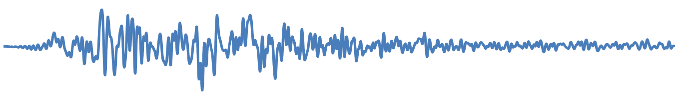
The Hashemite University
Department of Civil Engineering

Introduction to Earthquake Engineering

Introduction

Dr. Hazim Dwairi



Credits

This slide collection was developed by Dr. Hazim Dwairi, The Hashemite University, Jordan, 2007.

This presentation is based upon a seminar presentation prepared by EEFIT Team & Oxfam and presented by Dr Tiziana Rossetto at Chadwick Lecture Theatre, Dept of Civil and Environmental Engineering, University College London (UCL), after the Pakistan earthquake, 2006. The pictures concerning Turkey earthquake, 1999, are courtesy of Dr. Amr El Nashai, University of Illinois Urbana Champagne (UIUC). The rest of the pictures were taken from the files of Dr. Nigel Priestley, University of California San Diego (UCSD) and Dr. Mervyn Kowalsky, North Carolina State University (NCSU).

General Notes

- In the past century over One Million people have died due to earthquakes and earthquake related disasters.
- The economic losses estimated for the period 1929-1950 are in excess of US$ 10 billion. However the estimated losses between 2000-2010 are in excess of US$ 600 billion.
- For more developed countries the economic loss due to an earthquake can be enormous even if the death toll is fairly low
- Earthquakes cannot be prevented nor accurately predicted.

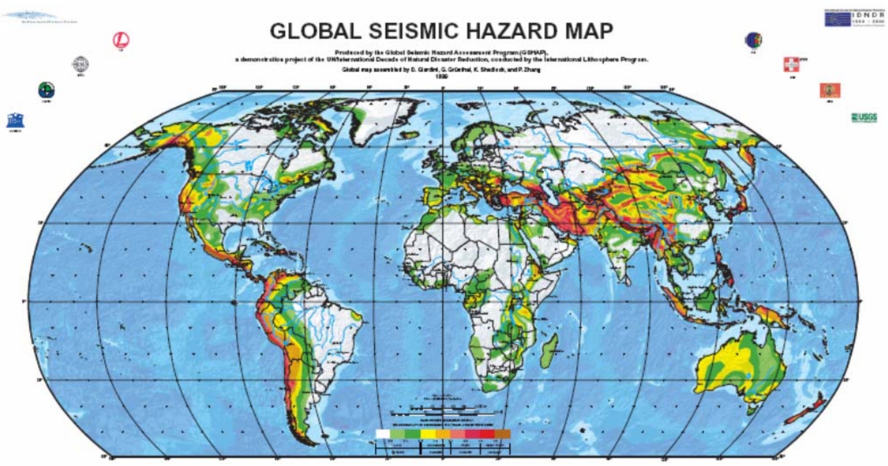
Number of EQs worldwide (2000-2012)

Magnitude	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012
8.0 to 9.9	1	1	0	1	2	1	2	4	0	1	1	1	2
7.0 to 7.9	14	15	13	14	14	10	9	14	12	16	23	19	12
6.0 to 6.9	146	121	127	140	141	140	142	178	168	144	150	185	108
5.0 to 5.9	1344	1224	1201	1203	1515	1693	1712	2074	1768	1896	2209	2276	1401
4.0 to 4.9	8008	7991	8541	8462	10888	13917	12838	12078	12291	6805	10164	13315	9534
3.0 to 3.9	4827	6266	7068	7624	7932	9191	9990	9889	11735	2905	4341	2791	2453
2.0 to 2.9	3765	4164	6419	7727	6316	4636	4027	3597	3860	3014	4626	3643	3111
1.0 to 1.9	1026	944	1137	2506	1344	26	18	42	21	26	39	47	43
0.1 to 0.9	5	1	10	134	103	0	2	2	0	1	0	1	0
No Magnitude	3120	2807	2938	3608	2939	864	828	1807	1922	17	24	11	3
Total	22256	23534	27454	31419	31194	30478	29568	29685	31777	14825	21577	* 22289	* 16667
Estimated Deaths	231	21357	1685	33819	228802	88003	6605	712	88011	1790	320120	21953	768

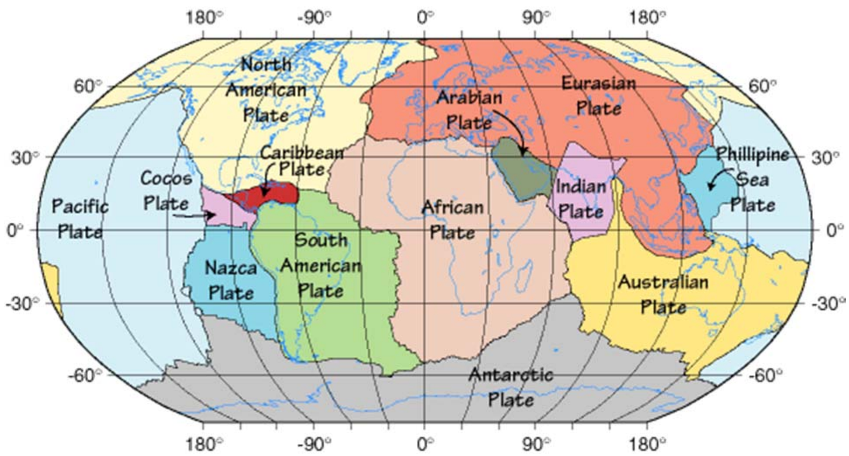
Major Earthquakes (2000-2010)

Popular name	Main countries affected	Date of event	Type of hazard	Total number of deaths	Total number of affected	Total damages US$
Japan earthquake	Japan	11/5/2011	EQ and tsunami	5,178	N/A	N/A
Haiti earthquake	Haiti	12/1/2010	EQ	222,570	3,400,000	N/A
Sichuan earthquake	China	12/5/2008	EQ	87,476	45,976,596	85 billion
Java earthquake	Indonesia	27/5/2006	EQ	5,778	3,177,923	3.1 billion
Kashmir earthquake	Pakistan	8/10/2005	EQ	73,338	5,128,000	5.2 billion
South Asian tsunami	Indonesia, Sri Lanka, India, Thailand, Malaysia, Maldives, Myanmar	26/12/2004	EQ and tsunami	226,408	2,321,700	9.2 billion
Bam earthquake	Iran	26/12/2003	EQ	26,796	267,628	500 million
Gujurat earthquake	India	26/1/2001	EQ	20,005	6,321,812	2.6 billion

Global Seismic Hazard Map



Major Tectonic Plates



Are Earthquakes Predictable?

- Many geophysicists believe that earthquake prediction is hopeless or plain wrong.
- prediction or forecasting must still have an important part to play in earthquake hazard mitigation: seismologists can and must predict how earthquakes can affect particular structures in specific locations.
- Answer the following:
  - When?
  - Where?
  - How big?
  - How often?

What's Earthquake Engineering?

- The application of Civil Engineering concepts to reduce/prevent life and economic losses due to earthquakes, i.e., mitigate seismic risk.
- Seismic Risk: the probability of losses occurring due to earthquakes within the design lifetime of a structure; these losses can include human lives, social and economic disruption as well as material damage.

SEISMIC RISK = SEISMIC HAZARD × VULNERABILITY

Earthquake Potential Damage

- Ground Shaking

Bhuj, India 2001

Loma Prieta, CA 1989

Earthquake Potential Damage

- Ground Shaking

Pakistan 2005

Earthquake Potential Damage

- Ground Shaking

Izmit , Turkey 1999

Earthquake Potential Damage

- Ground Shaking

Earthquake Potential Damage

- Ground Shaking
- Surface Rupture

Earthquake Potential Damage

- Ground Shaking
- Surface Rupture

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Earthquake Potential Damage

- Ground Shaking
- Surface Rupture

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Earthquake Potential Damage

- Ground Shaking
- Surface Rupture
- Landslides

Pakistan 2005

Earthquake Potential Damage

- Ground Shaking
- Surface Rupture
- Landslides
- Settlement & Liquefaction

Pakistan 2005

Earthquake Potential Damage

- Ground Shaking
- Surface Rupture
- Landslides
- Settlement & Liquefaction

Adapazari, Turkey 1999

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Earthquake Potential Damage

- Ground Shaking
- Surface Rupture
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Adapazari, Turkey 1999

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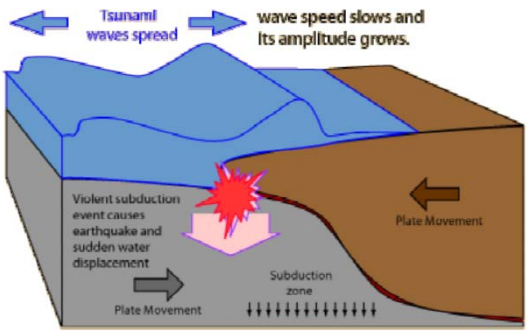
Izmit, Turkey 1999

Earthquake Potential Damage

- Ground Shaking
- Surface Rupture
- Landslides
- Settlement & Liquefaction
- Tsunami



What is Seismic Risk?

Probability of the occurrence of damage in a building when exposed to a particular earthquake effect. Determined by Man: Reduced Through Seismic Design

RISK = SEISMIC HAZARD × VULNERABILITY

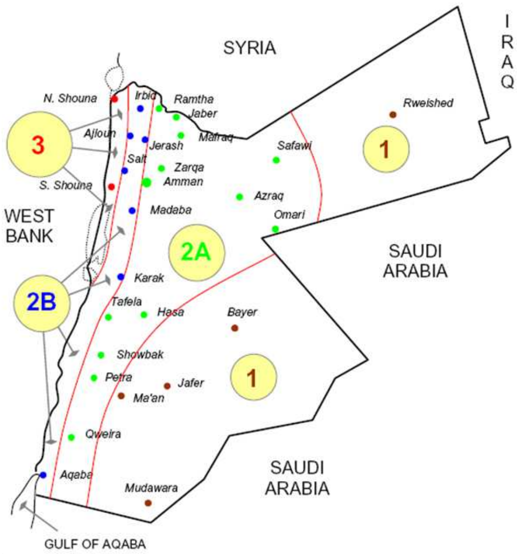
Probability of a potentially damaging earthquake effect occurring at the site of planned construction within its design life. Determined by Nature: Cannot be Reduced.

Design Sequence

- Engineers require seismic hazard assessment provided by seismologists.
- This assessment provides a description of the likely seismic loads (ground shaking) to be experienced by an engineering structure.
- Probabilities of occurrence are attached to these earthquake loads.
- Seismic loads are input into complicated dynamic inelastic time-history analysis to estimate possible damage to engineering structures.

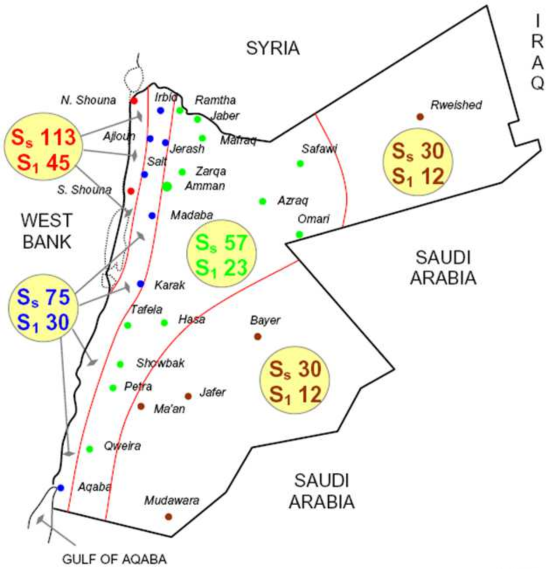
Code-type zonation map

- Official seismic zoning map of Jordan (2005)
- UBC'97 based map.
- Courtesy of N. Armouti



Code-type zonation map

- Tentative seismic zoning map of Jordan (2006)
- IBC based map.
- Courtesy of N. Armouti



Seismic Design Philosophy

- Modern building codes specify design EQs corresponding to a return period of 100 to 500 year for ordinary structures, such as offices.
- The corresponding design forces are too high, and if the structure is designed to behave elastically, the cost would be prohibitive.
- Thus, the structure is designed for a strength up to 15% ~ 25% of its elastic strength.
- The structure is expected to survive an earthquake by large inelastic deformations and energy dissipation corresponding to material distress.

Historical Perspective of EQ Eng.

Pre 1920	● Earthquakes not explicitly considered ● Structural survival tied to construction quality and sound engineering fundamentals
1920-1950	● Lateral forces explicitly calculated ● Analogous to lateral loading due to wind ● Lateral force corresponds to a level of 0.1g (10% of building weight) ● Deflections generally not considered in design
1950-1960	● Structural dynamics and response spectrum ● Significance of structural period on response

Historical Perspective of EQ Eng.

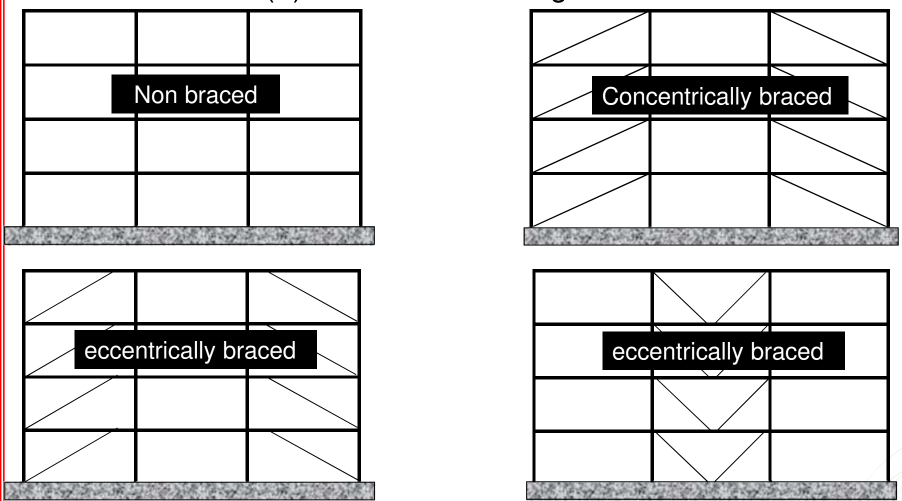
1960-1990	● Ductility concept was introduced ● Prescriptive ductility requirements introduced to building codes after 1971 San Fernando EQ ● Capacity design was introduced
1990-current	● Capacity design more widespread ● Importance of displacement as primary design parameter gained prominence ● SEAOC publishes vision 2000 in 1996 laying out framework for PBSE ● SEAOC publishes first 'Code-Based' PBSE guidelines in 1999 blue book

Simple Design Criteria

- **Choose an adequate load resisting system**
- Maintain regularity in plan and elevation and regularity of mass and stiffness distributions
- Ensure connection between structural elements
  - Anchoring of reinforcement
  - Use appropriate materials
- Avoid stress concentrations
- Avoid pounding between buildings
- Adopt capacity design to control failure mode
- Damage potential of non-structural elements

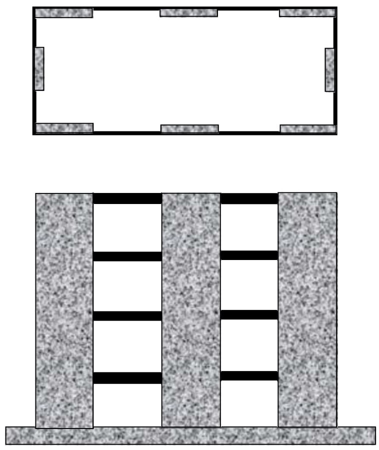
Load Resisting Systems

(1) Moment Resisting Frames

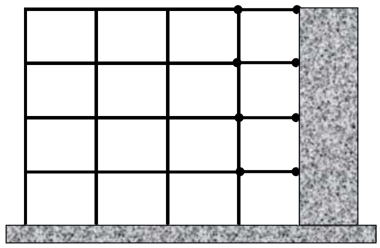


Load Resisting Systems

(2) Structural (Shear) Walls



(3) Duel (Hybrid) Systems



Simple Design Criteria

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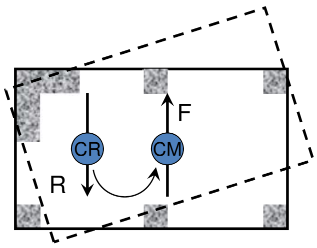
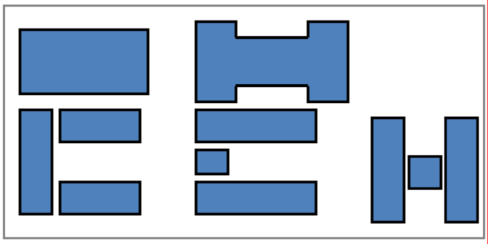
Plan Regularity

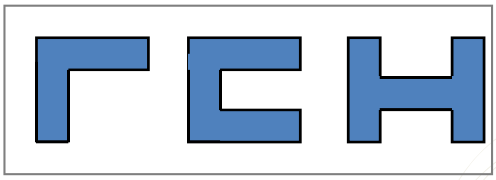


Torsional Vibration

Preferred Configurations

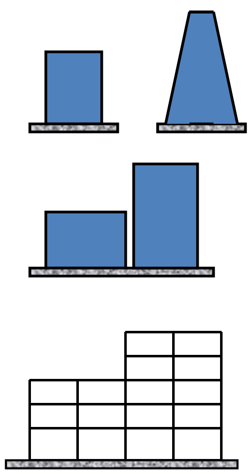


Undesirable Configurations

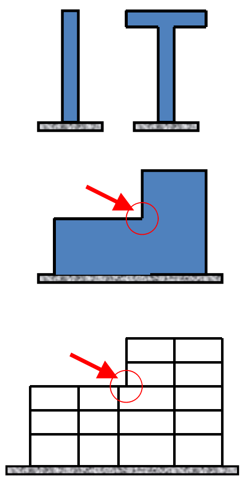


Elevation Regularity

Preferred Configurations

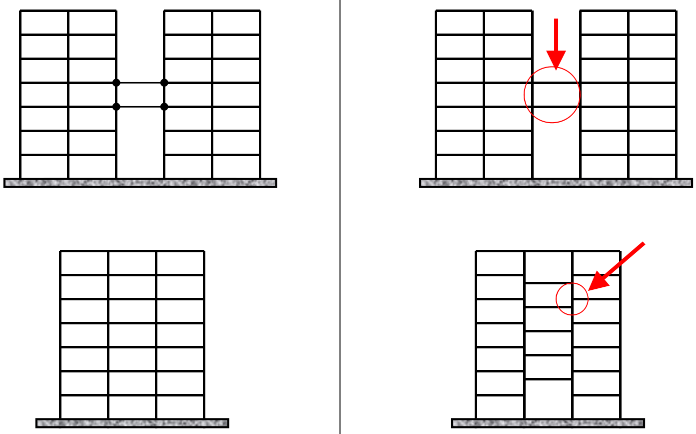


Undesirable Configurations



Elevation Regularity

Preferred Configurations | Undesirable Configurations



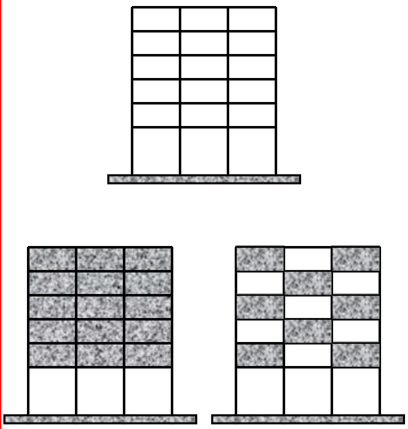
Regularity in Elevation

Pakistan 2005

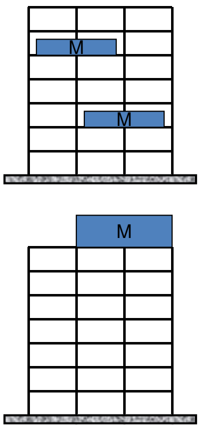
Regularity in Elevation

Elevation Regularity – Mass and Stiffness

Stiffness Irregularity



Mass Irregularity



Soft-Storey Mechanism

Soft-Storey Mechanism

Pakistan 2005

Soft-Storey Mechanism

Soft-Storey Mechanism

Soft-Storey Mechanism

Soft-Storey Mechanism

Simple Design Criteria

- Choose an adequate load resisting system
- Maintain regularity in plan and elevation and regularity of mass and stiffness distributions
- **Ensure connection between structural elements**
  - **Anchoring of reinforcement**
  - **Use appropriate materials**
- Avoid stress concentrations
- Avoid pounding between buildings
- Adopt capacity design to control failure mode
- Damage potential of non-structural elements

Poor Connection

Poor Connection

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Inappropriate Detailing

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Inappropriate Detailing

Inappropriate Detailing

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Simple Design Criteria

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- **Avoid stress concentrations**
- **Avoid pounding between buildings**
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Pounding

Simple Design Criteria

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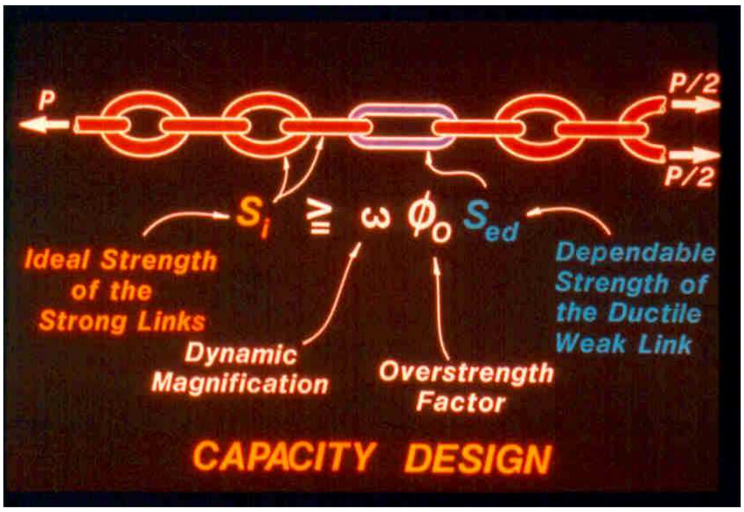
  - Use appropriate materials
- Avoid stress concentrations
- Avoid pounding between buildings
- **Adopt capacity design to control failure mode**
- Damage potential of non-structural elements

Capacity Design



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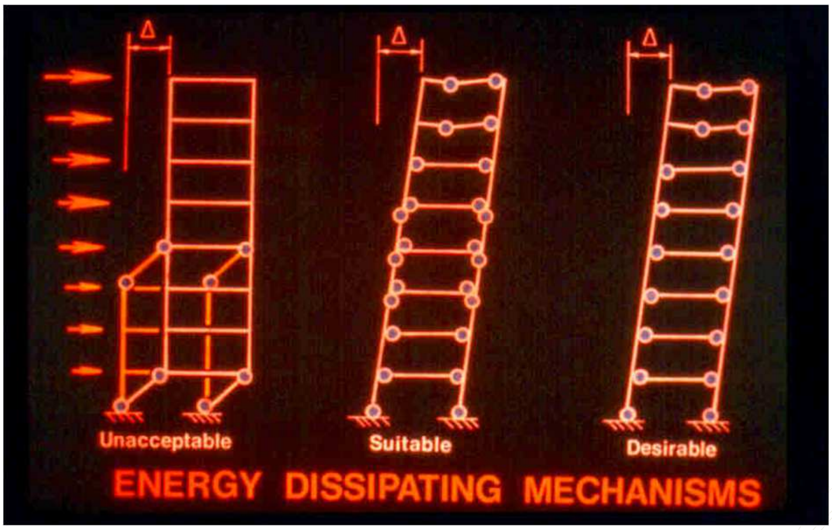
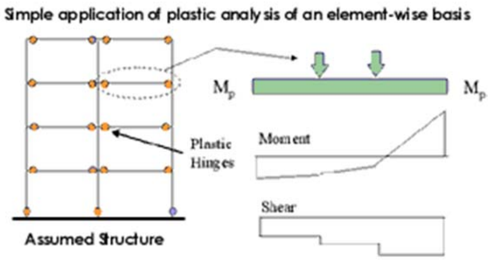
Capacity Design Mechanisms



Capacity Design Approach

- Explicitly considers determining the failure mechanism of members
- Force the member to fail in a ductile manner by making the capacity of the member in other possible failure modes greater
- Often, beams have significant overstrength in flexure which must be taken into account in order to determine what the actual plastic moments might be

Capacity Design Approach



- In the case of the beam shown at left:
- preferred ductile mode of failure is flexure, while brittle shear failure is to be avoided.
- The shear corresponding to the plastic moment in the beam is the design shear.
- The beam is then designed so that its nominal shear strength is greater than this shear.

Bad Design

Bad Design

Simple Design Criteria

- Choose an adequate load resisting system
- Maintain regularity in plan and elevation and regularity of mass and stiffness distributions
- Ensure connection between structural elements
  - Anchoring of reinforcement
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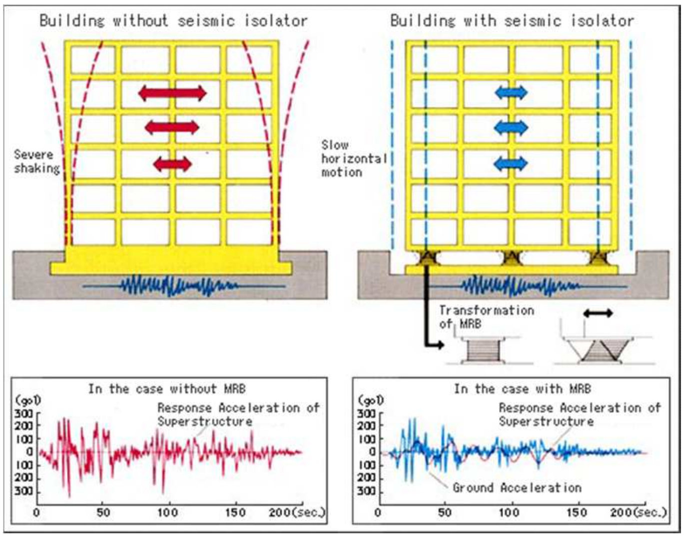
- Avoid stress concentrations
- Avoid pounding between buildings
- Adopt capacity design to control failure mode
- **Damage potential of non-structural elements**

Non-structural Elements

Base Isolation

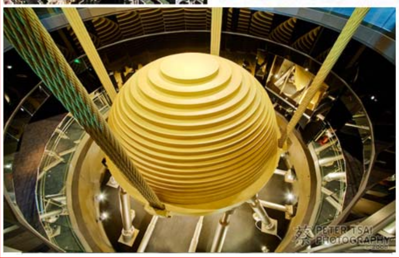


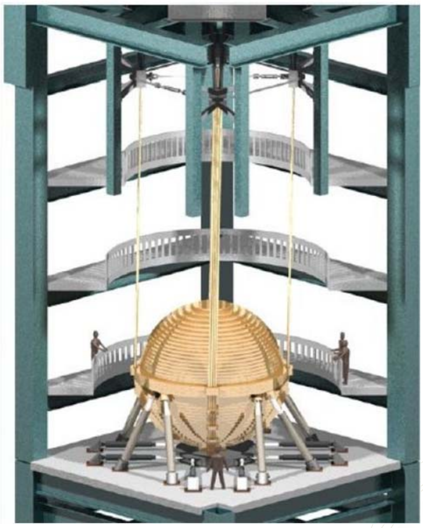
Base Isolation

Mechanical Dampers

Tuned-Mass Damper

Taipei, Taiwan (Wind Damper)





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Department of Civil Engineering

Introduction to Earthquake Engineering

Seismology

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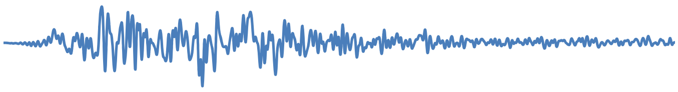
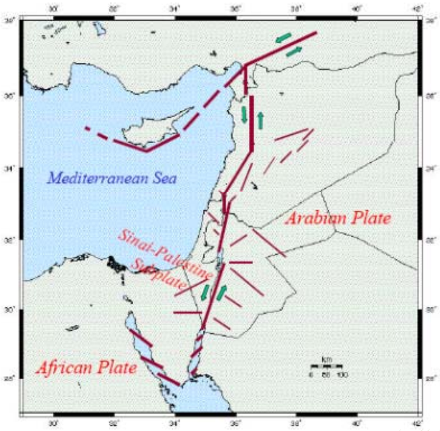


Introduction

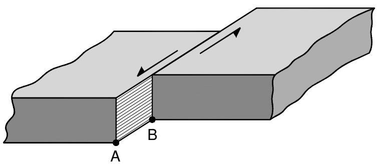
- Earthquakes may result of a number of natural and human-induced phenomena, including:
  - Meteoric impact
  - Volcanic activity
  - Underground nuclear explosions
  - Rock stress changes due to filling large human-made reservoirs
  - Relative deformations at the boundaries of crustal tectonic plates
- The vast majority of damaging EQs originate at or adjacent to the boundaries of tectonic plates.

Introduction

- Relative deformations between plates may reach several meters before faulting occur
- Typically, the boundaries of plates do not consist of simple single-fault surfaces.
- Its worth noting, that fault system maps are incomplete, with new faults being discovered continuously.

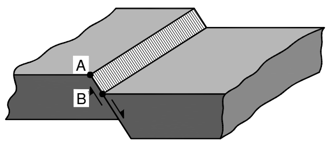


Fault Movement



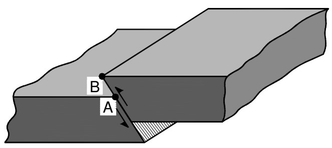
(a) Strike-slip fault

(left slip), AB = strike-slip



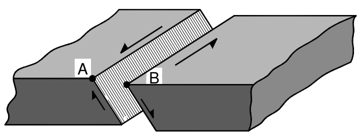
(b) Normal-slip fault

AB = dip-slip



(c) Reverse slip fault

(Thrust or Subduction)
AB = reverse slip



(d) Left-oblique-slip fault

AB = oblique slip

Fault Rupture (Chi

Chi EQ, Taiwan, 1999, M = 7.1)

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Terminology

- Hypocenter or focus or source: the rupture point within the earth crust.
- Epicenter: the point on the earth's surface above the hypocenter.
- Focal depth: distance between hypocenter and epicenter.
- Focal distance: distance between hypocenter and any reference point.

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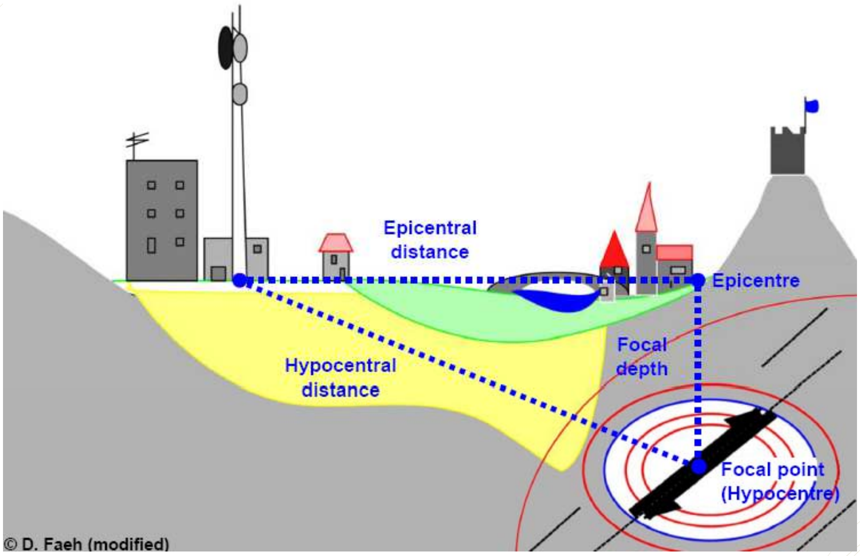


Seismic Waves

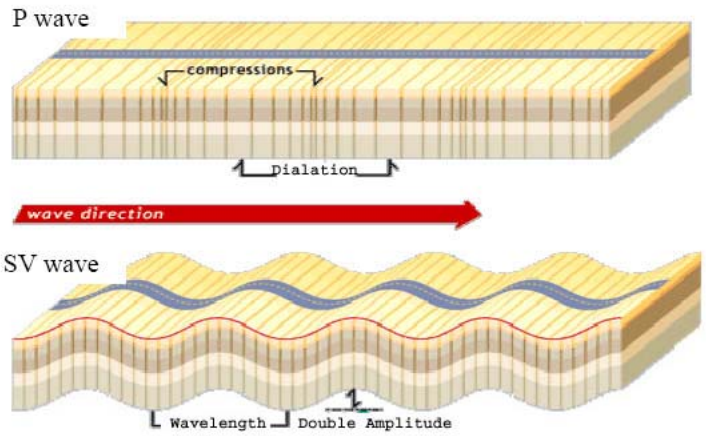
- Energy releases by earthquake propagates by different types of body waves:
  - P waves (primary or dilatation waves): arrive first which involve particle movement in the direction of propagation.

  P waves can travel though solids, liquids, and gases

  - S waves (secondary or shear wave) arrive second which involve particle movement perpendicular to the direction of propagation.

  S waves don't travel through liquids or gases since these don't have any shear strength.

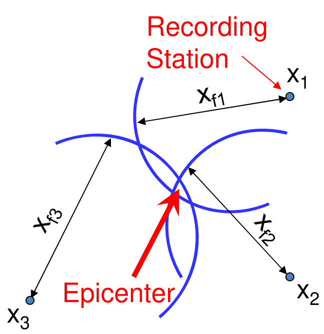
Body Waves



Locating Epicenter

- P and S waves have different velocities v_p and v_s. P and S waves at a given site are proportional to the focal distance x_f.
- Recording of the P-S time interval (ΔT)at three or more nonlinear stations enables the positioning of the epicenter.

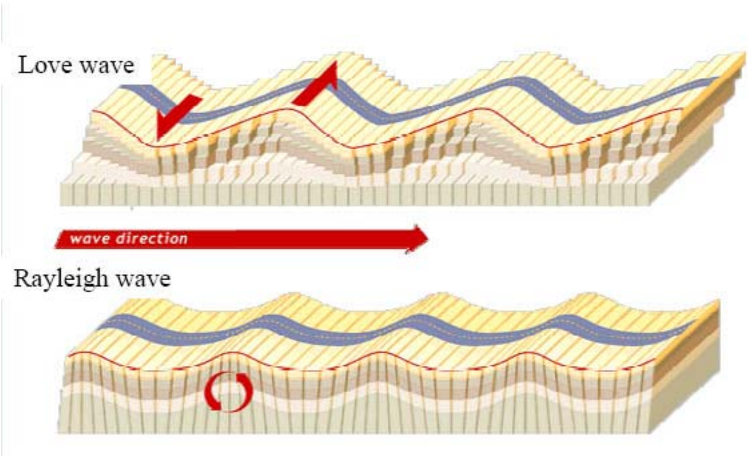
$$x_f = v_p \Delta T / (\sqrt{3} - 1)$$



Surface Waves

- When body waves reach the ground surface they reflect, but also generate surface waves which include:
  - Love waves (L waves): produce horizontal motion transverse to the direction of propagation.
  - Rayleigh waves (R waves): produce a circular motion analogous to the motion of ocean waves.
- In both cases, the amplitude of these waves reduces with depth from surface.
- Because surface wave velocities are low, they arrive after body waves.

Surface Waves



Magnitude and Intensity

- Magnitude: is a measure of the energy released during and earthquake, thus defines the size of the seismic event.
- Intensity: is a subjective assessment of the effect of the earthquake at a given location and is not directly related to magnitude.
- The accepted measure of magnitude is Richter scale. The magnitude (*M*) is related to the maximum trace deformation at a distance of 100 km from the epicenter.
- The accepted relationship between energy release (*E*) and Richter magnitude (*M*) is:

$$Log_{10}E = 11.4 + 1.5M$$

Magnitude and Potential Damage

Magnitude	Potential Damage
< M5	Rarely cause significant structural damage
M5 ~ M6	Cause structural damage over quite small area. 1986, M5.4, San Salvador EQ, caused damage over an area of = 100 km^2
M6 ~ M7	The area of potential damage is quite large. 1971 San Fernando EQ (M6.4) caused structural damage over an area = 2000 km^2
M7 ~ M8	Causes damage over an area of = 10,000 km^2 . Tangshan Earthquake, China, 1976.
> M8	*Great Earthquakes,* capable of causing damage over area of = 100,000 km^2. (Alaska 1964 & Chilea 1960)

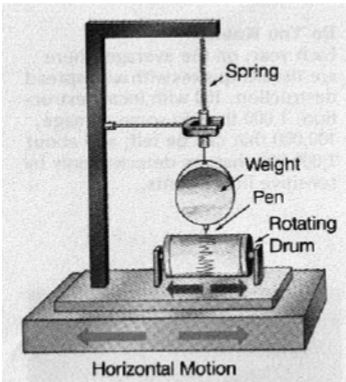
Mercalli Scale

- Acceptable measure of intensity is Mercalli scale.
- Developed by Mercalli in 1902 and modified by Wood and Neumann in 1931

MM I	Not felt
MM II	Felt by persons at rest on upper floors
MM III	Felt in doors. Hanging objects swing. Vibration like passing light trucks
MM IV	Hanging objects swing. Vibration like passing heavy trucks, …..
↓	↓
MM XII	Damage nearly total.

Seismometers

- Basic principle:
  – mass attached to a moveable frame, when frame is shaken by seismic waves the inertia of the mass causes it's motion to lag behind relative motion recorded on rotating drum, on magnetic tape or digitally
  – Mass is damped to prevent continued oscillation This limits the frequency response of the seismometer



Schematic of mechanical seismometer

Strong Motion Seismometer

- Designed to pickup strong, high-amplitude shaking close to quake source. Insensitive to weak shaking.
- Record horizontal and vertical ground accelerations
- These seismometers ground optical time-history record

Güralp Systems Ltd

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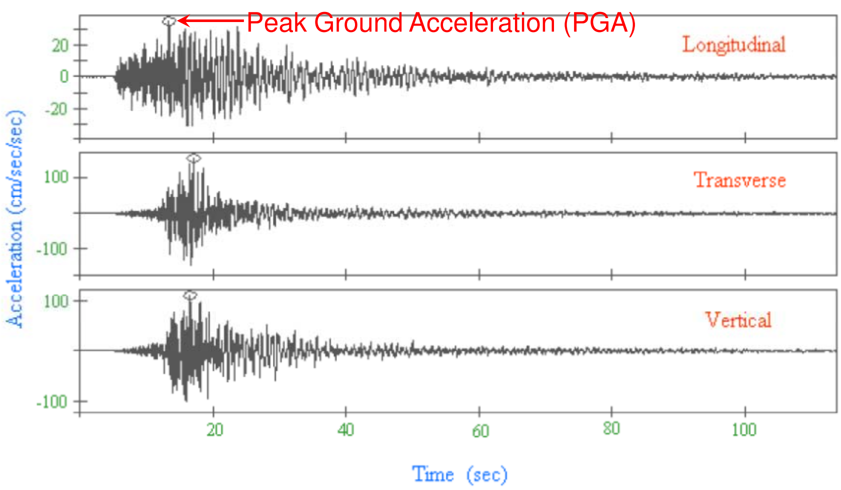
1995 Aqaba EQ Accelerations



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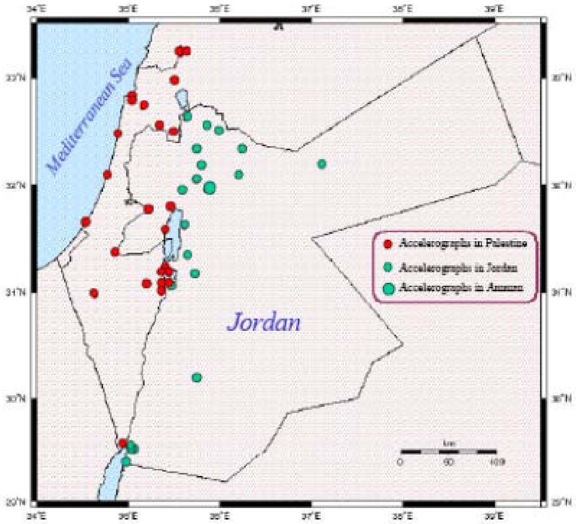
Accelerograph Stations in Jordan-Palestine Area



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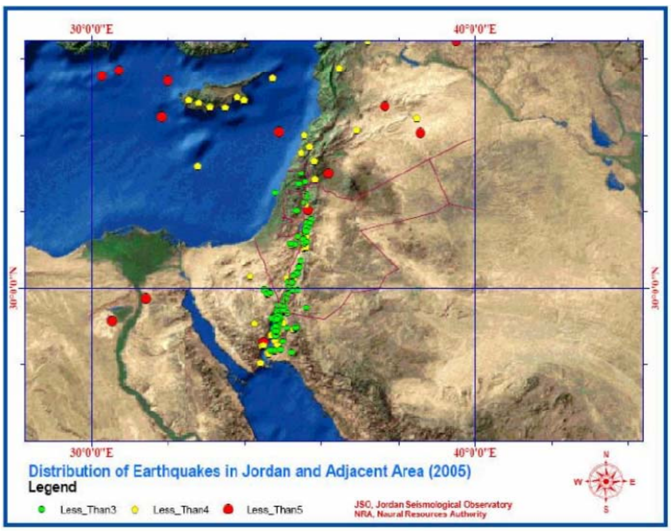
2005 Earthquakes



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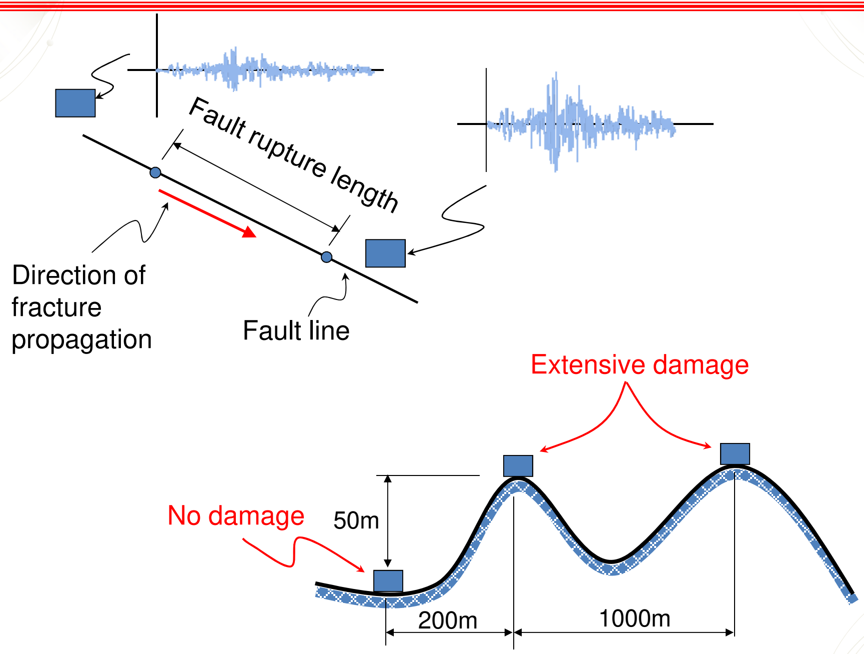
Remarks

- Influence of soil stiffness: soft soils tend to modify the characteristics of strong motion record by amplification in long period range.
- Directional effect: fracture initiates at a point and propagates in one or both directions. A building in the direction of propagation may experience enhanced peak accelerations.
- Geographical amplifications: steep ridges may amplify strong motion records.



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Introduction to Earthquake Engineering

Dynamics of Structures (Linear Analysis)

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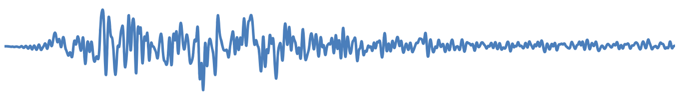
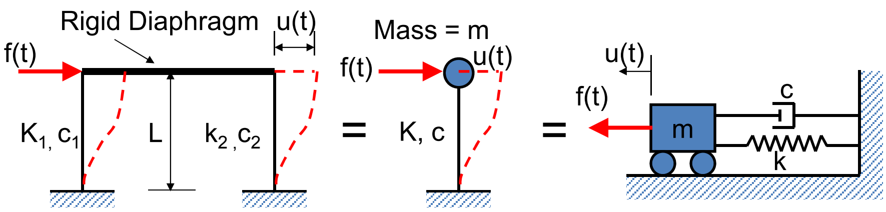


Free Vibration



Single-degree-of-freedom-oscillator (SDOF)

K_1 = stiffness of column (1) = $12EI_1/L^3$, $K = \sum k_i$, c = damping factor

Equation of Motion (Newton second law):

$$Inertia\ Forces + Damping\ Forces + Restoring\ Forces = Applied\ Forces$$

Forced Vibration :

$$m\ddot{u} + c\dot{u} + ku = f(t)$$

Free Vibration :

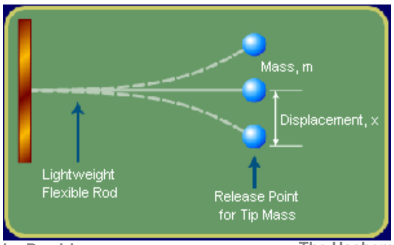
$$m\ddot{u} + c\dot{u} + ku = 0$$

$$\ddot{u} = \frac{d^2u}{dt^2}$$

$$\dot{u} = \frac{du}{dt}$$

Natural Frequency

- Natural frequency is the frequency at which a system naturally vibrates once it has been set into motion. So, natural frequency is the number of times a system will oscillate (move back and forth) between its original position and its displaced position, if there is no outside interference. For example, consider a simple beam fixed at one end and having a mass attached to its free end. If the beam tip is pulled downward, then released, the beam will oscillate at its natural frequency.



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Undamped Free Vibration

$$m\ddot{u} + ku = 0$$

$$u(0) = u_o \quad \& \quad \dot{u}(0) = v_o$$

$$u(t) = u_o \cos\omega_n t + \frac{v_o}{\omega_n}\sin\omega_n t$$

Where:

$$Natural\ circular\ frequency = \omega_n = \sqrt{k/m};(radians/sec)$$

$$Natural\ period = T_n = \frac{2\pi}{\omega_n};(\sec)$$

$$Natural\ cyclic\ frequency = f_n = \frac{1}{T_n} = \frac{\omega_n}{2\pi};(Hz)$$

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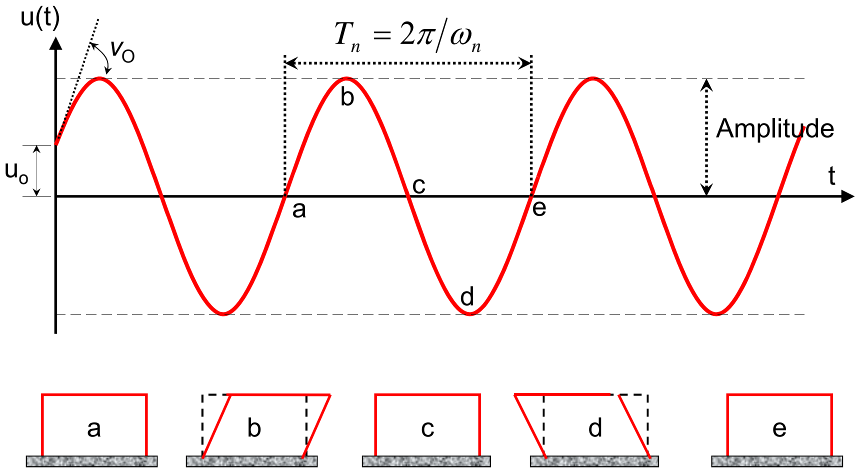
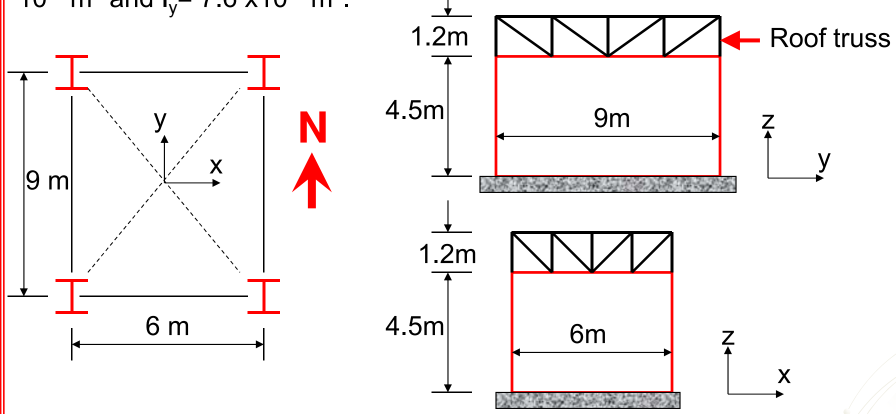
Undamped Free Vibration



Example 1.2, Chopra, pp. 16 (modified)

A small one storey industrial building 6m x 9m in plan. With moment frames in the N-S and E-W directions. The weight of the structure can be idealized as 1.5kN/m^2. All columns are W 8 x 24 sections where I_x = 34.3 x 10^{-6} m^4 and I_y= 7.6 x10^{-6} m^4.



$$The\ mass\ lumped\ at\ the\ roof\ \ is: m = \frac{w}{g} = \frac{6 \times 9 \times 1.5 \times 10^3}{9.81} = 8256.9 kg$$

(a) N - S Direction :

$$k_{N-S} = 4\left(\frac{12EI_x}{h^3}\right) = \frac{48 \times 200 \times 10^9 \times 34.3 \times 10^{-6}}{(4.5)^3} = 3.61 \times 10^6\ N/m$$

$$\omega_n = \sqrt{k/m} = \sqrt{3.61 \times 10^6 / 8256.9} \cong 21 rad/\sec$$

$$T_n = \frac{2\pi}{\omega_n} = 0.3\sec \quad ; \quad f_n = \frac{1}{T_n} = 3.33 Hz$$

(a) E - W Direction :

$$k_{E-W} = 4\left(\frac{12EI_y}{h^3}\right) = \frac{48 \times 200 \times 10^9 \times 7.6 \times 10^{-6}}{(4.5)^3} = 0.801 \times 10^6\ N/m$$

$$\omega_n = \sqrt{k/m} = \sqrt{0.801 \times 10^6 / 8256.9} \cong 9.85 rad/\sec$$

$$T_n = \frac{2\pi}{\omega_n} = 0.64\sec \quad ; \quad f_n = \frac{1}{T_n} = 1.56 Hz$$

Viscous Damped Free Vibration

$m\ddot{u} + c\dot{u} + ku = 0; divide\ by\ m:$

$\ddot{u} + 2\xi\omega_n\dot{u} + \omega_n^2 u = 0$

$where: \xi = damping\ ratio$

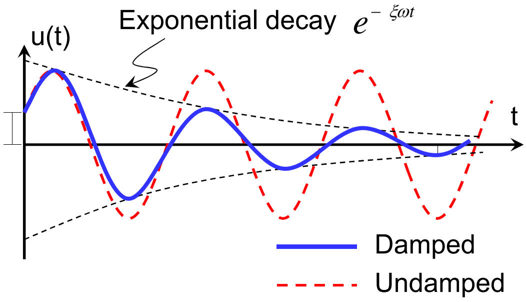
$$\xi = \frac{c}{2m\omega_n} = \frac{c}{c_{cr}}$$

$c_{cr} = 2m\omega_n = 2\sqrt{km} = 2k/\omega_n$

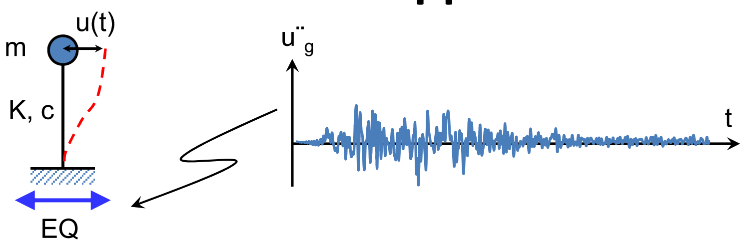
For $\xi < 1$:

$$u(t) = e^{-\xi\omega_n t}[u_o \cos\omega_D t + \frac{v_o + \xi\omega_n u_o}{\omega_D}\sin\omega_D t]$$

$$\omega_D = Damped\ Frequency = \omega_n\sqrt{1-\xi^2}$$



Forced Vibration – Support Excitation



$$m\ddot{u}_{tot} + c\dot{u} + ku = 0$$

$$m(\ddot{u} + \ddot{u}_g) + c\dot{u} + ku = 0$$

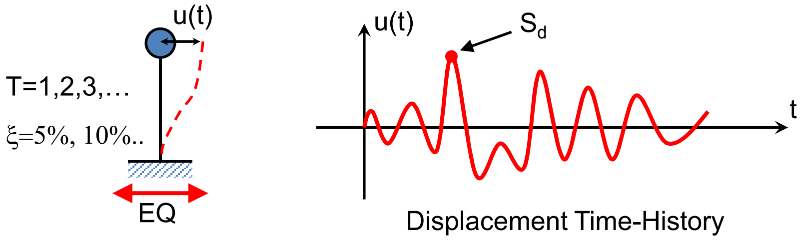
$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g$$

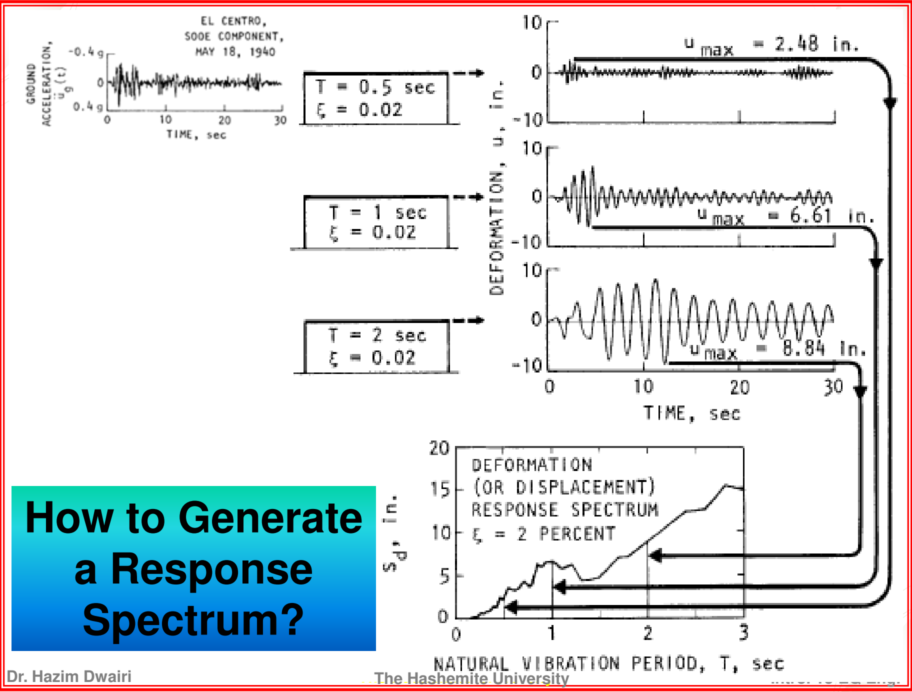
$$\ddot{u} + 2\xi\omega_n\dot{u} + \omega_n^2 u = -\ddot{u}_g$$

Solution for this equation is evaluated numerically. Methods like Newmark's or central difference are typically used.

Response Spectrum

- Solve the equation of motion for a specific EQ record for a range of SDOF oscillators with various periods, (say, 1 s to 5 s), and range of damping ratios (say 0% to 20%)
- Determine the absolute maximum displacement for each oscillator and plot it versus the period.





How to Generate a Response Spectrum?

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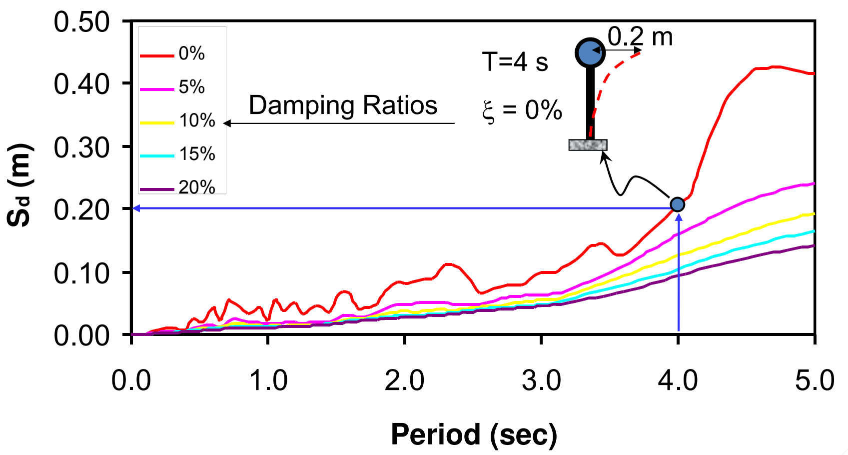
Displacement Spectra – Tabas EQ, Iran 1976



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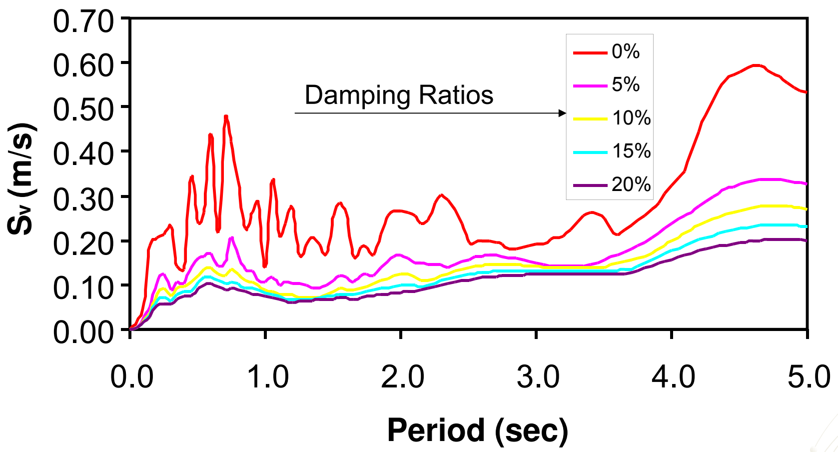
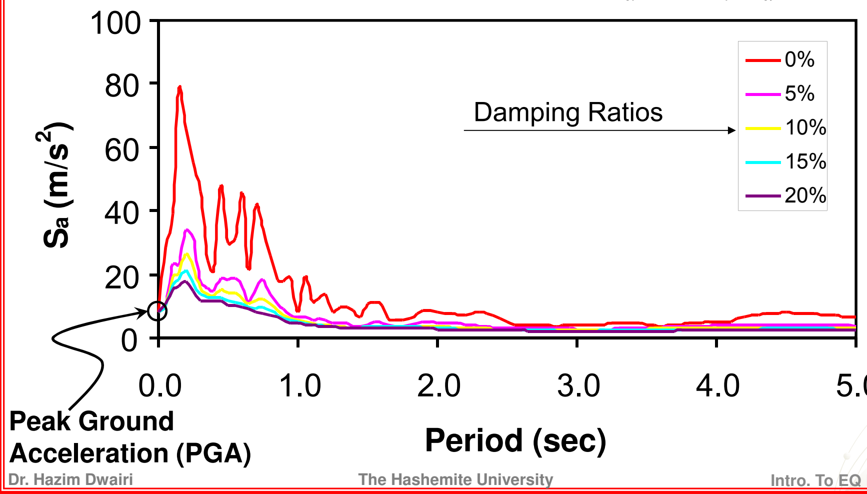
Velocity Spectra – Tabas EQ, Iran 1976

$$pseudo-velocity = S_v = \omega_n S_d$$

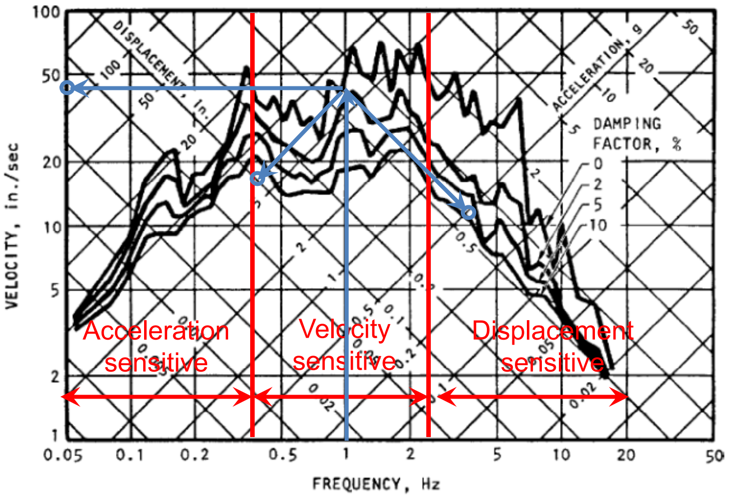


Acceleration Spectra – Tabas EQ, Iran 1976

$$pseudo-acceleration = S_a = \omega_n^2 S_d$$



Four-way logarithmic plot of El-centro, 1940, response spectra



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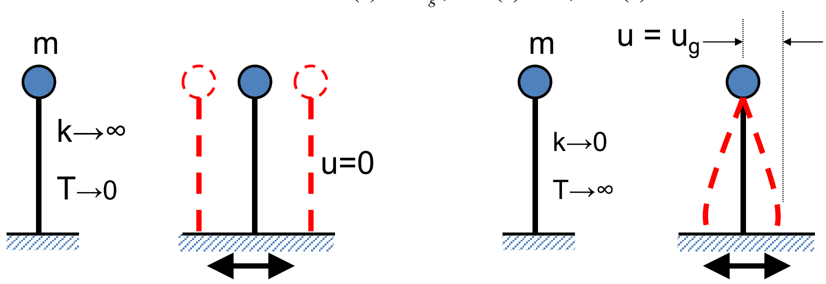
Limiting Elastic Response

- As T_n tends to zero, i.e., very-short-period or extremely rigid: $u(t) \approx 0; \quad \dot{u}(t) = 0; \quad \ddot{u}(t) = \ddot{u}_g$
- As T_n tend to infinity, i.e., very-long-period or extremely flexible: $u(t) \approx u_g; \quad \dot{u}(t) = 0; \quad \ddot{u}(t) = 0$



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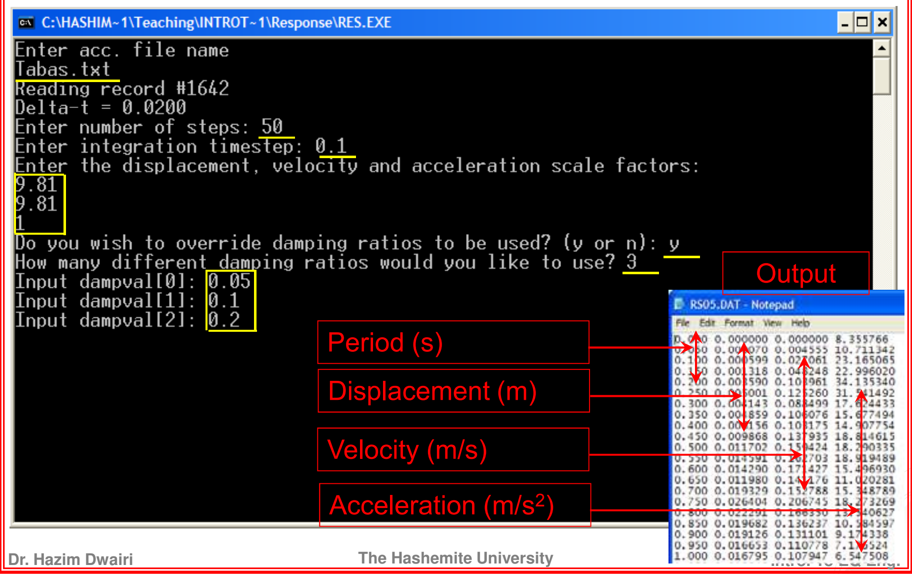
"RES" Program Example



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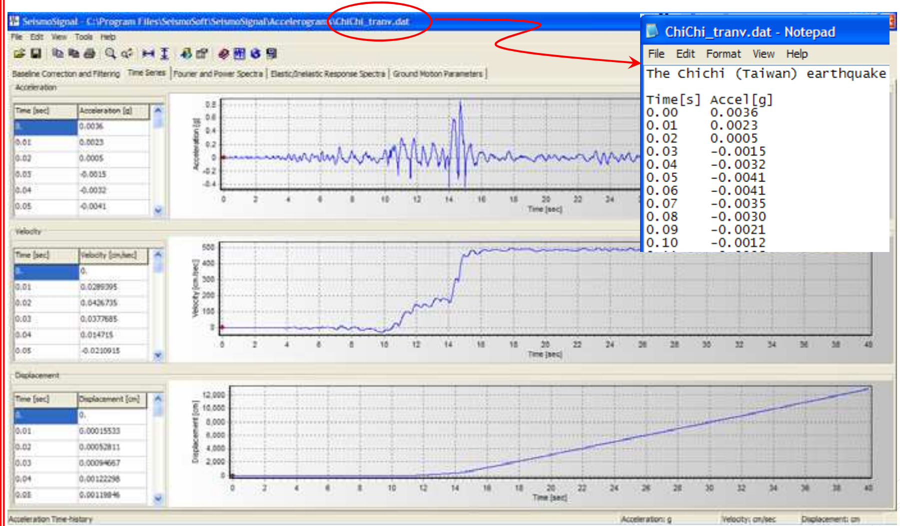
SeismoSignal Software Example



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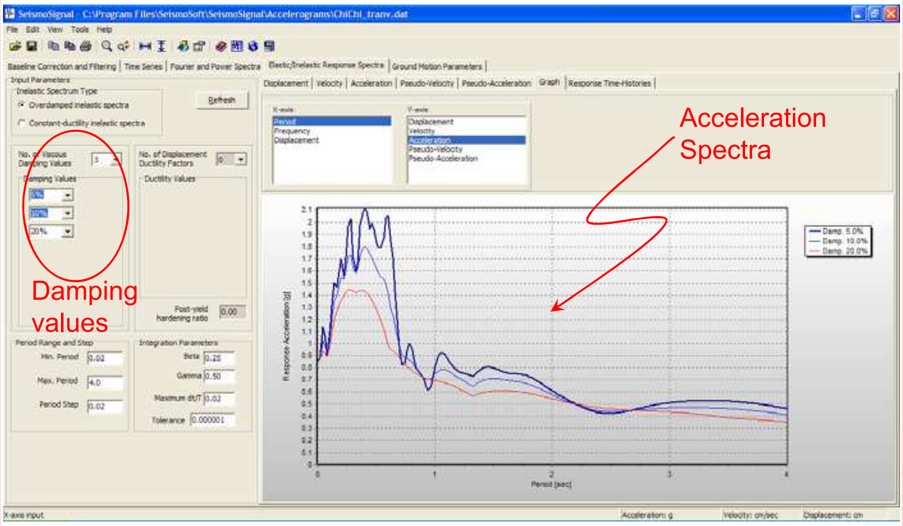
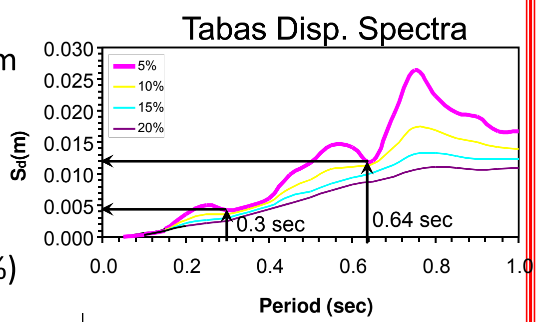
SeismoSignal Software Example



Example

- Determine the maximum displacement and base shear for the industrial building in the previous example if excited by Tabas record. (let ξ = 5%)



N-S Direction:

T_n=0.3 sec → S_d = 4mm

$V_{max} = k \times S_d = (3.61 \times 10^6)\times(0.004)$

= 14.44 kN

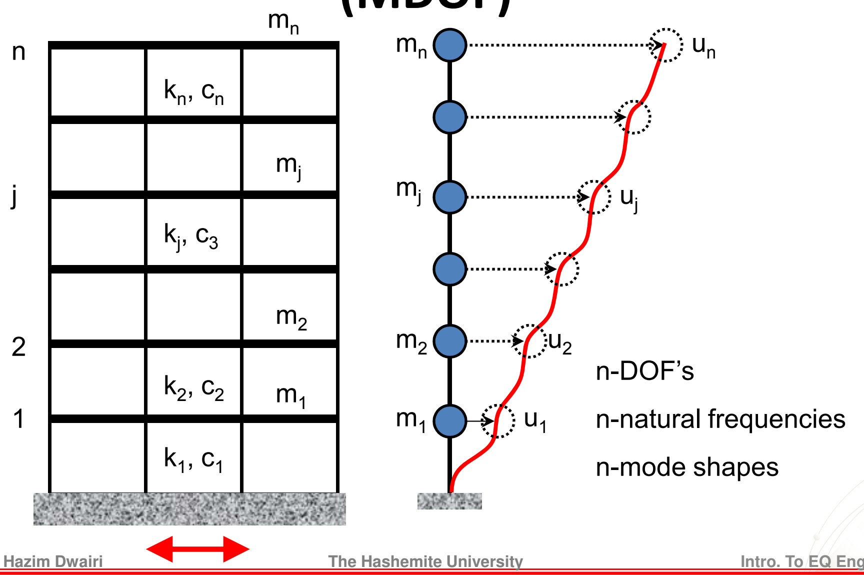
E-W Direction:

T_n=0.64 sec → S_d = 12mm

$V_{max} = k \times S_d = (0.801 \times 10^6)\times(0.012)$

= 9.61 kN

Multi-degree-of-freedom system (MDOF)



Equation of Motion: $M\ddot{U} + C\dot{U} + KU = -M\{1\}\ddot{u}_g$

For n=3, i.e., 3 degree of freedom system

$$\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix}\begin{bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \end{bmatrix} + \begin{bmatrix} c_{11} & c_{12} & c_{13} \\ c_{21} & c_{22} & c_{23} \\ c_{31} & c_{32} & c_{33} \end{bmatrix}\begin{bmatrix} \dot{u}_1 \\ \dot{u}_2 \\ \dot{u}_3 \end{bmatrix} + \begin{bmatrix} k_{11} & k_{12} & k_{13} \\ k_{21} & k_{22} & k_{23} \\ k_{31} & k_{32} & k_{33} \end{bmatrix}\begin{bmatrix} u_1 \\ u_2 \\ u_3 \end{bmatrix} = -\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix}\begin{bmatrix} 1 \\ 1 \\ 1 \end{bmatrix}\ddot{u}_g$$

Lumped mass matrix — Damping matrix — Stiffness matrix

Free Vibration Solution:

Let: $U = \Phi \sin \omega t$

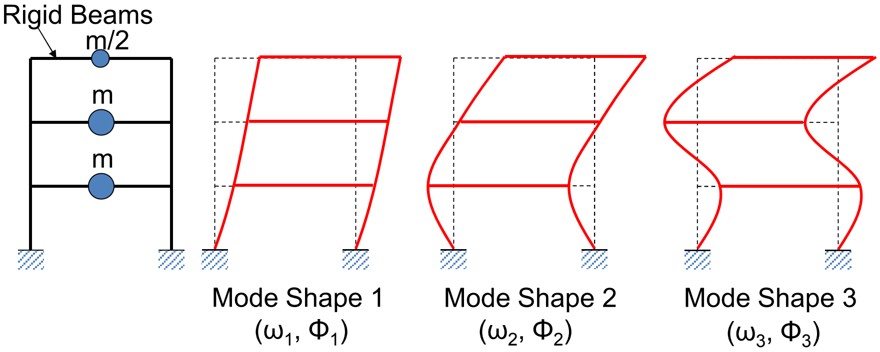
$[K - \omega^2 M]\Phi = 0 .... Eq.(1)$

Frequency Equation: $|K - \omega^2 M| = 0$

Kramer's rule:

Set Determinant = 0

The solution for the eignevalue problem yields 'n' natural frequencies and 'n' mode shapes (ϕ). Each frequency is associated with its mode shape. The Lowest frequency is called the fundamental frequency.



Total response of the structure is a combination of all mode shapes with different percentages. About 90% to 95% of the response is contributed to by mode shape 1.

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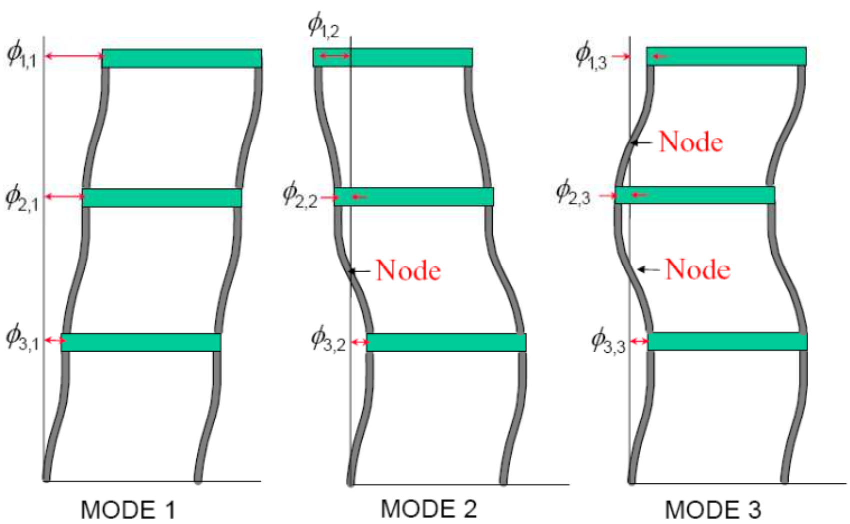
Mode Shapes for 3-DOF Structure



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Modal Analysis

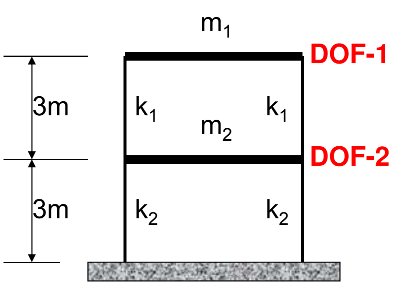
- Formulate the eigenvalue problem and solve to obtain natural frequencies (ω_n) and mode shapes (ϕ_n)
- For each of the natural periods $T_n=2\pi/\omega$ enter the response spectra and obtain the maximum modal responses, i.e., S_d, S_v, and S_a.
- The modal displacement are: $\{U_{n,\max}\} = \{\phi_n\}\Gamma_n S_{dn}$
- The displacement yielded from n-mode shapes are combined to give total response using the square root of the square of sums rule:

$$\{U_{max}\} = \sqrt{\sum_{1}^{n}\{U^2_{n,max}\}}$$

- Where Γ is the modal participation factor $\Gamma = \dfrac{\{\phi_n\}^T[M]\{R\}}{\{\phi_n\}^T[M]\{\phi_n\}}$

Modal Analysis Example

- The frame shown has very stiff beams, which can be idealized as 2-DOF. If the structure is excited by Kocaeli Earthquake, find the maximum displacements, max. storey forces, and max. base shear and base moment using the SRSS.



m_1 = 1 N.s^2/m

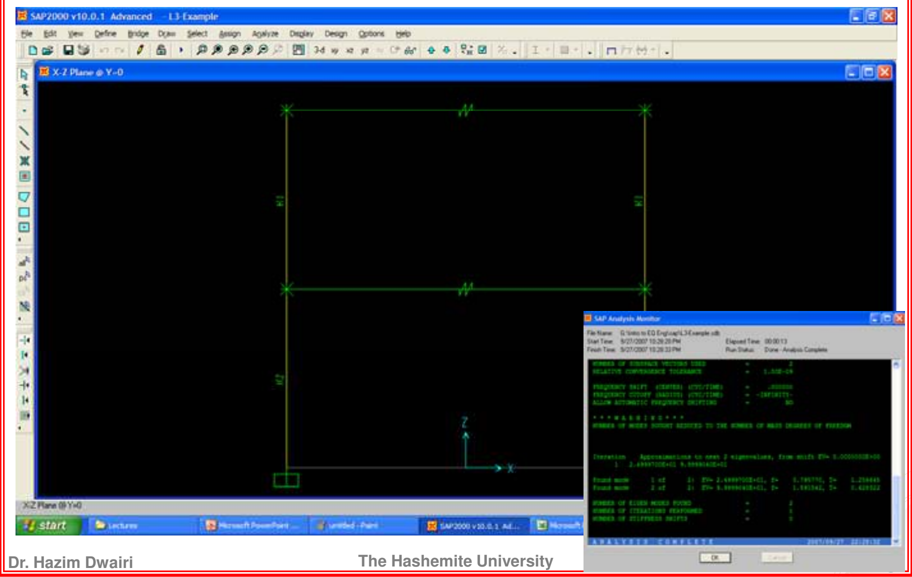
m_2 = 2 N.s^2/m

$k_1 = 12EI/L^3$ = 25 N/m

$k_2 = 12EI/L^3$ = 50 N/m

Use ξ = 5% for all modes

SAP2000 – modal analysis



SAP2000 – output

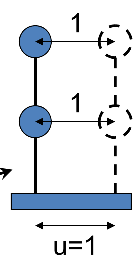
- Natural frequencies and mode shapes

$$T_{n1} = 1.26s \quad \phi_1 = \begin{Bmatrix} 0.8165 \\ 0.4083 \end{Bmatrix} \quad \phi_2 = \begin{Bmatrix} 0.5574 \\ -0.5574 \end{Bmatrix}$$
$$T_{n2} = 0.62s$$

$$normalized\ mode\ shapes: \phi_1 = \begin{Bmatrix} 1 \\ 0.5 \end{Bmatrix} \quad ; \quad \phi_2 = \begin{Bmatrix} 1 \\ -1 \end{Bmatrix}$$

- Lumped Mass Matrix = $\begin{bmatrix} 1 & 0 \\ 0 & 2 \end{bmatrix}$
- Modal participation factors:

$$\Gamma = \frac{\{\phi_n\}^T [M] \{R\}}{\{\phi_n\}^T [M] \{\phi_n\}}$$



Mode (1):

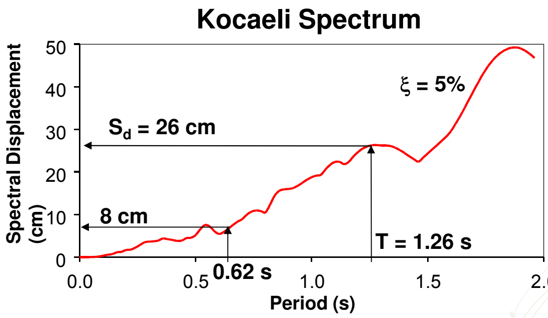
$$T_1 = 1.26s \quad ; \quad \xi_1 = 5\%$$

$$\psi_1 = \{\phi_1\}^T[M]\{R\} = \{1 \;\; 0.5\}\begin{bmatrix}1 & 0\\0 & 2\end{bmatrix}\begin{Bmatrix}1\\1\end{Bmatrix} = 2$$

$$m_1^* = \{\phi_1\}^T[M]\{\phi_1\} = \{1 \;\; 0.5\}\begin{bmatrix}1 & 0\\0 & 2\end{bmatrix}\begin{Bmatrix}1\\0.5\end{Bmatrix} = 1.5$$

$$\Gamma_1 = \psi_1 / m_1^* = 2/1.5 = 1.333$$

Enter the displacement spectra for T = 1.26 s, and ξ = 5%, then S_d = 26 cm.



$S_a = \omega^2 S_d = (2\pi/T_1)^2 S_d = (2\pi/1.26)^2 (0.26m) = 6.465 \text{ m/s}^2$

Hence, the modal responses are:

$$\{U_1\} = \{\phi_1\}\Gamma_1 S_{d1} = \begin{Bmatrix}u_{11}\\u_{21}\end{Bmatrix} = \begin{Bmatrix}1\\0.5\end{Bmatrix}(1.333)(260mm) = \begin{Bmatrix}347mm\\173mm\end{Bmatrix}$$

$$\{F_1\} = [M]\{\phi_1\}\Gamma_1 S_{a1} = \begin{Bmatrix}f_{11}\\f_{21}\end{Bmatrix} = \begin{bmatrix}1 & 0\\0 & 2\end{bmatrix}\begin{Bmatrix}1\\0.5\end{Bmatrix}(1.333)(6.465) = \begin{Bmatrix}8.62N\\8.62N\end{Bmatrix}$$

Mode (2):

$$T_2 = 0.62s \quad ; \quad \xi_2 = 5\%$$

$$\psi_2 = \{\phi_2\}^T[M]\{R\} = \{1 \;\; \text{-}1\}\begin{bmatrix}1 & 0\\0 & 2\end{bmatrix}\begin{Bmatrix}1\\1\end{Bmatrix} = -1$$

$$m_2^* = \{\phi_2\}^T[M]\{\phi_2\} = \{1 \;\; \text{-}1\}\begin{bmatrix}1 & 0\\0 & 2\end{bmatrix}\begin{Bmatrix}1\\-1\end{Bmatrix} = 3$$

$$\Gamma_2 = \psi_2 / m_2^* = -1/3 = -0.333$$

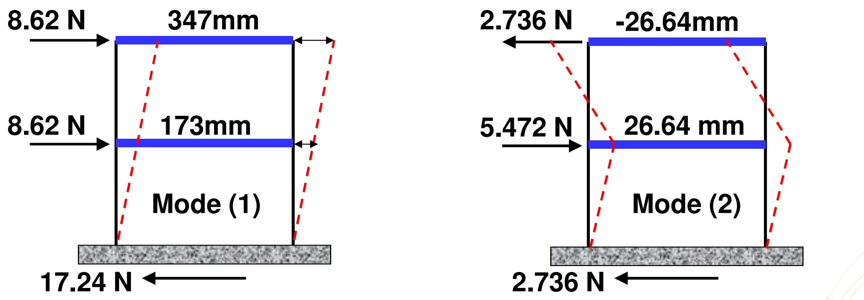
Enter the spectra for T = 0.62 s, and ξ = 5%, then S_d = 8 cm.

$S_a = \omega^2 S_d = (2\pi/T_2)^2 S_d = (2\pi/0.62)^2$ (0.08m) = 8.216 m/s²

Hence, the modal responses are:

$$\{U_2\} = \{\phi_2\}\Gamma_2 S_{d2} = \begin{Bmatrix} u_{12} \\ u_{22} \end{Bmatrix} = \begin{Bmatrix} 1 \\ -1 \end{Bmatrix}(-0.333)(80mm) = \begin{Bmatrix} -26.64mm \\ +26.64mm \end{Bmatrix}$$

$$\{F_2\} = [M]\{\phi_2\}\Gamma_2 S_{a2} = \begin{Bmatrix} f_{12} \\ f_{22} \end{Bmatrix} = \begin{bmatrix} 1 & 0 \\ 0 & 2 \end{bmatrix}\begin{Bmatrix} 1 \\ -1 \end{Bmatrix}(-0.333)(8.216) = \begin{Bmatrix} -2.736N \\ 5.472N \end{Bmatrix}$$



$$V_{1,\max} = 8.62 + 8.62 = 17.24N$$

$$V_{2,\max} = 5.472 - 2.736 = 2.736N$$

$$M_{1,\max} = 8.62(6) + 8.62(3) = 77.58N.m$$

$$M_{2,\max} = 5.472(3) - 2.736(6) = 0$$

Finally, the total responses are:

$$u_{1,\max} = \sqrt{(347)^2 + (-26.64)^2} = 348mm$$

$$u_{2,\max} = \sqrt{(173)^2 + (26.64)^2} = 175mm$$

$$V_{\max} = \sqrt{(17.24)^2 + (2.736)^2} = 17.45N$$

$$M_{\max} = \sqrt{(77.58)^2 + (0)^2} = 77.58N.m$$

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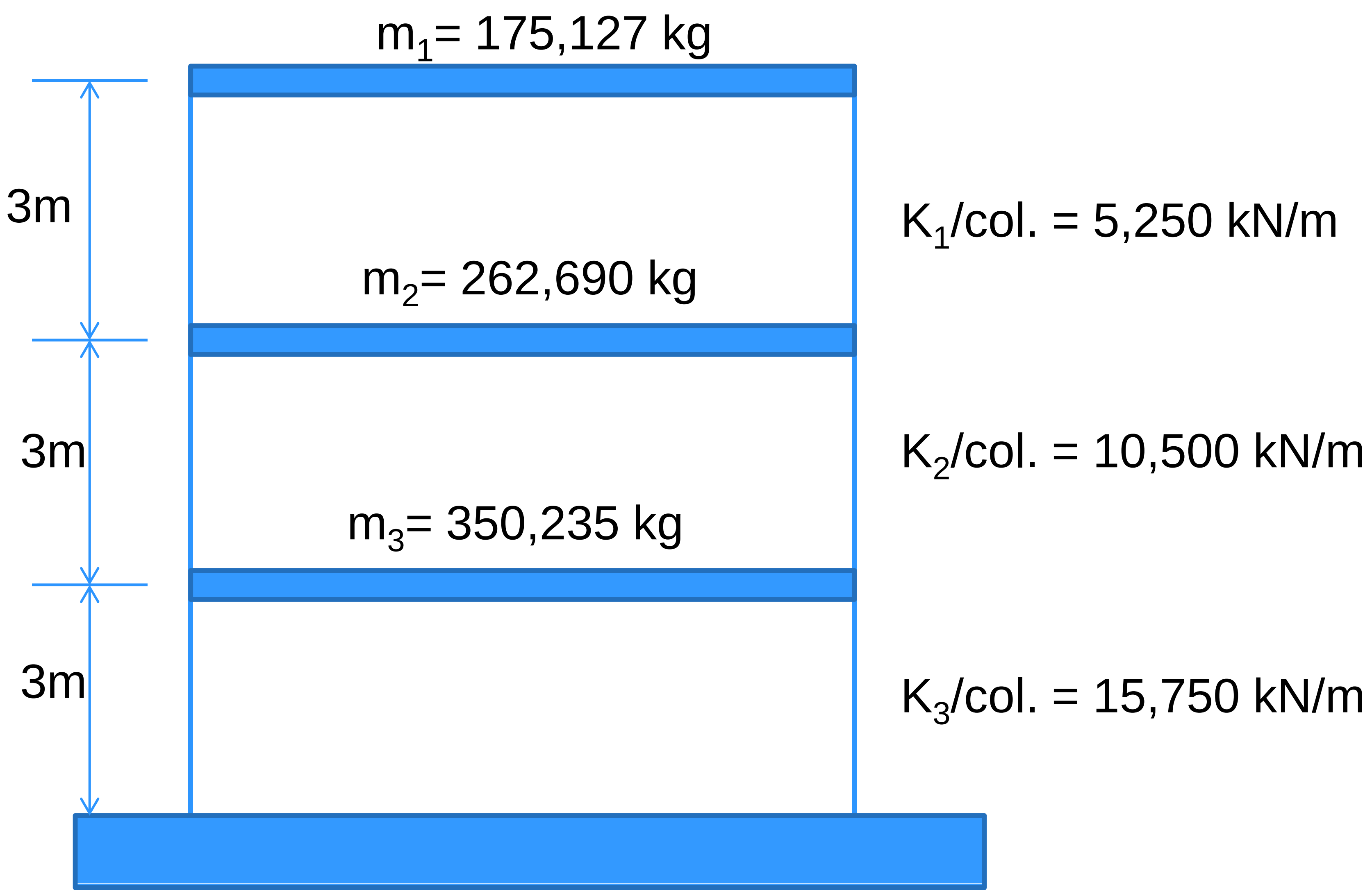
Modal Analysis Homework

Dr. Hazim Dwairi

Problem Statement

- A three story shear building can be idealized as 3-DOF structure. If the structure is excited by Tabas earthquake which took place in Iran 1976. Determine the maximum displacement for each story and maximum base shear using modal analysis and SRSS combination rule. (Use 5% viscous damping for all mode shapes)

Frame Building



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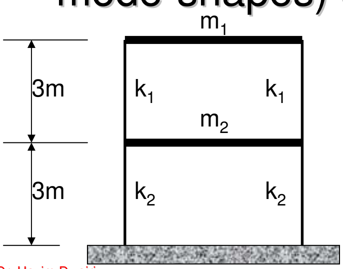
SAP2000 V12 – Modal Analysis Example

Dr. Hazim Dwairi

Modal Analysis Example

- The frame shown has very stiff beams, which can be idealized as 2-DOF. Determine the modal properties of the structure (i.e. natural frequencies and mode-shapes) using SAP2000 software



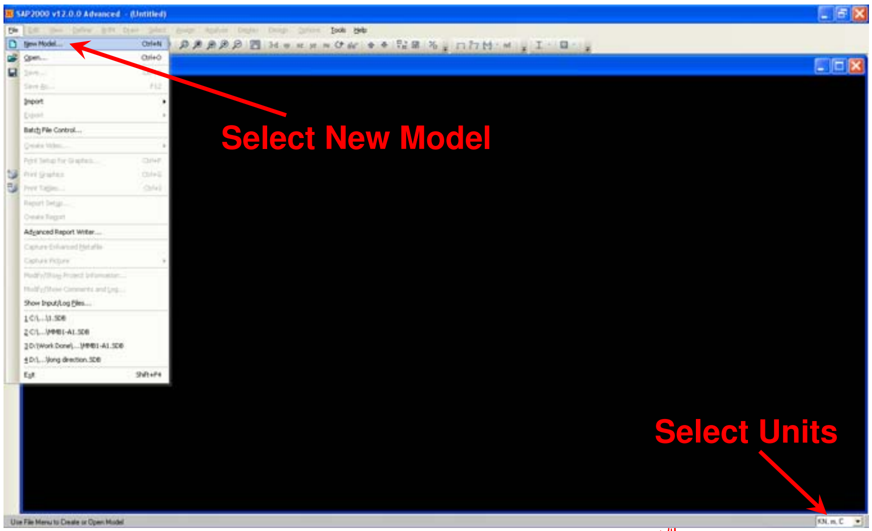
$m_1 = 1\ N.s^2/m$

$m_2 = 2\ N.s^2/m$

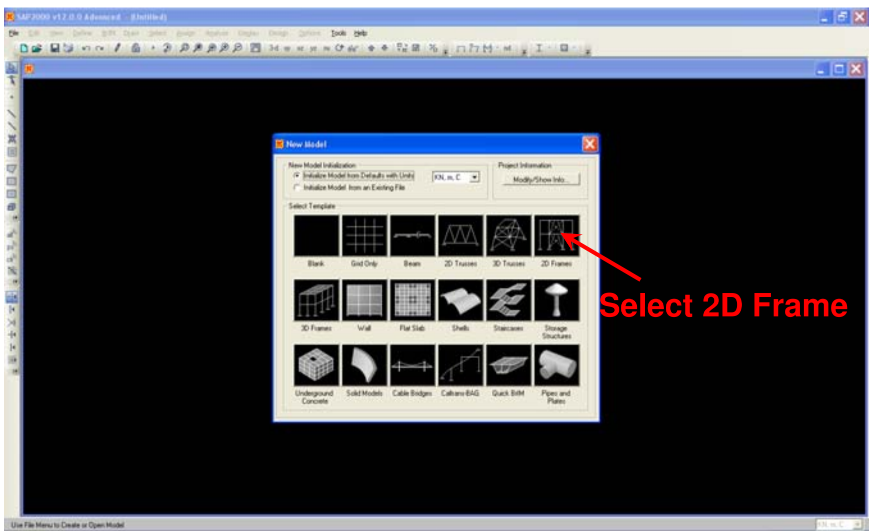
$k_1 = 12EI/L^3 = 25\ N/m$

$k_2 = 12EI/L^3 = 50\ N/m$

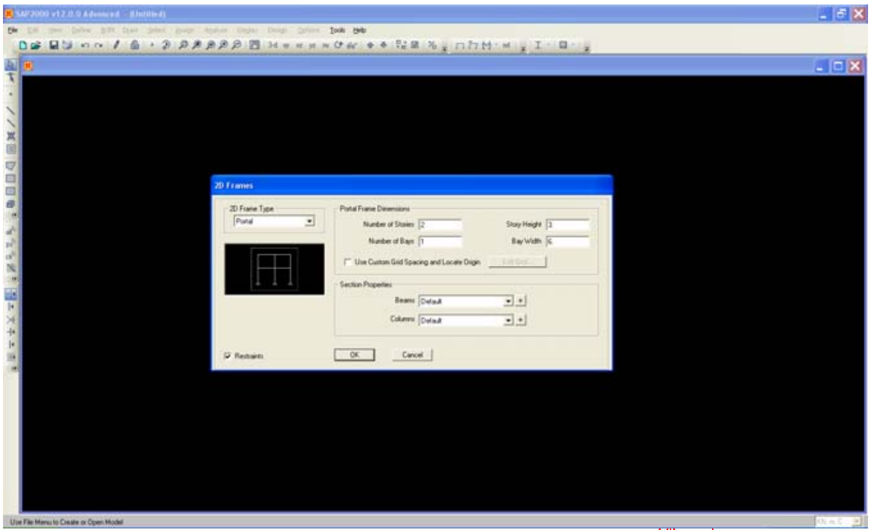
Create a new model



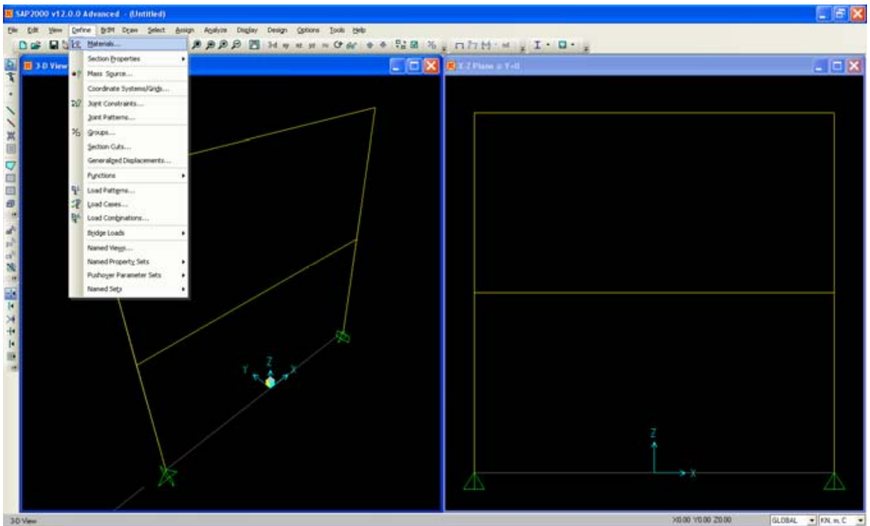
Select 2D Frames



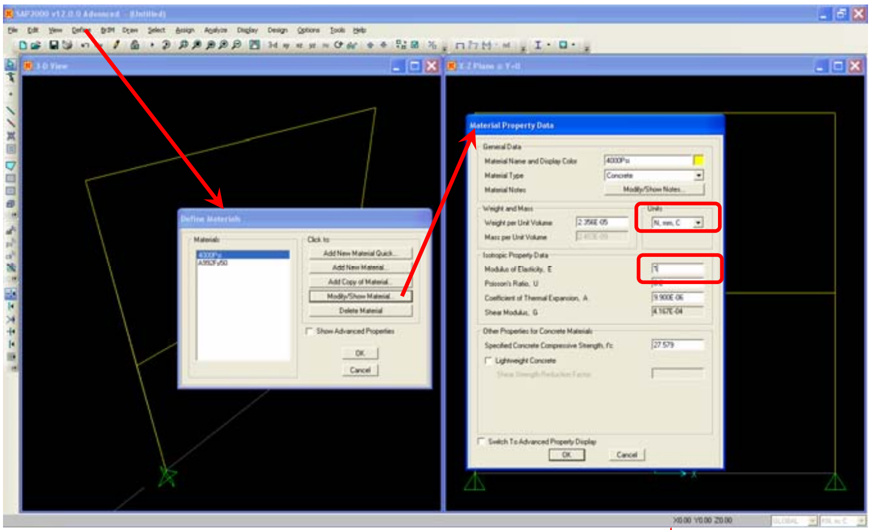
Input Geometry Data



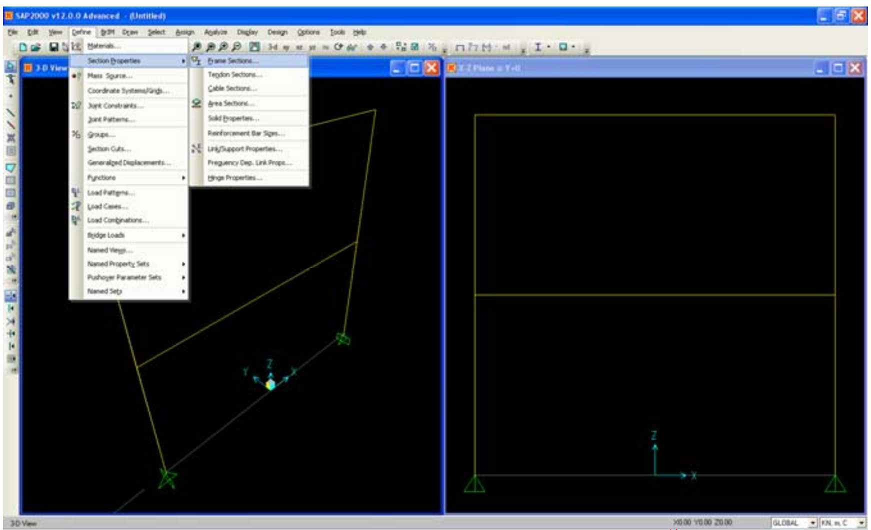
Define Material



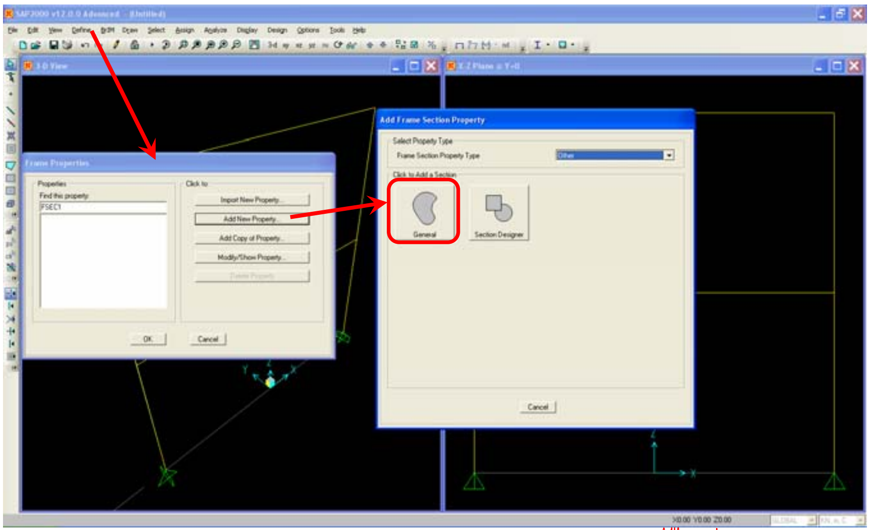
Define Material



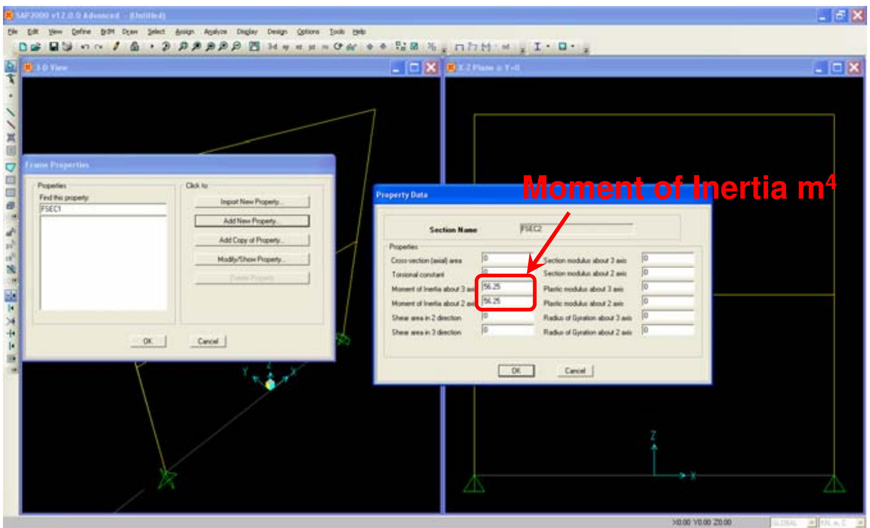
Define Frame Section 1



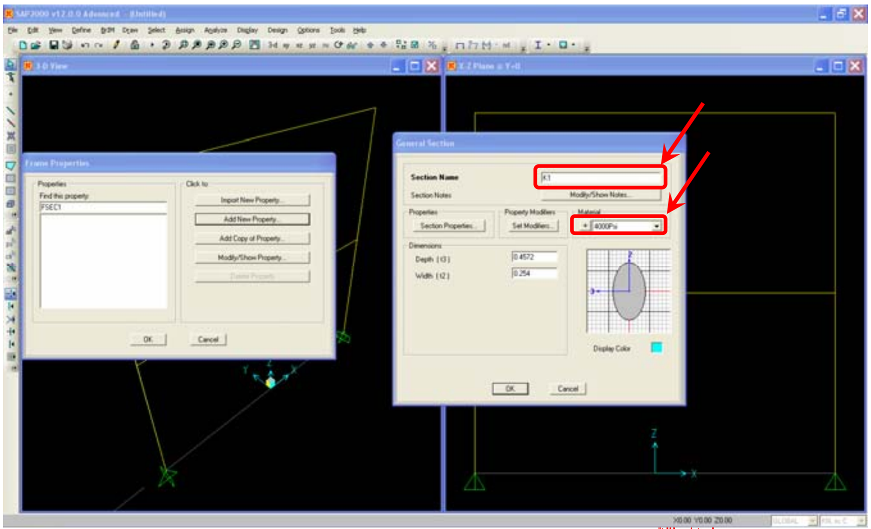
Add General Section



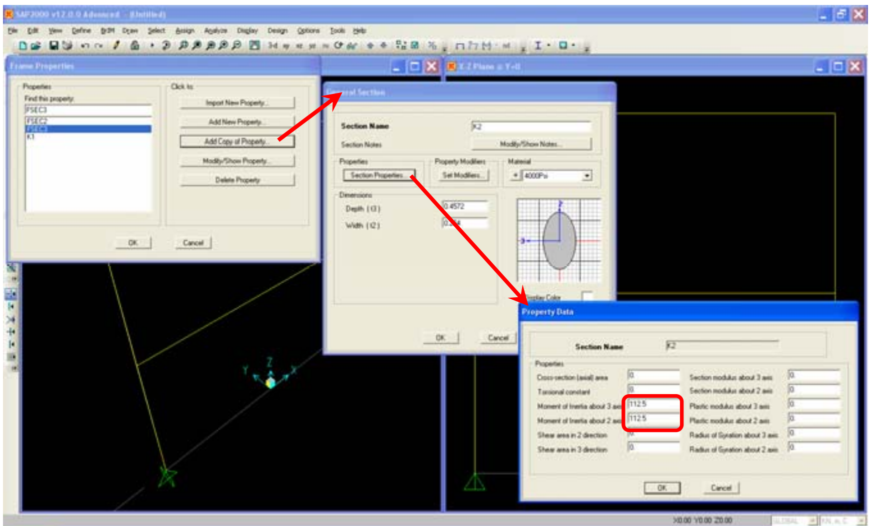
Input Moment of Inertia



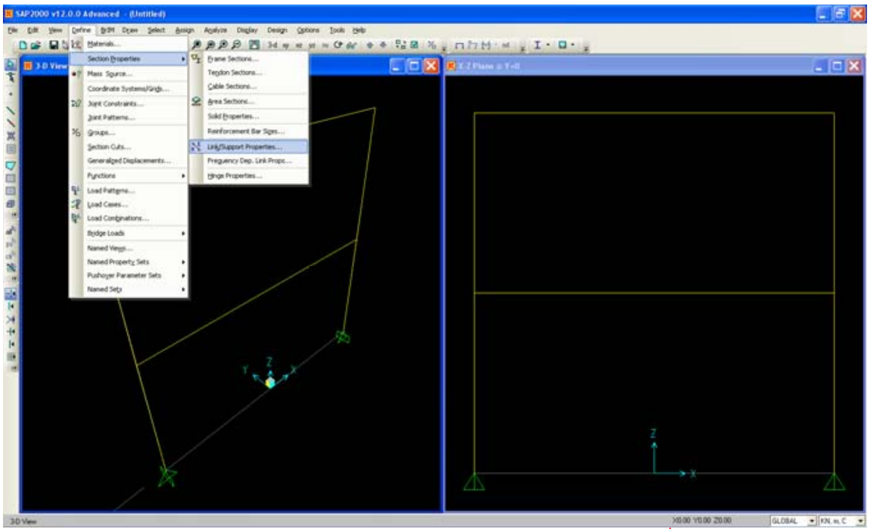
Change Section Name & Material



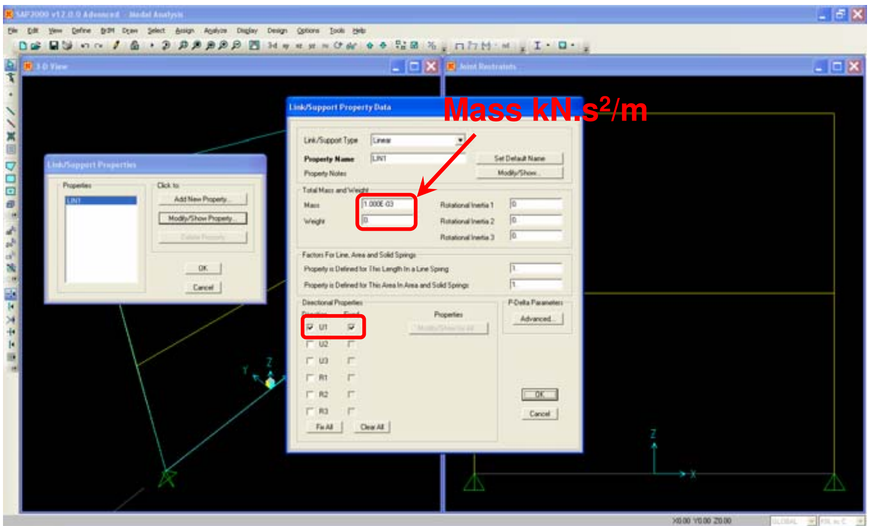
Define Frame Section 2



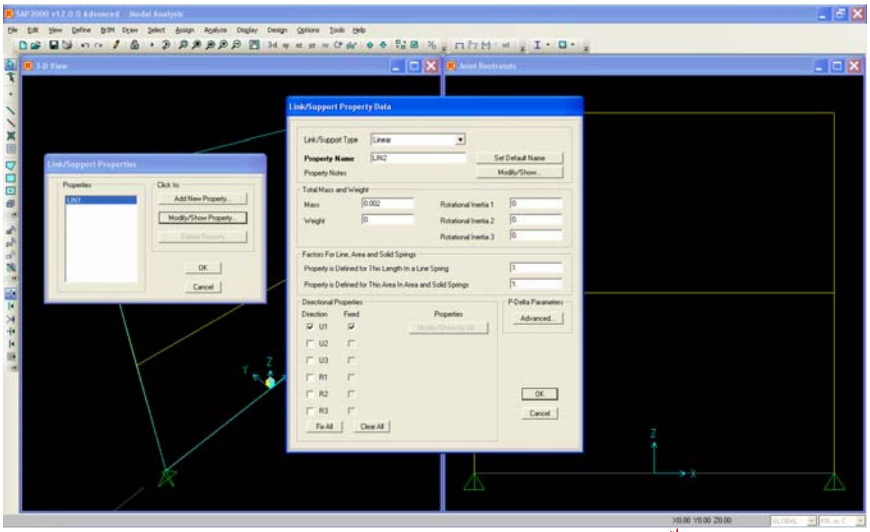
Define Link Section 1



Add New Link 1

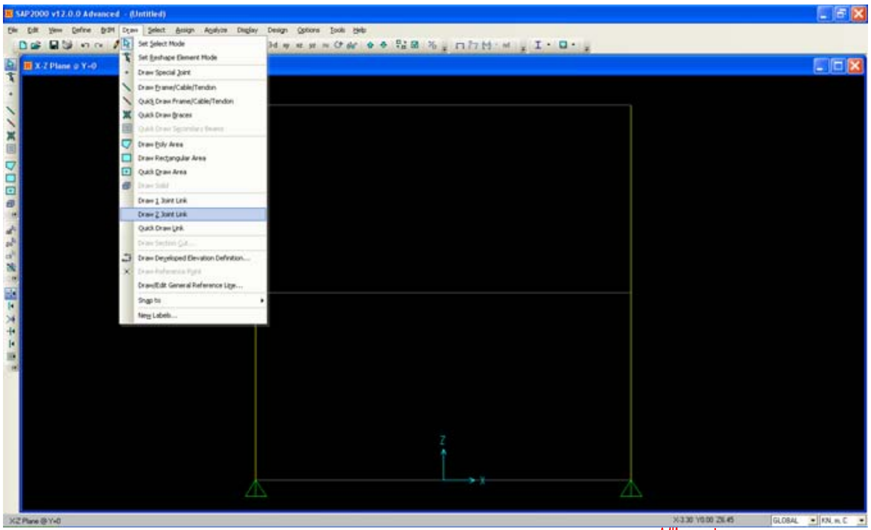


Add Link 2

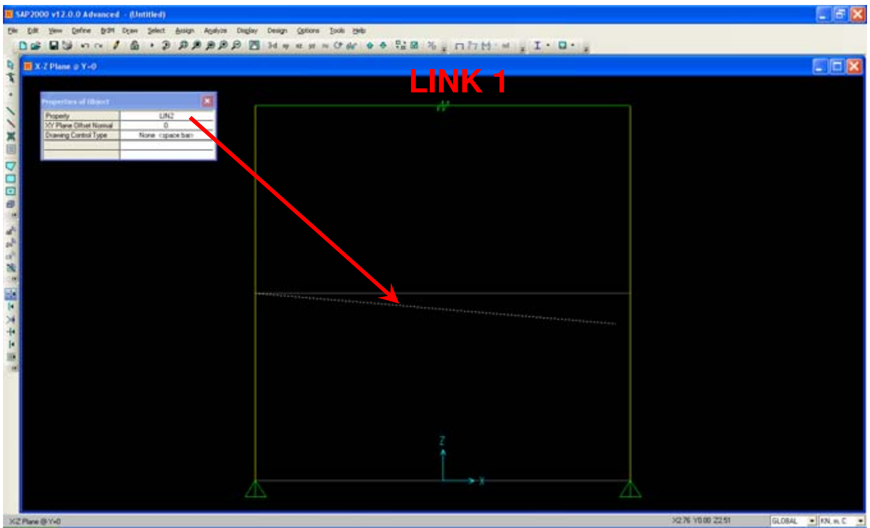


Delete Beam Elements

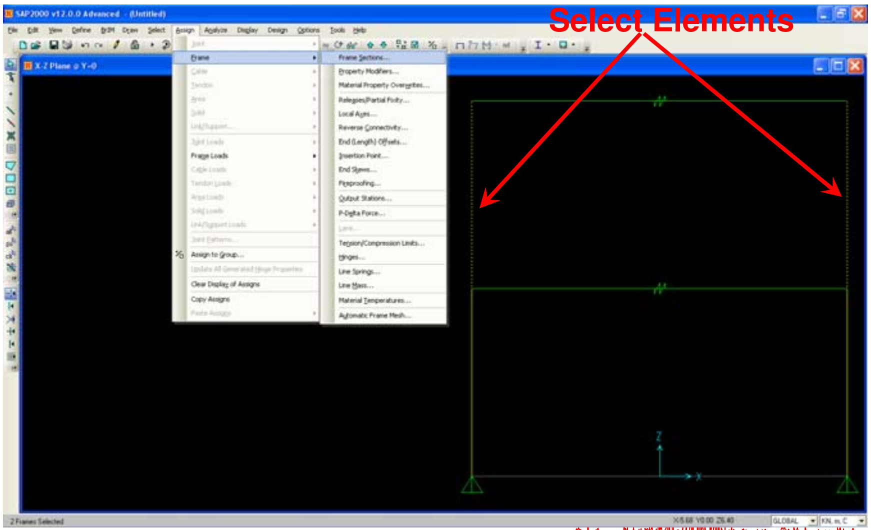
Draw Link Elements



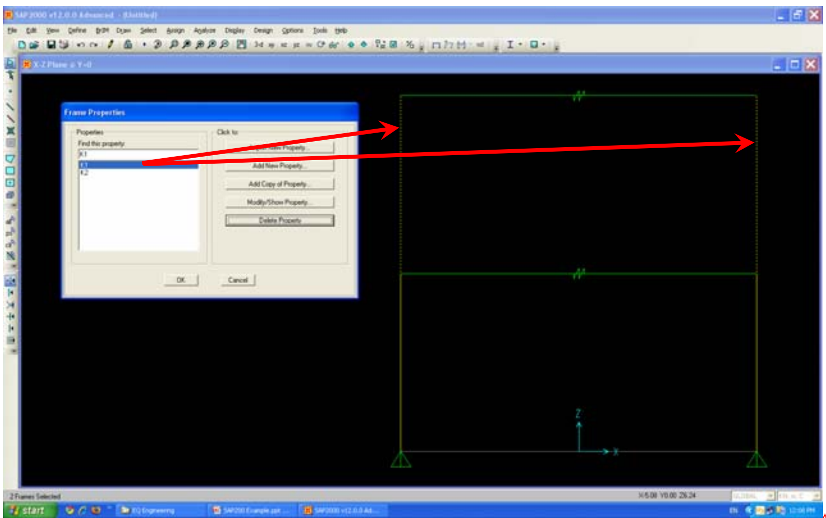
Draw Link 2 Element



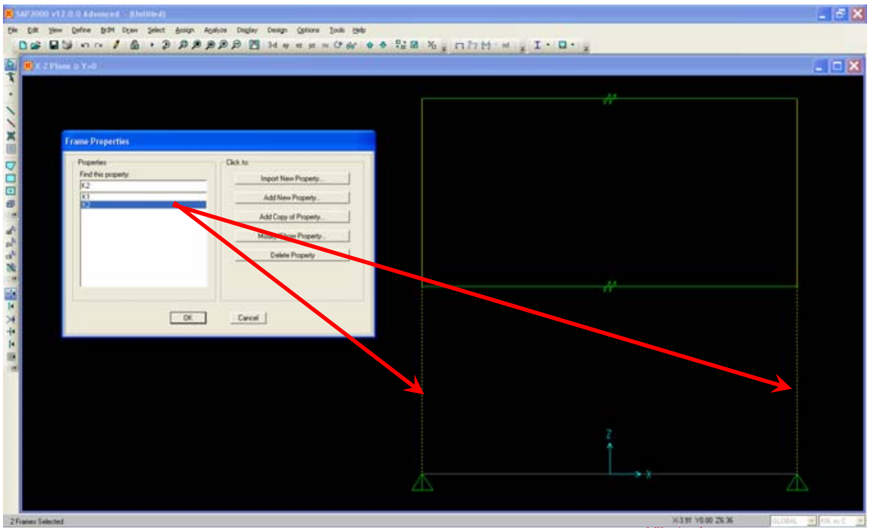
Select Top Story Columns



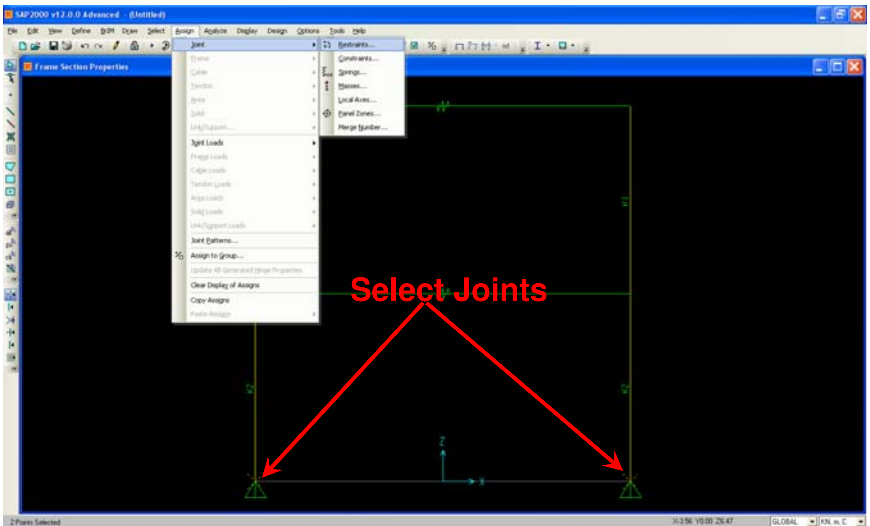
Assign K1 Frame Section



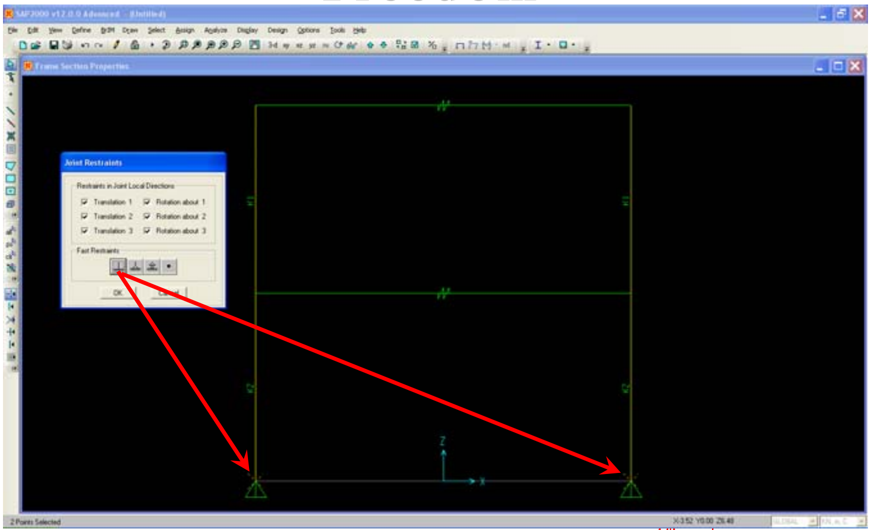
Assign K2 Frame Section



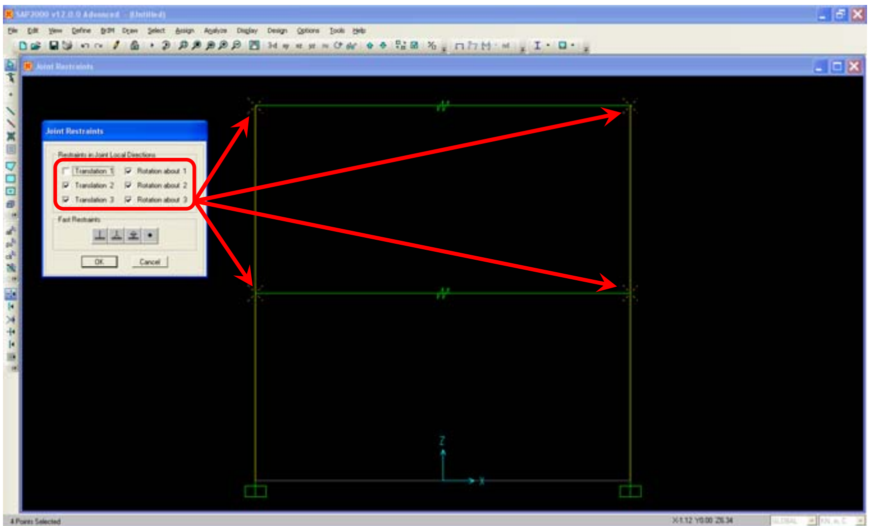
Assign Fixed Supports



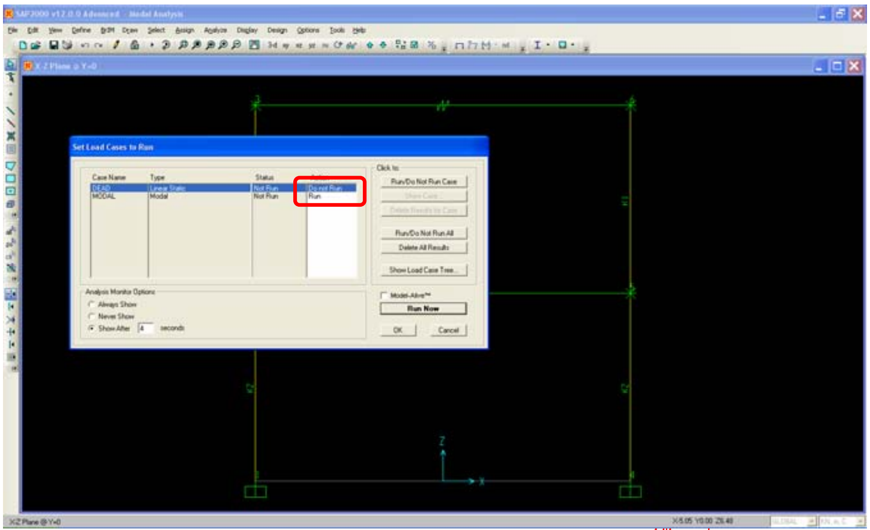
Assign Supports' Degrees of Freedom



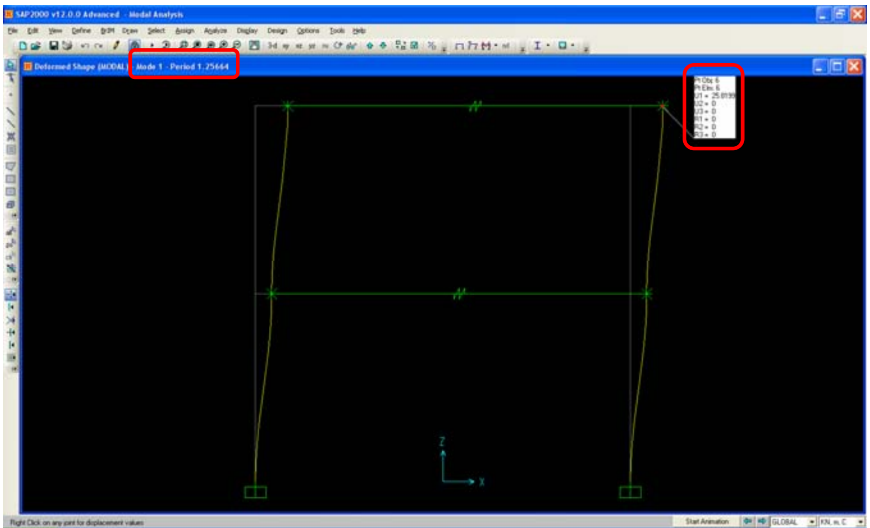
Assign Degrees of Freedom



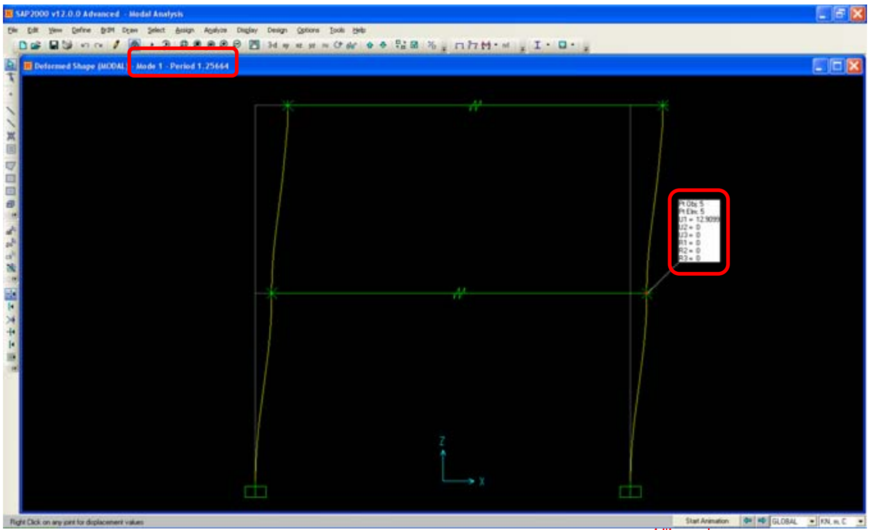
Run Software



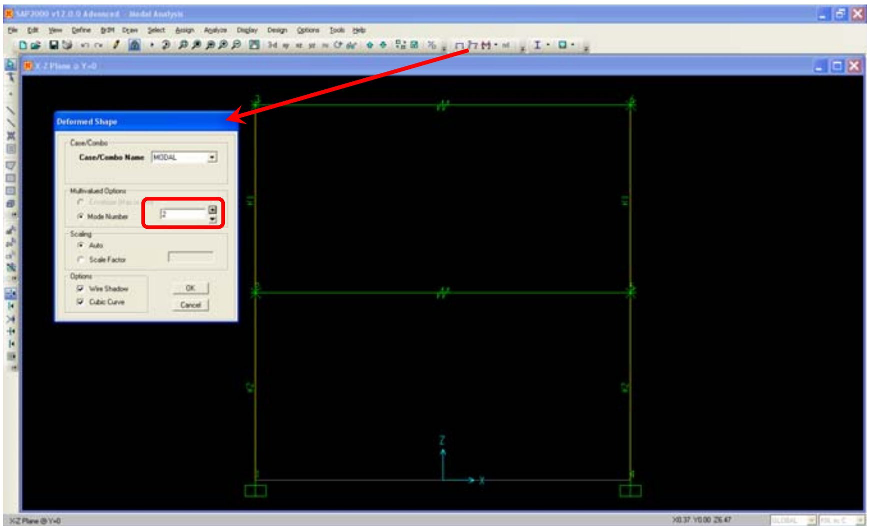
Mode-Shape 1



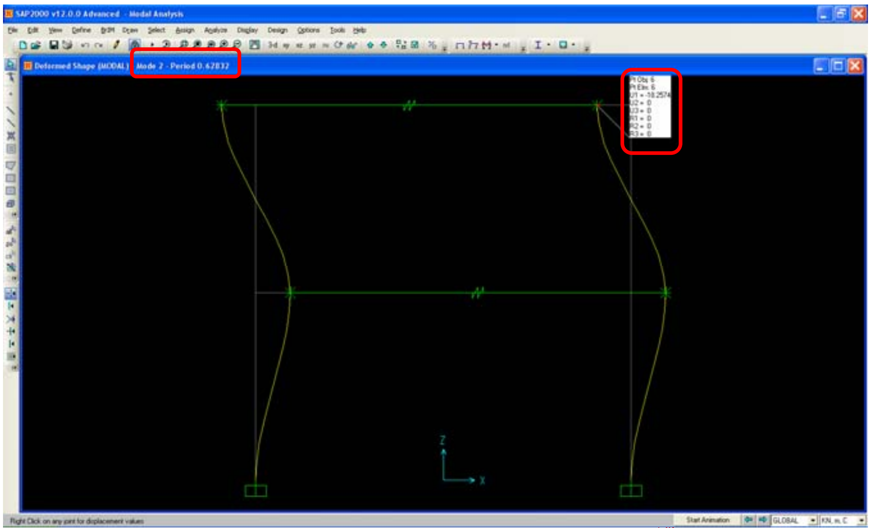
Mode-shape 1



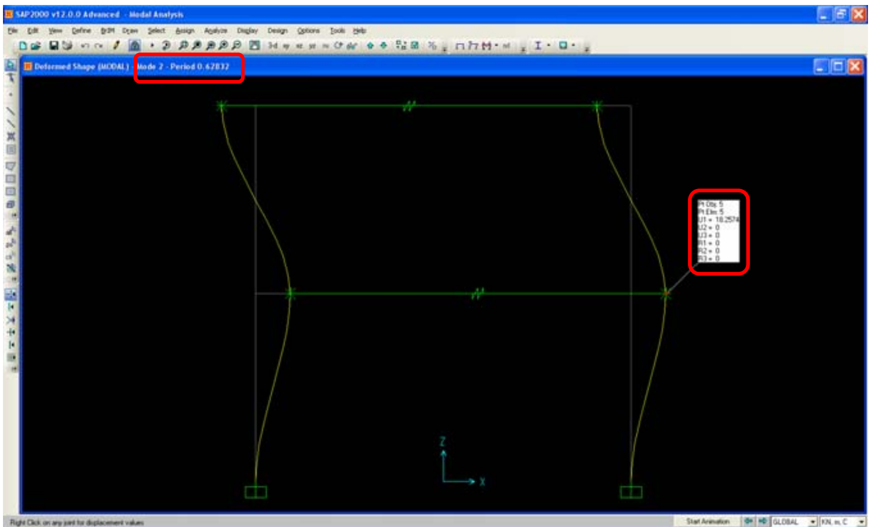
Mode-shape 2



Mode-shape 2



Mode-shape 2



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Modal Properties

- Natural Periods and Mode Shapes

$$T_{n1} = 1.26s \qquad \phi_1 = \begin{Bmatrix} 25.8199 \\ 12.9099 \end{Bmatrix} \qquad \phi_2 = \begin{Bmatrix} -18.2574 \\ 18.2574 \end{Bmatrix}$$
$$T_{n2} = 0.62s$$

- Normalized Mode Shapes

$$\phi_1 = \begin{Bmatrix} 1 \\ 0.5 \end{Bmatrix} \qquad \phi_2 = \begin{Bmatrix} -1 \\ 1 \end{Bmatrix}$$

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Dynamics of Structures – Nonlinear Analysis

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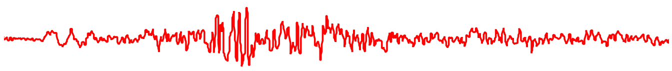
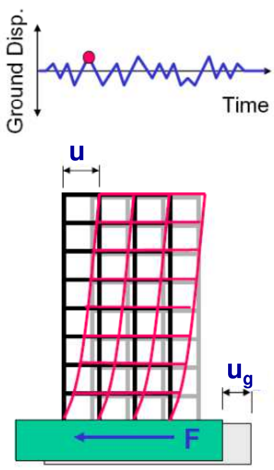
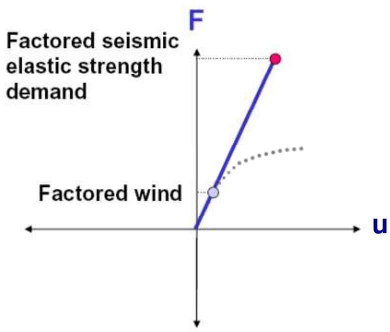


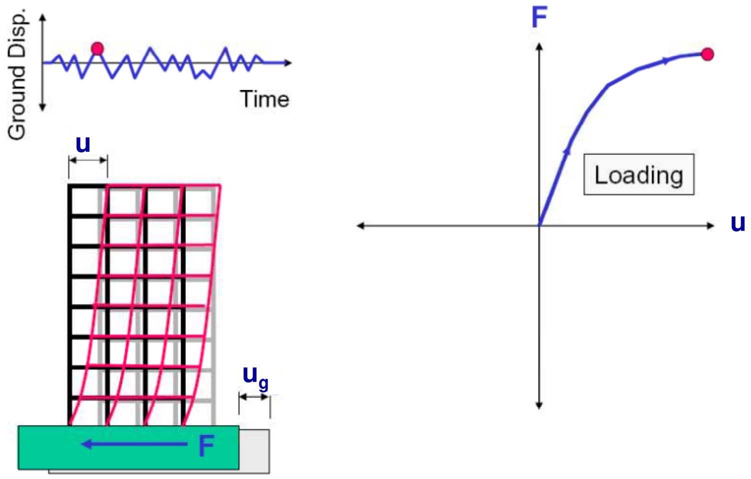
Behavior under Seismic Excitation (Elastic Response)





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

Behavior under Seismic Excitation (Inelastic Response)



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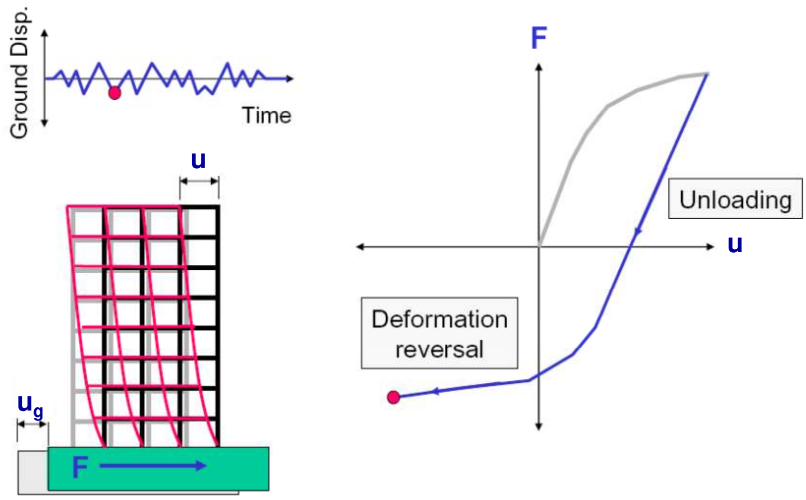
Behavior under Seismic Excitation (Inelastic Response)



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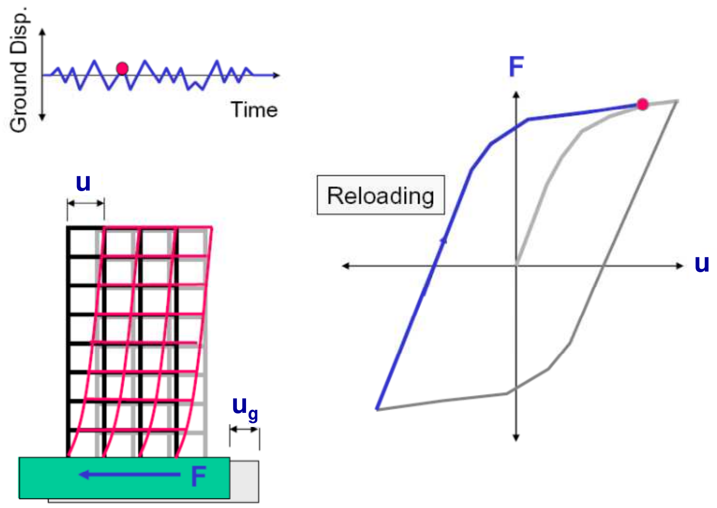
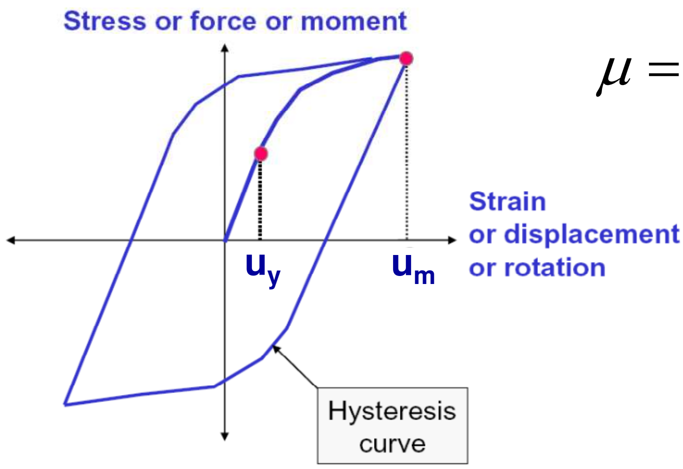
Behavior under Seismic Excitation (Inelastic Response)

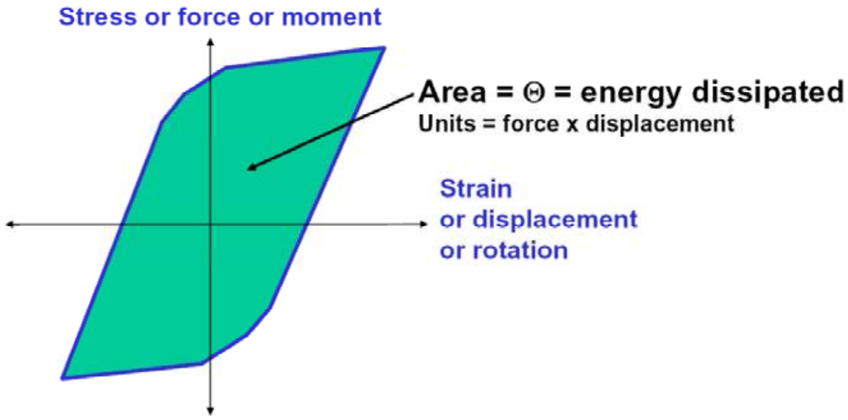


Basic Definition of Ductility



$$\mu = \frac{u_m}{u_y}$$

Definition of Energy Dissipation

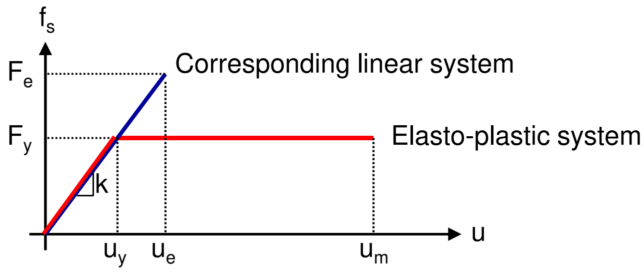


Inelastic Dynamic Analysis

- Equation of Motion for SDOF oscillator:

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

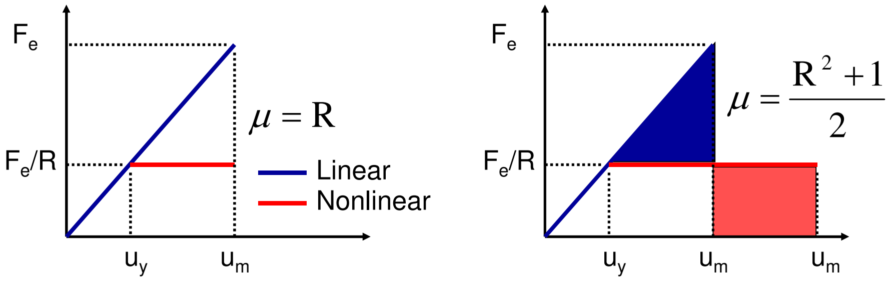
- f_s: is the restoring (resisting) force; for elasto-plastic system is shown below:



Displacement Ductility : $\mu = \frac{u_m}{u_y}$

Force Reduction Factor : $R = \frac{F_e}{F_y} = \frac{u_e}{u_y}$

Inelastic Displacement ratio : $C_\mu = \frac{u_m}{u_e}$



(a) Equal Displacement (b) Equal Energy

(R– μ– T) Relationship

- Newmark and Hall, during 70's

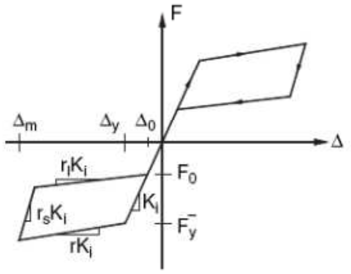
$$R = 1 \quad when \ T < 0.03s$$
$$R = \sqrt{2\mu - 1} \quad when \ 0.12\sec < T < 1s$$
$$R = \mu \quad when \ T > 1s$$

- Paulay and Priestley, in 1990

$$R = 1 + (\mu - 1)T/0.7 \quad when \ T < 0.7s$$
$$R = \mu \quad when \ T > 0.7s$$

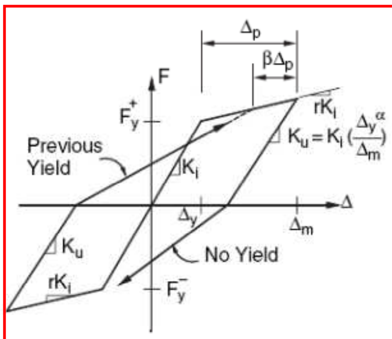
Note: the previous equations ignore the effect of hysteretic model as well as the effect of soil type. Other researchers introduced modified equations based on hysteretic model and type of soil.

Typical Hysteretic Models



(a) Ring-Spring (RS)

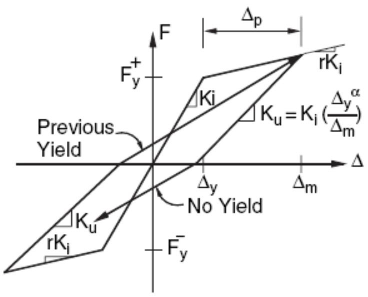
unbounded post-tensioned concrete system



(b) Large Takeda (LT)

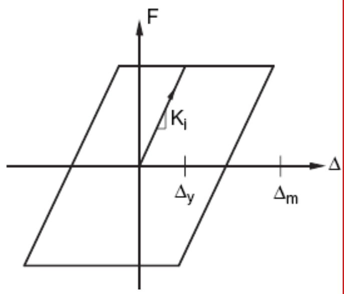
reinforced concrete beams

Typical Hysteretic Models



(c) Small Takeda (ST)

Reinforced concrete columns and walls

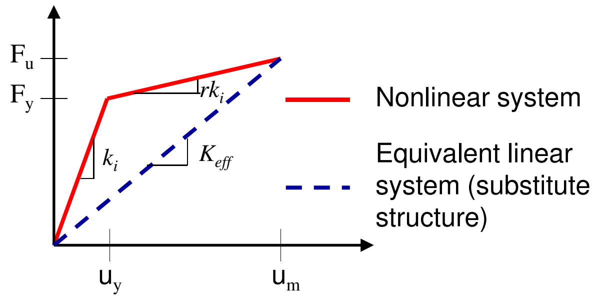


(d) Elasto-Plastic (EP)

Steel members

Substitute Structure Concept

- It is an approximate method used to determine the maximum response of a nonlinear system by an equivalent linear system with a reduced stiffness and equivalent viscous damping value.
- First Proposed by Gulkan and Sozen in 1974.



$$k_{eff} = k_i \left(\frac{r\mu - r + 1}{\mu} \right)$$

$$F_y = \frac{F_u}{r\mu - r + 1}$$

$$\mu = \frac{u_m}{u_y}$$

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Equivalent Viscous Damping

- First proposed by Jacobsen in 1930
- Jacobsen equated the energy dissipated by the nonlinear system to that dissipated by equivalent linear system during one cycle of response
- The equivalent linearization approach defined by Jacobsen's damping (ξ_{eq}) and the secant stiffness (k_{eff}) is referred to as the JDSS approach (Jacobsen's Damping Secant Stiffness)
- Equivalent viscous damping has two parts:
  - Elastic viscous damping (ξ_v)
  - Hysteretic damping due to energy dissipation (ξ_{hyst})

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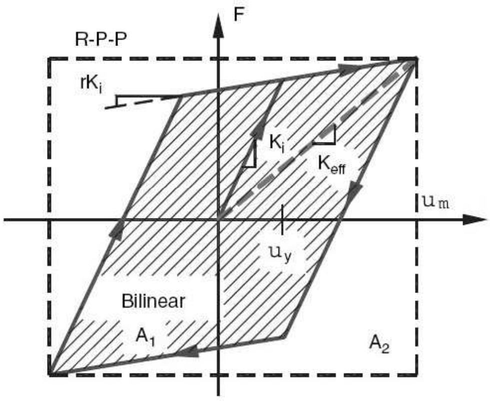
JDSS Approach

- The equivalent structure is defined by two parameters:
  - Effective stiffness (secant stiffness to max. response)
  - Equivalent damping



$$\xi_{eq} = \xi_v + \xi_{hyst}$$

$$\xi_{hyst} = \frac{2}{\pi}\frac{A_1}{A_2}$$

For Bilinear system:

$$\xi_{eq} = \xi_v + \frac{2}{\pi}\frac{(\mu-1)(1-r)}{\mu(1+r\mu-r)}$$

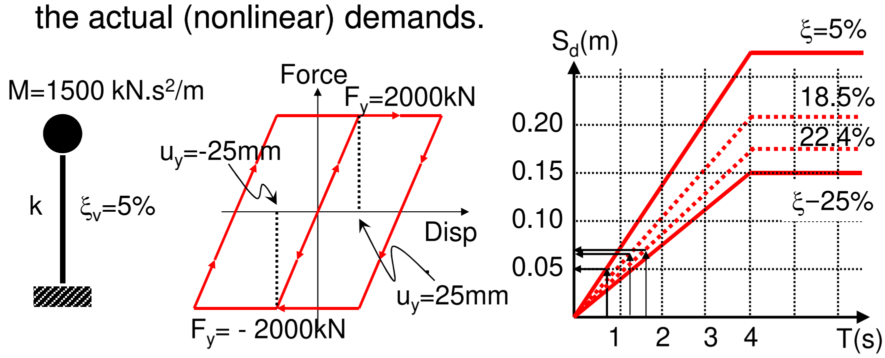
Equivalent Damping Relationships

- Modified damping equations (Dwairi *et.* al, 2007):

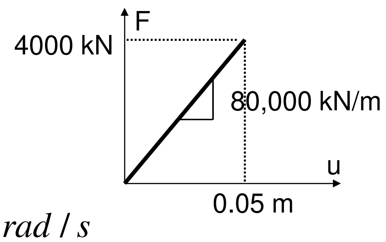
(1) Unbounded post-tensioned concrete system	$\xi_{eq} = \xi_v + 30\left(\frac{\mu-1}{\pi\mu}\right)$
(2) Reinforced concrete beams	$\xi_{eq} = \xi_v + 65\left(\frac{\mu-1}{\pi\mu}\right)$
(3) Reinforced concrete Columns and Walls	$\xi_{eq} = \xi_v + 50\left(\frac{\mu-1}{\pi\mu}\right)$
(4) Steel members:	$\xi_{eq} = \xi_v + 85\left(\frac{\mu-1}{\pi\mu}\right)$

Example

- An SDOF structure is excited by an earthquake with elastic response spectra as shown. If the structure follows a bilinear response. (a) find the elastic demands (b) find the actual (nonlinear) demands.



(1)Elastic Response



$$k_i = \frac{F_y}{u_y} = \frac{2000}{25/1000} = 80000 kN/m$$

$$\omega_{in} = \sqrt{k_i/m} = \sqrt{80000/1500} = 7.3 rad/s$$

$$T_i = 2\pi/\omega_i = 2\pi/7.3 = 0.86s$$

Enter response spectra with : $T_i = 0.86s$ *and* $\xi_v = 5\%$

$$\therefore S_d = 0.05m; S_v = \omega_n S_d = 0.365 m/s; S_a = \omega_n^2 S_d = 2.665 m/s^2$$

Thus,

$u_e = 0.05m$ *and*

$$F_e = mS_a = k_i u_e = 80000 \times 0.05 = 4000 kN$$

(2) Nonlinear (Inelastic) Response

Since max. displacement is unknown at this stage, iterative procedure is required. Assume initial dispalcement and loop over the force until it converges:

Trial 1: start with $u_m = u_e = 0.05m$

$k_{eff} = F_m / u_m = 2000/0.05 = 40{,}000 kN/m$

$T_{eff} = 2\pi\sqrt{m/k_{eff}} = 2\pi\sqrt{1500/40000} = 1.22s > T_i = 0.86s$

$Ductility = \mu = u_m/u_y = 0.05/0.025 = 2$

$$Equivalent\ Damping = \xi_v + 85\left(\frac{\mu-1}{\pi\mu}\right) = 5\% + 85\left(\frac{2-1}{2\pi}\right)\%$$

$\therefore \xi_{eq} = 5\% + 13.5\% = 18.5\%$

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Enter the spectra with $T = 1.22s$ and $\xi_{eq} = 18.5\%$

Interpolate between $\xi = 5\%$ and $\xi = 25\%$ NOT OK

$\therefore S_d = 0.065m = u_m;\quad F_m = k_{eff} u_m = 40{,}000 \times 0.065 = 2600kN$

Trial 2: start with $u_m = 0.07m$

$k_{eff} = F_m / u_m = 2000/0.07 = 28571 kN/m$

$T_{eff} = 2\pi\sqrt{m/k_{eff}} = 2\pi\sqrt{1500/28571} = 1.44s$

$\mu = u_m / u_y = 0.07/0.0025 = 2.8$

$$\xi_{eff} = 5 + 85\left(\frac{2.8-1}{2.8\pi}\right) = 22.4\%$$

Enter the spectra with $T = 1.44s$ and $\xi_{eq} = 22.4\%$ OK

$\therefore S_d = 0.07m = u_m;\quad F_m = k_{eff} u_m = 28571 \times 0.07 = 2000kN$

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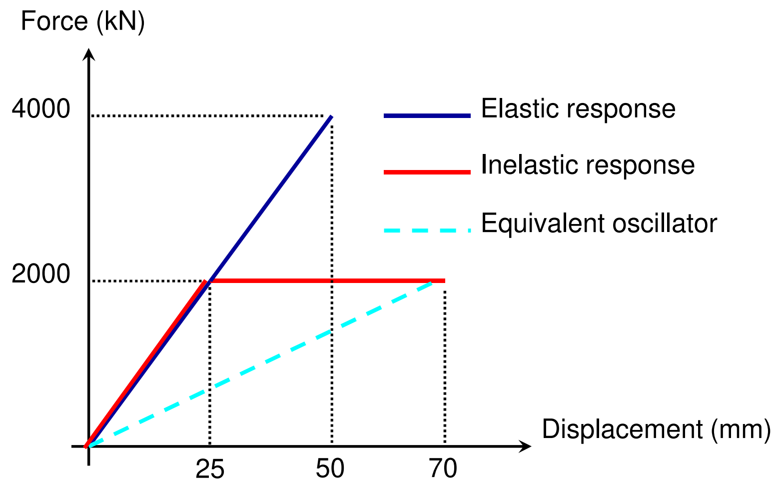
Elastic vs. Inelastic Response



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Member Strength and Ductility

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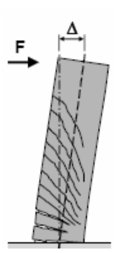
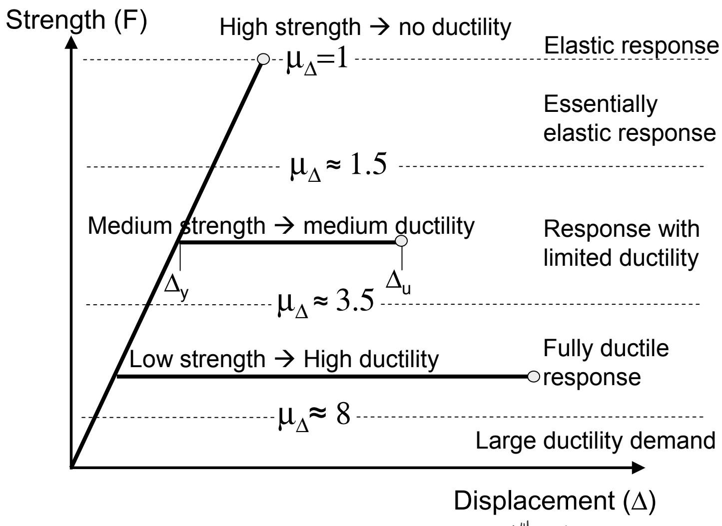
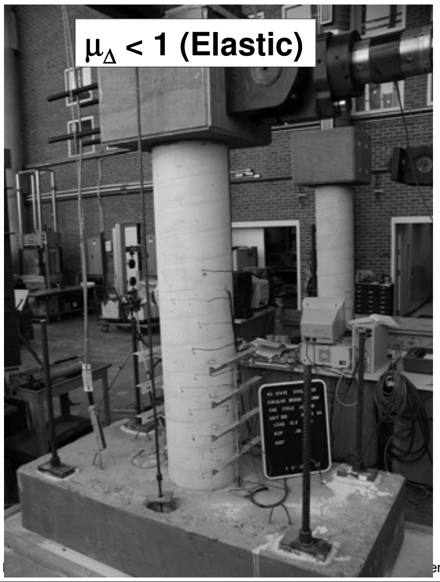
Strength and Ductility



© Alessandro D. (modified)

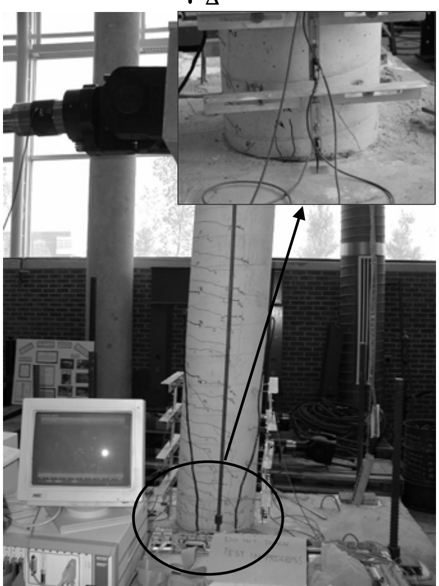


Bridge Column – Static Cyclic Test

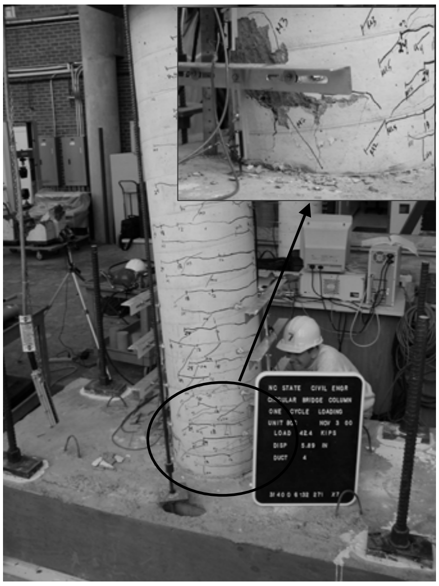


$\mu_\Delta = 2$

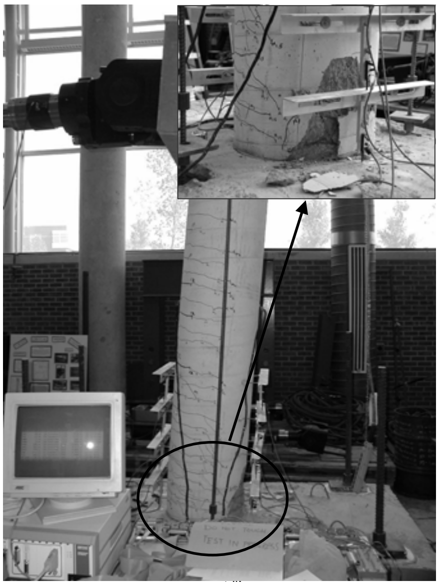
$\mu_\Delta = 3$



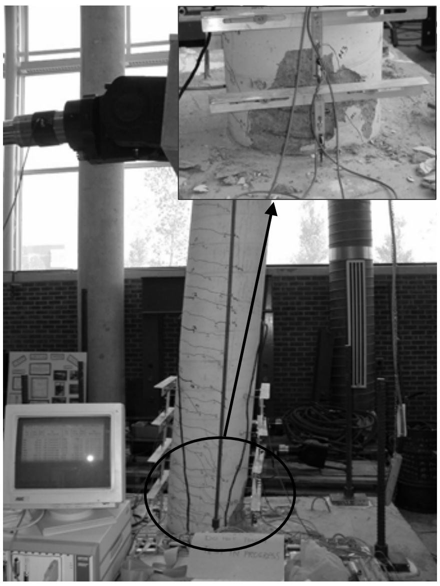
μ_Δ = 4



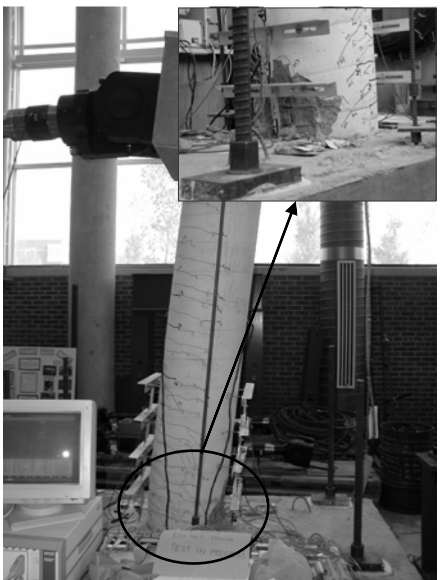
μ_Δ = 5



μ_Δ = 6



μ_Δ = 7



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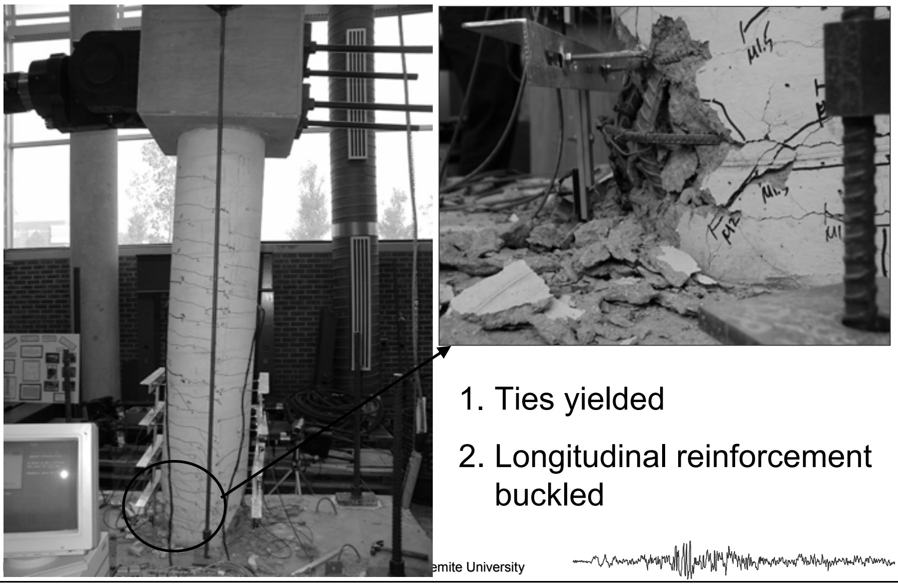
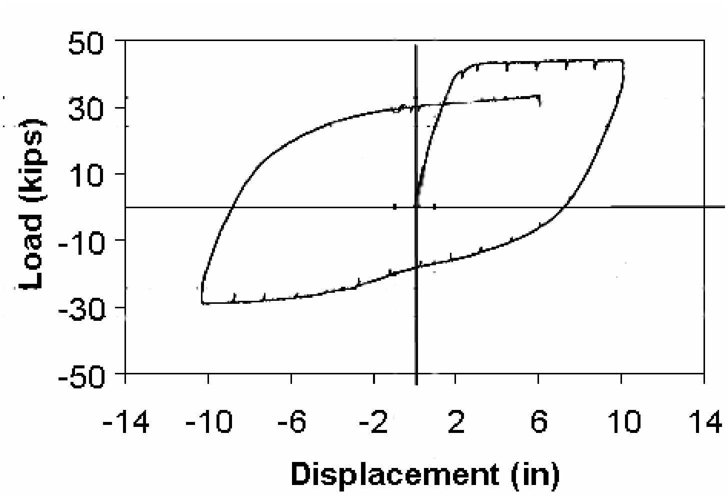
After a Full Cycle

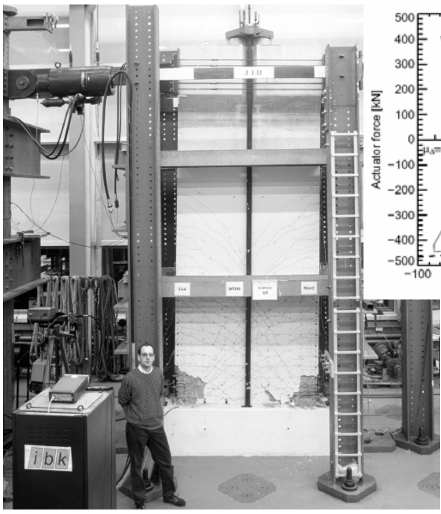


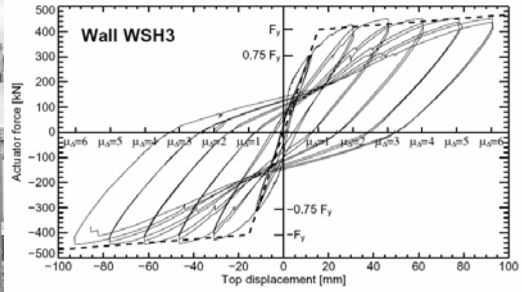
1. Ties yielded
2. Longitudinal reinforcement buckled

Load – Deformation Curve



RC Wall – static cyclic test





Load –Deformation Response
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Inelastic Behavior of Structures

From material

↓

to cross section

↓

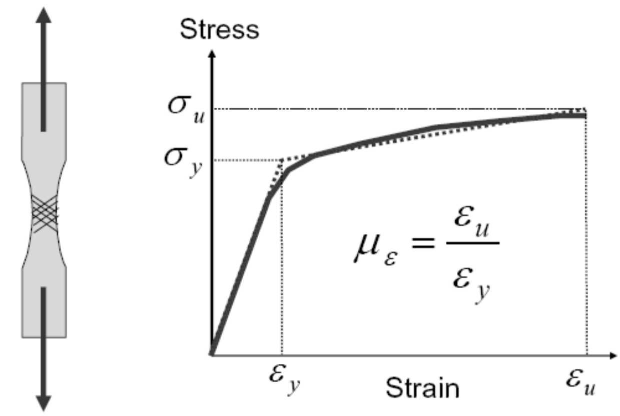
to critical region

↓

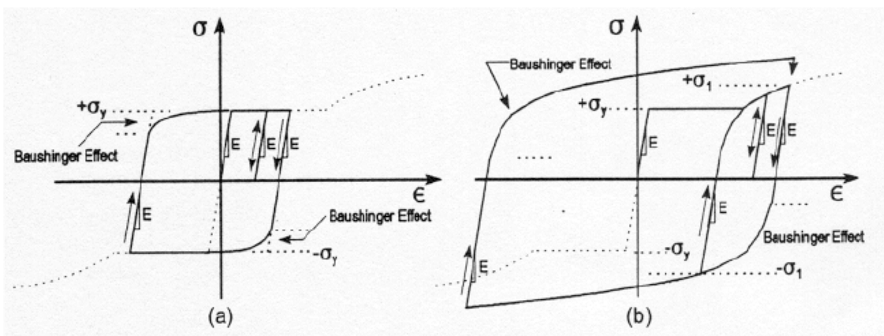
to structure

Idealized Inelastic Behavior

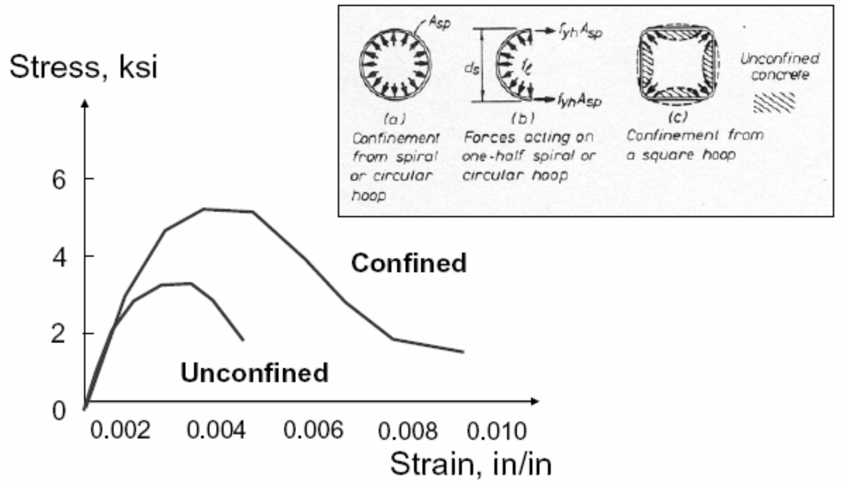
From Material



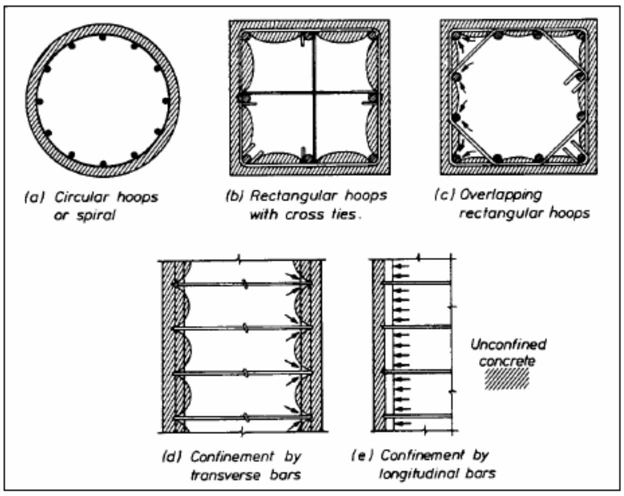
Stress-Strain Relationship for Steel



Stress-Strain Relationship for Concrete (confined and unconfined)

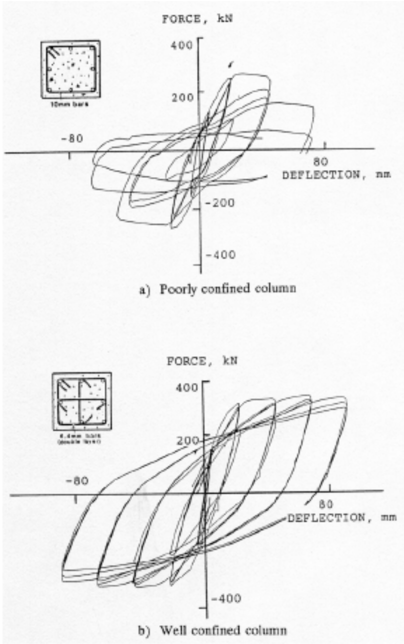


Confinement of Concrete



Unconfined

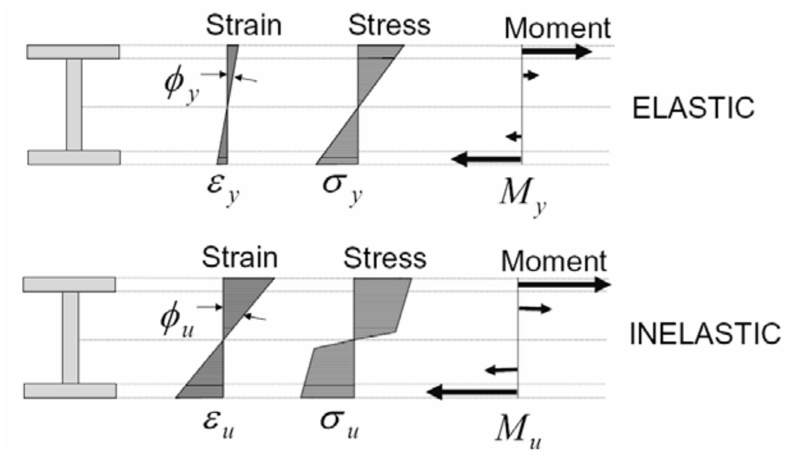
Confined



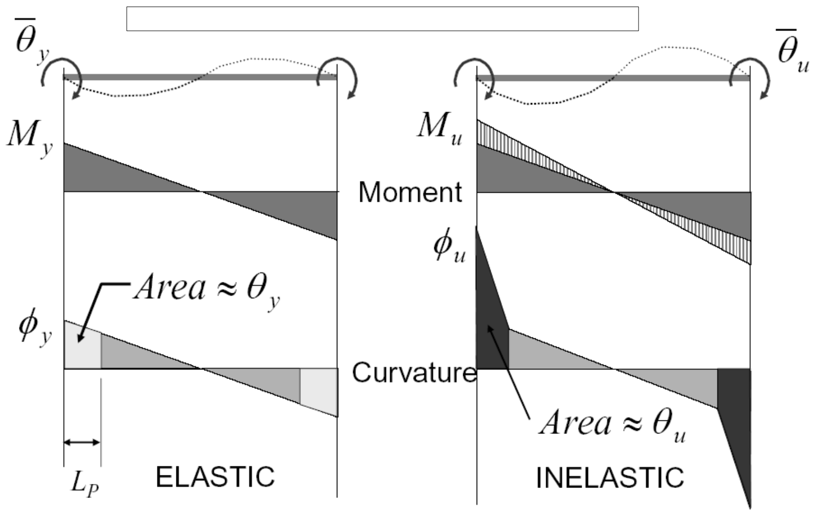
Benefits of Confinement

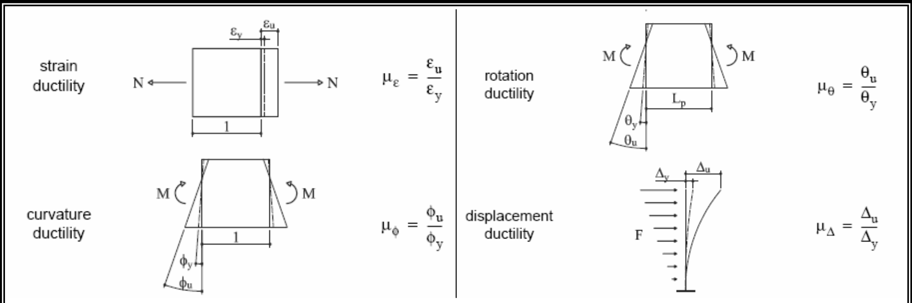
Olive View Hospital, 1971 San Fernando Valley earthquake

Idealized Inelastic Behavior to Section



Idealized Inelastic Behavior to Member

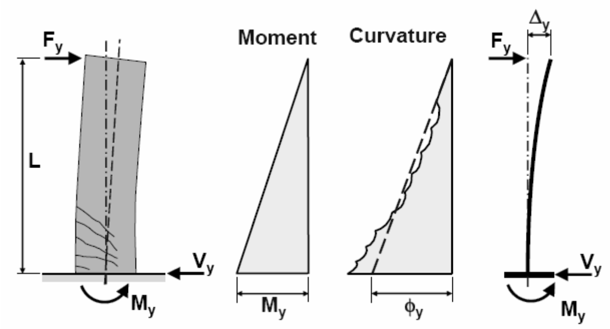




Ductility Concept

$$\frac{M}{EI} = \phi$$

$$\Delta_y = \frac{\phi_y L^2}{3}$$



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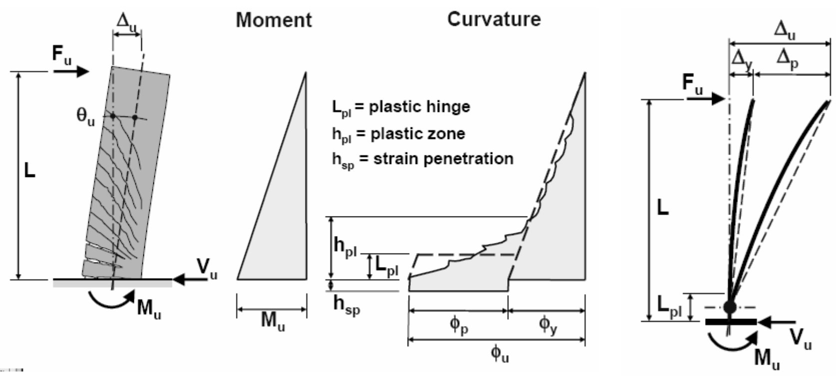
Plastic Hinge Model



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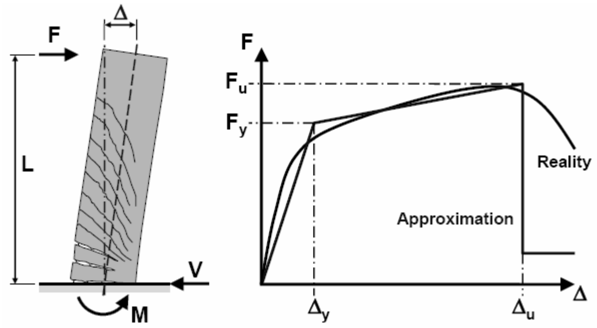
$$\Delta_p = (\phi_u - \phi_y) L_p (L - \frac{L_p}{2})$$

$$\Delta_u = \Delta_y + \Delta_p = \phi_y \frac{L^2}{3} + (\phi_u - \phi_y) L_p (L - \frac{L_p}{2})$$

Local and Global Ductility



Displacement Ductility (Global):

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = 1 + \frac{(\phi_u - \phi_y) L_p (L - L_p/2)}{(\phi_y L^2)/3} = 1 + \frac{(\mu_\phi - 1) L_p (L - L_p/2)}{L^2/3}$$

Hence, Curvature Ductility (Local):

$$\mu_\phi = \frac{\phi_u}{\phi_y} = 1 + \frac{\mu_\Delta - 1}{3\frac{L_p}{L}(1 - \frac{L_p}{2L})}$$

Plastic hinge length is calculated in a way such that the integration of the plastic curvature (ϕ_p) over the plastic hinge length is equal to the plastic deformation (Δ_p) in reality.

Pauly and Priestley (1992) proposed:

$$L_p = 0.08L + 0.022 d_b f_y \qquad [mm]$$

$$where: d_p = bar\ diameter\ in\ [mm]$$

$$f_y = yield\ strength\ in\ [MPa]$$

Ductility and Energy Dissipation Capacity

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

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.

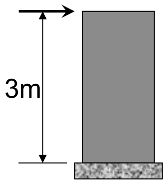
Ductility and Energy Dissipation Capacity

- 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.

Example 1

The R.C. column shown is subjected to an earthquake. The inelastic dynamic analysis results in terms of hysteretic response are shown as well.

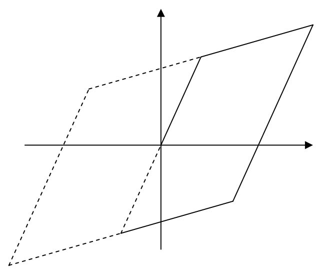
- Compute the global ductility demand
- Compute the local ductility demand



f'_c=30 MPa

f_y = 414 MPa

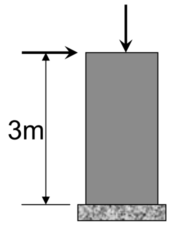
ε_{cu} = 0.004



Example 2

The R.C. column shown is subjected to lateral monotonic loading, F, and constant axial load, P = 1000 kN.

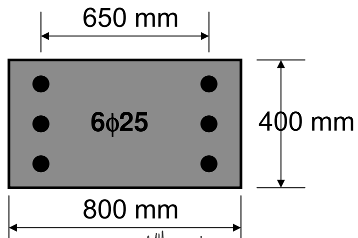
- Compute the yield curvature, and moment
- Compute the ultimate curvature and moment
- Compute local ductility demand
- Compute global ductility demand



f'_c = 30 MPa

f_y = 414 MPa

ε_{cu} = 0.004



What do you think?

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Department of Civil Engineering

Lecture 6 – Design Response Spectrum

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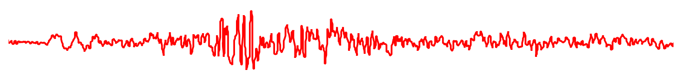
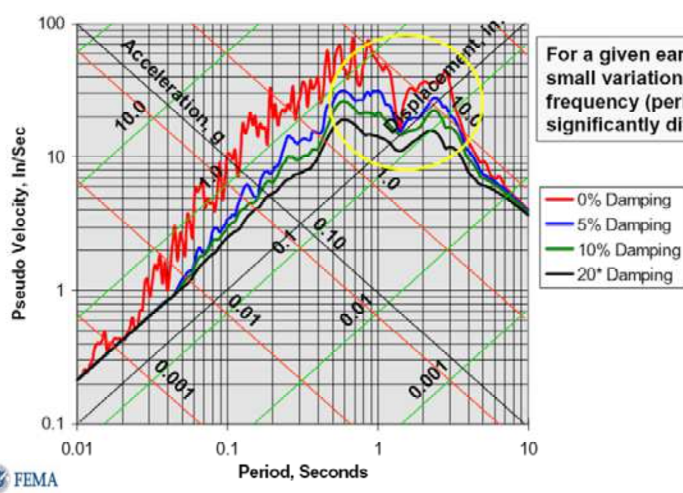


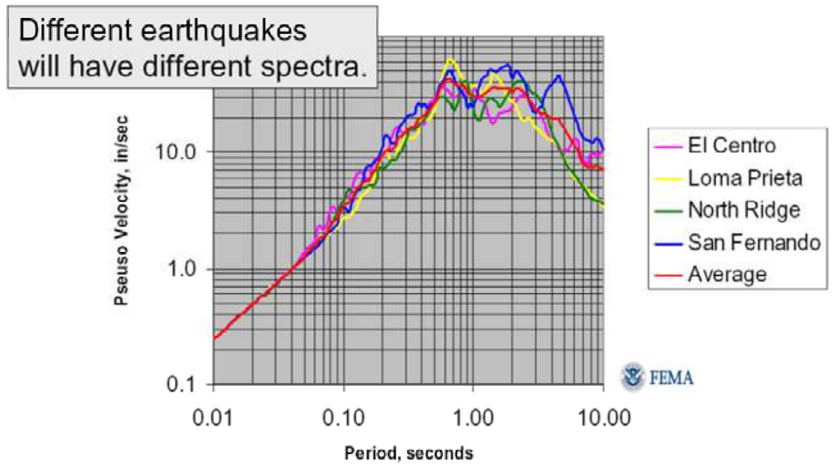
1940 El Centro, 0.35 g, N-S



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

FEMA

5% Damped Spectra for Four California Earthquakes – scaled to 0.4 g (PGA)



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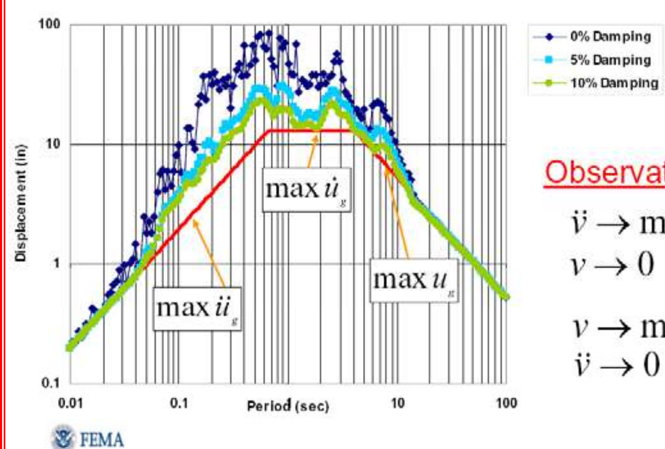
Smoothed Elastic Response Spectra – Newmark-Hall Spectra



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

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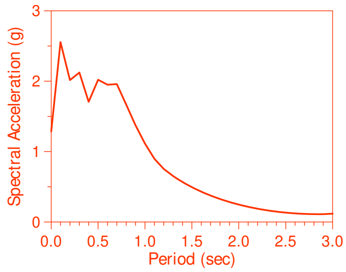
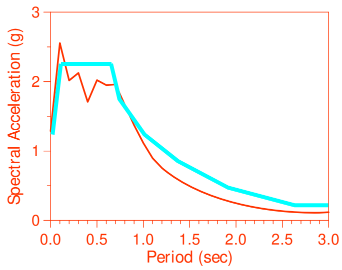
Design Spectra



- Actual spectra not used in design.
- Rough in shape.
- Specific to ground motion.

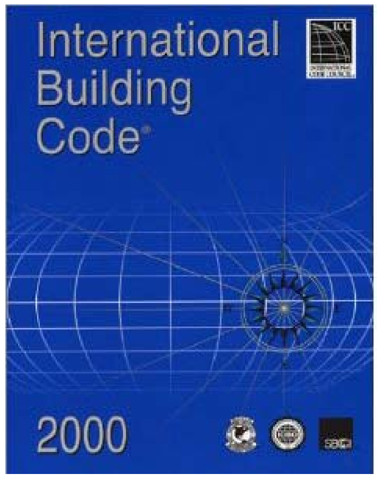


- Represents many EQs.
- Smooth shape.
- Guidelines for calculation in building codes - UBC, IBC.
- Older codes: Little change in shape, only magnitude.

Code-based Design Spectrum

- Is a uniform hazard spectrum based on probabilistic and deterministic seismic hazard analysis
- Design spectra considered in this course:
  - IBC 2000 Design Spectrum
  - UBC'97 Design Spectrum
  - Jordan Seismic Code (JSC) Design Spectrum

IBC 2000 Design Response Spectrum



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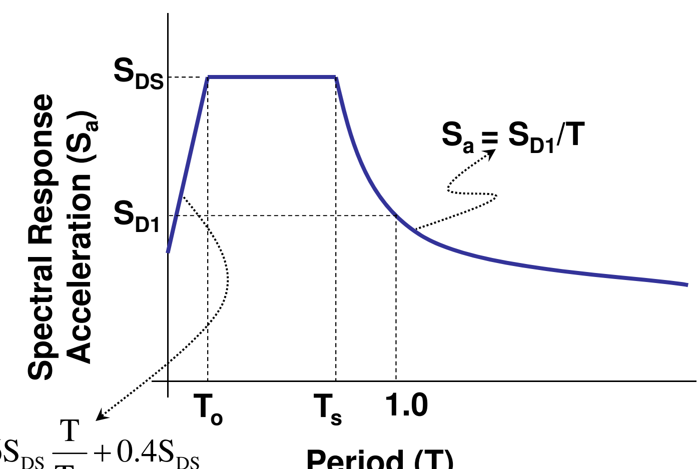
Design Response Spectrum – Fig 1615.1.4



$$S_a = 0.6S_{DS}\frac{T}{T_o} + 0.4S_{DS}$$

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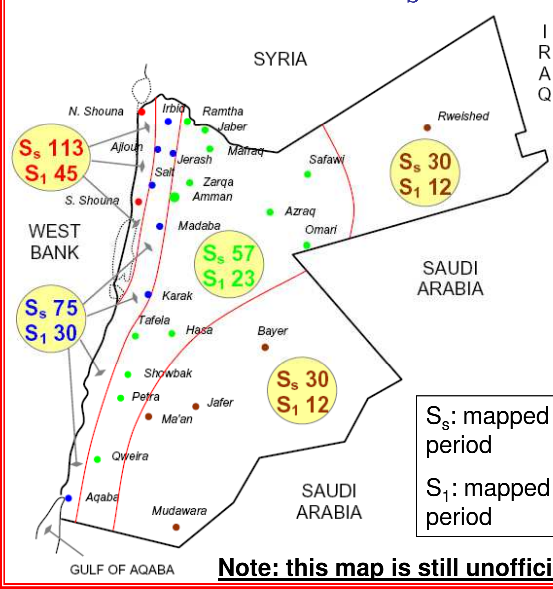
Design Spectrum Procedure

1. Determine basic ground motion parameters (S_s, S_1) – Zonation map
2. Determine site classification (A – F) – Table 1615.1.1
3. Determine site coefficient adjustment factors (F_a, F_v) – Tables 1615.1.2(1) & 1615.1.2(1)
4. Determine design ground motion parameters (S_{DS}, S_{D1})

1. S_s and S_1



S_s: mapped spectral acceleration for short period

S_1: mapped spectra acceleration for 1-sec period

Note: this map is still unofficial

2. Site Classes

A	Hard rock	$v_s > 5000$ ft/s
B	Rock	$2500 < v_s \leq 5000$ ft/s
C	Very dense soil or soft rock	$1200 < v_s \leq 2500$ ft/s
D	Stiff soil	$600 \leq v_s \leq 1200$ ft/s
E	Soft soil	$v_s < 600$ ft/s
F	Site specific requirements	

Note: v_s = soil shear wave velocity

3. Site Coefficients F_a & F_v

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a [a]

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

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_s.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_V [a]

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

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

4. Ground Motion Parameters

- Maximum expected earthquake coefficients:

  $S_{MS} = F_a S_s$

  $S_{M1} = F_v S_1$

- Design earthquake coefficients:

  $S_{DS} = 2/3 S_{MS}$

  $S_{D1} = 2/3 S_{M1}$

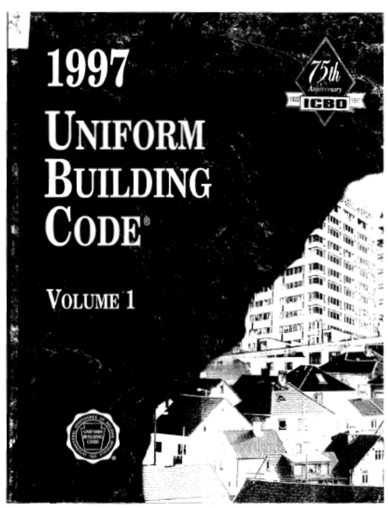
- Period:

  $T_o = 0.2\ S_{D1}/S_{DS}$

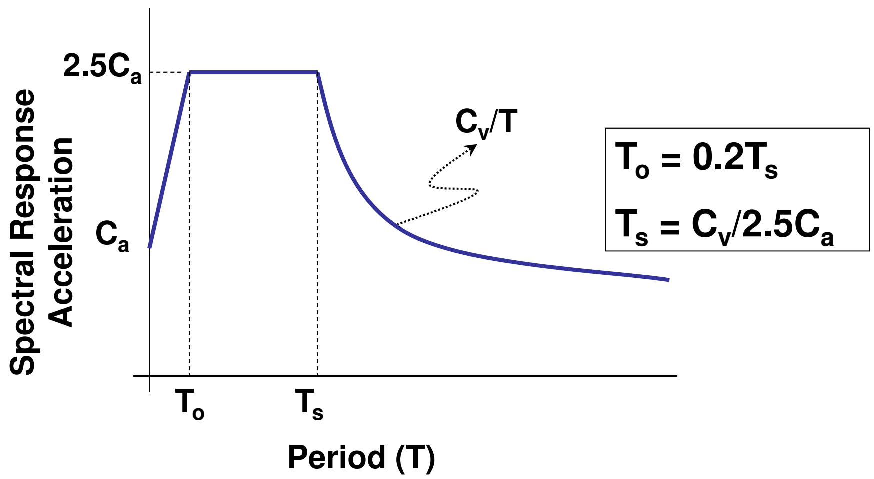
  $T_s = S_{D1}/S_{Ds}$

JSC Design Response Spectrum

(Uniform Building Code UBC 1997)



Design Response Spectrum – Figure 2-3



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Design Spectrum Procedure

1. Determine seismic site category – Zonation map
2. Determine seismic zone factor (Z) –Table 2-2
3. Determine site classification (S_A – S_F) – Table 2-1
4. Determine site coefficient adjustment factors (C_a, C_v) – Tables 2-3 & 2-4
5. Determine design ground motion parameters (T_0, T_s)

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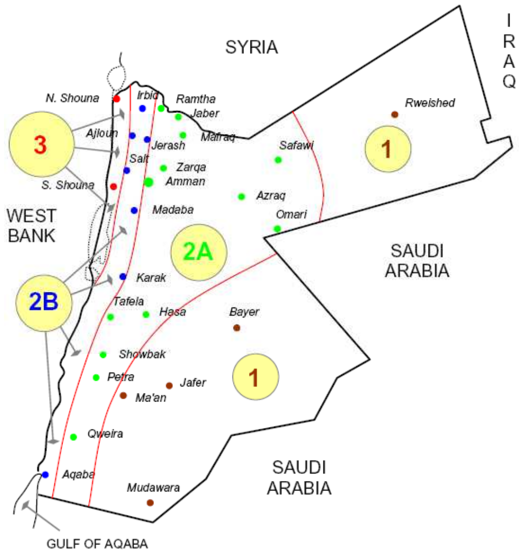
1. Seismic Site Category



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2. Seismic Zone Factor

Zone:	1	2A	2B	3
Z =	0.075	0.15	0.20	0.30

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3. Site Classes

S_A	Hard rock	$v_s > 1500$ m/s
S_B	Rock	$760 < v_s \leq 1500$ m/s
S_C	Very dense soil or soft rock	$360 < v_s \leq 760$ m/s
S_D	Stiff soil	$180 \leq v_s \leq 360$ m/s
S_E	Soft soil	$v_s < 180$ m/s
S_F	Site specific requirements	

Note: v_s = soil shear wave velocity

4. Site Factors – C_a

TABLE 16-Q—SEISMIC COEFFICIENT C_a

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

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

TABLE 16-S—NEAR-SOURCE FACTOR N_a[1]

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

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

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

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

4. Site Factors – C_v

TABLE 16-R—SEISMIC COEFFICIENT C_V

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

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

TABLE 16-T—NEAR-SOURCE FACTOR N_v[1]

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

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

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

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

Example

Utilizing the IBC-2000 and JSC'05 codes, determine design response spectra for Amman

Note: Assume soft soil site class

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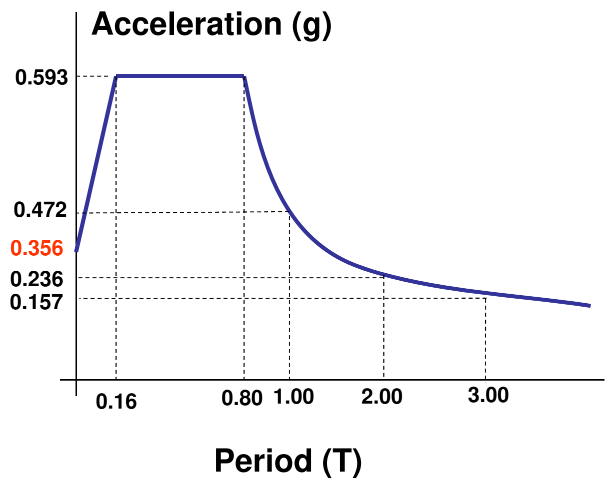
Amman IBC Spectrum

- S_s = 0.57g & S_1 = 0.23g
- F_a = 1.56 & F_v = 3.08
- S_{DS} = 2/3 x F_a x S_s = 0.593g
- S_{D1} = 2/3 x F_v x S_1 = 0.472g
- T_o = 0.2 S_{D1} / S_{DS} = 0.16 sec
- T_s = S_{D1} / S_{DS} = 0.80 sec

$$S_a = 0.6 S_{DS} \frac{T}{T_o} + 0.4 S_{DS} = 0.356g \quad @T = 0.0\,\text{sec}$$

Amman IBC Spectrum

Period (sec)	Acc. (g)
0.00	0.356
0.16	0.593
0.80	0.593
1.00	0.472
2.00	0.236
3.00	0.157

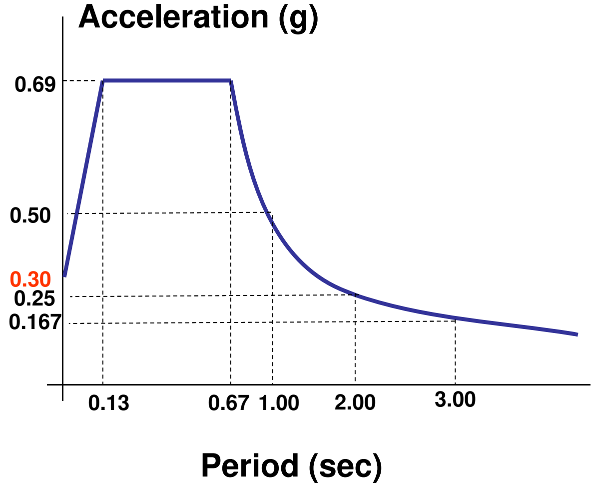


Amman UBC Spectrum

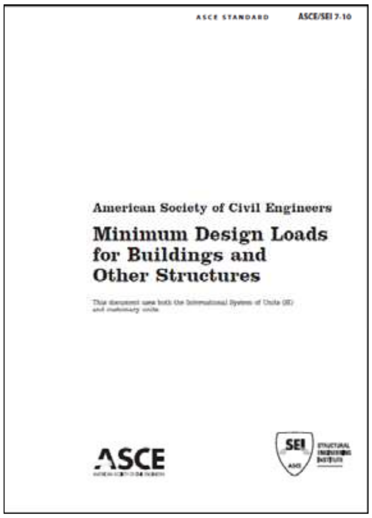
- Seismic Zone 2A
- Z = 0.15
- C_a = 0.30g
- C_v = 0.50g
- T_s = $C_v / 2.5C_a$ = 0.67 sec
- T_o = $0.2T_s$ = 0.13 sec

Amman UBC Spectrum

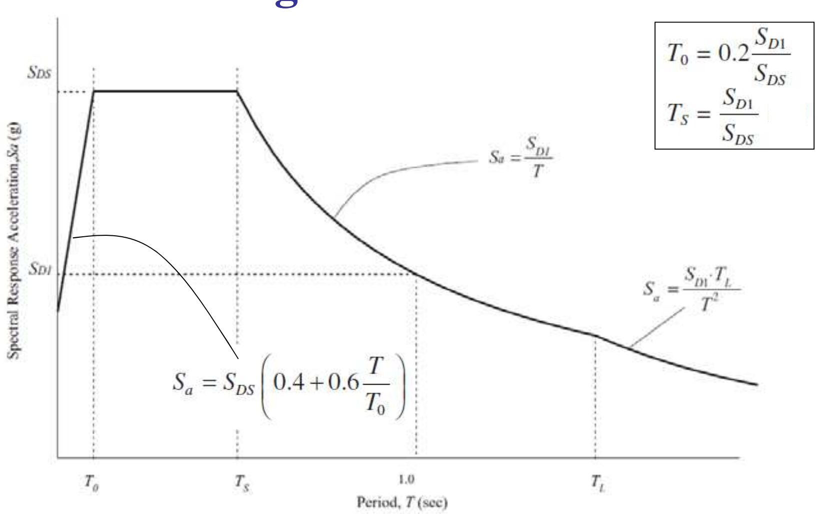
Period (sec)	Acc. (g)
0.00	0.300
0.13	0.690
0.67	0.690
1.00	0.500
2.00	0.250
3.00	0.167



ASCE7-2010 & IBC 2012 Design Response Spectrum



Design Response Spectrum – Figure 11.4-1



T_L = long-period transition period (s) shown in Figs. 22-12 through 22-16.

CHAPTER 22 SEISMIC GROUND MOTION LONG-PERIOD TRANSITION AND RISK COEFFICIENT MAPS

MINIMUM DESIGN LOADS

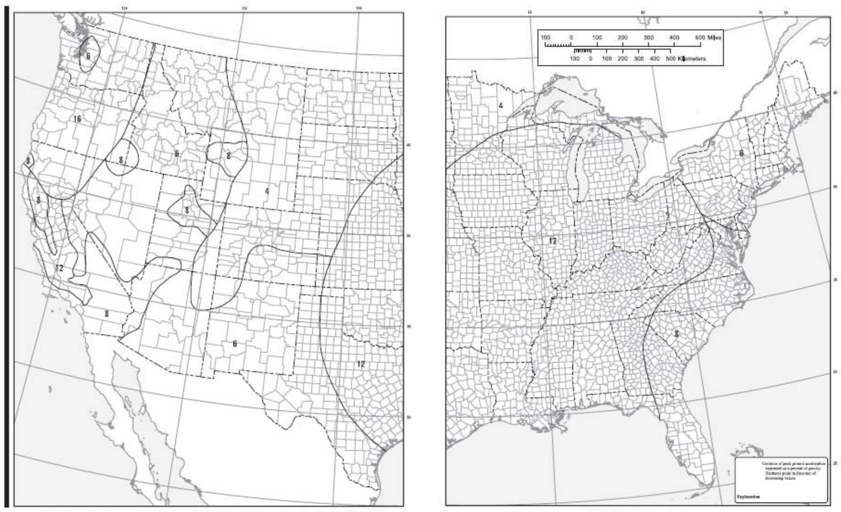


FIGURE 22-12 Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States.

FIGURE 22-12 (Continued)

224 225

EuroCode 8, 1998-2003
Design Response Spectrum

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

prEN 1998-1 : 2003
October 30 2003

UDC
Descriptors:

English version

Eurocode 8 : Design of structures for earthquake resistance

Part 1: General rules, seismic actions and rules for buildings

Calcul des structures pour leur résistance aux séismes

Auslegung von Bauwerken gegen Erdbeben

Partie 1 : Règles générales, actions sismiques et règles pour les bâtiments

Teil 1 : Grundlagen, Erdbebeneinwirkungen und Regeln für Hochbauten

Stage 49 draft

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Design Response Spectrum

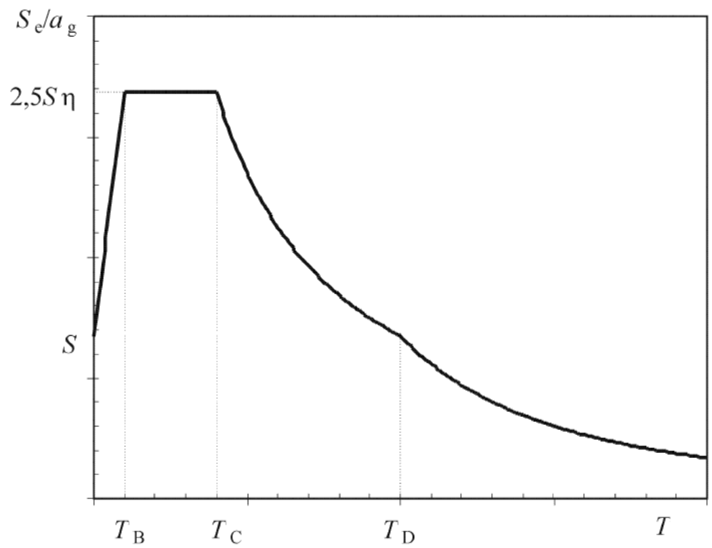


Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 3.3: Values of the parameters describing the recommended Type 2 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

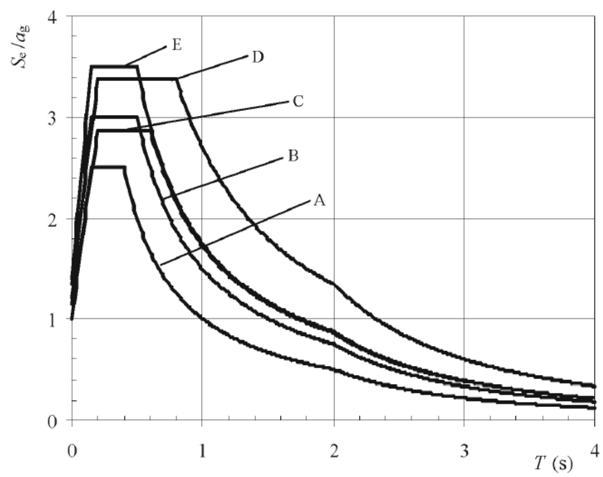


Figure 3.2: Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

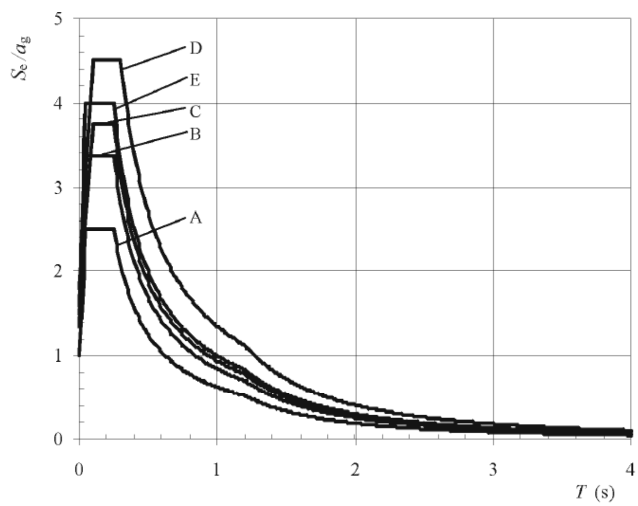


Figure 3.3: Recommended Type 2 elastic response spectra for ground types A to E (5% damping)

Example

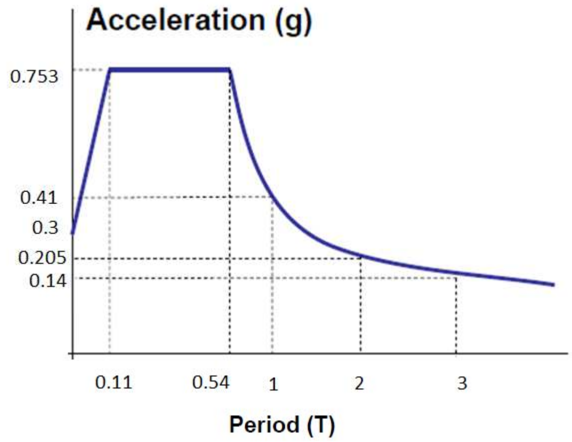
- Construct the [IBC-based & UBC-based] spectra for North Shouna , assuming soil type C

North Shouna IBC Spectrum

- $S_S = 1.13g$ & $S_1 = 0.45g$
- $F_a = 1.0$ & $F_v = 1.35$
- $S_{DS} = \frac{2}{3} x\ 1 \times 1.13 = 0.753$
- $S_{D1} = \frac{2}{3} x\ 1.35 \times 0.45 = 0.41$
- $T_o = 0.2 \times 0.41/0.753 = 0.11$ sec
- $T_s = 0.41/0.753 = 0.54$ sec

North Shouna IBC Spectrum

T (sec)	Sa (g)
0.0	0.3
0.11	0.753
0.54	0.753
1.0	0.41
2	0.205
3	0.14

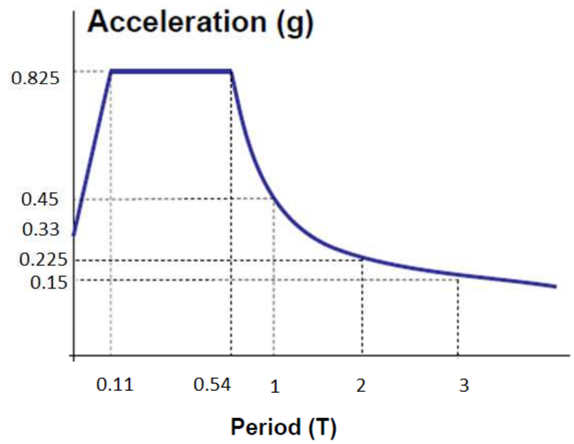


North Shouna UBC Spectrum

- Zone (3)
- Z=0.3g
- $C_a = 0.33g$ & $C_v = 0.45g$
- $T_o = 0.2\ T_s = 0.2x\ 0.54 = 0.11$ sec
- $T_s = \frac{Cv}{2.5\ Ca} = \frac{0.45}{2.5*0.33} = 0.54$ sec

North Shouna UBC Spectrum

T (sec)	Sa (g)
0.0	0.33
0.11	0.825
0.54	0.825
1.0	0.45
2.0	0.225
3.0	0.15



The Hashemite University
Department of Civil Engineering

Lecture 7 – Seismic Load Analysis

Dr. Hazim Dwairi

Credit:

Most of the sketches used in this presentation were acquired from the instructional material complementing FEMA 451, published by

FEMA

Current Model Codes

UBC, IBC:

- ✓ Provide minimum provisions for design and construction of structures to resist effects of seismic ground motions.
- ✓ "…to safeguard against major structural failures and loss of life, not to limit damage or maintain function." *(UBC '97, Section 1626)*

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Load Analysis Procedure, IBC 2000

1. Determine building occupancy category
2. Determine design response spectrum
3. Determine seismic design category
4. Determine importance factor
5. Select structural system and system parameters (R, C_d, Ω_o)
6. Examine system for configuration irregularities
7. Determine diaphragm flexibility (flexible, semi-rigid, and rigid)

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Load Analysis Procedure (continued)

8. Determine redundancy factor (ρ)
9. Determine lateral force analysis procedure
10. Compute lateral loads
11. Add torsional loads, as applicable
12. Add orthogonal loads as applicable
13. Perform analysis
14. Combine results
15. Check strength, deflection, and stability

1. Occupancy Category (IBC 2000)

I. Normal Hazard Occupancy: except those listed in categories II, III, IV

II. Substantial Hazard Occupancy:
   - High occupancy (more than 300 people in one room)
   - Schools and Universities
   - Health care more than 50 patient residents
   - Jails and detention facilities
   - Power stations
   - Water treatment plant
   - Waste water treatment plants

1. Occupancy Category (IBC 2000)

III. Essential Facilities:
- Hospitals and emergency facilities with surgery
- Fire, rescue, ambulance, police stations
- Designated emergency shelters
- Aviation control towers
- Critical national defense facilities

IV. Low hazard Occupancy:
- Agricultural facilities
- Temporary facilities
- Minor storage facilities

2. Design Response Spectrum

Recall Lecture 6

3. Seismic Design Category – Short Period Acceleration

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{DS} \ge 0.75g$	E	E	F

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3. Seismic Design Category – 1-second Period Acceleration

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

Value of S_{D1}	Seismic Use Group		
	I	II	III
$S_{D1} \ge 0.75g$	E	E	F

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4. Importance Factor (I_E)

Seismic Occupancy Category	Importance factor (I_E)
I	1.00
II	1.25
III	1.50
IV	1.00

*using IBC2000 – Table 1604.5

5. Structural System and System Parameters (R, C_d, Ω_o)

1. Bearing wall systems
2. Building frame systems
3. Moment-resisting frame system (MRF)
4. Duel systems with special MRF
5. Duel systems with intermediate MRF
6. Inversed pendulum system

System Parameters:

Response (strength) modification coefficient = R

System over-strength parameter = Ω_0

Deflection amplification factor = C_d

Structural Systems

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

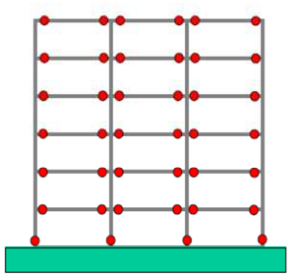
BASIC SEISMIC-FORCE-RESISTING SYSTEM	DETAILING REFERENCE SECTION	RESPONSE MODIFICATION COEFFICIENT, R^a	SYSTEM OVER-STRENGTH FACTOR, Ω_0^g	DEFLECTION AMPLIFICATION FACTOR, C_d^b	SYSTEM LIMITATIONS AND BUILDING HEIGHT LIMITATIONS (FEET) BY SEISMIC DESIGN CATEGORY[c] AS DETERMINED IN SECTION 1616.3				
					A or B	C	D[d]	E[e]	F[e]
1. Bearing Wall Systems									
A. Ordinary steel braced frames	(14)[j] 2211	4	2	3 1/2	NL	NL	160	160	160
B. Special reinforced concrete shear walls	1910.2.4	5 1/2	2 1/2	5	NL	NL	160	160	160
C. Ordinary reinforced concrete shear walls	1910.2.3	4 1/2	2 1/2	4	NL	NL	NP	NP	NP
D. Detailed plain concrete shear walls	1910.2.2	2 1/2	2 1/2	2	NL	NP	NP	NP	NP
E. Ordinary plain concrete shear walls	1910.2.1	1 1/2	2 1/2	1 1/2	NL	NP	NP	NP	NP
F. Special reinforced masonry shear walls	2106.1.1.5	5	2 1/2	3 1/2	NL	NL	160	160	100
G. Intermediate reinforced masonry shear walls	2106.1.1.4	3 1/2	2 1/2	2 1/4	NL	NL	NP	NP	NP
H. Ordinary reinforced masonry shear walls	2106.1.1.2	2 1/2	2 1/2	1 3/4	NL	160	NP	NP	NP
I. Detailed plain masonry shear walls	2106.1.1.3	2	2 1/2	1 3/4	NL	NP	NP	NP	NP
J. Ordinary plain masonry shear walls	2106.1.1.1	1 1/2	2 1/2	1 1/4	NL	NP	NP	NP	NP
K. Light frame walls with shear panels—wood structural panels/sheet steel panels	2306.4.1/ 2211	6	3	4	NL	NL	65	65	65
L. Light frame walls with shear panels—all other materials	2306.4.5	2	2 1/2	2	NL	NL	35	NP	NP
2. Building Frame Systems									
A. Steel eccentrically braced frames, moment-resisting, connections at columns away from links	(15)[j]	8	2	4	NL	NL	160	160	100
B. Steel eccentrically braced frames, nonmoment resisting, connections at columns away from links	(15)[j]	7	2	4	NL	NL	160	160	100
C. Special steel concentrically braced frames	(13)[j]	6	2	5	NL	NL	160	160	100
D. Ordinary steel concentrically braced frames	(14)[j]	5	2	4 1/2	NL	NL	160	100	100
E. Special reinforced concrete shear walls	1910.2.4	6	2 1/2	5	NL	NL	160	160	100
F. Ordinary reinforced concrete shear walls	1910.2.3	5	2 1/2	4 1/2	NL	NL	NP	NP	NP
G. Detailed plain concrete shear walls	1910.2.2	3	2 1/2	2 1/2	NL	NP	NP	NP	NP

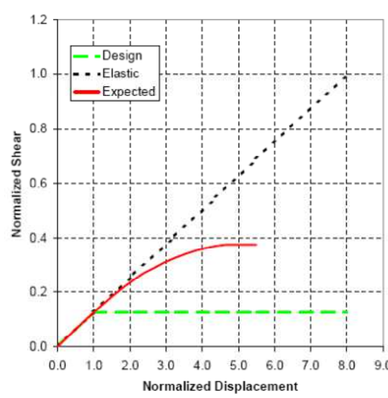
Special Moment Steel Frame

R	8
C_d	5.5
Ω_o	3

SDC

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





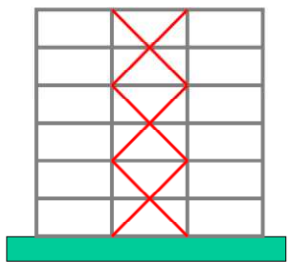
Advantages:
Architectural simplicity, relatively low base shear
Disadvantages:
Drift control, connection cost, connection testing

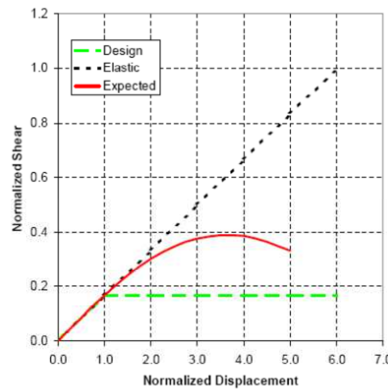
Special Steel Concentrically Braced Fame

R	6
C_d	5
Ω_o	2

SDC

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





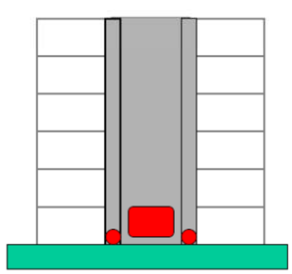
Advantages:
Lower drift, simple field connections
Disadvantages:
Higher base shear, high foundation forces, height limitations, architectural limitations

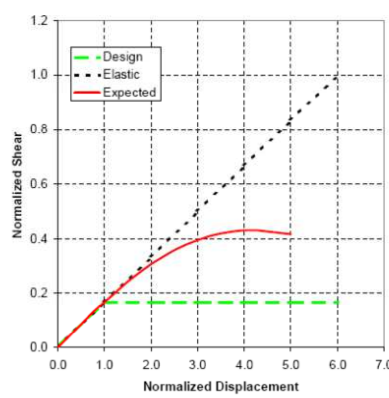
Special Reinforced Concrete Shear Wall

R	6
C_d	5
Ω_o	2.5

SDC

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





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

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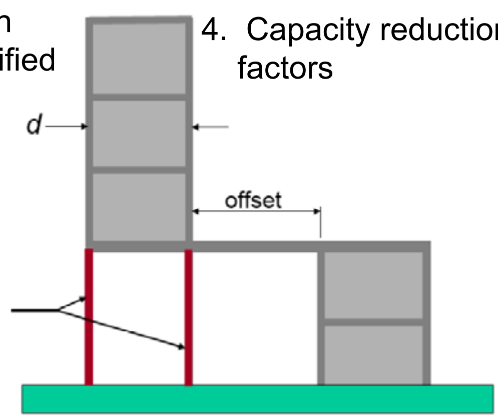
Response Modification Factor R

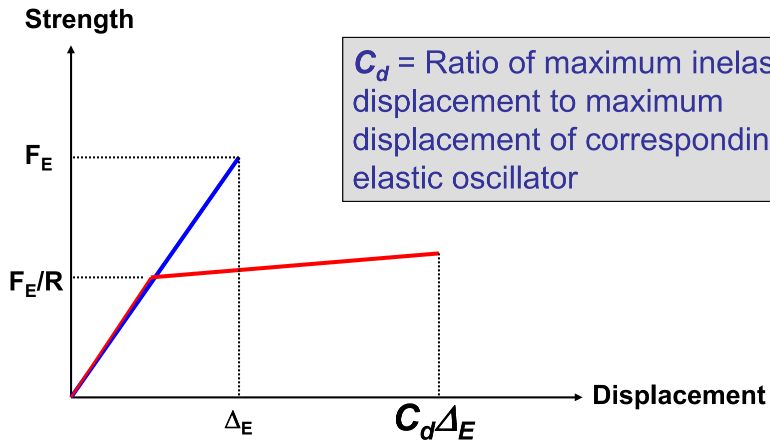
- Account for:
  - Ductility (inelastic action)
  - Overstrength
  - Redundancy
  - Damping
  - Past behavior
- Maximum = 8
  - Eccentrically braced frame with welded connections
  - Buckling restrained brace with welded connections
  - Special moment frame in steel or concrete
- Minimum = 1.5 (for cantilever systems)
  - Ordinary plain masonry shear walls

Overstrength Factor Ω_o

1. Sequential yielding of critical regions
2. Materials strength greater than specified values
3. Strength enhancement due to strain hardening
4. Capacity reduction (ϕ) factors



Deflection Amplification Factor C_d



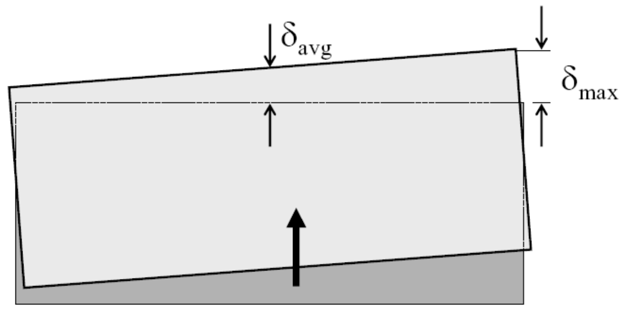
C_d = Ratio of maximum inelastic displacement to maximum displacement of corresponding elastic oscillator

6. Structure Irregularity

- Buildings shall be classified as regular or irregular based on the criteria in section 1616.5
- **Plan Irregularity**: buildings have one or more of the features listed in Table 1616.5.1
- **Vertical Irregularity**: buildings having one or more of the features listed in Table 1616.5.2

Horizontal Structural Irregularity

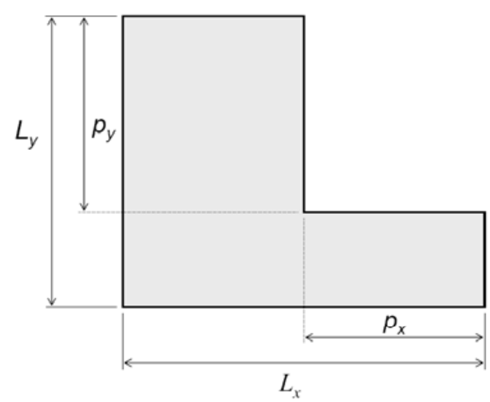
1a) and 1b) torsional irregularity



1a)	$\delta_{max} > 1.2\ \delta_{avg}$	Irregular
1b)	$\delta_{max} > 1.4\ \delta_{avg}$	Irregular

Horizontal Structural Irregularity

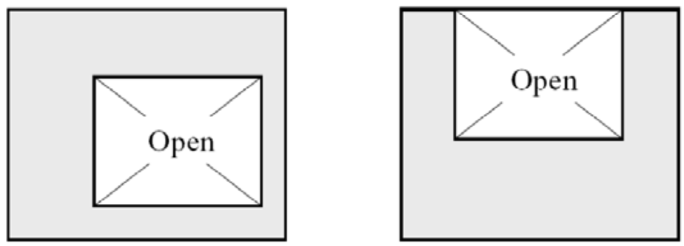
2) Re-entrant Corner Irregularity



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

Horizontal Structural Irregularity

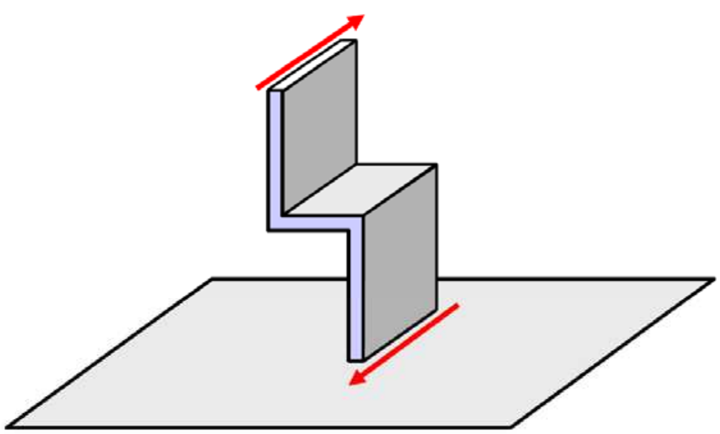
3) Diaphragm Discontinuity Irregularity



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.

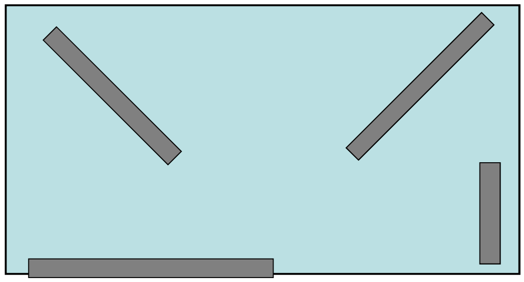
Horizontal Structural Irregularity

4) Out of Plane Offsets



Horizontal Structural Irregularity

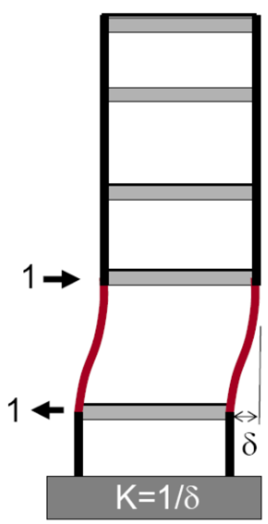
5) Non Parallel Systems Irregularity



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 Irregularity

1a) & 1b) Stiffness (Soft Storey) Irregularity



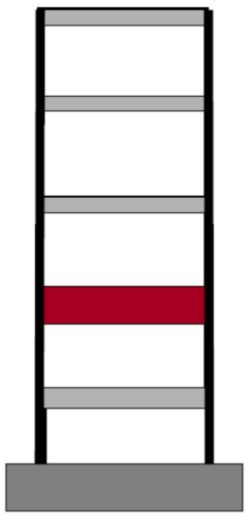
Irregularity **(1a)** exists if stiffness of any story is less than **70%** of the stiffness of the story above or less than **80%** of the average stiffness of the three stories above.

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 Irregularity
2) Weight (Mass) Irregularity

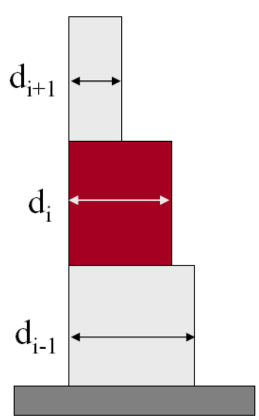


Irregularity exists if the effective mass of any story is more than **150%** of the effective mass of an adjacent story.

A roof that is lighter than the floor before need not be considered.

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

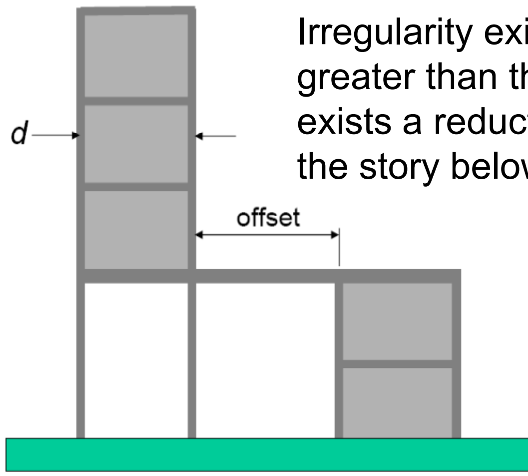
Vertical Structural Irregularity
3) Vertical Geometric Irregularity



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 Irregularity

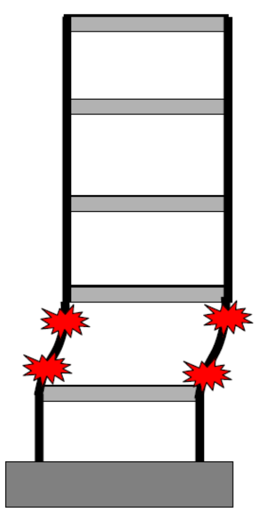
4) In-plane Discontinuity Irregularity



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

Vertical Structural Irregularity

5) Capacity (Weak-Storey) Irregularity



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

b) An extreme irregularity exists If the lateral strength of any story is less than **65%** of the strength of the story above. *(FEMA 450)*

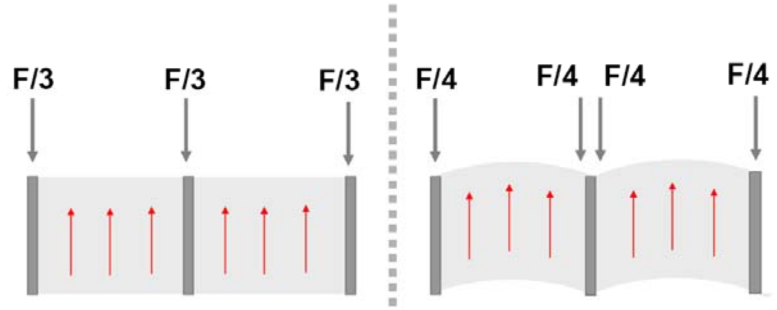
Irregularities (a) and (b) are NOT PERMITTED in SDC E or F. Irregularity (b) not permitted in SDC D.

7. 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 versus Flexible Diaphragms



RIGID
Center Wall Shear = F/3
Based on lateral stiffness ratios

FLEXIBLE
Center Wall Shear = F/2
Based on tributary area ratios

Diaphragm Flexibility

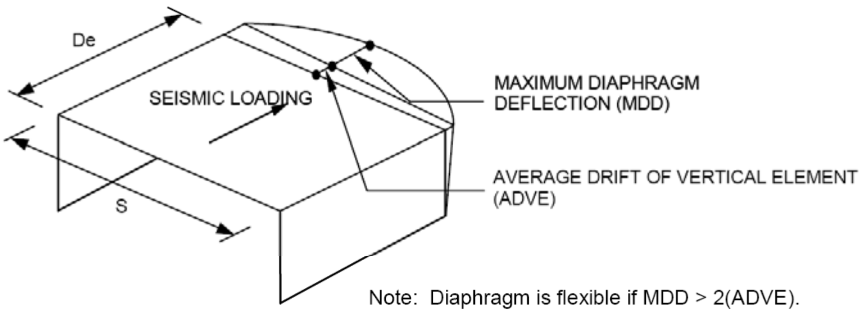


Diagram taken from ASCE 7-05

8. Redundancy factor (ρ)

- Seismic Design Categories **A, B, C** $\rho = 1.0$
- Seismic Design Categories **D, E, F** compute ρ_i for each storey and use the maximum:

  $$\rho_i = 2 - 6.1/r_{max,i}\sqrt{A_i}$$

  A_i = the floor area in m^2 of the diaphragm level immediately above the story in consideration

  r_i = the ratio of the design story shear resisted by the most heavily loaded element to the total story shear for a given direction of loading.

Structural system	r_i
Braced frames	Lateral load in the heavily loaded brace/story shear
Moment frames	Shear in two adjacent columns/story shear. In two bay frames multiply columns shear by 0.7
Shear walls	Wall shear x $3.3/L_w$
Dual systems	Multiply calculated ρ for all elements by 80% and use maximum

ρ shall not be less than 1.0 and need not exceed 1.5

Special moment resisting frames need to be arranged in away such that $\rho = 1.2$ for SDC D and $\rho = 1.1$ for SDC E & F

9. Lateral Force Analysis Procedure

- The equivalent lateral force (ELF) 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.

FEMA 450

9.1 Minimum Lateral Force

- Allowed for structures in SDC A
- Provide lateral force resisting system design to resist F_i applied at each floor level

$$F_i = 0.01\, w_i$$

F_i = design lateral force applied at each floor

w_i = portion of the total effective seismic gravity load of the structure, W, assigned to level 'i'

9.2 Simplified Analysis Procedure

- Allowed in the following cases:
  - Seismic use group I
  - Light frame building for up to three stories
  - Two story buildings
- The total design base shear, V, is given as:

$$V = 1.2 S_{DS} W/R$$

W = Total effective seismic gravity load of the structure

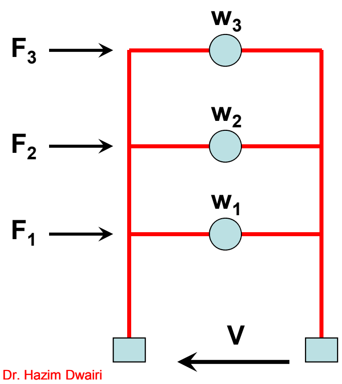
S_{DS} = Design spectral acceleration for short period

R = Response modification factor

9.2 Simplified Analysis Procedure

$$F_i = 1.2 S_{DS} w_i / R$$

w_i = portion of the total effective seismic gravity load of the structure, W, assigned to level 'i'



Design Drift: **unless exact analysis is provided, 1% of the storey height shall be assigned as relative inter-storey drift**

9.3 Equivalent Lateral Force Procedure (ELF)

Seismic Design Base Shear :

$V_B = C_s W \quad ; \quad$ Where

$$C_s = \frac{I_E}{R} S_{DS} \quad \Rightarrow \quad \begin{cases} C_{s,min} = 0.044 I_E . S_{DS} \\ C_{s,max} = \dfrac{I_E}{R.T_n} S_{D1} \end{cases}$$

For Structures in SDC = E or F and Structures with $S_{D1} \geq 0.6g$

$$C_{s,min} = \frac{0.5 I_E}{R} S_1$$

Effective Seismic Weight, W

- All Structural and nonstructural elements (Total dead load)
- 25% of the reduced storage live load, except in open parking structures and public garages it need not be considered.
- 500 N/m² minimum partition allowance
- Total weight of permanent equipments
- 20% of snow load when "flat roof" snow load exceeds 1.44 kN/m²

Approximate Period of Vibration, T_n

$$T_n = C_t (h_n)^{3/4}$$

C_t = 0.085 for steel moment frames

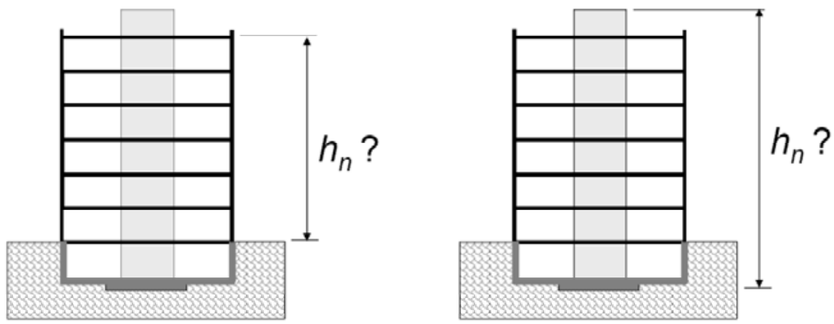
C_t = 0.073 for concrete moment frames and steel eccentrically braced frames

C_t = 0.049 for all other buildings

$$T_n = 0.1\ N$$

Buildings ONLY: For moment frames < 12 stories in height, minimum story height of 3 m. N = number of stories.

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



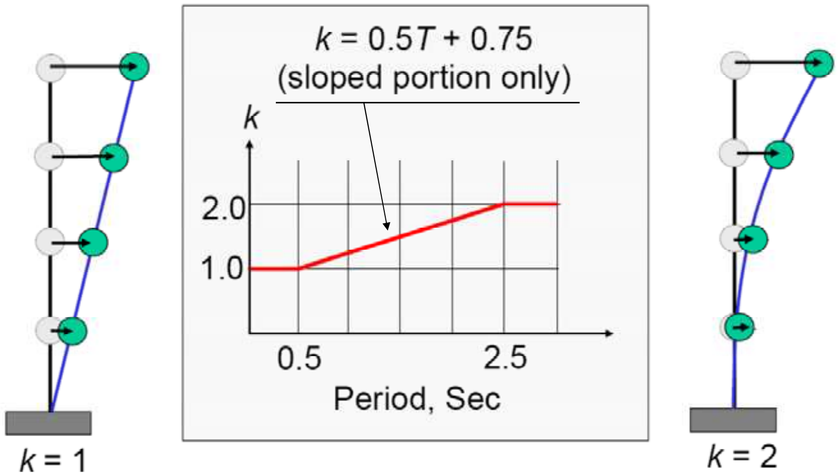
When in doubt use the lower (reasonable) value of h_n

10. Distribution of Forces along Height

$$F_x = C_{vx} V_B$$

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

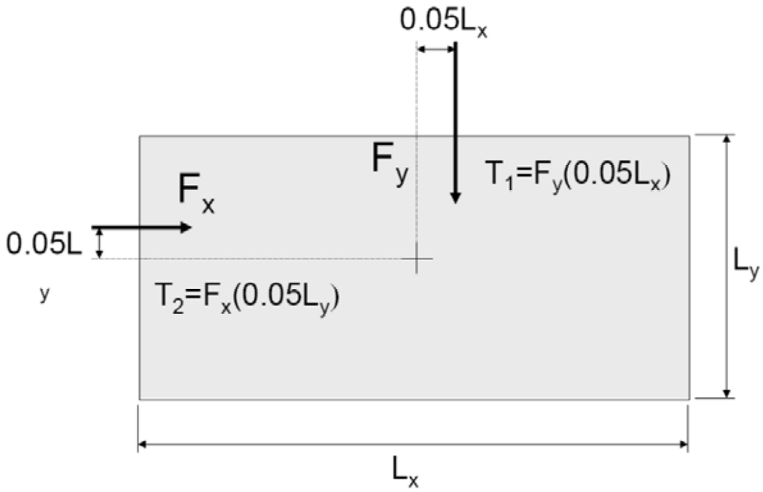
k Account for Higher Mode Effects



11. Torsional Effects

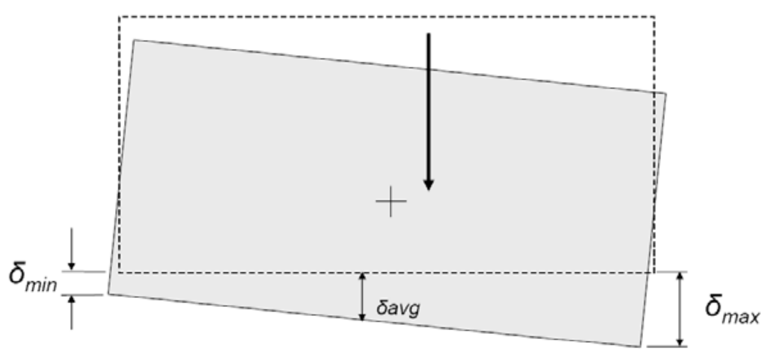
ALL	Induce inherent and accidental torsion effects
B	Ignore torsional amplification
C, D, E, F	Include torsional amplification where Type 1a or 1b irregularity exists

Accidental Torsion



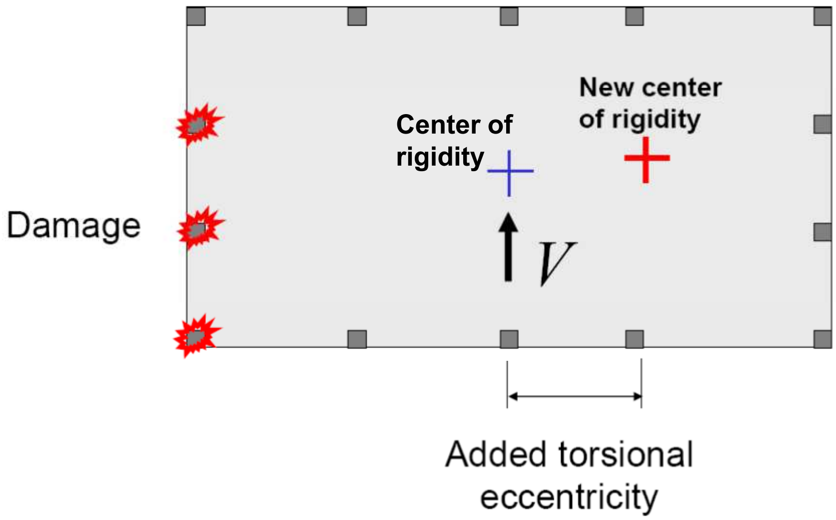
Uncertainty in the location of center of mass and center of rigidity

Amplification to Accidental Torsion



$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \leq 3.0$$

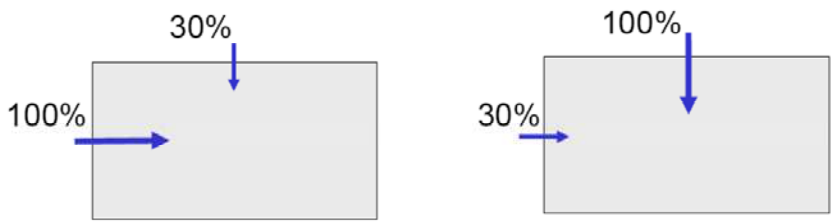
Why Amplifying Accidental Torsion?



12. Orthogonal Load Effects

- Earthquake can produce inertia forces in any direction
- Structures should be investigated for forces that act in the direction that causes the “critical load effect”
- Since this direction is not easily defined, seismic codes allow loading the structure with 100% of the seismic force in one direction and 30% of the force acting in the orthogonal direction.

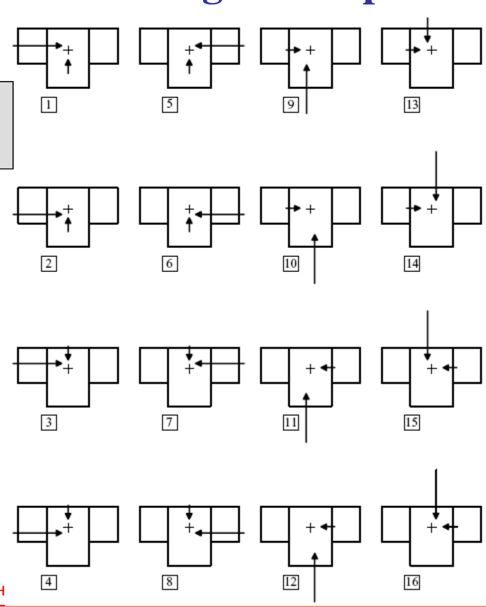
Orthogonal Load Effects, Q_E



- Applicable to S.D.C. **C**, **D**, **E**, **and F**
- Affect primarily column, especially corner columns

Nonsymmetrical Building Example

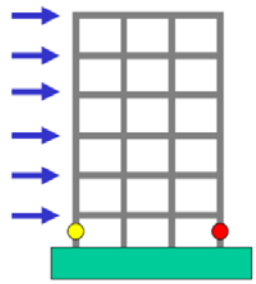
Orthogonal loading effects and accidental torsion



14. Basic Load Combinations

U = 1.2D + 1.0E + 0.5L + 0.2S

U = 0.9D + 1.0E



Note: **1.0L** instead of **0.5L** may be used when $L_o \geq 4.79$ kN/m^2 or in case of public assembly or parking garages.

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Combination of Load Effects

In load combinations, substitute the following for earthquake effect, E:

$$E = E_h \pm E_v$$

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

Resulting load combinations:

$$U = (1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L + 0.2S$$

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

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Maximum Seismic Load Effect

- Special combination for special members requires by the code:

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

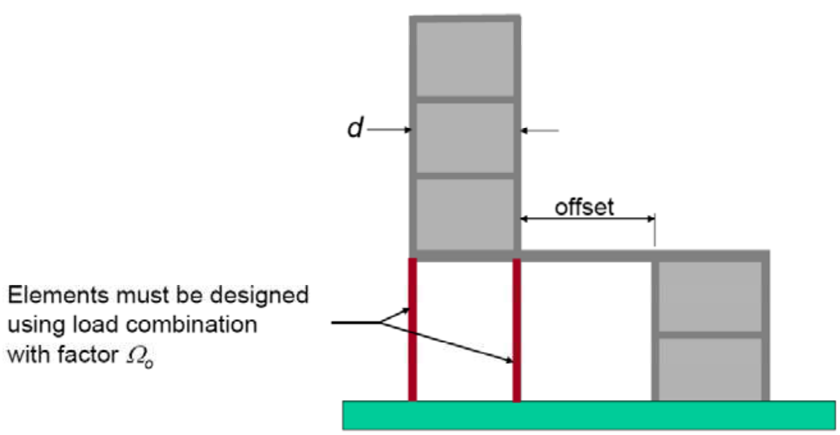
$$E_{mh} = \Omega_o Q_E \qquad E_v = 0.2 S_{DS} D$$

Resulting load combinations:

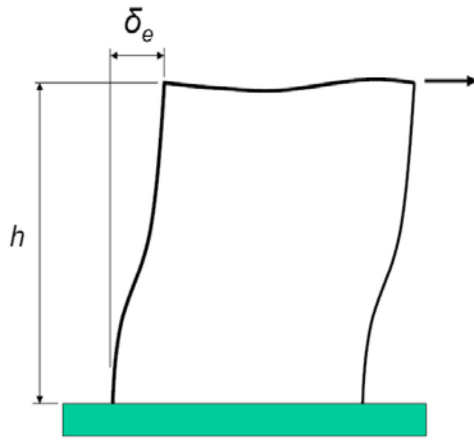
$$U = (1.2 + 0.2 S_{DS}) D + \Omega_o Q_E + 0.5L + 0.2S$$

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

Special Members



15. Storey Drift



Drift reported by analysis with strength level forces:

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

Inelastic Drift (amplified drift):

$$\Delta = C_d \Delta_e$$

$$\Delta_{P\text{-}\Delta} = \frac{\Delta}{1-\theta}$$

Drift computed at center of mass of story

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Drift Limits

Building	Seismic Use Group		
	I	II	III
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 buildings	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall buildings	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame buildings	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other buildings	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

h_{sx} is the story height below level x

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15. Overturning

The overturning moment at level x:

$$M_x = \tau \sum_{i=1}^{n} F_i (h_i - h_x)$$

F_i = portion of V_B induced at level i

h_i and h_x are the heights from the base to levels i and x

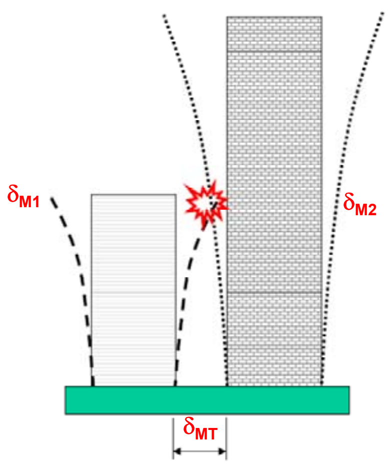
τ = the overturning moment reduction factor

= 1.0 for the top 10 stories

= 0.8 for the 20th storey from the top and below

= linear interpolation between 1.0 and 0.8 for the stories between the 10th and the 20th from top

15. Building Separation to Avoid Pounding

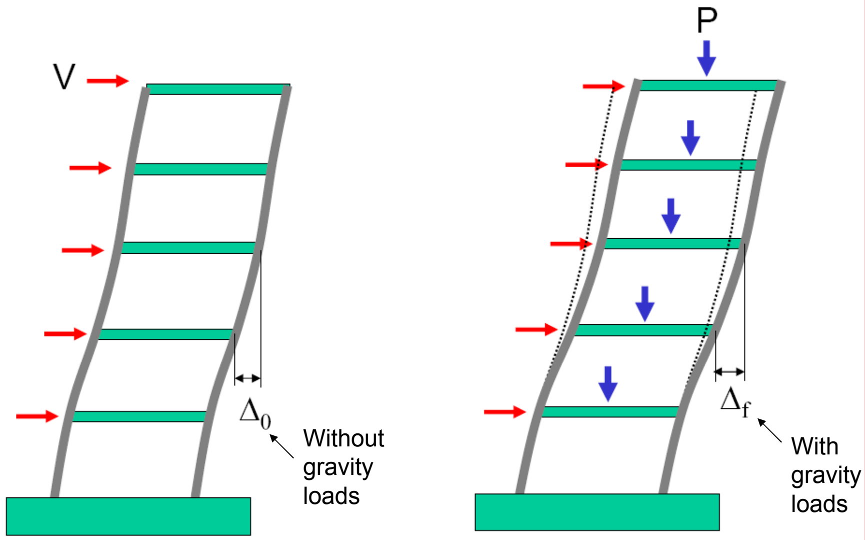


$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2}$$

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

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

15. P–Delta Effects



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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_{sx} = story height

If θ < 0.1, ignore P-delta effects

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P–Delta Effects

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

If θ > 0.1 and less than θ_{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.

Advanced Methods of Analysis

1. Modal response spectrum analysis

2. Time-history analysis

Modal Response Spectrum 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.

Modal Response Spectrum Analysis

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

Modal Response Spectrum Analysis

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
- For steel structures, include panel zone deformations
- 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.

Time-history Analysis

- 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.

Time-history Analysis

- 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

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

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Lecture 8 – Code Application Examples (UBC-97)

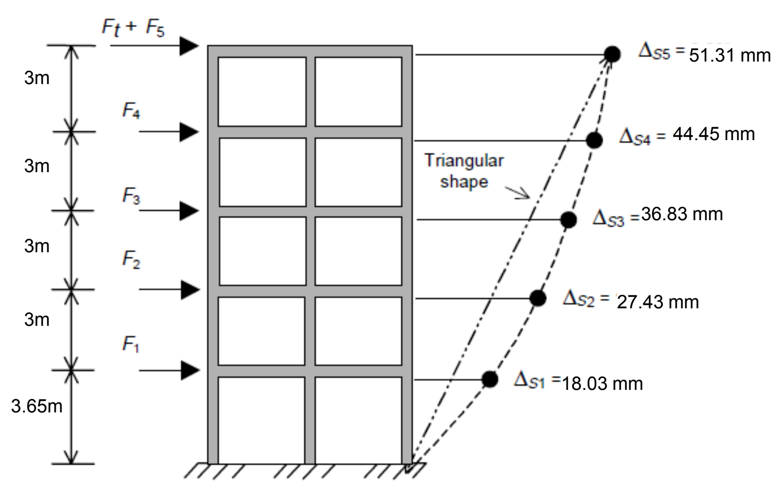
Dr. Hazim Dwairi

Code Application Examples by Structural Engineers Association of California (SEAOC)

Vertical Irregularity Type 1

- A five-story concrete special moment-resisting frame is shown below. The specified lateral forces F_x *has been applied and the* corresponding floor level displacements Δ_x *at the floor center of mass have been* found and are shown below
- **Determine if a Type 1 vertical irregularity—stiffness irregularity-soft story—exists in the first story.**

Vertical Irregularity Type 1



Vertical Irregularity Type 1

- **To determine if this is a Type 1 vertical irregularity—stiffness irregularity-soft story—here are two tests:**

1. The story stiffness is less than 70 percent of that of the story above.
2. The story stiffness is less than 80 percent of the average stiffness of the three stories above.

- If the stiffness of the story meets at least one of the above two criteria, the structure is considered to have a soft story, and a dynamic analysis is generally required under §1629.8.4 Item 2, unless the irregular structure is not more than five stories or 20-m in height

Vertical Irregularity Type 1

In terms of the calculated story drift ratios, the soft story occurs when one of the following conditions exists:

1. When 70 percent of $\frac{\Delta_{S1}}{h_1}$ exceeds $\frac{\Delta_{S2} - \Delta_{S1}}{h_2}$

   or

2. When 80 percent of $\frac{\Delta_{S1}}{h_1}$ exceeds

$$\frac{1}{3}\left[\frac{(\Delta_{S2} - \Delta_{S1})}{h_2} + \frac{(\Delta_{S3} - \Delta_{S2})}{h_3} + \frac{(\Delta_{S4} - \Delta_{S3})}{h_4}\right]$$

Vertical Irregularity Type 1

The story drift ratios are determined as follows:

$$\frac{\Delta_{S1}}{h_1} = \frac{(18.03 - 0)}{3650} = 0.00493$$

$$\frac{\Delta_{S2} - \Delta_{S1}}{h_2} = \frac{(27.43 - 18.03)}{3000} = 0.00313$$

$$\frac{\Delta_{S3} - \Delta_{S2}}{h_3} = \frac{(36.83 - 27.43)}{3000} = 0.00313$$

$$\frac{\Delta_{S4} - \Delta_{S3}}{h_4} = \frac{(44.45 - 36.83)}{3000} = 0.00254$$

$$\frac{1}{3}(0.00313 + 0.00313 + 0.00254) = 0.00293$$

Vertical Irregularity Type 1

Checking the 70 percent requirement:

$$0.70\left(\frac{\Delta_{S1}}{h_1}\right) = 0.70\,(0.00493) = 0.00345 > 0.00313$$

$\therefore$ Soft story exists

Checking the 80 percent requirement:

$$0.80\left(\frac{\Delta_{S1}}{h_1}\right) = 0.80\,(0.00493) = 0.00394 > 0.00293$$

$\therefore$ Soft story exists

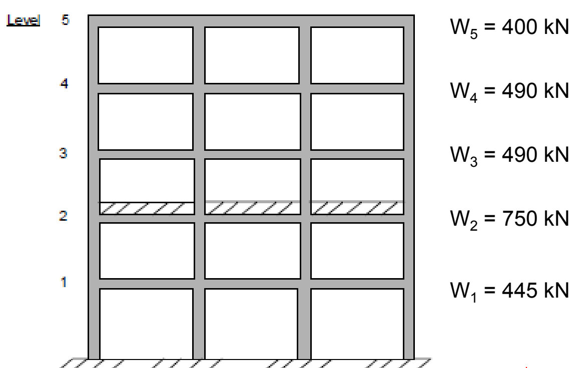
Vertical Irregularity Type 1

- Typically in practice, the check must be done for all stories.

Level	*Story Displacement*	*Story Drift*	*Story Drift Ratio*	*0.7x (Story Drift Ratio)*	*0.8x (Story Drift Ratio)*	*Avg. Story Drift Ratio of Next 3 Stories*	*Soft Story Status*
5	51.31	6.86	0.00229	0.00160	0.00183	---	No
4	44.45	7.62	0.00254	0.00178	0.00203	---	No
3	36.83	9.4	0.00313	0.00219	0.00251	---	No
2	27.43	9.4	0.00313	0.00219	0.00251	0.00265	No
1	18.03	18.03	0.00494	0.00346	0.00395	0.00294	Yes

Vertical Irregularity Type 2

- The five-story special moment frame office building has a heavy utility equipment installation at Level 2. This results in the floor weight distribution shown below:



Vertical Irregularity Type 2

A weight, or mass, vertical irregularity is considered to exist when the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. However, this requirement does not apply to the roof if the roof is lighter than the floor below.

Checking the effective mass of Level 2 against the effective mass of Levels 1 and 3

At Level 1

$$1.5 \times W_1 = 1.5\ (445)\ \text{kN} = 668\ \text{kN}$$

At Level 3

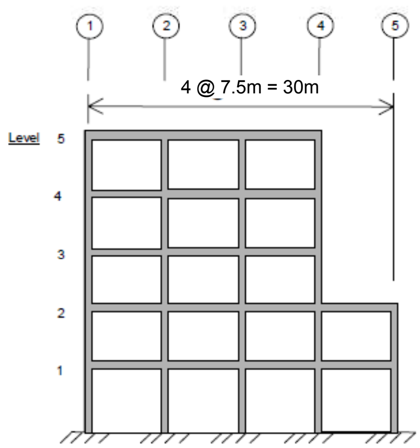
$$1.5 \times W_3 = 1.5\ (490)\ \text{kN} = 735\ \text{kN}$$

$$W_2 = 750\ \text{kN} > 668\ \text{kN} > 735\ \text{kN}$$

∴ Weight irregularity exists

Vertical Irregularity Type 3

The lateral force-resisting system of the five-story special moment frame building shown below has a 7.5m setback at the third, fourth and fifth stories.



Vertical Irregularity Type 3

A vertical geometric irregularity is considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in the adjacent story. One-story penthouses are not subject to this requirement.

In this example, the setback of Level 3 must be checked. The ratios of the two levels is

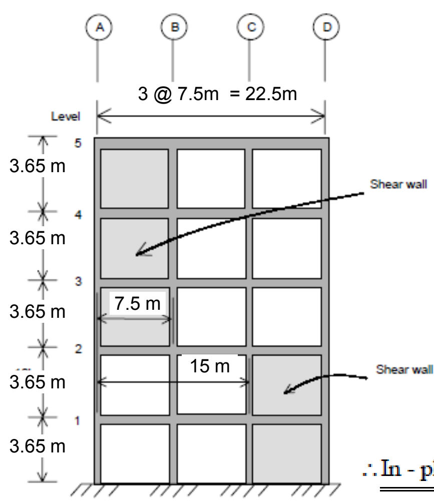
$$\frac{\text{Width of Level 2}}{\text{Width of Level 3}} = \frac{30\text{ m}}{22.5\text{ m}} = 1.33$$

$$133 \text{ percent} > 130 \text{ percent}$$

$$\therefore \underline{\underline{\text{Vertical geometric irregularity exists}}}$$

Vertical Irregularity Type 4

A concrete building has the building frame system shown below. The shear wall between Lines A and B has an in-plane offset from the shear wall between Lines C and D.



$$\therefore \underline{\underline{\text{In - plane discontinuity exists}}}$$

Vertical Irregularity Type 4

- Columns under wall A-B shall be designed for:

$$1.2D + f_1 L + 1.0E_m \quad (12\text{-}17)$$

$$0.9D \pm 1.0E_m \quad (12\text{-}18)$$

WHERE:

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 psf (4.79 kN/m^2), and for garage live load.

= 0.5 for other live loads.

Collector element B-C at level 2 shall be designed according to 1633.2.6

$$E = \rho E_h + E_v$$

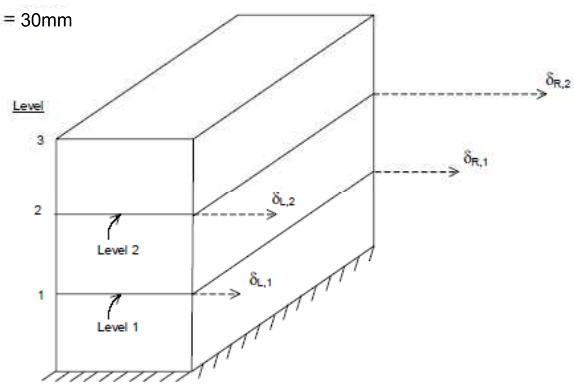
$$E_m = \Omega_o E_h$$

Plan Irregularity Type 1

A three-story special moment resisting frame building has rigid floor diaphragms. Under specified seismic forces, including the effects of accidental torsion, it has the following displacements at Levels 1 and 2:

$\delta_{L,2}$ = 33mm $\delta_{R,2}$ = 48mm

$\delta_{L,1}$ = 25mm $\delta_{R,1}$ = 30mm



Plan Irregularity Type 1

1. **Determine if a Type 1 torsional irregularity exists at the second story.**

   If it does:

2. **Compute the torsional amplification factor A_x for Level 2.**

Referring to the above figure showing the displacements δ due to the prescribed lateral forces, this irregularity check is defined in terms of story drift $\Delta\delta_X = (\delta_X - \delta_{X-1})$ at ends R (right) and L (left) of the structure. Torsional irregularity exists at level x when

$$\Delta_{max} = \Delta_{R,X} > \frac{1.2(\Delta_{R,x} + \Delta_{L,x})}{2} = 1.2(\Delta_{avg})$$

where

$$\Delta\delta_{L,2} = \delta_{L,2} - \delta_{L,1}$$

$$\Delta\delta_{R,2} = \delta_{R,2} - \delta_{R,1}$$

$$\Delta\delta_{max} = \Delta\delta_{R,X}, \quad \Delta\delta_{avg} = \frac{\Delta\delta_{L,X} + \Delta\delta_{R,X}}{2}$$

Determining story drifts at Level 2

$$\Delta_{L,2} = 33 - 25 = 8 \text{ mm}$$

$$\Delta_{R,2} = 48 - 30 = 18 \text{ mm}$$

$$\Delta_{avg} = \frac{8 + 18}{2} = 13 \text{ mm}$$

Checking 1.2 criteria

$$\frac{\Delta_{max}}{\Delta_{avg}} = \frac{\Delta_{R,2}}{\Delta_{avg}} = \frac{18}{13} = 1.4 > 1.2$$

∴ Torsional irregularity exists

Compute amplification factor A_x for Level 2.

When torsional irregularity exists at a level x, the accidental eccentricity, equal to 5 percent of the building dimension, must be increased by an amplification factor A_x. This must be done for each level, and each level may have a different A_x value. In this example, A_x is computed for Level 2.

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

$$\delta_{max} = \delta_{R,2} = 48 \text{ mm}$$

$$\delta_{avg} = \frac{\delta_{L,2} + \delta_{R,2}}{2} = \frac{33 + 48}{2} = 51 \text{ mm}$$

$$A_2 = \left(\frac{48}{1.2 \times 51} \right)^2 = 0.62 < 1.0$$

$\therefore$ use $A_x = 1.0$

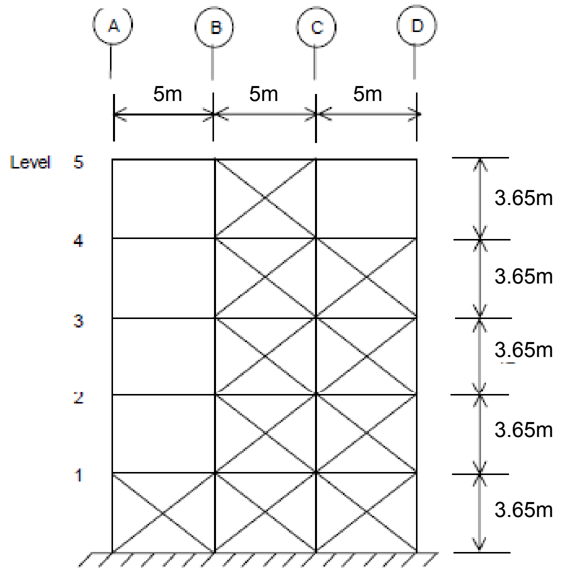
Reliability/Redundancy Factor

Evaluate the reliability/redundancy factor, ρ, for the three structural systems shown below. Given information for each system includes the story shears V_i due to the design base shear V, and the corresponding element forces E_h. The ρ factor is defined as

$$\rho_i = 2 - 6.1/r_{max,i}\sqrt{A_B}$$

where r_{max} is the largest of the element-story shear ratios, r_i, that occurs in any of the story levels at or below the two-thirds height level of the building; and A_B is the ground floor area of the structure in m^2. Once ρ has been determined, it is to be used in Equation (30-1) to establish the earthquake load E for each element of the lateral force-resisting system.

1. **Braced frame structure.**



The following information is given:

Story i	*Total Story Shear V_i (kN)*	*Brace Force E_h(kN)*	*Horizontal Component F_x (kN)*	*r_i = F_x/V_i*
1	4235	1215	972	0.230
2	3250	1300	1040	0.320
3	2300	500	400	0.174
4	1420	400	320	0.225
5	Not required above 2/3 height level			

A_B = 14.5m x 30.m = 435 m^2, where 30m is the building width

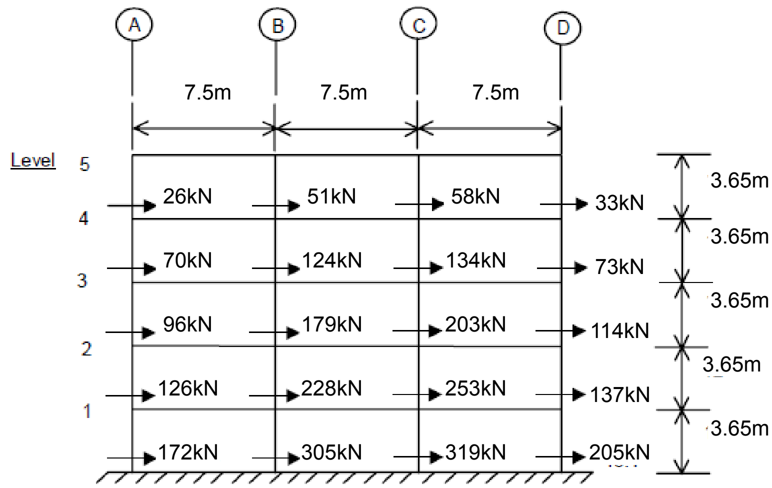
Horizontal component in each brace is

$F_x = 4/5\ E_h$ where E_h is the maximum force in a single brace element in story i.
For braced frames, the value of r_i is equal to the maximum horizontal force component F_x in a single brace element divided by the total story shear V_i.

$r_{max} = 0.320$ ➔

$\rho = 2 - 6.1 / (0.32 \sqrt{435}) = 1.09$

2. **Moment frame structure.**



Building first floor area:

A_B = 22.5m x 36.5m = 821.25 m^2

Column shears are given above.

$E_h = V_A, V_B, V_C, V_D$ in column lines A, B, C, D, respectively.

Column Lines B and C are common to bays on opposite sides.

For moment frames, r_i is maximum of the sum of $V_A + 0.7V_B$, *or* $0.7(V_B + V_C)$, *or* $0.7V_C + V_D$ divided by the story shear V_i.

Section 1630.1.1 requires that special moment-resisting frames have redundancy such that the calculated value of ρ does not exceed 1.25.

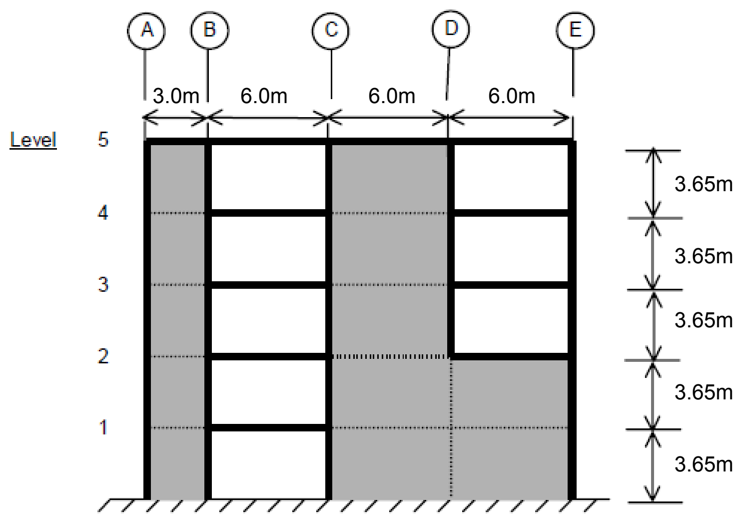
The story shears and r_i evaluations are:

Story i	*Vi* *kN*	$V_A + 0.7V_B$ *kN*	$0.7V_B + 0.7V_C$ *kN*	$0.7V_C + V_D$ *kN*	r_i
1	1726	385.5	436.8	428.3	0.253
2	1361	285.6	336.7	314.1	0.247
3	1014	221.3	267.4	256.1	0.264
4	672	156.8	180.6	166.8	0.269
5	Not required above 2/3 height level				

r_{max} = 0.269 ➔

$\rho = 2 - 6.1 /(0.269 \sqrt{821.25}) = 1.21 < 1.25$ *o.k.*

3. **Building frame system with shear walls.**



A_B = 21.5m x 36.5m = 784.75 m^2

E_h is the wall shear V_w

For shear walls, r_i is the maximum of $(V_{wi}/V_i)(3.3/L_w)$

The following information is given for the walls:

Story i	V_i (kN)	Wall A-B		Wall C-D-E and C-D	
		V_{wi} (kN)	L_{wi} (m)	V_{wi} (kN)	L_{wi} (m)
1	1615	152	3	411	12
2	1281	120	3	335	12
3	925	162	3	308	6
4	467	88	3	177	6
5	Not required above 2/3 height level				

Story i	V_i	Wall A-B	Wall C-D-E and C-D	r_i
		(V_{wi}/Vi) $(3.3/L_w)$	(V_{wi}/Vi) $(3.3/L_w)$	
1	1615	0.104	0.070	0.104
2	1281	0.103	0.072	0.103
3	925	0.193	0.183	0.193
4	467	0.207	0.208	0.208
5	Not required above 2/3 height level			

r_{max} = 0.208 ➔

$\rho = 2 - 6.1 /(0.208\ \sqrt{784.75}) = 0.953 < 1.0$

➔ <u>USE ρ = 1.0</u>

P-D Effect

A 15-story building has a steel special moment-resisting frame (SMRF). The following information is given:

Zone 4
$R = 8.5$

At the first story,

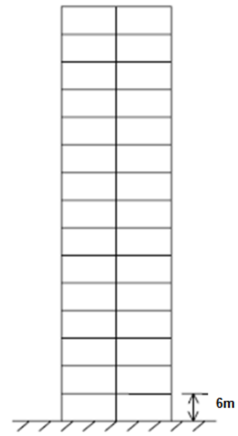
$\Sigma D = W =$ **38,446 kN**

$\Sigma L =$ **17,126 kN**

$V_1 = V = 0.042W =$ **1,615 kN**

$h_1 =$ **6.0 m**

Story drift $= \Delta_{S1} = 0.003h_1 =$ **0.018 m**



Determine the following:

1. **$P\Delta$ criteria for the building.**

2. **Check the first story for $P\Delta$ requirements.**

1. **$P\Delta$ criteria for the building.**

$P\Delta$ effects must be considered whenever the ratio of secondary moments to primary moments exceed 10 percent. As discussed in Section C105.1.3 of the 1999 SEAOC Blue Book Commentary, this ratio is defined as a stability coefficient θ:

$$\theta_x = \frac{P_x \Delta_{sx}}{V_x h_x}$$

where

θ_x = stability coefficient for story x

P_x = total vertical load (unfactored) on all columns in story x

Δ_{sx} = story drift due to the design base shear

V_x = design shear in story x

h_x = height of story x

$P\Delta$ effects must be considered when $\theta > 0.10$

Section 1630.1.3 requires that the total vertical load P_1 at the first story be considered as the total dead (ΣD) plus floor live (ΣL) and snow (S) load above the first story. These loads are unfactored for determination of $P\Delta$ effects.

$$P_1 = \Sigma D + \Sigma L + S$$

using $S = 0$ for the building site

$$P_1 = 38{,}446 + 17{,}126 = 55{,}572 \text{ kN}$$

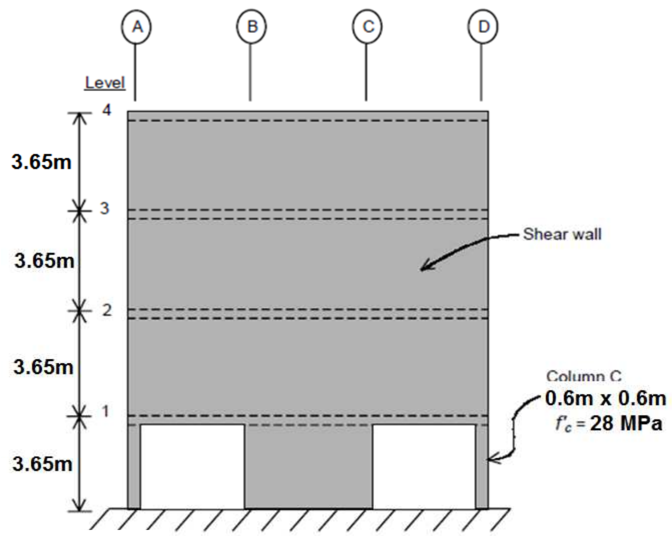
$$\theta_1 = \frac{P_1 \Delta_{S1}}{V_1 h_1} = \frac{(55{,}572)\,(0.018)}{(1{,}615)\,(6.0)} = 0.103 > 0.1$$

$\therefore$ <u>$P\Delta$ effects must be considered</u>

Elements Supporting Discontinuous Systems

A reinforced concrete building has the lateral force-resisting system shown below. Shear walls at the first floor level are discontinuous between Lines A and B and Lines C and D. The following information is given:

Zone 4
Concrete shear wall building frame system: $R = 5.5$ and $\Omega_o = 2.8$
Office building live load: $f_1 = 0.5$
Axial loads on column C: $D = 178$ kN $L = 90$ kN $E_h = 445$ kN



Determine the following for column C:

1. **Required strength.**

2. **Detailing requirements.**

1. **Required strength.**

Because of the discontinuous configuration of the shear wall at the first story, the first story columns on Lines A and D must support the wall elements above this level. Column "C" on Line D is treated in this example. Because of symmetry, the column on Line A would have identical requirements.

Section 1630.8.2 requires that the column strength be equal to or greater than

$$P_u = 1.2D + f_1 L + 1.0E_m$$

$$P_u = 0.9D \pm 1.0E_m$$

where

$$E_m = \Omega_o E_h = 2.8\ (445) = 1246\ \text{kN}$$

Substituting the values of dead, live and seismic loads

P_u = **1.2 (178) + 0.5 (90) + 1246 = 1505 kN** compression, and

P_u = **0.9 (178) - 1.0 (1246) = -1067 kN** tension

2. **Detailing requirements.**

The concrete column must meet the requirements of §1921.4.4.5. This section requires transverse confinement tie reinforcement over the full column height if

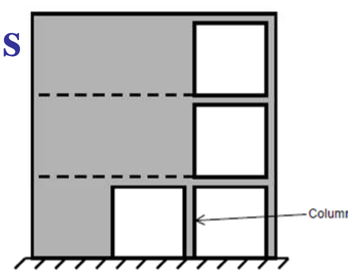
$$P_u > \frac{A_g f'_c}{10} = \frac{(0.6\text{m})^2(28\text{MPa})}{10} = 1008 \text{ kN}$$

$$P_u = 1505 \text{ kN} > 1008 \text{ kN}$$

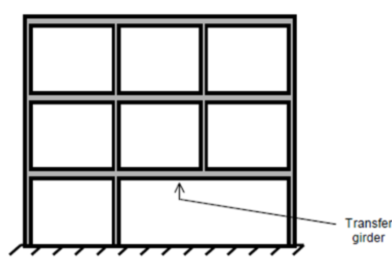
$\therefore$ Confinement is required over the full height

Vertical Irregularities

1. **Discontinuous shear wall.** The wall at left has a Type 4 vertical structural irregularity.

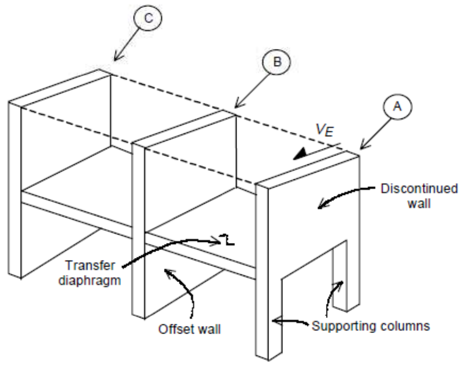


2. **Discontinuous column.** This frame has a Type 4 vertical structural irregularity.



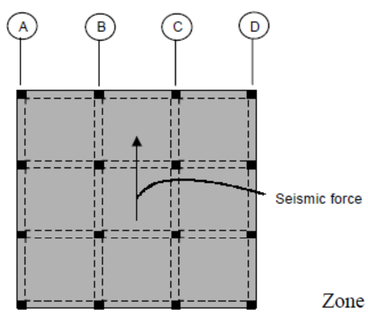
Vertical Irregularities

3. **Out-of-plane offset.** The wall on Line A at the first story is discontinuous. This structure has a Type 4 plan structural irregularity, and §1620.8.2 applies to the supporting columns. The portion of the diaphragm transferring shear (i.e., transfer diaphragm) to the offset wall must be designed for shear wall detailing requirements, and the transfer loads must use the reliability/redundancy factor ρ for the vertical-lateral-force-resisting system.



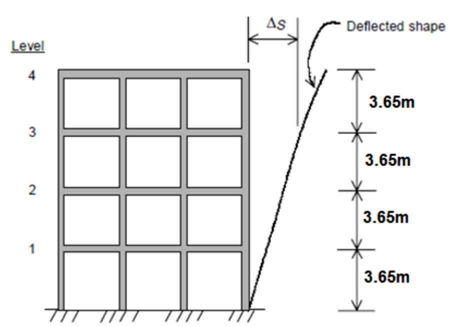
Story Drift

A four-story special moment-resisting frame (SMRF) building has the typical floor plan as shown below. The elevation of Line D is also shown, and the following information is given:



Typical floor plan

Zone 4
$I = 1.0$
$R = 8.5$
$\Omega_o = 2.8$
$T = 0.60$ sec



Elevation of Line D

The following are the design level response displacements Δ_S (total drift) for the frame along Line D. These values include both translational and torsional (with accidental eccentricity) effects. As permitted by §1630.10.3, Δ_S has been determined due to design forces based on the unreduced period calculated using Method B.

Level	Δ_S
4	**38 mm**
3	**26mm**
2	**16mm**
1	**7.6mm**

For the frame on Line D, determine the following:

1. **Maximum inelastic response displacements Δ_M.**
2. **Story drift in story 3 due to Δ_M.**
3. **Check story 3 for story drift limit.**

1. **Maximum inelastic response displacements Δ_M.**

These are determined using the Δ_S values and the R-factor

$$\Delta_M = 0.7R\Delta_S = 0.7(8.5)(\Delta_S) = 5.95\Delta_S$$

Therefore

Level	Δ_S	Δ_M
4	**38 mm**	**226mm**
3	**26mm**	**155mm**
2	**16mm**	**95mm**
1	**7.6mm**	**45mm**

2. **Story drift in story 3 due to Δ_M.**

Story 3 is located between Levels 2 and 3. Thus

Δ_M drift = **155 - 95 = 60 mm**

3. **Check story 3 for story drift limit.**

For structures with a fundamental period less than 0.7 seconds, §1630.10.2 requires that the Δ_M story drift not exceed 0.025 times the story height.

For story 3

Story drift using Δ_M = **60 mm**

Story drift limit = **0.025 x (3.65m) x 1000 = 91.3mm > 60 mm**

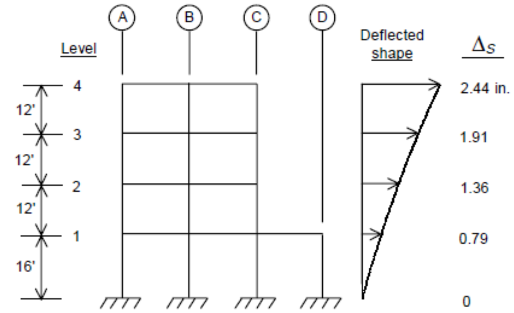
∴ Story drift is within limits

Story Drift Limits

For the design of new buildings, the code places limits on story drifts. The limits are based on the maximum inelastic response displacements and not the design level response displacements determined from the design base shear of §1630.2.

In the example given below, a four-story steel special moment-resisting frame (SMRF) structure has the design level response displacements Δ_S shown. These have been determined according to §1630.9.1 using a static, elastic analysis.

Zone 4
T = 0.60 sec.
R = 8.5



Determine the following:

1. **Maximum inelastic response displacements.**

2. **Compare story drifts with the limit value.**

1. **Maximum inelastic response displacements.**

Maximum inelastic response displacements, Δ_M , are determined from the following:

$$\Delta_M = 0.7R\Delta_S$$

$$\therefore\ \Delta_M = 0.7\,(8.5)\,\Delta_S = \underline{\underline{5.95\Delta_S}}$$

2. **Compare story drifts ith the limit value.**

Using Δ_M story displacements, the calculated story drift cannot exceed 0.025 times the story height for structures having a period less than 0.7 seconds.

Check building period.

$$T = .60\,\text{sec} < .70\,\text{sec}$$

Therefore, limiting story drift is 0.025 story height.

Determine drift limit at each level.

Levels 4, 3, and 2

$$\Delta_M \text{ drift} \leq .025h = .025\,(12\text{ ft} \times 12\text{ in./ft}) = 3.60\text{ in.}$$

Level 1

$$\Delta_M \text{ drift} \leq .025h = .025\,(16\text{ ft} \times 12\text{ in./ft}) = 4.80\text{ in.}$$

For Δ_M drift $= \Delta_{Mi} - \Delta_{Mi-1}$, check actual story drifts against limits:

Level i	Δ_S	Δ_M	Δ_M *drift*	*Limit*	*Status*
4	2.44 in.	14.52 in.	3.16 in.	3.60 in.	*o.k.*
3	1.91	11.36	3.27	3.60	*o.k.*
2	1.36	8.09	3.39	3.60	*o.k.*
1	0.79	4.70	4.70	4.80	*o.k.*

Therefore, the story drift limits of §1630.10 are satisfied.

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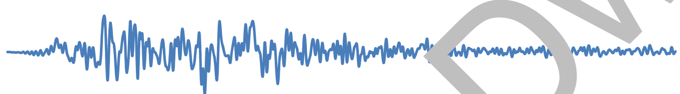
Department of Civil Engineering

Introduction to Earthquake Engineering

Equivalent Lateral Force According to UBC-97

Dr. Hazim Dwairi



Simplifies Equivalent Lateral Force

- ✓ Building with light frames & stories equal to 3.0 or less excluding basement .
- ✓ All other buildings of 2 stories .

$$V_B = \frac{3.0\,Ca}{R} W_D$$

Vertical Distribution

$$F_x = \frac{3.0\,Ca}{R} W_x$$

Equivalent Lateral Force

- Applicable only for:

✓ Regular & Irregular buildings in seismic zone1 and normal occupancy building in zone 2A , 2B

✓ Regular buildings 70m height or less.

✓ Irregular buildings of 7 stories or 25m in height or less.

Equivalent Lateral Force

- **Base shear**
- $V_B = \frac{C_v I}{R T_a} W_o$
- $V_{B,min} = 0.1 C_a I W_o$
- $V_{B,max} = \frac{2.5 C_a I}{R} W_o$

- **Vertical Distribution**
- $V_B = F_t + \sum_{x=1}^{n} F_x$

- $F_t = 0.07 T_a V_B$
- $F_t = 0.0$ if $T_a < 0.7$ sec

Equivalent Lateral Force

- $Fx = \frac{(V_B - F_t) W_x h_x}{\sum_{x=1}^{n} W_x h_x}$
- Natural Period Ta = $C_t \, (h_n)^{3/4}$

Ct = $\frac{1}{12}$, steel MRF
Ct = $\frac{1}{14}$, Rc-MRF
Ct = $\frac{1}{25}$, Bearing wall or MRF with infill panels
Ct = $\frac{1}{20}$, Other building

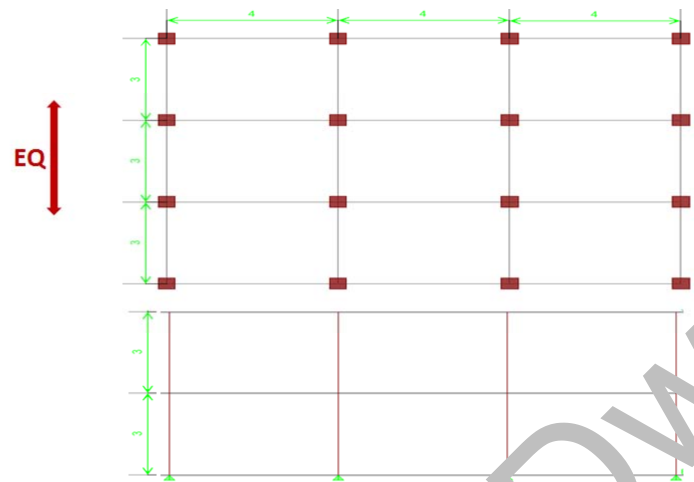
For shear wall :
Ct = $\frac{1}{13.5\sqrt{A_e}}$
Ae = $\sum A_w \, (0.2 + (l_w/h_n)^2$

l_w : length of shear walls in first floor
A_w : Area of shear wall in first floor

Example 1

- A two story building with story height = 3.0m, lies in seismic zone 2B on soil type S_D. The structural system consists of four RC-RMF in each direction . The interior columns have double the stiffness of exterior columns .
- The 1st & 2nd floors carry a total weight of q_D= 10 kN/m^2 & 7 kN/m^2 respectively.
- Find the seismic force for each of the four frames if the EQ is in the direction shown .

Example 1



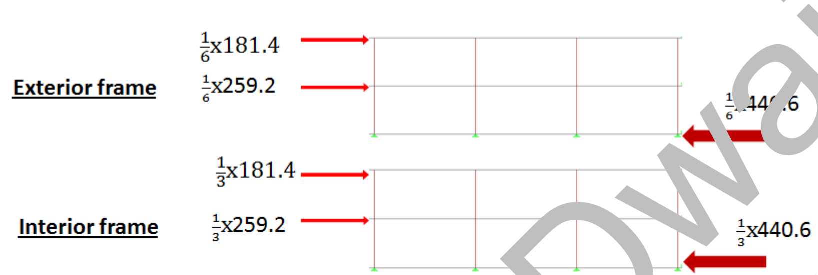
Example 1

- **<u>Use simplified Method :</u>**

- 2B , S_D → C_a= 0.28g.
- Ordinary RC-MRF → R= 3.5 .
- Seismic weight :
  Area per floor = 12x9=108 m^2.
  Wt. of 1st floor = 108x10=1080 kN.
  Wt. of 2nd floor = 108x7 = 756 kN.
  Total = 1,836 kN.

Example 1

- $V_B = \frac{3\,C_a}{R} W_D = \frac{3 \times 0.28}{3.5}(1836) = 440.6$ kN.
- $F_1 = \frac{3 \times 0.28}{3.5}(1080) = 259.2$ kN.
- $F_2 = \frac{3 \times 0.28}{3.5}(756) = 181.4$ kN.



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Example 2

- An eight story building weight story height =3m lies in siesmic zone 2B on soil type S_D. The structural system consists of light MRF for gravity loading & seven shear walls (ordinary) for lateral load resisting.
- The first seven floors carry atotal load weight of 10 kN/m^2 , while the eighth floor carry 7 kN/m^2.
- Find the siesmic force for each of the 3 shear walls marked (1,2,3) if the EQ is along walls . Stiffness of the walls is equal.

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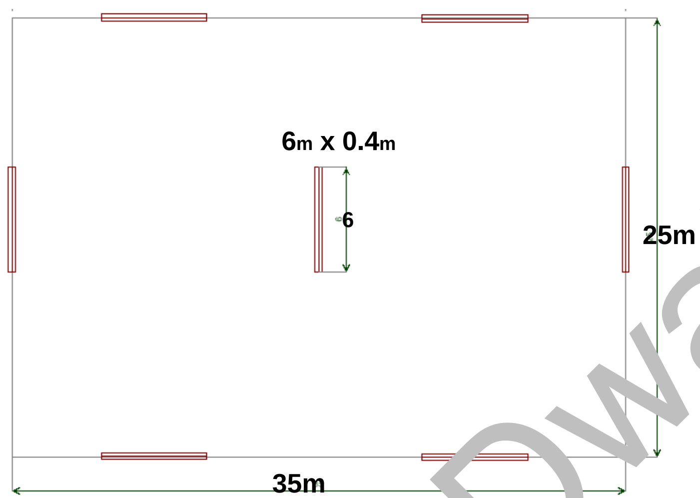
Example 2



Example 2

- C_a= 0.28g , C_v= 0.4g , R= 4.0
- Approximate Period Ta :

  $Ae = \sum A_w \left[0.2 + \left(\frac{l_{w1}}{h_n}\right)^2\right] = (3x.4x6)\left[0.2 + \left(\frac{3x6}{8x3}\right)^2\right] = 5.49m^2.$

  $Ct = \frac{1}{13.5\sqrt{A_e}} = \frac{1}{13.5\sqrt{5.49}} = 0.032$

  $Ta = 0.023\ (8x3)^{3/4} = 0.343$ sec.

- Mass weight :

  1st seven floor = 25x35x10 = 8750 kN.

  2nd floor = 25x35x7 = 6125 kN.

  Total =7x8750 + 6125 = 67,375 kN.

Example 2

- $V_B = \frac{C_v I}{R T_a} W_D = \frac{0.4 x 1}{4 x 0.343} x67,375 = 19,645\ kN$
- $V_{B,max} = \frac{2.5\ C_a I}{R} W_D$ = 11,791 kN
- $V_{B,min} = 0.11\ C_a\ I\ W_D$ = 2075 kN.
  → V_B = 11,791 kN.
- Ta < 0.7sec → Ft=0.0
- 1[st] floor → total V = V_B
  V1=V2=V3 = $\frac{1}{3}x$ 11,791 = 3930.2 kN

floor	Wi	hi	Fx (kN)
1	8750	3	319.7
2	8750	6	639.3
3	8750	9	959
4	8750	12	[illegible]
5	8750	15	[illegible]
6	8750	18	[illegible]
7	8750	21	[illegible]
8	[illegible]	[illegible]	2557.3

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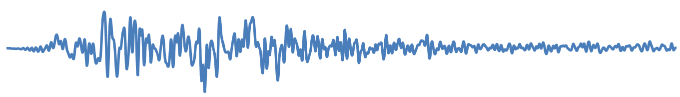
Department of Civil Engineering

Introduction to Earthquake Engineering

Structural Walls (Shear Walls)

Dr. Hazim Dwairi



Structural Walls

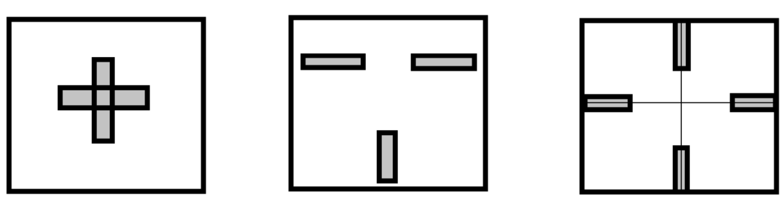
- Also known as Shear Walls .
- Term “shear” refers to historical perspective on how such walls behave.
- It also can be designed to behave as flexural walls.
- For buildings up to 20 stories , the use of structural walls is a matter of choice.
- For buildings over 30 stories , structural walls may become imperative from the point of view of economy and deflection control.

Structural Walls

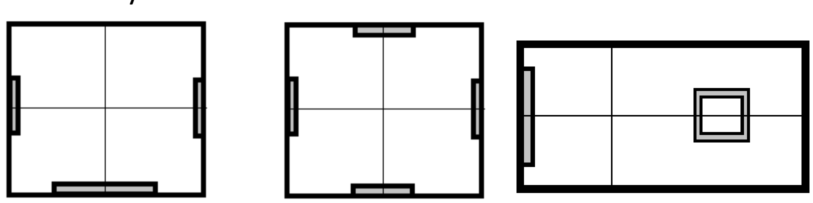
- Building braced by structural walls are stiffer than framed structures , which reduces the possibility of excessive deformations under small EQs.
- Necessary strength to avoid structural damage under moderate EQs can be achieved by properly detailed longitudinal & transverse reinforcement. Durable ductile response can be achieved under major EQs.
- The position of structural walls within a building are usually dictated by functional requirements . These may or may not suit structural planning .
- Building sites , architectural interests , or clients desires may position wall in undesirable location .

Layout Distribution

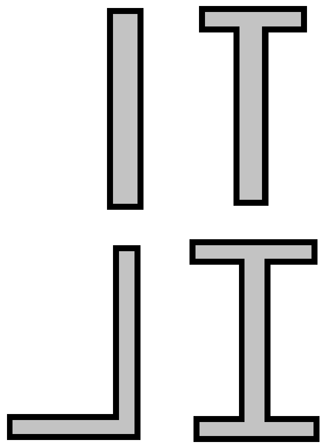
- Torsionally Unstable :



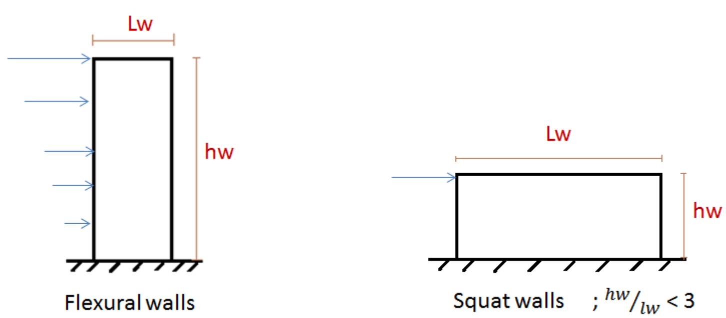
- Torsionally Stable:



Common wall section "top view"



Vertical Variation & Coupling Of Walls

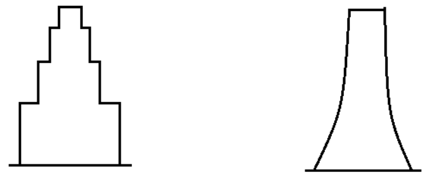


- For Squat walls , according to NZ code magnify lateral load on the wall by Z1 factor .

$$1.0 \le \left[Z1 = 2.5 - 0.5\frac{hw}{lw}\right] \le 2.0$$

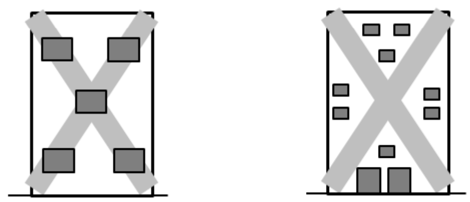
Vertical Variation & Coupling Of Walls

- Optimized Design :



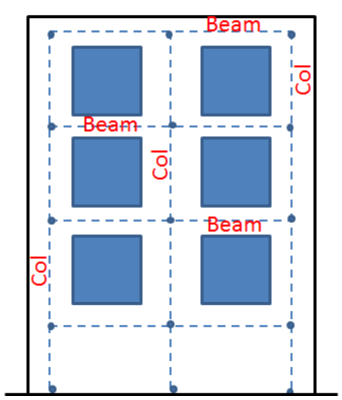
- Coupling Of Walls :

Wrong



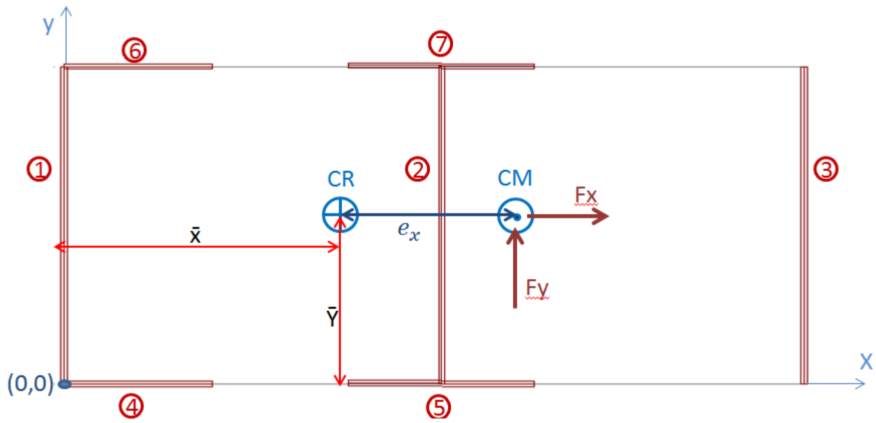
Stay away from Comp. field

Vertical Variation & Coupling Of Walls



- It works as a Frame .
- “ Weak Beam , Strong Col “

Distribution of story inertia forces between walls



- CM : center of mass
- CR : Center of rigidity
- Fx , Fy : Story shear in the (X,Y) - directions.
- e_x : eccentricity in the X- direction.

Distribution of story inertia forces between walls

- Under direct transition , story force is distributed propotional to wall stiffness .
  V_x= ¼ Fx ; walls 4,5,6,7
- Out of plane stiffness of walls 1,2,3 is Neglected.
  V_y= ⅓ Fy.
- <u>In General:</u>
- $V_{xi} = \frac{K_{xi}}{\sum K_{xi}}$ Fx , $V_{yi} = \frac{K_{yi}}{\sum K_{yi}}$ Fy
- If torsion exists ; CM≠CR ; then torsion in the building will alter the shear forces distribution.
- <u>Note</u> : even if CM=CR in design , we still assume **e= 0.05 L** in each direction , SO → Torsion always exists.

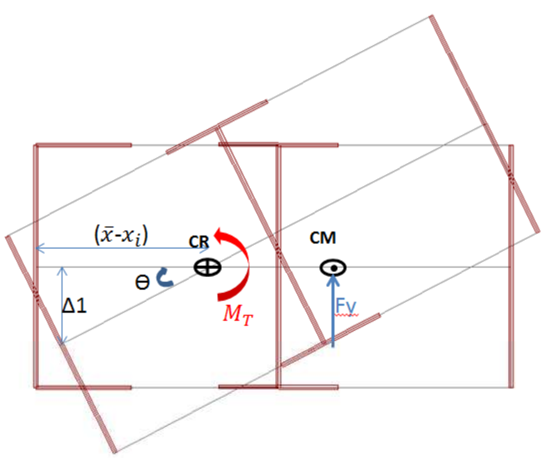
Distribution of story inertia forces between walls

- CR :
  $\bar{x} = \frac{K_{y1}X_1 + K_{y2}X_2 + K_{y3}X_3}{K_{y1} + K_{y2} + K_{y3}}$

- <u>In General</u> :
  $\bar{x} = \frac{\sum K_{yi}\, x_i}{\sum K_{yi}}$, $\bar{y} = \frac{\sum K_{xi}\, y_i}{\sum K_{xi}}$

- K_x, K_y: wall stiffness in X-Y direction .
- x_i, y_i : coordinates of center of wall "i".

Effect of Eccentricity



- $M_T = F_y\, e_x$
- $\tan\theta = \frac{\Delta 1}{\bar{x} - x_i} \approx \theta$;
  very small angle
- $\Delta i = \theta\,(\bar{x} - x_i)$
- $V_1 = k_{yi}\, \Delta 1$
- $V_1 = k_{yi}\, \theta\,(\bar{x} - x_i)$

Effect of Eccentricity

- Moment about CR for wall #1 :
  $m1 = V_1 (\bar{x} - x_i) = k_{y1}\, \Theta\, (\bar{x} - x_i)^2$
- Similar ror wall #4:
  $m4 = k_{x4}\, \Theta\, (\bar{y} - y_i)^2$

- For all walls to mention equilibriums :
  $M_T = \sum mi$
  $= \Theta \left[\sum k_{yi} (\bar{x} - x_i)^2 + \sum k_{xi} (\bar{y} - y_i)^2\right]$
  $= \Theta$ Jr ; Jr : torsional stiffness

  $\Theta = \frac{M_T}{Jr} = \frac{Fy \cdot e_x}{Jr}$

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Effect of Eccentricity

- Torsional shear on each wall :
  $V_{y1}" = k_{y1} (\bar{x} - x_1) \frac{Fy \cdot e_x}{Jr}$
  $V_{x4}" = k_{x4} (\bar{y} - y_4) \frac{Fy \cdot e_x}{Jr}$
- <u>Note that V" may be +ve or -ve</u>

- Wall shear:
  $V_{yi} = V_{yi}' + V_{yi}"$
  $V_{xi} = V_{xi}' + V_{xi}"$

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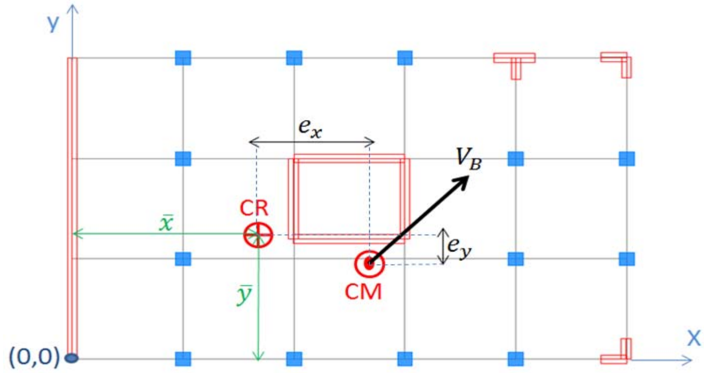
General Case



- $V_{ix} = \frac{K_{ix}}{\sum K_{ix}} V_x + \frac{(V_x e_y - V_y e_x)}{Jr} K_{ix} (y_i - \bar{y})$
- $V_{iy} = \frac{K_{iy}}{\sum K_{iy}} V_y + \frac{(V_x e_y - V_y e_x)}{Jr} K_{iy} (x_i - \bar{x})$

General Case

- Where :
  $(V_x e_y - V_y e_x)$: torsional moment of V about CR.

  Jr : torsional stiffness = $[\sum k_{yi}(\bar{x} - x_i)^2 + \sum k_{xi}(\bar{y} - y_i)^2]$

  e_x, e_y : eccentricities <u>measured from CR to CM</u>
  for example : e_x , positive & e_y , negative
  V : is the story shear = $\sum$ *inertia forces from top to story under consideration*

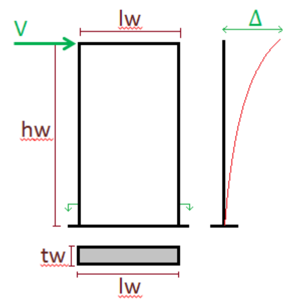
- Note : in Kx calculations use Iy
  in Ky calculations use Ix

Wall Flexural Stiffness

- $\Delta = \frac{V\,h_w^3}{3\,E\,I_{cr}} + \frac{V\,h_w}{A_e G}$
- $A_e = A_v \frac{I_{cr}}{I_g}$, G= 0.4 E
- Flexural Stiffness :

$$K = \frac{E}{h_w}\left(\frac{h_w^2}{3\,I_{cr}} + \frac{I_g}{0.4\,A_v I_{cr}}\right)^{-1}$$

$$I_{cr} = \left(\frac{100}{Fy} + \frac{P_u}{f_c' A_g}\right) I_g$$



A_g = tw.lw
$I_g = \frac{tw.lw^3}{12}$
E = $4700\sqrt{f_c'}$

- Pu : Gravity load cobination
  Axial load to act on the wall during an EQ
- Av : effective shear Area
  Av ≈ 0.9 A_g

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Example

- A five story building with story height equal to 3m as shown. The building lies in a seismic zone of S_S=1.0g , S_1=0.3g & soil type C. The structural system of the building consists of five ordinary shear walls spaced as shown in plane.
- Each floor carry a total mass weight intensity equal to 10 kN/m^2.
- It is required to find the design seismic force for each wall if the EQ direction is in the X-direction.
- Ignore accidental eccentricities & orthogonal loading & assume a light moment frames for gravity loading.

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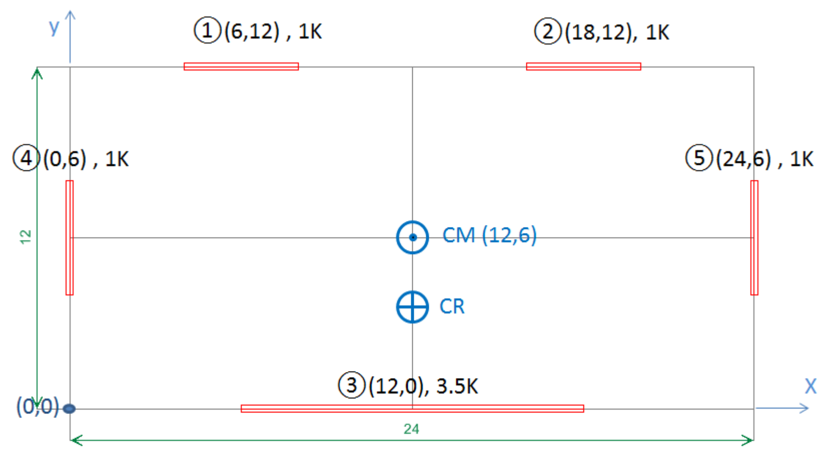
Example



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Example

- **Base shear calculation:**
  Fa= 1.0 ...... Table 1615.1.2(1) ,
  Fv= 1.5 ...... Table 1615.1.2(2) ,

  Response modification factor R=5 ; ordinary shear wall (Table 1617.6).

  Total mass weight of the building = (12x24)(10)(5) = 14,400 kN.
  Assume I_E= 1.0

  $V_B = \frac{S_{DS}}{R} I_E W = \frac{2/3}{5}(1)(14400) = 1930$ kN.
  $V_{B,min} = 0.044\ S_{DS}\ W = 0.044 \times 0.67 \times 14,400 = 425$ kN $< V_B$ → Okay

  $T_n = C_t\, h^{3/4} = 0.049 \times (5x3)^{3/4} = 0.37$ sec
  $V_{B,max} = \frac{S_{D1}}{R\,T_n} W = \frac{0.3}{5 \times 0.37}(14,400) = 2335$ kN $> V_B$ → Okay

  → $V_B = 1930$ kN.

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Example

- **Distribution of forces along height :**

  wi = (12x24)(10)= 2880 kN
  Tn = 0.37 sec → K = 1.0
  $\sum wi\, hi^{k}$ = 2880 (3+6+9+12+15) = 129600

  F1 = $\frac{2880x\,3^{1}}{129600}$ (1930) = 128.7 kN
  F2 = $\frac{2880x\,6^{1}}{129600}$ (1930) = 257.3 kN
  F3= 386 kN
  F4= 514.7 kN.
  F5= 643.3 kN

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Example

- **Horizontal Distribution :**

  Center of mass :
  CM (12,6)

  Center of Rigidity :
  $\bar{x} = 12m$ "symm."
  $\bar{y} = \frac{(3.5x0)+(1x12)+(1x12)}{3.5+1+1}$, CR (12 , 4.4)
  e_x=0 , e_y=6-4.4 = 1.6 m

  Fx = 1930 kN , Fy = 0.3x1930 = 579 kN

  $M_T = F_x e_y - F_y e_x$ = (1930x1.64) – (579x0)= 3165.2 kN.m

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Example

Wall	1	2	3	4	5
Xi	6	18	12	0	24
Yi	12	12	0	6	6
K_{xi}	1	1	3.5	0	0
K_{yi}	0	0	0	1	1
$\bar{y} - y_i$	-7.6	-7.6	4.4	-	-
$\bar{x} - x_i$	-	-	-	12	-12
$(\bar{y} - y_i)^2$	57.76	57.76	19.36	-	-
$(\bar{x} - x_i)^2$	-	-	-	144	144

- Jr = $\left[\sum k_{yi}(\bar{x}-x_i)^2 + \sum k_{xi}(\bar{y}-y_i)^2\right]$
  = $[1(4.4-12)^2]$+ $[1(4.4-12)^2]$+ $[3.5(4.4-0)^2]$+ $[1(12-0)^2]$+ $[1(12-24)^2]$
  = 471.3 kN.m

Example

- **Wall # 1 :**
  $V_1 = \frac{1}{5.5}(1930) + \frac{3165.2}{471.3}(1)(12-4.4) = 351 + 51.3 = 402.3\ kN$
- **Wall # 3 :**
  $V_3 = \frac{3.5}{5.5}(1930) + \frac{3165.2}{471.3}(3.5)|(0-4.4)| = 1228 - 102.4 = 1125.8\ kN$
- **Wall #5 :**
  $V_5 = \frac{1}{2}(579) + \frac{3165.2}{471.3}(1)(24-12) = 289.5 - 80.6 = 209\ kN$
- **Wall #2 :**
  V_2= 402.3 kN.

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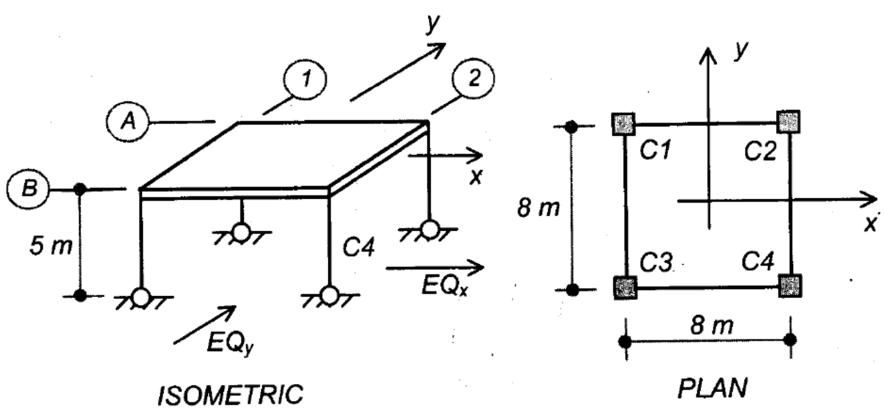
Lecture 8 – Load Combination Example

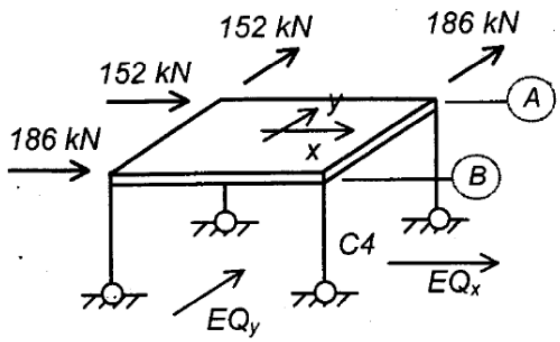
Dr. Hazim Dwairi

Problem Statement

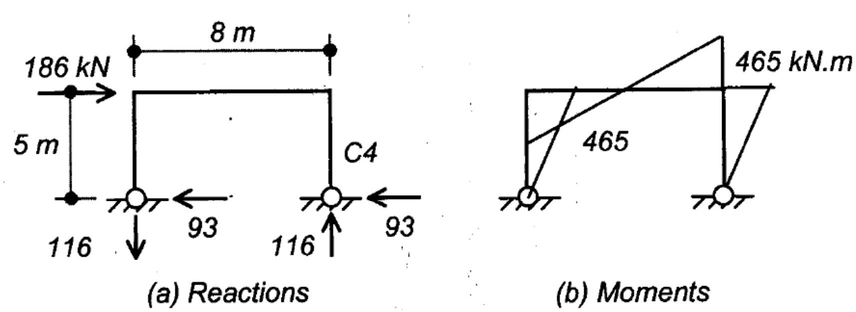


$q_D = 20\ kN/m^2$, including own weight of columns
$q_L = 10\ kN/m^2$,
$q_S = 2\ kN/m^2$

x- and y- EQ Forces



Ex7.Fig 2 Frame Forces

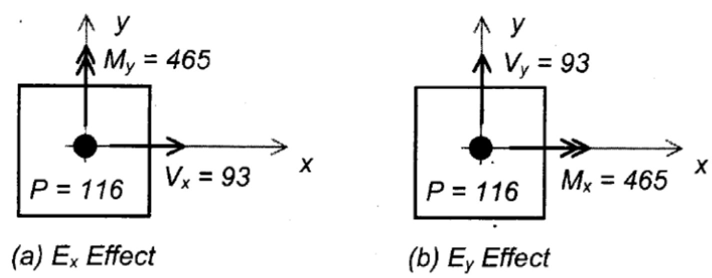


Ex7.Fig 3 Frame B Forces and Moments

Direction X: P_{Ex} = 116 kN, V_{Ex} = 93 kN, M_{Ex} = 465 kN.m

Direction Y: P_{Ey} = 116 kN, V_{Ey} = 93 kN, M_{Ey} = 465 kN.m

Directional Effect:



Ex7.Fig 4 Internal Actions due to Separate Excitation

Case (1):

$$P = P_{Ex} + 0.3\,P_{Ey} = 116 + 0.3\,(116) = 151\text{ kN}$$
$$V_x = V_{Ex} + 0.3\,V_{Ey} = 93 + 0.3\,(0) = 93\text{ kN}$$
$$V_y = V_{Ex} + 0.3\,V_{Ey} = 0 + 0.3\,(93) = 28\text{ kN}$$
$$M_x = M_{Ex} + 0.3\,M_{Ey} = 0 + 0.3\,(465) = 140\text{ kN}$$
$$M_y = M_{Ex} + 0.3\,M_{Ey} = 465 + 0.3\,(0) = 465\text{ kN}$$

Case (2):

$$P = 0.3\,P_{Ex} + P_{Ey} = 0.3\,(116) + 116 = 151\text{ kN}$$
$$V_x = 0.3\,V_{Ex} + V_{Ey} = 0.3\,(93) + 0 = 28\text{ kN}$$
$$V_y = 0.3\,V_{Ex} + V_{Ey} = 0.3\,(0) + 93 = 93\text{ kN}$$
$$M_x = 0.3\,M_{Ex} + M_{Ey} = 0.3\,(0) + 465 = 465\text{ kN}$$
$$M_y = 0.3\,M_{Ex} + M_{Ey} = 0.3\,(465) + 0 = 140\text{ kN}$$

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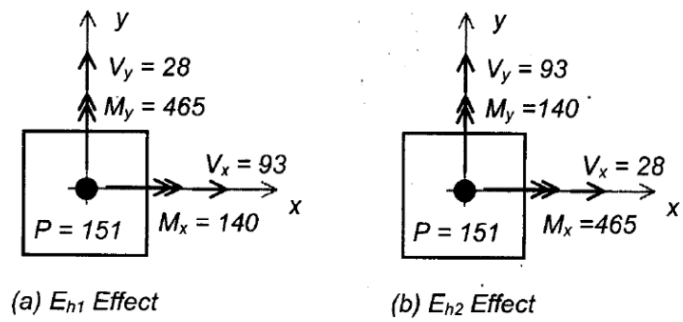
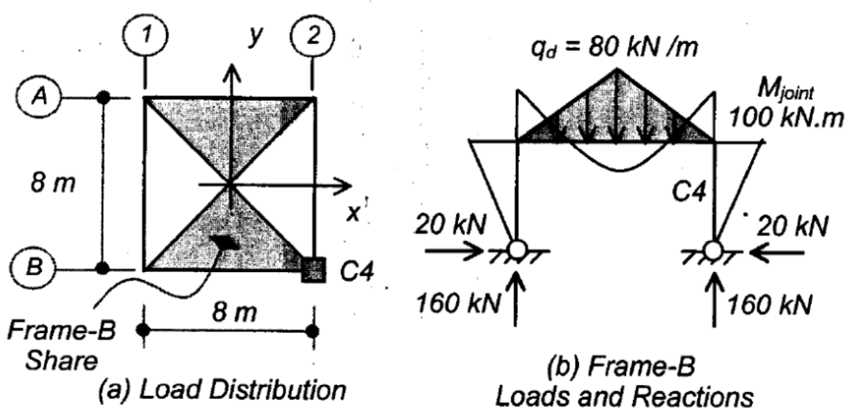


Ex7.Fig 5 Directinal Combination of Excitation

(7) Dead load: $(q_d)_{frame} = (q_D)_{slab}\,(4m) = 20\,(4) = 80\ kN/m^2$



Ex7.Fig 6 Dead Load Distribution to Frames

$P = 160 + 160 = 320$ kN
$V_x = 20$ kN
$V_y = 20$ kN

$M_x = 100$ kN
$M_y = 100$ kN

The effect of live load and snow loads will be proportional to the effect of the dead load since the load distribution will take the same path with different intensity, consequently,

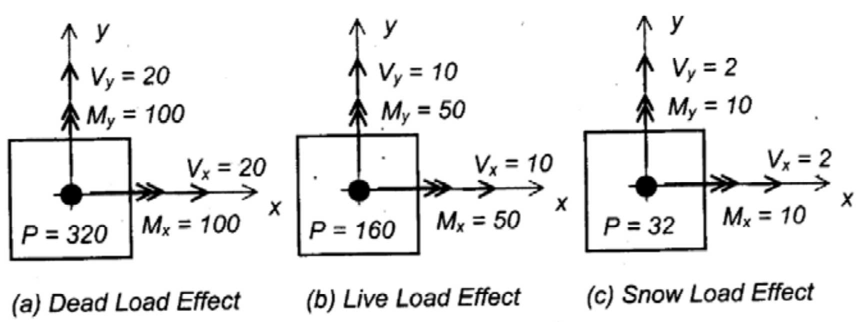
Due to live load: multiply by a factor of (10 / 20 = 0.5), hence

frame-B	$P = 80$ kN,	$V_x = 10$ kN,	$M_y = 50$ kN.m
frame-2	$P = 80$ kN,	$V_y = 10$ kN,	$M_x = 50$ kN.m

Due to snow load: multiply by a factor of (2 / 20 = 0.1), hence

frame-B	$P = 16$ kN,	$V_x = 2$ kN,	$M_y = 10$ kN.m
frame-2	$P = 16$ kN,	$V_y = 2$ kN,	$M_x = 10$ kN.m

The final gravity load effects are shown graphically in Ex7.Fig. 7.



Ex7.Fig 7 Gravity Load Effects in Column-C4

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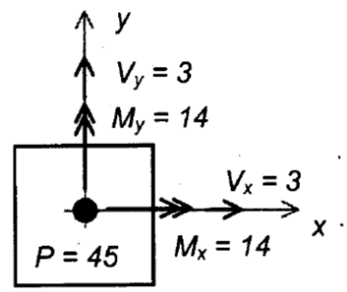
(**8**) Vertical earthquake excitation:

$$E_v = 0.5\, C_a\, I\, D = 0.5\, (0.28)\, (1)\, D = 0.14\, D$$

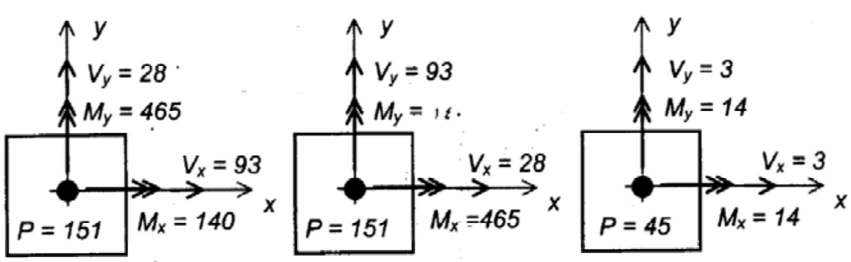
It must be noted that the vertical component of earthquake induces shears and moments in the structure, and not only vertical forces as usually is the general perception. Results are shown in Ex7.Fig. 8



Ex7.Fig. 8 E_v Effect

(**9**) Horizontal and vertical combination of earthquake effect:

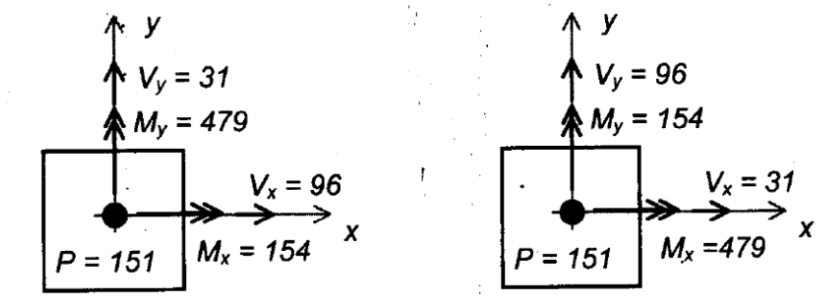
$$E = \rho E_h + E_v \qquad \text{since } \rho < 1, \qquad \rho = 1$$



E_{h1} Effect E_{h2} Effect E_v Effect

(a) Separate Effect of Earthquake Components

$E_1 = 1.0\, E_{h1} + E_v$
$E_2 = 1.0\, E_{h2} + E_v$



E_1 Combination E_2 Combination

(b) Combined Effect of Earthquake Components

(**10**) Load Combinations:

$U_1 = 1.2\, D + 1.0\, L + 0.2\, S + E_1$
$U_2 = 1.2\, D + 1.0\, L + 0.2\, S + E_2$

Accordingly, case U_1:

$P = 1.2\,(320) + 1.0\,(160) + 0.2\,(32) + 151 = 701$ kN
$V_x = 1.2\,(20) + 1.0\,(10) + 0.2\,(2) + 96 = 130$ kN
$V_y = 1.2\,(20) + 1.0\,(10) + 0.2\,(2) + 31 = 65$ kN
$M_x = 1.2\,(100) + 1.0\,(50) + 0.2\,(10) + 154 = 326$ kN.m
$M_y = 1.2\,(100) + 1.0\,(50) + 0.2\,(10) + 479 = 651$ kN.m

and, case U_2:

$P = 1.2\,(320) + 1.0\,(160) + 0.2\,(32) + 151 = 701$ kN
$V_x = 1.2\,(20) + 1.0\,(10) + 0.2\,(2) + 31 = 65$ kN
$V_y = 1.2\,(20) + 1.0\,(10) + 0.2\,(2) + 96 = 130$ kN
$M_x = 1.2\,(100) + 1.0\,(50) + 0.2\,(10) + 479 = 651$ kN.m
$M_y = 1.2\,(100) + 1.0\,(50) + 0.2\,(10) + 154 = 326$ kN.m