CE0043
Earthquake Engineering
Introduction to Basic Seismology and Earthquake Engineering

Module 1

OBJECTIVES

- At the end of the chapter, the learner should be able to:
  - *Familiarization on the basics of earthquake engineering.*
  - *Explain the importance of earthquake engineering in structural design.*
  - *Enumerate procedures in seismic analysis.*

Introduction to Basic Seismology and Earthquake Engineering

Basic Earthquake Engineering Definitions

Basic Terminologies in Earthquake Engineering

Base

- It is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure, as a dynamic vibrator, is supported.

Base Shear

- It is the total design lateral force or shear at the base of a structure.

Basic Terminologies in Earthquake Engineering

Bearing Wall System

- It is a structural system that does not have a compete vertical load-carrying space frame.

Boundary Element

- It is an element at edges of openings or at perimeters of shear walls or diaphragms.

Basic Terminologies in Earthquake Engineering

Braced Frame

- It is essentially a vertical truss system of the concentric or eccentric type that is provided to resist lateral forces.

Building Frame System

- It is essentially a complete space frame that provides support for gravity loads.

Basic Terminologies in Earthquake Engineering

Building (Enclosed)

- It is a building that does not comply with the requirements for open or partially enclosed buildings.

Building Envelope

- It refers to cladding, roofing, exterior wall, glazing, door assemblies, window assemblies, skylight assemblies and other components enclosing the building.

Basic Terminologies in Earthquake Engineering

Building (Flexible)

- It refers to slender buildings that have a fundamental natural frequency less than 1.0 Hz.

Building (Low Rise)

- It is an enclosed or partially enclosed building that complies with the following conditions: (a) mean roof height "h" less than or equal to 18m and (b) mean roof height "h" does not exceed least horizontal dimension.

Basic Terminologies in Earthquake Engineering

Building (Open)

- It refers to a building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$

Building (Regular Shaped)

- It refers to a building or other structure having no unusual geometrical irregularity in spatial form.

Basic Terminologies in Earthquake Engineering

Building (Partially Enclosed)

- It is a building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10%; and

2. The total area of openings in a wall that receives positive external pressure exceeds $0.5m^2$ or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

Basic Terminologies in Earthquake Engineering

Building (Partially Enclosed)

- It is a building that complies with both of the following conditions:

These conditions are expressed by the following equations:

1. $A_o > 1.10A_{oi}$

2. $A_o > Smaller\ of\ (0.5m^2\ or\ 0.01A_g)$

3. $A_{oi}/A_{gi} \leq 0.20$

Basic Terminologies in Earthquake Engineering

Building (Rigid)

- It refers to a building or other structure whose fundamental frequency is greater than or equal to 1.0 Hz.

Cantilevered Column Element

- It is a column element in a lateral-force-resisting system that cantilevers from a fixed base and has minimal moment capacity at the top, with lateral forces applied essentially at the top.

Basic Terminologies in Earthquake Engineering

Collector

- It is a member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral-force-resisting system.

Component

- It is a part or element of an architectural, electrical, mechanical or structural system.

Basic Terminologies in Earthquake Engineering

Component (Equipment)

- It is a mechanical or electrical component or element that is part of a mechanical and/or electrical system.

Component (Flexible)

- It is component, including its attachments, having fundamental period greater than 0.06s.

Basic Terminologies in Earthquake Engineering

Concentrically-Braced Frame

- It is a braced frame in which the members are subjected primarily to axial forces.

Cripple Wall

- It is a framed stud wall extending from the top of the foundation to the underside of floor framing for the lowest occupied level.

Basic Terminologies in Earthquake Engineering

Dead Loads

- It consists of the weight of all materials and fixed equipment incorporated into the building or other structure.

Deck

- It is an exterior floor system supported on at least two opposing sides by an adjacent structure and/or posts, piers, or other independent supports.

Basic Terminologies in Earthquake Engineering

Design Basis Ground Motion

- It is that ground motion that has a 10 percent chance of being exceeded in 50 years as determined by a site-specific hazard analysis or may be determined from a hazard map.

Design Response Spectrum

- It is an elastic response spectrum for 5 percent equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures.

Basic Terminologies in Earthquake Engineering

Design Seismic Force

- It is the minimum total strength design base shear, factored and distributed.

Diaphragm

- It is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. It includes horizontal bracing systems.

Basic Terminologies in Earthquake Engineering

Diaphragm (Blocked)

- It is a diaphragm in which all sheathing edges not occurring on framing members are supported on and connected to blocking.

Diaphragm Chord/Shear Wall Chord

- It is the boundary element of a diaphragm or shear wall that is assumed to take axial stresses analogous to the flanges of a beam.

Basic Terminologies in Earthquake Engineering

Diaphragm Strut

- It is the element of a diaphragm parallel to the applied load that collects and transfers diaphragm shear to the vertical resisting elements or distributed loads within the diaphragm. Such members may take axial tension or compression.

Diaphragm (Unblocked)

- It is the diaphragm that has edge nailing at supporting members only.

Basic Terminologies in Earthquake Engineering

Drift (Storey Drift)

- It is the lateral displacement of one level relative to the level above or below.

Dual System

- It is a combination of moment-resisting frames and shear walls or braced frames

Basic Terminologies in Earthquake Engineering

Elastic Response Parameters

- These are forces and deformations determined from an elastic dynamic analysis using an unreduced ground motion representation.

Essential Facilities

- These are buildings, towers and other vertical structures that are intended to remain operational in the event of extreme environmental loading from earthquakes.

Basic Terminologies in Earthquake Engineering

Flexible Element

- It is one whose deformation under lateral load is significantly larger than adjoining parts of the system.

Horizontal Bracing System

- It is a horizontal truss system that serves the same function as a diaphragm.

Basic Terminologies in Earthquake Engineering

Importance Factor

- It is a factor that accounts for the degree of hazard to human life and damage to property.

Lateral-Force-Resisting System

- It is that part of the structural system designed to resist the Design Seismic Forces.

Basic Terminologies in Earthquake Engineering

Limit State

- It is a condition beyond which a structure or member becomes unfit for service and is judged to be no longer useful for its intended functions (serviceability limit state) or to be unsafe (strength limit state)

Live Loads

- These are those loads produced by the use and occupancy of the building or other structure and do not include dead load, construction load, or environmental loads.

Basic Terminologies in Earthquake Engineering

Loads

- These are forces or other actions that results from the weight of all building materials, occupants and their possessions, environmental effects, differential movements, and restrained dimensional changes.

Moment-Resisting Frame

- It is a frame in which members and joints are capable of resisting forces primarily by flexure.

Basic Terminologies in Earthquake Engineering

Moment-Resisting Wall Frame (MRWF)

- It is a masonry wall frame especially detailed to provide ductile behavior.

Ordinary Moment-Resisting Frame (OMRF)

- It is a moment-resisting frame not meeting special detailing requirements for ductile behavior.

Basic Terminologies in Earthquake Engineering

Orthogonal Effects

- These are the earthquake load effect on structural elements simultaneously occurring to the lateral-force-resisting systems along two orthogonal axes.

PΔ Effect

- It is the secondary effect on shears, axial forces and moments of frame members induced by the horizontal displacement of vertical loads from various loading, when a structure is subjected to lateral forces.

Basic Terminologies in Earthquake Engineering

Rotation

- It is the torsional movement of a diaphragm about a vertical axis.

Shear Wall

- It is a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as vertical diaphragm or structural wall).

Basic Terminologies in Earthquake Engineering

Shear Wall-Frame Interactive System

- It uses combinations of shear walls and frames designed to resist lateral forces in proportion to their relative rigidities, considering interaction between shear walls and frames on all levels.

Subdiaphragm

- It is a portion of a diaphragm used to transfer wall anchorage forces to diaphragm cross ties.

Basic Terminologies in Earthquake Engineering

Soft Storey

- It is one in which the lateral stiffness is less than 70 percent of the stiffness of the storey above.

Storey

- It is the space between levels.

Basic Terminologies in Earthquake Engineering

Storey Drift Ratio

- It is the storey drift divided by the storey height.

Storey Shear

- It is the summation of design lateral forces above the storey under consideration.

Basic Terminologies in Earthquake Engineering

Strength

- It is an assemblage of framing members designed to support gravity loads and resist lateral forces.

Weak Storey

- It is one in which the storey strength is less than 80 percent of the storey above.

INTRODUCTION TO EARTHQUAKE ENGINEERING

Effects of Earthquakes

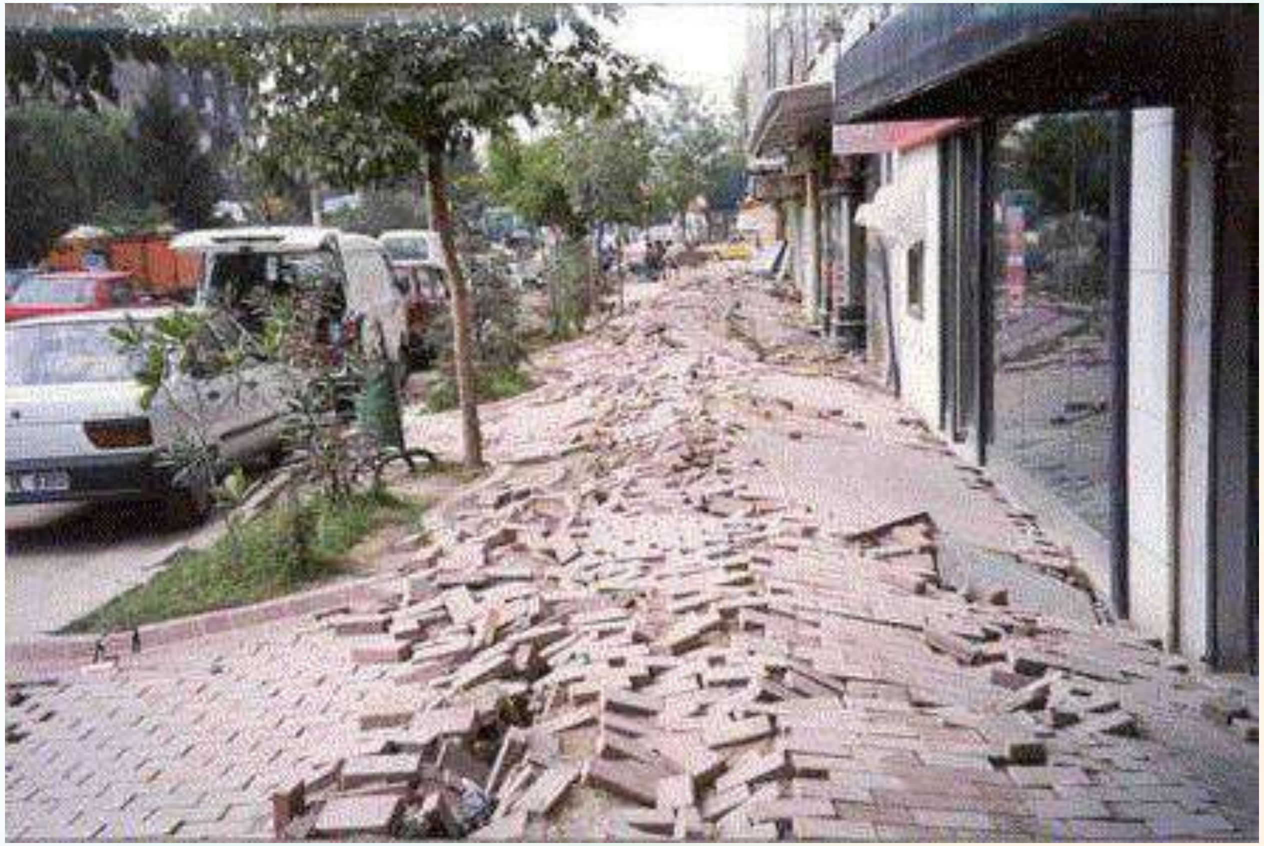
Earthquake Effects

- Earthquake Effects
  - Ground Failure
    - Surface Rupture
    - Ground Subsidence
    - Ground Cracking
    - Soil Liquefaction
    - Landslides
  - Indirect Effects
    - Tsunami
    - Seiches
    - Fires
  - Ground Shaking

Structural Effects of Earthquakes

Ground Failures

- These are generally considered part of geotechnical earthquake engineering, and they involve the movement of the ground surface at a location where geological fissures or zones of weakness in the crust of the earth (faults) slip slowly or suddenly.



(Bird and Bommer, 2004)

Structural Effects of Earthquakes

Surface Faulting

- Occurs when the relative movement of rocks on the two sides of a fault takes place deep within the earth and breaks through to the surface.
- Occur as slow movement in the form of fault creep or suddenly resulting in an earthquake.
- This type of ground failure typically follows a pre-existing fault line.

(Galli et al., 2017)

Structural Effects of Earthquakes

Ground Subsidence

- Occurs as loose soils rearrange and settle into a denser state during vibrations cause by earthquakes

(Glass, 2013)

Structural Effects of Earthquakes

Ground Cracking

- It is usually observed along the edges of ground subsidence.
- It is also be the result of slope failure or liquefaction, all of which cause the ground to lose its support and sink, with the ground surface breaking up into fissures, scarps, horsts and grabens.
- The most damaging effect of ground subsidence is differential settlement, which can severely disrupt the function of any infrastructure system near the vicinity of cracking locations, particularly those with long foundations that straddle the cracks.

(Hassan et al., 2016)

Structural Effects of Earthquakes

Soil Liquefaction

- It occurs when loose, saturated granular soils temporarily change from a solid to a liquid state, losing their shear strength, which corresponds to a loss in effective stress between soil particles.

- Loose saturated (or moderately saturated) sands and non-plastic silts are most susceptible to this ground failure; however, in rare cases, gravel and clay can also experience liquefaction.

- In all cases, poor drainage within the loose soil causes an increase in the pore water pressure as the soil is compressed by the vibratory effect of seismic waves.

https://www.britannica.com/science/soil-liquefaction

Structural Effects of Earthquakes

Landslides

- Landslides caused by earthquakes are uncommon. Consequently, in order for a structure to experience damage during the event, it must be located at the top or bottom of the soil mass that slides down; for this reason, damage resulting from earthquake-induced landslides is rare.

- Sloped land that is marginally stable under static conditions is most susceptible to sliding during the intense shaking of strong earthquakes.

- For the most severe cases, debris (soil, boulders, and other materials) flow can move at avalanche speeds and can travel long distances depending on the slope from which the landslide was formed.

https://research.engineering.ucdavis.edu/gpa/landslides/earthquake-induced-landslides/

Structural Effects of Earthquakes

Landslides

- Furthermore, earthquake-induced landslides can be sudden and unpredictable, producing the total destruction of communities in the path of the debris flow.

https://research.engineering.ucdavis.edu/gpa/landslides/earthquake-induced-landslides/

Indirect Effects of Earthquakes

Tsunamis

- These are long-period sea waves that are generated when an earthquake causes the vertical movement of the seafloor.

- Tsunamis travel far, at high speeds (over 500 mph) in the open ocean and are difficult to detect because of their small crest-to-trough height, and long wavelengths, which typically, are hundreds of miles long.

- Unobstructed, these waves can travel around the world and dissipate all their energy without causing damage.

https://www.nbcnews.com/mach/science/what-tsunami-ncna943571

Indirect Effects of Earthquakes

Tsunamis

- However, as they approach a shore, the water depth decreases causing an increase in wave speed and wave amplitude (height of wave run-ups).

- Wave run-ups of 75 feet have been observed at several locations.

- Wave run-ups can push water that rushes far inland, and have created devastating damage to infrastructure and great loss of life.

https://www.nbcnews.com/mach/science/what-tsunami-ncna943571

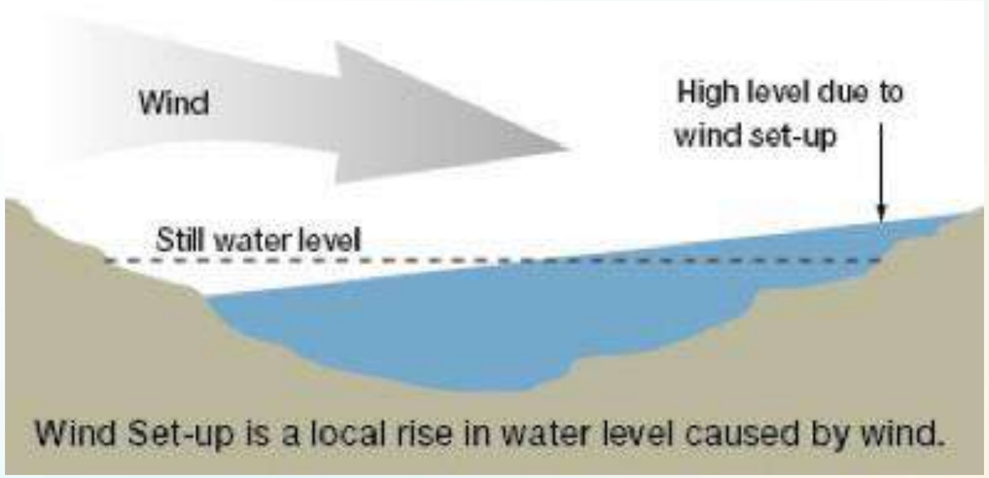
Indirect Effects of Earthquakes

Seiches

- These are earthquake-induced waves in an enclosed body of water, such as a lake or a reservoir, or one that is partially enclosed, such as a bay.

- These are are caused when long-period seismic waves resonate with oscillations of the enclosed water and cause standing waves.

- Earthquakes may happen within or far outside the perimeter of the body of water.



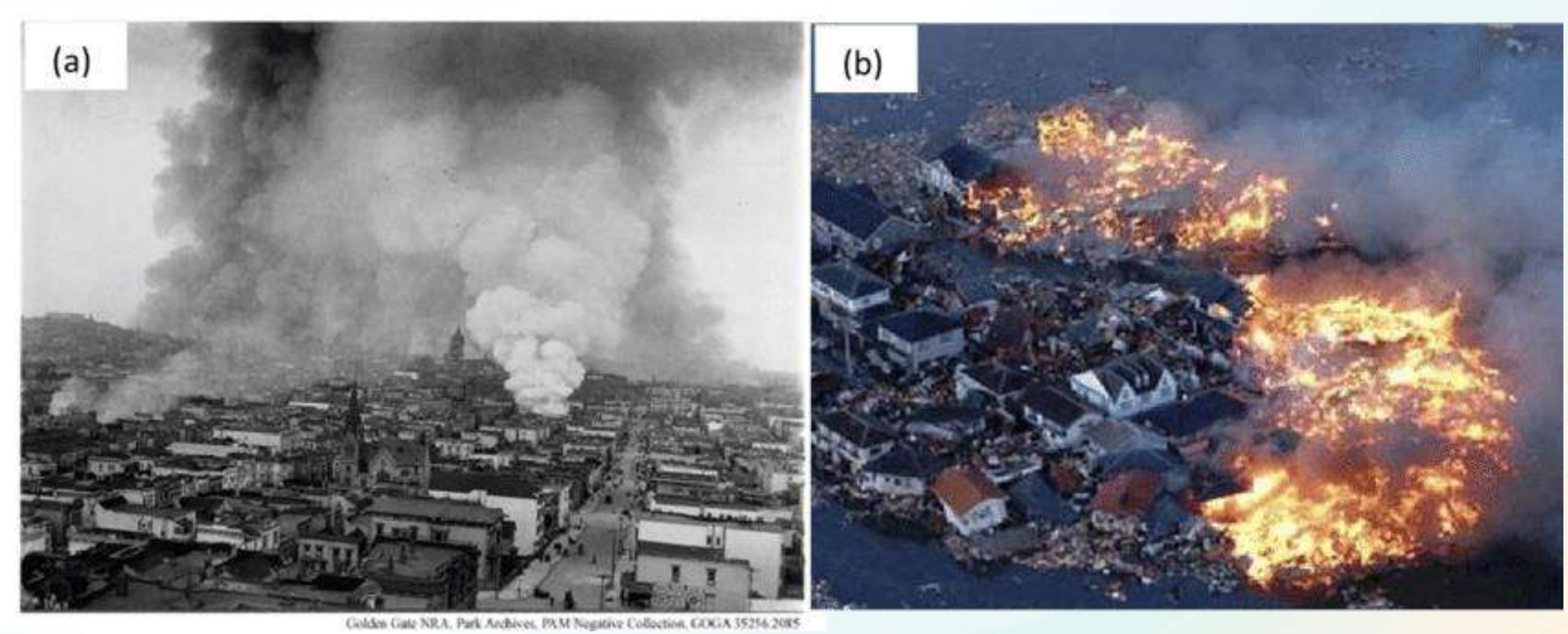
http://www.geo.mtu.edu/KeweenawGeoheritage/Lake/Seiches.html

- Although this type of wave has been observed during most earthquakes (even in swimming pools), related damage to infrastructure has been minimal.

Indirect Effects of Earthquakes

Fire

- It is probably the most terrifying indirect effect of earthquakes, particularly considering that people who survived in collapsed buildings, but were trapped in the debris, were burnt alive



Memari et al., 2013

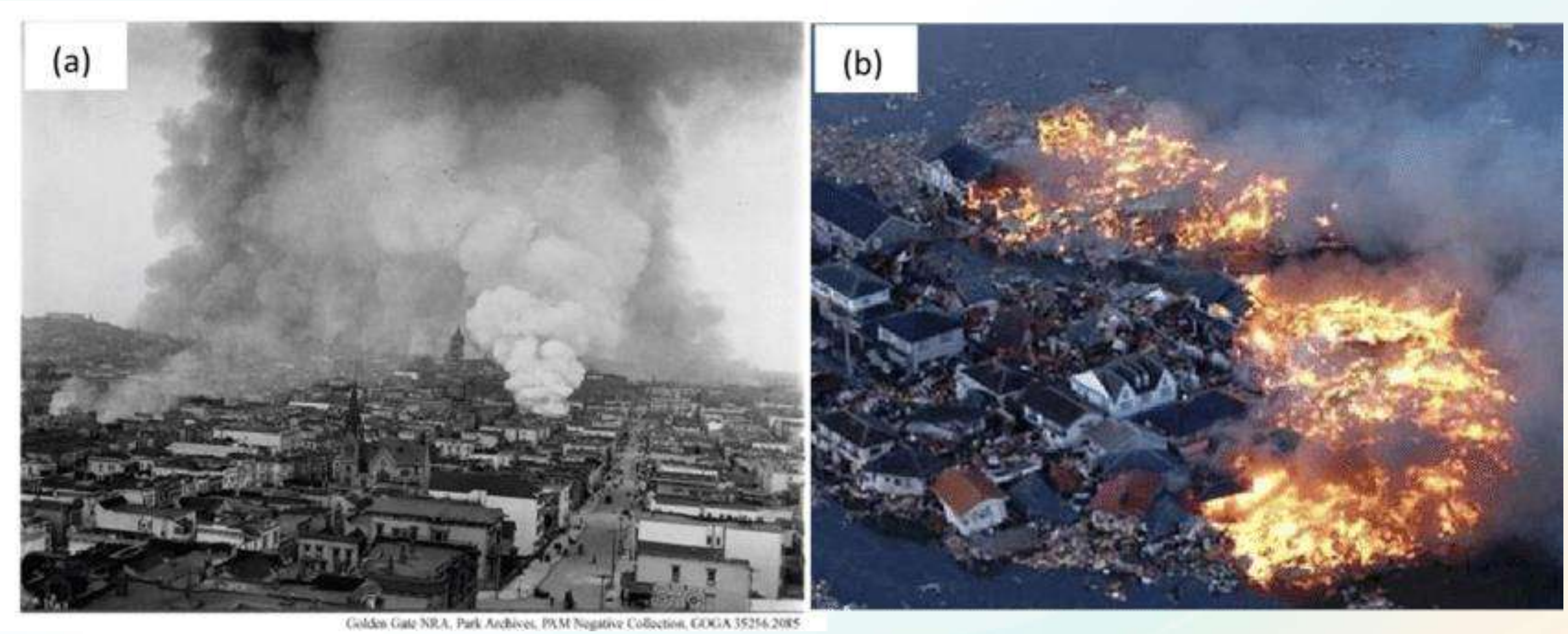
FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Indirect Effects of Earthquakes

Fire

- Traditional firefighting methods are often ineffective against earthquake-induced fires because most water mains that supply water hydrants break.

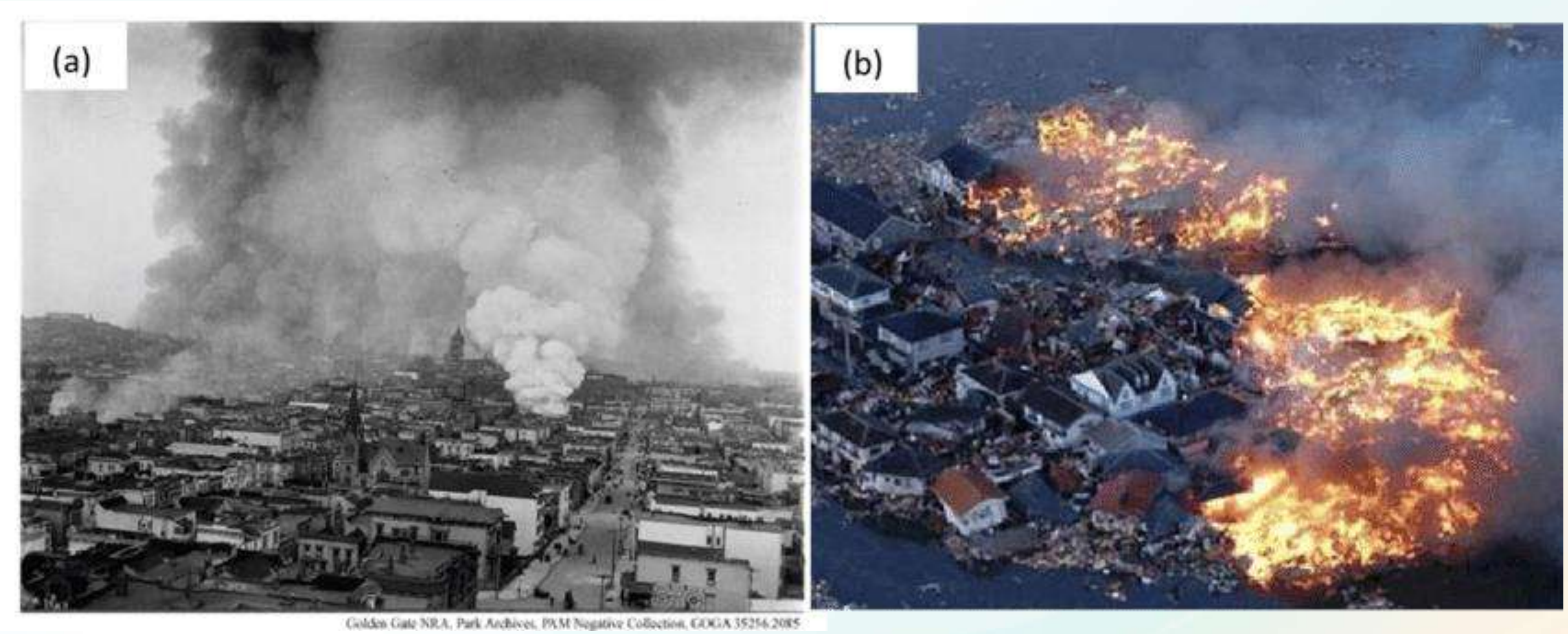


Memari et al., 2013

Indirect Effects of Earthquakes

Fire

- Earthquake-induced fires are started by ruptured combustible substance conduits (such as gas mains) or destroyed combustible substance storage containers (such as oil tanks), and then ignited by sparks from sources such as downed powerlines.



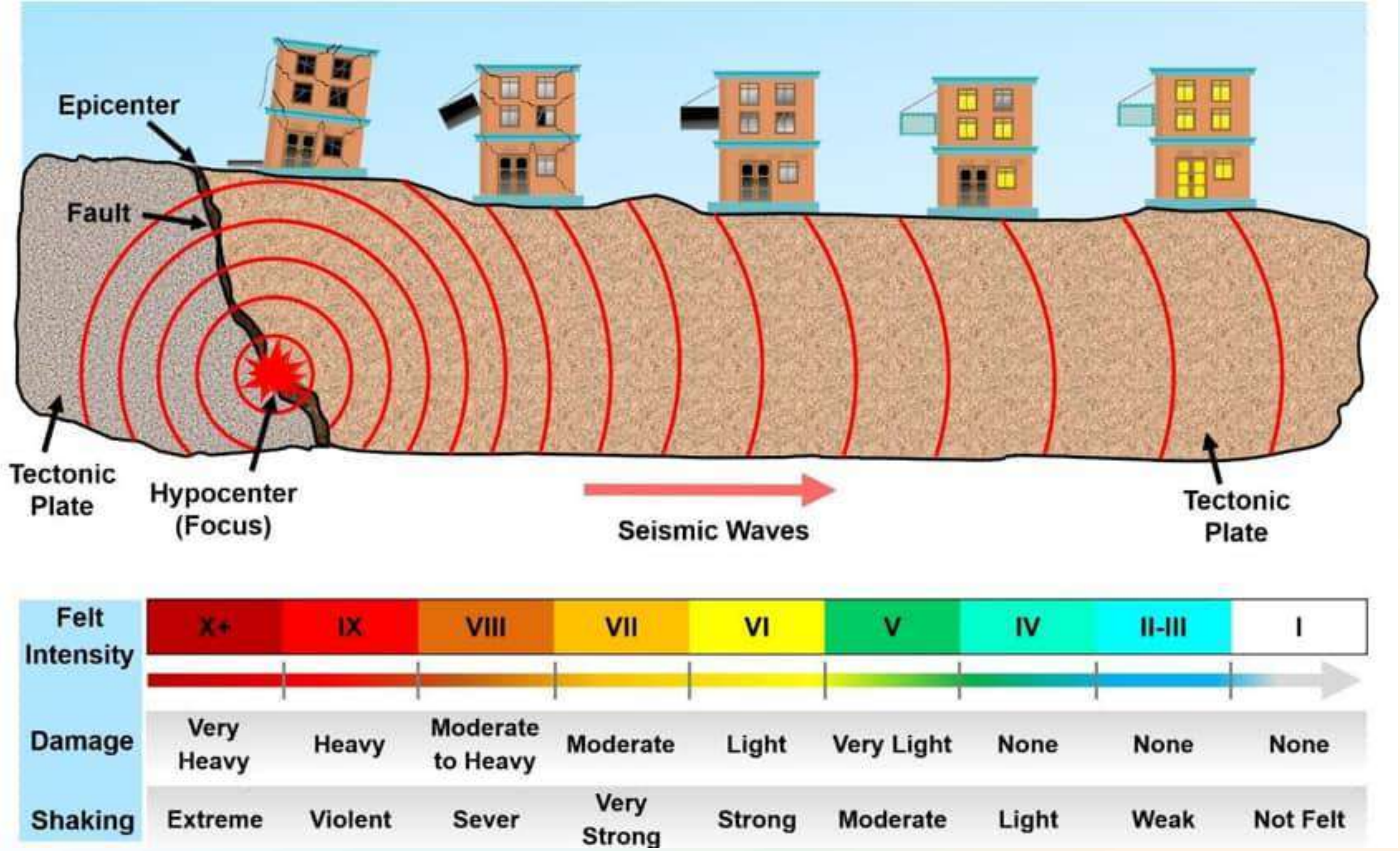
Memari et al., 2013

Ground Shaking

Ground Shaking

- It causes the majority of earthquake damage; additionally most of the aforementioned effects are caused by shaking. In fact, where the shaking intensity is low, the hazard of other effects can be minimal.

- Consequently, shaking is the only effect experienced by everyone within an afflicted area, and intense shaking can produce widespread damage from various seismic hazards.



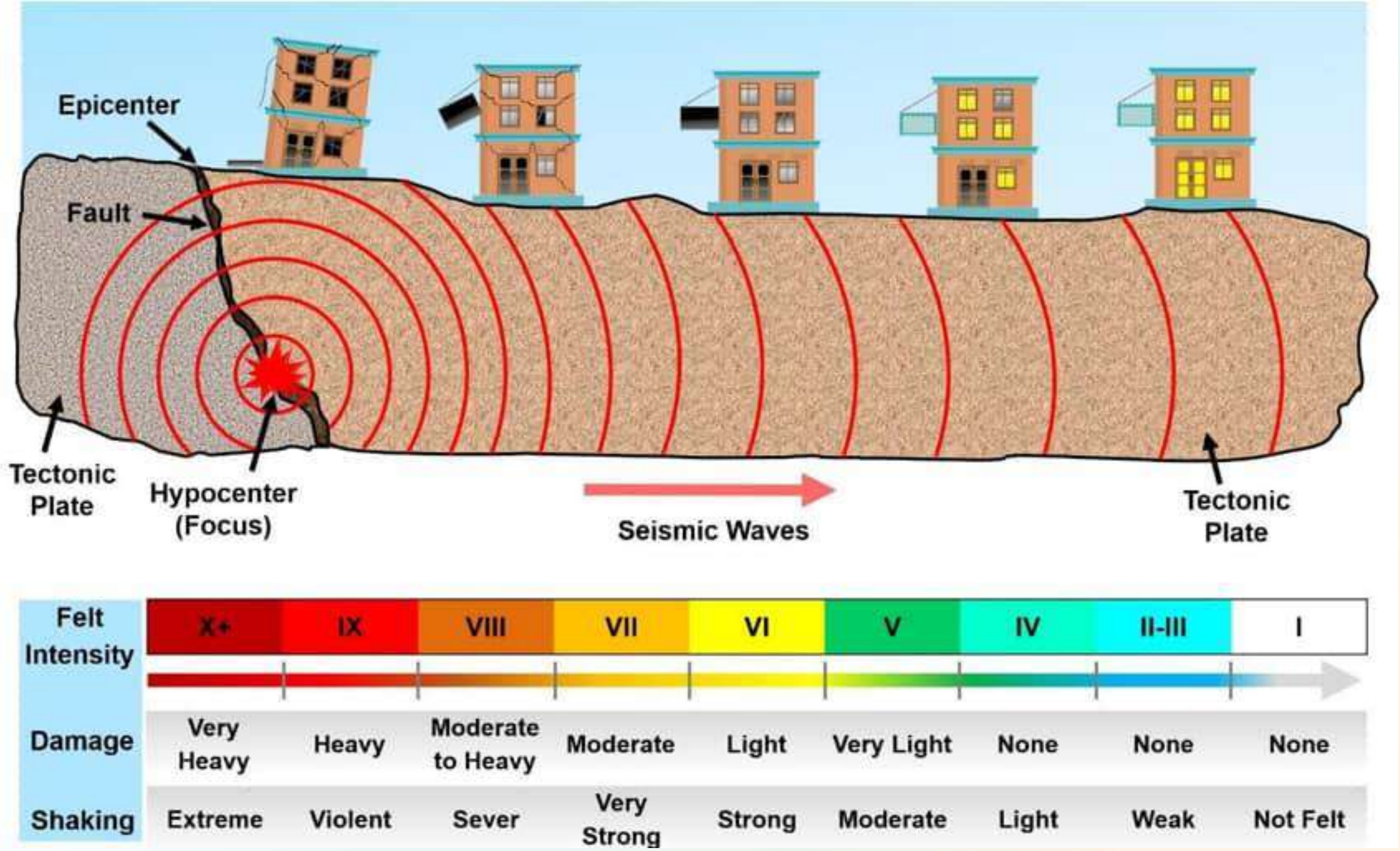
https://www.facebook.com/technologyreview.pk/photos/a.222627969 0731551/2762512310441617/?type=1&theater

Ground Shaking

Ground Shaking

- For this reason, ground shaking is the main focus of earthquake engineering.

- Although earthquakes are caused by numerous natural and human-induced phenomena, the events posing the highest seismic risks are caused by the relative deformation of crustal tectonic plates



https://www.facebook.com/technologyreview.pk/photos/a.222627969 0731551/2762512310441617/?type=1&theater

Ground Shaking

Seismic waves that radiate from the location where a fault ruptures (the *focus*) quickly travel throughout the earth's crust, producing ground shaking when they reach the ground surface. The intensity and duration of shaking experienced at a particular site during an earthquake are primarily because of three factors:

- *Earthquake size* (magnitude): It can be measured objectively or subjectively—larger earthquakes cause stronger shaking. A strong earthquake can cause ground shaking over widespread areas, suddenly affecting large numbers of structures. Even relatively small earthquakes can have a significant impact on large numbers of buildings.

- *Location* (distance from the focus or epicenter): Generally, the closer to the epicenter, the stronger the shaking. Structures near the epicenter of a strong earthquake can experience extensive damage, in some cases partial or total collapse;

Ground Shaking

Seismic waves that radiate from the location where a fault ruptures (the *focus*) quickly travel throughout the earth's crust, producing ground shaking when they reach the ground surface. The intensity and duration of shaking experienced at a particular site during an earthquake are primarily because of three factors:

- *The subsurface materials beneath the structure*: Soft soils amplify the shaking, while rocks do not. This is the most insidious of the three factors because the site can be located at a long distance from the *epicenter* and still experience extensive ground shaking due to local soil conditions. Seismic waves travel through rock for most of their trip from the focus to the surface; however, at many sites, the final part of the trip is through soil, the geological characteristics of which have a major influence on the nature of ground shaking. Some soils act as seismic wave filters, attenuating shaking at some frequencies while amplifying it at others.

Types of Earthquakes

Man – Made Earthquakes

- These generally have a much smaller magnitude than the other two types of earthquakes, and thus have a lesser impact on infrastructure. However, man-made earthquakes can lead to earlier fault ruptures (tectonic earthquakes) because the shaking can increase critical stresses at the plate boundaries.

- One of the most intense cases is due to explosions, both from conventional and nuclear weapons.

- For example, it is estimated that the Boxcar nuclear bomb explosion in 1968, with a yield of 1200,000 tons TNT equivalent, excited an earthquake of magnitude 5.0 that lasted for 10–12 s. This shook buildings in nearby communities, including Las Vegas, NV (30 miles away), but no serious damage or casualties occurred.

Types of Earthquakes

Volcanic Earthquakes

- These are caused by the same energy source as tectonic earthquakes, which is the heat from the earth's core. Volcanic seismicity affects limited areas near volcanic regions.

- The movement of magma through tubes below the volcanic vents creates pressure changes in the surrounding rock that can rupture, releasing elastic strain energy as seismic waves.

- These seismic waves have been successfully used to predict eruptions of volcanos such as Mount St. Helen in 1980 and Pinatubo in 1991.

- Other seismic waves can be induced by sudden, irregular movement of magma whose path has been obstructed, or by steady magma movement deep in the mantle.

- Damage from all these earthquakes is relatively minor compared with that produced by tectonic earthquakes.

Types of Earthquakes

Tectonic Earthquakes

- These are caused by a sudden dislocation of segments of the earth's crust, the structure of which is composed of plates (large and small) known as *tectonic plates* that float on a liquid layer, the mantle.

- This arrangement resulted from the formation of planet Earth five billion years ago, when hot gasses cooled into a semi-solid mass.

- It is estimated that after one to two billion years of cooling, the crust solidified and cracked forming tectonic plates (different ones than those that exist today).

- Damage from all these earthquakes is relatively minor compared with that produced by tectonic earthquakes.

Types of Earthquakes

Continental Drift Theory

- From the beginning, the plates have been in constant motion forming and breaking up continents over time, including the formation of supercontinents that contained most of the landmass. The latest supercontinent, Pangea, started separating approximately 200 million years ago, and its parts have drifted apart to the current configuration of the earth's surface. This process was originally proposed by Alfred Wegener in the early 1900s. He noted several different pieces of evidence to support his theory of the continental drift, including

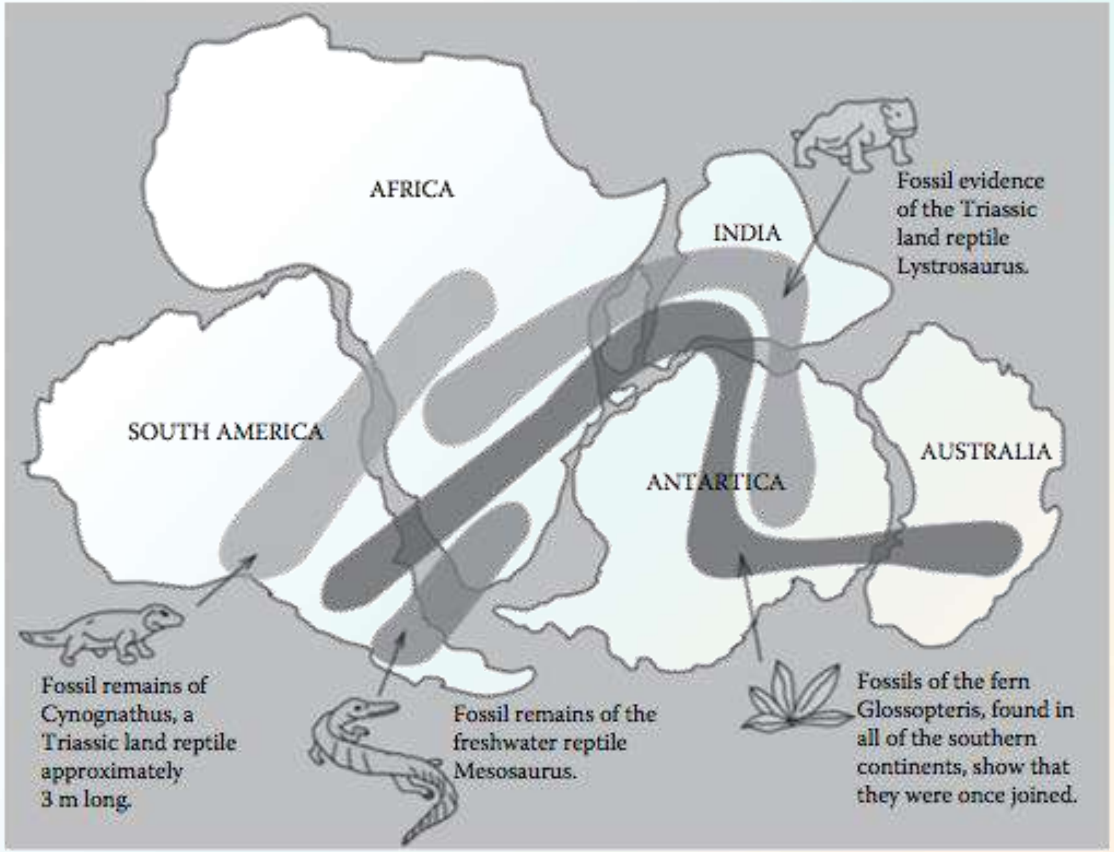


FIGURE 1.10 Continental drift evidence; distribution of fossils across the southern continents of Pangea. (Courtesy of USGS.)

Estrada and Lee, 2017

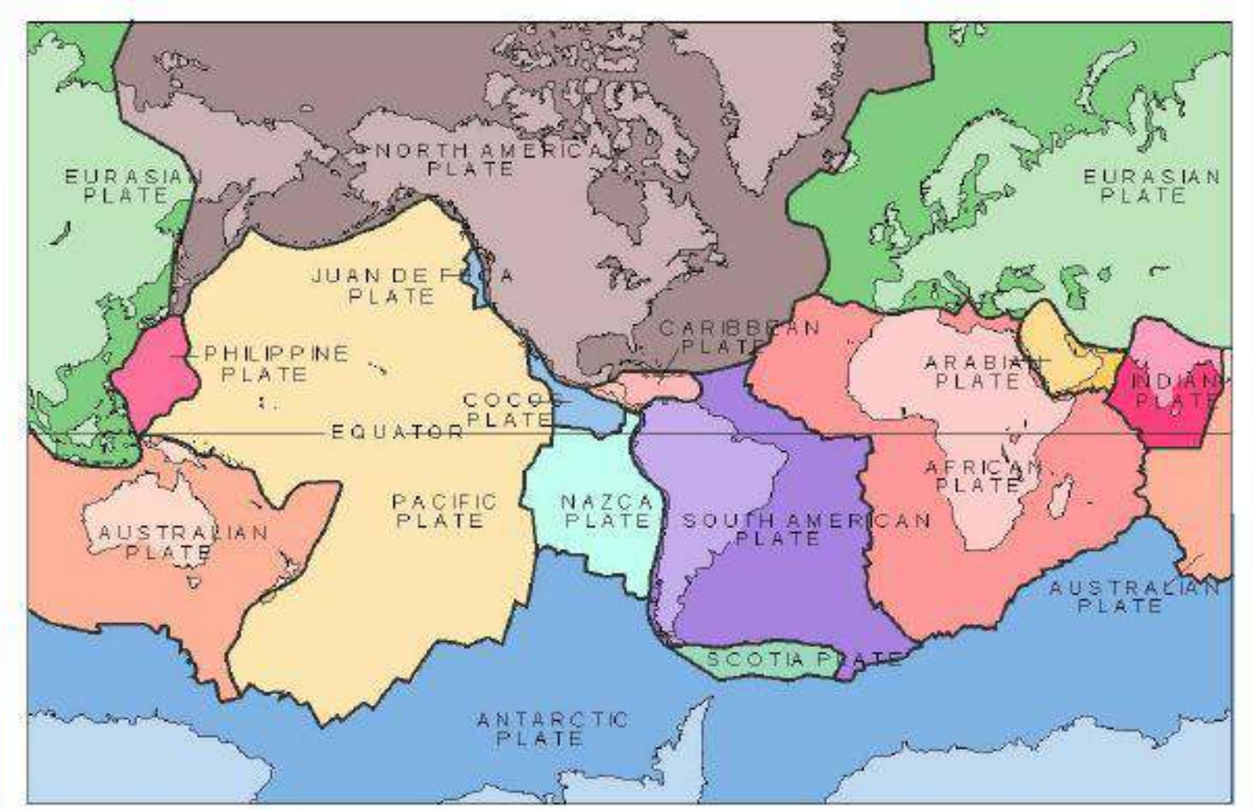
Types of Earthquakes

Continental Drift Theory

1. How the current shape of some continents appear to fit together, particularly the east coast of South America and the west coast of Africa?

2. The significant similarities between fossil records (both flora and fauna) found in several continents that could only have occurred if the continents were attached.

3. The similarity in geology across several continents, including grooves carved by glaciers and the sediments deposited by these glaciers.



Map Taken from the USGS Website

Basic Terminologies

Faults

- At the boundaries of the plates, rocks fracture, usually at many locations, creating a web of smaller plates with edges that rub and push relative to each other; these edges are called *faults*.

Slip

- When this energy is released with a sudden movement (*slip*), it causes brief strong ground vibrations.

Hypocenter/Focus

- The specific location (generally a volume of rock) where the movement or energy release occurs is known as the *focus,* or *hypocenter*.

Basic Terminologies

Epicenter

- The point on the earth's surface directly above the *hypocenter* is called the *epicenter.*

Aftershocks

- Usually, the vibrations cause the rocks near the focus to become unstable; and as these rocks settle into a new equilibrium state they cause *aftershocks*.

Seismology

- The discipline that studies seismic activity is known as *seismology*

Basic Terminologies

Epicenter

- The point on the earth's surface directly above the *hypocenter* is called the *epicenter.*

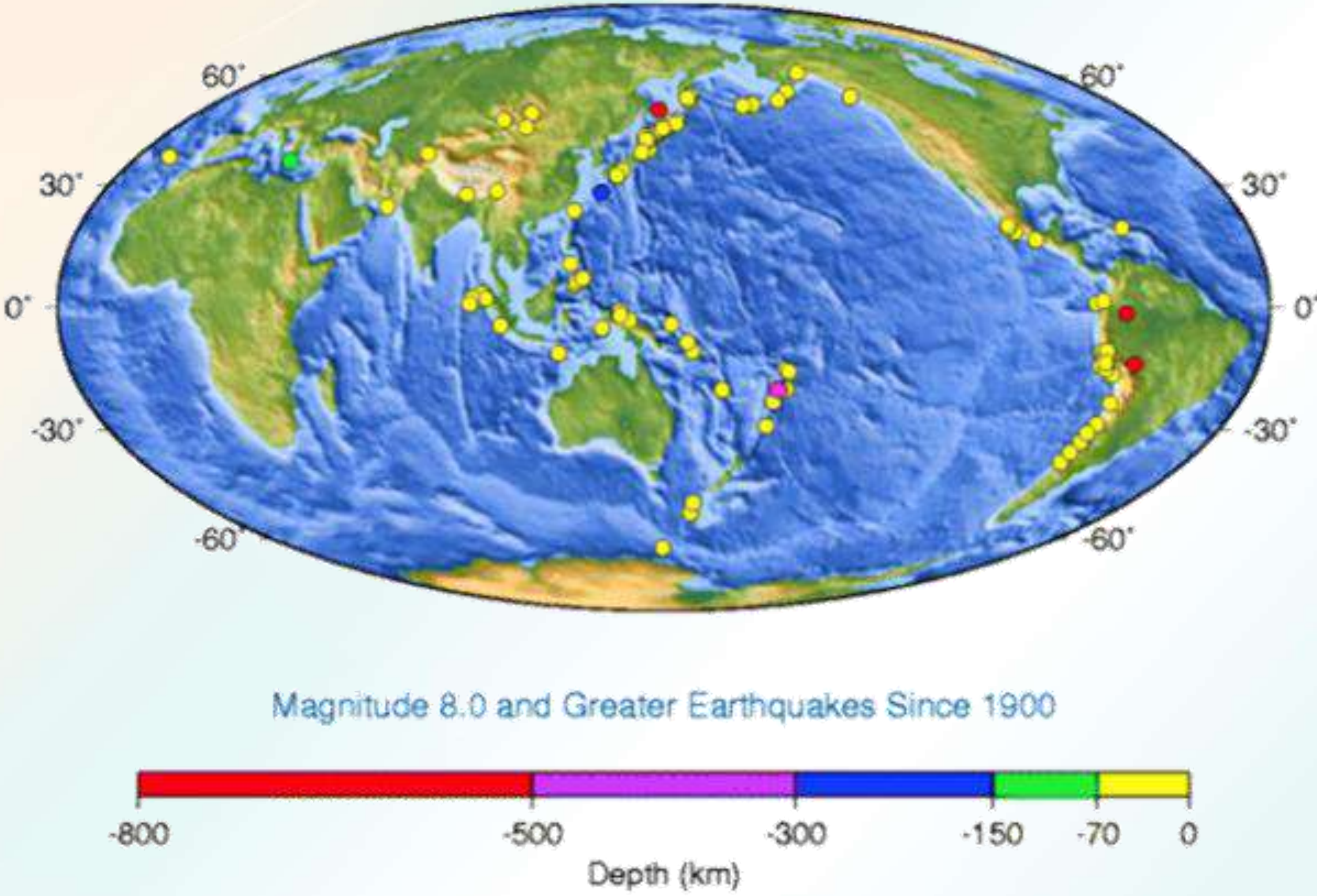
Aftershocks

- Usually, the vibrations cause the rocks near the focus to become unstable; and as these rocks settle into a new equilibrium state they cause *aftershocks*.

Seismology

- The discipline that studies seismic activity is known as *seismology*

Magnitude 8 and Greater Earthquakes Since 1900

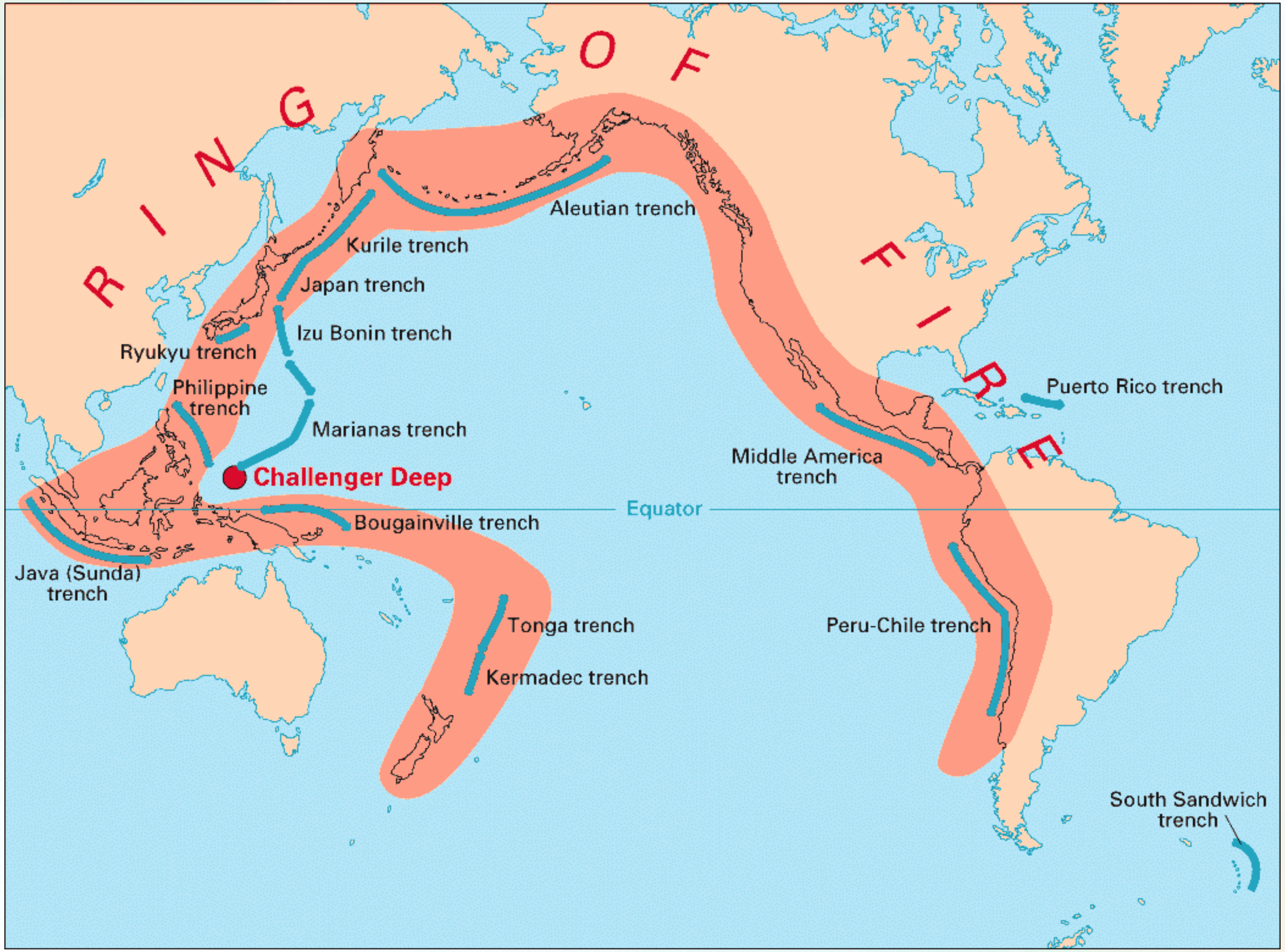


The May 22, 1960 Chile Earthquake is the highest magnitude earthquake recorded (9.5) since 1900.

The December 26, 2004 Sumatra, Indonesia Earthquake has the highest fatalities recorded since 1900.

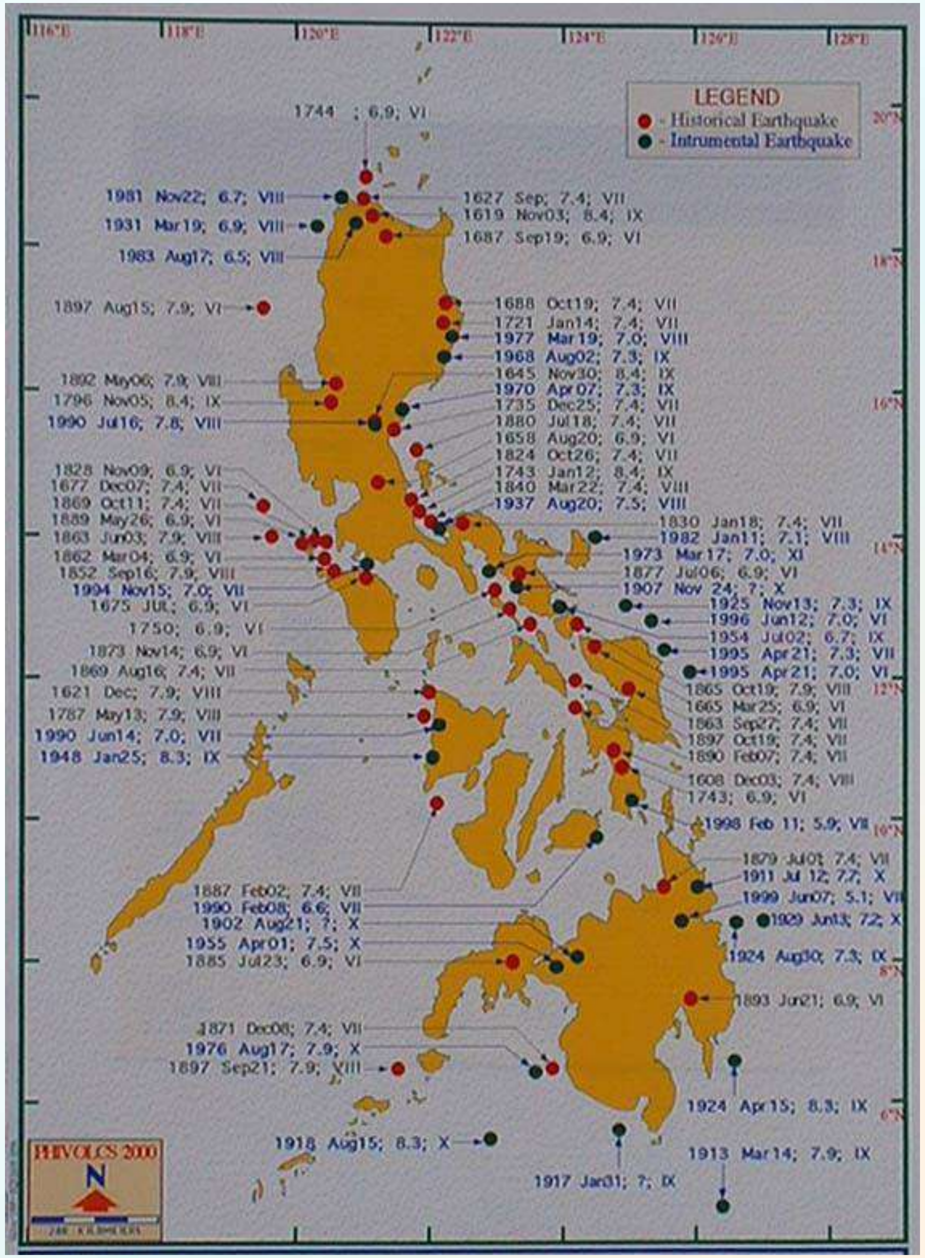
The April 14, 1924 Mindanao, Philippines Earthquake has the highest magnitude recorded (8.3) since 1900.

“Ring of Fire” – Regions of High Seismicity



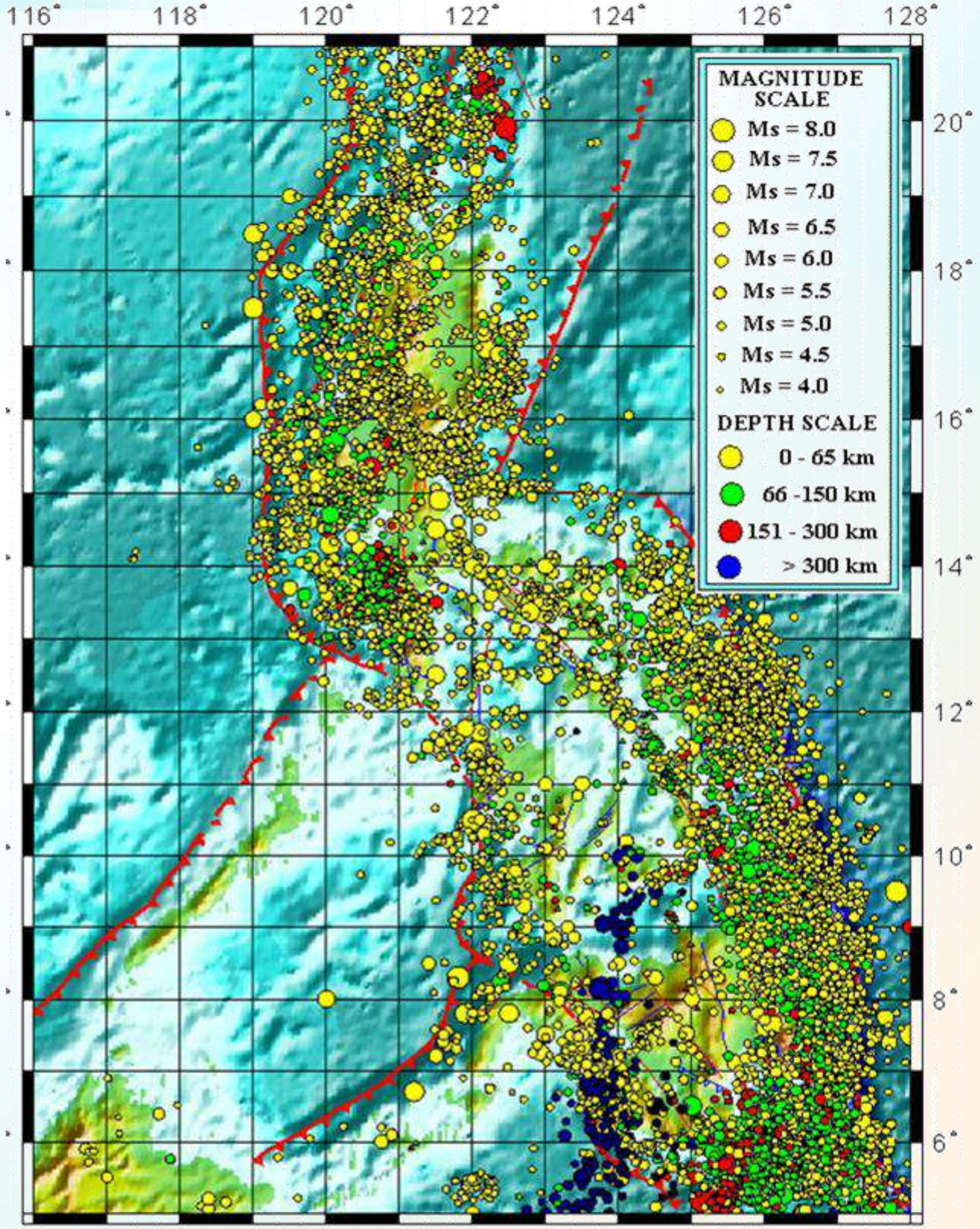
Significant Earthquakes in the Philippines

Excerpt from Pacheco, B.M. , Major Earthquake Zones in the Philippines presentation



Earthquakes for the Past 400 Years

Excerpt from Pacheco, B.M. , Major Earthquake Zones in the Philippines presentation



History of the Development of Mitigation Strategies for the Effects of Seismic Hazards

1. Development of construction practices to mitigate the effects of shaking.

2. The first half of the twentieth century focused on characterizing the effect of shaking using lateral forces.

3. In the 1940s, the development of the response spectrum theory marked a major step forward.

4. In the 1960s, concepts of structural dynamics were incorporated into design practice.

5. The last stage has implicitly been included in building codes since the 1970s and deals with performance-based design.

The dynamic forces are only equivalent to code-specified lateral forces in that a structure designed to resist these forces has the capability of deforming without overstressing from load reversals, and provide adequate member ductility, as well as provide connections with sufficient strength and resiliency to accomplish the following performance goals:

1. Resist minor earthquakes without damage.

2. Resist moderate earthquakes without structural damage, but with some nonstructural damage.

3. Resist major earthquakes without collapse, but with both structural and nonstructural damage.

INTRODUCTION TO BASIC SEISMOLOGY AND EARTHQUAKE ENGINEERING

Engineering Seismology

Engineering Seismology Terminology

Epicenter

- It is the geographical point on the ground surface where an earthquake is estimated to be centered.

Focal Depth

- Together with the epicenter, it gives the location where the rock ruptures at a fault (fault rupture) that generates the main earthquake, the *focus* or *hypocenter*.

- This is an area (not a point) that can extend for many miles along a fault.

Fault Plane

- The plane along which the rock ruptures and slips

Engineering Seismology Terminology

Dip Angle

- An angle with respect to the ground surface

Strike Angle

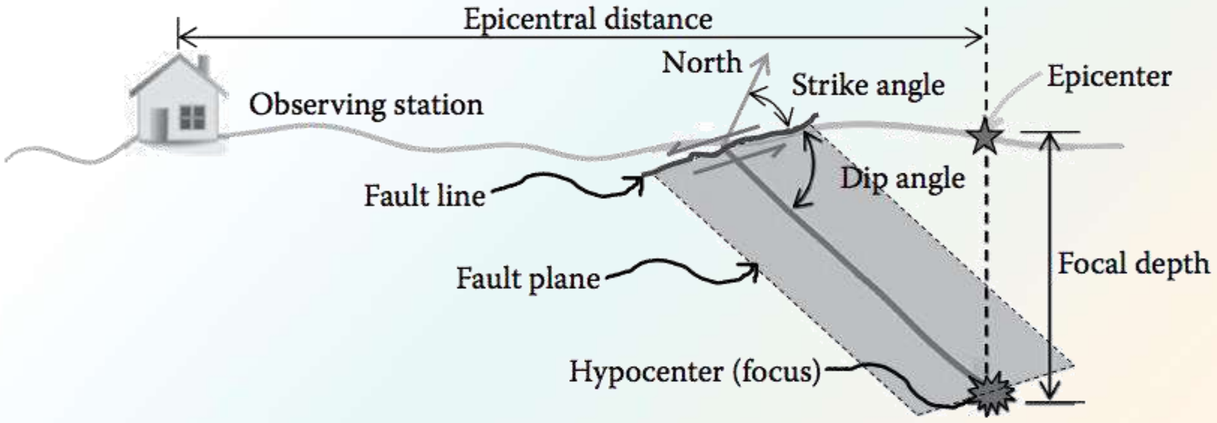
- The angle the fault plane makes with respect to the north direction along the surface

Fault Slip

- The relative displacement between the two sides of the fault plane

Epicentral Distance

- The radiating seismic waves are recorded using a seismometer at an *observation station* located at a distance



Estrada and Lee, 2017

Classification of Focal Depth

Shallow Focal Depth

- It is characterized by focal depths less than 70 km (43 miles)

Intermediate Focal Depth

- It is characterized by focal depths between 70km (43 miles) and 300km (186 miles)

Deep Distance

- It is characterized by focal depths greater than 300 km (186 miles)

Engineering Seismology Terminology

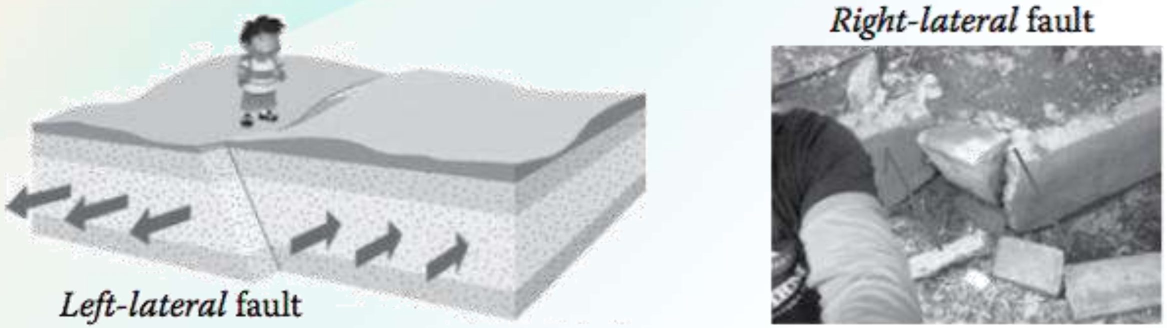


FIGURE 2.3 Horizontal movement, *strike-slip* fault.

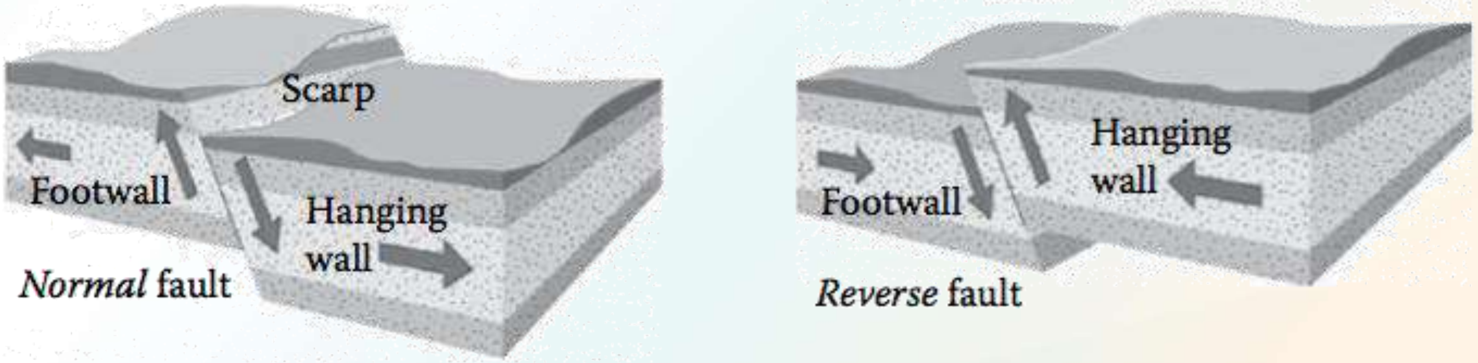


FIGURE 2.2 Vertical movement, *dip-slip* faults.

Estrada and Lee, 2017

Elastic Rebound Theory

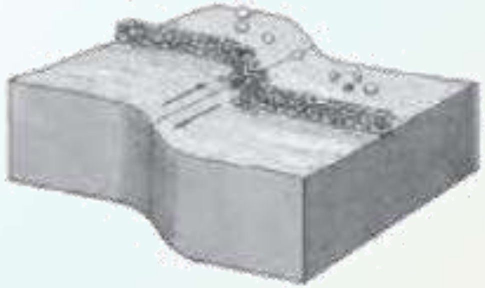
Elastic Rebound Theory: Tectonic Plate Movement

- The elastic-rebound theory is an explanation for how energy is released during an earthquake.

- As the Earth's crust deforms, the rocks which span the opposing sides of a fault are subjected to shear stress. Slowly they deform, until their internal rigidity is exceeded.

- Then they separate with a rupture along the fault; the sudden movement releases accumulated energy, and the rocks snap back almost to their original shape.

Unstrained rock



Strain buildup

Rebound to unstrained state

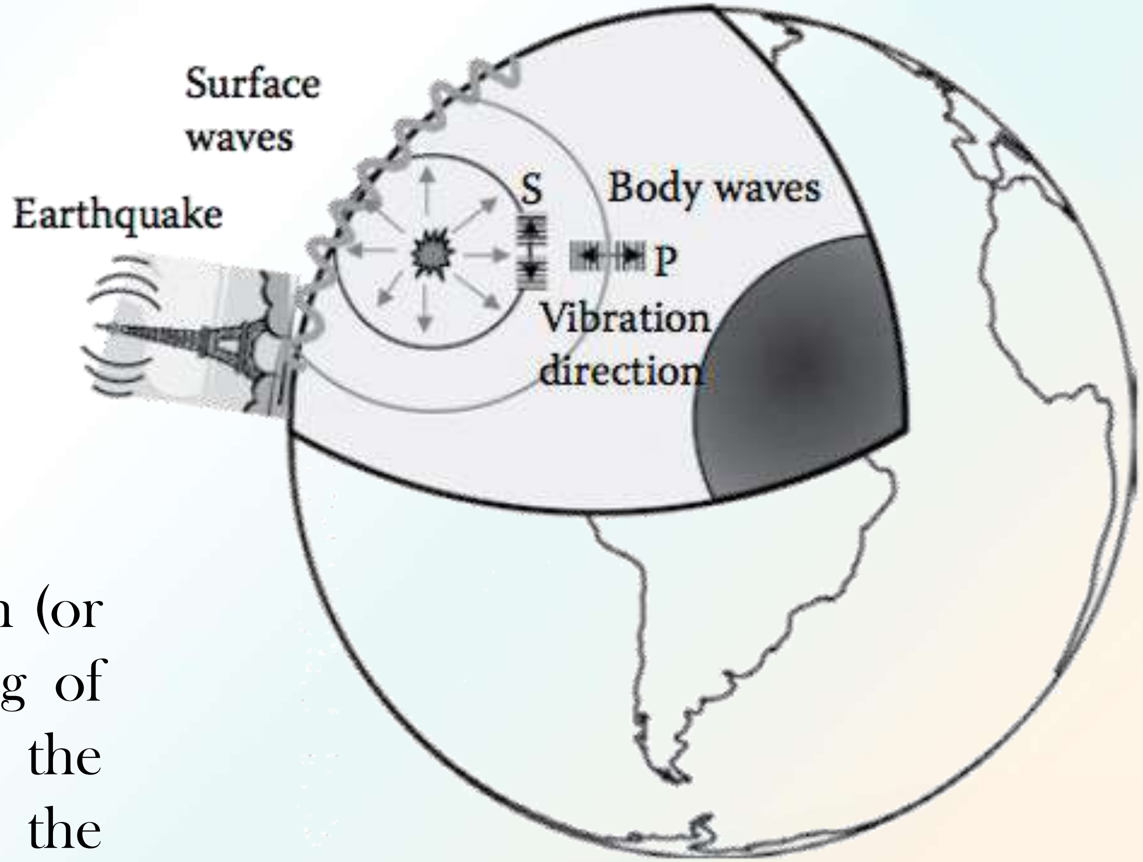
Estrada and Lee, 2017

Seismic Waves

- Seismic waves radiate from the focus and travel in every direction, as shown

- The portion of the energy released from a fault rupture as shaking first travels through the interior of the earth as *body waves*, following the shortest path to the ground surface where they are transformed into *surface waves*

- Thus, body waves arrive at an observation station (or site) before any surface waves. An understanding of these different waves is necessary to establish the epicenter and to characterize the size of the earthquake (earthquake magnitude and ground acceleration).



Estrada and Lee, 2017

Types of Seismic Waves

P-Waves (Primary or Pressure)

- P-waves travel faster than other seismic waves and hence are the first signal from an earthquake to arrive at any affected location or at a seismograph.

S-Waves (Secondary or Shear)

- S-waves are transverse waves, meaning that the oscillations of an S-wave's particles are perpendicular to the direction of wave propagation, and the main restoring force comes from shear stress

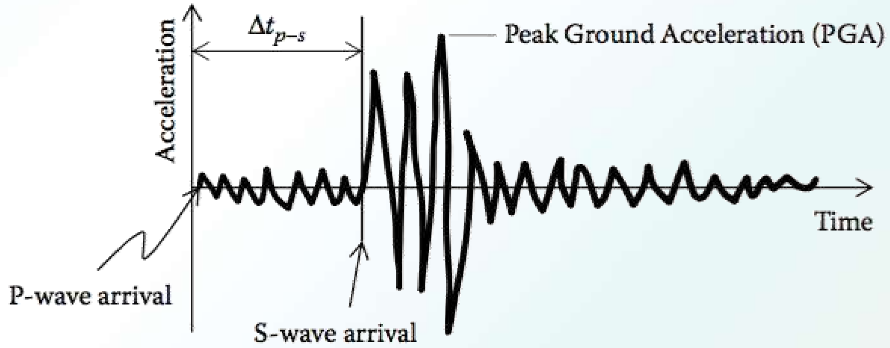


FIGURE 2.6 Seismograph—a strong-motion recording using a seismometer.

Estrada and Lee, 2017

Measuring Earthquakes

Earthquake Intensity

- The intensity of an earthquake is a subjective, non-empirical approach for estimating the size of an earthquake based on the subjective assessment of human observations of the effects of earthquake shaking on buildings (amount of damage sustained by structures and land surface) and on people.

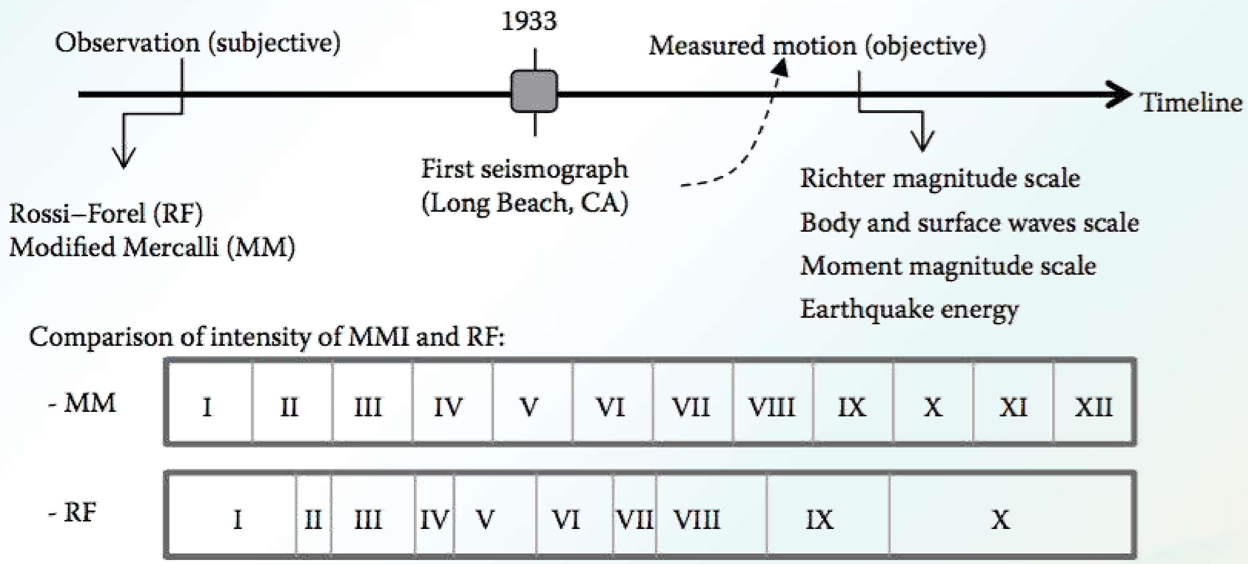


FIGURE 2.7 Scales used to estimate the relative size of earthquakes.

Estrada and Lee, 2017

Measuring Earthquakes

Earthquake Intensity

Estrada and Lee, 2017

TABLE 2.1
Modified Mercalli Intensity Scale, and Comparison to Magnitude and PGA

Degree	Intensity Description	Magnitude	PGA (*g*)
I	Not felt, except by some in rare circumstances.	<3.0	<0.03
II	Felt only by a few persons at rest on upper floors of buildings.	3.0–3.9	
III	Felt indoors, but many people do not recognize it as an earthquake.		
IV	During the day felt indoors by many, outdoors by few, while at night some awakened. Dishes, windows, doors are disturbed and walls make creaking sounds.	4.0–4.9	
V	Felt by nearly everyone during the day and many awakened at night. Some dishes, windows, etc. are broken; cracked plaster in a few places; unstable objects overturned.		0.03–0.08
VI	Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster and damaged chimneys. Damage slight.	5.0–5.9	0.08–0.15
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys damaged.		0.15–0.25
VIII	Damage slight in specially designed structures; considerable in ordinary buildings (some with partial collapse); great in poorly built structures. Chimneys, factory stack, columns, monuments, walls toppled. Heavy furniture overturned.	6.0–6.9	0.25–0.45
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great damage for others, including partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.	7.0–7.9	0.45–0.6
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed. Ground badly cracked.		0.6–0.8
XI	Few masonry structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service.	>8.0	0.8–0.9
XII	Damage total. Waves seen on ground surface. Objects thrown into the air.		>0.90

Measuring Earthquakes

Earthquake Magnitude

- Whereas the intensity of a given earthquake varies from one observation point to another, earth- quakes can be associated with a single value of magnitude.

- The consensus measure of magnitude is based on the Richter scale, which quantifies the size of an earthquake with an index of the amount of energy released, and while this approach is an improvement as compared to the intensity scales, the Richter scale does not accurately account for all factors that contribute to the actual size of an earthquake.

- This scale, however, does appropriately measure the relative strength of an earthquake and remains an important parameter in earthquake hazard analysis.

Measuring Earthquakes

Earthquake Magnitude

- In addition to being used for earthquake hazard analysis by seismologists and engineers, it is the preferred scale used to inform the public of the size of an earthquake.

- Because it was originally developed to quantify the strength of Southern California earthquakes, the Richter scale is also known as the local magnitude scale, M_L. Richter defined M_L using the base-10 logarithm of the peak trace amplitude (in micrometers, μm) of a standard Wood–Anderson seismograph (which has a magnification factor of 2800, a natural period of 0.8 s, and damping of 80%), located on firm ground at a distance of 100 km from the epicenter.

Measuring Earthquakes

Earthquake Magnitude

The following relationship gives the local magnitude:

$$M_L = \log_{10} \frac{A}{A_o}$$

where:

A = the peak amplitude in micrometer, measured from a seismogram.

A_o = the peak amplitude of a zero-magnitude earthquake in μm, which is used to adjust the variation of ground motion amplitude for epicentral distances other than 100 km

Measuring Earthquakes

Earthquake Magnitude

The previous equation is typically not used directly to determine the magnitude of an earthquake; instead, a correction nomogram provided by Richter is used. The nomogram was convenient when digital calculators were not available. The equation used to develop the nomogram is

$$M_L = \log_{10} A + 3\log_{10}(8\Delta t_{p-s}) - 2.92$$

where:

A = the peak amplitude in mm, measured from a seismogram.

Δt_{p-s} = the time between the arrival of P- and S-waves in seconds; this indirectly measures the epicentral distance

Measuring Earthquakes

Earthquake Magnitude

The Richter scale has a practical range from 0 to 9.0 (theoretically, the scale has no upper or lower limits). Also, the base-10 logarithmic scale indicates that each unit increase in M_L corresponds to a 10-fold increase of the earthquake wave amplitude. For example, a 7 M_L earthquake is 100 times stronger than a 5 M_L event ($10 \times 10 = 100$).

Measuring Earthquakes

Problem 1:

Estimate the local magnitude of a southern California earthquake recorded in two perpendicular directions at several stations using standard Wood–Anderson seismographs. The trace amplitudes and epicentral distances are as follows (Richter, 1958):

	Amplitude on the Seismograph, A (μm)		Epicentral Distance, Δ (km)
Station	N–S Component	E–W Component	
1	8400	6000	114
2	7900	8500	179
3	24,500	30,000	90
4	8100	7000	246

Measuring Earthquakes

Problem 1 - Solution

$$M_{LN-S} = \log_{10} A_{N-S} + 2.56\log_{10} D_{N-S} - 5.12$$

$$M_{LN-S} = \log_{10}\left(8.4x10^3\, mm\right) + 2.56\log_{10}\left(114km\right) - 5.12 = 4.07$$

$$M_{LE-W} = \log_{10}\left(6x10^3\, mm\right) + 2.56\log_{10}\left(114km\right) - 5.12 = 4.92$$

The average of these two values gives M_L at station 1. That is,

$$M_{L1} = \frac{4.07 + 3.92}{2} = 4.0$$

Measuring Earthquakes

Problem 1 - Solution

All other stations are treated similarly and the results are summarized as follows:

	Local Magnitude		
Station	M_{LN-S}	M_{LE-W}	Average M_L
1	4.07	3.92	4.0
2	4.54	4.58	4.6
3	4.27	4.36	4.3
4	4.91	4.84	4.9

Therefore, the magnitude of this particular earthquake can be determined as the average of the averages of each station:

$$M_L = \frac{4.0 + 4.6 + 4.3 + 4.9}{4} = 4.4$$

Measuring Earthquakes

Energy Radiated of an Earthquake (E) – Equation 1

The local energy associated with earthquakes during fault fracture growth is primarily trans- formed into heat, with only 1%–10% being released as seismic waves. The relationship between the fraction of energy radiated as waves and the local magnitude, M_L, is given as follows:

$$\log_{10} E = 11.8 + 1.5M_L$$

where:

E = in ergs, which is relatively small unit, 1 ft-lb = 1.356×10^7 ergs

Measuring Earthquakes

Energy Radiated of an Earthquake (E) – Equation 2

All of which can be estimated relatively accurately. Alternatively, the moment can be directly estimated from the amplitudes of long-period waves at large distances, with corrections for attenuation and directional effects. Although moment is an effective way to establish the size of an earth- quake, it is customarily convenient to convert it into a magnitude quantity so that it can be compared to M_L, m_b, and M_s. This can be accomplished by relating the seismic moment M_0 to the radiated energy E. That is,

$$E = \frac{\Delta\sigma}{2G} M_0$$

where:

$\Delta\sigma$ is the static stress drop in the earthquake, which ranges from 30 to 60 bars (1 bar = 100 kPa)

G is the shear modulus of the medium near the fault

Measuring Earthquakes

Body Wave and Surface Wave Scale

The wave train recorded on a seismograph for a deep earthquake is very different than for a shallow one, leading to two different values of M_L even for events that release equal amounts of total energy. Consequently, more accurate measures of magnitude had to be developed in order to improve the uniform coverage of earthquake size. Richter working with his colleague Gutenberg addressed the shortcomings of the local magnitude scale by developing the *body-wave* scale, m_b, to handle deep-focus earthquakes and the *surface-wave* scale, M_s, to handle distant earth- quakes. The three scales are related by the following empirical relationships:

$$M_s = 1.27(M_L - 1) - 0.016M_L^2$$

$$m_b = 0.63M_s + 2.5$$

Measuring Earthquakes

Moment Magnitude Scale (M_w)

The strength of an earthquake can be more accurately measured using the moment magnitude scale, M_W, which accurately measures a wide range of earthquake sizes and is applicable globally. This scale is a function of the total moment release by an earthquake. The moment is a measure of the total energy released. The concept of moment is adopted from mechanics and is defined as the product of the fault displacement and the force causing the displacement. A simple derivation of the moment, M_0, is presented in Villaverde and is based on the size of the fault rupture, the slip amount, and the stiffness of the fractured rocks. That is,

$$M_0 = G A_f D_s$$

where:

M_0 is in dyne cm, which is a relatively small unit, 1 dyne cm = 1×10^{-7} N m

G is the shear modulus of the rocks included in the fault in dyne/cm2, which ranges from 3.2×10^{11} dyne/cm2 in the crust to 7.5×10^{11} dyne/cm^2 in the mantle

A_f is the area of the fault rupture in cm^2

D_s is the average fault slip or displacement in cm

Measuring Earthquakes

Moment Magnitude Scale (M_w)

The moment magnitude, M_w, is then derived by substituting Equation 2 into Equation 1 and replacing M_L with M_w since the two scales give similar values for earthquakes of magnitude ranges from 3 to 5. For average values of $\Delta\sigma$ and G ($\Delta\sigma/G \cong 10^{-4}$), the relationship between M_0 and M_w is then given as

$$M_w = \frac{2}{3}\log_{10} M_0 - 10.7$$

where:

M_0 is in dyne cm, which is a relatively small unit, 1 dyne cm = 1×10^{-7} N m

The resulting M_w does not saturate at large magnitude values. M_w is also known as the Kanamori wave energy, after the scientist who developed this relationship (Villaverde, 2009).

Measuring Earthquakes

Return Period (RP)

$$\mathrm{RP} = \frac{-T}{\ln(1-P)}c$$

where:

P = the probability of exceedance in T years

Measuring Earthquakes

Problem 2:

Estimate the seismic moment and moment magnitude of the January 12, 2010 Haiti earthquake. It is estimated that the *blind thrust* fault (the slip plane ends before reaching the earth's surface) caused an average strike-slip displacement of 2 m over an area equal to 30 km long by 15 km deep (Eberhard et al. 2010). Assume that the rock along the fault has an average shear rigidity of 3.2×10^{11} dyne/cm^2.

Problem 2 - Solution

1. Determine the fault's rupture area A_f and fault slip, D_s in consistent units.

$$A_f = \left(30x10^5\,cm\right)\left(15x10^5\,cm\right) = 4.5x10^{12}\,cm^2$$

$$D_s = 2m = 20cm$$

Measuring Earthquakes

Problem 2 - Solution

2. Determine the seismic moment M_o

$$M_o = GA_fD_s$$

$$M_o = \left(3.2x10^{11}\frac{dyne}{cm^2}\right)\left(4.5x10^{12}cm^2\right)\left(200cm\right)$$

$$M_o = 2.88x10^{26}dyne - cm$$

3. Determine the moment magnitude M_w

$$M_w = \frac{2}{3}\log_{10}M_o - 10.7$$

$$M_w = \frac{2}{3}\log_{10}\left(2.88x10^{26}dyne - cm\right) - 10.7$$

$$M_w = 6.94$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Earthquake Hazard Assessment

The deterministic approach can be used in areas where seismic activity is frequent and its sources are well defined. The process is relatively simple and entails the following:

1. Identifying nearby seismic sources.
2. Identifying the distance to the structure site from nearby seismic sources.
3. Determining the magnitude and characteristics of nearby seismic sources.
4. Establishing the structural response to the effects from all nearby seismic sources.
5. Selecting the case that produces the largest structural response.

Q&A SESSION

ASK ANY QUESTION RELATED TO OUR TOPIC FOR TODAY.

HOMEWORK

Do the Exercises about the Introduction to Earthquake Engineering

Reference:

Bird, J. F., & Bommer, J. J. (2004). Earthquake losses due to ground failure. *Engineering geology*, *75*(2), 147-179.

Estrada, H., & Lee, L. S. (2017). *Introduction to Earthquake Engineering*. CRC Press.

Galli, Paolo & Castenetto, Sergio & Peronace, Edoardo. (2017). The macroseismic intensity distribution of the October 30, 2016 earthquake in central Italy (Mw 6.6). Seismotectonic implications.: 2016 central Italy earthquake intensity. Tectonics. 10.1002/2017TC004583.

Glass, C. E. (2013). *Interpreting aerial photographs to identify natural hazards*. Elsevier.

Hassan, Said & Al-Harbi, Tawfiq & Al-Yami, Mahdi & Al-Ghamdi, Ahmed & Al-Shammari, Mohammed. (2016). Earthquake Disaster Management Approach: The Case of Al-Ais, Medina Area in Saudi Arabia. Open Journal of Earthquake Research. 05. 219-235. 10.4236/ojer.2016.54018.

Reference:

Memari, Mehrdad & Turbert, Collin & Mahmoud, Hussam. (2013). Effects of Fire Following Earthquakes on Steel Frames with Reduced Beam Sections. Structures Congress 2013: Bridging Your Passion with Your Profession - Proceedings of the 2013 Structures Congress. 10.1061/9780784412848.223.

CEELECT1
Earthquake Engineering
Base Shear Computations

Module 2

OBJECTIVES

- At the end of the chapter, the learner should be able to:
  - *Interpret the code provisions regarding base shear and lateral forces.*
  - *Analyze the seismic data of the structure.*
  - *Solve the design base shear and distribution of lateral forces per level.*

INTRODUCTION TO EARTHQUAKE ENGINEERING

Base Shear Computation using the NSCP Code

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

208.5.1. Simplified Static Force Procedure

Structures conforming to the requirements of Section 208.4.8.1 may be designed using this procedure.

<u>Section 208.4.8.1. Simplified Static</u>

The simplified static lateral force procedure set forth in Section 208.5.1.1. may be used for the following structures of Occupancy Category IV or V.

1. Building of any occupancy (including single – family dwellings) not more than three stories in height excluding basements that use light frame construction.

2. Other buildings not more than two stories in height excluding basements.

208.5.1. Simplified Static Force Procedure

Structures conforming to the requirements of Section 208.4.8.1 may be designed using this procedure.

208.5.1.1. Simplified Design Base Shear

The total design base shear in a given direction shall be determined from the following equation:

$$V = \frac{3C_a}{R}W \qquad Eq.\ 208-5$$

Where the value of C_a shall be based on Table 208-7 for the soil profile type. When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used in Seismic Zone 4 and Type S_E shall be used in Seismic Zone 2. In Seismic Zone 4, the Near Source Factor, N_a, need not be greater than 1.2 if none of the following structural irregularities are present:

1. Type 1, 4 or 5 of Table 208-9 or
2. Type 1 or 4 of Table 208-10

208.5.1. Simplified Static Force Procedure

Structures conforming to the requirements of Section 208.4.8.1 may be designed using this procedure.

208.5.1.2. Vertical Distribution

The forces at each level shall be calculated using the following equation:

$$F_x = \frac{3C_a}{R} W_i \qquad Eq.\ 208-6$$

Where the value of C_a shall be determined as in Section 208.5.1.1.

Table 103-1: Occupancy Category

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTION OF STRUCTURE
1. Essential Facilities	✓ Occupancies having surgery and emergency treatment areas ✓ Fire and police stations ✓ Garages and shelters for emergency vehicles and emergency aircraft ✓ Structures and shelters in emergency preparedness centers ✓ Aviation control towers ✓ Structures and equipment in communication centers and other facilities required for emergency response ✓ Facilities for standby power-generating equipment for Category 1 structures ✓ Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category I, II, or III, IV and V structures ✓ Public school buildings ✓ Hospitals ✓ Designated evacuation centers and ✓ Power and communication transmission lines

Table 103-1: Occupancy Category

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTION OF STRUCTURE
2. Hazardous Facilities	✓ Occupancies and structures housing or supporting toxic or explosive chemicals or substances ✓ Non-building structures storing, supporting or containing quantities of toxic or explosive substances
3. Special Occupancy Structures	✓ Buildings with an assembly room with an occupant capacity of 1,000 or more ✓ Educational buildings such as museums, libraries, auditorium with a capacity of 300 or more occupants ✓ Institutional buildings with 50 or more incapacitated patients, but not included in Category 1 ✓ Mental hospitals, sanitariums, jails, prisons and other buildings where personal liberties of inmates are similarly restrained ✓ Churches, Mosques and other Religion Facilities ✓ All structures with an occupancy of 5,000 or more persons ✓ Structures and equipment in power-generating stations and other public utility facilities not included in Category 1 or Category 2 and required for continued operation

Table 103-1: Occupancy Category

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTION OF STRUCTURE
4. Standard Occupancy Structures	✓ All structures housing occupancies or having functions not listed in Category I, II or III, and Category V.
5. Miscellaneous Structures	✓ Private garages, carports, sheds and fences over 1.5m high

208.5.2. Static Force Procedure

208.5.2.1. Design Base Shear

The total design base shear in a given direction shall be determined from the following equation:

$$V = \frac{C_v I}{RT} W \qquad Eq.\ 208-8$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \qquad Eq.\ 208-9$$

208.5.2. Static Force Procedure

208.5.2.1. Design Base Shear

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

$$V = 0.11 C_a I W \qquad Eq.\ 208-10$$

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

$$V = \frac{0.8 Z N_v I}{R} W \qquad Eq.\ 208-11$$

Table 208-1: Seismic Importance Factors

OCCUPANCY CATEGORY	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE FACTOR, I_p[2]
1. Essential Facilities	1.50	1.50
2. Hazardous Facilities	1.25	1.50
3. Special Occupancy Structures	1.00	1.00
4. Standard Occupancy Structures	1.00	1.00
5. Miscellaneous Structures	1.00	1.00

2 The limitation of I_p for panel connections in Section 208.7.2.3 shall be 1.0 for the entire connector

3 Structural observation requirements are given in Section 107.9

4 For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5

208.4.3. Site Geology and Soil Characteristics

Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Section 208.4.3.1.1 and table 208-2.

Exception:

When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used. Soil Profile Type S_E or S_F need not be assumed unless the building official determines that Type S_E or S_F may be present at the site or in the event that Type S_E or S_F is established by geotechnical data.

Table 208-2: Soil Profile Types

Soil Profile Type	Soil Profile Name/Generic Description	Average Soil Properties for Top 30m of Soil Profile		
		Shear Wave Velocity, V_s (m/s)	SPT, N (blows/300mm)	Undrained Shear Strength, S_U (kPa)
S_A	Hard Rock	> 1500		
S_B	Rock	760 to 1500		
S_C	Very Dense Soil and Soft Rock	360 to 760	> 50	> 100
S_D	Stiff Soil Profile	180 to 360	15 to 50	50 to 100
S_E[1]	Soft Soil Profile	< 180	< 15	< 50
S_F	Soil Requiring Site-specific Evaluation. (See Section 208.4.3.1)			

1 Soil Profile Type S_E also includes any soil profile with more than 3.0m of soft clay defined as a soil with plasticity index, PI>20, $w_{mc} \geq 40\%$ and $s_u < 24$ kPa. The Plasticity Index, PI and the moisture content, w_{mc} shall be determined in accordance with approved national standards.

FEU ALABANG FEU DILIMAN FEU TECH

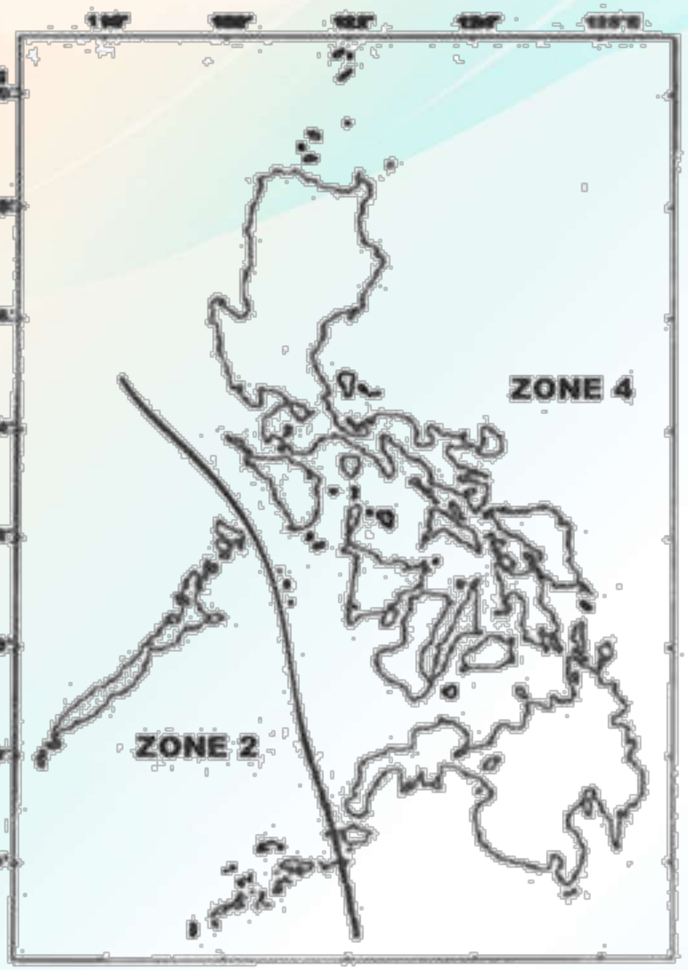
Technology Driven by Innovation

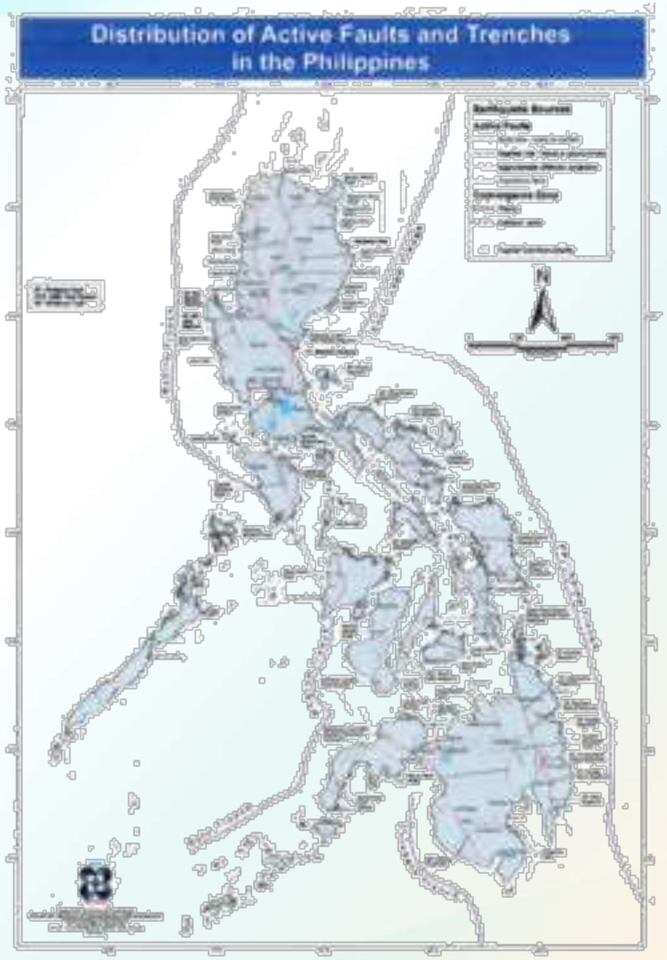
208.4.4. Site Seismic Hazard Characteristics

Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

208.4.4.1 Seismic Zone

The Philippine Archipelago is divided into two seismic zones only. Zone 2 covers the provinces of Palawan (except Busuanga), Sulu and Tawi-Tawi while the rest of the country is under Zone 4 as shown in figure 208-1. Each structure shall be assigned a seismic zone factor Z, in accordance with Table 208-3.

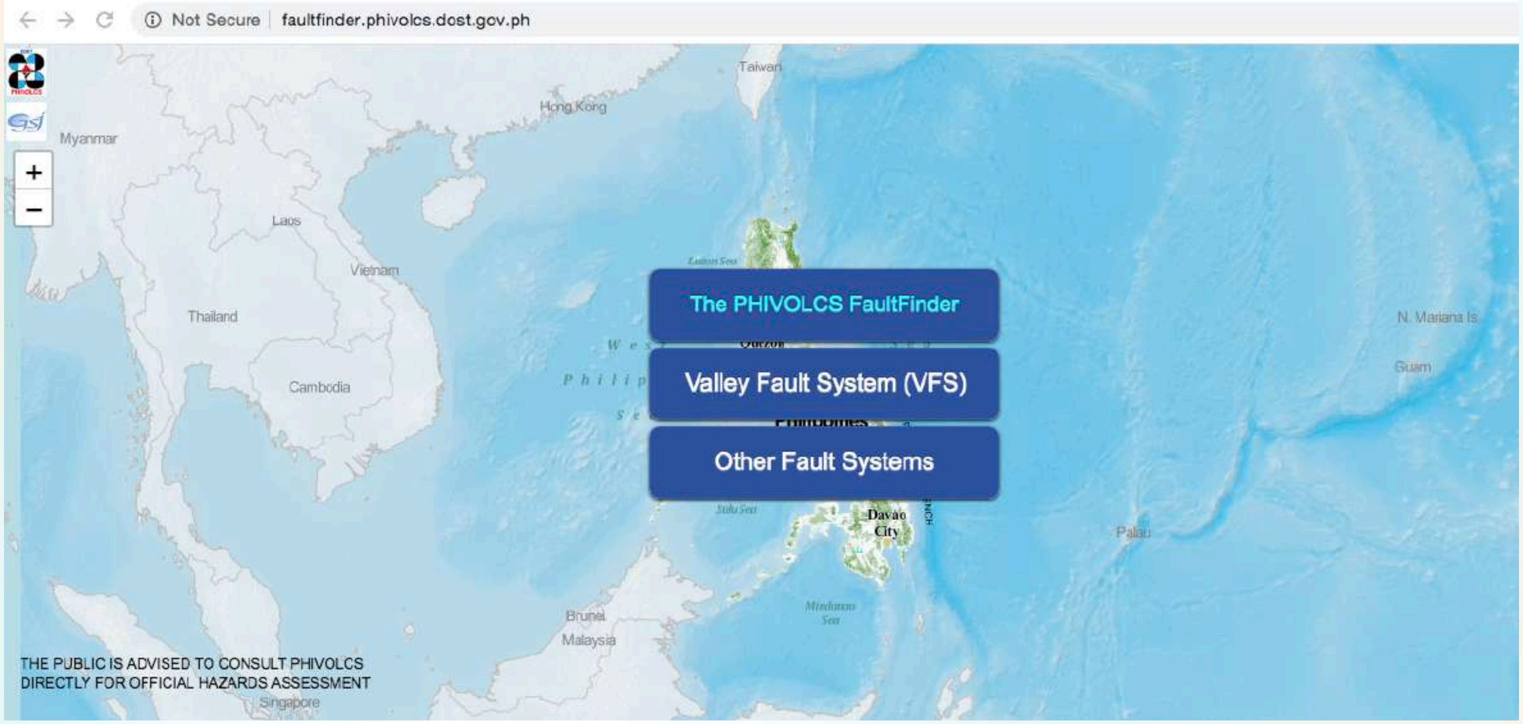




FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

PHIVOLCS Fault Finder



You can determine the distance from a known source using the link below:

http://faultfinder.phivolcs.dost.gov.ph/

Table 208-3: Seismic Zone Factor Z

Zone	2	4
Z	0.20	0.40

208.4.4.2 Seismic Source Types

Table 208-4 defines the types of seismic sources. The location and type of seismic sources to be used for design shall be established based on approved geological data. Type A sources shall be determined from figure 208-2B, 2C, 2D, 2E or the most recent mapping of active faults by the Philippine Institute of Volcanology and Seismology (PHIVOLCS).

Table 208-4: Seismic Sources Types

Seismic Source Type	Seismic Sources Description	Seismic Source Definition (Maximum Moment Magnitude, M)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity.	$7.0 \leq M \leq 8.4$
B	All faults other than Types A and C.	$6.5 \leq M < 7.0$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity.	$M < 6.5$

208.4.4.3. Seismic Zone 4 Near-Source Factor

In Seismic Zone 4, each site shall be assigned near-source factors in accordance with Tables 208-5 and 208-6 based on the Seismic Source Type as set forth in Section 208.4.4.2.

For high rise structures and essential facilities within 2.0km of a major fault, a site specific seismic elastic design response spectrum is recommended to be obtained for the specific area.

Table 208-5: Near-Source Factor, N_a[1]

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

Table 208-6: Near-Source Factor, N_v[1]

Seismic Source Type	Closest Distance to Known Seismic Source[2]			
	< 2km	5km	10km	> 10km
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 closest distance to seismic source shall be taken as the minimum distance between the size and the area described by the vertical projection of the source on the surface. The surface projection need not include portions of the source at depths of 10km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

The value of N_a used to determine C_a need not exceed 1.1 for structures complying with all the following conditions:

1. The soil profile type is S_A, S_B, S_C, or S_D
2. ρ = 1.0
3. Except in single storey structures, residential building accomodating 10 or fewer persons, private garages, carports, shed and agricultural buildings, moment frame systems designated as part of the lateral-force-resisting system shall be special moment-resisting frames.
4. The exceptions to Section 515.6.5 shall not apply, except for columns in one storey buildings or columns at the top storey of multi-storey buildings.
5. None of the following structural irregularities is present: Type 1, 4 or 5 of Table 208-9 and Type 1 or 4 of Table 208-10

208.4.4.4. Seismic Response Coefficients

Each Structure shall be assigned a seismic coefficient, C_a in accordance with Table 208-7 and a seismic coefficient, C_v in accordance with table 208-8.

1 *Site Specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients.*

FEU ALABANG FEU DILIMAN FEU TECH

Table 208-7: Seismic Coefficient, C_a

Soil Profile Type	Seismic Zone (Z)	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	See Footnote 1 of Table 208-8	

Table 208-8: Seismic Coefficient, C_v

Soil Profile Type	Seismic Zone (Z)	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_v$
S_B	0.20	$0.40N_v$
S_C	0.32	$0.56N_v$
S_D	0.40	$0.64N_v$
S_E	0.64	$0.96N_v$
S_F	See Footnote 1 of Table 208-8	

Technology Driven by Innovation

Table 208-11A: Earthquake-Force-Resisting Structural Systems of Concrete

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
✓Special Reinforced Concrete Shear Walls	4.5	2.8	NL	50
✓Ordinary Reinforced Concrete Shear Walls	4.5	2.8	NL	NP
B. Building Frame System				
✓Special Reinforced Concrete Shear Walls or Braced Frames (Shear Walls)	5.0	2.8	NL	75
✓Ordinary Reinforced Concrete Shear Walls or Braced Frames	5.6	2.2	NL	NP
✓Intermediate precast shear walls or braced frames	5.0	2.5	NL	10
C. Moment-Resisting Frame Systems				
✓Special Reinforced Concrete moment frames	8.5	2.8	NL	NL
✓ Intermediate reinforced concrete moment frames	5.5	2.8	NL	NP
✓ Ordinary Reinforced Concrete Moment Frames	3.5	2.8	NL	NP

Table 208-11A: Earthquake-Force-Resisting Structural Systems of Concrete

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
D. Dual Systems				
✓ Special Reinforced Concrete Shear Walls	8.5	2.8	NL	NL
✓ Ordinary Reinforced Concrete Shear Walls	6.5	2.8	NL	NP
E. Dual System with Intermediate Moment Frames				
✓ Special Reinforced Concrete Shear Walls	6.5	2.8	NL	50
✓ Ordinary Reinforced Concrete Shear Walls	5.5	2.8	NL	NP
✓ Shear wall frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	4.2	2.8	NP	NP
F. Cantilevered Column Building Systems				
✓ Cantilevered Column Elements	2.2	2.0	NL	10
G. Shear Wall-Frame Interaction Systems	5.5	2.8	NL	50

Table 208-11B: Earthquake-Force-Resisting Structural Systems of Steel

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
✓ Light Steel-Framed Bearing Walls with Tension Only Bracing	2.8	2.2	NL	20
✓ Braced frames where bracing carries gravity load				
✓ Light framed walls sheathed with steel sheets	4.4	2.2	NL	50
structural panels rated for shear resistance or steel sheets	5.5	2.8	NL	20
✓ Light-framed walls with shear panels of all other light materials	4.5	2.8	NL	20
✓ Light-framed wall systems using flat strap bracing	2.8	2.2	NL	NP

Table 208-11B: Earthquake-Force-Resisting Structural Systems of Steel

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
B. Building Frame Systems				
✓ Steel eccentrically braced frames (EBF), moment resisting connections at columns away from links	8.0	2.8	NL	30
✓ Steel eccentrically braced frames (EBF), non-moment-resisting connections at columns away from links	6.0	2.2	NL	30
✓ Special concentrically braced frames (SCBF)	6.0	2.2	NL	30
✓ Ordinary concentrically braced frames (OCBF)	3.2	2.2	NL	NP
✓ Light-framed walls sheathed with steel sheet structural panels/sheet steel panels	6.5	2.8	NL	20
✓ Light frame walls with shear panels of all other materials	2.5	2.8	NL	NP
✓ Buckling-restrained braced frames (BRBF), non-moment-resisting beam-column connection	7.0	2.8	NL	30
✓ Buckling-restrained braced frames, moment-resisting beam-column connections	8.0	2.8	NL	30
✓ Special steel plate shear walls (SPSW)	7.0	2.8	NL	30

Table 208-11B: Earthquake-Force-Resisting Structural Systems of Steel

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
C. Moment-Resisting Frame Systems				
✓Special moment-resisting frame (SMRF)	8.0	3.0	NL	NL
✓Intermediate steel moment frames (IMF)	4.5	3.0	NL	NP
✓Ordinary moment frames (OMF)	3.5	3.0	NL	NP
✓Special truss moment frames (STMF)	6.5	3.0	NL	NP
✓Special composite steel and concrete moment frames	8.0	3.0	NL	NL
✓Intermediate composite moment frames	5.0	3.0	NL	NP
✓Composite partially restrained moment frames	6.0	3.0	50	NP
✓Ordinary composite moment frames	3.0	3.0	NP	NP

Table 208-11B: Earthquake-Force-Resisting Structural Systems of Steel

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
D. Dual Systems with Special Moment Frames				
✓ Steel eccentrically braced frames	8.0	2.8	NL	NL
✓ Special steel concentrically braced frames	7.0	2.8	NL	NL
✓ Composite steel and concrete eccentrically braced frame	8.0	2.8	NL	NL
✓ Composite steel and concrete concentrically braced frame	6.0	2.8	NL	NL
✓ Composite steel plate shear walls	7.5	2.8	NL	NL
✓ Buckling-restrained braced frame	8.0	2.8	NL	NL
✓ Special steel plate shear walls	8.0	2.8	NL	NL
✓ Masonry shear wall with steel OMRF	4.2	2.8	NL	50
✓ Steel EBF with steel SMRF	8.5	2.8	NL	NL
✓ Steel EBF with steel OMRF	4.2	2.8	NL	50
✓ Special concentrically braced frames with steel SMRF	7.5	2.8	NL	NL
✓ Special concentrically braced frames with steel OMRF	4.2	2.8	NL	50

Table 208-11B: Earthquake-Force-Resisting Structural Systems of Steel

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
E. Dual Systems with Intermediate Moment Frames				
✓ Special steel concentrically braced frame	6.0	2.8	NL	NP
✓ Composite steel and concrete concentrically braced frame	5.5	2.8	NL	NP
✓ Ordinary composite braced frame	3.5	2.8	NL	NP
✓ Ordinary composite reinforced concrete shear walls with steel elements	5.0	3.0	NL	NP
F. Cantilevered Column Building Systems				
✓ Special steel moment frames	2.2	2.0	10	10
✓ Intermediate steel moment frames	1.2	2.0	10	NP
✓ Ordinary steel moment frames	1.0	2.0	10	NP
✓ Cantilevered column elements	2.2	2.0	NL	10
G. Steel systems not specifically detailed for seismic resistance, excluding cantilever systems	3.0	3.0	NL	NP

Table 208-11C: Earthquake-Force-Resisting Structural Systems of Masonry

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
✓ Masonry shear walls	4.5	2.8	NL	50
B. Building Frame Systems				
✓ Masonry shear walls	5.5	2.8	NL	50
C. Moment-Resisting Frame Systems				
✓ Masonry moment-resisting wall frames (MMRWF)	6.5	2.8	NL	50
D. Dual Systems				
✓ Masonry shear walls with SMRF	5.5	2.8	NL	50
✓ Masonry shear walls with steel OMRF	4.2	2.8	NL	50
✓ Masonry shear walls with concrete IMRF	4.2	2.8	NL	NP
✓ Masonry shear walls with masonry MMRWF	6.0	2.8	NL	50

Table 208-11D: Earthquake-Force-Resisting Structural Systems of Wood

Basic Seismic-Force Resisting System	R	Ω_o	System Limitation and Building Height Limitation by Seismic Zone, m	
			Zone 2	Zone 4
A. Bearing Wall Systems				
✓ Light-framed walls with shear panels: wood structural panel walls for structures three stories or less	5.5	2.8	NL	20
✓ Heavy timber braced frames where bracing carries gravity load	2.8	2.2	NL	20
✓ All other light framed walls	NA	NA		
B. Building Frame Systems				
✓ Ordinary heavy timber-braced frames	5.6	2.2	NL	20

208.5.2.2. Site Seismic Hazard Characteristics

The value of T shall be determined from one of the following methods:

1. Method A:

For all buildings, the value T may be approximated from the following equation:

$$T = C_t (h_n)^{\frac{3}{4}} \qquad Eq.\ 208-12$$

Where:

C_t = 0.0853 for steel moment-resisting frames

C_t = 0.0731 for reinforced concrete moment-resisting frames and eccentrically braced frames

C_t = 0.0488 for all other buildings

208.5.2.2. Site Seismic Hazard Characteristics

The value of T shall be determined from one of the following methods:

1. Method A:

For all buildings, the value T may be approximated from the following equation:

$$T = C_t (h_n)^{\frac{3}{4}} \qquad Eq.\ 208-12$$

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.0743/\sqrt{A_c}$

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

$$A_c = \sum A_e \left[0.2 + \left(\frac{D_e}{h_n} \right)^2 \right] \qquad Eq.\ 208-13$$

The value of D_e/h_n used in Equation 208-13 shall not exceed 0.9.

208.5.2.2. Site Seismic Hazard Characteristics

The value of T shall be determined from one of the following methods:

2. Method B:

The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 208.6.2. The value of T from Method B shall not exceed a value 30 percent greater than the value of T obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zone 2.

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

$$T = 2\pi \sqrt{\frac{\left(\sum_{i=1}^{n} w_i \delta_i^{\,2}\right)}{g\left(\sum_{i=1}^{n} wf_i \delta_i\right)}} \qquad Eq.\ 208-14$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

208.5.2.2. Site Seismic Hazard Characteristics

The value of T shall be determined from one of the following methods:

2. Method B:

The vaues of f_t represent any lateral force distributed approximately in accordance with the principles of Equations 208-15, 208-16 and 208-17 or any other rational distribution. The elastic deflections, $\delta_{i,}$ shall be calculated using the applied lateral forces, f_t.

208.5.2.3. Vertical Distribution of Force

The total force shall be distributed over the height of the structure in conformance with Equations 208-15, 208-16 and 208-17 in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_t \qquad Eq.\ 208-15$$

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

$$F_t = 0.07TV \qquad Eq.\ 208-16$$

208.5.2.3. Vertical Distribution of Force

The value of T used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Equation 208-4. F_t need not exceed 0.25V and may be considered as zero where T is 0.7s or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n, according to the following equation:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad Eq.\ 208-17$$

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

INTRODUCTION TO EARTHQUAKE ENGINEERING

Sample Problems for Base Shear Computation using the NSCP Code

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 1

Determine the design base shear for a five-storey concrete special moment resisting frame building. The following information is given:

Zone 4, $Z = 0.4$

Seismic Source Type A

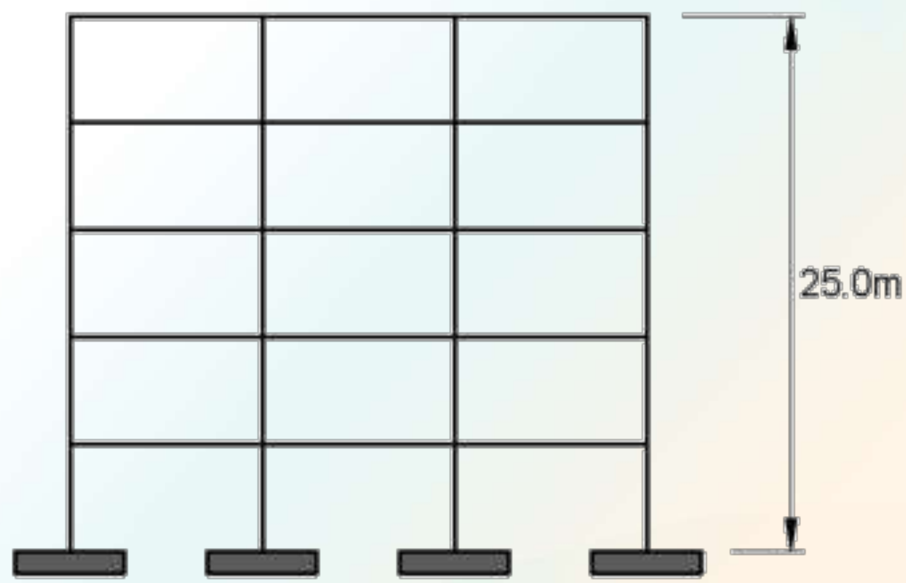
Distance to Seismic Source = 10km

Soil Profile Type = S_c

$I = 1.0$

$R = 8.5$

$W = 7300$ kN



Problem 1 - Solution

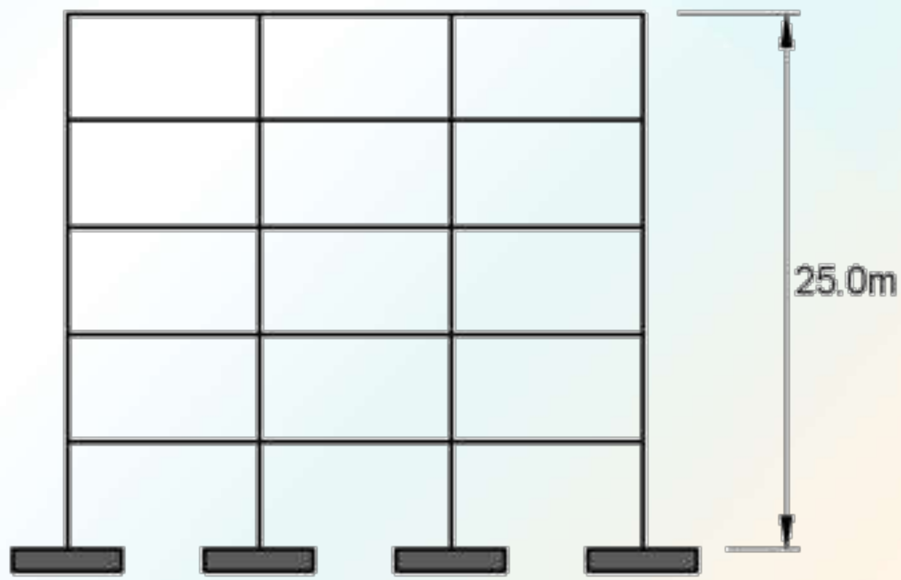
A. Solve for the structure period

For concrete moment-resisting frames, C_t = 0.0731

$$T = C_t (h_n)^{\frac{3}{4}}$$

$$T = 0.0731(25)^{\frac{3}{4}}$$

$$T = 0.817 \sec$$



- *$T > 0.7sec$, therefore, $F_t \neq 0$*

Problem 1 - Solution

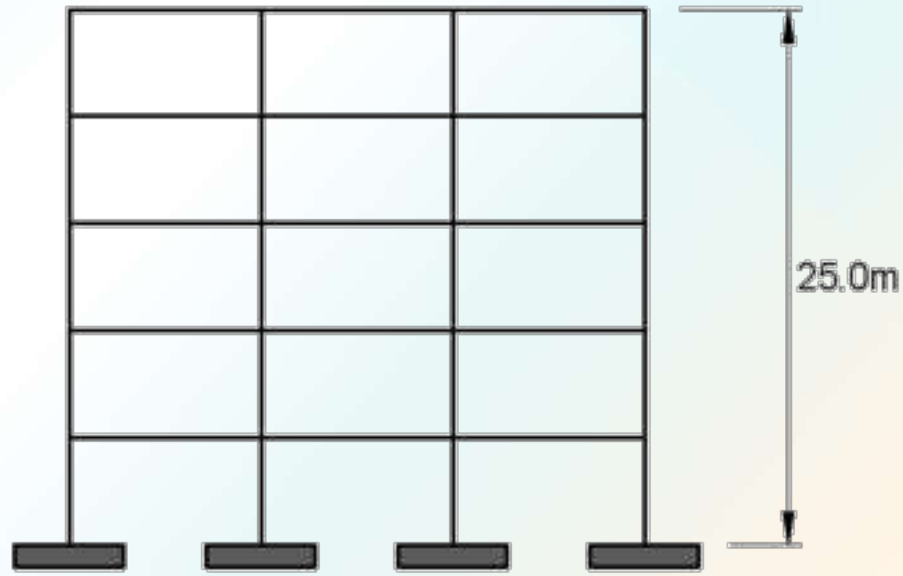
B. Find the near source factors N_a and N_v

Table 208-5: Near-Source Factor, N_a

Seismic Source Type	Closest Distance to Known Seismic Source[2]		
	< 2km	$\leq$ 5km	$\geq$ 10km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

Table 208-6: Near-Source Factor, N_v

Seismic Source Type	Closest Distance to Known Seismic Source[2]			
	< 2km	5km	10km	> 10km
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



$N_a = 1.0$

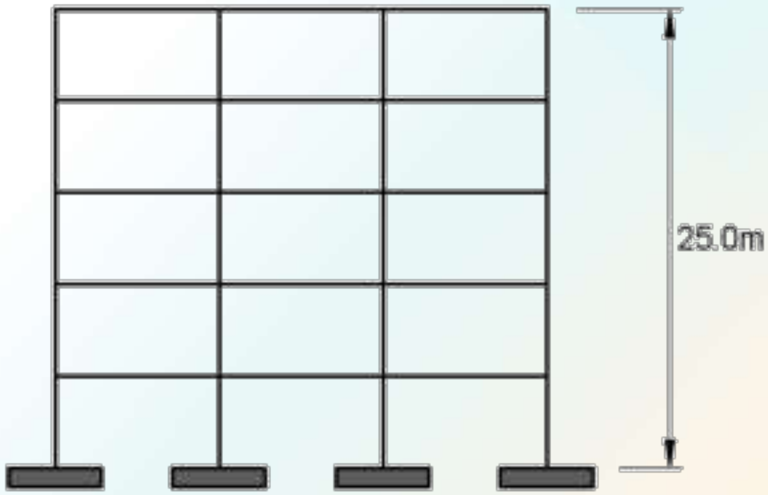
$N_v = 1.2$

Problem 1 - Solution

C. Determine the Seismic Coefficients C_a and C_v

Table 208-7: Seismic Coefficient, C_a

Soil Profile Type	Seismic Zone (Z)	
	Z = 0.2	$Z = 0.4$
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	See Footnote 1 of Table 208-8	



$$C_a = 0.40N_a$$

$$C_a = 0.40(1.0) = 0.40$$

$N_a = 1.0$

$N_v = 1.2$

FEU ALABANG FEU DILIMAN FEU TECH

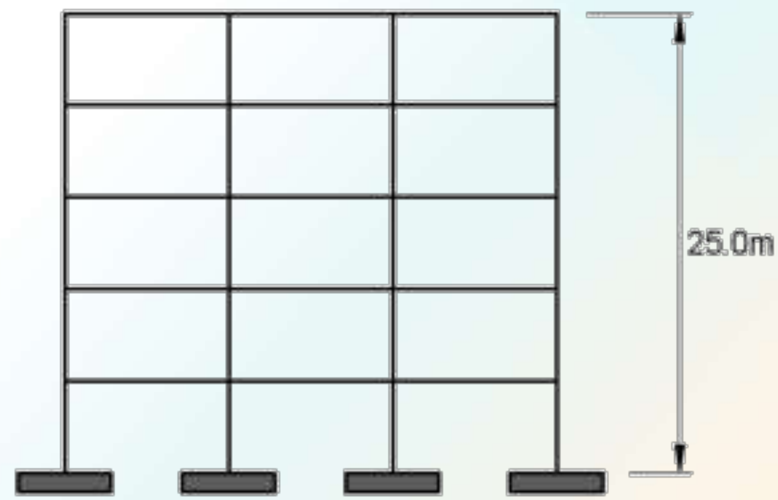
Technology Driven by Innovation

Problem 1 - Solution

C. Determine the Seismic Coefficients C_a and C_v

Table 208-8: Seismic Coefficient, C_v

Soil Profile Type	Seismic Zone (Z)	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_v$
S_B	0.20	$0.40N_v$
S_C	0.32	$0.56N_v$
S_D	0.40	$0.64N_v$
S_E	0.64	$0.96N_v$
S_F	See Footnote 1 of Table 208-8	



$$C_v = 0.56N_v$$

$$C_v = 0.56(1.2) = 0.672$$

$N_a = 1.0$

$N_v = 1.2$

Problem 1 - Solution

D. Determine the Base Shear

The total design base shear in a given direction is:

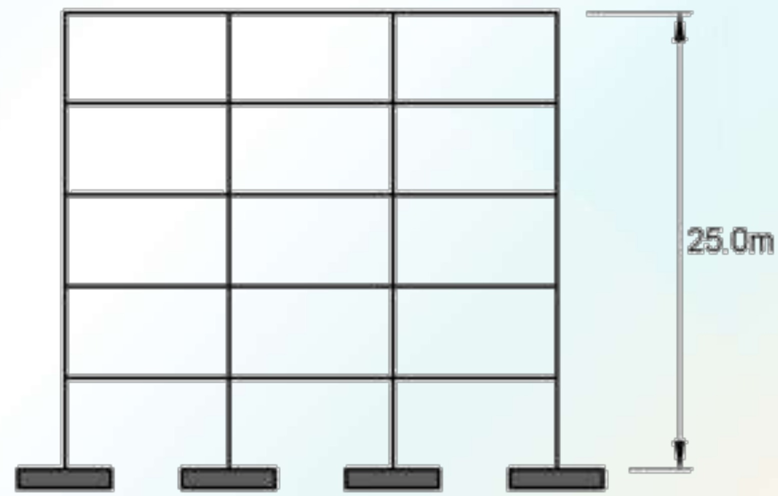
$$V = \frac{C_v I}{RT} W \qquad V = \frac{(0.672)(1.0)}{(8.5)(0.817)}(7300)$$

$$V = 706.401 kN$$

But the code indicates that the total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \qquad V = \frac{2.5(0.40)(1.0)}{(8.5)}(7300)$$

$$V = 858.824 kN$$



Values:

$C_a = 0.40$

$I = 1.0$

$R = 8.5$

$W = 7300kN$

Problem 1 - Solution

D. Determine the Base Shear

And that the base shear shall not be less than the following:

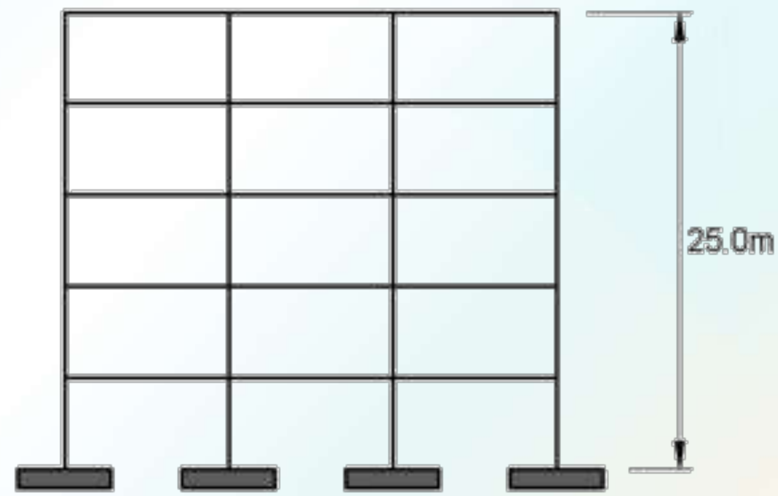
$$V = 0.11 C_a I W \qquad V = 0.11(0.40)(1.0)(7300)$$

$$V = 321.2 kN$$

And in Seismic 4, the total design base shear shall also be not less than:

$$V = \frac{0.8 Z N_v I}{R} W \qquad V = \frac{0.8(0.4)(1.20)(1.0)}{(8.5)}(7300)$$

$$V = 329.788 kN$$



Values:

N_v = 1.20

I = 1.0

R = 8.5

W = 7300kN

Z = 0.4

Problem 1 - Solution

D. Determine the Base Shear

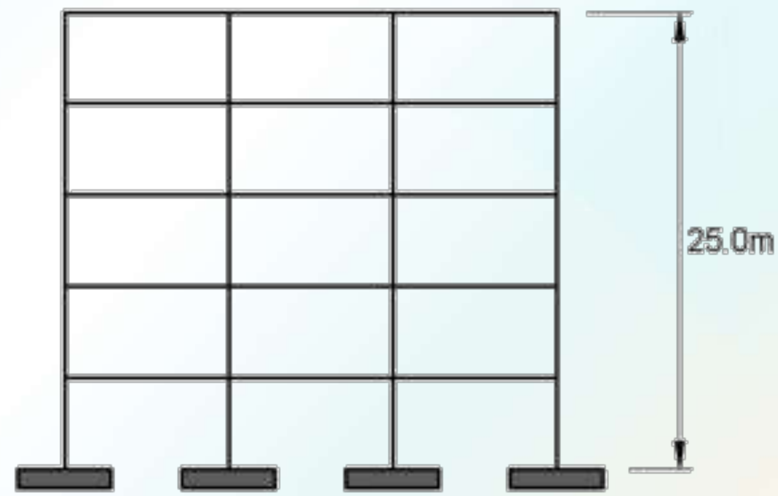
Therefore, the governing design base shear for this example is:

$$V = 706.401kN$$

$$V = 858.824kN$$

$$V = 321.2kN$$

$$V = 329.788kN$$



Values:

$N_v = 1.20$

$I = 1.0$

$R = 8.5$

$W = 7300kN$

$Z = 0.4$

Problem 2

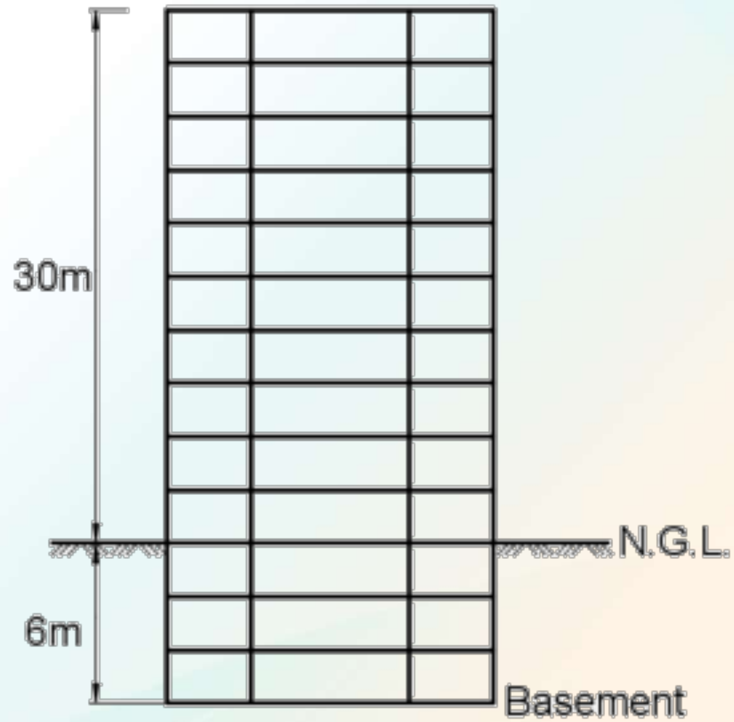
Determine the period for each of the structure shown below using Method A. Method A period calculations involves the following expression:

$$T = C_t \left(h_n\right)^{\frac{3}{4}}$$

1. Steel special moment-resisting frame structure

The height of the structure above its base is 30m with a basement 6m from the ground level.

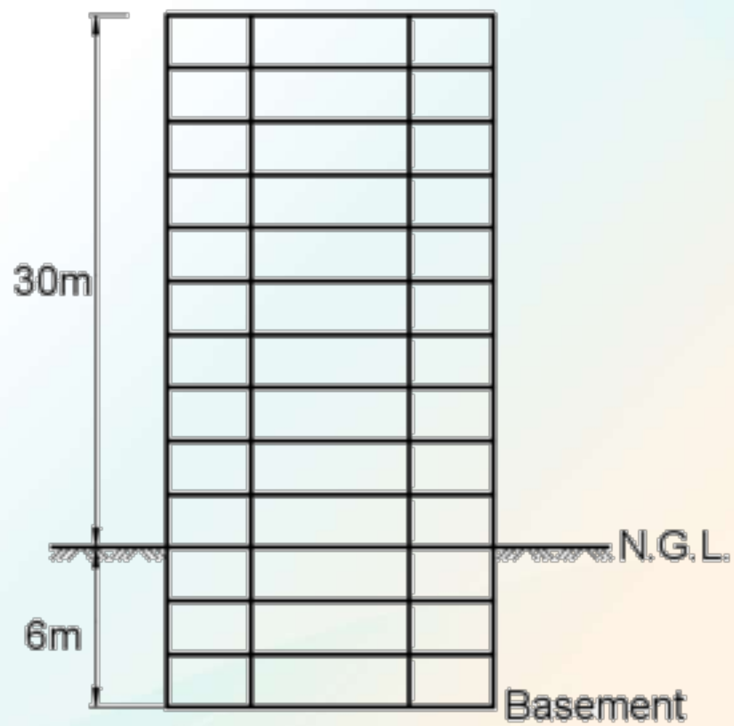
The height below the ground will not be included in determining h_n for calculating the period T.



Problem 2

Determine the period for each of the structure shown below using Method A. Method A period calculations involves the following expression:

$$T = C_t \left(h_n\right)^{\frac{3}{4}}$$



1. Steel special moment-resisting frame structure

Use C_t = 0.0853 for steel SMRF

$$T = 0.0853\left(30\right)^{\frac{3}{4}}$$

$$T = 1.09\,\text{sec}$$

Problem 2

Determine the period for each of the structure shown below using Method A. Method A period calculations involves the following expression:

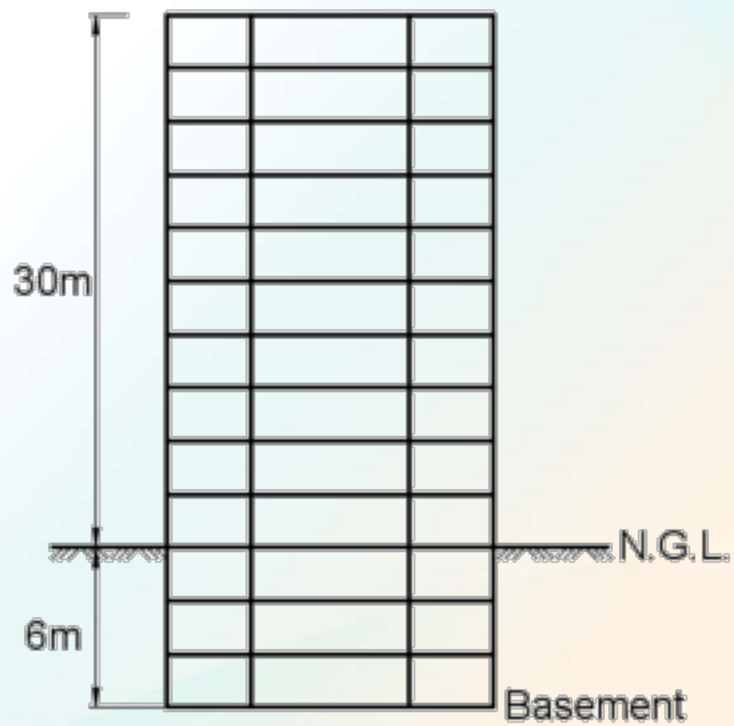
$$T = C_t \left(h_n\right)^{\frac{3}{4}}$$

2. Concrete special moment-resisting frame structure

The height of the tallest part of the building is 12meters. Roof Penthouses are generally not considered in determining h_n for period calculations, but heights of setbacks are included.

Use C_t = 0.0731 for concrete SMRF

$$T = 0.0853(12)^{\frac{3}{4}} = 0.471\text{sec}$$



FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 3

Determine the base shear and the design lateral forces for a two-storey reinforced SMRF office building using the simplified design base shear given the following information:

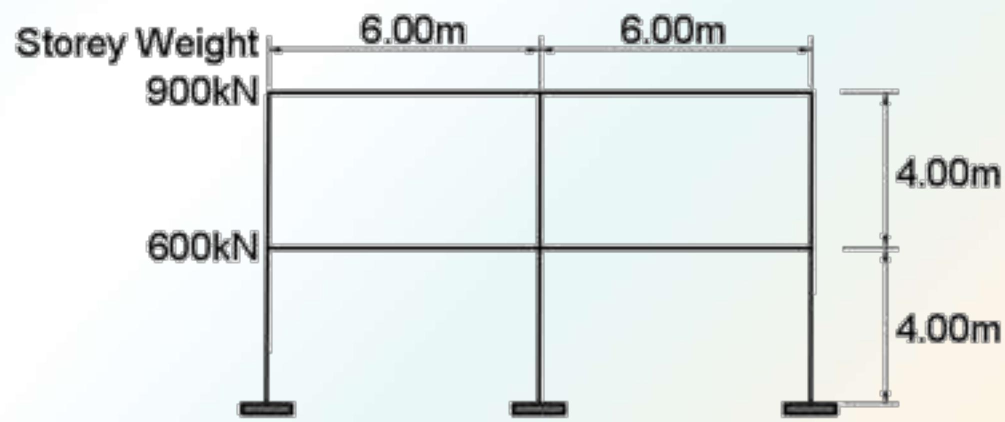
Zone 4, Z = 0.4

Seismic Source Type C

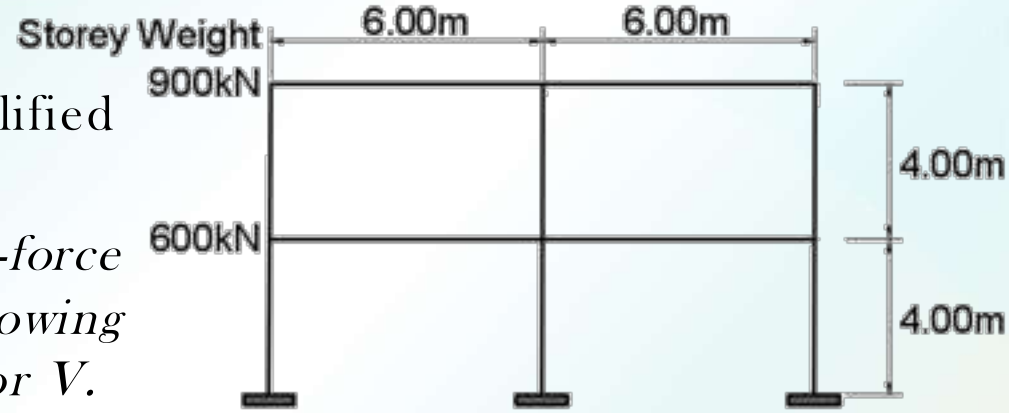
Soil Profile Type = unknown

R = 8.5

W = 1500 kN



Problem 3 - Solutions



1. Check the applicability of the simplified method:

The simplified static lateral-force procedure may be used for the following structures of Occupancy Category IV or V.

a. Buildings of any occupancy (including single-family dwellings) not more than three storeys in height excluding basements, that use light frame construction.

b. Other buildings not more than two stories in height excluding basements.

❖ *Since our building is covered by 1.b., we can use the simplified method.*

Problem 3 - Solutions

2. Determine Base Shear

Since soil properties are not known, the suggested soil profile type S_D shall be used per NSCP Section 208.4.3

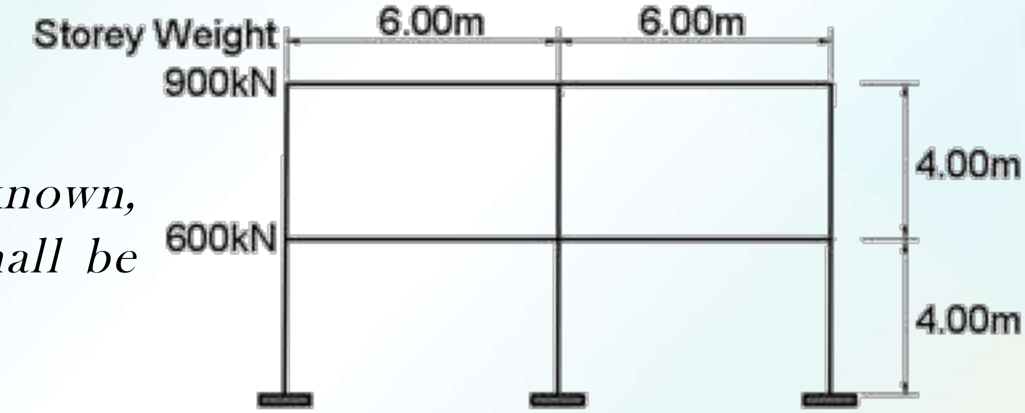


Table 208-5: Near-Source Factor, N_a

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

$$N_a = 1.0$$

Problem 3 - Solutions

2. Determine Base Shear

Since soil properties are not known, the suggested soil profile type S_D shall be used per NSCP Section 208.4.3

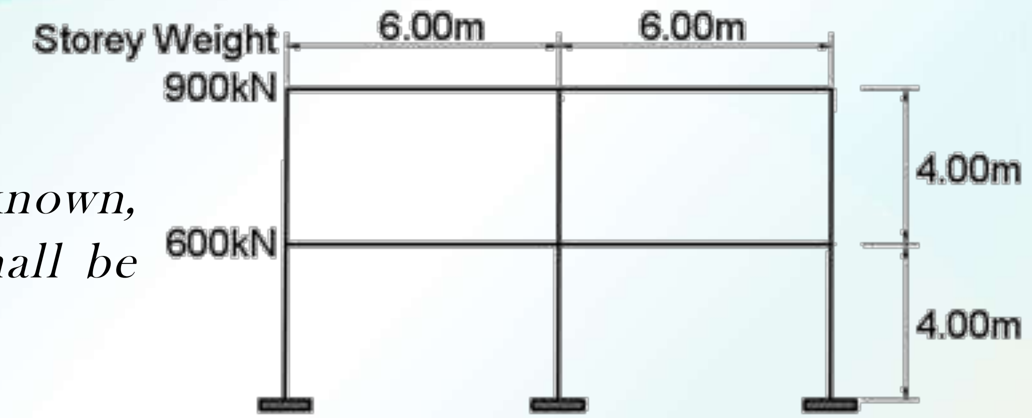


Table 208-7: Seismic Coefficient, C_a

Soil Profile Type	Seismic Zone (Z)	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	See Footnote 1 of Table 208-8	

$$N_a = 1.0$$

$$C_a = 0.44N_a = 0.44(1.0)$$

$$C_a = 0.44$$

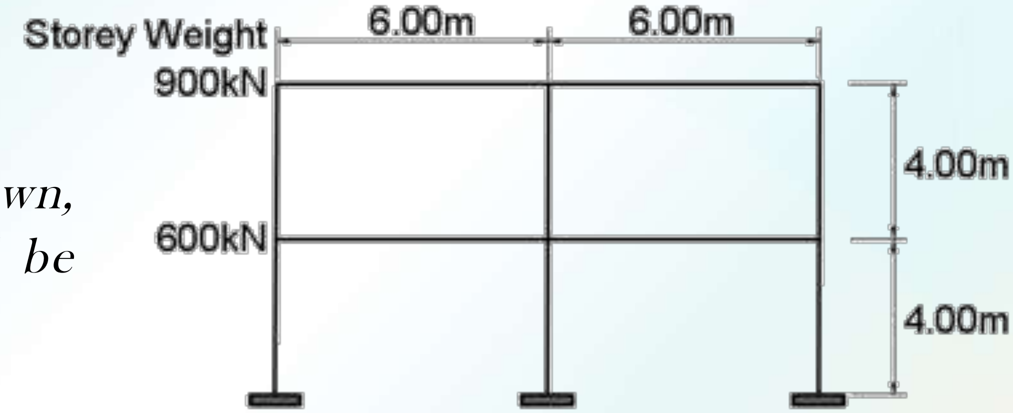
FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 3 - Solutions

2. Determine Base Shear

Since soil properties are not known, the suggested soil profile type S_D shall be used per NSCP Section 208.4.3



$$V = \frac{3.0C_a}{R}W$$

$$V = \frac{3.0(0.44)}{8.5}(1500)$$

$$V = 232.941kN$$

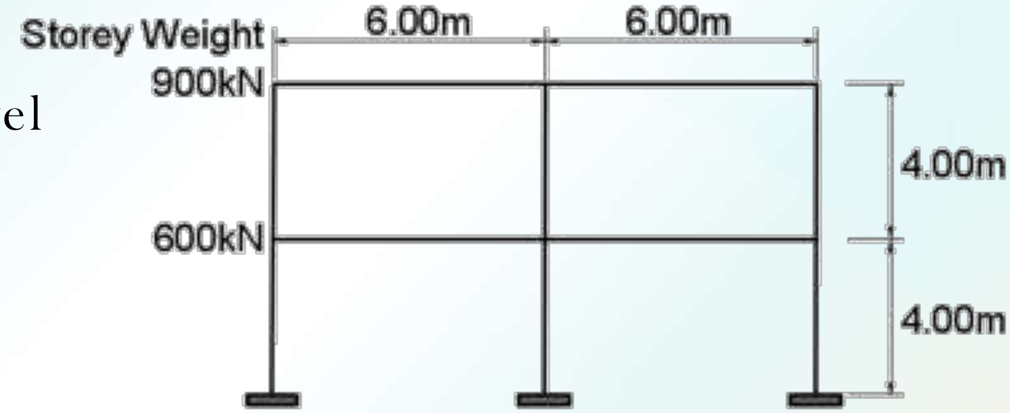
$N_a = 1.0$

$C_a = 0.44N_a = 0.44(1.0)$

$C_a = 0.44$

Problem 3 - Solutions

3. Determine the lateral force at each level



$$F_x = \frac{3.0C_a}{R} W_x$$

$$F_1 = \frac{3.0(0.44)}{8.5}(600)$$

$$F_1 = 93.176kN$$

$$F_2 = \frac{3.0(0.44)}{8.5}(900)$$

$$F_2 = 139.765kN$$

$N_a = 1.0$

$C_a = 0.44N_a = 0.44(1.0)$

$C_a = 0.44$

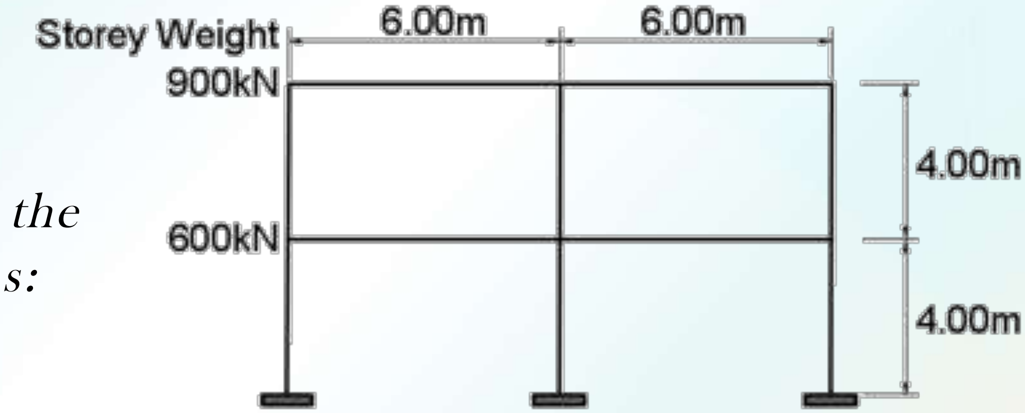
FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 3 - Solutions

Commentary:

Computing the design base shear using the standard method will yield smaller values:



$$V = \frac{2.5C_a I}{R}W$$

$$V = \frac{2.5(0.44)(1.0)}{8.5}(1500)$$

$$V = 194.118kN$$

$N_a = 1.0$

$C_a = 0.44N_a = 0.44(1.0)$

$C_a = 0.44$

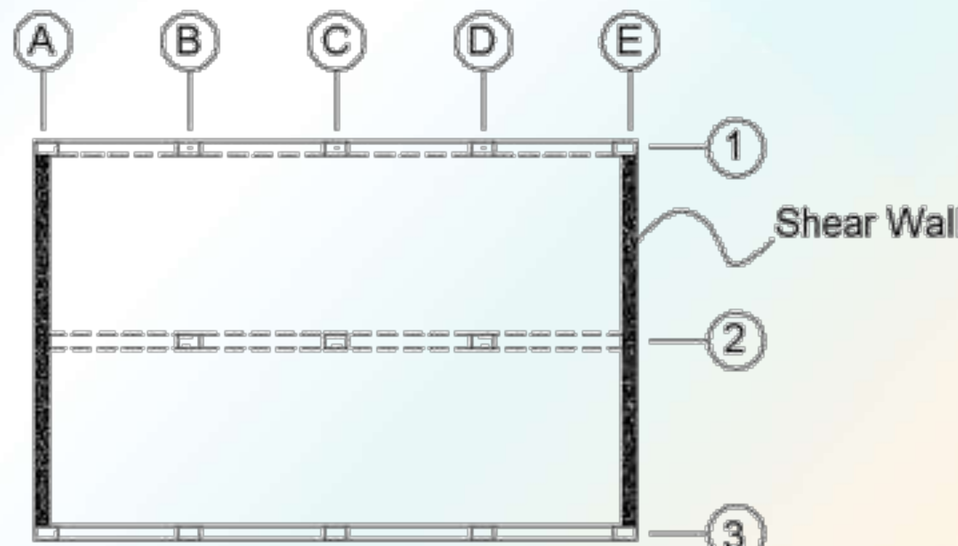
It is noticeable that from this example, the design base shear value using the simplified method is approximately 20% higher than that using the standard method.

$$\frac{232.941}{194.118} = 1.20$$

Problem 4 (Combination of Structural Systems: Along Different Axes)

This example illustrates determination of R values for a building that has different structural systems along different axes (i.e., directions) of the building.

In this example, a 3-storey building has concrete shear walls in one direction and concrete moment frames in the other. Floors are concrete slab, and the building is located in Zone 4. Determine the R value for each direction.



Typical Floor Plan

Lines A and E are reinforced concrete bearing walls: R = 4.5

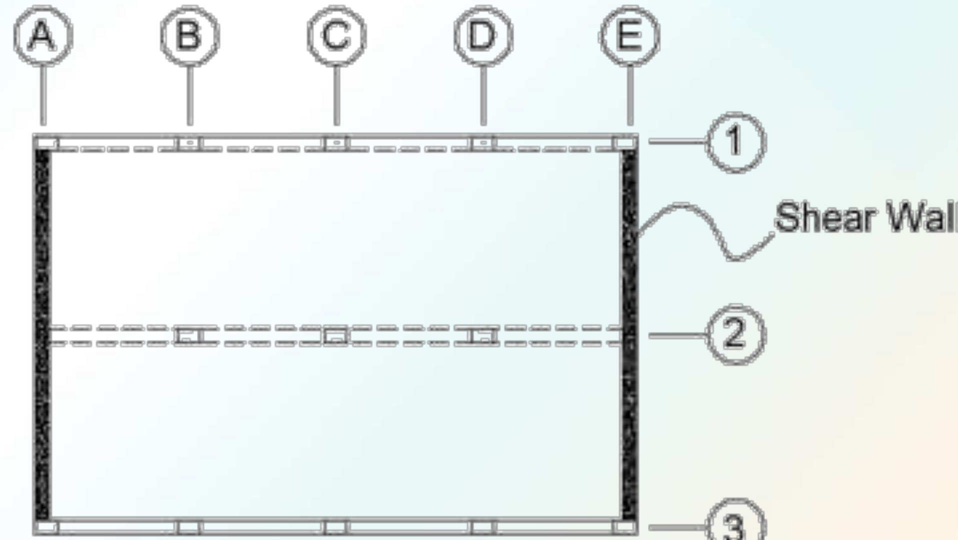
Lines 1,2 and 3 are concrete special moment-resisting frames: R=8.5

Problem 4 - Discussion

1. Determine the R value for each direction.

In Zone 4, the provisions of NSCP Section 208.5.4.2 require that when a structure has bearing walls in one direction, the R value used for the orthogonal direction cannot be greater than that for the bearing wall system.

- *Use R = 4.5 in both directions.*



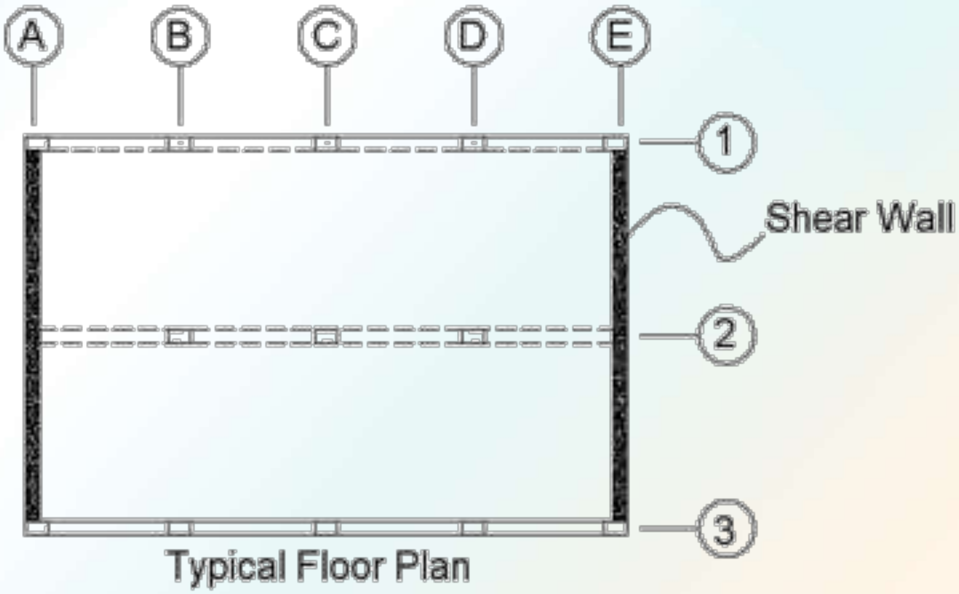
Typical Floor Plan

Problem 4 - Discussion

Commentary:

The reason for this orthogonal system requirement is to provide sufficient strength and stiffness to limit the amount of out-of-plane deformation of the bearing wall system. A more direct approach would be to design the orthogonal system such that the Δ_M value is below that would result in the loss of bearing wall capacity.

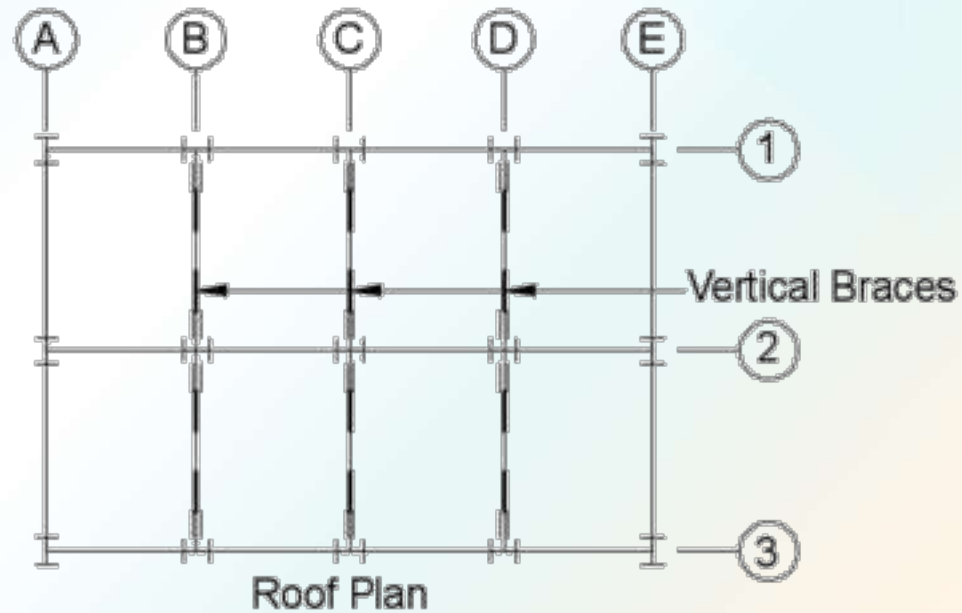
The design loads for the special moment-resisting frames are calculated using $R=4.5$. However, the frame details must comply with the requirements for the $R = 8.5$ system.



Problem 5 (Combination of Structural Systems: Along Different Axes)

Occasionally, it is necessary to have different structural systems in the same direction. This example shows how the R value is determined in such a situation.

One-storey steel frame structure has the roof plan shown below. The structure is located in Zone 4. Determine the R value for the N/S direction.



Lines A and E are steel ordinary moment-resisting frames: R = 4.5

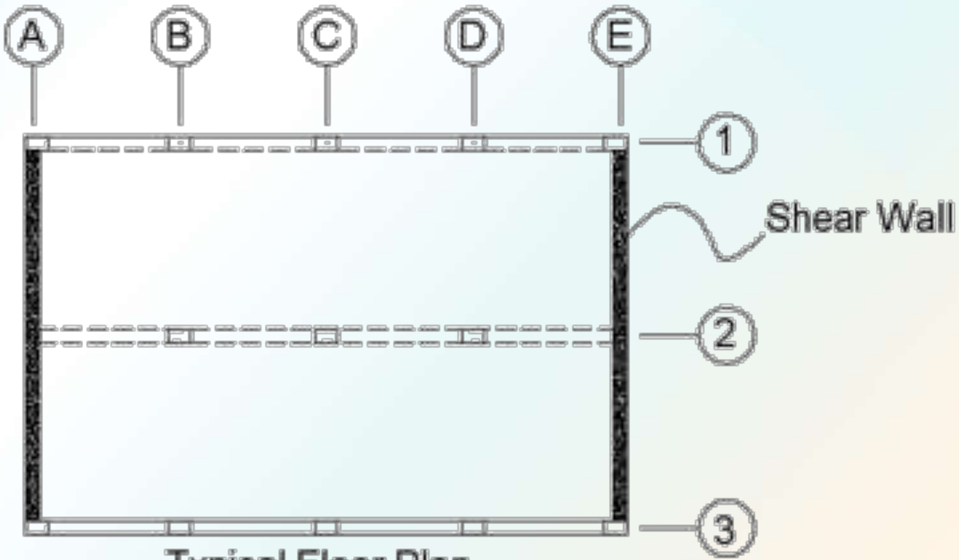
Lines B,C and D are steel ordinary braced frames: R = 5.6

Problem 5 - Discussion

1. Determine the R value for N/S Direction

When a combination of structural systems is used in the same direction, NSCP Section 208.5.4.3 requires that the value of R used be not greater than the least value of the system utilized.

❖ *Use R = 4.5 for the entire structure.*



Typical Floor Plan

Problem 6 (Vertical Distribution of Force)

A 10 Storey building has a moment resisting steel frame for a lateral force-resisting system. Find the vertical distribution of lateral forces F_x. The following information is given:

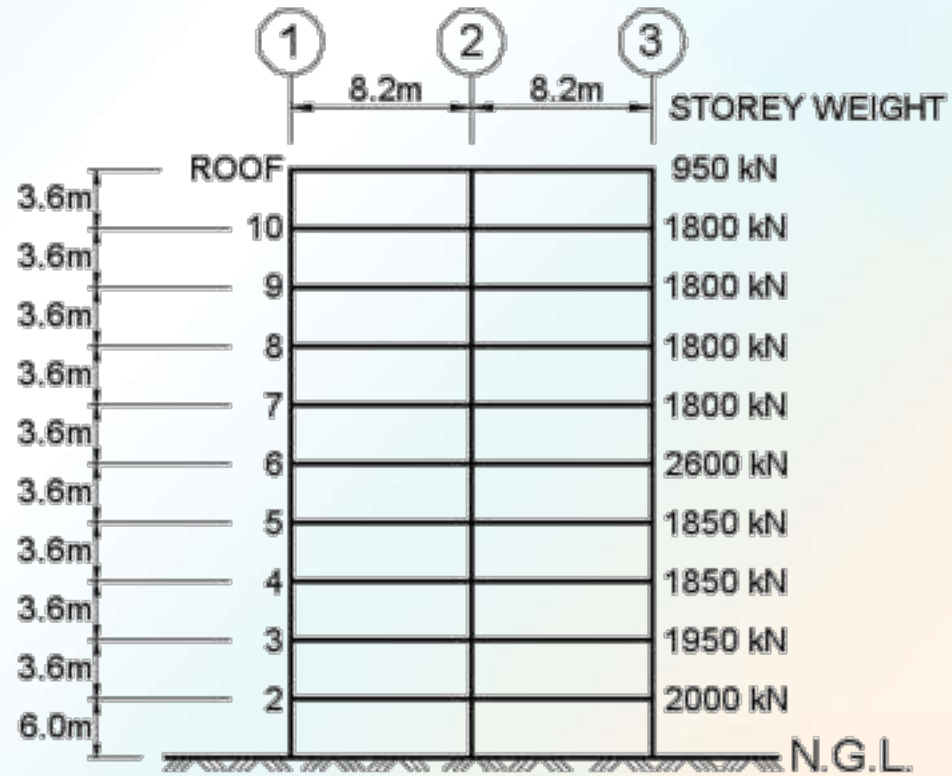
Zone 4

$W = 18,400$ kN

$C_v = 0.56$ $R = 8.5$ $I = 1.0$

$T = 1.32$ sec $V = 918.4$ kN

1. Determine F_t
2. Find F_x at each level.



FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 6 (Vertical Distribution of Force)

1. Determine F_t

This is the concentrated force applied at the top of the structure. It is determined as follows. First, check that F_t is not zero.

$$T = 1.32\,\text{sec} > 0.7\,\text{sec}$$

$$\therefore F_t \neq 0$$

$$F_t = 0.07TV \qquad F_t = 0.07(1.32)(918.4)$$

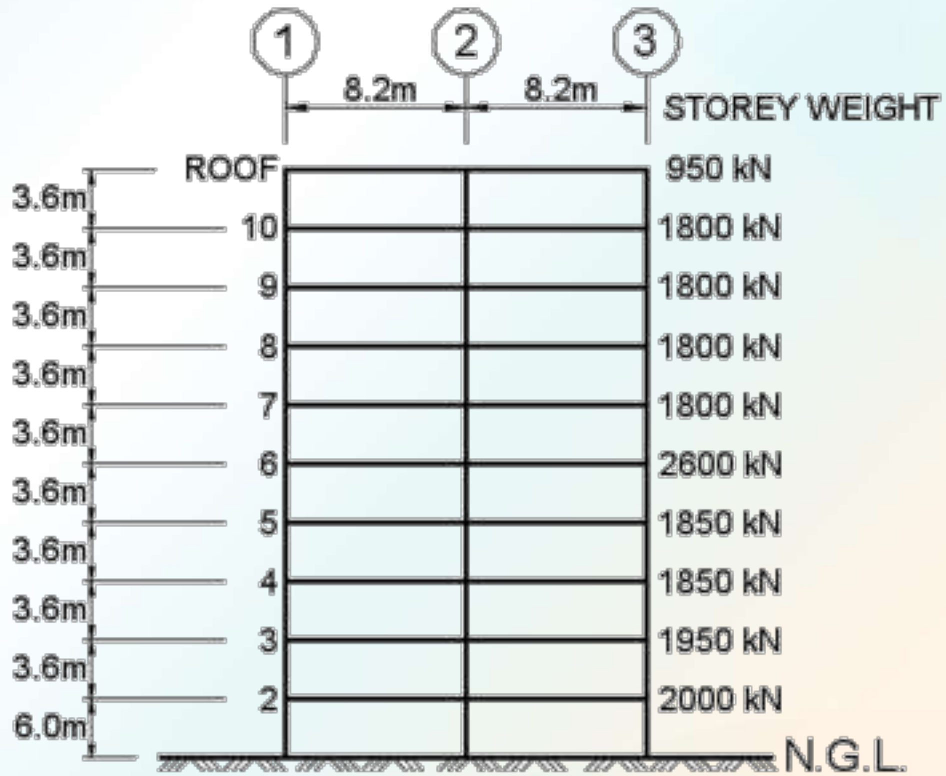
$$F_t = 84.86kN$$

2. Check if F_t is less than $0.25V$

$$0.25V = 0.25(918.4)$$

$$0.25V = 229.6$$

$$\therefore F_t < 0.25V$$ ❖ *OK!*

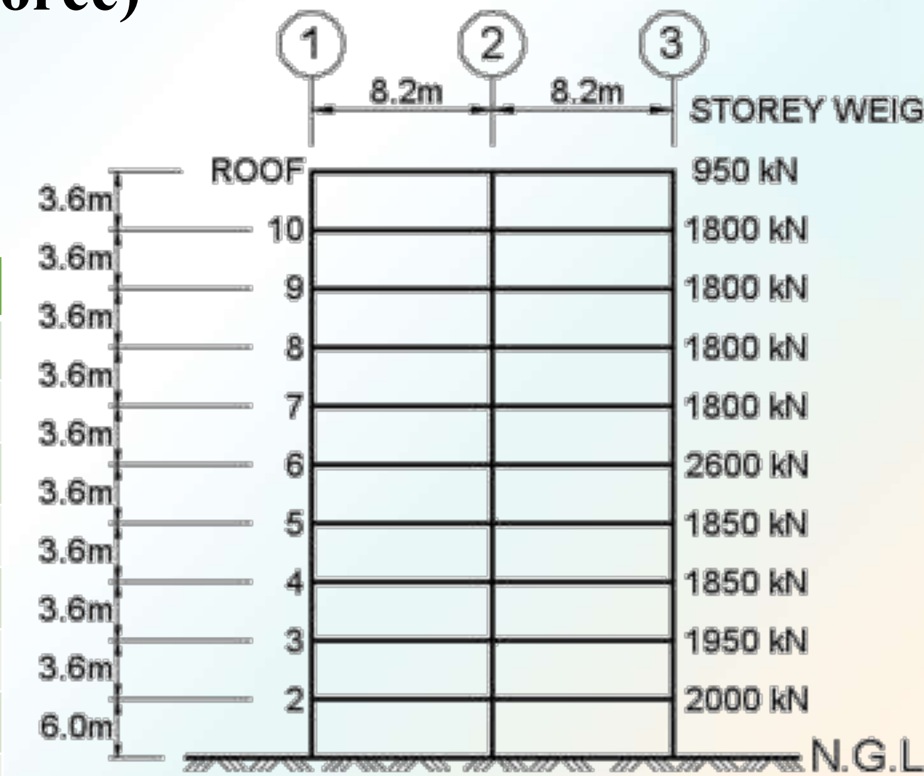


Problem 6 (Vertical Distribution of Force)

3. Find F_x at each level.

The vertical distribution of seismic forces is determined from NSCP Equation 208-15.

Level x	H_x	w_x	$w_x h_x$	F_x
Roof	38.4	950	36,480	$78.49 + 84.86 = 163.35$
10	34.8	1,800	62,640	134.77
9	31.2	1,800	56,160	120.83
8	27.6	1,800	49,680	106.89
7	24.0	1,800	43,200	92.95
6	20.4	2,600	53,040	114.12
5	16.8	1,850	31,080	66.87
4	13.2	1,850	24,420	52.54
3	9.6	1,950	18,720	40.28
2	6.0	2,000	12,000	25.82
			$\sum = 387{,}420$	$\sum = 918.4$



$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad F_t = 84.86kN$$

$$V = 918.4kN$$

Q&A SESSION

ASK ANY QUESTION RELATED TO OUR TOPIC FOR TODAY.

HOMEWORK

Do the Exercises about Base Shear Computations

Reference:

Association of Structural Engineers of the Philippines (2016). *National Structural Code of the Philippines 2015 (7th Edition) Volume 1 Buildings, Towers and Other Vertical Structures.*

Association of Structural Engineers of the Philippines (2003). *ASEP Earthquake Design Manual 2003 Volume 1: Code Provisions for Lateral Forces.*

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

CEELECT1
Earthquake Engineering
Center of Mass, Center of Rigidity and Eccentricities

Module 3

OBJECTIVES

- At the end of the chapter, the learner should be able to:
  - *Analyze structural plans of reinforced concrete and steel structures to locate its center of mass.*
  - *Determine the seismic dead load of the structure.*
  - *Choose for the governing value of the eccentricity*

Module 3

OBJECTIVES

■At the end of the chapter, the learner should be able to:

- *Center of Mass, Center of Rigidity and Eccentricities*
- *Determine the factor "F" for the lateral forces per frame.*

CENTER OF MASS, CENTER OF RIGIDITY AND ECCENTRICITIES

Computation of Dead Load “W”

Dead Load Computations for a Steel Structure

Example 1:

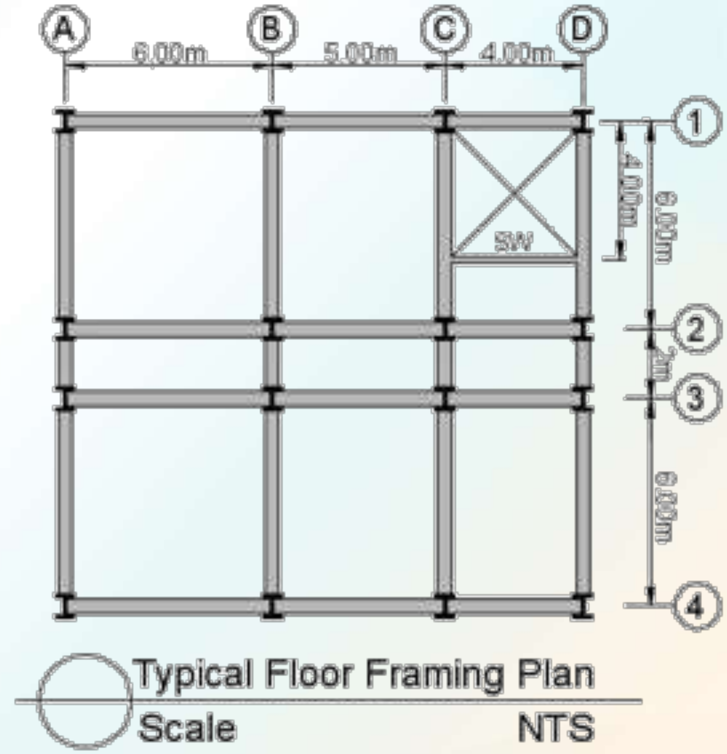
A sample set of plans were presented in the figure. Using the following minimum design loads from the NSCP:

A. Dead Load (DL)

a. 0.125m THK slab = 3.00 kPa
b. 0.050m THK Lean Concrete = 1.20 kPa
c. Marble = 1.58 kPa
d. Gypsum Board = 0.04 kPa
e. Suspended Steel Chanel = 0.10 kPa
f. Beam/Column = 1.00 kPa

Total = 6.92 kPa (without Movable Partition)
+1.00 kPa

7.92 kPa (with Movable Partition)

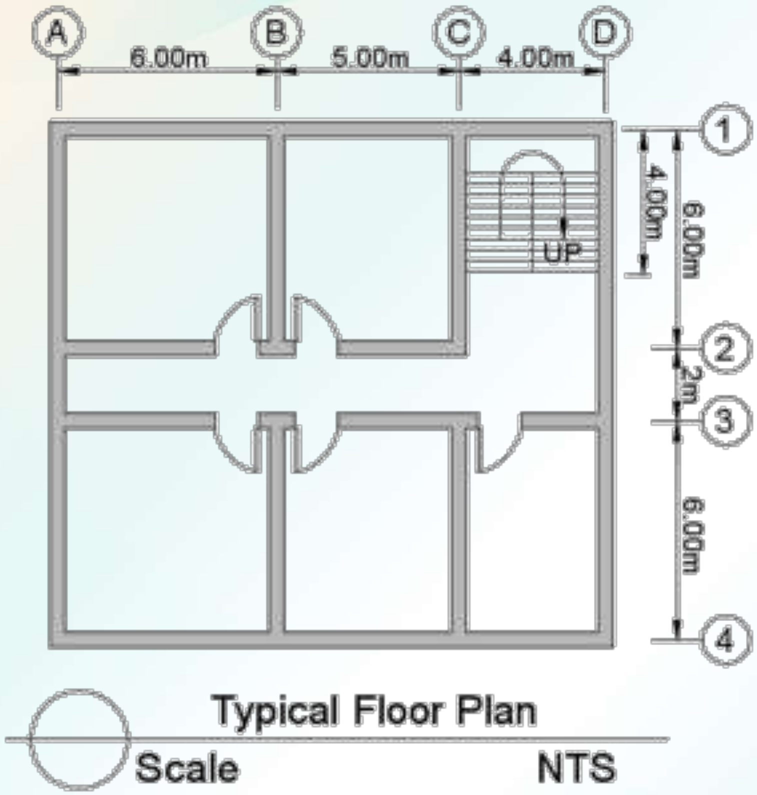


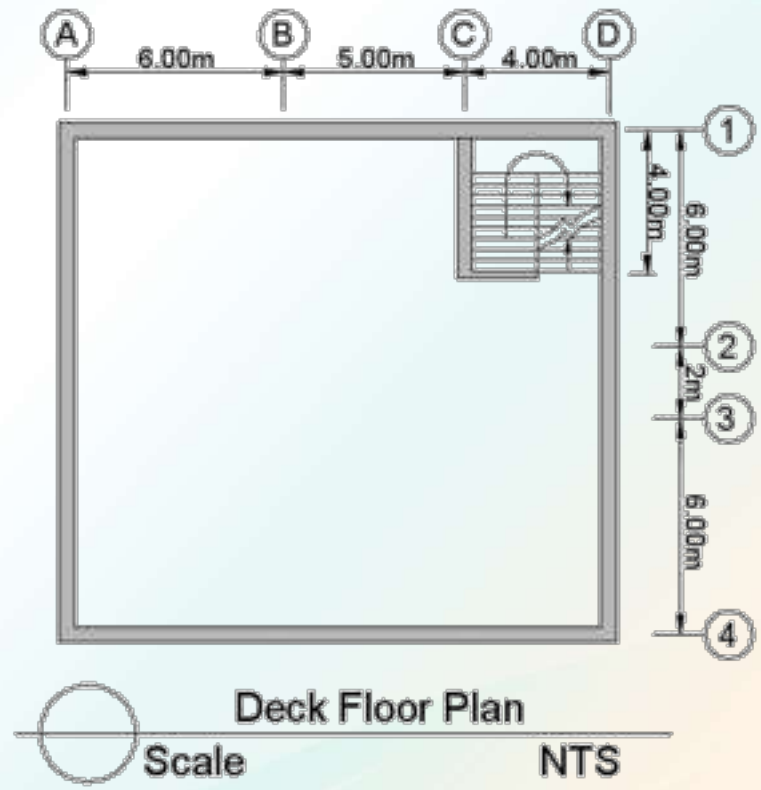
$W_{walls/parapet}$ = 2.70 kPa

Height of Walls & Parapet = 3.6m/1.5m

Dead Load Computations for a Steel Structure

Example 1:





Dead Load Computations for a Steel Structure

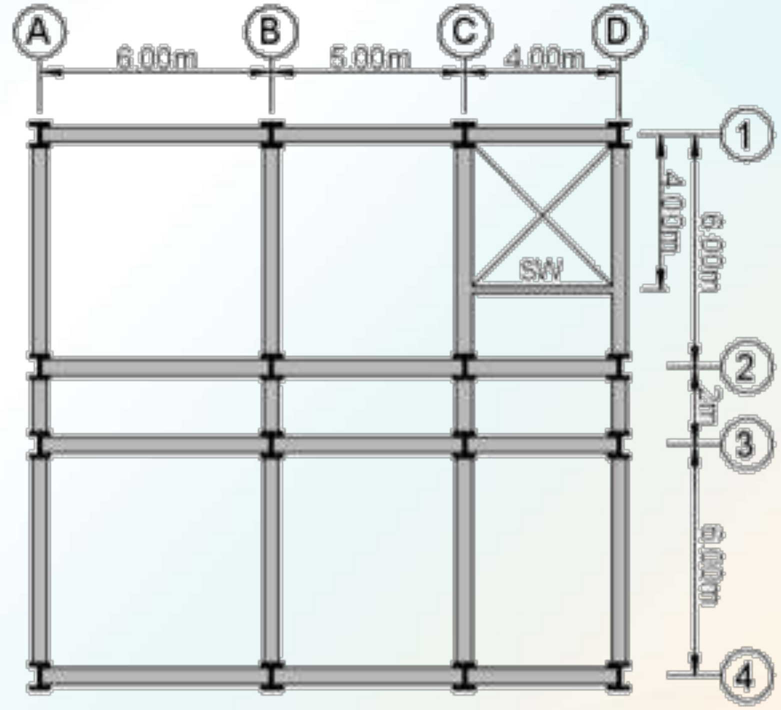
Example 1:

A. Solve for the Tributary Area

$$T.A. = 15(14) = 210m^2$$

B. Solve for the W_{DL}

$$W_{DL} = 6.92(210) = 1453.20kN$$



Typical Floor Framing Plan

Scale NTS

Dead Load Computations for a Steel Structure

Example 1:

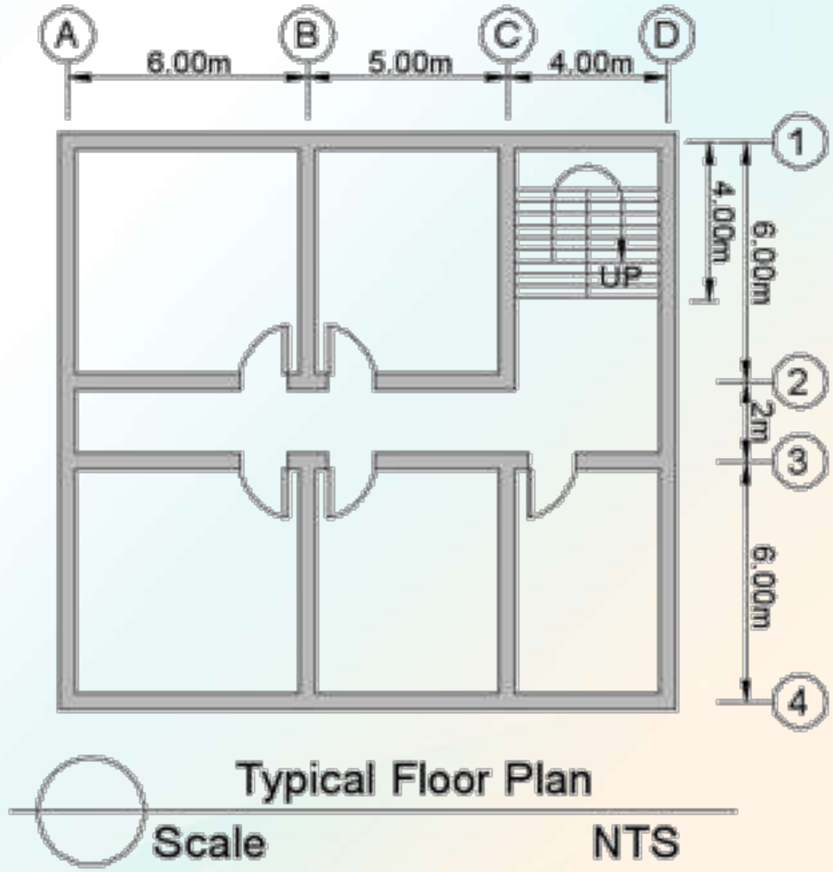
C. Solve for the Perimeter of the Walls (Typical Levels)

$$P = \begin{bmatrix} 15+11+15+15 \\ +14+12+12+14 \end{bmatrix}$$

$$P = 108m$$

D. Solve for the Weight of the Walls (Typical Levels)

$$W_{wall_{typical}} = 2.7(3.6)(108) = 1049.76kN$$



Dead Load Computations for a Steel Structure

Example 1:

C. Solve for the Weight of the Walls (Below Deck Level)

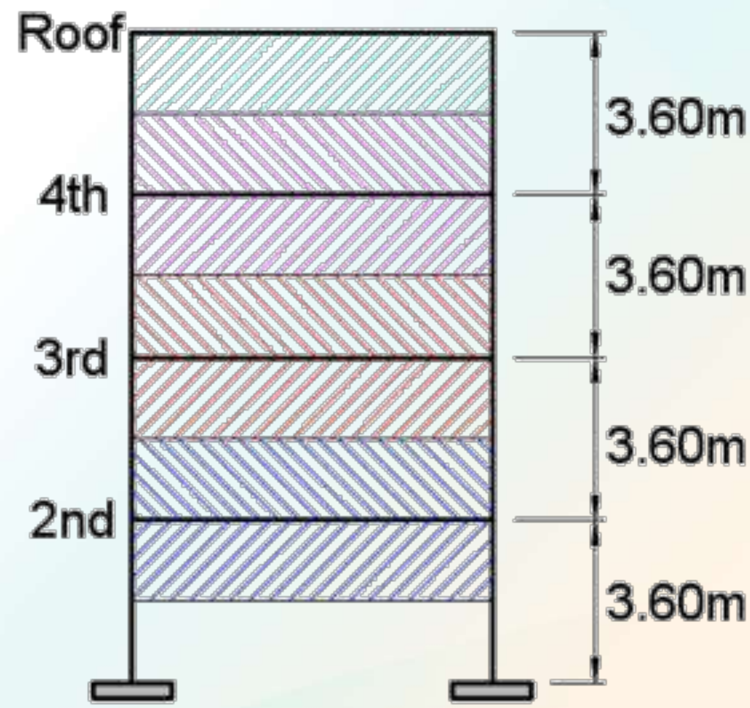
$$W_{DeckLevel_{below}} = 2.7(1.8)(108) = 524.88kN$$

D. Solve for the Perimeter of the walls at the deck level

$$P = 4 + 4 + 4 = 12m$$

E. Solve for the Weight of the walls at the deck level

$$W_{Walls_{DeckLevel}} = 2.7(3.6)(12) = 116.64kN$$



Dead Load Computations for a Steel Structure

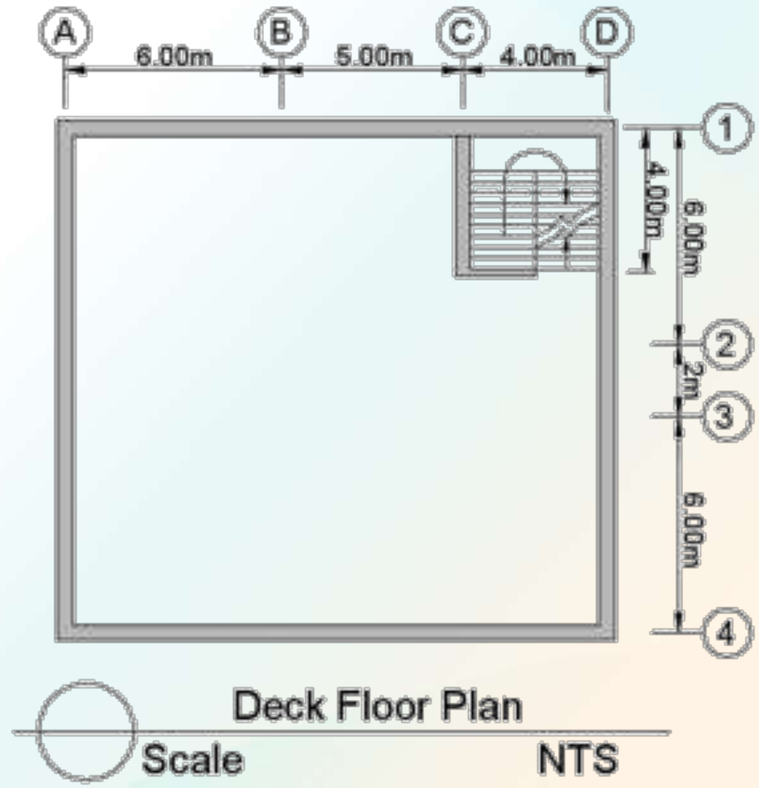
Example 1:

F. Solve for the Perimeter of the Parapet (Deck Level)

$$P_{parapet} = 11 + 14 + 15 + 10 = 50m$$

G. Solve for the Weight of the Parapet (Deck Level)

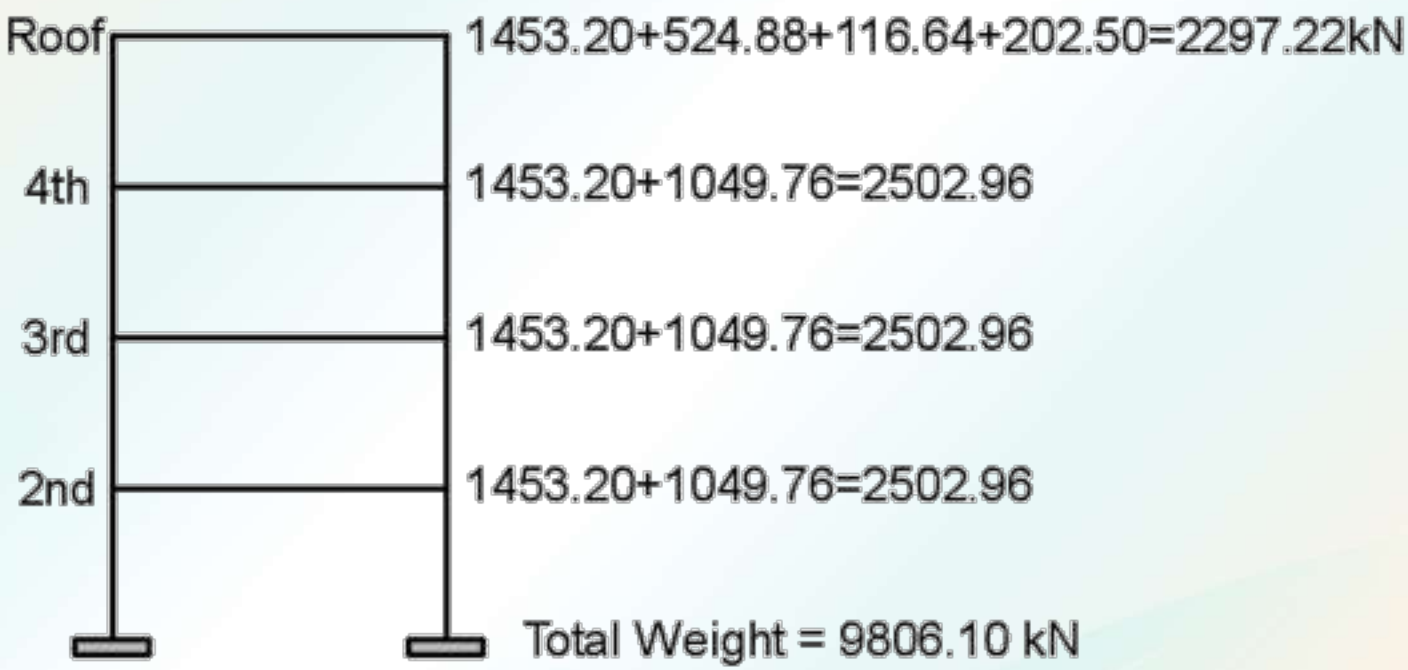
$$W_{parapet} = 2.7(1.5)(50) = 202.50kN$$



Dead Load Computations for a Steel Structure

Example 1:

H. Solve for the final weight of the structure



Dead Load Computations for a Steel Structure

Example 2:

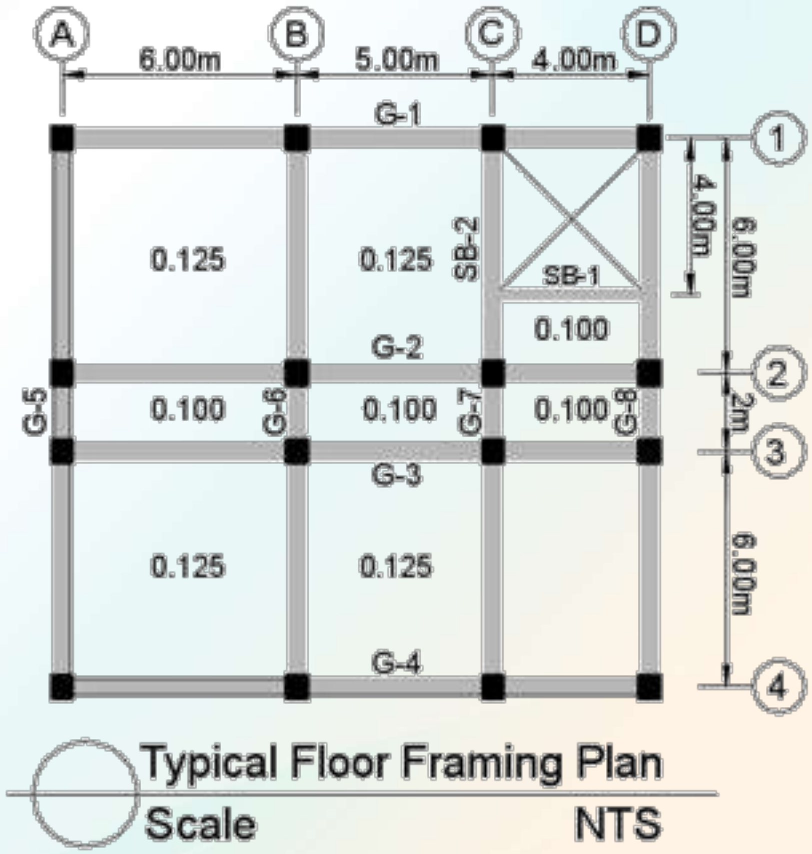
A sample set of plans were presented in the figure. Using the following values:

A. Beam/Column Sizes
   a. EB/SB = 0.25m x 0.40m
   b. Joist = 0.25m x 0.50m
   c. Girder = 0.35m x 0.60m
   d. Column = 0.40m x 0.40m

B. Design Loads
   a. Parapet/Wall = 2.70 kPa
   b. Superimposed DL = 1.60 kPa
   c. Unit Weight of Concrete = 24 kN/m^3
   d. Height of Wall = 3.2m
   e. Height of Parapet = 1.5m
   f. Stairs = 5 kPa

$$T.A. = (11.00x14.00) + (4.00x10.00)$$

$$T.A. = 194m^2$$

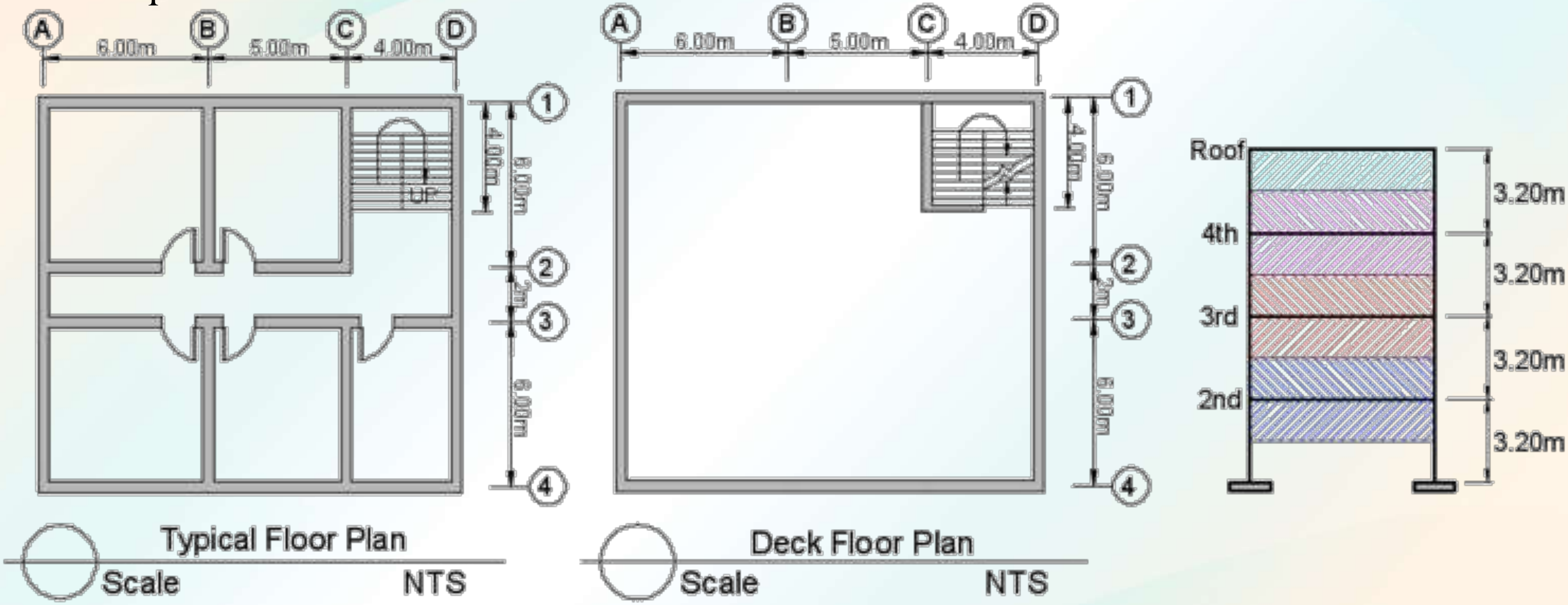


Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Dead Load Computations for a Steel Structure

Example 2:



Dead Load Computations for a Concrete Structure

Example 2:

Solve for the Perimeter of the Walls (Typical Levels)

$$P = \begin{bmatrix} 15+11+15+15 \\ +14+12+12+14 \end{bmatrix}$$

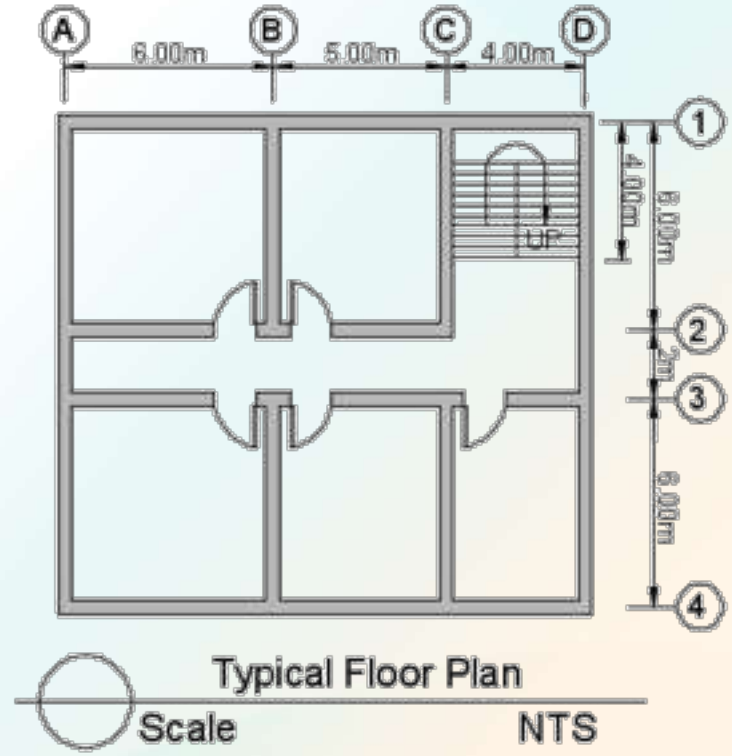
$$P = 108m$$

Solve for the Perimeter of the walls at the deck level

$$P = 4+4+4 = 12m$$

Solve for the Perimeter of the Parapet (Deck Level)

$$P_{parapet} = 11+14+15+10 = 50m$$



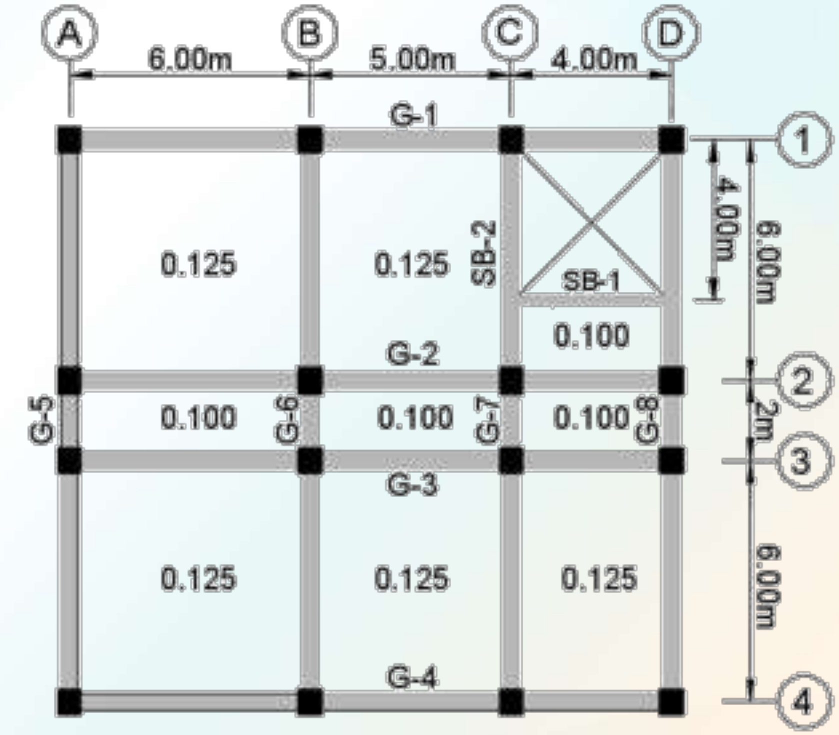
Dead Load Computations for a Concrete Structure

Example 2:

A. Deck Level

$$SB = 0.25x(0.400 - 0.100)x4.00x24 = 7.2kN$$

$$Girder = 0.35x(0.60 - 0.125)x11.00x24 = 43.89kN$$
$$= 0.35x0.60x4.00x24 = 20.16kN$$
$$= 0.35x(0.60 - 0.100)x15.00x24 = 63.00kN$$
$$= 0.35x(0.60 - 0.100)x15.00x24 = 63.00kN$$
$$= 0.35x(0.60 - 0.125)x15.00x24 = 59.85kN$$
$$= 0.35x(0.60 - 0.125)x12.00x24 = 47.88kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x12.00x24 = 47.88kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x4.00x24 = 15.96kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x6.00x24 = 23.94kN$$
$$= 0.35x0.60x4.00x24 = 20.16kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x6.00x24 = 23.94kN$$



Typical Floor Framing Plan

Scale NTS

Dead Load Computations for a Concrete Structure

Example 2:

A. Deck Level

$$Slab = 0.125x(11.00x6.00)x24 = 198kN$$

$$= 0.100x(4.00x2.00)x24 = 19.2kN$$

$$= 0.100x(15.00x2.00)x24 = 72.00kN$$

$$= 0.125x(15.00x6.00)x24 = 270kN$$

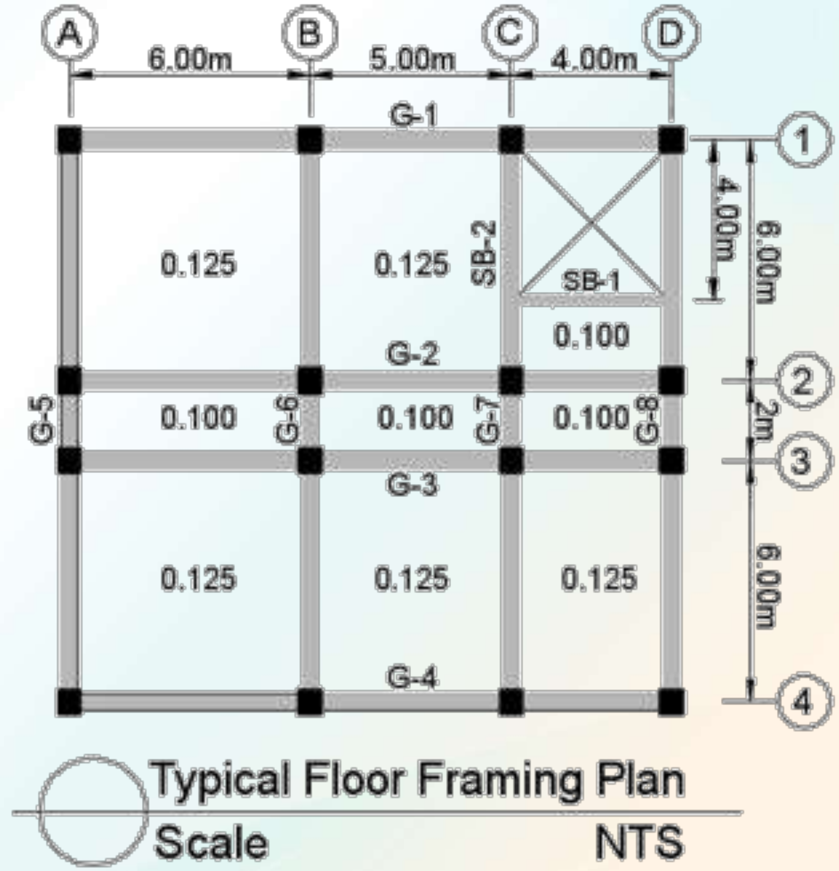
$$Parapet = 2.70x50x1.5 = 202.5kN$$

$$S_{DL} = 1.60x194.00 = 310.4kN$$

$$Columns = 0.40x0.40x1.6x24x16 = 98.30kN$$

$$Wall = 2.7x12x3.2 = 103.68kN$$

$$= 2.7x108x1.6 = 466.56kN$$

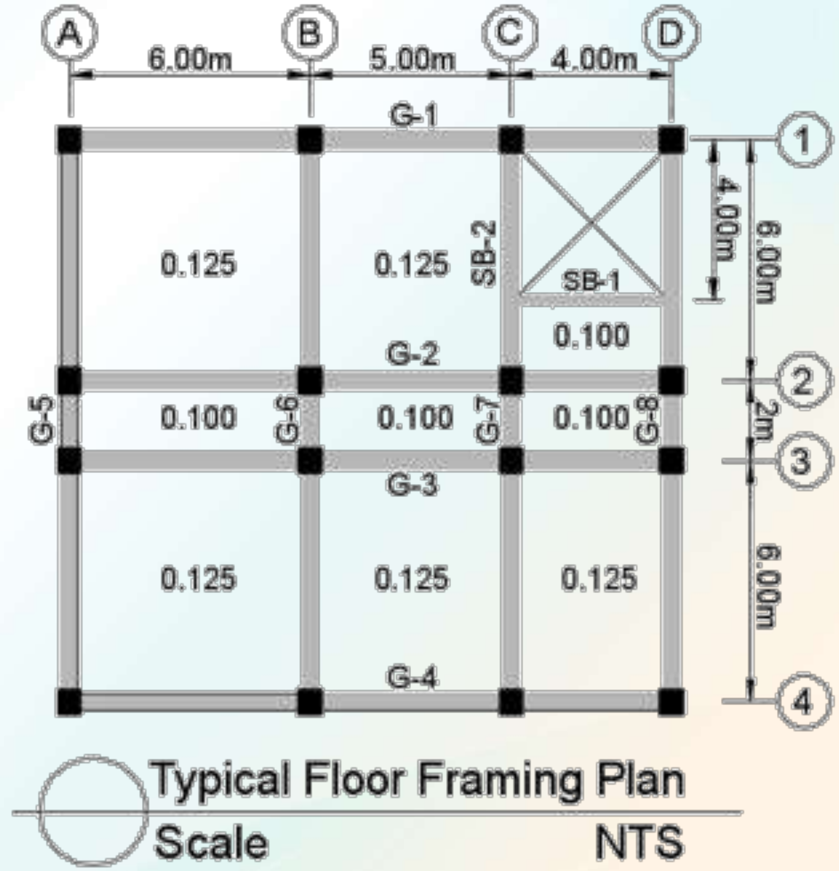


Dead Load Computations for a Concrete Structure

Example 2:

A. Deck Level

$$TotalWeight_{deck} = 2219.50kN$$



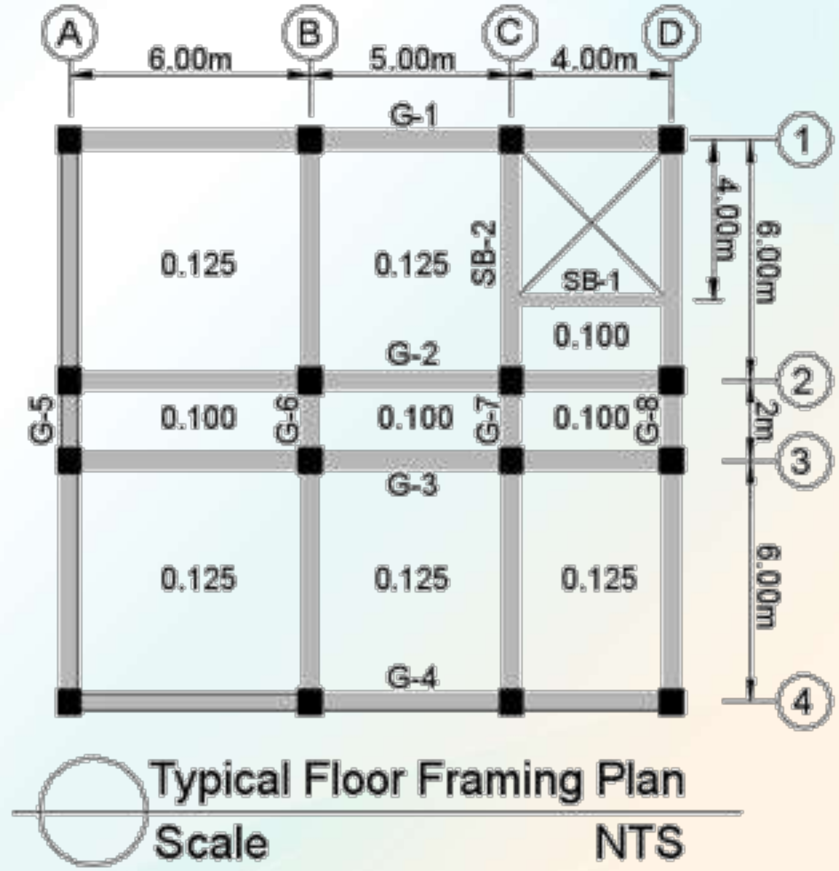
Dead Load Computations for a Concrete Structure

Example 2:

B. Typical Levels

$$SB = 0.25x(0.400 - 0.100)x4.00x24 = 7.2kN$$

$$Girder = 0.35x(0.60 - 0.125)x11.00x24 = 43.89kN$$
$$= 0.35x0.60x4.00x24 \quad = 20.16kN$$
$$= 0.35x(0.60 - 0.100)x15.00x24 = 63.00kN$$
$$= 0.35x(0.60 - 0.100)x15.00x24 = 63.00kN$$
$$= 0.35x(0.60 - 0.125)x15.00x24 = 59.85kN$$
$$= 0.35x(0.60 - 0.125)x12.00x24 = 47.88kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x12.00x24 = 47.88kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x4.00x24 = 15.96kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x6.00x24 = 23.94kN$$
$$= 0.35x0.60x4.00x24 \quad = 20.16kN$$
$$= 0.35x(0.60 - 0.100)x2.00x24 = 8.40kN$$
$$= 0.35x(0.60 - 0.125)x6.00x24 = 23.94kN$$



Dead Load Computations for a Concrete Structure

Example 2:

B. Typical Levels

$$Slab = 0.125x(11.00x6.00)x24 = 198kN$$

$$= 0.100x(4.00x2.00)x24 = 19.2kN$$

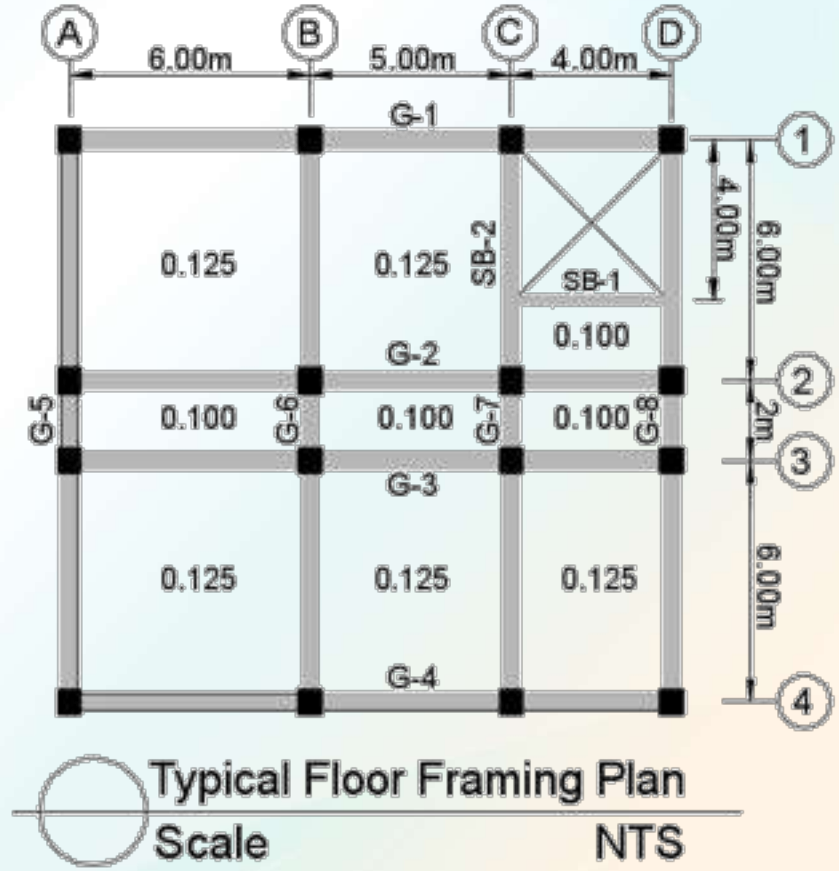
$$= 0.100x(15.00x2.00)x24 = 72.00kN$$

$$= 0.125x(15.00x6.00)x24 = 270kN$$

$$S_{DL} = 1.60x194.00 = 310.4kN$$

$$Columns = 0.4x0.4x3.2x24x16 = 196.608kN$$

$$Wall = 2.7x108x3.2 = 933.12kN$$

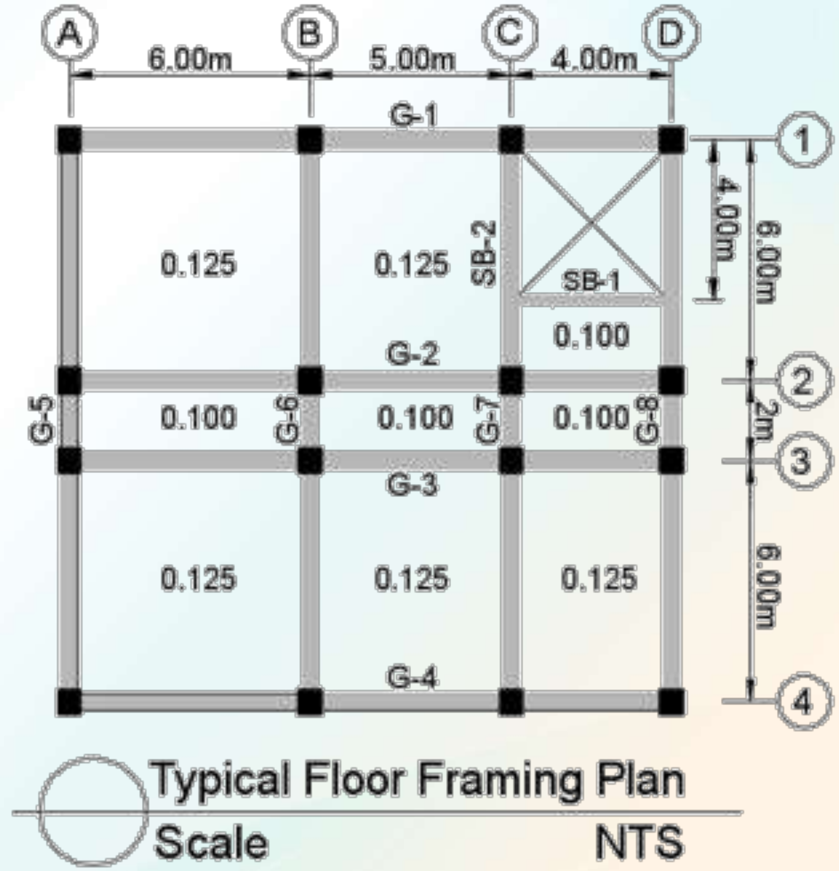


Dead Load Computations for a Concrete Structure

Example 2:

B. Typical Level

$$TotalWeight_{typical} = 2478.19kN$$



Dead Load Computations for a Concrete Structure

Example 2:

B. Solve for the total weight of the structure

CENTER OF MASS, CENTER OF RIGIDITY AND ECCENTRICITIES

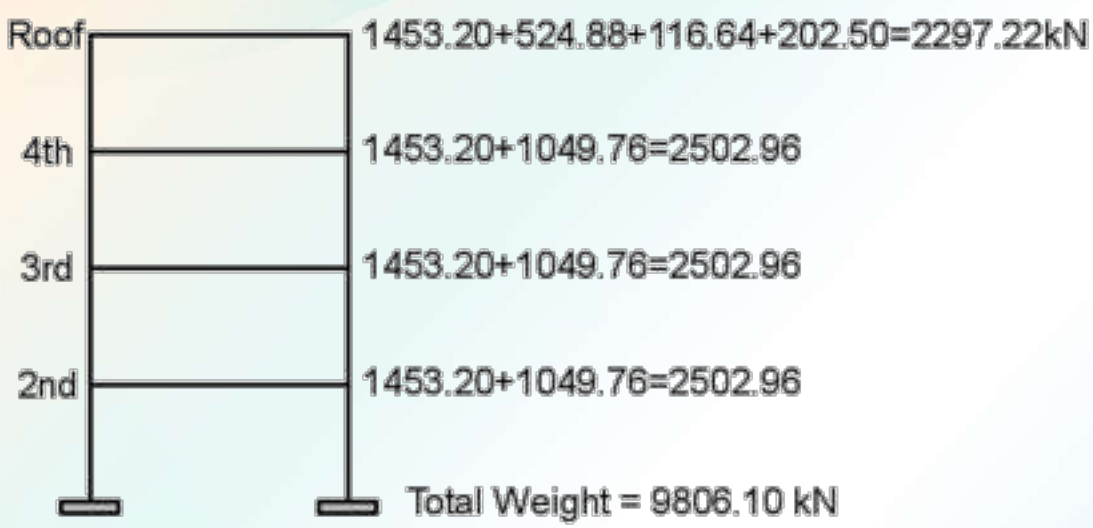
Distribution of Lateral Forces Per Frame

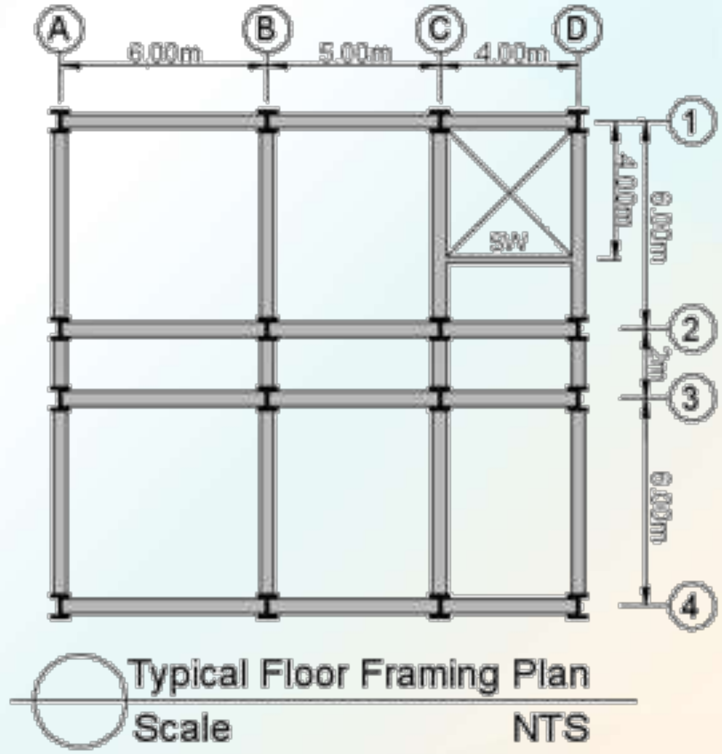
FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Horizontal Distrubution of Shear

Problem 3:





Given:

Zone 4, I = 1.0, Soil Profile Type A, Seismic Source Type A, Distance from the Source = 5km, R = 8.5, h = 3.6m, e_{acc} = 5% (Longitudinal Dimension)

Problem 3 - Solution

A. Solve for the structure period

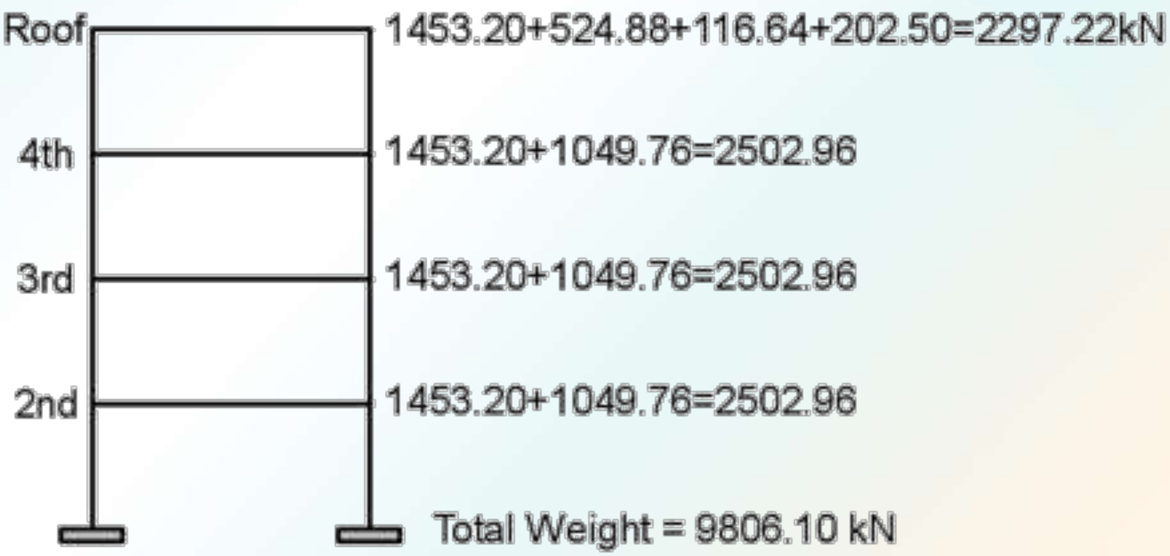
For steel SMRF, $C_t = 0.0853$

$$T = C_t\left(h_n\right)^{\frac{3}{4}}$$

$$T = 0.0853(14.4)^{\frac{3}{4}}$$

$$T = 0.631\sec < 0.7\sec$$

❖ $T < 0.7sec$, *therefore,* $F_t = 0$



Problem 3 - Solution

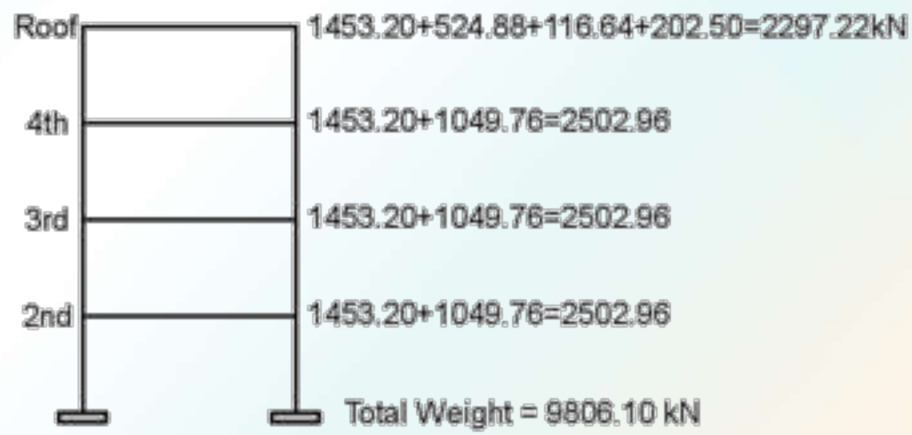
B. Find the near source factors N_a and N_v

Table 208-5: Near-Source Factor, N_a

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

Table 208-6: Near-Source Factor, N_v

Seismic Source Type	Closest Distance to Known Seismic Source[2]			
	< 2km	5km	10km	> 10km
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



$$N_a = 1.2$$

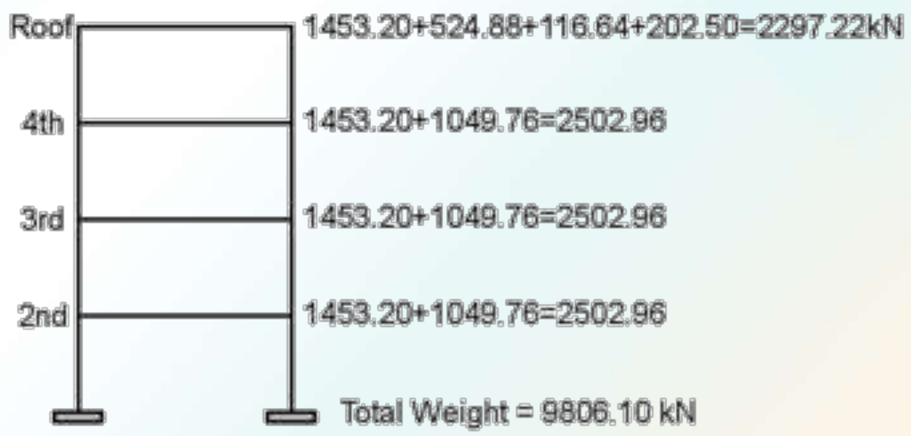
$$N_v = 1.6$$

Problem 3 - Solution

C. Determine the Seismic Coefficients C_a and C_v

Table 208-7: Seismic Coefficient, C_a

Soil Profile Type	Seismic Zone (Z)	
	Z = 0.2	$Z = 0.4$
S_A	0.16	$0.32N_a$
S_B	0.20	$0.40N_a$
S_C	0.24	$0.40N_a$
S_D	0.28	$0.44N_a$
S_E	0.34	$0.44N_a$
S_F	See Footnote 1 of Table 208-8	



$$C_a = 0.32N_a$$

$$C_a = 0.32(1.2) = 0.384$$

$N_a = 1.2$

$N_v = 1.6$

FEU ALABANG FEU DILIMAN FEU TECH

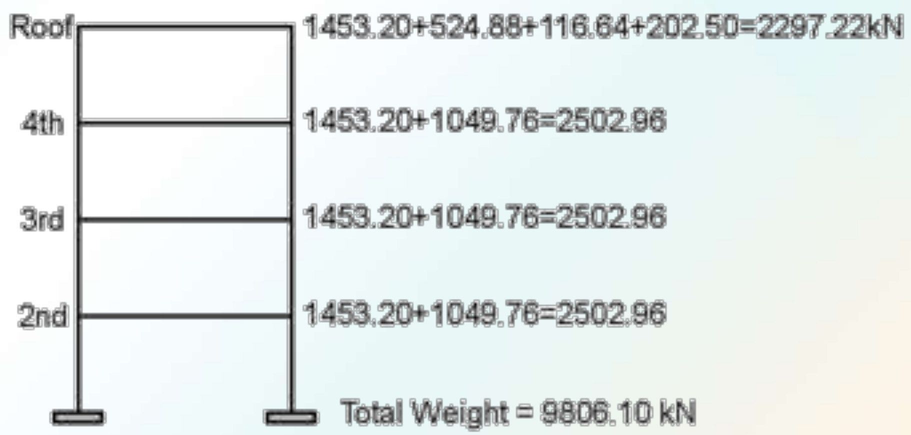
Technology Driven by Innovation

Problem 3 - Solution

C. Determine the Seismic Coefficients C_a and C_v

Table 208-8: Seismic Coefficient, C_v

Soil Profile Type	Seismic Zone (Z)	
	Z = 0.2	Z = 0.4
S_A	0.16	$0.32N_v$
S_B	0.20	$0.40N_v$
S_C	0.32	$0.56N_v$
S_D	0.40	$0.64N_v$
S_E	0.64	$0.96N_v$
S_F	See Footnote 1 of Table 208-8	



$$C_v = 0.32N_v$$

$$C_v = 0.32(1.6) = 0.512$$

N_a = 1.2

N_v = 1.6

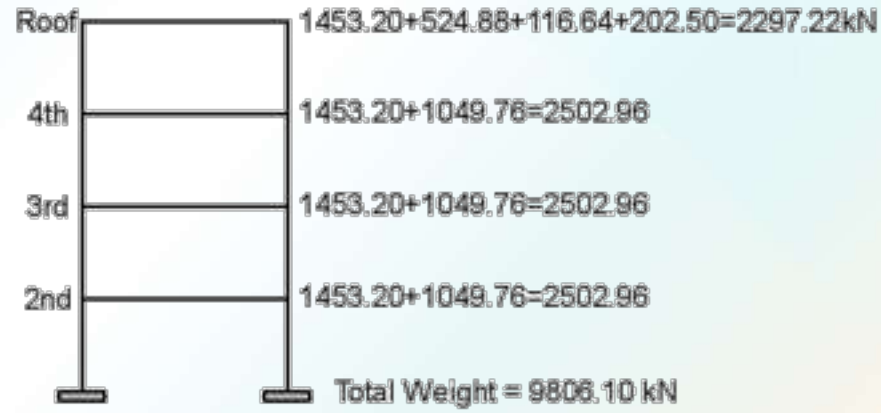
Problem 3 - Solution

D. Determine the Base Shear

The total design base shear in a given direction is:

$$V = \frac{C_v I}{RT} W \qquad V = \frac{(0.512)(1.0)}{(8.5)(0.631)}(9806.10)$$

$$V = 936.091 kN$$



Values:

$C_a = 0.384$

$I = 1.0$

$R = 8.5$

$C_v = 0.512$

$T = 0.631s$

But the code indicates that the total design base shear need not exceed the following:

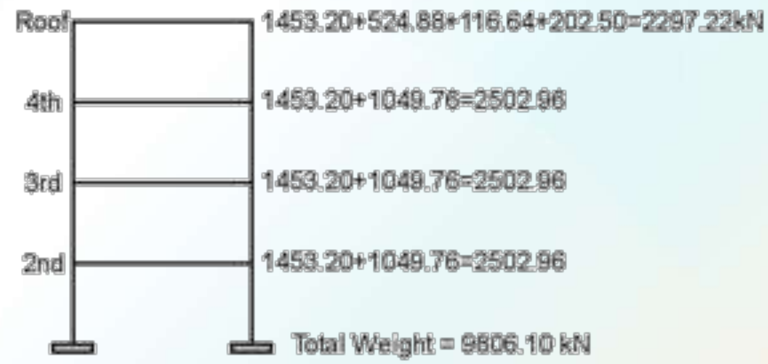
$$V = \frac{2.5 C_a I}{R} W \qquad V = \frac{2.5(0.384)(1.0)}{(8.5)}(9806.10)$$

$$V = 1107.512 kN$$

Problem 3 - Solution

D. Determine the Base Shear

And that the base shear shall not be less than the following:



$$V = 0.11 C_a I W \qquad V = 0.11(0.384)(1.0)(9806.10)$$

$$V = 414.210 kN$$

And in Seismic 4, the total design base shear shall also be not less than:

$$V = \frac{0.8 Z N_v I}{R} W \qquad V = \frac{0.8(0.4)(1.6)(1.0)}{(8.5)}(9806.10)$$

$$V = 590.673 kN$$

Values:

C_a = 0.384

I = 1.0

R = 8.5

N_v = 1.6

Z = 0.4

Problem 3 - Solution

D. Determine the Base Shear

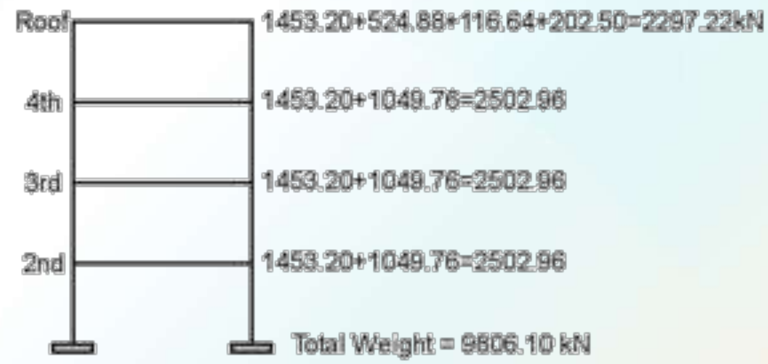
Therefore, the governing design base shear for this example is:

$$V = 936.091 kN$$

$$V = 1107.512 kN$$

$$V = 414.210 kN$$

$$V = 590.673 kN$$



Values:

$C_a = 0.384$

$I = 1.0$

$R = 8.5$

$N_v = 1.6$

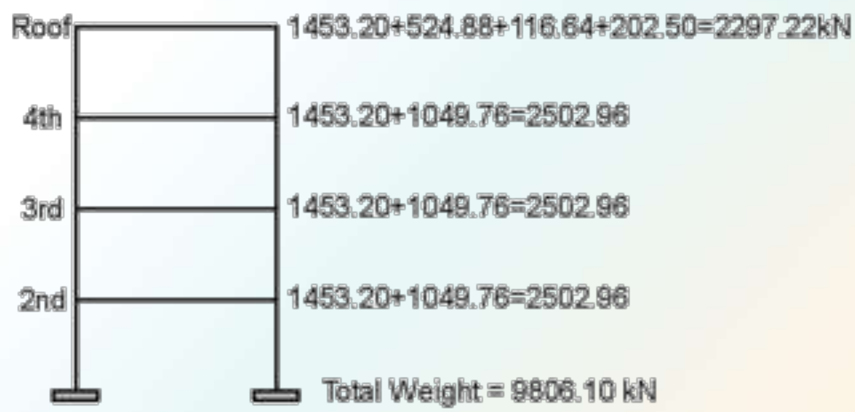
$Z = 0.4$

Problem 3 - Solution

E. Find F_x at each level.

The vertical distribution of seismic forces is determined from NSCP Equation 208-15.

Level x	H_x	w_x	$w_x h_x$	F_x
Roof	14.4	2297.22	33079.968	355.342
4	10.8	2502.96	27031.968	290.375
3	7.2	2502.96	18021.312	193.583
2	3.6	2502.96	9010.656	96.792
			$\sum = 87{,}143.904$	$\sum = 936.091kN$



$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad V = 936.091kN$$

Problem 3 - Solution

F. Distribution of Lateral Force in each Frame

Working Equations:

$$d_x = \left|\left(x - \sum xR_L\right)\right|$$

$$d_y = \left|\left(y - \sum yR_T\right)\right|$$

$$F = F_1 + F_2$$

$$F_1 = R_T \ \ or \ \ R_L$$

$$F_2 = \frac{M_T\left[Rd\right]}{\sum Rd^2}$$

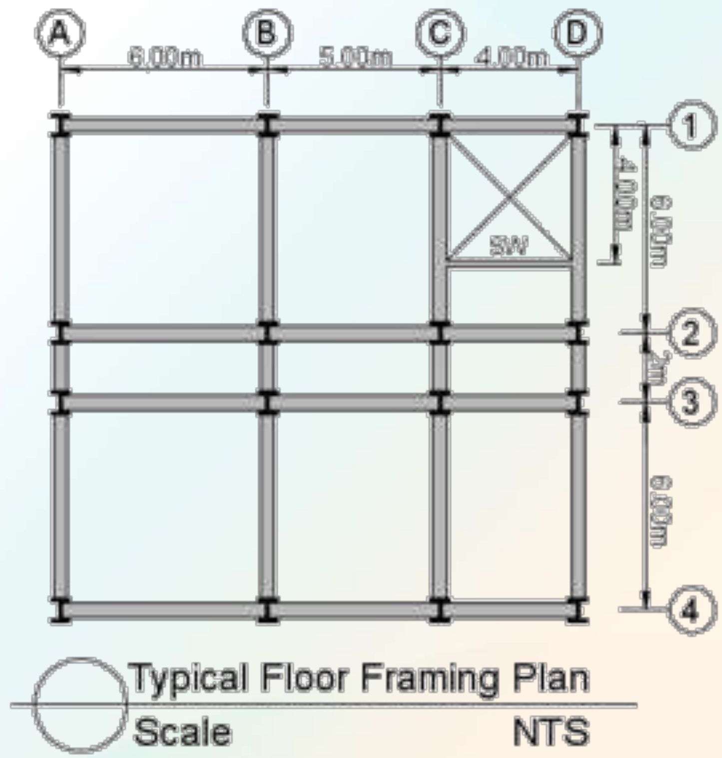
$$M_T = 1.0\left(e_{govern}\right)$$

$$e_x = \left|CM_x - CR_x\right|$$

$$e_y = \left|CM_y - CR_y\right|$$

$$e_{acc} = 5\%\begin{pmatrix} building\ dimension\ at\ that\ level\ perpendicular \\ to\ the\ direction\ of\ the\ force\ under\ consideration \end{pmatrix}$$

$$CR_x = \frac{\sum xR_L}{\sum R_L} \qquad CR_y = \frac{\sum yR_T}{\sum R_T}$$



Problem 3 - Solution

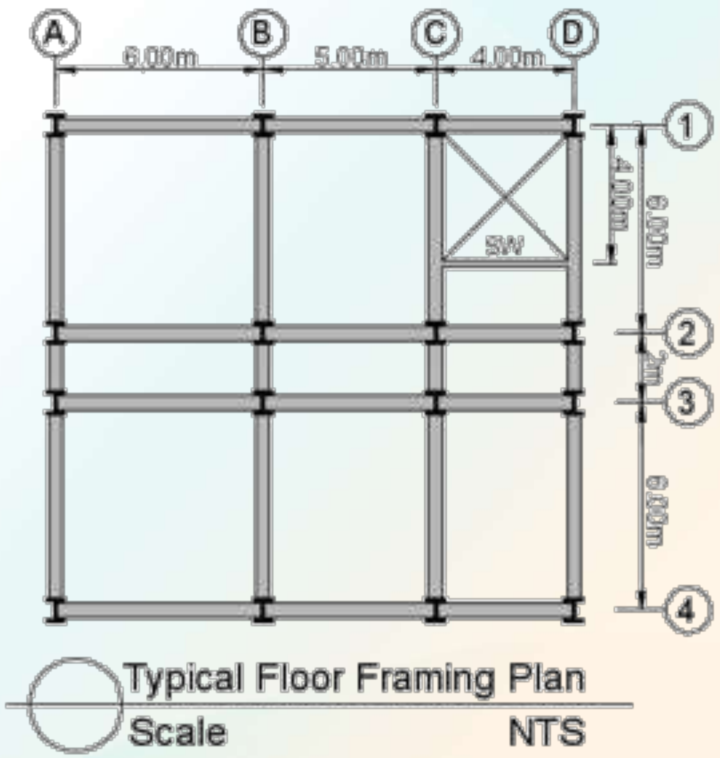
F. Distribution of Lateral Force in each Frame

a. West – East Direction

Frame Mark	R_L	x	xR_L	d_x	$R_L d_x$	$R_L d_x^2$	F_1	F_2	F
A	1/4	0	0	8.00	2.00	16.00	1/4		
B	1/4	6	1.5	2.00	0.50	1.00	1/4		
C	1/4	11	2.75	3.00	0.75	2.25	1/4		
D	1/4	15	3.75	7.00	1.75	12.25	1/4		
			$\sum = 8.00$			$\sum = 31.5$			

$$CM_x = \frac{15}{2} = 7.5m$$

$$CM_y = \frac{14}{2} = 7.0m$$



Problem 3 - Solution

F. Distribution of Lateral Force in each Frame

b. North to South Direction

Frame Mark	R_T	y	yR_T	d_y	R_Td_y	$R_Td_y^2$	F_1	F_2	F
A	1/4	0	0	7.00	1.75	12.25	1/4		
B	1/4	6	1.50	1.00	0.25	0.25	1/4		
C	1/4	8	2.00	1.00	0.25	0.25	1/4		
D	1/4	14	3.50	7.00	1.75	12.25	1/4		
			$\sum = 7.00$			$\sum = 25$			

$$CR_x = \frac{8.0}{1} = 8.0m$$

$$CR_y = \frac{7.0}{1} = 7.0m$$

$$e_x = |7.5 - 8.0| = 0.5m$$

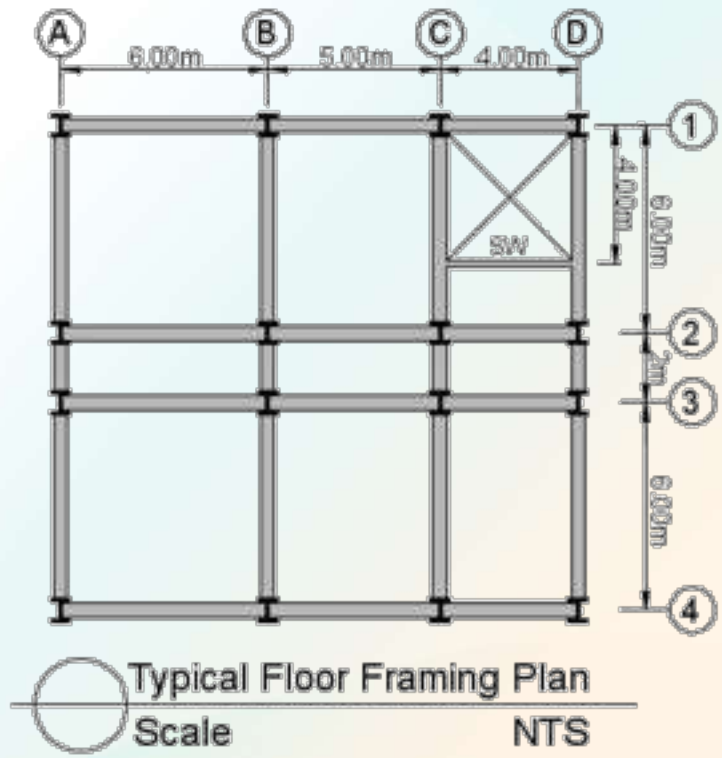
$$e_y = |7.0 - 7.0| = 0m$$

$$e_{acc} = 5\%(15) = 0.75m$$

$$\therefore e_{govern} = 0.75m$$

$$M_T = 1.0(0.75) = 0.75kN.m$$

$$F_2 = \frac{M_T(Rd)}{\sum Rd^2}$$



Problem 3 - Solution

F. Distribution of Lateral Force in each Frame

a. West – East Direction

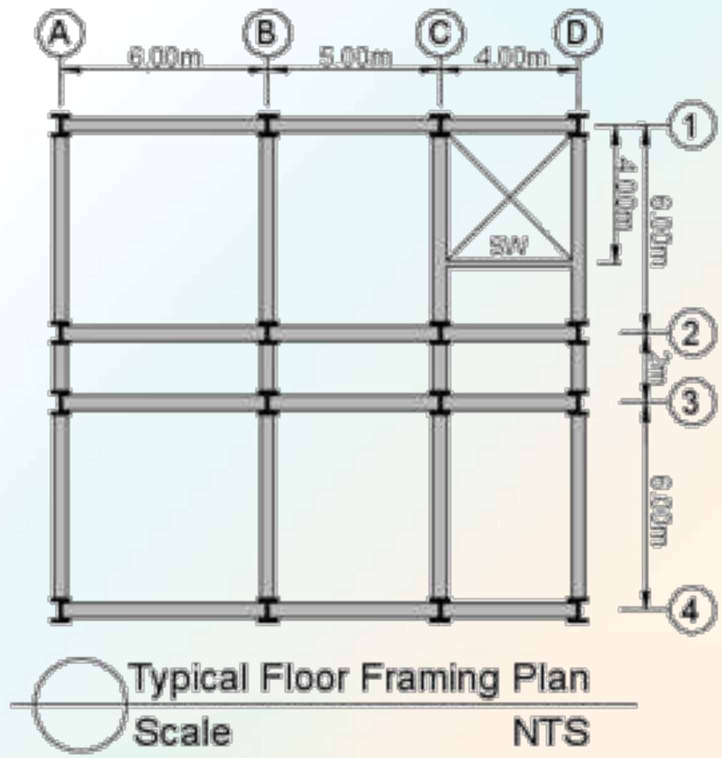
Frame Mark	R_L	x	xR_L	d_x	$R_L d_x$	$R_L d_x^2$	F_1	F_2	F
A	1/4	0	0	8.00	2.00	16.00	1/4	0.048	0.298
B	1/4	6	1.5	2.00	0.50	1.00	1/4	0.012	0.262
C	1/4	11	2.75	3.00	0.75	2.25	1/4	0.018	0.268
D	1/4	15	3.75	7.00	1.75	12.25	1/4	0.042	0.292
			$\sum = 8.00$			$\sum = 31.5$			

$$CM_x = \frac{15}{2} = 7.5m$$

$$CM_y = \frac{14}{2} = 7.0m$$

$$M_T = 1.0(0.75) = 0.75 kN.m$$

$$F_2 = \frac{M_T (Rd)}{\sum Rd^2}$$



Problem 3 - Solution

F. Distribution of Lateral Force in each Frame

b. North to South Direction

Frame Mark	R_T	y	yR_T	d_y	R_Td_y	$R_Td_y^2$	F_1	F_2	F
A	1/4	0	0	7.00	1.75	12.25	1/4	0.053	0.303
B	1/4	6	1.50	1.00	0.25	0.25	1/4	0.008	0.258
C	1/4	8	2.00	1.00	0.25	0.25	1/4	0.008	0.258
D	1/4	14	3.50	7.00	1.75	12.25	1/4	0.053	0.303
			$\sum = 7.00$			$\sum = 25$			

$$CR_x = \frac{8.0}{1} = 8.0m$$

$$CR_y = \frac{7.0}{1} = 7.0m$$

$$e_x = |7.5 - 8.0| = 0.5m$$

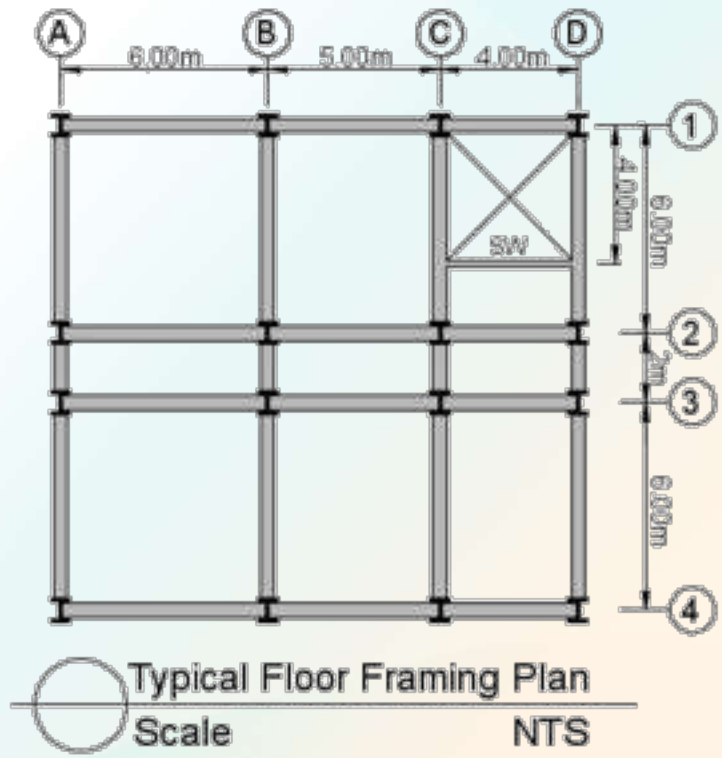
$$e_y = |7.0 - 7.0| = 0m$$

$$e_{acc} = 5\%(15) = 0.75m$$

$$\therefore e_{govern} = 0.75m$$

$$M_T = 1.0(0.75) = 0.75kN.m$$

$$F_2 = \frac{M_T(Rd)}{\sum Rd^2}$$



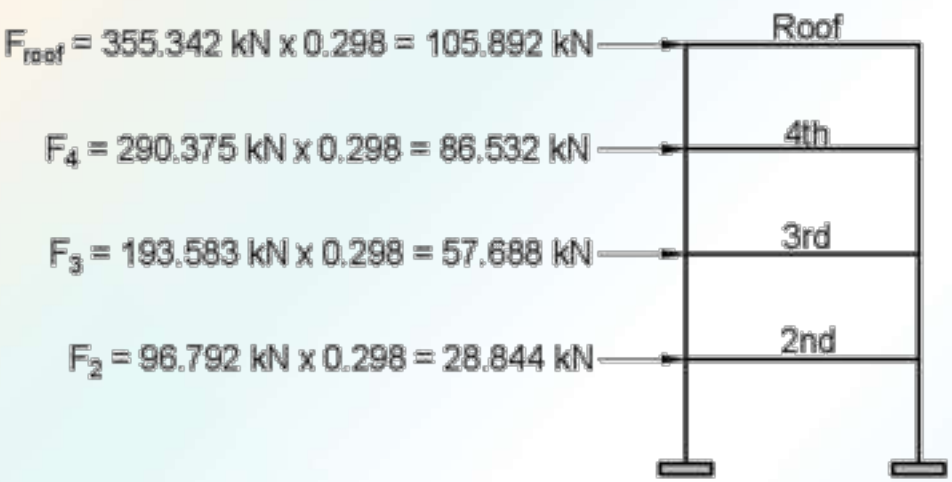
FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

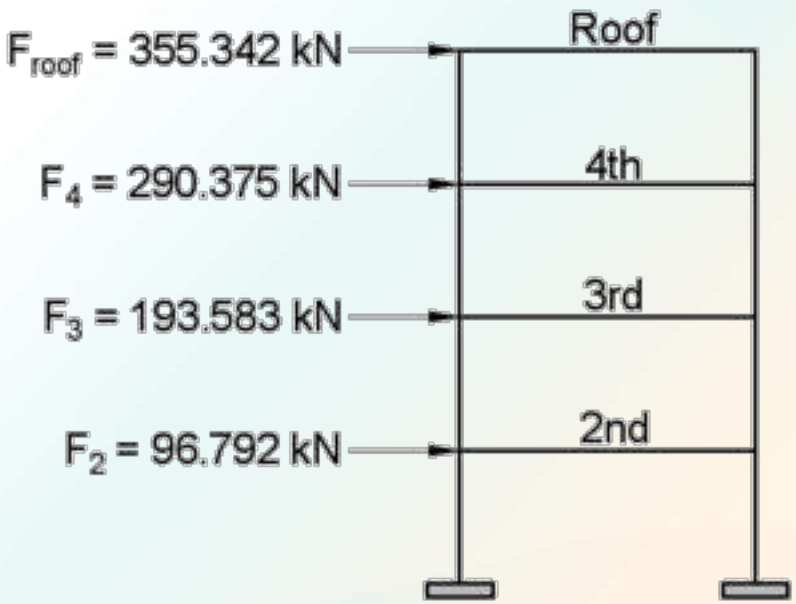
Problem 3 - Solution

F. Distribution of Lateral Force in each Frame

Grid A (F = 0.298)



Vertical Distribution of Forces



CENTER OF MASS, CENTER OF RIGIDITY AND ECCENTRICITIES

Stiffness of Columns

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Stiffness of Columns

Working Equations:

$$\delta_f = \frac{Ph^3}{12EI}$$

$$\delta_v = \frac{12Ph}{GA}$$

$$\delta_T = \delta_f + \delta_v$$

$$K = \frac{1}{\delta_T}$$

Where:

δ_f = Displacement due to flexure

δ_v = Displacement due to shear

δ_T = Total Displacement

P = 1000 kN

h = Height of Column

E = Modulus of Elasticity

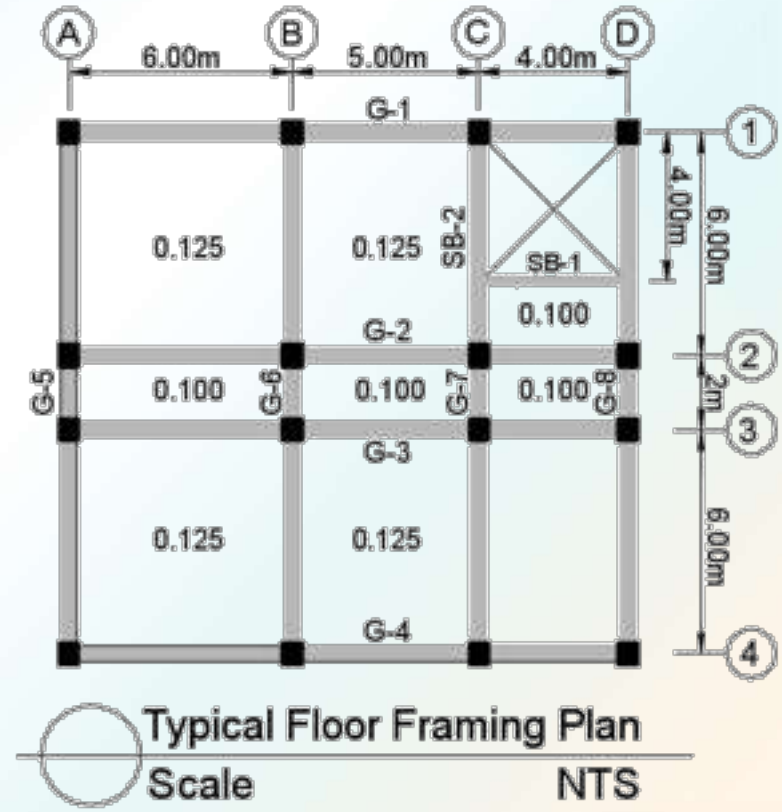
I = Moment of Inertia of the Column w/r to the N.A.

G = Shear Modulus

A = Column Area

K = Stiffness of Column

Problem 4



Given:

Zone 4, I = 1.0, Soil Profile Type A, Seismic Source Type A, Distance from the Source = 5km, R = 8.5, h = 3.2m, Column Dimension = 0.4 x 0.4m, E = 24.84×10^6 kPa, G=9.92×10^6 kPa

Problem 4 - Solution

A. Stiffness Computations (West to East Direction)

Frame	Column	B (m)	D (m)	H (m)	A (m^2)	I (m^4)	δ_f (m)	δ_v (m)	δ_T (m)	K	Total
1	A	0.4	0.4	3.2	0.16	0.00213	0.05161	0.00242	0.05403	18.50824	74.03296
	B									18.50824	
	C									18.50824	
	D									18.50824	
2	A									18.50824	74.03296
	B									18.50824	
	C									18.50824	
	D									18.50824	
3	A									18.50824	74.03296
	B									18.50824	
	C									18.50824	
	D									18.50824	
4	A									18.50824	74.03296
	B									18.50824	
	C									18.50824	
	D									18.50824	
										Total	296.13184

Problem 4 - Solution

B. Stiffness Computations (North to South Direction)

Frame	Column	B (m)	D (m)	H (m)	A (m^2)	I (m^4)	δ_f (m)	δ_v (m)	δ_T (m)	K	Total
A	1	0.4	0.4	3.2	0.16	0.00213	0.05161	0.00242	0.05403	18.50824	74.03296
	2									18.50824	
	3									18.50824	
	4									18.50824	
B	1									18.50824	74.03296
	2									18.50824	
	3									18.50824	
	4									18.50824	
C	1									18.50824	74.03296
	2									18.50824	
	3									18.50824	
	4									18.50824	
D	1									18.50824	74.03296
	2									18.50824	
	3									18.50824	
	4									18.50824	
										Total	296.13184

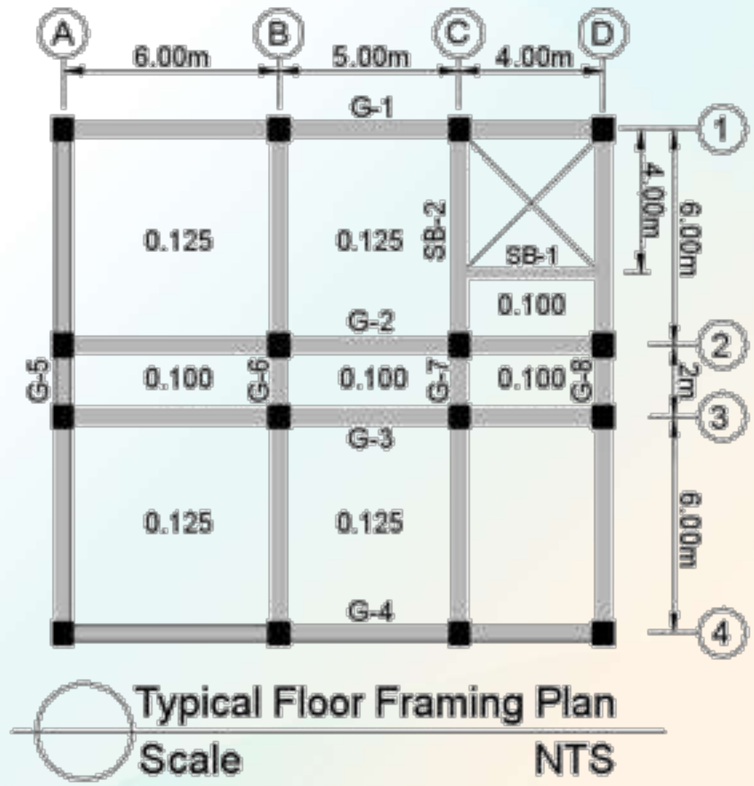
Problem 4 - Solution

C. Distribution of Lateral Forces per Frame (West to East Direction)

Frame	K	R_L
A	74.03296	1/4
B	74.03296	1/4
C	74.03296	1/4
D	74.03296	1/4
Total	**296.13184**	1.0

D. Distribution of Lateral Forces per Frame (North to South Direction)

Frame	K	R_T
1	74.03296	1/4
2	74.03296	1/4
3	74.03296	1/4
4	74.03296	1/4
Total	**296.13184**	1.0



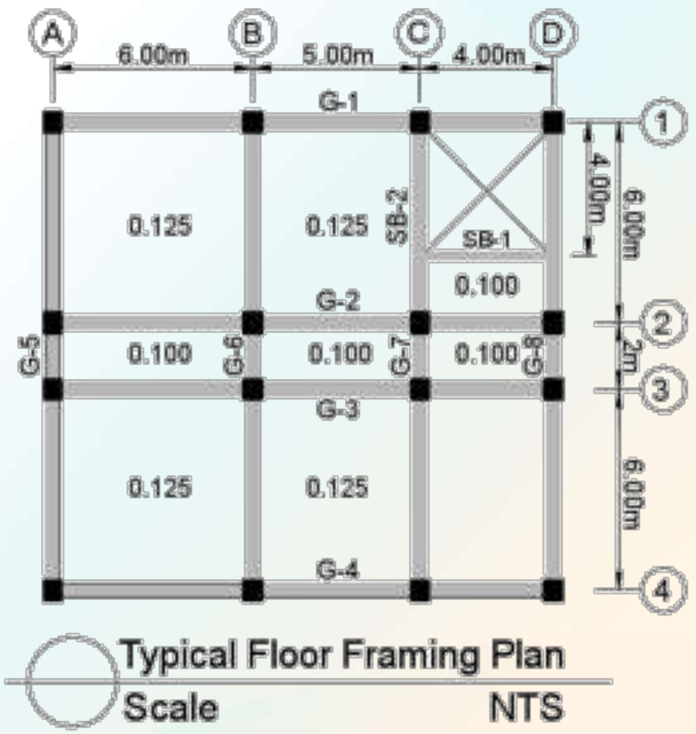
Problem 4 - Solution

Distribution of Lateral Force in each Frame

a. West – East Direction

Frame Mark	R_L	x	xR_L	d_x	R_Ld_x	$R_Ld_x^2$	F_1	F_2	F
A	1/4	0	0	8.00	2.00	16.00	1/4		
B	1/4	6	1.5	2.00	0.50	1.00	1/4		
C	1/4	11	2.75	3.00	0.75	2.25	1/4		
D	1/4	15	3.75	7.00	1.75	12.25	1/4		
			$\sum = 8.00$			$\sum = 31.5$			

$$CM_x = \frac{15}{2} = 7.5m \qquad CM_y = \frac{14}{2} = 7.0m$$



Problem 4 - Solution

F. Distribution of Lateral Force in each Frame

b. North to South Direction

Frame Mark	R_T	y	yR_T	d_y	$R_T d_y$	$R_T d_y^2$	F_1	F_2	F
A	1/4	0	0	7.00	1.75	12.25	1/4		
B	1/4	6	1.50	1.00	0.25	0.25	1/4		
C	1/4	8	2.00	1.00	0.25	0.25	1/4		
D	1/4	14	3.50	7.00	1.75	12.25	1/4		
			$\sum = 7.00$			$\sum = 25$			

$$CR_x = \frac{8.0}{1} = 8.0m$$

$$CR_y = \frac{7.0}{1} = 7.0m$$

$$e_x = |7.5 - 8.0| = 0.5m$$

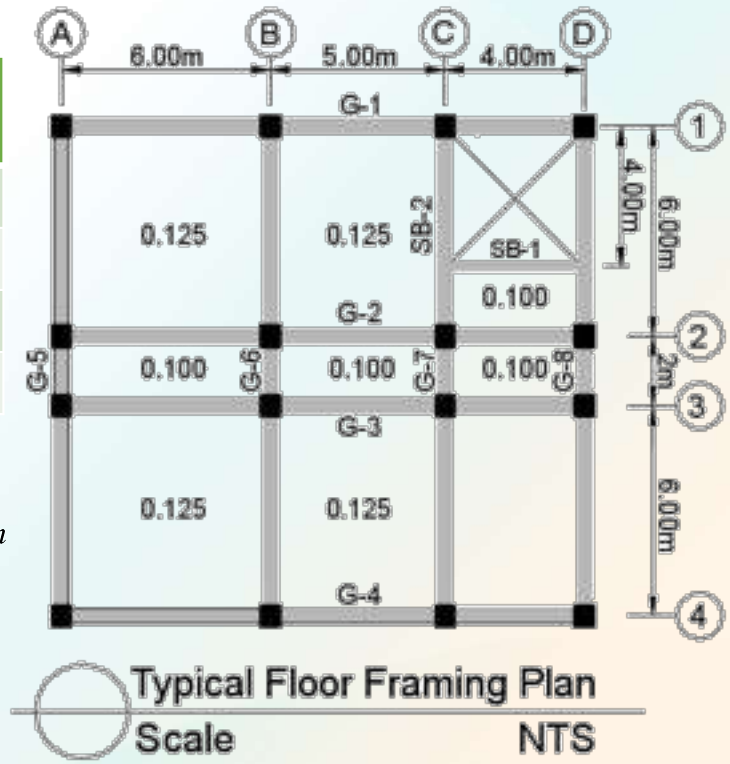
$$e_y = |7.0 - 7.0| = 0m$$

$$e_{acc} = 5\%(15) = 0.75m$$

$$\therefore e_{govern} = 0.75m$$

$$M_T = 1.0(0.75) = 0.75kN.m$$

$$F_2 = \frac{M_T(Rd)}{\sum Rd^2}$$



FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 4 - Solution

F. Distribution of Lateral Force in each Frame

a. West – East Direction

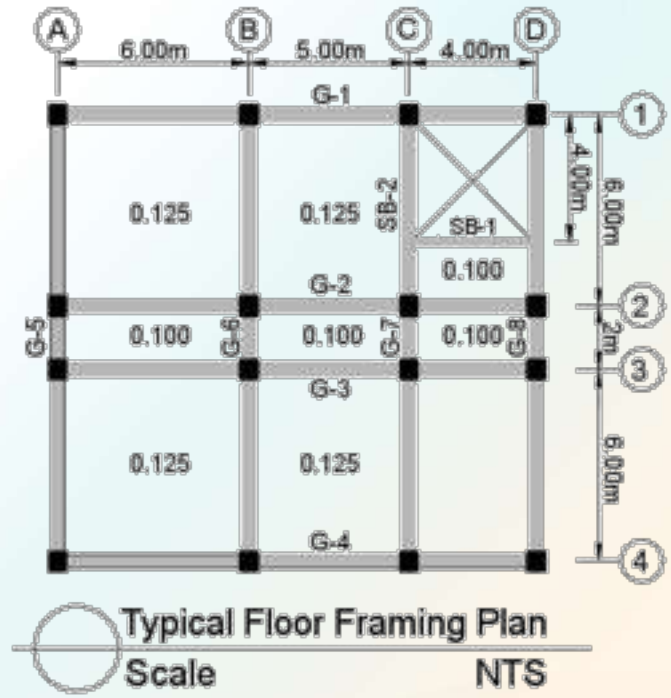
Frame Mark	R_L	x	xR_L	d_x	R_Ld_x	$R_Ld_x^2$	F_1	F_2	F
A	1/4	0	0	8.00	2.00	16.00	1/4	0.048	0.298
B	1/4	6	1.5	2.00	0.50	1.00	1/4	0.012	0.262
C	1/4	11	2.75	3.00	0.75	2.25	1/4	0.018	0.268
D	1/4	15	3.75	7.00	1.75	12.25	1/4	0.042	0.292
			$\sum = 8.00$			$\sum = 31.5$			

$$CM_x = \frac{15}{2} = 7.5m$$

$$CM_y = \frac{14}{2} = 7.0m$$

$$M_T = 1.0(0.75) = 0.75kN.m$$

$$F_2 = \frac{M_T(Rd)}{\sum Rd^2}$$



Problem 4 - Solution

F. Distribution of Lateral Force in each Frame

b. North to South Direction

Frame Mark	R_T	y	yR_T	d_y	$R_T d_y$	$R_T d_y^2$	F_1	F_2	F
A	1/4	0	0	7.00	1.75	12.25	1/4	0.053	0.303
B	1/4	6	1.50	1.00	0.25	0.25	1/4	0.008	0.258
C	1/4	8	2.00	1.00	0.25	0.25	1/4	0.008	0.258
D	1/4	14	3.50	7.00	1.75	12.25	1/4	0.053	0.303
			$\sum = 7.00$			$\sum = 25$			

$$CR_x = \frac{8.0}{1} = 8.0m$$

$$CR_y = \frac{7.0}{1} = 7.0m$$

$$e_x = |7.5 - 8.0| = 0.5m$$

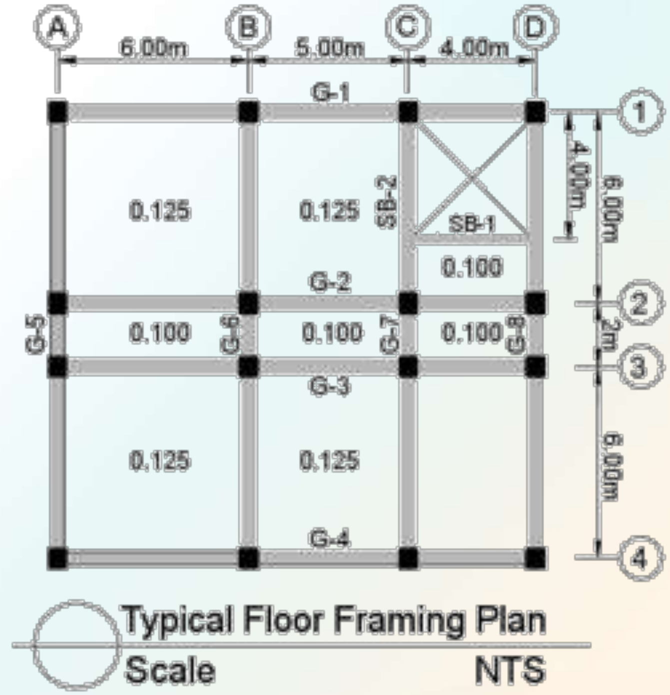
$$e_y = |7.0 - 7.0| = 0m$$

$$e_{acc} = 5\%(15) = 0.75m$$

$$\therefore e_{govern} = 0.75m$$

$$M_T = 1.0(0.75) = 0.75 kN.m$$

$$F_2 = \frac{M_T (Rd)}{\sum Rd^2}$$



Q&A SESSION

ASK ANY QUESTION RELATED TO OUR TOPIC FOR TODAY.

HOMEWORK

Do the Exercises about Base Shear Computations

Reference:

Association of Structural Engineers of the Philippines (2016). *National Structural Code of the Philippines 2015 (7th Edition) Volume 1 Buildings, Towers and Other Vertical Structures.*

Association of Structural Engineers of the Philippines (2003). *ASEP Earthquake Design Manual 2003 Volume 1: Code Provisions for Lateral Forces.*

CEELECT1
Earthquake Engineering
Direct and Torsional Forces and Moments

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Module 4

OBJECTIVES

- At the end of the chapter, the learner should be able to:
  - *Identify code provisions for the computation of direct and torsional forces.*
  - *Solve for the Torsions to the Base of the Building*

DIRECT AND TORSIONAL FORCES AND MOMENTS

Horizontal Distribution of Shear

208.5.1.3. Horizontal Distribution of Shear

The design storey shear, V_x in any storey is the sum of the forces F_t and F_x above that storey. V_x shall be distributed to the various elements of the vertical lateral force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction at a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the storey shear distribution shall be considered.

208.5.1.3. Horizontal Distribution of Shear

Diaphragms shall be considered flexible for the purposes of distribution of storey shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average storey drift of the associated storey. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the storey drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

DIRECT AND TORSIONAL FORCES AND MOMENTS

Sample Problem
Horizontal Distribution of Shear

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 1

A single storey building has a roof diaphragm. Shear walls resists lateral forces at both directions. The mass of the roof can be considered to be uniformly distributed, and in this example, the weight of the wall is neglected. In actual practice, particularly with concrete shear walls, the weight of the walls should be included in the determination of the center of mass (CM). The following information is given:

Design base shear: $V = 450$ kN

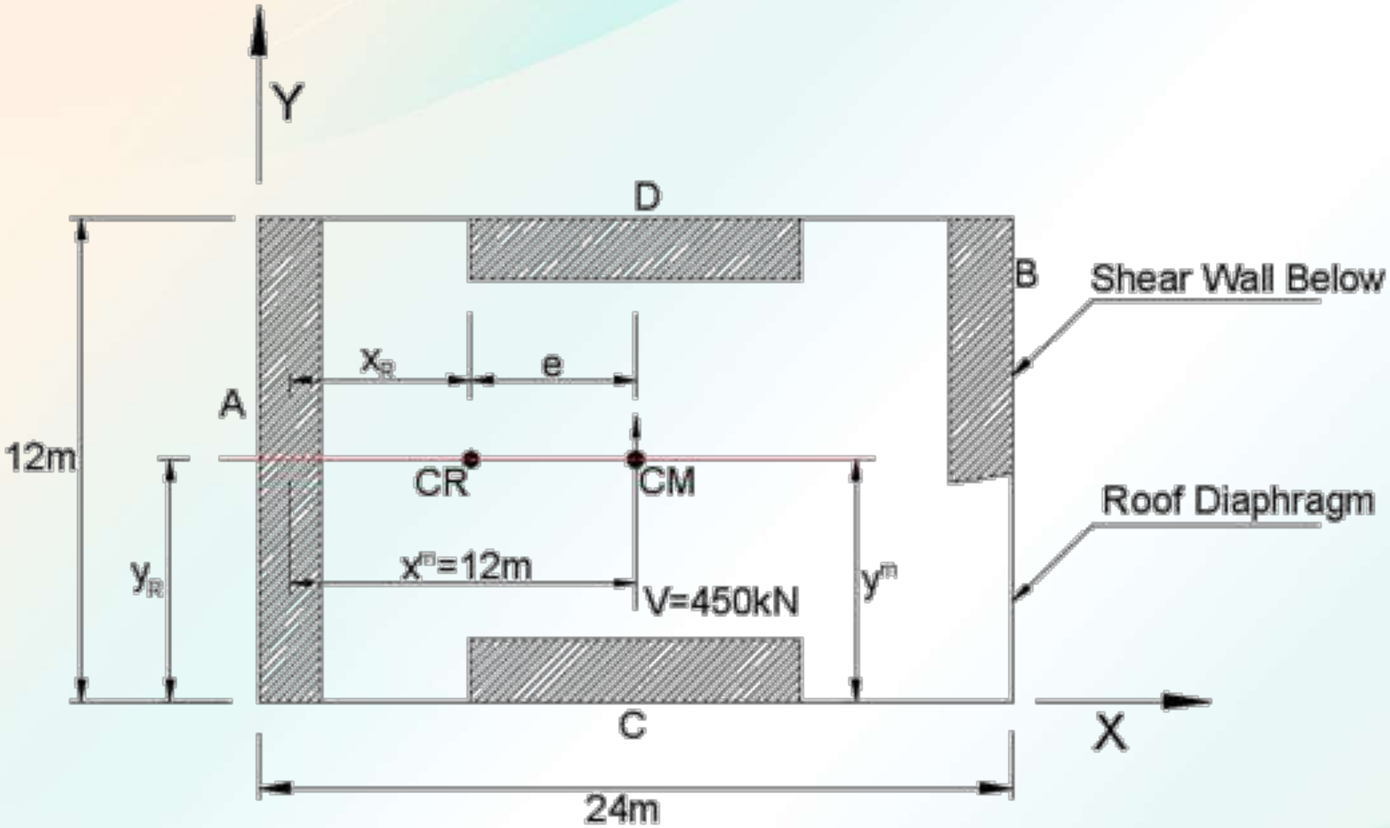
Wall Rigidities: $R_A = 54$ kN/mm

$R_B = 18$ kN/mm

$R_C = R_D = 36$ kN/mm

Center of Mass: $x_m = 24$m, $y_m = 12$m

Problem 1



Determine the following:

1. Eccentricity and rigidity properties.
2. Direct shear in walls A and B.
3. Plan irregularity requirements
4. Torsional shear in walls A and B
5. Total shear in walls A and B.

Problem 1 - Solution

Calculations and Discussions:

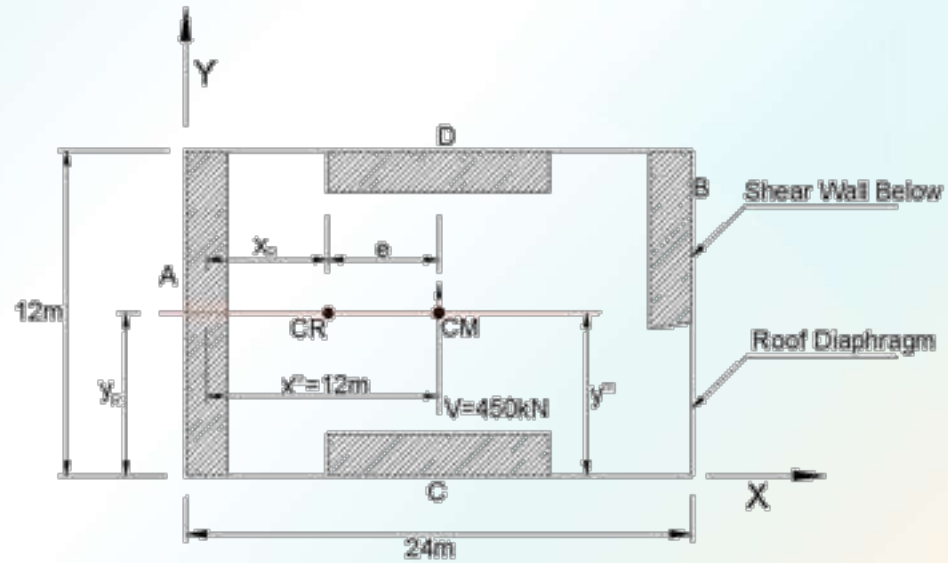
1. Eccentricity and rigidity properties

   The rigidity of the structures in the direction of applied force is the sum of the rigidities of the wall parallel to this force.

$$R = R_A + R_B$$

$$R = 54 + 18$$

$$R = 72\, kN/mm$$



Design base shear: $V = 450$ kN

Wall Rigidities: $R_A = 54$ kN/mm

$R_B = 18$ kN/mm

$R_C = R_D = 36$ kN/mm

Center of Mass: $x_m = 24$m, $y_m = 12$m

Problem 1 - Solution

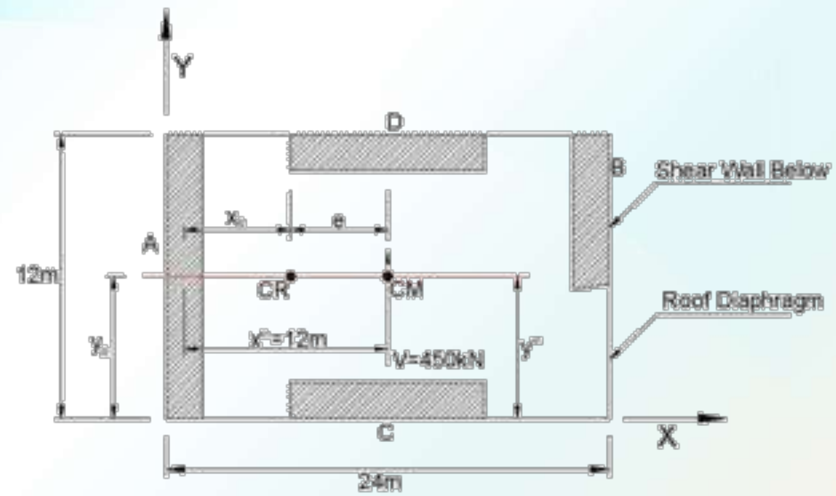
Calculations and Discussions:

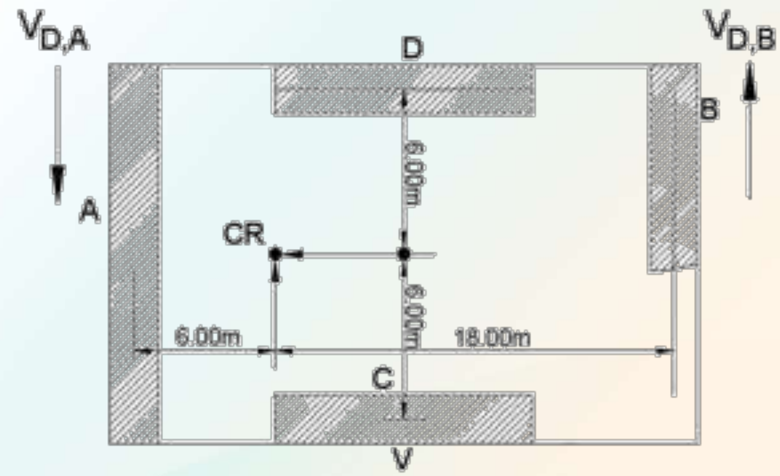
1. Eccentricity and rigidity properties

   The center of rigidity (CR) along the x and y axes are

$$x_R = \frac{R_B(24)}{R_A + R_B} = 6.0m$$

$$y_R = \frac{R_D(12)}{R_D + R_C} = 6.0m$$





Direct Shear Contribution

Problem 1 - Solution

Calculations and Discussions:

1. Eccentricity and rigidity properties

Eccentricity, $e = x_m - x_R$

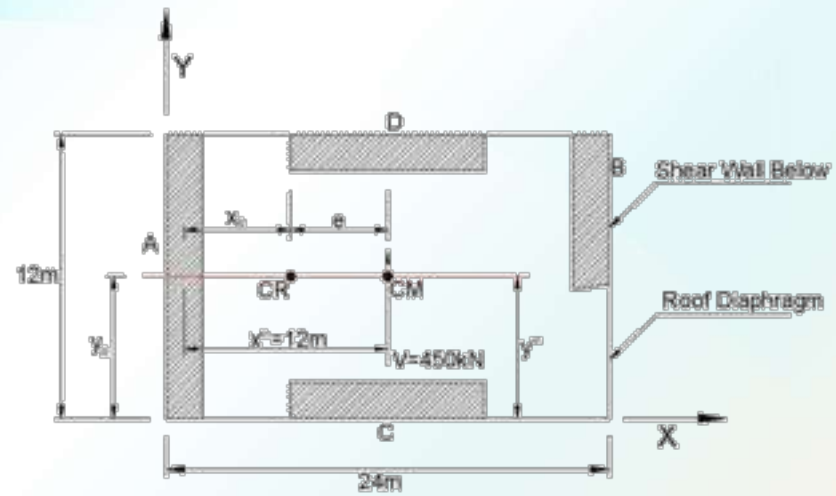
$$e = 12 - 6 = 6m$$

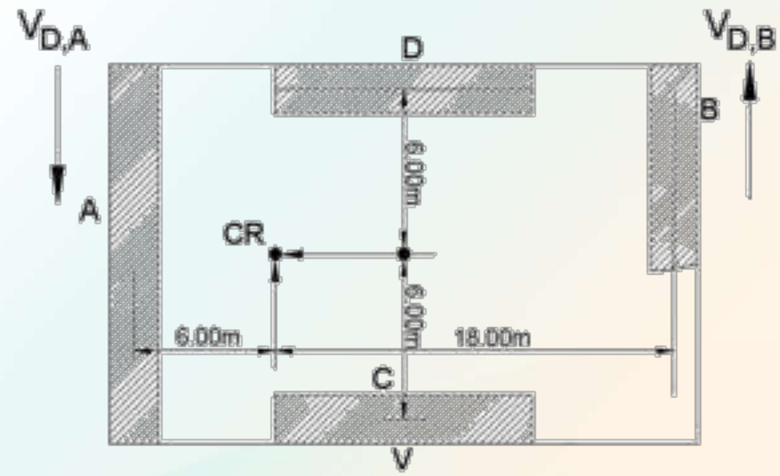
Torsional Rigidity about the center of rigidity is determined as:

$$J = R_A(6)^2 + R_B(18)^2 + R_C(6)^2 + R_D(6)^2$$

$$J = 54(6)^2 + 18(18)^2 + 36(6)^2 + 36(6)^2$$

$$J = 10{,}368(kN/mm)m^2$$





Direct Shear Contribution

FEU ALABANG FEU DILIMAN FEU TECH

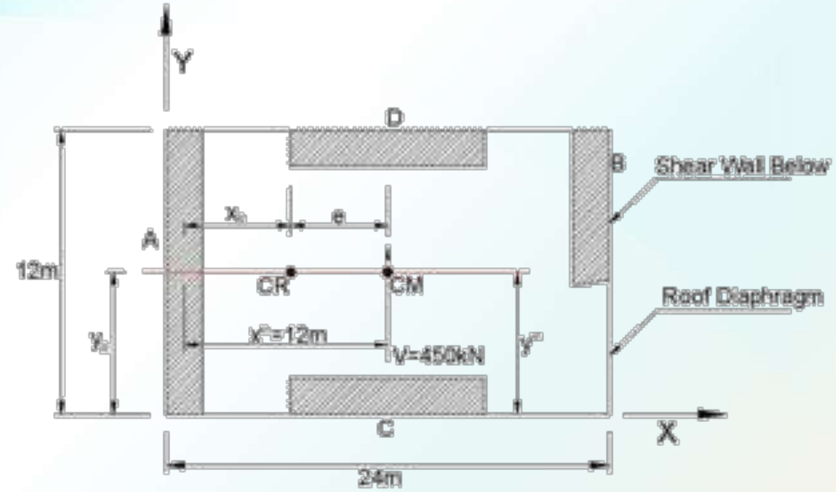
Technology Driven by Innovation

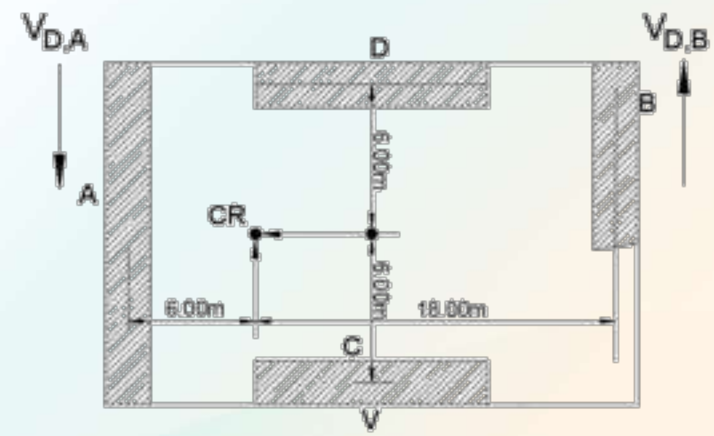
Problem 1 - Solution

Calculations and Discussions:

1. Eccentricity and rigidity properties

The seismic force V applied at the CM is equivalent to having V applied at the CR together with a counter-clockwise torsion T. With the requirements for accidental eccentricity e_{acc}, the total shear on walls A and B can be found by the addition of the direct and torsional load cases:





Direct Shear Distribution

Problem 1 - Solution

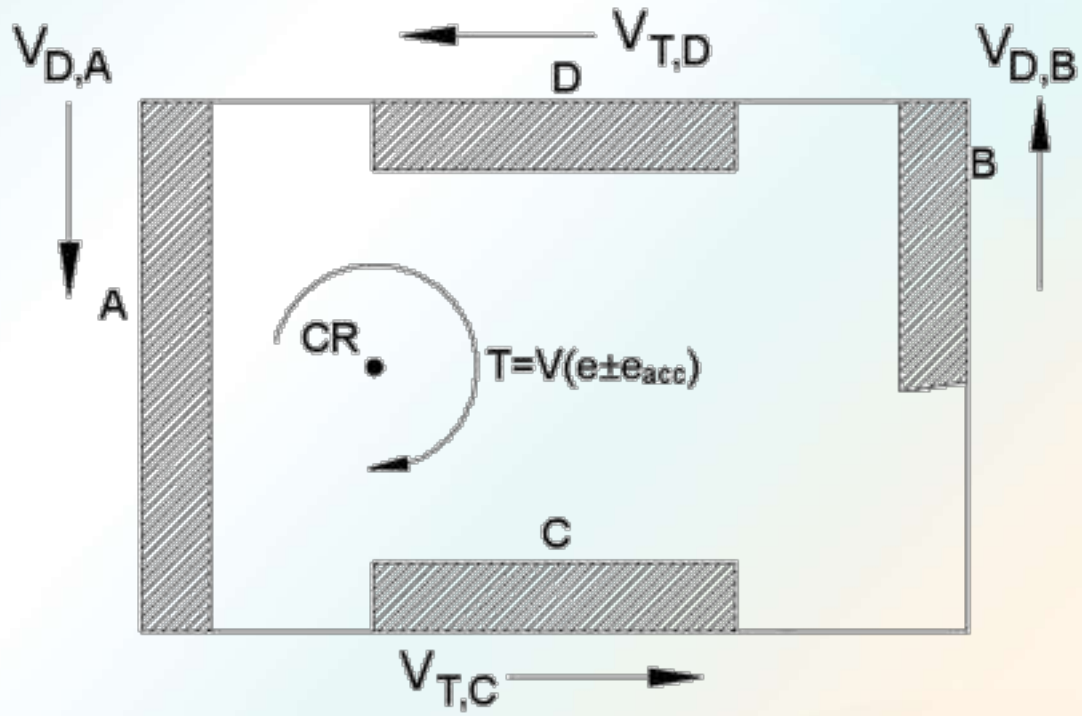
Calculations and Discussions:

2. Direct shear in walls A and B

$$V_{D,A} = \frac{R_A}{R_A + R_B} V$$

$$V_{D,A} = \frac{54}{54+18}(450)$$

$$V_{D,A} = 337.5kN$$



Torsional Shear Contribution

Problem 1 - Solution

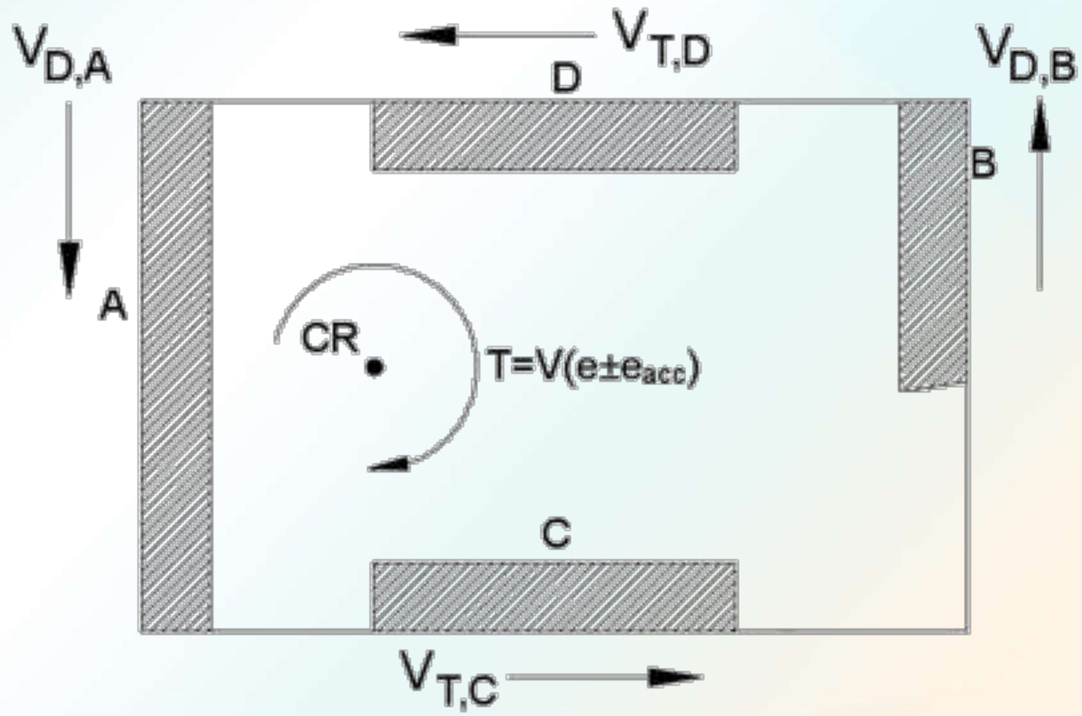
Calculations and Discussions:

2. Direct shear in walls A and B

$$V_{D,B} = \frac{R_B}{R_A + R_B} V$$

$$V_{D,B} = \frac{18}{54+18}(450)$$

$$V_{D,B} = 112.5 kN$$



Torsional Shear Contribution

Problem 1 - Solution

Calculations and Discussions:

3. Plan Irregularity Requirements

The determination of torsional irregularity, requires the evaluation of the storey drifts in walls A and B. This evaluation must include accidental torsion due to an eccentricity of 5 percent of the building dimension.

$$e_{acc} = 0.05(24)$$

$$e_{acc} = 0.96m$$

Problem 1 - Solution

Calculations and Discussions:

3. Plan Irregularity Requirements

The corresponding initial most severe torsional shears V' using e_{acc} = 0.96m are:

$$V'_{T,A} = \frac{V(e-e_{acc})(x_R)(R_A)}{J} \qquad V'_{T,A} = \frac{450(6-0.96)(6)(54)}{10,368}$$

$$V'_{T,A} = 70.875kN$$

$$V'_{T,B} = \frac{(e+e_{acc})(24-x_R)(R_B)}{J} \qquad V'_{T,B} = \frac{50(6+0.96)(18)(18)}{10,368}$$

$$V'_{T,B} = 97.875kN$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 1 - Solution

Calculations and Discussions:

3. Plan Irregularity Requirements

Note: These initial shears may need to be modified if torsional irregularity exists and the amplification factor $A_x > 1.0$.

The initial total shears are:

$$V'_A = V'_{D,A} - V'_{T,A}$$

$$V'_A = 337.5 - 70.875$$

$$V'_A = 266.625\,kN$$

Problem 1 - Solution

Calculations and Discussions:

3. Plan Irregularity Requirements

(Torsional shears may be subtracted if they are due to the reduced eccentricity $e - e_{acc}$)

$$V'_B = V'_{D,B} - V'_{T,B}$$

$$V'_B = 112.5 - 97.875$$

$$V'_B = 210.375 kN$$

Problem 1 - Solution

Calculations and Discussions:

3. Plan Irregularity Requirements

The resulting displacement Δ which for this single storey building are also drift values are:

$$\Delta'_A = \frac{V'_A}{R_A}$$

$$\Delta'_A = \frac{266.625}{54}$$

$$\Delta'_A = 4.94mm$$

$$\Delta'_B = \frac{V'_B}{R_B}$$

$$\Delta'_B = \frac{210.375}{18}$$

$$\Delta'_B = 11.69mm$$

$$\Delta'_{avg} = \frac{4.94 + 11.69}{2}$$

$$\Delta'_{avg} = 8.32mm$$

$$\Delta'_{\max} = \Delta'_B = 11.69mm$$

Problem 1 - Solution

Calculations and Discussions:

3. Plan Irregularity Requirements

The resulting displacement Δ which for this single storey building are also drift values are:

$$\Delta'_{avg} = \frac{4.94 + 11.69}{2}$$

$$\Delta'_{avg} = 8.32mm$$

$$\Delta'_{\max} = \Delta'_B = 11.69mm$$

$$\frac{\Delta'_{\max}}{\Delta'_{avg}} = \frac{11.69}{8.32}$$

$$\frac{\Delta'_{\max}}{\Delta'_{avg}} = 1.41 > 1.20$$

$$\therefore Torsional\ \ Irregularity\ \ Exist$$

Problem 1 - Solution

Calculations and Discussions:

3. Plan Irregularity Requirements

NSCP Section 208.5.7 requires accidental torsion amplification factor.

$$A_x = \left(\frac{\Delta'_{max}}{\Delta'_{avg}} \right)^2$$

$$A_x = \left[\frac{11.69}{1.2(8.32)} \right]^2$$

$$A_x = 1.37 < 3.0$$

Problem 1 - Solution

Calculations and Discussions:

4. Torsional Shear in Walls A and B

The final most severe torsional shears are determine by calculating the new accidental eccentricity and using this to determine the torsional shears

$$e_{acc} = A_x(0.96)$$

$$e_{acc} = (1.37)(0.96)$$

$$e_{acc} = 1.32m$$

Problem 1 - Solution

Calculations and Discussions:

4. Torsional Shear in Walls A and B

The final most severe torsional shears are determine by calculating the new accidental eccentricity and using this to determine the torsional shears

$$V_{T,A}=\frac{V\left(e-e_{acc}\right)\left(x_R\right)\left(R_A\right)}{J}$$

$$V_{T,B}=\frac{V\left(e+e_{acc}\right)\left(x_R\right)\left(R_B\right)}{J}$$

$$V_{T,A}=\frac{450\left(6-1.32\right)\left(6\right)\left(54\right)}{10,368}$$

$$V_{T,B}=\frac{50\left(6+1.32\right)\left(18\right)\left(18\right)}{10,368}$$

$$V_{T,A}=55.81kN$$

$$V_{T,B}=102.94kN$$

Problem 1 - Solution

Calculations and Discussions:

5. Total Shear in Walls A and B.

Total shear in each wall is the algebraic sum of the direct sum of the direct and torsional shear components.

$$V_A = V_{D,A} - V_{T,A} \qquad V_B = V_{D,B} - V_{T,B}$$

$$V_A = 337.5 - 65.81 \qquad V_B = 112.5 + 102.94$$

$$V_A = 271.69 kN \qquad V_B = 215.44 kN$$

Problem 1 - Solution

Commentary:

NSCP Section 208.5.7 requires that “the most severe load combination for each element shall be considered for the design.” This load combination involves the direct and torsional shears, and the “most severe” condition is as follows:

1. For the case where torsional shear has the same sense, and is therefore added to the direct shear, the torsional shear shall be calculated using actual eccentricity plus the accidental eccentricity so as to give the largest additive torsional shear.

Problem 1 - Solution

Commentary:

NSCP Section 208.5.7 requires that “the most severe load combination for each element shall be considered for the design.” This load combination involves the direct and torsional shears, and the “most severe” condition is as follows:

2. For the case where torsional shear has the opposite sense to that of the direct shear and is to be subtracted, the torsional shear shall be based on the actual eccentricity minus the accidental eccentricity so as to give the smallest subtractive shear.

DIRECT AND TORSIONAL FORCES AND MOMENTS

Horizontal Torsional Moments

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

208.5.1.4. Horizontal Torsional Moments

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

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

The accidental torsional moment shall be determined by assuming the mass displaced as required by Section 208.5.1.3.

208.5.1.4. Horizontal Torsional Moments

Where torsional irregularity exists, as defined in Table 208-10, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following equation:

$$A_x = \left[\frac{\delta_{\max}}{1.2\delta_{avg}} \right]^2 \qquad Eq.208-7$$

Where:

δ_{avg}= the average of the displacement at the extreme points of the structure at Level x, mm

δ_{max}= the maximum displacement at Level x, mm

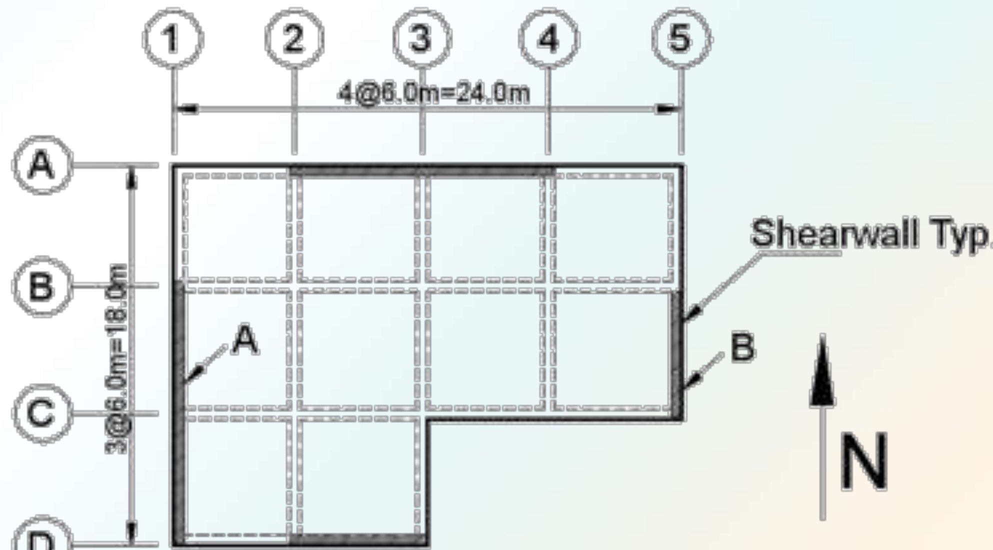
The value of A_x need not exceed 3.0.

DIRECT AND TORSIONAL FORCES AND MOMENTS

Sample Problem
Horizontal Torsional Moments

Problem 2

This example illustrates how to include the effects of accidental eccentricity in the lateral force analysis of a multi-storey building. The structure is a five-storey reinforced concrete building frame system. A three-dimensional rigid diaphragm model has been formulated per NSCP Section 208.5.1.2 for the evaluation of element actions and deformations due to prescribed loading conditions. Shear walls resist lateral forces in both directions.

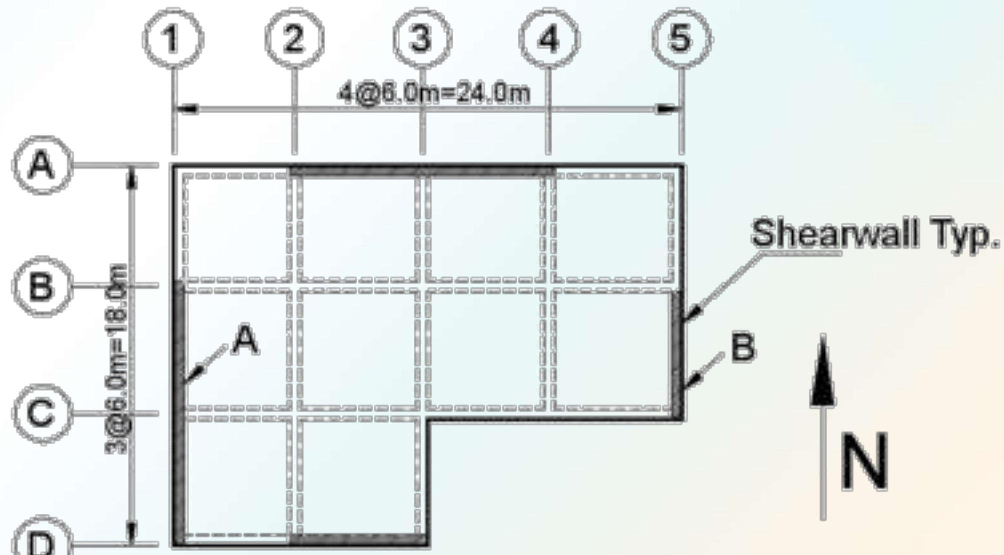


Floor Plan @ Level 2

Problem 2

The lateral seismic forces F_x in the north-south direction, structure dimensions and accidental eccentricity e_{acc} for each level x are given below:

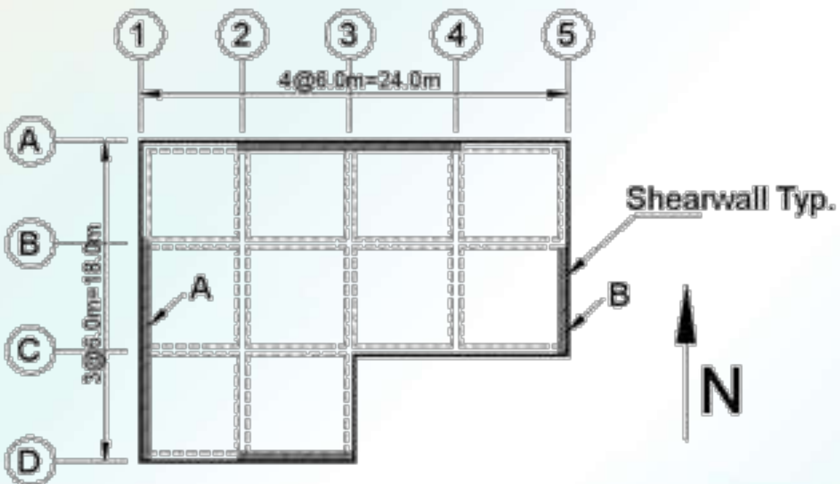
Level x	F_x	L_x	x_{cx}	$E_{acc}=0.05L_x$
5	490 kN	24.0 m	7.38 m	±0.96 m
4	370 kN	24.0 m	7.65 m	±0.96 m
3	290 kN	24.0 m	8.48 m	±0.96 m
2	190 kN	24.0 m	9.24 m	±0.96 m
1	105 kN	24.0 m	9.60 m	±0.96 m



Floor Plan @ Level 2

Problem 2

In addition, for the given lateral seismic forces F_x a computer analysis provides the following results for the second storey. Separate values are given for the application of the forces F_x at the centers of mass and the $\pm 0.05Lx$ displacements as required by NSCP Section 208.5.6.



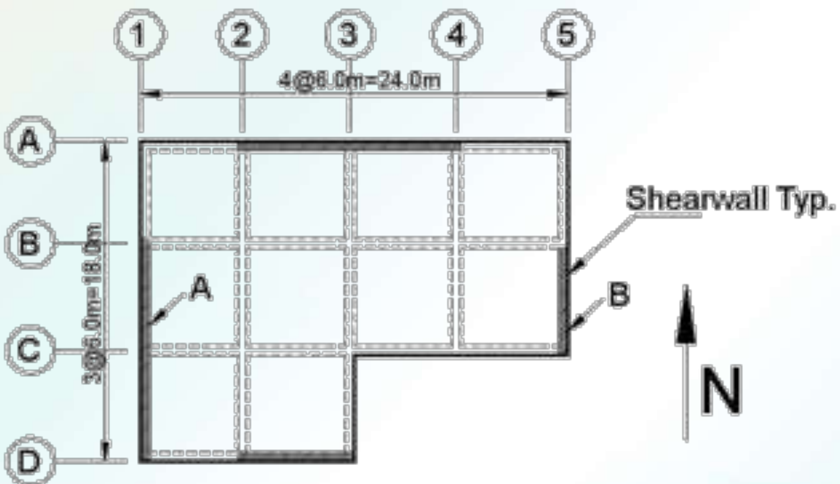
Floor Plan @ Level 2

	Force F_x Position		
	x_{c2}	$X_{c2} - e_{acc}$	$X_{c2} + e_{acc}$
Wall Shear V_A	825 kN	875 kN	775 kN
Wall Shear V_B	510 kN	465 kN	560 kN
Storey Drift $\Delta\delta_A$	8.75 mm	9.25 mm	8.25 mm
Storey Drift $\Delta\delta_B$	15.50 mm	14.00 mm	17.00 mm
Level 2 Displacement $\Delta\delta_A$	20.00 mm	21.25 mm	18.75 mm
Level 2 Displacement $\Delta\delta_B$	32.75 mm	29.50 mm	36.00 mm

Problem 2

For the second storey, find the following:

1. Maximum force in shear walls A and B.
2. Check if torsional irregularity exists.
3. Determine the amplification factor A_x.
4. New accidental torsion eccentricity.



Floor Plan @ Level 2

	Force F_x Position		
	x_{c2}	$X_{c2} - e_{acc}$	$X_{c2} + e_{acc}$
Wall Shear V_A	825 kN	875 kN	775 kN
Wall Shear V_B	510 kN	465 kN	560 kN
Storey Drift $\Delta\delta_A$	8.75 mm	9.25 mm	8.25 mm
Storey Drift $\Delta\delta_B$	15.50 mm	14.00 mm	17.00 mm
Level 2 Displacement $\Delta\delta_A$	20.00 mm	21.25 mm	18.75 mm
Level 2 Displacement $\Delta\delta_B$	32.75 mm	29.50 mm	36.00 mm

Problem 2 - Solution

Calculations and Discussion:

1. Maximum force in shear walls A and B.

   The maximum force in each shear wall is a result of direct shear and the contribution due to accidental torsion. From the above table, it is determined that

$$V_A = 875kN$$

$$V_B = 560kN$$

	Force F_x Position		
	x_{c2}	$X_{c2} - e_{acc}$	$X_{c2} + e_{acc}$
Wall Shear V_A	825 kN	875 kN	775 kN
Wall Shear V_B	510 kN	465 kN	560 kN
Storey Drift $\Delta\delta_A$	8.75 mm	9.25 mm	8.25 mm
Storey Drift $\Delta\delta_B$	15.50 mm	14.00 mm	17.00 mm
Level 2 Displacement $\Delta\delta_A$	20.00 mm	21.25 mm	18.75 mm
Level 2 Displacement $\Delta\delta_B$	32.75 mm	29.50 mm	36.00 mm

Problem 2 - Solution

Calculations and Discussion:

2. Check if torsional irregularity exists.

The building is L-shaped in plan. This suggests that it may have a torsion irregularity Type 1 of NSCP Table 208-10. The following is a check of the storey drifts.

$$\Delta\delta_{\max} = 17mm$$

$$\Delta\delta_{avg} = \frac{17 + 8.25}{2}$$

$$\Delta\delta_{avg} = 12.63mm$$

$$\frac{\Delta\delta_{\max}}{\Delta\delta_{avg}} = \frac{17}{12.63}$$

$$\frac{\Delta\delta_{\max}}{\Delta\delta_{avg}} = 1.35 > 1.2$$

$$\therefore Torsional\ Irregularity\ Exists$$

	Force F_x Position		
	x_{c2}	$X_{c2} - e_{acc}$	$X_{c2} + e_{acc}$
Wall Shear V_A	825 kN	875 kN	775 kN
Wall Shear V_B	510 kN	465 kN	560 kN
Storey Drift $\Delta\delta_A$	8.75 mm	9.25 mm	8.25 mm
Storey Drift $\Delta\delta_B$	15.50 mm	14.00 mm	17.00 mm
Level 2 Displacement $\Delta\delta_A$	20.00 mm	21.25 mm	18.75 mm
Level 2 Displacement $\Delta\delta_B$	32.75 mm	29.50 mm	36.00 mm

Problem 2 - Solution

Calculations and Discussion:

3. Determine the amplification factor A_x

Because a torsional irregularity exists, NSCP Section 208.5.7 requires that the second storey accidental eccentricity be amplified by the following factor:

$$A_x = \left[\frac{\delta_{\max}}{1.2\delta_{avg}}\right]^2$$

Where $\delta_{max} = \delta_B$ = 36mm

	Force F_x Position		
	x_{c2}	X_{c2} - e_{acc}	X_{c2} + e_{acc}
Wall Shear V_A	825 kN	875 kN	775 kN
Wall Shear V_B	510 kN	465 kN	560 kN
Storey Drift $\Delta\delta_A$	8.75 mm	9.25 mm	8.25 mm
Storey Drift $\Delta\delta_B$	15.50 mm	14.00 mm	17.00 mm
Level 2 Displacement $\Delta\delta_A$	20.00 mm	21.25 mm	18.75 mm
Level 2 Displacement $\Delta\delta_B$	32.75 mm	29.50 mm	36.00 mm

Problem 2 - Solution

Calculations and Discussion:

3. Determine the amplification factor A_x

The average storey displacement is computed as

$$\delta_{avg} = \frac{36+18.75}{2}$$

$$\delta_{avg} = 27.38mm$$

$$A_x = \left[\frac{36}{1.20(27.38)}\right]^2$$

$$A_x = 1.20$$

	Force F_x Position		
	x_{c2}	$X_{c2} - e_{acc}$	$X_{c2} + e_{acc}$
Wall Shear V_A	825 kN	875 kN	775 kN
Wall Shear V_B	510 kN	465 kN	560 kN
Storey Drift $\Delta\delta_A$	8.75 mm	9.25 mm	8.25 mm
Storey Drift $\Delta\delta_B$	15.50 mm	14.00 mm	17.00 mm
Level 2 Displacement $\Delta\delta_A$	20.00 mm	21.25 mm	18.75 mm
Level 2 Displacement $\Delta\delta_B$	32.75 mm	29.50 mm	36.00 mm

Problem 2 - Solution

Calculations and Discussion:

4. New Accidental Torsion Eccentricity

   Since A_2 (i.e. A_x for the second storey) is greater than unity, a second analysis for torsion must be done using the new accidental eccentricity.

$$e_{acc} = 1.20(0.96)$$

$$e_{acc} = 1.152m$$

Commentary:

Example calculations were given for the second storey. In practice, each storey requires an evaluation of the most severe element actions and a check for the torsional irregularity condition.

Problem 2 - Solution

Calculations and Discussion:

Commentary:

If torsional irregularity exists and A_x is greater than one at any level (or levels), then a second torsional analysis must be done using the new accidental eccentricities. However, it is not necessary to find the resulting new A_x values and repeat the process a second or third time (until the A_x iterates to a constant or reaches the limit of 3.0). The results of the first analysis with the use of A_x are sufficient for design purposes.

While this example involved the case of wall shear evaluation, the same procedure applies to the determination of the most severe element actions for any other lateral force-resisting system having rigid diaphragms.

Problem 2 - Solution

Calculations and Discussion:

Commentary:

When the dynamic analysis method of NSCP Section 208.6.5.1 is used, rather than static force procedure of NSCP Section 208.5.2, the following equivalent static force option may be used in lieu of performing the two extra dynamic analyses for mass positions at $x_{cx} \pm (0.05L_x)$ as per NSCP Section 208.6.5.6 (Torsion).

1. Perform the dynamic analysis with masses at the center of mass, and reduce results to those corresponding to the required design base shear.
2. Determine the F_x forces for the required design base shear, and apply pure torsion couple loads $F_x\ (0.05L_x)$ at each level x. Then add the absolute value of these couple load results to those of the reduced dynamic analysis.

DIRECT AND TORSIONAL FORCES AND MOMENTS

Storey Displacement

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Section 208.6.2. Storey Displacement

THE MATHEMATICAL MODEL OF THE PHYSICAL STRUCTURE SHALL INCLUDE ALL ELEMENTS OF THE LATERAL-FORCE-RESISTING SYSTEM. THE MODEL SHALL ALSO INCLUDE THE STIFFNESS AND STRENGTH OF ELEMENTS, WHICH ARE SIGNIFICANT TO THE DISTRIBUTION OF FORCES, AND SHALL REPRESENT THE SPATIAL DISTRIBUTION OF MASS AND STIFFNESS OF THE STRUCTURE. IN ADDITION, THE MODEL SHALL COMPLY WITH THE FOLLOWING:

Section 208.6.2. Storey Displacement

1. STIFFNESS PROPERTIES OF REINFORCED CONCRETE AND MASONRY ELEMENTS SHALL CONSIDER THE EFFECTS OF CRACKED SECTIONS.

2. FOR STEEL MOMENT FRAME SYSTEMS, THE CONTRIBUTION OF PANEL ZONE DEFORMATIONS TO OVERALL STORY DRIFT SHALL BE INCLUDED.

Section 208.6.4.

DRIFT OR HORIZONTAL DISPLACEMENTS OF THE STRUCTURE SHALL BE COMPUTED WHERE REQUIRED BY THIS CODE. FOR BOTH ALLOWABLE STRESS DESIGN AND STRENGTH DESIGN, THE MAXIMUM INELASTIC RESPONSE DISPLACEMENT, ΔM , OF THE STRUCTURE CAUSED BY THE DESIGN BASIS GROUND MOTION SHALL BE DETERMINED IN ACCORDANCE WITH THIS SECTION. THE DRIFTS CORRESPONDING TO THE DESIGN SEISMIC FORCES OF SECTION 208.5.2.1 OR SECTION 208.5.3.5, ΔS ,SHALL BE DETERMINED IN ACCORDANCE WITH SECTION.5.3.5,ΔS , SHALL BE DETERMINED IN ACCORDANCE WITH SECTION 208.6.4.1. TO DETERMINE ΔM, THESE DRIFTS SHALL BE AMPLIFIED IN ACCORDANCE WITH SECTION 208.6.4.2.

Section 208.6.4.1

A STATIC, ELASTIC ANALYSIS OF THE LATERAL FORCE-RESISTING SYSTEM SHALL BE PREPARED USING THE DESIGN SEISMIC FORCES FROM SECTION 208.5.2.1. ALTERNATIVELY, DYNAMIC ANALYSIS MAY BE PERFORMED IN ACCORDANCE WITH SECTION 208.6 WHERE ALLOWABLE STRESS DESIGN IS USED AND WHERE DRIFT IS BEING COMPUTED, THE LOAD COMBINATIONS OF SECTION 203.3 SHALL BE USED. THE MATHEMATICAL MODEL SHALL COMPLY WITH SECTION 208.6.4. THE RESULTING DEFORMATIONS, DENOTED AS ΔS SHALL BE DETERMINED AT ALL CRITICAL LOCATIONS IN THE STRUCTURE. CALCULATED DRIFT SHALL INCLUDE TRANSLATIONAL AND TORSIONAL DEFLECTIONS

Working Equations:

$$\Delta_x = \Delta_{x-1} + \frac{\sum_{x=1}^{n} F_x}{K_{Total}}$$

Δ_x= Storey Displacement at Level x

Δ_{x-1}= Storey Displacement below Level x

$\sum F_x$= Summation of Vertical Lateral Forces from Level x to Level n

K= Total Stiffness of Columns in N-S/W-E Direction

DIRECT AND TORSIONAL FORCES AND MOMENTS

Sample Problems in Storey Displacement

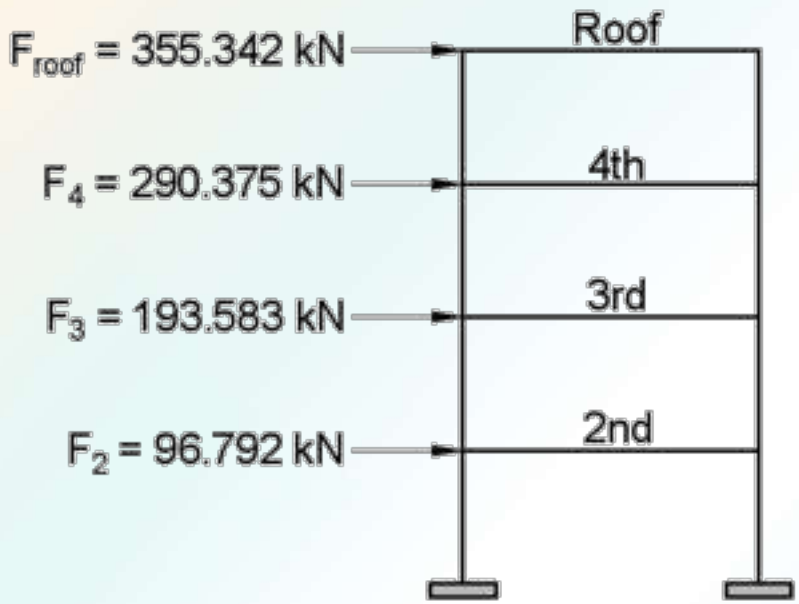
FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Problem 3

The vertical distribution of 4-storey with Roof Deck is shown below:

Vertical Distribution of Forces

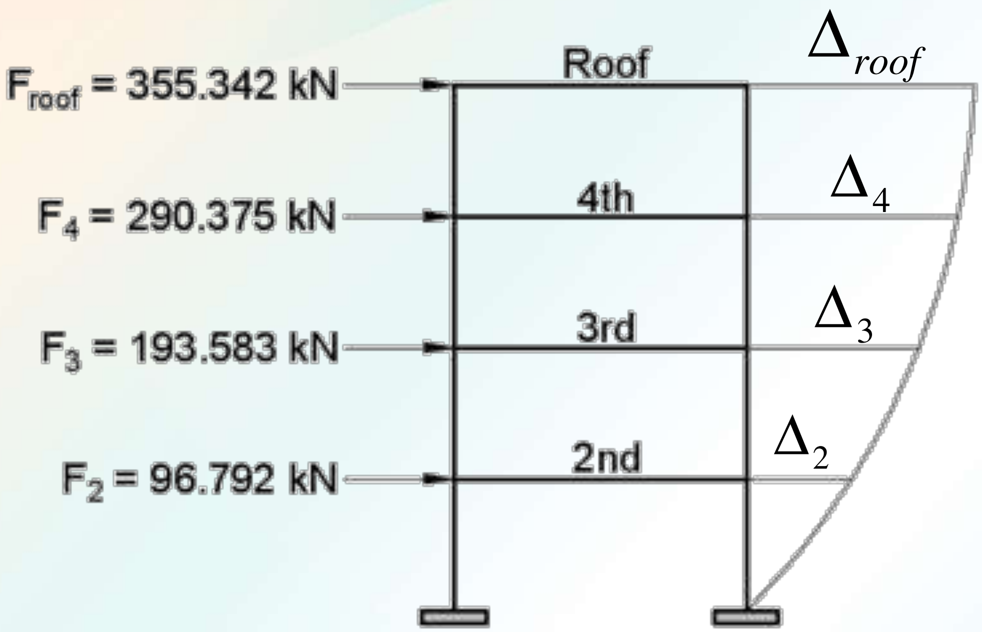


Total Stiffness (K) = 413.983 (N-S Direction) and 192.678 (W-E Direction)

Determine the displacement in each level.

Problem 3 - Solution

Given: Total Stiffness (K) = 413.983 (N-S Direction) and 192.678 (W-E Direction)



A. Solve for the displacement in N-S Direction

$$\Delta_G = 0$$

$$\Delta_2 = 0 + \frac{96.792 + 193.583 + 290.375 + 355.342}{413.983}$$

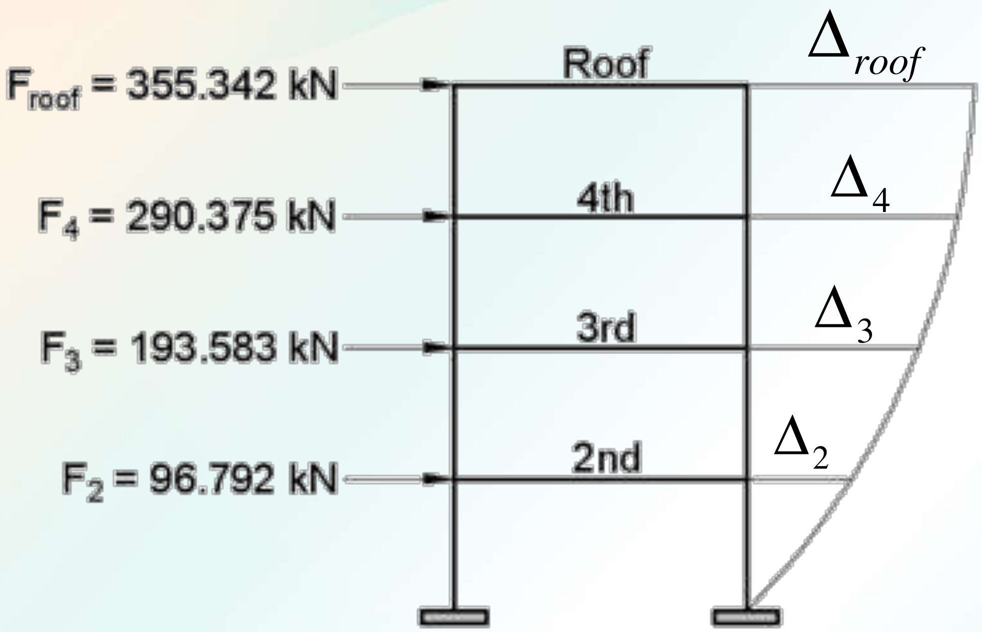
$$\Delta_2 = 2.261mm$$

$$\Delta_3 = 2.261 + \frac{193.583 + 290.375 + 355.342}{413.983}$$

$$\Delta_3 = 4.288mm$$

Problem 3 - Solution

Given: Total Stiffness (K) = 413.983 (N-S Direction) and 192.678 (W-E Direction)



A. Solve for the displacement in N-S Direction

$$\Delta_4 = 4.288 + \frac{290.375 + 355.342}{413.983}$$

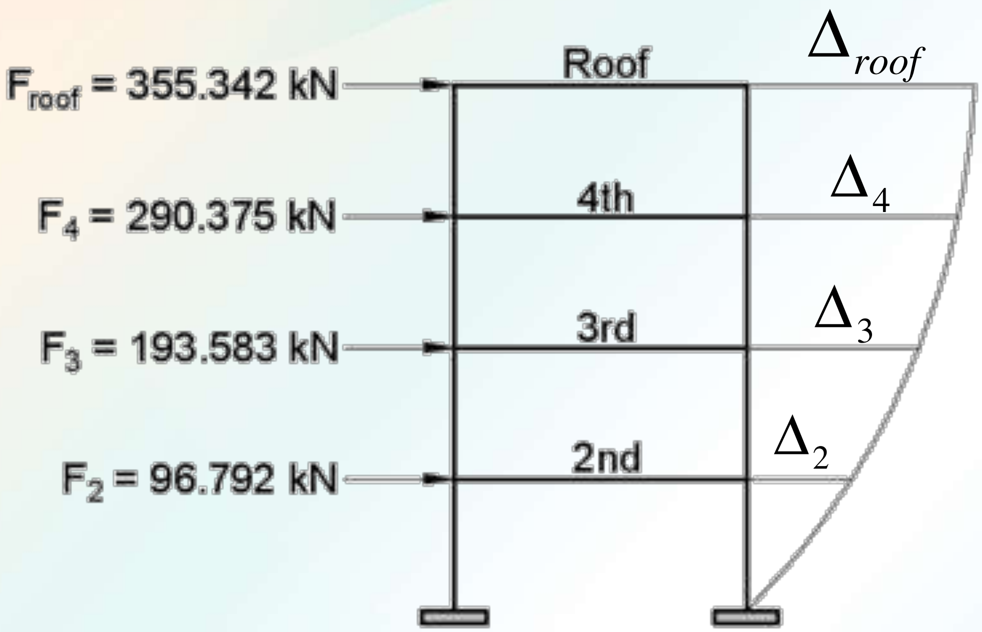
$$\Delta_4 = 5.848mm$$

$$\Delta_{roof} = 5.848 + \frac{355.342}{413.983}$$

$$\Delta_{roof} = 6.706mm$$

Problem 3 - Solution

Given: Total Stiffness (K) = 413.983 (N-S Direction) and 192.678 (W-E Direction)



B. Solve for the displacement in W-E Direction

$$\Delta_G = 0$$

$$\Delta_2 = 0 + \frac{96.792 + 193.583 + 290.375 + 355.342}{192.678}$$

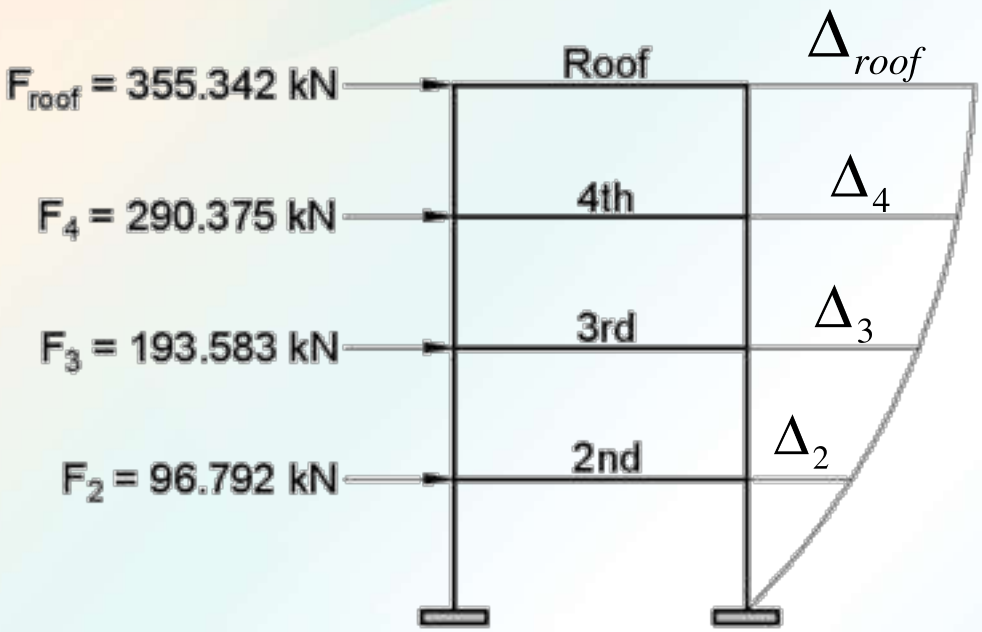
$$\Delta_2 = 4.858\,mm$$

$$\Delta_3 = 4.858 + \frac{193.583 + 290.375 + 355.342}{192.678}$$

$$\Delta_3 = 9.214\,mm$$

Problem 3 - Solution

Given: Total Stiffness (K) = 413.983 (N-S Direction) and 192.678 (W-E Direction)



B. Solve for the displacement in W-E Direction

$$\Delta_4 = 9.214 + \frac{290.375 + 355.342}{192.678}$$

$$\Delta_4 = 12.565mm$$

$$\Delta_{roof} = 12.565 + \frac{355.342}{192.678}$$

$$\Delta_{roof} = 14.409mm$$

Q&A SESSION

ASK ANY QUESTION RELATED TO OUR TOPIC FOR TODAY.

HOMEWORK

Do the Exercises about Base Shear Computations

Reference:

Association of Structural Engineers of the Philippines (2016). *National Structural Code of the Philippines 2015 (7th Edition) Volume 1 Buildings, Towers and Other Vertical Structures.*

Association of Structural Engineers of the Philippines (2003). *ASEP Earthquake Design Manual 2003 Volume 1: Code Provisions for Lateral Forces.*

CEELECT1
Earthquake Engineering

Computation of the Structures' Shears, Moments and Reactions due to Earthquake

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Module 5

OBJECTIVES

- At the end of the chapter, the learner should be able to:
  - *Analyze frames considering their individual lateral forces.*
  - *Determine the beam and column shears and moments.*
  - *Draw the shear and moments of each frames.*

COMPUTATION OF THE STRUCTURES' SHEARS, MOMENTS AND REACTIONS DUE TO EARTHQUAKE

Lateral Loads on Building Frames: Portal Method

Lateral Loads on Building Frames: Portal Method

The action of lateral loads on portal frames and found that for a frame fixed supported at its base, points of inflection occur at approximately the center of each girder and column and the columns carry equal shear loads. A building bent deflects in the same way as a portal frame, Fig. 7–12*a*, and therefore it would be appropriate to assume inflection points occur at the center of the columns and girders. If we consider each bent of the frame to be composed of a series of portals, Fig. 7–12*b*, then as a further assumption, the *interior columns* would represent the effect of *two portal columns* and would therefore carry twice the shear V as the two exterior columns.

Lateral Loads on Building Frames: Portal Method

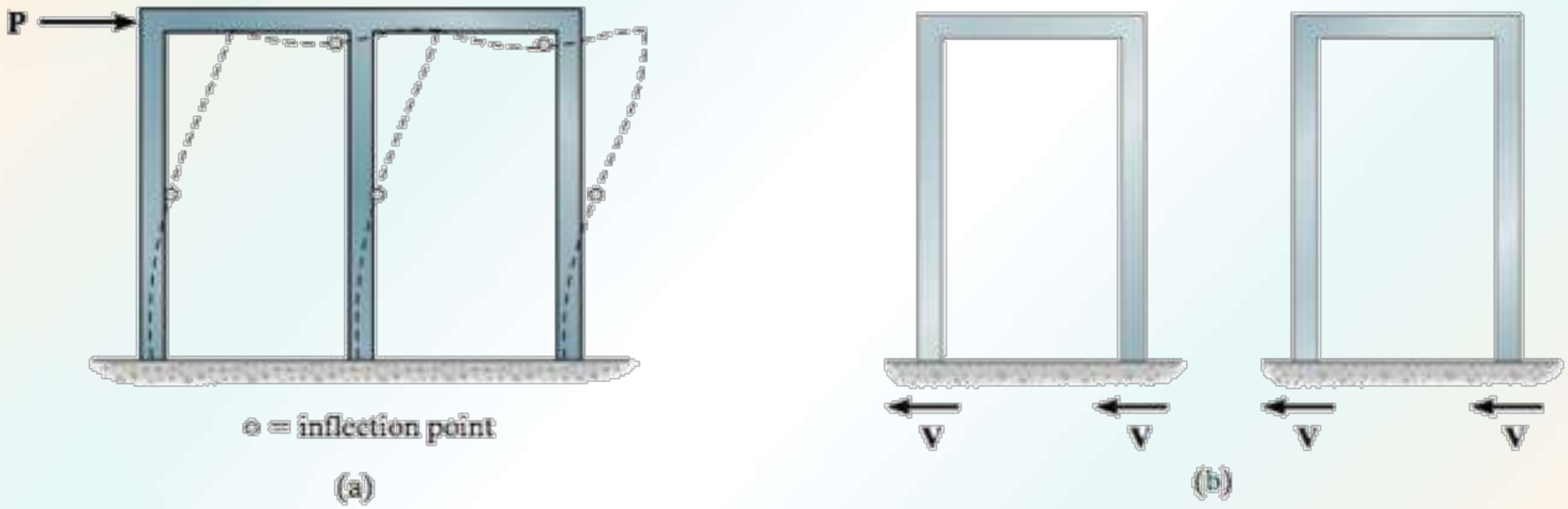


Fig. 7–12

Lateral Loads on Building Frames: Portal Method

In summary, the portal method for analyzing fixed-supported building frames requires the following assumptions:

1. A hinge is placed at the center of each girder, since this is assumed to be a point of zero moment.

2. A hinge is placed at the center of each column, since this is assumed to be a point of zero moment.

3. At a given floor level the shear at the interior column hinges is twice that at the exterior column hinges, since the frame is considered to be a superposition of portals.

Lateral Loads on Building Frames: Portal Method

These assumptions provide an adequate reduction of the frame to one that is statically determinate yet stable under loading.

By comparison with the more exact statically indeterminate analysis, *the* ***portal method*** *is most suitable for buildings having low elevation* and *uniform framing*. The reason for this has to do with the structure's action under load. In this regard, *consider the frame as acting like a cantilevered beam* that is fixed to the ground. Recall from mechanics of materials that *shear resistance* becomes more important in the design of *short* beams, whereas *bending* is more important if the beam is *long*. The portal method is based on the assumption related to shear as stated in item 3 above.

Lateral Loads on Building Frames: Portal Method

The portal method of analysis can be used to (approximately) perform a lateral-load analysis of this single-story frame.

COMPUTATION OF THE STRUCTURES' SHEARS, MOMENTS AND REACTIONS DUE TO EARTHQUAKE

Lateral Loads on Building Frames: Cantilever Method

Lateral Loads on Building Frames: Cantilever Method

The cantilever method is based on the same action as a long cantilevered beam subjected to a transverse load. It may be recalled from mechanics of materials that such a loading causes a bending stress in the beam that varies linearly from the beam's neutral axis, Fig. 7–15*a*. In a similar manner, the lateral loads on a frame tend to tip the frame over, or cause a rotation of the frame about a "neutral axis" lying in a horizontal plane that passes through the columns between each floor. To counteract this tipping, the axial forces (or stress) in the columns will be tensile on one side of the neutral axis and compressive on the other side, Fig. 7–15*b*. Like the cantilevered beam, it therefore seems reasonable to assume this axial stress has a linear variation from the centroid of the column areas or neutral axis. *The* ***cantilever method*** *is therefore appropriate if the frame is tall and slender, or has columns with different cross-sectional areas.*

Lateral Loads on Building Frames: Cantilever Method

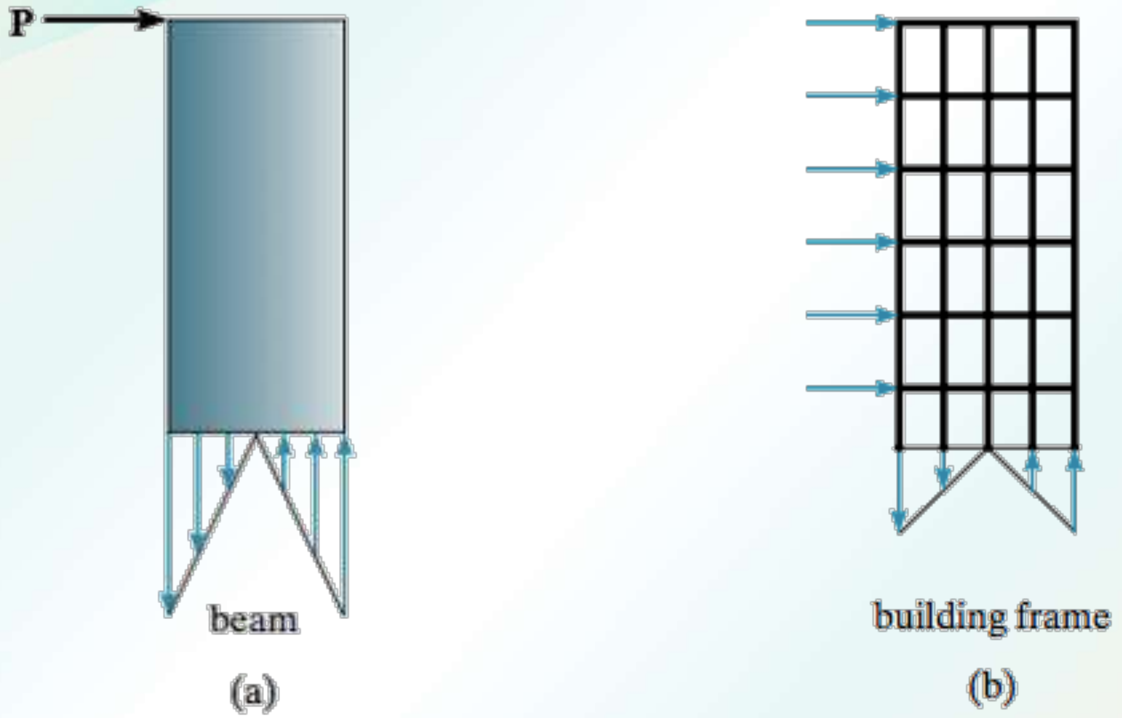


Fig. 7–15

Lateral Loads on Building Frames: Cantilever Method

In summary, using the cantilever method, the following assumptions apply to a fixed-supported frame.

1. A hinge is placed at the center of each girder, since this is assumed to be a point of zero moment.
2. A hinge is placed at the center of each column, since this is assumed to be a point of zero moment.
3. The axial *stress* in a column is proportional to its distance from the centroid of the cross-sectional areas of the columns at a given floor level. Since stress equals force per area, then in the special case of the *columns having equal cross-sectional areas,* the *force* in a column is also proportional to its distance from the centroid of the column areas.

Lateral Loads on Building Frames: Cantilever Method

These three assumptions reduce the frame to one that is both stable and statically determinate.

The following examples illustrate how to apply the cantilever method to analyze a building bent.

The building framework has rigid connections. A lateral-load analysis can be performed (approximately) by using the cantilever method of analysis.

COMPUTATION OF THE STRUCTURES' SHEARS, MOMENTS AND REACTIONS DUE TO EARTHQUAKE

Lateral Loads on Building Frames: Factor Method

Factor Method

- *Depend on certain assumptions regarding the elastic action of the structure which make possible an approximate slope-deflection analysis of the bent.*

- *More accurate than either the portal or the cantilever method.*

Note:

If K is not given; it can be computed by

$$K = I / L$$

It is not necessary to use absolute values of K, since the stress depend upon the relative stiffness of the members the bent. It is however, necessary for the K values for the various members to be in correct ratio to each other.

Steps in carrying out Factor Method

1. For each joint, compute the GIRDER FACTOR g by the following relation:

$$g = \Sigma\, Kc \,/\, \Sigma\, K$$

Σ Kc is the sum of the K values meeting at that joint.

Σ K is the sum of all K values for all members of that joint.

Write each value obtained at the near end of each girder meeting at the joint where it is computed.

2. For each joint, compute the COLUMN FACTOR c by the following relation:

$$c = 1 - g$$

Write each value of c thus obtained at the near end of each column meeting at the joint where it is computed.

For the fixed column bases of the first story, take

$$c = 1$$

3. From steps 1 and 2, there is a number at each end of each member of the bent.

To each of these numbers, *add half of the number at the other end of the member*.

4. Multiply each sum obtained from step 3 by the K value for the member in which the sum occurs. For columns, call this product the *column moment factor C*; for girders, call this product the girder moment factor G.

5. The column moment factors C from step 4 are actually the approximate relative values for column end moments for the story in which they occur.

The sum of the column end moments in a given story can be shown by static's to equal the horizontal shear on that storey multiplied by the story height

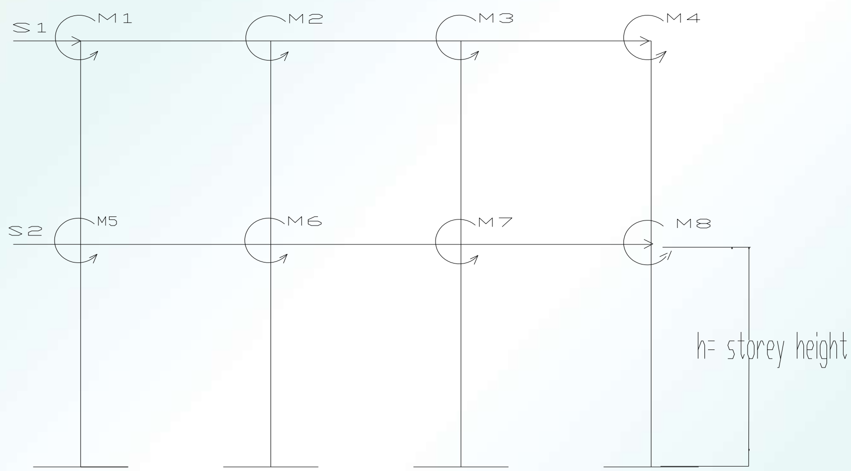
$$M = дC$$

M = column end moment

C = column end factor

Д = constant

EXAMPLE:



Actual Moment $M = Д\,C$ where C= common value for all columns

$$(S_1 + S_2 + S_3 + S_4)(h) = M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_7 + M_8$$

$$(S_1 + S_2 + S_3 + S_4)(h) = Д(\Sigma C)$$

6. The girder moment factors G from step 4 are actually approximate relative values for girder end moments for each joint.

The sum of the girder end moments at each joint is equal, by static's, to the sum of the column end moments at that joint, which can be obtain from step 5.

Hence, the girder moment factors G can be converted into girder end moments, by direct proportion, for each joint.

Determination of Girder Moments:

since the girder moment factors are relative value of girder end moments for a given joint, we can say that:

$$M = \beta(G)$$

Where:

M = girder moment

β = girder constant

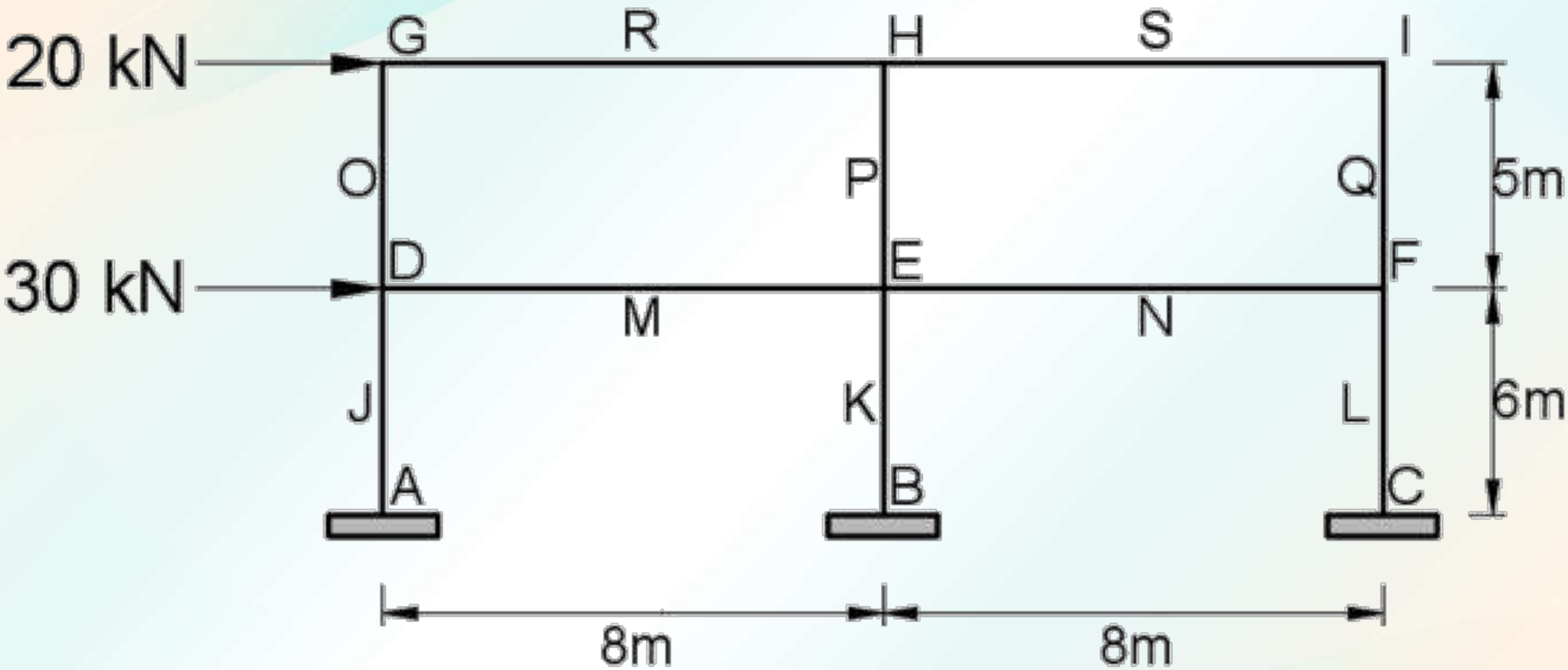
G = girder factor

Moreover, since at any joint the sum of the girder moments equals the sum of the column moments, β can be evaluated from the relation.

COMPUTATION OF THE STRUCTURES' SHEARS, MOMENTS AND REACTIONS DUE TO EARTHQUAKE

Sample Problems in Portal Method

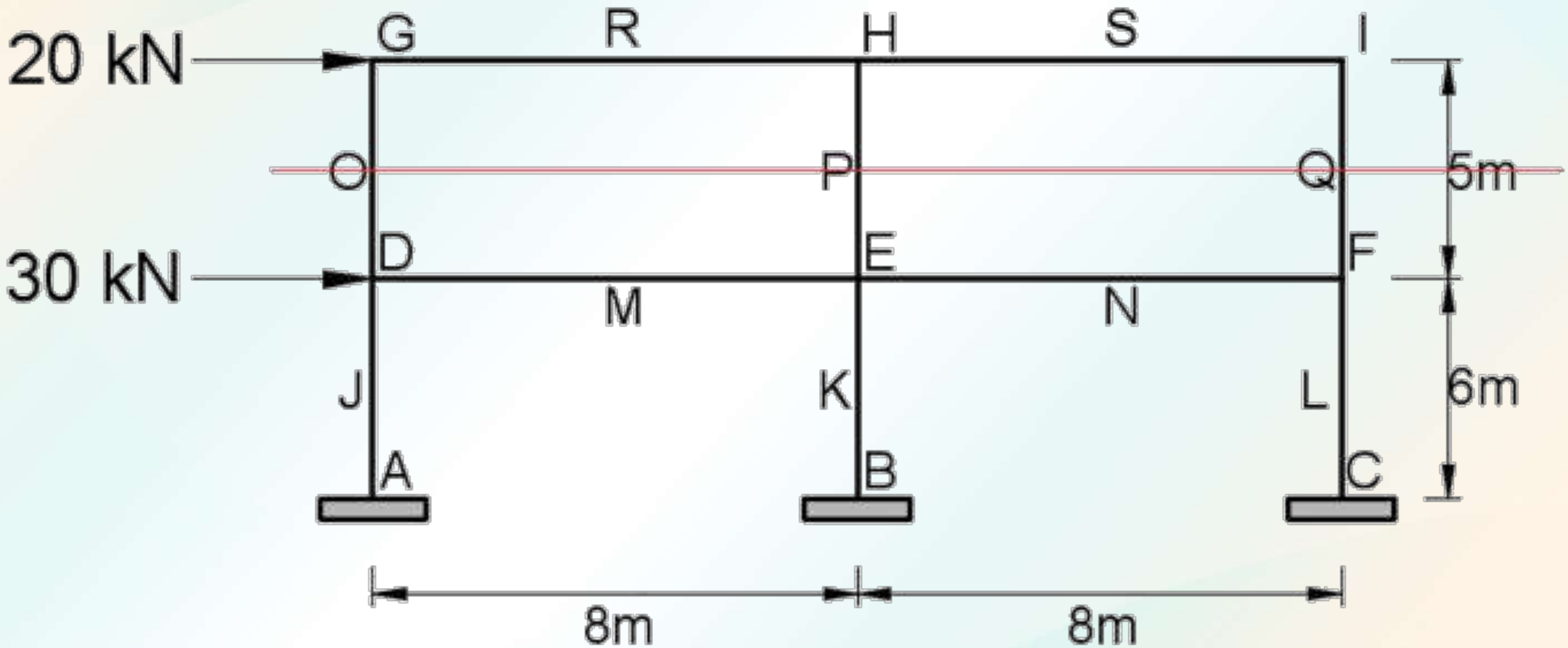
Problem 1



Solve the beam and column moments using Portal Method.

Problem 1 - Solution

Isolate the Upper Level:



Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Problem 1 - Solution

Isolate the Upper Level:

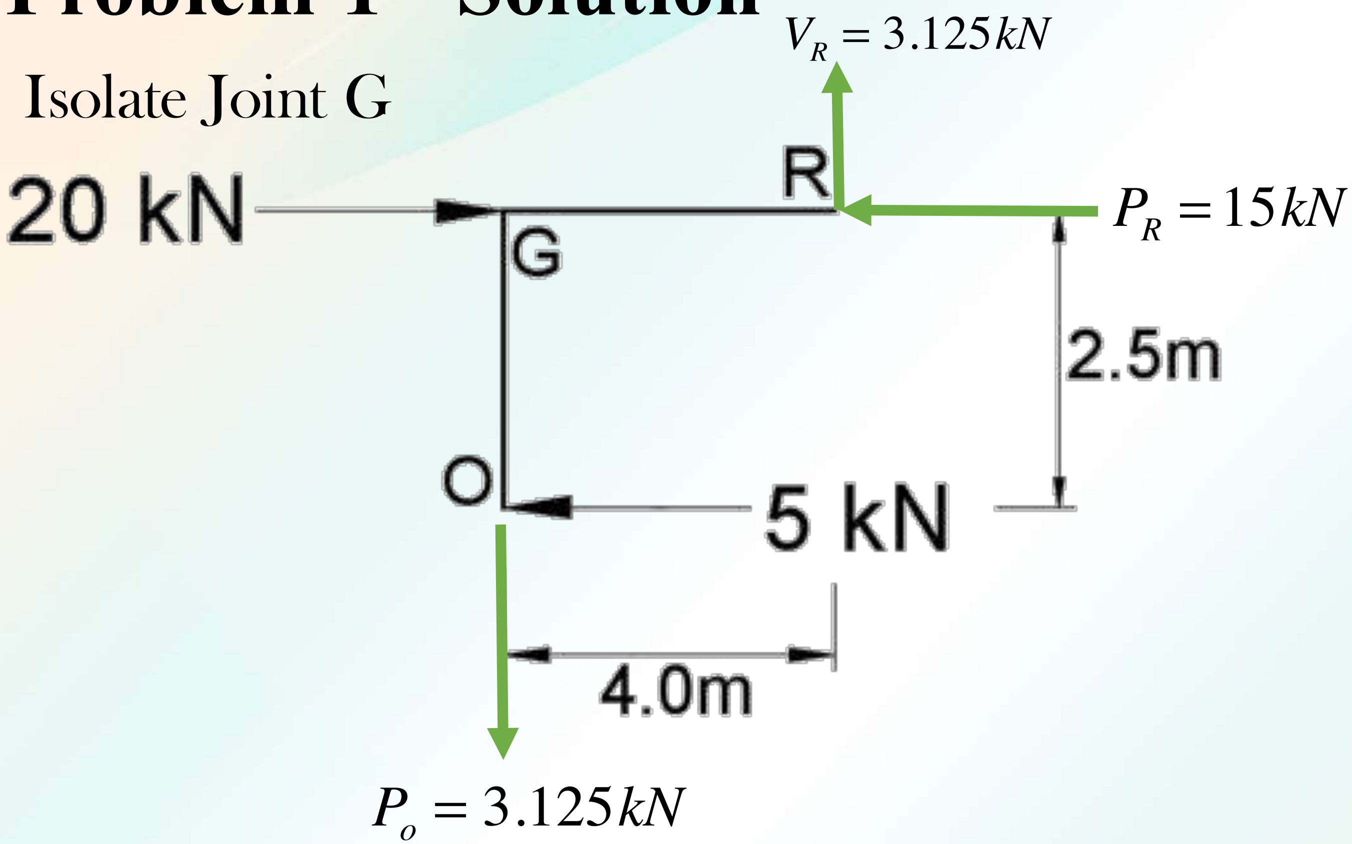
$$\sum F_H = 0$$

$$20 = V + 2V + V$$

$$V = 5kN$$

Problem 1 - Solution

Isolate Joint G



Use 3ME Equations

$$\sum M_R = 0$$

$$-P_o(4) + 5(2.5) = 0$$

$$P_o = 3.125kN$$

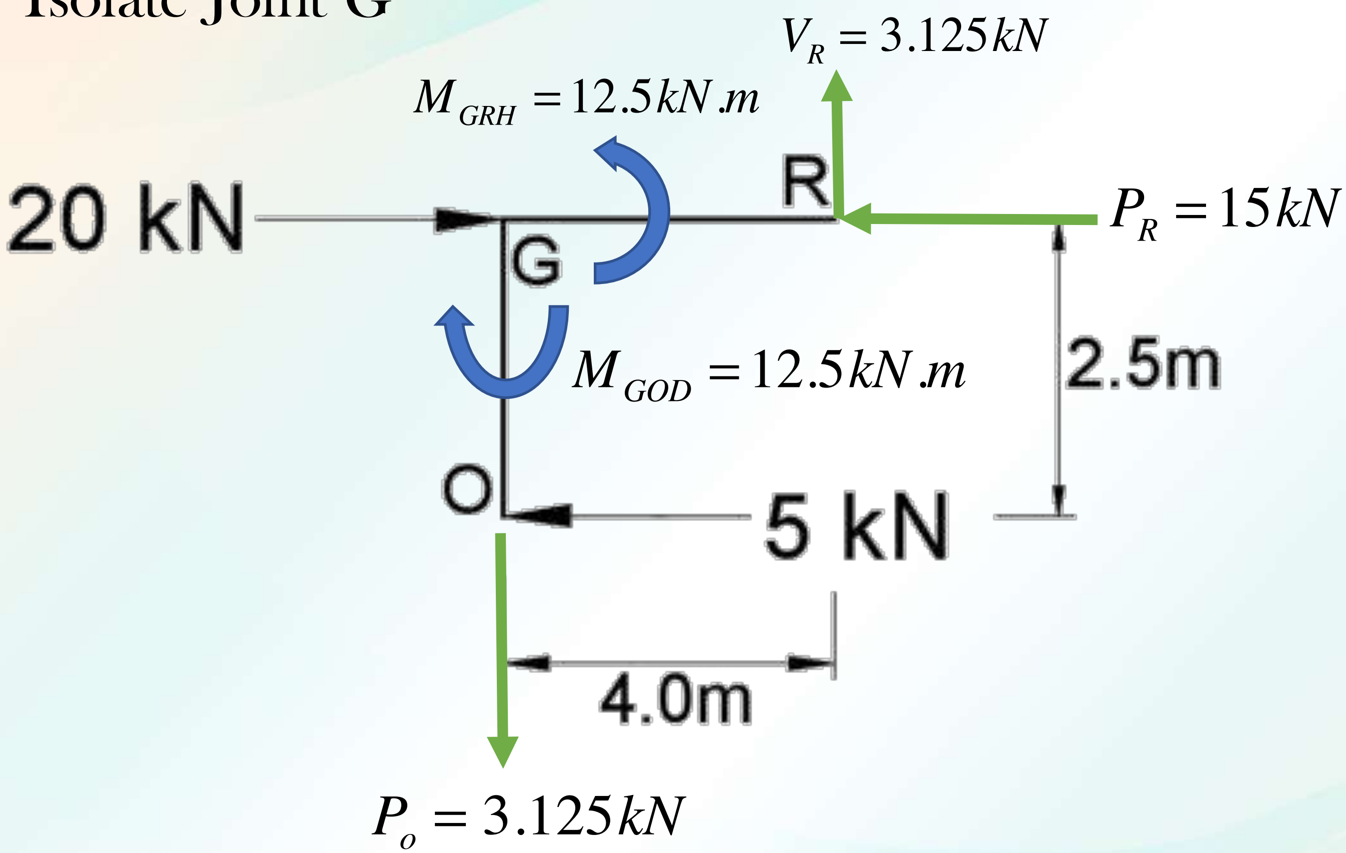
$$\sum F_H = 0$$

$$20 = 5 + V_R$$

$$V_R = 15kN$$

Problem 1 - Solution

Isolate Joint G



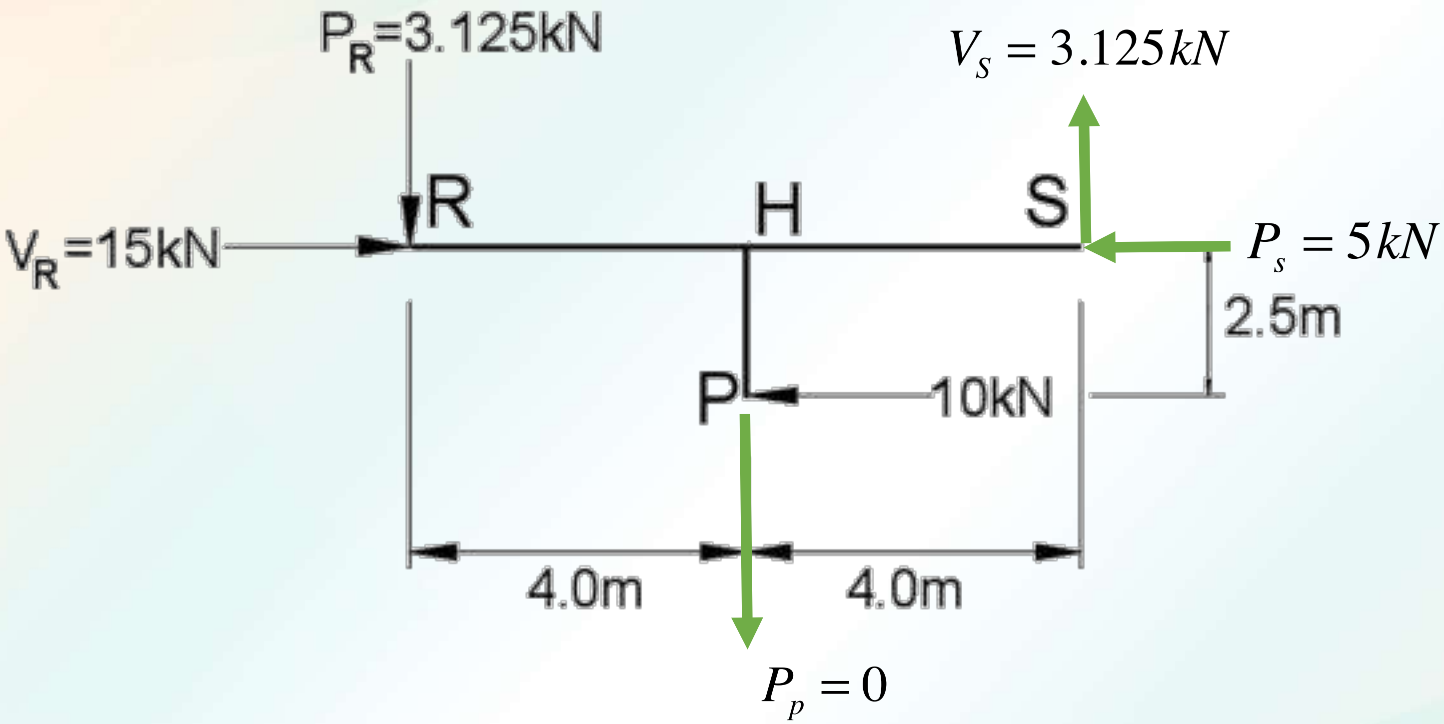
Solve for the Moment

$$M_{GOD} = 5(2.5)$$

$$M_{GOD} = 12.5kN.m$$

Problem 1 - Solution

Isolate Joint H



Use 3ME Equations

$$\sum M_s = 0$$

$$-3.125(8) + P_p(4) + 10(2.5) = 0$$

$$P_p = 0$$

$$\sum F_v = 0$$

$$-3.125 + V_s = 0$$

$$V_S = 3.125kN$$

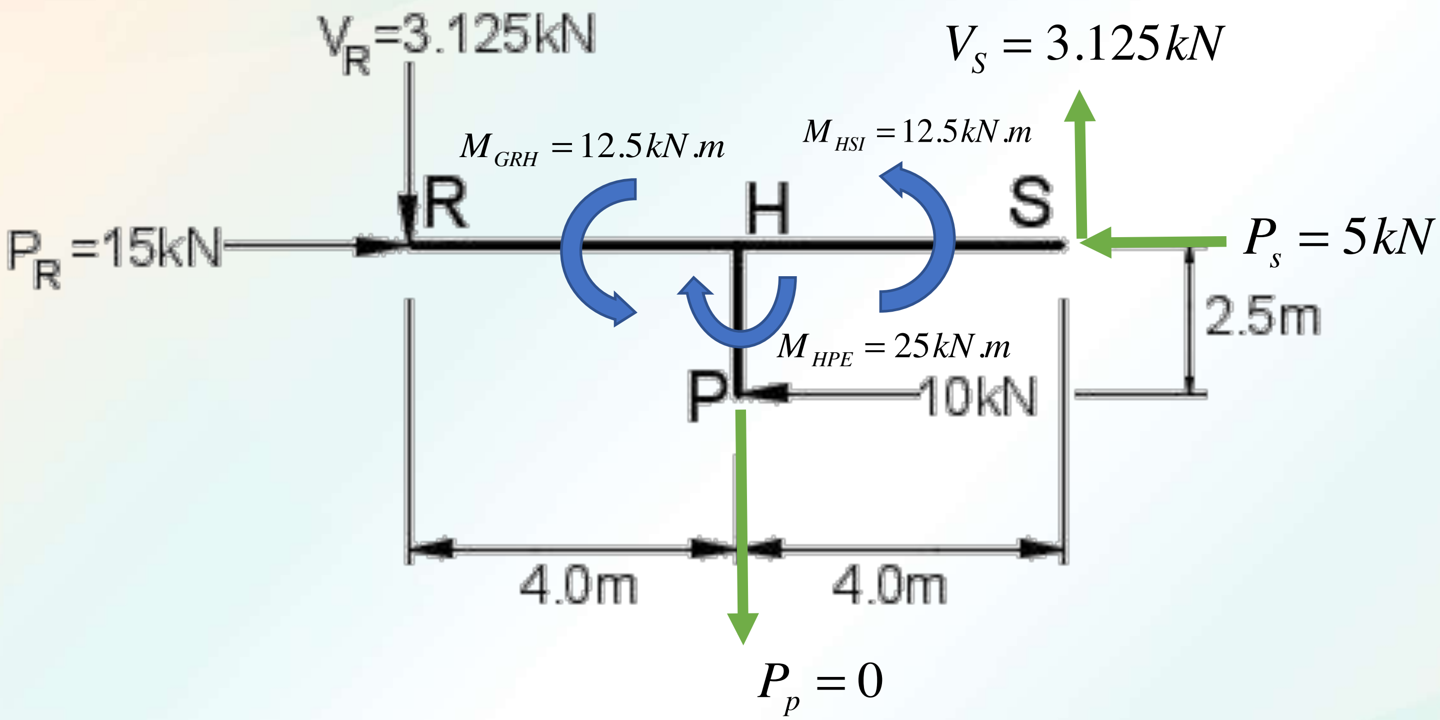
$$\sum F_H = 0$$

$$15 - 10 - P_s = 0$$

$$P_s = 5kN$$

Problem 1 - Solution

Isolate Joint H



Use 3ME Equations

$$\sum M_s = 0$$

$$-3.125(8) + P_p(4) + 10(2.5) = 0$$

$$P_p = 0$$

$$\sum F_v = 0$$

$$-3.125 + V_s = 0$$

$$V_S = 3.125kN$$

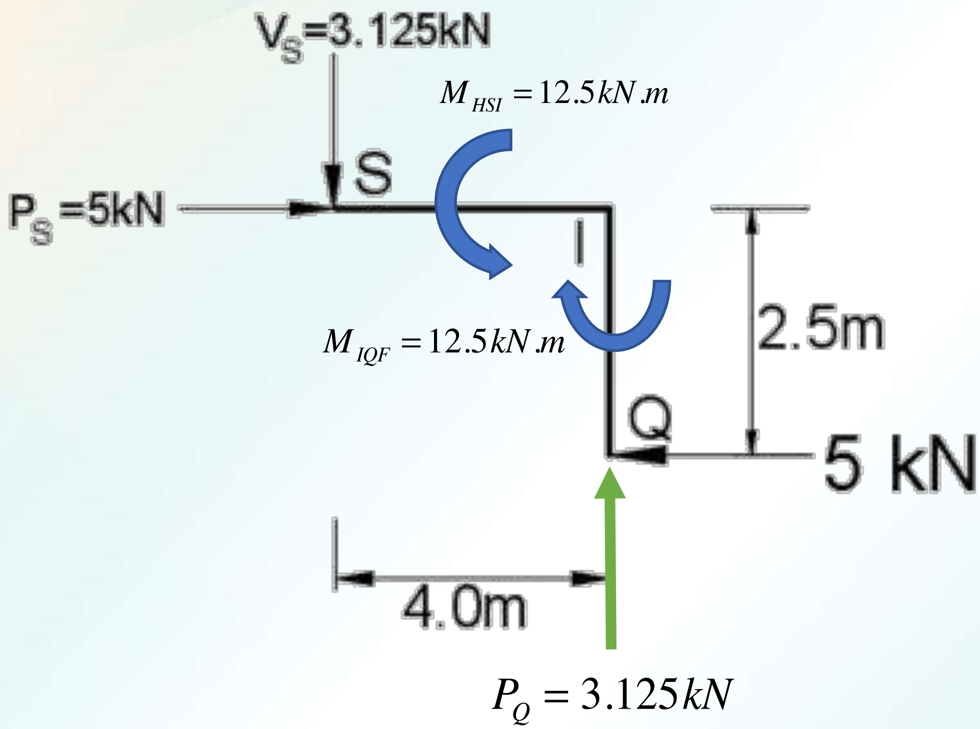
$$\sum F_H = 0$$

$$15 - 10 - P_s = 0$$

$$P_s = 5kN$$

Problem 1 - Solution

Isolate Joint I



Use 3ME Equations

$$\sum F_v = 0$$

$$-3.125 + P_Q = 0$$

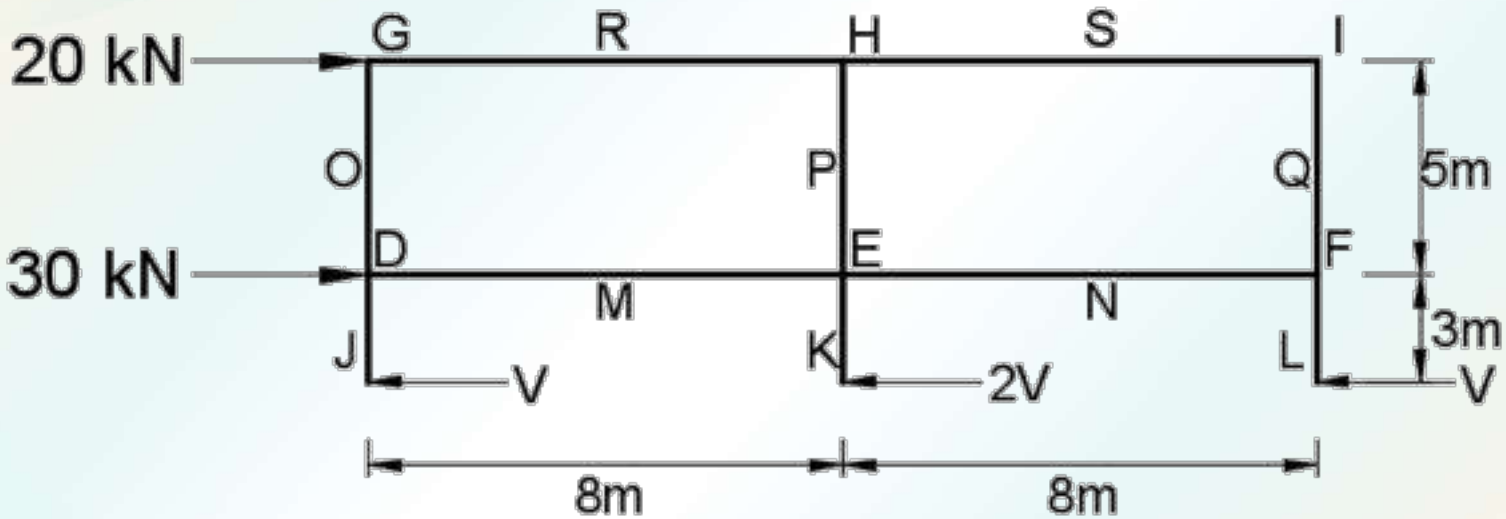
$$P_Q = 3.125kN$$

$$\sum F_H = 0$$

$$P_s - V = 0$$

Problem 1 - Solution

Isolate the Lower Level:



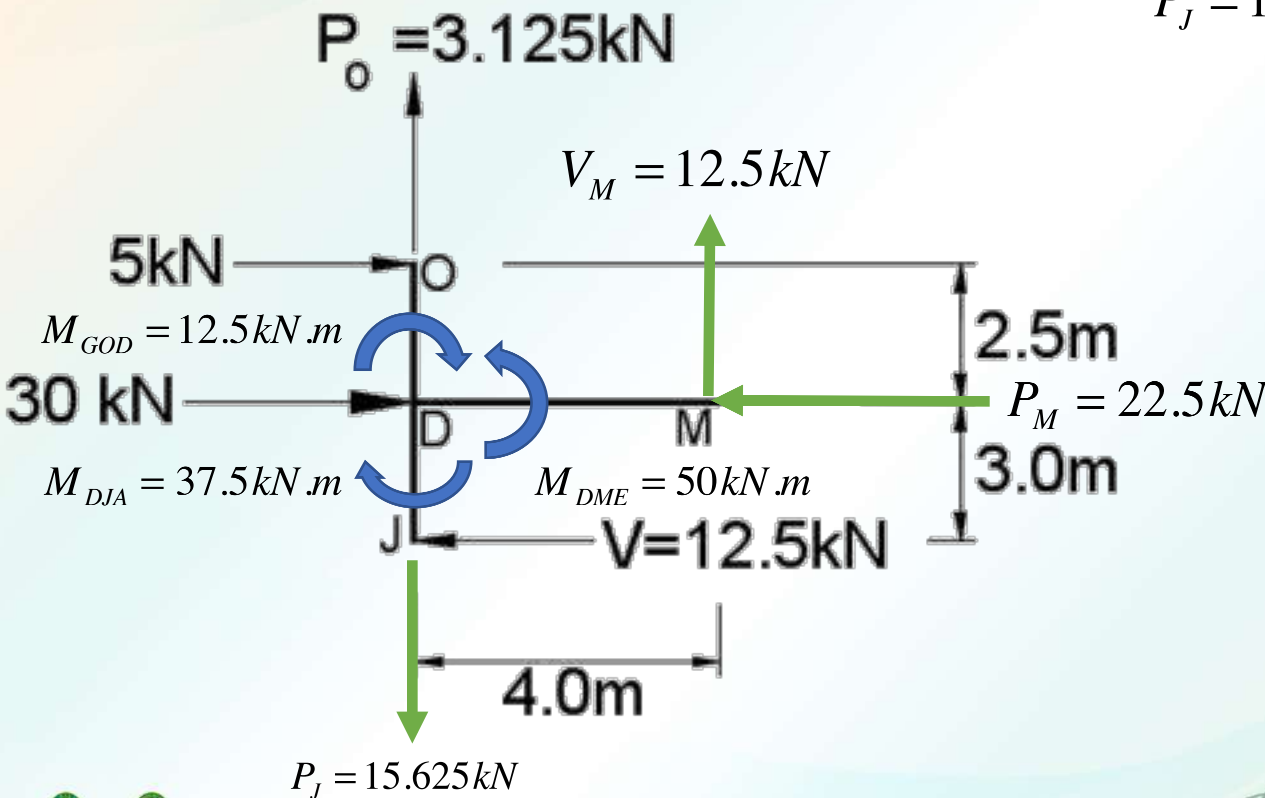
$$\sum F_H = 0$$

$$20 + 30 - V - 2V - V = 0$$

$$V = 12.5 kN$$

Problem 1 - Solution

Isolate Joint D:.



$$\sum M_M = 0$$

$$-P_J(4)+3.125(4)+5(2.5)+12.5(3)=0$$

$$P_J = 15.625kN$$

$$\sum F_v = 0$$

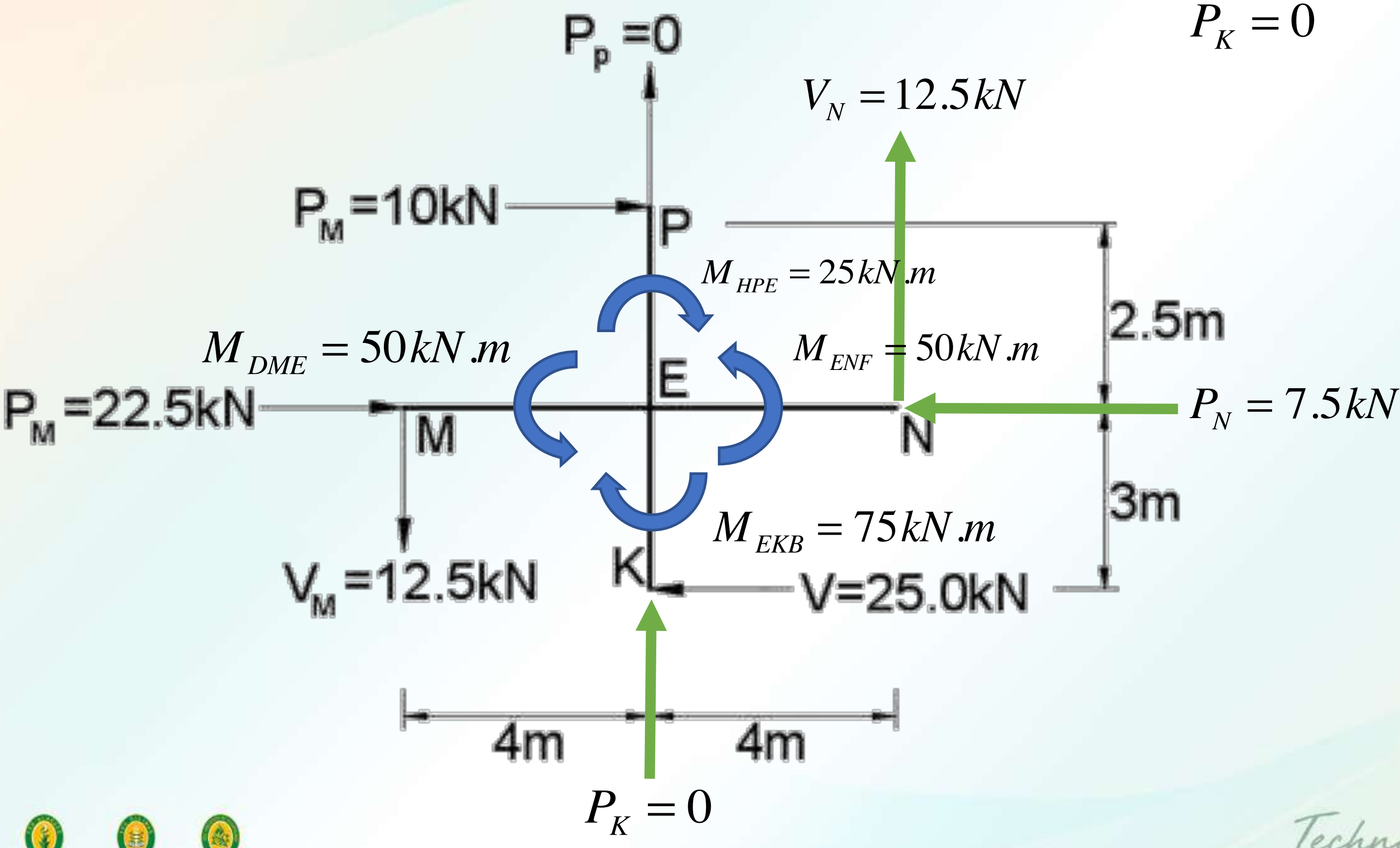
$$3.125-15.625+V_M=0$$

$$\sum F_H = 0$$

$$5+30-12.5-P_M=0$$

Problem 1 - Solution

Isolate Joint E:



$$\sum M_N = 0$$

$$-12.5(8) + P_K(4) + 10(2.5) + 25(3) = 0$$

$$P_K = 0$$

$$\sum F_V = 0$$

$$-12.5 + P_p + P_K + V_N = 0$$

$$V_N = 12.5kN$$

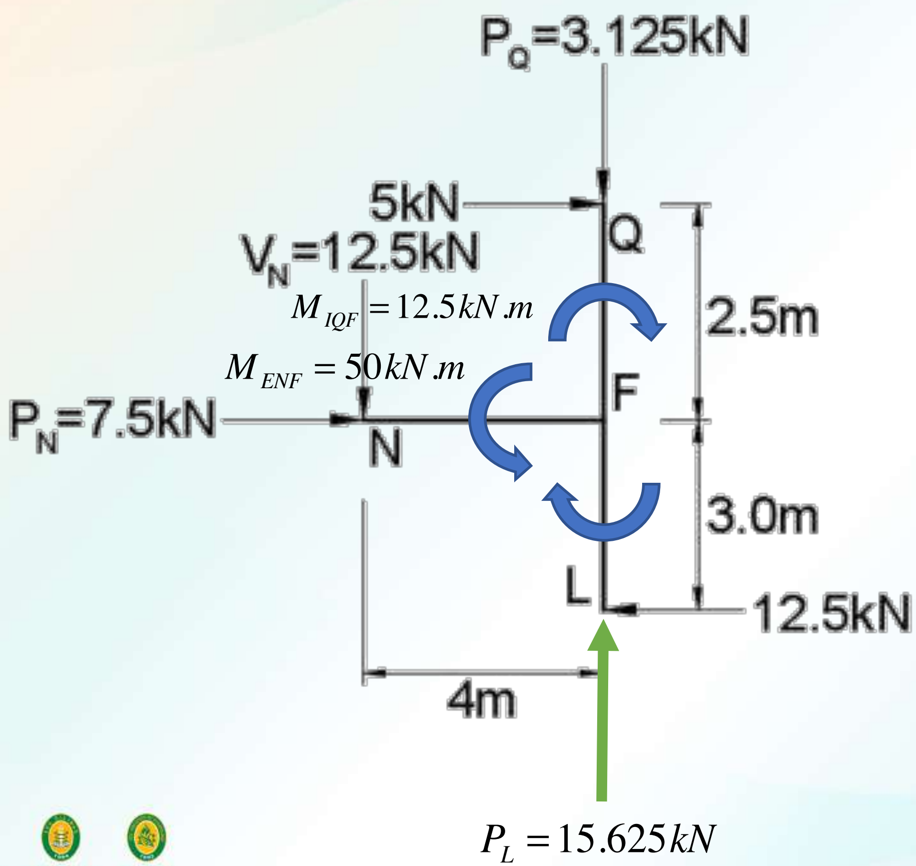
$$\sum F_H = 0$$

$$10 + 22.5 - 25.0 - P_N = 0$$

$$P_N = 7.5kN$$

Problem 1 - Solution

Isolate Joint F:



$$\sum F_v = 0$$

$$-3.125 - 12.5 + P_L = 0$$

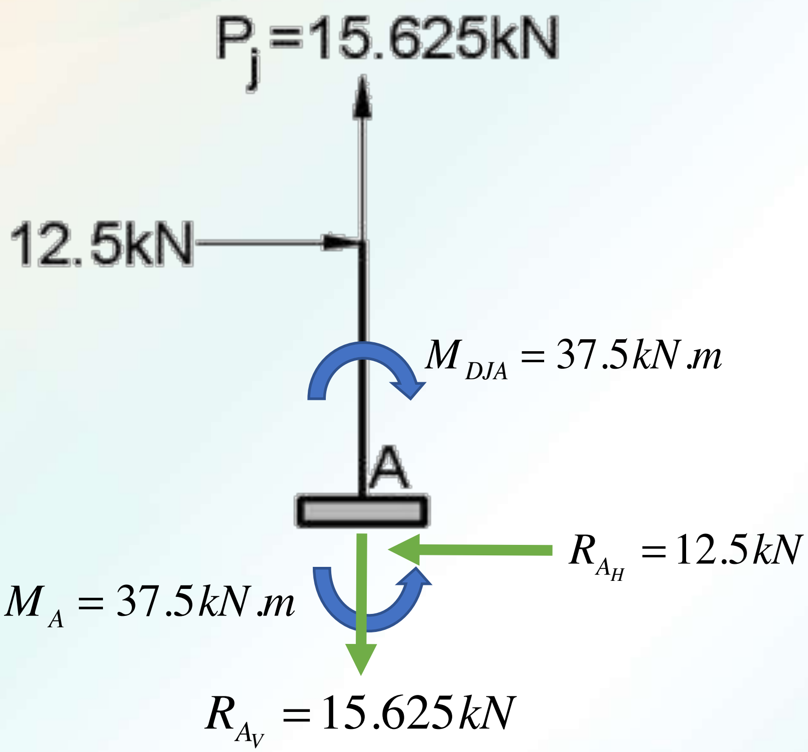
$$P_L = 15.625kN$$

Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Problem 1 - Solution

Isolate Joint A:



$$\sum F_V = 0$$

$$15.625 - R_{A_V} = 0$$

$$R_{A_V} = 15.625kN$$

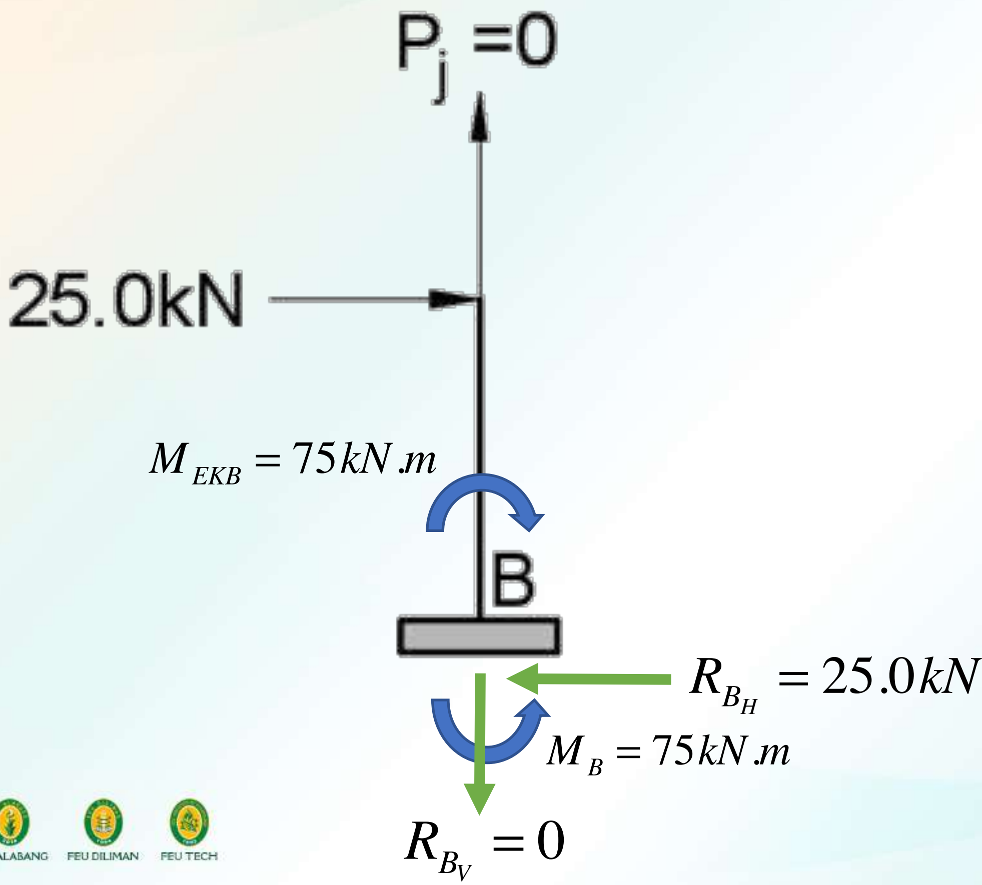
$$\sum F_H = 0$$

$$12.5 - R_{A_H} = 0$$

$$R_{A_H} = 12.5kN$$

Problem 1 - Solution

Isolate Joint B:



$$\sum F_v = 0$$

$$0 - R_{B_V} = 0$$

$$R_{B_V} = 0$$

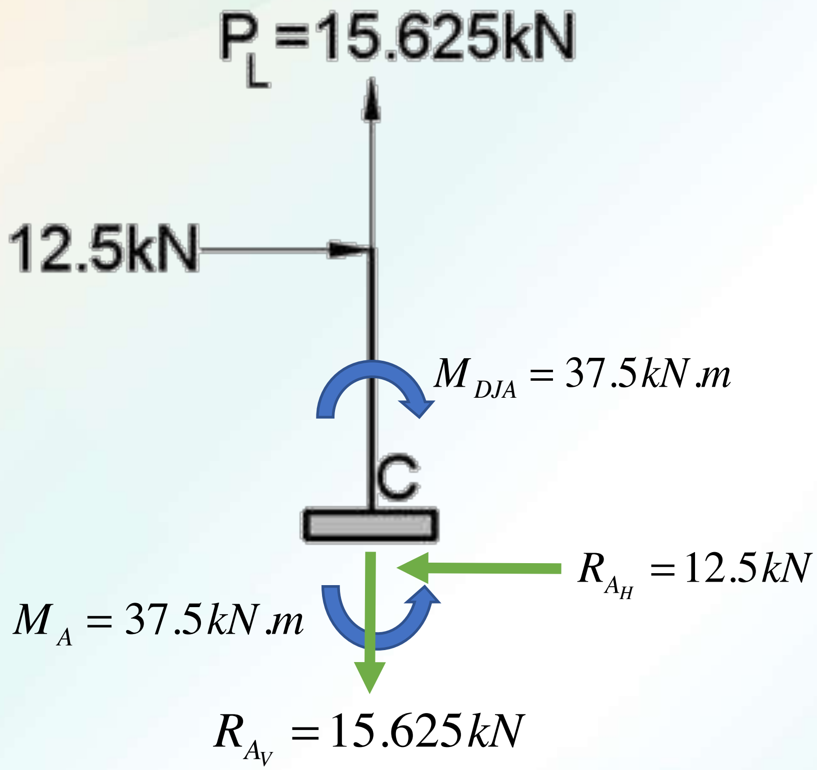
$$\sum F_H = 0$$

$$25.0 - R_{B_H} = 0$$

$$R_{B_H} = 25.0kN$$

Problem 1 - Solution

Isolate Joint C:



$$\sum F_V = 0$$

$$15.625 - R_{A_V} = 0$$

$$R_{A_V} = 15.625kN$$

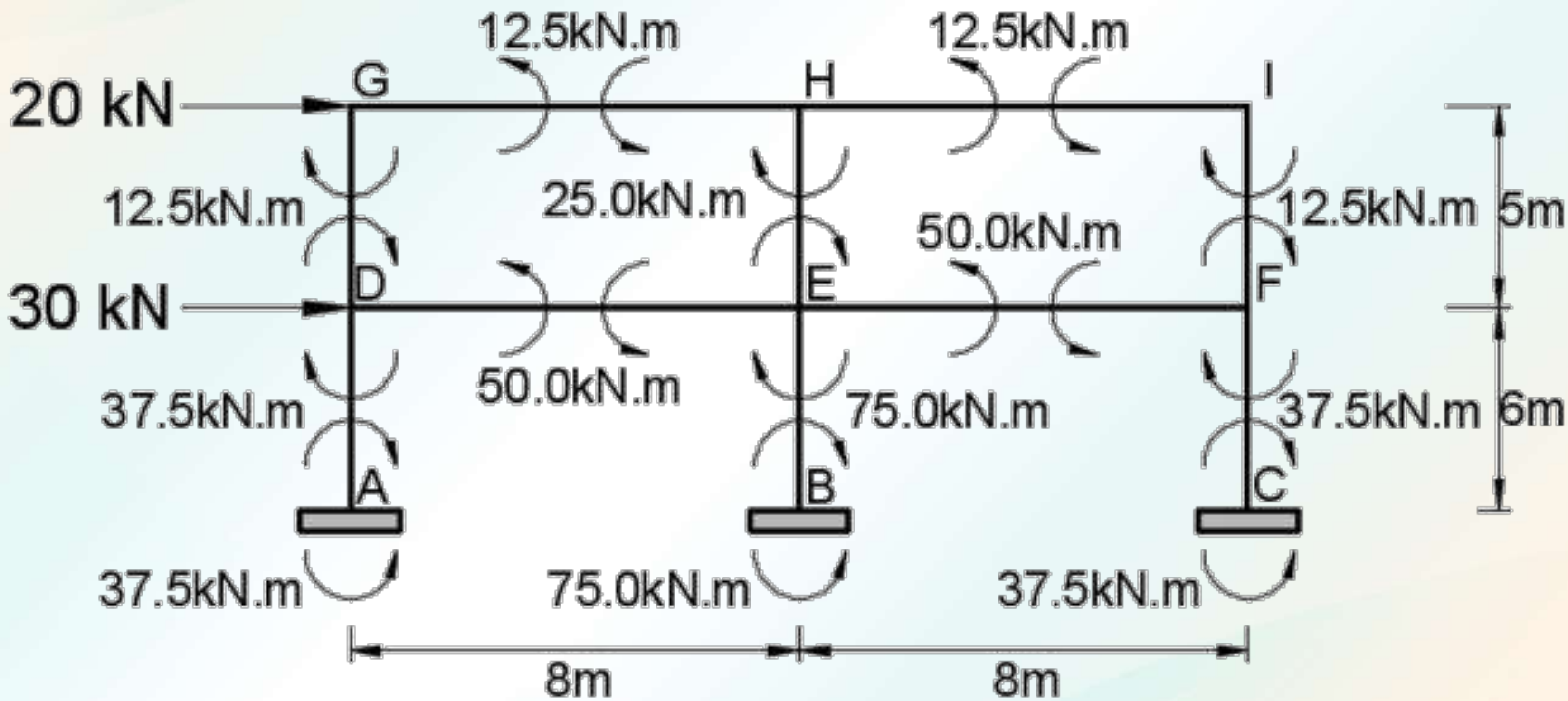
$$\sum F_H = 0$$

$$12.5 - R_{A_H} = 0$$

$$R_{A_H} = 12.5kN$$

Problem 1 - Solution

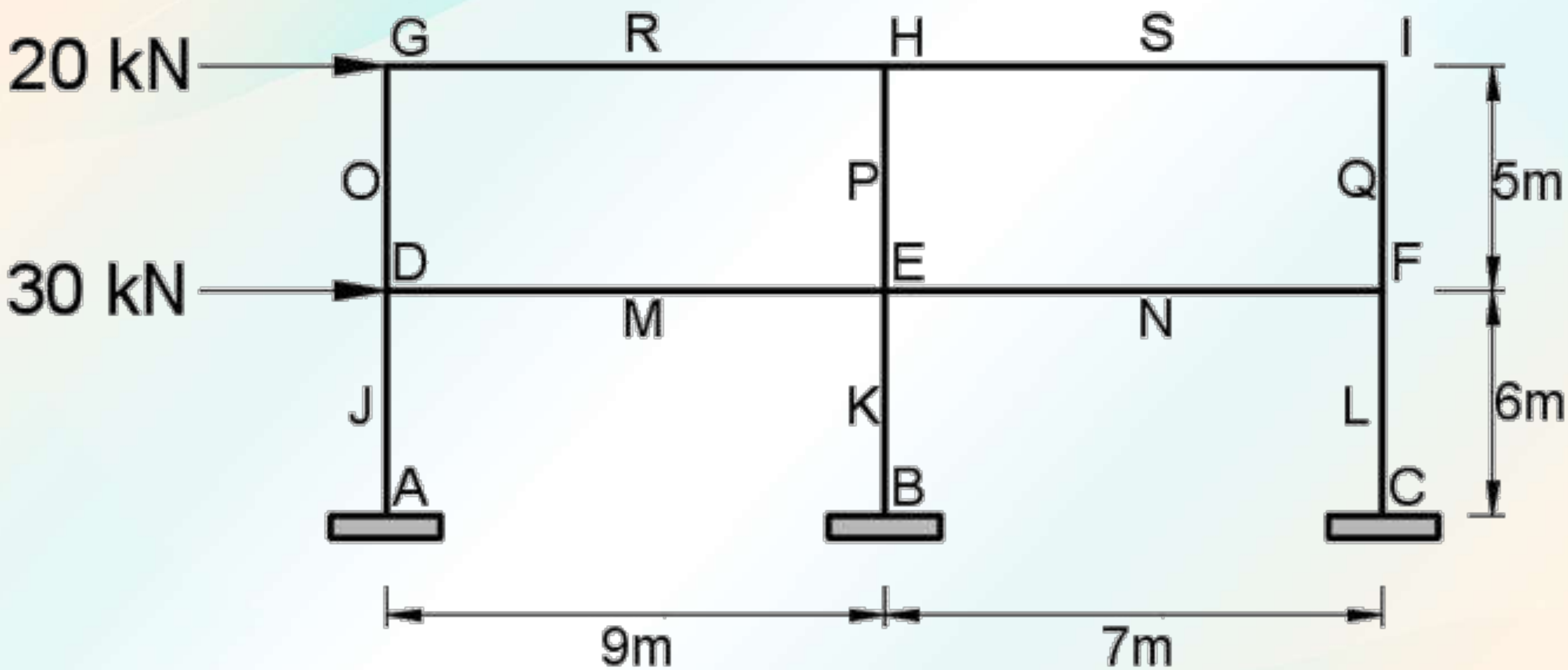
Final Beam and Column Moments:



COMPUTATION OF THE STRUCTURES' SHEARS, MOMENTS AND REACTIONS DUE TO EARTHQUAKE

Lateral Loads on Building Frames: Modified Portal Method

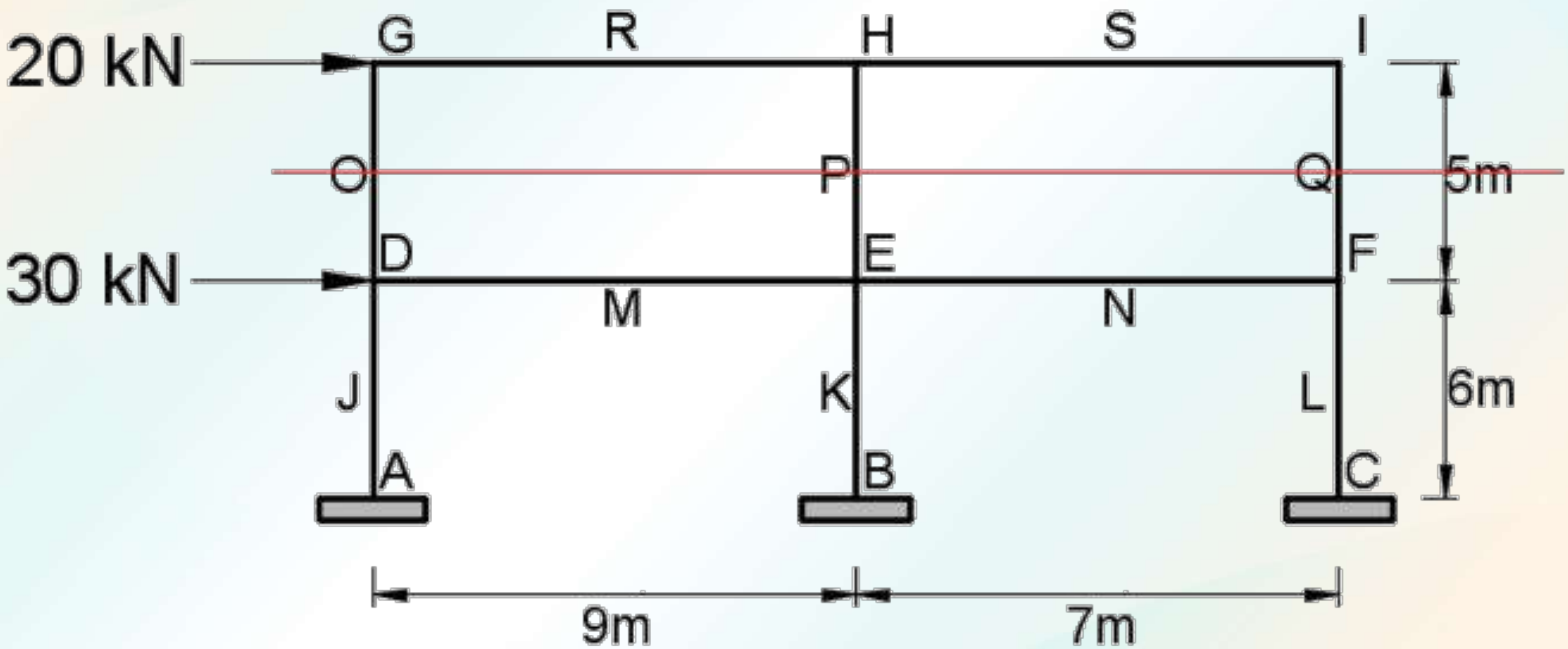
Problem 2



Solve the beam and column moments using Modified Portal Method.

Problem 2 - Solution

Isolate the Upper Level:

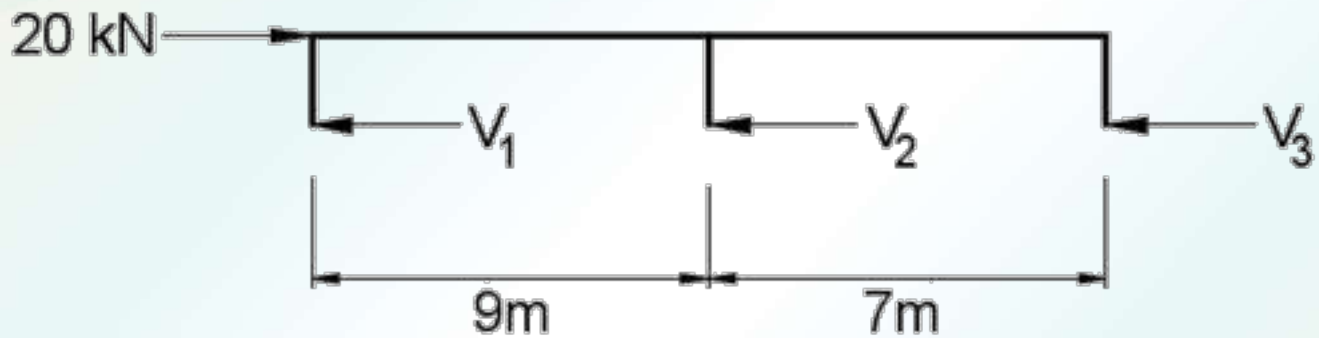


Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Problem 2 - Solution

Isolate the Upper Level:

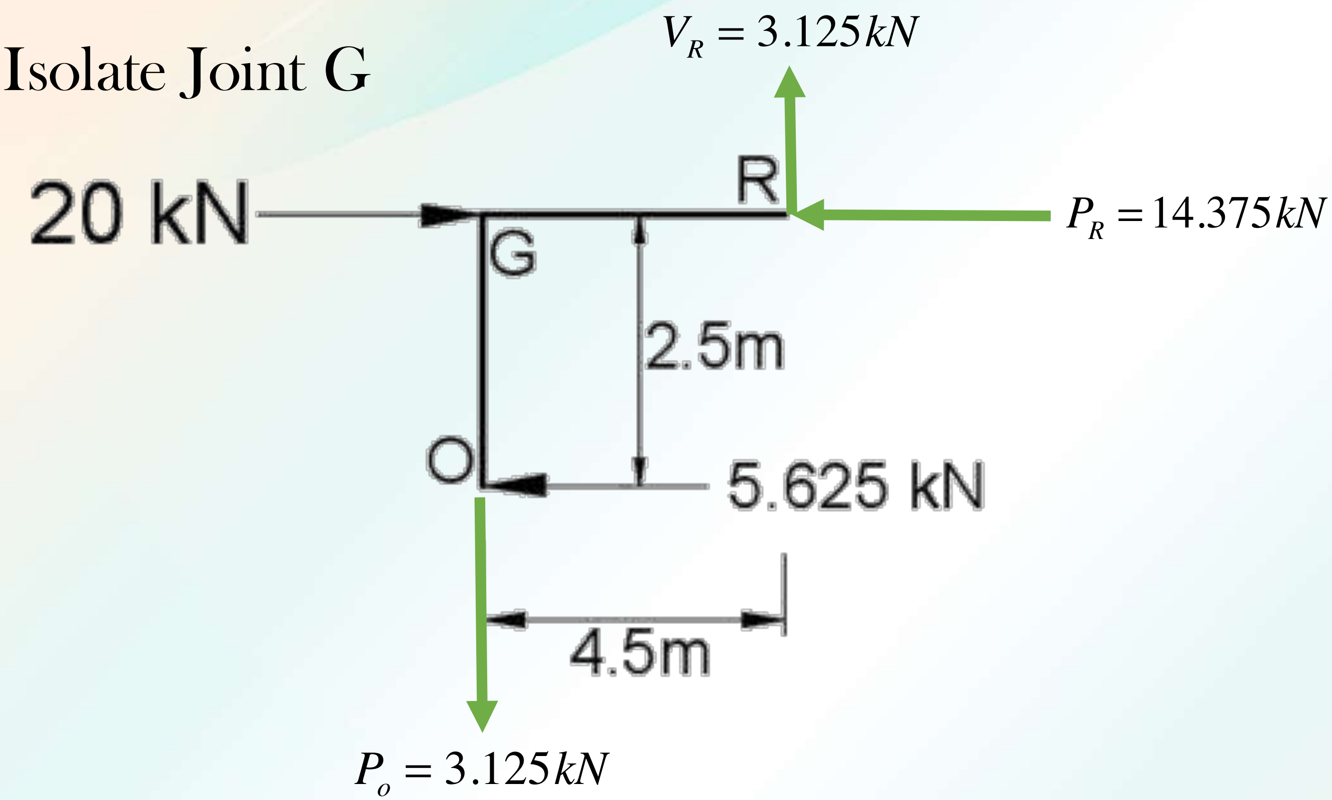


$$V_1 = 20\left(\frac{4.5}{16}\right) \qquad V_2 = 20\left(\frac{4.5+3.5}{16}\right) \qquad V_3 = 20\left(\frac{3.5}{16}\right)$$

$$V_1 = 5.625kN \qquad V_2 = 10kN \qquad V_3 = 4.375kN$$

Problem 2 - Solution

Isolate Joint G



Use 3ME Equations

$$\sum M_R = 0$$

$$-P_o(4.5) + 5.625(2.5) = 0$$

$$P_o = 3.125kN$$

$$\sum F_v = 0$$

$$-3.125 + V_R = 0$$

$$V_R = 3.125kN$$

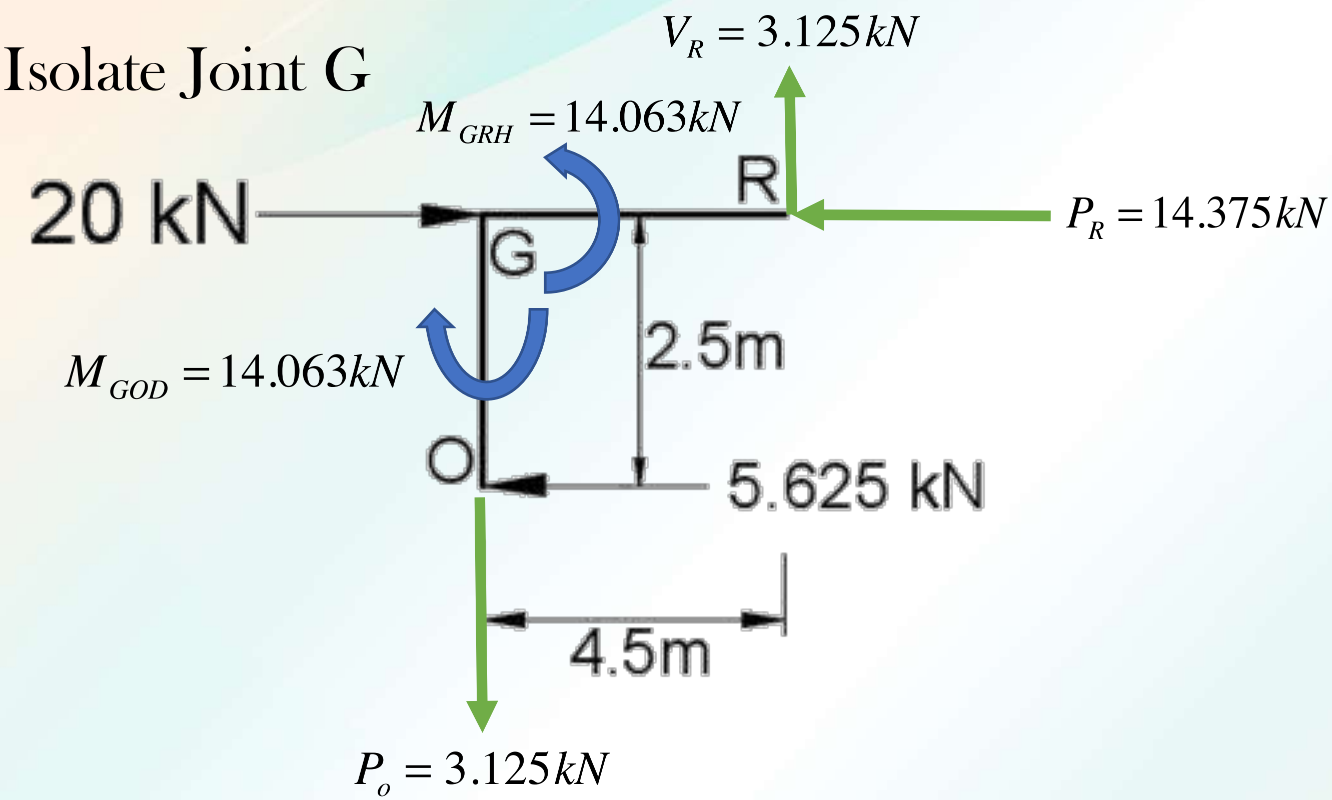
$$\sum F_H = 0$$

$$20 - 5.625 - P_R = 0$$

$$P_R = 14.375kN$$

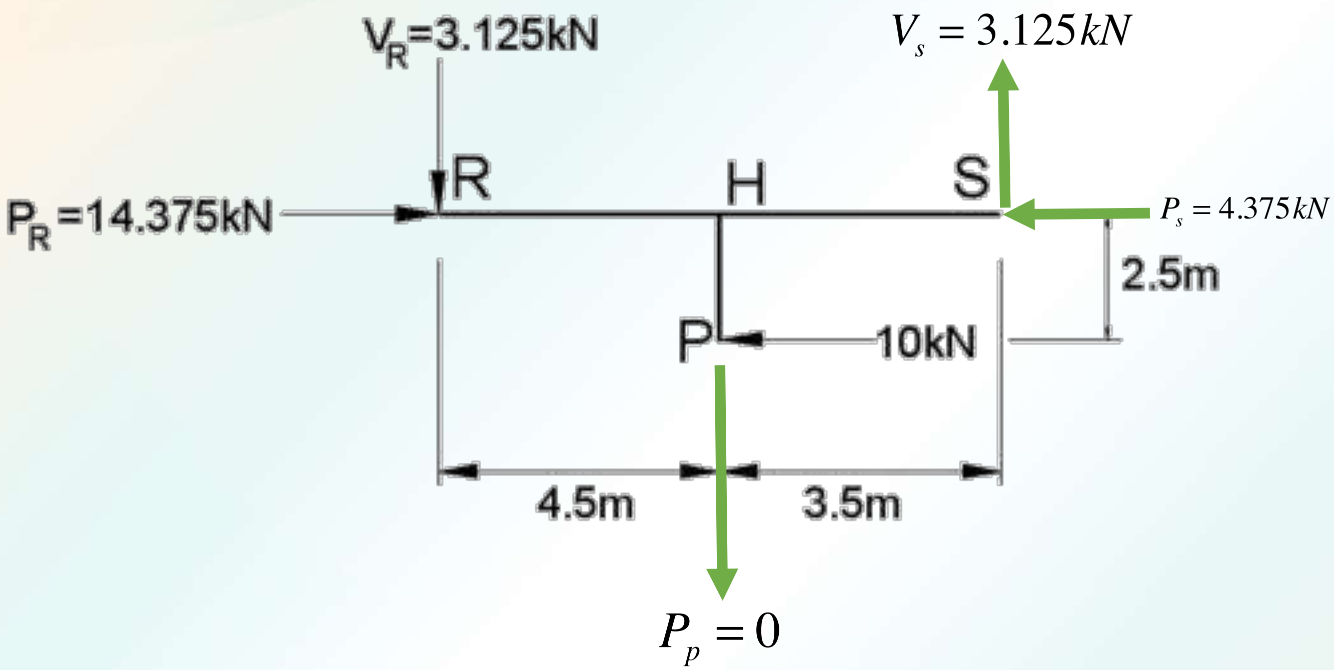
Problem 2 - Solution

Isolate Joint G



Problem 2 - Solution

Isolate Joint H



Use 3ME Equations

$$\sum M_s = 0$$

$$-P_p(3.5) - 3.125(8) + 10(2.5) = 0$$

$$P_p = 0$$

$$\sum F_v = 0$$

$$-3.125 + V_s = 0$$

$$V_s = 3.125kN$$

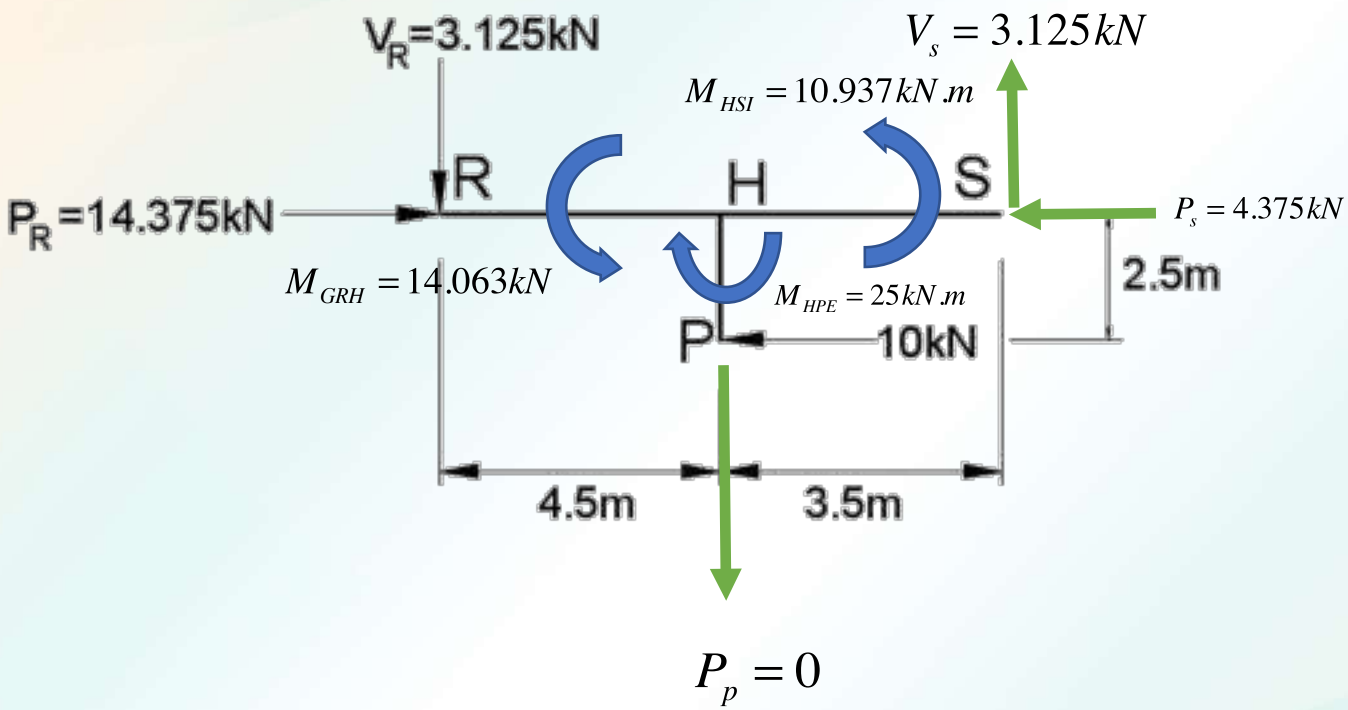
$$\sum F_H = 0$$

$$14.375 - 10 - P_s = 0$$

$$P_s = 4.375kN$$

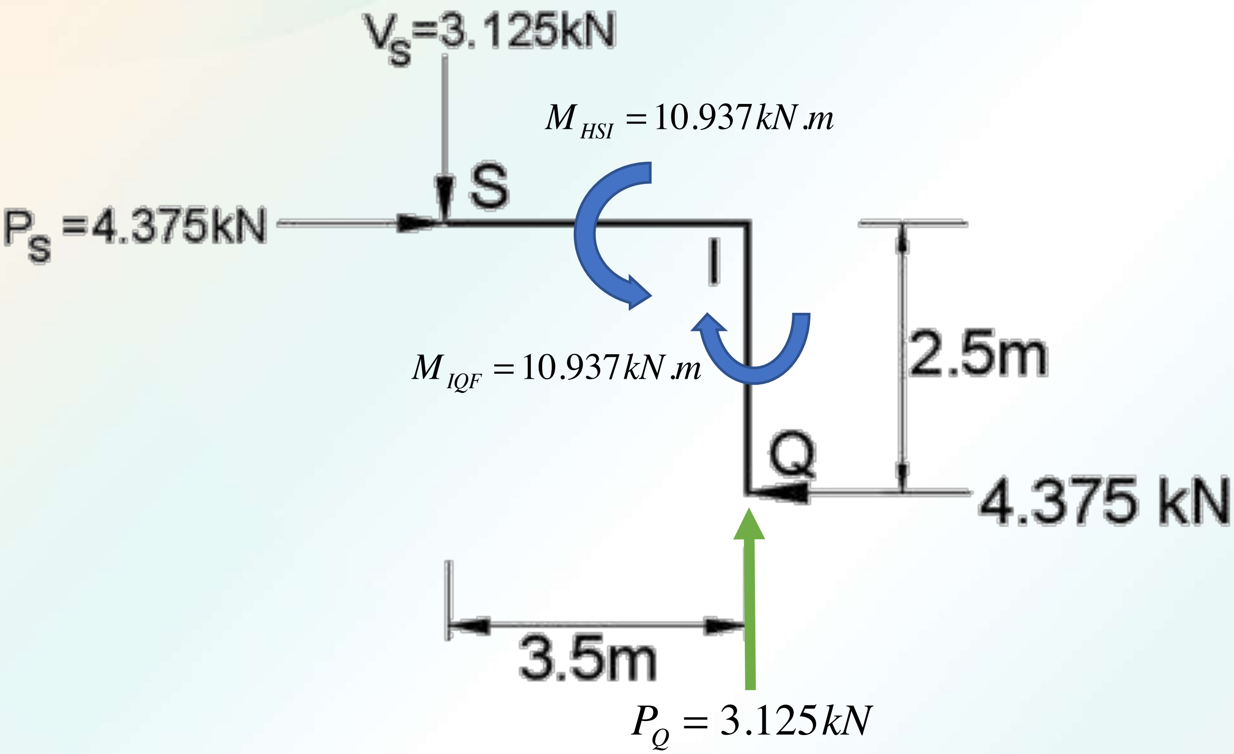
Problem 2 - Solution

Isolate Joint H



Problem 2 - Solution

Isolate Joint I



Use 3ME Equations

$$\sum F_v = 0$$

$$-3.125 + P_Q = 0$$

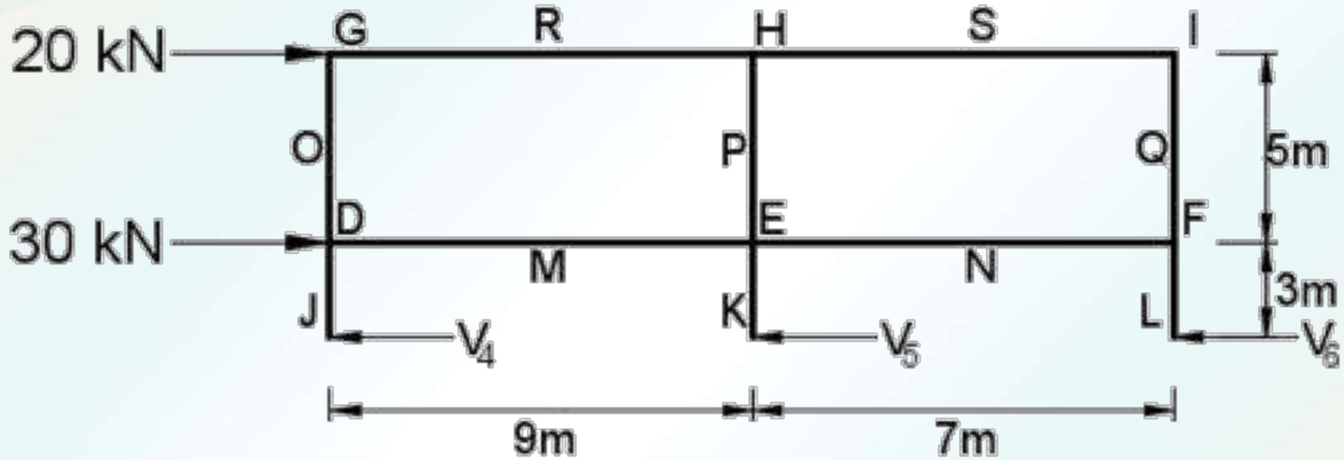
$$P_Q = 3.125 kN$$

$$\sum F_H = 0$$

$$P_s - V = 0$$

Problem 2 - Solution

Isolate the Lower Level:



$$V_4 = (20+30)\left(\frac{4.5}{16}\right)$$

$$V_4 = 14.063kN$$

$$V_5 = (20+30)\left(\frac{4.5+3.5}{16}\right)$$

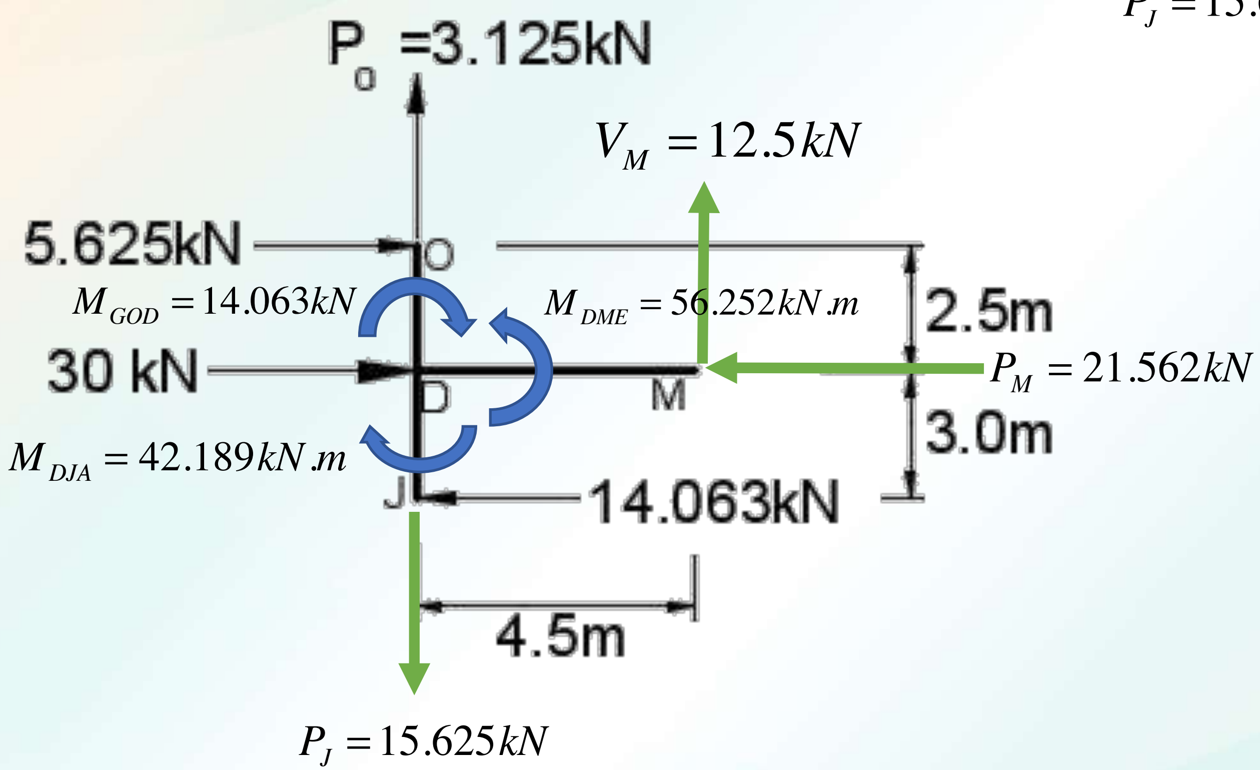
$$V_5 = 25kN$$

$$V_6 = (20+30)\left(\frac{3.5}{16}\right)$$

$$V_6 = 10.937kN$$

Problem 2 - Solution

Isolate Joint D:



$$\sum M_M = 0$$

$$-P_J(4.5) + 3.125(4.5) + 5.635(2.5) + 14.063(3) = 0$$

$$P_J = 15.625kN$$

$$\sum F_v = 0$$

$$3.125 - 15.625 + V_M = 0$$

$$V_M = 12.5kN$$

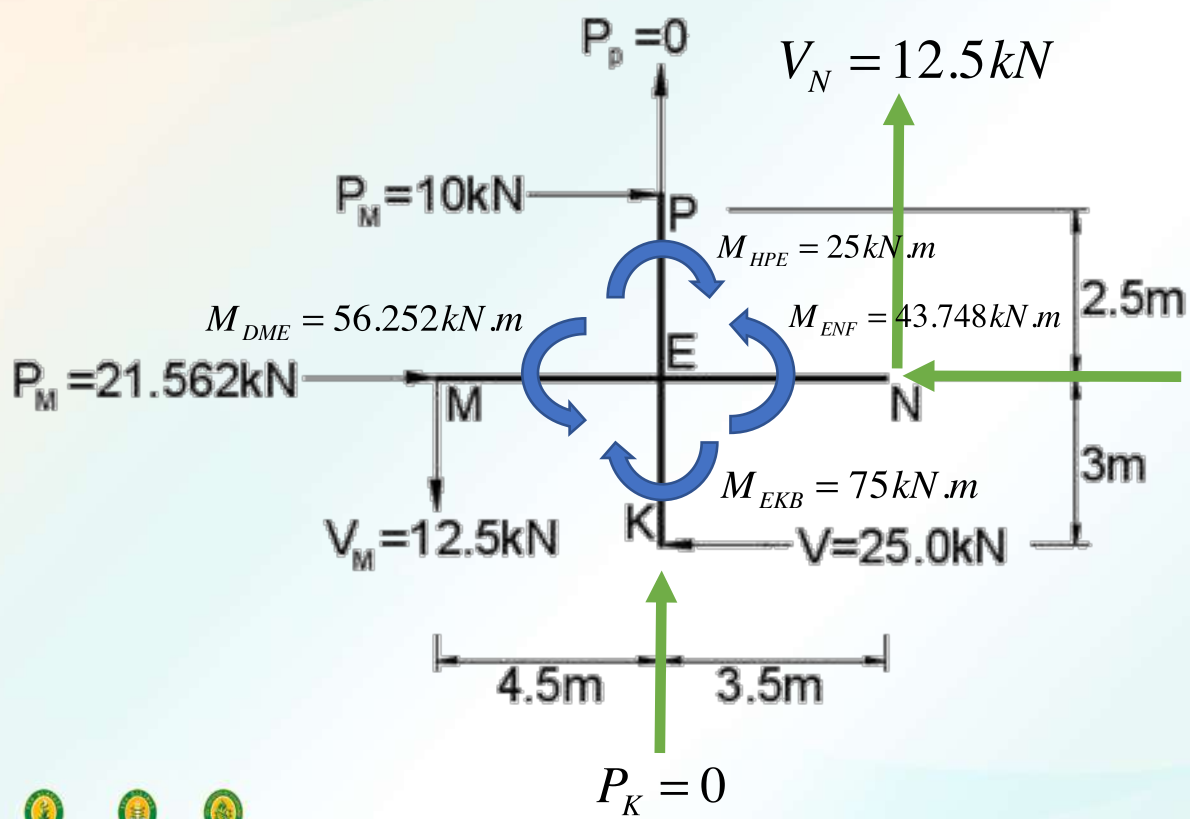
$$\sum F_H = 0$$

$$5.625 + 30 - 14.063 - P_M = 0$$

$$P_M = 21.562kN$$

Problem 2 - Solution

Isolate Joint E:



$$\sum M_N = 0$$

$$-12.5(8) + P_K(3.5) + 10(2.5) + 25(3) = 0$$

$$P_K = 0$$

$$\sum F_V = 0$$

$$-12.5 + V_N = 0$$

$$V_N = 12.5kN$$

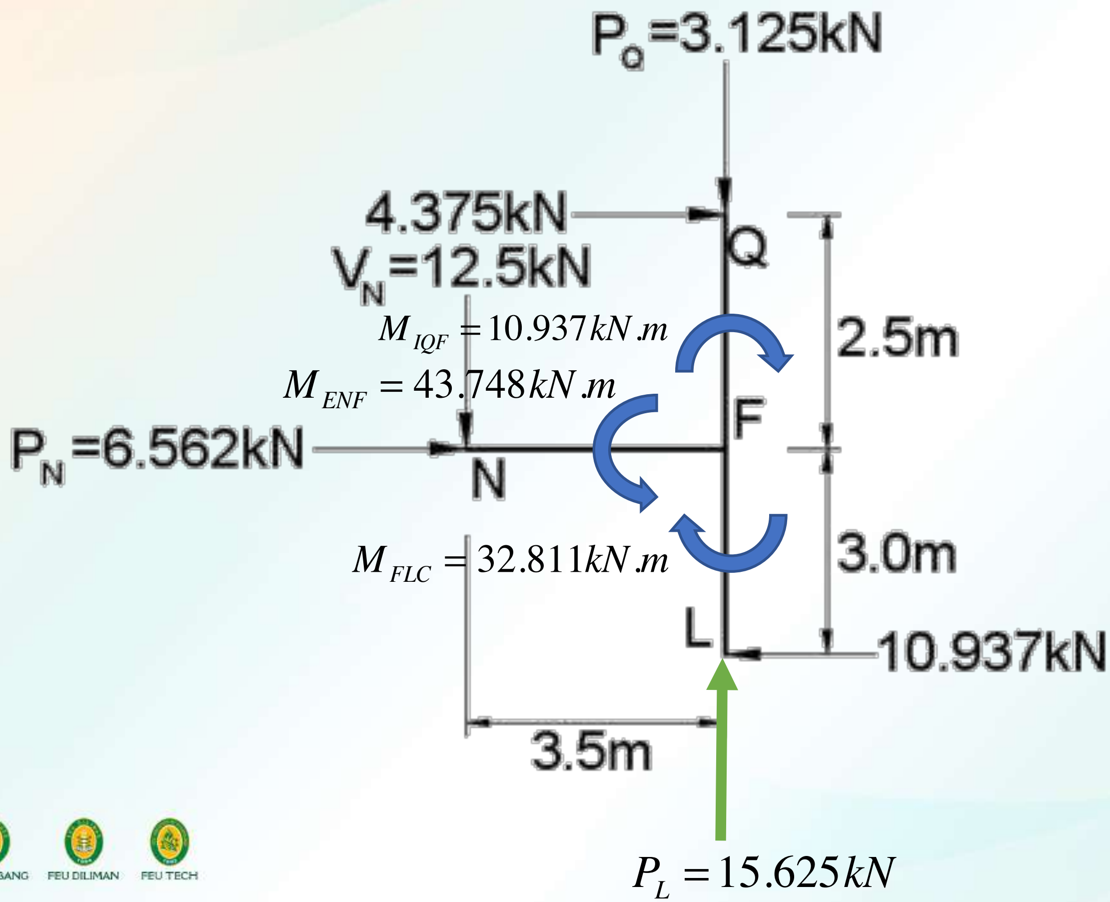
$$\sum F_H = 0$$

$$21.562 + 10 - 25 - P_N = 0$$

$$P_N = 6.562kN$$

Problem 2 - Solution

Isolate Joint F:



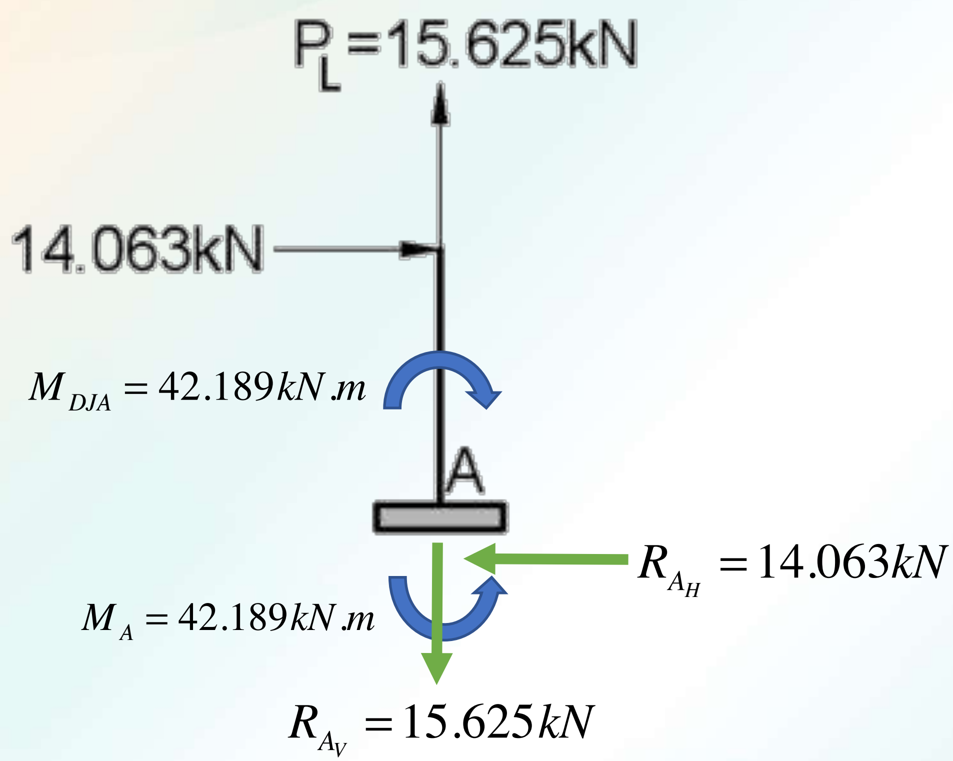
$$\sum F_v = 0$$

$$-3.125 - 12.5 + P_L = 0$$

$$P_L = 15.625 kN$$

Problem 2 - Solution

Isolate Joint A:



$$\sum F_V = 0$$

$$15.625 - R_{A_V} = 0$$

$$R_{A_V} = 15.625kN$$

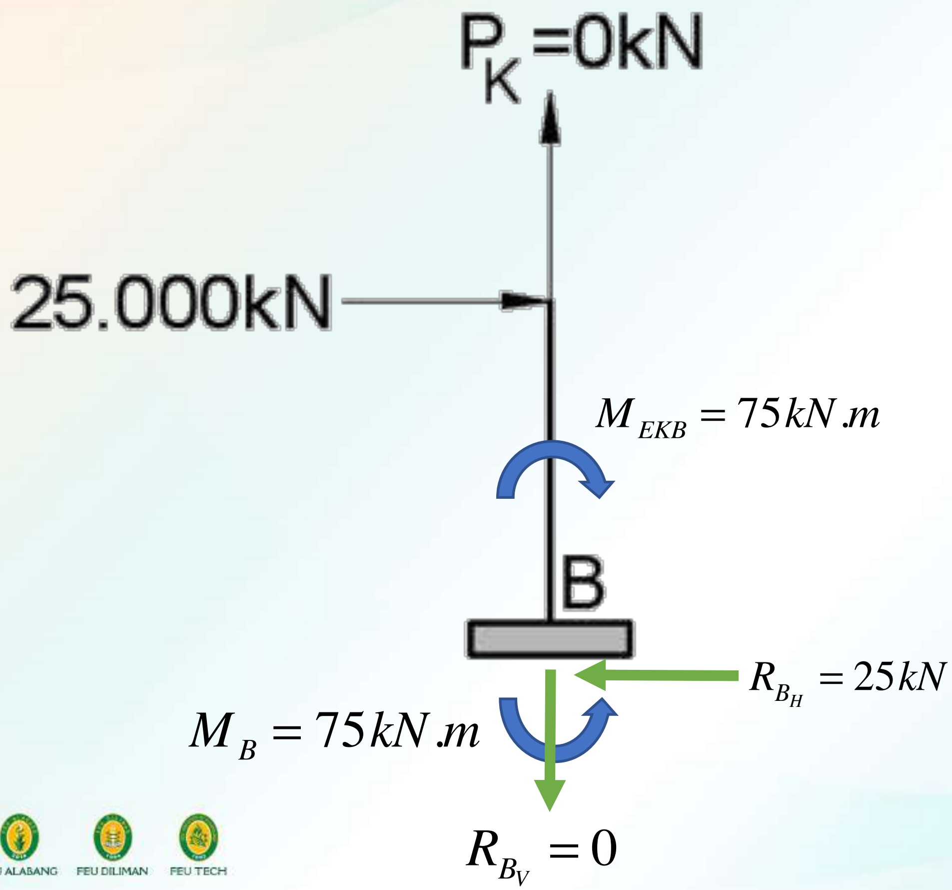
$$\sum F_H = 0$$

$$14.063 - R_{A_H} = 0$$

$$R_{A_H} = 14.063kN$$

Problem 2 - Solution

Isolate Joint B:



$$\sum F_v = 0$$

$$0 - R_{B_v} = 0$$

$$R_{B_v} = 0$$

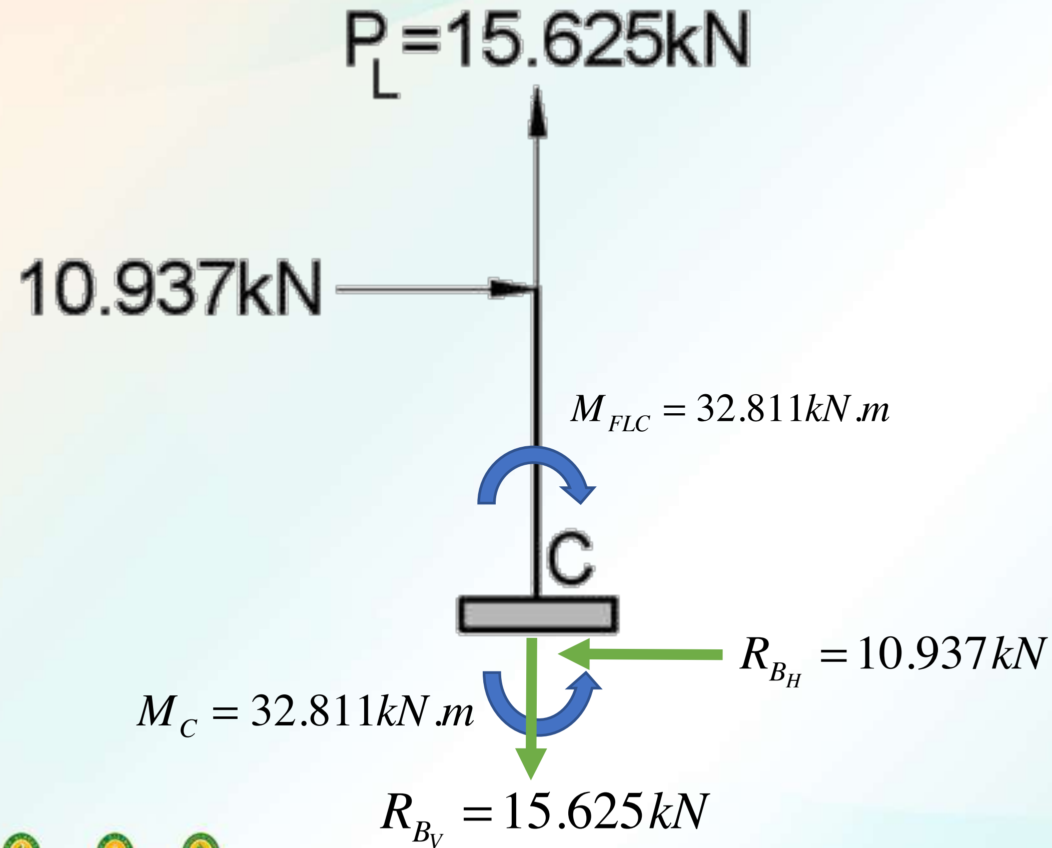
$$\sum F_H = 0$$

$$25 - R_{B_H} = 0$$

$$R_{B_H} = 25kN$$

Problem 2 - Solution

Isolate Joint C:



$$\sum F_V = 0$$

$$15.625 - R_{B_v} = 0$$

$$R_{B_V} = 15.625 kN$$

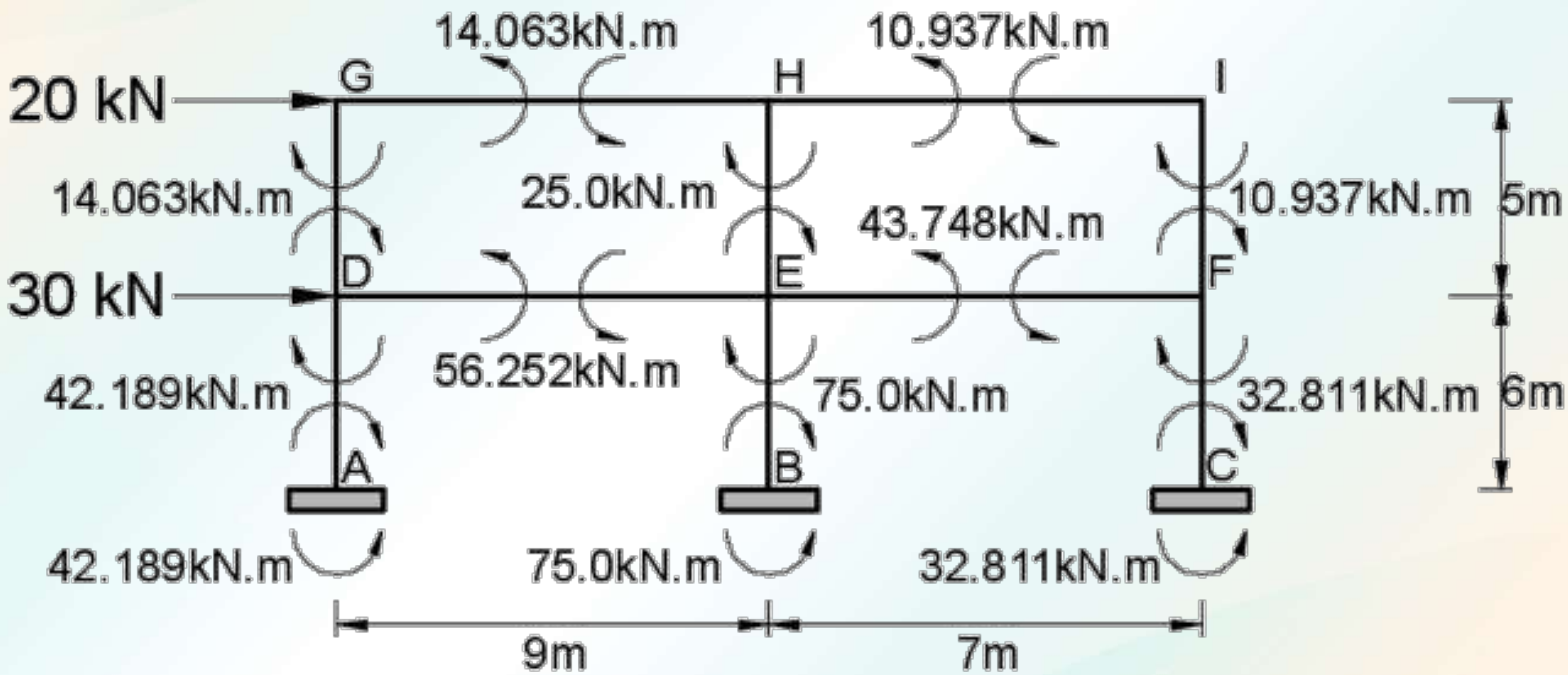
$$\sum F_H = 0$$

$$10.937 - R_{B_H} = 0$$

$$R_{B_H} = 10.937 kN$$

Problem 2 - Solution

Final Beam and Column Moments:



FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

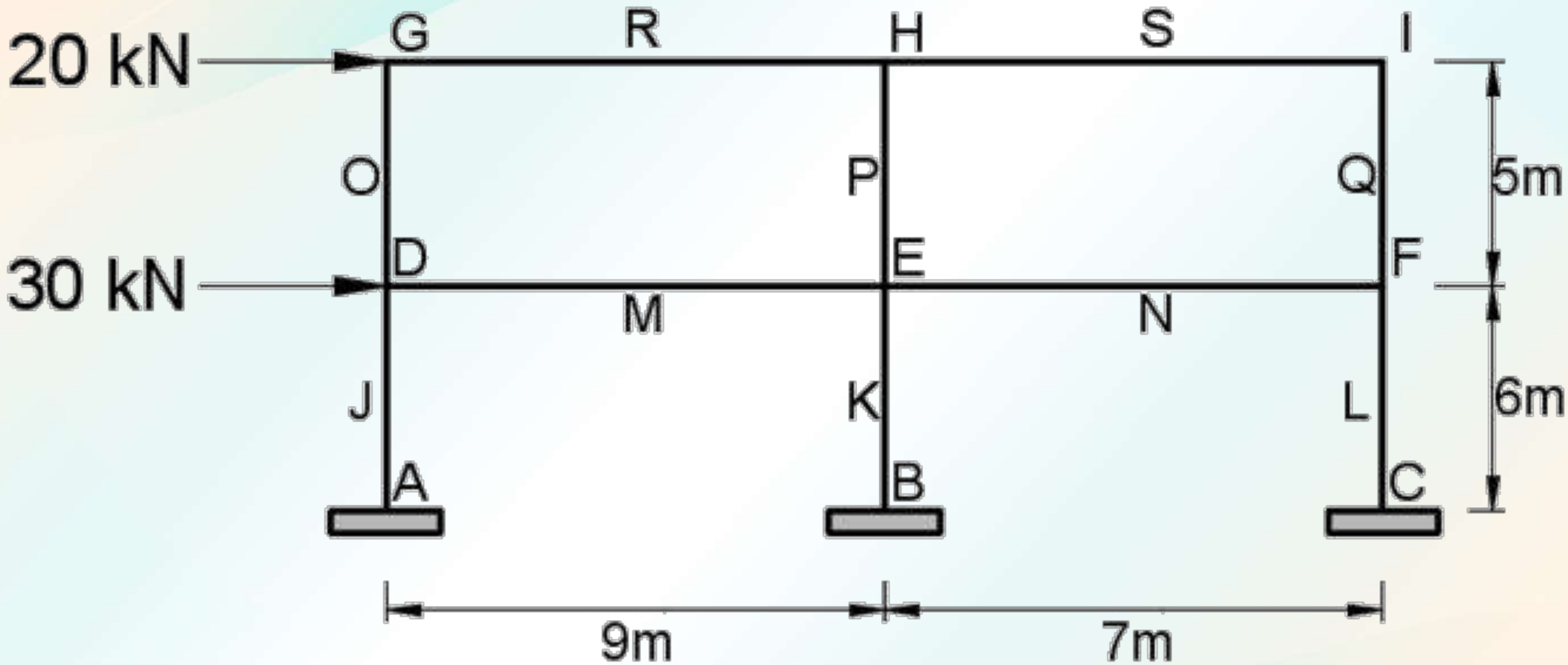
COMPUTATION OF THE STRUCTURES' SHEARS, MOMENTS AND REACTIONS DUE TO EARTHQUAKE

Sample Problems in Cantilever Method

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

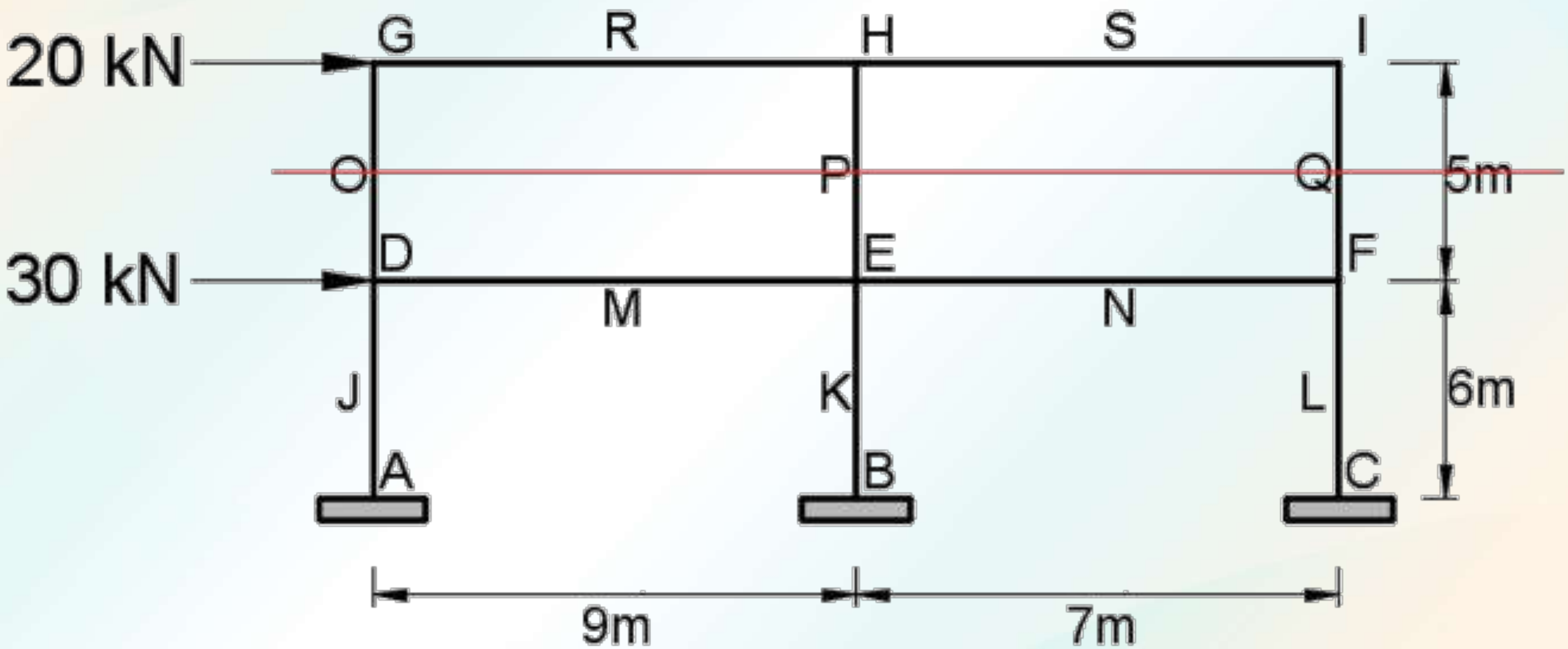
Problem 3



Solve the beam and column moments using Cantilever Method.

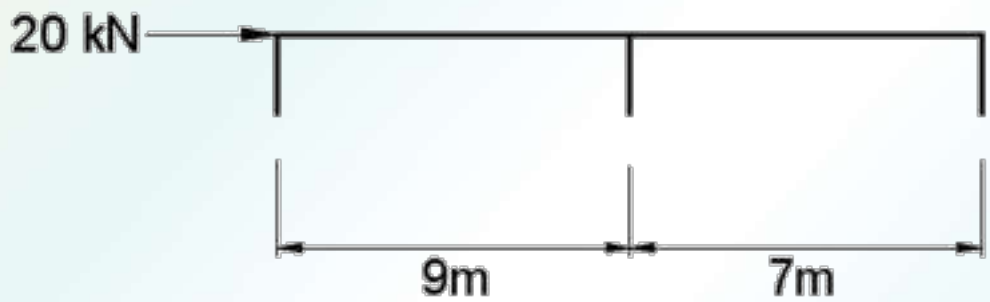
Problem 1 - Solution

Isolate the Upper Level:



Problem 1 - Solution

Isolate the Upper Level:



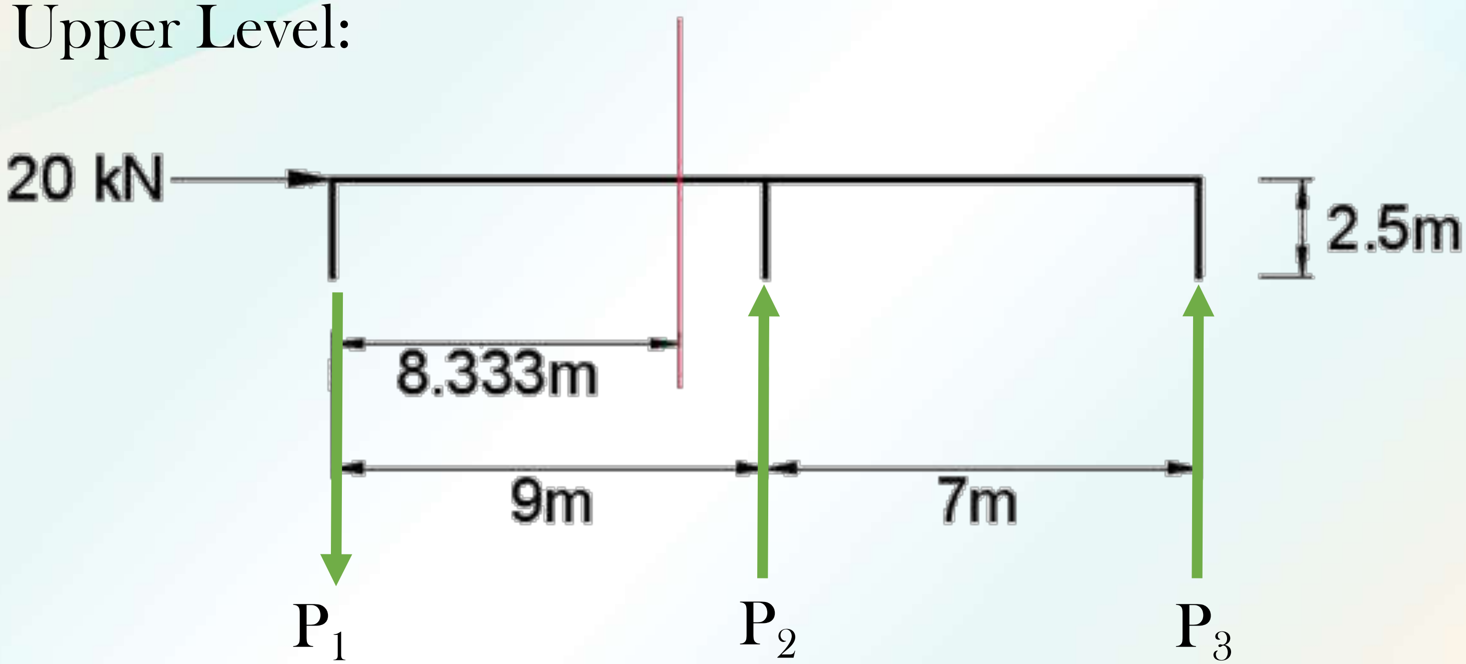
Locate the centroid of the group of columns:

$$3A\left(\bar{x}\right) = A(0) + A(9) + A(16)$$

$$\bar{x} = 8.333m$$

Problem 1 - Solution

Isolate the Upper Level:



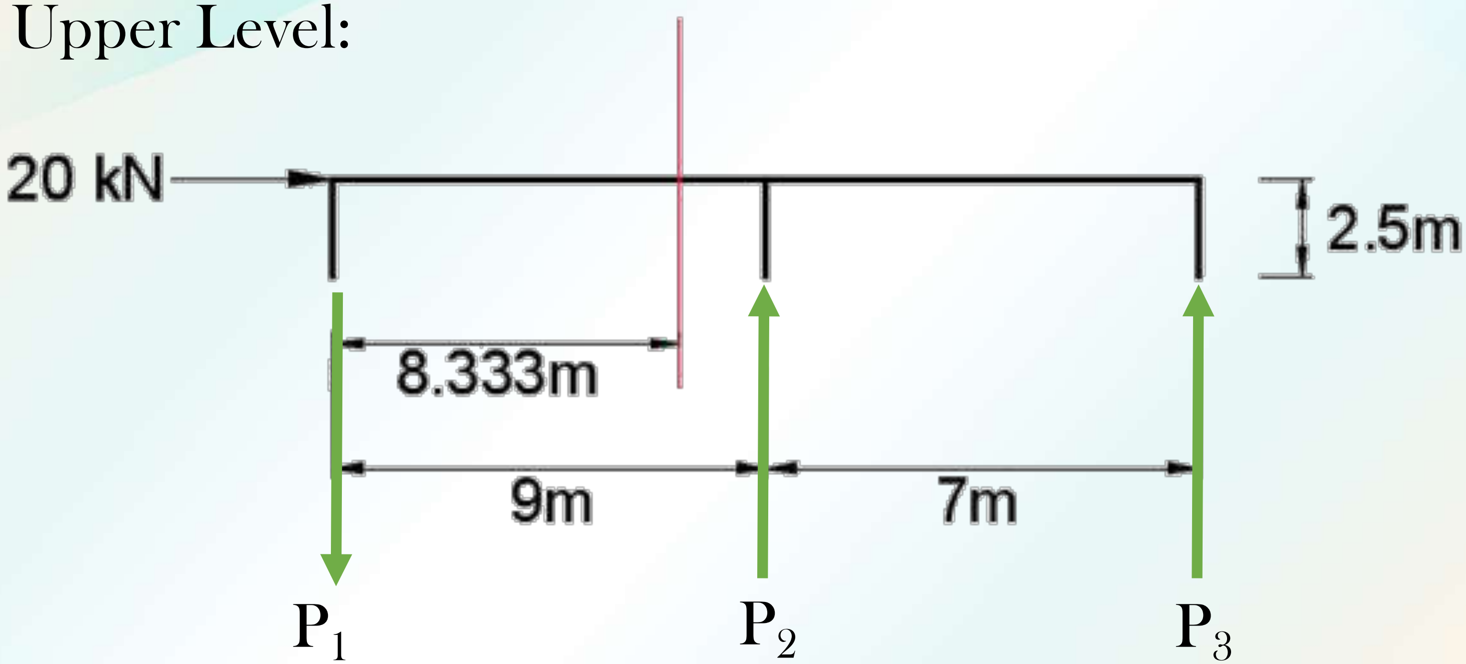
Solve for the Moment "M"

$$M = 20(2.5)$$

$$M = 50 kN.m$$

Problem 1 - Solution

Isolate the Upper Level:



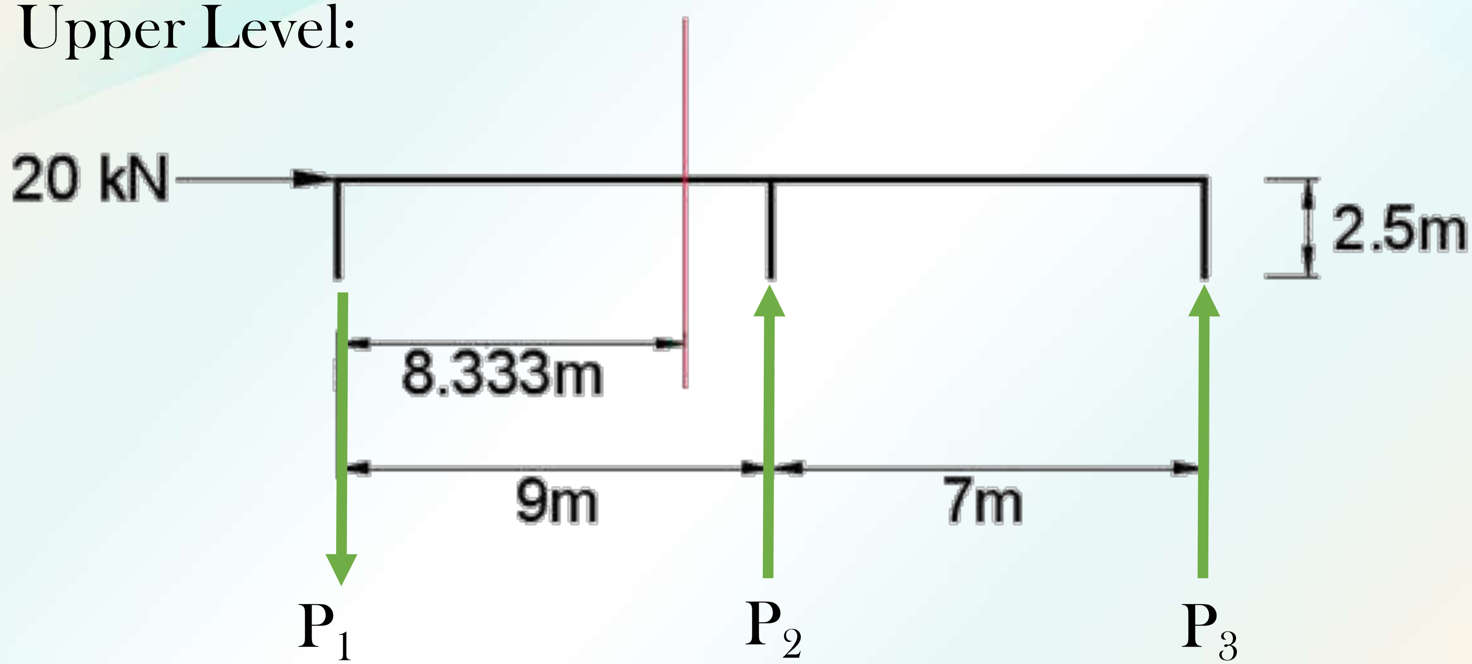
Solve for the Inertia "I"

$$I = 1.0(8.333)^2 + 1.0(0.667)^2 + 1.0(7.667)^2$$

$$I = 128.667$$

Problem 1 - Solution

Isolate the Upper Level:

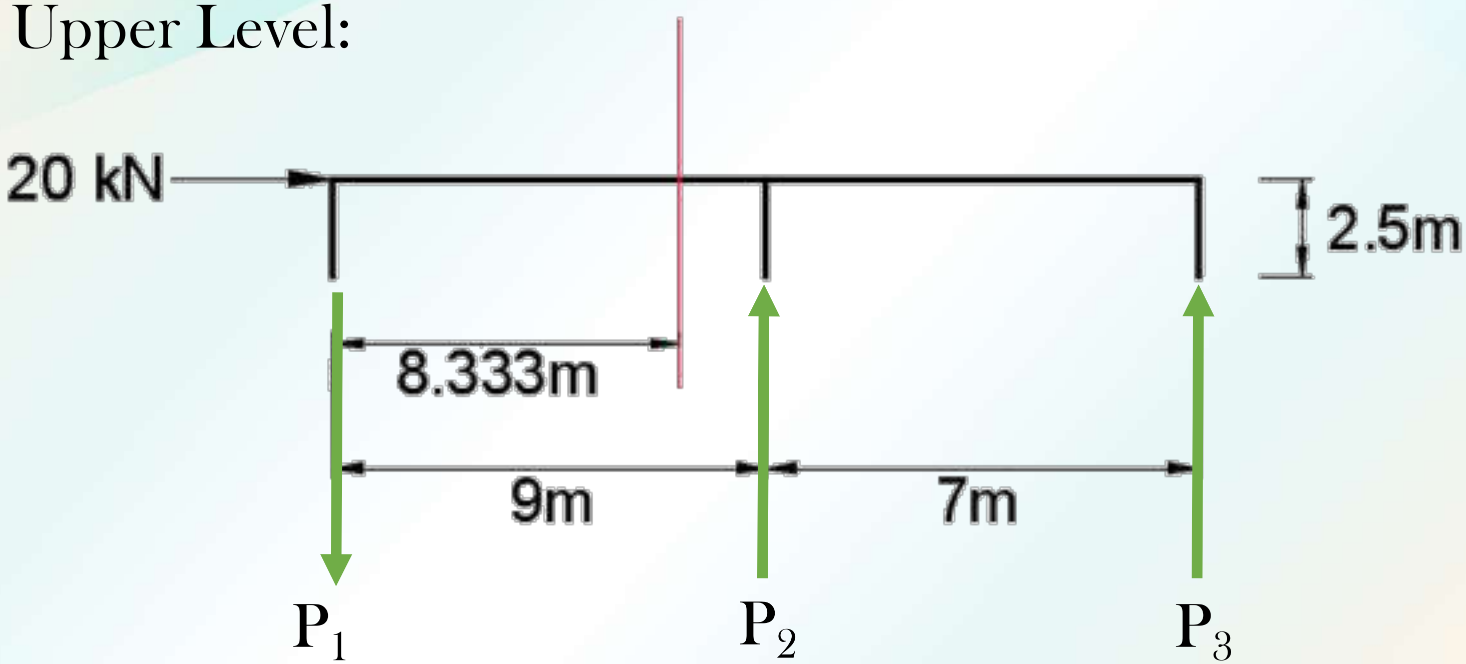


Solve for P_1

$$P_1 = f_1 A_1 \qquad P_1 = \left(\frac{Mc}{I}\right) A_1 \qquad P_1 = \left[\frac{50(8.333)}{128.667}\right](1.0) \qquad P_1 = 3.238\,kN$$

Problem 1 - Solution

Isolate the Upper Level:

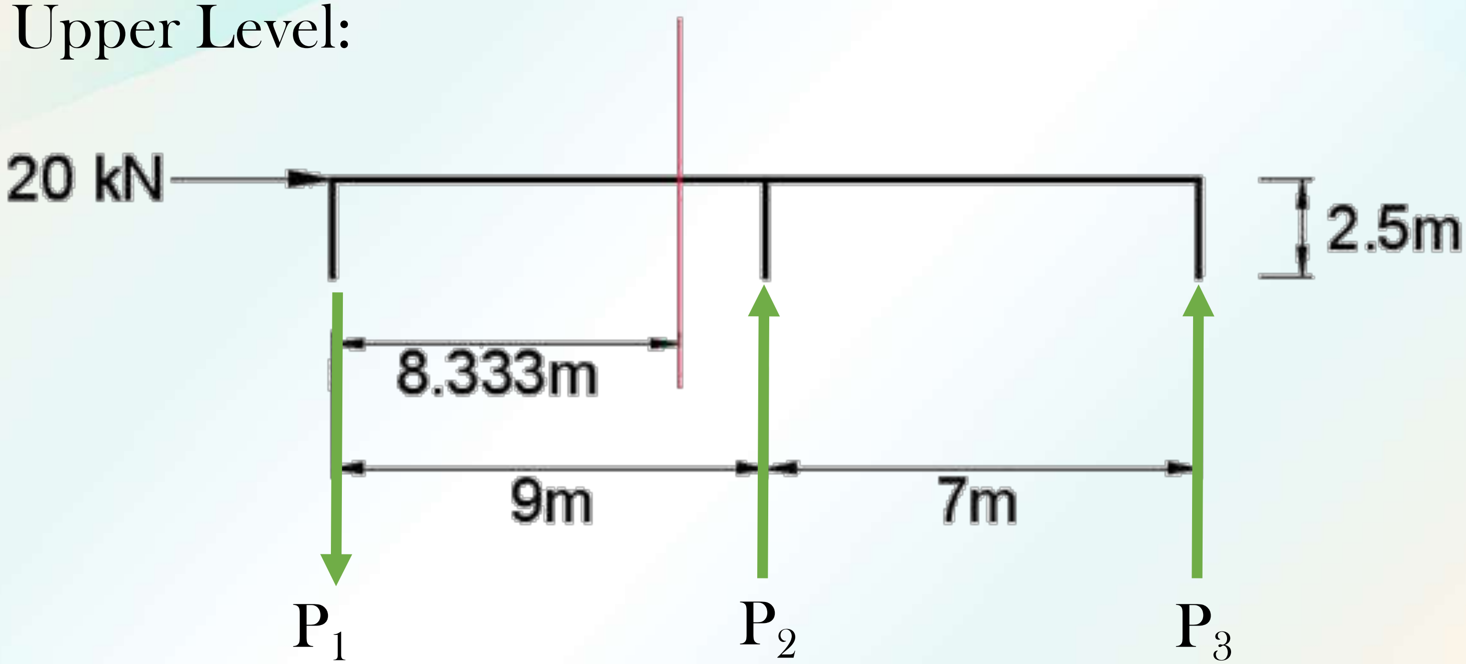


Solve for P_2

$$P_2 = f_2 A_2 \qquad P_2 = \left(\frac{Mc_2}{I}\right)A_2 \qquad P_2 = \left[\frac{50(0.667)}{128.667}\right](1.0) \qquad P_2 = 0.259kN$$

Problem 1 - Solution

Isolate the Upper Level:

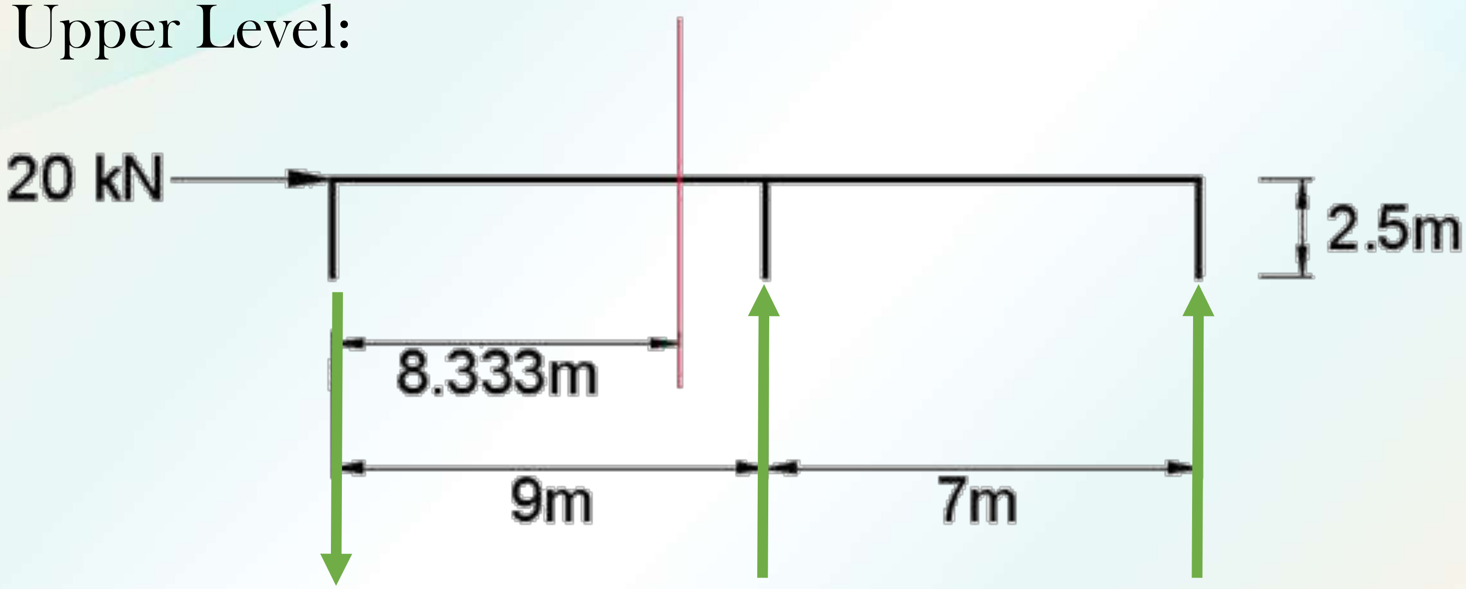


Solve for P_3

$$P_3 = f_3 A_3 \qquad P_3 = \left(\frac{Mc_3}{I}\right)A_3 \qquad P_3 = \left[\frac{50(7.667)}{128.667}\right](1.0) \qquad P_3 = 2.979kN$$

Problem 1 - Solution

Isolate the Upper Level:



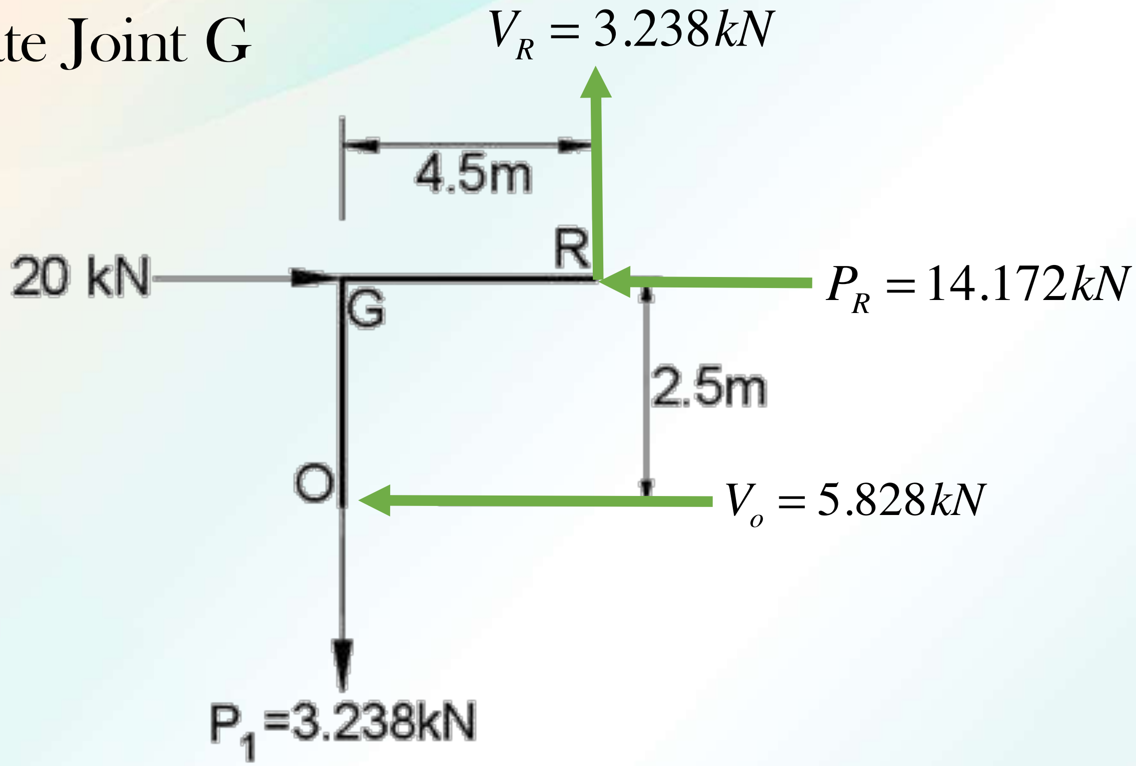
$P_1 = 3.238kN$ $P_2 = 0.259kN$ $P_3 = 2.979kN$

Checking:

$$P_1 = P_2 + P_3$$

Problem 2 - Solution

Isolate Joint G



Use 3ME Equations

$$\sum M_R = 0$$

$$-3.238(4.5) + V_o(2.5) = 0$$

$$V_o = 5.828kN$$

$$\sum F_v = 0$$

$$-3.238 + V_R = 0$$

$$V_R = 3.238kN$$

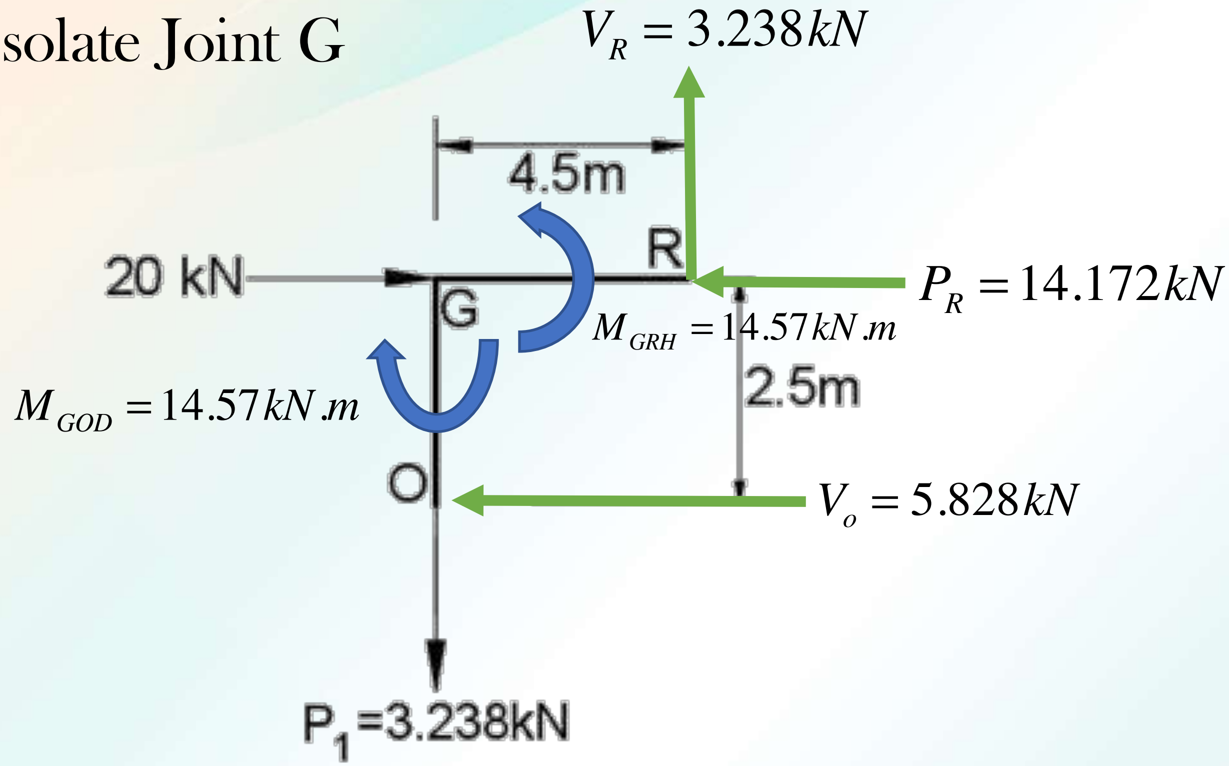
$$\sum F_H = 0$$

$$20 - 5.828 - P_R = 0$$

$$P_R = 14.172kN$$

Problem 2 - Solution

Isolate Joint G



Use 3ME Equations

$$\sum M_R = 0$$

$$-3.238(4.5) + V_o(2.5) = 0$$

$$V_o = 5.828kN$$

$$\sum F_v = 0$$

$$-3.238 + V_R = 0$$

$$V_R = 3.238kN$$

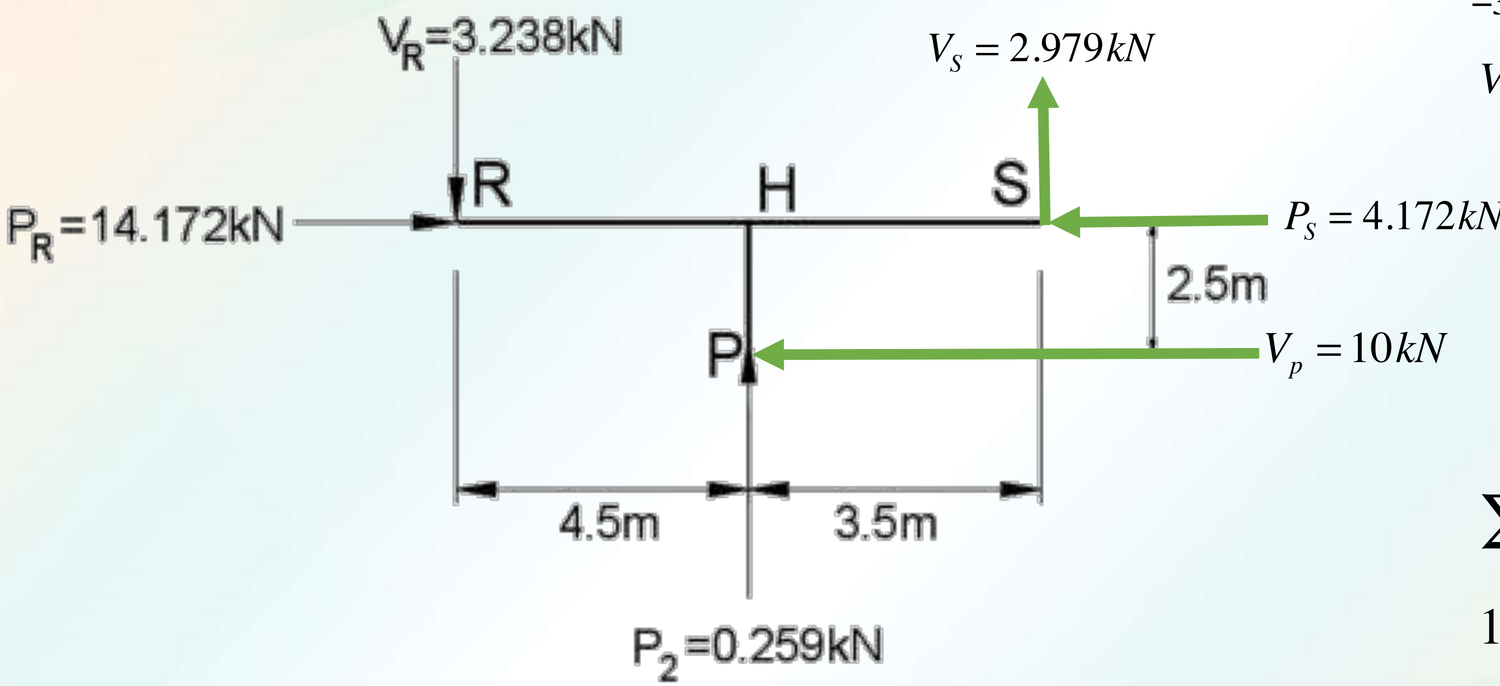
$$\sum F_H = 0$$

$$20 - 5.828 - P_R = 0$$

$$P_R = 14.172kN$$

Problem 2 - Solution

Isolate Joint H



Use 3ME Equations

$$\sum M_s = 0$$

$$-3.238(8)+0.259(3.5)+V_p(2.5)=0$$

$$V_p = 10kN$$

$$\sum F_v = 0$$

$$-3.238+0.259+V_S=0$$

$$V_S = 2.979kN$$

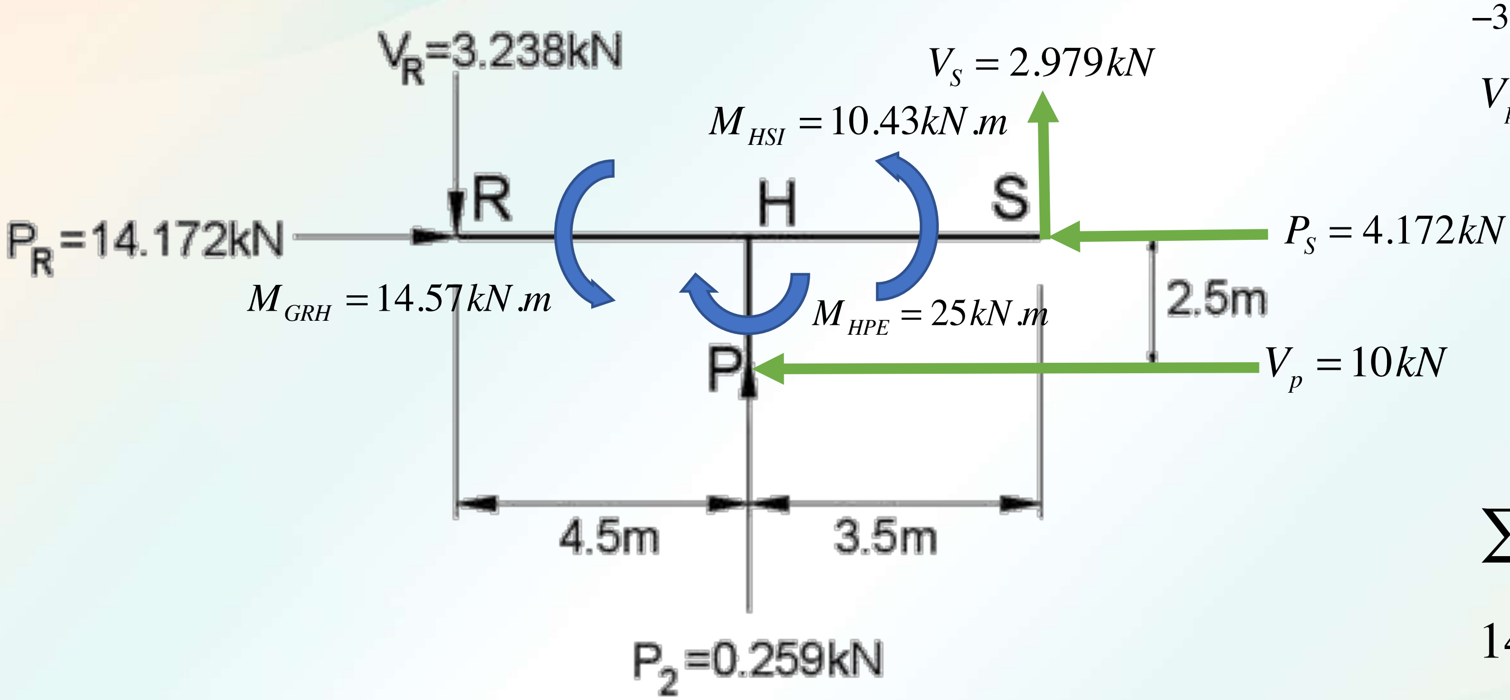
$$\sum F_H = 0$$

$$14.172-10-P_S=0$$

$$P_S = 4.172kN$$

Problem 2 - Solution

Isolate Joint H



Use 3ME Equations

$$\sum M_s = 0$$

$$-3.238(8)+0.259(3.5)+V_p(2.5)=0$$

$$V_p = 10kN$$

$$\sum F_v = 0$$

$$-3.238+0.259+V_S = 0$$

$$V_S = 2.979kN$$

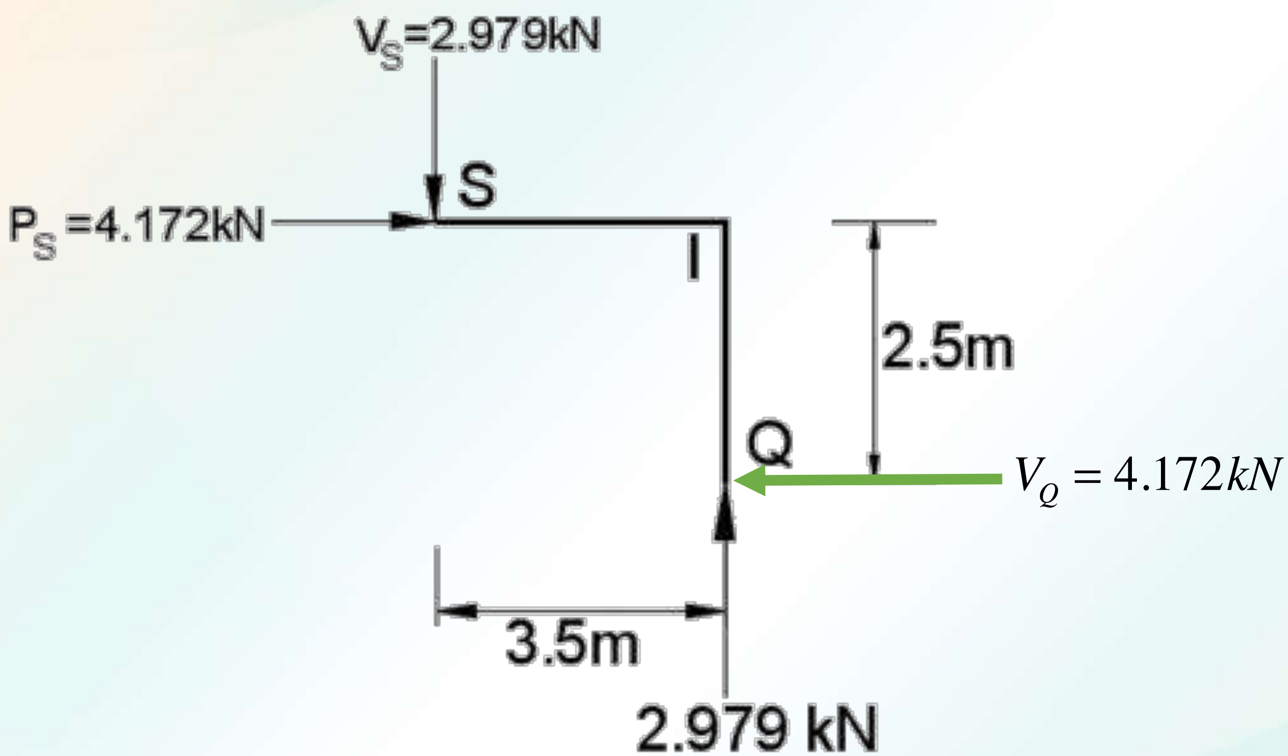
$$\sum F_H = 0$$

$$14.172-10-P_S = 0$$

$$P_S = 4.172kN$$

Problem 2 - Solution

Isolate Joint I



Use 3ME Equations

$$\sum F_v = 0$$

$$-V_s + 2.979 = 0$$

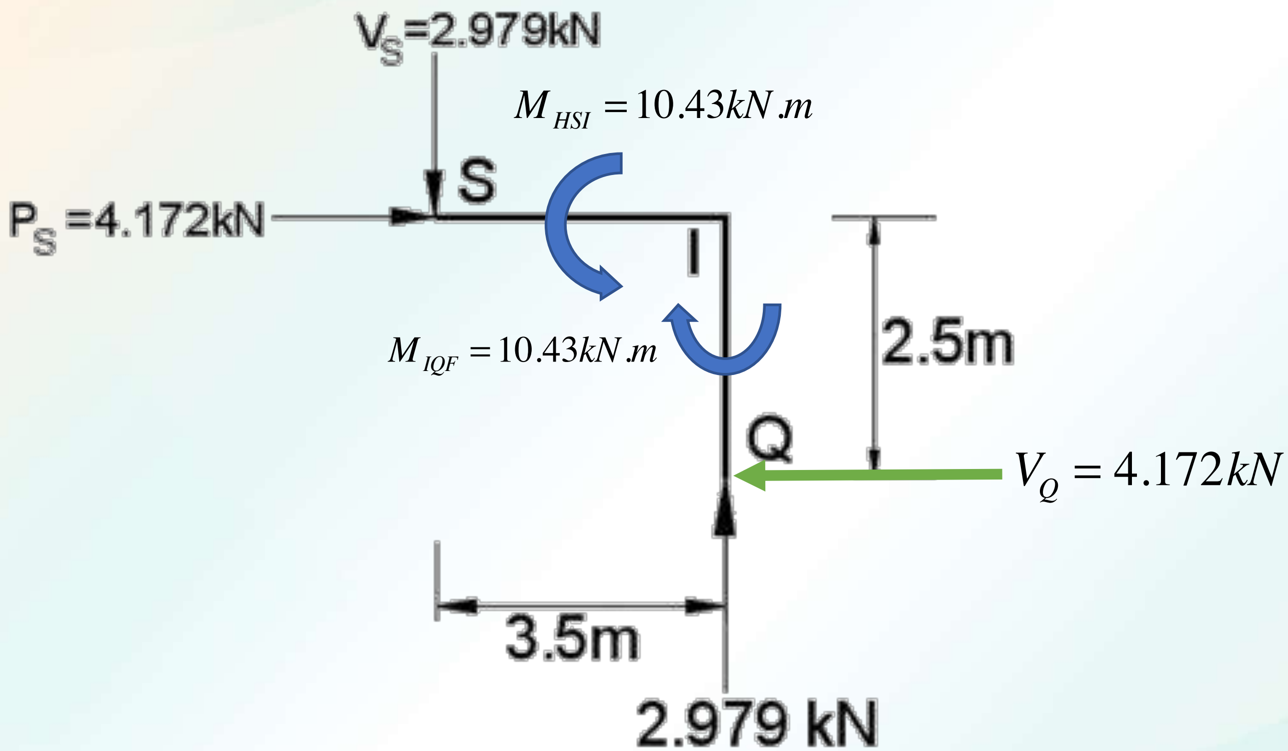
$$\sum F_H = 0$$

$$4.172 - V_Q = 0$$

$$V_Q = 4.172kN$$

Problem 2 - Solution

Isolate Joint I



Use 3ME Equations

$$\sum F_v = 0$$

$$-V_s + 2.979 = 0$$

$$\sum F_H = 0$$

$$4.172 - V_Q = 0$$

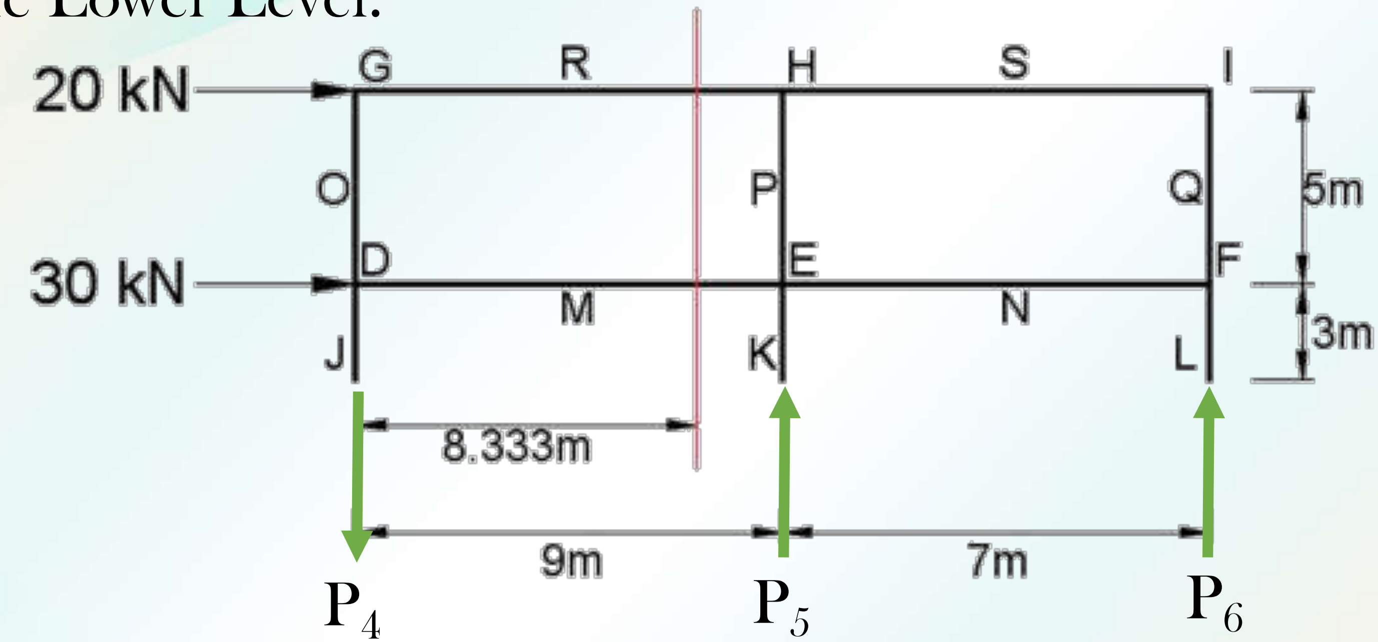
$$V_Q = 4.172 kN$$

Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Problem 2 - Solution

Isolate the Lower Level:



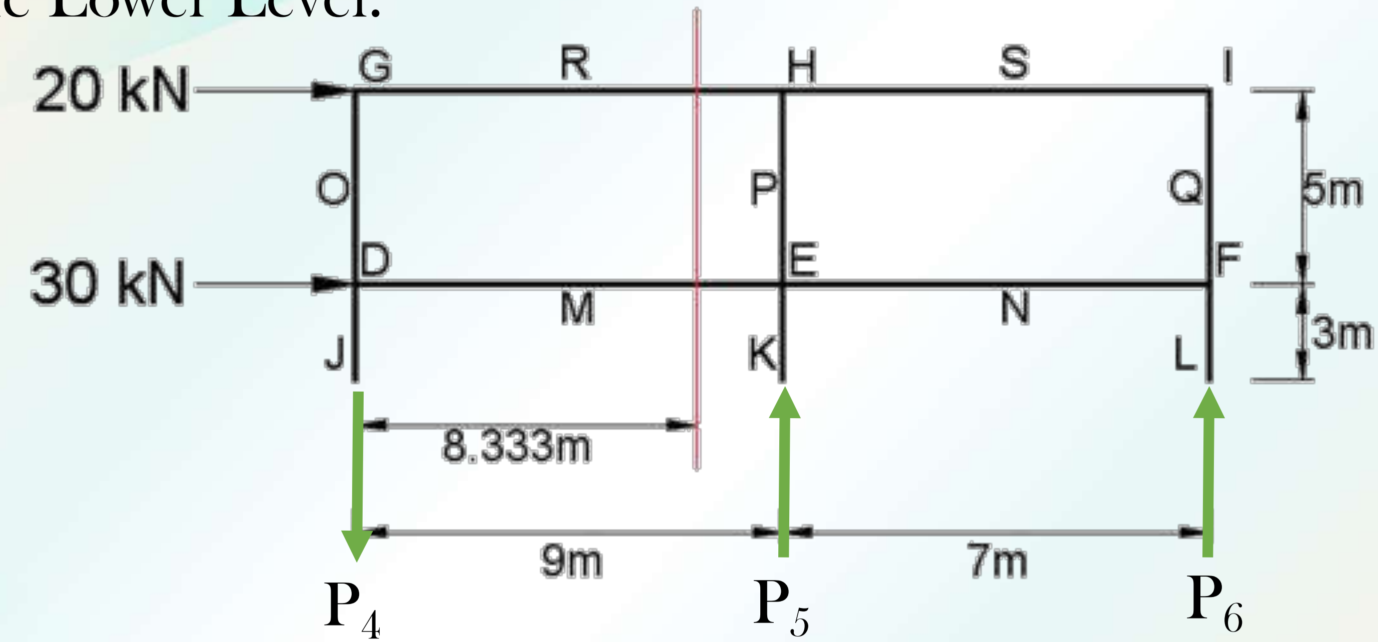
Locate the centroid of the group of columns:

$$3A\left(\bar{x}\right)=A\left(0\right)+A\left(9\right)+A\left(16\right)$$

$$\bar{x}=8.333m$$

Problem 2 - Solution

Isolate the Lower Level:



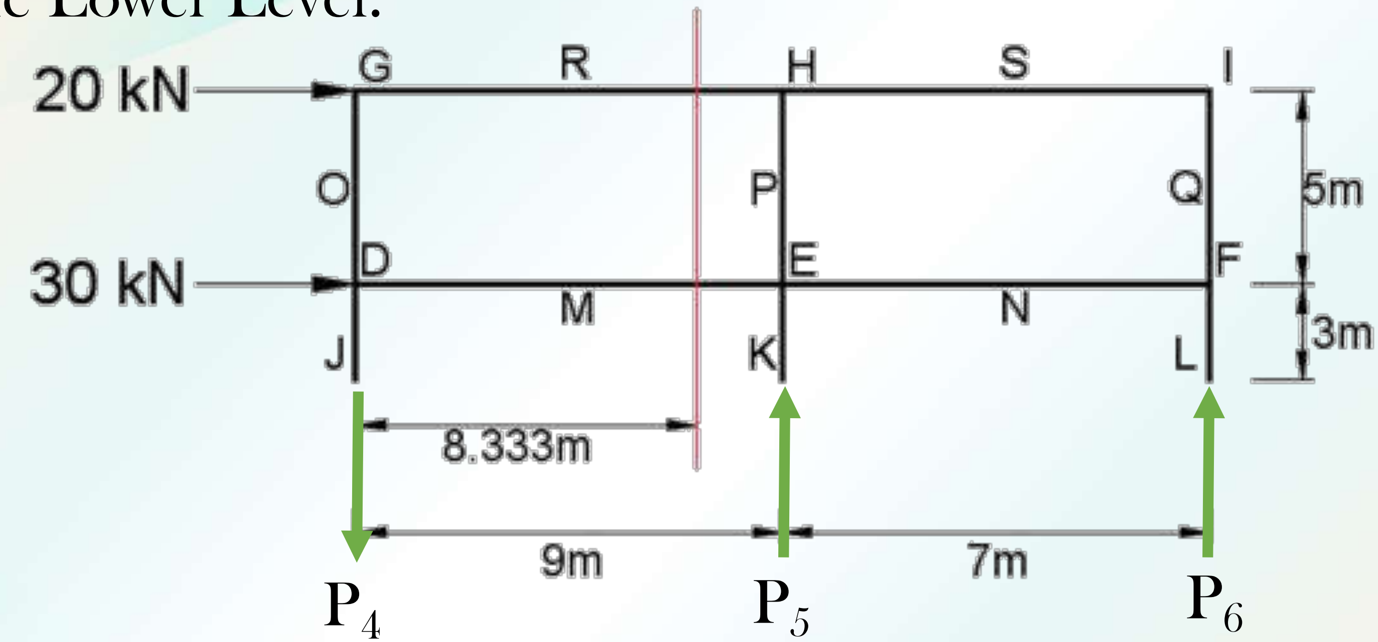
Solve for the Moment "M"

$$M = 20(8) + 30(3)$$

$$M = 250 kN.m$$

Problem 2 - Solution

Isolate the Lower Level:



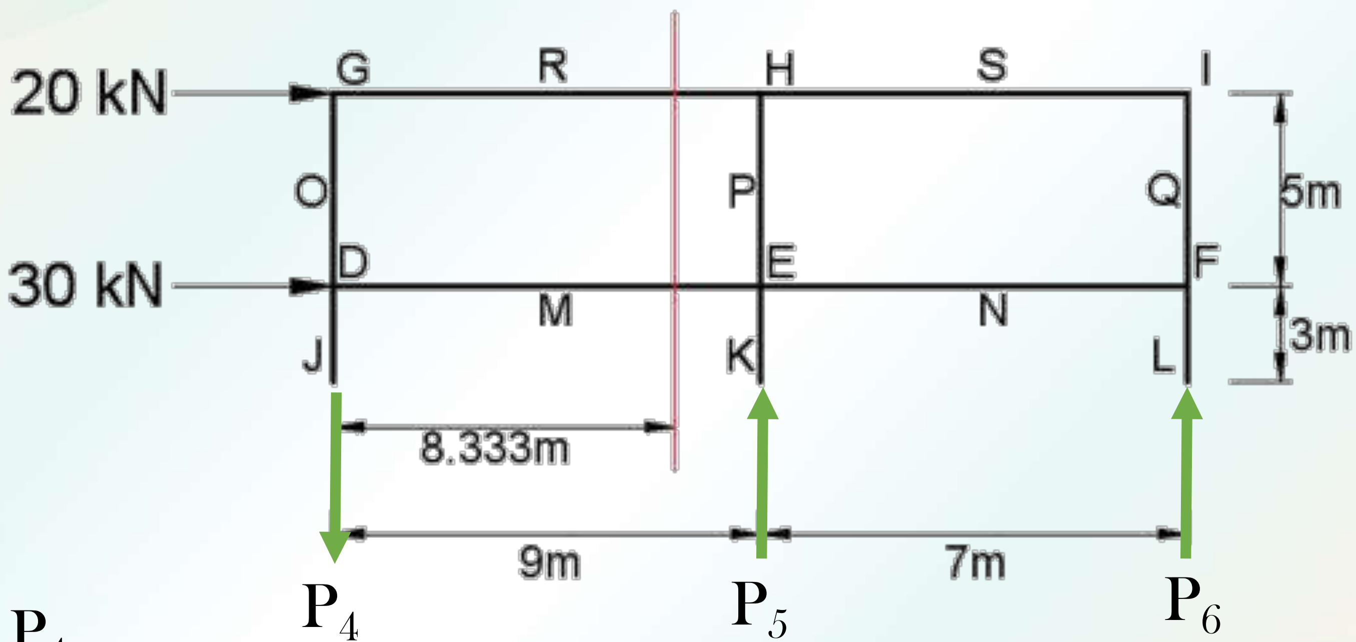
Solve for the Inertia "I"

$$I = 1.0(8.333)^2 + 1.0(0.667)^2 + 1.0(7.667)^2$$

$$I = 128.667$$

Problem 2 - Solution

Isolate the Lower Level:

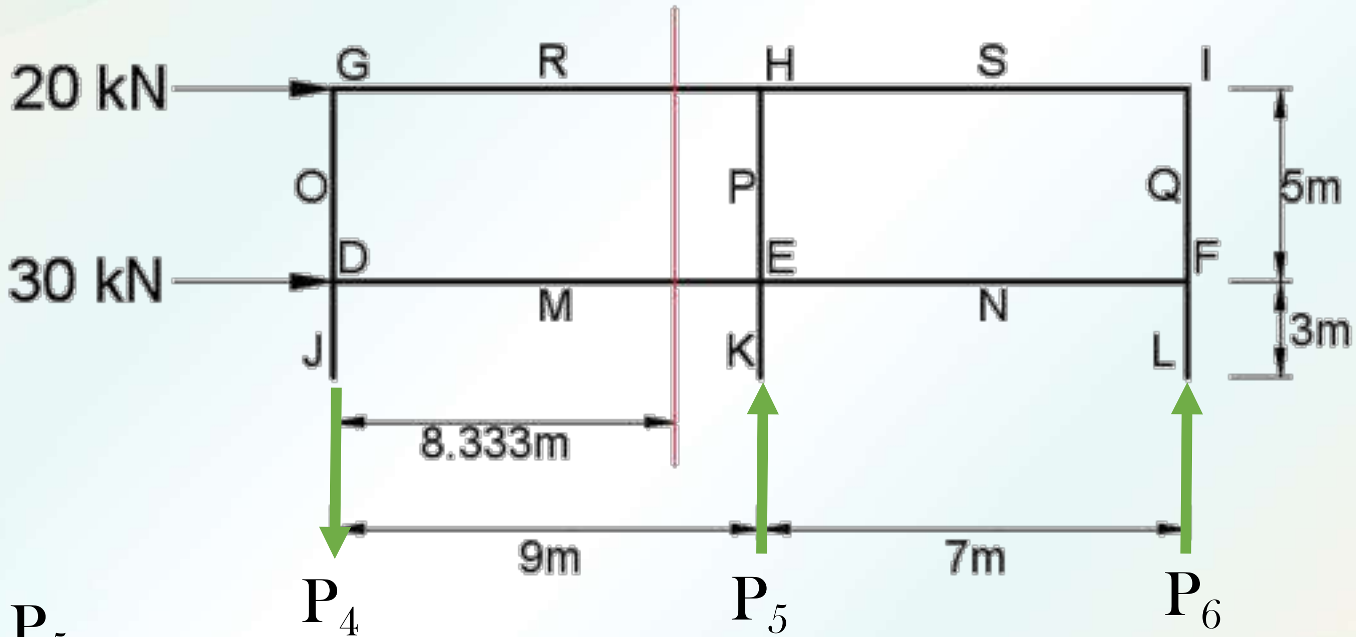


Solve for P_4

$$P_4 = f_4 A_4 \qquad P_4 = \left(\frac{Mc_4}{I}\right)A_4 \qquad P_4 = \left[\frac{(250)(8.333)}{128.667}\right](1.0) \qquad P_4 = 16.191kN$$

Problem 2 - Solution

Isolate the Lower Level:



Solve for P_5

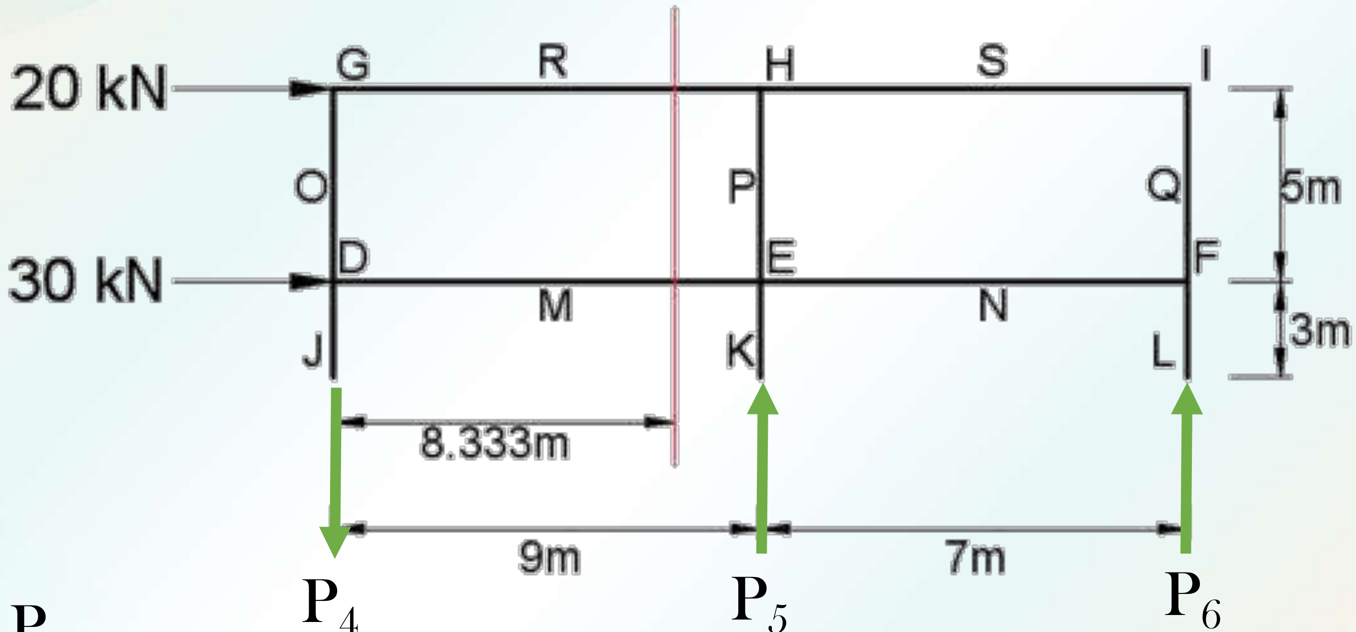
$$P_5 = f_5 A_5 \qquad P_5 = \left(\frac{Mc_5}{I}\right) A_5 \qquad P_5 = \left[\frac{(250)(0.667)}{128.667}\right](1.0) \qquad P_5 = 1.296kN$$

Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Problem 2 - Solution

Isolate the Lower Level:

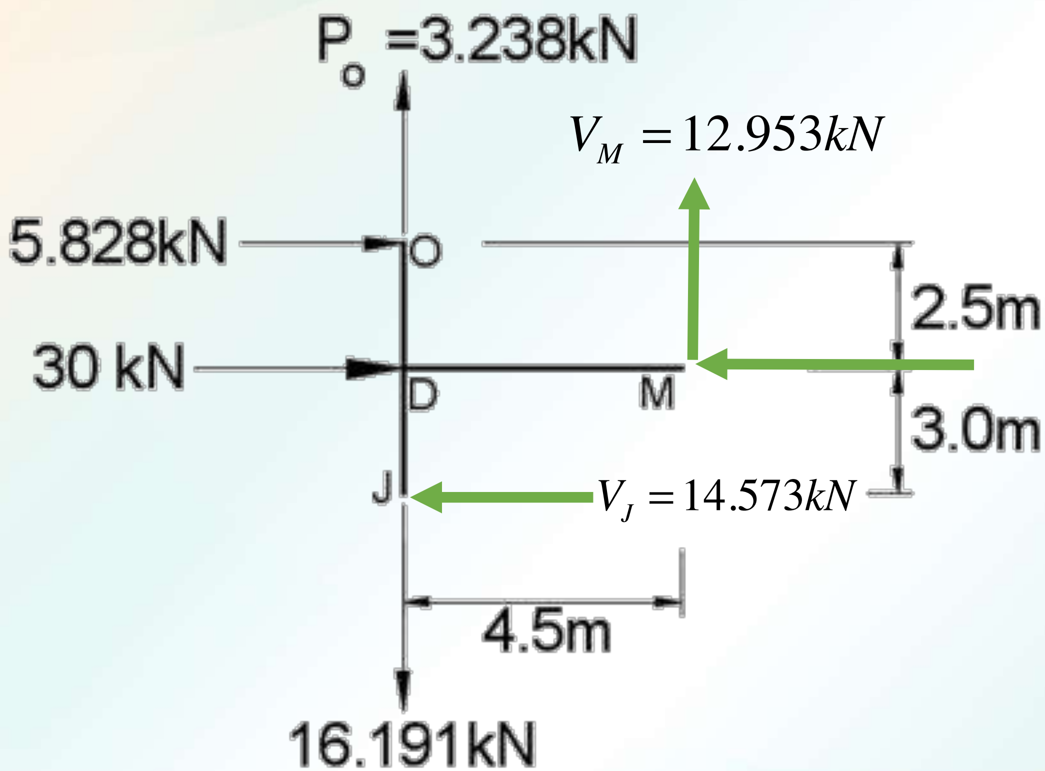


Solve for P_6

$$P_6 = f_6 A_6 \qquad P_6 = \left(\frac{Mc_6}{I}\right)A_6 \qquad P_6 = \left[\frac{(250)(7.667)}{128.667}\right](1.0) \qquad P_6 = 14.895\,kN$$

Problem 2 - Solution

Isolate Joint D:



$$\sum M_M = 0$$

$$-16.191(4.5) + 3.238(4.5) + 5.828(2.5) + V_J(3) = 0$$

$$V_J = 14.573kN$$

$$\sum F_v = 0$$

$$3.238 - 16.191 + V_M = 0$$

$$V_M = 12.953kN$$

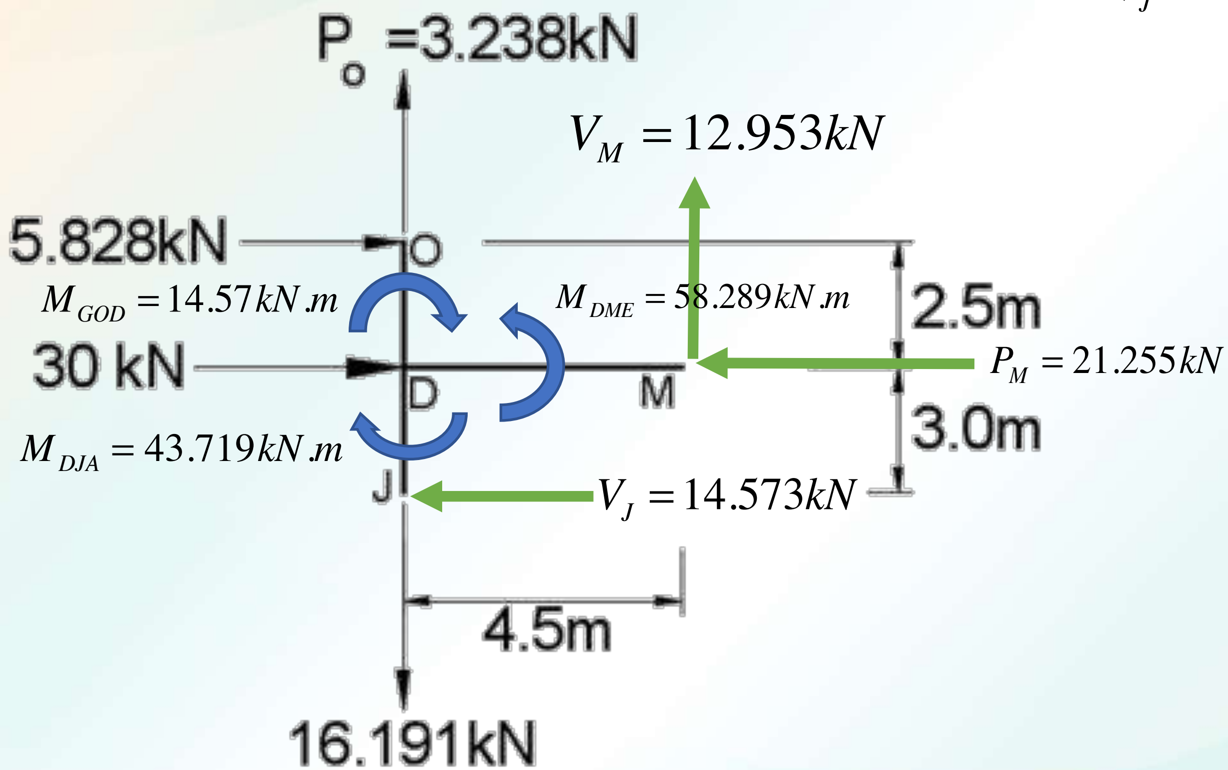
$$\sum F_H = 0$$

$$5.828 + 30 - 14.573 - P_M = 0$$

$$P_M = 21.255kN$$

Problem 2 - Solution

Isolate Joint D:



$$\sum M_M = 0$$

$$-16.191(4.5) + 3.238(4.5) + 5.828(2.5) + V_J(3) = 0$$

$$V_J = 14.573kN$$

$$\sum F_v = 0$$

$$3.238 - 16.191 + V_M = 0$$

$$V_M = 12.953kN$$

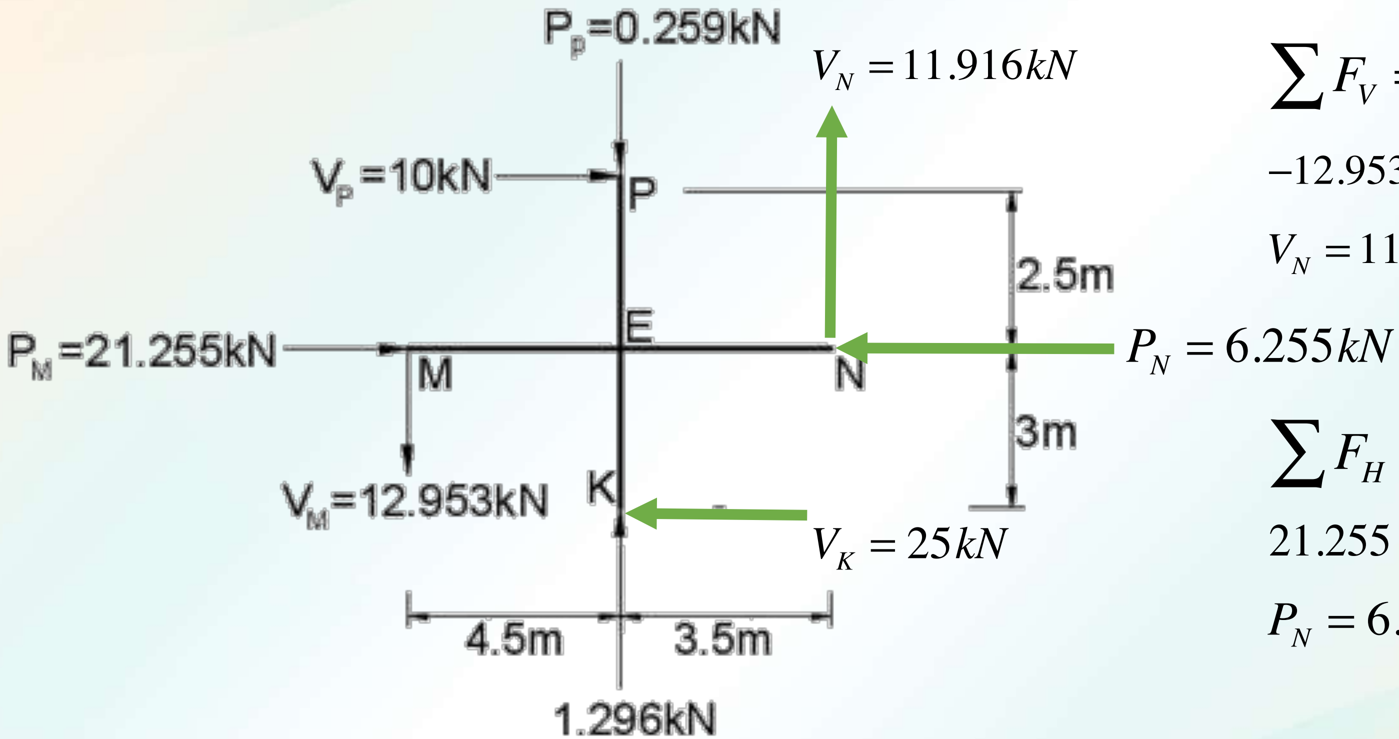
$$\sum F_H = 0$$

$$5.828 + 30 - 14.573 - P_M = 0$$

$$P_M = 21.255kN$$

Problem 2 - Solution

Isolate Joint E:



$$\sum M_N = 0$$

$$-12.953(8) - 0.259(3.5) + 1.296(3.5) + 10(2.5) + V_K(3) = 0$$

$$V_K = 25kN$$

$$\sum F_V = 0$$

$$-12.953 - 0.259 + 1.296 + V_N = 0$$

$$V_N = 11.916kN$$

$$\sum F_H = 0$$

$$21.255 + 10 - 25 - P_N = 0$$

$$P_N = 6.255kN$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

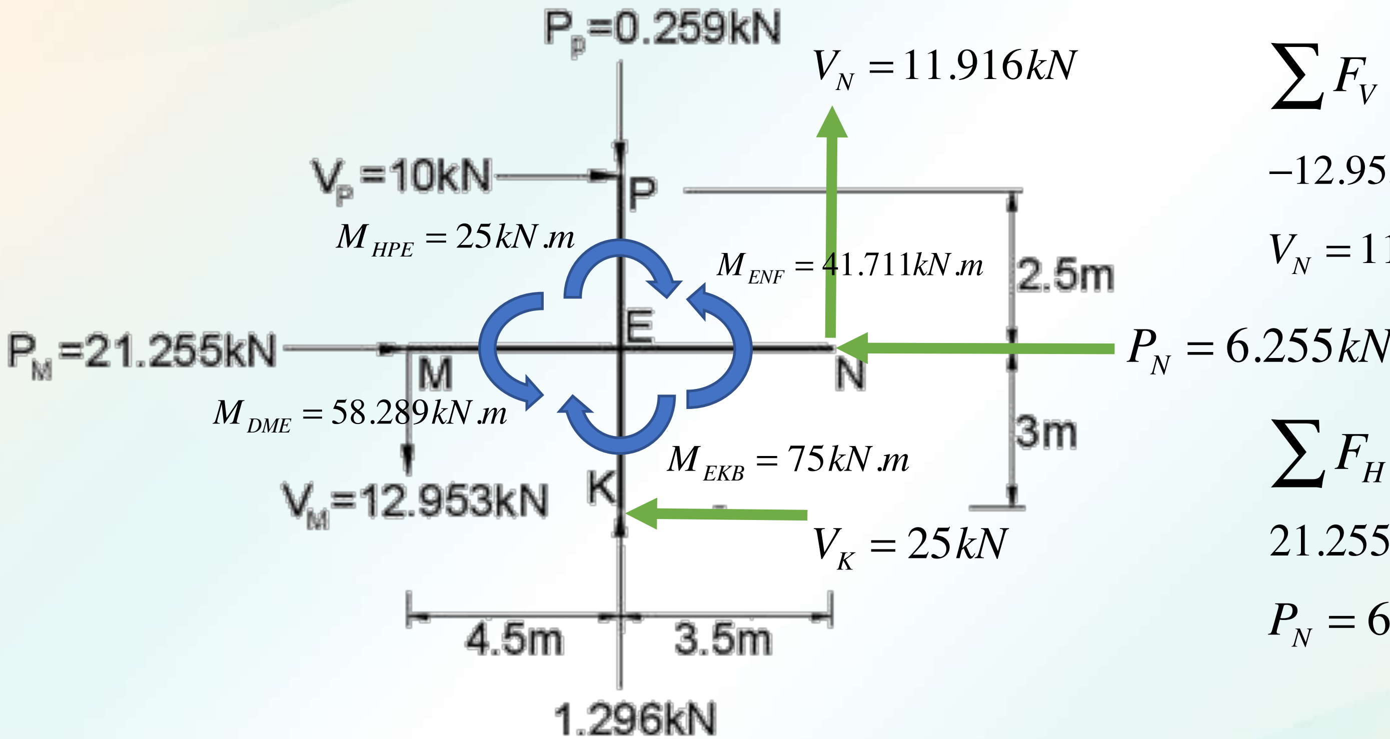
Problem 2 - Solution

Isolate Joint E:

$$\sum M_N = 0$$

$$-12.953(8) - 0.259(3.5) + 1.296(3.5) + 10(2.5) + V_K(3) = 0$$

$$V_K = 25kN$$



$$\sum F_V = 0$$

$$-12.953 - 0.259 + 1.296 + V_N = 0$$

$$V_N = 11.916kN$$

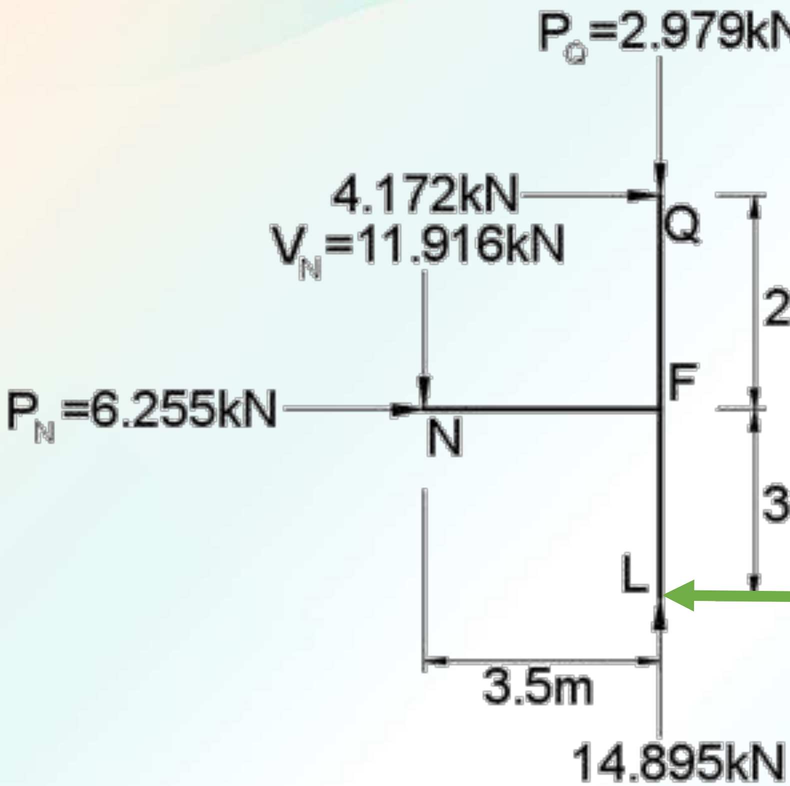
$$\sum F_H = 0$$

$$21.255 + 10 - 25 - P_N = 0$$

$$P_N = 6.255kN$$

Problem 2 - Solution

Isolate Joint F:



$V_L = 10.427kN$

$$\sum F_H = 0$$

$$4.172 + 6.255 - V_L = 0$$

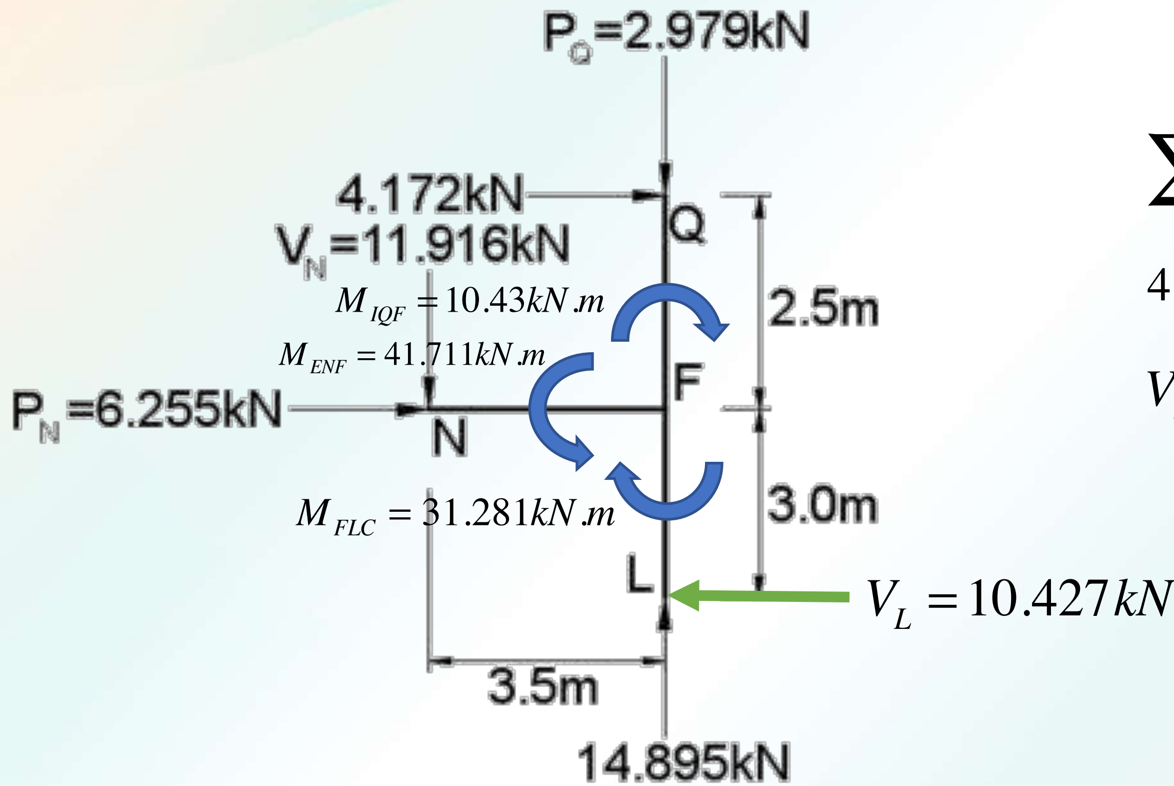
$$V_L = 10.427kN$$

Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Problem 2 - Solution

Isolate Joint F:



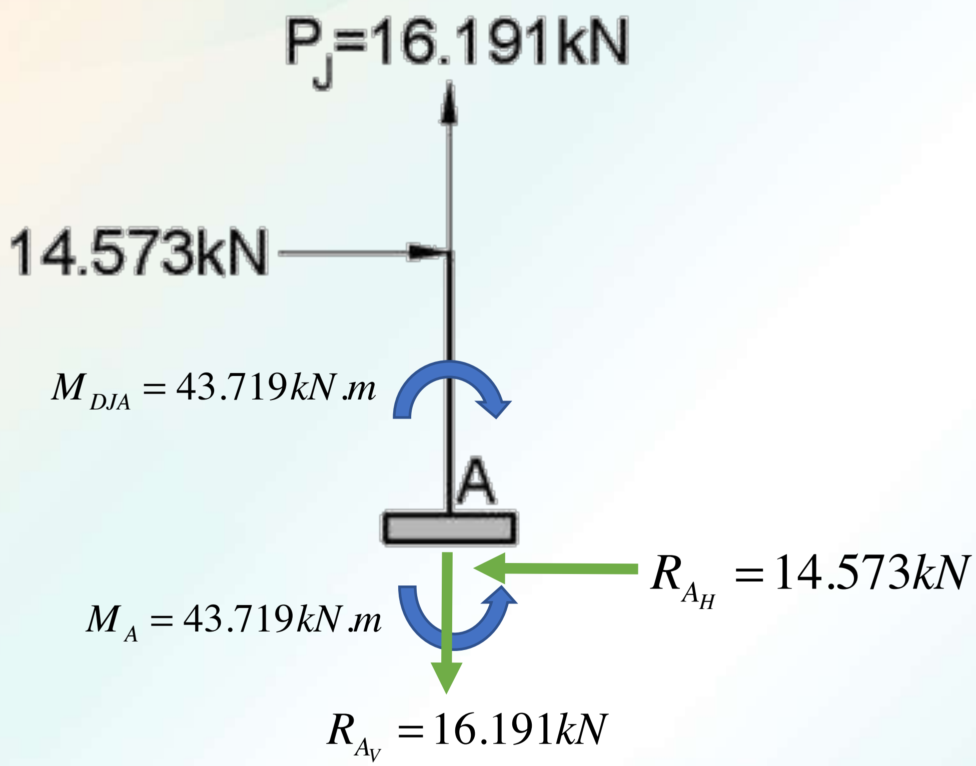
$$\sum F_H = 0$$

$$4.172 + 6.255 - V_L = 0$$

$$V_L = 10.427kN$$

Problem 2 - Solution

Isolate Joint A:



$$\sum F_V = 0$$

$$16.191 - R_{A_V} = 0$$

$$R_{A_V} = 16.191kN$$

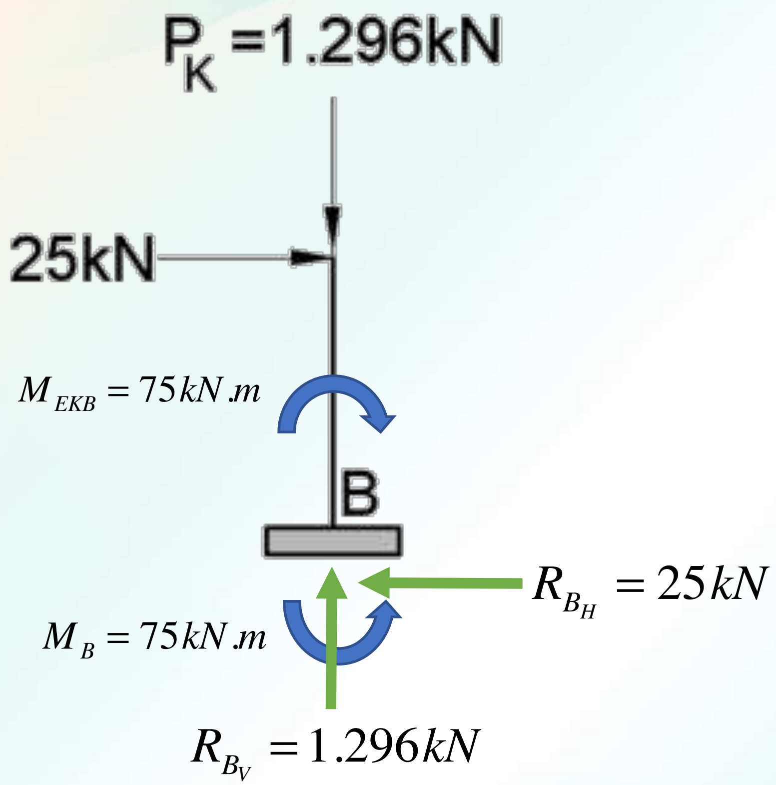
$$\sum F_H = 0$$

$$14.573 - R_{A_H} = 0$$

$$R_{A_H} = 14.573kN$$

Problem 2 - Solution

Isolate Joint B:



$$\sum F_V = 0$$

$$-1.296 + R_{B_V} = 0$$

$$R_{B_V} = 1.296kN$$

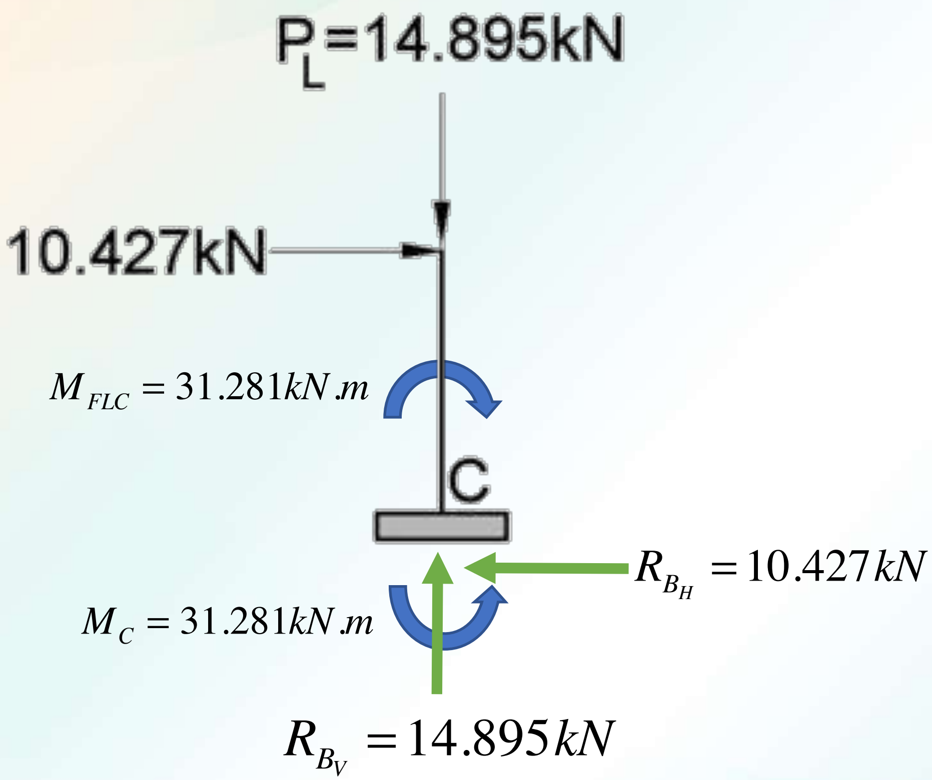
$$\sum F_H = 0$$

$$25 - R_{B_H} = 0$$

$$R_{B_H} = 25kN$$

Problem 2 - Solution

Isolate Joint C:



$$\sum F_V = 0$$

$$-14.895 + R_{B_V} = 0$$

$$R_{B_V} = 14.895 kN$$

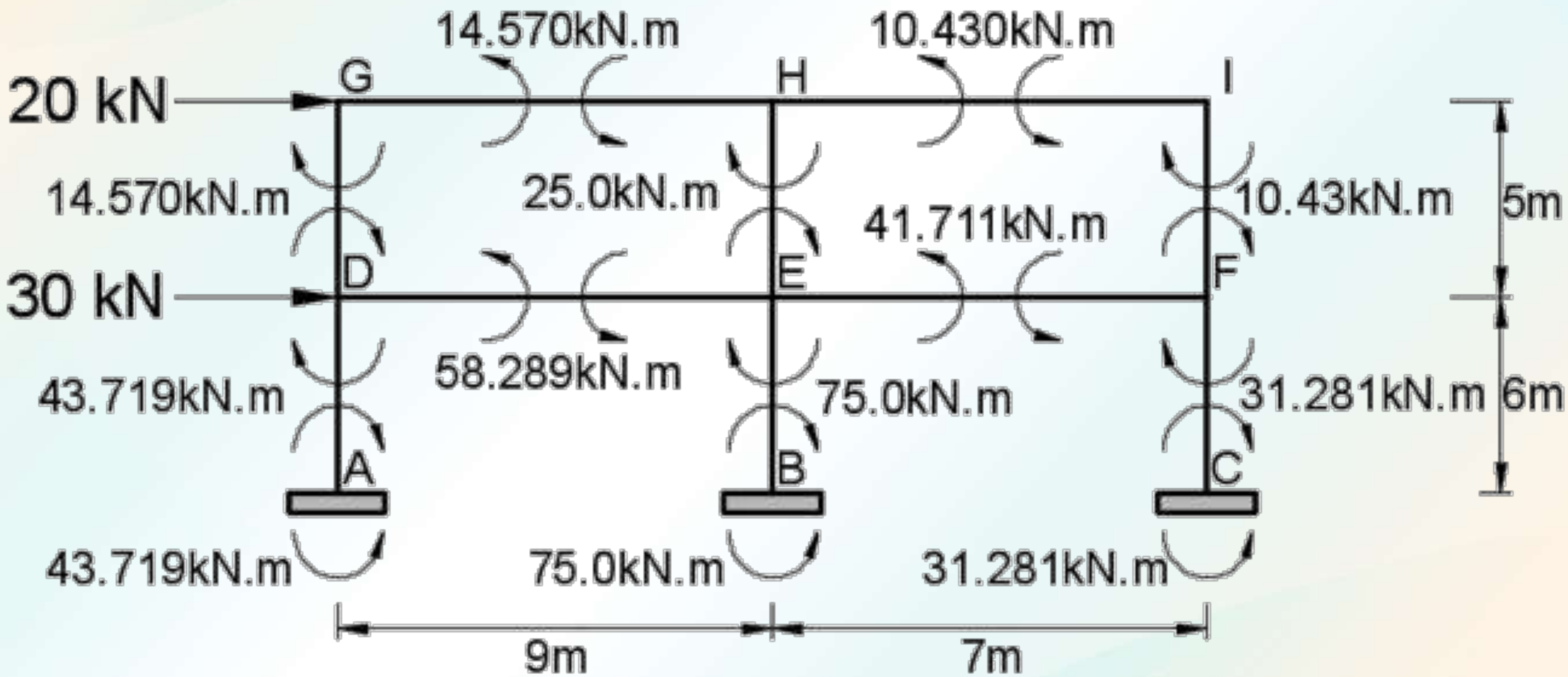
$$\sum F_H = 0$$

$$10.427 - R_{B_H} = 0$$

$$R_{B_H} = 10.427 kN$$

Problem 3 - Solution

Final Beam and Column Moments:

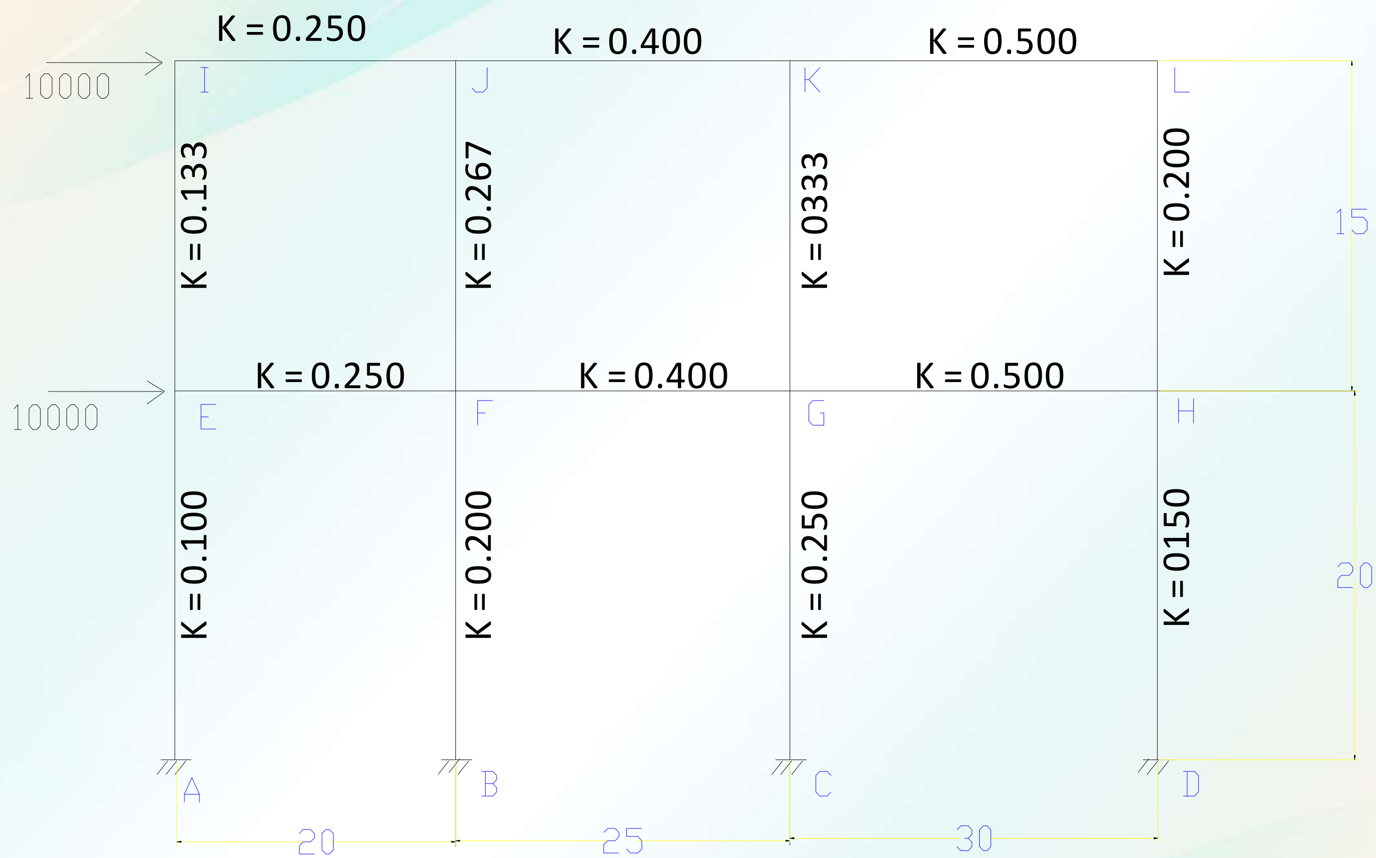


COMPUTATION OF THE STRUCTURES' SHEARS, MOMENTS AND REACTIONS DUE TO EARTHQUAKE

Sample Problems in Factor Method

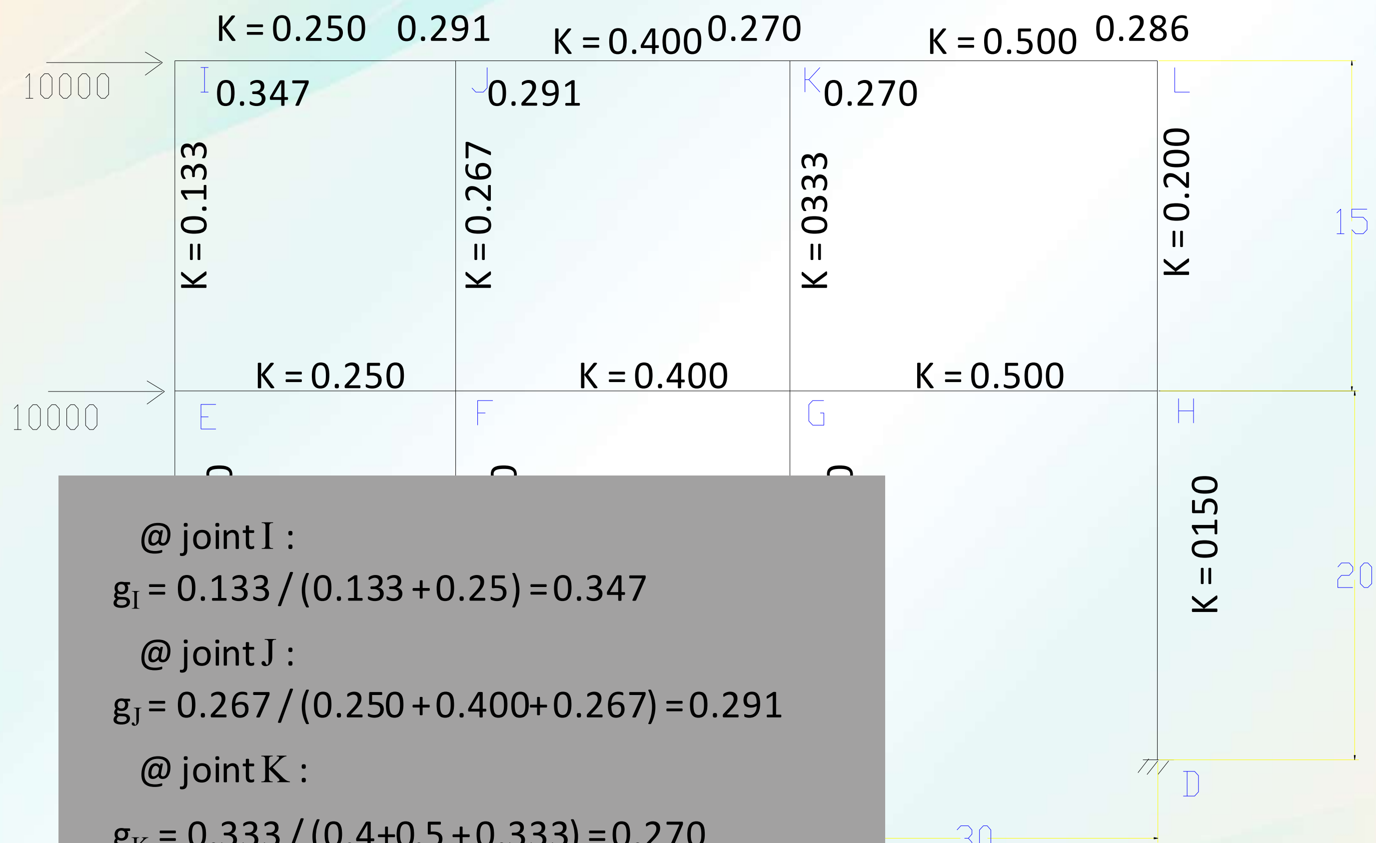
FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation



Problem. Determine the End Moments of the Frame shown using Factor Method.

Step 1. g = Σ Kc / Σ K



@ joint I :

$g_I = 0.133 / (0.133 + 0.25) = 0.347$

@ joint J :

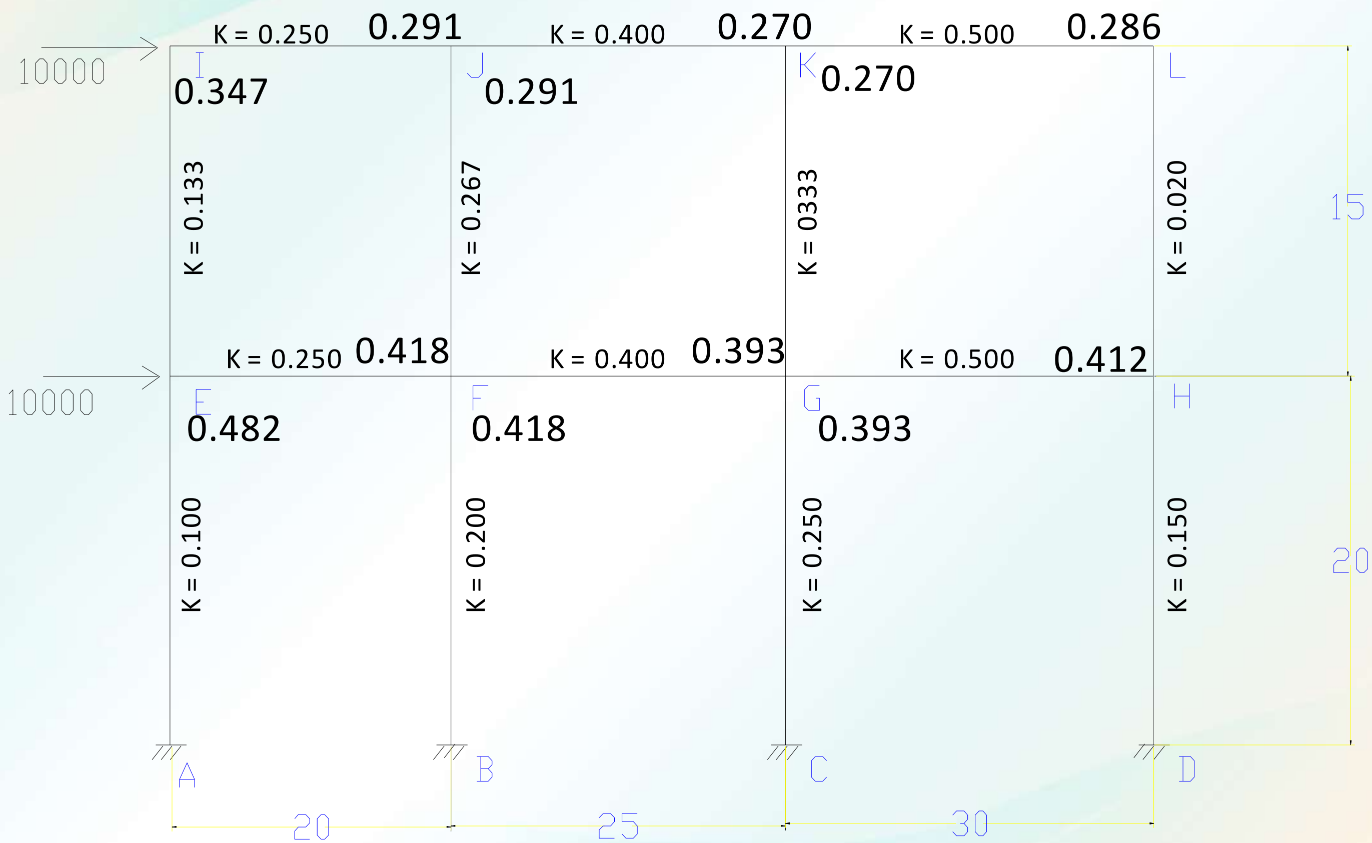
$g_J = 0.267 / (0.250 + 0.400 + 0.267) = 0.291$

@ joint K :

$g_K = 0.333 / (0.4 + 0.5 + 0.333) = 0.270$

@ joint L :

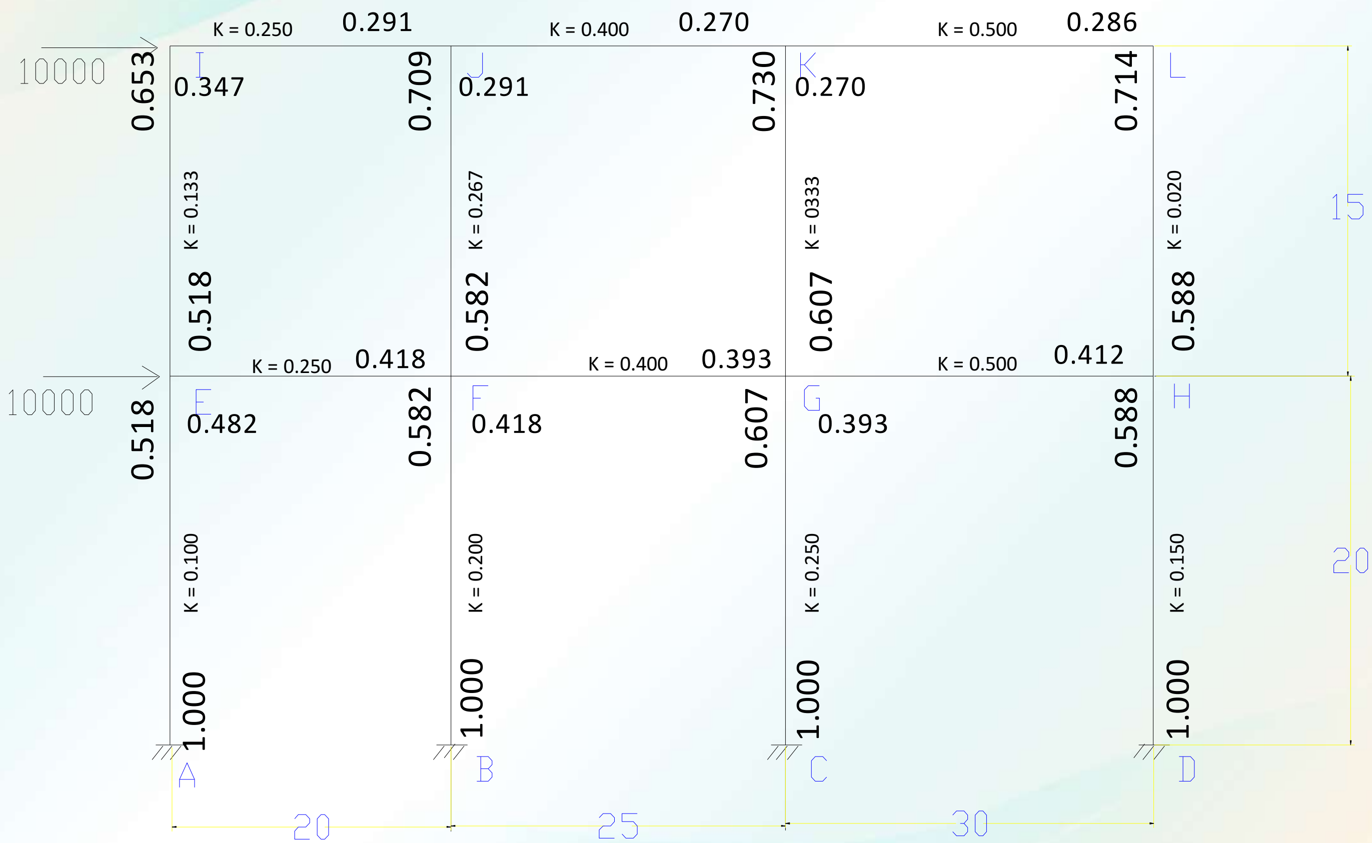
$g_L = 0.200 / (0.500 + 0.200) = 0.286$



FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

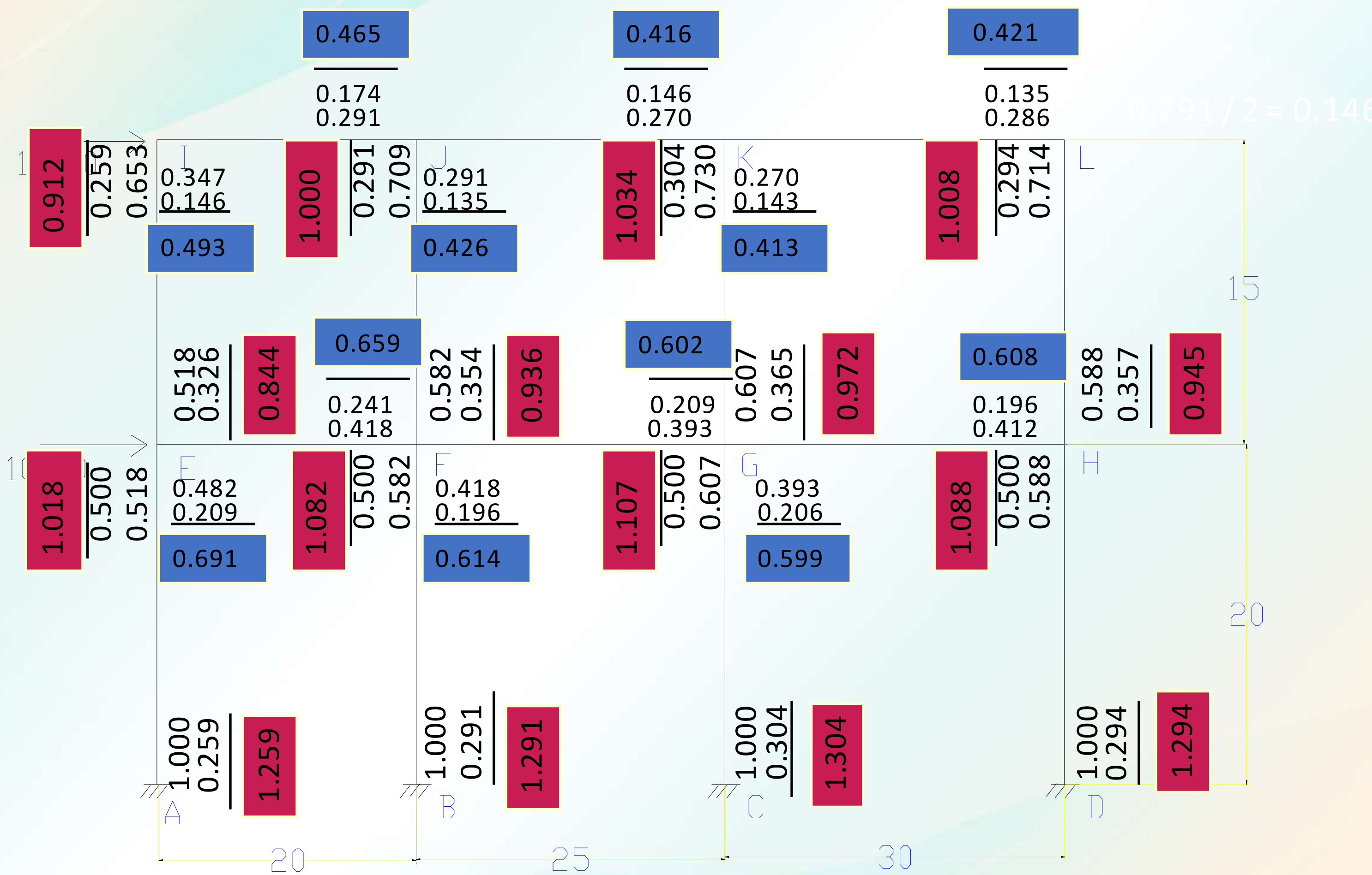
Step 2. $c = 1\text{-}g$



FEU ALABANG FEU DILIMAN FEU TECH

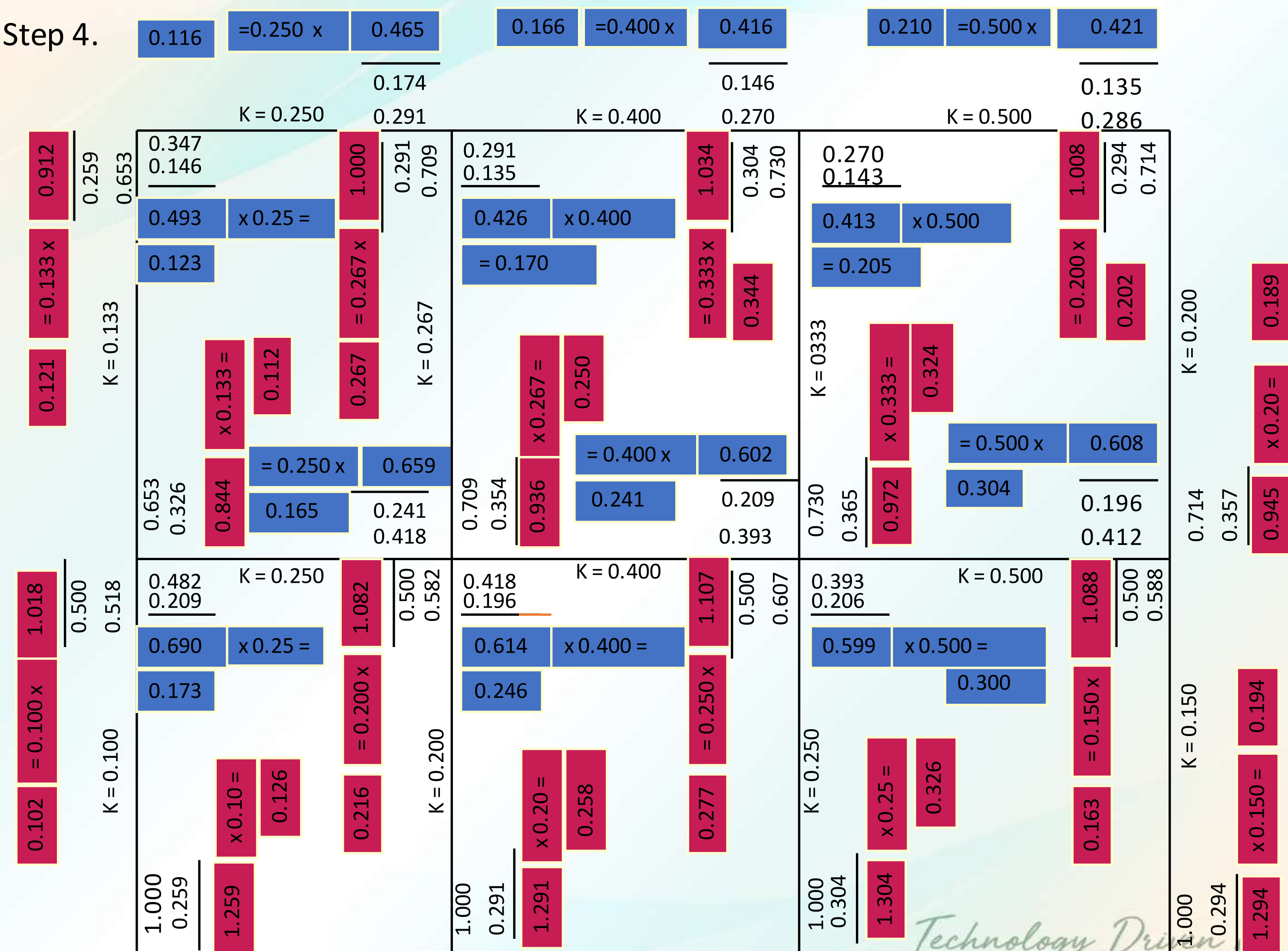
Technology Driven by Innovation

Step 3.



0.465
0.174
0.291
0.416
0.146
0.270
0.421
0.135
0.286
0.912
0.259
0.653
I
0.347
0.146
0.493
1.000
0.291
0.709
J
0.291
0.135
0.426
1.034
0.304
0.730
K
0.270
0.143
0.413
1.008
0.294
0.714
L
15
0.518
0.326
0.844
0.659
0.241
0.418
0.582
0.354
0.936
0.602
0.209
0.393
0.607
0.365
0.972
0.608
0.196
0.412
0.588
0.357
0.945
1.018
0.500
0.518
E
0.482
0.209
0.691
1.082
0.500
0.582
F
0.418
0.196
0.614
1.107
0.500
0.607
G
0.393
0.206
0.599
1.088
0.500
0.588
H
20
1.000
0.259
1.259
1.000
0.291
1.291
1.000
0.304
1.304
1.000
0.294
1.294
A
B
C
D
20
25
30

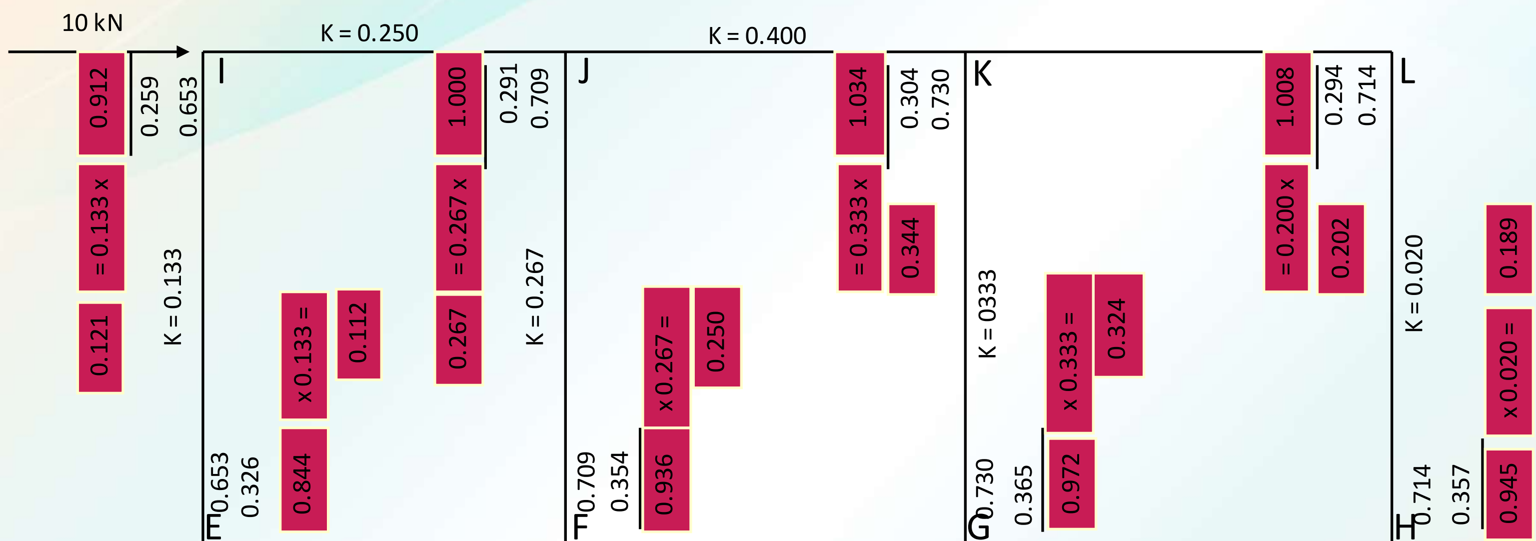
Step 4.



FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Step 5. Computing for Moment End of the 2nd floor :



$$10(15) = д(0.121 + 0.112 + 0.267 + 0.250 + 0.344 + 0.324 + 0.202 + 0.189)$$

$$Д = 82.92$$

$M_{IE} = 82.92(0.121) = 10.03$ KN-m

$M_{EI} = 82.92(0.112) = 9.29$ KN-m

$M_{JF} = 82.92(0.267) = 22.14$ KN-m

$M_{FJ} = 82.92(0.250) = 20.73$ KN-m

$M_{KG} = 82.92(0.344) = 28.52$ KN-m

$M_{GK} = 82.92(0.324) = 26.86$ KN-m

$M_{LH} = 82.92(0.202) = 16.75$ KN-m

$M_{HL} = 82.92(0.189) = 15.67$KN-m

FEU ALABANG FEU DILIMAN FEU TECH

Innovation

Computing for Moment End of the 1st floor :

$$(10+10)(20) = Д(0.126+0.102+0.258+0.216+0.326+0.277+0.194+0.163)$$

$$Д = 240.674$$

$M_{AE} = 240.674(0.126) = 30.325$ KN-m

$M_{EA} = 240.674(0.102) = 24.53$ KN-m

$M_{BF} = 240.674(0.258) = 62.094$ KN-m

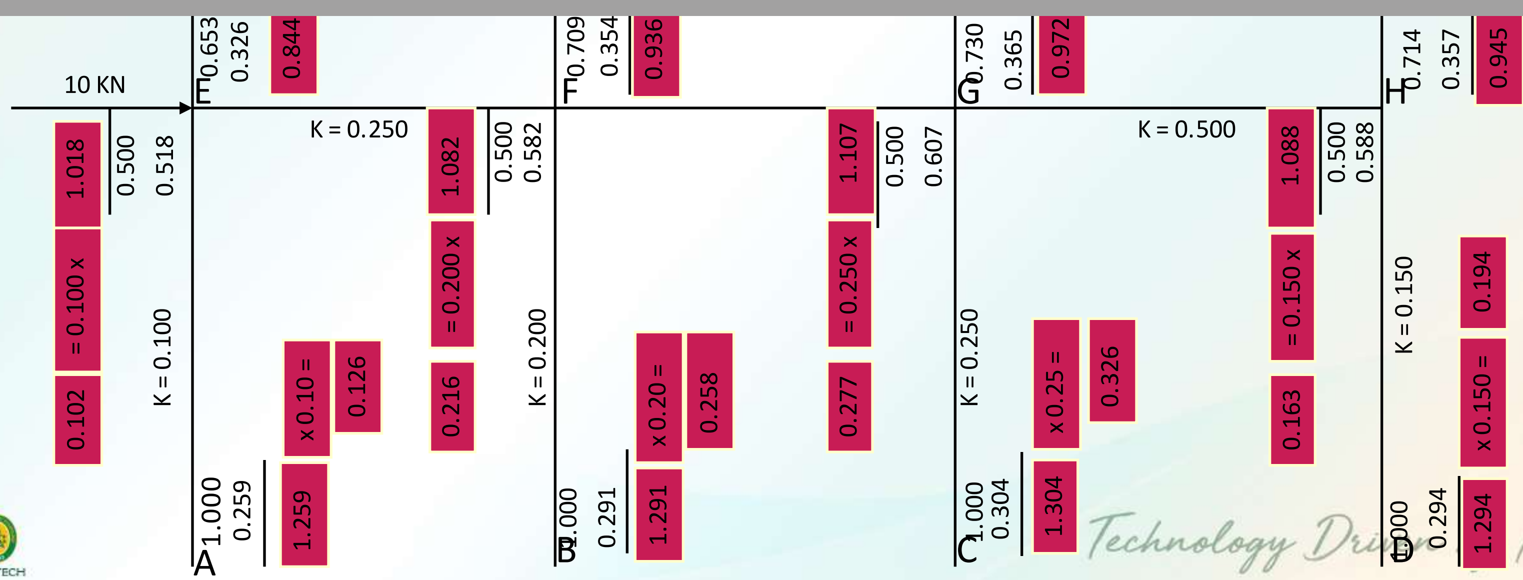
$M_{FB} = 240.674(0.216) = 51.986$ KN-m

$M_{CG} = 240.674(0.326) = 78.46$ KN-m

$M_{GC} = 240.674(0.277) = 66.67$ KN-m

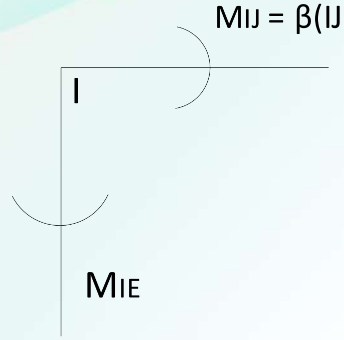
$M_{DH} = 240.674(0.194) = 46.69$ KN-m

$M_{HD} = 240.674(0.163) = 39.23$ KN-m

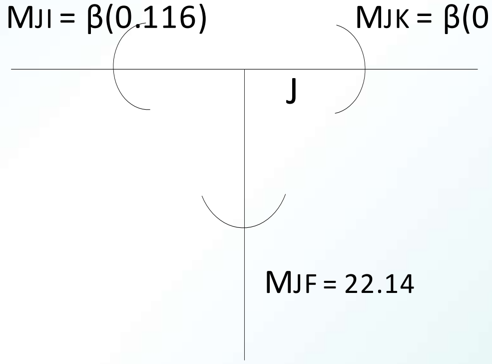


Ste 6. Computing Girder Moment

@ joint I



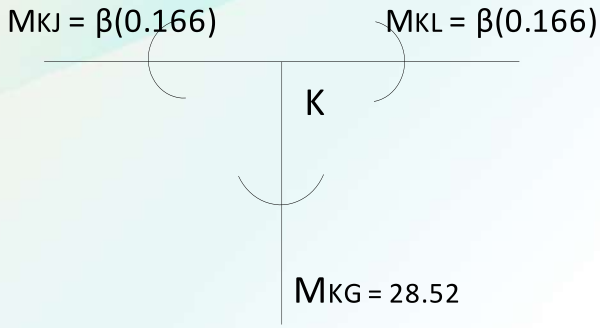
@ joint J



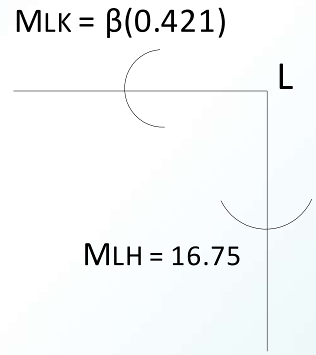
$$M_{IE} = M_{IJ}$$
$$10.03 = \beta(IJ)$$
$$10.03 = \beta(0.123)$$
$$\beta = 81.54$$
$$M_{IJ} = 81.54\,(0.123)$$
$$M_{IJ} = 10.03 \text{ KN-m}$$

$$M_{JF} = M_{JI} + M_{JK}$$
$$22.14 = \beta(JI + JK)$$
$$22.14 = \beta(0.116 + 0.170)$$
$$\beta = 77.41$$
$$M_{JI} = 77.41(0.116) \quad = 8.98 \text{ KN-m}$$
$$M_{JK} = 77.41\,(0.170) = 13.16 \text{ KN-m}$$

@ joint K



@ joint L



$$M_{KG} = M_{KJ} + M_{KL}$$

$$22.14 = \beta(JI + JK)$$

$$22.14 = \beta(0.116 + 0.207)$$

$$\beta = 76.46$$

$$M_{KJ} = 76.46(0.116) \quad = 12.69 \text{ KN-m}$$

$$M_{JK} = 76.46(0.207) 15.83 \text{ KN-m}$$

$$M_{LH} = M_{LK}$$

$$16.75 = \beta(LK)$$

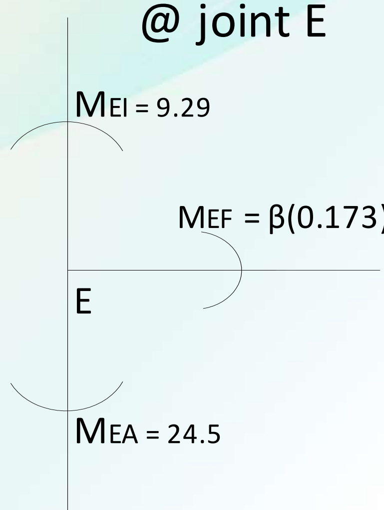
$$16.75 = \beta(0.421)$$

$$\beta = 39.79$$

$$M_{LK} = 39.79\,(0.421)$$

$$M_{LH} = 16.75 \text{ KN-m}$$

@ joint E



$M_{EI} + M_{EA} = M_{EF}$

$9.29 + 24.55 = \beta(EF)$

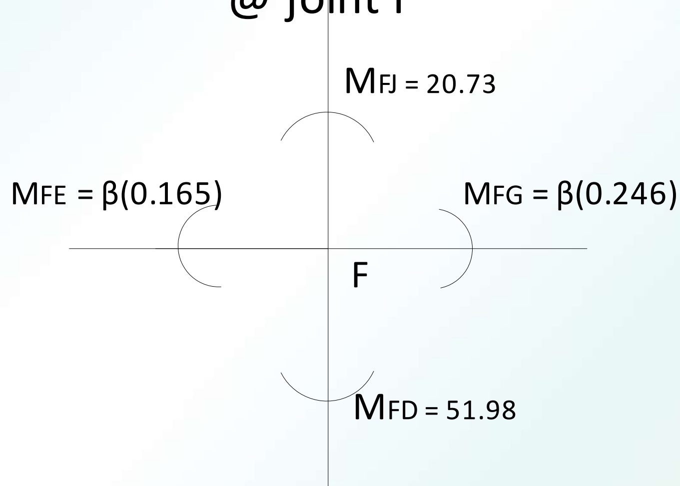
$9.29 + 24.55 = \beta(0.173)$

$\beta = 195.61$

$M_{EF} = 191.61(0.173)$

$M_{EF} = 33.84$ KN-m

@ joint F



$M_{FJ} + M_{FD} = M_{FE} + M_{FG}$

$20.73 + 51.98 = \beta(FE + FG)$

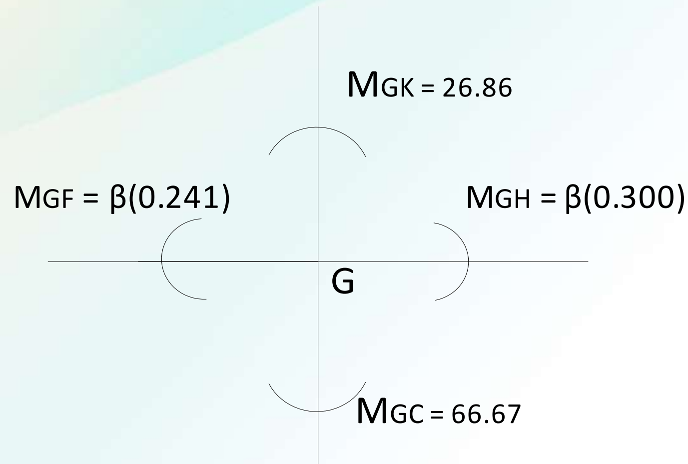
$20.73 + 51.98 = \beta(0.165 + 0.246)$

$\beta = 176.91$

$M_{KJ} = 176.91(0.165) = 43.52$ KN-m

$M_{JK} = 176.91(0.246) = 29.19$ KN-m

@ joint G



$$M_{GK} + M_{GC} = M_{GF} + M_{GH}$$

$$26.86 + 66.67 = \beta(GF + GH)$$

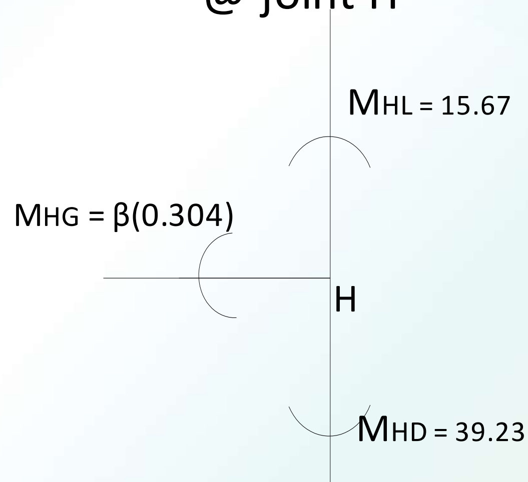
$$26.86 + 66.67 = \beta(0.241 + 0.300)$$

$$\beta = 172.88$$

$$M_{GF} = 172.88(0.241) \quad = 41.66 \text{ KN-m}$$

$$M_{GH} = 172.88(0.300) = 51.86 \text{ KN-m}$$

@ joint H



$$M_{HL} + M_{HD} = M_{HG}$$

$$15.67 + 39.23 = \beta(HG)$$

$$15.67 + 39.23 = \beta(0.304)$$

$$\beta = 180.59$$

$$M_{HG} = 180.59(0.304)$$

$$M_{HG} = 54.9 \text{ KN-m}$$

Q&A SESSION

ASK ANY QUESTION RELATED TO OUR TOPIC FOR TODAY.

HOMEWORK

Do the Exercises about Base Shear Computations

Reference:

Hibbeler, R. C., & Kiang, T. (2015). *Structural analysis*. Upper Saddle River: Pearson Prentice Hall.

CEELECT1
Earthquake Engineering
Elementary Structural Dynamics

Module 6

OBJECTIVES

- At the end of the chapter, the learner should be able to:
  - *Explain the basics of elementary structural dynamics.*
  - *Analyze using Newton's 2nd Laws of Motion*
  - *Interpret the results of computer simulations.*

ELEMENTARY STRUCTURAL DYNAMICS

Introduction to Structural Dynamics

Structural Dynamics

- ✓ Conventional structural analysis is based on the concept of statics, which can be derived from Newton's 1st Law of Motion.
- ✓ This law states that it is necessary for some force to act in order to initiate motion of a body at rest or to change the velocity of a moving body.
- ✓ Conventional structural analysis considers the external forces or joint displacements to be static and resisted only by the stiffness of the structure.

Structural Dynamics

- ✓Therefore, the resulting displacements and forces resulting from structural analysis do not vary with time.
- ✓Structural Dynamics is an extension of the conventional static structural analysis. It is the study of structural analysis that considers the external loads or displacements to vary with time and the structure to respond to them by its stiffness as well as inertia and damping
- ✓Newton's 2nd law of motion forms the basic principle of Structural Dynamics.

Structural Dynamics

- ✓The 2nd law of motion states that the resultant force on a body is equal to its mass times the acceleration induced.

- ✓Therefore, just as the 1st law of motion is a special case of the 2nd law, static structural analysis is also a special case of Structural Dynamics.

- ✓Although much less used by practicing engineers than conventional structural analysis, the use of Structural Dynamics has gradually increased with worldwide acceptance of its importance.

Structural Dynamics

✓At present it is being used for the analysis of tall buildings, bridges, towers due to wind, earthquake and for marine/offshore structures subjected wave, current, wind forces, vortex, etc.

Dynamic Force

- ✓The time-varying loads are called dynamic loads.
- ✓Structural dead loads and live loads have the same magnitude and direction throughout their application and are thus static loads.
- ✓However there are several examples of forces that vary with time, such as those caused by wind, vortex, water wave, vehicle, impact, blast or ground motion like earthquake.

Dynamic System

- ✓A dynamic system is a simple representation of physical systems and is modeled by mass, damping and stiffness.
- ✓Stiffness is the resistance it provides to deformations, mass is the matter it contains and damping represents its ability to decrease its own motion with time.
- ✓Mass is a fundamental property of matter and is present in all physical systems.

Dynamic System

- ✓This is simply the weight of the structure divided by the acceleration due to gravity.

- ✓Mass contributes an inertia force (equal to mass times acceleration) in the dynamic equation of motion.

- ✓Stiffness makes the structure more rigid, lessens the dynamic effects and makes it more dependent on static forces and displacements.

Dynamic System

- ✓This is simply the weight of the structure divided by the acceleration due to gravity.

- ✓Mass contributes an inertia force (equal to mass times acceleration) in the dynamic equation of motion.

- ✓Stiffness makes the structure more rigid, lessens the dynamic effects and makes it more dependent on static forces and displacements.

Damping

- ✓It is is often the least known of all the elements of a structural system.
- ✓Whereas the mass and the stiffness are well-known properties and measured easily, damping is usually determined from experimental results or values assumed from experience.
- ✓There are several sources of damping in a dynamic system.

Damping

- ✓Viscous damping is the most used damping system and provides a force directly proportional to the structural velocity.
- ✓This is a fair representation of structural damping in many cases and for the purpose of analysis, it is convenient to assume viscous damping (also known as linear viscous damping).
- ✓Viscous damping is usually an intrinsic property of the material and originates from internal resistance to motion between different layers within the material itself.

Damping

- ✓However, damping can also be due to friction between different materials or different parts of the structure (called frictional damping), drag between fluids or structures flowing past each other, etc.

- ✓Sometimes, external forces themselves can contribute to (increase or decrease) the damping.

- ✓Damping is also increased in structures artificially by external sources.

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Formulation of the Single-Degree-of-Freedom (SDOF) Equation

- ✓A dynamic system resists external forces by a combination of forces due to its stiffness (spring force), damping (viscous force) and mass (inertia force).
- ✓For the system shown in Fig. 1, k is the stiffness, c the viscous damping, m the mass and u(t) is the dynamic displacement due to the time-varying excitation force f(t).

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Formulation of the Single-Degree-of-Freedom (SDOF) Equation

✓ Such systems are called *Single-Degree-of-Freedom (SDOF) systems* because they have only one dynamic displacement [u(t) here].

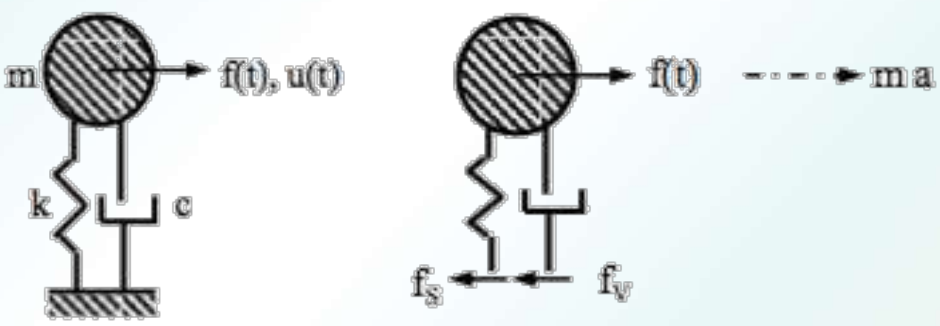


Fig. 1. *Dynamic SDOF system subjected to dynamic force f(t)*

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Formulation of the Single-Degree-of-Freedom (SDOF) Equation

✓ Considering the free body diagram of the system,

$$f(t) - f_S - f_V = ma \qquad Eq.1$$

Where:

$$f_S = Spring\ \ Force = (Stiffness)(Displacement) = k(u) \qquad Eq.2$$

$$f_V = Viscous\ \ Force = (ViscousDamping)(Velocity) = c(du/dt) \qquad Eq.3$$

$$f_t = Inertia\ \ Force = (Mass)(Acceleration) = m\left(d^2u/dt^2\right) \qquad Eq.4$$

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Formulation of the Single-Degree-of-Freedom (SDOF) Equation

- ✓ Combining equations 2 to 4 with Eq. 1, the equation of motion for a SDOF system is derived as

$$m\left(\frac{d^2u}{dt^2}\right)+c\left(\frac{du}{dt}\right)+ku=f(t) \qquad Eq.5$$

- ✓ This is a 2nd order ordinary differential equation (ODE), which needs to be solved in order to obtain the dynamic displacement u(t). As will be shown subsequently, this can be done analytically or numerically.

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Formulation of the Single-Degree-of-Freedom (SDOF) Equation

- ✓Eq. 5 has several limitations; e.g., it is assumed on linear input-output relationship [constant spring (k) and dashpot (c)].
- ✓It is only a special case of the more general equation 1, which is an equilibrium equation and is valid for linear or nonlinear systems.
- ✓Despite these, Eq. 5 has wide applications in Structural Dynamics. Several important derivations and conclusions in this field have been based on it.

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Free Vibration of Undamped Systems

- ✓Free Vibration is the dynamic motion of a system without the application of external force; i.e., due to initial excitement causing displacement and velocity.
- ✓The equation of motion of a general dynamic system with m, c and k is,

$$m\left(\frac{d^2u}{dt^2}\right)+c\left(\frac{du}{dt}\right)+ku=f(t) \qquad Eq.5$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Free Vibration of Undamped Systems

✓For undamped free vibration,

$$c = 0 \Rightarrow m\left(\frac{d^2u}{dt^2}\right) + ku = 0 \Rightarrow \left(\frac{d^2u}{dt^2}\right) + \omega_n{}^2 u = 0 \qquad Eq.6$$

Where:

$$\omega_n = \sqrt{\left(\frac{k}{m}\right)} \qquad Eq.7$$

is called the natural frequency of the system

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Free Vibration of Undamped Systems

✓ Assume u = e^{st}

$$\left(\frac{d^2u}{dt^2}\right) = s^2 e^{st}$$

From Eq. 6

$$s^2 e^{st} + \omega_n{}^2 e^{st} = 0 \Rightarrow s = \pm i\omega_n$$

Free Vibration of Undamped Single-Degree-of-Freedom (SDOF) System

Free Vibration of Undamped Systems

$$\Rightarrow u(t) = Ae^{i\omega_n t} + Be^{-i\omega_n t} = C_1 \cos(\omega_n t) + C_2 \sin(\omega_n t) \qquad Eq.8$$

$$\therefore v(t) = \frac{du}{dt} = -C_1\omega_n \sin(\omega_n t) + C_2\omega_n \cos(\omega_n t) \qquad Eq.9$$

If $u(0) = u_0$ and $v(0) = v_0$ then $C_1 = u_0$ and $C_2\omega_n = v_0 \Longrightarrow C_2 = v_0/\omega_n$ $\qquad Eq.10$

$$\therefore u(t) = u_0 \cos(\omega_n t) + \left(\frac{v_0}{\omega_n}\right)\sin(\omega_n t) \qquad Eq.11$$

ELEMENTARY STRUCTURAL DYNAMICS

Natural Frequency and Natural Period of Vibration

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Natural Frequency and Natural Period of Vibration

- ✓Equation 11 implies that the system vibrates indefinitely with the same amplitude at a frequency of ω_n radion/sec. Here, ω_n is the angular rotation (radians) traversed by a dynamic system in unit time (one second).
- ✓It is called the *natural frequency of the system (in radians/sec).*
- ✓Alternatively, the number of cycles completed by a dynamic system in one second is also called its *natural frequency (in cycles/sec or Hertz).*

Natural Frequency and Natural Period of Vibration

- ✓It is often denoted by f_n

$$f_n = \omega_n / 2\pi \qquad Eq.12$$

- ✓The time taken by a dynamic system to complete one cycle of revolution is called its natural period (T_n). It is the inverse of natural frequency.

$$\therefore T_n = 1/f_n = 2\pi/\omega_n \qquad Eq.13$$

ELEMENTARY STRUCTURAL DYNAMICS

Sample Problems in Natural Frequency and Natural Period of Vibration

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Example 1

An undamped structural system with stiffness (k) = 25 k/ft and mass (m) = 1 k-sec^2/ft is subjected to an initial displacement (u0)=1ft and an initial velocity (v_0) = 4 ft/sec.

i. Calculate the natural frequency and natural period of the system.

ii. Plot the free vibration of the system vs. time

Example 1 - Solution

i. For the system, natural frequency

$$\omega_n = \sqrt{\left(\frac{k}{m}\right)} = \sqrt{\left(\frac{25}{1}\right)} \qquad \omega_n = 5\,\frac{rad}{\sec}$$

$$\therefore f_n = \frac{\omega_n}{2\pi} = \frac{5}{2\pi} \qquad \therefore f_n = 0.796\,\frac{cycle}{\sec}$$

$$\therefore T_n = \frac{1}{f_n} = 1.257\sec$$

Example 1 - Solution

ii. The free vibration of the system is given by Eq. 11 as

$$u(t) = u_0 \cos(\omega_n t) + (v_0/\omega_n)\sin(\omega_n t)$$

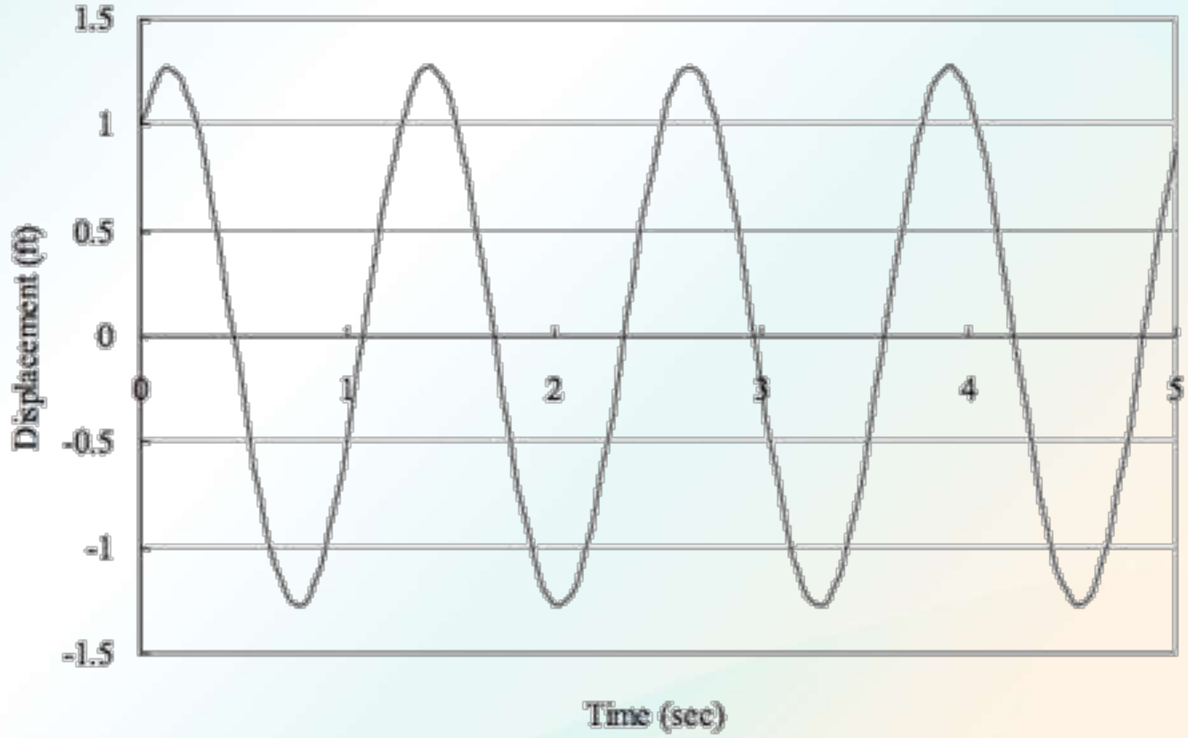
$$u(t) = (1)\cos(5t) + (4/5)\sin(5t)$$

The maximum value of u(t) = 1.281 ft

Example 1 - Solution

ii. The free vibration of the system is given by Eq. 11 as

The plot of u(t) vs time is shown below:



ELEMENTARY STRUCTURAL DYNAMICS

Free Vibration of Damped Systems

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Free Vibration of Damped Systems

The equation of motion of a dynamic system with mass (m), linear viscous damping (c) & stiffness (k) undergoing free vibration is,

$$m\left(\frac{d^2u}{dt^2}\right)+c\left(\frac{du}{dt}\right)+ku=0 \qquad Eq.5$$

Dividing both sides by m:

$$\left(\frac{d^2u}{dt^2}\right)+\left(\frac{c}{m}\right)\left(\frac{du}{dt}\right)+\left(\frac{k}{m}\right)u=0$$

$$\left(\frac{d^2u}{dt^2}\right)+2\xi\omega_n\left(\frac{du}{dt}\right)+\omega_n{}^2u=0 \qquad Eq.14$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Free Vibration of Damped Systems

Where:

$$\omega_n = \sqrt{k/m}$$ Natural Frequency of the System

and

$$\xi = c/2m\omega_n$$

$$\xi = c\omega_n/2k$$ Damping Ratio of the System

$$\xi = \frac{c}{2\sqrt{km}} \qquad Eq.15$$

Free Vibration of Damped Systems

Assume u = e^{st}

$$d^2u/dt^2 = s^2 e^{st}$$

$$s^2 e^{st} + 2\xi\omega_n s e^{st} + \omega_n{}^2 e^{st} = 0$$

$$s = \omega_n\left(-\xi \pm \sqrt{\xi^2 - 1}\right) \qquad Eq.16$$

Free Vibration of Damped Systems

1. If $\xi > 1$, the system is called an overdamped system. Here, the solution for s is a pair of different real numbers

$$\left[\omega_n\left(-\xi+\sqrt{\xi^2-1}\right),\ \omega_n\left(-\xi-\sqrt{\xi^2-1}\right)\right]$$

Such systems, however, are not very common. The displacement u(t) for such a system is

$$u(t)=e^{-\xi\omega_n t}\left(Ae^{\omega_1 t}+Be^{-\omega_1 t}\right) \qquad Eq.16$$

Where:

$$\omega_1=\omega_n\sqrt{\left(\xi^2-1\right)}$$

Free Vibration of Damped Systems

2. If ξ = 1, the system is called an critically damped system. Here, the solution for s is a pair of identical real numbers

$$\left[-\omega_n, -\omega_n\right]$$

Critically damped systems are rare and mainly of academic interest only. The displacement u(t) for such a system is

$$u(t) = e^{-\omega_n t}(A + Bt) \qquad Eq.17$$

Where:

$$\omega_1 = \omega_n\sqrt{\left(\xi^2 - 1\right)}$$

Free Vibration of Damped Systems

3. If $\xi < 1$, the system is called an underdamped system. Here, the solution for s is a pair of different complex numbers

$$\left[\omega_n\left(-\xi + i\sqrt{1-\xi^2}\right),\ \omega_n\left(-\xi - i\sqrt{1-\xi^2}\right)\right]$$

Practically, most structural systems are underdamped. The displacement u(t) for such as system is

$$u(t) = e^{-\xi\omega_n t}\left(Ae^{i\omega_d t} + Be^{-i\omega_d t}\right)$$

$$u(t) = e^{-\xi\omega_n t}\left[C_1\cos(\omega_d t) + C_2\sin(\omega_d t)\right] \qquad Eq.18$$

Where:

$\omega_d = \omega_n\sqrt{1-\xi^2}$ called the *damped natural frequency* of the system

Free Vibration of Damped Systems

Since underdamped systems are the most common of all structural systems, the subsequent discussion will focus mainly on those. Differentiating Eq. 18, the velocity of an underdamped system is obtained as:

$$v(t) = du/dt$$

$$v(t) = e^{-\xi\omega_n t}\left\{\omega_d\left[-C_1\sin(\omega_d t) + C_2\cos(\omega_d t)\right] - \xi\omega_n\left[C_1\cos(\omega_d t) + C_2\sin(\omega_d t)\right]\right\} \qquad Eq.19$$

If $u(0) = u_0$ and $v(0) = v_0$, then $C_1 = u_0$ and $\omega_d C_2 - \xi\omega_n C_1 = v_0$

$$\Rightarrow C_2 = \frac{(v_0 + \xi\omega_n u_0)}{\omega_d} \qquad \therefore u(t) = e^{-\xi\omega_n t}\left[u_0\cos(\omega_d t) + \frac{v_0 + \xi\omega_n u_0}{\omega_d}\sin(\omega_d t)\right]$$

Free Vibration of Damped Systems

Equation 19, the system vibrates at its damped natural frequency (i.e., a frequency of ω_d radian/sec). Since the damped natural frequency $\omega_d = \left[\omega_n\sqrt{1-\xi^2}\right]$ is less than $\omega_{n,}$ the system vibrates more slowly than the undamped system.

Moreover, due to the exponential term $e^{-\xi\omega_n t}$, the amplitude of the motion of an underdamped system decreases steadily and reaches zero after (a hypothetical) “infinite” time of vibration.

Similar equations can be derived for critically damped and overdamped dynamic systems in terms of their initial displacement, velocity and damping ratio.

Example 2

A damped structural system with stiffness (k) = 25 k/ft and mass (m) = 1 k-sec^2/ft is subjected to an initial displacement (u_0) = 1 ft and an initial velocity (v_0) = 4 ft/sec. Plot the free vibration of the system vs. time if the Damping Ratio is

a. 0.00 (undamped system)

b. 0.05

c. 0.50 (underdamped system)

d. 1.00 (critically damped system)

e. 1.50 (overdamped system)

Example 2 - Solution

The equations for u(t) are plotted against time for various damping ratios (DR) and shown below. The main features of these figures are

1. The underdamped systems have sinusoidal variations of displacement with time. Their natural periods are lengthened (more apparent for $\xi = 0.50$) and maximum amplitudes of vibration reduced due to damping.
2. The critically damped and overdamped systems have monotonic rather than harmonic (sinusoidal) variations of displacement with time. Their maximum amplitudes of vibration are less than the amplitudes of underdamped systems.

Example 2 - Solution

The equations for u(t) are plotted against time for various damping ratios (DR) and shown below. The main features of these figures are

a. ξ = 0.00 (undamped system) @ t=0

$$\omega_n = \sqrt{\frac{25}{1}} = 5\,\frac{rad}{\sec} \qquad \omega_d = \omega_n\sqrt{1-\xi^2} = 5\sqrt{1-0^2} = 5\,\frac{rad}{\sec}$$

$$u(t) = e^{-0(5)(0)}\left[1\cos(5x0) + \frac{4+0(5)(1)}{5}\sin(5x0)\right]$$

$$u(t) = 0$$

Technology Driven by Innovation

FEU ALABANG FEU DILIMAN FEU TECH

Example 2 - Solution

The equations for u(t) are plotted against time for various damping ratios (DR) and shown below. The main features of these figures are

a. ξ = 0.00 (undamped system) @ t=1

$$\omega_n = \sqrt{\frac{25}{1}} = 5\,\frac{rad}{\sec} \qquad \omega_d = \omega_n\sqrt{1-\xi^2} = 5\sqrt{1-0^2} = 5\,\frac{rad}{\sec}$$

$$u(t) = e^{-0(5)(1)}\left[1\cos(5x1) + \frac{4+0(5)(1)}{5}\sin(5x1)\right]$$

$$u(t) = -0.483\,ft$$

Example 2 - Solution

The equations for u(t) are plotted against time for various damping ratios (DR) and shown below. The main features of these figures are

Redo the succeeding Damping Ratios and Time
using the same equation utilized in the previous slide.

Example 2 - Solution

The equations for u(t) are plotted against time for various damping ratios (DR) and shown below. The main features of these figures are

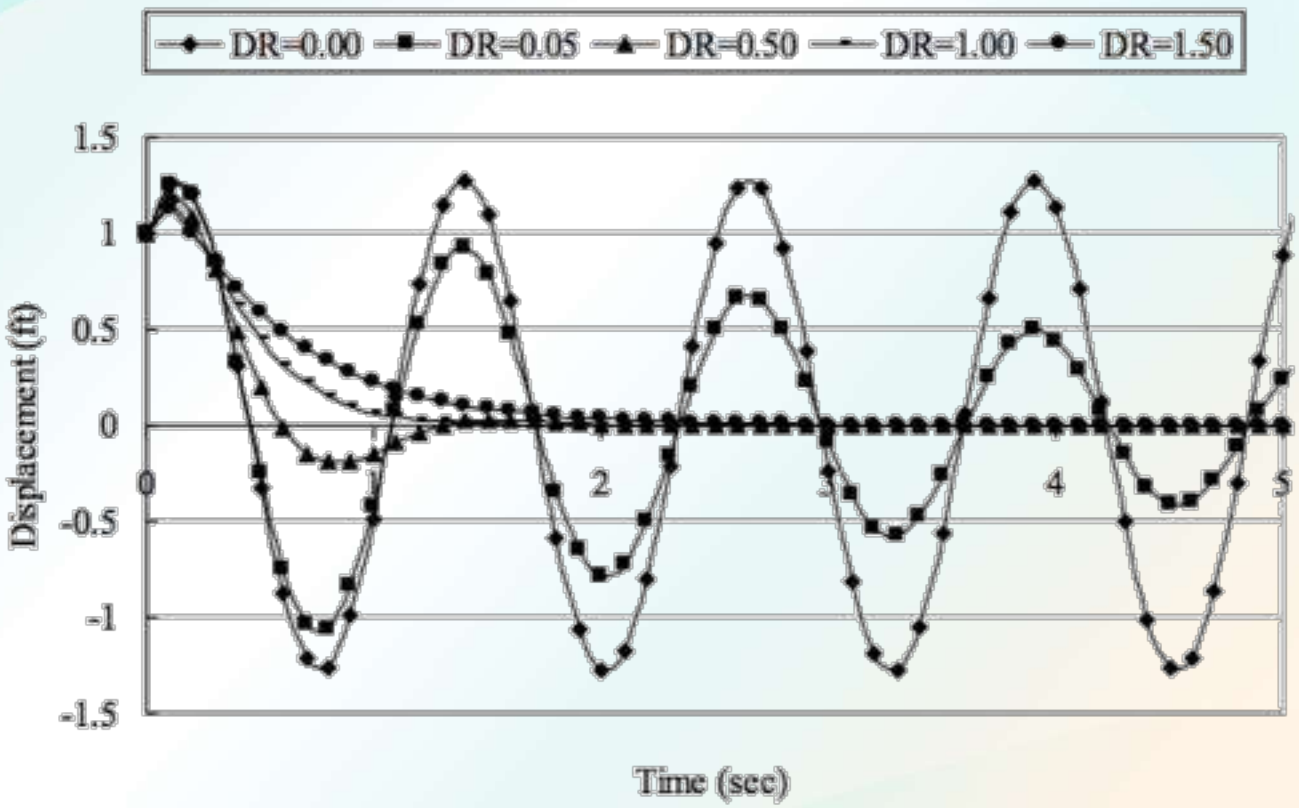


Fig. 3.1: Displacement vs. Time for free Vibration of Damped Systems

Damping of Structures

- ✓Damping is the element that causes impedence of motion in a structural system.

- ✓There are several sources of damping in a dynamic system.

- ✓Damping can be due to internal resistance to motion between layers, friction between different materials or different parts of the structure (called frictional damping), drag between fluids or structures flowing past each other, etc.

Damping of Structures

✓Damping is also increased in structures artificially by external sources like dampers acting as control systems.

Viscous Damping of SDOF systems

- ✓Viscous damping is the most used damping and provides a force directly proportional to the structural velocity.

- ✓This is a fair representation of structural damping in many cases and for the purpose of analysis it is convenient to assume viscous damping (also known as linear viscous damping).

- ✓Viscous damping is usually an intrinsic property of the material and originates from internal resistance to motion between different layers within the material itself.

Viscous Damping of SDOF systems

- ✓ While discussing different types of viscous damping, it was mentioned that underdamped systems are the most common of all structural systems.

- ✓ This discussion focuses mainly on underdamped SDOF systems, for which the free vibration response was found to be

$$\therefore u(t) = e^{-\xi\omega_n t}\left[u_0 \cos(\omega_d t) + \frac{v_0 + \xi\omega_n u_0}{\omega_d}\sin(\omega_d t)\right]$$

Viscous Damping of SDOF systems

- ✓The system vibrates at its damped natural frequency (i.e., a frequency of ω_d radian/sec).
- ✓Since $\omega_d = \left[\omega_n\sqrt{1-\xi^2}\right]$ is less than ω_n, the system vibrates more slowly than the undamped system.
- ✓Due to the exponential term $e^{-\xi\omega_n t}$ the amplitude of motion decreases steadily and reaches zero after (a hypothetical) "infinite" time of vibration.

Viscous Damping of SDOF systems

✓However, the displacement at N time periods ($T_d = 2\pi/\omega_d$) later than u(t) is:

$$u\left(t+NT_d\right)=e^{-\xi\omega_n\left(t+\frac{2\pi N}{\omega_d}\right)}\left[u_0\cos\left(\omega_d t+2\pi N\right)+\left\{\frac{\left(v_0+\xi\omega_n u_0\right)}{\omega_d}\right\}\sin\left(\omega_d t+2\pi N\right)\right]$$

$$u\left(t+NT_d\right)=e^{-\xi\omega_n\left(\frac{2\pi N}{\omega_d}\right)}u\left(t\right) \qquad Eq.19$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Viscous Damping of SDOF systems

✓From which, using

$$\omega_d = \omega_n \sqrt{1-\xi^2} \Rightarrow \xi \Big/ \sqrt{1-\xi^2} = \frac{\ln\left[\dfrac{u(t)}{u(t+NT_d)}\right]}{2\pi N} = \delta$$

$$\Rightarrow \xi = \frac{\delta}{\sqrt{1+\delta^2}} \qquad Eq.20$$

Viscous Damping of SDOF systems

✓ For lightly damped structures (i.e., ξ<< 1),

$$\xi \cong \delta = \frac{\ln\left[\dfrac{u(t)}{u(t+NT_d)}\right]}{2\pi N} \qquad Eq.21$$

Viscous Damping of SDOF systems

✓Table 1: Recommended Damping Ratios for different Structural Elements

Stress Level	Type and Condition of Structure	ξ (%)
Working Stress	Welded steel, pre-stressed concrete, RCC with slight cracking	2-3
	RCC with considerable cracking	3-5
	Bolted/Riveted Steel or Timber	5-7
Yield Stress	Welded steel, pre-stressed concrete	2-3
	RCC	7-10
	Bolted/Riveted Steel or Timber	10-15

Example 3

Determine the Damping Ratio (DR) if the free vibration amplitude of a SDOF system decays from 1.5” to 0.5” in 3 cycles.

Example 3 - Solution

Solve for the Damping Ratio:

$$\xi \cong \delta = \frac{\ln\left[\dfrac{u(t)}{u(t+NT_d)}\right]}{2\pi N}$$

$$\xi = \frac{\ln\left[\dfrac{1.5}{0.5}\right]}{2\pi(3)} x100$$

$$\xi = 5.829\%$$

Q&A SESSION

ASK ANY QUESTION RELATED TO OUR TOPIC FOR TODAY.

HOMEWORK

Do the Exercises about Base Shear Computations

Reference:

STRUCTURAL DYNAMICS, DYNAMIC FORCE AND DYNAMIC SYSTEM
http://www.uap-bd.edu/ce/anam/Anam_files/Structural%20Dynamics%20and%20Earthquake%20Engineering.pdf

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

CEELECT1

Earthquake Engineering

Design Response Spectra and Structural Detailing for Earthquake and Structural Systems

Module 7

OBJECTIVES

- At the end of the chapter, the learner should be able to:
  - *Explain the Design Response Spectra*
  - *Analyze and Compute the Design Response Spectra of Buildings using Tripartite Curve and NSCP Code*
  - *Modify Building Irregularities*

DESIGN RESPONSE SPECTRA AND STRUCTURAL DETAILING FOR EARTHQUAKE AND STRUCTURAL SYSTEMS

Dynamic Analysis Procedures

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

208.5.3 Dynamic Analysis Procedures

208.5.3.1. General

Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics.

Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

208.5.3 Dynamic Analysis Procedures

208.5.3.2. Ground Motion

The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. An elastic design response spectrum constructed in accordance with Figure 208-3, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 9.815 m/s^2.

208.5.3 Dynamic Analysis Procedures

208.5.3.2. Ground Motion

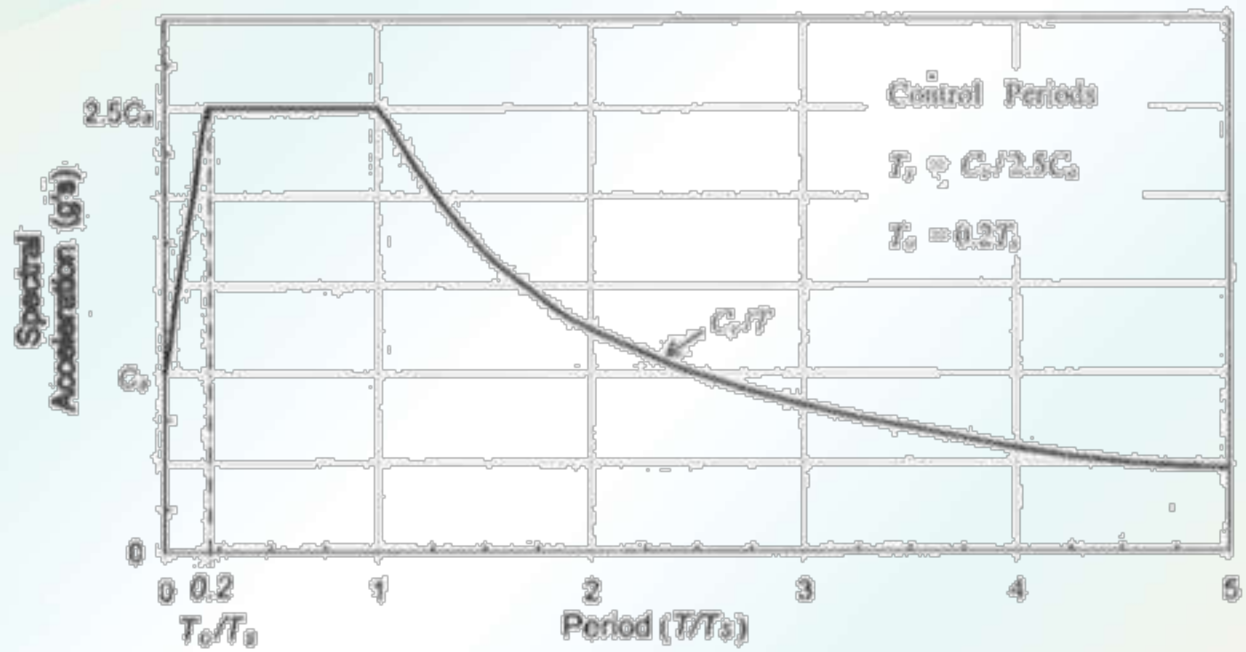


Figure 208-3
Design Response Spectra

208.5.3 Dynamic Analysis Procedures

208.5.3.2. Ground Motion

The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

2. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and snil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

208.5.3 Dynamic Analysis Procedures

208.5.3.2. Ground Motion

The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 208.5.3.2, Item 2.

208.5.3 Dynamic Analysis Procedures

208.5.3.2. Ground Motion

4. For structures on Soil Profile Type S_F, the following requirements shall apply when required by Section 208.4.8.3, Item 4:

4.1. The ground motion representation shall be developed in accordance with Items 2 and 3.

4.2. Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.

208.5.3 Dynamic Analysis Procedures

208.5.3.2. Ground Motion

The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

5. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site-specific data. Where the Near-Source Factor, N_a, is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

208.5.3 Dynamic Analysis Procedures

208.5.3.3. Mathematical Model

A mathematical model of the physical structure shall represent the spatial distribution of' the mass and stiffness of the structure to an extent that is adequate for the calculation o f the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of' structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 208-10 and having a rigid or semi-rigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accondance with Section 208.6.2.

208.5.3 Dynamic Analysis Procedures

208.5.3.4. Description of Analysis Procedures

208.5.3.4.1. Response Spectrum Analysis

An elastic dynamic analysis o f a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses arc calculated using the ordinates o f the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions arc combined in a statistical manner to obtain an approximate total structural response.

208.5.3 Dynamic Analysis Procedures

208.5.3.4. Description of Analysis Procedures

208.5.3.4.2. Time History Analysis

An analysis of' the dynamic response o f a structure at each increment of time when the base is subjected to a specific ground motion time history.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.1. Response Spectrum Representation and Interpretation of Results

The ground motion representation shc1ll be in accordance with Section 208.5.3.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 208.5.3.5.4.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.1. Response Spectrum Representation and Interpretation of Results

The base shear for a given direction, determined using dynamic analysis must not be less than the value obtained by the equivalent lateral force method of Section 208.5.2. In this case, all corresponding response parameters are adjusted proportionately.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.2. Number of Modes

The requirement of Section 208.5.3.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered. al least 90 percent or the participating mass of the structure is included in the calculation or response for each principal horizontal direction.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.3. Combining Modes

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

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.4. Reduction of Elastic Response Parameters for Design

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

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.4. Reduction of Elastic Response Parameters for Design

1. For all regular structures where the ground motion representation complies with Section 208.5.3.2, Item 1, Elastic response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with Section 208.5.2.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.4. Reduction of Elastic Response Parameters for Design

2. For all regular structures where the ground motion representation complies with Section 208.5.3.2, Item 2, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with Section 208.5.2.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.4. Reduction of Elastic Response Parameters for Design

3. For all irregular structures, regardless of the ground motion representation, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 208.5.2.

The corresponding reduced design seismic forces shall be used for design in accordance with Section 203.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.5. Directional Effects

Directional effects for horizontal ground motion shall conform to the requirements of Section 208.6. The effects of vertical ground motions on horizontal cantilevers and pre-stressed elements shall he considered in accordance with Section 208.6. Alternately, vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.6. Torsion

The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 208.5.1.4. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 208.5.1.3.

208.5.3 Dynamic Analysis Procedures

208.5.3.5. Response Spectrum Analysis

208.5.3.5.7. Dual Systems

Where the lateral forces are resisted by a dual system as defined in Section 208.4.6.4, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to Section 208.4.6.4, Item 2, and may be analyzed using either the procedures of Section 208.5.2.3 or those of Section 208.5.3.5.

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.1. Time History

Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time- history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake).

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.1. Time History

Where three appropriate recorded ground- motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used lo make up the total number required. For each pair of horizontal ground- motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components shall be constructed.

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.1. Time History

The motions shall be scaled such that the average value of the SRSS spectra docs not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from $0.2T$ second to $1.5T$ seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.2. Elastic Time History Analysis

Elastic time history shall conform to Sections 208.5.3.1 208.5.3.2, 208.5.3.3. 208.5.3.5.2, 208.5.3.5.4: 208.6.5.3.5.5, 208.6.5.3.5.6, 208.5.3.5.7 and 208.5.3.6.1 and 208.6.6.1. Response parameters from elastic time- history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 208.5.3.5.4.

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.3. Nonlinear Time History Analysis

208.5.3.6.3.1 Nonlinear Time History

Nonlinear time history analysis shall meet the requirements of Section 208.4.8.4, and time histories shall be developed and results determined in accordance with the requirements of Section 208.5.3.6.l. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with Section 208.6.5

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.3. Nonlinear Time History Analysis

208.5.3.6.3.2 Design Review

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

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.3. Nonlinear Time History Analysis

208.5.3.6.3.2 Design Review

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

208.5.3 Dynamic Analysis Procedures

208.5.3.6. Time History Analysis

208.5.3.6.3. Nonlinear Time History Analysis

208.5.3.6.3.2 Design Review

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

DESIGN RESPONSE SPECTRA AND STRUCTURAL DETAILING FOR EARTHQUAKE AND STRUCTURAL SYSTEMS

Vertical Structural Irregularities

Introduction to Vertical Irregularities

Vertical irregularities are identified in the NSCP 2015 Table 208-9. These can be divided into two categories. The first are dynamic force distribution irregularities. These are irregularities Types 1, 2 and 3. The second category is irregularities in load path or force transfer, and these are Types 4 and 5. The five vertical irregularities are as follows:

1. Stiffness irregularity – Soft Storey
2. Weight (Mass) Irregularity
3. Vertical Geometric Irregularity

Introduction to Vertical Irregularities

Vertical irregularities are identified in the NSCP 2015 Table 208-9. These can be divided into two categories. The first are dynamic force distribution irregularities. These are irregularities Types 1, 2 and 3. The second category is irregularities in load path or force transfer, and these are Types 4 and 5. The five vertical irregularities are as follows:

4. In-plane discontinuity in vertical lateral-force resisting element

5. Discontinuity in capacity-weak storey

Introduction to Vertical Irregularities

The first category, dynamic force distribution irregularities, requires that the distribution of lateral forces be determined by combined dynamic modes of vibration. For regular structures without abrupt changes in stiffness or mass (i.e., structures without "vertical structural irregularities"), this shape can be assumed to be linearly-varying or a triangular shape as represented by the code force distribution pattern. However, for irregular structures, the pattern can be significantly different and must be determined by the combined mode shapes from the dynamic analysis procedure of NSCP 2015 Section 208.6. The designer may opt to go directly to the dynamic analysis procedure and thereby bypass the checks for vertical irregularity Types 1,2 and 3.

Introduction to Vertical Irregularities

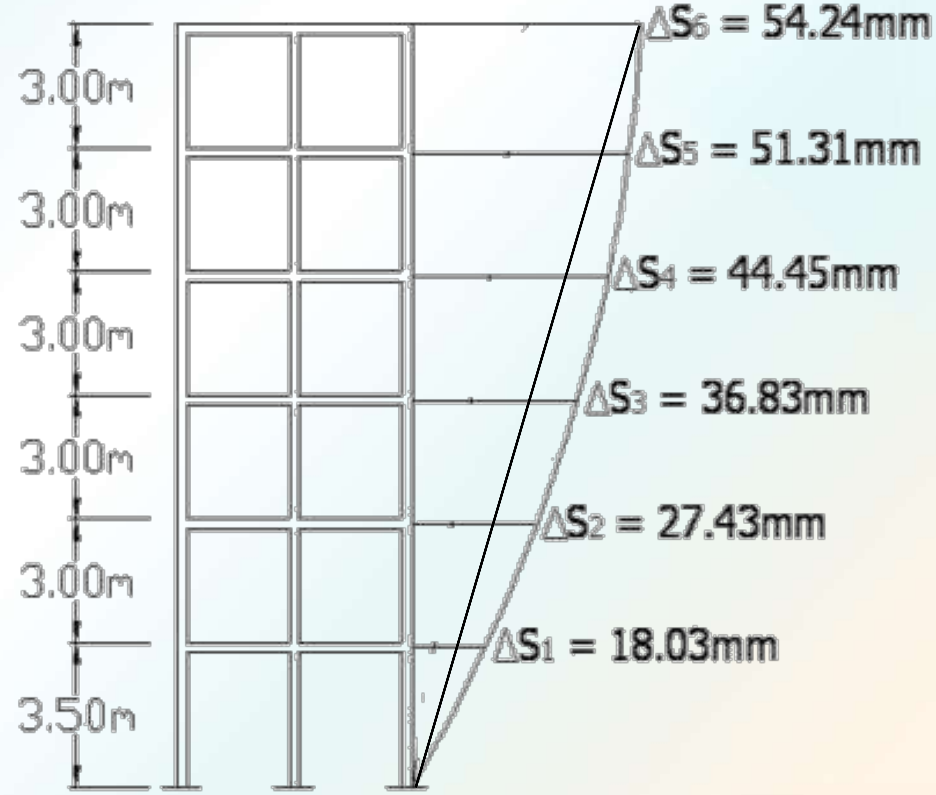
Regular structures are assumed to have a reasonably uniform distribution of inelastic behavior in elements throughout the lateral force resisting system. When vertical irregularity Types 4 and 5 exist, there is the possibility of having localized concentrations of excessive inelastic deformations due to the irregular load path or weak storey. In this case, the code prescribes additional strengthening to correct the deficiencies.

Vertical Irregularity Type 1 (Stiffness Irregularity – Soft Storey)

A soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80 percent of the average stiffness of the three storeys above.

Example 1

A six – storey concrete special moment – resisting frame is shown below. The specified lateral forces F_x from NSCP 2015 Equations 208-14 and 208-15 have been applied and the corresponding floor level displacement 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 storey) exists in each storey.

Example 1 - Discussion

To determine if this is a Type 1 vertical irregularity (stiffness irregularity-soft storey) here are two tests:

1. The storey stiffness is less than 70 percent of that of the storey above.

2. The storey stiffness is less than 80 percent of the average stiffness of the three storeys above.

If the stiffness of the storey meets at least one of the above criteria, the structure is considered to have a soft storey, and a dynamic analysis is generally required under Section 208.4.8.3 Item 2, unless the irregular structure is not more than five storeys or 20 meters in height.

Example 1 - Discussion

The definition of soft in the code compares values of the lateral stiffness of individual stories. Generally, it is not practical to use stiffness properties unless these can be easily determined. There are many structural configurations where the evaluation of storey stiffness is complex and is often not an available output from computer programs. Recognizing that the basic intent of this irregularity check is to determine if the lateral force distribution will differ significantly from the linear pattern prescribed by NSCP 2015 Equation 208-15, which assumes a triangular shape for the first dynamic mode of response, this type of irregularity can also be determined by comparing values of lateral storey displacements or drift ratios due to the prescribed lateral forces.

Example 1 - Discussion

This deformation comparison may even be more effective than the stiffness comparison because of the shape of the first mode shape is often closely approximated by the structure displacements due to the specified triangular load pattern. Floor level displacements and corresponding storey drift ratios are directly available from computer programs. To compare displacements rather than stiffness, it is necessary to use the reciprocal of the limiting percentage ratios of 70 and 80 percent as they apply to storey stiffness, or reverse their applicability to the storey or storeys above. The following examples shows this equivalent use of the displacement properties.

Example 1 - Discussion

From the given displacements, storey drifts and the storey drift ratio values are determined. The storey drift ratio is the storey drift divided by the storey height. These will be used for the required comparisons, since these better represent the changes in the slope of the mode shape when there are significant differences in inter-storey heights. (Note: storey displacements can be used if the storey heights are nearly equal.)

Example 1 - Discussion

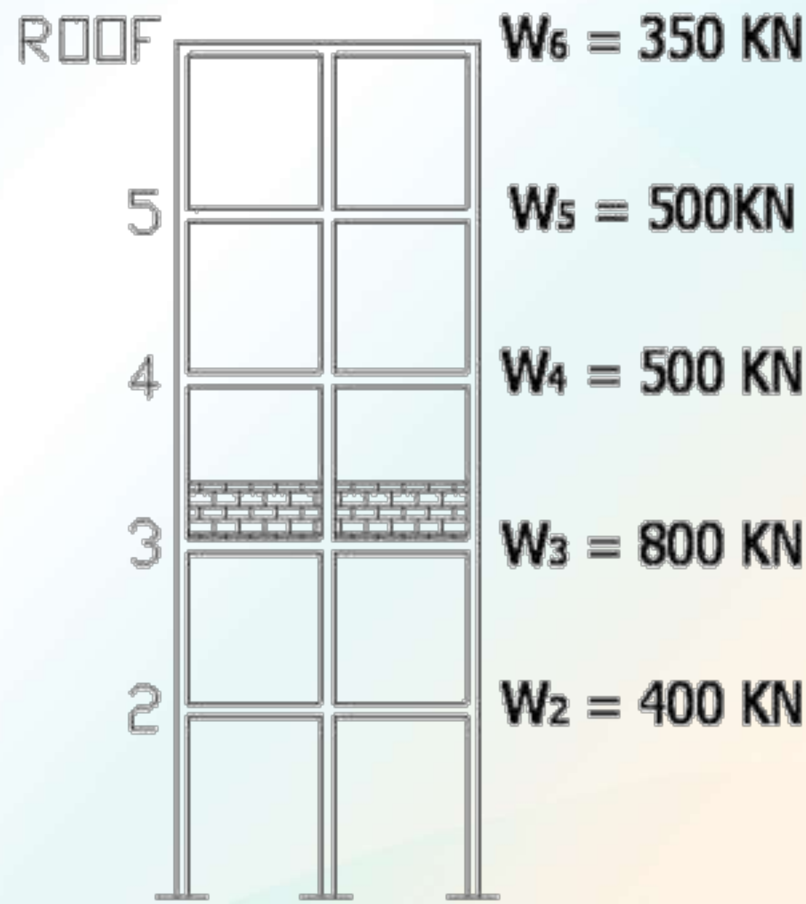
Level	Height (m)	Storey Displacement (mm)	Storey Drift (mm)	Storey Drift Ratio	0.7 SDR	0.8 SDR	Avg. SDR of the Next 3 Storeys	Soft Storey Status
6	3.0	57.24	5.93	0.00198	0.00139	0.00158	-	*No*
5	3.0	51.31	6.86	0.00229	0.00160	0.00183	-	*No*
4	3.0	44.45	7.62	0.00254	0.00178	0.00203	-	*No*
3	3.0	36.83	9.40	0.00313	0.00219	0.00250	0.00227	*Yes*
2	3.0	27.43	9.40	0.00313	0.00219	0.00250	0.00265	*No*
1	3.5	18.03	18.03	0.00515	0.00361	0.00412	0.00293	*Yes*

Vertical Irregularity Type 2 – Weight (Mass) Irregularity

Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150% of the effective mass of an adjacent storey. A roof that is lighter than the floor below need not be considered.

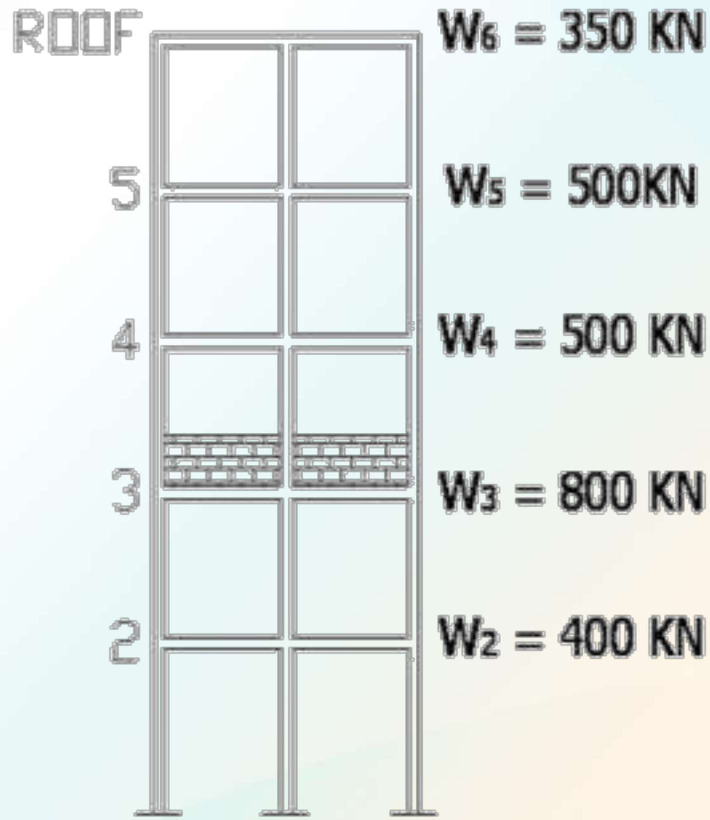
Example 2

The five-storey special moment frame office building has a heavy utility equipment installation at Level 3. This results in the floor weight distribution shown below. Determine if there is a type 2 vertical weight (mass) irregularity.



Example 2 – Calculation and Discussion

Under NSCP 2015 Table 208-9 Vertical Irregularities, Item 2, Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150% of the effective mass of an adjacent storey. A roof that is lighter than the floor below need not be considered.



Example 2 – Calculation and Discussion

Checking the effective mass of Level 3 against the effective mass of levels 2 and 4.

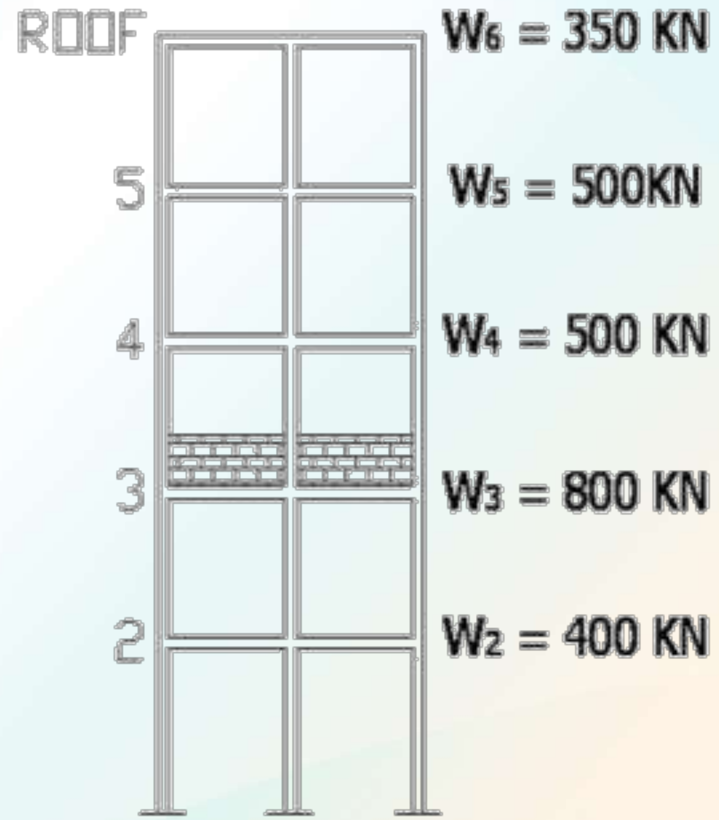
At Level 2

$$1.5xW_1 = 1.5(400)$$

$$1.5xW_1 = 600kN$$

$$W_2 = 800kN > 600kN$$

$$\therefore Weight\ \ Irregularity\ \ Exists!$$



Example 2 – Calculation and Discussion

Checking the effective mass of Level 3 against the effective mass of levels 2 and 4.

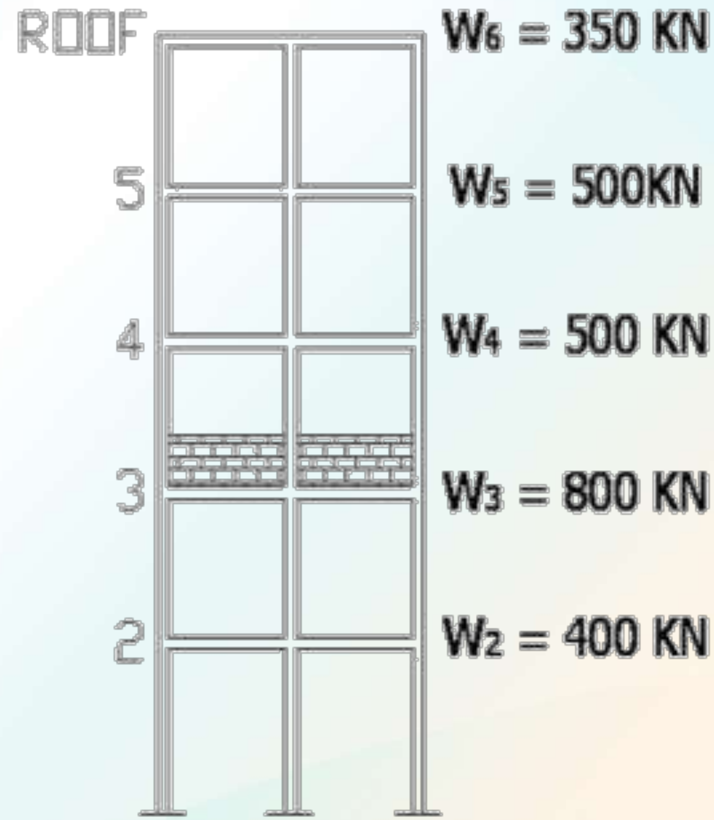
At Level 4

$$1.5xW_3 = 1.5(500)$$

$$1.5xW_3 = 750kN$$

$$W_2 = 800kN > 750kN$$

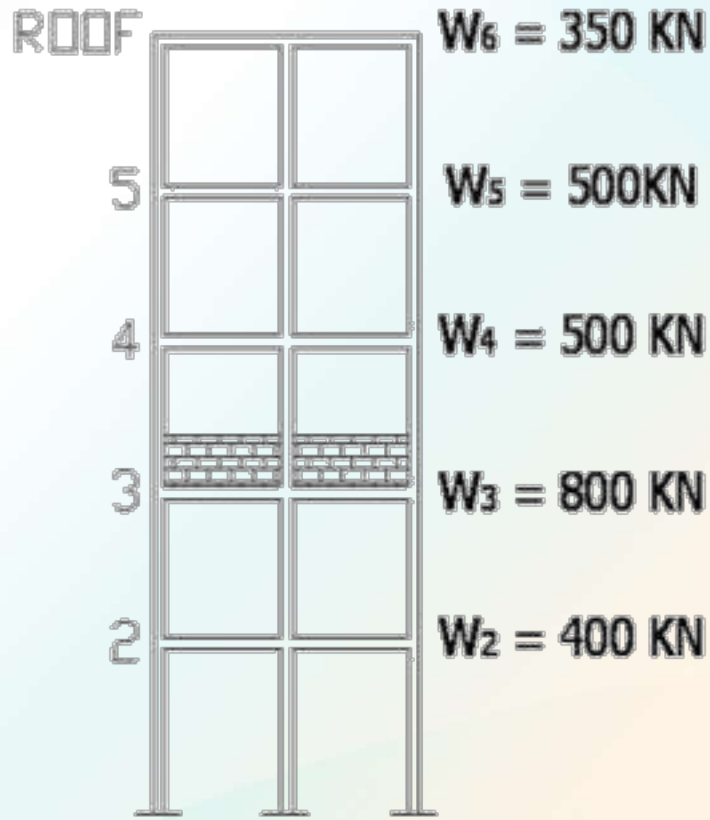
$$\therefore Weight\ \ Irregularity\ \ Exists!$$



Example 2 – Calculation and Discussion

Commentary:

As in the case of vertical irregularity type 1, this type of irregularity also results in a primary mode shape that can be substantially different from the triangular shape and lateral load distribution given by NSCP 2015 Equation 208-15. Consequently, the appropriate load distribution must be determined by the dynamic analysis procedure of Section 208.6, unless the irregular structure is less than five storeys or 20 meters in height.

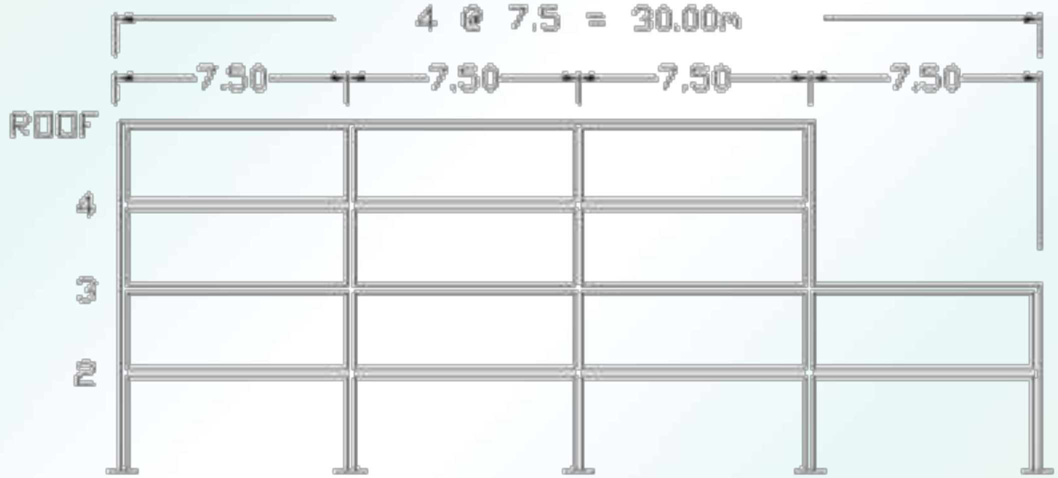


Vertical Irregularity Type 3 – Vertical Geometric Irregularity

Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any storey is more than 130% of that in an adjacent storey. One – storey penthouses need not be considered.

Example 3

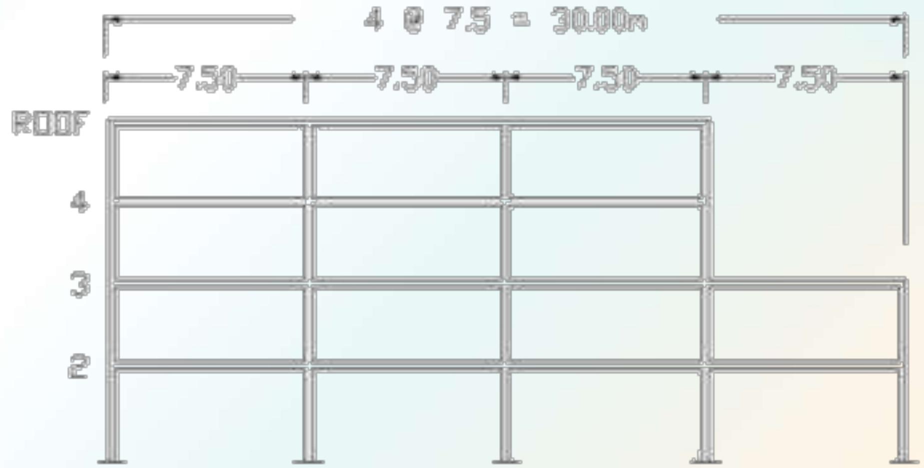
The lateral force-resisting system of the four-storey special moment frame building shown below has 7.5m setback at the third and fourth storeys.



Determine if a Type 3 vertical irregularity, vertical geometric irregularity, exists.

Example 3 – Calculation and Discussion

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



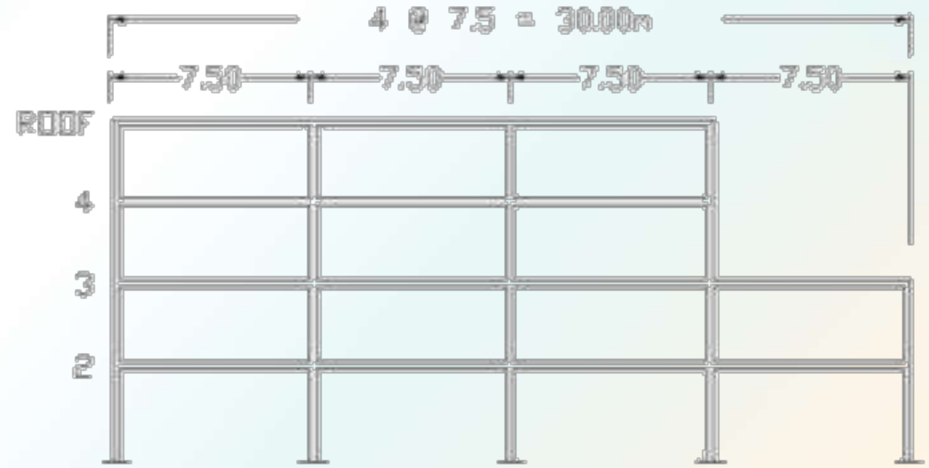
Example 3 – Calculation and Discussion

In this example, the setback of Level 4 must be checked. The ratios of the two levels is

$$\frac{Width\ of\ Level\ 3}{Width\ of\ Level\ 4} = \frac{30m}{22.5m}$$

$$\frac{Width\ of\ Level\ 3}{Width\ of\ Level\ 4} = 1.33$$

$$133\ percent > 130\ percent$$



$$\therefore Vertical\ geometric\ irregularity\ exists!$$

Example 3 – Calculation and Discussion

Commentary:

The more than 130 percent change in the width of the lateral force-resisting system between adjacent storeys could result in a primary mode shape that is substantially different from the triangular shape assumed for NSCP 2015 Equation 208-15. If the change is a decrease in width of the upper adjacent storey (the usual situation), the mode shape difference can be mitigated by designing for an increased stiffness in the storey with a reduced width.

Example 3 – Calculation and Discussion

Commentary:

Similarly, if the width decrease is in the lower adjacent storey (the unusual situation), the Type 1 soft storey irregularly can be avoided by a proportional increase in the stiffness of the lower storey. However, when the width decrease is in the lower storey, there could be an overturning moment load transfer discontinuity that would require the application of NSCP 2015 Section 208.5.8.1.

Example 3 – Calculation and Discussion

Commentary:

Where there is a large decrease in the width of the structure above the first storey along with a corresponding large change in storey stiffness that creates a flexible tower, then NSCP 2015 Section 208.4.8.2, Item 4 and Section 208.5.4.1, Item 2 may apply.

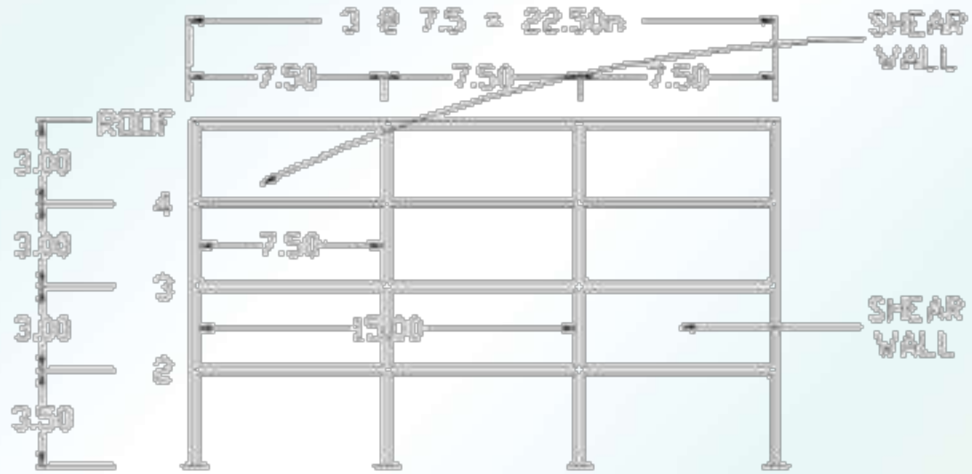
Note that if the frame elements in the bay between lines 4 and 5 were not included as a part of the designated lateral force resisting system, then the vertical geometric irregularity would not exist. However, the effects of this adjoining frame would have to be considered under the adjoining rigid elements requirements of NSCP 2015 Section 208.8.2.3.1.

Vertical Irregularity Type 4 – In Plane Discontinuity in Vertical Lateral Force Resisting Element irregularity

Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any storey is more than 130% of that in an adjacent storey. One – storey penthouses need not be considered.

Example 4

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



Determine if there is a Type 4 vertical irregularity, in-plane discontinuity in the vertical lateral force-resisting element

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Example 4 – Calculation and Discussion

A Type 4 vertical irregularity exists where there is an in-plane of the lateral load resisting elements greater than the length of those elements. In this example, the left side of the lower shear wall (between lines C and D). This 15m offset is greater than the 7.5m length of the offset wall elements.

$$\therefore In - plane\ discontinuity\ exists!$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Example 4 – Calculation and Discussion

Commentary:

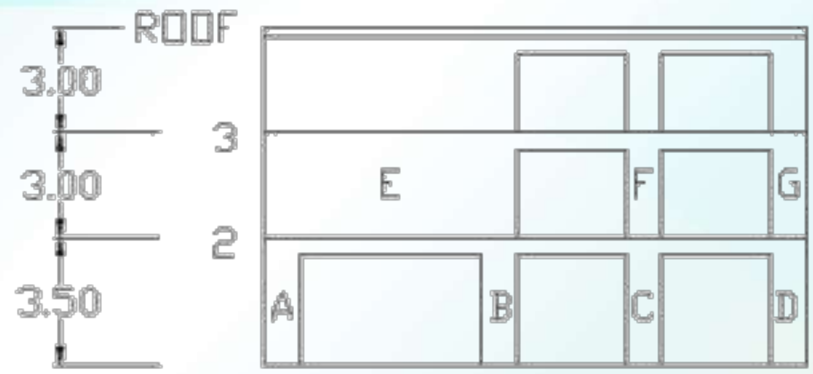
The intent of this irregularity check is to provide correction of force transfer or load path deficiencies. It should be noted that any in-plane offset, even those less or equal to the length of bay width of the resisting element, can result in an overturning moment load transfer discontinuity that requires application of NSCP 2015 Section 208.5.3.1. When the offset exceeds the length of the resisting element, there is also a shear transfer discontinuity that requires application of NSCP 2015 Section 208.8.2.5 for the strength of collector elements along the offset. In this example, the columns under wall A-B are subject to the provisions of NSCP 2015 Section 208.5.8 and the collector element between Lines B and C at Level 3 is subject to the provisions of NSCP 2015 Section 208.8.2.5.

Vertical Irregularity Type 5 – Discontinuity In Capacity – Weak Storey Irregularity

A weak storey is one in which the storey strength is less than 80% of that in the storey above. The storey strength is the total strength of all seismic – resisting elements sharing the storey for the direction under consideration.

Example 5

A concrete bearing wall building has the typical transverse shear wall configuration shown below. All walls in this direction are identical and the individual piers have the shear contribution given below. V_n is the nominal shear strength calculated in accordance with NSCP 2015 and V_m is the shear corresponding to the development of the nominal flexure strength calculated in accordance with NSCP 2015.



PIER	V_n	V_m
A	20 KN	30 KN
B	30 KN	40 KN
C	15 KN	10 KN
D	20 KN	15 KN
E	80 KN	120 KN
F	15 KN	10 KN
G	20 KN	15 KN

FEU ALABANG FEU DILIMAN FEU TECH

DESIGN RESPONSE SPECTRA AND STRUCTURAL DETAILING FOR EARTHQUAKE AND STRUCTURAL SYSTEMS

Horizontal Structural Irregularities

Introduction to Horizontal Structural Irregularities

Horizontal structural irregularities are identified in NSCP 2015 Table 208-10. These are five types of plan irregularities.

1. Torsional irregularity – to be considered when diaphragms are not flexible.
2. Re-entrant corners
3. Diaphragm discontinuity
4. Out-of-plane offsets
5. Non-parallel systems

Introduction to Horizontal Structural Irregularities

These irregularities can be categorized as being either special response conditions or cases of irregular load path. Types 1,2,3 and 5 are special response conditions:

Type 1:

When the ratio of maximum drift to average drift exceeds the given limit, there is the potential for an unbalance in the inelastic deformation demands at the two extreme sides of a storey. As a consequence, the equivalent stiffness of the side having maximum deformation will be reduced, and the eccentricity between the centers of mass and rigidity will be increased along with the corresponding torsions. An amplification factor A_x is to be applied to the accidental eccentricity to represent the effects of this unbalanced stiffness.

Introduction to Horizontal Structural Irregularities

These irregularities can be categorized as being either special response conditions or cases of irregular load path. Types 1,2,3 and 5 are special response conditions:

Type 2:

The opening and closing deformation response or flapping action of the projecting legs of the building plan adjacent to re-entrant corners can result in concentrated forces at the corner point. Elements must be provided to transfer these forces into the diaphragms.

Introduction to Horizontal Structural Irregularities

These irregularities can be categorized as being either special response conditions or cases of irregular load path. Types 1,2,3 and 5 are special response conditions:

Type 3:

Excessive openings in a diaphragm can result in a flexible diaphragm response along with force concentrations and load path deficiencies at the boundaries of the openings. Elements must be provided to transfer the forces into the diaphragm and the structural system

Introduction to Horizontal Structural Irregularities

These irregularities can be categorized as being either special response conditions or cases of irregular load path. Types 1,2,3 and 5 are special response conditions:

Type 4:

The type 4 plan irregularity, out-of-plane offset, represents the irregular load path category. In this case, shears and overturning moments must be transferred from the level above the offset, and there is a horizontal “offset” in the load path for the shears.

Introduction to Horizontal Structural Irregularities

These irregularities can be categorized as being either special response conditions or cases of irregular load path. Types 1,2,3 and 5 are special response conditions:

Type 5:

The response deformations and load patterns on a system with non-parallel lateral force-resisting elements can have significant differences from that of a regular system. Further analysis of deformation and load behavior may be necessary.

Horizontal Irregularity Type 1 – To be considered when Diaphragms not Flexible

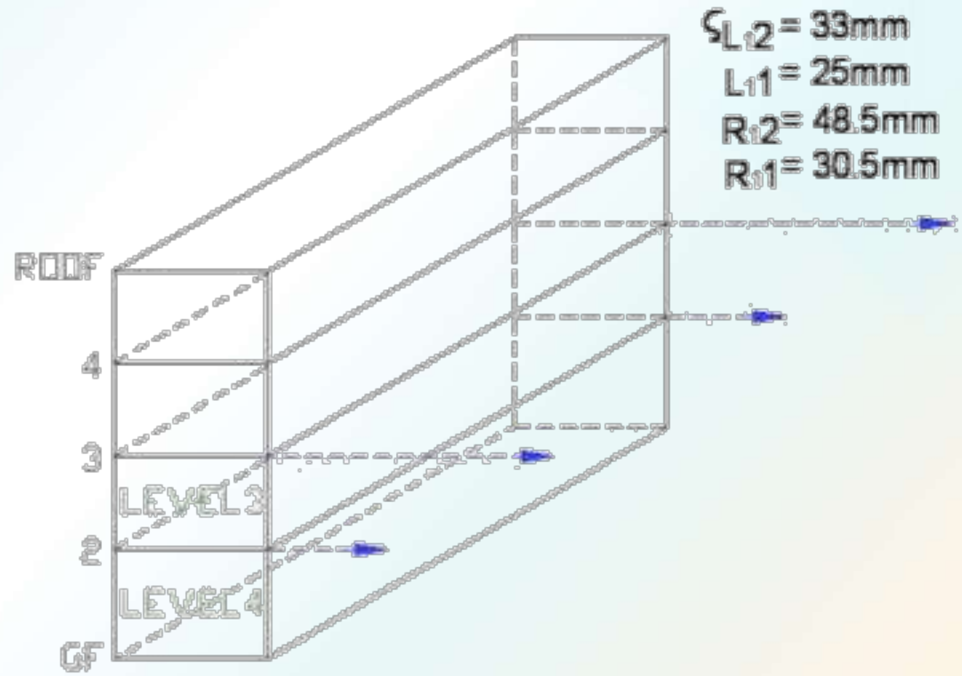
Torsional irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

Example 5

A four-storey 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.

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

2. Compute the torsional amplification factor A_x for Level 3.



$$\delta_{L,2} = 33mm \qquad \delta_{R,2} = 48.5mm$$

$$\delta_{L,1} = 25mm \qquad \delta_{R,1} = 30.5mm$$

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

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

Referring to the above figure showing the displacements δ due to the prescribed lateral forces, this irregularity check is defined in terms of storey 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.

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

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

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

$$\Delta_{\max} > 1.2\left(\Delta_{Avg}\right)$$

FEU ALABANG FEU DILIMAN FEU TECH

Technology Driven by Innovation

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

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

Where:

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

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

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

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

Determining storey drifts at Level 3

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

$$\Delta_{R,2} = 48.5 - 30.5 = 18mm$$

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

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

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

Checking 1.2 criteria

$$\frac{\Delta_{\max}}{\Delta_{avg}} = \frac{\Delta_{R,2}}{\Delta_{avg}}$$

$$\frac{\Delta_{\max}}{\Delta_{avg}} = \frac{18}{13} = 1.4 > 1.2 \qquad \therefore \textit{Torsional Irregularity Exists!}$$

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

2. Compute amplification factor A_x for Level 3

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 may have a different A_x value. In this example, A_x is computed for Level 2.

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

2. Compute amplification factor A_x for Level 3

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

$$\delta_{\max} = \delta_{R,2} = 48.5mm$$

$$\delta_{avg} = \frac{\delta_{L,2} + \delta_{R,2}}{2}$$

$$\delta_{avg} = \frac{33 + 48.5}{2}$$

$$\delta_{avg} = 40.75mm$$

Example 5 – Calculations and Discussion

A Type 1 torsional plan irregularity is considered to exist when the maximum storey drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

2. Compute amplification factor A_x for Level 3

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

$$A_x = \left[\frac{48.5}{1.2(40.75)}\right]^2$$

$$A_x = 0.98 < 1.0 \qquad \therefore use\ A_x = 1.0$$

Example 5 – Calculations and Discussion

Commentary:

In NSCP 2015 Section 208.5.7, there is the provision that "the most severe load combination must be considered." The interpretation of this for the case of the storey drift and displacements to be used for the average values $\Delta\delta_{avg}$ and δ_{avg} is as follows. The most severe condition is when both $\delta_{R,x}$ and $\delta_{L,x}$ are computed for the same accidental center of mass displacement that causes the maximum displacement δ_{max}. For the condition shown in this example where $\delta_{R,x}$=δ_{max}, the center of mass at all levels should be displaced by the accidental eccentricity to the right side R, and both $\delta_{R,x}$ and $\delta_{L,x}$ should be evaluated for this load condition.

Example 5 – Calculations and Discussion

Commentary:

While NSCP Table 208-10 calls only for NSCP Section 208.8.2.8, Item 6 (regarding diaphragm connections) to apply of this irregularity exists, there is also NSCP Section 208.5.7, which requires the accidental torsion amplification factor A_x given by NSCP Equation 208-16. It is important to recognize that torsional irregularity is defined in terms of storey drift $\Delta\delta_x$ while the evaluation of A_x by Equation 208-16 is in terms of displacements δ_x. There can be instances where the storey drift values indicate torsional irregularity and where the related displacement values produce an A_x value less than one. This result is not the intent of the provision, and the value of A_x used to determine accidental torsion should not be less than 1.0.

Example 5 – Calculations and Discussion

Commentary:

The displacement and storey drift values should be obtained by the equivalent lateral force method with the specified lateral forces. Theoretically, if the dynamic analysis procedure were to be used, the values of $\Delta\delta_{max}$ and $\Delta\delta_{avg}$ would have to be found for each dynamic mode, then combined by the appropriate SRSS or CQC procedures, and then scaled to the specified base shear. However, in view of the complexity of this determination and the judgmental nature of the 1.2 factor, it is reasoned that the equivalent static force method is sufficiently accurate to detect torsional irregularity and evaluate the A_x factor.

Example 5 – Calculations and Discussion

Commentary:

If the dynamic analysis procedure is either elected or required, then NSCP Section 208.6.3 requires the use of a three – dimensional model if there are any of the plan irregularities listed in NSCP Table 208-10. For cases of large eccentricity and low torsional rigidity, the static force procedure can result in a negative displacement on one side and a positive on the other. For example, this occurs if $\delta_{L,3}$=-10mm and $\delta_{R,3}$=46mm. The value of δ_{avg} in NSCP Equation 208-16 should be calculated as the algebraic average:

$$\delta_{avg} = \frac{\delta_{L,3} + \delta_{R,3}}{2} \qquad \delta_{avg} = \frac{(-10) + 46}{2} \qquad \delta_{avg} = \frac{36}{2} = 18mm$$

Example 5 – Calculations and Discussion

Commentary:

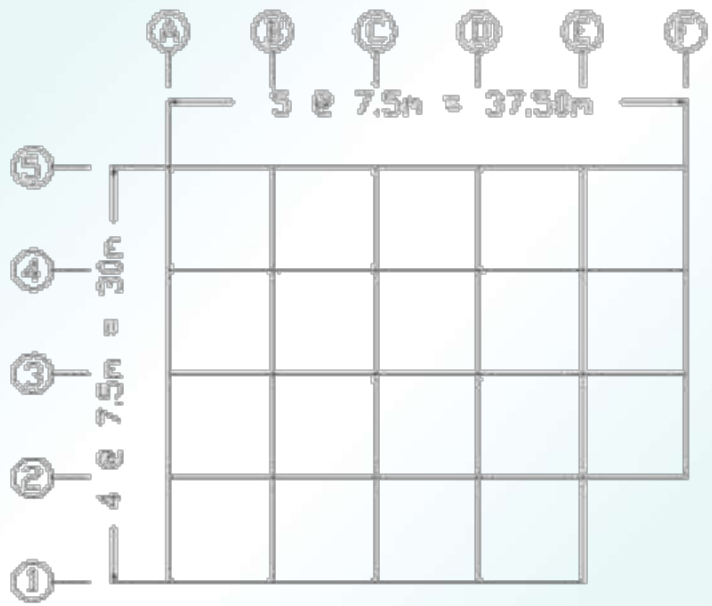
When dynamic analysis is used, the algebraic average value δ_{avg} should be found for each mode, and the individual modal results must be properly combined to determine the total response value for δ_{avg}

Horizontal Irregularity Type 2 – Re-Entrant Corner Irregularity

Torsional irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.

Example 5

The plan configuration of a ten – storey special moment frame building is as shown below:



Determine if there is a Type 2 re-entrant corner irregularity.

Example 5 – Calculation and Discussion

A Type 2 Re – Entrant Corner Plan Irregularity exists when the plan configuration of a structure and its lateral force resisting system contain re – entrant corners where both projections of the structure beyond a re – entrant corner are greater than 15 percent of the plan dimension of the structure in the direction considered.

The plan consideration of this building and its lateral force resisting system; have identical re – entrant corner dimensions. For the sides on lines 1 and 5 the projection beyond the re – entrant corner is

Example 5 – Calculation and Discussion

37.5m – 30m = **7.5m**

This is $\frac{7.5}{37.5}$ or 20% of the 37.5 plan dimension.

For the sides on lines A and F, the projection is 30m – 22.5m = 7.5m,

This is $\frac{7.5}{30}$ or 25% of the 30m plan dimension.

Since both projections exceed 15% there is a re – entrant corner irregularity.

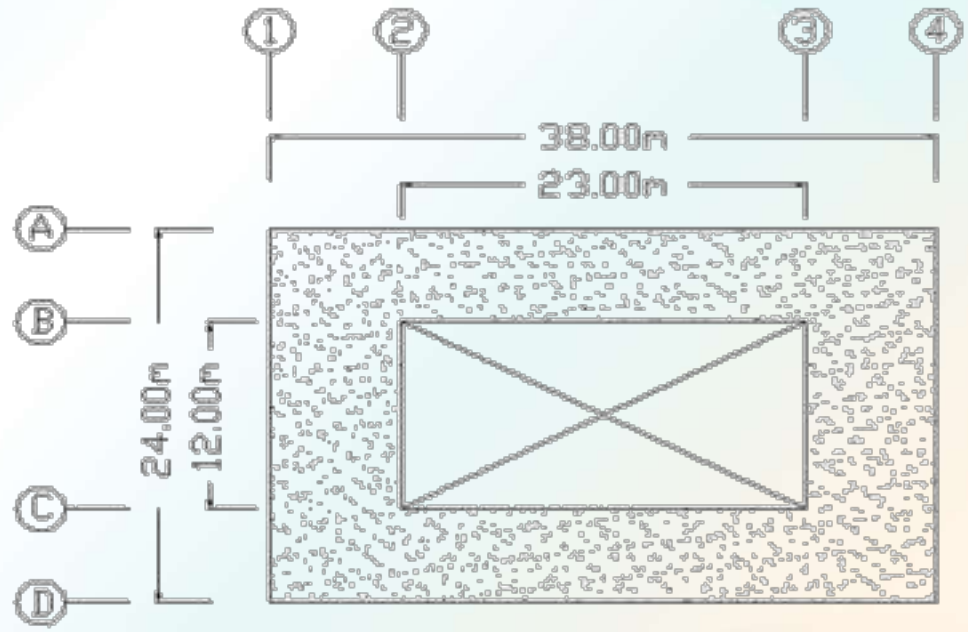
- ✓ **RE – ENTRANT CORNER IRREGULARITY EXISTS**

Horizontal Irregularity Type 3 – Diaphragm Discontinuity Irregularity

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open area greater than 50% of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50% from one storey to the next.

Example 6

A five storey concrete building has a bearing wall system located around the perimeter of the building. Lateral forces are resisted by the bearing walls acting as shear walls. The floor plan of the second floor of the building is as shown below. The symmetrically placed open area in the diaphragm is for an atrium, and has dimensions of 12m x 23m. All diaphragms above the second floor are without significant openings.



Determine if a Type 3 diaphragm discontinuity exists at the second floor level.

Example 6 – Calculation and Discussion

Gross Enclosed Area of the Diaphragm = 24m X 38m

Gross Enclosed Area of the Diaphragm = 912 m^2

Area of Opening = 12m X 23m

Area of Opening = 276 m^2

50% of Gross Area = 0.5 (912)

50% of Gross Area = 456 m^2

Example 6 – Calculation and Discussion

Area of Opening = 276 m^2 < 50% of Gross Area = 456 m^2

✓ **NO DIAPHRAGM DISCONTINUITY IRREGULARITY EXISTS**

Commentary:

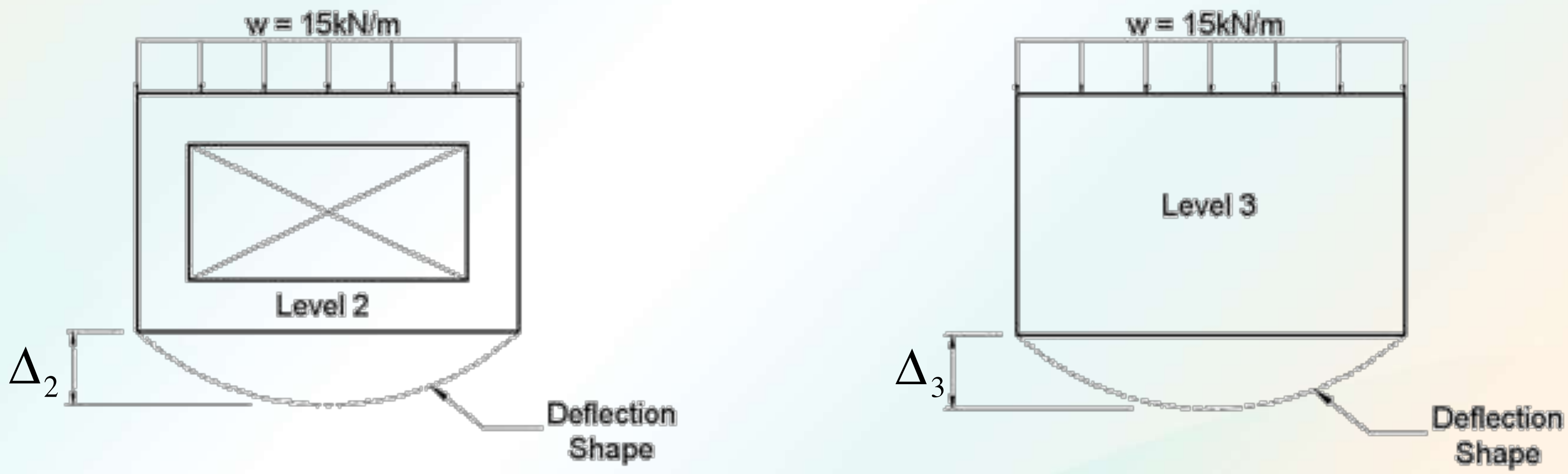
The stiffness of the second floor diaphragm with its opening must be compared with the stiffness of the solid diaphragm at the third floor. If the change in stiffness exceeds 50 percent, then a diaphragm discontinuity irregularity exists for the structure.

Example 6 – Calculation and Discussion

Commentary:

This comparison can be performed as follows:

Find the simple beam mid-span deflections Δ_2 and Δ_3 for the diaphragms at Levels 2 and 3, respectively, due to a common distributed load w, such as 15 kN/m.



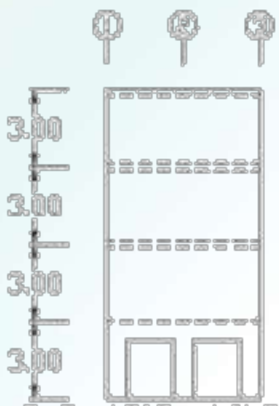
If $\Delta_2 > 1.5\Delta_2$, then there is diaphragm discontinuity

Horizontal Irregularity Type 4 – Out-Of-Plane Offsets Irregularity

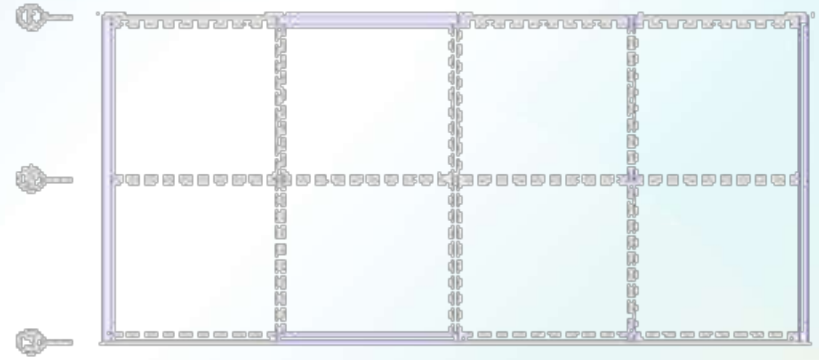
Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.

Example 7

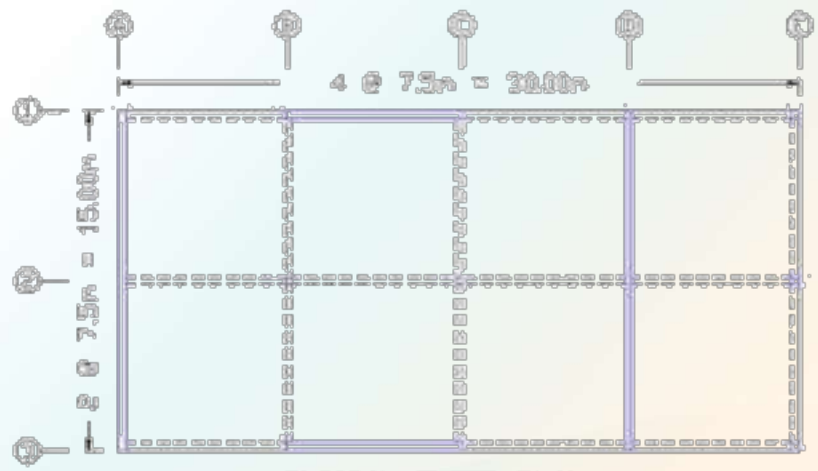
A four – storey building has a concrete shear wall lateral force-resisting system in a building frame system configuration. The plan configuration of the shear walls is shown below.



ELEVATION LINE E



TYPICAL FLOOR PLAN



TYPICAL FLOOR PLAN

Determine if there is a Type 4 out-of-plane offset plan irregularity between the first and second storeys.

Example 7 – Calculation and Discussion

An out-of-plane offset plan irregularity exists when there are discontinuities in a lateral force path, for example: out-of-plane offsets of vertical resisting elements such as shear walls. The first storey shear wall on Line D has 7.5m out-of-plane offset to the shear wall on line E at the second storey and above. This constitutes an out-of-plane offset irregularity, and the referenced sections in NSCP Table 208-10 apply to the design.

$$\therefore \textit{Offset Irregularity Exists!}$$

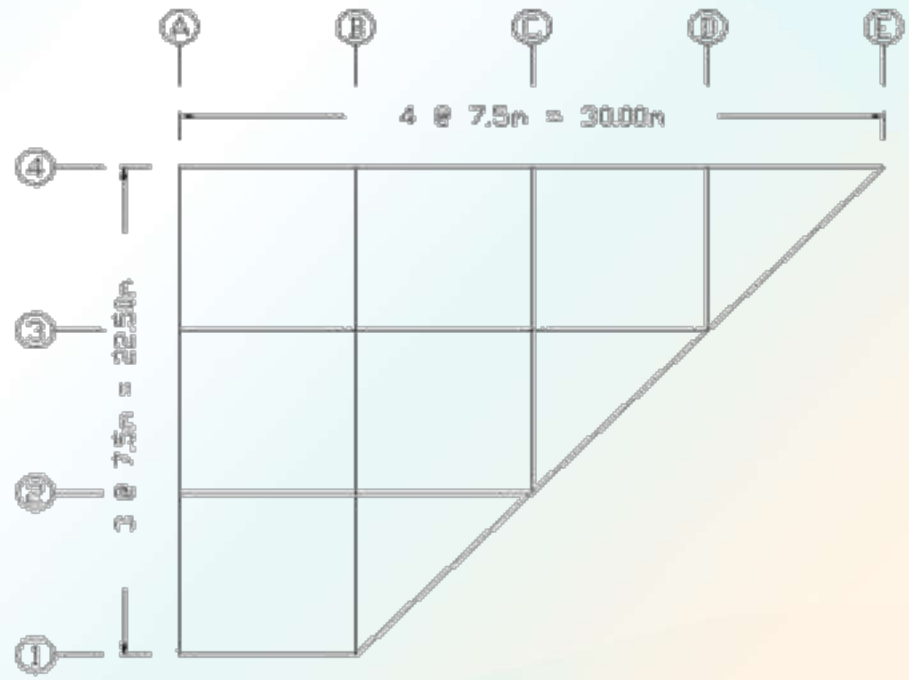
Horizontal Irregularity Type 5 – Non-Parallel Systems Irregularity

The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting systems.

Example 8

A ten – storey building has the floor plan shown below at all levels. Special moment resisting frames are located on the perimeter of the building on Lines 1, 4, A and F.

Determine if a Type 5 non-parallel system irregularity exists.



Example 8 – Calculation and Discussion

A Type 5 non-parallel system irregularity is considered to exist when the vertical lateral load resisting elements are not parallel to or symmetric about major orthogonal axes of the building's lateral force-resisting system.

The vertical lateral force-resisting frame elements located on Line F are not parallel to the major orthogonal axes of the building (i.e., Lines 4 and A). Therefore, a non-parallel system irregularity exists, and the referenced section in NSCP Table 208-10 applies to the design.

$$\therefore A\ Non - parallel\ System\ Irregularity\ Exists!$$

Q&A SESSION

ASK ANY QUESTION RELATED TO OUR TOPIC FOR TODAY.

HOMEWORK

Do the Exercises about Base Shear Computations

Reference:

Association of Structural Engineers of the Philippines (2016). *National Structural Code of the Philippines 2015 (7th Edition)Volume 1 Buildings, Towers and Other Vertical Structures.*

Association of Structural Engineers of the Philippines (2003). *ASEP Earthquake Design Manual 2003 Volume 1: Code Provisions for Lateral Forces.*