / \delta_{avg}$ is less than 1.2, then torsional amplification will not be required.

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

The 3D analysis results, as shown in FEMA 451, indicate that none of the $\delta_{max} / \delta_{avg}$ ratios exceed 1.2; therefore, there is no torsional amplification.

SMF Example – Member Design NEHRP Guide

Member Design Considerations - Because $P_u/\varphi P_n$ is typically less than 0.4 for the columns, combinations involving Ω_0 factors do not come into play for the special steel moment frames (re: AISC Seismic Sec. 8.3). In sizing columns (and beams) for strength one should satisfy the most severe value from interaction equations. However, the frame in this example is controlled by drift. So, with both strength and drift requirements satisfied, we will check the column-beam moment ratio and the panel zone shear.

SMF Example – Column-Beam Moment Ratio

Per AISC Seismic Sec. 9.6

$$\frac{\Sigma M_{pc}^{*}}{\Sigma M_{pb}^{*}} > 1.0$$

where ΣM^*_{pc} = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. ΣM^*_{pc} is determined by summing the projections of the nominal flexural strengths of the columns above and below the joint to the beam centerline with a reduction for the axial force in the column.

ΣM^*_{pb} = the sum of the moments in the beams at the intersection of the beam and column centerlines.

SMF Example – Column-Beam Moment Ratio

Column – W14x370; beam – W33x141

$$\Sigma M_{pc}^{*} = \Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right) = 2\left[736in^2 \left(50ksi - \frac{500kips}{109in^2} \right) \right]$$

$$\Sigma M_{pc}^{*} = 66{,}850in - kips$$

Adjust this by the ratio of average story height to average clear height between beams.

$$\Sigma M_{pc}^{*} = 66{,}850in - kips \left(\frac{268in. + 160in.}{251.35in. + 128.44in.} \right) = 75{,}300in - kips$$

SMF Example – Column-Beam Moment Ratio

For beams:

$$\Sigma M^*_{pb} = \Sigma(1.1R_y M_p + M_v)$$

$$\text{with } M_v = V_p S_h$$

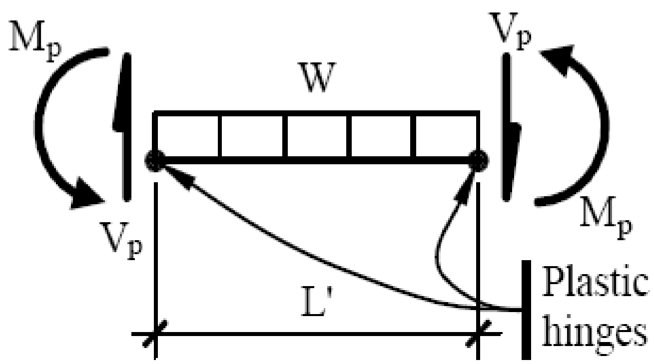
$$S_h = \text{dist. from col. centerline to plastic hinge}$$

$$= d_c/2 + d_b/2 = 25.61 in.$$

$$V_p = \text{shear at plastic hinge location}$$

$$V_p = \left[2M_p + \left(wL'^2/2\right)\right] = \frac{2M_p + \dfrac{wL'^2}{2}}{L'}$$

$$= \frac{(2)(25{,}700 in - kips) + \left(\dfrac{(1.046 klf)}{12}\dfrac{(248.8 in.)^2}{2}\right)}{248.8 in.} = 221.2 kips$$



SMF Example – Column-Beam Moment Ratio

$$M_v = V_p S_h = (221.2 kips)(25.61 in.) = 5{,}665 in - kips$$

and

$$\Sigma M^*_{pb} = \Sigma(1.1 R_y M_p + M_v)$$

$$= 2[(1.1)(1.1)(25{,}700 in - kips) + 5{,}665 in - kips] = 73{,}500 in - kips$$

The ratio of column moment strengths to beam moment strengths is computed as:

$$Ratio = \frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = \frac{75{,}300 in - kips}{73{,}500 in - kips} = 1.02 > 1.00 \quad \therefore OK$$

Other ratios are also computed to be greater than 1.0

SMF Example –Panel Zone Check

The 2005 AISC Seismic specification is used to check the panel zone strength. Note that FEMA 350 contains a different methodology, but only the most recent AISC provisions will be used. From analysis shown in the NEHRP *Design Examples* volume (FEMA 451), the factored strength that the panel zone at Story 2 of the frame in the EW direction must resist is 1,883 kips.

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right] = (0.6)(50ksi)(17.92in.)(t_p)\left[1 + \frac{(3)(16.475in.)(2.66)^2}{(33.3in.)(17.92in.)(t_p)}\right]$$

$$R_v = 537.6t_p + 315$$

The required total (web plus doubler plate) thickness is determined *by*:

$$\phi R_v = R_u$$

$$(1.0)(537.6t_p + 315) = 1{,}883kips$$

$$t_{p_{required}} = 2.91in.$$

The column web thickness is 1.66*in*., *therefore the required doubler plate thickness is*:

$$t_{p_{doubler}} = 1.25in. \quad (\text{therefore use one } 1.25in. \text{ plate or two } 0.625in. \text{ plates})$$

SMF Example – Connection Configuration

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

(3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h \qquad (9\text{-}1)$$

SMF Example – Connection Configuration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 9.2a by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Appendix P.

(c) Provision of qualifying cyclic test results in accordance with Appendix S. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix S.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S.

Special Moment Frames
Summary

Beam to column connection capacity

Select preliminary member sizes

Check member local stability

Check deflection and drift

Check torsional amplification

Check the column-beam moment ratio rule

Check shear requirement at panel zone

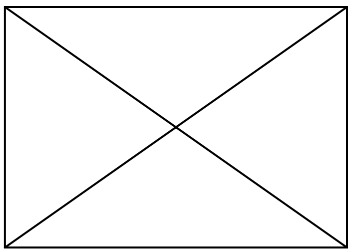
Select connection configuration

- Prequalified connections
- Testing

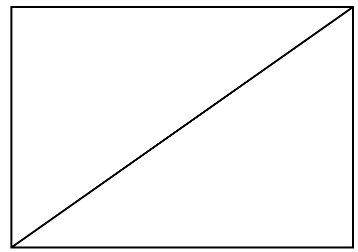
NEHRP Recommended Provisions
Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Seismic design category requirement
- Moment resisting frames
- Braced frames

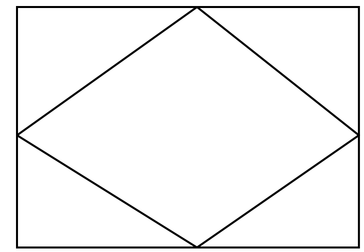
Concentrically Braced Frames
Basic Configurations



X



Diagonal



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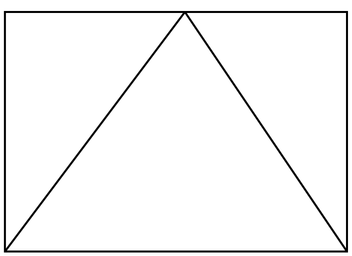
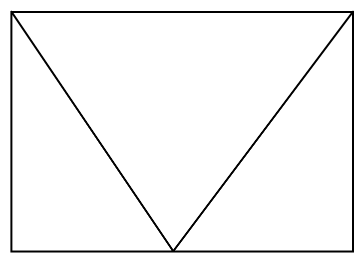
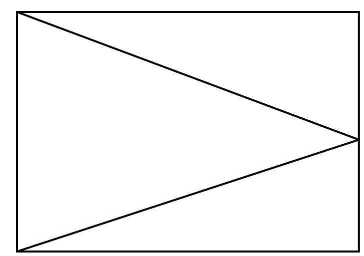


Inverted V



V



K

Braced Frame Under Construction

Braced Frame Under Construction

Concentrically Braced Frames

Special AISC Seismic R = 6

Chapter 13

Ordinary AISC Seismic R = 3.25

Chapter 14

Not Detailed for Seismic R = 3

AISC LRFD

Concentrically Braced Frames

Dissipate energy after onset of global buckling by avoiding brittle failures:

- Minimize local buckling
- Strong and tough end connections
- Better coupling of built-up members

Concentrically Braced Frames
Special and Ordinary

Bracing members:

- Compression capacity = $\phi_c P_n$
- Width / thickness limits

  Generally compact

  Angles, tubes and pipes very compact

- Overall $\dfrac{KL}{r} < 4\sqrt{\dfrac{E}{F_y}}$
- Balanced tension and compression

Concentrically Braced Frames

Special concentrically braced frames

Brace connections

Axial tensile strength > smallest of:

- Axial tension strength = $R_y F_y A_g$
- Maximum load effect that can be transmitted to brace by system.

Axial compressive strength $\geq 1.1 R_y P_n$ where P_n is the nominal compressive strength of the brace.

Flexural strength > $1.1 R_y M_p$ or rotate to permit brace buckling while resisting $A_g F_{CR}$

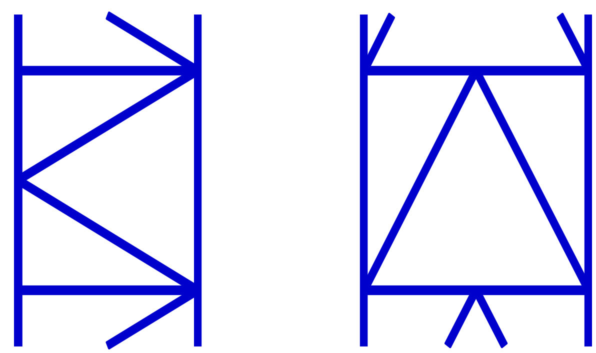
Concentrically Braced Frames

<u>V bracing:</u>

- Design beam for D + L + unbalanced brace forces, using $0.3\phi P_c$ for compression and $R_y F_y A_g$ in tension
- Laterally brace the beam
- Beams between columns shall be continuous.

<u>K bracing:</u>

- Not permitted



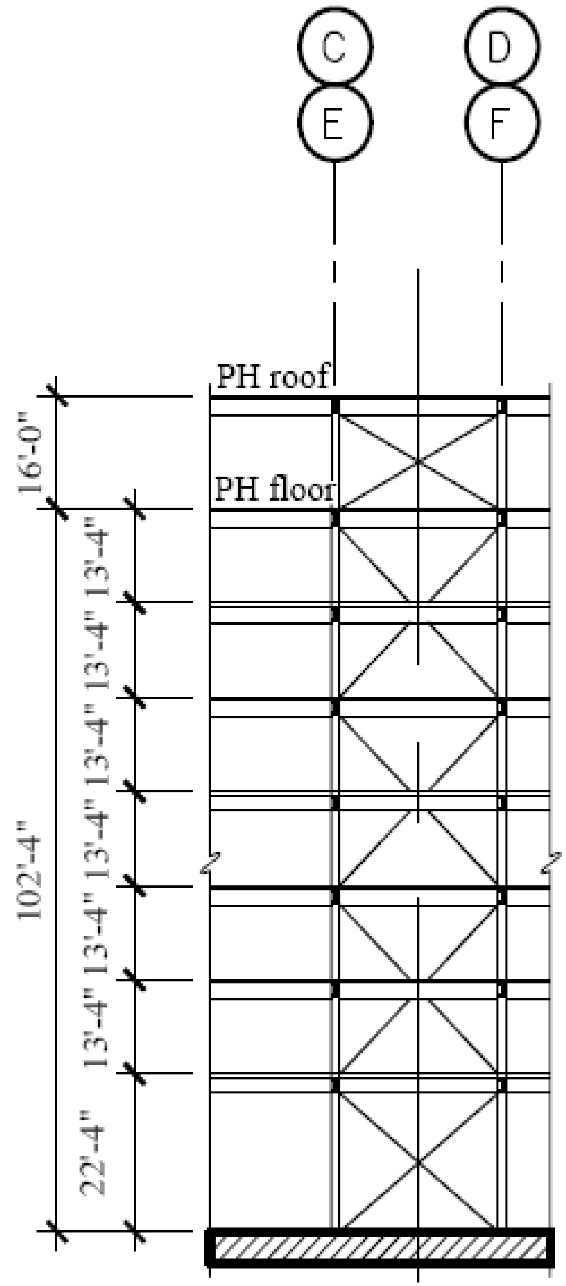
Concentrically Braced Frames

Built-up member stitches:

- Spacing < 40% KL/r
- No bolts in middle quarter of span
- Minimum strengths related to P_y

Column in CBF:

- Same local buckling rules as brace members
- Splices resist moments



Concentrically Braced Frame Example

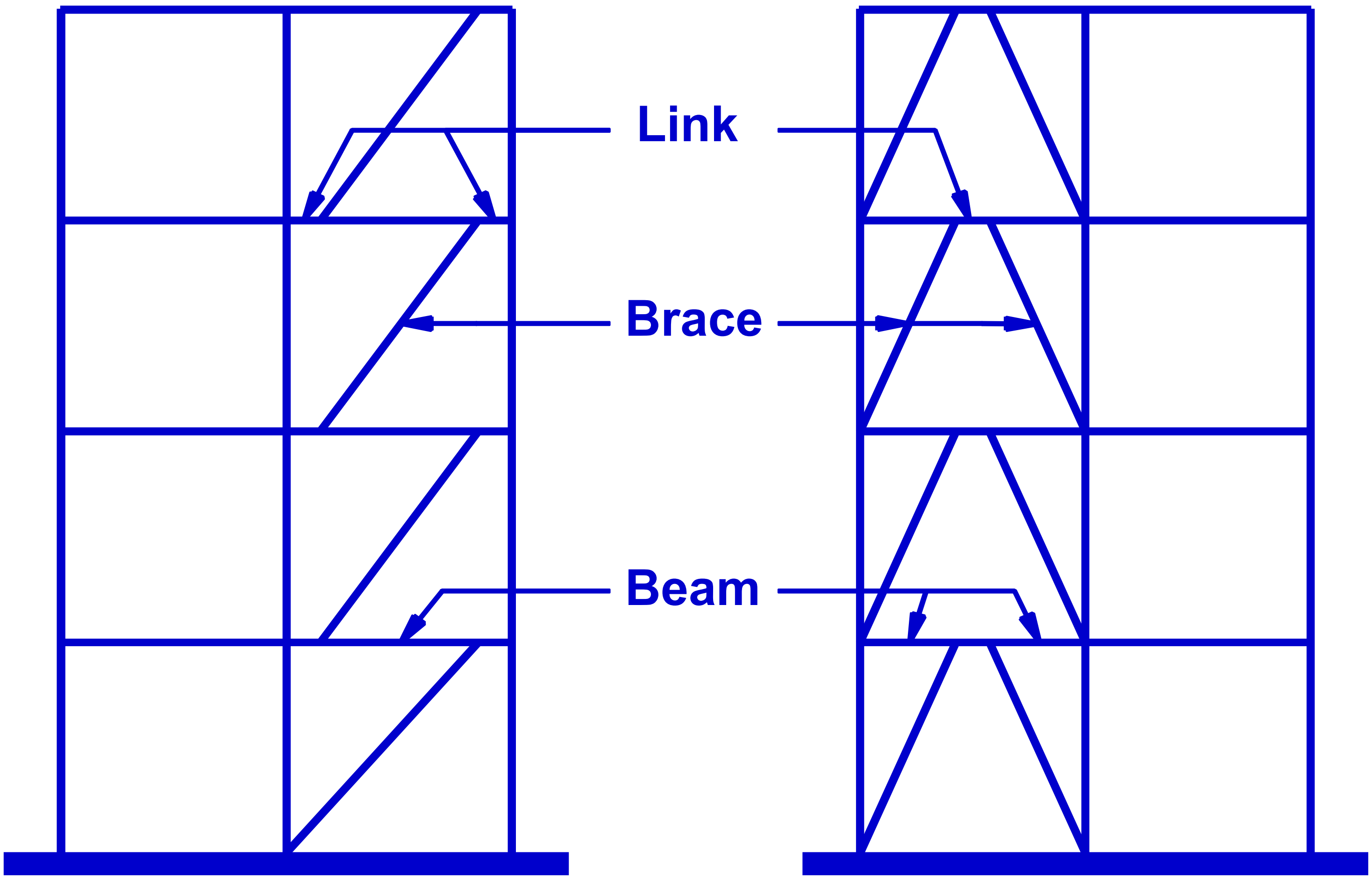
E-W direction

Concentrically Braced Frame Example

The following general design steps are required:

- Selection of preliminary member sizes
- Check strength
- Check drift
- Check torsional amplification
- Connection design

Eccentrically Braced Frames

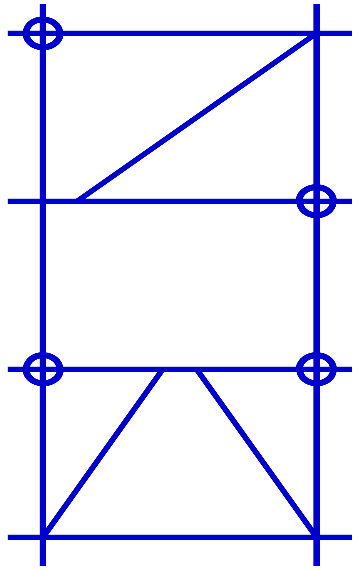


Eccentrically Braced Frame Under Construction

Eccentrically Braced Frame Under Construction

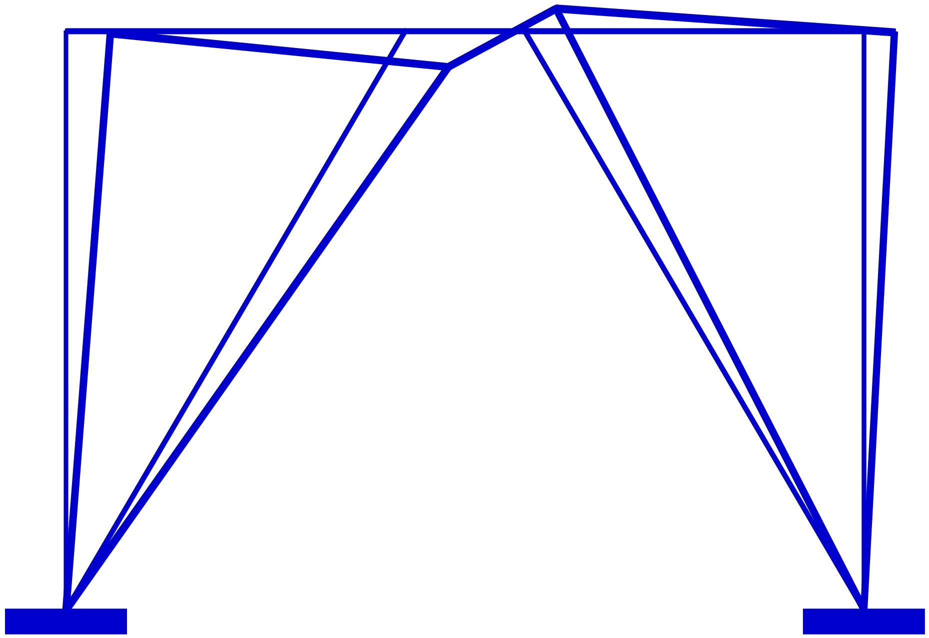
Eccentrically Braced Frames

Eccentric bracing systems	R	C_d
Building frame system or part of dual system w/ special moment frame		
With moment resisting connections at columns away from links	8	4
Without moment resisting connections at columns away from links	7	4



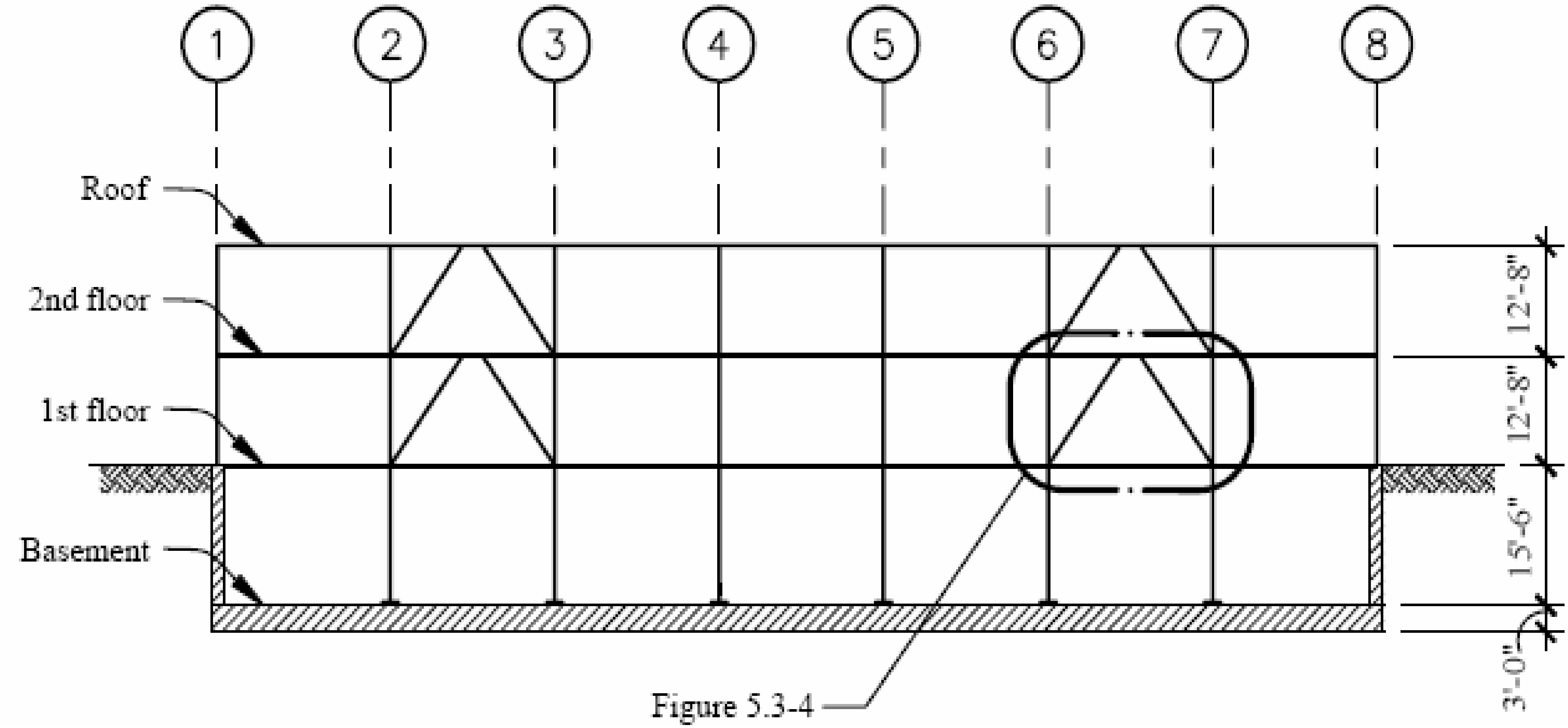
These connections determine classification

Eccentrically Braced Frames
Design Procedure



1. Elastic analysis
2. Check rotation angle; reproportion as required
3. Design check for strength
4. Design connection details

Eccentrically Braced Frames
Example



Eccentrically Braced Frames
Rotation Angle

1. Compute total $\Delta = C_d \Delta_E$

2. Deform model as rigid-plastic mechanism with hinges at ends of line

3. Compute rotation angle at end of link

4. Check limits (Sec. 15.2g)

$$\alpha \le 0.08 \text{ radians when } L \le \frac{1.6M_p}{V_p}$$

$$\alpha \le 0.02 \text{ radians when } L \ge \frac{2.6M_p}{V_p}$$

$$\text{Interpolate for } \alpha \text{ when } \frac{1.6M_p}{V_p} < L < \frac{2.6M_p}{V_p}$$

Eccentrically Braced Frames
Rotation Angle Example

From computer analysis:

$$\Delta_e = 0.247\ in$$

Total drift:

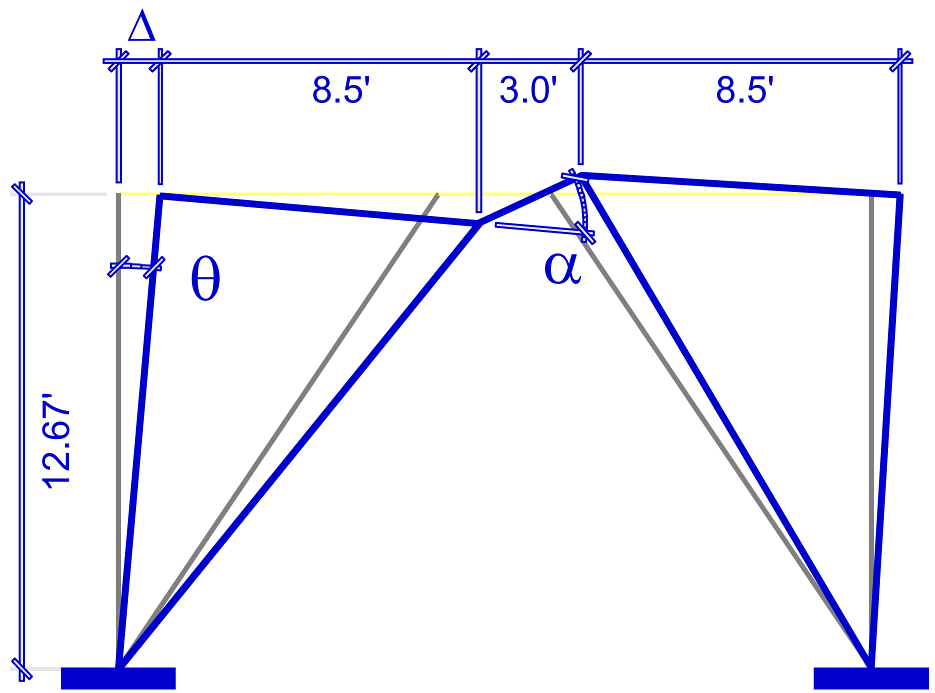
$$\Delta = C_d \Delta_e = 4(0.247) = 0.99\ in.$$

From geometry:

$$\alpha = \left(\frac{L}{e}\right)\theta = \left(\frac{20}{3}\right)\left(\frac{0.99}{12.67(12)}\right) = 0.043 \quad rad$$

$$Because \quad e = 3.0' < \frac{1.6M_p}{F_y} = 3.52'$$

$$\alpha_{\max} = 0.08\ rad > 0.043\ rad \quad OK$$



Eccentrically Braced Frames
Rotation Angle

- Rotation angle limits based on link beam equivalent length
  - Short links yield in shear and are allowed greater rotation
- Rotation angle may be reduced in design by:
  - Increasing member size (reducing Δ_e)
  - Changing geometric configuration (especially changing length of link beam)

Eccentrically Braced Frames
Link Design

- Provide strength V and M per load combinations
- Check lateral bracing per AISC L_{pd}
- Local buckling (width to thickness of web and flange) per AISC Seismic
- Stiffeners (end and intermediate) per AISC Seismic

Eccentrically Braced Frames
Brace Design

$$\text{Strength} > 1.25R_y \cdot \begin{pmatrix} \text{axial force from design} \\ \text{shear strength of link} \end{pmatrix}$$

Eccentrically Braced Frames

Brace Design Example

Check axial strength of 15.26 ft long TS 8 x 8 x 5/8 F_y = 46 ksi:

$$\frac{KL}{r} = \frac{(1)(15.26)(12)}{2.99} = 61.2$$

$$61.2 < 4.71\sqrt{\frac{E}{F_y}} = 118.3 \qquad \therefore F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000)}{61.2^2} = 76.4\ ksi$$

$$F_{cr} = \left(0.658^{\frac{46}{76.4}}\right)46 = 35.8ksi$$

$$\phi_c P_n = \phi_c A_g F_{cr} = 0.9(16.4)(35.8) = 528\ kip$$

Eccentrically Braced Frames
Brace Design Example

$$\phi V_n = 0.9(0.6F_y)d\,t_w = 0.9\left[0.6(50)(16.4)(0.43)\right] = 190\ kip$$

or

$$\phi V_n = 2(0.9)M_p / e = \frac{2(0.9)(50)(105)}{3(12)} = 262.5\ kip$$

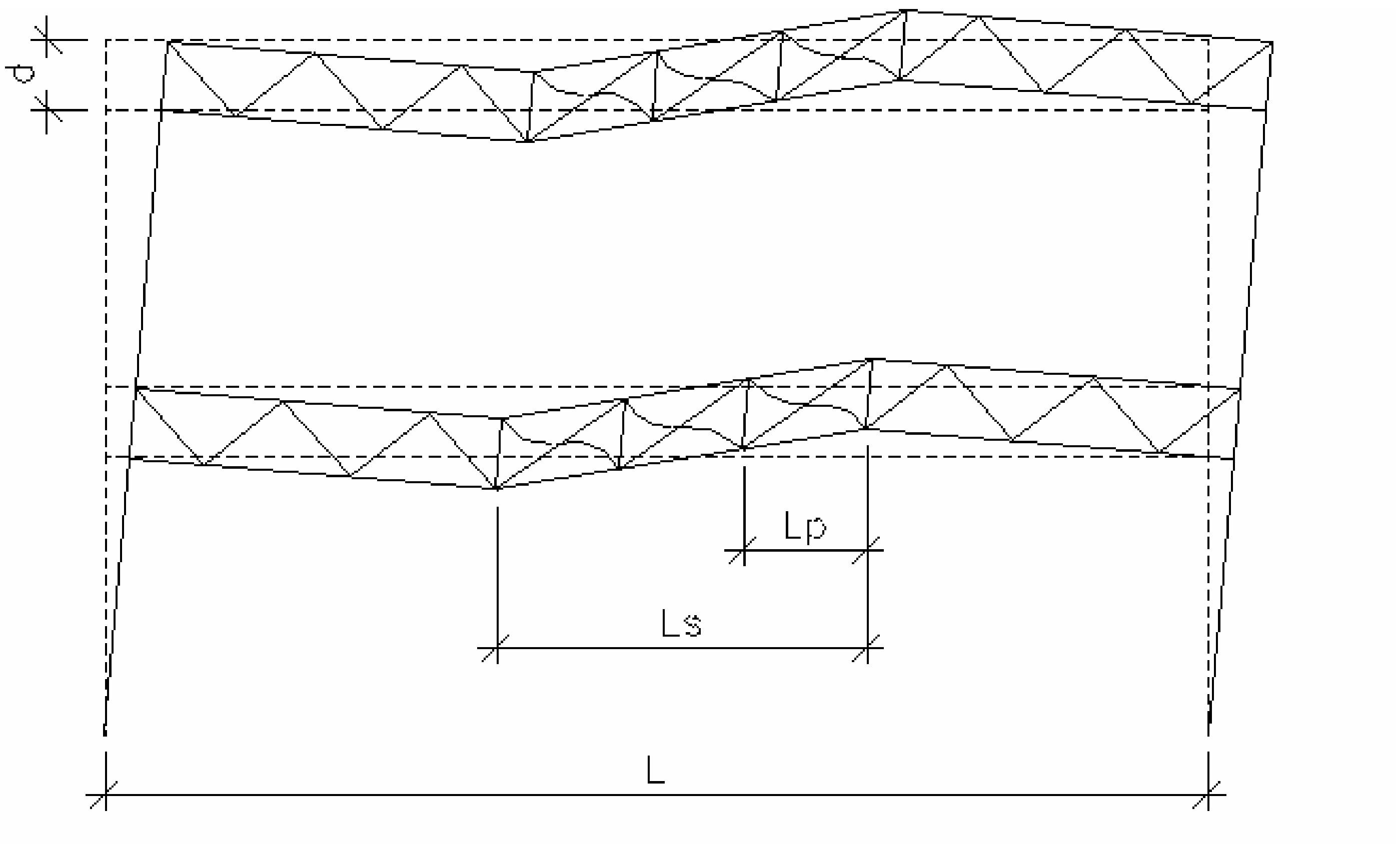
$$V_{e(link)} = 85.2\ kip \quad and \quad P_{e(brace)} = 120.2\ kip$$

$$\therefore P_u = 1.25(1.1)\left(\frac{190}{85.2}\right)(120.2) = 369 < 528 \quad OK$$

NEHRP Recommended Provisions
Steel Design

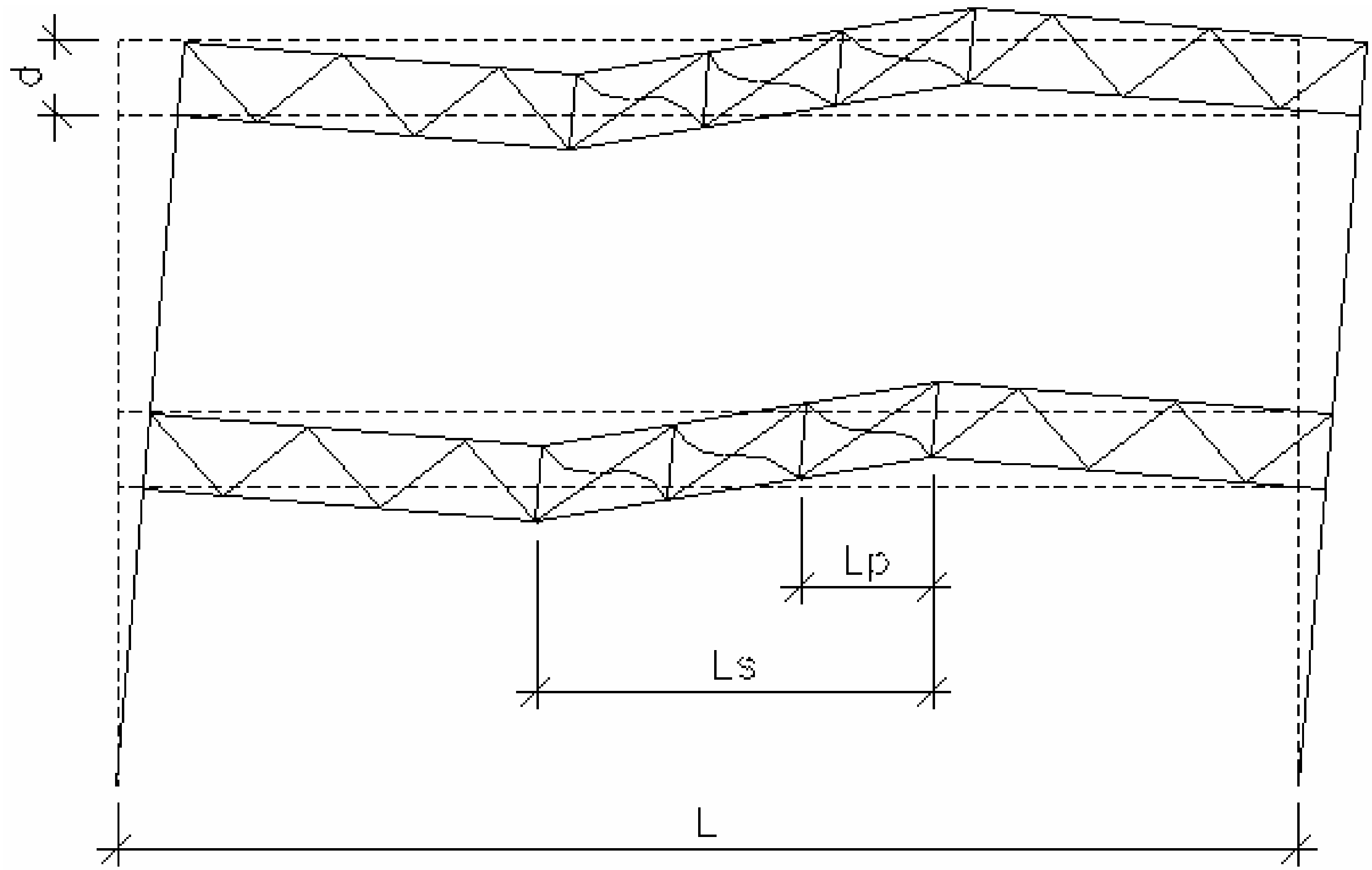
- Context in *NEHRP Recommended Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics

Special Truss Moment Frame



- **Buckling and yielding in <u>special</u> section**
- **Design to be elastic outside special section**
- **Deforms similar to EBF**
- **Special panels to be symmetric X or Vierendeel**

Special Truss Moment Frame



Geometric Limits:

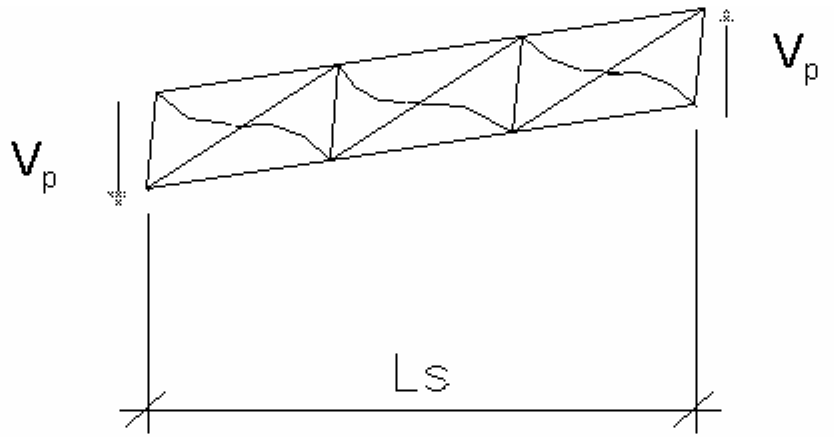
$L \leq 65'$ $d \leq 6'$

$$0.1 < \frac{L_s}{L} < 0.5$$

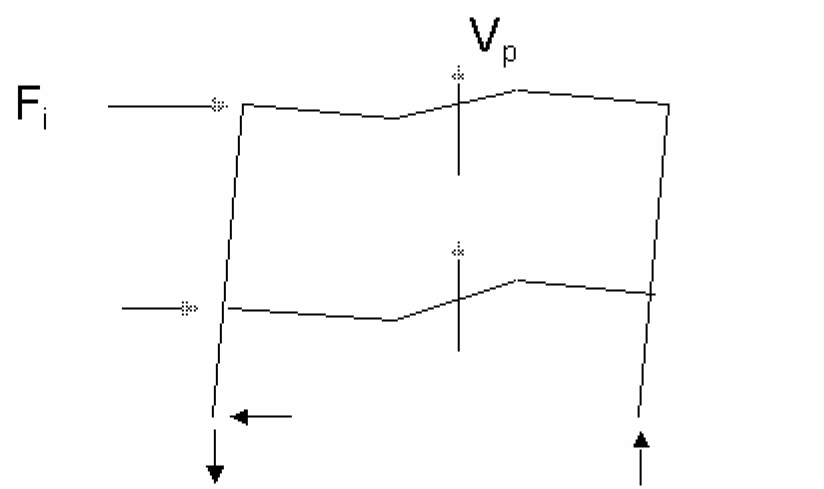
$$\frac{2}{3} < \frac{L_p}{d} < \frac{3}{2}$$

Flat bar diagonals, $\frac{b}{t} \leq 2.5$

Special Truss Moment Frame



$$V_p = 2\left(\frac{2\,M_{pc}}{L_s}\right) + \sin\alpha\,\left(P_{nt} + 0.3P_{cd}\right)$$



$$\sum F_i\,h_i = \sum V_p\,L$$

Special Truss Moment Frame

Special Truss Moment Frame

General Seismic Detailing

Materials:

- Limit to lower strengths and higher ductilities

Bolted Joints:

- Fully tensioned high strength bolts
- Limit on bearing

General Seismic Detailing

Welded Joints:

- AWS requirements for welding procedure specs
- Filler metal toughness
  - CVN > 20 ft-lb @ -20°F, or AISC Seismic App. X
- Warning on discontinuities, tack welds, run offs, gouges, etc.

Columns:

- Strength using Ω_o if $P_u / \phi P_n > 0.4$
- Splices: Requirements on partial pen welds and fillet welds

Steel Diaphragm
Example

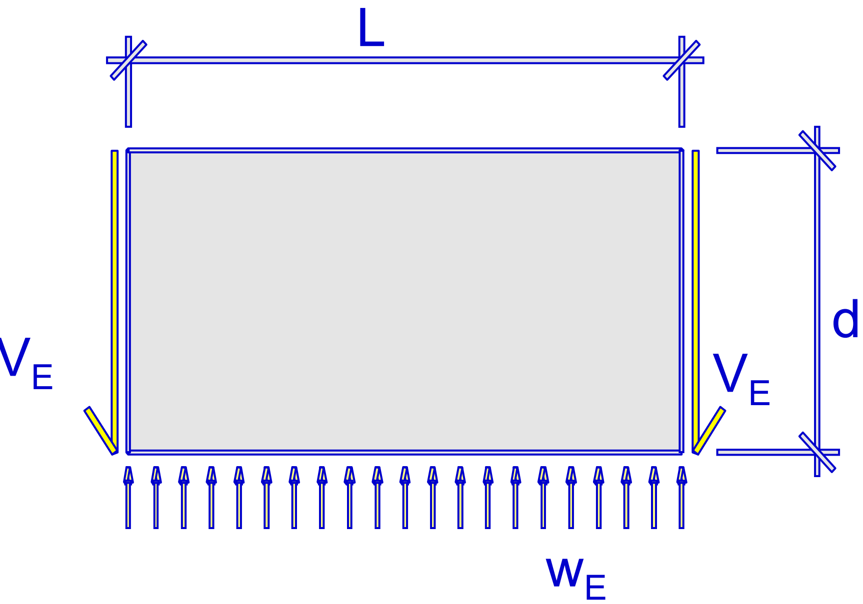
$\phi V_n = \phi$ (approved strength)

$\phi = 0.6$

For <u>example only:</u>

Use approved strength as 2.0 x working load in SDI *Diaphragm Design Manual*

Steel Deck Diaphragm Example



$$L = 80' \qquad d = 40'$$

$$w_D = w_L = 0 \qquad w_E = 500 \text{ plf}$$

$$V_E = \frac{w_E L}{2} = 20 \text{ kip}; \quad v_E = \frac{20000}{40} = 500 \text{ plf}$$

$$v_{SDI} = \frac{v_E}{2\phi} = \frac{500}{2(0.6)} = 417 \text{ plf}$$

Deck chosen:
1½ “, 22 gage with welds on 36/5 pattern and 3 sidelap fasteners, spanning 5’-0”

Capacity = 450 > 417 plf

Welded Shear Studs

Shear Stud Strength - AISC 2005 Specification

$$Q_n = 0.5\, A_{sc}\,(f_c'\, E_c)^{1/2} \leq R_g\, R_p\, A_{sc}\, F_u$$

R_g = stud geometry adjustment factor

R_p = stud position adjustment factor

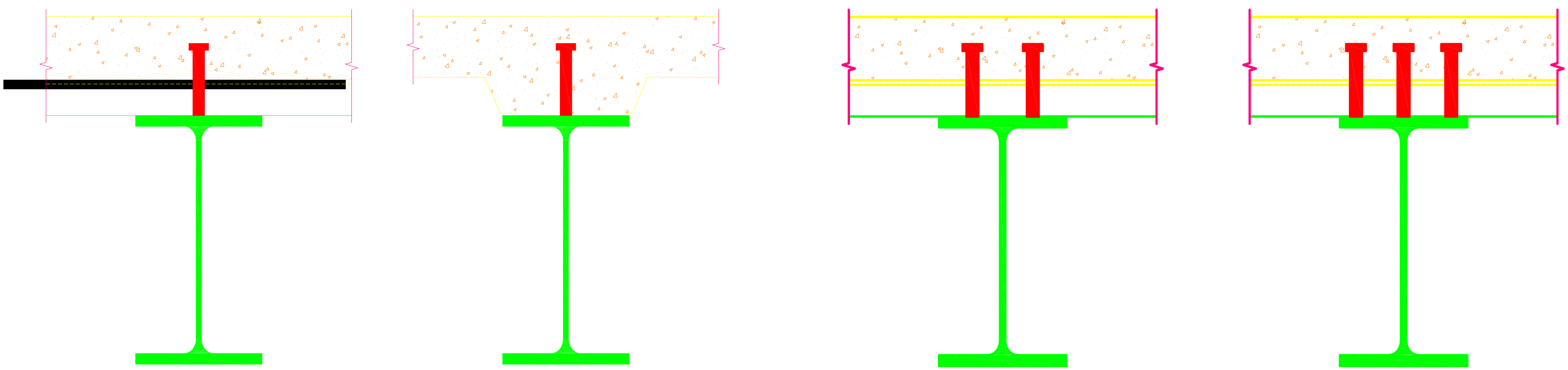
Note that the strength reduction factor for bending has been increased from 0.85 to 0.9. This results from the strength model for shear studs being more accurate, although the result for Q_n is lower in the 2005 specification.

Shear Studs – Group Adjustment Factor

$$Q_n = 0.5\, A_{sc}\, (\, f_c'\, E_c)^{1/2} \le R_g\, R_p\, A_{sc}\, F_u$$

R_g = stud group adjustment factor

$R_g = 1.0$ | $R_g = 1.0^*$ | $R_g = 0.85$ | $R_g = 0.7$



*0.85 if $w_r/h_r < 1.5$

Shear Studs – Position Adjustment Factor

$$Q_n = 0.5\, A_{sc}\, (f_c'\, E_c)^{1/2} \leq R_g\, R_p\, A_{sc}\, F_u$$

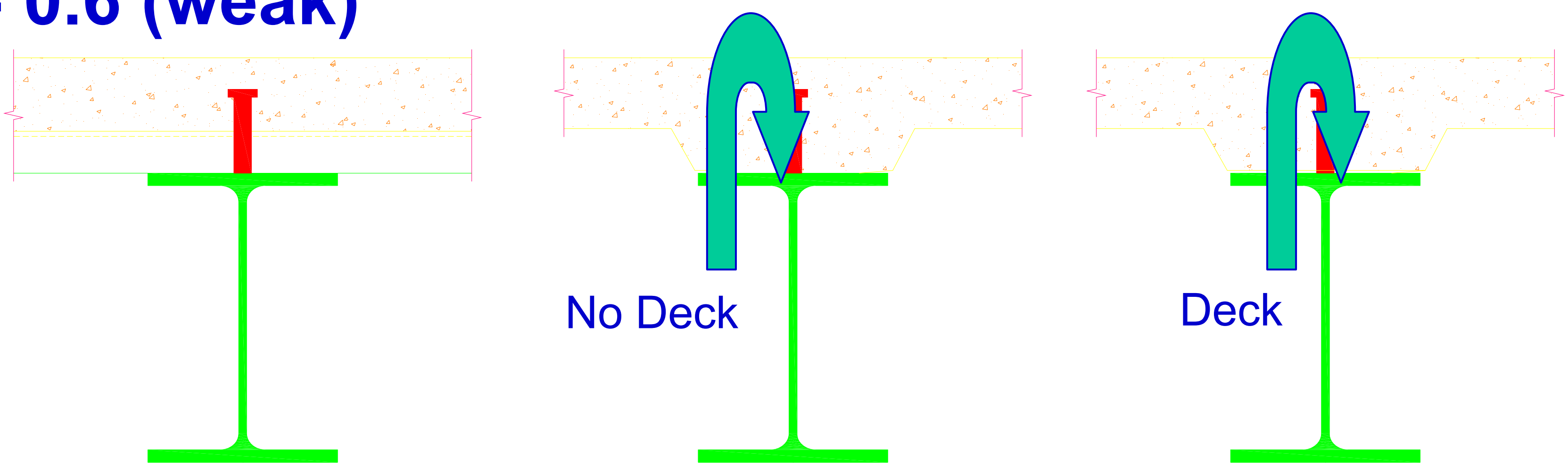
R_p = stud position adjustment factor

$R_p = 0.75$ (strong)

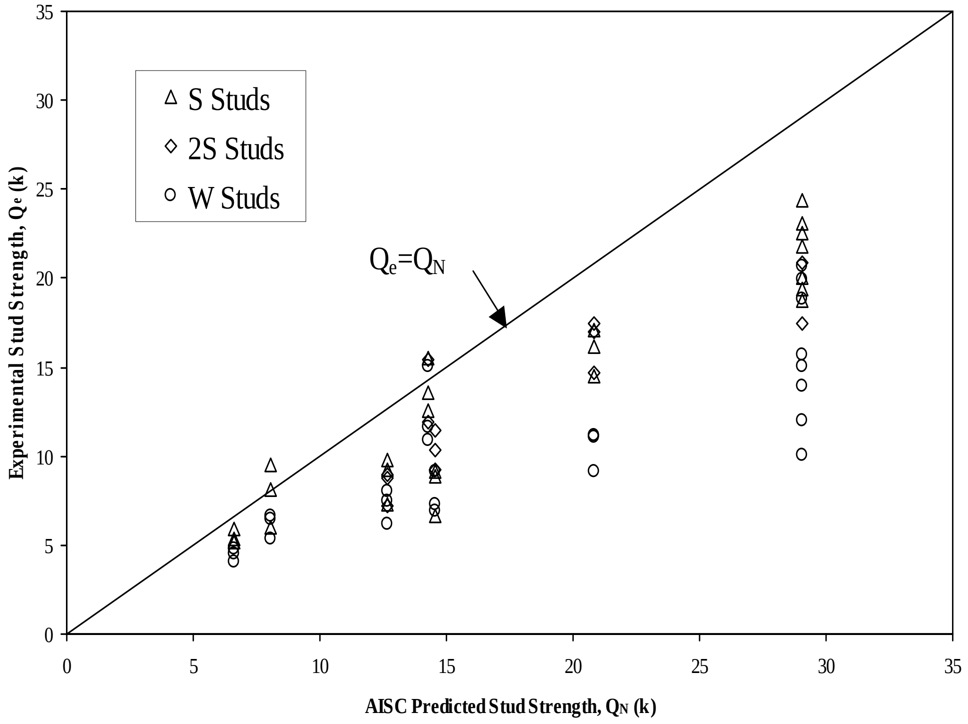
$= 0.6$ (weak)

$R_p = 1.0$

$R_p = 0.75$

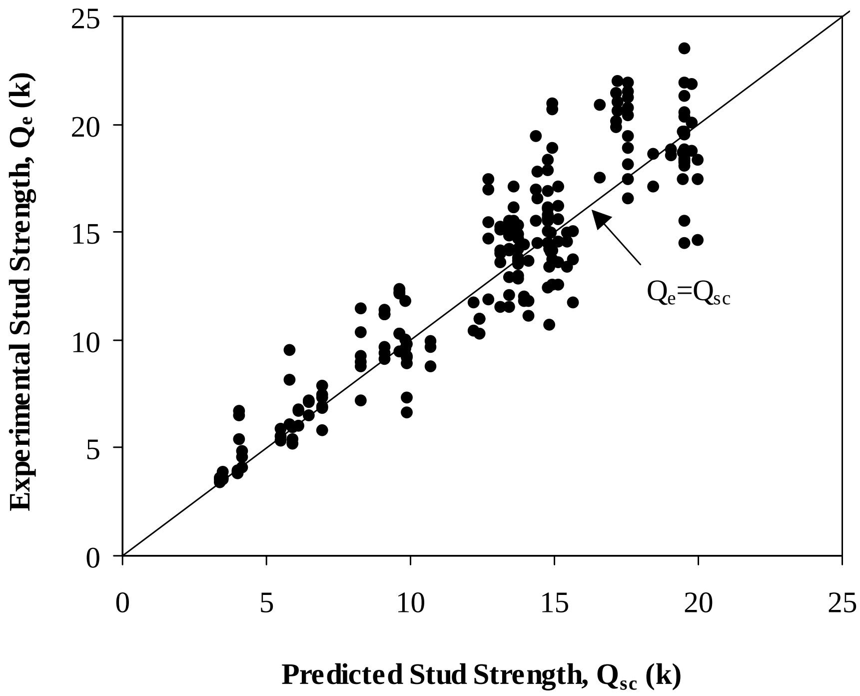


Shear Studs – Strength Calculation Model Comparison



AISC Seismic prior to 2005

Virginia Tech strength model



Shear Studs – Diaphragm Applications

Shear studs are often used along diaphragm collector members to transfer the shear from the slab into the frame. The shear stud calculation model in the 2005 AISC specification can be used to compute the nominal shear strengths. A strength reduction factor should be used when comparing these values to the factored shear. There is no code-established value for the strength reduction factor. A value of 0.8 is recommended pending further development.

Inspection and Testing
Inspection Requirements

- Welding:
  - Single pass fillet or resistance welds
    > PERIODIC
  - All other welds
    > CONTINUOUS

- High strength bolts:
    > PERIODIC

Inspection and Testing
Shop Certification

- Domestic:
  - AISC
  - Local jurisdictions

- Foreign:
  - No established international criteria

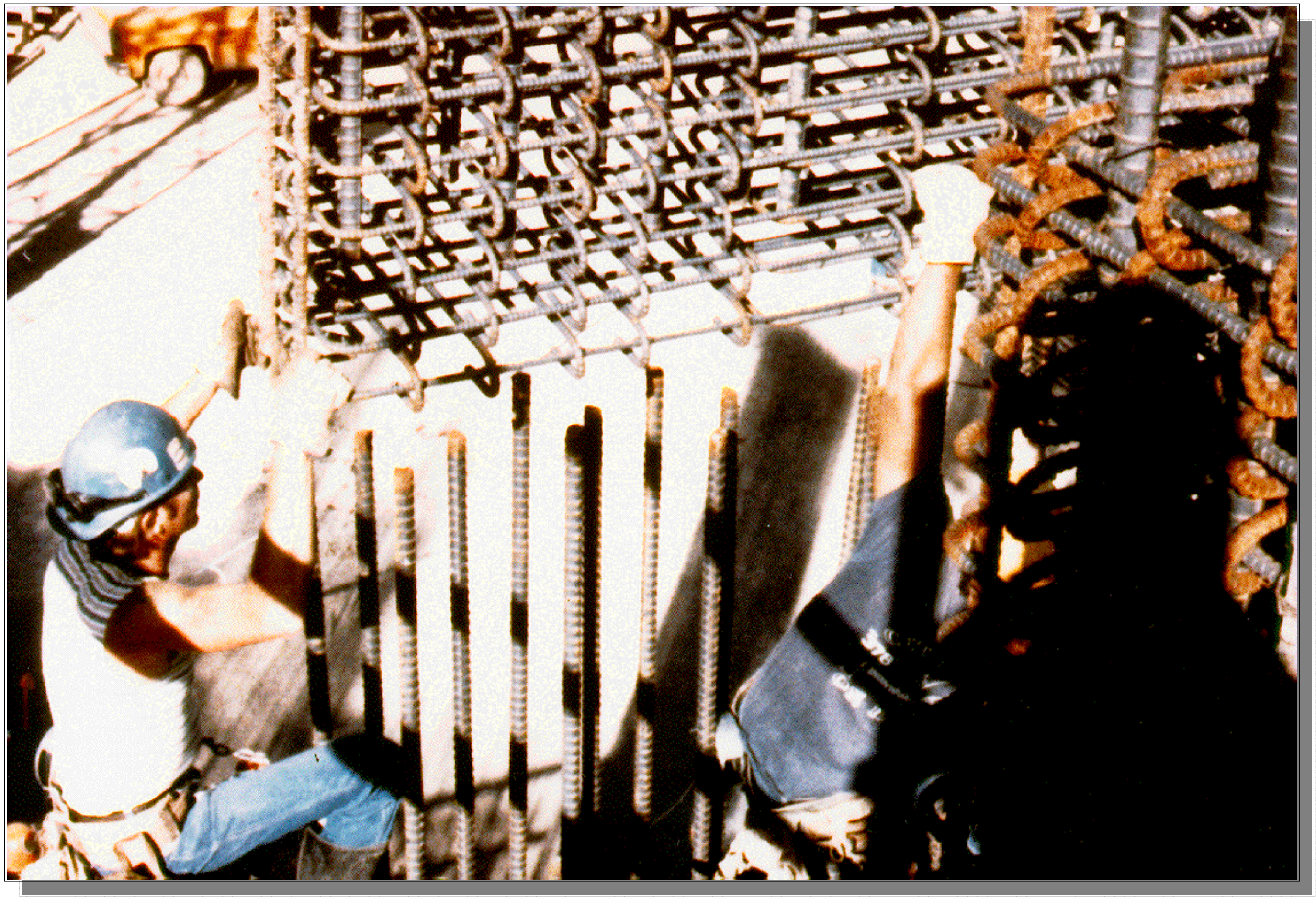
Inspection and Testing
Base Metal Testing

- More than 1-1/2 inches thick
- Subjected to through-thickness weld shrinkage
- Lamellar tearing
- Ultrasonic testing

NEHRP Recommended Provisions Steel Design

- Context in *Provisions*
- Steel behavior
- Reference standards and design strength
- Moment resisting frames
- Braced frames
- Other topics
- Summary

SEISMIC DESIGN OF
REINFORCED CONCRETE STRUCTURES



NEHRP Recommended Provisions Concrete Design Requirements

- Context in the *NEHRP Recommended Provisions*
- Concrete behavior
- Reference standards
- Requirements by Seismic Design Category
- Moment resisting frames
- Shear walls
- Other topics
- Summary

Context in *NEHRP Recommended Provisions*

Design basis: Strength limit state

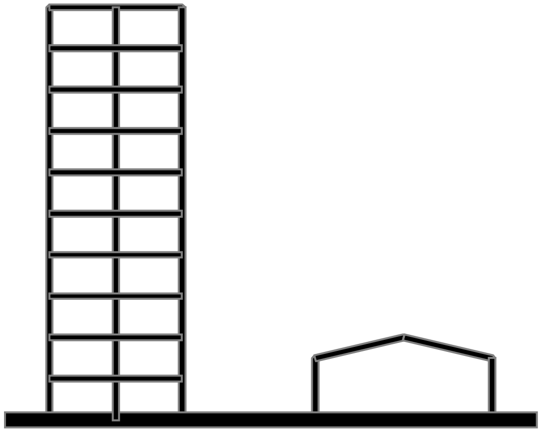
<u>Using *NEHRP Recommended Provisions*</u>:

Structural design criteria:	**Chap. 4**
Structural analysis procedures:	**Chap. 5**
Components and attachments:	**Chap. 6**
Design of concrete structures:	**Chap. 9 and ACI 318**

Seismic-Force-Resisting Systems
Reinforced Concrete

Unbraced frames (with rigid “moment resisting” joints):

- Three types
  - Ordinary
  - Intermediate
  - Special



R/C shear walls:

- Ordinary
- Special

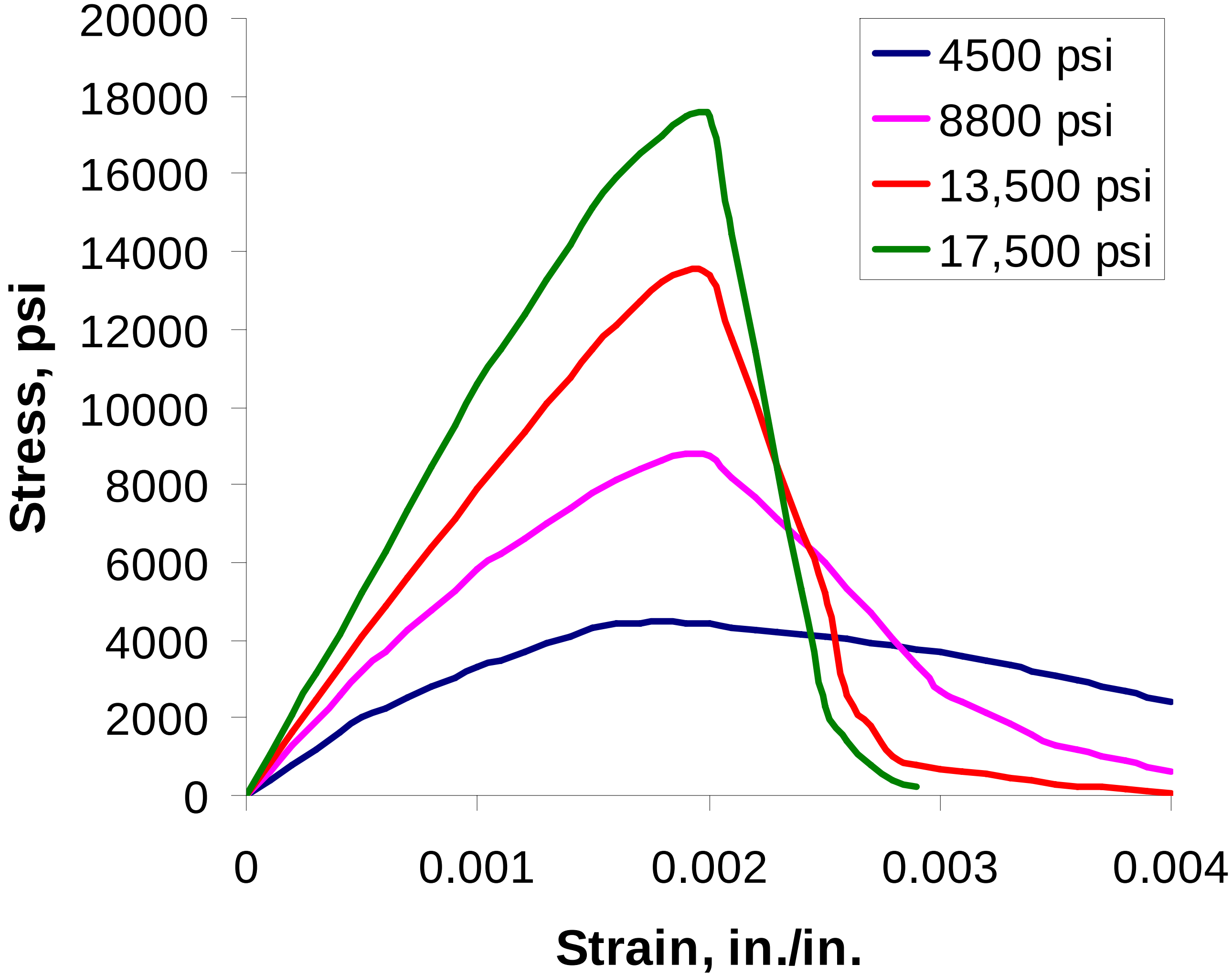
Precast shear walls:

- Special
- Intermediate
- Ordinary

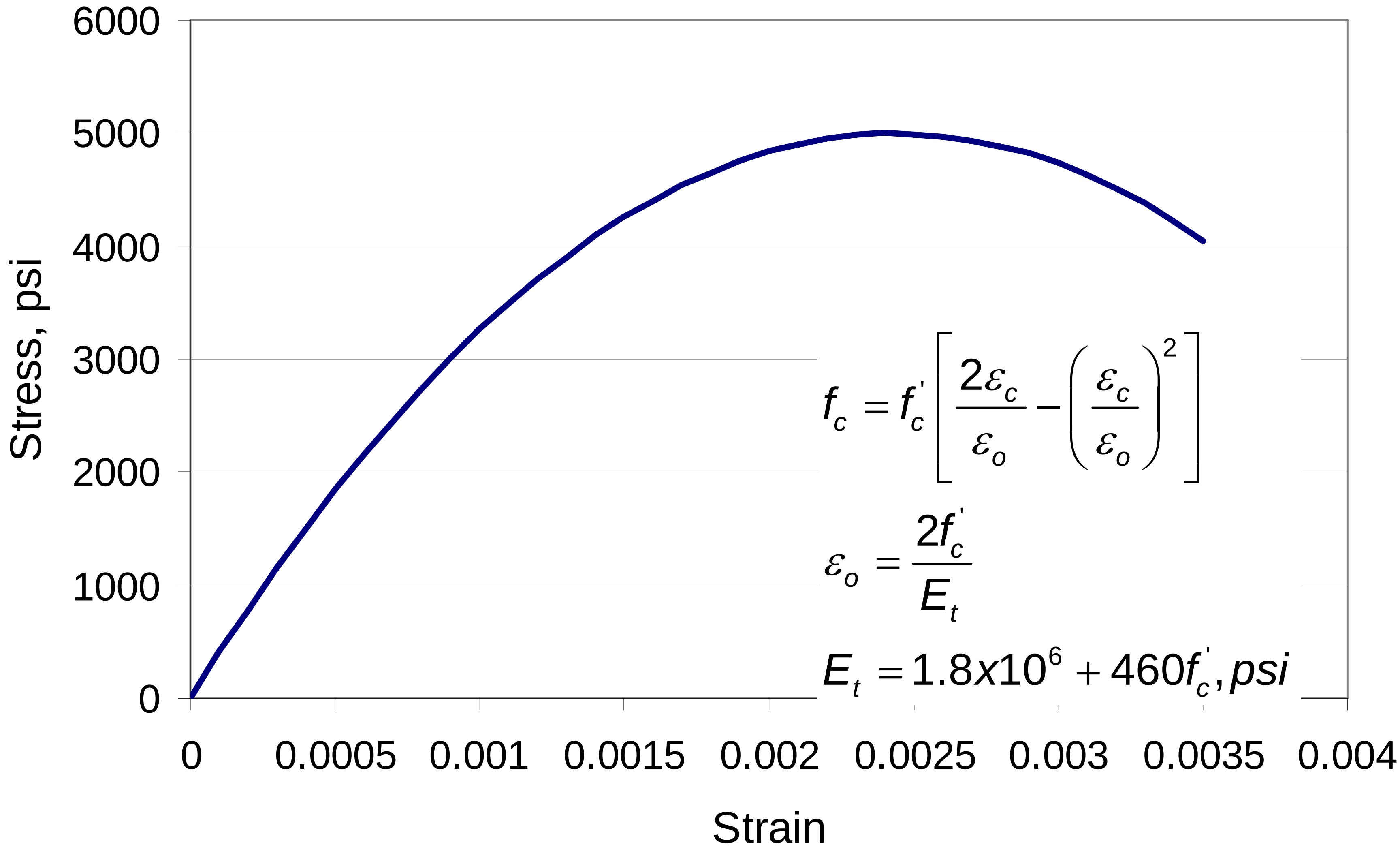
NEHRP Recommended Provisions Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**

Unconfined Concrete Stress-Strain Behavior



Idealized Stress-Strain Behavior of Unconfined Concrete

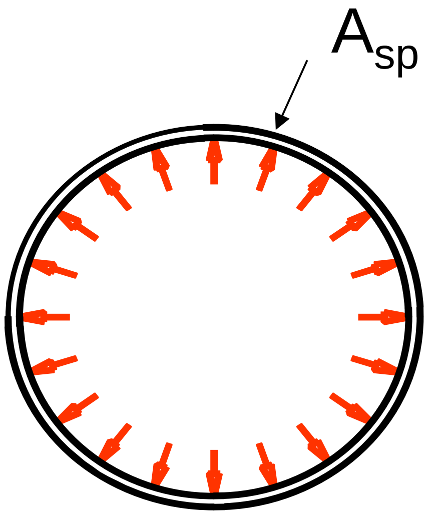


$$f_c = f_c^{'}\left[\frac{2\varepsilon_c}{\varepsilon_o} - \left(\frac{\varepsilon_c}{\varepsilon_o}\right)^2\right]$$

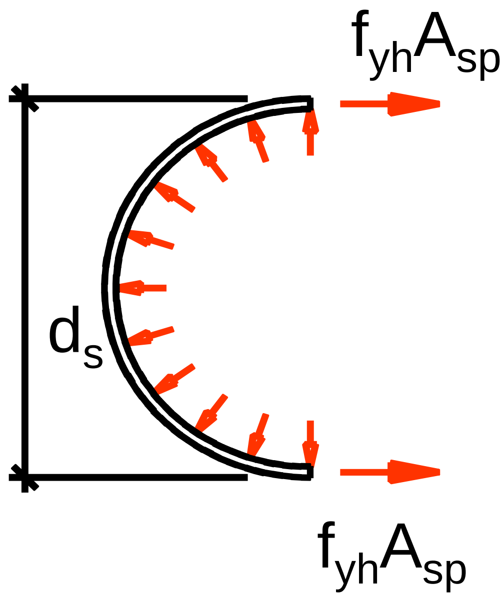
$$\varepsilon_o = \frac{2f_c^{'}}{E_t}$$

$$E_t = 1.8x10^6 + 460f_c^{'}, psi$$

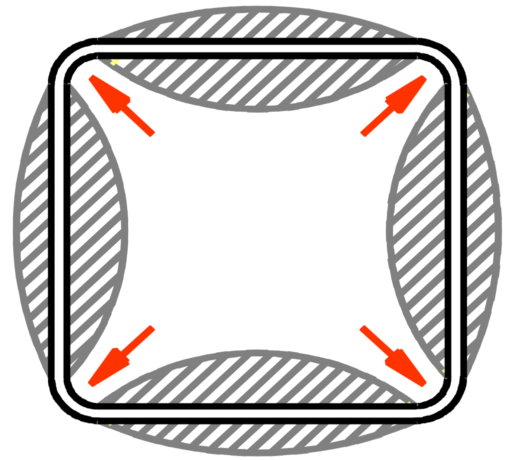
Confinement by Spirals or Hoops



Confinement from spiral or circular hoop

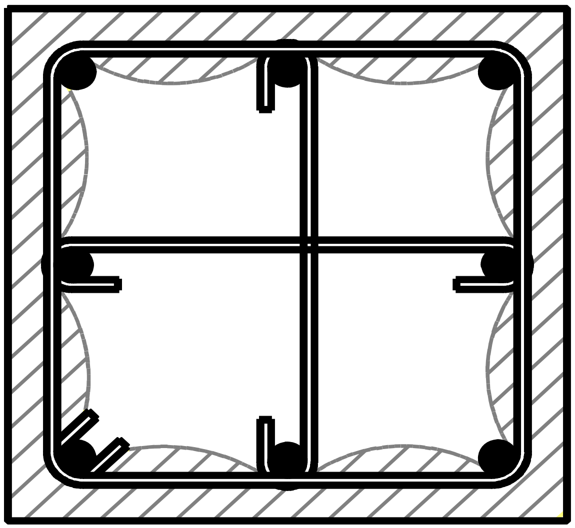


Forces acting on 1/2 spiral or circular hoop

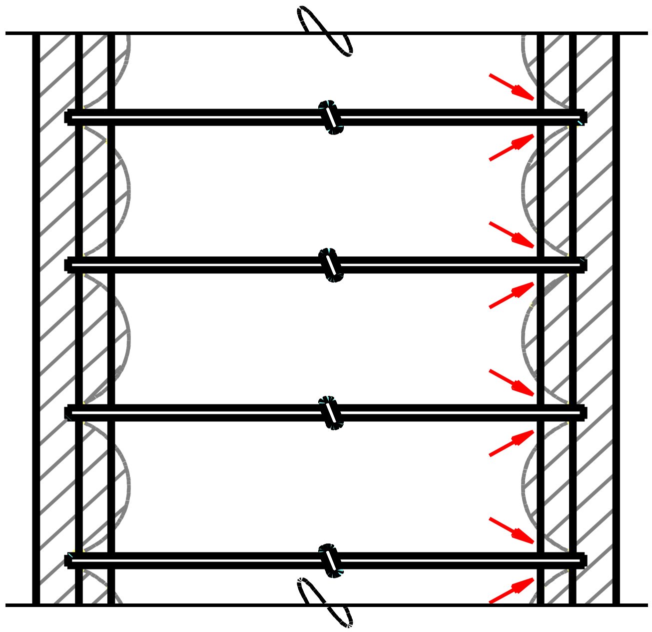


Confinement from square hoop

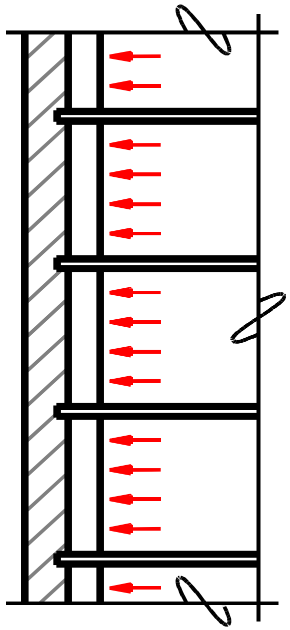
Confinement



Rectangular hoops with cross ties



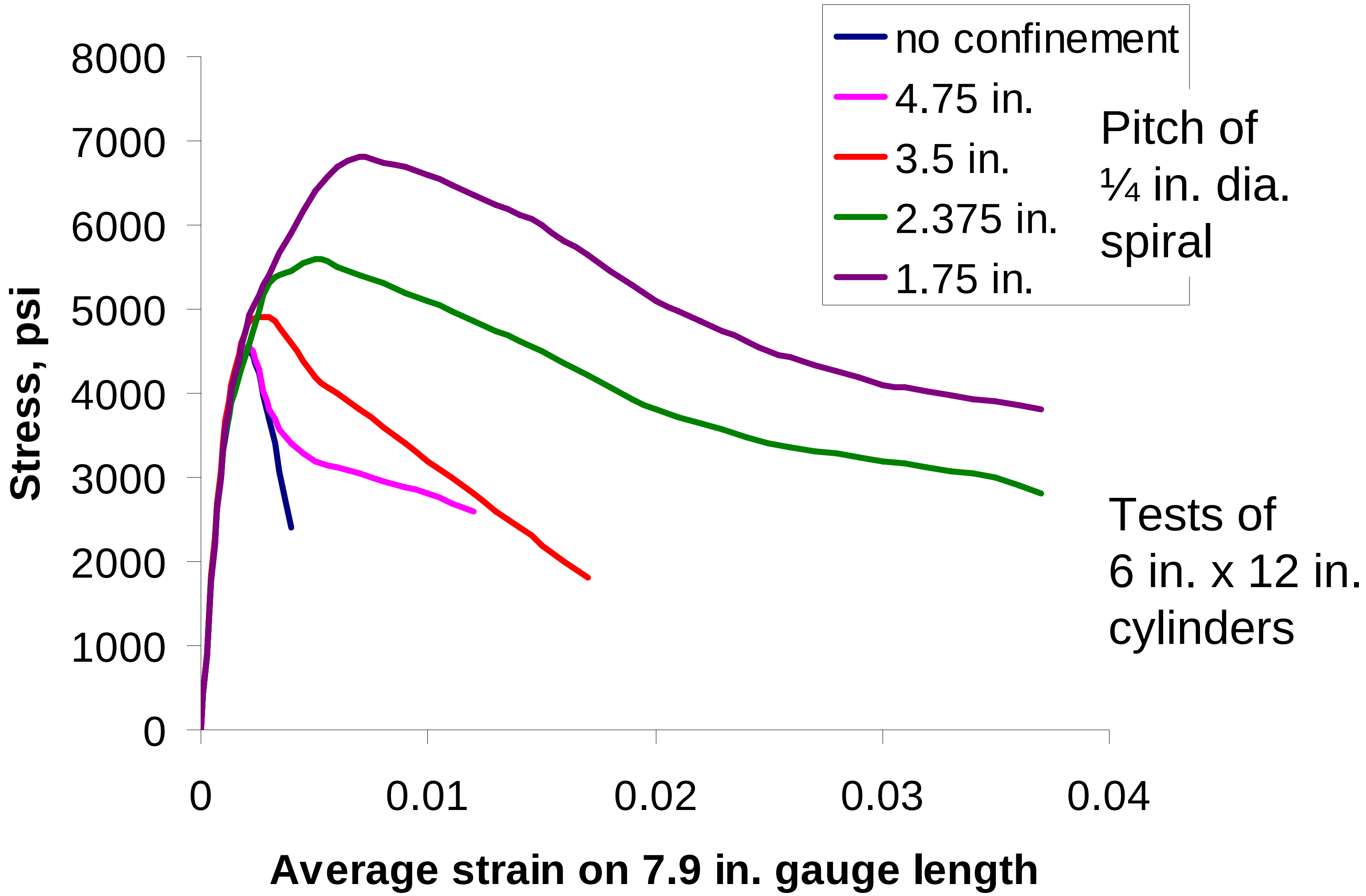
Confinement by transverse bars



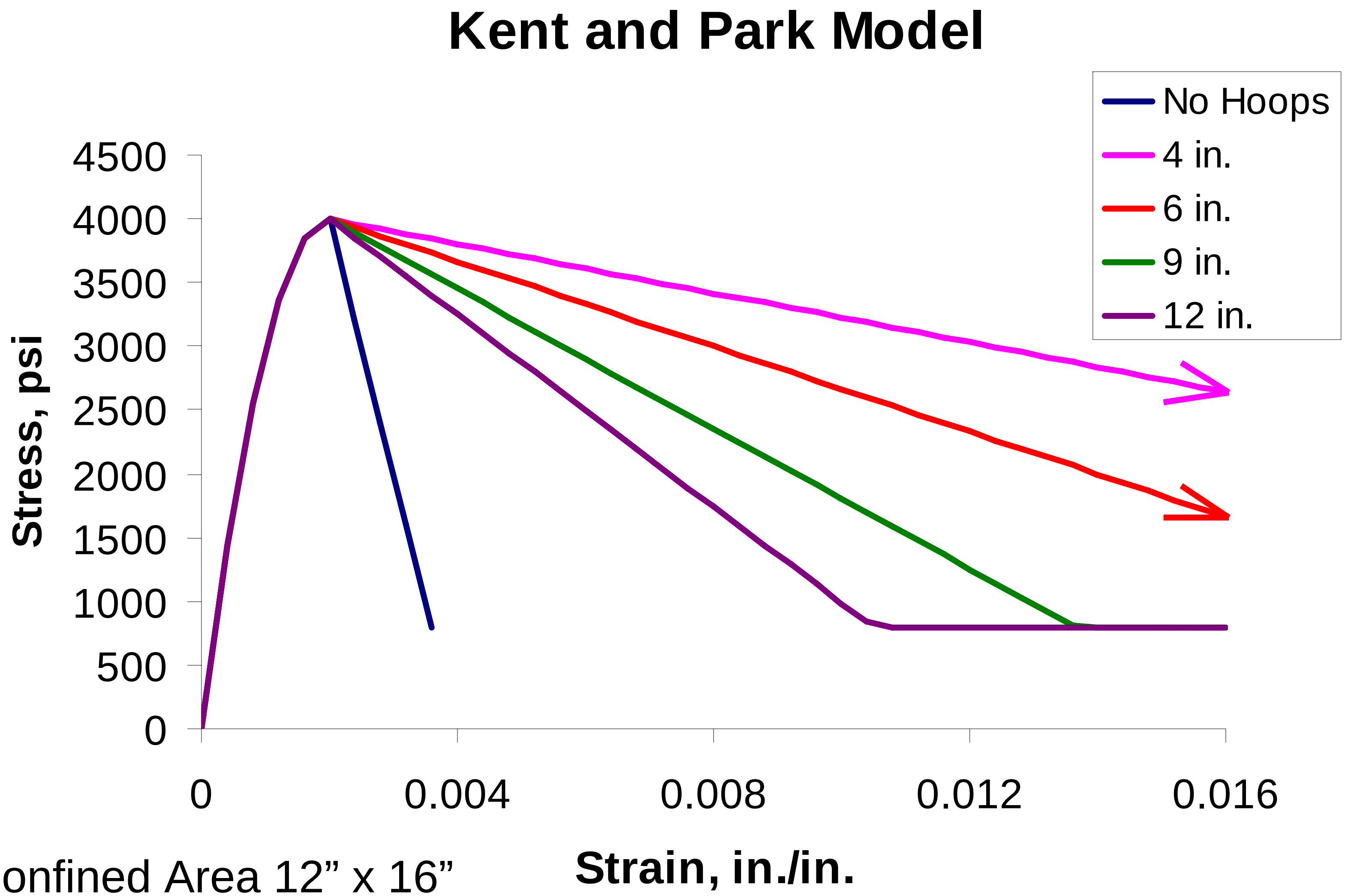
Confinement by longitudinal bars

Opened 90° hook on hoops

Confined Concrete Stress-Strain Behavior

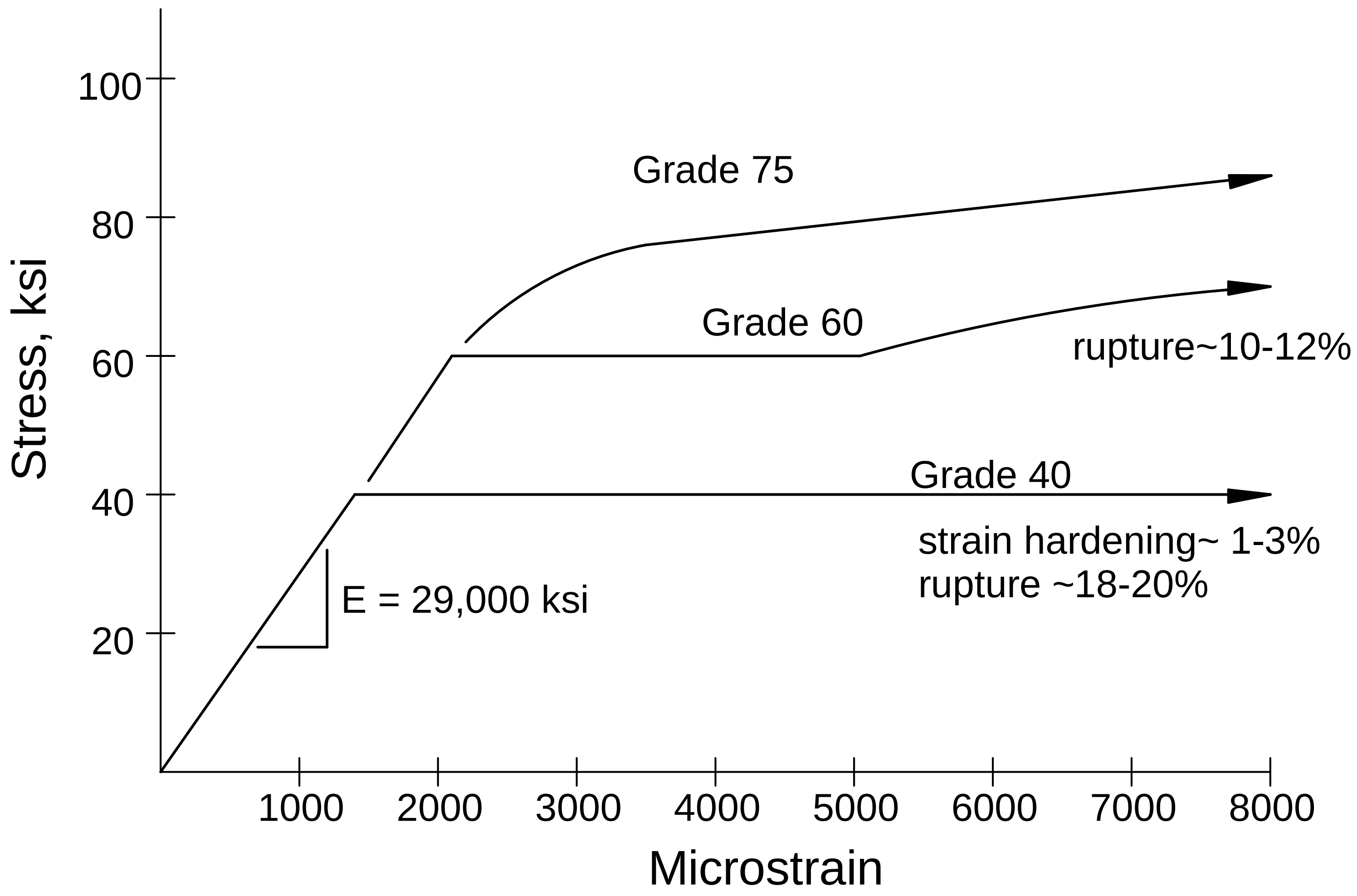


Idealized Stress-Strain Behavior of Confined Concrete

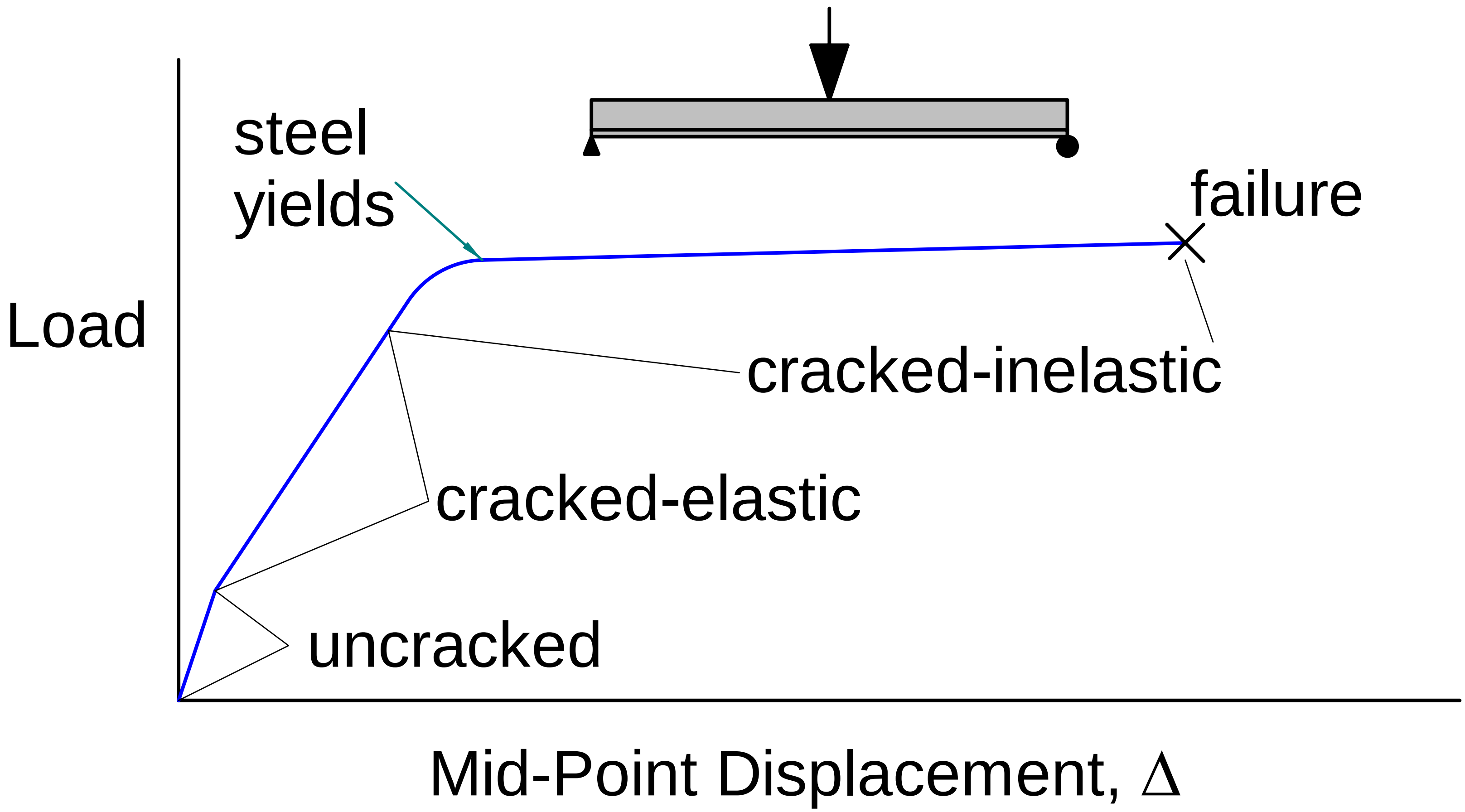


Confined Area 12” x 16”

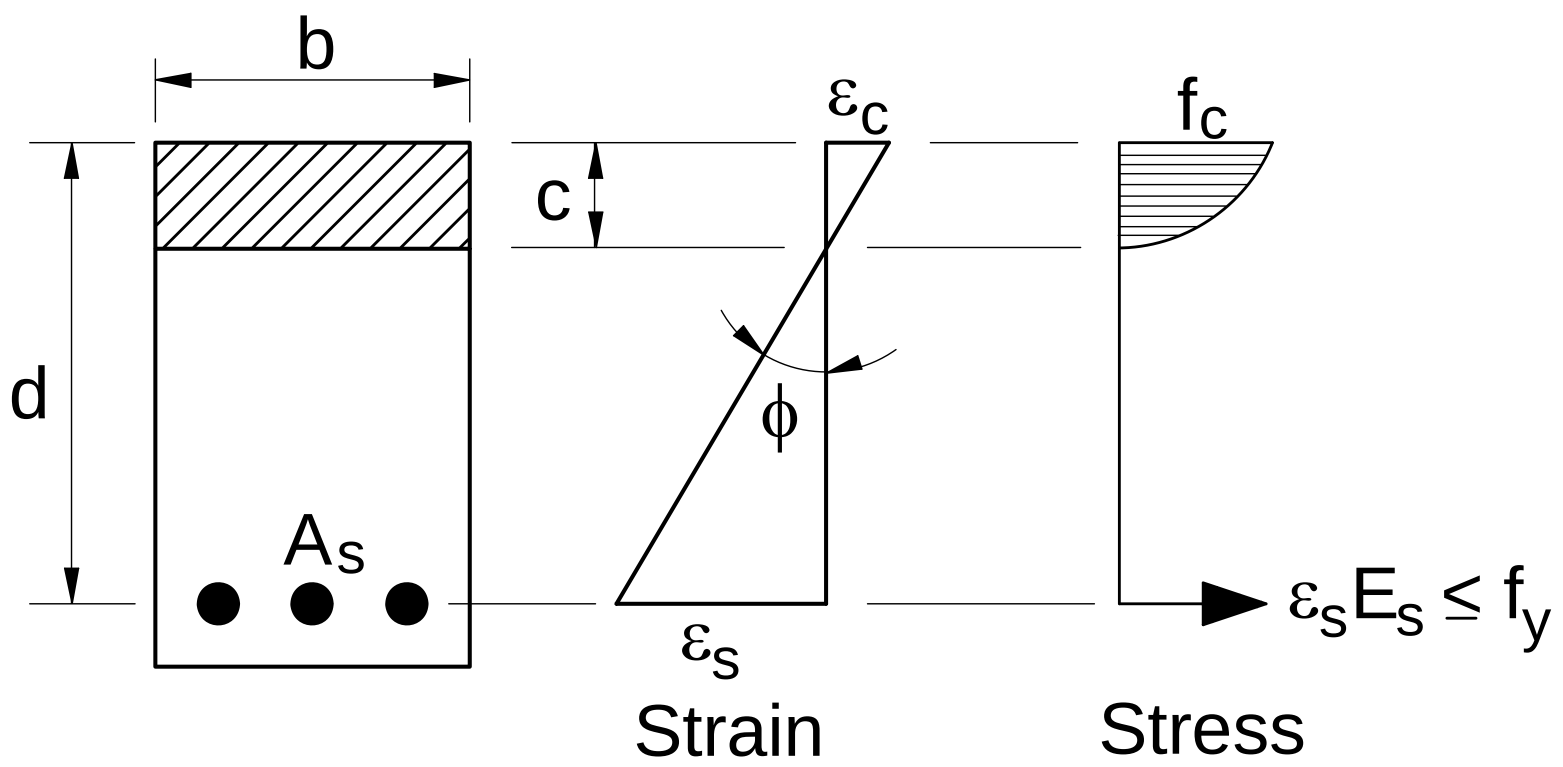
Reinforcing Steel Stress-Strain Behavior



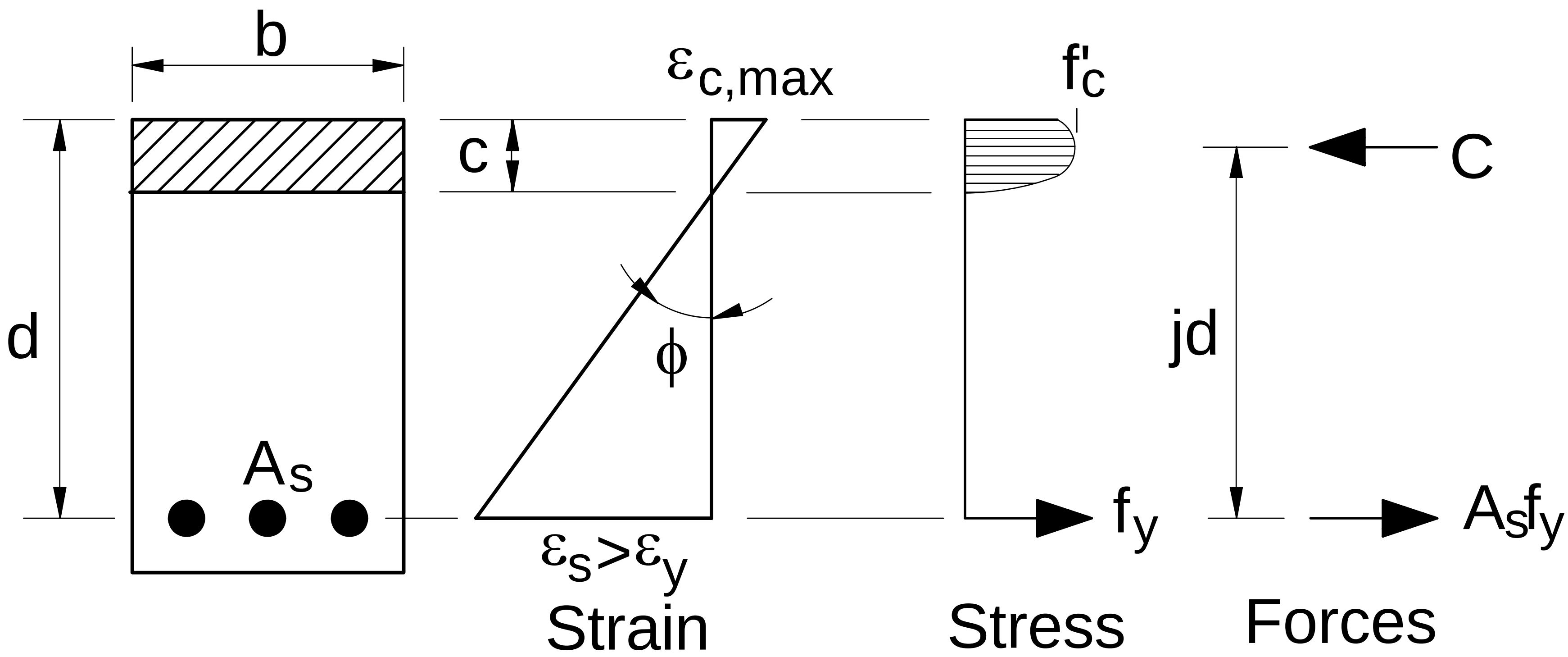
Reinforced Concrete Behavior



Behavior Up to First Yield of Steel

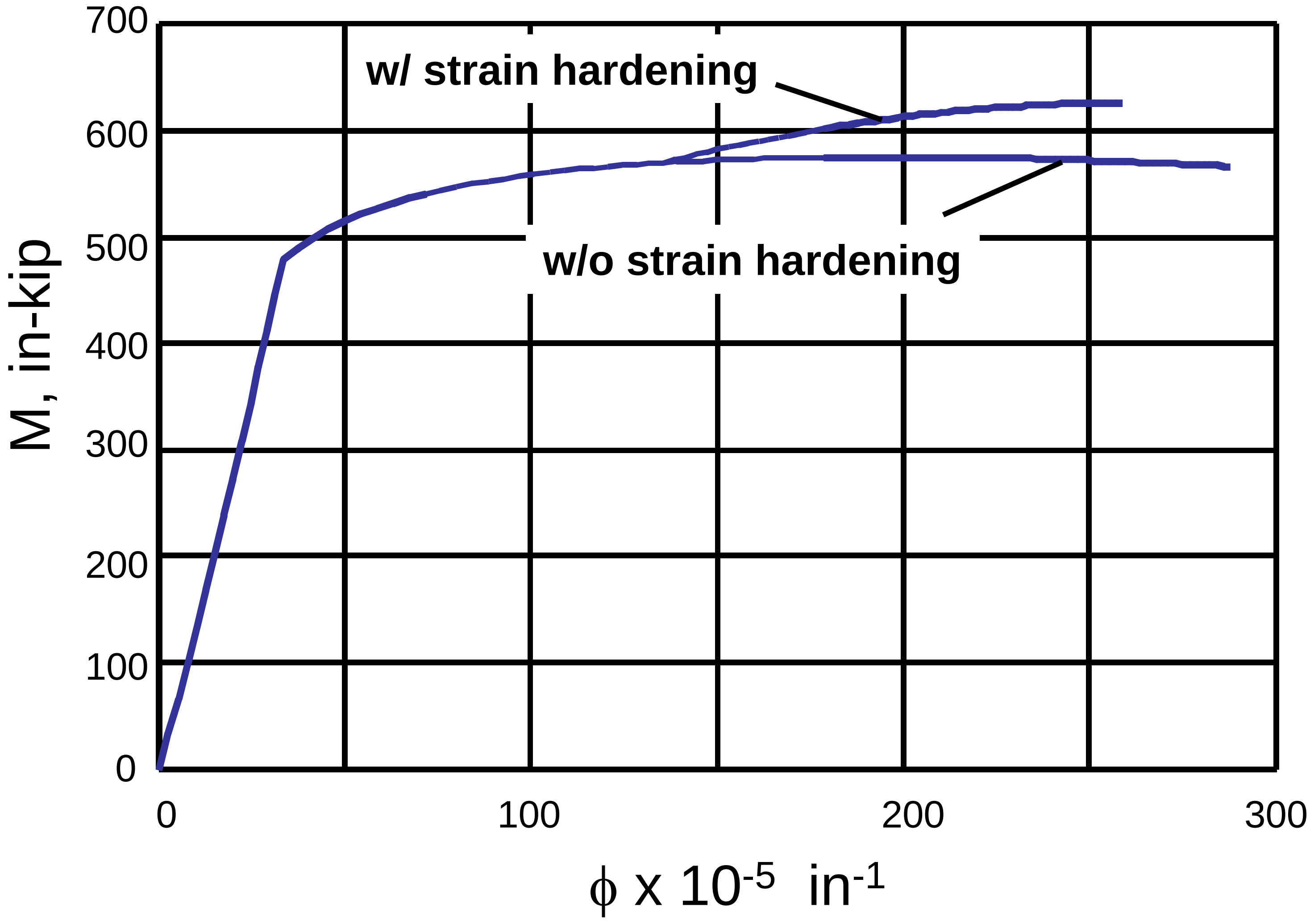


Behavior at Concrete Crushing



$$M_n = A_s f_y jd$$

Typical Moment Curvature Diagram



f'_c = 4 ksi

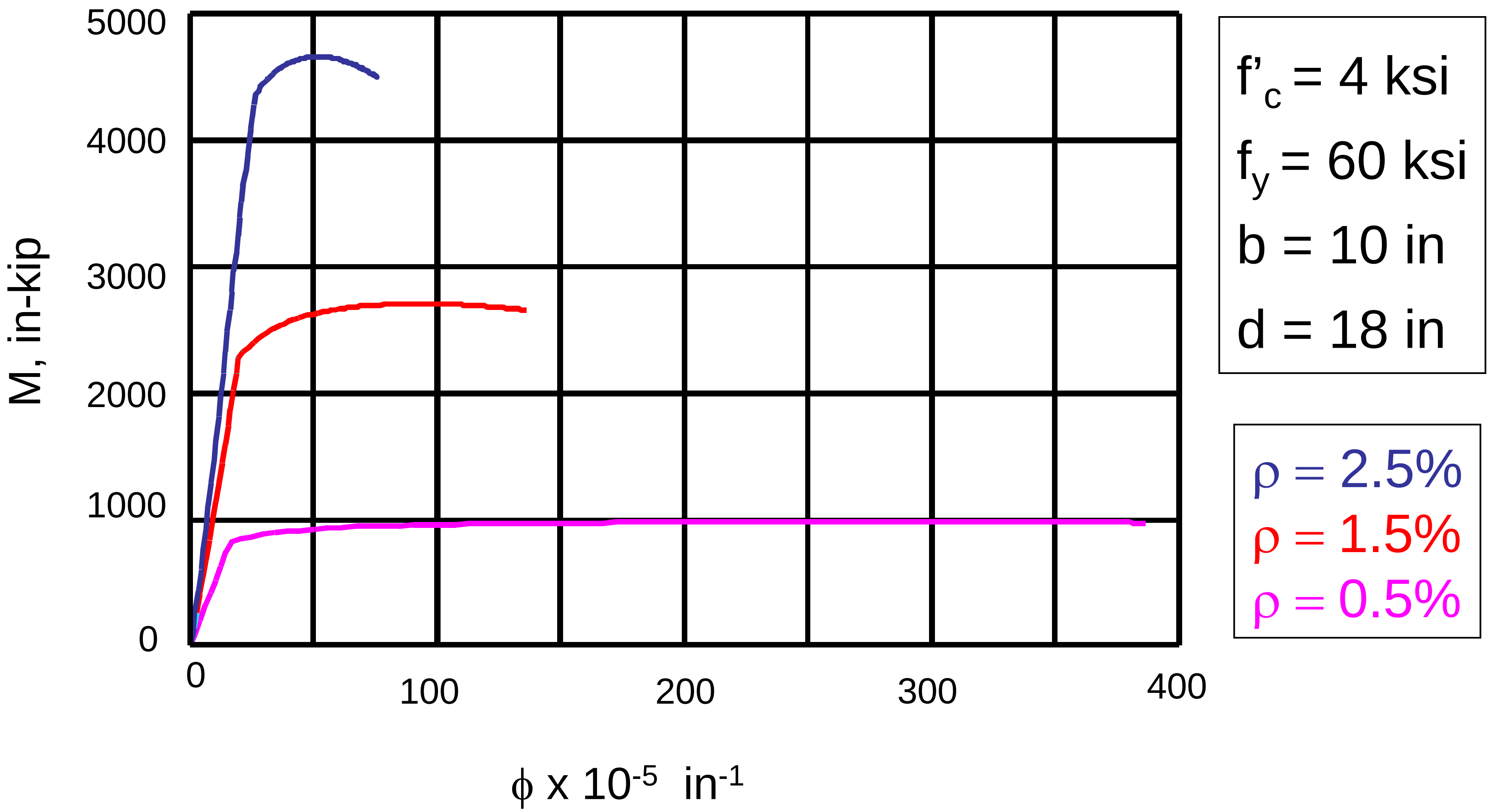
f_y = 60 ksi

b = 8 in

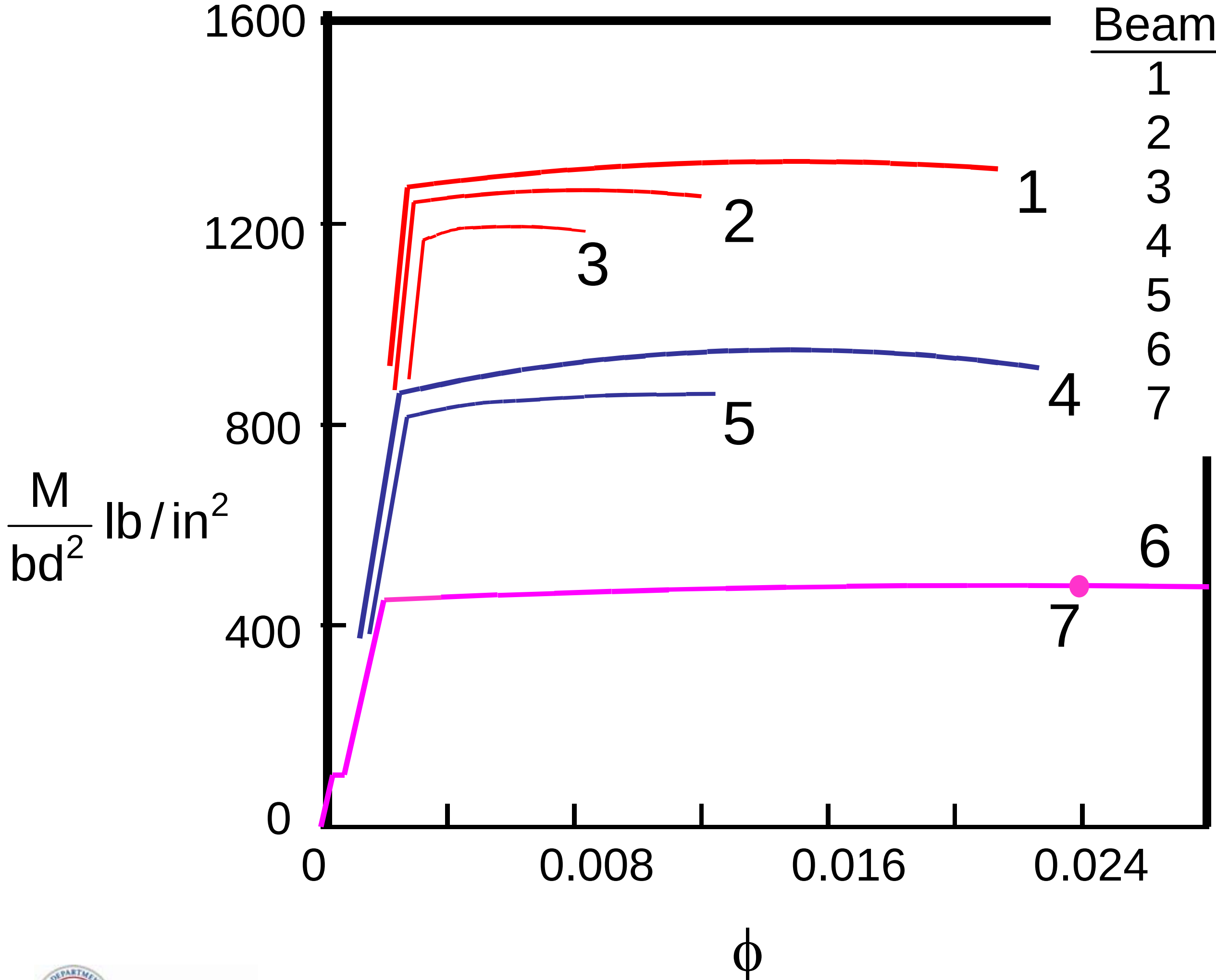
d = 10 in

ρ = 0.0125

Influence of Reinforcement Ratio

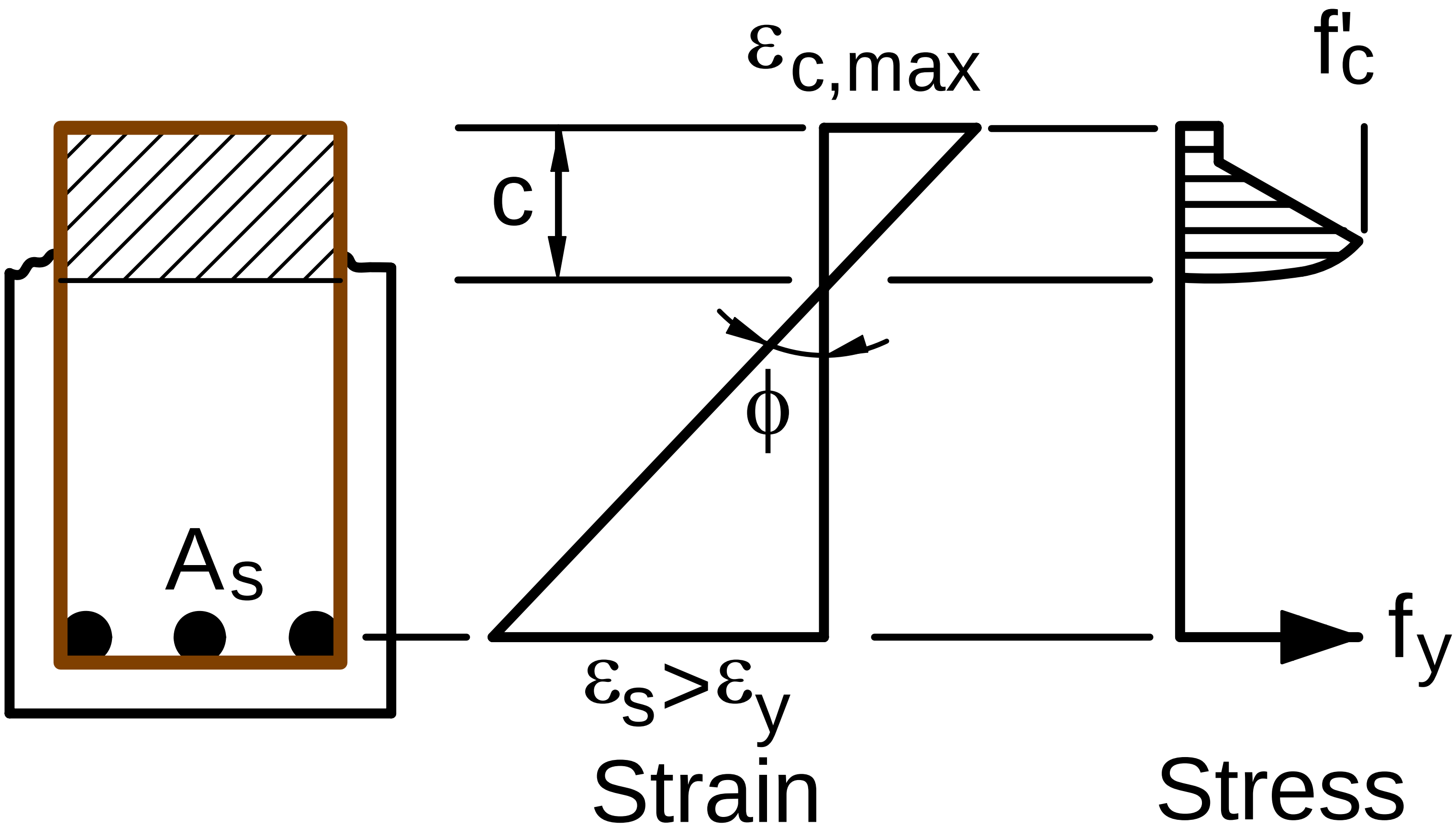


Influence of Compression Reinforcement

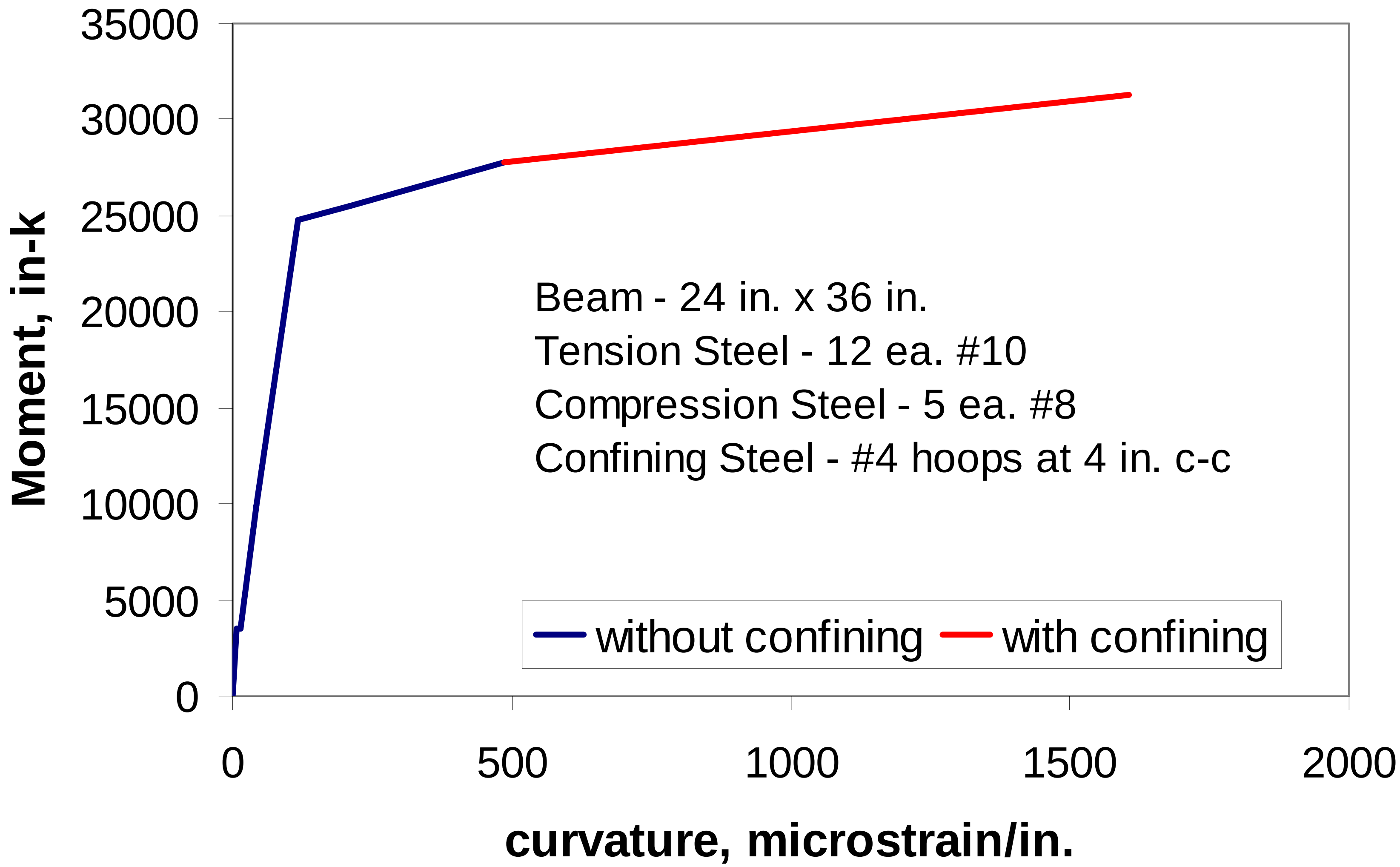


Beam	ρ	ρ'
1	0.0375	0.0250
2	0.0375	0.0125
3	0.0375	0
4	0.0250	0.0125
5	0.0250	0
6	0.0125	0.0125
7	0.0125	0

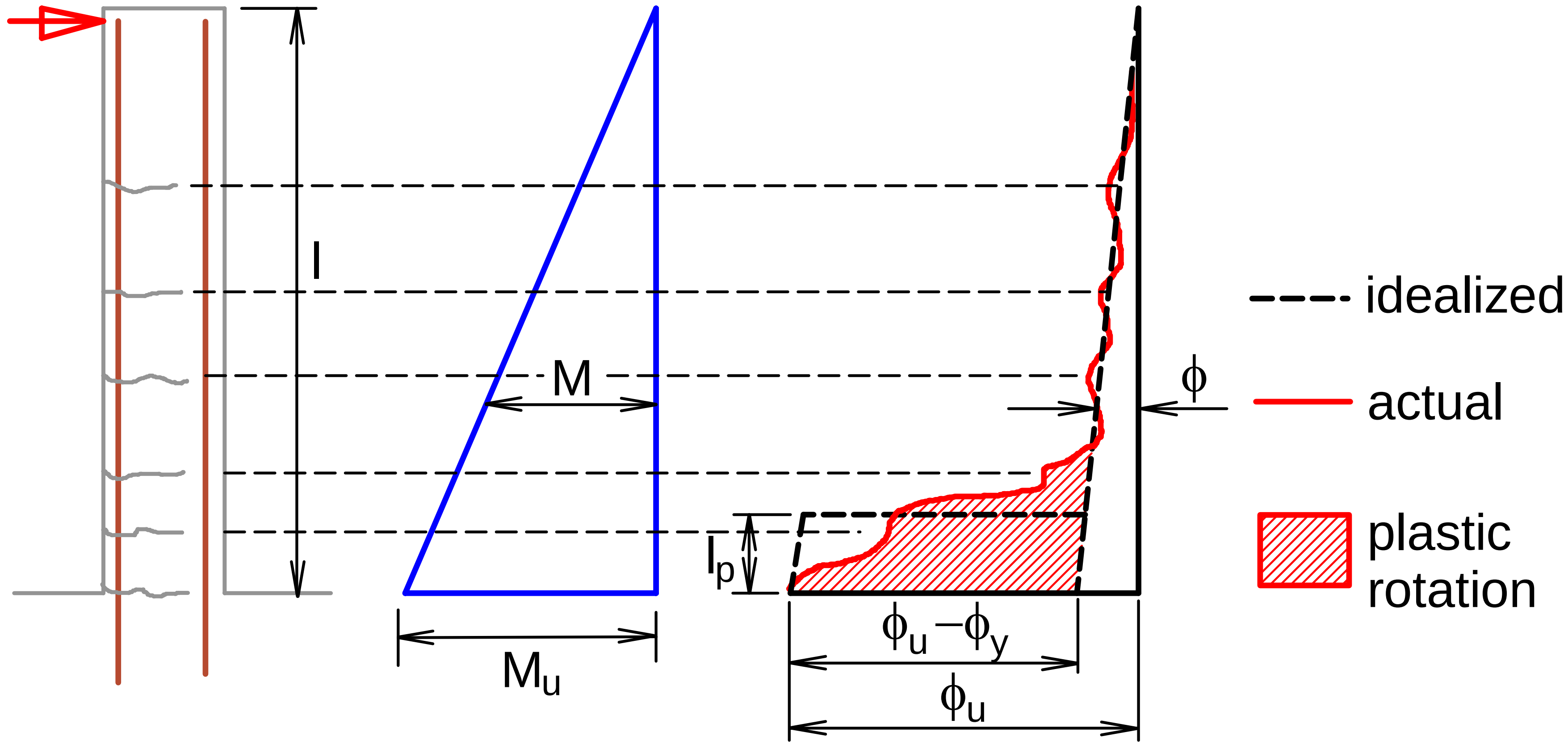
Moment-Curvature with Confined Concrete



Moment-Curvature with Confined Concrete



Plastic Hinging



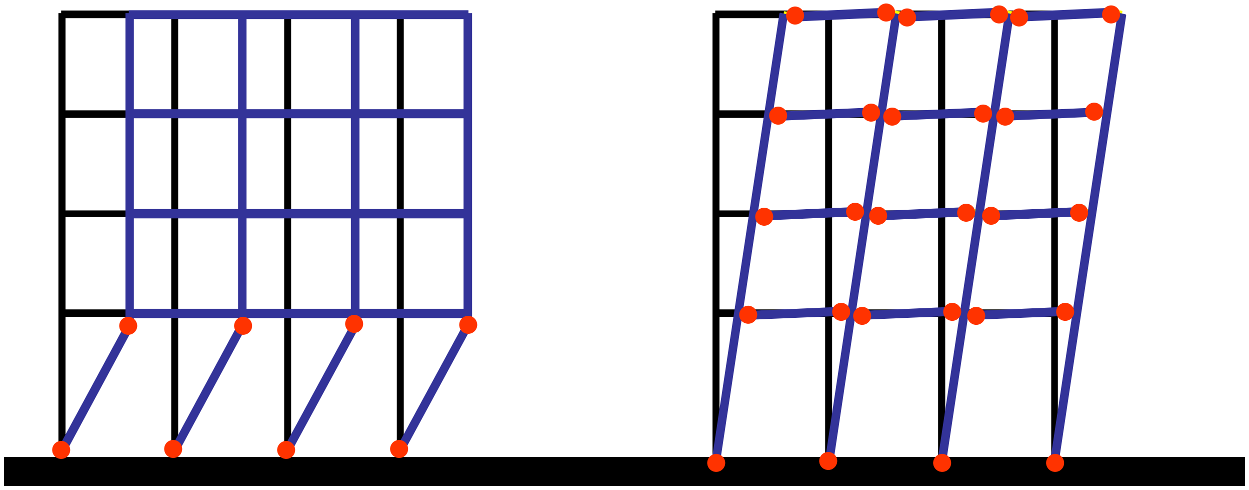
Strategies to Improve Ductility

- **Use low flexural reinforcement ratio**
- **Add compression reinforcement**
- **Add confining reinforcement**

Other Functions of Confining Steel

- **Acts as shear reinforcement**
- **Prevents buckling of longitudinal reinforcement**
- **Prevents bond splitting failures**

Structural Behavior
Frames

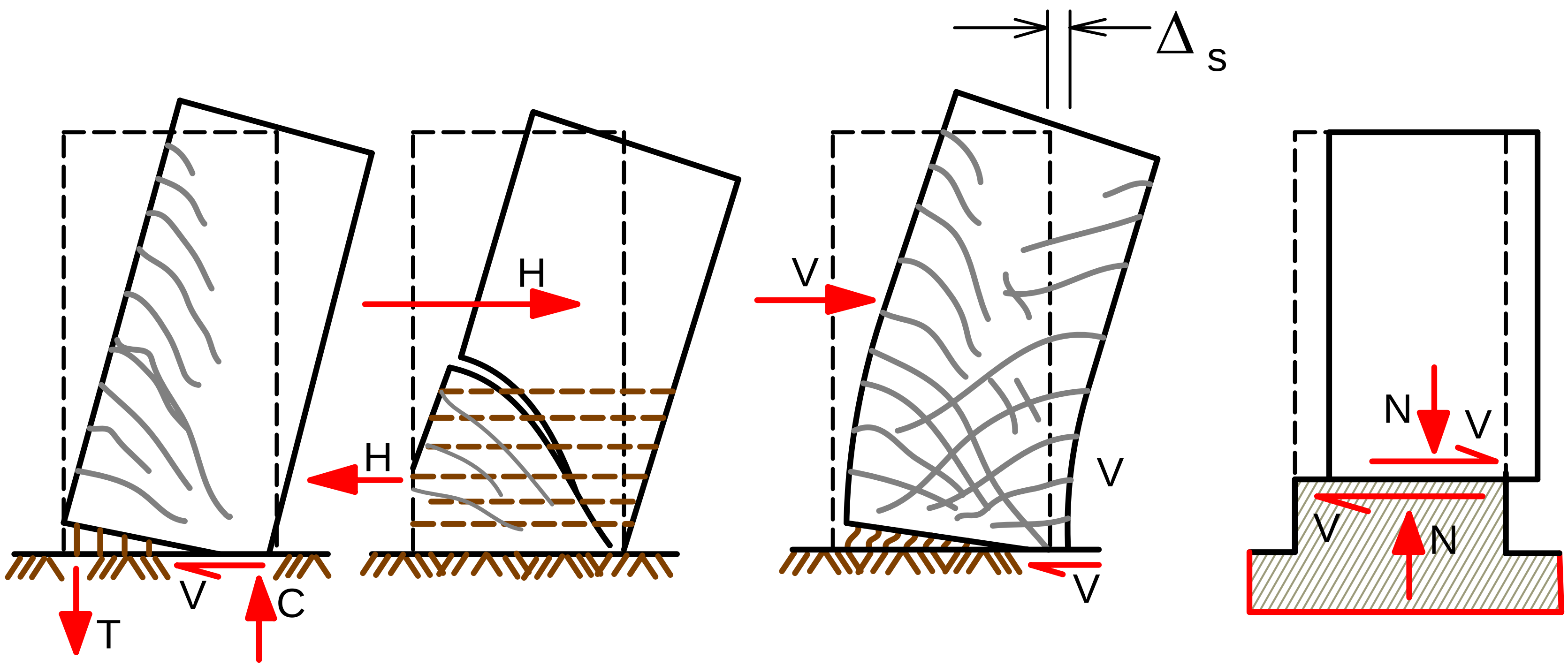


Story Mechanism

Sway Mechanism

Story Mechanism

Structural Behavior - Walls



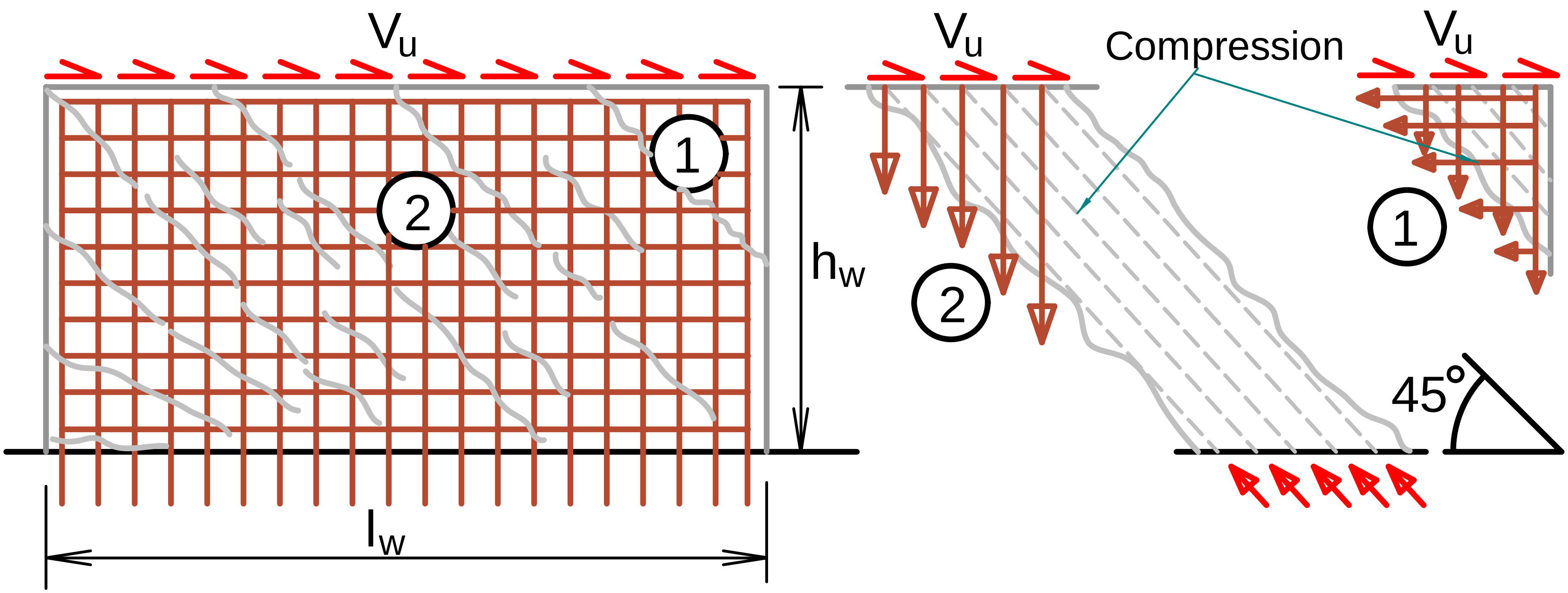
Flexural failure

Horizontal tension

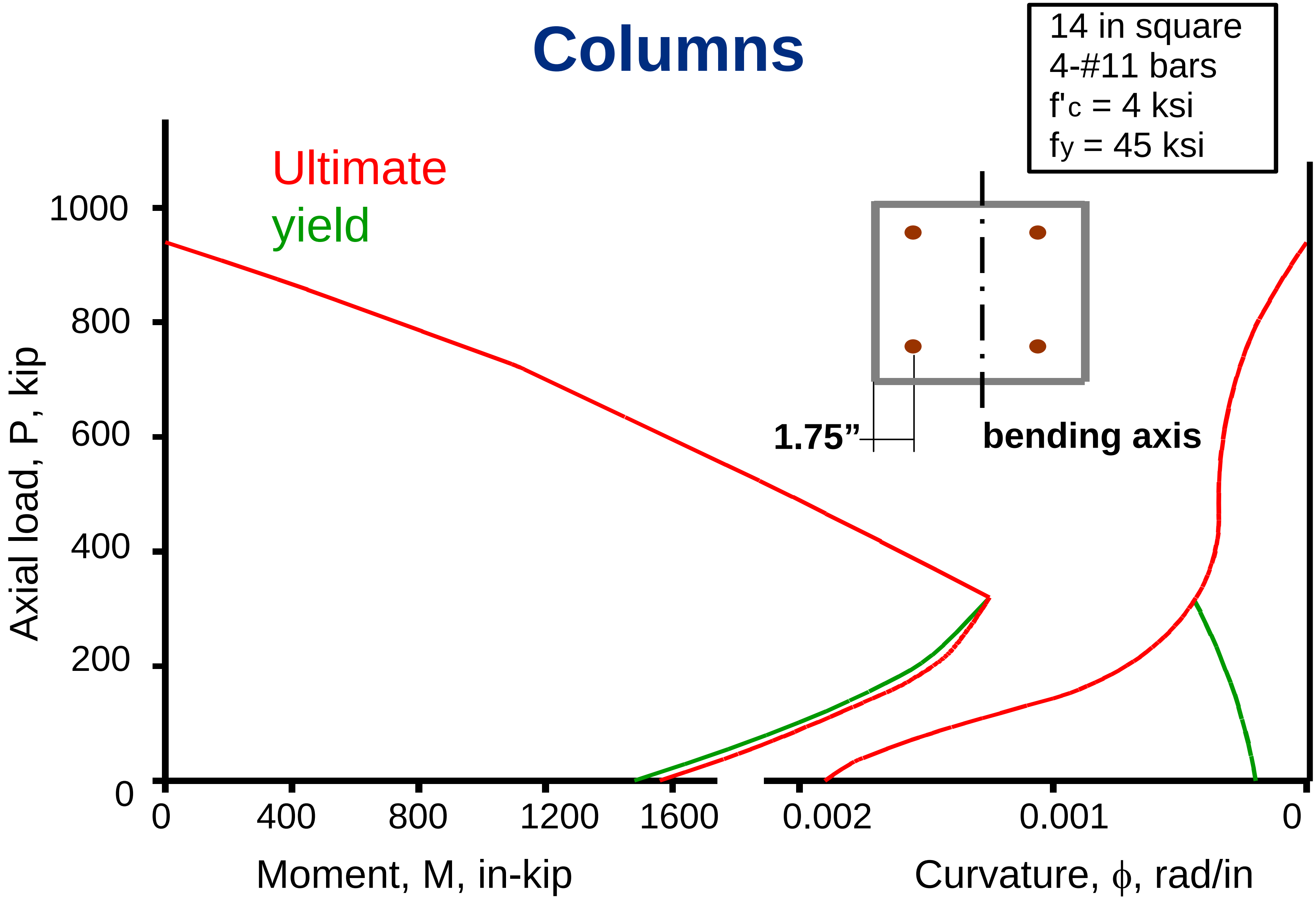
Sliding on flexural cracks

Sliding on construction joint

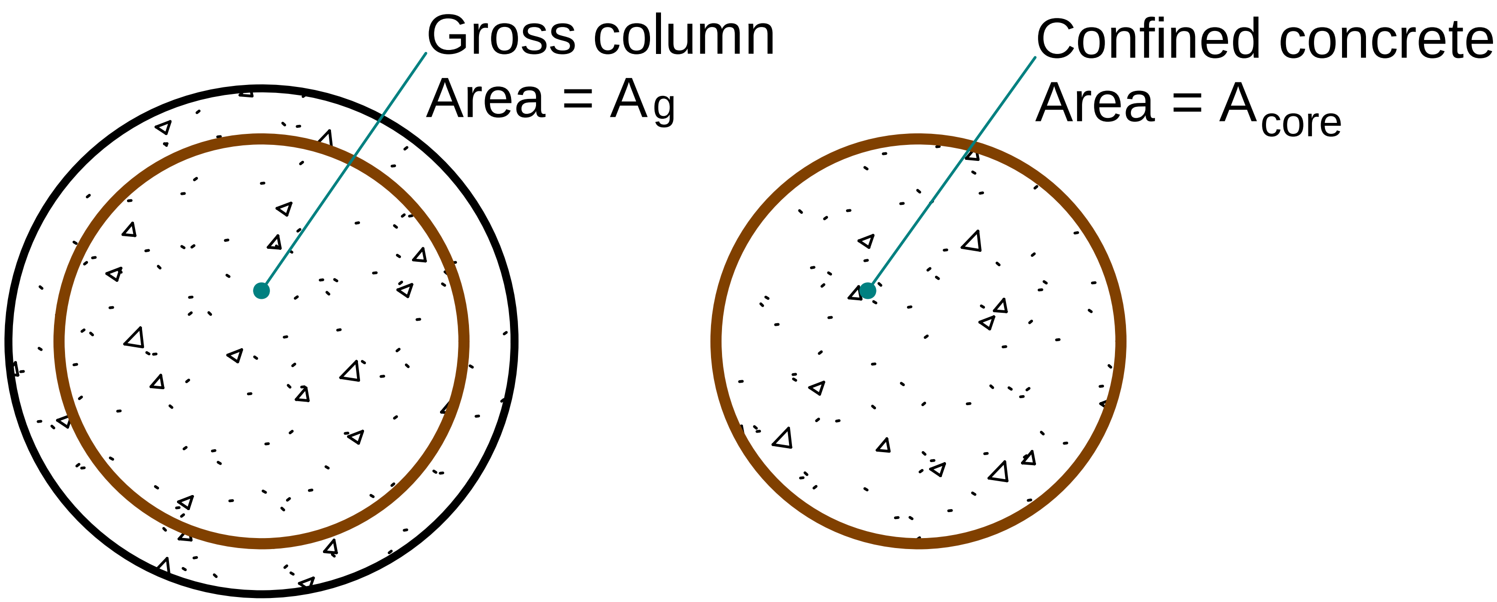
Structural Behavior
Walls



Structural Behavior
Columns



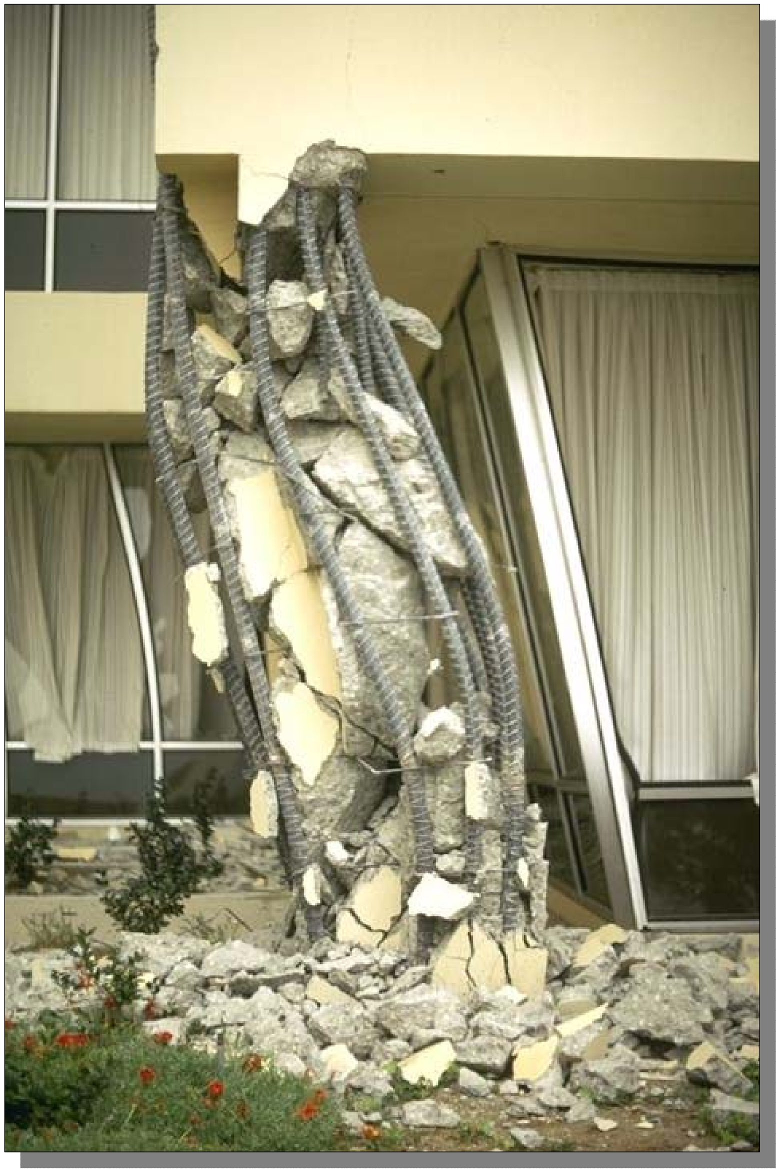
Influence of Hoops on Axial Strength



Before spalling-
$P = A_g f'_c$

After spalling-
$P = A_{core}(f'_c + 4\ f_{lat})$

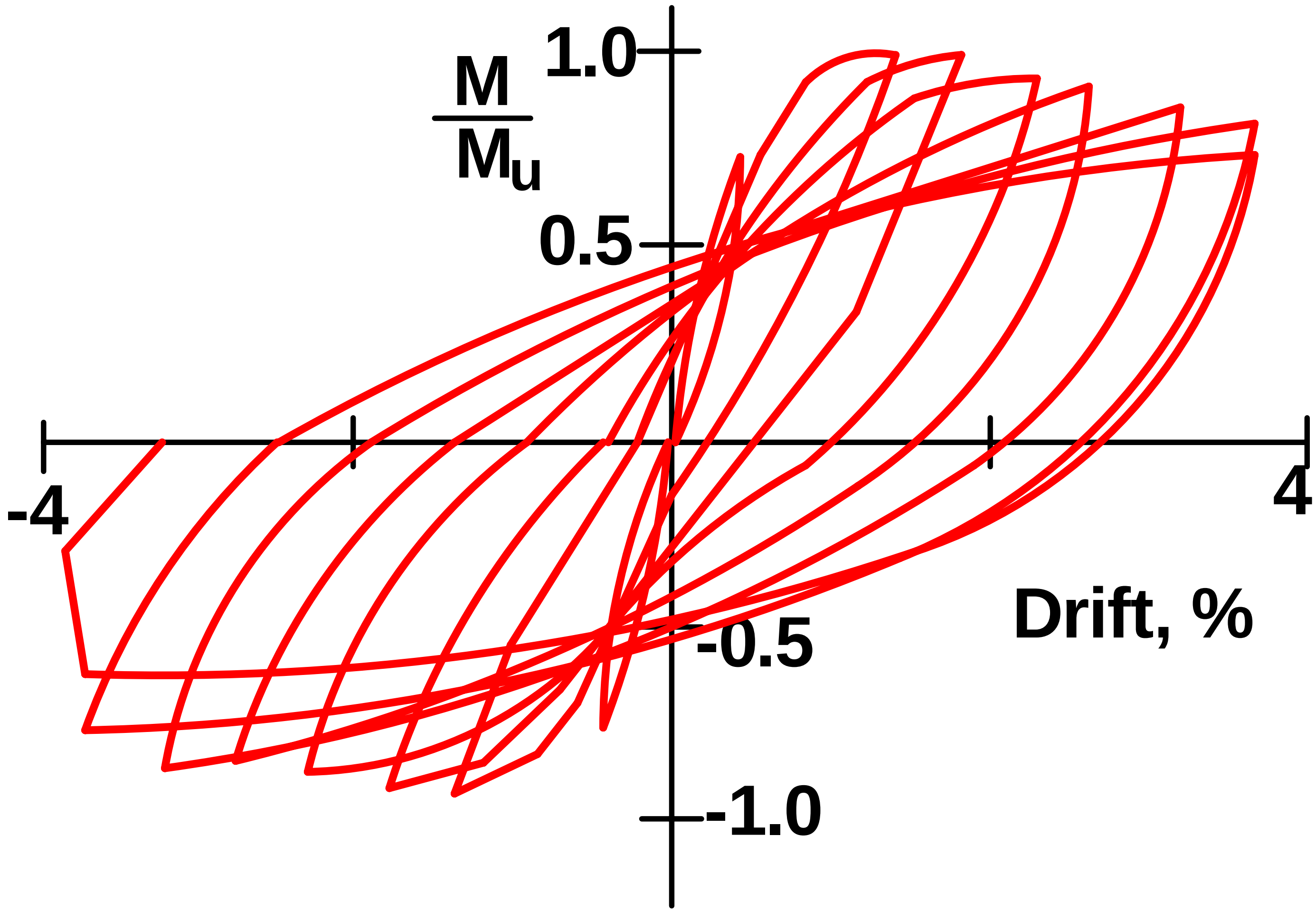
After spalling $\geq$ Before spalling



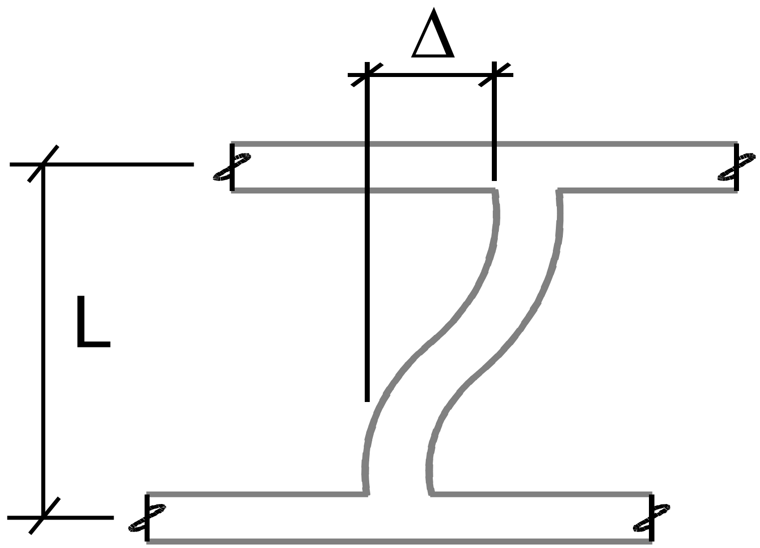
Column with Inadequate Ties

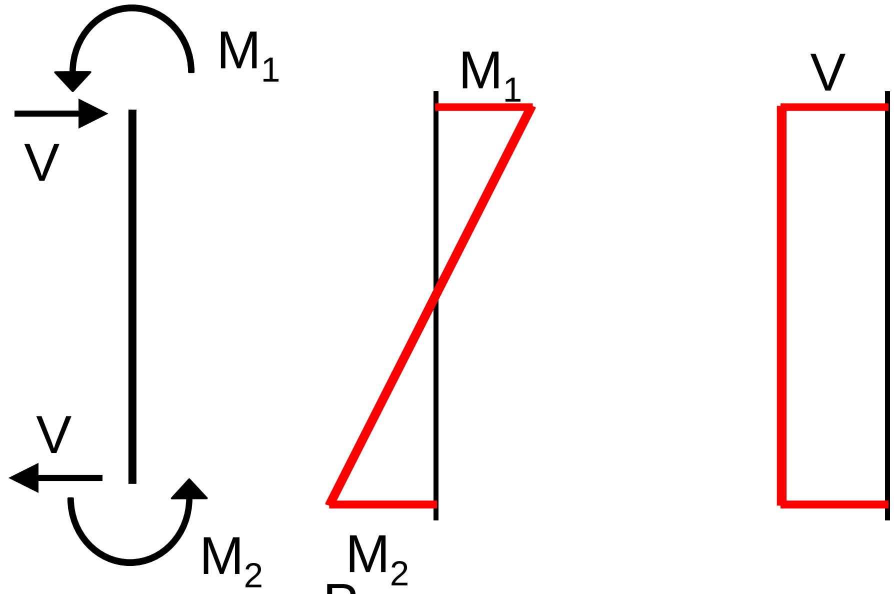
Well Confined Column

Hysteretic Behavior of Well Confined Column

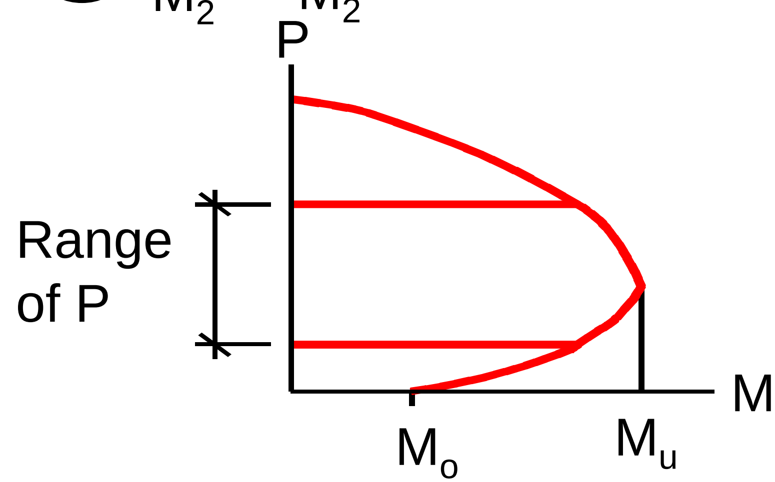


Structural Behavior
Columns



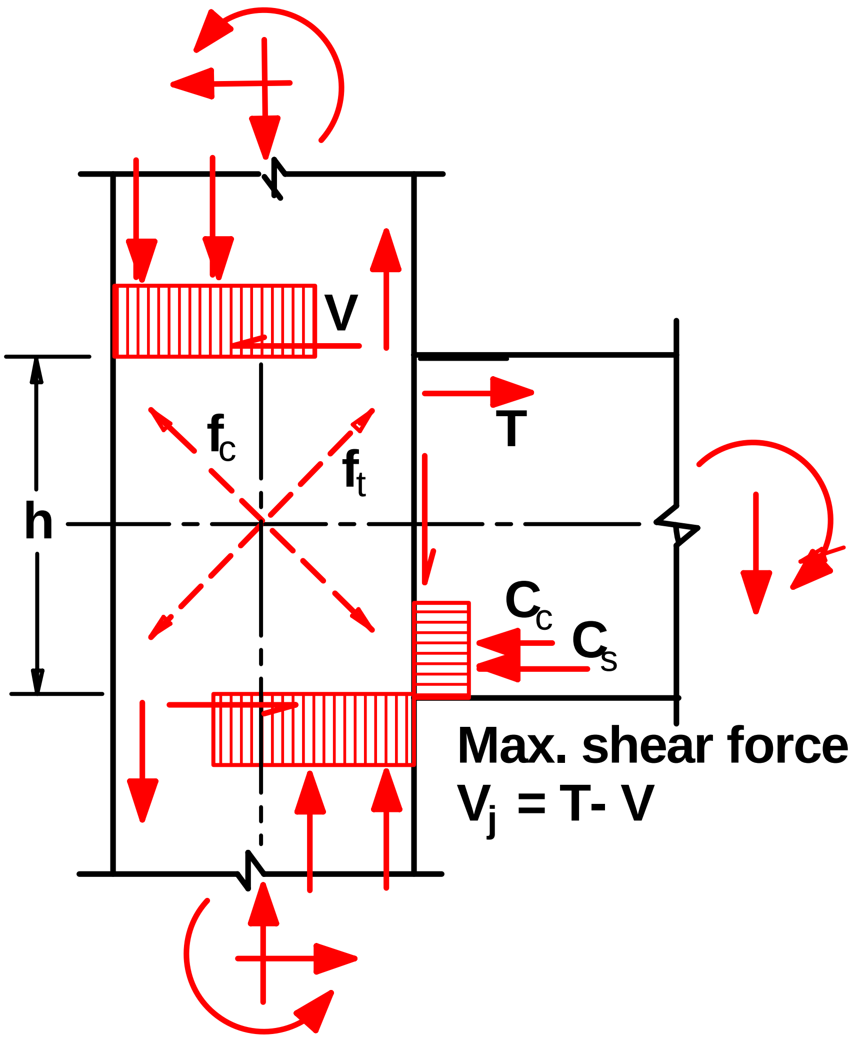


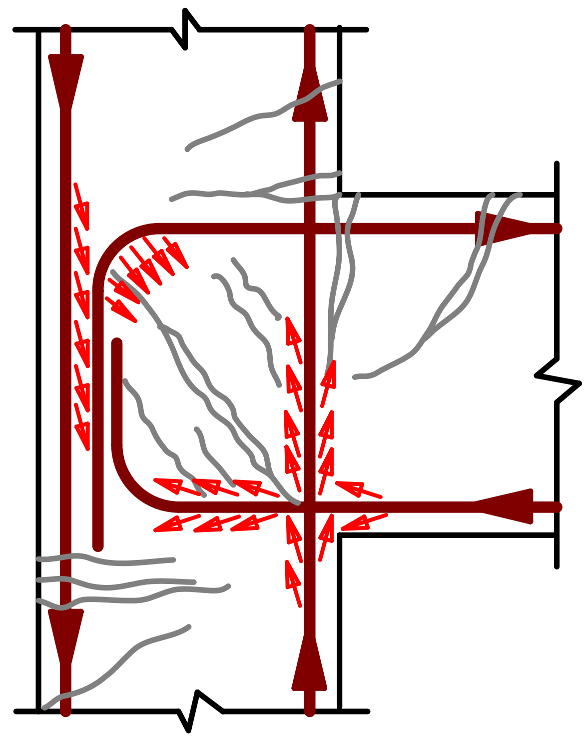
$$V = \frac{M_1 + M_2}{L} = \frac{2M_u}{L}$$



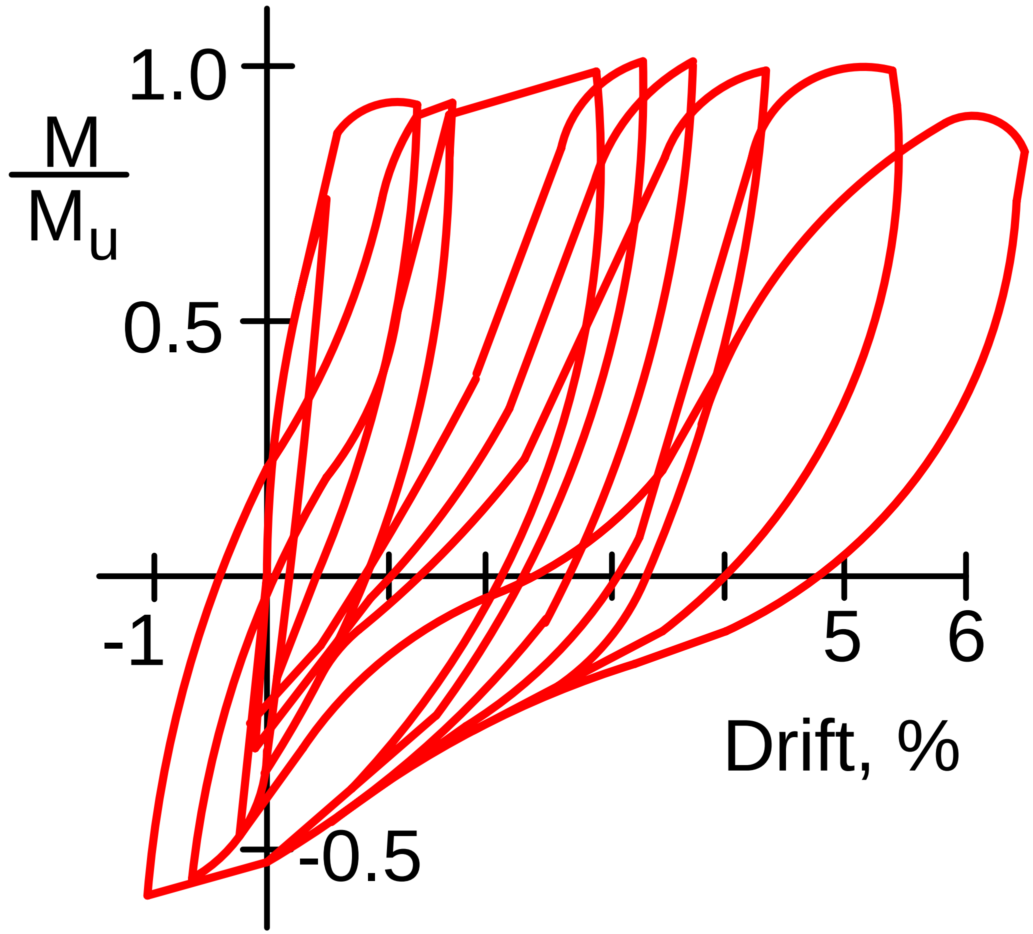
Column Shear Failure

Structural Behavior
Joints

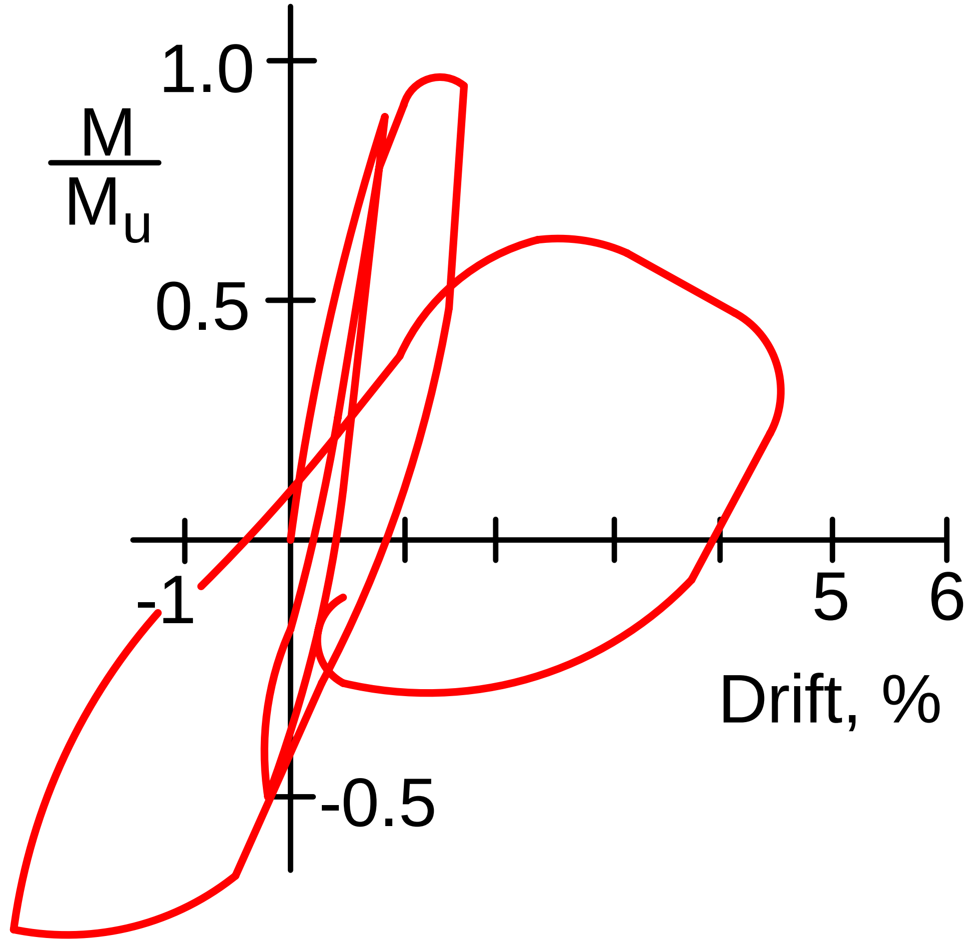




Hysteretic Behavior of Joint with Hoops



Hysteretic Behavior of Joint with No Hoops



Joint Failure – No Shear Reinforcing

Anchorage Failure in Column/Footing Joint

Summary of Concrete Behavior

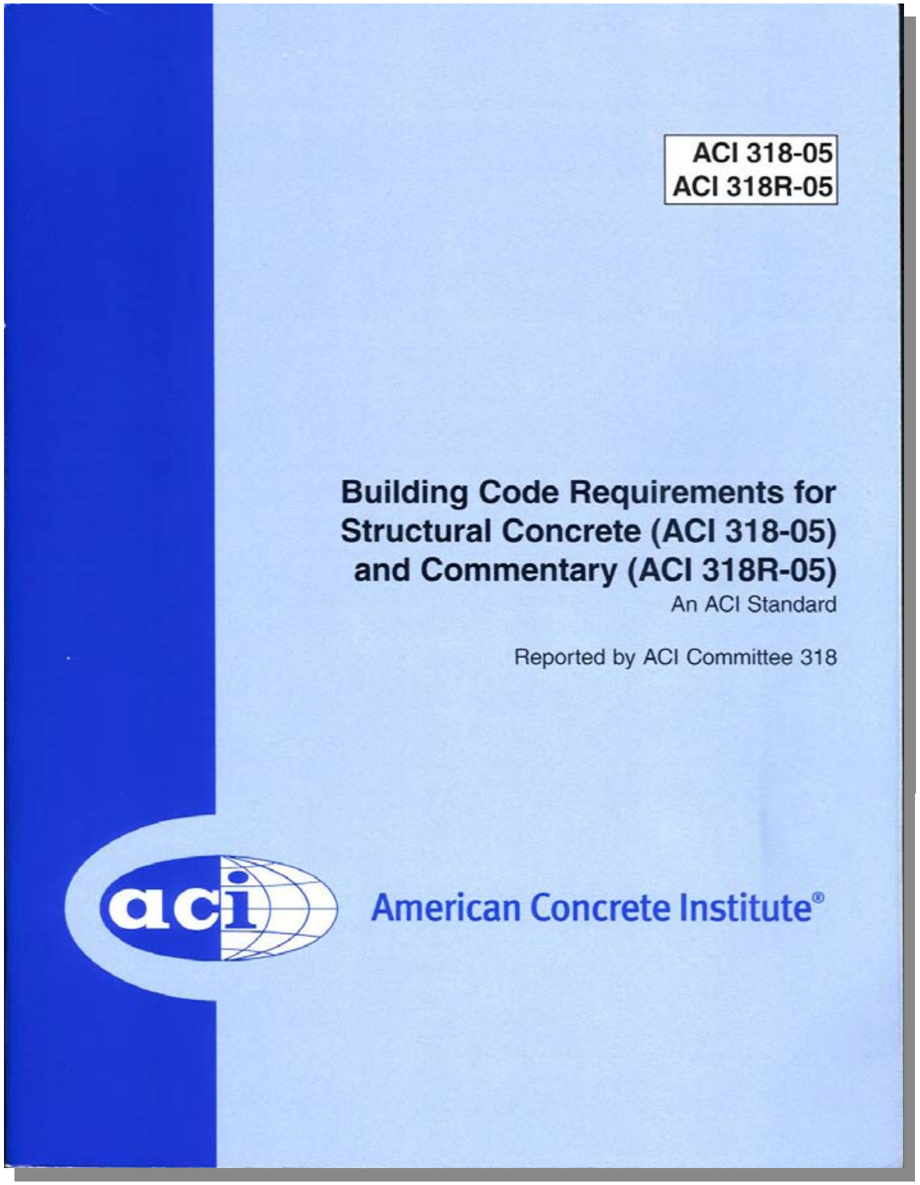
- **Compressive Ductility**
  - Strong in compression but brittle
  - Confinement improves ductility by
    - Maintaining concrete core integrity
    - Preventing longitudinal bar buckling
- **Flexural Ductility**
  - Longitudinal steel provides monotonic ductility at low reinforcement ratios
  - Transverse steel needed to maintain ductility through reverse cycles and at very high strains (hinge development)

Summary of Concrete Behavior

- **Damping**
  - Well cracked: moderately high damping
  - Uncracked (e.g. prestressed): low damping
- **Potential Problems**
  - Shear failures are brittle and abrupt and must be avoided
  - Degrading strength/stiffness with repeat cycles
    - Limit degradation through adequate hinge development

NEHRP Recommended Provisions Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**
- **Reference standards**


ACI 318-05
ACI 318R-05
Building Code Requirements for
Structural Concrete (ACI 318-05)
and Commentary (ACI 318R-05)
An ACI Standard
Reported by ACI Committee 318
aci
American Concrete Institute®

ACI 318-05

Use of Reference Standards

- **ACI 318-05**
  - Chapter 21, Special Provisions for Seismic Design
- **NEHRP Chapter 9, Concrete Structures**
  - General design requirements
  - Modifications to ACI 318
  - Seismic Design Category requirements
  - Special precast structural walls
  - Untopped precast diaphragms (Appendix to Ch.9)

Detailed Modifications to ACI 318

- Modified definitions and notations
- Scope and material properties
- Special moment frames
- Special shear walls
- Special and intermediate precast walls
- Foundations
- Anchoring to concrete

NEHRP Recommended Provisions Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**
- **Reference standards**
- **Requirements by Seismic Design Category**

Design Coefficients - Moment Resisting Frames

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Special R/C Moment Frame	8	5.5
Intermediate R/C Moment Frame	5	4.5
Ordinary R/C Moment Frame	3	2.5

Design Coefficients
Shear Walls (Bearing Systems)

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Special R/C Shear Walls	5	5
Ordinary R/C Shear Walls	4	4
Intermediate Precast Shear Walls	4	4
Ordinary Precast Walls	3	3

Design Coefficients
Shear Walls (Frame Systems)

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Special R/C Shear Walls	6	5
Ordinary R/C Shear Walls	5	4.5
Intermediate Precast Shear Walls	5	4.5
Ordinary Precast Walls	4	4

Design Coefficients
Dual Systems with Special Frames

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Dual System w/ Special Walls	8 (7)	6.5 (5.5)
Dual System w/ Ordinary Walls	6	5

(ASCE 7-05 values where different)

Frames

Seismic Design Category	Minimum Frame Type	ACI 318 Requirements
A and B	Ordinary	Chapters 1 thru 18 and 22
C	Intermediate	ACI 21.2.1.3 and ACI 21.12
D, E and F	Special	ACI 21.2.1.4 and ACI 21.2, 21.3, 21.4, and 21.5

Reinforced Concrete Shear Walls

Seismic Design Category	Minimum Wall Type	ACI 318 Requirements
A, B and C	Ordinary	Chapters 1 thru 18 and 22
D, E and F	Special	ACI 21.2.1.4 and ACI 21.2 and 21.7

Precast Concrete Shear Walls

Seismic Design Category	Minimum Wall Type	ACI 318 Requirements
A and B	Ordinary	Chapters 1 thru 18 and 22
C	Intermediate	ACI 21.2.1.3 and ACI 21.13
D, E and F	Special	ACI 21.2.1.4 and ACI 21.2, 21.8

Additional *Provisions* Requirements

- **Category C**
  - Discontinuous members
  - Plain concrete
    - Walls
    - Footings
    - Pedestals (not allowed)

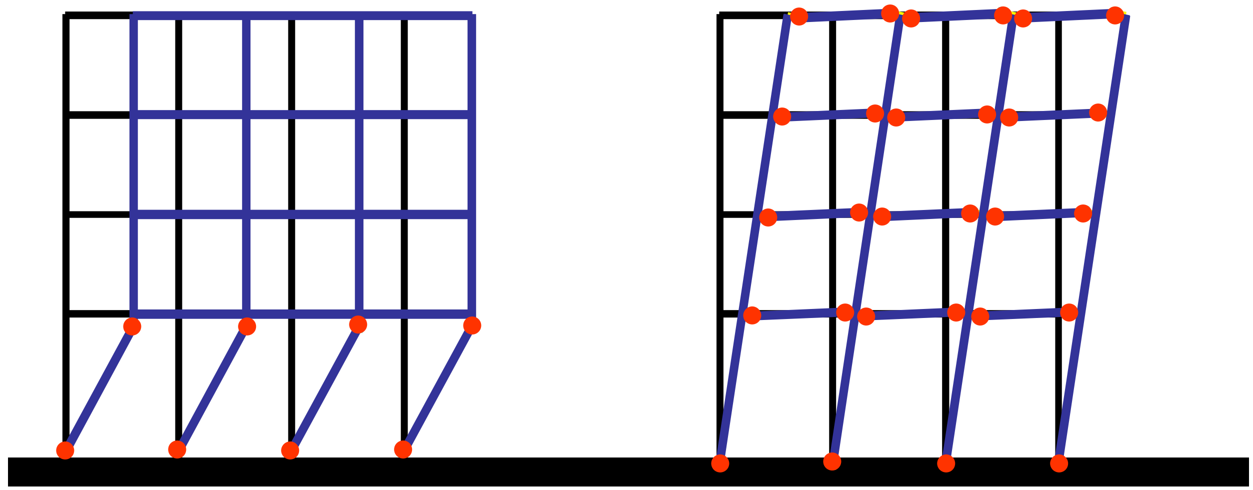
NEHRP Recommended Provisions Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**
- **Reference standards**
- **Requirements by Seismic Design Category**
- **Moment resisting frames**

Performance Objectives

- **Strong column**
  - Avoid story mechanism
- **Hinge development**
  - Confined concrete core
  - Prevent rebar buckling
  - Prevent shear failure
- **Member shear strength**
- **Joint shear strength**
- **Rebar development**

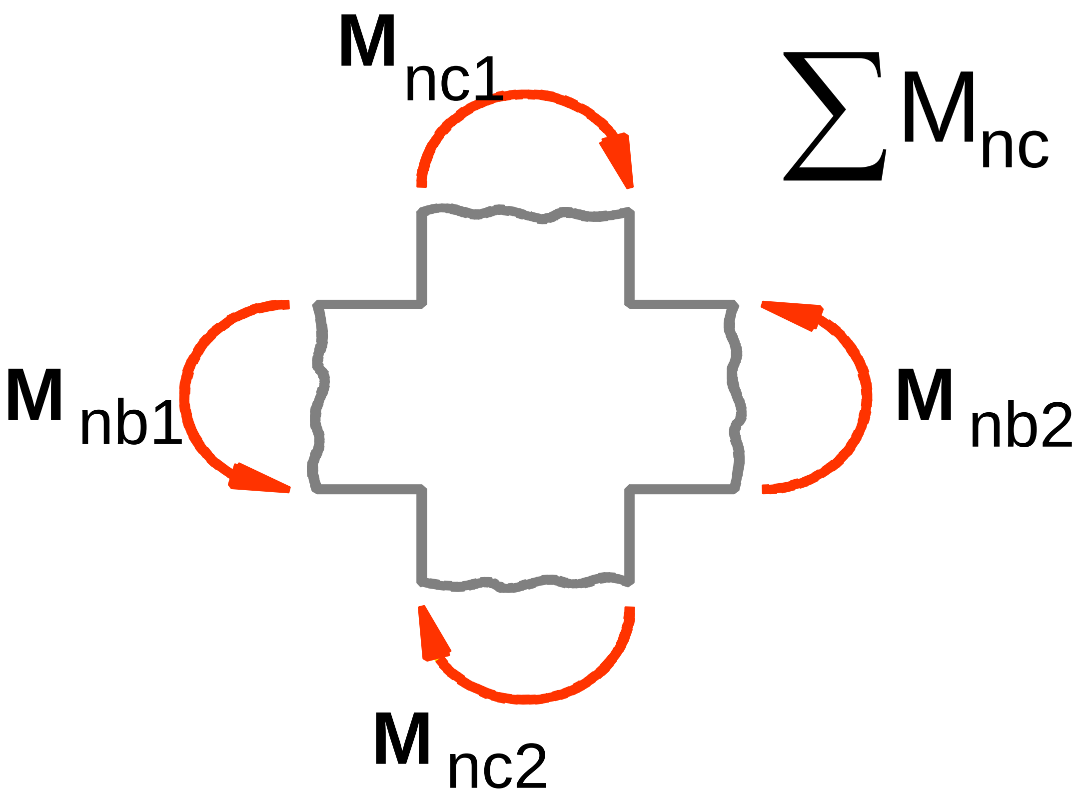
Frame Mechanisms
“strong column – weak beam”



Story mechanism

Sway mechanism

Required Column Strength

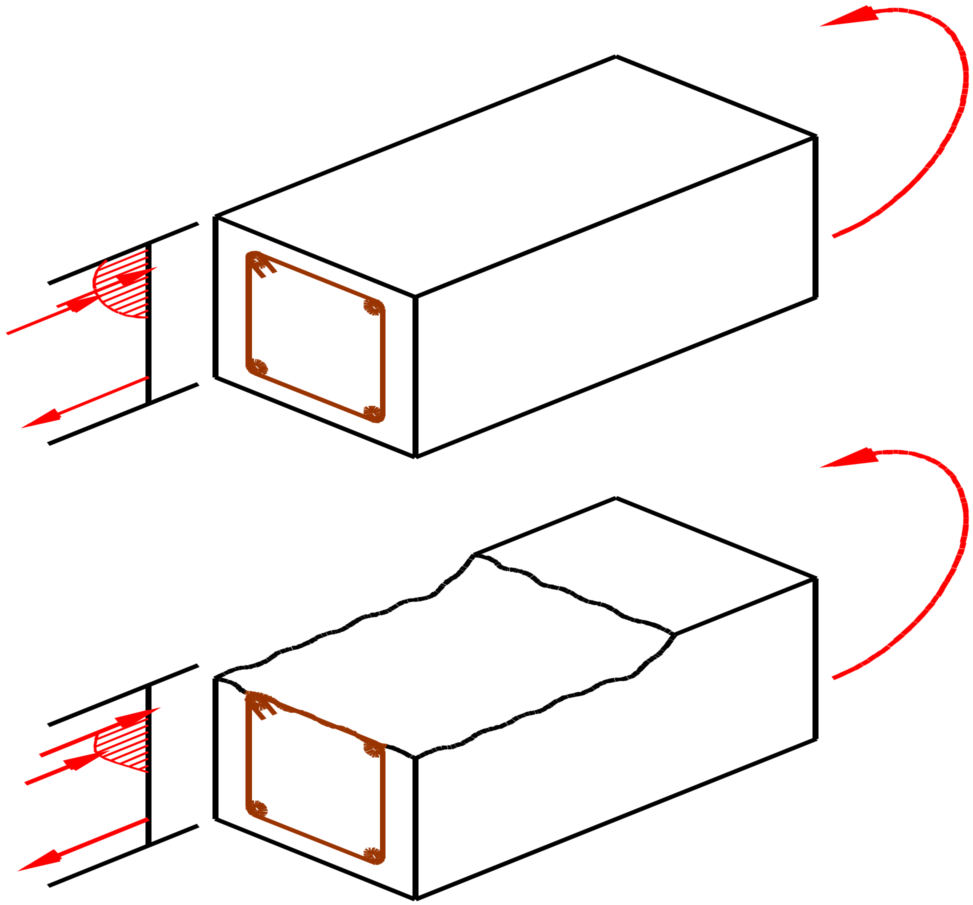


$$\sum M_{nc} \geq 12 \sum M_{nb}$$

Hinge Development

- **Tightly Spaced Hoops**
  - Provide confinement to increase concrete strength and usable compressive strain
  - Provide lateral support to compression bars to prevent buckling
  - Act as shear reinforcement and preclude shear failures
  - Control splitting cracks from high bar bond stresses

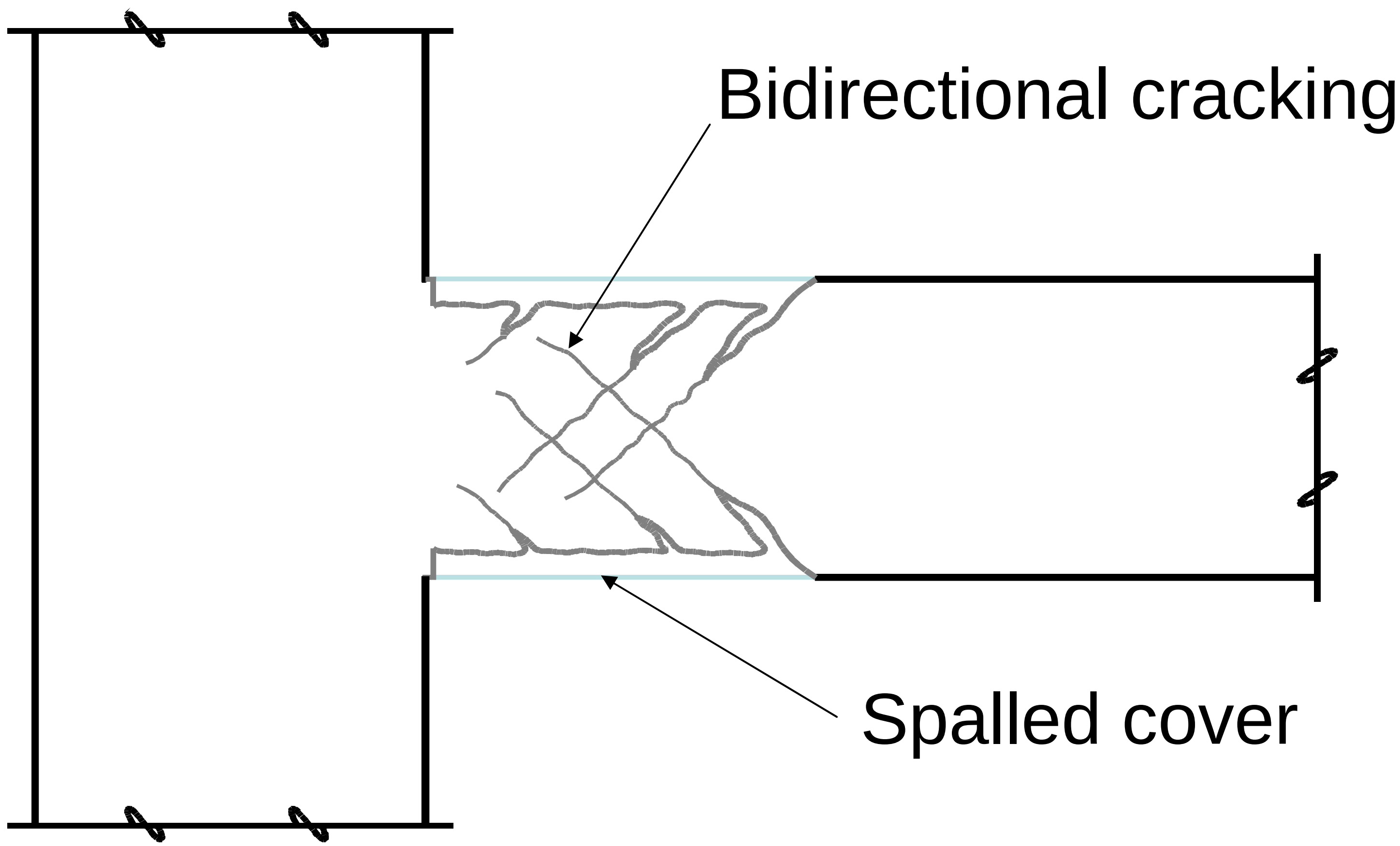
Hinge Development



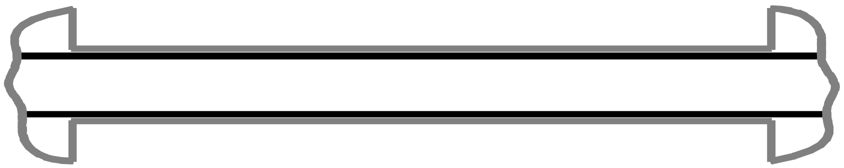
Before spalling

After spalling

Hinge Development

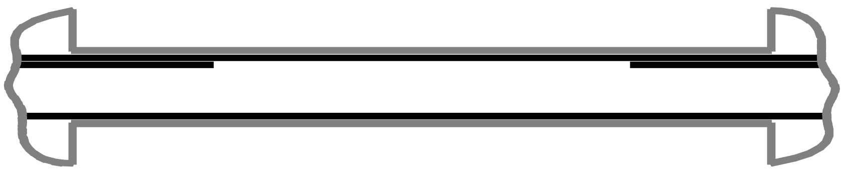


ACI 318-05, Overview of Frames:
Beam Longitudinal Reinforcement

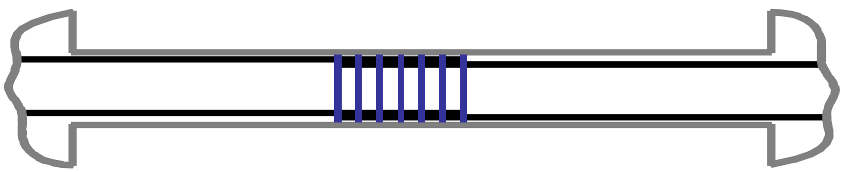


$$\frac{200}{f_y} \leq \rho \leq 0.025$$

At least 2 bars continuous top & bottom

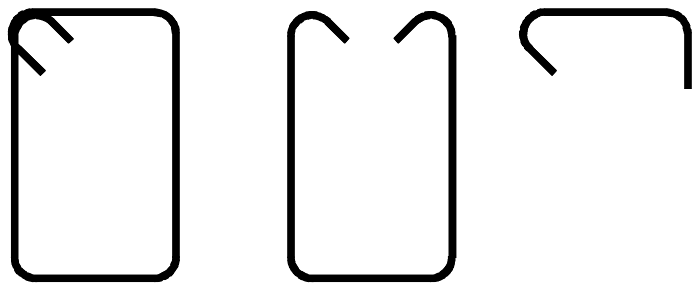


Joint face M_n^+ not less than 50% M_n^-
Min. M_n^+ or M_n^- not less than 25% max. M_n at joint face



Splice away from hinges and enclose within hoops or spirals

ACI 318-05, Overview of Frames:
Beam Transverse Reinforcement

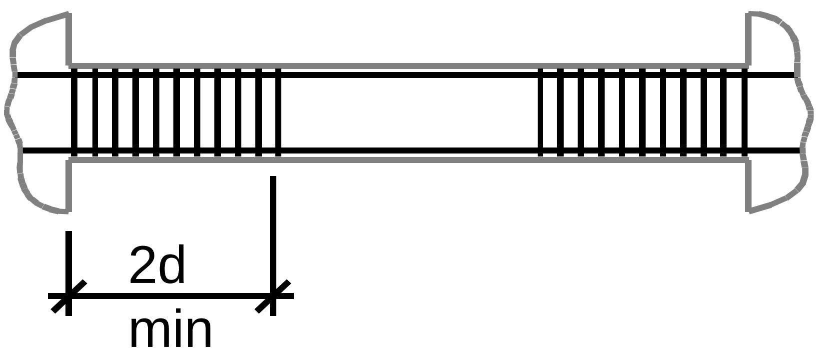


Closed hoops at hinging regions with "seismic" hook

135º hook, $6d_h \geq 3$" extension

Maximum spacing of hoops:

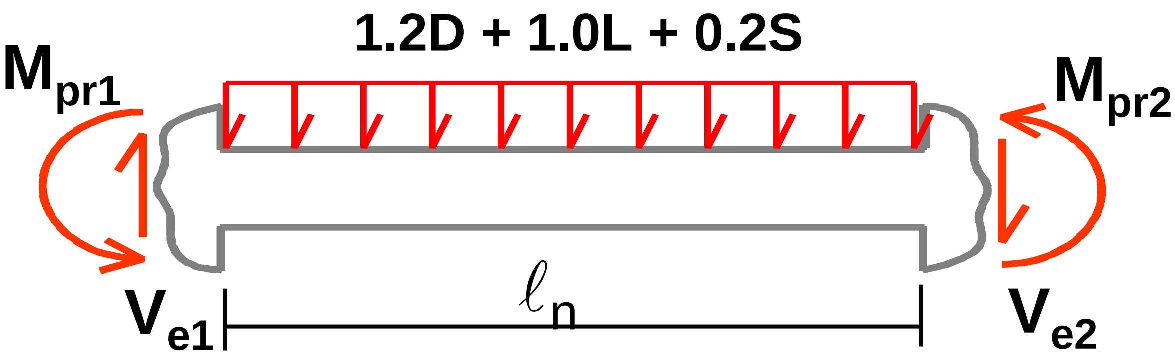
d/4 $8d_b$ $24d_h$ 12"



Longitudinal bars on perimeter tied as if column bars

Stirrups elsewhere, $s \leq d/2$

ACI 318-05, Overview of Frames:
Beam Shear Strength



$$M_{pr} = M_n \quad \text{with}$$

$$f_s = 1.25 f_y, \quad \phi = 1.0$$

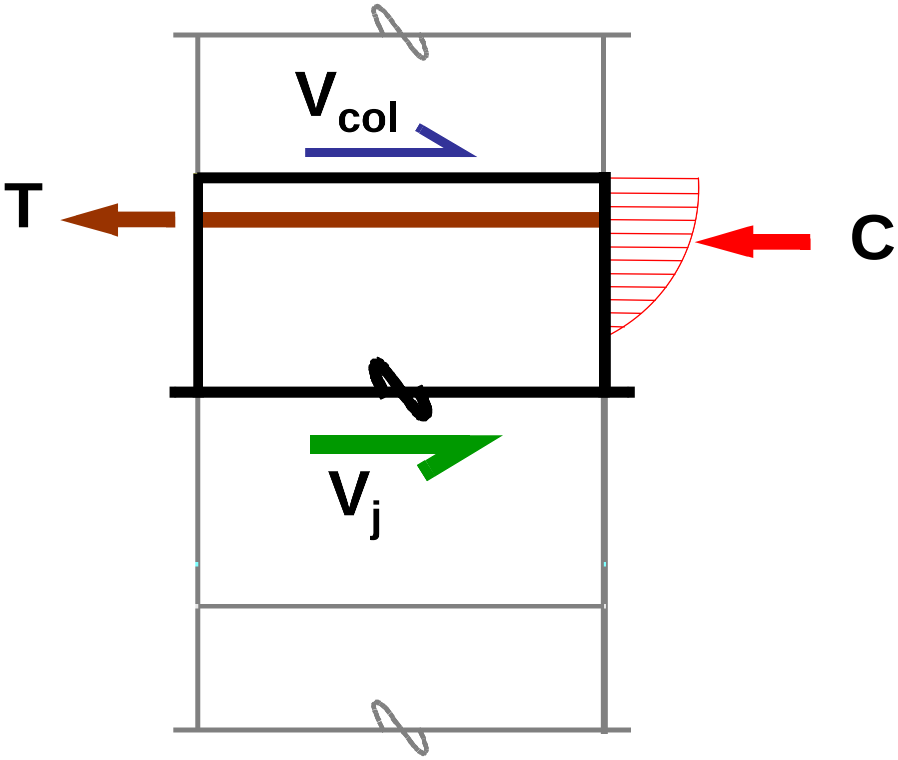
$$V_e = \frac{M_{pr1} + M_{pr2}}{\ell_n} \pm \frac{w_u \ell_n}{2} \geq V_e \quad \text{by analysis}$$

If earthquake-induced shear force $> \frac{1}{2} V_e$

and $P_u < \frac{A_g f_c'}{20}$

then $V_c = 0$

ACI 318-05, Overview of Frames:
Beam-Column Joint



$$V_j = T + C - V_{col}$$

$$T = 1.25 f_y A_{s,top}$$

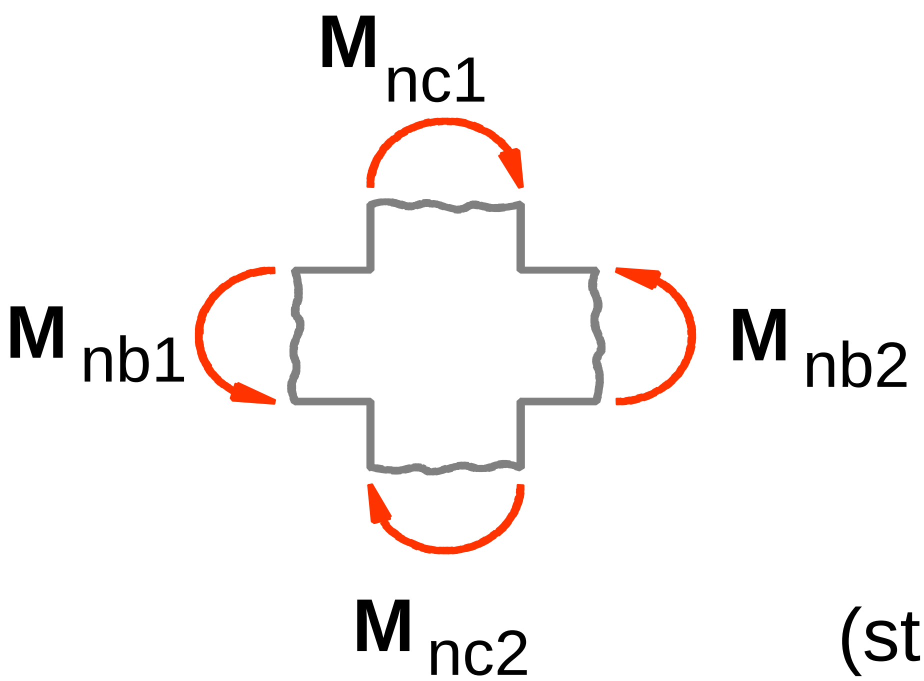
$$C = 1.25 f_y A_{s,bottom}$$

ACI 318-05, Overview of Frames:
Beam-column Joint

$$V_n = \begin{Bmatrix} 20 \\ 15 \\ 12 \end{Bmatrix} \sqrt{f'_c} A_j$$

- V_n controls size of columns
- Coefficient depends on joint confinement
- To reduce shear demand, increase beam depth
- Keep column stronger than beam

ACI 318-05: Overview of Frames:
Column Longitudinal Reinforcement



$$0.01 \leq \rho \leq 0.06$$

$$\sum M_{nc} \geq 1.2 \sum M_{nb}$$

At joints
(strong column-weak beam)

M_{nc} based on factored axial force, consistent with direction of lateral forces

ACI 318-05, Overview of Frames:
Column Transverse Reinforcement at Potential Hinging Region

Spirals

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

and

$$\rho_s \geq 0.12\frac{f'_c}{f_{yt}}$$

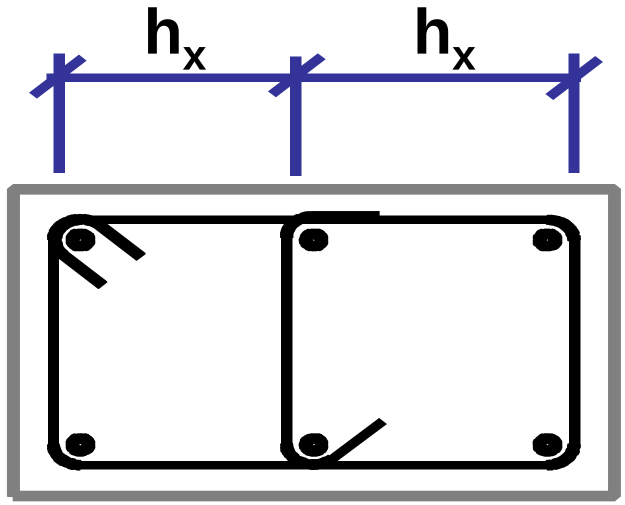
Hoops

$$A_{sh} \geq 0.3\left(sb_c \frac{f'_c}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1\right)$$

and

$$A_{sh} \geq 0.09 sb_c \frac{f'_c}{f_{yt}}$$

ACI 318-05, Overview of Frames:
Column Transverse Reinforcement at Potential Hinging Region



$$s_o = 4 + \left(\frac{14 - h_x}{3} \right)$$

Spacing shall not exceed the smallest of:

b/4 or 6 d_b or s_o (4” to 6”)

Distance between legs of hoops or crossties, $h_x \leq 14$”

ACI 318-05, Overview of Frames: Potential Hinge Region

- For columns supporting stiff members such as walls, hoops are required over full height of column if

$$P_e > \frac{f'_c A_g}{10}$$

- For shear strength- same rules as beams (concrete shear strength is neglected if axial load is low and earthquake shear is high)
- Lap splices are not allowed in potential plastic hinge regions

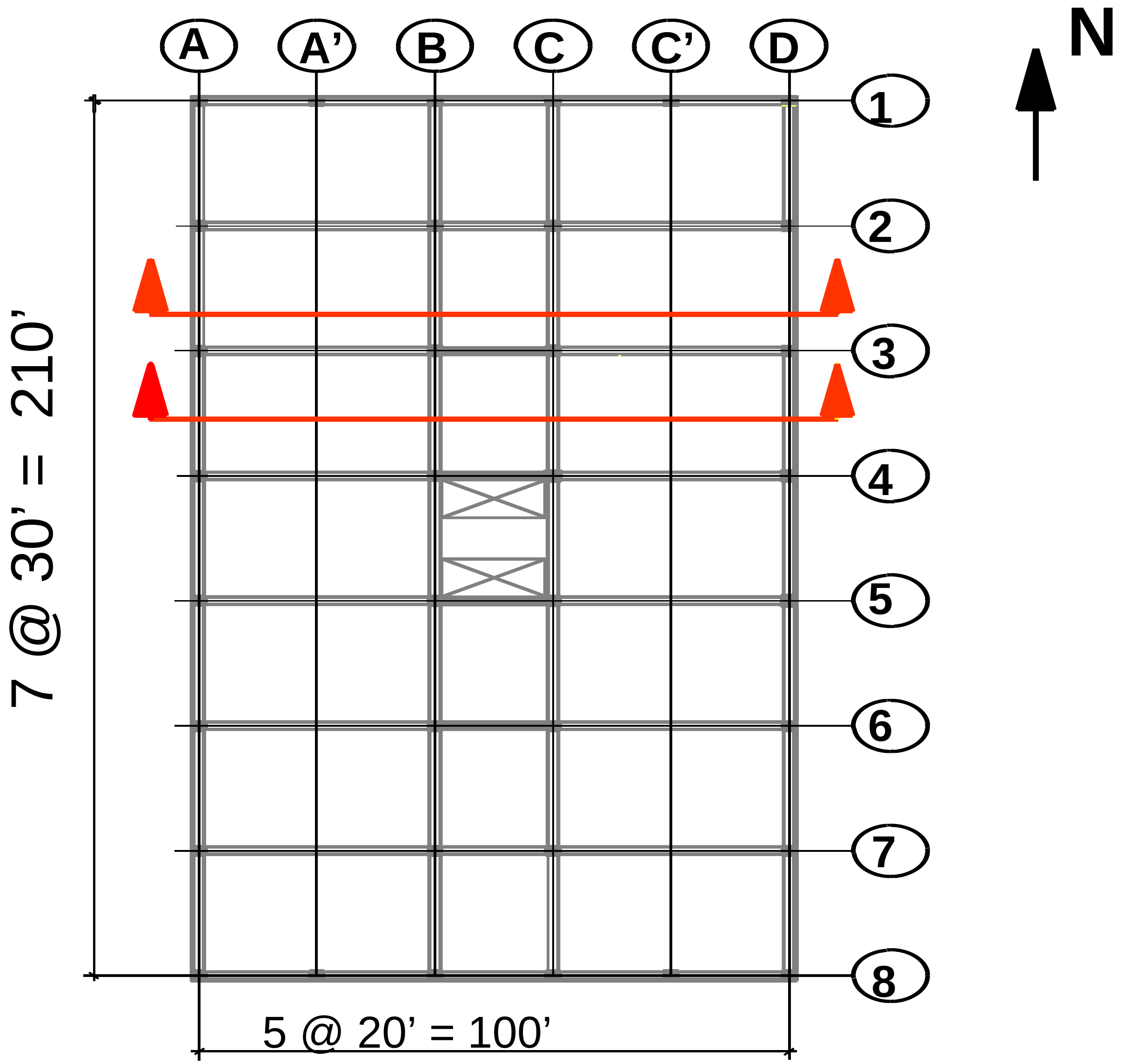
Splice in Hinge Region

Terminating bars

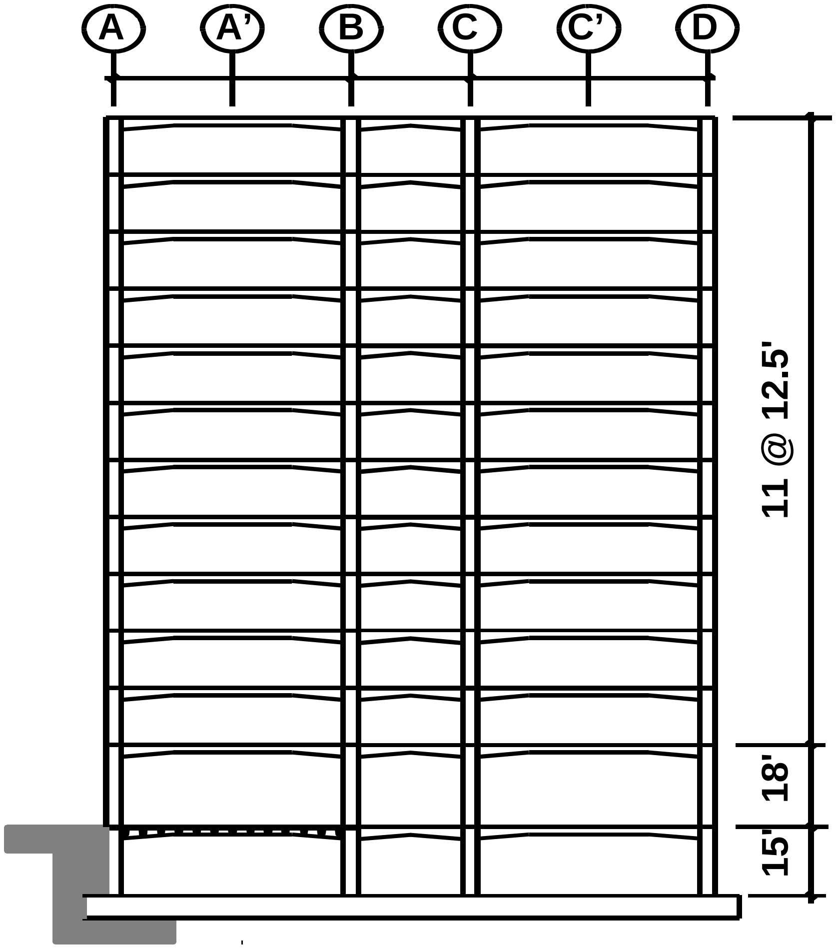
ACI 318-05, Overview of Frames:
Potential Hinge Region

$$\ell_o \geq \begin{cases} d \\ \dfrac{clear\ height}{6} \\ 18'' \end{cases}$$

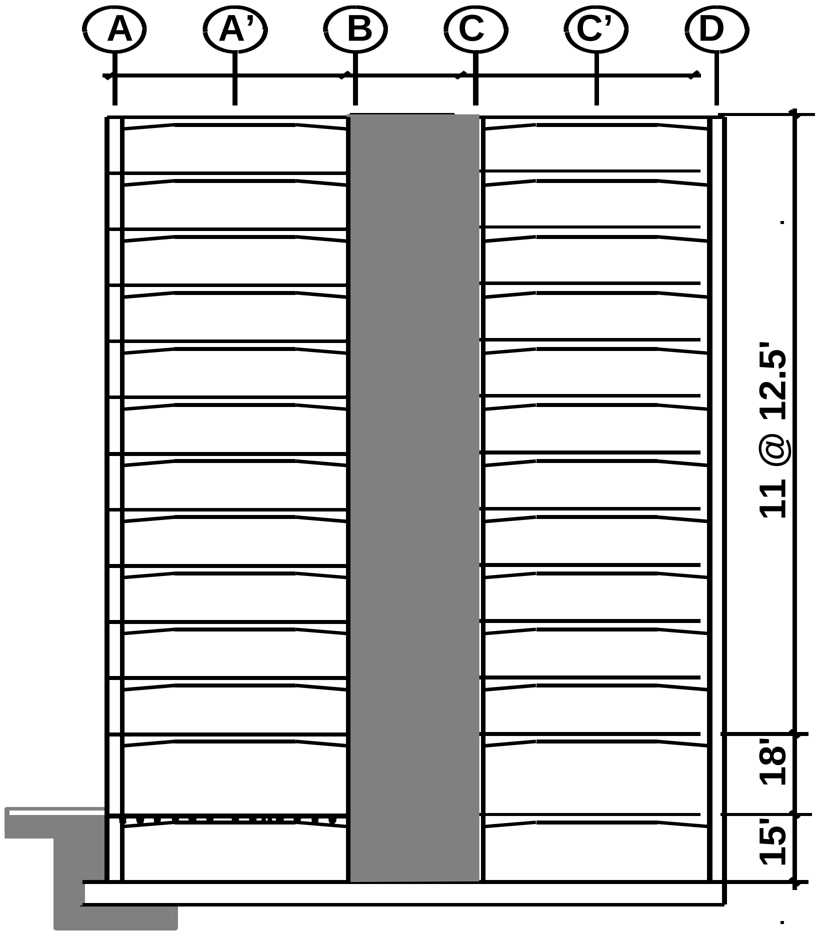
Moment Frame Example



Frame Elevations

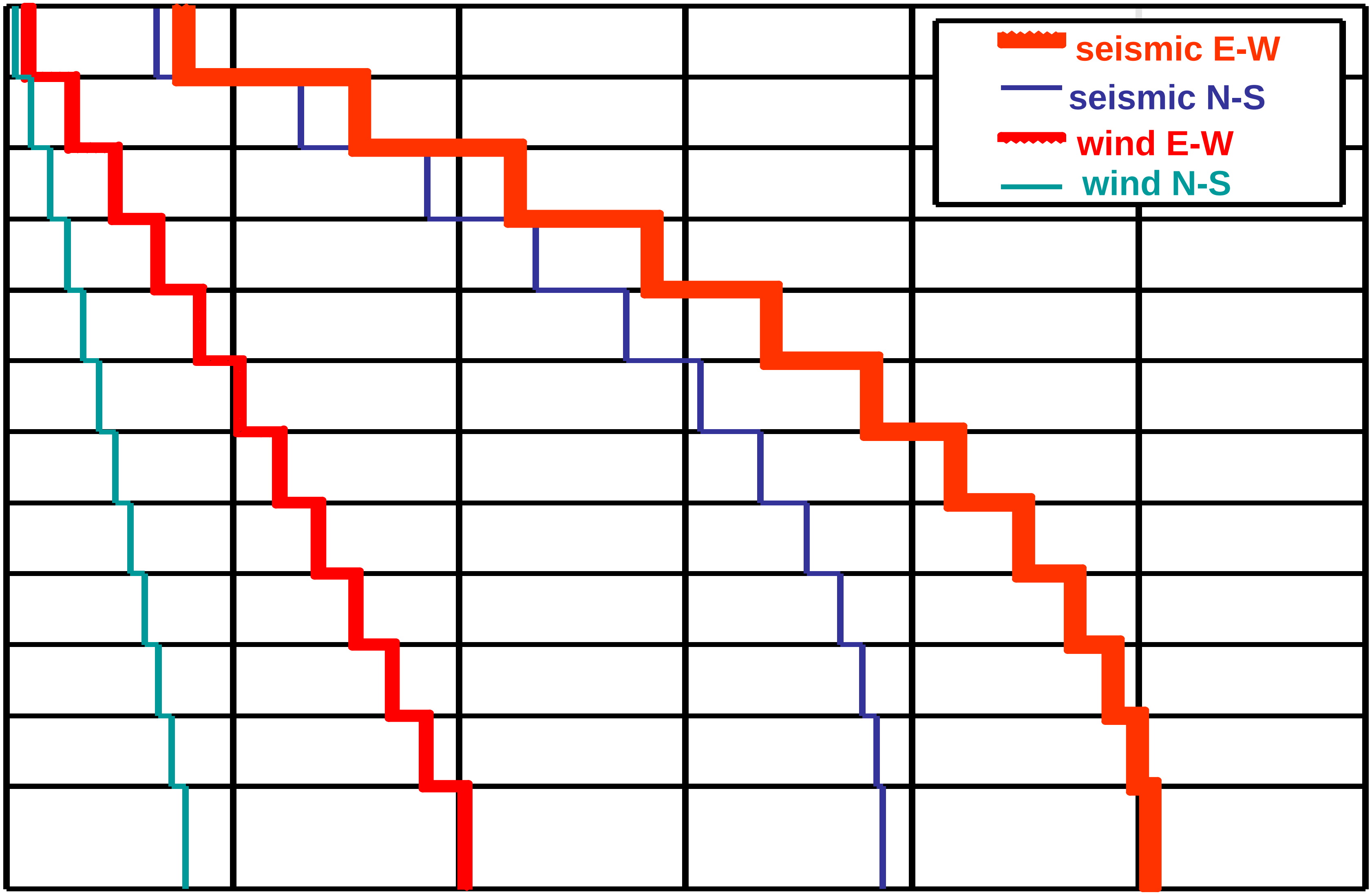


Column Lines 2 and 7

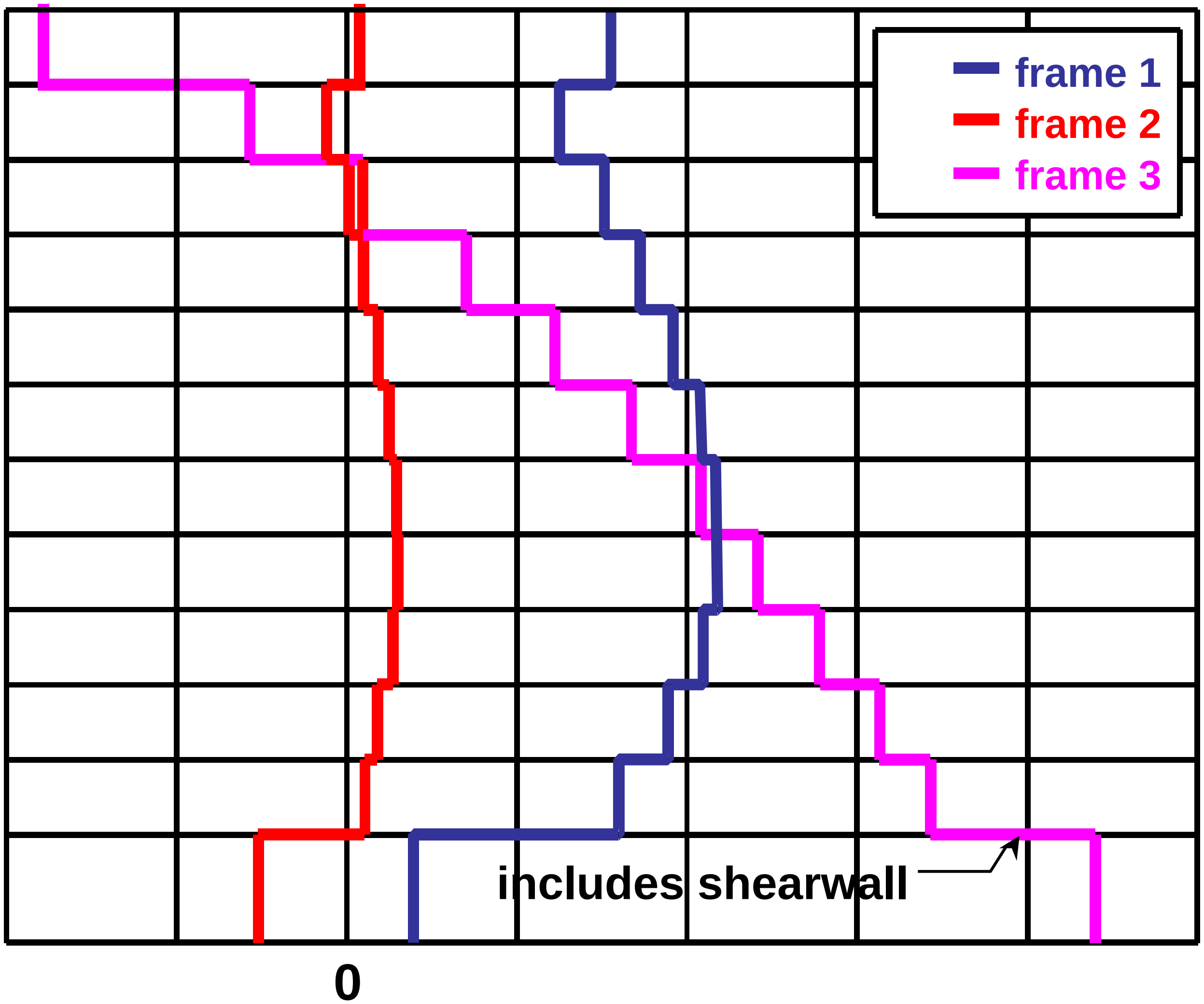


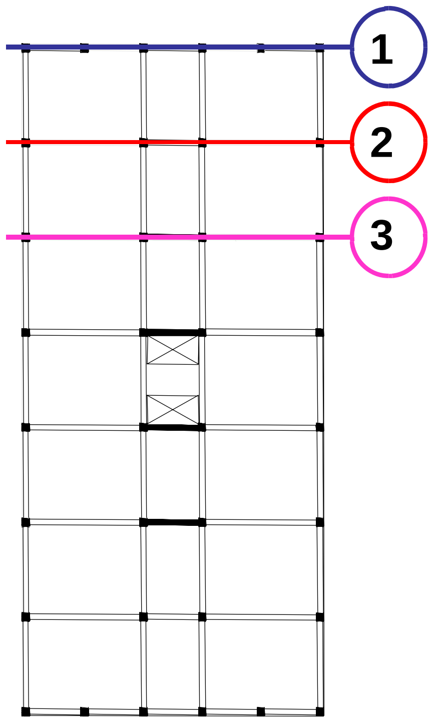
Column Lines 3 to 6

Story Shears: Seismic vs Wind

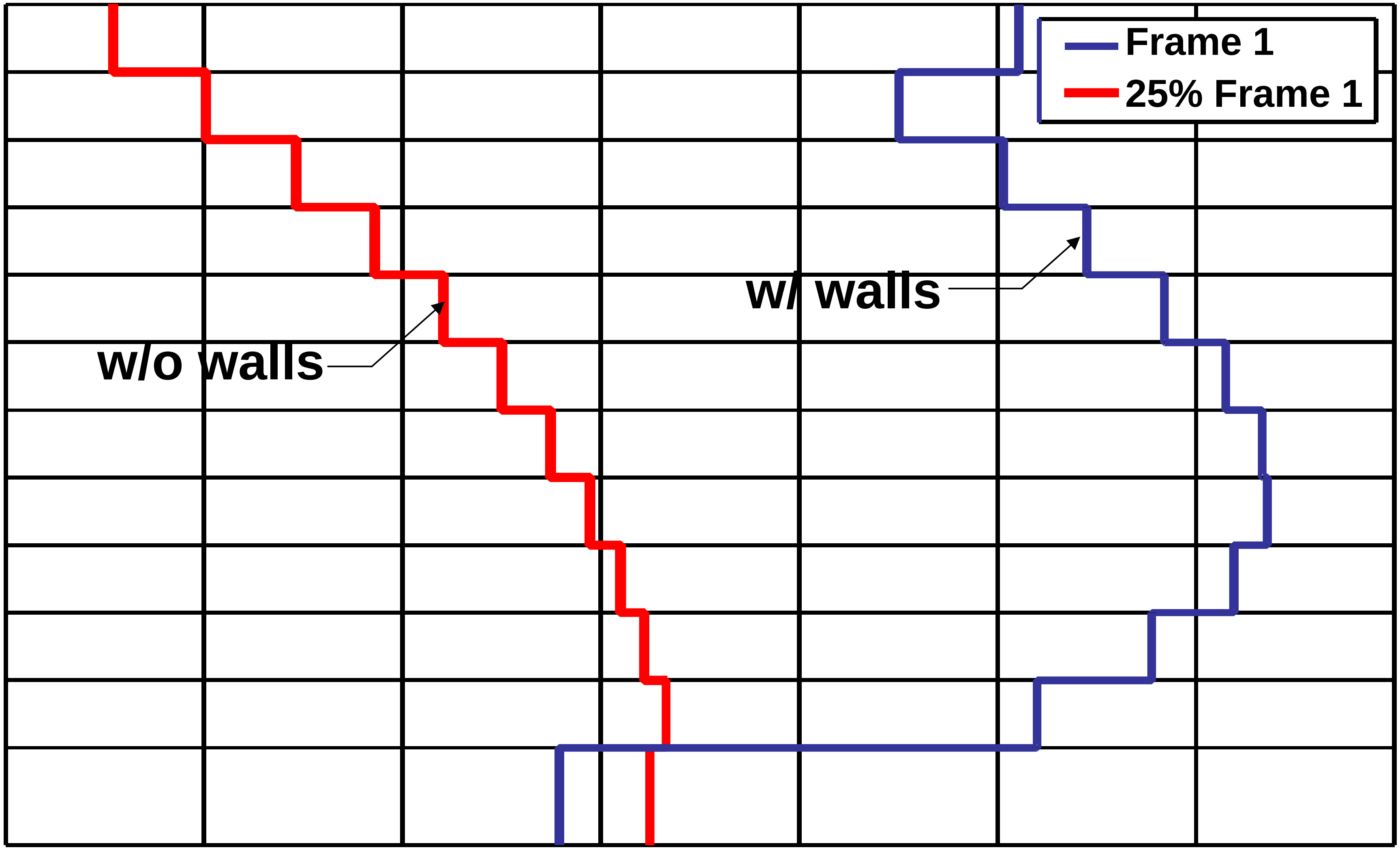


Story Shears: E-W Loading

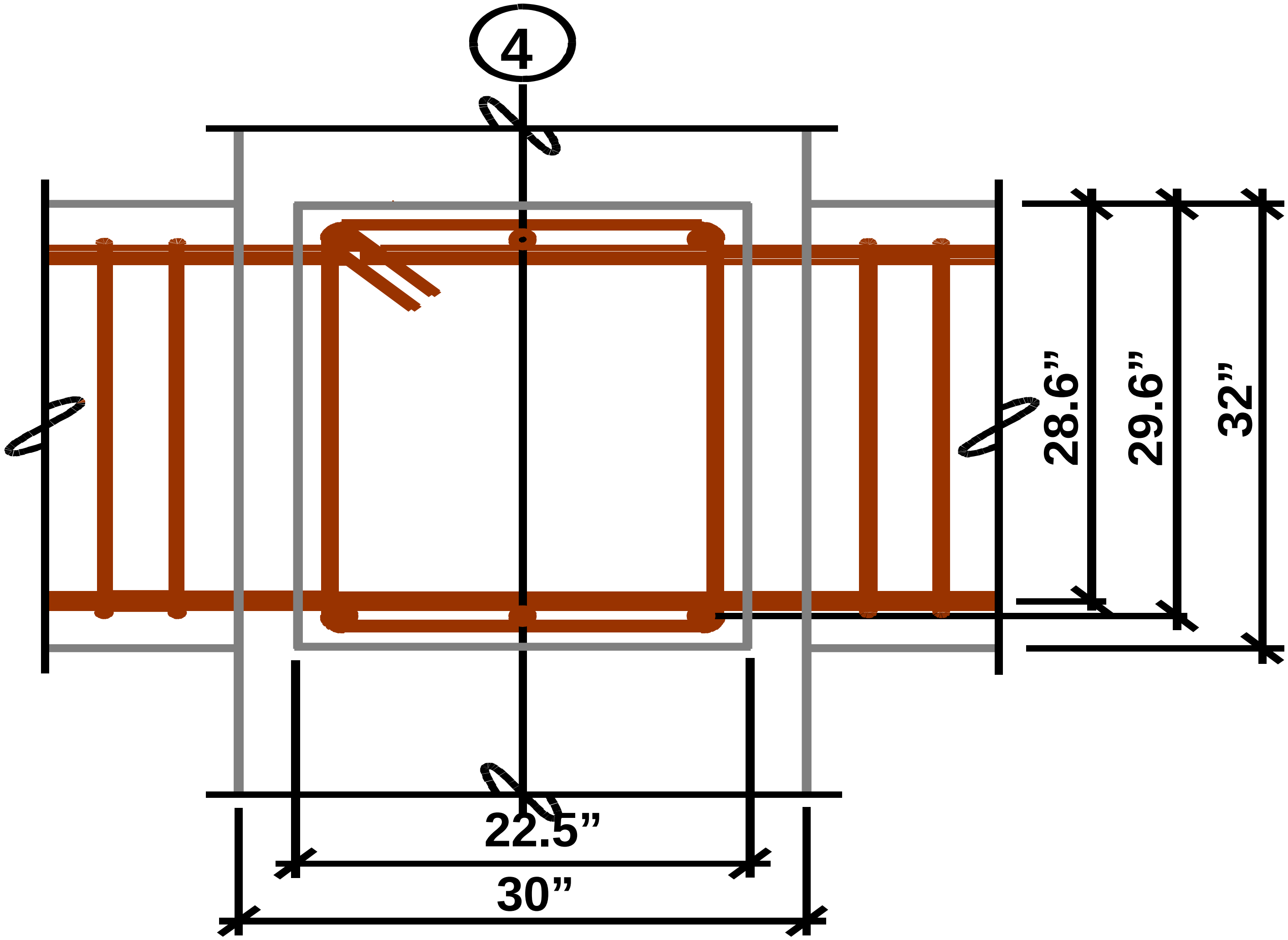




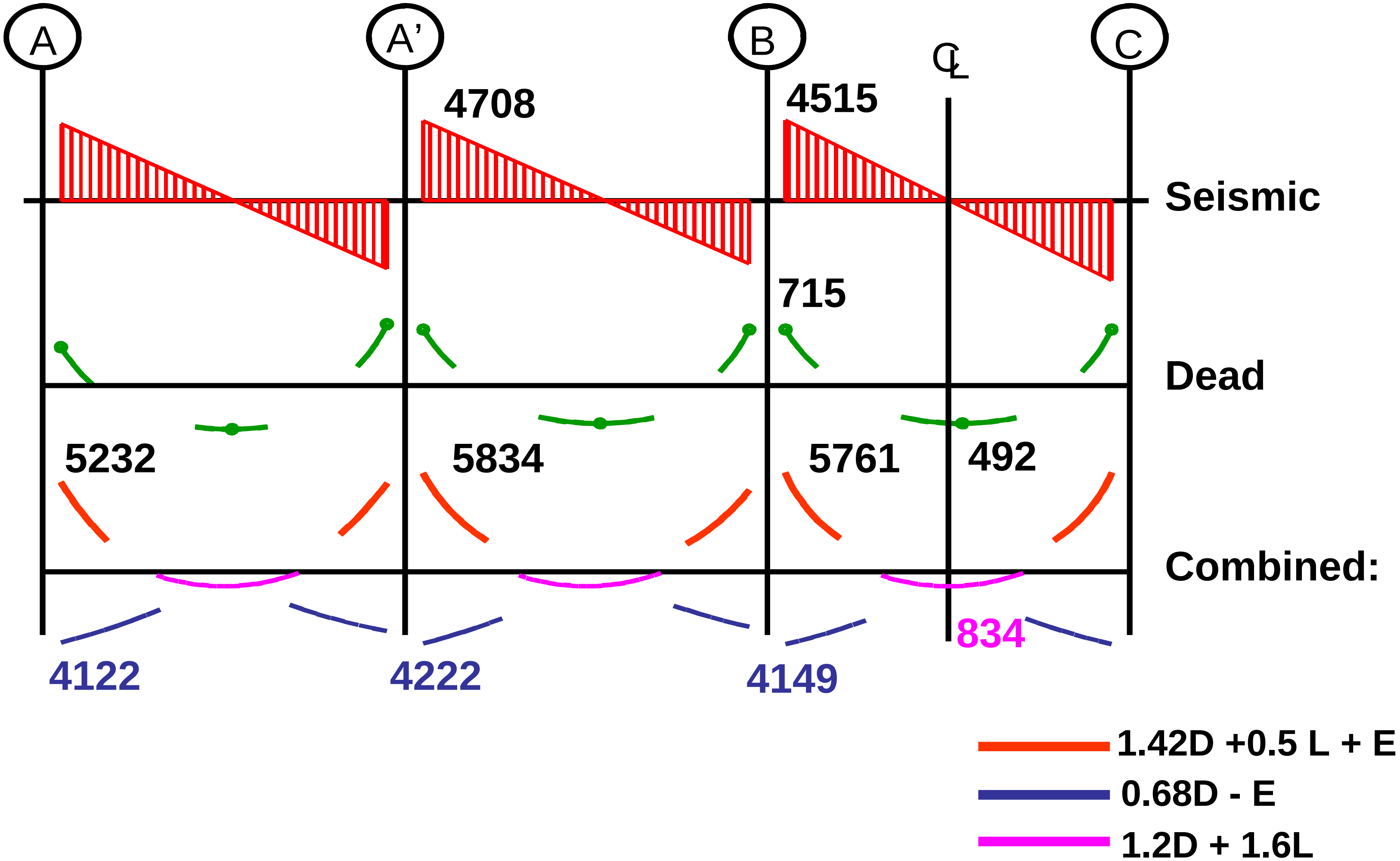
Story Shears: 25% rule



Layout of Reinforcement



Bending Moment Envelopes: Frame 1 Beams



Beam Reinforcement: Longitudinal

Max negative M_u = 5834 in-kips

b = 22.5" d = 29.6" f'_c = 4 ksi f_y = 60 ksi

$$A_{s\,req'd} = \frac{M_u/\phi}{f_y(0.875d)} = \frac{5834/0.9}{60 \cdot 0.875 \cdot 29.6} = 4.17 in^2$$

Choose: 2 #9 and 3 #8 A_s = 4.37 in^2

ρ = 0.0066 < 0.025   <u>OK</u>

ϕM_n = 6580 in-kips   <u>OK</u>

Beam Reinforcement: Longitudinal (continued)

Positive M_u at face of column = 4222 in-kips (greater than ½(5834) = 2917)

b for negative moment is the sum of the beam width (22.5 in.) plus 1/12 the span length (20 ft x 12 in./ft)/12, b = 42.5 in.

$$A_{s\,req'd} = \frac{M_u/\phi}{f_y(0.9d)} = \frac{4222/0.9}{60 \cdot 0.9 \cdot 29.6} = 2.94 in^2$$

Beam Reinforcement: Longitudinal (continued)

Choose 2 #7 and 3 #8 $A_s = 3.57$ in^2

$\phi M_n = 5564$ in-kips   <u>OK</u>

Run 3 #8s continuous top and bottom

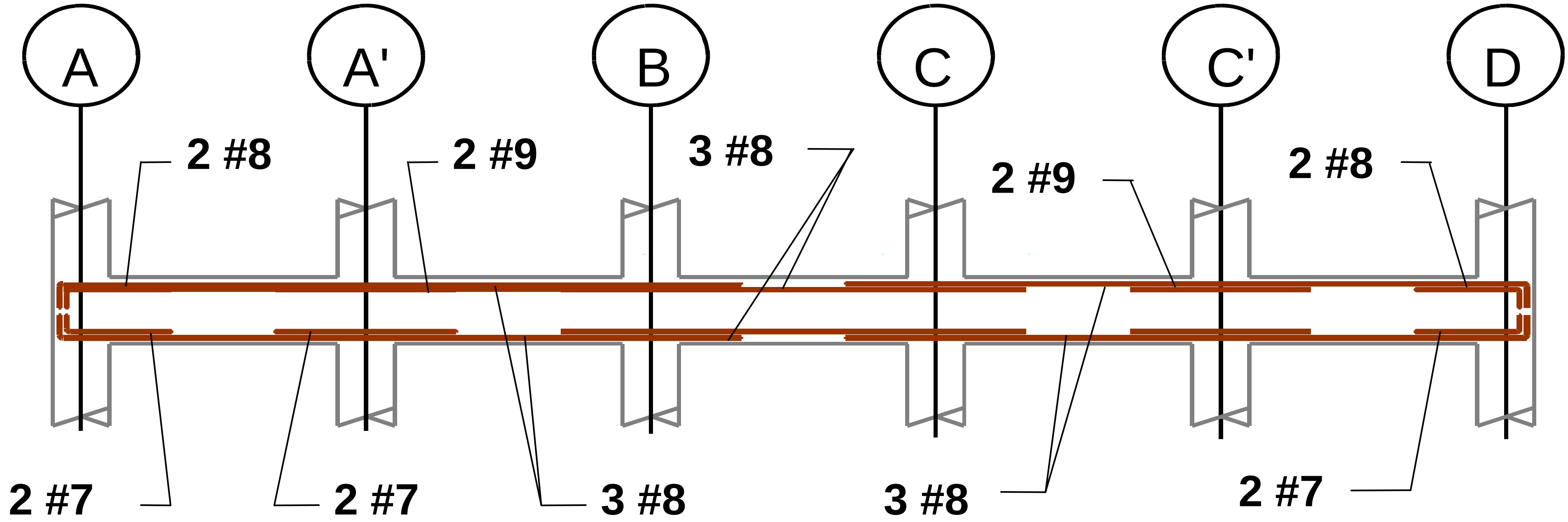
$\phi M_n = 3669$ in-kips

This moment is greater than:

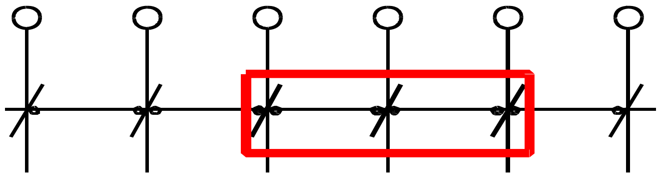
25% of max negative $M_n = 1459$ in-kips

Max required $M_u = 834$ in-kips

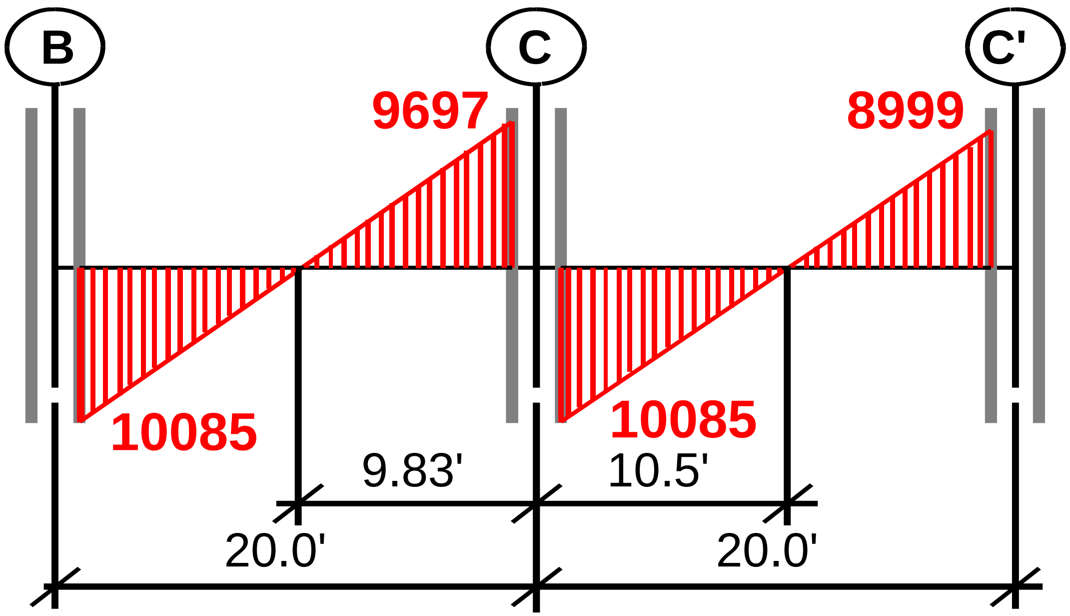
Beam Reinforcement: Preliminary Layout



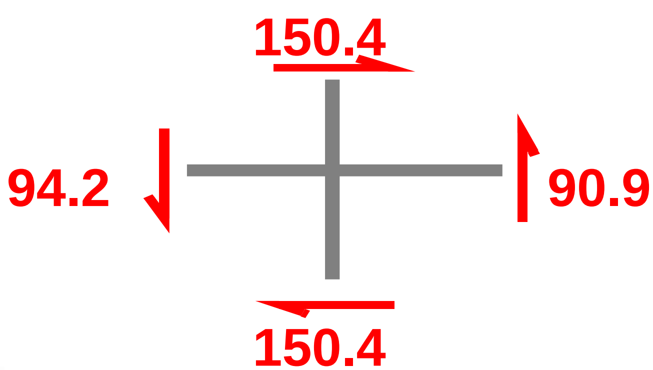
Moments for Computing Shear



Hinging mechanism

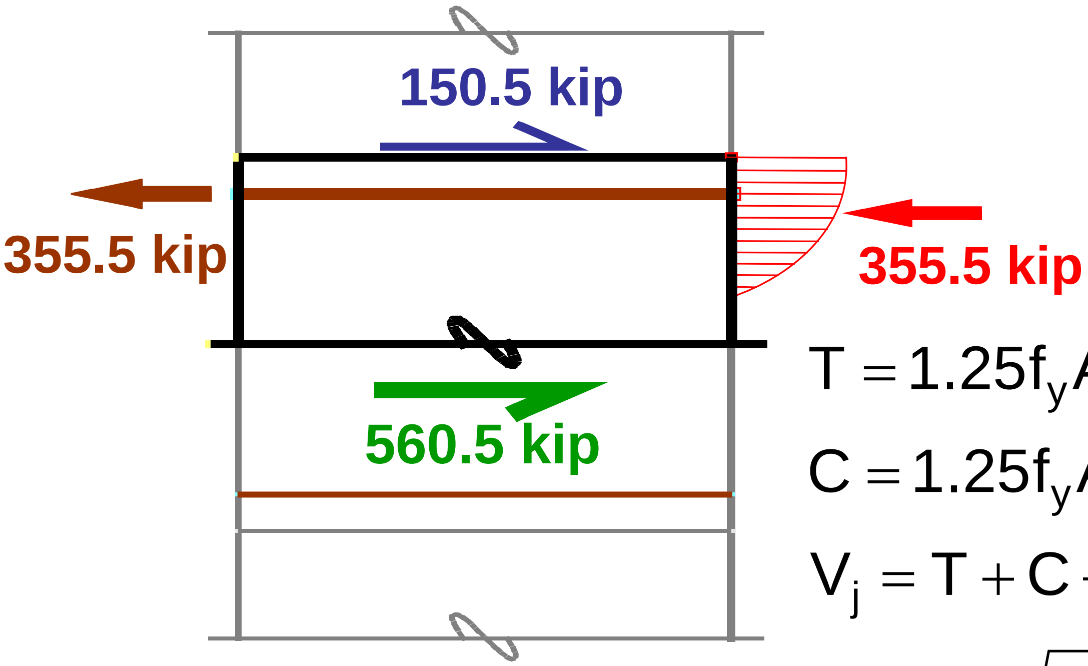


Plastic moments (in-kips)



Girder and column shears (kips)

Joint Shear Force



$$T = 1.25 f_y A_{s,top} = 355.5 \text{ kips}$$

$$C = 1.25 f_y A_{s,bot} = 355.5 \text{ kips}$$

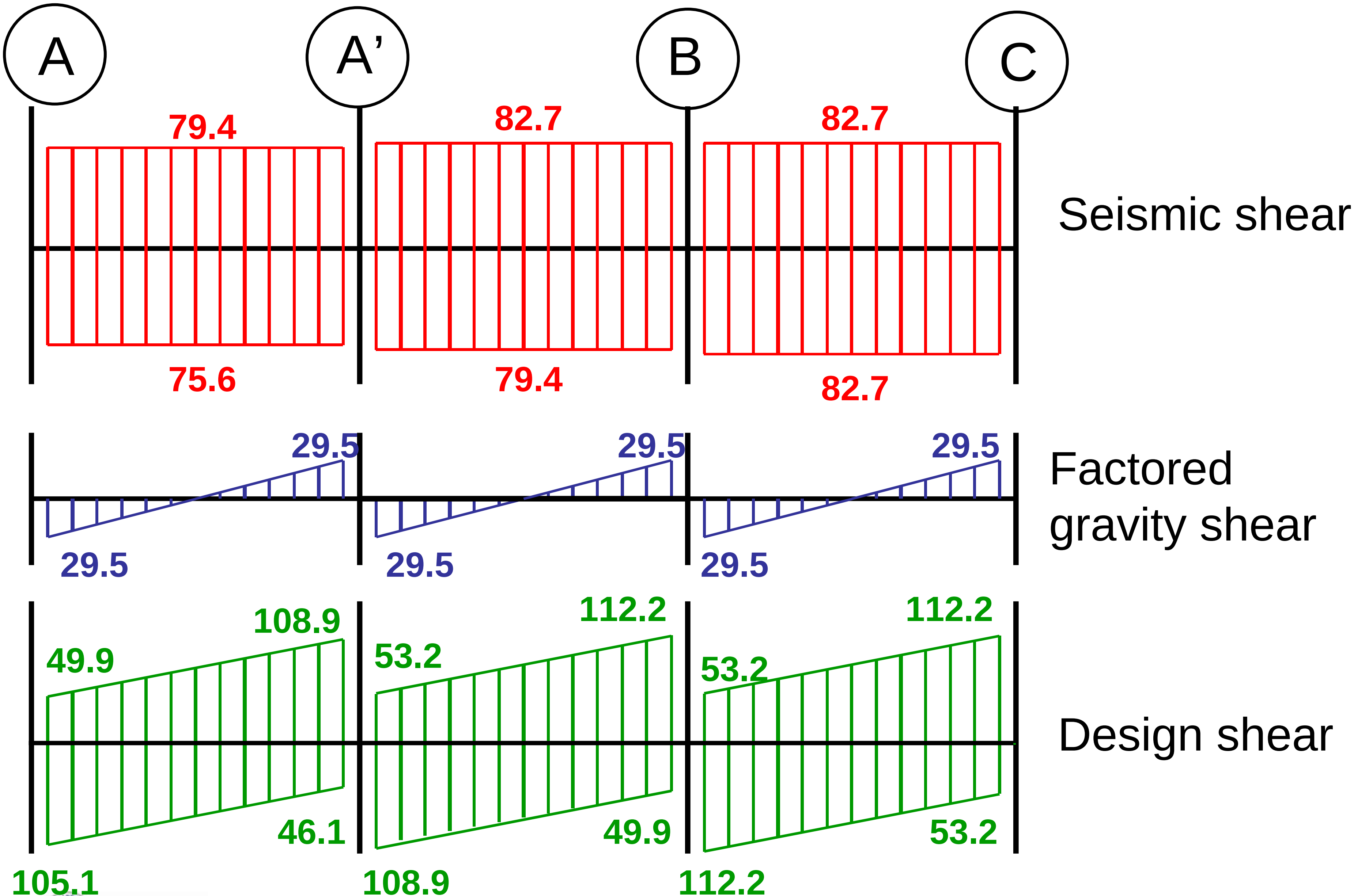
$$V_j = T + C - V_{col} = 560.5 \text{ kips}$$

$$v_j = 15\sqrt{f'_c} = 949 \text{ psi}$$

$$V_n = v_j A_j = 949 \cdot 30 \cdot 30 = 854 \text{ kips}$$

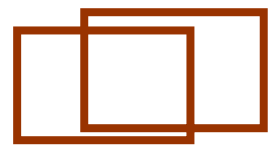
$$\phi V_n = 0.85 \cdot 854 = 726 \text{ kips} > 560.5 \text{ kips}$$

Beam Shear Force



Beam Reinforcement: Transverse

$V_{seismic} > 50\%\ V_u$ therefore take $V_c = 0$
82.7 kips = 73%(112.2)

Use 4 legged #3 stirrups 

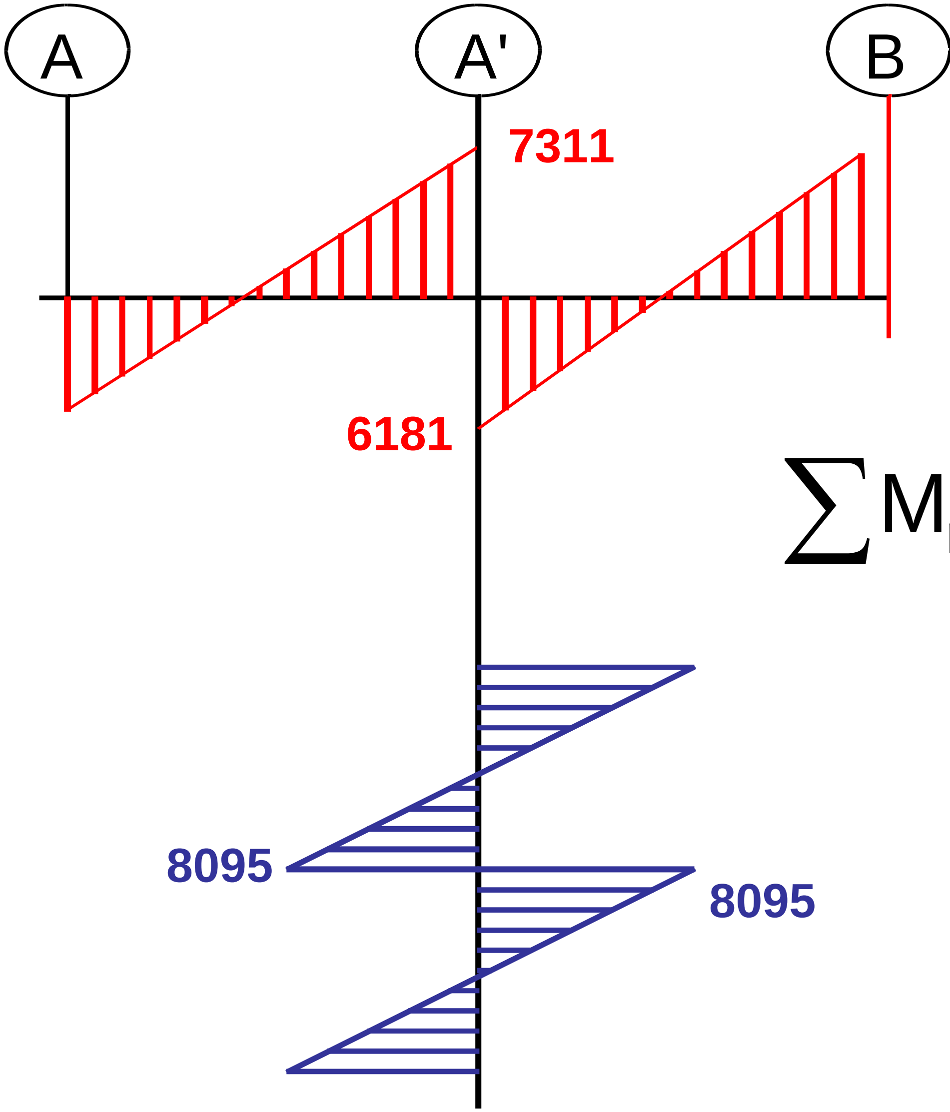
$$V_s = \frac{A_v f_y d}{s}$$

At ends of beam s = 5.5 in.
Near midspan s = 7.0 in.

Beam Reinforcement: Transverse

- **Check maximum spacing of hoops within plastic hinge length (2d)**
  - $d/4 = 7.4$ in.
  - $8d_b = 7.0$ in.
  - $24d_h = 9.0$ in.

Column Design Moments



Girder moments (Level 7)

$$\sum M_{nc} = 1.2(7311 + 6181)$$
$$= 16190 \text{ in} - \text{k}$$

Column moments (Level 7)

Column Design Moments

$$\text{if} \quad P_u > \frac{f'_c A_g}{10}$$

$$\sum M_{nc} > 1.2 \sum M_{nb}$$

Distribute relative to stiffness of columns above and below:

M_{nc} = 8095 in-kips (above)
M_{nc} = 8095 in-kips (below)

Design Strengths

Design Aspect	Strength Used
Beam rebar cutoffs	Design strength
Beam shear reinforcement	Maximum probable strength
Beam-column joint strength	Maximum probable strength
Column flexural strength	1.2 times nominal strength
Column shear strength	Maximum probable strength

Column Transverse Reinforcement

$$A_{sh} = 0.3\left(sb_c \frac{f'_c}{f_{yt}}\right)\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right]$$

and

$$A_{sh} = 0.09sb_c \frac{f'_c}{f_{yt}}$$

A_g = gross area of column
A_{ch} = area confined within the hoops
b_c = transverse dimension of column core measured center to center of outer legs

Second equation typically governs for larger columns

Column Transverse Reinforcement

Maximum spacing is smallest of:

- One quarter of minimum member dimension
- Six times the diameter of the longitudinal bars
- s_o calculated as follows:

$$s_o = 4 + \frac{14 - h_x}{3}$$

h_x = maximum horizontal center to center spacing of cross-ties or hoop legs on all faces of the column, not allowed to be greater the 14 in.

Column Transverse Reinforcement

For max s = 4 in.

$$A_{sh} = 0.3\left(sb_c \frac{f'_c}{f_{yt}}\right)\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right] = 0.3\left(4 \cdot 26.5 \cdot \frac{4}{60}\right)\left(\frac{900}{702} - 1\right)$$

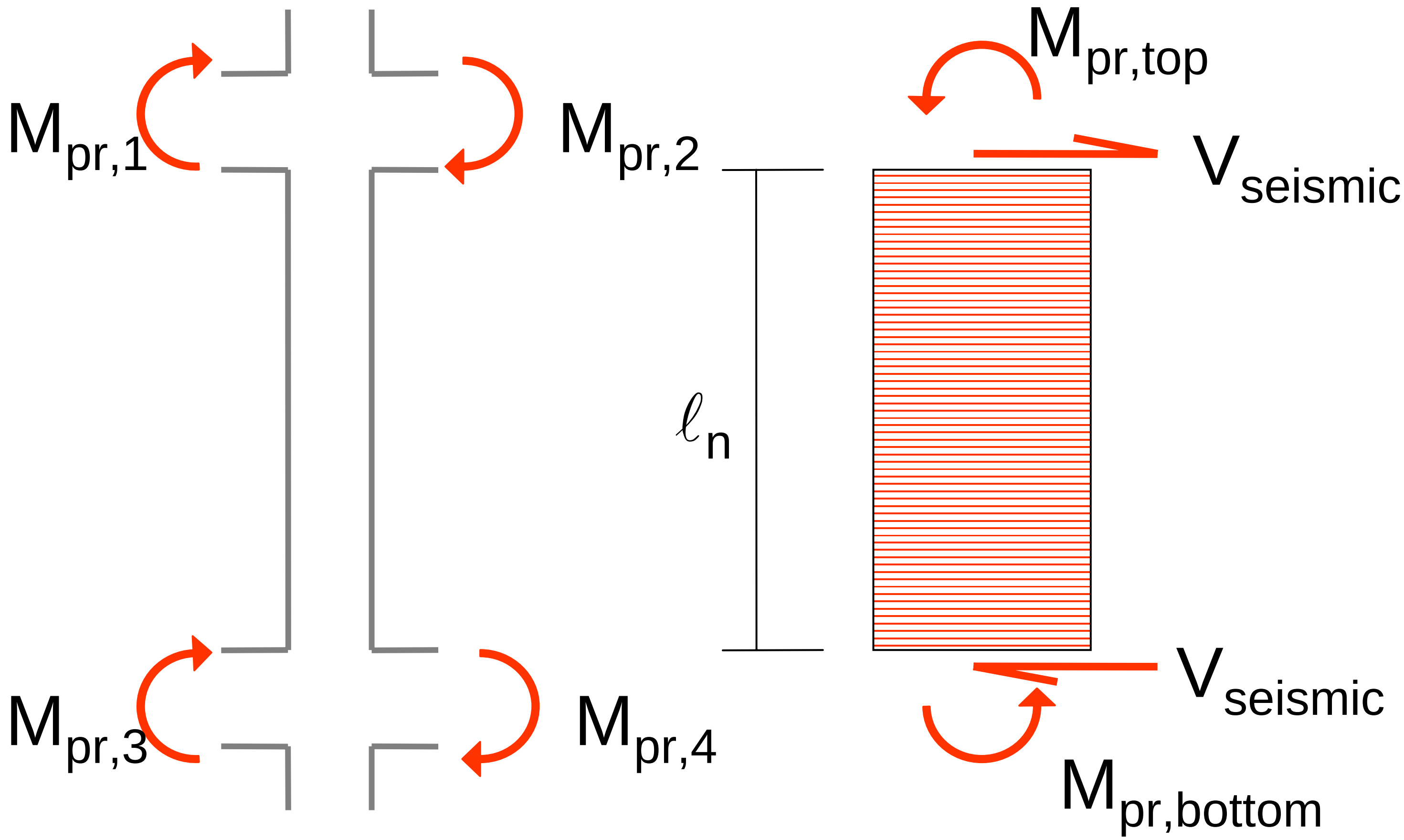
$$A_{sh} = 0.60 \text{ in}^2$$

and

$$A_{sh} = 0.09 sb_c \frac{f'_c}{f_{yt}} = 0.09 \cdot 4 \cdot 26.5 \cdot \frac{4}{60} = 0.64 \text{ in}^2$$

Use 4 legs of #4 bar – A_{sh} = 0.80 in^2

Determine Seismic Shear



Column Transverse Reinforcement Shear Demand from M_{pr} of Beams

$M_{pr,1}$ = 9000 in-k (2 #9 and 3 #8)
$M_{pr,2}$ = 7460 in-k (2 #7 and 3 #8)

Assume moments are distributed equally above and below joint

$$V_{seismic} = \frac{8230 \cdot 2}{(12.5 \cdot 12) - 32} = 139 \text{ kips}$$

Note $V_{seismic}$~100%V_u

$$V_c = 0, if\ P_{\min} < \frac{f'_c A_g}{20} = 180\ kips$$

For 30 in. square column
P_{min} = 266 kips OK

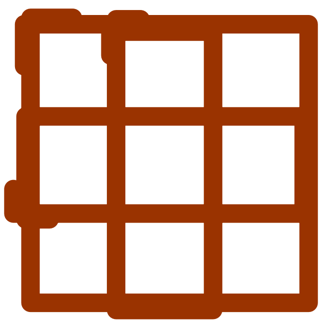
Column Transverse Reinforcement Shear Demand from M_{pr} of Beams

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd = 0.75 \cdot 2 \cdot 0.85\sqrt{4000} \cdot 30 \cdot 27.5 = 66.5 \text{ kips}$$

$$\phi V_{s,required} = 139 - 66.5 = 72.5 \text{ kips}$$

$$\phi V_{s,provided} = \phi\frac{A_v f_y d}{s} = 0.75\frac{4 \cdot 0.2 \cdot 60 \cdot 29.6}{4} = 266.4 \text{ kips}$$

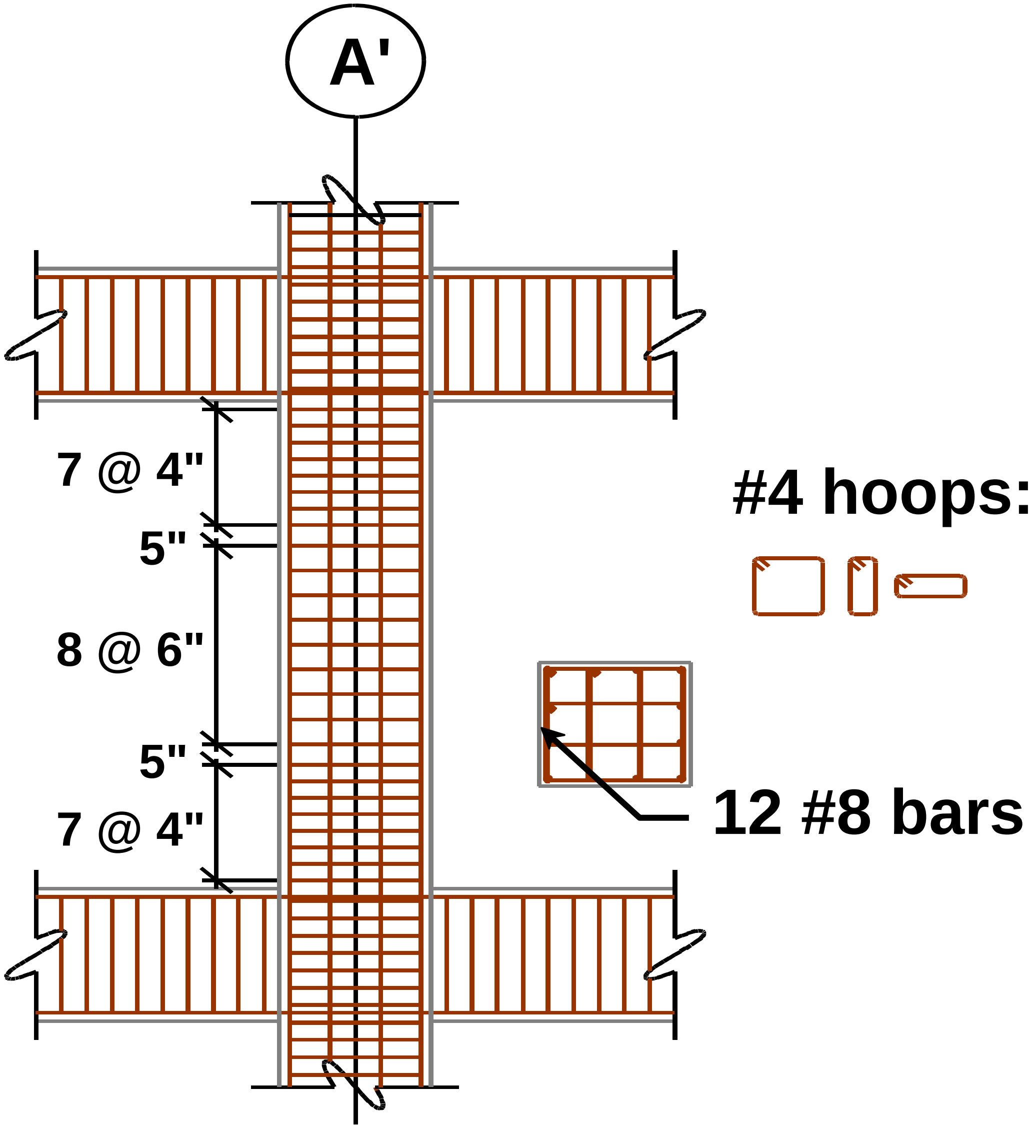
Hoops



4 legs #4

s = 4"

Column Reinforcement



Levels of Seismic Detailing for Frames

Issue	Ordinary	Intermediate	Special
Hinge development and confinement		minor	full
Bar buckling		lesser	full
Member shear		lesser	full
Joint shear	minor	minor	full
Strong column			full
Rebar development	lesser	lesser	full
Load reversal	minor	lesser	full

NEHRP Recommended Provisions Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**
- **Reference standards**
- **Requirements by Seismic Design Category**
- **Moment resisting frames**
- **Shear walls**

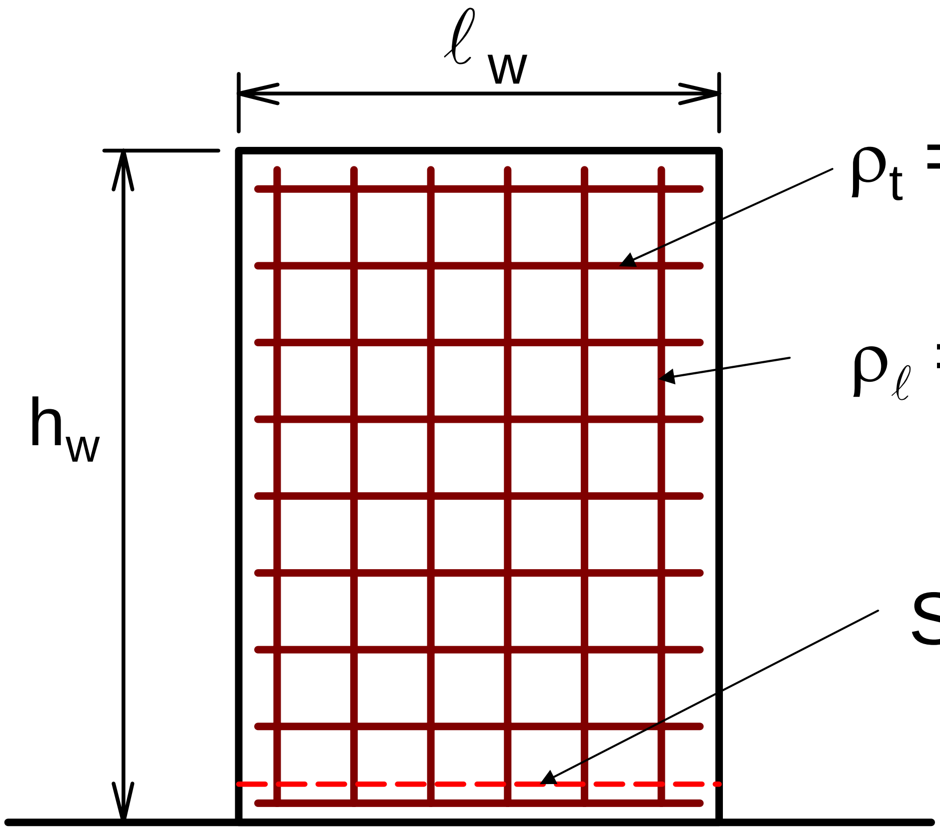
Performance Objectives

- **Resist axial forces, flexure and shear**
- **Boundary members**
  - Where compression strains are large, maintain capacity
- **Development of rebar in panel**
- **Discontinuous walls: supporting columns have full confinement**

Design Philosophy

- **Flexural yielding will occur in predetermined flexural hinging regions**
- **Brittle failure mechanisms will be precluded**
  - Diagonal tension
  - Sliding hinges
  - Local buckling

ACI 318-05, Overview of Walls: General Requirements



ρ_t = parallel to shear plane

ρ_ℓ = perpendicular to shear plane

Shear plane, A_{cv} = web thickness x length of wall

ACI 318-05, Overview of Walls: General Requirements

- ρ_ℓ and ρ_t not less than 0.0025 unless $V_u < A_{cv}\sqrt{f'_c}$

  then as allowed in 14.3
- Spacing not to exceed 18 in.
- Reinforcement contributing to V_n shall be continuous and distributed across the shear plane

ACI 318-05, Overview of Walls: General Requirements

- **Two curtains of reinforcing required if:**

$$V_u > 2A_{cv}\sqrt{f'_c}$$

- **Design shear force determined from lateral load analysis**

ACI 318-05, Overview of Walls: General Requirements

- **Shear strength:**

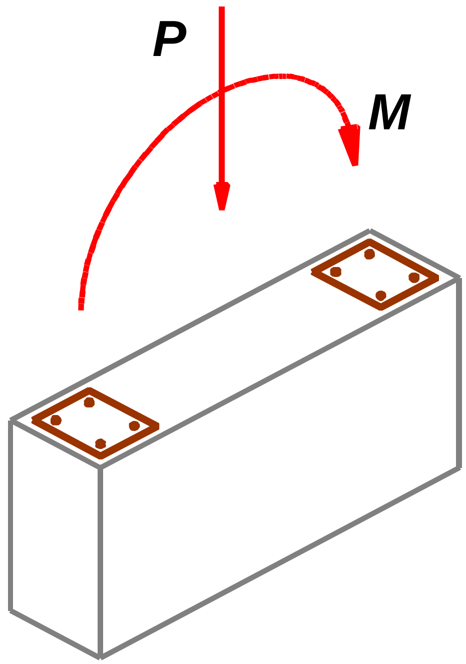
$$V_n = A_{cv}\left(\alpha_c\sqrt{f'_c} + \rho_t f_y\right)$$

α_c = 3.0 for $h_w/\ell_w \le 1.5$
α_c = 2.0 for $h_w/\ell_w \ge 2.0$
Linear interpolation between

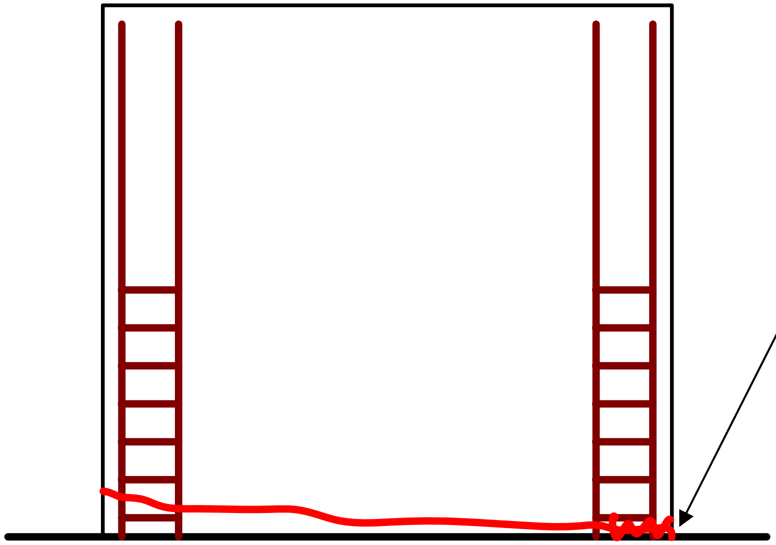
- **Walls must have reinforcement in two orthogonal directions**

ACI 318-05, Overview of Walls: General Requirements

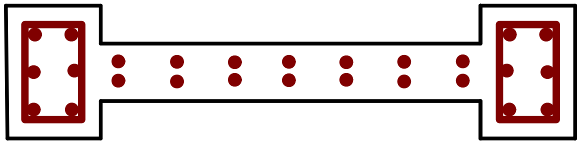
- For axial load and flexure, design like a column to determine axial load – moment interaction



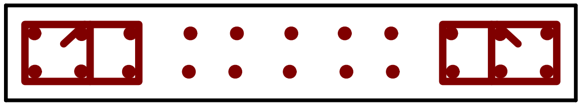
ACI 318-05, Overview of Walls: Boundary Elements



For walls with a high compression demand at the edges – Boundary Elements are required



Widened end with confinement



Extra confinement and/or longitudinal bars at end

ACI 318-05, Overview of Walls: Boundary Elements

- **Boundary elements are required if:**

$$c \geq \frac{\ell_w}{600\left(\delta_u / h_w\right)}$$

δ_u = Design displacement

c = Depth to neutral axis from strain compatibility analysis with loads causing δ_u

ACI 318-05, Overview of Walls: Boundary Elements

- **Where required, boundary elements must extend up the wall from the critical section a distance not less than the larger of:**

$$\ell_w \quad \text{or} \quad M_u/4V_u$$

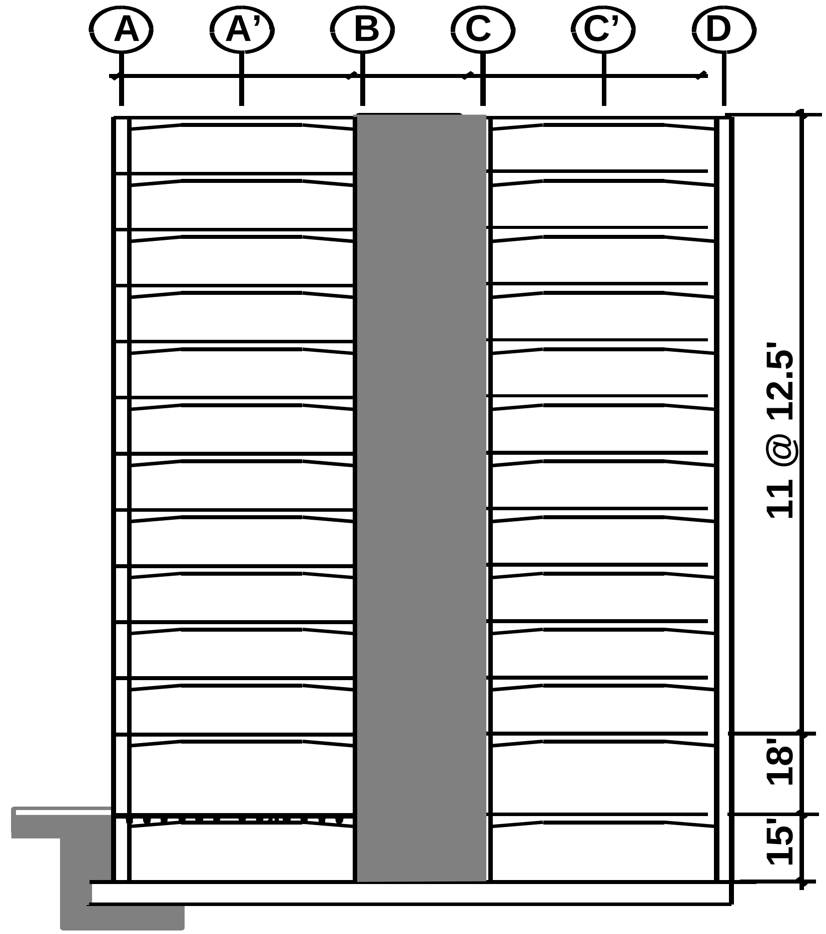
ACI 318-05: Overview of Walls Boundary Elements

- **Boundary elements are required where the maximum extreme fiber compressive stress calculated based on factored load effects, linear elastic concrete behavior and gross section properties, exceeds 0.2 f'_c**
- **Boundary element can be discontinued where the compressive stress is less than 0.15f'_c**

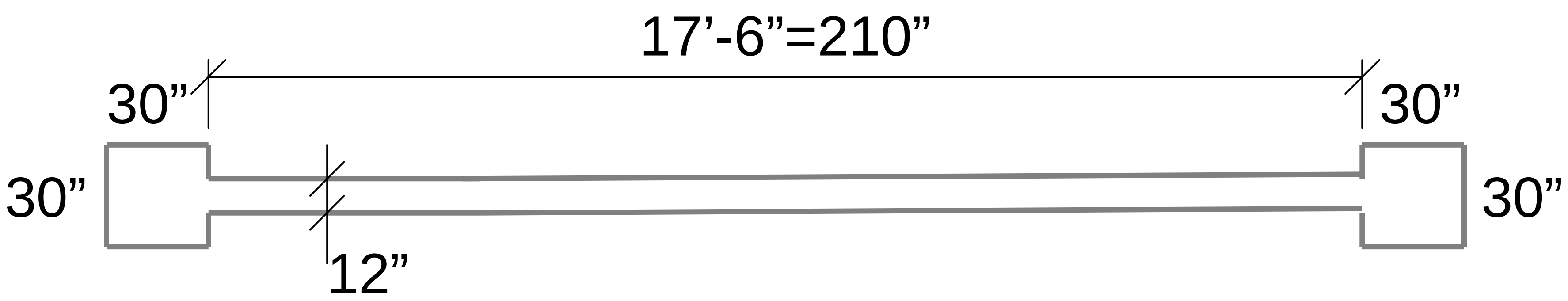
ACI 318-05: Overview of Walls Boundary Elements

- **Boundary elements must extend horizontally not less than the larger of c/2 or c-0.1ℓ_w**
- **In flanged walls, boundary element must include all of the effective flange width and at least 12 in. of the web**
- **Transverse reinforcement must extend into the foundation**

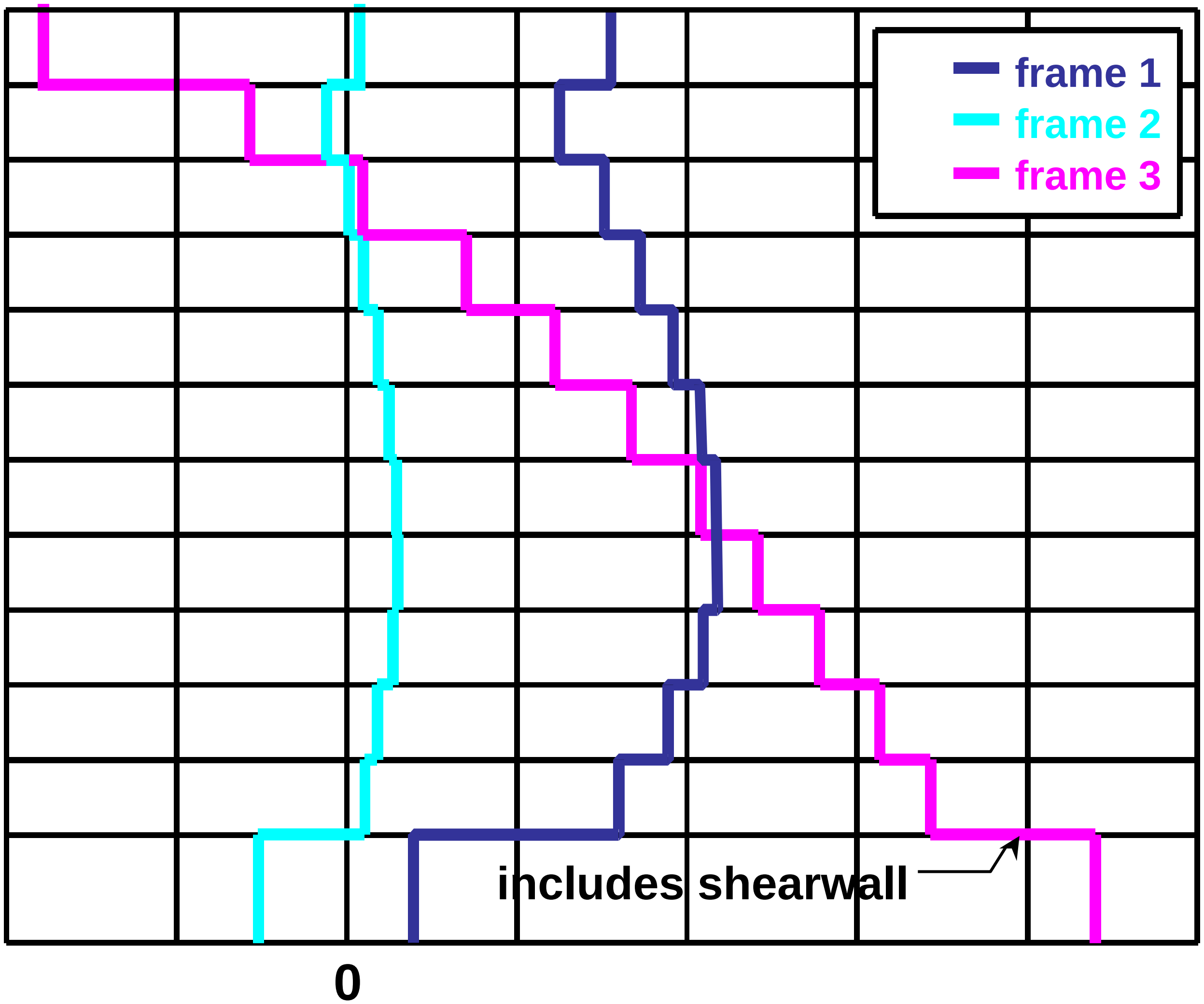
Wall Example

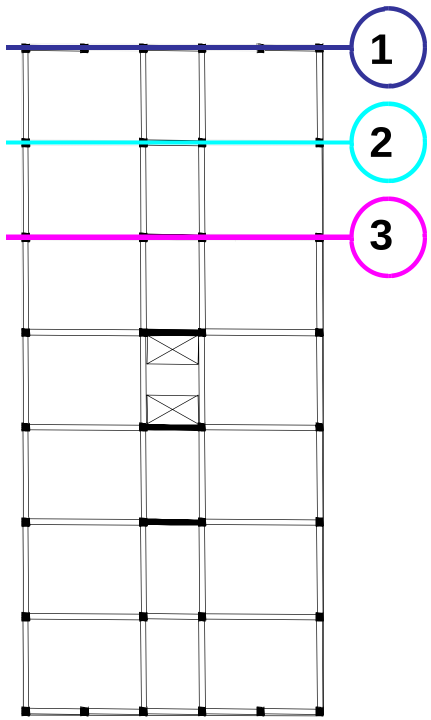


Wall Cross-Section



Story Shears
E-W Loading





Boundary Element Check

Required if: $f_c > 0.2 f_c'$ based on gross concrete section

Axial load and moment are determined based on factored forces, including earthquake effects

At ground P_u = 5550 kip

M_u from analysis is 268,187 in-kip

The wall has the following gross section properties:

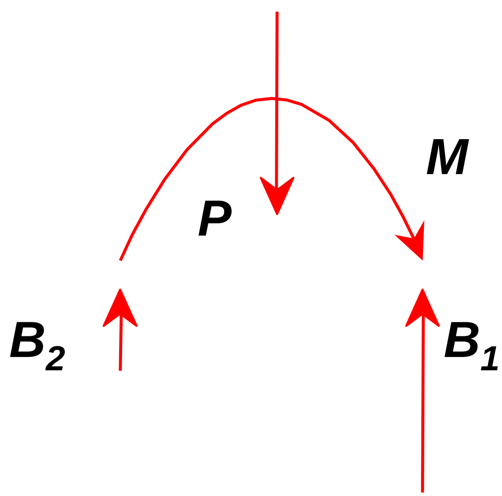
$A = 4320 \text{ in}^2$ $S = 261{,}600 \text{ in}^3$

f_c = 2.3 ksi = 38% of f'_c = 6 ksi

$\therefore$ **Need boundary element**

Boundary Element Design

Determine preliminary reinforcing ratio in boundary elements by assuming only boundary elements take compression



M = 268,187 in-k

P = 5550 k

$$B_1 = \frac{P}{2} + \frac{M}{240} = 3892 \text{ kip}$$

$$B_2 = \frac{P}{2} - \frac{M}{240} = 1658 \text{ kip}$$

Need $\quad 0.8P_o = 0.8(0.7)A_g\left[0.85 f_c'(1-\rho) + \rho f_y\right] > 3892 \text{ kip}$

For A_g = 30(30) = 900 in^2

For f_c' = 4 ksi $\Rightarrow$ ρ = 7.06% Too large

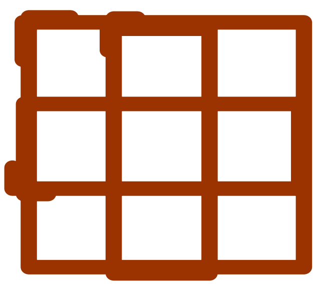
For f_c' = 6 ksi $\Rightarrow$ ρ = 4.18% Reasonable; 24 #11

Boundary Element Confinement

Transverse reinforcement in boundary elements is to be designed essentially like column transverse reinforcement

$$A_{sh} = 0.09\, s b_c \frac{f_c'}{f_y} = 1.08 \text{ in}^2 \text{ at } s = 4"$$

4 legs of #5



Shear Panel Reinforcement

$$V_n = A_{cv}\left(2\lambda\sqrt{f_c^{'}} + \rho_t f_y\right)$$

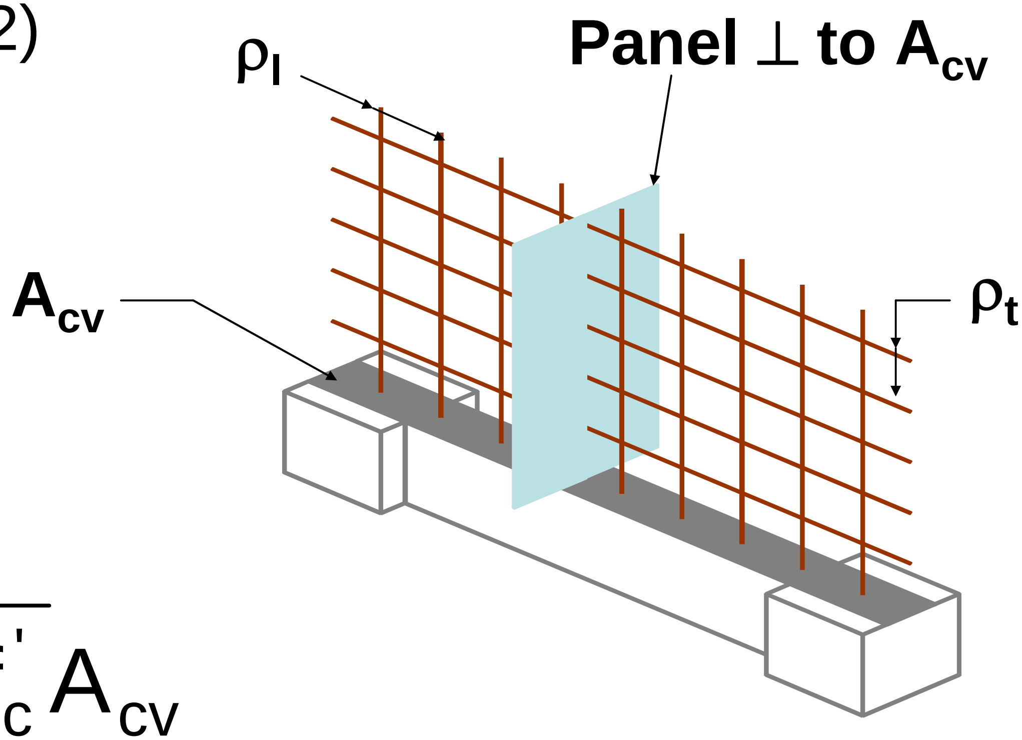
V_u = 539 kips (below level 2)

ϕ = 0.6 (per ACI 9.3.4(a))

ρ_t = 0.0036 for f_y = 40 ksi

Min ρ_ℓ (and ρ_t) = 0.0025

2 curtains if $$V_u > 2\sqrt{f_c^{'}}A_{cv}$$



Shear Panel Reinforcement

Select transverse and longitudinal reinforcement:

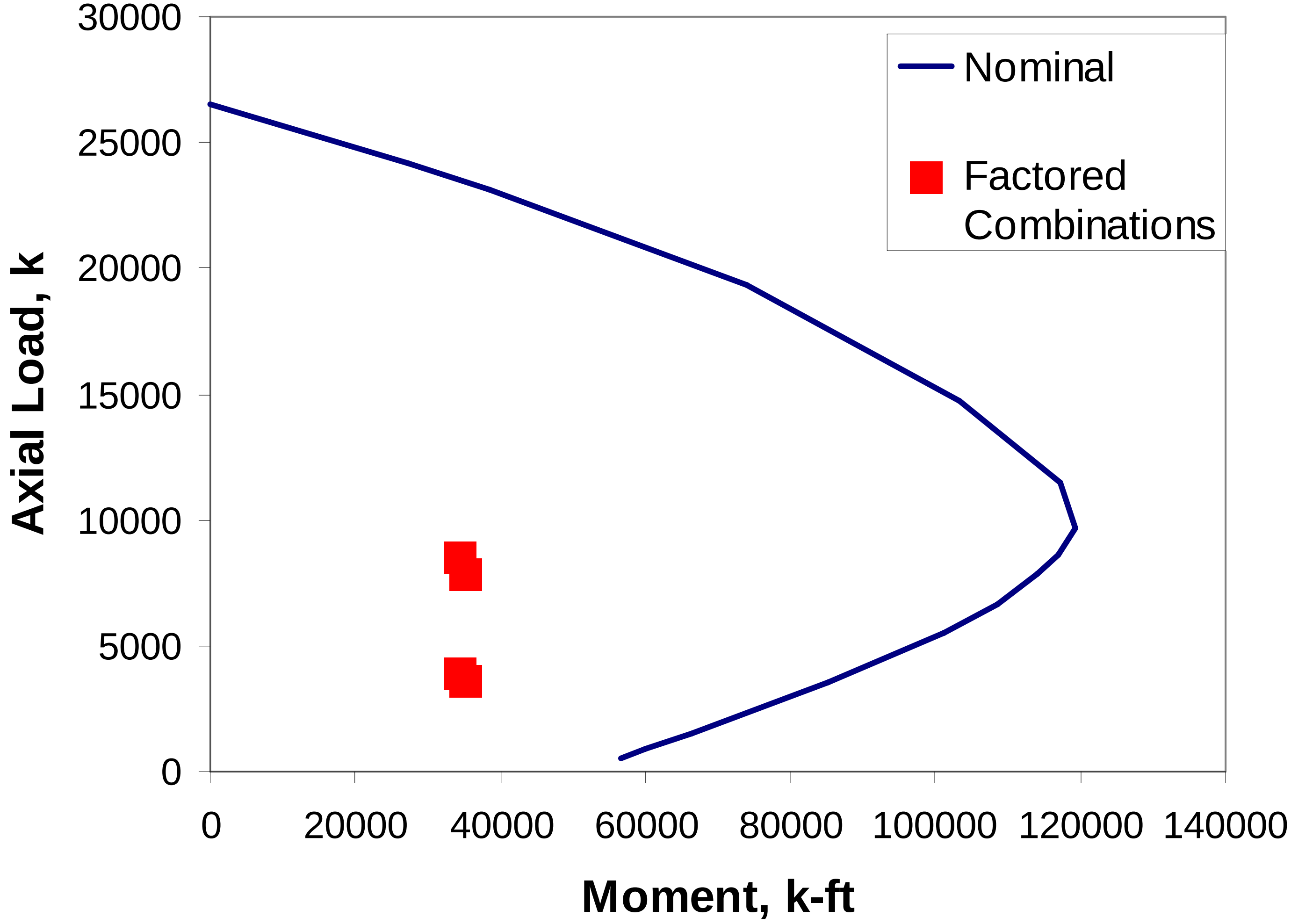
$$\text{longitudinal:}$$

$$\#4@12'' \Rightarrow \frac{0.2 \cdot 2}{12 \cdot 12} = 0.0028 > 0.0025$$

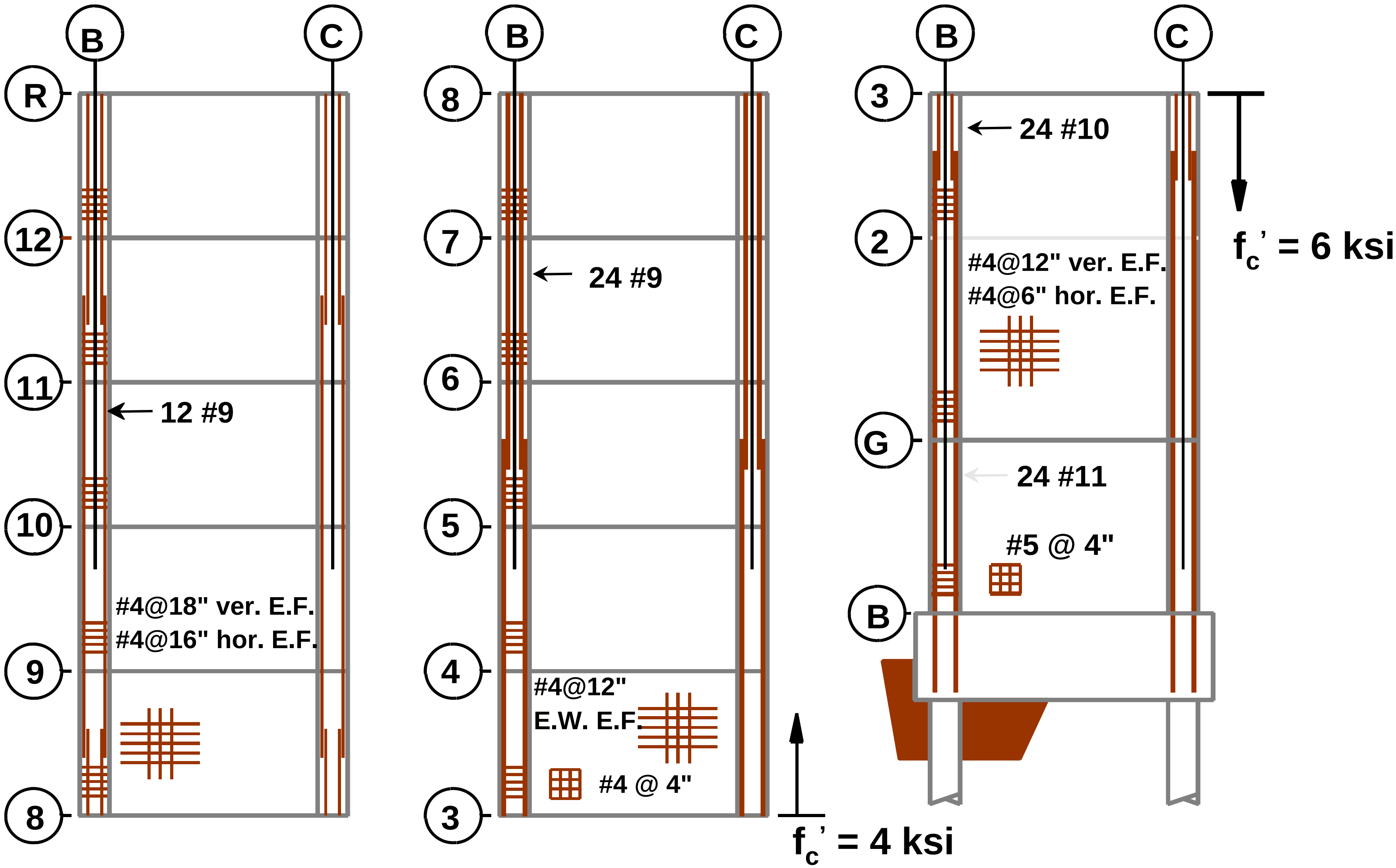
$$\text{transverse:}$$

$$\#4@9'' \Rightarrow \frac{0.2 \cdot 2}{12 \cdot 9} = 0.0037 > 0.0036$$

Check Wall Design



Shear Wall Reinforcement



NEHRP Recommended Provisions Concrete Design

- **Context in the *Provisions***
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- **Requirements by Seismic Design Category**
- **Moment resisting frames**
- **Shear walls**
- **Other topics**

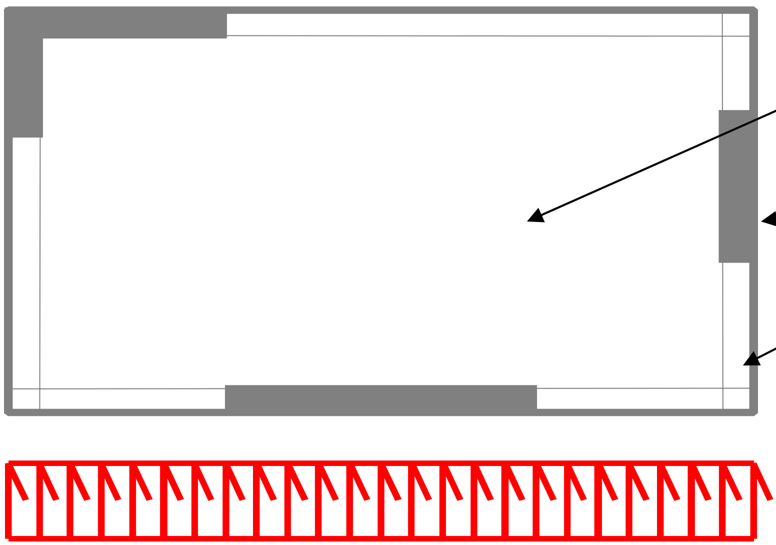
Members Not Part of SRS

- **In frame members not designated as part of the lateral-force-resisting system in regions of high seismic risk:**
  - Must be able to support gravity loads while subjected to the design displacement
  - Transverse reinforcement increases depending on:

    Forces induced by drift

    Axial force in member

Diaphragms



Diaphragm

Shear walls

Collectors, if req'd to transfer force from diaphragm to shear walls

Load from analysis in accordance With design load combinations

<u>Check</u>:

- Shear strength and reinforcement (min. slab reinf.)
- Chords (boundary members)

  - Force = M/d Reinforced for tension

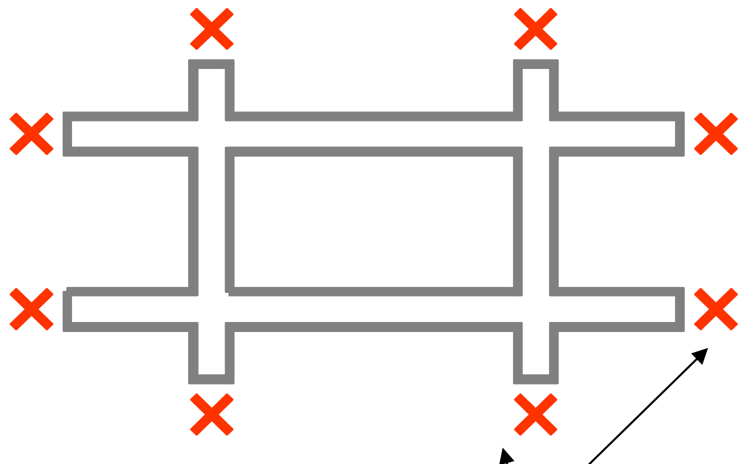
  (Usually don't require boundary members)

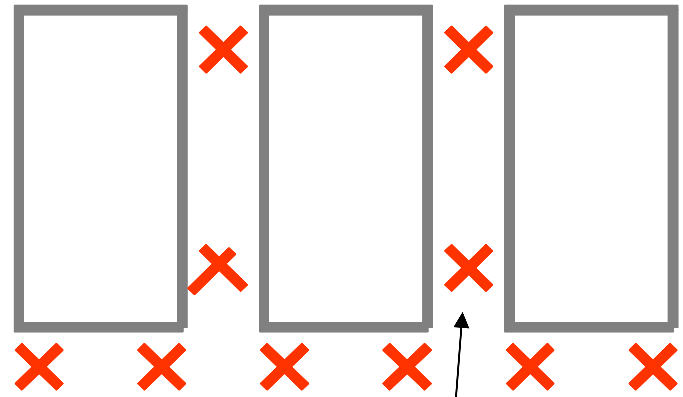
Struts and Trusses
performance objectives

- **All members have axial load (not flexure), so ductility is more difficult to achieve**

- **Full length confinement**

Precast performance objectives





Field connections at points of low stress

Field connections must yield

<u>Strong connections</u>

- Configure system so that hinges occur in factory cast members away from field splices

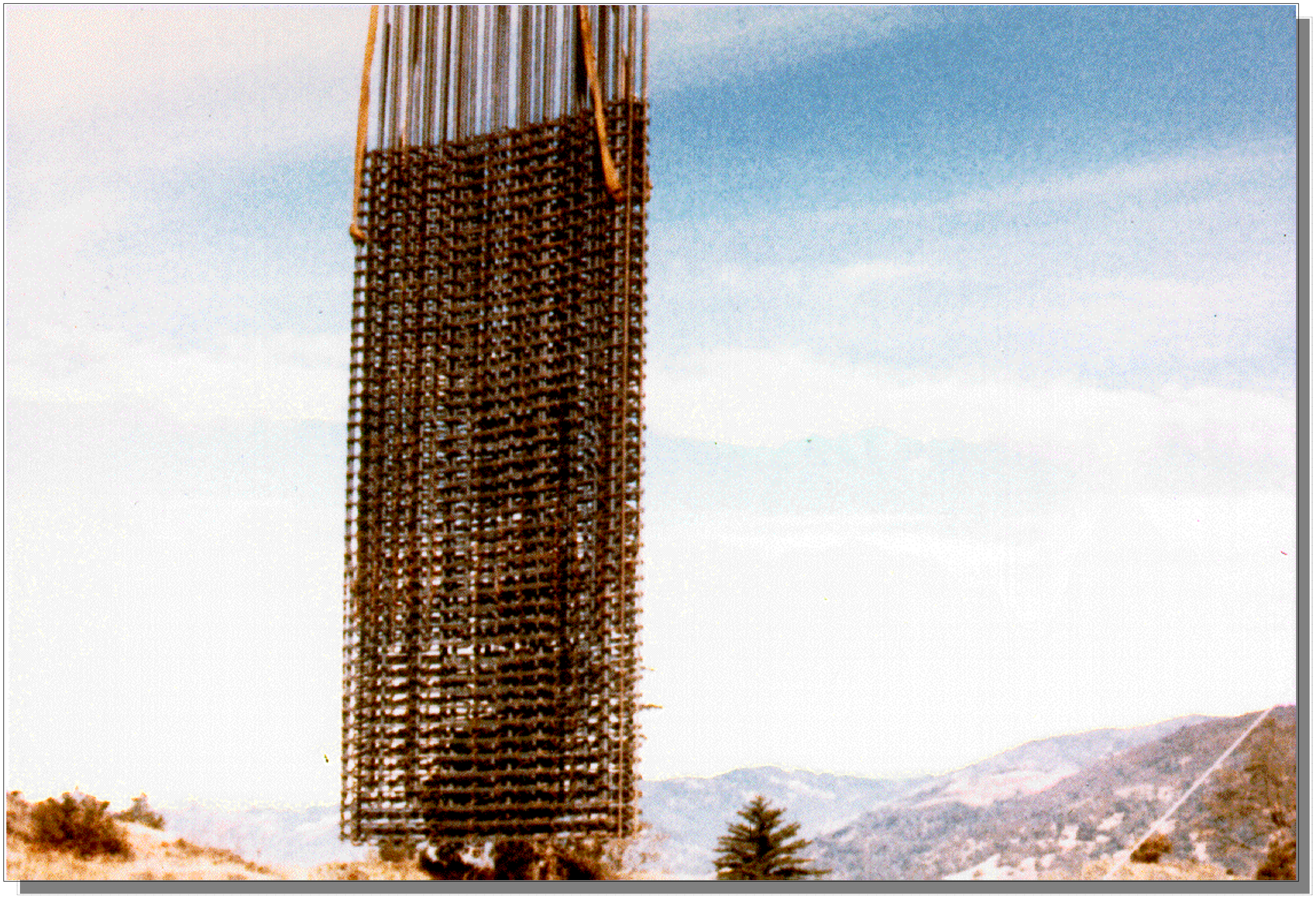
<u>Ductile connections</u>

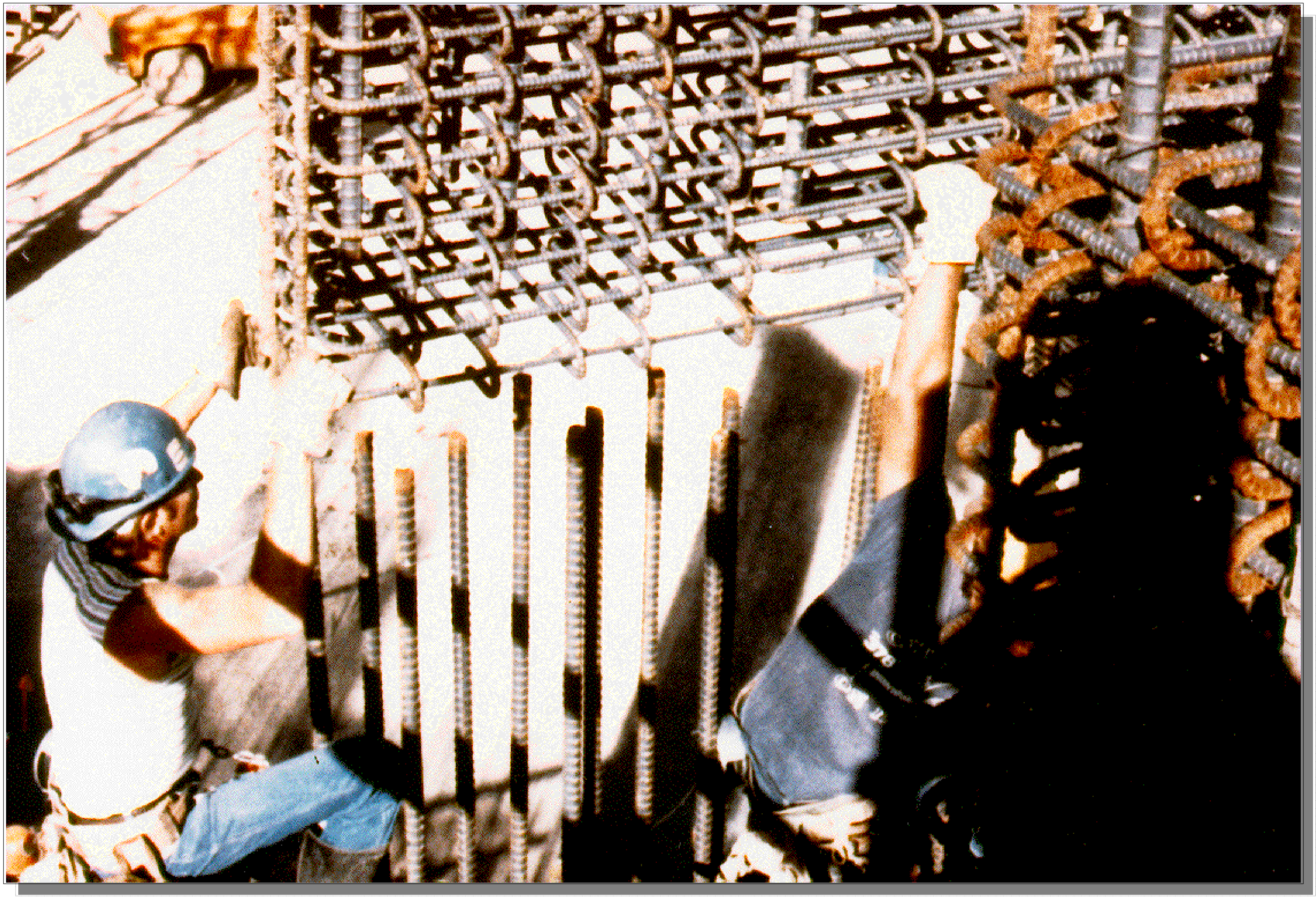
- Inelastic action at field splice

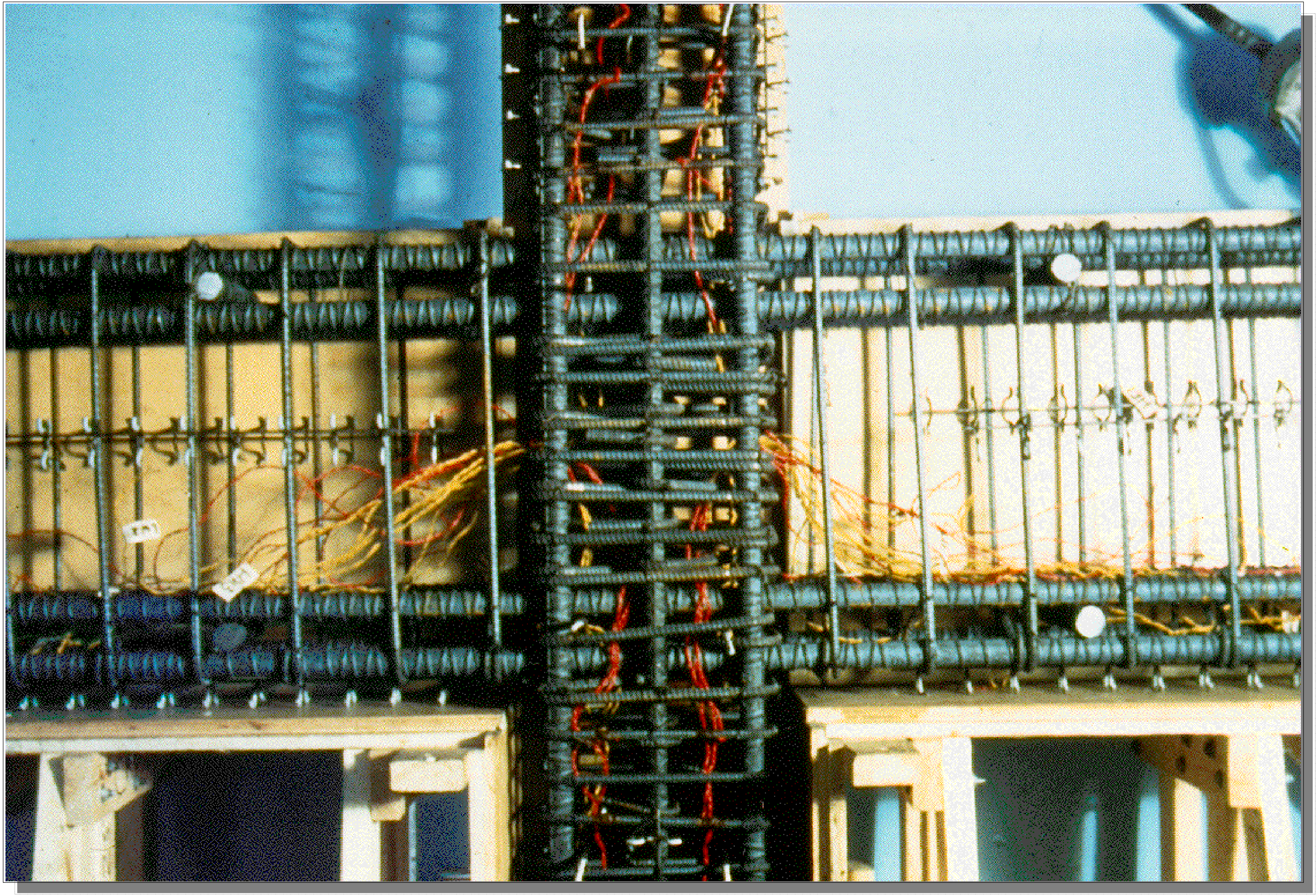
Quality Assurance
Rebar Inspection

- **Continuous**
  – Welding of rebar
- **Periodic**
  – During and upon completion of placement for special moment frames, intermediate moment frames and shear walls

Shear panel reinforcement cage







Quality Assurance: Reinforcing Inspection - Prestressed

- **Periodic**
  – Placing of prestressing tendons (inspection required upon completion)
- **Continuous**
  – Stressing of tendons
  – Grouting of tendons

Quality Assurance: Concrete Placement Inspection

- **Continuous**
  - Prestressed elements
  - Drilled piers
  - Caissons
- **Periodic**
  - Frames
  - Shear walls

Quality Assurance: Precast Concrete (plant cast)

- Manufacturer may serve as special inspector if plant's quality control program is approved by regulatory agency
- If no approved quality control program, independent special inspector is required

Quality Assurance: PCI Certification Program

- **Review of plant operations**
  - Scheduled and surprise visits
  - Qualified independent inspectors
  - Observed work of in-plant quality control
  - Check results of quality control procedures
  - Periodic – specific approvals requiring renewal

Quality Assurance: ACI Inspector Certification

- **Specialized training available for:**
  - Laboratory and in situ testing
  - Inspection of welding
  - Handling and placement of concrete
  - Others

Quality Assurance: Reinforcement Testing

- **Rebar**
  - Special and intermediate moment frames
  - Boundary elements
- **Prestressing steel**
- **Tests include**
  - Weldability
  - Elongation
  - Actual to specified yield strength
  - Actual to specified ultimate strength

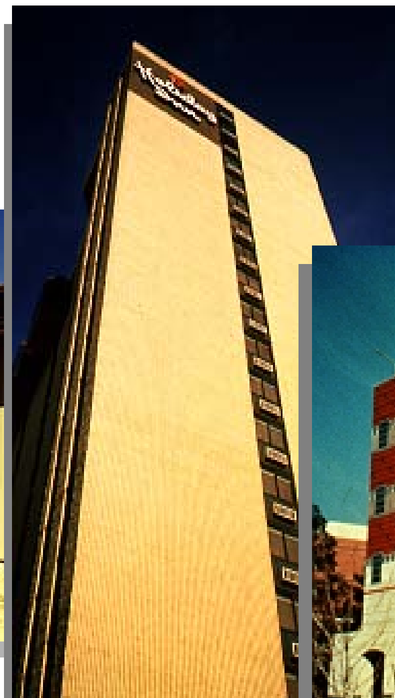
Quality Assurance: Concrete Testing

- **Sample and test according to ACI 318-05**
  - Slump
  - Air content
  - 7 and 28 day strengths
  - Unit weight
- **Rate**
  - Once per day per class

NEHRP Recommended Provisions: Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**
- **Reference standards**
- **Requirements by Seismic Design Category**
- **Moment resisting frames**
- **Shear walls**
- **Other topics**
- **Summary**

SEISMIC DESIGN OF MASONRY STRUCTURES



NEHRP Recommended Provisions Masonry Design

- **Context in the *NEHRP Recommended Provisions***
- **Masonry behavior**
- **Reference standards**
- **Seismic resisting systems**
- **Component design**
- **Quality assurance**
- **Summary**

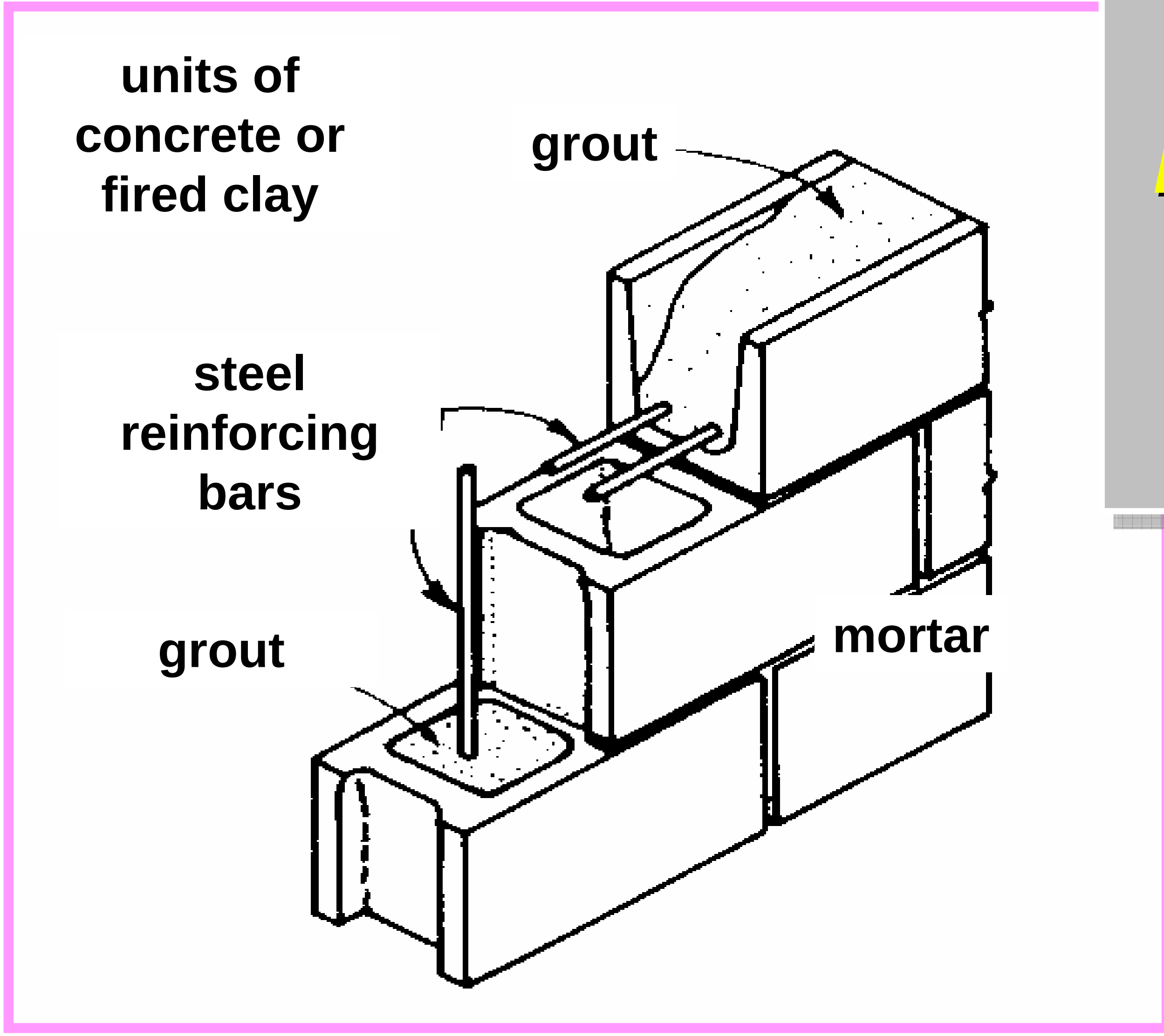
Objectives of Module

- Basics of masonry behavior
- Basics of masonry specification
- The MSJC code and specification and their relationship to the *NEHRP Recommended Provisions* documents
- Earthquake design of masonry structures and components using the 2005 MSJC code and specification
- Example of masonry shear wall design

Context in the *NEHRP Recommended Provisions*

- **Design seismic loads**
  - **Load combinations** — **Chap. 5**
  - **Loads on structures** — **Chap. 5**
  - **Loads on components & attachments** — **Chap. 6**
- **Design resistances** — **Chap. 11**
  - **Strength design (mostly references the 2002 MSJC)**

... typical materials in reinforced masonry



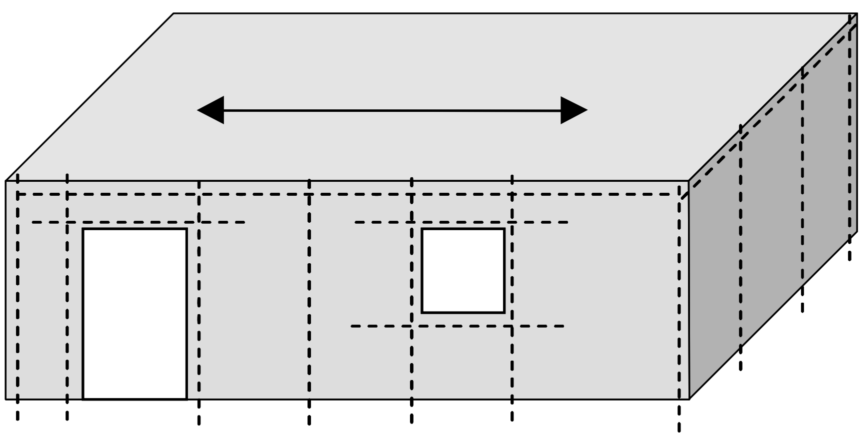
Essential Elements of Simplified Design for Wall-type Structures

- Starting point for design
- Design of vertical strips in walls perpendicular to lateral loads
- Design of walls parallel to lateral loads
- Design of lintels
- Simplified analysis for lateral loads
- Design of diaphragms
- Detailing

Starting Point for Wall-type Masonry Structures

No beams or columns

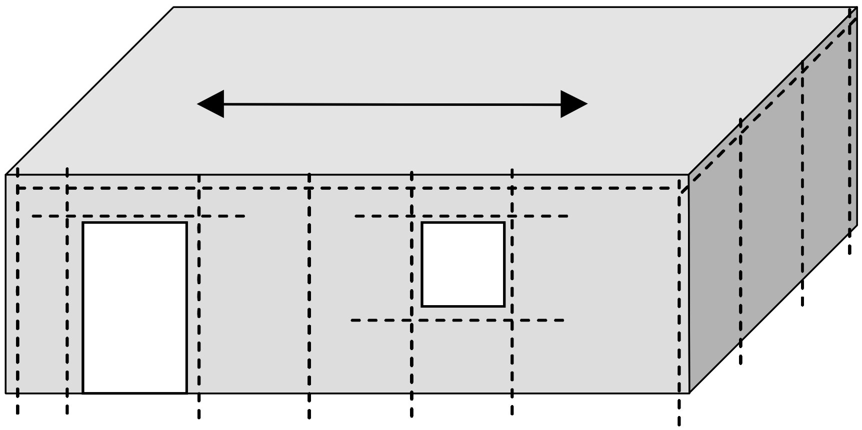
(Example of direction of span)



Vertical reinforcement of #4 bars at corners and jambs

Horizontal reinforcement of two #4 bars in bond beam at top of wall, and over and under openings (two #5 bars with span > 6 ft)

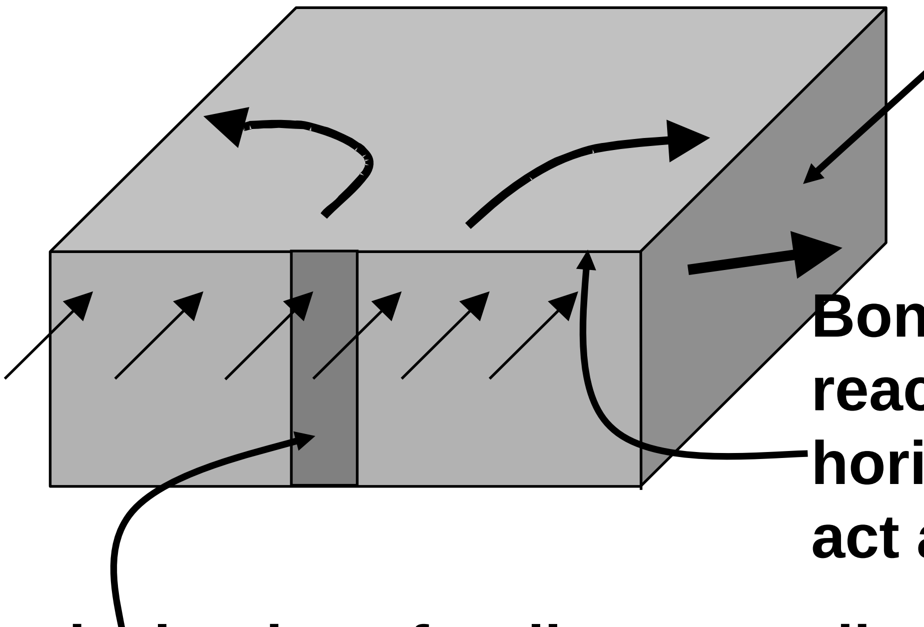
Essential Function of Walls in Resisting Gravity Loads



Bearing walls resist axial loads (concentric and eccentric) as vertical strips

Nonbearing walls resist concentric axial load as vertical strips

Essential Function of Walls in Resisting Lateral Forces

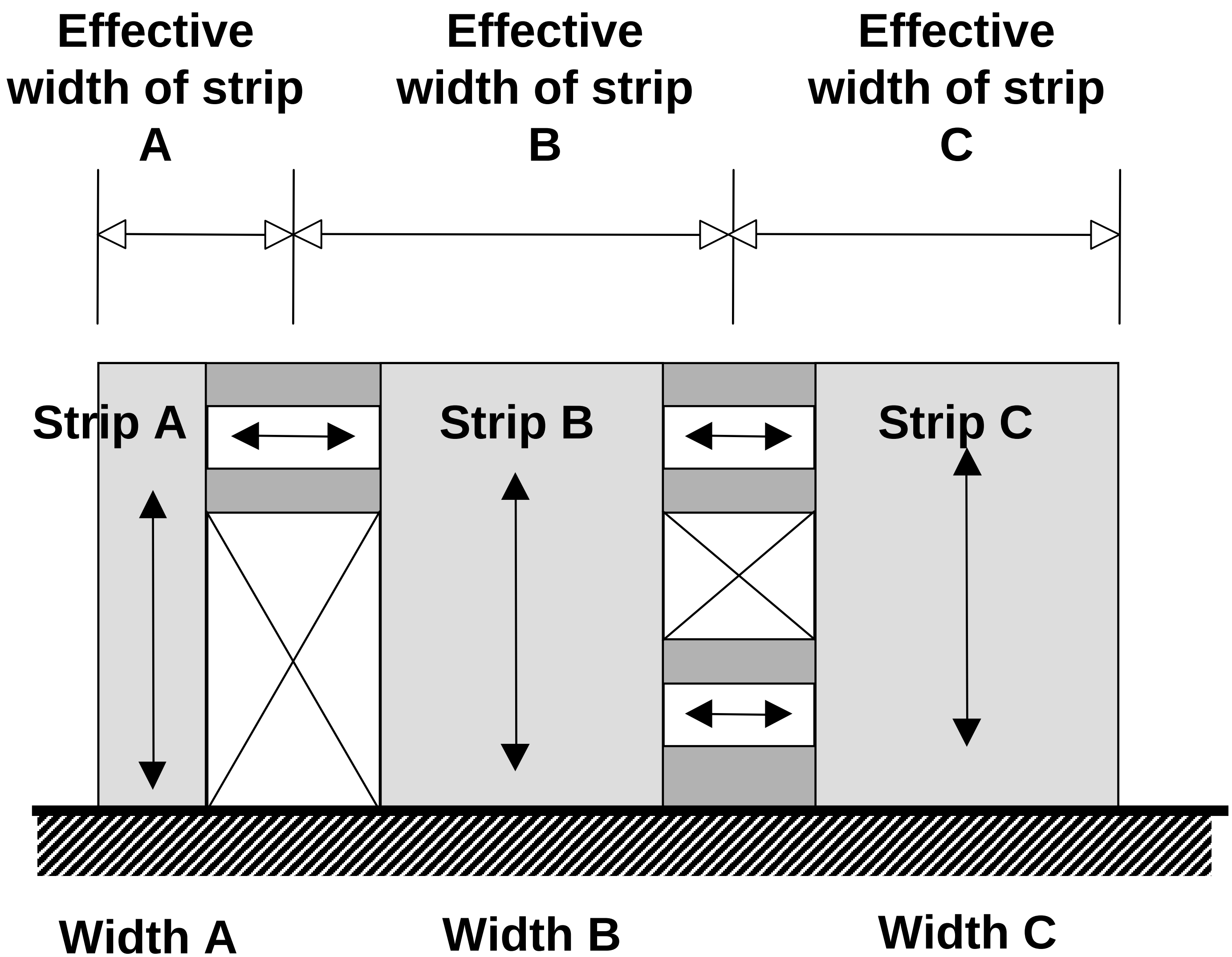


Walls parallel to lateral forces act as shear walls

Bond beams transfer reactions from walls to horizontal diaphragms and act as diaphragm chords

Vertical strips of walls perpendicular to lateral forces resist combinations of axial load and out-of-plane moments, and transfer their reactions to horizontal diaphragms

Effect of Openings



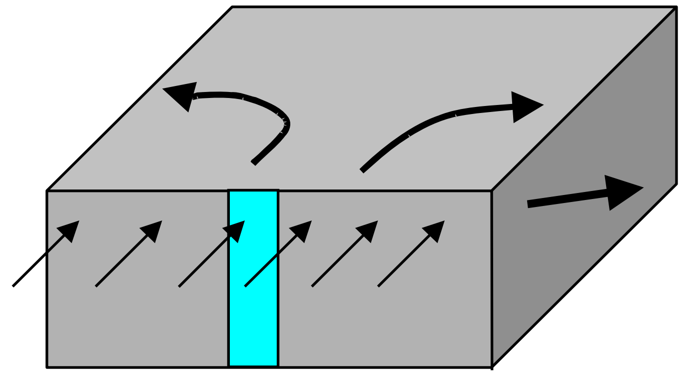
Effect of Openings

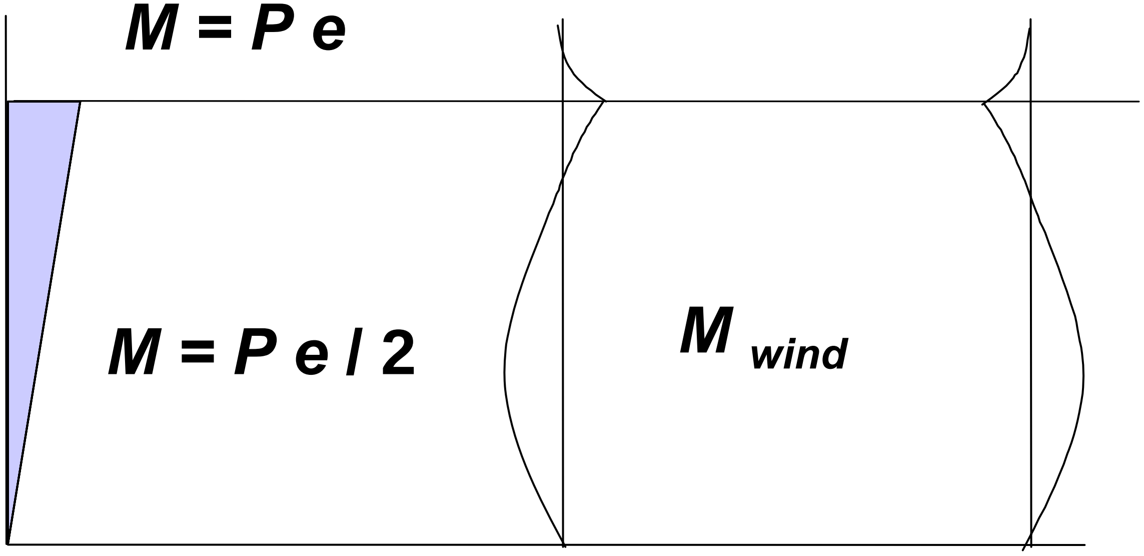
Openings increase original design actions on each strip by a factor equal to the ratio of the effective width of the strip divided by the actual width:

$$Actions\ in\ Strip\ B = Original\ \ Actions\left(\frac{Effective\,Width\ B}{Actual\ Width\ B}\right)$$

Design of Vertical Strips in Perpendicular Walls

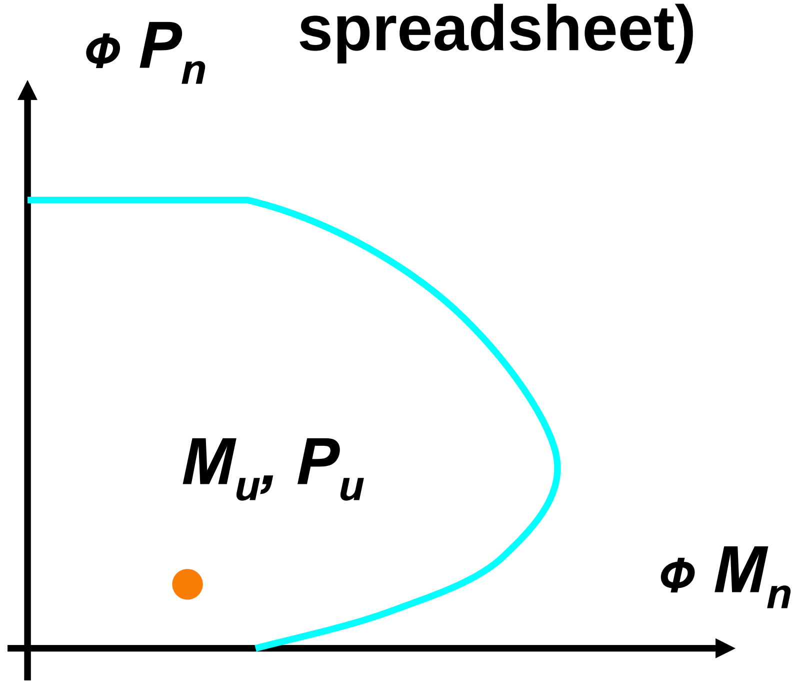
Moments and axial forces due to combinations of gravity and lateral load

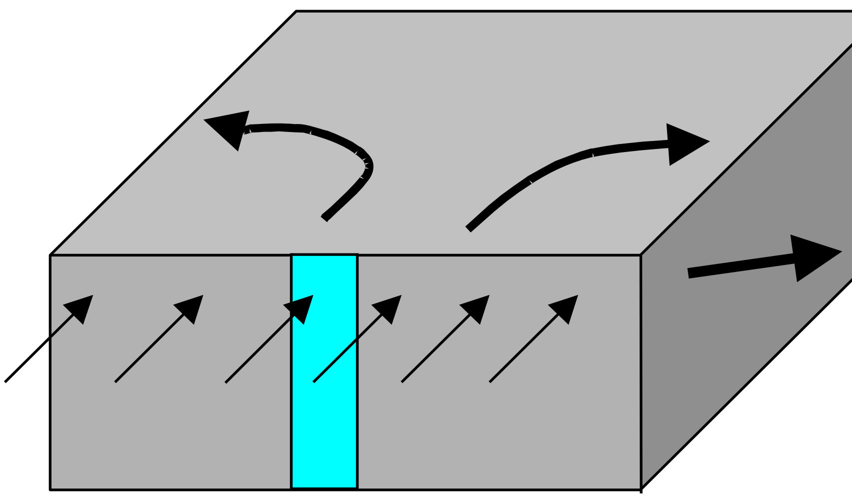




Design of Vertical Strips in Perpendicular Walls

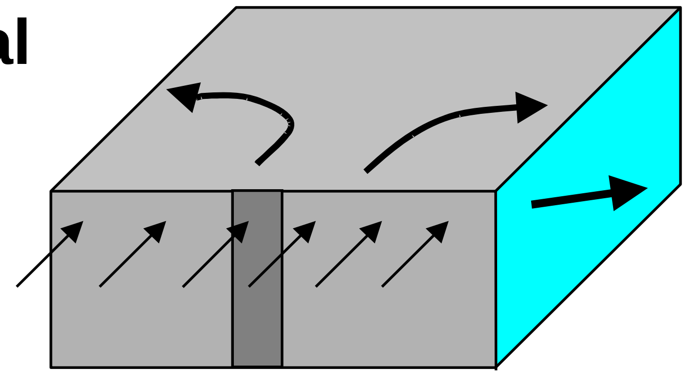
Moment-axial force interaction diagram (with the help of a spreadsheet)

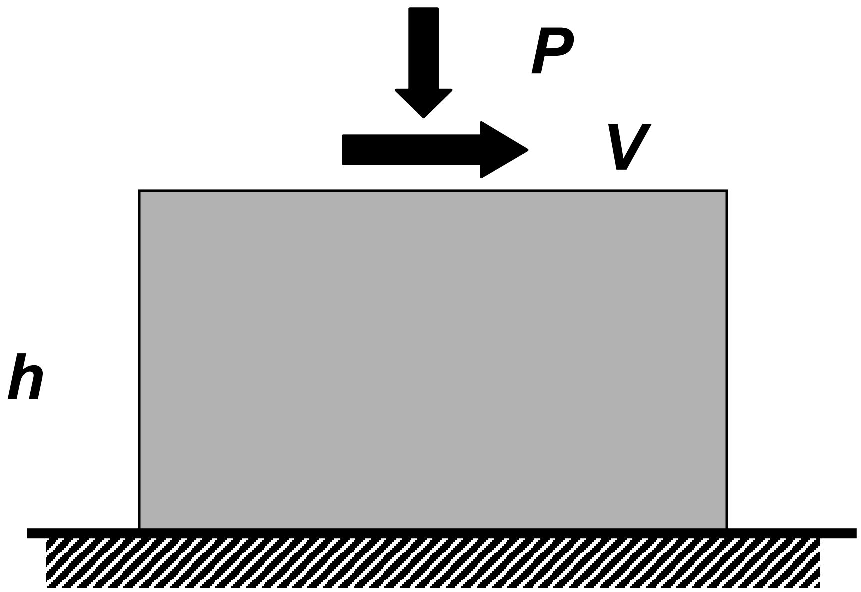




Design of Parallel Walls

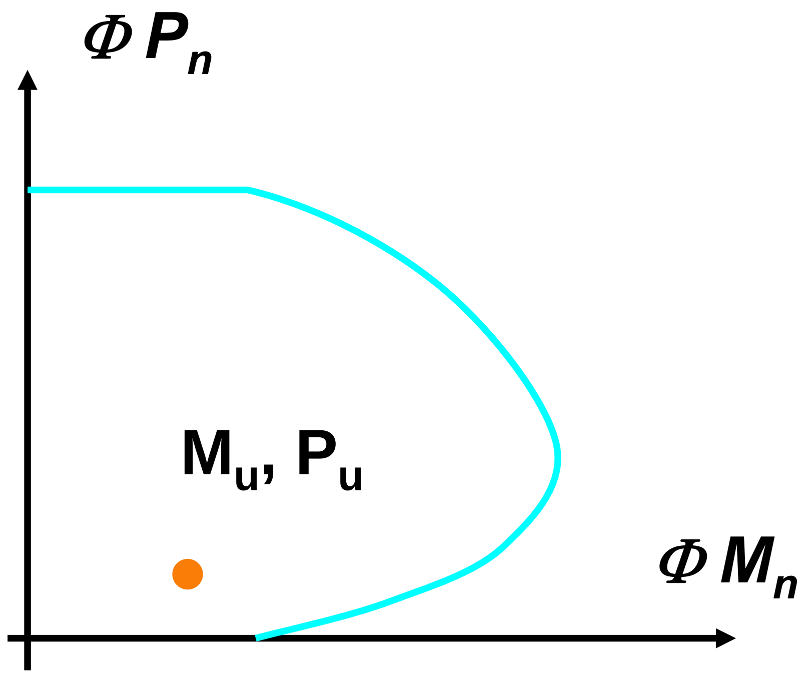
Moments, axial forces, and shears due to combinations of gravity and lateral loads



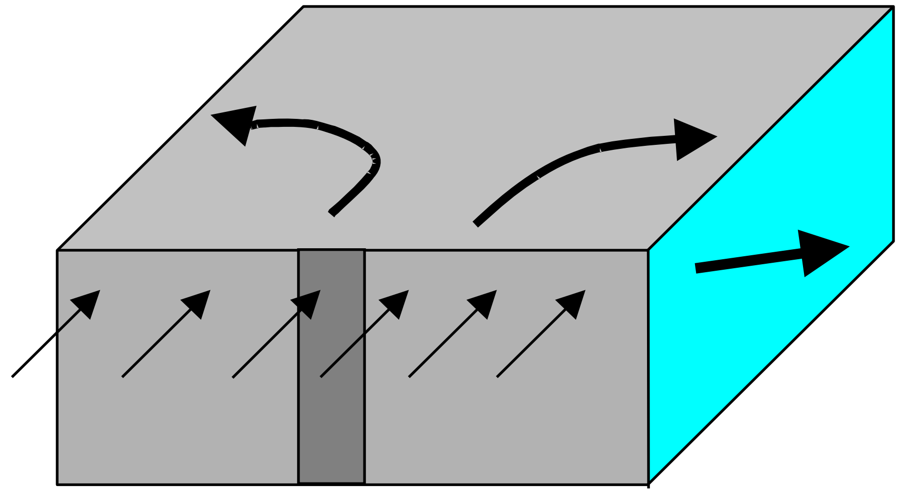


Design of Parallel Walls

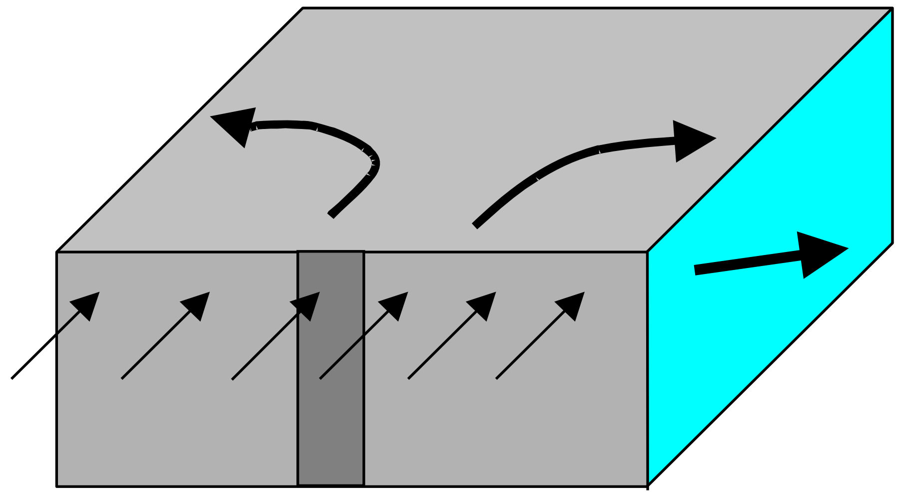
Moment-axial force interaction diagram (with the help of a spreadsheet)



Sufficient lateral capacity comes from wall density.



Design of Parallel Walls



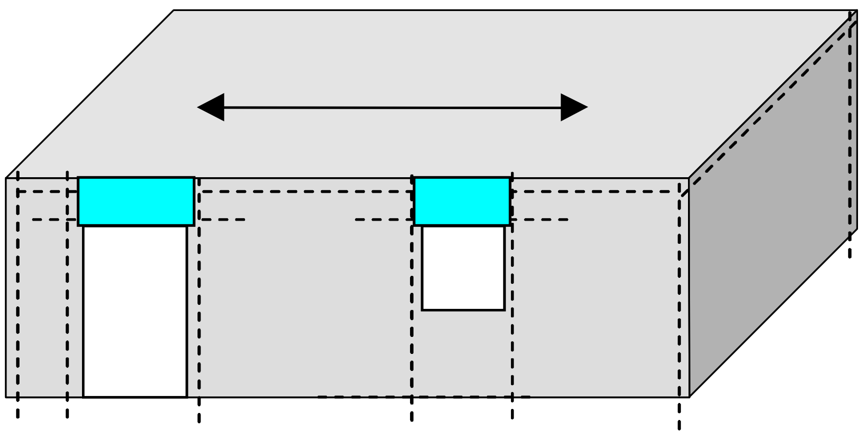
Shearing resistance:

$$V_n = V_m + V_s$$

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u\, d_v}\right)\right] A_n \sqrt{f'_m} + 0.25\, P_u$$

Design of Lintels

(Example of direction of span)

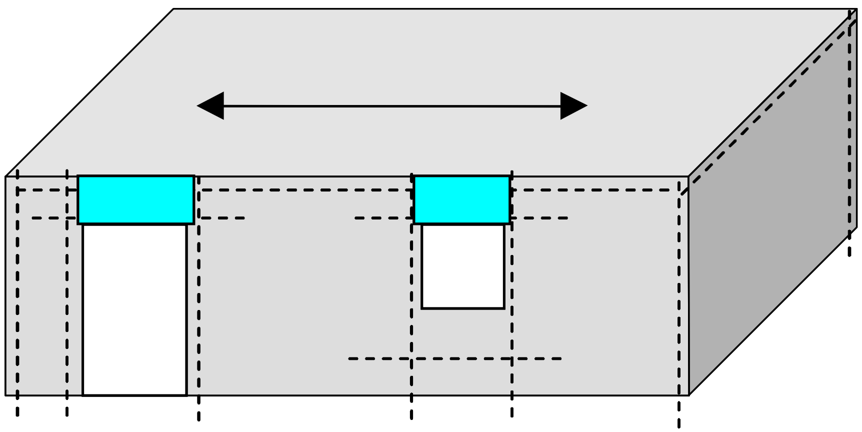


Moments and shears due to gravity loads:

$$M_u = \frac{w\,\ell^2}{8}$$

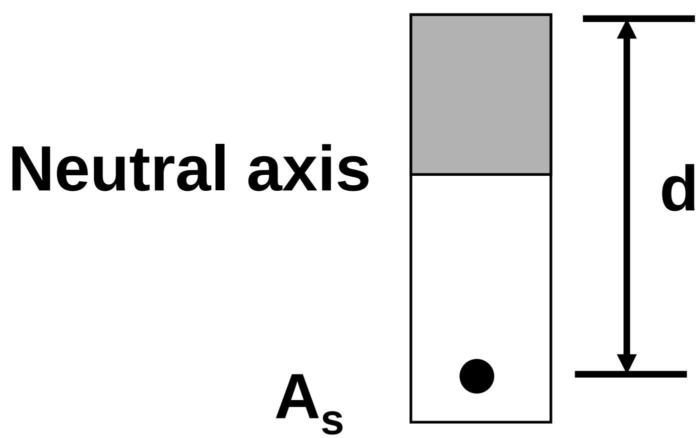
$$V_u = \frac{w\,\ell}{2}$$

Design of Lintels



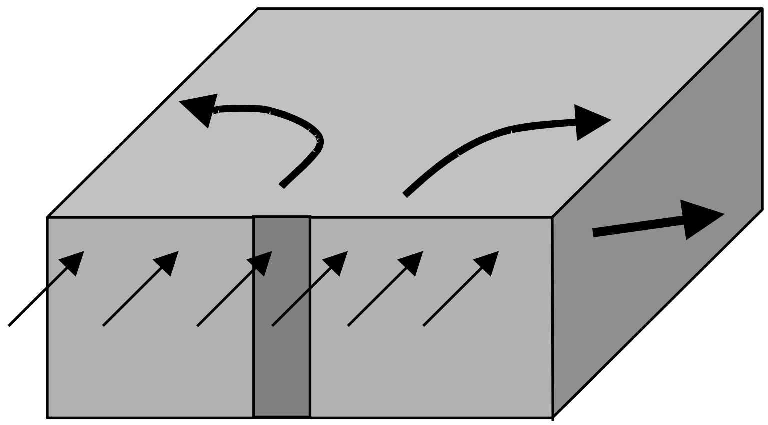
Shear design: Provide enough depth so that shear reinforcement is not needed.

Flexural design:



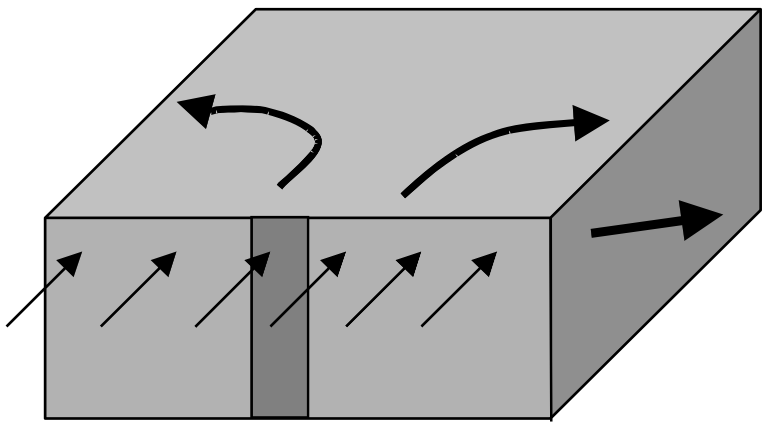
$$A_s \approx \frac{M_u}{\phi \times f_y \times 0.9\, d}$$

Distribution of Shears to Shear Walls



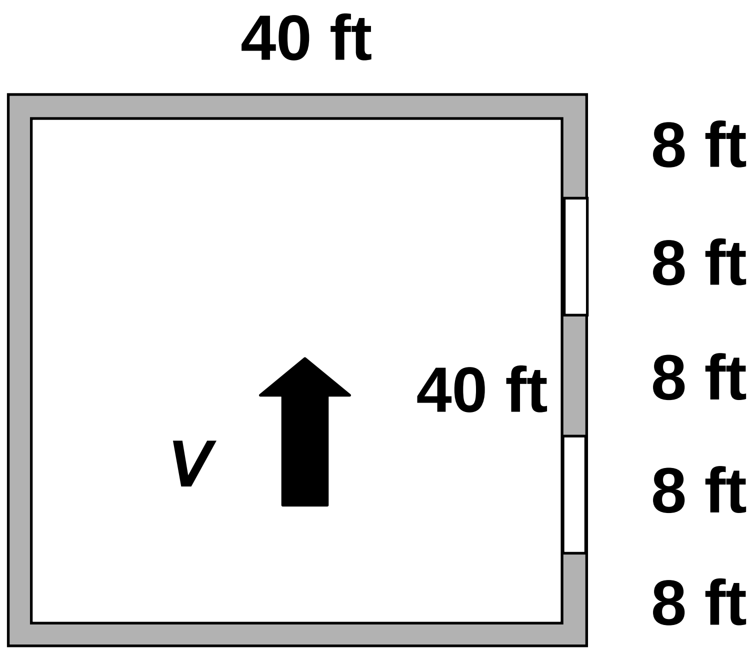
- Classical approach
  - Determine whether the diaphragm is "rigid" or "flexible"
  - Carry out an appropriate analysis for shears

Classical Analysis of Structures with Rigid Diaphragms



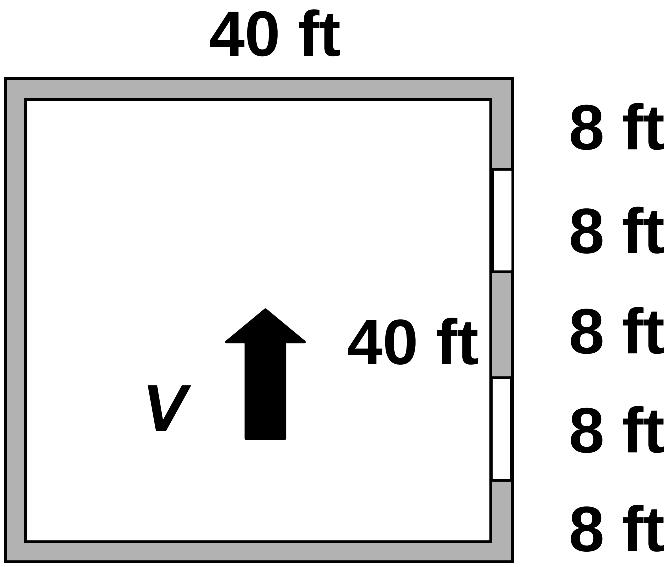
- Locate center of rigidity
- Treat the lateral load as the superposition of a load acting through the center of rigidity and a torsional moment about that center of rigidity

Simplified Analysis of Structures with Rigid Diaphragms



- Consider only the shearing stiffness, which is proportional to plan length
- Neglect plan torsion

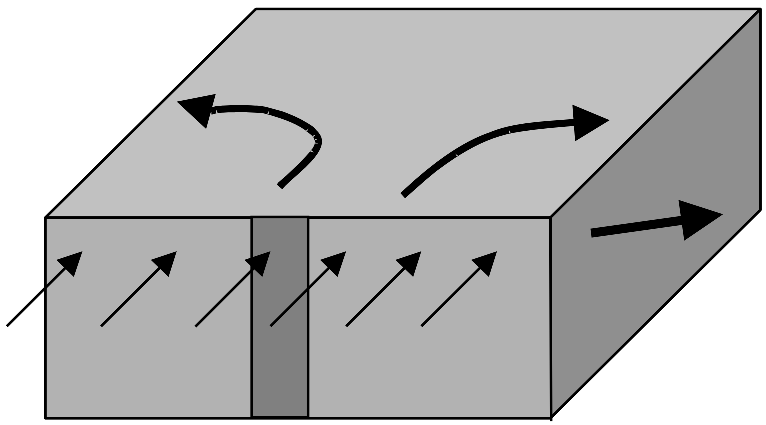
Simplified Analysis of Structures with Rigid Diaphragms



$$V_{left} = \frac{40\ ft}{(40+8+8+8)\ ft} \times V_{total} = \frac{5}{8} V_{total}$$

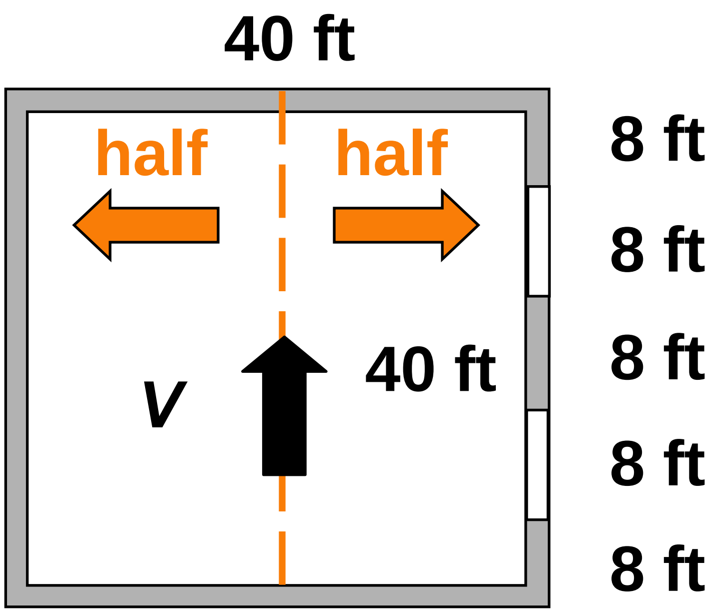
$$V_{right} = \frac{(8+8+8)\ ft}{(40+8+8+8)\ ft} \times V_{total} = \frac{3}{8} V_{total}$$

Classical Analysis of Structures with Flexible Diaphragms



- Distribute shears according to tributary areas of the diaphragm independent of the relative stiffnesses of the shear walls

Classical Analysis of Structures with Flexible Diaphragms



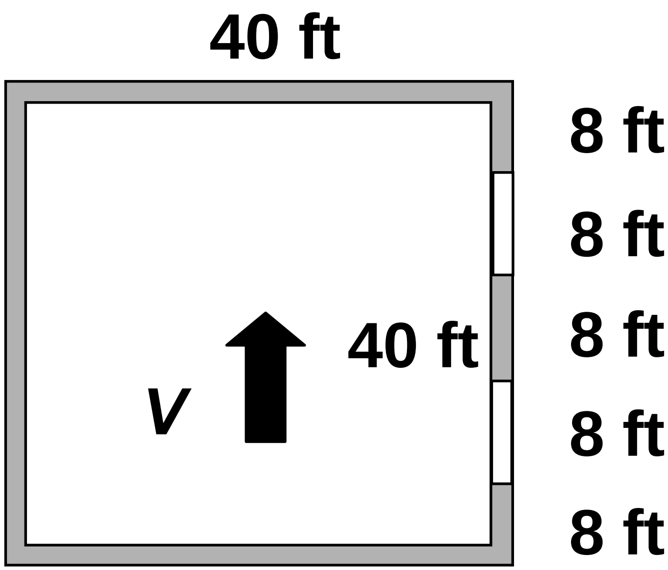
$$V_{left} = \frac{1}{2} V_{total}$$

$$V_{right} = \frac{1}{2} V_{total}$$

Simplified Diaphragm Analysis

Design for the worse of the two cases:

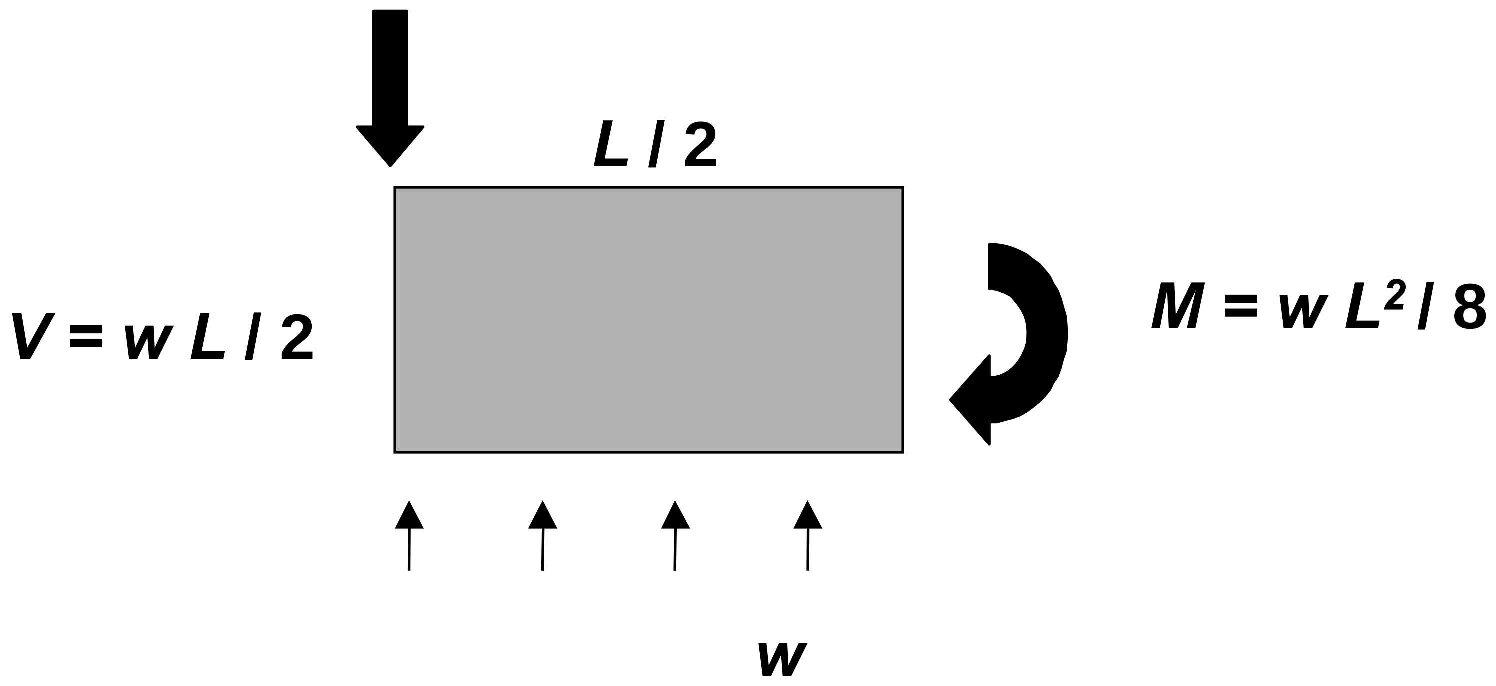
5 / 8 *V*
1 / 2 *V*



3 / 8 *V*
1 / 2 *V*

Diaphragm Design

- Diaphragm shears are resisted by total depth or by cover (for plank diaphragms). Diaphragm moments are resisted by diaphragm chords in bond beams.



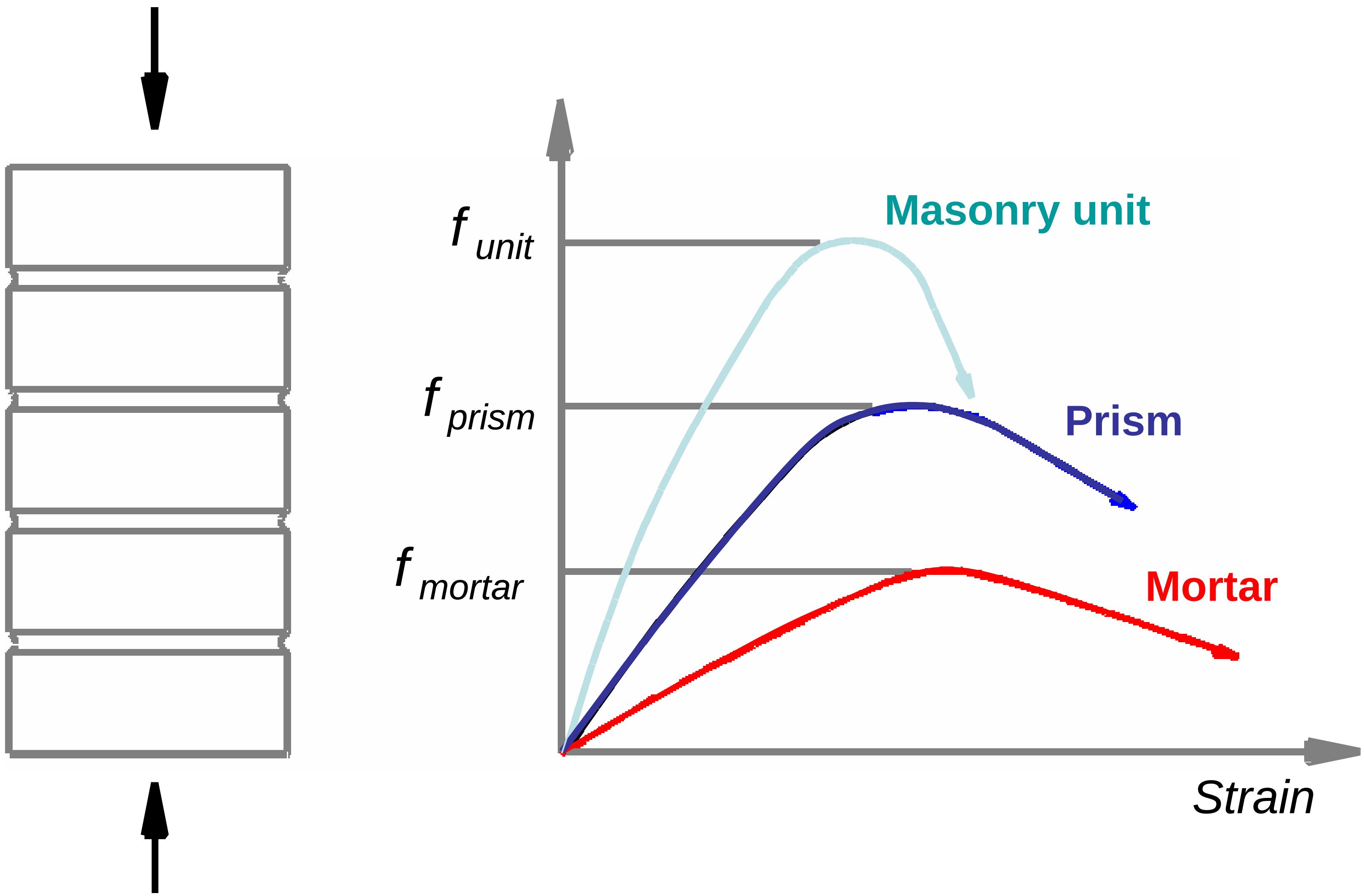
Details

- Wall-diaphragm connections
- Design of lintels for out-of-plane loads between wall-diaphragm connections
- Connections between bond beam and walls
- Connections between walls and foundation

Masonry Behavior

- On a local level, masonry behavior is nonisotropic, nonhomogeneous, and nonlinear.
- On a global level, however, masonry behavior can be idealized as isotropic and homogeneous. Nonlinearity in compression is handled using an equivalent rectangular stress block as in reinforced concrete design.
- A starting point for masonry behavior is to visualize it as very similar to reinforced concrete. Masonry capacity is expressed in terms of a specified compressive strength, f_m', which is analogous to f_c'.

Masonry Behavior Stress-Strain Curve for Prism Under Compression

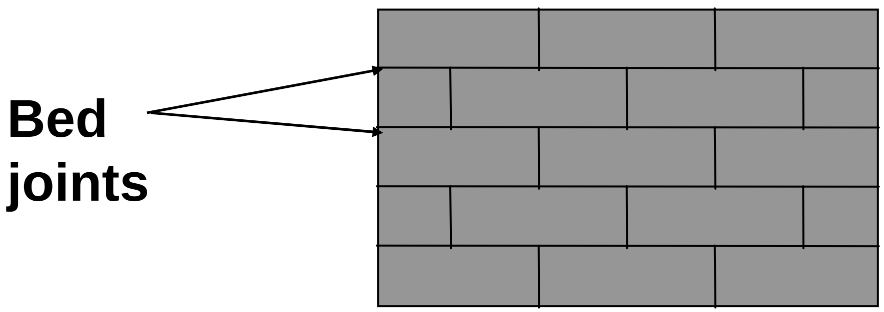


Review Masonry Basics

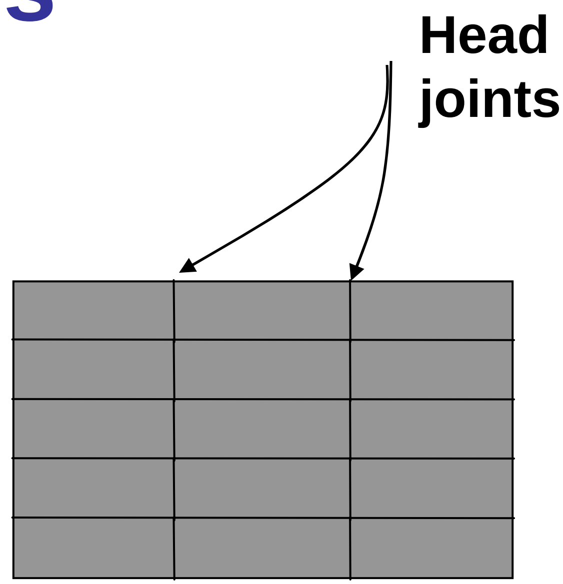
- Basic terms
- Units
- Mortar
- Grout
- Accessory materials
  - Reinforcement (may or may not be present)
  - Connectors
  - Flashing
  - Sealants
- Typical details

Basic Terms

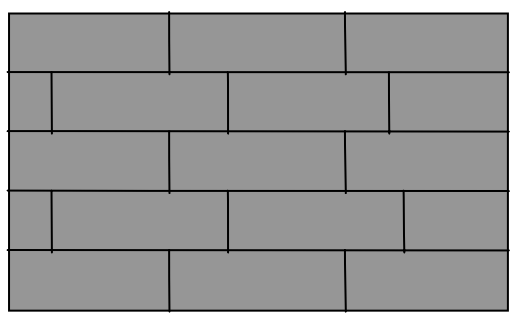
- Bond patterns (looking at wall):



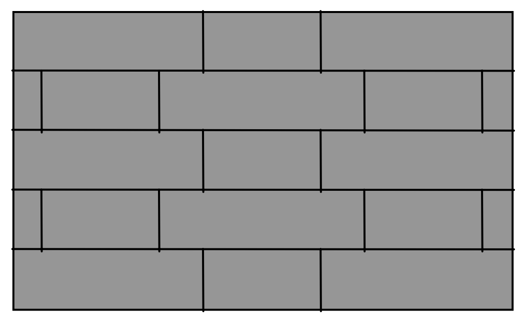
Running bond



Stack bond



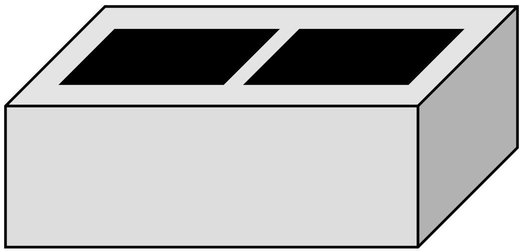
1/3 Running bond



Flemish bond

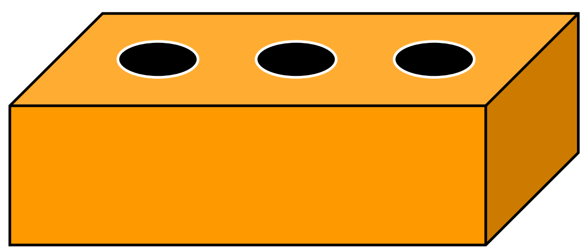
Masonry Units

- Concrete masonry units (CMU):
  - Specified by ASTM C 90
  - Minimum specified compressive strength (net area) of 1900 psi (average)
  - Net area is about 55% of gross area
  - Nominal versus specified versus actual dimensions
  - Type I and Type II designations no longer exist



Masonry Units

- **Clay masonry units:**
  - **Specified by ASTM C 62 or C 216**
  - **Usually solid, with small core holes for manufacturing purposes**
  - **If cores occupy ≤ 25% of net area, units can be considered 100% solid**



Masonry Mortar

- Mortar for unit masonry is specified by ASTM C 270
- Three cementitious systems
  - Portland cement – lime mortar
  - Masonry cement mortar
  - Mortar cement mortar

Masonry Mortar

- **Within each cementitious system, mortar is specified by type (M a S o N w O r K):**
  - **Going from Type K to Type M, mortar has an increasing volume proportion of portland cement. It sets up faster and has higher compressive and tensile bond strengths.**
  - **As the volume proportion of portland cement increases, mortar is less able to deform when hardened.**
  - **Types N and S are specified for modern masonry construction.**

Masonry Mortar

- Under ASTM C270, mortar can be specified by proportion or by property.
- If mortar is specified by proportion, compliance is verified only by verifying proportions. For example:
  - Type S PCL mortar has volume proportions of 1 part cement to about 0.5 parts hydrated mason's lime to about 4.5 parts mason's sand.
  - Type N masonry cement mortar (single-bag) has one part Type N masonry cement and 3 parts mason's sand.

Masonry Mortar

- **Under ASTM C270, mortar can be specified by proportion or by property:**
  - **Proportion specification is simpler -- verify in the field that volume proportions meet proportion limits.**
  - **Property specification is more complex: (1) establish the proportions necessary to produce a mortar that, tested at laboratory flow, will meet the required compressive strength, air content, and retentivity (ability to retain water) requirements and (2) verify in the field that volume proportions meet proportion limits.**

Masonry Mortar

- The proportion specification is the default. Unless the property specification is used, no mortar testing is necessary.
- The proportion of water is not specified. It is determined by the mason to achieve good productivity and workmanship.
- Masonry units absorb water from the mortar decreasing its water-cement ratio and increasing its compressive strength. Mortar need not have high compressive strength.

Grout

- Grout for unit masonry is specified by ASTM C 476
- Two kinds of grout:
  - Fine grout (cement, sand, water)
  - Coarse grout (cement, sand, pea gravel, water)
- ASTM C 476 permits a small amount of hydrated lime, but does not require any. Lime is usually not used in plant – batched grout.

Grout

- Under ASTM C476, grout can be specified by proportion or by compressive strength:
  - Proportion specification is simpler. It requires only that volume proportions of ingredients be verified.
  - Specification by compressive strength is more complex. It requires compression testing of grout in a permeable mold (ASTM C 1019).

Grout

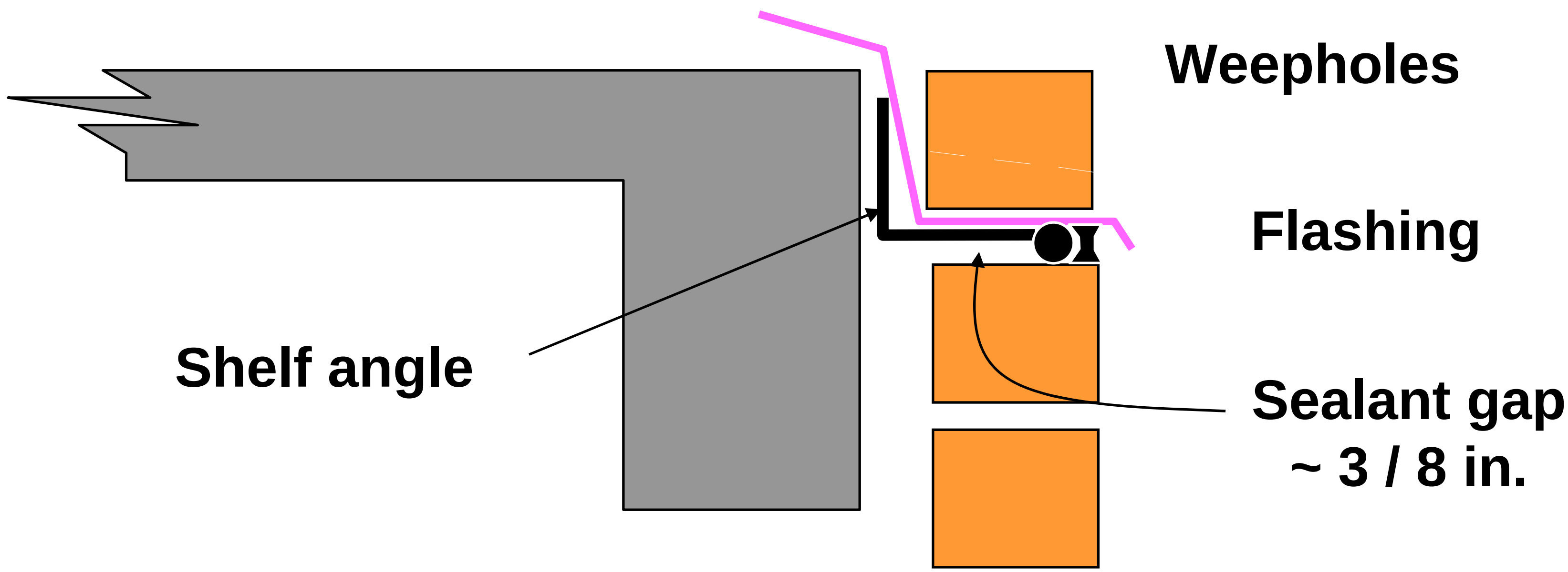
- If grout is specified by proportion, compliance is verified only by verifying proportions. For example:
  - Fine grout has volume proportions of 1 part cement to about 3 parts mason's sand.
  - Coarse grout has volume proportions of 1 part cement to about 3 parts mason's sand and about 2 parts pea gravel.
- Unless the compressive-strength specification is used, no grout testing is necessary.

Grout

- The proportion of water is not specified. The slump should be 8 to 11 in.
- Masonry units absorb water from the grout decreasing its water-cement ratio and increasing its compressive strength. High-slump grout will still be strong enough.

Accessory Materials

Horizontally oriented expansion joint under shelf angle:



MASONRY DESIGN CODES IN THE US

ANSI process (balance of interests, letter ballots, resolution of Negatives, public comment)

Technical Organizations

Industry Groups

MSJC develops provisions

MSJC Code

NEHRP

model codes reference those provisions

ICC (International Building Code)

Other Model Codes (NFPA)

local authorities adopt those model codes

Building Code (legal standing) (contract between society and the designer)

ASTM (Material Specifications)

MSJC Specification (QA, materials, execution)

(part of a civil contract between owner and contractor)

What is the MSJC Code and Specification... ?

ACI
(ACI 530-05)
(ACI 530.1-05)

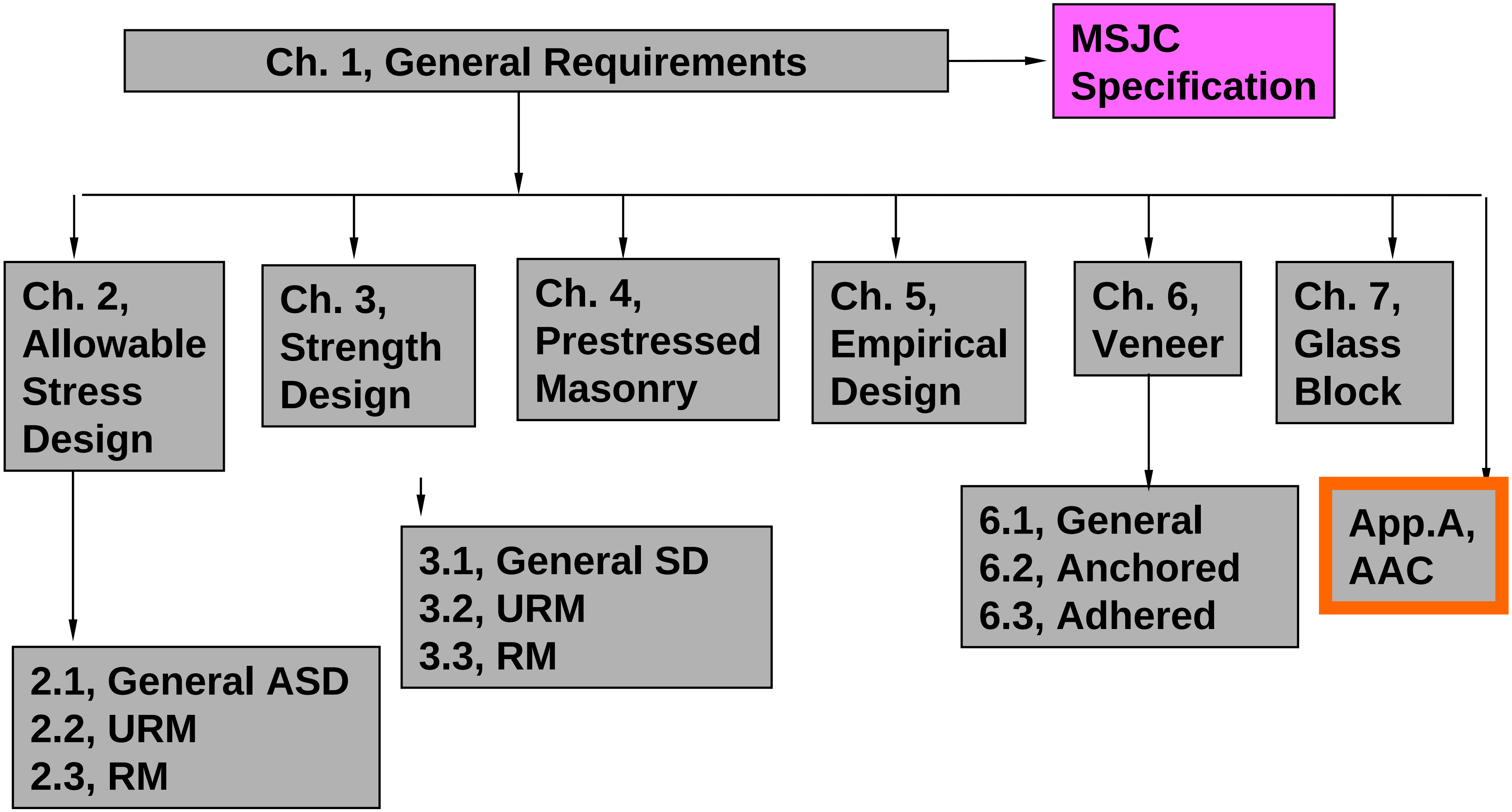
2005 MSJC Code and Specification

TMS
(TMS 402-05)
(TMS 602-05)

ASCE
(ASCE 5-05)
(ASCE 6-05)

"Masonry Standards Joint Committee"

2005 MSJC Code



Relation Between Code and Specification

- **Code:**
  - **Design provisions are given in Chapters 1-7 and Appendix A**
  - **Sections 1.2.4 and 1.14 require a QA program in accordance with the specification**
  - **Section 1.4 invokes the specification by reference.**
- **Specification:**
  - **Verify compliance with specified f_m'**
  - **Comply with required level of quality assurance**
  - **Comply with specified products and execution**

Role of f_m'

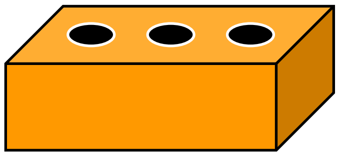
- **Concrete:**
  - **Designer states assumed value of f_c'**
  - **Compliance is verified by compression tests on cylinders cast in the field and cured under ideal conditions**
- **Masonry**
  - **Designer states assumed value of f_m'**
  - **Compliance is verified by "unit strength method" or by "prism test method"**

Verify Compliance with Specified f_m'

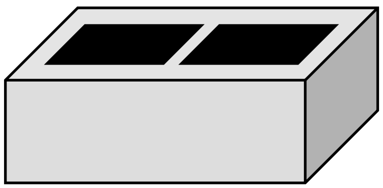
- Unit strength method (Spec 1.4 B 2):
  - Compressive strengths from unit manufacturer
  - ASTM C 270 mortar
  - Grout meeting ASTM C 476 or 2,000 psi
- Prism test method (Spec 1.4 B 3):
  - Pro -- can permit optimization of materials
  - Con -- require testing, qualified testing lab, and procedures in case of non-complying results

Example of Unit Strength Method (Specification Tables 1, 2)

- **Clay masonry units (Table 1):**
  - **Unit compressive strength ≥ 4150 psi**
  - **Type N mortar**
  - **Prism strength can be taken as 1500 psi**



- **Concrete masonry units (Table 2):**
  - **Unit compressive strength ≥ 1900 psi**
  - **Type S mortar**
  - **Prism strength can be taken as 1500 psi**



Application of Unit Strength Method (Spec Tables 1, 2)

- **Design determines required material specification:**
  - **Designer states assumed value of f_m'**
  - **Specifier specifies units, mortar and grout that will satisfy "unit strength method"**
- **Compliance with f_m' can be verified with no tests on mortar, grout, or prisms**

Comply with Specified Products and Execution

- **Products -- Specification Article 2:**
  - **Units, mortar, grout, accessory materials**
- **Execution -- Specification Article 3**
  - **Inspection**
  - **Preparation**
  - **Installation of masonry, reinforcement, grout, prestressing tendons**

Organization of MSJC Code Chapter 1

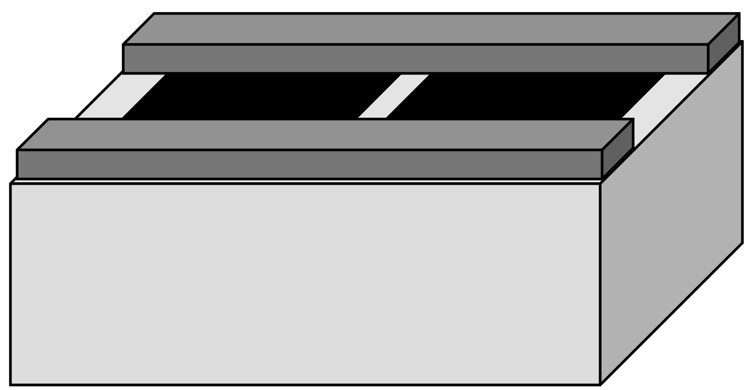
- 1.1 – 1.6 Scope, contract documents and calculations, special systems, reference standards, notation, definitions
- 1.7 Loading
- 1.8 Material properties
- 1.9 Section properties
- 1.10 Deflections
- 1.11 Stack bond masonry
- 1.12 Corbels
- 1.13 Details of reinforcement
- 1.14 Seismic design requirements
- 1.15 Quality assurance program
- 1.16 Construction

Code 1.8, Material Properties

- Chord modulus of elasticity, shear modulus, thermal expansion coefficients, and creep coefficients for clay, concrete, and AAC masonry
- Moisture expansion coefficient for clay masonry
- Shrinkage coefficients for concrete masonry

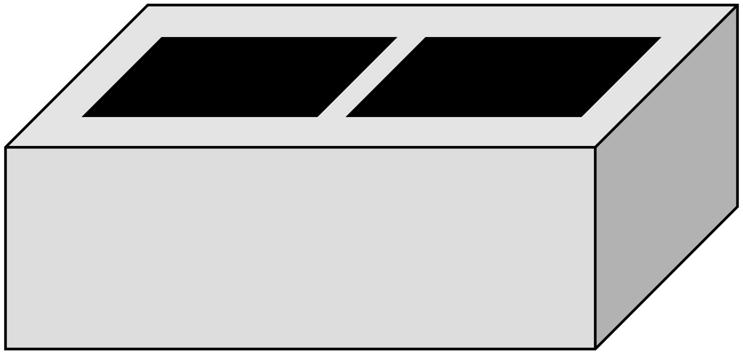
Code 1.9, Section Properties

- **Use minimum (critical) area for computing member stresses or capacities**
  - **Capacity is governed by the weakest section; for example, the bed joints of face-shell bedded hollow masonry**



Code 1.9, Section Properties

- **Radius of gyration and member slenderness are better represented by the average section; for example, the net area of units of face-shell bedded masonry**



Organization of MSJC Code Chapter 1

- 1.1 – 1.6 Scope, contract documents and calculations, special systems, reference standards, notation, definitions
- 1.7 Loading
- 1.8 Material properties
- 1.9 Section properties
- 1.10 Deflections
- 1.11 Stack bond masonry
- 1.12 Corbels
- 1.13 Details of reinforcement
- 1.14 Seismic design requirements
- 1.15 Quality assurance program
- 1.16 Construction

Code 1.13, Details of Reinforcement

- Reinforcing bars must be embedded in grout; joint reinforcement can be embedded in mortar
- Placement of reinforcement
- Protection for reinforcement
- Standard hooks

Organization of MSJC Code Chapter 1

1.1 – 1.6 Scope, Contract documents and calculations, special systems, reference standards, notation, definitions

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1.15 Quality assurance program

1.16 Construction

Code 1.14, Seismic Design

- Applies to all masonry except
  - Glass unit masonry
  - Veneers
- Seeks to improve performance of masonry structures in earthquakes
  - Improves ductility of masonry members
  - Improves connectivity of masonry members
- Different requirements for AAC masonry

Code 1.14, Seismic Design

- Define a structure's Seismic Design Category (SDC) according to ASCE 7-02
  – SDC depends on seismic risk (geographic location), importance, underlying soil
- SDC determines
  – Required types of shear walls (prescriptive reinforcement)
  – Prescriptive reinforcement for other masonry elements
  – Permitted design approaches for LFRS (lateral force-resisting system)

Code 1.14, Seismic Design

- Seismic design requirements are keyed to ASCE 7-02 Seismic Design Categories (from A up to F).
- Requirements are cumulative; requirements in each "higher" category are added to requirements in the previous category.

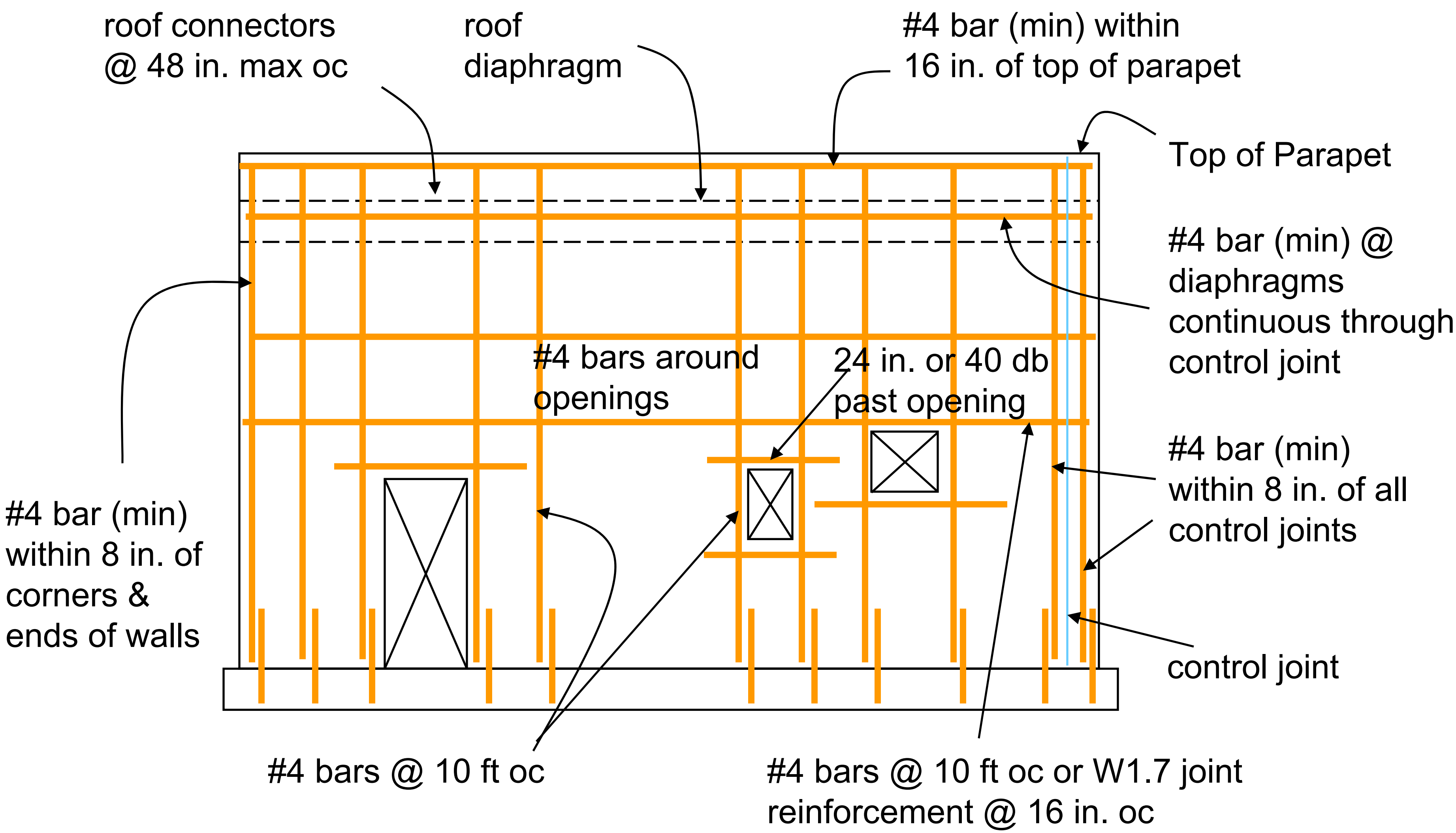
Code 1.14, Seismic Design

- **Seismic Design Category A:**
  - **Drift limit = 0.007**
  - **Minimum design connection force for wall-to roof and wall-to-floor connections**
- **Seismic Design Category B:**
  - **Lateral force resisting system cannot be designed empirically**

Code 1.14, Seismic Design

- Seismic Design Category C:
  - All walls must be considered shear walls unless isolated
  - Shear walls must meet minimum prescriptive requirements for reinforcement and connections (ordinary reinforced, intermediate reinforced, or special reinforced)
  - Other walls must meet minimum prescriptive requirements for horizontal or vertical reinforcement

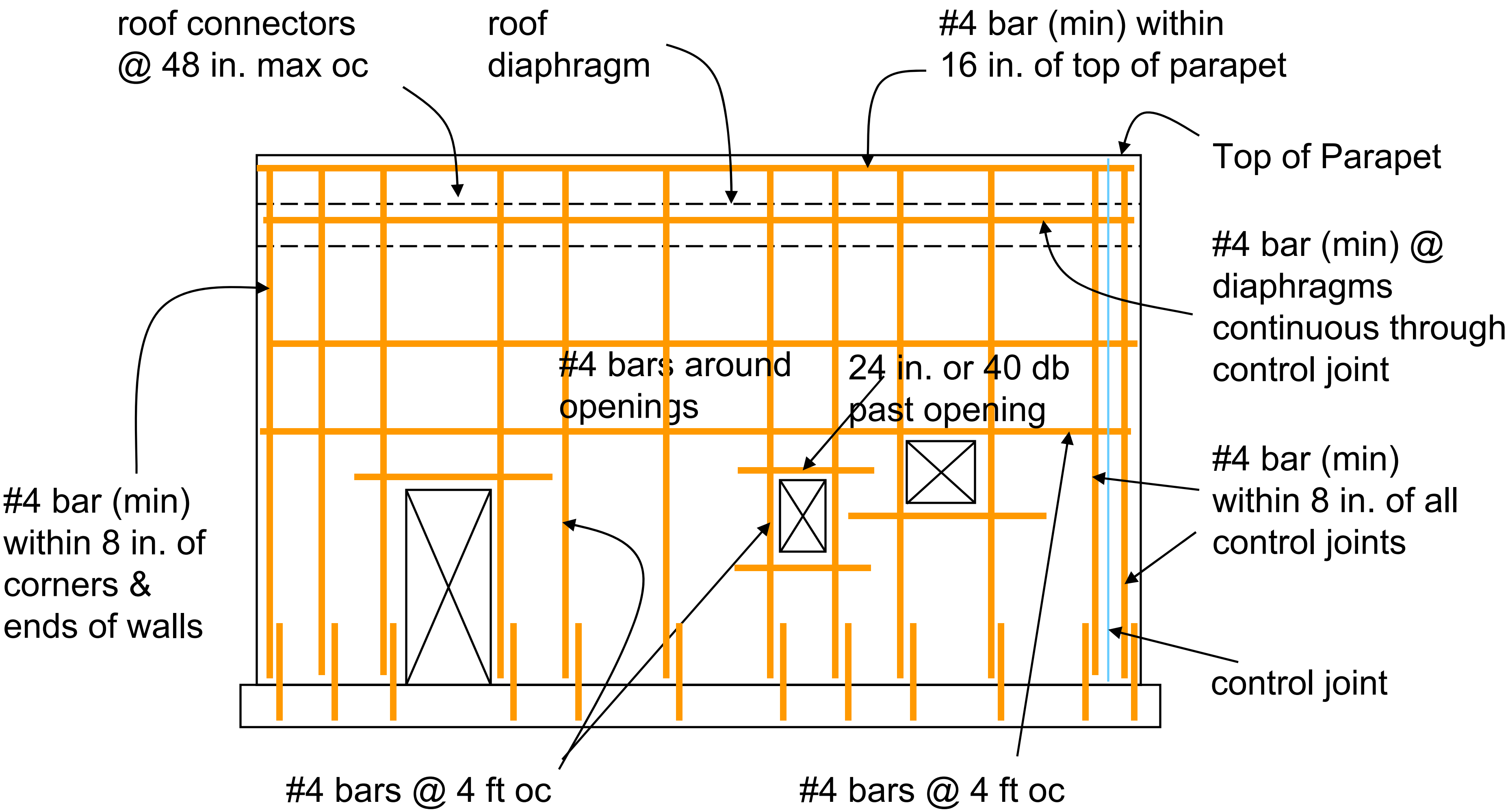
Minimum Reinforcement for Detailed Plain Shear Walls for SDC C



Code 1.14, Seismic Design

- Seismic Design Category D:
  - Masonry that is part of the lateral force-resisting system must be reinforced so that $\rho_v + \rho_h \geq 0.002$, and ρ_v and $\rho_h \geq 0.0007$
  - Type N mortar and masonry cement mortars are prohibited in the lateral force-resisting system
  - Shear walls must meet minimum prescriptive requirements for reinforcement and connections (special reinforced)
  - Other walls must meet minimum prescriptive requirements for horizontal and vertical reinforcement

Minimum Reinforcement for Special Reinforced Shear Walls



Code 1.14, Seismic Design

- Seismic Design Categories E and F:
  - Additional reinforcement requirements for stack-bond masonry

Minimum Reinforcement, SW Types

SW Type	Minimum Reinforcement	SDC
Empirically Designed	none	A
Ordinary Plain	none	A, B
Detailed Plain	Vertical reinforcement = 0.2 in.^{2} at corners, within 16 in. of openings, within 8 in. of movement joints, maximum spacing 10 ft; horizontal reinforcement W1.7 @ 16 in. or #4 in bond beams @ 10 ft	A, B
Ordinary Reinforced	same as above	A, B, C
Intermediate Reinforced	same as above, but vertical reinforcement @ 4 ft	A, B, C
Special Reinforced	same as above, but horizontal reinforcement @ 4 ft, and $\rho = 0.002$	any

Organization of MSJC Code Chapter 1

1.1 – 1.6 Scope, contract documents and calculations, special systems, reference standards, notation, definitions

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1.8 Material properties

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1.12 Corbels

1.13 Details of reinforcement

1.14 Seismic design requirements

1.15 Quality assurance program

1.16 Construction

Code 1.15, Quality Assurance

- Requires a quality assurance program in accordance with the MSJC Specification:
  - Three levels of quality assurance (A, B, C)
  - Compliance with specified f_m'
  - Increasing levels of quality assurance require increasingly strict requirements for inspection, and for compliance with specified products and execution

Code 1.15, Quality Assurance

- **Minimum requirements for inspection, tests, and submittals:**
  - **Empirically designed masonry, veneers, or glass unit masonry**
    - Table 1.14.1.1 for nonessential facilities
    - Table 1.14.1.2 for essential facilities
  - **Other masonry**
    - Table 1.14.1.2 for nonessential facilities
    - Table 1.14.1.3 for essential facilities

Organization of MSJC Code Chapter 1

- **1.1 – 1.6 Scope, contract documents and calculations, special systems, reference standards, notation, definitions**
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- **1.16 Construction**

1.16, Construction

- Minimum grout spacing (Table 1.16.2)
- Embedded conduits, pipes, and sleeves:
  - Consider effect of openings in design
  - Masonry alone resists loads
- Anchorage of masonry to structural members, frames, and other construction:
  - Show type, size, and location of connectors on drawings

... Organization of MSJC Code Chapter 3, Strength Design (SD)

- Fundamental basis
- Loading combinations
- Design strength
- Deformation requirements
- *Φ*-factors
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

Fundamental Basis for Strength Design

- Factored design actions must not exceed nominal capacities, reduced by Φ factors
- Quotient of load factor divided by the Φ factor is analogous to safety factor of allowable-stress design, and should be comparable to that safety factor.

Organization of MSJC Code Chapter 3, Strength Design

- Fundamental basis
- Loading combinations
- Design strength
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Code 3.1.2, Loading Combinations for SD

- From governing building code
- From ASCE 7-02

Organization of MSJC Code Chapter 3

- Fundamental basis
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- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
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- Reinforced masonry

Code 3.1.3, Design Strength for SD

- Design strength must exceed required strength
- Extra caution against brittle shear failure:
  - Design shear strength shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength
  - Nominal shear strength need not exceed 2.5 times required shear strength

Organization of MSJC Code
Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- *Φ* factors
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- Bearing strength
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- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

Code 3.1.4, Strength-reduction Factors for SD

Action	Reinforced Masonry	Unreinforced Masonry
Combinations of flexure and axial load	0.90	0.60
Shear	0.80	0.80
Anchorage and splices of Reinforcement	0.80	---
Bearing	0.60	0.60

Code 3.1.4, Strength-reduction Factors for SD

Capacity of Anchor Bolts as Governed by	Strength-reduction Factor
Steel yield and fracture	0.90
Masonry breakout	0.50
Pullout of bent-bar anchors	0.65

Organization of MSJC Code Chapter 3

- Fundamental basis
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- Unreinforced masonry
- Reinforced masonry

Code 3.1.5, Deformation Requirements

- Drift limits from ASCE 7-02
- Deflections of unreinforced masonry (URM) based on uncracked sections
- Deflections of reinforced masonry (RM) based on cracked sections

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
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- Reinforced masonry

Code 3.1.6, Anchor Bolts

- Tensile capacity governed by:
  - Tensile breakout
  - Yield of anchor in tension
  - Tensile pullout (bent-bar anchor bolts only)
- Shear capacity governed by:
  - Shear breakout
  - Yield of anchor in shear
- For combined tension and shear, use linear interaction

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
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- *Φ* factors
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Reinforced masonry
- Unreinforced masonry

Code 3.1.7.1.1, Compressive Strength of Masonry

- For concrete masonry, 1,500 psi $\leq f_m' \leq$ 4,000 psi
- For clay masonry, 1,500 psi $\leq f_m' \leq$ 6,000 psi

Code 3.1.7.1.2, Compressive Strength of Grout

- **For concrete masonry, $f_m' \leq f_g' \leq$ 5,000 psi**
- **For clay masonry, $f_g' \leq$ 6,000 psi**

Organization of MSJC Code Chapter 3

- Fundamental basis
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- Bearing strength
- Compressive strength
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- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

Code 3.1.8.2, Modulus of Rupture

- **In-plane and out-of-plane bending**
  - **Table 3.1.8.2.1**
  - **Lower values for masonry cement and air-entrained portland cement-lime mortar**
  - **Higher values for grouted masonry**
  - **For grouted stack-bond masonry, f_r = 250 psi parallel to bed joints for continuous horizontal grout section**

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- Deformation requirements
- *Φ* factors
- Anchor bolts
- Bearing strength
- Compressive strength
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- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

Code 3.1.8.3, Strength of Reinforcement

- $f_y \leq$ **60 ksi**
- **Actual yield strength shall not exceed 1.3 times the specified value**
- **Compressive strength of reinforcement shall be ignored unless the reinforcement is tied in compliance with Code 2.1.6.5**

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- Deformation requirements
- *Φ* factors
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

Code 3.3, Reinforced Masonry

- **Masonry in flexural tension is cracked**
- **Reinforcing steel is needed to resist tension**
- **Similar to strength design of reinforced concrete**

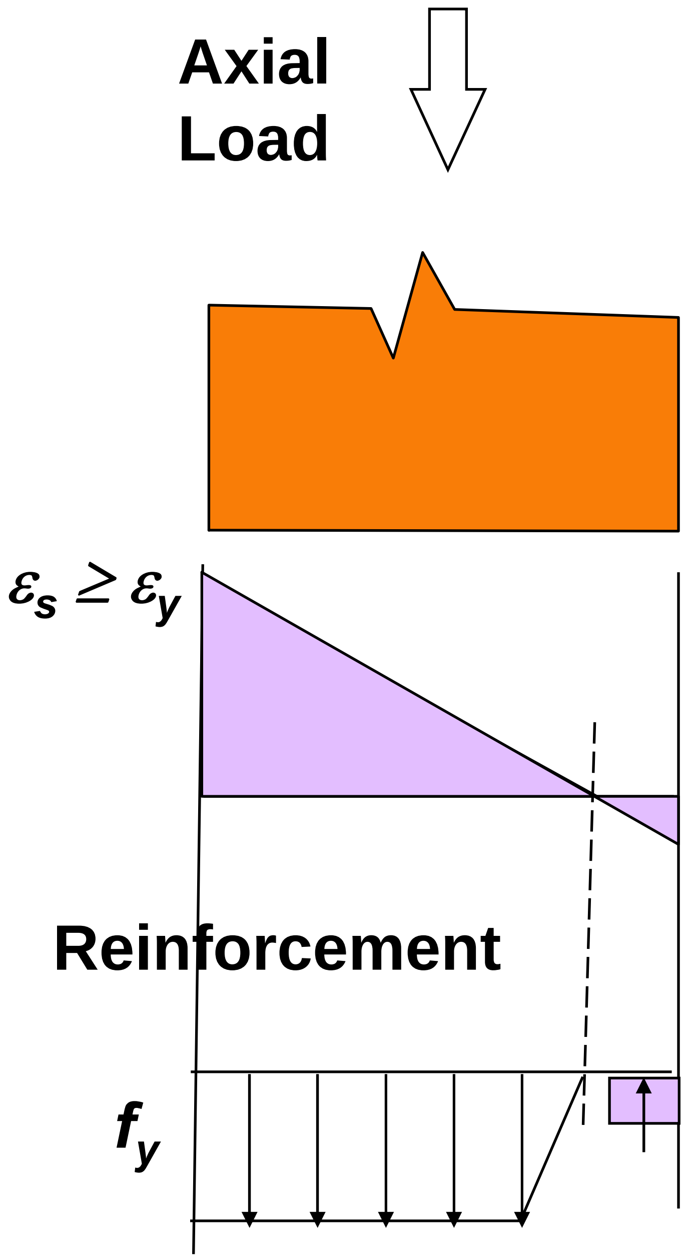
Code 3.3, Reinforced Masonry

- 3.3.2 Design assumptions
- 3.3.3 Reinforcement requirements and details, including maximum steel percentage
- 3.3.4 Design of piers, beams and columns:
  - Nominal axial and flexural strength
  - Nominal shear strength
- 3.3.5 Design of walls for out-of-plane loads
- 3.3.6 Design of walls for in-plane loads

Code 3.3.2, Design Assumptions

- Continuity between reinforcement and grout
- Equilibrium
- ε_{mu} = 0.0035 for clay masonry, 0.0025 for concrete masonry
- Plane sections remain plane
- Elasto-plastic stress-strain curve for reinforcement
- Tensile strength of masonry is neglected
- Equivalent rectangular compressive stress block in masonry, with a height of 0.80 f_m' and a depth of 0.80 c

Flexural Assumptions



- Locate neutral axis based on extreme-fiber strains
- Calculate compressive force, *C*
- $P = C - T$
- $M = \sum F_i y_i$ (y_i from plastic centroid)

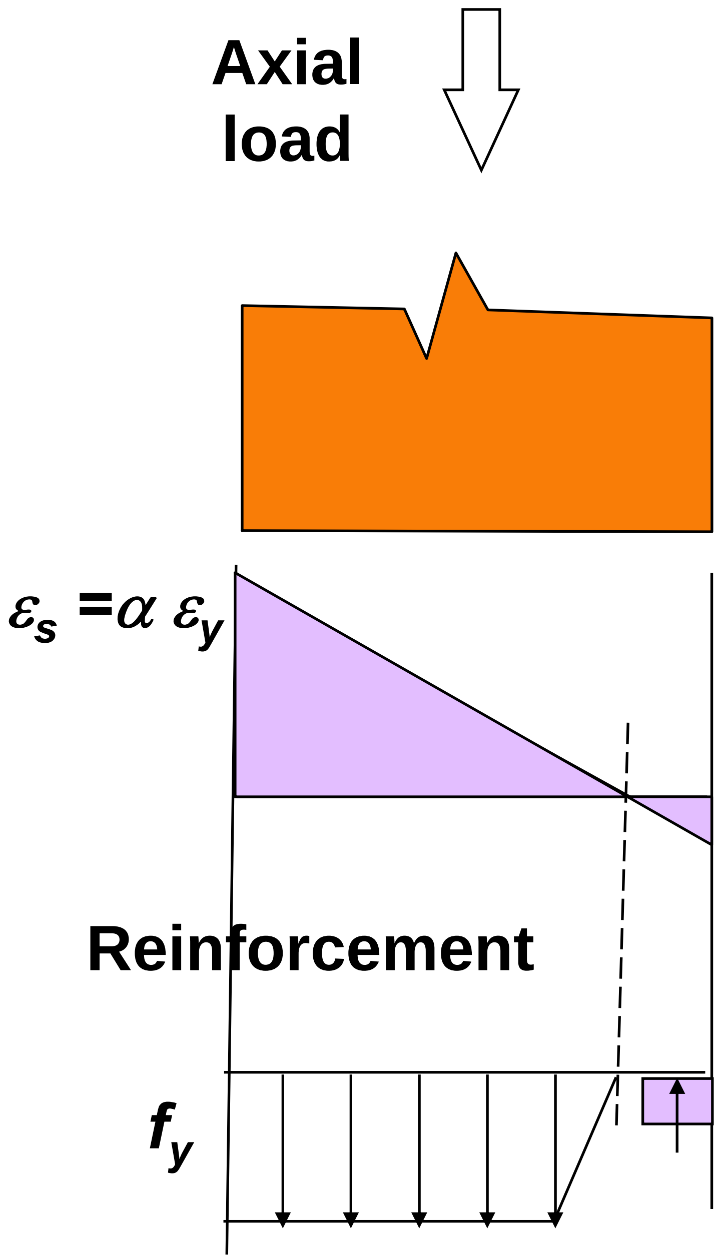
$\varepsilon_{mu} =$ 0.0035 clay
0.0025 concrete

$0.80\, f_m'$
$\beta_1 = 0.80$

Code 3.3.3, Reinforcement Requirements and Details

- **Bar diameter ≤ 1 / 8 nominal wall thickness**
- **Standard hooks and development length:**
  - **Development length based on pullout and splitting**
- **In walls, shear reinforcement must be bent around extreme longitudinal bars**
- **Splices:**
  - **Lap splices based on required development length**
  - **Welded and lap splices must develop 1.25 f_y**

Code 3.3.3.5, Maximum Reinforcement



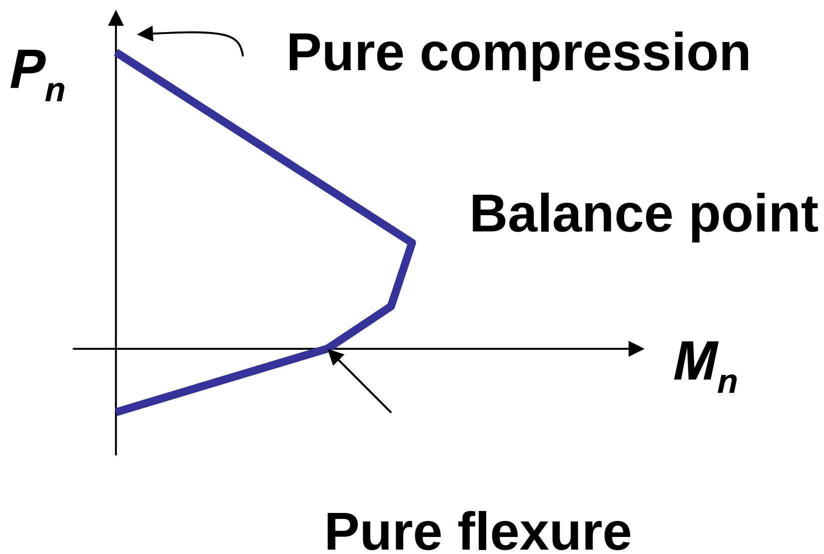
- **Locate neutral axis based on extreme-fiber strains**
- **Calculate compressive force, *C* (can include compressive reinforcement)**
- **Reinforcement + axial Load = C**

ε_{mu} = **0.0035 clay**
0.0025 concrete

0.80 f_m'
$\beta_1 = 0.80$

Code 3.3.4, Design of Beams, Piers, and Columns

- Capacity under combinations of flexure and axial load is based on the assumptions of Code 3.3.2 (interaction diagram)



Code 3.3.4, Design of Beams, Piers, and Columns

- **Slenderness is addressed by multiplying axial capacity by slenderness-dependent modification factors**

$$\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{for } \frac{h}{r} \le 99$$

$$\left(\frac{70r}{h}\right)^2 \quad \text{for } \frac{h}{r} > 99$$

Code 3.3.4, Nominal Shear Strength

- $V_n = V_m + V_s$
- V_n shall not exceed:
  - $M / V d_v \leq 0.25$ $\qquad V_n \leq 6 A_n \sqrt{f_m'}$
  - $M / V d_v \geq 1.0$ $\qquad V_n \leq 4 A_n \sqrt{f_m'}$
  - Linear interpolation between these extremes
  - Objective is to avoid crushing of diagonal strut

Code 3.3.4, Nominal Shear Strength

- V_m **and** V_s **are given by:**

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u \qquad (3-21)$$

$$\left(\frac{M_u}{V_u d_v}\right) \leq 1.0$$

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v \qquad (3-22)$$

Code 3.3.4.2, Requirements for Beams

- $P_u \leq 0.05\, A_n\, f_m'$
- $M_n \geq 1.3\, M_{cr}$
- **Lateral bracing spaced at most 32 times beam width**
- **Nominal depth not less than 8 in.**

Code 3.3.4.3, Requirements for Piers

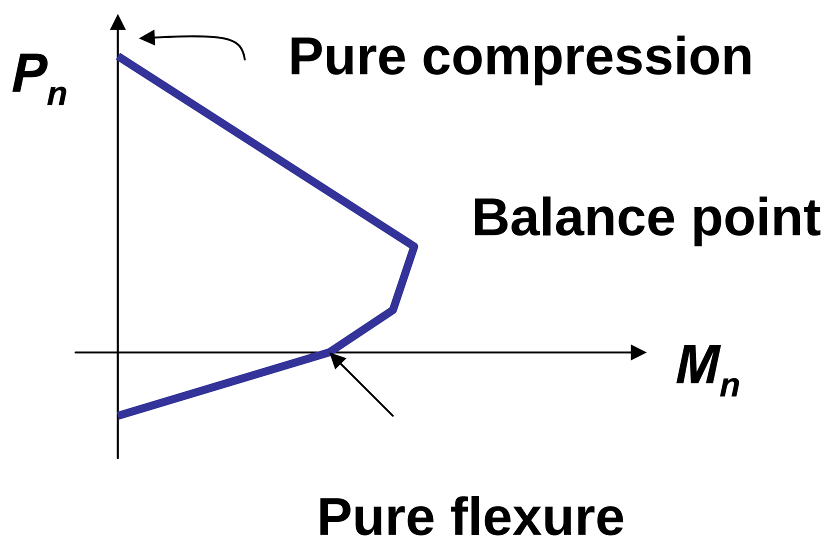
- Isolated elements (wall segments are not piers)
- $P_u \leq 0.3\, A_n\, f_m'$
- Nominal thickness between 6 and 16 in.
- Nominal plan length between 3 and 6 times the nominal thickness
- Clear height not more than 5 times the nominal plan length

Code 3.3.4.4, Requirements for Columns

- **Isolated elements (wall segments are not columns)**
- $\rho_g \geq 0.0025$
- $\rho_g \leq 0.04$, **and also meet Code 3.3.3.5**
- **Lateral ties in accordance with Code 2.1.6.5**
- **Solid-grouted**
- **Least cross-section dimension** $\geq$ **8 in.**
- **Nominal depth not greater than 3 times the nominal width**

Code 3.3.5, Design of Walls for Out-of-plane Loads

- **Capacity under combinations of flexure and axial load is based on the assumptions of Code 3.3.2 (interaction diagram)**

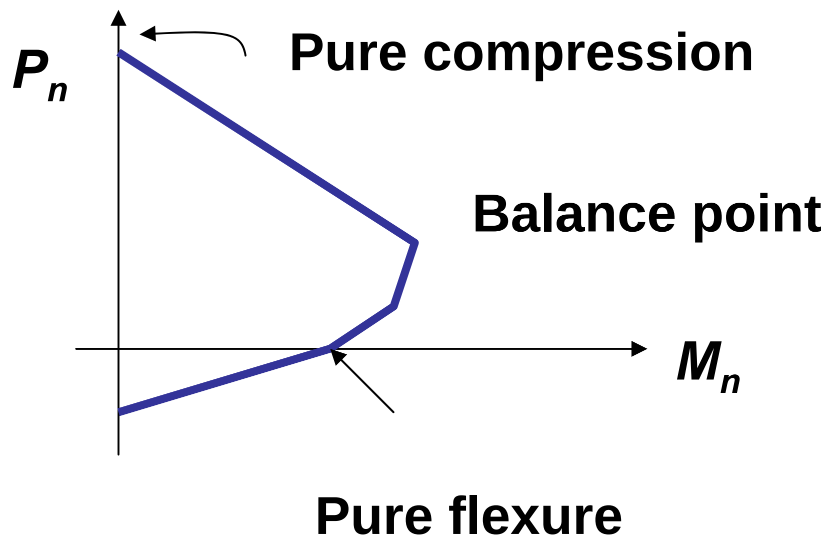


Code 3.3.5, Design of Walls for Out-of-plane Loads

- Maximum reinforcement by Code 3.3.3.5
- Procedures for computing out-of-plane moments and deflections (moment magnifier, vary depending on axial load)
- Nominal shear strength by Code 3.3.4.1.2

Code 3.3.6, Design of Walls for In-plane Loads

- **Capacity under combinations of flexure and axial load is based on the assumptions of Code 3.3.2 (interaction diagram)**



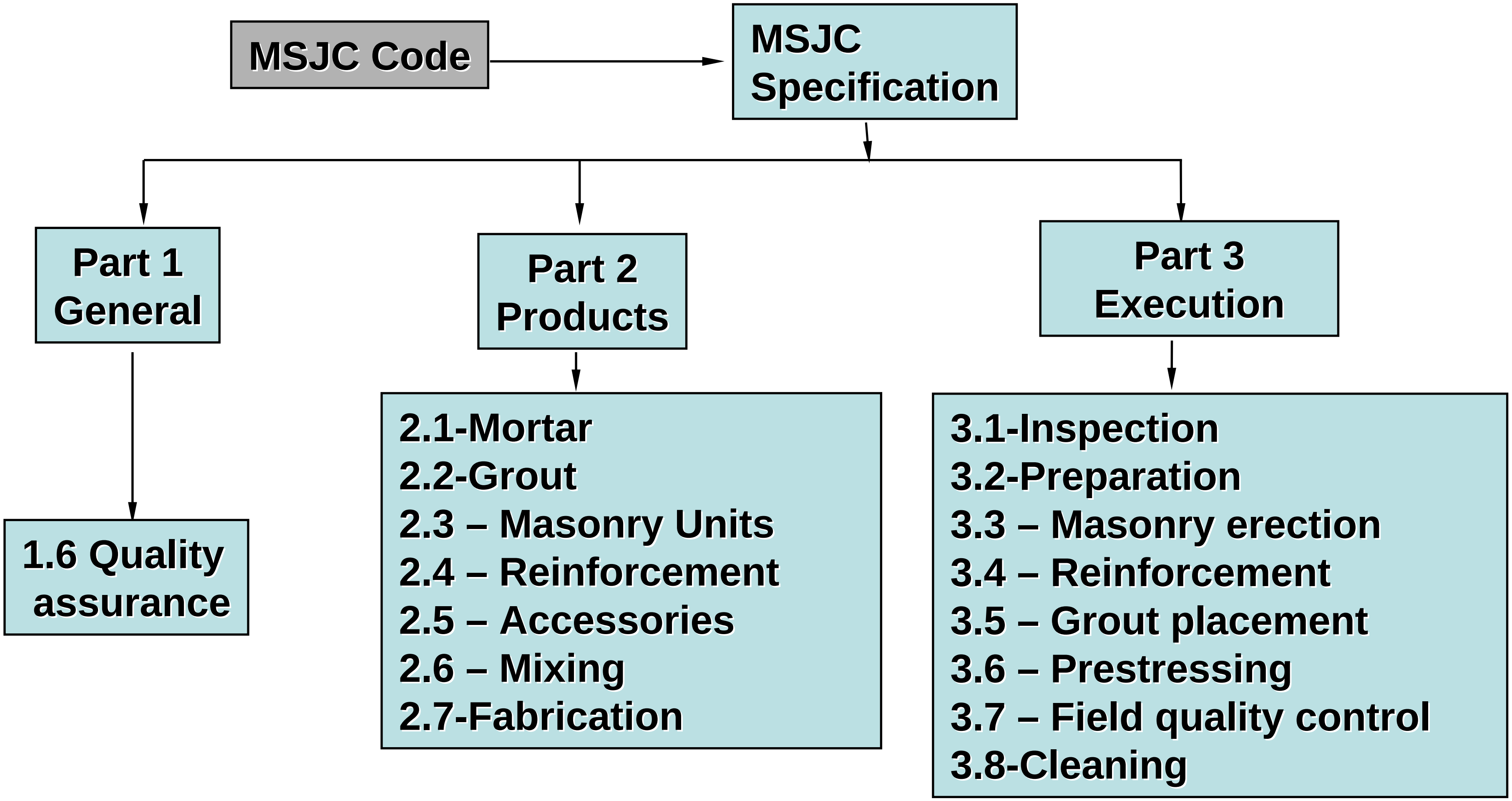
Code 3.3.6, Design of Walls for In-plane Loads

- **Maximum reinforcement by Code 3.3.3.5**
- **Vertical reinforcement not less than one-half the horizontal reinforcement**
- **Nominal shear strength by Code 3.3.4.1.2**

Code 3.3.6, Alternative Approach to Maximum Reinforcement

- **For walls expected to have flexural ductility in plane, provide confined boundary elements in hinging regions (this is another way of preventing toe crushing)**
- **Detailing requirements for boundary elements have yet to be developed**

Organization of MSJC Specification

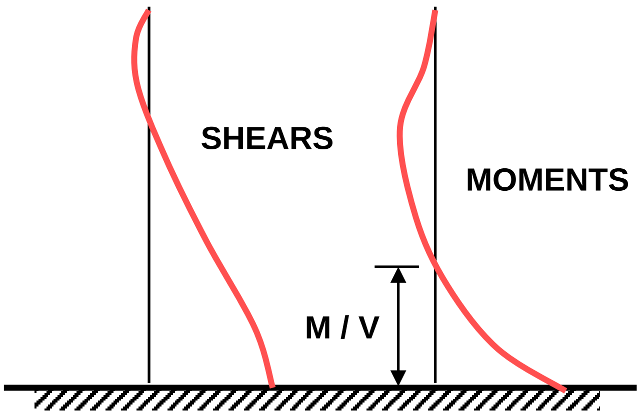


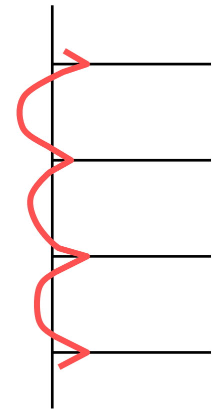
Strength Design of Reinforced Masonry Shear Walls

- Compute factored design moments and shears for in- and out-of-plane loading.
- Given practical thickness for wall, design flexural reinforcement as governed by out-of-plane loading.
- Design flexural reinforcement as governed by in-plane loading and revise design as necessary.
- Check shear capacity using capacity design if required.
- Check detailing.

Compute Factored Design Moments and Shears

- Factored design moments and shears for in-plane loading depend on actions transferred to shear walls by horizontal diaphragms at each floor level.
- Factored design moments and shears for out-of-plane loading depend on wind or earthquake forces acting between floor levels



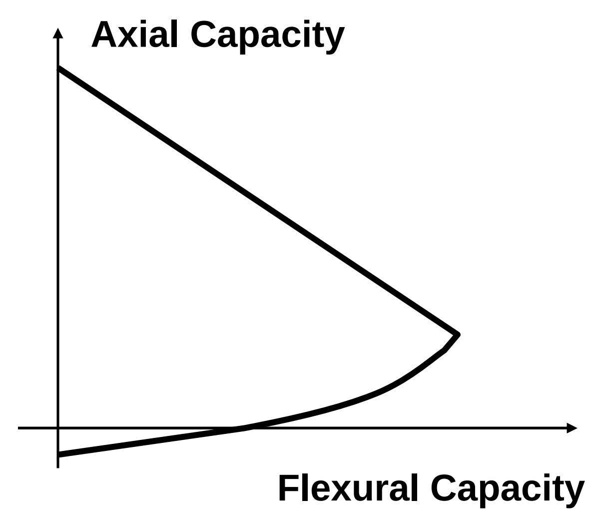


Design Flexural Reinforcement as Governed by Out-of-plane Loading

- Practical wall thickness is governed by available unit dimensions:
  - 8- by 8- by 16-in. nominal dimensions
  - Specified thickness = 7-5/8 in.
  - One curtain of bars, placed in center of grouted cells
- Practical wall thickness = 7-5/8 in.
- Proportion flexural reinforcement to resist out-of-plane wind or earthquake forces

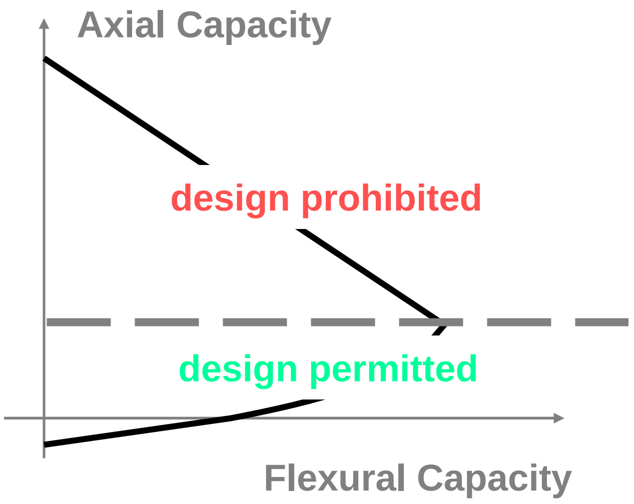
Design Flexural Reinforcement as Governed by In-plane Loading

- **Construct moment – axial force interaction diagram**
  - **Initial estimate (more later)**
  - **Computer programs**
  - **Spreadsheets**
  - **Tables**



Strict Limits on Maximum Flexural Reinforcement

- Objective -- Keep compressive stress block from crushing:
  - Walls must be below balance point.
  - Maximum steel percentage decreases as axial load increases, so that design above balance point is impossible.

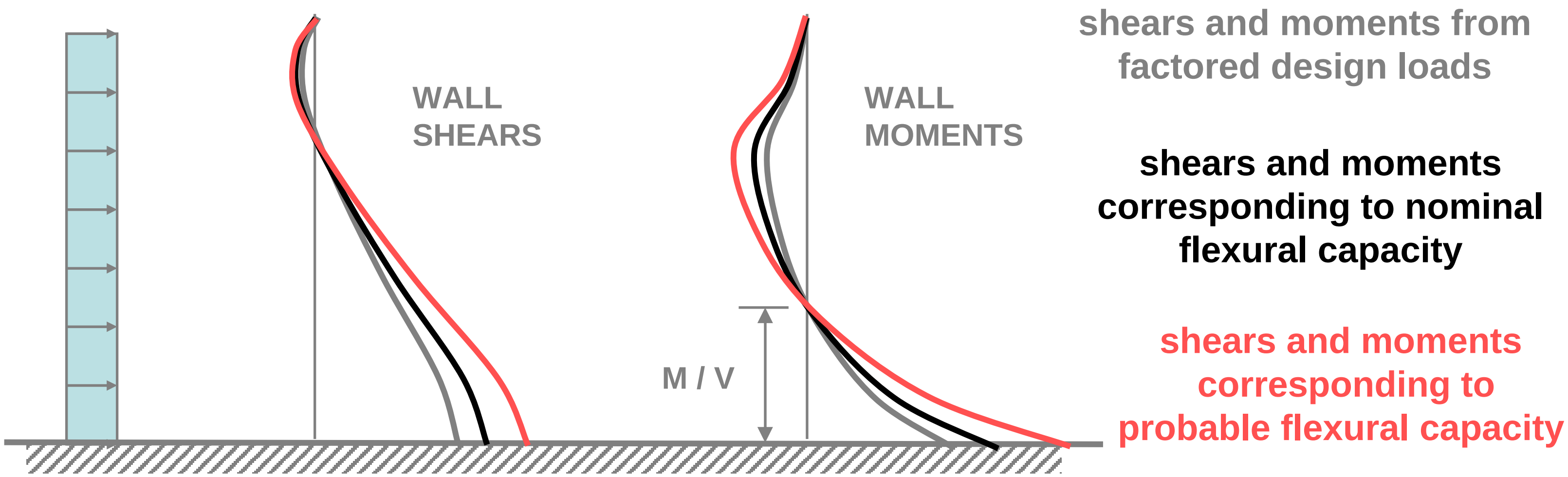


Revise Design as Necessary

- If flexural reinforcement required for out-of-plane moments is less than or equal to that required for in-plane moments, no adjustment is necessary. Use the larger amount.
- If flexural reinforcement required for out-of-plane moments exceeds that required for in-plane moments, consider making the wall thicker so that in-plane flexural capacity does not have to be increased. Excess in-plane capacity increases shear demand.

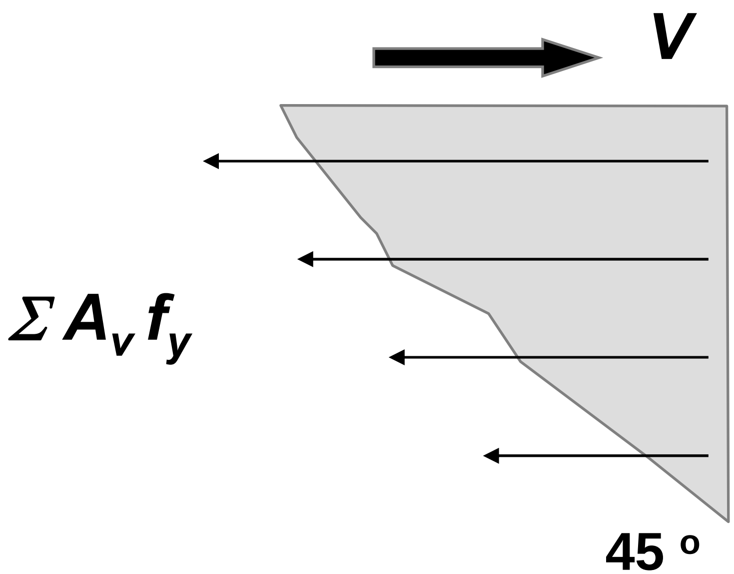
Check Shear Capacity (1)

- Elastic structures or those with considerable shear overstrength:
  - Compute factored design shears based on factored design actions.
- Inelastic structures:
  - Compute design shears based on flexural capacity



Check Shear Capacity (2)

- $V_n = V_m + V_s$
- V_m depends on $(M_u / V_u d_v)$ ratio
- $V_s = (0.5)\, A_v f_y$ (note efficiency factor)

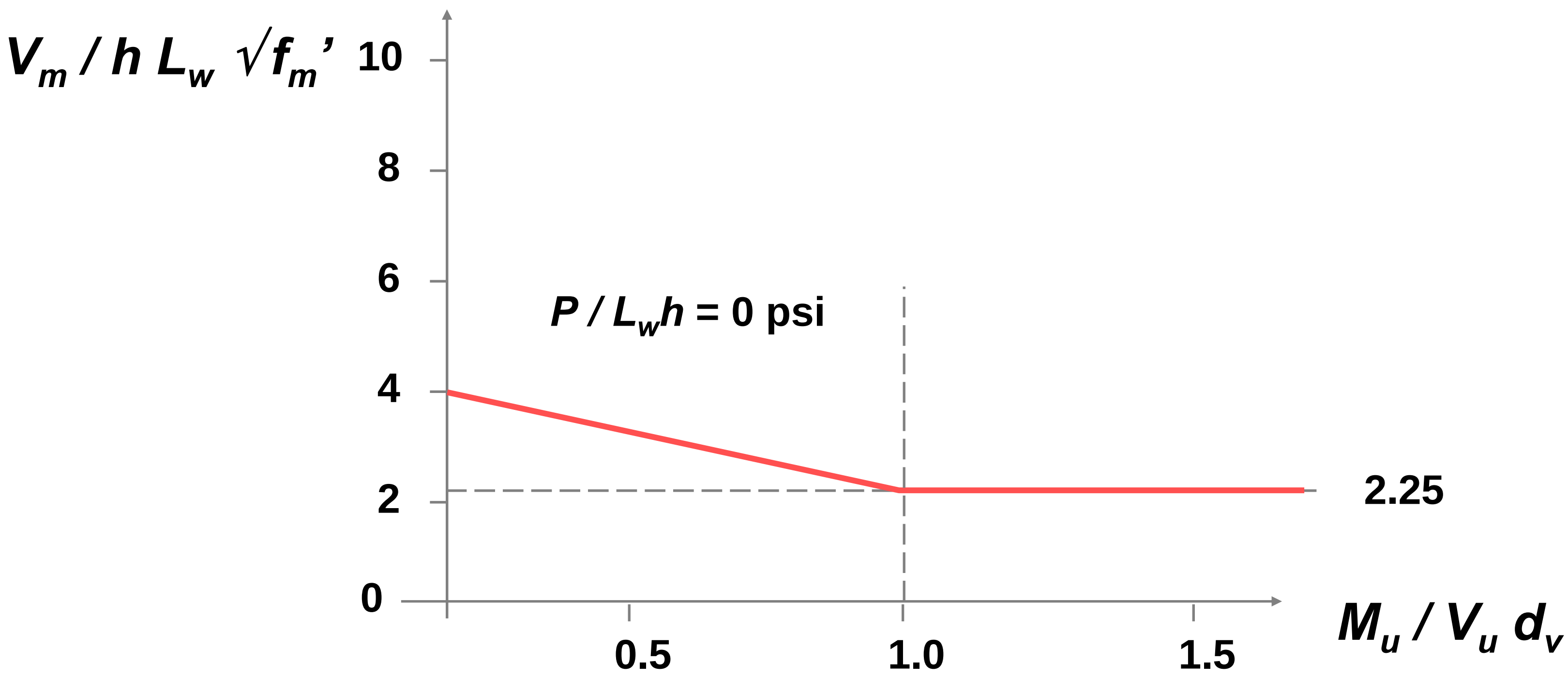


Shear Resistance from Masonry, V_m (1)

- V_m **depends on (**$M_u / V_u d_v$**) ratio and axial force**
- **(**$M_u / V_u d_v$**) need not be taken greater than 1.0**

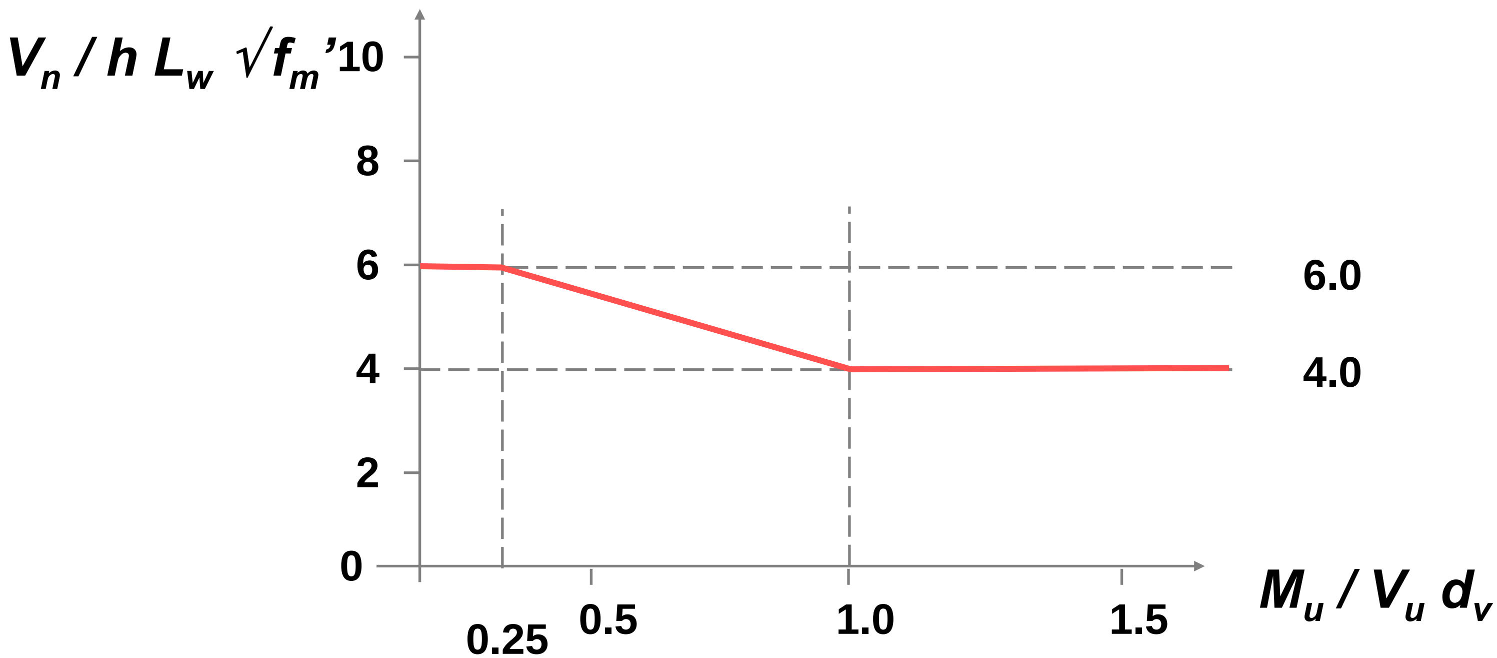
$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u\, d_v}\right)\right] A_n \sqrt{f_m'} + 0.25P_u$$

Shear Resistance from Masonry, V_m (2)



Total Shear Resistance, V_n (1)

- Resistance from masonry (V_m) plus resistance from reinforcement (V_s)
- Upper limit on V_m depends on ($M_u / V_u d_v$) ratio



Check Detailing

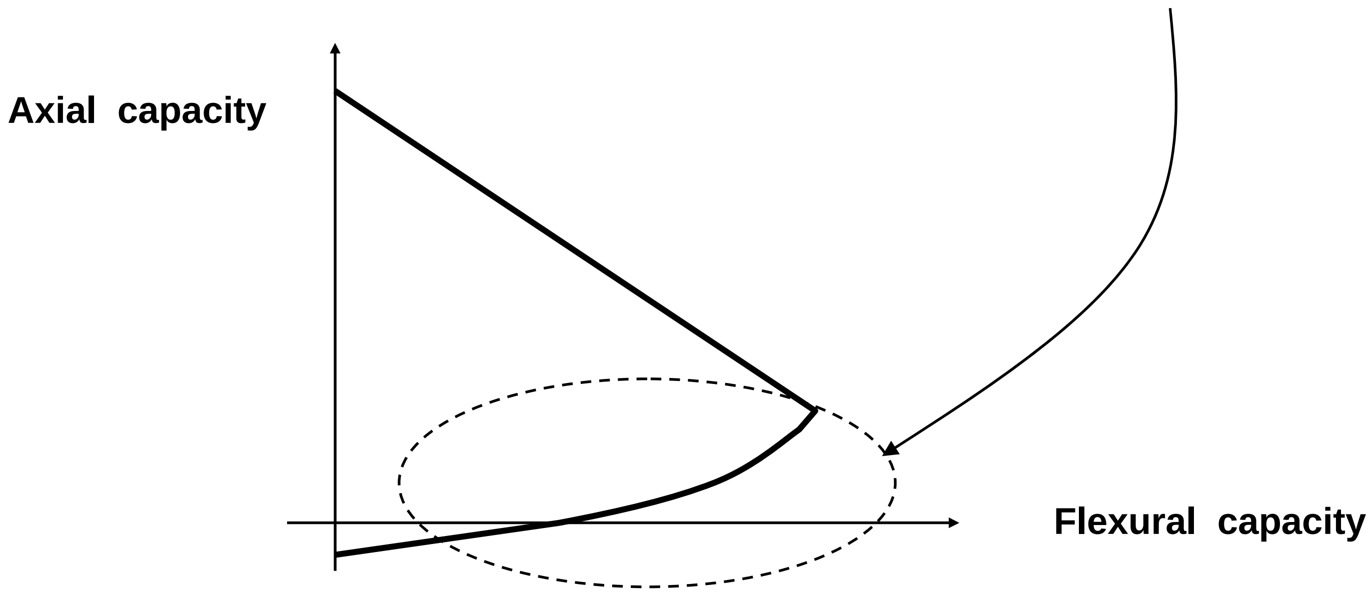
- Cover
- Placement of flexural and shear reinforcement
- Boundary elements not required:
  - Ductility demand is low
  - Maximum flexural reinforcement is closely controlled

Detailing (1)

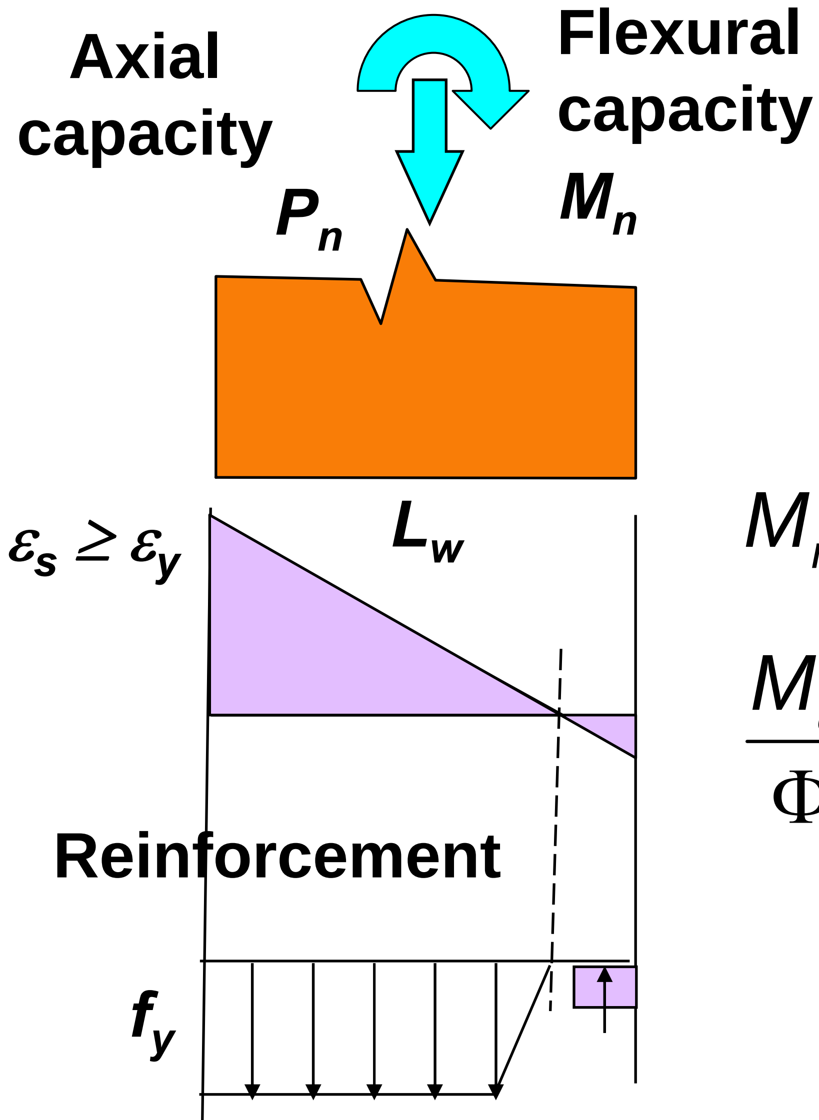
- **Cover:**
  - **Automatically satisfied by putting reinforcement in grouted cells**
- **Placement of flexural and shear reinforcement:**
  - **Minimum flexural reinforcement and spacing dictated by Seismic Design Category**
  - **Flexural reinforcement placed in single curtain. Typical reinforcement would be at least #4 bars @ 48 in.**
  - **Place horizontal reinforcement in single curtain. Typical reinforcement would be at least #4 bars @ 48 in.**
  - **Add more flexural reinforcement if required, usually uniformly distributed.**

Flexural Strength of Lineal Walls (1)

- **Approximation to moment-axial force interaction diagram for low axial load**



Flexural Strength of Lineal Walls (2)



- **Sum moments about centroid of compressive stress block**

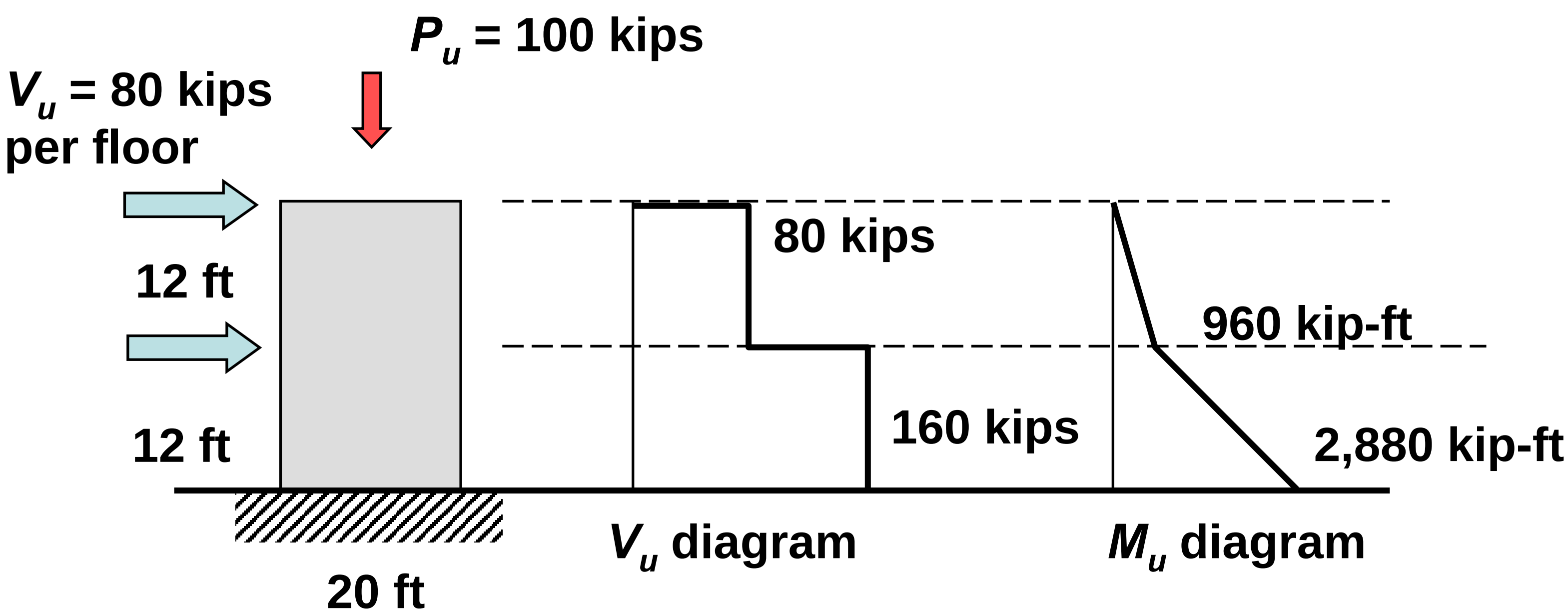
$$M_n \approx 0.9 A_s f_y \frac{0.9L_w}{2} + P_n \frac{0.9L_w}{2}$$

$$\frac{M_u}{\Phi} \approx 0.41 A_s f_y L_w + 0.45 \frac{P_u}{\Phi} L_w$$

- **Given M_u and P_u, solve for A_s**

Design Example (1)

- **Carry out the preliminary design of the masonry shear wall shown below. Use f_m' = 1500 psi.**



Design Example (2)

- **Assume out-of-plane flexure is OK.**
- **Check in-plane flexure using initial estimate.**

$$\frac{M_u}{\Phi} \approx 0.41 A_s \, f_y \, L_w + 0.45 \frac{P_u}{\Phi} L_w$$

$$\frac{2880 \; kip - ft \times 12 \; in./ft}{0.90} \approx 0.41 A_s \times 60 \; ksi \times 240 \; in. + 0.45 \frac{100 \; kips}{0.9} \times 240 \; in.$$

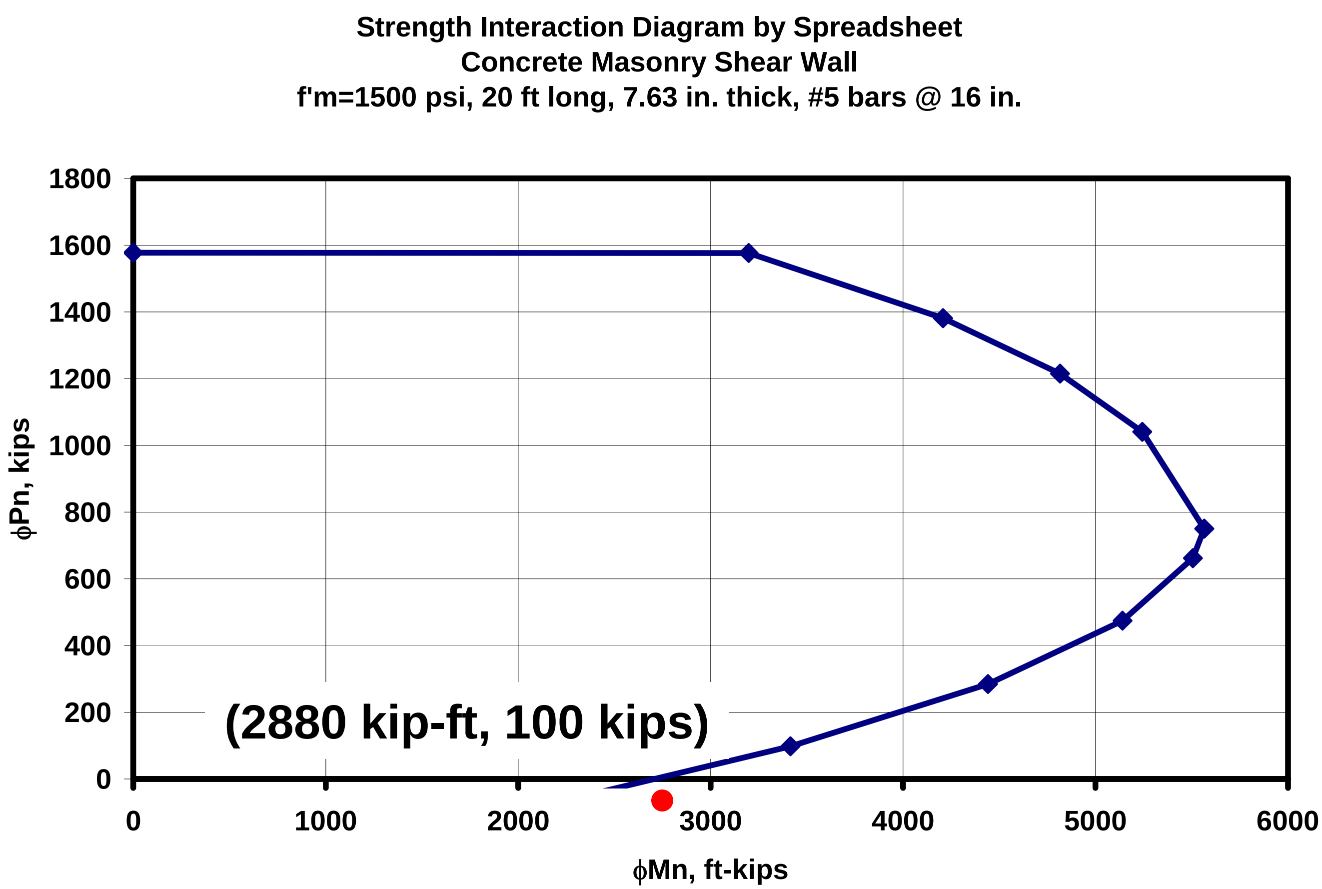
$$38,400 \approx 5,900 \; A_s + 12,000$$

$$A_s \approx 4.47 \; in.^2$$

- **This is equivalent to #5 bars @ 12 in.**

Design Example (3)

- **Refine flexural reinforcement using spreadsheet-based interaction diagram -- use #5 bars @ 16 in.**



Design Example (4)

- **Now check shear:**

$$\frac{M_u}{V_u\, d} = \frac{2{,}880\ kip - ft}{160\ kips \cdot 20 \cdot 0.8\ ft} = 1.13$$

$$V_m = 2.25\sqrt{f_m^{'}}\ h\, L_w + 0.25P$$

$$V_m = 2.25\sqrt{1500} \cdot 7.63\ in. \cdot 240\ in. + 0.25 \cdot 100\ kips$$

$$V_m = 159.6\ kips + 25.0\ kips = 184.6\ kips$$

Design Example (5)

- **Compute required shear reinforcement, including capacity design:**

$$V_u = 160\ kips \left(\frac{1.25\, M_n}{M_u} \right)$$

$$V_u = 160\ kips \left[\frac{1.25 \times 3427\ kip-ft \times \left(\dfrac{1}{0.9} \right)}{2880\ kip-ft} \right] = 160\ kips \times 1.65$$

$$V_n^{required} = \frac{V_u}{\Phi} = \frac{V_u}{0.8} = \frac{160\ kips \times 1.65}{0.8} = 2.07\, V_u \leq 2.5\, V_u$$

Design Example (6)

- **Compute required shear reinforcement including capacity design:**

$$V_s^{required} \geq \frac{V_u}{\Phi} - V_m = \frac{160 \times 1.65}{0.8} - 184.6 = 145.4 \; kips$$

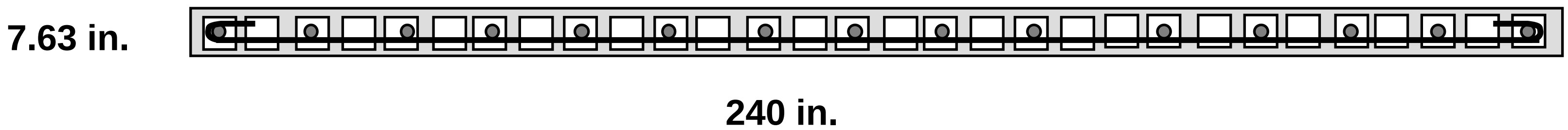
$$V_s^{required} = 2 \times A_v f_y \frac{d}{s}$$

$$A_v^{required} = \frac{V_s^{required} s}{d \; f_y} = \frac{145.4 \; kips \cdot 16 \; in.}{0.8 \cdot 240 \; in. \cdot 60 \; ksi} = 0.202 \; in.^2$$

- **Use #4 bars every 16 in.**

Design Example (7)

- **Now finish detailing:**
  - **Use #5 bars @ 16 in. vertically**
  - **Use #4 bars @ 16 in. horizontally**
  - **Hook #4 horizontal bars around end #5 vertical bars**



Web sites for more information

- BSSC = www.bssconline.org
- TMS = www.masonrysociety.org
- ACI = www.aci-int.org
- ASCE / SEI = www.seinstitute.org
- MSJC = www.masonrystandards.org

Acknowledgements

- This material on masonry was prepared by Prof. Richard E. Klingner, University of Texas at Austin.
- Some of the material was originally prepared by Prof. Klingner for a US Army short course.
- Some of the material was originally prepared by Prof. Klingner for The Masonry Society and is used with their permission.

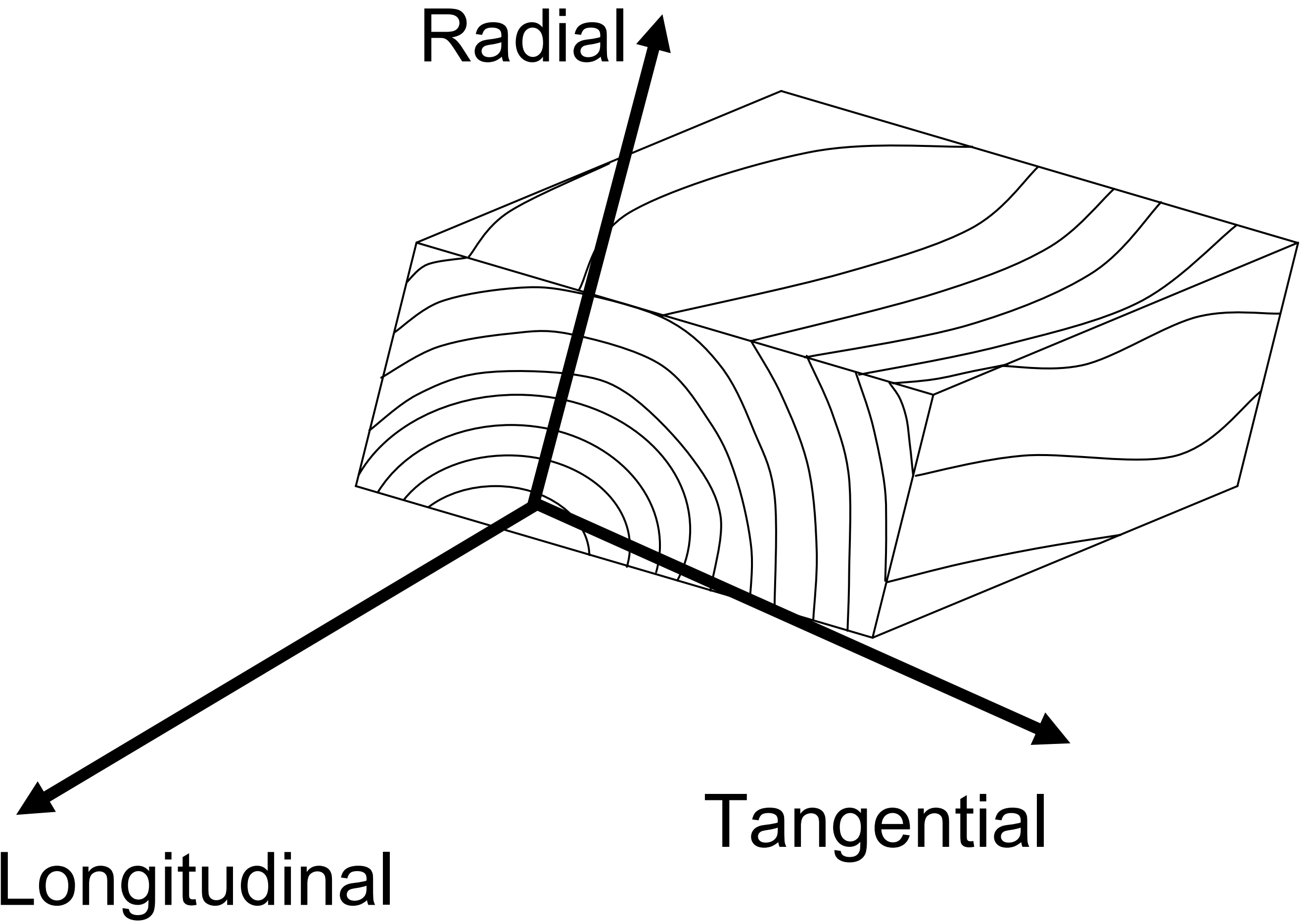
WOOD STRUCTURES

Objectives of Topic

Understanding of:

- **Basic wood behavior**
- **Typical framing methods**
- **Main types of lateral force resisting systems**
- **Expected response under lateral loads**
- **Sources of strength, ductility and energy dissipation**
- **Basic shear wall construction methods**
- **Shear wall component behavior**
- **Analysis methods**
- **Code requirements**

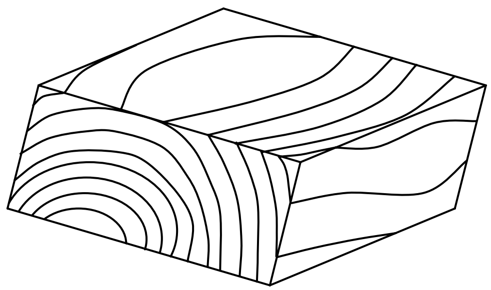
Basic Wood Material Properties



Wood is orthotropic

- **Varies with moisture content**
- **Main strength axis is longitudinal - parallel to grain**
- **Unique, independent, mechanical properties in 3 different directions**
- **Radial and tangential are "perpendicular" to the grain – substantially weaker**

Basic Wood Material Properties



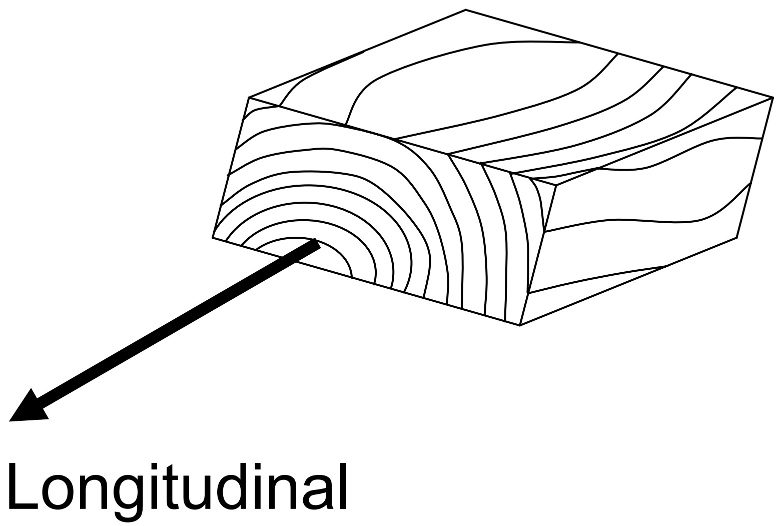
"Timber is as different from wood as concrete is from cement."

– Madsen, Structural Behaviour of Timber

Concept of "wood" as "clear wood": design properties used to be derived from clear wood with adjustments for a range of "strength reducing characteristics."

- Concept of "timber" as the useful engineering and construction material: "In-grade" testing (used now) determines engineering properties for a specific grade of timber based on full-scale tests of timber, a mixture of clear wood and strength reducing characteristics.

Basic Wood Material Properties



Sample DFL longitudinal design properties:

- **Modulus of elasticity: 1,800,000 psi**
- **Tension (parallel to grain): 1,575 psi**
- **Bending: 2,100 psi**
- **Compression (parallel to grain): 1875 psi**

Basic Wood Material Properties

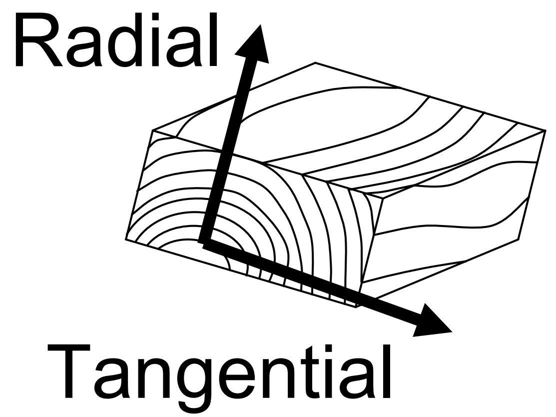
Sample DFL perpendicular to grain design properties:

- **Modulus of elasticity: 45,000 psi (2.5 ~ 5 % of $E_{||}$!)**
- **Tension (perpendicular to grain): 180 to 350 psi**

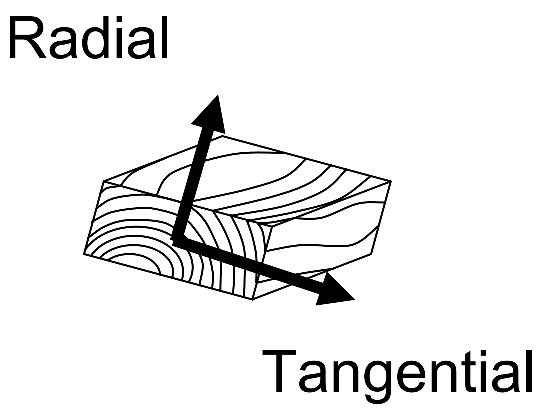
<u>FAILURE</u> stresses. Timber is extremely weak for this stress condition. It should be avoided if at all possible, and mechanically reinforced if not avoidable.

- **Compression (perpendicular to grain): 625 psi.**

Note that this is derived from a serviceability limit state of ~ 0.04" permanent deformation under stress in contact situations. This is the most "ductile" basic wood property.



Basic Wood Material Properties



<u>Shrinkage</u>

- **Wood will shrink with changes in moisture content.**
- **This is most pronounced in the radial and tangential directions (perpendicular to grain).**
- **May need to be addressed in the LFRS.**

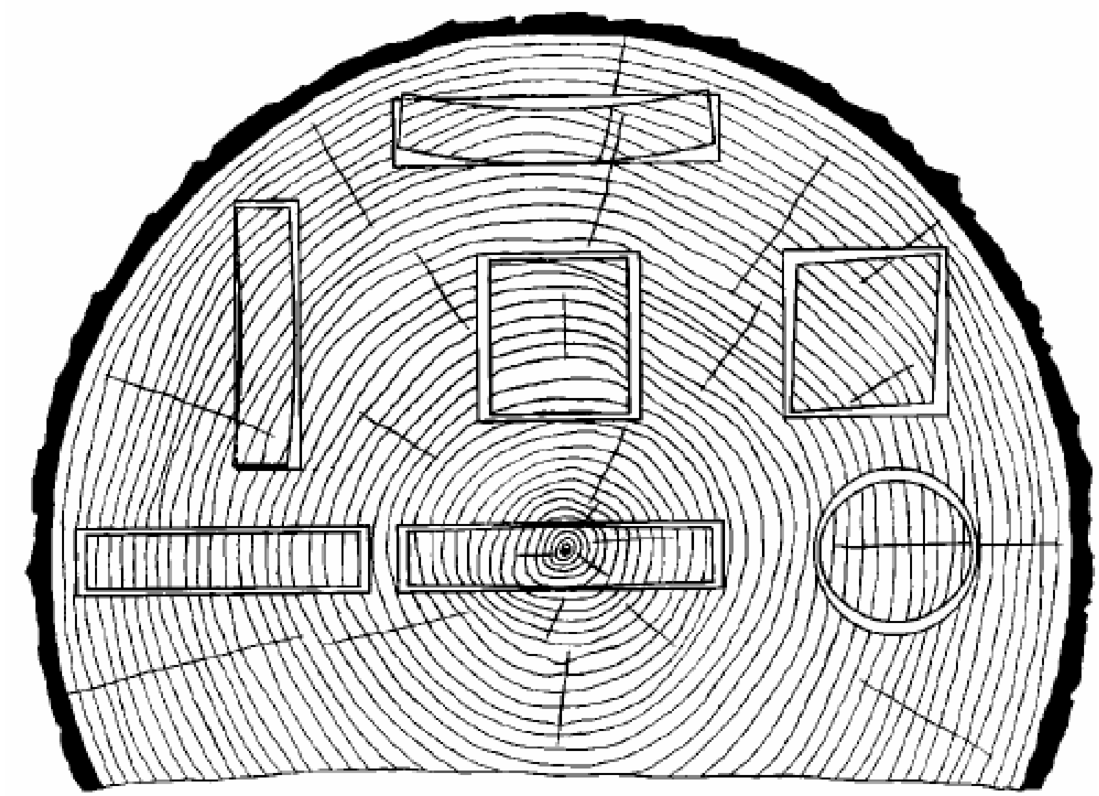
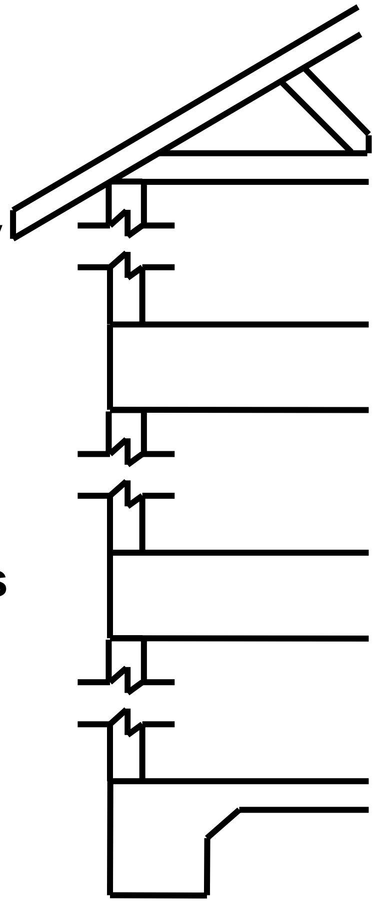


Figure 3–3. Characteristic shrinkage and distortion of flat, square, and round pieces as affected by direction of growth rings. Tangential shrinkage is about twice as great as radial.

(Wood Handbook, p. 58)

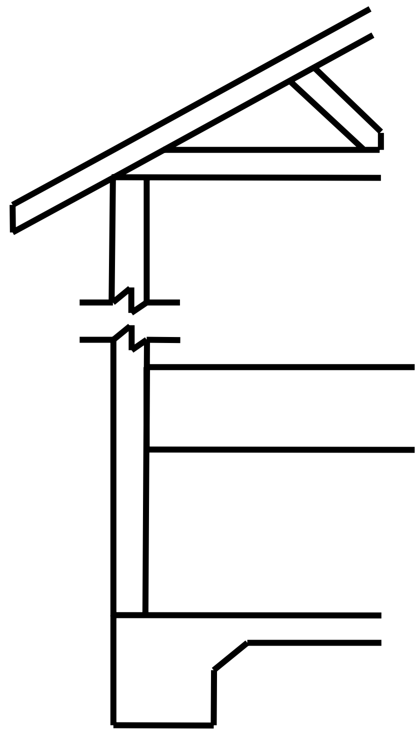
Wood Structure Construction Methods: Gravity

Platform



- Walls are interrupted by floor "platforms."
- Floors support walls.
- Most common type of light-frame construction today.
- Economical but creates discontinuity in the load path.
- Metal connectors essential for complete load path.

Balloon



- Walls feature foundation to roof framing members.
- Floors supported by ledgers on walls or lapped with studs.
- Not very common today.

Wood Structure Construction Methods: Gravity

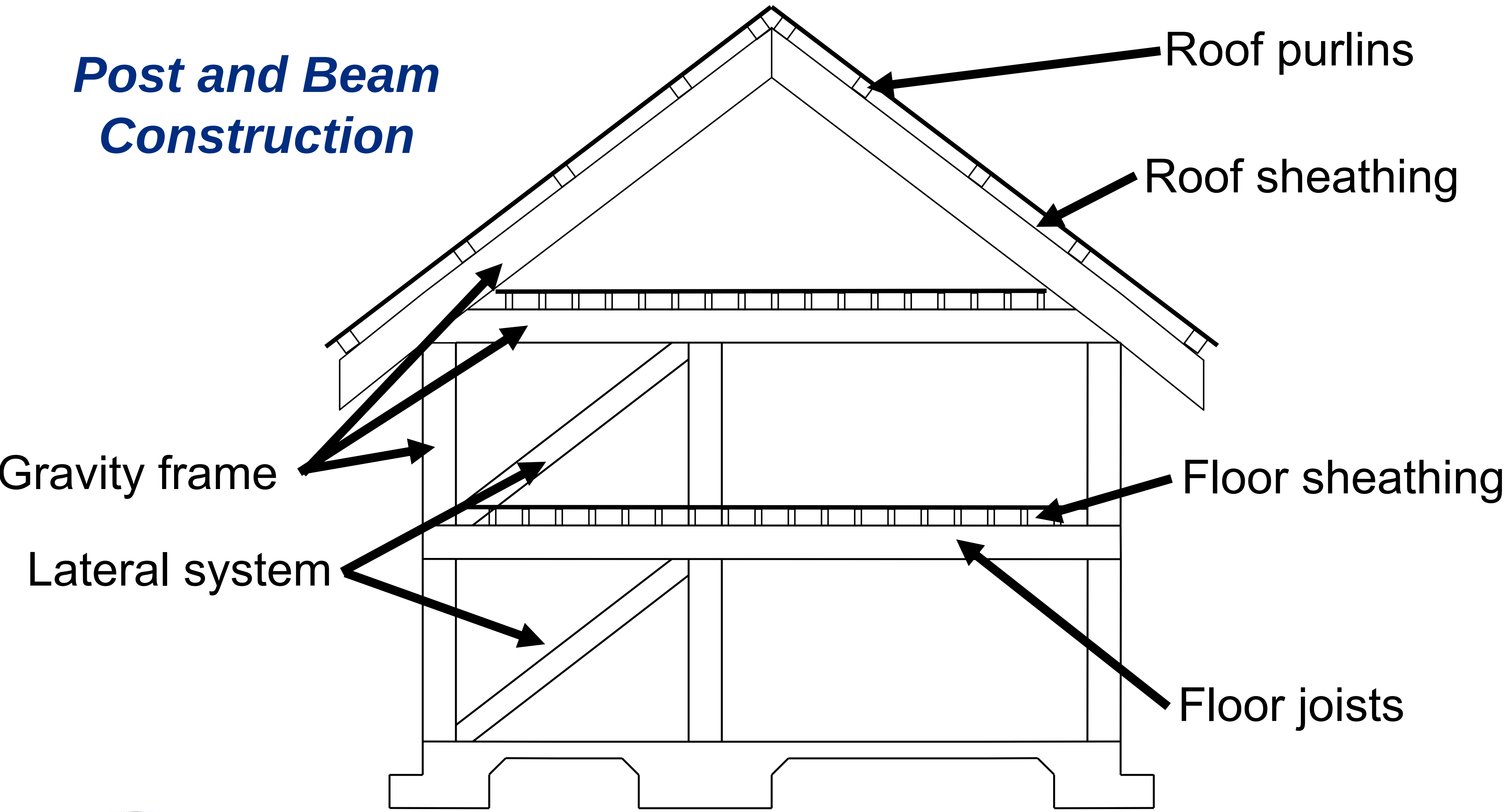
Post and Beam

- Space frame for gravity loads.
- Moment continuity at joint typically only if member is continuous through joint.
- Lateral resistance through vertical diaphragms or braced frames.
- Knee braces as seen here for lateral have no code design procedure for seismic.

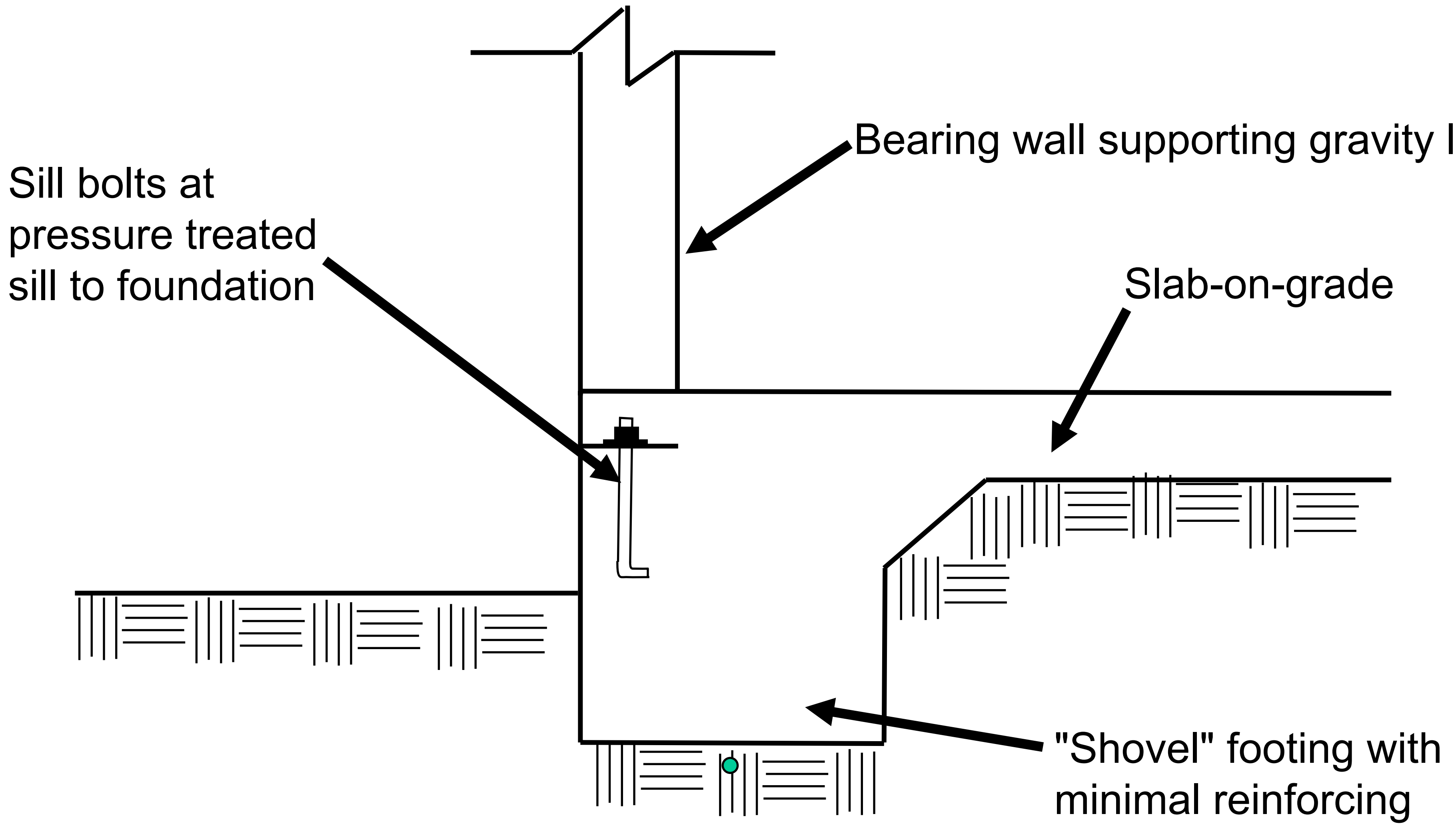
Six story main lobby Old Faithful Inn, Yellowstone, undergoing renovation work in 2005. Built in winter of 1903-1904, it withstood a major 7.5 earthquake in 1959.

Wood Structure Construction Methods: Gravity

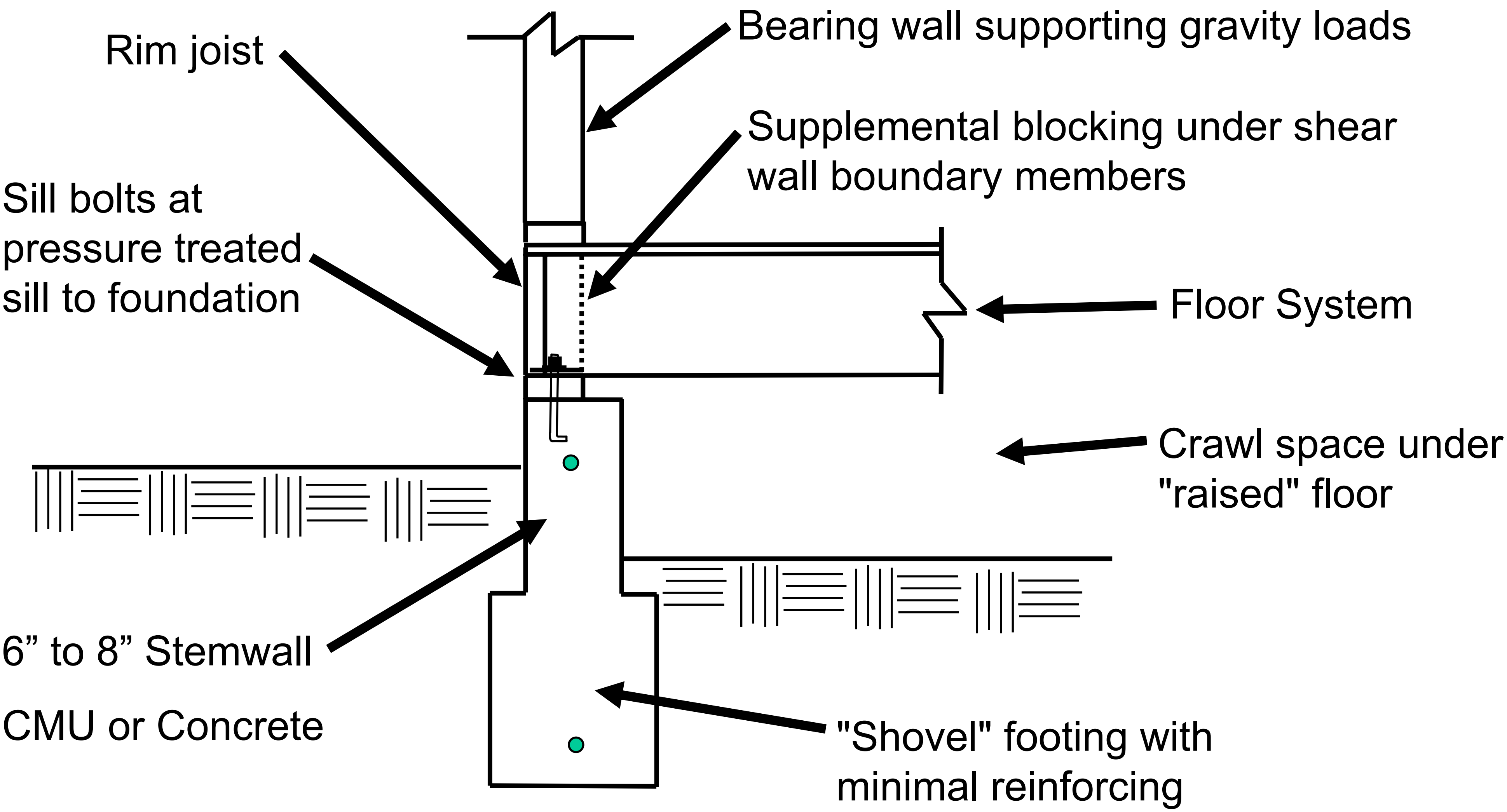
Post and Beam Construction



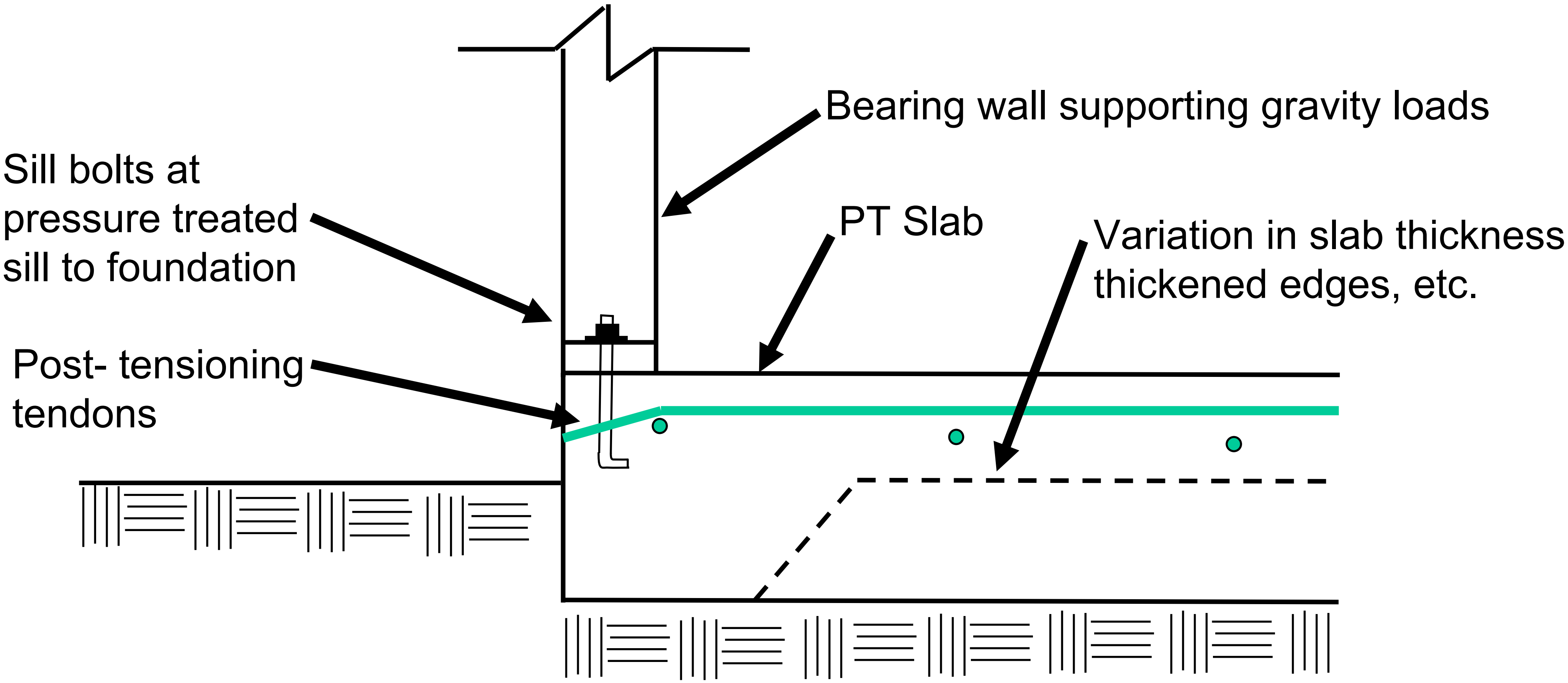
Typical Light-Frame Foundation: Slab-On-Grade



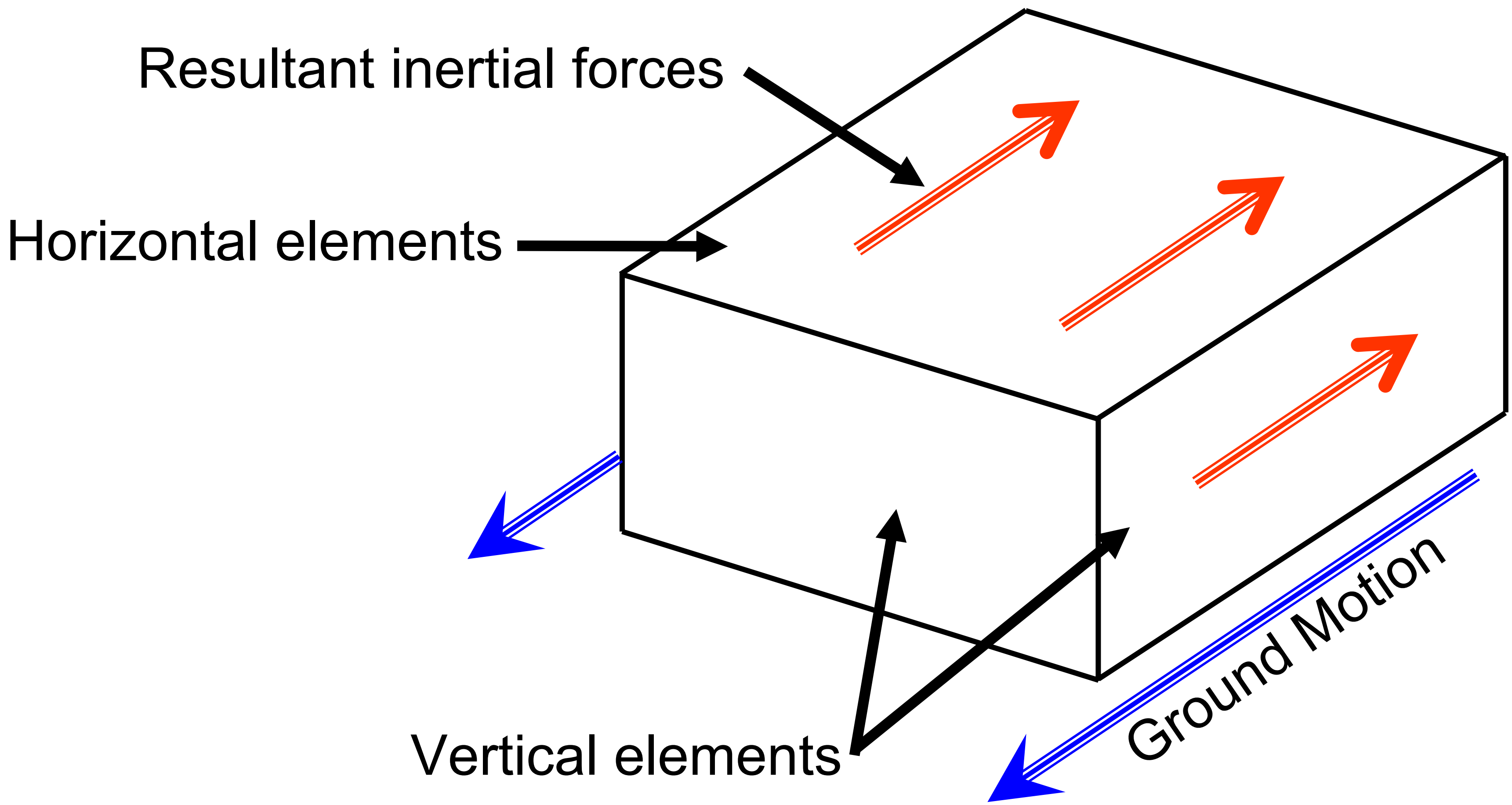
Typical Light-Frame Foundation: Raised Floor



Typical Light-Frame Foundation: Post Tensioning



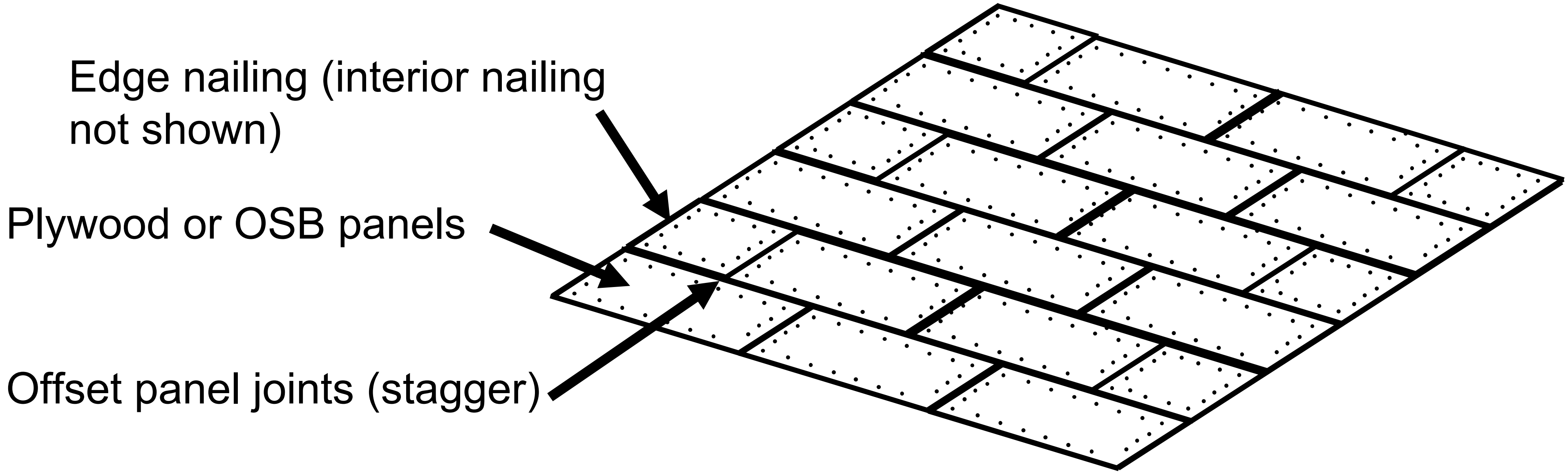
Wood Structure Construction Methods: Lateral



- The basic approach to the lateral design of wood structures is the same as for other structures.

Wood Structure Construction Methods: Lateral

Horizontal elements of LFRS

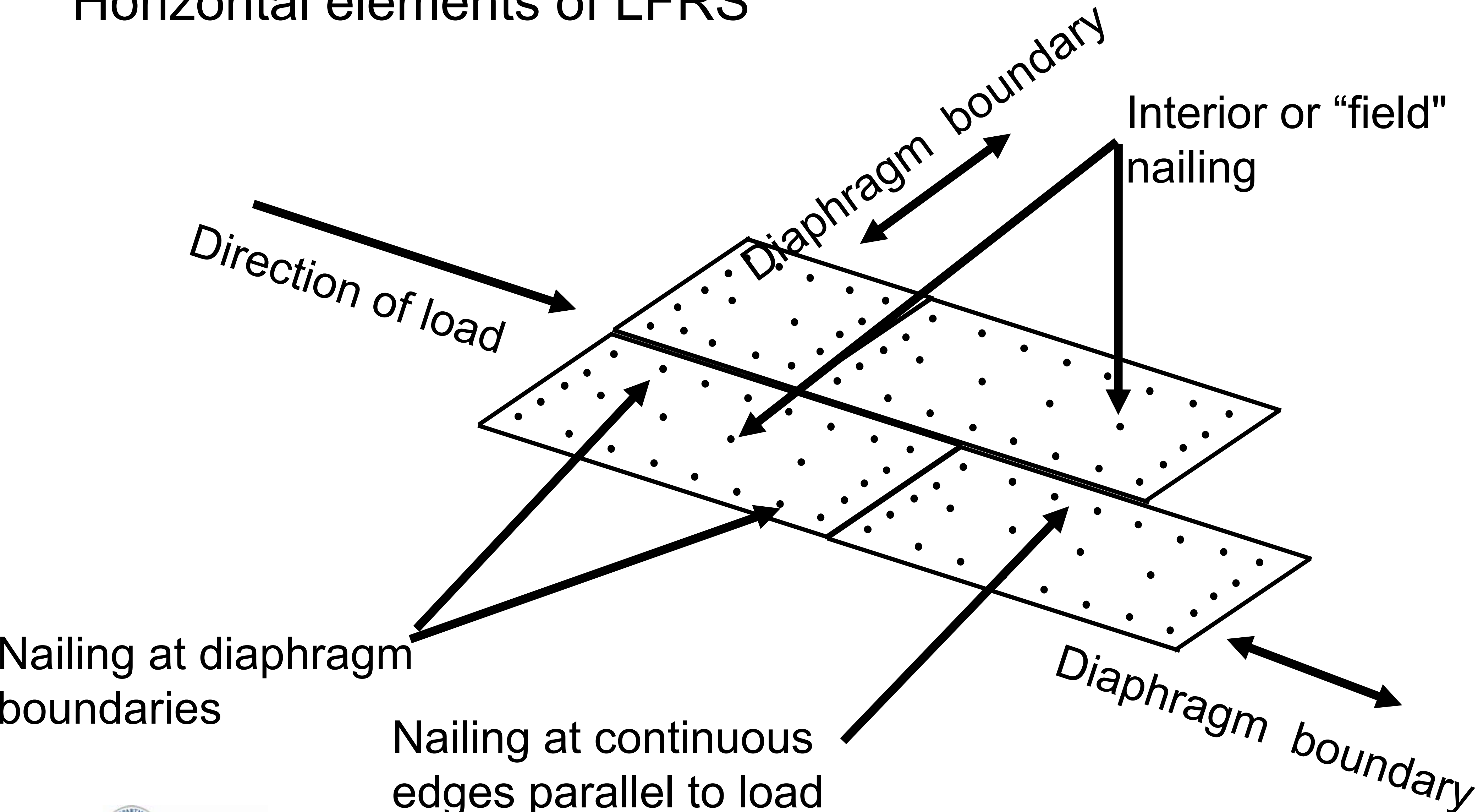


- **Most structures rely on some form of nailed wood structural panels to act as diaphragms for the horizontal elements of the LFRS (plywood or oriented strand board – OSB).**

- **Capacity of diaphragm varies with sheathing grade and thickness, nail type and size, framing member size and species, geometric layout of the sheathing (stagger), direction of load relative to the stagger, and whether or not there is blocking behind every joint to ensure shear continuity across panel edges.**

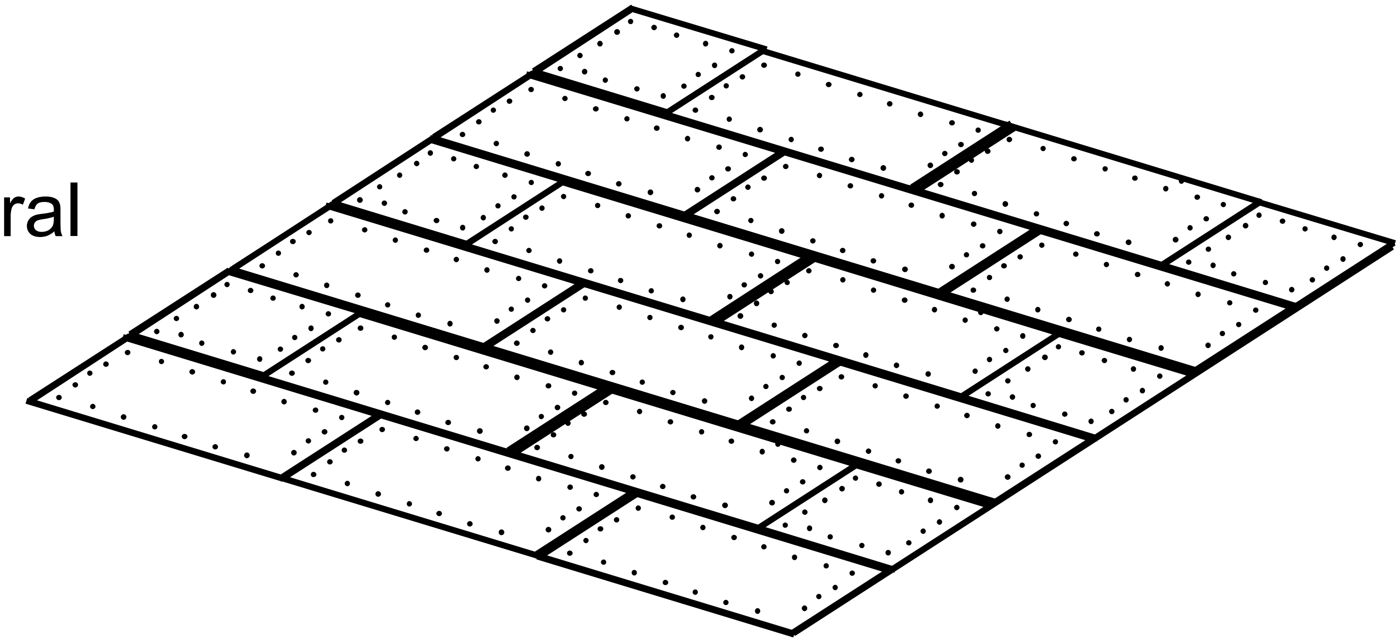
Wood Structure Construction Methods: Lateral

Horizontal elements of LFRS



Wood Structure Construction Methods: Lateral

Horizontal element: nailed wood structural panel diaphragm



- **The building code has tables of diaphragm design capacity (at either ASD or LRFD resistance levels) relative to all of the factors mentioned above.**

Wood Structure Construction Methods: Lateral

Vertical element: nailed wood structural panel diaphragm

- **Shear capacities for vertical plywood/OSB diaphragms are also given in the codes, with similar variables impacting their strength.**

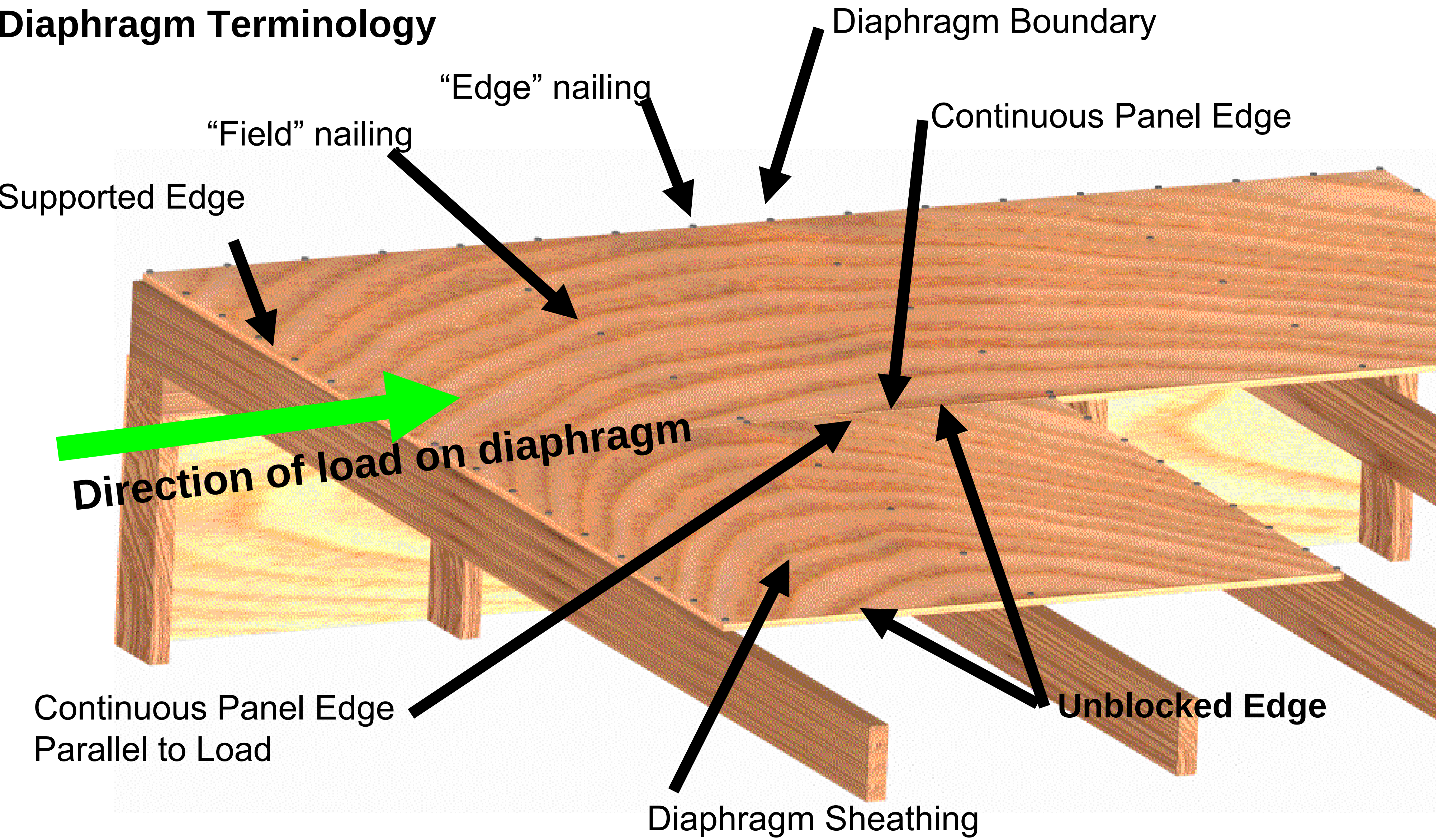
- **Heavy timber braced frames (1997 UBC) and singly or doubly diagonal sheathed walls are also allowed, but rare.**

Wood Structure LFRS Design Methods: Engineered

- If a structures does not meet the code requirements for "prescriptive" or "conventional" construction, it must be "engineered."
- As in other engineered structures, wood structures are only limited by the application of good design practices applied through principles of mechanics (and story height limitations in the code).
- A dedicated system of horizontal and vertical elements, along with complete connectivity, must be designed and detailed.

Wood Structure LFRS Design Methods: Engineered

Diaphragm Terminology



Wood Structure LFRS Design Methods: Engineered

Diaphragm Design Tables

- **Tables are for DFL or SYP – need to adjust values if framing with wood species with lower specific gravities.**
- **Partial reprint of engineered wood structural panel diaphragm info in 2003 IBC Table 2306.3.1.**
- **Major divisions: Structural 1 vs. Rated Sheathing and Blocked vs. Unblocked panel edges.**

RECOMMENDED SHEAR (POUNDS PER FOOT) FOR HORIZONTAL APA PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS-FIR, LARCH OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING

Panel Grade	Common Nail Size	Minimum Nail Penetration in Framing (inches)	Minimum Nominal Panel Thickness (inch)	Minimum Nominal Width of Framing Member (inches)	Blocked Diaphragms: Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)[b] 6; Nail Spacing (in.) at other panel edges (Cases 1, 2, 3 & 4) 6	4 / 6	2-1/2[c] / 4	2[c] / 3	Unblocked Diaphragms: Nails Spaced 6" max. at Supported Edges[b] – Case 1 (No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 & 6)
APA STRUCTURAL I grades	6d[e]	1-1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1-1/2	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[d]	1-5/8	15/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240
APA RATED SHEATHING, APA RATED STURD-I-FLOOR	6d[e]	1-1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
			3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
	8d	1-1/2	7/16	2	255	340	505	575	230	170

(a) For framing of other species: (1) Find specific gravity for species of lumber in AFPA National Design Specification. (2) Find shear value from table above for nail s[illegible] for Structural I panels (regardless of a[illegible]ual grade). (3) Multiply value by [illegible] for species with specific gravity of [illegible] or greater, or 0.65 for all other species..

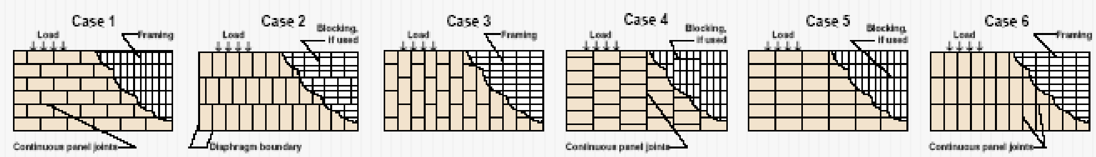
(b) Space nails maximum 12 in. o.c. along intermediate framing members (6 in. o.c. when supports are spaced 48 in. o.c.).

(c) Framing at adjoining panel edges shall be 3-in. nominal or wider, and nails shall be staggered where nails are spaced 2 inches o.c. or 2-1/2 inches o.c.

(d) Framing at adjoining panel edges shall be 3-in. nominal or wider, and nails shall be staggered where 10d nails having penetration into framing of more than 1-5/8 [illegible] are spaced 3 inches o.c.

(e) 8d is recomme[illegible]d minimum for roofs due to negat[illegible] pressures of high winds.

Notes: Design for diaphragm stresses depends on direction of continuous panel joints with reference to load, not on direction of long dimension of sheet. Continuous framing may be in either direction for blocked diaphragms.



Wood Structure LFRS Design Methods: Engineered

Shear Wall Design Tables

- Partial reprint of engineered wood structural panel diaphragm info in 2003 IBC Table 2306.4.1.
- Tables are for DFL or SYP – need to adjust values if framing with wood species with lower specific gravities.
- Major divisions: Structural 1 vs. Rated Sheathing and Panels Applied Directly to Framing vs. Panels Applied Over Gypsum Wallboard.
- NO UNBLOCKED edges allowed.

RECOMMENDED SHEAR (POUNDS PER FOOT) FOR APA PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS-FIR, LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b]

Panel Grade	Minimum Nominal Panel Thickness (in.)	Minimum Nail Penetration in Framing (in.)	Panels Applied Direct to Framing: Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.): 6	4	3	2[e]	Panels Applied Over 1/2" or 5/8" Gypsum Sheathing: Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.): 6	4	3	2[e]
APA STRUCTURAL I grades	5/16	1-1/4	6d	200	300	390	510	8d	200	300	390	510
	3/8			230[d]	360[d]	460[d]	610[d]					
	7/16	1-1/2	8d	255[d]	395[d]	505[d]	670[d]	10d[f]	280	430	550	730
	15/32			280	430	550	730					
	15/32	1-5/8	10d[f]	340	510	665	870	—	—	—	—	—
APA RATED SHEATHING; APA RATED SIDING[g] and other APA grades except species Group 5	5/16 or 1/4[c]	1-1/4	6d	180	270	350	450	8d	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8			220[d]	320[d]	410[d]	530[d]					
	7/16	1-1/2	8d	240[d]	350[d]	450[d]	585[d]	10d[f]	260	380	490	640
	15/32			260	380	490	640					
	15/32	1-5/8	10d[f]	310	460	600	770	—	—	—	—	—
	19/32			340	510	665	870	—	—	—	—	—
APA RATED SIDING 303[g] and other APA grades except species Group 5			Nail Size (galvanized casing)					Nail Size (galvanized casing)				
	5/16[c]	1-1/4	6d	140	210	275	360	8d	140	210	275	360
	3/8	1-1/2	8d	160	240	310	410	10d[f]	160	240	310	410

(a) For framing of other species: (1) Find specific gravity for species of lumber in the AFPA National Design Specification. (2)(a) For common or galvanized box nails, find shear value from table above for nail size for STRUCTURAL I panels (regardless of actual grade). (b) For galvanized casing nails, take shear value directly from table above. (3) Multiply this value by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species.

(b) All panel edges backed with 2-inch nominal or wider framing. Install panels either horizontally or vertically. Space nails maximum 6 inches o.c. along intermediate framing members for 3/8-inch and 7/16-inch panels installed on studs spaced 24 inches o.c. For other conditions and panel thicknesses, space nails maximum 12 inches o.c. on intermediate supports.

(c) 3/8-inch or APA RATED SIDING 16 oc is minimum recommended when applied direct to framing as exterior siding.

(d) Shears may be increased to values shown for 15/32-inch sheathing with same nailing provided (1) studs are spaced a maximum of 16 inches o.c., or (2) if panels are applied with long dimension across studs.

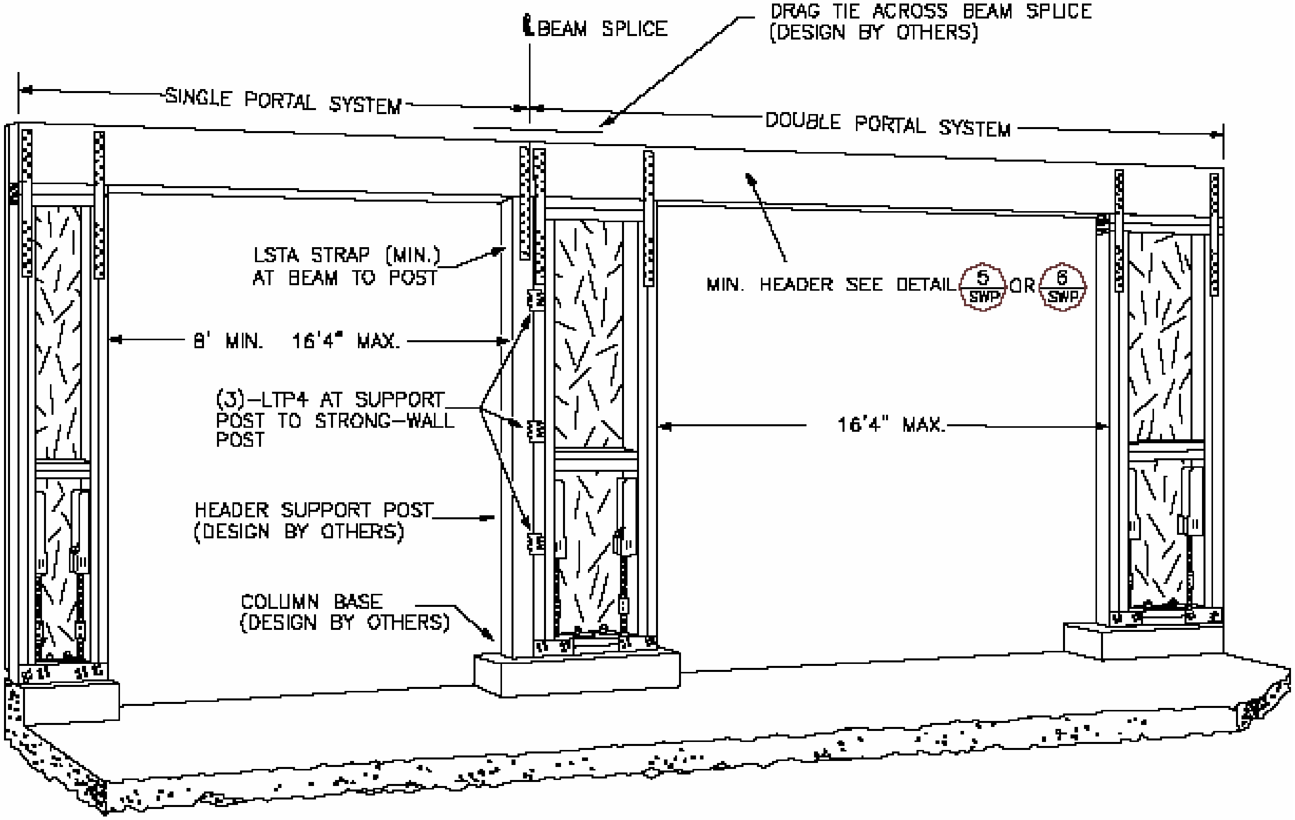
(e) Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where nails are spaced 2 inches o.c.

(f) Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches o.c.

(g) Values apply to all-veneer plywood APA RATED SIDING panels only. Other APA RATED SIDING panels may also qualify on a proprietary basis. APA RATED SIDING 16 oc plywood may be 11/32 inch, 3/8 inch or thicker. Thickness at point of nailing on panel edges governs shear values.

Wood Structure LFRS Design Methods: Engineered

Proprietary Moment Frames



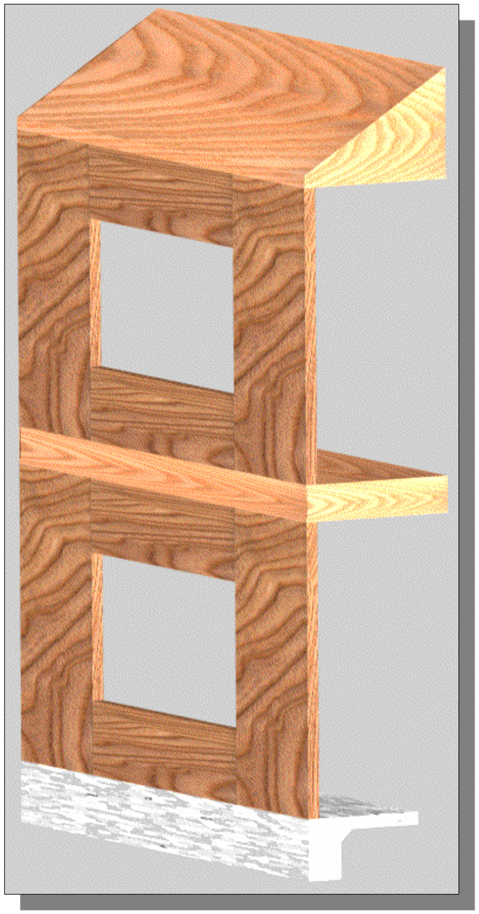
SINGLE & DOUBLE PORTAL ASSEMBLY

- **Traditional vertical diaphragm shear walls less effective at high aspect ratios.**
- **Prefabricated proprietary code-approved solutions available.**

Wood Structure LFRS Design Methods: Engineered

Complete Load Path

- Earthquakes move the foundations of a structure.
- If the structure doesn't keep up with the movements of the foundations, failure will occur.
- Keeping a structure on its foundations requires a complete load path from the foundation to all mass in a structure.
- Load path issues in wood structures can be complex.
- For practical engineering, the load path is somewhat simplified for a "good enough for design" philosophy.

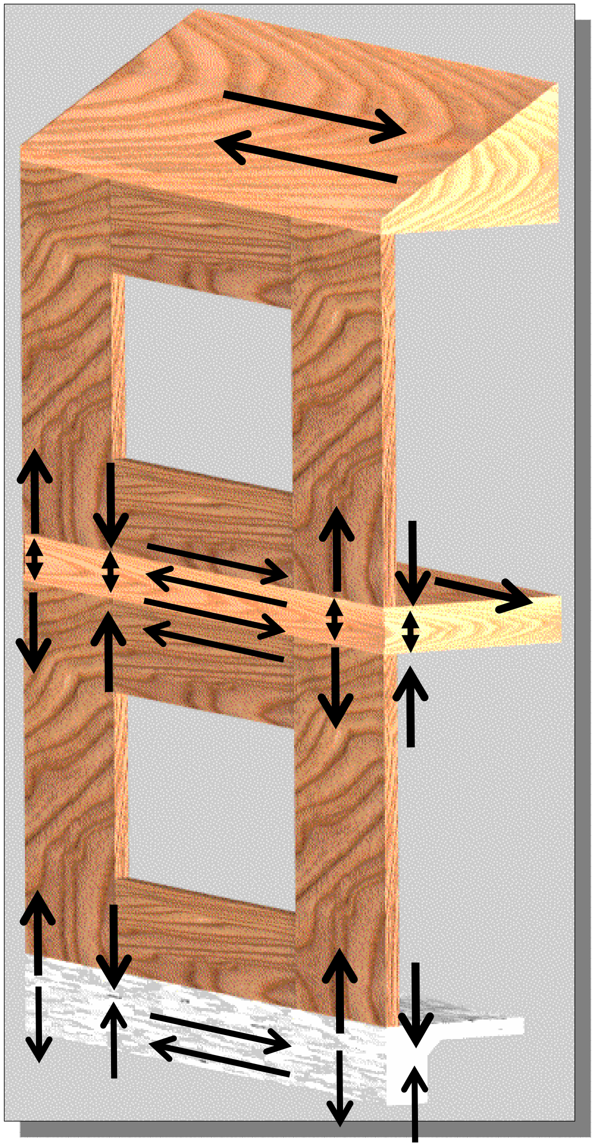


Wood Structure LFRS Design Methods: Engineered

Complete Load Path

- Diaphragm to shear wall

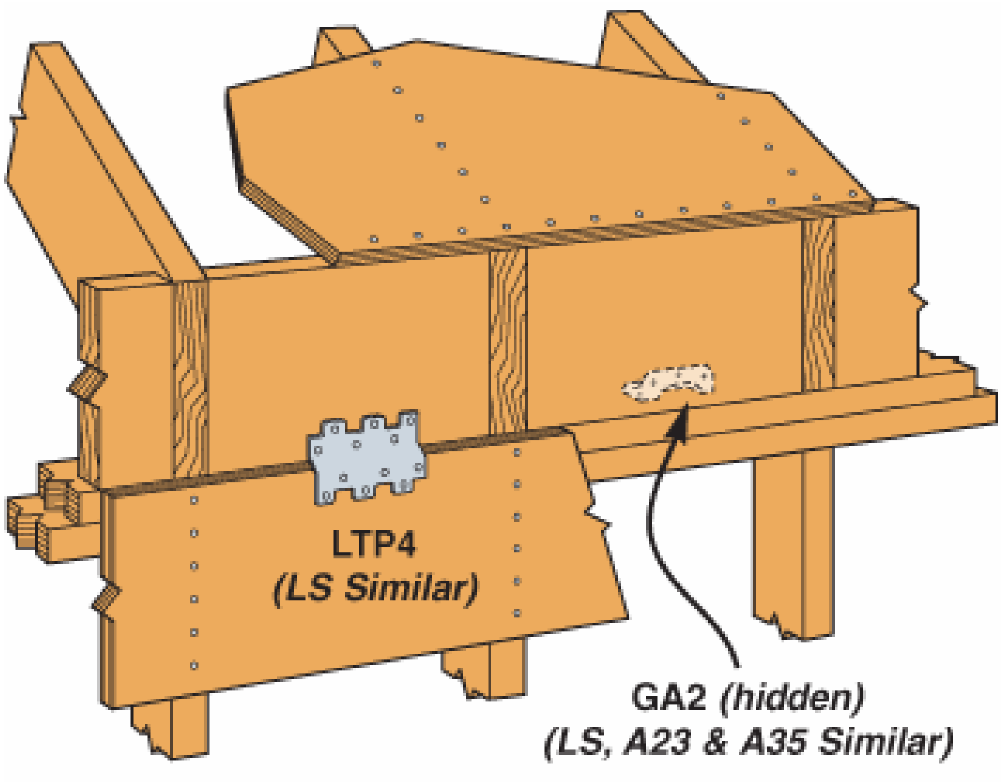
- Shear wall overturning
- Diaphragm to shear wall
- Overturning tension/compression through floor
- Shear transfer through floor
- Overturning tension/compression to foundation
- Shear transfer to foundation

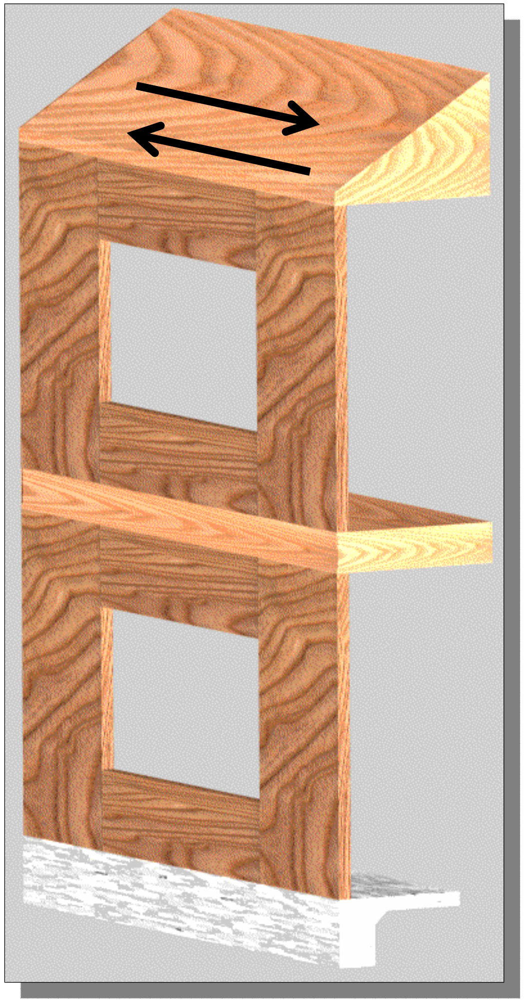


Wood Structure LFRS Design Methods: Engineered

Complete Load Path

- Diaphragm to shear wall



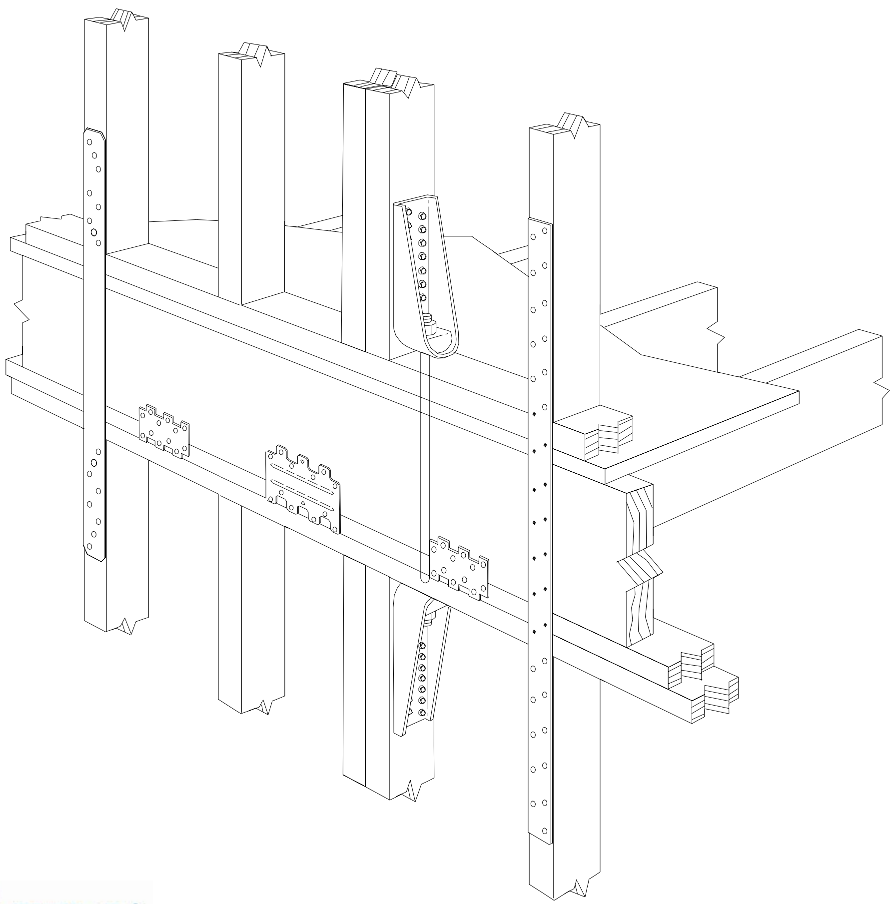


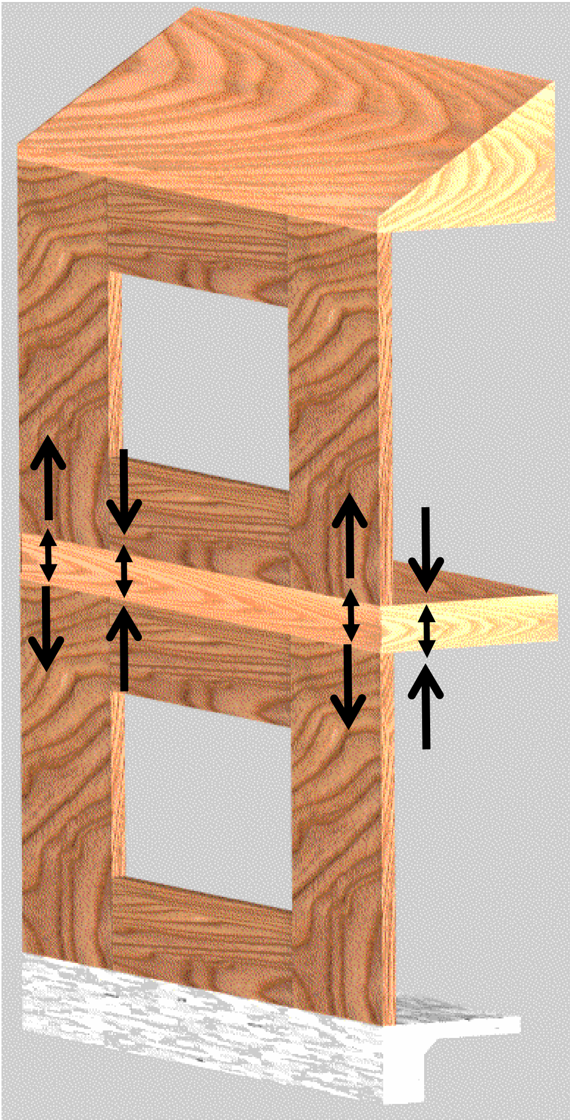
Toe nails: 2003 IBC 2305.1.4 150 plf limit in SDCs D-F.

Wood Structure LFRS Design Methods: Engineered

Complete Load Path

- Shear wall overturning / transfer of vertical forces through floor

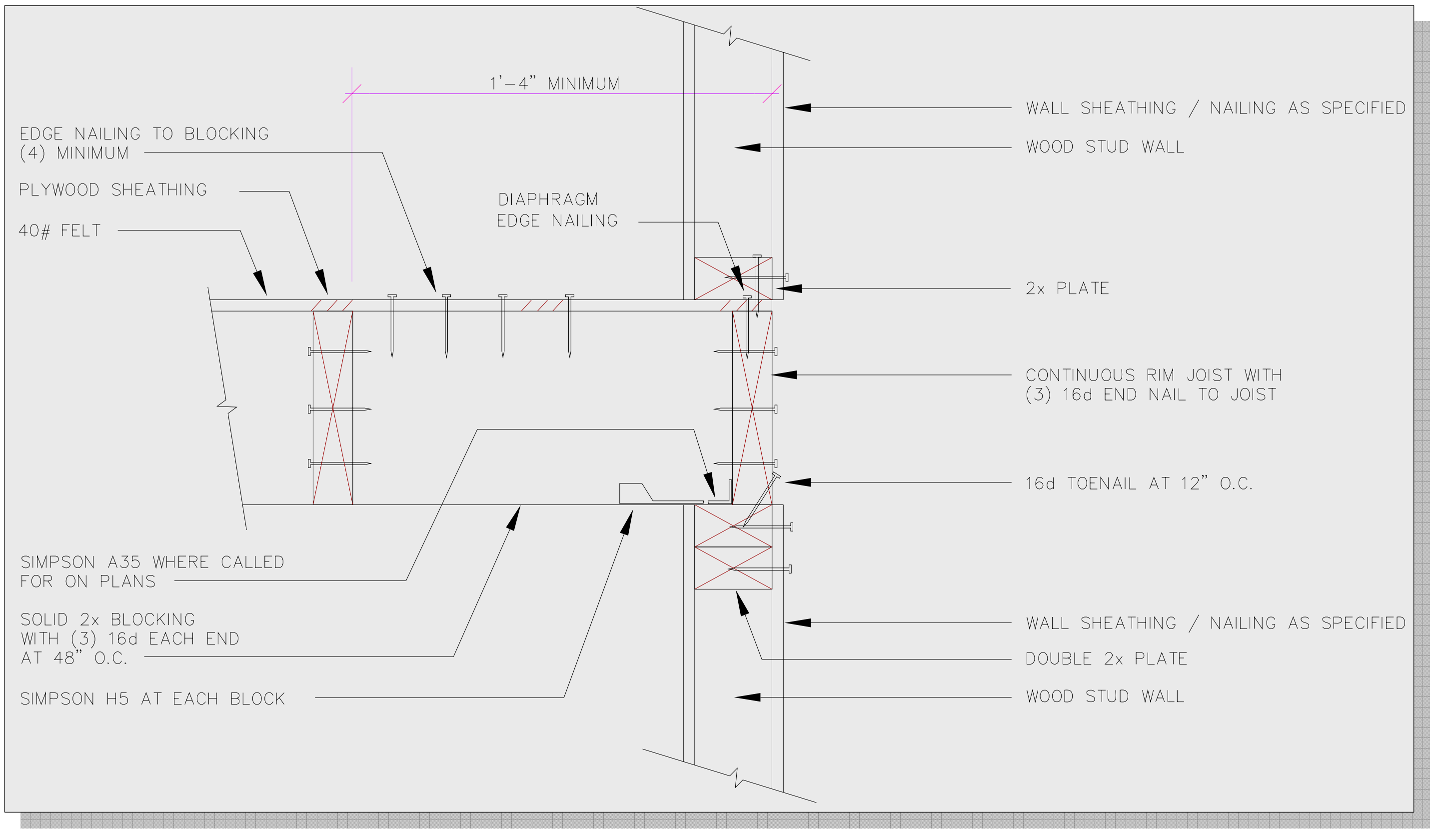


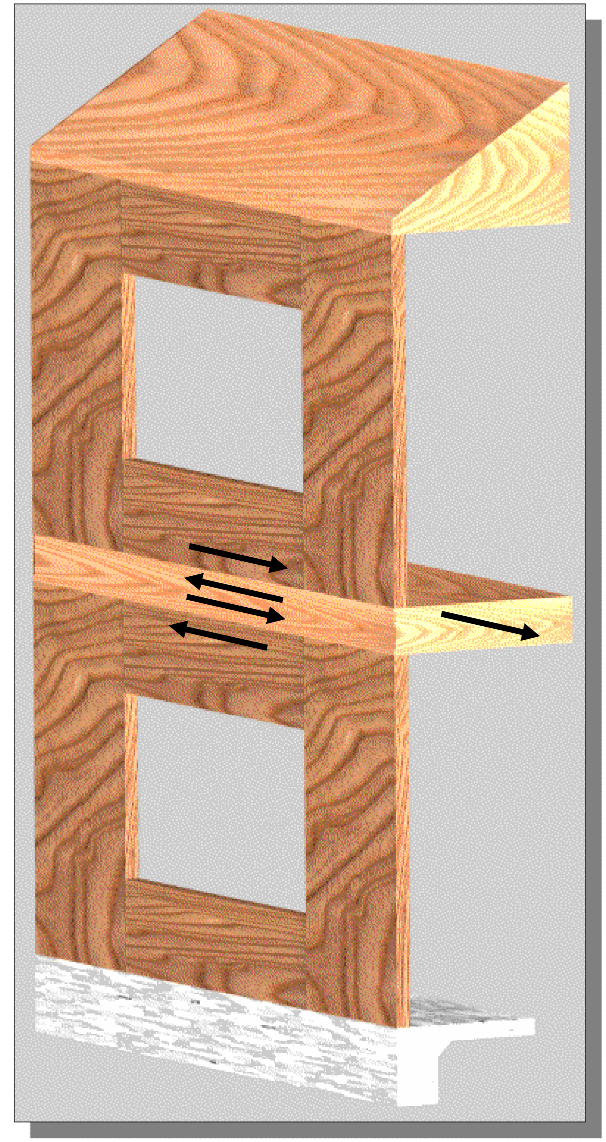


Wood Structure LFRS Design Methods: Engineered

Complete Load Path

- Diaphragm to shear wall / shear transfer through floor

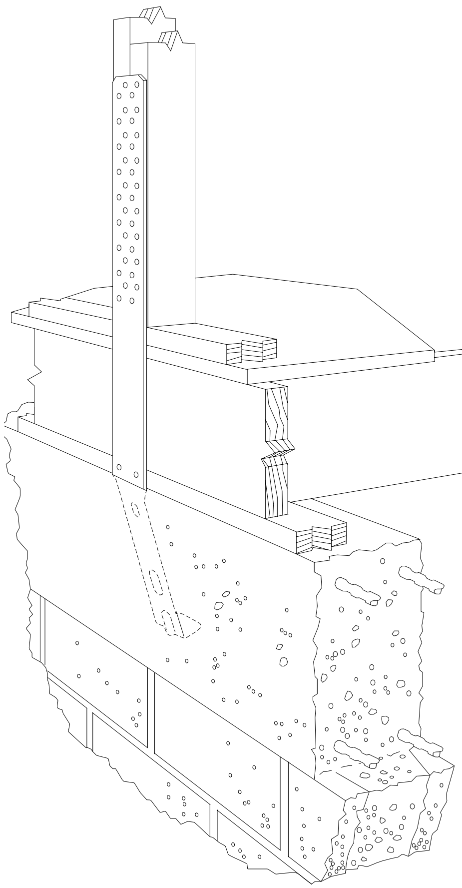


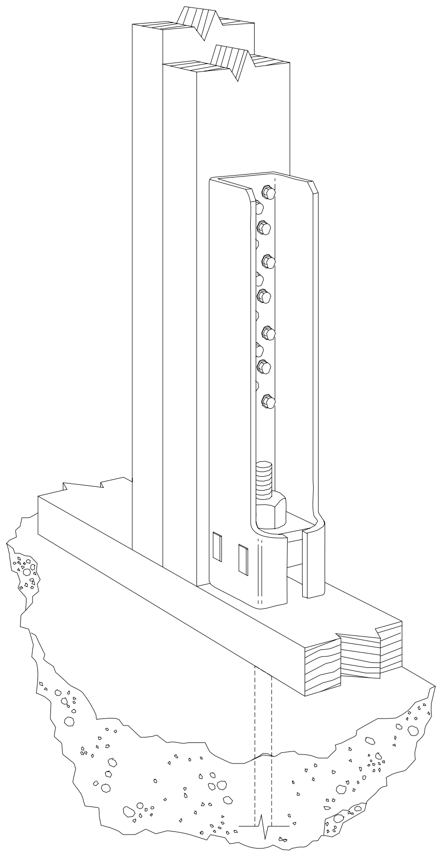


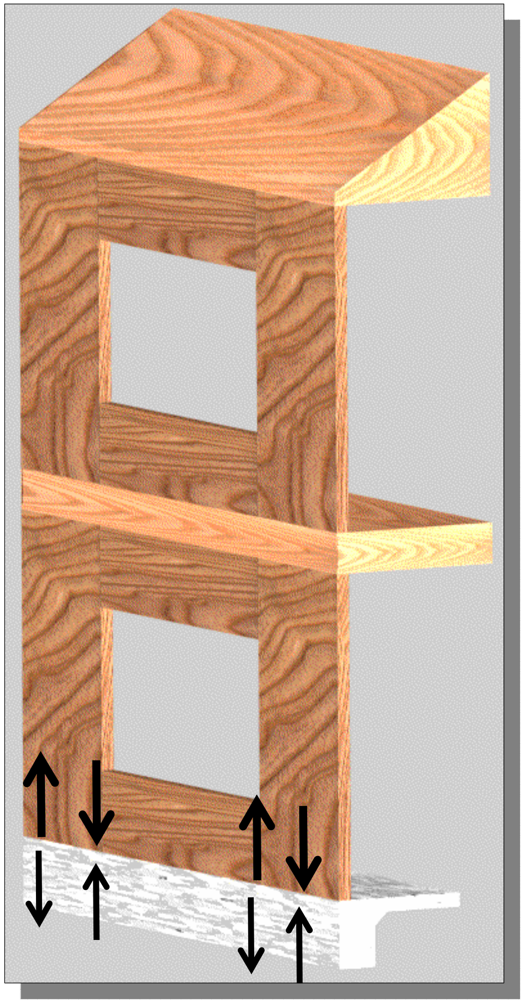
Wood Structure LFRS Design Methods: Engineered

Complete Load Path

- Overturning tension/compression to foundation



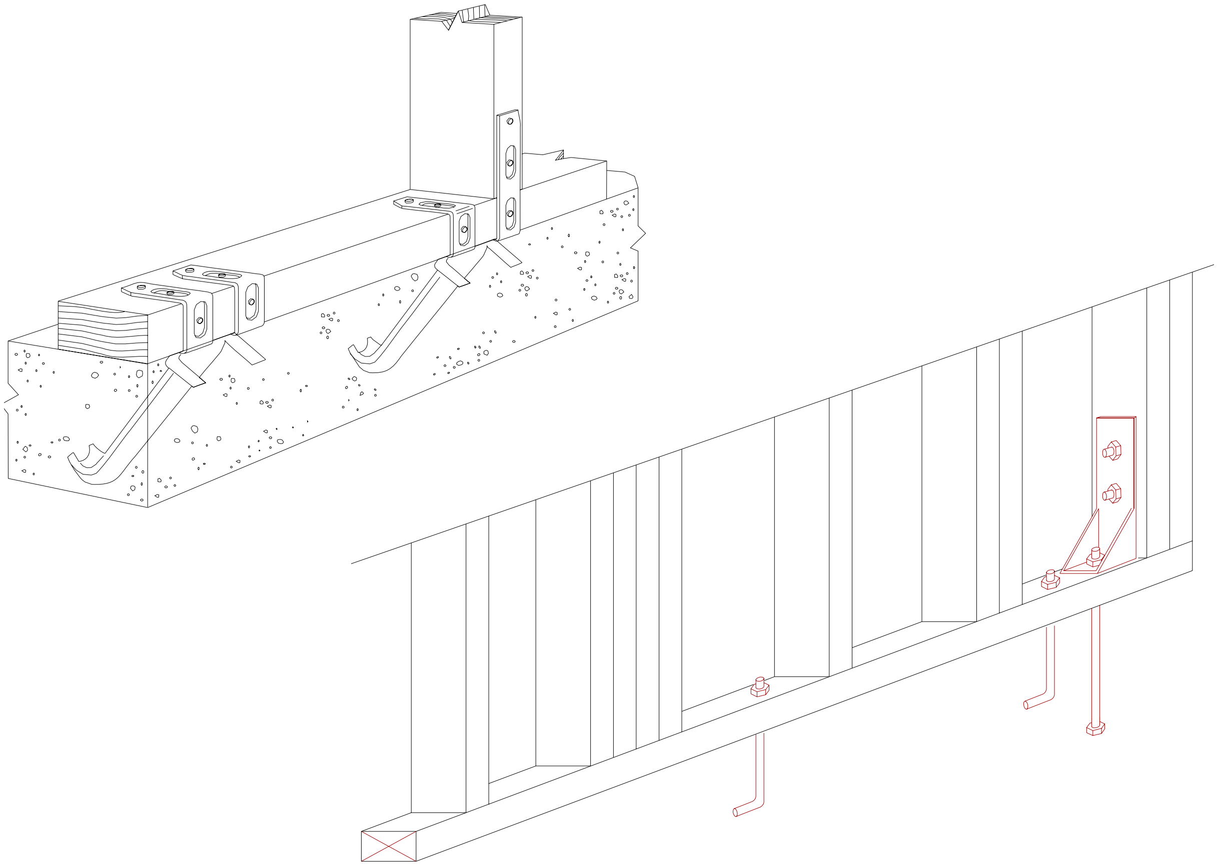


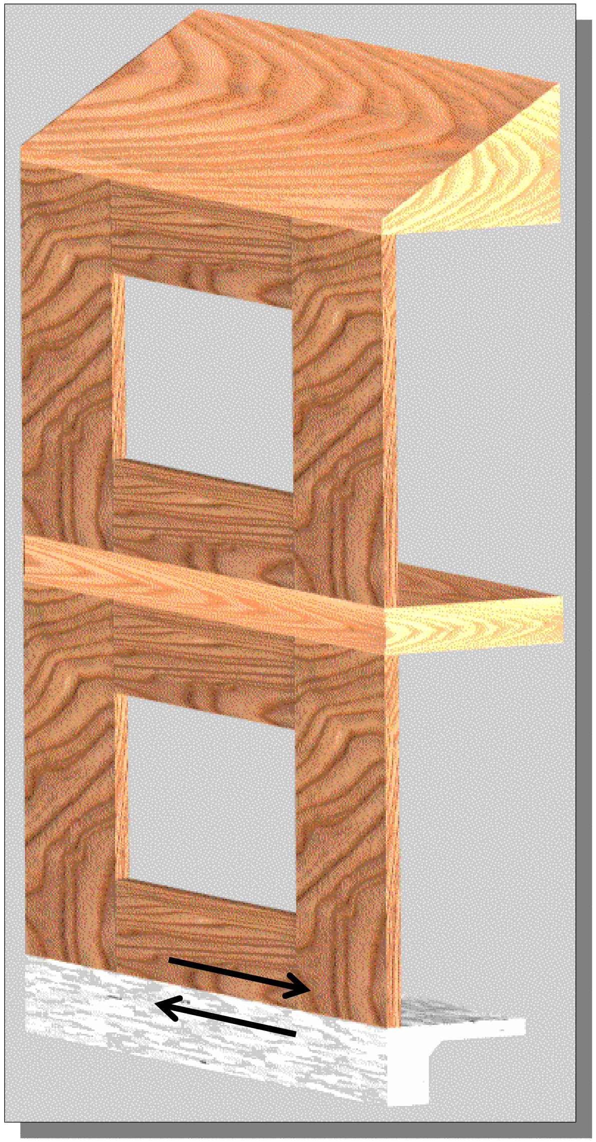


Wood Structure LFRS Design Methods: Engineered

Complete Load Path

- Shear transfer to foundation





Wood Structure LFRS Design Methods: Prescriptive

- **Also referred to as "*Conventional Construction*" or "*Deemed to Comply*"**

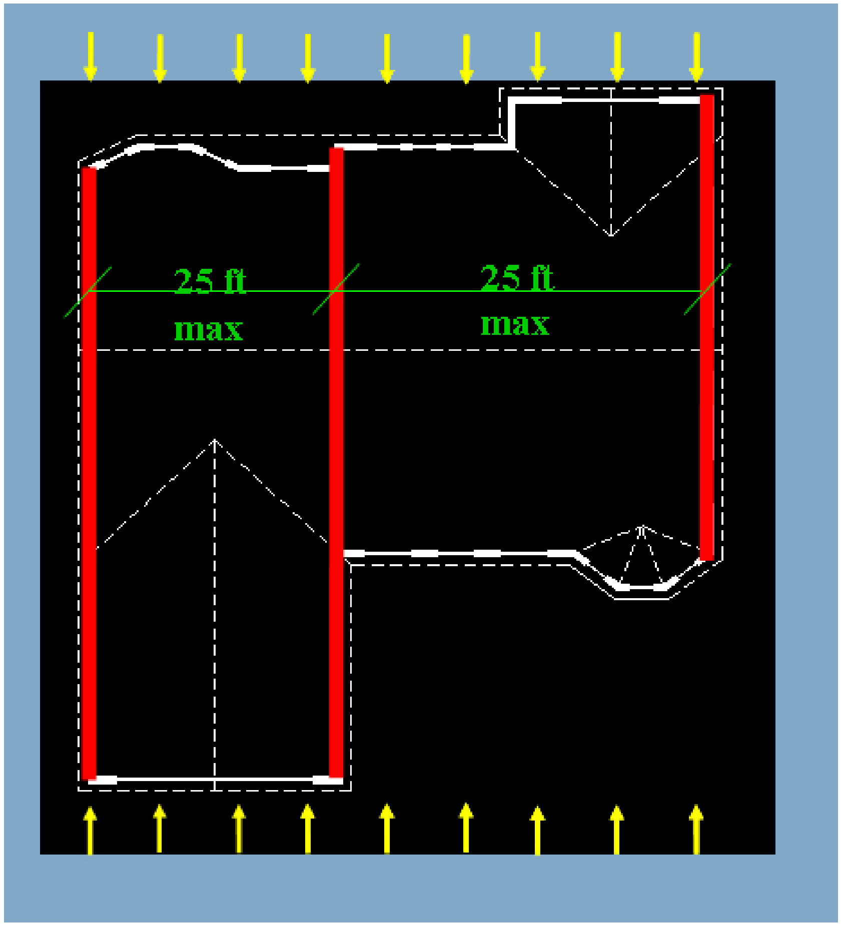
- **Traditionally, many simple wood structures have been designed without "engineering."**
- **Over time, rules of how to build have been developed, most recently in the 2003 International Residential Code (IRC).**
- **For the lateral system, the "dedicated" vertical element is referred to as a *braced wall panel*, which is part of a *braced wall line.***
- **Based on SDC and number of stories, rules dictate the permissible spacing between braced wall lines, and the spacing of braced wall panels within braced wall lines.**

Wood Structure LFRS Design Methods: Prescriptive

- While rules exist for the "dedicated" elements, testing and subsequent analysis has show these structures do not "calc out" based on just the strength of braced wall panels.
- In reality, the strength, stiffness, and energy dissipation afforded by the "nonstructural" elements (interior and exterior sheathing) equal or exceed the braced wall panels in their contribution to achieving "life safety" performance in these structures.
- Load path not explicitly detailed.

Wood Structure LFRS Design Methods: Prescriptive

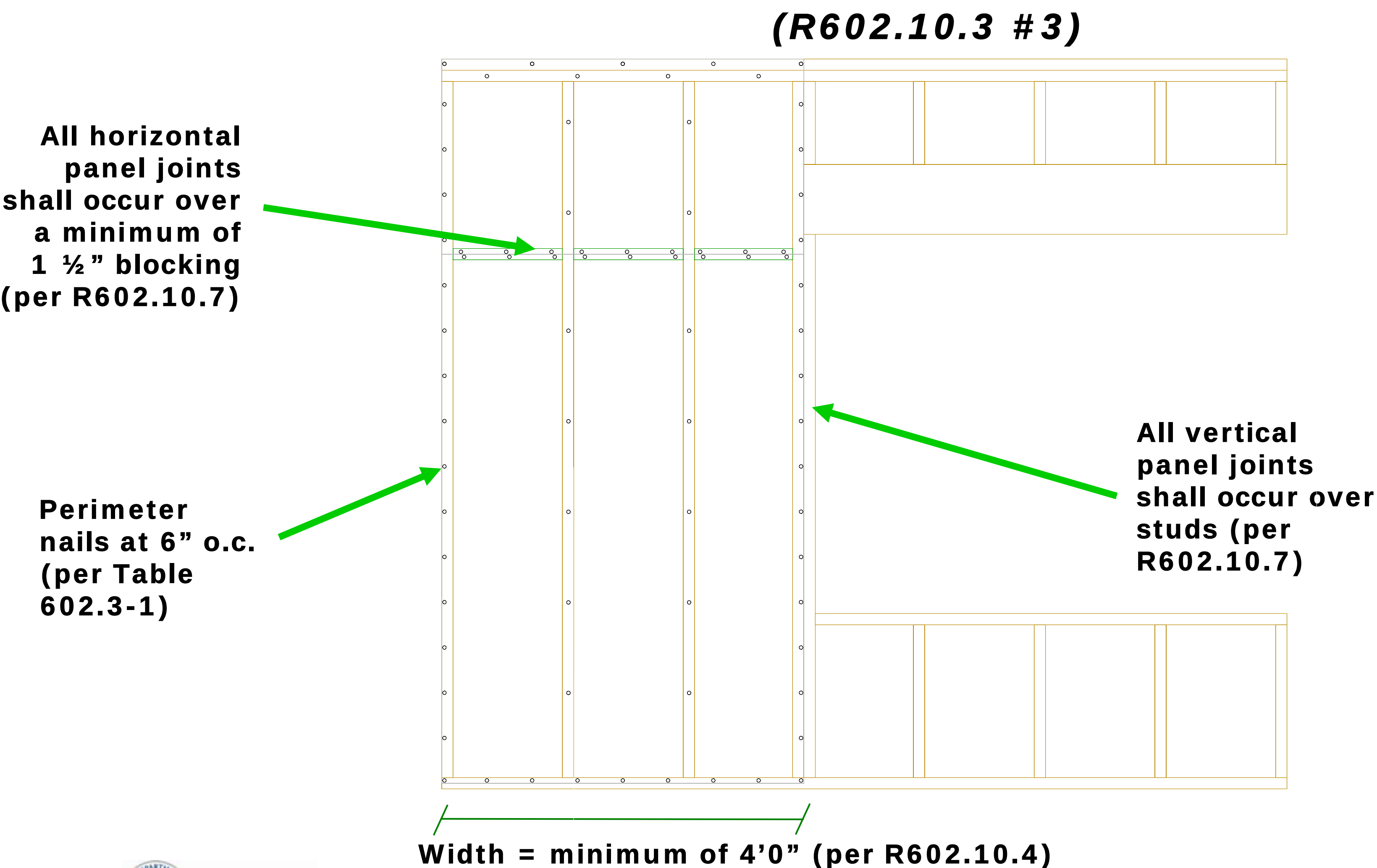
(Seismic Design Category D1 or D2 and/or Wind Speeds < 110 mph)



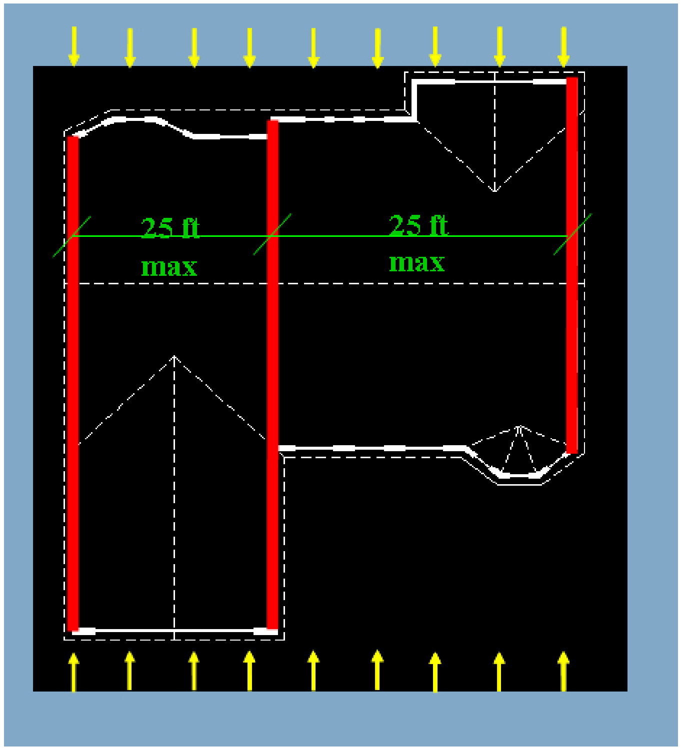
- Example 2003 IRC Spacing Requirements for *Braced Wall Lines*

Wood Structure LFRS Design Methods: Prescriptive

- Example of *Braced Wall Panel* construction (2003 IRC references)

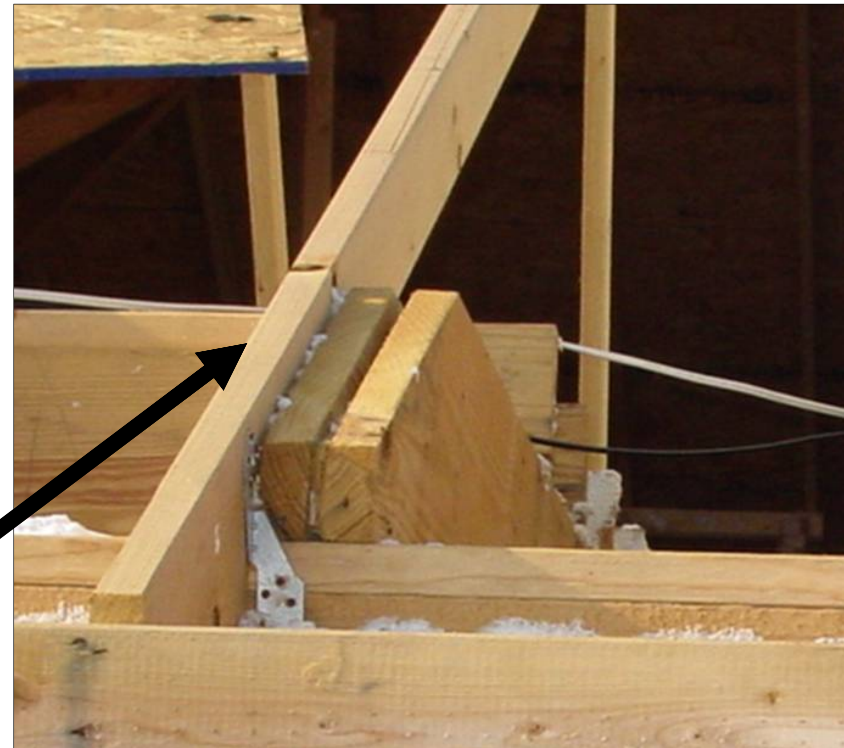


Wood Structure LFRS Design Methods: Prescriptive



- Prescriptive provisions in the 2003 IRC are more liberal than in the 2003 NEHRP *Provisions.*
- The NEHRP Provisions and Commentary can be downloaded from http://www.bssconline.org/. Also available from FEMA and at the BSSC website is FEMA 232, an up to date version of the *Homebuilders' Guide to Earthquake-Resistant Design and Construction*.

Expected Response Under Lateral Load: Wind



- Unlike seismic *design* loads, wind *design* loads are representative of the real expected magnitude.
- When built properly, structural damage should be low.
- Missile or wind born projectile damage can increase damage (this could potentially breach openings and create internal pressures not part of the design).

Expected Response Under Lateral Load: Seismic

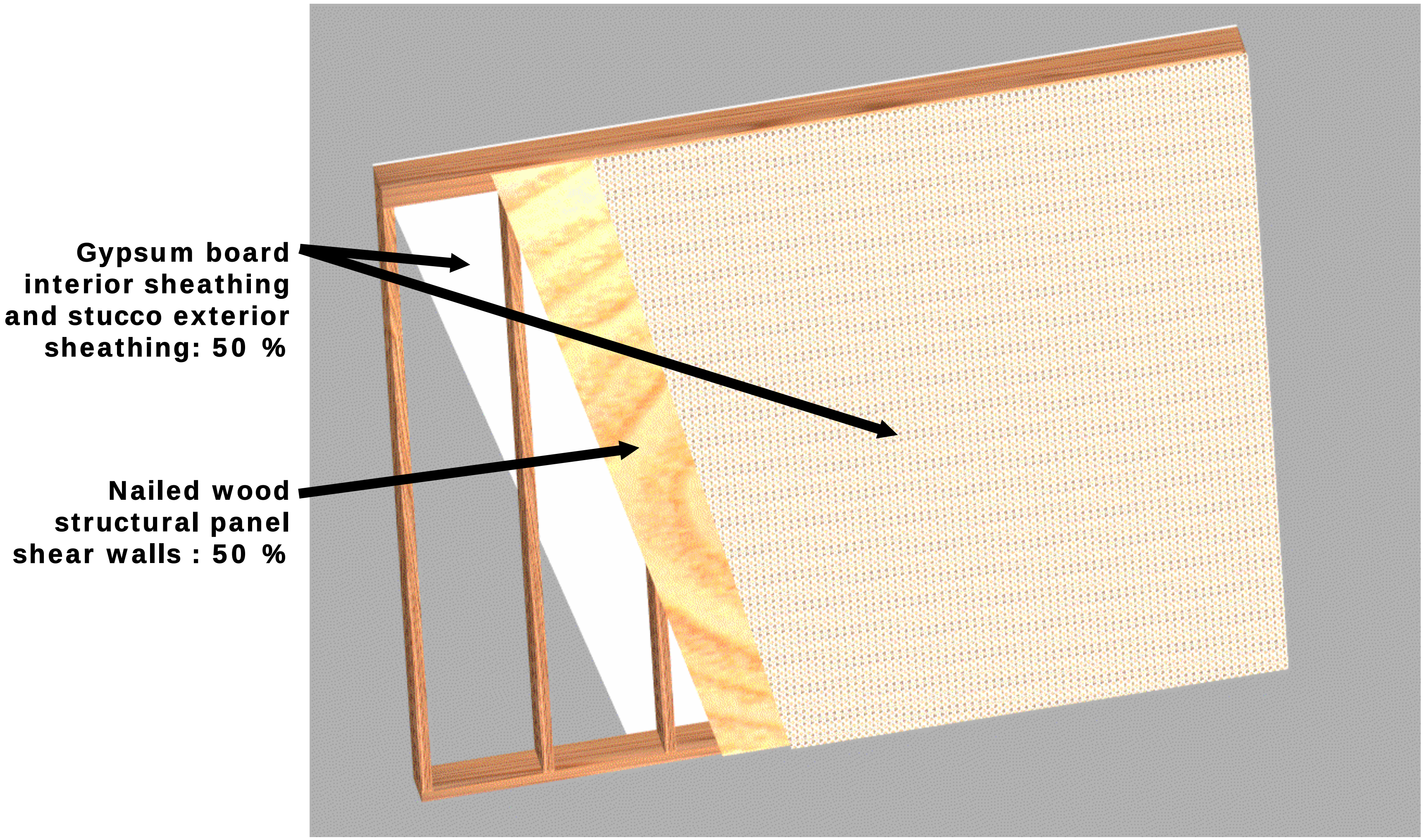
5.3 Daly City, CA March 22, 1957

7.0 Imperial Valley, CA Oct 15, 1957

- **Engineered wood structures are thought of as having good flexibility/ductility, but can also be quite brittle.**
- **Wood structures can be engineered with either "ductile" nailed wood structural panel shear walls or "brittle" gypsum board and/or stucco shear walls as their primary LFRS.**
- **2003 IBC *R* factors: Wood – 6.5; All Others – 2.0.**

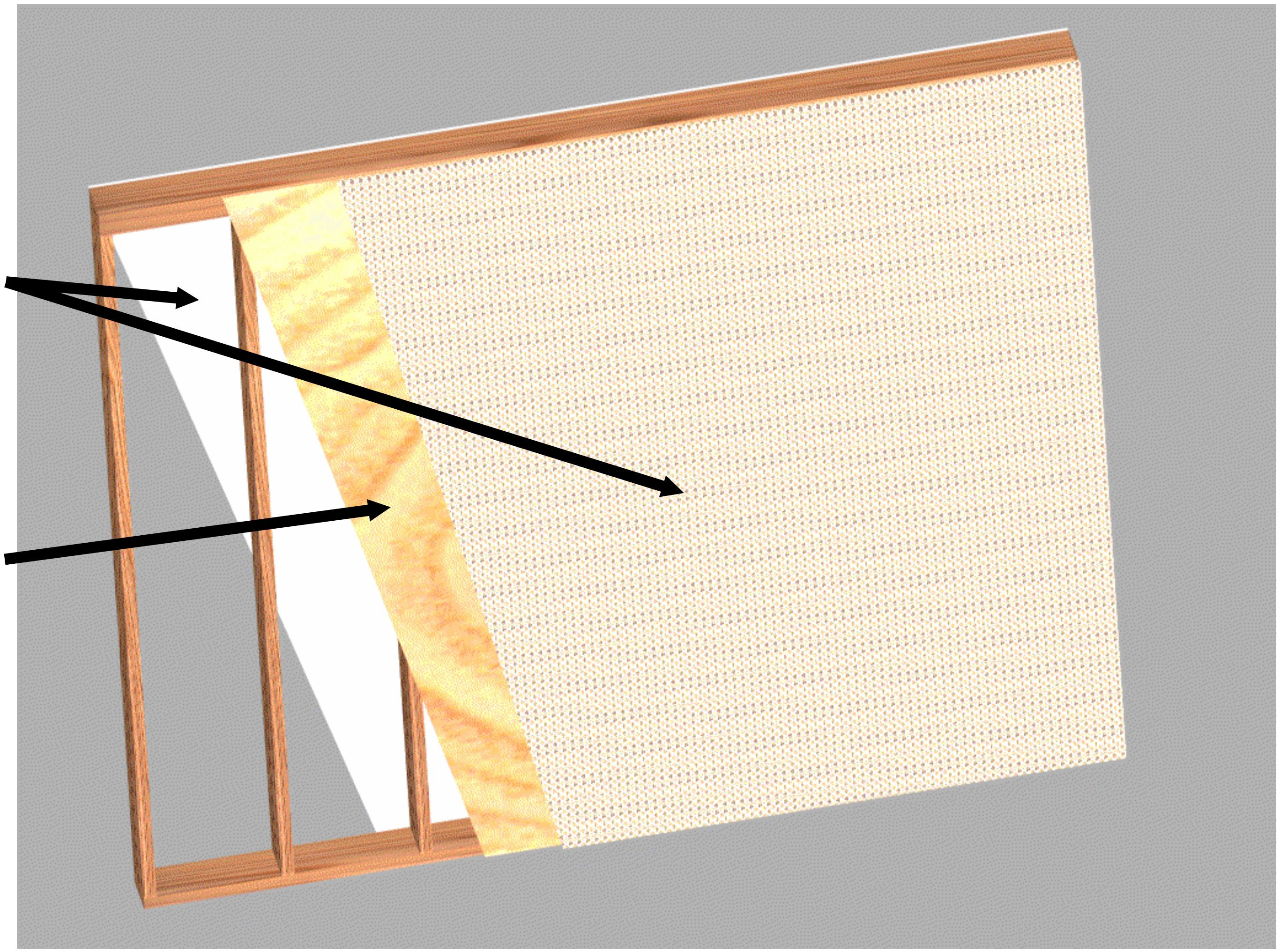
Sources of Strength for Seismic Lateral Resistance

- **Rough** estimates for ***engineered single family home***



Sources of Strength for Seismic Lateral Resistance

- **<u>Rough</u> estimates for *prescriptive single family home***

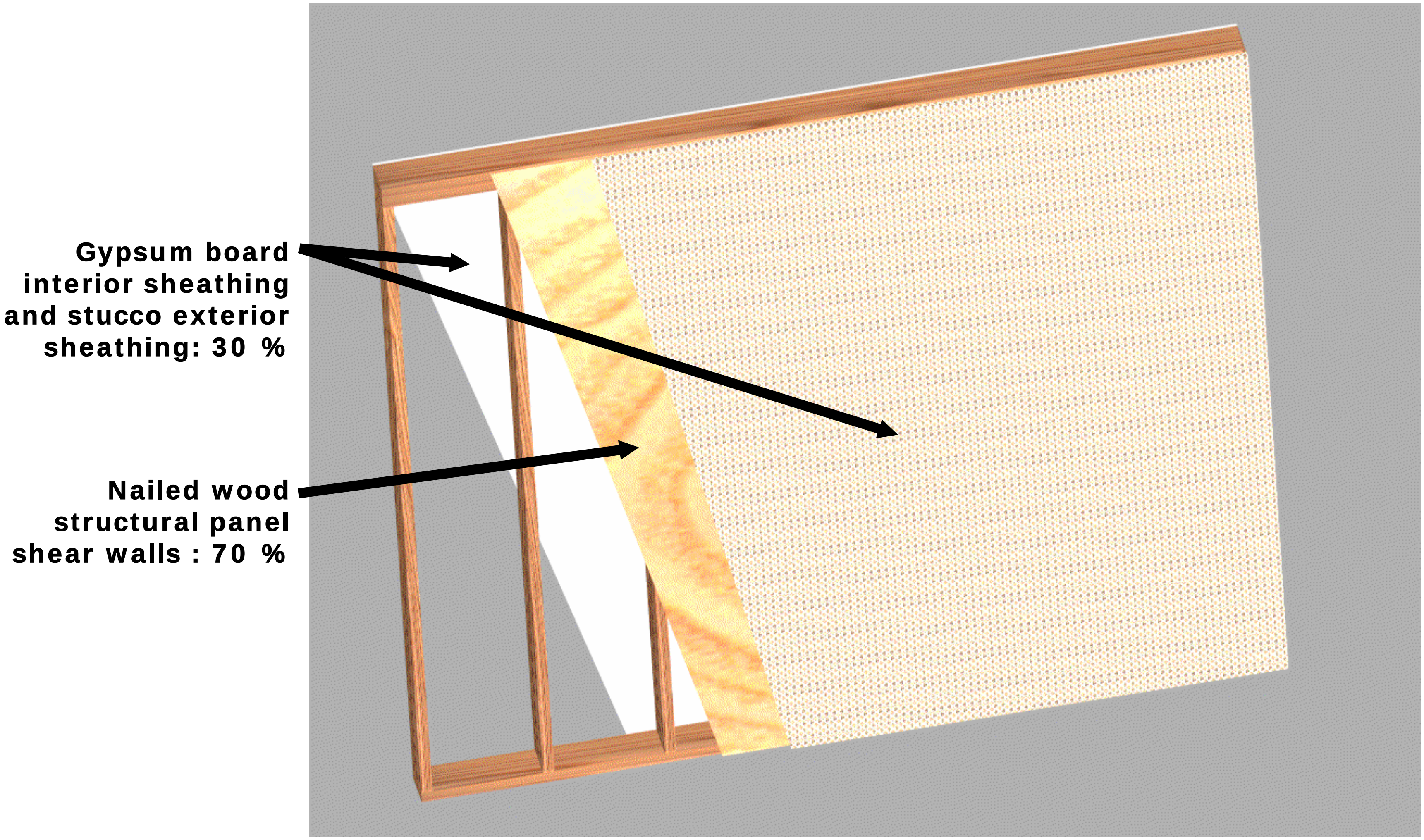


Gypsum board interior sheathing and stucco exterior sheathing: 70 %

Braced Wall Panels : 30 %

Sources of Strength for Seismic Lateral Resistance

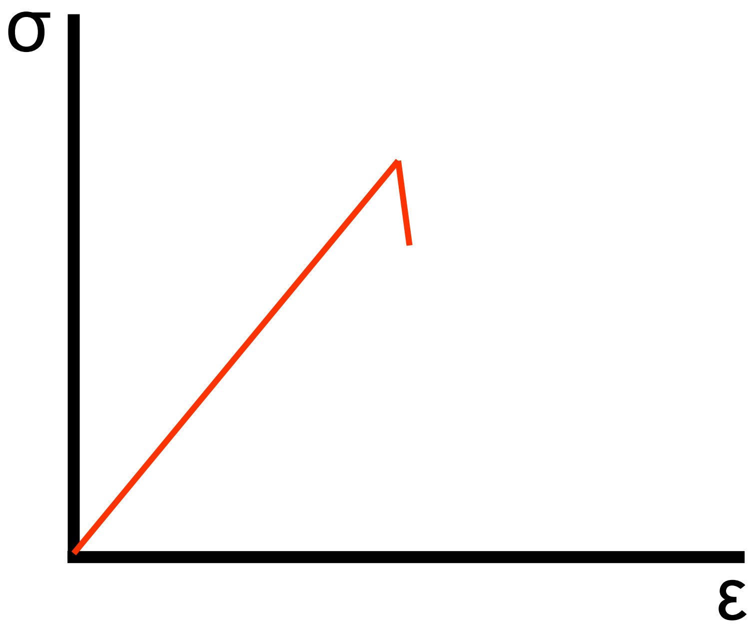
- **<u>Rough</u>** **estimates for *engineered light commercial* structures**



Sources of Ductility and Energy Dissipation in Wood Structures

Stress in the wood

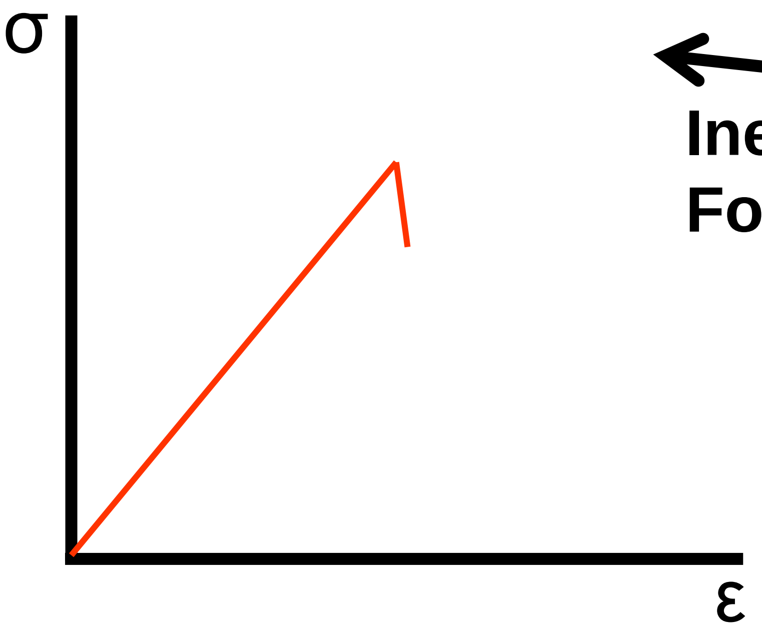
- **Tension parallel to the grain: not ductile, low energy dissipation**



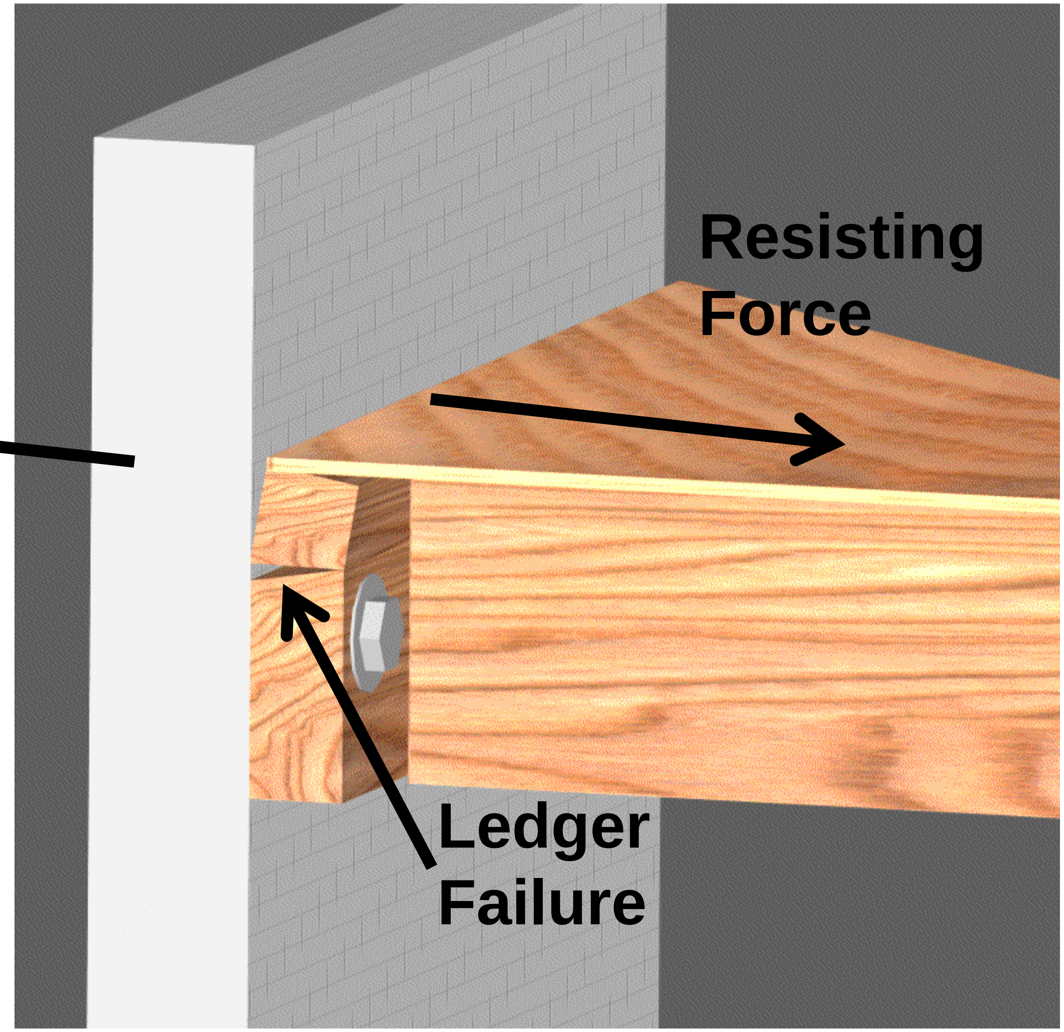
Sources of Ductility and Energy Dissipation in Wood Structures

Stress in the wood

- Tension perpendicular to the grain: not ductile, low energy dissipation



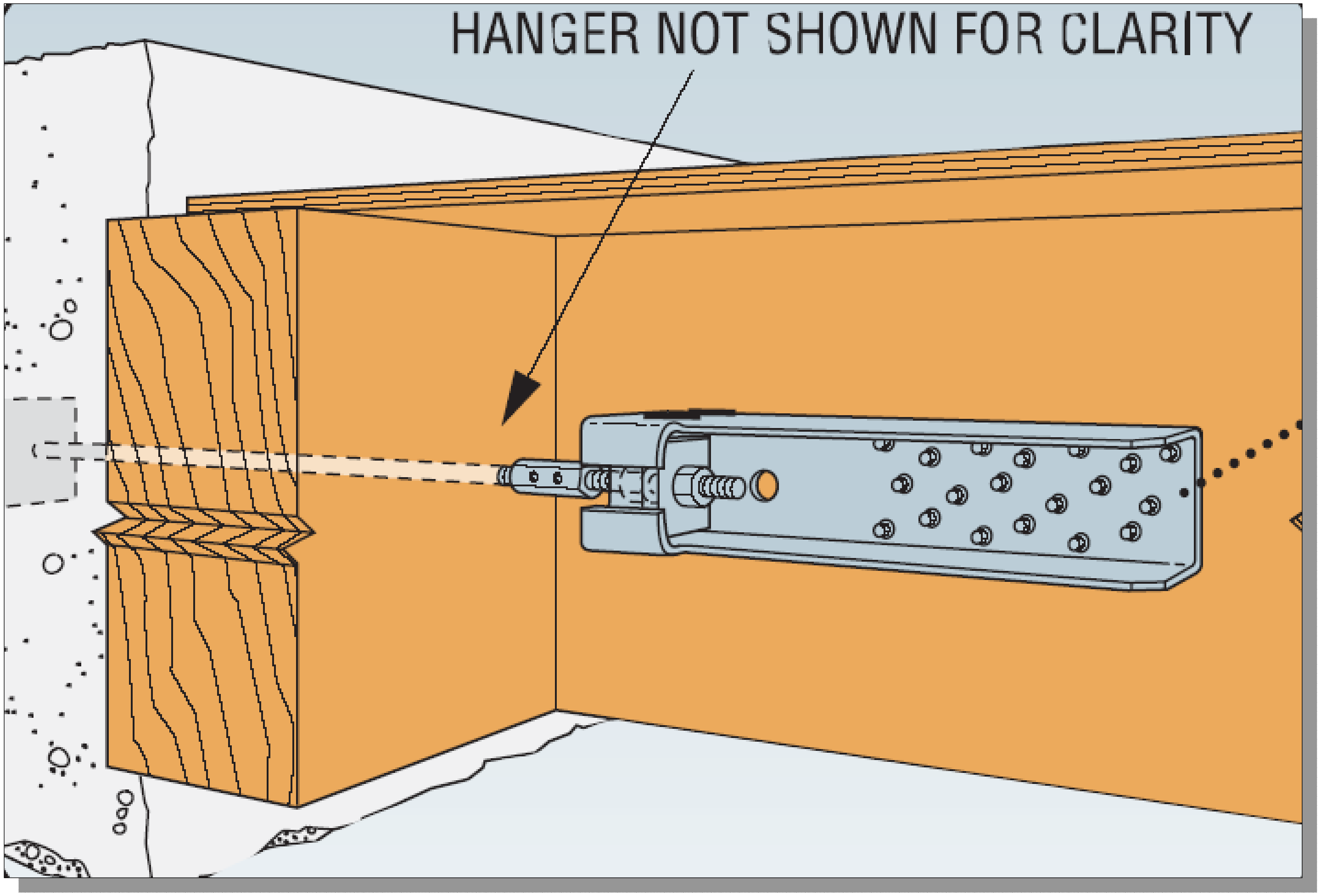
Inertial Force



- Need to have positive wall ties to perpendicular framing

Sources of Ductility and Energy Dissipation in Wood Structures

Positive Wall Tie



Sources of Ductility and Energy Dissipation in Wood Structures

Stress in the wood

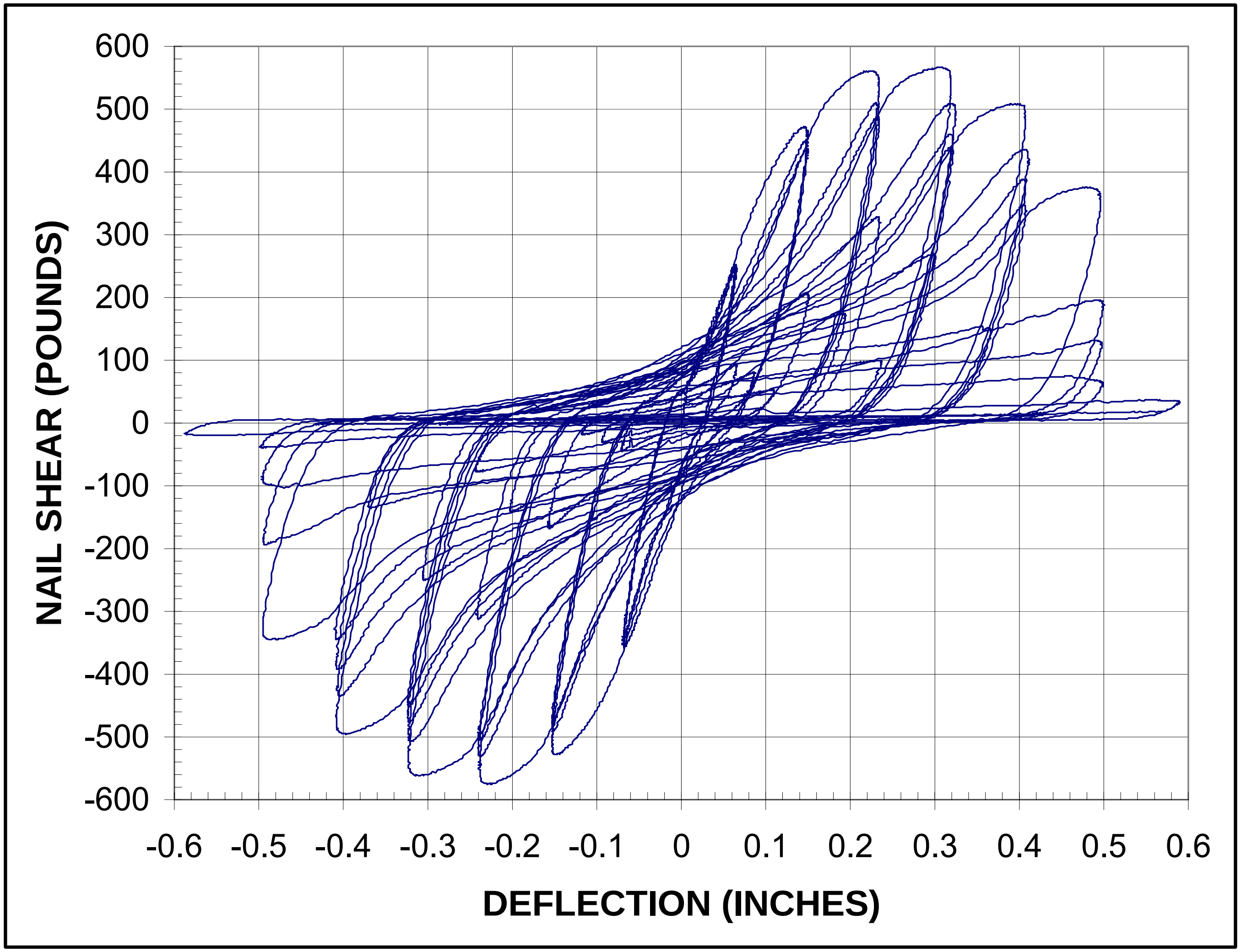
- Compression perpendicular to the grain: ductile, but not recoverable during and event – one way crushing similar to tension only braced frame behavior – ductile, but low energy dissipation
- Design allowable stress should produce ~0.04” permanent crushing

Sources of Ductility and Energy Dissipation in Wood Structures

Stress in the fastener

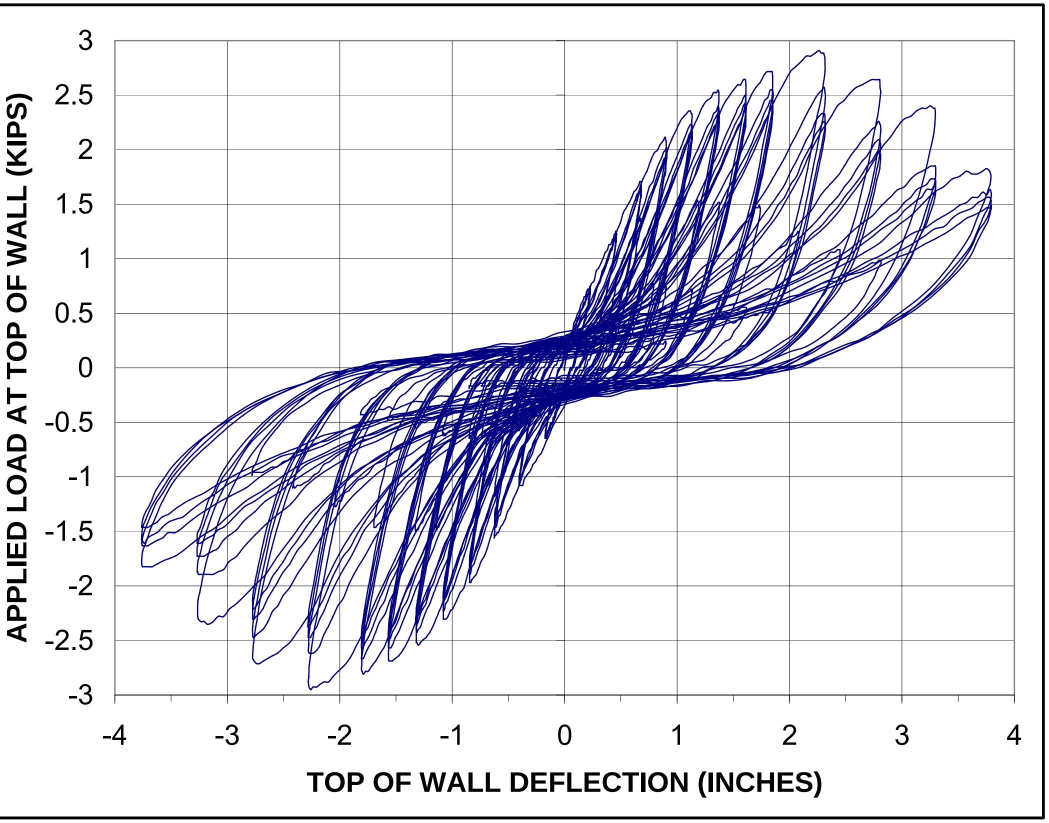
- Nailed joint between sheathing and framing is source of majority of ductility and energy dissipation for nailed wood structural panel shear walls.
- The energy dissipation is a combination of yielding in the shank of the nail, and crushing in the wood fibers surrounding the nail.
- Since wood crushing is nonrecoverable, this leads to a partial "pinching" effect in the hysteretic behavior of the joint.
- The pinching isn’t 100% because of the strength of the nail shank undergoing reversed ductile bending yielding in the wood.
- As the joint cycles, joint resistance climbs above the pinching threshold when the nail "bottoms out" against the end of the previously crushed slot forming in the wood post.

Sources of Ductility and Energy Dissipation in Wood Structures

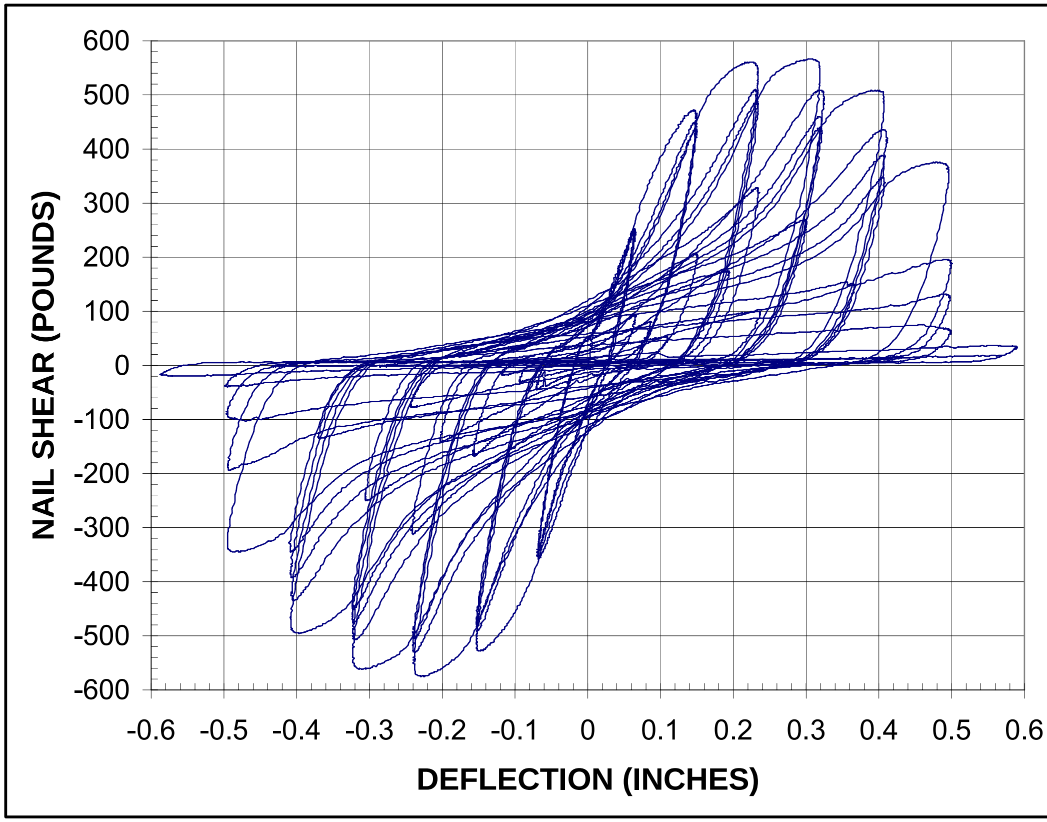


Individual nail test

Sources of Ductility and Energy Dissipation in Wood Structures



Full-scale shear wall test



Individual nail test

Vertical Elements of the LFRS: Prescriptive

NEHRP Section 12.4

- Numerous geometry limitations
- Two types of braced wall panel construction: gypsum wall board and wood structural panel

IRC 2003 Methods

- Numerous geometry limitations
- Numerous types of braced wall panel construction: NEHRP methods + ~10 more

Vertical Elements of the LFRS: Engineered

NEHRP Methods

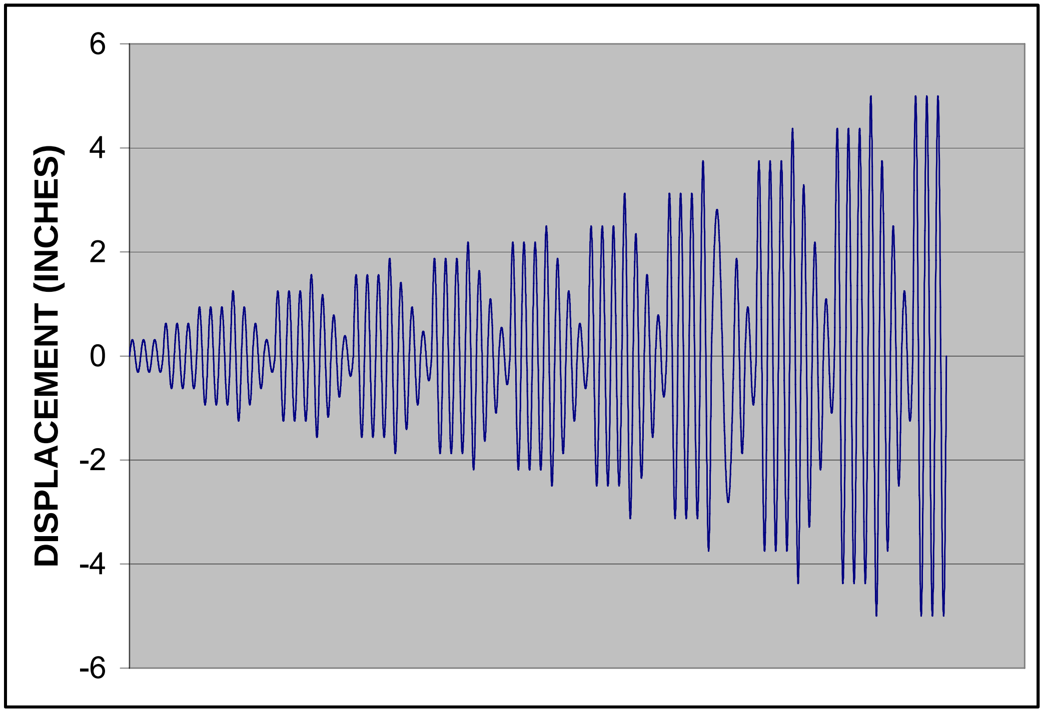
- Nailed/stapled wood structural panel
- Cold-formed steel with flat strap tension-only bracing
- Cold-formed steel with wood structural panel screwed to framing

IBC 2003 Methods

- Nailed wood structural panel shear walls
- Sheet steel shear walls
- Ordinary steel braced frames
- All others: gypsum and stucco
- Proprietary shear walls

Wall Performance Based on Testing

- **First cyclic protocol to be adopted in the US for cyclic testing of wood shear walls.**
- **62 *post yield* cycles.**
- **Found to demand too much energy dissipation compared with actual seismic demand.**
- **Can result in significant underestimation of peak capacity and displacement at peak capacity.**



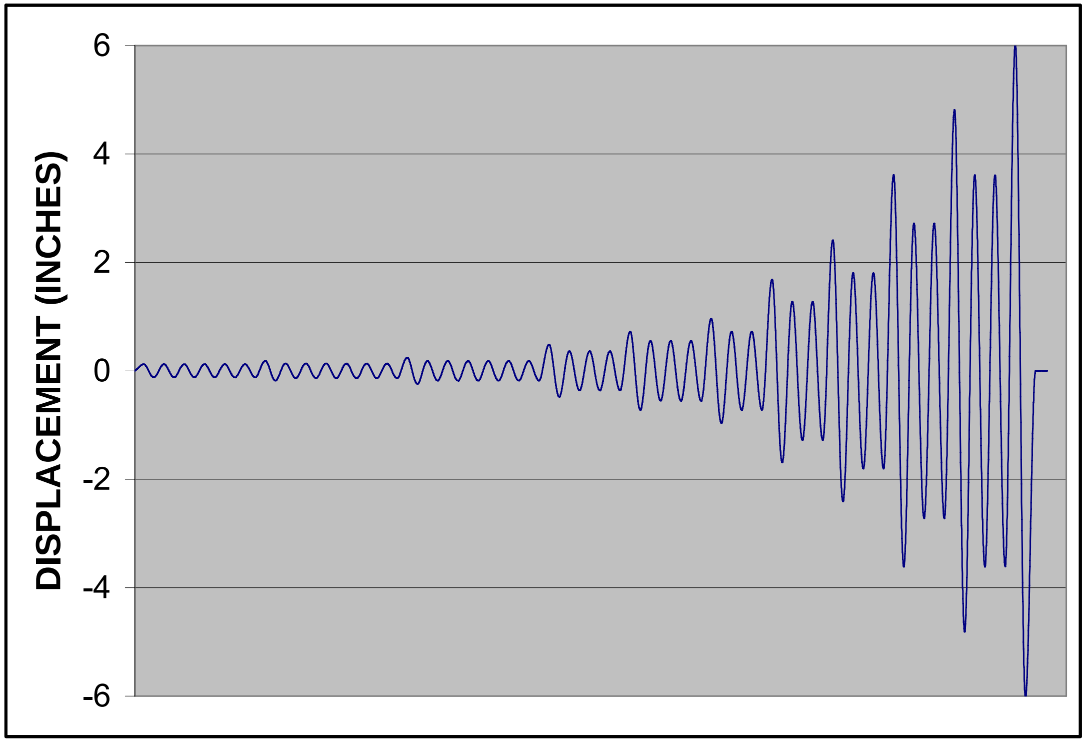
Cyclic Test Protocols
TCCMAR (SPD)

Wall Performance Based on Testing

- Developed by researchers at Stanford University as part of the CUREE/Caltech Woodframe Project

.

- Based on nonlinear time history analysis of wood structures considering small "non-design" vents preceding the "design event."
- Currently the "state-of-the-art" in cyclic test protocols.
- More realistically considers actual energy and displacement demands from earthquakes.



Cyclic Test Protocols -- CUREE

Code Basis of Design Values

Nailed Wood Structural Panel Shear Walls

- Values currently in the code were developed by the APA – The Engineered Wood Association (used to be the American Plywood Association) in the 1950s.
- These values are based on a principles of mechanics approach.
- Some monotonic testing was run to validate procedure.
- Testing was conducted on 8'x8' walls (1:1 aspect ratio), with very rigid overturning restraint.
- Test was more of a sheathing test, not shear wall system test.
- Extrapolation of use down to 4:1 aspect ratio panels proved problematic on 1994 Northridge earthquake.
- Code now contains provisions to reduce the design strength of walls with aspect ratios (AR's) > 2:1 by multiplying the base strength by a factor of 2 / AR.

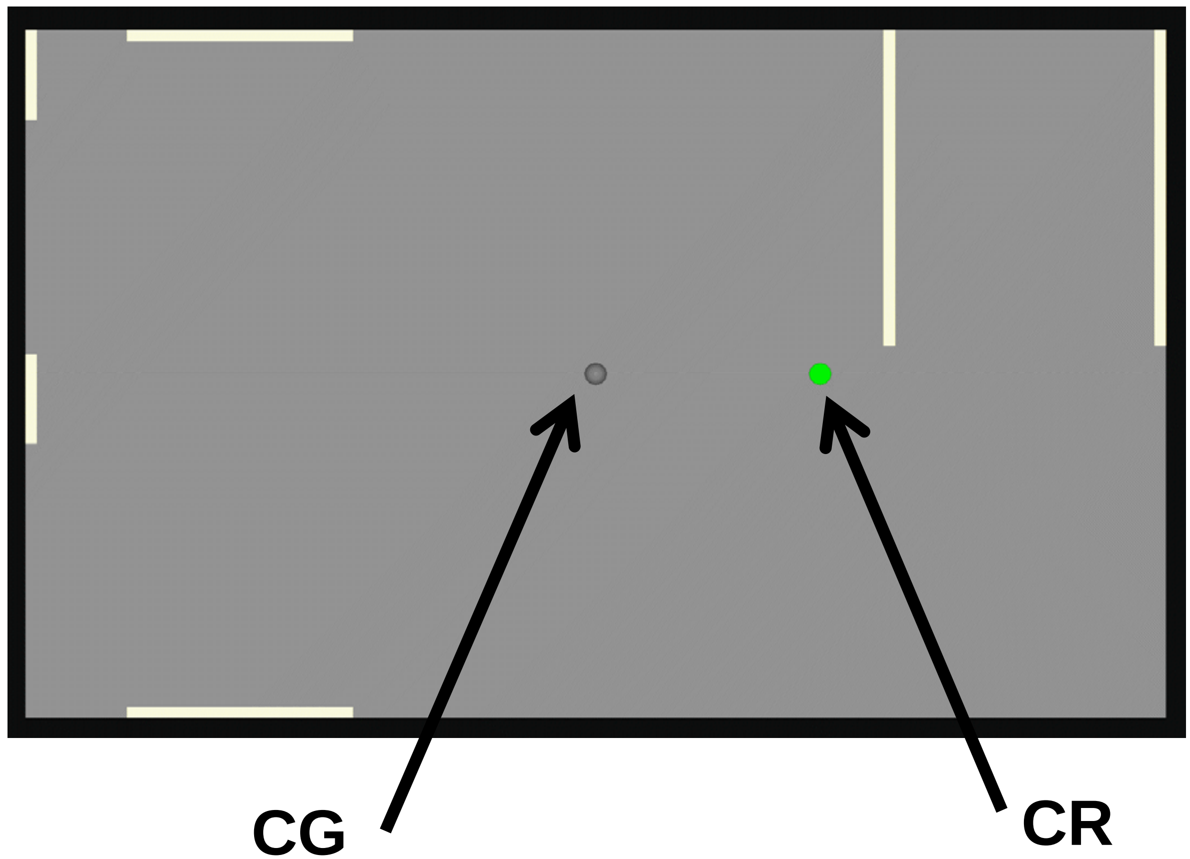
Code Basis of Design Values

Proprietary Wood Structural Panel Shear Walls:

- Proprietary shear wall systems for light frame construction have been developed to provide higher useable strength when the AR exceeds 2:1.
- Values are determined according to Acceptance Criteria 130 (AC130) developed by the International Code Council Evaluation Services (ICC ES).
- AC130 requires full-scale cyclic testing of the wall seeking approval based on either SPD or CUREE protocols.
- Design rating based on either strength (ultimate / safety factor) or displacement (deflection which satisfies code deflection limits based on C_d, the deflection amplification factor associated with the rated R factor, and the appropriate maximum allowed inelastic drift ratio).

Typical Woodframe Analysis Methods

- **Flexible diaphragm analysis**
- **Rigid diaphragm analysis**

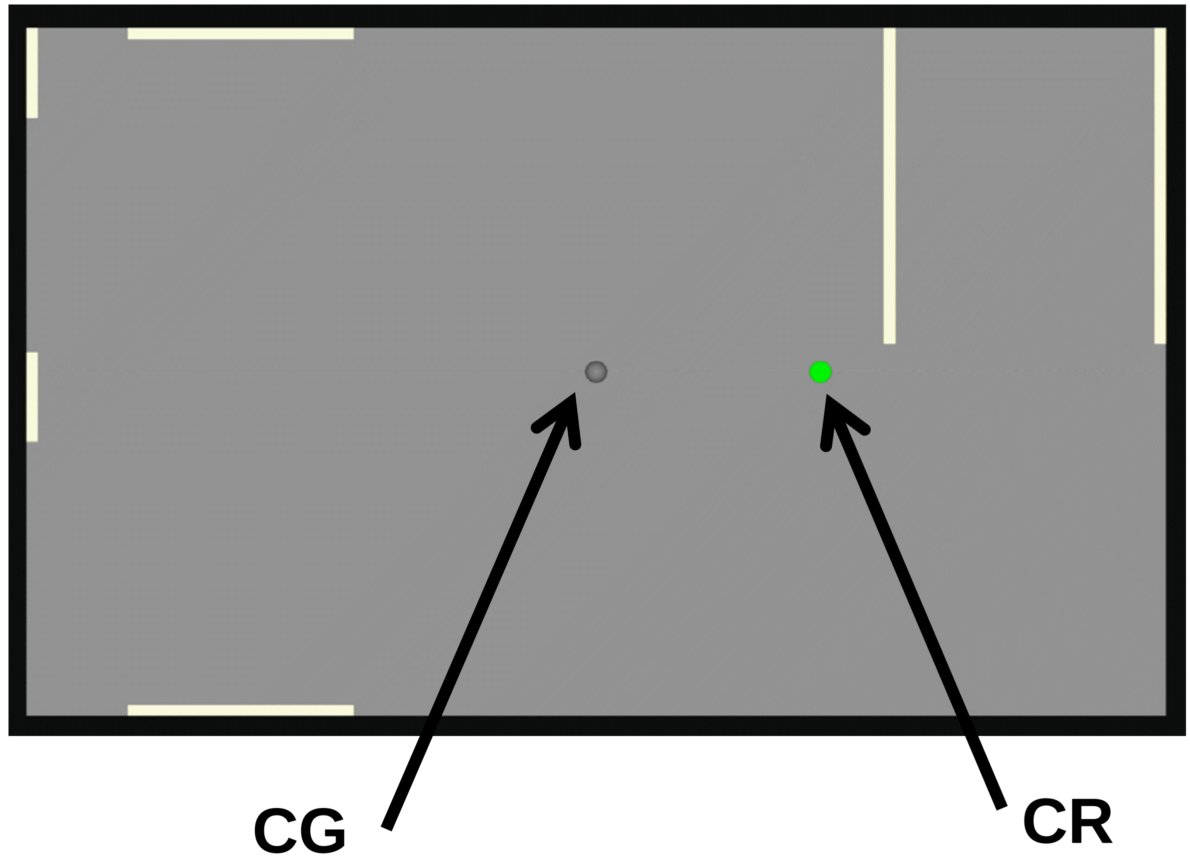


- **Worry about it??**

Typical Woodframe Analysis Methods

Flexible Diaphragm Analysis

- Lateral loads distributed as if diaphragm is a simple span beam between lines of lateral resistance.
- Diaphragm loads are distributed to lines of shear resistance based on tributary area between lines of shear resistance.

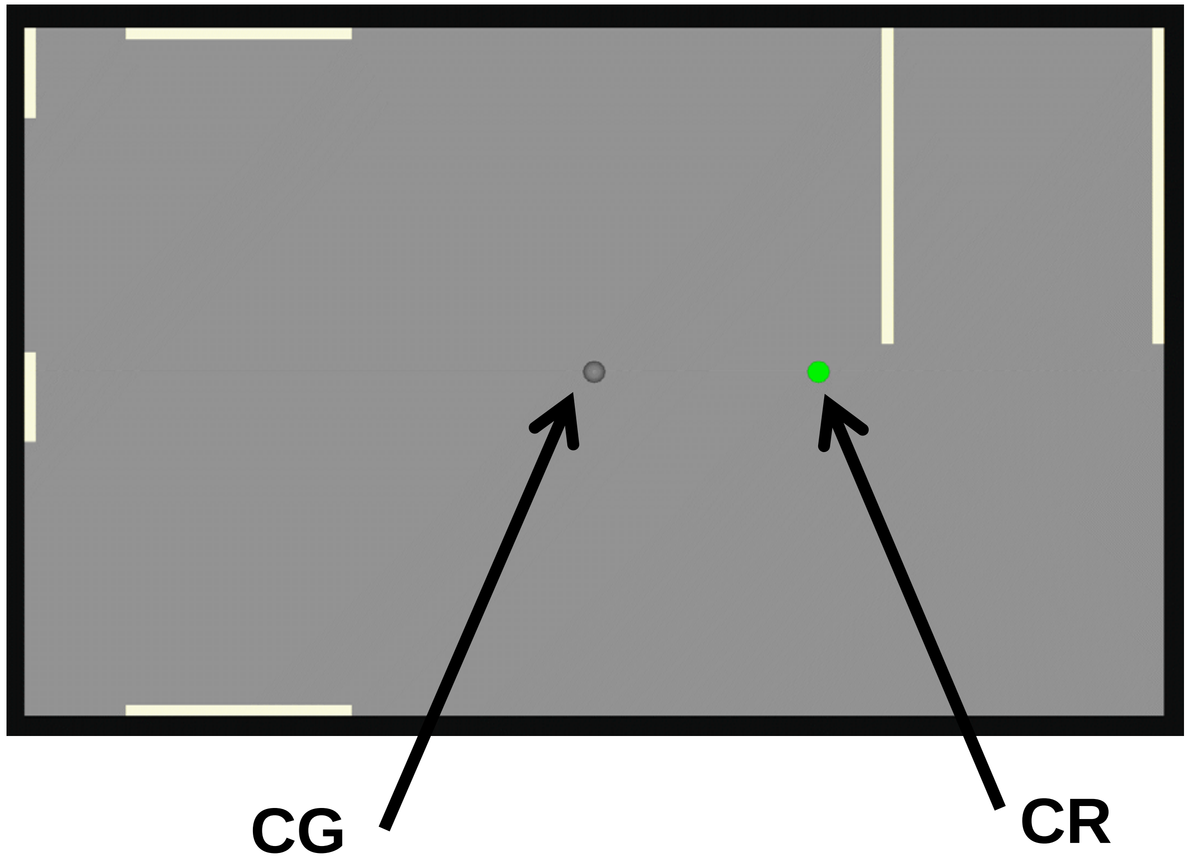


- Worry about it??
- No

Typical Woodframe Analysis Methods

Rigid Diaphragm Analysis

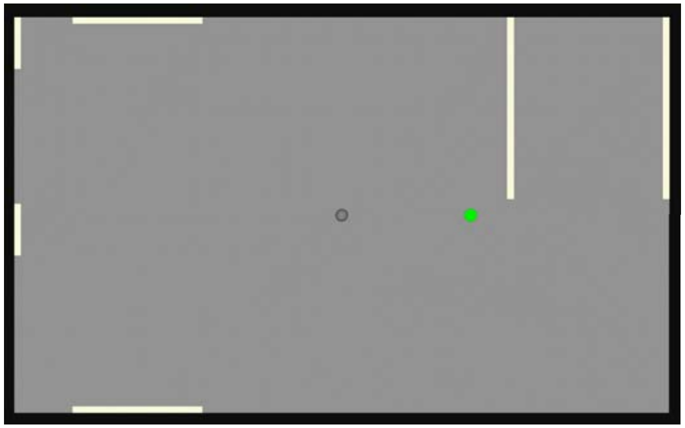
- **Lateral loads distributed as if diaphragm is rigid, rotating around the CR.**
- **Force in shear walls is a combination of translational and rotational shear.**



- **Worry about it??**
- **Yes**

Typical Woodframe Analysis Methods

Comments on Analysis Methods



- Neither the rigid nor flexible diaphragm methods really represent the distribution of lateral resistance in a typical structure.
- Both methods (typically) ignore the stiffness distribution of interior and exterior wall finishes.

- Wood structural diaphragms are neither "flexible" or "rigid" – they are somewhere in between. "Glued and screwed" floor sheathing makes floors more rigid than flexible. The nailing of interior wall sill plates across sheathing joints has the same effect. Exterior walls can act as "flanges", further stiffening the diaphragm.
- However, encouraging rigid diaphragm analysis is also encouraging the design of structures with torsional response – may not be a good thing!

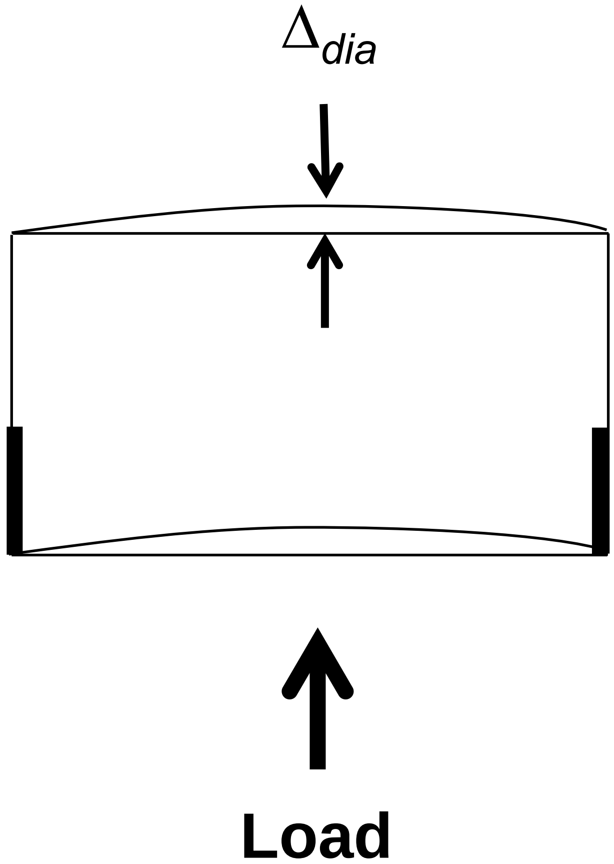
Rigid Diaphragms: When are they Rigid?

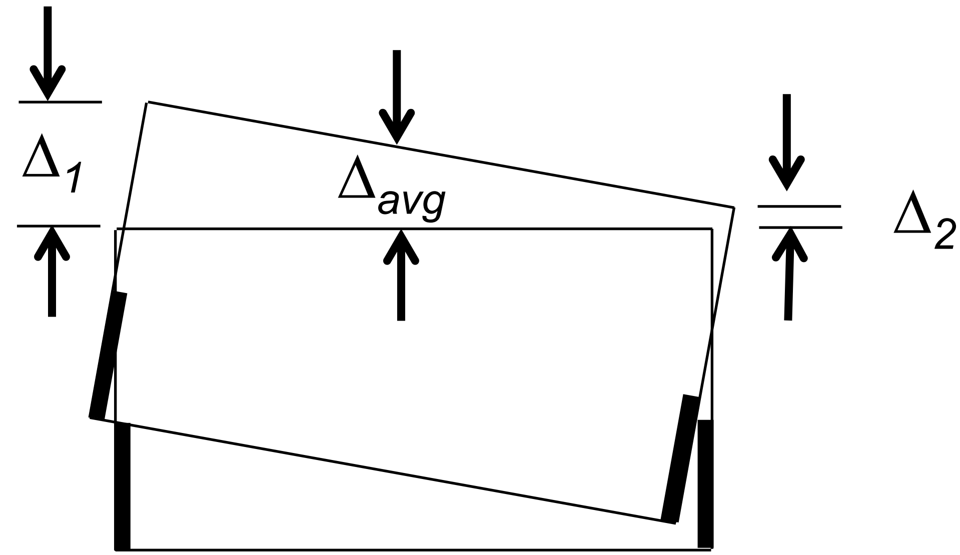
- **2003 *NEHRP Recommended Provisions* in Sec. 12.1.2.1 refers to the ASD/LRFD Supplement, *Special Design Provisions for Wind and Seismic*, American Forest and Paper Association, 2001:**

  "A diaphragm is rigid for the purposes of distribution of story shear and torsional moment when the computed maximum in-plane deflection of the diaphragm itself under lateral load is less than or equal to two times the average deflection of adjoining vertical elements of the lateral force-resisting system of the associated story under equivalent tributary lateral load." (Section 2.2, Terminology)

- **Same definition in 2003 IBC Sec. 1602.**

Rigid Diaphragms: When Are They Rigid?

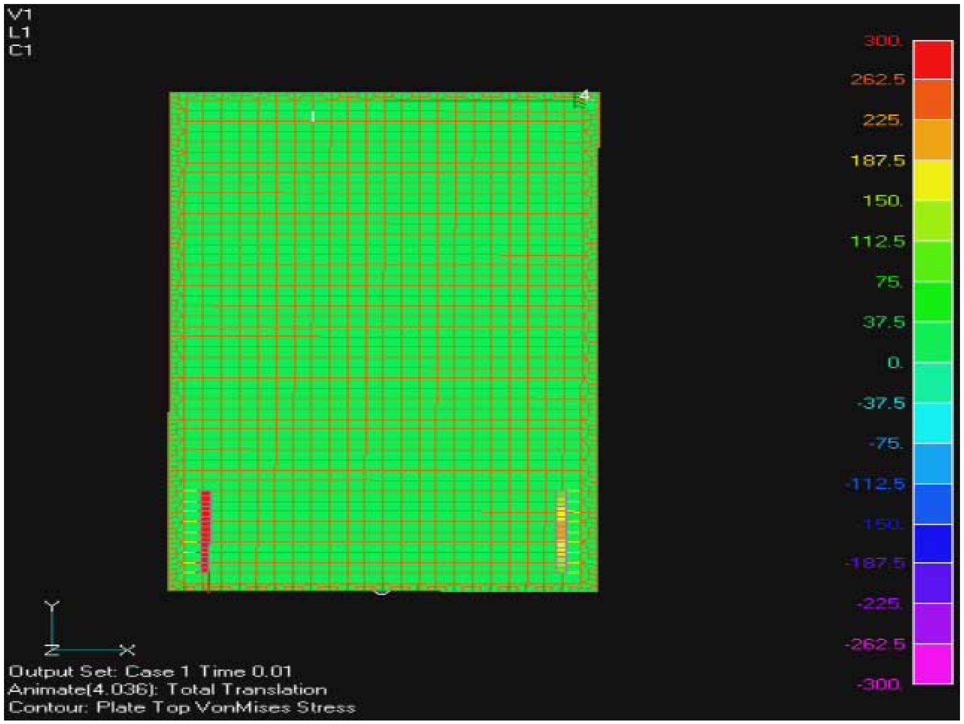


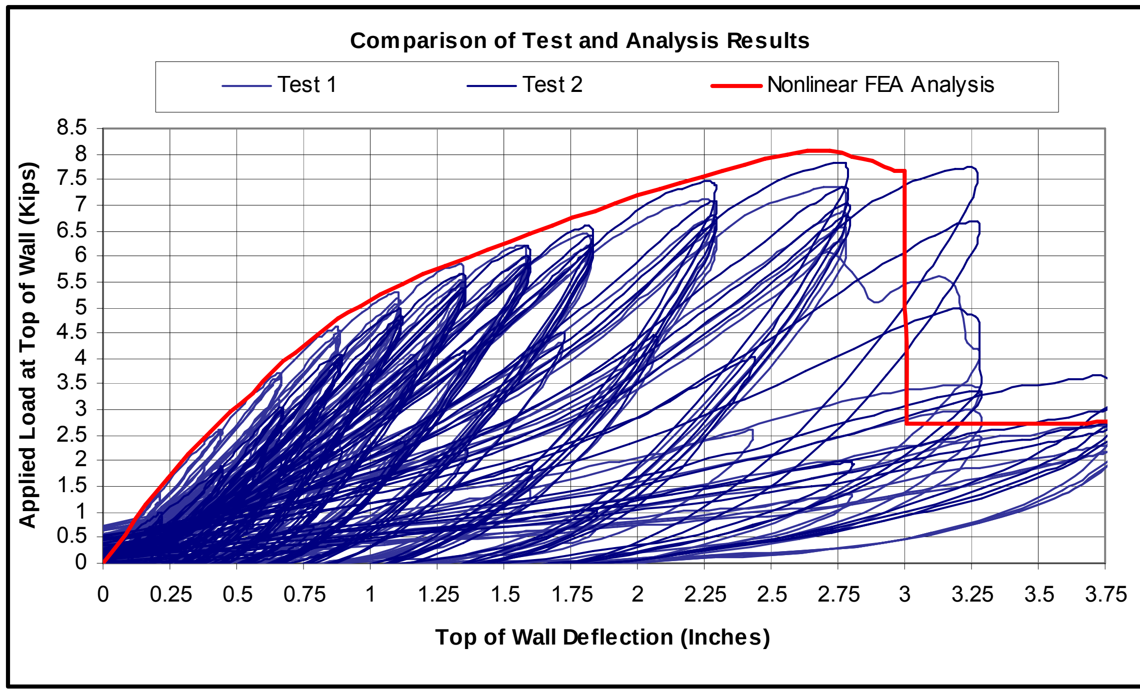


If Δ_{dia} [$2(\Delta_{avg})$ then diaphragm is classified as rigid

Advanced Analysis

- FEA : nail-level modeling is possible, with good correlation to full-scale testing.
- Requires a "true direction" nonlinear spring for the nails, as opposed to paired orthogonal springs.





Advanced Analysis

- NLTHA: rules based phenomenological elements fitted to full scale test data to predict structural response.
- Good correlation to simple tests – more work needed for complex, full structures.

Max Rel Disp		
Story	Predicted	Tested
1	1.14	1.57
2	2.65	2.3
3	1.76	1.92

Summary

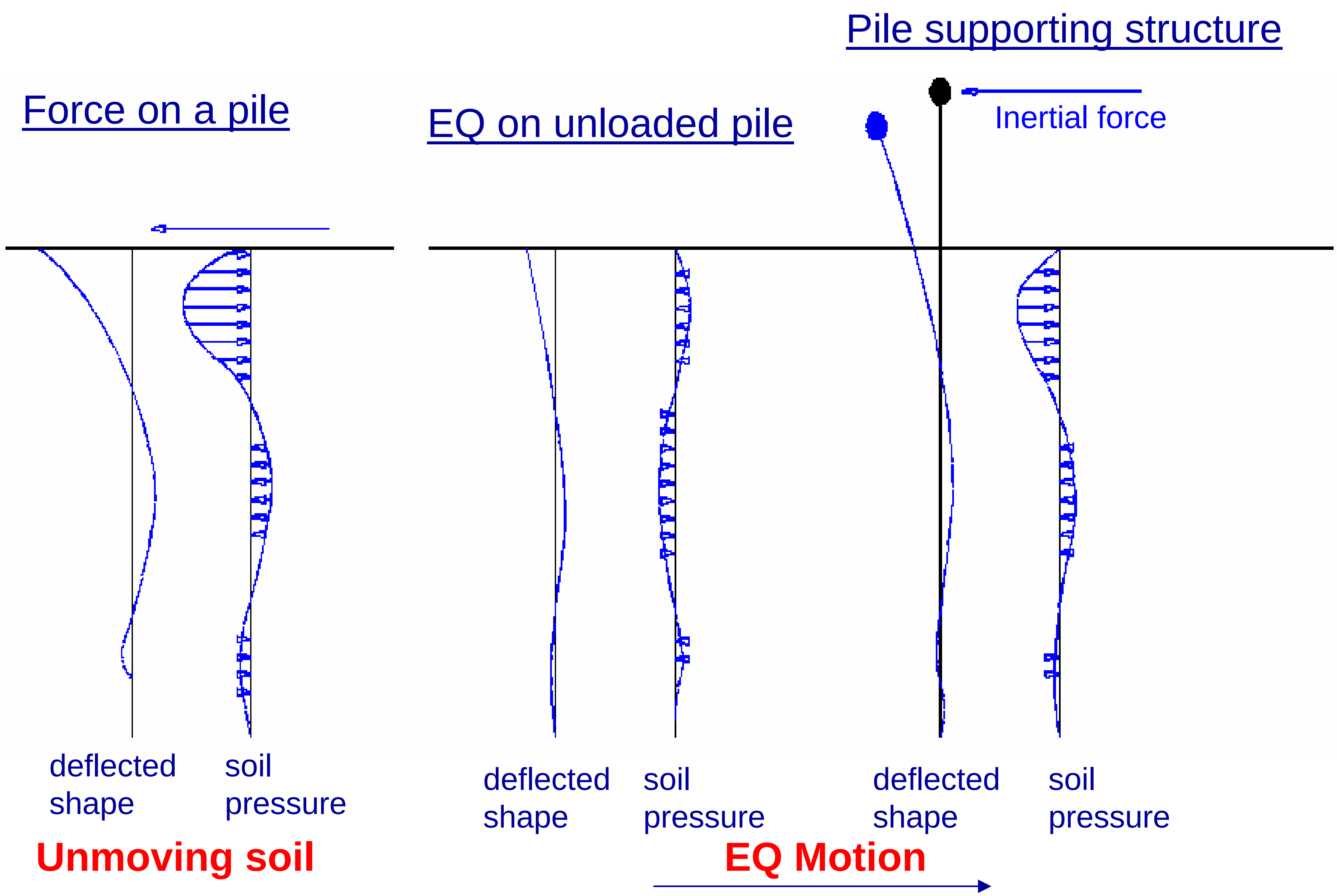
- Timber structures have a good track record of performance in major earthquakes
- Their low mass and good damping characteristics help achieve this.
- The orthotropic nature of wood, combined with the discontinuous methods of framing wood structures, requires careful attention to properly detailing the load path.
- There is still much room for improvement in our understanding of force distribution within wood structures, and the development of design tools to better model this.

FOUNDATION DESIGN

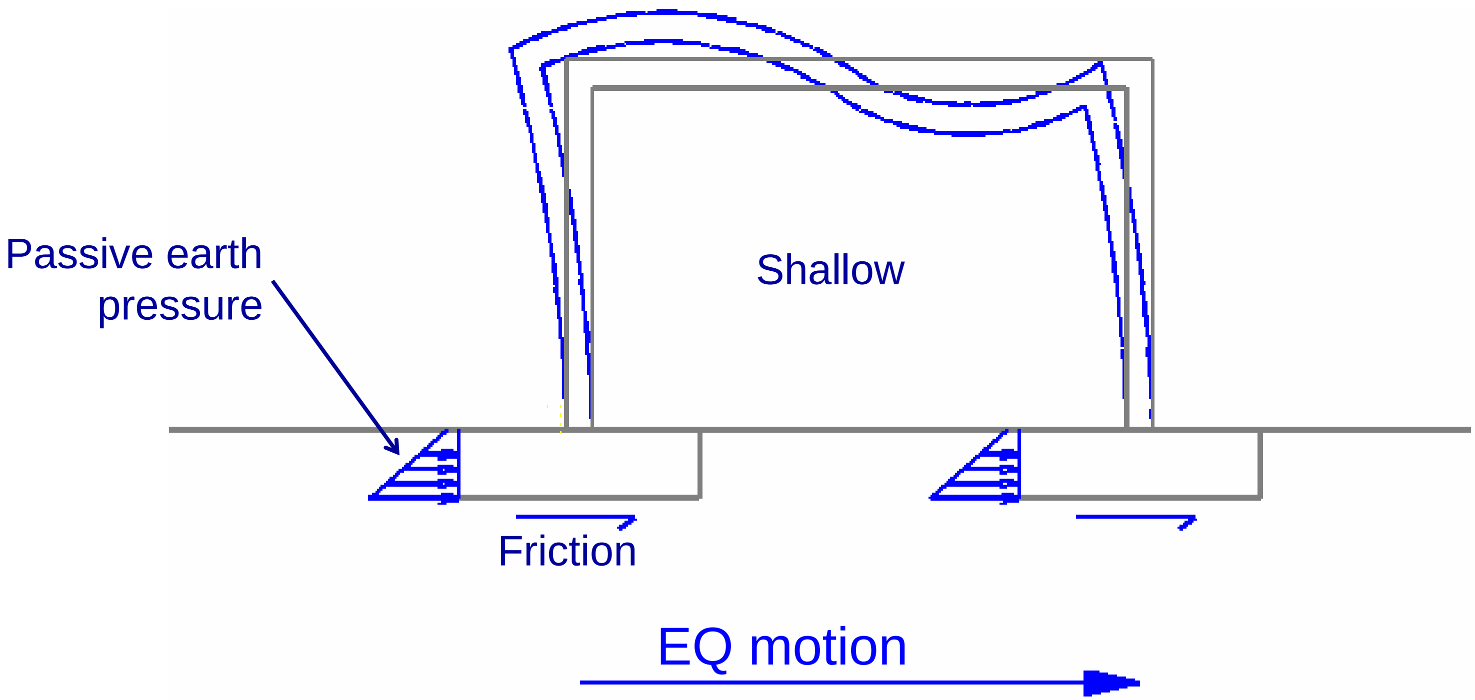
Proportioning elements for:

- **Transfer of seismic forces**
- **Strength and stiffness**
- **Shallow and deep foundations**
- **Elastic and plastic analysis**

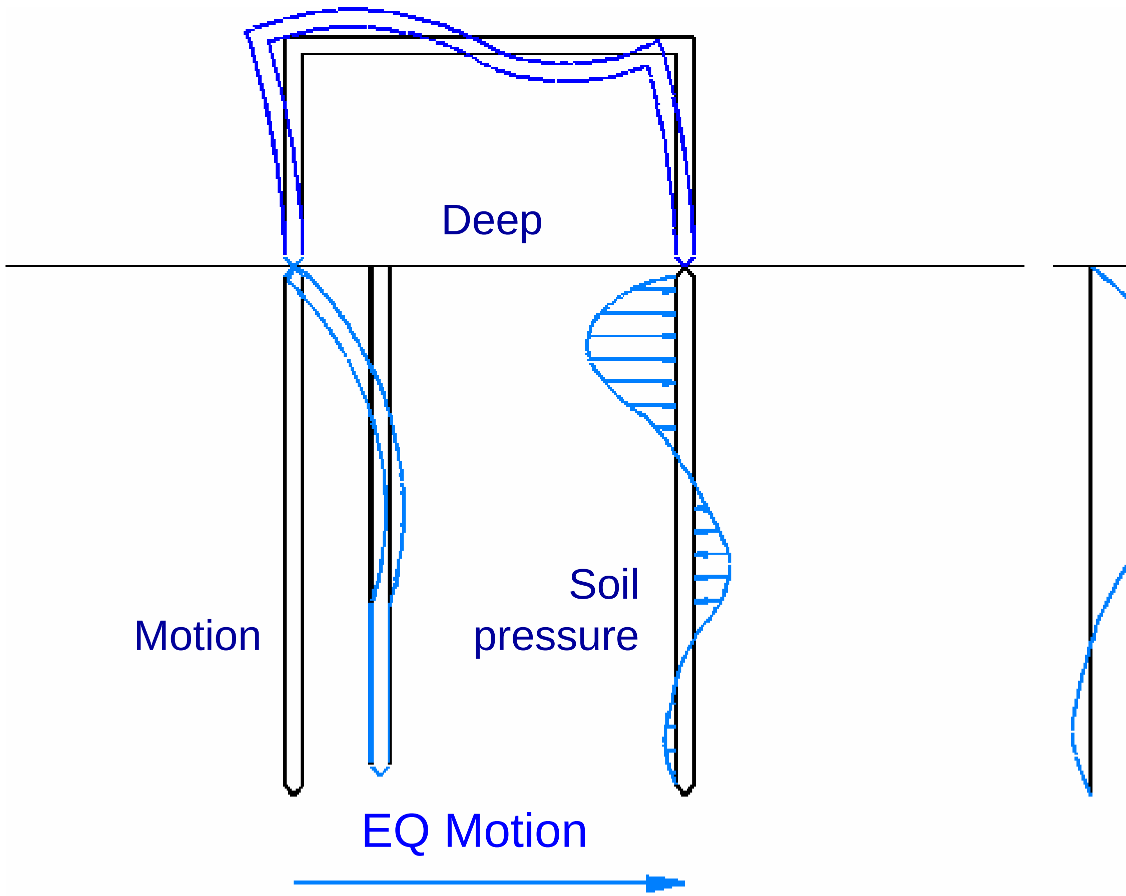
Load Path and Transfer to Soil
Soil Pressure



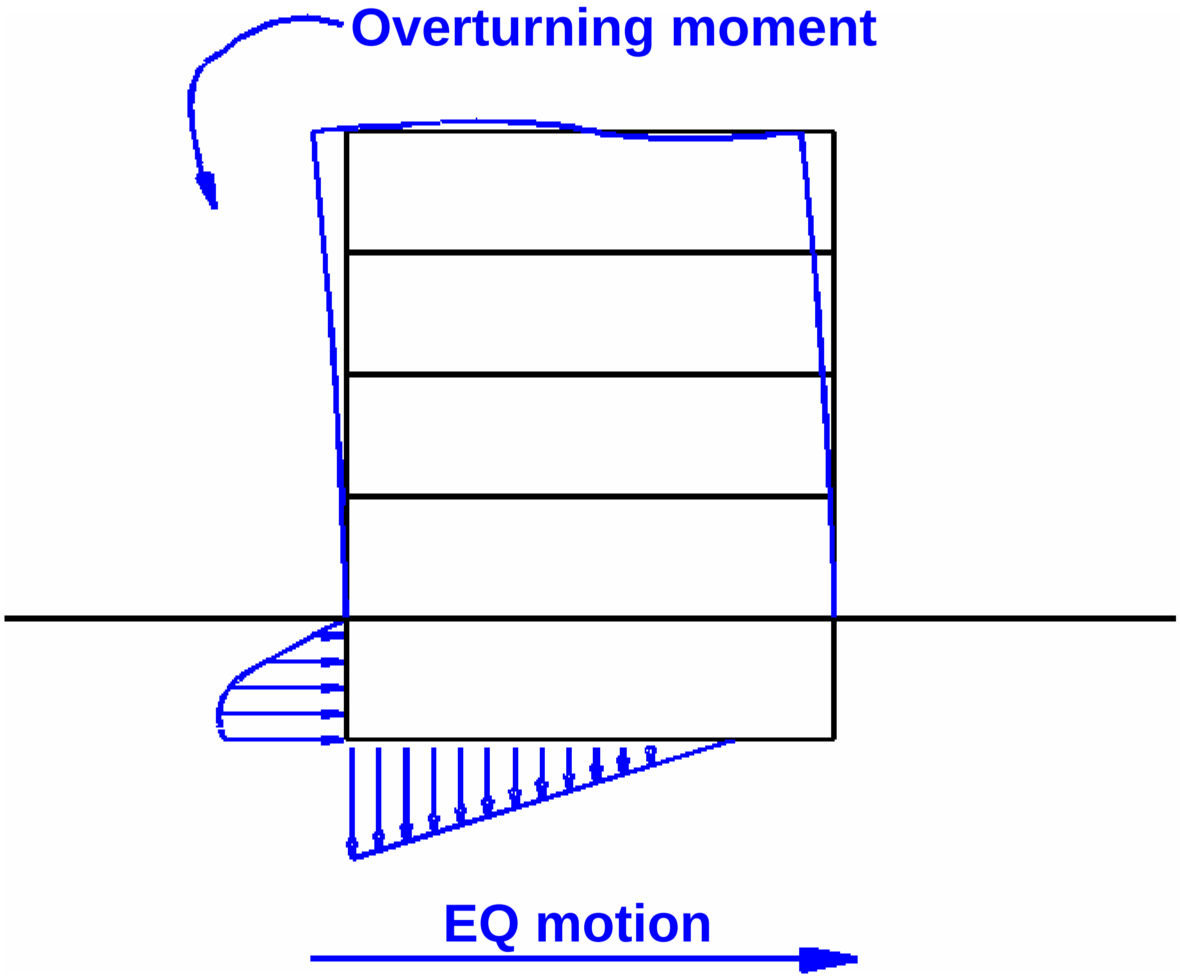
Load Path and Transfer to Soil
Soil-to-foundation Force Transfer



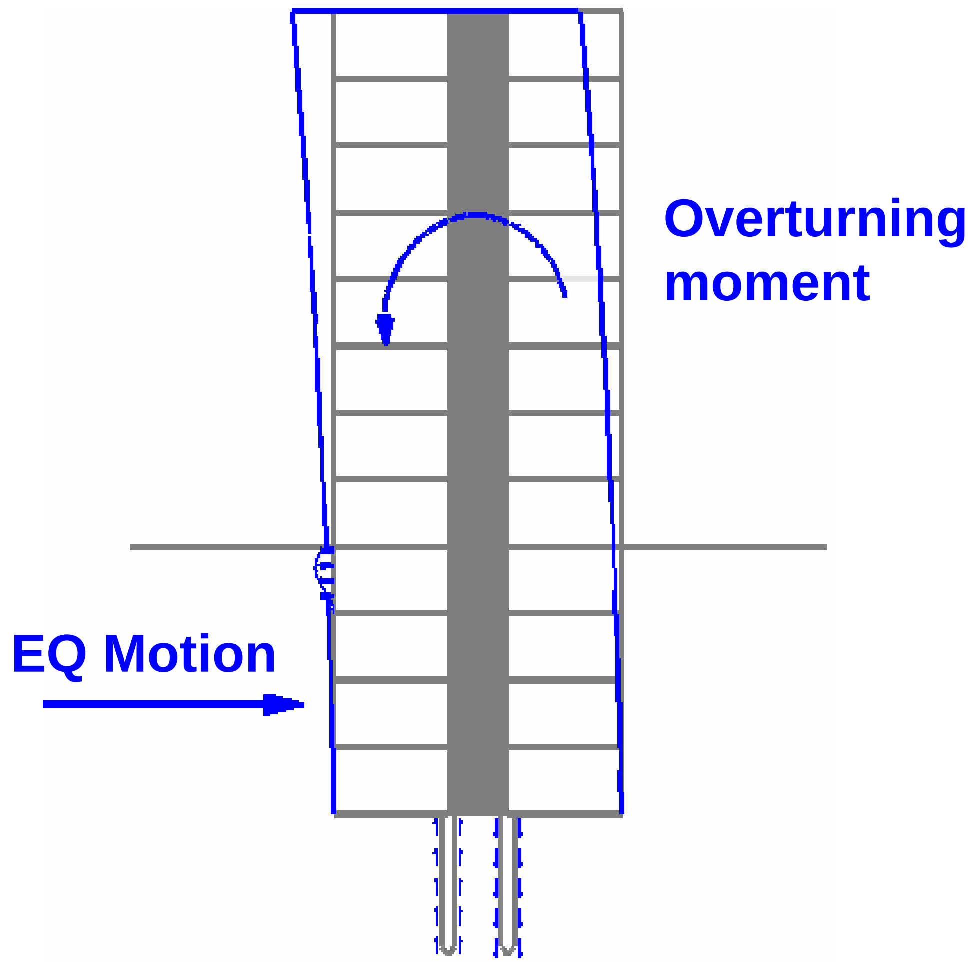
Load Path and Transfer to Soil
Soil-to-foundation Force Transfer



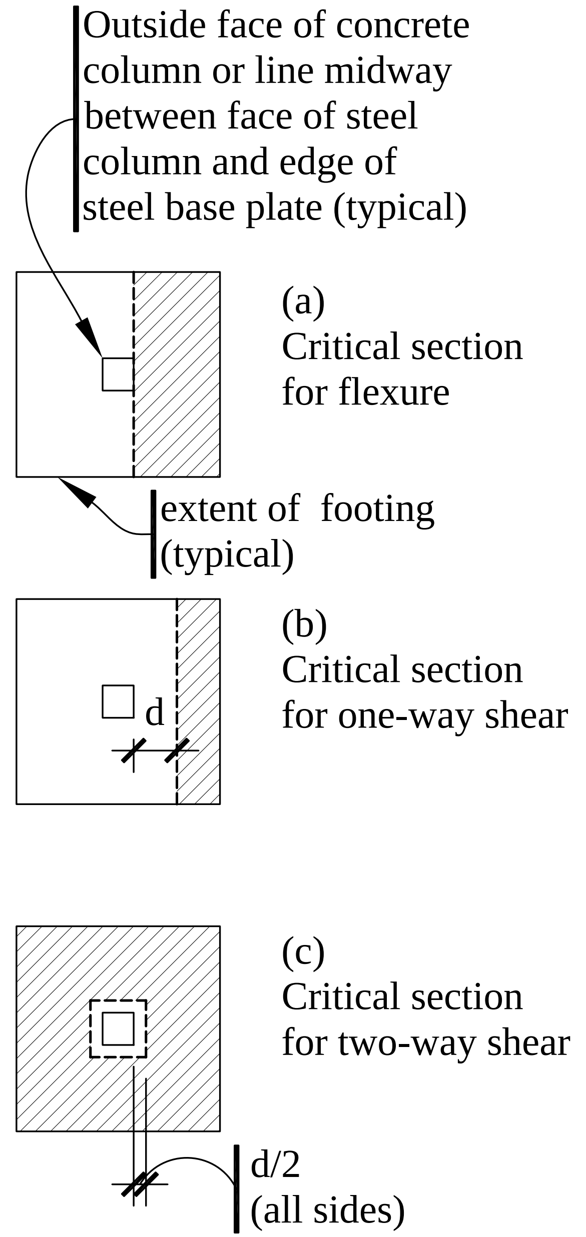
Load Path and Transfer to Soil
Vertical Pressures - Shallow



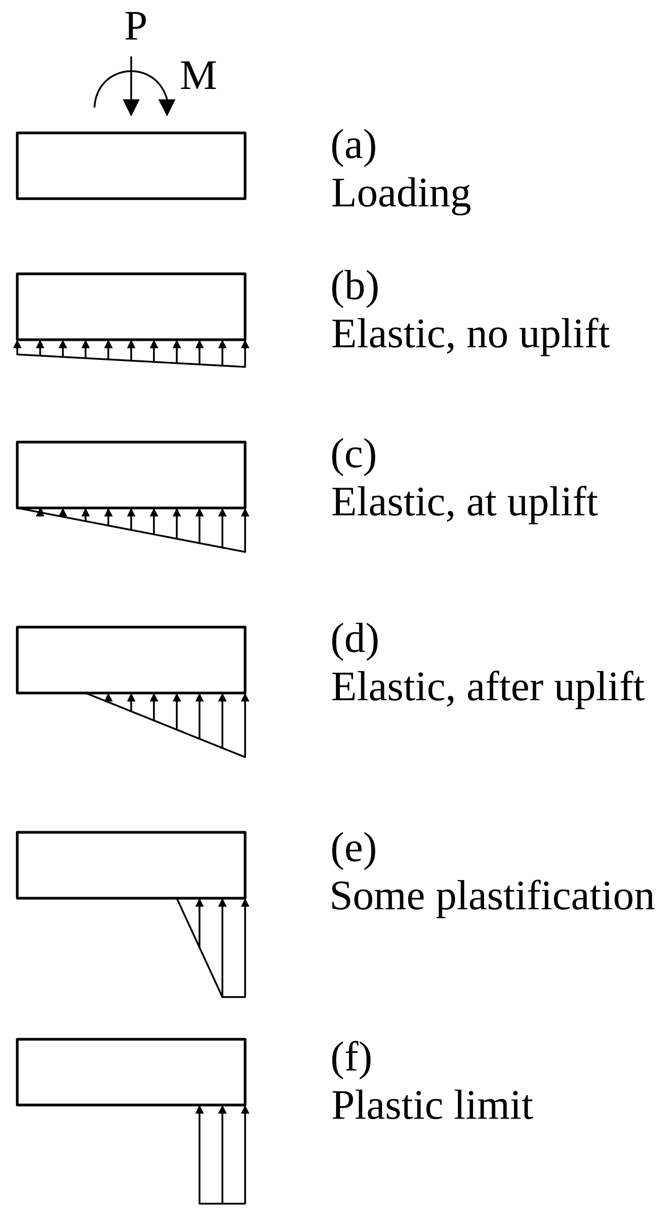
Load Path and Transfer to Soil
Vertical Pressures - Deep

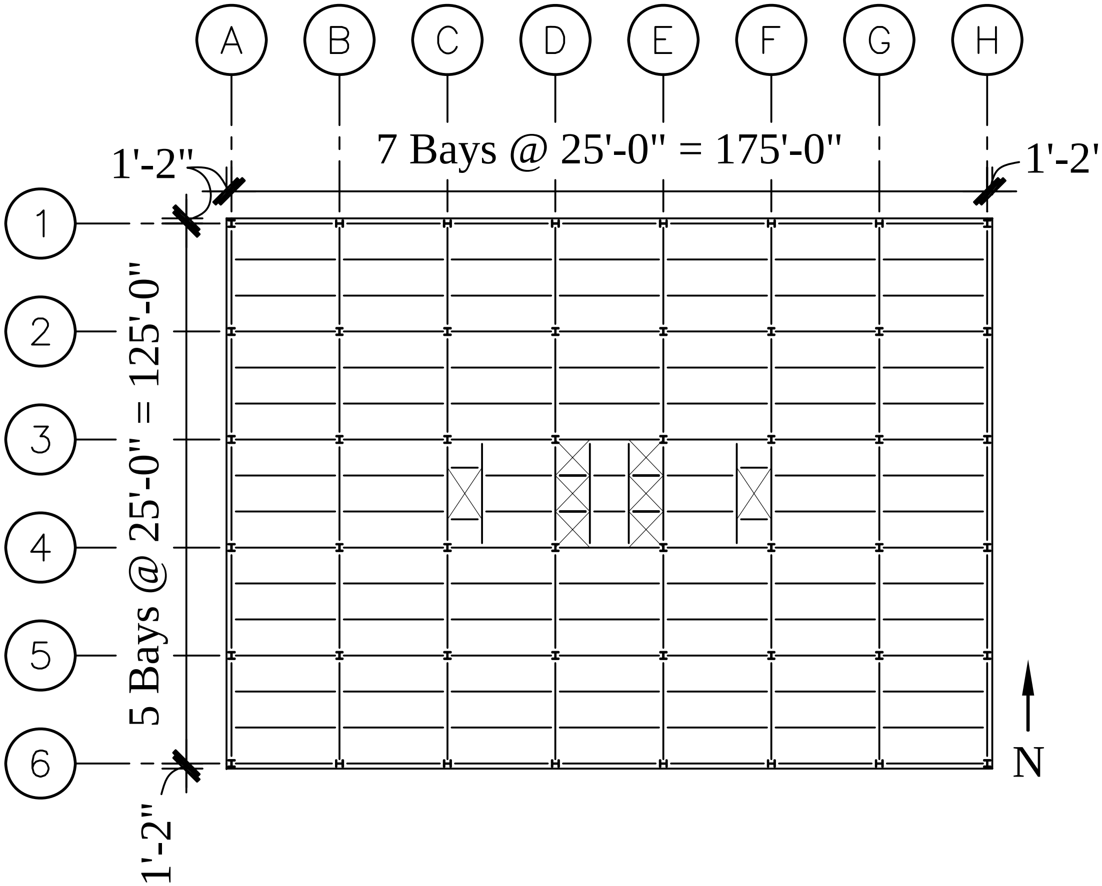


Reinforced Concrete Footings: Basic Design Criteria (concentrically loaded)



Footing Subject to Compression and Moment: Uplift Nonlinear





Example 7-story Building: Shallow foundations designed for perimeter frame and core bracing.

Shallow Footing Examples

Soil parameters:

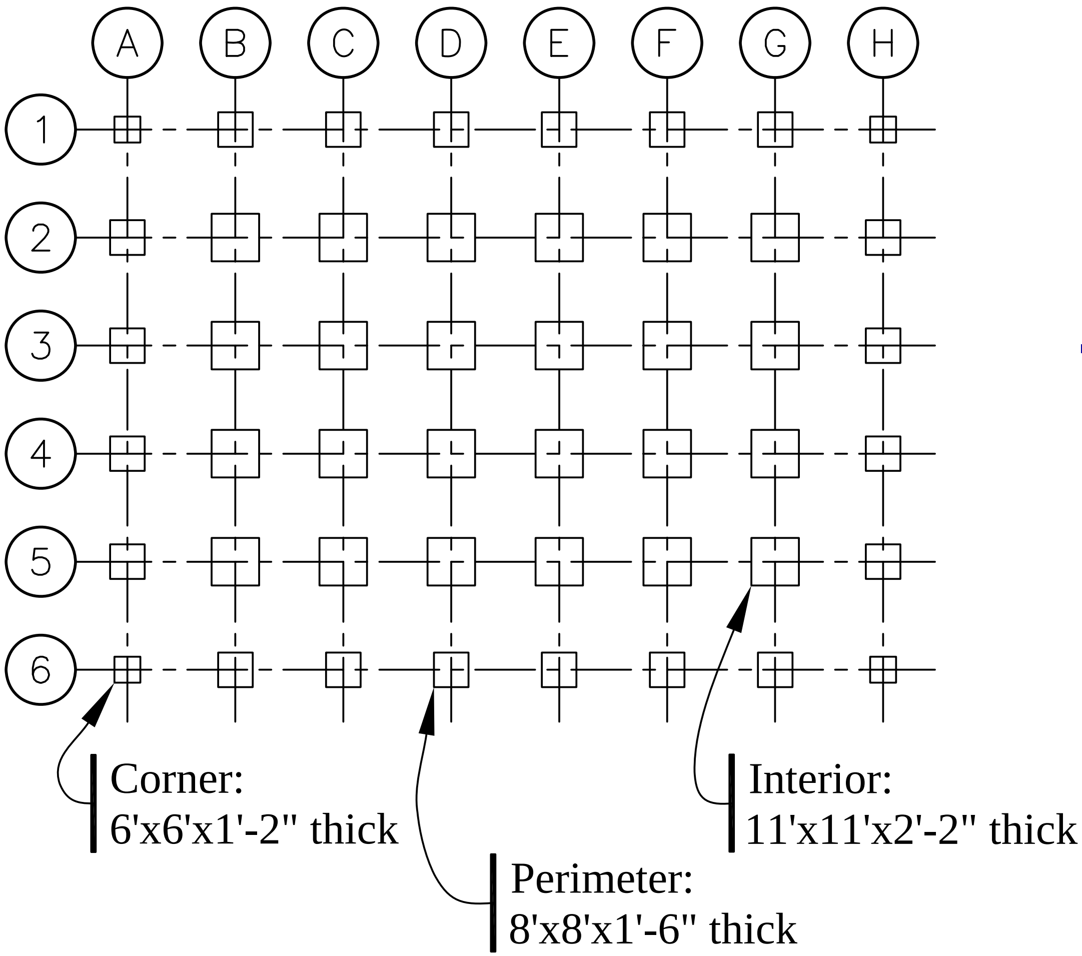
- Medium dense sand
- (SPT) N = 20
- Density = 120 pcf
- Friction angle = 33º

Gravity load allowables

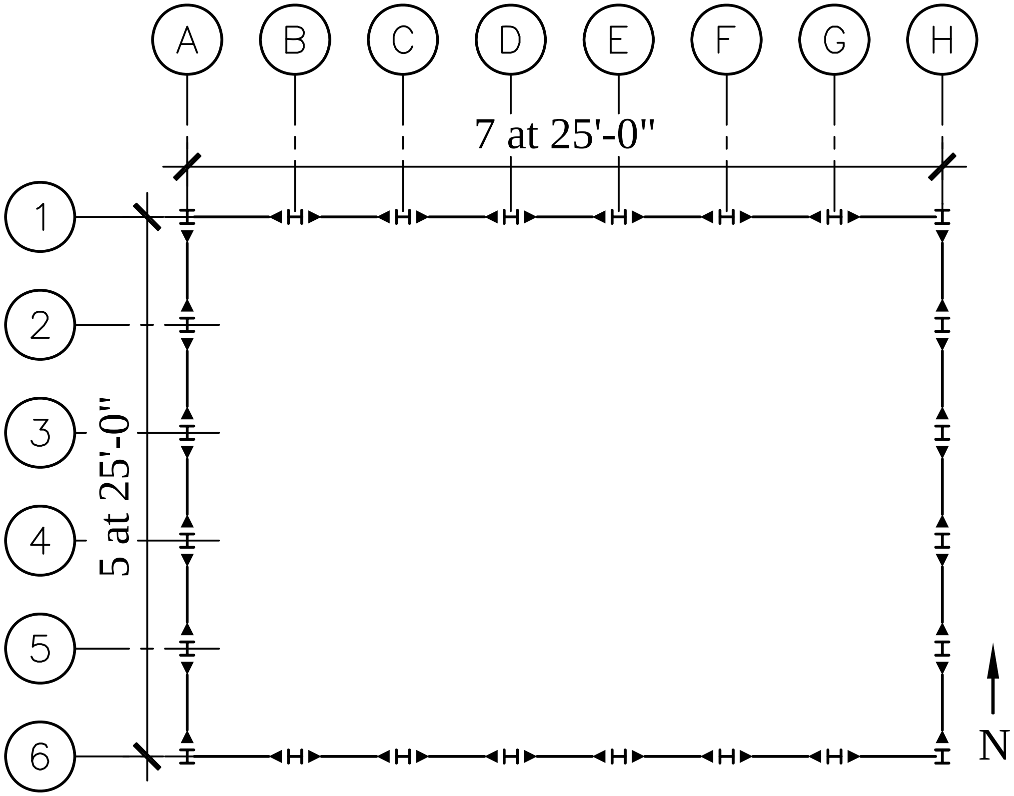
- 4000 psf, B < 20 ft
- 2000 psf, B > 40 ft

Bearing capacity (EQ)

- 2000*B* concentric sq.
- 3000*B* eccentric
- $\phi = 0.6$

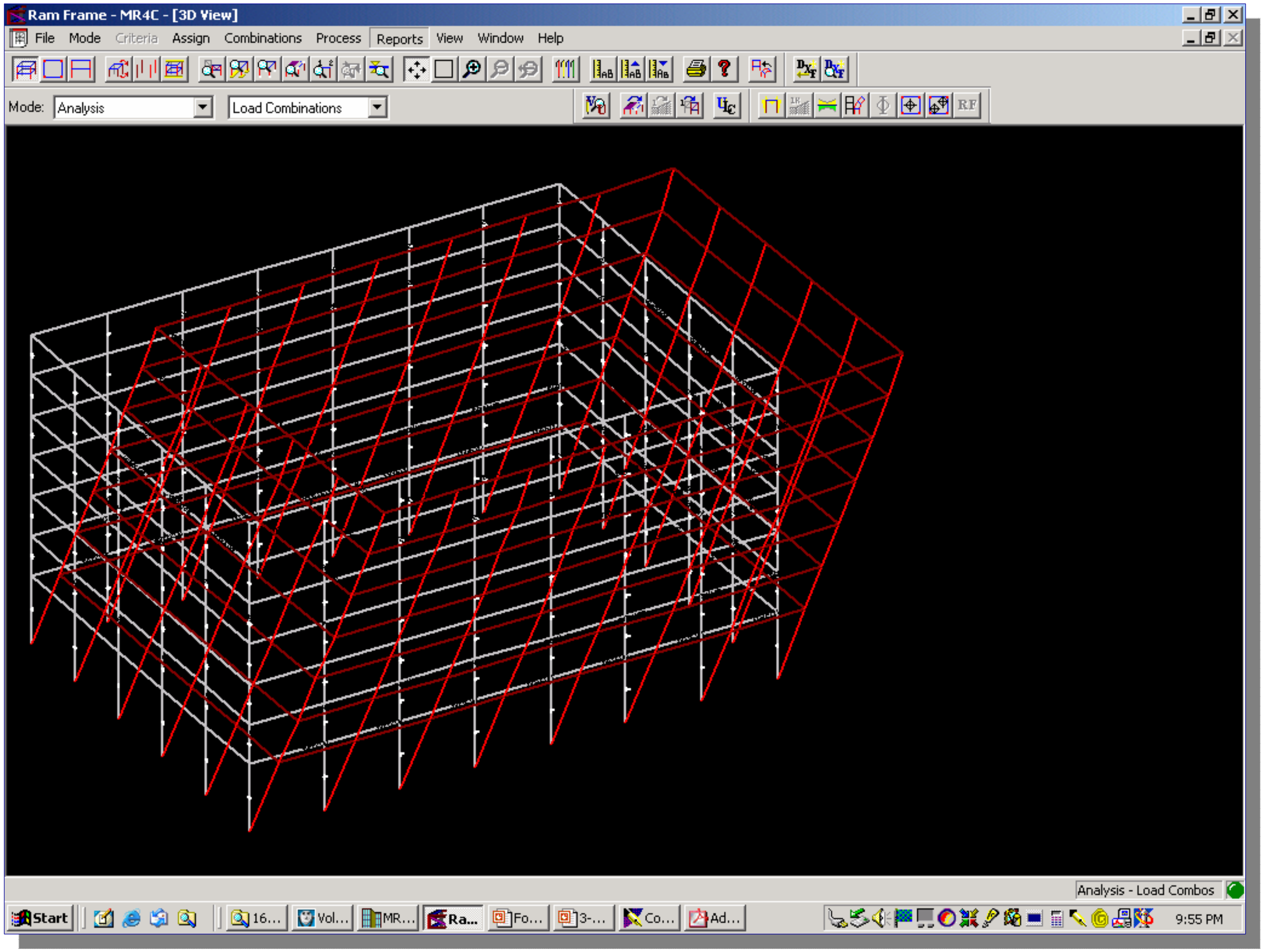


Footings proportioned for gravity loads alone



Design of Footings for Perimeter Moment Frame

7-Story Frame, Deformed



Combining Loads

- Maximum downward load:

  $1.2D + 0.5L + E$

- Minimum downward load:

  $0.9D + E$

- Definition of seismic load effect E:

  $E = \rho_1 Q_{E1} + 0.3\ \rho_2 Q_{E2} +/- 0.2\ S_{DS} D$

  $\rho_x = 1.08 \quad \rho_y = 1.11$ and $S_{DS} = 1.0$

Reactions

Grid		Dead	Live	E_x	E_y
A-5	P M_{xx} M_{yy}	203.8 k	43.8 k	-3.8 k 53.6 k-ft -243.1 k-ft	21.3 k -1011.5 k-ft 8.1 k-ft
A-6	P M_{xx} M_{yy}	103.5 k	22.3 k	-51.8 k 47.7 k-ft -246.9 k-ft	-281.0 k -891.0 k-ft 13.4 k-ft

Reduction of Overturning Moment

- *NEHRP Recommended Provisions* allow base overturning moment to be reduced by 25% at the soil-foundation interface.
- For a moment frame, the column vertical loads are the resultants of base overturning moment, whereas column moments are resultants of story shear.
- Thus, use 75% of seismic vertical reactions.

Additive Load w/ Largest Eccentricity

- At A5: $P = 1.4(203.8) + 0.5(43.8) + 0.75(0.32(-3.8) + 1.11(21.3)) = 324$ k

  $M_{xx} = 0.32(53.6) + 1.11(-1011.5) = -1106$ k-ft
- At A6: $P = 1.4(103.5) + 0.5(22.3) + 0.75(0.32(-51.8) + 1.11(-281)) = -90.3$ k

  $M_{xx} = 0.32(47.7) + 1.11(-891) = -974$ k-ft
- Sum $M_{xx} = 12.5(-90.3-324) -1106 -974 = -7258$

Counteracting Load with Largest *e*

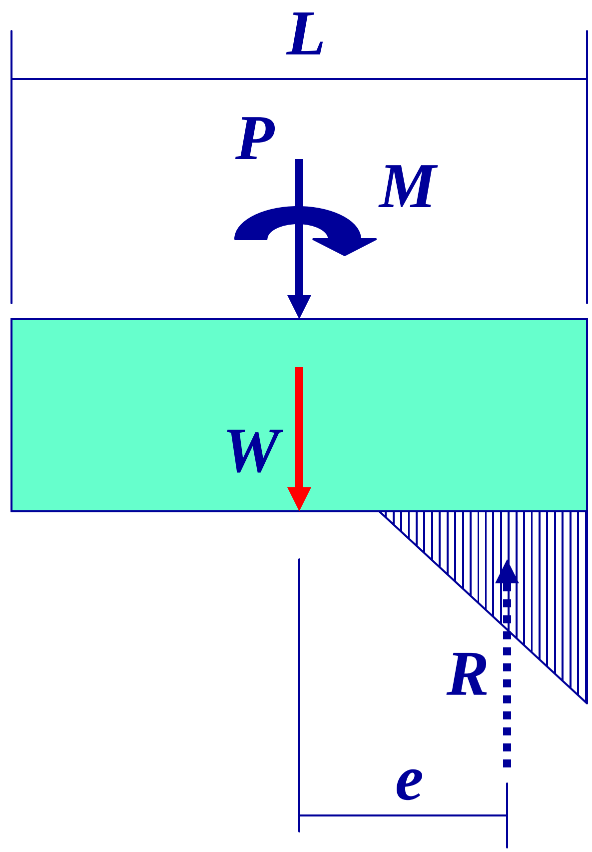
- At A-5: P = 0.7(203.8) + 0.75(0.32(-3.8) + 1.11(21.3)) = 159.5 k

  M_{xx} = 0.32(53.6) + 1.11 (-1011.5) = -1106 k-ft
- At A-6: P = 0.7(103.5) + 0.75(0.32(-51.8) + 1.11(-281)) = -173.9 k

  M_{xx} = 0.32(47.7) + 1.11(-891) = -974 k-ft
- Sum M_{xx} = 6240 k-ft

Elastic Response

- Objective is to set *L* and *W* to satisfy equilibrium and avoid overloading soil.
- Successive trials usually necessary.



Additive Combination

Given P = 234 k, M =7258 k-ft

Try 5 foot around, thus L = 35 ft, B = 10 ft

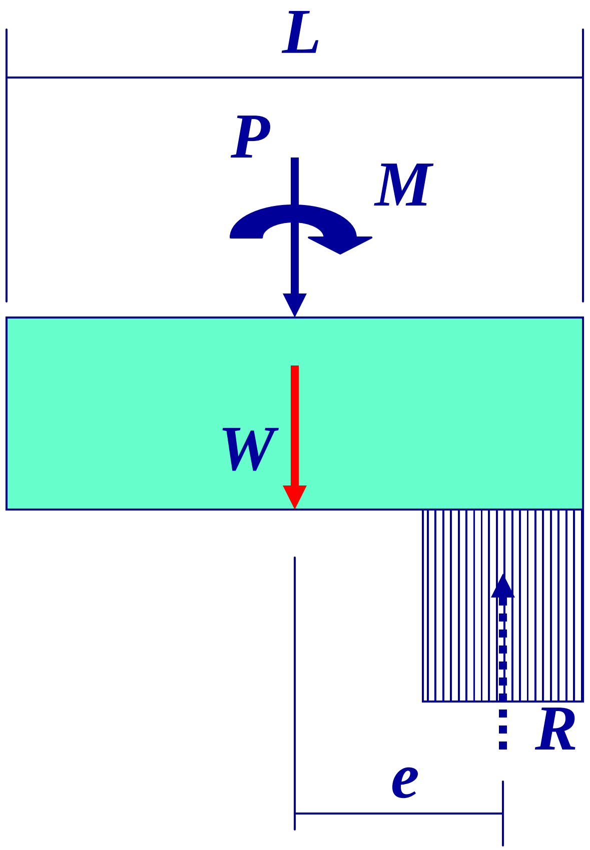
- Minimum $W = M/(L/2) – P$ = 181 k = 517 psf

Try 2 foot soil cover & 3 foot thick footing

- W = 245 k; for additive combo use 1.2W
- $Q_{max} = (P + 1.2W)/(3(L/2 – e)B/2)$ = 9.4 ksf
- $\phi Q_n = 0.6(3)B_{min}$ = 10.1 ksf, OK by Elastic

Plastic Response

- Same objective as for elastic response.
- Smaller footings can be shown OK thus:



Counteracting Case

Given P = -14.4 k; M = 6240

Check prior trial; W = 245 k (use 0.9W)

- e = 6240/(220.5 – 14.4) = 30.3 > 35/2 NG

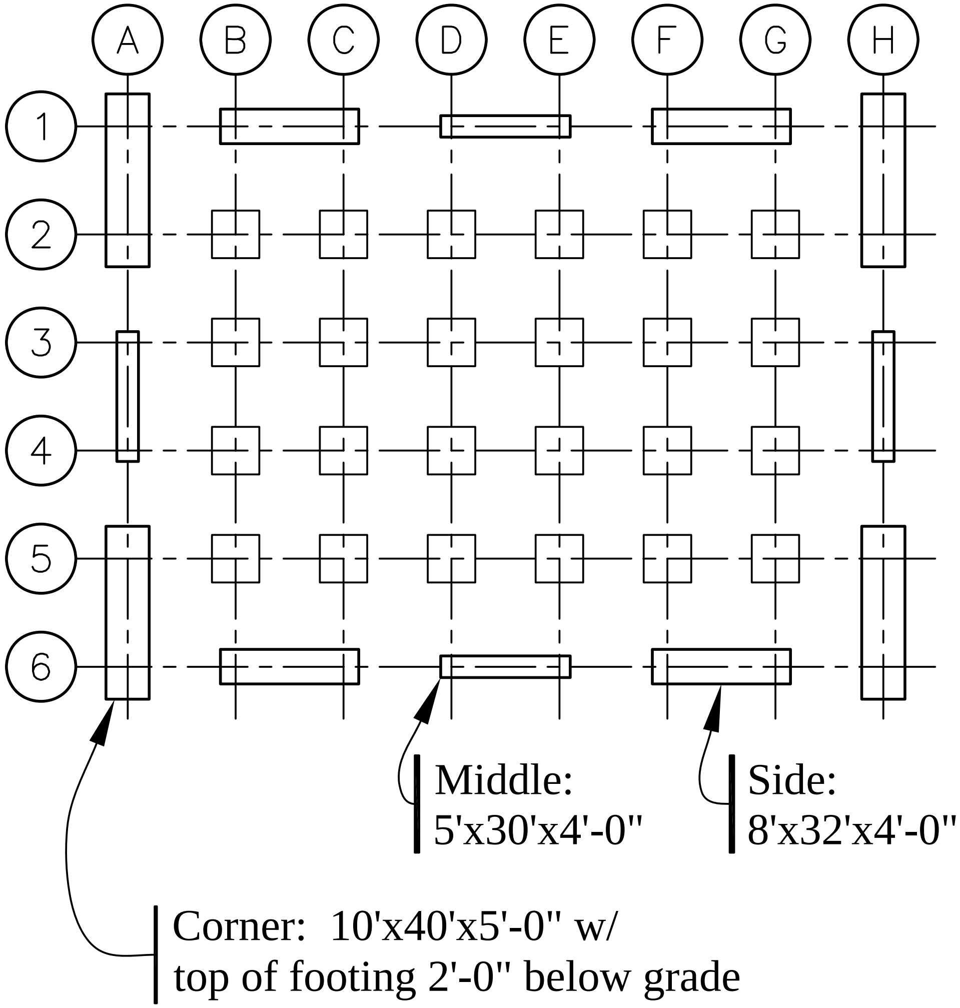
New trial: L = 40 ft, 5 ft thick

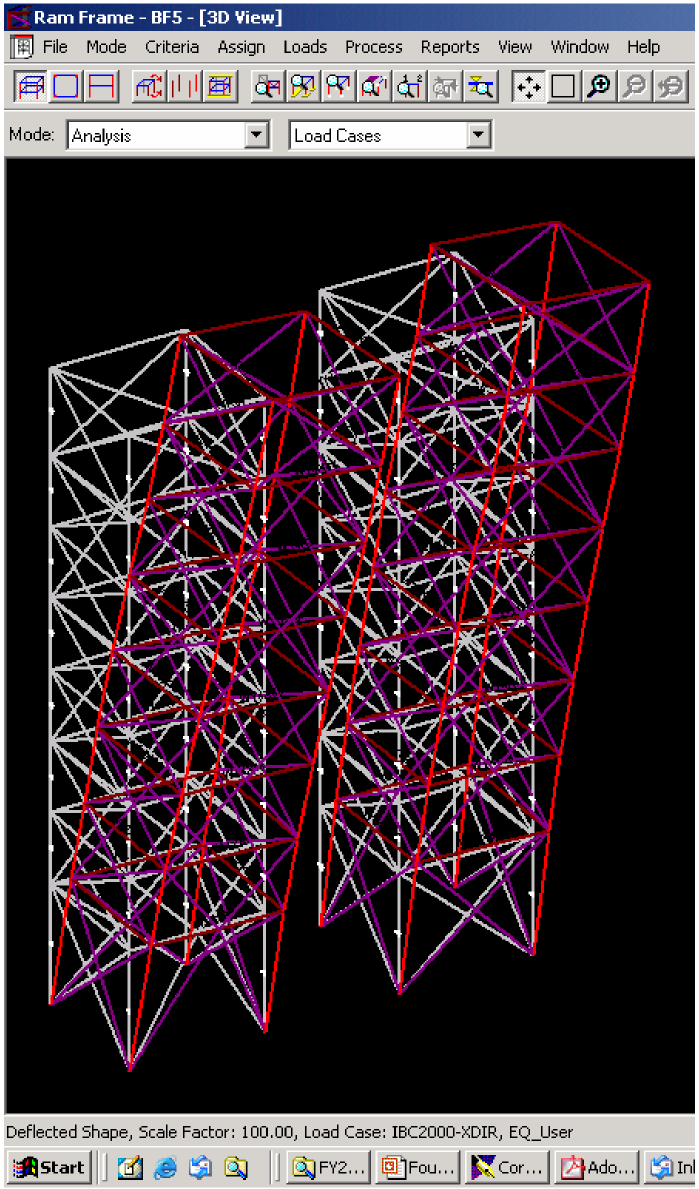
- W = 400 k; e = 18.0 ft; plastic Q_{max}= 8.6 ksf
- ϕQ_n = 0.6(3)4 = 7.2 ksf, close
- Solution is to add 5 k, then e = 17.8 ft and $Q_{max} = \phi Q_n$ = 7.9 ksf

Additional Checks

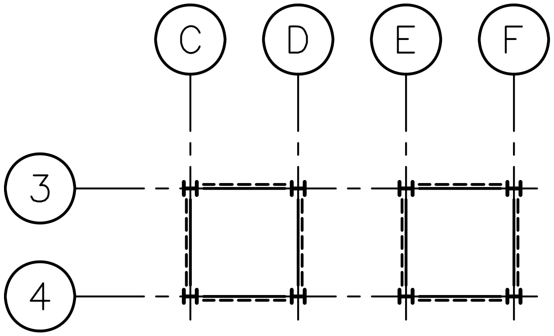
- Moments and shears for reinforcement should be checked for the overturning case.
- Plastic soil stress gives upper bound on moments and shears in concrete.
- Horizontal equilibrium: $H_{max} < \phi\mu(P+W)$
  in this case friction exceeds demand; passive could also be used.

Results for all SRS Footings



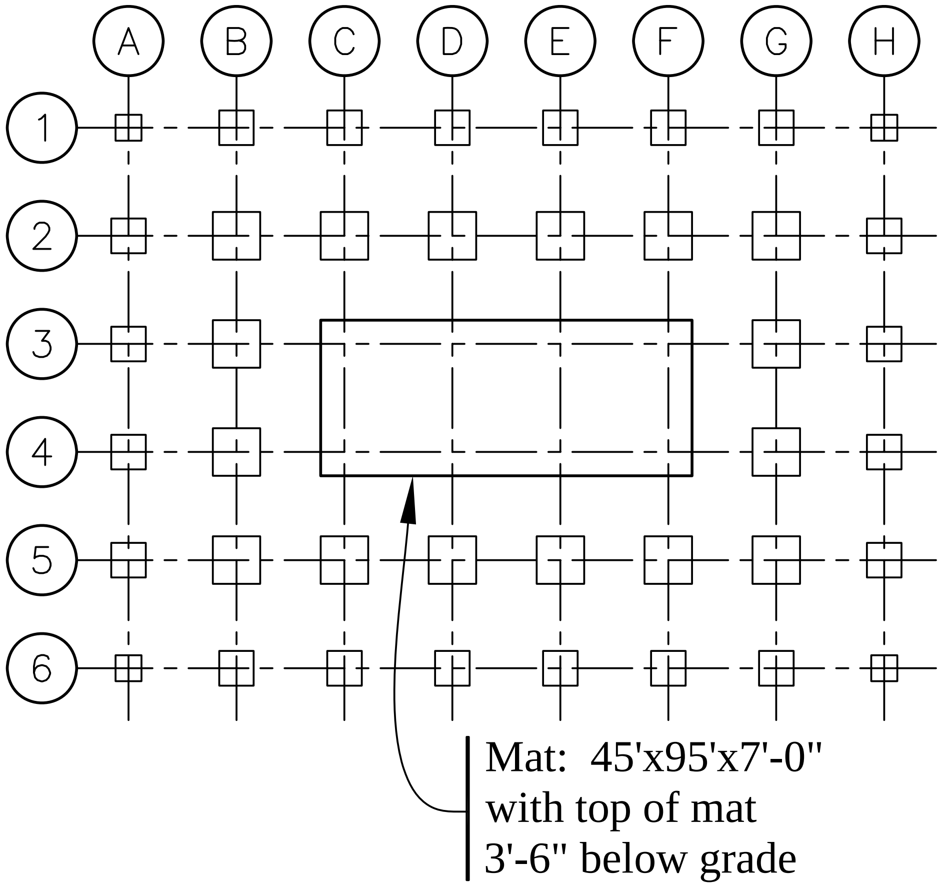


Design of Footings for Core-braced 7-story Building



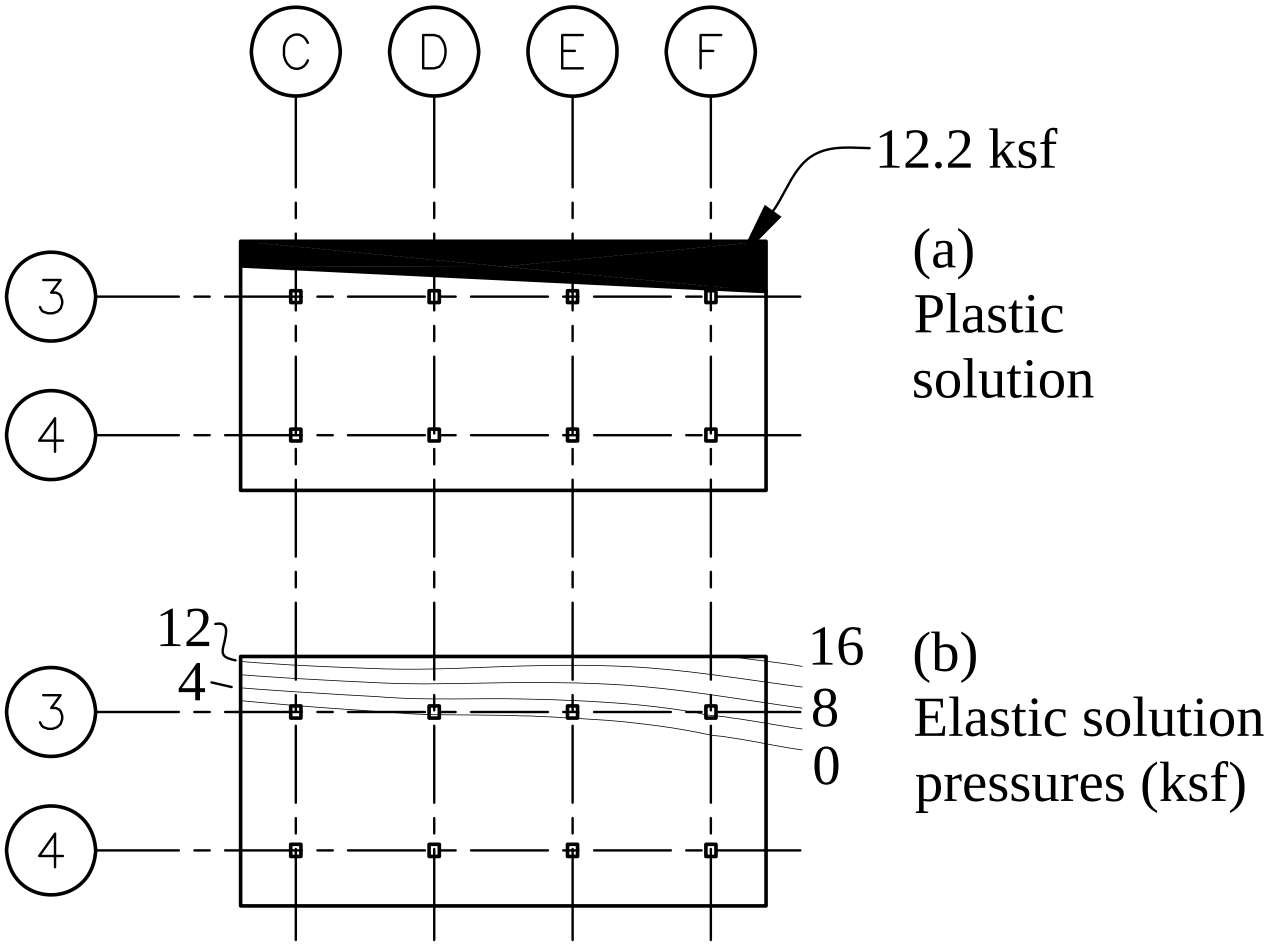
25 foot square bays at center of building

Solution for Central Mat



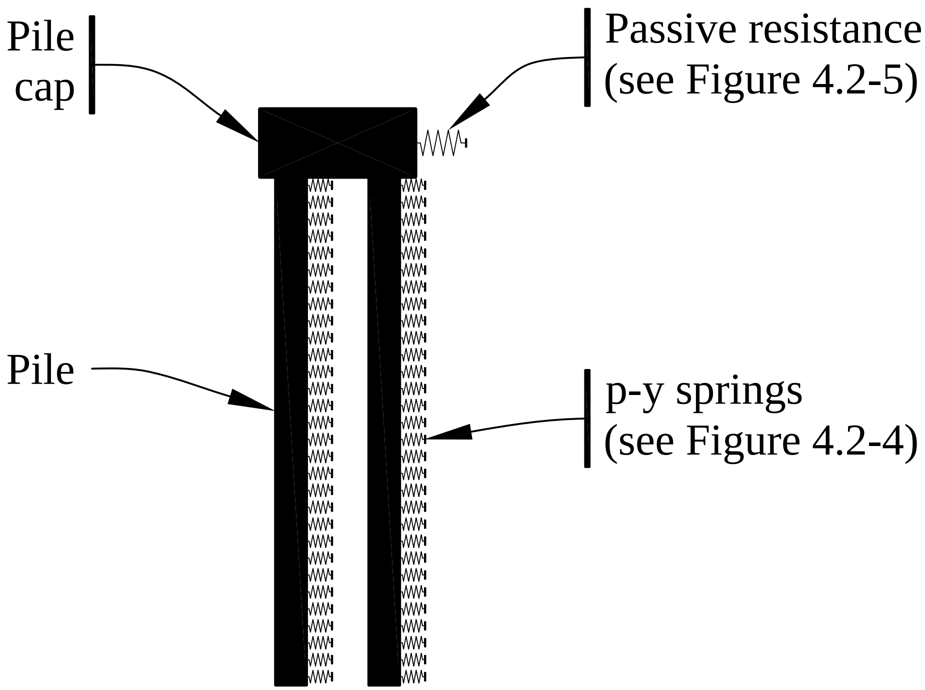
Very high uplifts at individual columns; mat is only practical shallow foundation.

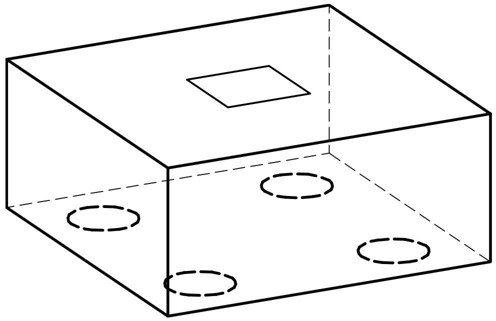
Bearing Pressure Solution



Plastic solution is satisfactory; elastic is not; see linked file for more detail.

Pile/Pier Foundations





View of cap with column above and piles below.

Pile/Pier Foundations

Pile Stiffness:

- Short (rigid)
- Intermediate
- Long

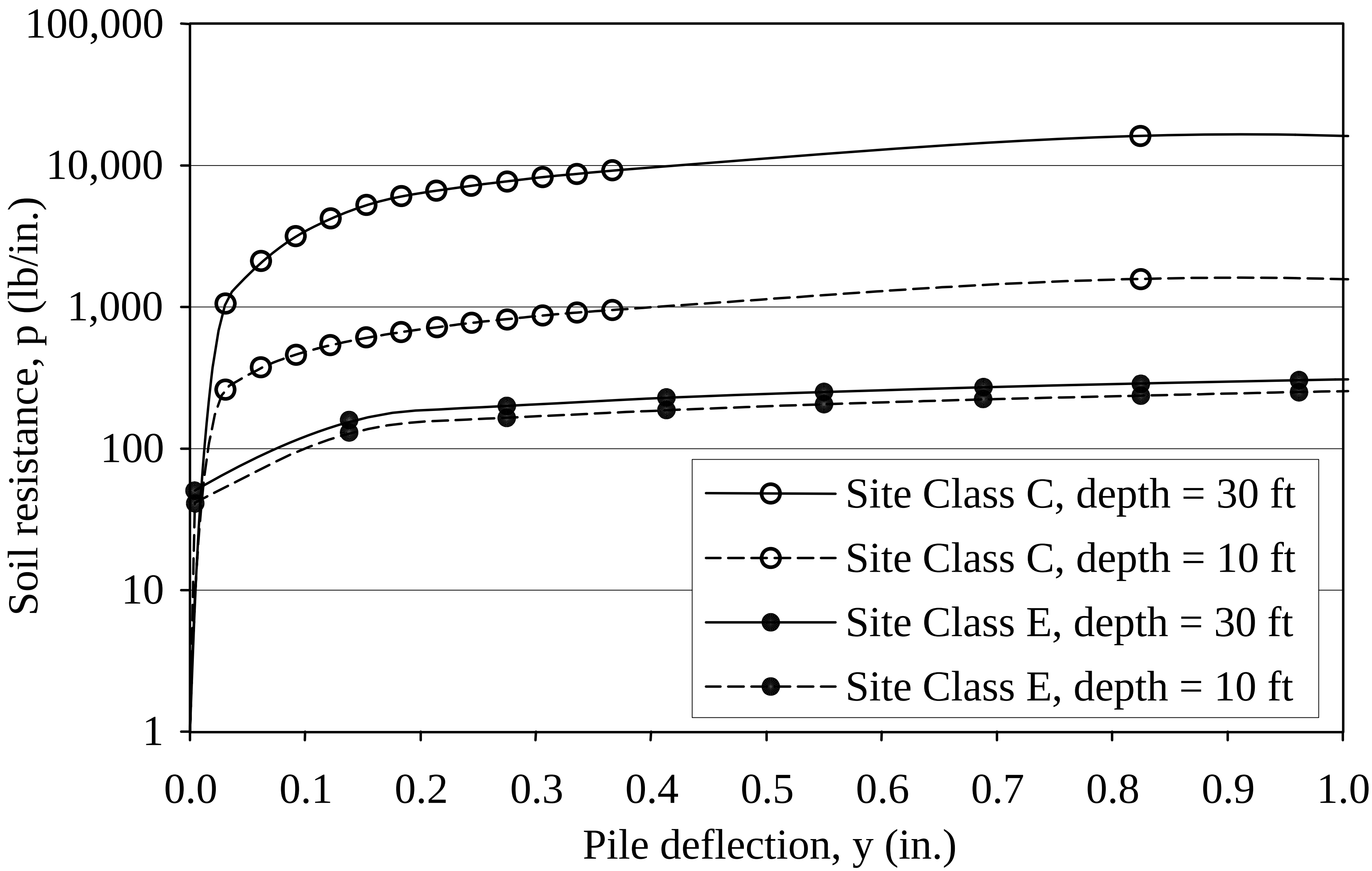
Cap influence

Group action

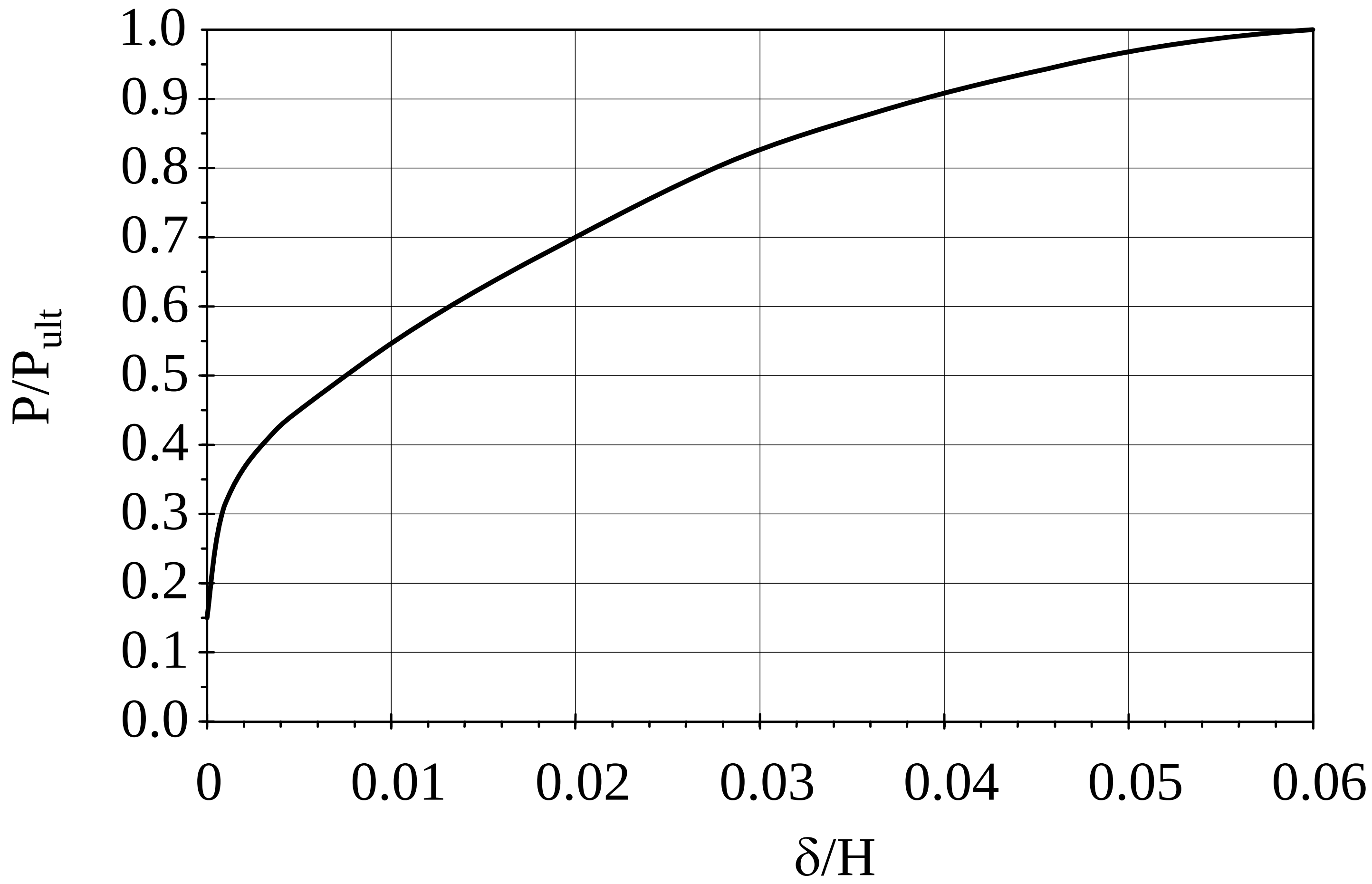
Soil Stiffness

- Linear springs – nomographs e.g. NAVFAC DM7.2
- Nonlinear springs – LPILE or similar analysis

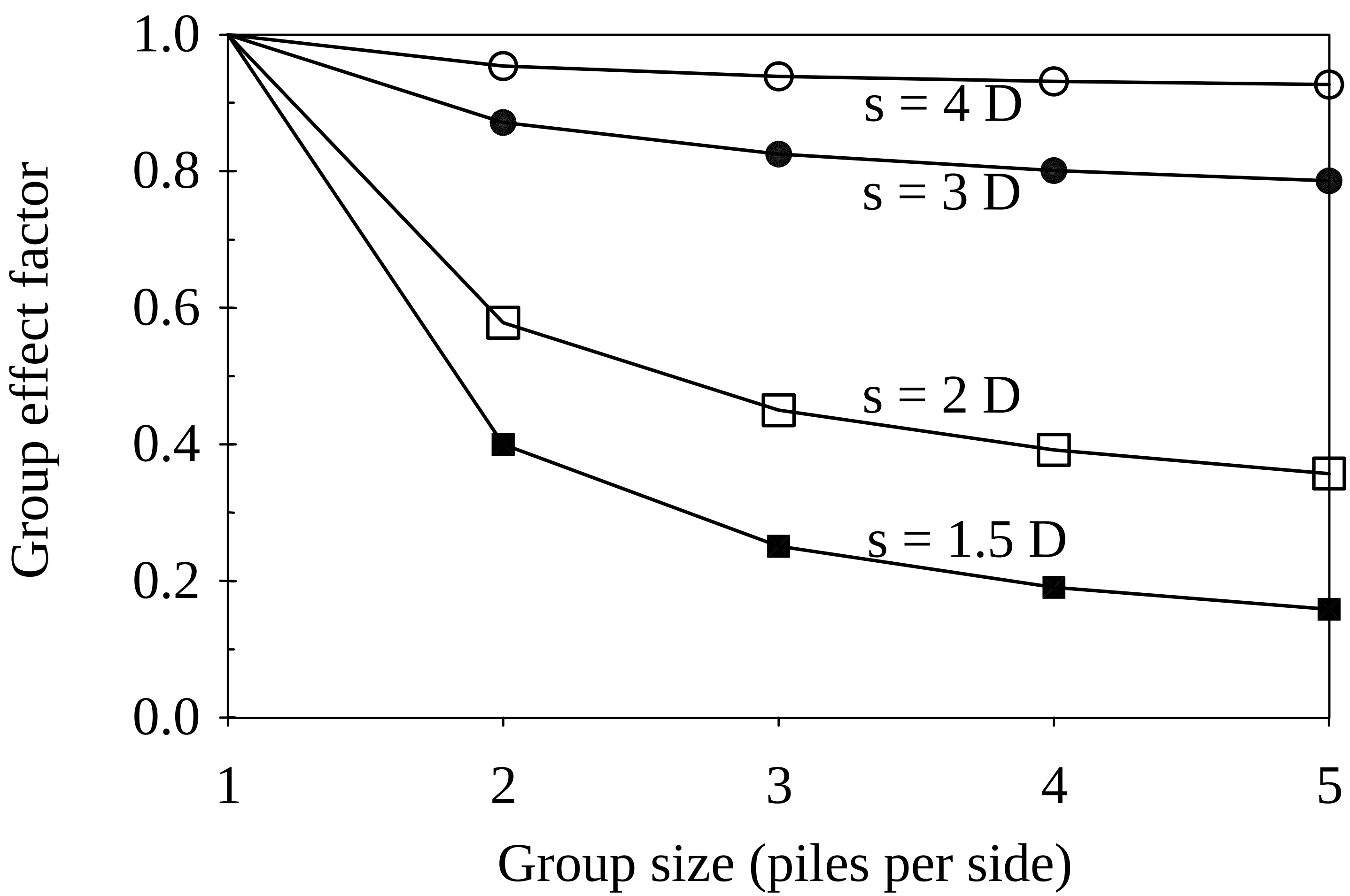
Sample *p-y* Curves



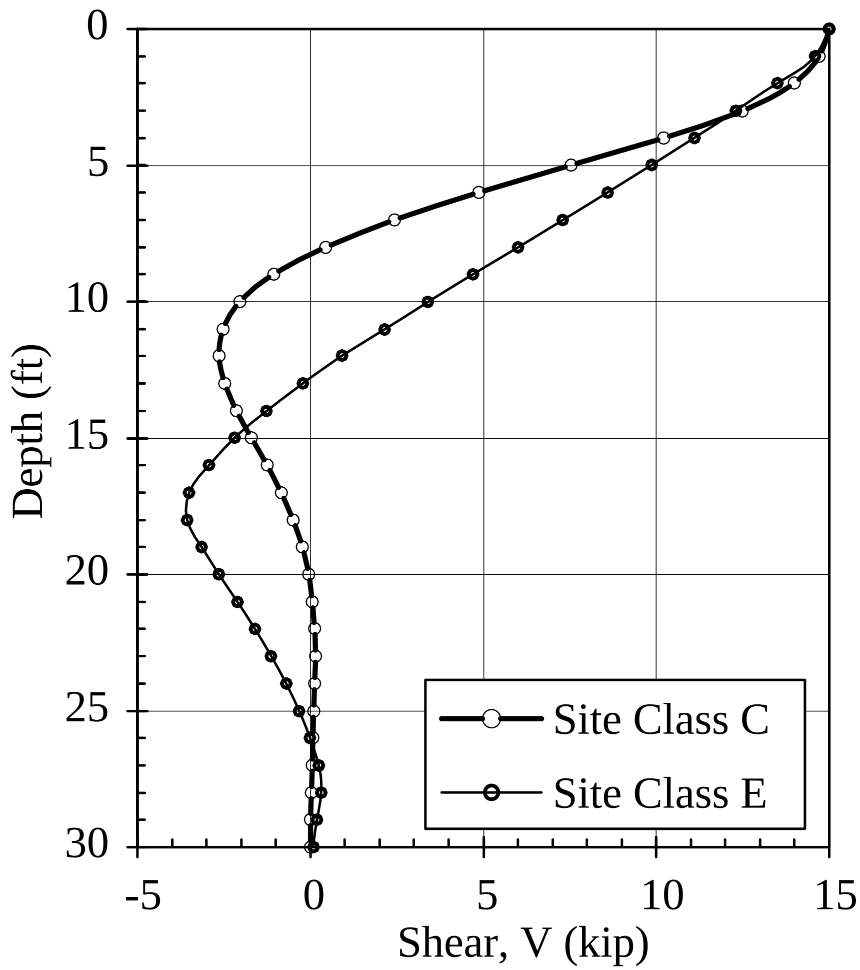
Passive Pressure



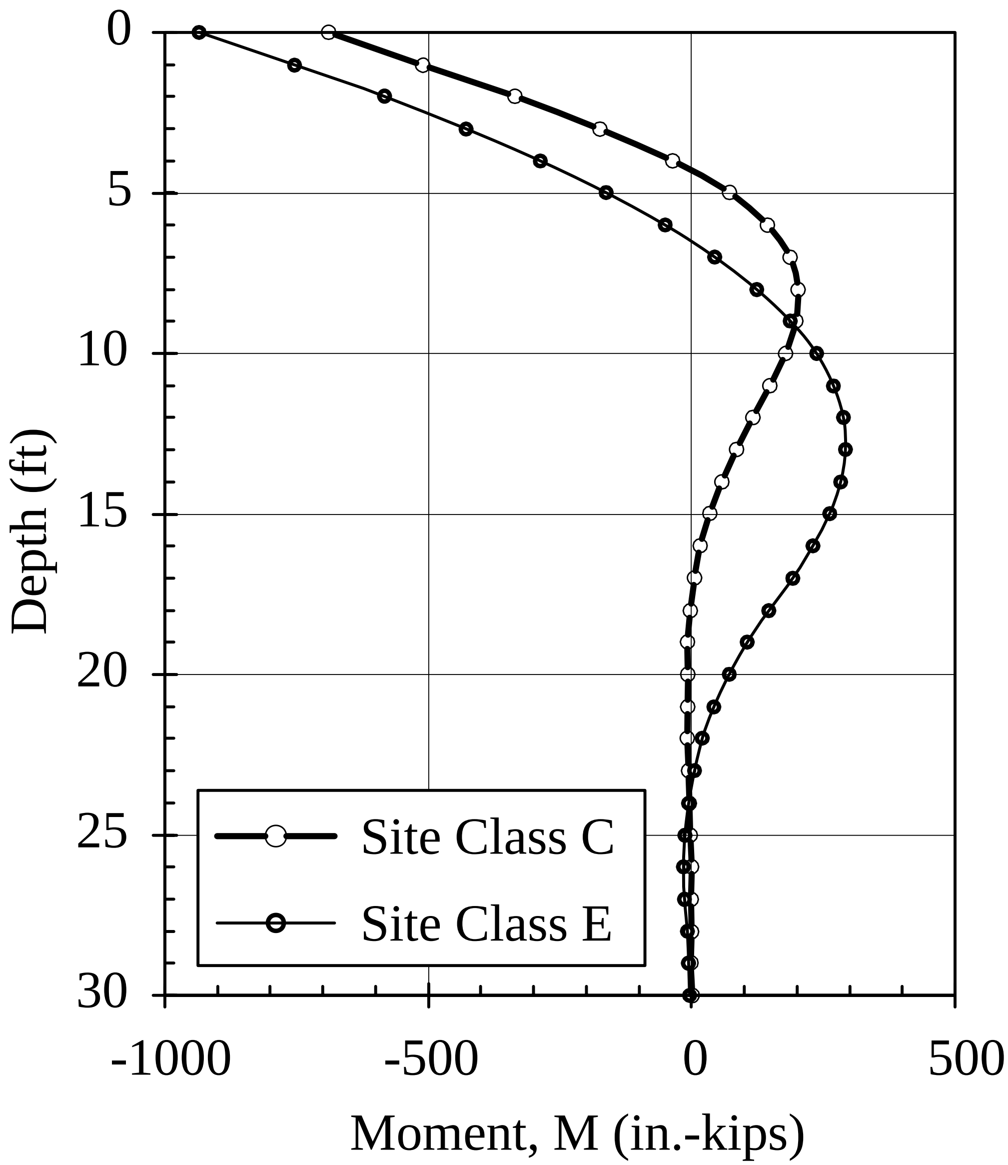
Group Effect

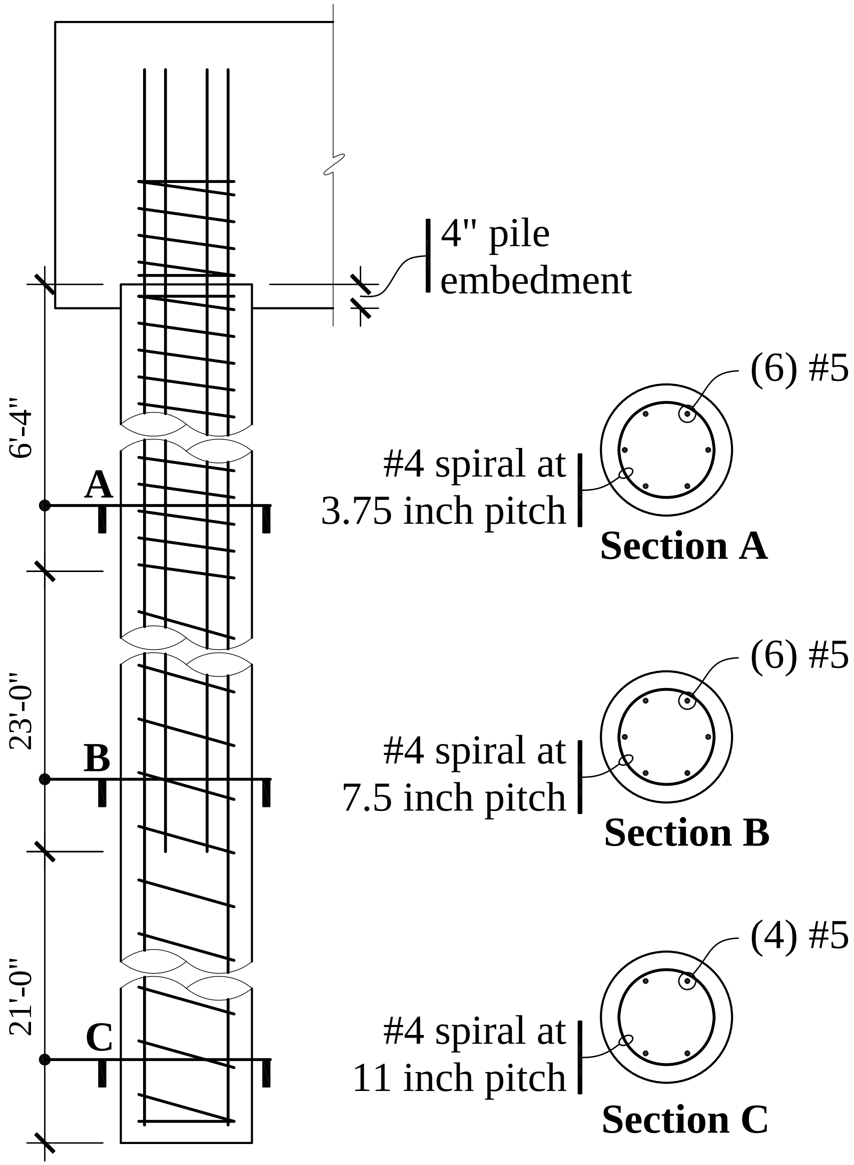


Pile Shear: Two Soil
Stiffnesses



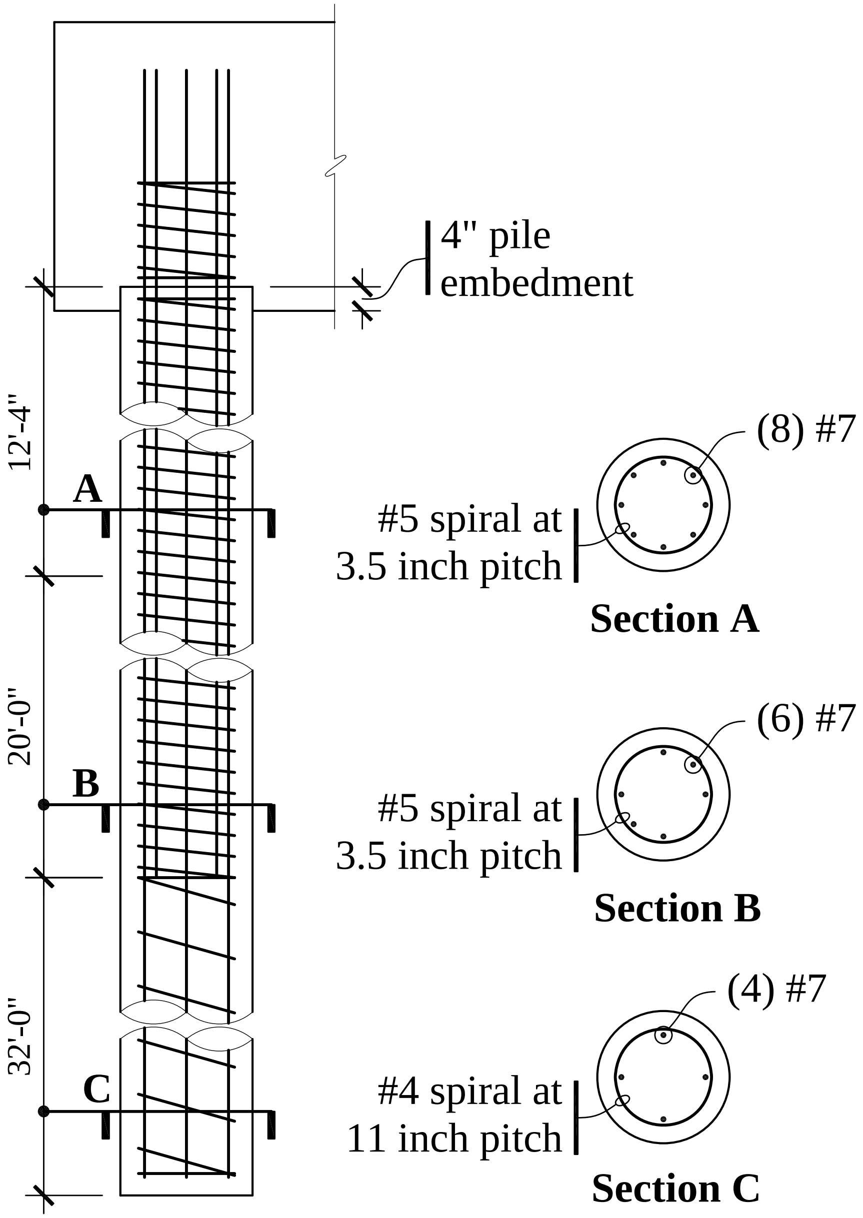
Pile Moment vs Depth





Pile Reinforcement

- Site Class C
- Larger amounts where moments and shears are high
- Minimum amounts must extend beyond theoretical cutoff points
- “Half” spiral for 3D



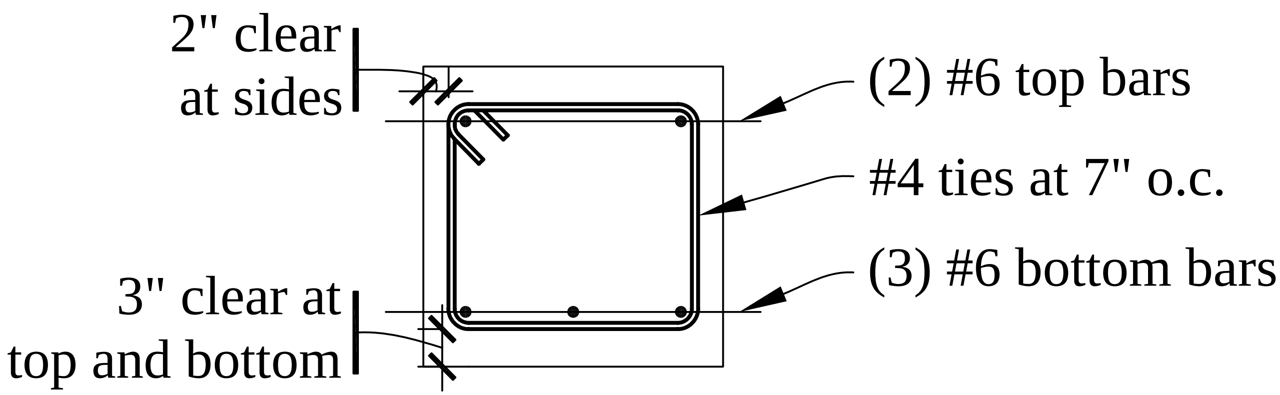
Pile Design

- Site Class E
- Substantially more reinforcement
- "Full" spiral for 7D
- Confinement at boundary of soft and firm soils (7D up and 3D down)

Other Topics for Pile Foundations

- Foundation Ties: $F = P_G(S_{DS}/10)$
- Pile Caps: high shears, rules of thumb; look for 3D strut and tie methods in future
- Liquefaction: another topic
- Kinematic interaction of soil layers

Tie Between Pile Caps



- Designed for axial force (+/-)
- Pile cap axial load times S_{DS}/10
- Often times use grade beams or thickened slabs one grade

TOPICS IN PERFORMANCE-BASED EARTHQUAKE ENGINEERING

Topics Covered

- Principles of performance-based earthquake engineering
- Seismic hazard and seismic risk snalysis
- Geotechnical earthquake engineering
- Methods of analysis
  - Pushover-based methods
  - Nonlinear response history methods
- Passive energy systems
  - Displacement dependent
  - Velocity dependent
- Seismic isolation
- Nonbuilding structures

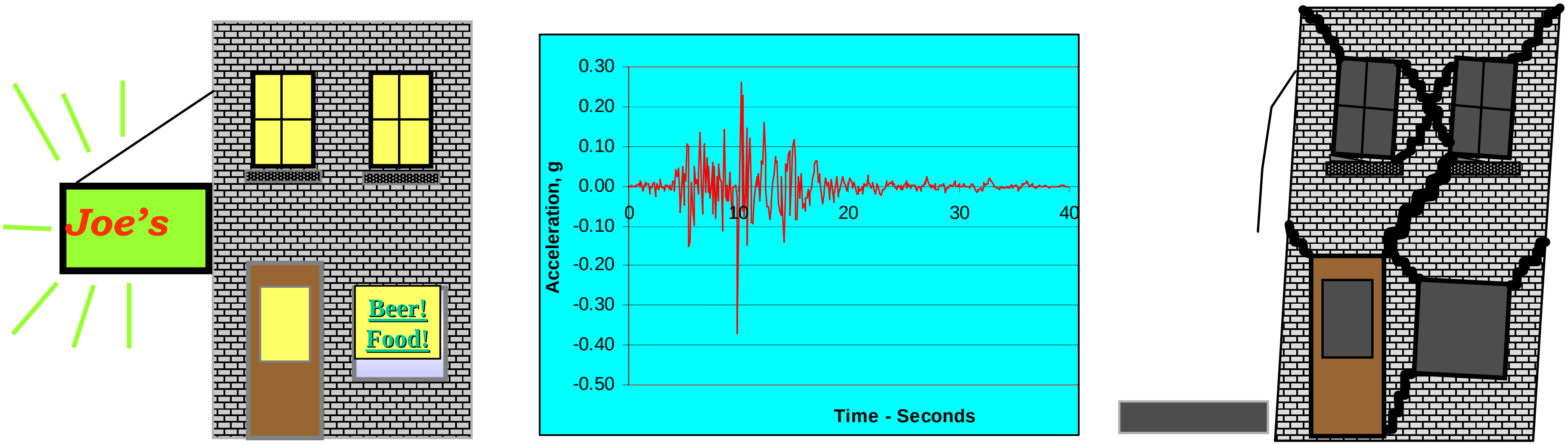
Structural engineering is
the Art of using materials
that have properties which can only be estimated

to build real structures
that can only be approximately analyzed

to withstand forces
that are not accurately known

so that our responsibility to the public safety is satisfied

PERFORMANCE-BASED ENGINEERING



Performance Approach

- The fundamental reason for the creation of a structure is placed at the forefront.
- Innovation is permitted, even encouraged.
- Characterization, measurement, and prediction of performance are fundamental concepts.

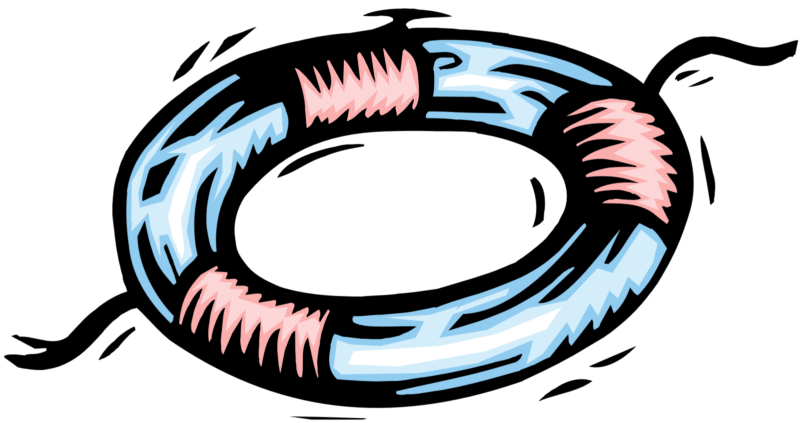
Performance-Based Structural Engineering

- Historical review
- Motivation
- Communications
- *ICC Performance Code*

- Modern trends in earthquake engineering
  - Performance levels
  - Global v local evaluation
  - Primary and secondary
  - Uncertainty

Performance Requirement

- A qualitative statement of a human need, usually in the form of an attribute that some physical entity, process, or person should possess.



Early Performance Requirement

- From the Code of Hammurabi (circa 1700 BCE):

  "If a builder has built a house for a man and his work is not strong and if the house he has built falls in and kills the householder, the builder shall be slain . . ."

Two Opposite Poles

- Performance:

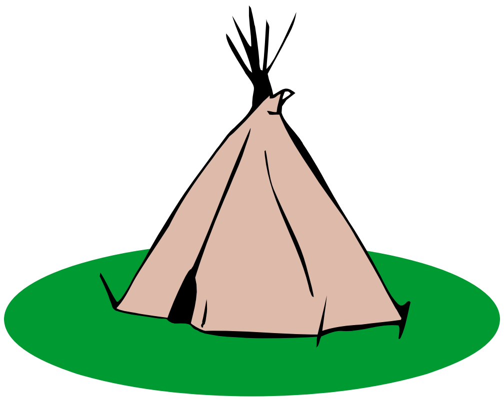
  An acceptable level of protection against structural failure under extreme load shall be provided.

- Prescriptive:

  ½” diameter bolts spaced no more than 6 feet on center shall anchor the wood sill of an exterior wall to the foundation.

Why Prescriptive?

- Simple to design and check.
- Simple can be economical.
- No need to "re-invent the wheel" on every new project.



What Is Wrong with Prescriptive?

- Loss of rationale leads to loss of ability to change.
- Loss of innovation leads to loss of economy.
- Loss of rationale can lead to loss of compliance.

What's Wrong with Performance Standards?

- Quantitative criteria:
  - Sometimes difficult to develop
  - Often difficult to achieve consensus
- Evaluation procedures:
  - Measurement is the key – it is essential to find a way to measure (analytically or experimentally) a meaningful quantity

Early Performance Standards at NBS (now NIST)

- 1969: Performance concept and its application
- 1970: Criteria for Operation Breakthrough
- 1971: PBS performance criteria for office buildings
- 1975: Interim performance criteria for solar
- 1977: Performance criteria resource document for innovative housing

NBS Format

R • A set of performance requirements

C • A set of quantitative performance criteria for each performance requirement

E
C • One evaluation procedure for each performance criterion

• A commentary if appropriate

Performance Requirements Circa 1976

1. The structural system shall support all loads expected during its service life without failure.
2. The structure shall support the service loads…without impairing function…or appearance…or causing discomfort.
3. Floor and wall surfaces shall resist service loadings without damage.

Criteria for Requirement 1 (Safety)

1.1 Resistance to ultimate load

Eight items to evaluate

Based on probabilistic reliability

1.2 Resistance to progressive collapse

No real evaluation; mostly commentary

1.3 Resistance to repeated loads

Evaluation focused on physical testing

Evaluations for Resistance to Maximum Load

- Load combinations for additive and counteracting loads
- Computations of load effects
- Foundation settlements
- Factored resistance, mean and variation in resistance
- Ductility

Maximum Loads

$$U = 1.1\ D + 1.45[Q + \Sigma\Psi_i F_i]$$

where:

D = dead load

Q mean maximum variable load (= 1.25L, 1.2S, 1.0H, 0.85W, 1.4E, or 1.0T)

Ψ_i = factor for arbitrary point in time load

F_i = L, S, H, W, E, or T

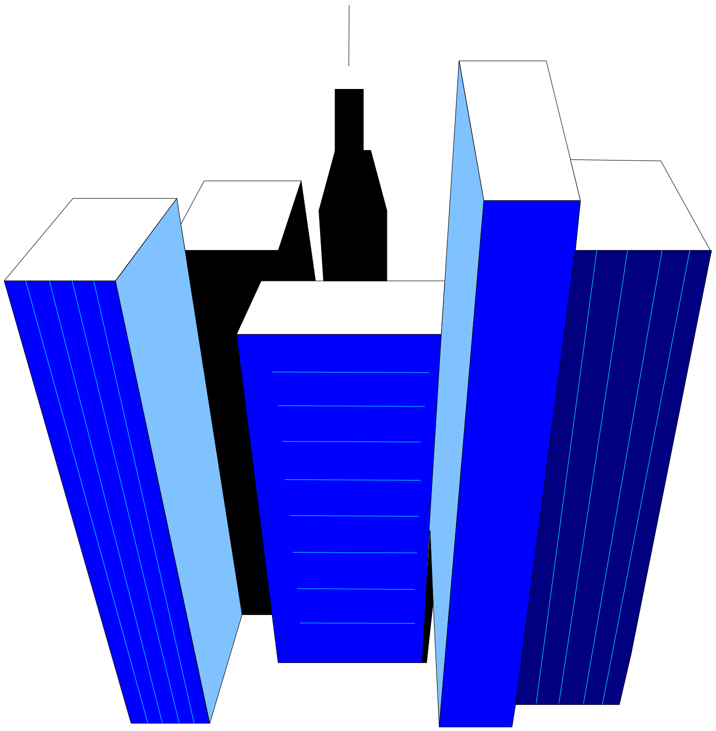
“Partial vs. Pure Performance”

- Specification of the load factors creates a “procedural standard” whereas specification of a reliability level would be more purely “performance”
- Analytical evaluation
- Experimental evaluation ($$$)

Performance-based Design

- Design specifically intended to limit the consequences of one or more perils to defined acceptable levels
- Perils addressed: wind, fire, snow, earthquake, live loads

All Design Is Intended to Achieve Performance . . .



- Protect the public safety by minimizing the chance for:
  - Uncontrolled or inescapable fire
  - Structural collapse
  - Spread of disease
- Limit occupant discomfit by controlling:
  - Noise
  - Vibration
  - Environment

… But Most Building Code Provisions Are Not Performance-based

- Codes typically prescribe design and construction rules:
  - Believed capable of attaining desired performance
  - Largely based on past poor performance

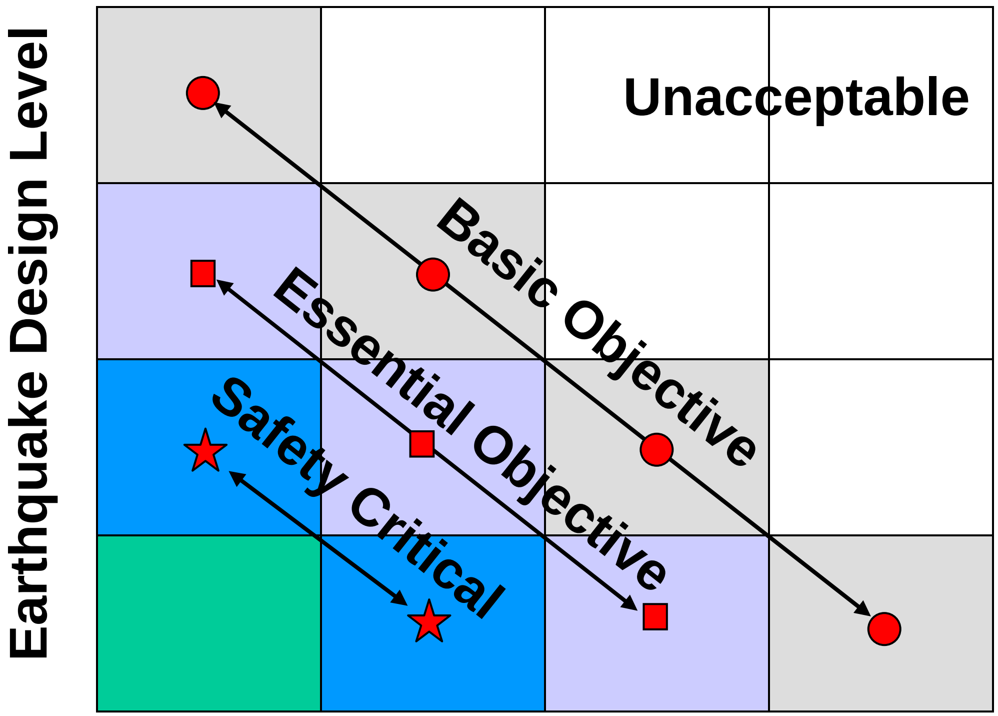
Designers Following These Codes . . .

- Learn to follow the rules, but often:
  - Don’t know why the rules require certain things.
  - Don’t understand the performance intended.
  - Don’t know how to adjust the rules to get different performance.

Performance-based Design

- Requires the designer to understand:
  - Intended performance
  - Relationship between design features and performance
- Forces the designer to predict expected performance given a design event

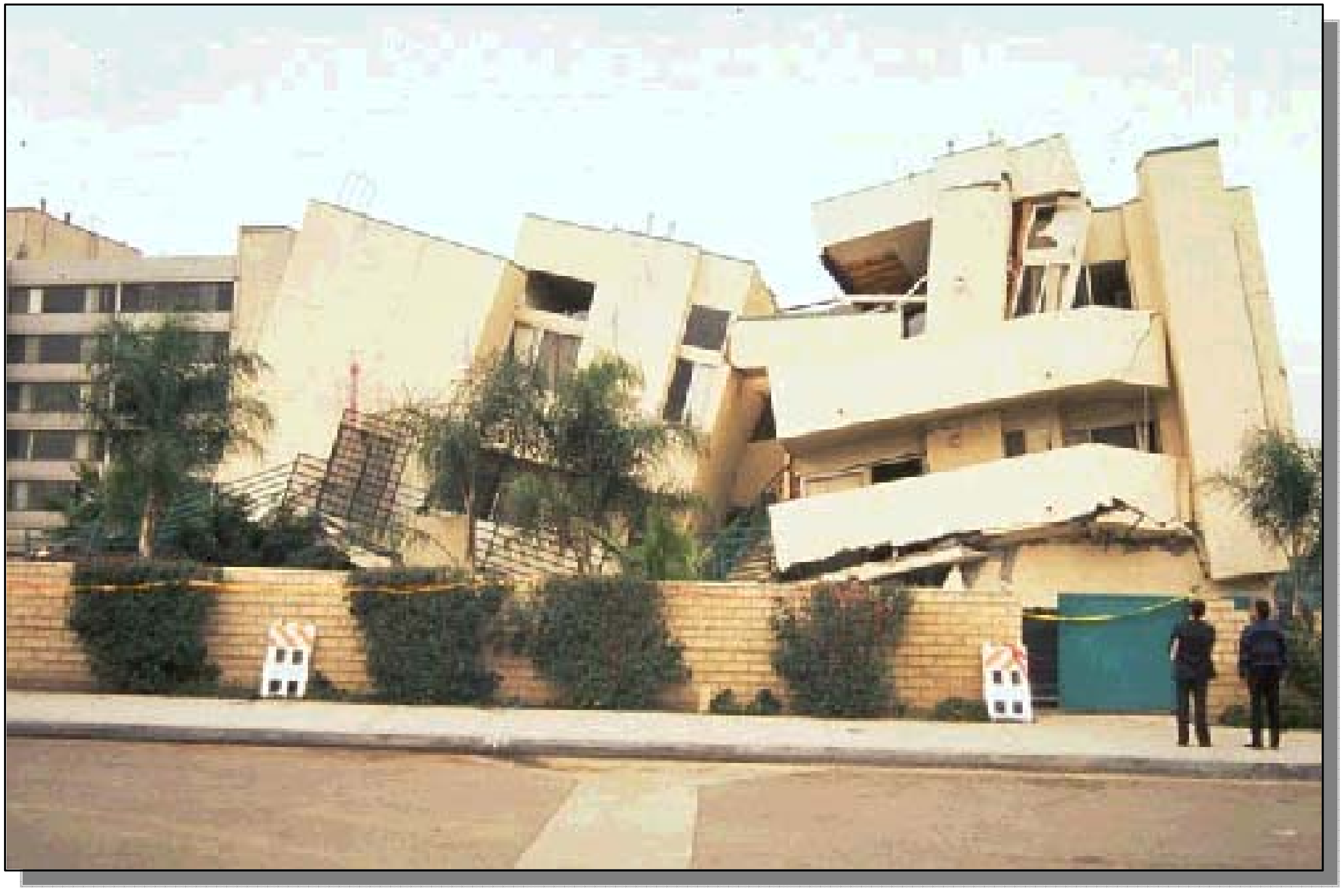
SEAOC's Vision 2000



Motivation for PBE (Structural)

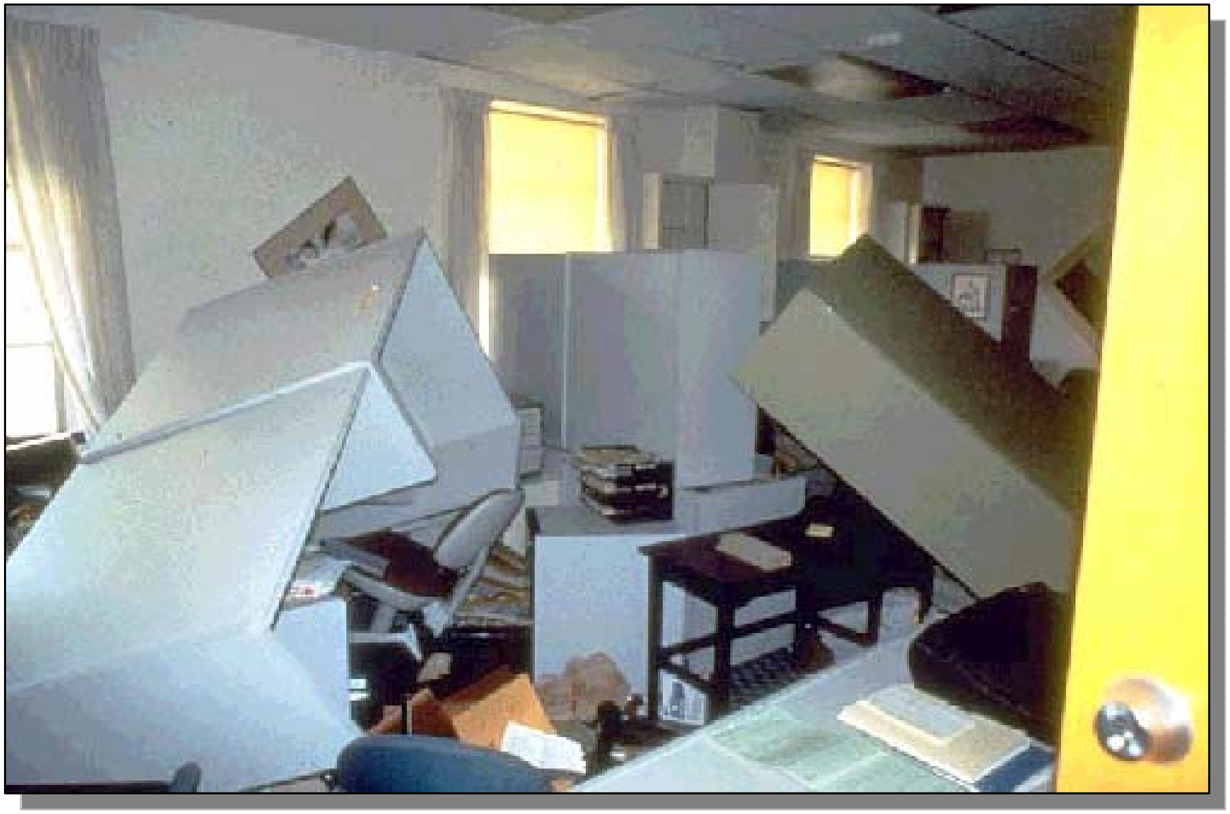
A modern garage at Cal State Northridge.

Motivation for PBE (Structural)



A modern wood-frame residential building on Sherman Way.

Motivation for PBE (Nonstructural)



Veterans Administration Medical Center in Sepulveda.

Motivation for PBE

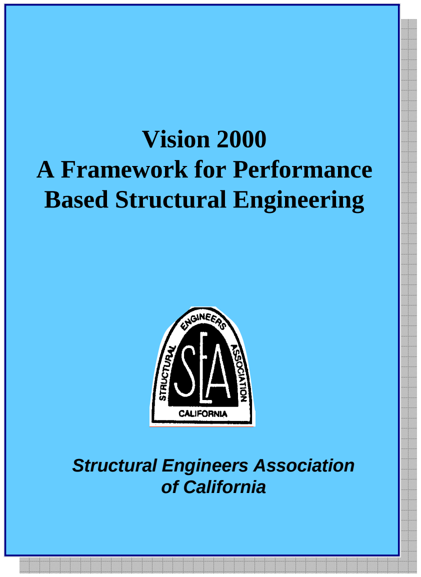
What is wrong with current building codes?

- Only a single performance level is checked.
- Only a single seismic event is applied.
- Linear static or dynamic analysis.
- No local acceptance criteria.

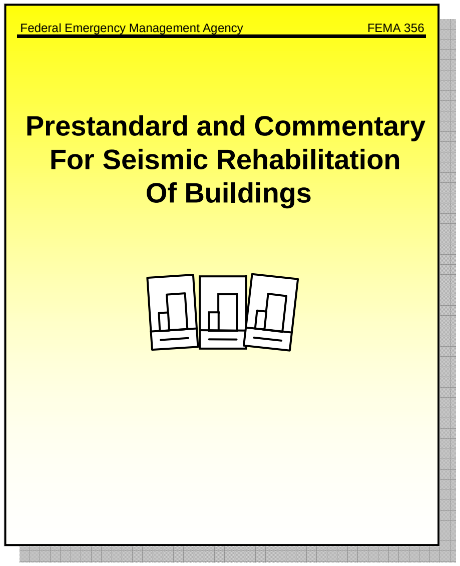
Concepts Incorporated within PBE

- Multiple performance levels are checked.
- Multiple seismic events are applied.
- May utilize nonlinear analysis.
- Detailed local acceptance criteria
  - For structural elements
  - For nonstructural elements

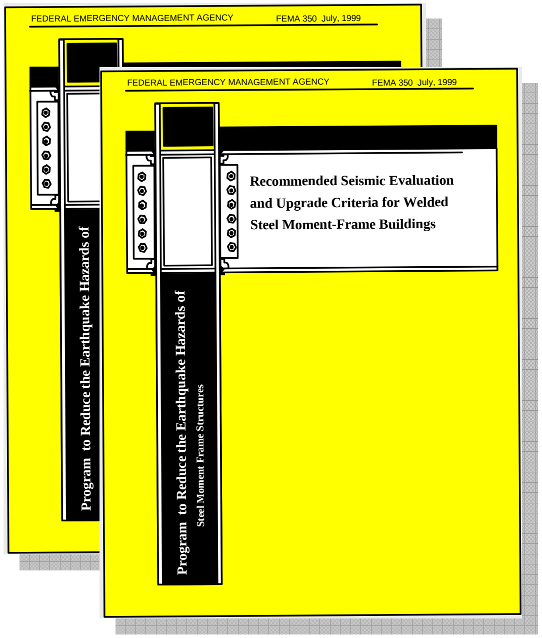
Basic Resource Documents Performance-based Seismic Design



Vision 2000
(new buildings)

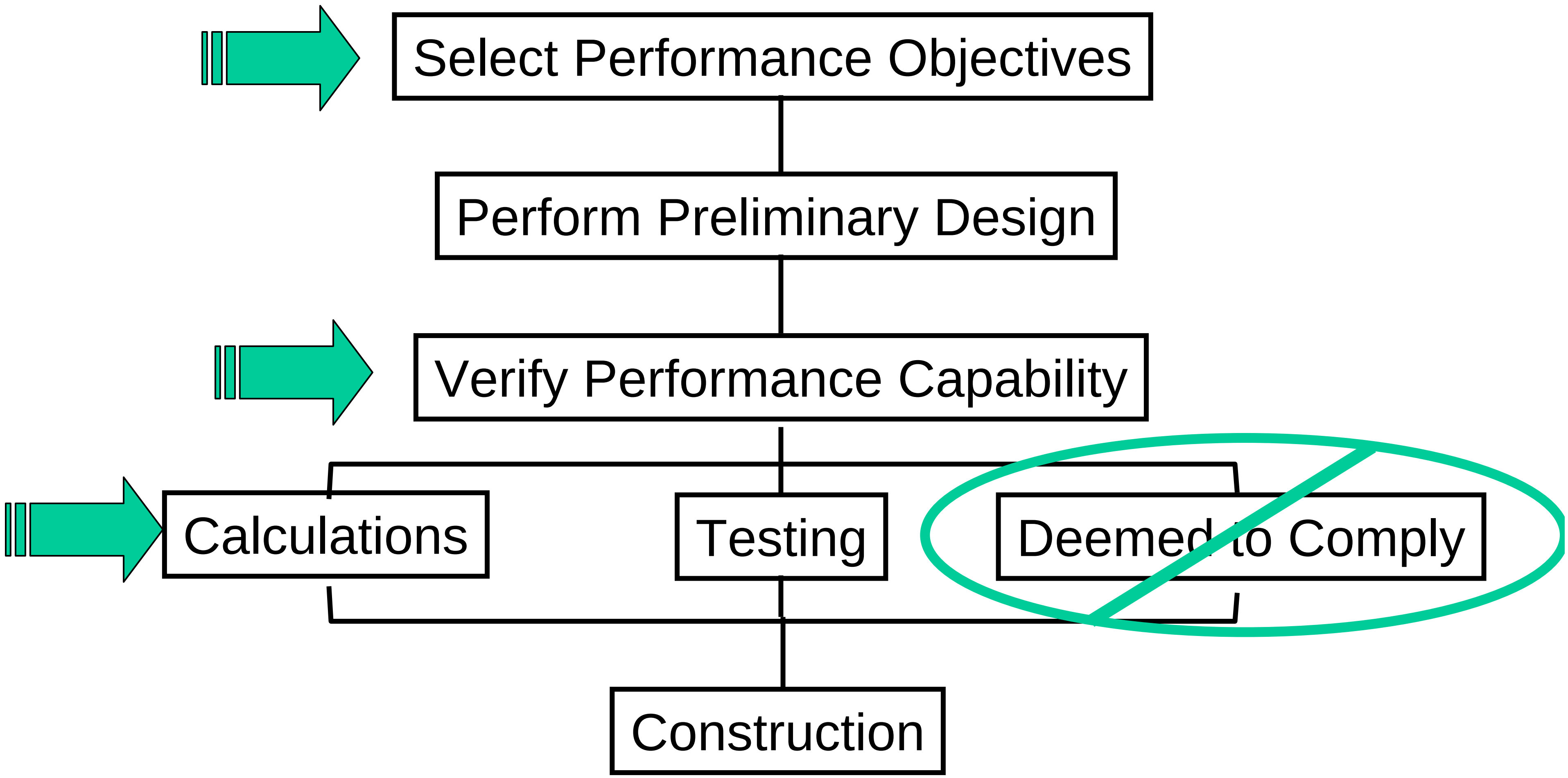


FEMA 356
(existing buildings)



FEMA 350/351
(steel moment frame buildings)

The PBD Process

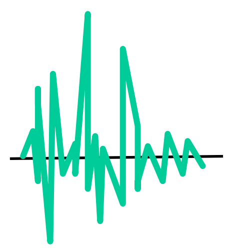


Vision 2000 / FEMA 356 Performance Objectives

Specification of:

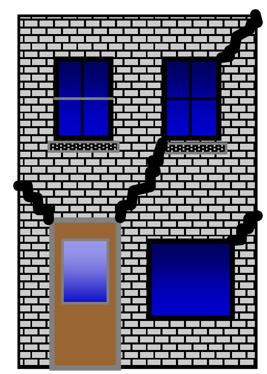
- ***Design Hazard (earthquake ground shaking)***
- ***Acceptable Performance Level (maximum acceptable damage given that shaking occurs)***

Performance Objective =



Ground Motion
x% - 50 years

\+



Performance Level

Performance Objectives

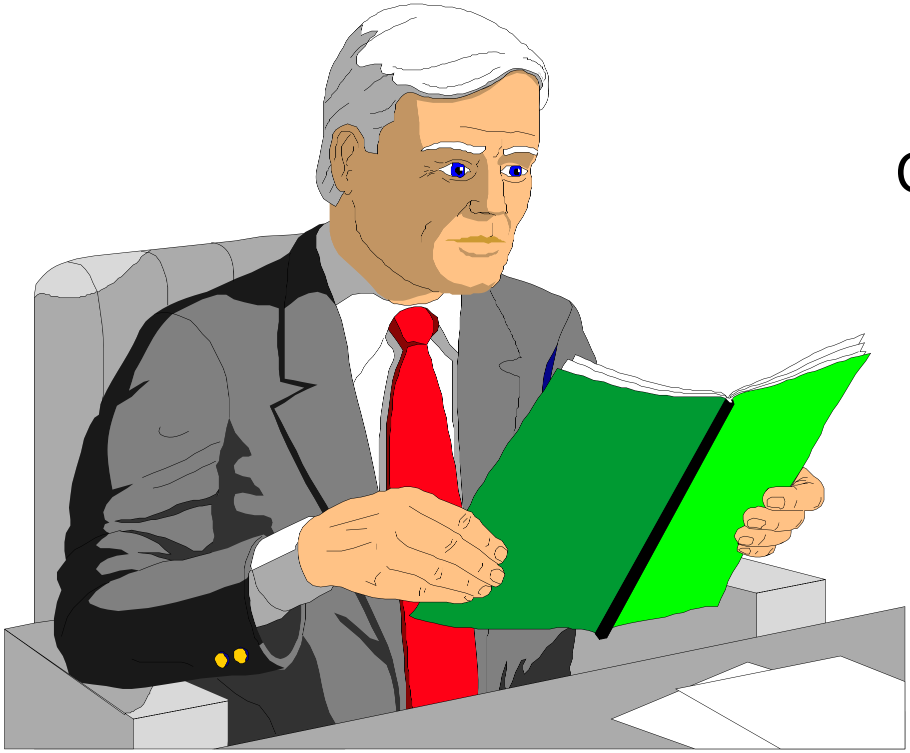
- For performance-based design to be successful, the needs of both the client and engineer must be satisfied.

Engineer --

Hazard must be quantifiable and performance must be quantifiable

Performance Objectives

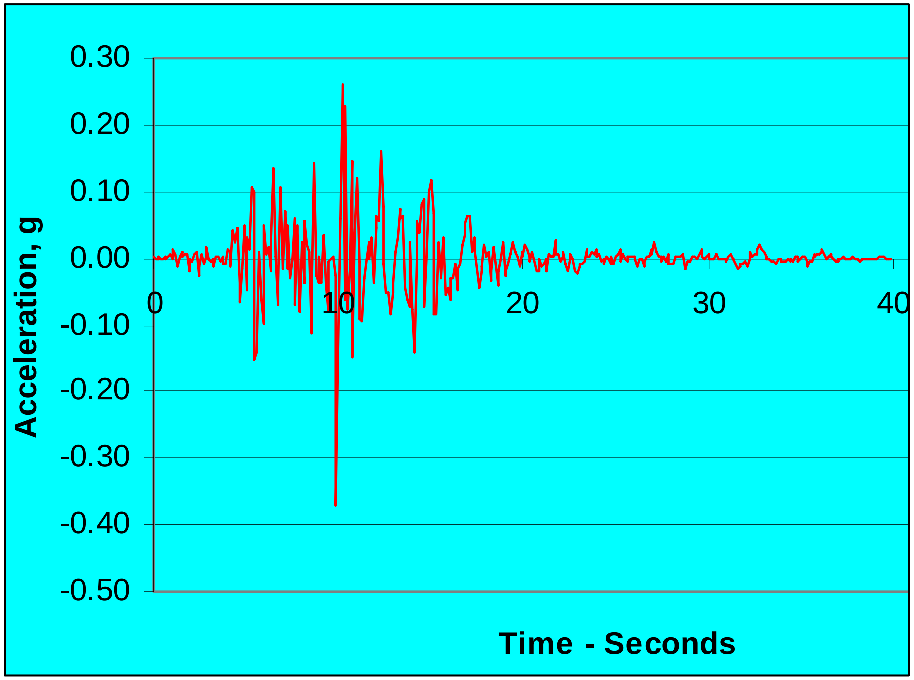
- For performance-based design to be successful, both the client and engineer must be satisfied



Owner --

Hazard must be understandable and performance must be understandable and useful

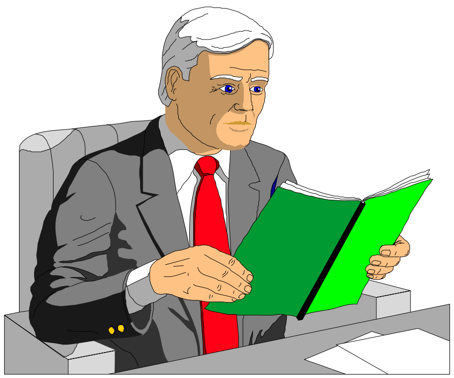
Hazard



The intensity and characteristics of ground shaking that design is developed to resist.

Hazard

- Two methods of expression:
  - Deterministic
    - Magnitude “x” earthquake on “y” fault

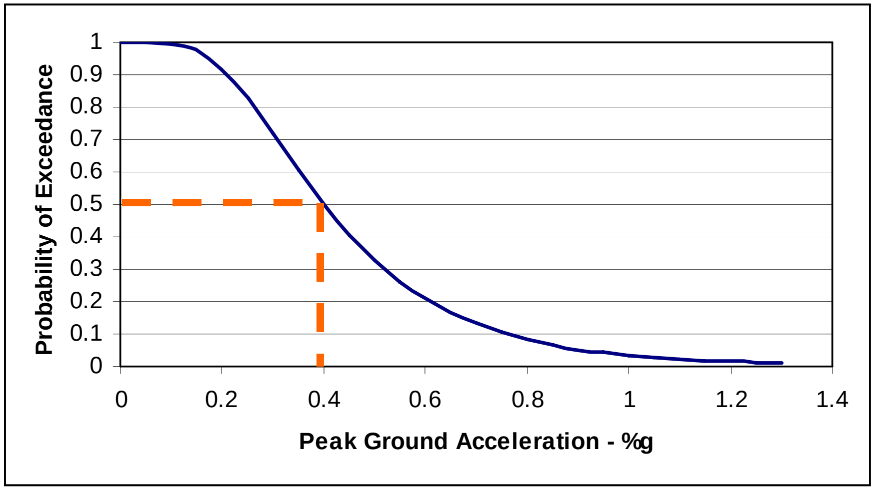


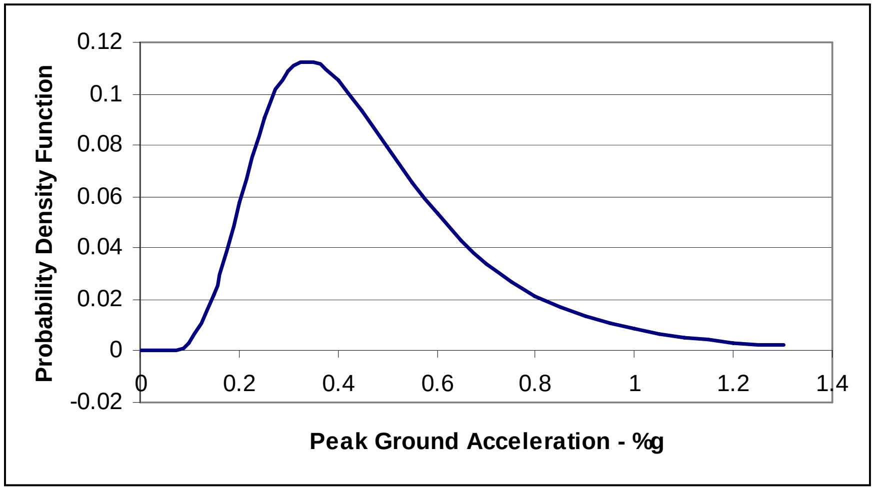
  - Probabilistic
    - “x” % probability of exceedance in “y” years for design event

Deterministic Hazards

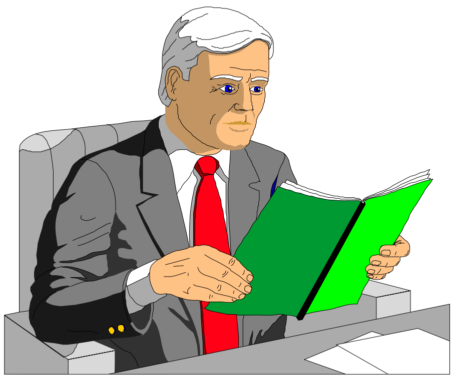
- Easy to understand but . . .

  there is considerable uncertainty as to how strong the motion from such an event actually is.





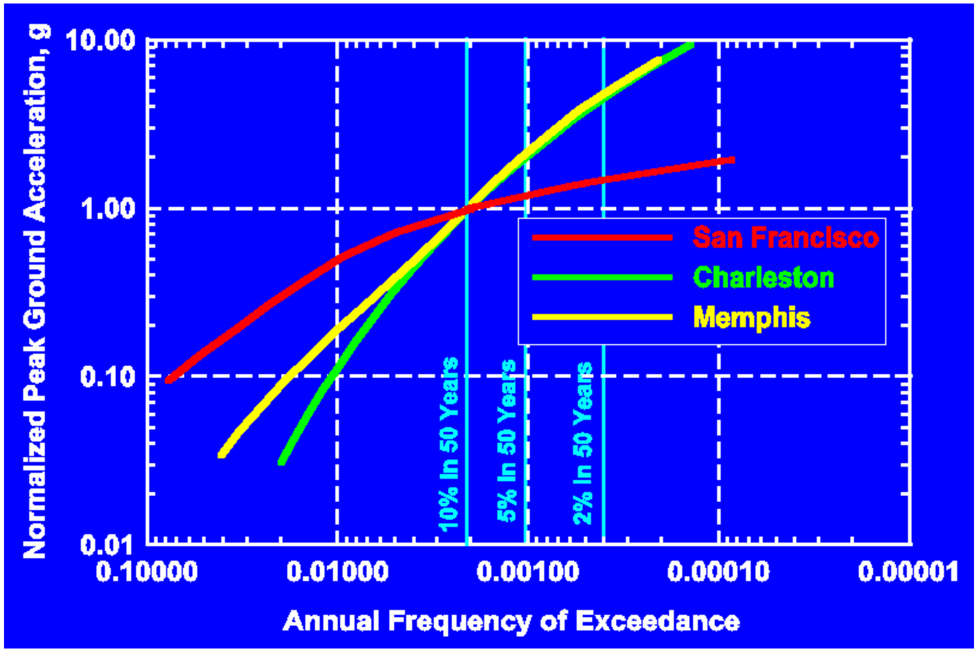
Probabilistic Hazards



- Need to move clients to "probabilistic" mind set.
- Commonly used for other considerations such as:
  - Probable occupancy rates,
  - Probable cost of construction, and
  - Probable return on investment.

Probabilistic Hazards

- Low intensity shaking occurs frequently.
- Moderate intensity shaking occurs occasionally.
- Severe shaking occurs rarely.



Probabilistic Hazards

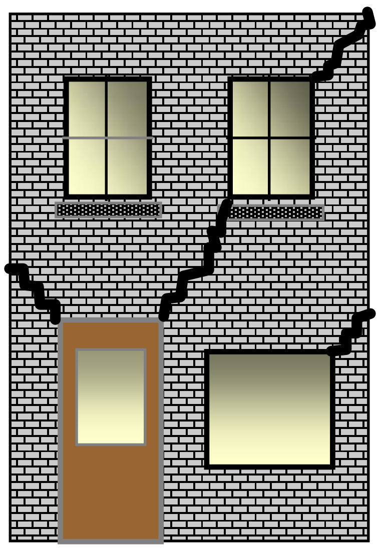
- Probability of exceedance for design event:
  - 10%/50 years
    (500 year mean return) traditionally taken as hazard for “life safety protection”
  - 2%/50 years
    (2,500 year mean return) traditionally taken as hazard for collapse avoidance
  - Hazard for economic loss protection can be taken at any level based on cost-benefit considerations.

Earthquake Hazard Levels (FEMA 273)

Probability	MRI	Frequency
50%-50 Year	72 Years	Frequent
20%-50 Year	225 Years	Occasional
10%-50 Year **(BSE-1)**	474 Years	Rare
2%-50 Year* **(BSE-2)**	2475 Years	Very Rare

*NEHRP Maximum Considered Earthquake.

Performance Level



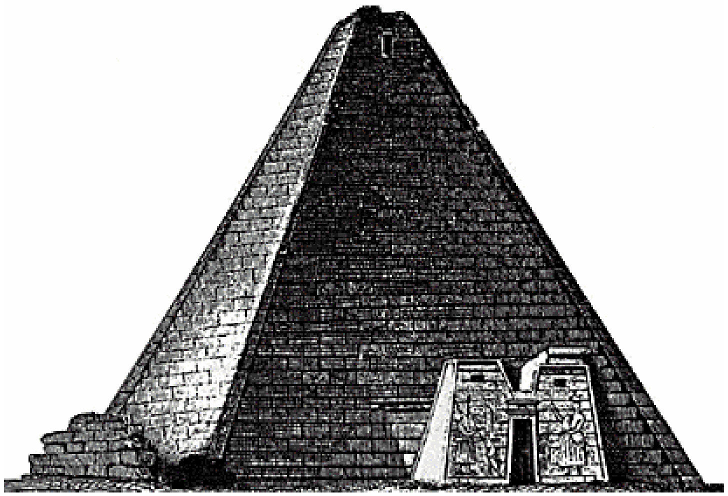
The permissible amount of damage, given that design hazards are experienced.

ICC Performance Code

- “Allows user to systematically achieve various solutions.”
- “Prescriptive code deemed to be acceptable.”
- “Procedure to address the alternate materials and methods clause of code.”
- Commentary highly recommended.

ICC Performance Code

- “Committee envisions limited code changes in the future, except that “acceptable methods” will be an evolving process.



ICC Performance Code

- “Purpose -- To provide appropriate health, safety, welfare, and social and economic value, while promoting innovative, flexible and responsive solutions.”
- “Intent -- A structure that will withstand loads associated with normal use and of the severity associated the location….”

ICC: Administrative Provisions

- Functional statements:
  - Design professional qualifications
  - Design documents required for review
  - Construction compliance to be verified
  - Maintenance of performance-based design over life of building

ICC Administrative Provisions

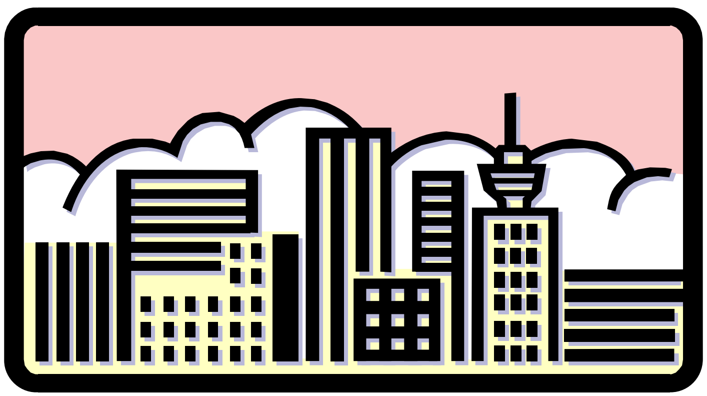
“Performance” requirements

- Building owner responsibilities
- Design professional qualifications
- Special expert responsibilities
- Documentation
  - Concept report and design reports
  - O & M manual

ICC Use Groups

Basis for assignment:

- Function
- Risks to users



Risk factors:

- Nature of hazard
- Number of people
- Length of time occupied
- Sleep facility
- Familiarity
- Vulnerable groups
- Relationships

ICC Performance Groups

Performance Group	Description
I	Low hazard to humans
II	Normal buildings
III	Hazardous contents
IV	Essential facilities

ICC Design Performance (Damage) Levels

"Size" of event	Perf. Group I	Perf. Group II	Perf. Group III	Perf. Group IV
V. Large (v.rare)	Severe	Severe	High	Mod
Large (rare)	Severe	High	Mod	Mild
Medium	High	Mod	Mild	Mild
Small (frequent)	Mod	Mild	Mild	Mild

Mild Damage Level

- No structural damage; safe to occupy
- Necessary nonstructural is operational
- Minimal number of minor injuries
- Minimal damage to contents

Moderate Damage Level

- Structural damage, but repairable; delay in reoccupancy
- Necessary nonstructural operational
- Locally significant injuries but low likelihood of death
- Moderate cost of damage
- Minimal risk from hazardous materials

High Damage Level

- Significant structural damage, but no large falling debris; repair possible but long-term
- Necessary nonstructural damaged significantly
- Injury and death possible
  but moderate numbers
- Hazardous materials release
  locally

Severe Damage Level

- Substantial structural damage, but collapse is avoided; repair may be infeasible
- Necessary nonstructural not functional
- Likely single life loss; moderate probability of multiple lives lost
- Damage may “total” the building
- Hazardous materials release requires relocation

MRI for Environmental Loads

Event Size	Flood	Wind	Snow	Ice	Earth-quake
Small	20 **100**	50	25	25	25
Medium	50 **500**	75	30	50	72
Large	100 **SS**	100	50	100	475
V. large	500 **SS**	125	100	200	2475

ICC Performance Code Appendices

A. Use classification related to main code
B. Worksheet for assignment to performance groups
C. Individually substantiated design method
D. Qualification characteristics
E. Use of computer models

Performance-Based Structural Engineering

- Historical review
- Motivation
- Communications
- *ICC Performance Code*

- Modern trends in earthquake engineering
  - Performance levels
  - Global v local evaluation
  - Primary and secondary
  - Uncertainty

Performance-Based Earthquake Engineering

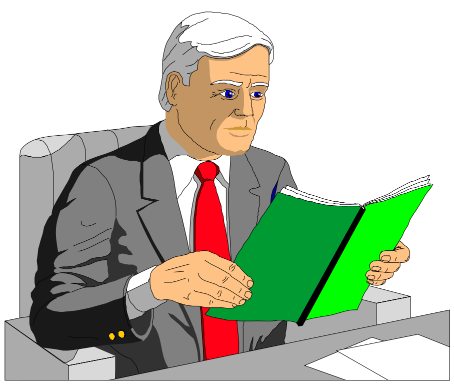
Two driving factors:

- High cost of upgrading existing structures now considered unsafe
  - Requires more exacting assessment
- High cost of damage and associated impacts from structural performance in earthquakes
  - Higher performance criteria

Performance Levels

Engineer --

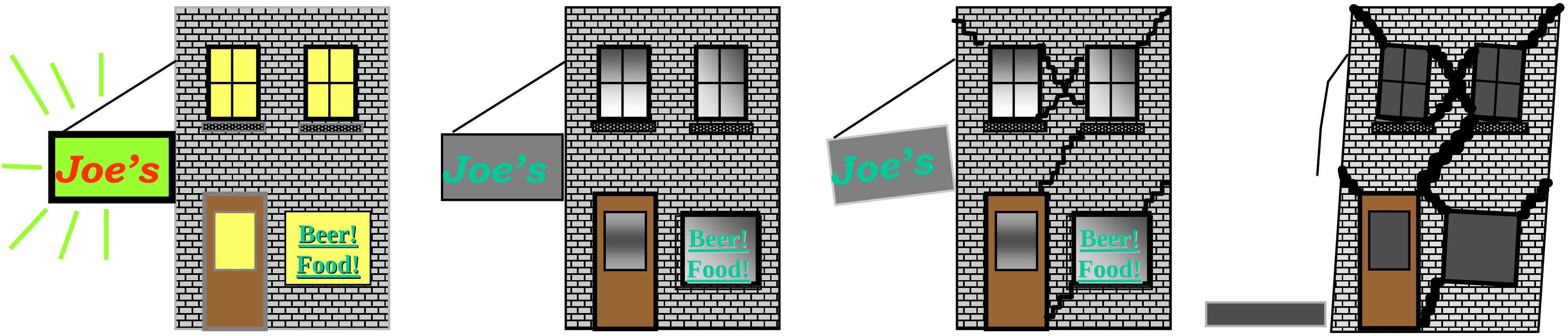
amount of yielding, buckling, cracking, permanent deformation that structure experiences



Owner --

Will the building be safe?
Can I use the building after the earthquake?
How much will repair cost?
How long will it take to repair?

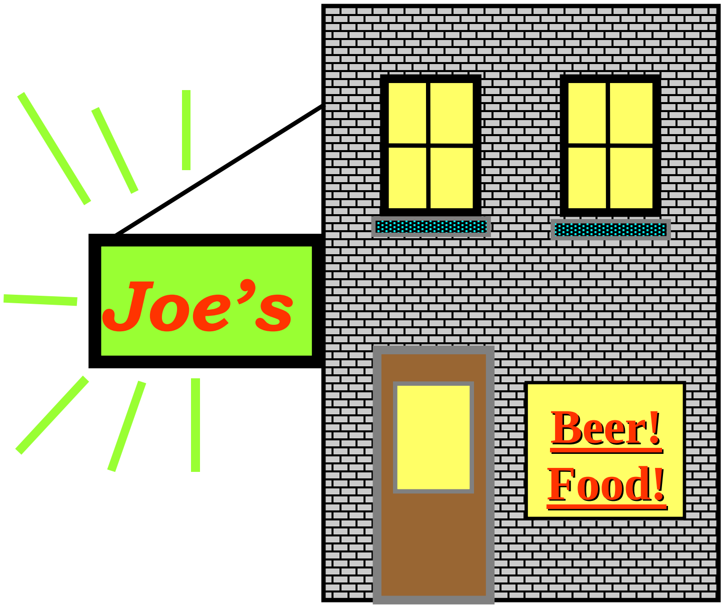
“Standard” Structural Performance Levels



Operational | *Immediate Occupancy* | *Life Safety* | *Collapse Prevention*

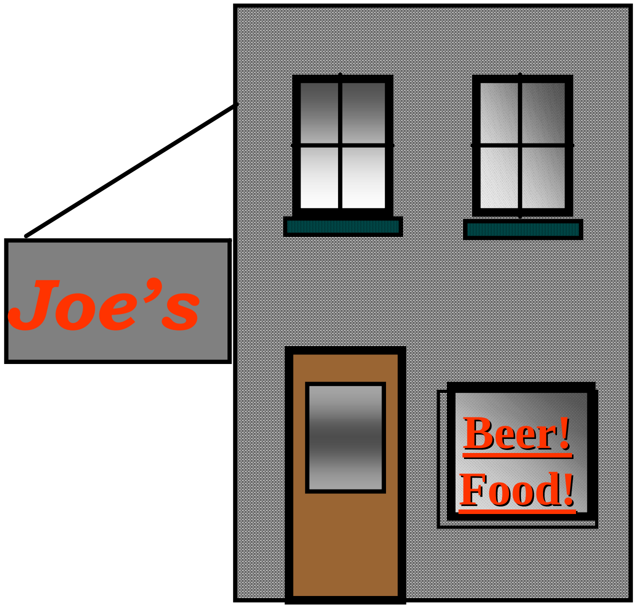
0% — *Damage or Loss* — 99%

Operational Level



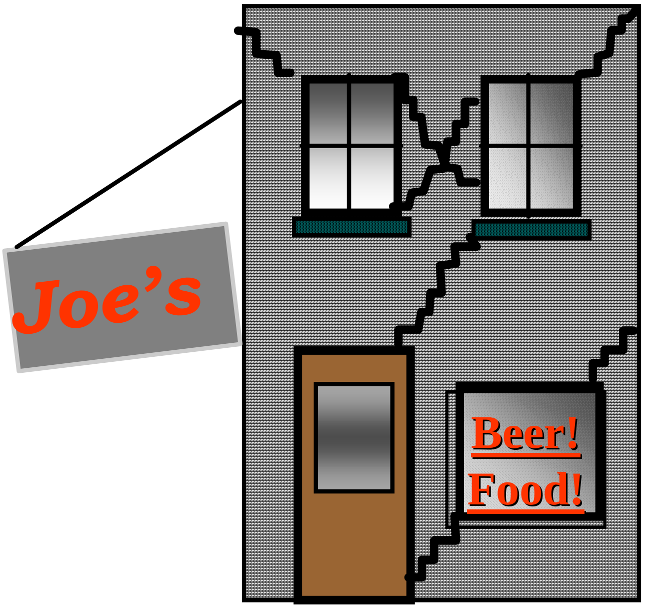
- Negligible structural and nonstructural damage
- Occupants are safe during event
- Utilities are available
- Facility is available for immediate re-use (some cleanup required)
- Loss < 5% of replacement value

Immediate Occupancy Level



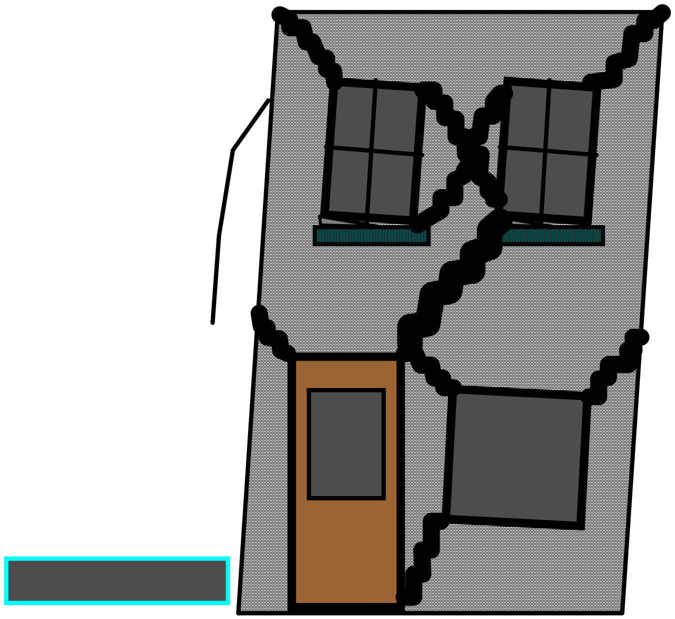
- Negligible structural damage
- Occupants safe during event
- Minor nonstructural damage
- Building is safe to occupy but may not function
- Limited interruption of operations
- Losses < 15%

Life Safety Level



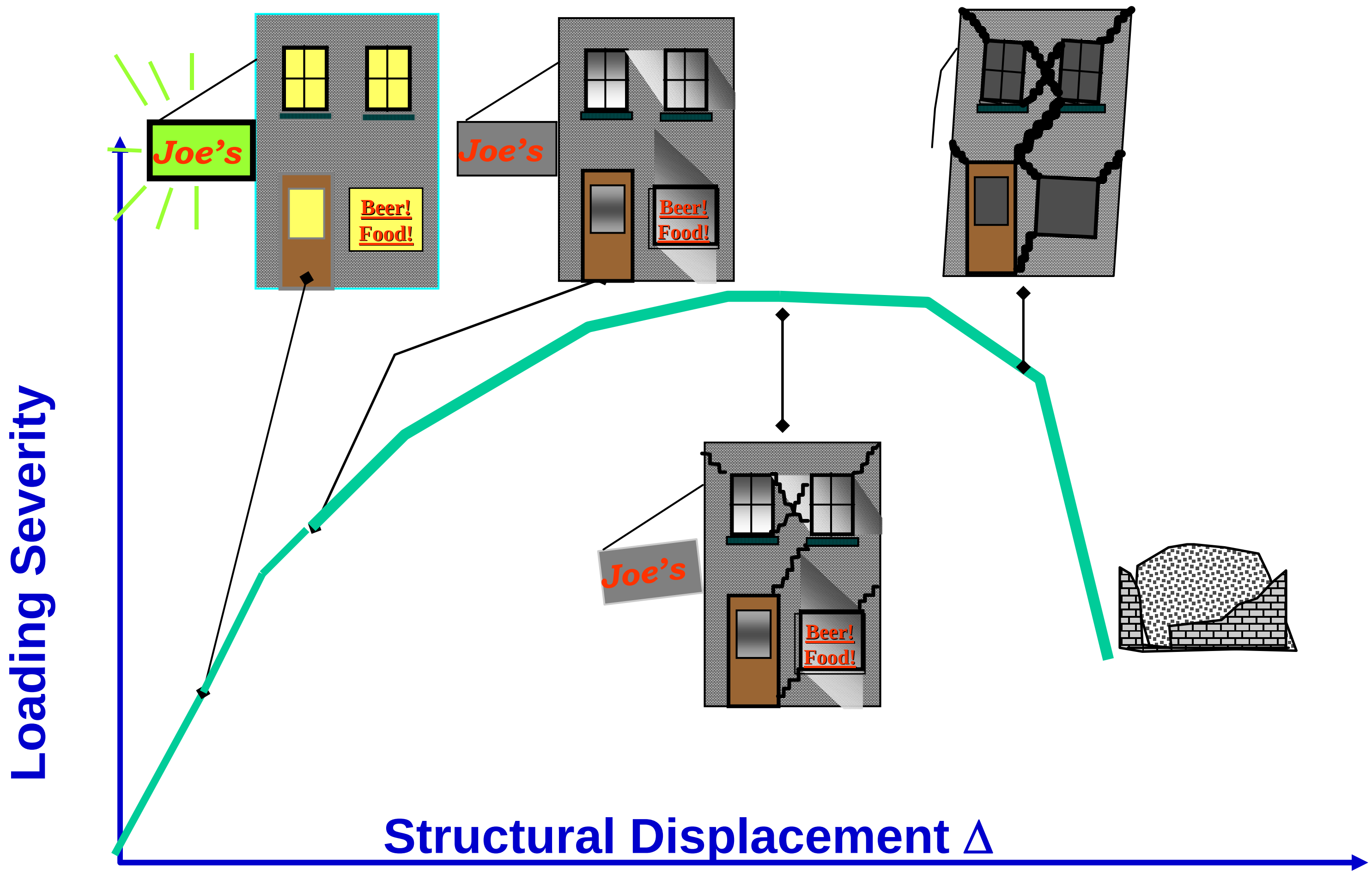
- Significant structural damage
- Some injuries may occur
- Extensive nonstructural damage
- Building not safe for reoccupancy until repaired
- Losses < 30%

Collapse Prevention Level

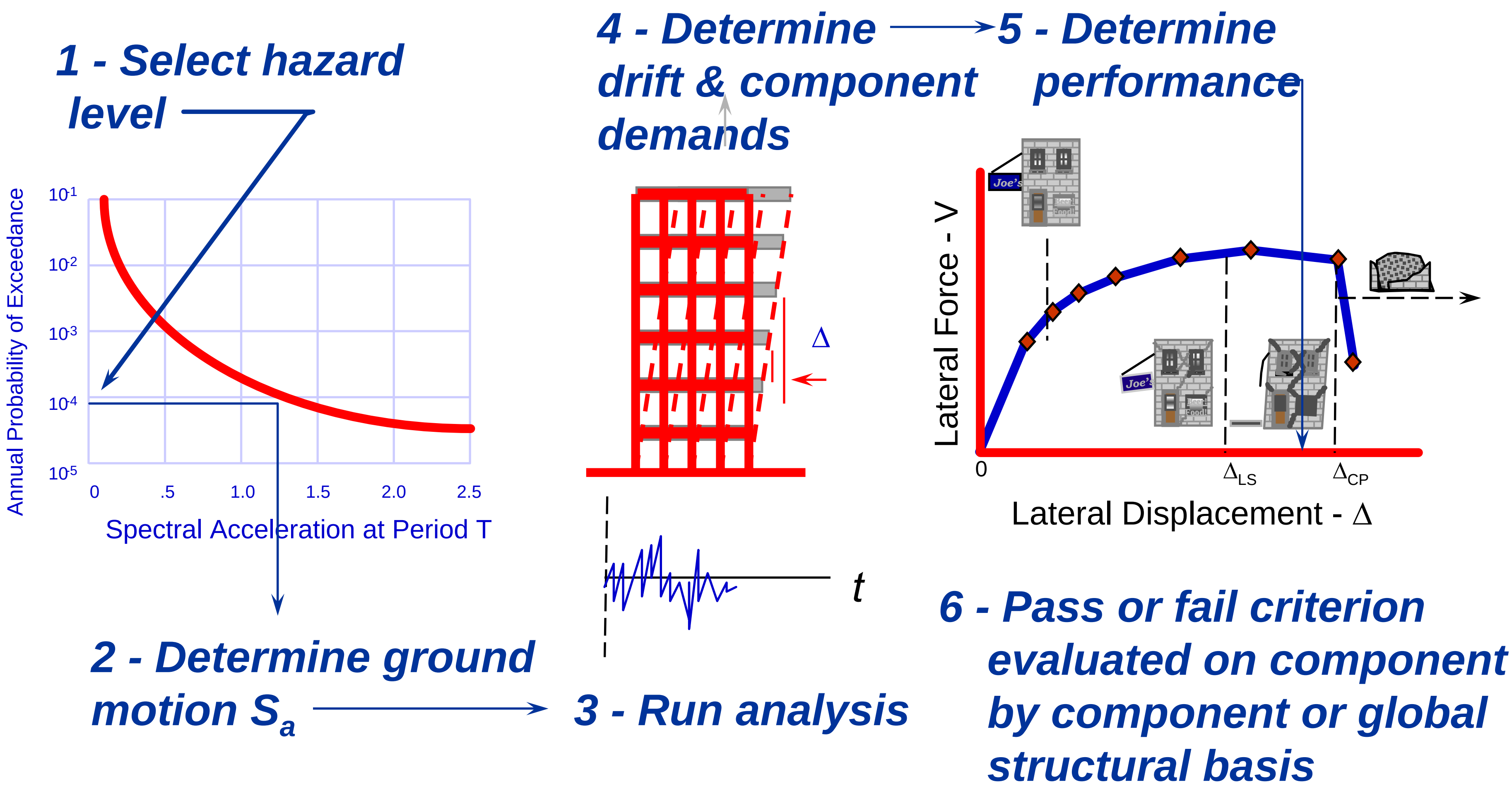


- Extensive (near complete) structural and nonstructural damage
- Significant potential for injury but not wide scale loss of life
- Extended loss of use
- Repair may not be practical
- Loss >> 30%

Global Response and Performance



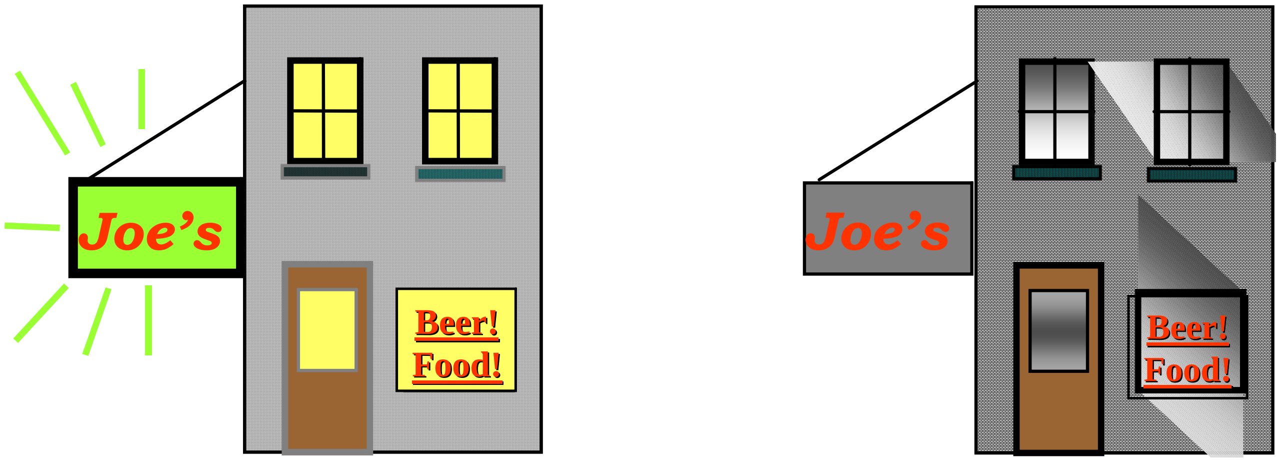
Evaluation Approach



What Type of Analysis?

- The answer depends on:
  - What performance level you are hoping to achieve.
  - The configuration of the structure.
  - How accurate you need to be.
- A wide range of choices are available.

Superior Performance Levels



- Behavior will be essentially elastic
  - Regular structures with short periods
    - Linear static procedures are fine
  - Regular structures with long periods and all irregular structures - linear dynamic procedures are better
    - Response spectra accurate enough

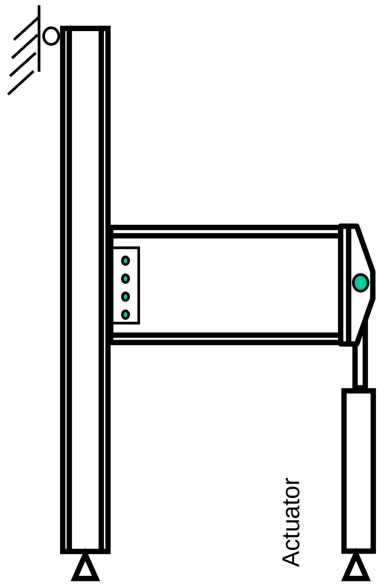
Poorer Performance Levels

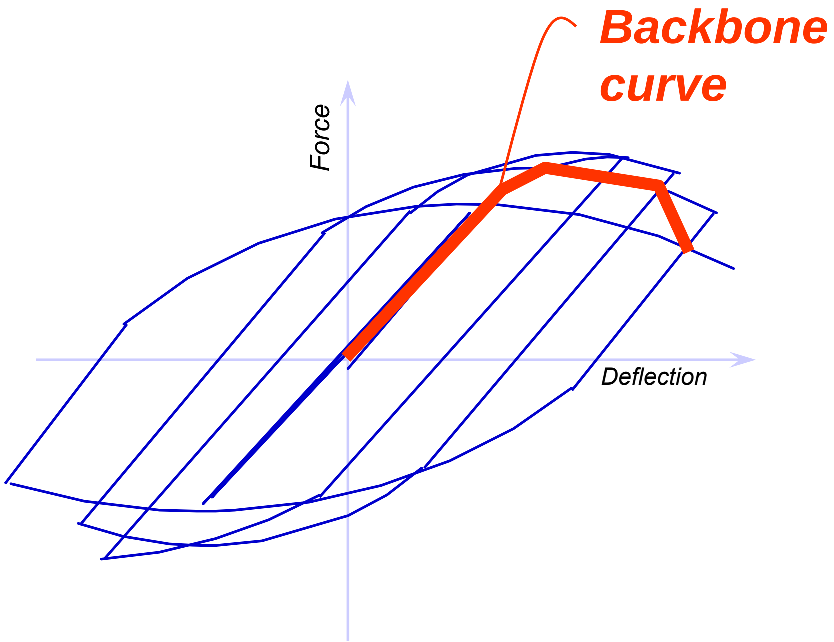
- Inelastic behavior is significant (elastic analyses are the wrong approach!)
  - Structures dominated by first mode response
    - Pushover analysis may be adequate
  - Structures with significant higher mode response
    - Nonlinear time history necessary

Judging Performance Acceptability

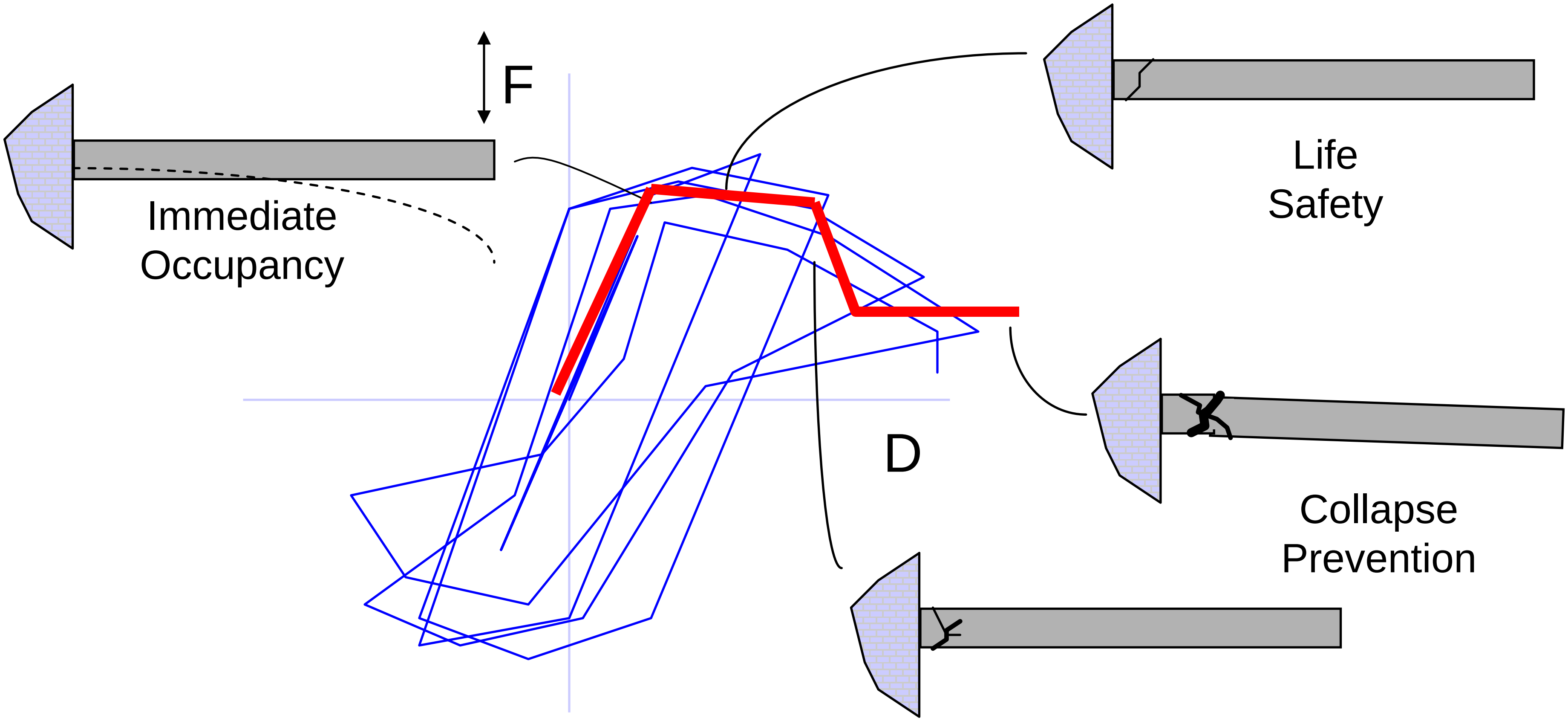
- Acceptance criteria are indicators of whether the predicted performance is adequate
  - Local (component-based)
  - Global (overall structure-based)

Local Response and Performance

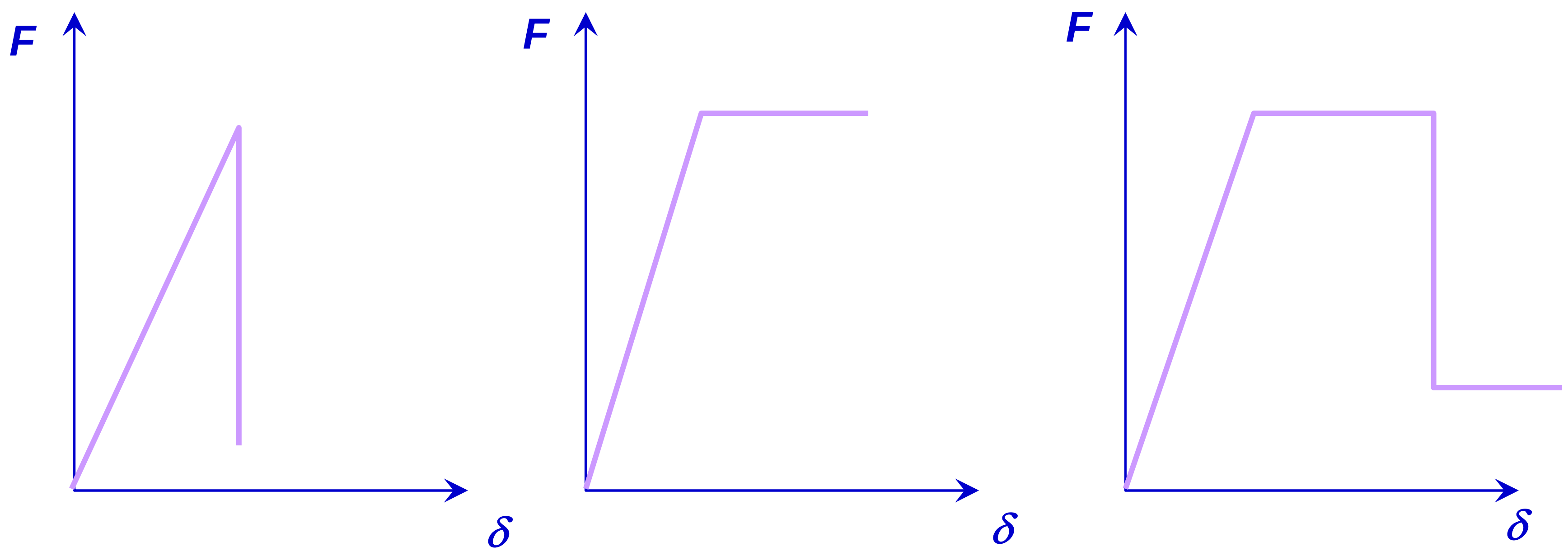




Local (Component-based) Acceptance Criteria



Component Backbones and Acceptance Criteria

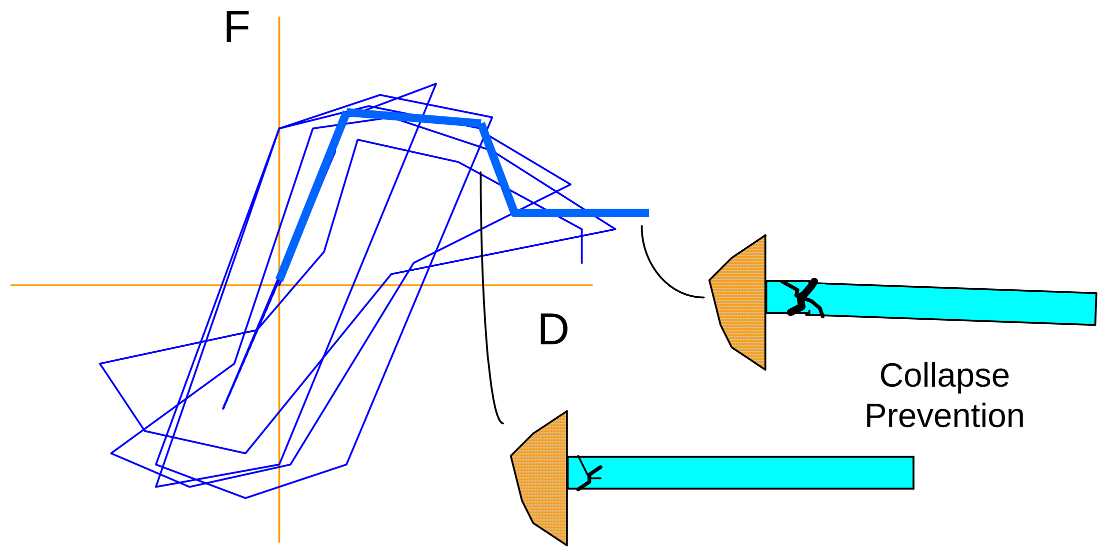


Brittle Behavior (Force Controlled)

Ductile Behavior (Deformation Controlled)

Disadvantages Associated with Local Acceptance Criteria

- The “weakest” or “most highly damageable” element controls the structure’s performance.
- The effect on global stability is difficult to judge.

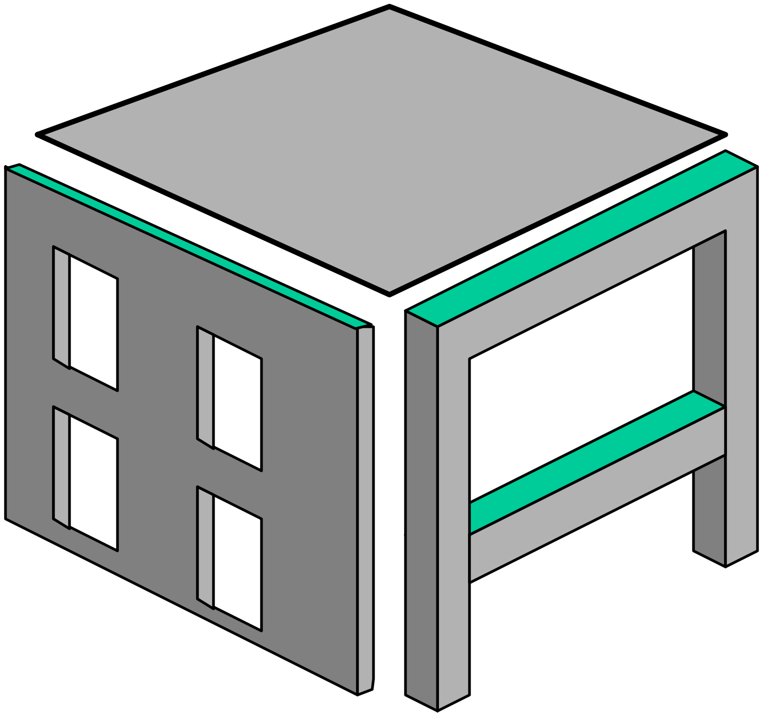


Building Configuration

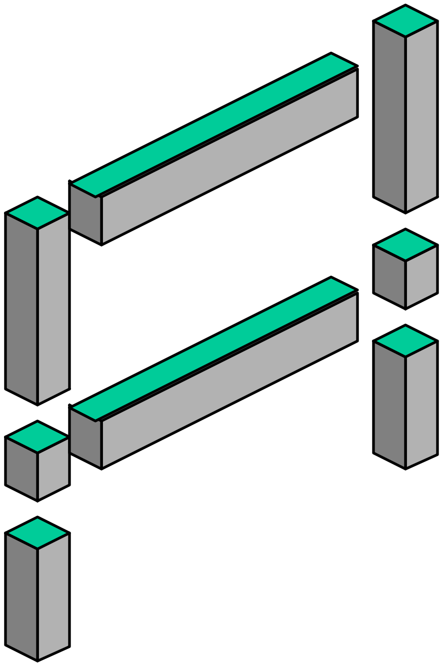
- Hierarchy of “parts” that comprise a building:
  - Elements
  - Components
  - Actions

Elements

- Horizontal or vertical subassemblies that comprise a structure:
  - Braced frame
  - Moment frame
  - Shear wall
  - Diaphragm



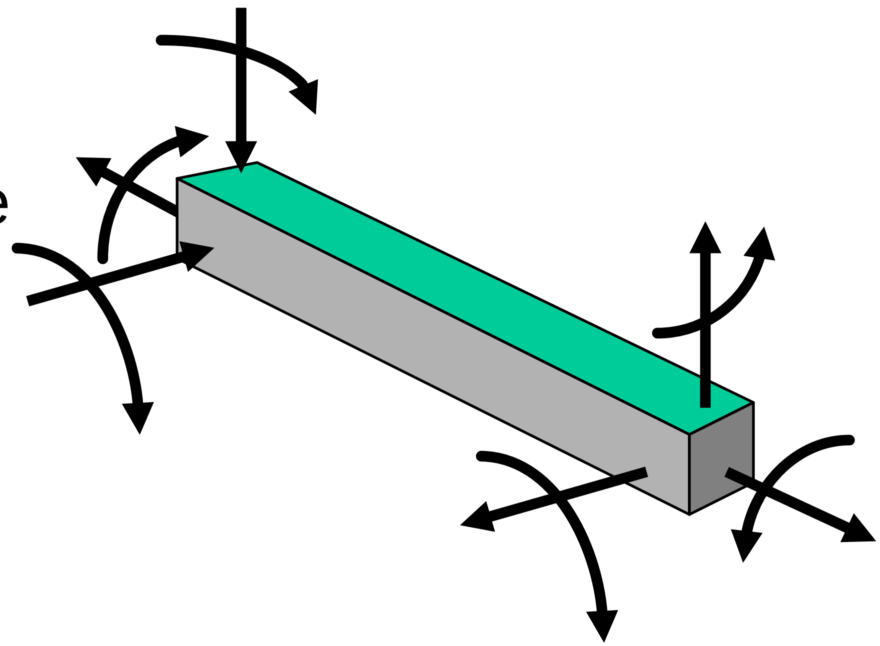
Components



- Individual members that comprise an element:
  - Beam
  - Column
  - Joint
  - Brace
  - Pier
  - Footing
  - Damper

Actions

- Independent degrees of freedom associated with a component, each with an associated force and deformation:
  - Axial force - elongation
  - Moment - rotation
  - Torsional moment - twist



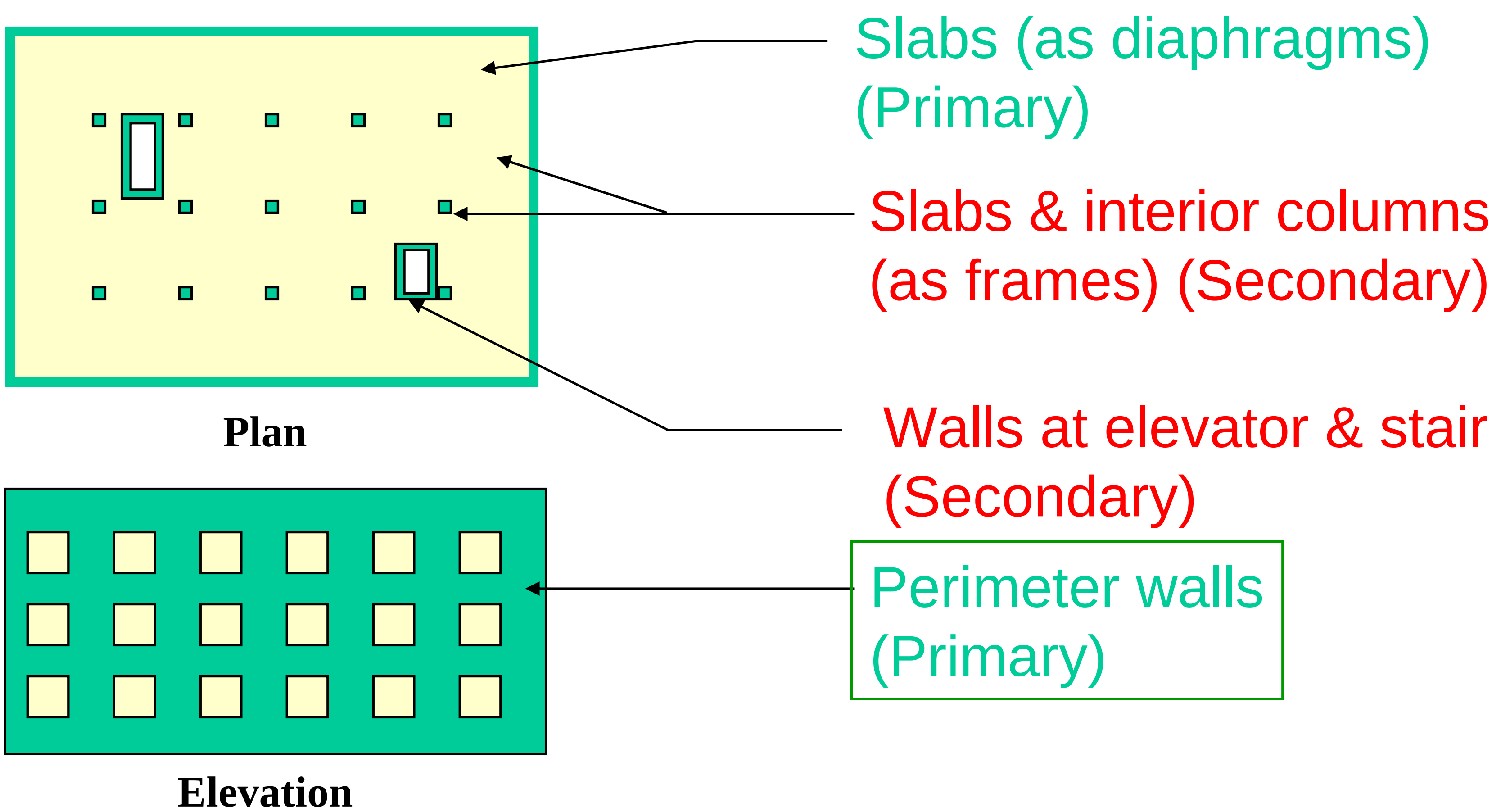
Primary and Secondary Parts

- Primary Elements:
  - Any element (component) {action} required to provide the building's basic lateral resistance.
    - Similar to the concept of a "*participating*" element in the building code.
- Secondary:
  - Any element (component) {action} that is not required to provide the building's basic lateral resistance.
    - May "*participate*" but is not required to do so.

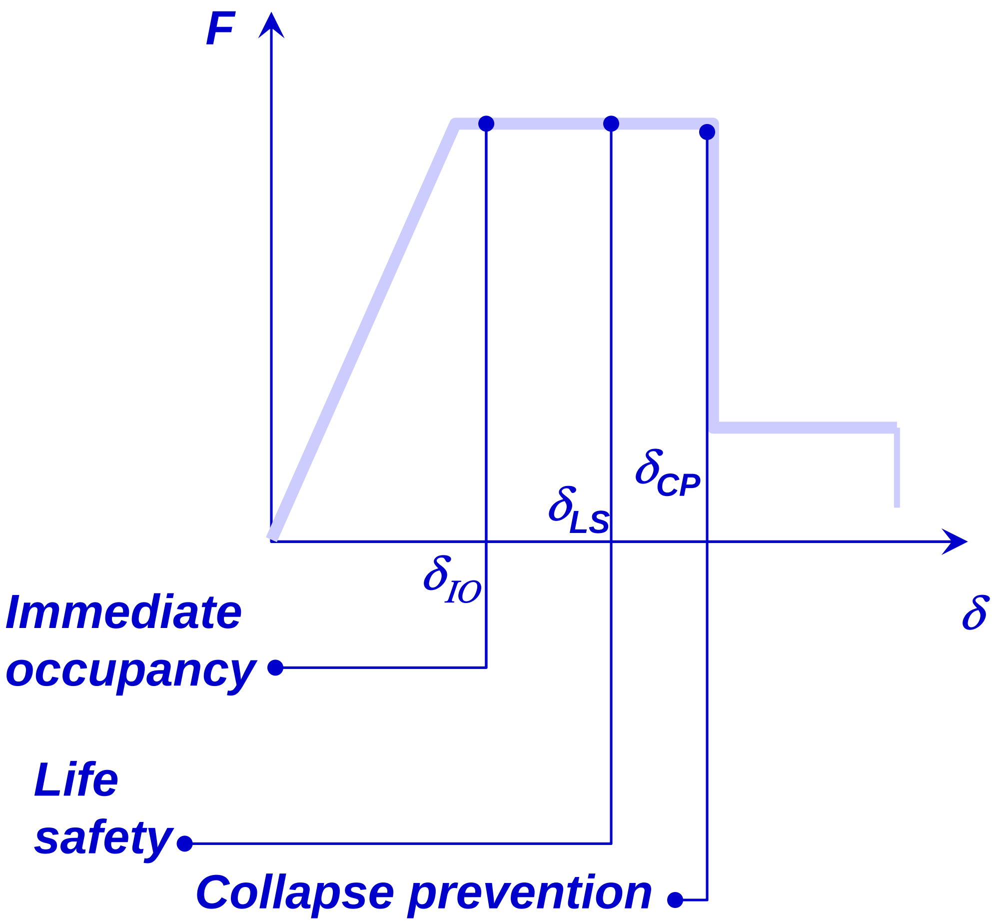
Primary and Secondary

- Permits engineer to utilize judgment in determining whether a building meets the intended performance levels.
  - Secondary elements are permitted to experience more damage than primary elements.
  - Acceptance criteria for secondary elements are more permissive than for primary elements.

Primary & Secondary



Performance Evaluation Primary Components

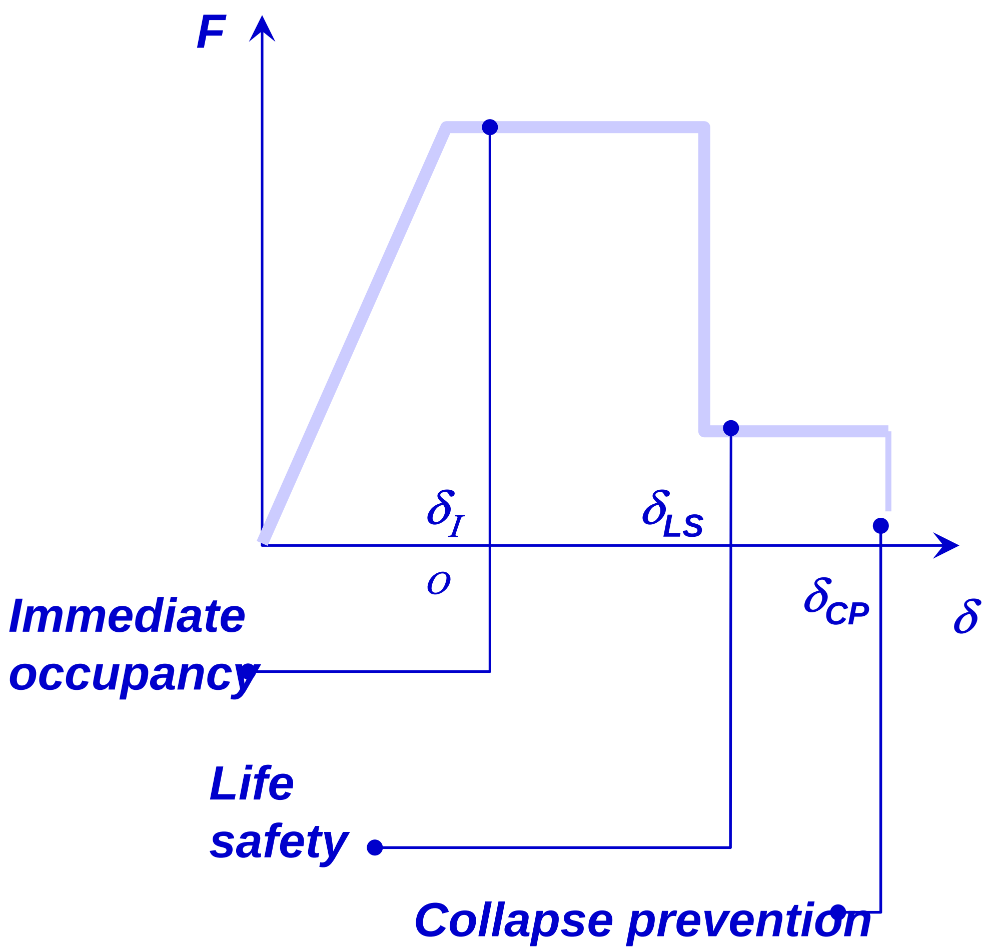


δ_{IO} - based on appearance of damage

δ_{CP} - based on loss of lateral load resisting capacity

δ_{LS} - 75 % δ_{CP}

Performance Evaluation Secondary Components



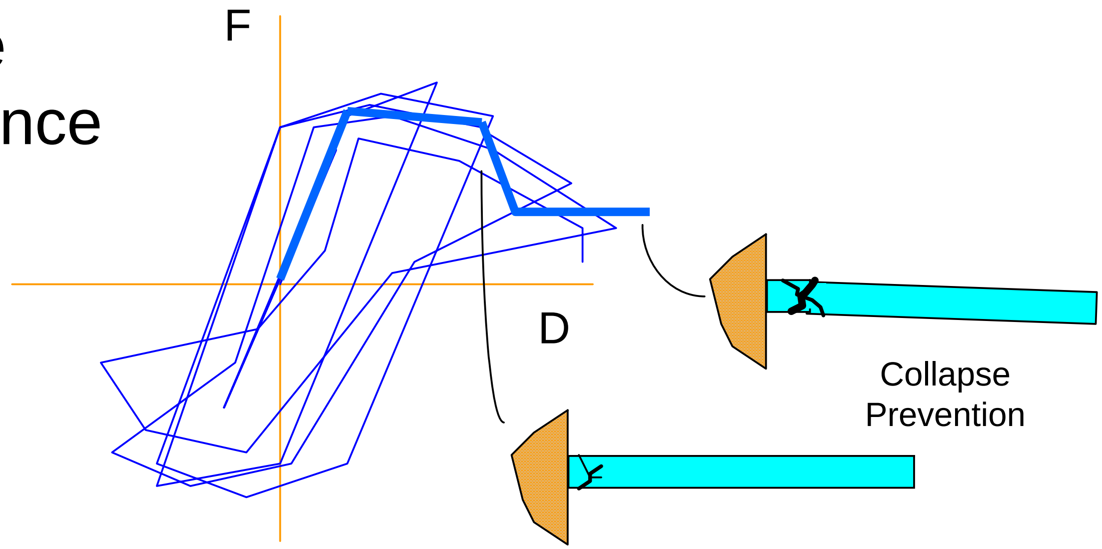
δ_{IO} - based on appearance of damage

δ_{CP} - based on complete failure of element

δ_{LS} - 75% δ_{CP}

Disadvantages Associated with Local Acceptance Criteria

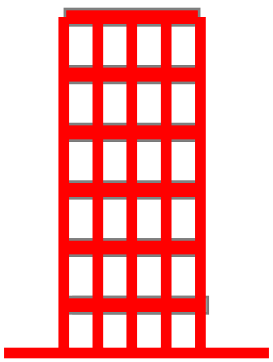
- The "weakest" or "most highly damageable" element controls the structure's performance
- The effect on global stability is difficult to judge



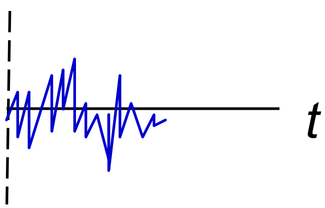
Incremental Dynamic Analysis

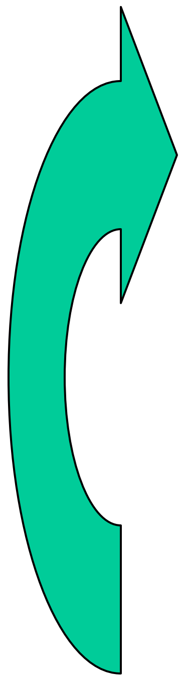
Determining Capacity Limited by Global Stability

1 - Build analytical model

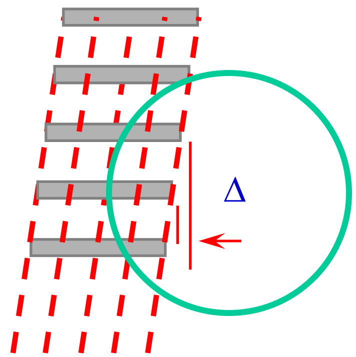


2- Select a ground motion



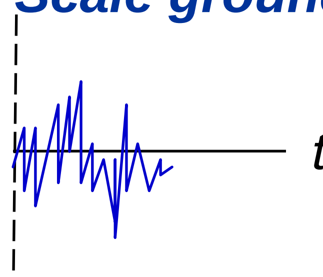


3- Nonlinear time history analysis



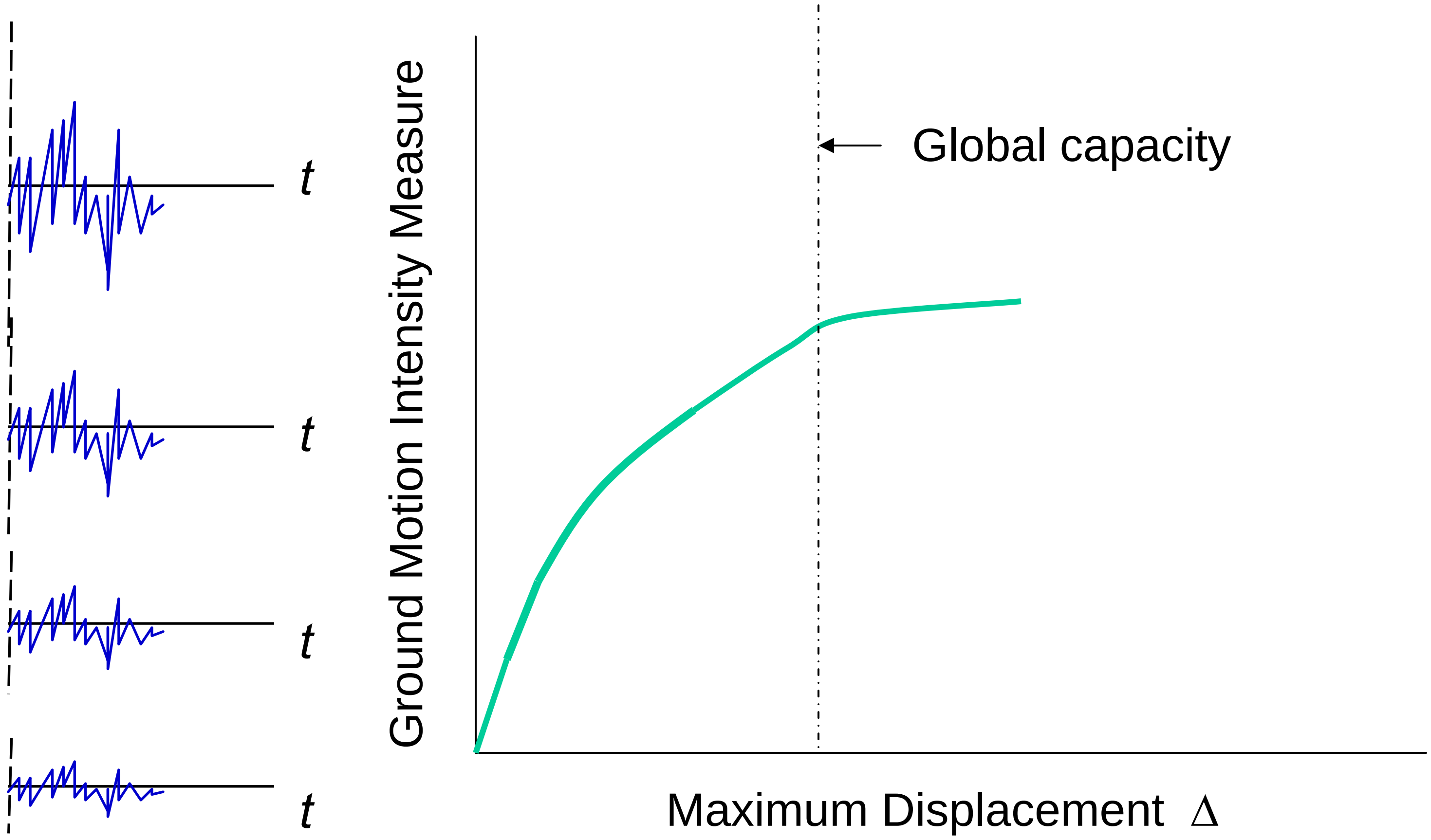
4- Find maximum displacement

5- Scale ground motion up & repeat



Incremental Dynamic Analysis

Determining Capacity Limited by Global Stability

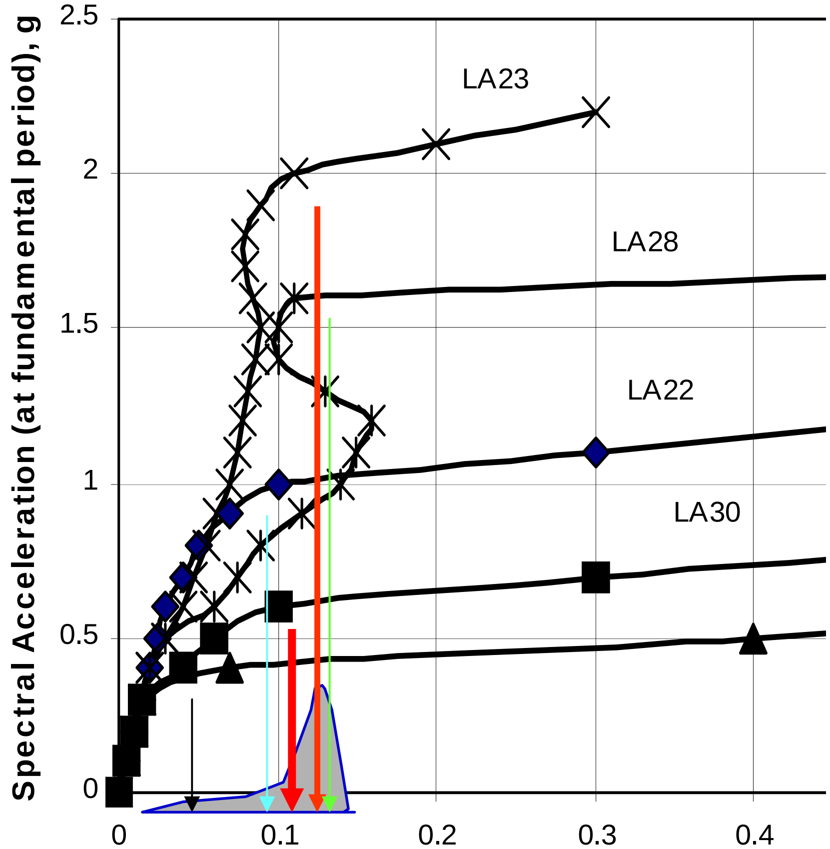


Perception of a Guarantee

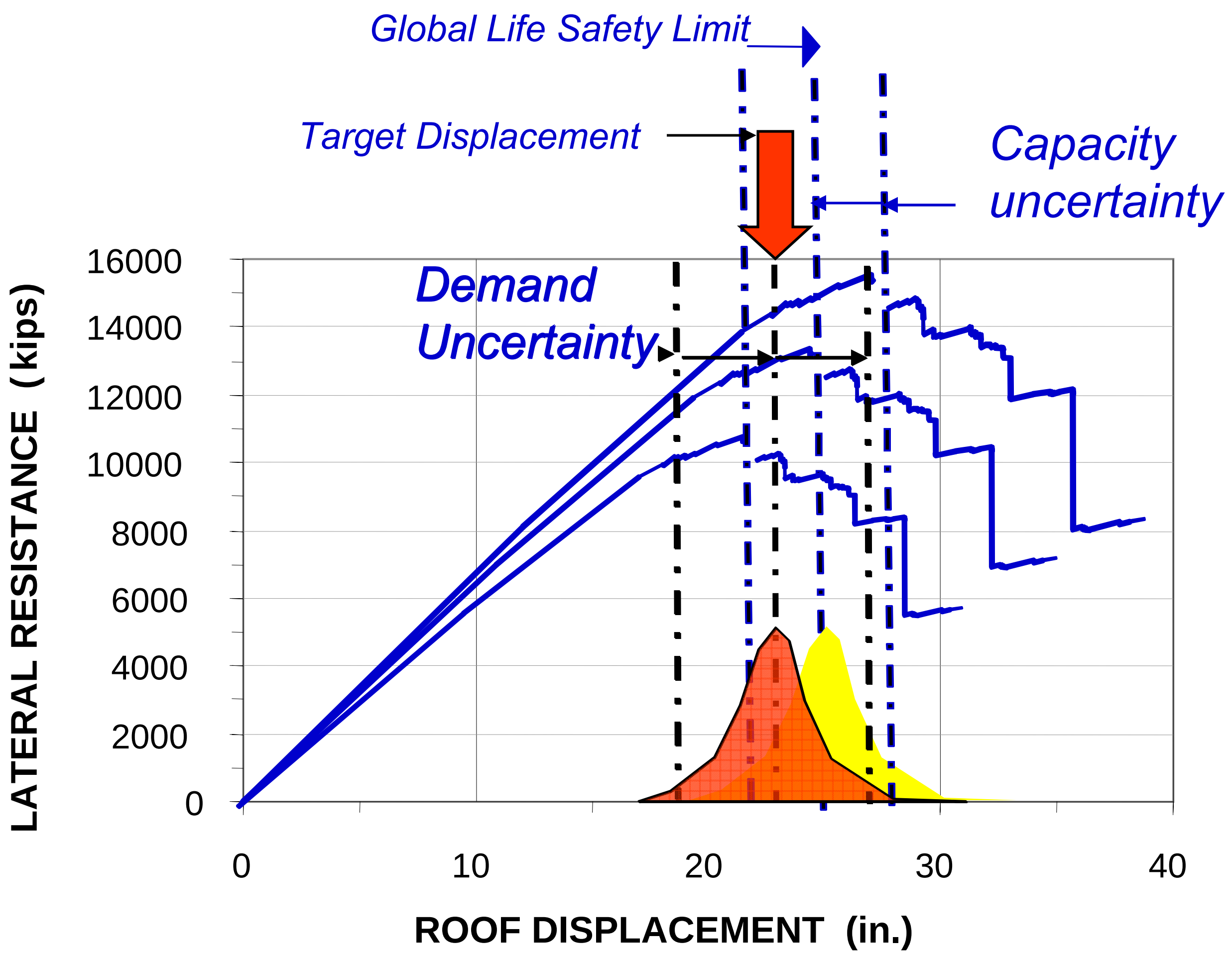
How Could This Happen?

- Loading that will occur in the future is uncertain.
- Actual strength of materials and quality of construction is variable.
- Neither the real demands nor the capacity of the structure to resist these demands can be perfectly defined.

Ground Motion and Capacity are Uncertain and Variable



Capacity, Demand, and Performance Prediction



Performance Objective Redefined

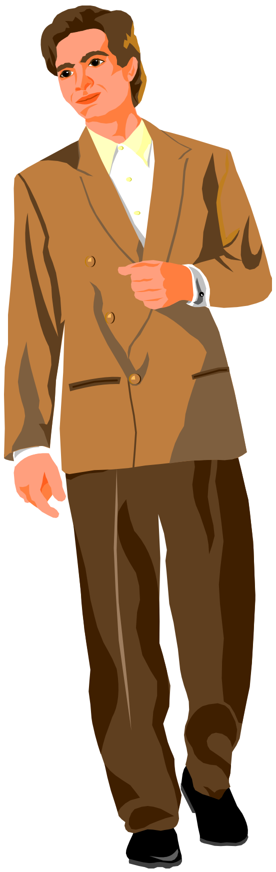
- Vision 2000 / FEMA 273/356:
  - Damage will not exceed desired level, given that ground motion of specified probability is experienced.
- SAC Approach:
  - Total probability of damage exceeding a desired level, will not exceed a specified amount, given our understanding of site hazards.
    - Confidence level associated with achieving this performance is defined.

Performance Objectives Redefined

highly
I am moderately confident
not very

that there is less than x%
chance in 50 years

that damage will be worse than
Immediate occupancy
Collapse prevention



Total Probability of Damage Exceeding Specified Level

$$P(Damage > PerLev) = \int P\left|D > C\right|GM\left|P\left(GM\right)\right.$$

D = demand (drift, or force) = b (GM) - random variable β_D

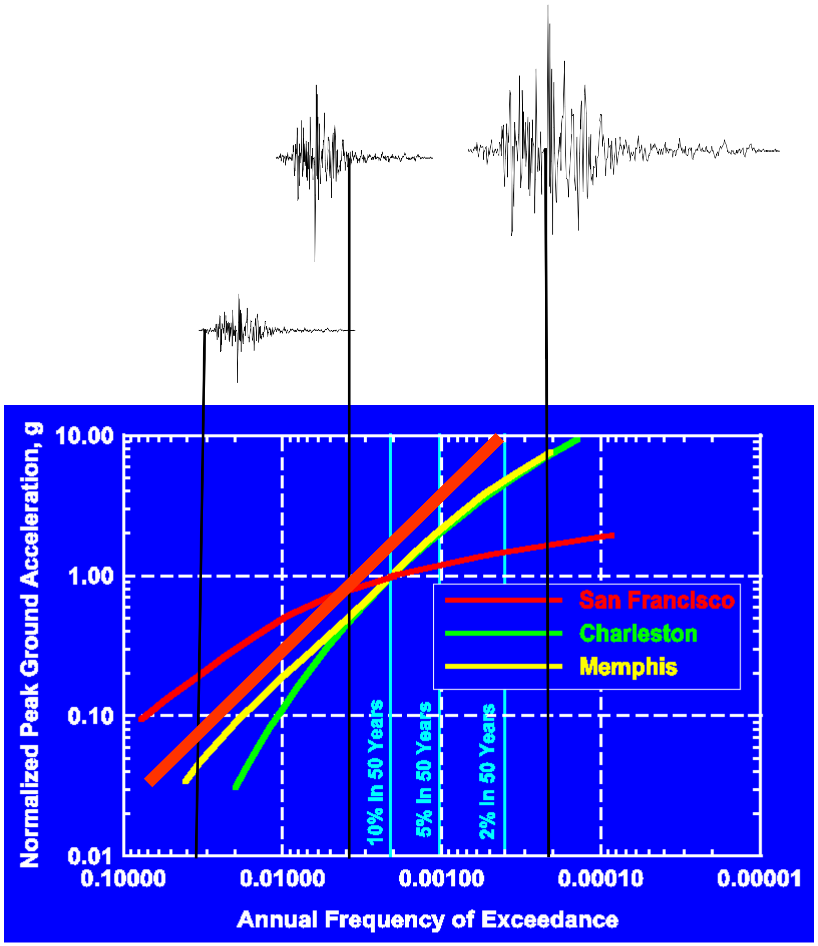
C = capacity (function of drift or force) - random variable β_c

ln(GM) = kln(PE)

β_D, β_C defined in terms of random and uncertain components

Load and resistance factors derived as products of integration

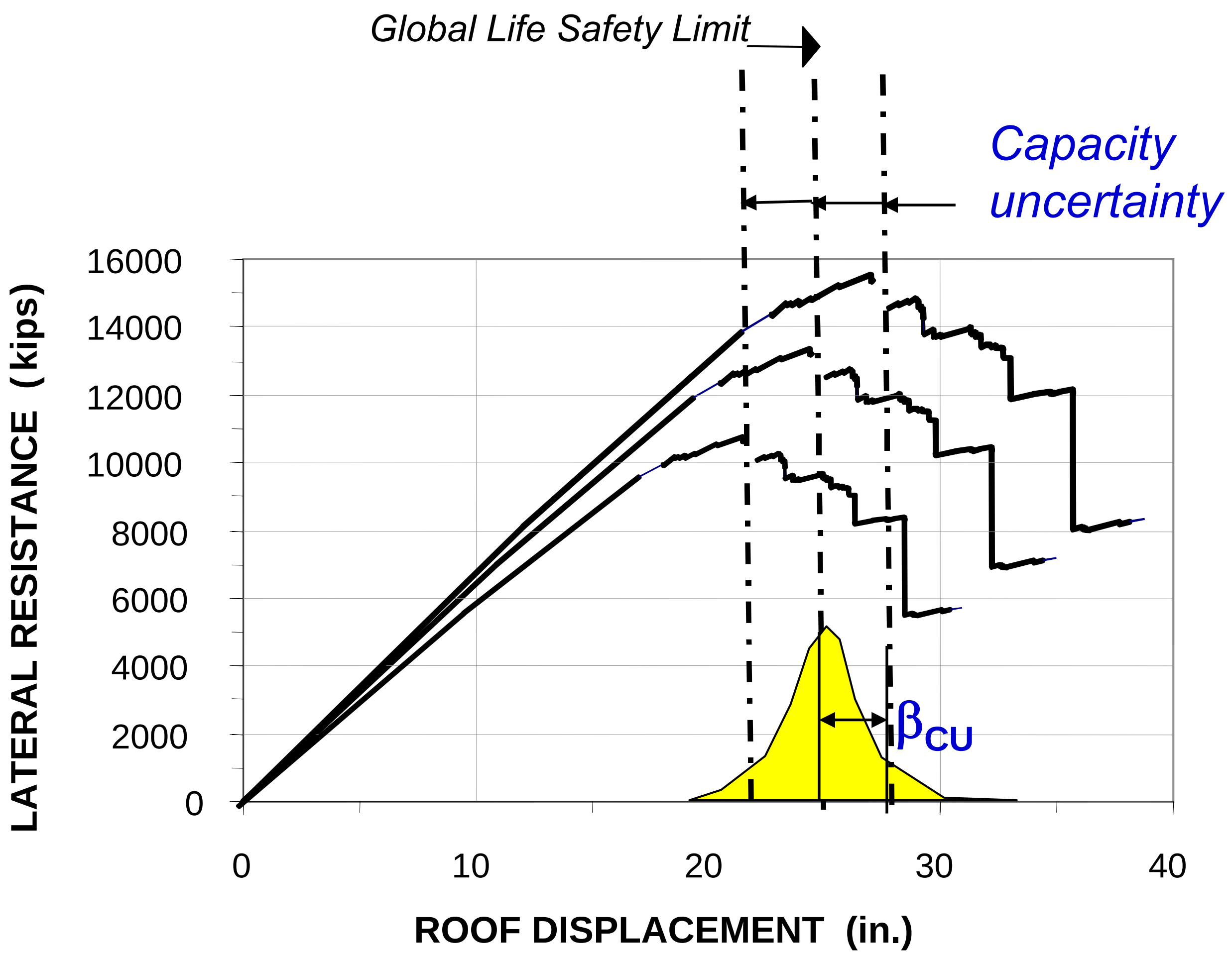
Hazard Level and Load Severity



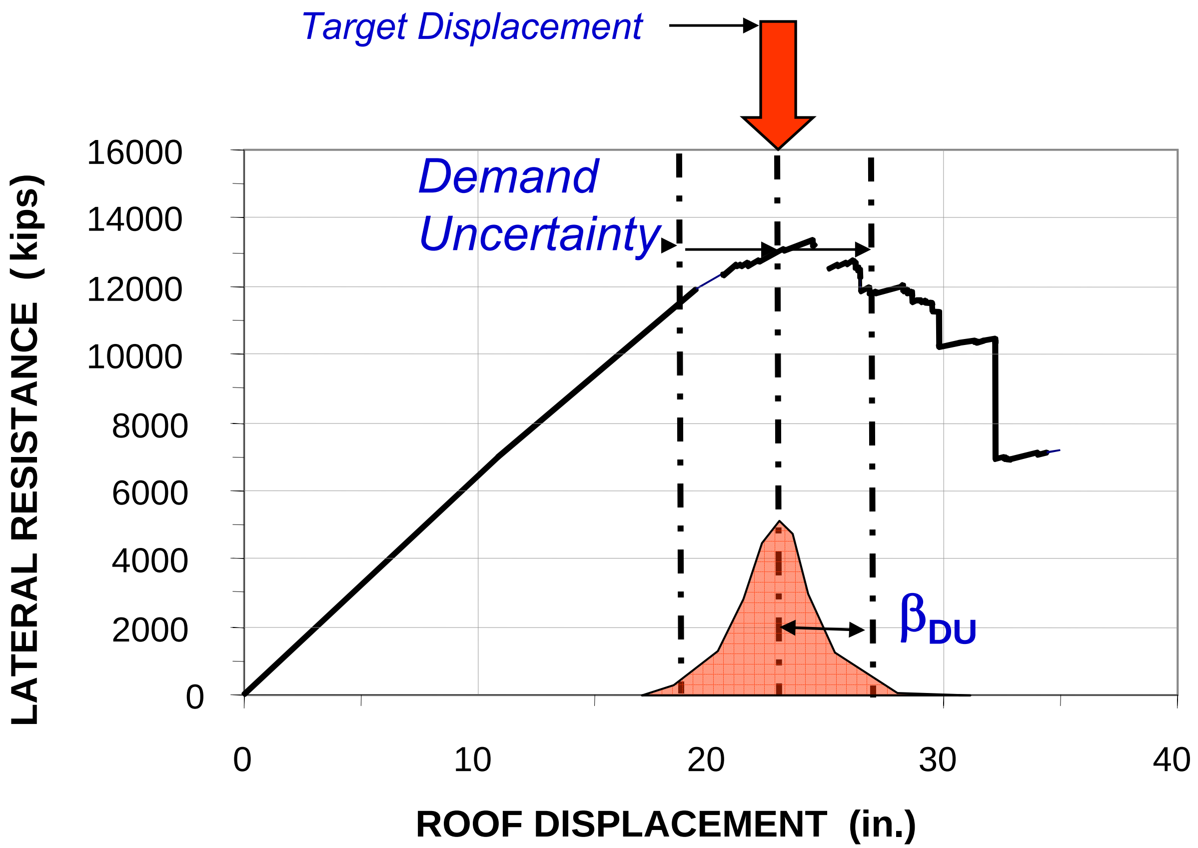
- Minor earthquakes occur frequently.
- Moderate earthquakes occur occasionally.
- Major earthquakes occur rarely.

Mathematically, "*k*" is the slope of the hazard curve and indicates how much more intense motion gets with decreasing probability of exceedance.

Uncertainty in Capacity b_{CU}



Uncertainty in Demand b_{DU}



Demand and Resistance Factor Procedure

- Demand and resistance factors computed as products of integration, functions of hazard, randomness and uncertainty

$$\gamma = e^{\frac{k}{2b}\beta_{DR}^2};\gamma_a = C_B e^{\frac{k}{2b}\beta_{DU}^2};\phi = e^{\frac{-k}{2b}\left(\beta_{CU}^2+\beta_{CR}^2\right)}$$

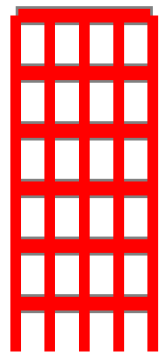
- Factored demand -- Capacity ratio used to determine confidence of successful performance

$$\lambda = \frac{\gamma\gamma_a D}{\phi C}$$

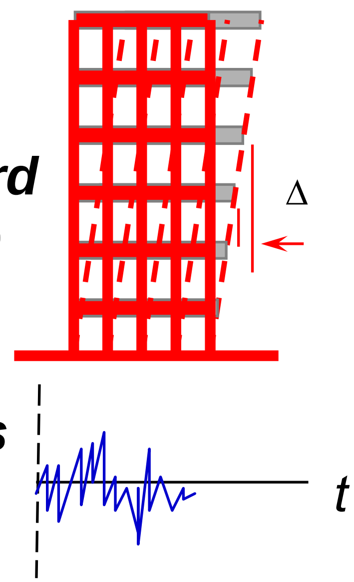
- λ = 1 indicates mean confidence (on order of 60%)
- < 1 indicates higher than mean confidence
- > 1 indicates less than mean confidence

Procedure

1. *Start with frame design:*
   - *Configuration*
   - *Member sizes*
   - *Connection details*



2. *Analyze frame :*
   - *Use ground motion at appropriate hazard level (x% - 50 years)*
   - *Predict maximum drift, member deformations, forces*



3. *Correct predicted maximum demands for known inaccuracies in prediction method to obtain median estimate of demand.*

$$\gamma \gamma_a D$$

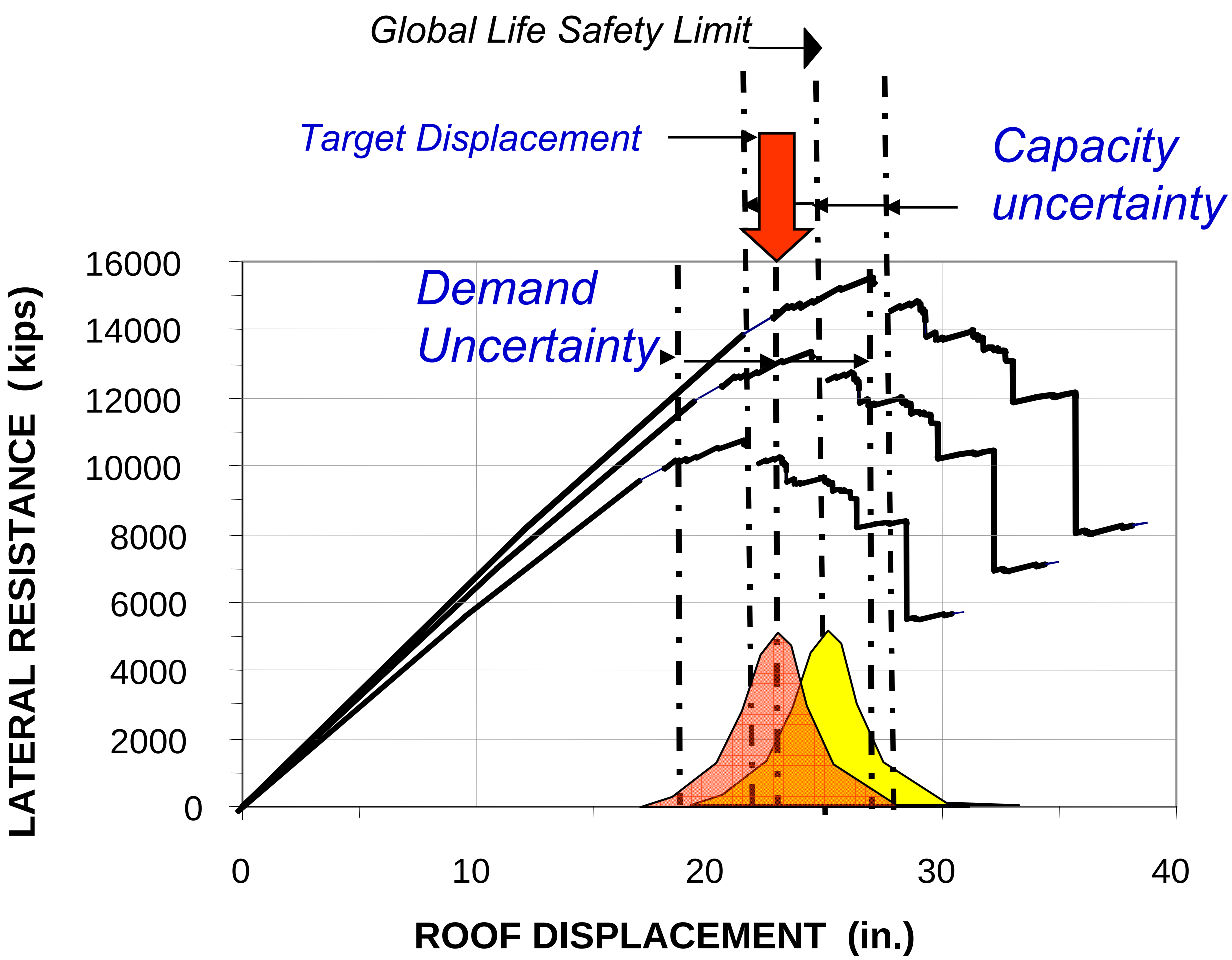
Procedure

4. ***Compute factored demand to capacity ratio (DCR)***

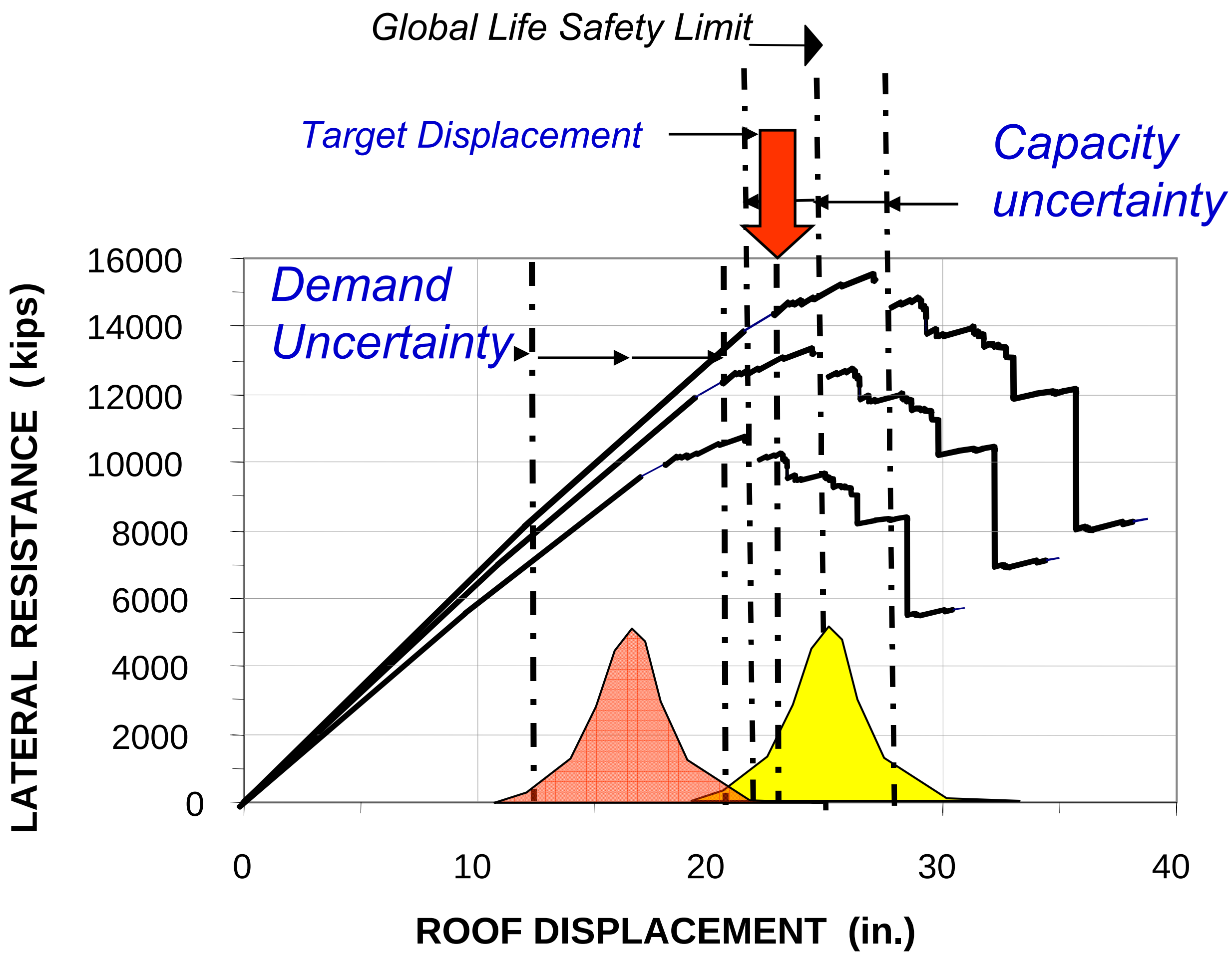
$$\lambda = \frac{\gamma\left(\gamma_a D\right)}{\phi C}$$

Confidence	2%	5%	10%	20%	30%	40%	50%	60%	70%	80%	90%	95%	98%
λ	3.0	2.6	2.2	1.9	1.6	1.5	1.3	1.2	1.1	0.95	0.8	0.7	0.5

Low Confidence λ > 1



High Confidence λ < 1



Summary

- Performance-based design for earthquake resistance is possible.
    - There is considerable uncertainty associated with prediction of performance.
- LRFD approach developed for steel moment frame buildings allows the engineer to be honest as to confidence that performance may (or may not) be achieved.
- Communication is more complex but less dangerous.
- Extensive work necessary to derive demand and resistance factors for various structural systems for general application.

SEISMIC HAZARD AND SEISMIC RISK ANALYSIS

- Seismotectonics
- Fault mechanics
- Ground motion considerations for design
- Deterministic and probabilistic analysis
- Estimation of ground motions
- Scaling of ground motions and design and analysis tools (i.e., NONLIN)

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Seismic Activity > M5 Since 1980

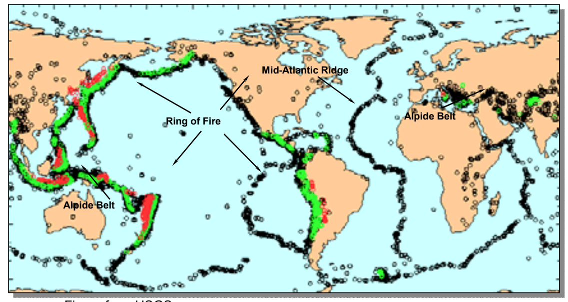

Figure from USGS

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Crustal Plate Boundaries

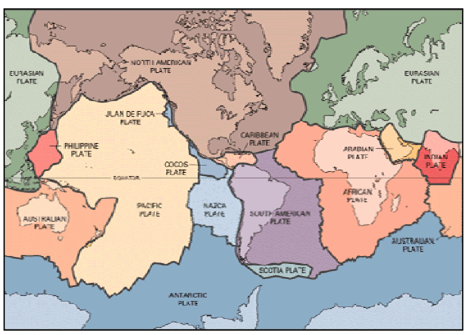

Figure from USGS

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Convection Drives the Plates

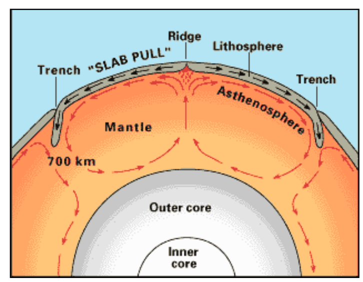

Figure credit: USGS.

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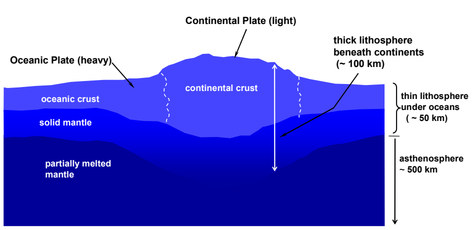
Oceanic and Crustal Plates



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Continental-Continental Collision
(orogeny)

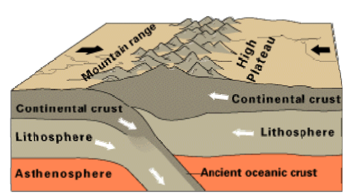

Figure credit: USGS.

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Oceanic-Continental Collision
(subduction)

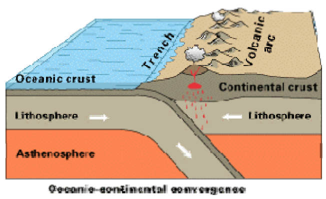


Figure credit: USGS.

Types of Earthquakes

About 90% of the earth's seismicity occurs at plate boundaries on faults directly forming the interface between two plates. These are called **plate-boundary or <u>interplate</u>** earthquakes.

The other 10% occur away from the plate boundary, in the interior of plates. These are called **<u>intraplate</u>** earthquakes.

Plate-boundary Earthquakes

A plate-boundary (interplate) earthquake is an earthquake that occurs along a fault associated with an active plate boundary. An example of this type of boundary is the San Andreas Fault in California.

⇒ Frequent occurrence, relatively well understood behavior, as per plate tectonic theory.

San Andreas Fault – Well Known Plate Boundary

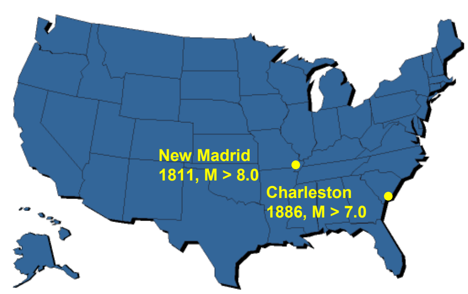
Photo courtesy of: USGS.

Intraplate Earthquakes

An intraplate earthquake is an earthquake that occurs along a fault within the stable region of a plate's interior (SICR). Examples are the 1811-12 Madrid, MO earthquakes, the 1886 Charleston, South Carolina, earthquake, and, more recently, the Bhuj, India, earthquake in 2001.

⇒ Infrequent occurrence, poorly understood, difficult to study.

Historical Large Intraplate Earthquakes

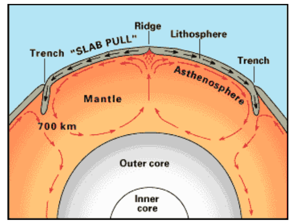


* Largest historical earthquakes in contiguous United States occurred east of the Mississippi!!

Why Intraplate Earthquakes?

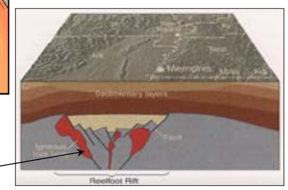
- Ancient "Rifts" – very old fractures in crust related to previous episodes of continental spreading.
- "Weak Spots" – heating up and thinning of lower crust such that the brittle-ductile transition (molten rock/crust boundary) migrates to a higher level. Because the overlying crust becomes thinner, stresses become more concentrated in the crust.

Why Intraplate Earthquakes?



Figures from USGS

Example of 700 million year old rift zone:



Rift allows stress concentrations

Why Intraplate Earthquakes?

- Thermal destabilization -- sinking of mafic rock mass (rock mass of heavy minerals) into underlying molten rock. As mafic block sinks, stresses are concentrated in overlying crust. Process thought to be due to rock density anomalies combined with thermal processes.
- Other localized mechanisms? (meteor impact craters, etc.)

Seismicity of North America

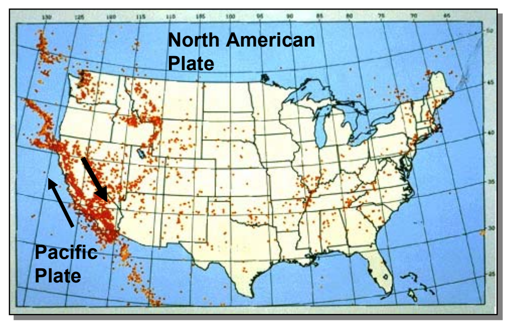


Figure credit: USGS.

California Seismicity

Seismicity relatively well understood

Figure credit: USGS.

Pacific Northwest – Cascadia Subduction Zone

Ultimate magnitude potential?

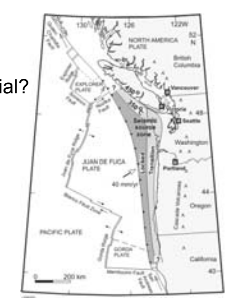


Figure Credit: USGS

Idaho, Utah, Wyoming

Recurring events along Wasatch Fault

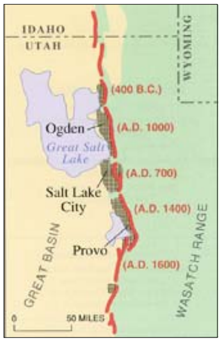
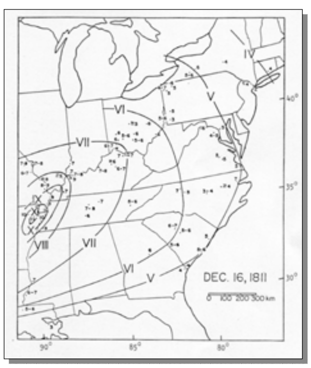


Figure credit: USGS.

Central US Seismic Zones

- Who really knows for sure?
- The Reelfoot Rift is associated with many events in this region.

Figure credit: USGS.



Isoseismal Map from New Madrid Earthquake, Dec. 16, 1811

Figure credit: USGS.

Reelfoot Rift Associated with Central US Earthquakes

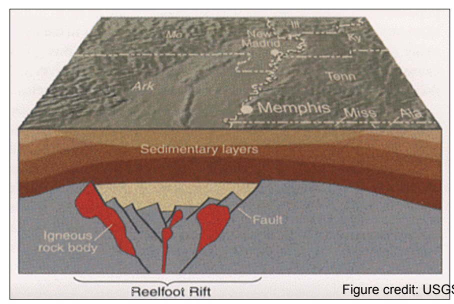
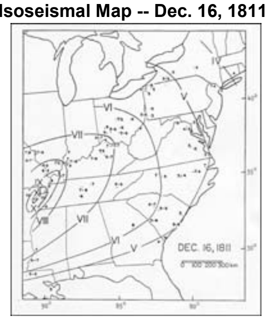


Figure credit: USGS.

1811-12 New Madrid Earthquakes (three M8+)

Isoseismal Map -- Dec. 16, 1811



Reelfoot Lake, Tennessee, was created due to subsidence and tectonic change

Figure and photo credit: USGS.

How Big is the CEUS Problem?

New Madrid Seismic Zone

- Highest hazard in the US outside the WUS
- M1-2 every other day (200 per year)
- M3 every year (felt)
- M4 every 1.5 years (local minor damage)
- M5 every 10 years (damaging event)
- M6 every 80 years (last one in 1895)
- M8+ every 400-600 years? (last one in 1812)

- M6-7.5 has 25-40% chance in 50 years
- M8+ has 4-10% chance in 50 years

How Big Is the CEUS Problem?

- A recurrence of the New Madrid earthquake, postulated with a 4-10% probability in the next 50 years, has been estimated to cause a total loss potential of $200 billion with 26 states affected.
- Approximately 2/3 of the projected losses will be due to **interruptions in business operations and the transport** of goods across mid-America.
- This **economic loss is of the same order as that caused by the terrorist attacks of September 11**, 2001 (NRC, 2003).

Southeastern Seismicity

- Tennessee relatively active
- 1886 South Carolina event not fully explained
- Magnetic signature from North Carolina to Georgia similar to Charleston area; same potential?

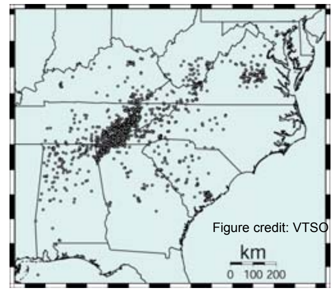


Figure credit: VTSO

Epicenters of earthquakes (M > 0.0) in the southeastern US from 1977 through 1999.

Isoseismal Map from the 1886 Charleston Earthquake

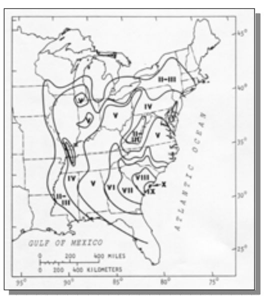


Figure credit: USGS.

Isoseismal Map for the Giles County, Virginia, Earthquake of May 31, 1897; M ≈ 6?

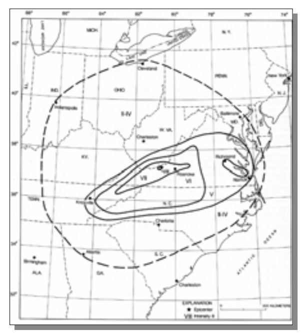


Figure credit: USGS.

Recent Paleoseismological Studies

- Studies in the central and southeastern United States indicate recurring large prehistoric earthquakes – this has increased hazard
- Studies in Pacific Northwest debatable

Isoseismal Map from the 1886 Charleston Earthquake

Figure credit: USGS.

1886 Charleston Earthquake

Photo credit: USGS

1886 Liquefaction Feature

Photo credit: USGS

Prehistoric Sand Crater in Trench Wall

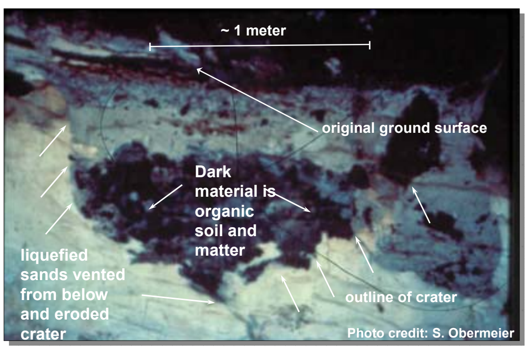


Photo credit: S. Obermeier

Schematic of Ancient Sand Crater

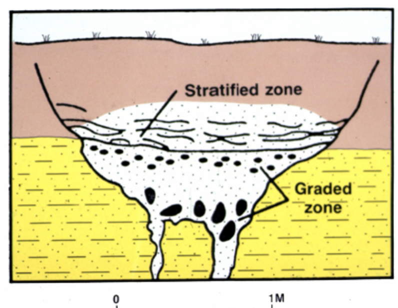


Figure from Obermeier, 1998.

Ages of Earthquake-induced Liquefaction Features Found in Charleston Region*

600 ybp

1250 ybp

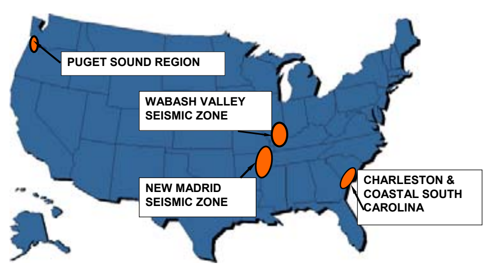
3250 ybp

5150 ybp

> 5150 ybp

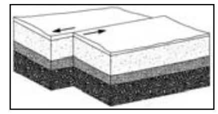
* Study led to increased seismic design values in South Carolina.

Virginia Tech Paleoliquefaction Studies

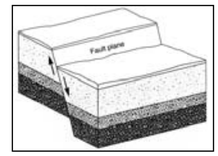


Artesian Condition?

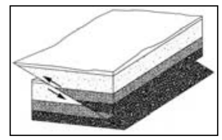
Types of Faults



(a) Strike-slip fault



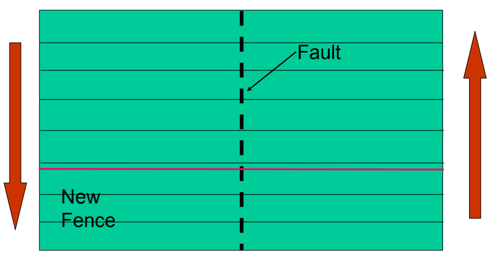
(b) Normal fault



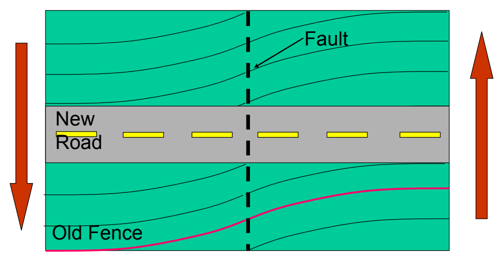
(c) Reverse fault

Elastic Rebound Theory

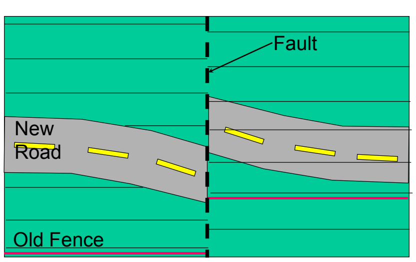
Time = 0 Years



Time = 40 Years (strain building)



Time = 41 Years (strain energy released)



San Andreas Fault, San Francisco, 1906

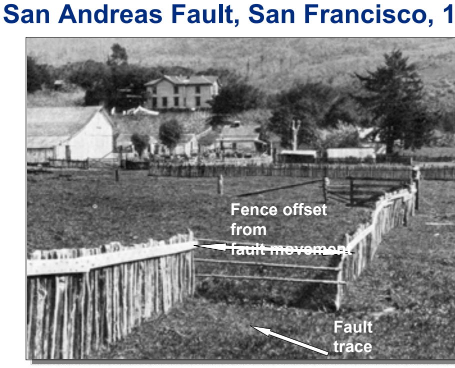


Photo credit: USGS.

Moment Magnitude

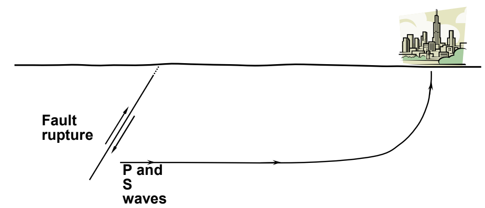
•Seismic Moment = $M_O = \mu A D$ [Units = Force x Distance]

where:

μ = modulus of rigidity (~ 3.5x10^{11} dynes/cm^2 typical)
A = fault rupture area (W x L); where typical L for big earthquake ≈ 100 km, and W ≈ 10 to 20 km
D = fault displacement (typical ≈ 2 m for big quake)

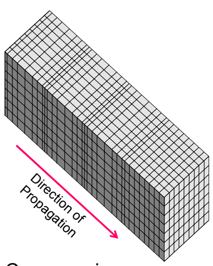
•Moment magnitude: M_W= 2/3(Log$_{10}$ M_O/1.5) 10.7

Earthquake Source and Seismic Waves

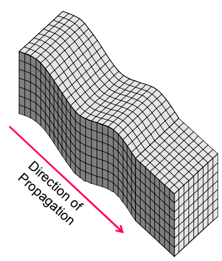


- Body waves are generated at the source and they radiate in all directions.
- As they go through layers, they are *reflected*, *refracted* and *transformed*.

Seismic Wave Forms (Body Waves)

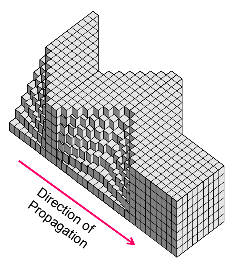


Compression eave
(P wave)

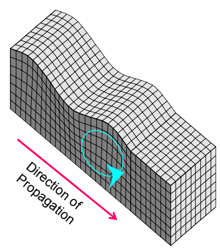


Shear wave
(S wave)

Seismic Wave Forms (Surface Waves)

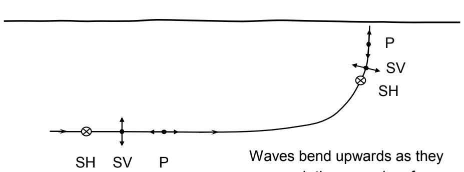


Love wave



Rayleigh wave

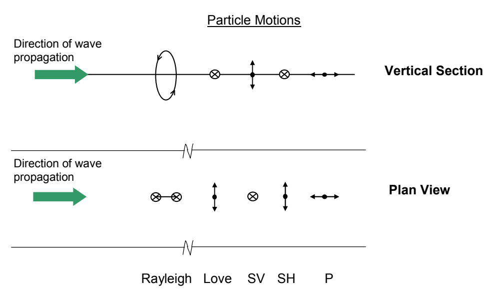
Earthquake Source and Seismic Waves



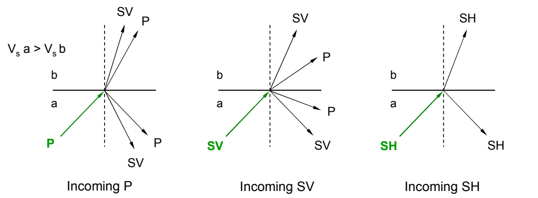
Waves bend upwards as they approach the ground surface because of less competent material near the surface – Snell's Law

P – Primary waves
SH – Horizontally polarized S waves
SV – Vertically polarized S waves

Seismic Waves

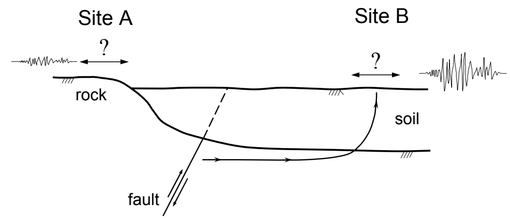


Reflection and Refraction at Boundary



- Amplitude and direction of reflected and refracted waves with respect to the incoming wave is given by Snell's Law
- Earth's crust is layered, with seismic velocities increasing with depth; therefore as waves approach ground surface wave path will get near-vertical

Ground Motion Estimation



What ground motions at Site A and B? Two steps:

1. Define earthquake scenario
2. Estimate site response and ground motions

⇒ Must be done in context of structure, type of analysis

Different Structures, Responses, Analyses, and Issues

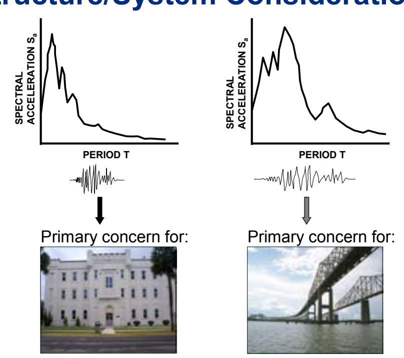
Ground Motion Estimation

- No "universal" set of ground motions for any region.
- Uncertainties are inherent to the process and will cause differences in results.
- Judgment is required, even with probability.
- Inconsistency among governing agencies.

Ground Motion Estimation

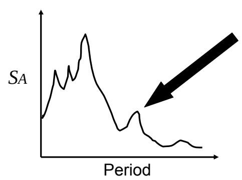
- Two analyses using same models and basic parameters can give different answers (EPRI vs. NRC/LLNL studies in 1980s).
- Where time and effort are focused during the process is function of structure/system being analyzed.
- Not possible to predict actual motion that will occur at a site; mainly concerned with capturing characteristics important to performance of project.
- Seismologist and engineers must have continuous feedback!

Structure/System Considerations



Structure/System Issues

- Place emphasis on issues important to the specific project.



Example: If this is not an important part of the spectrum, do not spend extra time and effort on issues that affect this.

- Also, think in terms of <u>system</u> performance.

Consider Performance of Entire <u>System</u>

Internal systems

Site effects, liquefaction, etc.

Structure/System Considerations

- Type of structure (building, embankment dam, etc.)
- Type and purpose of analysis – (linear elastic? time history? liquefaction?)
- Parameters that are important (pga? duration?)
- Typical process: seismologist ⇒ geotech engineer ⇒ structural engineer
- Seismologists and end user must be closely involved with continuous feedback
- Selection of earthquake scenario is most important task – (do not want precise analysis of inaccurate model)

Seismic Hazard and Seismic Risk

<u>Seismic hazard evaluation</u> ⇒ involves establishing earthquake ground motion parameters for use in evaluating a site/facility during seismic loading. By assessing the vulnerability of the site and the facility under various levels of these ground motion parameters, the *seismic risk* for the site/facility can then be evaluated.

- Seismic hazard – the *<u>expected occurrence</u>* of future seismic events
- Seismic risk – the *<u>expected consequences</u>* of future seismic events

Approaches to Seismic Hazard Analysis

Deterministic:
"The earthquake hazard for the site is a peak ground acceleration of 0.35 g resulting from an earthquake of magnitude 7 on the Woodstock Fault at a distance of 18 miles from the site. "

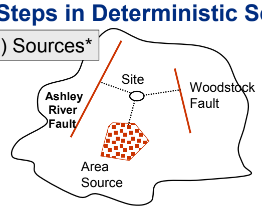
Probabilistic:
"The earthquake hazard for the site is a peak ground acceleration of 0.25 g, with a 2 percent probability of being exceeded in 50 years."

Deterministic Hazard Analysis

- Identify and characterize source zones that may produce significant ground shaking at the site
- Determine the distance from each source zone to the site
- Select the controlling earthquake scenario(s)
- Calculate the ground motions at the site using a regional attenuation relationship

Steps in Deterministic Seismic Hazard Analysis

1) Sources*

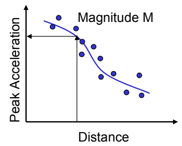


2) Controlling earthquake

Fixed Distance R*

Fixed Magnitude M*

3) Ground motion attenuation

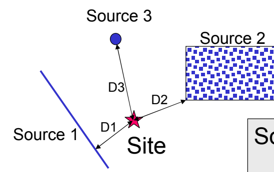


4) Hazard at site

"The earthquake hazard for the site is a pga of 0.35 g resulting from an earthquake of M7 on the Woodstock Fault at a distance of 18 miles from the site. "

*Can use probability to help define these.

Example Deterministic Analysis (Kramer, 1996)



Source	M	D (km)	PGA (g)
1	7.3	23.7	0.42
2	7.7	25.0	0.57
3	5.0	60.0	0.02

Maximum on source

Closest distance

From attenuation relationship

Advantages of Deterministic Approach

- Analysis is relatively "transparent"; effects of individual elements can be understood and judged more readily.
- Requires less expertise than probabilistic analysis.
- Anchored in reality.

Disadvantages of Deterministic Approach

- Does not consider inherent uncertainties in seismic hazard estimation (i.e., maximum magnitude, ground motion attenuation).
- Relative likelihood of events not considered (EUS vs. WUS); therefore, inconsistent levels of risk.
- Does not allow rational determination of scenario design events in many cases.
- More dependent upon analyst.

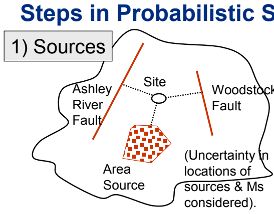
Probabilistic Seismic Hazard Analysis

⇒ Considers where, how big, and how often.

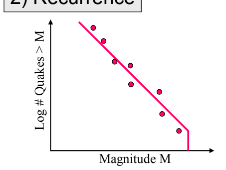
- Identify and characterize source zones that may produce significant ground shaking at the site including the spatial distribution and probability of eq's in each zone.
- Characterize the temporal distribution and probability of earthquakes in each source zone via a recurrence relationship and probability model.
- Select a regional attenuation relationship and associated uncertainty to calculate the variation of ground motion parameters with magnitude & distance.
- Calculate the hazard by integrating over magnitude and distance for each source zone.

Steps in Probabilistic Seismic Hazard Analysis

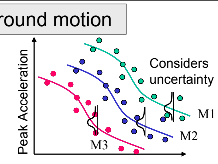
1) Sources



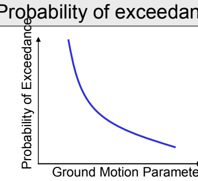
2) Recurrence



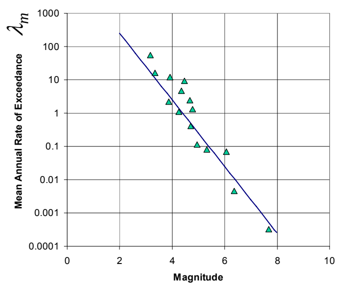
3) Ground motion



4) Probability of exceedance



Empirical Gutenberg-Richter Recurrence Relationship



$$\log \lambda_m = a - bm$$

λ_m = mean rate of recurrence (events/year)

$1/\lambda_m$ = return period

a and b to be determined from data; b is typically about 1.0

Uncertainties Included in Probabilistic Analysis

Attenuation Laws

Recurrence Relationship

Distance to Site

$$\lambda_{y*} = \sum_{i=1}^{N_S} \sum_{j=1}^{N_M} \sum_{k=1}^{N_R} v_i \, P[Y > y^* | m_j, r_k] \, P[M = m_j] \, P[R = r_k]$$

We Commonly Use Two Approaches to Predict the Likelihood of Earthquakes

- Time-independent (Poisson Model)
- Time-dependent Models

Poisson Model

- The simplest, most used model for earthquake probability.
- It is a time-independent model -- the probability that an earthquake will occur in an interval of time starting from now does not depend on when "now" is, because a Poisson process has no "memory."

Poisson Distribution (general form)

$$\mathbf{P\,(X = k) = \frac{(\lambda t)k\; e^{-(\lambda t)}}{k!}}$$

where λ = rate (events/year)

t = exposure interval

k = no. of events

Poisson Distribution (for one event)

$$P = 1 - e^{-\lambda t}$$

where λ = rate (events/year) ⇐ key!!

t = exposure interval

$1/\lambda$ = return period

Poisson Model

- Note that the probabilistic earthquake risk level can be put in the form of an earthquake return interval:

Earthquake Return Period = t/-ln(1-PE)

PE	t	Return Period
10%	50 yrs.	475
5%	50 yrs.	975
2%	50 yrs.	2475

Note that when the exponent of the equation, λt, is small, then $P \approx \lambda t$.

Example- Poisson Model

Is a 2%/50-year event the same as a 10%/250-year event?

– For 2%/50 years, we have 50/(-ln(1-0.02))= 2,475 year return period

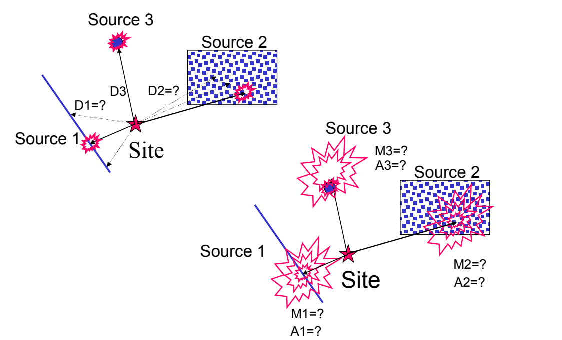
– For 10%/250 years, we have 250/(-ln(1-0.10))= 2,372 year return period

⇒ These events (probabilities) are not exactly equal, but are "equal" from design standpoint.

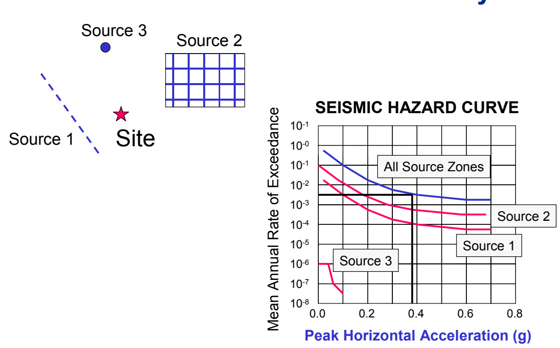
Time-Dependent Models

- Used less than simpler Poisson model
- Time-dependent means that the probability of a large earthquake is small immediately after the last, and then grows with time.
- Such models use various probability density functions to describe the time between earthquakes including Gaussian, log-normal, and Weibull distributions.

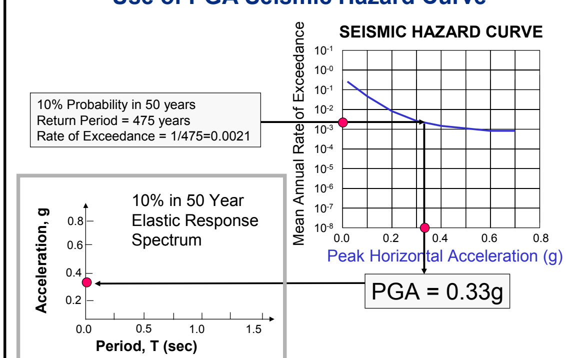
Example Probabilistic Analysis (Kramer)



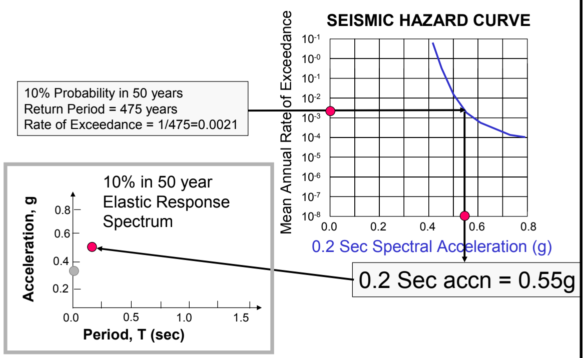
Result of Probabilistic Hazard Analysis



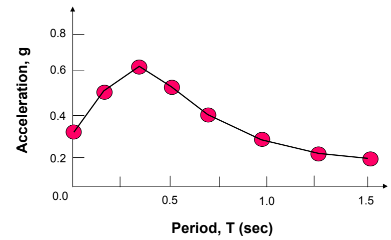
Use of PGA Seismic Hazard Curve



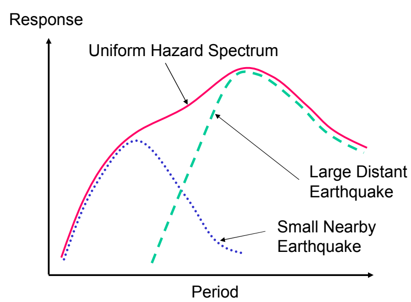
Use of 0.2 Sec. Seismic Hazard Curve



10% in 50 year Elastic Response Spectrum (UHS)



Uniform Hazard Spectrum (UHS)



Uniform Hazard Spectrum

- Developed from *probabilistic* analysis.
- Represents contributions from small local and large distant earthquakes.
- May be overly conservative for modal response spectrum analysis.
- May not be appropriate for artificial ground motion generation, especially in CEUS.

Advantages of Probabilistic Approach

- Reflects true state of knowledge and lack thereof.
- Consider inherent uncertainties in seismic hazard estimation (i.e., maximum magnitude, ground motion attenuation).
- Considers likelihood of events considered; basis for consistent levels of risk established.
- Allows more rationale comparison among many scenarios and to other hazards.
- Less dependent upon analyst.

Disadvantages of Probabilistic Approach

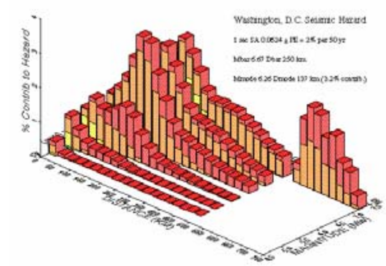
- Analyses are not transparent; the effects of individual parameters cannot be easily recognized and understood.
- “Quantitatively seductive” -- encourages use of precision that is out of proportion with the accuracy with which the input is known.
- Requires special expertise.
- May provide unrealistic scenarios (i.e., probabilistic design event could correspond to location where actual fault does not exist).
- Analyst still has big influence (methods, etc.).

Probabilistic vs. Deterministic

- Results of probabilistic and deterministic analyses are often similar in the WUS; not true for CEUS.
- Deterministic scenarios typically very difficult to define in CEUS.
- Best to use integrated or hybrid method that combines both approaches.

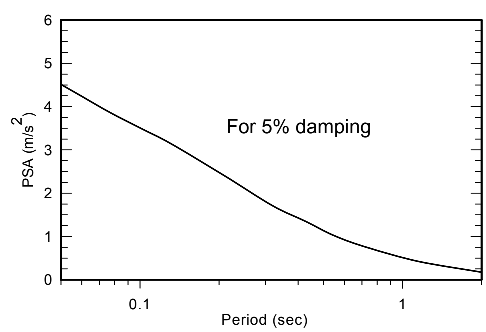
Deaggregation of the PSHA

- Each bar represents an event that exceeds a specified ground motion at 1 Hz – Washington, DC, example.; note mean and modal values.



Hazard Scenario – Example

1,950 Year Uniform Hazard Spectrum for Site



Deaggregation Plots for 1,950 Year Event (5%/100 yr)

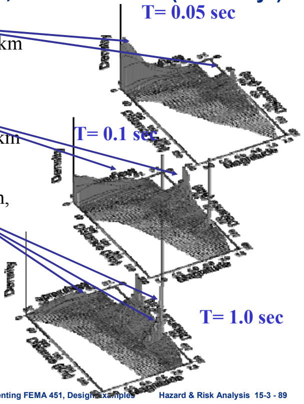
Scenarios A & B
M6@25 km & M7.5 @101 km

T= 0.05 sec

Scenarios A & B
M6@25 km & M7.5 @101 km

T= 0.1 sec

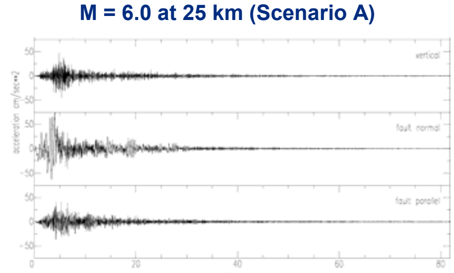
Scenarios A, B, & C?
M6@25 km, M7.5 @101 km, and M7.5@ 200 km



T= 1.0 sec

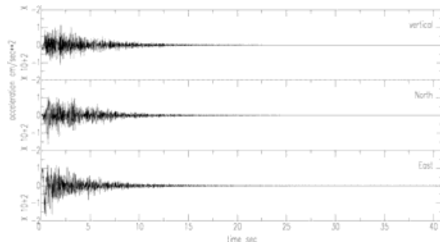
⇒ Scenarios A & B selected based on T of structure (< 1.0 sec.)

Stochastic Simulations of Ground Acceleration for M = 6.0 at 25 km (Scenario A)



From the top, vertical, North-South and East-West components

Stochastic Simulations of Ground Acceleration for M = 7.5 at 101 km (Scenario B)



Vertical, fault normal and fault parallel refer to finite fault calculations, and show 3-orthogonal components of motion, oriented with respect to source

Discussion of Selected Scenarios A & B

- What kind of analysis to be performed?
- Is duration important, or just pga?
- Basic question: "*Does it matter which event caused motions to be exceeded*?"
- Seismologist and end user should be closely linked from the beginning!!

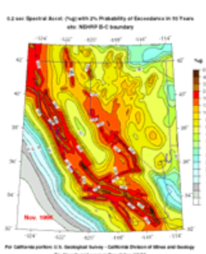
National Seismic Hazard Maps

- Developed by U.S. Geological Survey.
- Adopted (almost exactly) by building codes and reference standards (i.e., IBC2003) and, therefore, very important!!!
- Based on probability ⇒ maps show contours of maximum expected ground motion for a given level of certainty (90%, 98%, etc.) in 50 years; or, said differently, contours of ground motions that have a common given probability of exceedance, PE, in 50 years (10%, 2%, etc.).

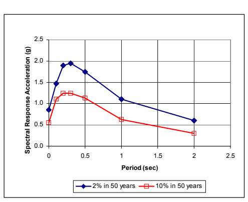
Earthquake Probability Levels

- Note that the term "2500 year earthquake" does not indicate an event that occurs once every 2,500 years!
- Rather, this term reflects a **probability**, that is, the earthquake event that has a probability of 1 in 2500 of occurring in one year.
- For instance, the "100-year flood" can actually occur several years in a row or even several times in one year (as occurred in the 1990s in Virginia).

USGS PROBABILISTIC HAZARD MAPS (2002/2003 versions most recent)*



HAZARD MAP



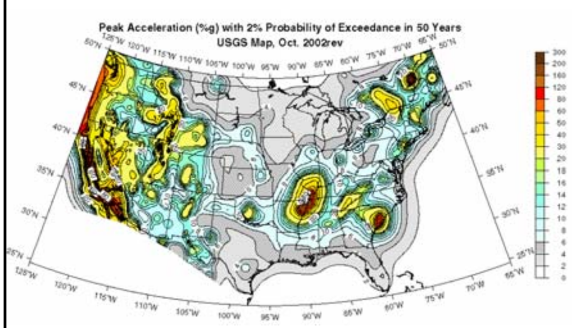
Uniform Hazard Spectra

*2002 versions revised April 2003

USGS PROBABILISTIC HAZARD MAPS (and *NEHRP Provisions* Maps)

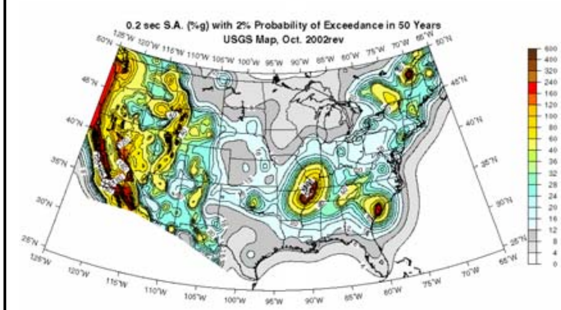
Earthquake Spectra
Theme Issue : Seismic Design Provisions and Guidelines
Volume 16, Number 1
February, 2000

USGS SEISMIC HAZARD MAP (PGA)



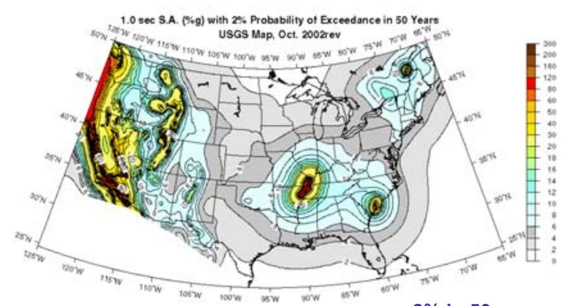
2% in 50 years

USGS SEISMIC HAZARD MAP OF US (0.2 sec)



2% in 50 years

USGS SEISMIC HAZARD MAP OF US (1.0 sec)



2% in 50 years

USGS Website: ZIP CODE Values

http://eqint.cr.usgs.gov/eq/html/zipcode.html

```
   The input zip-code is 80203. (DENVER)
   ZIP CODE                         80203
   LOCATION                         39.7310 Lat. -104.9815 Long.
   DISTANCE TO NEAREST GRID POINT  3.7898 kms
   NEAREST GRID POINT               39.7 Lat. -105.0 Long.
   Probabilistic ground motion values, in %g, at the Nearest Grid
point are:

               10%PE in 50 yr   5%PE in 50 yr   2%PE in 50 yr
      PGA        3.299764         5.207589        9.642159
   0.2 sec SA    7.728900        11.917400       19.921591
   0.3 sec SA    6.178438         9.507714       16.133711
   1.0 sec SA    2.334019         3.601994        5.879917
```

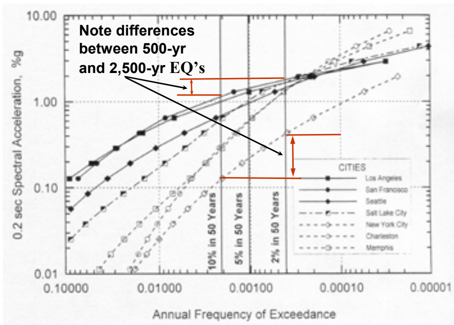

USGS Seismic Hazard Maps

- Hazard in some areas increased relative to previous maps due to recent studies.
- Maps developed for motions on B-C soil boundary (soft rock).
- Maps do not account for regional geological effects such as deep profiles of unconsolidated sediments– this is big effect in CEUS (i.e., in Charleston ~1 km thick).
- New 2002 versions of maps revised in April 2003.

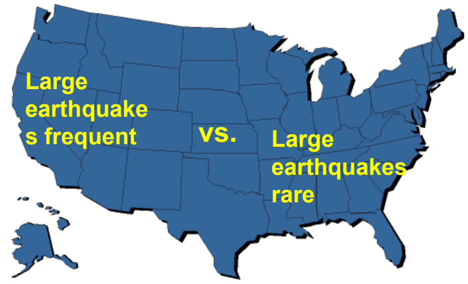
National Seismic Hazard Maps Uses:

- can illustrate relative probability of a given level of earthquake ground motion of one part of the country relative to another.
- illustrate the relative demand on structures in one region relative to another, at a given probability level.
- as per building codes, use maps as benchmark to determine the resistance required by buildings to resist damaging levels of ground motion.
- with judgment and sometimes special procedures, use maps to determine the input ground motions for geotechnical earthquake analyses (liquefaction,etc.)

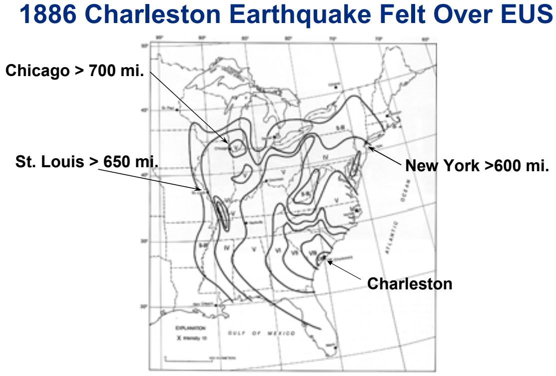
USGS Seismic Hazard Curves for Various Cities



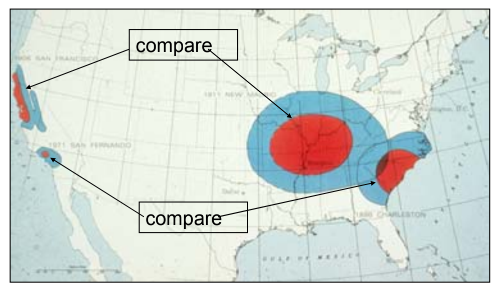
How Does CEUS and WUS Seismic Risk Compare?



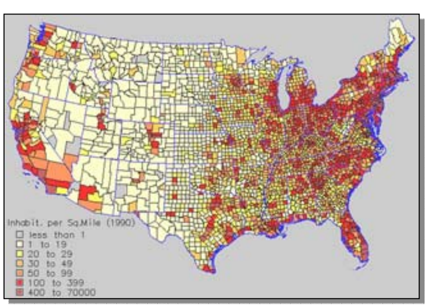
1886 Charleston Earthquake Felt Over EUS!



WUS vs. CEUS Attenuation



US Population Density



California Seismicity Well Understood

Seismicity relatively well understood

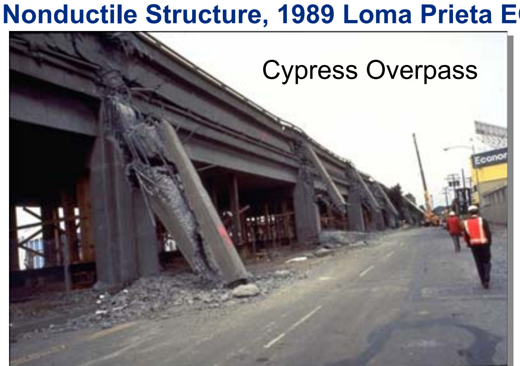
Seismically Weak Infrastructure in CEUS

WUS and CEUS Risk Comparison

- CEUS has potential for recurring large earthquakes
- Attenuation lower in CEUS
- Weak structures not “weeded out” in CEUS
- “Adolescent” seismic practice in CEUS
- “Human inertia” in CEUS
- Much more uncertainty in CEUS

- Bottom line ⇒ *seismic <u>risk</u> in CEUS and WUS is comparable!*

Example of Inadequately Reinforced, Nonductile Structure, 1989 Loma Prieta EQ



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FEMA Instructional Material Complementing FEMA 451, Design Examples Hazard & Risk Analysis 15-3 - 116

FEMA Instructional Material Complementing FEMA 451, Design Examples Hazard & Risk Analysis 15-3 - 117

FEMA Instructional Material Complementing FEMA 451, Design Examples Hazard & Risk Analysis 15-3 - 118

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This Type of Non-Ductile Infrastructure is Common in CEUS!

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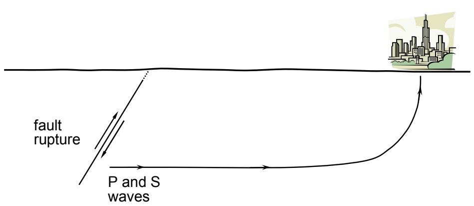
WUS and CEUS Risk Comparison Summary

- CEUS has potential for recurring large EQs
- Attenuation lower in CEUS
- Abundance of weak, non-ductile structures in CEUS; weakest not “weeded out”
- Immature seismic practice in CEUS
- “Human inertia” in CEUS; little awareness
- Much more uncertainty in CEUS
- Areas with poor soils in CEUS
- Bottom line ⇒ ***seismic <u>risk</u> in CEUS and WUS is comparable!***

Issues To Think About

- Good analogy ⇒ Kobe is to Tokyo, as CEUS is to the WUS
- Kobe M6.9 (> $120 billion losses); weaker infrastructure, poor soil conditions
- Remember ⇒ most expensive US natural disaster (Northridge, EQ ~$30 billion) was moderate earthquake on minor fault on fringe of Los Angeles

Estimation of Ground Motions



Estimation of Ground Motions

We typically need one or more of these:

- Peak ground motion parameters (peak ground accelerations, peak velocities); or, duration.
- Spectral parameters (response spectra, Fourier spectra, uniform hazard spectra)
- Time history of acceleration, velocity, etc. ⇒ needed for advanced and/or specialized analyses.
- We typically need these parameters for ground surface

Ground Motions at a Site Are Related To:

- Source conditions– amount of energy released,nature of fault rupture,etc.
- Path effects – anelastic attenuation, geometrical spreading,etc.
- Site effects – site response, soil amplification, etc.

Source Conditions Include:

- Stress drop
- Source depth
- Size of the rupture area
- Slip distribution (amount and distribution of static displacement on the fault plane)
- Rise time (time for the fault slip to complete at a given point on the fault plane)
- Type of faulting
- Rupture directivity

Transmission Path Includes:

- Crustal structure
- Shear-wave velocity (or Q) and damping characteristics of the crustal rock

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Site Conditions Include:

- Rock properties beneath the site to depths of up to about 2 km (hard crystalline rock)
- Local soil conditions at the site to depths of up to several hundred feet (typically)
- Topography of the site

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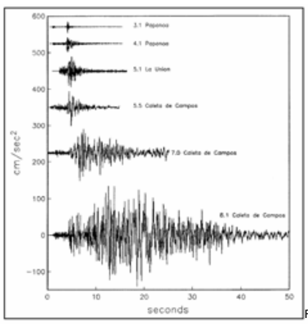
Effects of Magnitude



From USACE, 2000

Figure 3-1. An example of accelerograms recorded in 1985 and 1986 on the Guerrero accelerograph array (Anderson and Quaas (1988), courtesy of Earthquake Engineering Research Institute, Oakland, CA)

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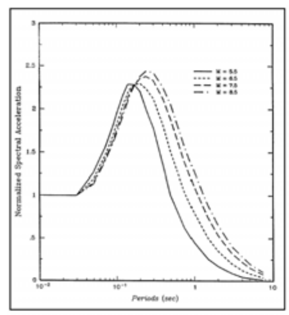
Effects of Magnitude



From USACE, 2000

Figure 3-3. Effect of magnitude *M* on response spectral shape of rock motions based on attenuation relationships of Sadigh et al. (1993), 30-km distance from source to site, 5 percent damping

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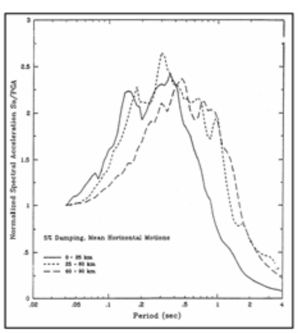
Effects of Distance



From USACE, 2000

Figure 3-4. Variation of spectral shape with distance for rock recordings of the October 17, 1989, Loma Prieta earthquake

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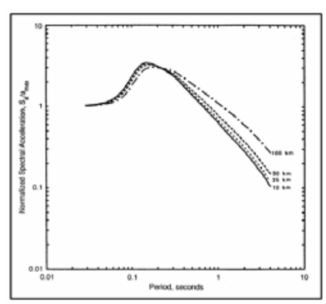
Effects of Distance



From USACE, 2000

Figure 3-5. Effect of distance on response spectral shapes for a moment magnitude M_w 6.5 earthquake using western North American parameters (Silva and Green 1989, courtesy of Earthquake Engineering Research Institute, Oakland, CA)

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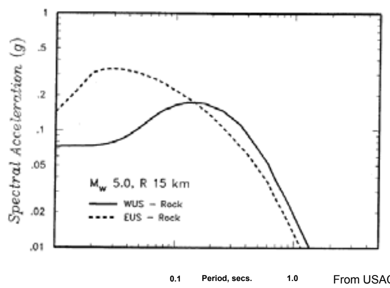
Regional Effects



From USACE, 2000

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Effect of Local Site Conditions

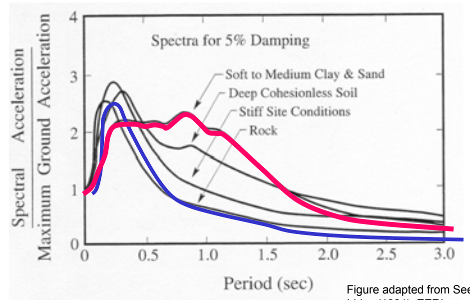

Figure adapted from Seed and Idriss (1984); EERI.

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Special Near-source Effects

"Near-source" can be interpreted differently. For many engineering applications, a zone within about 20 km of the fault rupture is considered near-source. Other cases near-source is considered within a distance roughly equal to the ruptured length of the fault; 20 to 60 km typical

Near-source effects:

- Directiviity
- Fling
- Radiation pattern

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Important Near-Fault Effects

Two Causes of large velocity pulses:

- Directivity
- Fling

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Causes of Velocity Pulses

Directivity:

- Related to the direction of the rupture front
  - Forward directivity: rupture toward the site (site away from the epicenter)
  - Backward directivity: rupture away from the site (site near the epicenter)

Fling:

- Related to the permanent tectonic deformation at the site

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Velocity Pulses

- Directivity
  - Two-sided velocity pulse due to constructive interference of SH waves from generated from parts of the rupture located between the site and epicenter; affects fault-normal component
  - Occurs at sites located close to the fault but away from the epicenter
- Fling
  - One-sided velocity pulse due to tectonic deformation; affects fault-parallel component
  - Occurs at sites located near the fault rupture independent of the epicenter location

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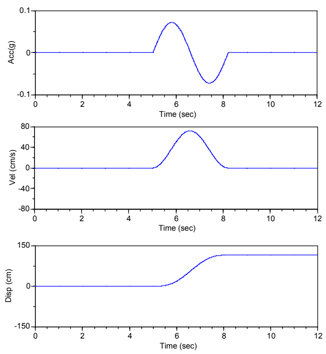
Preliminary Model for Fling



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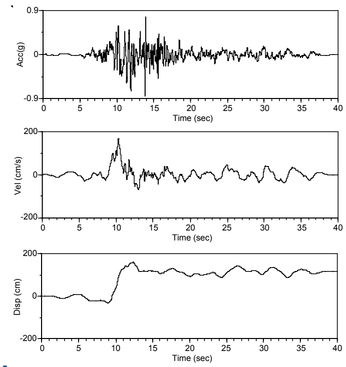
Vibratory Ground Motion + Fling



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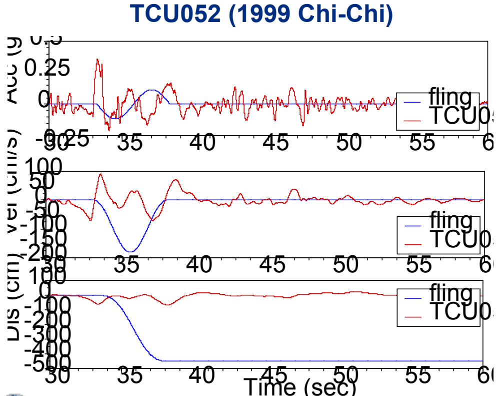
TCU052 (1999 Chi-Chi)



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Effects of Fling

- Not currently known which types of structures are sensitive to fling ground motions.
- Preliminary results indicate some long-span structure may be sensitive to fling.
- Need to evaluate various types of structures to ground motions with and without fling to determine the effect.

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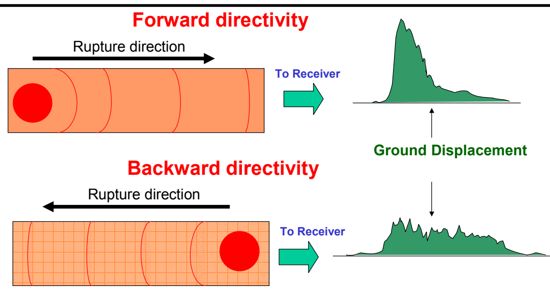

The areas under the far-field displacement pulses are equal, but the amplitudes and durations differ. This has major effects on the ground velocity and acceleration.

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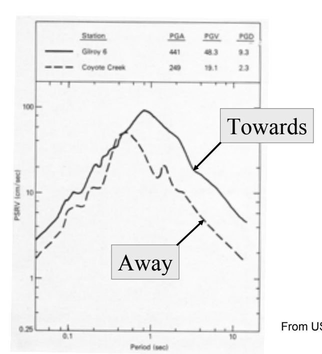
Effect of Directivity on Response Spectra



From USACE, 2000

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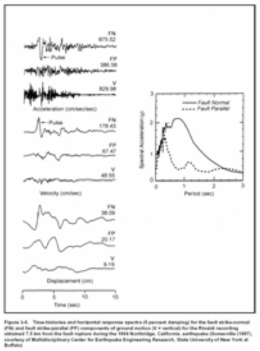
Effect of Directivity

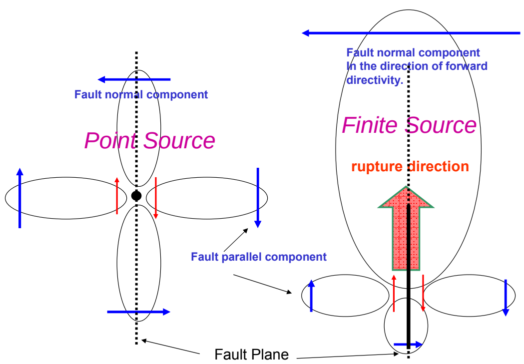


From USACE, 2000

Effects of Fling and Directivity

- Directivity can cause amplification of motions for sites close to the fault rupture.
- Unclear as to engineering significance of fling.
- Current attenuation relations do not include these effects.

SH Radiation Pattern for Vertical Strike-slip



Other Important Effects

- Also, vertical motions tend to be higher than 2/3 maximum horizontal motions when near-source.
- Subduction zone EQs vs. shallow EQs
- Topographical effects (especially basins).
- Surface waves may be important for certain long-span structures (relative motion among supports).
- Others…

Three Classes of Methods for Ground Motion Estimates

- Generalized, simplified (i.e., IBC2003)⇐
- Site-specific, simplified (i.e., attenuation curves, site amplification factors)
- Site-specific, rigorous (time history analysis)

Generalized, Simplified (i.e., IBC 2003)

- Simple to use.
- Based on probabilistic maps.
- Does not account for regional geological effects (maps assume standard depth for B-C boundary and profile layering) ⇒ in WUS, B-C boundary is shallow bedrock, but in some CEUS areas the B-C boundary is deep as 1 km.
- Accounts for local site effects in general manner– cannot handle special site conditions.
- Not well-suited to many geotechnical analyses (no magnitude, UHS approach, etc.).

IBC 2003 - Overview

- Developed from a combination of three legacy model codes (UBC, BOCA, & SBC).
- Based largely on FEMA 368 and 369, *NEHRP Recommended Provisions* and *Commentary.*
- Adopted in 45 states (as of July 2004) and by the DoD.
- Incorporates most recent (2002/2003) USGS seismic hazard maps; USGS map values capped in some areas by IBC 2003.

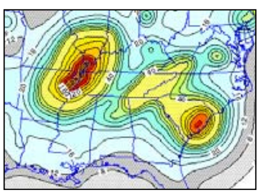
IBC 2003 – General Procedure

- Maximum Considered Earthquake (MCE) based on 2002/2003 USGS probabilistic hazard maps (deterministic limits used in high seismicity areas – here hazard can be driven by tails of distributions).
- Maps provide and spectral accelerations for T = 0.2 sec (S_s), and T = 1.0 sec (S_1) for B-C boundary.
- Local soil conditions considered using site coefficients (F_a and F_v)
- Develop design spectrum using S and F values

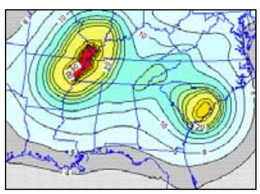
IBC 2003 – General Procedure

- Determine S_s and S_1 from the maps

S_s (0.2 sec) map



S_1 (1.0 sec) map



IBC 2003 – General Procedure

- Determine site class based on top 30 m:

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, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	Not applicable	Not applicable
B	Rock	$2{,}500 < \bar{v}_s \le 5{,}000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \le 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \ge 2{,}000$
D	Stiff soil profile	$600 \le \bar{v}_s \le 1{,}200$	$15 \le \bar{N} \le 50$	$1{,}000 \le \bar{s}_u \le 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$; 2. Moisture content $w \ge 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ 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 m², 1 pound per square foot = 0.0479 kPa.

IBC 2003 – General Procedure

- Determine F_a & F_v values from S_s, S_1 and site class:

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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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

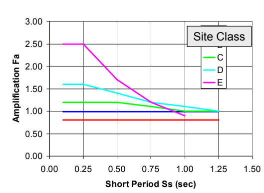
a. Use straight line interpolation for intermediate values of mapped spectral acceleration at short period, S_S.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

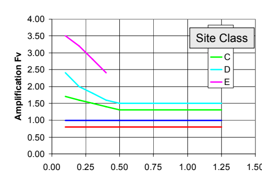
TABLE 1615.1.2(2)
VALUES OF SITE COEFFICIENT F_V AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD (S_1)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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

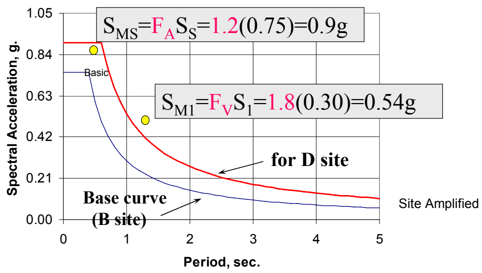
a. Use straight line interpolation for intermediate values of mapped spectral acceleration at 1-second period, S_1.
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

NEHRP Provisions Site Amplification for Site Classes A through E





Example: 2% in 50 Year Spectrum Modified for Site Class D (5% Damping)



IBC 2003 - General Procedure

- Adjust MCE values of S_s and S_1 for local site effects:

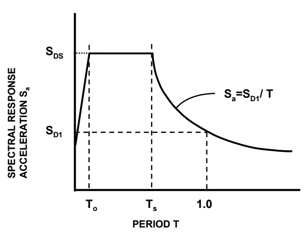
  $S_{MS} = F_a \bullet S_s \quad S_{M1} = F_v \bullet S_1$

- Calculate the spectral design values S_{DS} and S_{D1}:

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

IBC2003 – General Procedure

- From S_{DS} and S_{MS}, develop the design response spectrum



DESIGN RESPONSE SPECTRUM

Scaling of Spectra by 2/3 for "Margin of Performance"

- Design with current 2%/50-yr. maps but scale by 2/3.
- Buildings designed according to current procedures assumed to have margin of collapse of 1.5.
- Judgment of "lower bound" margin of collapse given by current design procedures.
- Results in 2/3 x 1.5 = 1.0 deterministic earthquake (where applicable).
- 2/3 (2500-yr. EQ) = 500-year motions in WUS, but 2/3 (2500-yr. EQ) ≈ 1600-year motions in EUS
- 2/3 factor not related to geotechnical performance!

Three Classes of Methods for Ground Motion Estimates

- Generalized, simplified (i.e., IBC 2003)
- Site-specific, simplified (i.e., attenuation curves, site amplification factors) ⇐
- Site-specific, rigorous (time history analysis)

Site-Specific, Simplified

- Relatively simple (chart-based procedures).
- Based on probabilistic motions or deterministic scenarios.
- Can account for regional geological effects (within 2 km of surface; USGS maps assume standard depth for B-C boundary and hard rock).
- Accounts for local site (within few hundred feet of surface) effects in simplified, but more specific manner.
- Better-suited to many geotechnical analyses.

Site-Specific, Simplified: Comments

- Note IBC 2003 limits site-specific "benefit" (in terms of reduced design) motions to 20% for A-E sites.
- Site-specific analysis in some CEUS area less than probabilistic maps values; opposite may be true in WUS.

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Site-Specific Simplified Procedures

Typical deterministic scenario:

1. Knowing fault location and earthquake magnitude, estimate ground motion parameter (i.e, pga or spectral values) for hard rock from attenuation relationships.
2. If appropriate, correct for regional geological conditions such as deep unconsolidated sediments (Vs >700m/s and typically within 2 km of surface)
3. Modify motions for near-surface soils (V_s < 700 m/s and within few hundred of surface)*

*covered in detail in a following lecture.

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1. Estimating Motions on Hard Rock

- Typically use region-specific attenuation curve (but can use probabilistic maps also).
- Curves developed from empirical data from recorded motions in most regions.
- Curves in CEUS developed from few small EQs, plus stochastic simulations using methods developed in WUS but with CEUS geological parameters (Q, stress drop, etc.).
- Most curves provide PGA, PGV, and spectral values.

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Ground Motion Attenuation Basic Empirical Relationships

$$\ln \hat{Y} = \ln b_1 + f_1(M) + \ln f_2(R) + \ln f_3(M,R) + \ln f_4(P_i) + \ln \varepsilon$$

$\hat{Y}$ Ground Motion Parameter (e.g. PGA)

b_1 Scaling factor

$f_1(M)$ Function of Magnitude

$f_2(R)$ Function of Distance

$f_3(M,R)$ Function of Magnitude and Distance

$f_4(P_i)$ Other Variables

ε Error Term

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Ground Motion Attenuation Relationships for Different Conditions

- Central and Eastern US
- Subduction Zone Earthquakes
- Shallow Crustal Earthquakes
- Near-Source Attenuation
- Extensional Tectonic Regions
- Many Others
- Most are for hard rock, some for "soil"

May be developed for any desired quantity (PGA, PGV, Spectral Response)

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Ground Motion Attenuation Relationships

Seismological Research Letters
Volume 68, Number 1
January/February, 1997

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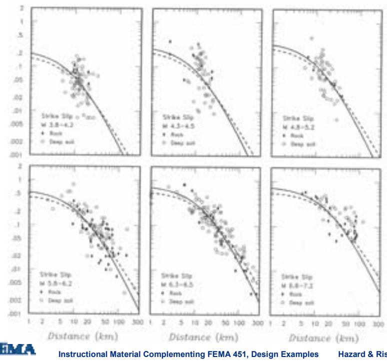
Attenuation Relation for Shallow Crustal Earthquakes
(Sadigh, Chang, Egan, Makdisi, and Youngs; for Rock and "Soil")



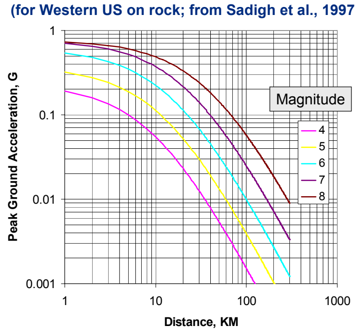
Attenuation Relation for Shallow Crustal Earthquakes
(Sadigh, Chang, Egan, Makdisi, and Youngs)

$$\ln(y) = C_1 + C_2 M + C_3(8.5 - M) + C_4 \ln(r_{rup} + \exp(C_5 + C_6 M)) + C_7 (r_{rup} + 2)$$

T	C_1	C_2	C_3	C_4	C_5	C_6	C_7
PGA	-0.624	1.000	0.000	-2.100	1.296	0.250	0.000
0.07	0.110	1.000	0.006	-2.128	1.296	0.250	-0.082
0.1	0.275	1.000	0.006	-2.148	1.296	0.250	-0.041
0.2	0.153	1.000	-0.004	-2.080	1.296	0.250	0.000
0.3	-0.057	1.000	-0.017	-2.028	1.296	0.250	0.000
0.4	-0.298	1.000	-0.028	-1.990	1.296	0.250	0.000
0.5	-0.588	1.000	-0.040	-1.945	1.296	0.250	0.000
0.75	-1.208	1.000	-0.050	-1.865	1.296	0.250	0.000
1	-1.705	1.000	-0.055	-1.800	1.296	0.250	0.000
1.5	-2.407	1.000	-0.065	-1.725	1.296	0.250	0.000
2	-2.945	1.000	-0.070	-1.670	1.296	0.250	0.000
3	-3.700	1.000	-0.080	-1.610	1.296	0.250	0.000
4	-4.230	1.000	-0.100	-1.570	1.296	0.250	0.000

Table for Magnitude <= 6.5

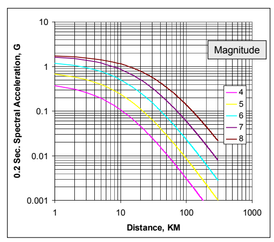
Attenuation Relation for Shallow Crustal Earthquakes
(for Western US on rock; from Sadigh et al., 1997)



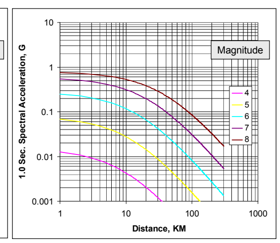
- typically use mean or 84th percentile (+1σ) values

Attenuation Relation for Shallow Crustal Earthquakes
(Western US, rock conditions; Sadigh et al., 1997)

0.2 Second Acceleration



1.0 Second Acceleration

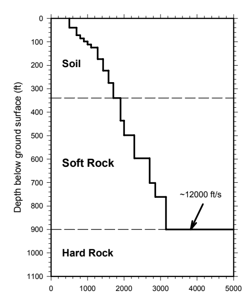


2. Adjustment for Regional Geology

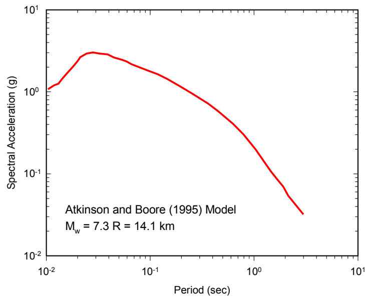
In some regions, the presence of deep unconsolidated sediments ("soil" to geologists, "soft rock" to engineers; Vs ≈ 700 m/s) require correction of hard rock values for these conditions. Can use:

- Regional correction curve to adjust hard rock curve; or,
- A "soil" attenuation curve in Step 1 that already includes the effect of the "soil" as soil attenuation curve. In this case, the correction here for Step 2 is not required.

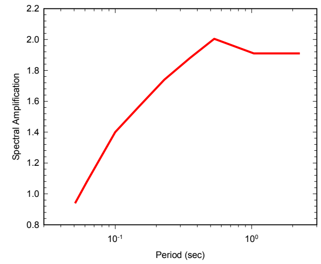
Example: CEUS Geological Condition Requiring Adjustment:



EUS Hard Rock Response Spectrum (adjust with regional soil amplification curve)



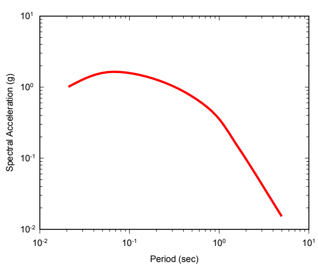
Regional "Soil" Amplification Factors (use to adjust hard rock curve)



Atkinson and Boore (1997)

- Amplification with respect to hard rock
- Deep soil profile representative of Site Class C soil profile

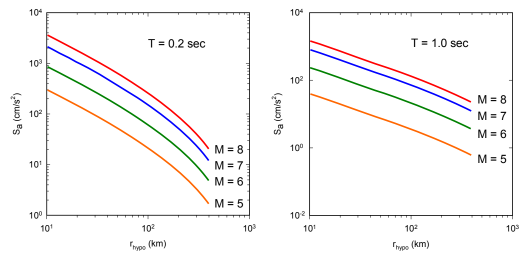
Adjusted Curve for Regional Geology



EPRI (1993)

"Soil" Attenuation Relationships

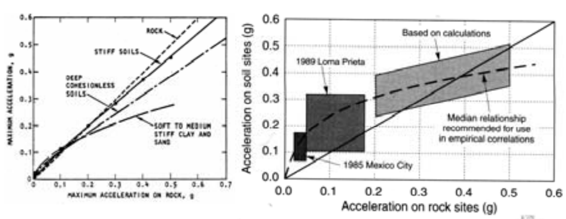
- Can use these directly where appropriate and available in lieu of two-step procedure:



Boore and Joyner (1991)

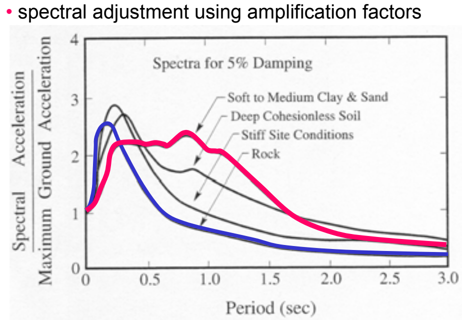
3. Adjustment for near-surface soil conditions (within ~30 m depth)

- pga adjustment using amplification factors



3. Adjustment for local soil conditions

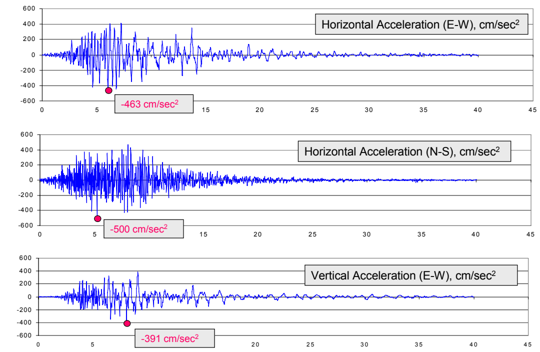
- spectral adjustment using amplification factors



Three Classes of Methods for Ground Motion Estimates

- Generalized, simplified (i.e., IBC 2003)
- Site-specific, simplified (i.e., attenuation curves, site amplification factors)
- Site-specific, rigorous (time history analysis) ⇐

Typical Earthquake Acceleration Time History Set



Loma Prieta Earthquake

Time History analyses

- Allows best possible analysis (usually)
- Increasing in usage
- Time histories can be obtained from:
  - Databases of recorded motions such as
    - National and state data catalogs (NSMDS)
    - USGS web page
    - other sources (i.e., NONLIN)
  - By developing the motions using
    - modified recorded motions
    - synthetic motions

Obtaining Time Histories

Conditions for which there are few records available:

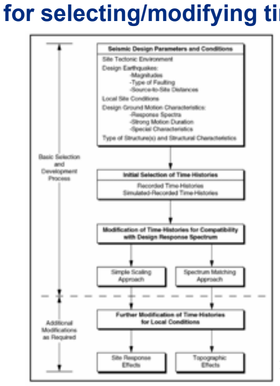
- Moderate to large earthquakes in CEUS
- Large-magnitude (8+) shallow crustal events
- Near-source, large-magnitude (7.5+) events

Time History Analysis

- Objective: develop a set or sets of time-histories, usually acceleration time histories, that are representative of site ground motions for the design earthquake(s)* and that are appropriate for the type of analyses planned.
- Will not be able to predict actual motions, rather interested in *representing characteristics most important for design.*

* Discussed earlier. The design earthquake can be from deterministic or probabilistic analysis; but, if probabilistic, the uniform hazard spectrum should probably not be used as the target spectrum. Rather, deterministic scenarios should be developed from deaggregation of the PSHA.

Process for selecting/modifying time histories:



From: (USACE, 2000)

How many time histories are needed for a typical analysis?

- ***For linear analysis, typically 2 or 3***
  (linear system is more influenced by frequency-domain aspects of motion)
- ***For non-linear analysis, typically 4 or 5***
  (non-linear systems more influenced by time-domain aspects of record- shape and sequences of pulses, etc.)

1. Selecting time histories – key factors:

Most logical procedure is to select available time histories from databases that are reasonably consistent with the design parameters and conditions. Factors to consider include in selection:

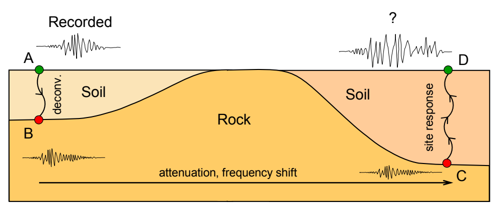
- tectonic environment (subduction, shallow crustal, intraplate,etc.)
- earthquake magnitude and fault type
- distance from recording site to fault rupture – want distances within a factor of 2

1. Selecting time histories – key factors:

- site conditions at recording site (want similar)
- response spectra of motions (want similar shape and level to design spectra; also, want to achieve reasonable match by scaling by factor $\leq$ 2.0 (especially if scaling record motions to higher level)
- duration of strong shaking
- if site is near-field (within about 15 km) then acceleration record should contain strong motion pulse similar to that caused directivity, etc.

2. Modifying and scaling time histories:

What is the motion at D?

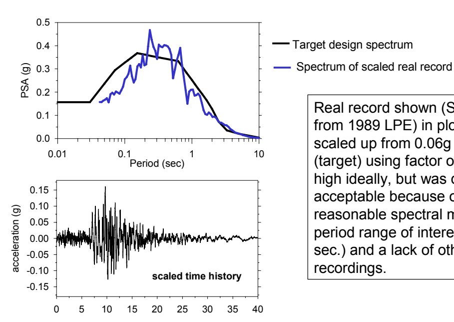


2. Modifying and scaling time histories:

(a) Simple scaling – scale motions by single factor to match target spectrum; again limiting the scaling factor to 2.0.

- The required degree–of-fit to target spectrum is project-dependent, but typically want suite of candidate spectra to have average visual fit to target. More important to have conservative fit in period range of interest.

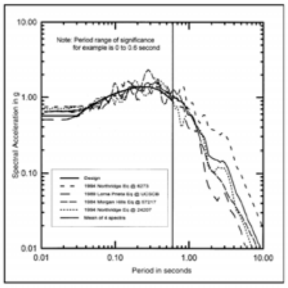
Simple Scaling to Match Design (Target) Spectrum



Real record shown (Sierra point from 1989 LPE) in plot was scaled up from 0.06g to 0.16g (target) using factor of 2.8-- too high ideally, but was deemed acceptable because of reasonable spectral match in period range of interest (~ 1 sec.) and a lack of other recordings.

Degree-of-fit for Suite of Motions:

- Required degree of fit is project dependent and often mandated



From USACE, 2000

2. Modifying and scaling time histories –

(b) Spectrum matching– adjustments made in either time domain or frequency domain to change characteristics of the motions:

- Want to maintain time-domain character of recorded motion
- Best to begin with candidate motion that has spectral shape similar to target spectrum
- Best to first scale motion to approximate level of target spectrum before modification

Spectrum Matching Methods

(i) Time-Domain Approach: (Lilhanand and Tseng, 1988; Abrahamson, 1992).

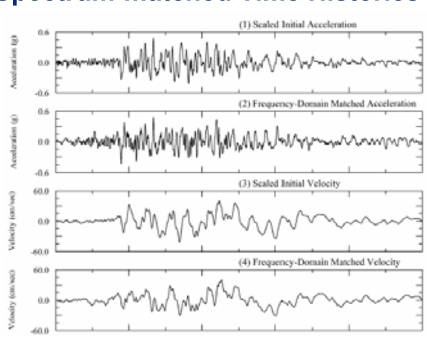
- Matching accomplished by adding (or subtracting) finite-duration wavelets to (or from) the initial time-history.
- Normally provides a close fit to the target. Best to being with candidate motion has spectral shape similar to target spectrum.
- Best to first scale motion to approximate level of target spectrum before modification.

Spectrum Matching Methods

(ii) Frequency-Domain Approach: (Gasparini and Vanmarcke 1976; Silva and Lee 1987; Bolt and Gregor 1993).

- Adjusts only the Fourier amplitudes while the Fourier phases are kept unchanged.
- Procedure equivalent to adding or subtracting sinusoids (with the Fourier phases of the initial time-history) in the time domain.
- Does not always provide as close a fit as time-domain approach.

Spectrum-matched Time Histories



From USACE, 2000

Spectra of spectrum-matched time histories:

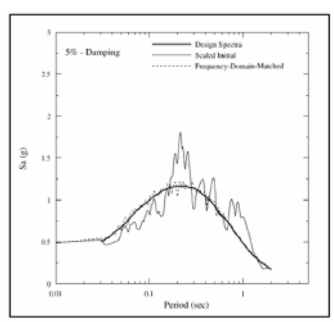


Figure 5-9. Comparisons of response spectra from the scaled 1971 San Fernando

From USACE, 2000

Other corrections…

- Ensure records are instrument and base-line corrected, etc.

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3. Modification for local site conditions

- Dynamic site response analysis is best approach (discussed in following lecture).

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Real vs. Synthetic Time Histories

- What is considered a "real"record? (i.e., how much modification is allowed?)
- Un-scaled record motion vs. scaled recorded motion vs. synthetic.
- Synthetic motions developed using Fourier phase spectra from real earthquake probably "real" in most important ways.

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Synthetic Time Histories – Pros and Cons

- One main concern: Is true character of real motion present?
- One main advantage: Can develop motions to match regional and site conditions (i.e., motion recorded on outcrops actually have surface wave energy included but we commonly input this to base).
  - there are many data gaps in database of motions (no strong motions for CEUS)
  - certainly better to have reasonable region-specific synthetic motion than inappropriate real motion

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Developing Synthetic Motions

- Process should be performed by expert, typically seismologist.
- Seismologists typically develop a suite of time histories for hard rock or B-C (soft rock) boundary.
- Geotechnical engineers typically generate top-of-profile motions using site response analysis.

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Synthetic Ground Motion Development

The computational model for generating synthetic seismograms consists of:

- The seismic source process;
- The process of seismic wave propagation from the source region to the design site; and
- Shallow site response (site response is discussed later).

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Synthetic Ground Motion Development

Source Parameters Required

- Rupture velocity, rupture initiation point, and slip-time functions over the ruptured area are the primary source parameters needed.

Propagation (Path) Parameters Required

- Average propagation usually developed with Green's functions -- requires knowledge of the crustal parameters such as the P and S-wave velocities, density, and damping factor (or seismic Q factor, where Q = 0.5/damping ratio).

Synthetic Ground Motion Generation

- To model complexity of seismogram, randomness (stochastic model) is often introduced, either in the source process or in the wave propagation.
  - very erratic, irregular high-frequency waves from rupture process usually characterized as a "stochastic" process that must be modeled with randomness
  - deterministic process often used for low-frequency portion of motion
- Hybrid models combine deterministic with random process.

Synthetic Ground Motion Generation

- With fault slip model and Green's functions, ground motions are computed using the representation theorem (deconvolution process); see Aki and Richards 1980; Hartzell, Frazier, and Brune 1978.
- Simulation procedure simply sums a suite of Green's functions lagged in time (delay caused by the rupture propagation plus the time needed for the seismic waves to travel from the corresponding point source to the site).

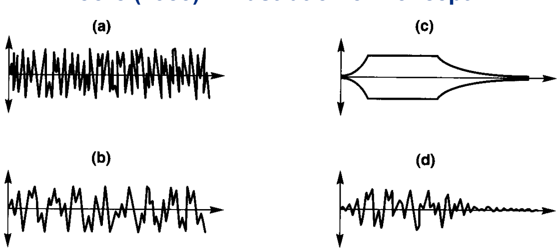
⇒ Green's Function is heart of the process.

Synthetic Ground Motion Methods

(1) Boore (1983): developed Band-Limited-White-Noise model for stochastic simulation of high-frequency ground motions.

- This simulation procedure does not use stochastic slip model.
- Procedure generates random white noise, multiplies it by a window function appropriate for the expected source duration, and then filters the windowed white noise to obtain a time-history having a band-limited Fourier amplitude spectrum specified by the ω^2-source Brune (1970) model.
- Incorporates wave propagation effects of a homogeneous crust with $1/R$ geometrical attenuation.

Boore (1983) – Illustration of Concept*:



Boore (1983): Example of time-domain generation of synthetic time history: (a) time history of white noise is filtered in the time domain to produce (b) time history of filtered white noise. Filtered white noise is multiplied by envelope function in (c) to produce the artificial ground motion shown in (d).

***Figure adapted from Kramer (1996)**

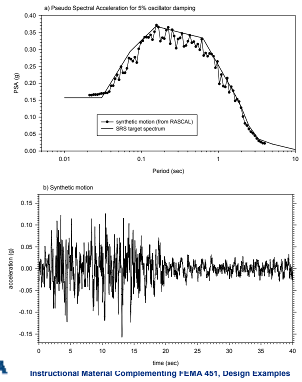
Synthetic Ground Motion Methods

(2) Silva and Lee (1987): method uses formulation for the Fourier amplitude spectrum similar to Boore, but the phase spectrum from a natural time-history to generate the synthetic time-history.

(3) Publicly available computer codes: Some public domain simulation codes are: RASCAL (Silva and Lee 1987) and SMSIM (Boore 1996).

⇒ The above methods (1 through 3) are well-established.

Example: Synthetic Motion development with RASCAL



Source Modeling for Synthetic Motions

1) Point source models (i.e., Brune source spectrum):
- Simple model where the source is represented by a point.
- Assumes "stationary" signal; provides average component.
- Need Magnitude, stress drop $\Delta\sigma$, density, crust modulus.

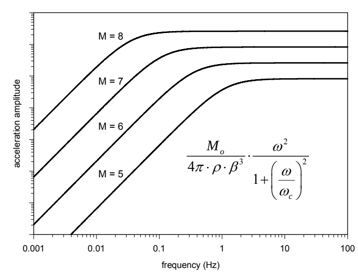
2) Finite fault models – modeling the actual rupture:
- Fault is divided into segments and each segment ruptures after another simulating energy release.
- Energy radiation from each segment is modeled using Green's Function.
- Motion from all segments added up to generate motion at a point from the fault.
- It models directivity, radiation, and non-stationarity.

1) Point Source Modeling – Brune Model

$$\text{Source}\ (\omega) = \frac{M_o}{4\pi\cdot\rho\cdot\beta^3}\cdot\frac{\omega^2}{1+\left(\frac{\omega}{\omega_c}\right)^2}$$

M_o : seismic moment
ρ : mass density of earth's crust
β : shear wave velocity of earth's crust
ω_c : corner frequency ($2\pi f_c$) $\quad f_c = 4.91\times10^6\,\beta\left(\frac{\Delta\sigma}{M_o}\right)^{1/3}$

Modeling Source – Brune Model



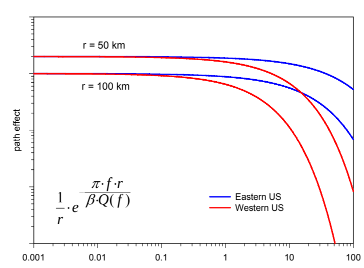
- Source spectrum for different magnitude earthquakes
- Corner frequency (ω_c) decreases for larger magnitudes (duration α $1/\omega_c$)

Modeling Path Effects

$$\text{Path}\ (\omega) = \frac{1}{r}\cdot e^{-\frac{\pi\cdot f\cdot r}{\beta\cdot Q(f)}}$$

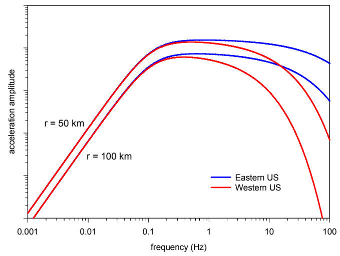
r : distance to the source
f : frequency
β : shear wave velocity of earth's crust
Q : quality factor (1/2D, D = damping ratio)
Q = 200 $f^{0.2}$ – Western US
Q = 680 $f^{0.34}$ – Eastern US

Modeling Path Effects



- Frequency dependent attenuation
- Smaller attenuation for Eastern US

Combined Source and Path Effects



- Wider band spectrum for Eastern US
- Larger high frequency components for Eastern US even at large distances

2) Finite Fault Model

Total far-field S displacement is constructed by summation of displacement pulses for a large number of sub-faults, randomly distributed on the fault plane.

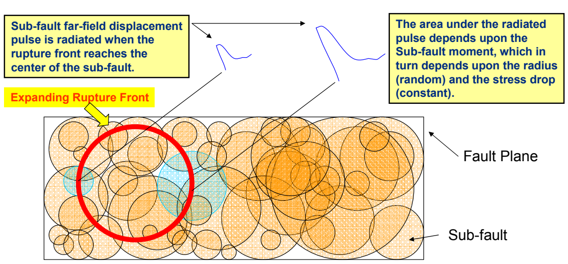
- Approach taken is similar to that described originally by Zeng et al., *Geophysical Research Letters*, 1994.
- Can model some near-field effects, provides 3 components

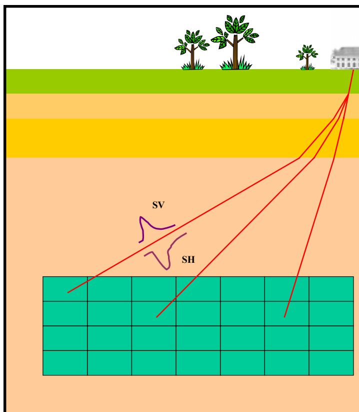
Important Input Parameters:

1) Total Seismic Moment
2) Fault dimensions
3) Maximum and minimum (circular) sub-fault radii
4) Sub-fault stress drop (not necessarily the static stress drop)
5) Rupture velocity (spatially constant, etc.)

The sub-faults are allowed to overlap spatially. Superposition of the radiated pulses from the sub-faults models the spatial and temporal variability of fault slip velocity.

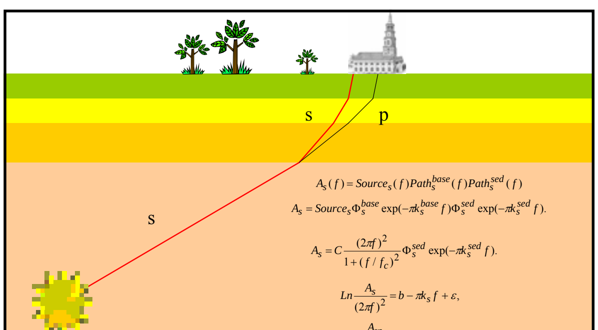
To Receiver





Path effects are calculated using a reduced number of SV and SH Greens functions corresponding to the center points of a number of fault grid elements.

These are combined with the summation of source pulses from sub-faults lying within each grid element.



$$A_s(f) = Source_s(f) Path_s^{base}(f) Path_s^{sed}(f)$$

$$A_s = Source_s \Phi_s^{base} \exp(-\pi k_s^{base} f) \Phi_s^{sed} \exp(-\pi k_s^{sed} f).$$

$$A_s = C \frac{(2\pi f)^2}{1 + (f/f_c)^2} \Phi_s^{sed} \exp(-\pi k_s^{sed} f).$$

$$Ln \frac{A_s}{(2\pi f)^2} = b - \pi k_s f + \varepsilon,$$

$$Ln \frac{A_{sp}}{(2\pi f)^2} = b - \pi k_p f + \varepsilon.$$

$$Ln(A_s / A_{sp}) = b - \pi (k_s - k_p) f + \varepsilon.$$

$$Ln(Y^i) = \sum_{j=1}^{n} b_j G_j - a\pi f.$$

Modeling Considerations – CEUS

- Recurrence rates lower and uncertainties in source mechanisms, locations in CEUS.
- Stronger crustal structure in CEUS, therefore less attenuation.
- Stress drop?
- Too few strong motion recordings.

GEOTECHNICAL EARTHQUAKE ENGINEERING

Typically concerned with:

- *Determining ground motions* – especially as to effects of local site conditions
- *Liquefaction and liquefaction-related evaluations* – (settlements, lateral spreading movements, etc.)
- Slope/landslide evaluation
- Dams/embankments
- Design of retaining structures
- Deep and shallow foundation analysis
- Underground structures (tunnels, etc.)

Key Reference

Kramer, Steven L. 1996. *Geotechnical Earthquake Engineering.* Prentice Hall, 653 pp.

Historical Perspective

"While many cases of soil effects had been observed and reported for many years, it was not until a series of catastrophic failures, involving landslides at Anchorage, Valdez and Seward in the 1964 Alaska earthquake, and extensive liquefaction in Niigata, Japan, during the earthquake in 1964, caused geotechnical engineers to become far more aware of, and eventually engaged in understanding, these phenomena."

(I. M. Idriss, 2002)

Important Learning Opportunities

- 1964 Niigata and 1964 Alaska
- 1967 Caracas
- 1971 San Fernando
- 1979 Imperial valley
- 1985 Mexico City
- 1989 Loma Prieta
- 1995 Kobe (Japan)
- 1999 Kocaeli (Turkey)
- 1999 Chi Chi (Taiwan)

Site Effects – Some History

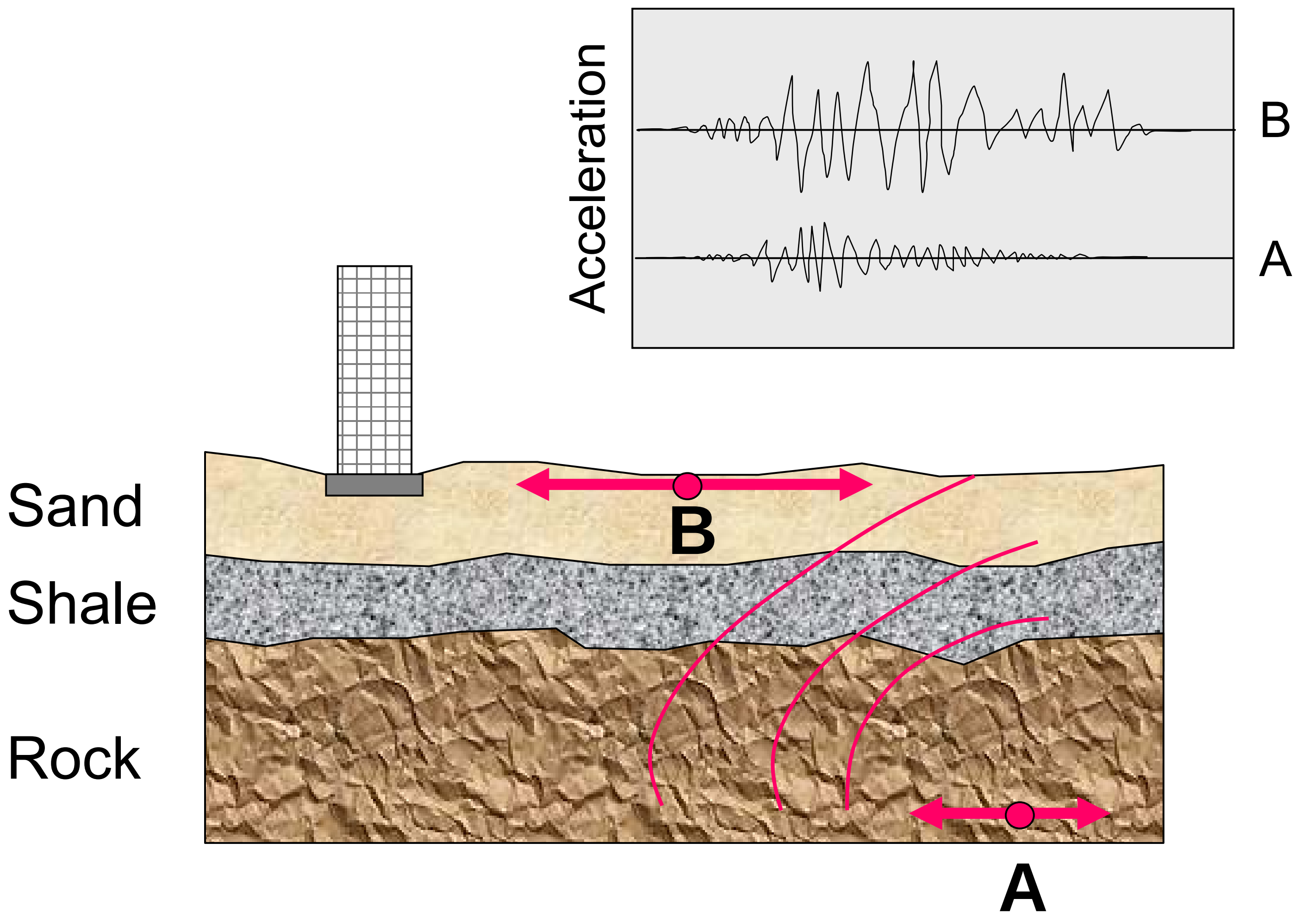
"… a movement … must be modified while passing through media of different constitutions. Therefore, the earthquake effects will arrive to the surface with higher or lesser violence according to the state of aggregation of the terrain which conducted the movement. This seems to be, in fact, what we have observed in the Colchagua Province (of Chile) as well as in many other cases."

- from Del Barrio (1855) in Toro and Silva (2001)

Site Effects on Ground Motions

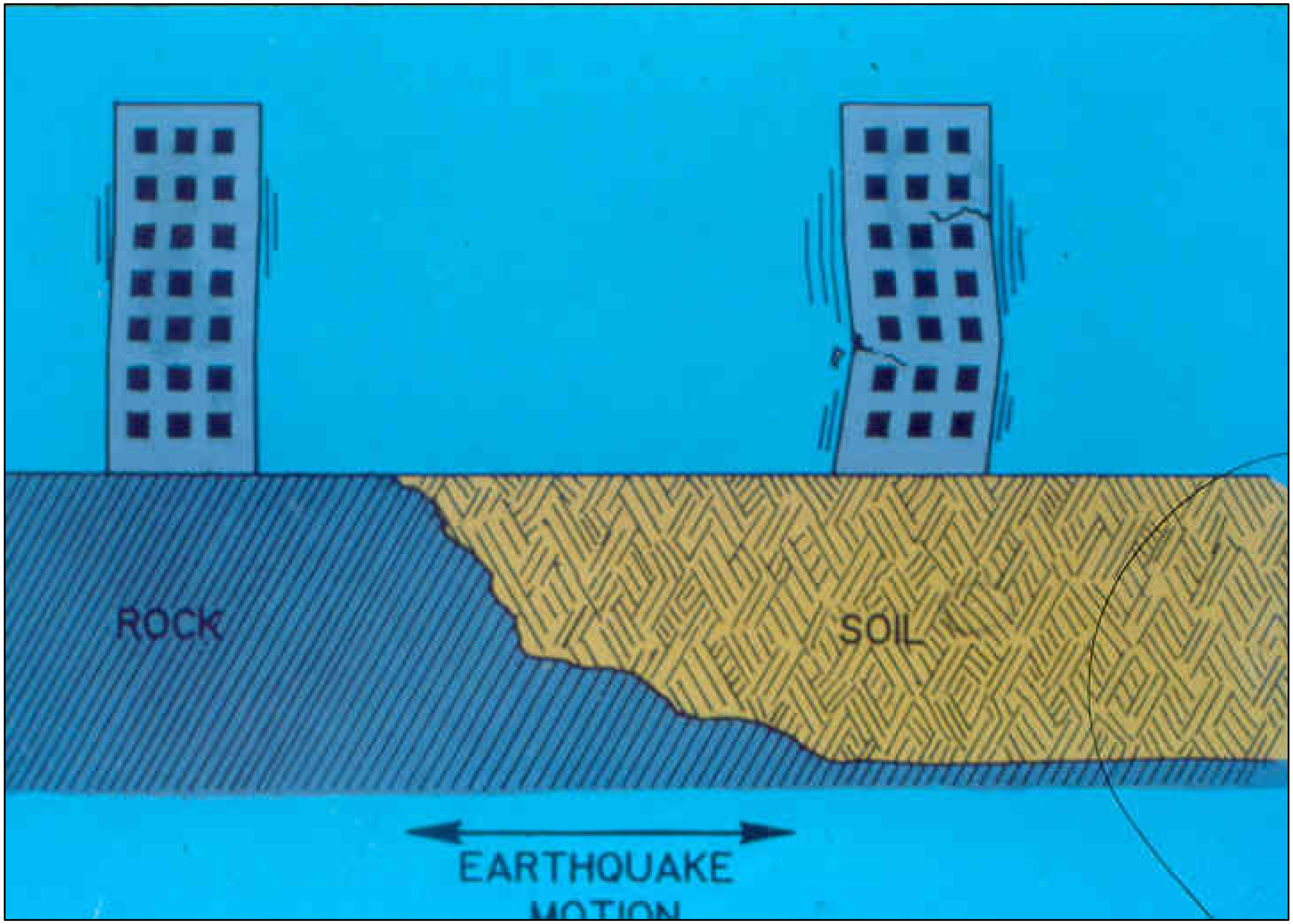
- Soil profile acts as filter
- Change in frequency content of motion
- Layering complicates the issue
- Amplification or de-amplification of ground motions can occur
- Duration of motion is increased

Site Amplification Is Common

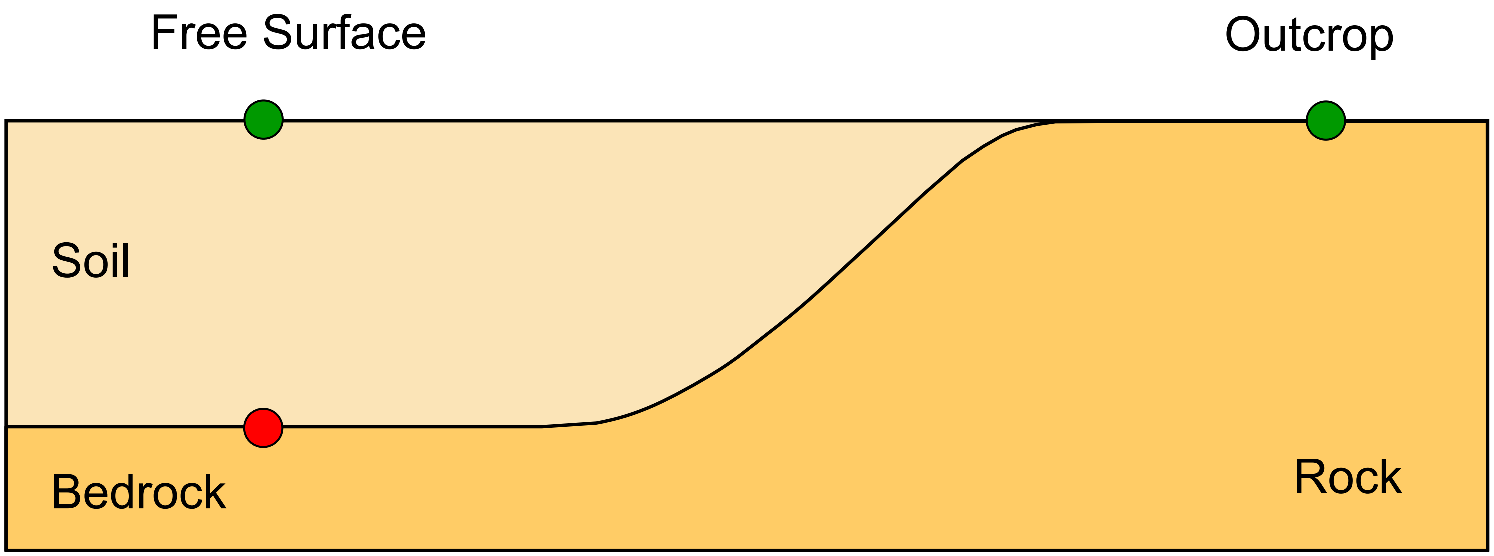


Site Effects on Ground Motions

Conservation of energy drives amplification



Amplification Definitions



$$\text{Amplification} = \frac{\text{Free Surface}}{\text{Bedrock}} \qquad \text{Amplification} = \frac{\text{Free Surface}}{\text{Outcrop}}$$

Figure adapted from Rix, G. J., (2001)

Amplification Definitions

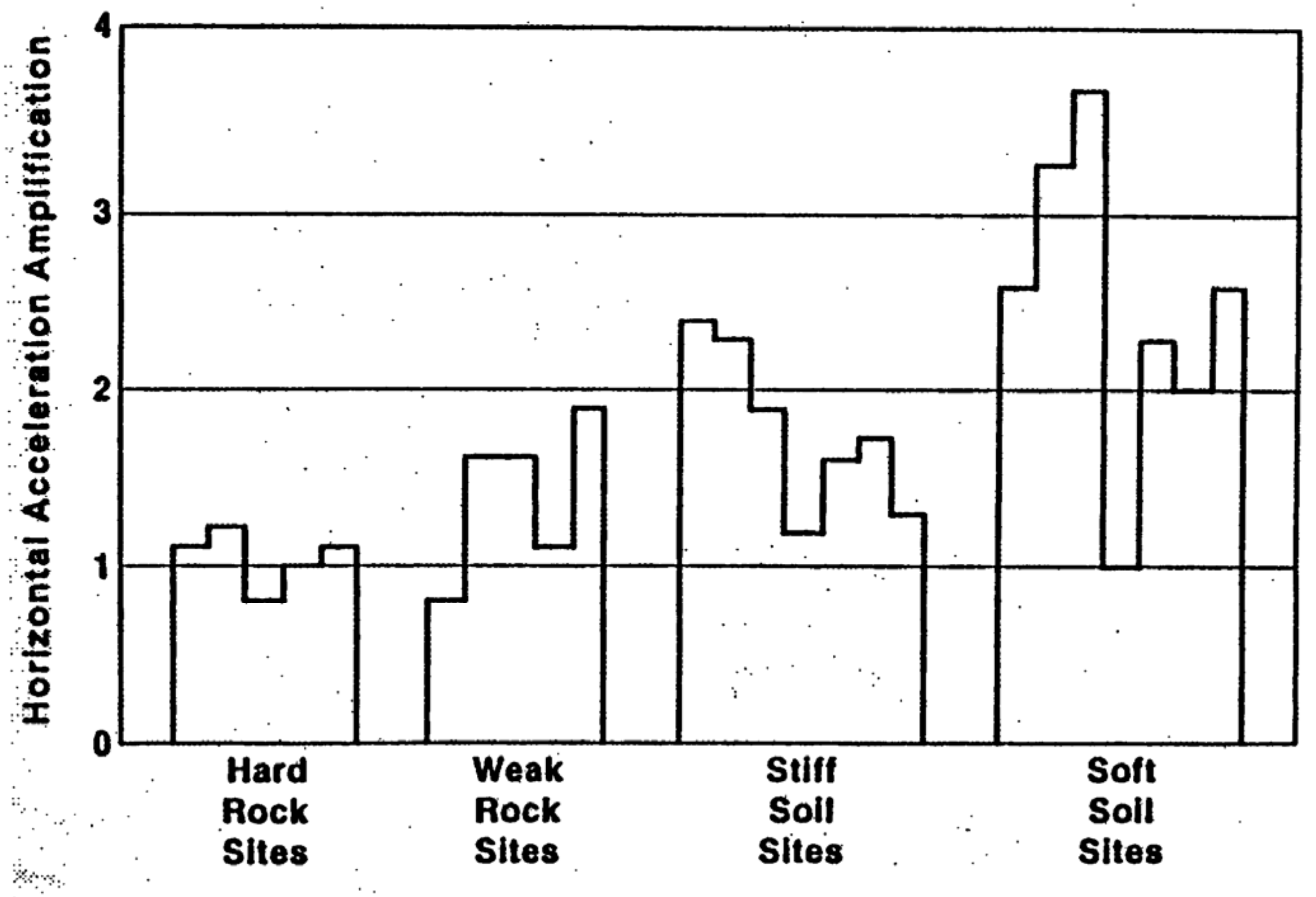
- Fourier amplification spectra

$$\left|\frac{a_{\text{free surface}}(f)}{a_{\text{outcrop}}(f)}\right|$$

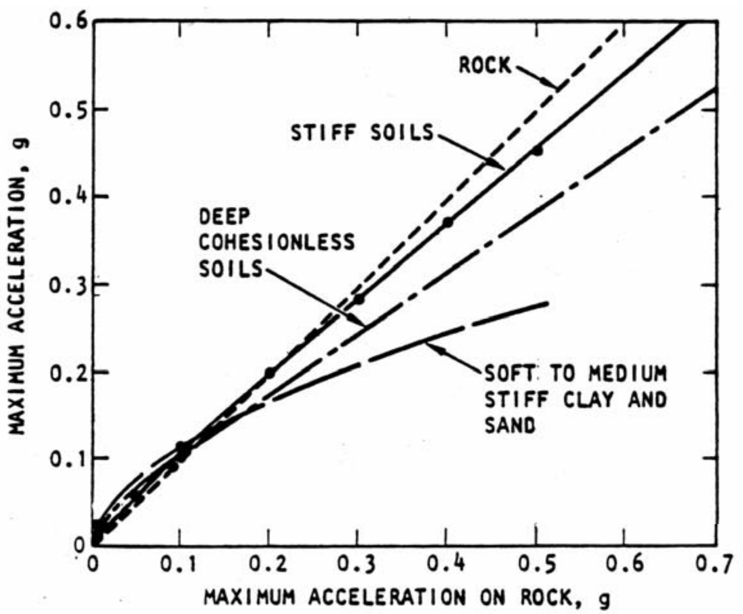
- Spectral amplification

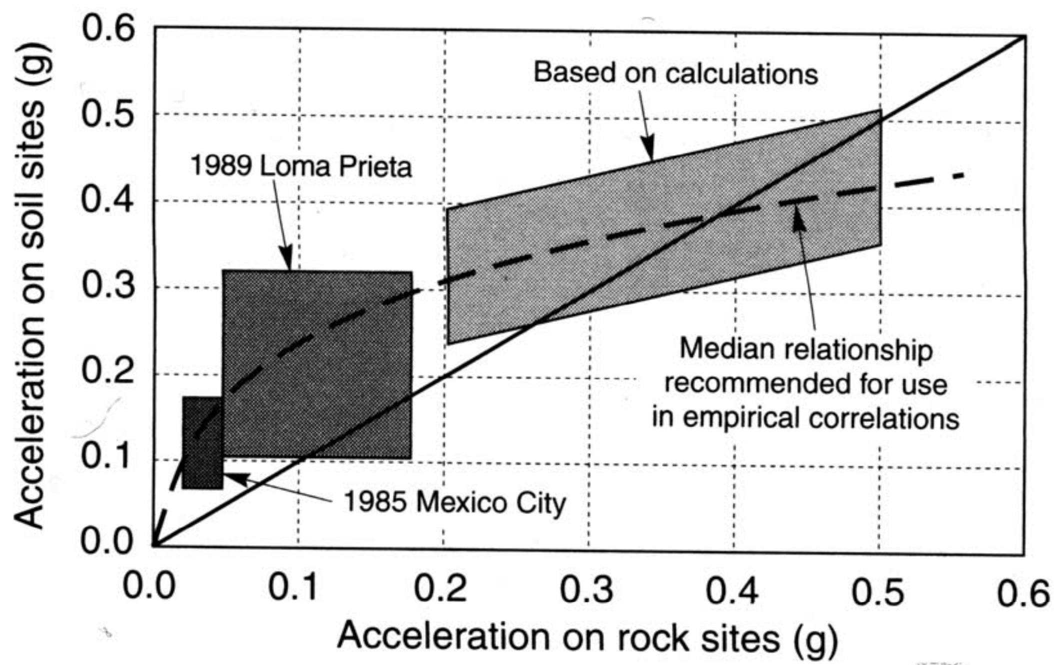
$$\left|\frac{S_{\text{a, free surface}}(T)}{S_{\text{a, outcrop}}(T)}\right|$$

Soft Soils Commonly Amplify Motions Relative To Bedrock

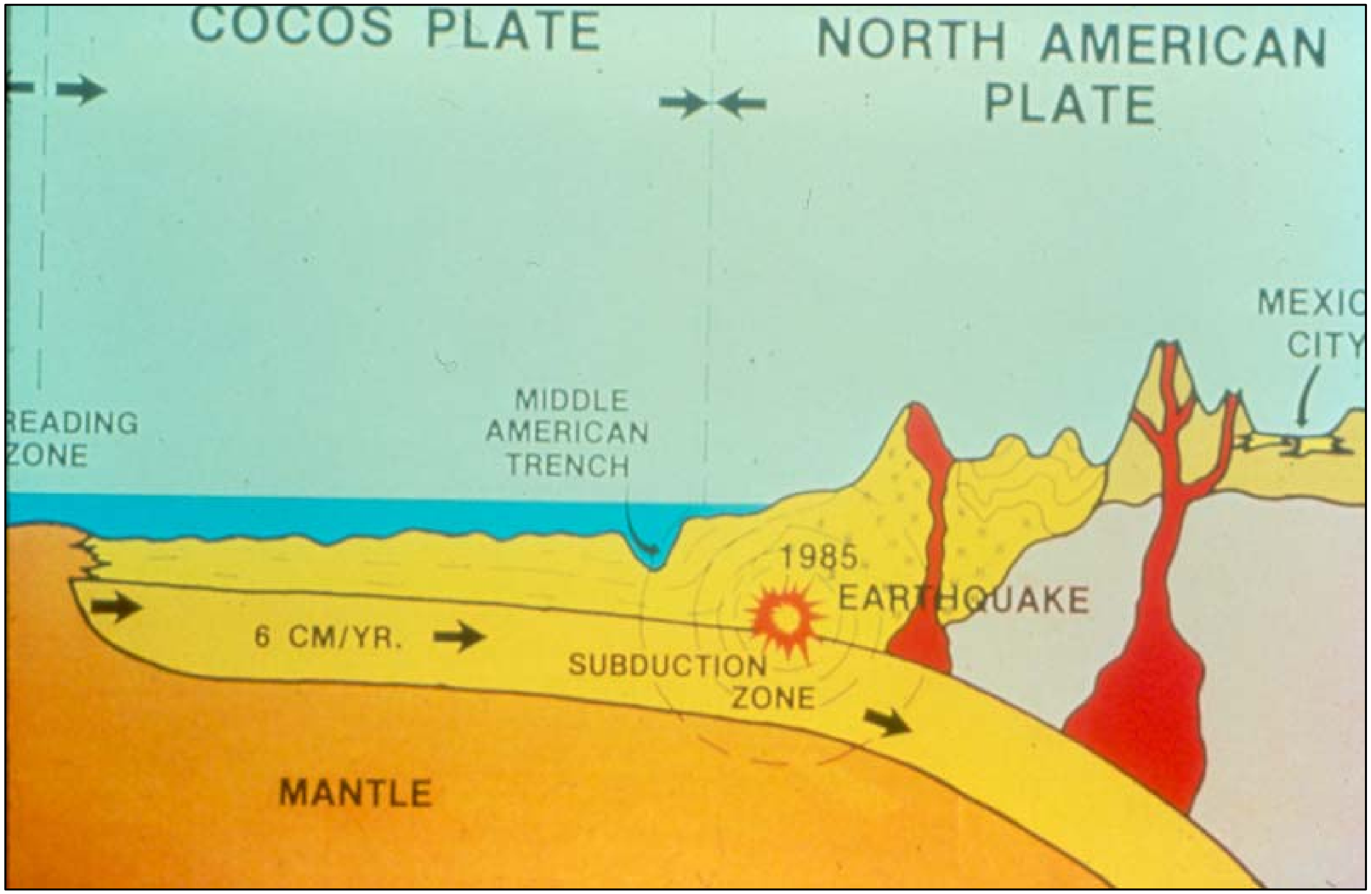


Effects of Local Soil Conditions

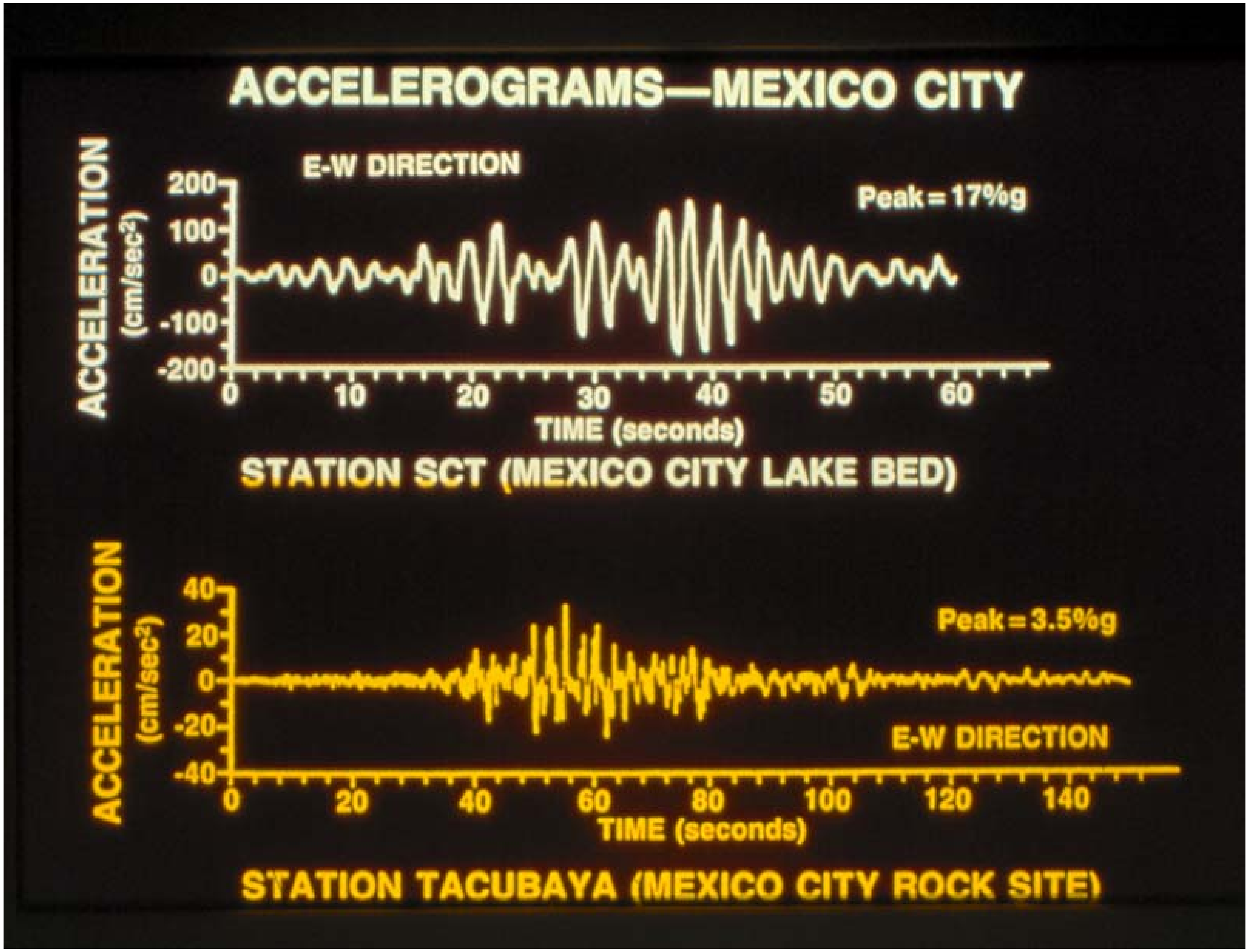




1985 Mexico City Earthquake

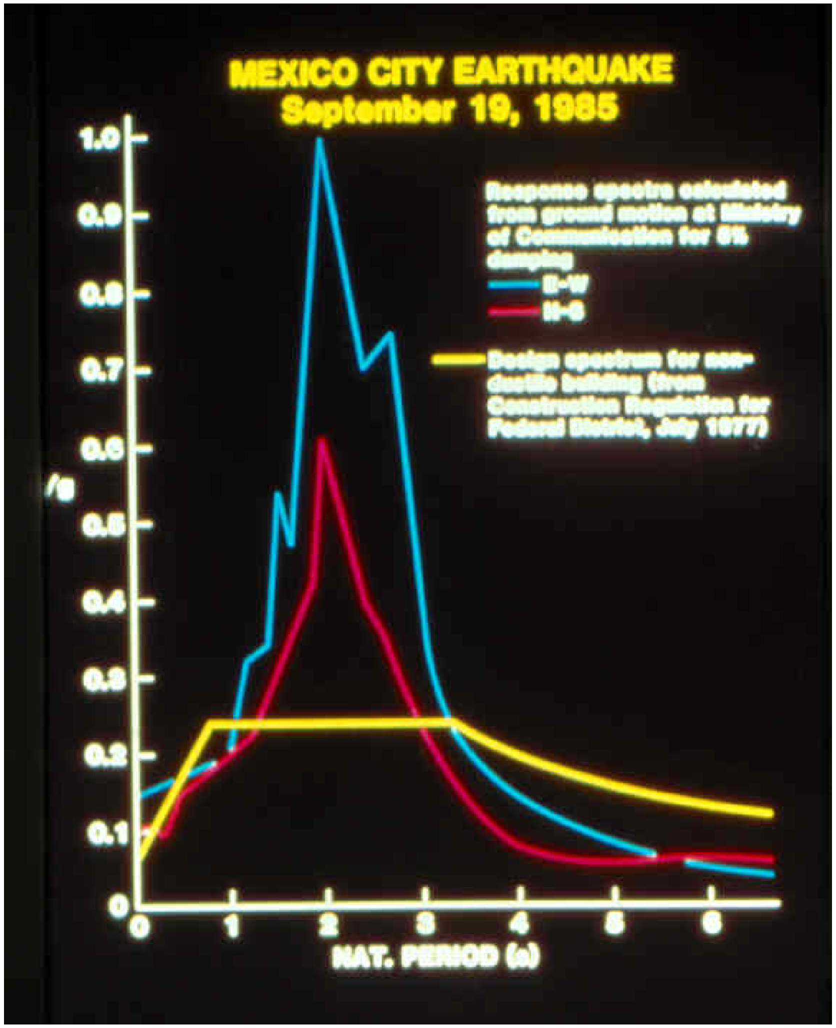


1985 Mexico City Accelerograms

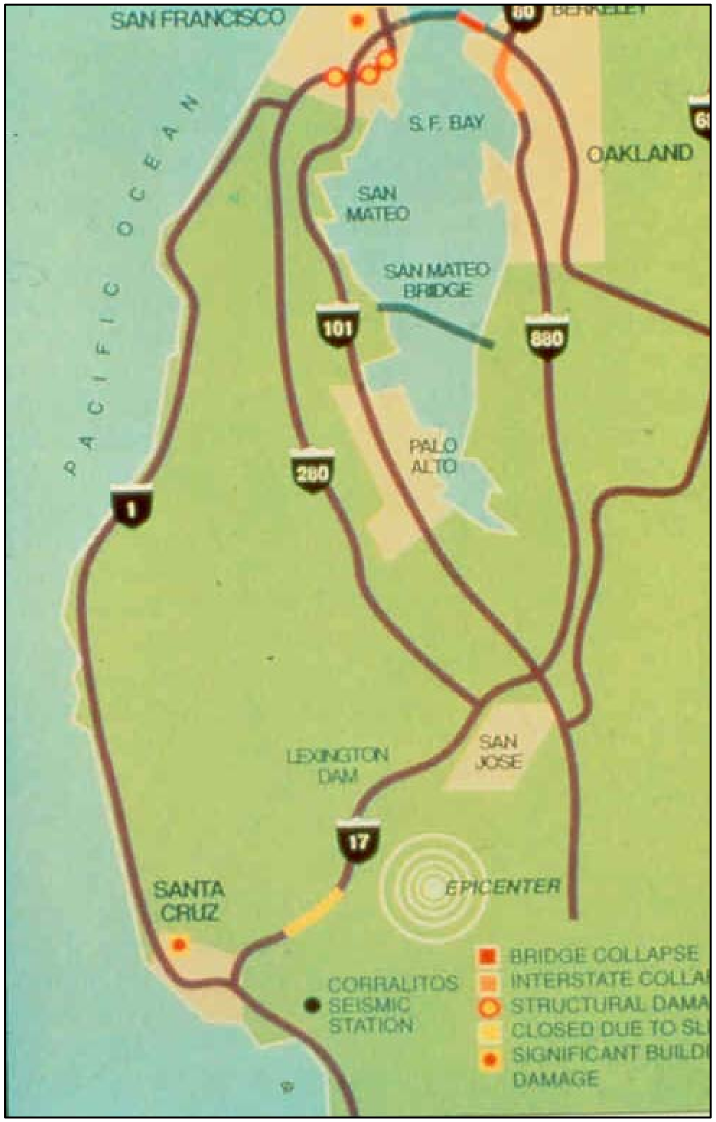


1985 Mexico City – Juarez Hospital

1985 Mexico City – Response Spectra

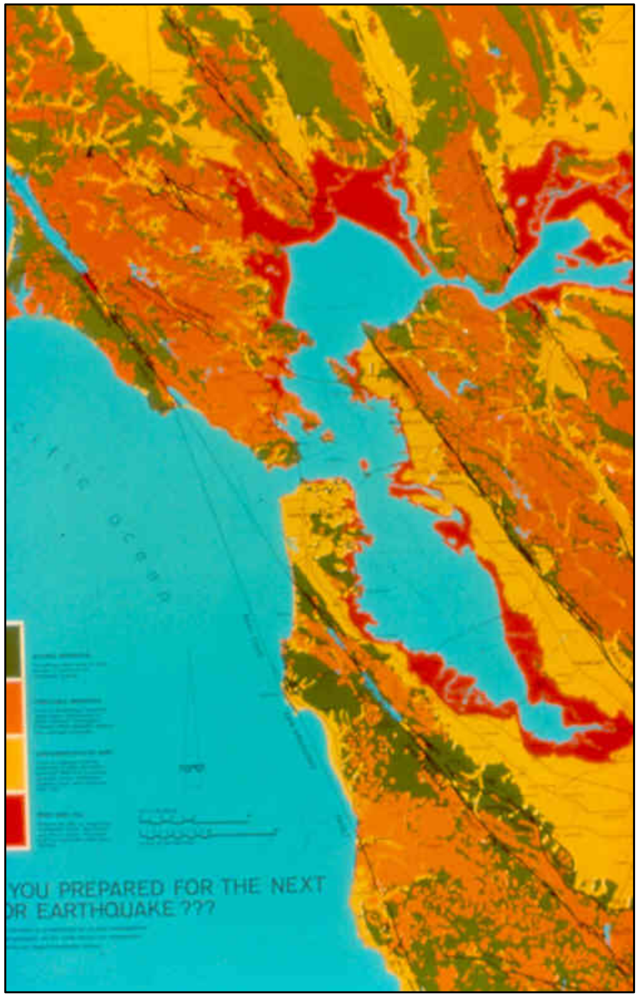


1989 Loma Prieta Earthquake



San Francisco Bay Geological Map

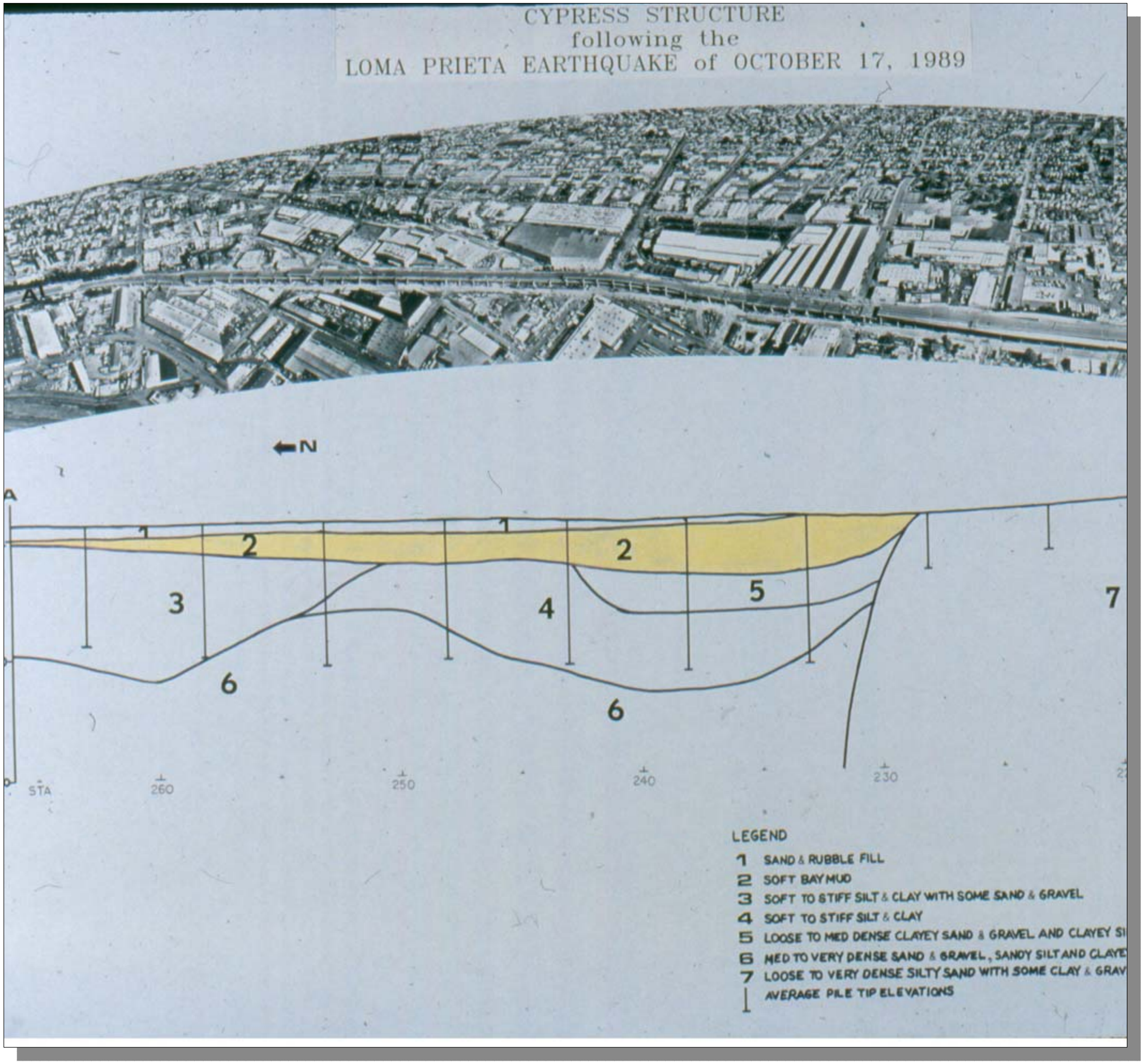
- Soft deposits in red (Bay mud)



San Francisco Marina District

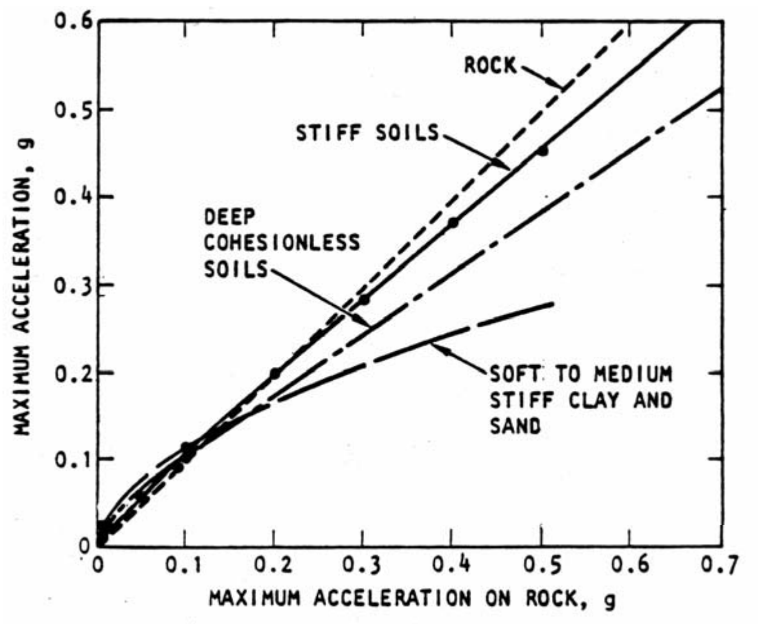
Damage in Marina District

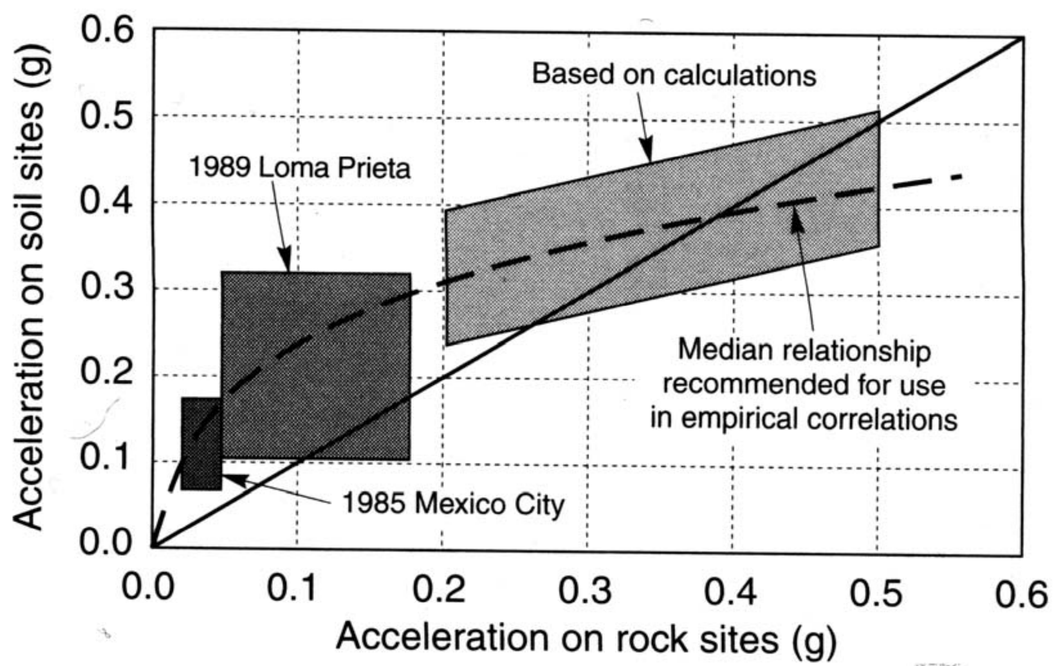
Cypress Structure Collapse


CYPRESS STRUCTURE
following the
LOMA PRIETA EARTHQUAKE of OCTOBER 17, 1989
N
A
1
2
3
4
5
6
7
STA
260
250
240
230
LEGEND
1 SAND & RUBBLE FILL
2 SOFT BAY MUD
3 SOFT TO STIFF SILT & CLAY WITH SOME SAND & GRAVEL
4 SOFT TO STIFF SILT & CLAY
5 LOOSE TO MED DENSE CLAYEY SAND & GRAVEL AND CLAYEY SI
6 MED TO VERY DENSE SAND & GRAVEL, SANDY SILT AND CLAYE
7 LOOSE TO VERY DENSE SILTY SAND WITH SOME CLAY & GRAV
AVERAGE PILE TIP ELEVATIONS

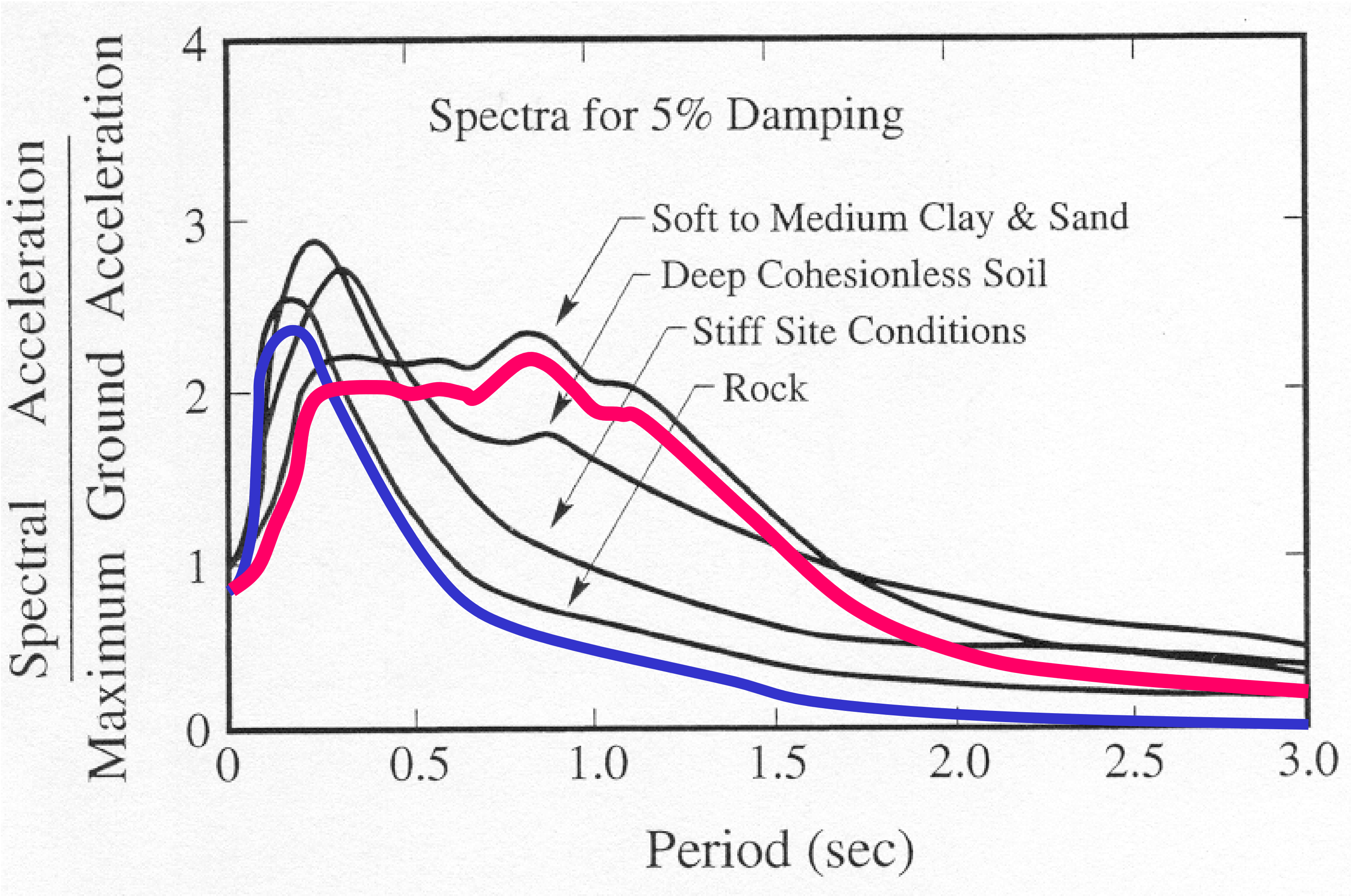
Cypress Structure Collapse

Effects of Local Soil Conditions

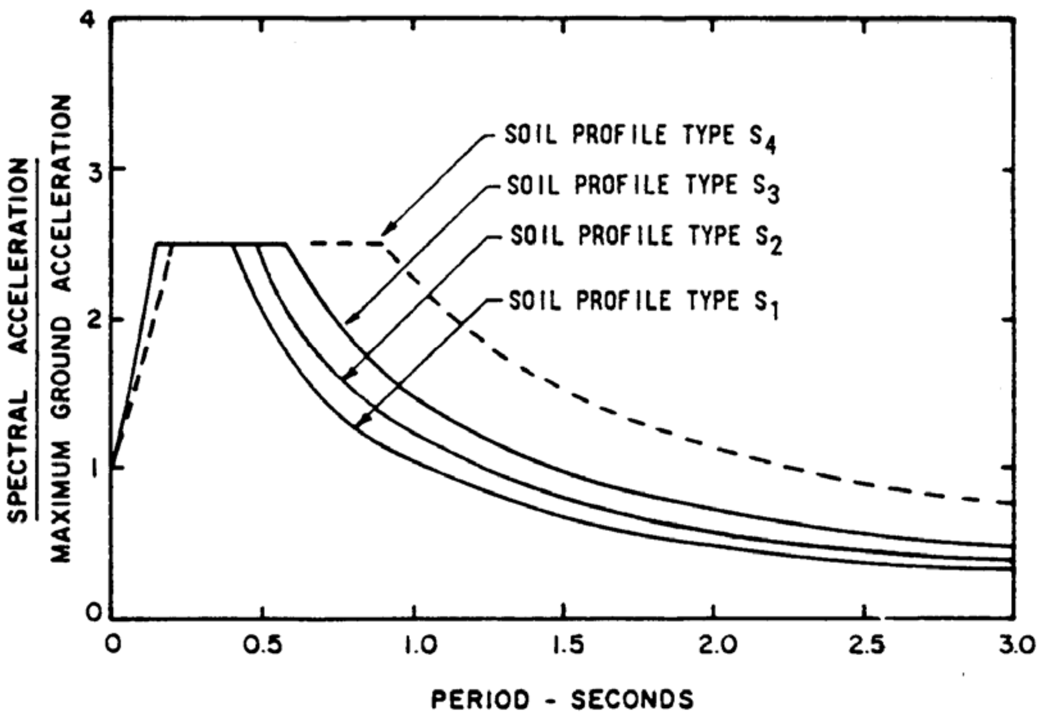




Effects of Local Soil Conditions



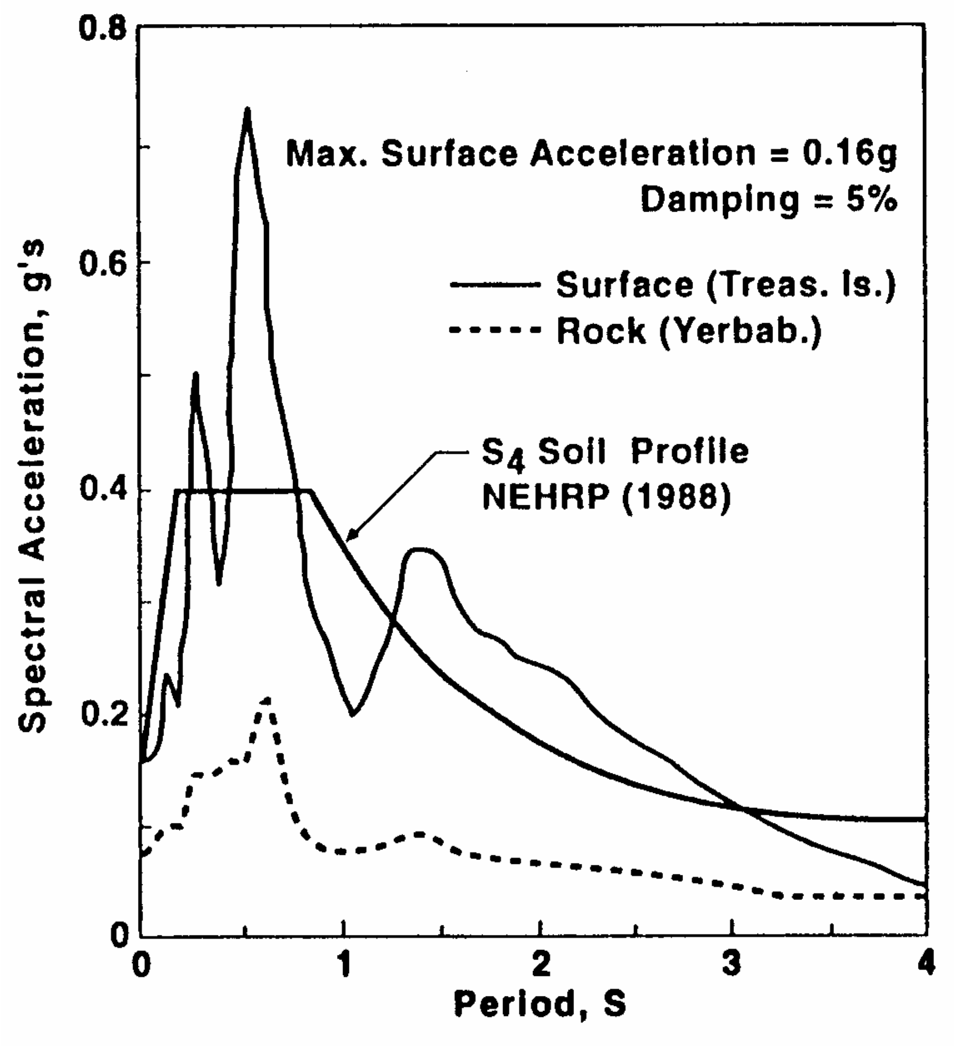
Pre-Loma Prieta Design Spectra



Soil Factor, S (NEHRP, 1988)

Type	Description	NEHRP
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity greater than 2,500 feet per second or by other suitable means of classification, or (b) stiff or dense soil condition where the soil depth is less than 200 feet.	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 200 feet or more.	1.2
S_3	A soil profile 70 feet or more in depth and containing more than 20 feet of soft to medium stiff clay but not more than 40 feet of soft clay.	1.5
S_4	A soil profile, characterized by a shear wave velocity less than 500 feet per second, containing more than 40 feet of soft clay.	2.0

Spectrum from 1989 Loma Prieta at Deep Soft Soil Site



Reason for F Category in IBC 2003

IBC2003 – "F" Requires Site-specific Analysis

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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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.

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				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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.

IBC2003 – "F" Requires Site-specific Analysis

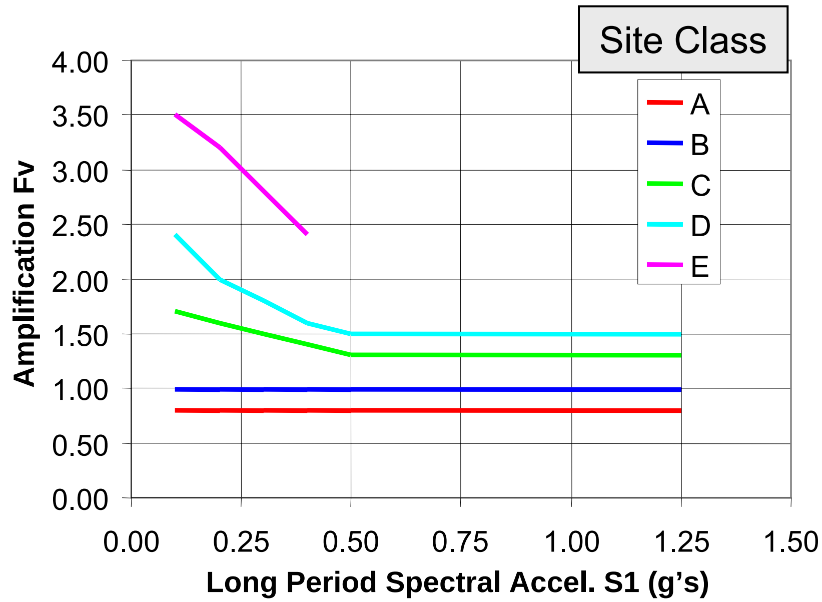
- Determine site class based on top 30 m:

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, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	Not applicable	Not applicable
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	Not applicable	Not applicable
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
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 $\bar{s}_u < 500$ 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 m², 1 pound per square foot = 0.0479 kPa.

NEHRP Provisions Site Amplification for Site Classes A through E

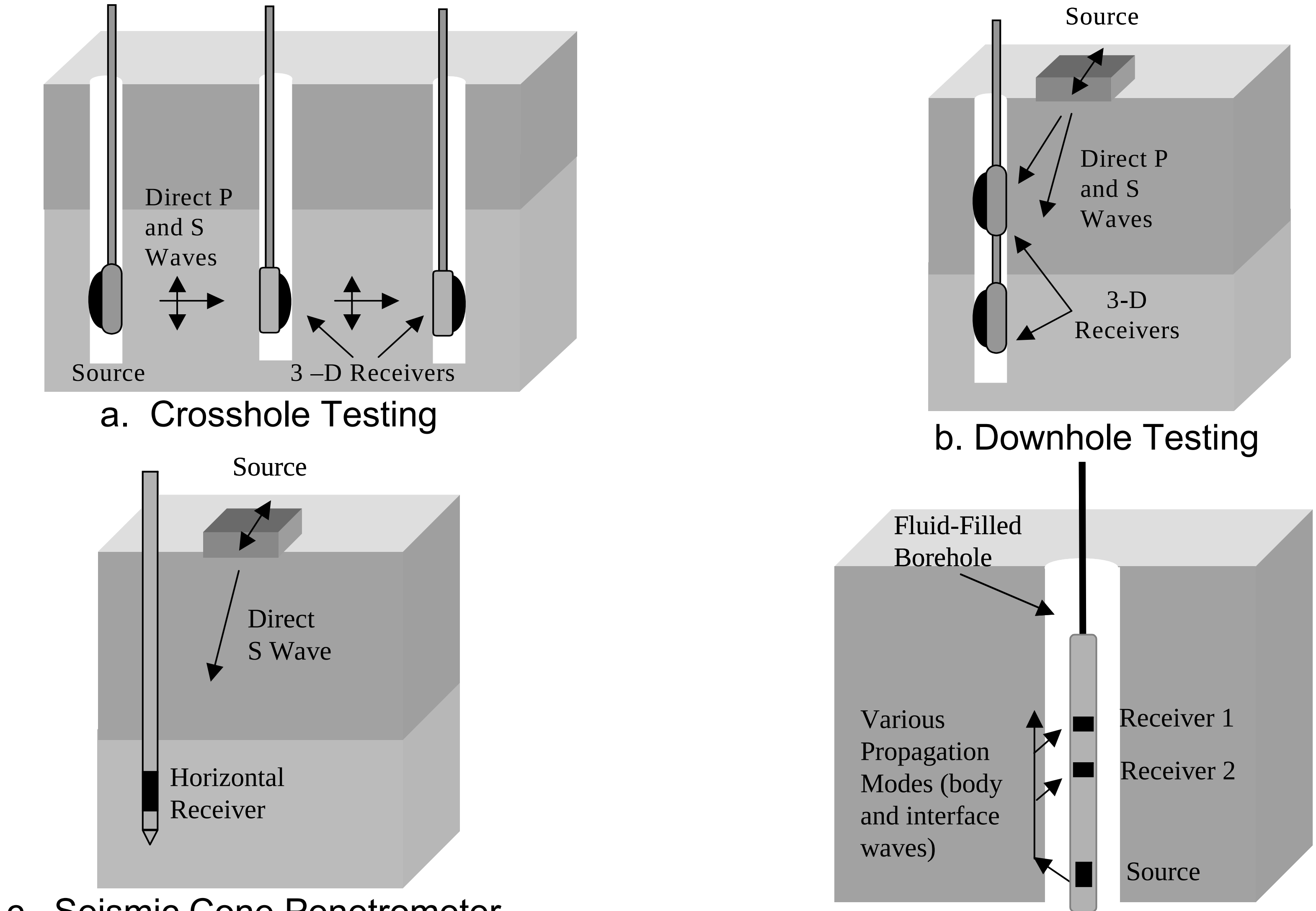


Site Classification from?

- *NEHRP Provisions* allow site classification to determined from various geotechnical data, such as SPT blowcounts, undrained shear strength, and shear wave velocity measurements (V_s)

- Best approach $\Rightarrow$ in situ V_s measurement

Field Tests To Measure Seismic Wave Velocities



a. Crosshole Testing

b. Downhole Testing

c. Seismic Cone Penetrometer

d. Suspension Logging

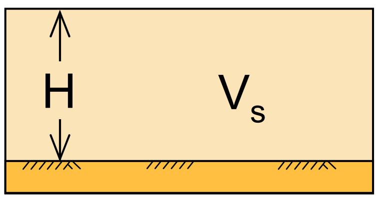
Courtesy of K. H. Stokoe II

Site Response Mechanisms

- Constant flux rate – impedance

$$\rho V_s \mathring{u}^2 = \text{constant}$$

- Resonances within the soil column

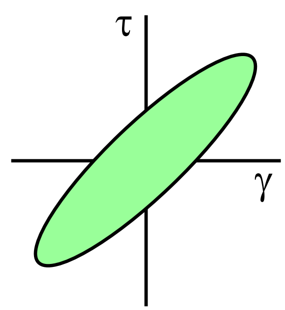


$$f_n = \frac{V_s}{4H}$$

Amplification

- Low-strain damping and apparent attenuation in soil

- Nonlinear soil behavior



Deamplification

Figure adapted from Rix, G. J., (2001)

Site Response Analysis - Two Steps

(1) Modeling the soil profile

(2) Calculating the site-modified time histories or other motions at various level within the profile, typically, at the ground surface

(1) Modeling the Soil Profile

- The stratigraphy and dynamic properties (dynamic moduli and damping characteristics) of the soil profile are modeled.
- If soil depth is reasonably constant beneath the structure and the soil layers and ground surface reasonably flat, then a one-dimensional analysis can be used.
- Two- or three-dimensional models of the site can be used where above conditions are not met.
- Unless soil properties are well constrained a range of properties should be defined for the soil layers to account for uncertainties.

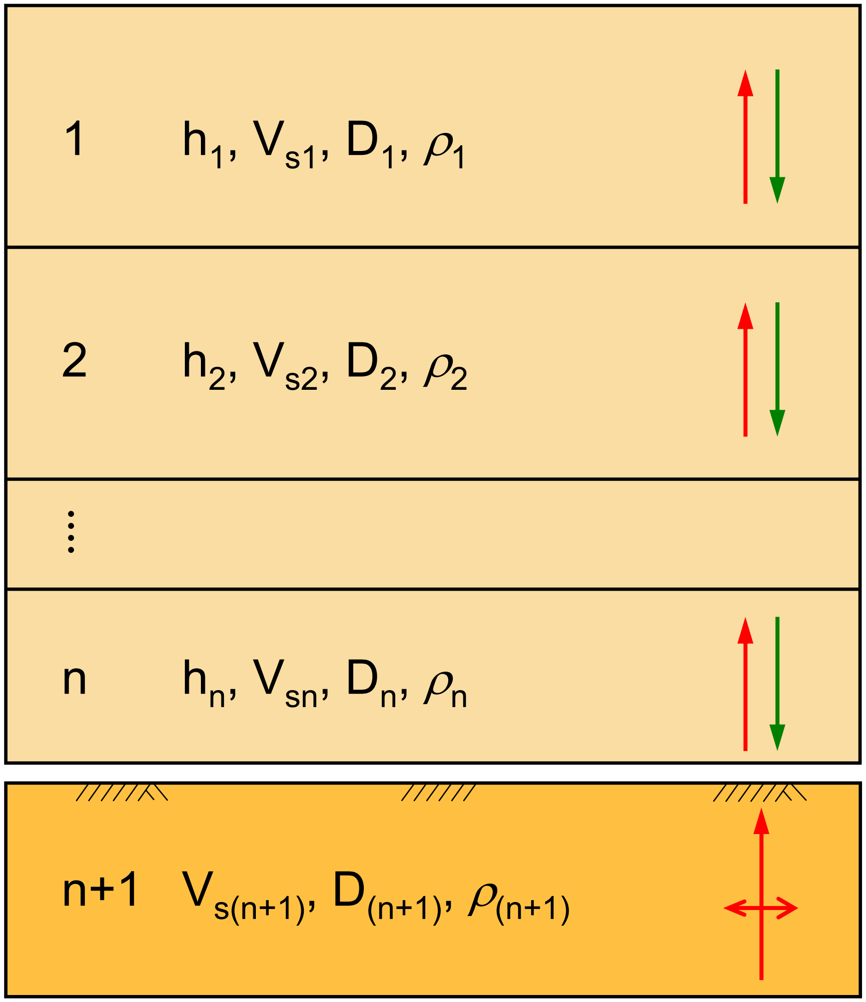
(2) Calculating top-of-profile motions:

- Typically the design bedrock time-histories are input to the soil model and the corresponding top-of-soil time-histories are obtained.
- Analysis should incorporate nonlinear soil behavior either through the equivalent linear method or true nonlinear analysis methods.
- Ensure program properly accounts for motion recorded on outcrop being input at base, etc.
- Issue: where to assume base or halfspace? (V_s = 2000 fps is often assumed but not always OK)

Site Response Analysis Techniques

- Linear analyses
- Quarter-wavelength approximation
- Equivalent linear analyses
- Nonlinear analyses

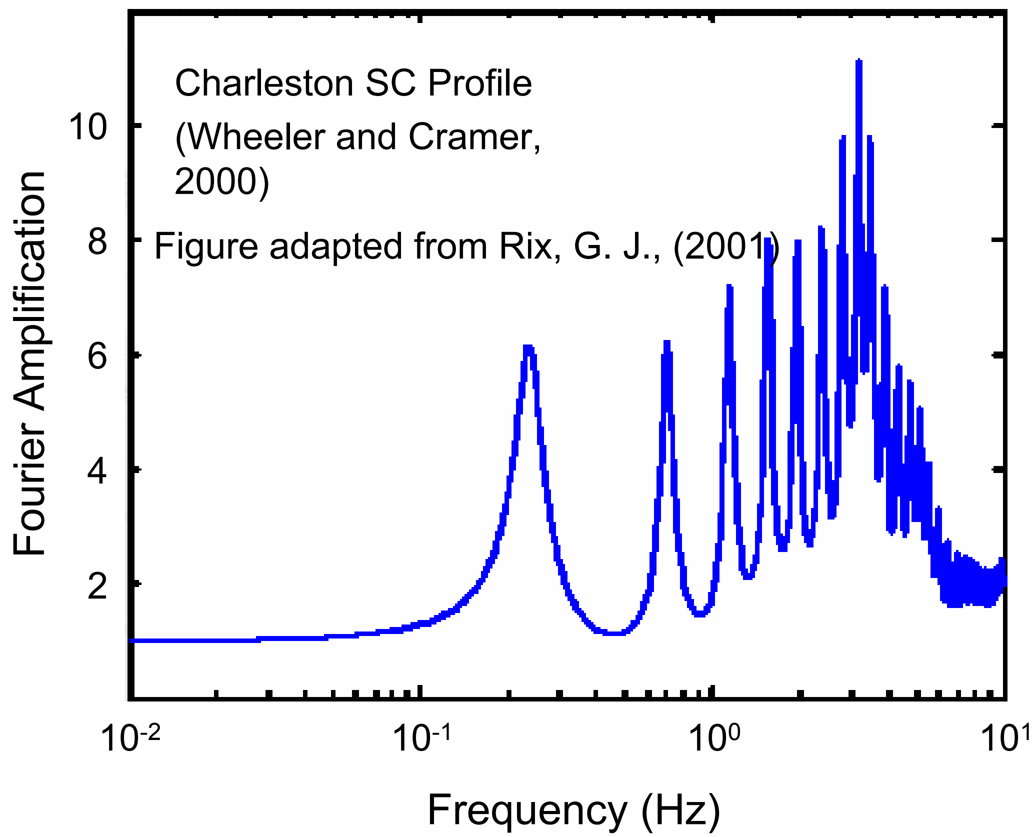
Site Response Calculations



- Layered profile
- Vertically propagating, horizontally polarized shear waves
- Calculate the amplitude of up-going and down-going waves in each layer by enforcing the compatibility of displacements and stresses at layer interface

Figure adapted from Rix, G. J., (2001)

Linear Analysis



- Constant V_s (i.e., G) and *D* (i.e., Q)

- Amplification from Pre-Cretaceous outcrop (hard rock) to ground surface. Soil profile is ~1 km thick.

Equivalent-Linear Analysis (i.e., SHAKE)

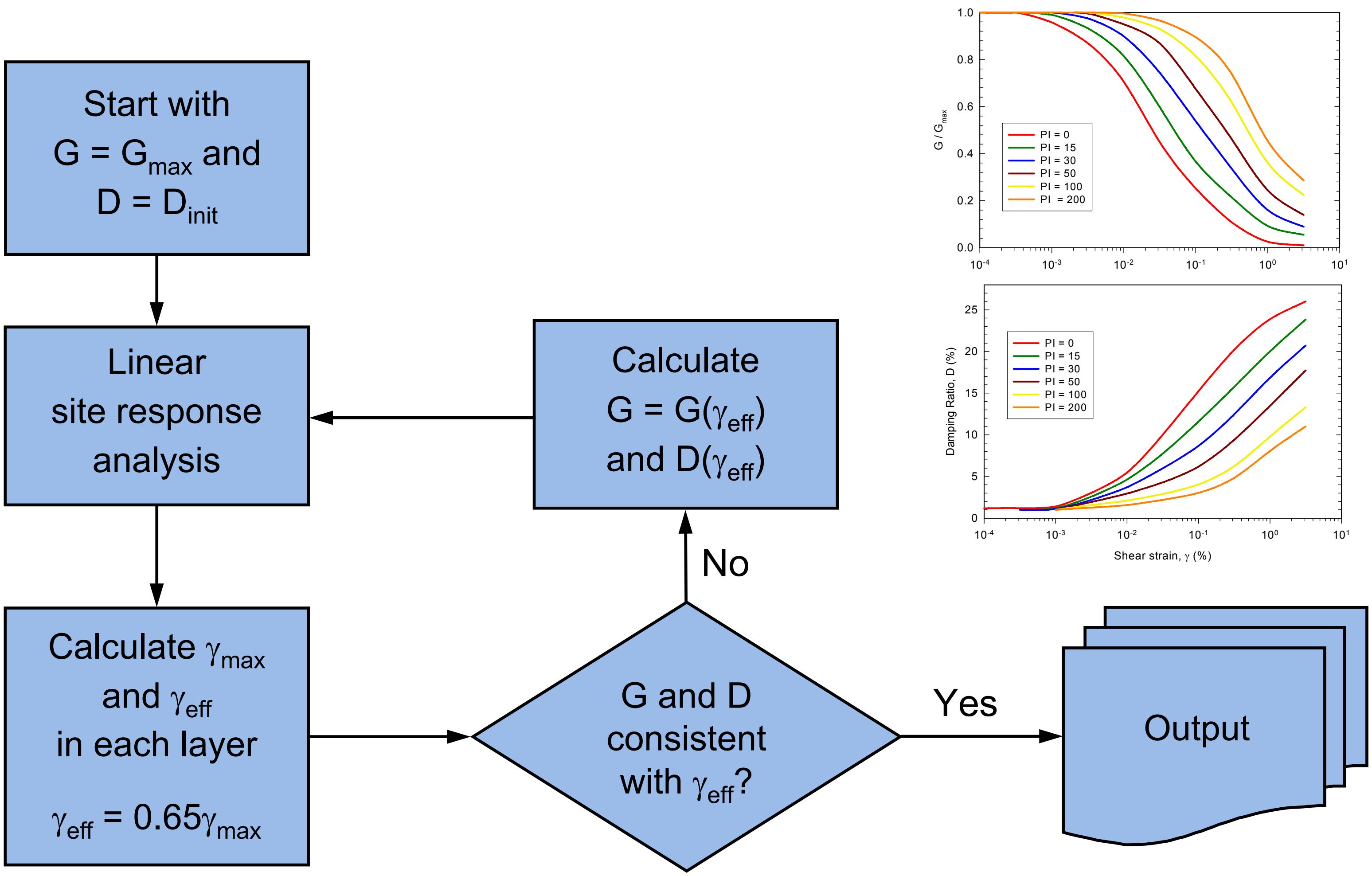
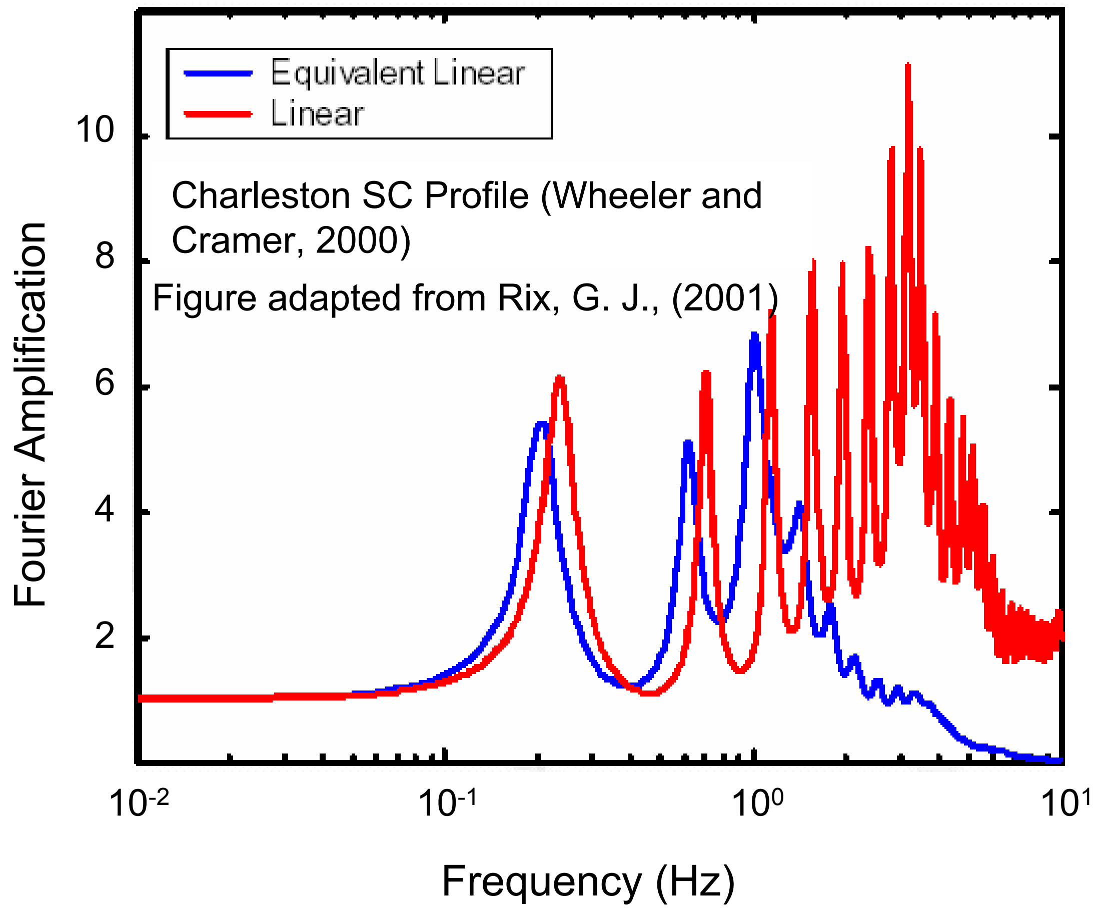
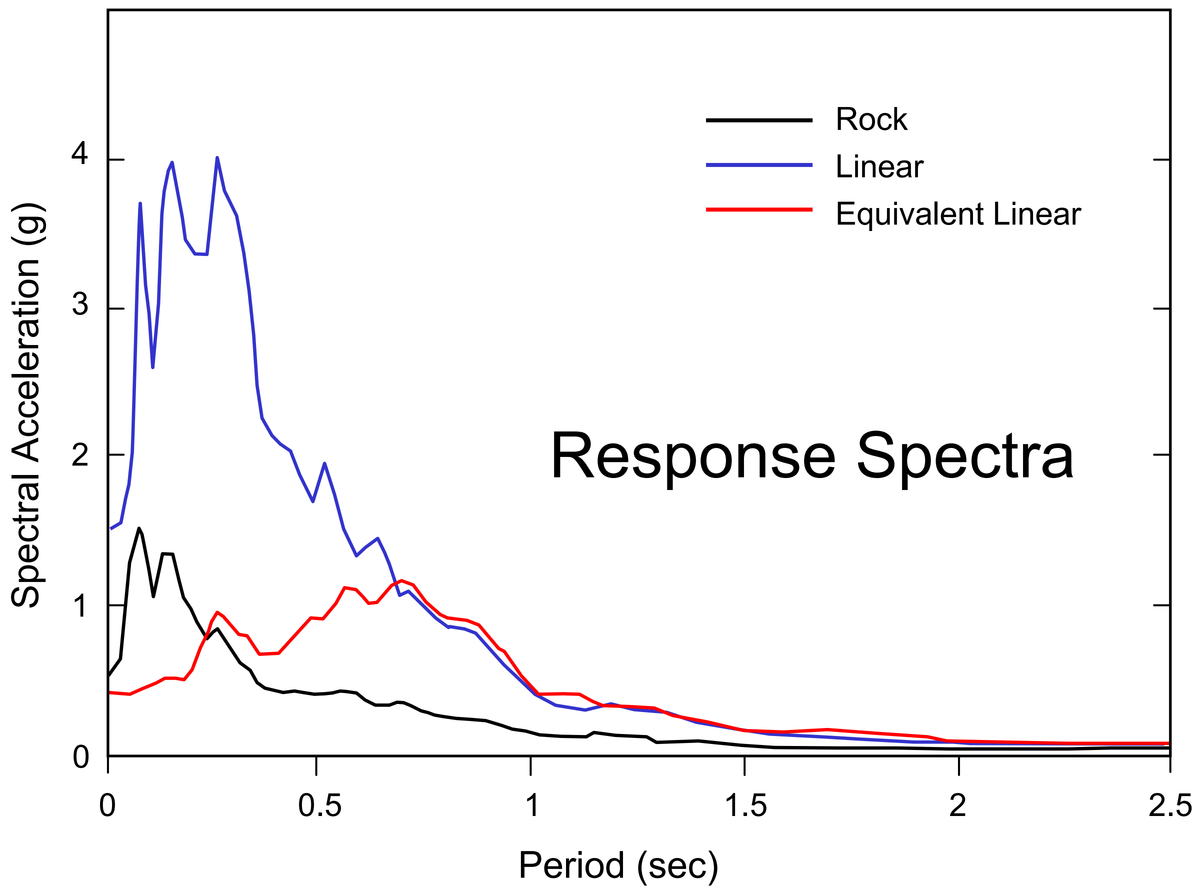


Figure adapted from Rix, G. J., (2001)

Equivalent Linear Analysis

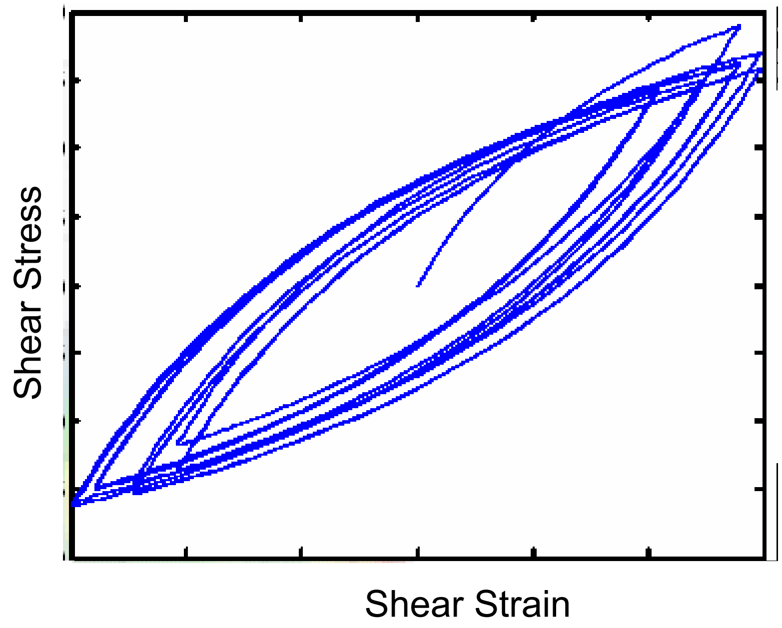


Equivalent Linear Analysis



Nonlinear Analysis

- Choose a constitutive model representing nonlinear cyclic soil behavior (nonlinear inelastic, cyclic plasticity, pore pressure generation)
- Integrate the equation of motion for vertically propagating shear waves in time domain
- Programs available are DESRA, FLAC, DYNAFLOW, SUMDES, etc.



Equivalent Linear vs. Nonlinear

- The inherent linearity of equivalent linear analyses can lead to “spurious” resonances.
- The use of effective shear strain can lead to an over-softened and over-damped system when the peak shear strain is not representative of the remainder of the shear-strain time history and vice versa.
- Nonlinear methods can be formulated in terms of effective stress to model generation of excess pore pressures.
- Nonlinear methods require a robust constitutive model that may require extensive field and lab testing to determine the model parameters.
- Difference between equivalent linear and nonlinear analyses depend on the degree of nonlinearity in the soil response. For low to moderate strain levels (i.e. weak input motions and/or stiff soils), equivalent linear methods provide satisfactory results.

-- from Kramer (1996)

Site Response Analysis Codes

A. One-dimensional equivalent-linear codes:

- SHAKE (Schnabel, Seed, and Lysmer 1972; Idriss and Sun 1992)
- WESHAKE (Sykora, Wahl, and Wallace 1992);

Site Response Analysis Codes

B. One-dimensional nonlinear codes:

- DESRA-2 (Lee and Finn 1978), DESRA-MUSC (Qiu 1998)
- SUMDES (Li, Wang, and Shen 1992)
- MARDES (Chang et al. 1990)
- D-MOD (Matasovic 1993)
- TESS (Pyke 1992)

Site Response Analysis Codes

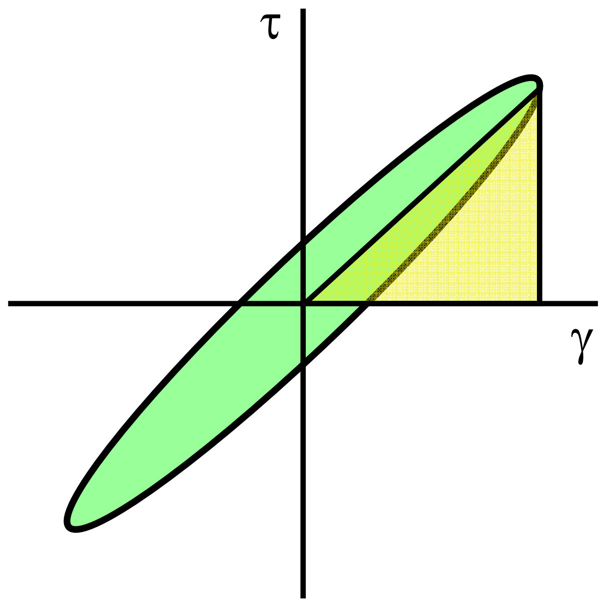
C. 2-D and 3-D equivalent linear codes:

- FLUSH (2-D) (Lysmer et al. 1975)
- QUAD4M (Hudson, Idriss, and Beikae 1994)
- SASSI (2-D or 3-D) (Lysmer et al. 1991)

Dynamic Soil Properties

- Shear wave velocity profile

$$G_{\max} = \rho \cdot V_s^2$$



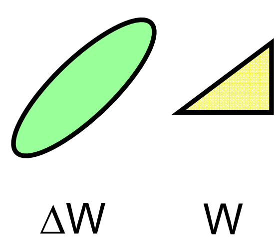
- Nonlinear soil behavior

Modulus reduction curve

$$G_{\sec} / G_{\max} = f\left(\gamma_{cyclic}\right)$$

Material damping ratio curve

$$D = \frac{1}{4\pi} \frac{\Delta W}{W} = f\left(\gamma_{cyclic}\right)$$



Laboratory Methods

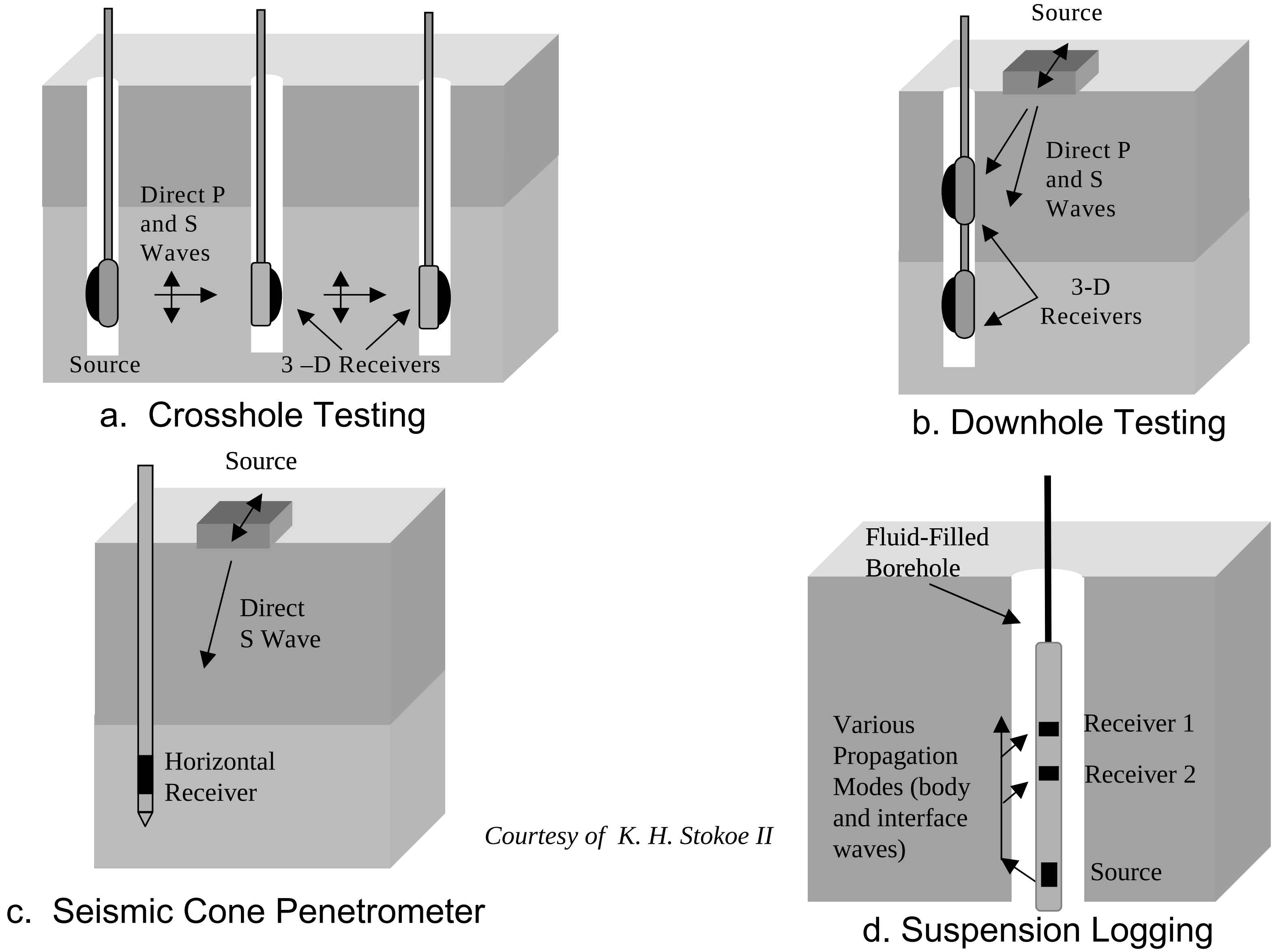
- Resonant column
- Torsional shear
- Cyclic simple shear
- Cyclic triaxial
- Bender elements

In Situ Methods

- Invasive methods
  - Crosshole
  - Downhole/SCPT
  - P-S suspension logger
- Invasive methods for nonlinear soil properties
- Vertical arrays

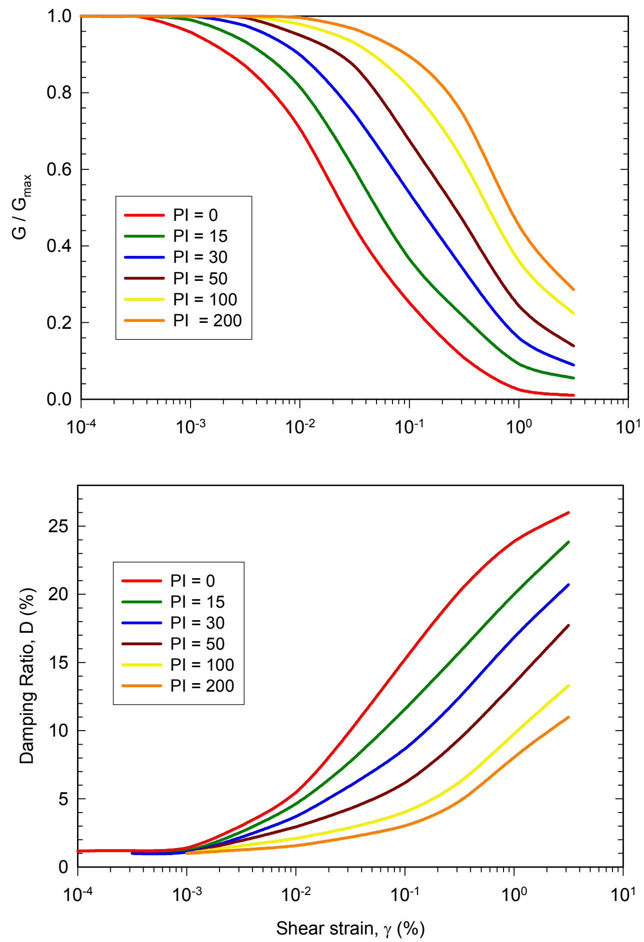
- Noninvasive methods
  - Refraction
  - High-resolution seismic reflection
  - Surface wave methods
- Empirical correlations with SPT and CPT

In Situ Tests to Measure Seismic Wave Velocities



Courtesy of K. H. Stokoe II

Modulus Reduction and Damping



Vucetic and Dobry (1991)

- Seed et al. (1986)
- Sun et al. (1988)
- Ishibashi and Zhang (1993)
- EPRI (1993)
- Hwang (1997)
- Assimaki et al. (2000)
- Toro and Silva (2001)

Liquefaction

"If a saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume.

If drainage is unable to occur, the tendency to decrease in volume results in an increase in pore pressure.

If the pore water pressure builds up to the point at which it is equal to the overburden pressure, the effective stress becomes zero, the sand loses its strength completely, and liquefaction occurs."

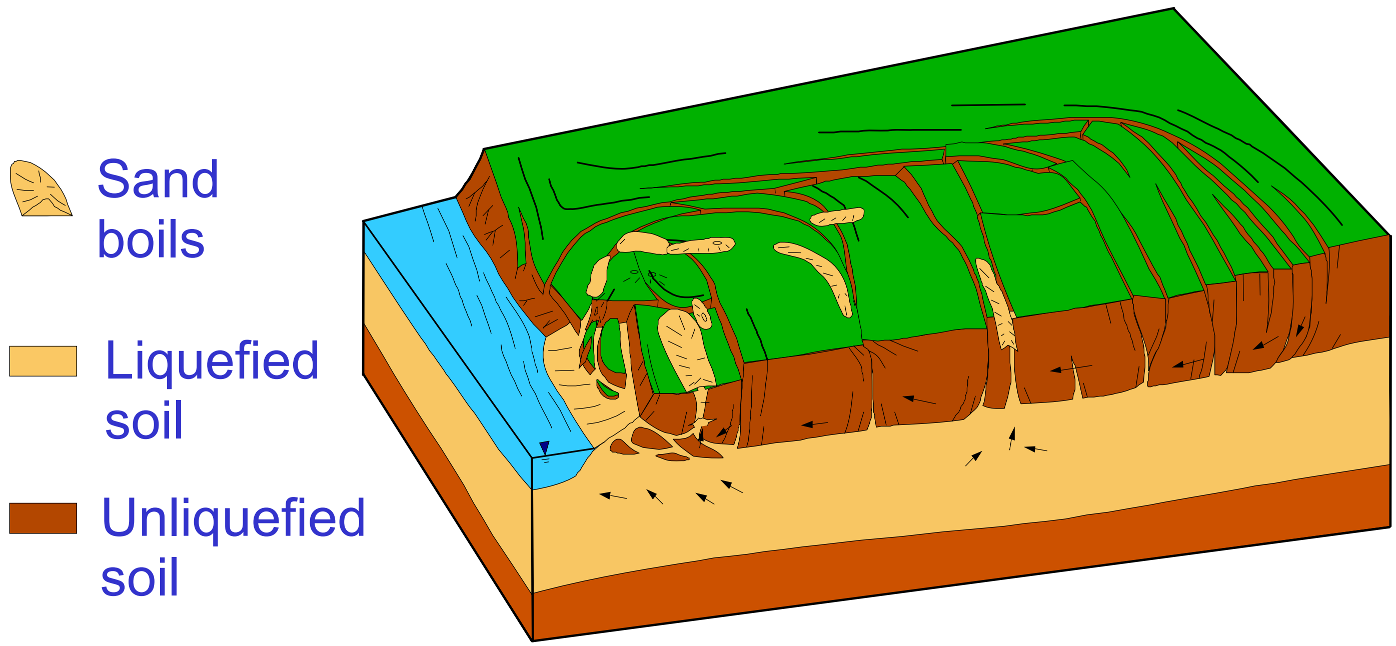
Seed and Idriss

Liquefaction - Field of Sand Boils

Liquefaction Damage, Niigata, Japan, 1964

Liquefaction Damage, Adapazari, Turkey, 1999

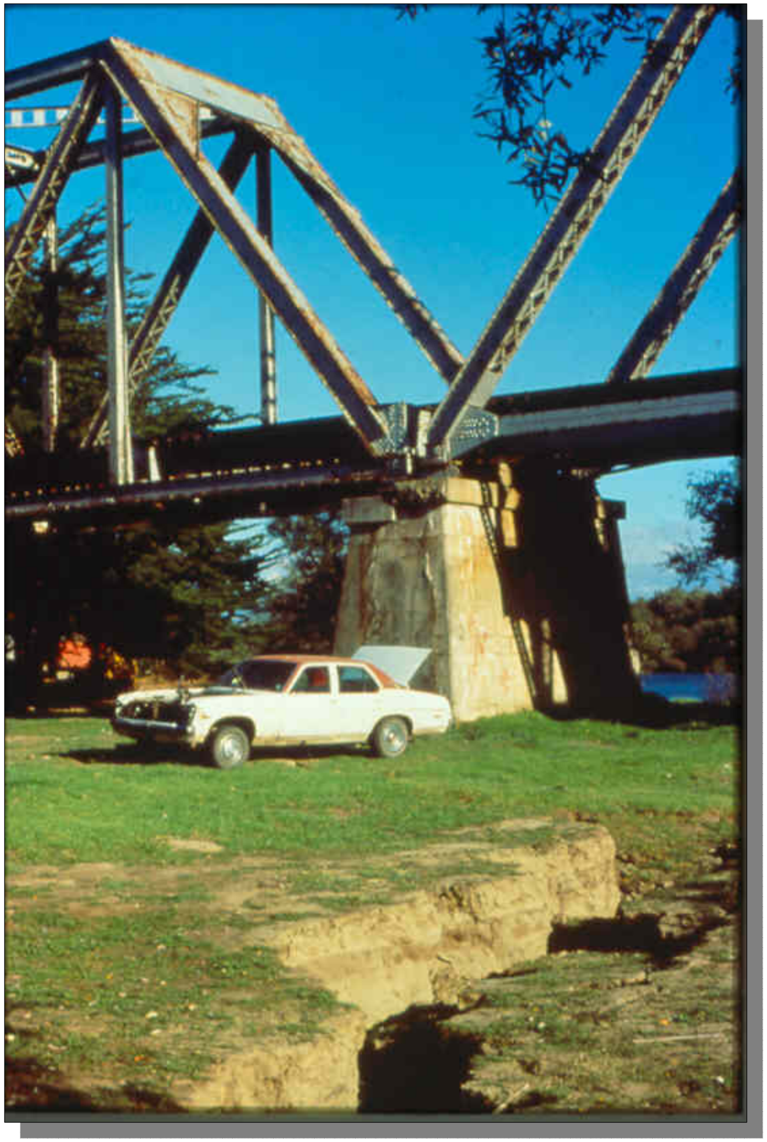
Lateral Spreading



- Mostly horizontal deformation of gently-sloping ground (< 5%) resulting from soil liquefaction
- One of most pervasive forms of ground damage; especially troublesome for lifelines

Liquefaction and Lateral Spreading, Kobe, Japan, 1995

Lateral Spreading, Loma Prieta, 1989

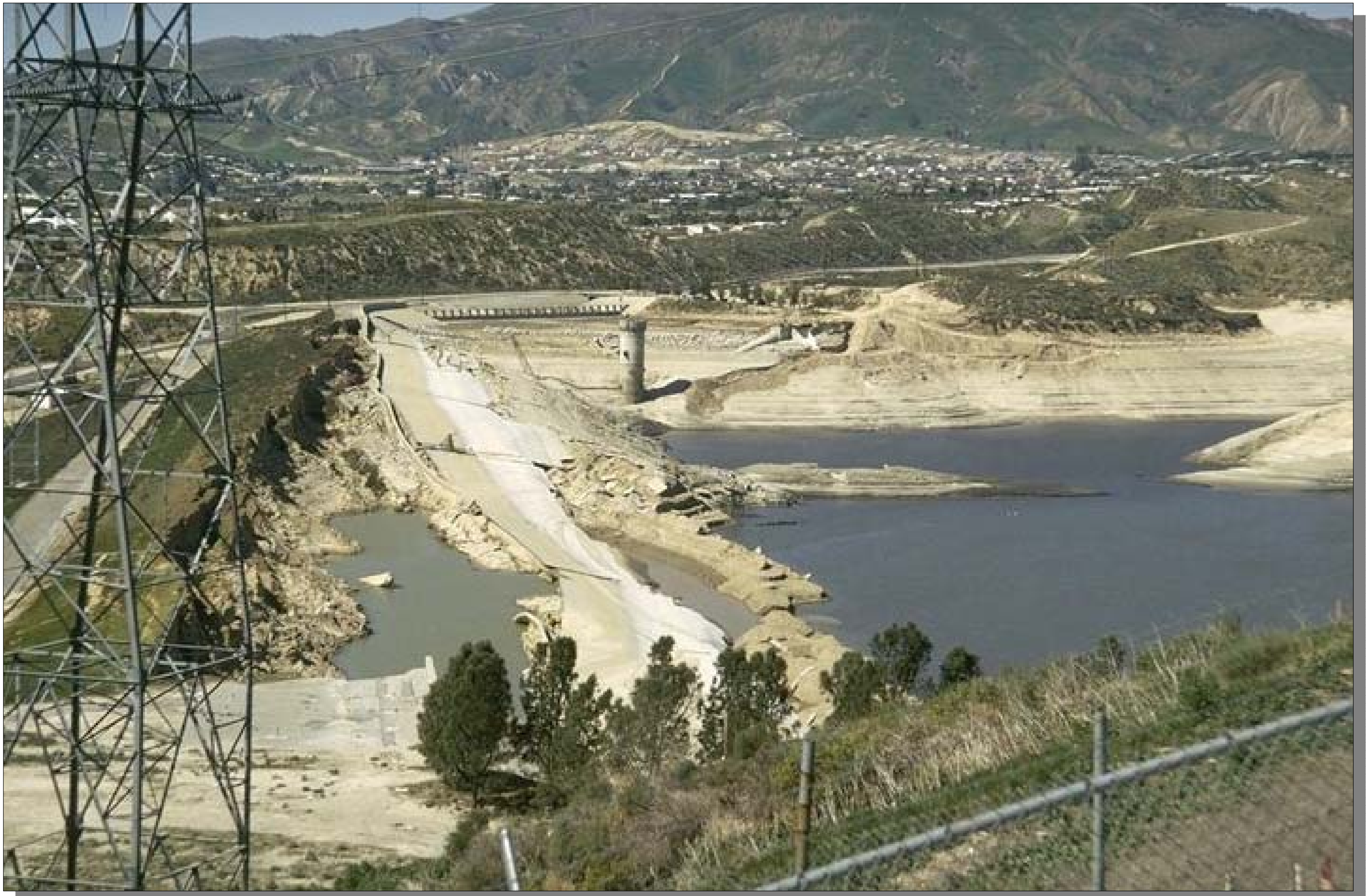


Pile Damage Beneath Building by Lateral Spread 1964, Niigata, Japan

Photo courtesy of Professor T. L. Youd from Elgamal (2002)

Lower San Fernando Dam

Lower San Fernando Dam



Liquefaction Damage

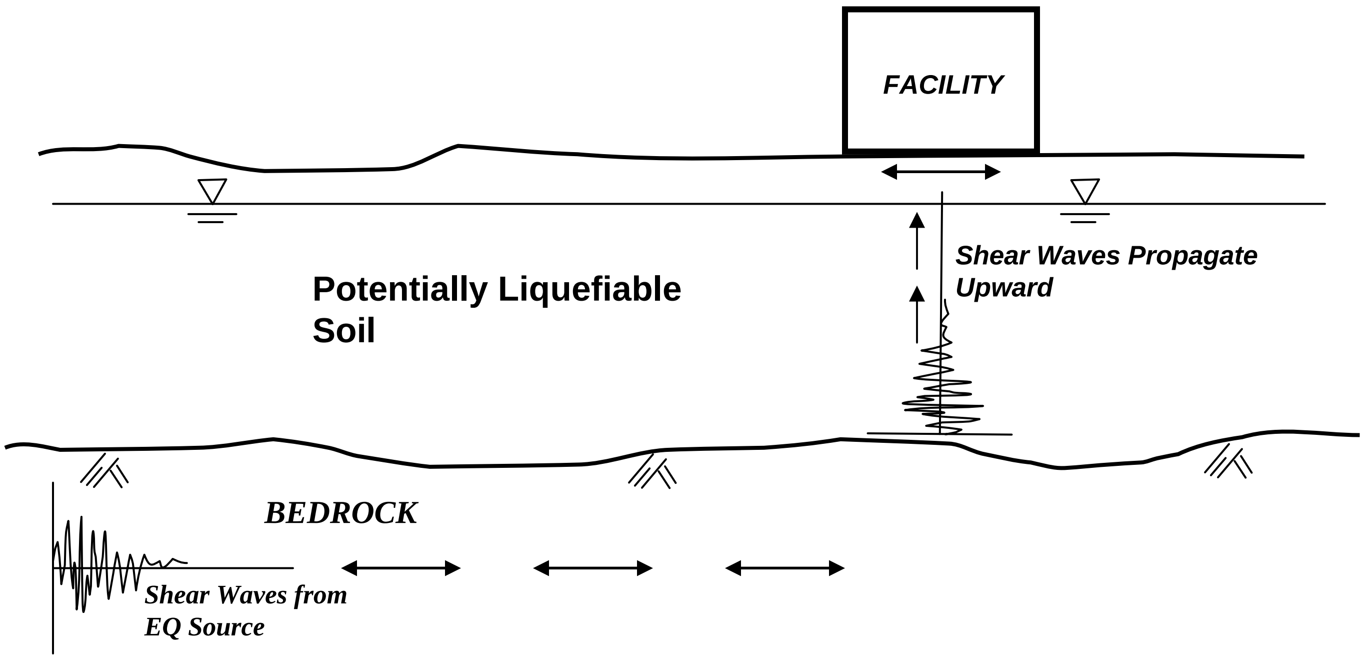
- In the 1994 Northridge earthquake, homes damaged by liquefaction or ground failure were 30 times more likely to require demolition than those homes only damaged by ground shaking (ABAG)
- In the 1995 Kobe Japan Earthquake, significant damages occurred to port facilities due to liquefaction; after almost 10 years post trade still 10-15% off

Key Reference

Youd et al. 2001. "Liquefaction Resistance Of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironmental Engineering,* October, pp. 817-833.

Liquefaction Analysis

Saturated loose sands, silty sands, sandy silts, nonplastic silts, and some gravels are susceptible to liquefaction in an earthquake.



Liquefaction Analysis

- A quantified measure of seismically induced shaking within a soil profile is termed the earthquake demand. The most commonly used measure of demand in current practice is the c*yclic stress ratio* (**CSR**).

- The soil's ability to resist this shaking without liquefaction is determined by one or more methods, and is indicated by its *cyclic resistance ratio* (**CRR**).

Liquefaction Analysis Steps

Step 1 -- Estimate the maximum acceleration at the ground surface, a_{max}:

This can be obtained from: (a) an actual acceleration record from nearby; (b) from "attenuation" relationships that relate a_{max} to the earthquake magnitude and include the effects of soil directly; (c) from a site response analysis using a series of time histories (if this is done, CSR can be determined directly from the output); (d) soft soil amplification factors such as Idriss (1990); and (e) national seismic hazard maps.

Liquefaction Analysis

Step 2 -- Determine the cyclic shear stress ratio, CSR, according to:

$$CSR = \frac{\tau_{ave}}{\sigma_{vo}'} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma_{vo}'} r_d$$

in which

τ_{ave} = average cyclic shear stress

σ'_{vo} = vertical effective stress (total vertical stress minus the pore water pressure) at the depth of interest

σ_{vo} = total vertical stress at the depth of interest

g = acceleration due to gravity

r_d = depth reduction factor (see Figure 1)

Figure 1 – R_d vs. Depth

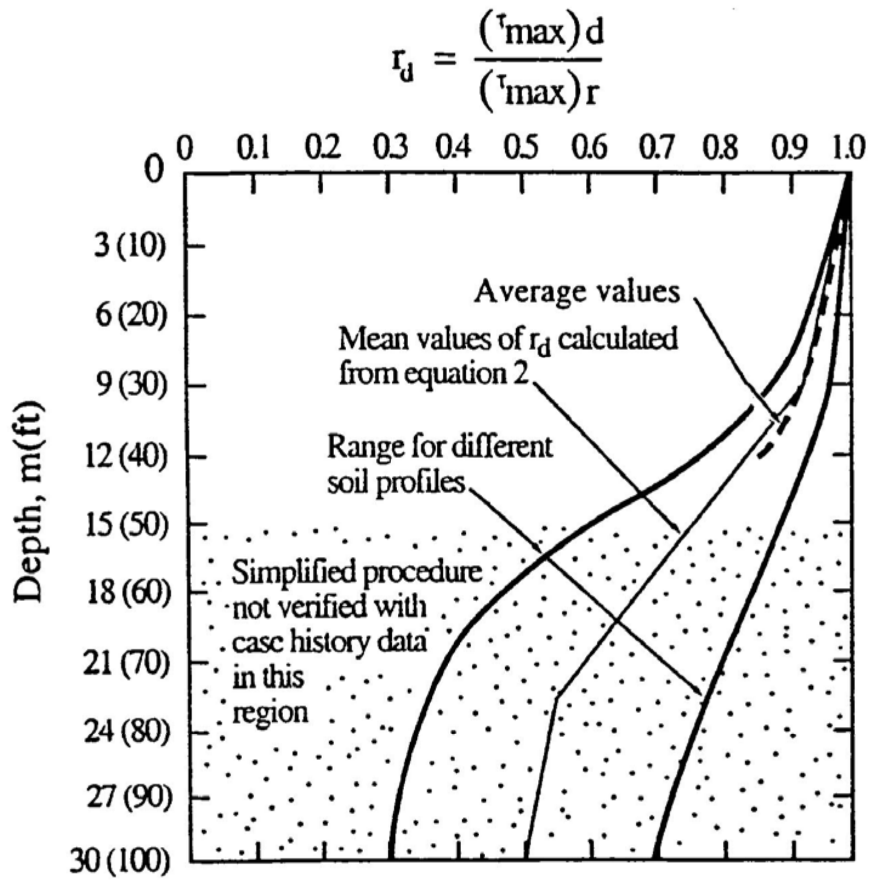


Fig. 1. r_d Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines Plotted from Eq. 2

Liquefaction Analysis

Step 3 -- Determine the soil resistance to liquefaction, CRR.

CRR can be determined from the results of Standard Penetration Tests (SPT) – see Figure 2, Cone Penetration Tests (CPT) – see Figure 3, or Shear Wave Velocity Measurements (V_s) - see Figure 4, may be used. Characteristics and comparisons of these test methods are given in Table 1.

$\Rightarrow$ The SPT N-value method is described here for level ground.

Figure 2- $N_{1,60}$ vs. CSR/CRR

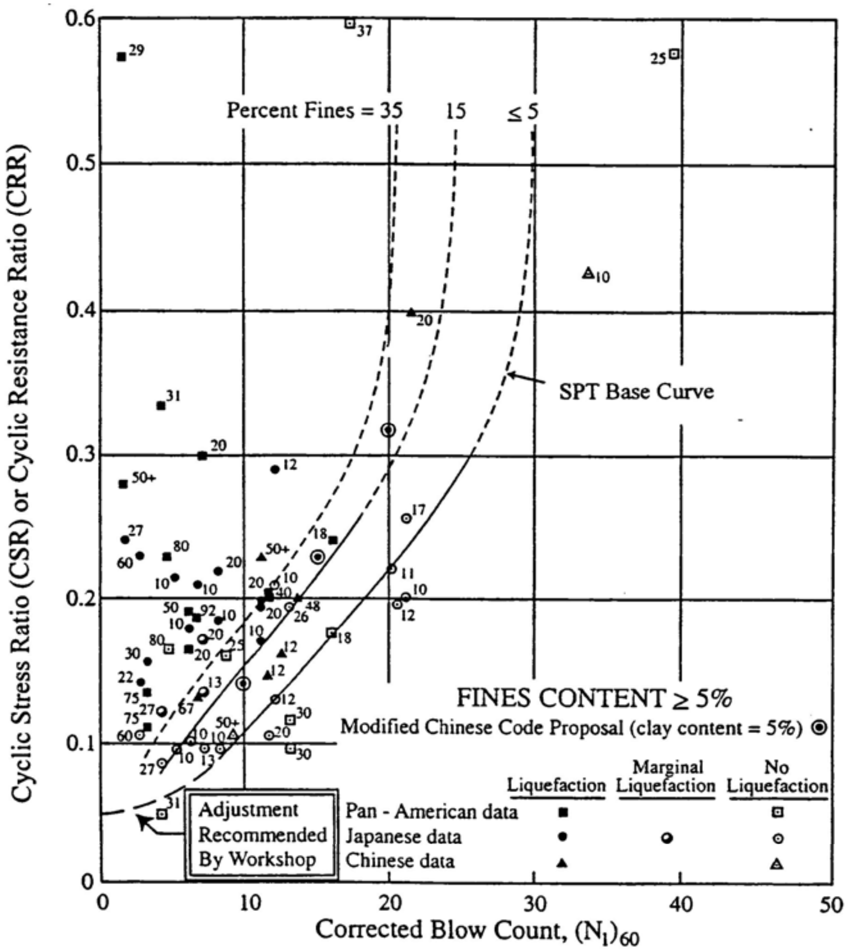


Fig. 2 . Base Curve for Magnitude 7.5 Earthquakes Recommended for Calculation of CRR from SPT Data with Data From Compiled Liquefaction Case Histories (modified From Seed et al., 1985)

Liquefaction Analysis

Step 4 -- Determine SPT N-values at several depths over the range of interest. These values must be corrected to account for depth (overburden pressure) and several other factors as listed in Table 2 to give the normalized penetration resistance $(N_1)_{60}$ which corresponds to a hammer efficiency of 60%.

$$(N_1)_{60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S$$

where:

N = measured penetration resistance, blows per foot

C_N = correction for overburden pressure = $(P_a/\sigma'_{vo})^{0.5}$

P_a = atmospheric pressure in same units as σ'_{vo} = 1 tsf, 100 kPa, 1 kg/cm^2

C_E = energy correction (see Table 2)

C_B = borehole diameter correction (see Table 2)

C_R = correction for rod length (see Table 2)

C_S = correction for sampling method (see Table 2)

Table 2. SPT Correction Factors

Factor	Test Variable	Term	Correction
Overburden Pressure[1]		C_N	$(P_a / \sigma_{vo}')^{0.5}$ $C_N \leq 1.7$
Energy Ratio	Donut Hammer Safety Hammer Automatic-Trip DonutType Hammer	C_E	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole Diameter	65 mm to 115 mm 150 mm 200 mm	C_B	1.0 1.05 1.15
Rod Length[2]	< 3 m 3 m to 4 m 4 m to 6 m 6 m to 10 m 10 m to 30 m > 30 m	C_R	0.75 0.8 0.85 0.95 1.0 >1.0
Sampling Method	Standard Sampler Sampler without Liners	C_S	1.0 1.1 to 1.3

Liquefaction Analysis

Step 5 -- Locate $(N_1)_{60}$ on Figure 2. If the earthquake magnitude is 7.5 and the depth of the point being evaluate corresponds to an effective overburden pressure of 1 tsf, 100 kPa, or 1 kg/cm^2, then the cyclic resistance ratio (CRR) is given by the corresponding value from the curve that separates the zones of liquefaction and no liquefaction (note that the appropriate curve to use depends on the fines content of the soil).

Liquefaction Analysis

Step 6 -- If the effective overburden pressure (σ'_{vo}) is greater than 1 tsf, 100 kPa or 1 kg/sq. cm, then the CRR should be reduced according to Figure 5 by:

$$(CRR)_{(\sigma' vo)} = (CRR)_{(\sigma' vo)=1} \times K_{\sigma}$$

If the earthquake magnitude is less than 7.5, then the CRR should be increased according to:

$$(CRR)_{M<7.5} = (CRR)_{M=7.5} \times MSF$$

The Magnitude Scaling Factor (MSF) is given by the shaded zone in Figure 6. Similarly, if the magnitude is greater than 7.5, then the CRR should be reduced according to the relationship in Figure 4.

Liquefaction Analysis

Step 7 --If the soil contains more than 5% fines, Fines content (FC corrections for soils with >5% fines may be made using (with engineering judgment and caution) the following relationships. $(N_1)_{60cs}$ is the clean sand value for use with base curve in Fig. 2.

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$$

$\alpha = 0$	for FC $\leq$ 5%
$\alpha = \exp[1.76 - (190/FC^2)]$	for 5% $\leq$ FC $\leq$ 35%
$\alpha = 5.0$	for FC $\geq$ 35%
$\beta = 1.0$	for FC $\leq$ 5%
$\beta = [0.99 + (FC^{1.5}/1000)]$	for 5% $\leq$ FC $\leq$ 35%
$\beta = 1.2$	for FC $\geq$ 35%

Liquefaction Analysis

Step 8 -- The factor of safety against liquefaction is defined by:

$$\boxed{FS_{LIQ'N} = CRR/CSR}$$

Typically want FS >1.35 or so.

Table 1- Comparison of In Situ Tests

Feature	Test Type			
	SPT	CPT	V_s	BPT
Data base from past EQ's	Abundant	Abundant	Limited	Sparse
Type of stress-deformation in test	Partly drained, large strain	Drained, large strain	Small strain	Partly drained, large strain
Quality control, repeatability	Poor to good	Very good	Good	Poor
Detection of heterogeneity	Good if tests closely spaced	Very good	Fair	Fair
Most suitable soil types	Gravel free	Gravel free	All	Gravelly soil
Soil sample obtained	Yes	No	No	Possibly
Index value or property measured directly	Index	Index	Property	Index
Data suitable for theoretical interpretation/analysis	No	Yes	Yes	No

Figure 3 - CPT vs. CSR/CRR

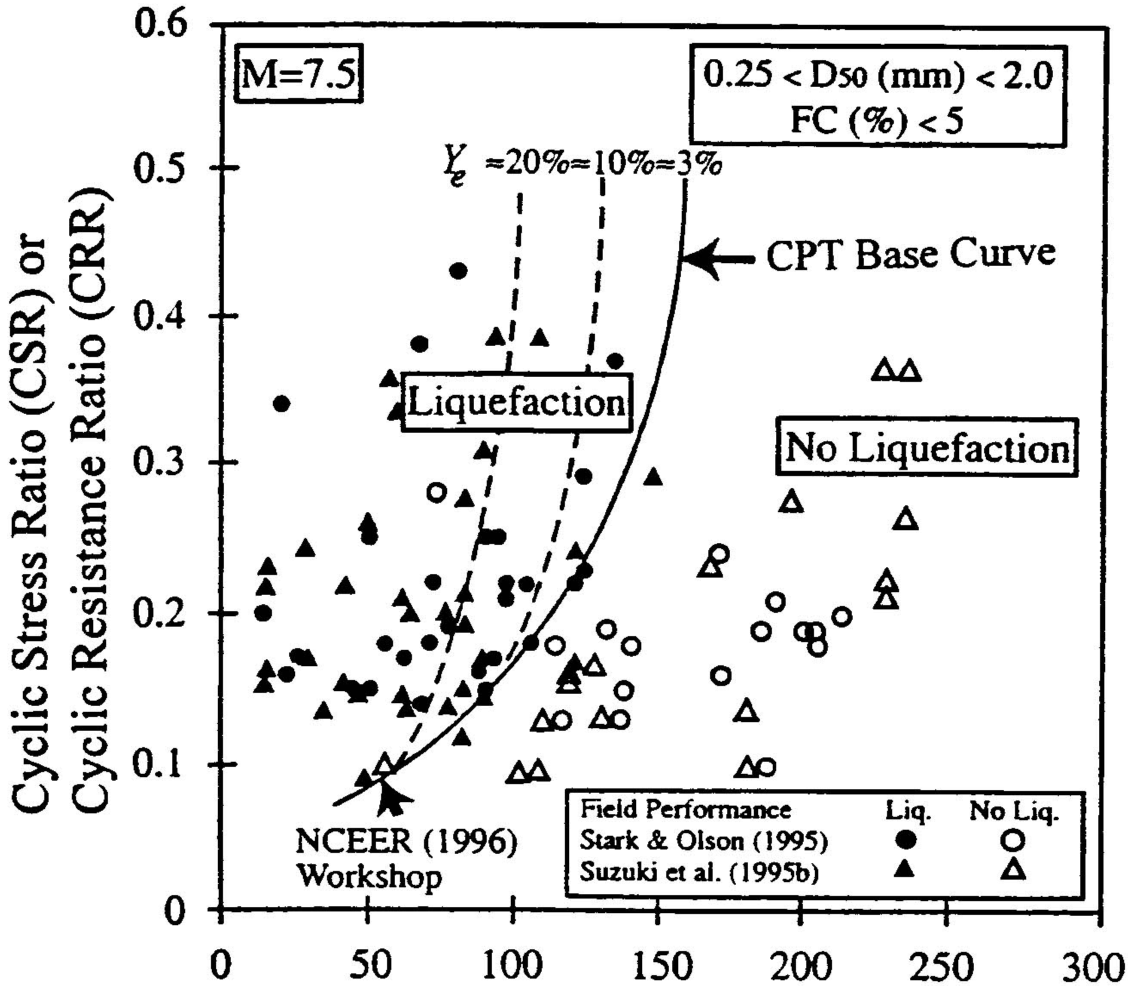


Figure 4 - Shear Wave Velocity

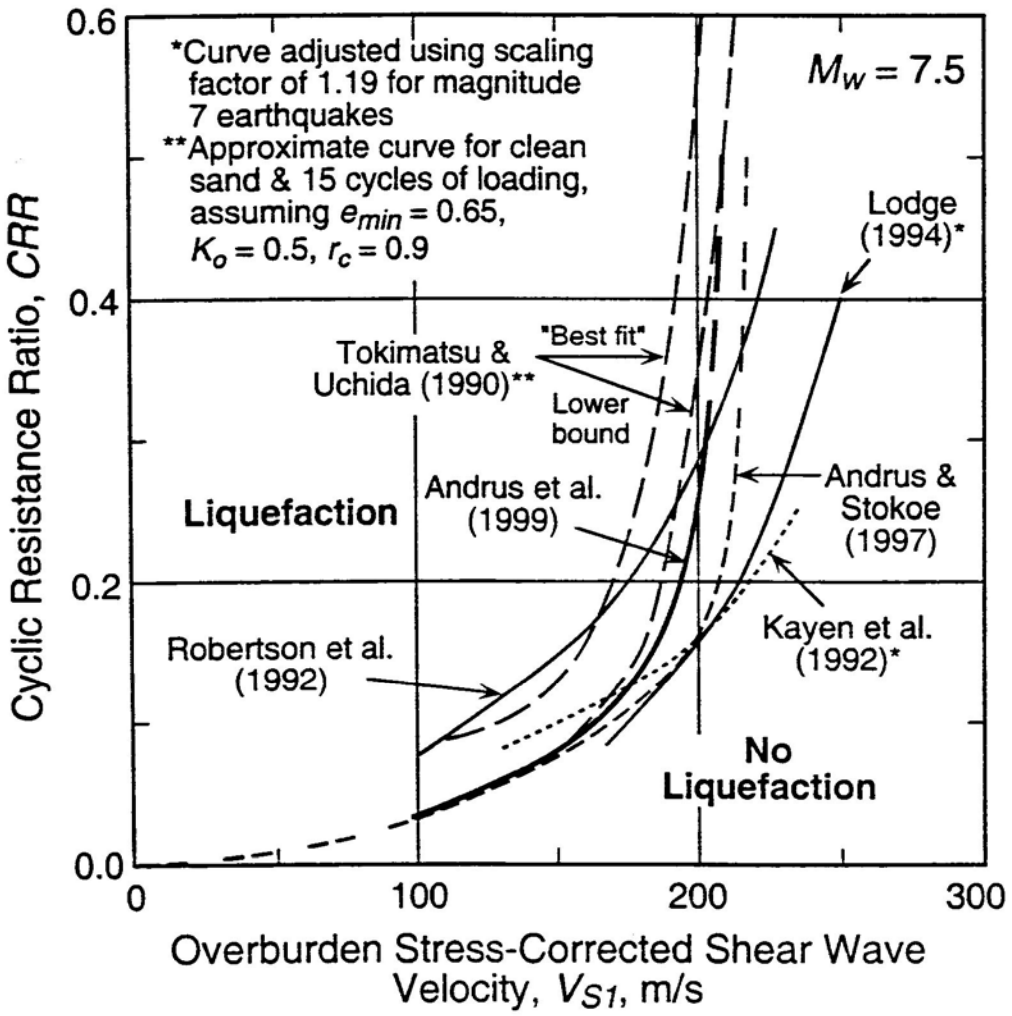


Figure 5 - Recommended Factors for Kσ

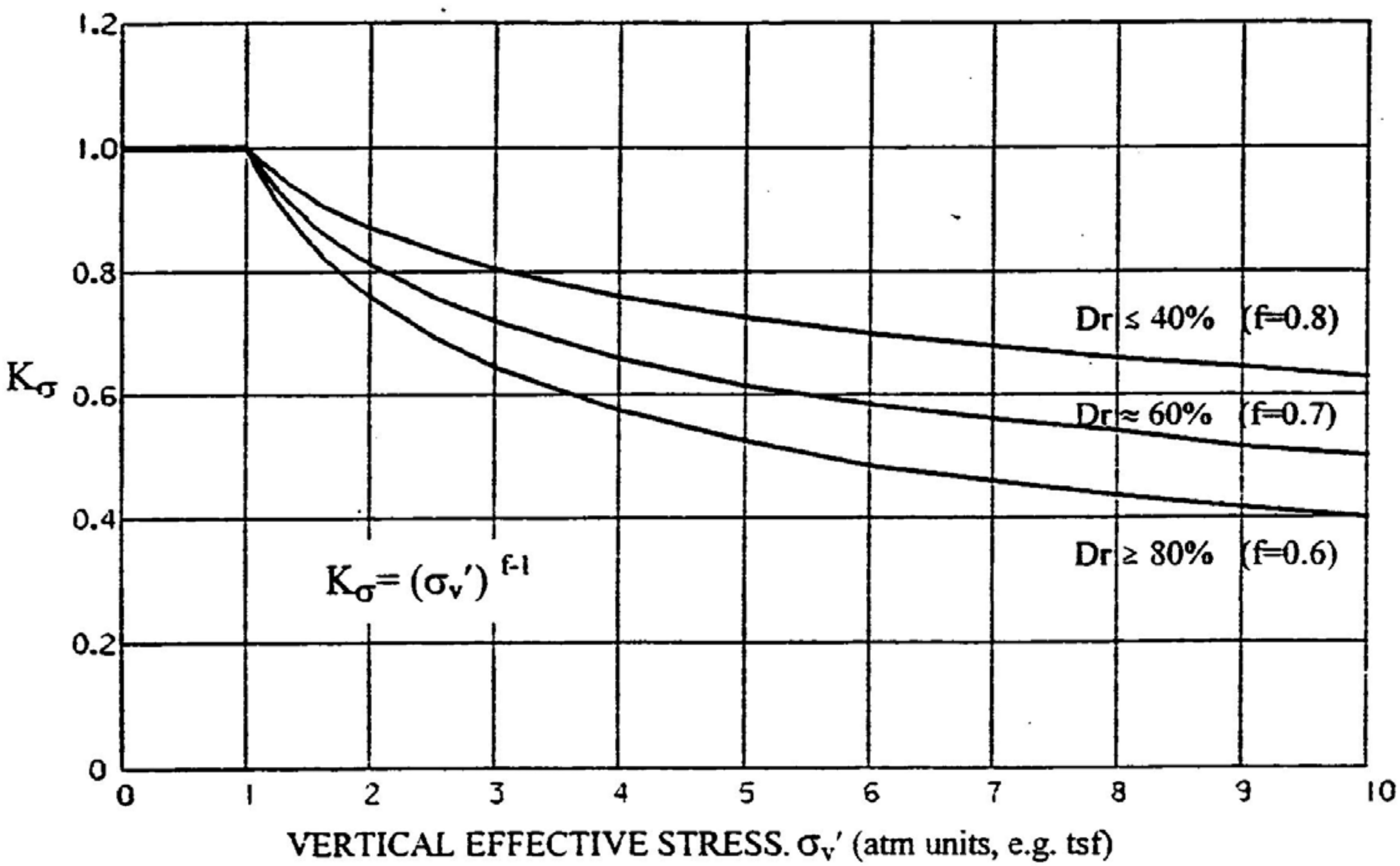
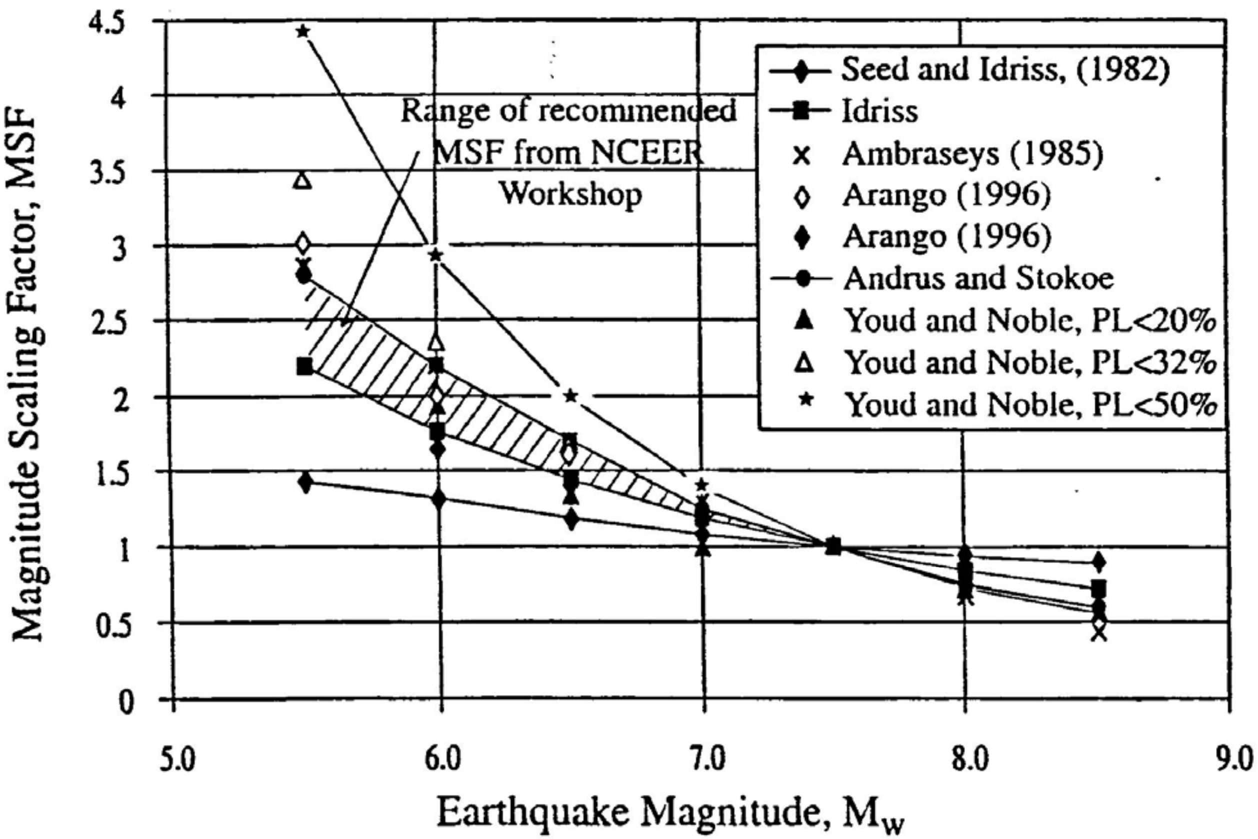


Figure 6 - Magnitude Scaling Factors



Soils With Plastic Fines: Chinese Criteria

Clayey Sands

Potentially liquefiable clayey soils need to meet all of the following characteristics (Seed et al., 1983):

- Percent finer than 0.005 mm < 15
- Liquid Limit (LL) < 35
- Water content > 0.9 x LL

If soil has these characteristics (and plot above the A-Line for the fines fraction to be classified as clayey), cyclic laboratory tests may be required to evaluate liquefaction potential. Recent work suggests latter two criteria work well to distinguish liquefiable soil, but the criterion of "percent finer than 0.005" does not match recent field experience (Martin et al., 2004).

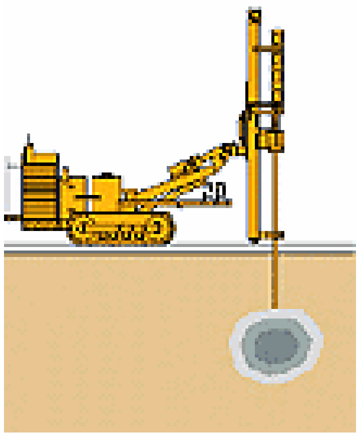
Liquefaction Remediation

- Basic approach is to either increase capacity (i.e., increase density, bind particles together), or decrease demand (i.e., soil reinforcement)
- Recent studies indicate cost/benefit ratio of liquefaction and site remediation is generally > 1.0
- Excellent summary of performance and techniques available from:

http://www.ce.berkeley.edu/~hausler/home.html

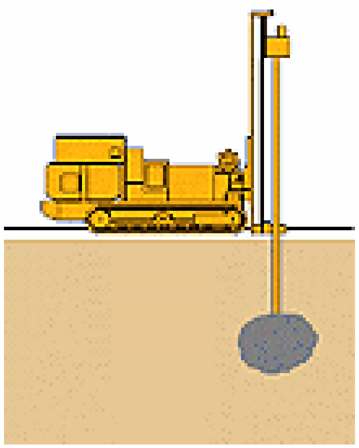
Liquefaction Remediation – Brief Summary

Source of following slides: http://www.haywardbaker.com/



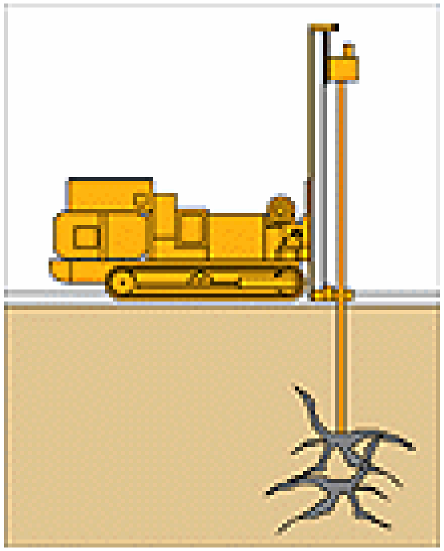
Compaction Grouting

When low-slump compaction grout is injected into granular soils, grout bulbs are formed that displace and densify the Surrounding loose soils. The technique is ideal for remediating or preventing structural settlements, and for site improvement of loose soil strata.

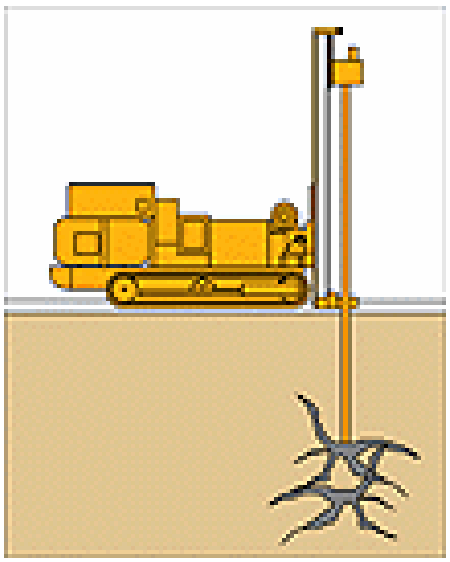


Chemical Grouting

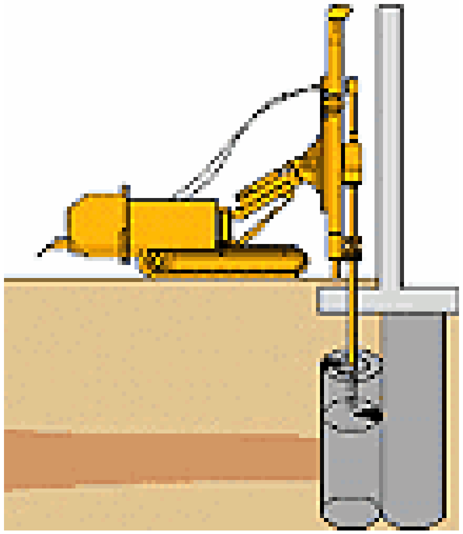
The permeation of very low-viscosity chemical grout into granular soil improves the strength and rigidity of the soil to limit ground movement during construction. Chemical grouting is used extensively to aid soft ground tunneling and to control groundwater intrusion. As a remedial tool, chemical grouting is effective in waterproofing leaking subterranean structures.



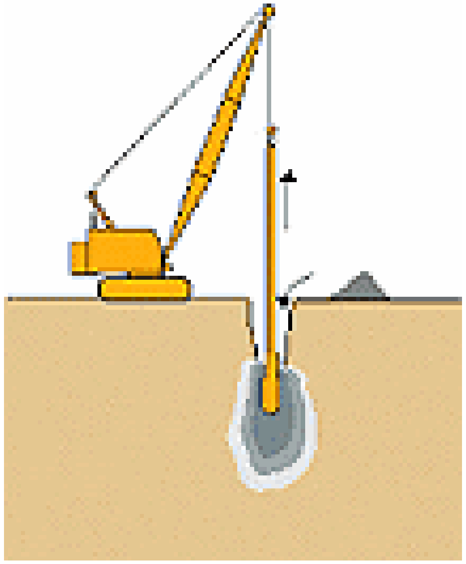
Cement Grouting Primarily used for water control in fissured rock, Portland and microfine cement grouts play an important role in dam rehabilitation, not only sealing water passages but also strengthening the rock mass. Fast-set additives allow cement grouting in moving water and other hard-to-control conditions.



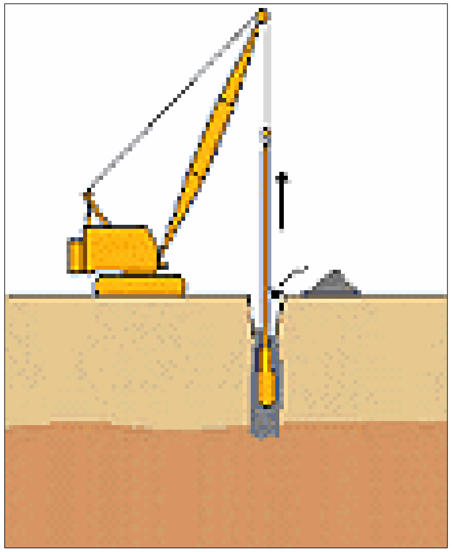
Soilfrac Grouting Soilfrac^{sm} grouting is used where a precise degree of settlement control is required in conjunction with soft soil stabilization. Cementitious or chemical grouts are injected in a strictly controlled and monitored sequence to fracture the soil matrix and form a supporting web beneath at-risk structures.



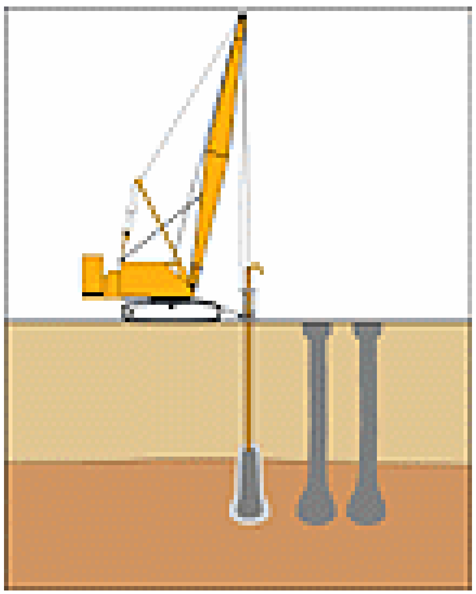
Jet Grouting Jet grouting is an erosion/replacement system that creates an engineered, in situ soil/cement product known as Soilcrete[sm]. Effective across the widest range of soil types, and capable of being performed around subsurface obstructions and in confined spaces, jet grouting is a versatile and valuable tool for soft soil stabilization, underpinning, excavation support and groundwater control.



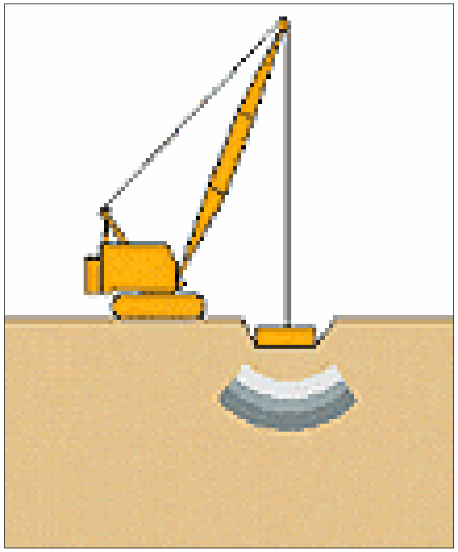
Vibro-Compaction A site improvement technique for granular material, Vibro-Compaction uses company-designed probe-type vibrators to densify soils to depths of up to 120 feet. Vibro-Compaction increases bearing capacity for shallow-footing construction, reduces settlements and also mitigates liquefaction potential in seismic areas.



Vibro-Replacement Related to Vibro-Compaction, Vibro-Replacement is used in clays, silts, and mixed or stratified soils. Stone backfill is compacted in lifts to construct columns that improve and reinforce the soil strata and aid in the dissipation of excess pore water pressures. Vibro-Replacement is well suited for stabilization of bridge approach soils, for shallow footing construction, and for liquefaction mitigation.

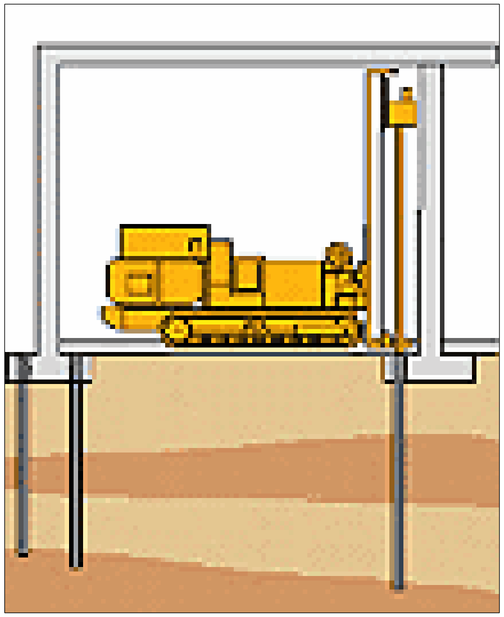


Vibro Concrete Columns Very weak, cohesive and organic soils that are not suitable for standard Vibro techniques can be improved by the installation of Vibro Concrete Columns. Beneath large area loads, Vibro Concrete Columns reduce settlement, increase bearing capacity, and increase slope stability.



Dynamic Deep Compaction Dynamic Deep Compaction™ is an economic site improvement technique used to treat a range of porous soil types and permit shallow, spread footing construction. Soils are densified at depth by the controlled impact of a crane-hoisted, heavy weight (15-35 tons) on the ground surface in a pre-determined grid pattern. Dynamic Deep Compaction is also successful in densifying landfill material for highway construction or recreational landscaping.

Soil Mixing Typically used in soft soils, the soil mixing technique relies on the introduction of an engineered grout material to either create a soil-cement matrix for soil stabilization, or to form subsurface structural elements to support earth or building loads. Soil mixing can be accomplished by many methods, with a wide range of mixing tools and tool configurations available.



Minipiles Underpinning of settling or deteriorating foundations, and support of footings for increased capacity are prime candidates for minipile installation, particularly where headroom is limited or access restricted. These small diameter, friction and/or end bearing elements can transfer ultimate loads of up to 350 tons to a competent stratum.

Extensive literature is available at the Hayward Baker Web-site: http://www.haywardbaker.com/

Vibrocompaction/Vibroreplacement

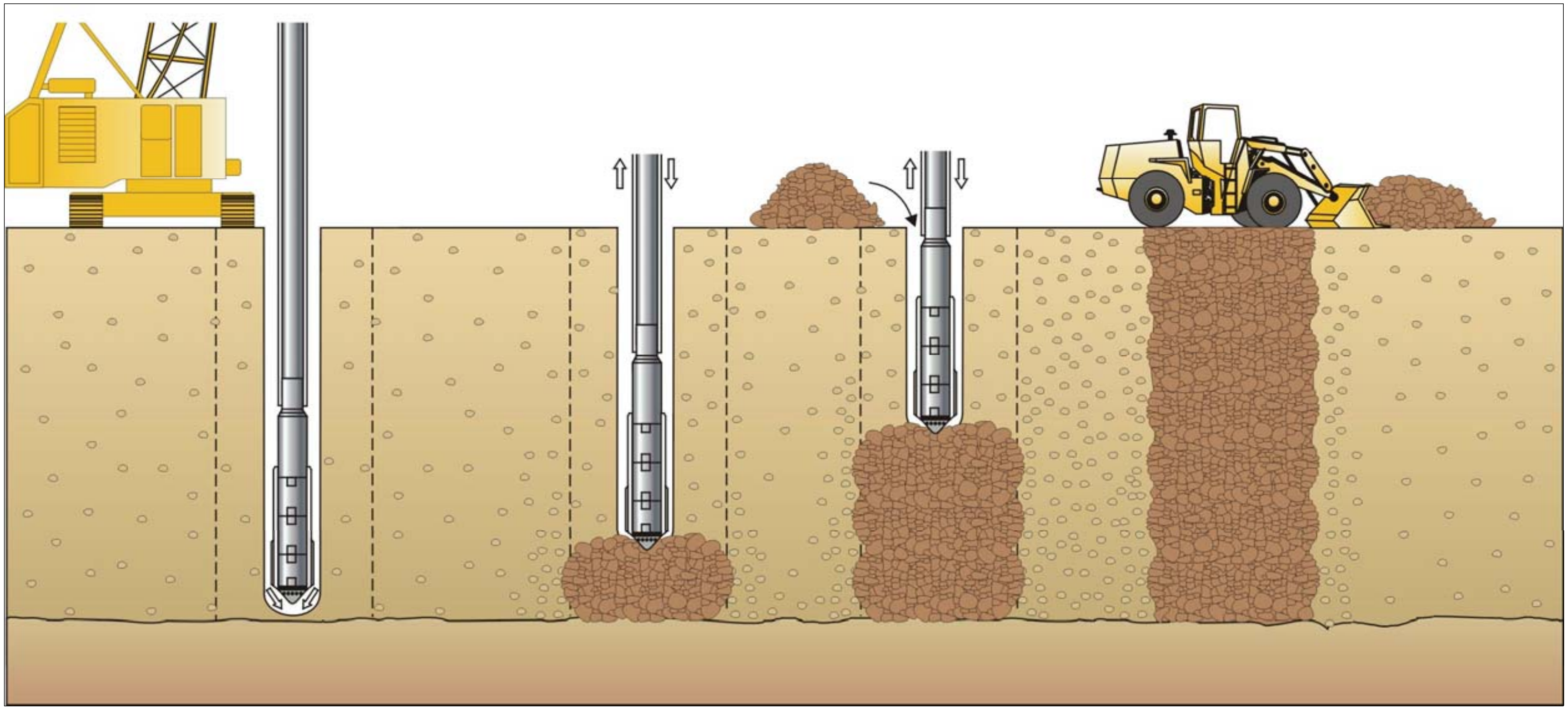
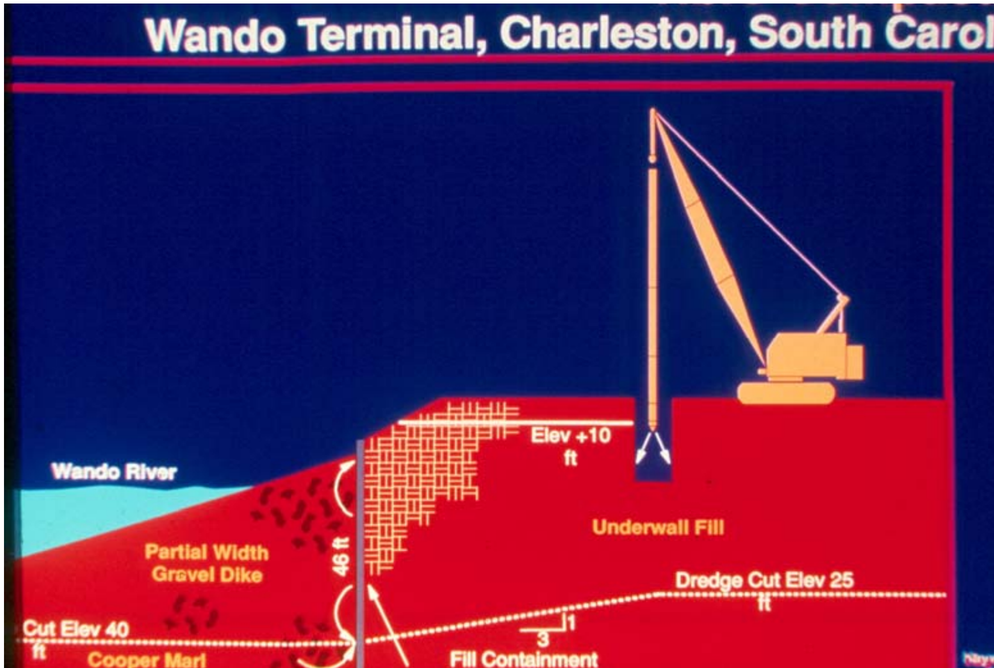


Figure adapted from
Hayward Baker, Inc.

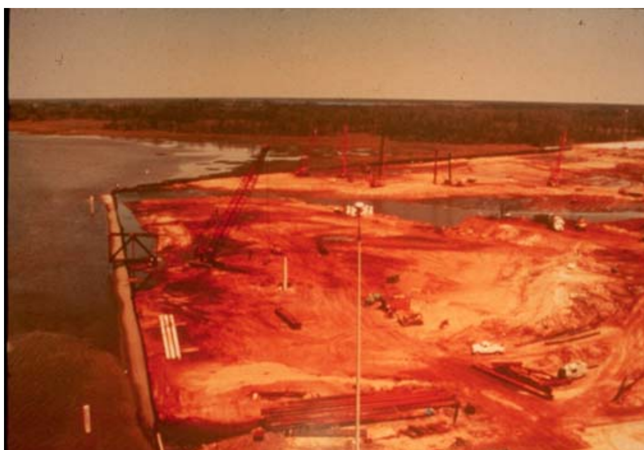
Vibrocompaction/Vibroreplacement

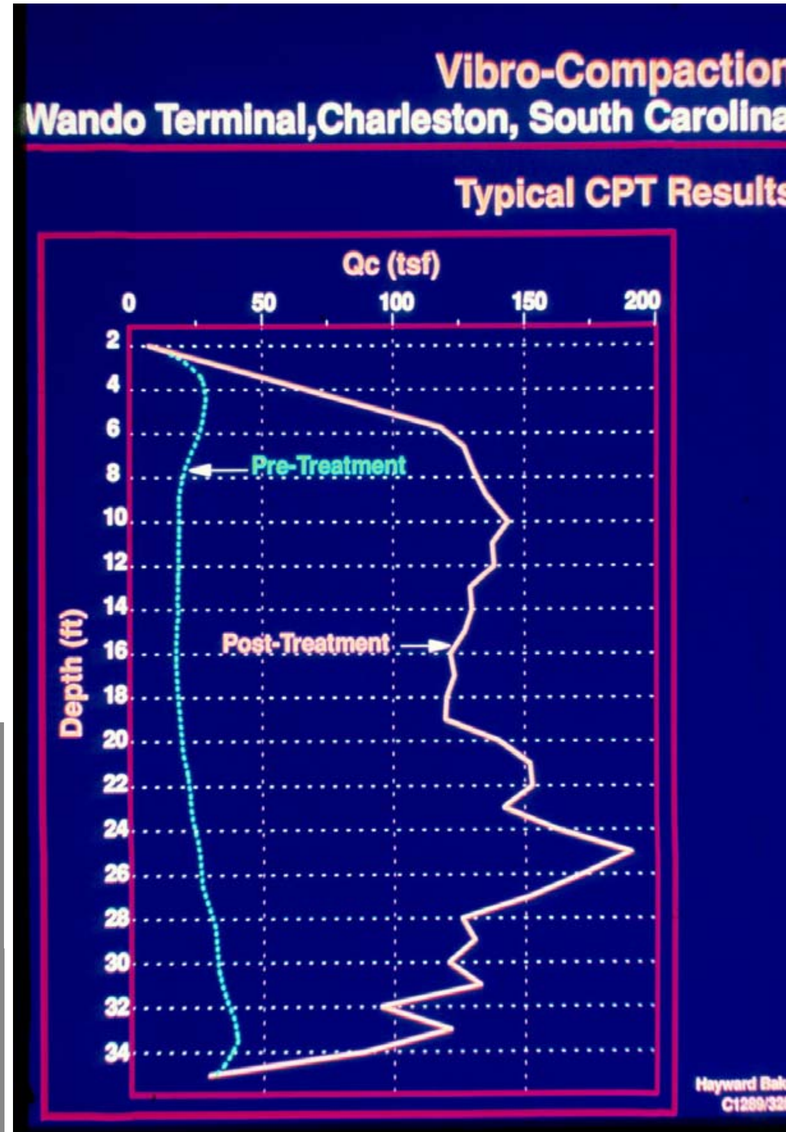
Vibroreplacement

Vibrocompaction in Charleston, SC



Photos adapted from Hayward Baker, Inc.





Deep Dynamic Compaction

Jet Grouting Systems

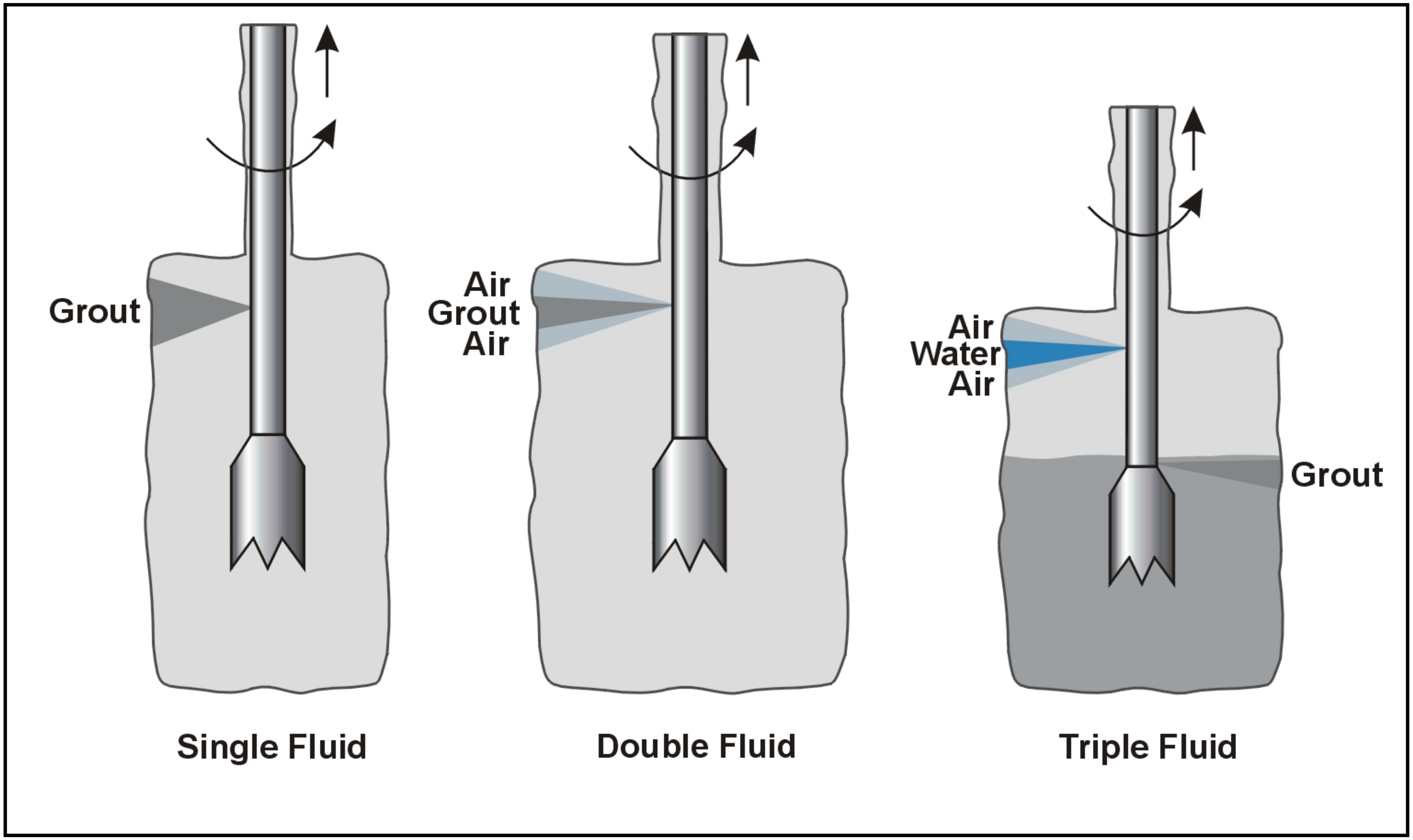


Figure adapted from Hayward Baker, Inc.

Jet Grouting Process

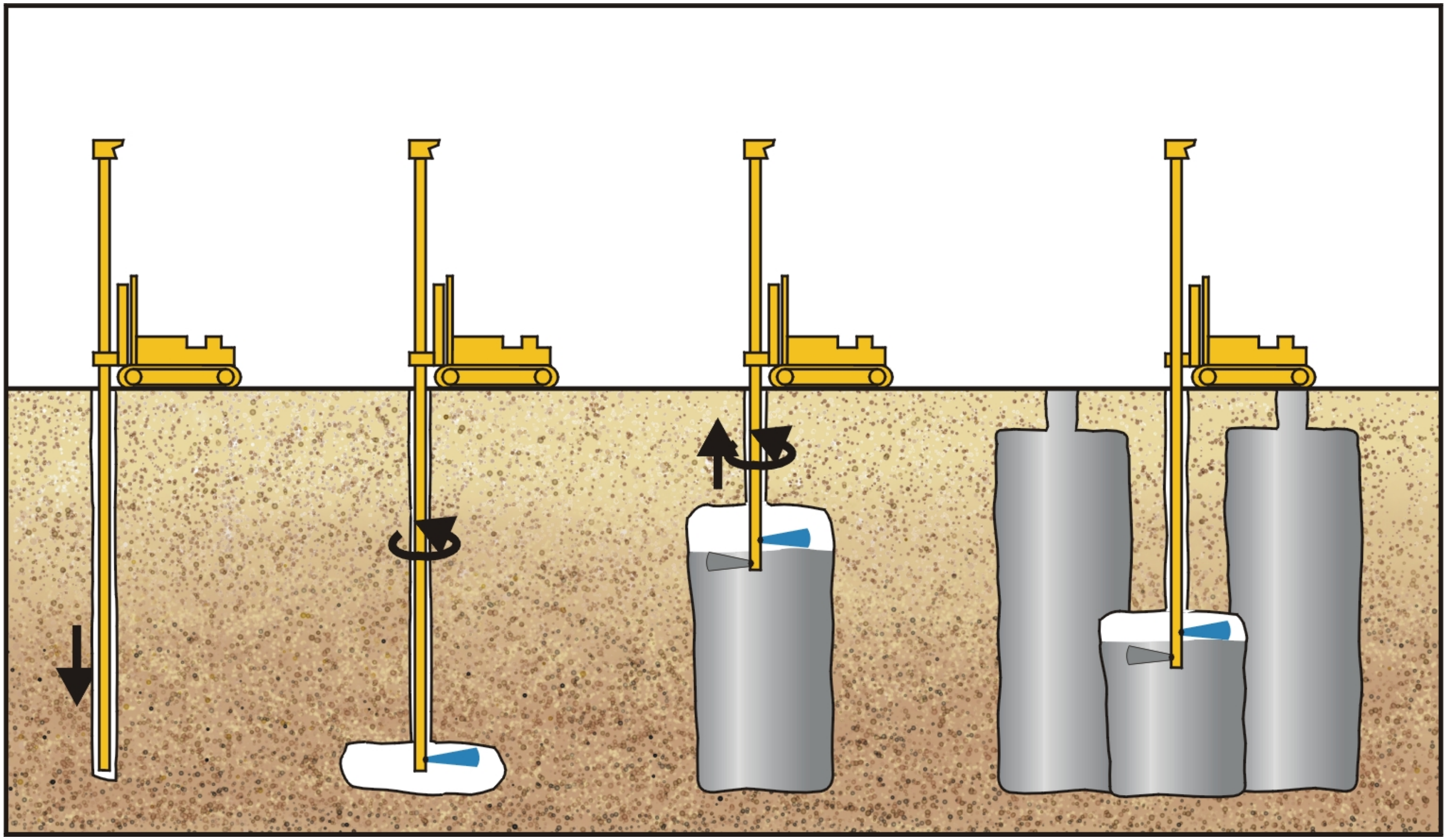
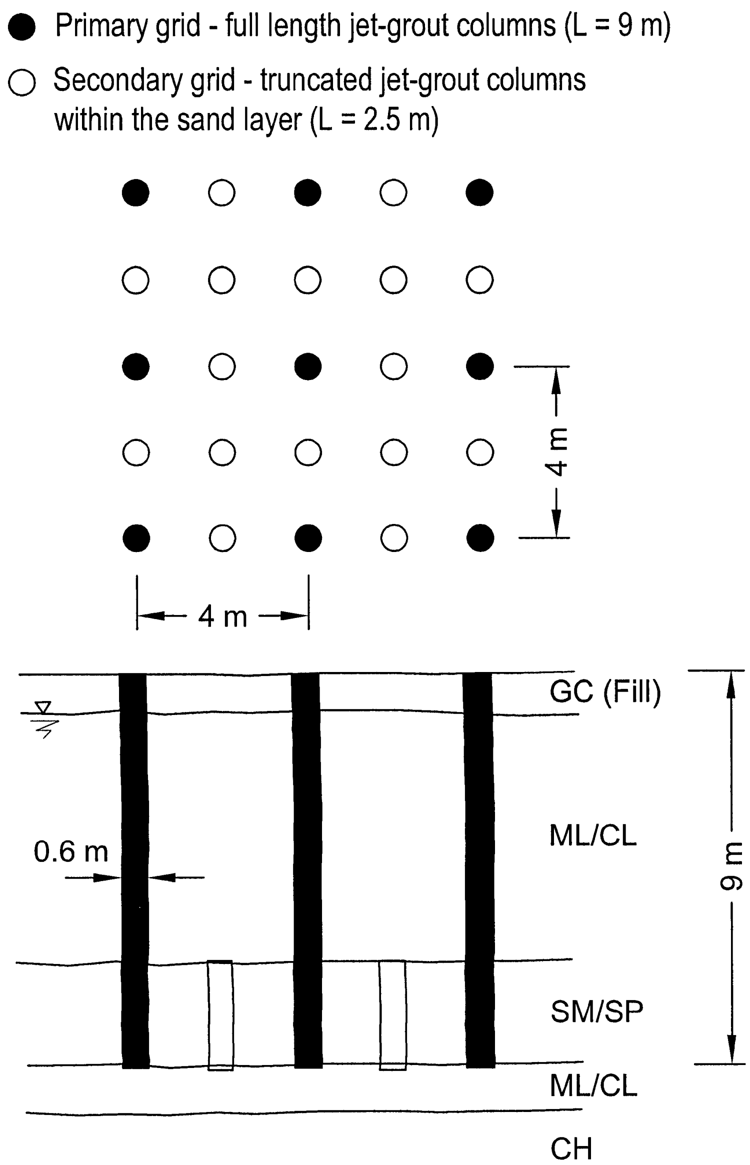


Figure adapted from Hayward Baker, Inc.

Jet Grouting for Liquefaction Mitigation



Jet Grouting Machine

Excavated Jet-Grout Columns

Deep Soil Mixing

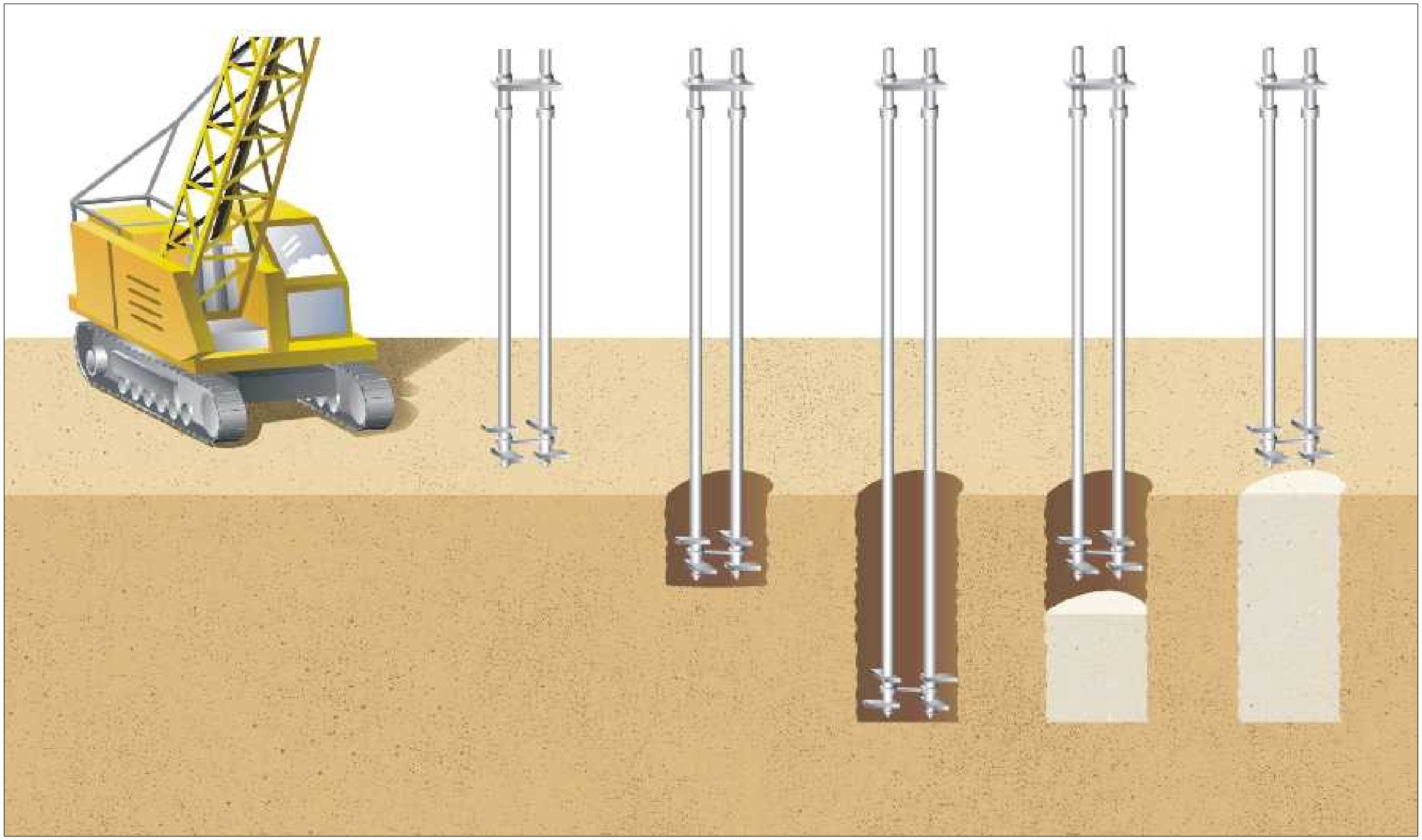
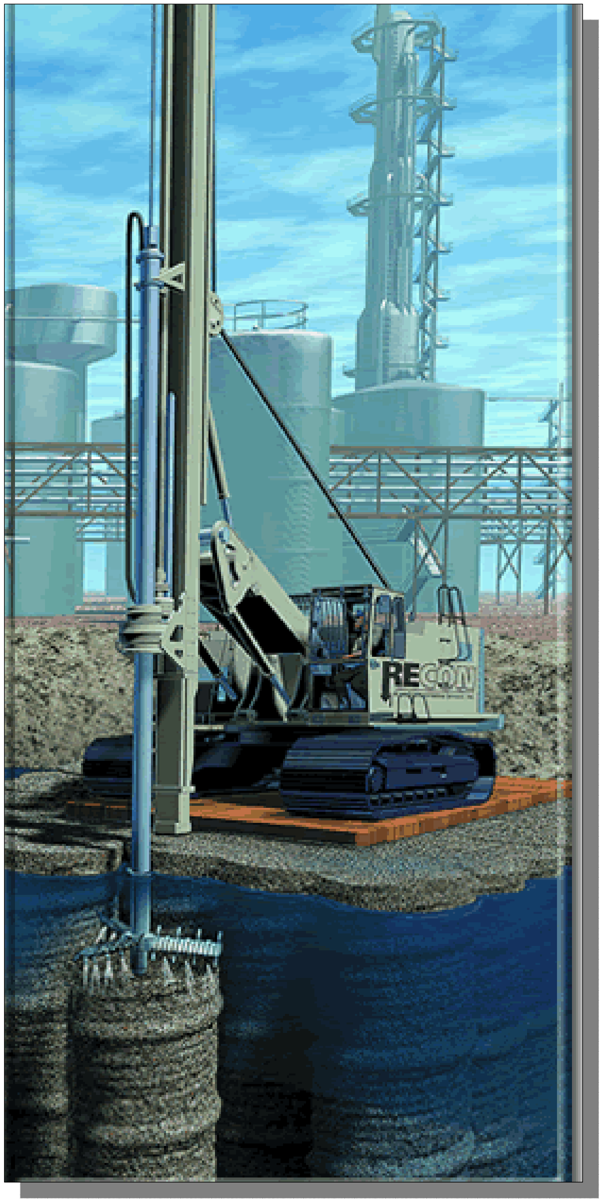


Figure adapted from Hayward Baker, Inc.

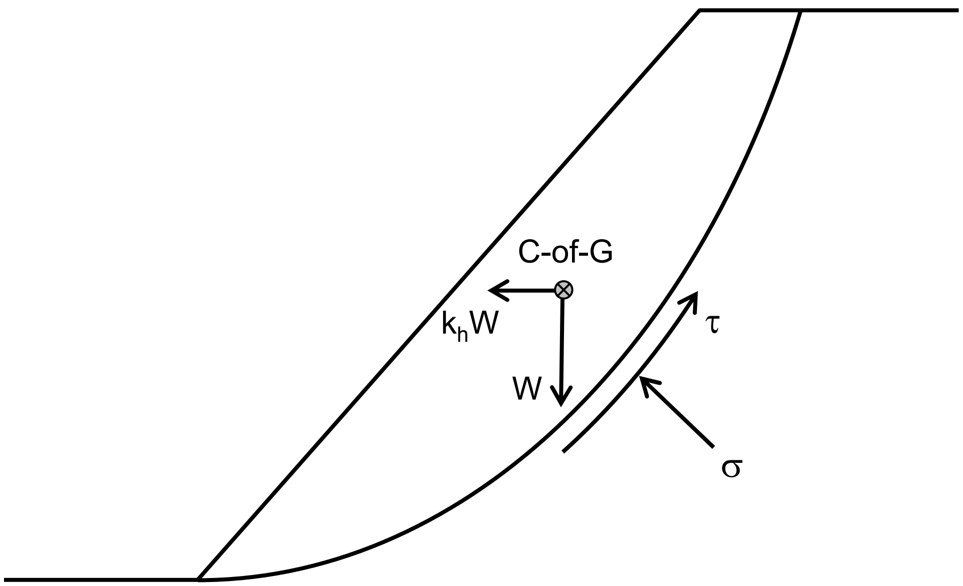
Deep Soil Mixing



Deep Soil Mixing

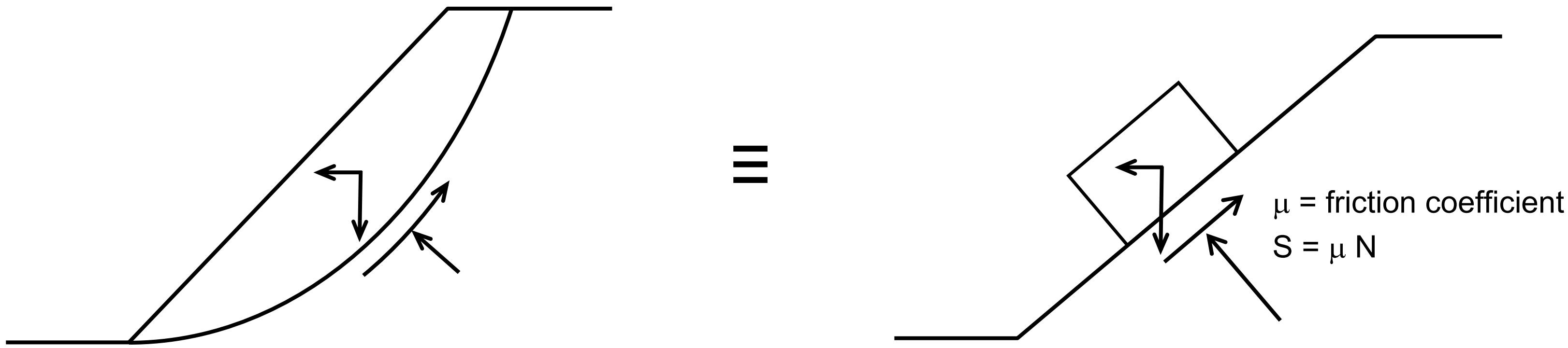
Slopes and Dams

Pseudostatic Analysis



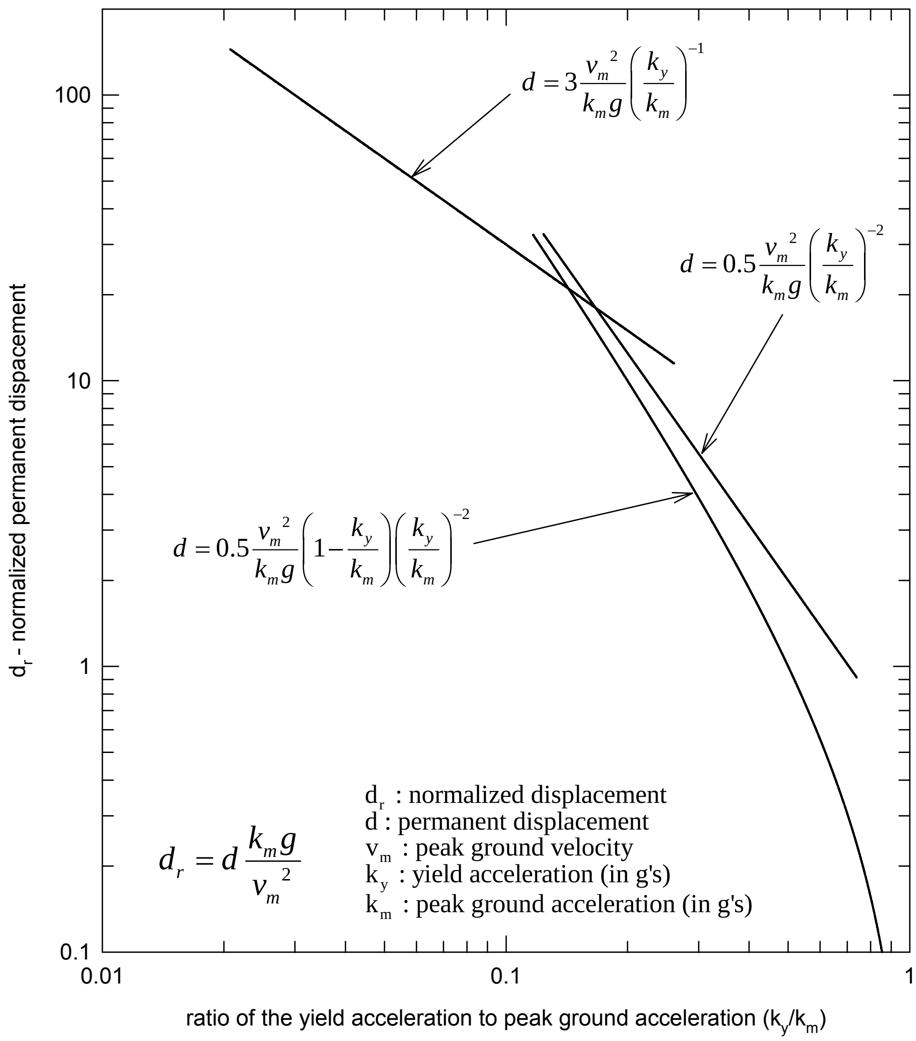
- stability is related to the resisting forces (soil strength) and driving forces (inertial forces)
- seismic coefficient (k_h) to represent horizontal inertia forces from earthquake
- seismic coefficient is related to PGA
- insufficient to represent dynamics of the problem

Displacement Analysis



- Estimate the acceleration (i.e. k_h) that would overcome the available friction and start moving the block down the plane – critical acceleration, yield acceleration
- Bracket the acceleration time history with yield acceleration in one direction (i.e. downward movement only), double integrate the portion of the acceleration history to estimate permanent displacement
- Or use simplified charts to relate permanent displacements to yield acceleration and peak ground acceleration

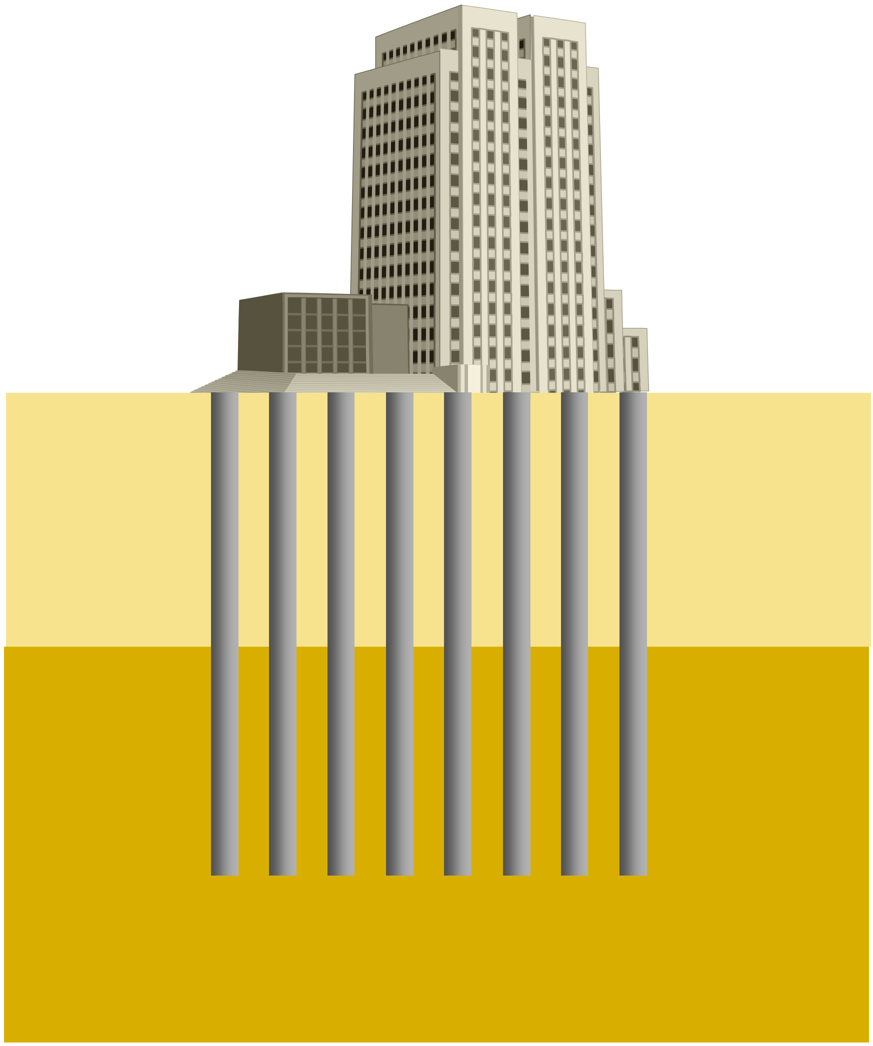
Displacement Analysis



Soil-Structure Foundation Interaction- SSFI

- Traditionally considered conservative to ignore (flexible foundations transmit less motion to superstructure, vice versa);
- However, recent studies from (i.e., 1995 Kobe, Japan EQ) suggest SSFI effects may actually increase ductility demand in some structures

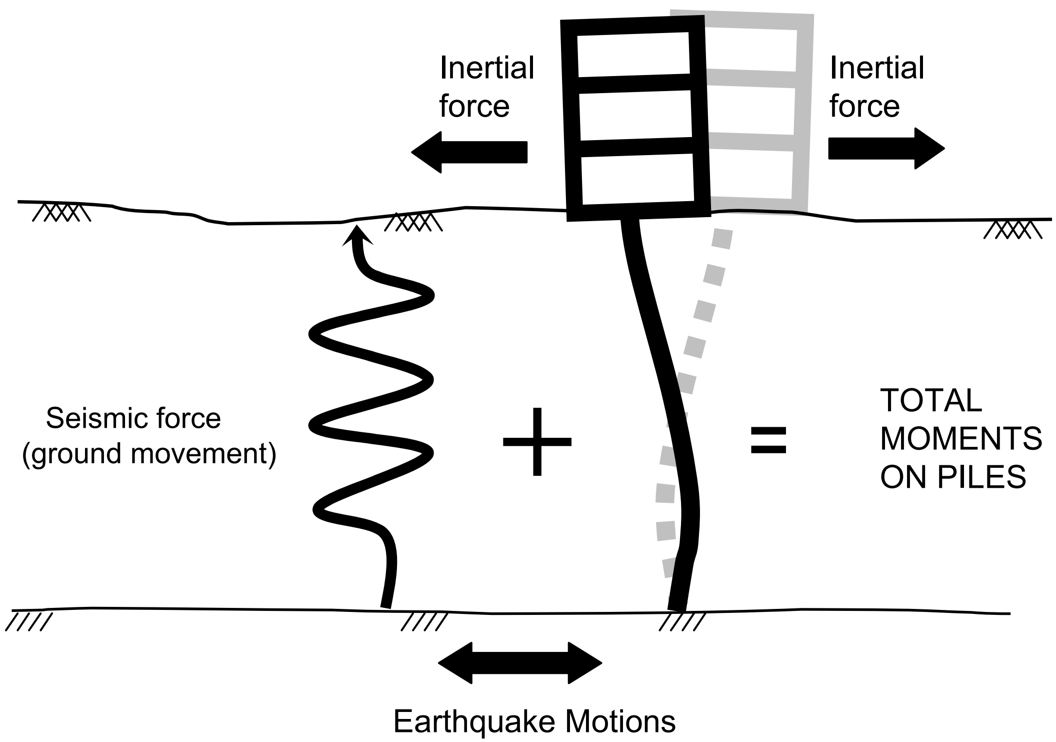
Seismic Design of Pile Foundations - SSFI



- The piles have to withstand forces due to the movement of the soil around and also inertial forces due to the building above

SSFI- Example: Earthquake Loadings on Piles

1. Seismic force;
2. Inertial force;
3. Soil failure (liquefaction, etc.)



Deep Foundations in Soft Soils

IBC 2003 Primary Geotechnical Issues

- Map-based procedure not ideally suited for geotechnical analyses

- Interpretation of soil categories not straight forward (i.e., What is “F” site?)

National Seismic Hazard Maps & IBC Issues for Geotechnical Use

- Maps generalized and not originally intended for site-specific analysis that account for the effects of local soil conditions, such as liquefaction.
- Map-based site classification procedure does not work as well for complex, layered soil profiles (site class based on average of top 30 m or 100 ft.)– think of 30 ft. of medium clay on top of hard rock– should this really be a "C" site?
- Modifications of ground motions for the effects of local soil conditions using the maps is not well-established
- Maps do not account for regional geology

National Seismic Hazard Maps & IBC Issues for Geotechnical Use

- Further away from original design intent, the fewer guidelines are available (structural engineer⇒ geotech engineer ⇒ seismologist)
- Maps developed mainly for structural design
- Earthquake magnitude/duration not provided directly, only pga's (M requires deaggregation)
- For structures with elastic response, duration is not as important per se
- Magnitude/duration is very important for most geotechnical analyses (non-linear behavior)

IBC 2003 Geotechnical Design Issues

- Provisions (Chap. 18) recommend $S_{DS}/2.5$ for liquefaction analysis $\Rightarrow$ SDS factored by 2/3, and 2/3 is from structural considerations, not soil-- this is inconsistent!!
- Structures can factor MCE by 2/3, but not soils $\Rightarrow$ new IBC Provisions affect geotechnical analyses more than structural analyses
- 20% limitation in reduction of map-based design motions based on site-specific analysis, but no simplified approach available for Class “F” sites $\Rightarrow$ leads to loophole.
- What is “F” site not always clear (i.e. “liquefaction”)

IBC Geotechnical issues

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				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
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.

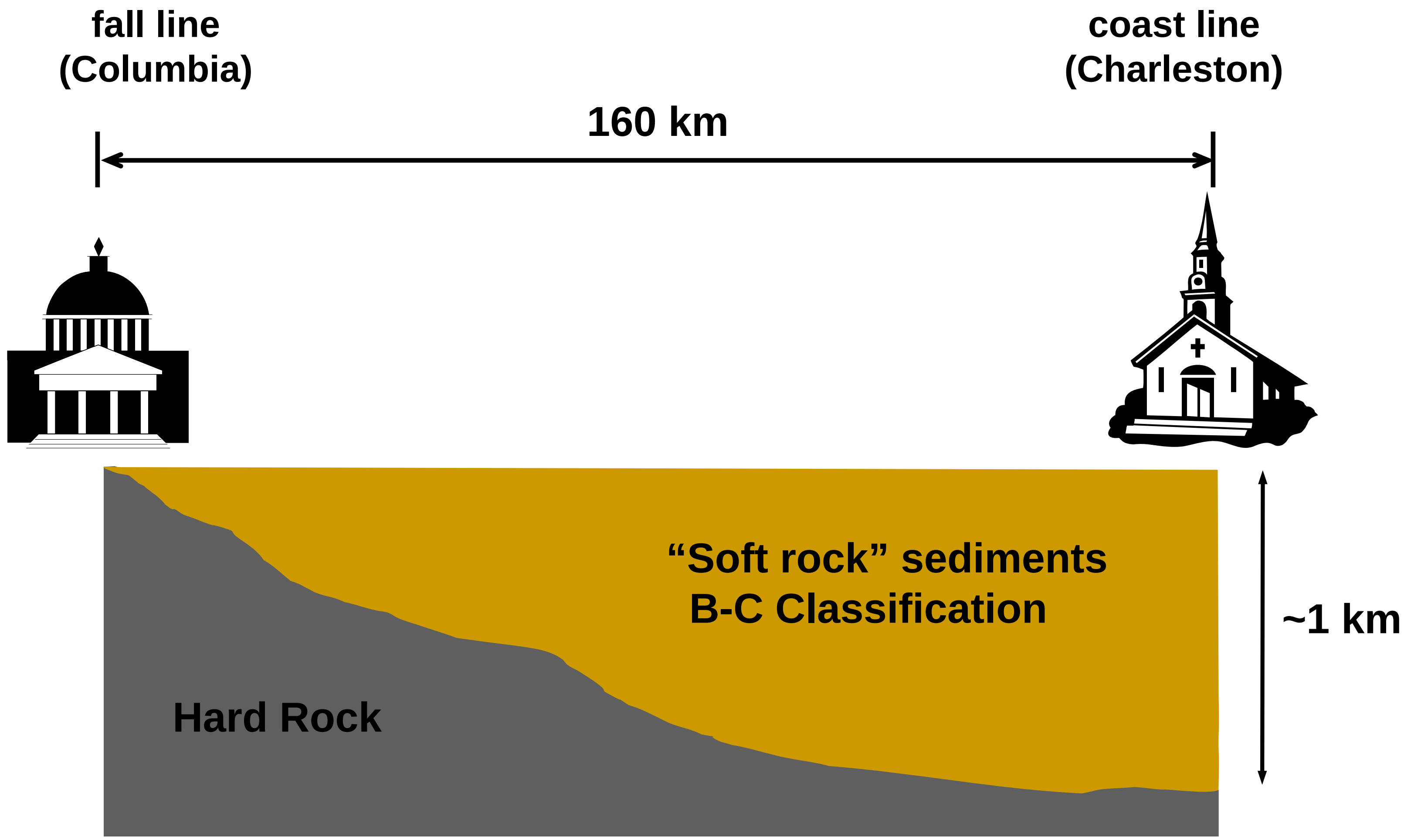
TABLE 1615.1.2(2)
VALUES OF SITE COEFFICIENT F_V AS A FUNCTION OF SITE CLASS
AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD (S_1)[a]

SITE CLASS	MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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.

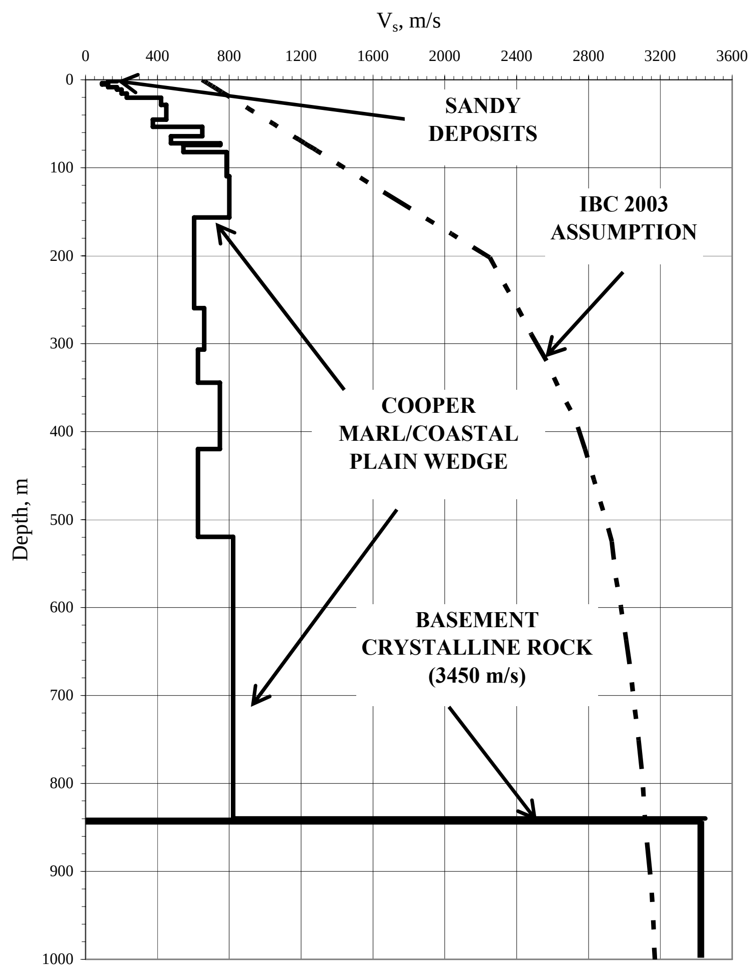
b. Site-specific geotechnical investigation and dynamic site response analyses shall be performed to determine appropriate values.

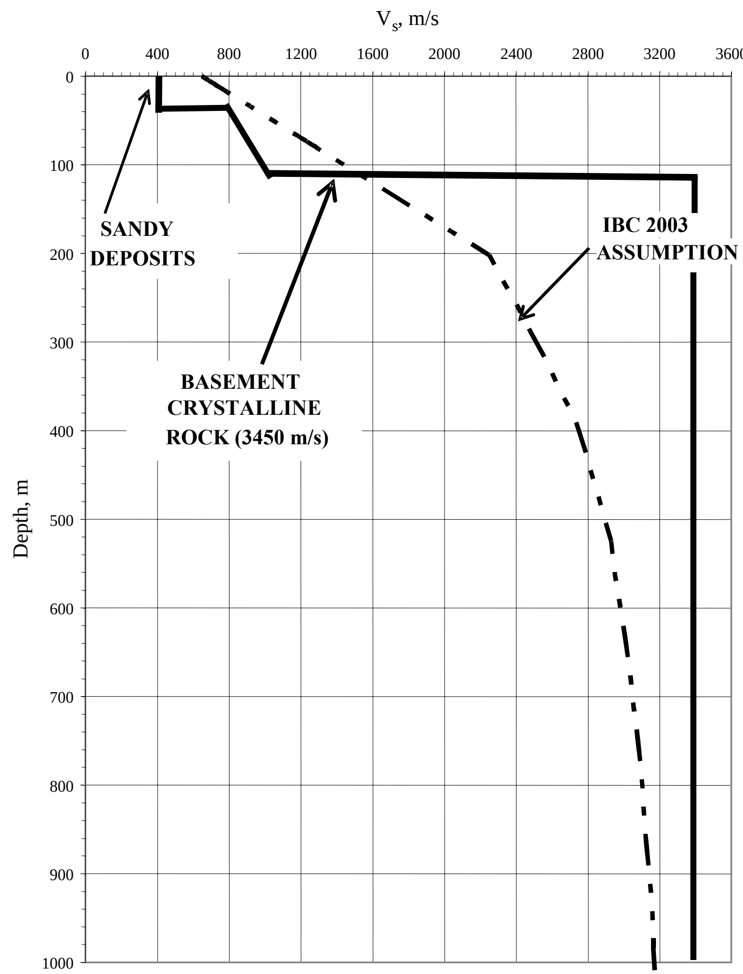
Example of Conditions Different from Those Assumed by Current USGS Maps



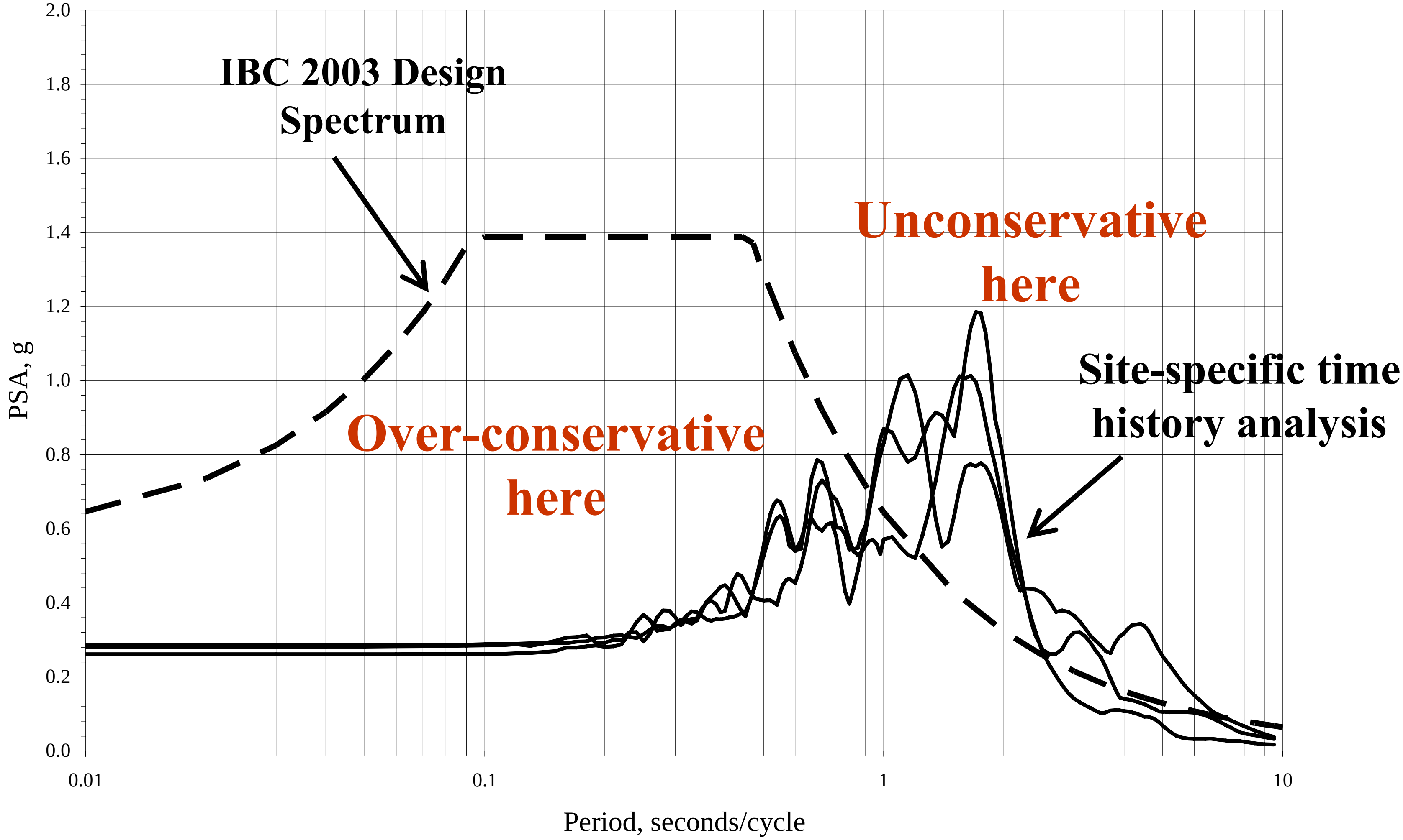
Charleston, SC

Columbia, SC

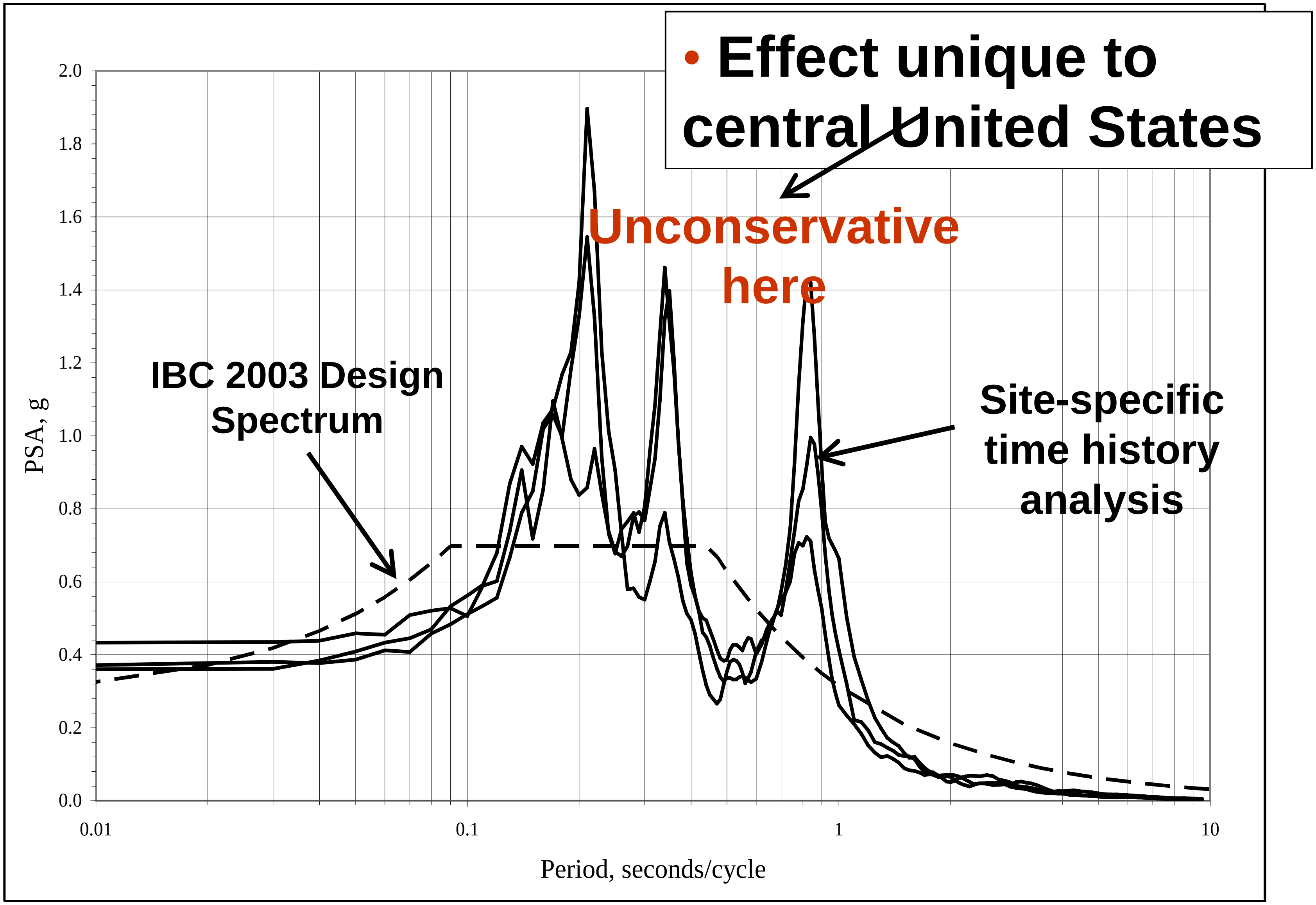




Charleston, SC, Response Spectra

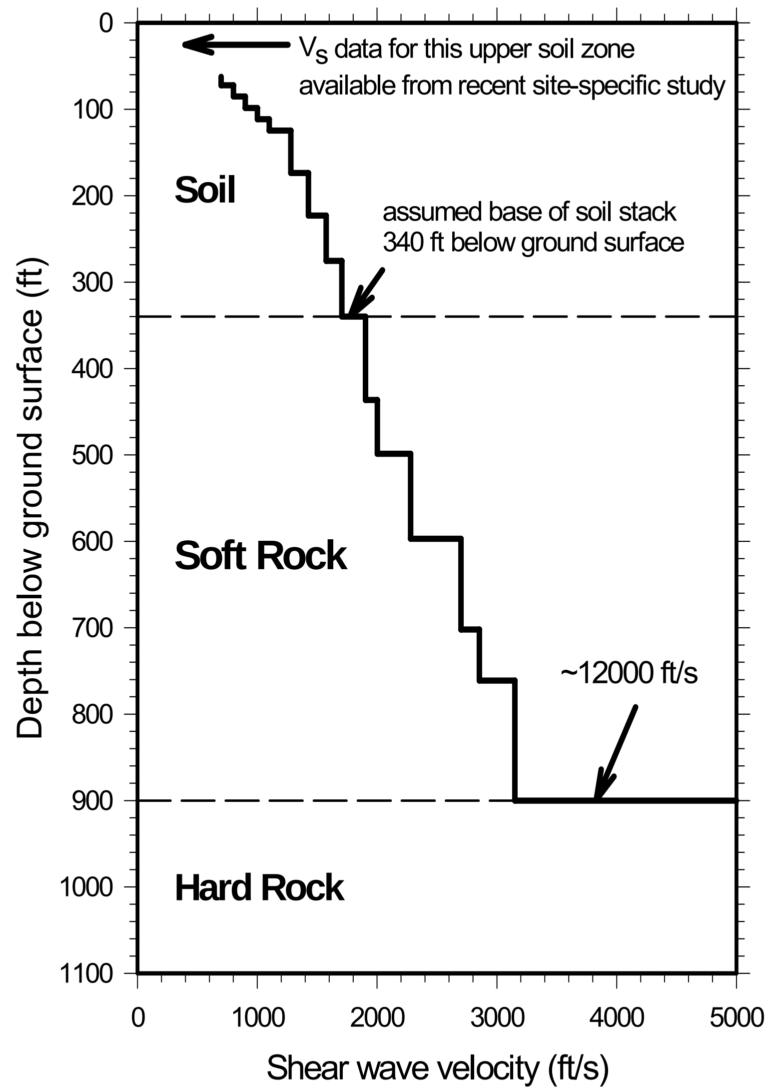


Columbia, SC, Response Spectra -- High Impedance

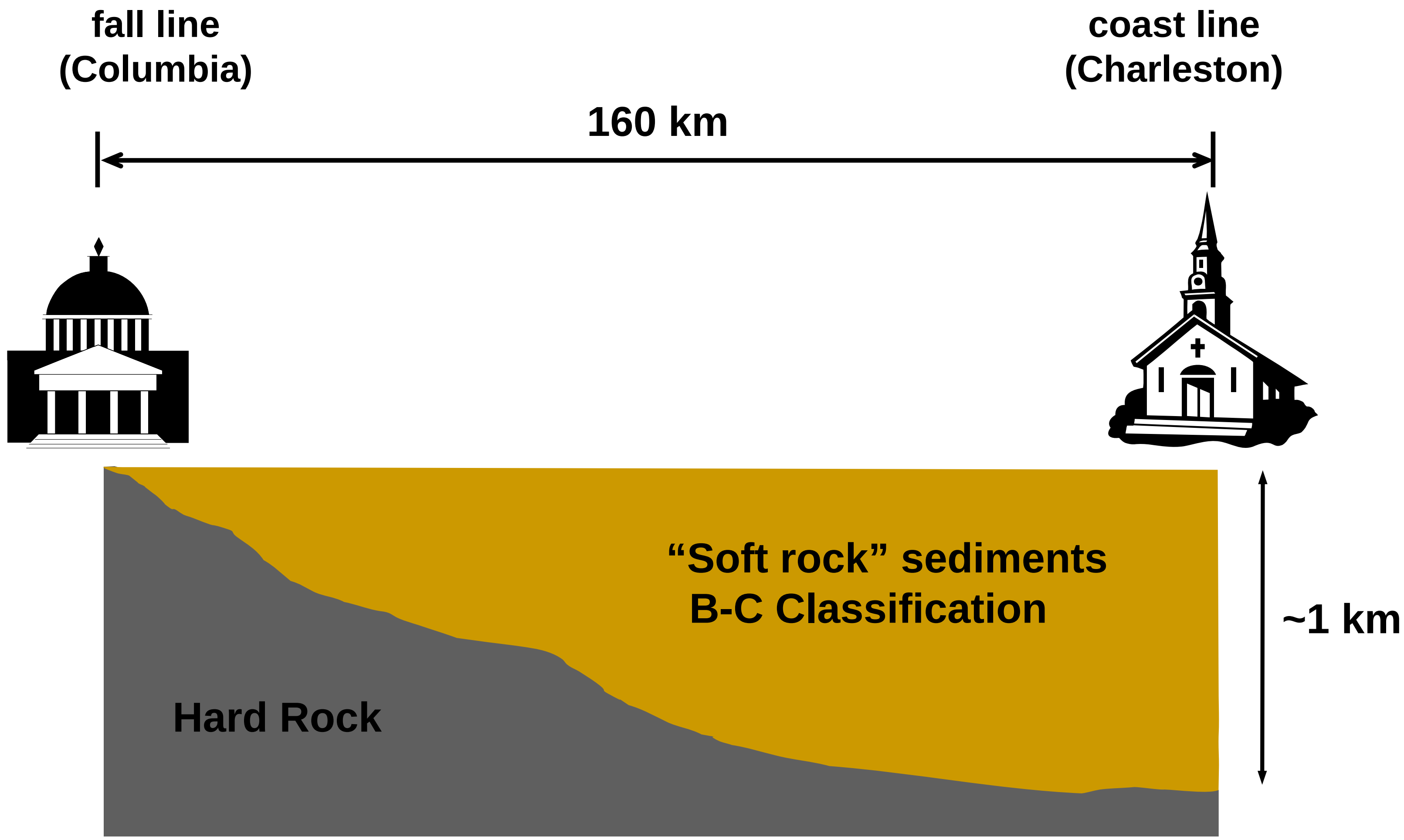


IBC 2003 Site-Specific Example

- **Typical South Carolina Coastal Plain Site**



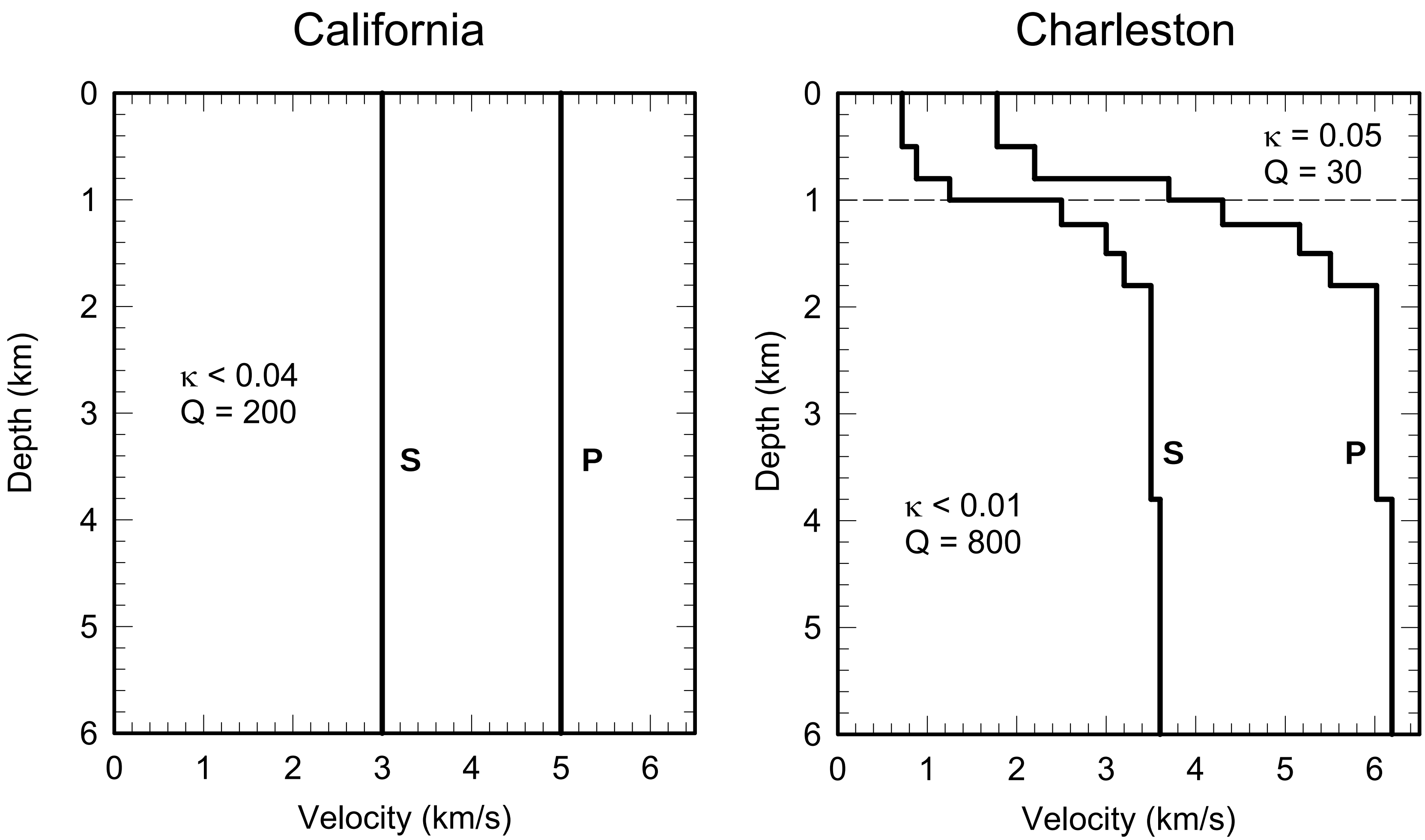
South Carolina Coastal Plain



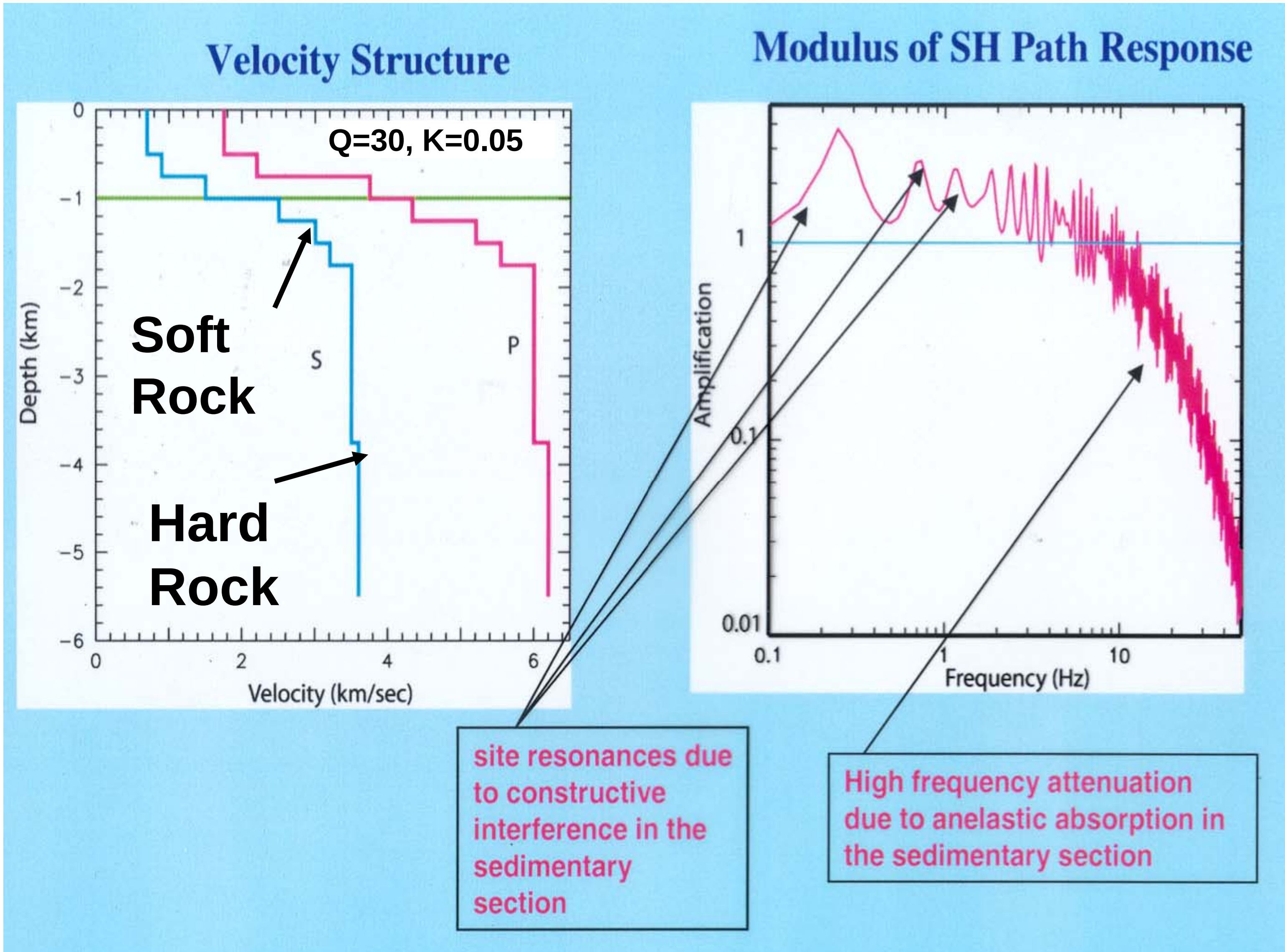
SC Coastal Plain Geology

- SC coastal plain sediments ("soft rock") difficult to characterize
- Q & κ (f of damping) are two big unknowns
- Sediments filter high frequencies and decrease peak motions
- "Effective" κ values in Eastern US soft rock similar to κ values for Western US hard rock
- "Soft rock" motions in coastal SC may be similar to Western US "hard" rock motions

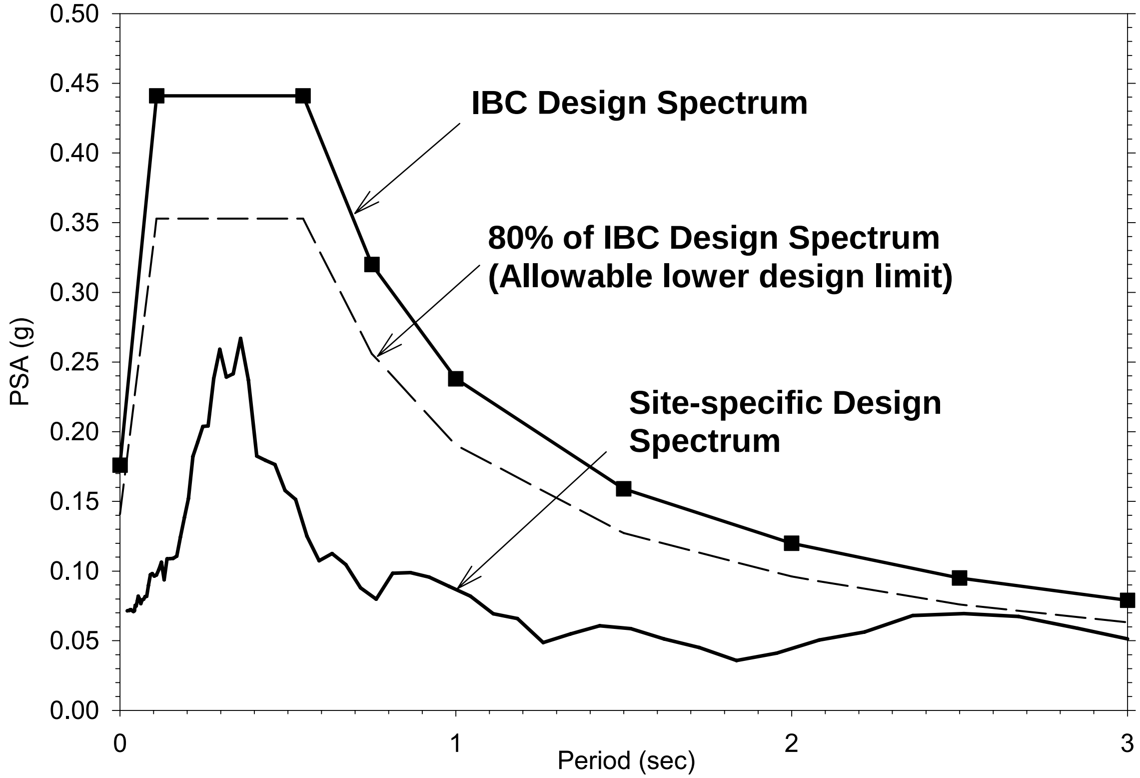
WUS vs. EUS Crustal Models



Effect of SC Coastal Plain on Ground Motions



Results of Site Specific Analysis*

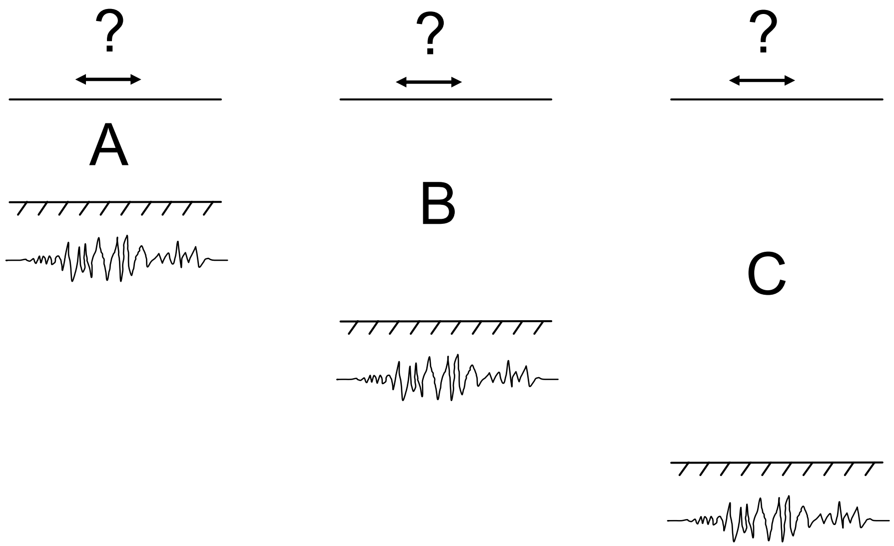


* Includes effect of coastal plain sediments plus near-surface soils in top 30 m. Plots developed for typical site in coastal SC

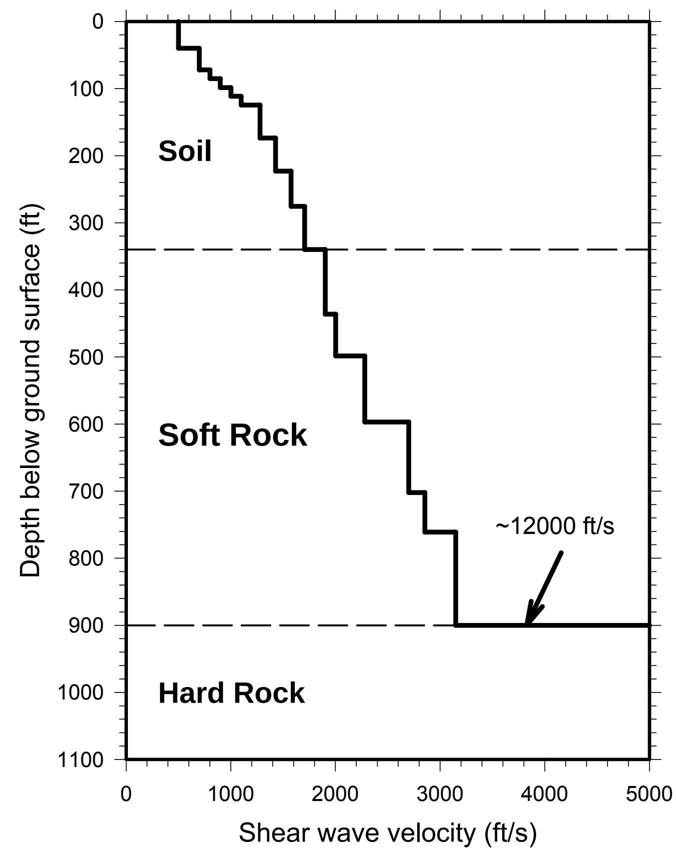
Special Comments on Site Response Analysis in CEUS

- Analysis techniques common in WUS, may not apply in many cases in CEUS
- Site response (i.e., SHAKE) analyses not as straight-forward in CEUS
- SHAKE has depth limitations (600 ft.? CEUS sites can be deeper)
- Where is halfspace? (Vs = 2000 ft/sec rule of thumb not always applicable in CEUS)

Where Is the Halfspace?



Typical EUS Site:



- Surface motions obtained from A, B, & C would be different, unless base motion modified for the different halfspace depths.
- Deeper profile is probably better to use, if base motion is appropriately developed and if damping is not too high.

A Final Point to Remember….

Relative PGAs in the United States

Soil is the great equalizer:

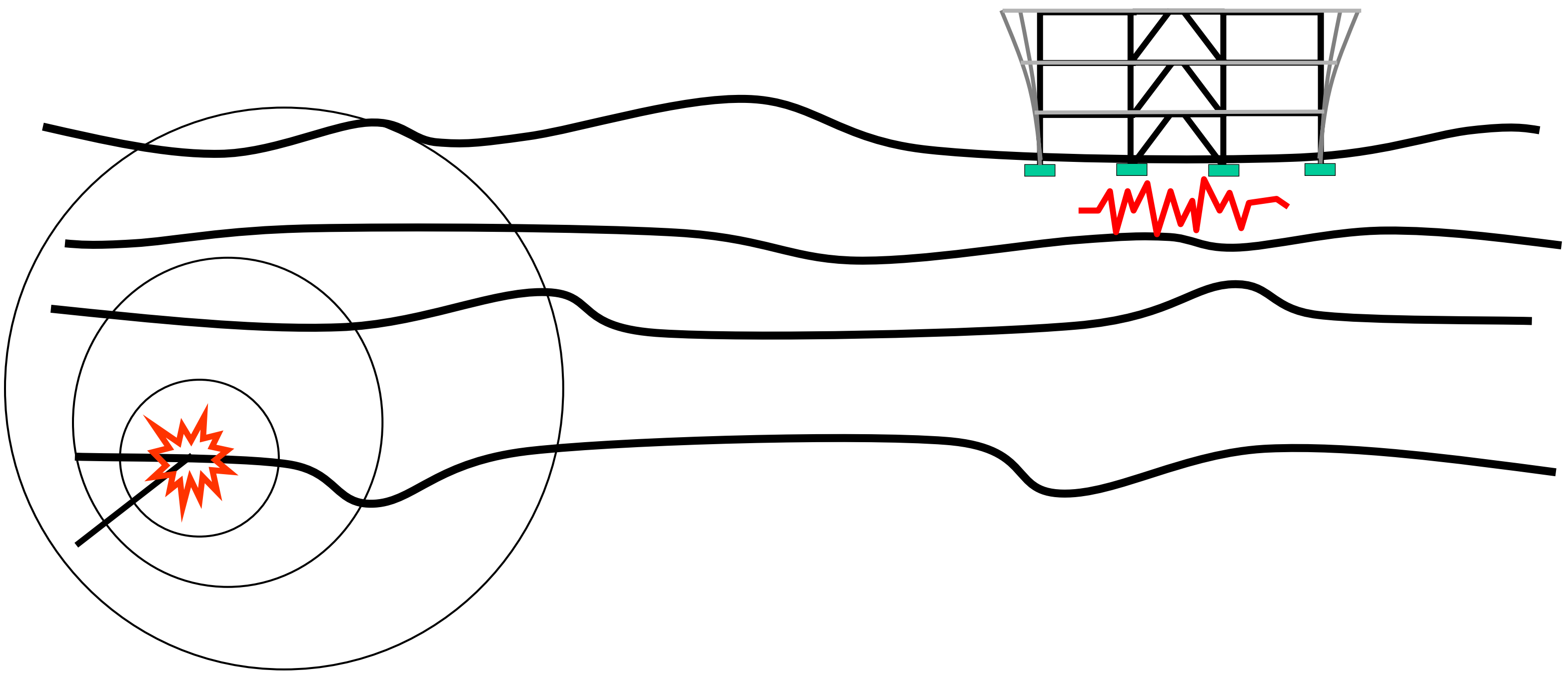
Summary

- Losses from earthquakes continue to exceed those from other natural hazards (with the exception of megadisasters like Hurricane Katrina).
- Poor soils tend to increase damages from earthquakes.
- Earthquake soil mitigation, especially for soil liquefaction, is effective.

Summary

- Current IBC 2003 procedures are based on WUS practice and experience.
- IBC provisions may not yet adequately account for unique CEUS conditions.
- Soil conditions in CEUS increase hazard.

STRUCTURAL ANALYSIS FOR PERFORMANCE-BASED EARTHQUAKE ENGINEERING



Structural Analysis for Performance-Based Earthquake Engineering

- Basic modeling concepts
- Nonlinear static pushover analysis
- Nonlinear dynamic response history analysis
- Incremental nonlinear dynamic analysis
- Probabilistic approaches

Disclaimer

- The “design” ground motion cannot be predicted.
- Even if the motion can be predicted it is unlikely than we can precisely predict the response. This is due to the rather long list of things we do not know and can not do, as well as uncertainties in the things we do know and can do.
- The best we can hope for is to predict the characteristics of the ground motion and the characteristics of the response.

How to Compute Performance-Based Deformation Demands?

Increasing Value
of
Information

- ✗ Linear Static Analysis
- ✗ Linear Dynamic Modal Response Spectrum Analysis
- ✗ Linear Dynamic Modal Response History Analysis
- ✗ Linear Dynamic Explicit Response History Analysis

- √ Nonlinear Static "Pushover" Analysis
- √ Nonlinear Dynamic Explicit Response History Analysis

✗ = Not Reliable in Predicting Damage

FEMA 368 Analysis Requirements (SDC D, E, F)

Analysis Method

		Linear Static	Response Spectrum	Linear Resp. Hist.	Nonlinear Resp. Hist.
$T \le T_s$	Regular Structures	YES	YES	YES	YES
	Plan Irreg. 2,3,4,5 Vert. Irreg. 4, 5	YES	YES	YES	YES
	Plan Irreg. 1a ,1b Vert. Irreg. 1a, 1b 2, or 3	NO	YES	YES	YES
All Other Structures		NO	YES	YES	YES

Nonlinear Static Analysis Limitations not Stated

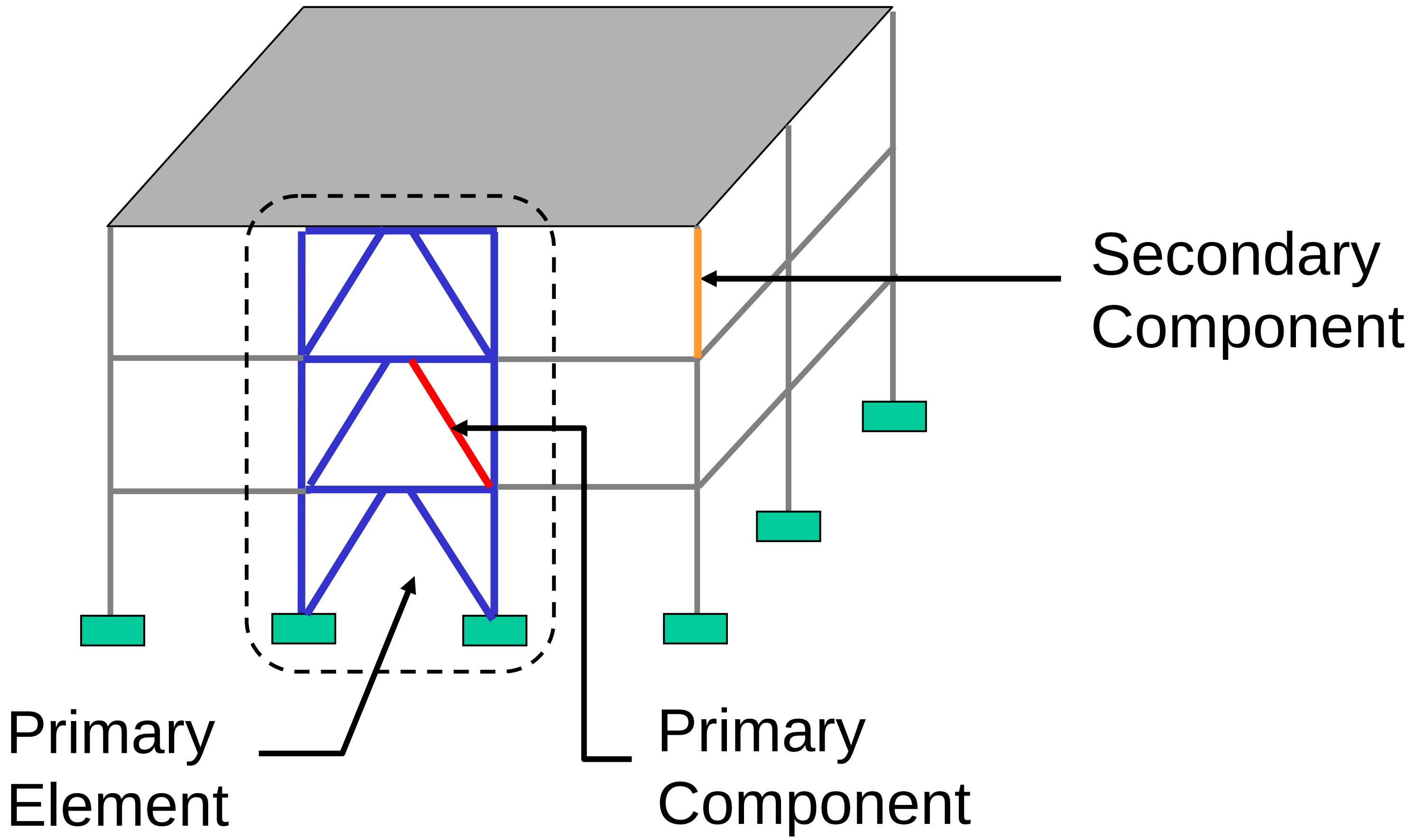
FEMA 350 Analysis Requirements (Collapse Prevention)

Analysis Method

			Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
$T \le T_s$	Regular	Strong Column	YES	YES	YES	YES
		Weak Column	NO	NO	YES	YES
	Irregular	Any Condition	NO	NO	YES	YES

			Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
$T > T_s$	Regular	Strong Column	NO	YES	NO	YES
		Weak Column	NO	NO	NO	YES
	Irregular	Any Condition	NO	NO	NO	YES

Definition for "Elements" and "Components"



Primary elements or components are critical to the buildings ability to resist collapse

Basic Modeling Concepts

In general, a model should include the following:

- Soil-Structure-Foundation System
- Structural (Primary) Components and Elements
- Nonstructural (Secondary) Components and Elements
- Mechanical Systems (if performance of such systems is being assessed)
- Reasonable Distribution and Sequencing of gravity loads
- P-Delta (Second Order) Effects
- Reasonable Representation of Inherent Damping
- Realistic Representation of Inelastic Behavior
- Realistic Representation of Ground Shaking

Basic Modeling Concepts

- In general, a three-dimensional model is necessary. However, due to limitations in available software, 3-D inelastic time history analysis is still not practical (except for very special and important structures).

- In this course we will concentrate on 2-D analysis.

- We will use the computer program NONLIN-Pro which is on the course CD. Note that the analysis engine behind NONLIN-Pro is DRAIN-2Dx.

- DRAIN-2Dx is old technology, but it represents the basic state of the practice. The state of the art is being advanced through initiatives such as PEER's OpenSees Environment.

Steps in Performing Nonlinear Response History Analysis (1)

1) Develop Linear Elastic Model, *without P-Delta Effects*
   - a) Mode Shapes and Frequencies (Animate!)
   - b) Independent Gravity Load Analysis
   - c) Independent Lateral Load Analysis

2) Repeat Analysis (1) but *include P-Delta Effects*

3) Revise model to include Inelastic Effects. *Disable P-Delta.*
   - a) Mode Shapes and Frequencies (Animate!)
   - b) Independent Gravity Load Analysis
   - c) Independent Lateral Load (Pushover)Analysis
   - d) Gravity Load followed by Lateral Load
   - e) Check effect of variable load step

4) Repeat Analysis (3) but *include P-Delta Effects*

Steps in Performing Nonlinear Response History Analysis (2)

5) Run Linear Response History Analysis, *disable P-Delta*
   a) Harmonic Pulse followed by Free Vibration
   b) Full Ground Motion
   c) Check effect of variable time step

6) Repeat Analysis (5) but *include P-Delta Effects*

7) Run Nonlinear Response History Analysis, *disable P-Delta*
   a) Harmonic Pulse followed by Free Vibration
   b) Full Ground Motion
   c) Check effect of variable time step

8) Repeat Analysis (7) but *include P-Delta Effects*

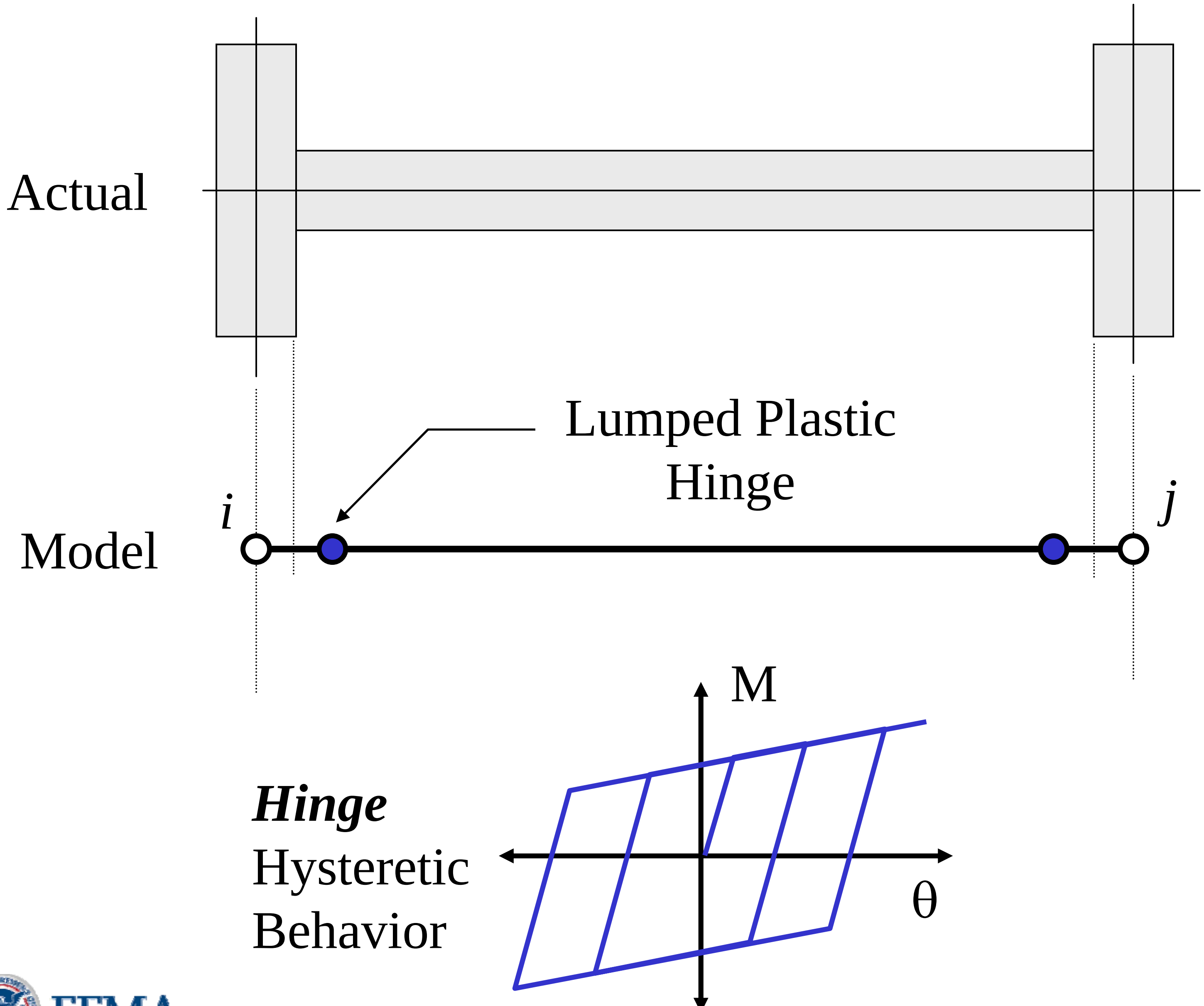
Basic Component Model Types

Phenomenological

All of the inelastic behavior in the yielding region of the component is “lumped” into a single location. Rules are typically required to model axial-flexural interaction.

Very large structures may be modeled using this approach. Nonlinear dynamic analysis is practical for most 2D structures, but may be too computationally expensive for 3D structures.

Phenomenological Model



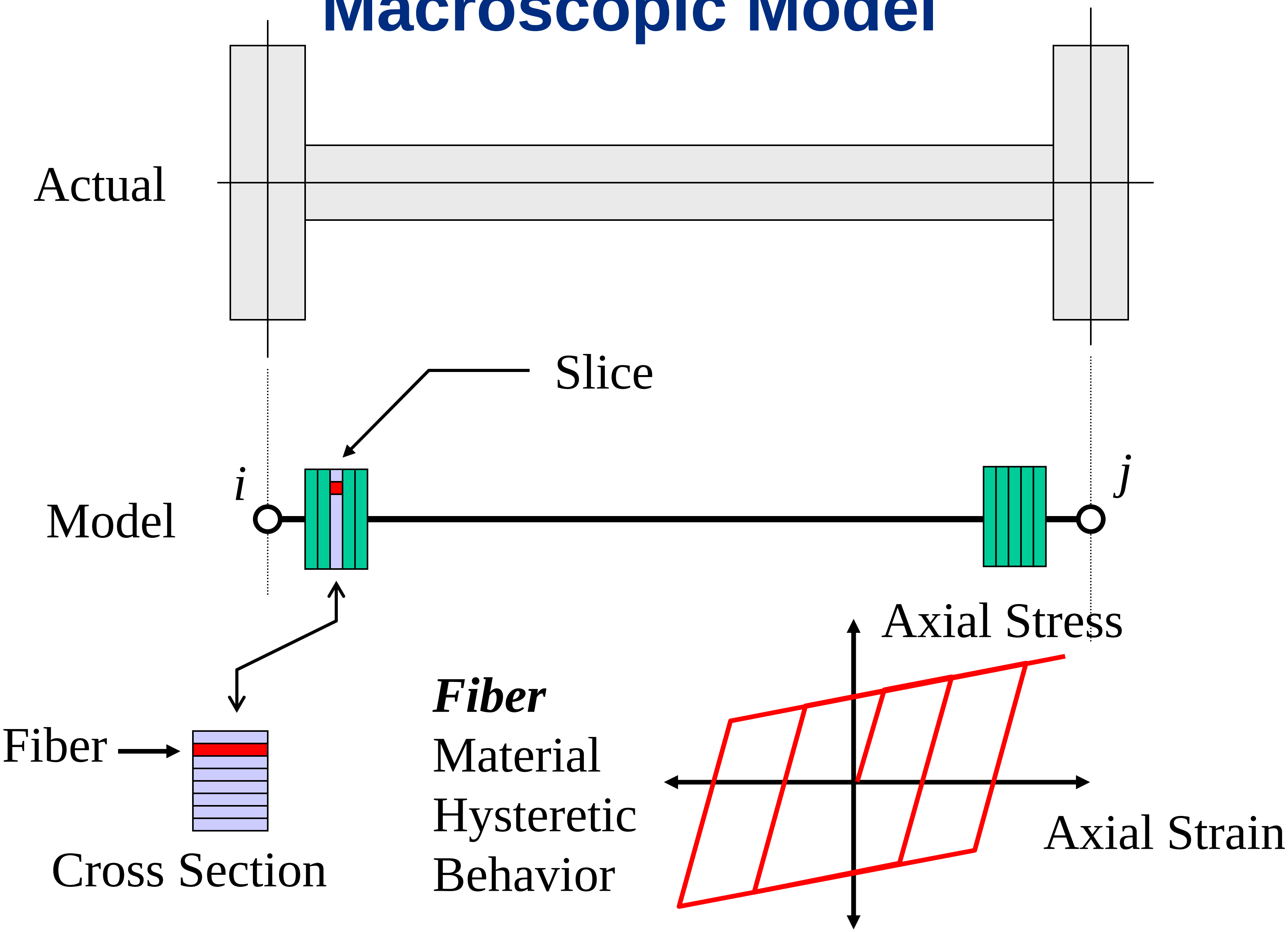
Basic Component Model Types

Macroscopic

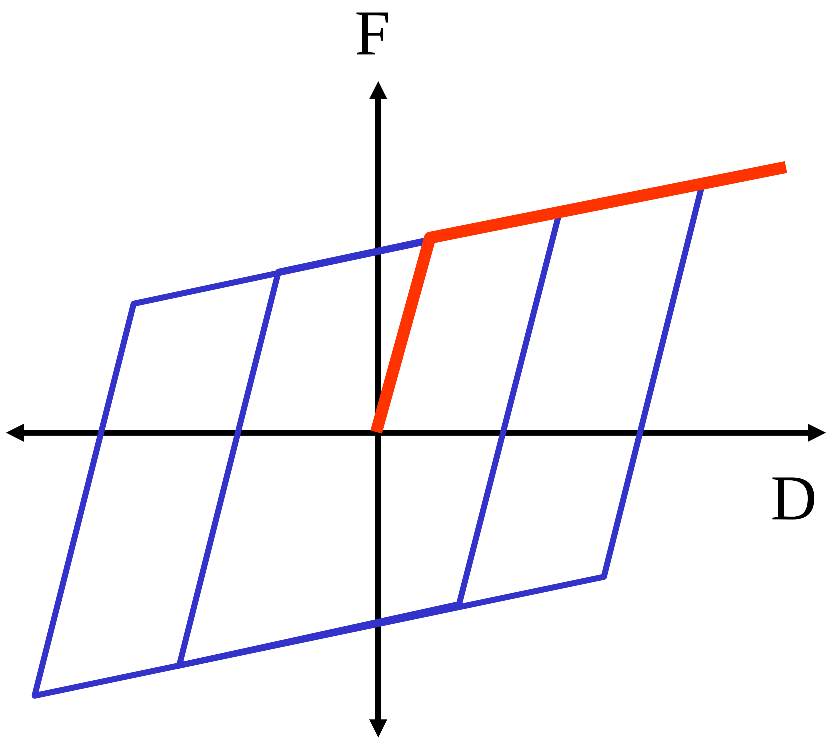
The yielding regions of the component are highly discretized and inelastic behavior is represented at the material level. Axial-flexural interaction is handled automatically.

These models are reasonably accurate, but are very computationally expensive. Pushover analysis may be practical for some 2D structures, but nonlinear dynamic time history analysis is not currently feasible for large 2D structures or for 3D structures.

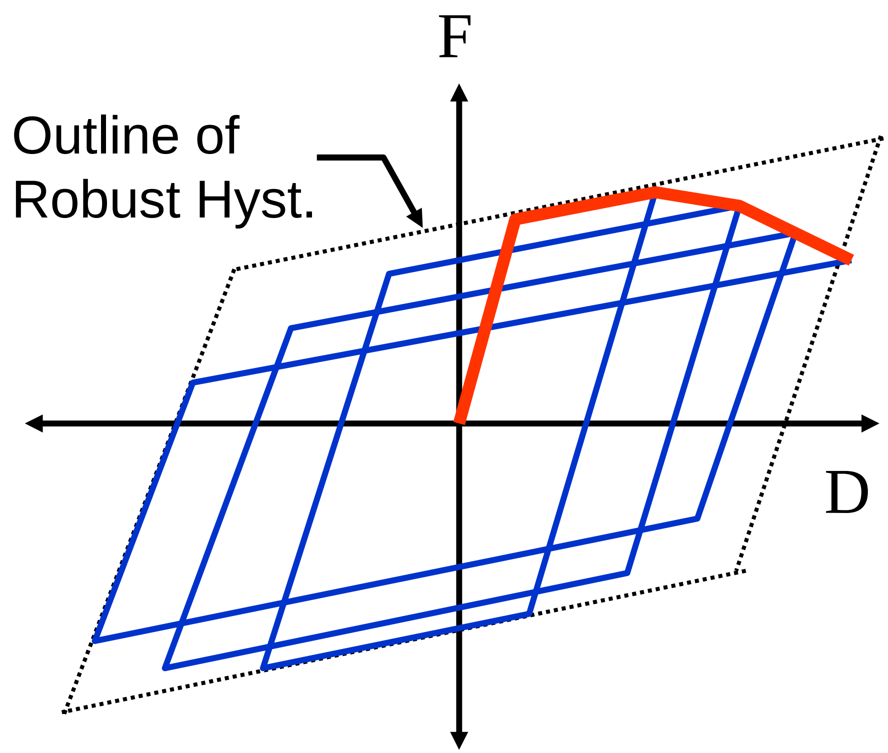
Macroscopic Model



Rule-Based Hysteretic Models and Backbone Curves (1)

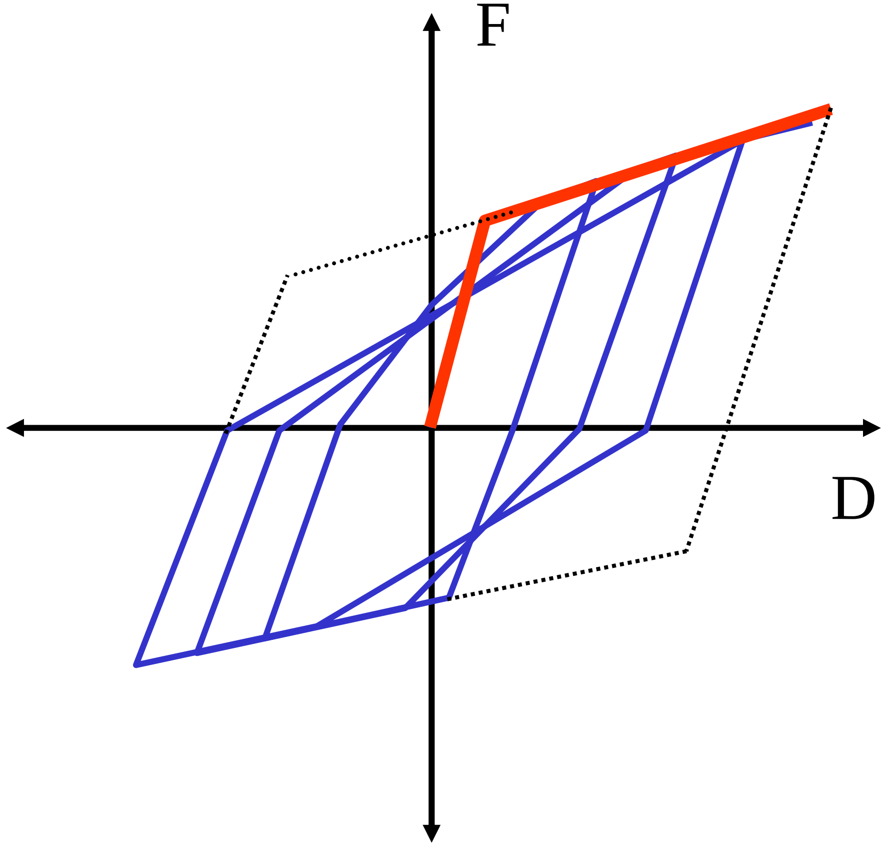


Simple Yielding
(Robust)

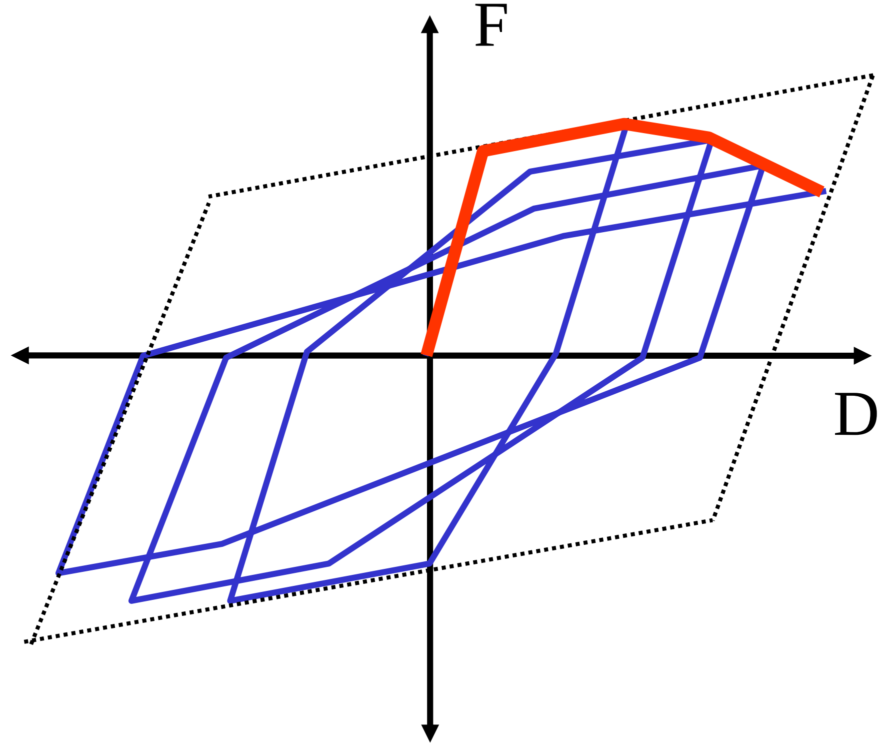


(Ductile) Loss of Strength

Rule-Based Hysteretic Models and Backbone Curves (2)

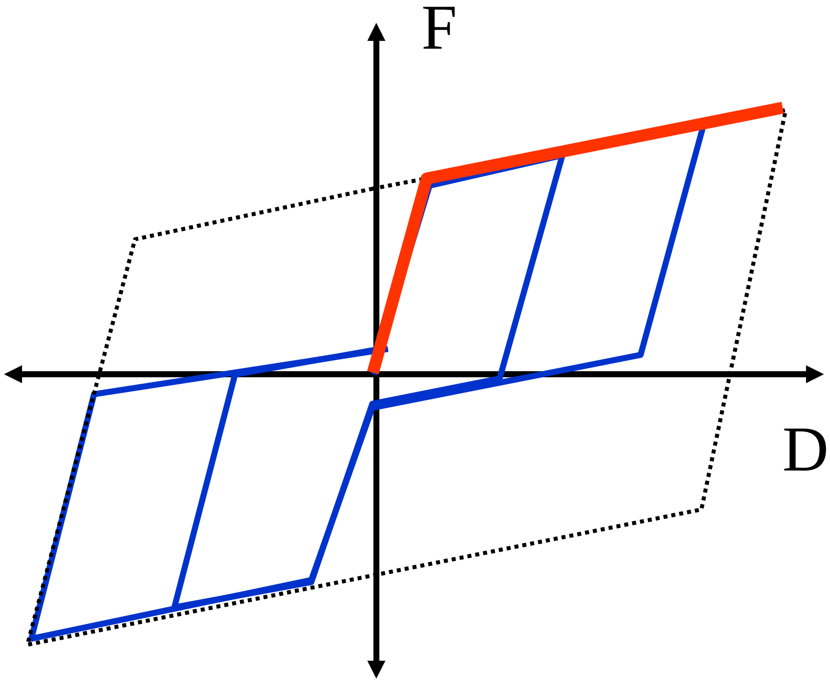


Loss of Stiffness

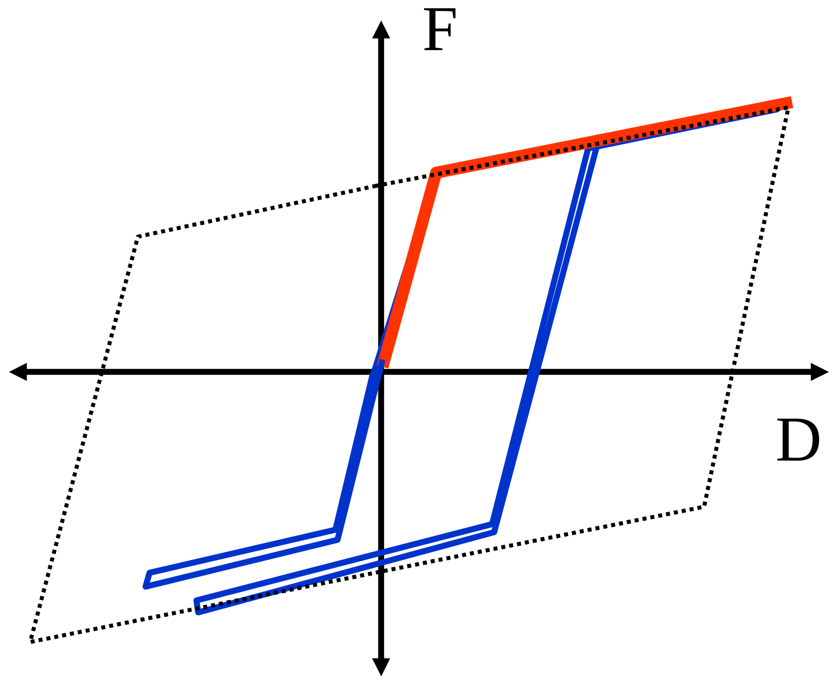


Loss of Strength and Stiffness

Rule-Based Hysteretic Models and Backbone Curves (3)

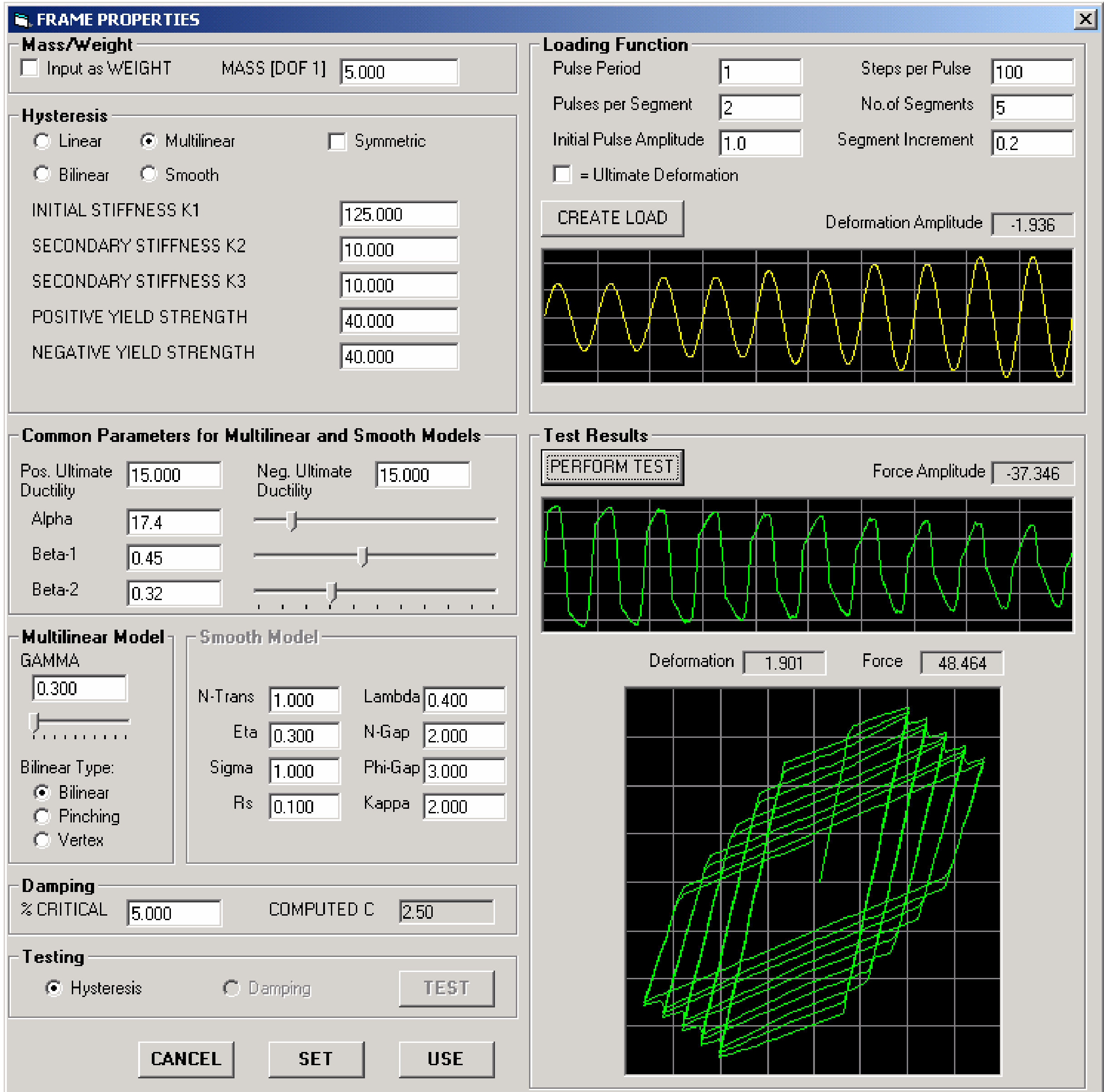


Pinched



Buckling

Sivaselvan and Reinhorn Models in NONLIN (MDOF MODEL)

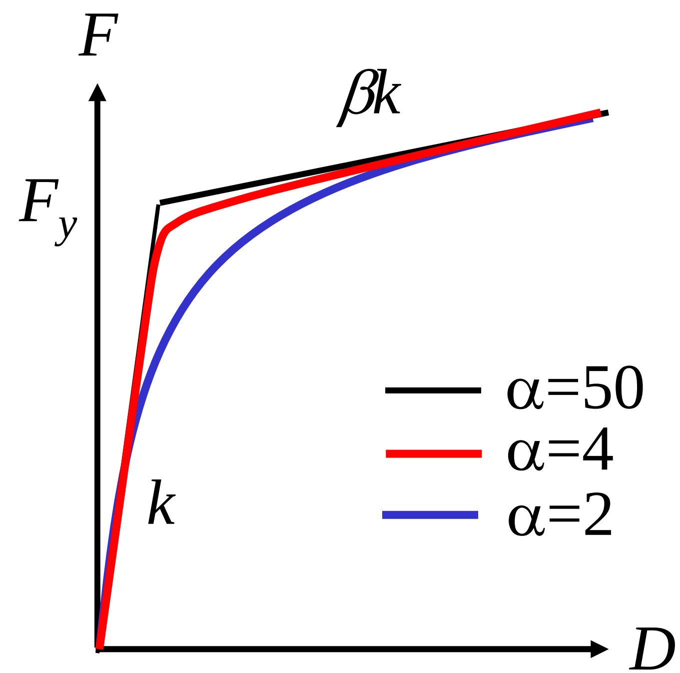


NONLIN

Parametric Models, e.g., SAP2000

$$F = \beta k D + (1-\beta) F_y Z$$

$$\dot{Z} = \frac{k}{F_y} \begin{Bmatrix} \dot{D}(1-|Z|^{\alpha}) \text{ if } \dot{D}Z > 0 \\ \dot{D} \quad \text{otherwise} \end{Bmatrix}$$



Degrading Stiffness, Degrading Strength, and Pinching Models also available. See Sivaselvan and Reinhorn for Details.

The *NONLIN-Pro* Structural Analysis Program

- A Pre-and Post-Processing Environment for DRAIN 2Dx
- Developed by Advanced Structural Concepts, Inc., of Blacksburg, Virginia
- Formerly Marketed as RAM XLINEA
- Provided at no cost to MBDSI Participants
- May soon be placed in the Public Domain through NISEE.

The *DRAIN-2DX* Structural Analysis Program

- Developed at U.C. Berkeley under direction of Graham H. Powell
- *Nonlin-Pro* Incorporates Version 1.10, developed by V. Prakash, G. H. Powell, and S. Campbell, EERC Report Number UCB/SEMM-93/17.
- A full User's Manual for DRAIN may be found on the course CD, as well as in the *Nonlin-Pro* online Help System.
- FORTAN Source Code for the version of DRAIN incorporated into *Nonlin-Pro* is available upon request

DRAIN-2DX Capabilities/Limitations

- Structures may be modeled in TWO DIMENSIONS ONLY. Some 3D effects may be simulated if torsional response is not involved.
- Analysis Capabilities Include:
  - Linear Static
  - Mode Shapes and Frequencies
  - Linear Dynamic Response Spectrum*
  - Linear Dynamic Response History
  - Nonlinear Static: Event-to-Event (Pushover)
  - Nonlinear Dynamic Response History

* Not fully supported by Nonlin-Pro

DRAIN-2DX Capabilities/Limitations

- Small Displacement Formulation Only
- P-Delta Effects included on an element basis using linearized formulation
- System Damping is Mass and Stiffness Proportional
- Linear Viscous Dampers may be (indirectly) modeled using stiffness Proportional Damping
- Response-History analysis uses Newmark constant average acceleration scheme
- Automatic time-stepping with energy-based error tolerance is provided

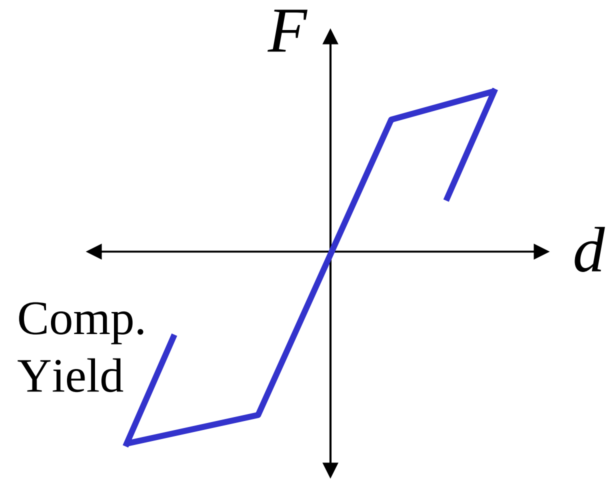
DRAIN-2DX Element Library

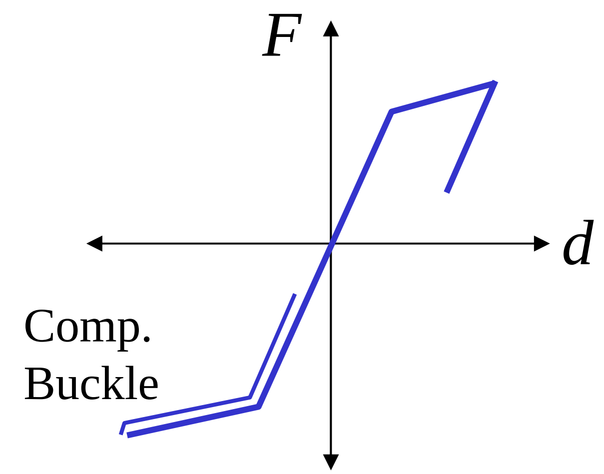
TYPE 1: Truss Bar
TYPE 2: Beam-Column
TYPE 3: Degrading Stiffness Beam-Column*
TYPE 4: Zero Length Connector
TYPE 6: Elastic Panel
TYPE 9: Compression/Tension Link
TYPE 15: Fiber Beam-Column*

* Not fully supported by Nonlin-Pro

DRAIN 2Dx Truss Bar Element

- Axial Force Only
- Simple Bilinear Yield in Tension or Compression
- Elastic Buckling in Compression
- Linearized Geometric Stiffness
- May act as linear viscous damper (some trickery required)

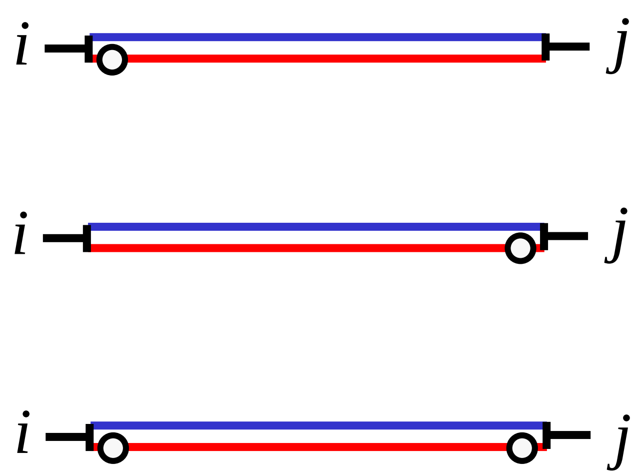




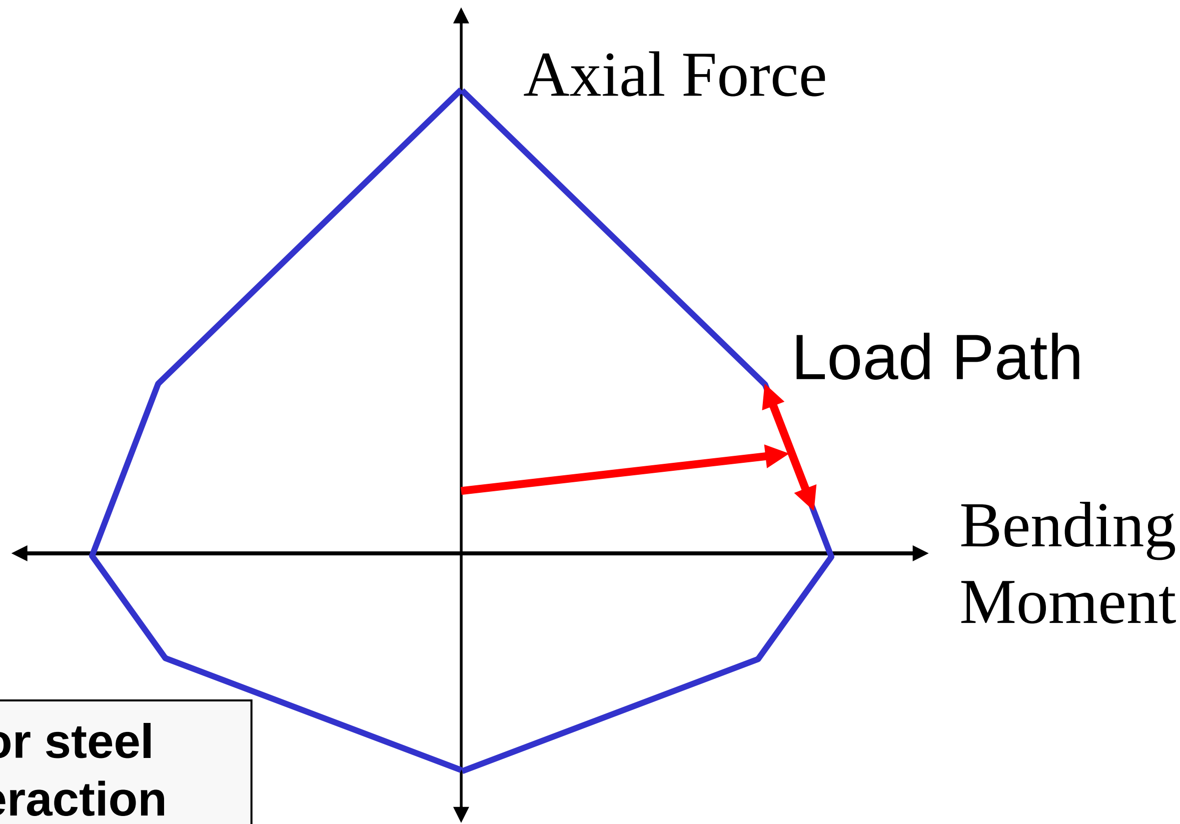
DRAIN 2Dx Beam-Column Element

- Two Component Formulation
- Simple Bilinear Yield in Positive or Negative Moment. *Axial yield is NOT provided.*
- Simple Axial-Flexural Interaction
- Linearized Geometric Stiffness
- Nonprismatic properties and shear deformation possible
- Rigid End Zones Possible

Possible Yield States

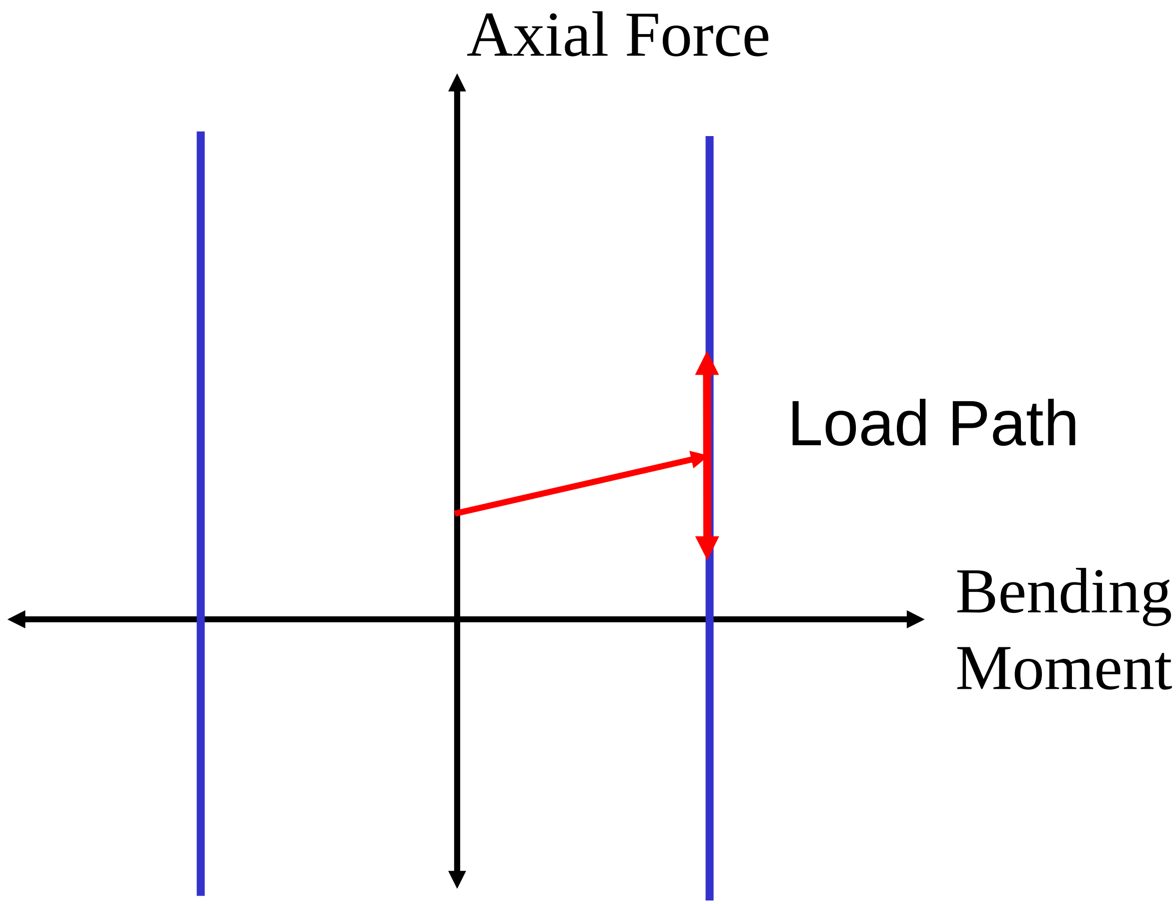


DRAIN 2Dx Beam-Column Element Axial-Flexural Interaction

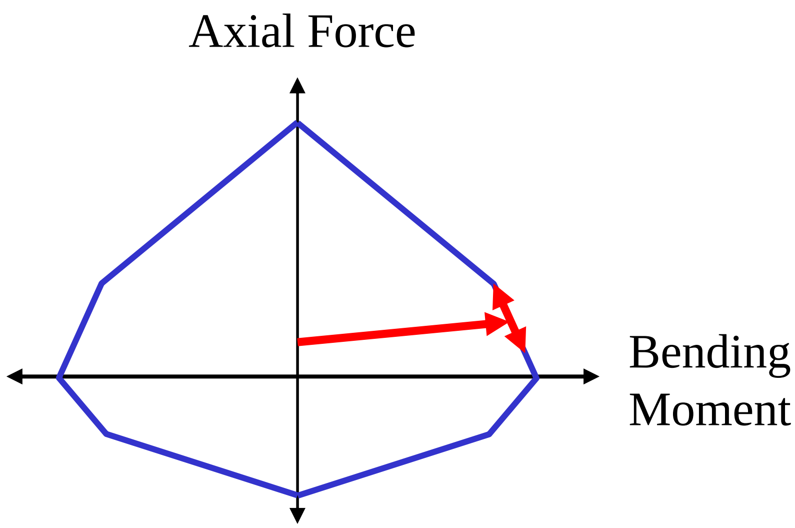


Note: Diagram is for steel sections. NOo interaction and reinforced concrete type interaction is also possible

DRAIN 2Dx Beam-Column Element NO Axial-Flexural Interaction



DRAIN 2Dx Beam-Column Element Axial-Flexural Interaction



Note: This Model is not known for its accuracy or reliability. Improved models based on plasticity theory have been developed. See, for example, The RAM-Perform Program.

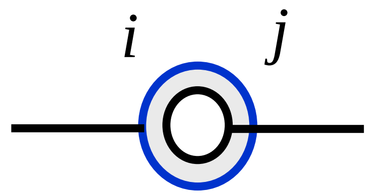
DRAIN 2Dx
Connection Element

- Zero Length Element
- Translational or Rotational Behavior
- Variety of Inelastic Behavior, including:
  - Bilinear yielding with inelastic unloading
  - Bilinear yielding with elastic unloading
  - Inelastic unloading with gap
- May be used to model linear viscous dampers

Using a Connection Element to Model a Rotational Spring

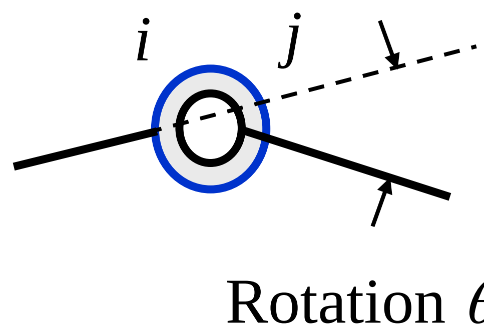
- Nodes i and j have identical X and Y coordinates. The pair of nodes is referred to as a "compound node"

- Node j has X and Y displacements slaved to those of node i



- A rotational connection element is placed "between" nodes i and j

- Connection element resists relative rotation between nodes i and j

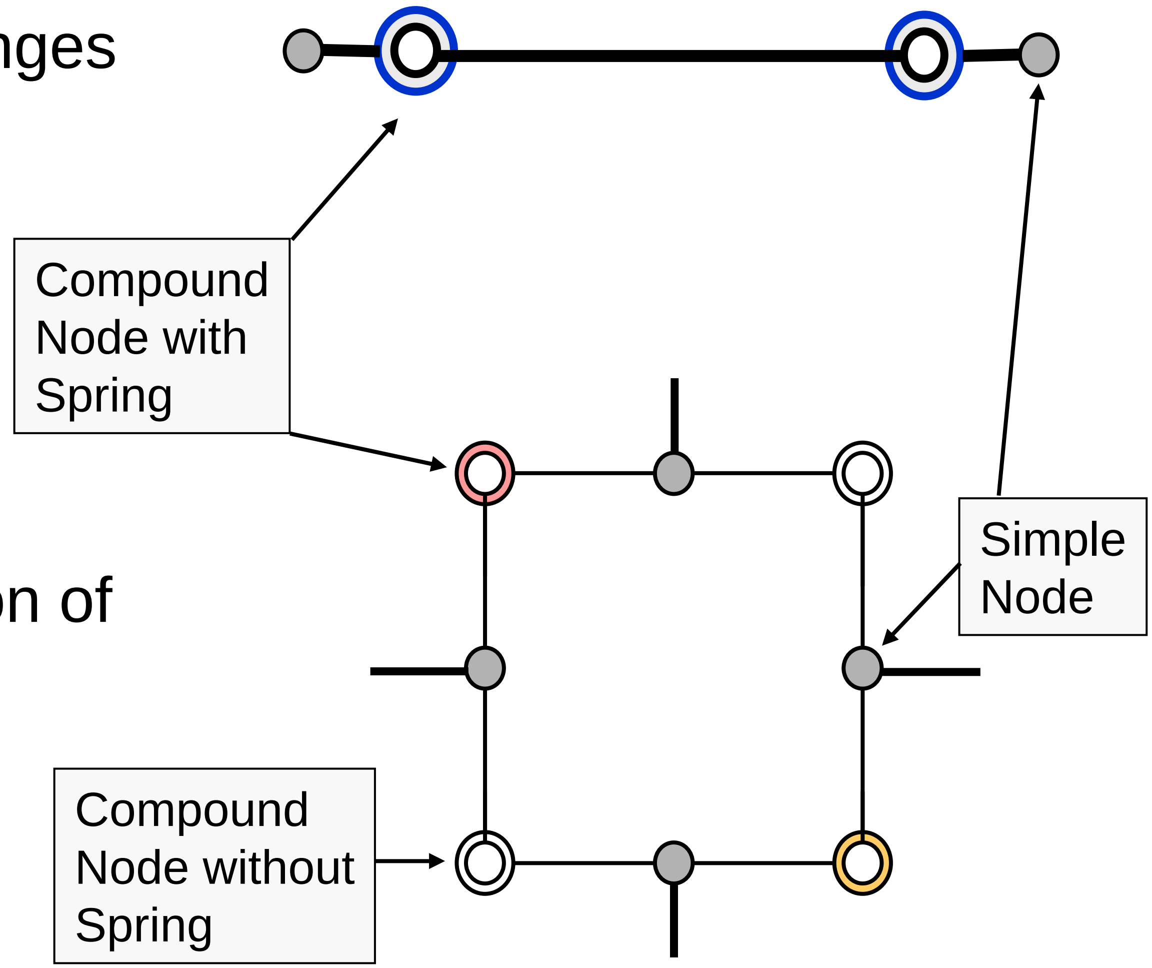


- ***NEVER use Beta Damping unless you are explicitly modeling a damper.***

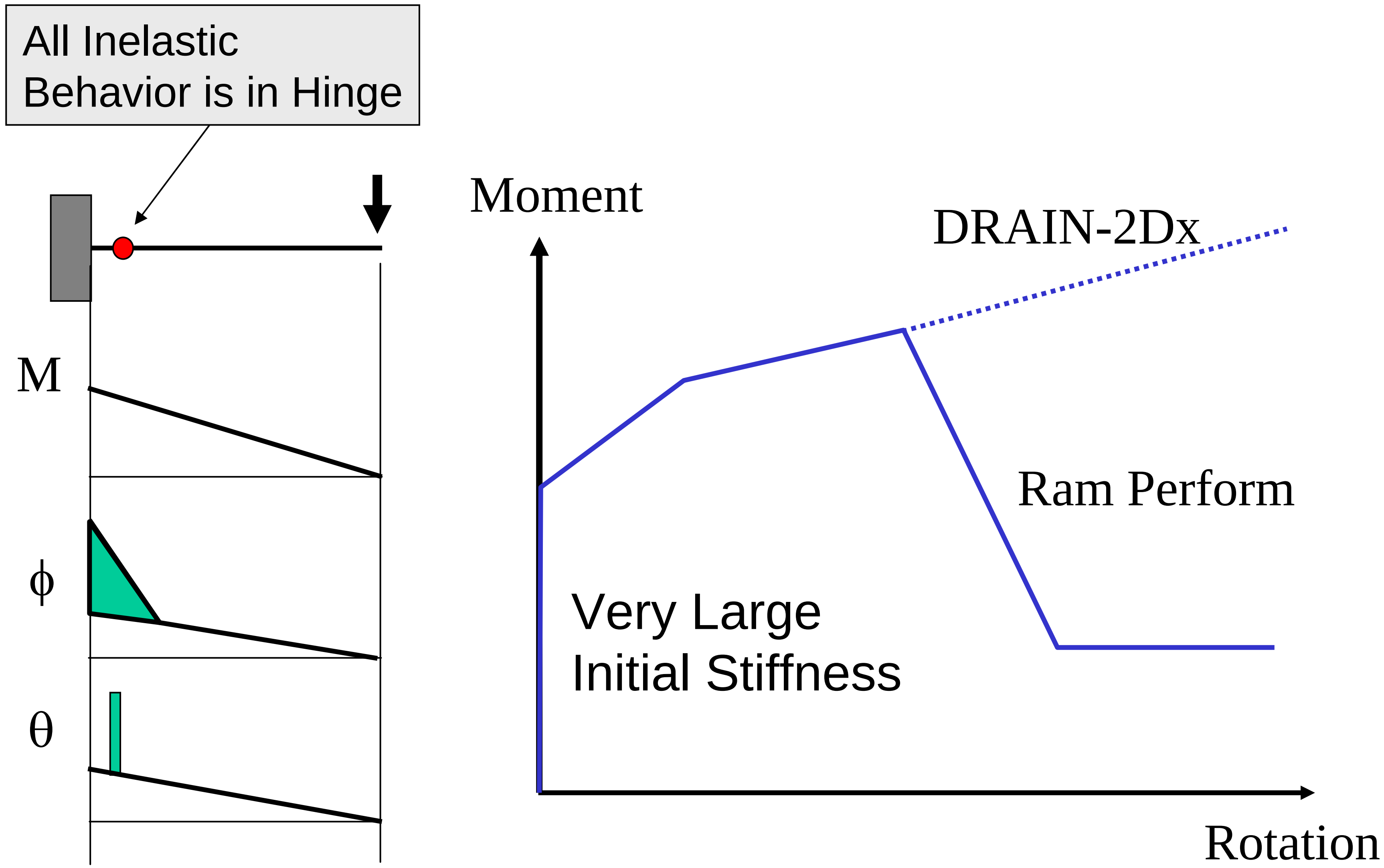
Uses of Compound Nodes

Girder Plastic Hinges

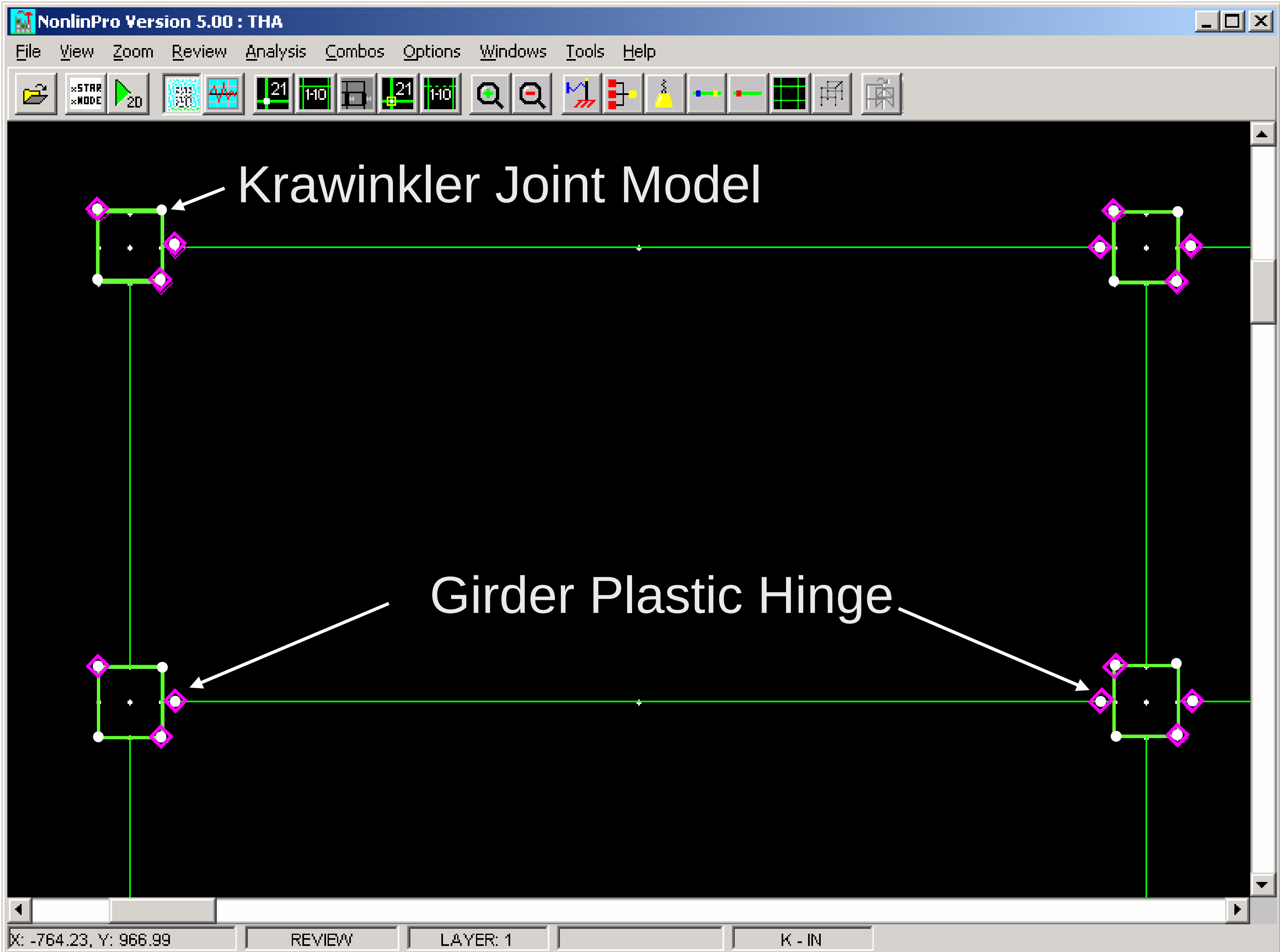
Panel Zone region of Beam-Column Joint



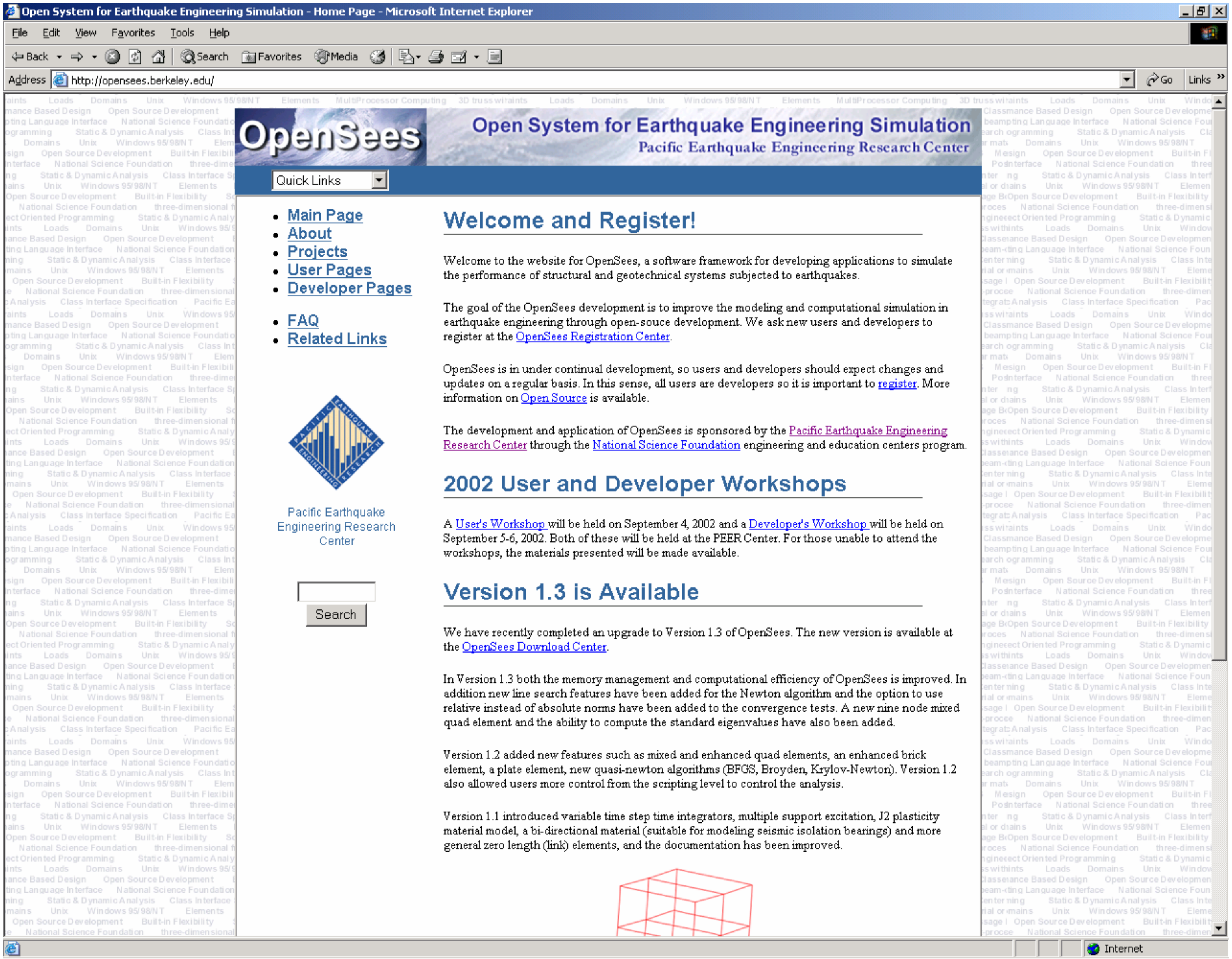
Development of Girder Hinge Model



Girder and Joint Modeling in NONLIN-Pro



The OpenSees Computational Environment

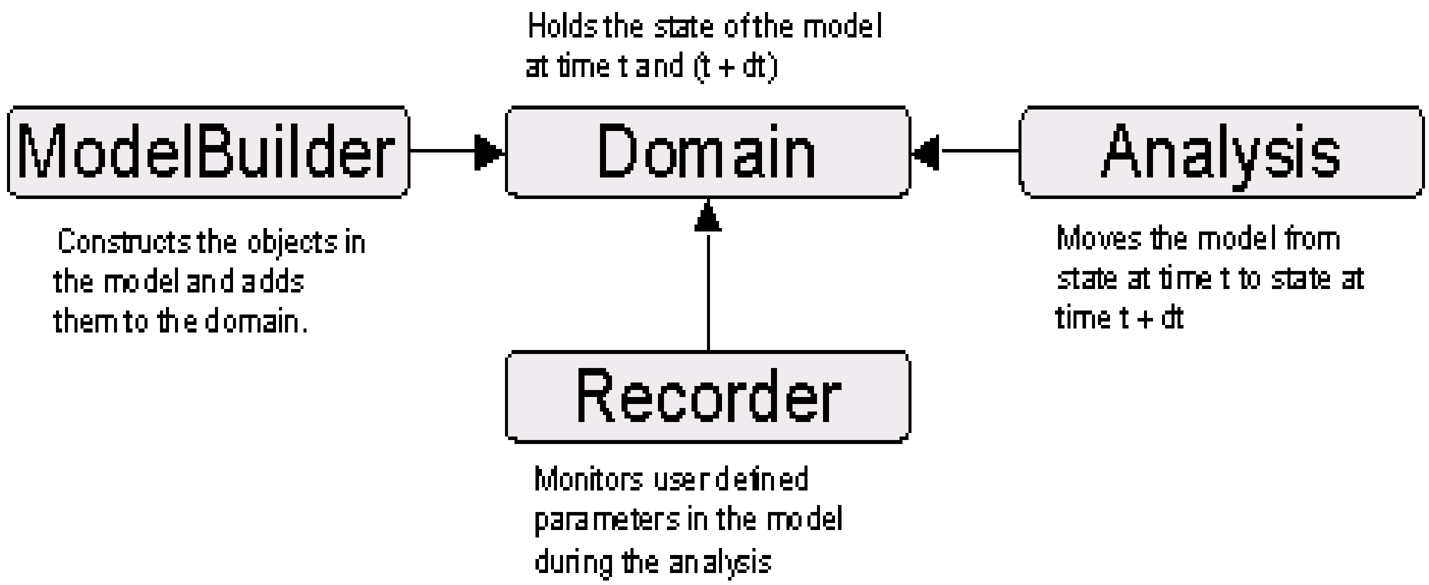


What is *OpenSees*?

- OpenSees is a multi-disciplinary open source structural analysis program.
- Created as part of the Pacific Earthquake Engineering Research (PEER) center.
- The goal of OpenSees is to improve modeling and computational simulation in earthquake engineering through open-source development

OpenSees Program Layout

- OpenSees is an object oriented framework for finite element analysis
- OpenSees consists of 4 modules for performing analyses:



OpenSees Modules

- **Modelbuilder** - Performs the creation of the finite element model
- **Analysis** – Specifies the analysis procedure to perform on the model
- **Recorder** – Allows the selection of user-defined quantities to be recorded during the analysis
- **Domain** – Stores objects created by the Modelbuilder and provides access for the Analysis and Recorder modules

OpenSees Element Types

- Elements

Truss elements	Corotational truss
Elastic beam-column	Nonlinear beam-column
Zero-length elements	Quadrilateral elements
Brick elements	

- Sections

Elastic section	Uniaxial section
Fiber section	Section aggregator
Plate fiber section	Bidirectional section
Elastic membrane plate section	

OpenSees Material Properties

- Uniaxial Materials

Elastic	Elastic perfectly plastic
Parallel	Elastic perfectly plastic gap
Series	Hardening
Steel01	Concrete01
Hysteretic	Elastic-No tension
Viscous	Fedeas

OpenSees Analysis Types

- **Loads**: Variable time series available with plain, uniform, or multiple support patterns
- **Analyses**: Static, transient, or variable-transient
- **Systems of Equations**: Formed using banded, profile, or sparse routines
- **Algorithms**: Solve the SOE using linear, Newtonian, BFGS, or Broyden algorithms
- **Recording**: Write the response of nodes or elements (displacements, envelopes) to a user-defined set of files for evaluation

OpenSees Applications

- Structural modeling in 2 or 3D, including linear and nonlinear damping, hysteretic modeling, and degrading stiffness elements
- Advanced finite element modeling
- Potentially useful for advanced earthquake analysis, such as nonlinear time histories and incremental dynamic analysis
- Open-source code allows for increased development and application

OpenSees Disadvantages

- No fully developed pre or post processors yet available for model development and visualization
- Lack of experience in applications
- Code is under development and still being fine-tuned.

OpenSees Information Sources

- The program and source code:
  http://millen.ce.berkeley.edu/
- Command index and help:
  http://peer.berkeley.edu/~silva/Opensees/manual/html/
- OpenSees Homepage:
  http://opensees.berkeley.edu/OpenSees/related.html

Other Commercially Available Programs

SAP2000/ETABS

Both have 3D pushover capabilities and linear/nonlinear dynamic response history analysis. P-Delta and large displacement effects may be included. These are the most powerful commercial programs that are specifically tailored to analysis of buildings(ETABS) and bridges (SAP2000).

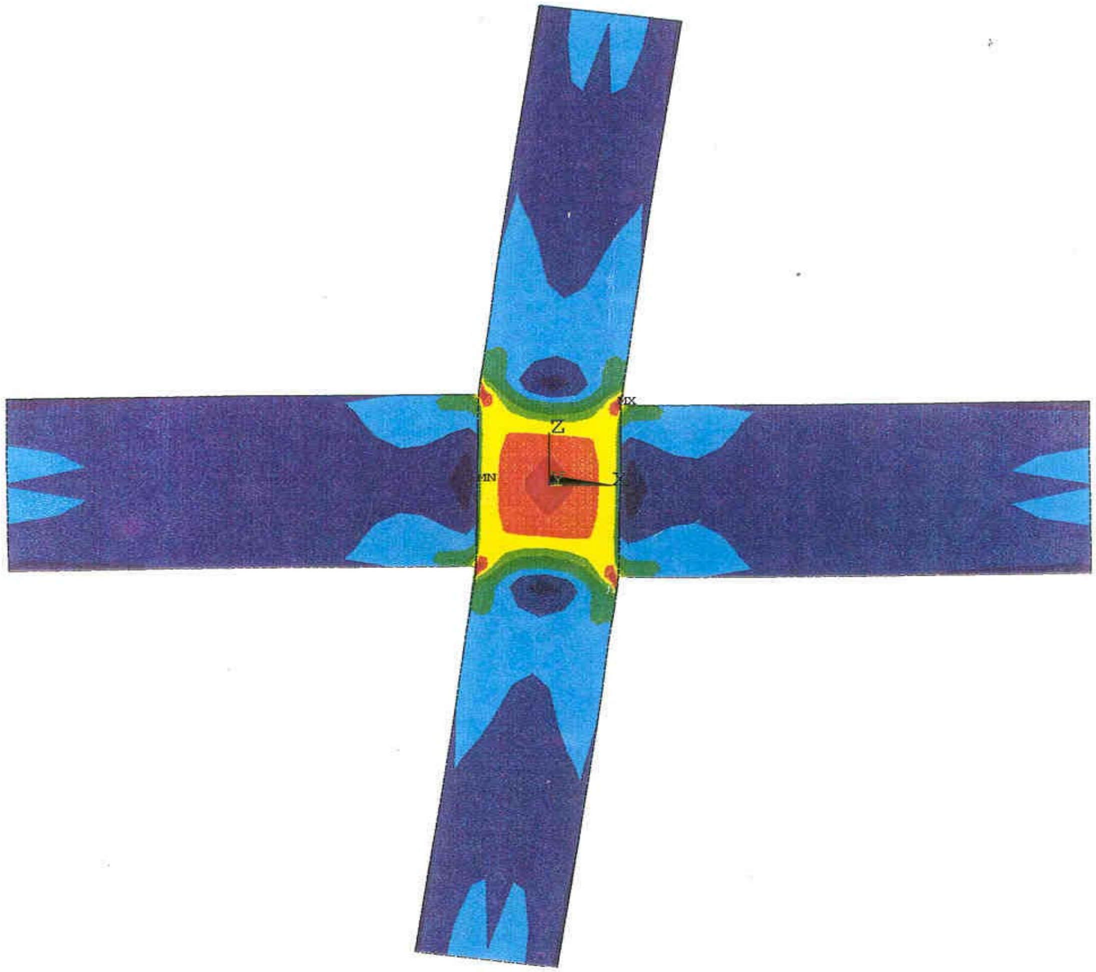
RAM/Perform

Currently 2D program, but a 3D version should be available soon. Developed by G. Powell, and is based on DRAIN-3D technology. Some features of program (e.g. model building) are hard-wired and not easy to override.

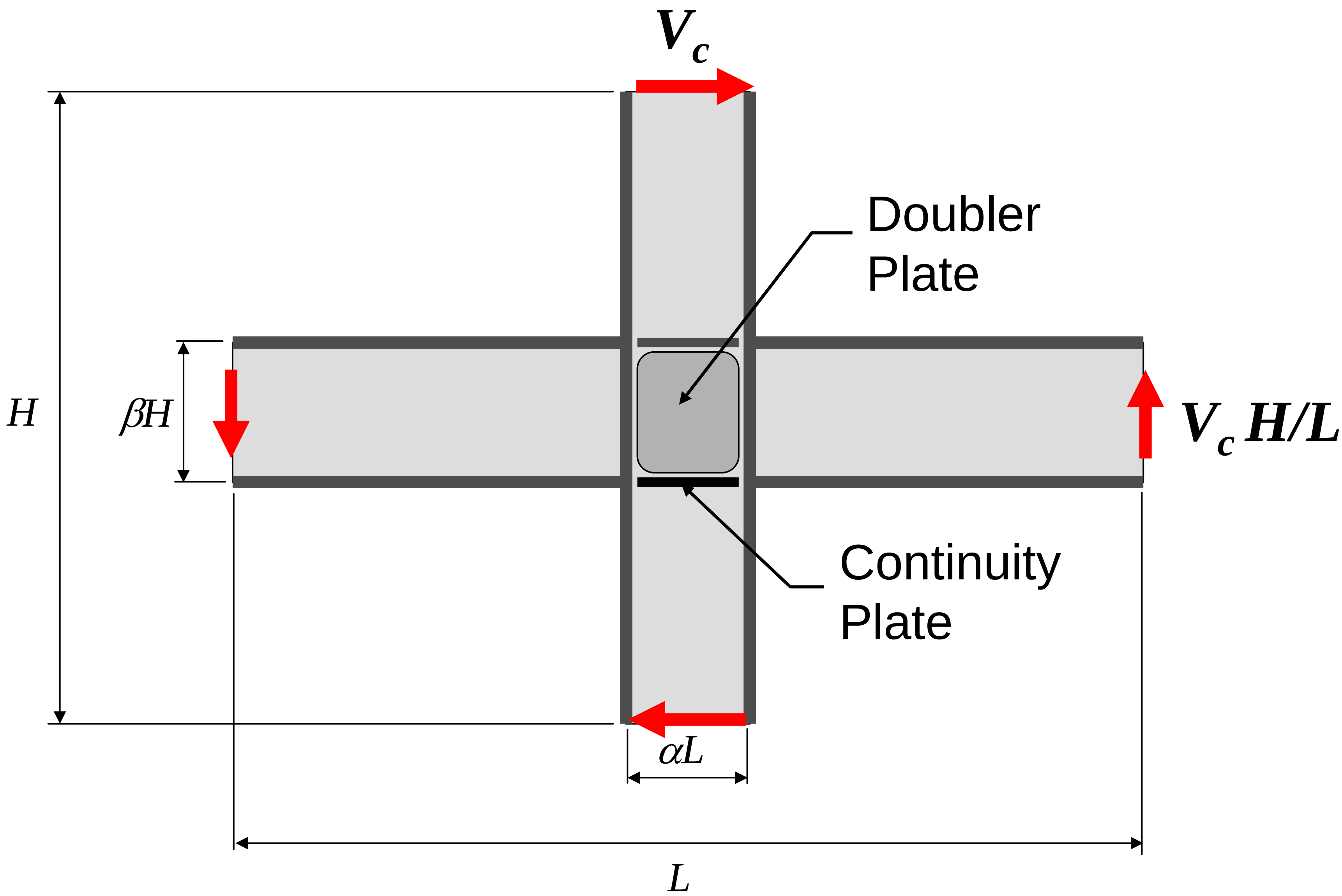
ABAQUS,ADINA, ANSYS, DIANA,NASTRAN

These are extremely powerful FEA programs but are not very practical for analysis of building and bridge structures.

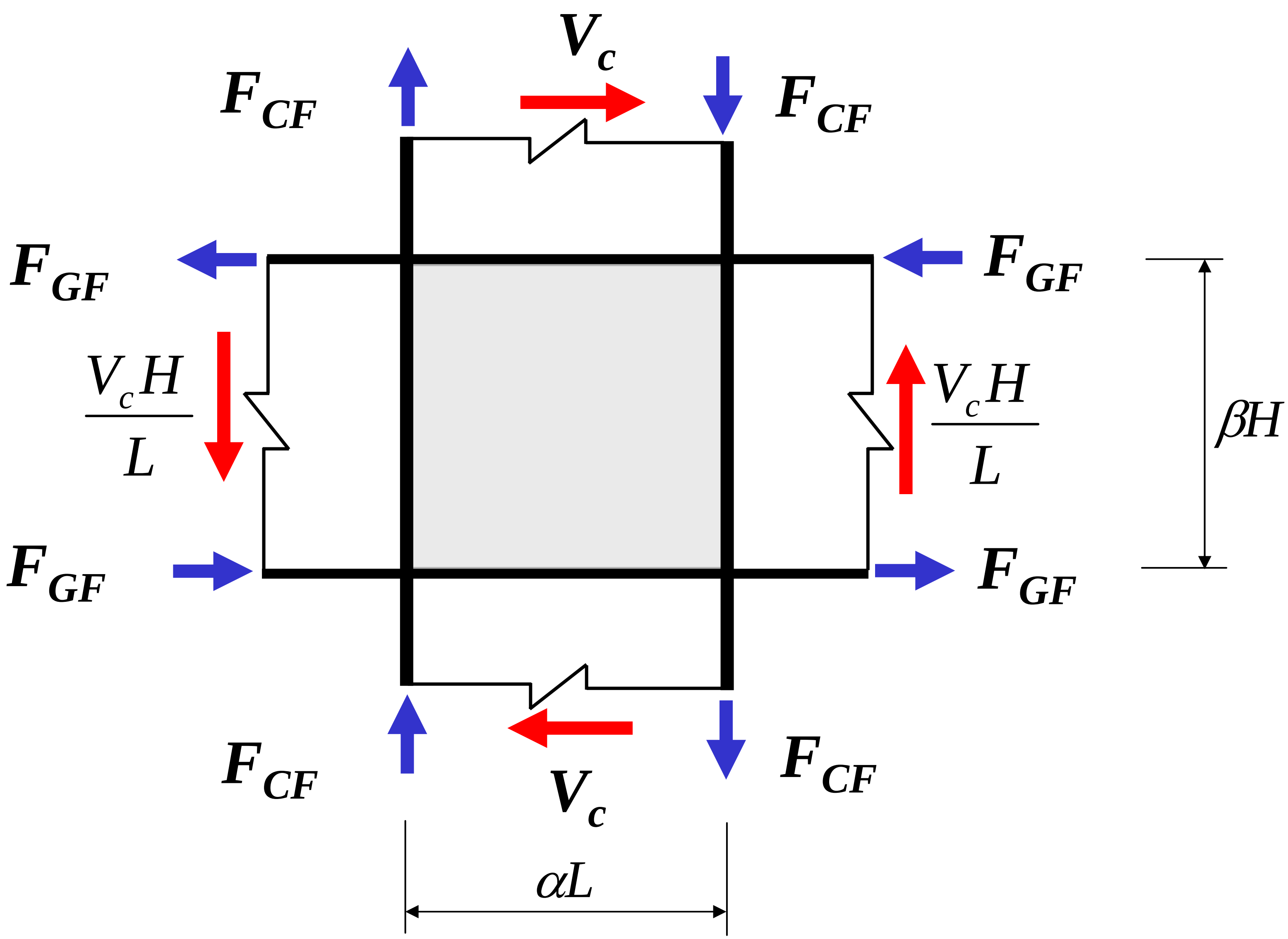
Modeling Beam-Column Joint Deformation In Steel Structures



Typical Interior Subassemblage



Equilibrium in Beam-Column Joint Region



Forces and Stresses in Panel Zone

Horizontal Shear in Panel Zone:

$$V_P = V_c \frac{(1-\alpha-\beta)}{\beta}$$

Note: PZ shear can be 4 to 6 times the column shear

Shear Stress in Panel Zone:

$$\tau_P = V_c \frac{(1-\alpha-\beta)}{\alpha\beta L t_P}$$

t_p is panel zone thickness including doubler plate

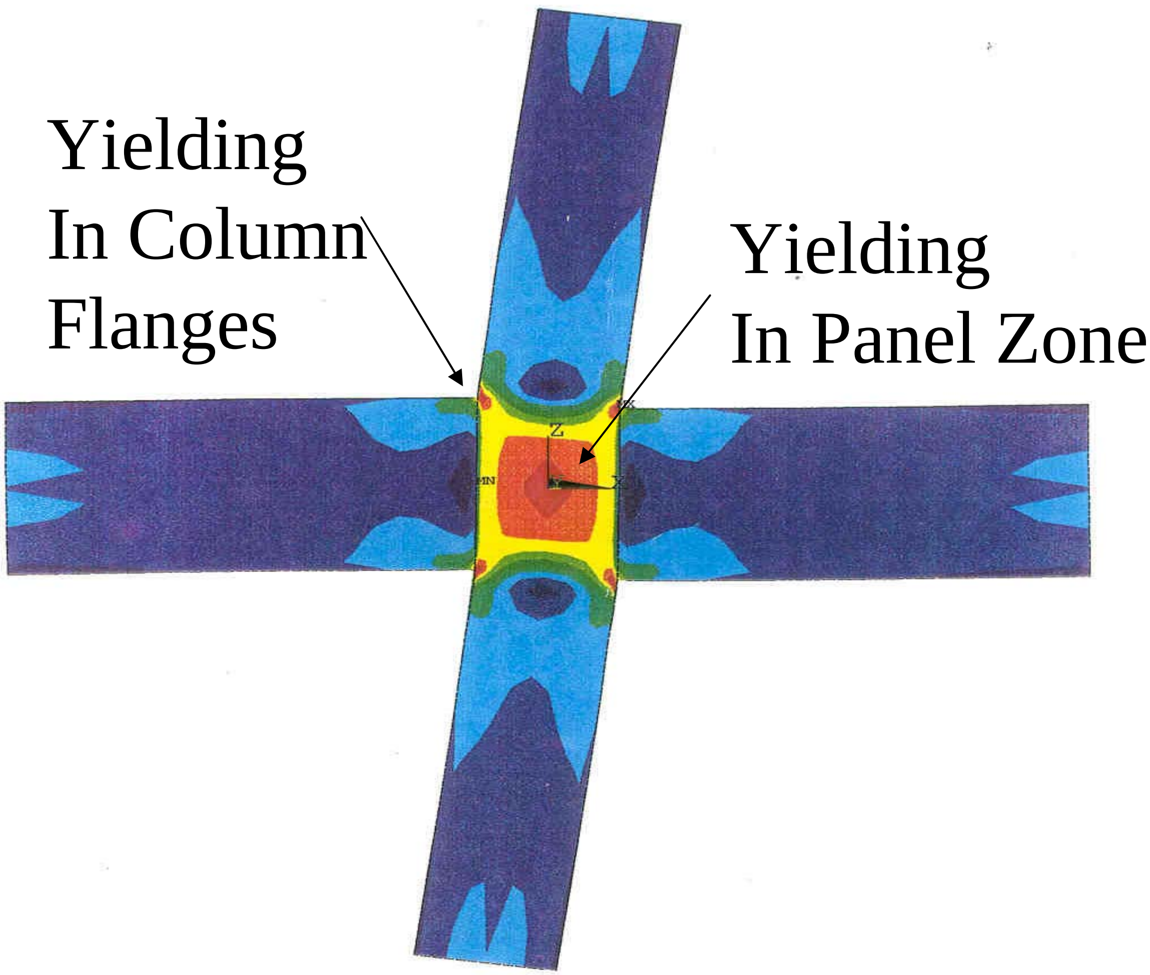
Effects of High Panel Zone Stresses

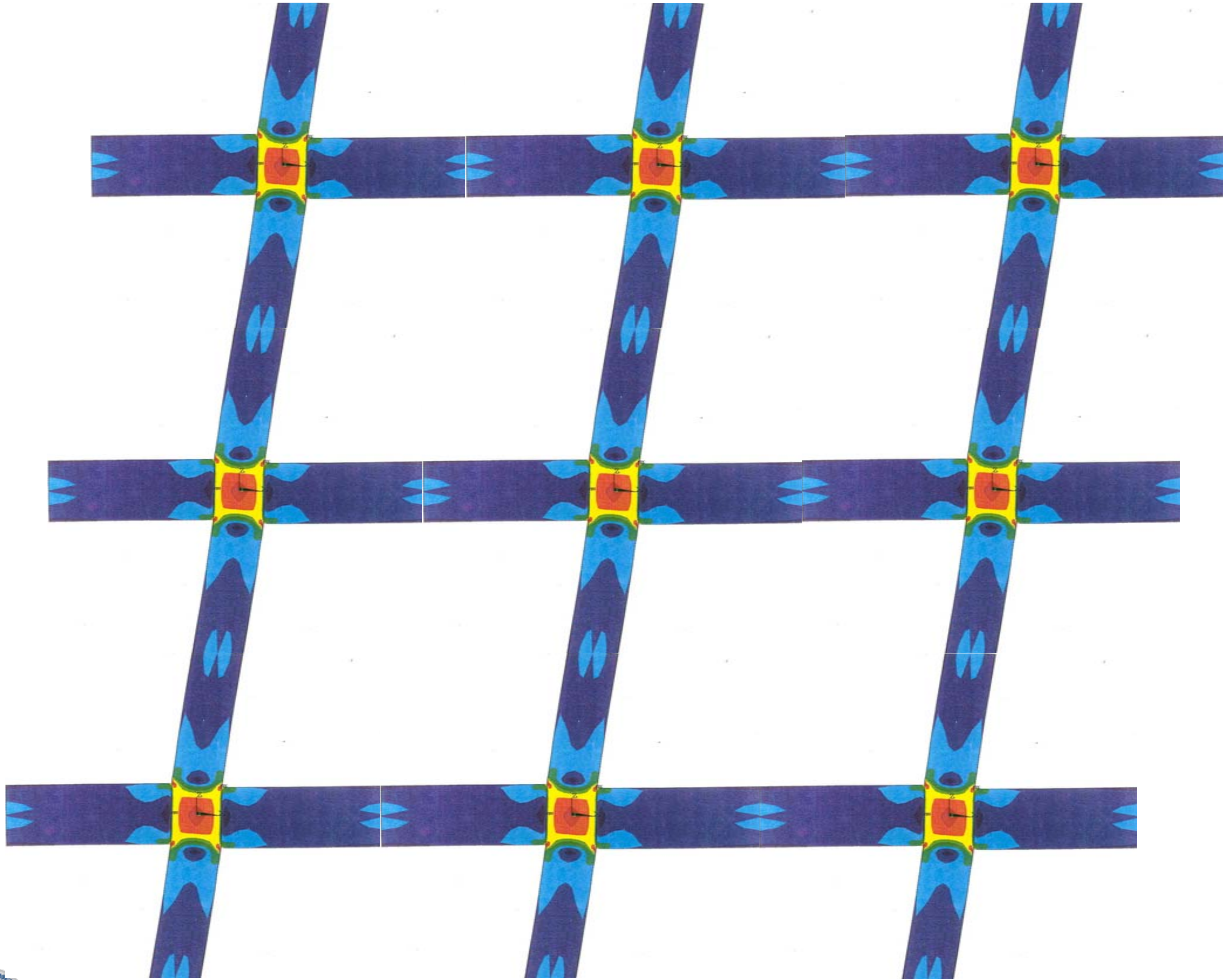
- Shear deformations in the panel zone can be responsible for 30 to 40 percent of the story drift. FEMA 350's statement that use of centerline dimensions in analysis will overestimate drift is *incorrect* for joints *without* PZ reinforcement.

- Without doubler plates, the panel zone will almost certainly yield before the girders do. Although panel zone yielding is highly ductile, it imposes high strains at the column flange welds, and may contribute to premature failure of the connection.

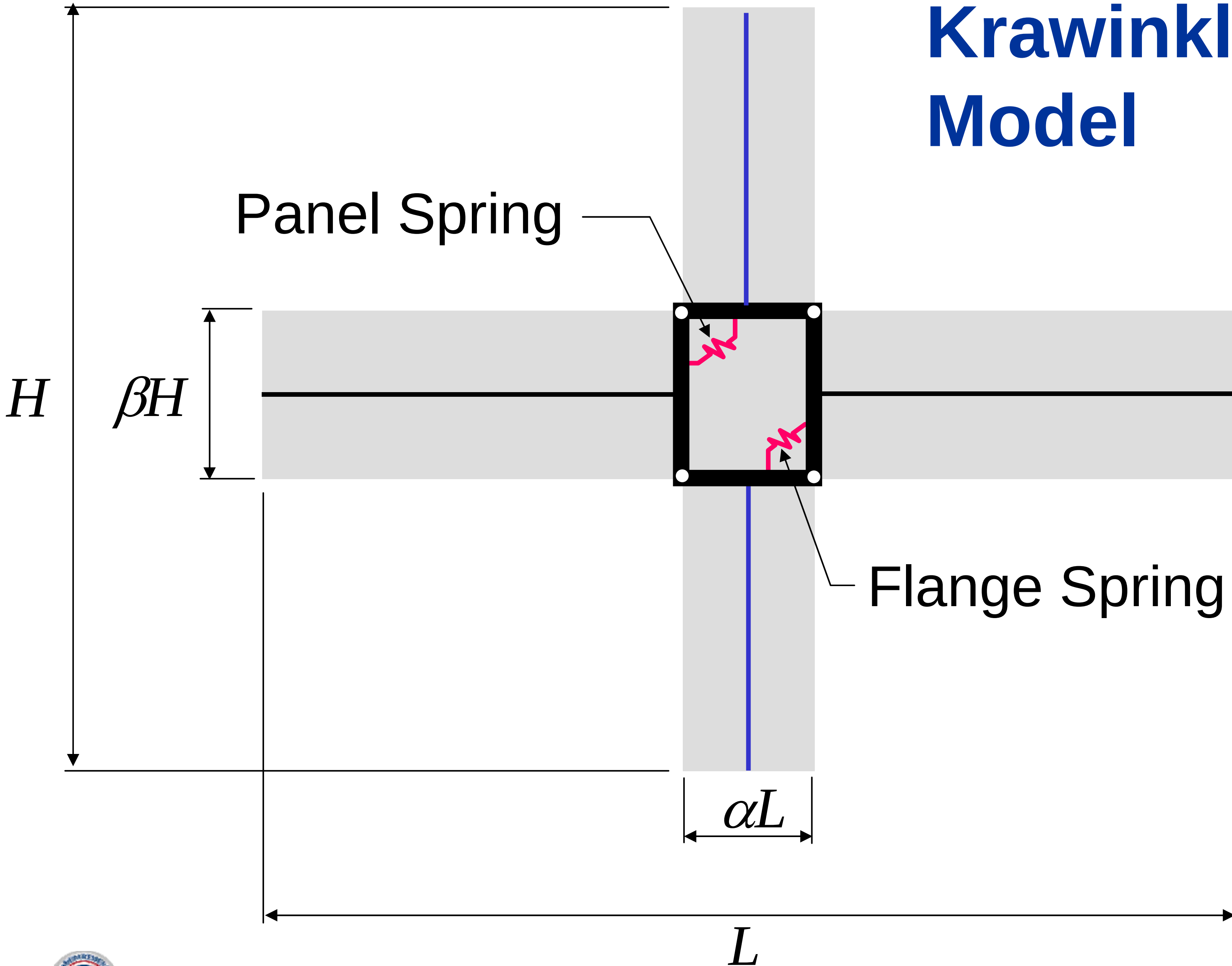
- Even with doubler plates, panel zones may yield. This inelastic behavior must be included in the model.

Sources of Inelastic Deformation in Typical Joint

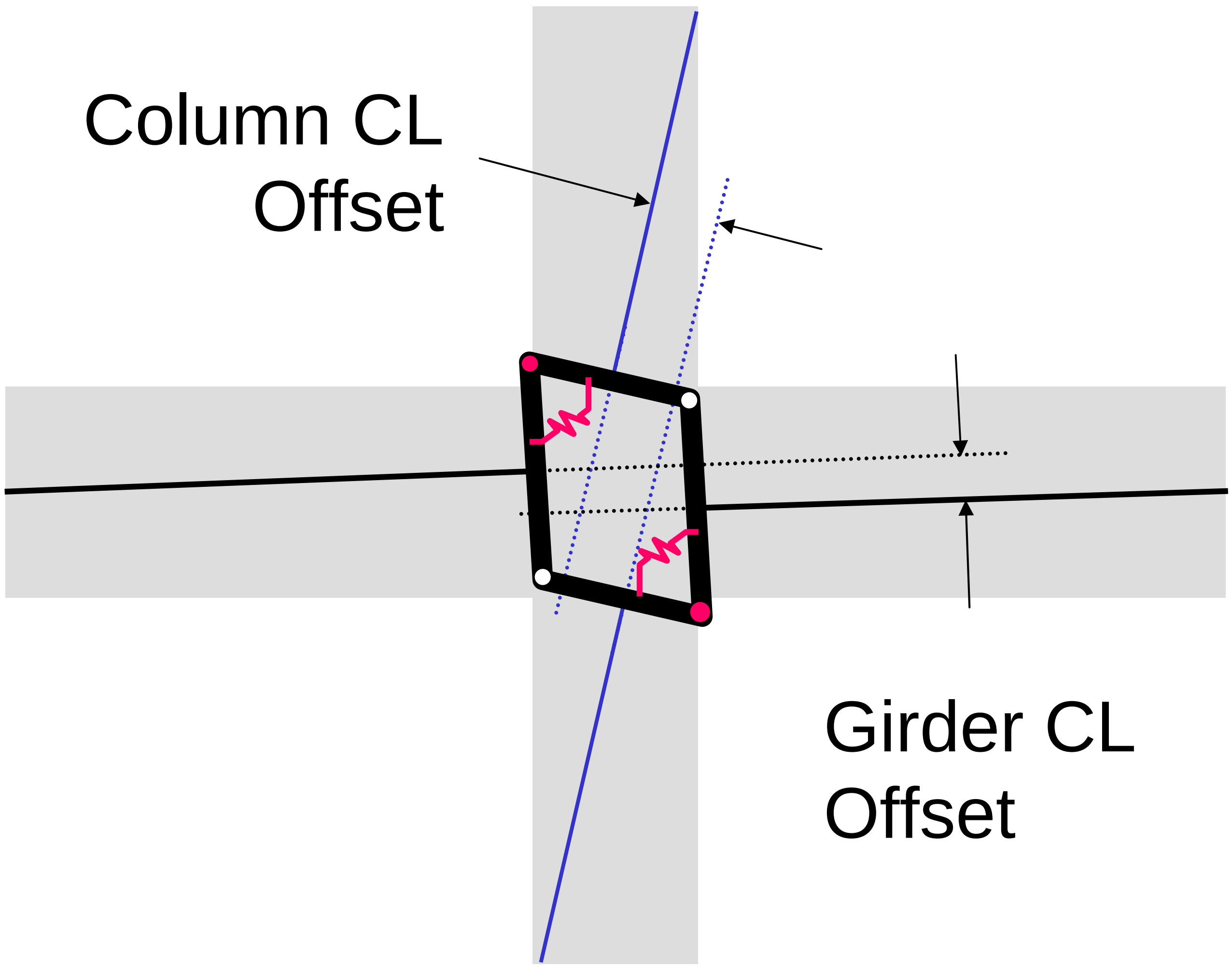




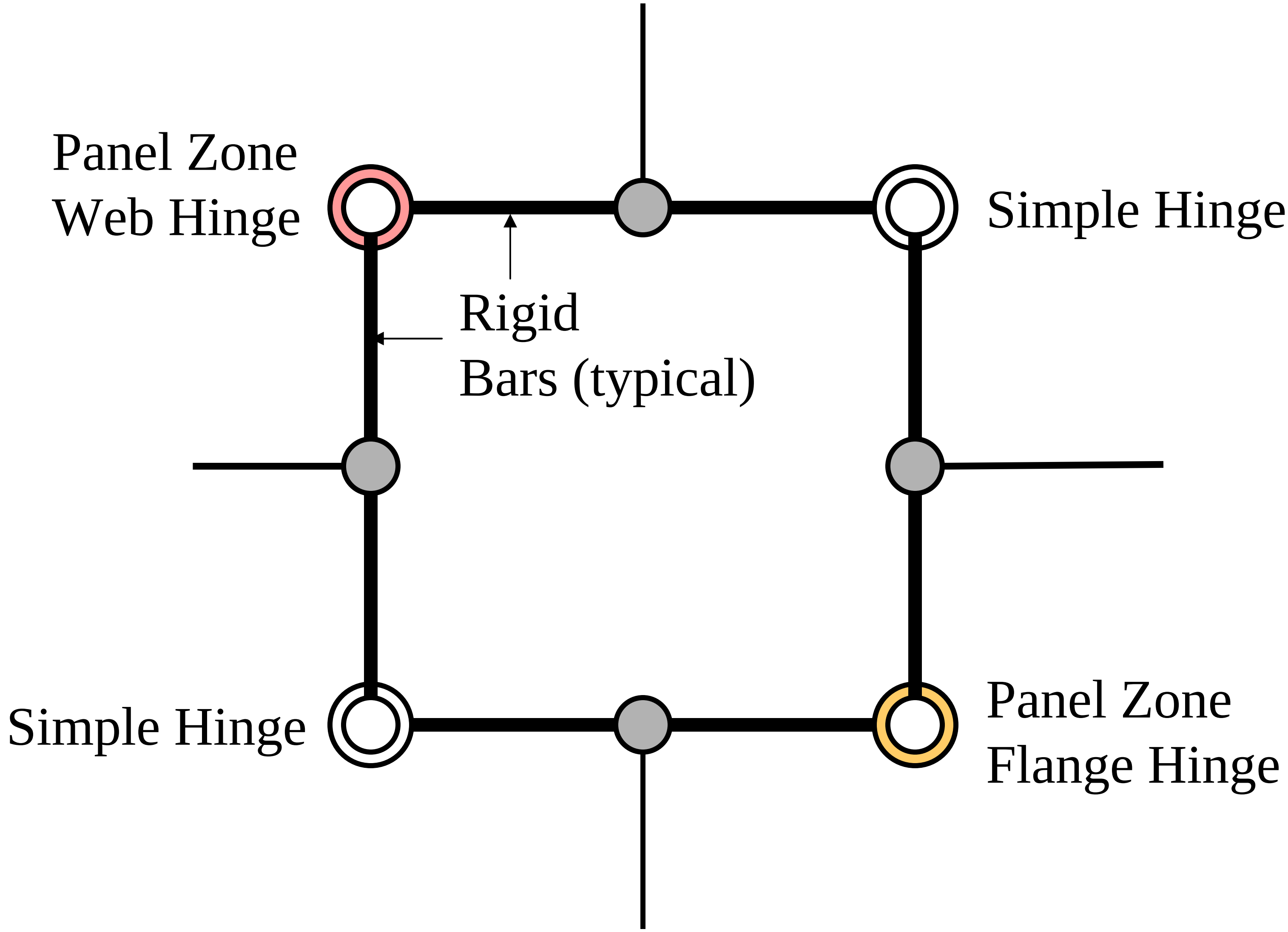
Krawinkler Model



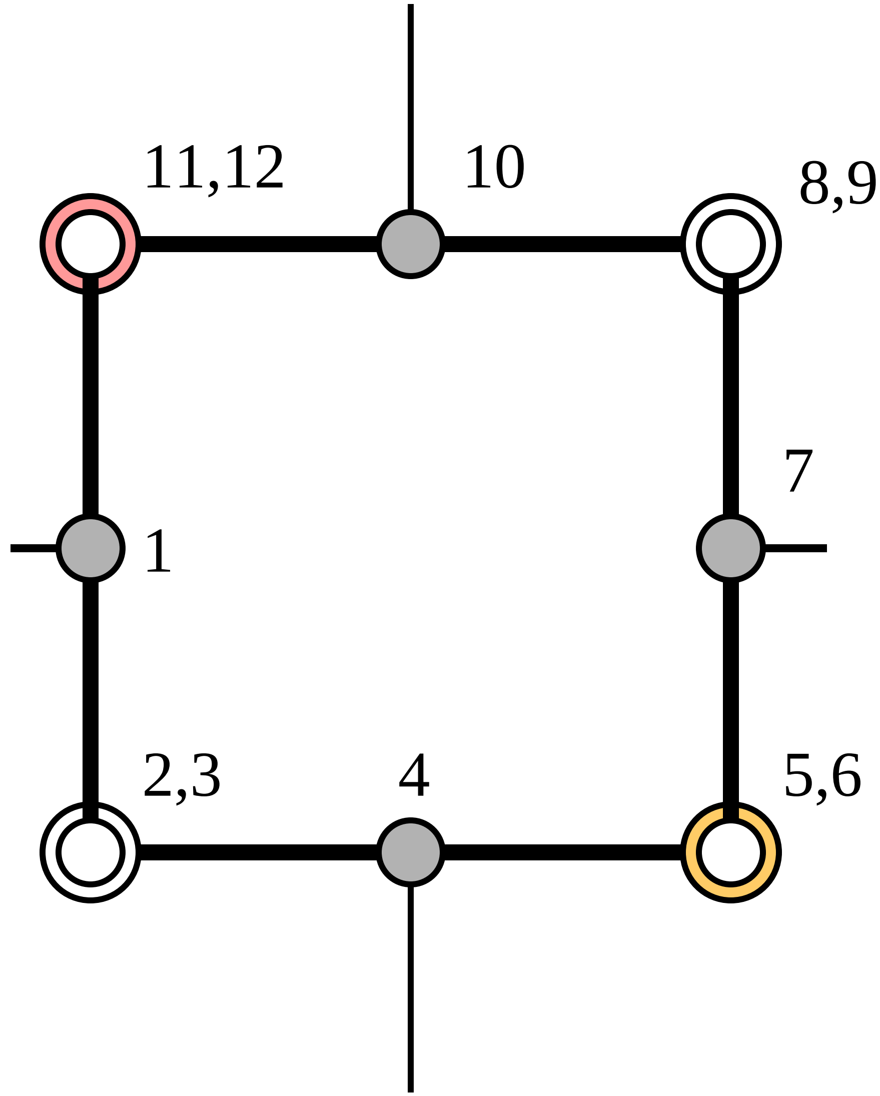
Kinematics of Krawinkler Model



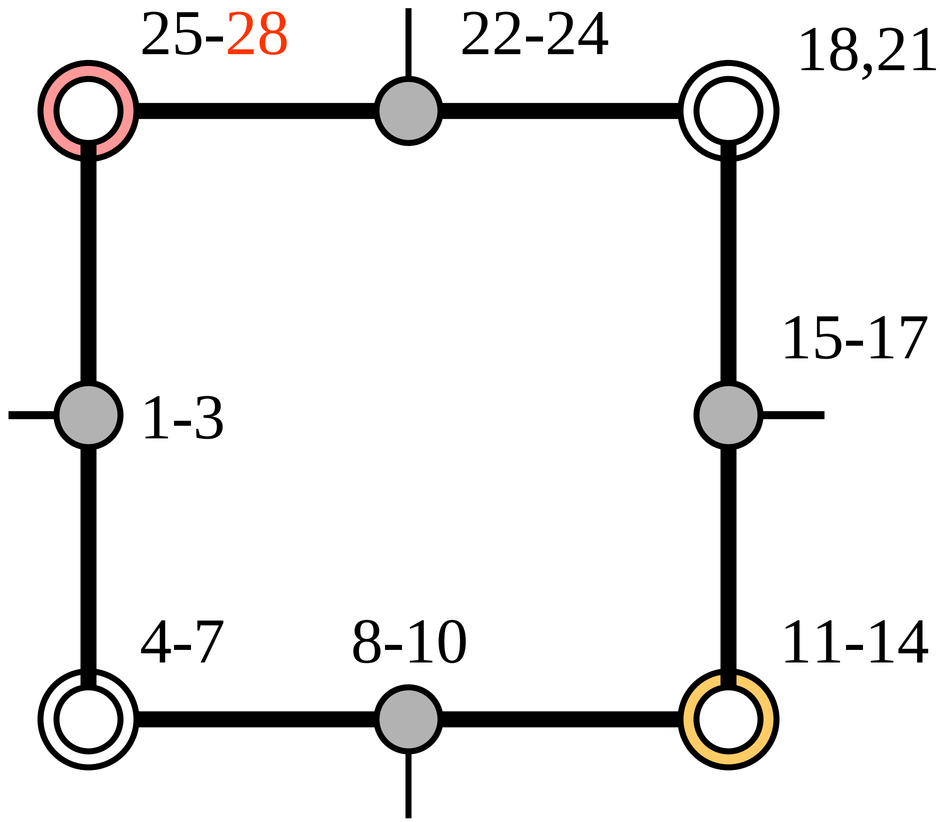
Krawinkler Joint Model



Nodes in Krawinkler Joint Model

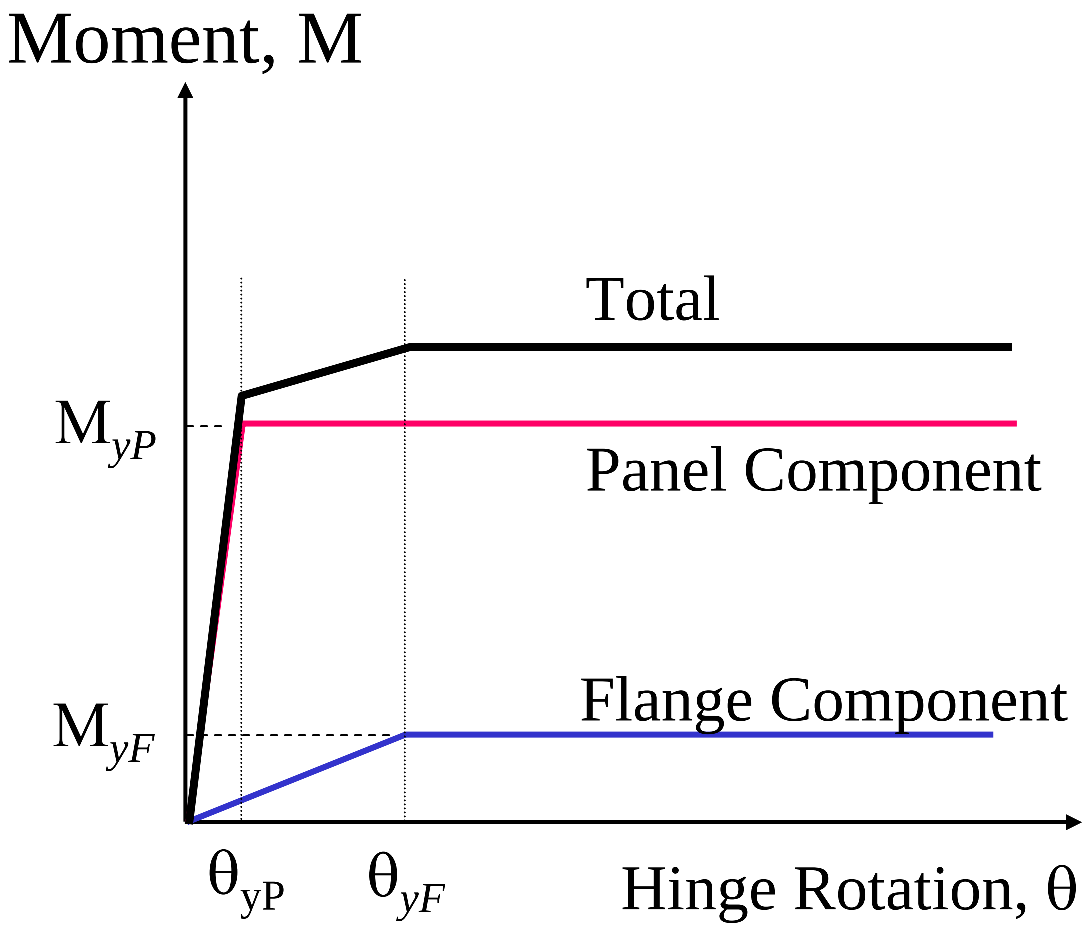


DOF in Krawinkler Joint Model

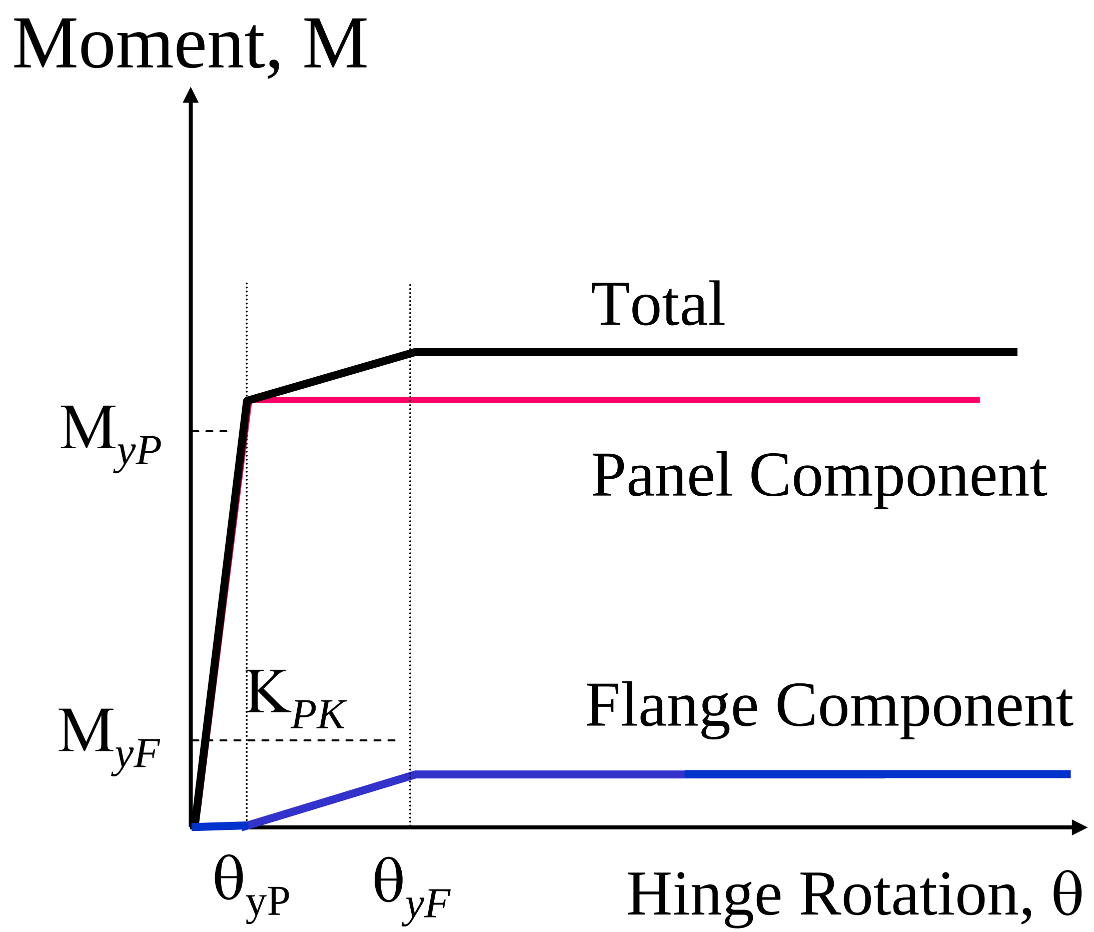


Note: Only FOUR DOF are truly independent.

Moment-Rotation Relationships in Krawinkler Model



Moment-Rotation Relationships in Krawinkler Model (Alternate)



Krawinkler Model Properties (Panel Component)

$$\mathrm{M}_{yP,K} = 0.6F_y \alpha L \beta H (t_{wc} + t_d)$$

$$K_{P,K} = G \alpha L \beta H (t_{wc} + t_d)$$

$$\theta_{yP,K} = \frac{0.6F_y}{G}$$

Krawinkler Model Properties (Panel Component)

$$My_{P,K} = 0.6F_y \boxed{\alpha L \beta H (t_{wc} + t_d)}$$

Volume of Panel

$$K_{P,K} = G \boxed{\alpha L \beta H (t_{wc} + t_d)}$$

Krawinkler Model Properties (Flange Component)

$$\mathrm{M}y_{F,K} = 1.8 F_y b_{cf} t_{cf}^2$$

$$\theta_{yF,K} = 4\theta_{yP,K}$$

Advantages of Krawinkler Model

- Physically mimics actual panel zone distortion and thereby accurately portrays true kinematic behavior

- Corner hinge rotation is the same as panel shear distortion

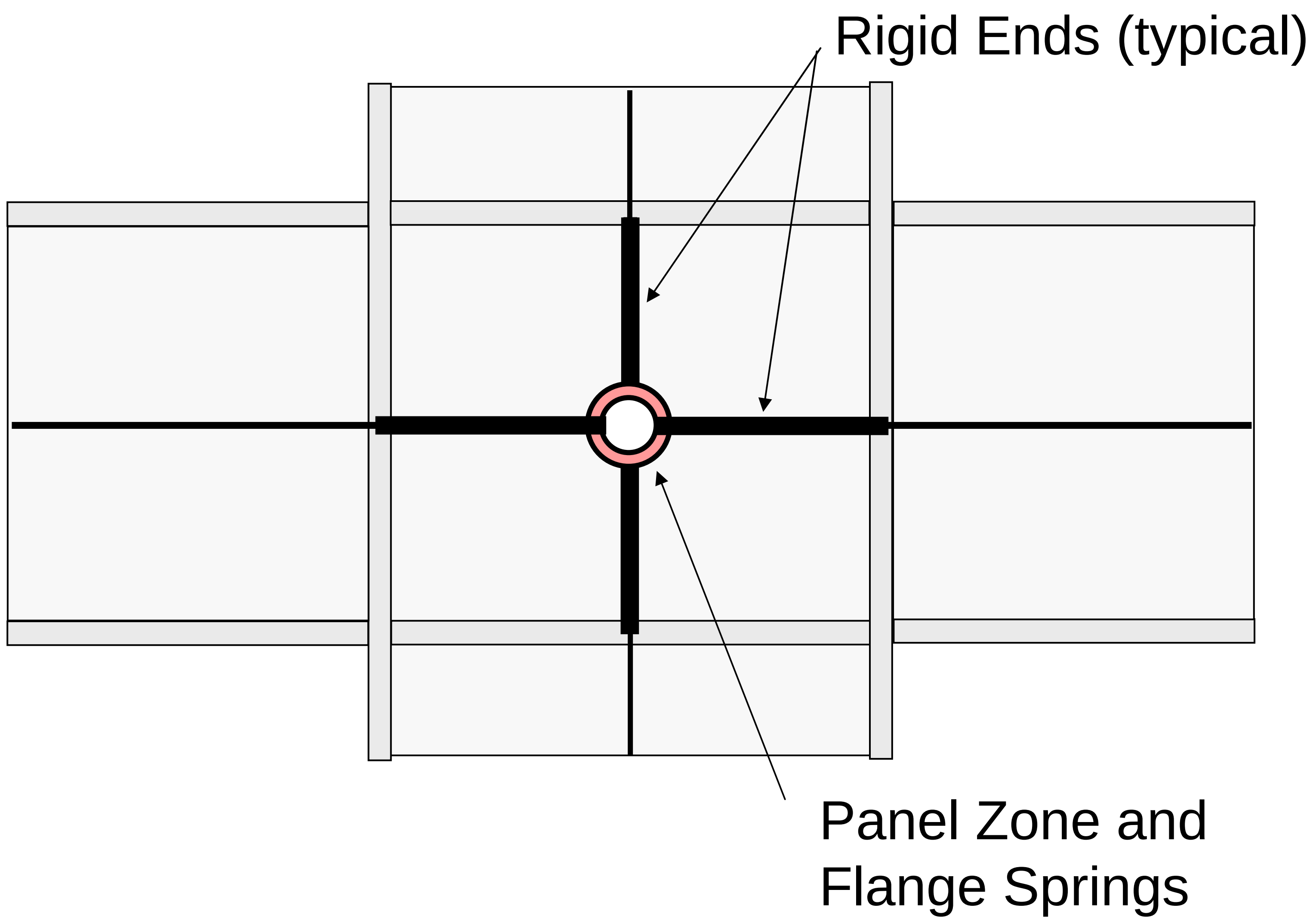
- Modeling parameters are independent of structure outside of panel zone region

Disadvantages of Krawinkler Model

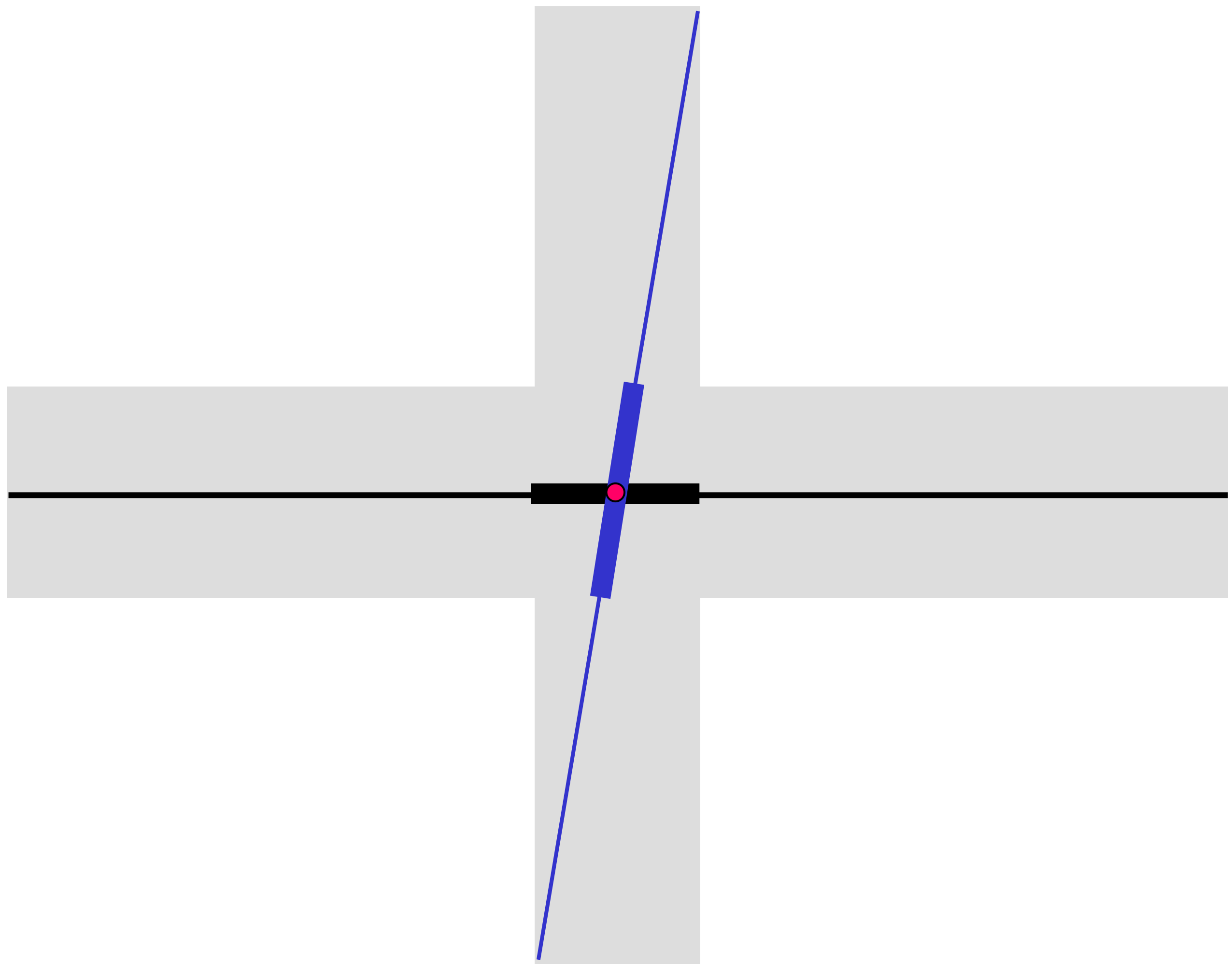
- Model is relatively complex
- Model does not include flexural deformations in panel zone region
- Requires 12 nodes, 12 elements, and 28 degrees of freedom

Note: Degrees of freedom can be reduced to four (4) through proper use of constraints, if available.

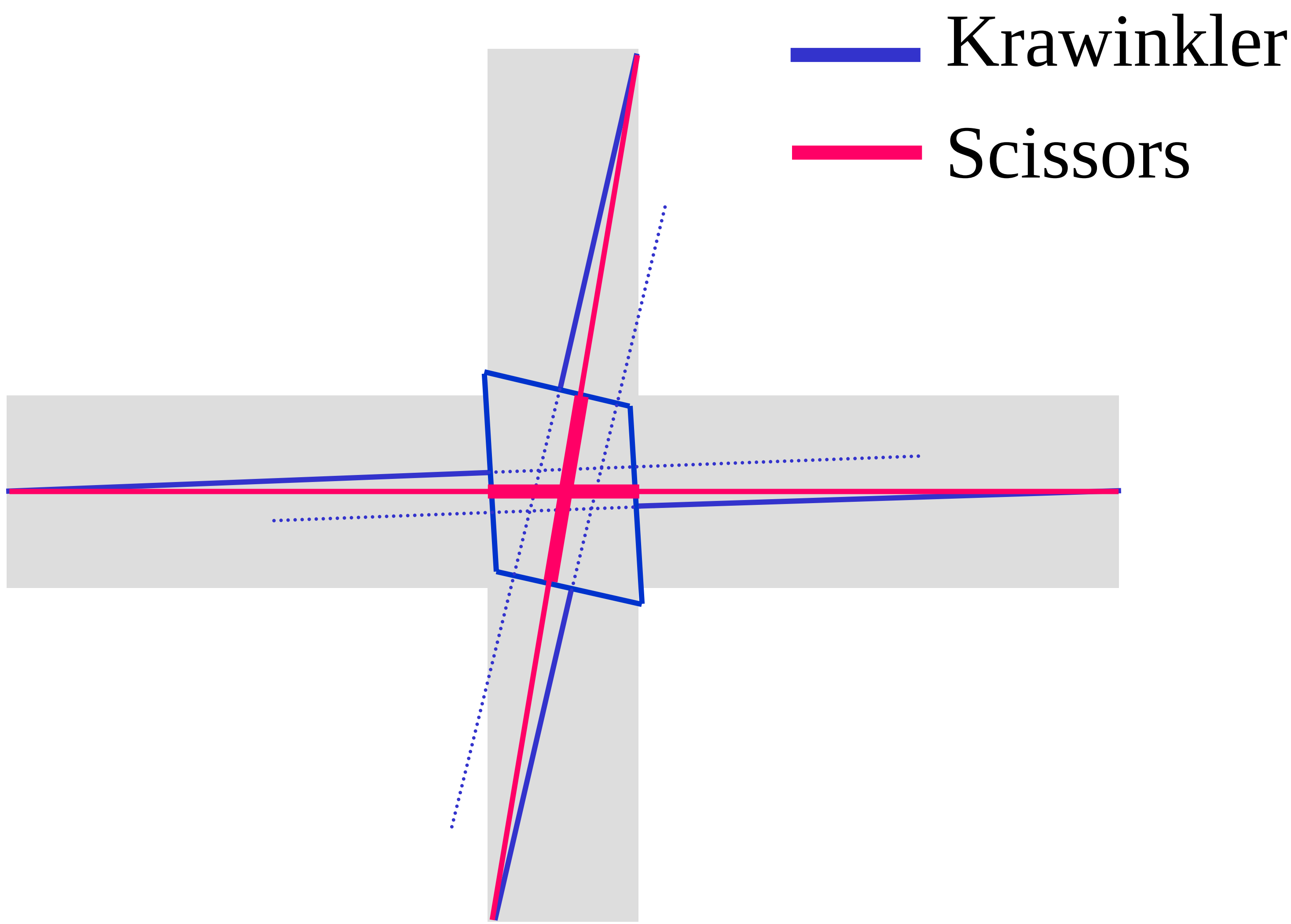
Scissor Joint Model



Kinematics of Scissors Model



Model Comparison: Kinematics



Mathematical Relationship Between Krawinkler and Scissors Models

$$K_{Scissors} = \frac{K_{Krawinkler}}{(1-\alpha-\beta)^2}$$

$$M_{y,Scissors} = \frac{M_{y,Krawinkler}}{(1-\alpha-\beta)}$$

Advantage of Scissors Model

- Relatively easy to model (compared to Krawinkler). Only 4 DOF per joint, and only two additional elements.

- Produces almost identical results as Krawinkler.

Disadvantages of Scissors Model

- Does not model true behavior in joint region.
- Does not include flexural deformations in panel zone region
- Not applicable to structures with unequal bay width (model parameters depend on α and β)

Modeling Beam-Column Joint Deformation in Concrete Structures

- Accurate modeling is much more difficult (compared to structural steel) due to pullout and loss of bond of reinforcement and due to loss of stiffness and strength of concrete in the beam-column joint region.

- Physical models similar to the Krawinkler Steel Model are under development. See reference by Lowes and Altoontash.

When to Include P-Delta Effects?

2000 NEHRP Provisions 5A.1.1:

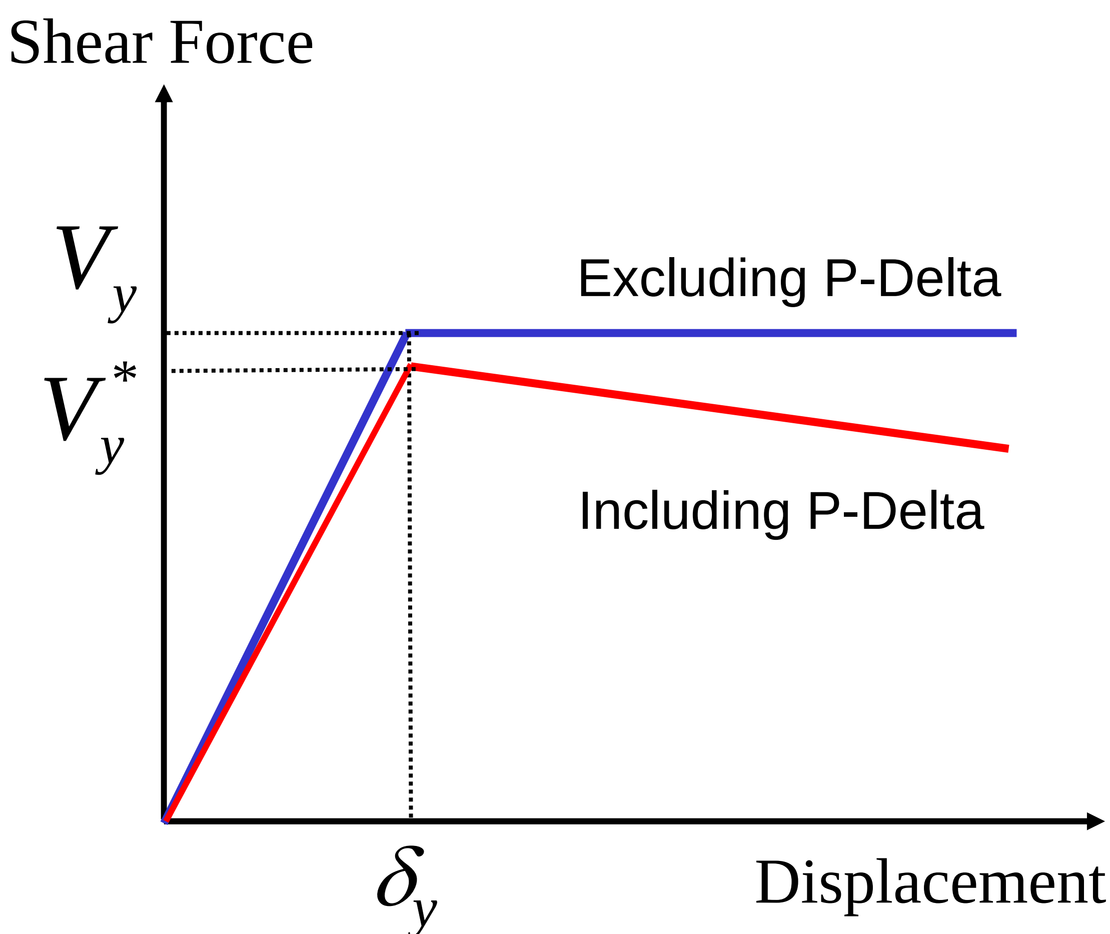
" The models for columns should reflect the influence of axial load when axial loads exceed 15 percent of the buckling load"

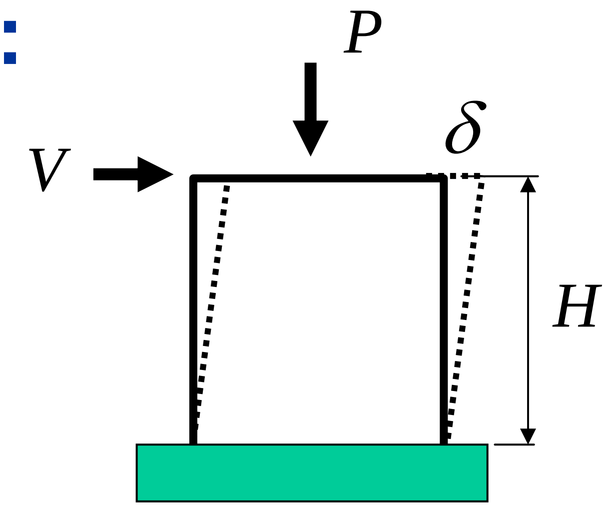
Recommended Revision:

"P-Delta effects must be explicitly included in the computer model of the structure."

Influence of P-Delta Effects:

1) Loss of Stiffness and increased displacements



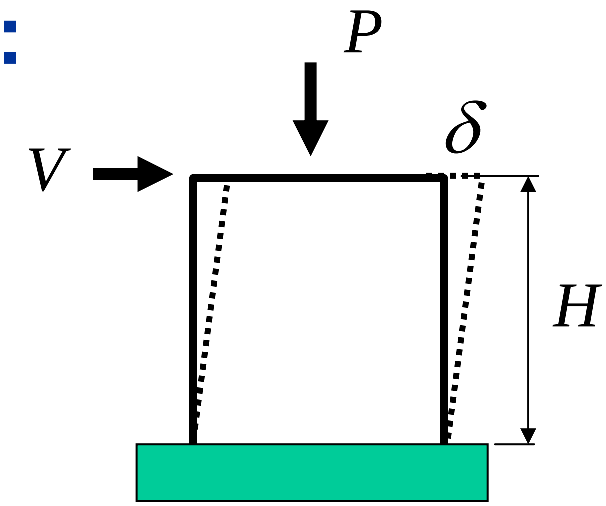


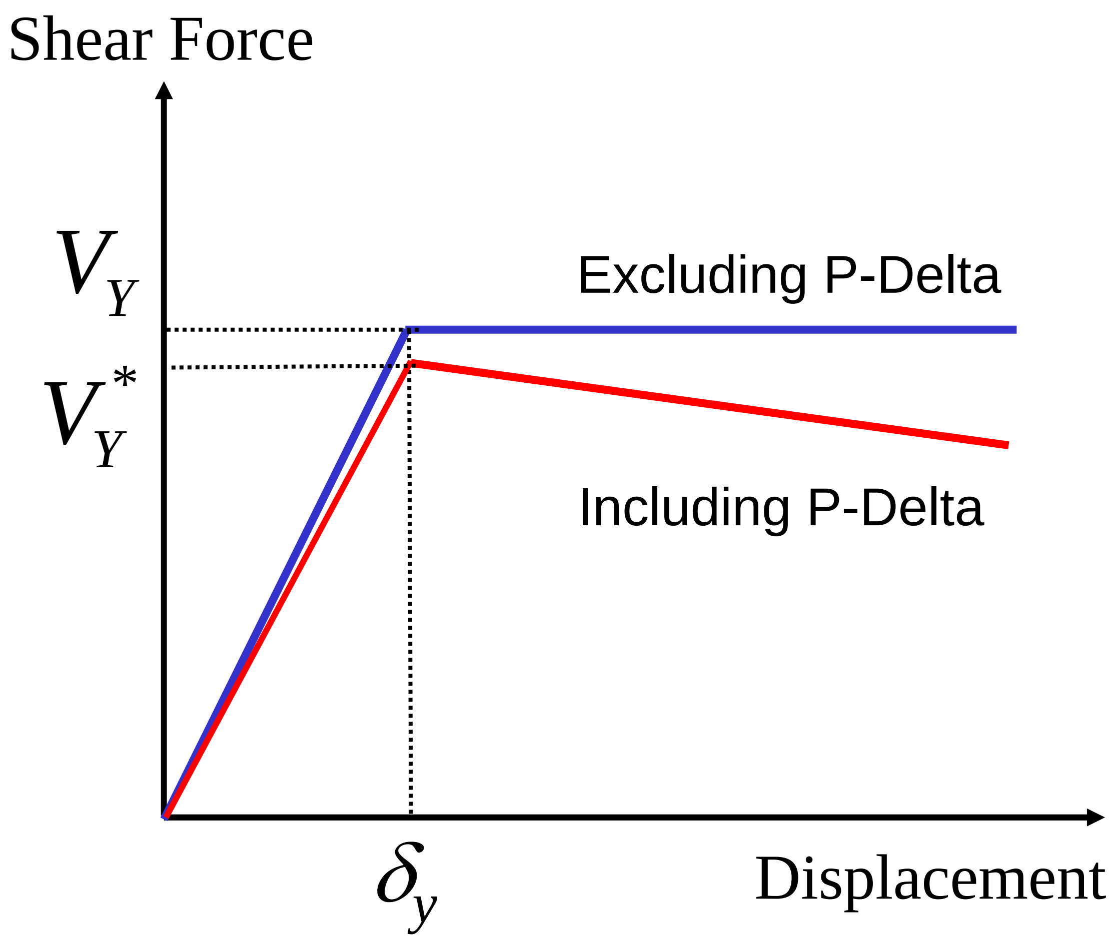
$$K_G = -\frac{P}{H}$$

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

$$K = K_E + K_G$$

Influence of P-Delta Effects:
2) Loss of Strength



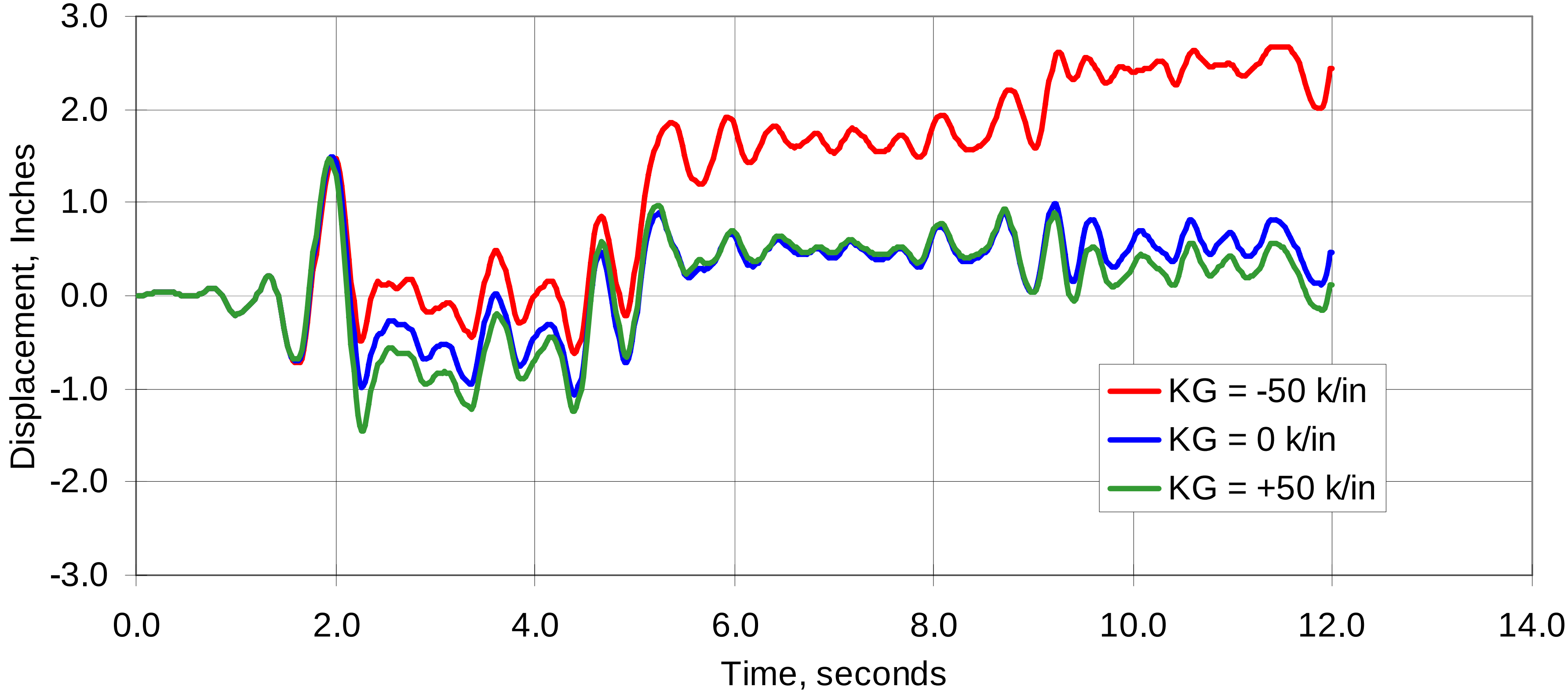


$$\theta = \frac{P\delta_y}{V_y H}$$

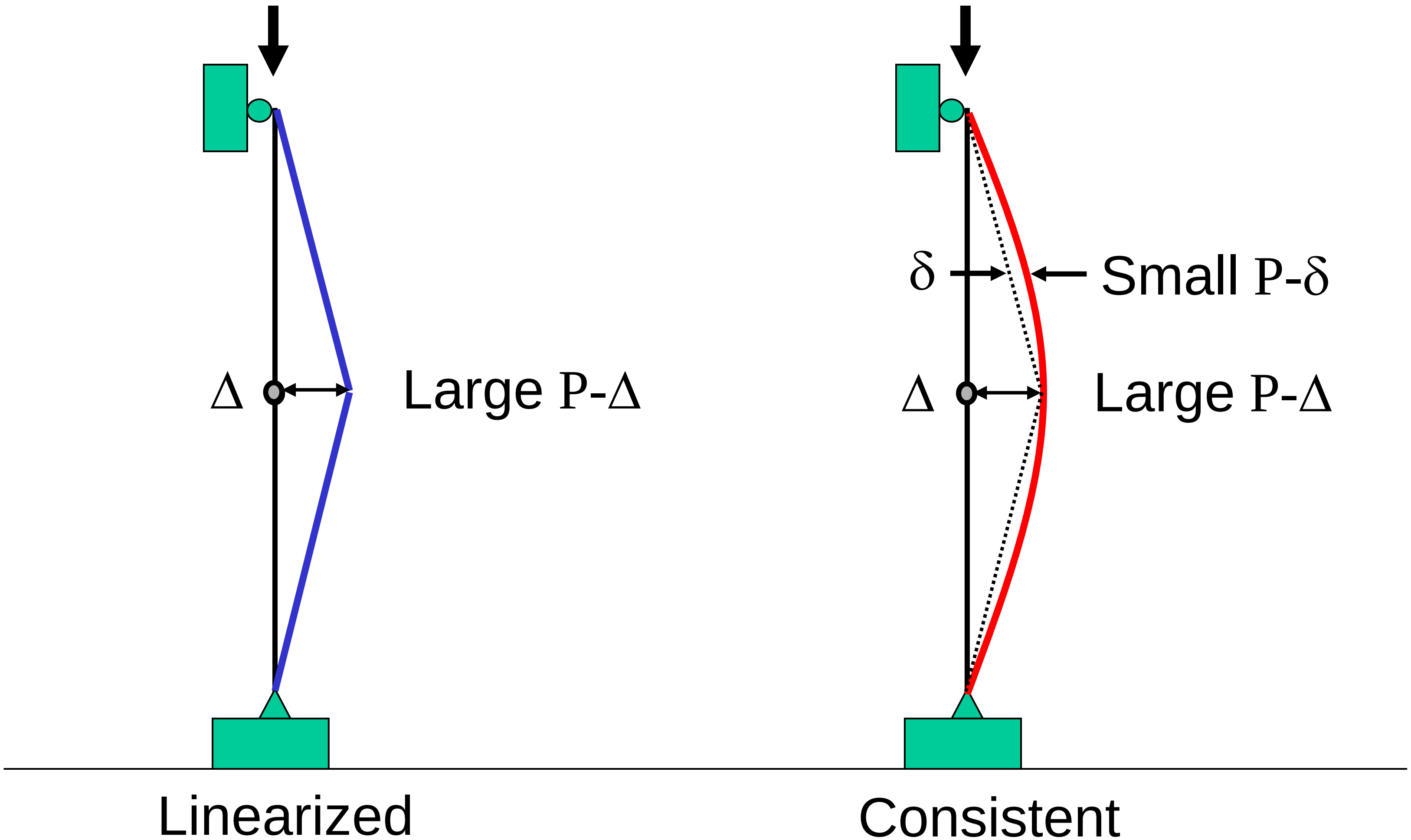
$$V_y^* = V_y(1-\theta)$$

Influence of P-Delta Effects:

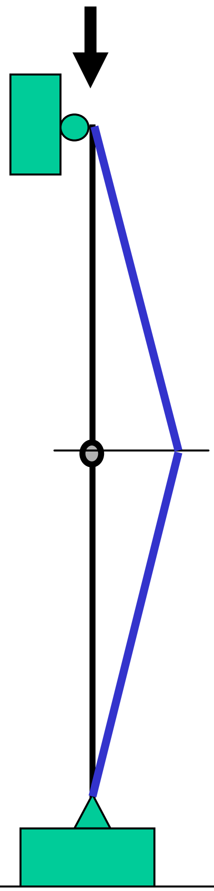
3) Larger residual deformations and increased tendency towards dynamic instability



Modeling P-Delta Effects
Linearized vs Consistent Geometric Stiffness



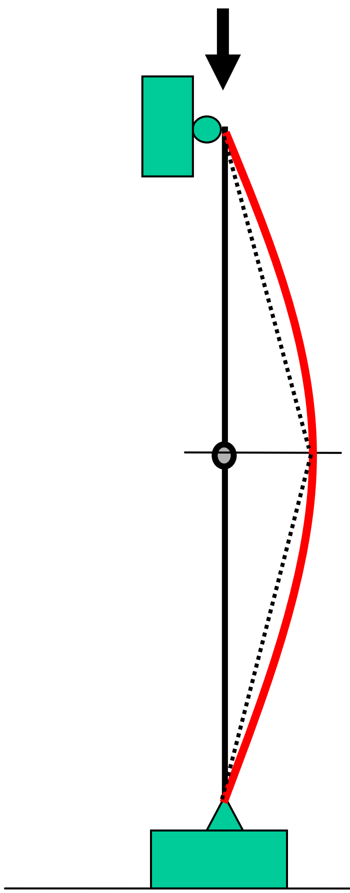
Modeling P-Delta Effects
Linearized Geometric Stiffness



- Uses linear shape function to represent displaced shape. No iteration required for solution.
- Solution based on undeformed geometry
- Significantly overestimates buckling loads for individual columns
- Useful ONLY for considering the "Large P-Delta" Effect on a story-by-story basis

Linearized

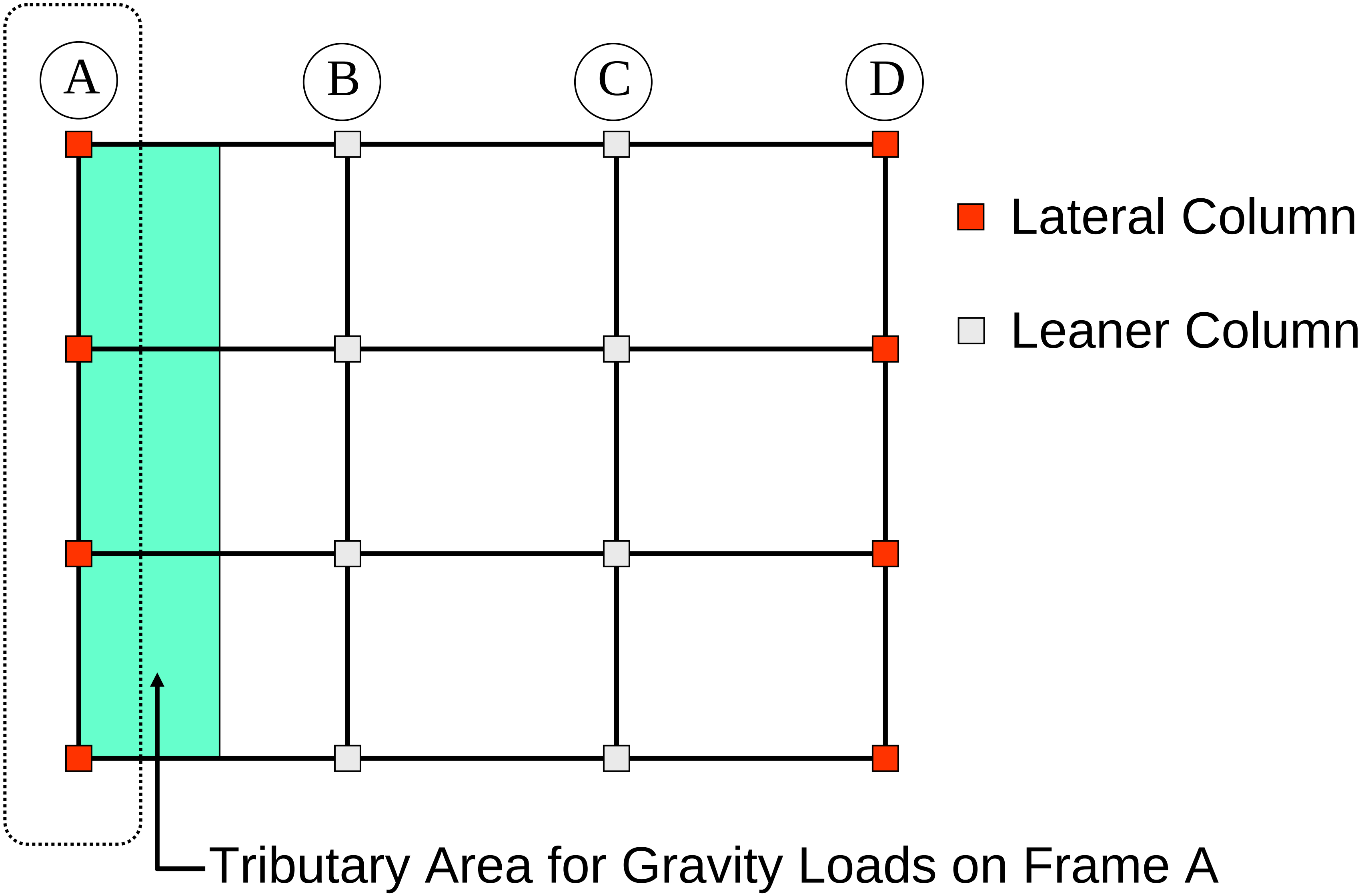
Modeling P-Delta Effects Consistent Geometric Stiffness



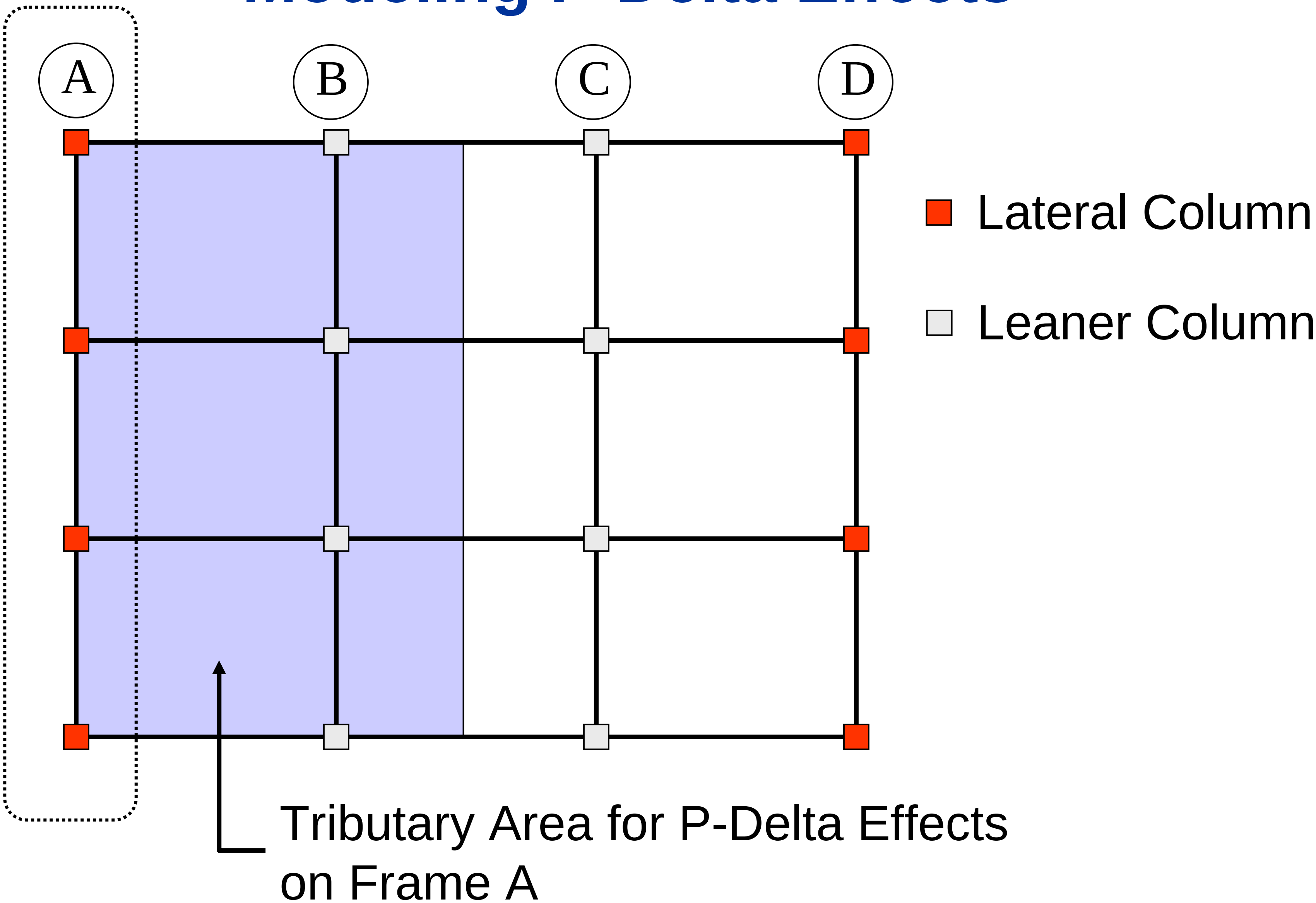
Consistent

- Uses cubic shape function to represent displaced shape. Iteration required for solution.
- Solution based on undeformed geometry
- Accurately estimates buckling loads for individual columns *only if each column is subdivided into two or more elements.*
- Does not provide significant increase in accuracy (compared to linearized model) if being used only for considering the "Large P-Delta" effect in moment resisting frame structures.

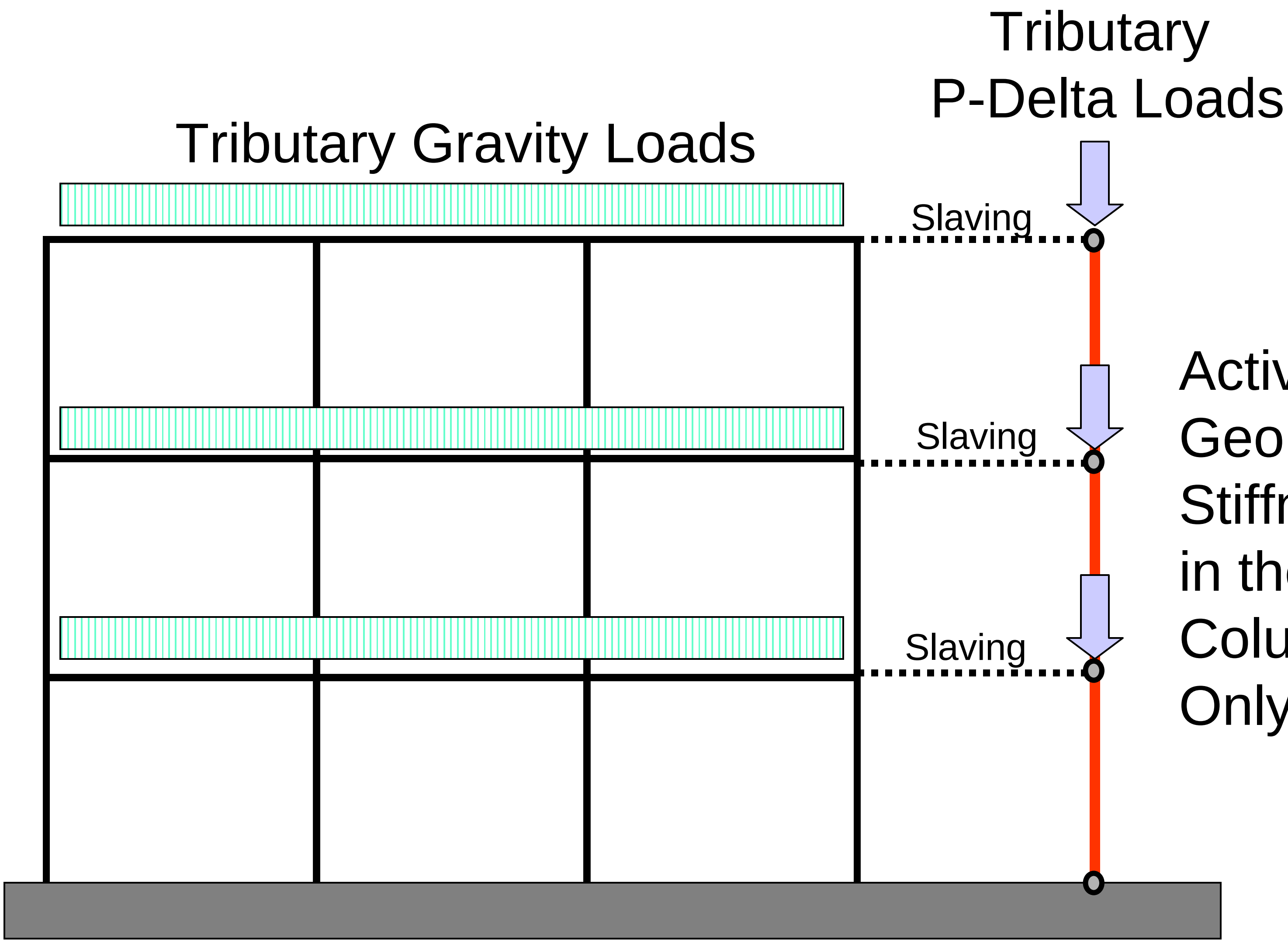
Modeling P-Delta Effects



Modeling P-Delta Effects



Modeling P-Delta Effects



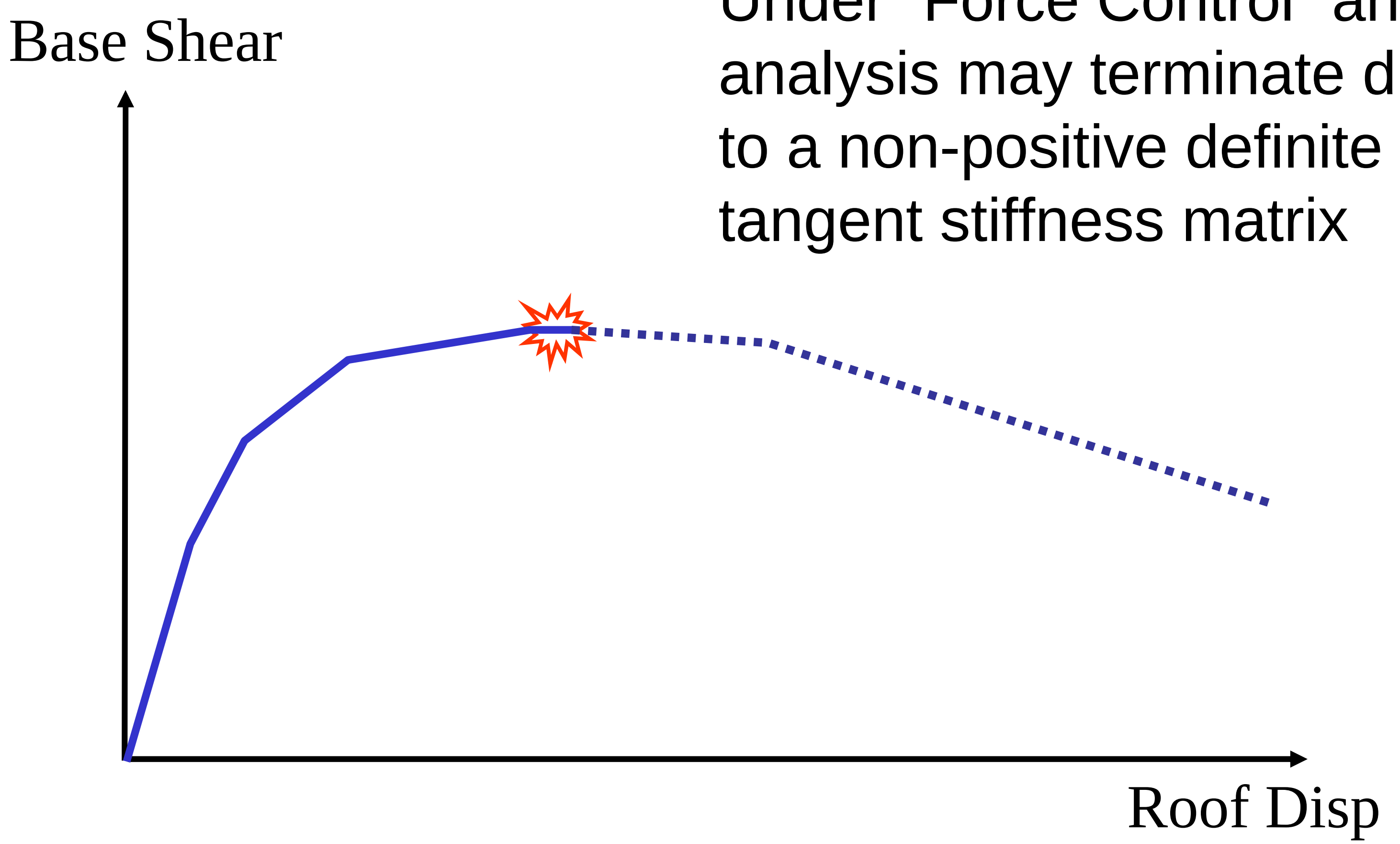
Activate Geometric Stiffness in these Columns Only.

How Much Gravity Load to Include for P-Delta Analysis?

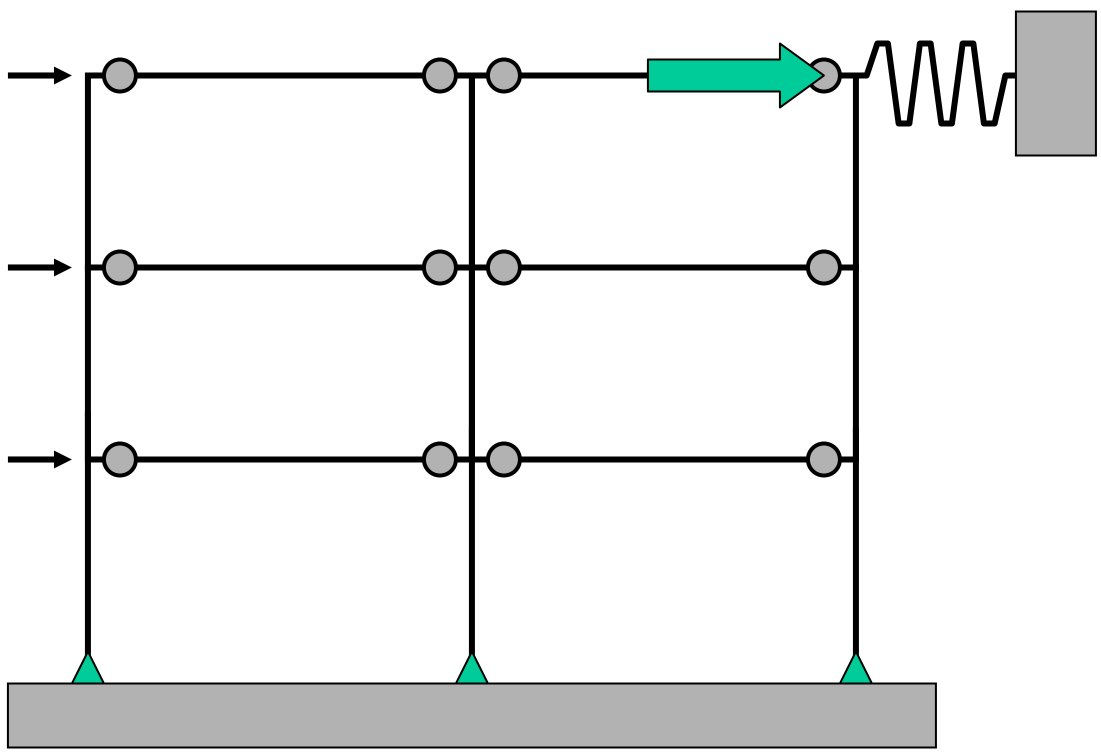
- Full Dead Load
- 10 PSF Partition Load (or computed value if available)
- Full Reduced Live Load (as would be used for column design).
- Reduced Live Load based on most probable live load. See for example Commentary of ASCE 7.
- Effect of Vertical Accelerations?

Modeling P-Delta Effects

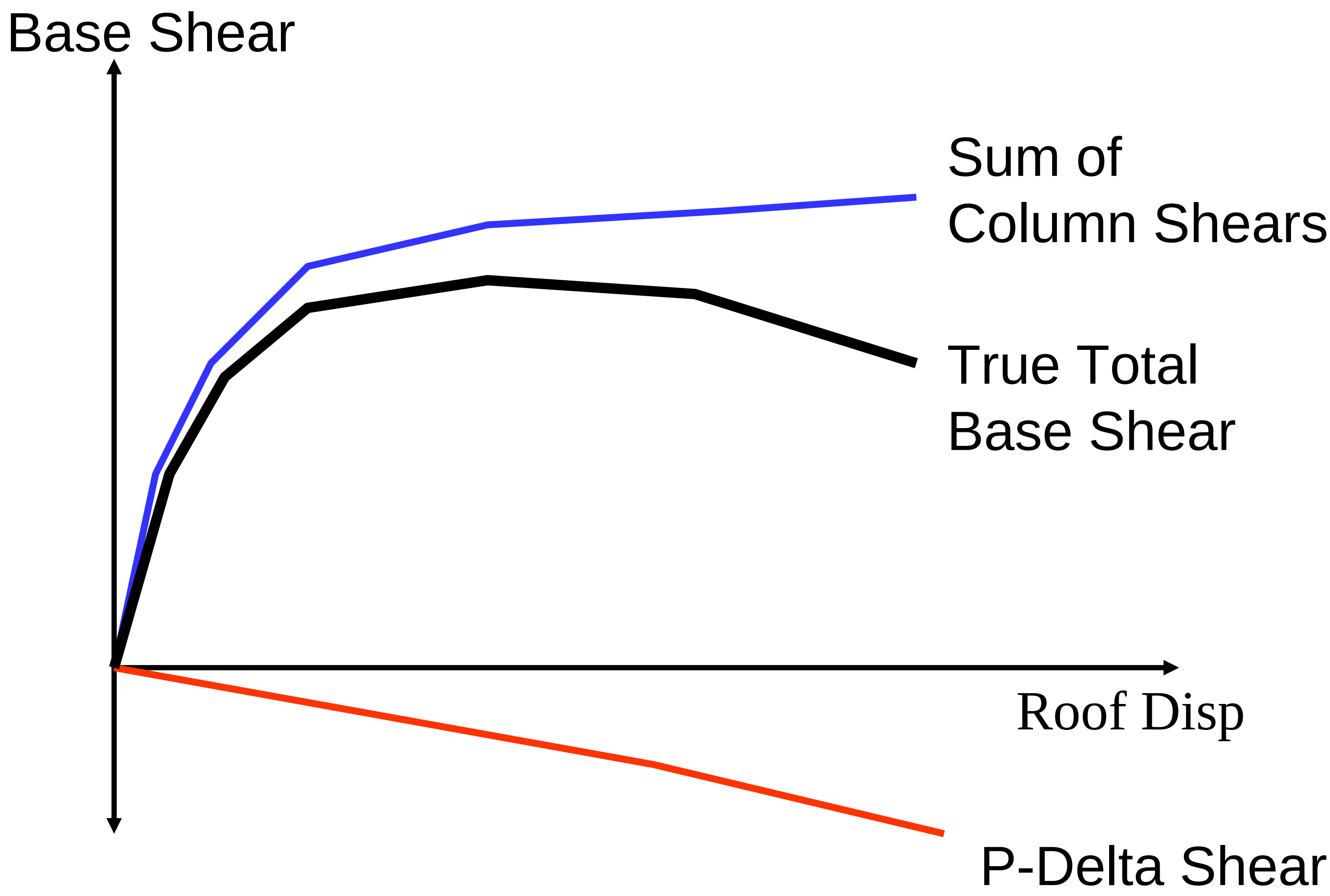
Under "Force Control" an analysis may terminate due to a non-positive definite tangent stiffness matrix



Must Use Displacement Controlled Analysis to Obtain Complete Response



When Using Displacement Control (or response-history analysis), do not recover base shears from column forces.



Structural Analysis for Performance-Based Earthquake Engineering

- Basic modeling concepts
- **Nonlinear static pushover analysis**
- Nonlinear dynamic response history analysis
- Incremental nonlinear analysis
- Probabilistic approaches

Nonlinear Static Pushover Analysis

- Why pushover analysis?
- Basic overview of method
- Details of various steps
- Discussion of assumptions
- Improved methods

Why Pushover Analysis?

- Performance-based methods require reasonable estimates of inelastic deformation or damage in structures.

- Elastic Analysis is not capable of providing this information.

- Nonlinear dynamic response history analysis is capable of providing the required information, but may be very time-consuming.

Why Pushover Analysis?

- Nonlinear static pushover analysis may provide reasonable estimates of location of inelastic behavior.

- Pushover analysis alone is not capable of providing estimates of maximum deformation. Additional analysis must be performed for this purpose. The fundamental issue is…
How Far to Push?

Why Pushover Analysis?

- It is important to recognize that the purpose of pushover analysis is not to predict the actual response of a structure to an earthquake. (It is unlikely that nonlinear dynamic analysis can predict the response.)

- The minimum requirement for any method of analysis, including pushover, is that it must be "good enough for design".

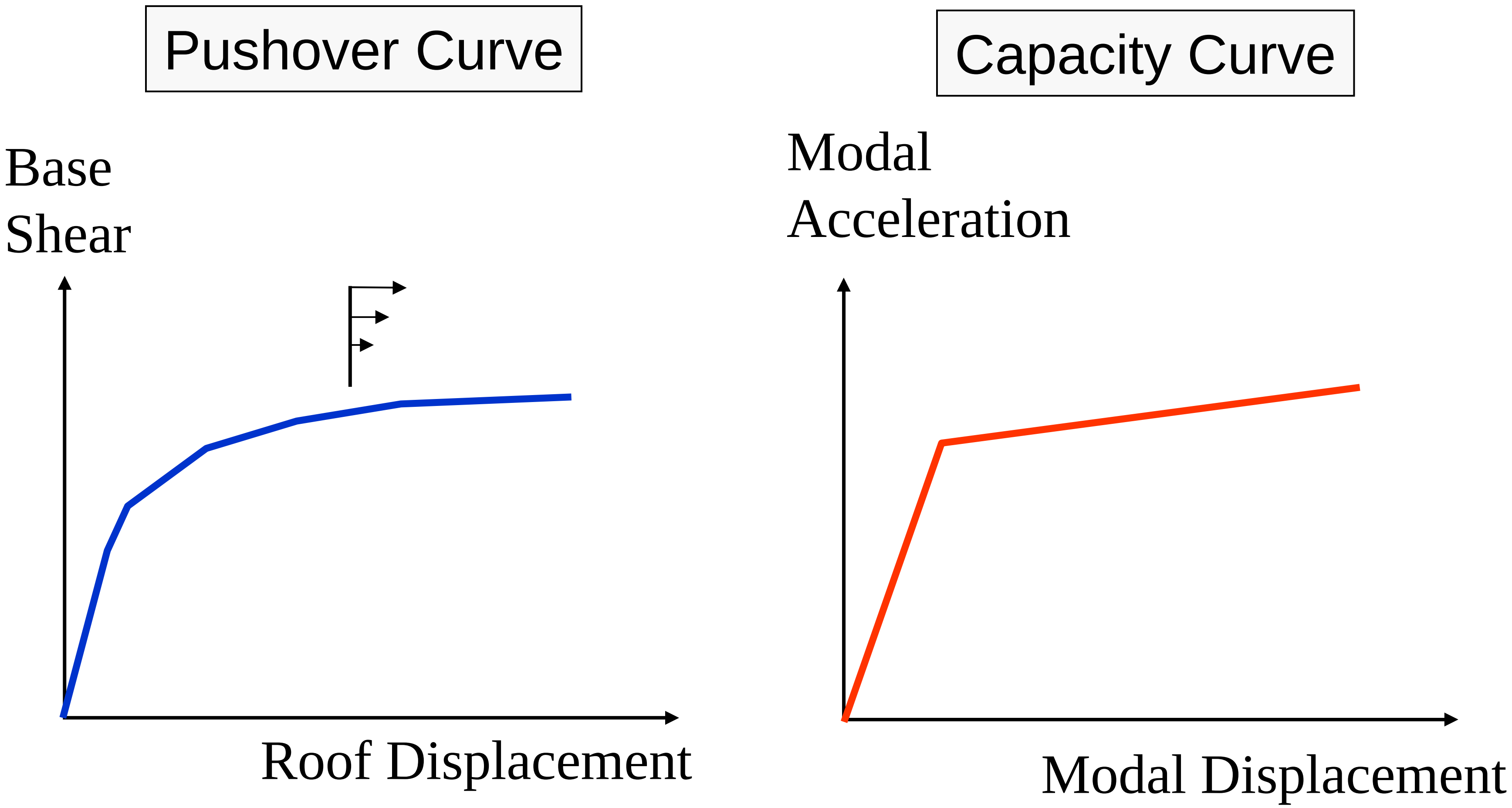
Basic Overview of Method

- Development of Capacity Curve
- Prediction of “Target Displacement”
  - Capacity-Spectrum Approach (ATC 40)
  - Simplified Approach (FEMA 273, NEHRP)
  - Uncoupled Modal Response History
  - Modal Pushover

Development of the Capacity Curve (ATC 40 Approach)

1. Develop Analytical Model of Structure Including:
   - Gravity loads
   - Known sources of inelastic behavior
   - P-Delta Effects
2. Compute Modal Properties:
   - Periods and Mode Shapes
   - Modal Participation Factors
   - Effective Modal Mass
3. Assume Lateral Inertial Force Distribution
4. Construct Pushover Curve
5. Transform Pushover Curve to 1st Mode Capacity Curve
6. Simplify Capacity Curve (Use bilinear approximation)

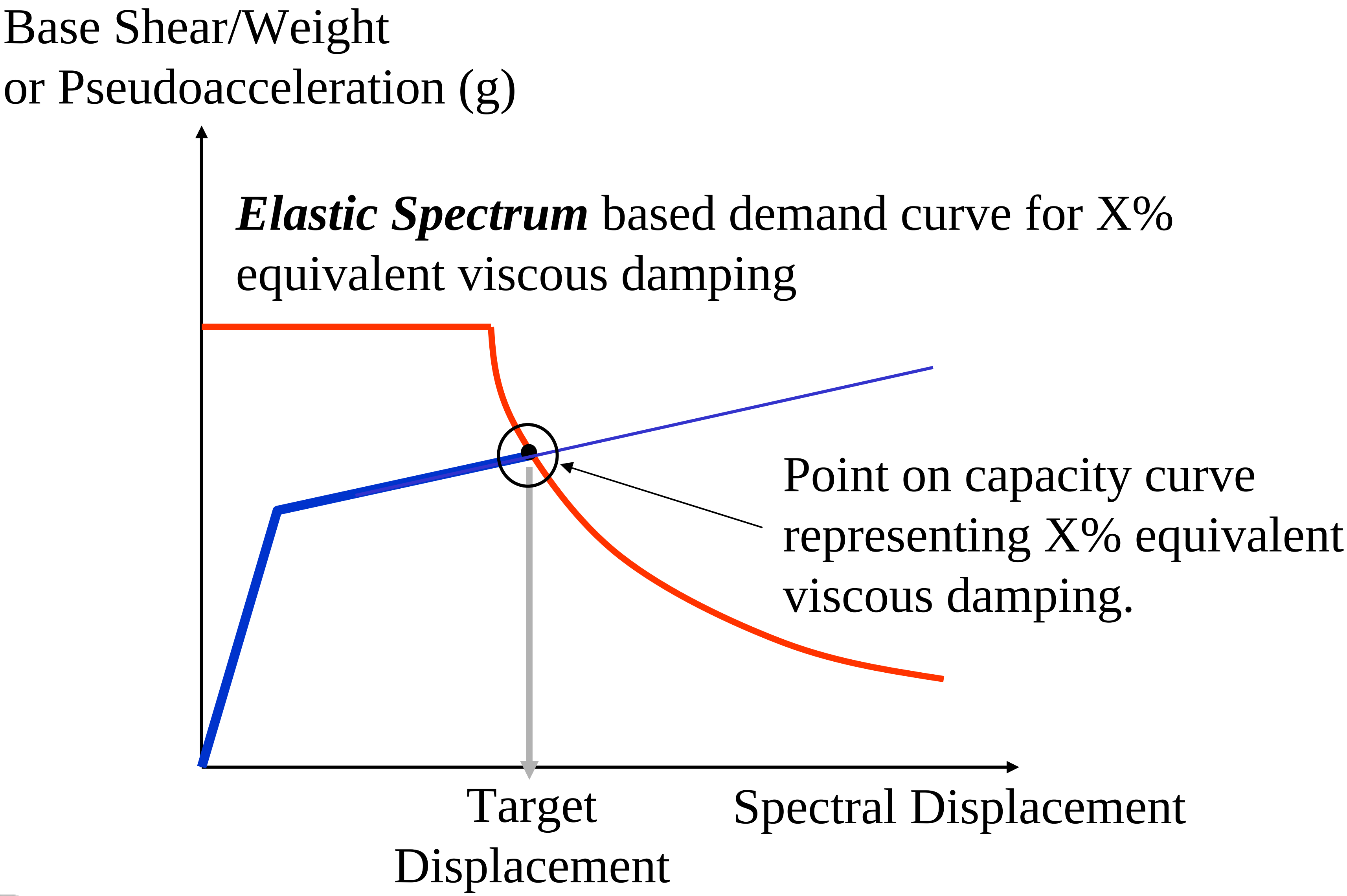
Development of the Capacity Curve



Development of the Demand Curve

1. Assume Seismic Hazard Level (e.g 2% in 50 years)
2. Develop 5% Damped *ELASTIC* Response Spectrum
3. Modify for Site Effects
4. Modify for Expected Performance and Equivalent Damping
5. Convert to Displacement-Acceleration Format

Elastic Spectrum Based Target Displacement



Review of MDOF Dynamics (1)

Original Equations of Motion:

$$M\ddot{u} + C\dot{u} + Ku = -MR\ddot{u}_g \qquad K\Phi = M\Phi\Omega^2 \qquad R = \begin{Bmatrix} 1 \\ 1 \\ \cdot \\ 1 \end{Bmatrix}$$

Transformation to Modal Coordinates:

$$u = \Phi y$$

$$\Phi = \left[\phi_1 \ \phi_2 \ \phi_3 \ \ldots \ \phi_n\right] \qquad y = \begin{Bmatrix} y_1 \\ y_2 \\ \cdot \\ y_n \end{Bmatrix}$$

$$\boxed{M\Phi\ddot{y} + C\Phi\dot{y} + K\Phi y = -MR\ddot{u}_g}$$

Review of MDOF Dynamics (2)

Use of Orthogonality Relationships:

$$\Phi^{\mathrm{T}} M \Phi \ddot{y} + \Phi^{\mathrm{T}} C \Phi \dot{y} + \Phi^{\mathrm{T}} K \Phi y = -\Phi^{\mathrm{T}} M R \ddot{u}_g$$

$$\Phi^{\mathrm{T}} M \Phi = M^* \qquad \phi_i^{\mathrm{T}} M \phi_i = m_i^*$$

$$\Phi^{\mathrm{T}} C \Phi = C^* \qquad \phi_i^{\mathrm{T}} C \phi_i = c_i^*$$

$$\Phi^{\mathrm{T}} K \Phi = K^* \qquad \phi_i^{\mathrm{T}} K \phi_i = k_i^*$$

SDOF equation in Mode i :

$$\boxed{m_i^* \ddot{y}_i + c_i^* \dot{y}_i + k_i^* y_i = -\phi_i^{\mathrm{T}} M R \ddot{u}_g}$$

Review of MDOF Dynamics (3)

Simplify by dividing through by m_i^*

and noting

$$\frac{c_i^*}{m_i^*} = 2\xi_i\omega_i \qquad \frac{k_i^*}{m_i^*} = \omega_i^2$$

$$\boxed{\ddot{y}_i + 2\xi_i\omega_i\dot{y}_i + \omega_i^2 y_i = -\frac{\phi_i^{\mathrm{T}} MR}{\phi_i^{\mathrm{T}} M\phi_i}\ddot{u}_g = -\Gamma_i\ddot{u}_g}$$

Review of MDOF Dynamics (4)

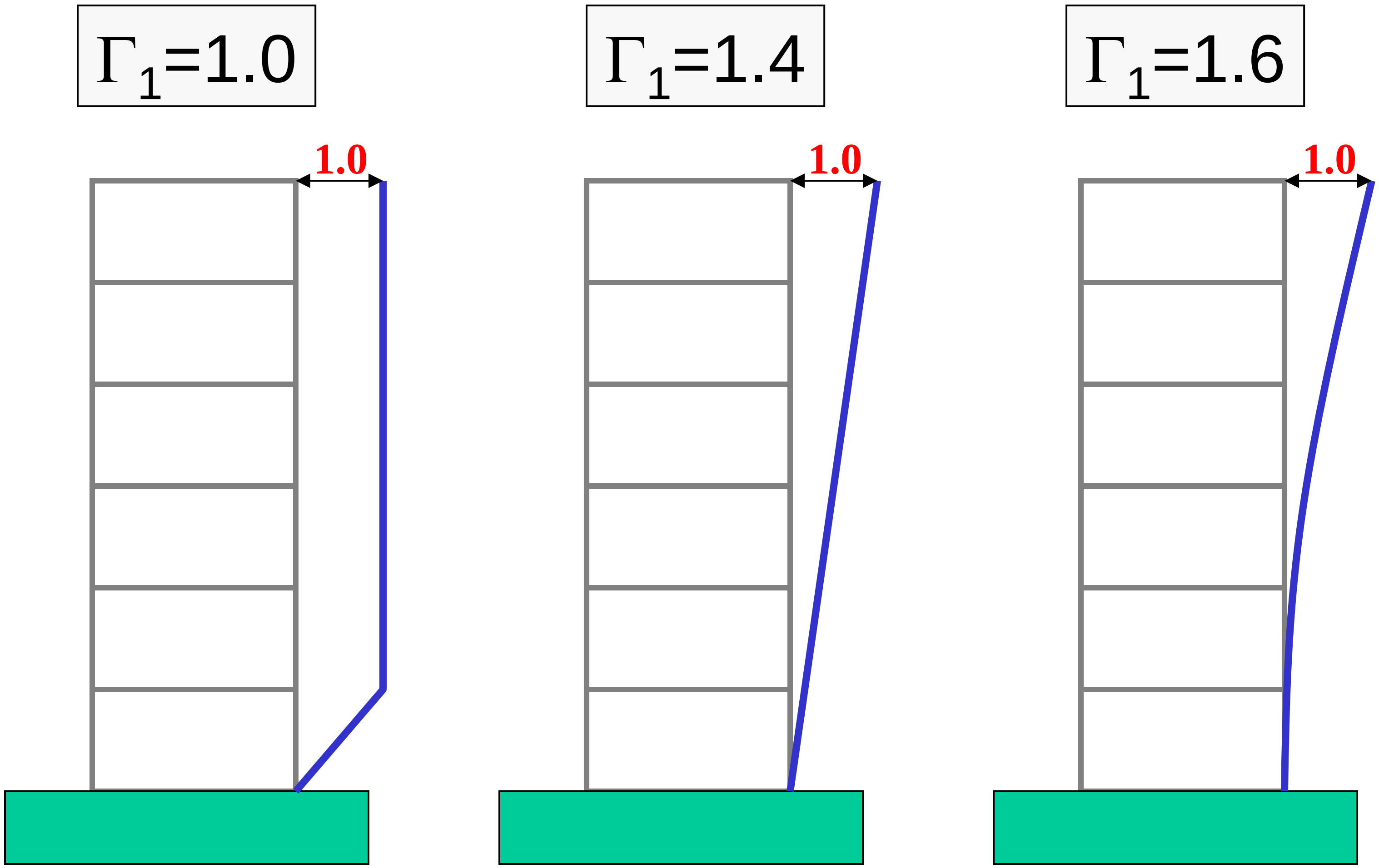
$$\ddot{y}_i + 2\xi_i\omega_i\dot{y}_i + \omega^2 y_i = -\frac{\phi_i^{\mathrm{T}} MR}{\phi_i^{\mathrm{T}} M\phi_i}\ddot{u}_g = -\Gamma_i\ddot{u}_g$$

Modal Participation Factor:

$$\boxed{\Gamma_i = \frac{\phi_i^{\mathrm{T}} MR}{\phi_i^{\mathrm{T}} M\phi_i}}$$

Important Note: Γ_i depends on mode shape scaling

Variation of First Mode Participation Factor with First Mode Shape



Review of MDOF Dynamics (5)

Any Mode of MDOF system

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

SDOF system

$$\ddot{D}_i + 2\xi_i\omega_i\dot{D}_i + \omega_i^{\,2} D_i = -\ddot{u}_g$$

If we obtain the displacement $D_i(t)$ from the response of a SDOF we must multiply by Γ_1 to obtain the modal amplitude response $y_i(t)$. history

$$\boxed{y_1(t) = \Gamma_1 D_i(t)}$$

Review of MDOF Dynamics (6)

If we run a SDOF Response history analysis:

$$\boxed{y_i(t) = \Gamma_i D_i(t)}$$

If we use a response spectrum:

$$\boxed{y_{i,\max} = \Gamma_i D_{i,\max}}$$

Review of MDOF Dynamics (7)

In general

$$\boxed{y_i(t) = \Gamma_i D_i(t)}$$

Recalling

$$\boxed{u_i(t) = \phi_i y_i(t)}$$

Substituting

$$\boxed{u_i(t) = \Gamma_i \phi_i D_i(t)}$$

Review of MDOF Dynamics (8)

Applied “static” forces required to produce $u_i(t)$:

$$F_i(t) = Ku_i(t) = \Gamma_i K\phi_i D_i(t)$$

Recall $K\phi_i = \omega_i^2 M\phi_i$

$$F_i(t) = \Gamma_i M\phi_i\omega_i^2 D_i(t) = \Gamma_i M\phi_i a_i(t)$$

$$\boxed{F_i(t) = S_i a_i(t)}$$

where $S_i = \Gamma_i M\phi_i$

Review of MDOF Dynamics (9)

Total shear in mode:

$$V_i = F_i^T R$$

$$V_i(t) = \Gamma_i (M\phi_i)^T R a_i(t) = \Gamma_i \phi_i^T M R a_i(t)$$

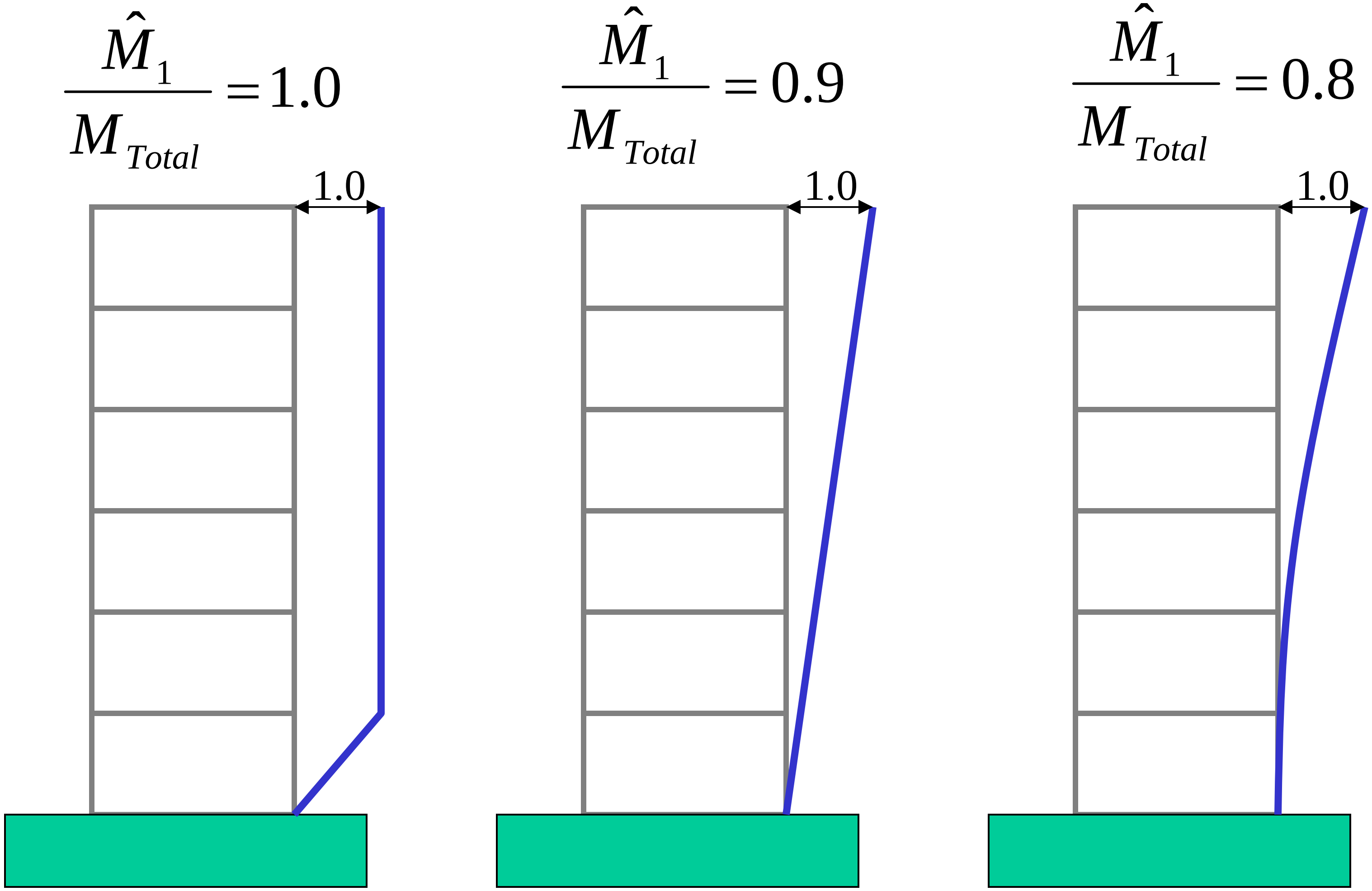
$$\boxed{V_i(t) = \hat{M}_i a_i(t)}$$

Effective Modal Mass:

$$\boxed{\hat{M}_i = \frac{[\phi_i^{\mathrm{T}} M R]^2}{\phi_i^{\mathrm{T}} M \phi_i}}$$

Important Note: $\hat{M}_i$ ***does NOT*** depend on mode shape scaling

Variation of First Mode Effective Mass with First Mode Shape



Review of MDOF Dynamics (10)

$$S_1 + S_2 + \ldots + S_n = MR$$

$$\sum_{k=1}^{n} S_{i,k} = \hat{M}_i$$

Simple Numerical Example

$$K = \begin{bmatrix} 50 & -50 & 0 \\ -50 & 110 & -60 \\ 0 & -60 & 130 \end{bmatrix} \qquad M = \begin{bmatrix} 1.0 & 0 & 0 \\ 0 & 1.1 & 0 \\ 0 & 0 & 1.2 \end{bmatrix}$$

$$S_1 = \begin{Bmatrix} 1.267 \\ 1.060 \\ 0.600 \end{Bmatrix} \qquad S_2 = \begin{Bmatrix} -0.338 \\ 0.223 \\ 0.428 \end{Bmatrix} \qquad S_3 = \begin{Bmatrix} 0.071 \\ -0.183 \\ 0.172 \end{Bmatrix}$$

$$\sum_{k=1}^{3} S_{1,k} = 2.927 \qquad \sum_{k=1}^{3} S_{2,k} = 0.313 \qquad \sum_{k=1}^{3} S_{3,k} = 0.060$$

$$S_1 + S_2 + S_3 = \begin{Bmatrix} 1.0 \\ 1.1 \\ 1.2 \end{Bmatrix}$$

Review of MDOF Dynamics (11)

Displacement Response in single mode:

$$u_i(t) = \Gamma_i \phi_i D_i(t)$$

From Response-History
or Response Spectrum
Analysis

Total shear in single mode:

$$V_i(t) = \hat{M}_i a_i(t)$$

First Mode Response as Function of System Response

Modal Displacement:

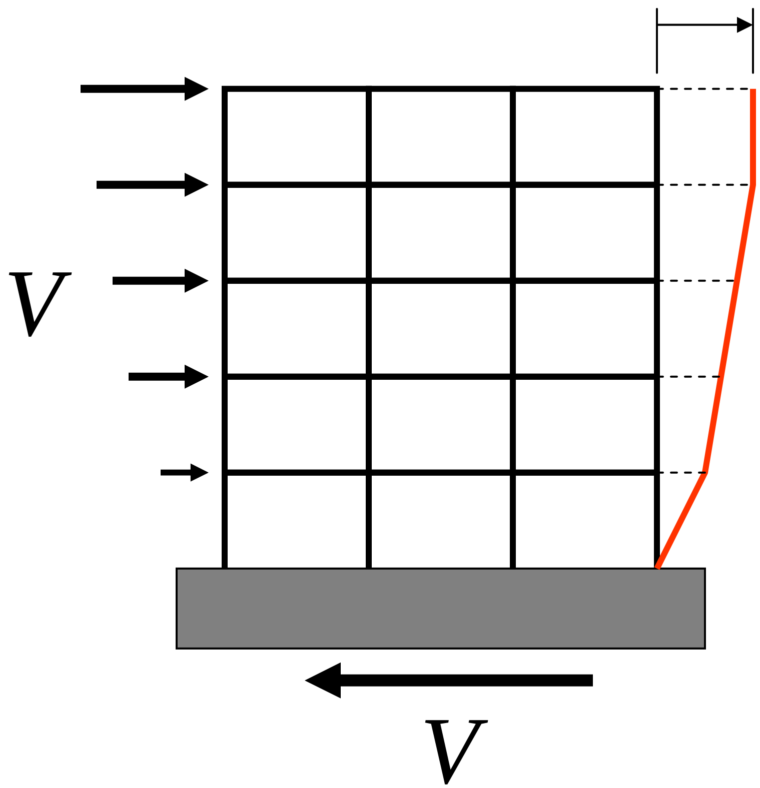
$$D_1(t) = \frac{u_{1,roof}(t)}{\Gamma_1 \phi_{1,roof}}$$

Modal Acceleration:

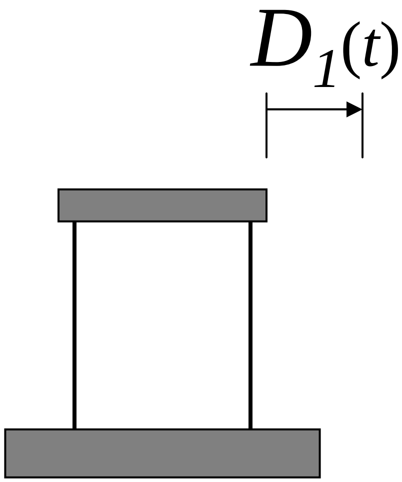
$$a_1(t) = \frac{V_1(t)}{\hat{M}_1}$$

Converting Pushover Curve to Capacity Curve

$$u_{1,roof}(t) = \Gamma_1 \phi_{1,roof} D_1(t)$$

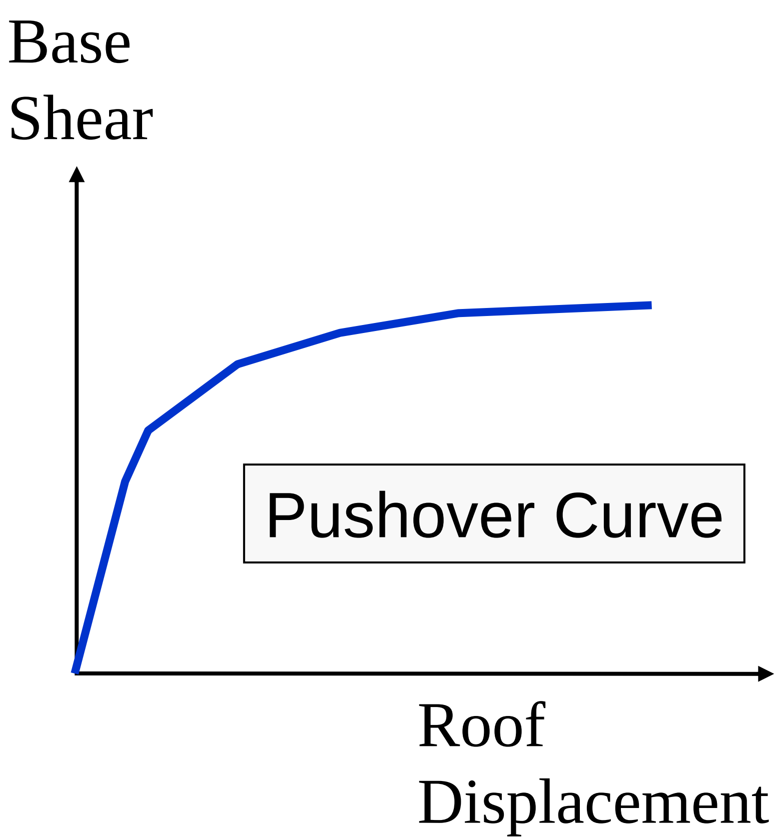


First Mode System (natural coords)



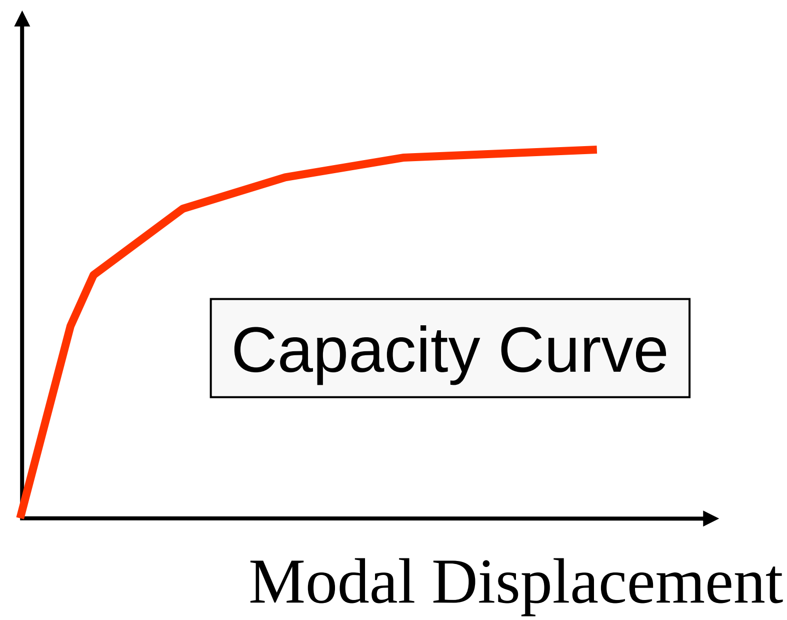
First Mode SDOF System
(modal coords)

Converting Pushover Curve to Capacity Curve



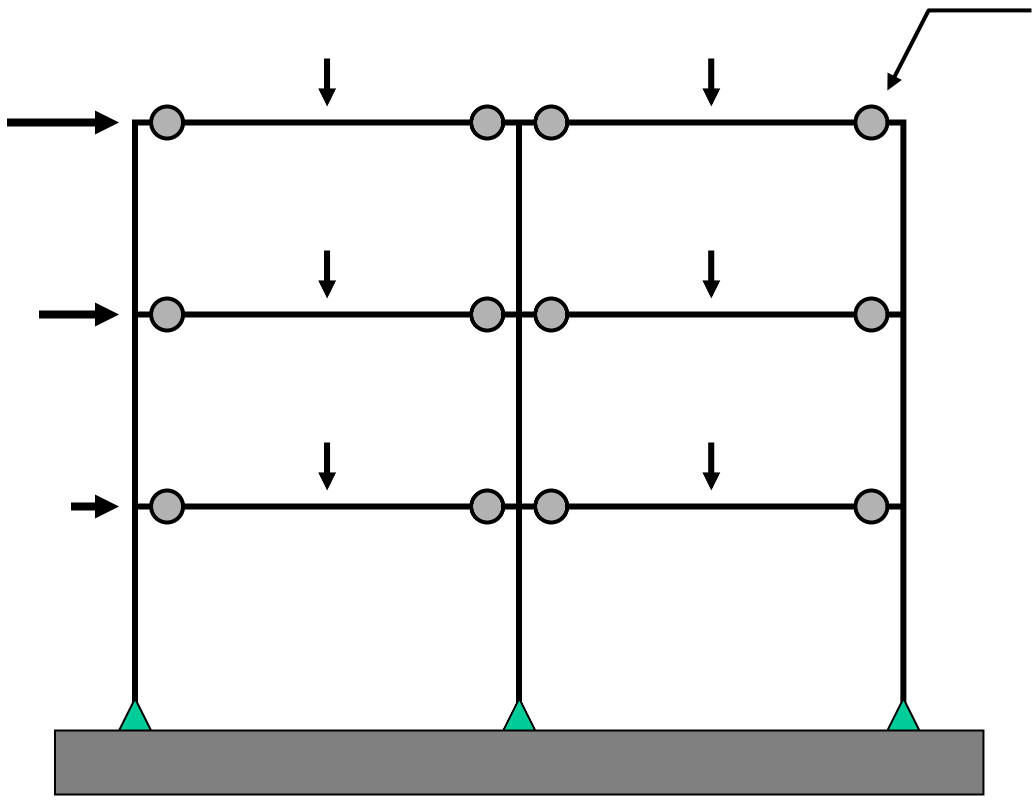
Modal
Acceleration

$$a_1(t) = \frac{V_1(t)}{\hat{M}_1}$$



$$D_1(t) = \frac{u_1(t)}{\Gamma_1 \phi_{1,roof}}$$

Development of Pushover Curve



Potential Plastic
Hinge Location
(Must be predicted
and possibly corrected)

Event-to-Event Pushover Analysis

Create Mathematical Model

↓

Apply gravity load to determine
initial nodal displacements and member forces

↓

Apply lateral load sufficient to produce
single yield event

↓

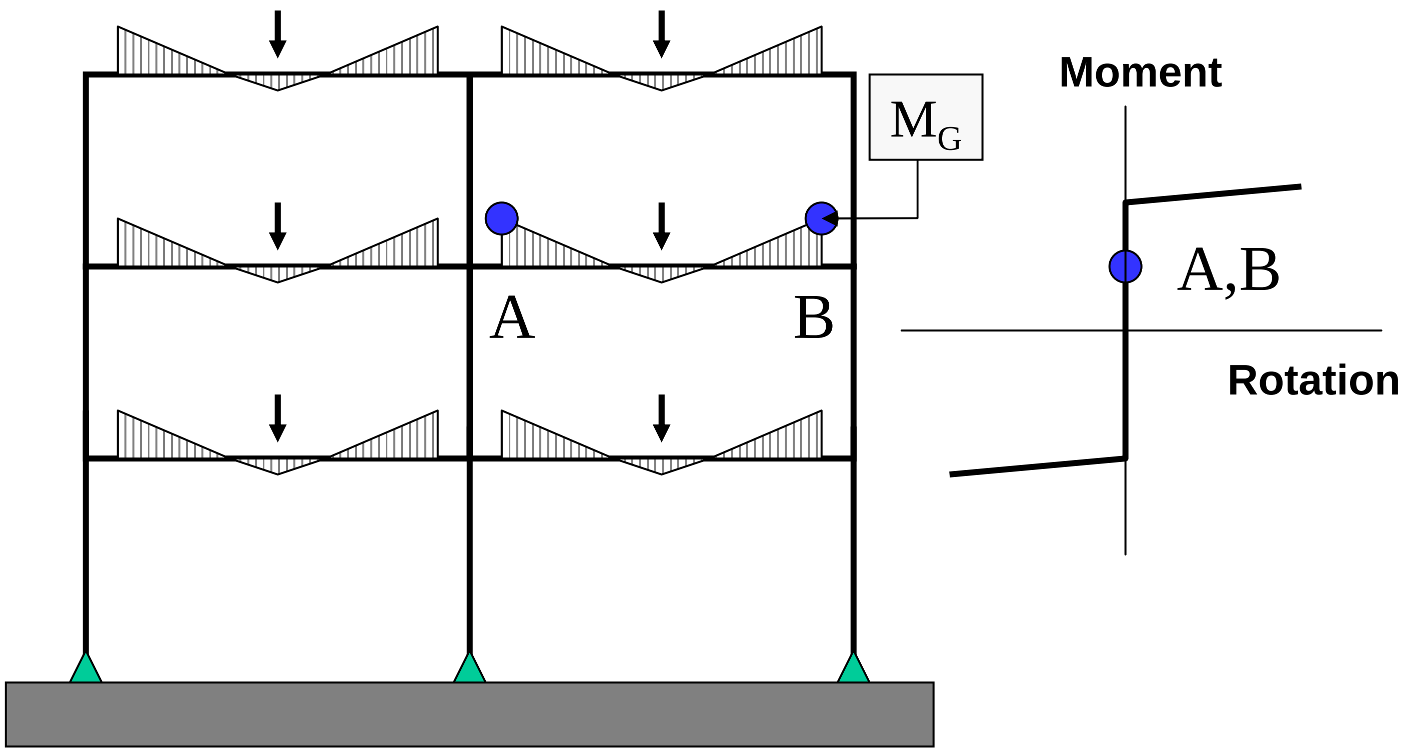
Update nodal displacements and member forces

↓

Modify structural stiffness to represent yielding

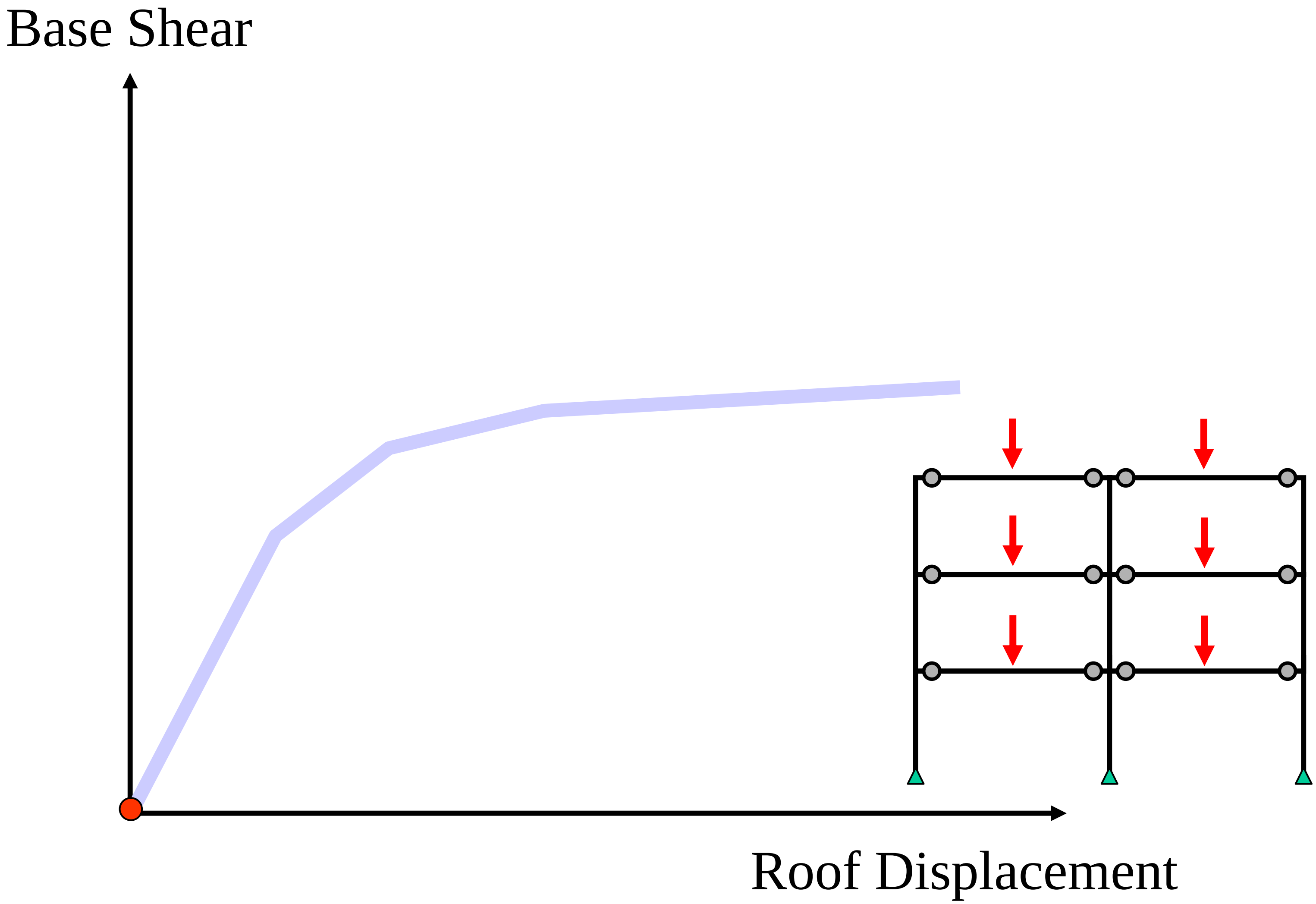
Continue
Until
Sufficient
Load or
Displacement
is obtained.

Initial Gravity Load Analysis

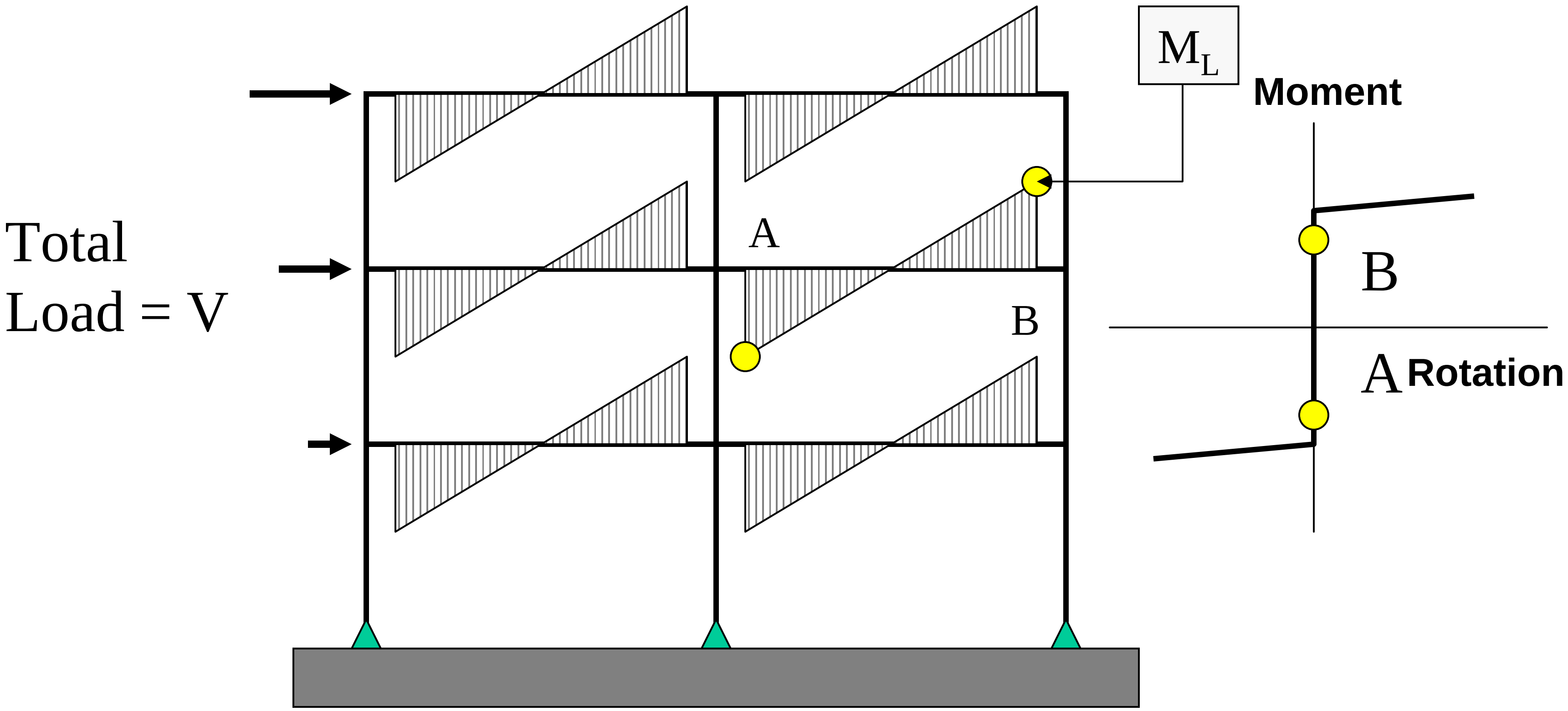


Moments plotted on tension side.

Analysis 1: Gravity Analysis



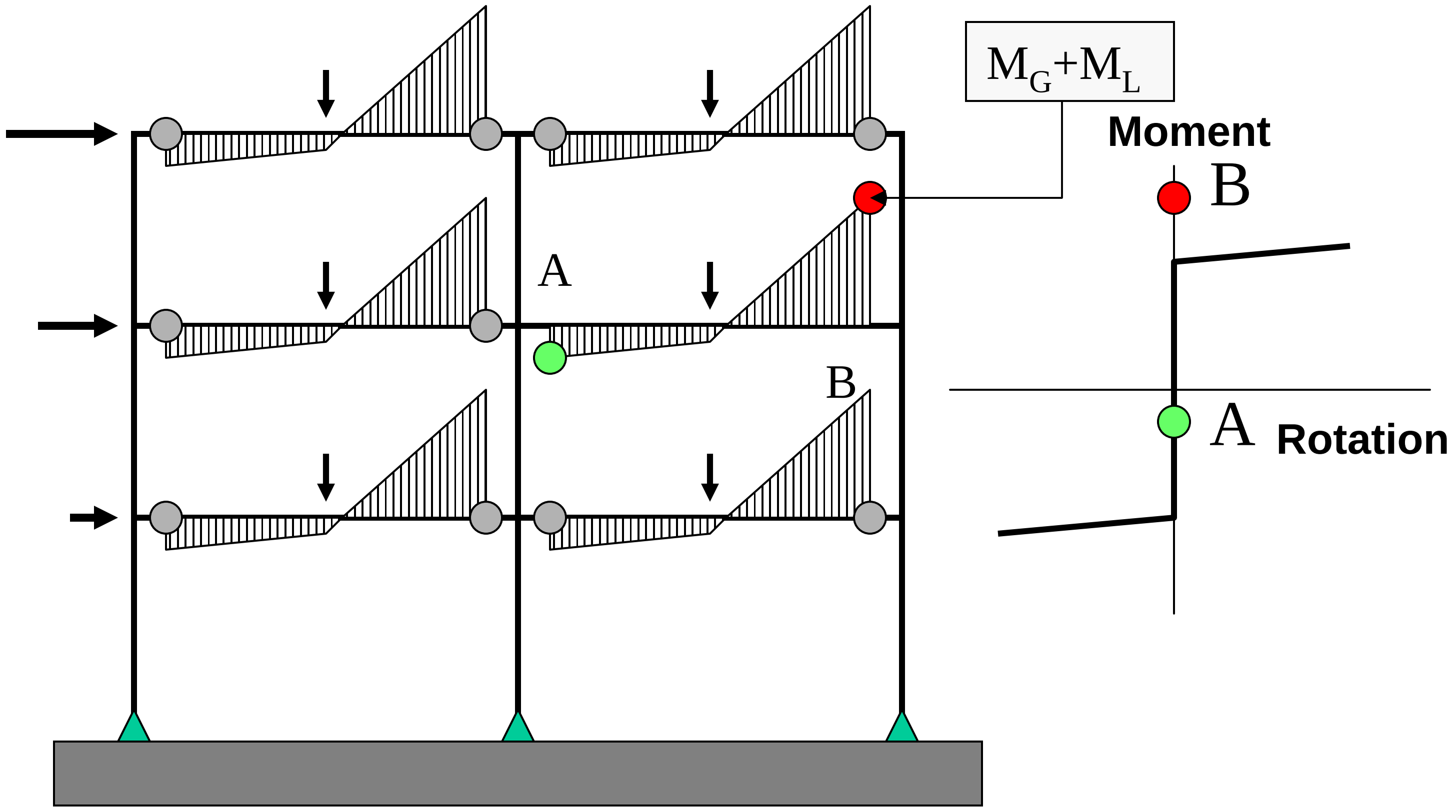
Lateral Load Analysis (Acting Alone)



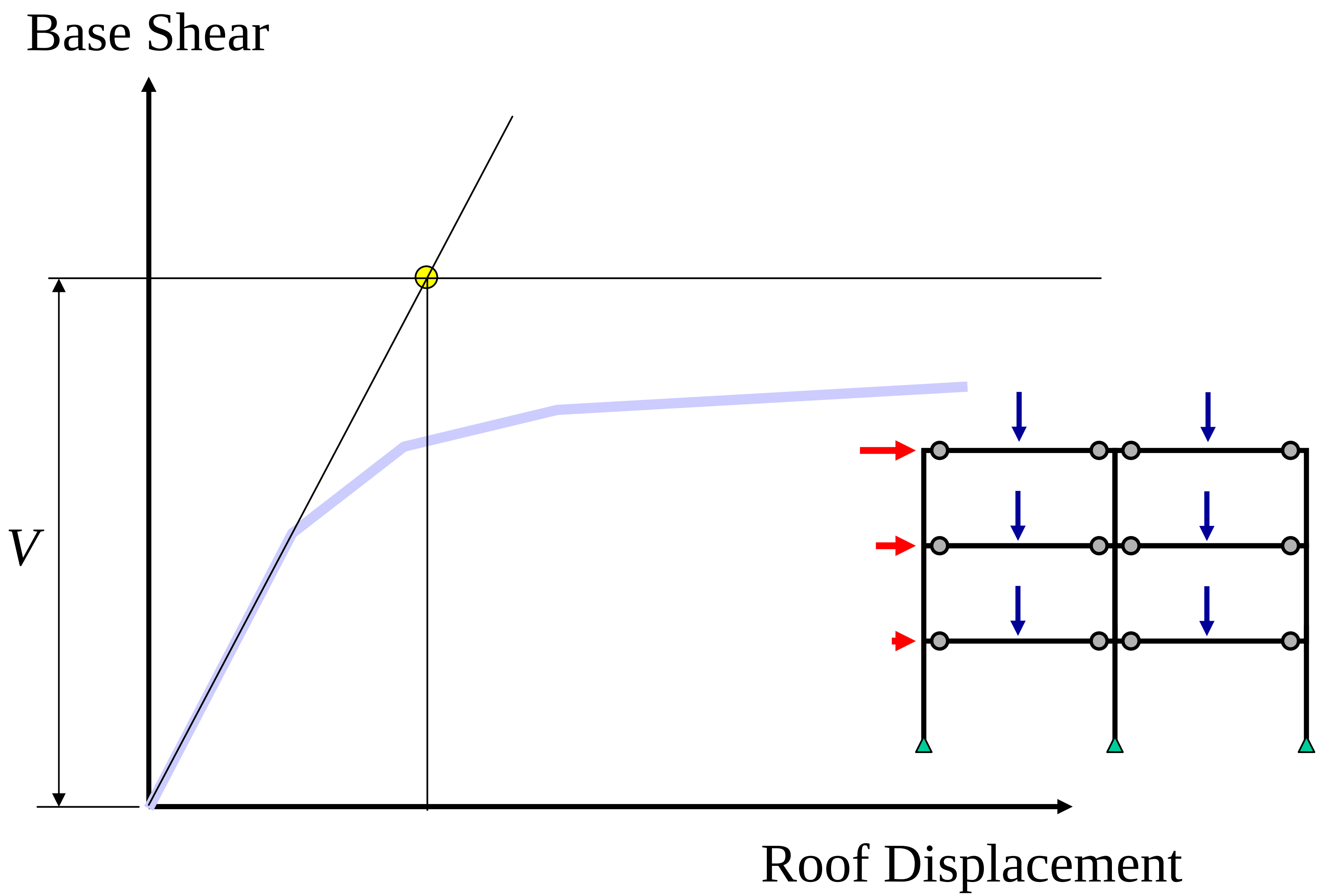
Moments plotted on tension side.

Combined Load Analysis Including Total Load V

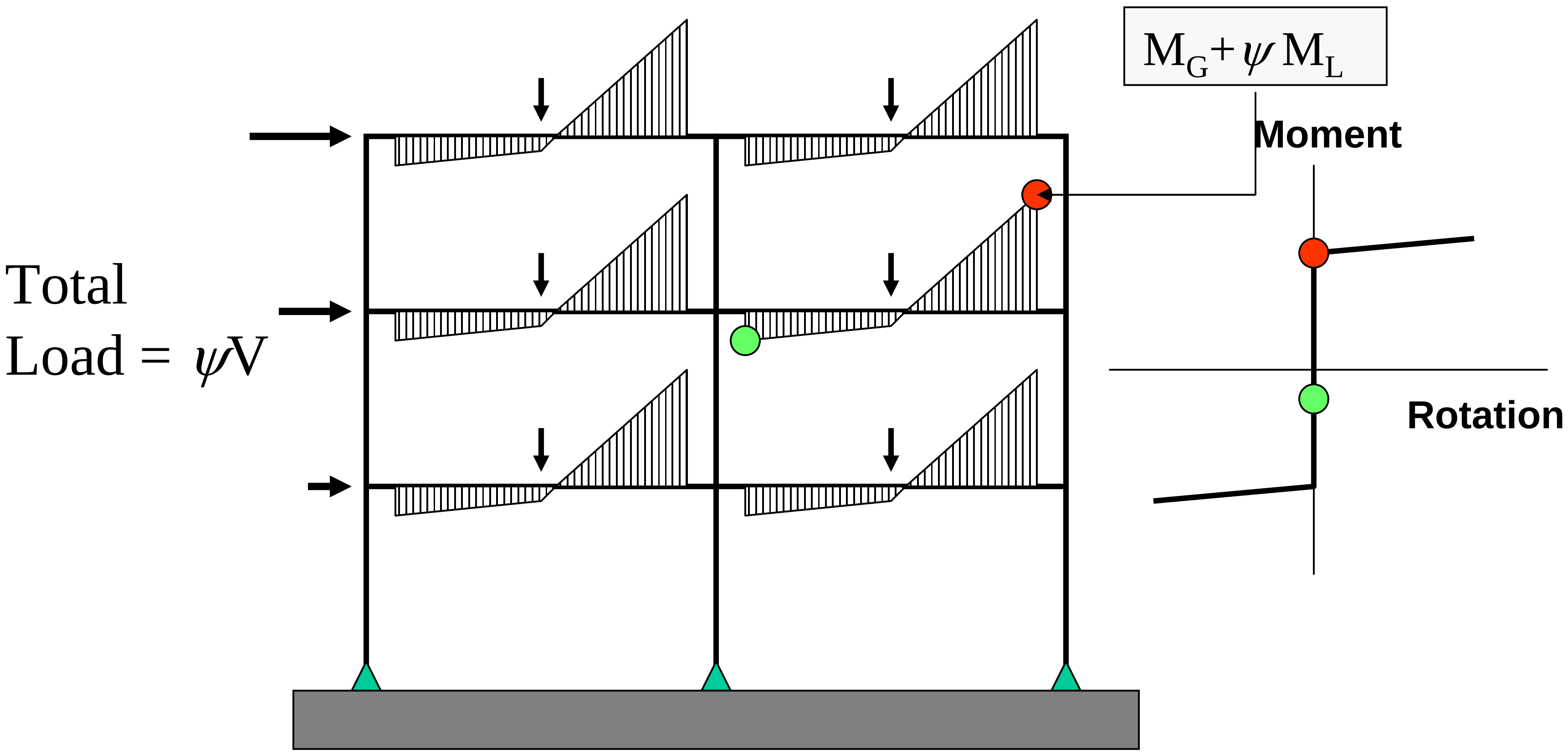
Total Load = V



Analysis 2a First Lateral Analysis



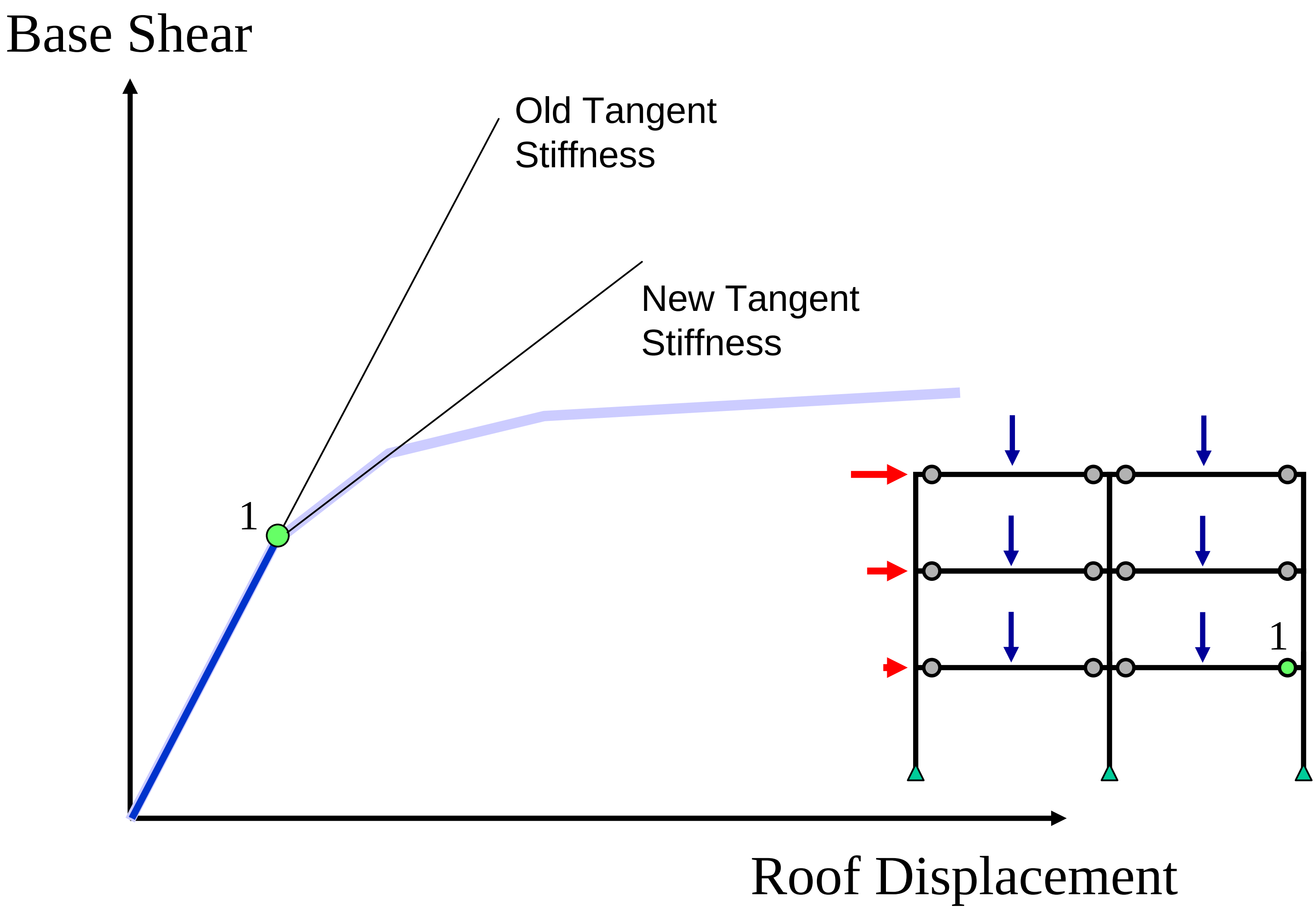
Combined Load Analysis:
Determine amount of Lateral Load Required to Produce First Yield



For all potential hinges (i) find ψ_i such that

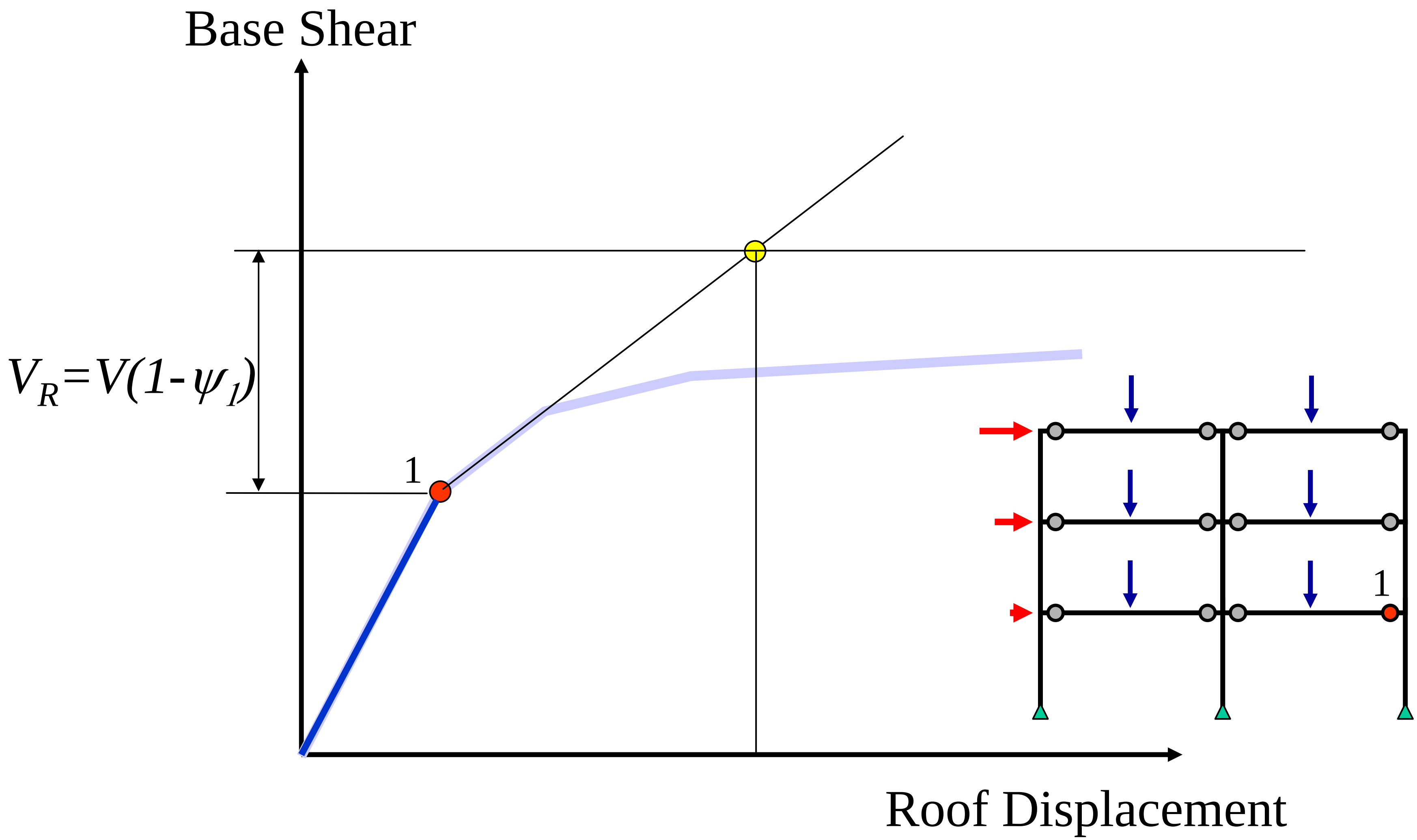
$$M_{G,i} + \psi_i M_{L,i} = M_{P,i}$$

Analysis 2b
Adjust Load to First Yield



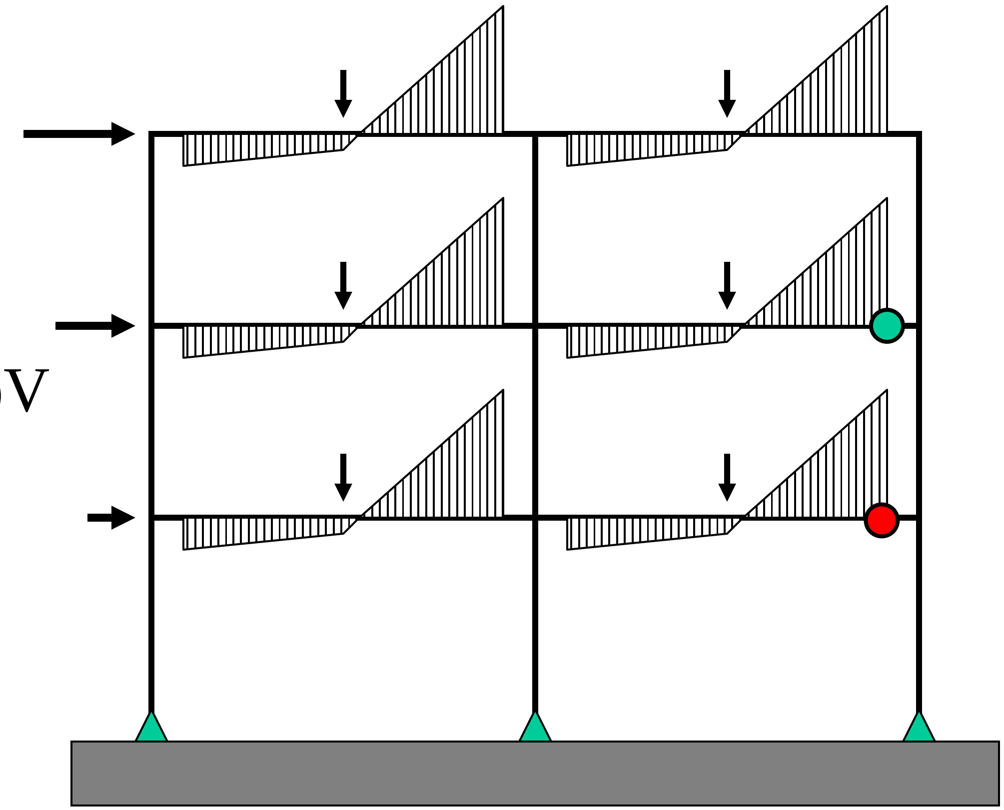
Analysis 3a

Modify System Stiffness Apply Remainder of Load

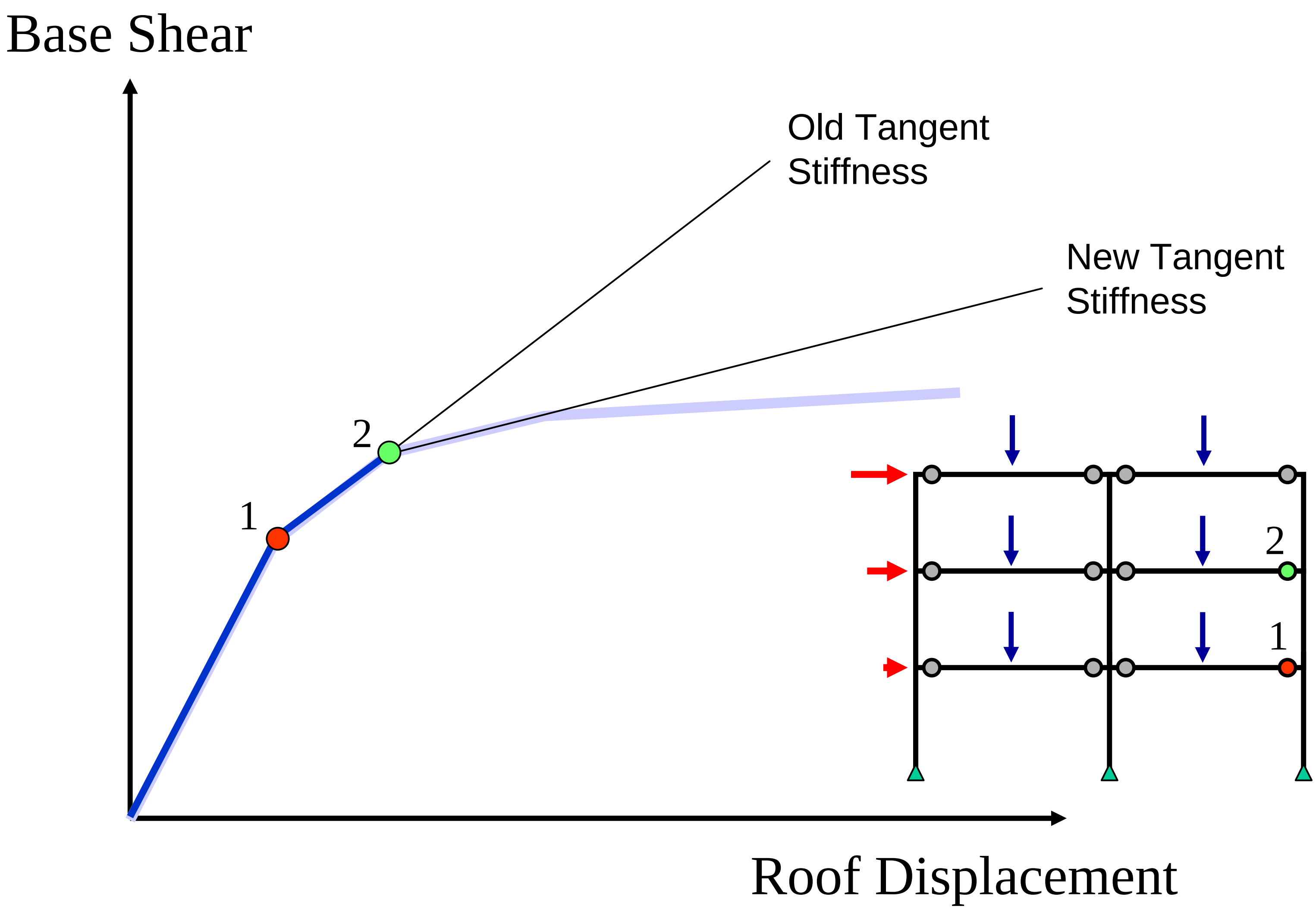


Determine amount of Lateral Load Required to Produce Second Yield

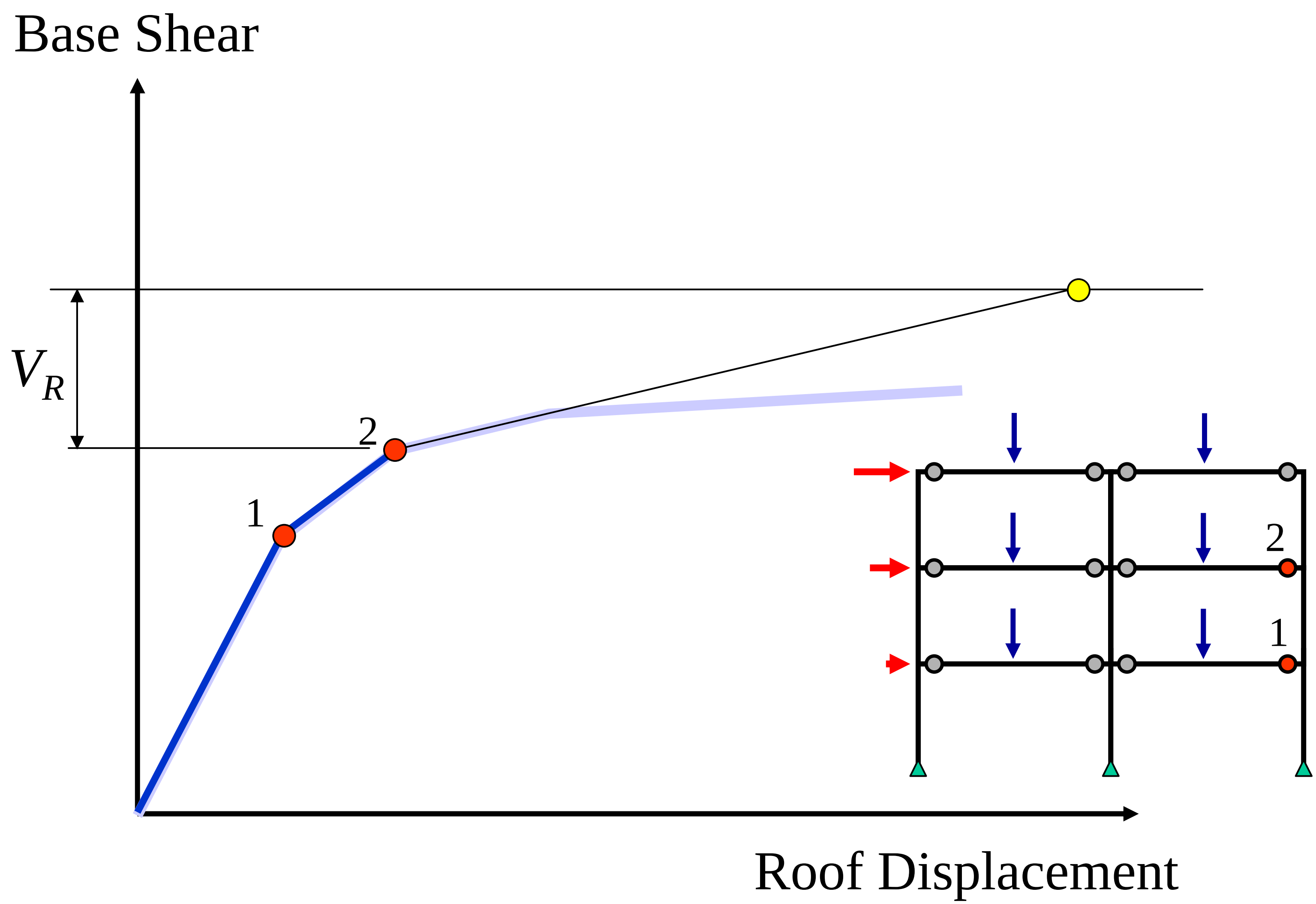
Total
Load = $(1-\psi_1)V$



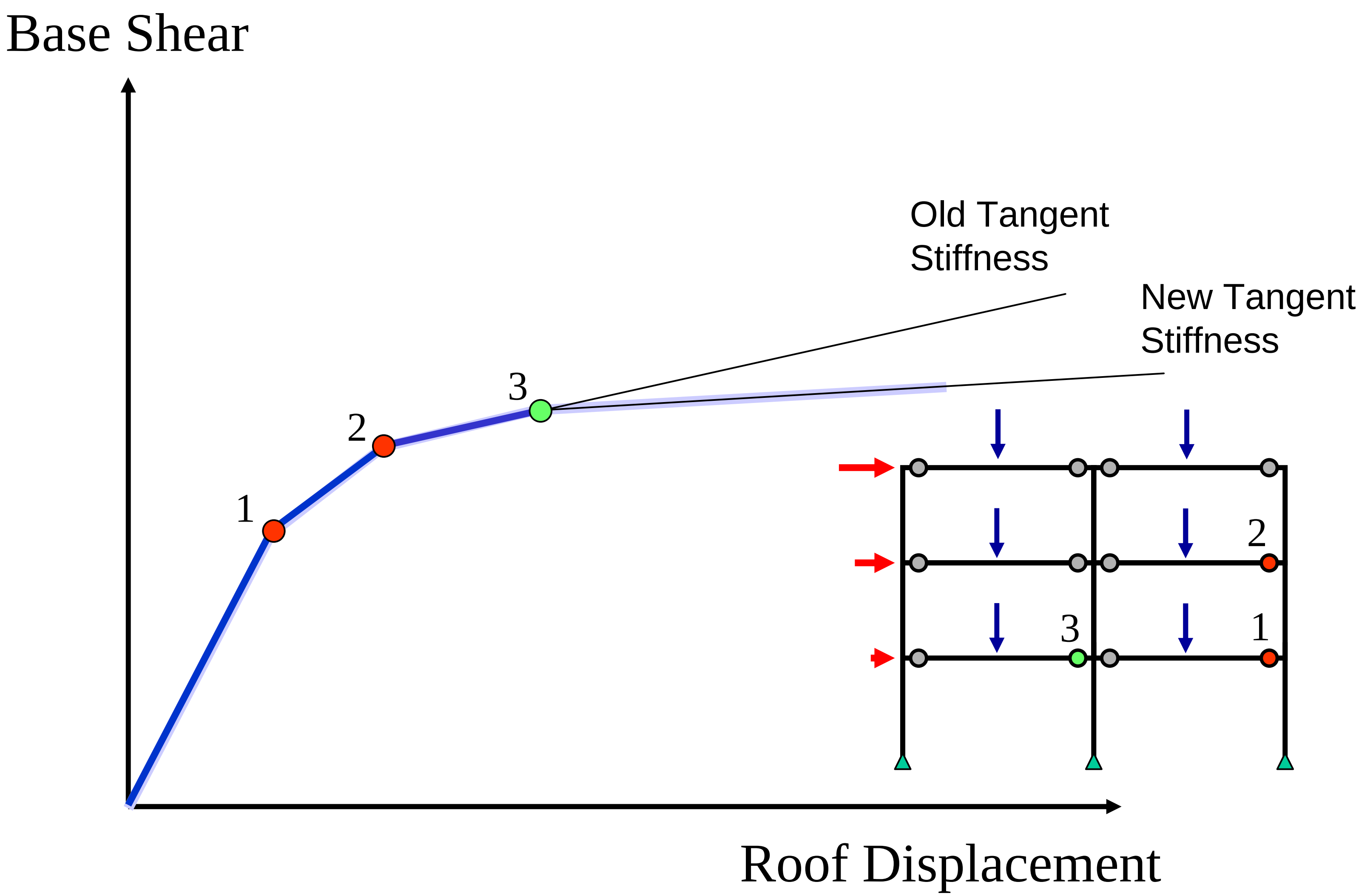
Analysis 3b
Adjust Load to Second Yield



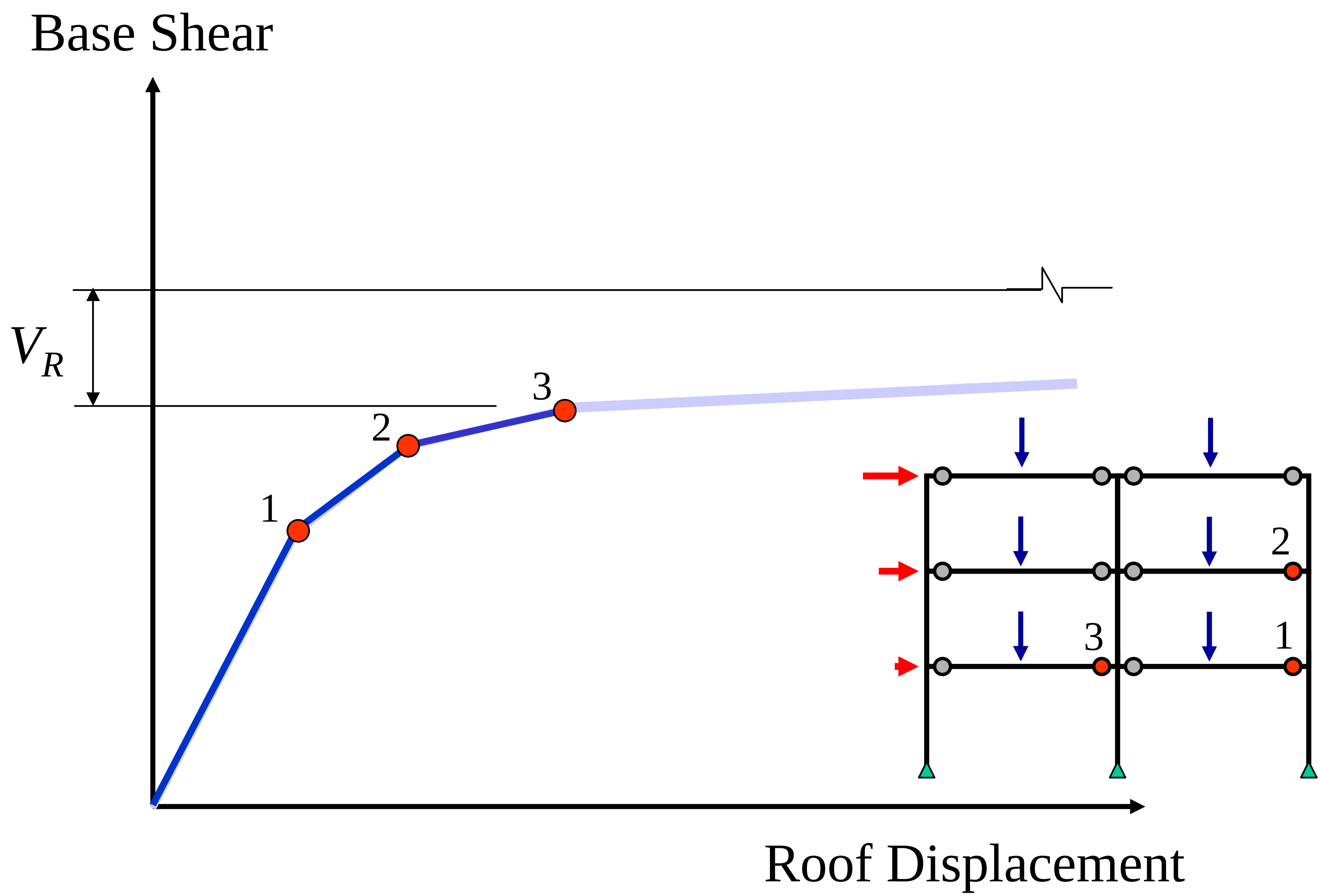
Analysis 4a
Modify System Stiffness Apply Remainder of Load



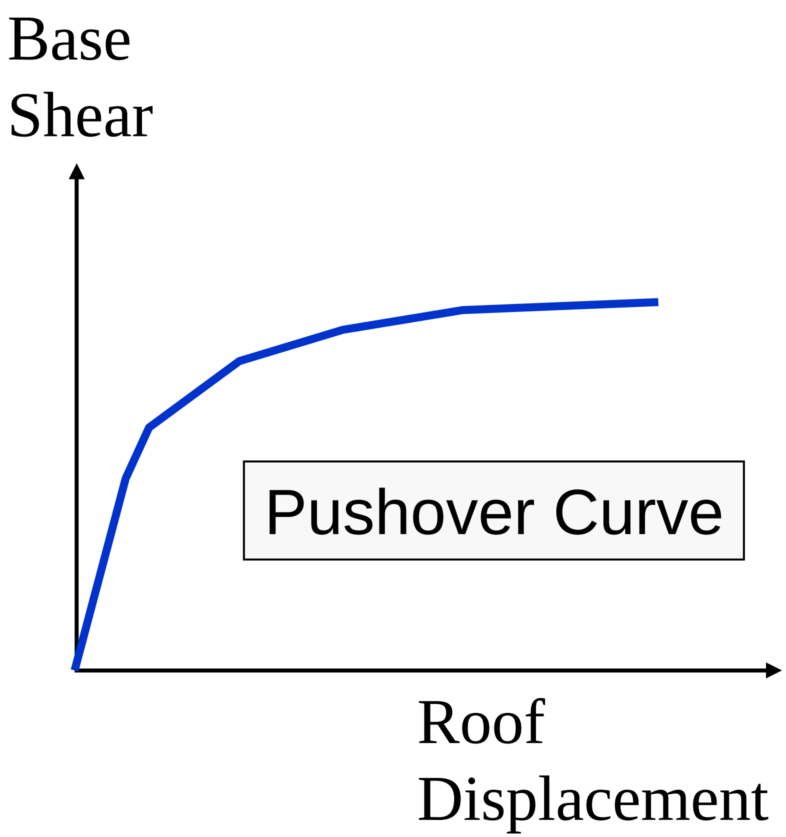
Analysis 4b
Adjust Load to Third Yield



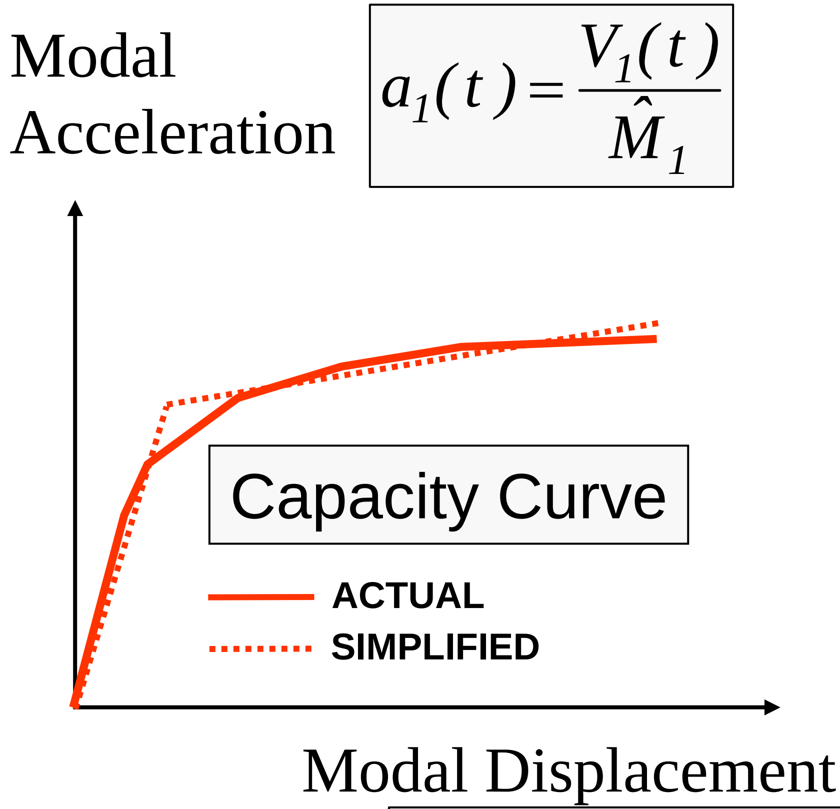
Analysis 5a…..



Convert Pushover Curve to Capacity Curve

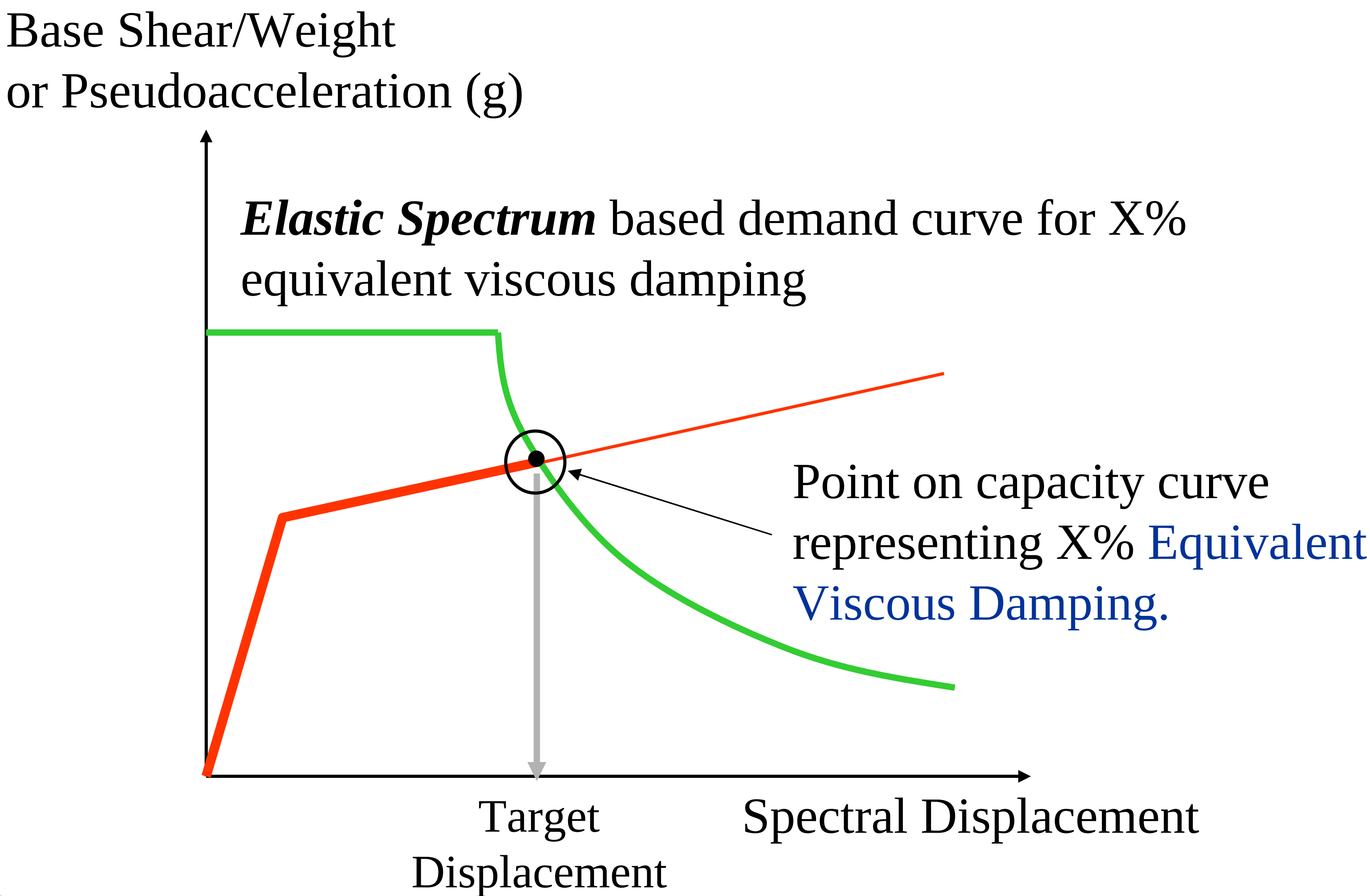


$$a_1(t) = \frac{V_1(t)}{\hat{M}_1}$$

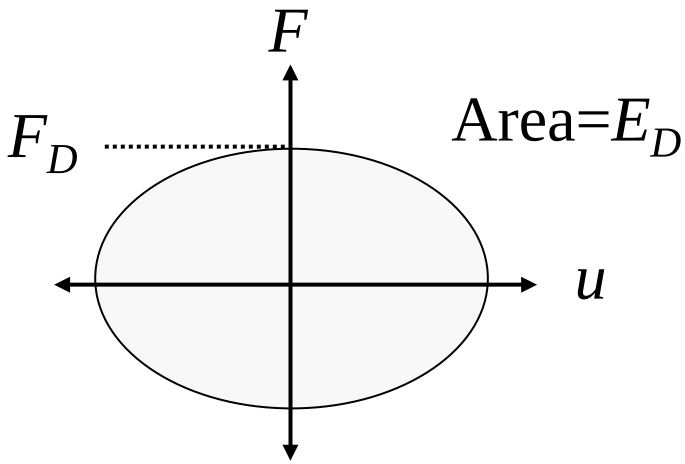


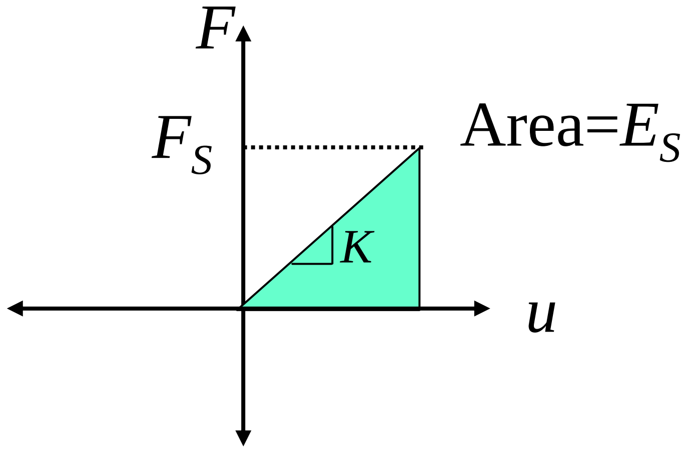
$$D_1(t) = \frac{u_1(t)}{\Gamma_1 \phi_{1,roof}}$$

Equivalent Viscous Damping



Computing Damping Ratio from Damping Energy and Strain Energy



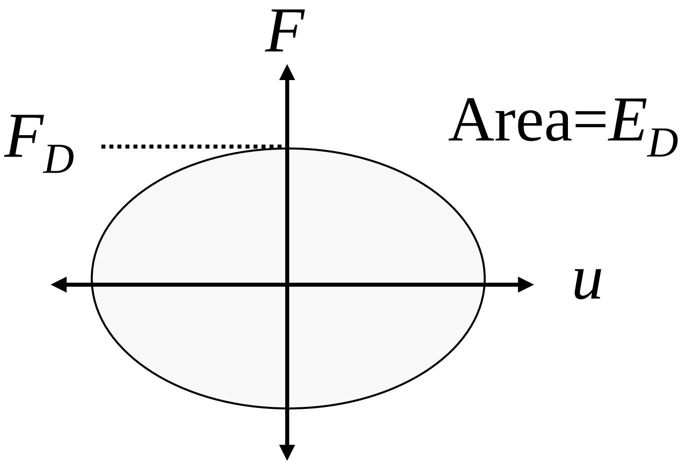


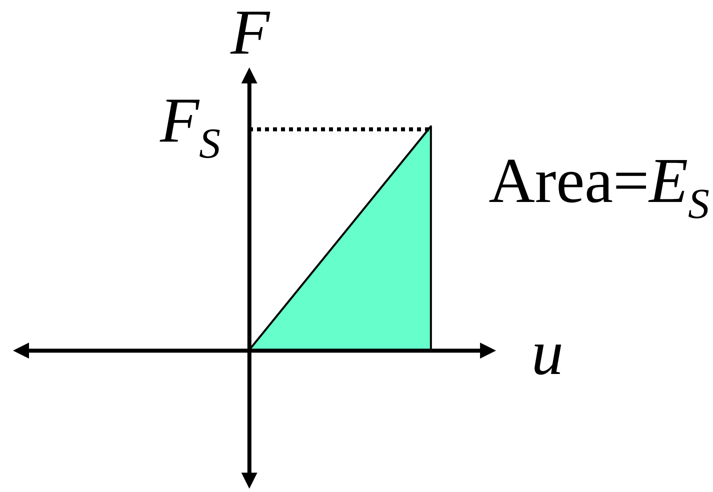
$$E_D = \pi F_D u = \pi C u^2 \omega = 2\pi\xi m \omega^2 u^2$$

$$E_S = 0.5 F_s u = 0.5 K u^2 = 0.5 m \omega^2 u^2$$

$$\boxed{\xi = \frac{E_D}{4\pi E_S}}$$

Computing Damping Ratio from Damping Force and Elastic Force





$$E_D = \pi F_D u$$

$$E_S = \frac{1}{2} F_s u$$

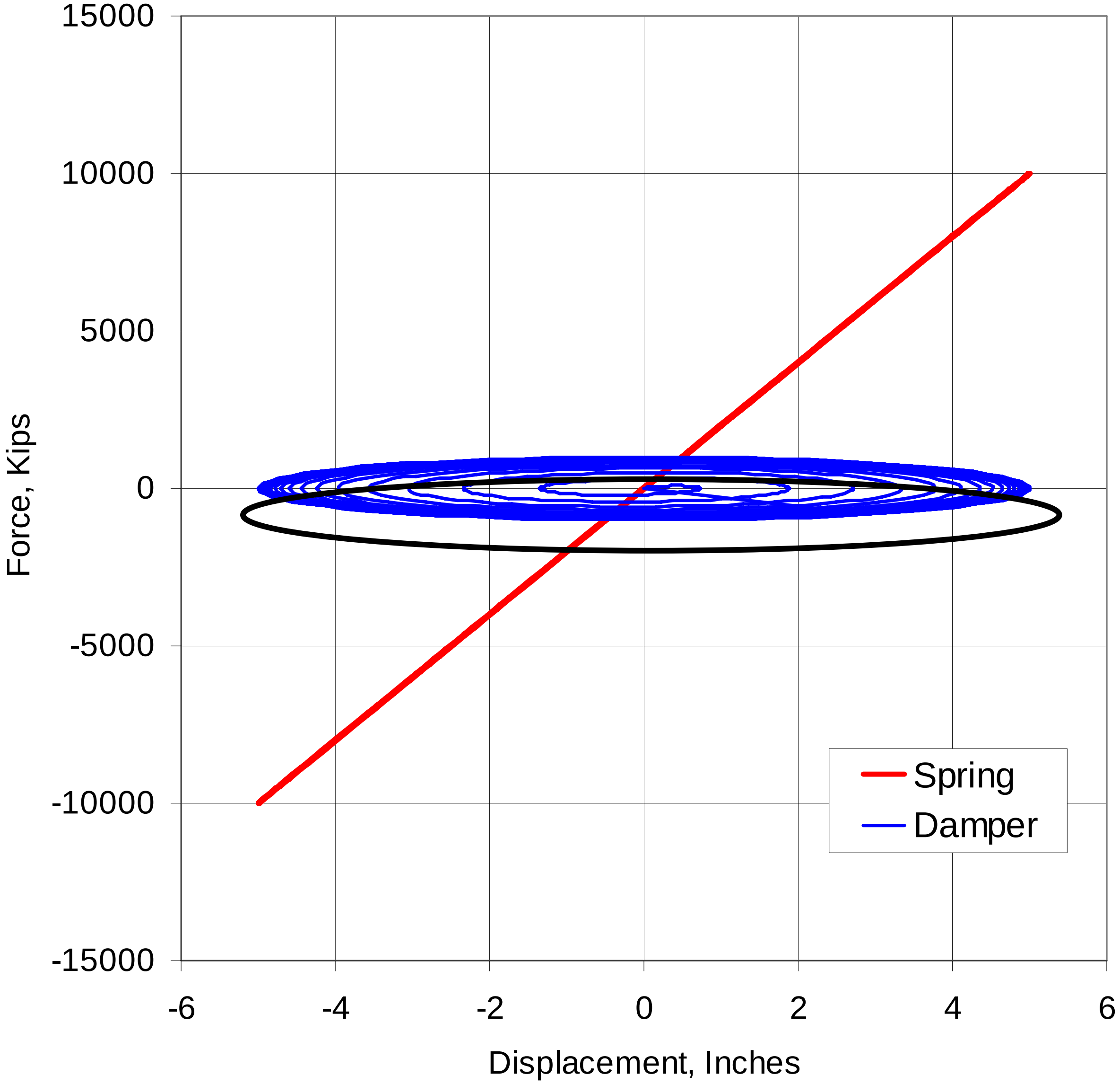
$$\boxed{\xi = \frac{E_D}{4\pi E_S} = \frac{F_D}{2F_S}}$$

Computing "True" Viscous Damping Ratio from Damping Energy and Strain Energy

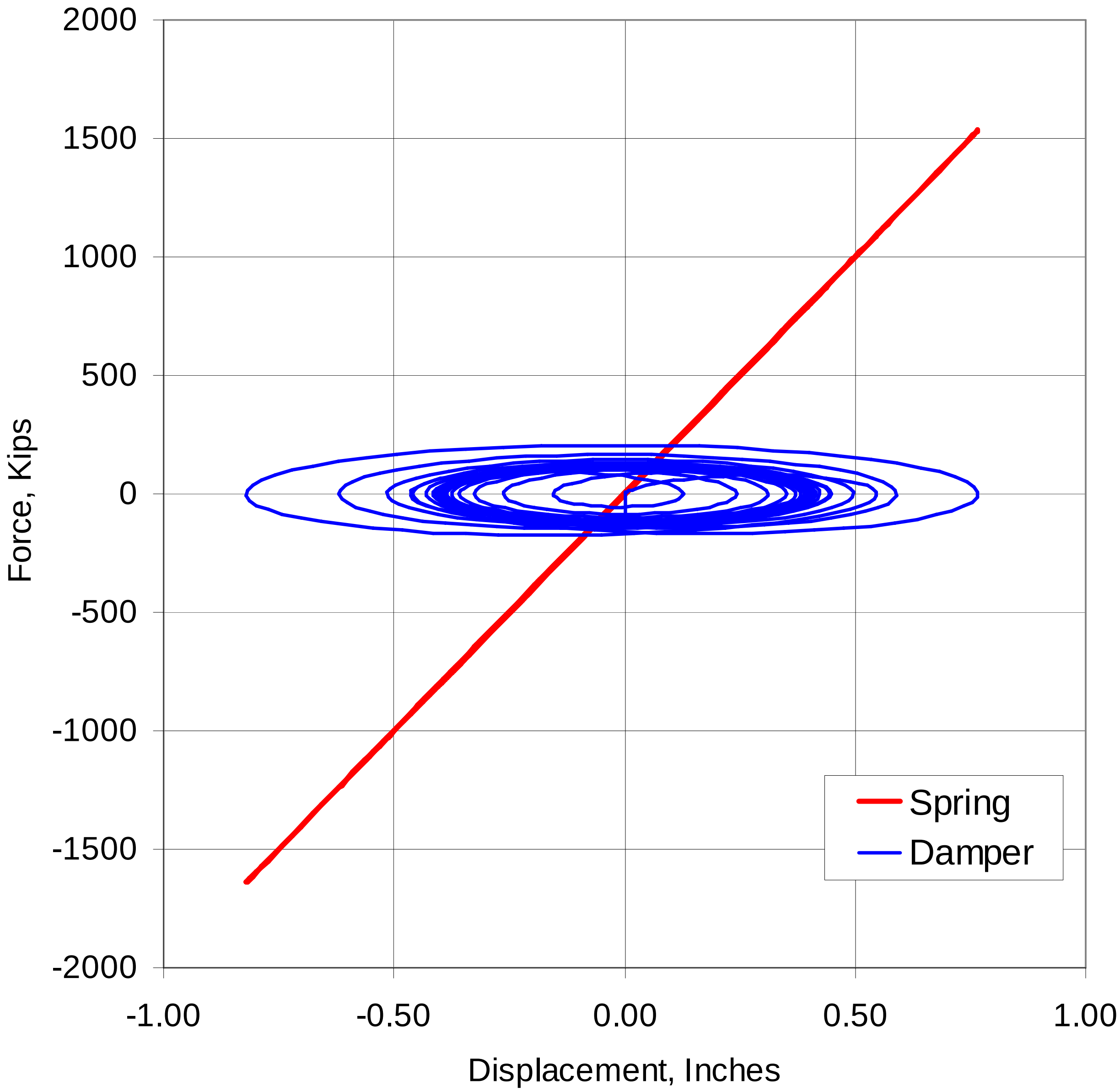
$$\xi = \frac{E_D}{4\pi E_S} = \frac{F_D}{2F_S}$$

Note:
System must be in steady state harmonic **RESONANT** response for this equation to work.

Harmonic Resonant Response from NONLIN



Harmonic Non-Resonant Response from NONLIN



Results from NONLIN Using: $\xi = \dfrac{E_D}{4\pi E_S} = \dfrac{F_D}{2F_S}$

System Period = 0.75 seconds
Harmonic Loading
Target Damping Ratio 5% Critical

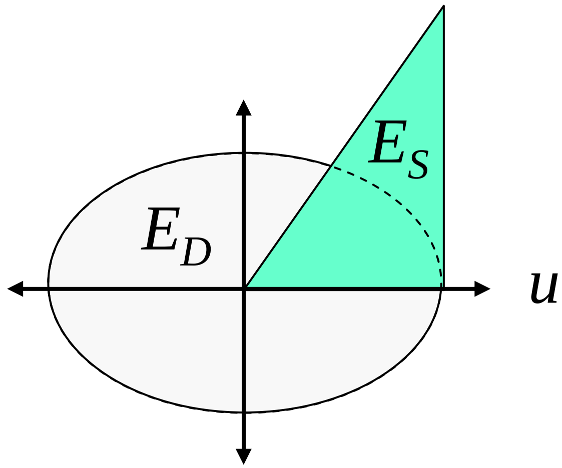
Loading Period (sec)	Damping Force (k)	Spring Force (k)	Damping Ratio %		
0.50	118	787	**7.50**	X	
0.75	984	9828	**5.00**	✓	**Resonant**
1.00	197	2251	**3.75**	X	

Results from NONLIN Using: $\xi = \dfrac{E_D}{4\pi E_S} = \dfrac{F_D}{2F_S}$

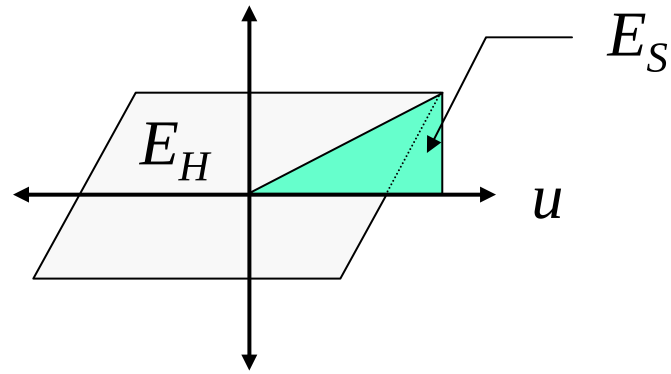
System Period = 0.75 seconds
Harmonic Loading
Target Damping Ratio 20% Critical

Loading Period (sec)	Damping Force (k)	Spring Force (k)	Damping Ratio %	
0.50	430	717	**30.0** ✗	
0.75	999	2498	**20.0** ✓	**Resonant**
1.00	1888	5666	**16.7** ✗	

Computing *Equivalent* Viscous Damping Ratio from Yield-Based Hysteretic Energy and Strain Energy



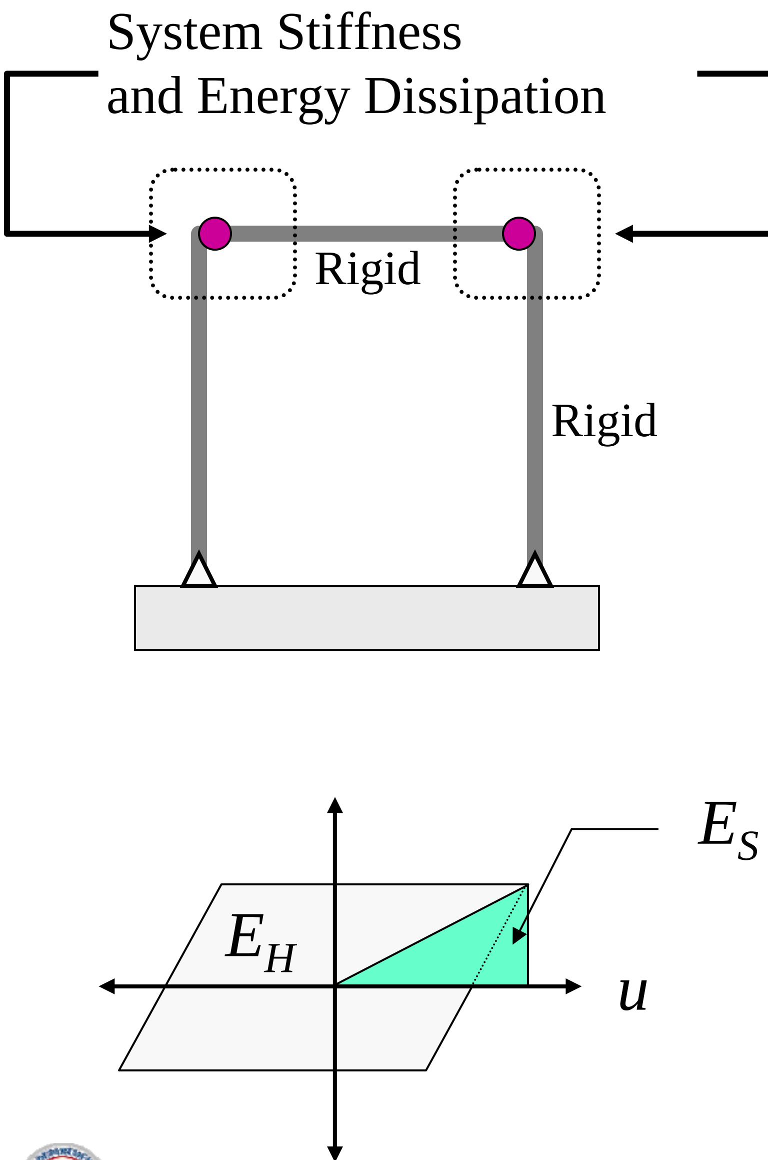
Viscous System



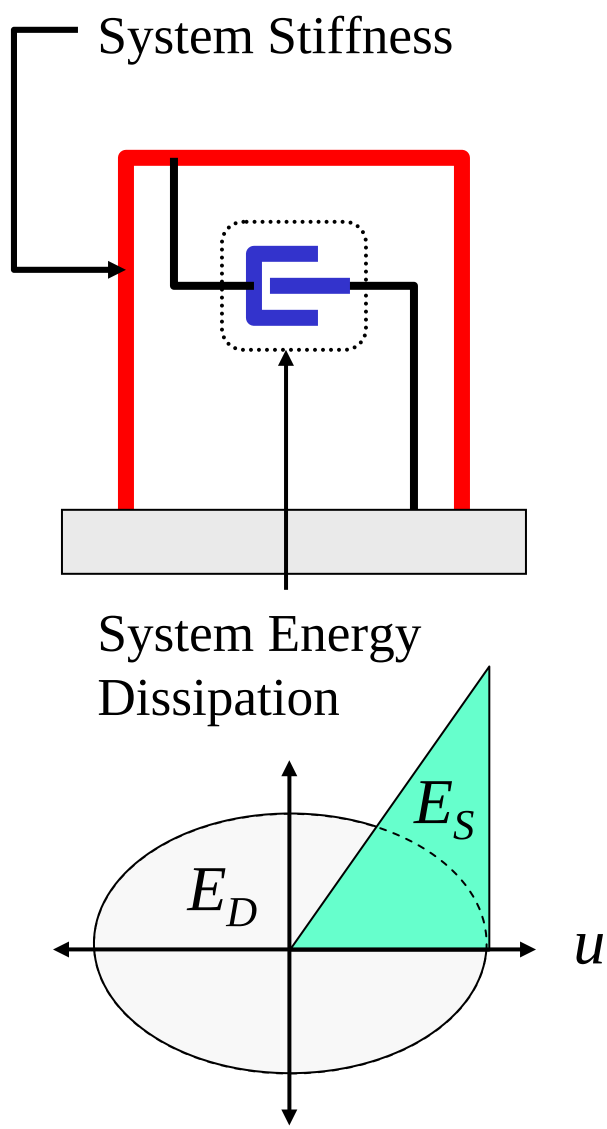
Yielding System

$$\xi \equiv \frac{E_H}{4\pi E_S}$$

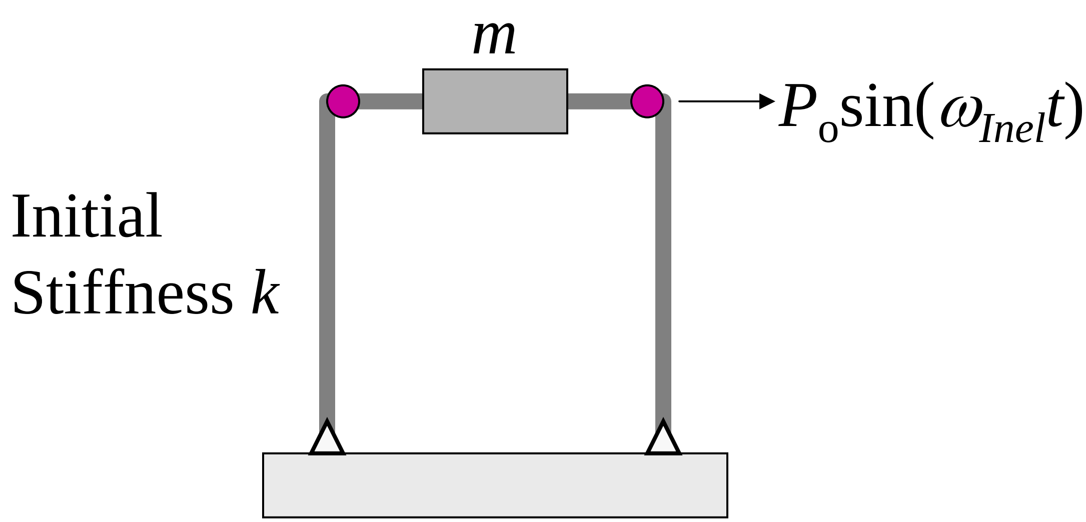
Actual Yielding System

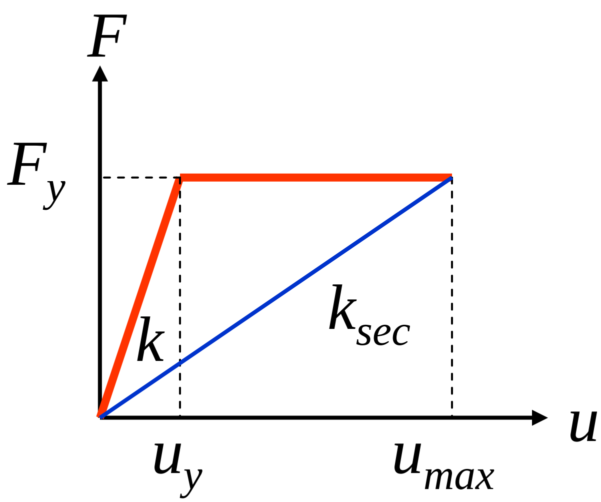


“Equivalent” Elastic System



Original Yielding System





Initial Frequency:

$$\omega = \sqrt{\frac{k}{m}}$$

Resonant Frequency:

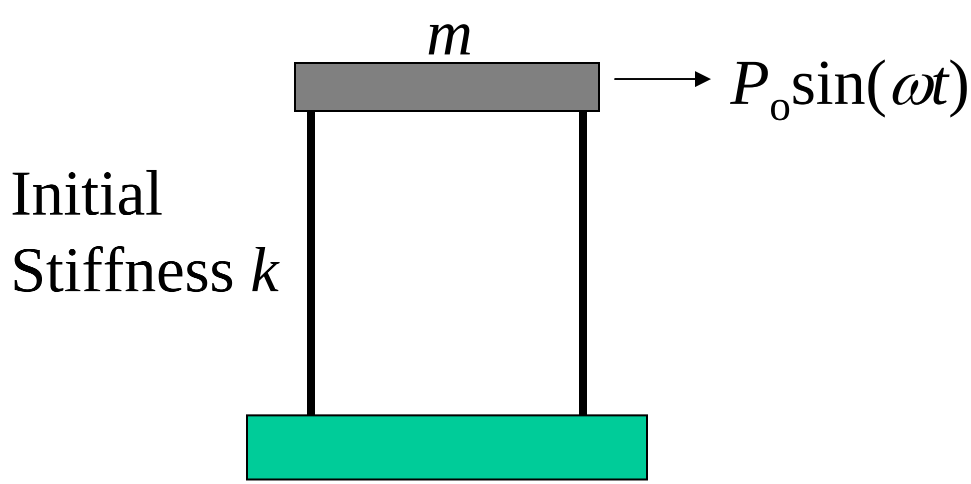
$$\omega_{Inelastic} = \frac{\omega^2}{\pi}(\theta - 0.5\sin 2\theta)$$

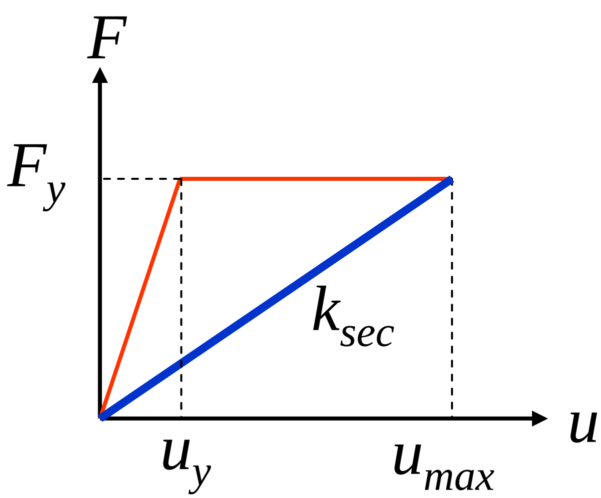
$$\theta = \cos^{-1}\left(1 - 2\frac{u_y}{u_{\max}}\right)$$

Maximum Steady State Response (loaded at $\omega_{Inelastic}$):

$$u_{\max} = \frac{4u_y}{4 - \dfrac{P_o \pi}{k u_y}}$$

“Equivalent” Elastic System





Resonant Frequency:

$$\omega_{\text{sec}} = \sqrt{\frac{k_{\text{sec}}}{m}}$$

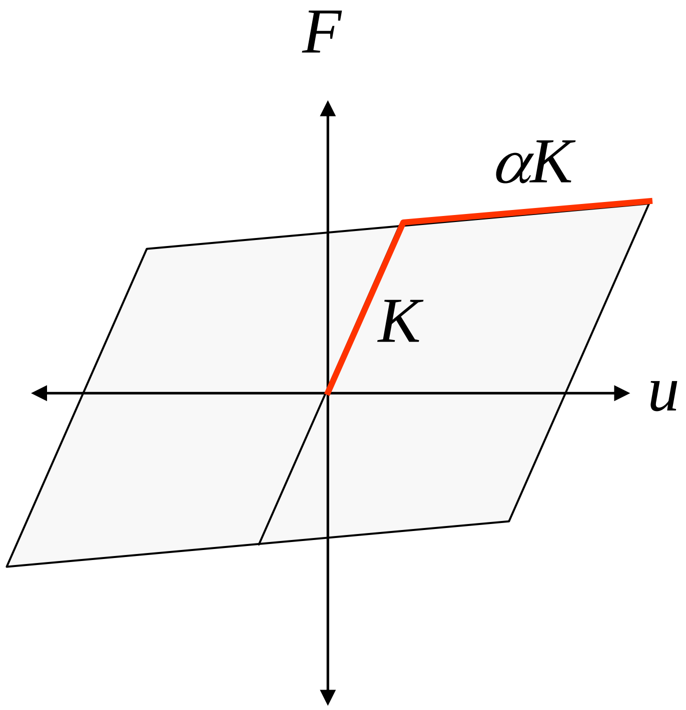
Maximum Steady State Resonant Response:

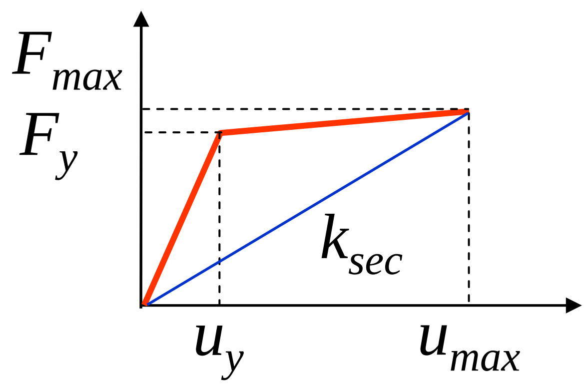
$$u_{\max} = \frac{P_o}{2\xi_{\text{sec}} k_{\text{sec}}}$$

Equivalent Damping:

$$\xi_{\text{sec}} = 0.637\left(1 - \frac{u_y}{u_{\max}}\right) = 0.637\left(1 - \frac{1}{\mu_\Delta}\right)$$

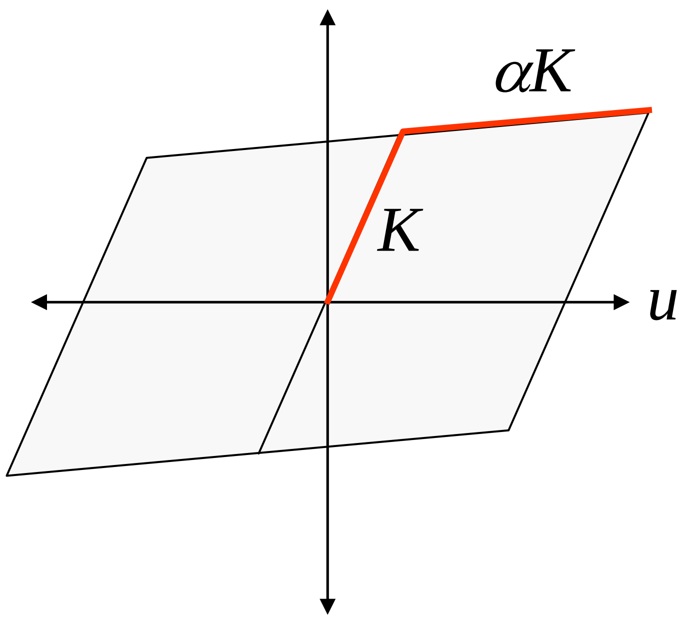
"Equivalent" Elastic System when Strain Hardening is Included

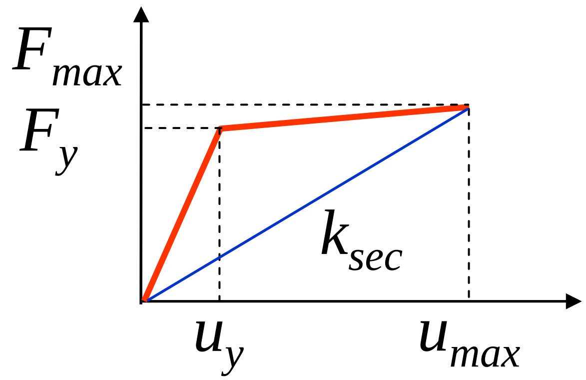




$$\xi_{sec} \equiv 0.637 \frac{(F_y u_{\max} - F_{\max} u_y)}{F_{\max} u_{\max}}$$

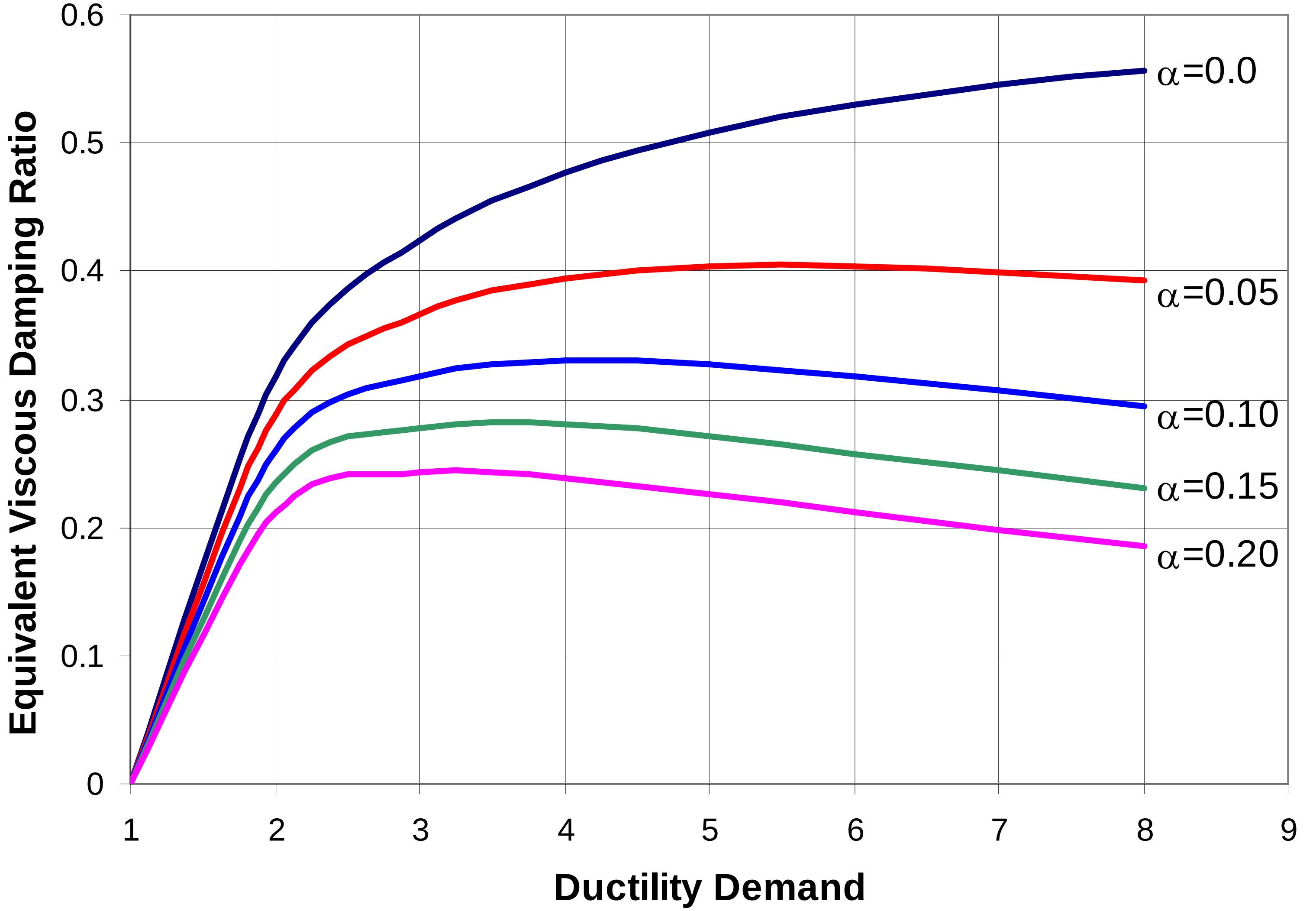
"Equivalent" Elastic System when Strain Hardening is Included





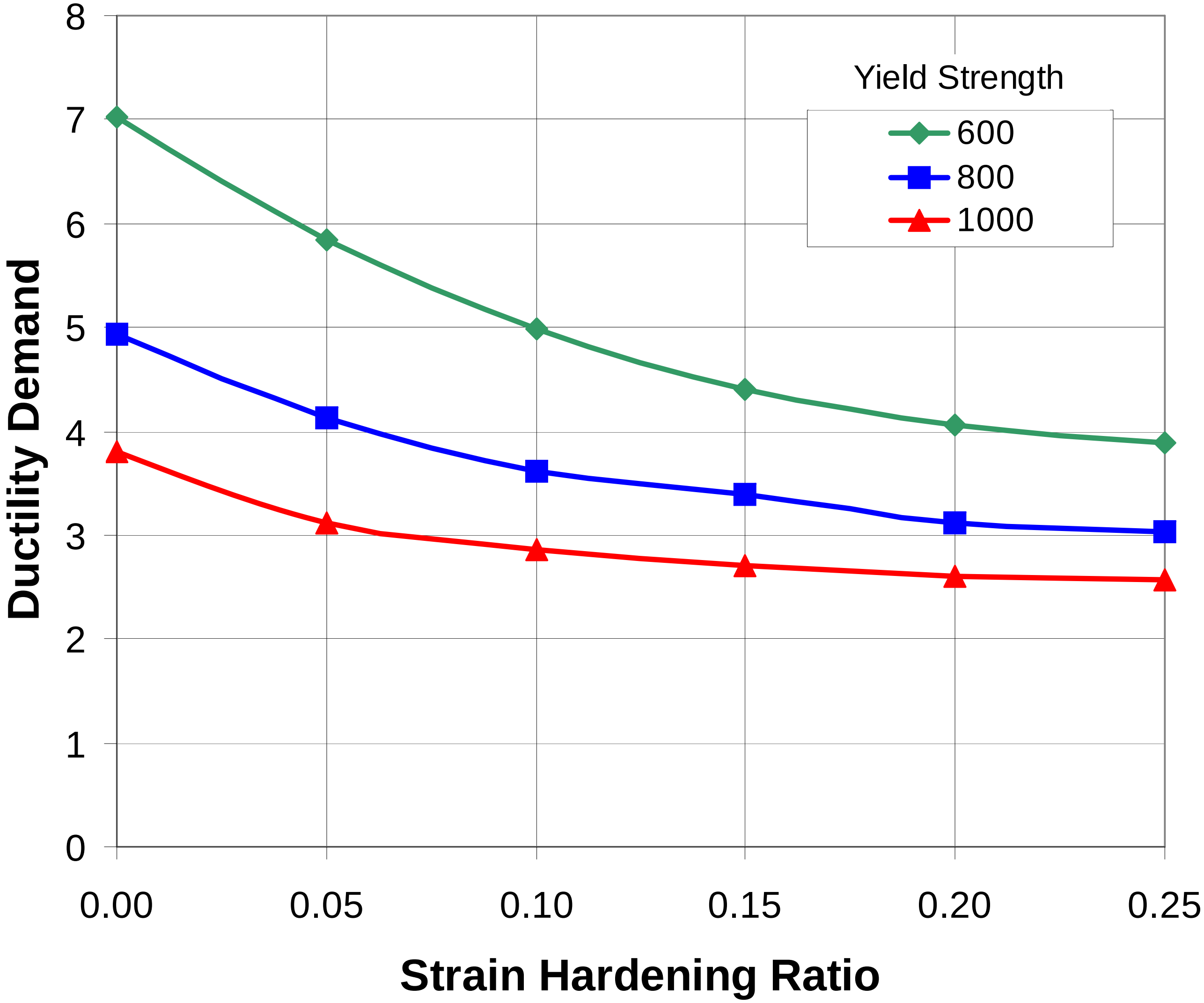
$$\xi_{Equiv} \equiv 0.637\left[\frac{F_y}{F_{\max}} - \frac{u_y}{u_{\max}}\right] = 0.637\left[\frac{1}{\alpha(\mu_\Delta - 1) + 1} - \frac{1}{\mu_\Delta}\right]$$

Effect of Secondary Stiffness On Equivalent Viscous Damping



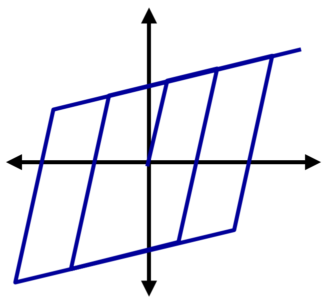
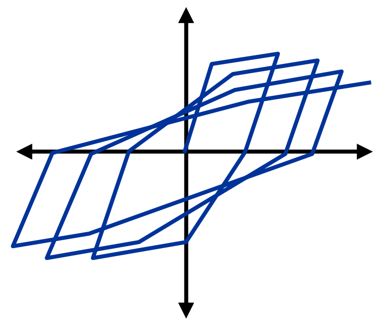
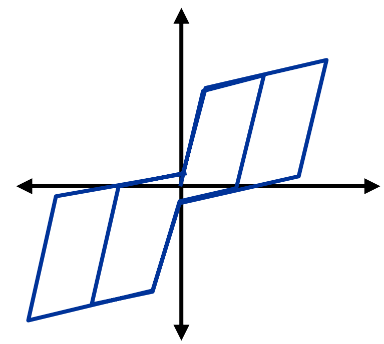
Reduction in Ductility Demand with Strain Hardening Ratio

(W = 11250 k, K = 918 k/in., T=1.0 sec, El Centro Ground Motion)



Total System Damping (% Critical)

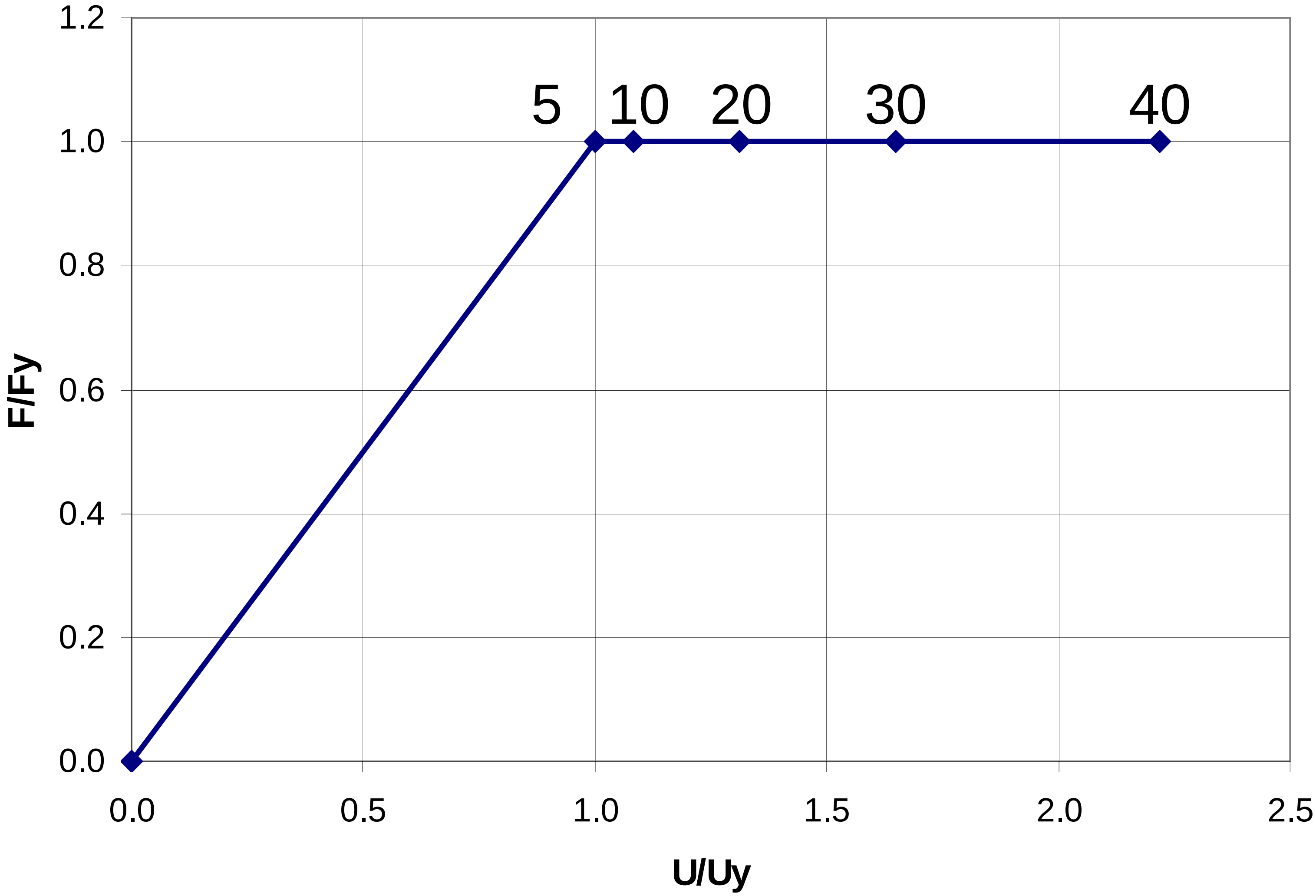
$$\xi_{Total} = 5 + \kappa \xi_{Equiv}$$

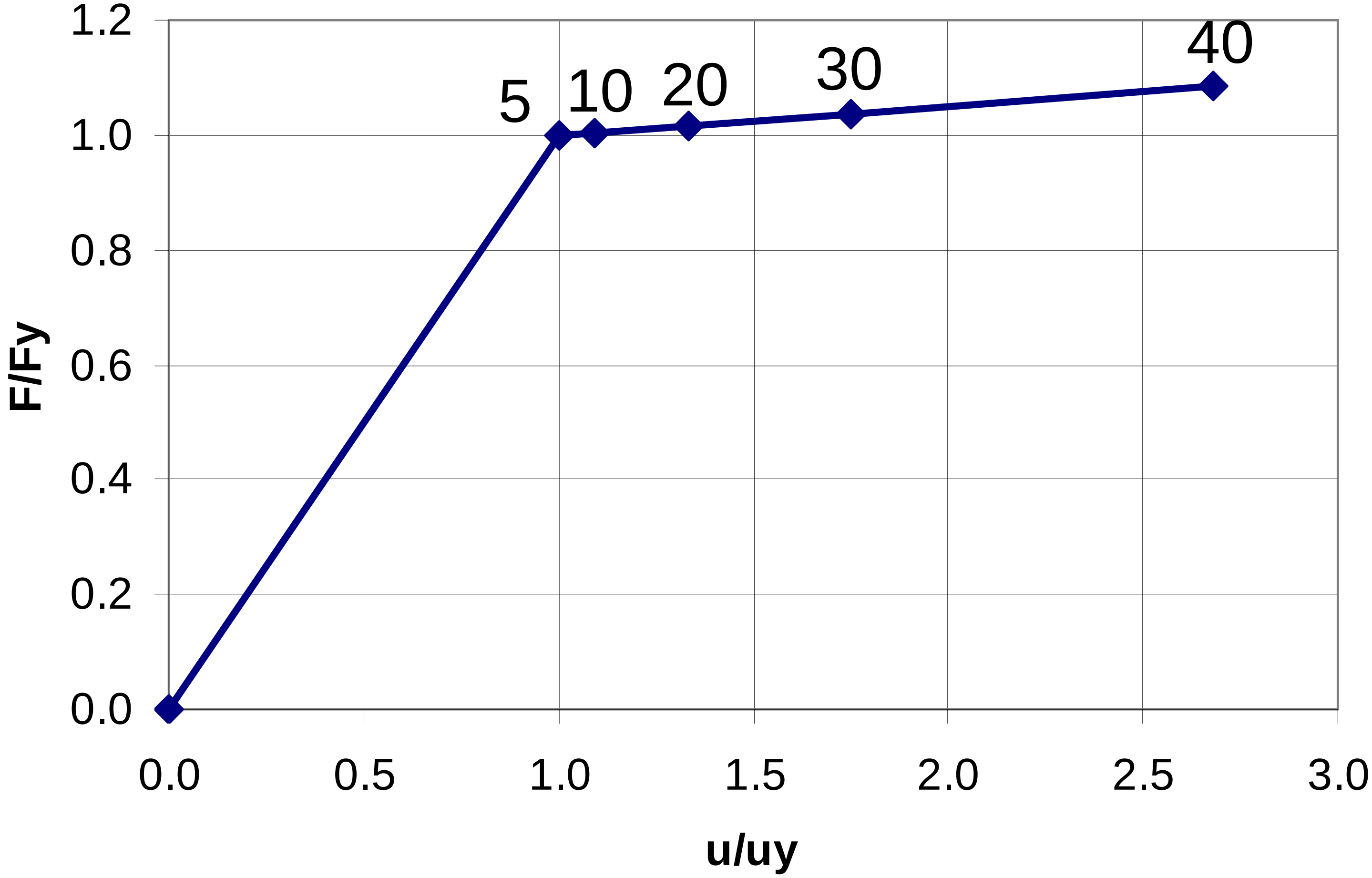
Shaking Duration	Robust	Moderately Robust	Pinched Or Brittle
Short	$\kappa = 1$	$\kappa = .7$	$\kappa = .7$
Long	$\kappa = .7$	$\kappa = .33$	$\kappa = .33$

See ATC 40 for Exact Values

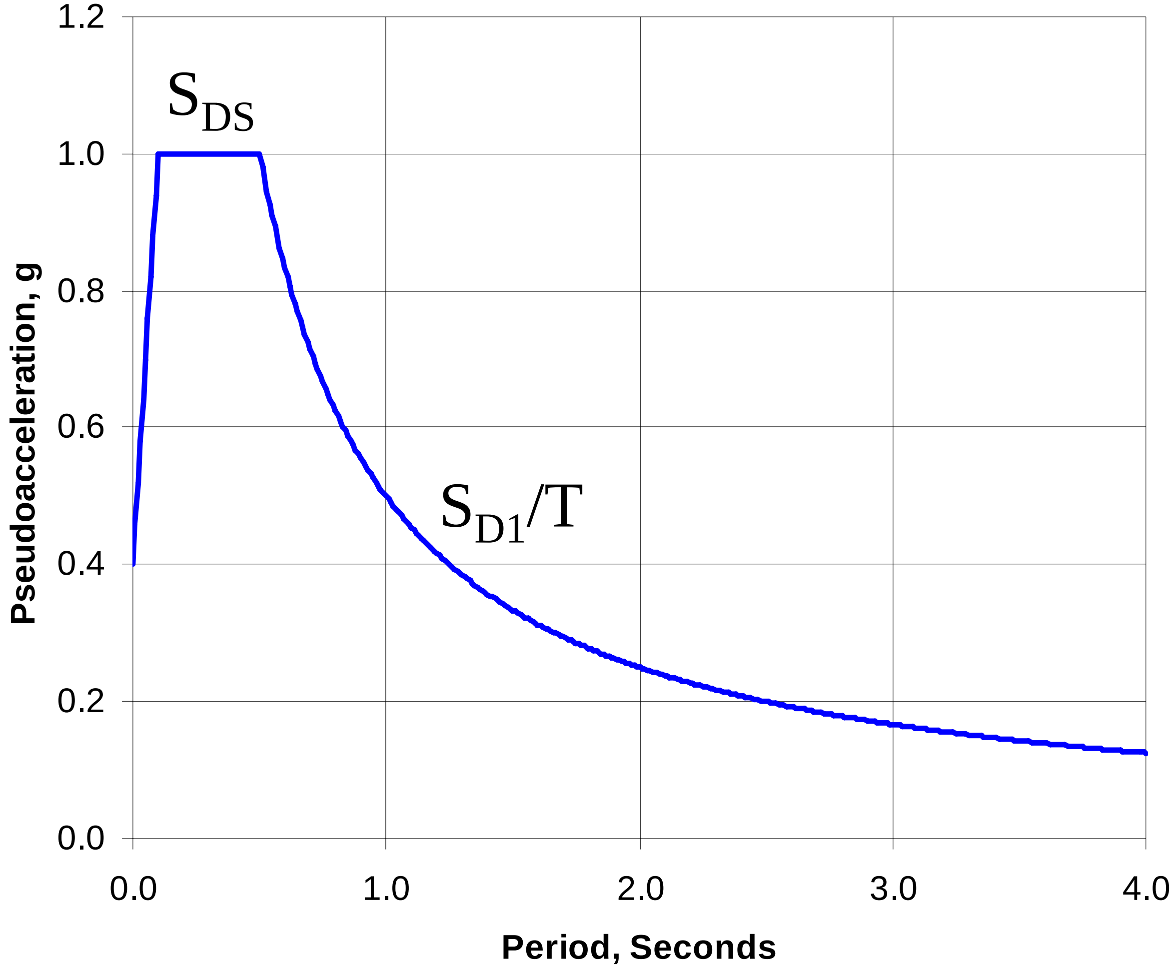
Equivalent Viscous Damping Values for EPP System
(Values Shown are Percent Critical)



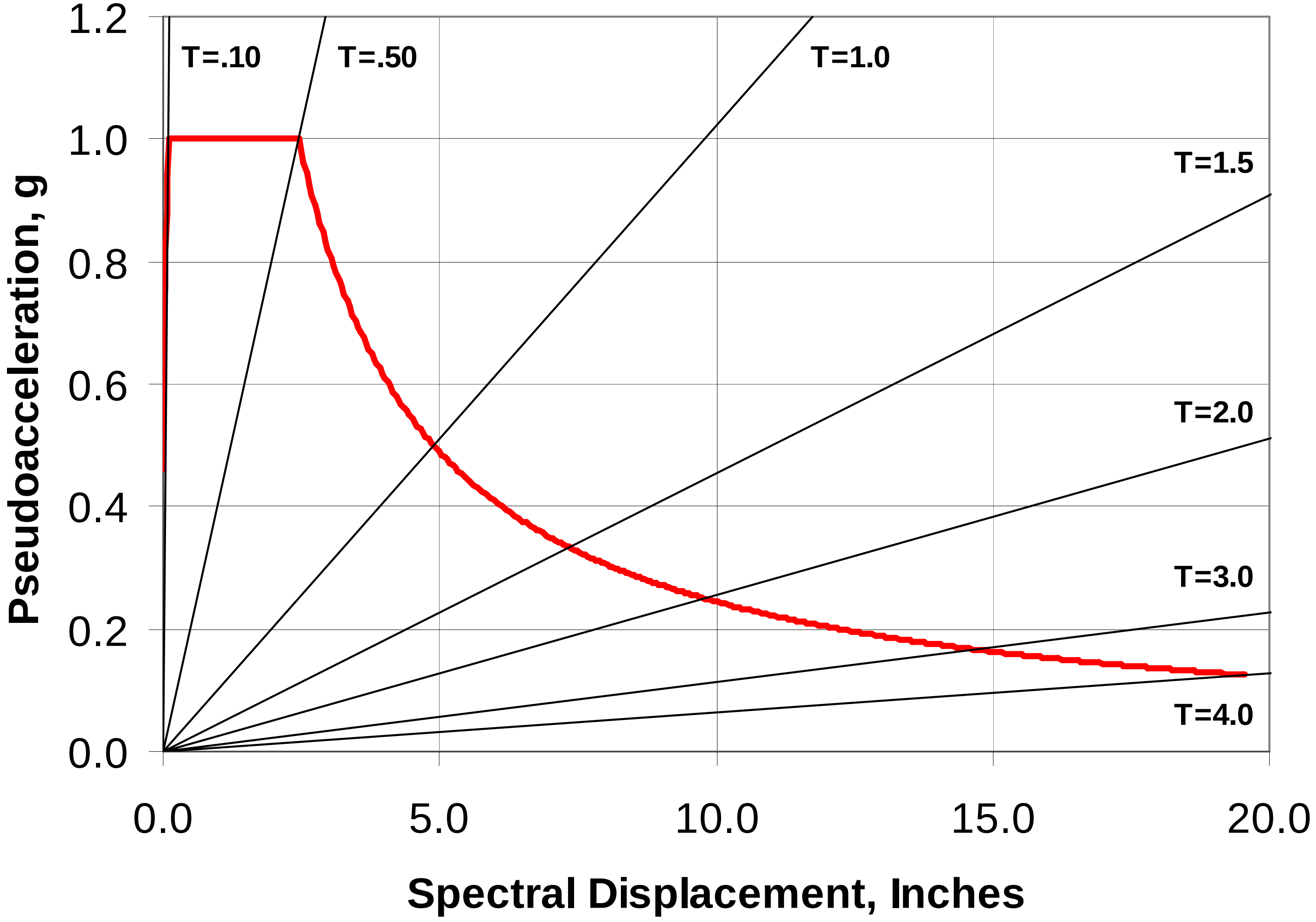
Equivalent Viscous Damping Values for System
With 5% Strain Hardening Ratio
(Values Shown are Percent Critical)



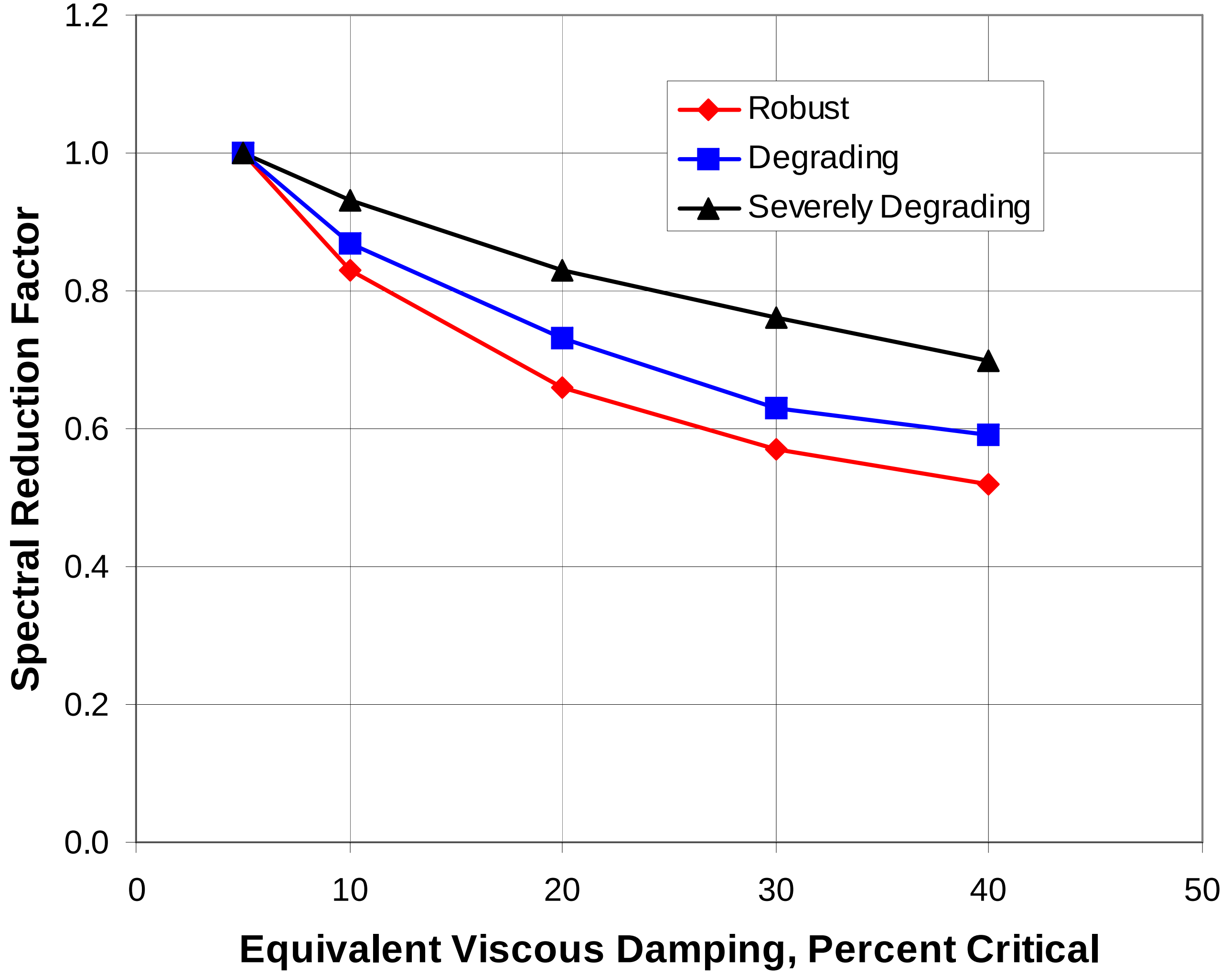
Pseudoacceleration Spectrum in Traditional Format



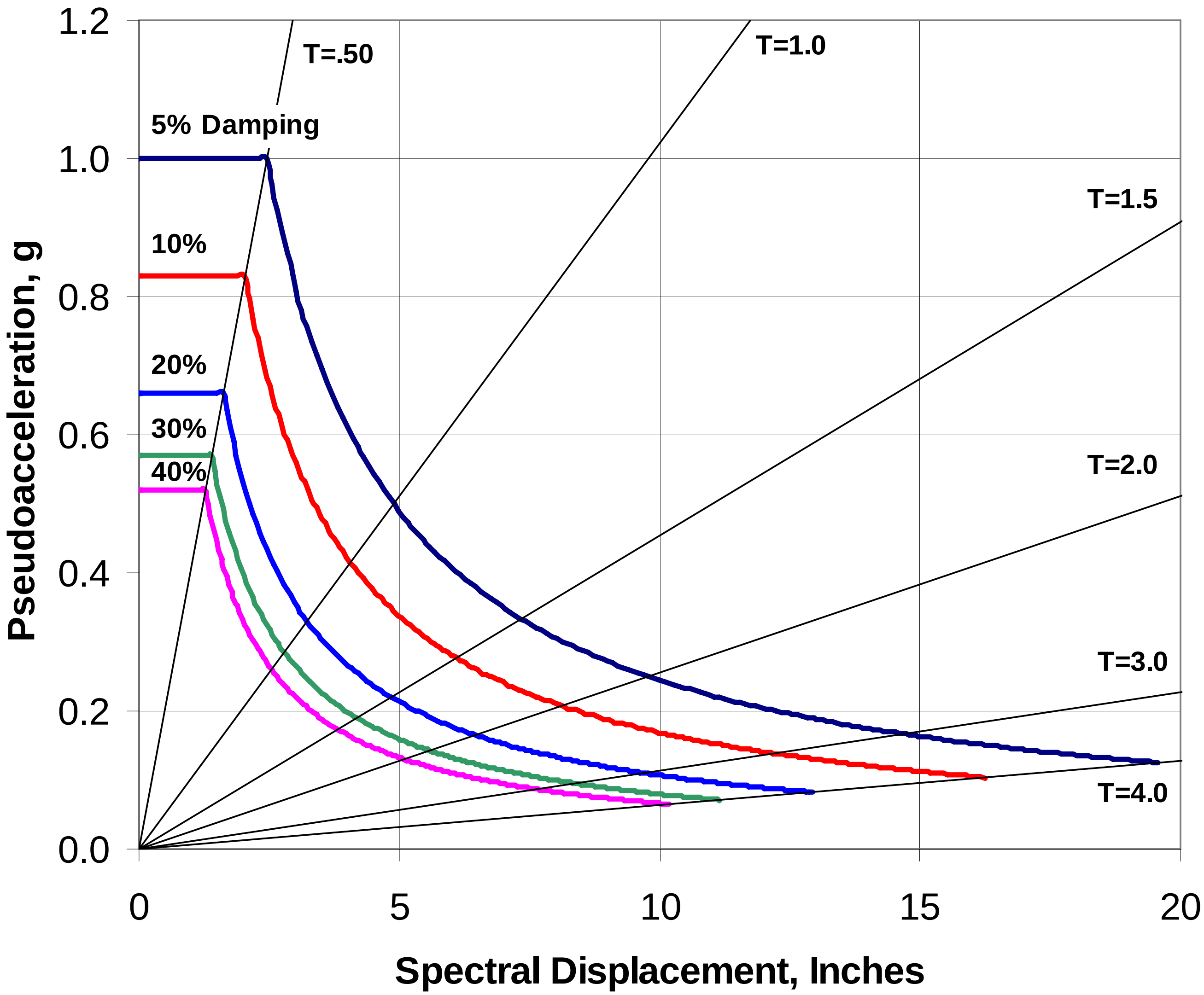
Pseudoacceleration (Demand) Spectrum in ADRS Format (5% Damping)



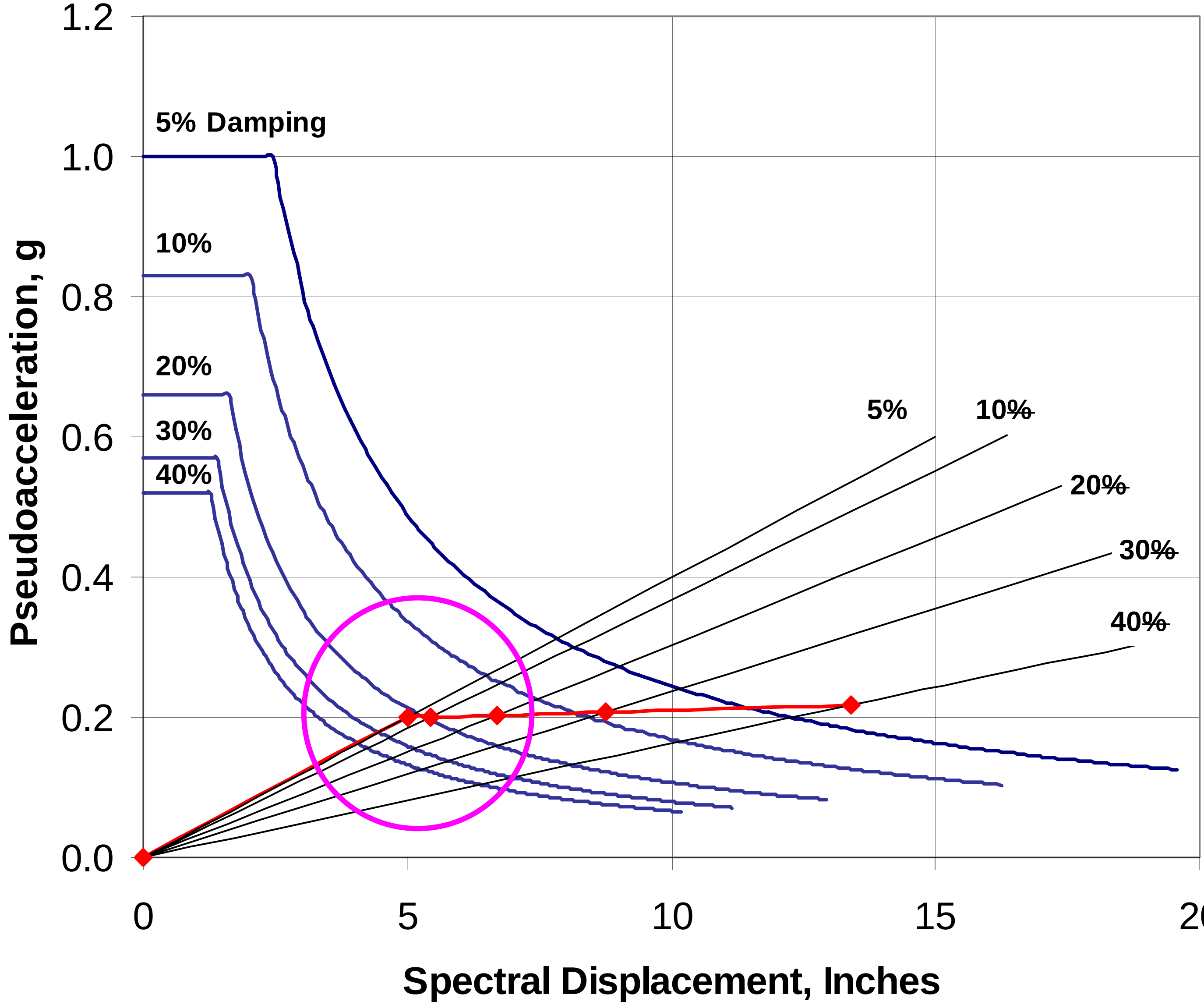
Spectral Reduction Factors for Increased Equivalent Damping



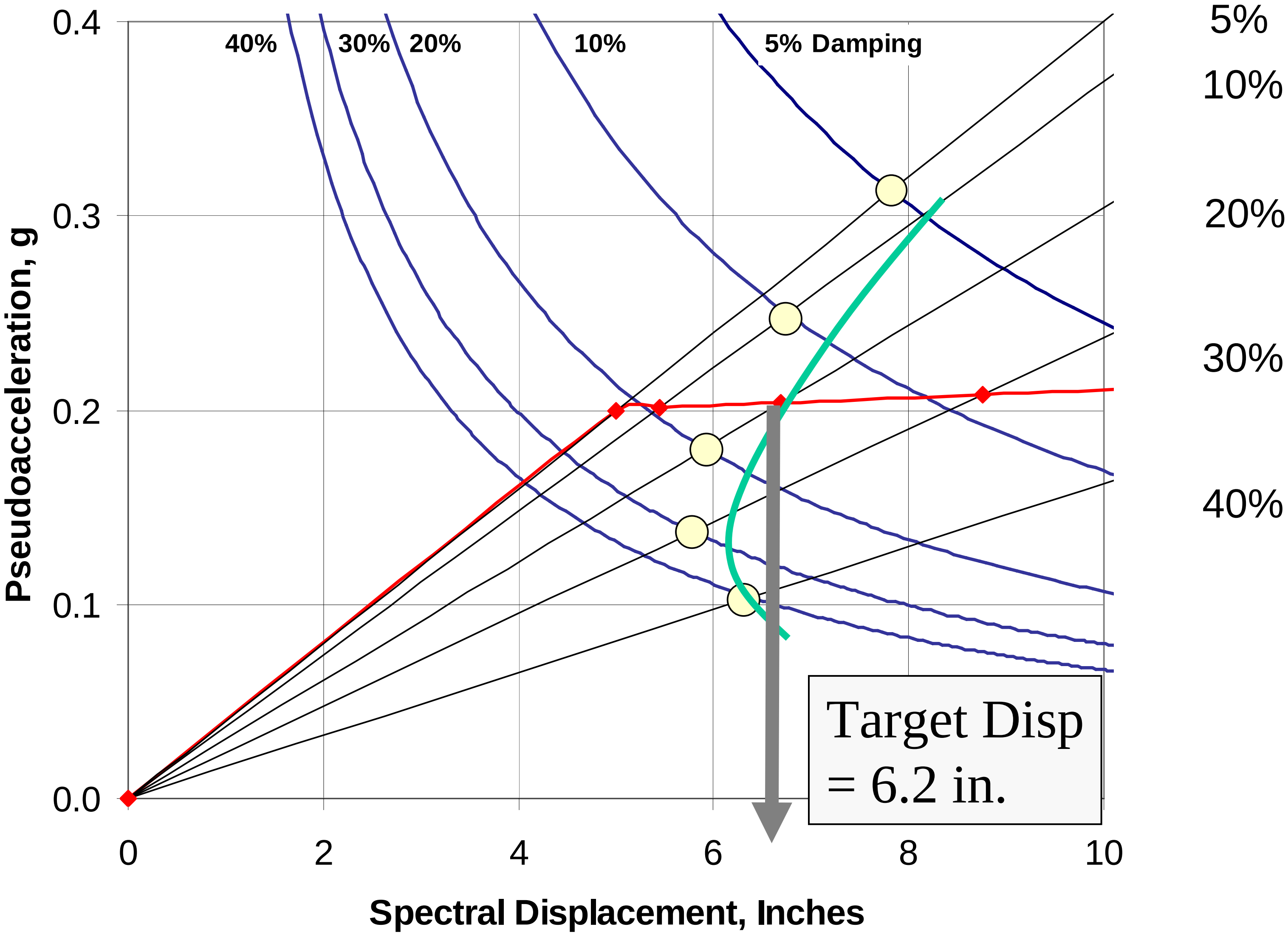
Demand Spectra for Various Damping Values



Combined Capacity-Demand Spectra



Finding the Target Displacement



You are Not Done Yet!

- Note: The target displacement from the Capacity-Demand diagram corresponds to a first mode SDOF system. It must be multiplied by the first mode modal participation factor and the modal amplitude of the first mode mode shape at the roof to determine displacements or deformations in the original system.

  Hinge rotations may then be obtained for comparison with performance criterion.

- Knowing the target displacement, the base shear can be found from the original pushover curve.

“There is sometimes cause to fear that scientific technique, that proud servant of engineering arts, is trying to swallow its master”

Professor Hardy Cross

Simplified Pushover Approaches: 2003 *NEHRP Recommended Provisions*

- Procedure is presented in Appendix to Chapter 5
- Gravity Loads include 25% of live load (but Provisions are not specific on P-Delta Modeling Requirements)
- Lateral Loads Applied in a "First Mode Pattern"
- Structure is pushed to 150% of target displacement
- Target displacement is assumed equal to the displacement computed from a first mode response spectrum analysis, multiplied by the factor C_i
- C_i adjusts for "error" in equal displacement theory when structural period is low

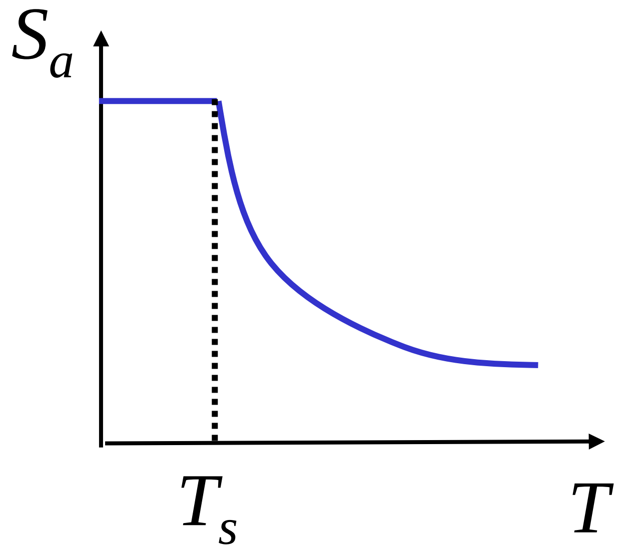
Simplified Pushover Approaches: 2003 *NEHRP Provisions* (2)

$$C_i = \frac{(1 - T_s / T_1)}{R_d} + (T_s / T_1)$$

C_i=1 if T_s/T_1 <1

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

$$R_d = \frac{1.5R}{\Omega_0}$$



Simplified Pushover Approaches: 2003 *NEHRP Provisions* (3)

- Member strengths need not be evaluated
- Component deformation acceptance based on laboratory tests
- Maximum story drift may be as high as 1.25 times standard limit
- Nonlinear Analysis must be Peer Reviewed

Simplified Pushover Approaches: FEMA 356*. (Also used in FEMA 350)

- Procedure presented in Chapter 3
- More detailed (thoughtful) treatment than in *NEHRP Recommended Provisions*

Principal Differences:

> Apply 25% of unreduced Gravity Load
> Use of two different lateral load patterns
> P-Delta effects included
> Consideration of Hysteretic Behavior

*** FEMA 273 in Prestandard Format**

Simplified Pushover Approaches: FEMA 356 (2)

Spectral Displacement

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

δ_t = Target Displacement

C_0 = Modification factor to relate roof displacement to first mode spectral displacement.

C_1 = Modification factor to relate expected maximum inelastic displacement to displacement calculated from elastic response (similar to NEHRP *Provisions* C_i)

Simplified Pushover Approaches: FEMA 356 (3)

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

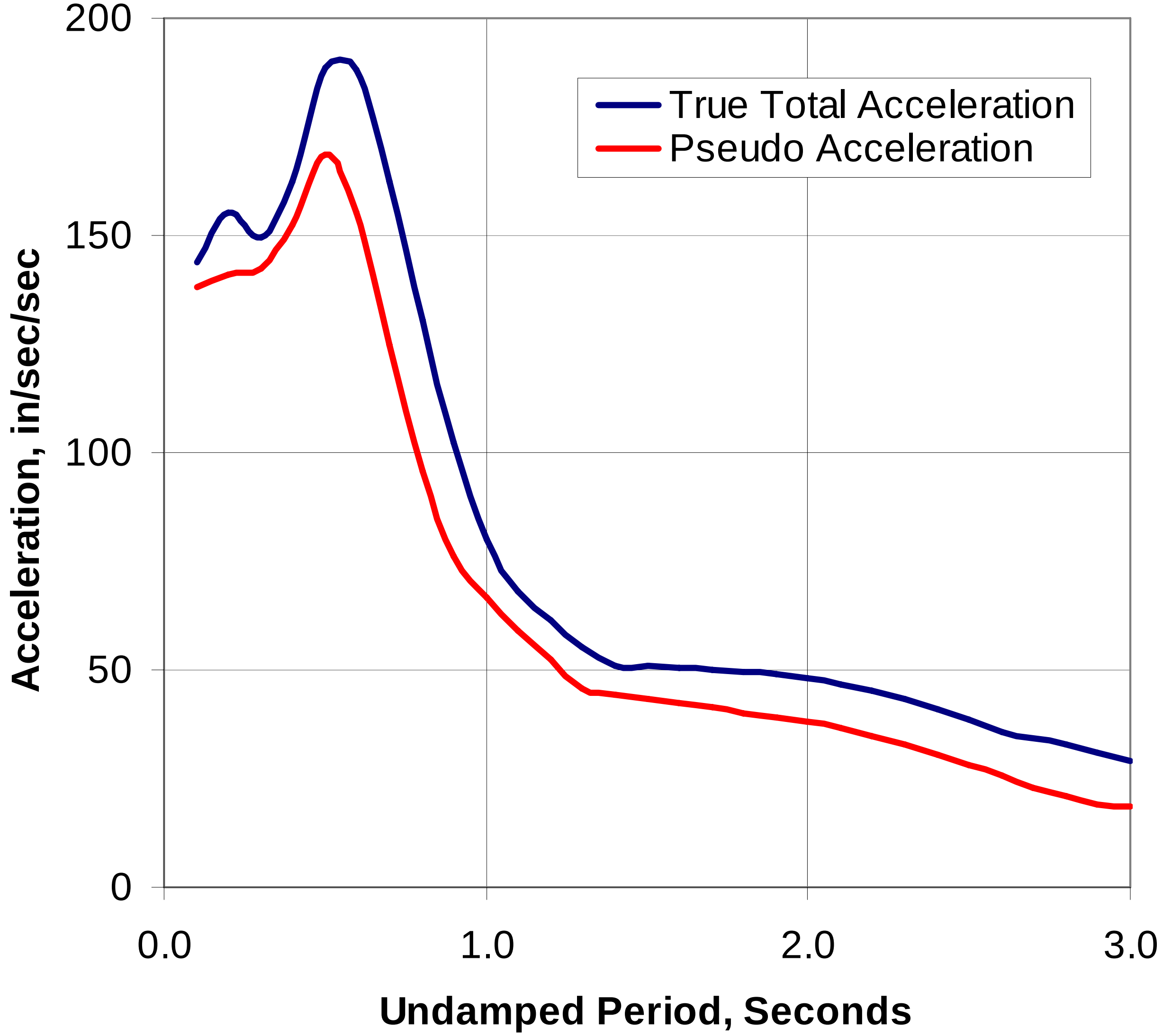
C_2 = Modification factor to represent effect of pinched hysteretic loop, stiffness degradation, and strength loss.

C_3 = Modification factor to represent increased displacements due to dynamic P-Delta effect

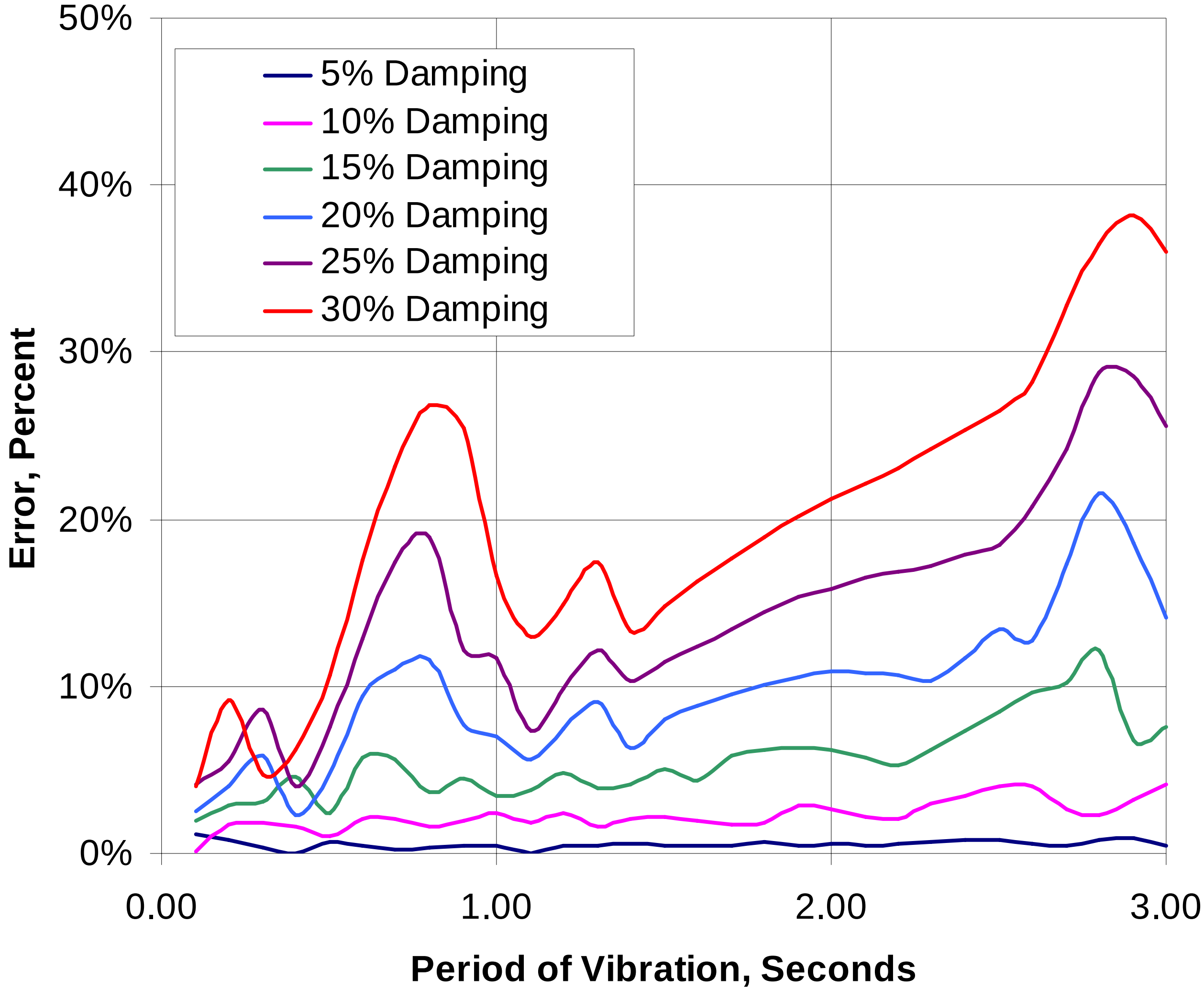
Discussion of Assumptions

1. Dynamic effects are ignored
2. Duration effects are ignored
3. Choice of lateral load pattern
4. Only first mode response included
5. Use of elastic response spectrum
6. Use of equivalent viscous damping
7. Modification of response spectrum for higher damping

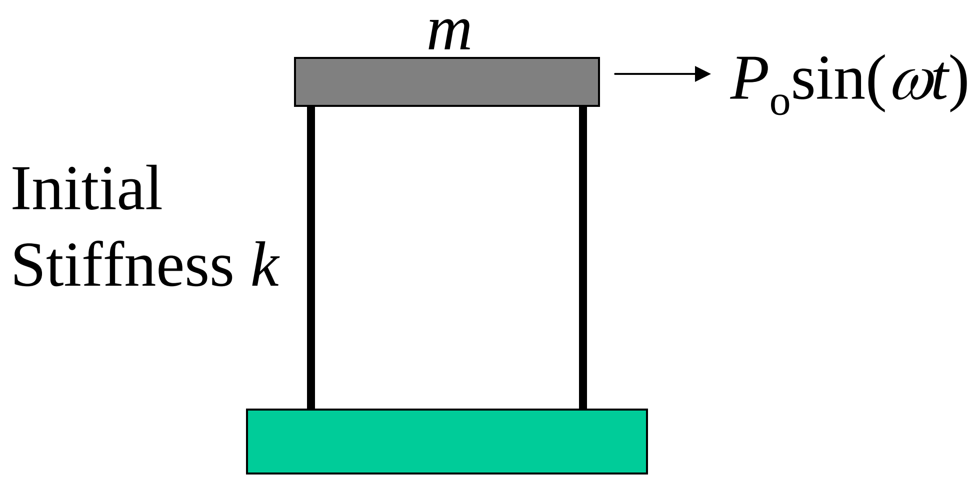
True Acceleration vs Pseudoacceleration
30% Critical Damping

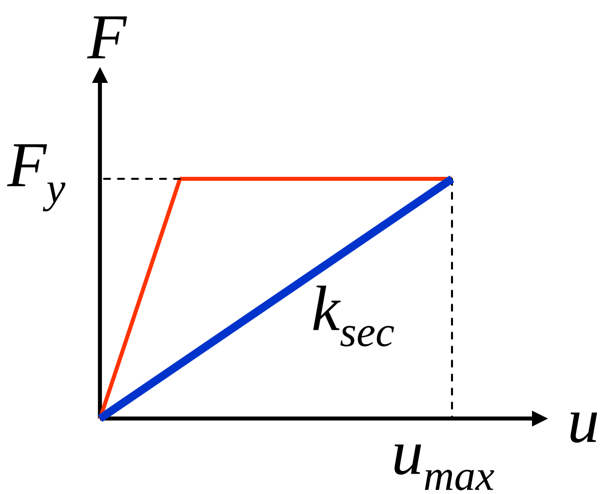


Relative Error Between True Acceleration and Pseudoacceleration



“Equivalent” Elastic System





Resonant Frequency:

$$\omega_{\sec} = \sqrt{\frac{k_{\sec}}{m}}$$

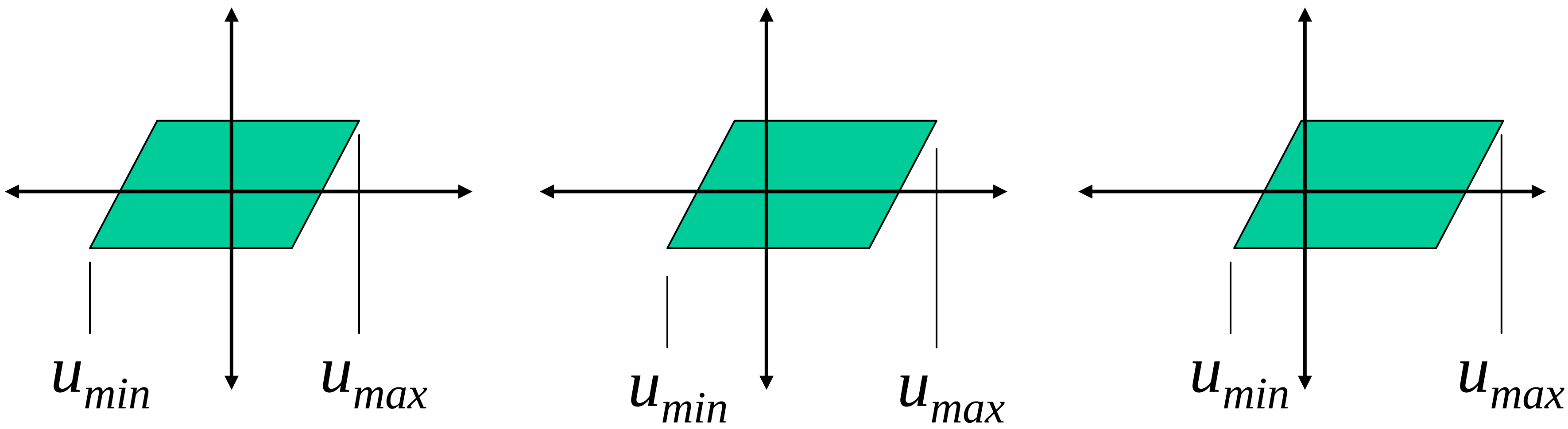
Maximum Steady State Resonant Response:

$$u_{\max} = \frac{k_{\sec}}{2\xi_{\sec} P_o}$$

Equivalent Damping:

$$\xi_{\sec} = 0.637\left(1 - \frac{u_y}{u_{\max}}\right)$$

These systems have the same hysteretic Energy Dissipation, the same AVERAGE (+/-) displacement, but considerably DIFFERENT maximum displacement.

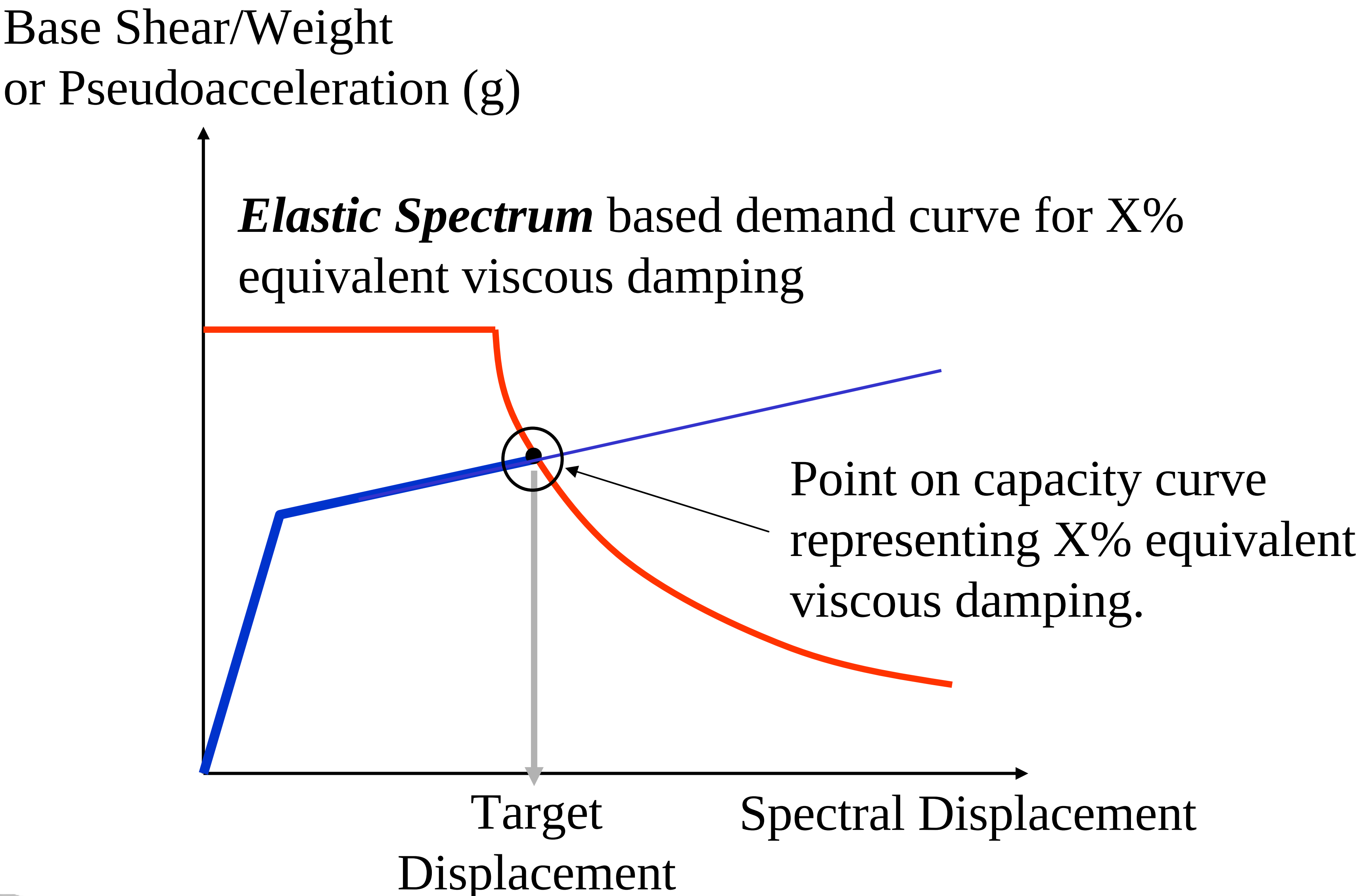


The equivalent viscous damping (see previous slide) is good at predicting the AVERAGE displacement, but CAN NOT predict the true maximum displacement.

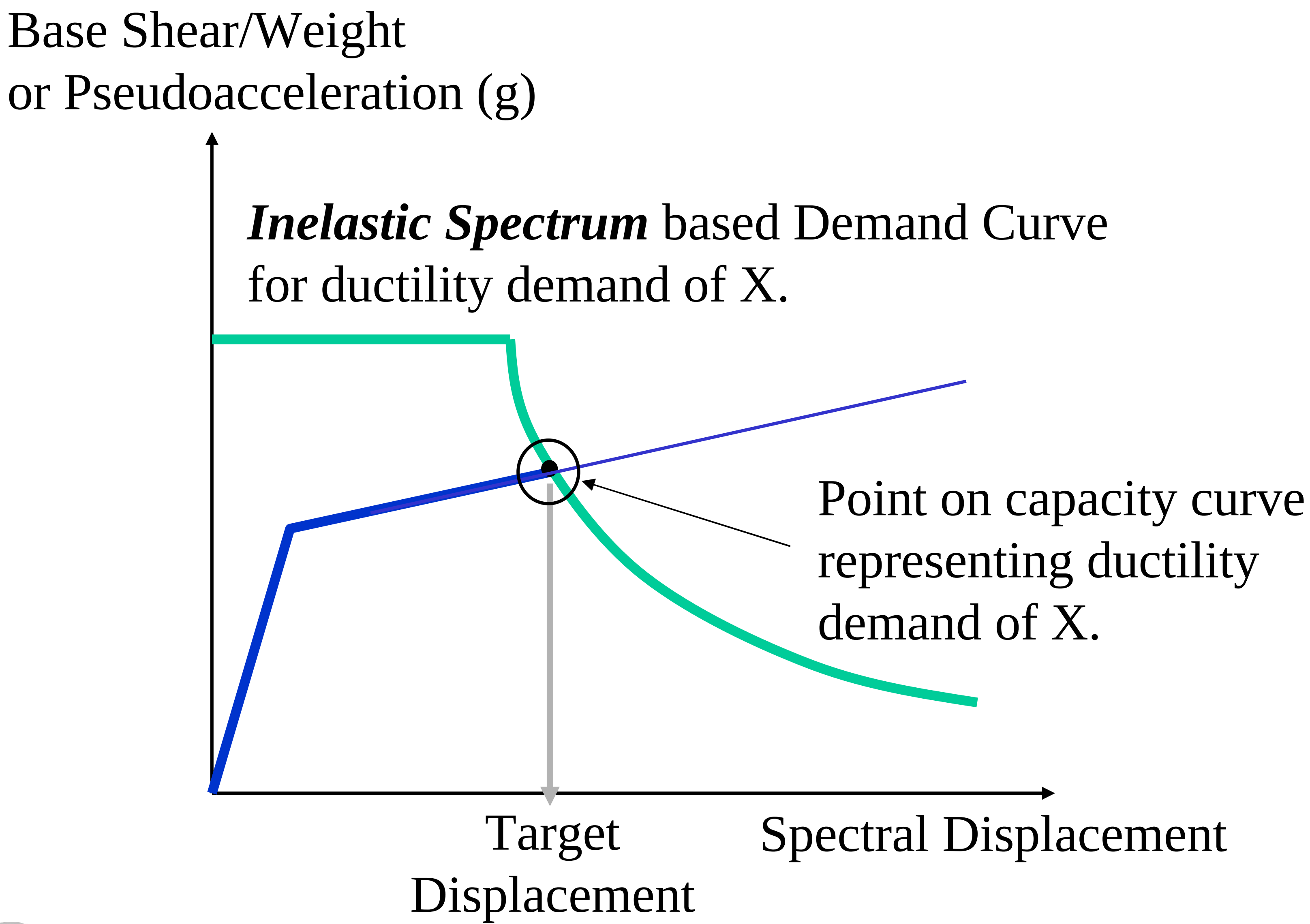
“Improved” Pushover Methods

- Use of Inelastic Response Spectrum
- Adaptive Load Patterns
- Use of SDOF Response History Analysis
- Inclusion of Higher Mode Effects

Elastic Spectrum Based Target Displacement



Inelastic Response Spectrum Based Target Displacement



Inelastic Spectrum Based Target Displacement

- Gives the same results as the equal displacement theory for (longer period) EPP systems

- When compared to inelastic response history analysis, the use of inelastic spectra gives better results than ATC 40 procedure.

Computing Target Displacements from Response History Analysis of SDOF Systems

- Method called "Uncoupled Modal Response History Analysis" (UMRHA) is described by Chopra and Goel. See, for example, Appendix A of PEER Report 2001/03, entitled *Modal Pushover Analysis Procedure to Estimate Seismic Demands for Buildings*.
- In the UMHRA method, the undamped mode shapes are used to determine a static load pattern for each mode.
- Using these static lateral loads, a series of pushover curves and corresponding bilinear capacity curves are obtained for the first few modes. This is done using the procedures described earlier for the ATC 40 approach.

Computing Target Displacements from Response History Analysis of SDOF Systems (2)

- Using an appropriate ground motion, a nonlinear dynamic response history analysis is computed for each modal bilinear system. This may be accomplished using NONLIN or NONLIN-Pro.
- The modal response histories are transformed to system coordinates and displacement (and deformation) response histories are obtained for each mode.
- The modal response histories are added algebraically to determine the final displacement (deformations). In the Modal Pushover approach, the individual response histories are combined using SRSS.

Computing Target Displacements from Response History Analysis of SDOF Systems (3)

- Results from such an analysis are detailed in PEER Report 2001/16, entitled *Statistics of SDF-System Estimate of Roof Displacement for Pushover Analysis of Buildings*.

Conclusions from above report (paraphrased by F. Charney):

- For larger ductility demands the SDOF method, using only the first mode, overestimates roof displacements and the bias increases for longer period buildings.

- For small ductility demand systems, the SDOF system, using only the first mode, underestimates displacement, and the bias increases for longer period systems.

Conclusions (continued)

- First mode SDOF estimates of roof displacements due to individual ground motions can be alarmingly small (as low as 0.31 to 0.82 times "exact") to surprisingly large (1.45 to 2.15 times exact).
- Errors increase when P-Delta effects are included. (Note: the method includes P-Delta effects only in the first mode).
- The large errors arise because for individual ground motions the first mode SDOF system may underestimate or overestimate the residual deformation due to yield-induced permanent drift.
- <u>The error is not improved significantly by including higher mode contributions</u>. However, the dispersion is reduced when elastic or nearly elastic systems are considered.

Computing Target Displacements from Response History Analysis of SDOF Systems

Problems with the method:

- **No rational basis**
- Does not include P-Delta effects in higher modes
- Can not consider differences in hysteretic behavior of individual components
- No reduction in effort compared to full time-history analysis
- Problem of ground motion selection and scaling still exists

Structural Analysis for Performance-Based Earthquake Engineering

- Basic modeling concepts
- Nonlinear static pushover analysis
- Nonlinear dynamic response history analysis
- Incremental nonlinear dynamic analysis
- Probabilistic approaches

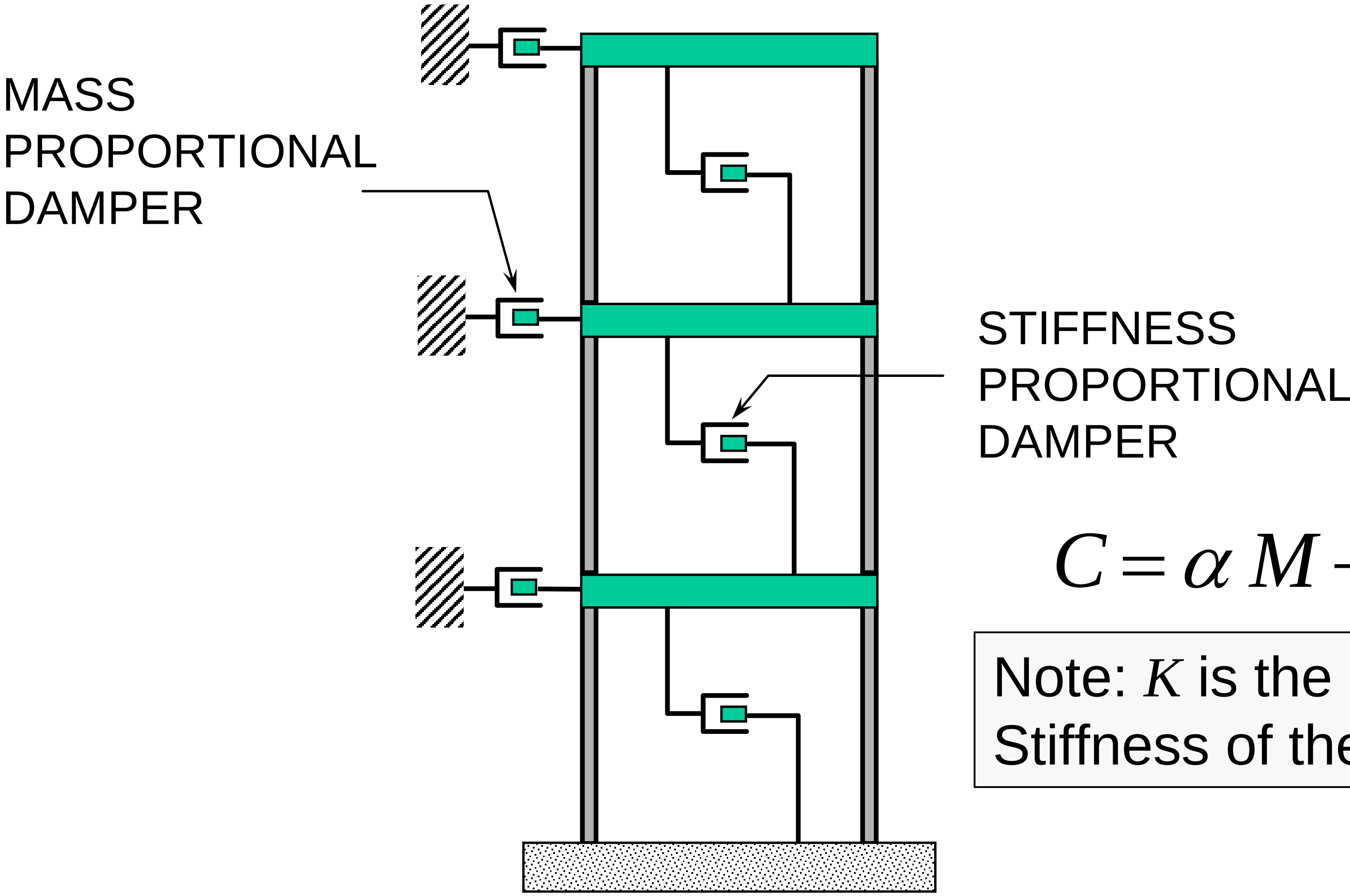
Nonlinear Dynamic Response History Analysis

Principal Advantage: **All** problems with pushover analysis are eliminated. However, new problems may arise.

Main Concerns in Nonlinear Dynamic Analysis:

1) Modeling of hysteretic behavior
2) Modeling inherent damping
3) Selection and scaling of ground motions
4) Interpretation of results
5) Results may be very sensitive to seemingly minor perturbations

Modeling Inherent Damping Using Rayleigh Proportional Damping



$$C = \alpha M + \beta K$$

Note: K is the INITIAL Stiffness of the system

Rayleigh Proportional Damping

Select Damping value in two modes, ξ_k and ξ_n

Compute Coefficients α and β:

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = 2\frac{\omega_k \omega_n}{\omega_n^2 - \omega_k^2} \begin{bmatrix} \omega_n & -\omega_k \\ -1/\omega_n & 1/\omega_k \end{bmatrix} \begin{Bmatrix} \xi_k \\ \xi_n \end{Bmatrix}$$

Form Damping Matrix $C = \alpha M + \beta K$

Rayleigh Proportional Damping (Example)

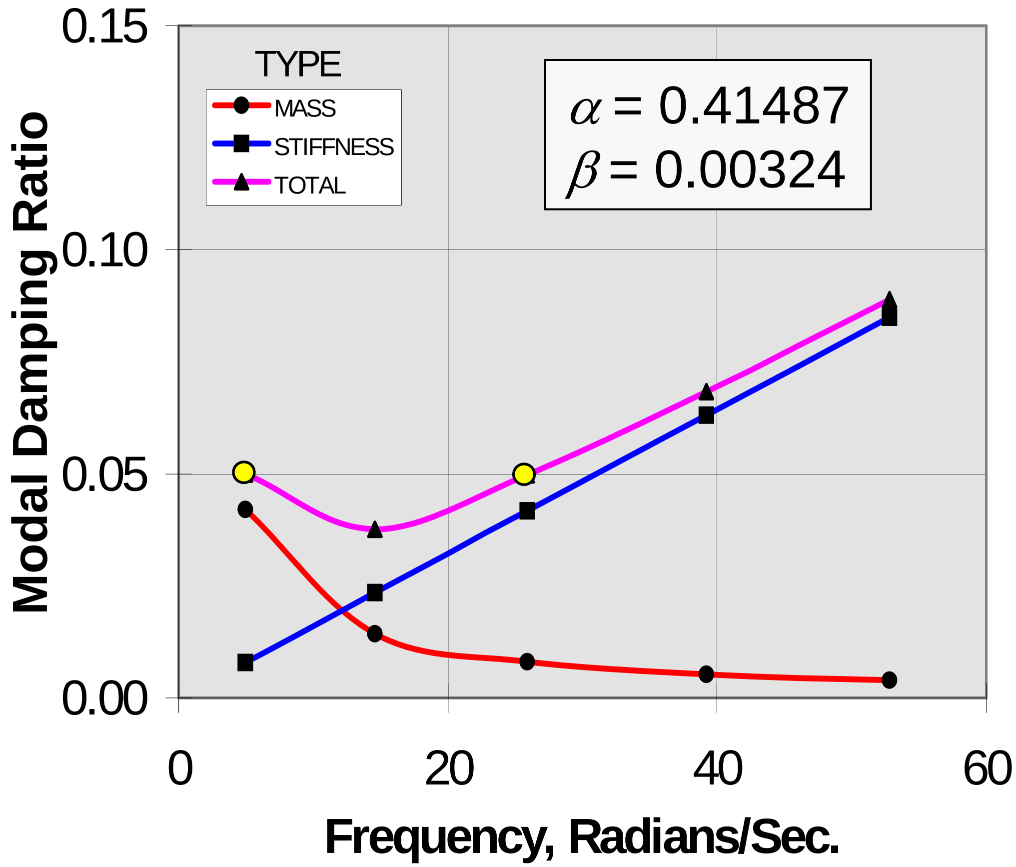
5% Critical in Modes 1 and 3

Damping in any other Mode m:

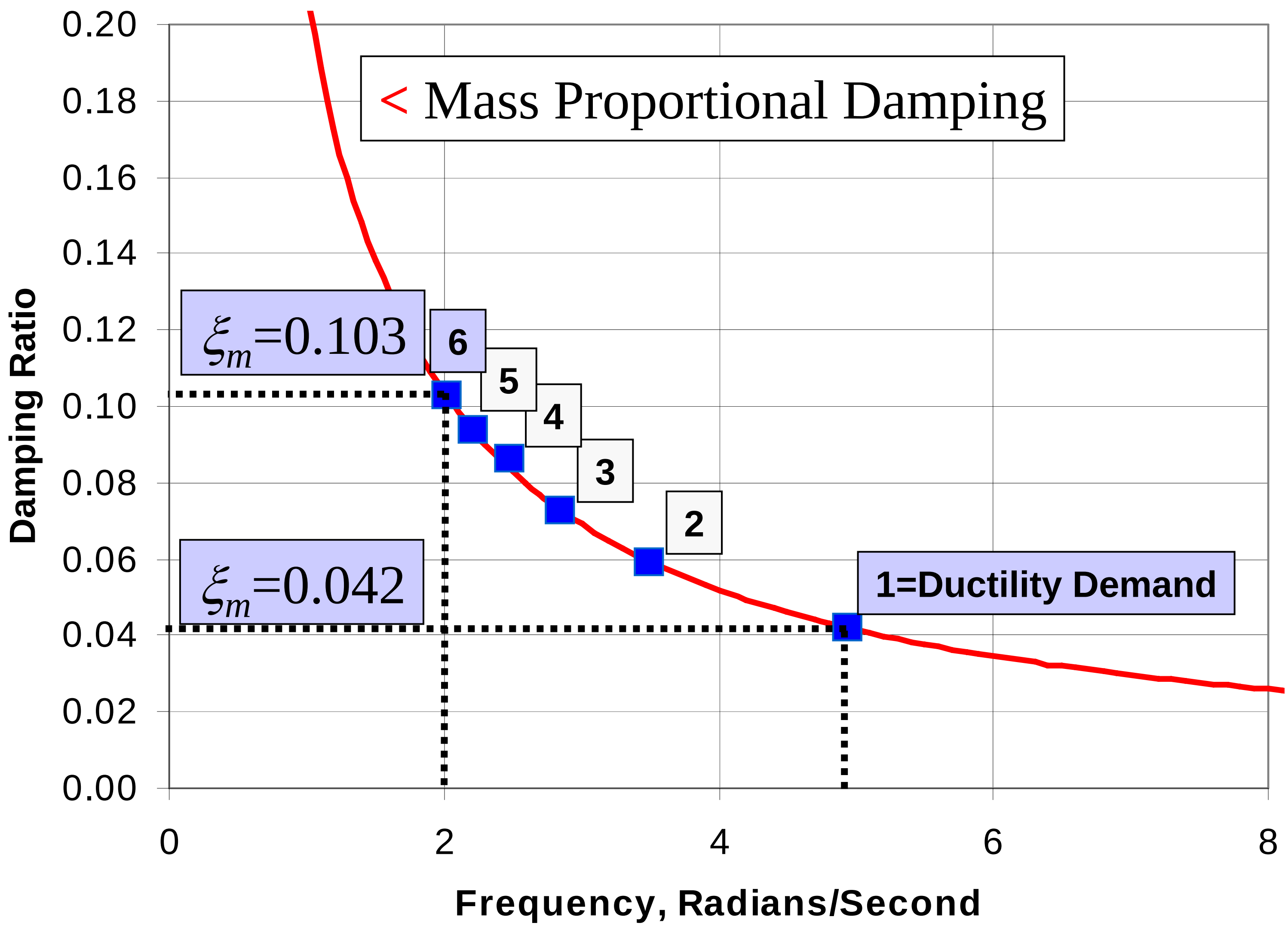
$$\xi_m = 0.5 \begin{bmatrix} \dfrac{1}{\omega_m} & \omega_m \end{bmatrix} \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix}$$

Structural Frequencies

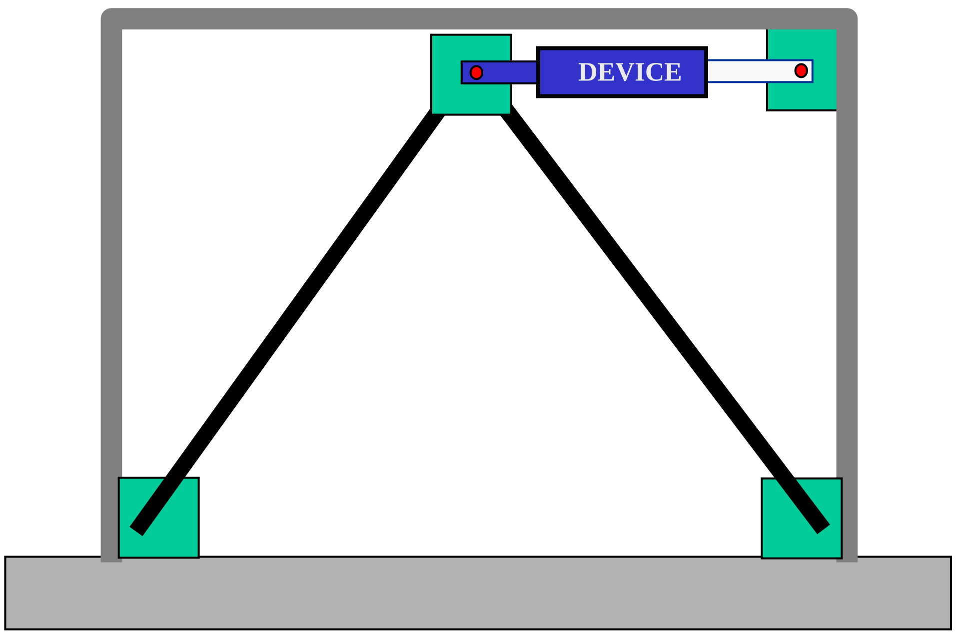
Mode	ω
1	**4.94**
2	14.6
3	**25.9**
4	39.2
5	52.8



Loss of stiffness, frequency shift, and higher mass proportional damping

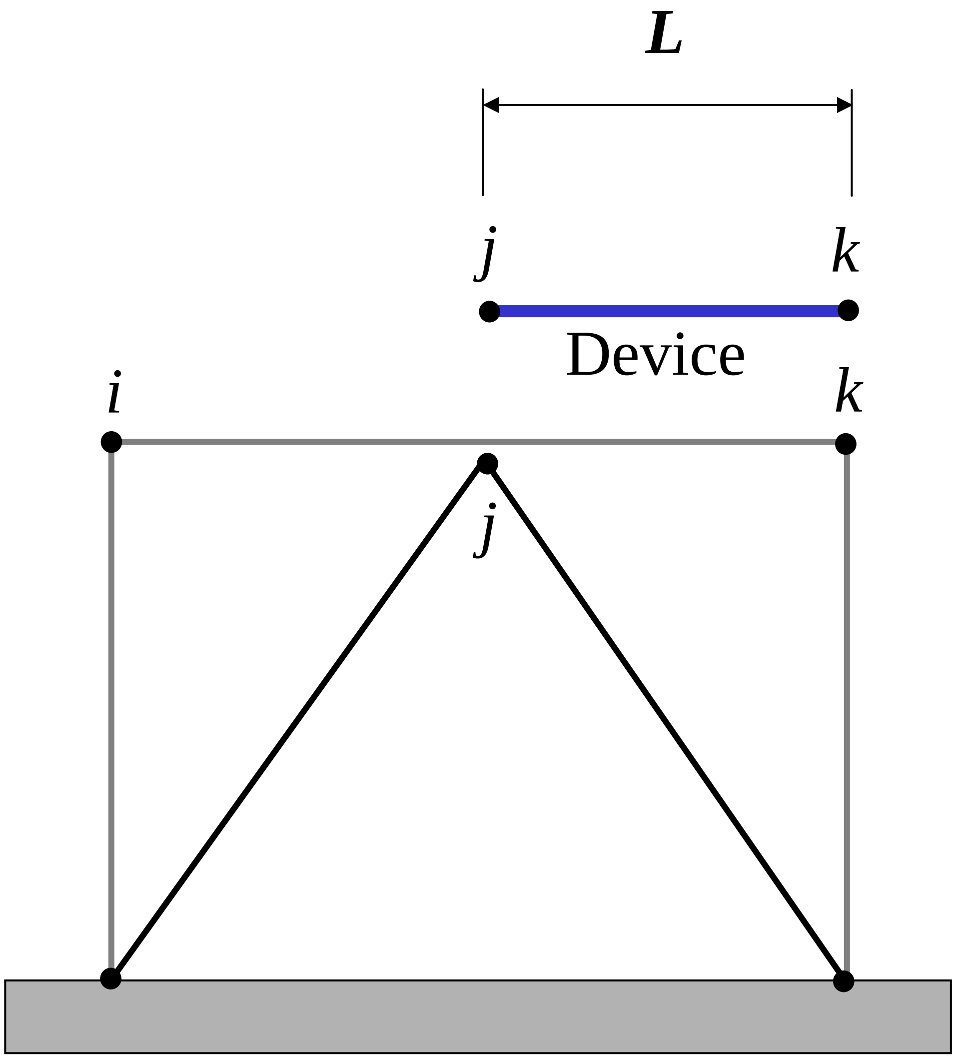


Modeling Linear Viscous Dampers in DRAIN



Note: Nonlinear Damping is NOT Available in DRAIN.

Modeling Linear Viscous Dampers in DRAIN



Use element stiffness proportional damping.

$$K_{Damper} = \frac{AE}{L}$$

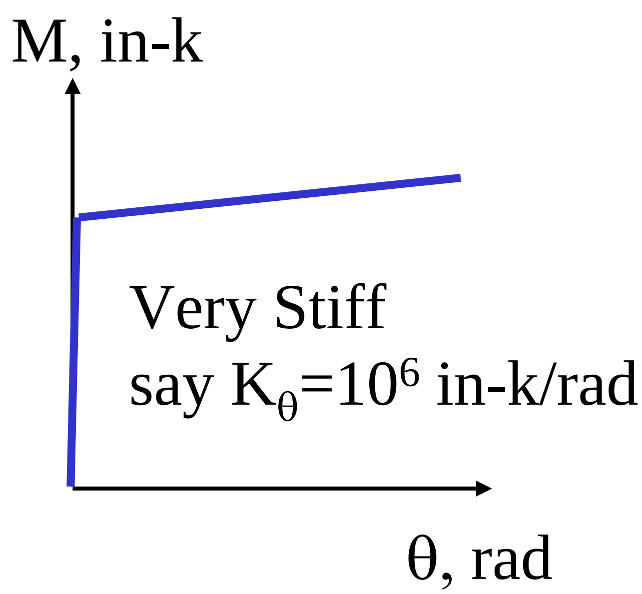
$$C_{Damper} = \beta K_{Damper}$$

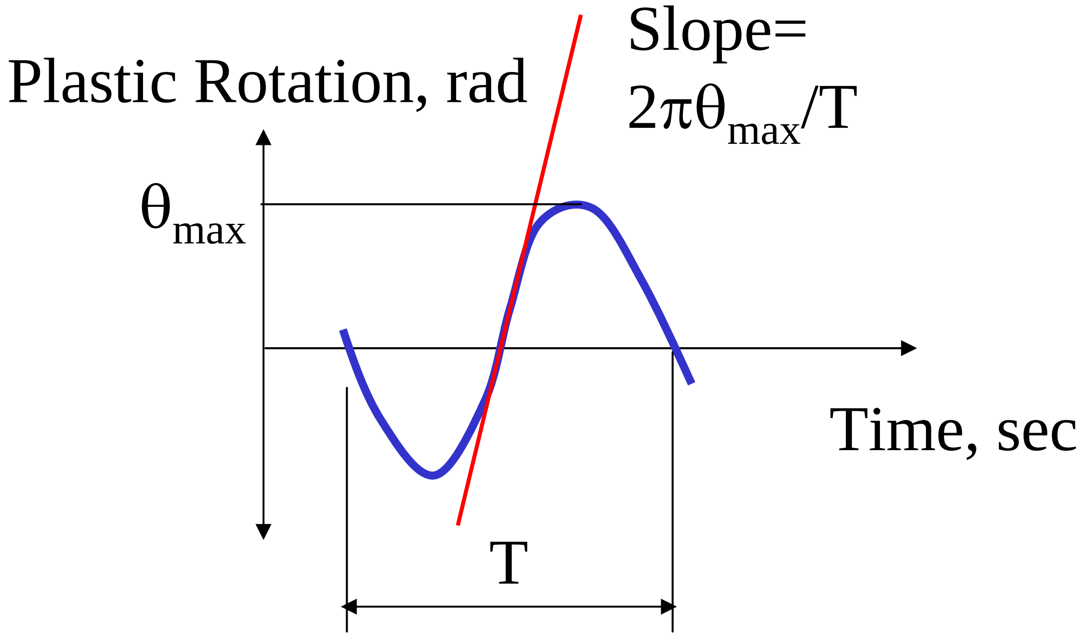
For low damper stiffness:
Set $A=L$, $E=0.01$

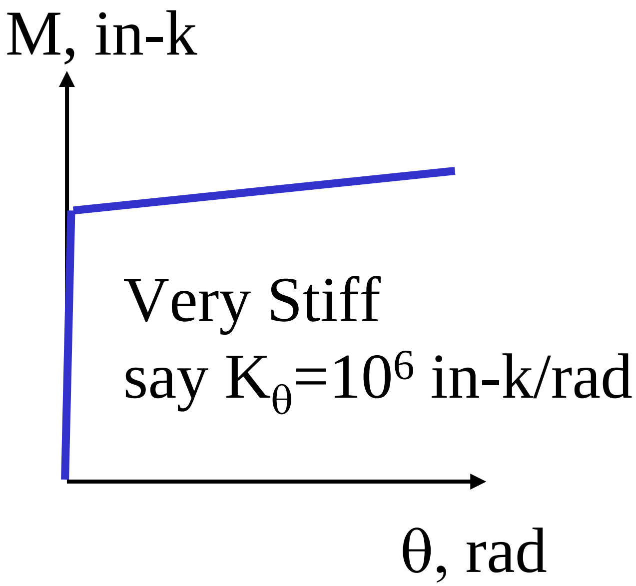
use $\beta = C_{Damper}/0.01$

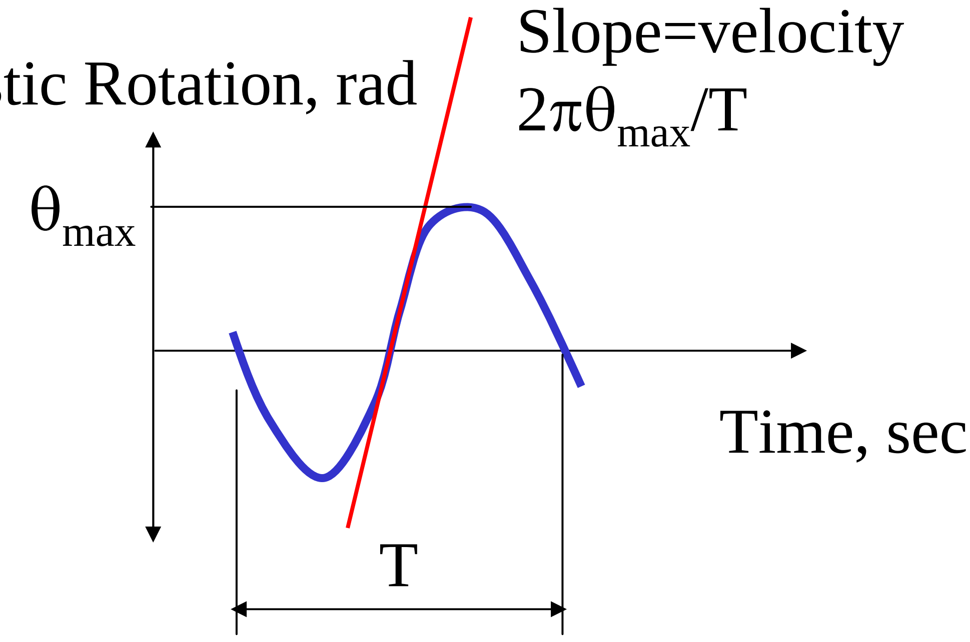
Caution Regarding Stiffness Proportional Damping

NEVER use stiffness proportional damping in association with ANY elements that have artificially high stiffness and that may yield.









Viscous Moment in Hinge = $K_\theta \beta (2\pi\theta_{max}/T)$

Assume θ_{max} = .03 rad, T=1.0 sec, β=0.004

$M=10^6(0.004)(2\pi(.03)/1.0))=7540$ in-k

NEHRP Ground Motion Selection

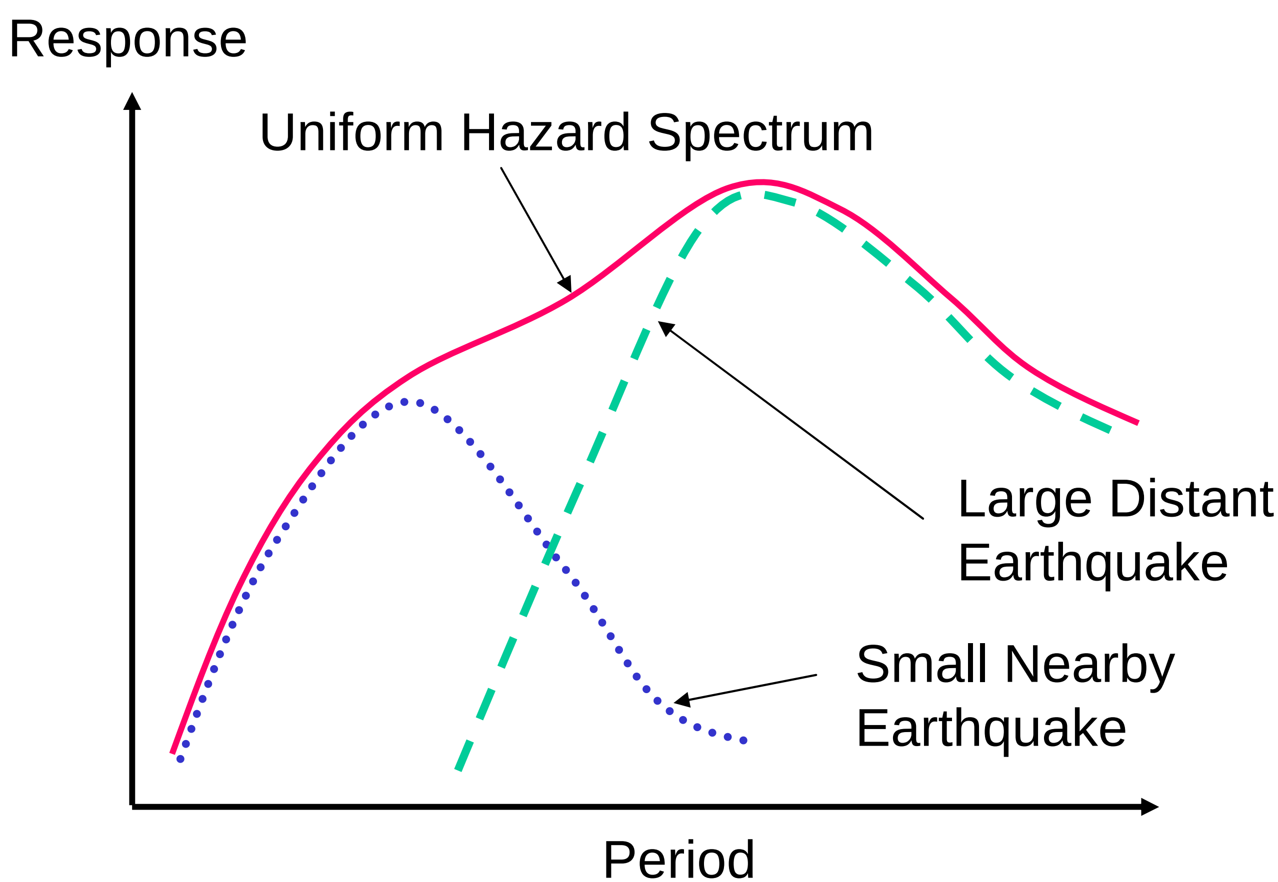
- Ground motions must have magnitude, fault mechanism, and fault distance consistent with the site and must be representative of the maximum considered ground motion
- Where the required number of motions are not available simulated motions (or modified motions) may be used

(Parenthesis by F. Charney)

How many records should be used?
Where does one get the records?
How can the records be modified to match site conditions?

Use of Simulated Ground Motions

Simulated records should **NOT** be used if they have been created on the basis of spectrum matching where the target spectrum is a uniform hazard spectrum.



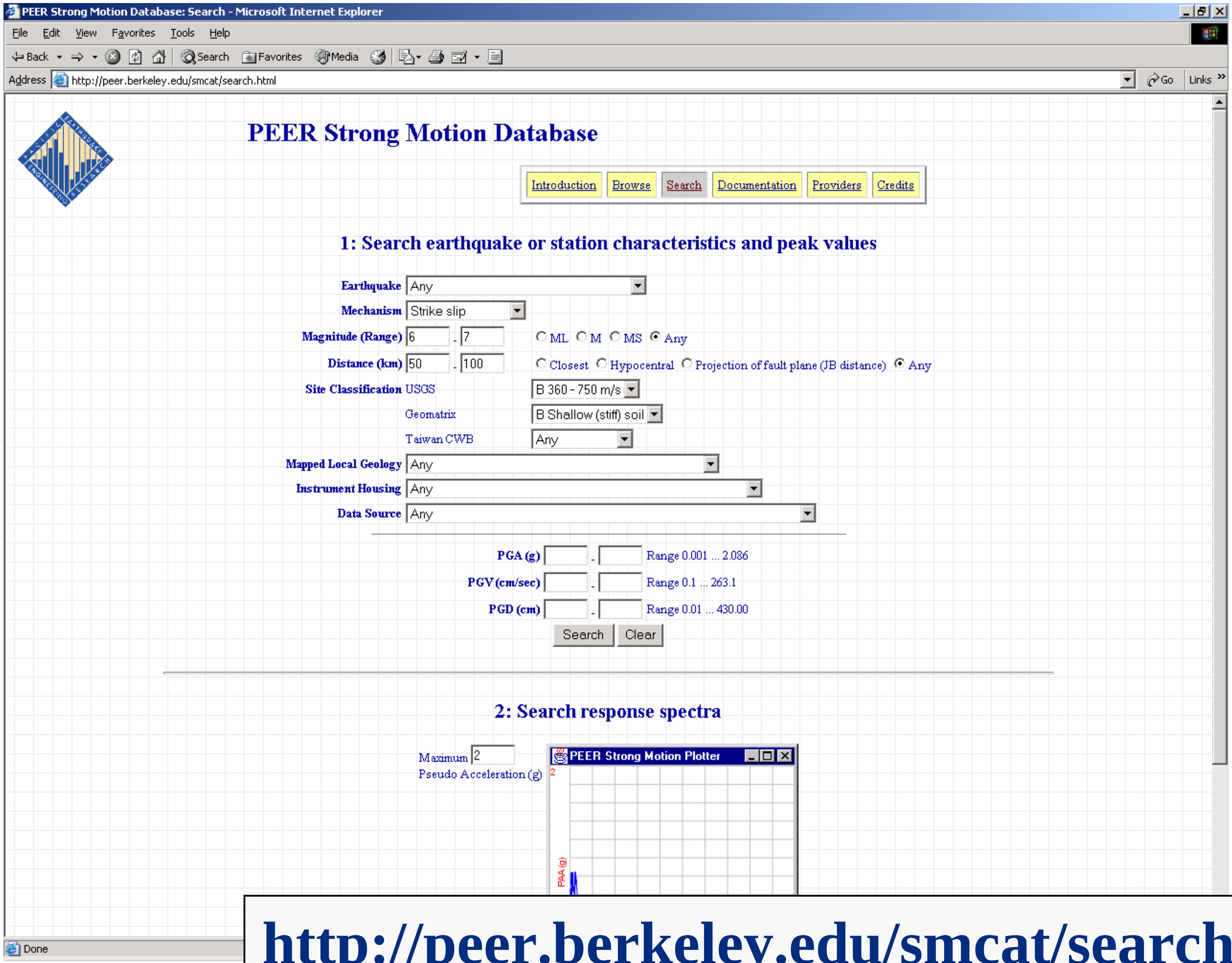
Use of Simulated Ground Motions

Reference:

"On the use of Design Spectrum Compatible Time Histories", by Farzad Naiem and Marshall Lew, Earthquake Spectra, Volume 11, No.1.

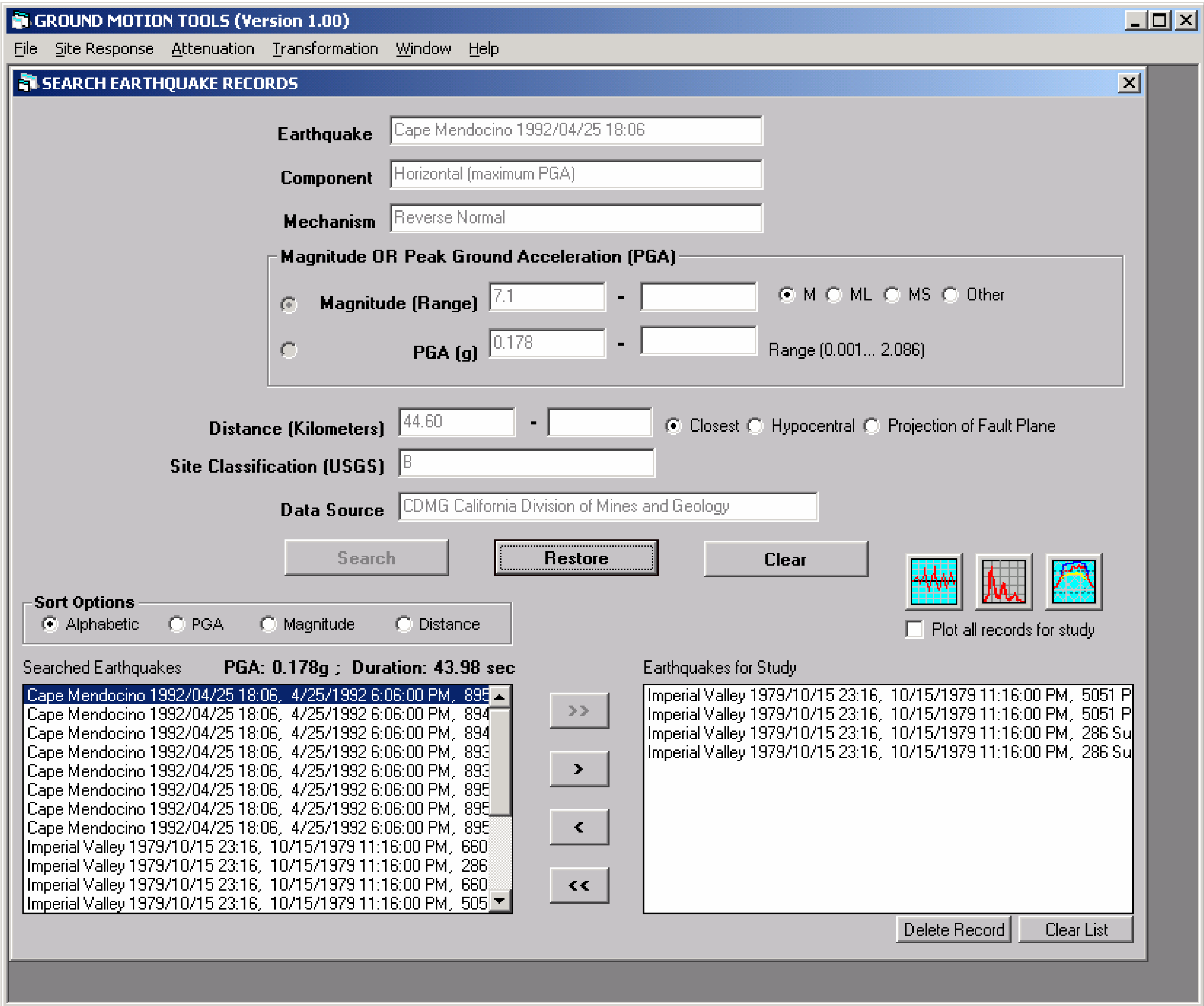
"Frequency domain scaled Design Spectrum Compatible Time Histories (DSCTH) are based on an erroneous understanding of the role of design spectra and can suffer from a multitude of major problems. They may represent velocities, displacements, and high energy content which are very unreliable. The authors urge extreme caution in the use of DSCTH in the design of earthquake resistant structures."

PEER Ground Motion Search Engine

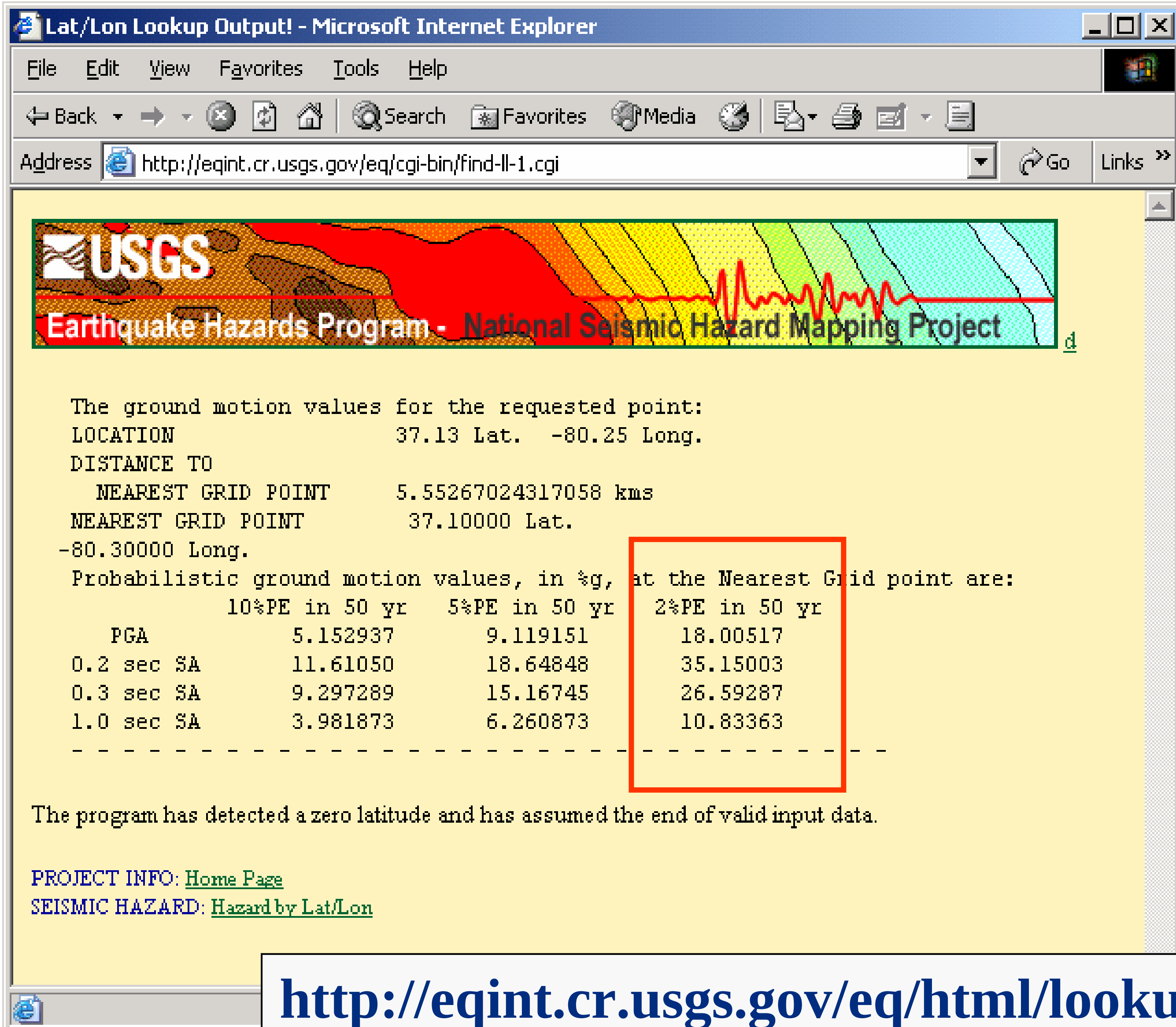


http://peer.berkeley.edu/smcat/search.html

NONLIN Ground Motion Tools (EQTOOLS)

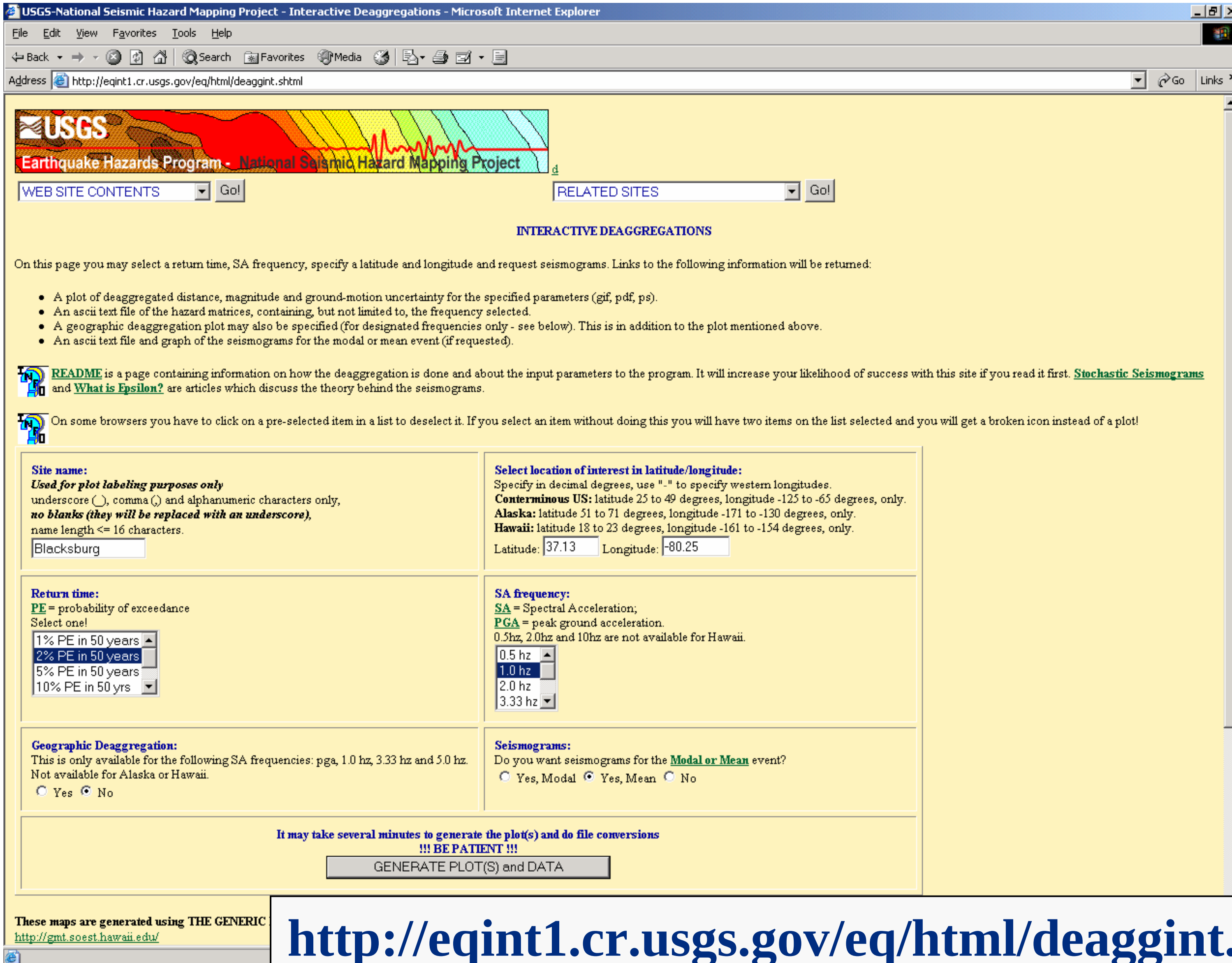


Uniform Hazard Spectrum Coordinates



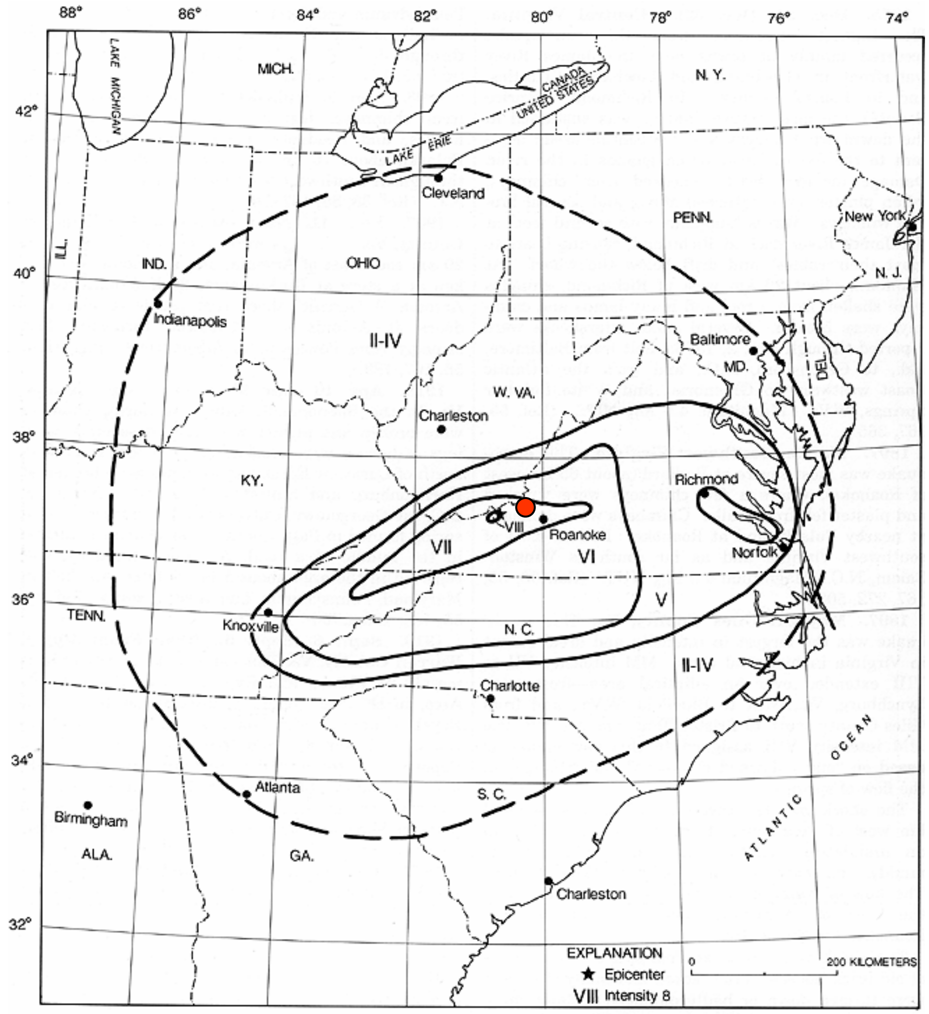
http://eqint.cr.usgs.gov/eq/html/lookup.shtml

Ground Motion Generator



http://eqint1.cr.usgs.gov/eq/html/deaggint.shtml

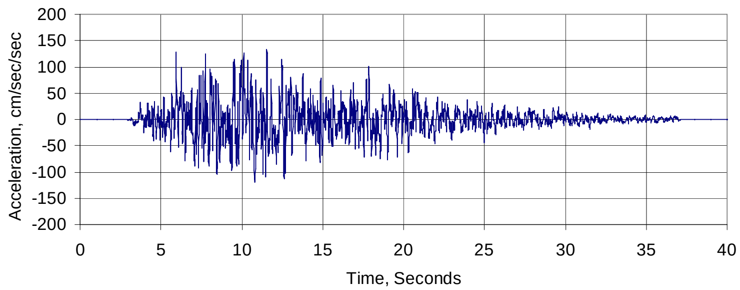
Isoseismal Map for the Giles County, Virginia, Earthquake of May 31, 1897.



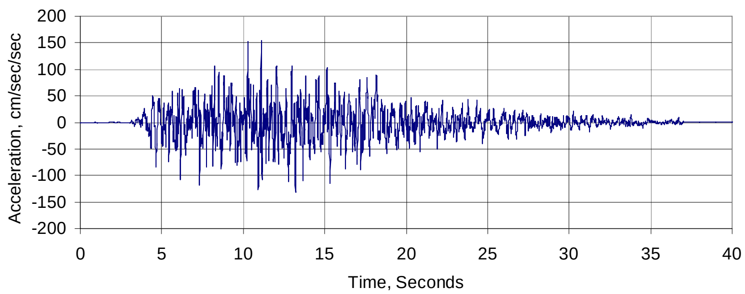
● Blacksburg
N 37.1
W -80.25

Blacksburg 2%-50 Ground Motions from USGS Web Site

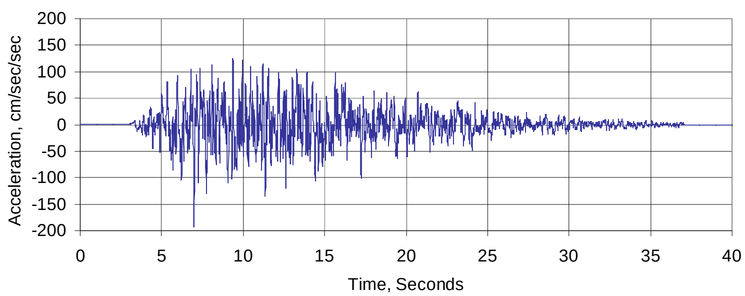
MOTION 1



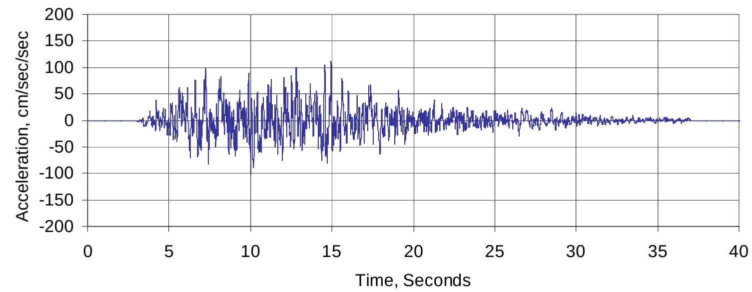
MOTION 2



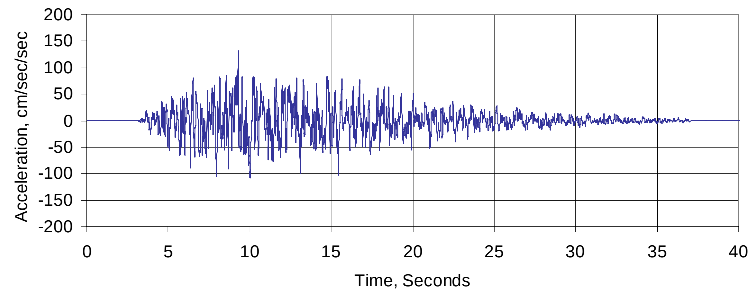
MOTION 3



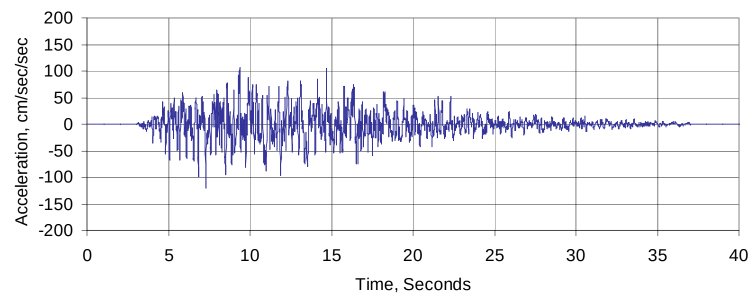
MOTION 4



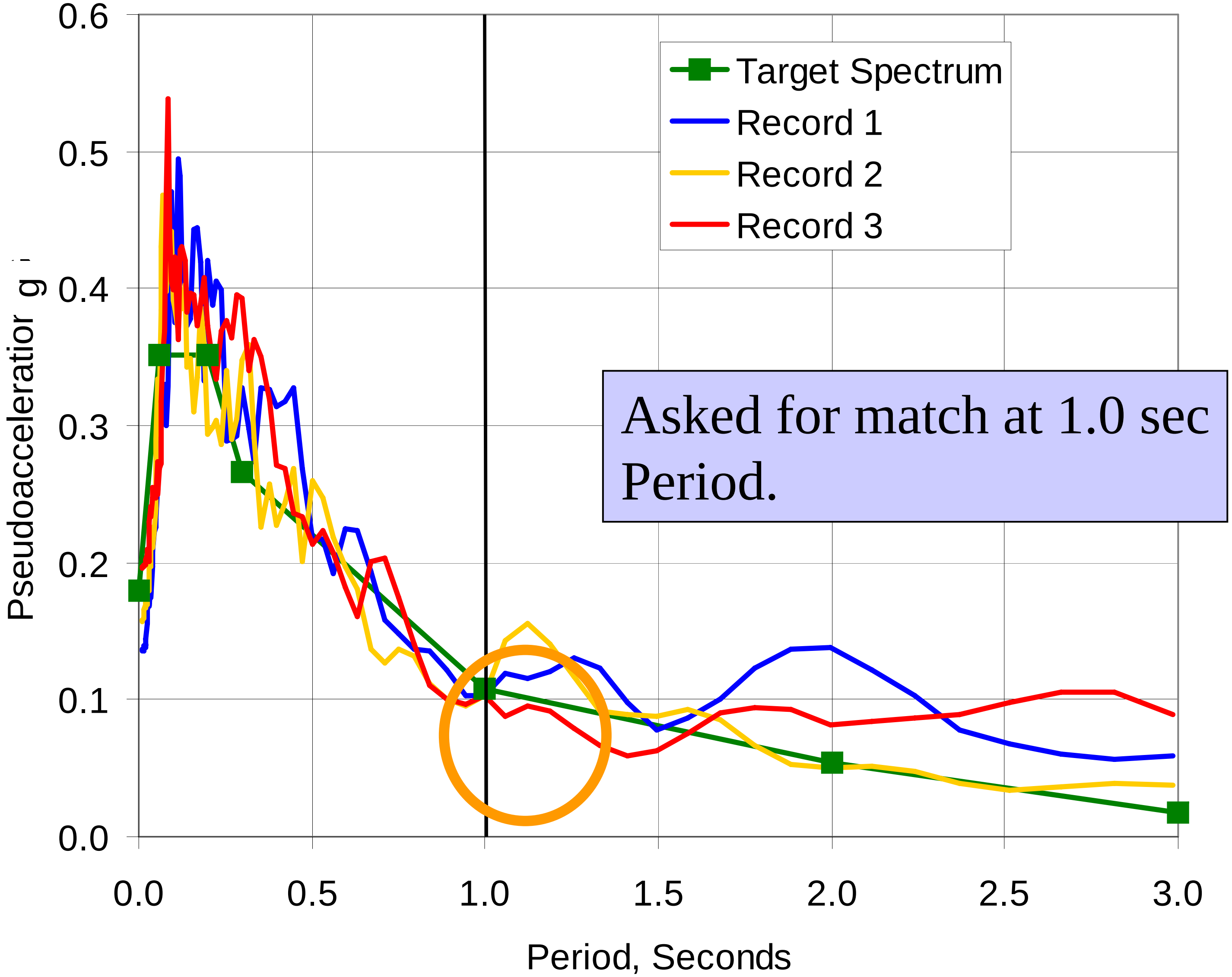
MOTION 5



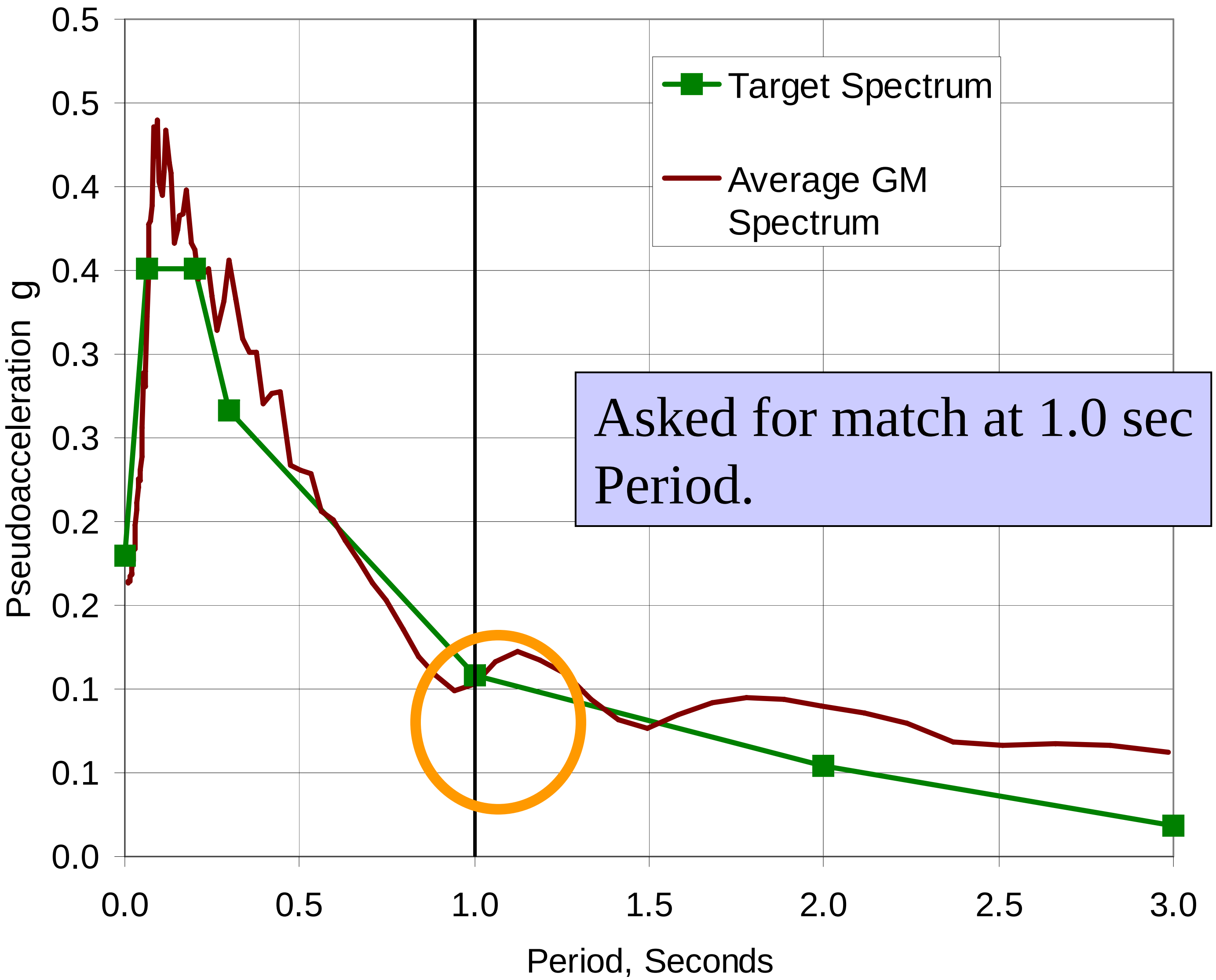
MOTION 6



USGS Ground Motion Spectra and Target Spectrum

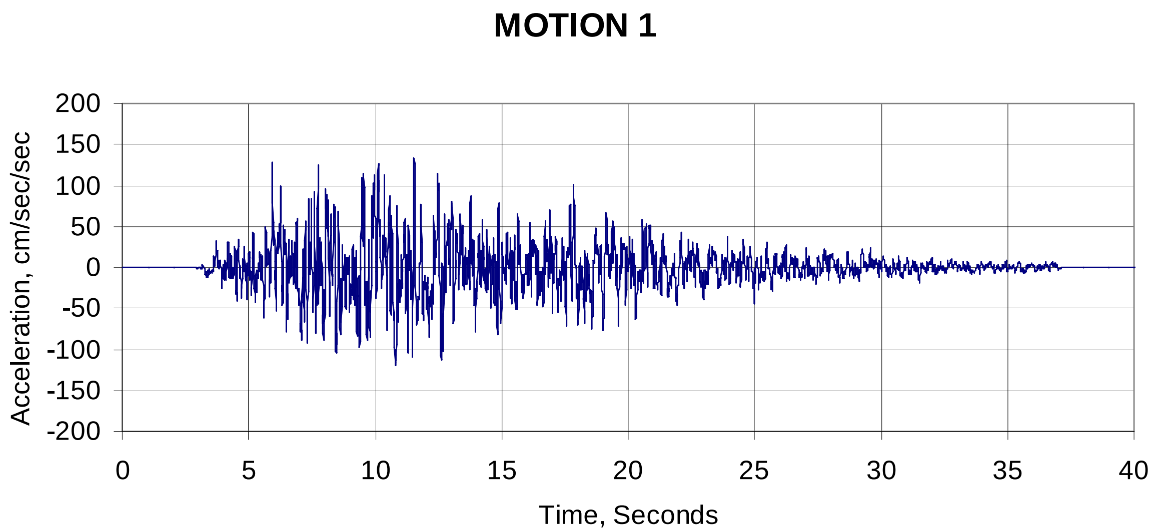


Average USGS Ground Motion Spectrum and Target Spectrum



Ground Modification Modifications

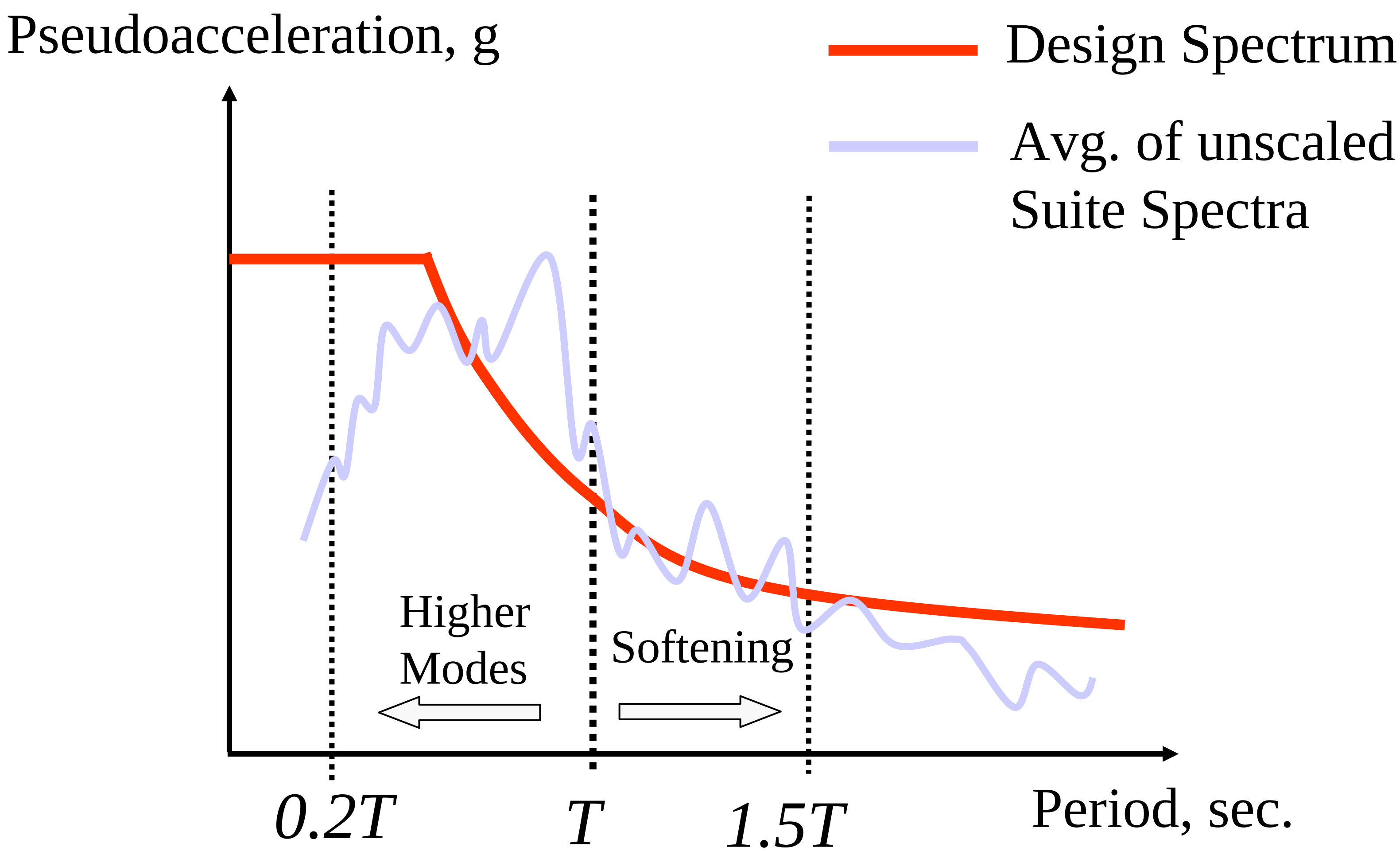
1. **Scale a given record to a higher or lower acceleration (e.g to produce a record that represents a certain hazard level)**
2. Modify a record for distance
3. Modify a record for site classification (usually from hard rock to softer soil)
4. Modify a record for fault orientation



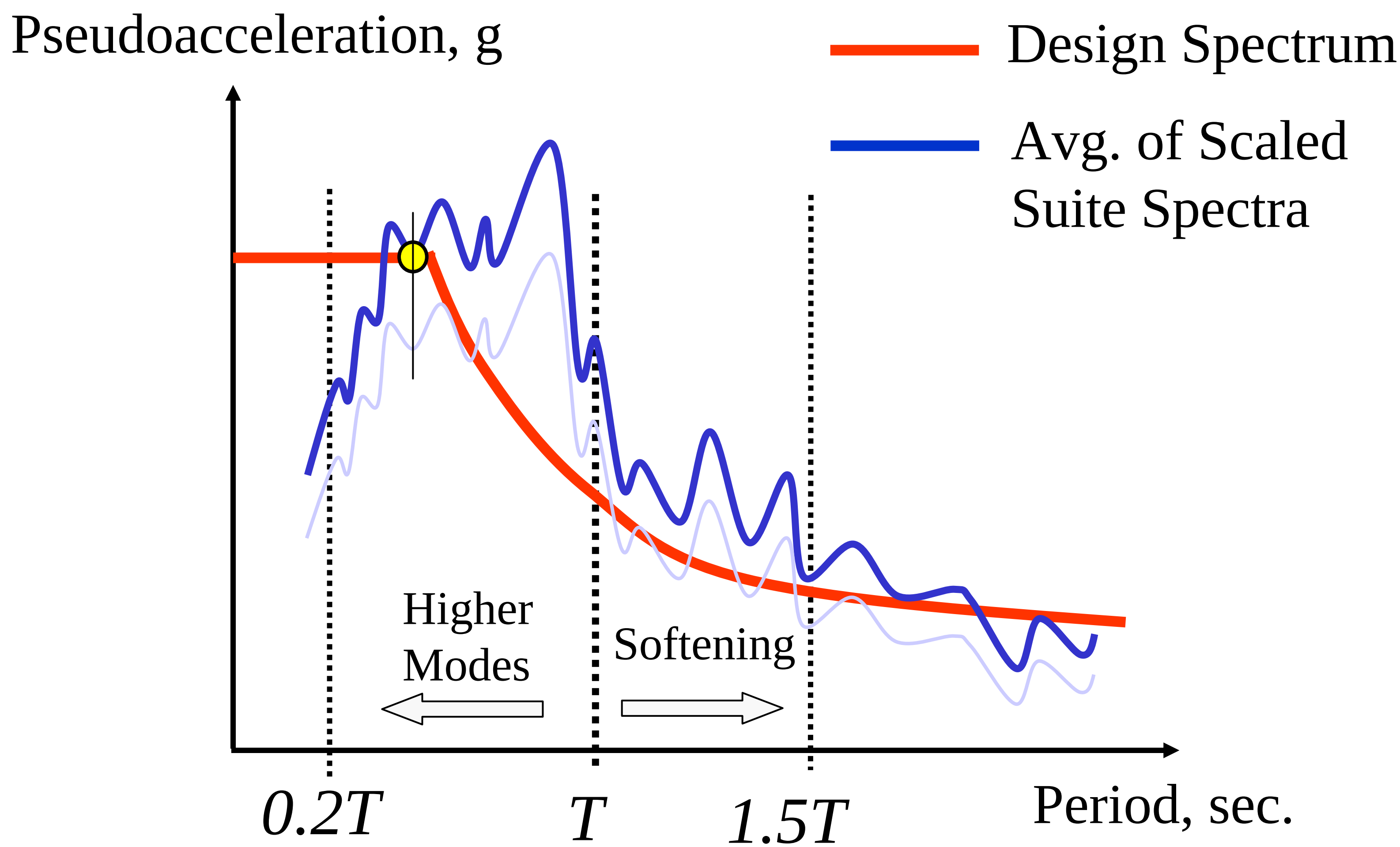
NEHRP Ground Motion Scaling
(2-D Analysis)

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

NEHRP Scaling for 2-D Analysis



NEHRP Scaling for 2-D Analysis



NEHRP Ground Motion Selection and Scaling (3-D Analysis)

1. The Square Root of the Sum of the Squares of the 5% damped spectra of each motion pair (N-S and E-W components) is constructed.

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

Potential Problems with NEHRP Scaling

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

How Many Records to Use?

NEHRP Recommended Provisions:

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

5.6.3 If at least seven ground motions are used evaluation may be based on the average responses from the different analyses. If less than seven motions are used the evaluation must be based on the maximum value obtained from all analyses.

Normalization and Scaling Accelerograms For Nonlinear Analysis

Nilesh Shome and Allin Cornell
6th U.S. Conference on Earthquake Engineering
Seattle, Washington, September, 1997

Ground Motion Scaling for Nonlinear Analysis

(Shome and Cornell)

Bin:

A suite of ground motions with similar source, distance, and magnitude.

Bin Normalization:

Adjusting individual bin records to the same "intensity"

Bin Scaling:

Adjusting records from one bin (say a lower magnitude) to the intensity of the records from a different (usually higher) intensity bin.

Normalization Procedures

(Shome and Cornell)

- Normalize to PGA (NOT RECOMMENDED)
- Normalize to a Single Frequency at low damping (e.g. 2%)
- Normalize to a Single Frequency at a higher damping (e.g 5% to 20%) (RECOMMENDED)
- Normalize over a Range of Frequencies

How Many Records to Use?

(Shome and Cornell)

For records normalized to first mode spectral acceleration it may typically require about **4 to 6 records** to obtain about a one sigma (plus or minus 10 to 15 percent) confidence band.

Can records from a low intensity bin be scaled to represent higher intensity earthquakes?

(Shome and Cornell)

When the records are scaled from one intensity level to a higher intensity there is a mild dependency of scaling on computed ductility demand. The median ductility demand may vary 10 to 20 percent for one unit change in magnitude. *The effect of scaling on nonlinear hysteretic energy demand is more significant.*

Recommendations (Charney):

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

Ground Modification Modifications

1. **Scale a given record to a higher or lower acceleration (e.g to produce a record that represents a certain hazard level)**
2. Modify a record for distance (SRL Attenuation Issue)
3. Modify a record for site classification, usually from hard rock to softer soil. (WAVES by Hart and Wilson)
4. Modify a record for fault orientation (Somerville, et al)

See Also: *Ground Motion Evaluation Procedures for Performance Based Design*, by J.P. Stewart, et al, PEER Report 2001/09

Damage Prediction

Performance based design requires a quantification of the damage that might be incurred in a structure.

The "damage index" must be calibrated such that it may predict and quantify damage at all performance levels.

While inter-story drift and inelastic component deformation may be useful measures of damage, a key characteristic of response is missing… the effect of the duration of ground motion on damage.

A number of different damage measures have been proposed which are dependent on duration.

Damage Prediction

Park and Ang (1985)

$$DI_{PA} = \frac{u_{max}}{u_{cap}} + \lambda \frac{E_H}{u_{cap} F_y}$$

u_{max} = maximum attained deformation

u_{cap} = monotonic deformation capacity

E_H = hysteretic energy dissipated

F_y = monotonic yield strength

λ = calibration factor

See Reference List for Additional Info on Damage Measures

Energy Balance

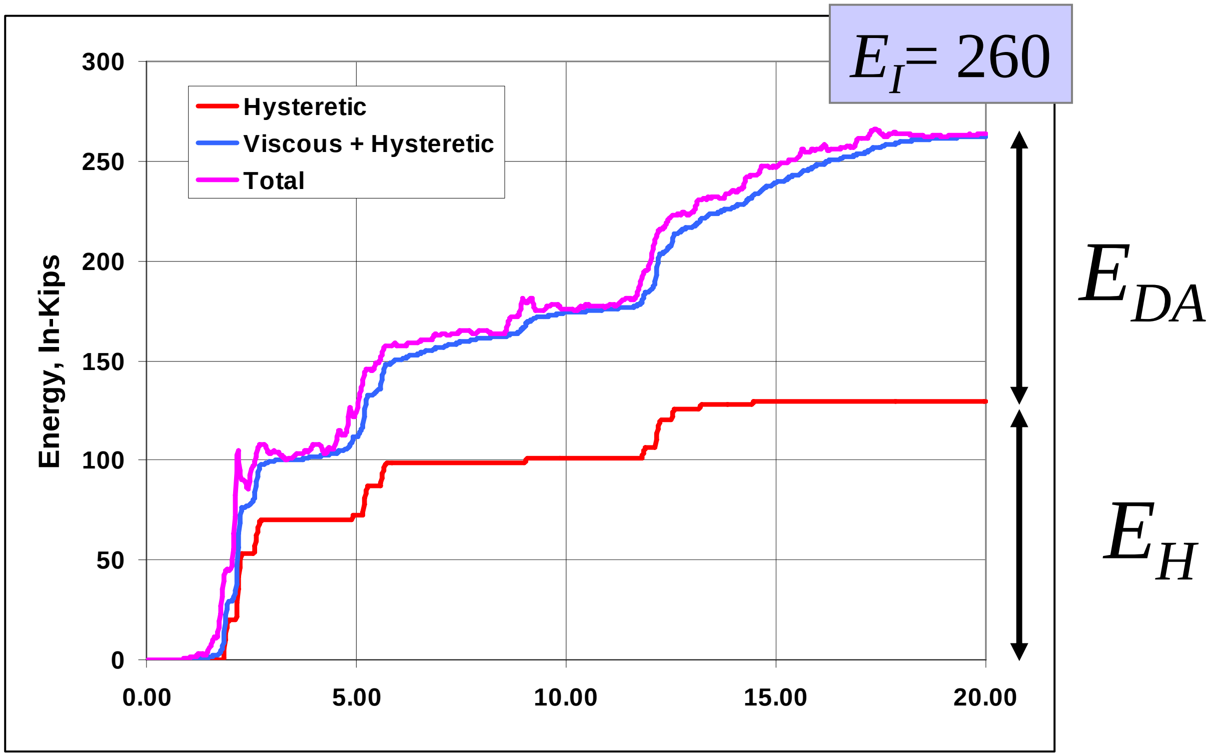
Hysteretic Energy

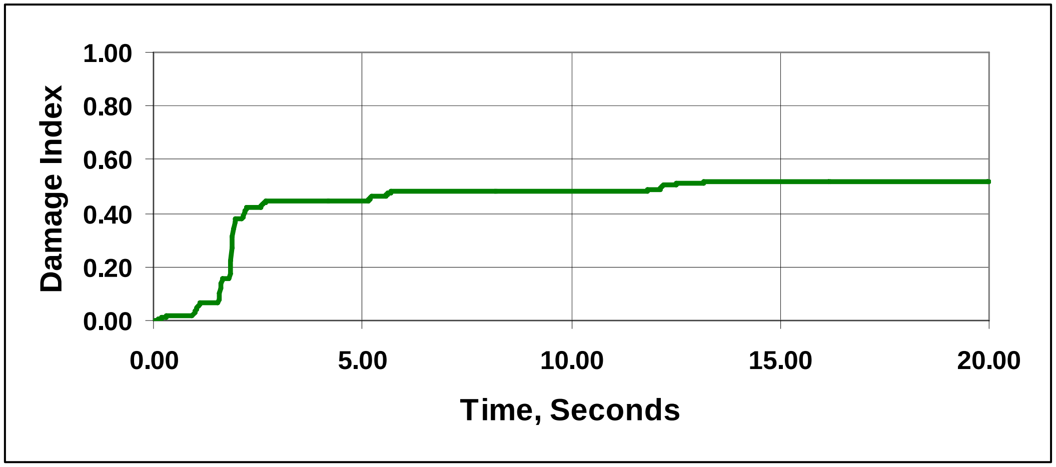
$$E_I = E_S + E_K + (E_{DI} + E_{DA}) + E_H$$

Inherent Damping

Added Damping

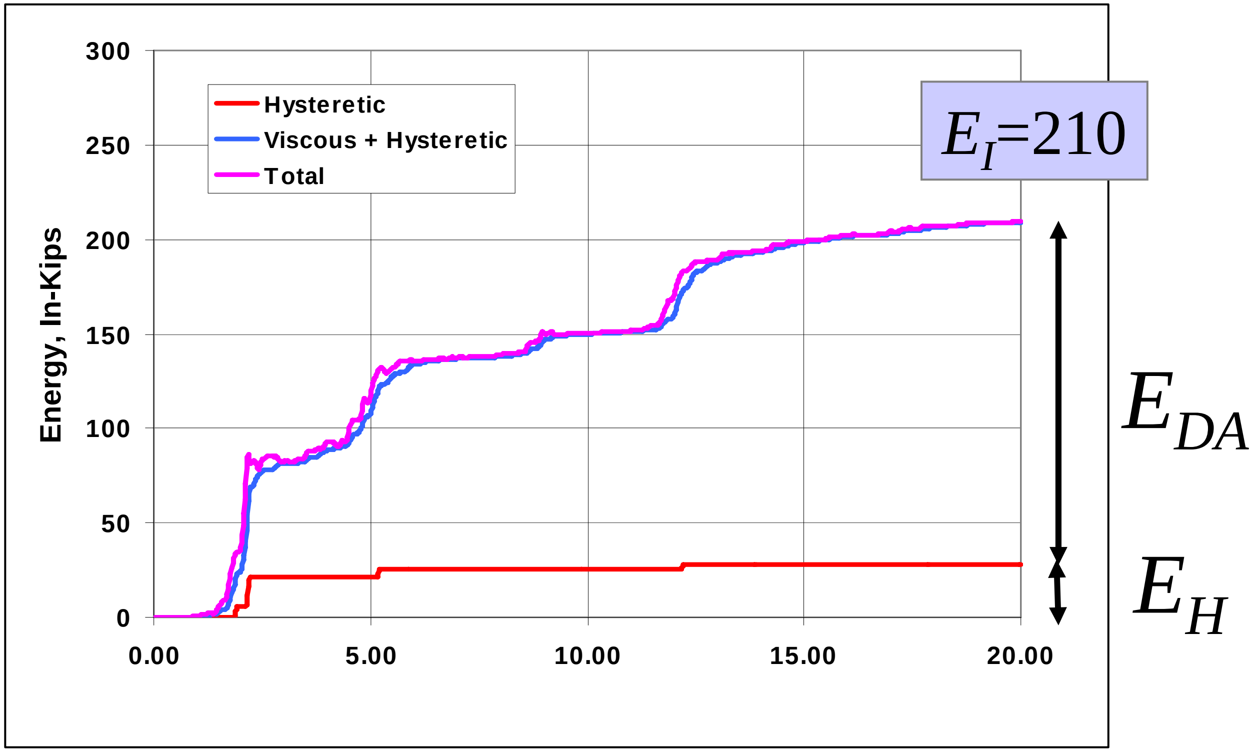
Energy and Damage Histories, 5% Damping

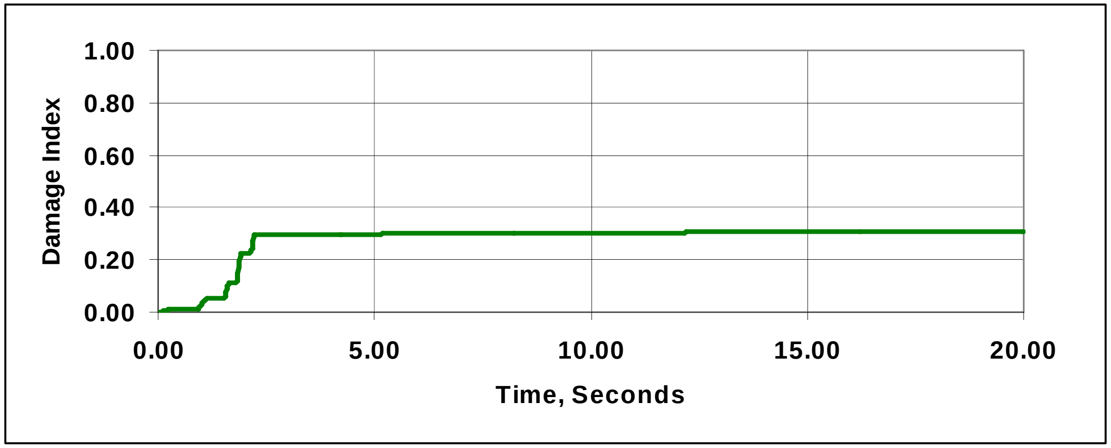




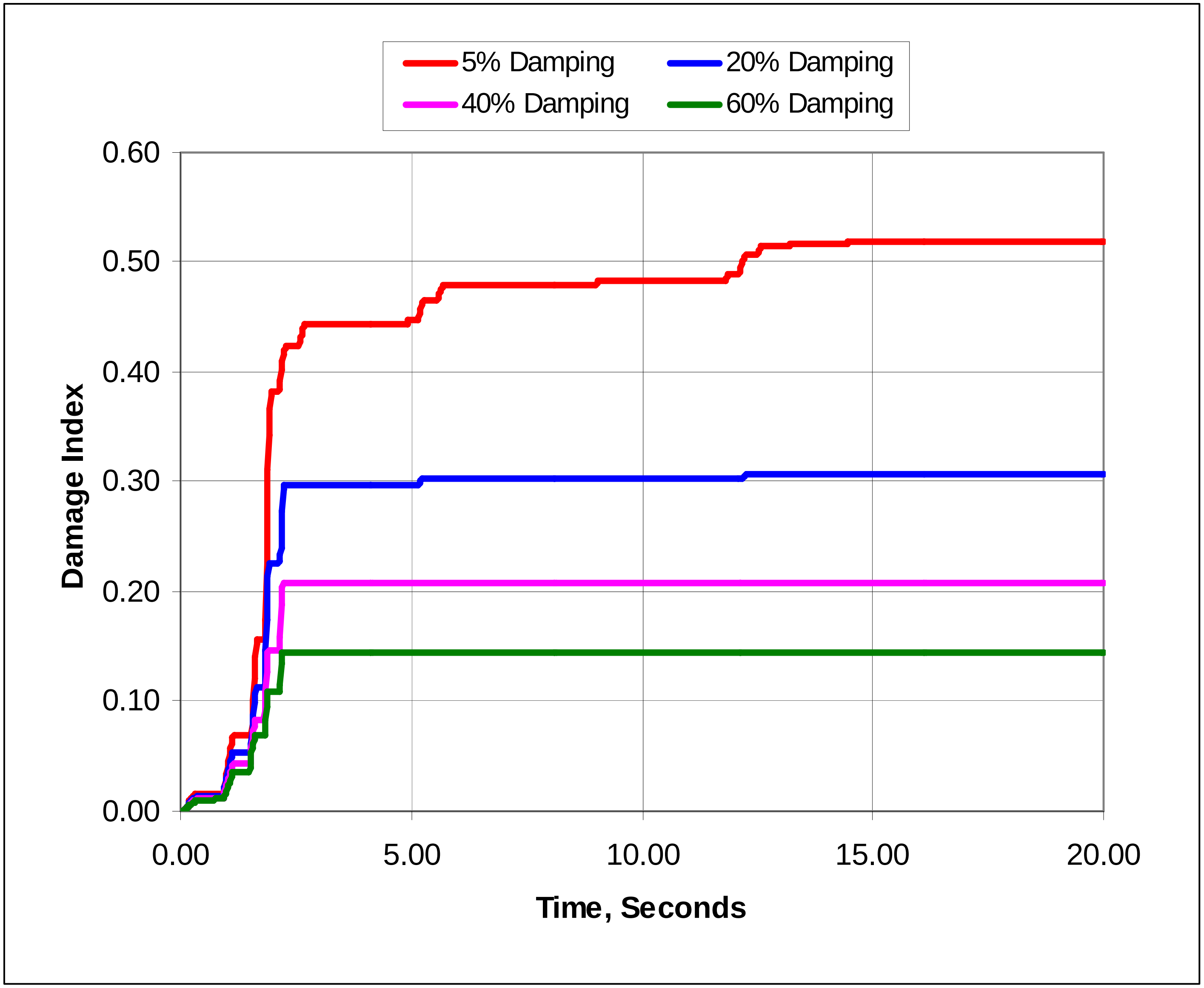
Analysis performed on NONLIN

Energy and Damage Histories, 20% Damping





Reduction in Damage with Increased Damping

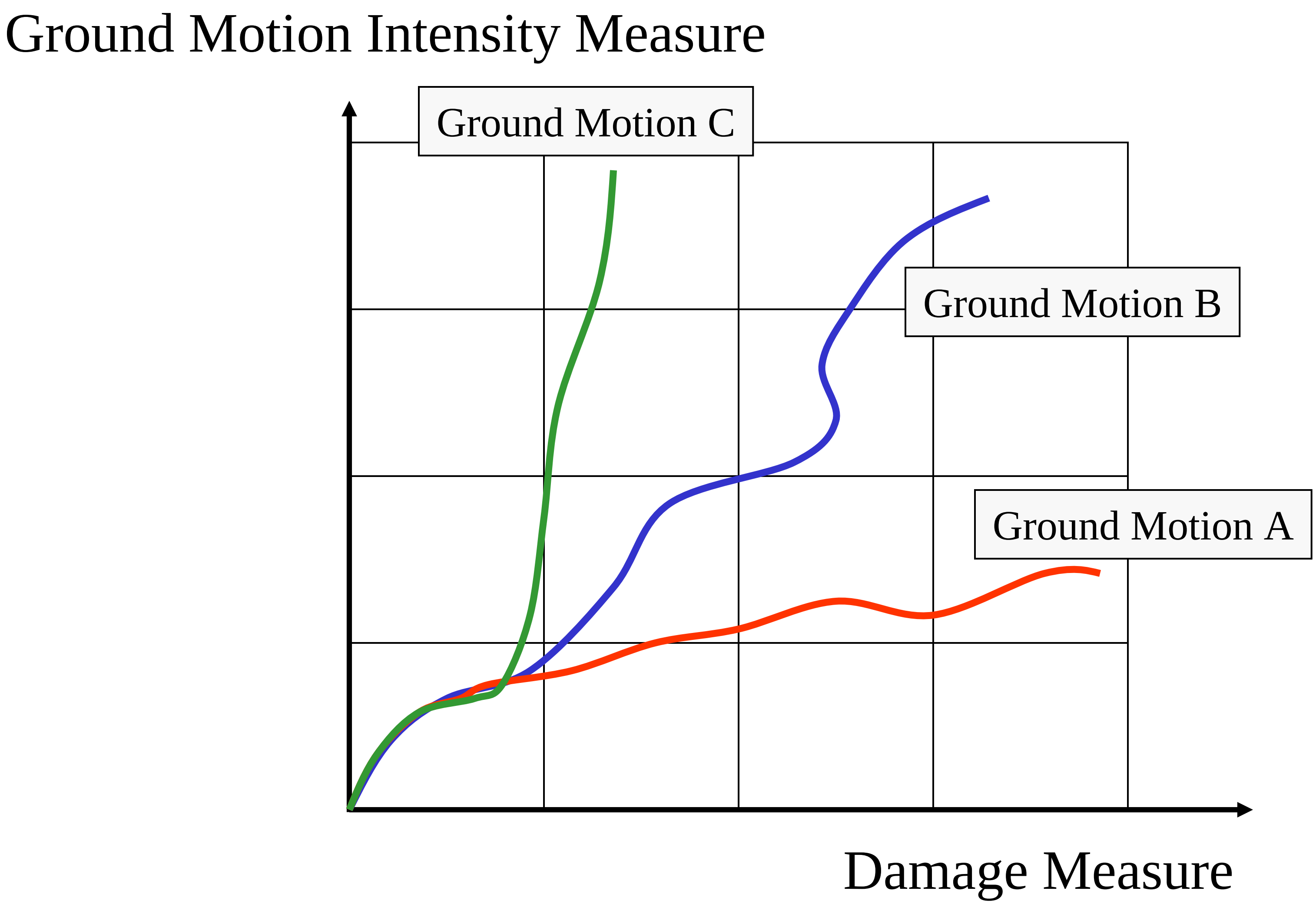


Incremental Nonlinear Dynamic Analysis

Seismic Performance, Capacity, and Reliability of Structures as Seen Through Incremental Dynamic Analysis

Ph.D. Dissertation of Dimitros Vamvatsikos,
Department of Civil and Environmental Engineering
Stanford University
July 2002.

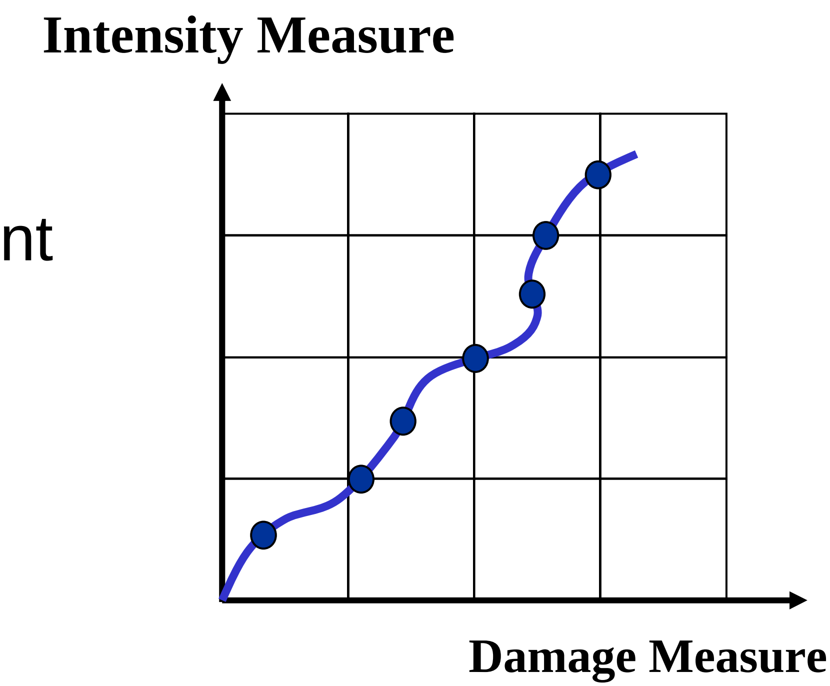
Incremental Nonlinear Dynamic Analysis



Incremental Nonlinear Dynamic Analysis

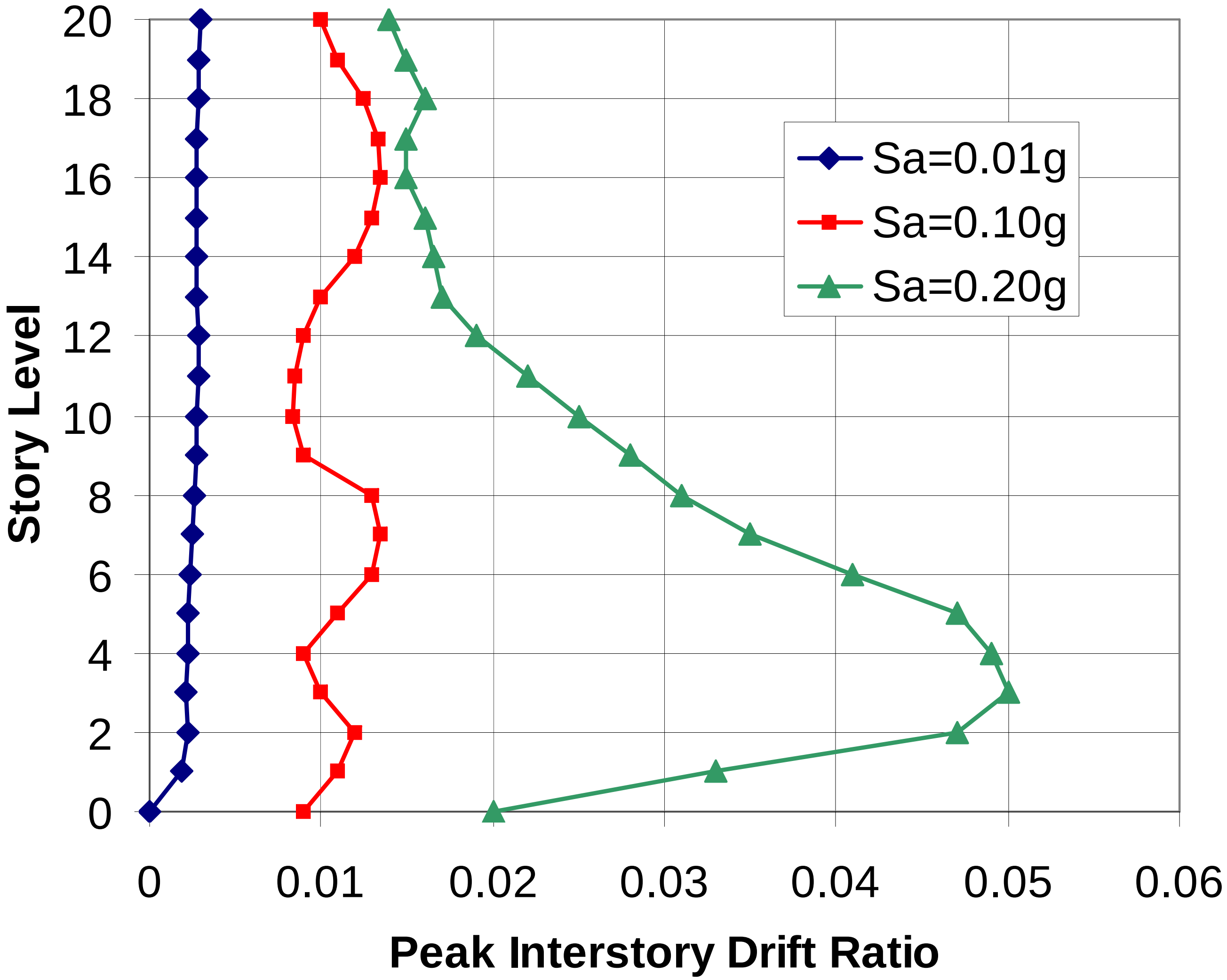
An *IDA study* is produced by subjecting a single structure to a series of time history analyses, where each subsequent analysis uses a higher ground motion intensity.

An *IDA Curve* is a plot of a damage measure (DM) versus the ground motion intensity (IM) at which it occurred.

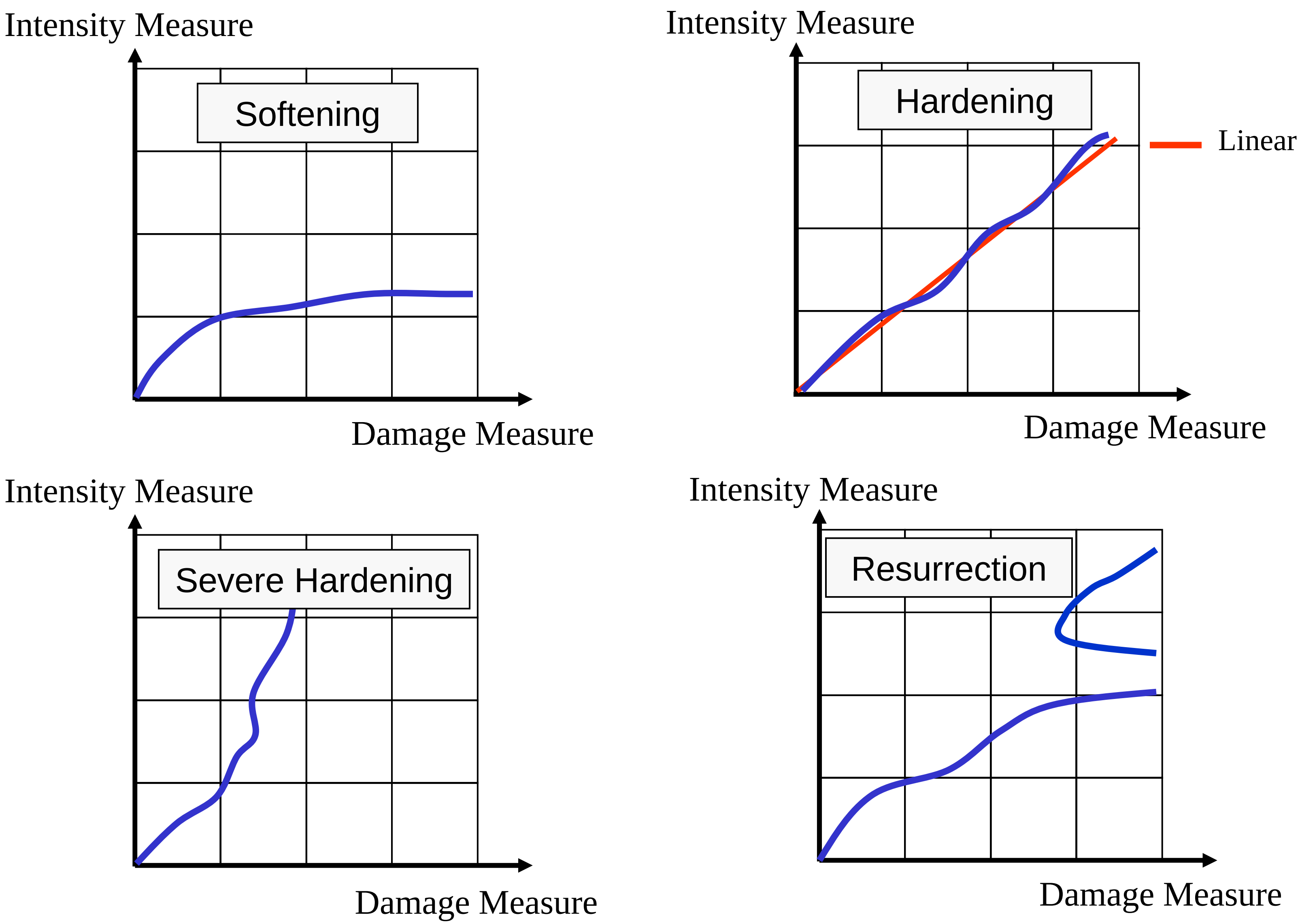


IDA Results for a Particular Ground Motion

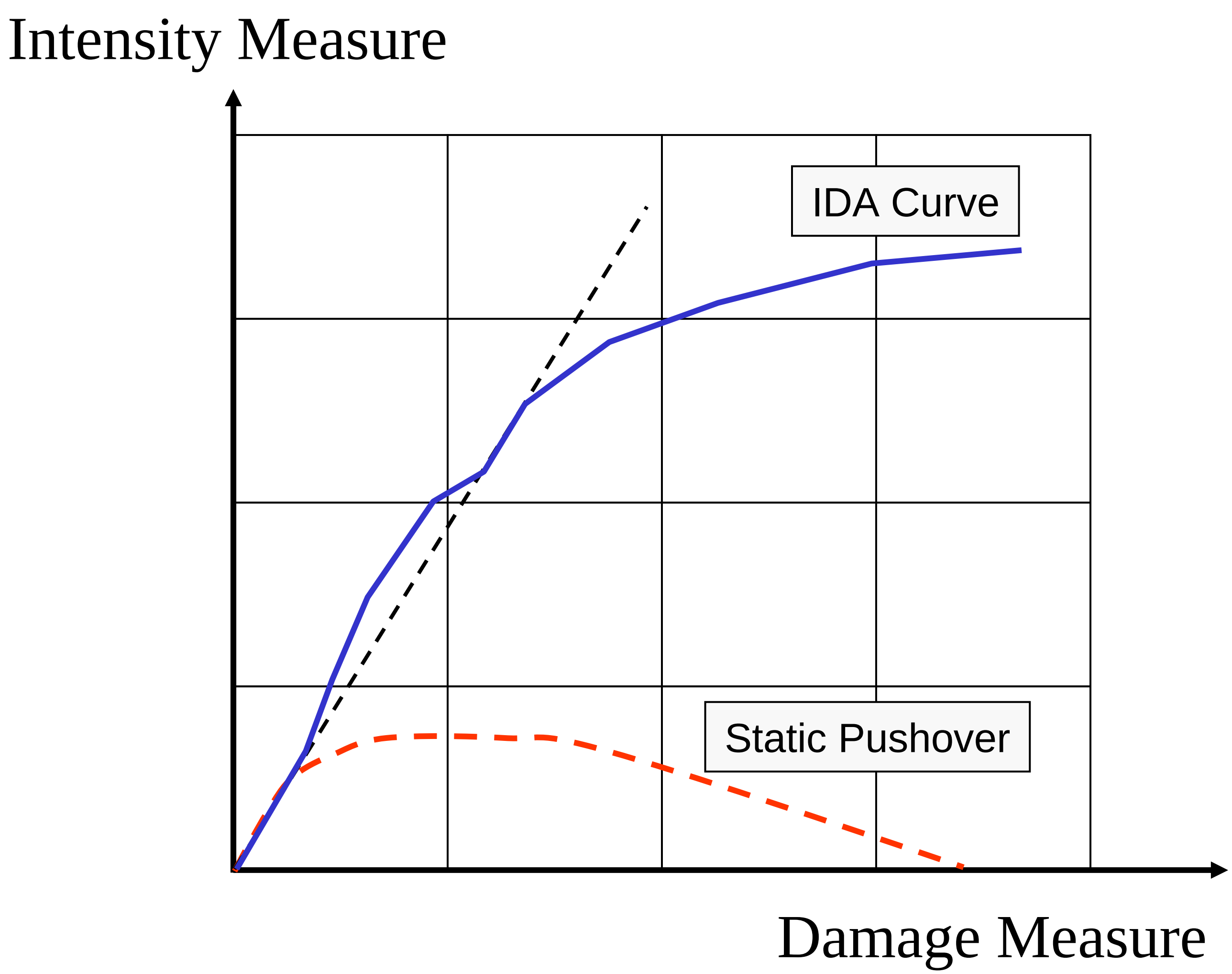
(after Vamvatsikos and Cornell)



Typical IDA Curve Characteristics



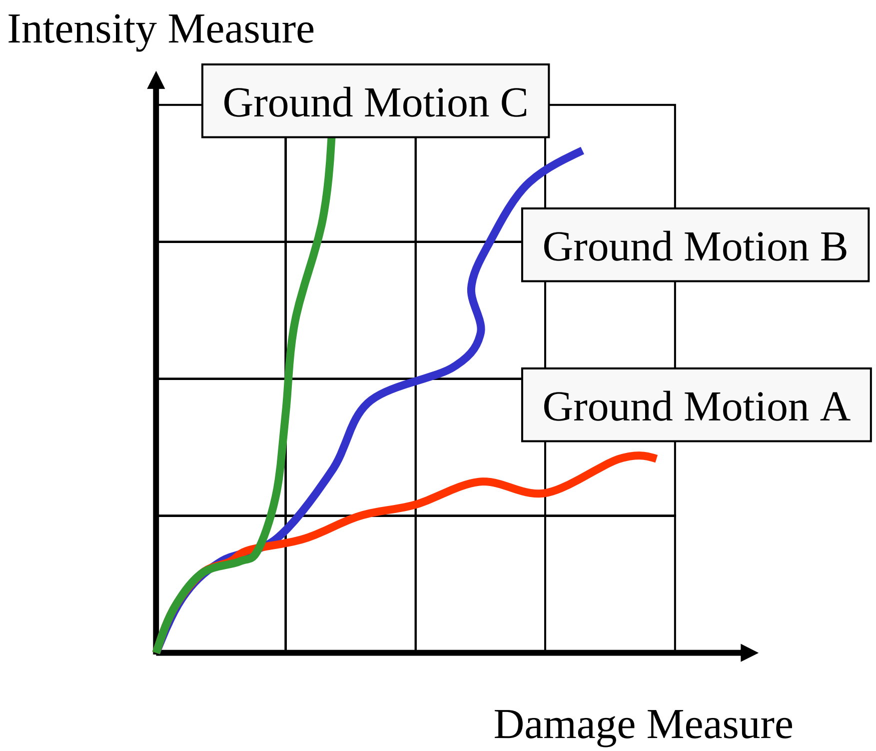
Typical IDA Curve Characteristics



Incremental Nonlinear Dynamic Analysis
(using Multiple Ground Motions)

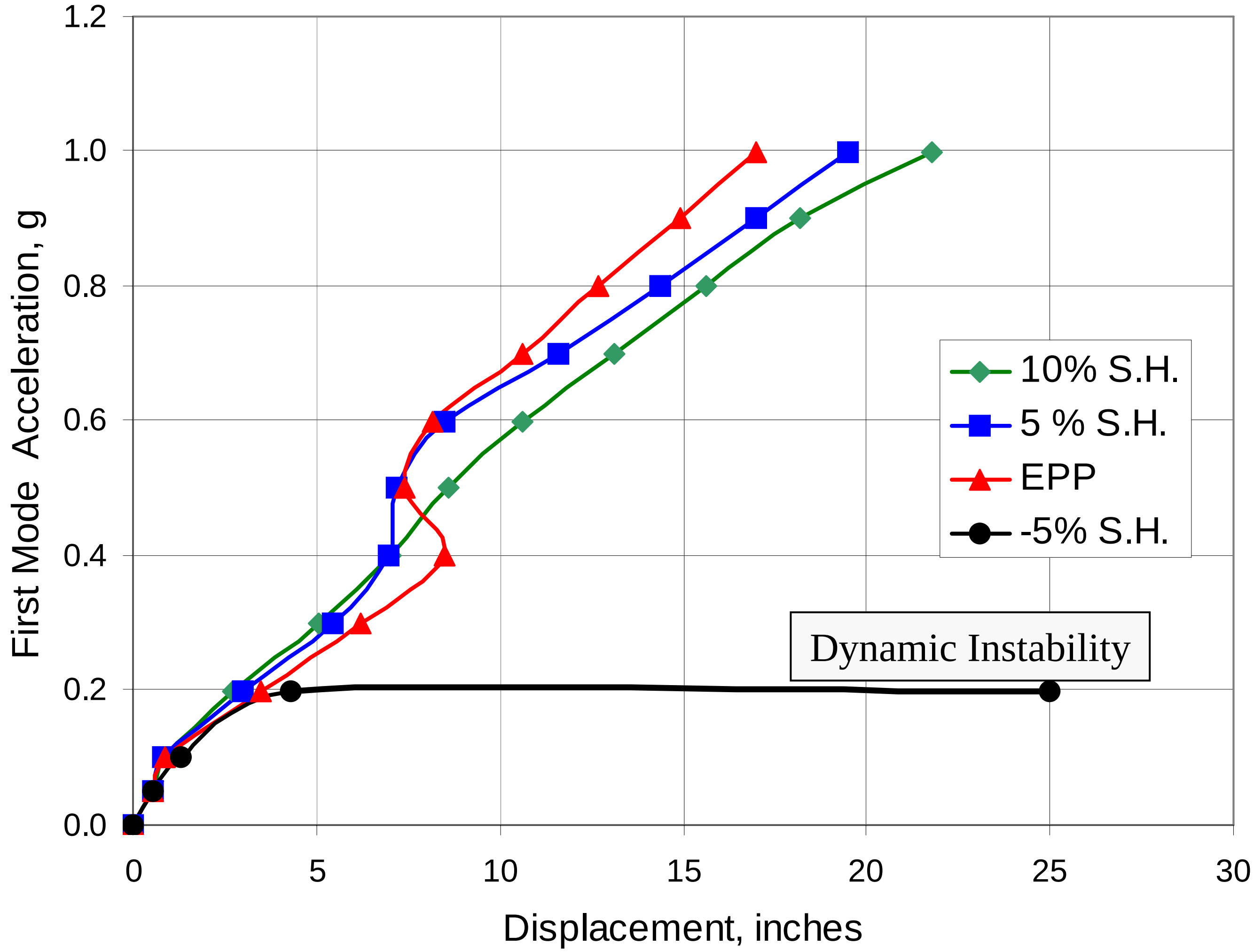
Usually, a study compares the response of the structure to a suite of ground motions.

An IDA study may also be used to assess the effect of a design change (or uncertainty) on the response of a structure to a particular ground motion.



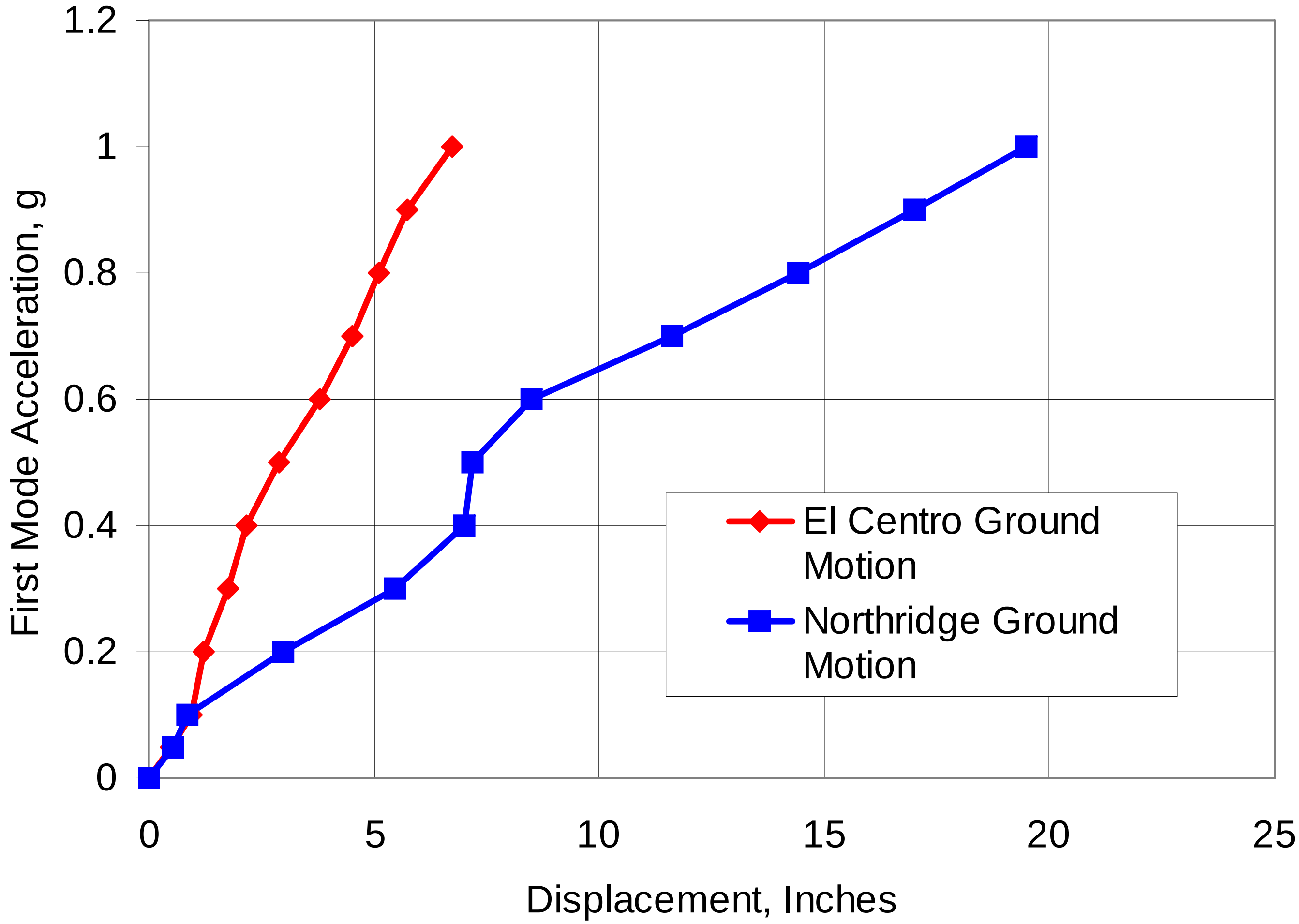
IDA Curves to Investigate Sensitivity of SDOF System Response to Strain Hardening Ratio

Analyzed on NONLIN Using Northridge (Slymar) Ground Motion.

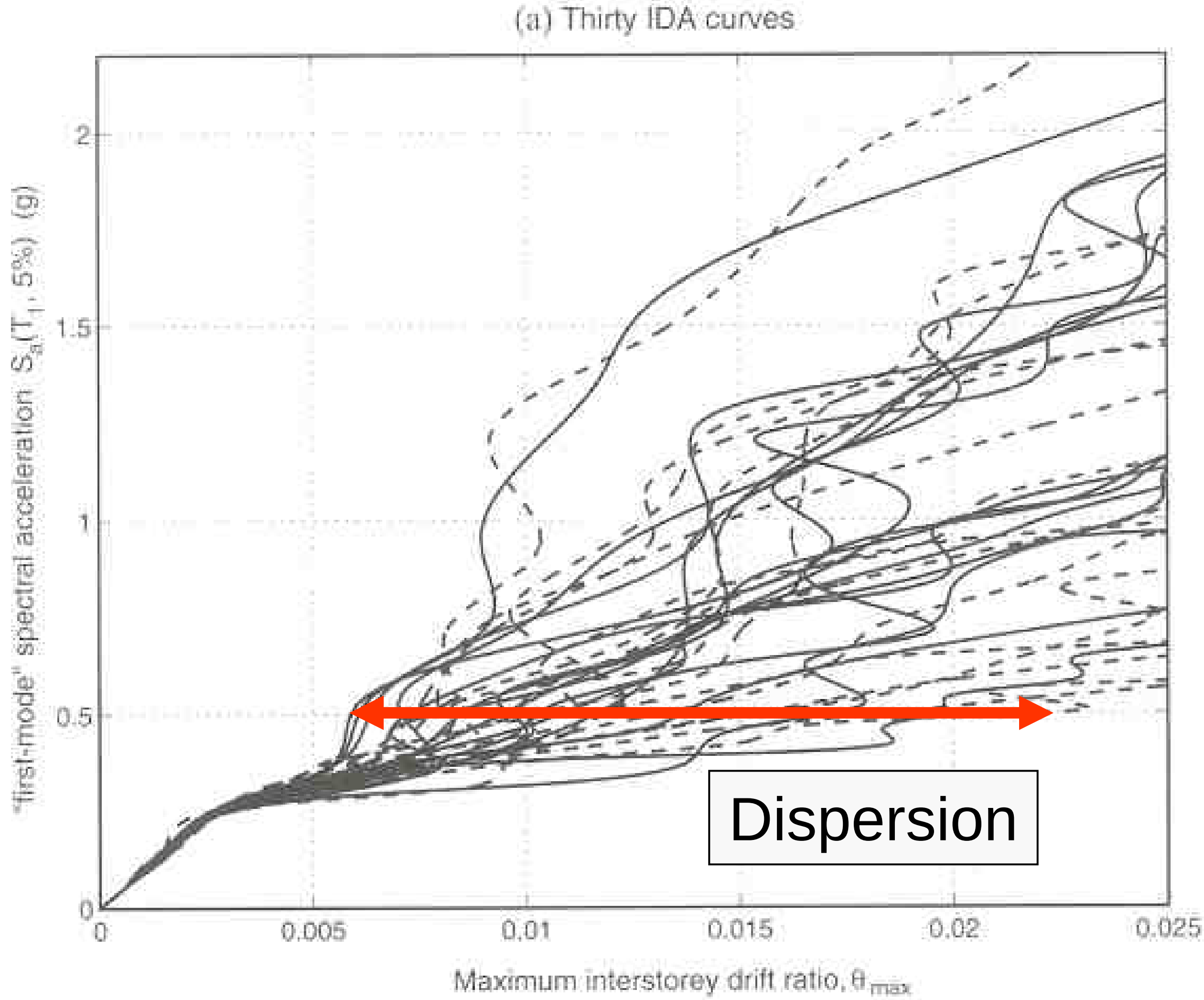


IDA Curves to Investigate Sensitivity of SDOF System Response to Choice of Ground Motion

2% Damping, 5% Strain Hardening

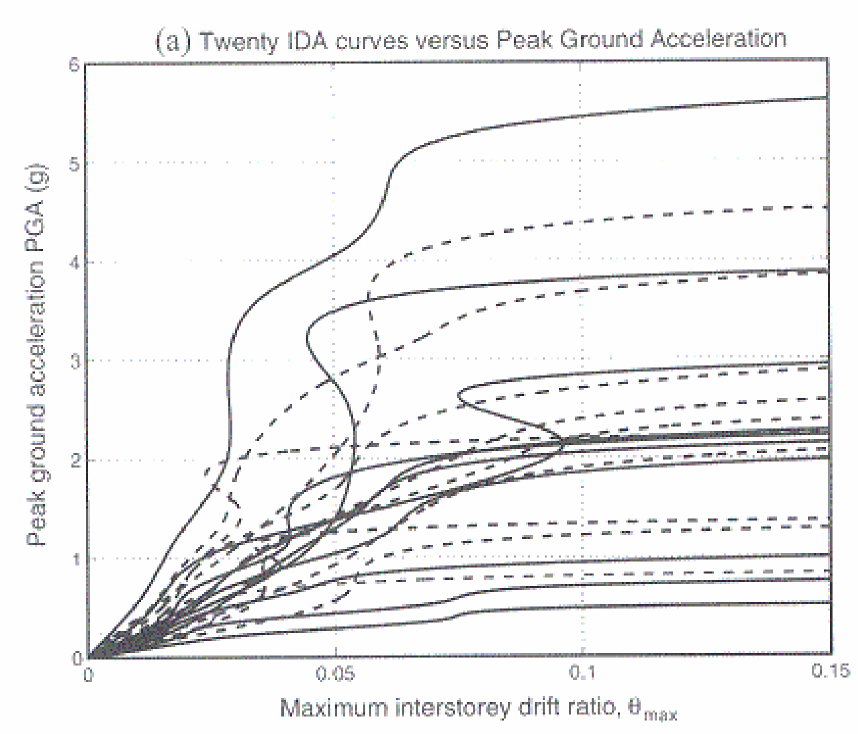


A Family of IDA Curves of the Same Building Subjected to Thirty Earthquakes

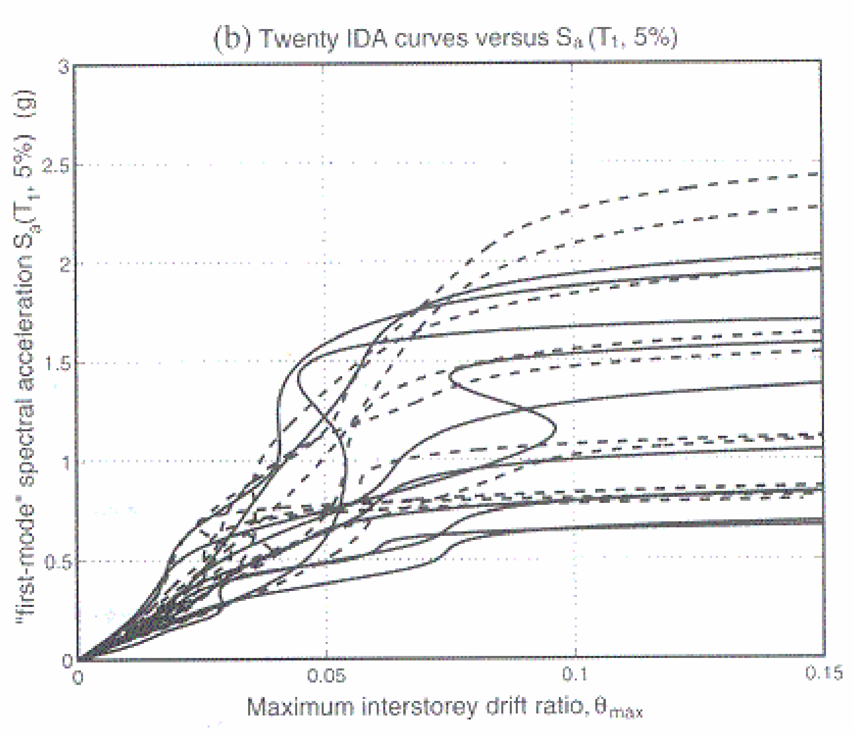


IDA Curves of the Same Building Subjected to Suite of Earthquakes

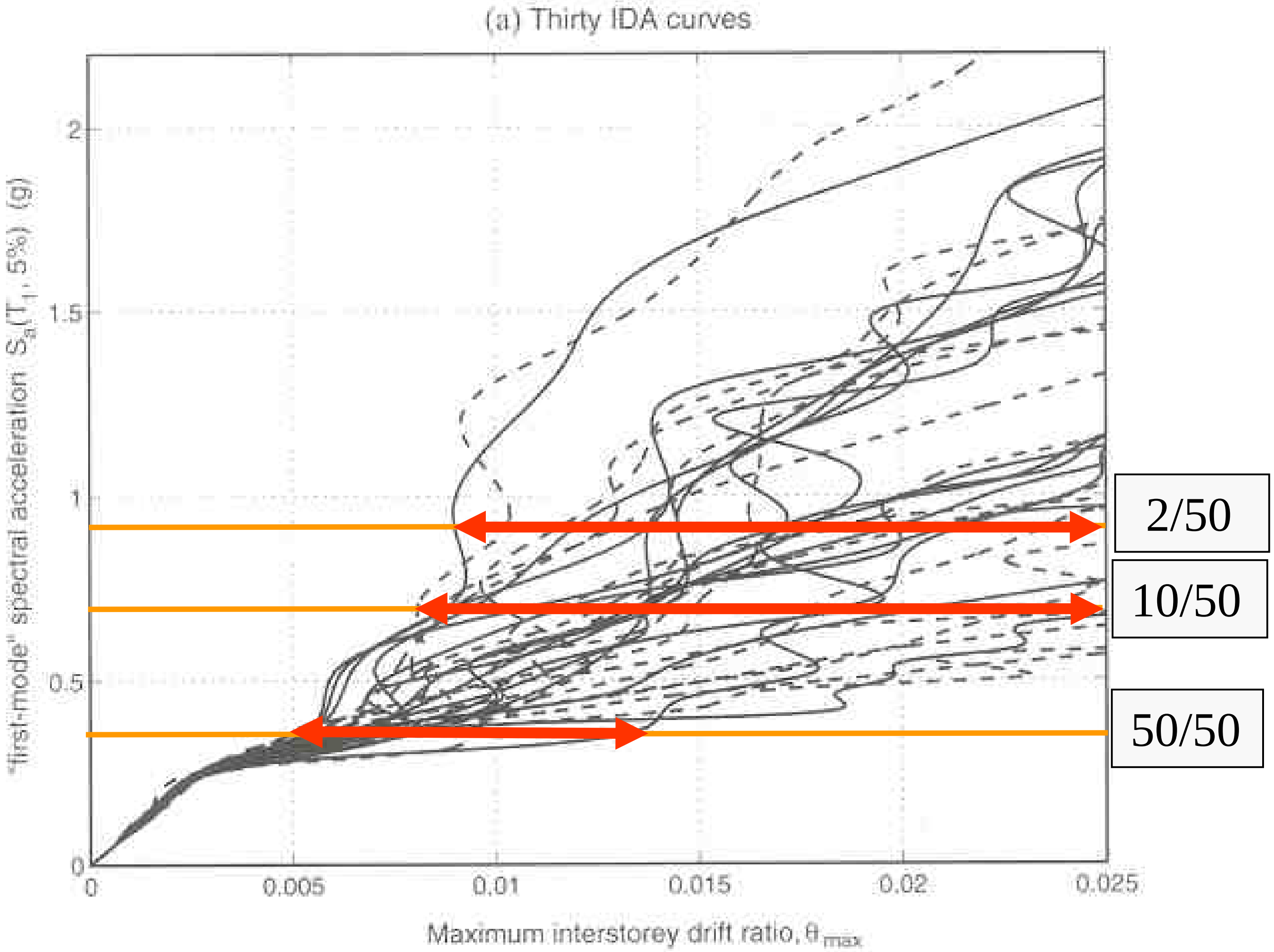
NORMALIZED to PGA



NORMALIZED to SA



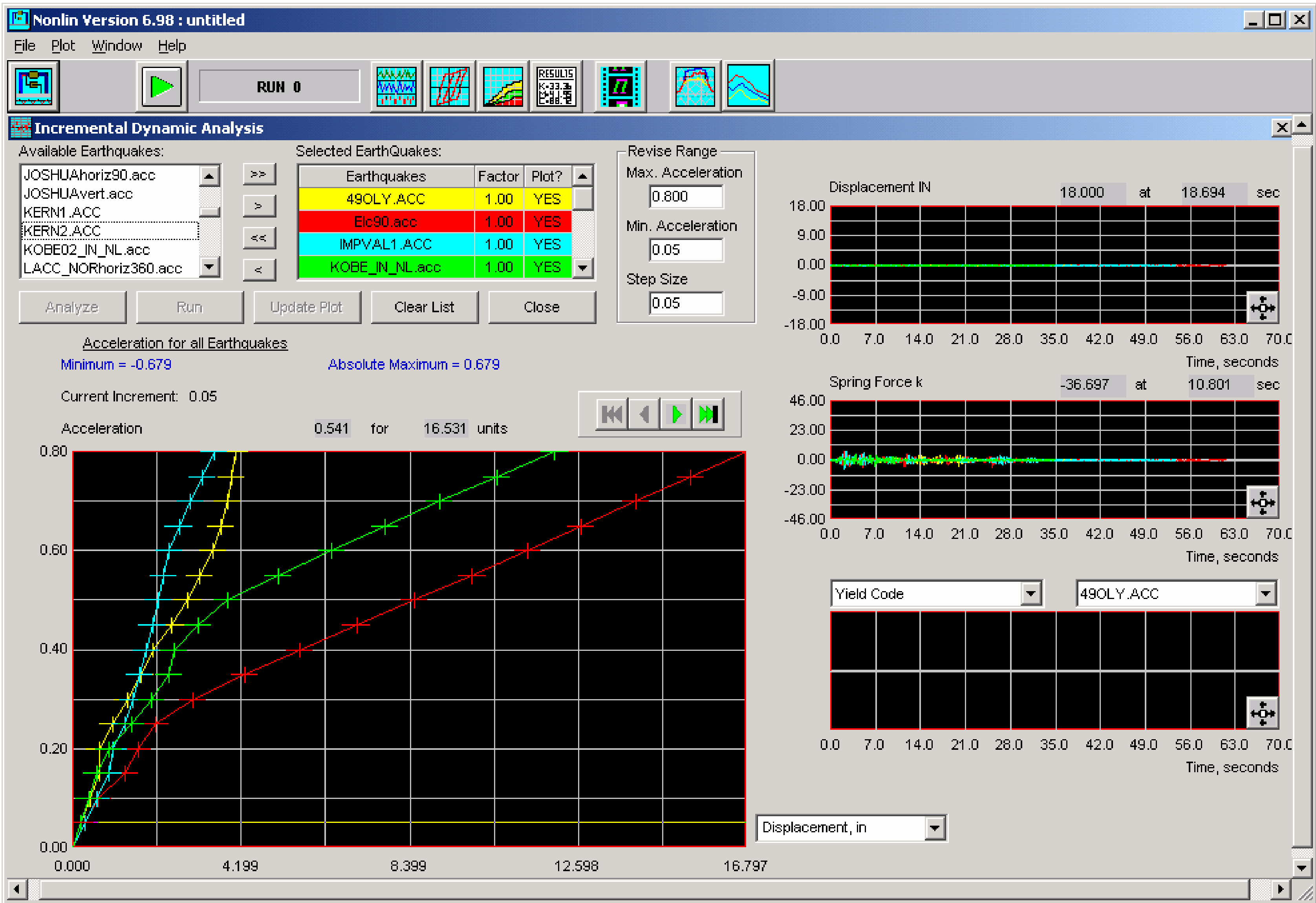
A Family of IDA Curves of the Same Building Subjected to Thirty Earthquakes



Incremental Nonlinear Dynamic Analysis

- Use of IDA shows the **EXTREME** sensitivity of damage to ground motion intensity, as well as the **EXTREME** sensitivity of damage to the chosen ground motion.
- Dispersion in multiple ground motion IDA may be reduced by scaling each base ground motion to a target spectral intensity computed at the structure's fundamental frequency of vibration.
- Even with such scaling, it is clear that PBE assessments based on response history analysis is problematic if carried out in a purely deterministic framework. Probabilistic methods must be employed to adequately handle the randomness of the input and the apparent "chaos" in the results.

NONLIN Version 7 IDA Tool



Probabilistic Approaches to Performance-Based Engineering

The Most Daunting Task:

Identifying and Quantifying Uncertainties

Demand Side (Ground Motion)

1) Magnitude
2) Source Mechanism
3) Wave Propagation Direction
4) Attenuation
5) Site Amplification
6) Frequency Content
7) Duration
8) Sequence (foreshocks, aftershocks)

…

Probabilistic Approaches
The Most Daunting Task:
Identifying and Quantifying Uncertainties

Capacity Side (Soil/Foundation/Structure Behavior)

1) Strength
2) Stiffness
3) Inherent Damping
4) Hysteretic Behavior
5) Gravity Load
6) Built-in Imperfections

…

Analysis Uncertainties

PEER's Probabilistic Framing Equation

$$\lambda(DV) = \iint G(DV \mid DM)\ \left| dG(DV \mid IM) \right| \left| d\lambda(IM) \right|$$

$\lambda(DV)$ Likelihood of exceeding a certain limit state

IM	Intensity Measure
DM	Damage Measure
DV	Decision Variable

Probabilistic Approaches: FEMA 350

$$P(D > PL) = \int P_{D>PL}(x)h(x)dx$$

$P(D > PL)$ Probability of damage exceeding a performance level in a period of *t* years

$P_{D>PL}(x)$ Probability of damage exceeding a performance level given that the ground motion intensity is level *x*, as a function of *x*.

$h(x)dx$ Probability of experiencing a ground motion intensity of level (x) to ($x+dx$) in a period of *t* years

Probabilistic Approaches: FEMA 350

$$P(D > PL) = \int P_{D>PL}(x)h(x)dx$$

Simplified Method

Detailed Method

Probabilistic Approaches: FEMA 350

$$\lambda = \frac{\gamma \, \gamma_a \, D}{\phi C}$$

λ	Capacity to Demand Ratio
γ	Demand Variability Factor
γ_a	Analysis Uncertainty Factor
C	Tabulated Capacity for the Component
ϕ	Capacity Resistance Factor
D	Calculated Demand for the Component

β_{UT} Total Coefficient of Variation

Probabilistic Approaches: FEMA 350

Table 4-7
Recommended Minimum Confidence Levels

Behavior	Performance Level	
	Immediate Occupancy	Collapse Prevention
Global Interstory Drift	50%	90%
Local Interstory Drift	50%	50%
Column Compression	50%	90%
Splice Tension	50%	50%

Probabilistic Approaches: FEMA 350

Table 4-8
Interstory Drift Angle Analysis Uncertainty Factor γ_a

Analysis Procedure		LSP		LDP		NSP		NDP	
System Characteristic		I.O	C.P.	I.O	C.P.	I.O	C.P.	I.O	C.P.
Special	Low Rise (<4 stories)	0.94	0.70	1.03	0.83	1.13	0.89	1.02	1.03
Special	Mid Rise (4-12 stories)	1.15	0.97	1.14	1.25	1.45	0.99	1.02	1.06
Special	High Rise (> 12 stories)	1.12	1.21	1.21	1.14	1.36	0.95	1.04	1.10
Ordinary	Low Rise (<4 stories)	0.79	0.98	1.04	1.32	0.95	1.31	1.02	1.03
Ordinary	Mid Rise (4-12 stories)	0.85	1.14	1.10	1.53	1.11	1.42	1.02	1.06
Ordinary	High Rise (> 12 stories)	0.80	0.85	1.39	1.38	1.36	1.53	1.04	1.10

Probabilistic Approaches: FEMA 350

Table 4-9
Interstory Drift Angle Demand Variability Factor γ

	Building Height	γ	
		I.O.	C.P.
Special	Low Rise (< 4 stories)	1.5	1.3
Special	Mid Rise (4-12 stories)	1.4	1.2
Special	High rise (>12 stories)	1.4	1.5
Ordinary	Low Rise (< 4 stories)	1.4	1.4
Ordinary	Mid Rise (4-12 stories)	1.3	1.5
Ordinary	High rise (>12 stories)	1.6	1.8

Probabilistic Approaches: FEMA 350

Table 4-10
Global Interstory Drift Angle Capacity Factors (C) and Resistance Factors (ϕ)

	Building Height	I.O.		C.P.	
		C	ϕ	C	ϕ
Special	Low Rise (<4 stories)	0.02	1.00	0.10	0.90
Special	Mid Rise (4-12 stories)	0.02	1.00	0.10	0.85
Special	High Rise (> 12 stories)	0.02	1.00	0.09	0.75
Ordinary	Low Rise (<4 stories)	0.01	1.00	0.10	0.85
Ordinary	Mid Rise (4-12 stories)	0.01	0.90	0.08	0.70
Ordinary	High Rise (> 12 stories)	0.01	0.85	0.06	0.60

Probabilistic Approaches: FEMA 350

Table 4-11
Uncertainty Coefficient β_{UT} for Global Interstory Drift Evaluation

	Building Height	Perf. Level	
		I.O.	C.P.
Special	Low Rise (< 4 stories)	0.20	0.30
Special	Mid Rise (4-12 stories)	0.20	0.40
Special	High rise (>12 stories)	0.20	0.50
Ordinary	Low Rise (< 4 stories)	0.20	0.35
Ordinary	Mid Rise (4-12 stories)	0.20	0.45
Ordinary	High rise (>12 stories)	0.20	0.55

Probabilistic Approaches: FEMA 350

Table 4-6
Confidence Levels for Various Values of λ and β_{UT}

Confidence Level	10	20	30	40	50	60	70	80	90	95	99
λ for $\beta_{UT} = 0.2$	1.37	1.26	1.18	1.12	1.06	1.01	0.96	0.90	0.82	0.76	0.67
λ for $\beta_{UT} = 0.3$	1.68	1.48	1.34	1.24	1.14	1.06	0.98	0.89	0.78	0.70	0.57
λ for $\beta_{UT} = 0.4$	2.12	1.79	1.57	1.40	1.27	1.15	1.03	0.90	0.76	0.66	0.51
λ for $\beta_{UT} = 0.5$	2.76	2.23	1.90	1.65	1.45	1.28	1.12	0.95	0.77	0.64	0.46
λ for $\beta_{UT} = 0.6$	3.70	2.86	2.36	1.99	1.72	1.48	1.25	1.03	0.80	0.64	0.43

Probabilistic Approaches: FEMA 350

Example Calculations for 4-12 Story Frame (DL is "Allowable" Interstory Drift Limit)

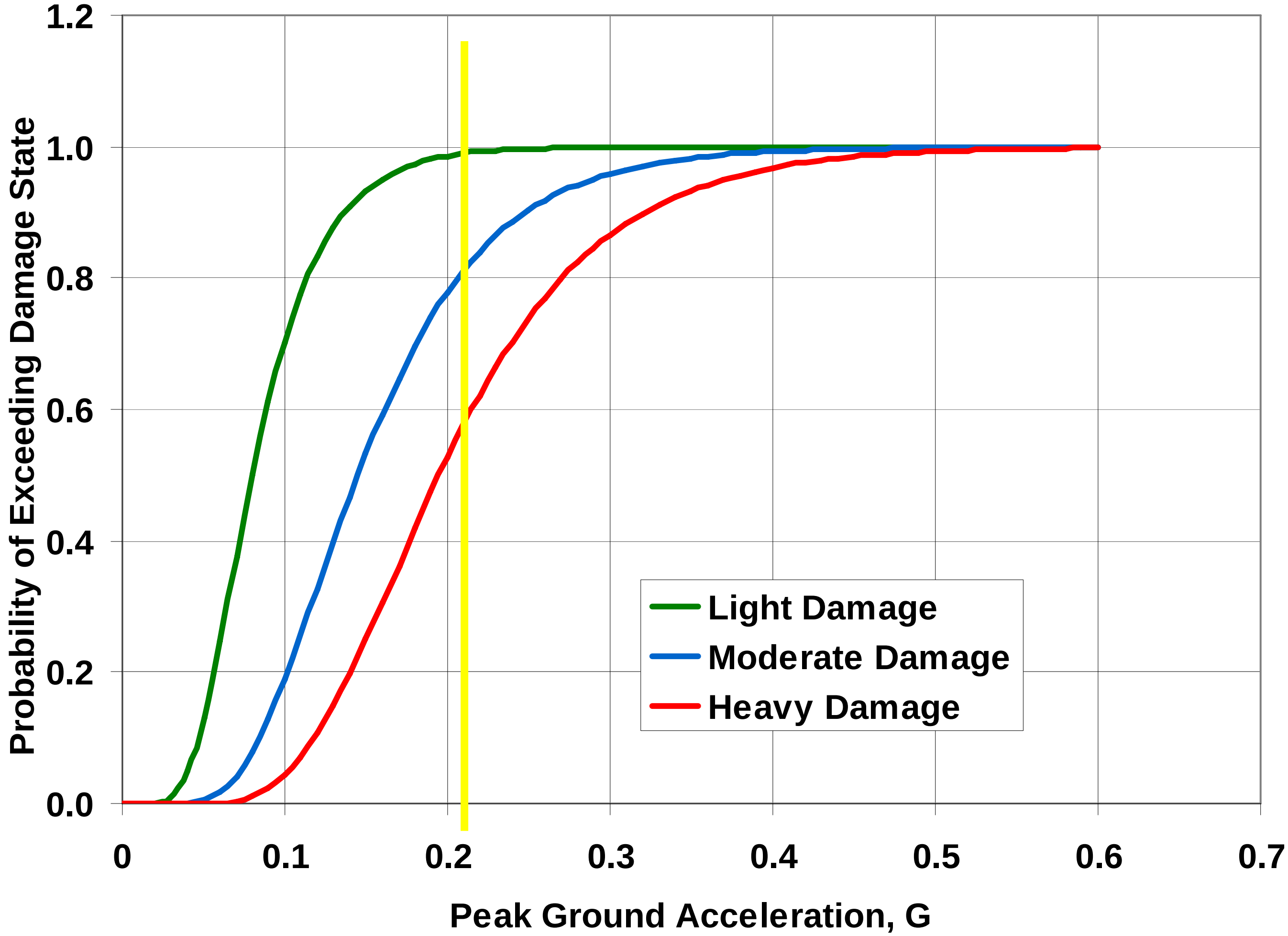
Type	PERF	Analysis	Confidence	γ	γ_a	ϕ	C	β_{UT}	λ	DL
SPECIAL	IO	NSP	50%	1.4	1.45	1	0.02	0.2	1.06	0.0104
SPECIAL	IO	NDP	50%	1.4	1.02	1	0.02	0.2	1.06	0.0148
SPECIAL	CP	NSP	90%	1.2	0.99	0.85	0.1	0.4	0.76	0.0544
SPECIAL	CP	NDP	90%	1.2	1.06	0.85	0.1	0.4	0.76	0.0508
ORDINARY	IO	NSP	50%	1.3	1.11	0.9	0.01	0.2	1.06	0.0066
ORDINARY	IO	NDP	90%	1.3	1.02	0.9	0.01	0.2	1.06	0.0072
ORDINARY	CP	NSP	50%	1.5	1.42	0.7	0.08	0.45	0.765	0.0201
ORDINARY	CP	NDP	90%	1.5	1.06	0.7	0.08	0.45	0.765	0.0269

Problem with FEMA 350 Approach?

Even though the method provides the owner a "Level of Confidence" that a certain performance criteria will be met, the engineer is likely to be bewildered by the arrays of coefficients. Hence, it is difficult for the engineer to obtain a feel for the validity of the results.

Given this, how confident is the engineer with the value of confidence provided?

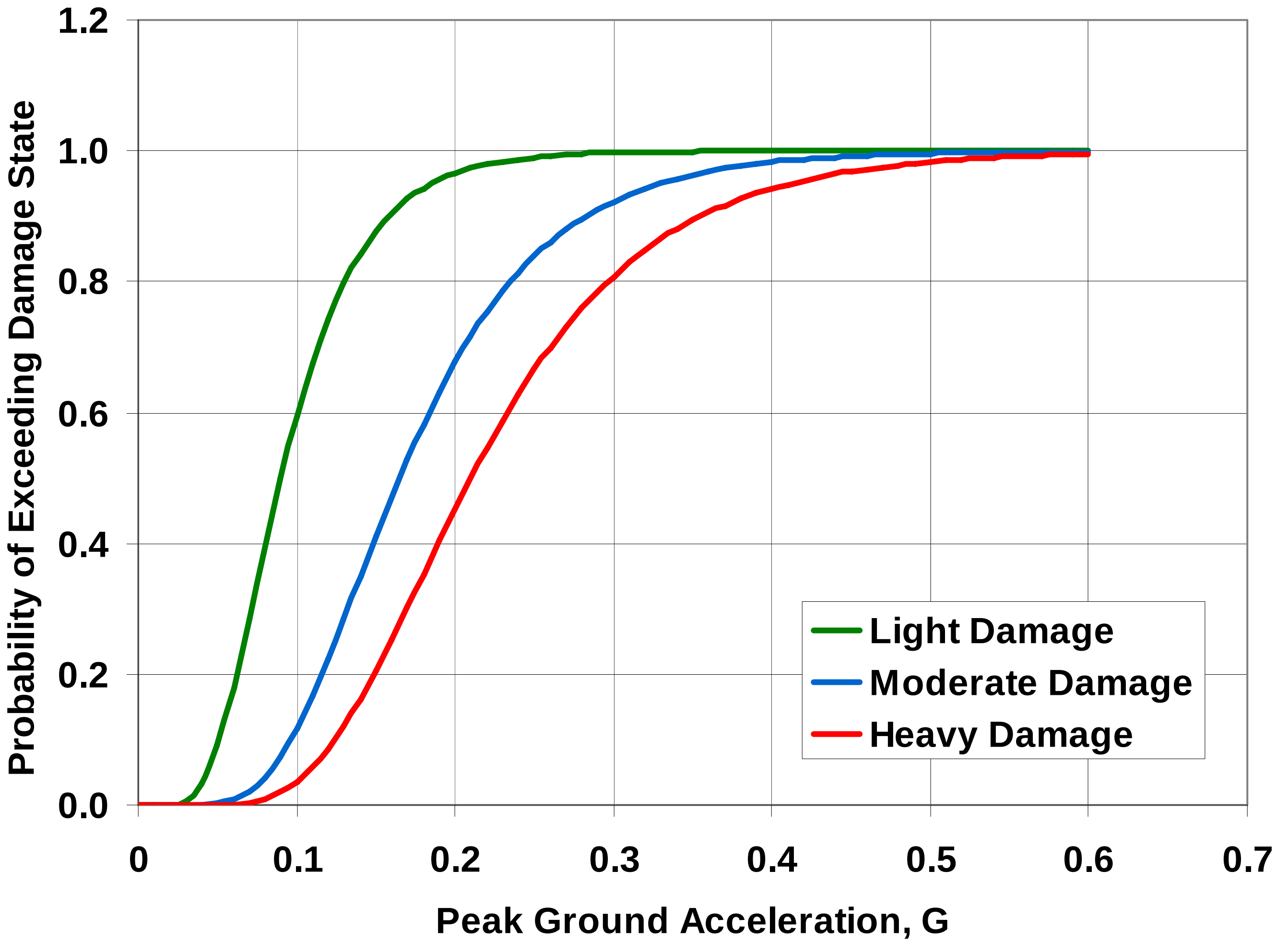
Probabilistic Approaches: Fragility Curves
Unreinforced Masonry



http://www.ceri.memphis.edu/~hwang/

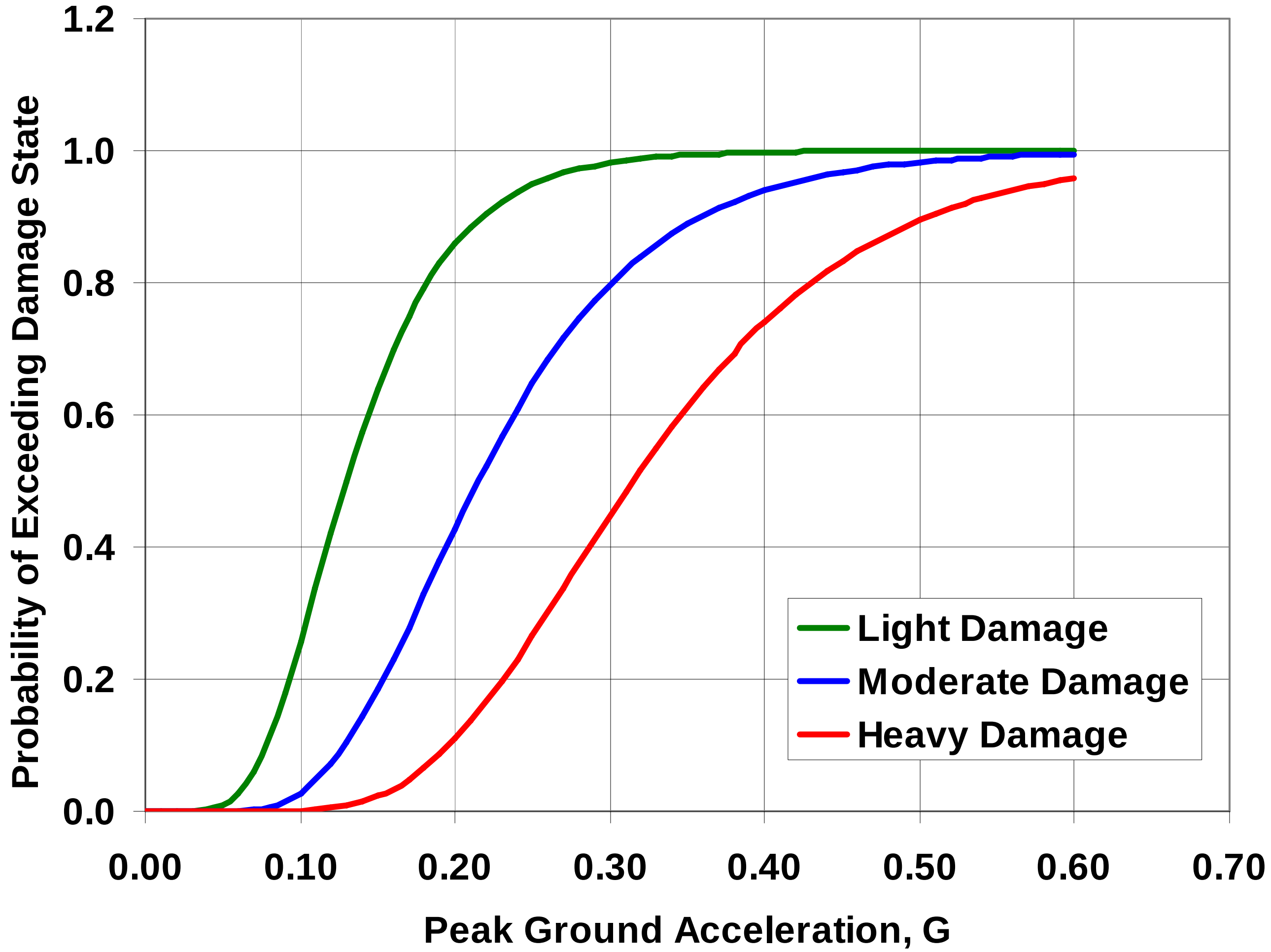
Probabilistic Approaches: Fragility Curves

Reinforced Masonry



http://www.ceri.memphis.edu/~hwang/

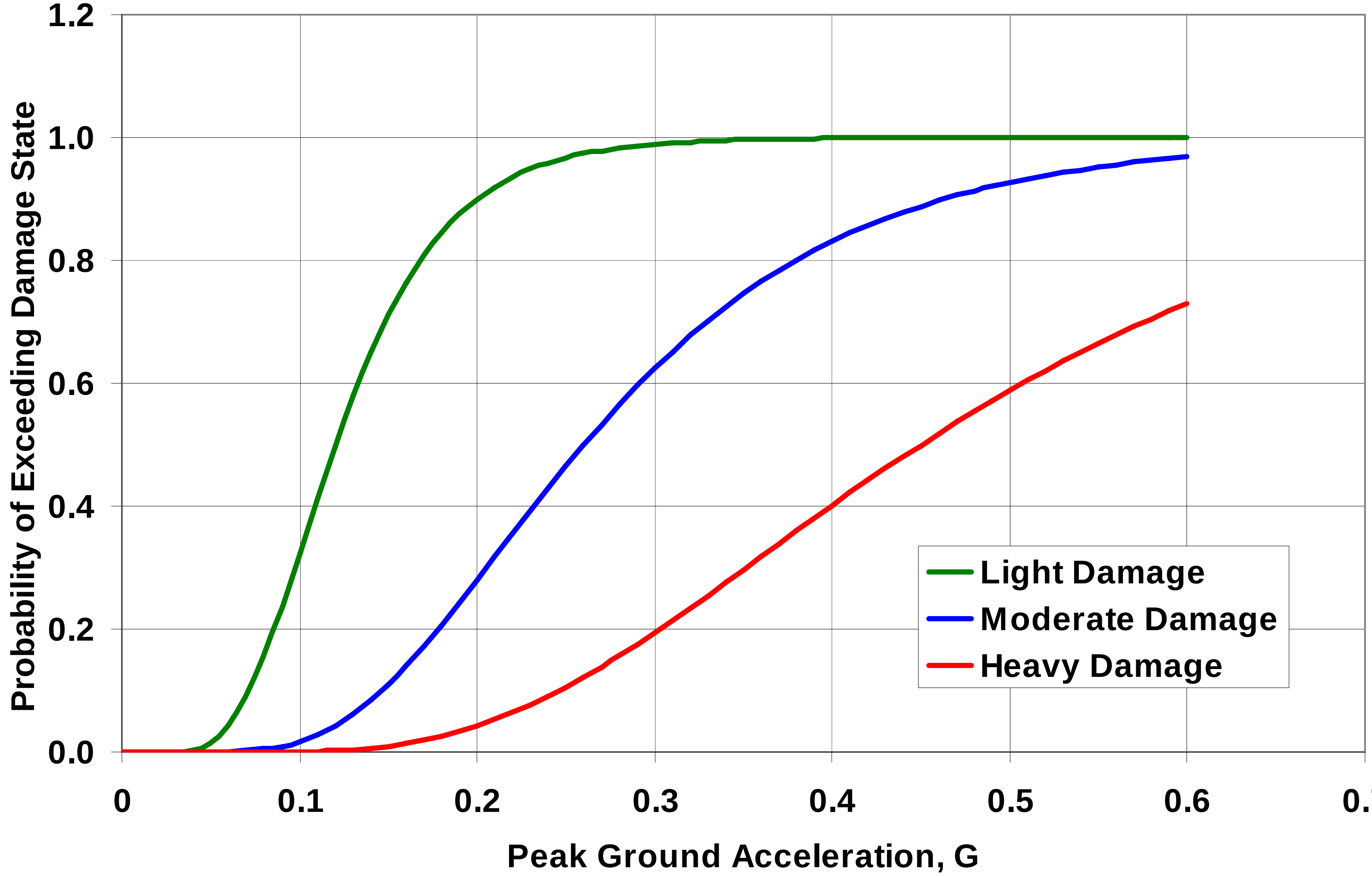
Probabilistic Approaches: Fragility Curves
Reinforced Concrete



http://www.ceri.memphis.edu/~hwang/

Probabilistic Approaches: Fragility Curves

Wood Frame



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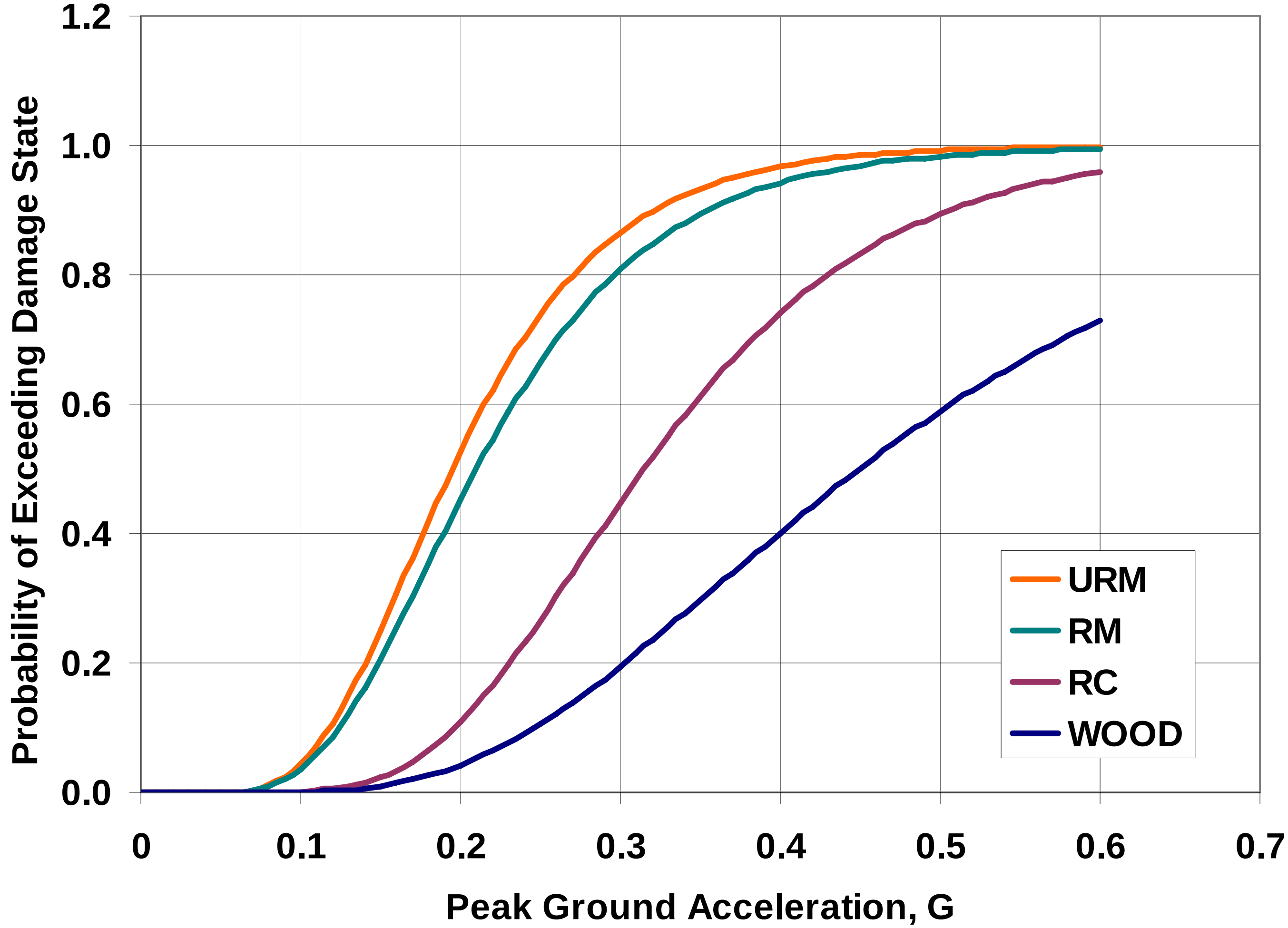
Probabilistic Approaches: Fragility Curves (Heavy Damage)



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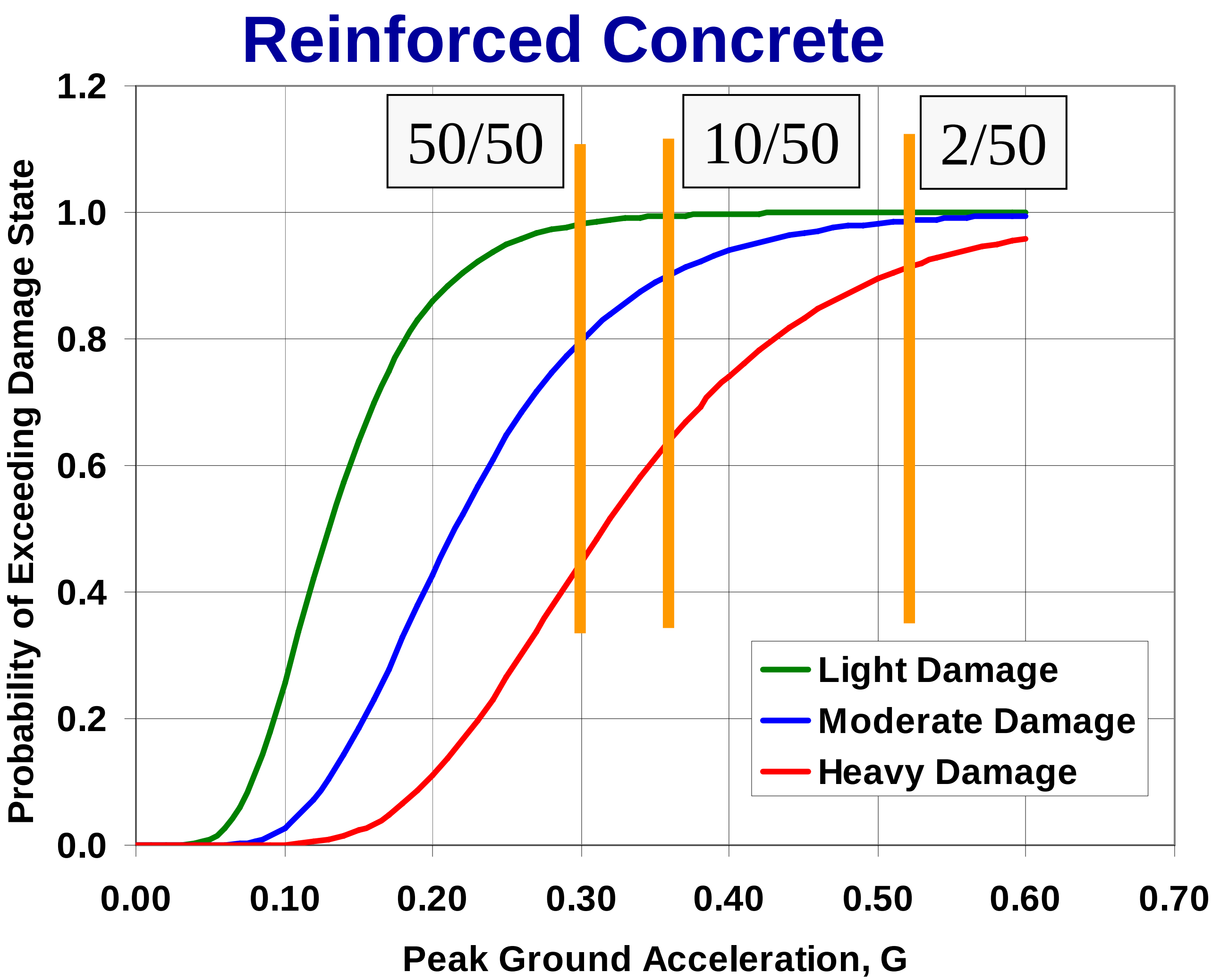
Probabilistic Approaches: Fragility Curves



http://www.ceri.memphis.edu/~hwang/

Where are We Headed with Performance Based Engineering?

- **Performance Basis: Minimize Life Cycle Costs**
  - Realistic Damage Measures
  - Realistic Forecasting of Cost of Repairing Damage
  - Realistic Forecasting of Cost of Loss of Use

- **Analysis Procedures**
  - Incremental Nonlinear Dynamic Response History Analysis
  - Sensitivity Analysis (Deterministic)
  - Probabilistic Assessment of Performance
  - Deaggregation of Probabilistic Results (Deterministic)

What We Need

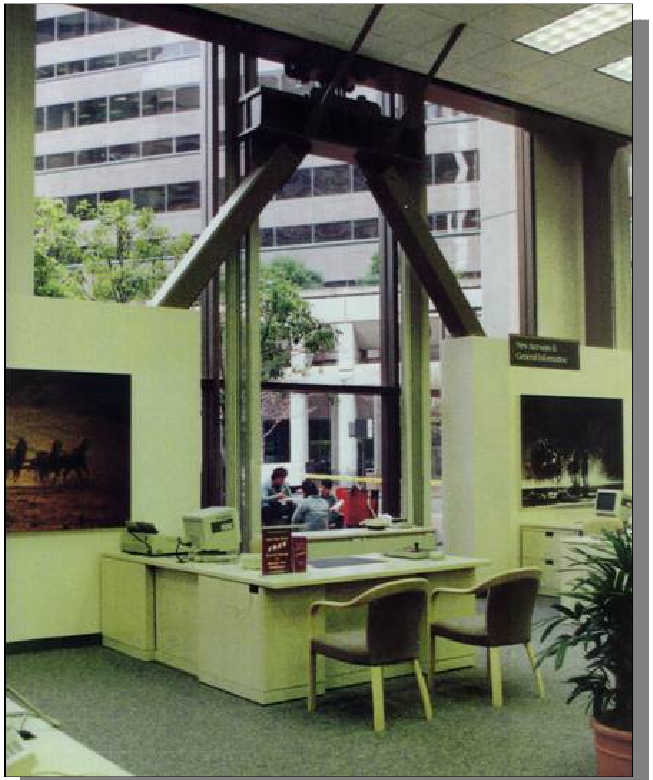
- Ground motion search, scaling, and modification tools for development of suites for nonlinear dynamic analysis
- Reliable damage measures which (hopefully) minimize dispersion in results
- **Rapid** but reliable methods of analysis, including
  - Multiple Ground Motions [7 motions]
  - Incremental Nonlinear Dynamic Analysis [20 increments]
  - Systematic Sensitivity Analysis [10 uncert. X 8 values]
  - Deterministic/Probabilistic Assessment Tools
- Big, **Fast** (Parallel Processing) Computers

SEISMIC PROTECTIVE SYSTEMS: PASSIVE ENERGY DISSIPATION

Presented & Developed by:

Michael D. Symans, PhD

Rensselaer Polytechnic Institute



Initially Developed by:

Finley A. Charney, PE, PhD

Virginia Polytechnic Institute & State University

Major Objectives

- Illustrate why use of passive energy dissipation systems may be beneficial
- Provide overview of types of energy dissipation systems available
- Describe behavior, modeling, and analysis of structures with energy dissipation systems
- Review developing building code requirements

Outline: Part I

- Objectives of Advanced Technology Systems and Effects on Seismic Response
- Distinction Between Natural and Added Damping
- Energy Distribution and Damage Reduction
- Classification of Passive Energy Dissipation Systems

Outline: Part II

- Velocity-Dependent Damping Systems: Fluid Dampers and Viscoelastic Dampers
- Models for Velocity-Dependent Dampers
- Effects of Linkage Flexibility
- Displacement-Dependent Damping Systems: Steel Plate Dampers, Unbonded Brace Dampers, and Friction Dampers
- Concept of Equivalent Viscous Damping
- Modeling Considerations for Structures with Passive Energy Dissipation Systems

Outline: Part III

- Seismic Analysis of MDOF Structures with Passive Energy Dissipation Systems
- Representations of Damping
- Examples: Application of Modal Strain Energy Method and Non-Classical Damping Analysis
- Summary of MDOF Analysis Procedures

Outline: Part IV

- MDOF Solution Using Complex Modal Analysis
- Example: Damped Mode Shapes and Frequencies
- An Unexpected Effect of Passive Damping
- Modeling Dampers in Computer Software
- Guidelines and Code-Related Documents for Passive Energy Dissipation Systems

Outline: Part I

- Objectives of Advanced Technology Systems and Effects on Seismic Response
- Distinction Between Natural and Added Damping
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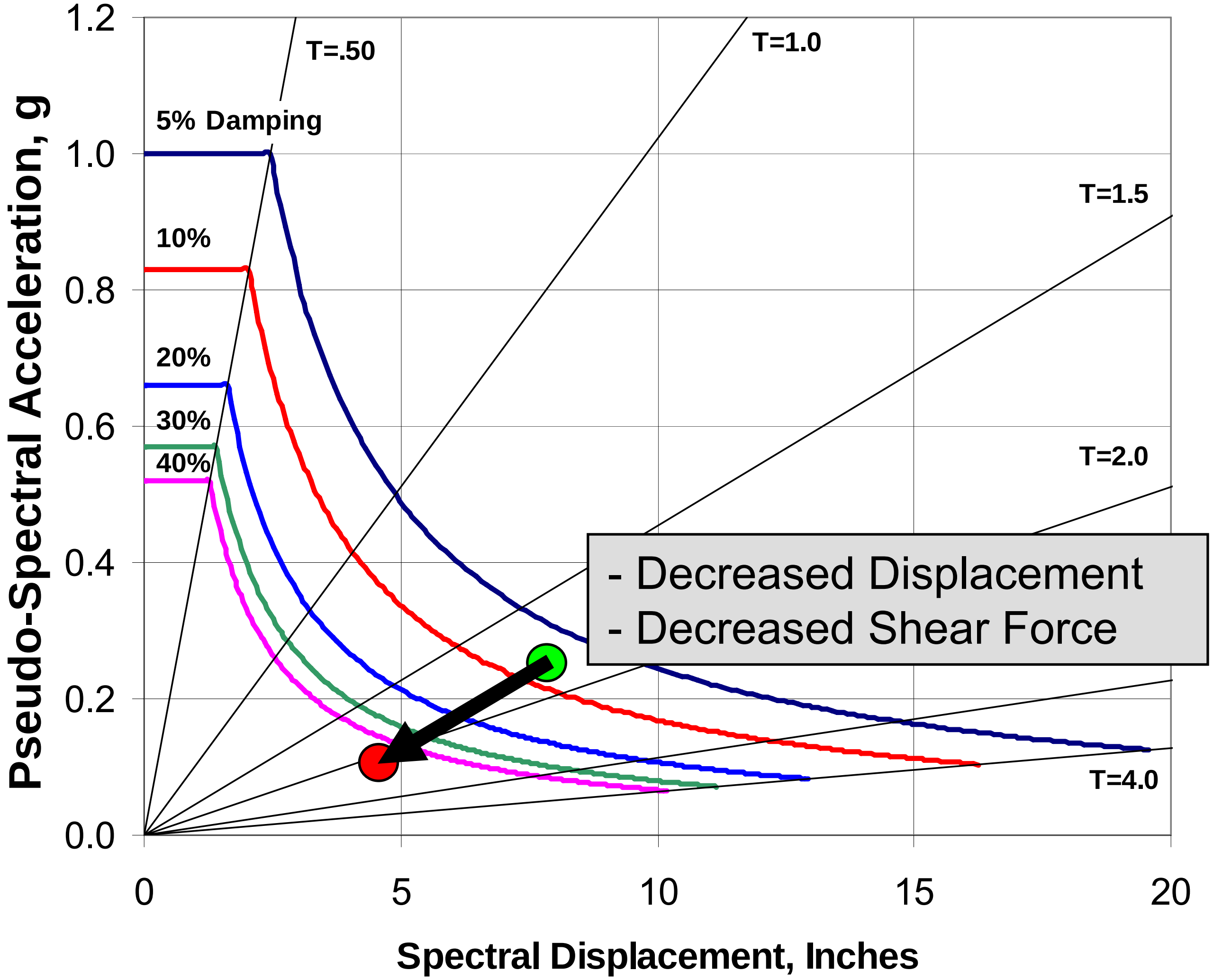
- Classification of Passive Energy Dissipation Systems

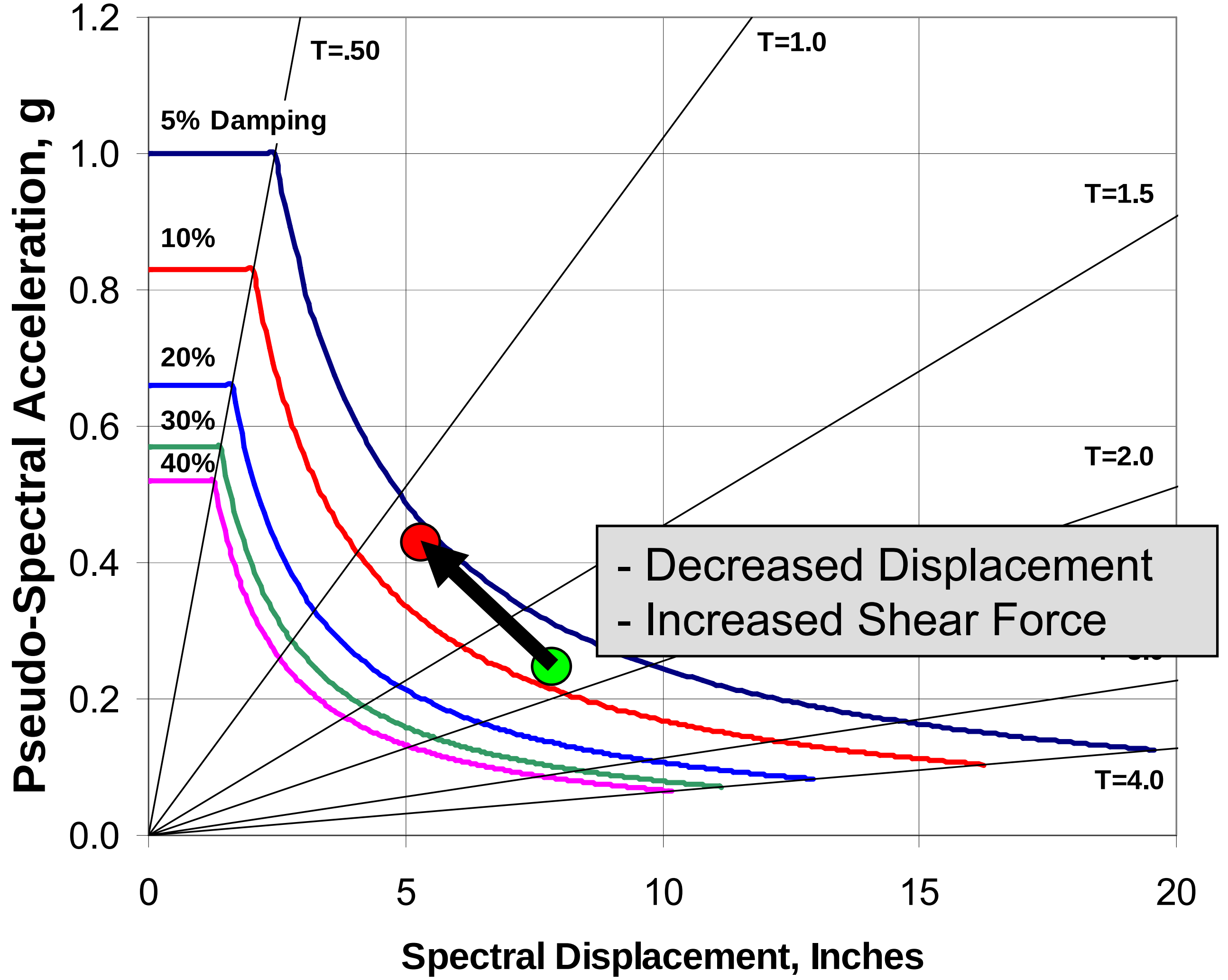
Objectives of Energy Dissipation and Seismic Isolation Systems

- Enhance performance of structures at all hazard levels by:
  - Minimizing interruption of use of facility *(e.g., Immediate Occupancy Performance Level)*
  - Reducing damaging deformations in structural and nonstructural components
  - Reducing acceleration response to minimize contents-related damage

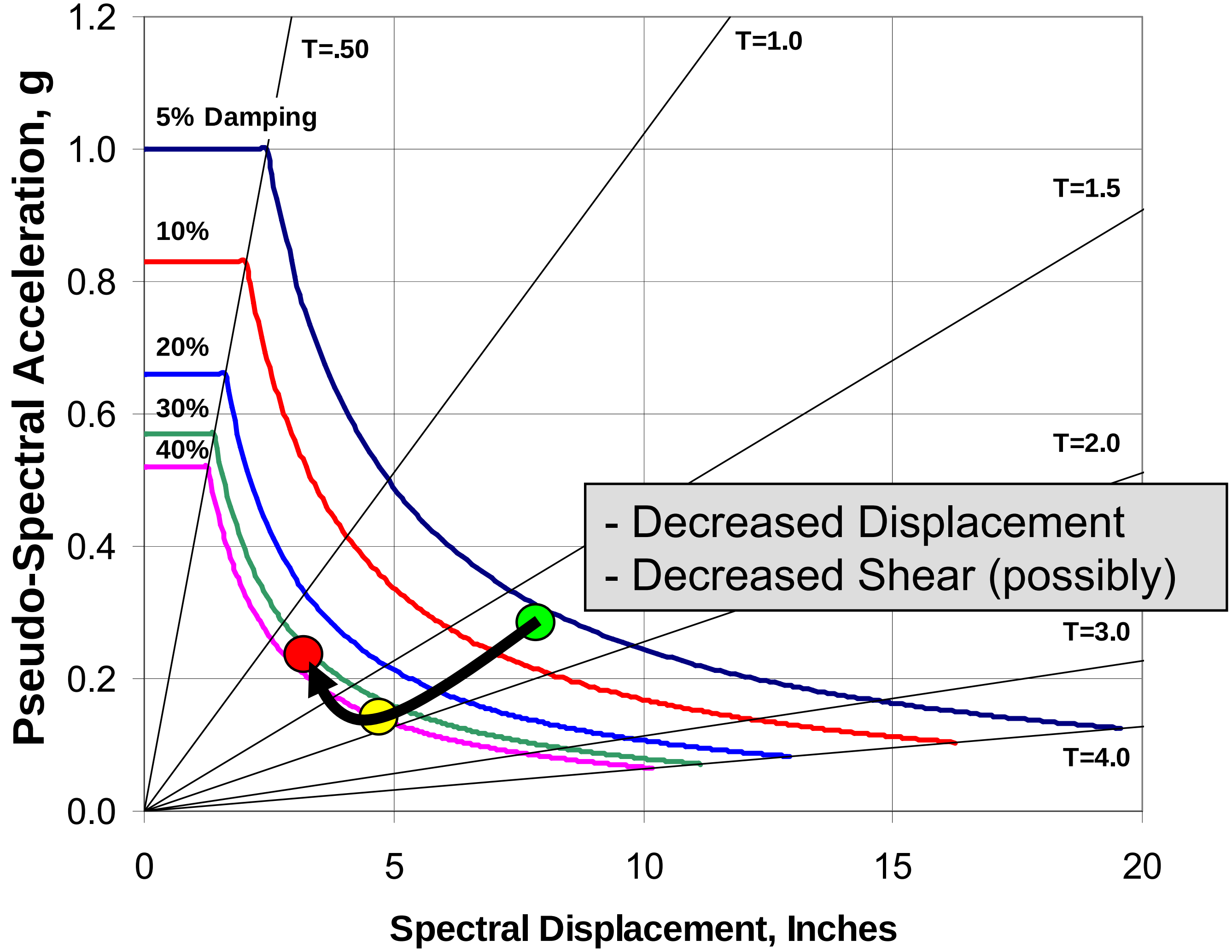
Effect of Added Damping
(Viscous Damper)



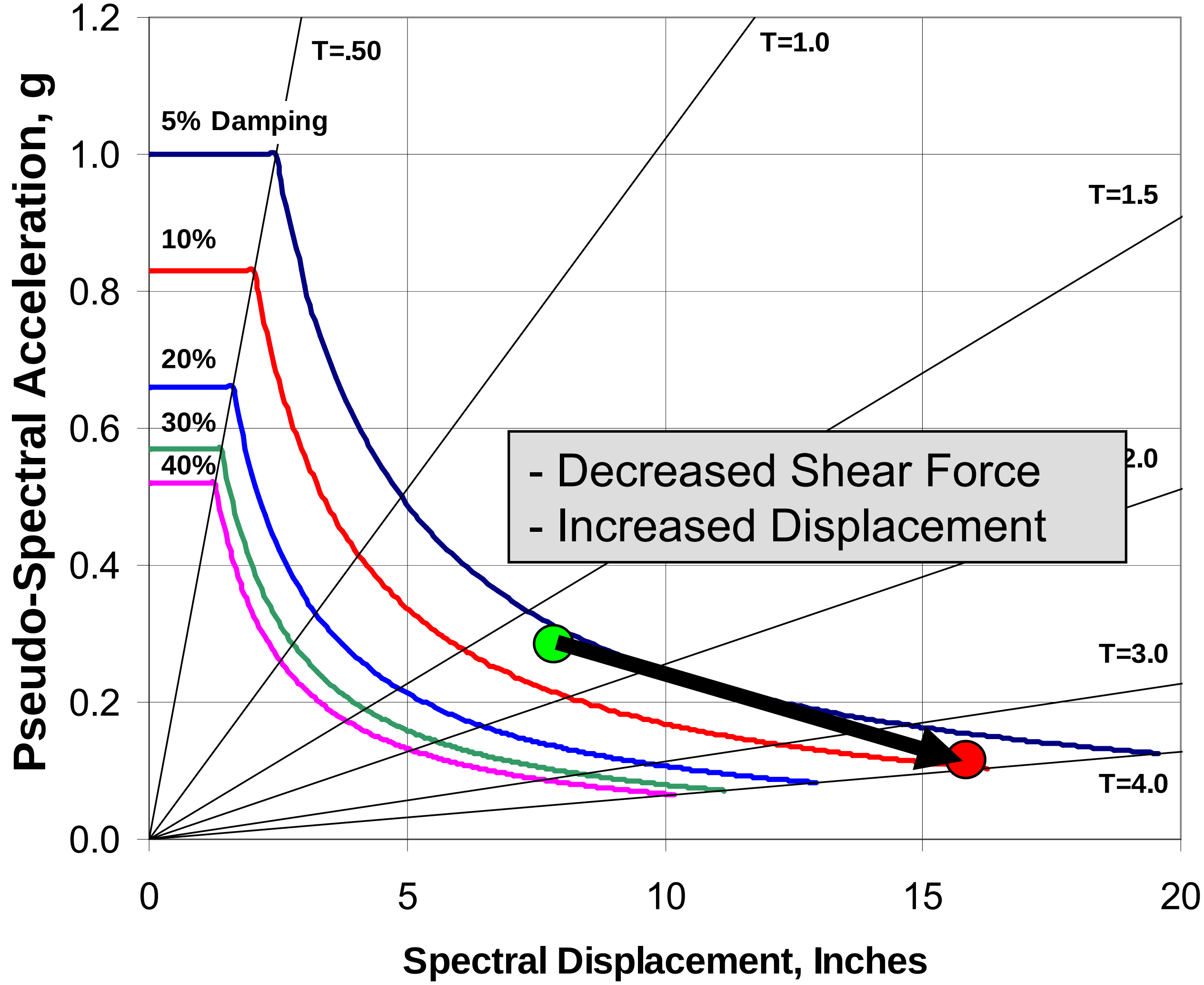
Effect of Added Stiffness
(Added Bracing)



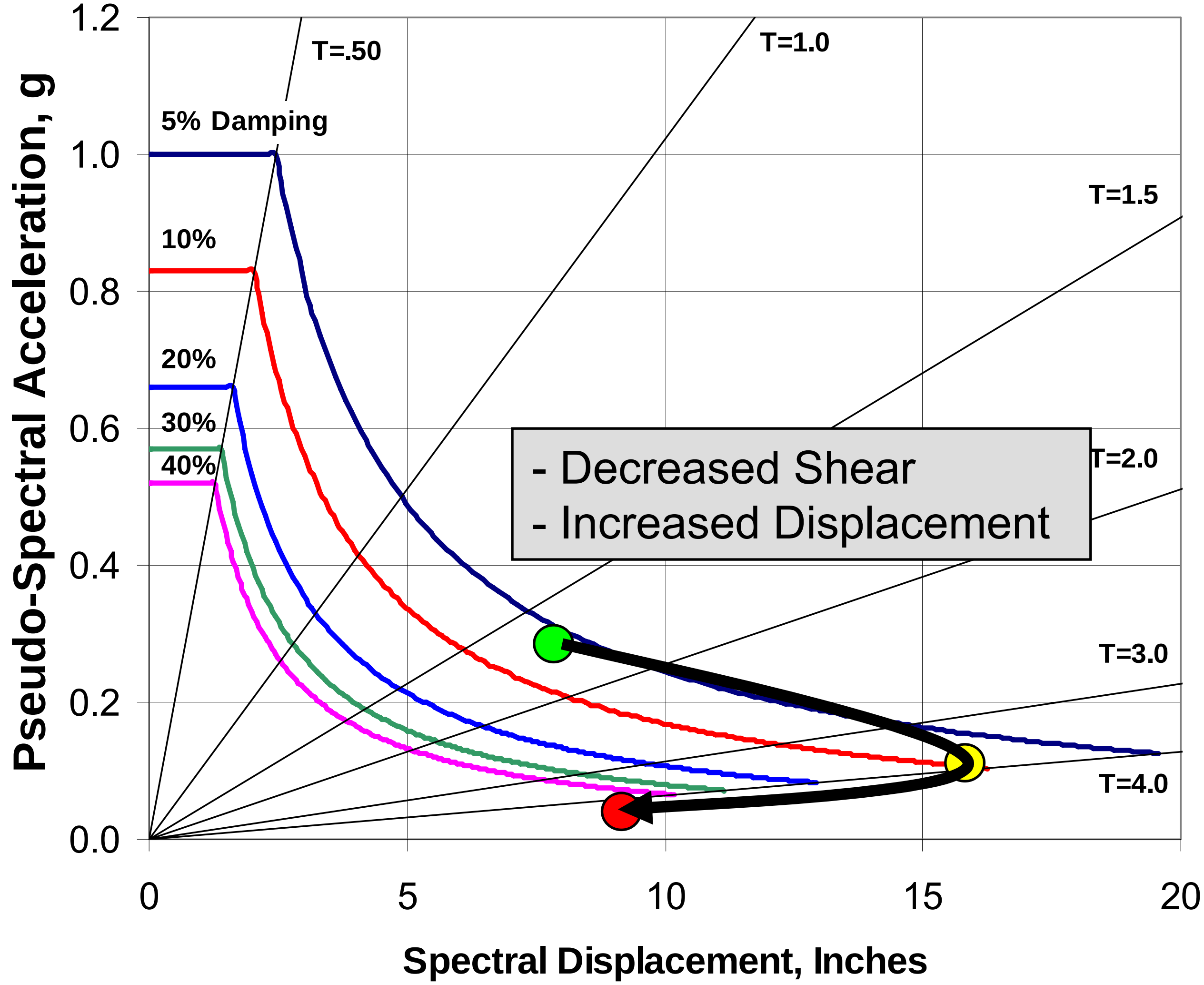
Effect of Added Damping and Stiffness (ADAS System)



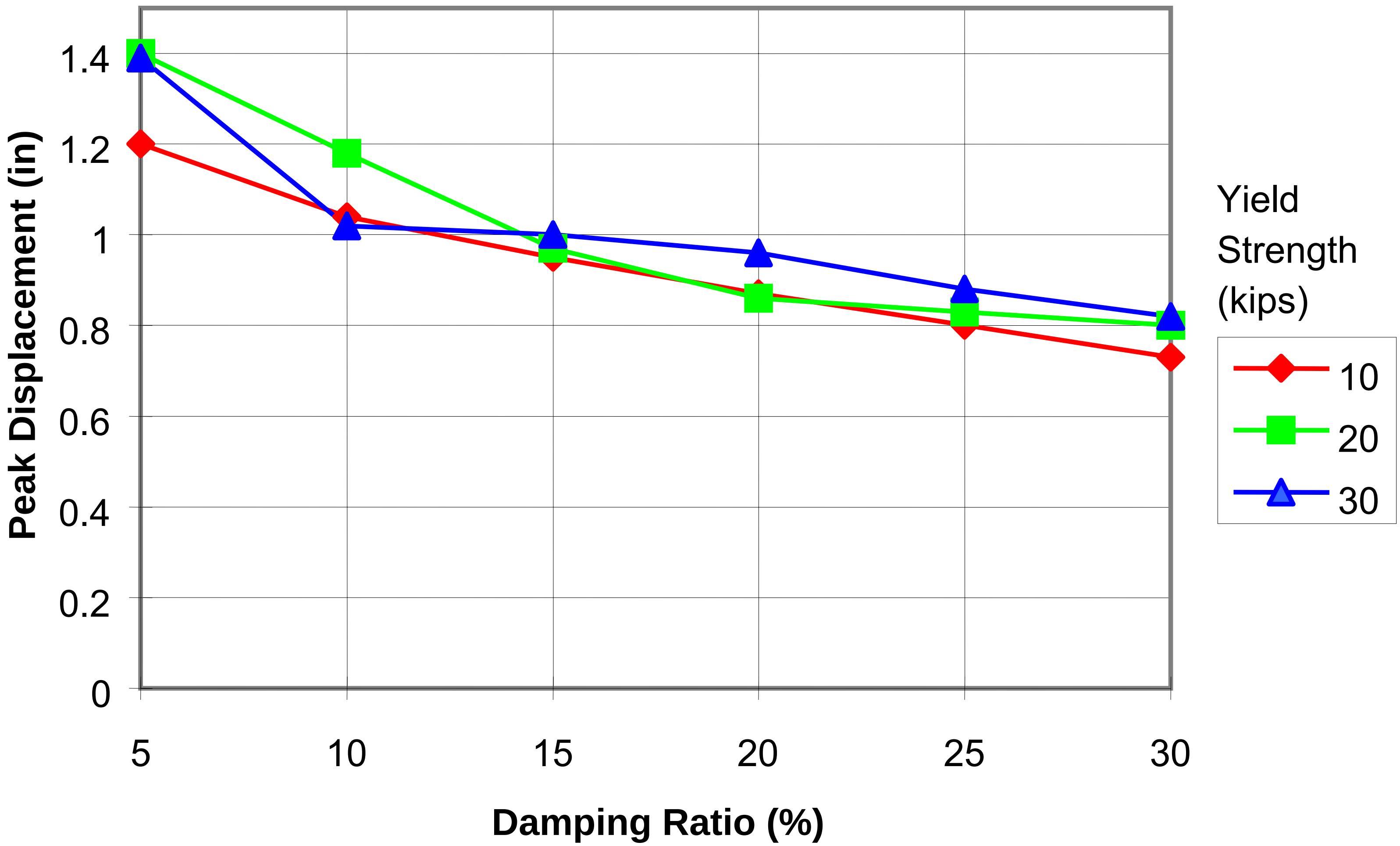
Effect of Reduced Stiffness (Seismic Isolation)



Effect of Reduced Stiffness (Seismic Isolation with Dampers)



Effect of Damping and Yield Strength on Deformation Demand



Outline: Part I

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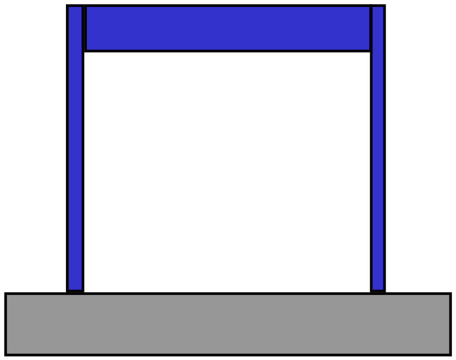
Distinction Between Natural and Added Damping

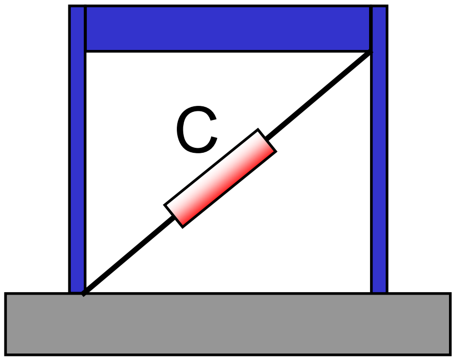
Natural (Inherent) Damping



ξ is a structural property, dependent on system mass, stiffness, and inherent energy dissipation mechanisms

$$\xi_{\mathrm{NATURAL}} = 0.5 \text{ to } 7.0\%$$

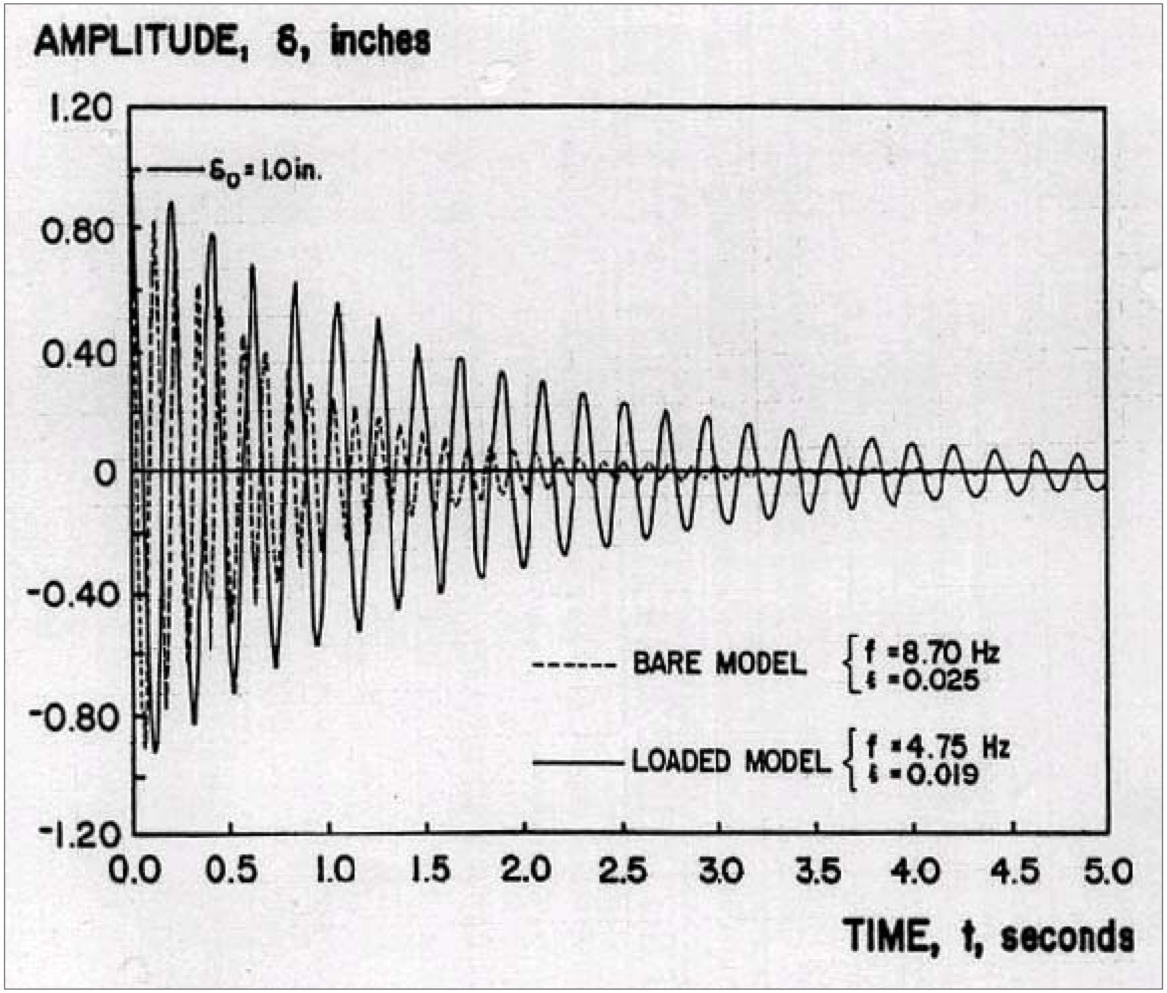
Added Damping

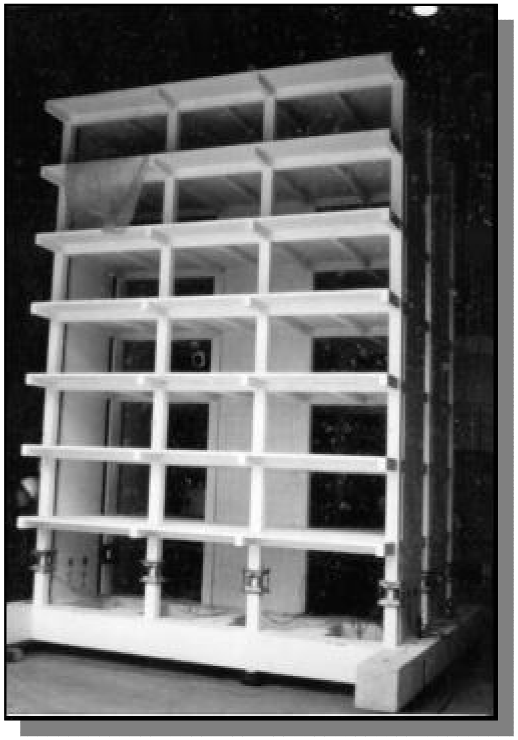


ξ is a structural property, dependent on system mass, stiffness, and the added damping coefficient C

$$\xi_{\mathrm{ADDED}} = 10 \text{ to } 30\%$$

1981/1982 US-JAPAN PROJECT
Response of Bare Frame Before and After Adding Ballast



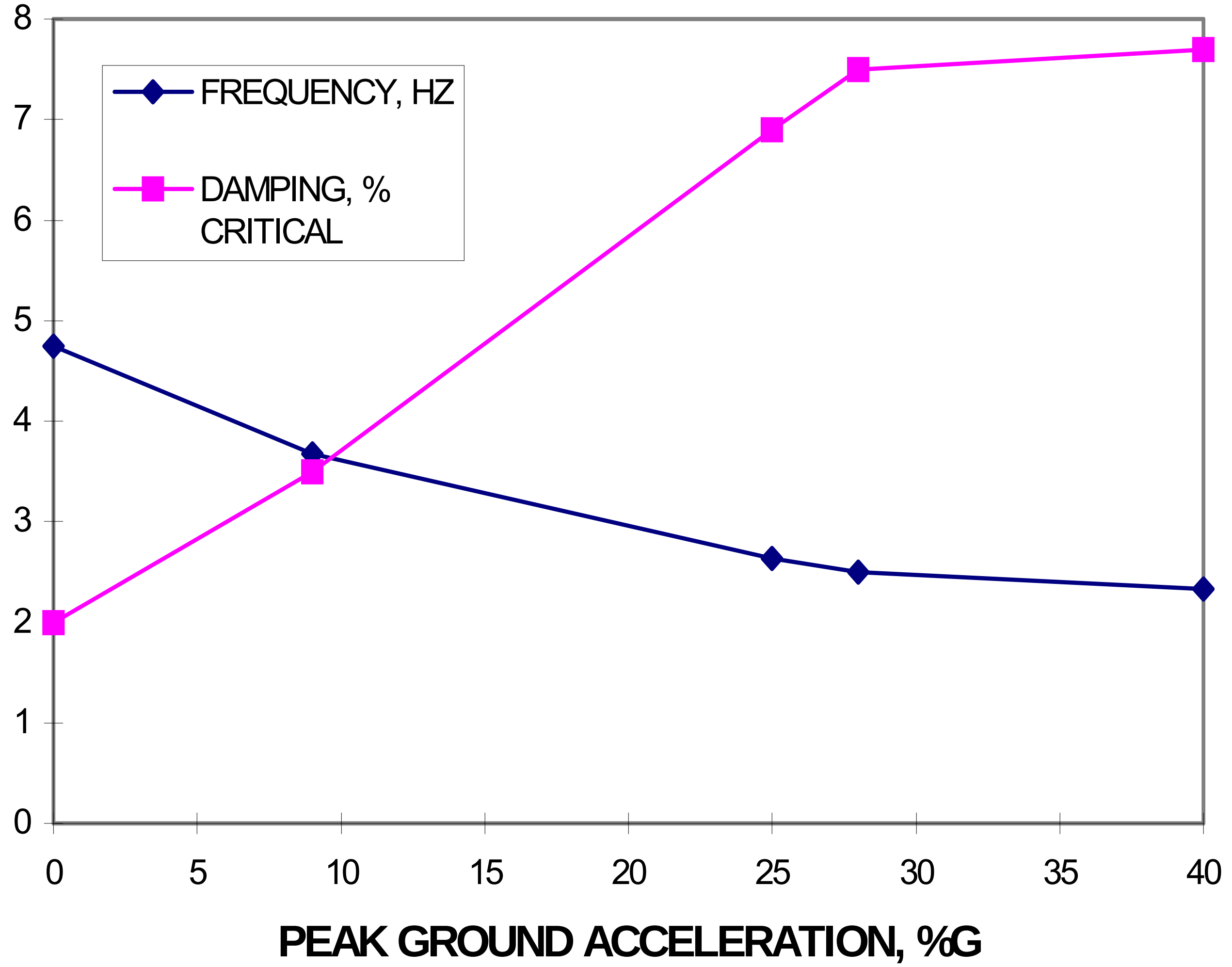


Model Weight

Bare Model	**18 kips**
Loaded Model	**105 kips**

1981/1982 US-JAPAN PROJECT

Change in Damping and Frequency with Accumulated Damage



Outline: Part I

- Objectives of Advanced Technology Systems and Effects on Seismic Response
- Distinction Between Natural and Added Damping
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- Classification of Passive Energy Dissipation Systems

Reduction in Seismic Damage

Hysteretic Energy

Energy Balance:

$$E_I = E_S + E_K + (E_{DI} + E_{DA}) + E_H$$

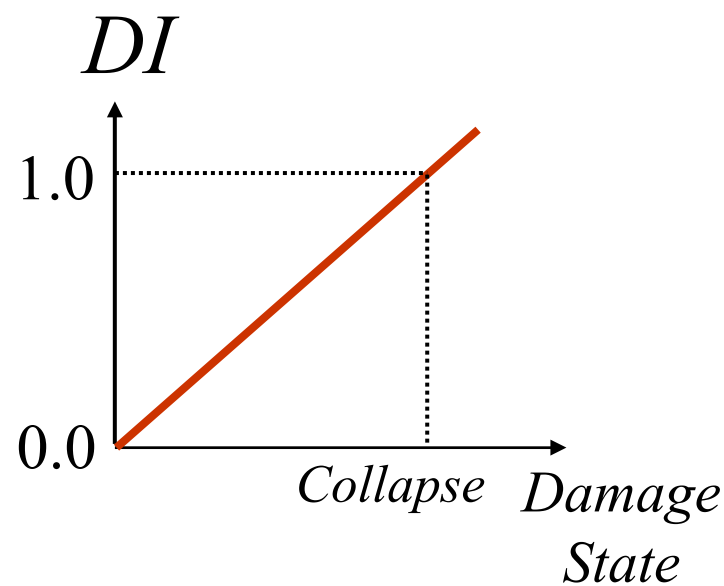
Inherent Damping

Added Damping

Damage Index:

$$DI(t) = \frac{u_{max}}{u_{ult}} + \rho \frac{E_H(t)}{F_y u_{ult}}$$

Source: Park and Ang (1985)



Duration-Dependent Damage Index

$$DI(t) = \frac{u_{max}}{u_{ult}} + \rho \frac{E_H(t)}{F_y u_{ult}}$$

Source: Park and Ang (1985)

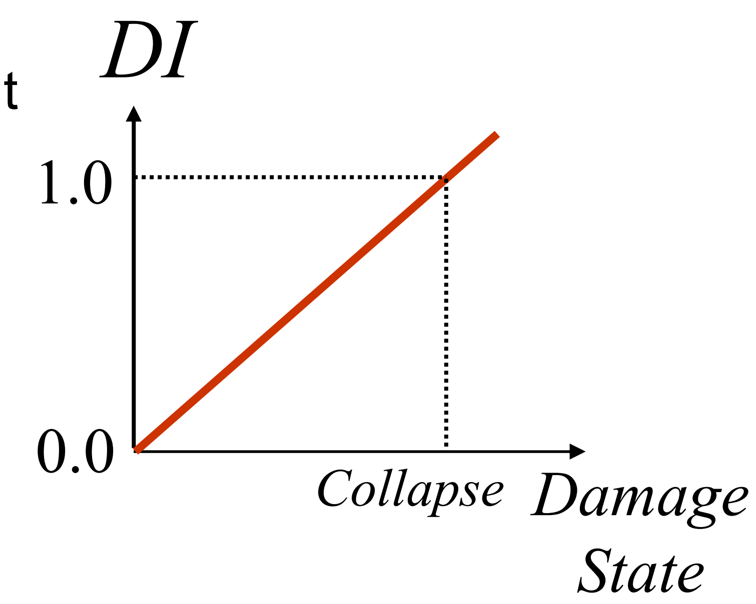
u_{max} = maximum displacement

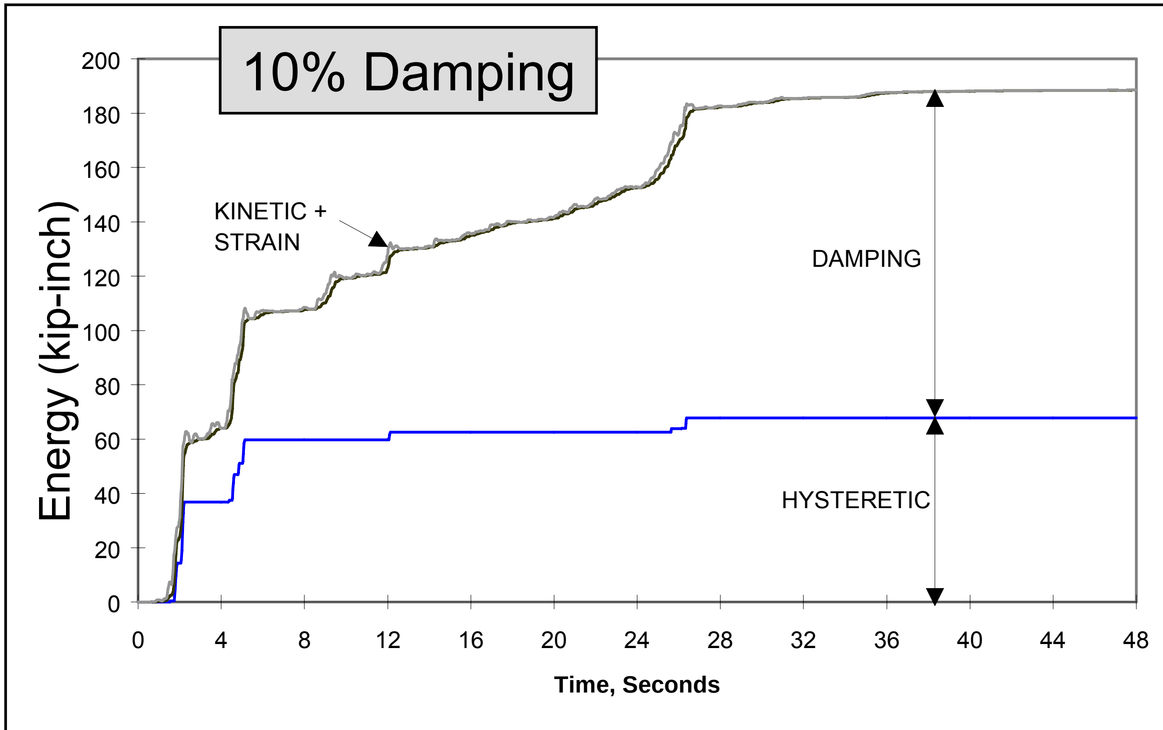
u_{ult} = monotonic ultimate displacement

ρ = calibration factor

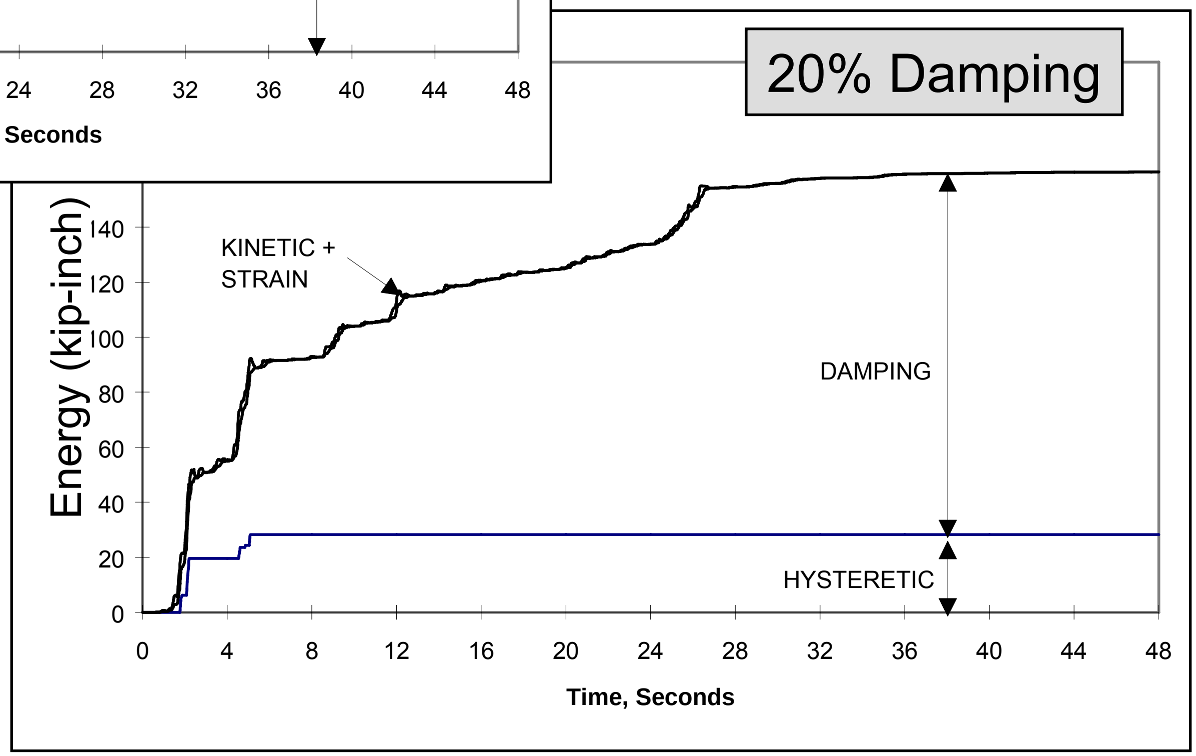
E_H = hysteretic energy dissipated

F_y = monotonic yield force

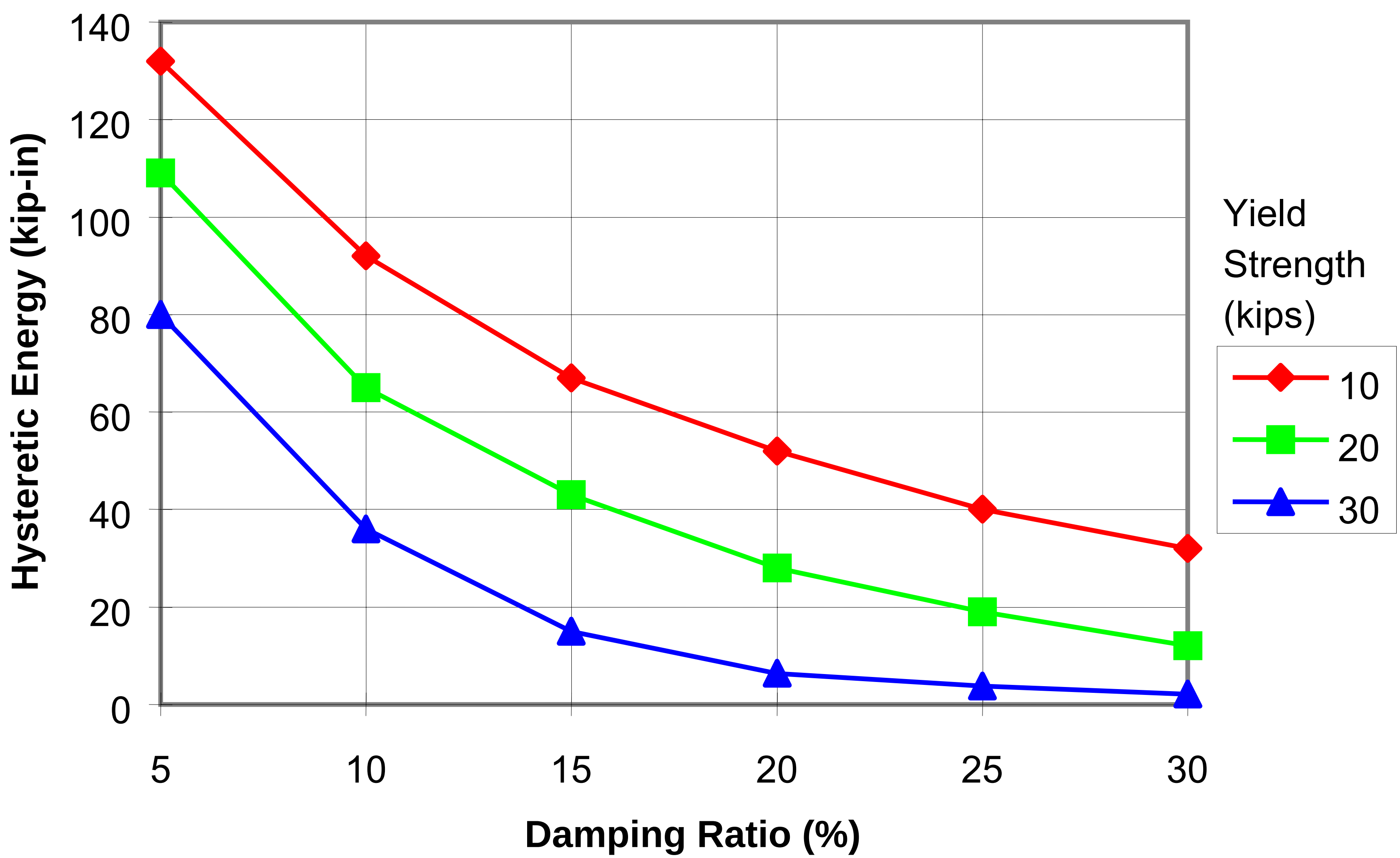




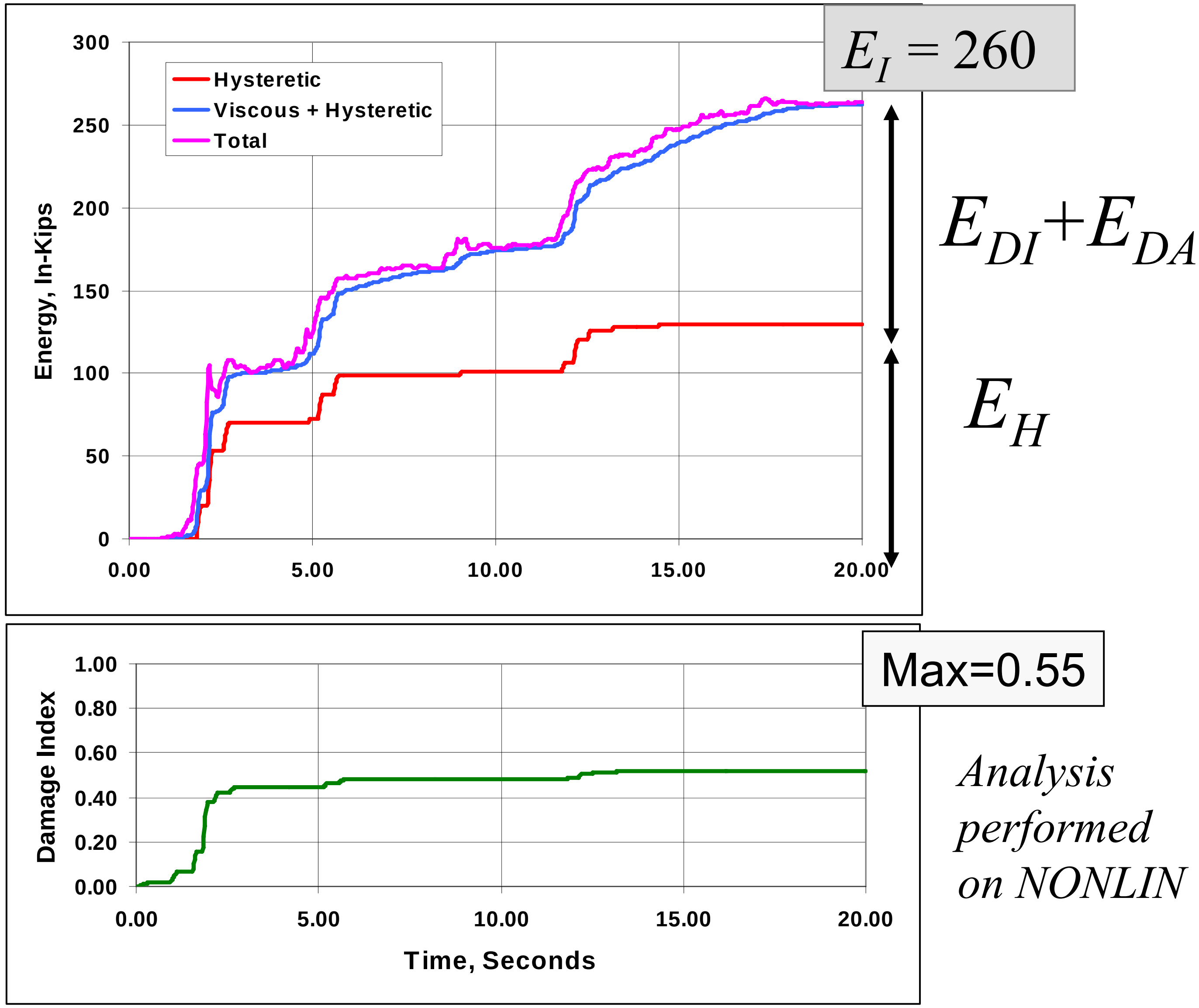
Damping Reduces Hysteretic Energy Dissipation Demand



Effect of Damping and Yield Strength on Hysteretic Energy

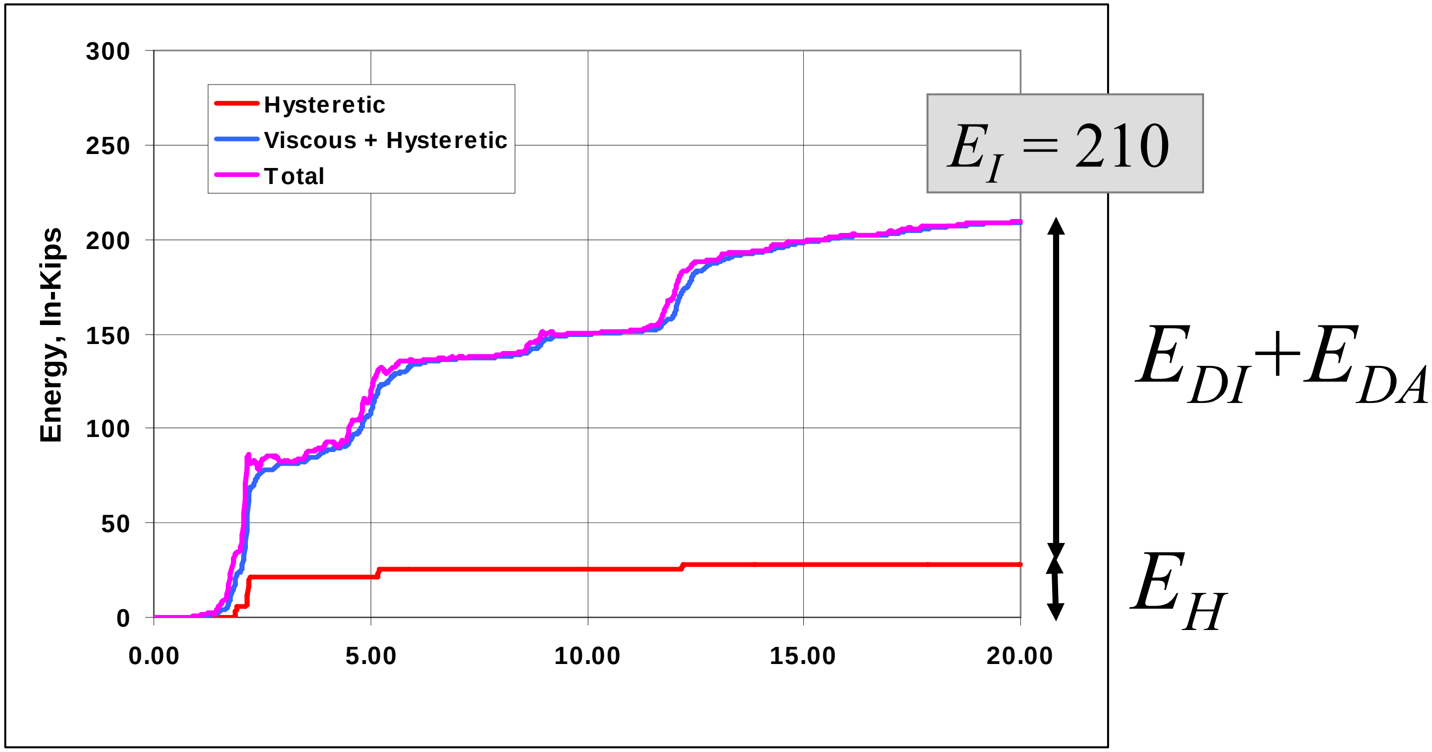


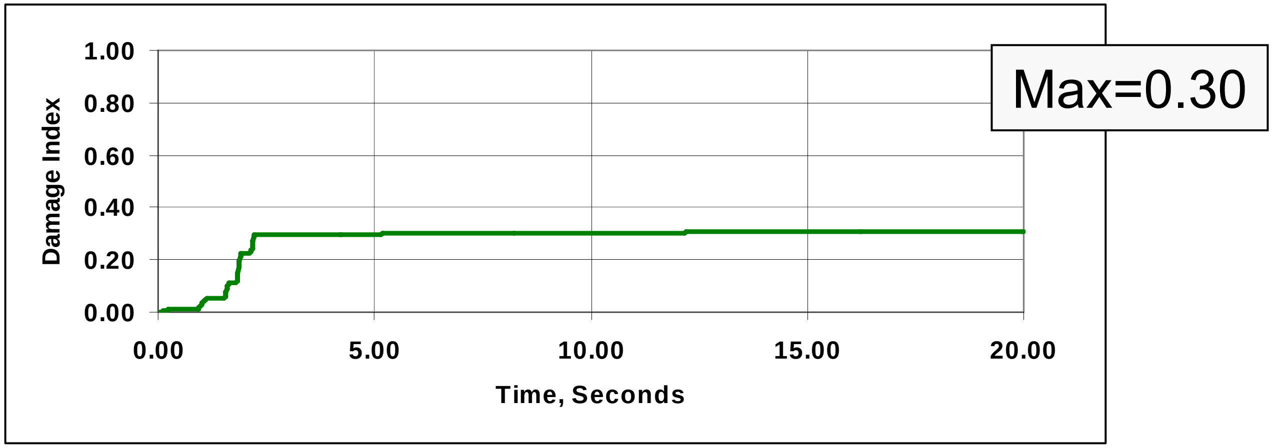
Energy and Damage Histories, 5% Damping



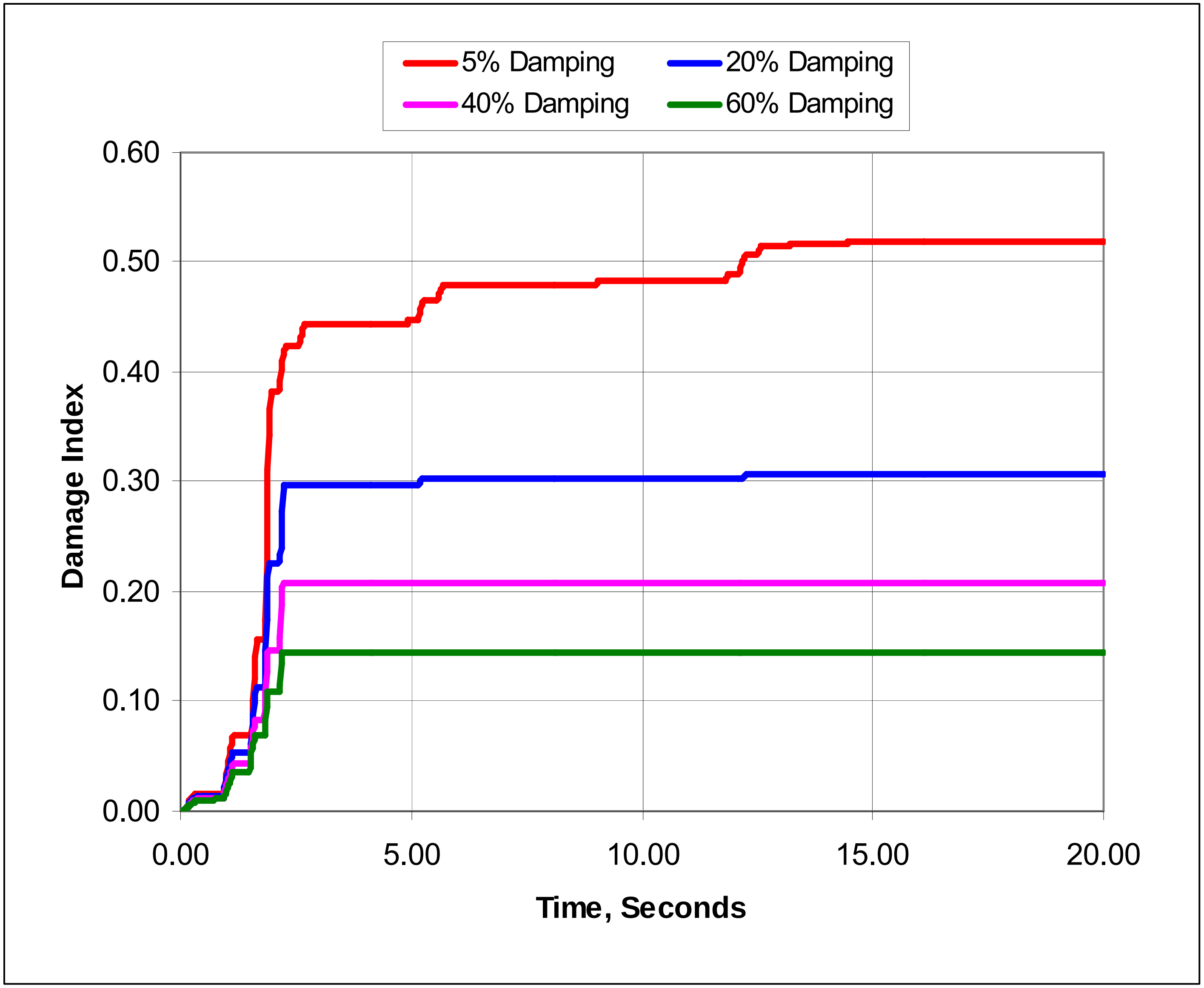
Analysis performed on NONLIN

Energy and Damage Histories, 20% Damping





Reduction in Damage with Increased Damping



Outline: Part I

- Objectives of Advanced Technology Systems and Effects on Seismic Response
- Distinction Between Natural and Added Damping
- Energy Distribution and Damage Reduction
- Classification of Passive Energy Dissipation Systems

Classification of Passive Energy Dissipation Systems

Velocity-Dependent Systems

- Viscous fluid or viscoelastic solid dampers
- May or may not add stiffness to structure

Displacement-Dependent Systems

- Metallic yielding or friction dampers
- Always adds stiffness to structure

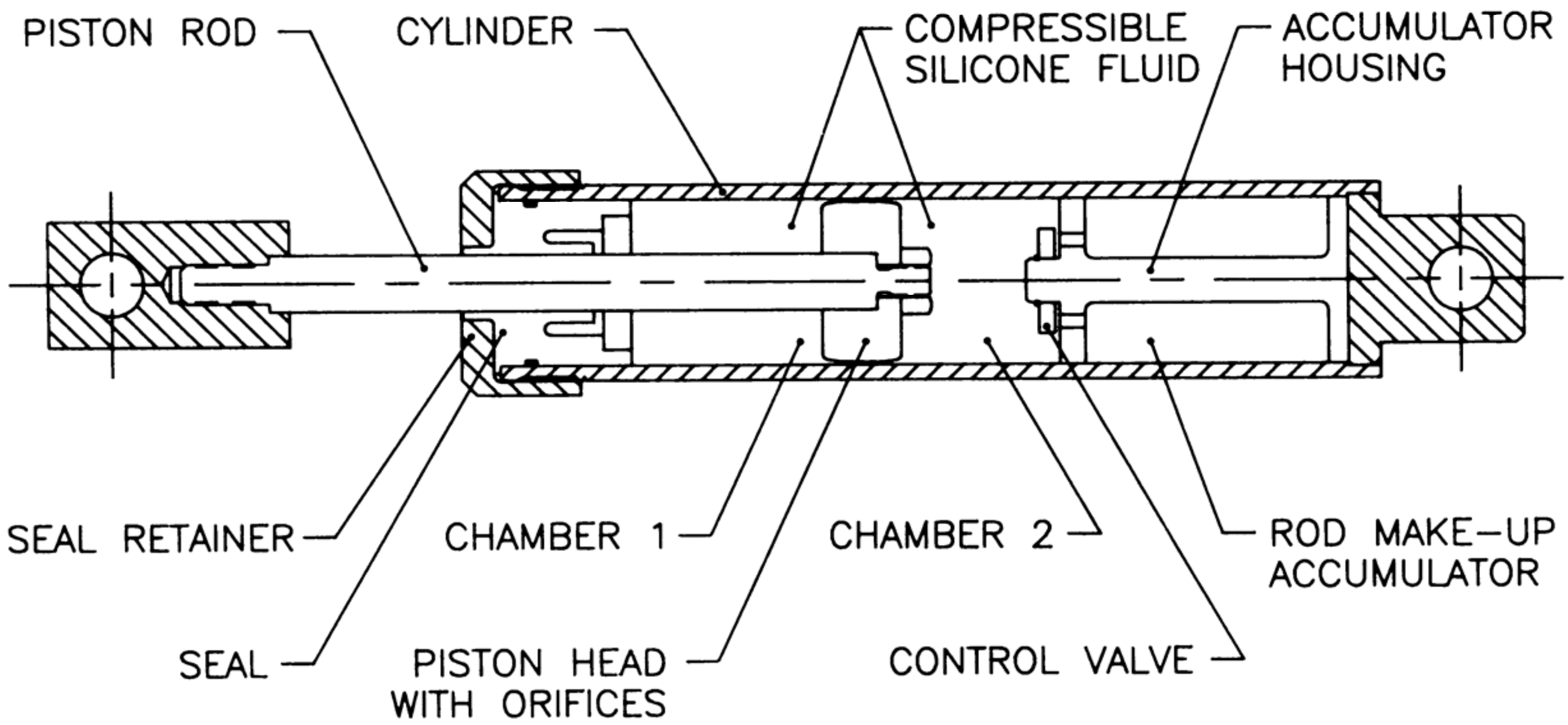
Other

- Re-centering devices (shape-memory alloys, etc.)
- Vibration absorbers (tuned mass dampers)

Outline: Part II

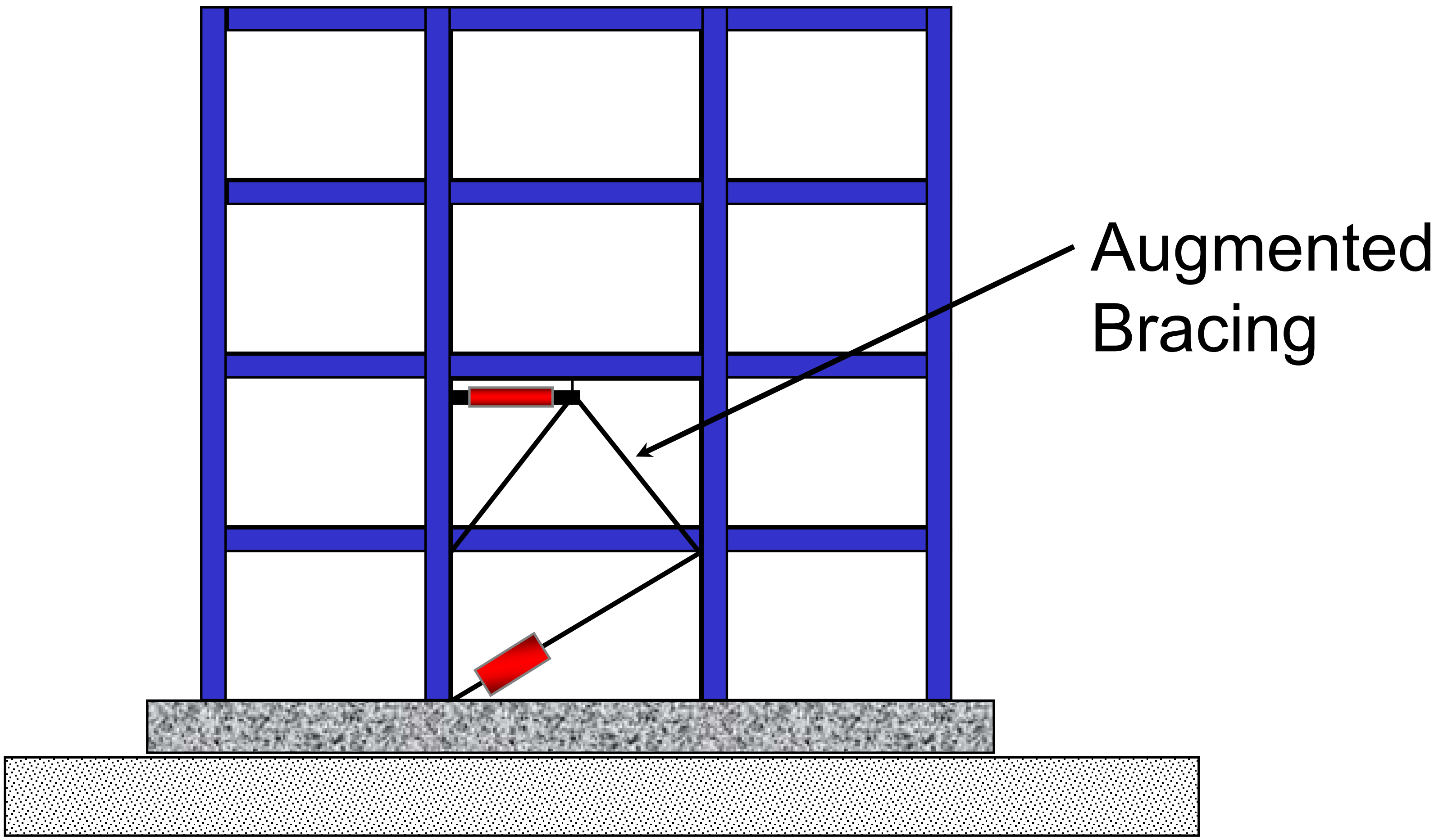
- Velocity-Dependent Damping Systems: Fluid Dampers and Viscoelastic Dampers
- Models for Velocity-Dependent Dampers
- Effects of Linkage Flexibility
- Displacement-Dependent Damping Systems: Steel Plate Dampers,Unbonded Brace Dampers, and Friction Dampers
- Concept of Equivalent Viscous Damping
- Modeling Considerations for Structures with Passive Damping Systems

Cross-Section of Viscous Fluid Damper

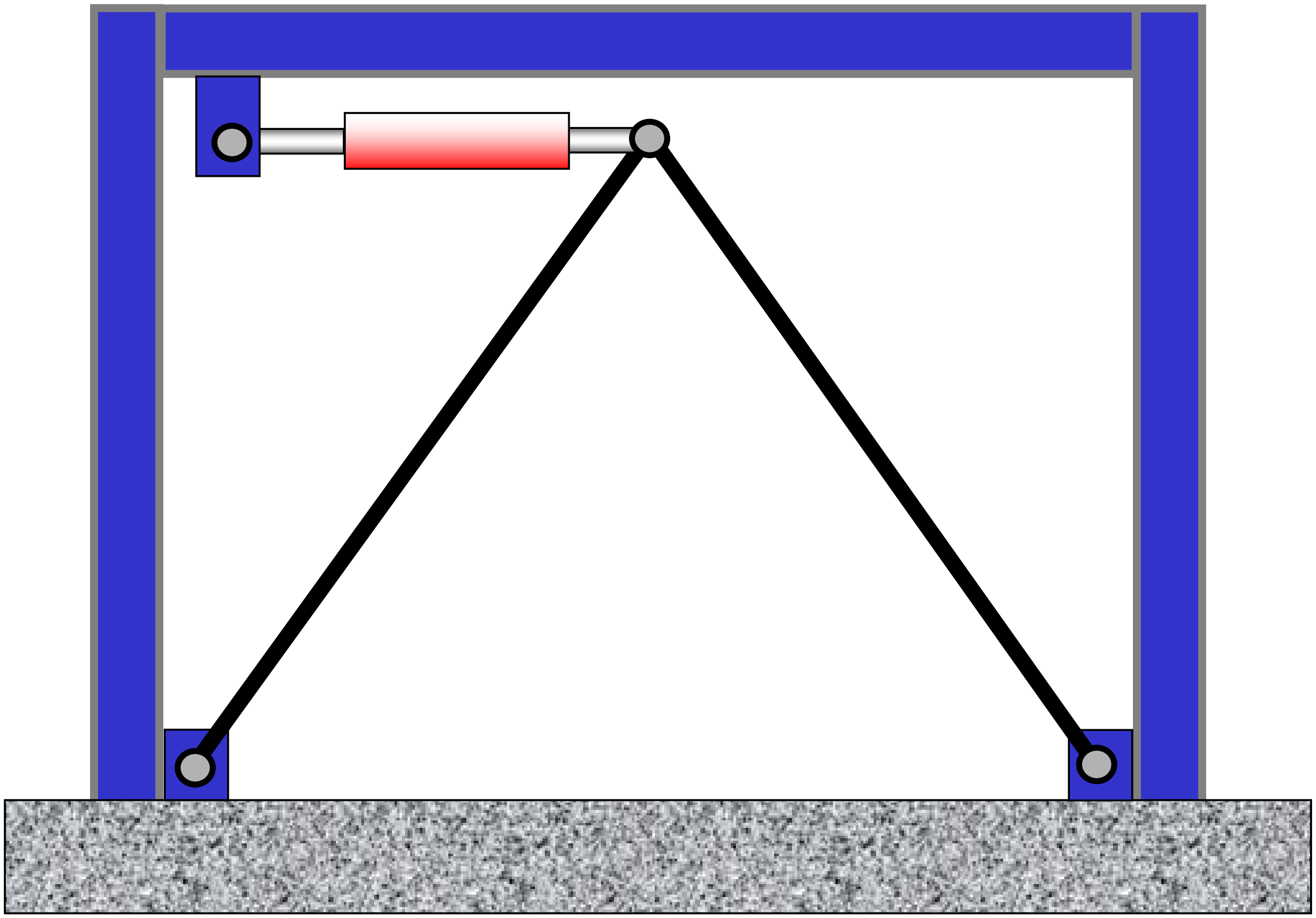


Source: Taylor Devices, Inc.

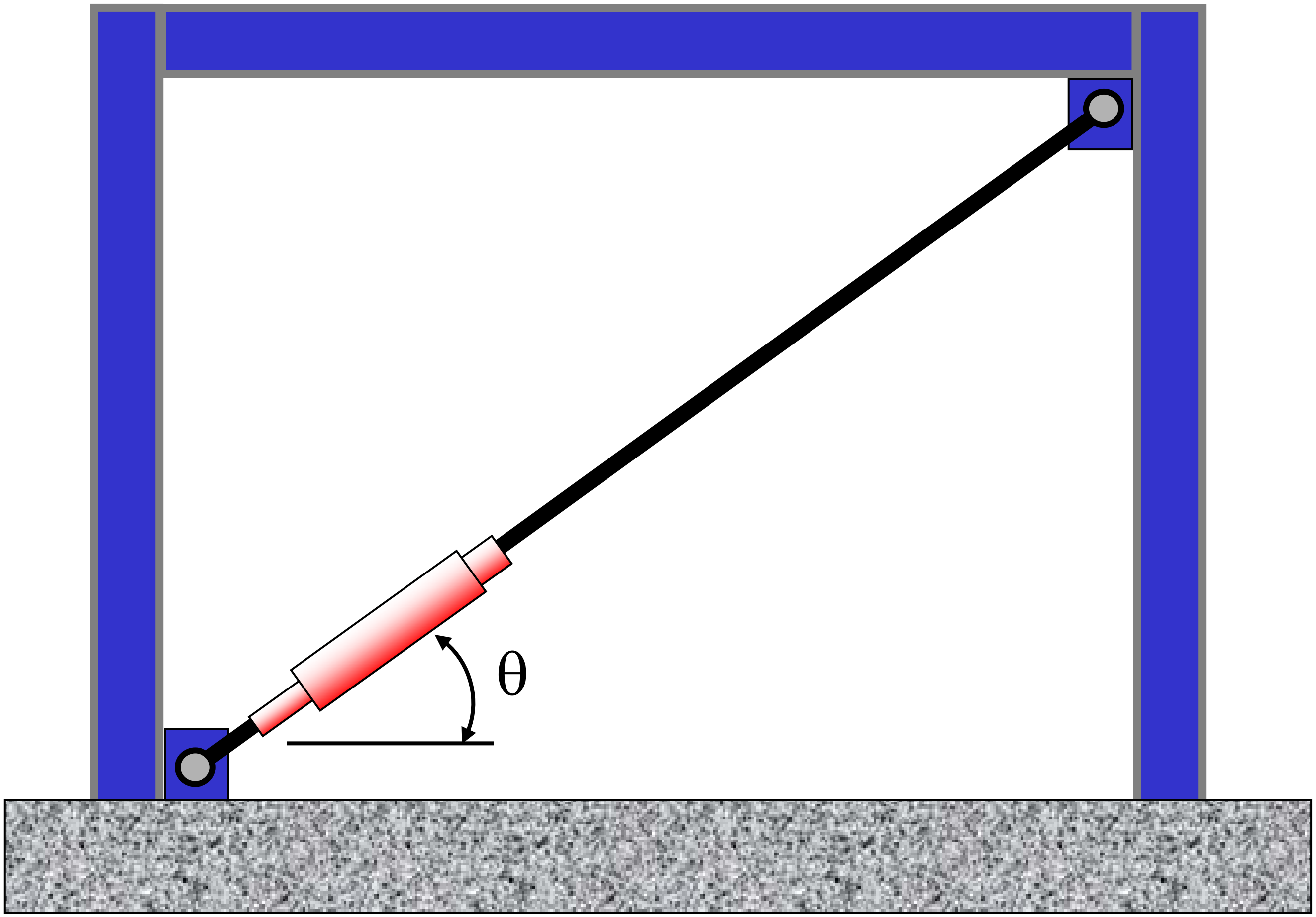
Possible Damper Placement Within Structure



Chevron Brace and Viscous Damper



Diagonally Braced Damping System



Fluid Dampers within Inverted Chevron Brace

Pacific Bell North Area Operation Center (911 Emergency Center)
Sacramento, California
(3-Story Steel-Framed Building Constructed in 1995)

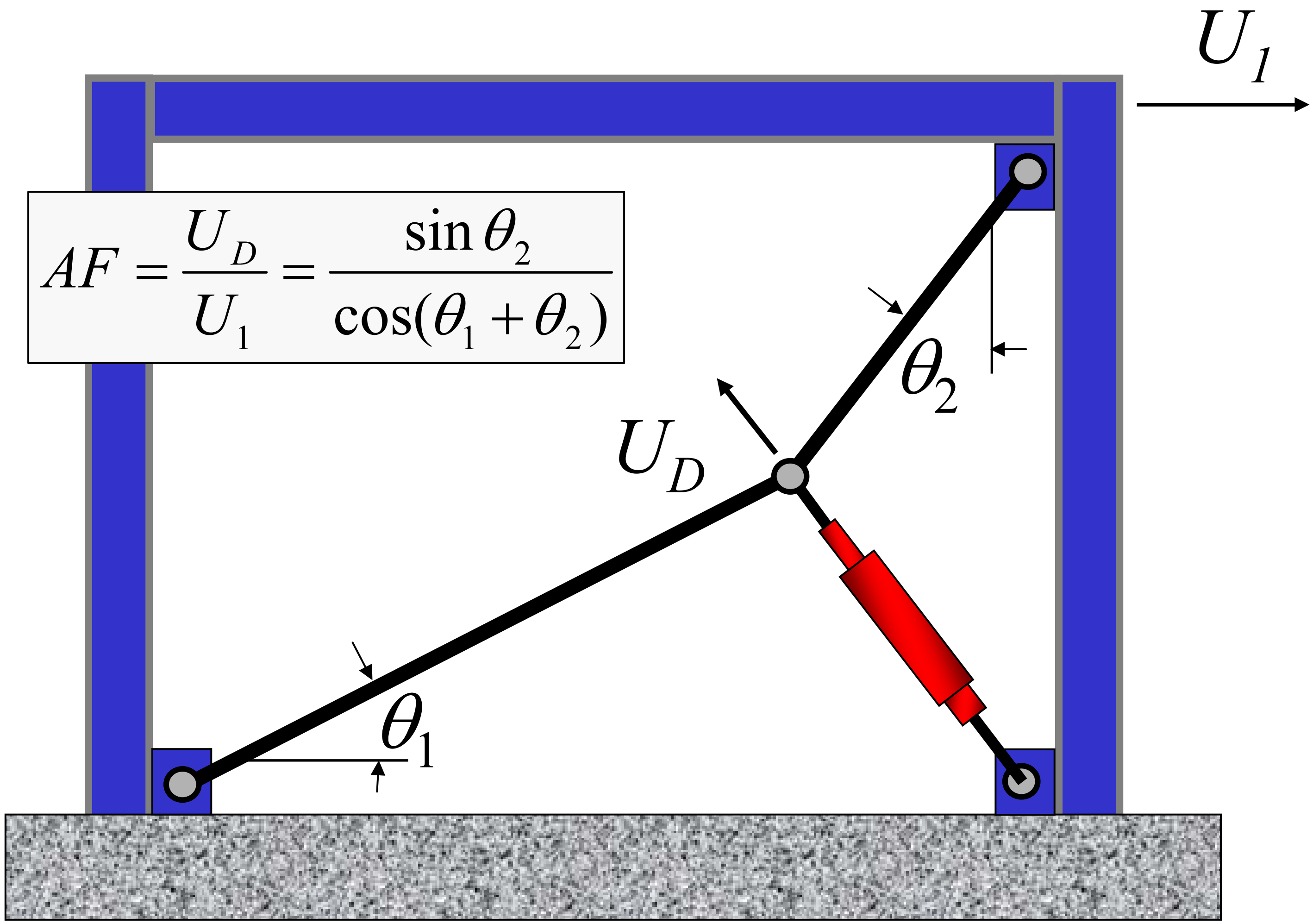
62 Dampers: 30 Kip Capacity, +/-2 in. Stroke

Fluid Damper within Diagonal Brace

Huntington Tower
Boston, MA

San Francisco State
Office Building
San Francisco, CA

Toggle Brace Damping System



$$AF = \frac{U_D}{U_1} = \frac{\sin\theta_2}{\cos(\theta_1 + \theta_2)}$$

Toggle Brace Deployment

Huntington Tower, Boston, MA

- **New 38-story steel-framed building**
- **100 direct-acting and toggle-brace dampers**
- **1300 kN (292 kips), +/- 101 mm (+/- 4 in.)**
- **Dampers suppress wind-induced vibration**

Harmonic Behavior of Fluid Damper

$$u(t) = u_0 \sin(\overline{\omega} t)$$

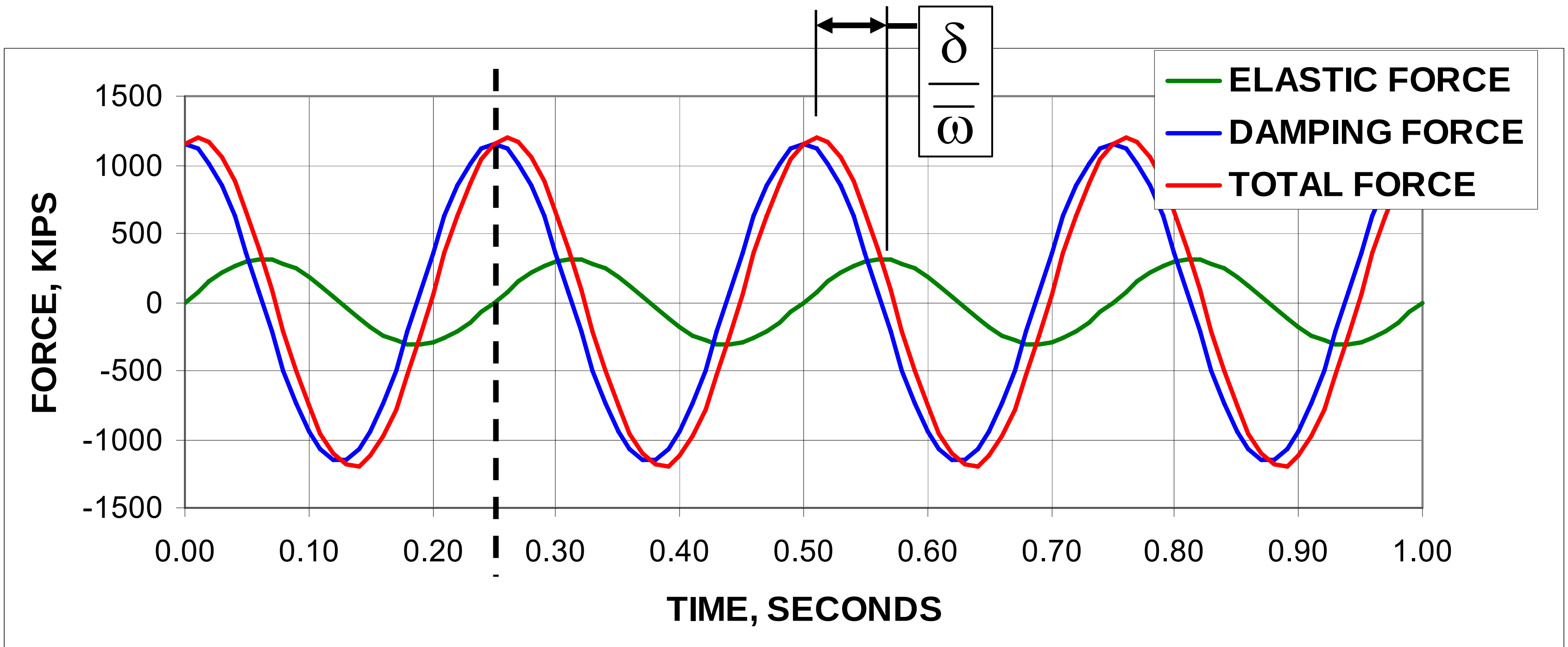
Imposed Motion

Loading Frequency

Phase Angle (Lag)

Total Force

$$P(t) = P_0 \sin(\overline{\omega} t)\cos(\delta) + P_0 \cos(\overline{\omega} t)\sin(\delta)$$



Note: Damping force 90º out-of-phase with elastic force.

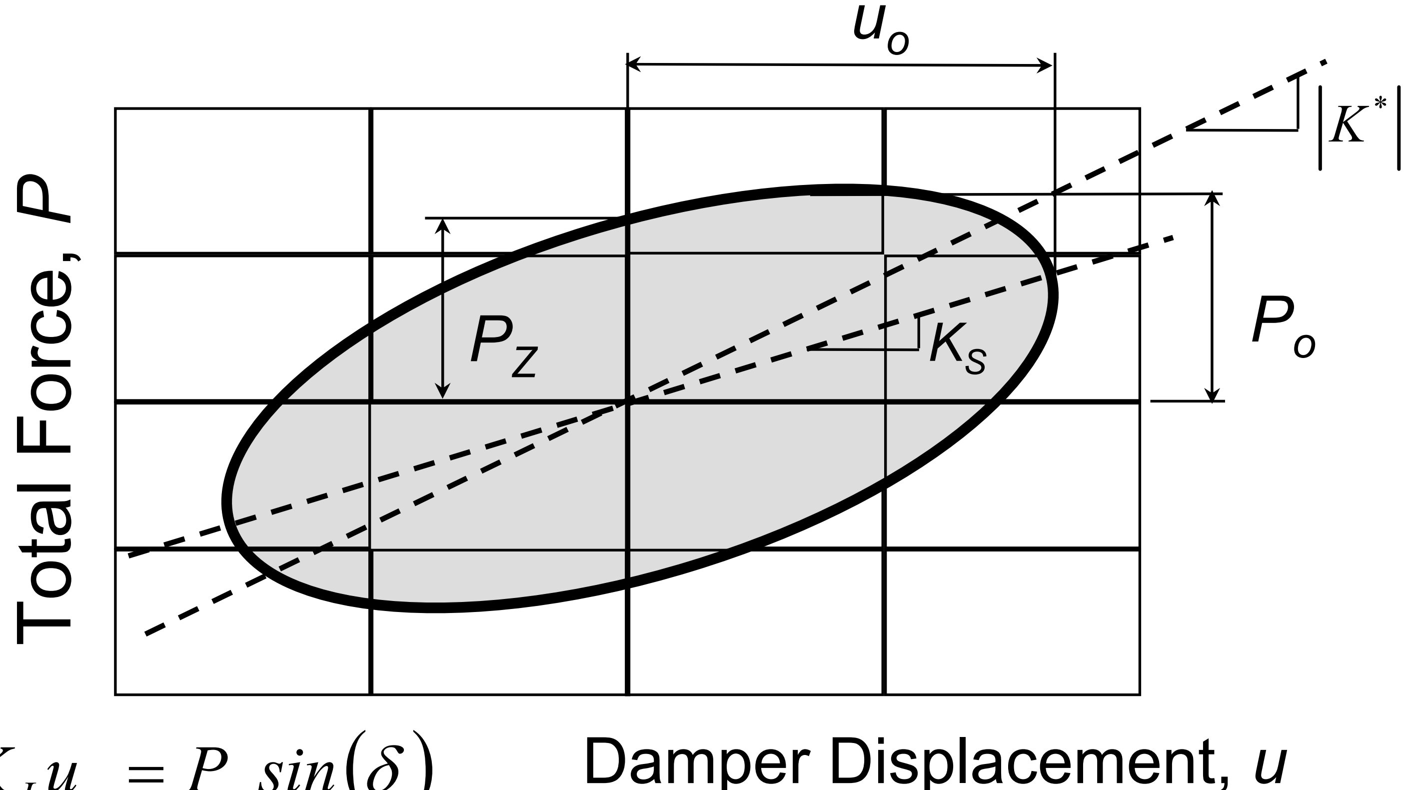
$$P(t) = K_S u(t) + C\dot{u}(t)$$

$$K_S = \frac{P_0}{u_0}\cos(\delta)$$
Storage Stiffness

$$K_L = \frac{P_0}{u_0}\sin(\delta)$$
Loss Stiffness

$$C = \frac{K_L}{\overline{\omega}}$$
Damping Coeff.

$$\delta = \sin^{-1}\left(\frac{P_Z}{P_0}\right)$$
Phase Angle



$$P_Z = K_L u_o = P_o \sin(\delta)$$

$$E_D = \pi P_Z u_o = \pi P_o u_o \sin(\delta)$$

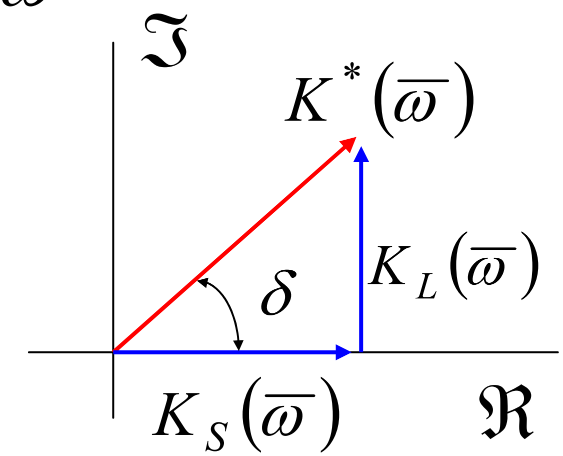
Frequency-Domain Force-Displacement Relation

$$P(t) = K_S u(t) + C\dot{u}(t)$$

Apply Fourier Transform:

$$P(\overline{\omega}) = K_S u(\overline{\omega}) + K_L i\overline{\omega} u(\overline{\omega}) / \overline{\omega}$$

$$P(\overline{\omega}) = \left[K_S + iK_L\right] u(\overline{\omega})$$



Complex Stiffness:

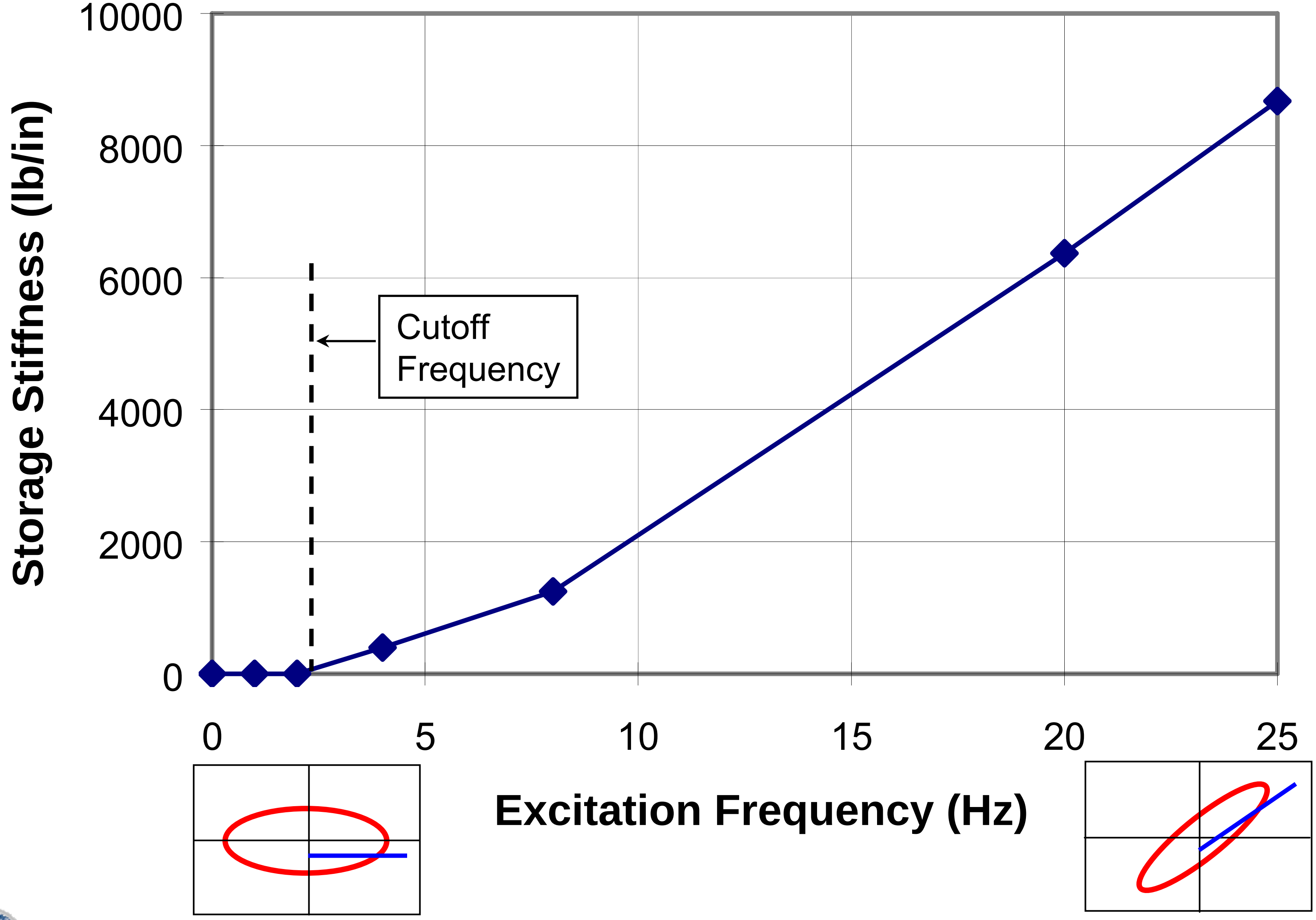
$$K^*(\overline{\omega}) = \frac{P(\overline{\omega})}{u(\overline{\omega})}$$

$$P(\overline{\omega}) = K^*(\overline{\omega}) u(\overline{\omega}) \longleftarrow$$

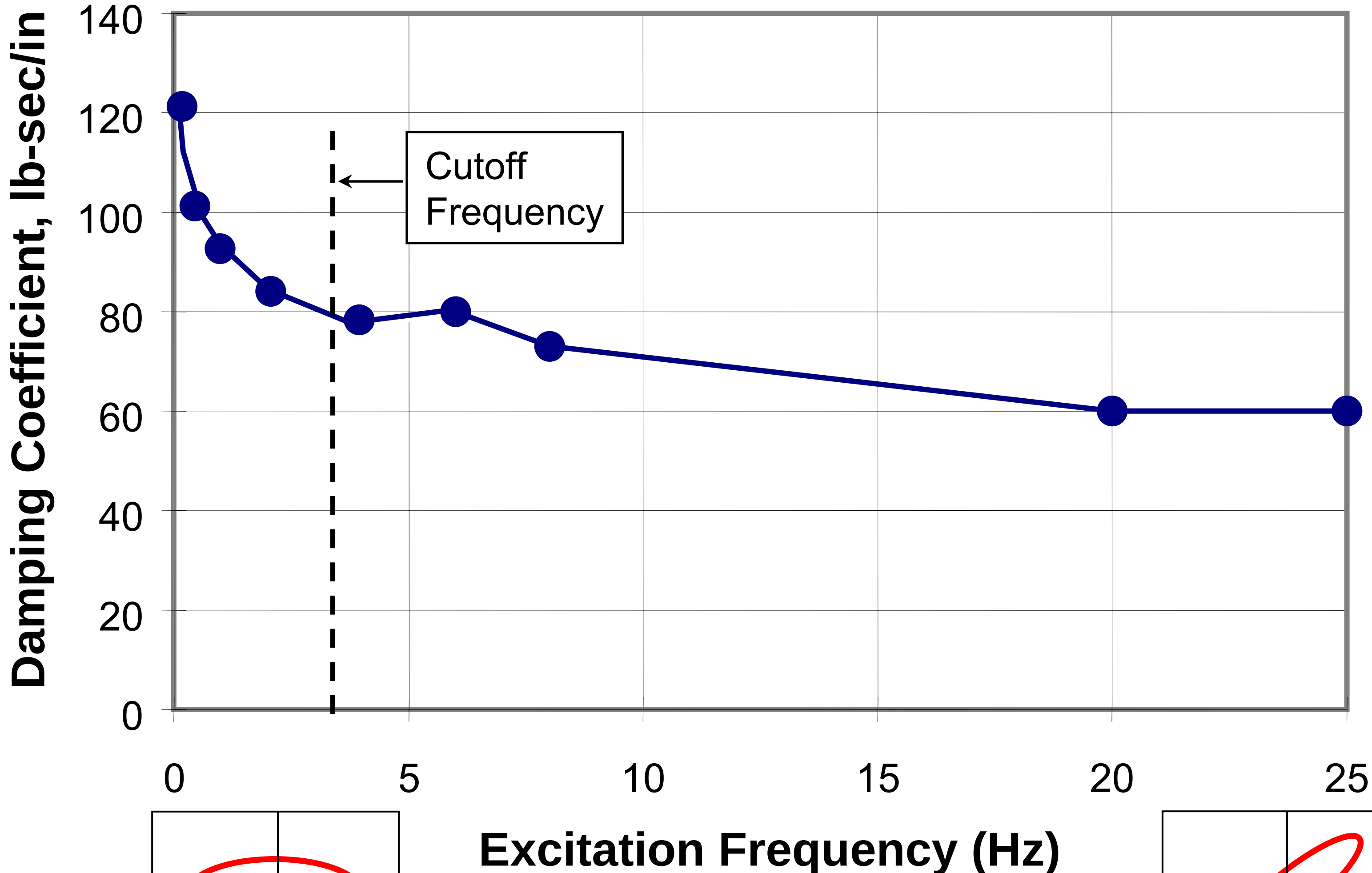
Compact Force-Displ. Relation for Viscoelastic Dampers

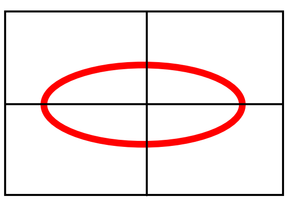
Note: $\Re\left(K^*\right) = K_S$ and $\Im\left(K^*\right) = K_L$

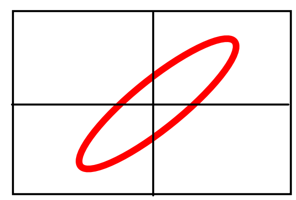
Dependence of Storage Stiffness on Frequency for Typical "Single-Ended" Fluid Damper



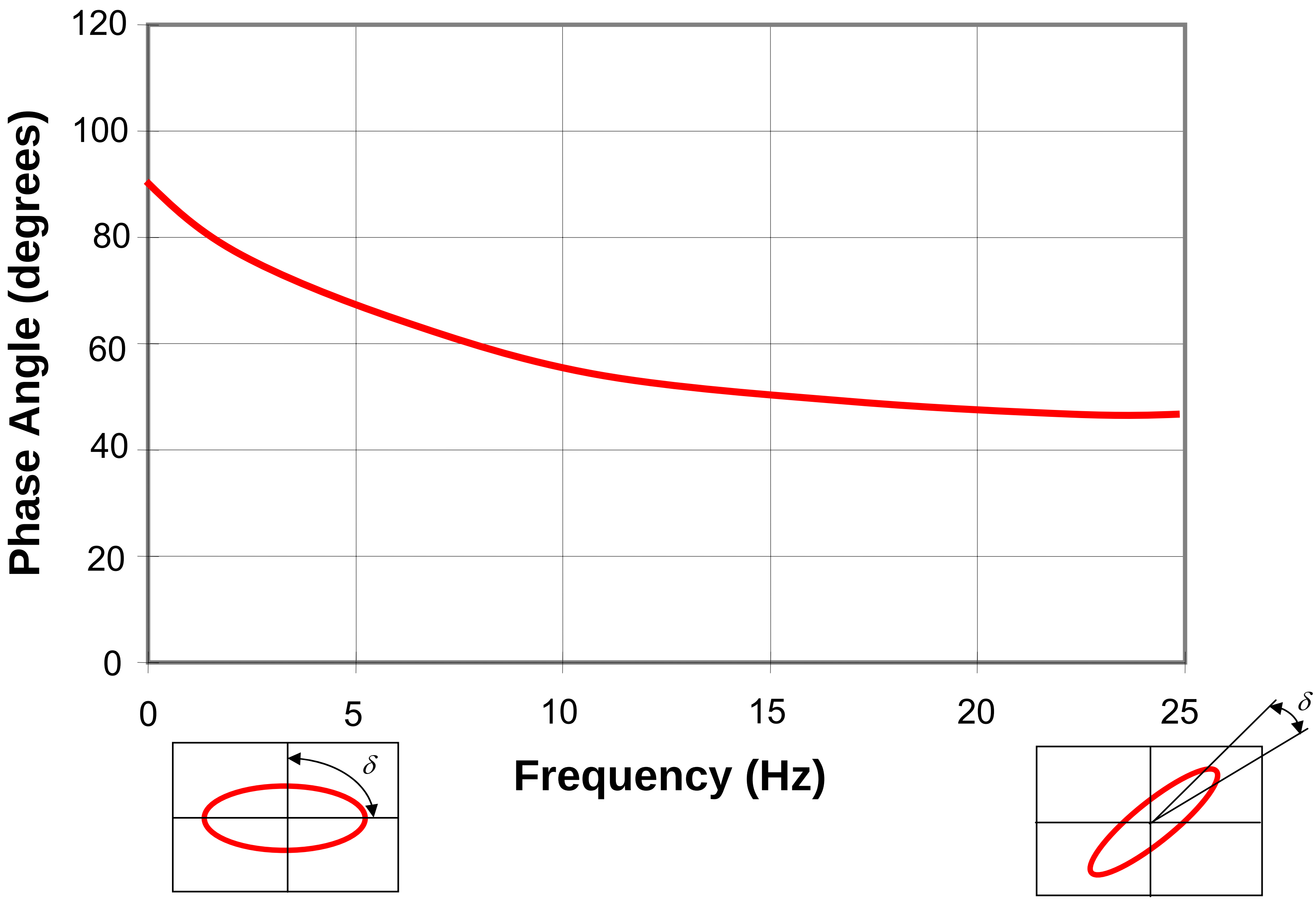
Dependence of Damping Coefficient on Frequency for Typical "Single-Ended" Fluid Damper



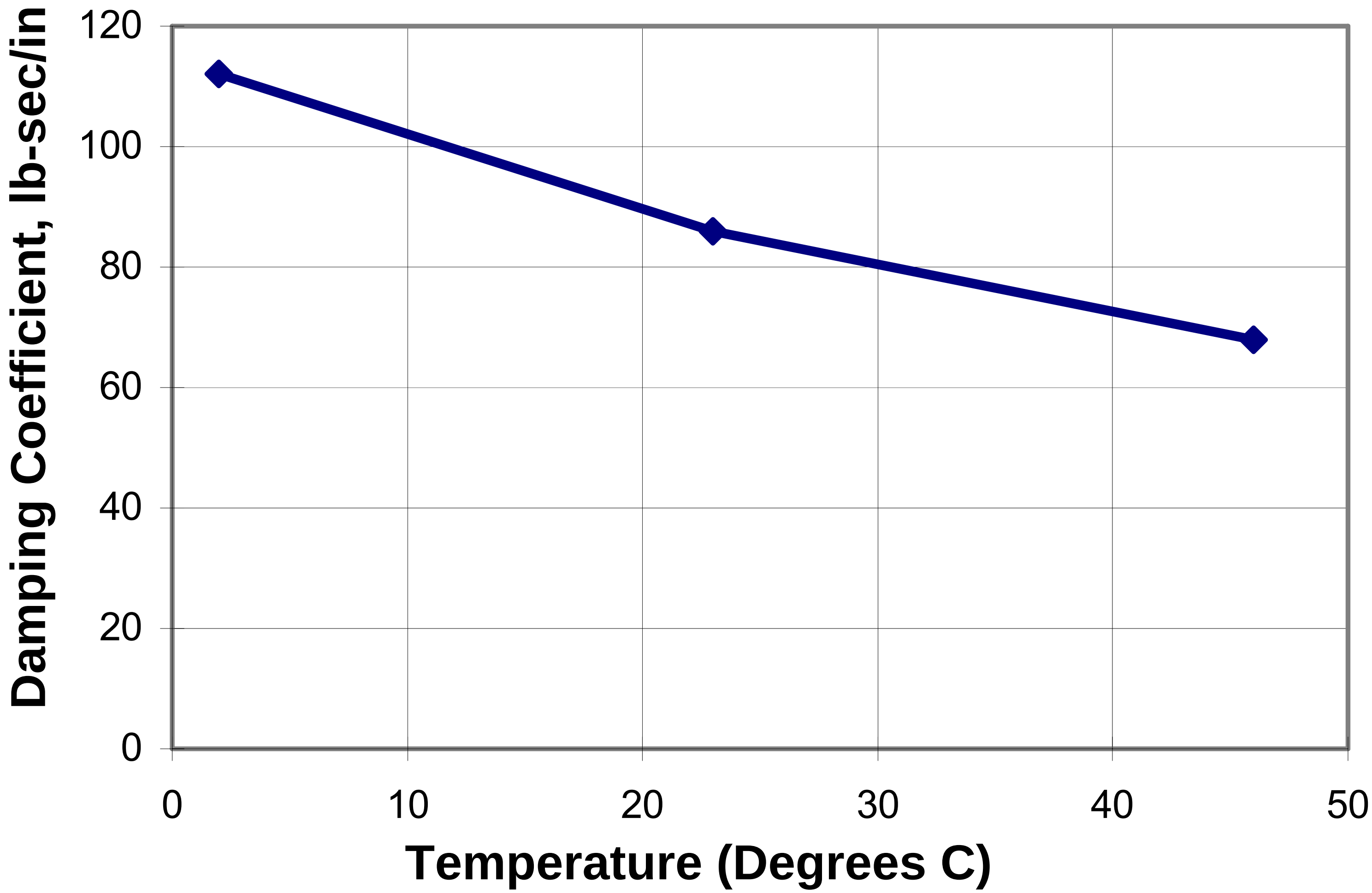




Dependence of Phase Angle on Frequency for Typical "Single-Ended" Fluid Damper



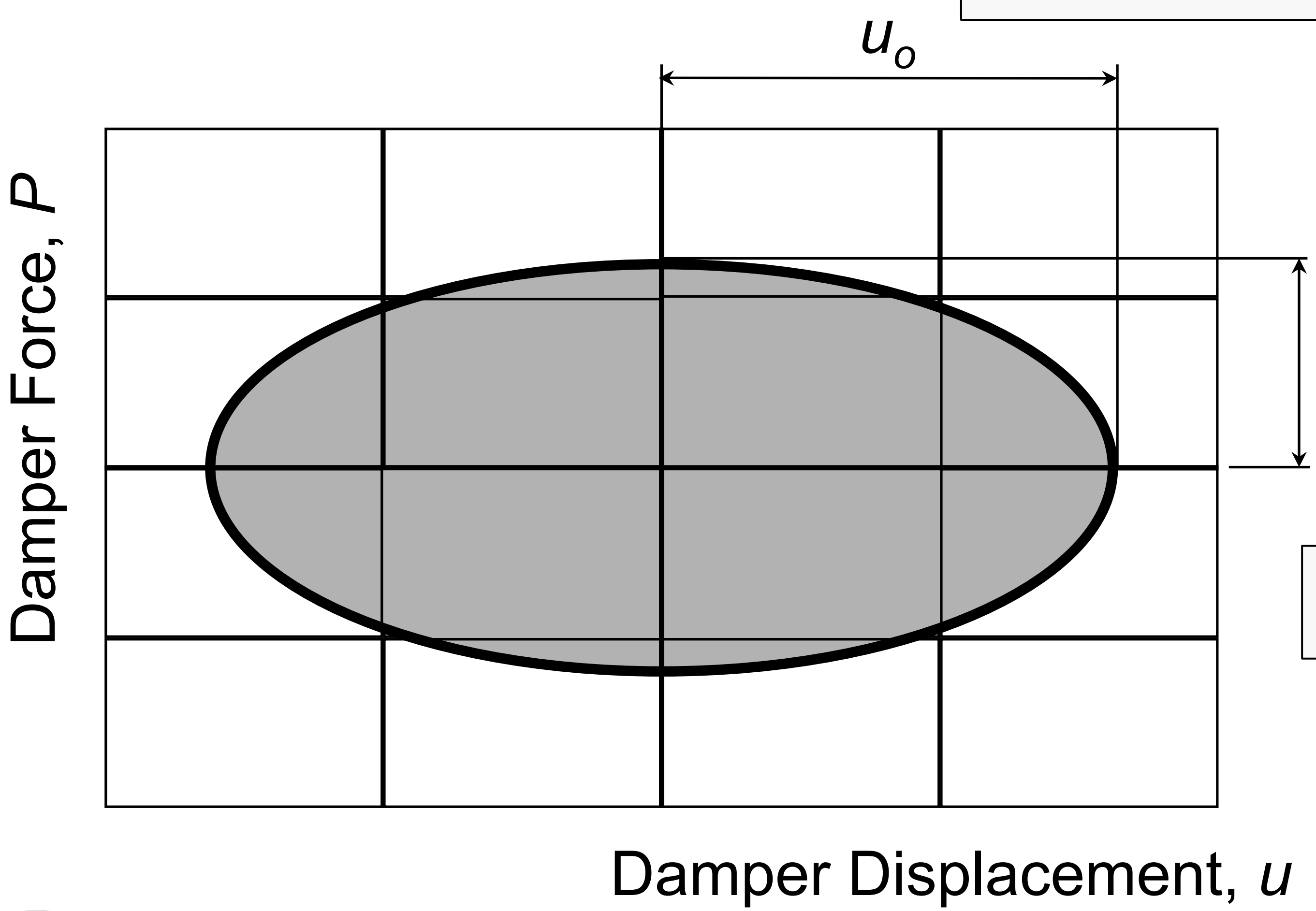
Dependence of Damping Coefficient on Temperature for Typical Fluid Damper



Behavior of Fluid Damper with Zero Storage Stiffness

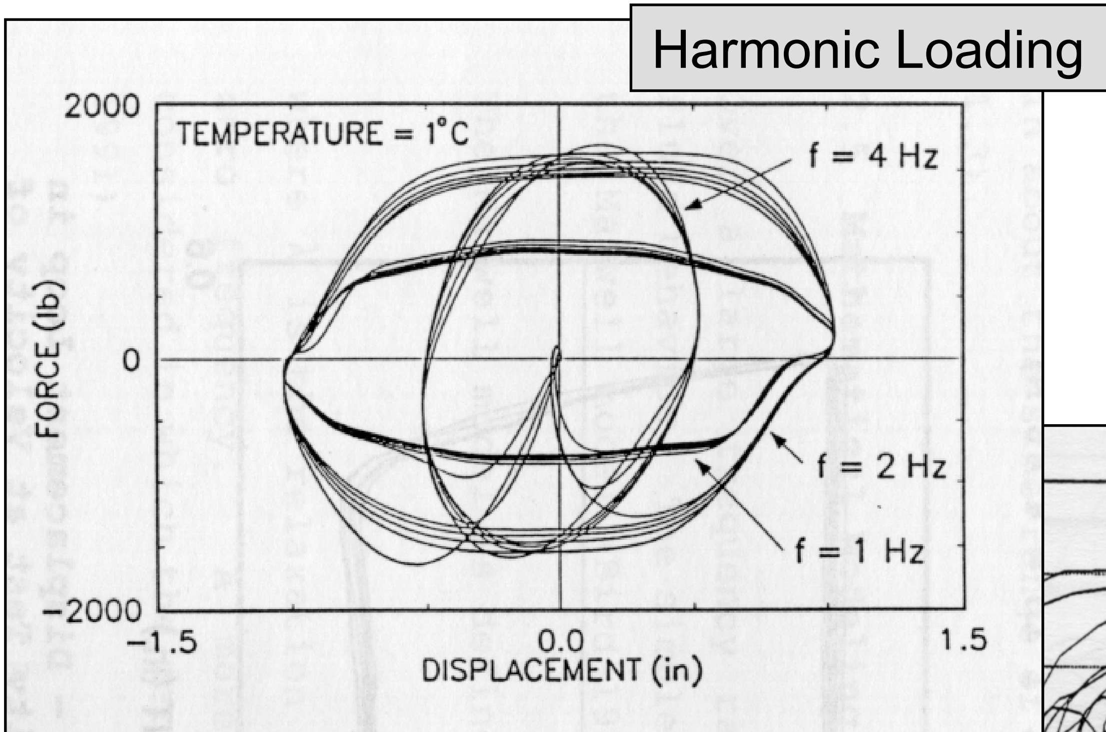
$$K_S = 0 \Rightarrow \delta = 90^o$$

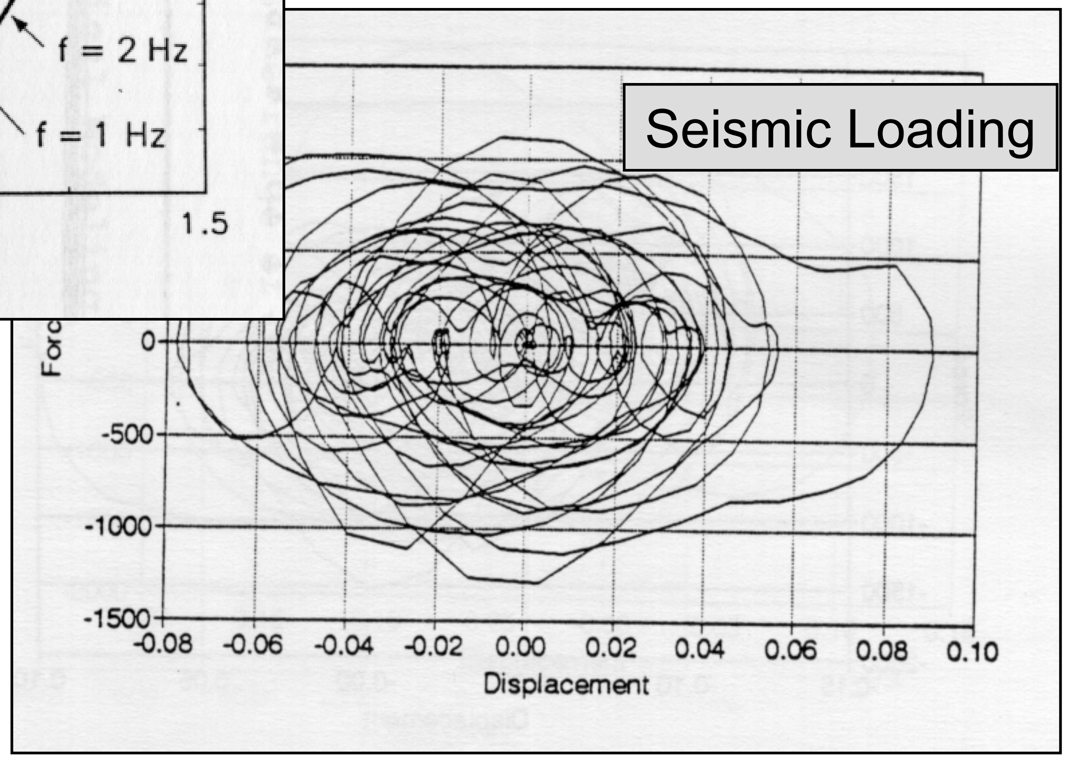
$$P(t) = C\dot{u} = \frac{K_L}{\overline{\omega}}\dot{u}$$



$$E_d = \pi P_o u_o$$

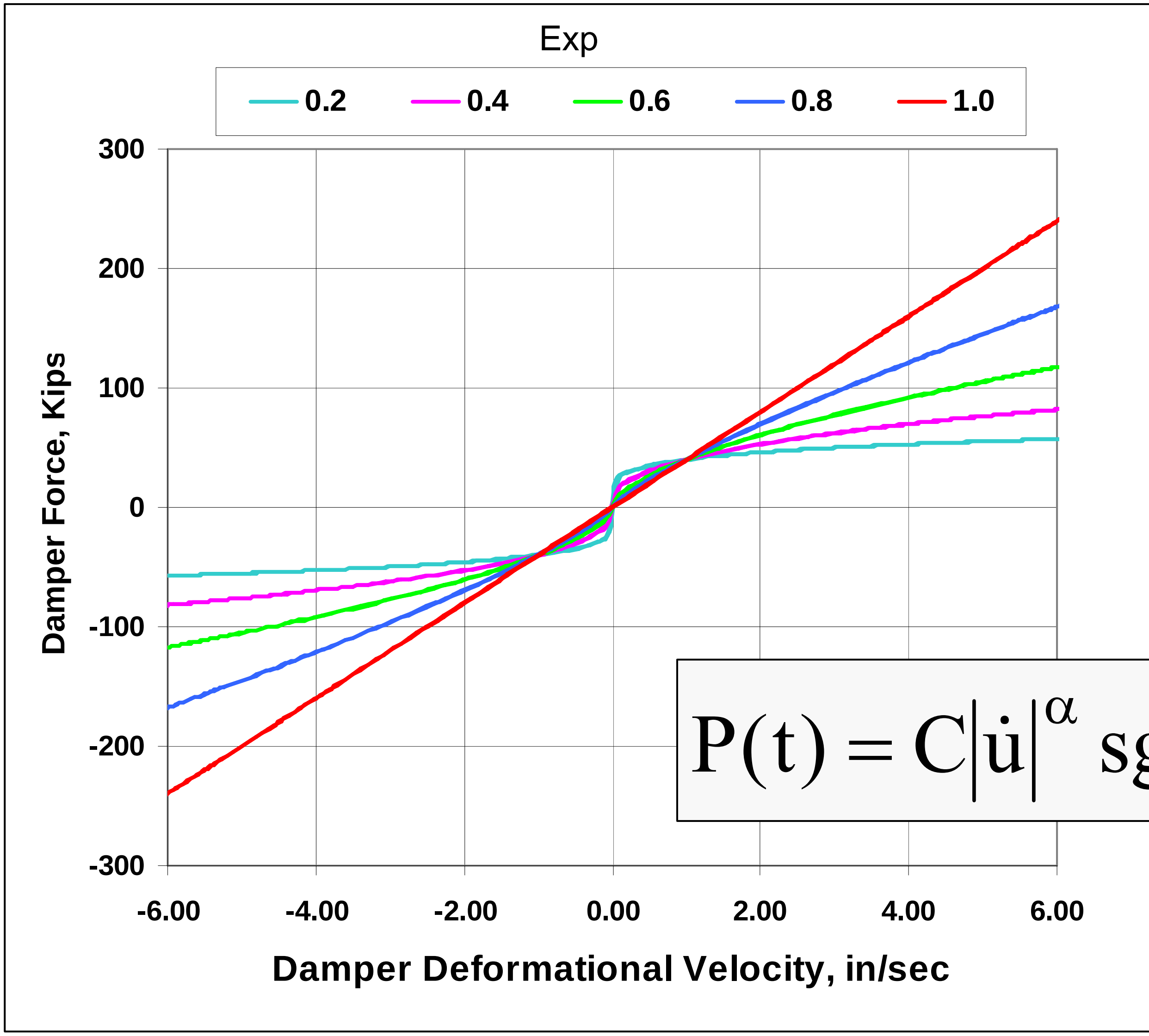
Actual Hysteretic Behavior of Fluid Damper





Source:
Constantinou and Symans (1992)

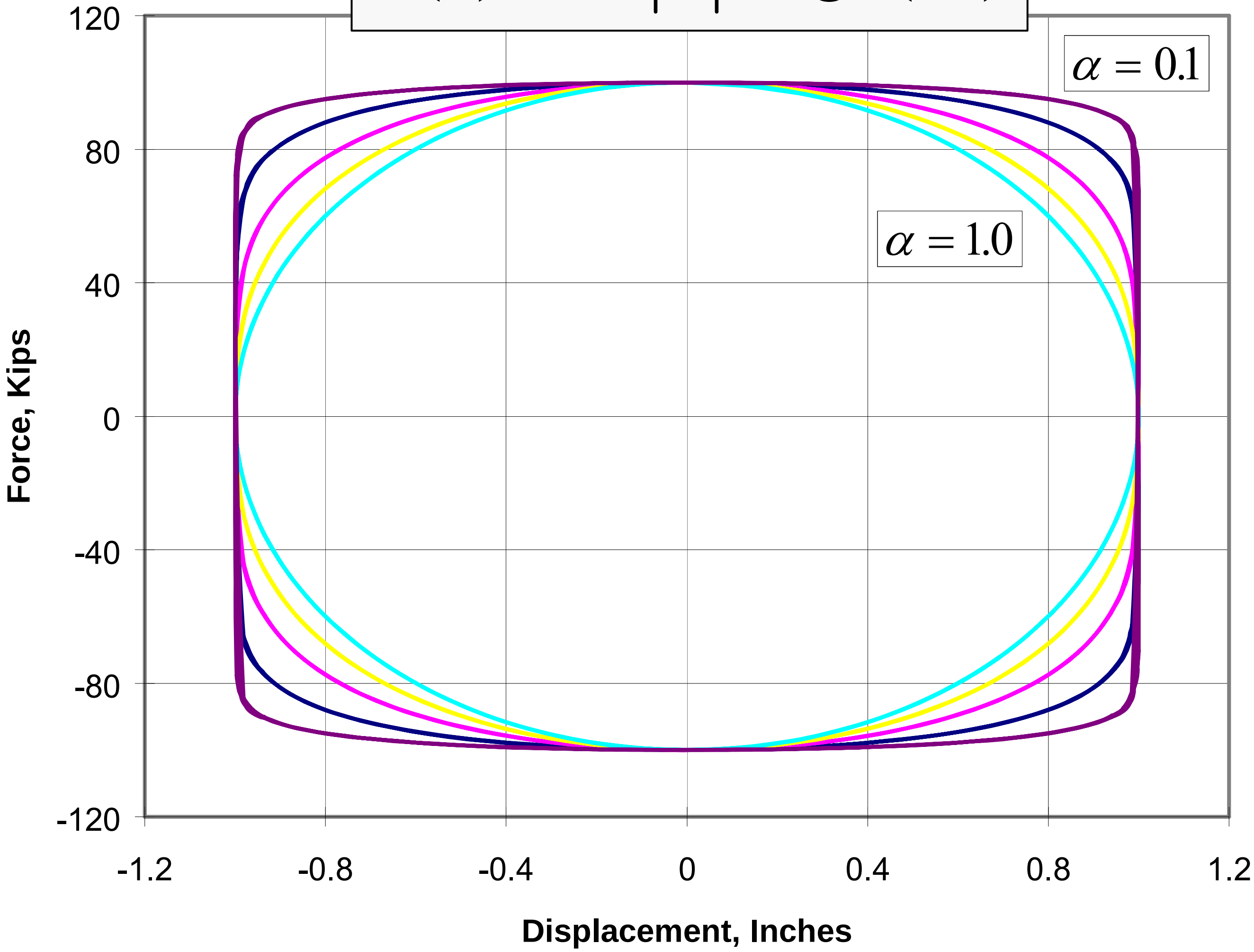
Force-Velocity Behavior of Viscous Fluid Damper



$$P(t) = C|\dot{u}|^{\alpha}\,\mathrm{sgn}(\dot{u})$$

Nonlinear Fluid Dampers

$$P(t) = C|\dot{u}|^{\alpha}\,\mathrm{sgn}(\dot{u})$$



Energy Dissipated Per Cycle for Linear and Nonlinear Viscous Fluid Dampers

Linear Damper: $$E_D = \pi P_o u_o$$

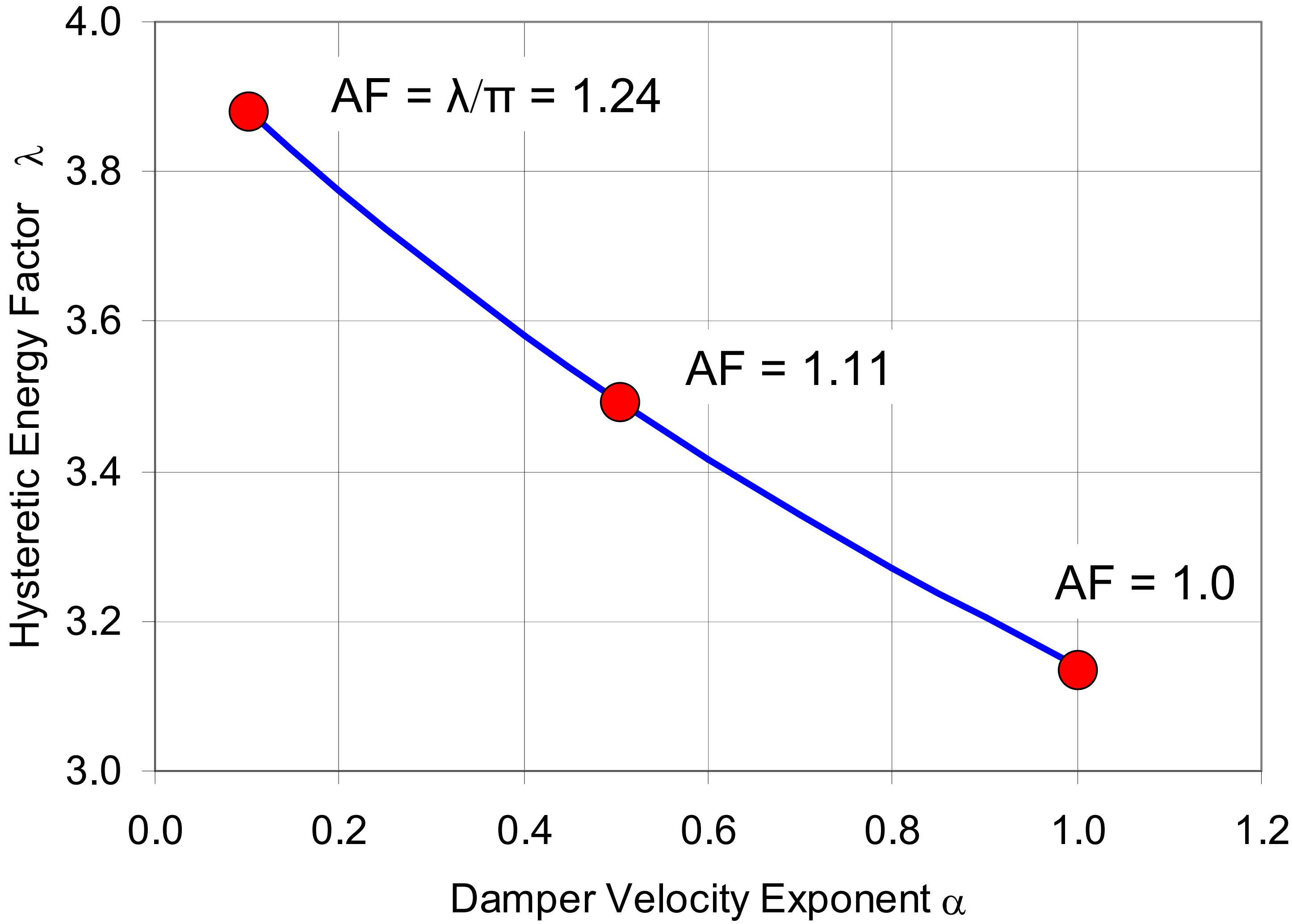
Hysteretic Energy Factor

Nonlinear Damper: $$E_D = \lambda P_o u_o$$

$$\lambda = 4 \times 2^{\alpha} \frac{\Gamma^2\left(1+\frac{\alpha}{2}\right)}{\Gamma(2+\alpha)}$$

Γ = Gamma Function

Relationship Between λ and α for Viscous Fluid Damper

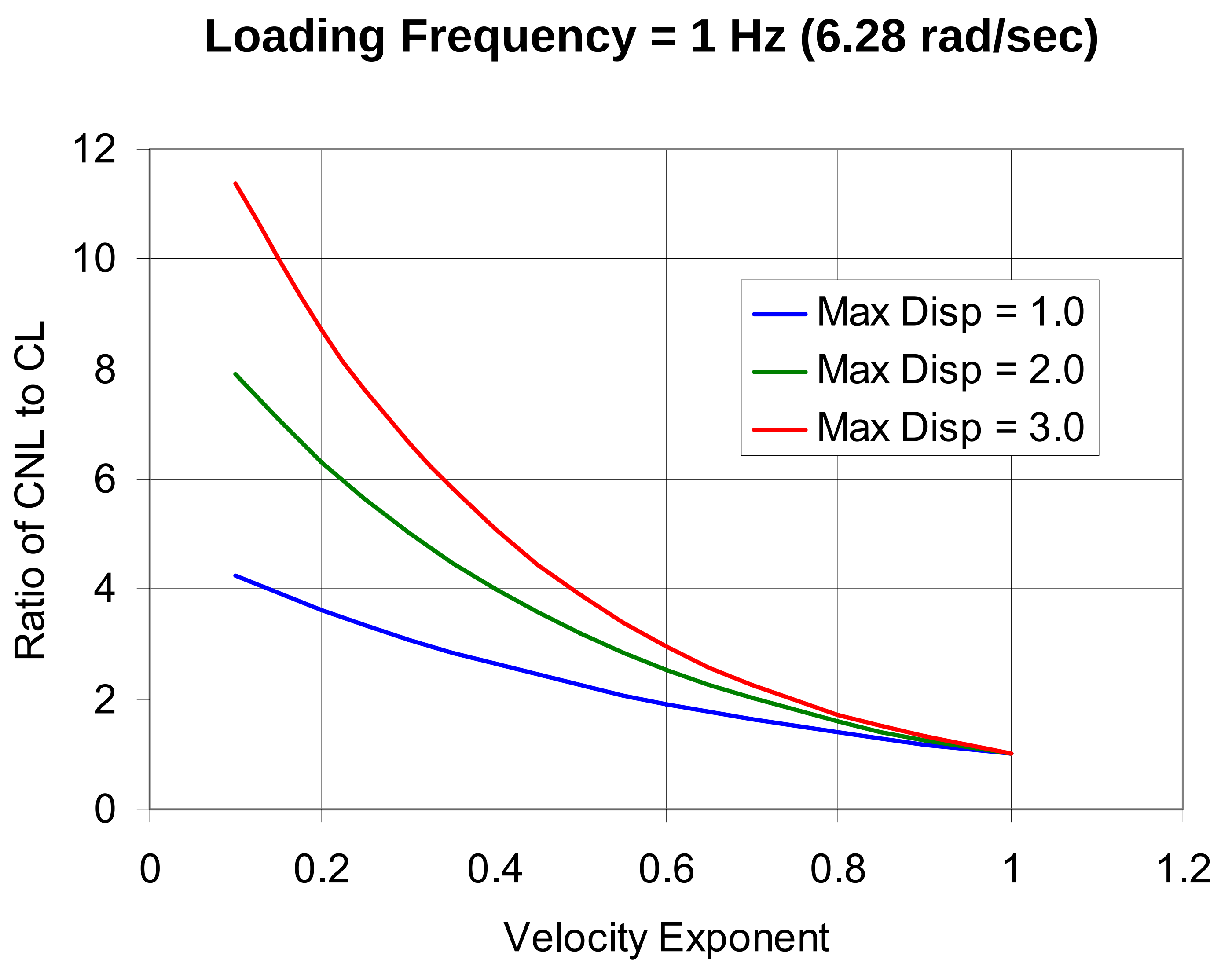


Relationship Between Nonlinear and Linear Damping Coefficient for Equal Energy Dissipation Per Cycle

$$\frac{C_{NL}}{C_L} = \frac{\pi}{\lambda}(u_o\overline{\omega})^{1-\alpha}$$

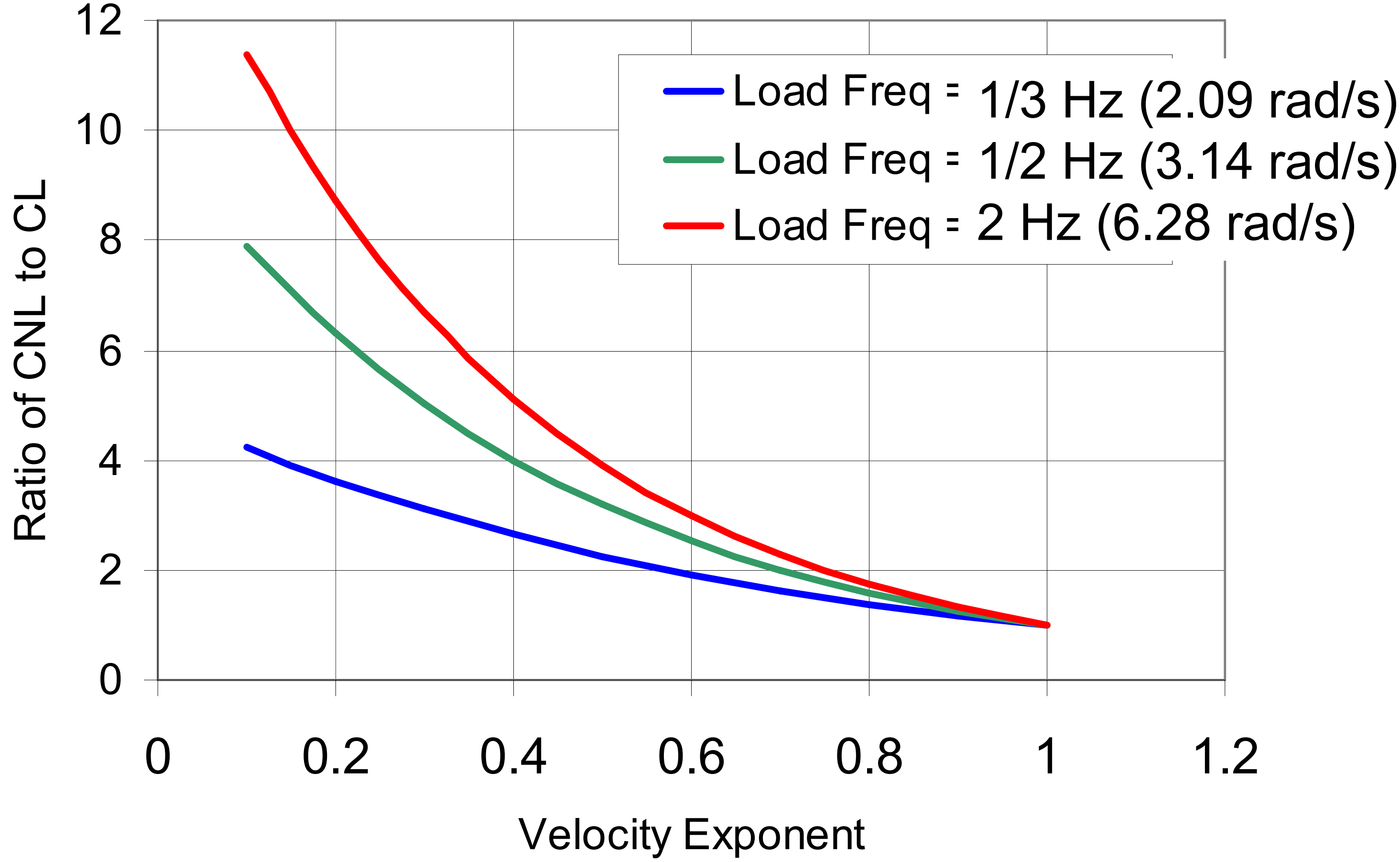
Note: Ratio is frequency- and displacement-dependent and is therefore meaningful only for steady-state harmonic response.

Ratio of Nonlinear Damping Coefficient to Linear Damping Coefficient (For a Given Loading Frequency)

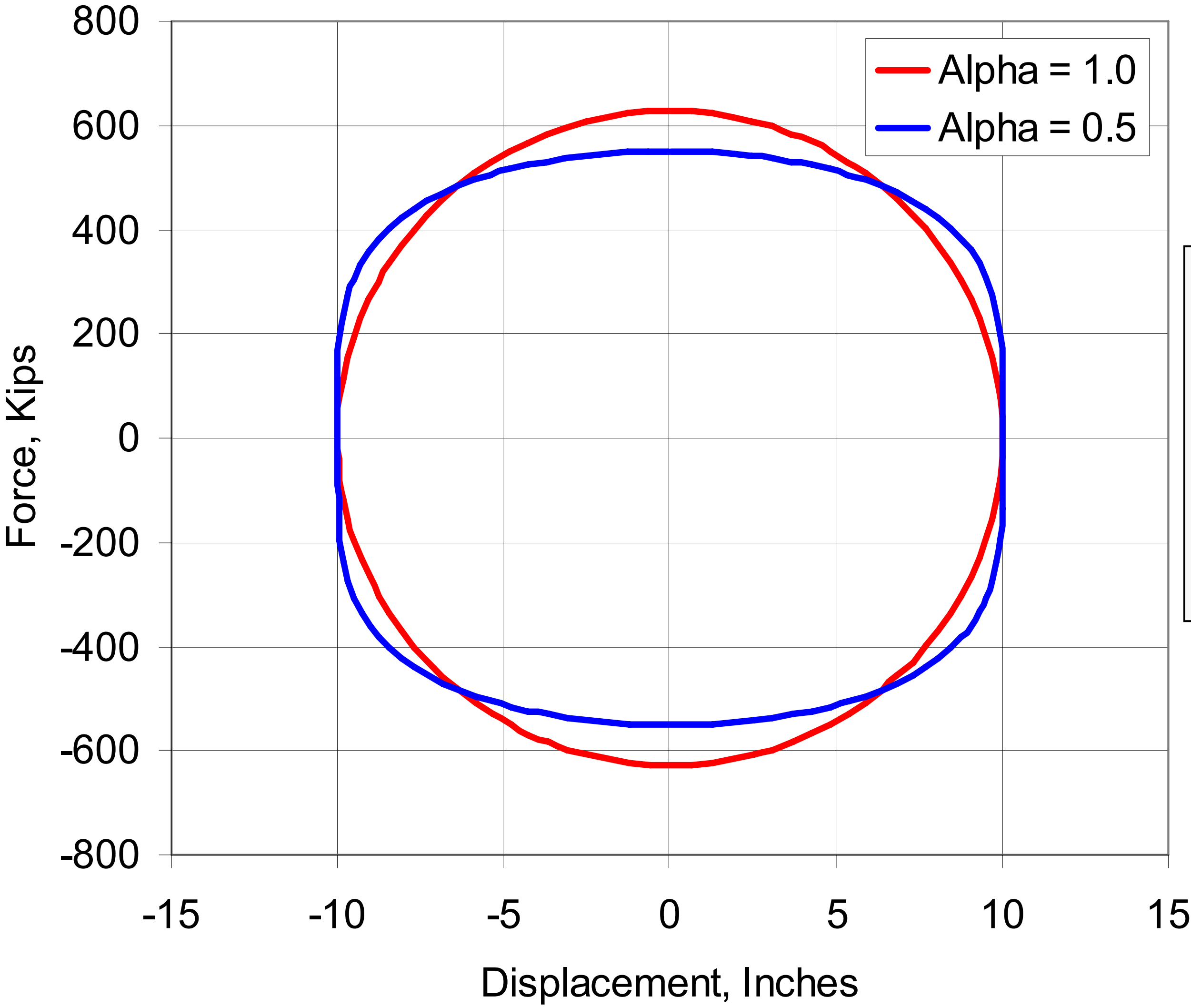


Ratio of Nonlinear Damping Constant to Linear Damping Constant (For a Given Maximum Displacement)

Maximum Displacement = 1



Example of Linear vs Nonlinear Damping



Frequency = 1 Hz (6.28 rad/s)
Max. Disp. = 10.0 in.

λ = 3.58

C_{Linear} = 10.0 k-sec/in
$C_{Nonlinear}$ = 69.5 k-sec$^{0.5}$/in$^{0.5}$

Recommendations Related to Nonlinear Viscous Dampers

- Do NOT attempt to linearize the problem when nonlinear viscous dampers are used. Perform the analysis with discrete nonlinear viscous dampers.

- Do NOT attempt to calculate effective damping in terms of a damping ratio (ξ) when using nonlinear viscous dampers.

- DO NOT attempt to use a free vibration analysis to determine equivalent viscous damping when nonlinear viscous dampers are used.

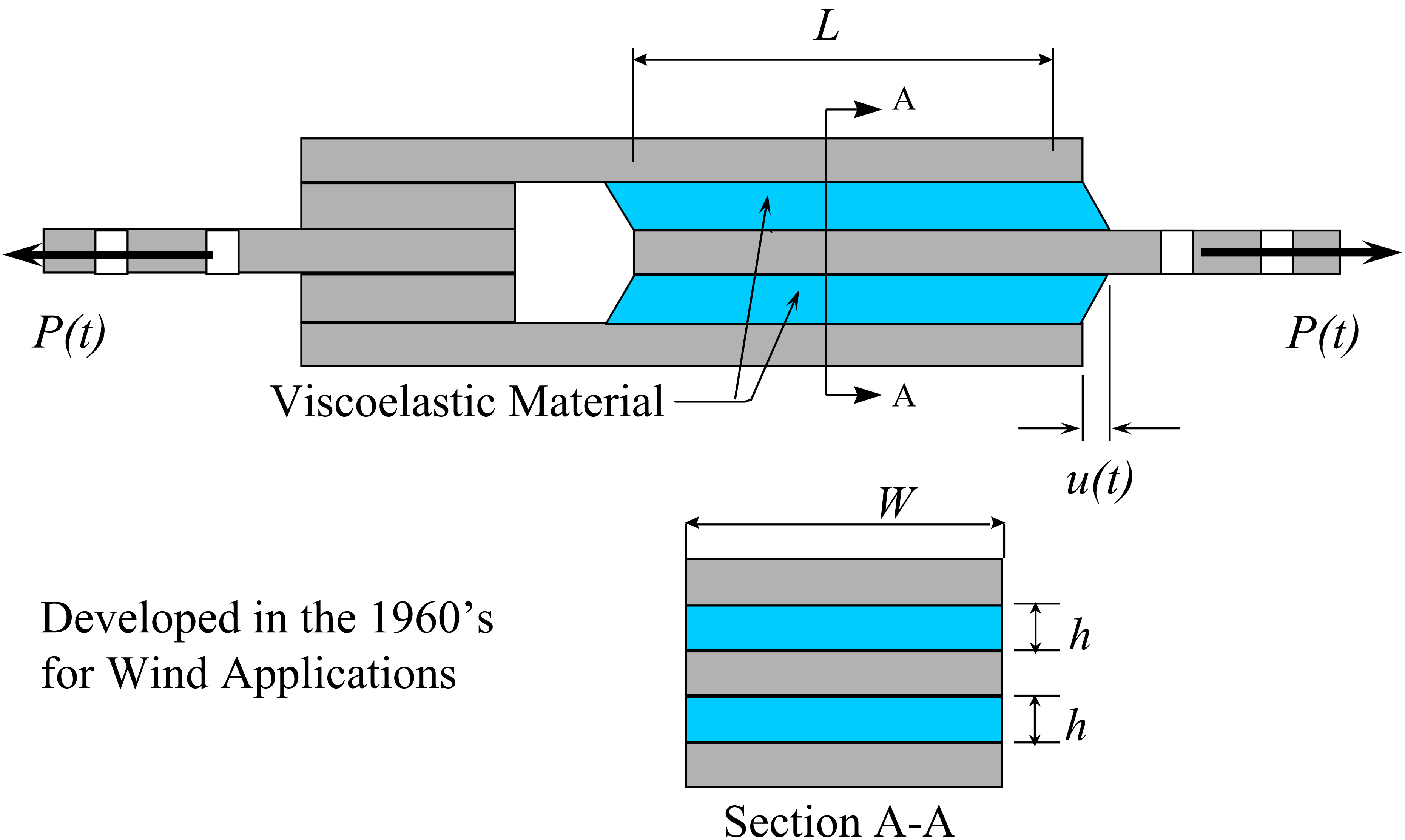
Advantages of Fluid Dampers

- High reliability
- High force and displacement capacity
- Force Limited when velocity exponent < 1.0
- Available through several manufacturers
- No added stiffness at lower frequencies
- Damping force (possibly) out of phase with structure elastic forces
- Moderate temperature dependency
- *May* be able to use linear analysis

Disadvantages of Fluid Dampers

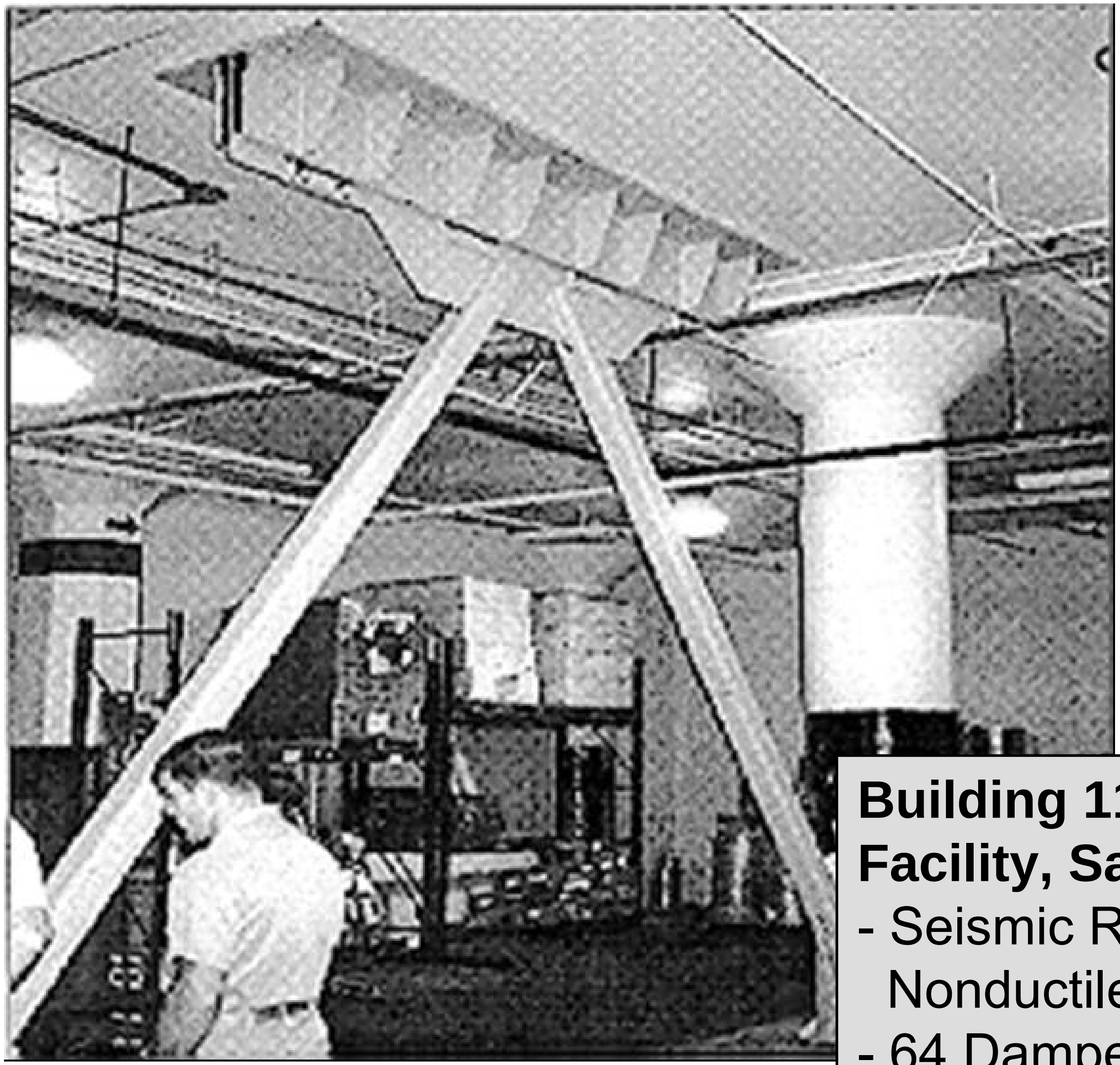
- Somewhat higher cost
- Not force limited (particularly when exponent = 1.0)
- Necessity for nonlinear analysis in most practical cases (as it has been shown that it is generally not possible to add enough damping to eliminate all inelastic response)

Viscoelastic Dampers



Developed in the 1960's
for Wind Applications

Implementation of Viscoelastic Dampers



Building 116, US Naval Supply Facility, San Diego, CA
- Seismic Retrofit of 3-Story Nonductile RC Building
- 64 Dampers Within Chevron Bracing Installed in 1996

Harmonic Behavior of Viscoelastic Damper

$$u(t) = u_0 \sin(\overline{\omega} t)$$

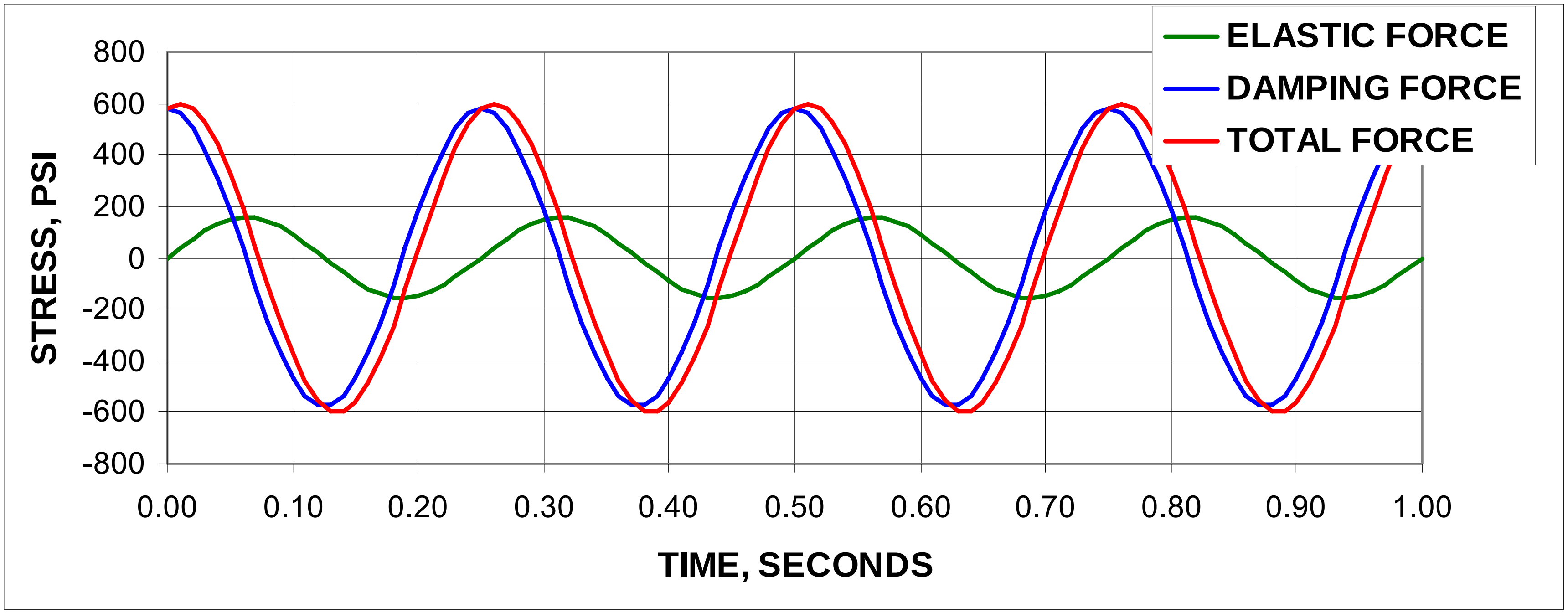
Imposed Motion

Loading Frequency

Phase Angle (Lag)

Total Force

$$P(t) = P_0 \sin(\overline{\omega} t)\cos(\delta) + P_0 \cos(\overline{\omega} t)\sin(\delta)$$



$$P(t) = K_S u(t) + C\dot{u}(t)$$

$$\boxed{K_S = \frac{G' A}{h}}$$

Storage Stiffness

$$\boxed{K_L = \frac{G'' A}{h}}$$

Loss Stiffness

$$\boxed{C = \frac{K_L}{\overline{\omega}}}$$

Damping Coeff.

$$\boxed{\delta = \sin^{-1}\left(\frac{\tau_Z}{\tau_0}\right)}$$

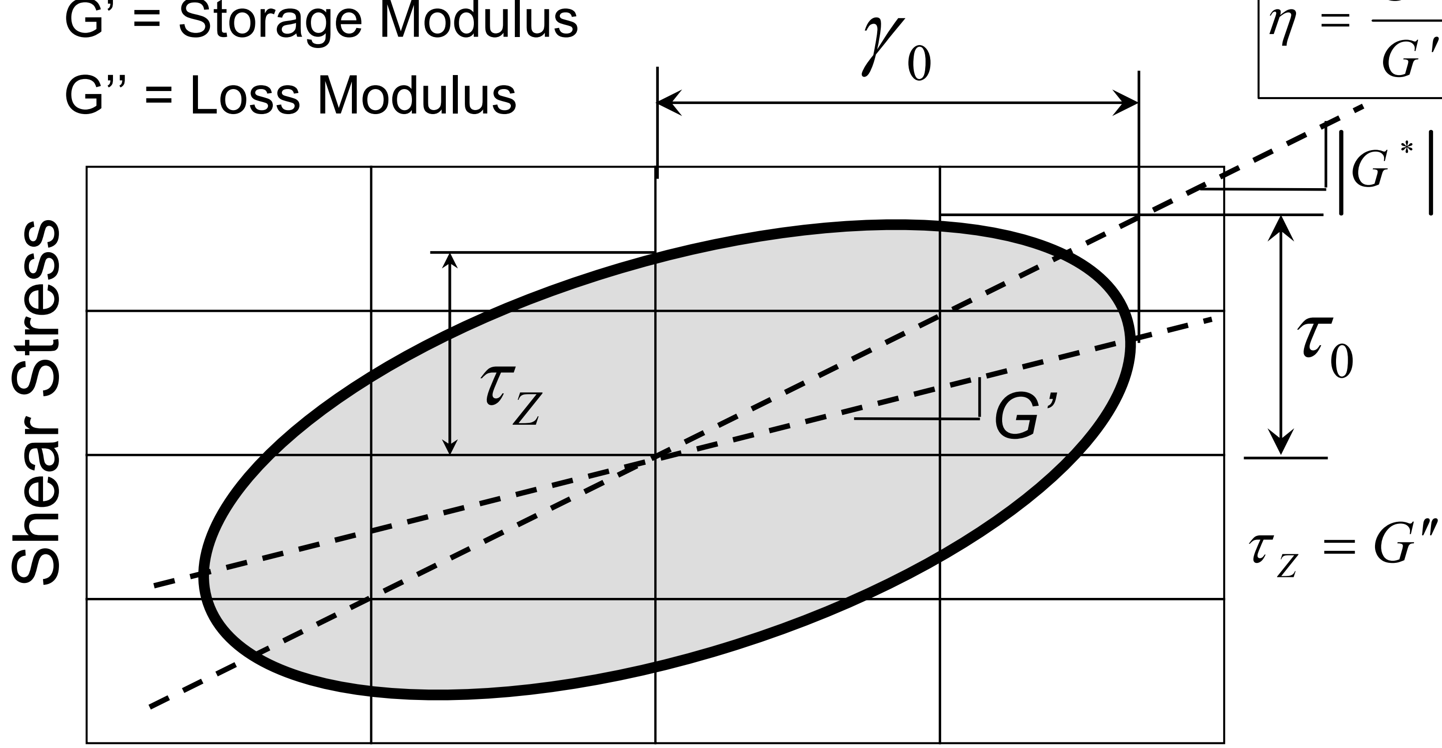
Phase Angle

G’ = Storage Modulus

G’’ = Loss Modulus

$$\boxed{\eta = \frac{G''(\overline{\omega})}{G'(\overline{\omega})} = tan\,(\delta)}$$

Loss Factor



$$\tau_Z = G''\gamma_o = \tau_o\, sin(\delta)$$

$$\boxed{\tau(t) = G'\gamma(t) + G''\dot{\gamma}(t)/\overline{\omega}}$$

Shear Strain

$$E_D = \pi\tau_Z\gamma_o Ah = \pi\tau_o\gamma_o Ah\, sin(\delta) = \pi G''\gamma_o^2 V$$

Frequency-Domain Stress-Strain Relation

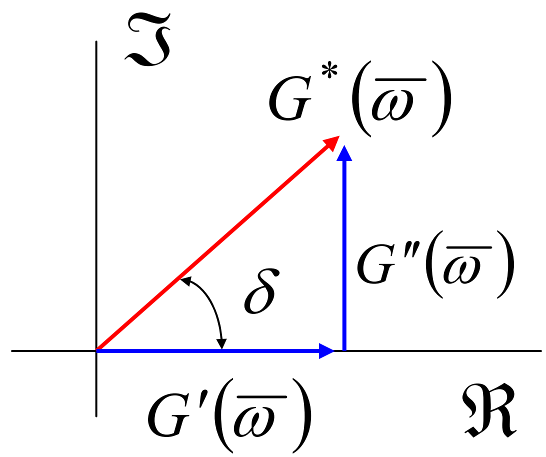
$$\tau(t) = G'\gamma(t) + G''\dot{\gamma}(t)/\overline{\omega}$$

Apply Fourier Transform:

$$\tau(\overline{\omega}) = G'\gamma(\overline{\omega}) + G''i\overline{\omega}\gamma(\overline{\omega})/\overline{\omega}$$

$$\tau(\overline{\omega}) = [G' + iG'']\gamma(\overline{\omega})$$

$$\tau(\overline{\omega}) = G'[1 + i\eta]\gamma(\overline{\omega})$$



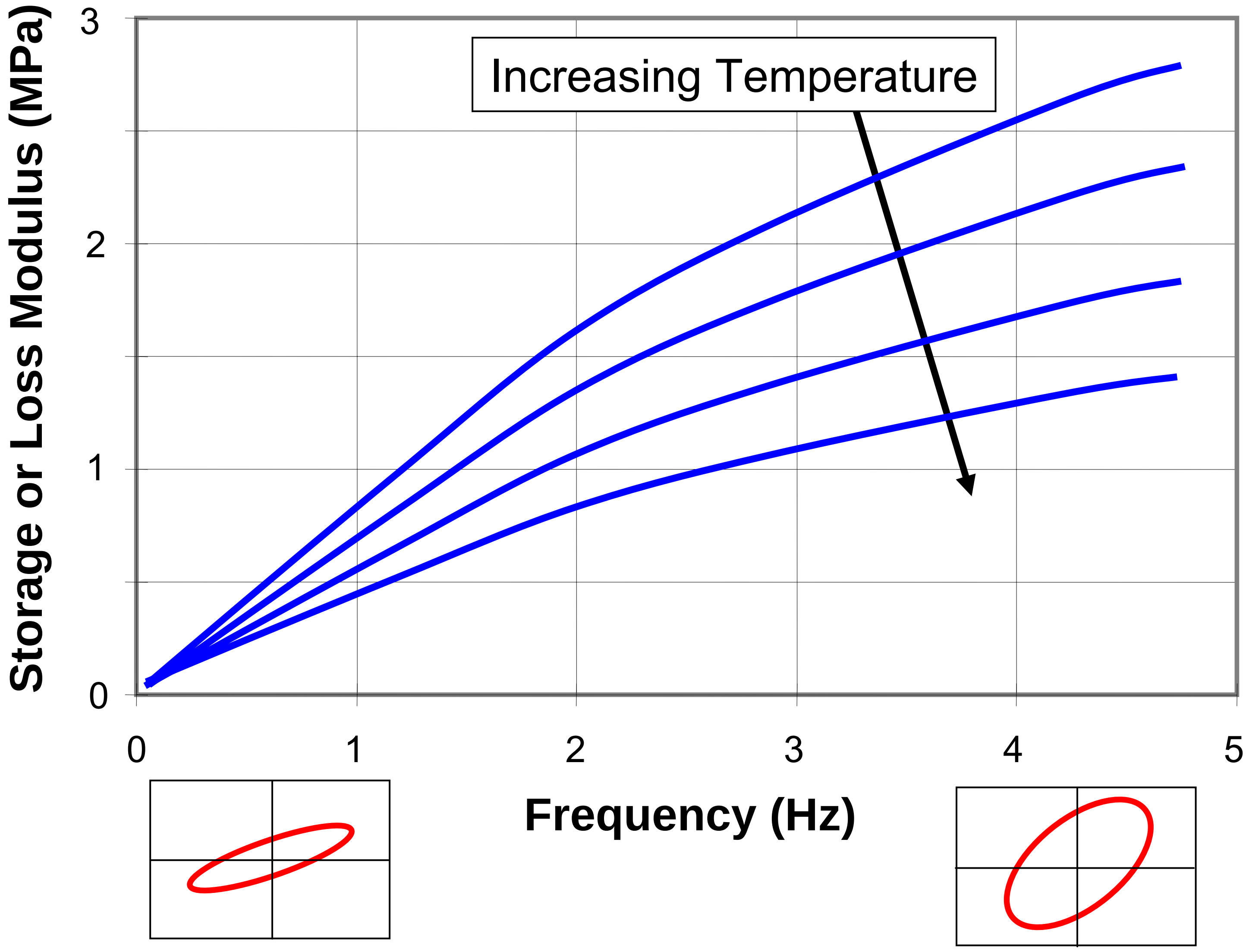
Complex Shear Modulus:

$$G^*(\overline{\omega}) = \frac{\tau(\overline{\omega})}{\gamma(\overline{\omega})} = G'[1 + i\eta]$$

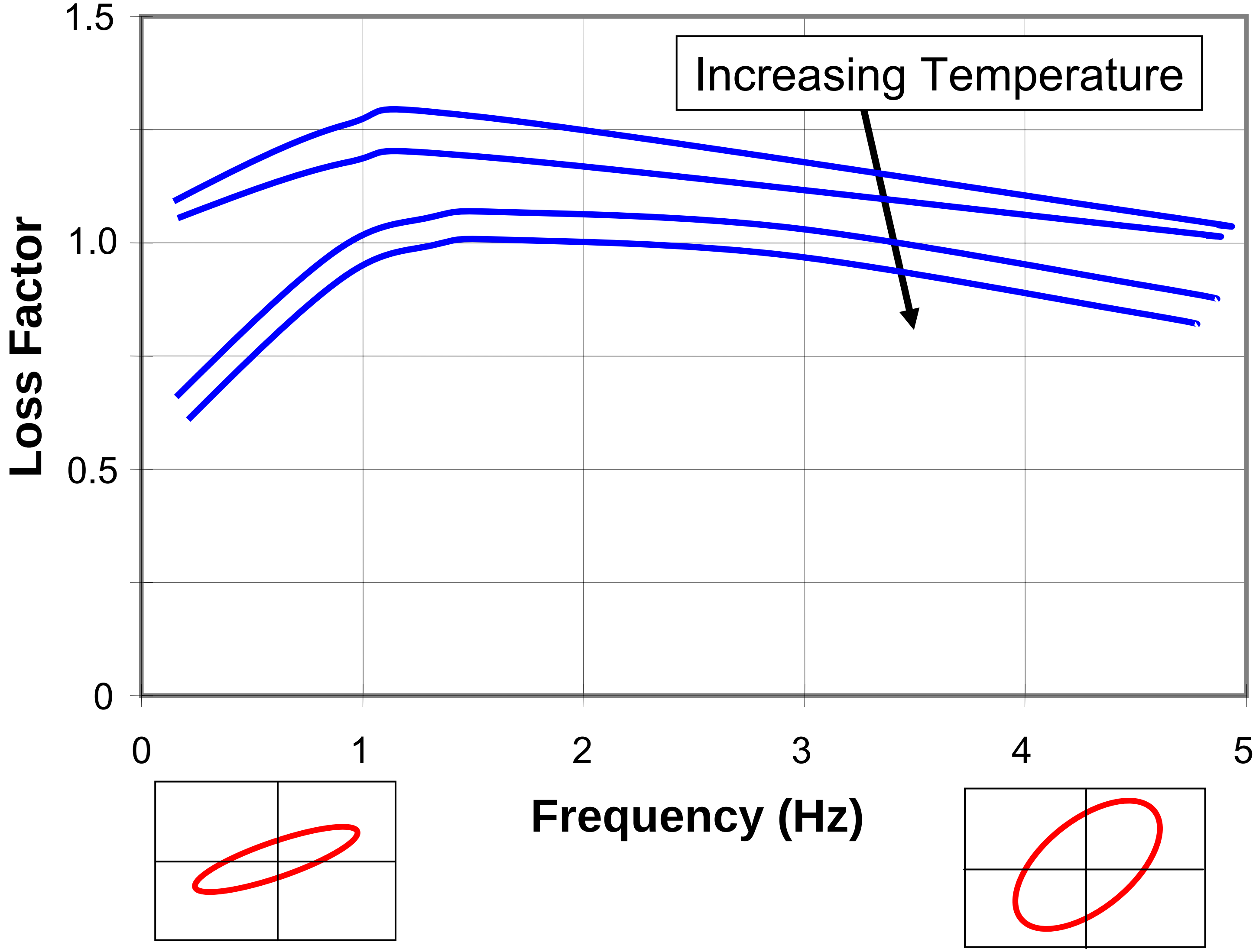
$$\tau(\overline{\omega}) = G^*(\overline{\omega})\gamma(\overline{\omega})$$

Compact Stress-Strain Relation for Viscoelastic Materials

Dependence of Storage and Loss Moduli on Temperature and Frequency for Typical Viscoelastic Damper

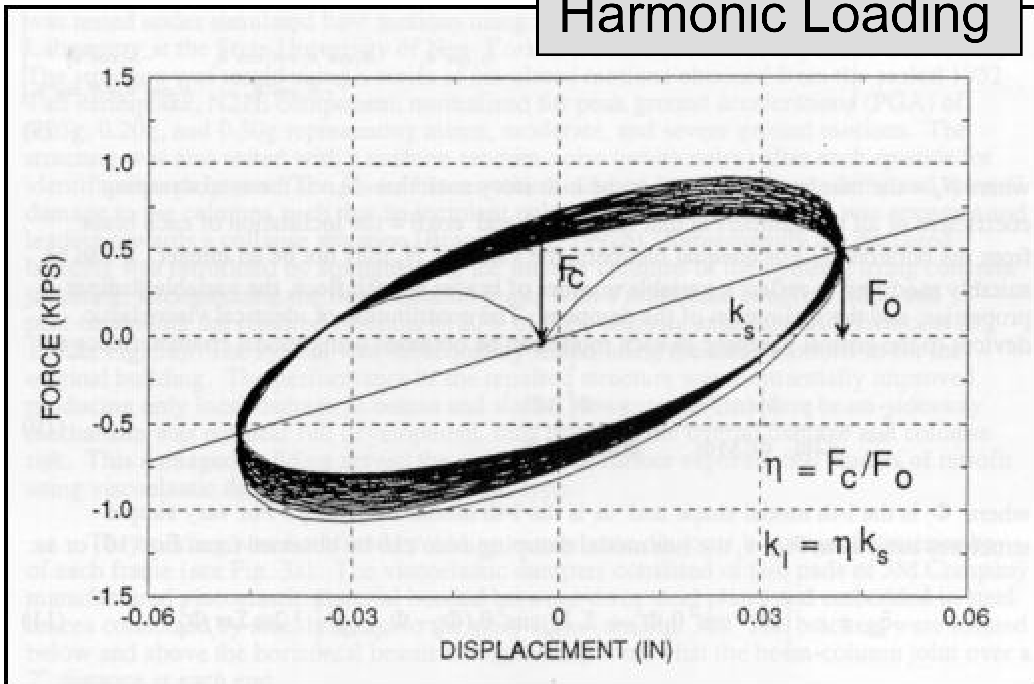


Dependence of Loss Factor on Temperature and Frequency for Typical Viscoelastic Damper

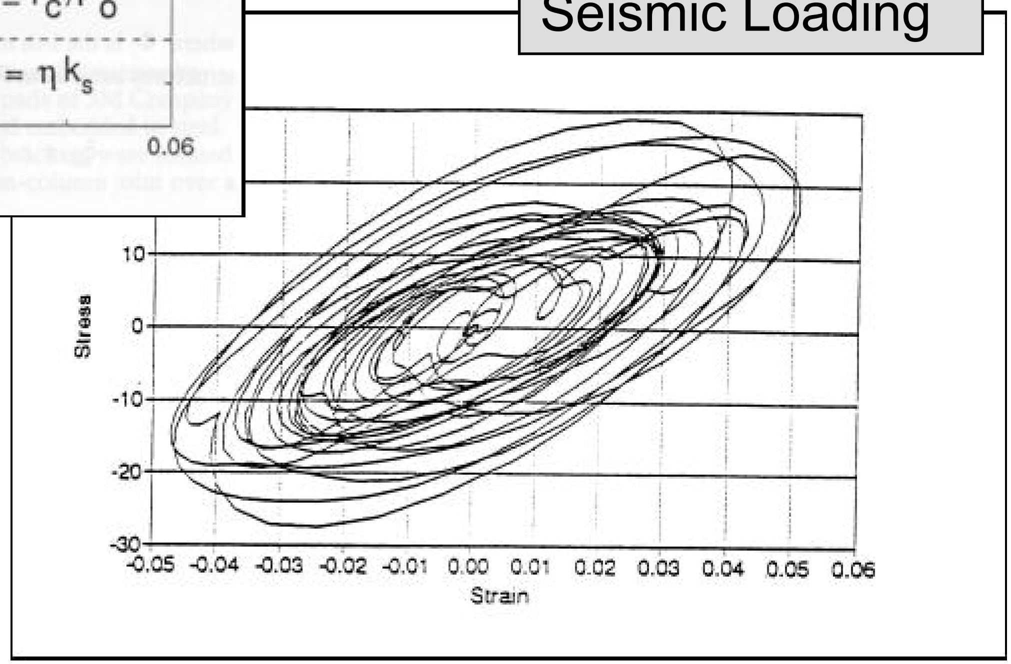


Actual Hysteretic Behavior of Viscoelastic Damper

Harmonic Loading



Seismic Loading



Advantages of Viscoelastic Dampers

- High reliability
- *May* be able to use linear analysis
- Somewhat lower cost

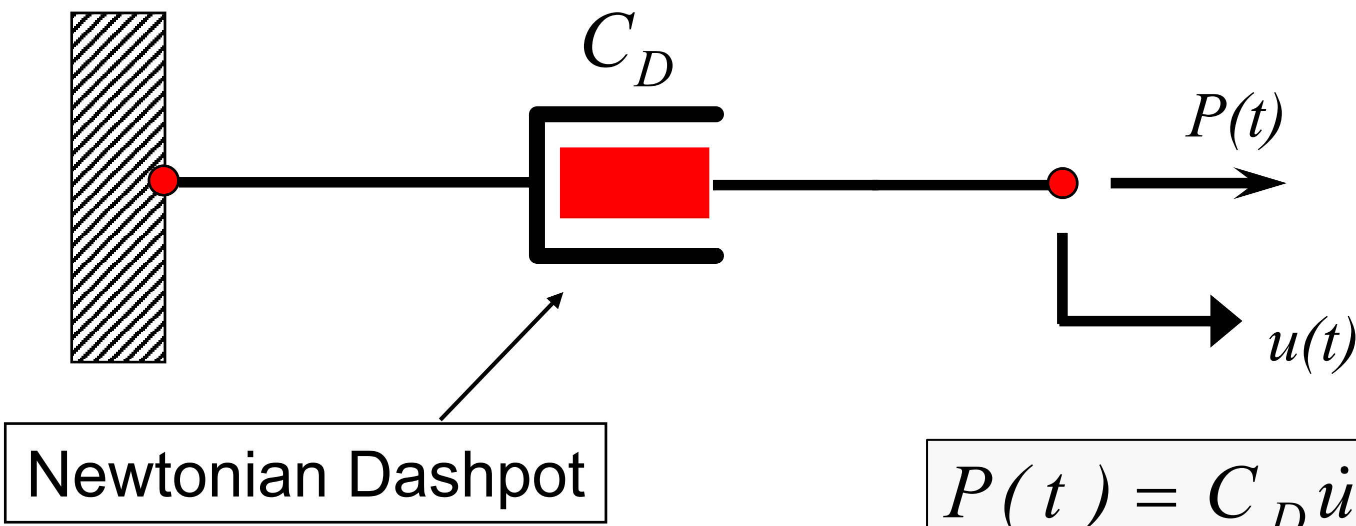
Disadvantages of Viscoelastic Dampers

- Strong Temperature Dependence
- Lower Force and Displacement Capacity
- Not Force Limited
- Necessity for nonlinear analysis in most practical cases (as it has been shown that it is generally not possible to add enough damping to eliminate all inelastic response)

Outline: Part II

- Velocity-Dependent Damping Systems: Fluid Dampers and Viscoelastic Dampers
- Models for Velocity-Dependent Dampers
- Effects of Linkage Flexibility
- Displacement-Dependent Damping Systems: Steel Plate Dampers, Unbonded Brace Dampers, and Friction Dampers
- Concept of Equivalent Viscous Damping
- Modeling Considerations for Structures with Passive Damping Systems

Modeling Viscous Dampers:
Simple Dashpot

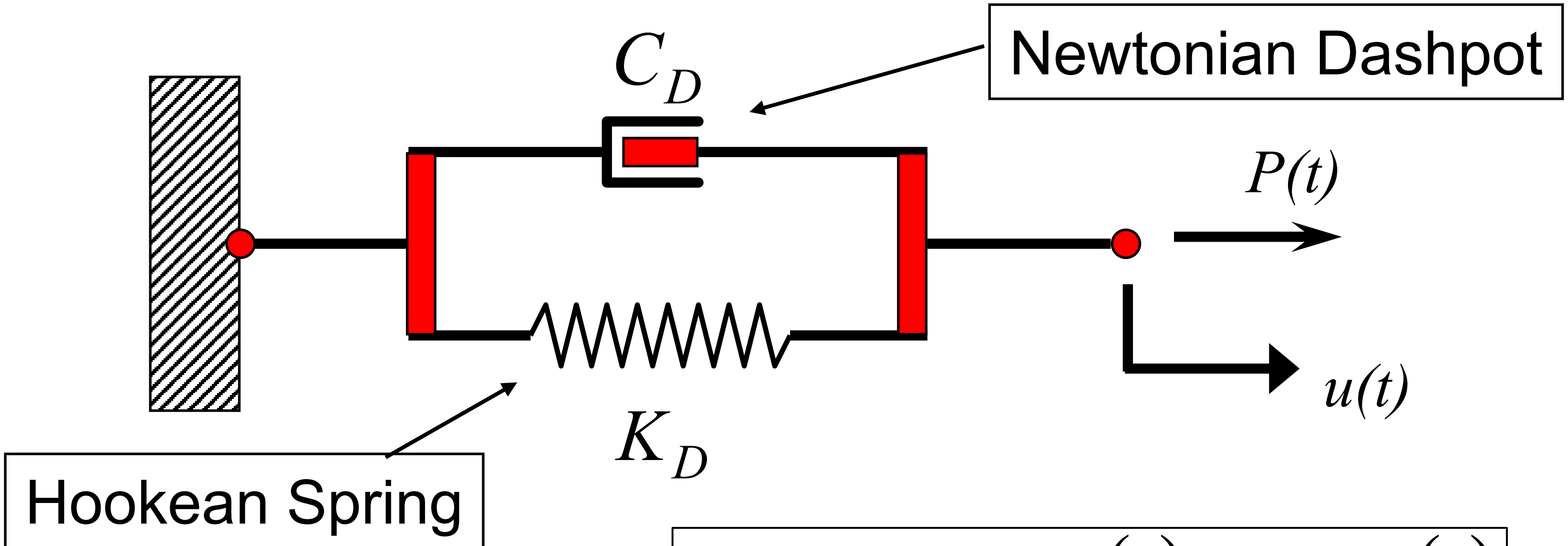


$$P(t) = C_D \dot{u}(t)$$

Useful For :
Fluid Dampers with Zero Storage Stiffness

This Model Ignores Temperature Dependence

Modeling Linear Viscous/Viscoelastic Dampers: Kelvin Model



$$P(t) = K_D u(t) + C_D \dot{u}(t)$$

Useful For :
Viscoelastic Dampers and Fluid Dampers with Storage Stiffness and Weak Frequency Dependence.

This Model Ignores Temperature Dependence

Kelvin Model (Continued)

$$\boxed{P(t) = K_D u(t) + C_d \dot{u}(t)}$$

Apply Fourier Transform:

$$P(\overline{\omega}) = \left[K_D + i\overline{\omega}C_d\right]u(\overline{\omega})$$

Complex Stiffness:

$$K^*(\overline{\omega}) = K_D + i\overline{\omega}C_d$$

Storage Stiffness:

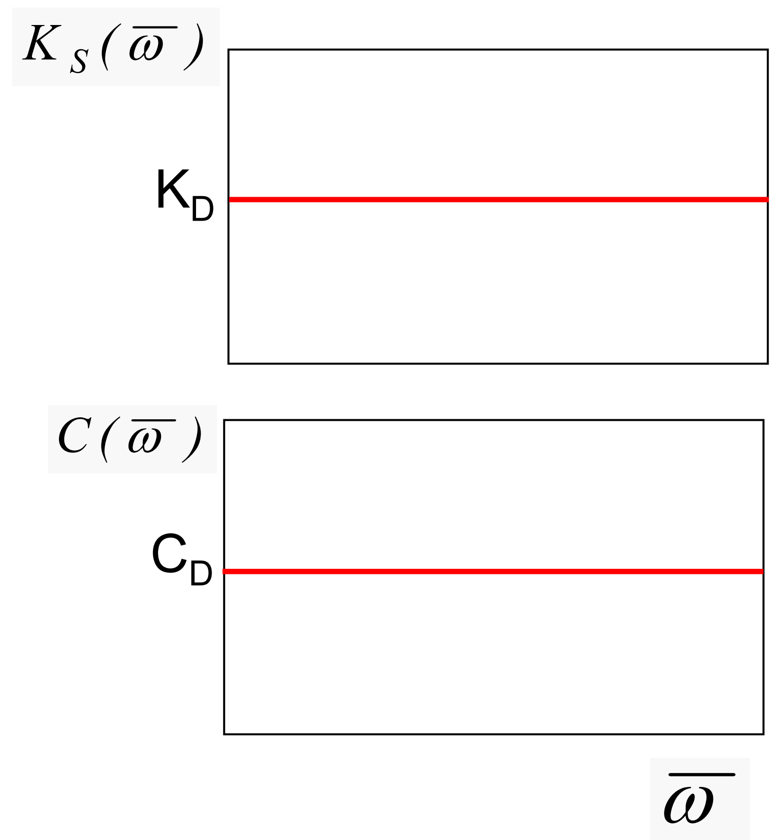
$$K_S(\overline{\omega}) = \Re\left[K^*(\overline{\omega})\right] = K_D$$

Loss Stiffness:

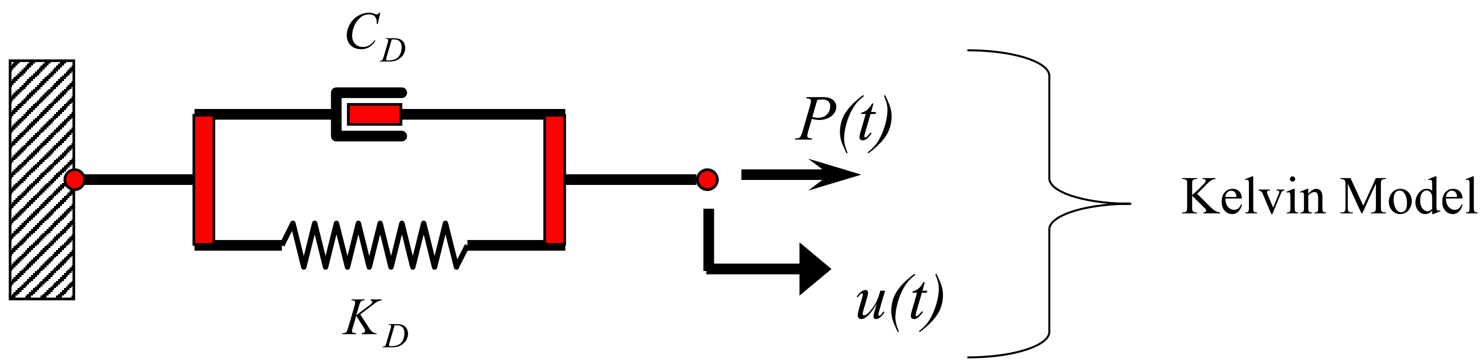
$$K_L(\overline{\omega}) = \Im\left[K^*(\overline{\omega})\right] = C_D\overline{\omega}$$

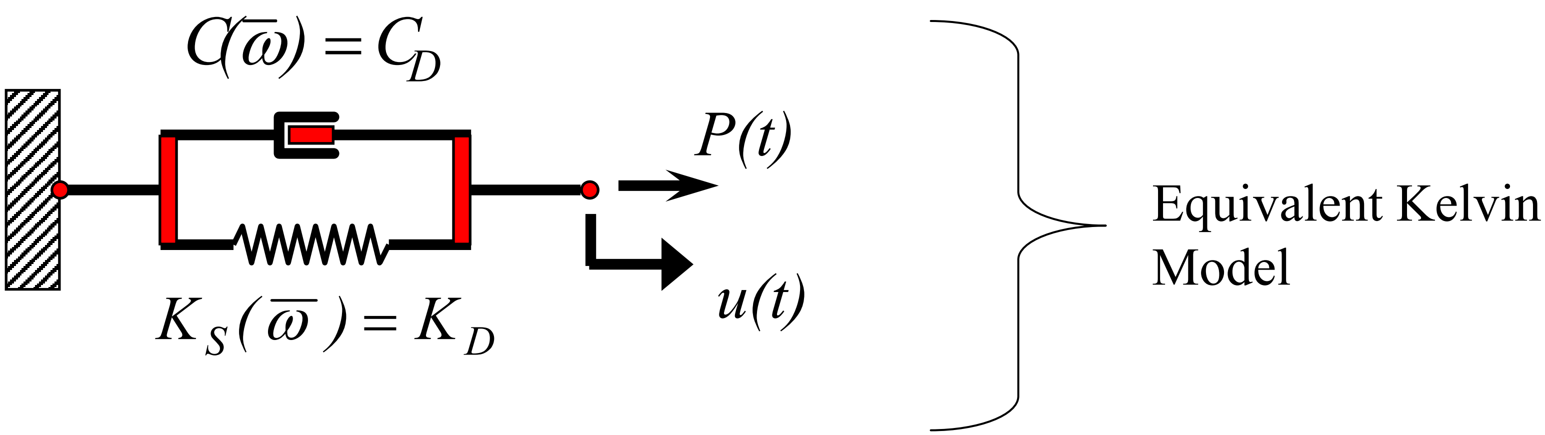
Damping Coefficient:

$$C(\overline{\omega}) = \frac{K_L(\overline{\omega})}{\overline{\omega}} = C_D$$

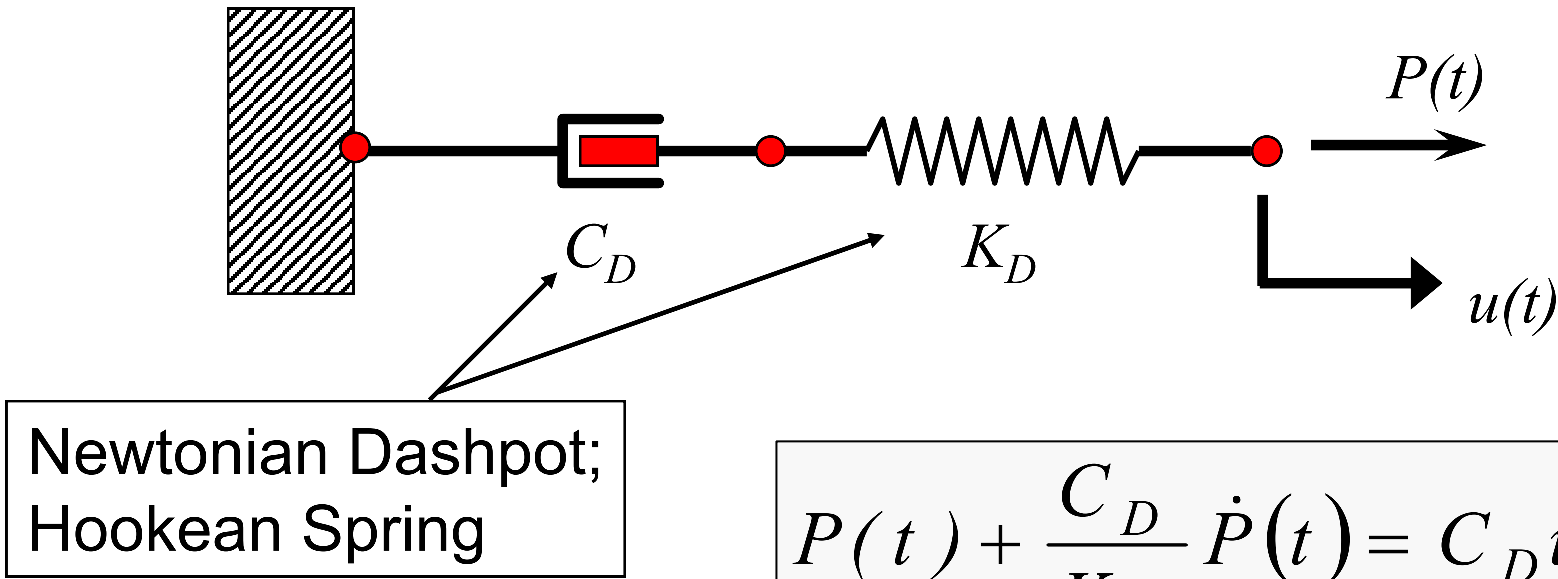


Kelvin Model (Continued)





Modeling Linear Viscous/Viscoelastic Dampers: Maxwell Model



$$P(t) + \frac{C_D}{K_D}\dot{P}(t) = C_D \dot{u}(t)$$

Useful For :

Viscoelastic Dampers and Fluid Dampers with Strong Frequency Dependence.

This Model Ignores Temperature Dependence

Maxwell Model (Continued)

$$\boxed{P(t) + \frac{C_D}{K_D}\dot{P}(t) = C_d\dot{u}(t)}$$

Apply Fourier Transform:

$$P(\overline{\omega}) + i\overline{\omega}\frac{C_D}{K_D}P(\overline{\omega}) = i\overline{\omega}C_d u(\overline{\omega})$$

Complex Stiffness:

$$K^*(\overline{\omega}) = \frac{C_D\lambda\overline{\omega}^2}{1+\lambda^2\overline{\omega}^2} + i\frac{C_D\overline{\omega}}{1+\lambda^2\overline{\omega}^2}$$

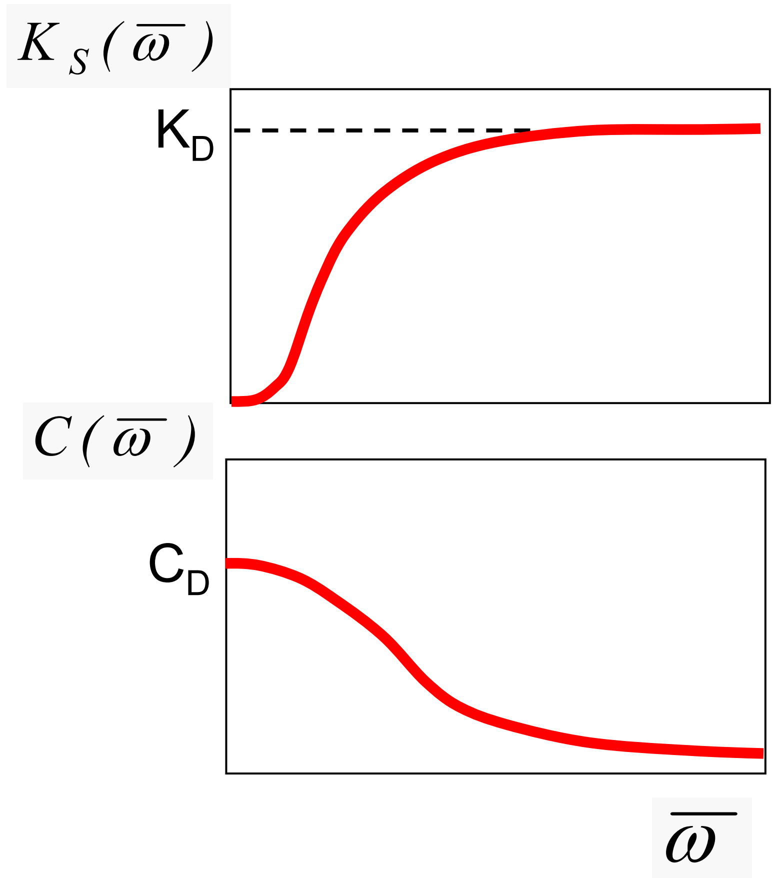
Relaxation Time: $\lambda = C_D / K_D$

Storage Stiffness:

$$K_S(\overline{\omega}) = \Re\left[K^*(\overline{\omega})\right] = \frac{K_D\lambda^2\overline{\omega}^2}{1+\lambda^2\overline{\omega}^2}$$

Loss Stiffness:

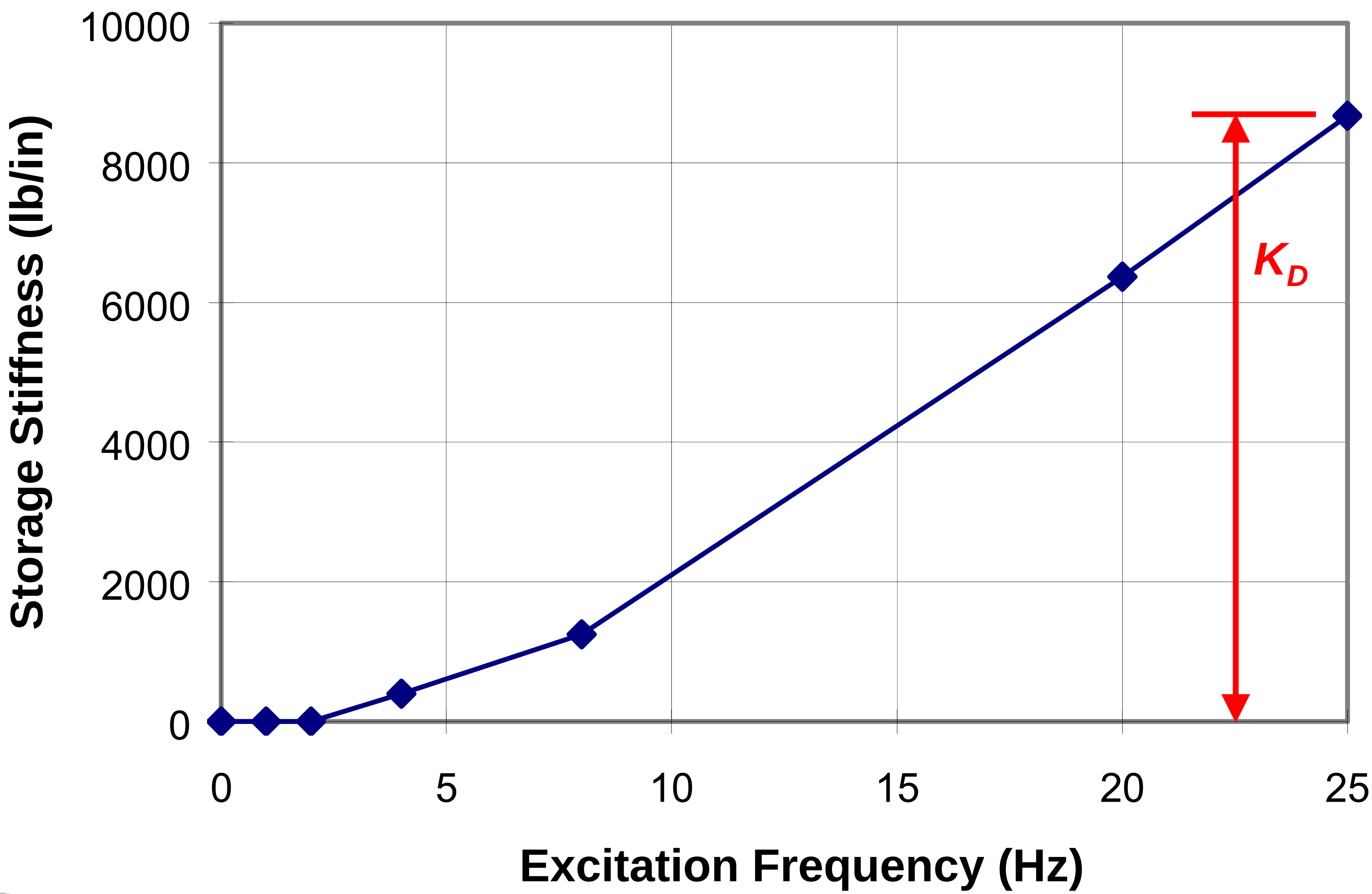
$$K_L(\overline{\omega}) = \Im\left[K^*(\overline{\omega})\right] = \frac{C_D\overline{\omega}}{1+\lambda^2\overline{\omega}^2}$$



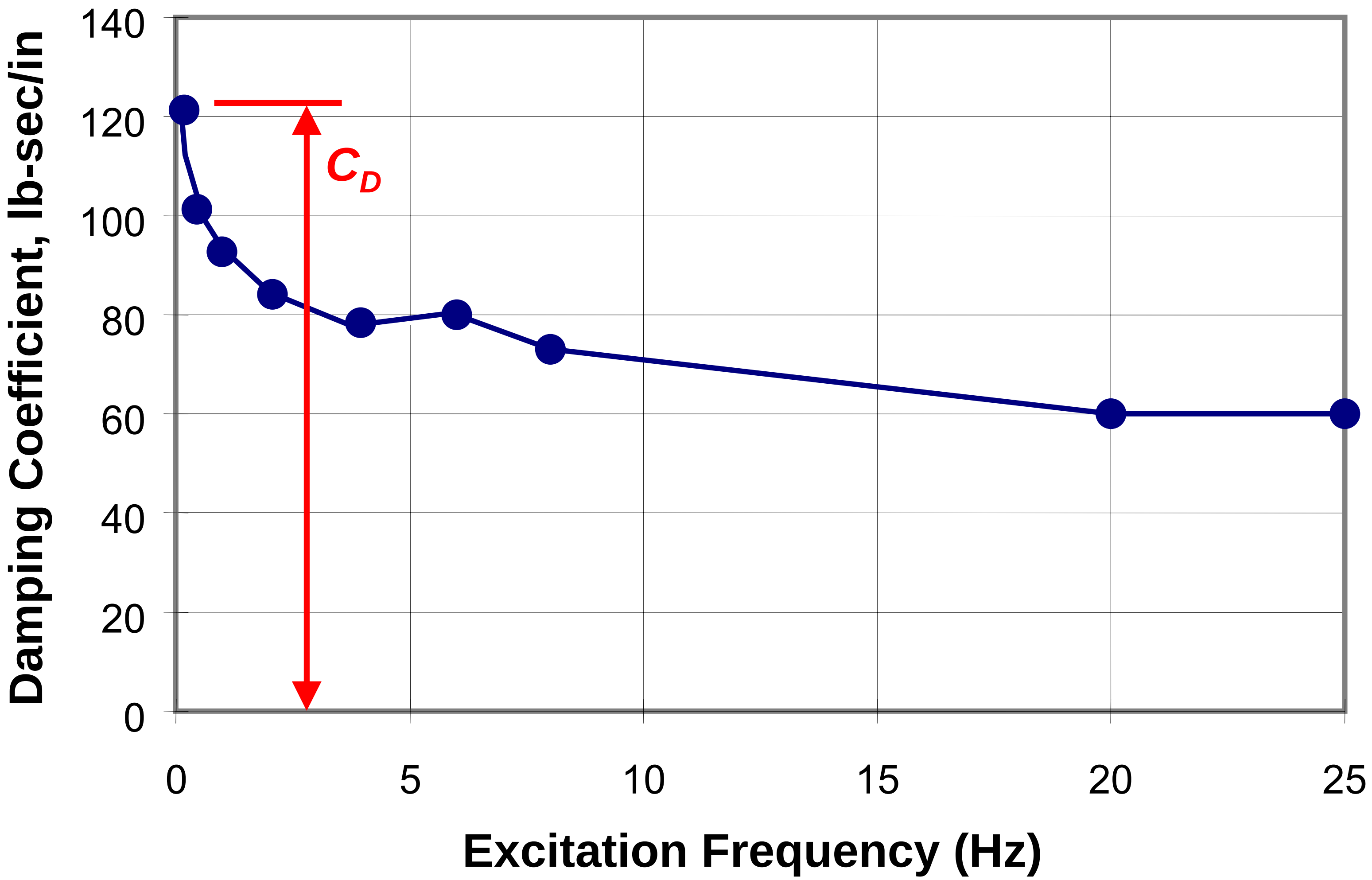
Damping Coefficient:

$$C(\overline{\omega}) = \frac{K_S(\overline{\omega})}{\overline{\omega}} = \frac{C_D}{1+\lambda^2\overline{\omega}^2}$$

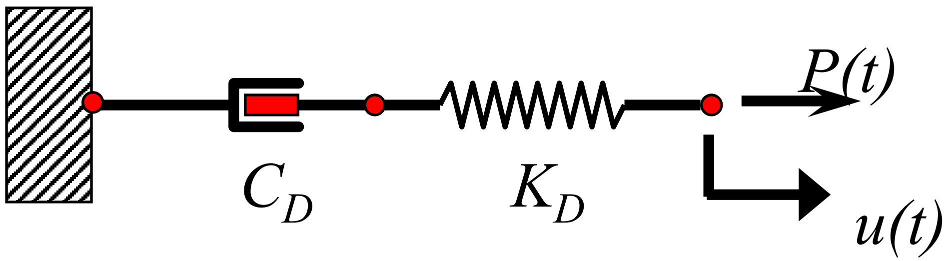
Maxwell Model Parameters from Experimental Testing of Fluid Viscous Damper



Maxwell Model Parameters from Experimental Testing of Fluid Viscous Damper

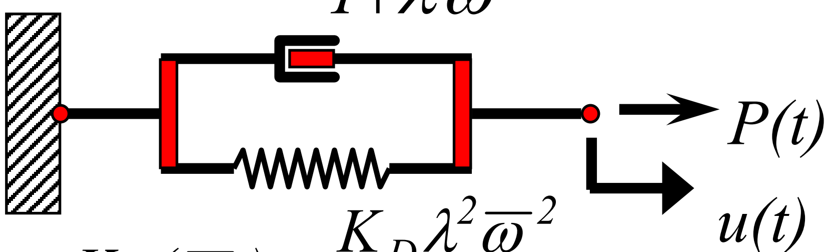


Maxwell Model (Continued)



Maxwell Model

$$C(\overline{\omega}) = \frac{C_D}{1+\lambda^2\overline{\omega}^2}$$

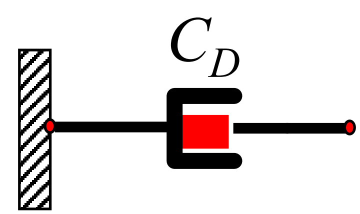


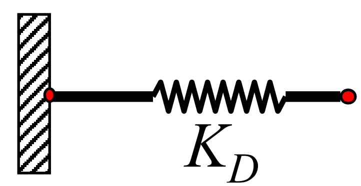
$$K_S(\overline{\omega}) = \frac{K_D\lambda^2\overline{\omega}^2}{1+\lambda^2\overline{\omega}^2}$$

Equivalent Kelvin Model

Note:

- If K_D is very large, λ is very small, K_S is small and $C = C_D$
- If C_D is very small, λ is very small, K_S is small and $C = C_D$
- If K_D is very small, λ is very large, C is small and $K_S = K_D$.





Outline: Part II

- Velocity-Dependent Damping Systems: Fluid Dampers and Viscoelastic Dampers
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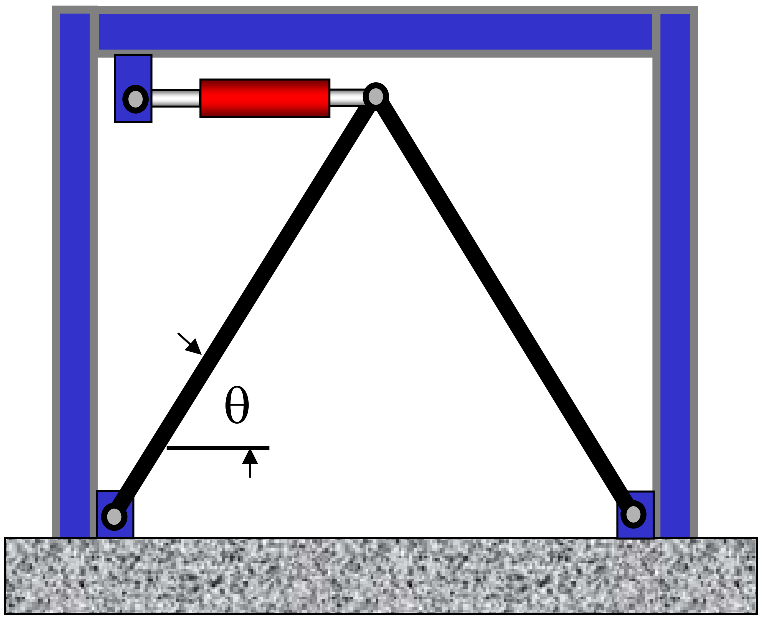
Effect of Linkage Flexibility on Damper Effectiveness



$$K_{Brace,Effective} = 2\frac{AE}{L}\cos^2\theta$$

Because the damper is always in series with the linkage, the damper-brace assembly acts like a Maxwell model.

Hence, the effectiveness of the damper is reduced. The degree of lost effectiveness is a function of the structural properties and the loading frequency.

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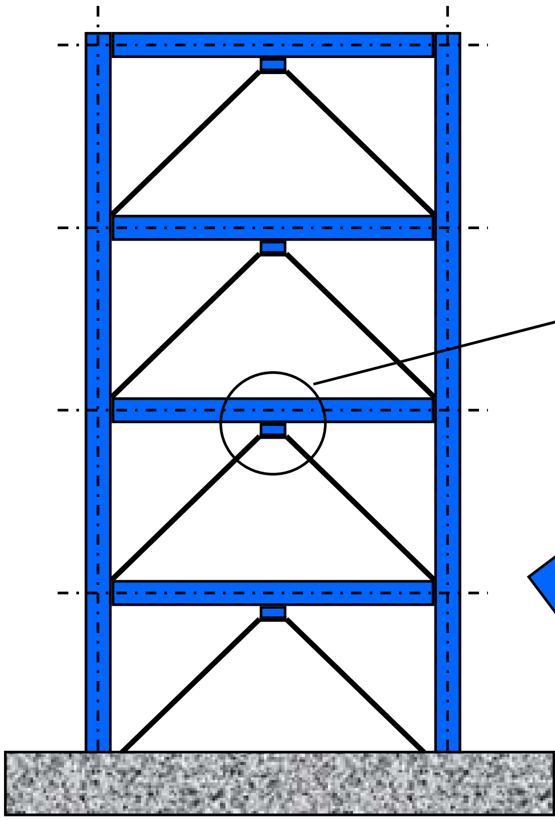
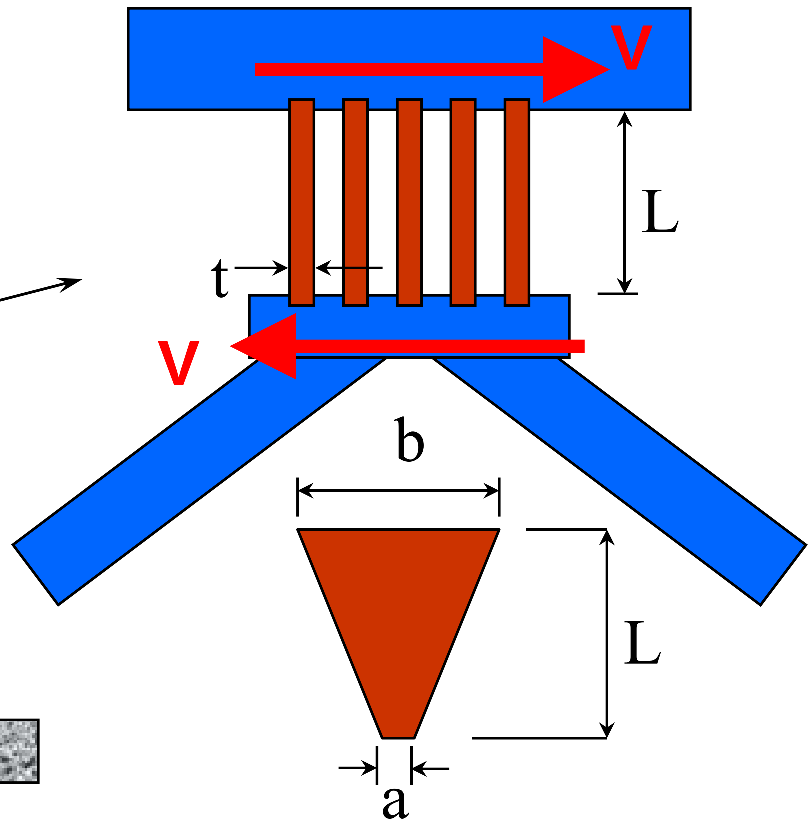
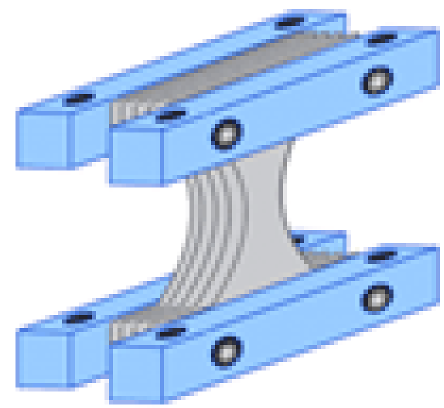
Steel Plate Dampers
(<u>A</u>dded <u>D</u>amping <u>a</u>nd <u>S</u>tiffness System - ADAS)





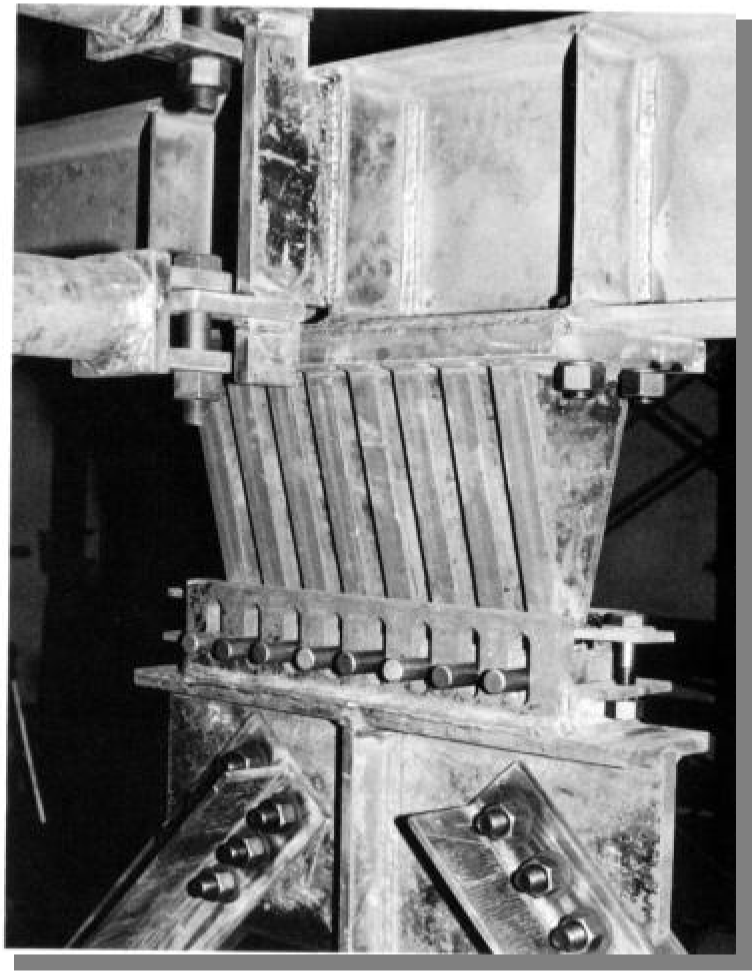


Implementation of ADAS System

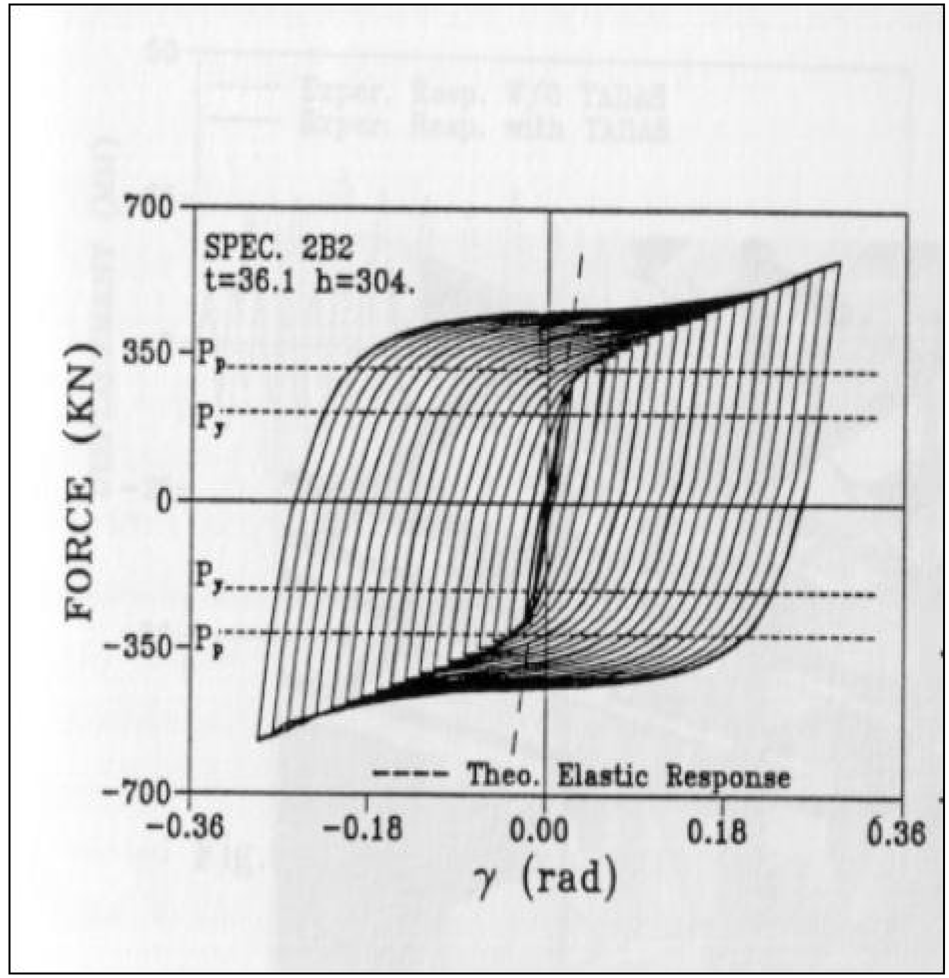
Wells Fargo Bank,
San Francisco, CA

- Seismic Retrofit of Two-Story Nonductile Concrete Frame; Constructed in 1967
- 7 Dampers Within Chevron Bracing Installed in 1992
- Yield Force Per Damper: 150 kips

Hysteretic Behavior of ADAS Device



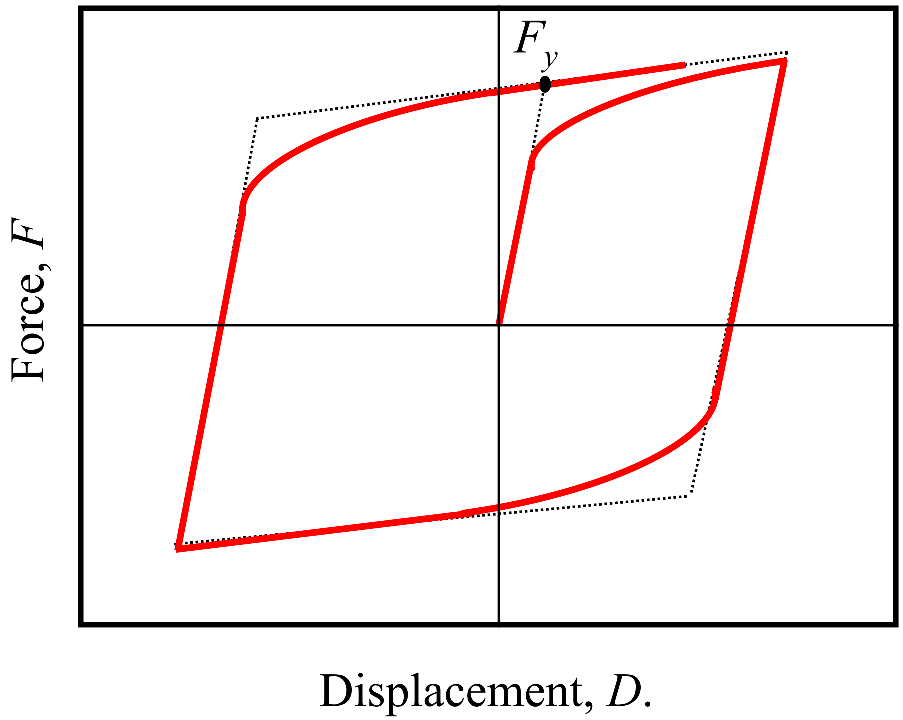
ADAS Device
(Tsai et al. 1993)



Experimental Response (Static)
(**Source:** Tsai et al. 1993)

Ideal Hysteretic Behavior of ADAS Damper

(SAP2000 and ETABS Implementation)



Initial Stiffness

Secondary Stiffness Ratio

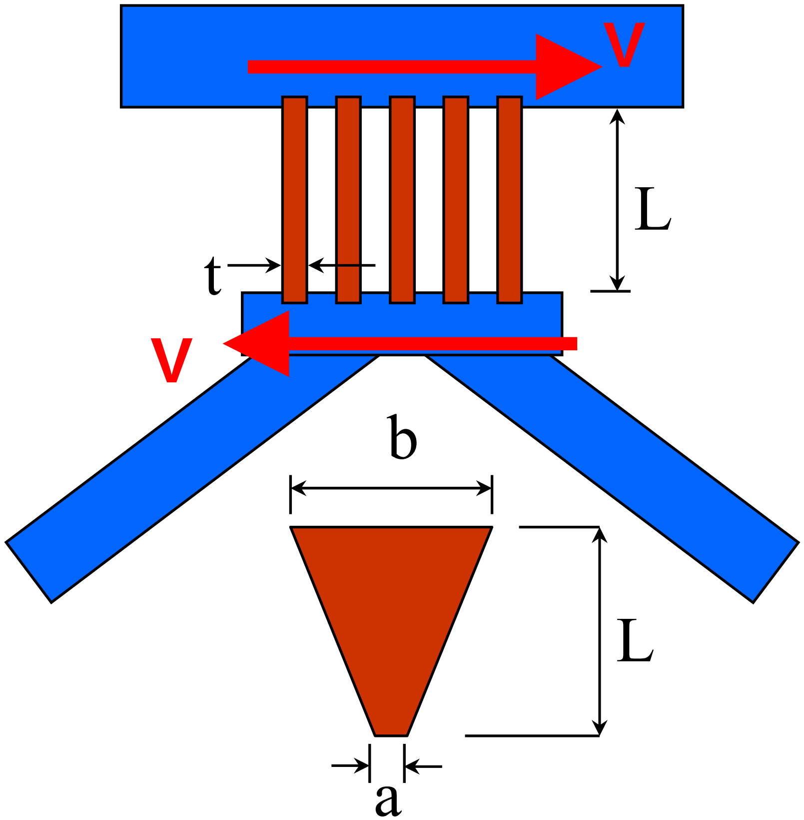
$$F = \beta k D + (1-\beta) F_y Z$$

$$\dot{Z} = \frac{k}{F_y} \begin{cases} \dot{D}\left(1 - |Z|^{\alpha}\right) & \textbf{if } \dot{D} \cdot Z > 0 \\ \dot{D} & \textbf{\textit{otherwise}} \end{cases}$$

Yield Sharpness

Z is a Path Dependency Parameter

Parameters of Mathematical Model of ADAS Damper



$$k = \frac{n\left(2 + a/b\right)EI_b}{L^3}$$

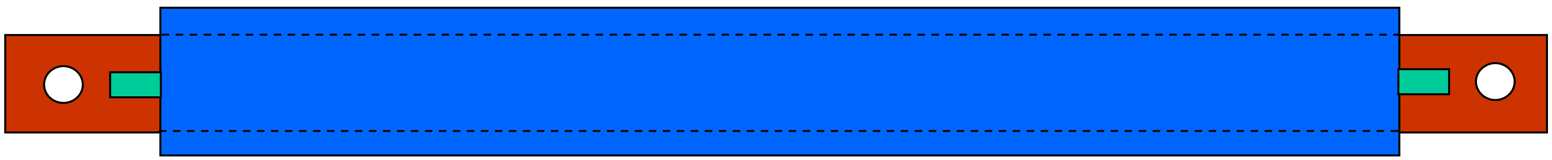
$$F_y = \frac{nf_y bt^3}{4L}$$

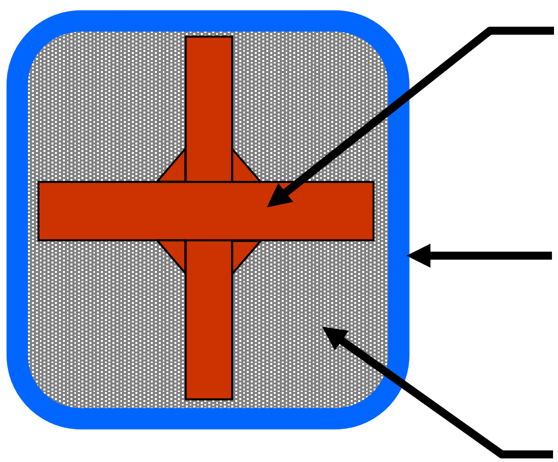
$n =$ Number of plates

$f_y =$ Yield force of each plate

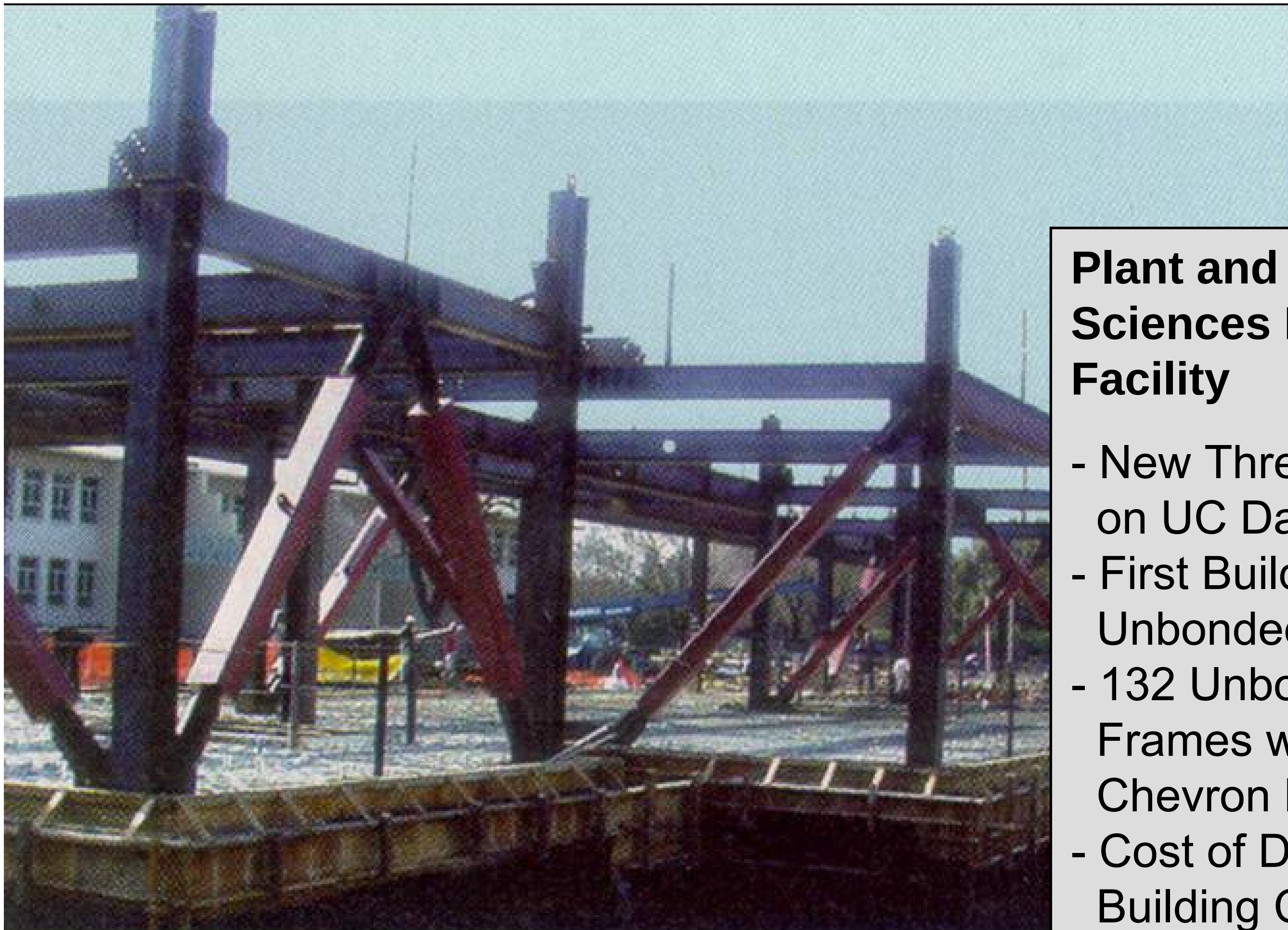
$I_b =$ Second moment of area of each plate at b (i.e, at top of plate)

Unbonded Brace Damper





Implementation of Unbonded Brace Damper

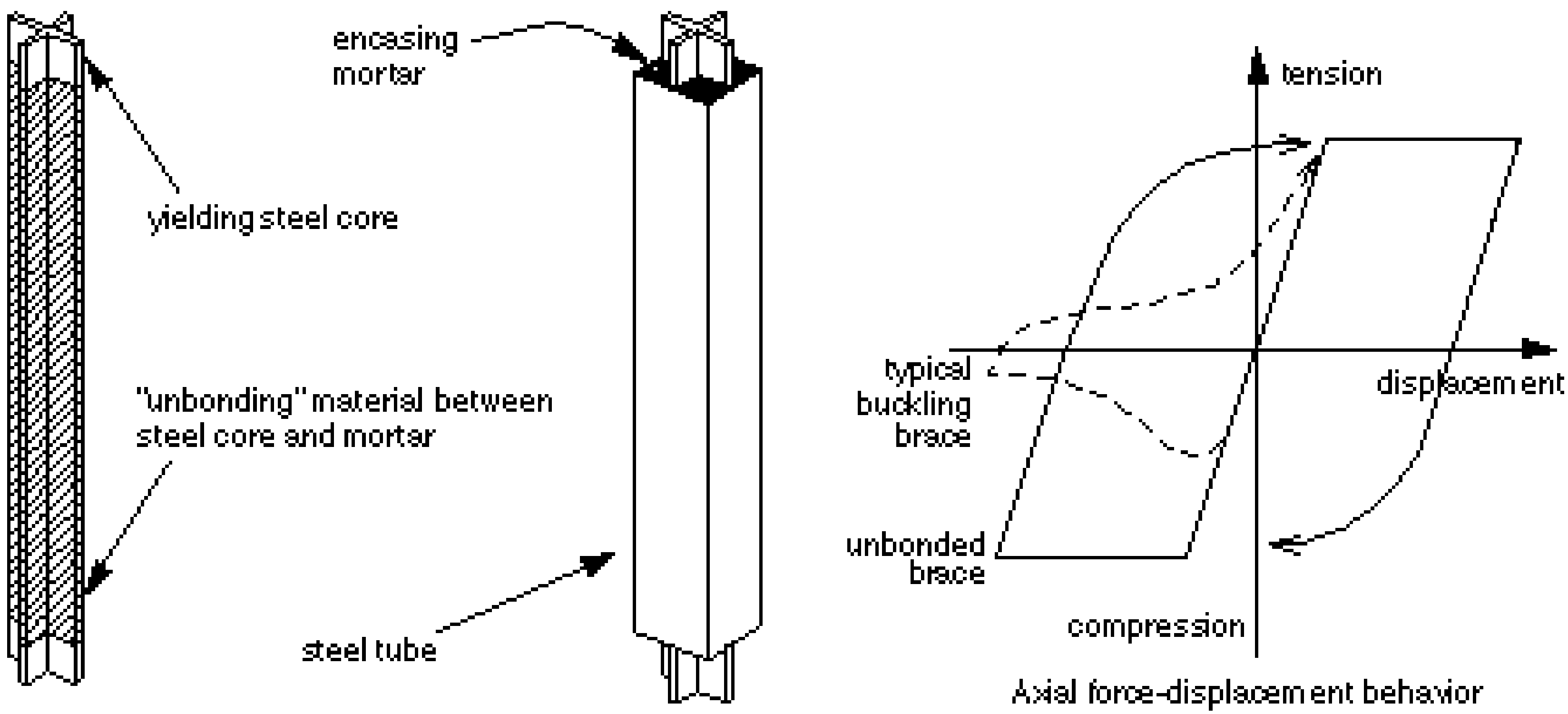


Plant and Environmental Sciences Replacement Facility

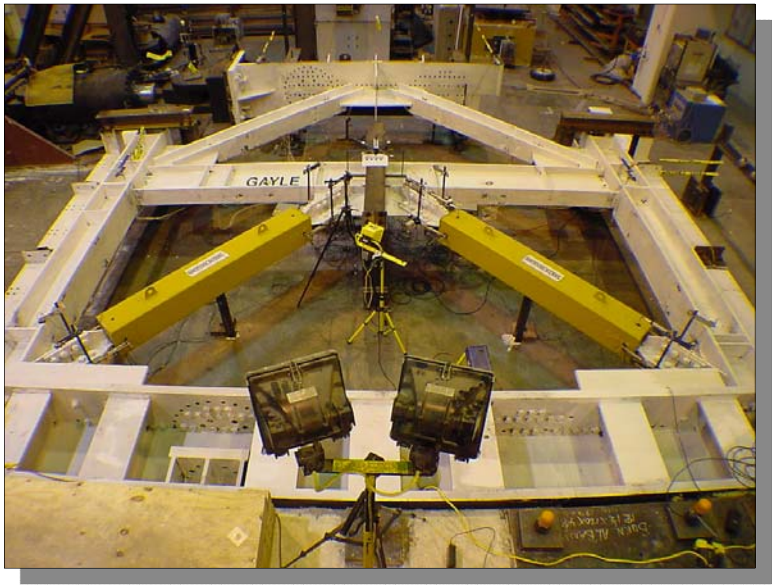
- New Three-Story Building on UC Davis Campus
- First Building in USA to Use Unbonded Brace Damper
- 132 Unbonded Braced Frames with Diagonal or Chevron Brace Installation
- Cost of Dampers = 0.5% of Building Cost

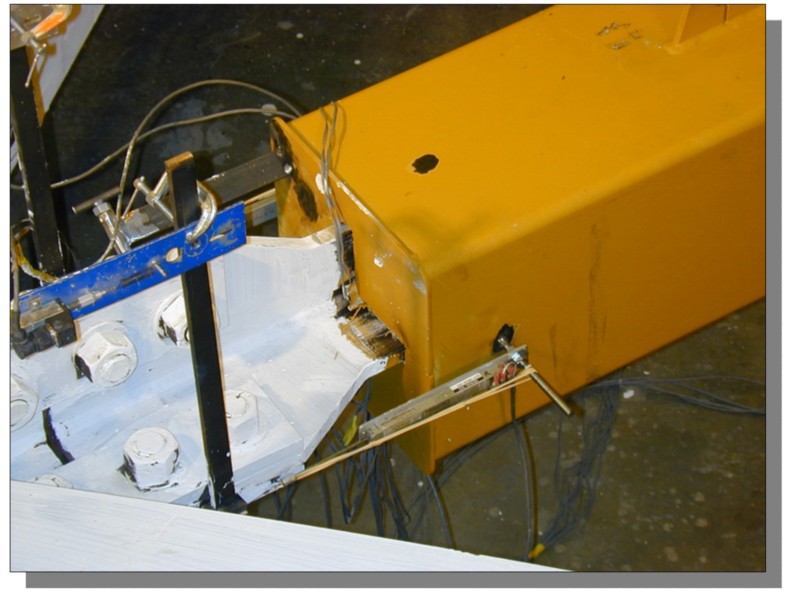
Source: ASCE Civil Engineering Magazine, March 2000.

Hysteretic Behavior of Unbonded Brace Damper



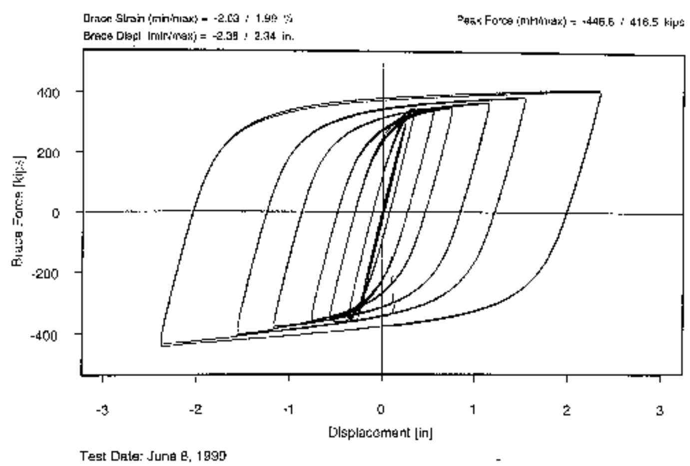
Testing of Unbonded Brace Damper





Testing Performed
at UC Berkeley

Typical Hysteresis
Loops from
Cyclic Testing



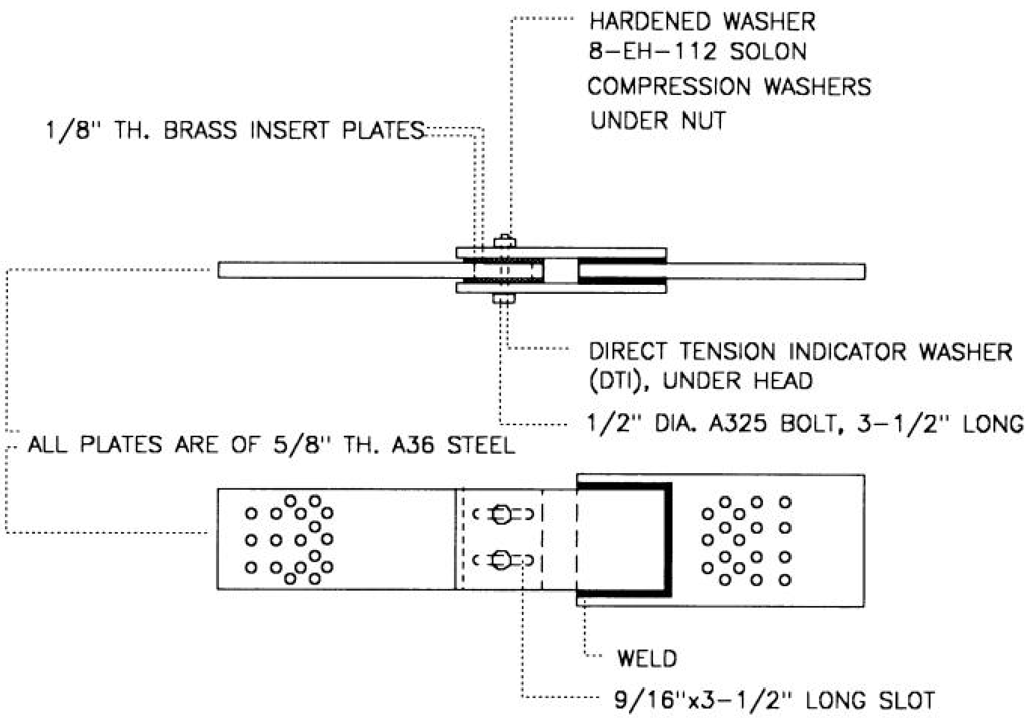
Advantages of ADAS System and Unbonded Brace Damper

- Force-Limited
- Easy to construct
- Relatively Inexpensive
- Adds both "Damping" and Stiffness

Disadvantages of ADAS System and Unbonded Brace Damper

- Must be Replaced after Major Earthquake
- Highly Nonlinear Behavior
- Adds Stiffness to System
- Undesirable Residual Deformations Possible

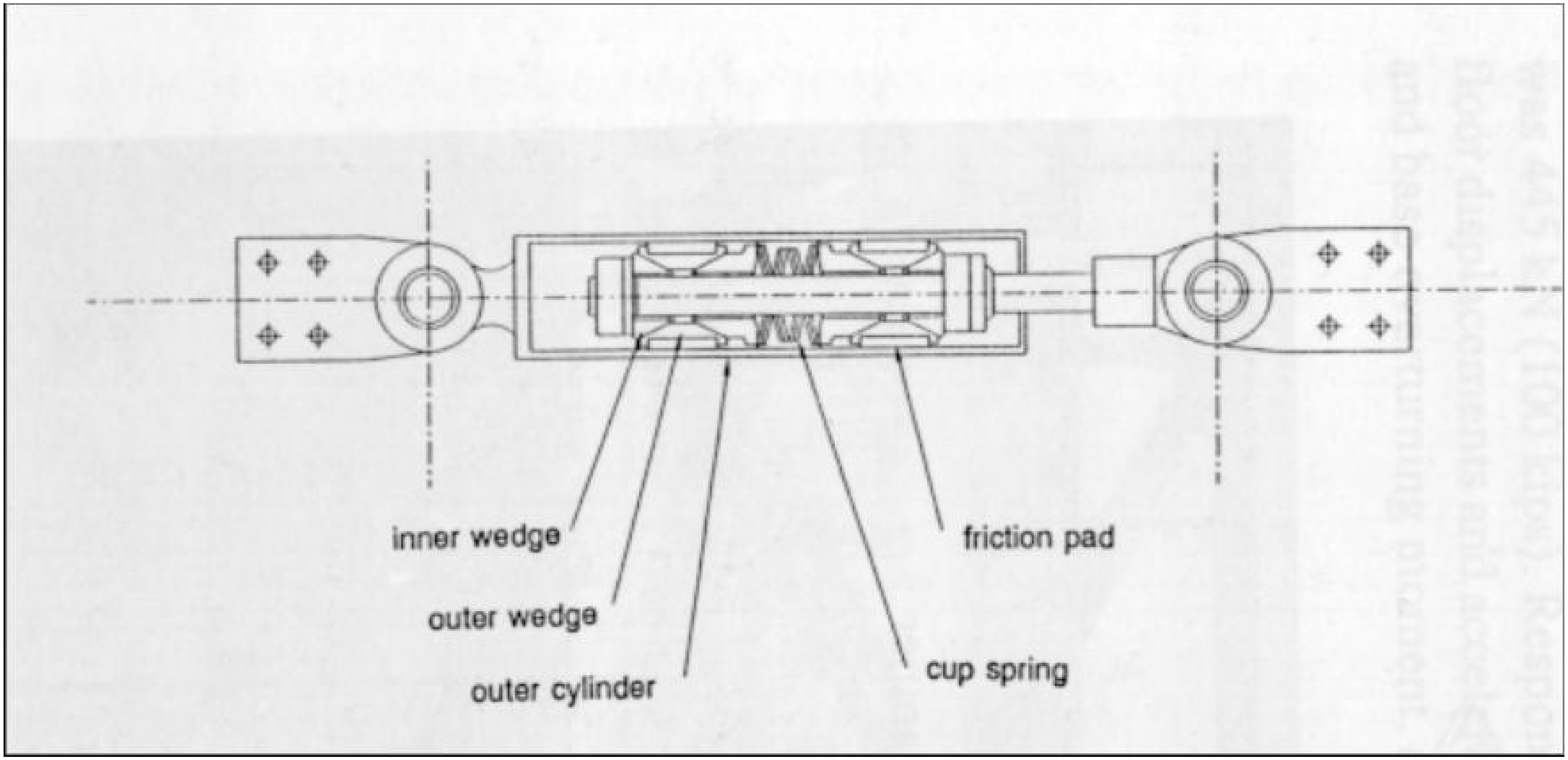
Friction Dampers: Slotted-Bolted Damper



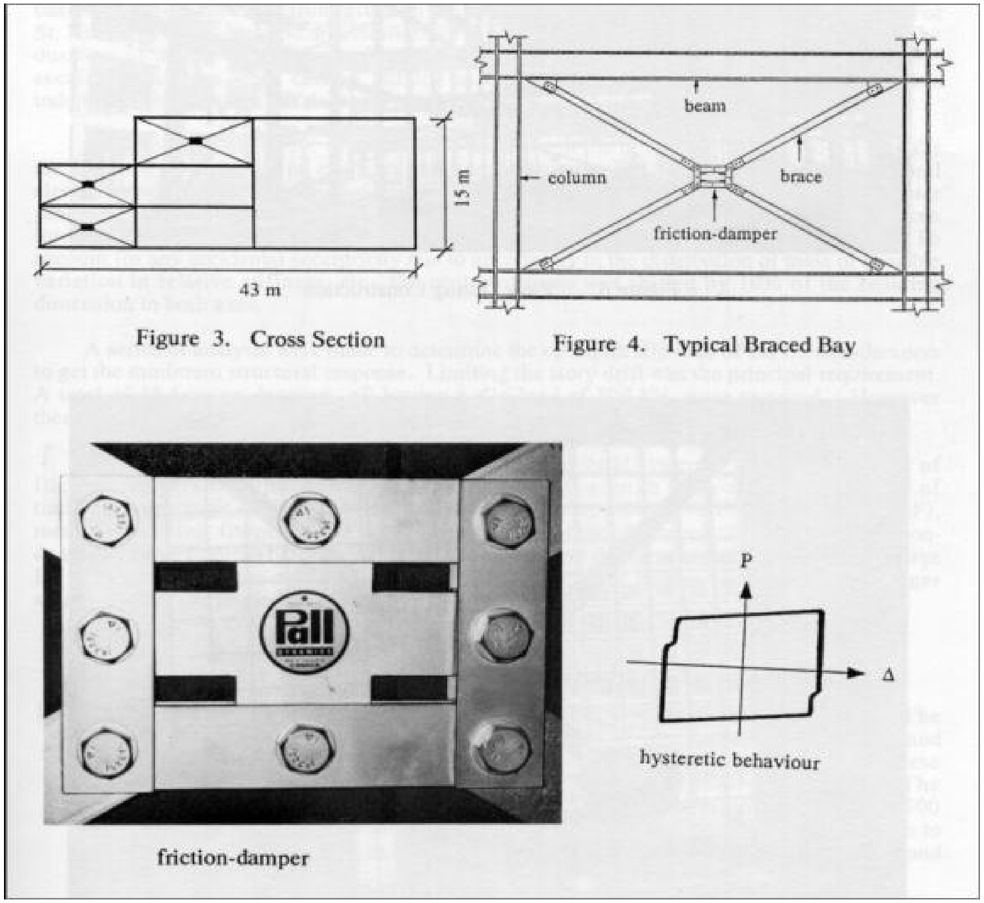
Pall Friction Damper

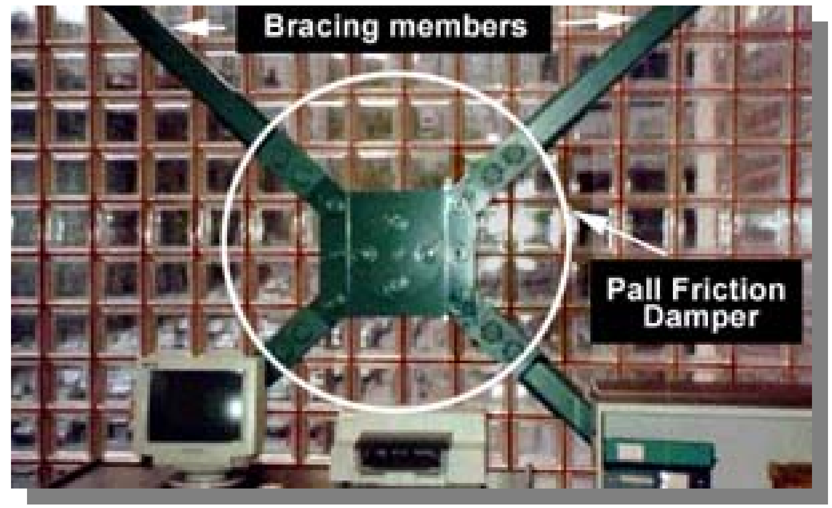
Sumitomo Friction Damper

(Sumitomo Metal Industries, Japan)



Pall Cross-Bracing Friction Damper





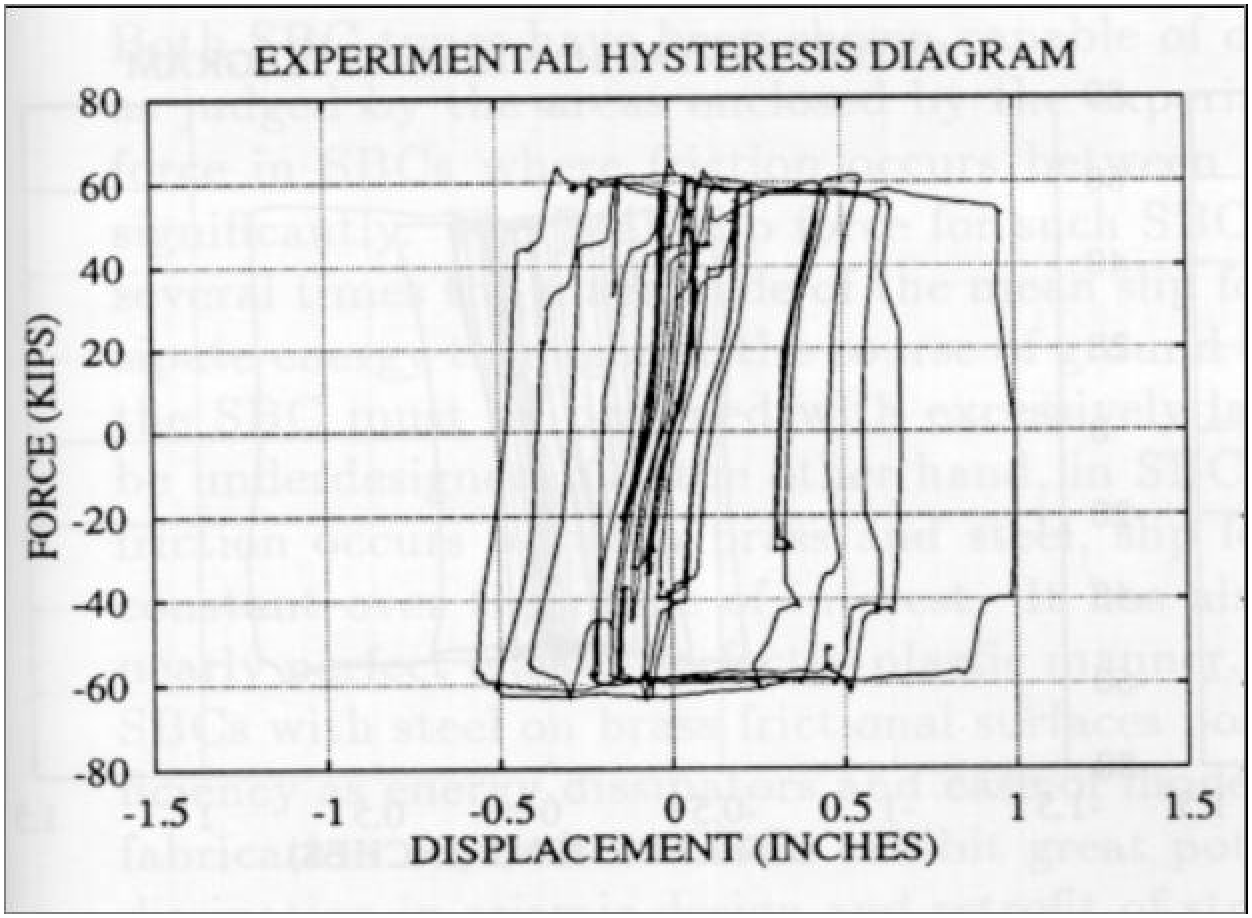
Interior of Webster Library at Concordia University, Montreal, Canada

Implementation of Pall Friction Damper

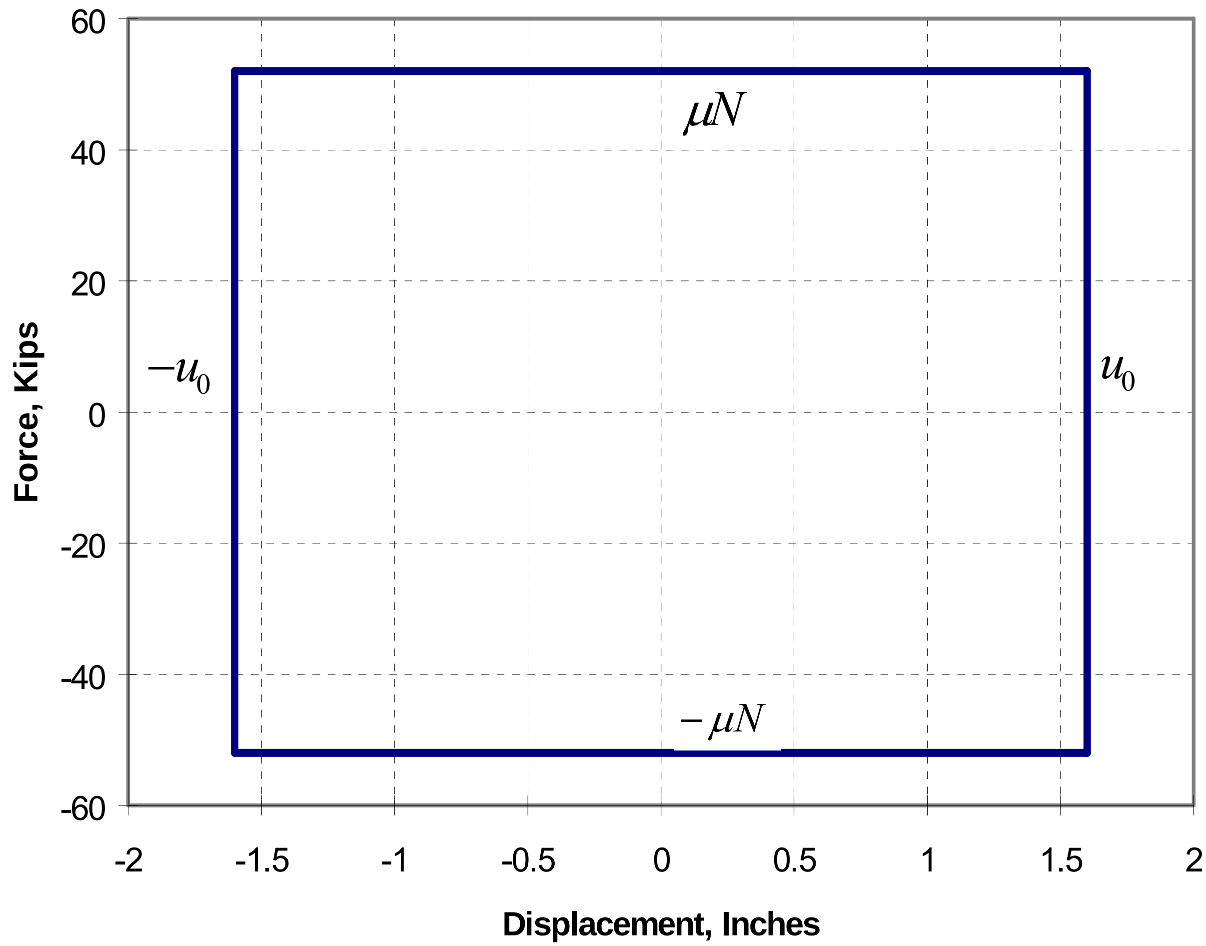
McConnel Library at Concordia University, Montreal, Canada

- Two Interconnected Buildings of 6 and 10 Stories
- RC Frames with Flat Slabs
- 143 Cross-Bracing Friction Dampers Installed in 1987
- 60 Dampers Exposed for Aesthetics

Hysteretic Behavior of Slotted-Bolted Friction Damper



Ideal Hysteretic Behavior of Friction Damper



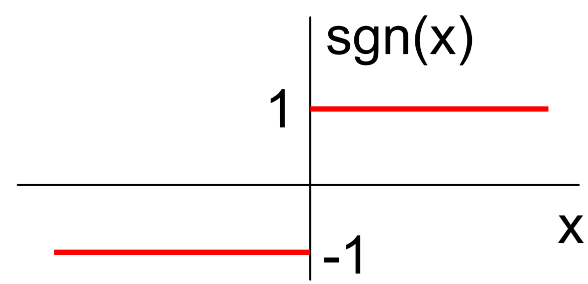
Normal Force

$$F_D = N\mu \frac{\dot{u}(t)}{|\dot{u}(t)|}$$

Coefficient of Friction

Alternatively,

$$F_D = N\mu \, sgn[\dot{u}(t)]$$



Advantages of Friction Dampers

- Force-Limited
- Easy to construct
- Relatively Inexpensive

Disadvantages of Friction Dampers

- May be Difficult to Maintain over Time
- Highly Nonlinear Behavior
- Adds Large Initial Stiffness to System
- Undesirable Residual Deformations Possible

Outline: Part II

- Velocity-Dependent Damping Systems: Fluid Dampers and Viscoelastic Dampers
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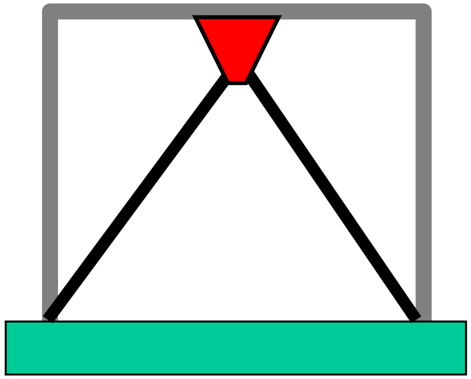
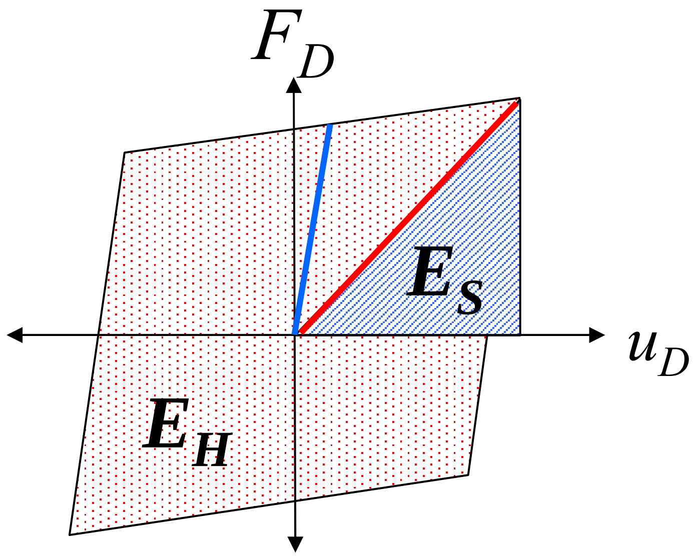
Equivalent Viscous Damping:
Damping System with Inelastic or Friction Behavior



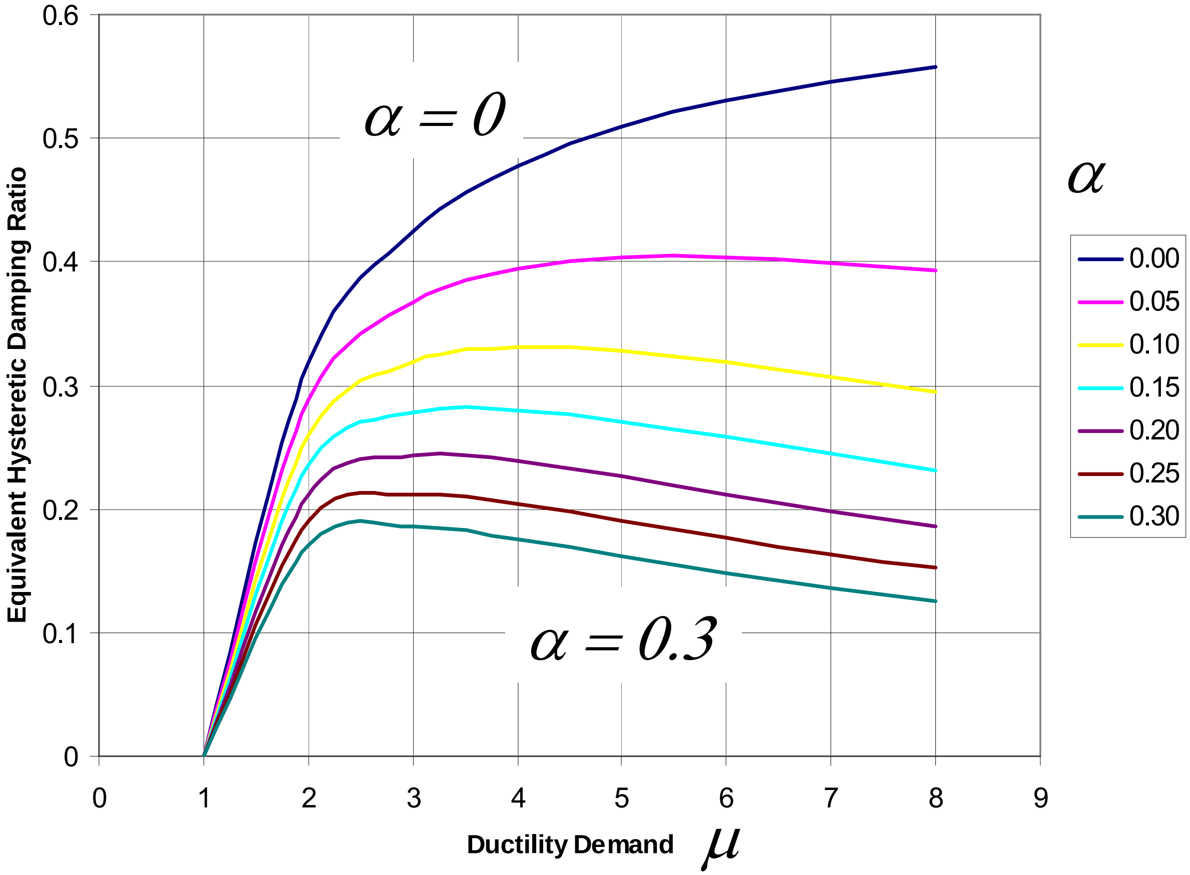


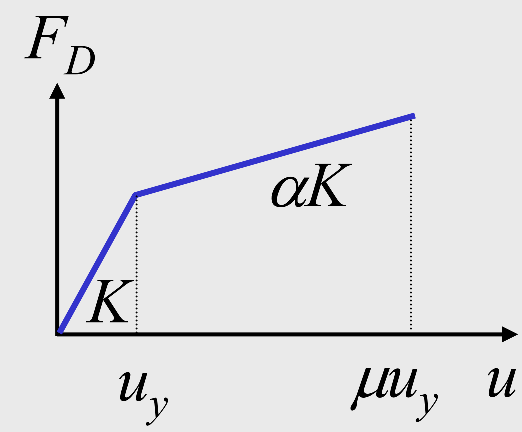
E_s based on secant stiffness

$$\xi_H = \frac{E_H}{4\pi E_S}$$

Note: Computed damping ratio is *displacement-dependent*

Effect of Inelastic System Post-Yielding Stiffness on Equivalent Viscous Damping

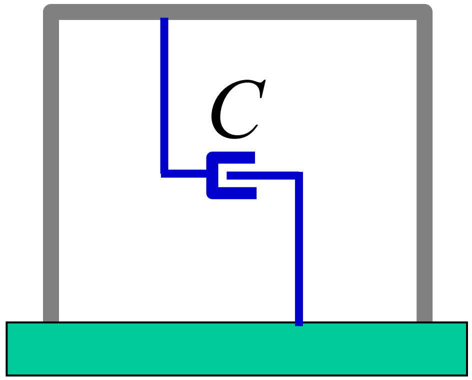


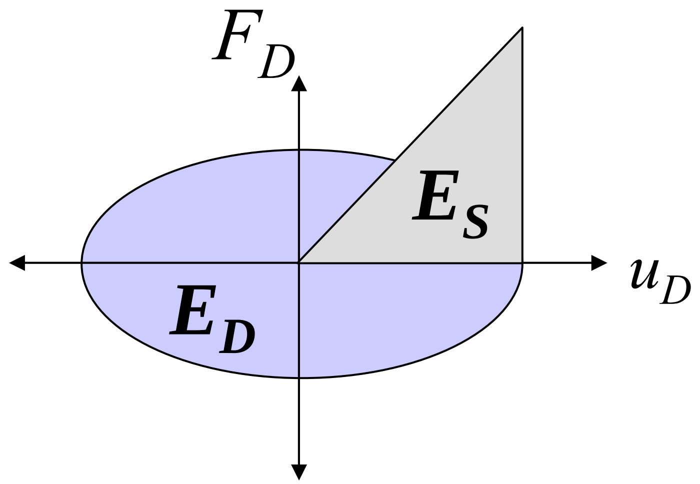


$$\xi_H = \frac{2(\mu-1)(1-\alpha)}{\pi\mu(1+\mu\alpha-\alpha)}$$

Note: May be Modified **(κ)** for Other (less Robust) Hysteretic Behavior

Equivalent Viscous Damping:
"Equivalent" System with Linear Viscous Damper





$$\boxed{C = 2m\omega\xi_H}$$

E_s and ω are based on Secant Stiffness of Inelastic System

Equivalent Viscous Damping: Caution!

- It is not possible, on a device level, to “replace” a displacement-dependent device (e.g. a Friction Damper) with a velocity-dependent device (e.g. a Fluid Damper).
- Some simplified procedures allow such replacement on a structural level, wherein a “smeared” equivalent viscous damping ratio is found for the whole structure. This approach is marginally useful for preliminary design, and should not be used under any circumstances for final design.

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Modeling Considerations for Structures with Passive Energy Dissipation Devices

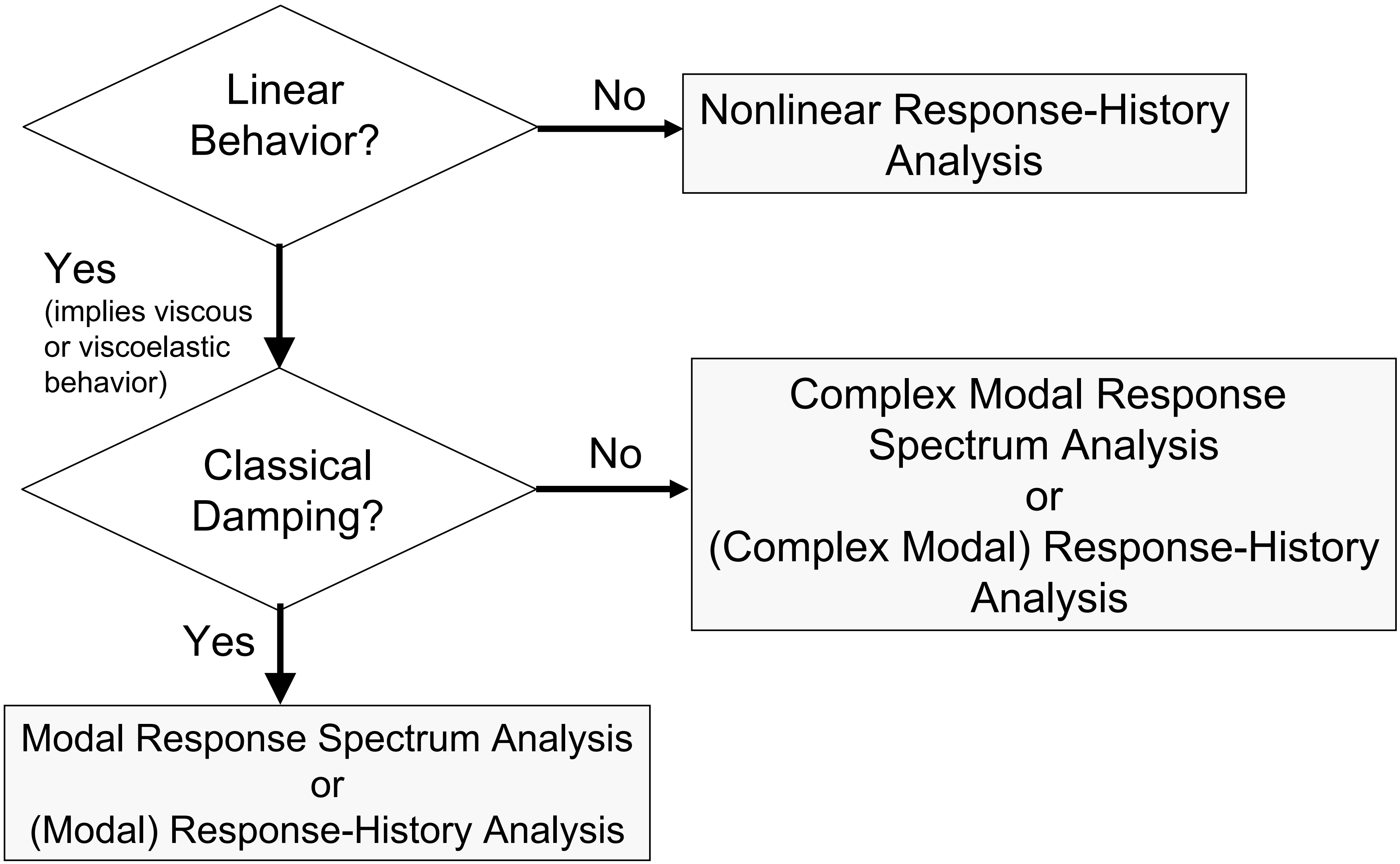
- Damping is almost always nonclassical (Damping matrix is not proportional to stiffness and/or mass)
- For seismic applications, system response is usually partially inelastic
- For seismic applications, viscous damper behavior is typically nonlinear (velocity exponents in the range of 0.5 to 0.8)

Conclusion: This is a ***NONLINEAR*** analysis problem!

Outline: Part III

- Seismic Analysis of MDOF Structures with Passive Energy Dissipation Systems
- Representations of Damping
- Examples: Application of Modal Strain Energy Method and Non-Classical Damping Analysis
- Summary of MDOF Analysis Procedures

Seismic Analysis of Structures with Passive Energy Dissipation Systems



Seismic Analysis of MDOF Structures with Passive Energy Dissipation Systems

$$M\ddot{v}(t)+C_I\dot{v}(t)+C_A\dot{v}(t)+F_S(t)=-MR\ddot{v}_g(t)$$

Inherent Damping:
Linear

Added Viscous Damping:
Linear or ***Nonlinear***

Restoring Force:
(May include Added Devices)
Linear or ***Nonlinear***

$$\boxed{C_A \neq f(\omega)}$$

MDOF Solution Techniques

$$M\ddot{v}(t)+C_I\dot{v}(t)+C_A\dot{v}(t)+F_S(t)=-MR\ddot{v}_g(t)$$

Explicit integration of fully coupled equations:

- Treat C_I as Rayleigh damping and model C_A explicitly.
- Use Newmark solver (with iteration) to solve full set of coupled equations.

System may be linear or nonlinear.

MDOF Solution Techniques

$$M\ddot{v}(t) + C_I\dot{v}(t) + C_A\dot{v}(t) + F_S(t) = -MR\ddot{v}_g(t)$$

Fast Nonlinear Analysis:
Treat C_I as modal damping and model C_A explicitly. Move C_A (and any other nonlinear terms) to right-hand side. Left-hand side may be uncoupled by Ritz Vectors. Iterate on unbalanced right-hand side forces.

System may be linear or nonlinear.

Fast Nonlinear Analysis

$$M\ddot{v}(t)+C_I\dot{v}(t)+C_A\dot{v}(t)+K_E v(t)+F_H(t)=-MR\ddot{v}_g(t)$$

Nonlinear Restoring Force

Move Added Damper Forces
and Nonlinear Forces to RHS:

$$\underbrace{M\ddot{v}(t)+C_I\dot{v}(t)+K_E v(t)}_{\text{Linear Terms}}=-MR\ddot{v}_g(t)\underbrace{-F_H(t)-C_A\dot{v}(t)}_{\text{Nonlinear Terms}}$$

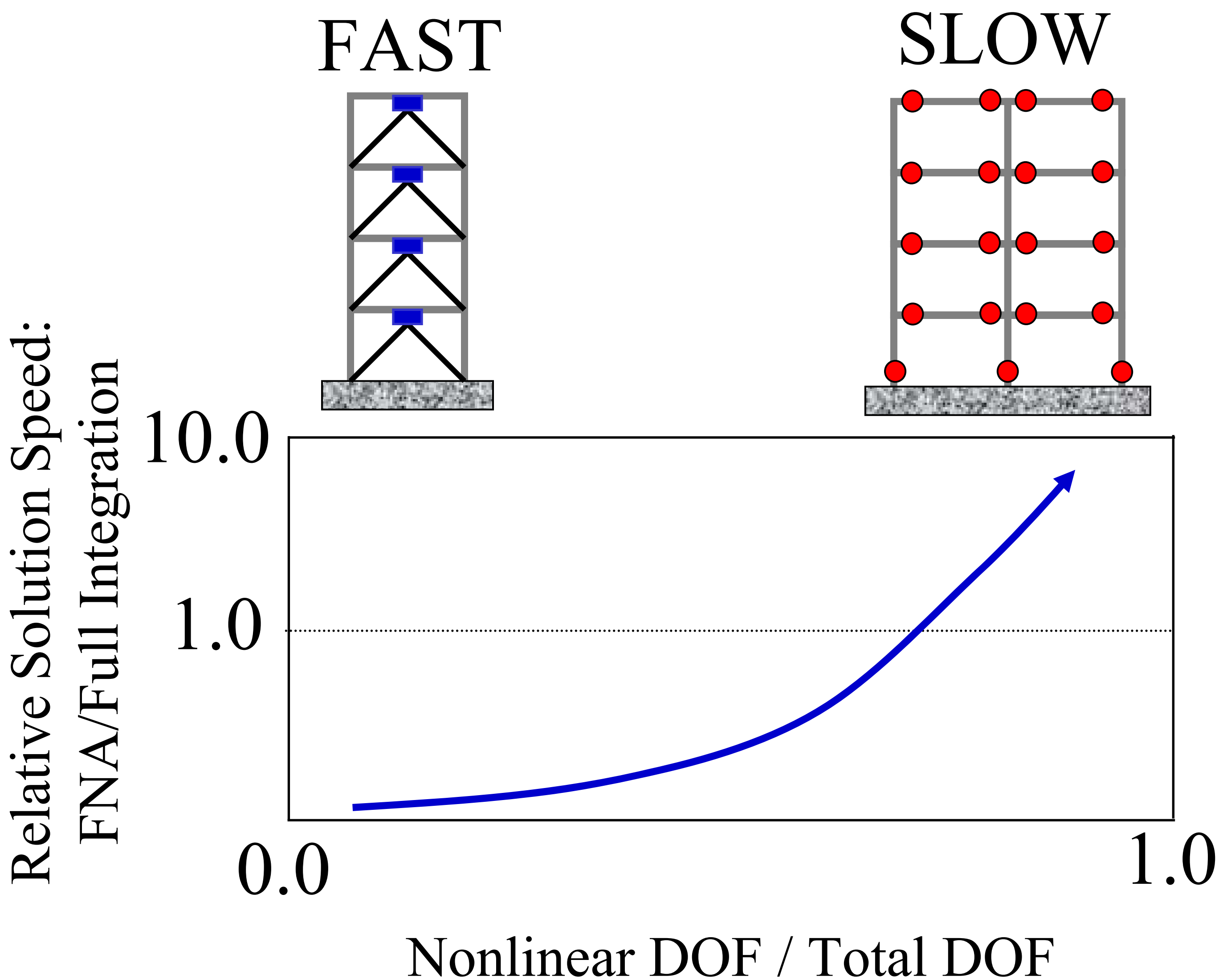
Transform Coordinates: $v(t)=\Phi y(t)$

Orthogonal basis of Ritz vectors:
Number of vectors << N

Apply Transformation:

$$\underbrace{\tilde{M}\ddot{y}(t)+\tilde{C}_I\dot{y}(t)+\tilde{K}_E y(t)}_{\text{Uncoupled}}=\underbrace{-\Phi^T MR\ddot{v}_g(t)-\Phi^T F_H(t)-\tilde{C}_A\dot{y}(t)}_{\text{Coupled}}$$

Fast Nonlinear Analysis



MDOF Solution Techniques

$$M\ddot{v}(t)+C_I\dot{v}(t)+C_A\dot{v}(t)+F_S(t)=-MR\ddot{v}_g(t)$$

Explicit integration or response spectrum analysis of first few uncoupled modal equations:

- Treat C_I as modal damping or Rayleigh damping
- Use Modal Strain Energy method to represent C_A as modal damping ratios.

System must be linear.
Applicable only to viscous (or viscoelastic) damping systems.

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Modal Damping Ratios

$$M\ddot{v} + C\dot{v} + Kv = -MR\ddot{v}_g$$

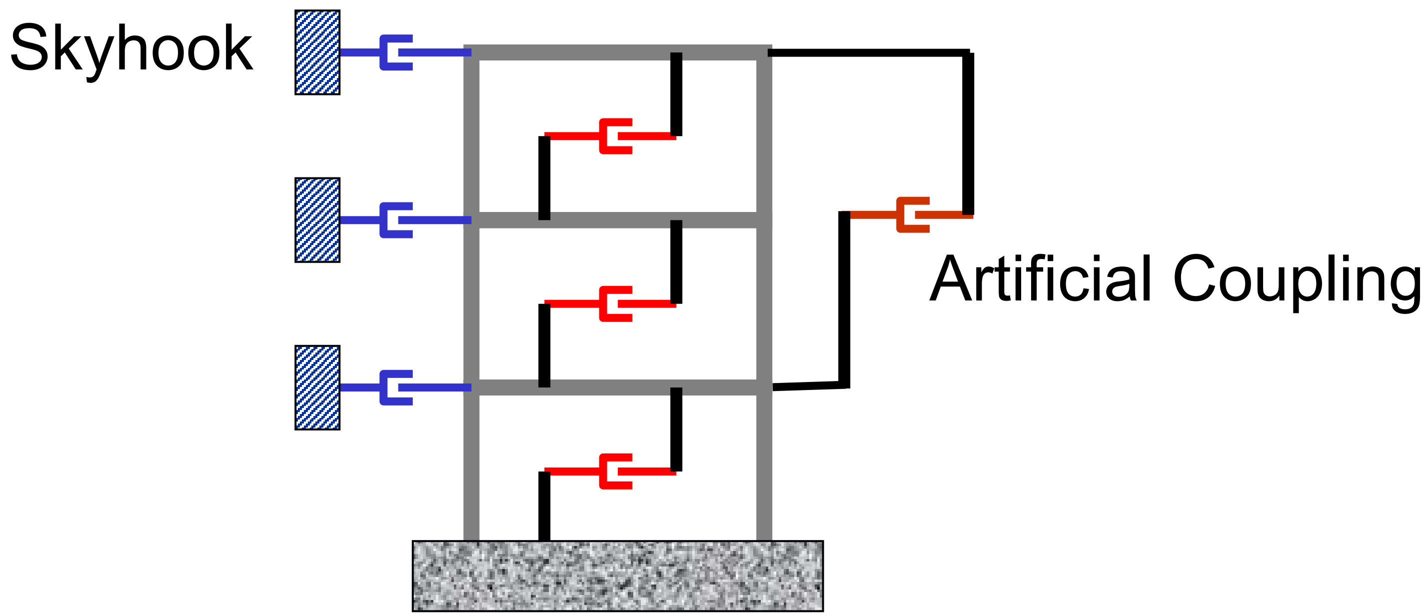
$$v = \Phi y$$

$$\ddot{y}_i + 2\xi_i \omega_i \dot{y}_i + \omega_i^{\,2} y_i = \Gamma_i \ddot{v}_g$$

Specify modal damping values directly

Modal Superposition Damping

$$C = M\left[\sum_{i=1}^{n} \frac{2\xi_i \omega_i}{\phi_i^T M \phi_i} \phi_i^T \phi_i\right] M$$

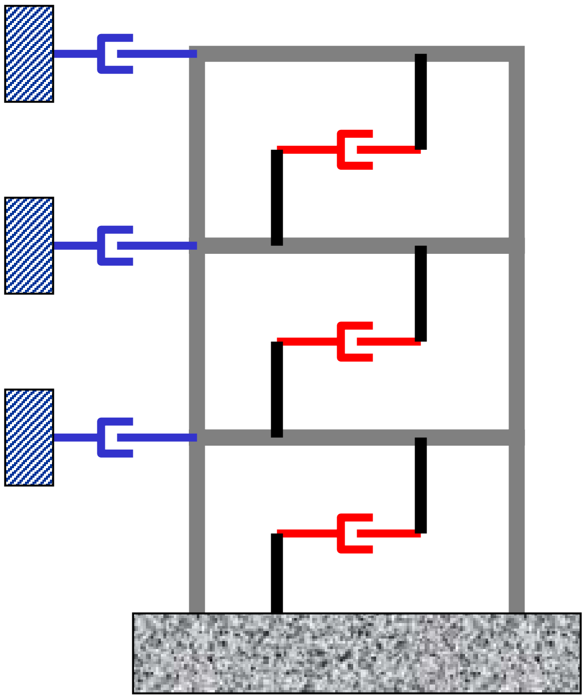


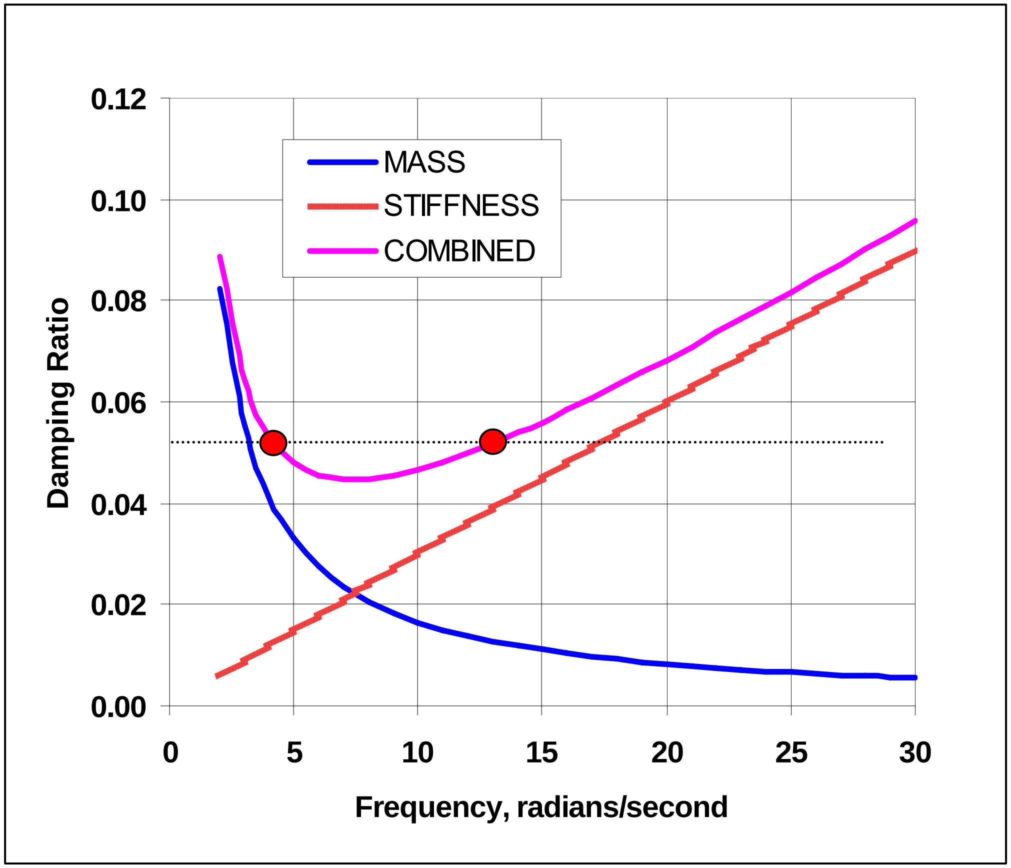
Note: There is no need to develop C explicitly.

Rayleigh Proportional Damping

$$C_R = \alpha M + \beta K$$

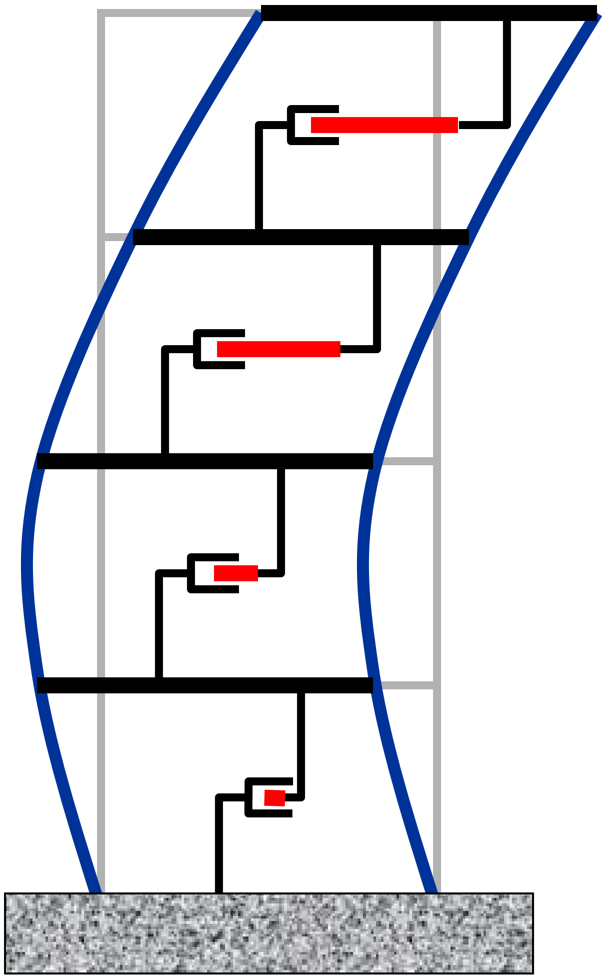
Skyhook





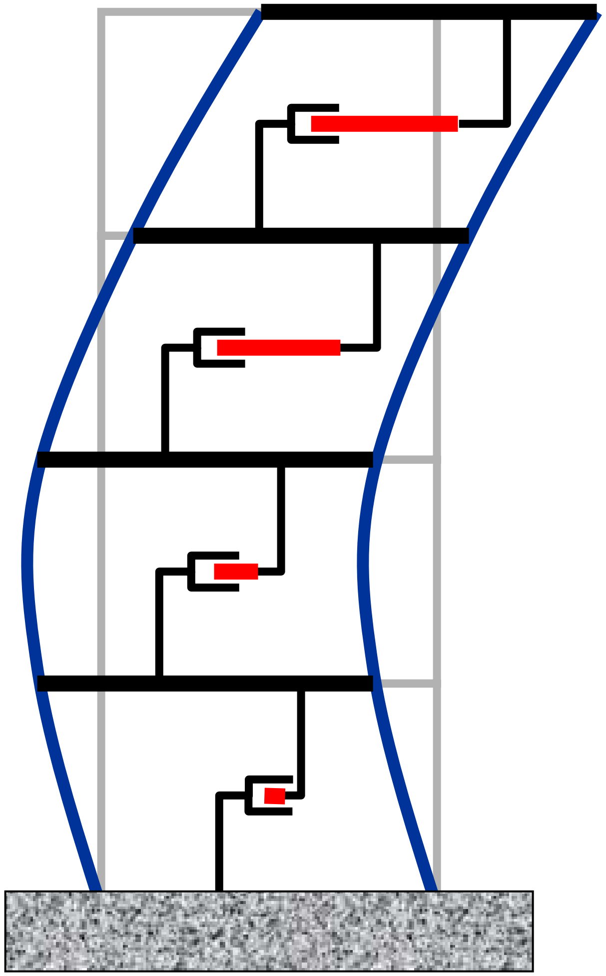
Derivation of Modal Strain Energy Method

Deformation of Structure
in its Second Mode



Floor Displacement	Damper Deformation
$\phi_{2,1}$	$\phi_{2,1} - \phi_{2,2}$
$\phi_{2,2}$	$\phi_{2,2} - \phi_{2,3}$
$\phi_{2,3}$	$\phi_{2,3} - \phi_{2,4}$
$\phi_{2,4}$	$\phi_{2,4}$

Derivation of Modal Strain Energy Method

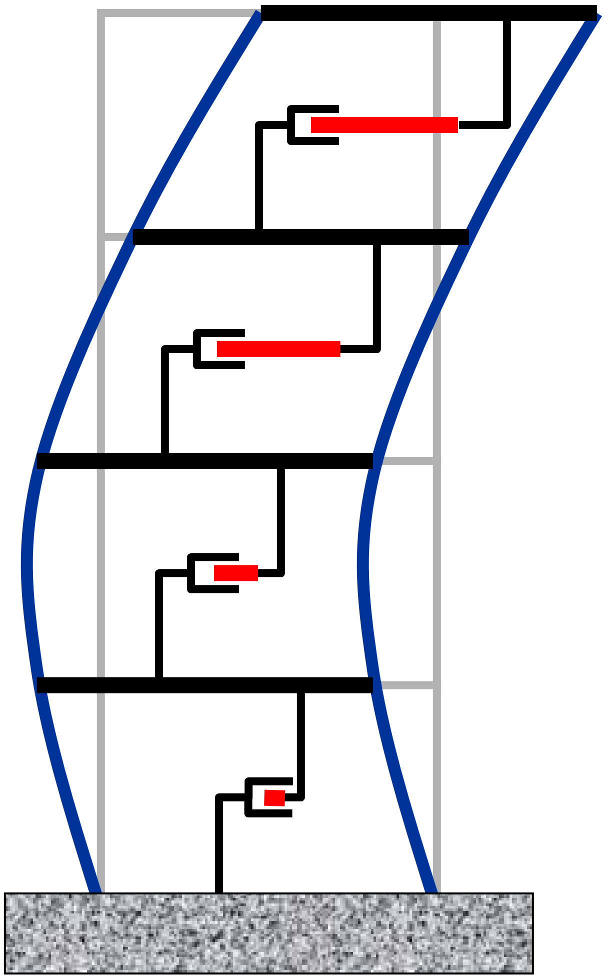


$$E_{D,i,story\,k} = \pi\omega_i C_k(\phi_{i,k} - \phi_{i,k-1})^2$$

$$E_{S,i} = \frac{1}{2}\phi_i^T K\phi_i = \frac{1}{2}\omega_i^2\phi_i^T M\phi_i$$

$$\xi_i = \frac{\sum_{k=1}^{n\ stories} E_{D,i,story\,k}}{4\pi E_{S,i}}$$

Derivation of Modal Strain Energy Method



$$\xi_i = \frac{\sum_{k=1}^{N\ stories} C_k (\phi_{i,k} - \phi_{i,k-1})^2}{2\omega_i \phi_i^T M \phi_i}$$

$$\xi_i = \frac{\phi_i^T C_A \phi_i}{2\omega_i \phi_i^T M \phi_i} = \frac{\phi_i^T C_A \phi_i}{2 m_i^* \omega_i}$$

Note: IF C_A is diagonalized by Φ,
THEN

$$\xi_i = \frac{c_i^*}{2 m_i^* \omega_i}$$

Modal Strain Energy Damping Ratio

$$\xi_i = \frac{\phi_i^T C_A \phi_i}{2 m_i^* \omega_i}$$

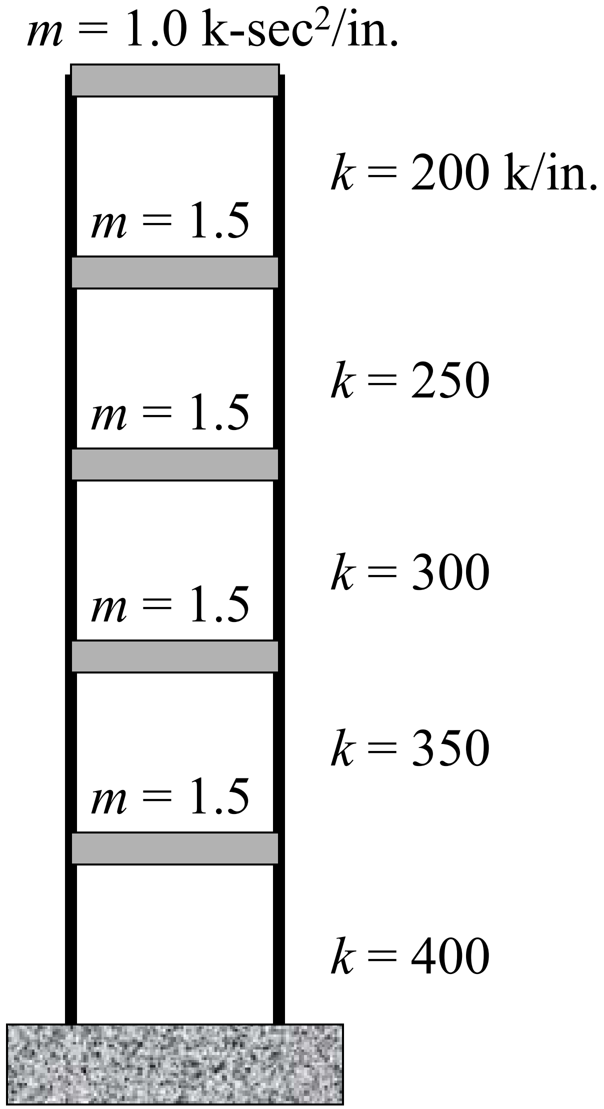
Note: ϕ is the *Undamped* Mode Shape

The Modal Strain Energy Method is approximate if the structure is non-classically damped since the undamped and damped mode shapes are different.

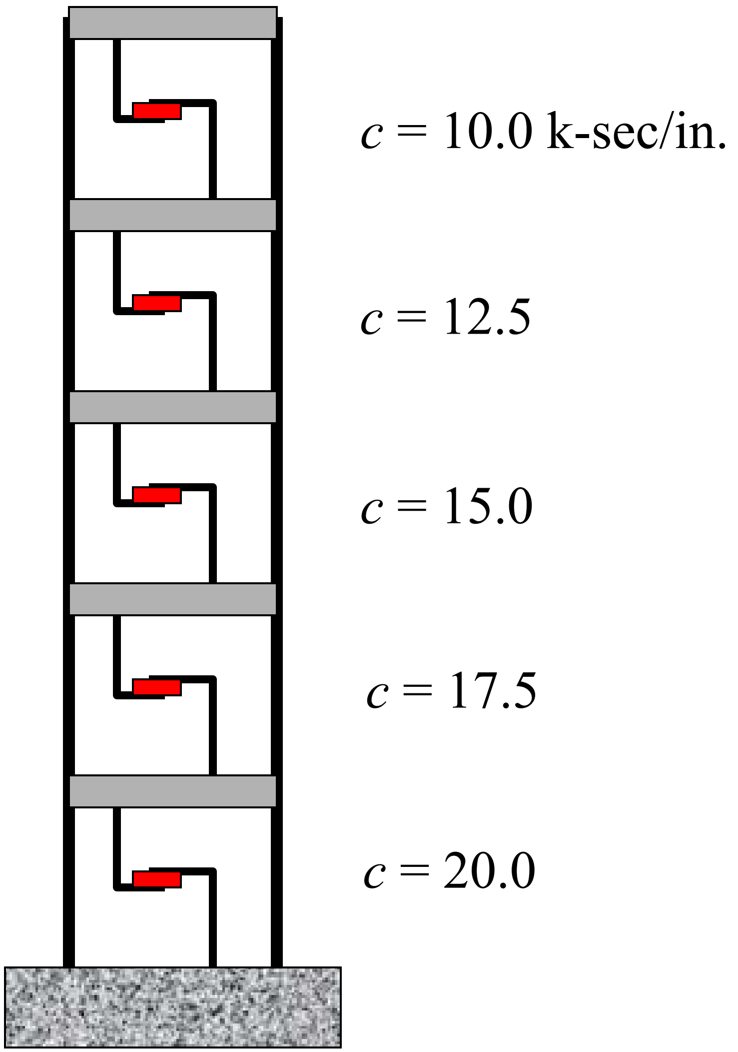
Outline: Part III

- Seismic Analysis of MDOF Structures with Passive Energy Dissipation Systems
- Representations of Damping
- Examples: Application of Modal Strain Energy Method and Non-Classical Damping Analysis
- Summary of MDOF Analysis Procedures

Example: Application of Modal Strain Energy Method

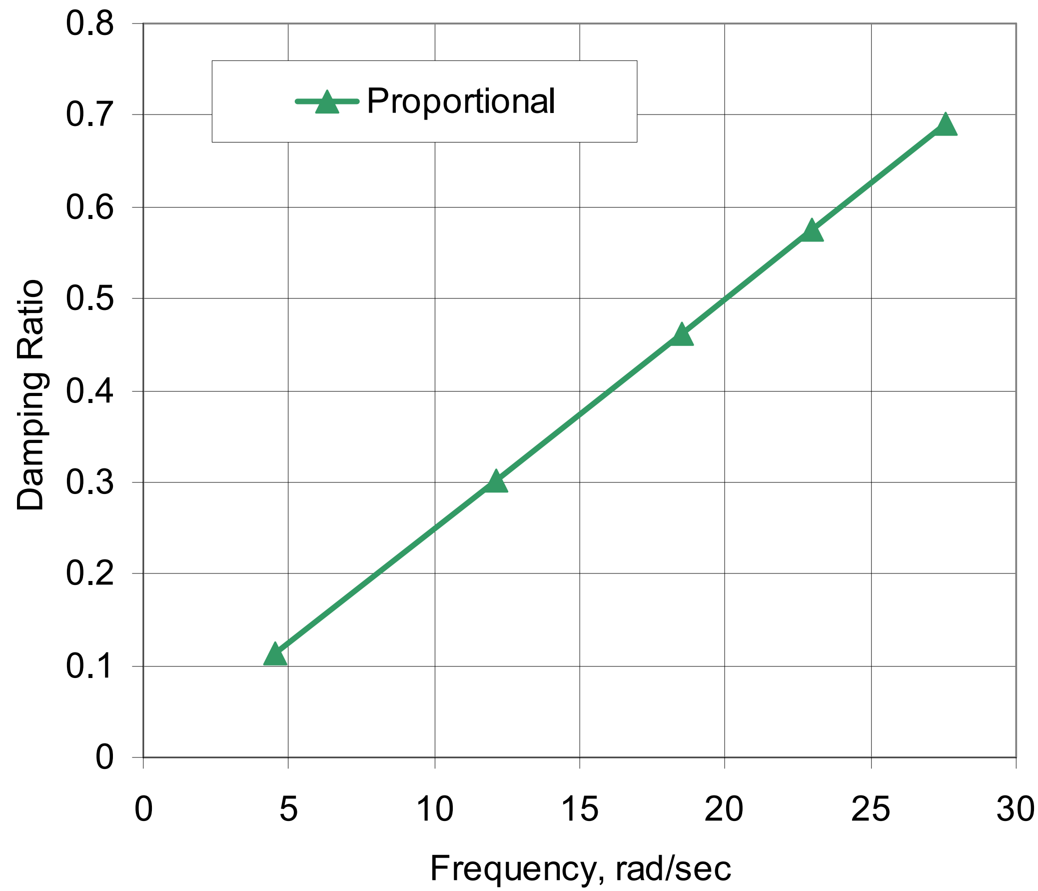


Proportional Damping

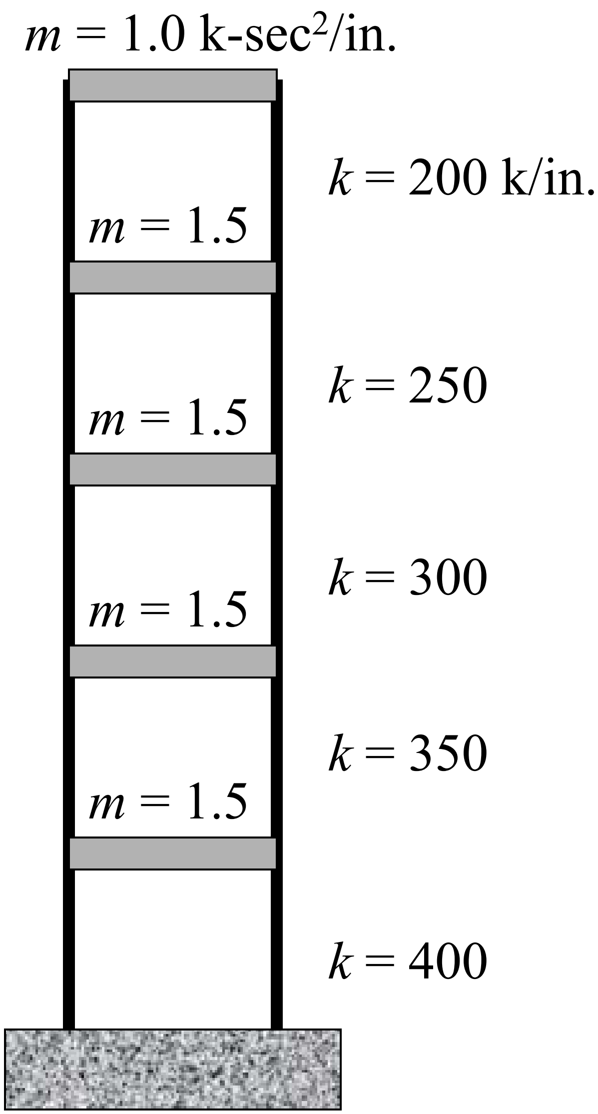


Modal Damping Ratios from Modal Strain Energy Method for *Proportional* Damping Distribution

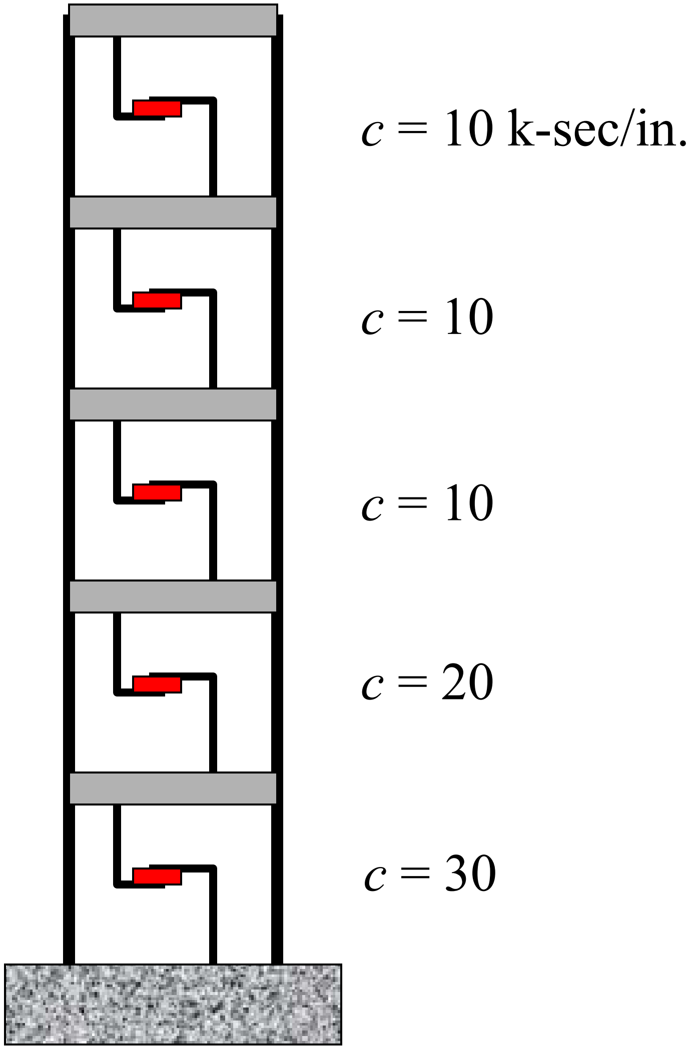
Frequency (rad/sec)	Damping Ratio, ξ
4.54	0.113
12.1	0.302
18.5	0.462
23.0	0.575
27.6	0.690



Example: Application of Modal Strain Energy Method

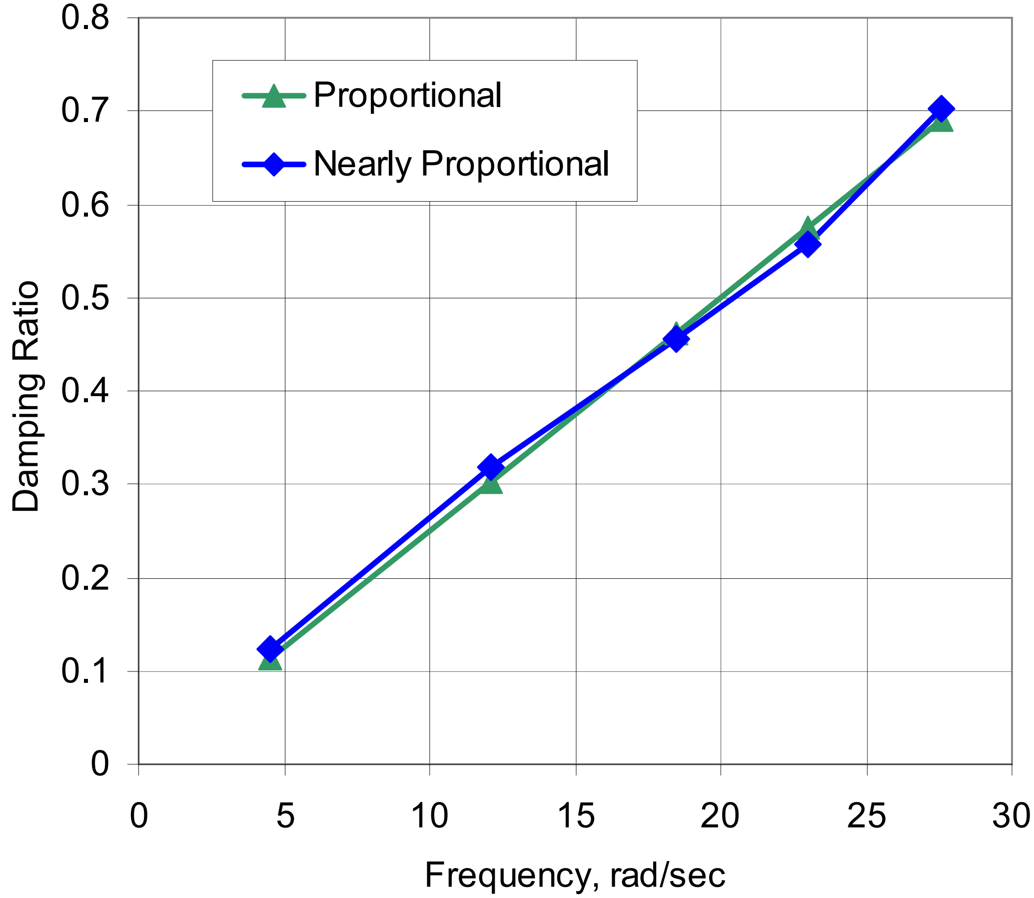


Nearly Proportional Damping

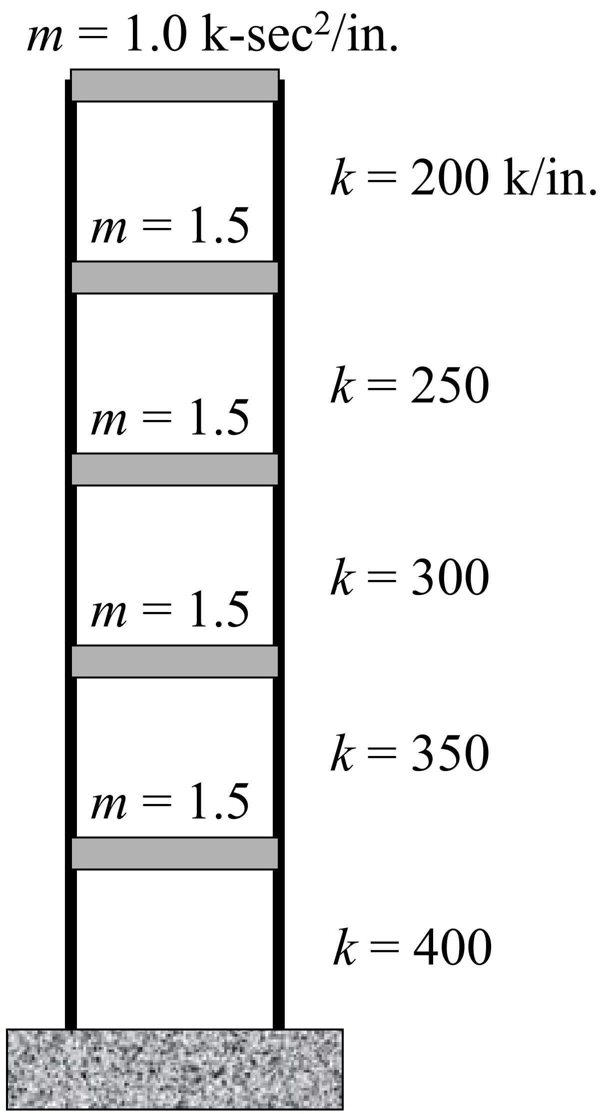


Modal Damping Ratios from Modal Strain Energy Method for *Nearly Proportional* Damping Distribution

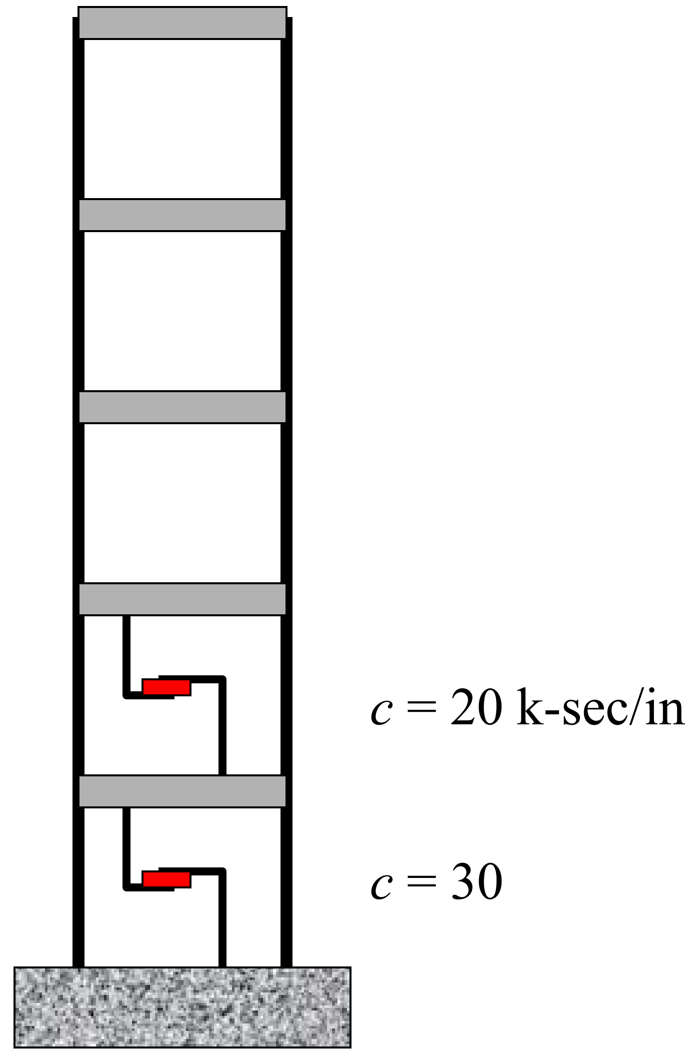
Frequency (rad/sec)	Damping Ratio, ξ
4.54	0.123
12.1	0.318
18.5	0.455
23.0	0.557
27.6	0.702



Example: Application of Modal Strain Energy Method

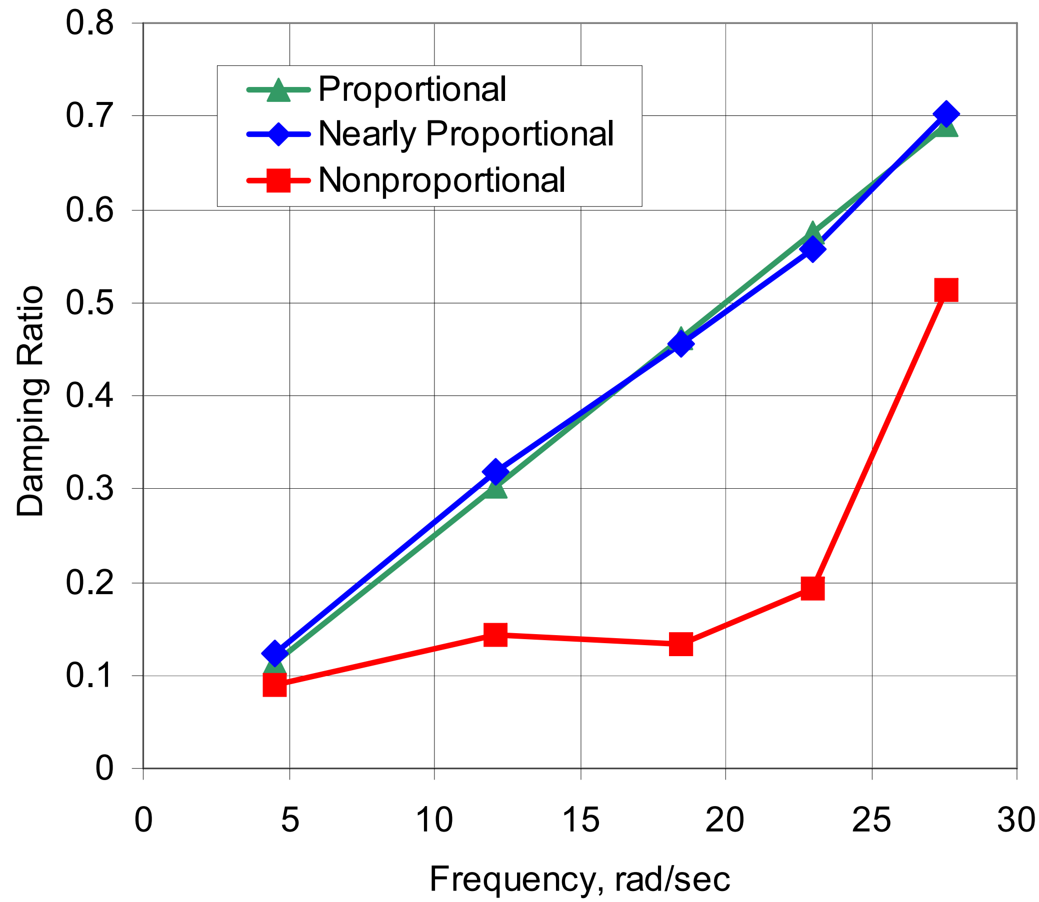


Nonproportional Damping



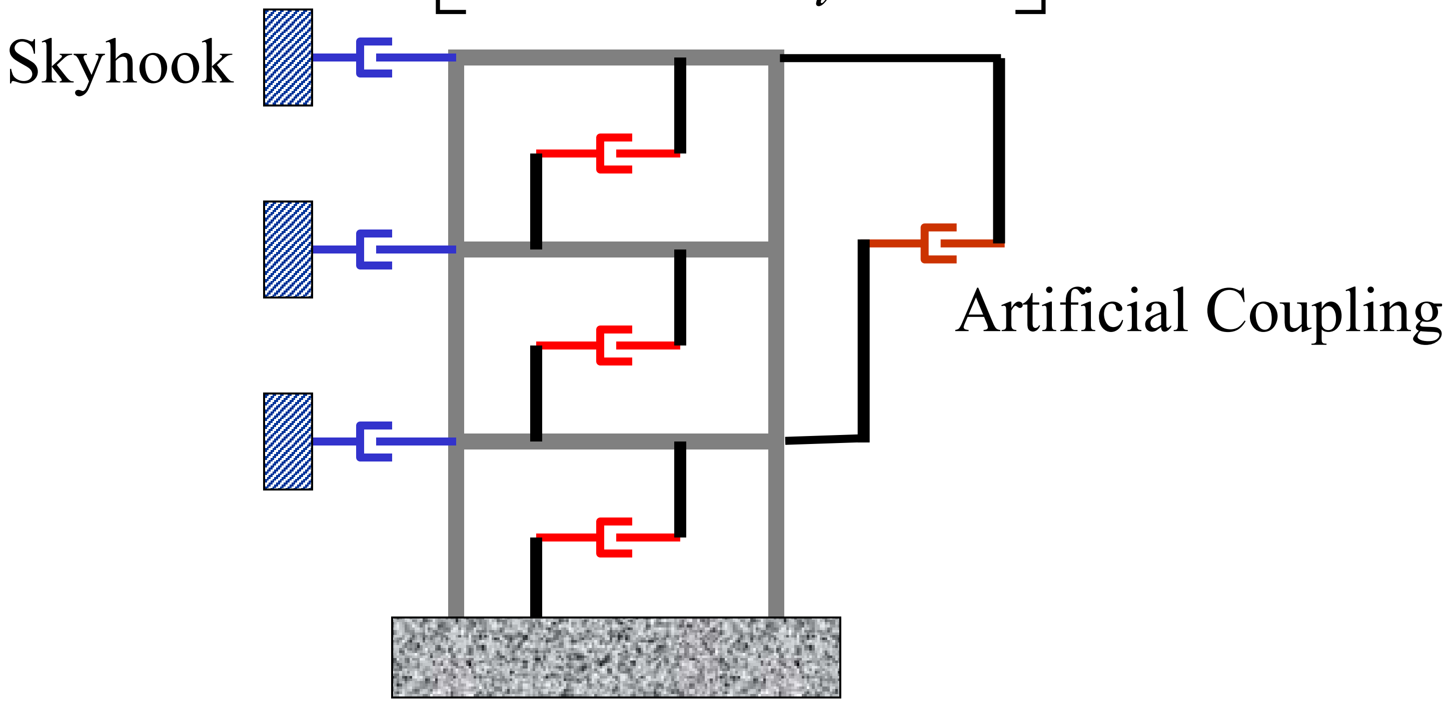
Modal Damping Ratios from Modal Strain Energy Method for *Nonproportional* Damping Distribution

Frequency (rad/sec)	Damping Ratio. ξ
4.54	0.089
12.1	0.144
18.5	0.134
23.0	0.194
27.6	0.514



Modal Superposition Damping

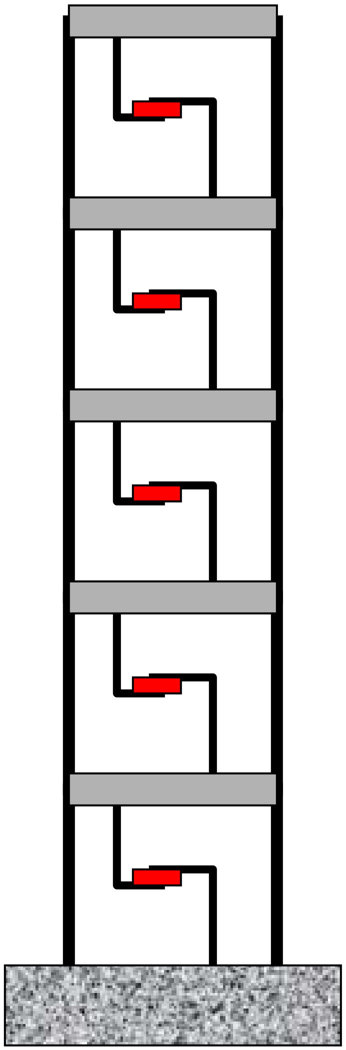
$$C = M\left[\sum_{i=1}^{n} \frac{2\xi_i\omega_i}{\phi_i^T M \phi_i} \phi_i^T \phi_i\right] M$$



Modal Superposition Damping can be used to construct the damping matrix from the modal damping ratios obtained via the Modal Strain Energy Method

Comparison of Actual Damping Matrix and Damping Matrix Obtained from MSE Damping Ratios

Proportional Damping



$c = 10.0$ k-sec/in.

$c = 12.5$

$c = 15.0$

$c = 17.5$

$c = 20.0$

Actual Damping Matrix

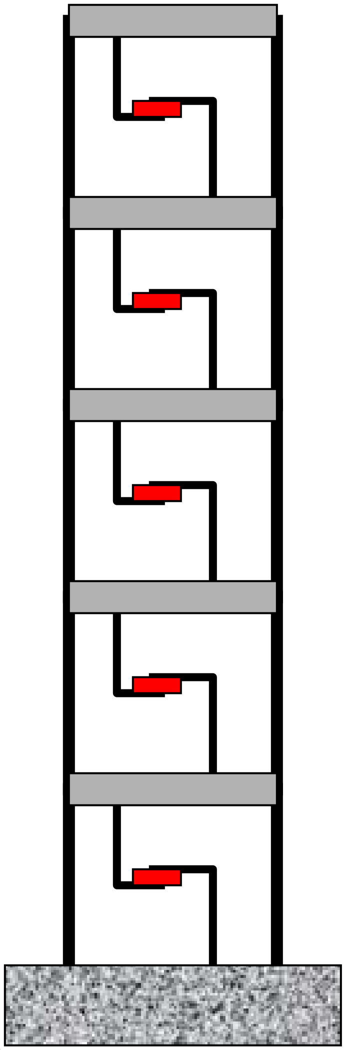
$$C_A = \begin{bmatrix} 10.0 & -10.0 & 0 & 0 & 0 \\ -10.0 & 22.5 & -12.5 & 0 & 0 \\ 0 & -12.5 & 27.5 & -15.0 & 0 \\ 0 & 0 & -15.0 & 32.5 & -17.5 \\ 0 & 0 & 0 & -17.5 & 37.5 \end{bmatrix}$$

Modal Superposition Damping Matrix Using MSE Damping Ratios

$$C = \begin{bmatrix} 10.0 & -10.0 & 0 & 0 & 0 \\ -10.0 & 22.5 & -12.5 & 0 & 0 \\ 0 & -12.5 & 27.5 & -15.0 & 0 \\ 0 & 0 & -15.0 & 32.5 & -17.5 \\ 0 & 0 & 0 & -17.5 & 37.5 \end{bmatrix}$$

Comparison of Actual Damping Matrix and Damping Matrix Obtained from MSE Damping Ratios

Nearly Proportional Damping



Actual Damping Matrix

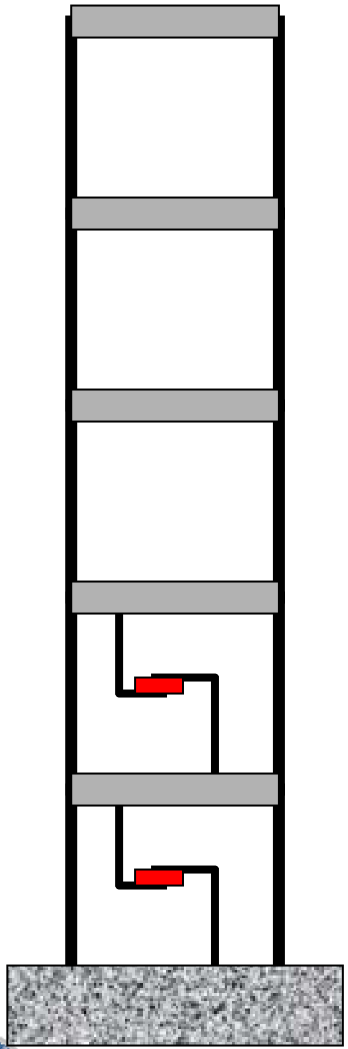
$$C_A = \begin{bmatrix} 10.0 & -10.0 & 0 & 0 & 0 \\ -10.0 & 20.0 & -10.0 & 0 & 0 \\ 0 & -10.0 & 20.0 & -10.0 & 0 \\ 0 & 0 & -10.0 & 30.0 & -20.0 \\ 0 & 0 & 0 & -20.0 & 50.0 \end{bmatrix}$$

Modal Superposition Damping Matrix Using MSE Damping Ratios

$$C = \begin{bmatrix} 10.0 & -9.66 & -0.166 & -0.228 & -0.010 \\ -9.66 & 22.0 & -12.2 & -0.169 & -0.422 \\ -0.166 & -12.2 & 27.3 & -15.1 & -0.731 \\ -0.228 & -0.169 & -15.1 & 33.1 & -17.8 \\ -0.010 & -0.422 & -0.731 & -17.8 & 37.5 \end{bmatrix}$$

Comparison of Actual Damping Matrix and Damping Matrix Obtained from MSE Damping Ratios

Nonproportional Damping



$c = 20.0$ k-sec/in.

$c = 30.0$

Actual Damping Matrix

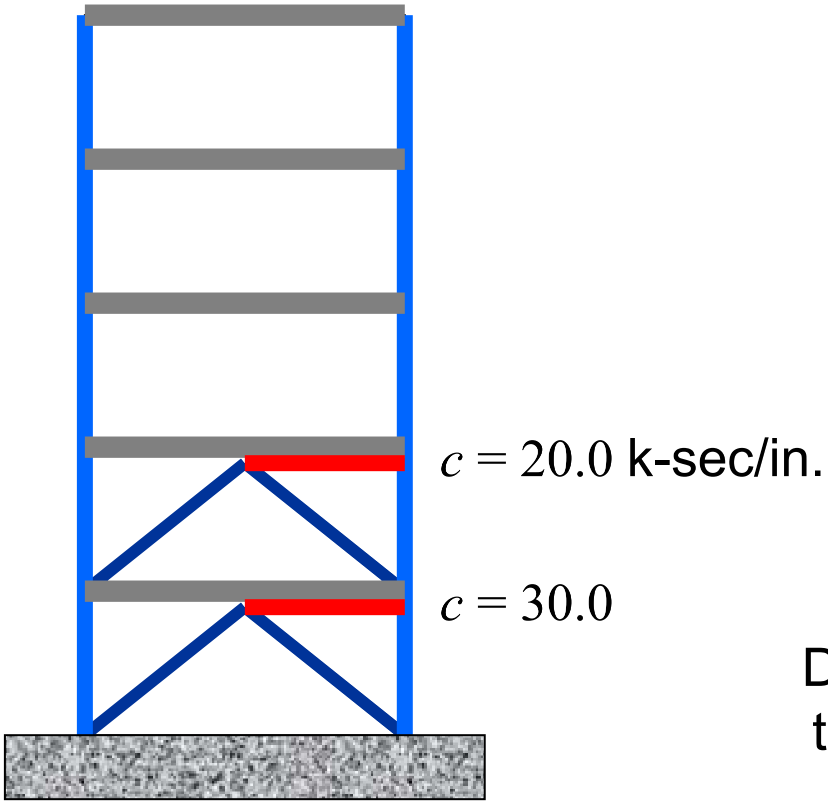
$$C_A = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 20.0 & -20.0 \\ 0 & 0 & 0 & -20.0 & 50.0 \end{bmatrix}$$

Modal Superposition Damping Matrix Using MSE Damping Ratios

$$C = \begin{bmatrix} 3.65 & -2.96 & 0.456 & -1.098 & 0.066 \\ -2.96 & 8.27 & -5.92 & 2.72 & -2.07 \\ 0.456 & -5.92 & 13.4 & -10.9 & 6.21 \\ -1.098 & 2.72 & -10.9 & 21.9 & -15.1 \\ 0.066 & -2.07 & 6.21 & -15.1 & 20.9 \end{bmatrix}$$

Example: Seismic Analysis of a Structure with Nonproportional Damping

- Discrete Damping vs Rayleigh Damping
- Discrete Damping: Rigid vs Flexible Braces

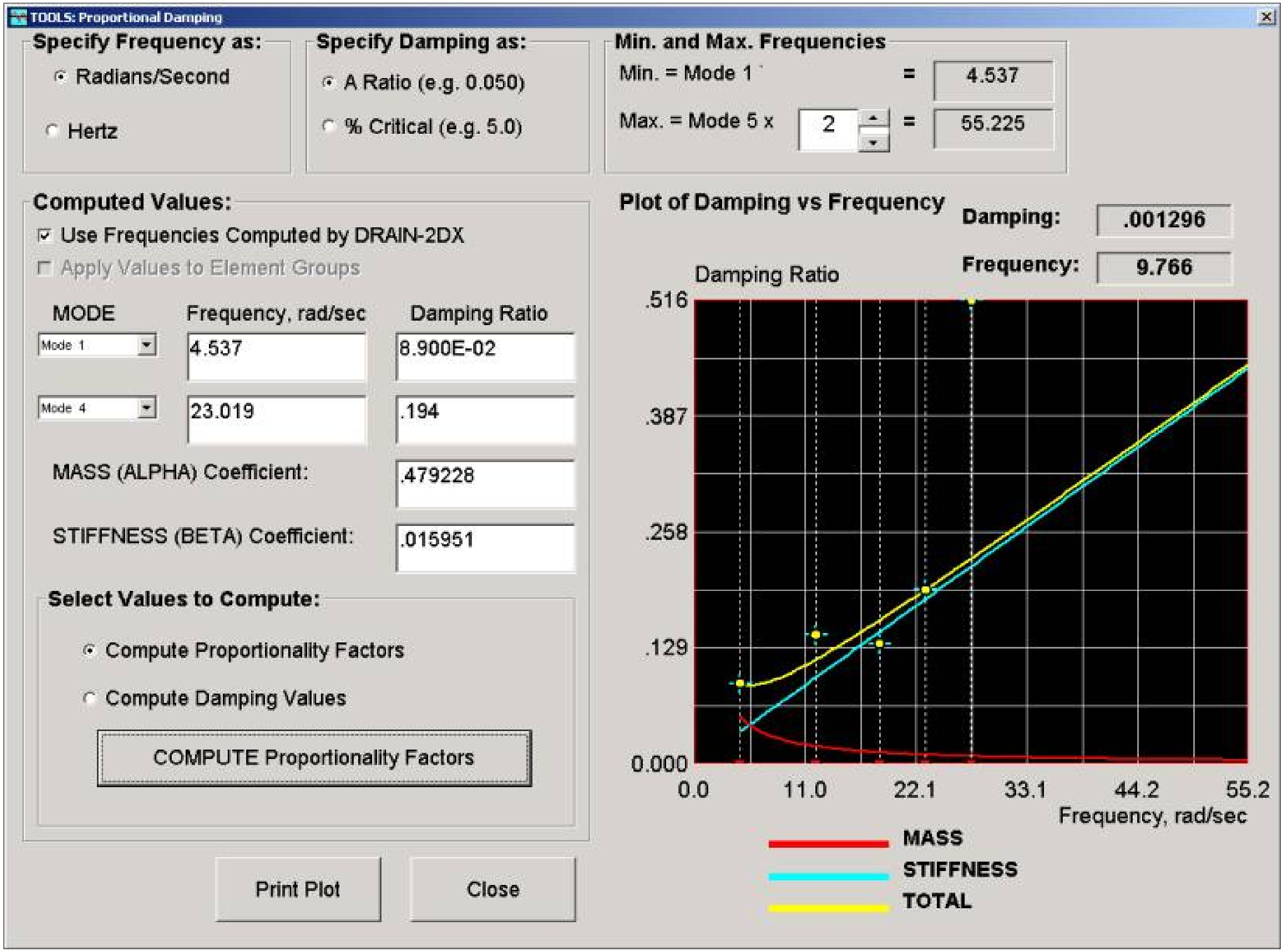


MSE Results

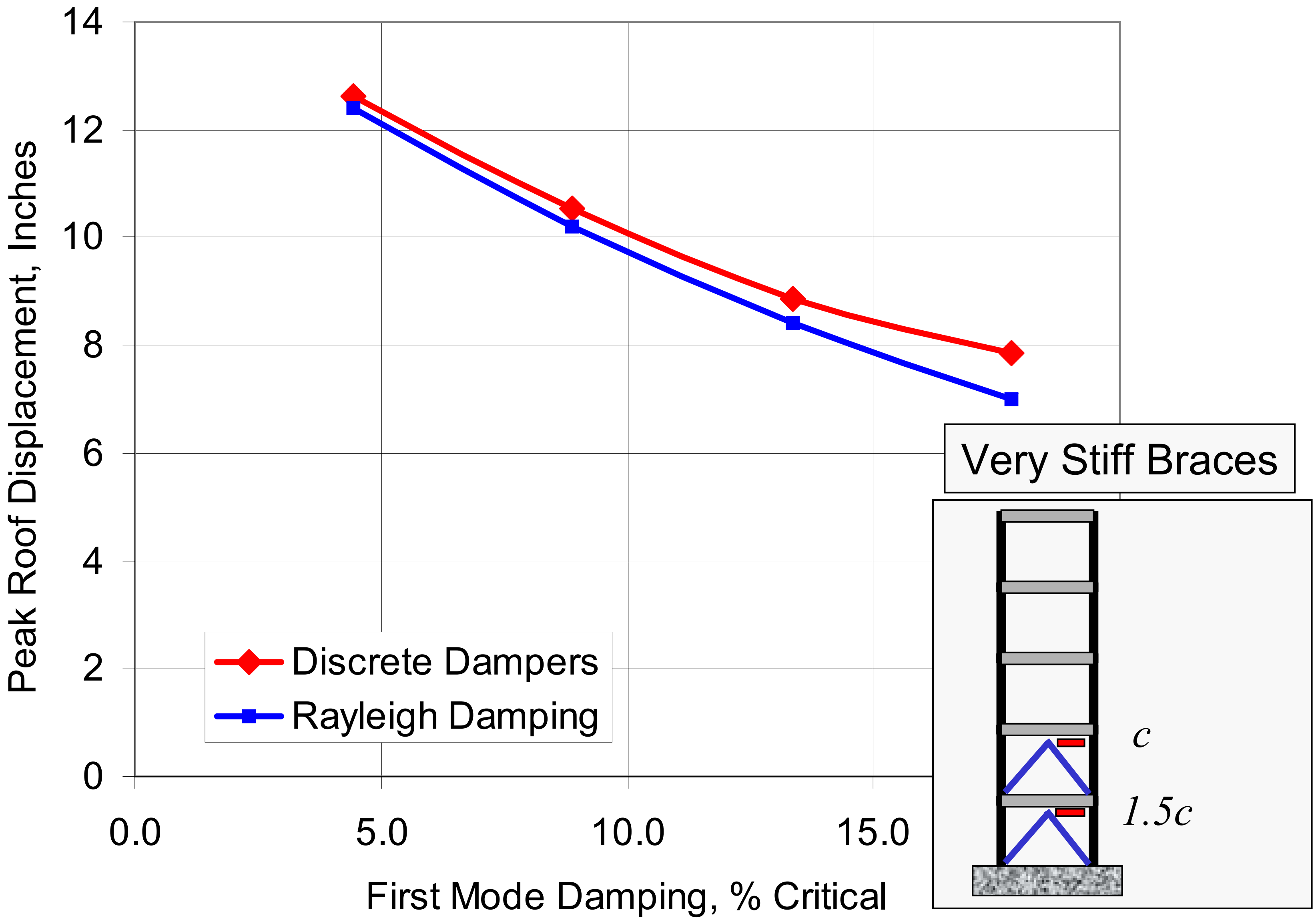
Frequency (rad/sec)	Damping Ratio, ξ
4.54	0.089
12.1	0.144
18.5	0.134
23.0	0.194
27.6	0.514

Damping ratios in modes 1 and 4 used to construct Rayleigh damping matrix.

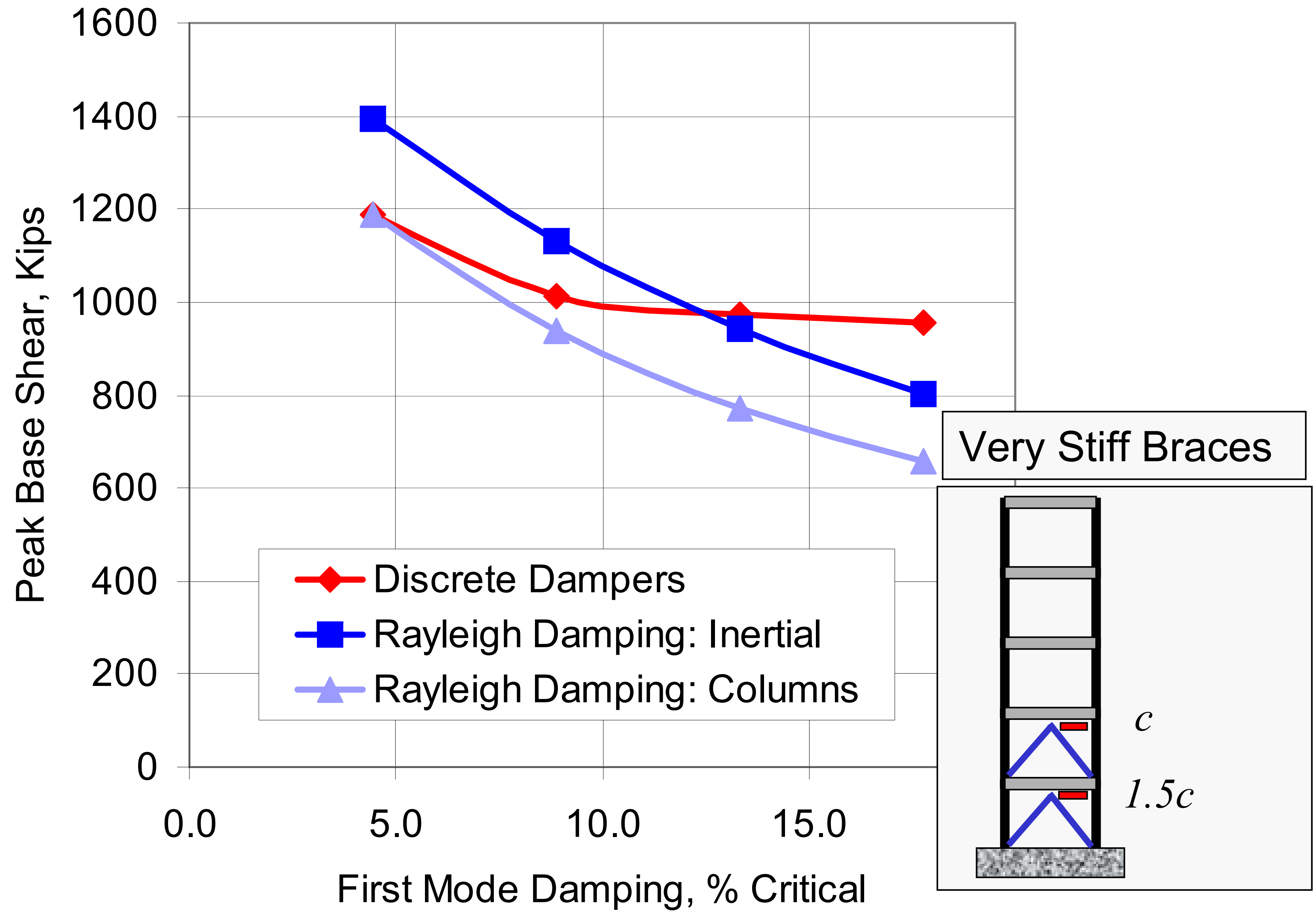
Computing Rayleigh Damping Proportionality Factors (Using NONLIN Pro)



Example: Discrete (Stiff Braces) vs Rayleigh Damping

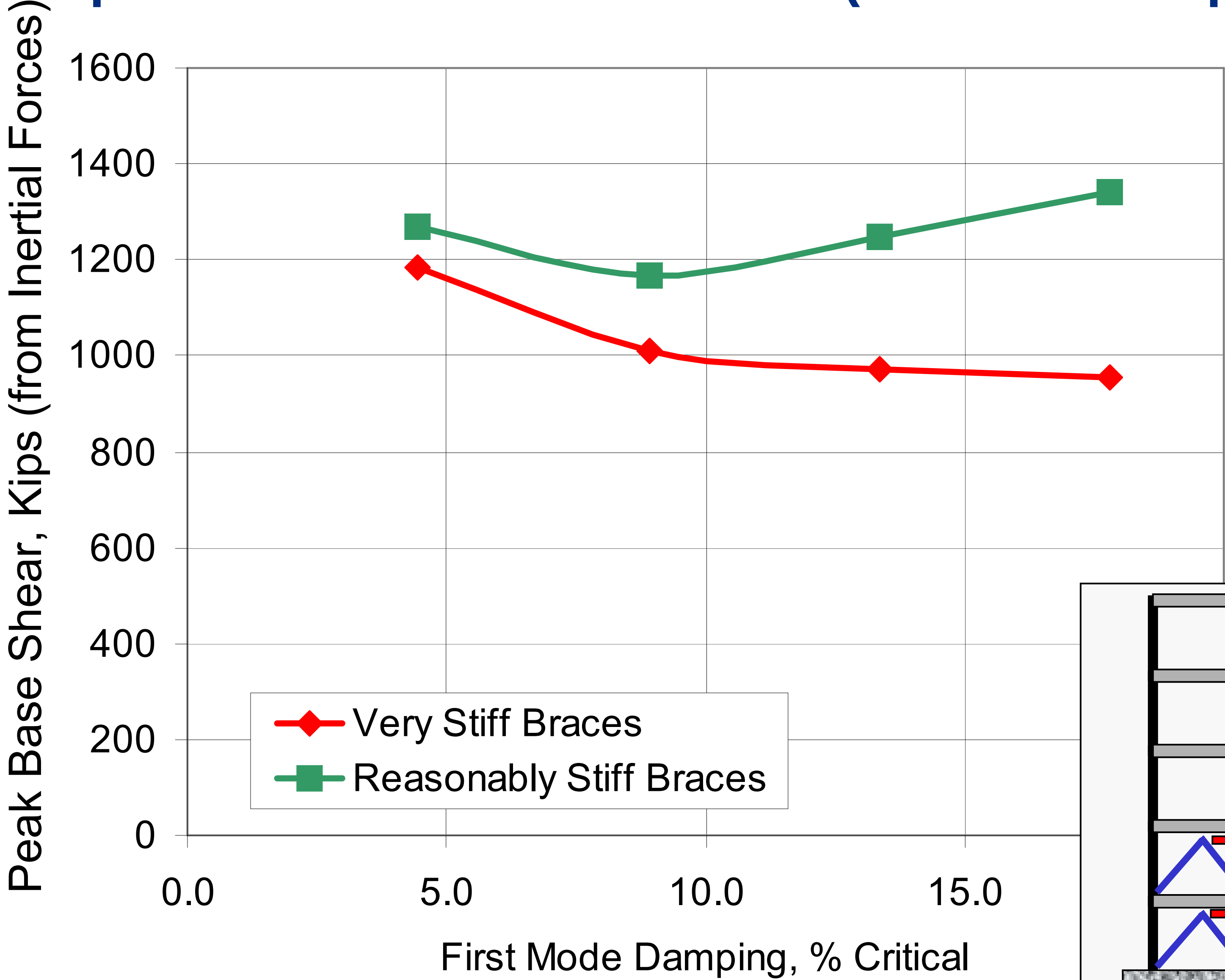


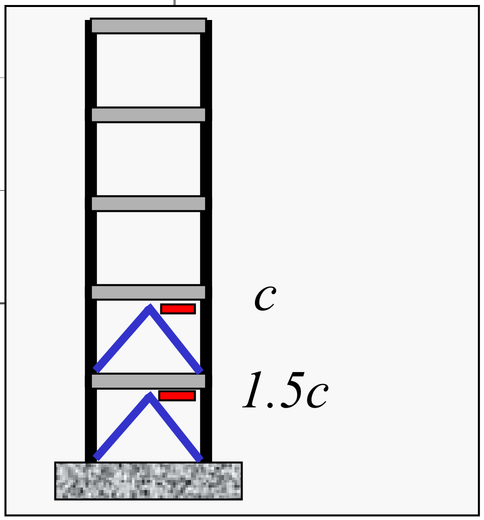
Example: Discrete (Stiff Braces) vs Rayleigh Damping



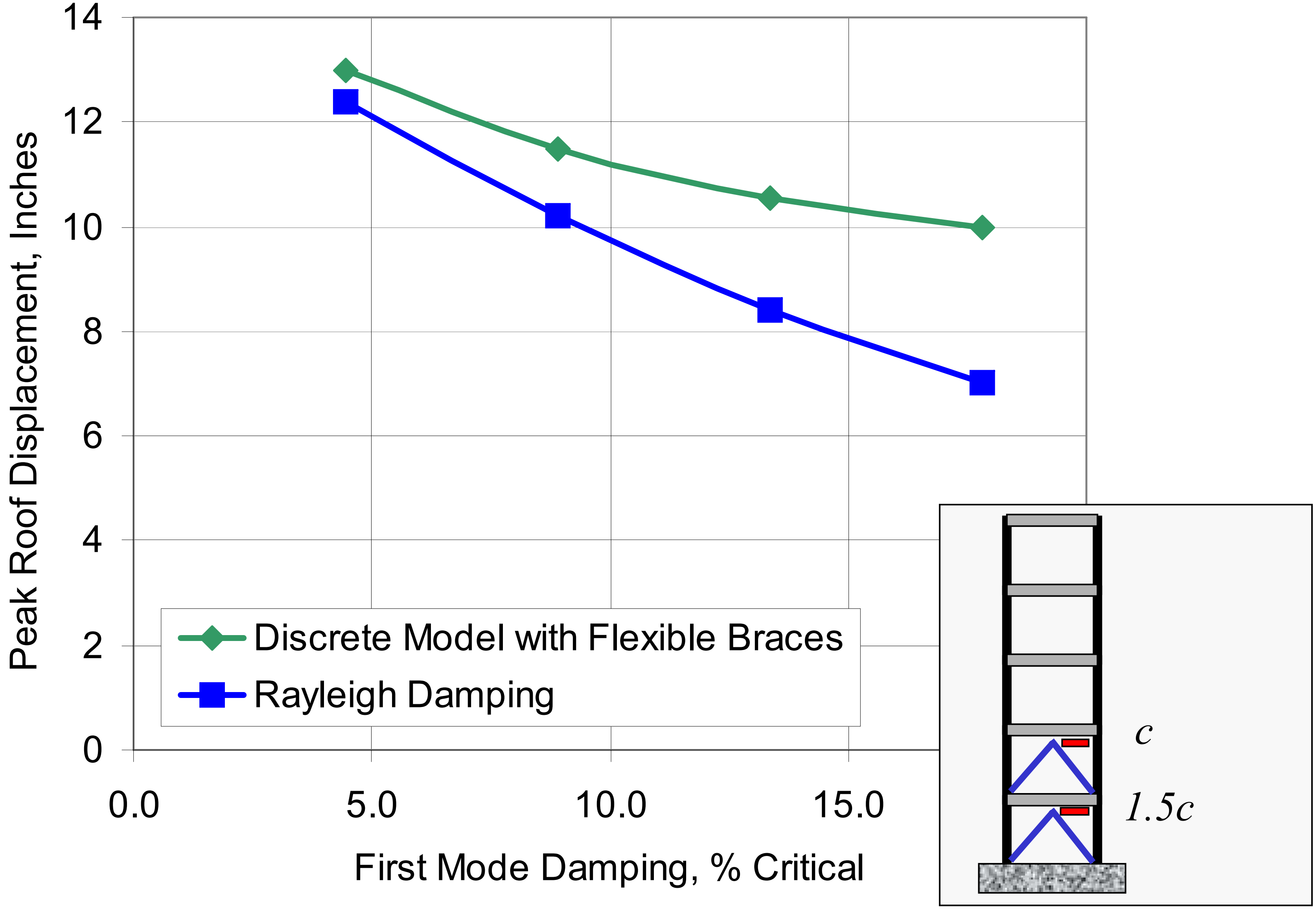
Example: Effect of Brace Stiffness (Discrete Damping Model)

Example: Effect of Brace Stiffness (Discrete Damping Model)

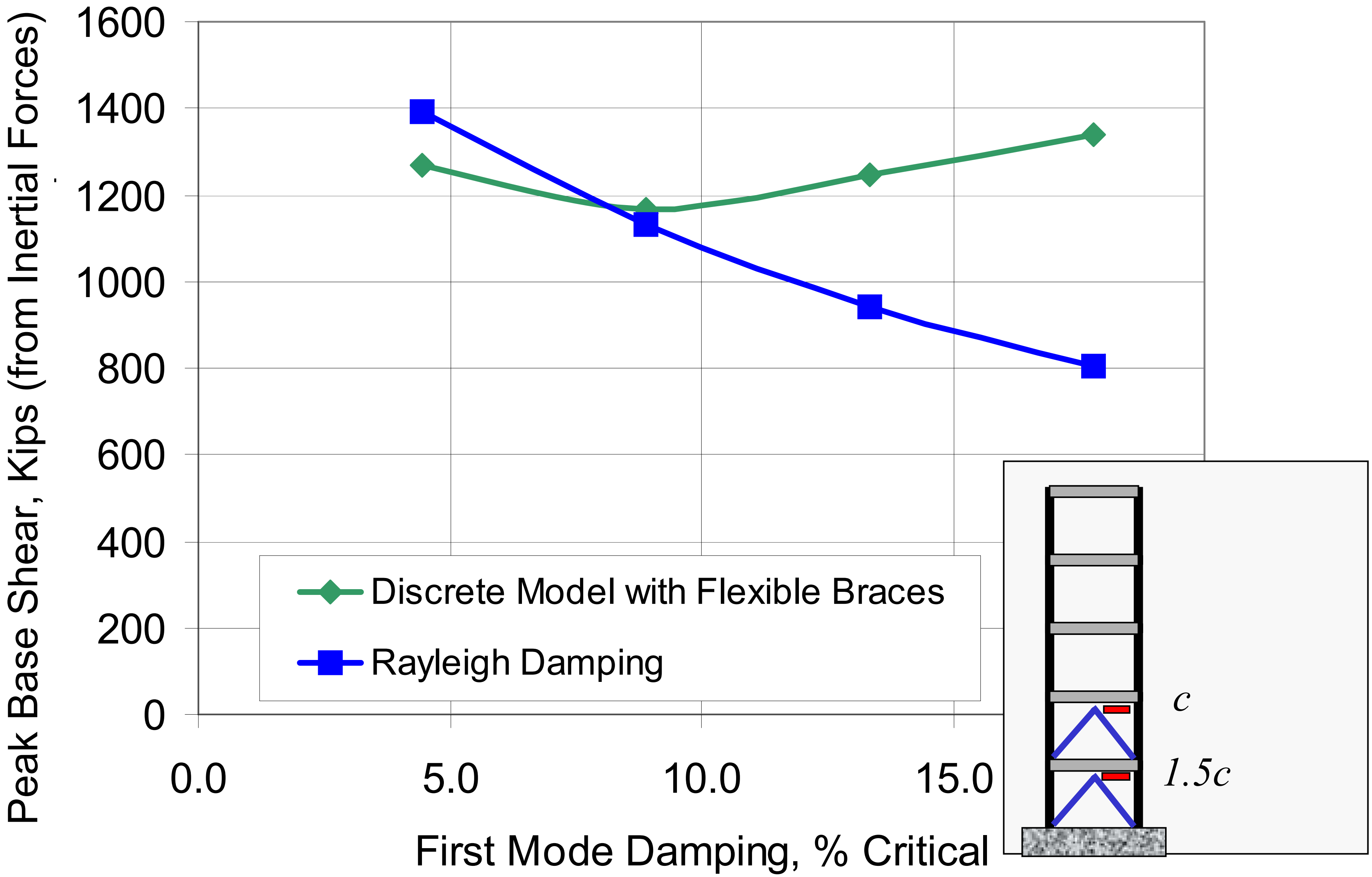




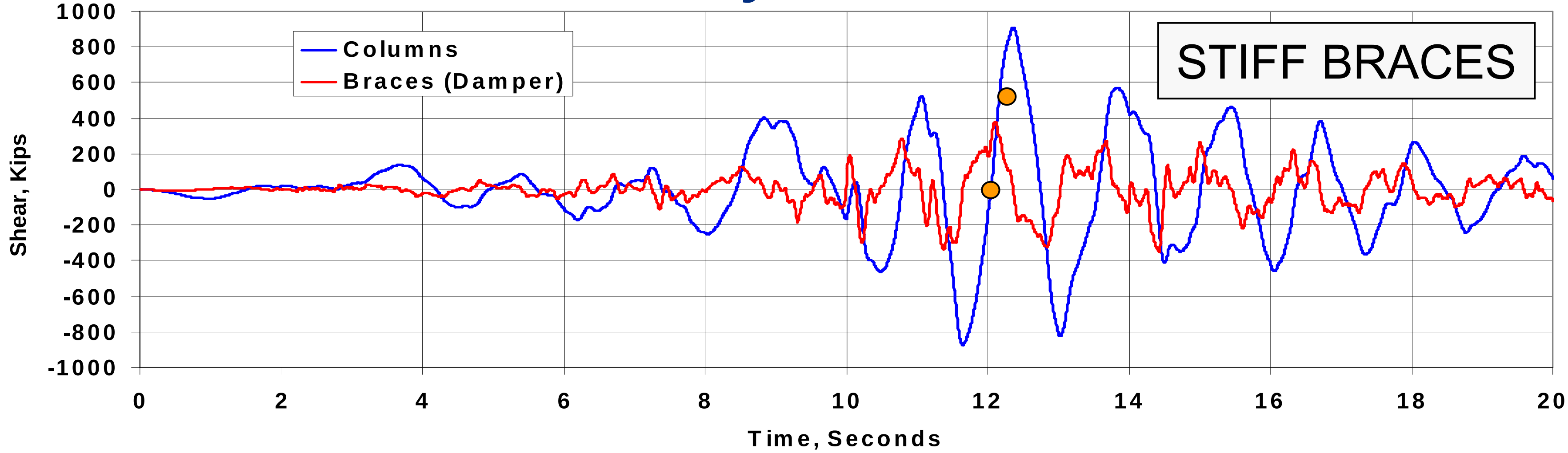
Example: Discrete (Flexible Braces) vs Rayleigh Damping

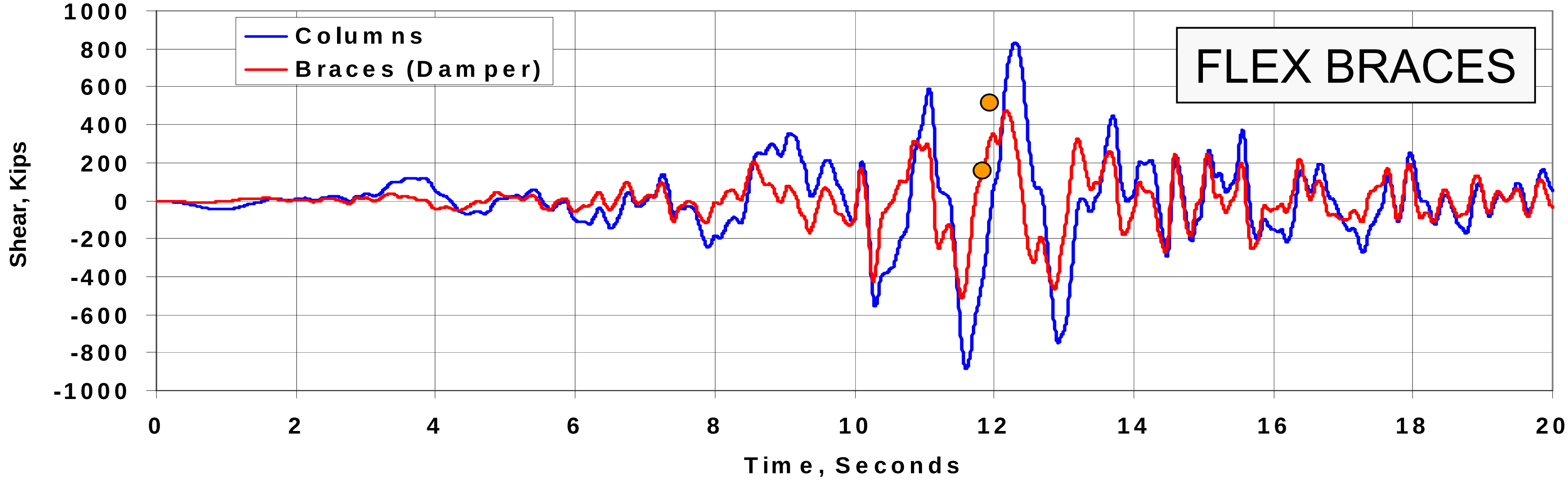


Example: Discrete (Flexible Braces) vs Rayleigh Damping



Example: Effect of Brace Stiffness on Peak Story Shear Forces





Outline: Part III

- Seismic Analysis of MDOF Structures with Passive Energy Dissipation Systems
- Representations of Damping
- Examples: Application of Modal Strain Energy Method and Non-Classical Damping Analysis
- Summary of MDOF Analysis Procedures

Summary: MDOF Analysis Procedures (Linear Systems and Linear Dampers)

- Use discrete damper elements and explicitly include these dampers in the system damping matrix. Perform response history analysis of full system. **Preferred.**

- Use discrete damper elements to estimate modal damping ratios and use these damping ratios in modal response history or modal response spectrum analysis. **Dangerous.**

- Use discrete damper elements to estimate modal damping ratios and use these damping ratios in a response history analysis which incorporates Rayleigh proportional damping. **Dangerous.**

Summary: MDOF Analysis Procedures (Linear Systems with Nonlinear Dampers)

- Use discrete damper elements and explicitly include these dampers in the system damping matrix. Perform response history analysis of full system. **Preferred.**

- Replace nonlinear dampers with "equivalent energy" based linear dampers, and then use these equivalent dampers in the system damping matrix. Perform response history analysis of full system. **Very Dangerous.**

- Replace nonlinear dampers with "equivalent energy" based linear dampers, use modal strain energy approach to estimate modal damping ratios, and then perform response spectrum or modal response history analysis. **Very Dangerous.**

Summary: MDOF Analysis Procedures (Nonlinear Systems with Nonlinear Dampers)

- Use discrete damper elements and explicitly include these dampers in the system damping matrix. Explicitly model inelastic behavior in superstructure. Perform response history analysis of full system. **Preferred.**

- Replace nonlinear dampers with "equivalent energy" based linear dampers and use modal strain energy approach to estimate modal damping ratios. Use pushover analysis to represent inelastic behavior in superstructure. Use capacity-demand spectrum approach to estimate system deformations. **Do This at Your Own Risk!**

Outline: Part IV

- MDOF Solution Using Complex Modal Analysis
- Example: Damped Mode Shapes and Frequencies
- An Unexpected Effect of Passive Damping
- Modeling Dampers in Computer Software
- Guidelines and Code-Related Documents for Passive Energy Dissipation Systems

MDOF Solution for Non-Classically Damped Structures Using Complex Modal Analysis

$$M\ddot{v}(t)+C_I\dot{v}(t)+C_A\dot{v}(t)+F_S(t)=-MR\ddot{v}_g(t)$$

Modal Analysis using Damped Mode Shapes:

- Treat C_I as modal damping and model C_A explicitly.
- Solve by modal superposition using damped (complex) mode shapes and frequencies.

System (dampers and structure) must be linear.

Damped Eigenproblem

$$M\ddot{v}(t)+C_A\dot{v}(t)+Kv(t)=0$$

EOM for Damped Free Vibration

Assume C_I is negligible

Linear Structure

State Vector: $Z=\begin{Bmatrix}\dot{v}\\ v\end{Bmatrix}$

State-Space Transformation: $\dot{Z}=HZ$

State Matrix: $H=\begin{bmatrix}-M^{-1}C_A & -M^{-1}K\\ I & 0\end{bmatrix}$

Solution of Damped Eigenproblem

Assume Harmonic Response for n-th mode: $Z_n = P_n e^{\lambda_n t}$

Substitute Response into State Space Equation: $P_n \lambda_n = H P_n \leftarrow$ Damped Eigenproblem for n-th Mode

$P\Lambda = HP \leftarrow$ Damped Eigenproblem for All Modes

Eigenvalue Matrix:
(* = complex conjugate)

$$\Lambda = \begin{bmatrix} \lambda & \\ & \lambda^* \end{bmatrix} \qquad \lambda = diag\left[\lambda_n\right]$$

Eigenvector Matrix:

$$P = \begin{bmatrix} \Phi\lambda & \Phi^*\lambda^* \\ \Phi & \Phi^* \end{bmatrix}$$

Extracting Modal Damping and Frequency from Complex Eigenvalues

Complex Eigenvalue for Mode n: $$\lambda_n = -\xi_n \omega_n \pm i\omega_n \sqrt{1-\xi_n^2}$$

Analogous to Roots of Characteristic Equation for SDOF Damped Free Vibration Problem

Modal Frequency: $$\omega_n = \left|\lambda_n\right|$$

Modal Damping Ratio: $$\xi_n = -\frac{\Re\left(\lambda_n\right)}{\left|\lambda_n\right|}$$

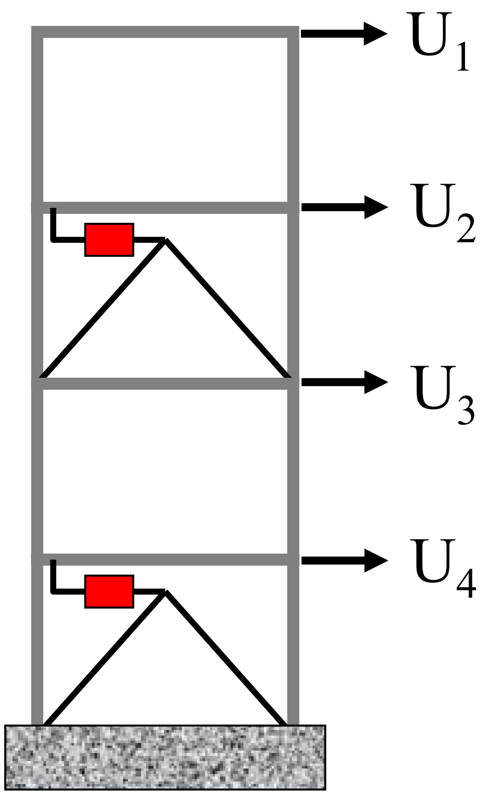
Note: $i = \sqrt{-1}$

Extracting Damped Mode Shapes

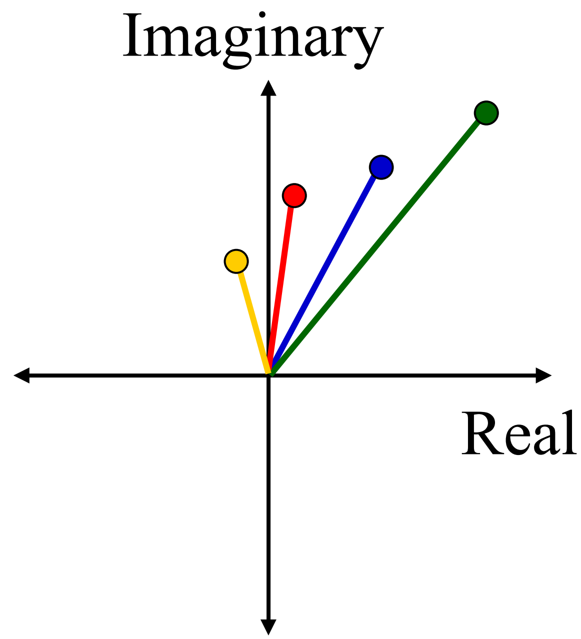
$$P = \begin{bmatrix} \Phi\Lambda & \Phi^{*}\Lambda^{*} \\ \boxed{\Phi} & \Phi^{*} \end{bmatrix}$$

Damped Mode Shapes

Damped Mode Shapes



$$\phi = \begin{Bmatrix} \boxed{a_1 + ib_1} \\ \boxed{a_2 + ib_2} \\ \boxed{a_3 + ib_3} \\ \boxed{a_4 + ib_4} \end{Bmatrix}$$



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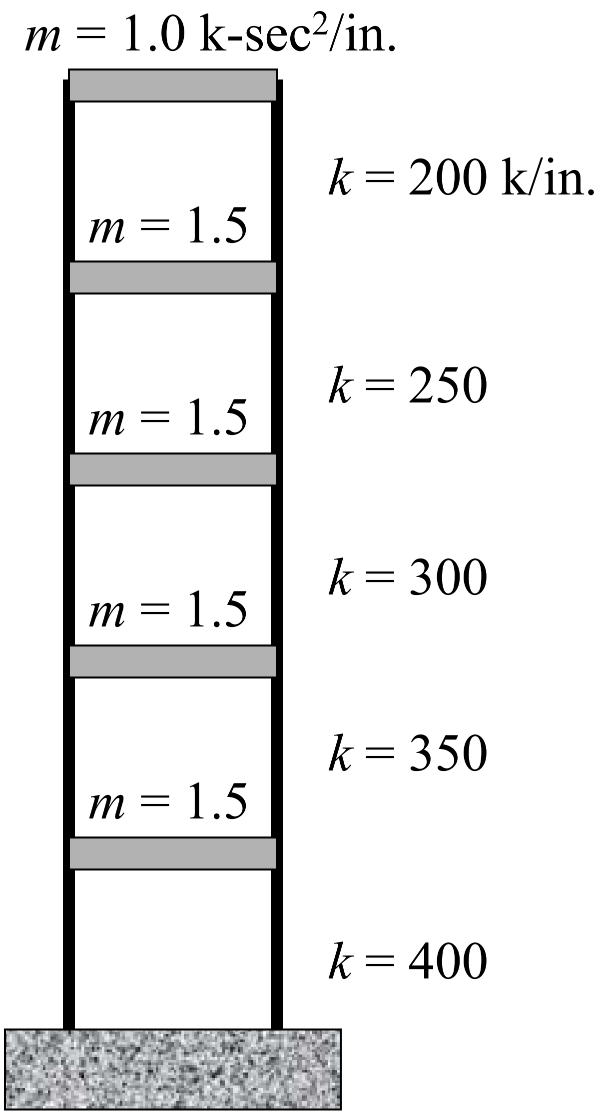
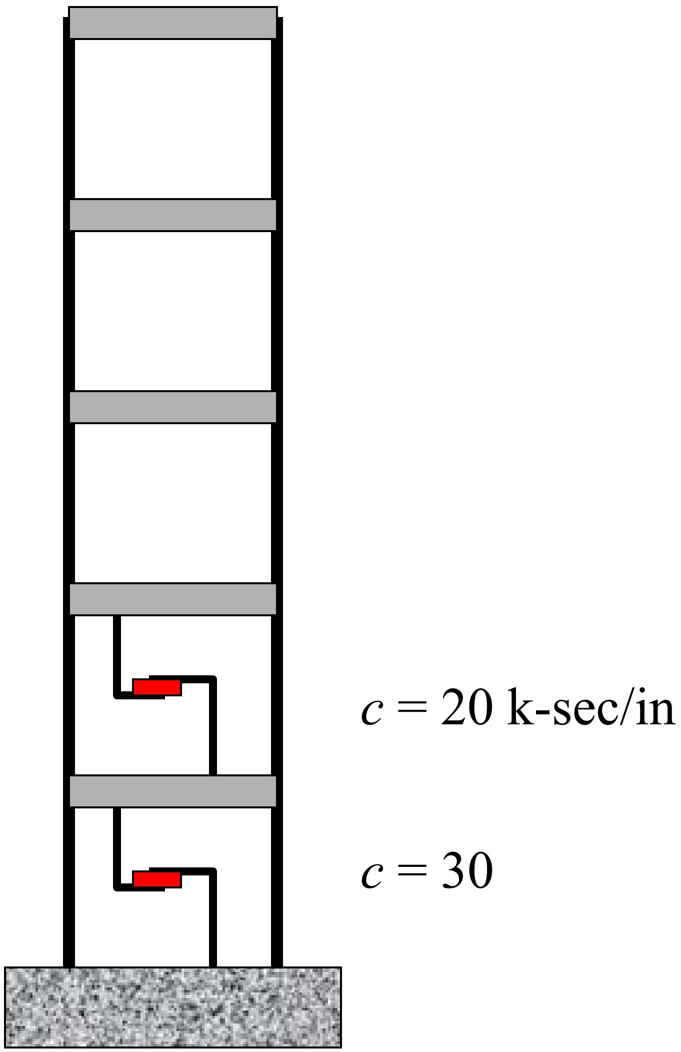
Example: Damped Mode Shapes and Frequencies



Nonproportional Damping



Example: Damped Mode Shapes and Frequencies
System with Non-Classical Damping

Using UNDAMPED MODE SHAPES

	Frequency (rad/sec)	Damping Ratio
1	4.54	0.089
2	12.1	0.144
3	18.4	0.134
4	23.0	0.194
5	27.6	0.516

Obtained from MSE Method

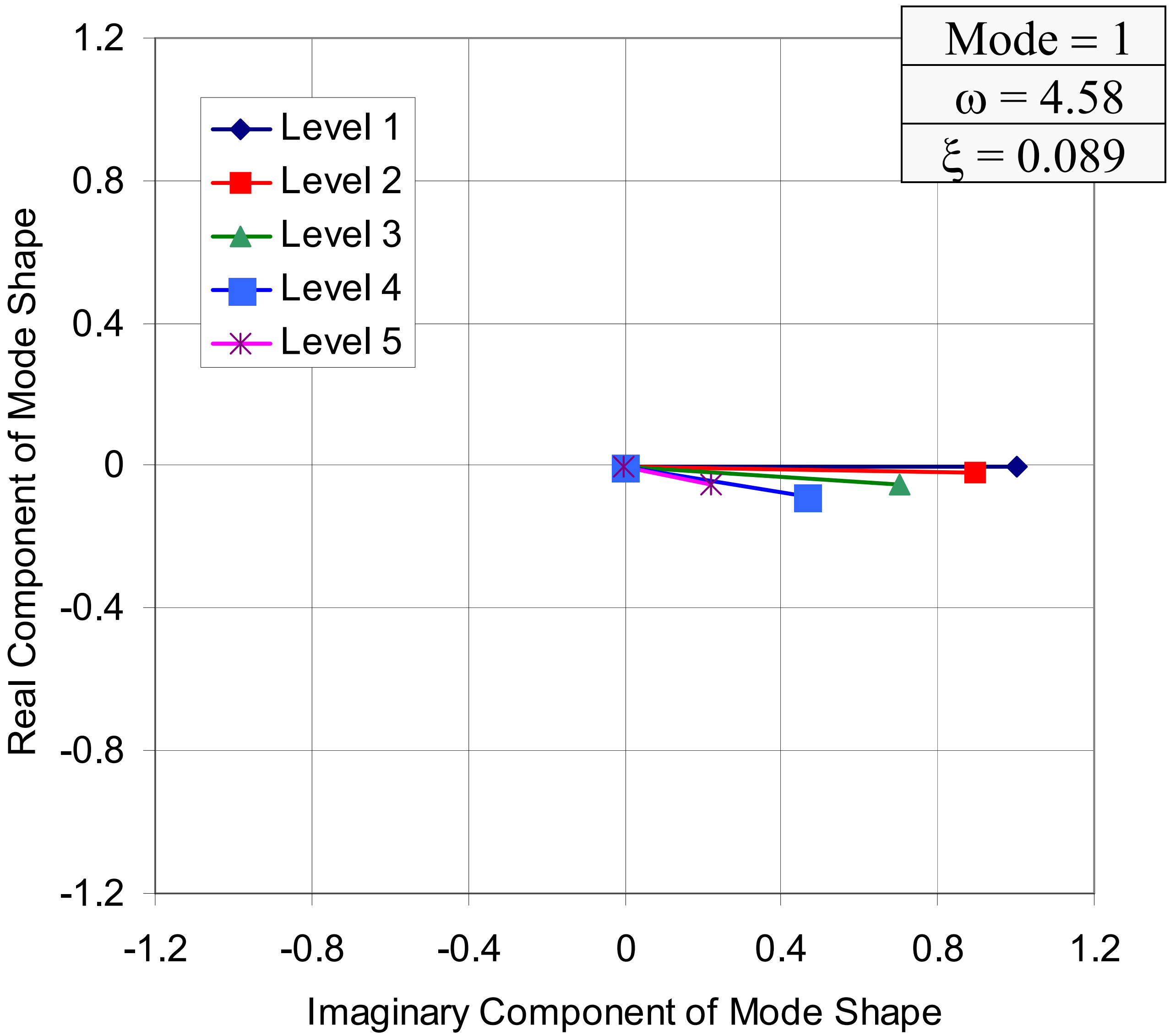
Using DAMPED MODE SHAPES*

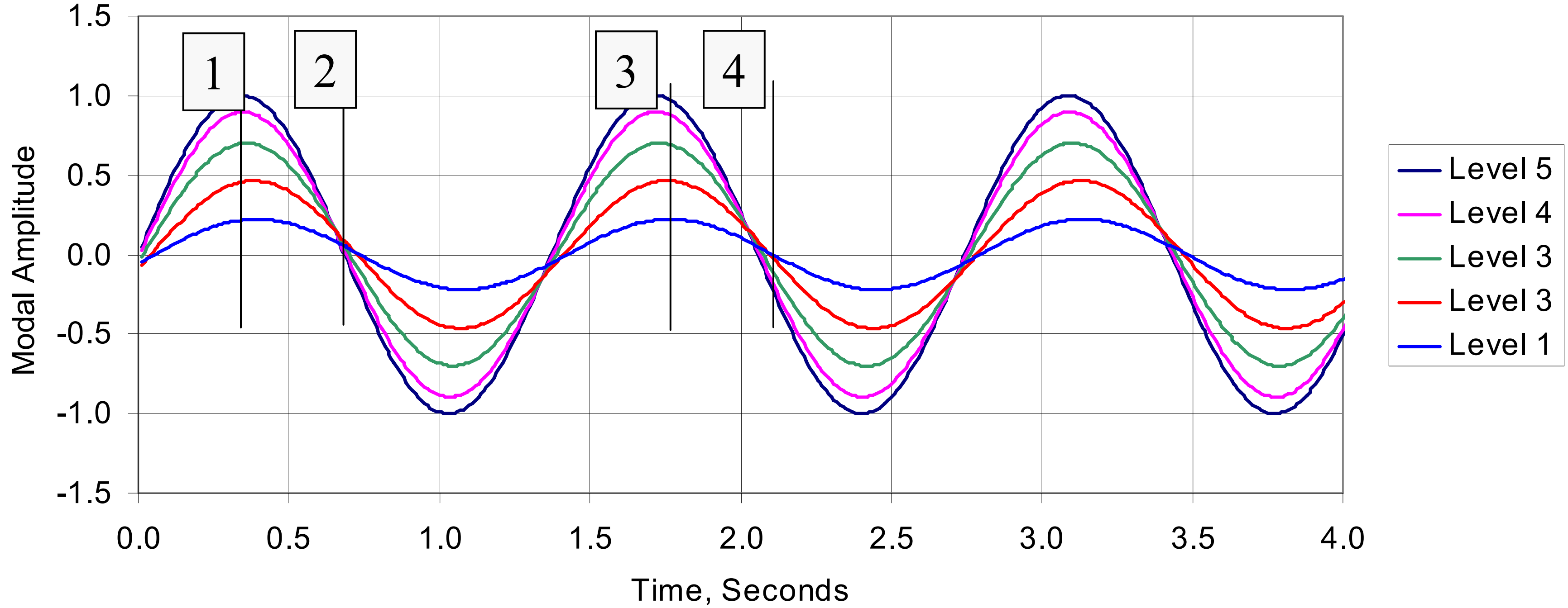
	Frequency (rad/sec)	Damping Ratio
1	4.58	0.089
2	12.3	0.141
3	18.9	**0.064**
4	24.0	**0.027**
5	25.1	**0.770**

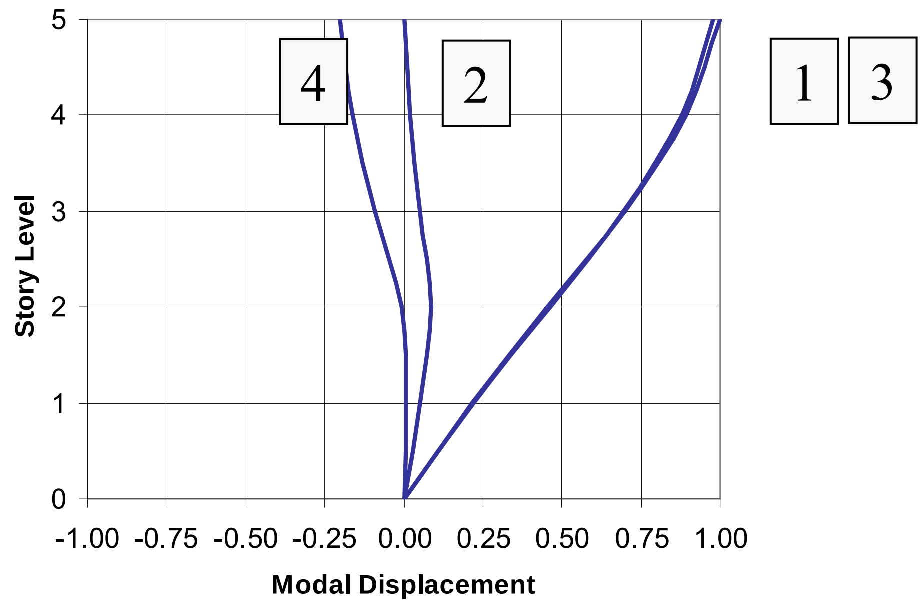
***Table is for model with VERY STIFF braces.**

Significant Differences in Higher Mode Damping Ratios

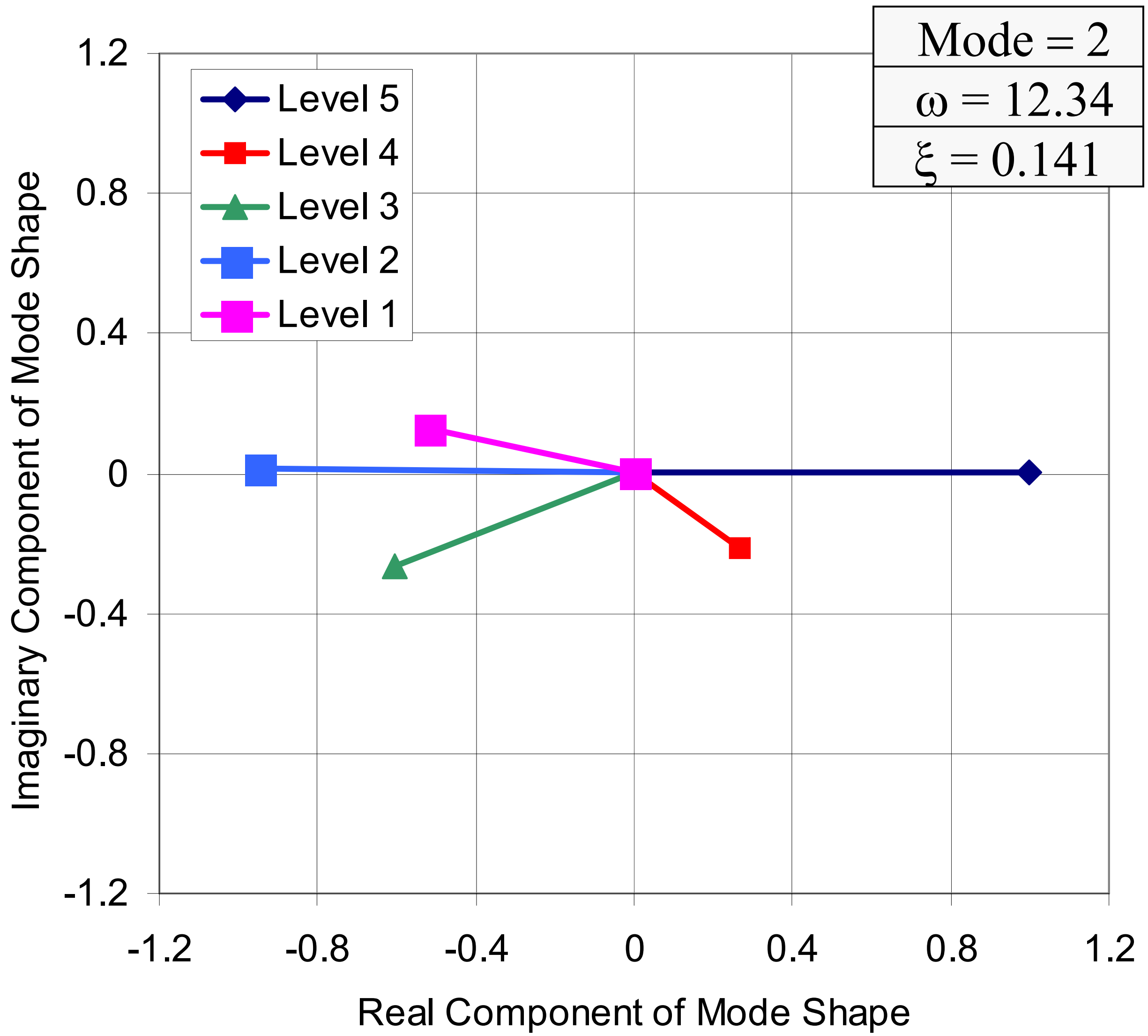
Example: Damped Mode Shapes and Frequencies
System with Non-Classical Damping




1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
1
2
3
4
Modal Amplitude
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
Time, Seconds
Level 5
Level 4
Level 3
Level 3
Level 1


5
4
3
2
1
0
Story Level
4
2
1
3
-1.00
-0.75
-0.50
-0.25
0.00
0.25
0.50
0.75
1.00
Modal Displacement

Example: Damped Mode Shapes and Frequencies
System with Non-Classical Damping



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An Unexpected Effect of Passive Damping

The larger the damping coefficient ***C***, the *smaller* the damping ratio ξ.

Why?

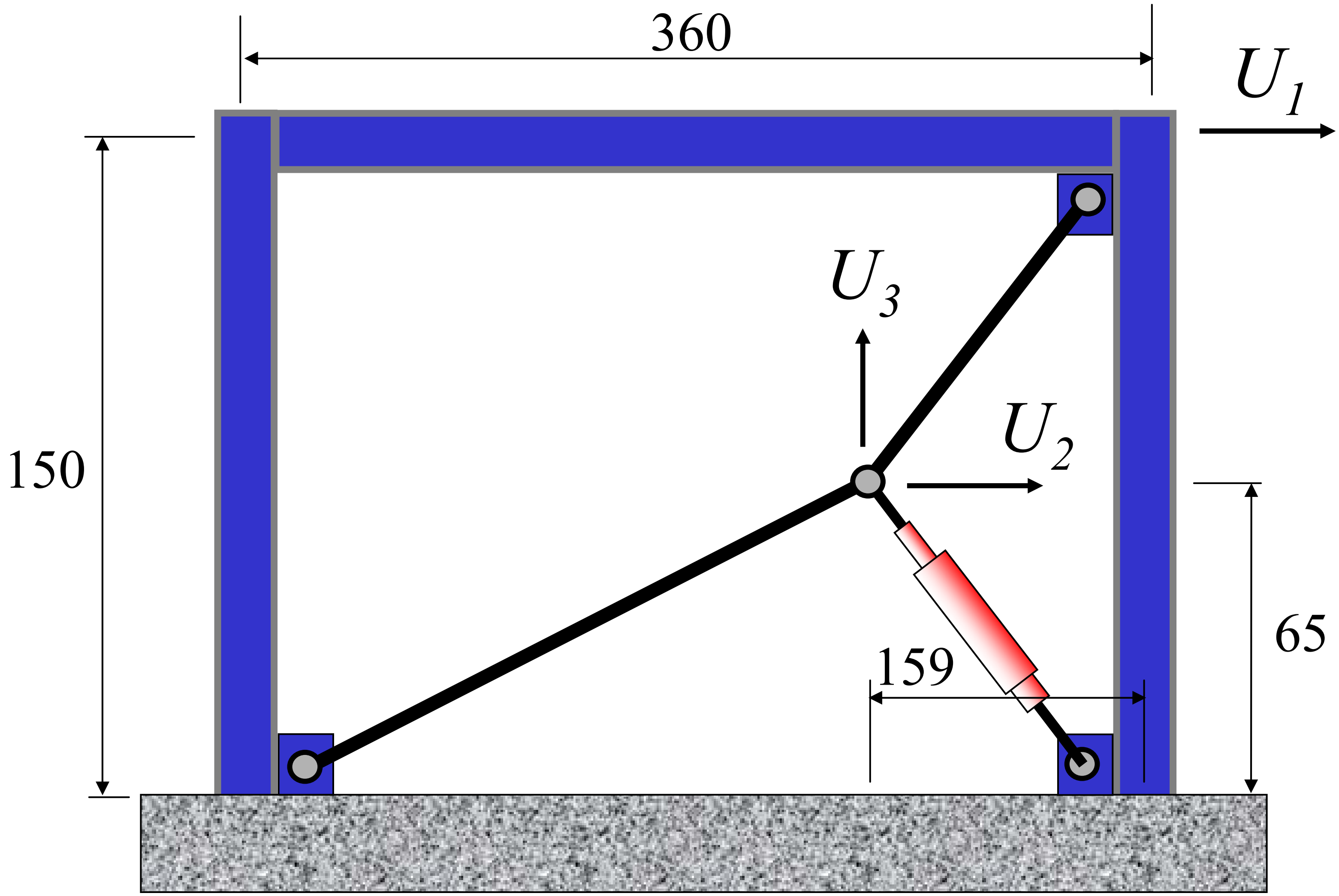
Note:
Occurs for toggle-braced systems only.

Huntington Tower
- **111 Huntington Ave, Boston, MA**
- **New 38-story steel-framed building**
- **100 Direct-acting and toggle-brace dampers**
- **1300 kN (292 kips), +/- 101 mm (+/- 4 in.)**
- **Dampers suppress wind vibration**

Toggle Brace Deployment

Huntington Tower

Example: Toggle Brace Damping System



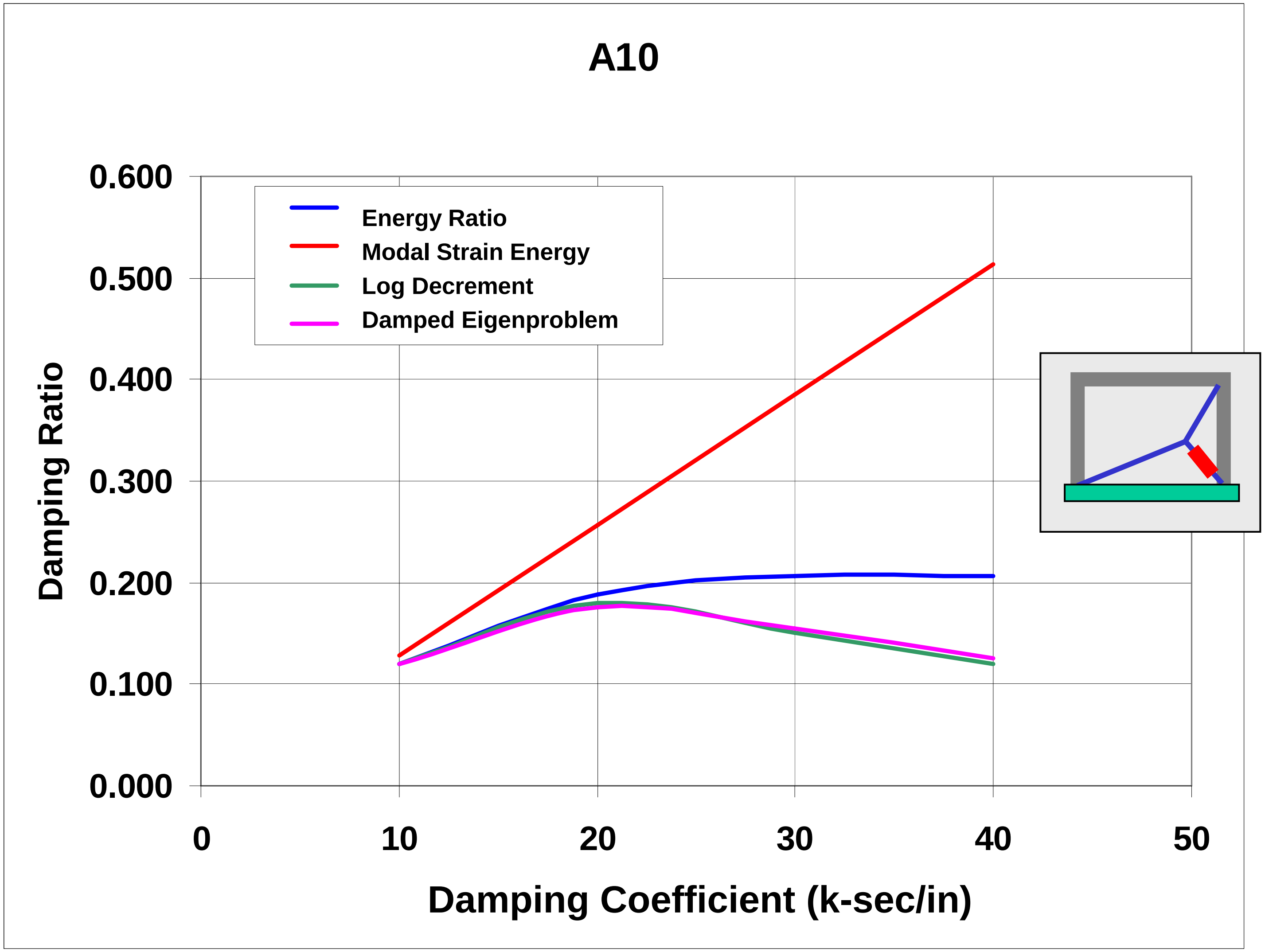
Units: inches

Methods of Analysis Used to Determine Damping Ratio

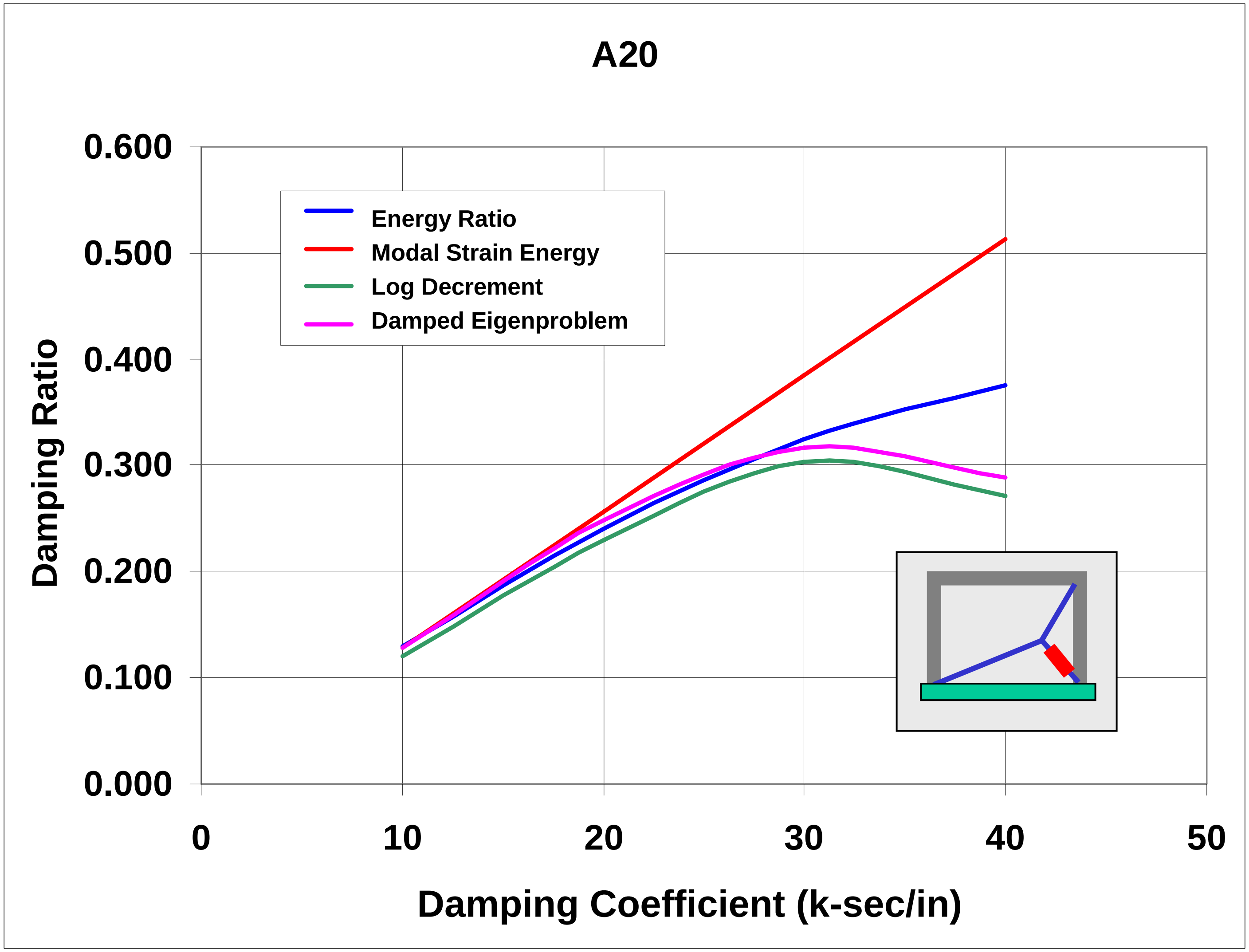
- Energy Ratios for Steady-State Harmonic Loading: $\xi = E_D / 4\pi E_S$
- Modal Strain Energy
- Free Vibration Log Decrement
- Damped Eigenproblem

$\boldsymbol{C}$ =10 to 40 k-sec/in (increments of 10)
$\boldsymbol{A}$ =10 to 100 in^2 (increments of 10)

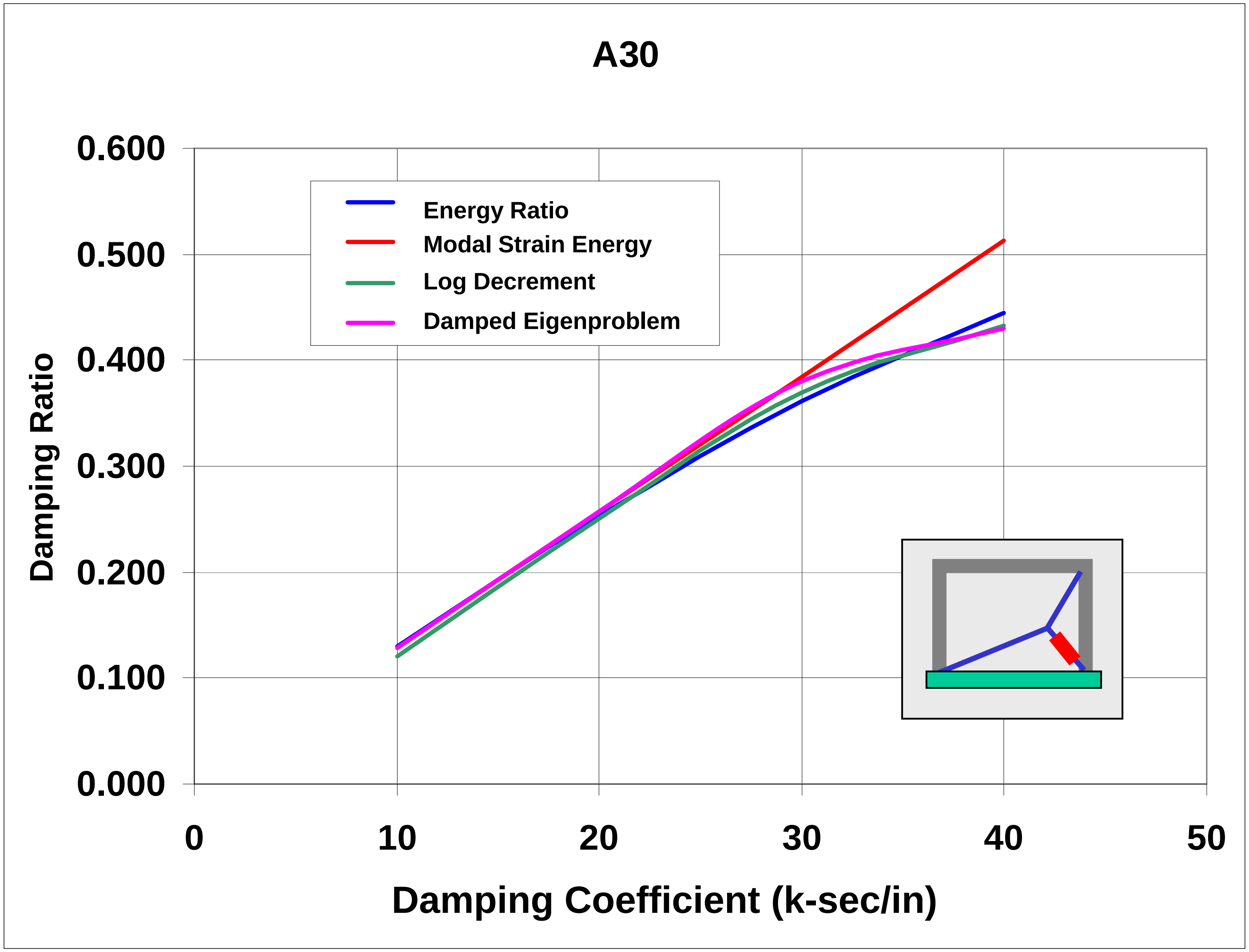
Computed Damping Ratios for System With A = 10



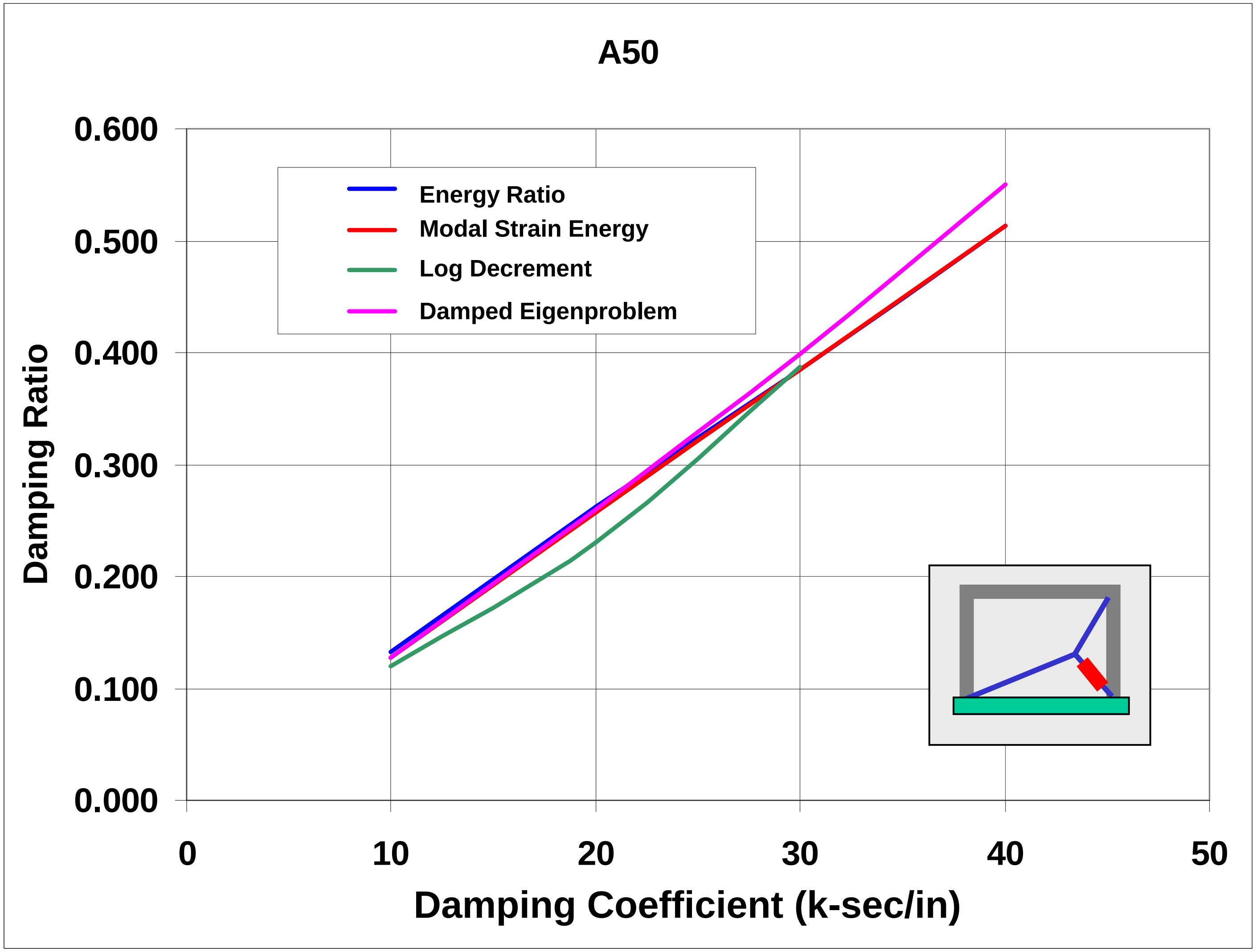
Computed Damping Ratios for System With A = 20



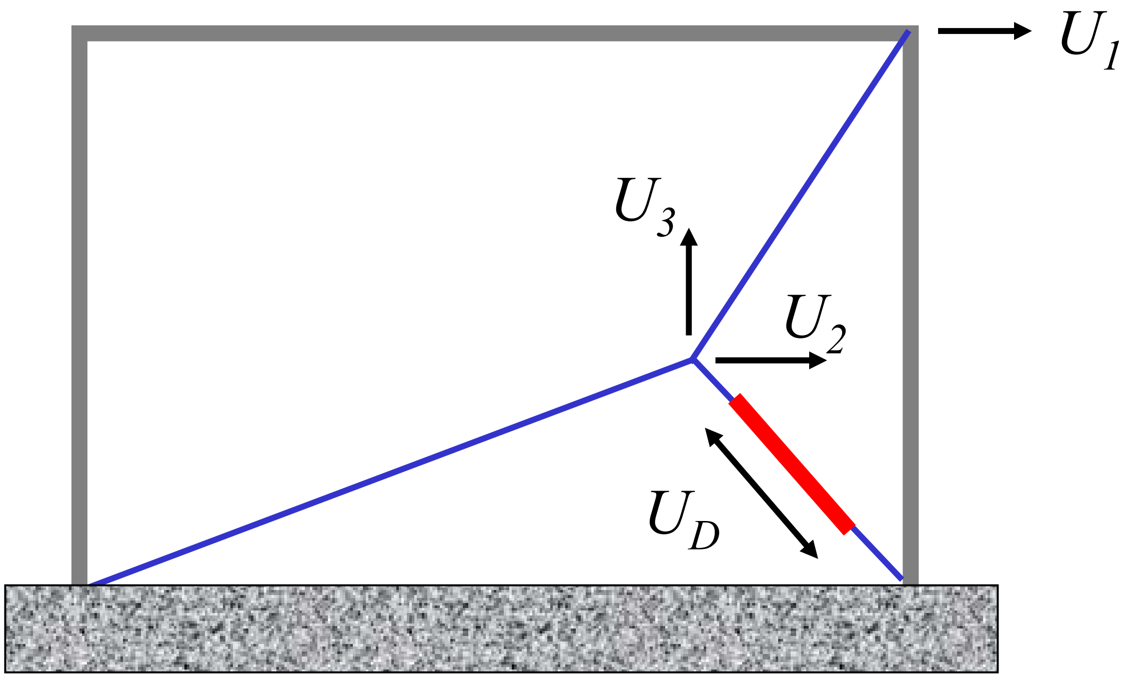
Computed Damping Ratios for System With A = 30



Computed Damping Ratios for System With A = 50

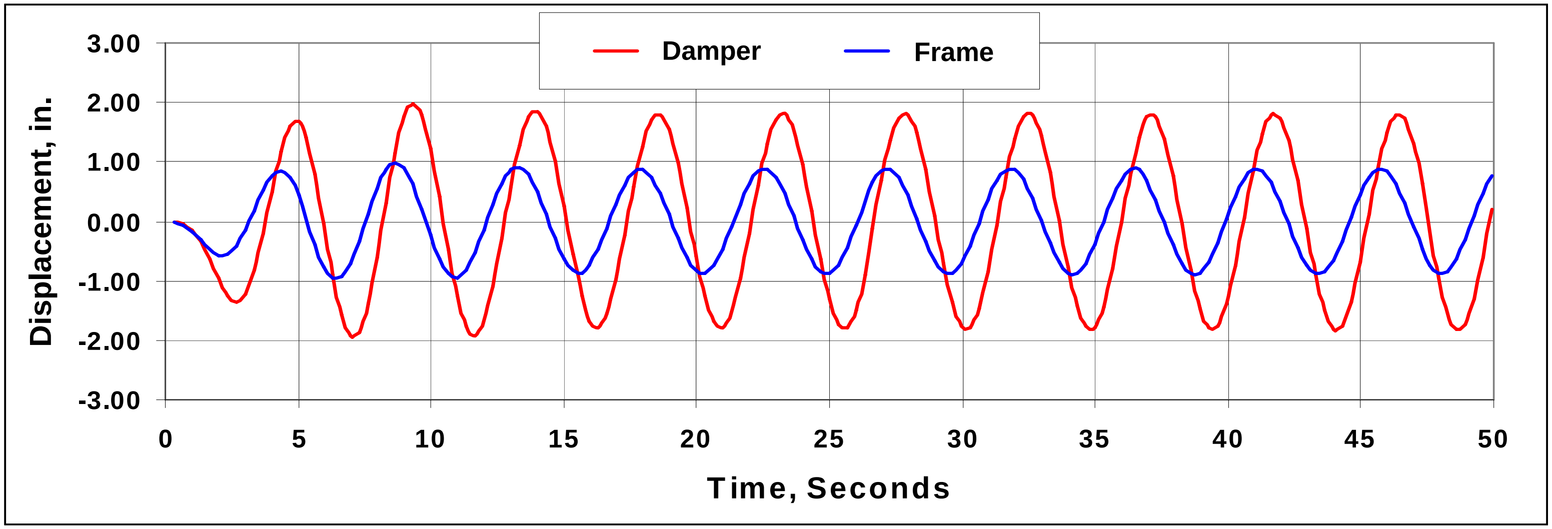


Why Does Damping Ratio Reduce for Low Brace Area/Damping Coefficient Ratios?



Displacement in Damper is *Out-of-Phase* with Displacement at DOF 1

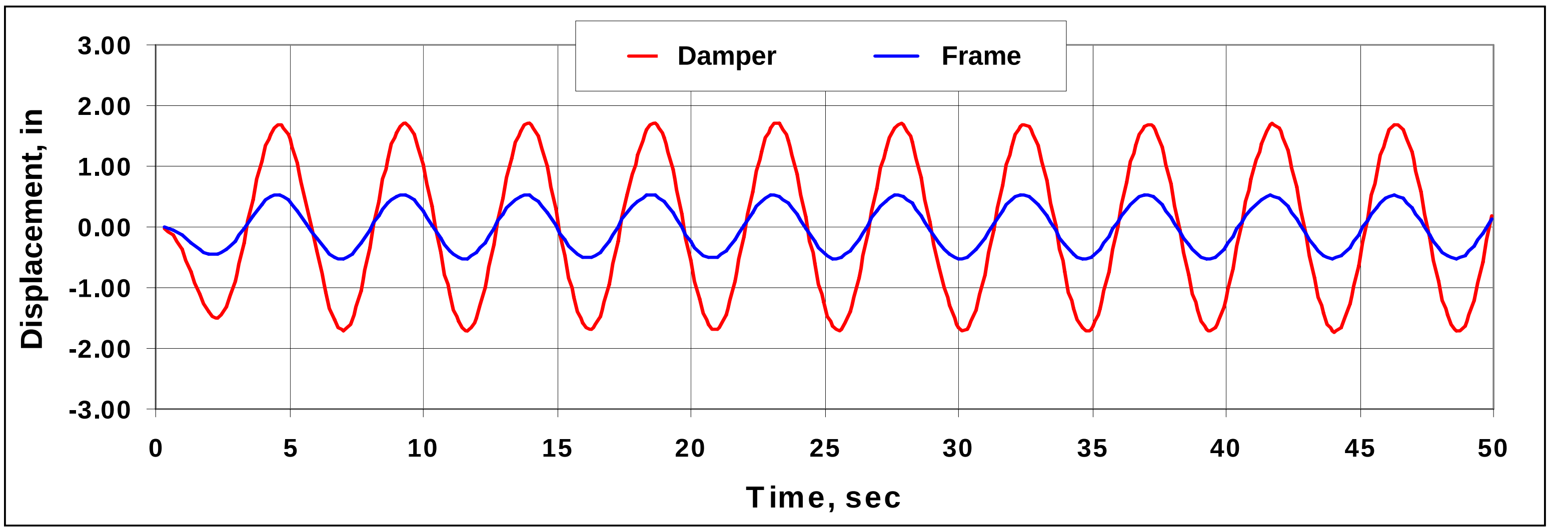
Phase Difference Between Damper Displacement and Frame Displacement



A=10

C=40

A/C=0.25

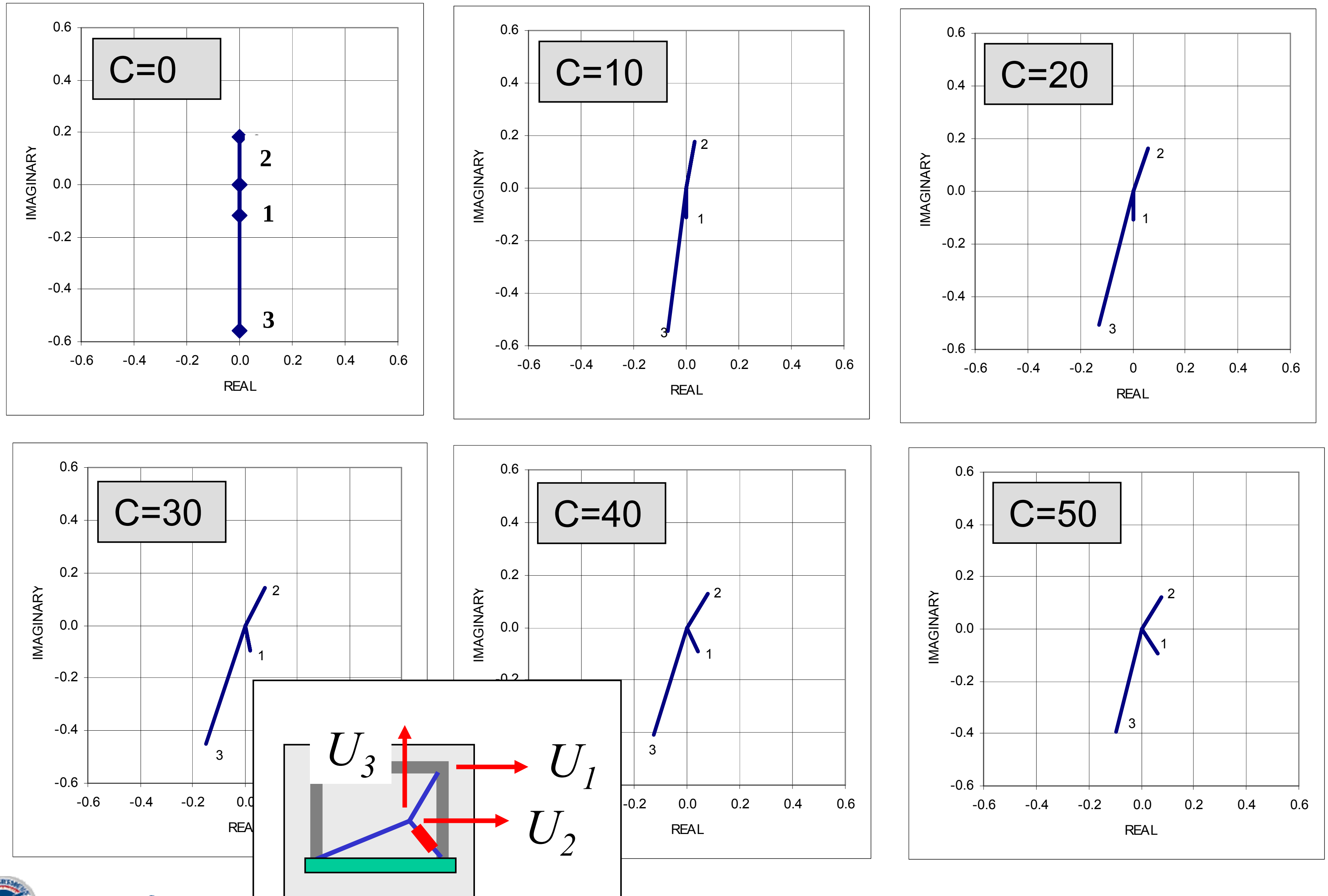


A=50

C=40

A/C=1.25

Damped Mode Shapes for System With A=20 in^2



Interim Summary Related to Modeling and Analysis (1)

- Viscously damped systems are very effective in reducing damaging deformations in structures.
- With minor exceptions, viscously damped systems are non-classical, and *must* be modeled explicitly using dynamic time history analysis.
- Avoid the use of the Modal Strain Energy method (it may provide unconservative results)

Interim Summary Related to Modeling and Analysis (2)

- Damped mode shapes provide phase angle information that is essential in assessing and improving the efficiency of viscously damped systems. This is particularly true for linkage systems (e.g. toggle-braced systems).

- If damped eigenproblem analysis procedures are not available, use overlayed response history plots of damper displacement and interstory displacement to assess damper efficiency. (This would be required for nonlinear viscously damped systems.)

Outline: Part IV

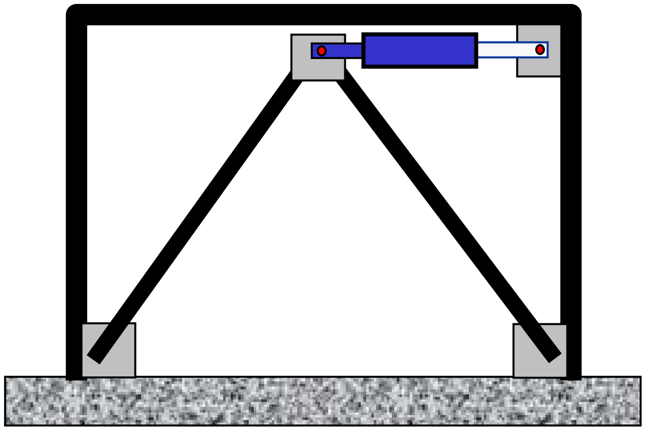
- MDOF Solution Using Complex Modal Analysis
- Example: Damped Mode Shapes and Frequencies
- An Unexpected Effect of Passive Damping
- Modeling Dampers in Computer Software
- Guidelines and Code-Related Documents for Passive Energy Dissipation Systems

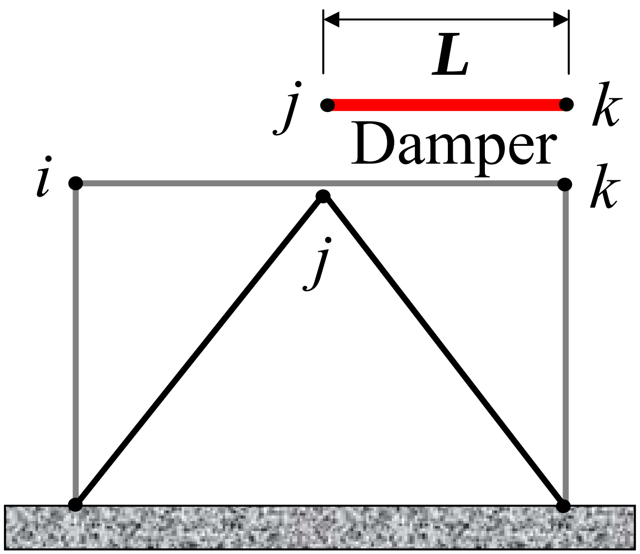
Computer Software Analysis Capabilities

	SAP2000; ETABS	DRAIN	RAM Perform
Linear Viscous Fluid Dampers	Yes	Yes	Yes
Nonlinear Viscous Fluid Dampers	Yes	NO	Yes*
Viscoelastic Dampers	Yes	Yes	Yes
ADAS Type Systems	Yes	Yes	Yes
Unbonded Brace Systems	Yes	Yes	Yes
Friction Systems	Yes	Yes	Yes
General System Yielding	Pending	Yes	Yes

***Piecewise Linear**

Modeling Linear Viscous Dampers in DRAIN





Use a Type-1 truss bar element with stiffness proportional damping:

$$K = \frac{AE}{L} \qquad C = \beta K$$

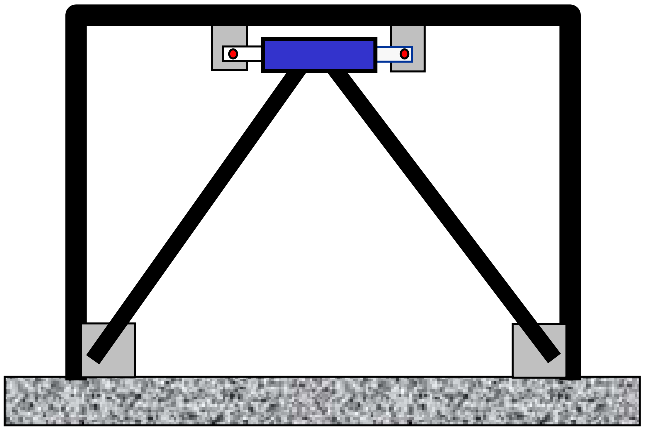
For dampers with low stiffness:
Set $A = L$, $E = 0.01$ and
$\beta = C_{Damper}/0.01$

Result:

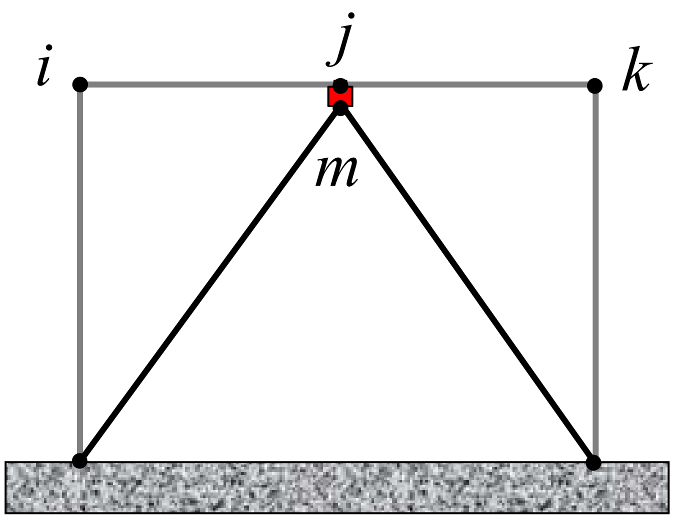
$$K = 0.01 \qquad C = C_{Damper}$$

$$F = C\dot{u} = \beta K\dot{u} = C_{Damper}\dot{u}$$

Modeling Linear Viscous Dampers in DRAIN

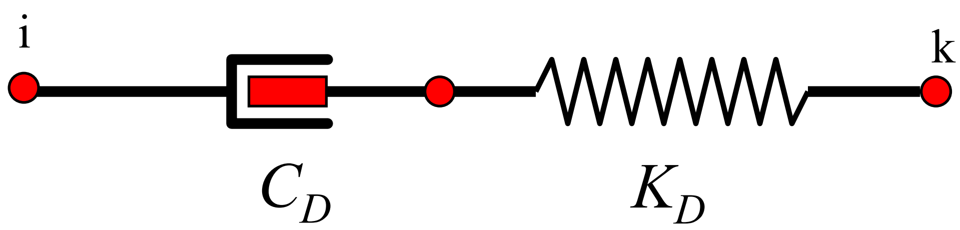


Dampers may be similarly modeled using the zero-length “Type-4” connection element.



Nodes *j* and *m* have the same coordinates

Modeling Viscous/Viscoelastic Dampers Using the SAP2000 NLLINK Element



The damper is modeled as a Maxwell Element consisting of a linear or nonlinear dashpot in series with a linear spring.

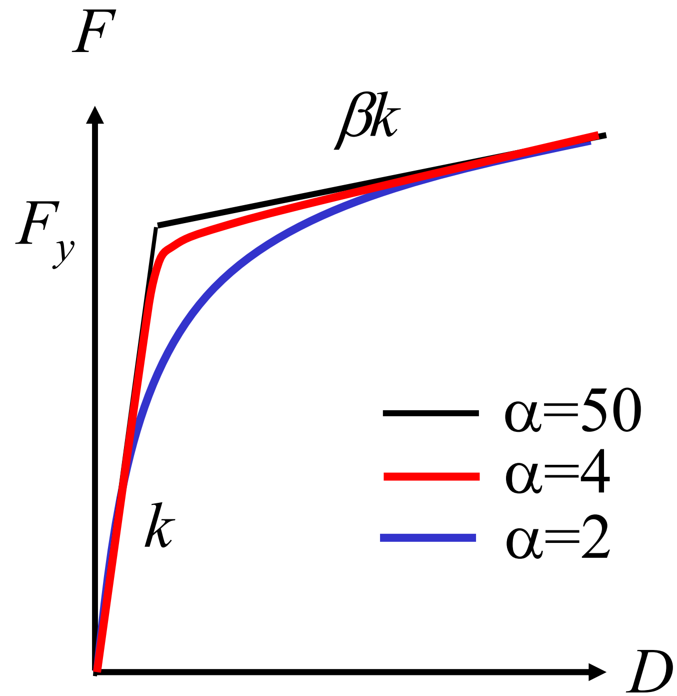
To model a linear viscous dashpot, K_D must be set to a large value, but not *too* large or convergence will not be achieved. To achieve this, it is recommended that the relaxation time
($\lambda = C_D/K_D$) be an order of magnitude less than the loading time step Δt.
For example, let $K_D = 100 C_D/\Delta t$. Sensitivity to K_D should be checked.

SAP2000 often has difficulty converging when nonlinear dampers are used and the velocity exponent is less than 0.4.

Modeling ADAS, Unbonded Brace, and Friction Dampers using the SAP2000 NLLINK Element

$$F = \beta k D + (1-\beta) F_y Z$$

$$\dot{Z} = \frac{k}{F_y}\begin{cases} \dot{D}\left(1-\left|Z\right|^{\alpha}\right) & if\ \dot{D}Z > 0 \\ \dot{D} & otherwise \end{cases}$$



Note: Z is an internal hysteretic variable with magnitude less than or equal to unity. The yield surface is associated with a magnitude of unity.

For bilinear behavior, use α of approximately 50. Larger values can produce strange results.

Outline: Part IV

- MDOF Solution Using Complex Modal Analysis
- Example: Damped Mode Shapes and Frequencies
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- Guidelines and Code-Related Documents for Passive Energy Dissipation Systems

1993 - Tentative General Requirements for the Design and Construction of Structures Incorporating Discrete Passive Energy Dissipation Devices (1 of 3)

- Draft version developed by Energy Dissipation Working Group (EDWG) of Base Isolation Subcommittee of Seismology Committee of SEAONC (Not reviewed/approved by SEAOC; used as basis for 1994 NEHRP Provisions)

- Philosophy: For Design Basis Earthquake (10/50), confine inelastic behavior to energy dissipation devices (EDD); gravity load resisting system to remain elastic

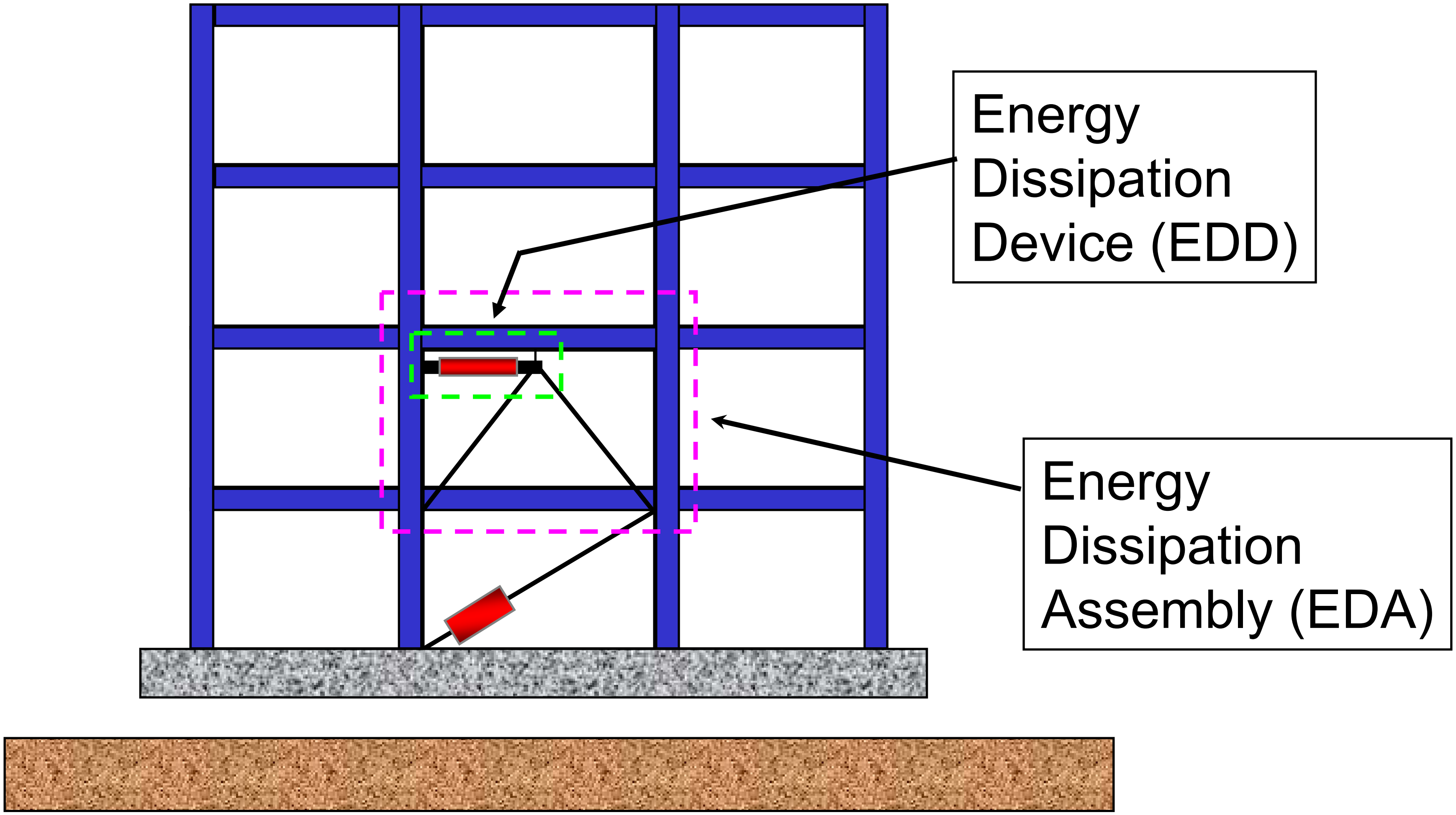
- Established terminology and nomenclature for energy dissipation systems (EDS)

- Classified systems as rate-independent or rate-dependent (included metallic, friction, viscoelastic, and viscous dampers)

- Required at least two vertical lines of dampers in each principal direction of building; dampers to be continuous from the base of the building

- Prescribed analysis and testing procedures

1993 - Tentative General Requirements for the Design and Construction of Structures Incorporating Discrete Passive Energy Dissipation Devices (2 of 3)



Energy Dissipation Nomenclature

1993 - Tentative General Requirements for the Design and Construction of Structures Incorporating Discrete Passive Energy Dissipation Devices (3 of 3)

- Elastic structures with rate-dependent devices: Linear dynamic procedures (response spectrum or response history analysis)

- Inelastic structures or structures with rate-independent devices: Nonlinear dynamic response history analysis

- Prototype tests on full-size specimens (not required if previous tests performed and documented by ICBO)

- General acceptability criteria for energy dissipation systems:
    - Remain stable at design displacements
    - Provide non-decreasing resistance with increasing displacement (for rate-independent systems)
    - Exhibit no degradation under repeated cyclic load at design displ.
    - Have quantifiable engineering parameters

- Independent engineering review panel required to oversee design and testing

1994 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (1 of 4)

Part 1 – Provisions & Part 2 – Commentary (FEMA 222A & 223A)

- Includes Appendix to Chapter 2 entitled: Passive Energy Dissipation Systems

- Material is based on:
  - 1993 draft SEAONC EDWG document
  - Proceedings of ATC 17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control (March 1993)
  - Special issue of Earthquake Spectra (August 1993)

- Applicable to wide range of EDD's; therefore requires EDD performance verification via prototype testing

- Performance objective identical to conventional structural system (i.e., life-safety for design EQ)

- At least two EDD per story in each principal direction, distributed continuously from base to top of building unless adequate performance (drift limits satisfied and member curvature capacities not exceeded) with incomplete vertical distribution can be demonstrated

- Members that transmit damper forces to foundation designed to remain elastic

1994 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (2 of 4)

Part 1 – Provisions & Part 2 – Commentary (FEMA 222A & 223A)

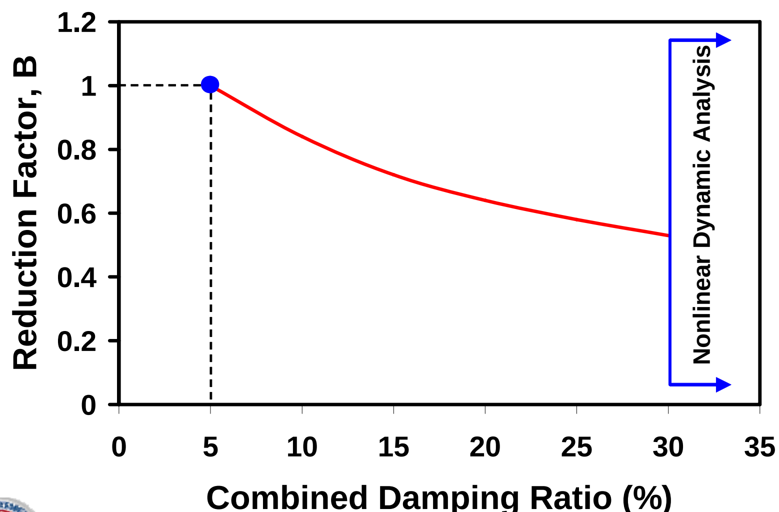
Analysis/Design Procedure for Linear Viscous Energy Dissipation Systems

$$V_{min} = BV = BC_S W$$

V_{min} = Minimum base shear for design of structure with EDS
[Use for linear static (ELF) or linear dynamic (Modal) analysis]

V = Minimum base shear for design of structure without EDS

B = Reduction factor to account for energy dissipation provided by EDS
(based on combined, inherent plus added damping, damping ratio)



***Note:** After publication, it was recognized that this procedure may not be appropriate since it allows reduction in forces due to both inelastic structural response (R-factor) and added damping (B-factor). For yielding structures, added damping will not reduce forces.*

1994 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (3 of 4)

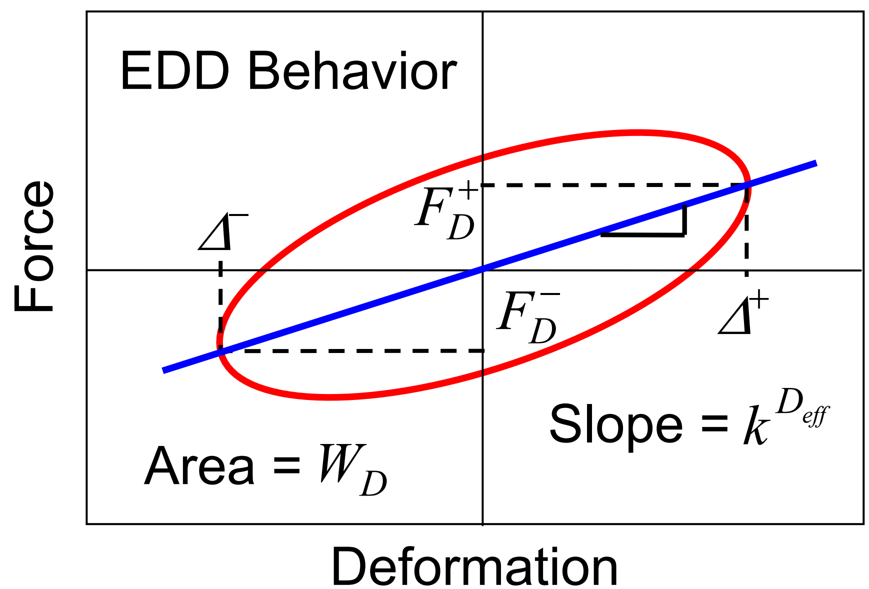
Part 1 – Provisions & Part 2 – Commentary (FEMA 222A & 223A)

Analysis/Design Procedure for EDD's other than Linear Viscous Dampers

1) Preliminary Design: Linear dynamic modal analysis using effective stiffness and damping coefficient of energy dissipation devices. Use B-factor to reduce modal base shears.

$$k^{D_{eff}} = \frac{\left|F_D^+\right| + \left|F_D^-\right|}{\left|\Delta^+\right| + \left|\Delta^-\right|}$$

Eq. (C2A.3.2.1a)
Effective Device Stiffness at Design Displacement



$$c_{eq} = 2m\omega_n\xi_{eq} = \frac{2m\omega_n W_D}{4\pi W_S} = \frac{W_D T}{2\pi^2\Delta^2}$$

Eq. (2A.3.2.1)
Equivalent Device Damping Coefficient

$$\xi_{combined} = \xi_{str} + \frac{\sum W_D}{4\pi SE}$$

Eq. (C2A.3.2.1c)
Combined Equivalent Damping Ratio

2) Performance Verification: Nonlinear response history analysis

1994 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (4 of 4)

Part 1 – Provisions & Part 2 – Commentary (FEMA 222A & 223A)

- For nonlinear response-history analysis, mathematical modeling should account for:
  - Plan and vertical spatial distribution of EDD’s
  - Dependence of EDD’s on loading frequency, temperature, sustained loads, nonlinearities, and bilateral loads

- Prototype Tests on at least two full-size EDD’s (unless prior testing has been documented)
  - 200 fully reversed cycles corresponding to wind forces
  - 50 fully reversed cycles corresponding to design earthquake
  - 10 fully reversed cycles corresponding to maximum capable earthquake

- Acceptability criteria from prototype testing of EDD’s:
  - Hysteresis loops have non-negative incremental force-carrying capacities (for rate-independent systems only)
  - Exhibit limited effective stiffness degradation under repeated cyclic load
  - Exhibit limited degradation in energy loss per cycle under repeated cyclic load
  - Have quantifiable engineering parameters
  - Remain stable at design displacements

1997 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures

Part 1 – Provisions & Part 2 – Commentary (FEMA 302 & 303)

- Includes an appendix to Chapter 13 entitled:
Passive Energy Dissipation

- The appendix in the 1994 NEHRP Provisions was deleted since it was deemed to be insufficient for design and regulation. It was replaced with 3 paragraphs that provide very general guidance on passive energy dissipation systems.

1997 - NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273)
1997 - NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA 274) (1 of 9)

- Chapter 9 entitled: Seismic Isolation and Energy Dissipation
  (Developed by New Technologies Team under ATC Project 33)

- Performance-based document
    - Rehabilitation objectives based on desired performance levels for selected hazard levels

- Global Structural Performance Levels
    - Operational (OP)
    - Immediate Occupancy (IO)
    - Life-Safety (LS)
    - Collapse Prevention (CP)

Most Applicable Performance Levels

- Hazard levels
    - Basic Safety Earthquake 1 (BSE-1): 10/50 event
    - Basic Safety Earthquake 2 (BSE-2): 2/50 event (Maximum Considered EQ - MCE)

- Rehabilitation Objectives
    - Limited Objectives (less than BSO)
    - Basic Safety Objective (BSO): LS for BSE-1 and CP for BSE-2
    - Enhanced Objectives (more than BSO)

Applicable Rehabilitation Objectives

1997 - NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273)
1997 - NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA 274) (2 of 9)

-Simplified vs. Systematic Rehabilitation
- Simplified: For simple structures in areas of low to moderate seismicity
- Systematic: Considers all elements needed to attain rehabilitation objective

- Systematic Rehabilitation methods of analysis:
  - Linear static procedure (LSP)
  - Linear dynamic procedure (LDP)
  - Nonlinear static procedure (NSP)
  - Nonlinear dynamic procedure (NDP)

Coefficient Method

Capacity Spectrum Method

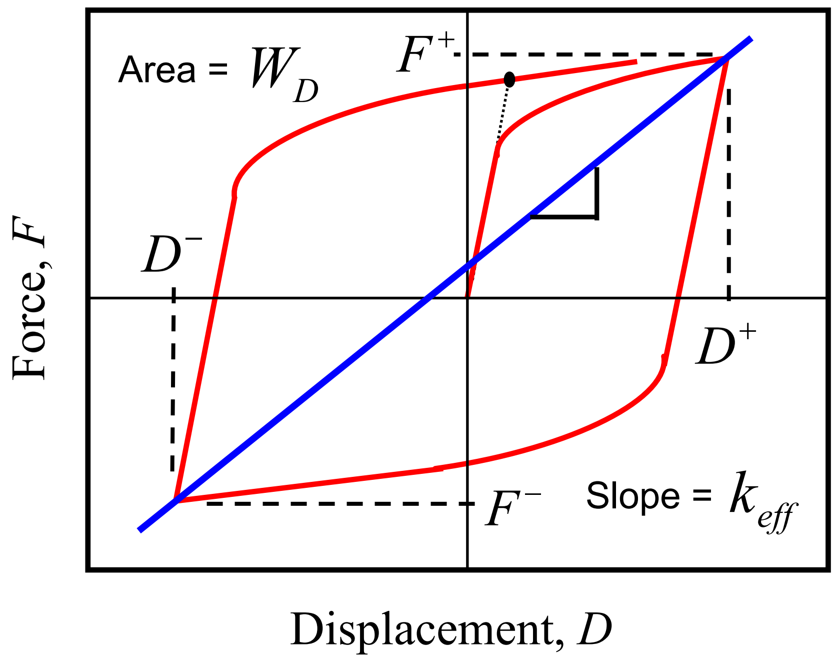
1997 - NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273) 1997 - NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA 274) (3 of 9)

- **Basic Principles:**
  - Dampers should be spatially distributed (at each story and on each side of building)
  - Redundancy (at least two dampers along the same line of action; design forces for dampers and damper framing system are reduced as damper redundancy is increased)
  - For BSE-2, dampers and their connections designed to avoid failure (i.e, *not* weak link)
  - Members that transmit damper forces to foundation designed to remain elastic

- **Classification of EDD's**
  - Displacement-dependent
  - Velocity-dependent
  - Other (e.g., shape memory alloys and fluid restoring force/damping dampers)

Manufacturing quality control program should be established along with prototype testing programs and independent panel review of system design and testing program

Mathematical Modeling of Displacement-Dependent Devices



$$F = k_{eff} D$$

Eq. (9-20)
Force in Device

$$k_{eff} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|}$$

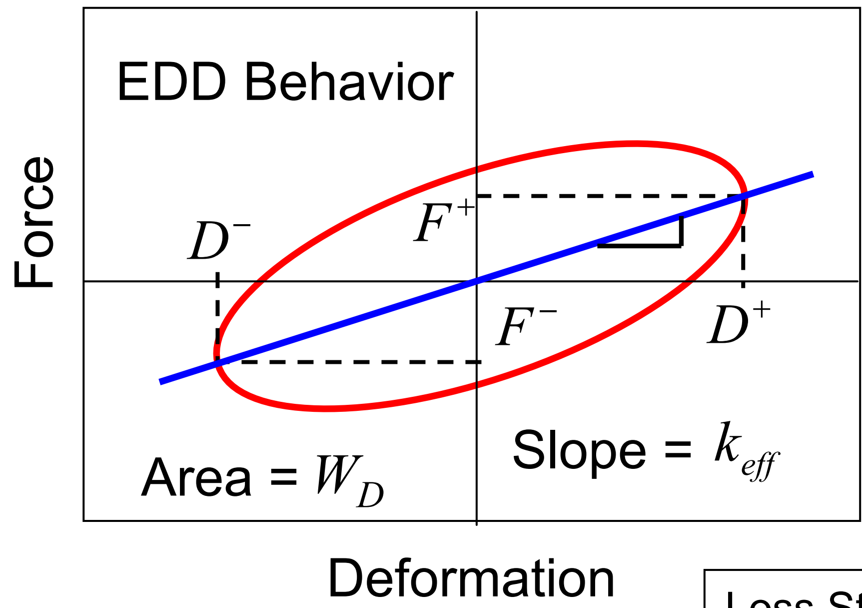
Eq. (9-21)
Effective Stiffness of Device

$$\beta_{eff} = \frac{1}{2\pi} \frac{W_D}{k_{eff} D_{ave}^2}$$

Eq. (9-39)
Equivalent Viscous Damping Ratio of Device

1997 - NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273)
1997 - NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA 274) (5 of 9)

Mathematical Modeling of Solid Viscoelastic Devices



$$F = k_{eff} D + C\dot{D}$$

Eq. (9-22)
Force in Device

$$k_{eff} = \frac{|F^+| + |F^-|}{|D^+| + |D^-|} = K'$$

Eq. (9-23)
Effective Stiffness of Device

Storage Stiffness

Loss Stiffness

$$C = \frac{W_D}{\pi \omega_1 D_{ave}^2} = \frac{K''}{\omega_1}$$

Eq. (9-24)
Damping Coefficient of Device

Average Peak Displ.

Circular frequency of mode 1

1997 - NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273)
1997 - NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA 274) (6 of 9)

Mathematical Modeling of Fluid Viscoelastic and Fluid Viscous Devices

Fluid Viscoelastic Devices:

$$F + \lambda \dot{F} = C\dot{D}$$

Maxwell Model

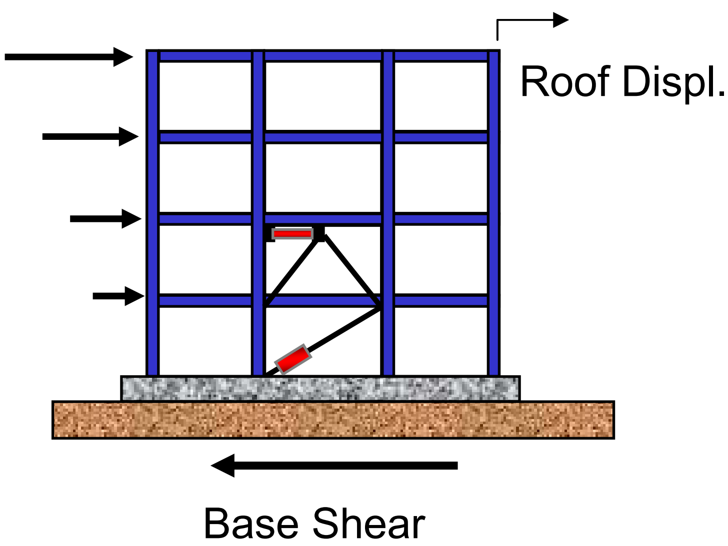
Fluid Viscous Devices:

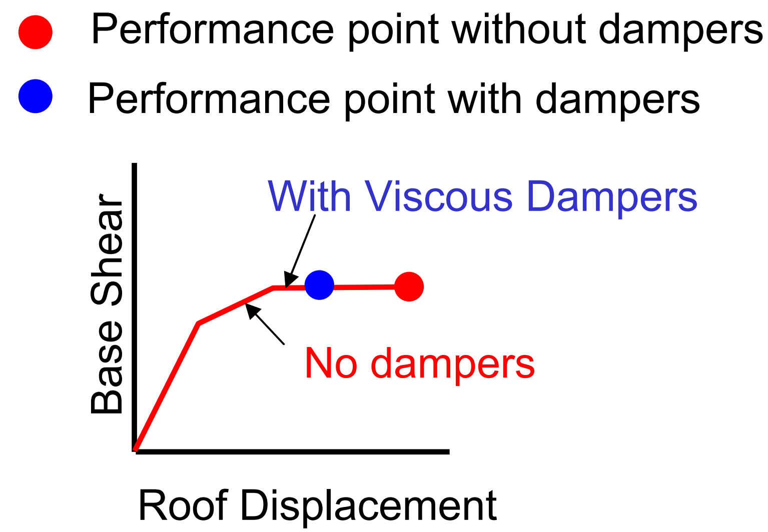
$$F = C_0 \left|\dot{D}\right|^{\alpha} sgn\left(\dot{D}\right)$$

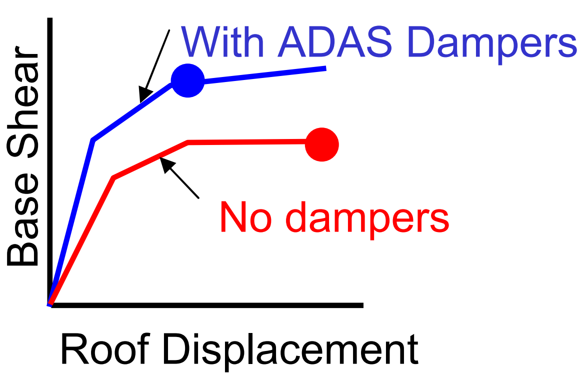
Eq. (9-25)
Linear or Nonlinear Dashpot Model

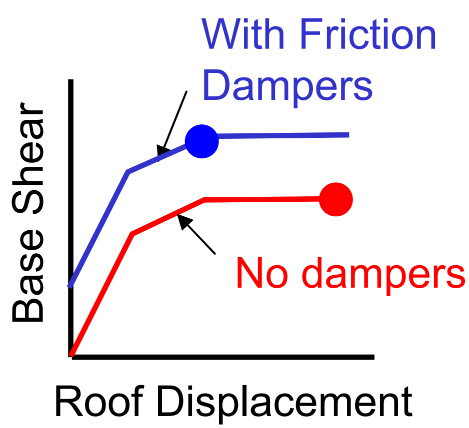
<u>Caution:</u> Only use fluid viscous device model if $K' \approx 0$ for frequencies between 0.5 f_1 and 2.0 f_1; Otherwise, use fluid viscoelastic device model.

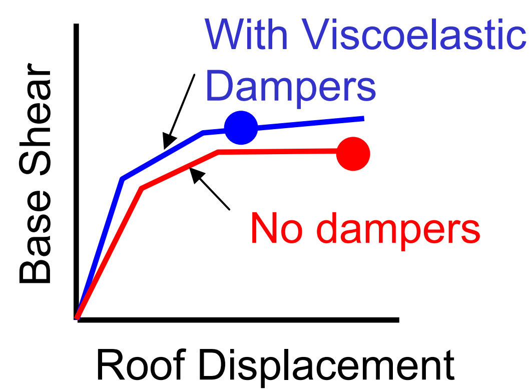
Pushover Analysis for Structures with EDD's (Part of NSP)











Reduced Displacement → **Reduced Damage**

Design Process for Velocity-Dependent Dampers using NSP

Steps

1) Estimate Target Displacement (performance point)
2) Calculate Effective Damping Ratio and Secant Stiffness of building with dampers at Target Displacement
3) Use Effective Damping and Secant Stiffness to calculate revised Target Displacement
4) Compare Target Displacement from Steps 1 and 4.
   If within tolerance, stop. Otherwise, return to Step 1.

$$\beta_{eff} = \beta + \frac{\sum_j W_j}{4\pi W_k}$$

Effective damping ratio of building with dampers at Target Displ.;
j = index over devices

$$W_k = \frac{1}{2}\sum_i F_i \delta_i$$

Maximum strain energy in building with dampers at Target Displ.;
i = index over floor levels

Design Process for Velocity-Dependent Dampers using NSP (2)

$$W_j = \frac{2\pi^2}{T_S} C_j \delta_{rj}^2$$

Work done by j-th damper with building subjected to Target Displacement (assumes harmonic motion with amplitude equal to Target Displacement and frequency corresponding to Secant Stiffness at Target Displacement)

$$\beta_{eff} = \beta + \frac{T_S \sum_j C_j \cos^2 \theta_j \phi_{rj}^2}{4\pi \sum_i m_i \phi_i^2}$$

Alternate expression for Effective Damping Ratio that uses modal amplitudes of first mode shape

Checking Building Component Behavior (Forces and Deformations)

For velocity-dependent dampers, must check component behavior at three stages:

1) Maximum Displacement
2) Maximum Velocity
3) Maximum Acceleration

2000 – Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356)

- Prestandard version of 1997 NEHRP Guidelines and Commentary for the Seismic Rehabilitation of Buildings (FEMA 273 & 274)

- Prepared by ASCE for FEMA

- Prestandard = Document has been accepted for use as the start of the formal standard development process (i.e., it is an initial draft for a consensus standard)

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (1 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)

- **Appendix to Chapter 13 entitled *Structures with Damping Systems***
(completely revised/updated version of 1994 and 1997 Provisions; Brief commentary provided)

- **Intention:**
  - **Apply to all energy dissipation systems (EDS)**
  - **Provide design criteria compatible with conventional and enhanced seismic performance**
  - **Distinguish between design of members that are part of EDS and members that are independent of EDS.**

- **The seismic force resisting system must comply with the requirements for the system's Seismic Design Category, except that the damping system may be used to meet drift limits.**

  No reduction in detailing is thereby allowed, even if analysis shows that the damping system is capable of producing significant reductions in ductility demand or damage.

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (2 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)

- Members that transmit damper forces to foundation designed to remain elastic

- Prototype tests on at least two full-size EDD's (reduced-scale tests permitted for velocity-dependent dampers)

- Production testing of dampers prior to installation.

- Independent engineering panel for review of design and testing programs

- Residual mode concept introduced for linear static analysis. This mode, which is in addition to the fundamental mode, is used to account for the combined effects of higher modes. Higher mode interstory-velocities can be significant and thus are important for velocity-dependent dampers.

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (3 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)

Methods of Analysis:

- Linear Static (Equivalent Lateral Force*)
  - *OK for Preliminary Design*
- Linear Dynamic (Modal Response Spectrum*)
  - *OK for Preliminary Design*
- Nonlinear Static (Pushover*)
  - *May Produce Significant Errors*
- Nonlinear Dynamic (Response History)
  - *Required if S_1 > 0.6 g and may be used in all other cases*

*The *Provisions* allow final design using these procedures, but only under restricted circumstances.

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (4 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)

Effective Damping Ratio
(used to determine factors, B, that reduce structure response)

$$\beta_m = \beta_I + \underbrace{\beta_{Vm}\sqrt{\mu}} + \beta_H$$

Hysteretic Damping Due to Post-Yield Behavior in Structure

Equivalent Viscous Damping of EDS in the m-th Mode

Inherent Damping Due to Pre-Yield Energy Dissipation of Structure
(β_I = 5% or less unless higher values can be justified)

Effective Damping Ratio in m-th mode of vibration

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (5 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)

Equivalent Viscous Damping from EDS

$$\beta_m = \beta_I + \boxed{\beta_{Vm}\sqrt{\mu}} + \beta_H$$

$$\beta_{Vm} = \frac{\sum_j W_{mj}}{4\pi W_m}$$

Equivalent Viscous Damping in m-th mode (due to EDS)

$$W_m = \frac{1}{2}\sum_i F_{im}\delta_{im}$$

Maximum Elastic Strain Energy of structure in m-th mode

$$\sqrt{\mu}$$

Adjustment factor that accounts for dominance of post-yielding inelastic hysteretic energy dissipation

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (6 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)

Base Shear Force

Minimum base shear for design of structure without EDS

$$V_{min} = max\left\{\frac{V}{B_{V+I}};\ 0.75V\right\}$$

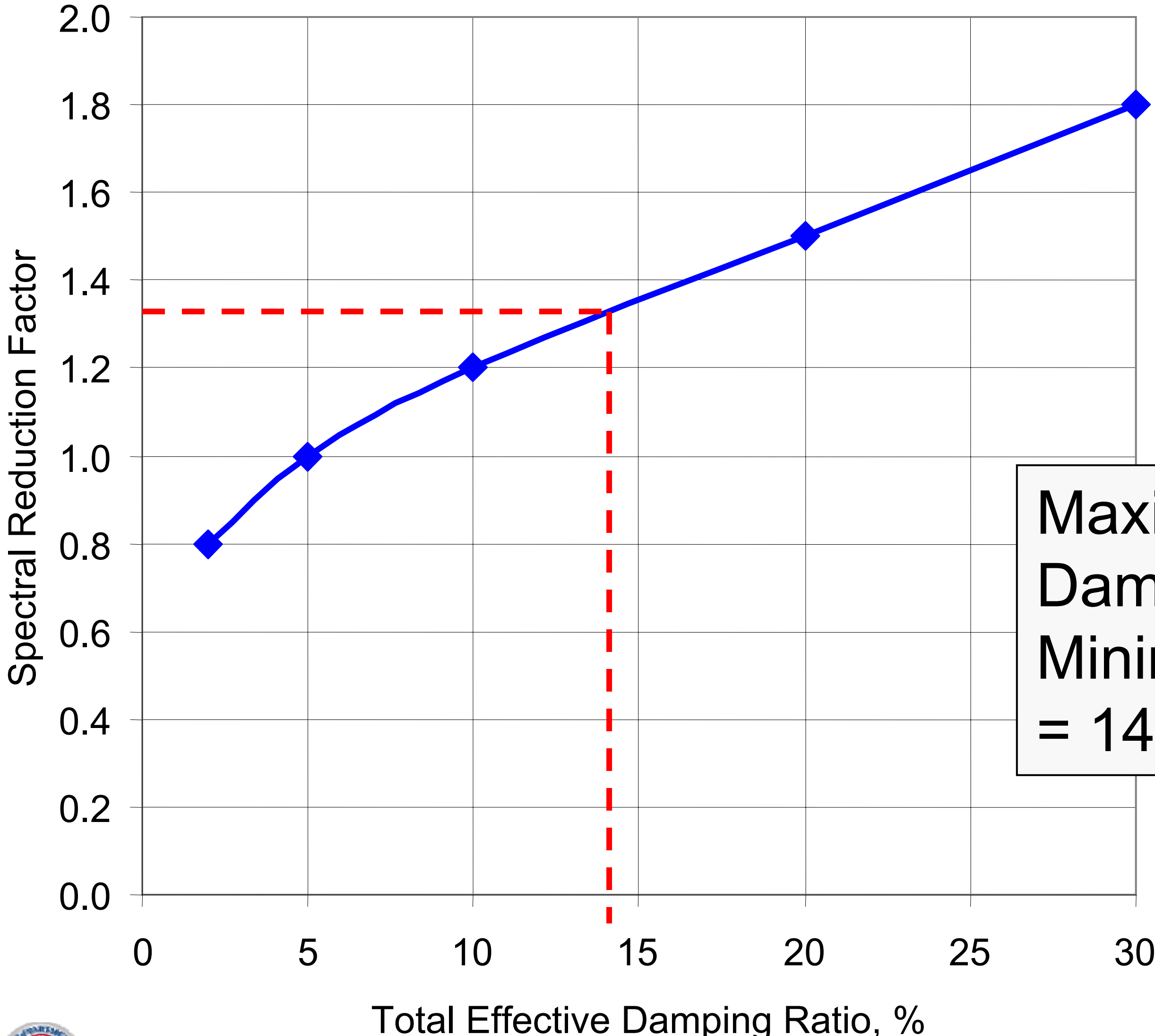
Minimum base shear for design of seismic force resisting system

Spectral reduction factor based on the sum of viscous and inherent damping

To protect against damper system malfunction, maximum reduction in base shear over a conventional structure is 25%

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (7 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)



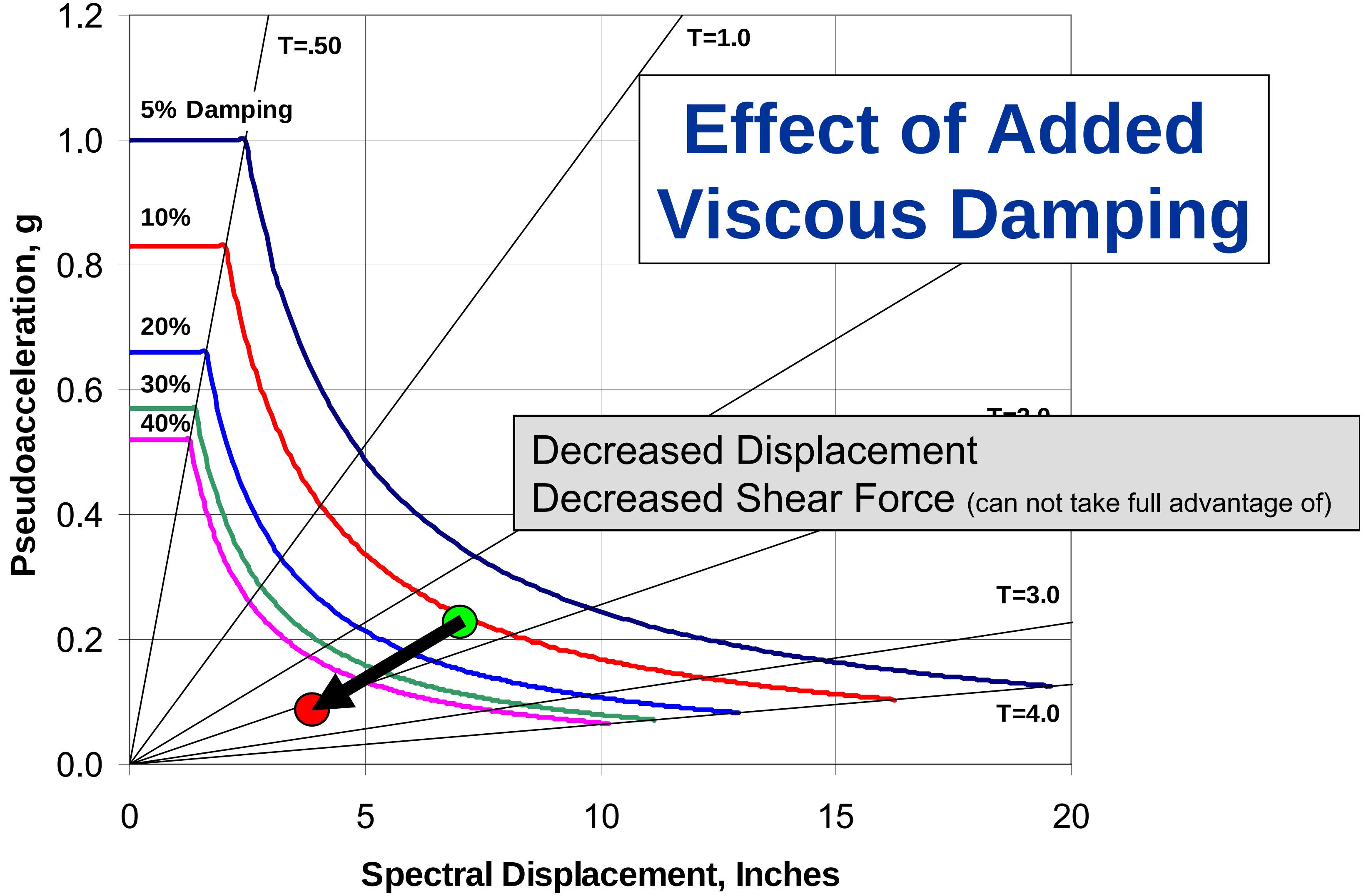
$$V_{min} \geq 0.75V = \frac{V}{1.33}$$

Maximum base shear reduction factor

Maximum Added Damping WRT Minimum Base Shear = 14 - 5 = 9%

2000 - NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (8 of 8)

Part 1 – Provisions & Part 2 – Commentary (FEMA 368 & 369)

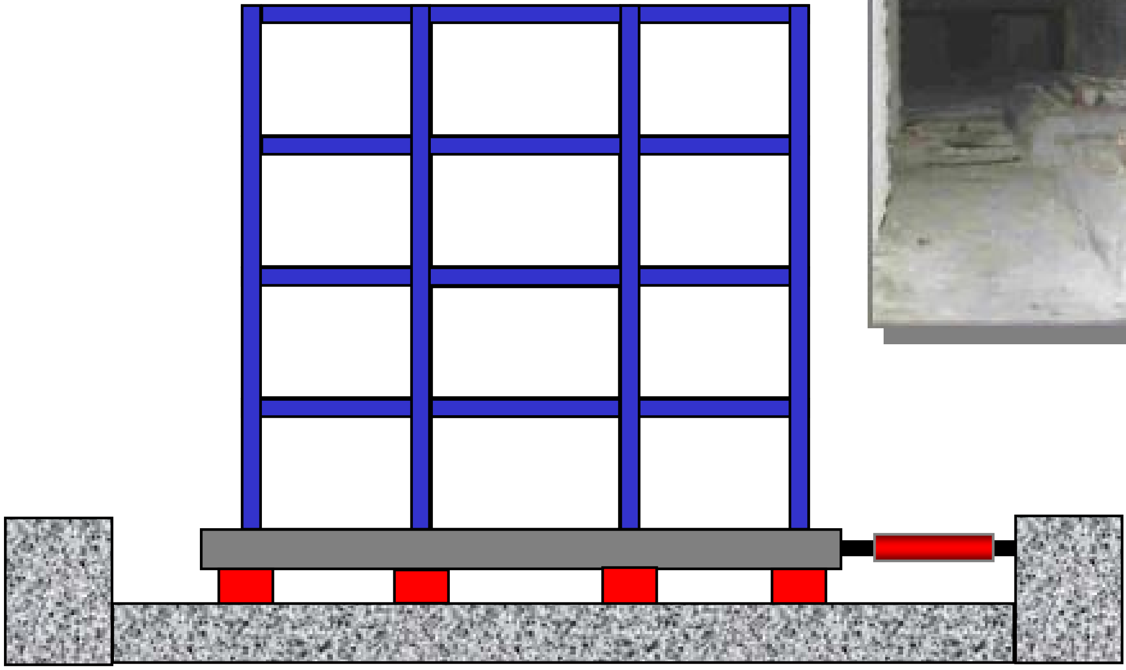


SEISMIC PROTECTIVE SYSTEMS: SEISMIC ISOLATION

Developed by:

Michael D. Symans, PhD

Rensselaer Polytechnic Institute



Major Objectives

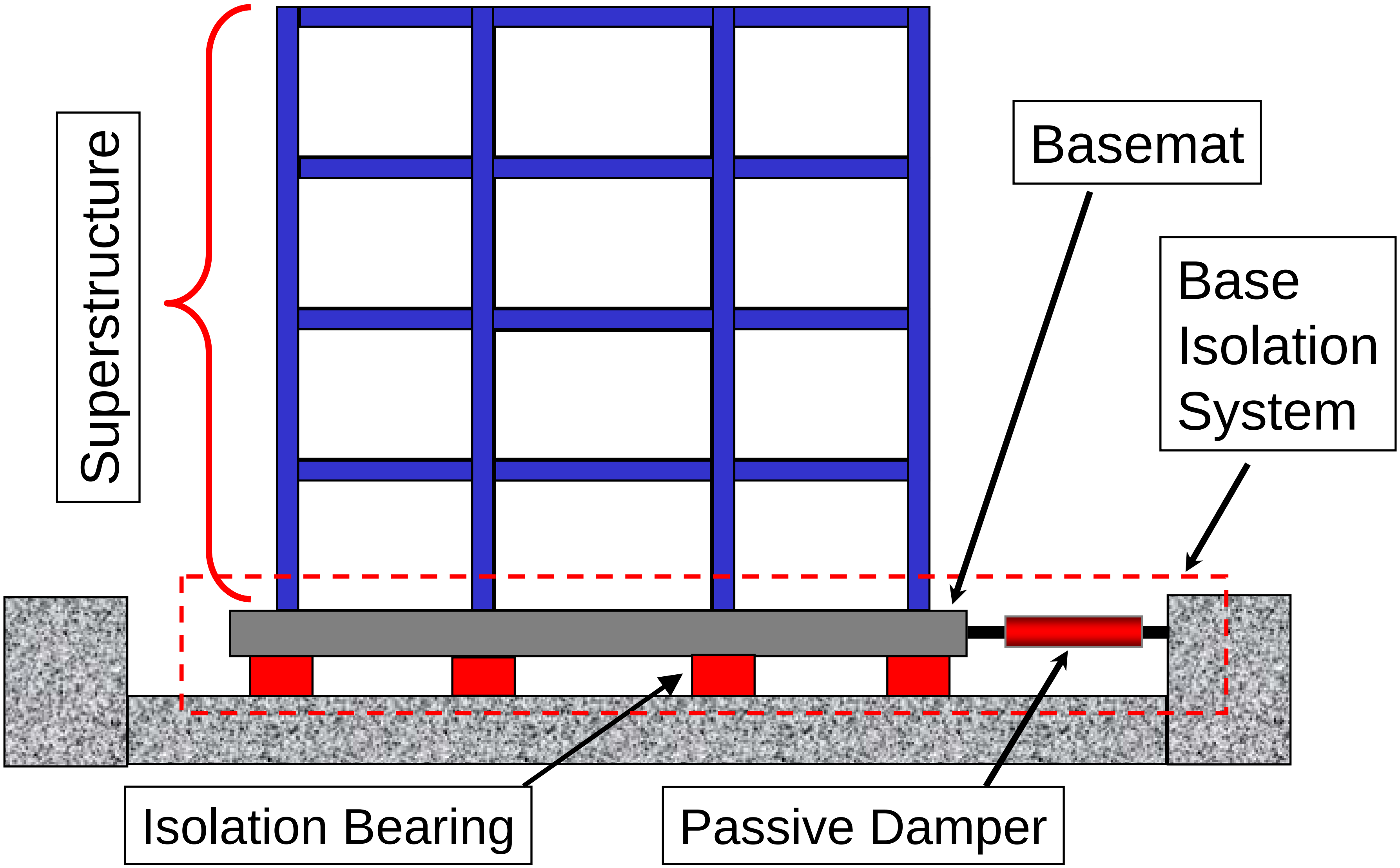
- Illustrate why use of seismic isolation systems may be beneficial
- Provide overview of types of seismic isolation systems available
- Describe behavior, modeling, and analysis of structures with seismic isolation systems
- Review building code requirements

Outline

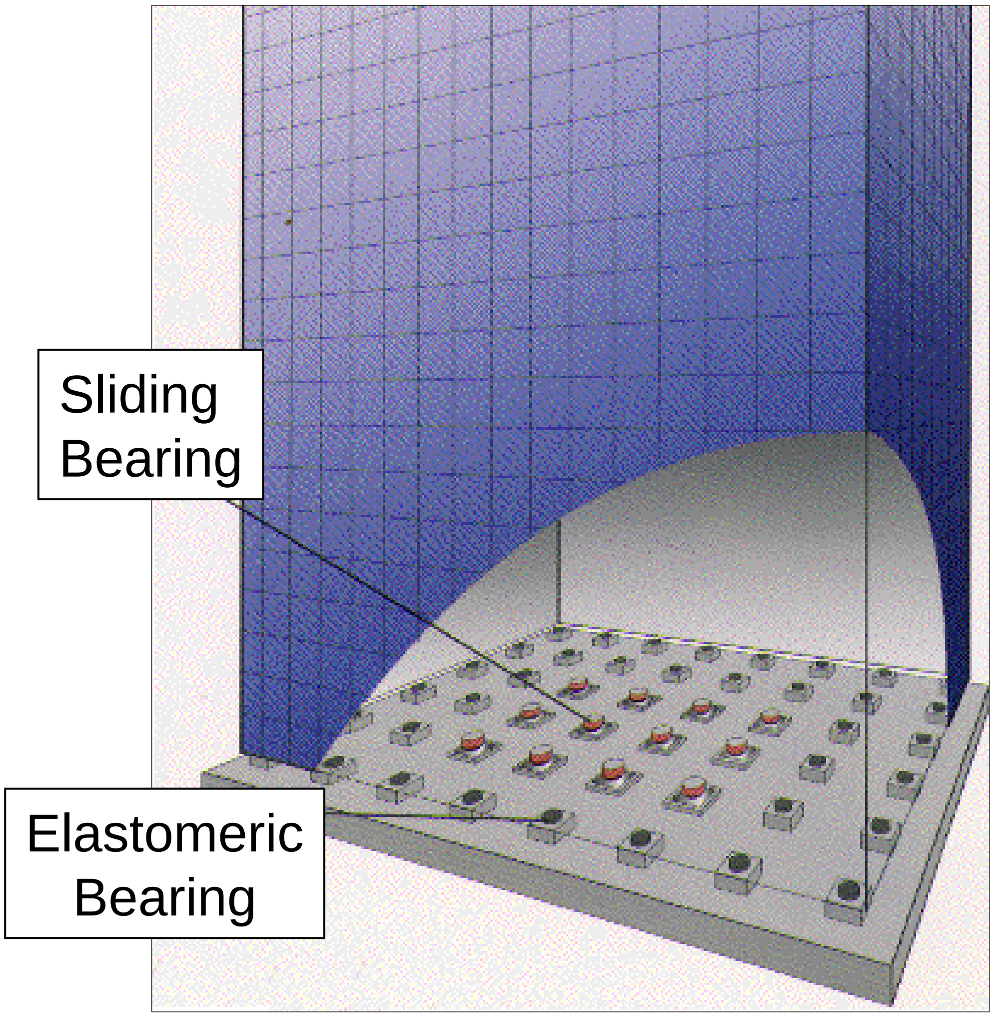
Seismic Base Isolation

– Configuration and Qualitative Behavior of Isolated Building
– Objectives of Seismic Isolation Systems
– Effects of Base Isolation on Seismic Response
– Implications of Soil Conditions
– Applicability and Example Applications of Isolation Systems
– Description and Mathematical Modeling of Seismic Isolation Bearings
  - Elastomeric Bearings
  - Sliding Bearings
– Modeling of Seismic Isolation Bearings in Computer Software
– Code Provisions for Base Isolation

Configuration of Building Structure with Base Isolation System

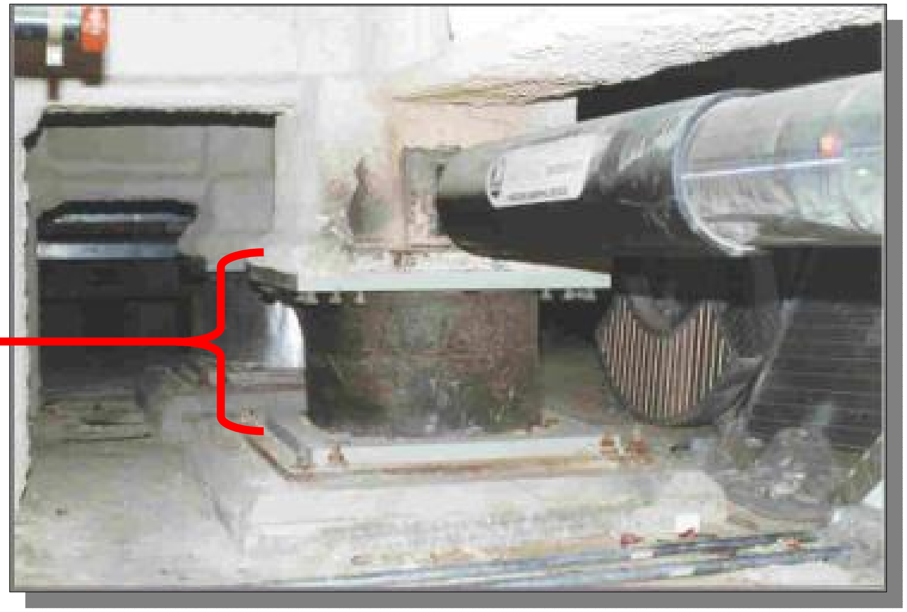


Three-Dimensional View of Building Structure with Base Isolation System

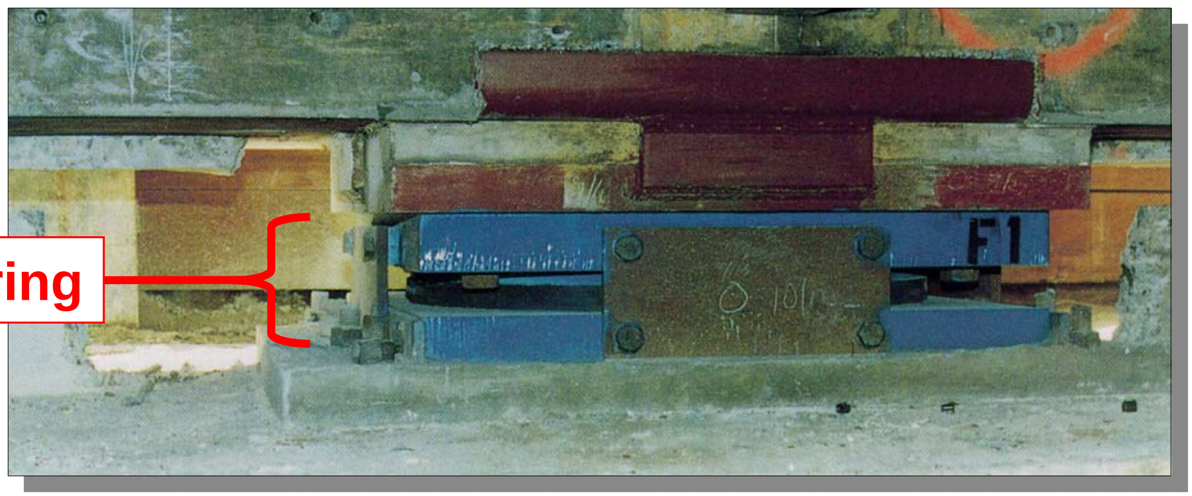


Installed Seismic Isolation Bearings

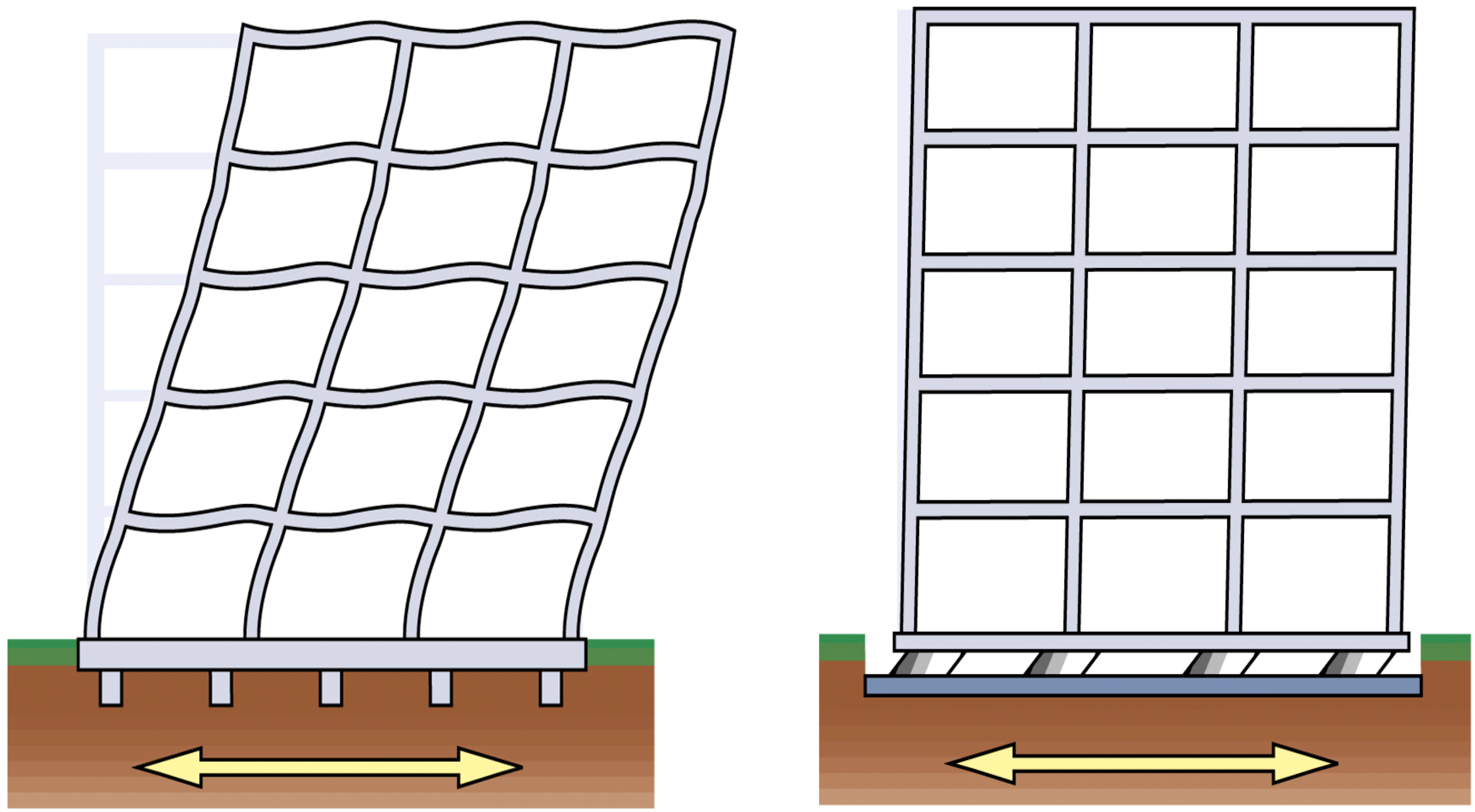
Elastomeric Bearing



Sliding Bearing



Behavior of Building Structure with Base Isolation System



Conventional Structure

Base-Isolated Structure

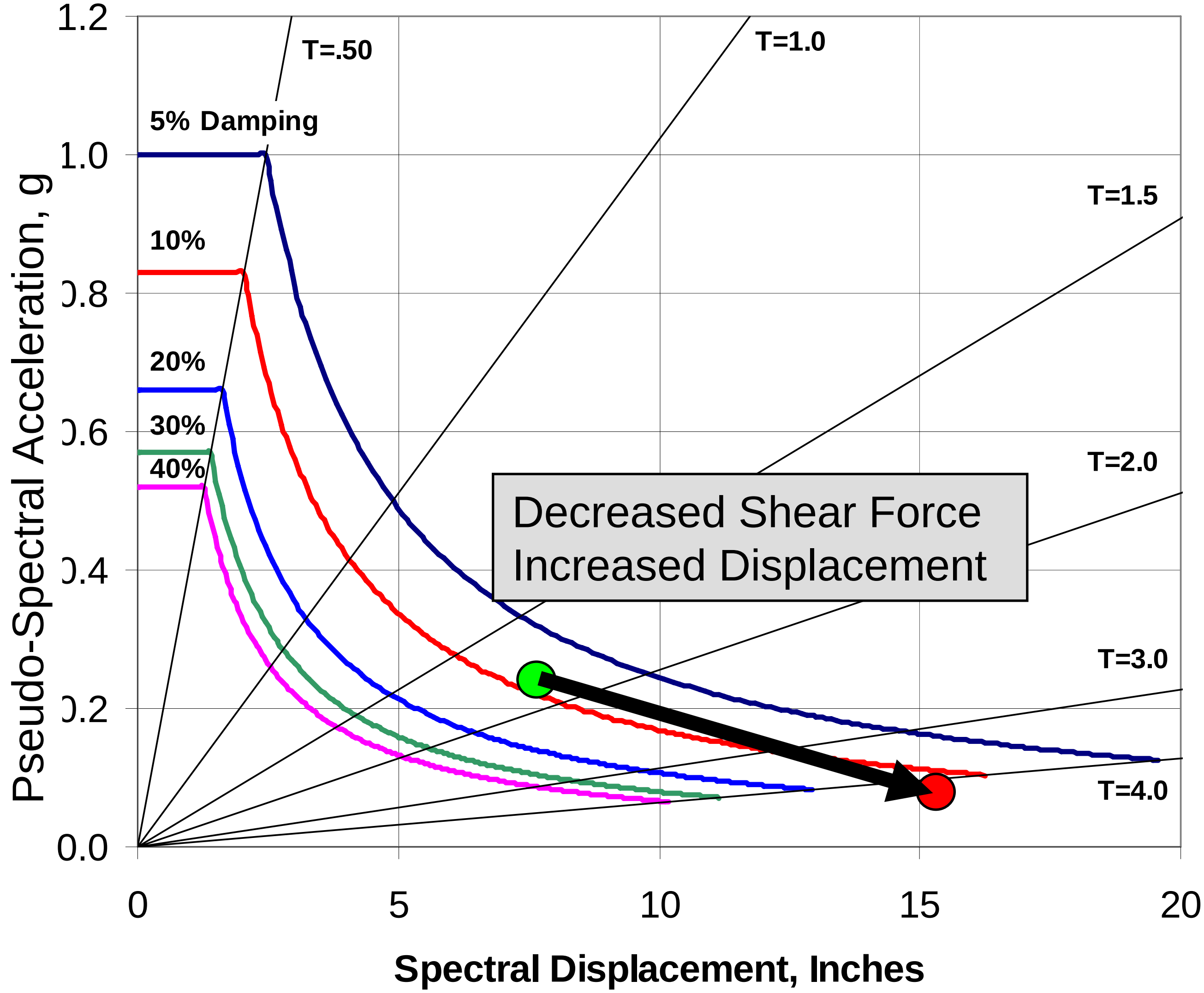
Objectives of Seismic Isolation Systems

- Enhance performance of structures at all hazard levels by:
  - Minimizing interruption of use of facility (*e.g., Immediate Occupancy Performance Level)*
  - Reducing damaging deformations in structural and nonstructural components
  - Reducing acceleration response to minimize contents-related damage

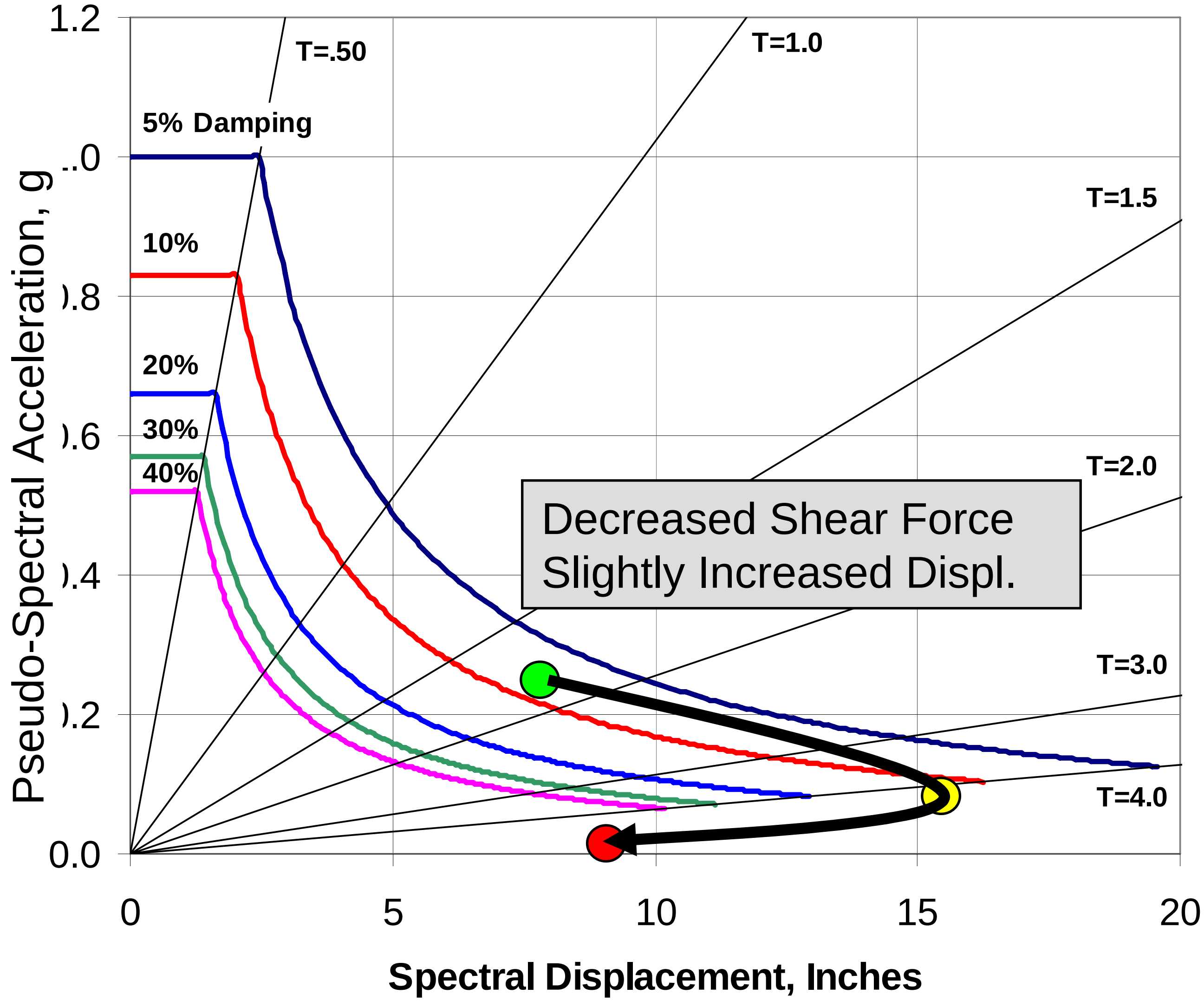
Characteristics of Well-Designed Seismic Isolation Systems

- Flexibility to increase period of vibration and thus reduce force response
- Energy dissipation to control the isolation system displacement
- Rigidity under low load levels such as wind and minor earthquakes

Effect of Seismic Isolation (ADRS Perspective)

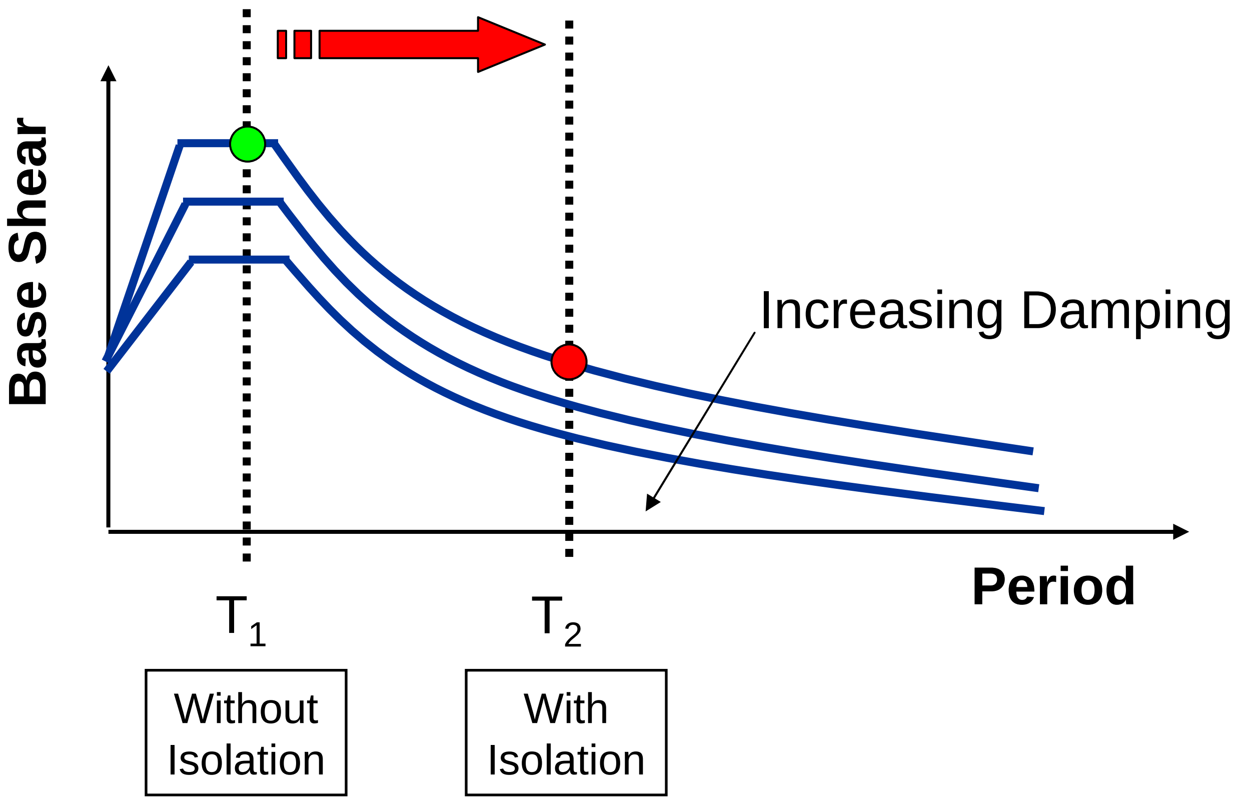


Effect of Seismic Isolation with Supplemental Dampers
(ADRS Perspective)



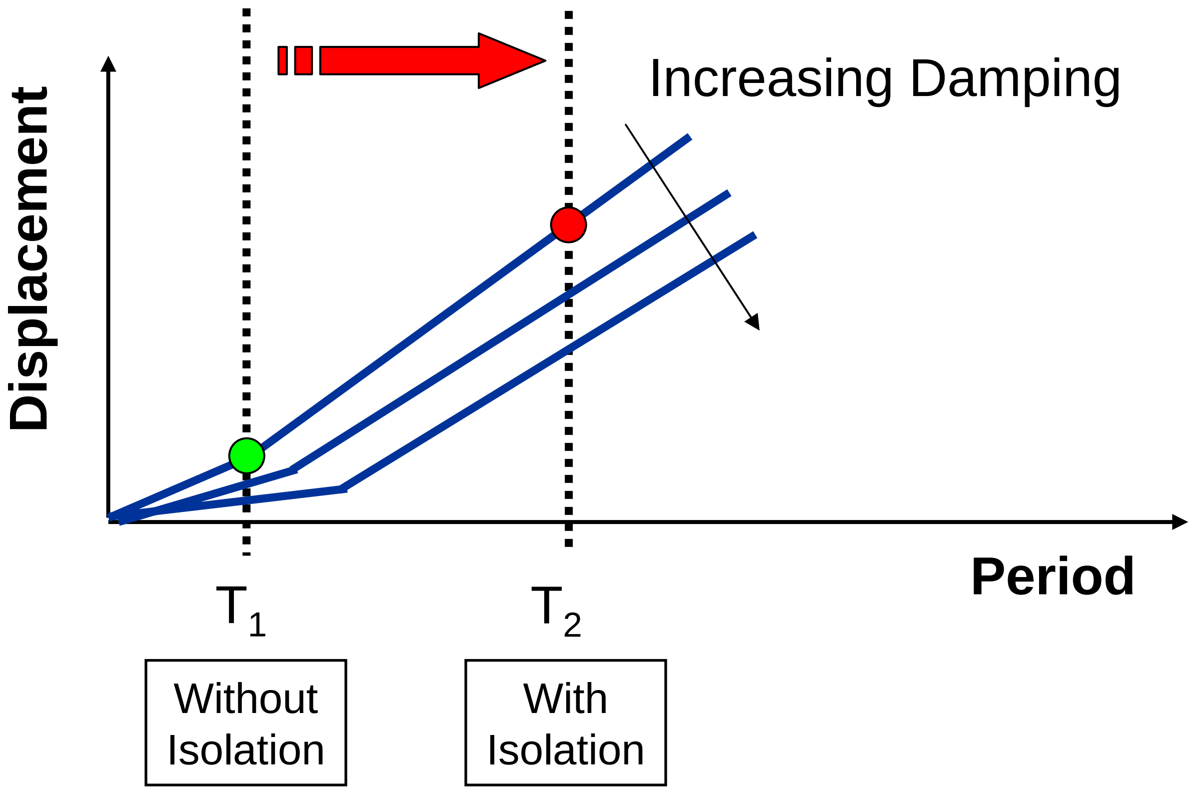
Effect of Seismic Isolation
(Acceleration Response Spectrum Perspective)

Increase Period of Vibration of Structure
to Reduce Base Shear

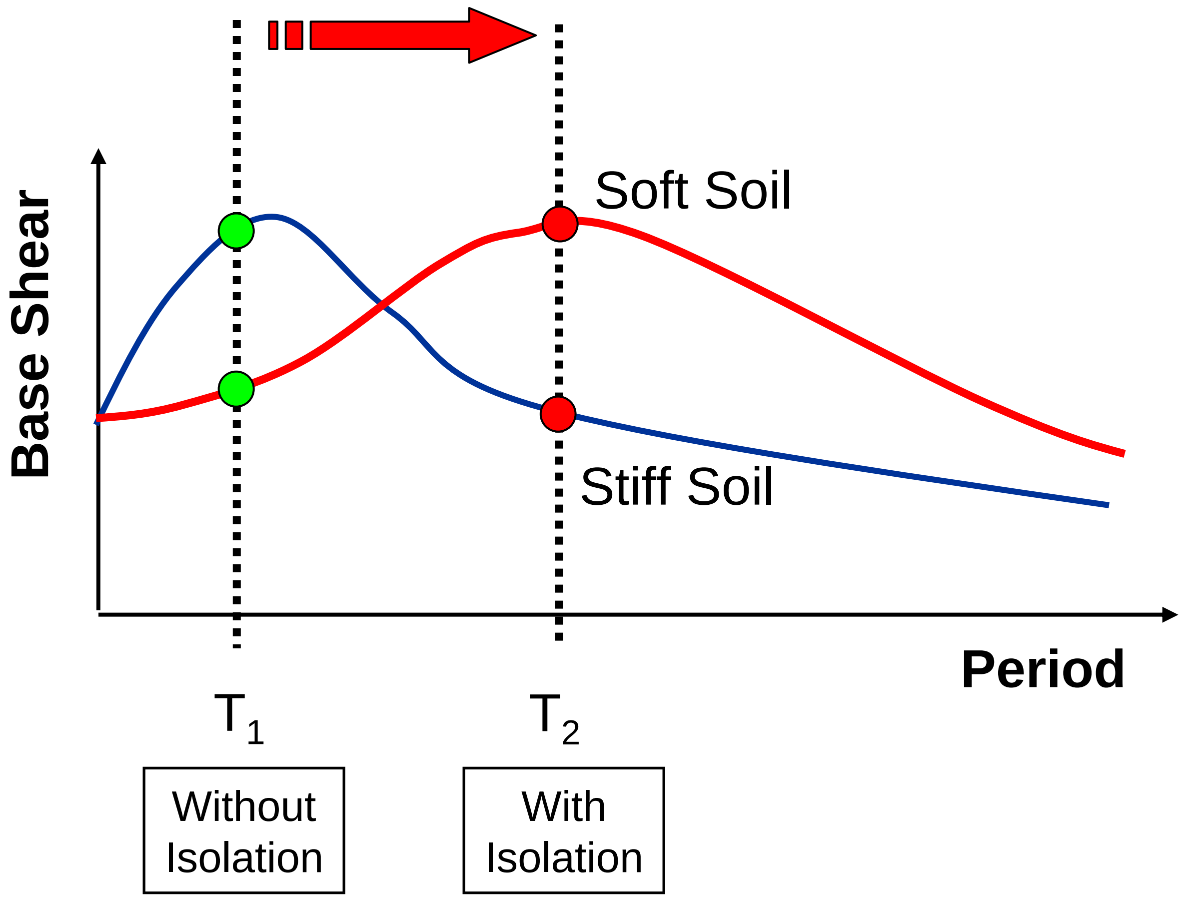


Effect of Seismic Isolation
(Displacement Response Spectrum Perspective)

Increase of period increases displacement demand (now concentrated at base)



Effect of Soil Conditions on Isolated Structure Response



Applicability of Base Isolation Systems

MOST EFFECTIVE

- Structure on Stiff Soil
- Structure with Low Fundamental Period (Low-Rise Building)

LEAST EFFECTIVE

- Structure on Soft Soil
- Structure with High Fundamental Period (High-Rise Building)

First Implementation of Seismic Isolation

Foothill Community Law and Justice Center, Rancho Cucamonga, CA

- Application to *new building* in 1985
- 12 miles from San Andreas fault
- Four stories + basement + penthouse
- Steel braced frame
- Weight = 29,300 kips
- 98 High damping elastomeric bearings
- 2 sec fundamental lateral period
- 0.1 sec vertical period
- +/- 16 inches displacement capacity
- Damping ratio = 10 to 20% (dependent on shear strain)

Application of Seismic Isolation to Retrofit Projects

Motivating Factors:

- Historical Building Preservation
  (minimize modification/destruction of building)

- Maintain Functionality
  (building remains operational after earthquake)

- Design Economy
  (seismic isolation may be most economic solution)

- Investment Protection
  (long-term economic loss reduced)

- Content Protection
  (Value of contents may be greater than structure)

Example of Seismic Isolation Retrofit

U.S. Court of Appeals,
San Francisco, CA

- Original construction started in 1905
- Significant historical and architectural value
- Four stories + basement
- Steel-framed superstructure
- Weight = 120,000 kips
- Granite exterior & marble, plaster, and hardwood interior
- Damaged in 1989 Loma Prieta EQ
- Seismic retrofit in 1994
- 256 Sliding bearings (FPS)
- Displacement capacity = +/-14 in.

Types of Seismic Isolation Bearings

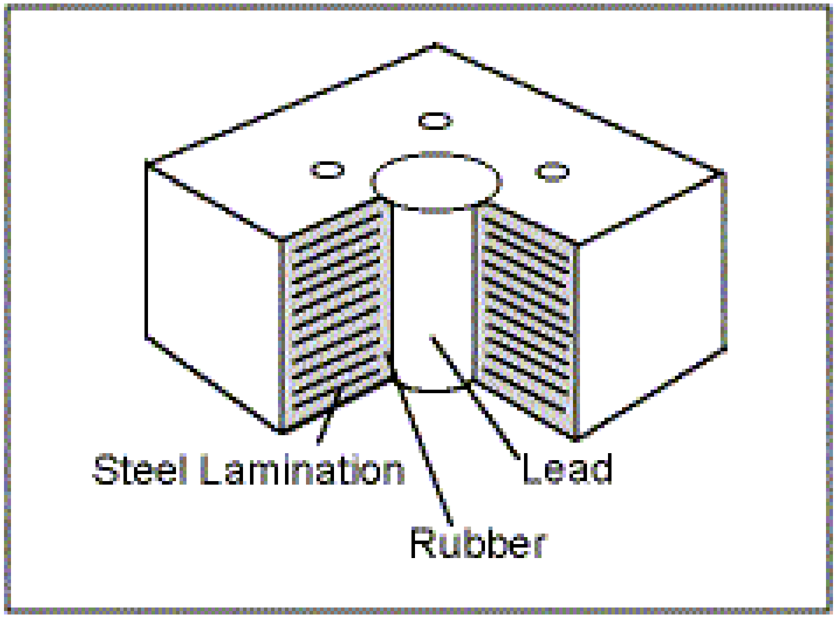
Elastomeric Bearings

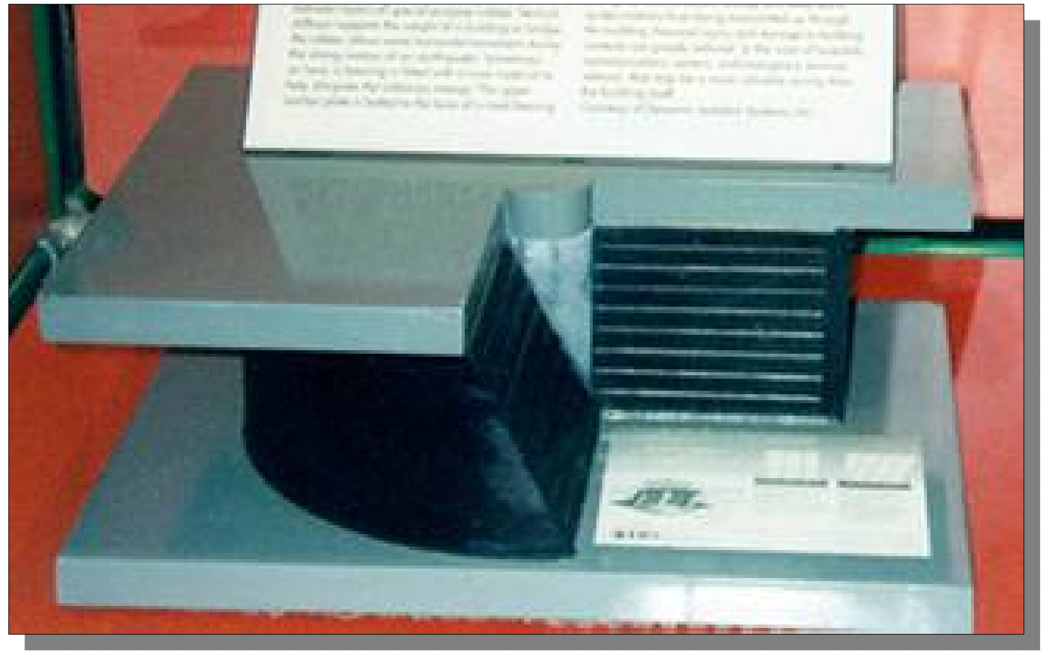
- Low-Damping Natural or Synthetic Rubber Bearing
- High-Damping Natural Rubber Bearing
- Lead-Rubber Bearing
  (Low damping natural rubber with lead core)

Sliding Bearings

- Flat Sliding Bearing
- Spherical Sliding Bearing

Geometry of Elastomeric Bearings

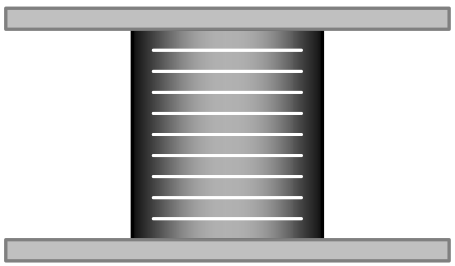




Major Components:

- Rubber Layers: Provide lateral flexibility
- Steel Shims: Provide vertical stiffness to support building weight while limiting lateral bulging of rubber
- Lead plug: Provides source of energy dissipation

Low Damping Natural or Synthetic Rubber Bearings



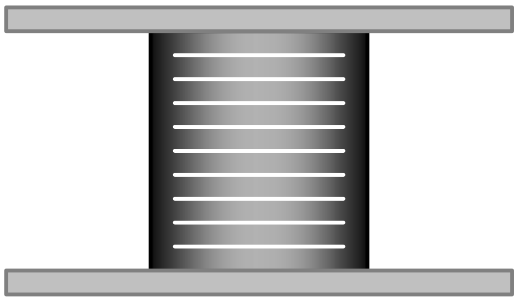
Linear behavior in shear for shear strains up to and exceeding 100%.

Damping ratio = 2 to 3%

Advantages:
- Simple to manufacture
- Easy to model
- Response not strongly sensitive to rate of loading, history of loading, temperature, and aging.

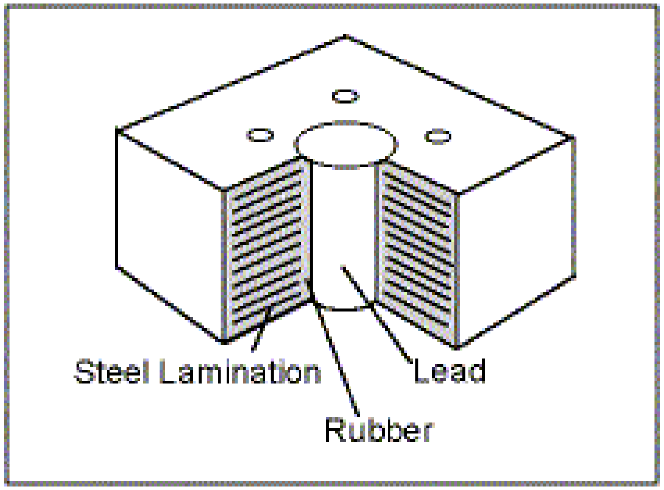
Disadvantage:
Need supplemental damping system

High-Damping Natural Rubber Bearings



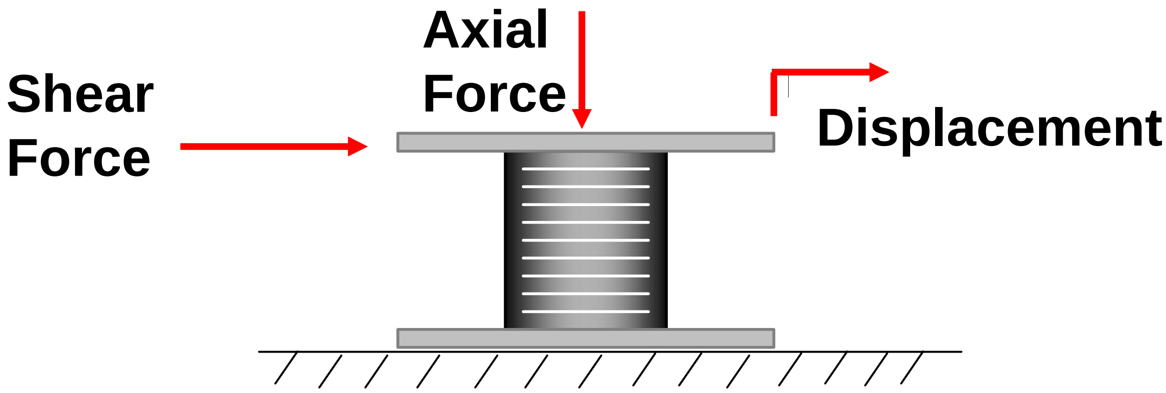
- Maximum shear strain = 200 to 350%
- Damping increased by adding extrafine carbon black, oils or resins, and other proprietary fillers
- Damping ratio = 10 to 20% at shear strains of 100%
- Shear modulus = 50 to 200 psi
- Effective Stiffness and Damping depend on:
  - Elastomer and fillers
  - Contact pressure
  - Velocity of loading
  - Load history (scragging)
  - Temperature

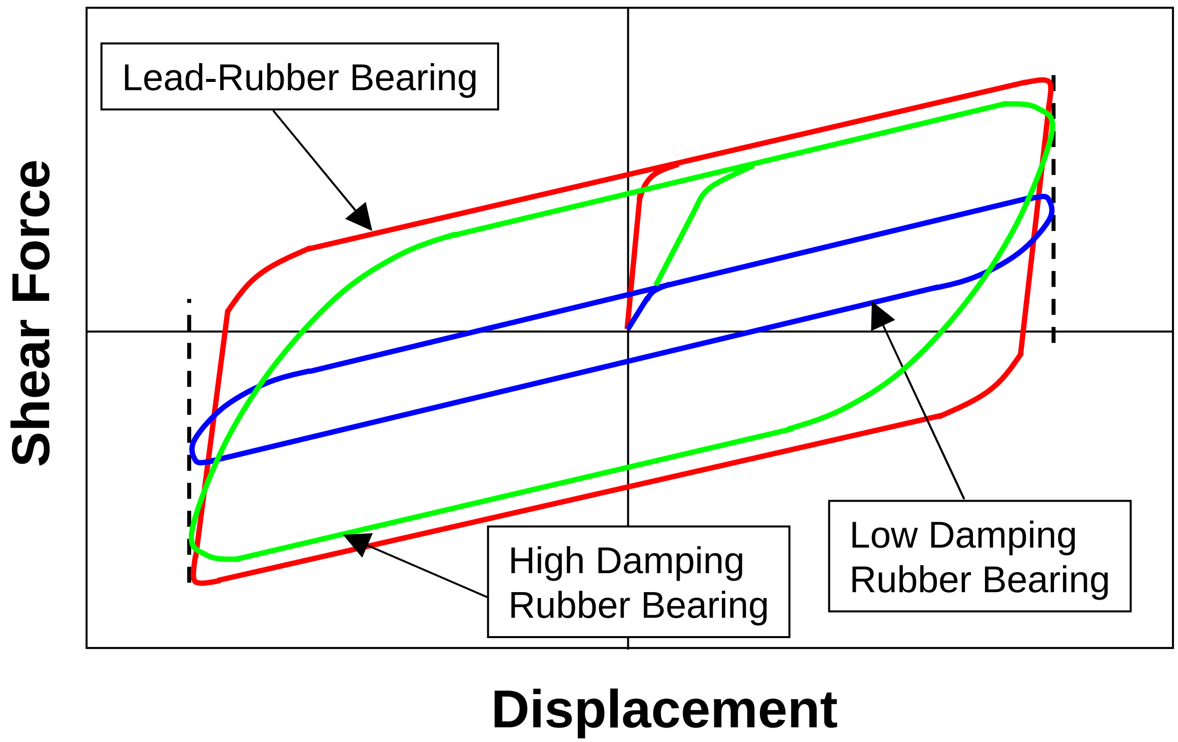
Lead-Rubber Bearings



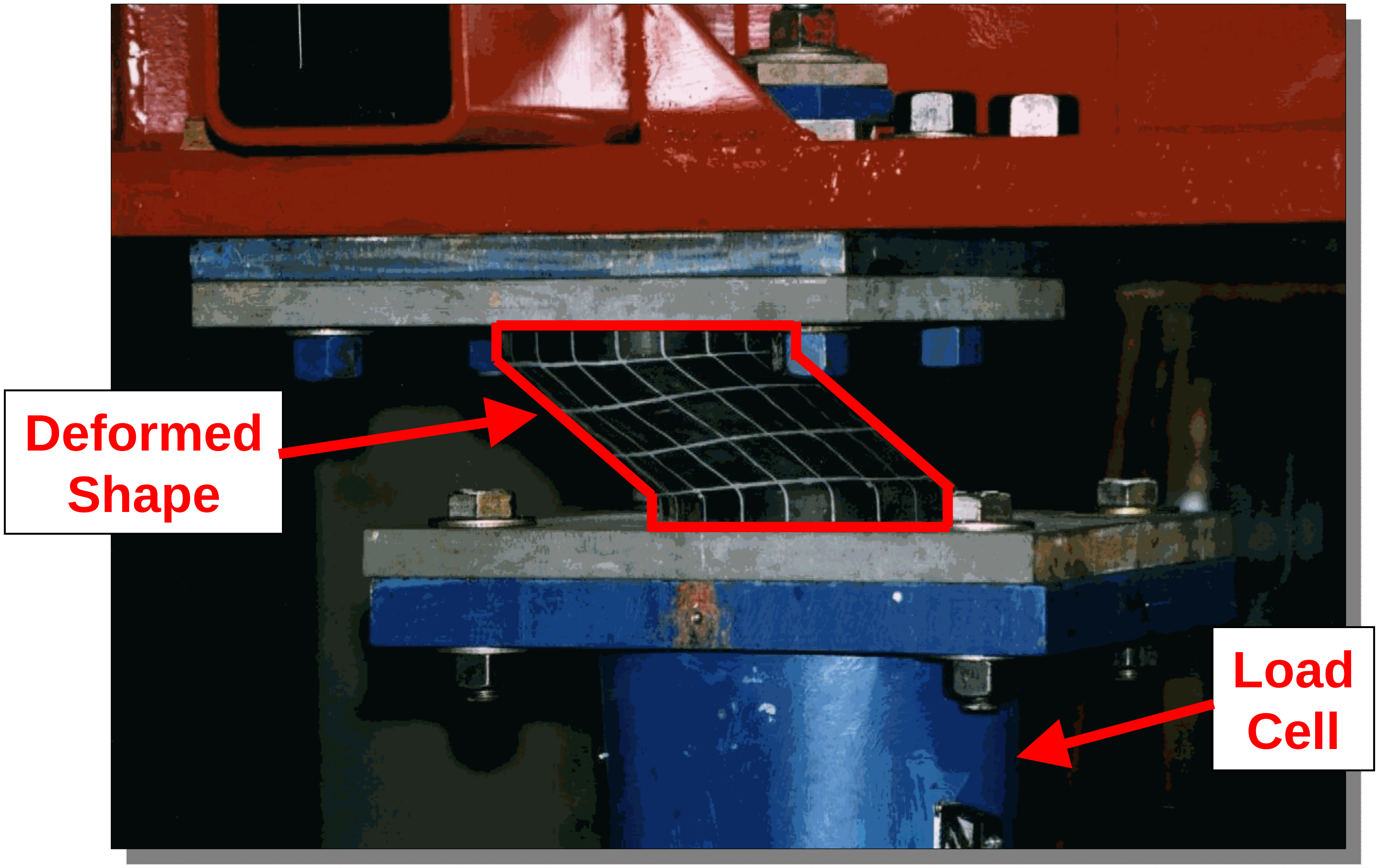
- Solid lead cylinder is press-fitted into central hole of elastomeric bearing
- Lead yield stress = 1500 psi (results in high initial stiffness)
- Invented in 1975 in New Zealand and used extensively in New Zealand, Japan, and the United States.
- Low damping rubber combined with central lead core
- Shear modulus = 85 to 100 psi at 100% shear strain
- Maximum shear strain = 125 to 200% (since max. shear strain is typically less than 200%, variations in properties are not as significant as for high-damping rubber bearings)
- Yield stress reduces with repeated cycling due to temperature rise
- Hysteretic response is strongly displacement-dependent

Elastomeric Bearing Hysteresis Loops



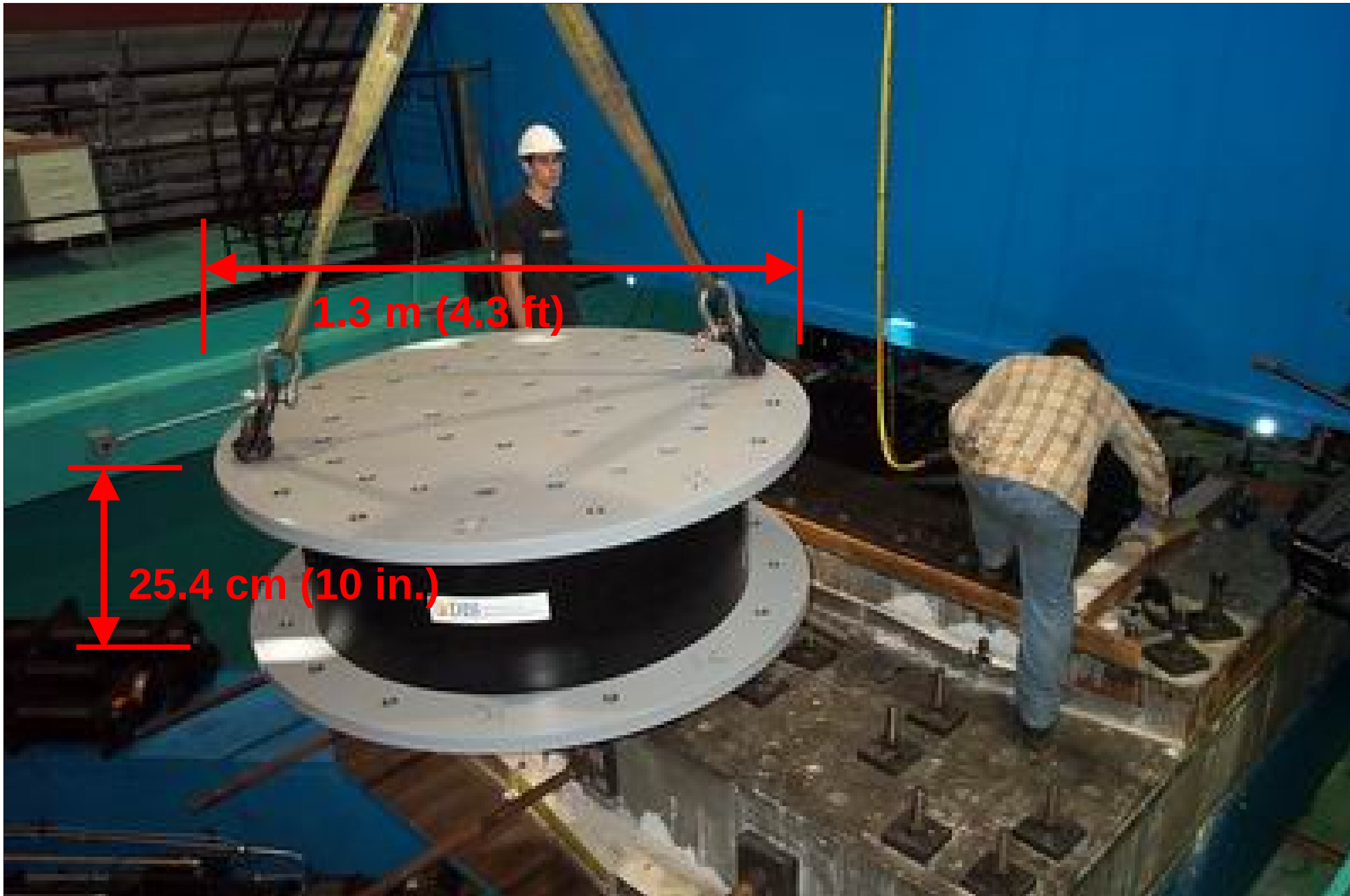


Shear Deformation of Elastomeric Bearing



- Bearing Manufactured by Scougal Rubber Corporation.
- Test Performed at SUNY Buffalo.
- Shear strain shown is approximately 100%.

Full-Scale Bearing Prior to Dynamic Testing



Cyclic Testing of Elastomeric Bearing

Bearing Manufactured by
Dynamic Isolation Systems Inc.

Testing of Full-Scale Elastomeric Bearing at UC San Diego
- Compressive load = 4000 kips
- 400% Shear Strain [1.0 m (40 in.) lateral displacement]
- Video shown at 16 x actual speed of 1.0 in/sec

Harmonic Behavior of Elastomeric Bearing

$$u(t) = u_0 \sin(\overline{\omega} t)$$

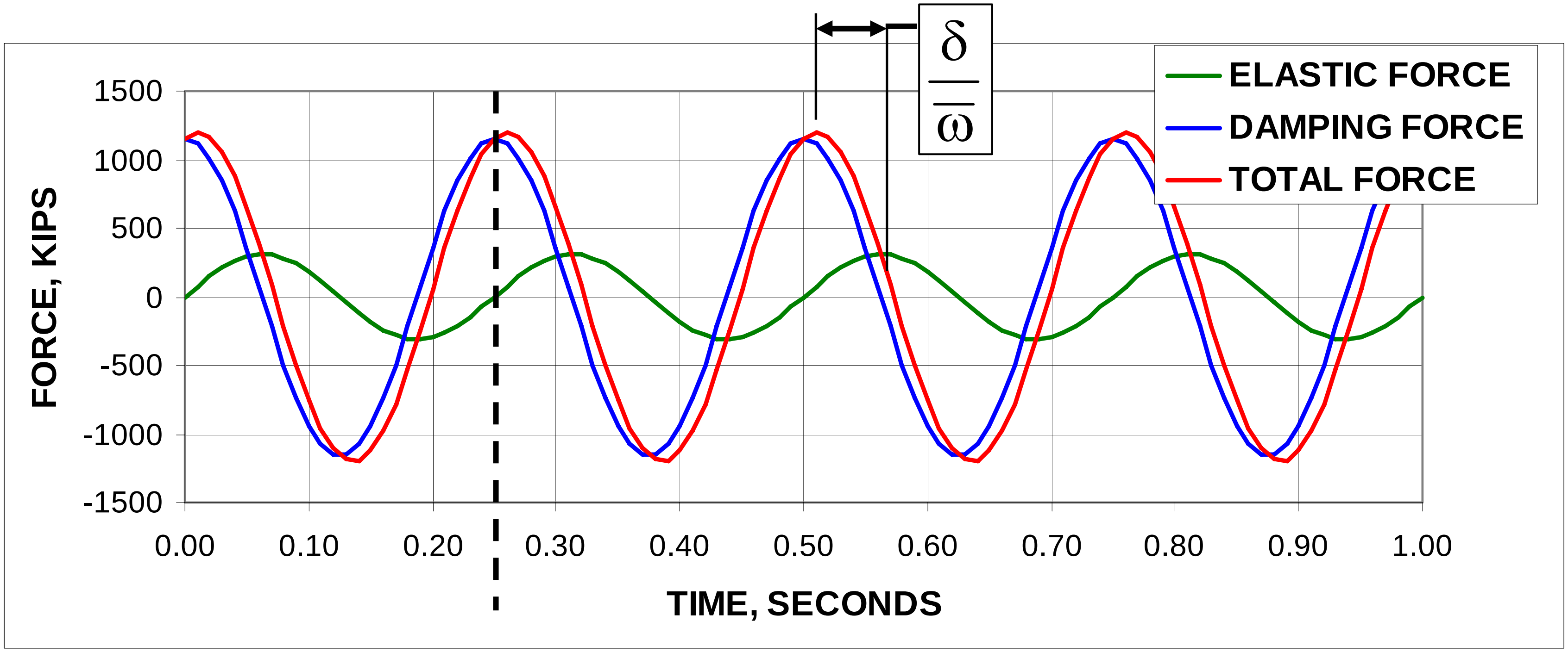
Imposed Motion

Loading Frequency

Phase Angle (Lag)

Assumed Form of Total Force

$$P(t) = P_0 \sin(\overline{\omega} t)\cos(\delta) + P_0 \cos(\overline{\omega} t)\sin(\delta)$$



Note: Damping force 90º out of phase with elastic force.

$$P(t) = K_S u(t) + C\dot{u}(t)$$

$$K_S = \frac{P_0}{u_0}\cos(\delta)$$

Storage Stiffness

$$K_L = \frac{P_0}{u_0}\sin(\delta)$$

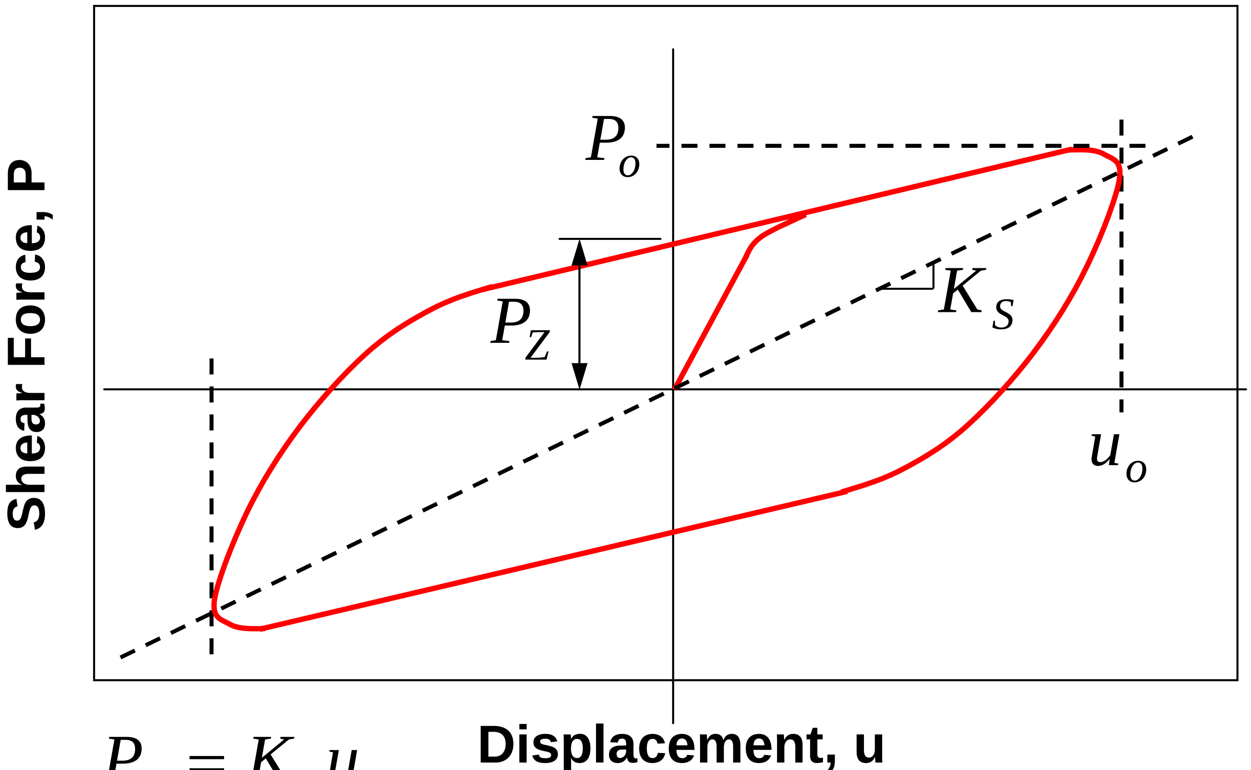
Loss Stiffness

$$C = \frac{K_L}{\overline{\omega}}$$

Damping Coeff.

$$\delta = \sin^{-1}\left(\frac{P_Z}{P_0}\right)$$

Phase Angle



$$\xi = \frac{1}{2}\tan(\delta)$$

$$P_Z = K_L u_o$$

$$P(t) = K_S u(t) + C\dot{u}(t)$$

$$K_S = \frac{G' A}{t_r}$$

Storage Stiffness

$$K_L = \frac{G'' A}{t_r}$$

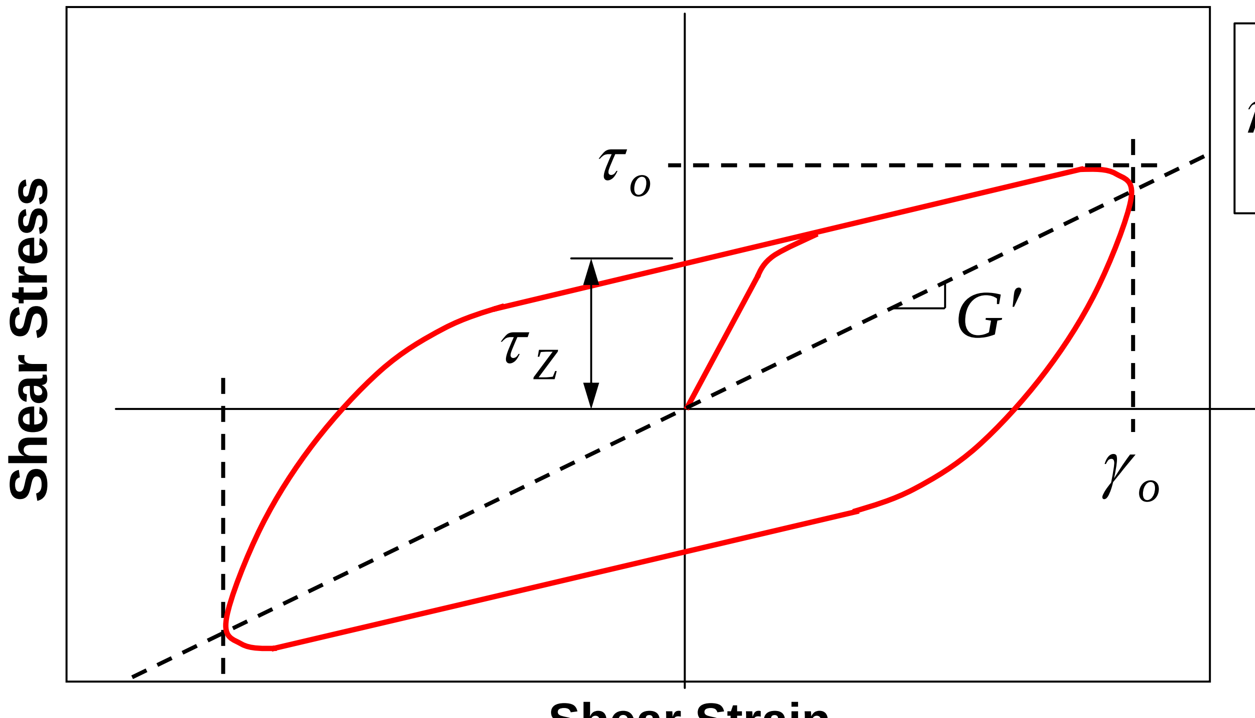
Loss Stiffness

$$C = \frac{K_L}{\overline{\omega}}$$

Damping Coeff.

$$\delta = \sin^{-1}\left(\frac{\tau_Z}{\tau_0}\right)$$

Phase Angle



$$\eta = \frac{G''(\overline{\omega})}{G'(\overline{\omega})} = tan(\delta)$$

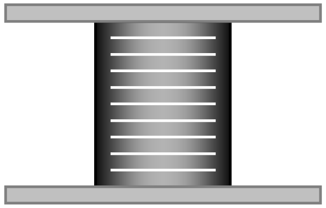
Loss Factor

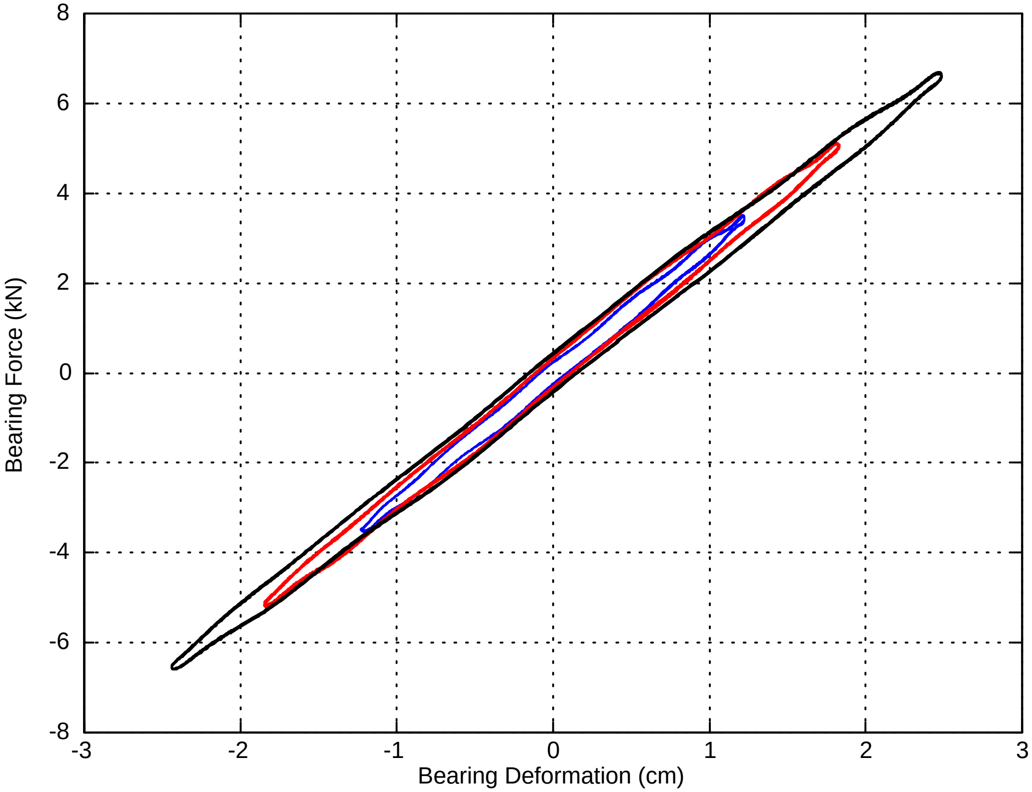
$$\tau(t) = G'\gamma(t) + G''\dot{\gamma}(t) / \overline{\omega}$$

$$\xi = \frac{\eta}{2} = \frac{1}{2} tan(\delta)$$

Damping Ratio

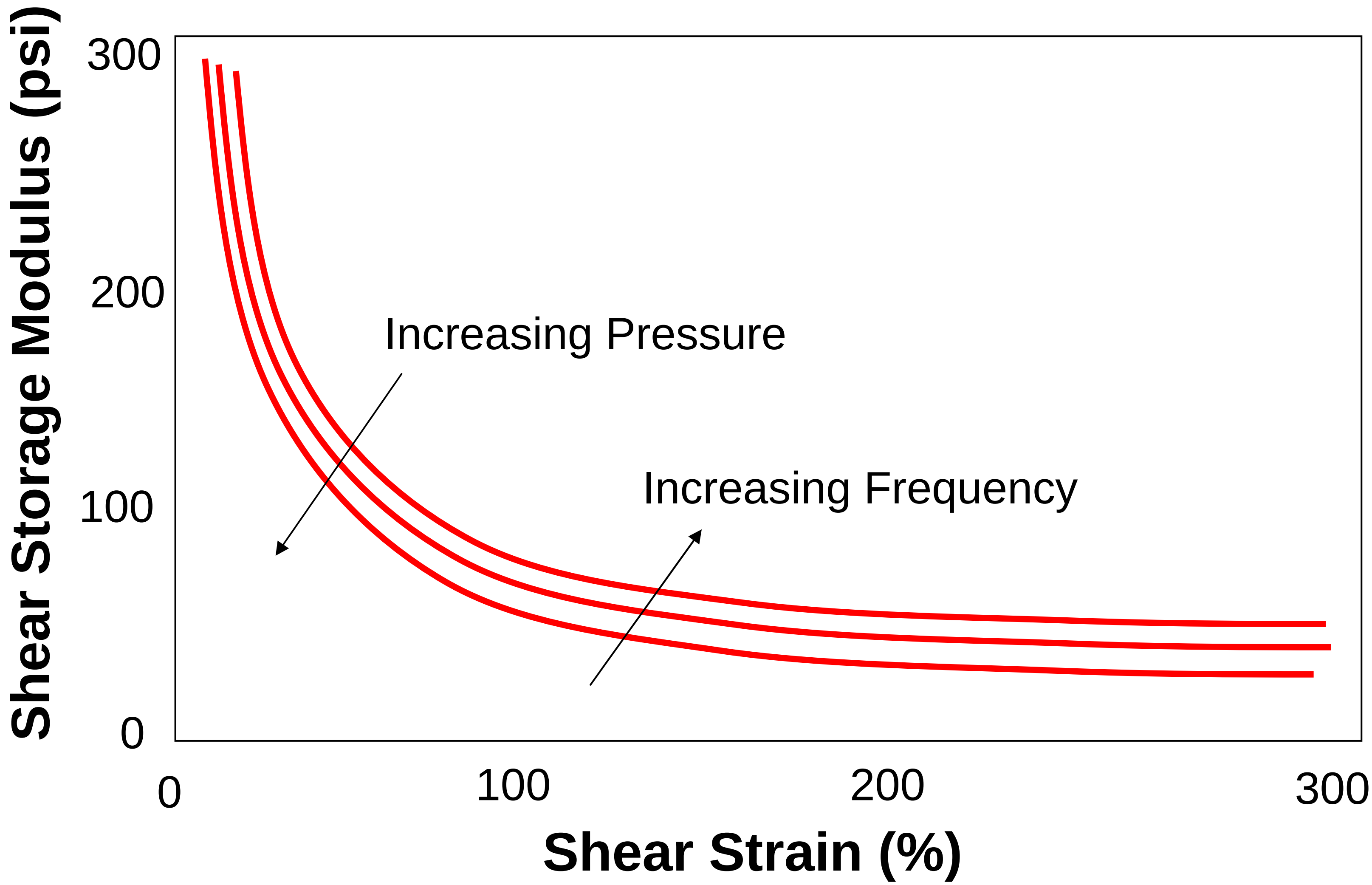
Experimental Hysteresis Loops of Low Damping Rubber Bearing



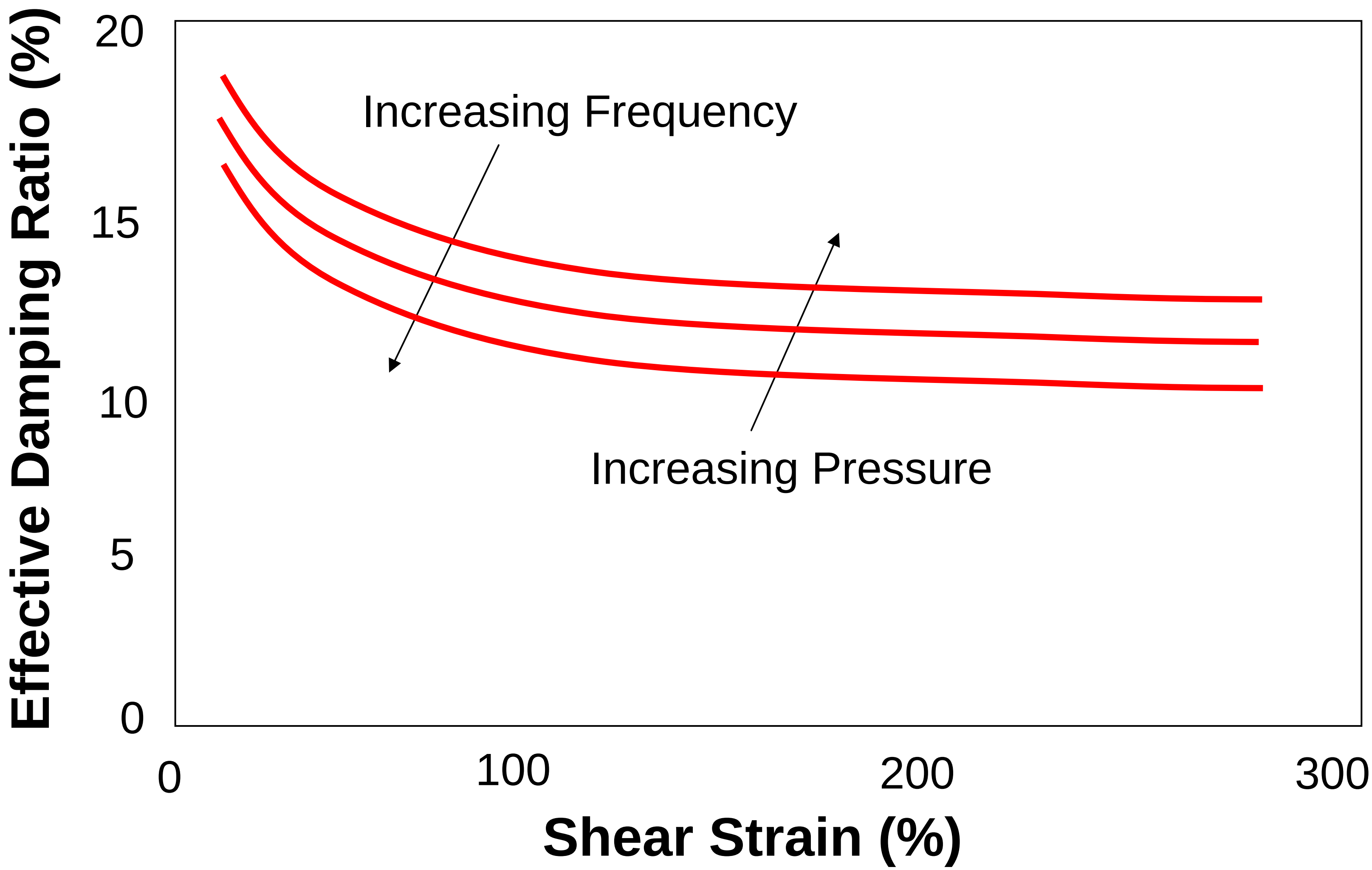


Low Damping Rubber Bearing
- Reduced scale bearing for ¼-scale building frame
- Diameter and height approx. 5 in.
- Prototype fundamental period of building = 1.6 sec

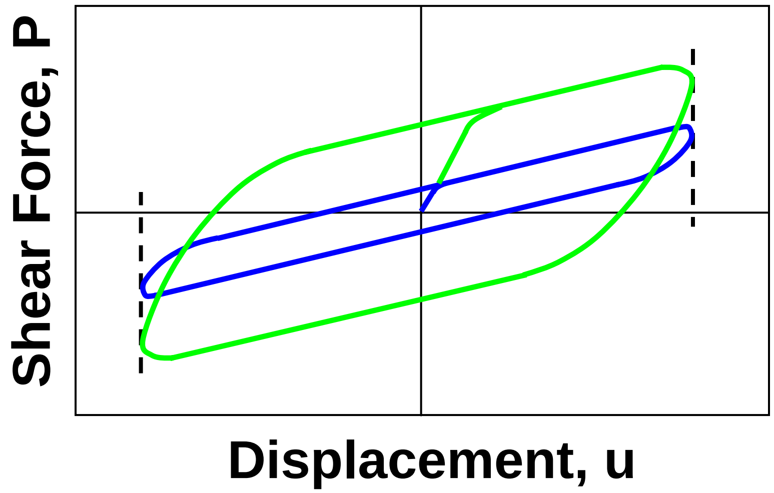
Shear Storage Modulus of High-Damping Natural Rubber



Effective Damping Ratio of High-Damping Natural Rubber

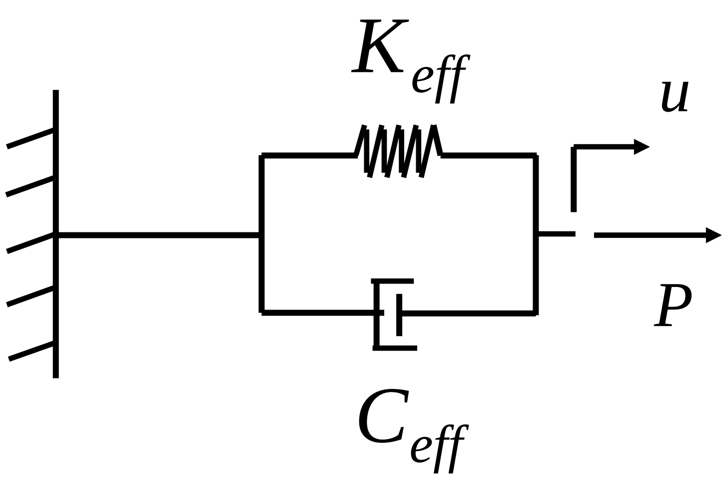


Linear Mathematical Model for Natural and Synthetic Rubber Bearings



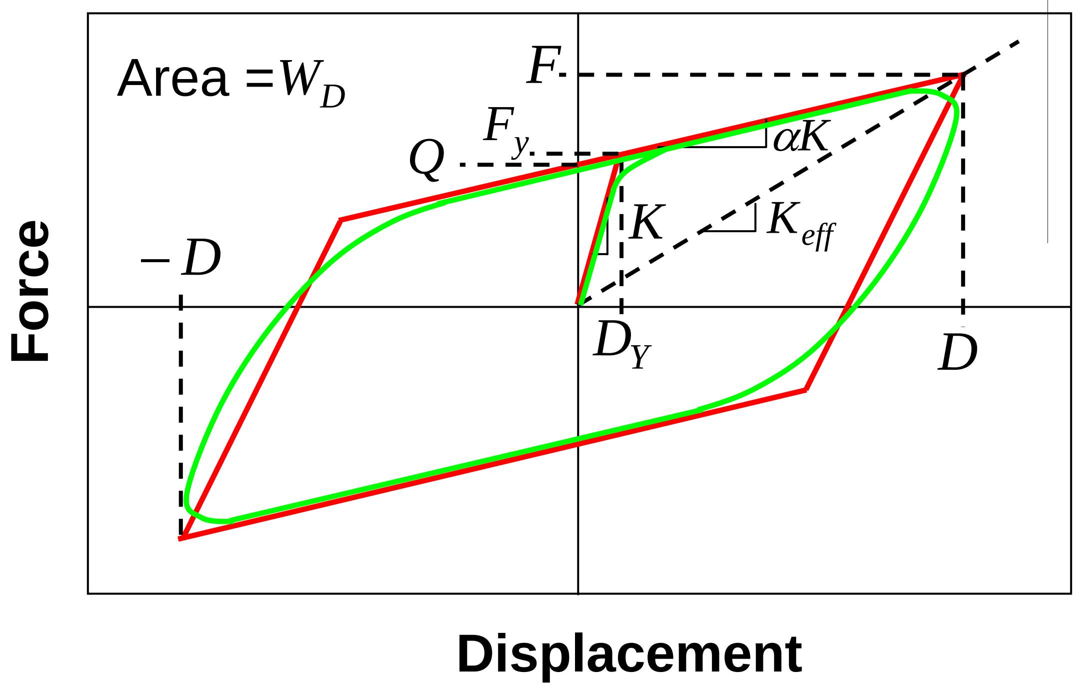
k_{eff} = Effective stiffness at design displacement

c_{eff} = Effective damping coefficient associated with design displacement



$$P(t) = k_{eff}u(t) + c_{eff}\dot{u}(t)$$

Equivalent Linear Properties from Idealized Bilinear Hysteresis Loop



$$k_{eff} = \frac{F}{D}$$

$$\boxed{k_{eff} = \frac{F}{D} = \alpha K + \frac{Q}{D}}$$

$$\xi_{eff} = \frac{W_D}{4\pi W_S}$$

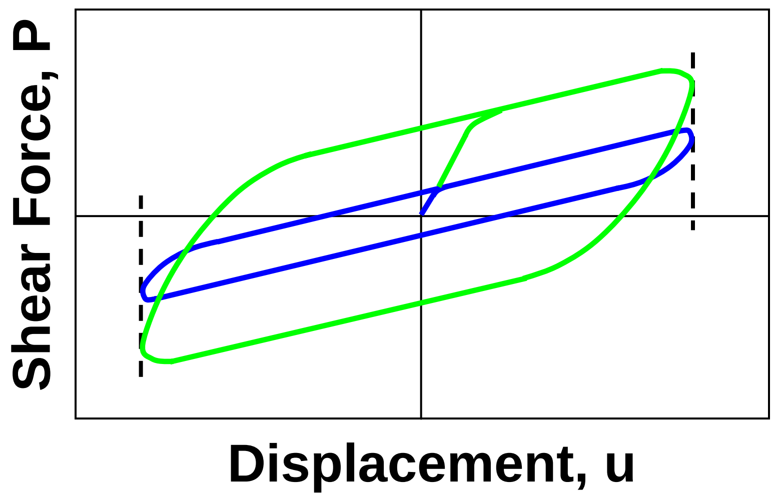
$$W_S = \frac{1}{2} K_{eff} D^2$$

$$W_D = 4Q(D - D_Y)$$

If $Q >> D_Y$, then: $W_D \approx 4QD$

$$\boxed{\xi_{eff} = \frac{2Q(D - D_Y)}{\pi D(Q + \alpha KD)}}$$

Refined Nonlinear Mathematical Model for Natural and Synthetic Rubber Bearings



α = Post-to-pre yielding stiffness ratio

P_y = Yield force

u_y = Yield displacement

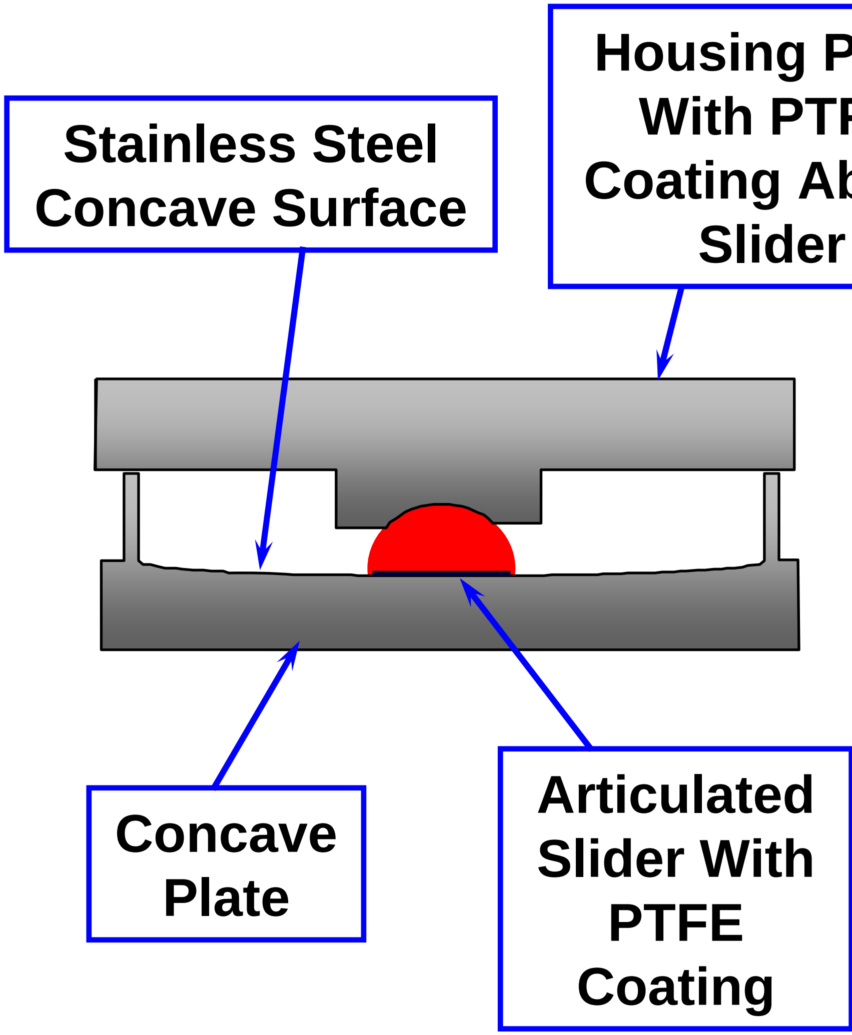
Z = Evolutionary variable

$\gamma, \beta, \eta, \theta$ = Calibration constants

$$P(t) = \alpha \frac{P_y}{u_y} u(t) + (1 - \alpha) P_y Z(t)$$ *Shear Force in Bearing*

$$u_y \dot{Z} + \gamma |\dot{u}| Z |Z|^{\eta - 1} + \beta \dot{u} |Z|^{\eta} - \theta \dot{u} = 0$$ *Evolutionary Equation*

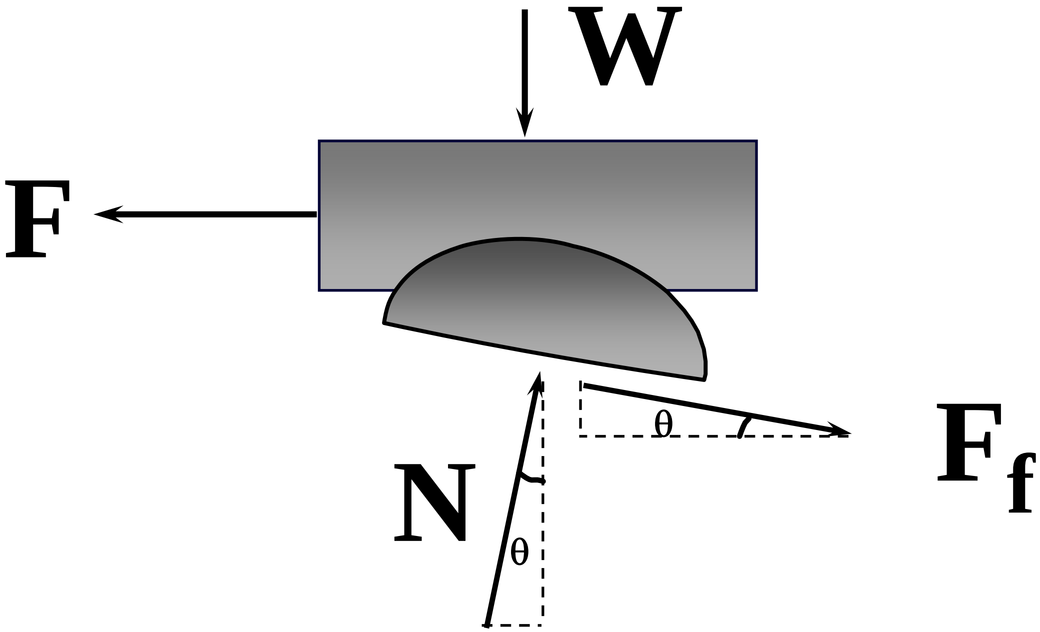
Spherical Sliding Bearing:
Friction Pendulum System (FPS)



Concave Plate and Slider for FPS Bridge Bearing

- **Seismic retrofit of Benicia-Martinez Bridge, San Francisco, CA**
- **7.5 to 13 ft diameters**
- **Displ. Capacity of 13 ft bearings = +/- 4.3 ft**

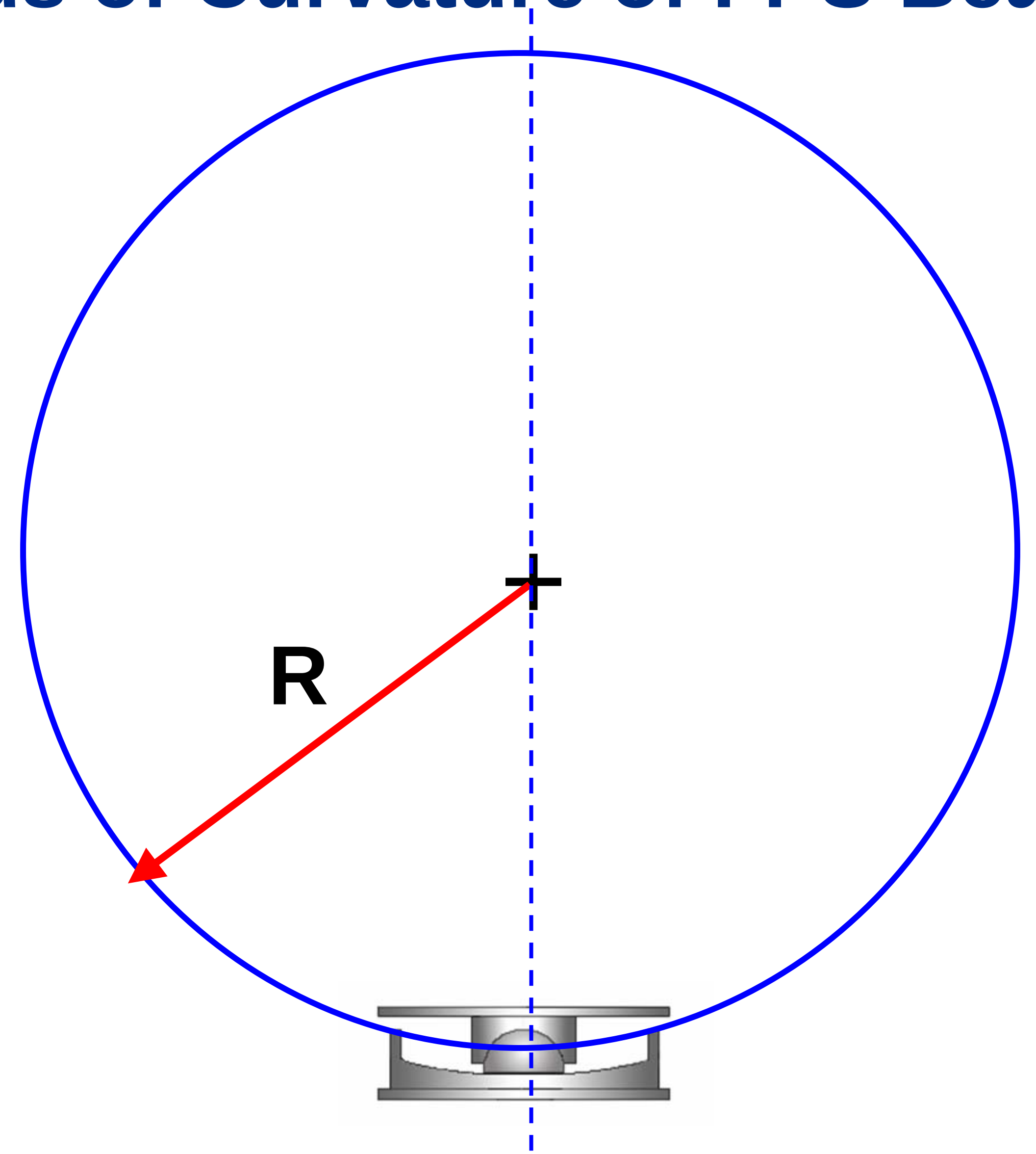
Mathematical Model of Friction Pendulum System Bearings



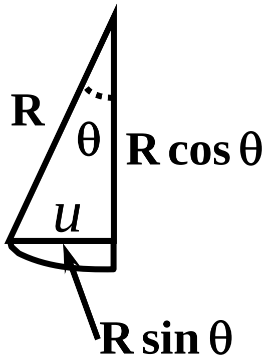
Free-Body Diagram of Top Plate and Slider Under Imposed Lateral Force F

$$F = W \tan\theta + \frac{F_f}{\cos\theta}$$

Radius of Curvature of FPS Bearings



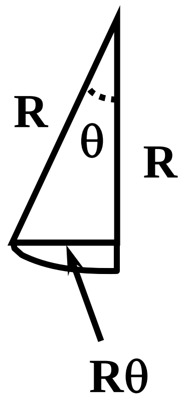
Mathematical Model of Friction Pendulum System Bearings



For u < 0.2R, θ is small (2% error in u)

$$\sin\theta = \theta - \frac{\theta^3}{3!} + \ldots \approx \theta$$

$$\cos\theta = 1 - \frac{\theta^2}{2!} + \ldots \approx 1$$

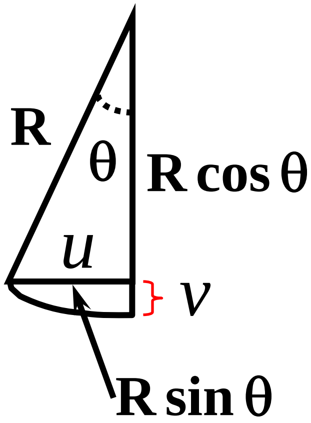


$$\theta \approx \frac{u}{R} \qquad N = \frac{W}{\cos\theta} \approx W$$

$$F_f = \mu N \, sgn(\dot{u})$$

$$\boxed{F = W\tan\theta + \frac{F_f}{\cos\theta}} \longrightarrow \boxed{F = \frac{W}{R}u + \mu W \, sgn(\dot{u})}$$

Vertical Displacement of FPS Bearings

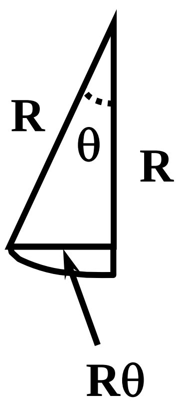


$$v = R(1 - \cos\theta) = R\left[1 - \cos\left(\sin^{-1}\left(\frac{u}{R}\right)\right)\right]$$

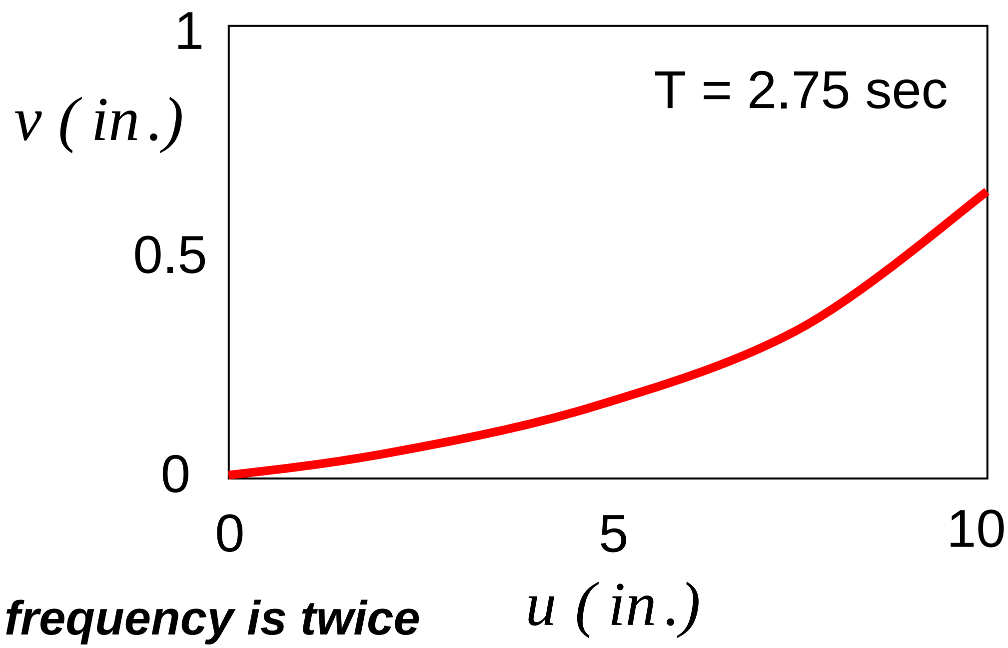
$$v \approx \frac{R\theta^2}{2} \approx \frac{u^2}{2R}$$

$$\sin\theta = \theta - \frac{\theta^3}{3!} + \ldots \approx \theta$$

$$\cos\theta = 1 - \frac{\theta^2}{2!} + \ldots \approx 1$$



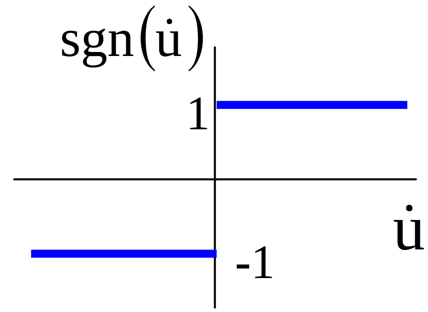
$$\theta \approx \frac{u}{R}$$

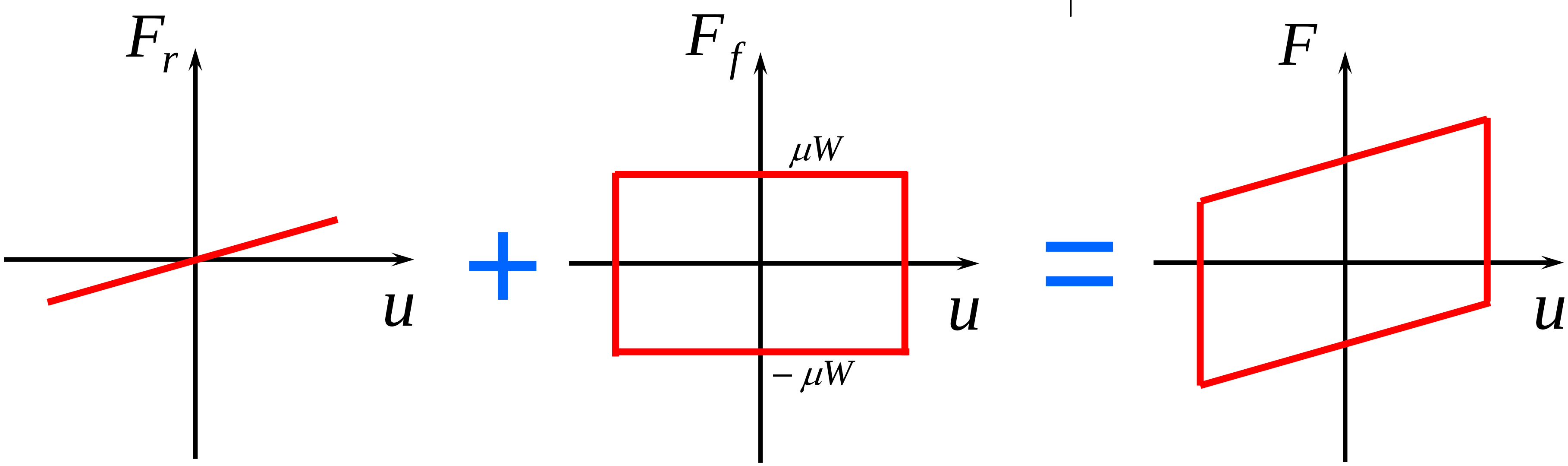


Note: Vertical frequency is twice that of lateral frequency

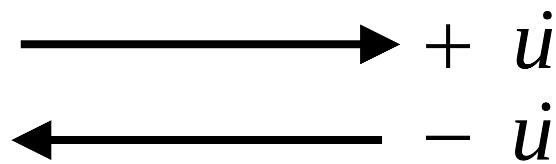
Components of FPS Bearing Lateral Force

$$\boxed{F = \frac{W}{R}u + \mu W\ sgn(\dot{u}) = F_r + F_f}$$





Slope = $\dfrac{W}{R}$

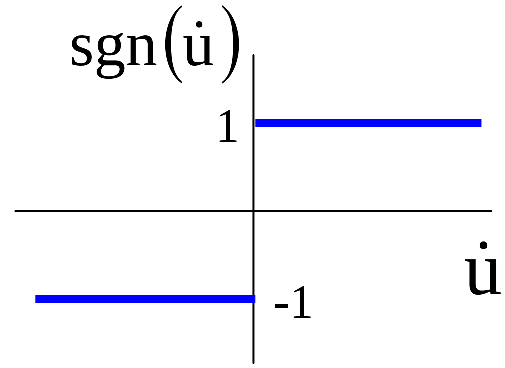


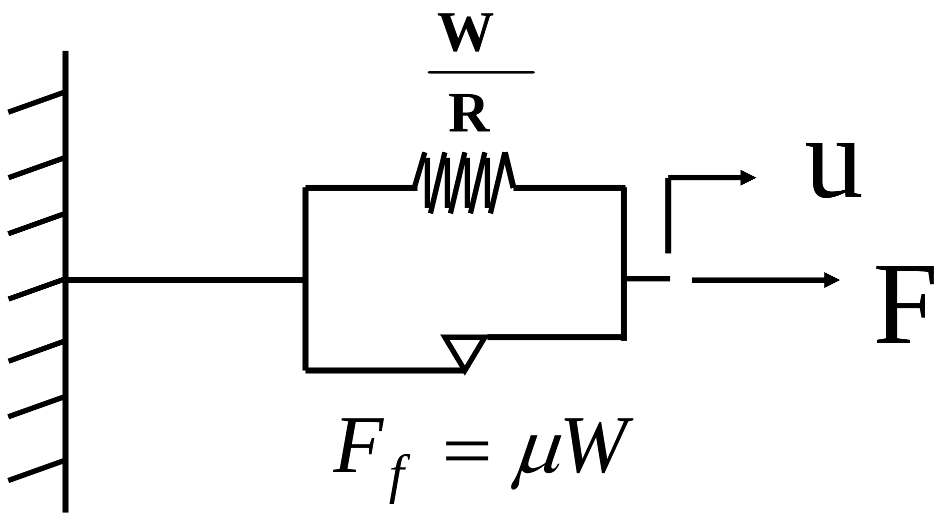
Note: Bearing will not recenter if $F_r < F_f\ (u < \mu R)$

For large T, and thus large R, this can be a concern.

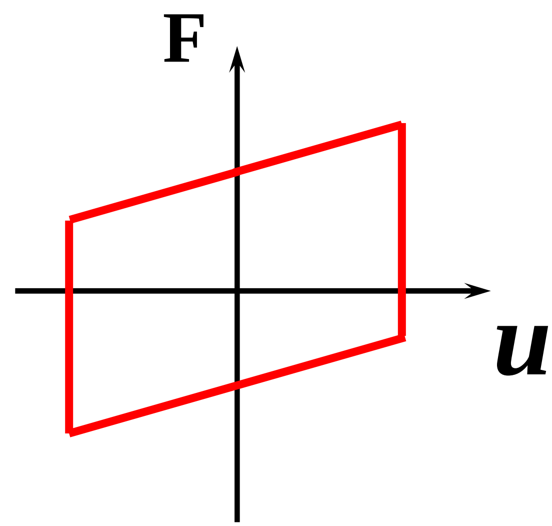
Mechanical Model of Friction Pendulum System Bearings

$$F = \frac{W}{R}u + \mu W\, sgn(\dot{u})$$





Rigid Model with Strain Hardening

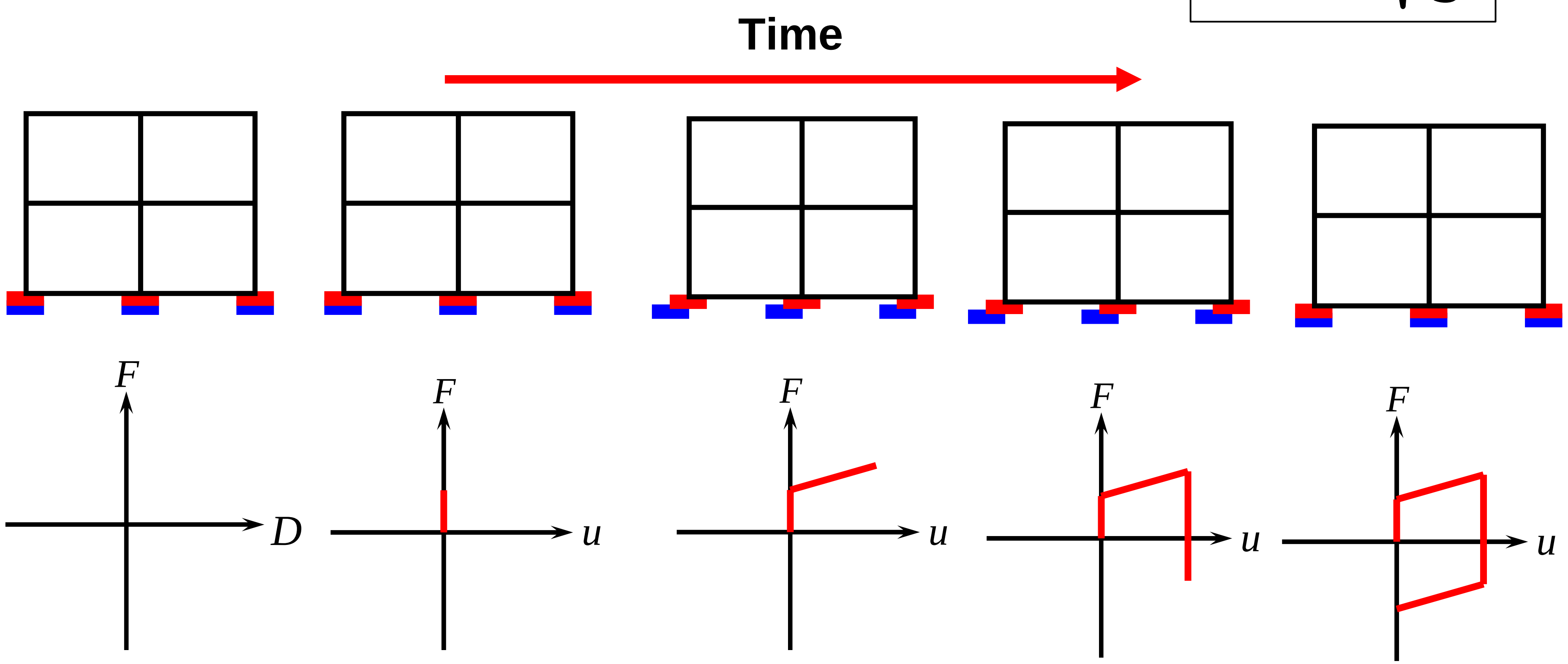


Hysteretic Behavior of Friction Pendulum System Bearings

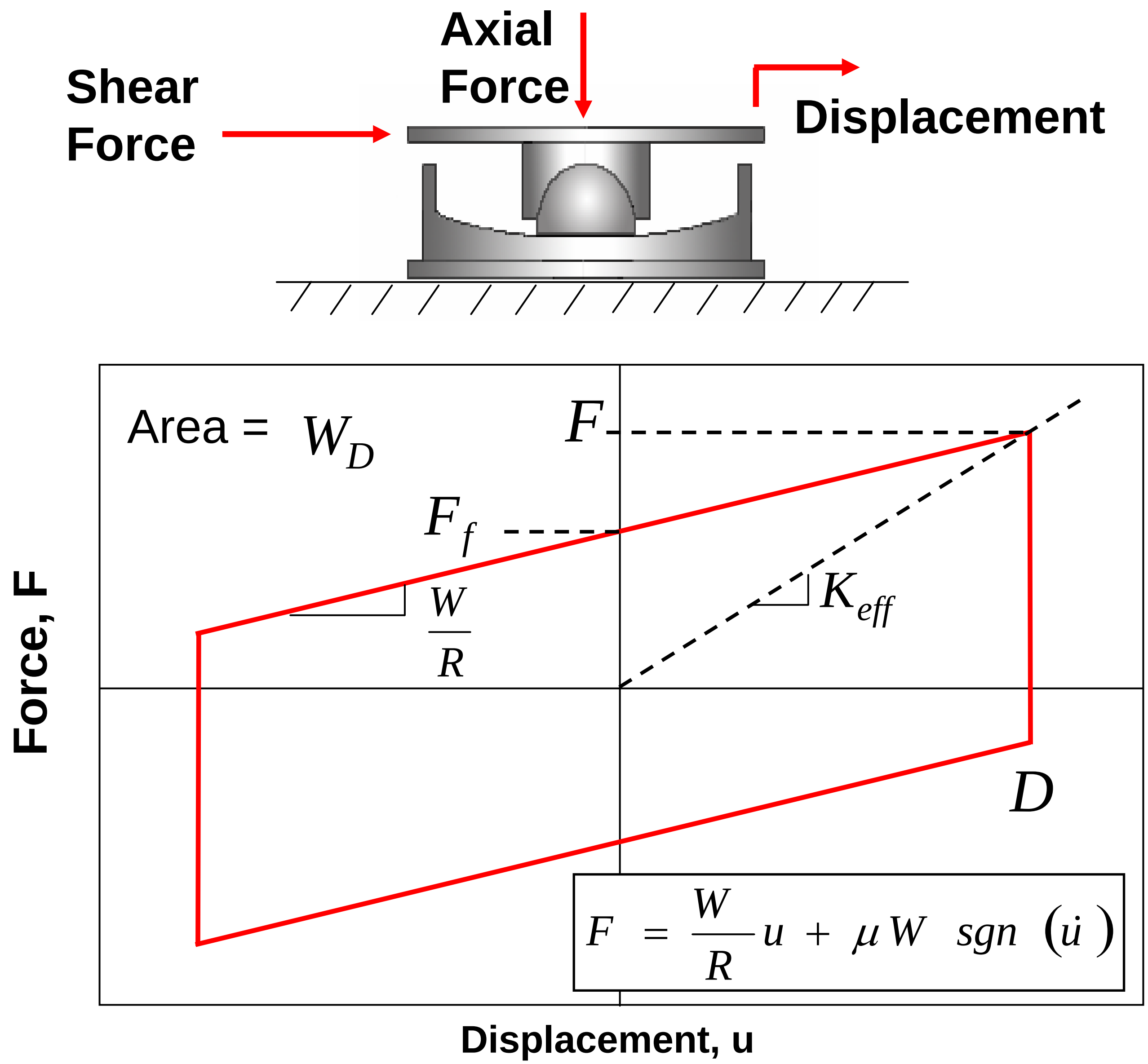
$$F = \frac{W}{R}u + \mu W \, sgn(\dot{u})$$

Free Vibration Period

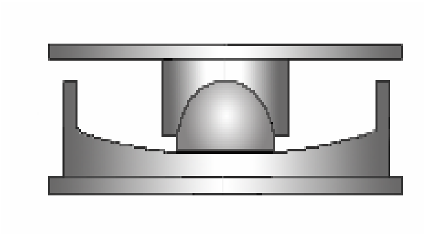
$$T = 2\pi\sqrt{\frac{R}{g}}$$

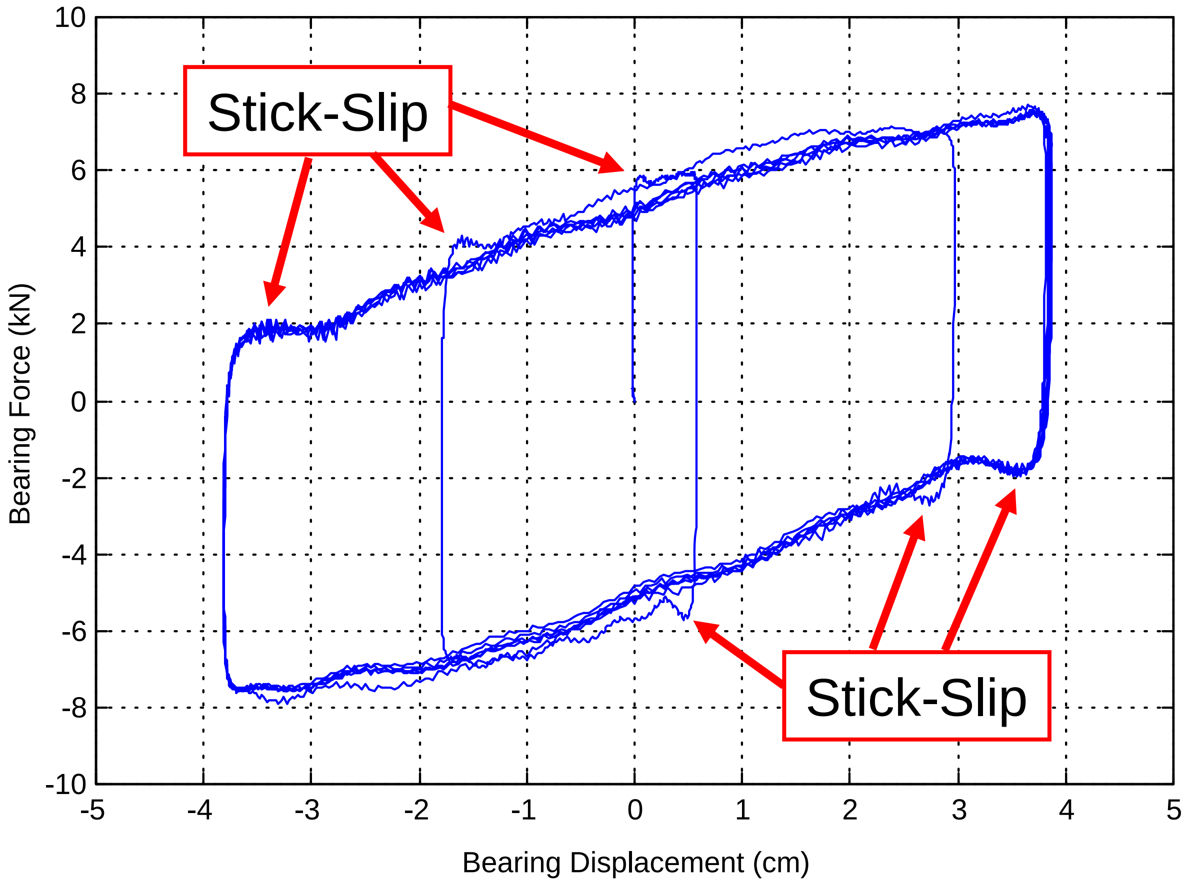


Idealized FPS Bearing Hysteresis Loop



Actual FPS Bearing Hysteresis Loop



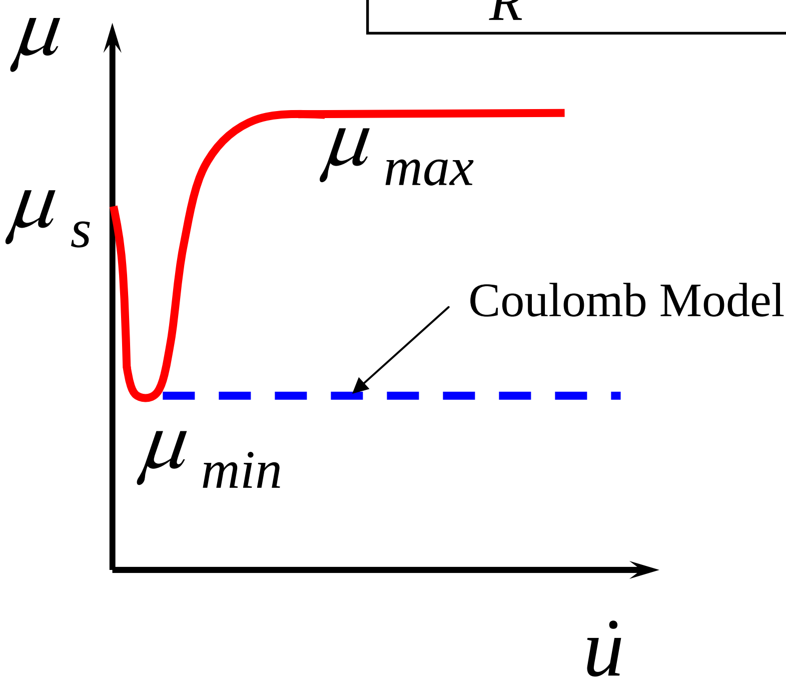


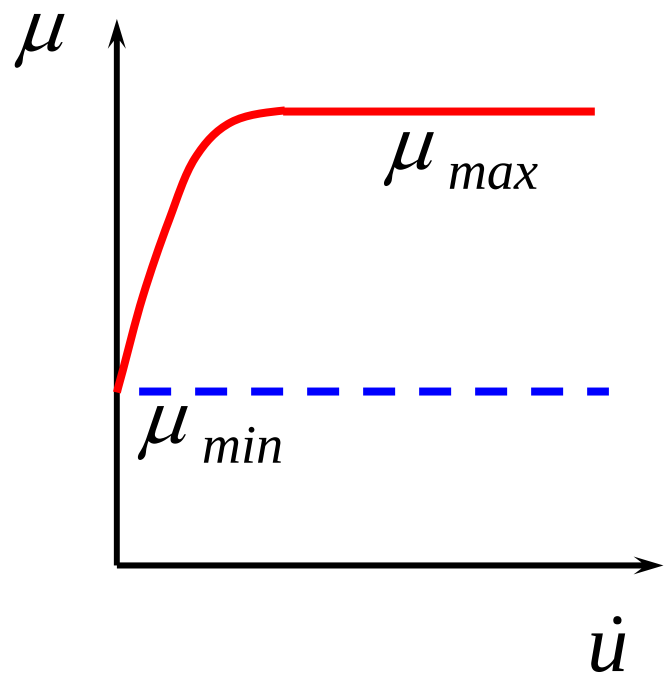
FPS Bearing

- Reduced-scale bearing for ¼-scale building frame
- R = 18.6 in; D = 11 in.; H = 2.5 in. (reduced scale)
- Prototype fundamental period of building = 2.75 sec (R = 74.4 in. = 6.2 ft)

Velocity-Dependence of Coefficient of Friction

$$\boxed{F = \frac{W}{R}u + \mu W \, sgn(\dot{u})}$$





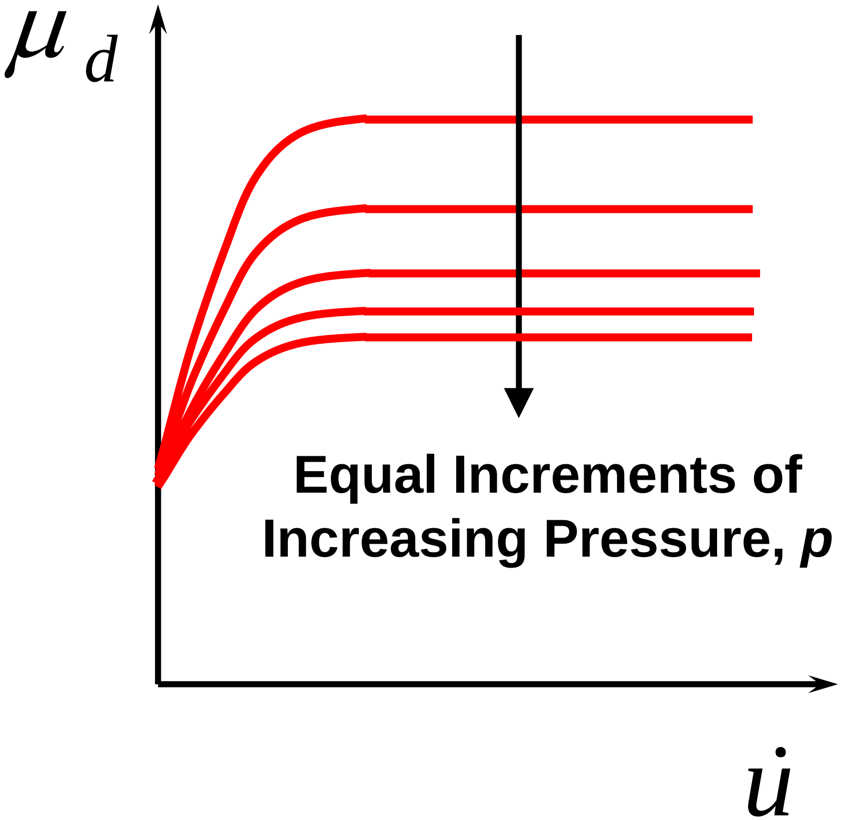
Actual Velocity-Dependence

Approximate Velocity-Dependence

$$\mu = \mu_{max} - (\mu_{max} - \mu_{min}) exp(-a|\dot{u}|)$$

- Shear strength of PTFE depends on rate of loading.

Pressure-Dependence of Coefficient of Friction



$$p = \frac{W}{A}\left(1 + \underbrace{\frac{\ddot{u}_v}{g} + \frac{P_s}{W}}_{\text{Typically Neglected}}\right)$$

Pressure- and Velocity-Dependence

Pressure-Dependence of Coefficient of Friction

$$\boxed{\mu = \mu_{max} - (\mu_{max} - \mu_{min})\exp(-a|\dot{u}|)}$$

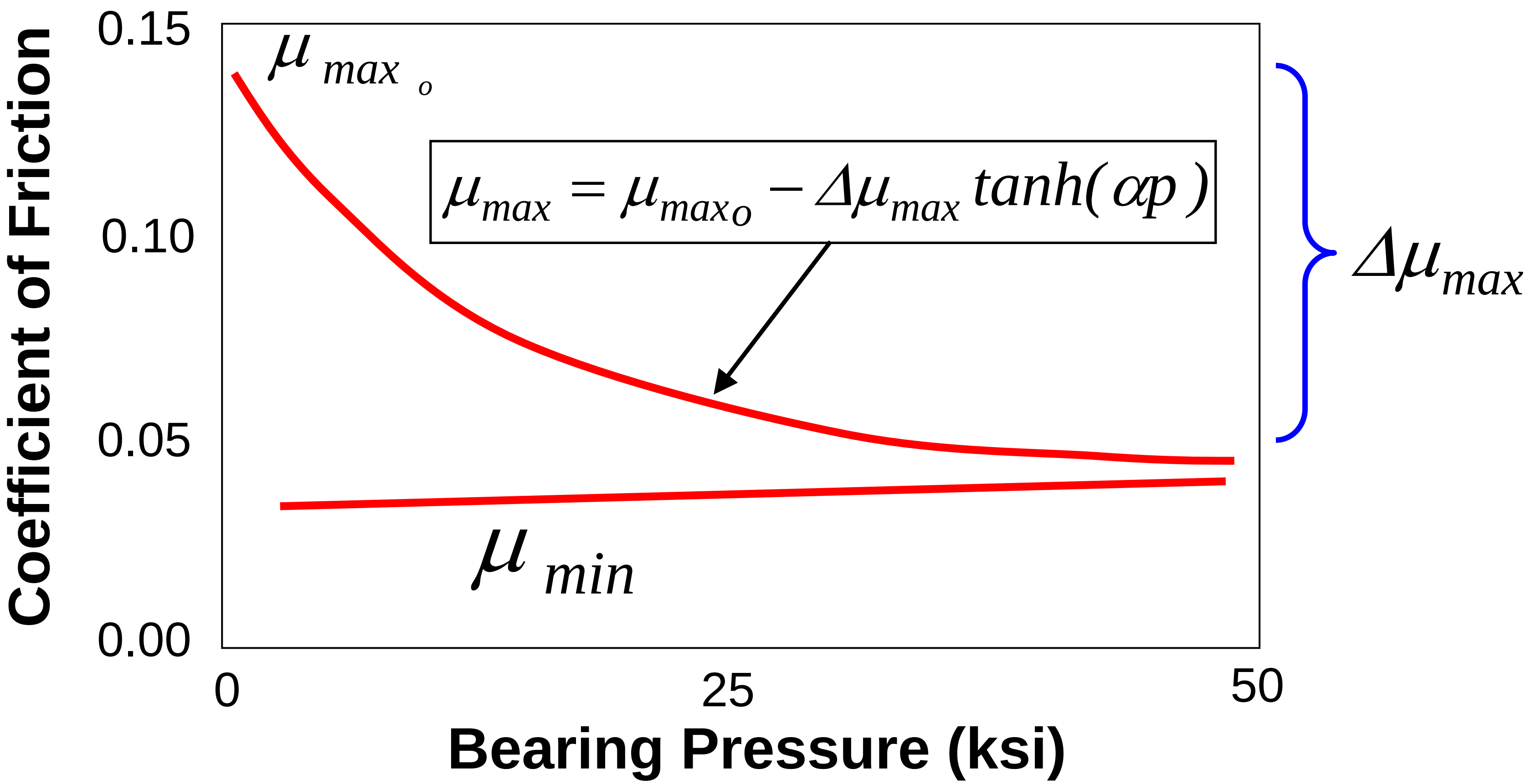
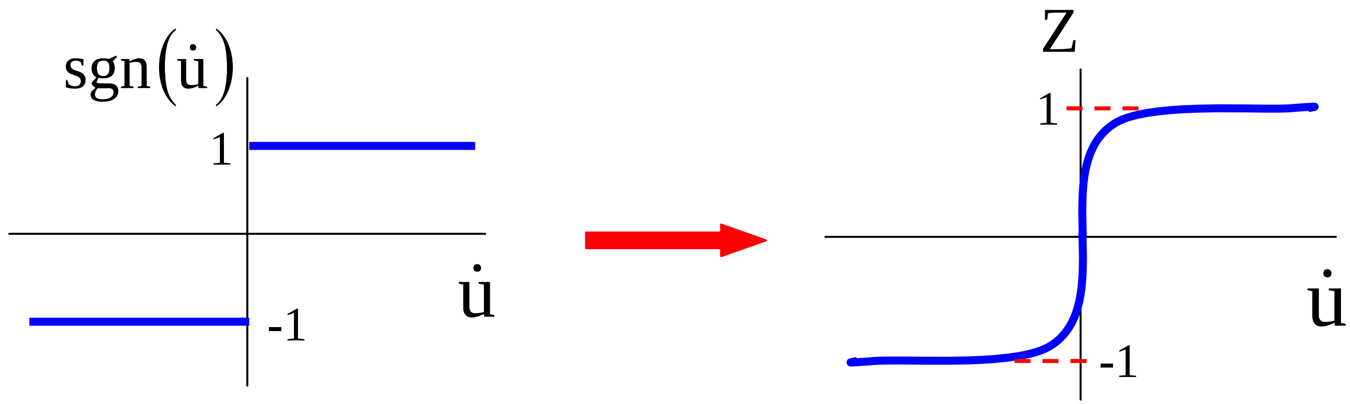


Figure is based on studies of PTFE-based self-lubricating composites used in FPS bearings.

Refined Model of FPS Bearing Behavior



Viscoplasticity Model

$$Y\dot{Z} + \alpha\,|\dot{u}|\,|Z|\,|Z|^{\eta-1} + \beta\dot{u}\,|Z|^{\eta} - \gamma\dot{u} = 0$$

Evolutionary Equation

Coefficient of Friction

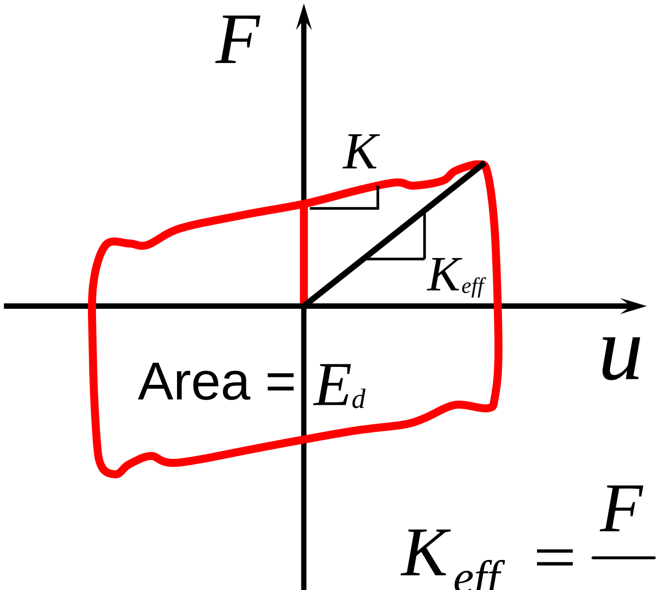
$$\mu = \mu_{max} - \left(\mu_{max} - \mu_{min}\right)exp\left(-a|\dot{u}|\right)$$

$$\boxed{F(t) = \frac{W}{R}u(t) + \mu W\, sgn(\dot{u})} \longrightarrow \boxed{F(t) = \frac{W}{R}u(t) + \mu W Z(t)}$$

Evaluation of Dynamic Behavior of Base-Isolated Structures

- Isolation Systems are Almost Always Nonlinear and Often Strongly Nonlinear
- Equivalent Linear Static Analysis Using Effective Bearing Properties is Commonly Utilized for Preliminary Design
- Final Design Should be Performed Using Nonlinear Dynamic Response History Analysis

Equivalent Linear Properties of FPS Isolation Bearings



$$F(t) = \frac{W}{R} u(t) + \mu W \, sgn(\dot{u})$$

$$K_{eff} = \frac{F}{u} = \frac{W}{R} + \frac{\mu W}{u}$$

Effective (Secant) Stiffness at Displacement u

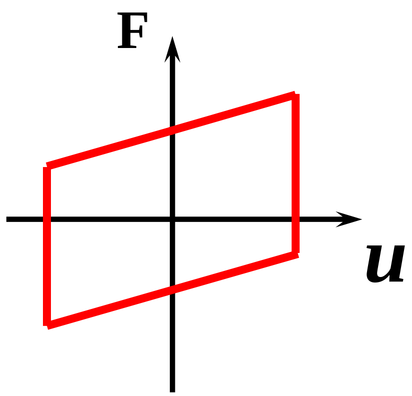
$$\xi_{eff} = \frac{E_d}{4\pi E_s} = \frac{4\mu W u}{4\pi\left(0.5 K_{eff} u^2\right)} = \frac{2\mu R}{\pi\left(\mu R + u\right)}$$

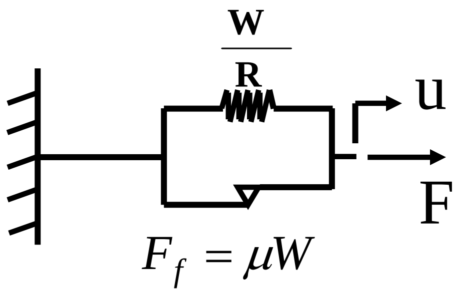
Effective Damping Ratio at Displacement u

Effective linear properties are displacement-dependent. Therefore, design using effective linear properties is an iterative process.

Seismic Analysis using Nonlinear and Equivalent Linear Models

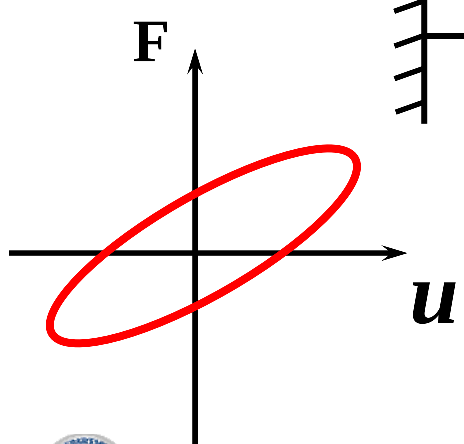
Nonlinear Model

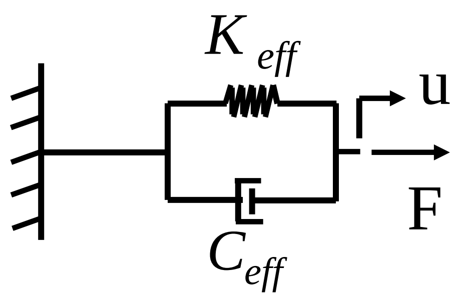




$$\boxed{F(t) = \frac{W}{R}u(t) + \mu W \, sgn(\dot{u})}$$

Linear Model





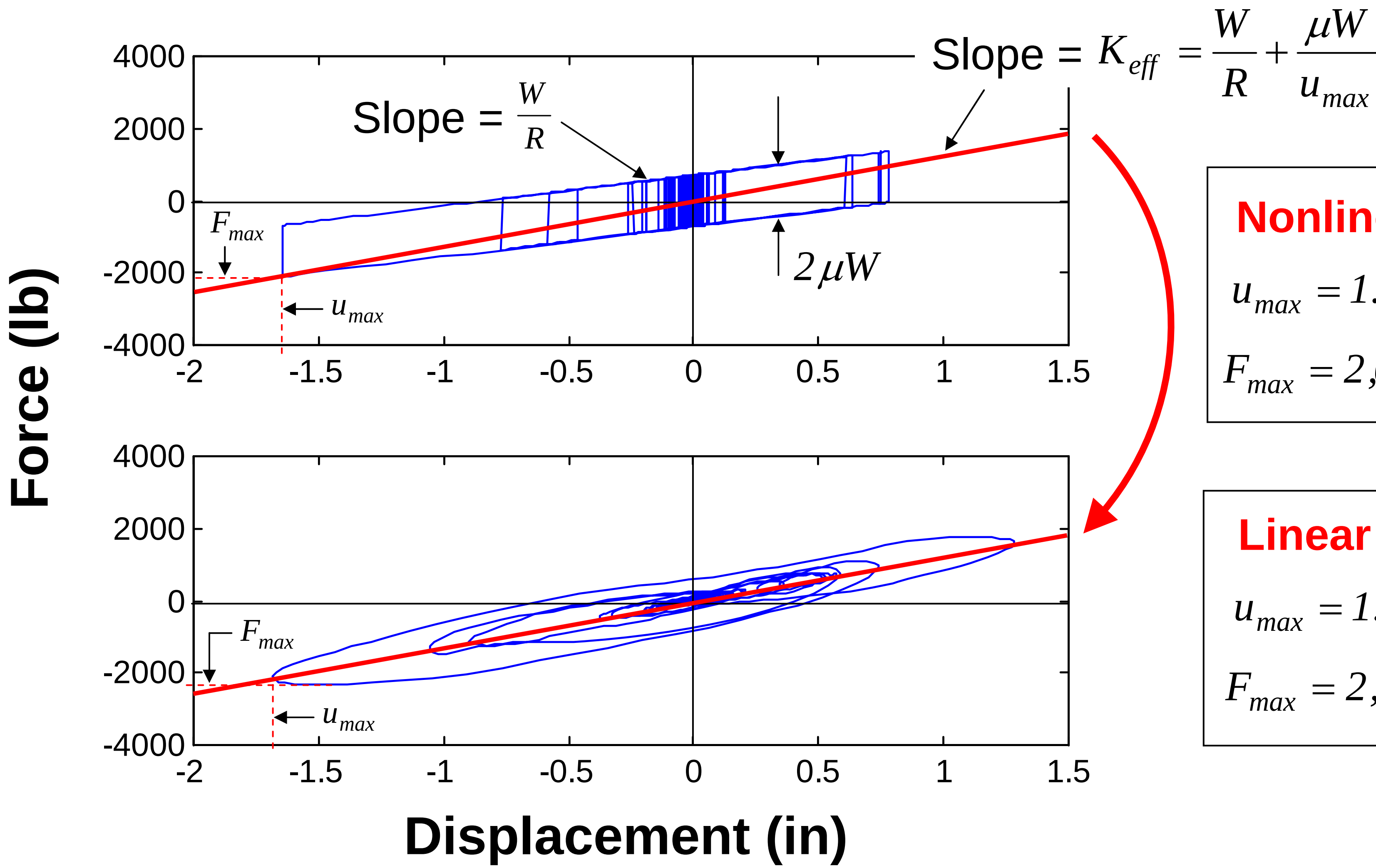
$$\boxed{F(t) = K_{eff}\,u(t) + C_{eff}\,\dot{u}(t)}$$

$$K_{eff} = \frac{W}{R} + \frac{\mu W}{u}$$

$$\xi_{eff} = \frac{2\mu R}{\pi(\mu R + u)}$$

$$C_{eff} = 2m\omega_{n_{eff}}\xi_{eff}$$

Example: Seismic Response Using Nonlinear and Linear Models



Nonlinear

$u_{max} = 1.65$ in.

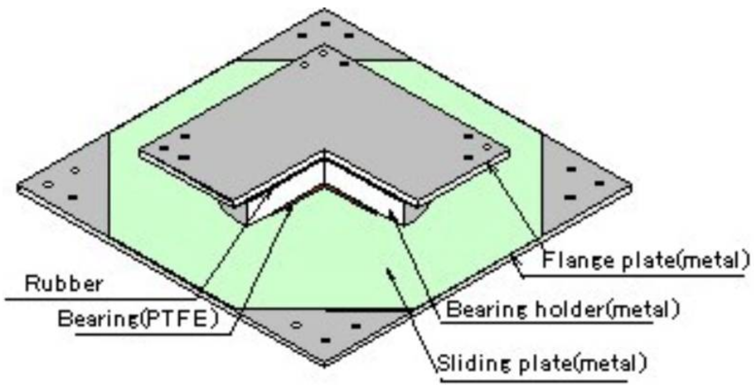
$F_{max} = 2{,}069$ lb

Linear

$u_{max} = 1.68$ in.

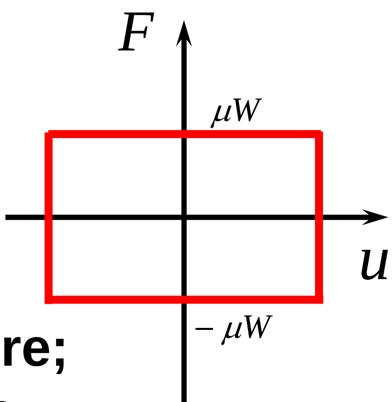
$F_{max} = 2{,}261$ lb

Flat Sliding Bearings



For Spherical Bearings:

$$F(t) = \frac{W}{R} u(t) + \mu W \, sgn(\dot{u})$$



- **Flat Bearings:** $R \rightarrow \infty \quad \therefore \quad F(t) = \mu W \, sgn(\dot{u})$
- **Bearings do NOT increase natural period of structure; Rather they limit the shear force transferred into the superstructure**
- **Requires supplemental self-centering mechanism to prevent permanent isolation system displacement**
- **Not commonly used in building structures**

Examples of Computer Software for Analysis of Base-Isolated Structures

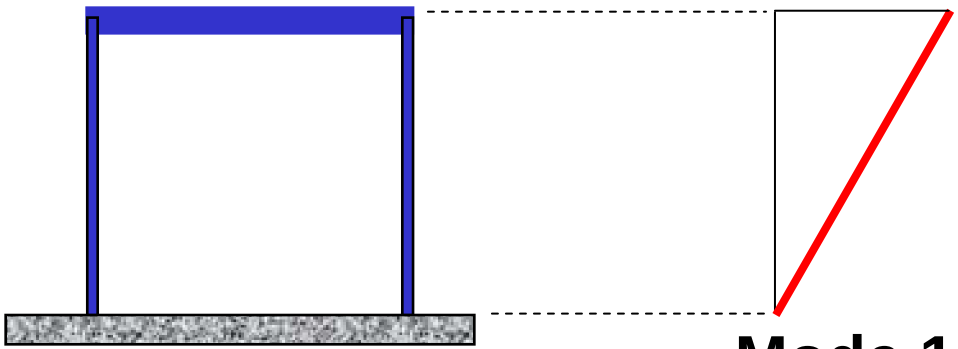
- **ETABS**
  Linear and nonlinear analysis of buildings

- **SAP2000**
  General purpose linear and nonlinear analysis

- **DRAIN-2D**
  Two-dimensional nonlinear analysis

- **3D-BASIS**
  Analysis of base-isolated buildings

Simplified Evaluation of Dynamic Behavior of Base-Isolated Structures



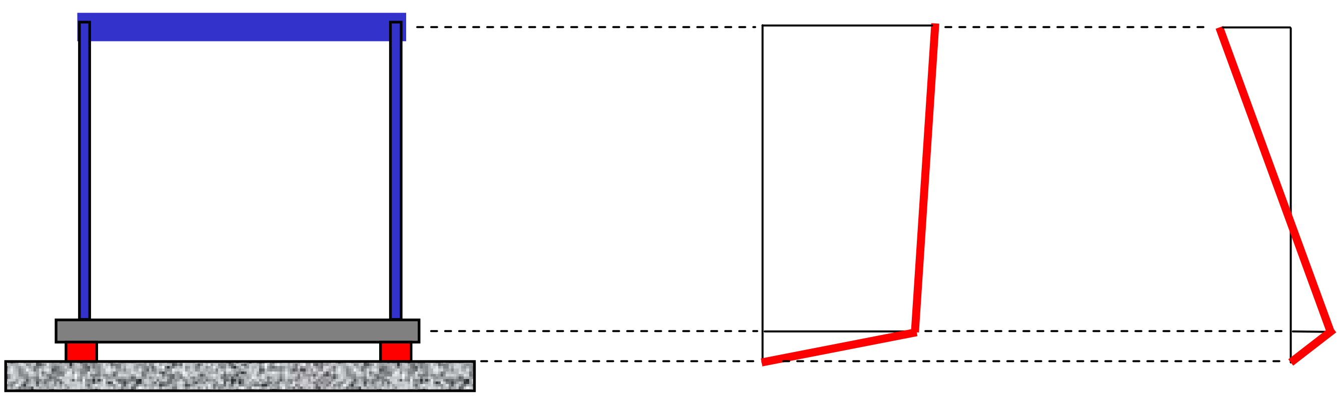
Fixed-Base

Mode 1
($T = T_f$)

Eigenproblem Analysis Results:

$T_{I1} >> T_f$

$T_{I1} >> T_{I2}$



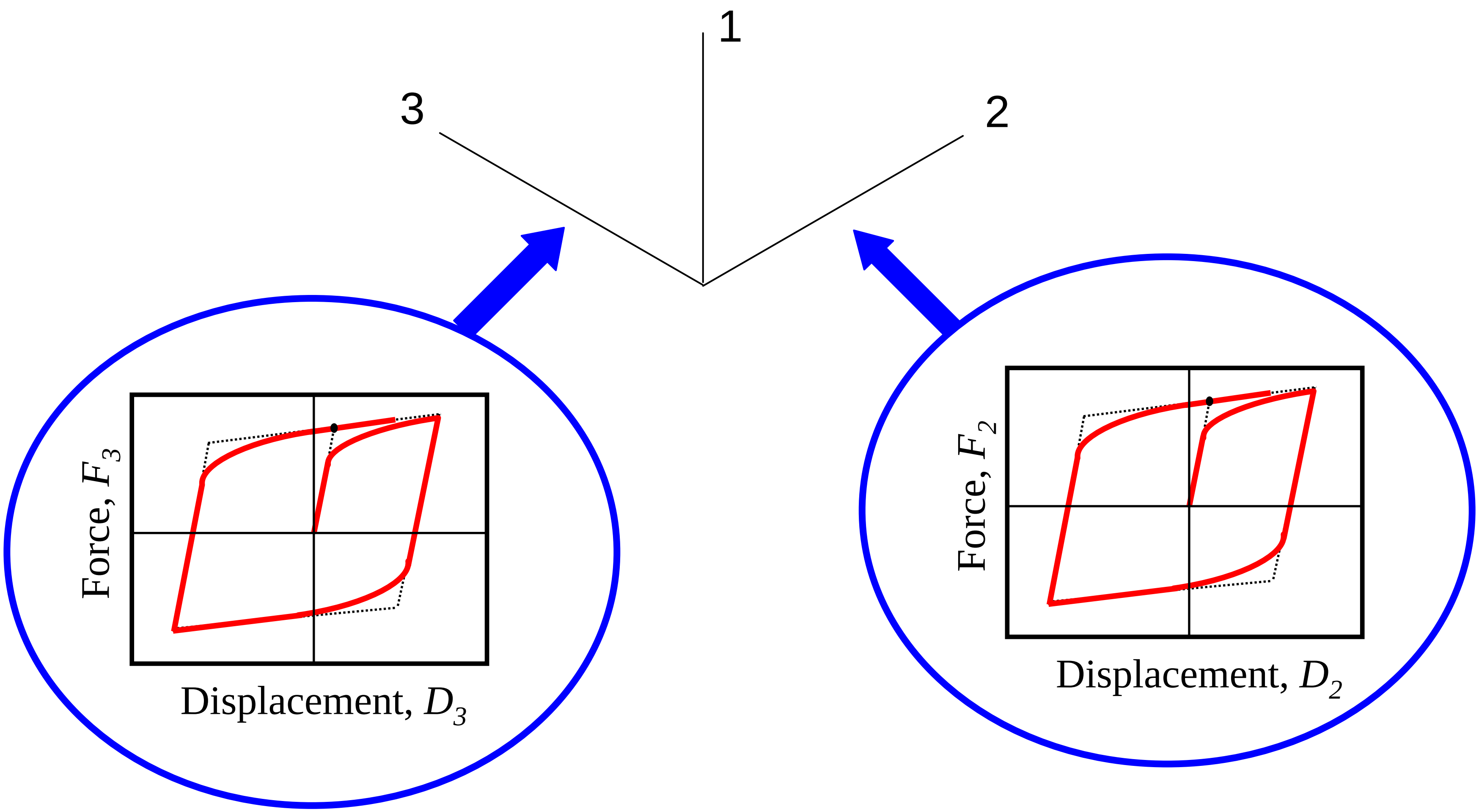
Base-Isolated

Mode 1
($T = T_{I1}$)

Mode 2
($T = T_{I2}$)

Modeling Isolation Bearings Using the SAP2000 NLLINK Element

ISOLATOR1 Property – Biaxial Hysteretic Isolator



Coupled Plasticity Equations

$$\left.\begin{aligned} F_2 &= \beta_2 k_2 D_2 + (1-\beta_2) F_{y2} Z_2 \\ F_3 &= \beta_3 k_3 D_3 + (1-\beta_3) F_{y3} Z_3 \end{aligned}\right\}$$

Shear Force Along Each Orthogonal Direction

$$\begin{Bmatrix} \dot{Z}_2 \\ \dot{Z}_3 \end{Bmatrix} = \begin{bmatrix} 1-a_2 Z_2^2 & -a_3 Z_2 Z_3 \\ -a_2 Z_2 Z_3 & 1-a_3 Z_3^2 \end{bmatrix} \begin{Bmatrix} \dfrac{k_2}{F_{y2}} \dot{D}_2 \\ \dfrac{k_3}{F_{y3}} \dot{D}_3 \end{Bmatrix}$$

Coupled Evolutionary Equations

$$a_2 = \begin{cases} 1 & if\; \dot{D}_2 Z_2 > 0 \\ 0 & otherwise \end{cases}$$

$$a_3 = \begin{cases} 1 & if\; \dot{D}_3 Z_3 > 0 \\ 0 & otherwise \end{cases}$$

$$\sqrt{Z_2^2 + Z_3^2} \le 1$$

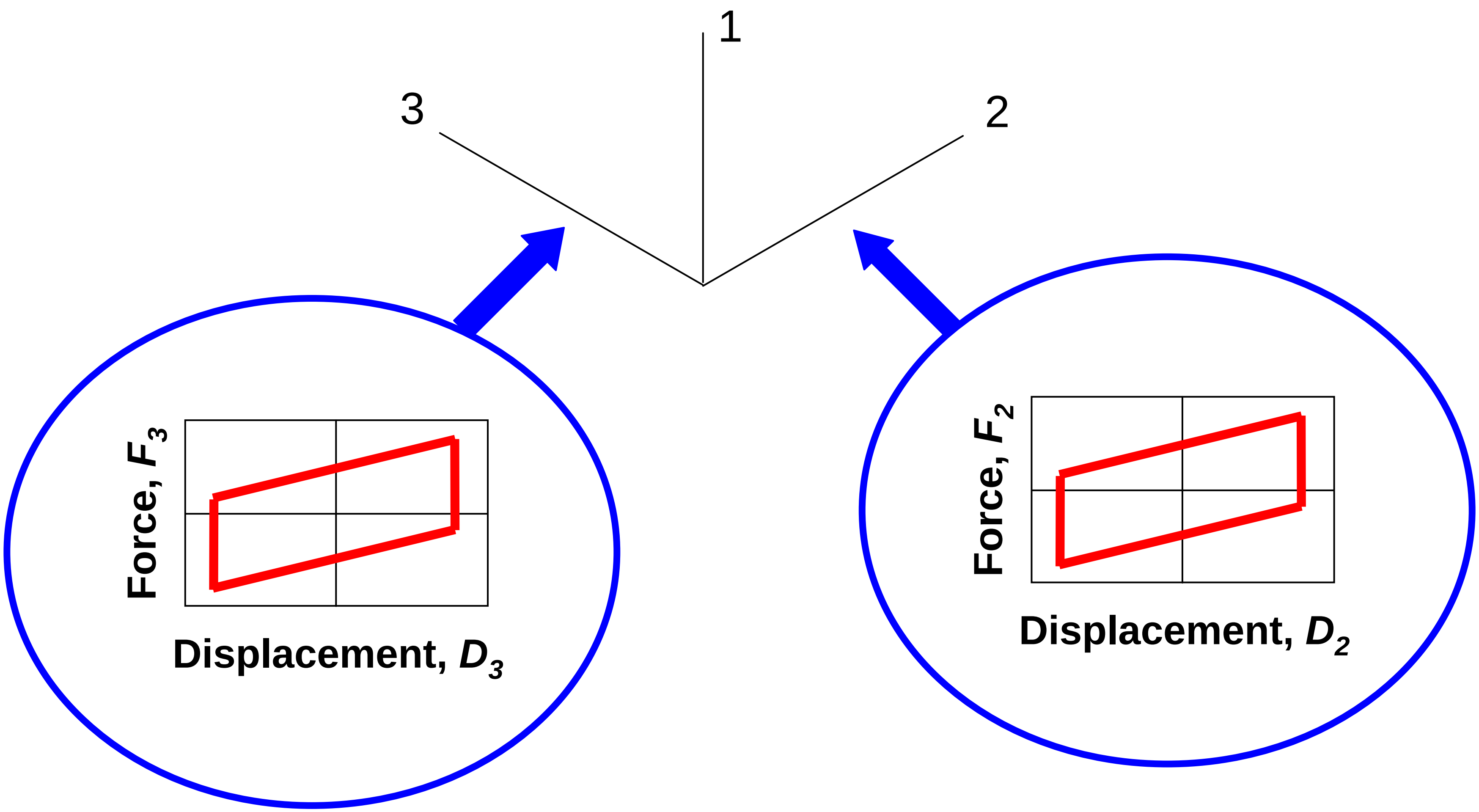
Range of Evolutionary Variables

$$\sqrt{Z_2^2 + Z_3^2} = 1$$

Defines Yield Surface

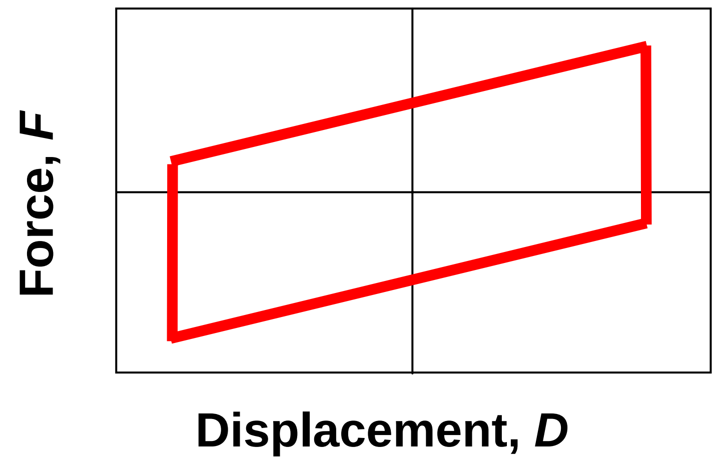
Modeling Isolation Bearings Using the SAP2000 NLLINK Element

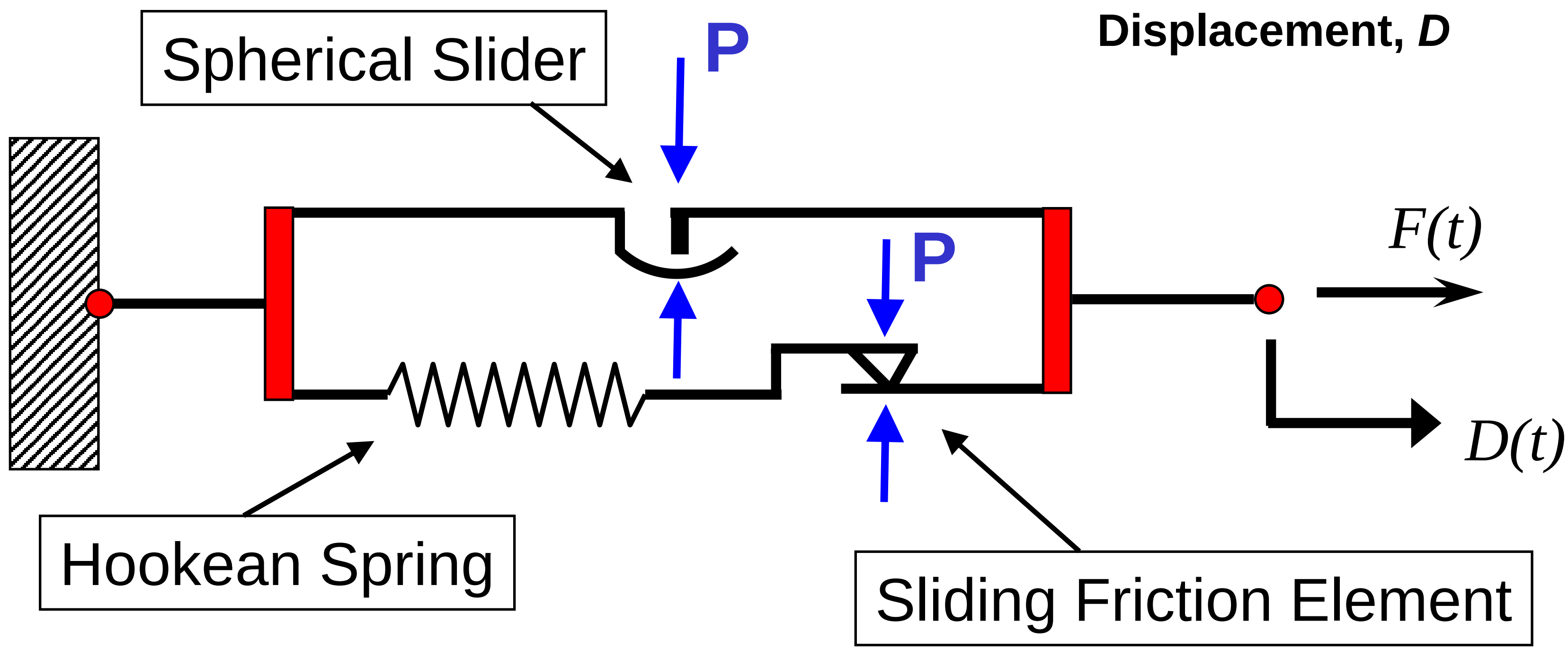
ISOLATOR2 Property – Biaxial Friction Pendulum Isolator



Mechanical Model of FPS Bearing in SAP2000

ISOLATOR2 Property
– Biaxial Friction Pendulum Isolator

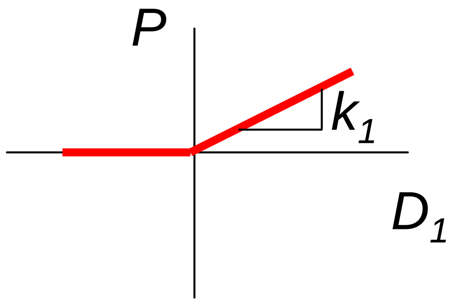




Forces in Biaxial FPS Isolator

$$F_1 = P = \begin{cases} k_1 D_1 & \text{if } D_1 < 0 \\ 0 & \text{otherwise} \end{cases}$$

Axial Force:
+ = Comp.
- = Tension



$$\left. \begin{aligned} F_2 &= \frac{P}{R_2} D_2 + P\mu_2 Z_2 \\ F_3 &= \frac{P}{R_3} D_3 + P\mu_3 Z_3 \end{aligned} \right\}$$

Shear Force Along Each Orthogonal Direction

For FPS Bearing, $R_2 = R_3$

$$\left. \begin{aligned} \mu_2 &= \mu_{max\,2} - (\mu_{max\,2} - \mu_{min\,2}) e^{-rv} \\ \mu_3 &= \mu_{max\,3} - (\mu_{max\,3} - \mu_{min\,3}) e^{-rv} \end{aligned} \right\}$$

Friction Coefficients

$$v = \sqrt{\dot{D}_2^2 + \dot{D}_3^2}$$

Resultant Velocity

$$r = \frac{r_2 \dot{D}_2^2 + r_3 \dot{D}_3^2}{v^2}$$

Effective Inverse Velocity

Forces in Biaxial FPS Isolator

$$\begin{Bmatrix} \dot{Z}_2 \\ \dot{Z}_3 \end{Bmatrix} = \begin{bmatrix} 1 - a_2 Z_2^2 & -a_3 Z_2 Z_3 \\ -a_2 Z_2 Z_3 & 1 - a_3 Z_3^2 \end{bmatrix} \begin{Bmatrix} \dfrac{k_2}{P\mu_2} \dot{D}_2 \\ \dfrac{k_3}{P\mu_3} \dot{D}_3 \end{Bmatrix}$$

Coupled Evolutionary Equations

$$a_2 = \begin{cases} 1 & \text{if } \dot{D}_2 Z_2 > 0 \\ 0 & \text{otherwise} \end{cases}$$

$$\sqrt{Z_2^2 + Z_3^2} \le 1$$

Range of Evolutionary Variables

$$a_3 = \begin{cases} 1 & \text{if } \dot{D}_3 Z_3 > 0 \\ 0 & \text{otherwise} \end{cases}$$

$$\sqrt{Z_2^2 + Z_3^2} = 1$$

Defines Yield Surface

$k_2, \quad k_3$ *Elastic Shear Stiffnesses (stiffness prior to sliding)*

Note: Flat Bearings: Set R = 0 for both directions (restoring forces will be set equal to zero).
Cylindrical Bearings: Set R = 0 for one direction.

Historical Development of Code Provisions for Base Isolated Structures

- Late 1980's: BSB (Building Safety Board of California) "An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation"

- 1986 SEAONC "Tentative Seismic Isolation Design Requirements"
  - Yellow book [emphasized equivalent lateral force (static) design]

- 1990 SEAOC "Recommended Lateral Force Requirements and Commentary"
  - Blue Book
  - Appendix 1L: "Tentative General Requirements for the Design and Construction of Seismic-Isolated Structures"

- 1991 and 1994 Uniform Building Code
  - Appendix entitled: "Earthquake Regulations for Seismic-Isolated Structures"
  - Nearly identical to 1990 SEAOC Blue Book

- 1994 NERHP Recommended Provisions for Seismic Regulations for New Buildings (FEMA 222A – Provisions; FEMA 223A - Commentary)
  - Section 2.6: Provisions for Seismically Isolated Structures
  - Based on 1994 UBC but modified for strength design and national applicability

Historical Development of Code Provisions for Base Isolated Structures

- **1996 SEAOC "Recommended Lateral Force Requirements and Commentary"**
  - **Chapter 1, Sections 150 to 161 (chapters/sections parallel those of 1994 UBC)**

- **1997 Uniform Building Code**
  - **Appendix entitled: "Earthquake Regulations for Seismic-Isolated Structures"**
  - **Essentially the same as 1991 and 1994 UBC**

- **1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 302 – Provisions; FEMA 303 - Commentary)**
  - **Chapter 13: Seismically Isolated Structures Design Requirements**
  - **Based on 1997 UBC (almost identical)**

- **1997 NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273 – Guidelines; FEMA 274 - Commentary)**
  - **Chapter 9: Seismic Isolation and Energy Dissipation**
  - **Introduces Nonlinear Static (pushover) Analysis Procedure**
  - **Isolation system design is similar to that for new buildings but superstructure design considers differences between new and existing structures**

Historical Development of Code Provisions for Base Isolated Structures

- **1999 SEAOC "Recommended Lateral Force Requirements and Commentary"**
  - Chapter 1, Sections 150 to 161 (chapters/sections parallel those of 1997 UBC)

- **2000 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 368 – Provisions; FEMA 369 - Commentary)**
  - Chapter 13: Seismically Isolated Structures Design Requirements

- **2000 Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356)**
  - Chapter 9: Seismic Isolation and Energy Dissipation

- **2000 International Building Code (IBC)**
  - Section 1623: Seismically Isolated Structures
  - Based on 1997 NEHRP Provisions
  - Similar to FEMA 356 since same key persons prepared documents

General Philosophy of Building Code Provisions

- No specific isolation systems are described

- All isolation systems must:
  - Remain stable at the required displacement
  - Provide increasing resistance with increasing displacement
  - Have non-degrading properties under repeated cyclic loading
  - Have quantifiable engineering parameters

Design Objectives of 2000 NEHRP and 2000 IBC Base Isolation Provisions

- Minor and Moderate Earthquakes
  - No damage to structural elements
  - No damage to nonstructural components
  - No damage to building contents

- Major Earthquakes
  - No failure of isolation system
  - No significant damage to structural elements
  - No extensive damage to nonstructural components
  - No major disruption to facility function
  - Life-Safety

2000 NEHRP and 2000 IBC Base Isolation Provisions

General Design Approach

EQ for Superstructure Design

Design Earthquake

10%/50 yr = 475-yr return period

- Loads reduced by up to a factor of 2 to allow for limited Inelastic response; a similar fixed-base structure would be designed for loads reduced by a factor of up to 8

EQ for Isolation System Design (and testing)

Maximum Considered Earthquake

2%/50 yr = 2,500-yr return period

- No force reduction permitted for design of isolation system

Analysis Procedures of 2000 NEHRP and 2000 IBC Base Isolation Provisions

- **Equivalent Lateral Response Procedure**
  - **Applicable for final design under limited circumstances**
  - **Provides lower bound limits on isolation system displacement and superstructure forces**
  - **Useful for preliminary design**

Presented Herein

- **Dynamic Lateral Response Procedure**
  - **May be used for design of any isolated structure**
  - **Must be used if structure is geometrically complex or very flexible**
  - **Two procedures:**
    - **Response Spectrum Analysis (linear)**
    - **Response-History Analysis (linear or nonlinear)**

Isolation System Displacement (Translation Only)

Design Displacement

Design Spectral Acceleration at One-Second Period (g)

$$D_D = \left(\frac{g}{4\pi^2} \right) \frac{S_{D1} T_D}{B_D}$$

Effective Period of Isolated Structure at Design Displacement

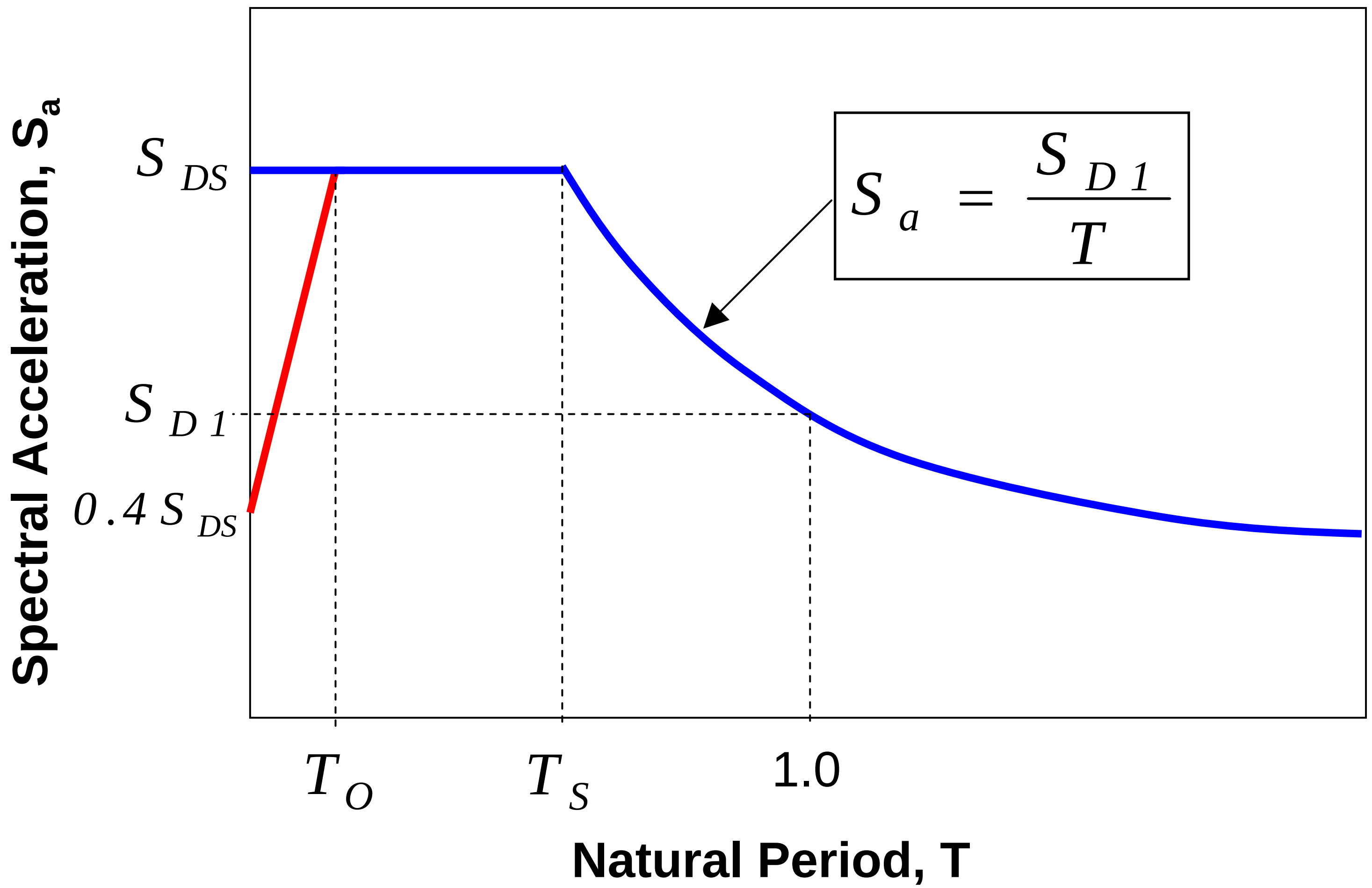
Damping Reduction Factor for Isolation System at Design Displacement

Design is evaluated at two levels:

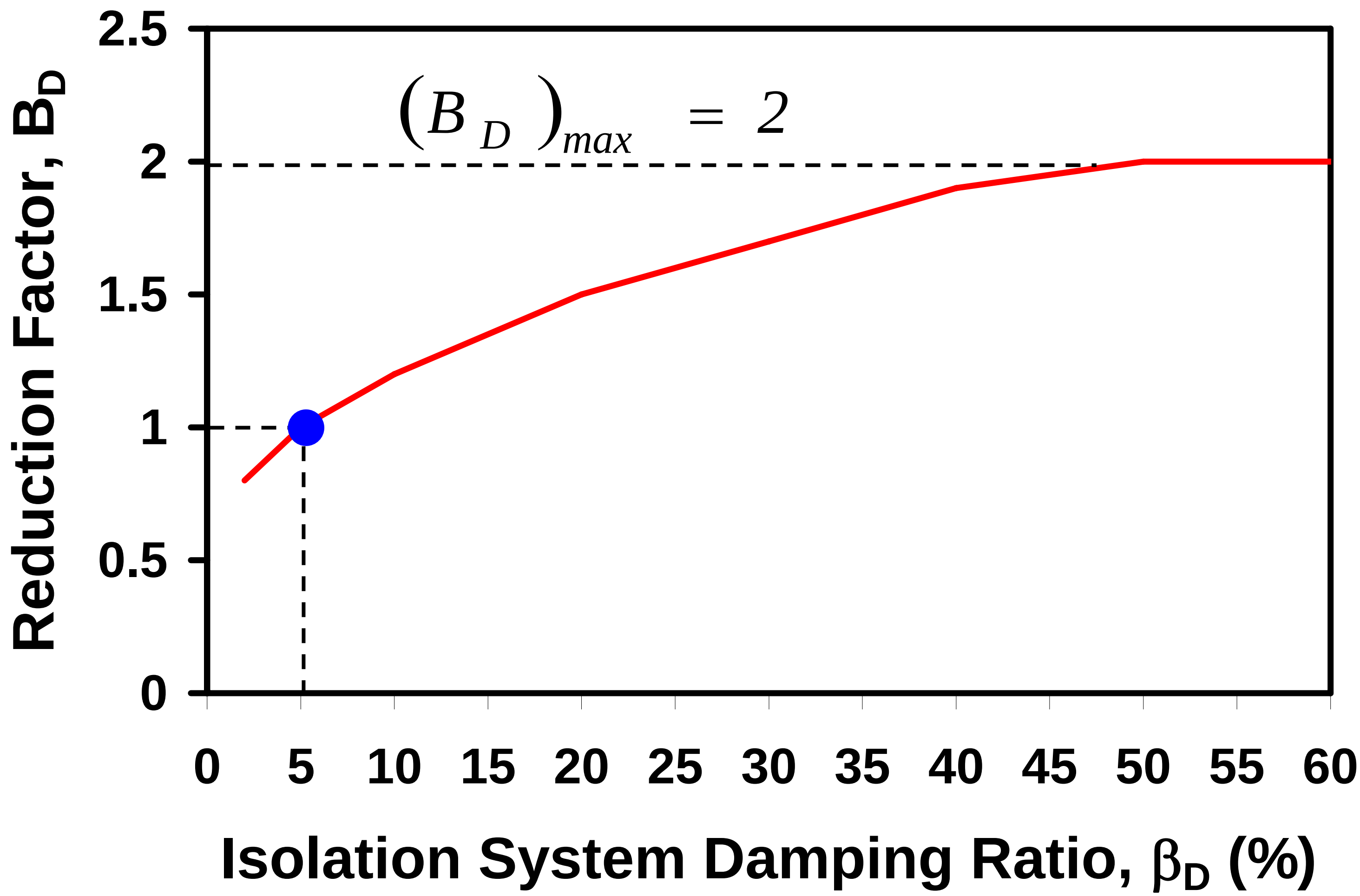
Design Earthquake: 10% / 50 yr = 475-yr return period

Maximum Considered Earthquake: 2% / 50 yr = 2,500-yr return period

Design Response Spectrum



Damping Reduction Factor



Effective Isolation Period

Effective Period

$$T_D = 2\pi \sqrt{\frac{W}{k_{D\,min}\, g}}$$

Total Seismic Dead Load Weight

Minimum Effective Stiffness of Isolation System at Design Displacement

Minimum stiffness used so as to produce largest period and thus most conservative design displacement.

Isolation System Displacement (Translation and Rotation)

Total Design Displacement

Eccentricity (actual + accidental) Between CM of Superstructure and CR of Isolation System

$$D_{TD} = D_D\left[1 + y\left(\frac{12e}{b^2 + d^2}\right)\right]$$

Use only if isolation system has uniform spatial distribution of lateral stiffness

Distance Between CR of Isolation System and Element of Interest

Shortest and Longest Plan Dimensions of Building

Note: A smaller total design displacement may be used (but not less than 1.1D_D) provided that the isolation system can be shown to resist torsion accordingly.

Base Shear Force

Isolation System and Elements Below Isolation System

$$V_b = k_{D\,max} D_D$$

No Force Reduction; Therefore Elastic Response Below Isolation System

Maximum Effective Isolation System Stiffness

Shear Force Above Isolation System

Structural Elements Above Isolation System

$$V_S = \frac{k_{D\,max} D_D}{R_I}$$

Response Modification Factor for Isolated Superstructure

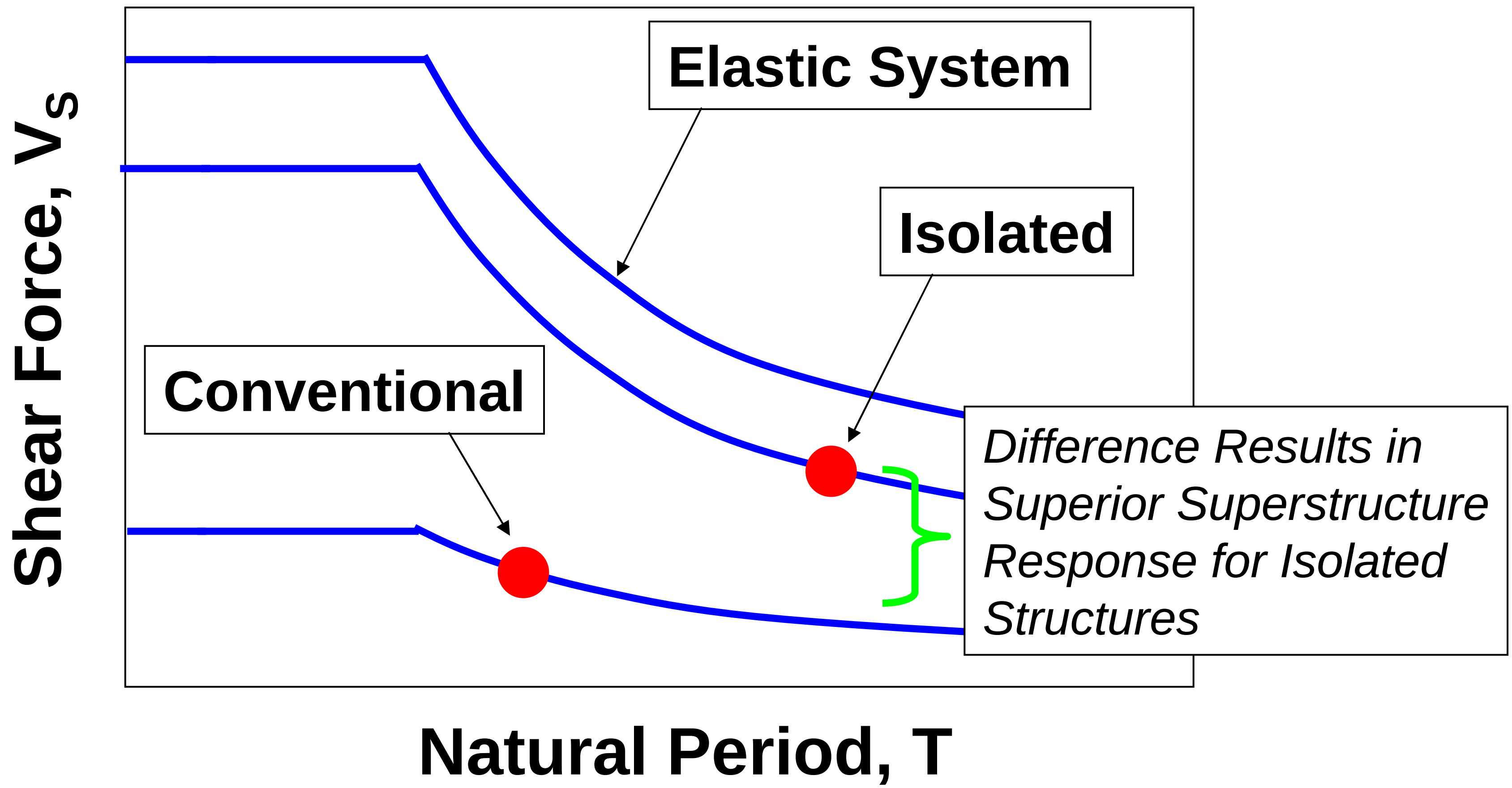
$$R_I = \frac{3}{8} R = \frac{R}{2.67} \leq 2$$

Ensures essentially elastic superstructure response

Minimum Values of V_S:

- Base shear force for design of conventional structure of fixed-base period T_D
- Shear force for wind design.
- 1.5 times shear force that activates isolation system.

Design Shear Force for Conventional and Isolated Structures



Example: Evaluation of Design Shear Force

Base Shear Coefficient

$$BSC_I = \frac{V_S}{W} = \frac{k_{D\,max} D_D}{W R_I} = \frac{S_{D1}}{B_D R_I T_D}$$ *Isolated Structure*

$$BSC_C = \frac{V_S}{W} = C_S = \frac{S_{D1}}{T(R/I)}$$ *Conventional Structure Having Period of One-Second or More*

$$\frac{BSC_I}{BSC_C} = \frac{T(R/I)}{B_D R_I T_D}$$

Example:

- Fire Station (I = 1.5)
- Conventional: Special steel moment frame (R = 8.5) and T = 1.0 sec
- Isolated: T_D = 2.0 sec, damping ratio = 10% (B_D = 1.2), R_I = 2

Result: $$\frac{BSC_I}{BSC_C} = 1.18$$

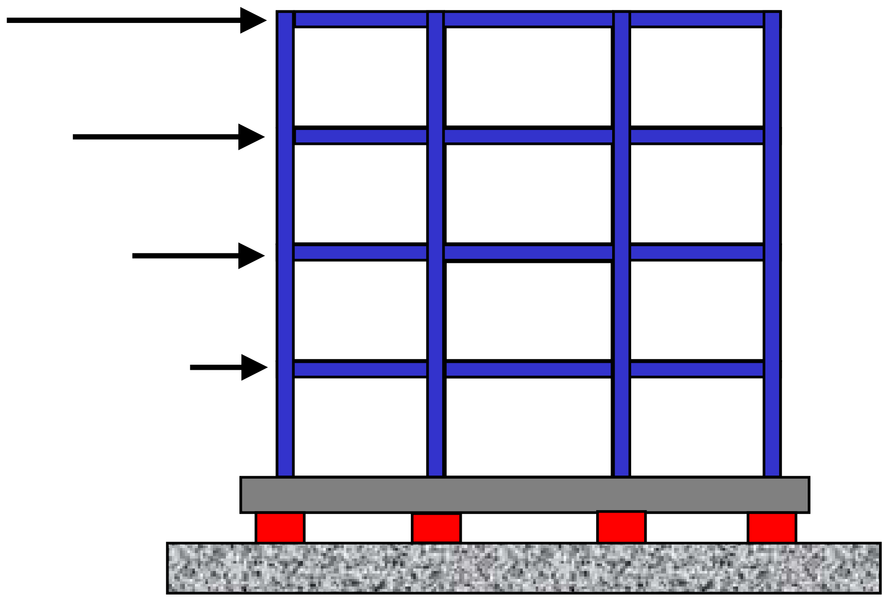
Isolating structure results in 18% *increase* in shear force for design of superstructure

Distribution of Shear Force

$$F_x = \frac{V_S w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

Standard Inverted Triangular Distribution of Base Shear

Lateral Force at Level x of the Superstructure



Interstory Drift Limit

Displacement at Level x of Superstructure

Deflection Amplification Factor

Displacement at Level x of Superstructure Based on Elastic Analysis

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

Occupancy Importance Factor

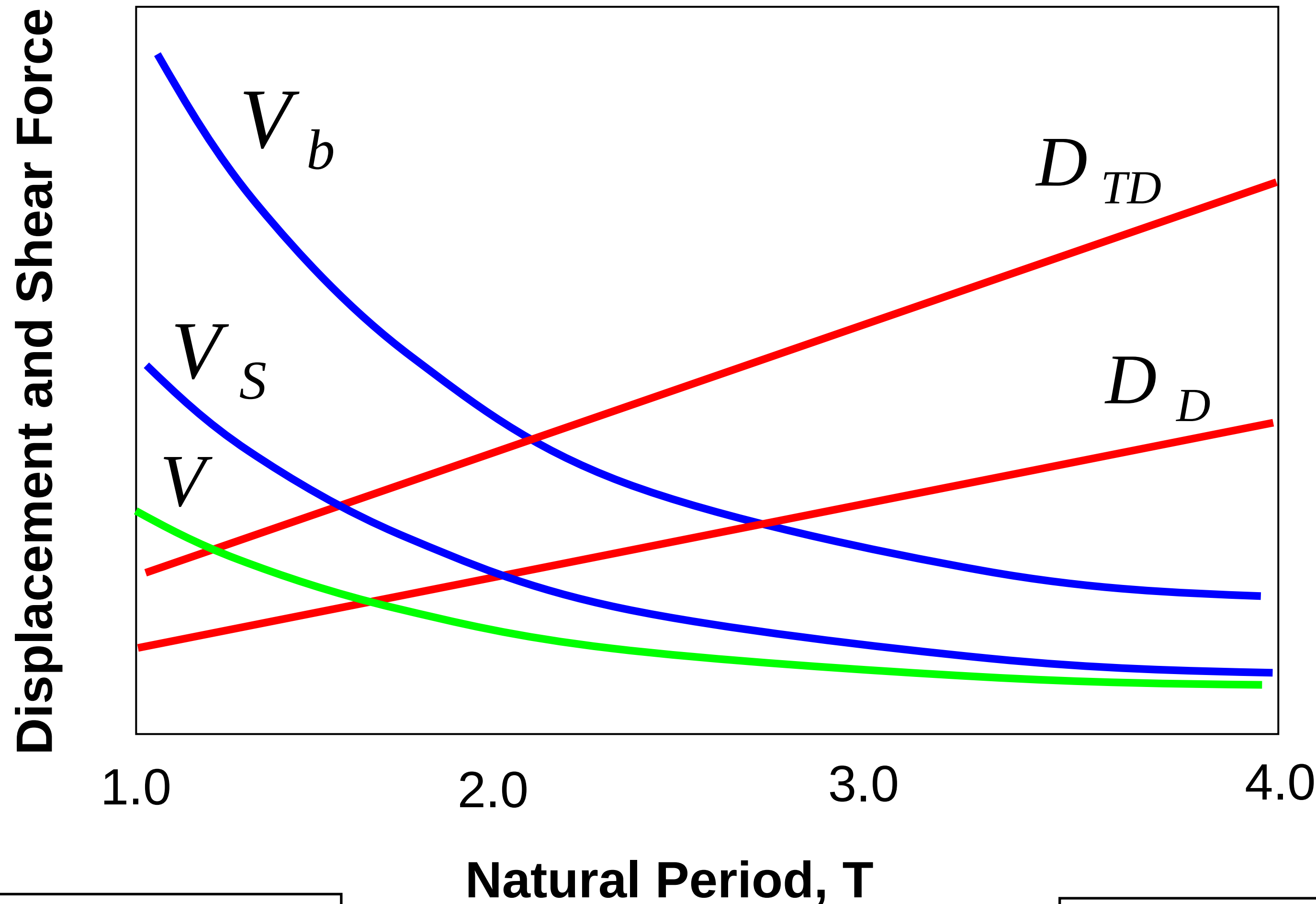
Note: For Isolated Structures, C_d is replaced by R_I.

Interstory Drift of Story x

$$\Delta_x \leq 0.015 \; h_{sx}$$

Height of Story x

Displacement and Shear Force Design Spectrum



$$V_b = k_{D\,max} D_D$$

$$V_S = \frac{k_{D\,max} D_D}{R_I}$$

$$V = C_S W$$

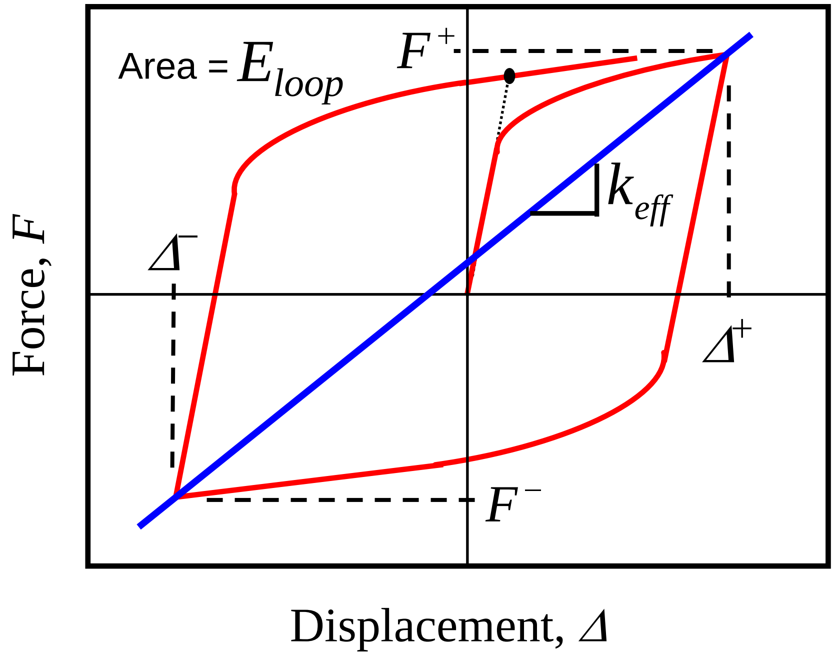
Required Tests of Isolation System

Prototype Tests on Two Full-Size Specimens of Each Predominant Type of Isolation Bearing

- **Check Wind Effects**
  - **20 fully reversed cycles at force corresponding to wind design force**
- **Establish Displacement-Dependent Effective Stiffness and Damping**
  - **3 fully reversed cycles at 0.25D_D**
  - **3 fully reversed cycles at 0.5D_D**
  - **3 fully reversed cycles at 1.0D_D**
  - **3 fully reversed cycles at 1.0D_M**
  - **3 fully reversed cycles at 1.0 D_{TM}**
- **Check Stability**
  - **Maximum and minimum vertical load at 1.0 D_{TM}**
- **Check Durability**
  - **30$S_{D1}B_D/S_{DS}$, but not less than 10, fully reversed cycles at 1.0 D_{TD}**

For cyclic tests, bearings must carry specified vertical (dead and live) loads

Effective Linear Properties of Isolation Bearing from Cyclic Testing



$$k_{eff} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|}$$

Effective Stiffness of Isolation Bearing

$$\beta_{eff} = \frac{2}{\pi} \frac{E_{loop}}{k_{eff}\left(|\Delta^+| + |\Delta^-|\right)^2}$$

Equivalent Viscous Damping Ratio of Isolation Bearing

Effective properties determined for each cycle of loading

Effective Linear Properties of Isolation System from Cyclic Testing

Absolute Maximum Force at Positive D_D over 3 Cycles of Motion at 1.0D_D

$$k_{D\,max} = \frac{\sum \left| F_D^+ \right|_{max} + \sum \left| F_D^- \right|_{max}}{2D_D}$$

Maximum Effective Stiffness of Isolation System

$$k_{D\,min} = \frac{\sum \left| F_D^+ \right|_{min} + \sum \left| F_D^- \right|_{min}}{2D_D}$$

Minimum Effective Stiffness of Isolation System

Use smallest value from cyclic tests

$$\beta_D = \frac{1}{2\pi} \frac{\sum E_D}{k_{D\,max} D_D^2}$$

Equivalent Viscous Damping Ratio of Isolation System

Additional Issues to Consider

- Buckling and stability of elastomeric bearings
- High-strain stiffening of elastomeric bearings
- Longevity (time-dependence) of bearing materials
  (Property Modification Factors to appear in 2003 NEHRP Provisions)
- Displacement capacity of non-structural components that cross isolation plane
- Displacement capacity of building moat
- Second-order (P-Δ) effects on framing above and below isolation system

Example Design of Seismic Isolation System Using 2000 NEHRP Provisions

Seismically Isolated Structures by Charles A. Kircher
Chapter 11 of *Guide to the Application of the 2000 NEHRP Provisions; Note: The Guide is in final editing. Chapter 11 is in the handouts.*

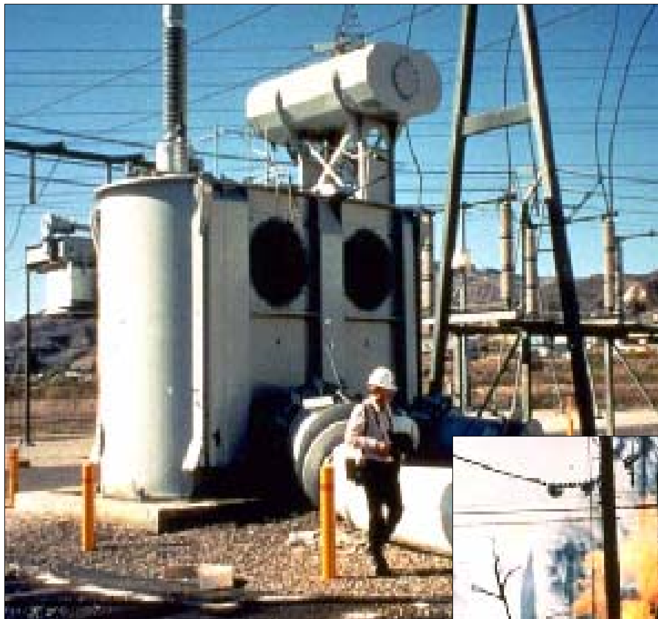
Structure and Isolation System

- "Hypothetical" Emergency Operations Center, San Fran., CA
- Three-Story Steel Braced-Frame with Penthouse
- High-Damping Elastomeric Bearings

Design Topics Presented:

- Determination of seismic design parameters
- Preliminary design of superstructure and isolation system
- Dynamic analysis of isolated structure
- Specification of isolation system design and testing criteria

NONBUILDING STRUCTURES



Nonbuilding Structures

Same:

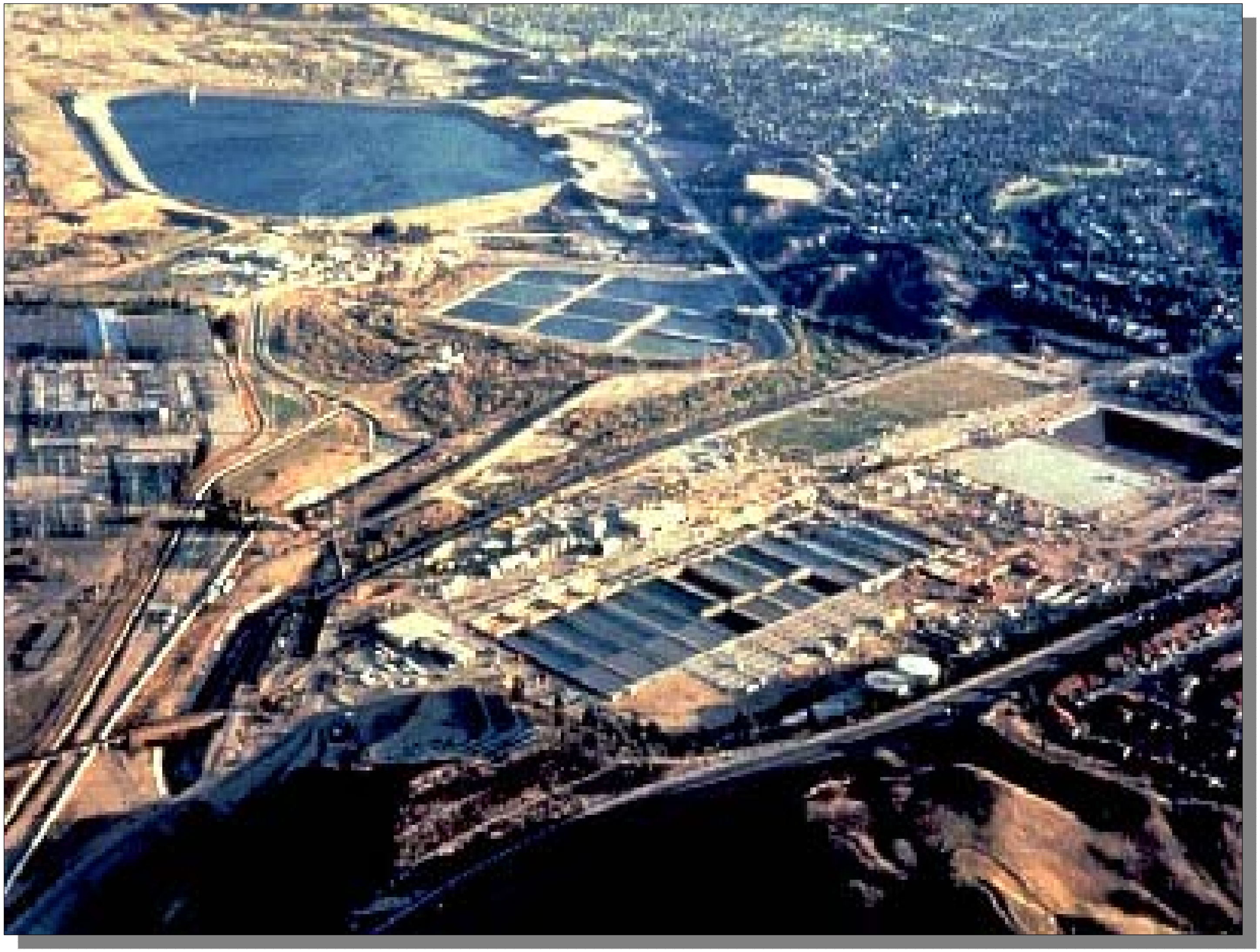
- Basic ground motion hazards
- Basic structural dynamics

Different:

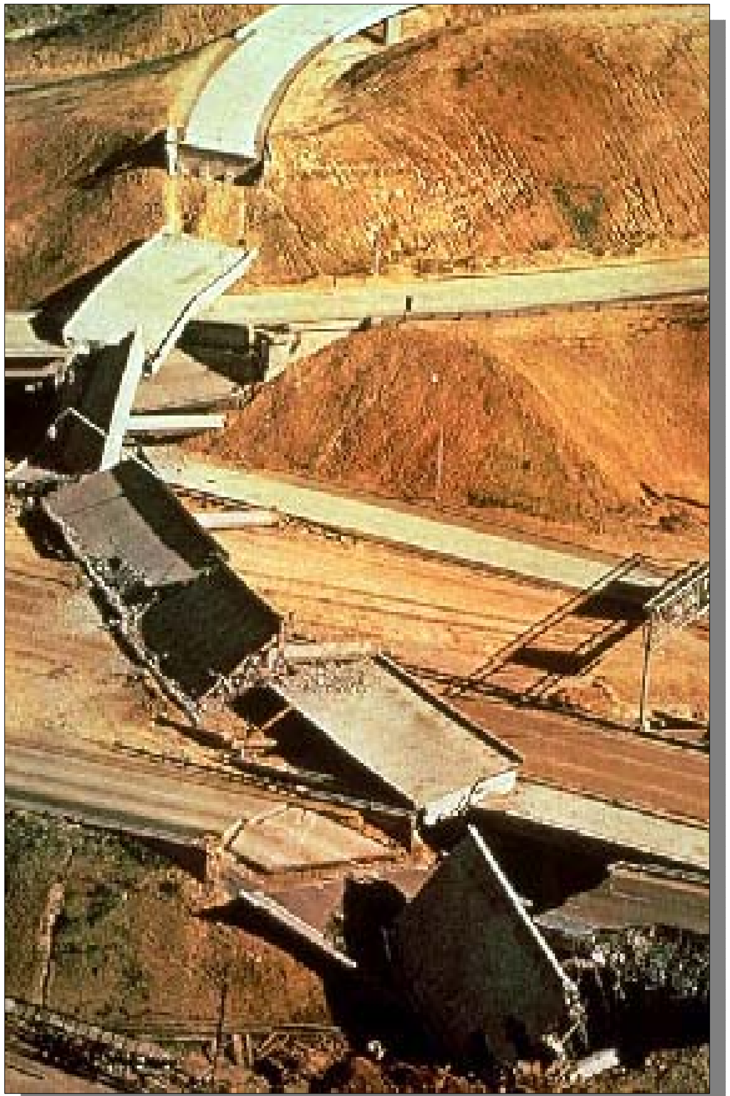
- Structural characteristics
- Fault rupture
- Fluid dynamics
- Performance objectives
- Networked systems

Dams with Damage

Dam and Water Treatment Plant



Bridges



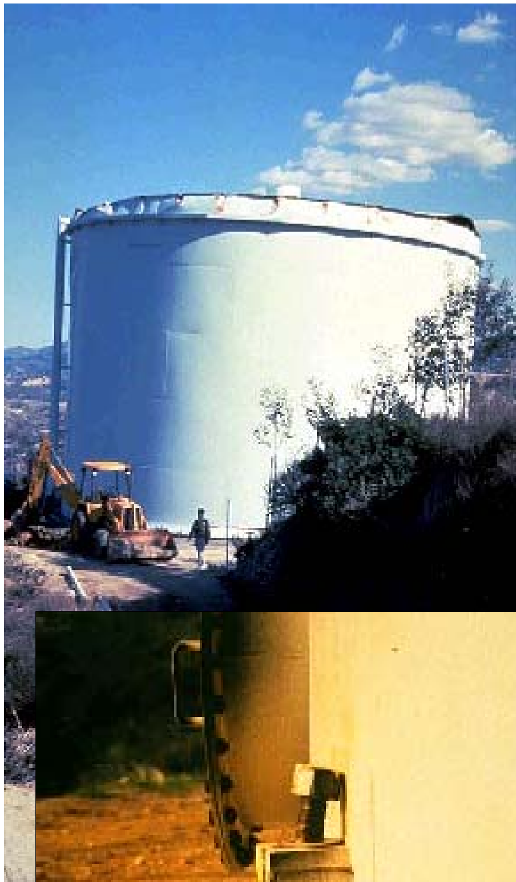
Joints at Long Spans

Elevated Roadways (1)

Elevated Roadways (2)

Lack of Redundancy

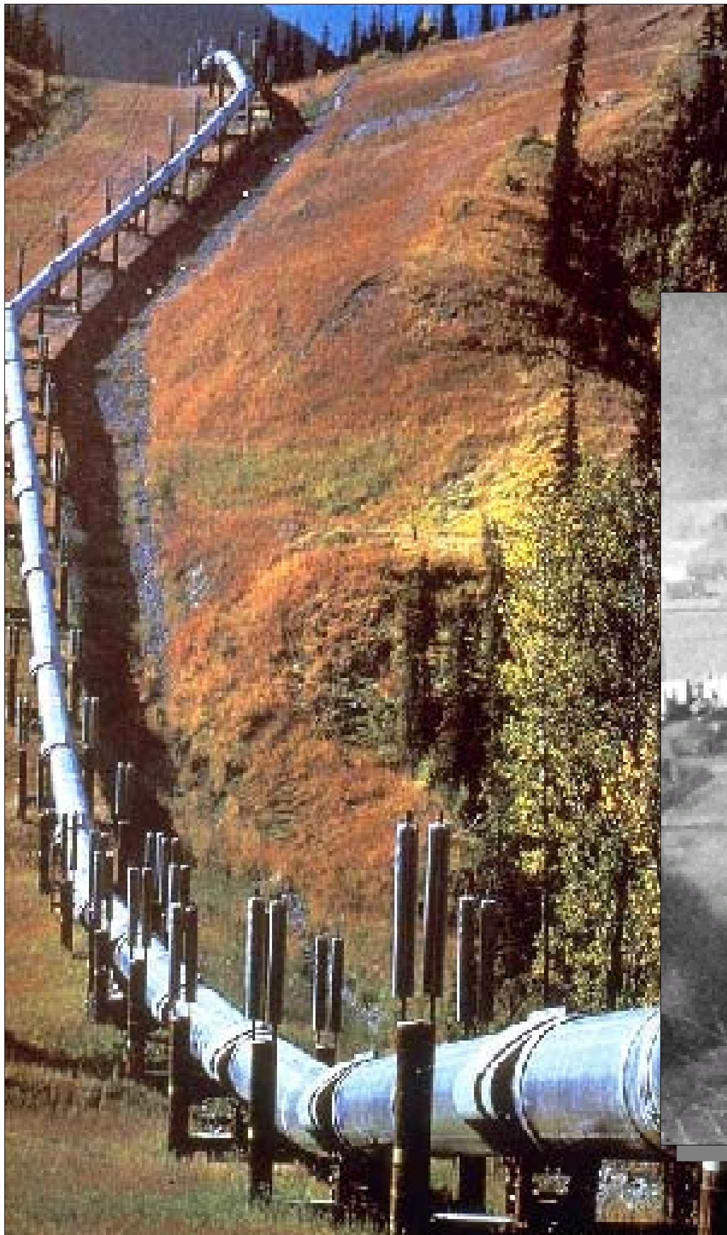
Tanks



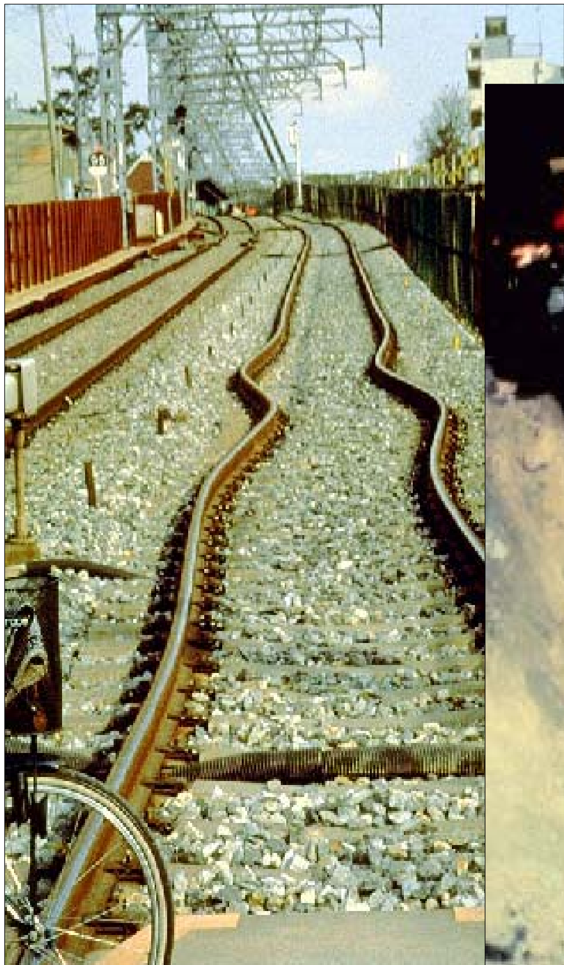
Elephant’s foot buckling

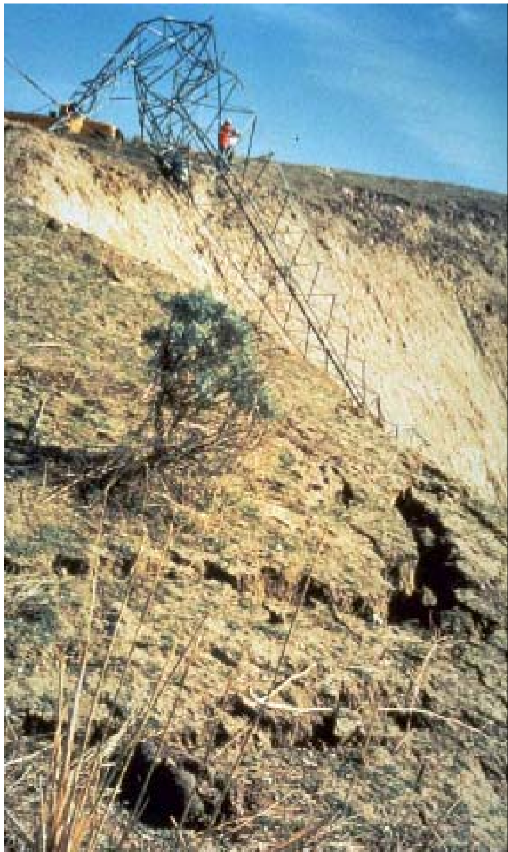
Tanks & Towers

Piplines

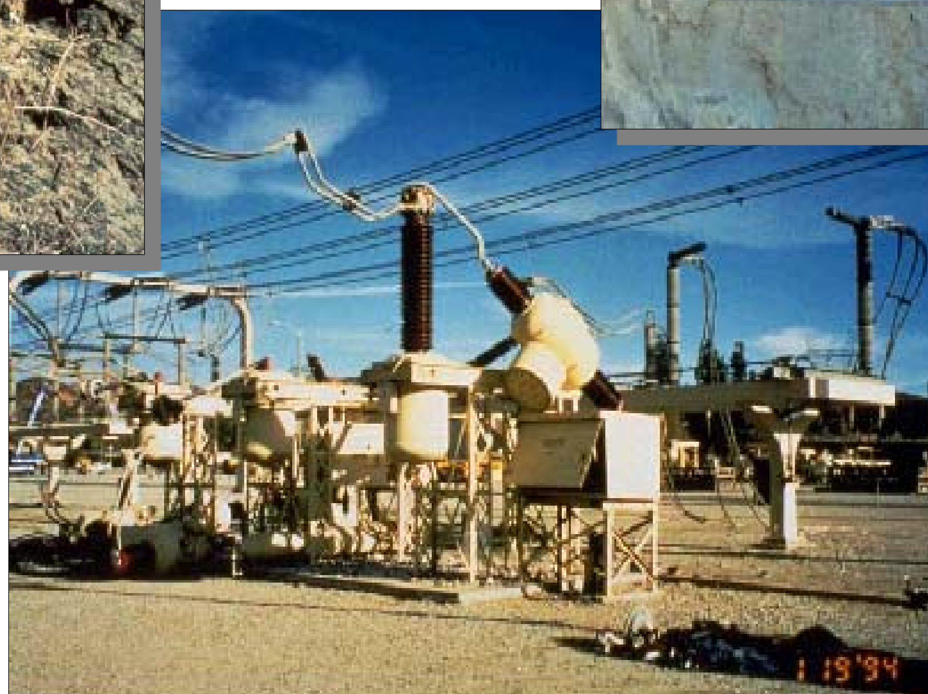


On-Grade and Buried





Electrical Towers and



Substations

Nonbuilding Structures in the *NEHRP Recommended Provisions*

SCOPE of Chapter 14:

- Self supporting structures that carry gravity loads.
- Nonbuilding structures may be supported by earth or
  by other structures.

EXCLUSIONS:

- Vehicular and railroad bridges
- Nuclear power plants
- Offshore platforms
- Dams

Nonbuilding Structures

TWO CLASSIFICATIONS included in *Provisions*

1. Nonbuilding structures similar to buildings

- Dynamic response similar to buildings
- Structural systems are designed and constructed similar to buildings
- Use provisions of Chapter 14 and applicable parts of Chapter 5, 7, 8, 9, ...

2. Nonbuilding structures not similar to buildings

- Design and construction results in dynamic response different from buildings
- Use Chapter 14 and "approved standards" for design

Nonbuilding Structures defined similar to buildings (2000)

Examples:

- Pipe racks
- Steel storage racks
- Electric power generation facilities
- Structural towers for tanks & vessels

(Many of these have changed in the 2003 edition)

Nonbuilding Structures not similar to buildings

- Use "approved standards" for design. Loads and load distributions shall not be less than those given by NEHRP RP.

Examples:

- Earth retaining structures
- Tanks and vessels
- Telecommunication towers
- Stacks and chimneys
- Buried structures (tanks, tunnels, pipes)

Nonbuilding Structures not similar to buildings

Examples of approved design standards:

- Telecommunications structures:
  - ASCE 7, *Minimum Design Loads for Buildings and Other Structures,* 1995.
  - TIA/EIA 222F, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures, 1996.
- Steel Stacks and Chimneys:
  - ANSI/ASME STS-1-1992, *Steel Stacks*

Nonbuilding Structures
Design Requirements

- LOADS
  - Weight, W, for calculating seismic forces includes all dead loads and all normal operating contents
  - (grain, water, etc. for bins and tanks)
- DRIFT LIMITATIONS
  - Drift limits of Section 5.2.8 do not apply - but must maintain stability. P-Δ check required.
- FUNDAMENTAL PERIOD
  - Calculate using same methods for buildings (5.3.3)

Nonbuilding Structures
Design Requirements

- **VERTICAL DISTRIBUTION OF SEISMIC FORCES**
  - Use same methods for buildings:
  - ELF or Modal Analysis
- **NONBUILDING STRUCTURES SUPPORTED BY OTHER STRUCTURES**
  - If $W_{nb} < 25\%$ of W_{tot} treat nonbuilding structure as component and design per Chapter 6
  - If $W_{nb} \geq 25\%$ of W_{tot} determine seismic forces considering effects of combined structural systems

Nonbuilding Structures
Design Requirements

- **SEISMIC COEFFICIENTS AND HEIGHT LIMITS**
  – Use smaller R factor from Table 5.2.2 or Table 14.2.1.1
  – In general, height limits for nonbuilding structures are less stringent than those for buildings

Nonbuilding Structures
Design Requirements

Table 14.2.1.1: Seismic Coefficients and Height Limits

Structural System	R	Ω_o	C_d	HL	X
Steel storage racks	4	2	3½	NL	--
Elevated tanks on braced legs	3	2	2½	NL	--
Reinf conc tanks (nonsliding base)	2	2	2	NL	--
Conc silos, stacks…w/ walls to fdn	3	1 ¾	3	NL	--
Trussed towers, guyed stacks…	3	2	2 ½	NL	--
Self-supporting, not covered by other standards and not similar to bldgs	1 ¼	2	2 ½	C	--

Nonbuilding Structures
Design Requirements

- **IMPORTANCE FACTOR AND SEISMIC USE GROUP**
  - Based on relative hazard of contents and function
  - Use largest value from Table 14.2.1.2 or from approved standard

Nonbuilding Structures
Design Requirements

- **Table 14.2.1.2: Importance Factor and SUG**

Importance Factor	I=1.0	I=1.25	I=1.5
Seismic Use Group	I	II	III
Hazard	H-I	H-II	H-III
Function	F-I	F-II	F-III

H-I, H-II and H-III: Relative hazard of stored product

F-III: Communication towers, fuel storage tanks, cooling towers etc., required for the operation of SUG III buildings

F-II: Not applicable

Nonbuilding Structures
Chapter 14 Appendix

Additional design procedures and recommendations for:

- Electrical transmission, substation and distribution structures
- Buried structures
- represents current industry accepted design practice
- info not ready for inclusion in main body of chapter

Nonstructural Components

Architectural, Mechanical and Electrical Components supported by or located within buildings or other structures.

In 2003 *NEHRP Recommended Provisions*:

- Chapter 6 – Architectural, Mechanical, and Electrical Component Design Requirements

In ASCE 7-05:

- Chapter 13 – Seismic Design Requirements for Nonstructural Components
- Section 12.11.2 Anchorage of Concrete and Masonry Structural Walls

See also Chapter 13 of
FEMA 451, *NEHRP Recommended Provisions: Design Examples*

ASCE 7 Chapter 13
Nonstructural Components

1. Scope and Application
2. Design Requirements
3. Seismic Force and Imposed Displacements
4. Component Anchorage
5. Architectural Components
6. Mechanical and Electrical

Nonstructural Limits of Applicability

- ASCE 7-05 – Applies throughout the United States with following exceptions:
  1. Mechanical and Electrical Components in SDC A and B
  2. Mechanical and Electrical in SDC if I_p = 1.0
  3. Architectural in SDC A
  4. Architectural in SDC B if I_p = 1.0 except certain parapets

- Other exceptions for light items, piping and ductwork in both

There is a important errata regarding this section.

See: www.seinstitute.org/pdf/erratasheet7-05.pdf

(these changes are not in the 2003 *Provisions*)

Nonstructural Demands

- Equivalent Static Forces – F_p Equation – Independent of building structural properties
    1. Strength Level Forces in both codes
    2. ASCE 7-05 provides building dependent option of determining.

- Relative Displacements for Selected Components
    1. Anticipated Relative Displacements at Design Earthquake Level in both codes (Δ_m or D_p).
    2. ASCE 7-05 provides explicit equations and option of determining using building structural properties.

Nonstructural Force Demand

- ASCE 7-05 – Based on 2003 NEHRP

$$Fp = \frac{0.4 a_p S_{DS}}{\left(R / I_p \right)} \left(1 + 2 \frac{z}{h} \right) W_p$$

F_p (min) = 0.3 S_{DS} I_p W_p *for* S_{DS} *= 1.00,* F_p *= 0.30* $I_p W_p$
F_p (max) = 1.6 S_{DS} I_p W_p *for* S_{DS} *= 1.00,* F_p *= 1.60* $I_p W_p$

Nonstructural Importance Factor - I_p

- ASCE 7-05 has assigned Nonstructural Component Importance Factor, I_p
- The values of I_p is either 1.0 or 1.5
- In ASCE 7-05, the value of I_p is based

1. Requirements of the component to function after a DBE

or

2. Occupancy Category of the structure or facility

- In ASCE 7-05, nonstructural components/systems which are assigned an I_p = 1.5 are called *Designated Seismic Systems*.

Nonstructural Component a_p and R_p Factors

- ASCE 7-05 have a_p and R_p factors assigned in tables that are used in F_p equation.
- The values of a_p range from 1.0 to 2.5 in both codes and values of a_p can be taken as less than 2.5 based on dynamic analysis.
- In ASCE 7-05, R_p values range from 1.0 to 12.0.
- The values of R_p can be assigned based on the ductility and deformability capacity.
- In ASCE 7-05 there only 2 footnotes to the a_p and R_p tables.

Changes in Nonstructural Component a_p and R_p Factors

- Many values of a_p and R_p for equipment and distributed systems in ASCE 7-05 differ from those in the NEHRP *Provisions*
- Changes are highlighted in the following Tables

Comparison of a_p and R_p values for Selected Architectural Components

Architectural Component	2003 NEHRP		ASCE 7-05	
	a_p	R_p	a_p	R_p
Cantilever Parapets	2.5	2.5	2.5	2.5
Exterior Walls* (rigid diaph.)	1.0	2.5	1.0	2.5
Partitions* (rigid diaph.)	1.0	2.5	1.0	2.5
Ceilings	1.0	2.5	1.0	2.5
Penthouses (not an extension)	2.5	3.5	2.5	3.5
Signs and Billboards	2.5	2.5	2.5	2.5
Access Floors* (special)	1.0	2.5	1.0	2.5
Storage Cabinets	1.0	2.5	1.0	2.5

Comparison of F_p/W_p values for Architectural Components for $C_a = 0.40$, $S_{DS} = 1.0$, $I_p = 1.0$

Architectural Component	2003 NEHRP		ASCE 7-05	
	z = 0	z = h	z = 0	z = h
Cantilever Parapets	0.40	1.20	0.40	1.20
Exterior Walls	0.30	0.48	0.30	0.48
Partitions	0.30	0.48	0.30	0.48
Ceilings	0.30	0.48	0.30	0.48
Penthouses (not an extension)	0.30	0.86	0.30	0.86
Signs and Billboards	0.40	1.20	0.40	1.20
Access Floors (special)	0.30	0.48	0.30	0.48
Storage Cabinets	0.30	0.48	0.30	0.48

Comparison of a_p and R_p values for Selected Mechanical and Electrical Components

Mech./Elect. Component	2003 NEHRP		ASCE 7-05	
	a_p	R_p	a_p	R_p
Tanks and Vessels w/o skirts	1.0	2.5	1.0	2.5
Air side equipment	1.0	2.5	2.5	6.0
Wet side equipment	1.0	2.5	1.0	2.5
Emergency Battery Racks	1.0	2.5	1.0	2.5
Stacks, towers braced below cg	2.5	2.5	2.5	3.0
Vibration Isolated Equipment*	2.5	2.5	2.5	2.0 – 2.5
Piping – ASME Welded	1.0	3.5	2.5	12.0
Piping – threaded joints	1.0	2.5	2.5	4.5

Comparison of F_p/W_p Values for Mech./Elect. Components for $C_a = 0.40$, $S_{DS} = 1.0$, $I_p = 1.0$

Mech./Elect. Component	2003 NEHRP		ASCE 7-05	
	z = 0	z = h	z = 0	z = h
Tanks and Vessels w/o skirts	0.30	0.48	0.30	0.48
Air side equipment	0.30	0.48	0.30	0.50
Wet side equipment	0.30	0.48	0.30	0.48
Emergency Batteries/Tanks	0.30	0.48	0.30	0.48
Stacks, towers braced below cg	0.40	1.20	0.33	1.00
Vibration Isolated Equipment*	0.40	1.20	0.40-0.50	1.20-1.50
Piping – ASME Welded	0.30	0.34	0.30	0.30
Piping – threaded joints	0.30	0.48	0.30	0.67

Nonstructural Relative Displacement Demand

- ASCE 7-05 – Maximum Relative Displacements for DBE level motions are to be considered
- In ASCE 7-05
  - Required for Architectural Components which pose a life safety hazard including exterior wall elements and glazing
  - Required for Mech/Elect components and systems where I_p greater than 1.0.

Except for glazing – no specific acceptance criteria is provided

No requirement to stay within elastic limits or allowables

Load Combinations

- In ASCE 7-05, the redundancy factor, *ρ*, is specified as 1.0 for nonstructural components.
- In ASCE 7-05, Ω_o is not specified and load combinations with Ω_o are not used with nonstructural components (including penthouses)
- Vertical seismic forces need not be considered for lay-in access floor panels and lay-in ceiling panels (exception is not in the *Provisions*)

Anchorage of Nonstructural Components

In ASCE 7-05

All anchor forces based on R_p of 1.5 unless:

- Anchorage governed by ductile steel element or
- Post installed for Seismic Applications per ACI 355.2 or
- Anchors design in accordance with ACI 318-05, App. D

Additional 1.3 factor or maximum transferable force

Special requirement – vibration isolated equipment –
2 F_p if gap > 0.25”

Load path analysis to primary structure shall be performed.

Design and Detailing Requirements of Architectural Components

In ASCE 7-05:

- Specific demands exterior walls and connections
- Suspending Ceilings – CISCA & ASTM standards
- Glazing – Drift capacity AAMA 501.6
- Access Floors – special access floor details
- Tall Partitions – independent bracing

Design and Detail Requirements for Mechanical and Electrical Equipment

In ASCE 7-05:

- Sprinkler systems – NFPA 13 with amendments
- Escalators and Elevators – ASME A17.1
- Vessels – ASME B& PV
- Piping – ASCE B 31.1
- HVAC Ducting – (SMACNA not specifically referenced)
- Lighting fixtures – Prescriptive detail requirements
- Many specific prescriptive details for Mechanical and
- Electrical Equipment – Section 13.6.5.5

Certification Requirements for Certain Designated Seismic Systems (I_p=1.5) in the 2003 *Provisions*

- Testing permitted in lieu of analysis methods to determine the seismic capacity of components and their supports and attachments.
- Certification Requirements are found in Section 2.4.5
  - Registered design professional in responsible charge must state the applicable requirements on the construction documents.
  - Each manufacturer of designated seismic system components must test or analyze the component and its mounting system or anchorage
  - Evidence of compliance must be submitted for review and acceptance by the registered design professional and for approval by the authority having jurisdiction.
  - The evidence of compliance shall be by
    - Actual test on a shake table, or
    - Three-dimensional shock tests, or
    - Analytical methods using dynamic characteristics and forces, or
    - Use of experience, or
    - Rigorous analysis providing for equivalent safety

ASCE 7-05 Special Certification Requirements for Certain Designated Seismic Systems (I_p=1.5)

- In ASCE 7-05 - Seismic qualification required for
  1. Active mechanical and electrical equipment that are required to function following a DBE
  2. Components containing hazardous contents
- Qualification to demonstrate functionality after being subject to a DBE to be determined by either:
  1. Shake table testing – ICC-ES AC-156 , 2004
  2. Experience Data
  3. Analysis (extremely difficult for active equipment)
- Certification required by supplier indicating compliance

Final Comments about Nonstructural Components

- Much implementation detail in ASCE 7-05
- Additional documentation requirements in ASCE 7-05
- Much more QA requirements in ASCE 7-05. Note there are specific requirements for supports and attachments to be shown on the construction documents.

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