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Performance-based Design

Modeling for Nonlinear Analysis

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Why Nonlinear Analysis

Why Nonlinear Analysis Needed

- Structures are Damaged in Earthquakes
  - Not because of large forces, but because of large deformations
- Structures do not behave linearly or elastically
  - Infinite load can produce infinite deformation
  - Capacity of member is not accounted for in analysis
- Reinforced Concrete is a highly nonlinear–inelastic material
- The Performance Determination requires either Nonlinear Static Pushover Analysis or Non-linear Dynamic Analysis
-

Conditions of Linearity

- Stress-strain relationship must be linear and elastic. Most materials exhibit a change in stiffness or modulus before inelastic or plastic behavior starts.

- Displacements and rotations must be small such that the assumption "plane remain plane after deformation" is still valid. Mathematically, it is being approximated as $\sin(\theta) = \theta$ or $\tan(\theta) = 0$.

- The magnitude, orientation or direction and distribution of loads must not change.

Symptoms of Nonlinear Behavior

- Stress levels approach the yield point.
  - Most materials exhibit a significant range of nonlinear elastic behavior long before the yield stress is reached.
  - When a material is strained beyond its proportional limit, the stress-strain relationship is no longer linear.
  - Yield stress value after 0.2% offsetting the linear slope of the stress-strain curve may be higher or lower than the elastic limit.
- However, maximum stress approaching and/or exceeding yield point may be highly localized, which can be redistributed and dissipated to less stressed geometry around it, thus nonlinear analysis may not be necessary. It needs engineering judgment and expertise.

Symptoms of Nonlinear Behavior

- Large displacement.
  - Excessive displacement is usually considered a failure condition, regardless of the stress levels.
- Coupled displacements are restrained.
  - the degree of nonlinearity due to displacements will be small in a lightly constrained case and larger as the constraints restrict the natural movement of the material.

Comprehensive Equilibrium Equation

Mass-Acceleration

Stiffness-Displacement

External Force

$$M\ddot{u} + C\dot{u} + Ku + F_{NL} = F$$

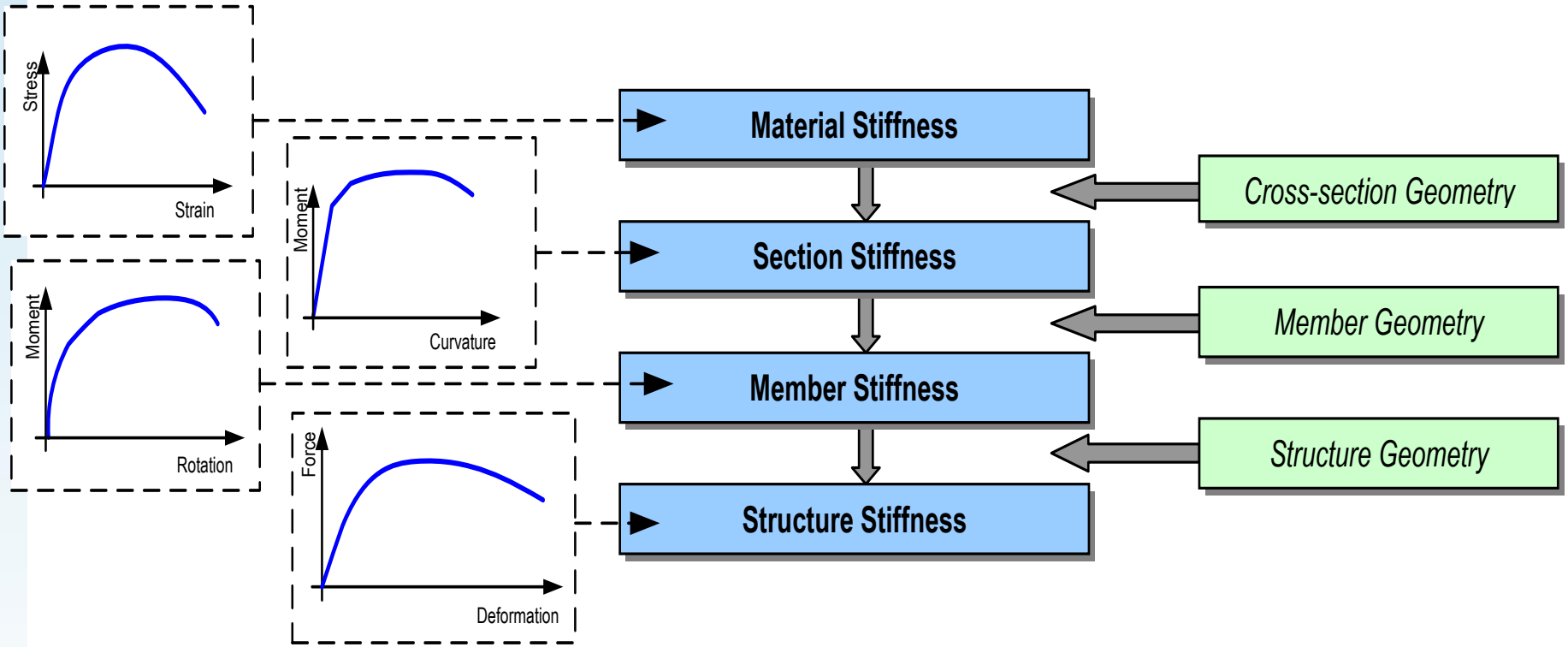
Damping-Velocity

Nonlinearity

$$M\ddot{u} + C\dot{u} + Ku$$

The basic variable is displacement and its derivatives

Structure Stiffness and Response



ANALYSIS PROCEDURES

- LINEAR PROCEDURE (LP)
  - STATIC (LSP)
  - DYNAMIC (LDP)
- NONLINEAR PROCEDURE (DP)
  - STATIC (NLSP)
  - DYNAMIC (NLDP, NLTH)

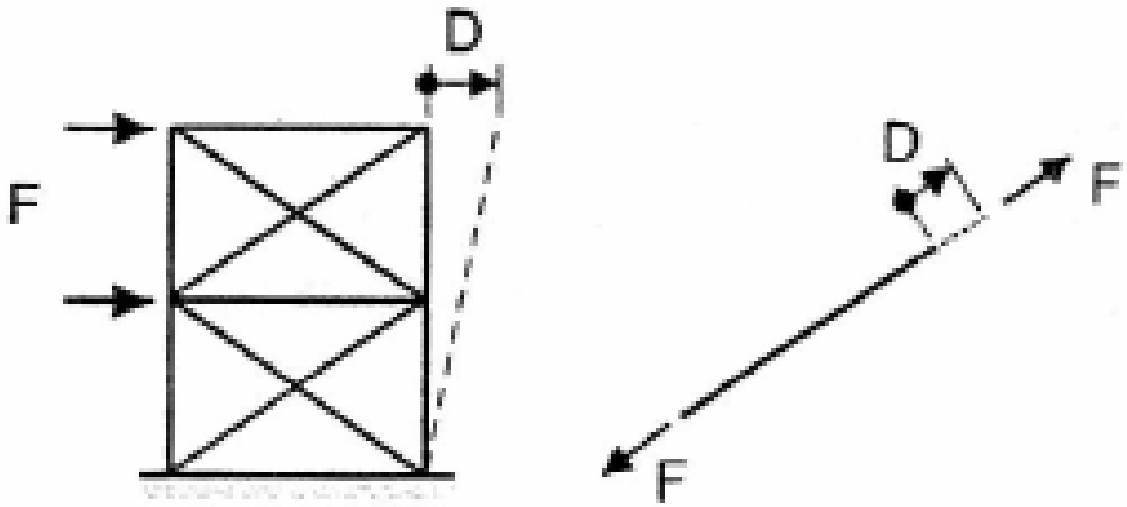
What is Nonlinearity

Action – Deformation Curves

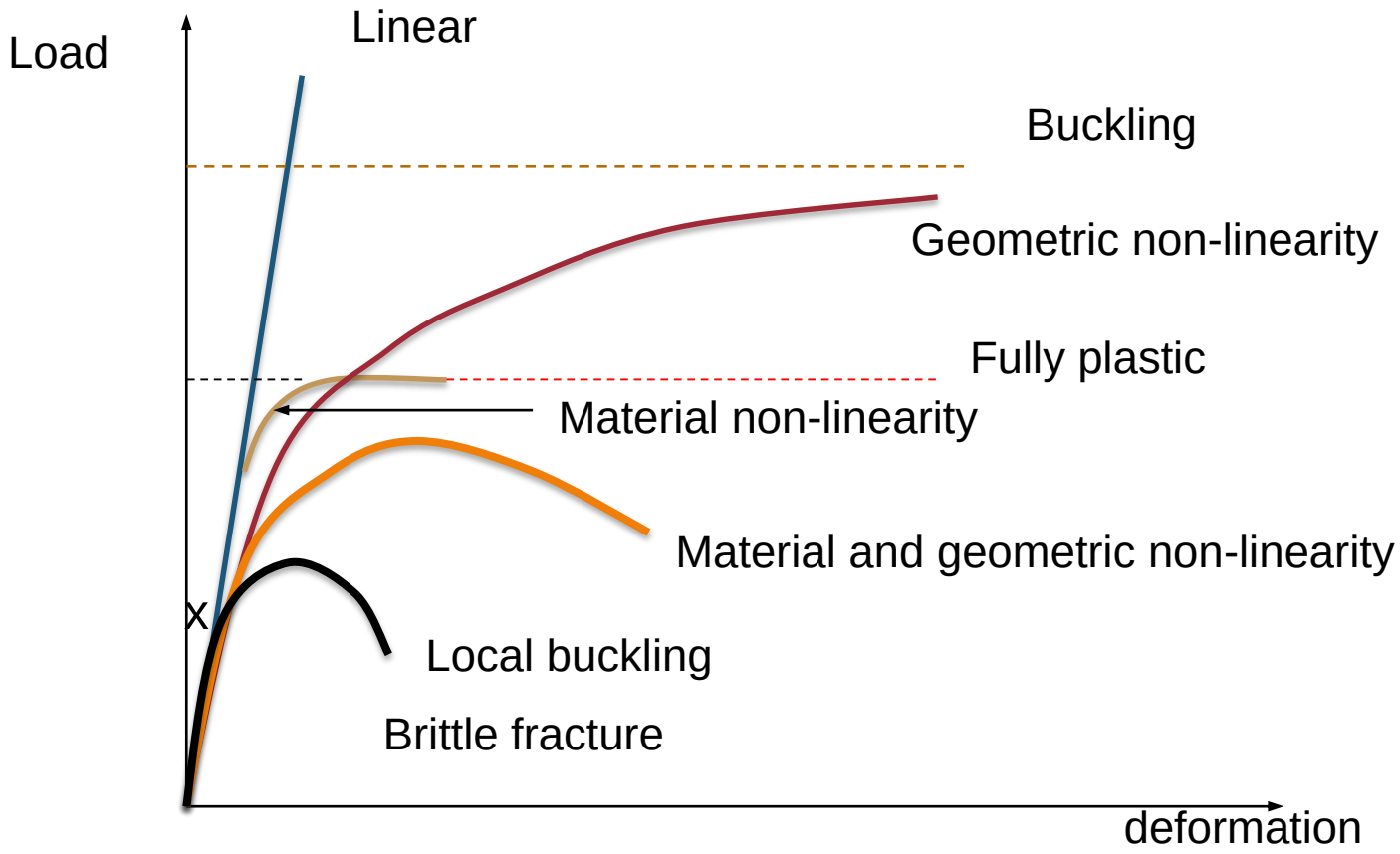
- Relationship between action and corresponding deformation
- These relationships can be obtained at several levels
  - The Structural Level: Load - Deflection
  - The Member Level: Moment - Rotation
  - The Cross-section Level: Moment - Curvature
  - The Material Level: Stress-Strain
- The Action-Deformation curves show the entire response of the structure, member, cross-section or material

Structure and Structural Components

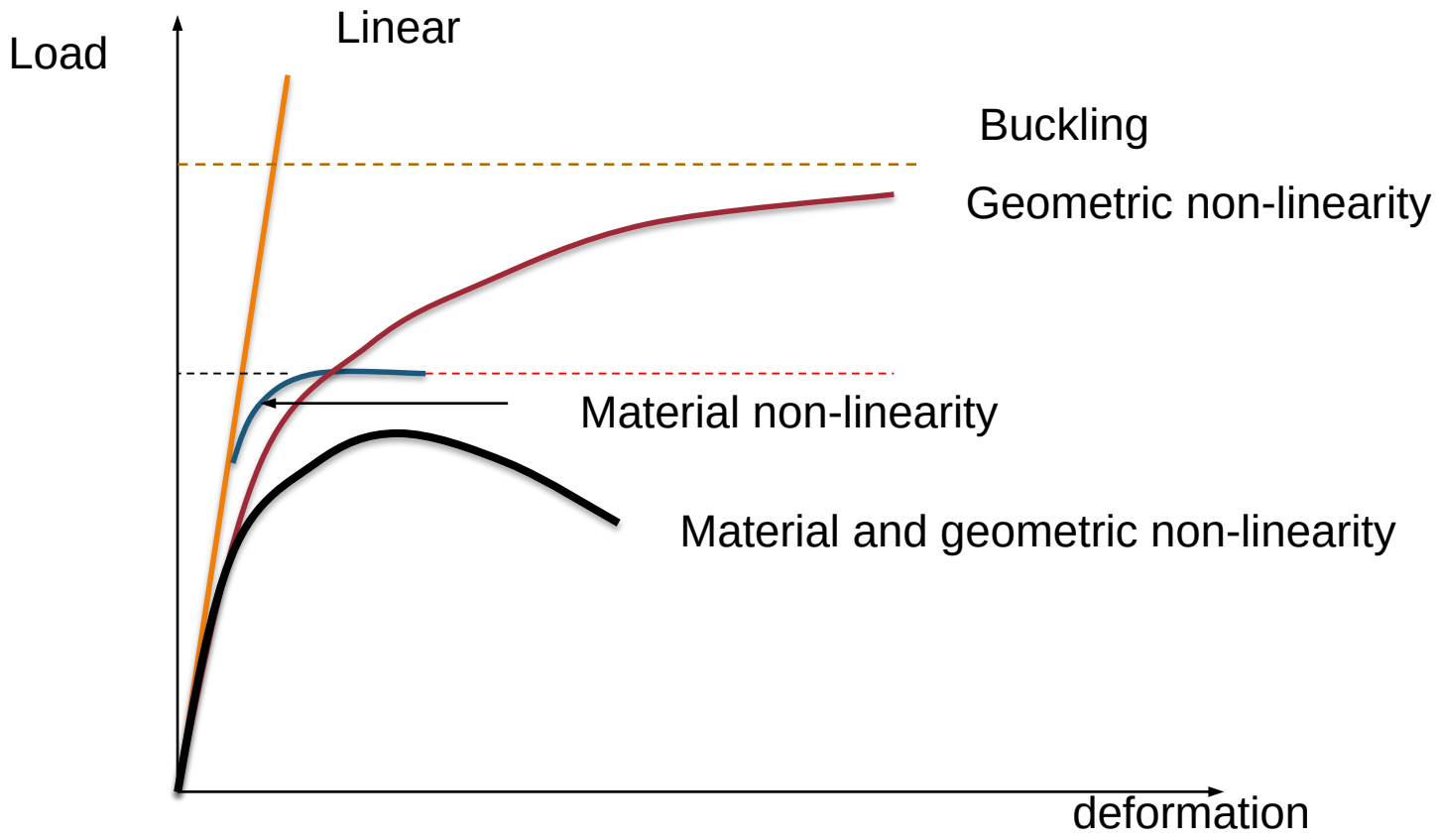
- For a structure, F = load, D = deflection.
- For a component, F depends on the component type, D is the corresponding deformation.
- The component F-D relationships must be known.
- The Structure F-D relationship is obtained by structural analysis.

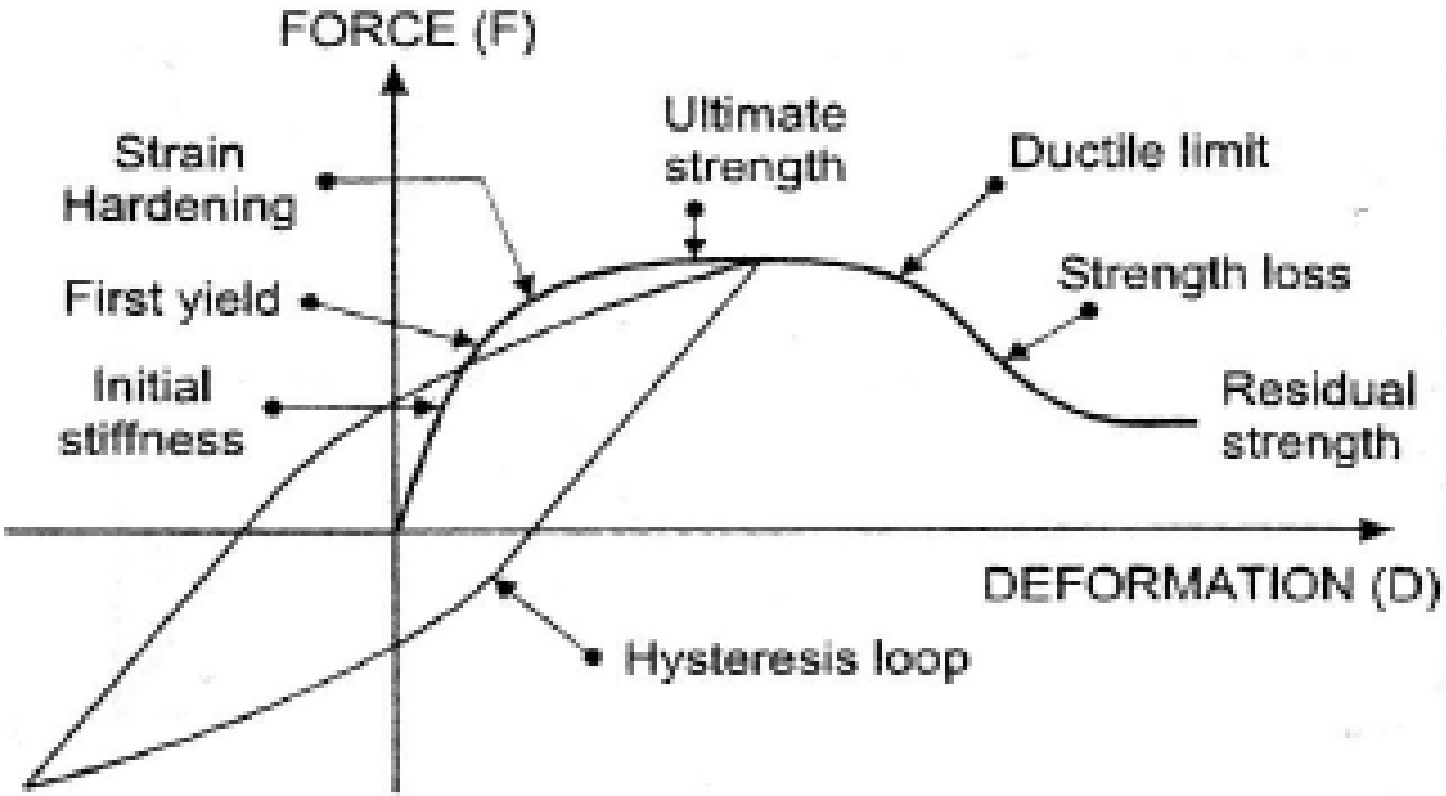


Member Behavior

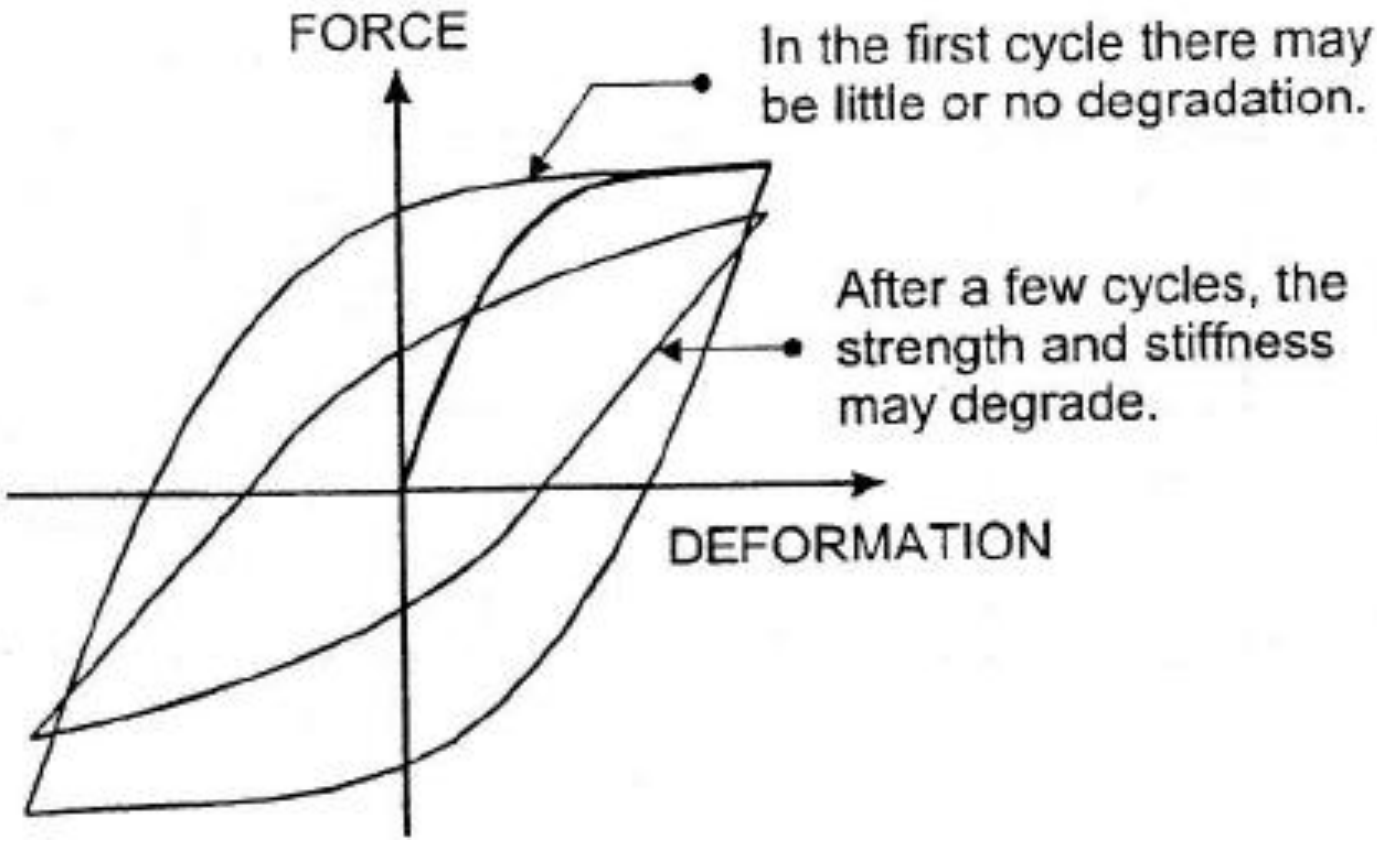


Structural Behavior

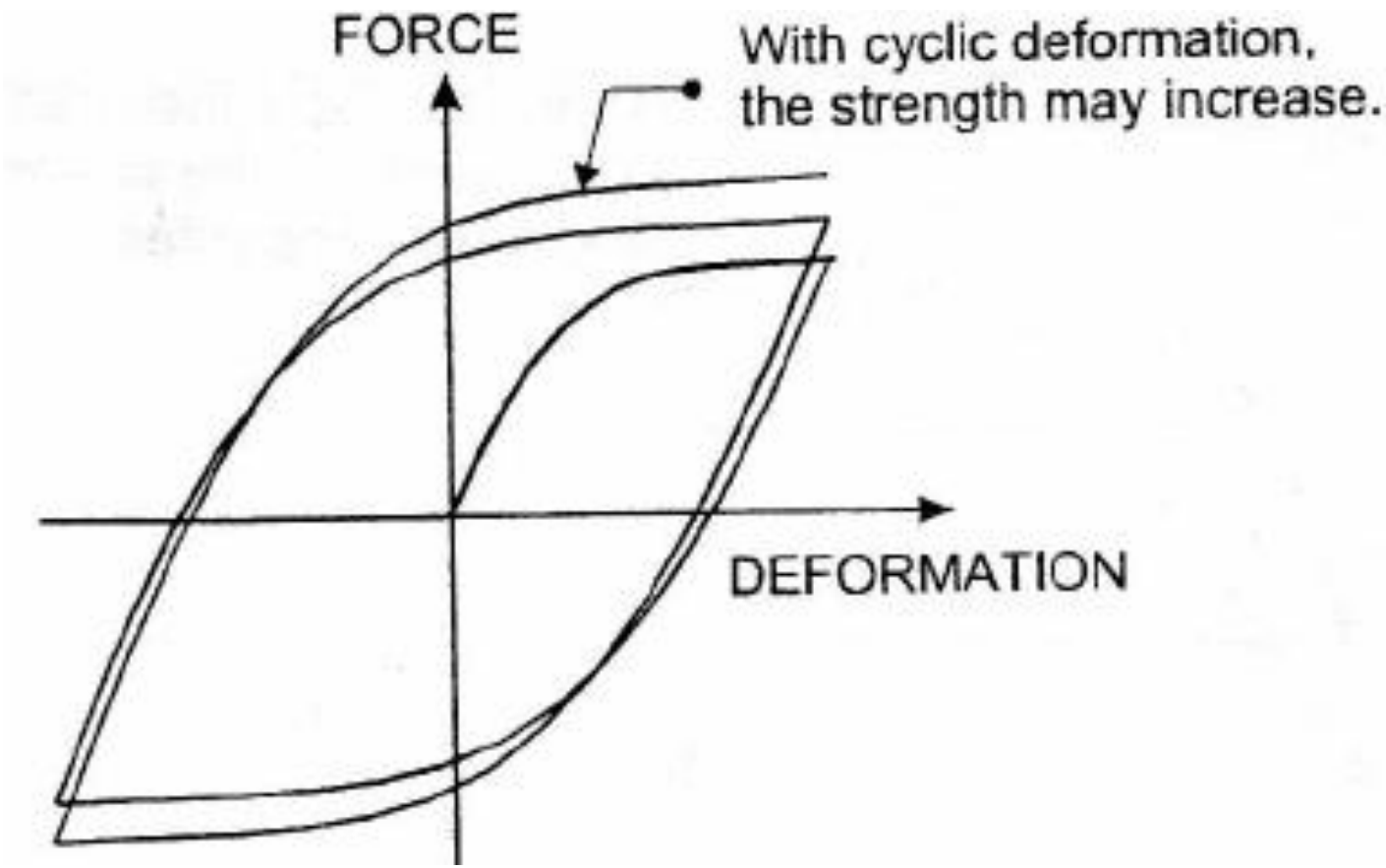




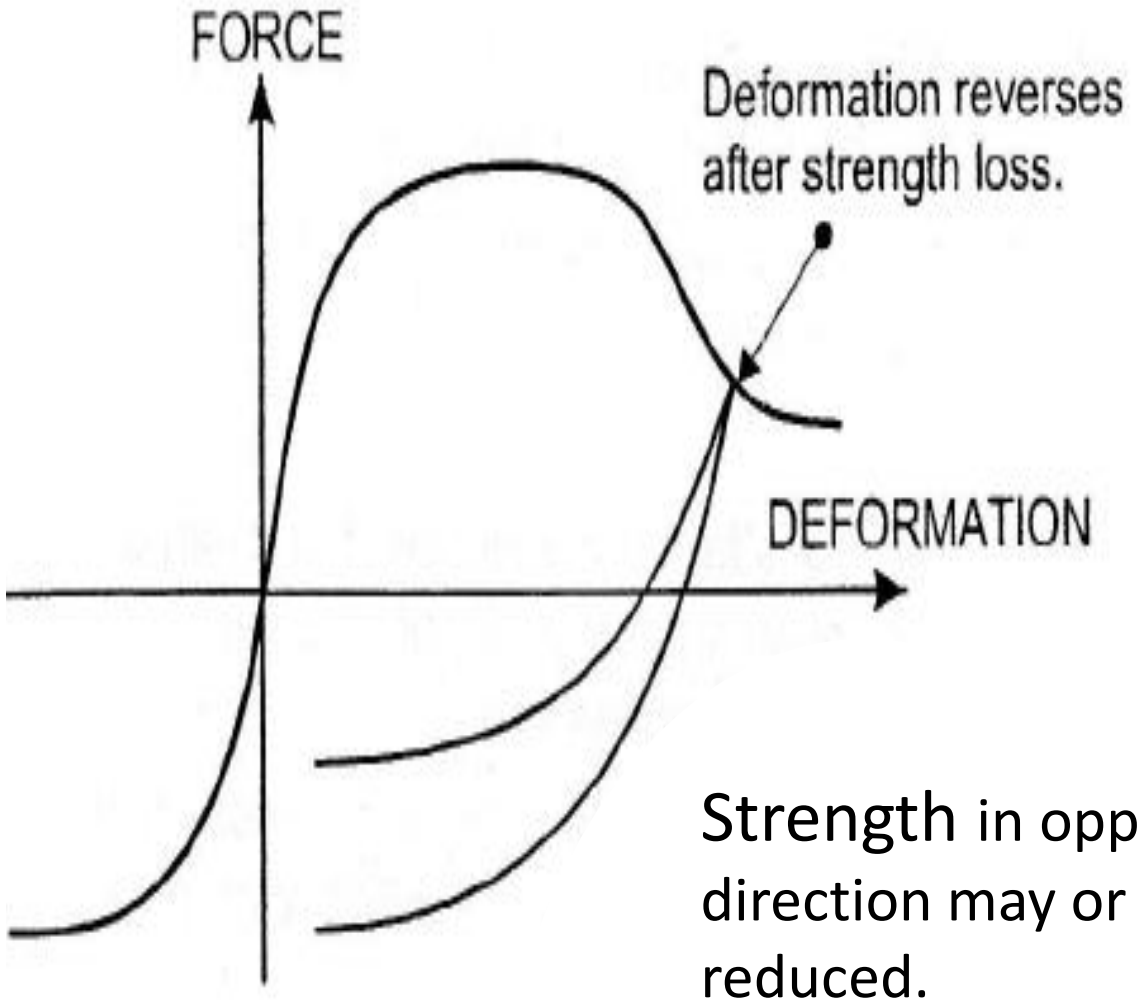
Main Aspects of F-D Relationship



Complication – Cyclic Degredation

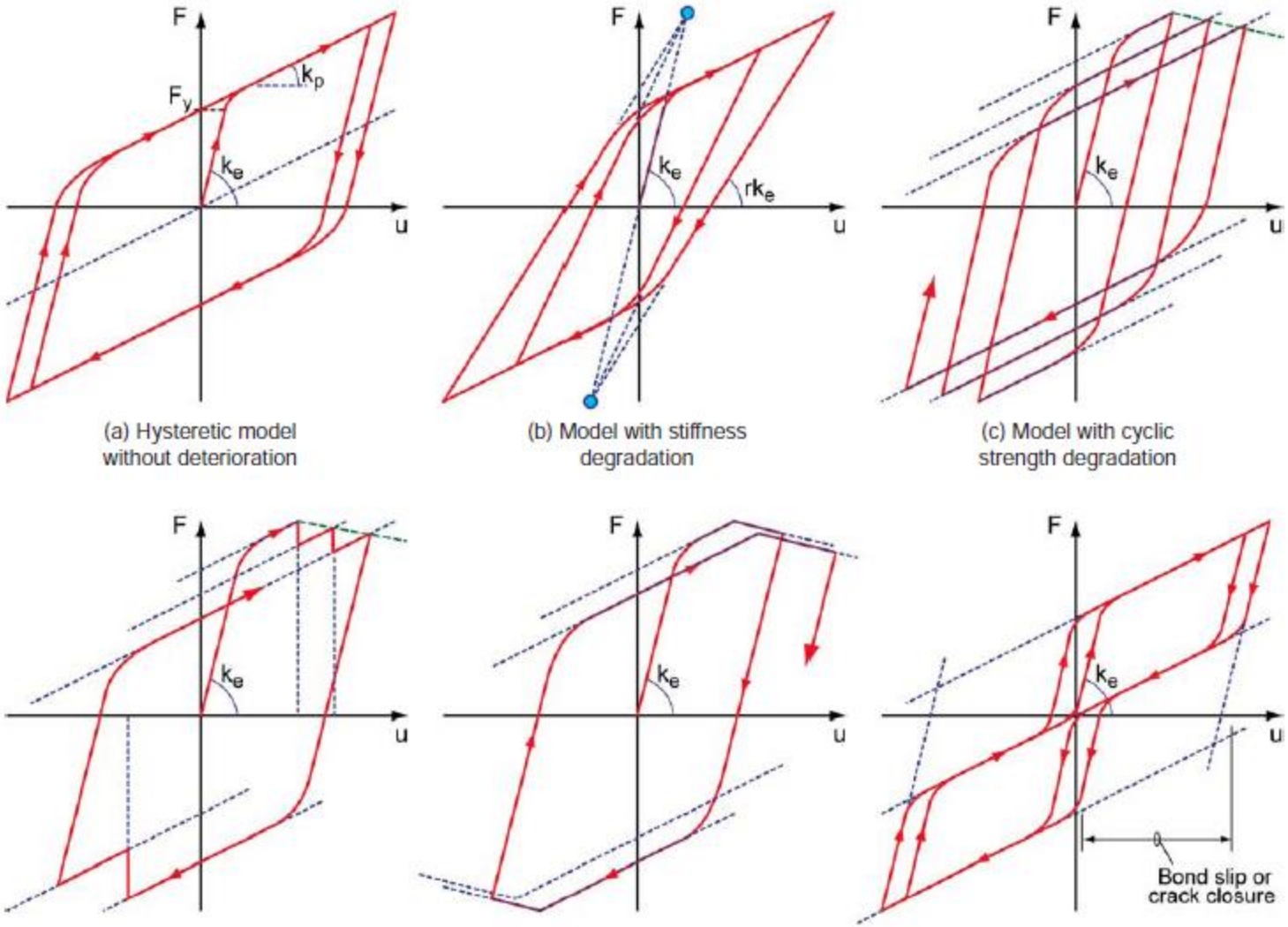


Complication – Cyclic Strength Gain



Strength in opposite direction may or may not reduced.

Complication – Effect of Strength Loss



(a) Hysteretic model without deterioration

(b) Model with stiffness degradation

(c) Model with cyclic strength degradation

(d) Model with fracture

(e) Model with post-capping gradual strength deterioration

(f) Model with bond slip or crack closure (pinching)

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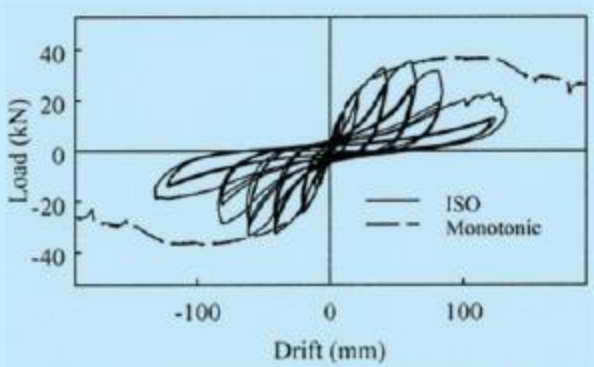
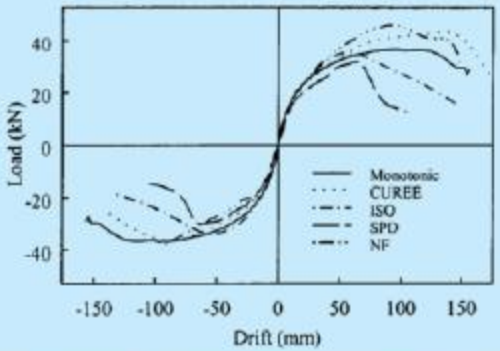
From Cyclic to Backbone Curve

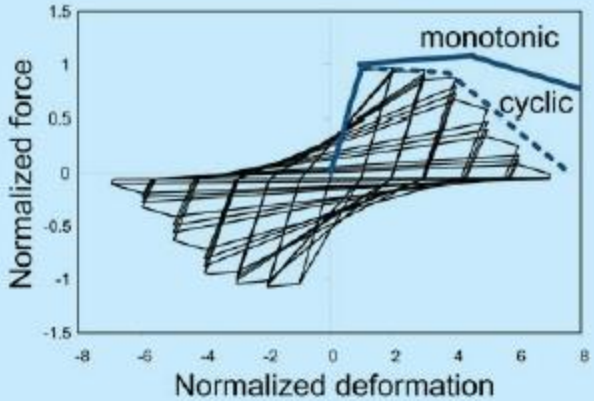


(a) Cyclic versus monotonic results



(b) Monotonic and cyclic envelope curves (ASCE 2007)

Figure 2-4 – Load versus displacement data from wood shear walls.



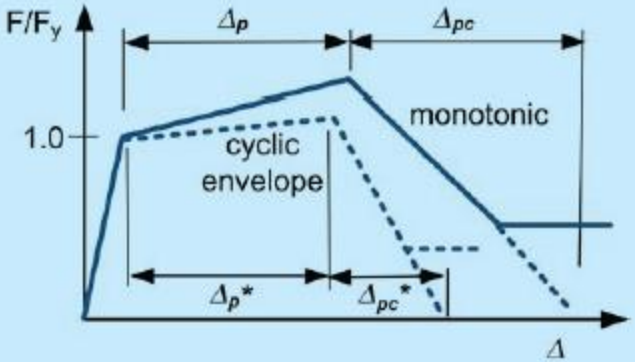


Figure 2-5 – Idealized model backbone curves derived from monotonic and cyclic envelope curves (PEER/ATC 2010).

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Effect of other Nonlinearities

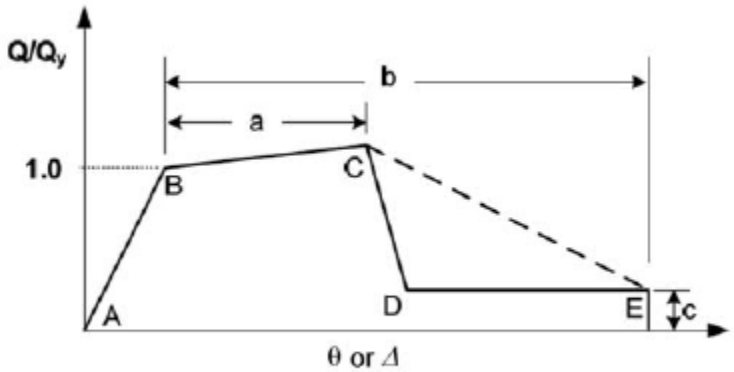

Figure 2-7 – Generalized force-deformation curve (PEER/ATC 2010).

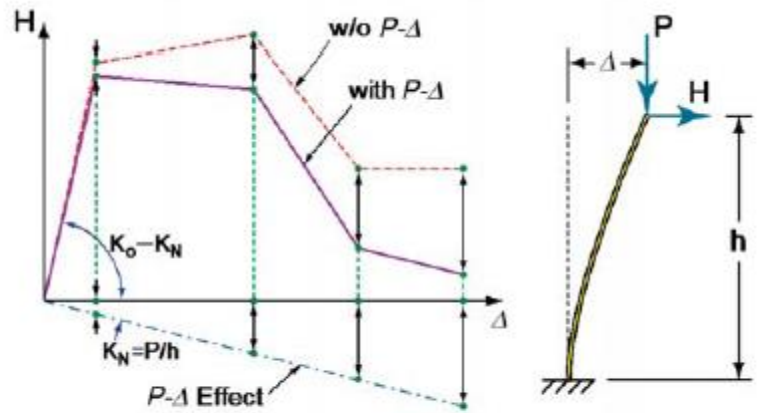

Figure 2-8 – Force-deformation curve with and without the *P*-Δ effect (PEER/ATC 2010).

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Linear Vs. Nonlinear

Feature	Linear problems	Nonlinear problems
Load displacement relationship	Displacements are linearly dependent on the applied loads.	The load-displacement relationships are usually nonlinear.
Stress-strain relationship	A linear relationship is assumed between stress and strain.	In problems involving material nonlinearity, the stress-strain relationship is often a nonlinear function of stress, strain and/or time.
Magnitude of displacement	Changes in geometry due to displacement are assumed to be small and hence ignored, and the original (undeformed) state is always used as the reference state.	Displacements may not be small, hence an updated reference state may be needed.

Linear Vs. Nonlinear

Material properties	Linear elastic material properties are usually easy to obtain	Nonlinear material properties may be difficult to obtain and may require additional experimental testing
Reversibility	The behaviour of the structure is completely reversible upon removal of the external loads	Upon removal of the external loads, the final state may be different from the initial state.
Boundary Conditions	Boundary conditions remain unchanged throughout the analysis	Boundary conditions may change, e.g. a change in the contact area.
Loading Sequence	Loading sequence is not important, and the final state is unaffected by the load history	The behaviour of the structure may depend on the load history
Iterations and increments	The load is applied in one load step with no iterations	The load is often divided into small increment with iterations performed to ensure that equilibrium is satisfied at every load increment

Linear Vs. Nonlinear

Computation time	Computation time is relatively small in comparison to nonlinear problems	Due to the many solution steps required for load incrementation and iterations, computation time is high, particularly if a high degree of accuracy is sought
Robustness of solutions	A solution can easily be obtained with no interaction from the user	In difficult nonlinear problems, the FE code may fail to converge without some interaction from the user
Use of results	Superposition and scaling allow results to be factored and combined as required	Factoring and combining of results is not possible
Initial state of stress/strain	The initial state of stress and/or strain is unimportant	The initial state of stress and/or strain is usually required for material nonlinearity problems.

Three Types of Nonlinearity

- Material Nonlinearity
  - Due to inelastic behavior of constituent materials such as concrete and steel when strained beyond proportional limit resulting to cracking, crushing, sliding, yielding, fracture, etc.
- Geometric Nonlinearity
  - Due to Due to change in shape of the structure.
  - Includes P-Δ and large displacement/rotation effects.
- Nonlinear boundary conditions
  - Due to contact such as constraints and restraints
- In many cases, if material nonlinearity is encountered, one or both of the other types will be required as well.

Nonlinear Modeling

- Types
  - Truss – Yielding and Buckling
  - 3D Beam – Major direction Flexural and Shear Hinging
  - 3D Column – P-M-M Interaction and shear Hinging
  - Panel Zone – Shear Yielding
  - In-Fill Panel – Shear Failure
  - Shear Wall – P-M-Shear Interaction!
  - Spring – for foundation modeling

Nonlinear Model

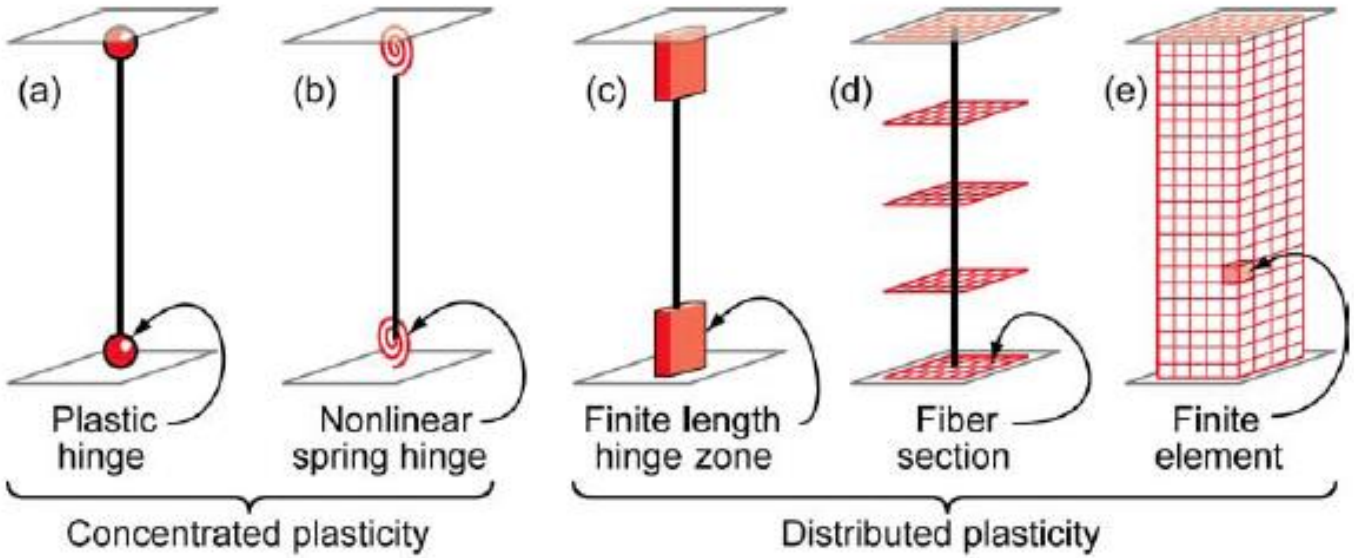

Figure 2-1 – Idealized models of beam-column elements.

Nonlinear Structural Analysis For Seismic Design: A Guide for Practicing Engineers

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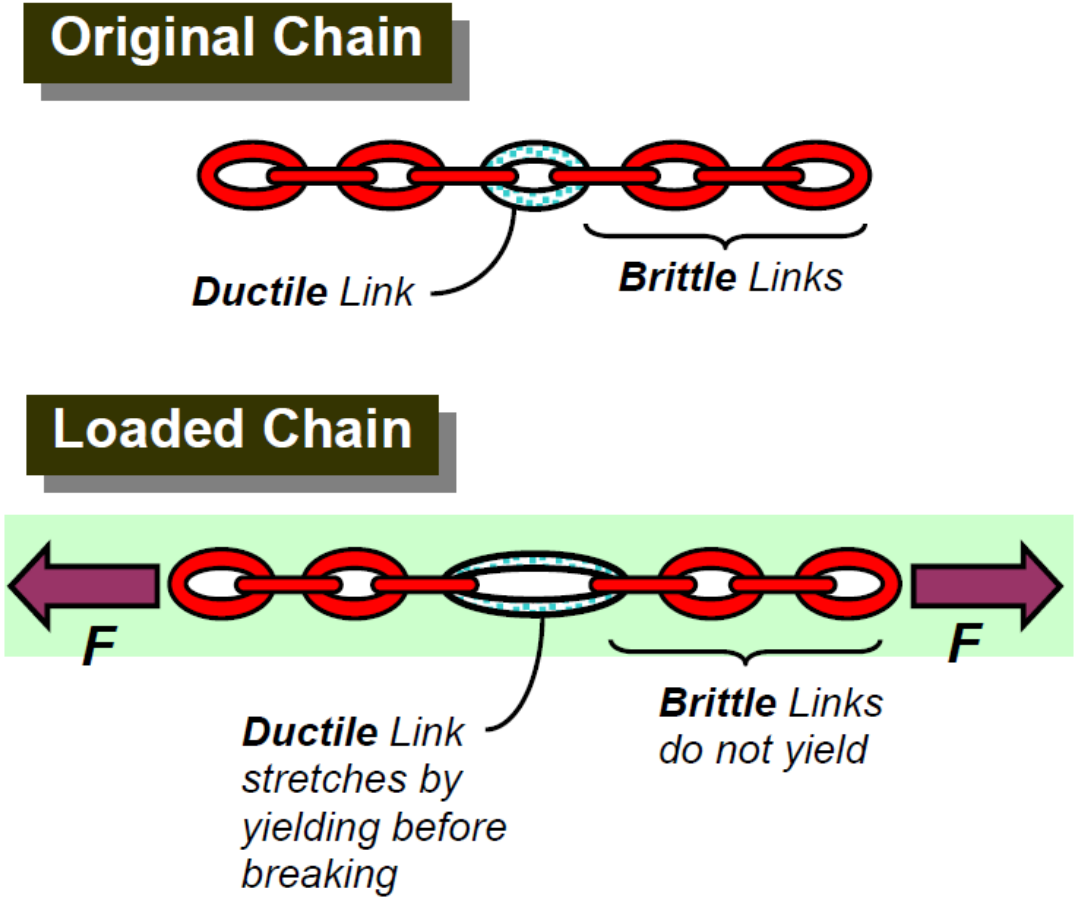
Ductile Link Analogy



Ductile chain design

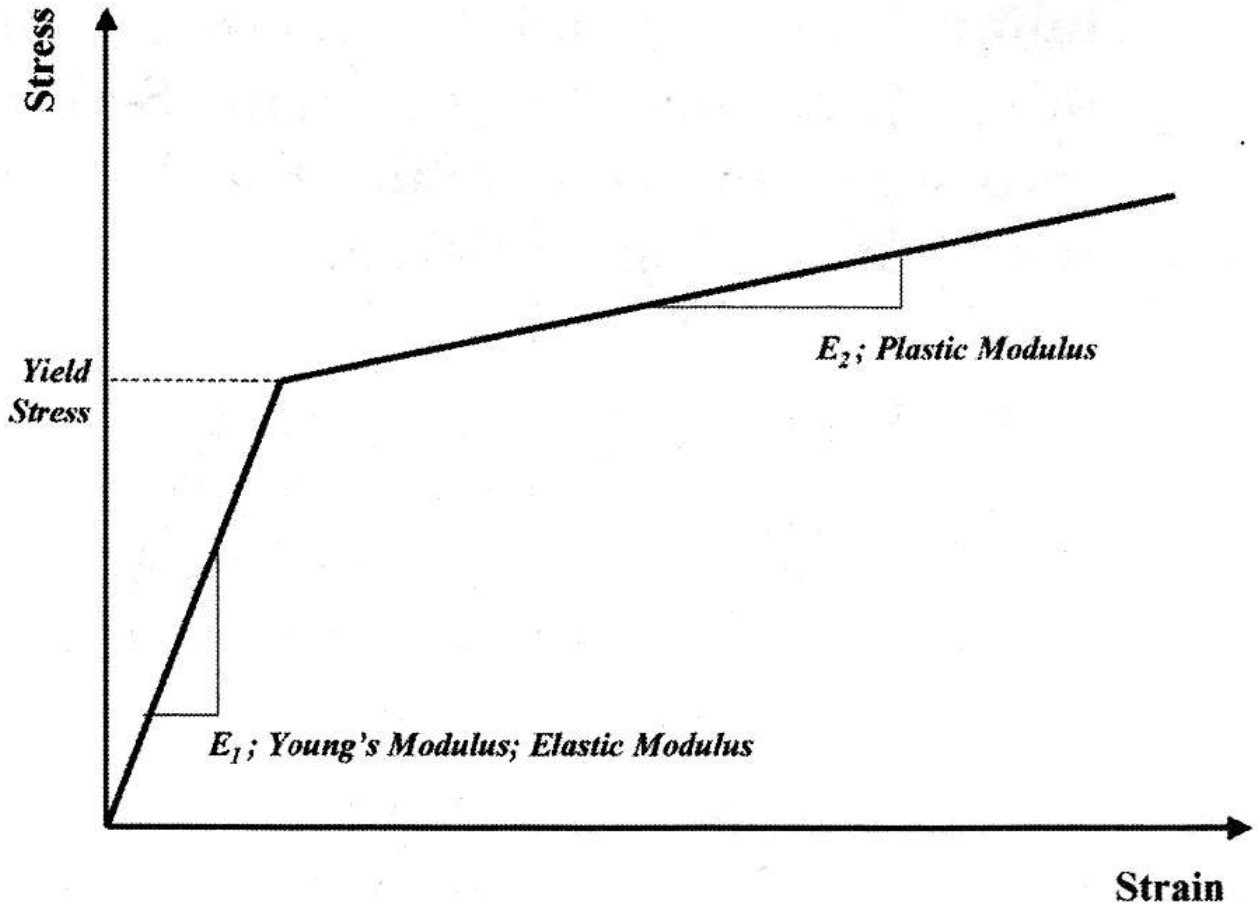
How to Get Action-Deformation Curves

- By actual measurements
  - Apply load, measure deflection
  - Apply load, measure stress and strain
- By computations
  - Use material models, cross-section dimensions to get Moment-Curvature Curves
- By combination of measurement and computations
  - Calibrate computation models with actual measurements
  - Some parameters obtained by measurement and some by computations

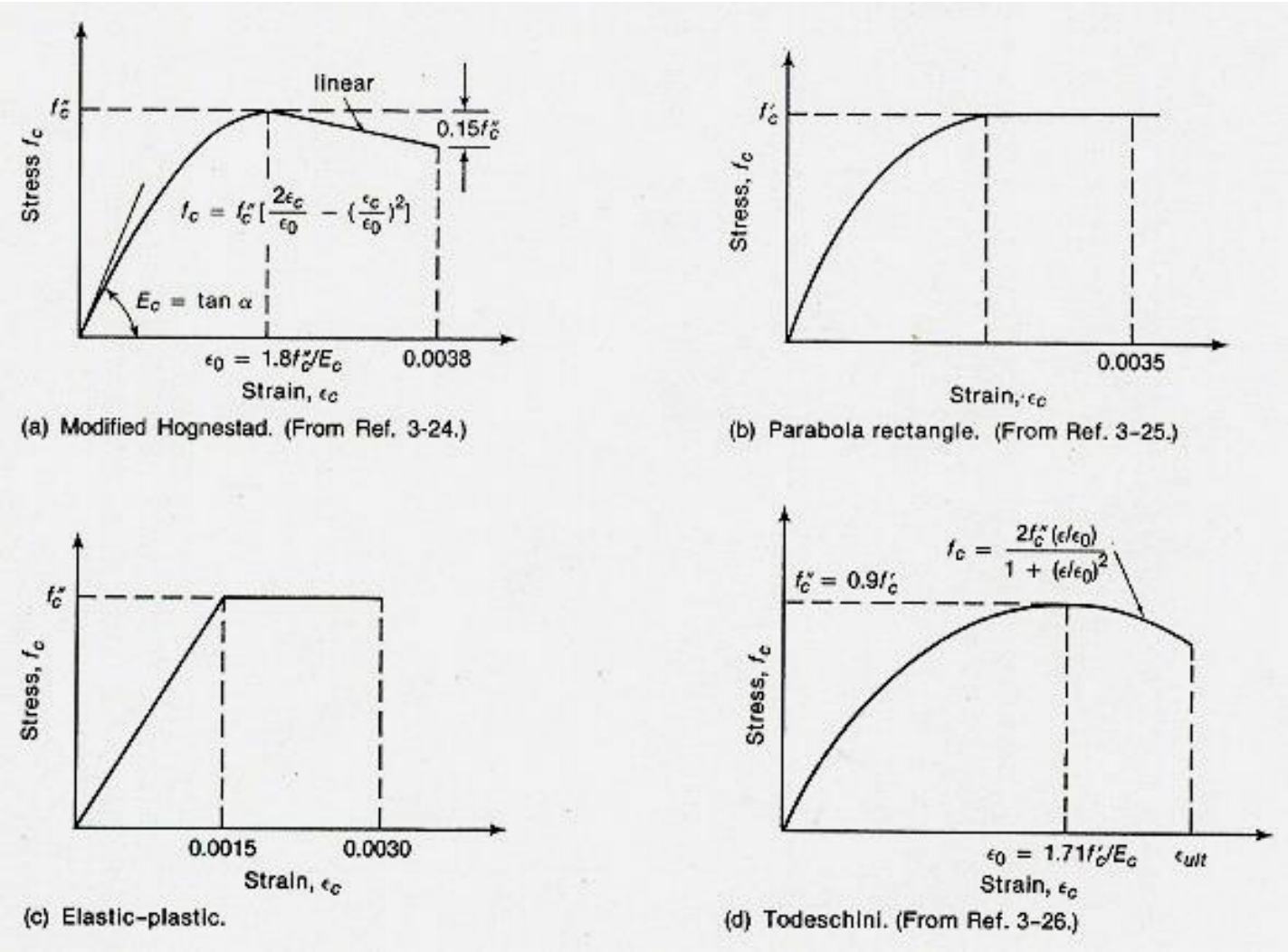
Modeling Nonlinearity in Materials

Basic of Material Model

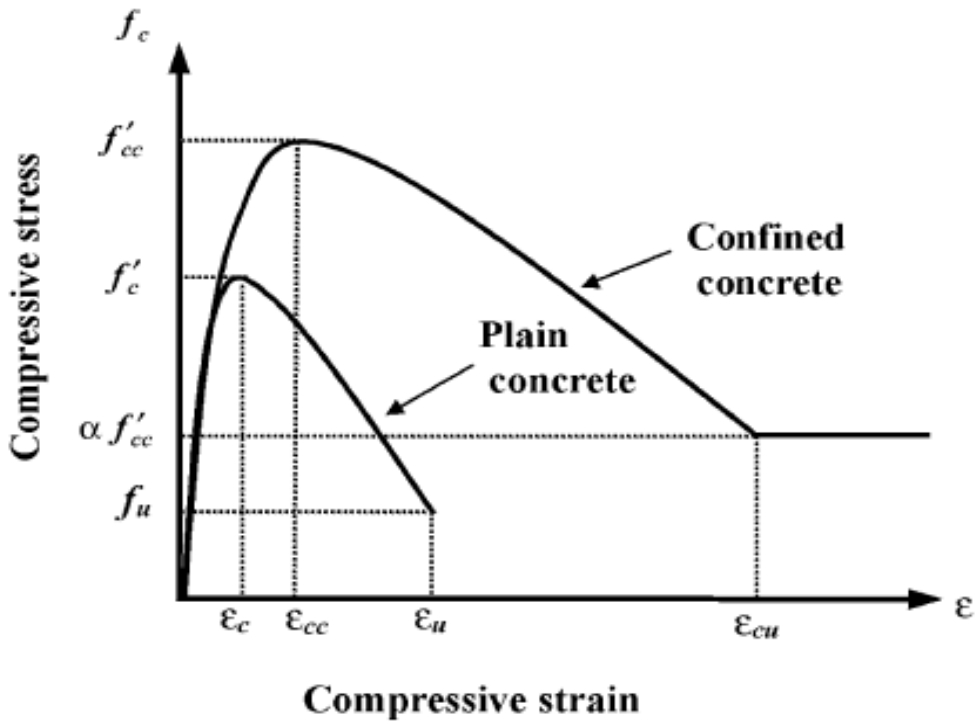
- Bilinear material model



Various Models



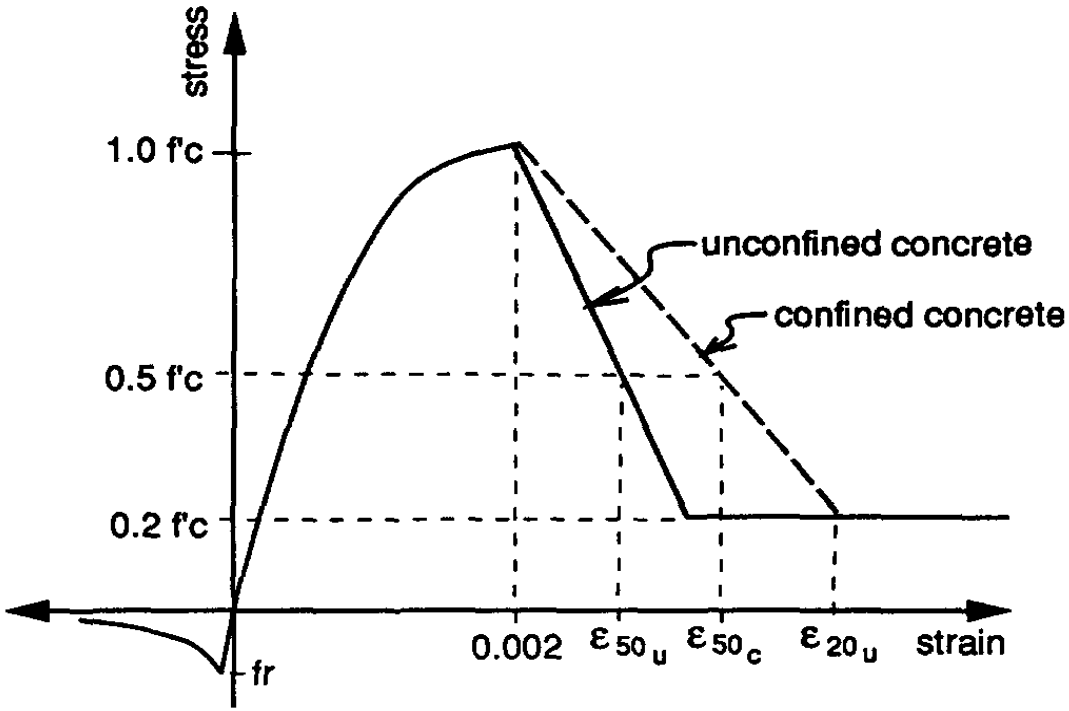
Stress Strain Relationship – Confined Concrete



Concrete: General stress-strain curves for confined and unconfined concrete

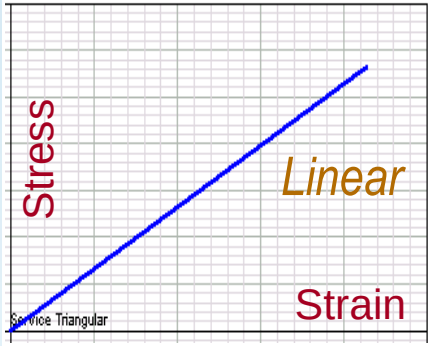
Material Model for Concrete

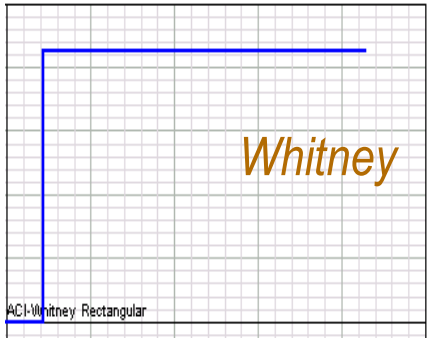
- With RSB and structural steel

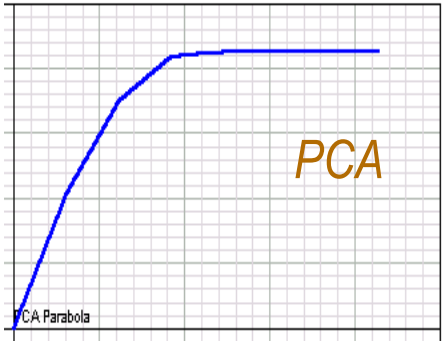


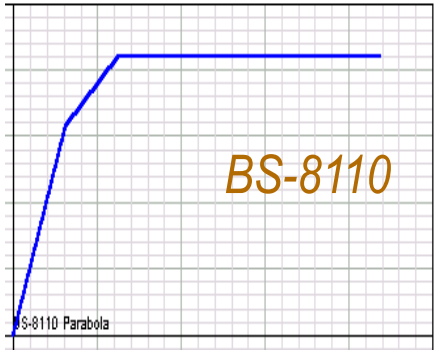
Source: El-Tawil et al. (1994)

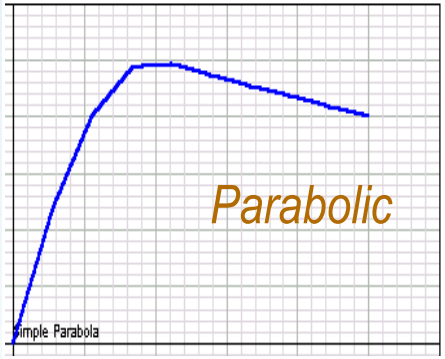
Unifying Material Models

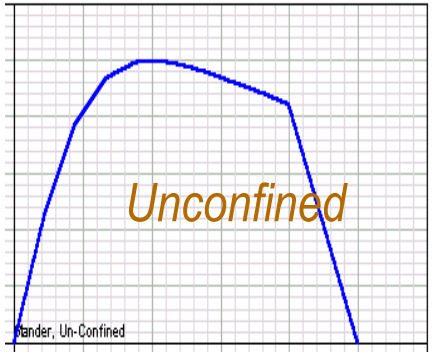


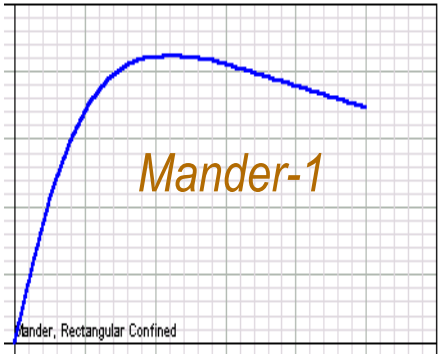


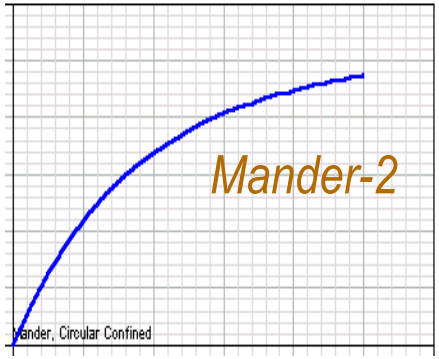






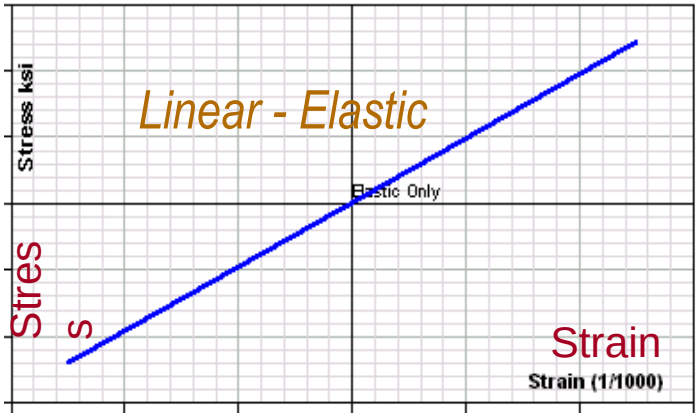


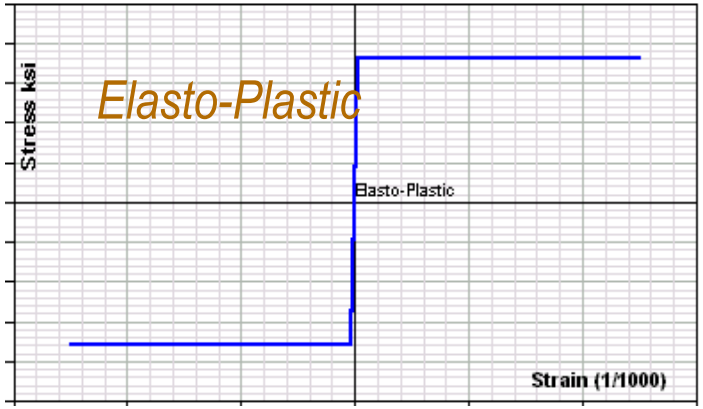


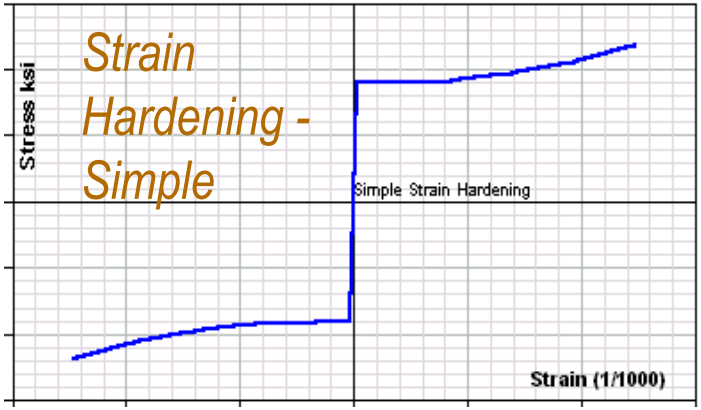


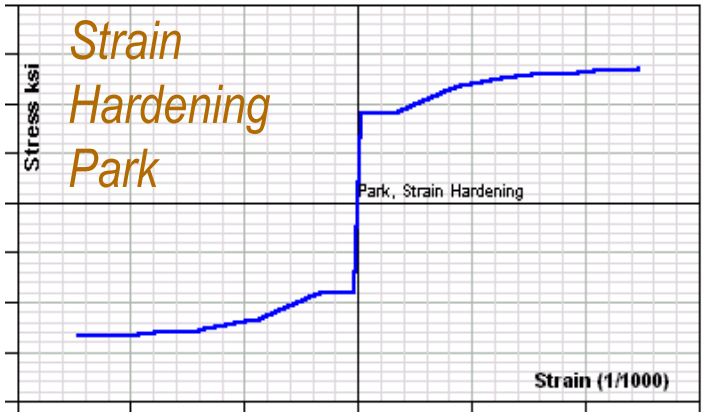
Concrete Stress-Strain Relationships

Unifying Material Models



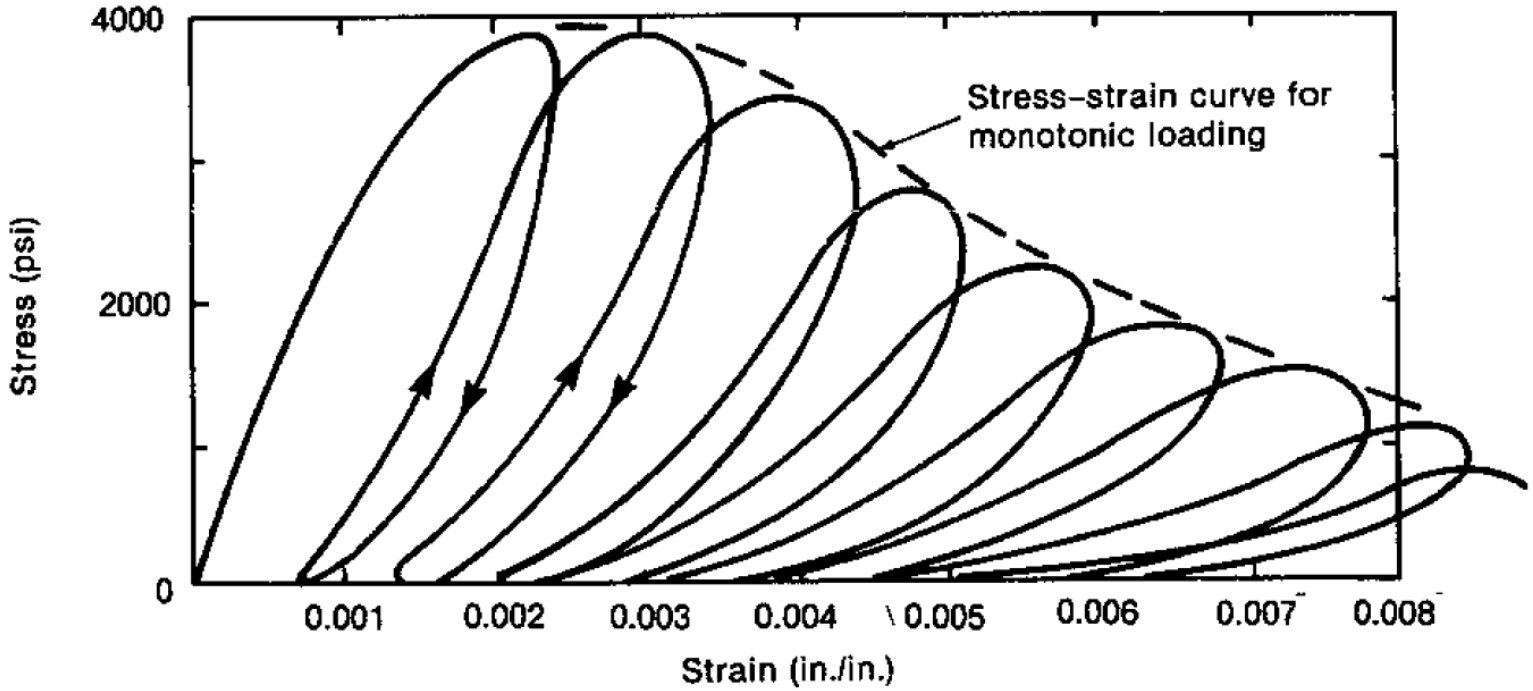






Steel Stress-Strain Relationships

Stress Strain Relationship - Cyclic Load



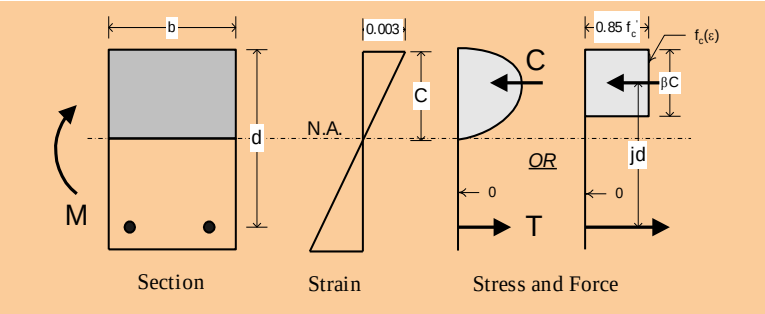
Reference: James G. Macgregor
Reinforced Concrete: Mechanics and Design, 3rd Edition

Modeling Nonlinearity In Sections

Flexural Theory: Stress Resultants

The Most Simple Case

$$M_n = \phi \quad f_y A_{st}\left(d - \frac{a}{2}\right)$$

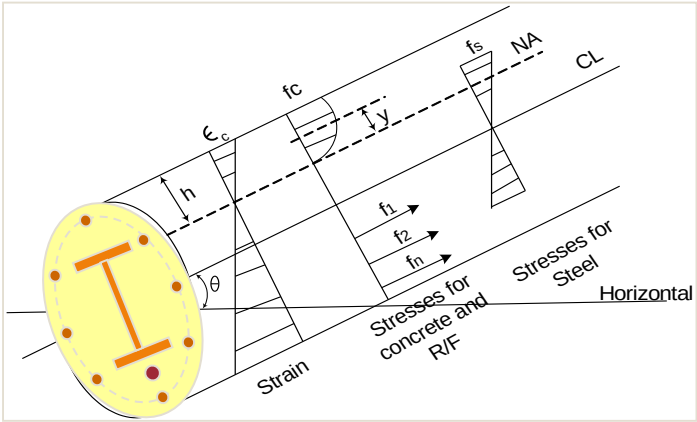


The Most Comprehensive Case

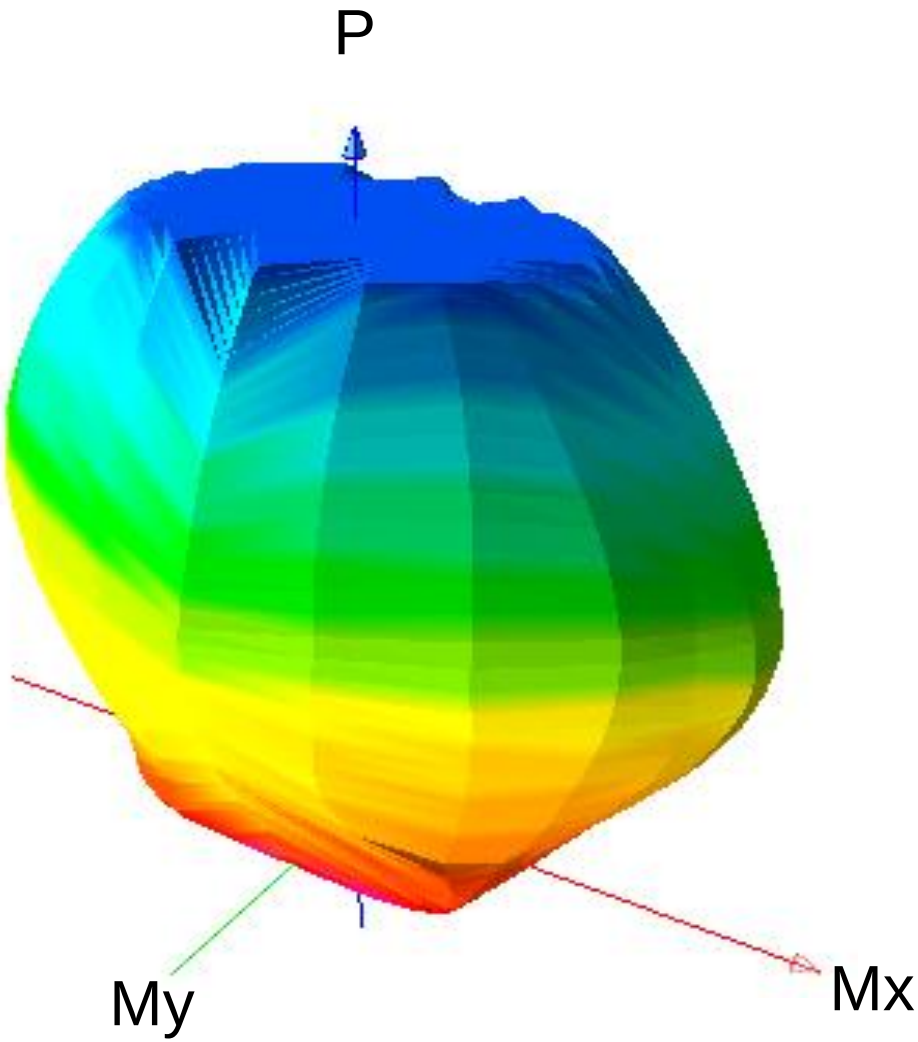
$$N_z = \phi_1\left[\frac{1}{\gamma_1}\int_x\int_y \sigma(x,y)\,dx\,dy \ldots + \frac{1}{\gamma_2}\sum_{i=1}^{n} A_i\sigma_i(x,y) \ldots\right]$$

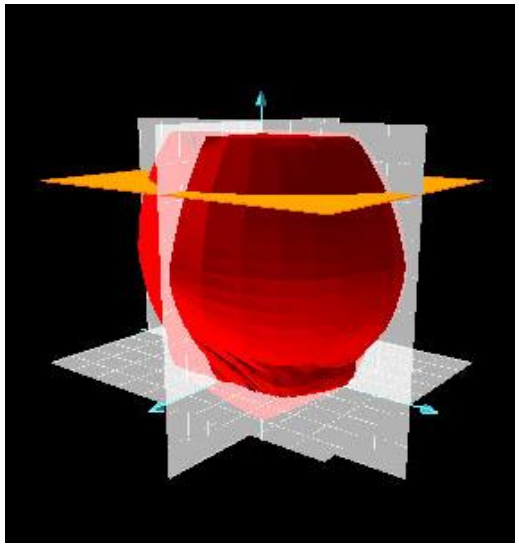
$$M_x = \phi_2\left[\frac{1}{\gamma_1}\int_x\int_y \sigma(x,y)\,dx\,dy.y \ldots + \frac{1}{\gamma_2}\sum_{i=1}^{n} A_i\sigma_i(x,y)\ y_i \ldots\right]$$

$$M_y = \phi_3\left[\frac{1}{\gamma_1}\int_x\int_y \sigma(x,y)\,dx\,dy.x \ldots + \frac{1}{\gamma_2}\sum_{i=1}^{n} A_i\sigma_i(x,y)\ x_i \ldots\right]$$

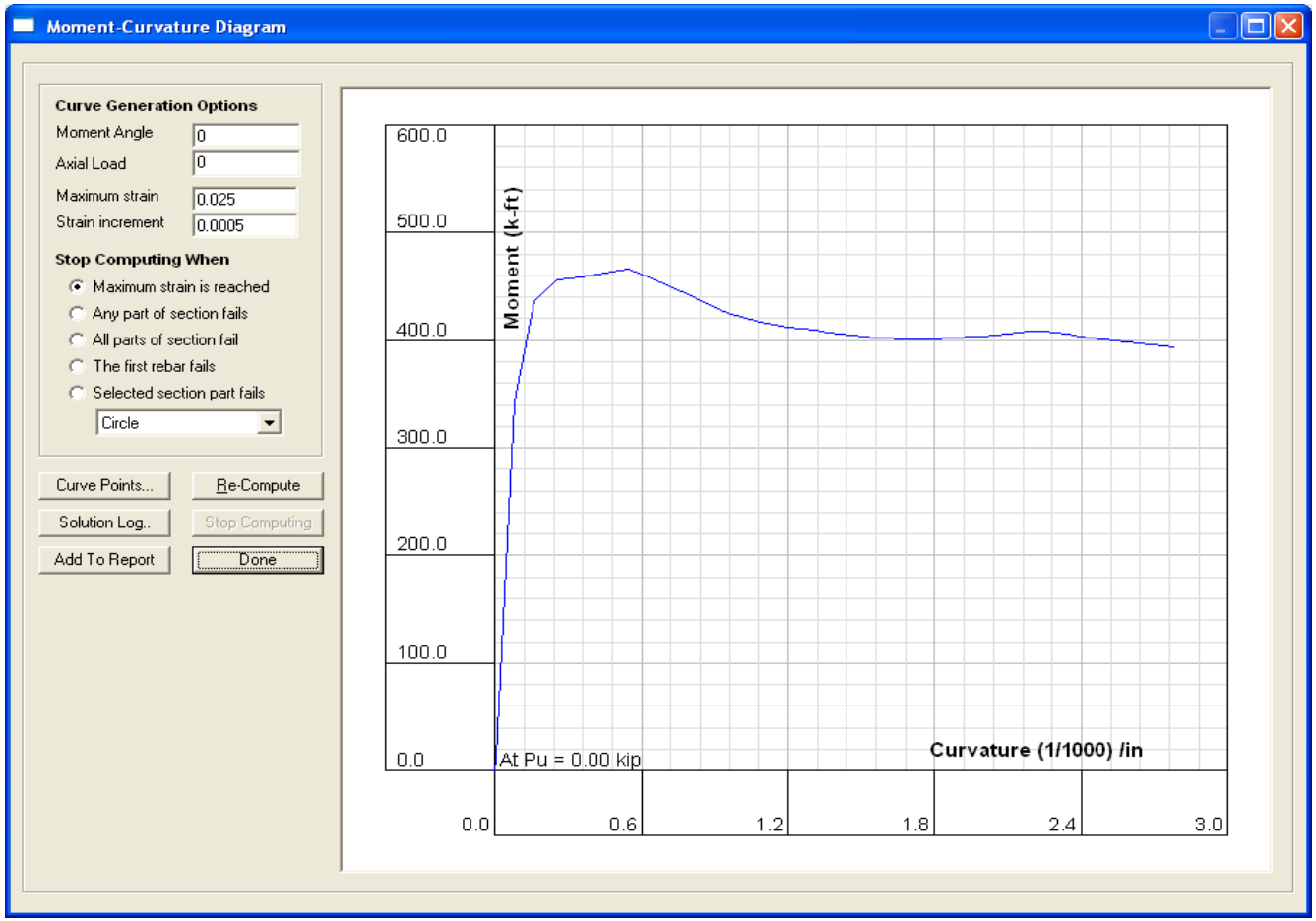


Capacity Interaction Surface



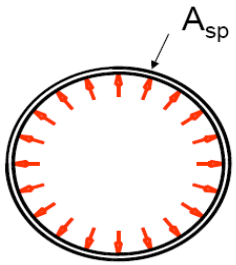


The Moment Curvature Curve

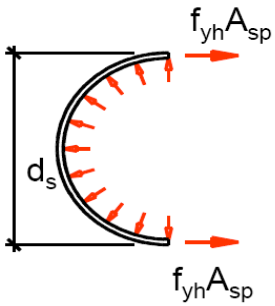


The Moment-Curvature Curve

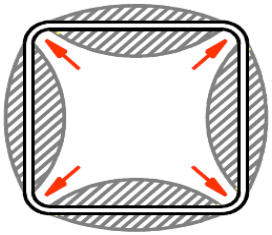
- Probably the most important action-deformation curve for beams, columns, shear walls and consequently for building structures
- Significant information can be obtained from Moment Curvature Curve to compute:
  - Yield Point
  - Failure Point
  - Ductility
  - Stiffness
  - Crack Width
  - Rotation
  - Deflection
  - Strain



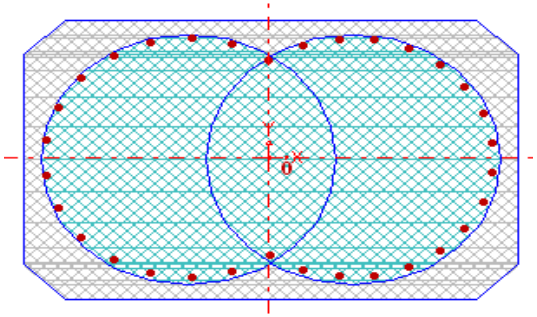
Confinement from spiral or circular hoop

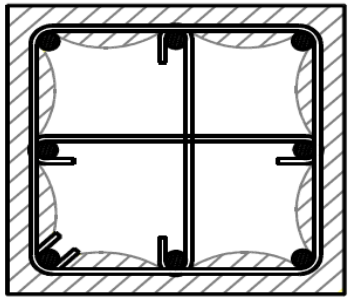


Forces acting on 1/2 spiral or circular hoop

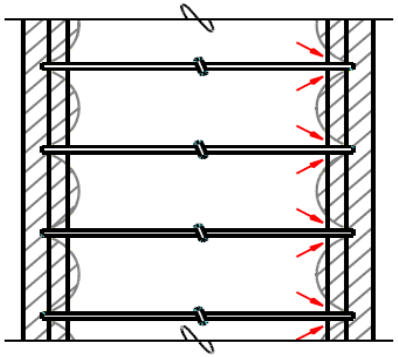


Confinement from square hoop

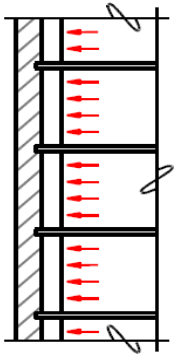




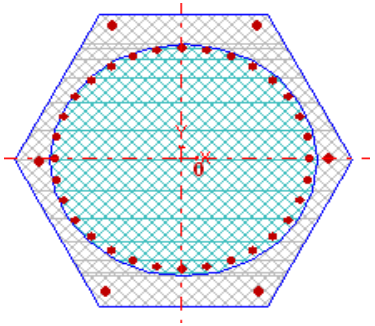
Rectangular hoops with cross ties



Confinement by transverse bars

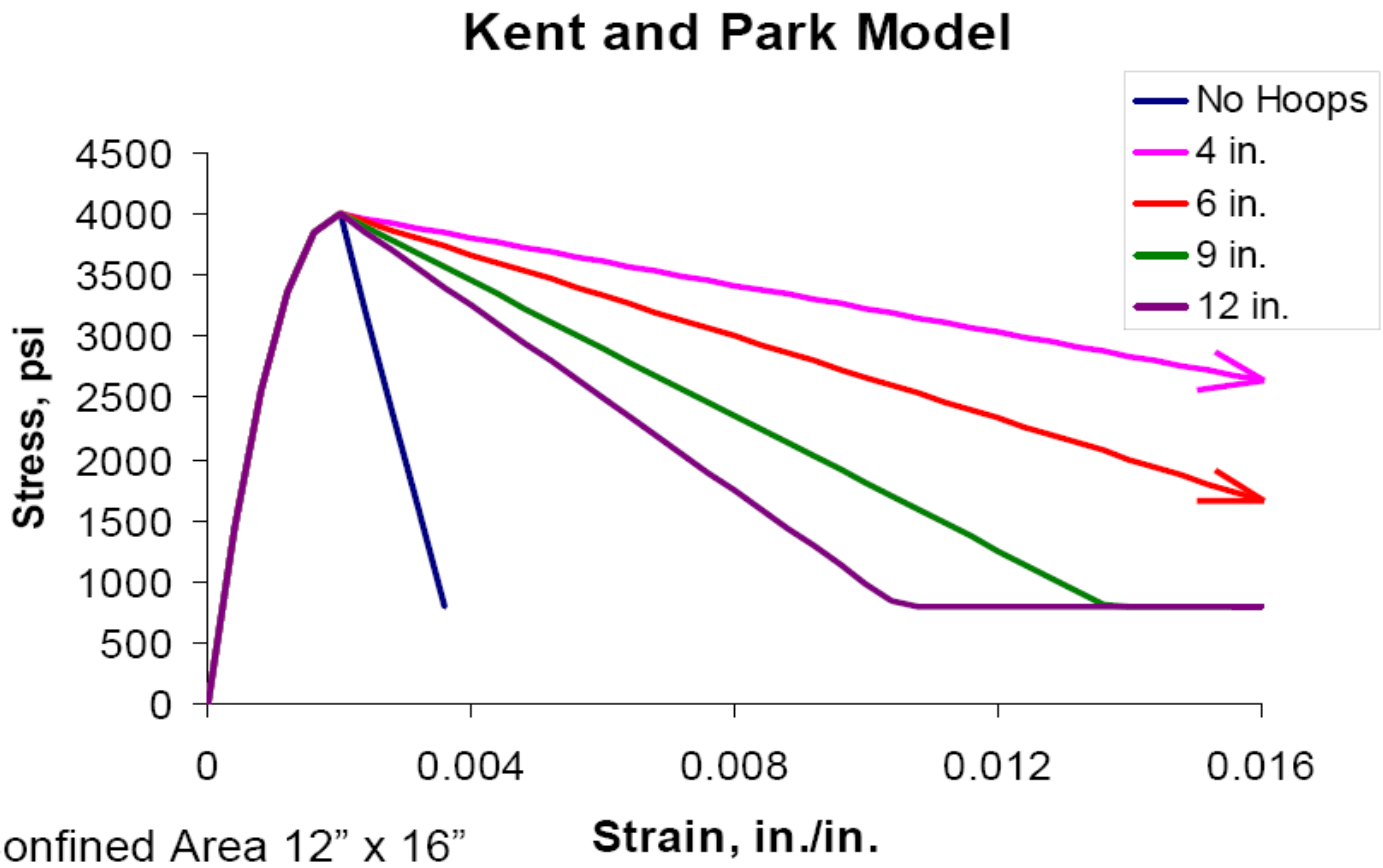


Confinement by longitudinal bars



Confinements

Concrete Behavior and Confinement

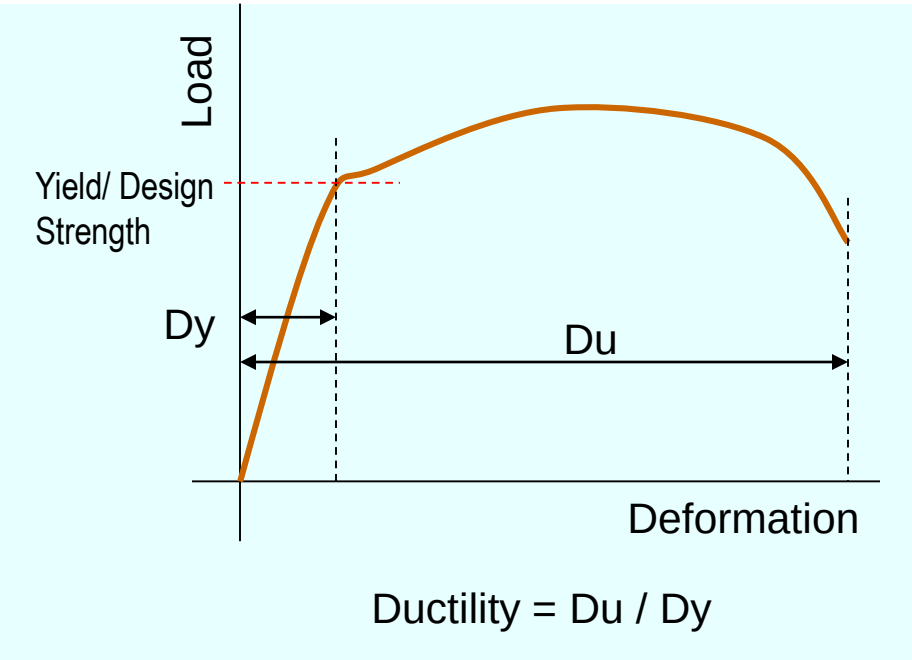


Confined Area 12" x 16"

Idealized Stress-Strain Behavior of Confined Concrete

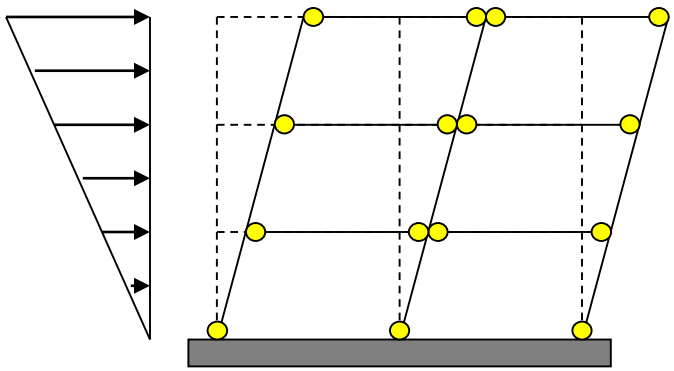
Ductility – Definition and Usage

- Ductility can be defined as the "ratio of deformation and a given stage to the maximum deformation capacity"
- Normally ductility is measured from the deformation at design strength to the maximum deformation at failure

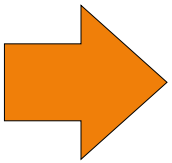


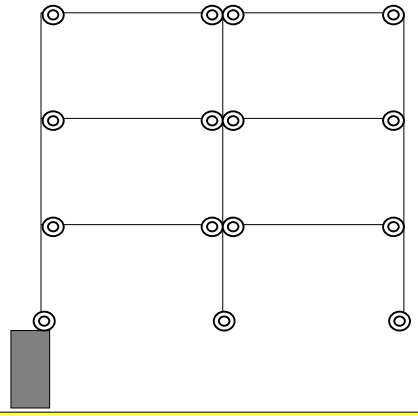
Modeling for Of Members

Modeling of Seismic Resistant Structures



RC Frame Subjected to EQ Motions





Lumped Plasticity Model

location of plastic hinge

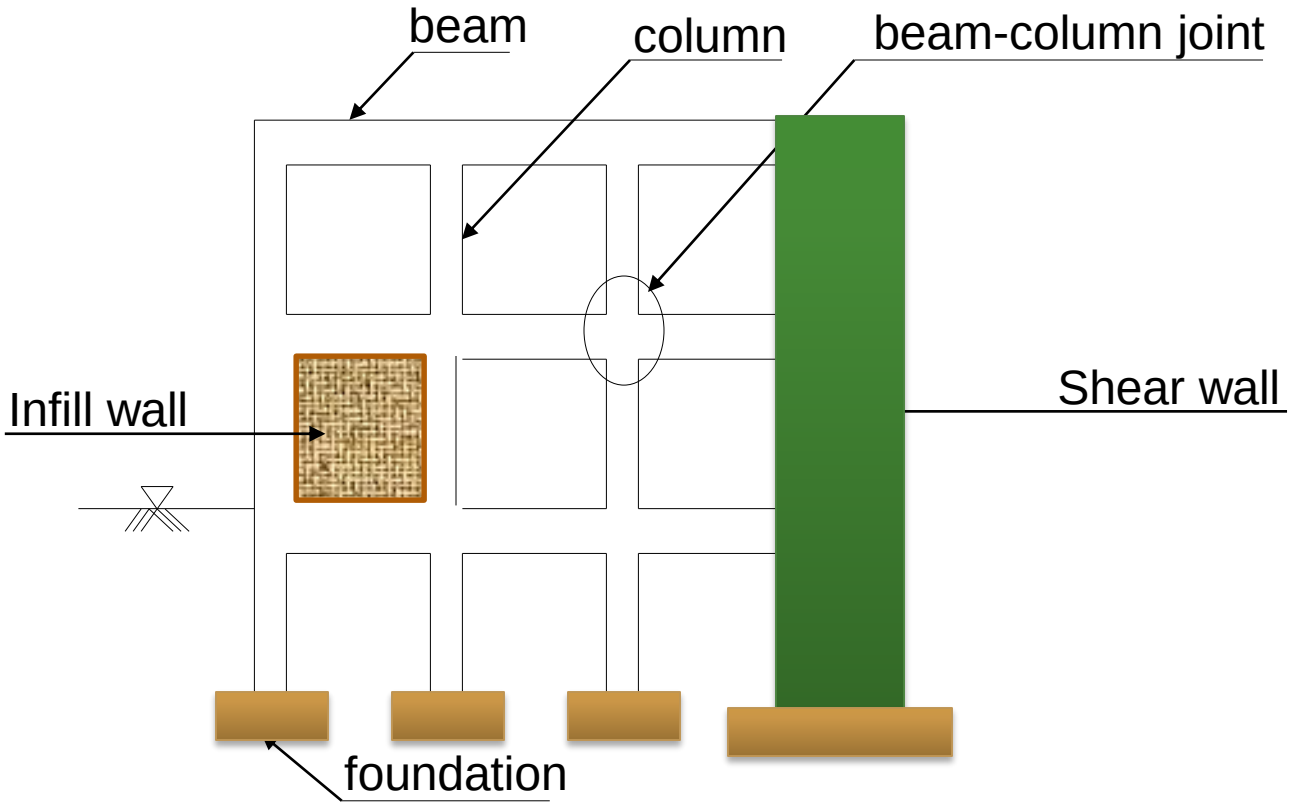
elastic frame member

plastic hinge spring

(nonlinear rotational spring)

elastic frame element

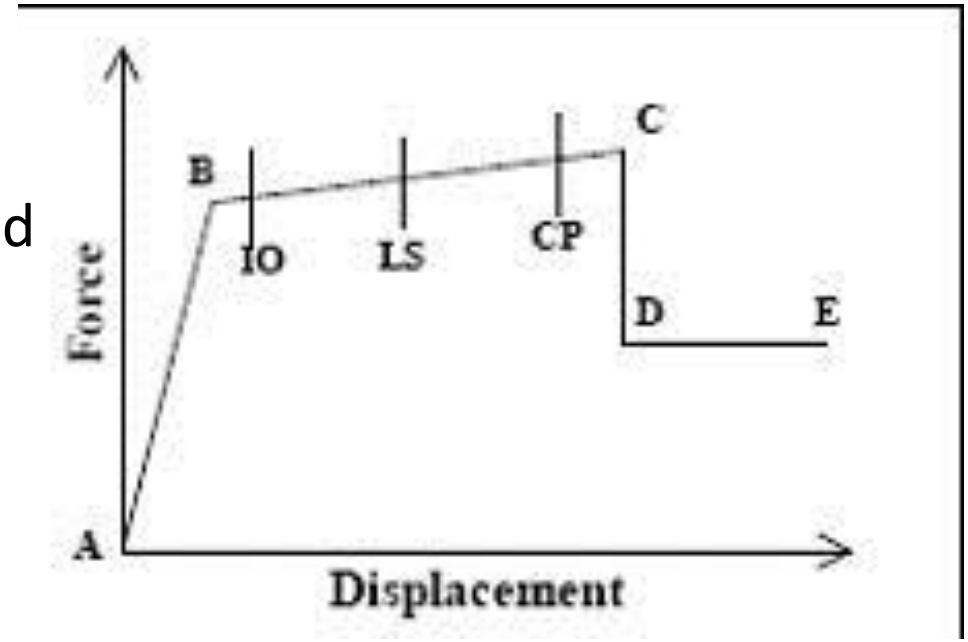
Nonlinear Model for Building Components



$$\underline{M}\,\underline{\ddot{u}} + \underline{C}\,\underline{\dot{u}} + \underline{K}\,\underline{u} = -\underline{M}(\underline{r}\,\ddot{u}_g)$$

Hinge Properties

- Five points labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge
- Three points labeled IO, LS and CP are used to define the acceptance criteria for the hinge
- IO- Immediate Occupancy
- LS- Life Safety
- CP-Collapse Prevention

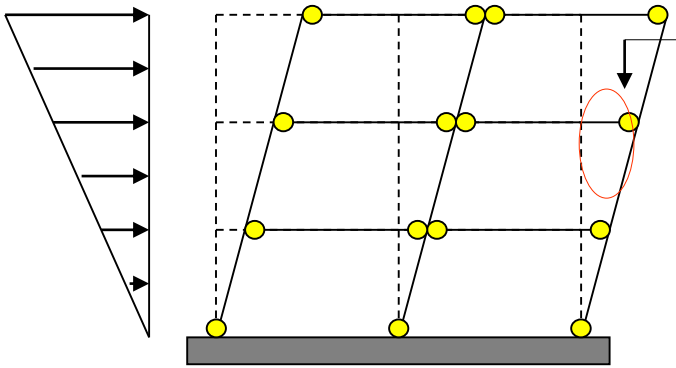


Hinge Properties

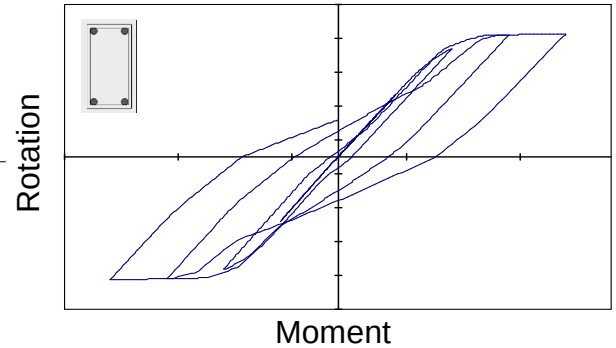
- Point A is always the origin
- Point B represents yielding. No deformation occurs in the hinge up to point B, regardless of the deformation value specified for point B. The displacement (rotation) at point B will be subtracted from the deformations at points C, D, and E. Only the plastic deformation beyond point B will be exhibited by the hinge
- Point C represents the ultimate capacity for Pushover analysis
- Point D represents a residual strength for Pushover analysis
- Point E represents total failure. Beyond point E the hinge will drop load down to point F (not shown) directly below point E on the horizontal axis. To prevent this failure in the hinge, specify a large value for the deformation at point E

Plastic Hinges

- Seismic energy dissipation mechanism of well-designed cast-in-place RC frames is usually relied on plastic flexural-deformation of beams and 1-st story columns (Plastic Hinge, PH).



Energy Dissipation Mechanism

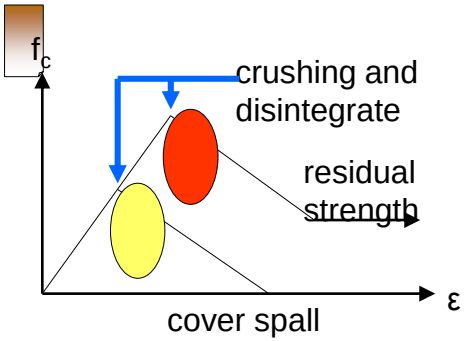


Plastic Hinge in Beams

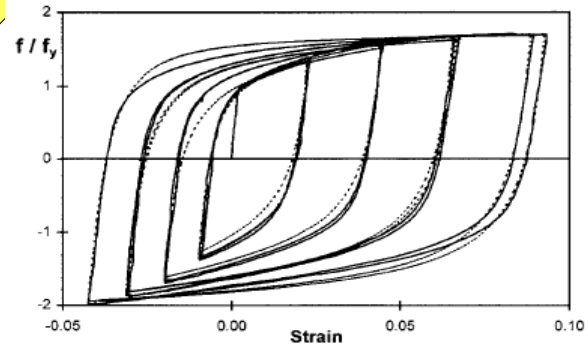
1) reliable predicted behavior
2) most ductile plastic deformation
3) highest equivalent damping

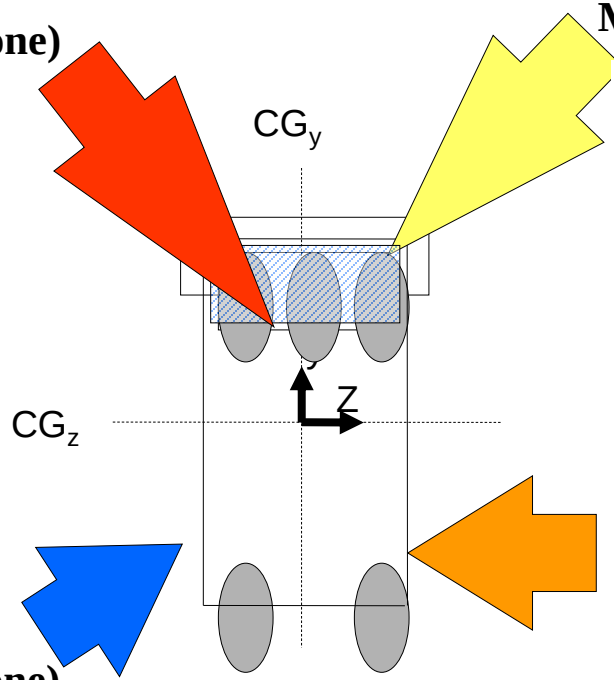
Plastic Hinges – Material Functions

Core Concrete (Confined Zone)



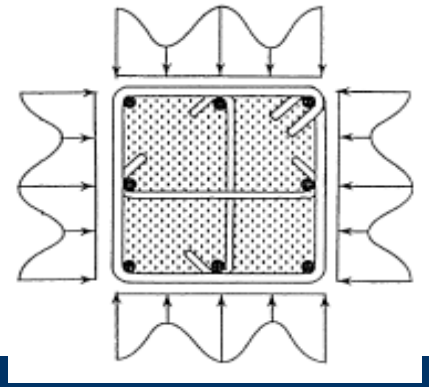
MAIN STEEL= energy dissipater





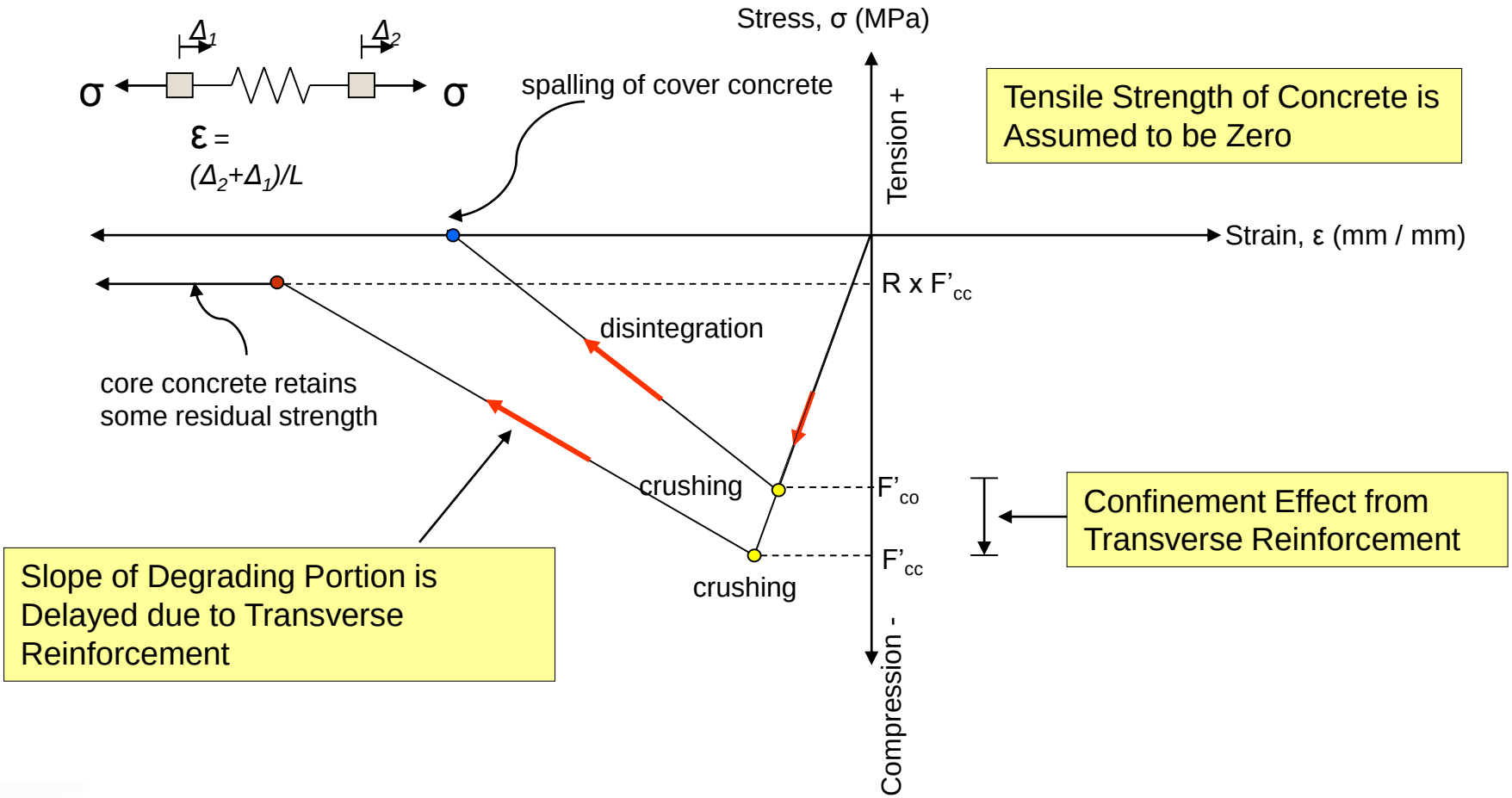
BEAM CROSS SECTION

STIRRUP = confinement + shear capacity

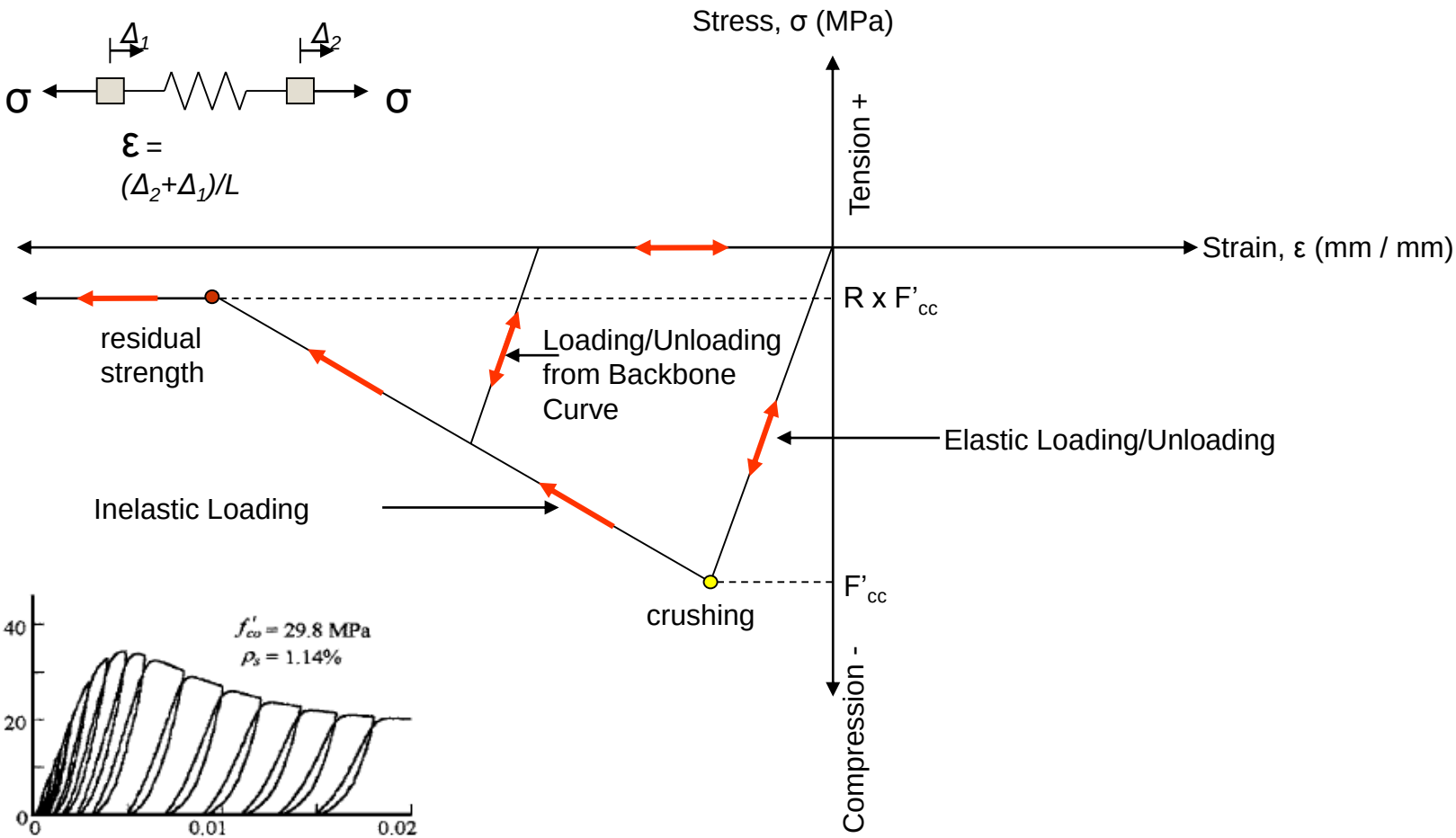


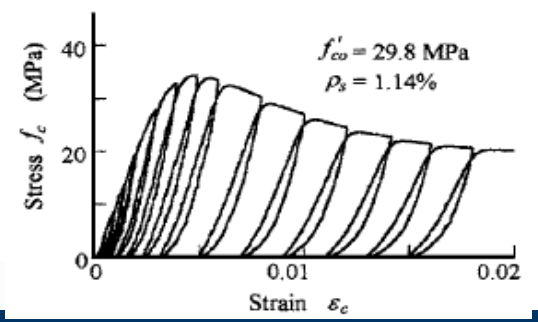
Cover Concrete (Unconfined Zone)

Core/Cover Concrete – Monotonic Response

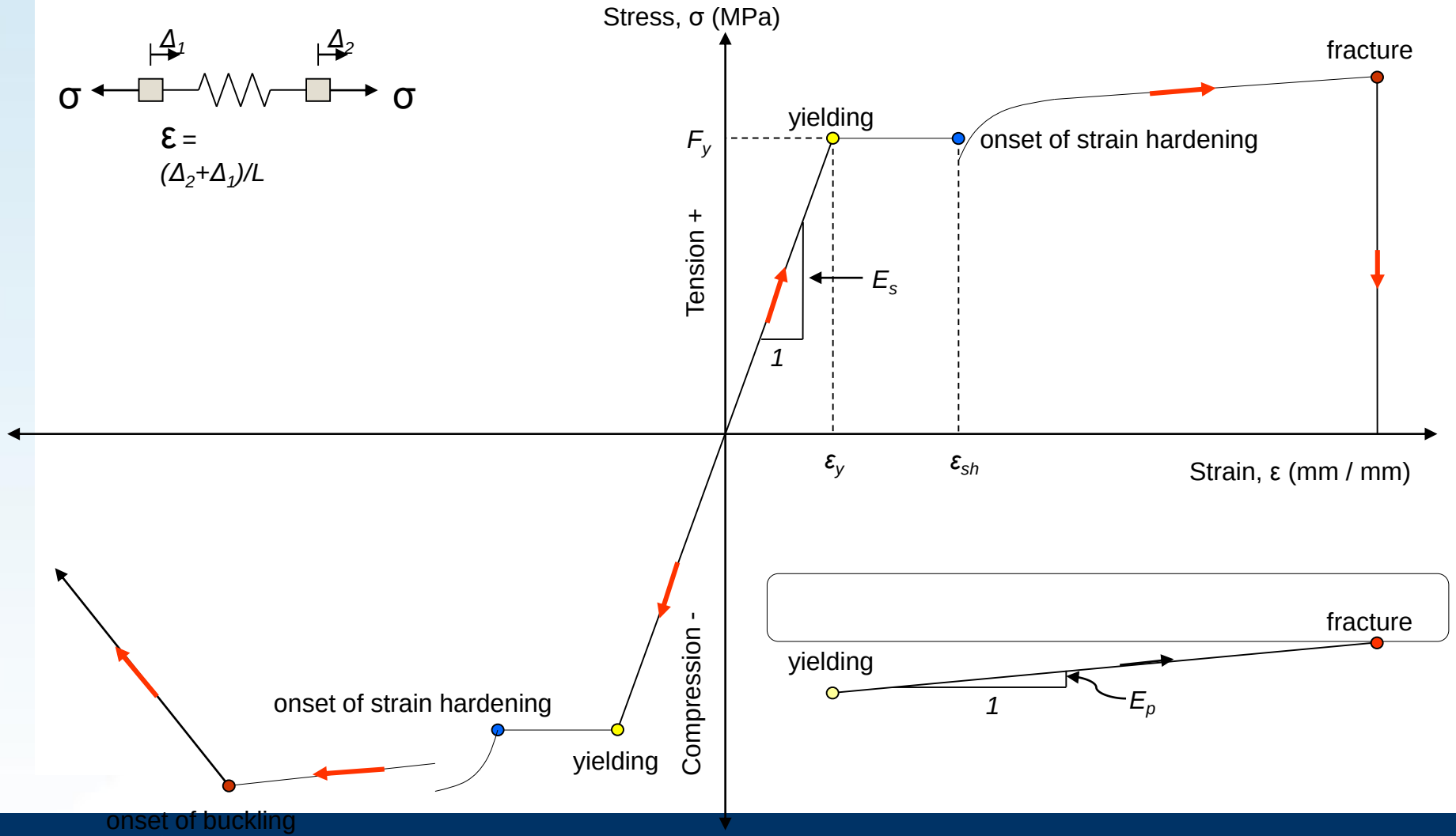


Core/Cover Concrete – Hysteretic Response

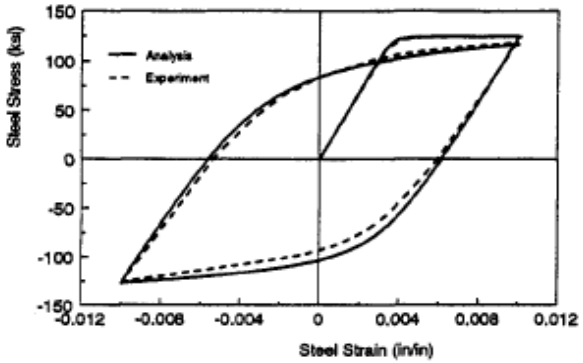


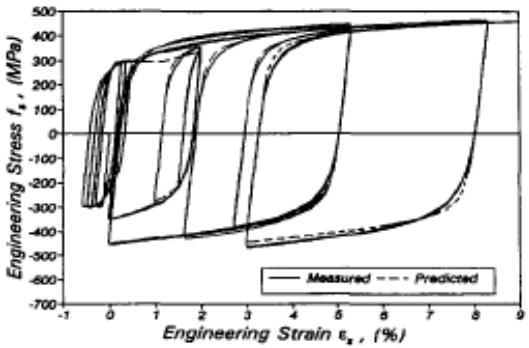


Steel Reinforcement – Monotonic Response



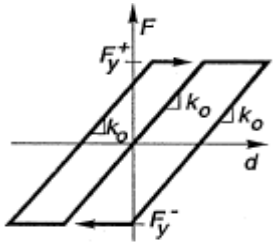
Steel Reinforcement – Hysteretic Response



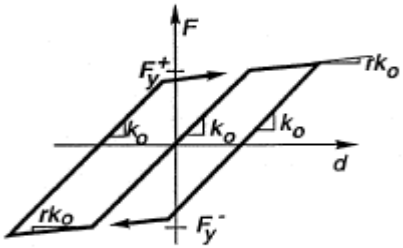


Hysteretic response of reinforcing steel

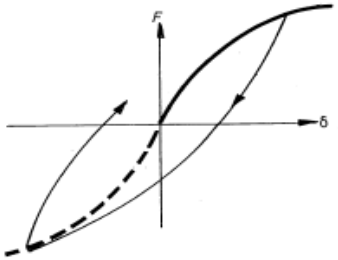
[L.L. Dodd and J.I. Restrepo-Posada, 1995]



EPP-Model

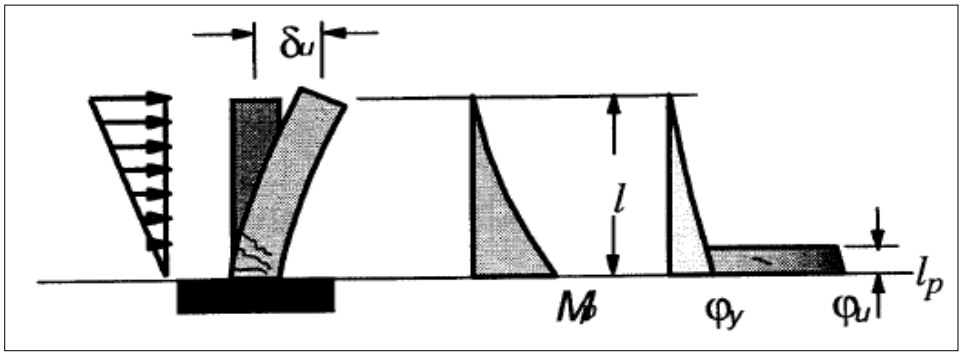


Bilinear Model (Clough)



Ramberg-Osgood Model

Equivalent Plastic Hinge Length, L_{PH}



Concept of "Equivalent Plastic Hinge Length, L_{PH}"

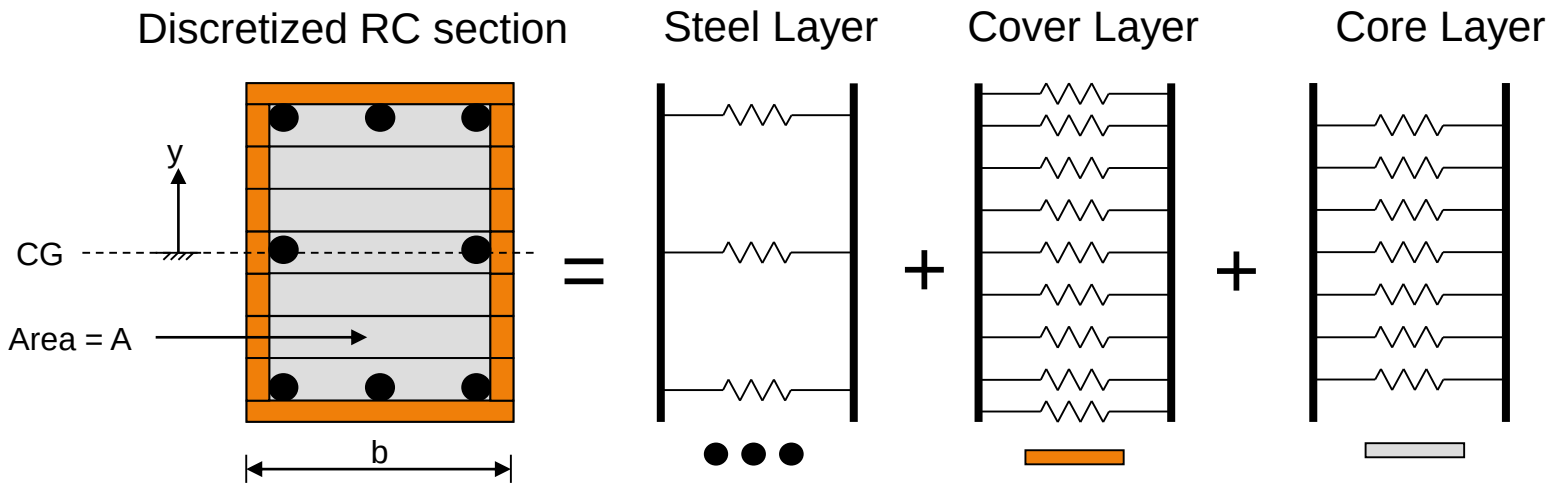
$$\theta_{PH} = \int_{PH-length} \phi(x)dx \approx \phi_{\max} L_{PH}$$

Empirical equations for L_{ph} can be found in the literature

$$L_{PH} = 0.08L + 0.022 d_b F_y$$

[Paulay and Priestley, 1992]

Zero-Length Fiber Section Model



- An RC section can be represented by sub-divided layers (fibers). Each layers is modeled using uniaxial nonlinear springs which, in turn, classified into 3 groups according to their material hysteretic response, i.e., steel springs, cover-concrete springs, and core-concrete springs.
- Theoretical formulation of the fiber section model can be explained through the following equations.

$$N = \int f_s(\varepsilon) b dy$$

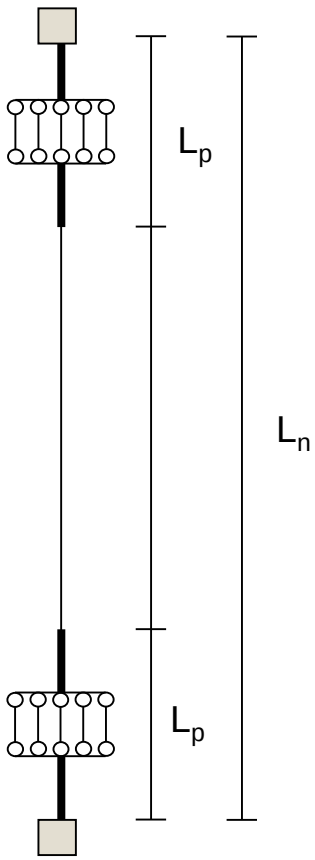
$$N \approx \sum_{i=1}^{n} (f_s(\varepsilon) A_{fiber})_i$$

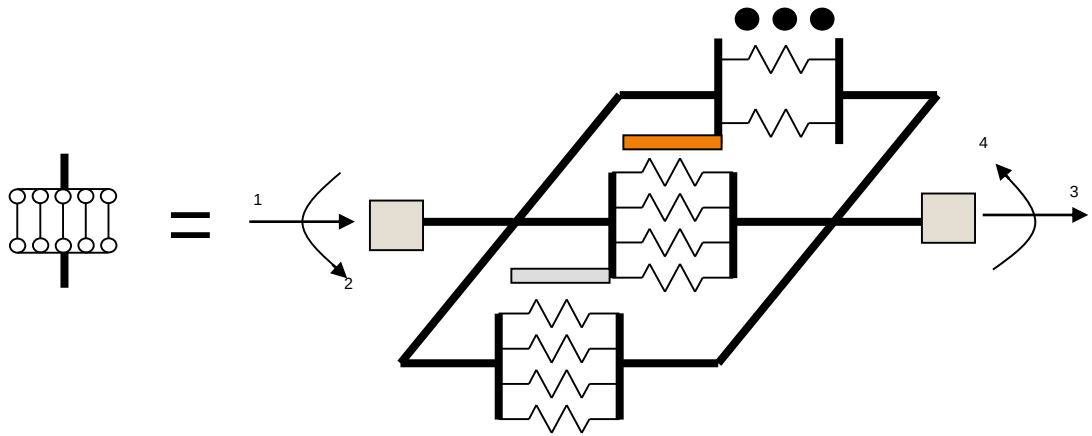
$$M = \int f_s(\varepsilon) y b dy$$

$$M \approx \sum_{i=1}^{n} (f_s(\varepsilon) y_{cg} A_{fiber})_i$$

$$K_{spring} = \frac{EA}{L} = \frac{E_t A_{fiber}}{L_{PH}}$$

Zero-Length Fiber Section Model

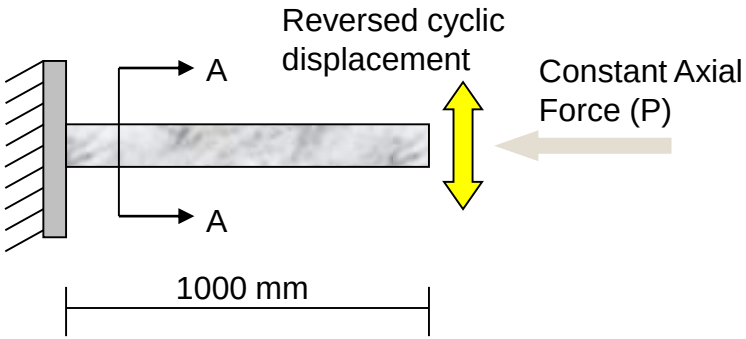


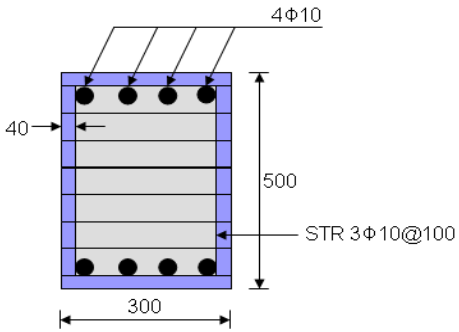


= uniaxial nonlinear spring

- Axial stiffness of each spring is defined by area of fiber, equivalent plastic hinge length, and tangent stiffness of the corresponding material
- To define the tangent stiffness, material hysteretic model as discussed earlier can be directly assign to these springs

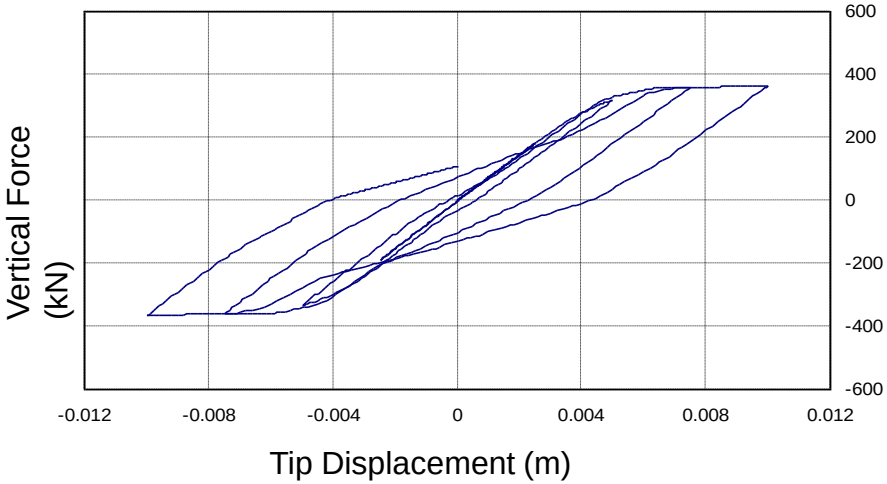
Analytical Plastic Hinge Response



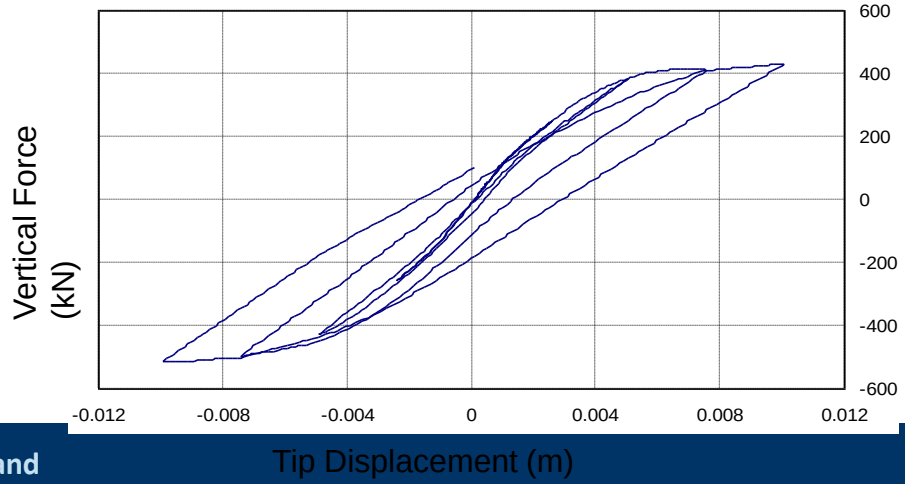


SECTION A-A
(dimension in mm)

CASE A – [P = 0 kN]

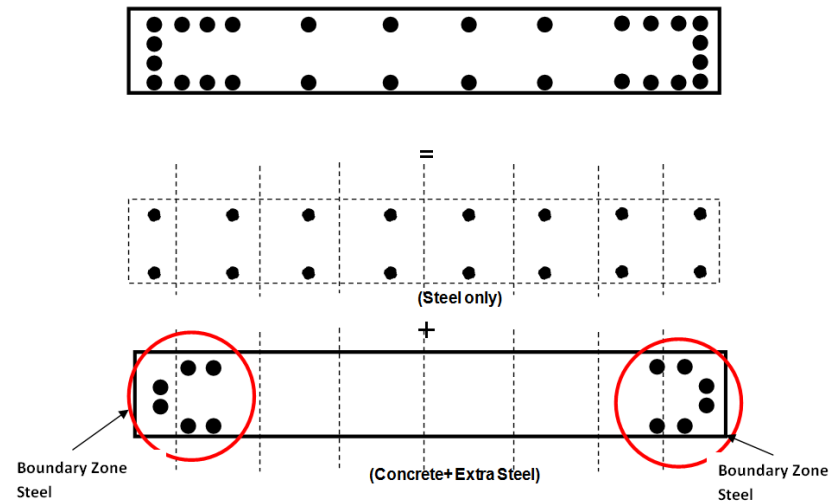


CASE B – [P = 100 kN]



Modeling of Shear Walls

- Fiber modeling
  - Two parallel fiber sections are used
  - Shear behavior is modeled as elastic
- Nonlinear Shell Element
  - 7 layer NL shell with explicit cover, mid portion, vertical and horizontal bars



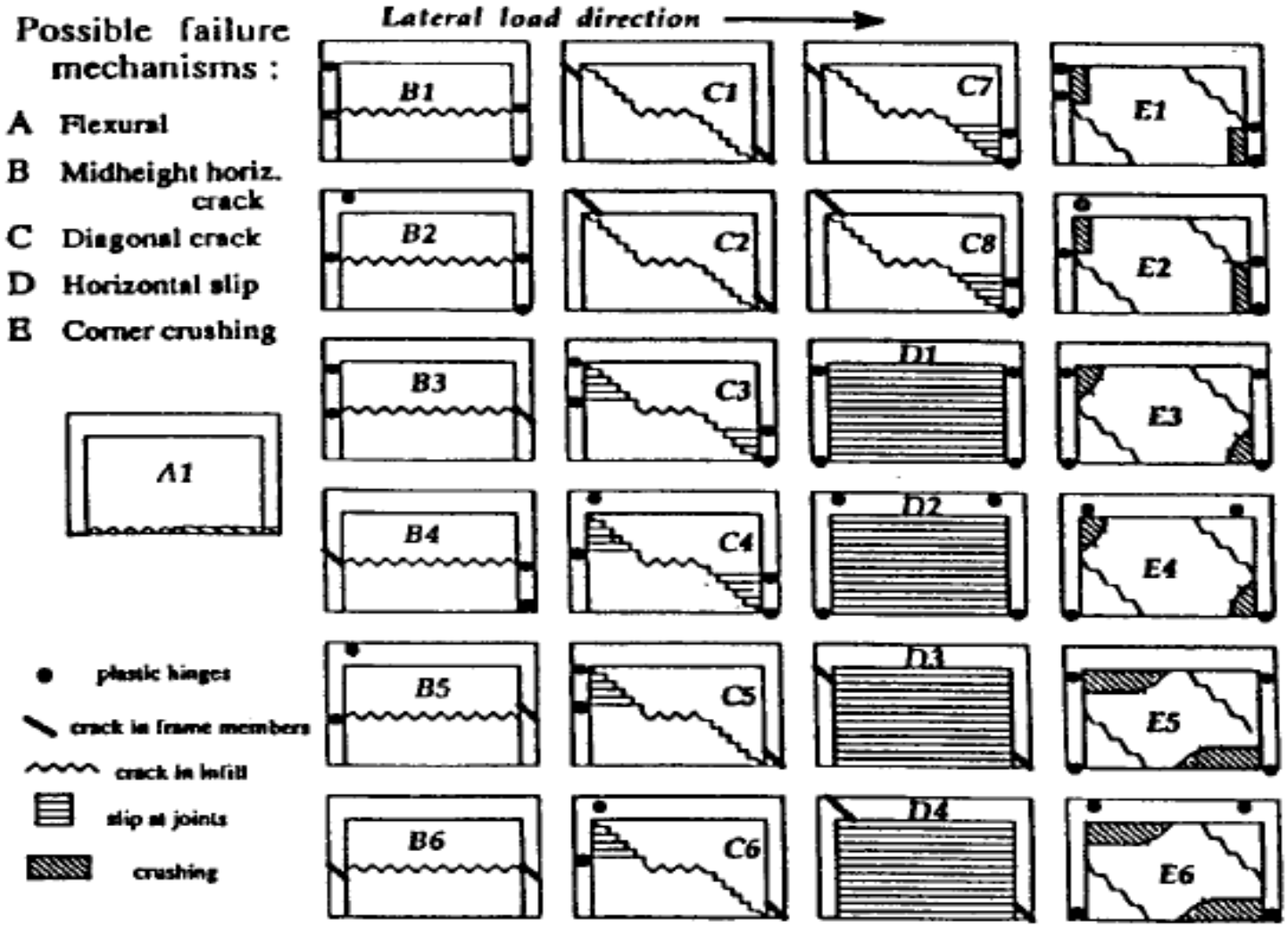
Masonry Infill Walls

- Masonry infill walls are typically used in reinforced concrete buildings and are considered by engineers as nonstructural components
- Even if they are relatively weak when compared with structural components, they can drastically alter the response of structure.
- The presence of masonry infill walls can modify lateral stiffness, strength, and ductility of structure

Masonry Infill Walls

- At low level of in-plane lateral force, the frame and infill panel will act in a fully composite fashion, as a structural wall with boundary elements
- When lateral deformations increase, frame attempts to deform in a flexural mode. But, infill panel attempts to deform in a shear mode.
- These lead to separation between frame and panel at the corners on the tension diagonal, and the development of a diagonal compression strut on the compression diagonal.

Failure mechanisms of infill frames



Modeling of Masonry Infill Walls

- Masonry infill walls are modeled using equivalent strut concept based on recommendations of FEMA-273 (1997)
- Based on this concept, the stiffness contribution of infill wall is represented by an equivalent diagonal compression strut

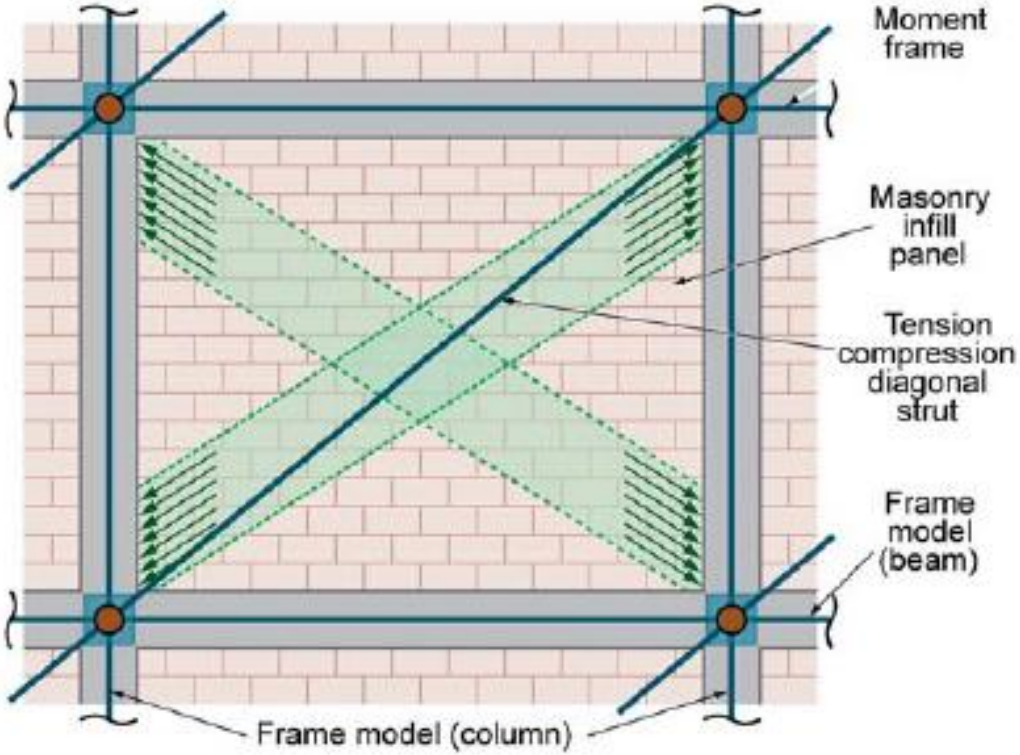
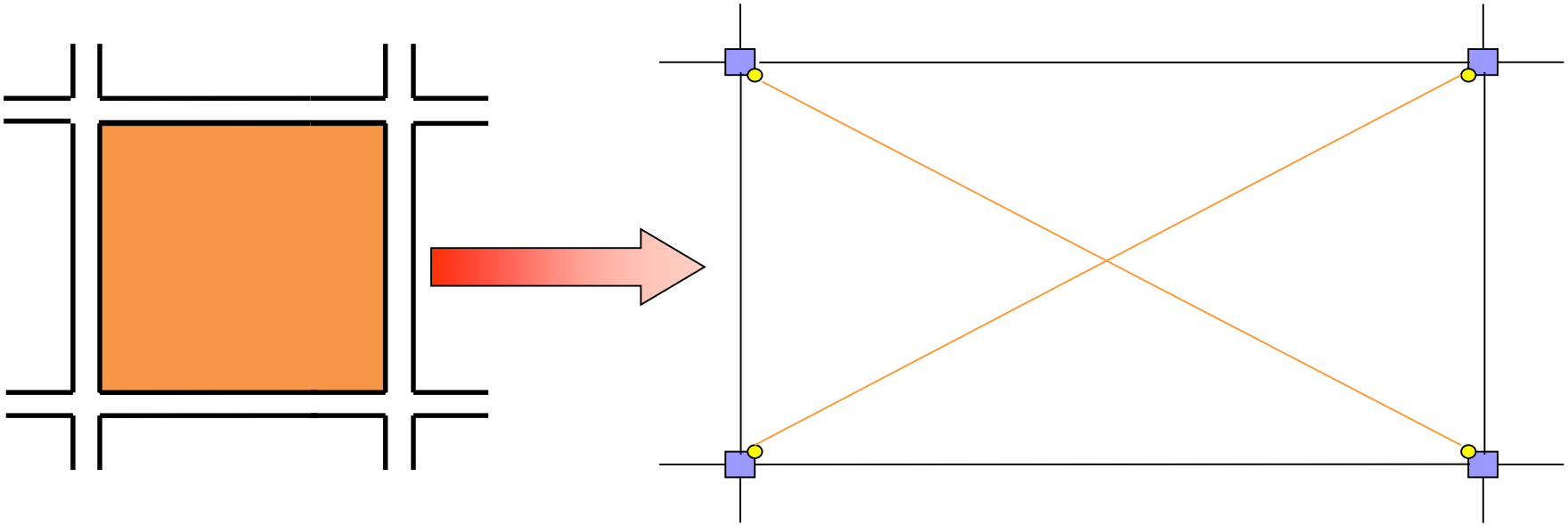


Figure 3-3 – Masonry infill wall panel and model.

NEHRP Seismic Design Technical Brief No. 4

Equivalent Diagonal Compression Strut



Modeling of Masonry Infill Walls

- Thickness and modulus of elasticity of strut are assumed to be the same as those of infill wall.
- Width of equivalent strut, a, is determined by using Equations was suggested by FEMA-273

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf}$$

$$\lambda_1 = \left[\frac{E_m t_{\inf} \sin 2\theta}{4 E_c I_c h_{\inf}} \right]^{\frac{1}{4}}$$

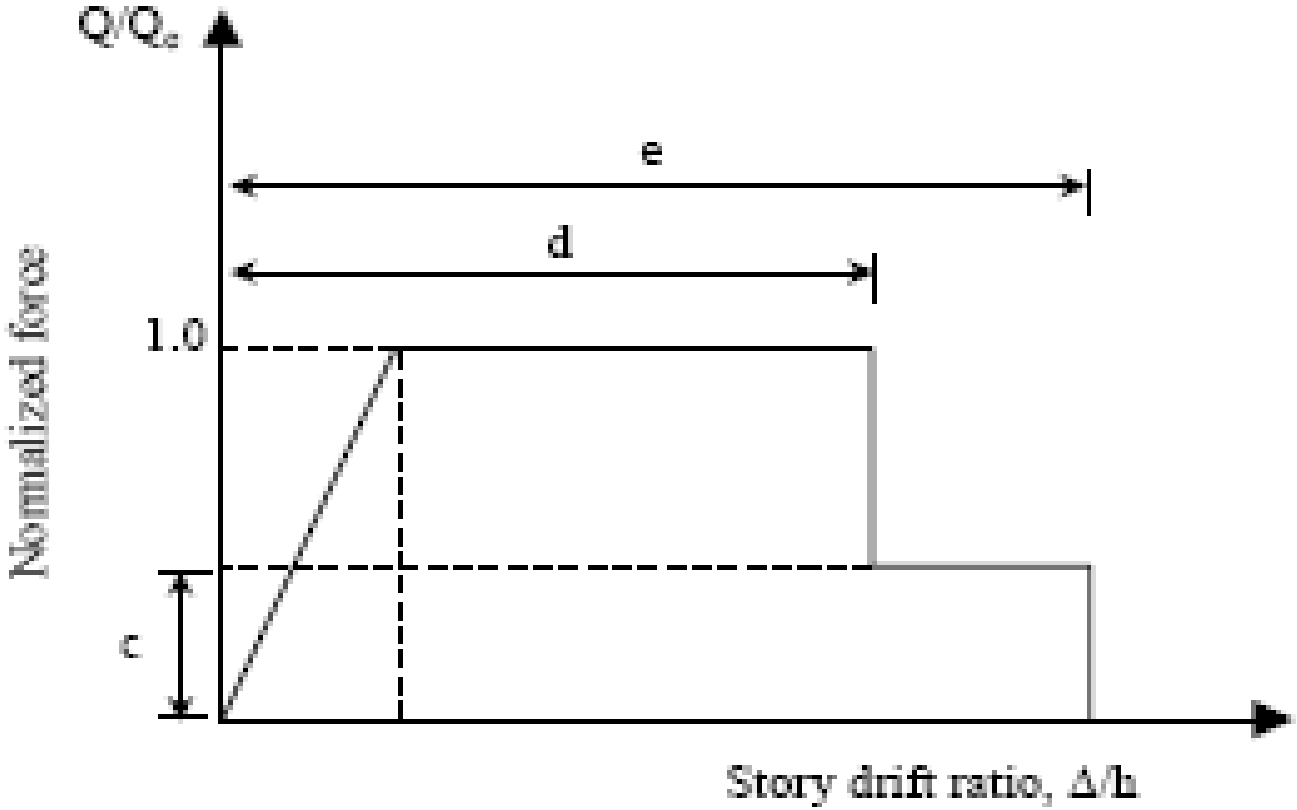
$$where: E_c I_c \text{ is the bending stiffness of the columns}$$

$$E_m \text{ is the modulus of Elasticity of masonry}$$

Modeling of Masonry Infill Walls

- In SAP 2000, equivalent diagonal compression strut will be modeled as an axial element having a nonlinear axial hinge along its length.
- According to FEMA-273, idealized force-displacement relations for infill wall are defined by a series of straight-line segment
- These relations are plotted between normalized force and story drift ratio.

Modeling of Masonry Infill Walls



Idealized force-displacement relation of infill wall (FEMA-273)

P-Delta (Second Order) Effects

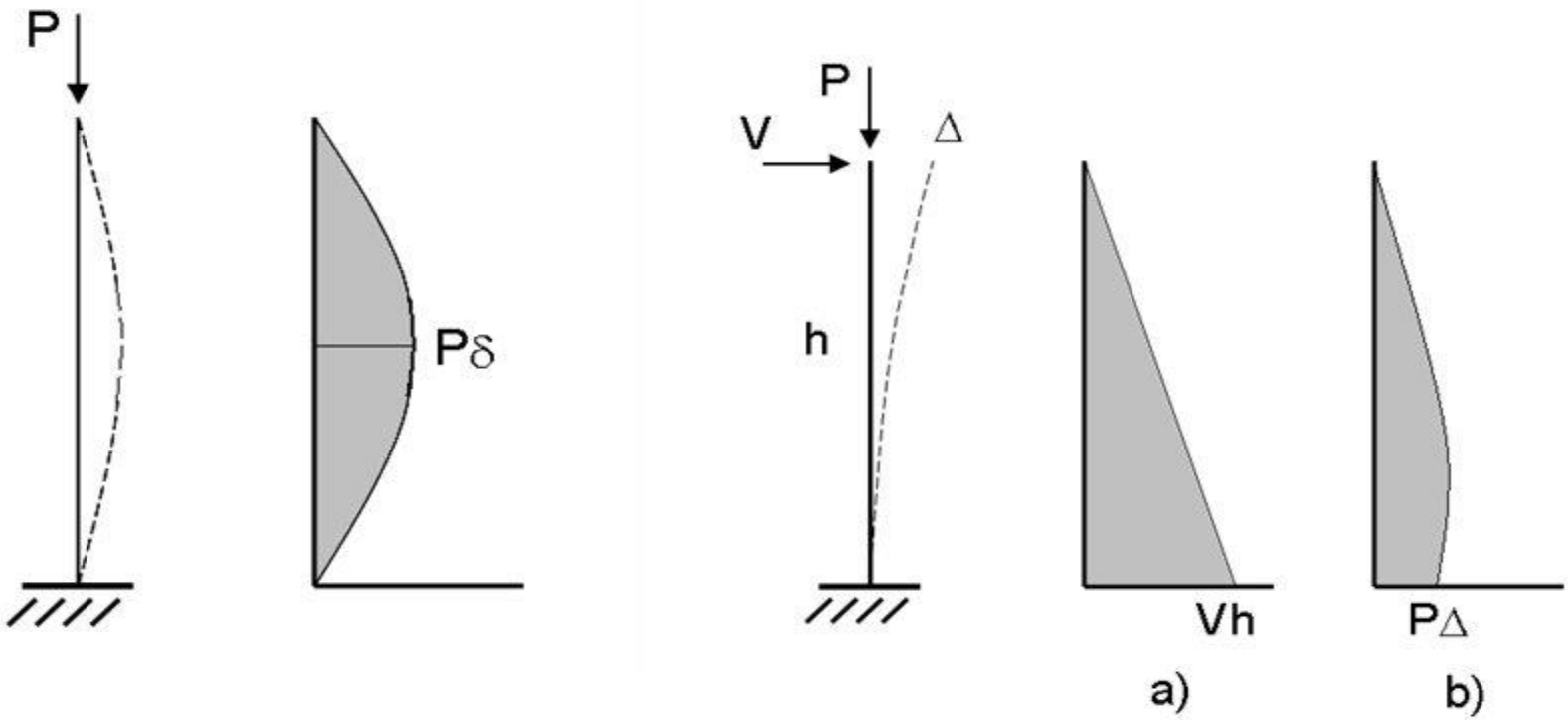
Second Order Analysis

- Second order analysis combines two effects to reach a solution:-
  - Large displacement theory; the resulting forces and moments take full account of the effects due to the deformed shape of both the structure and its members.
  - "Stress stiffening"; the effect of element axial loads on structure stiffness, tensile loads stiffening an element and compressive loads softening an element.

P – Delta Effect

- P-Delta is a non-linear (second order) effect that occurs in every structure where elements are subject to axial load. It is a genuine “effect” that is associated with the magnitude of the applied axial load (P) and a displacement (delta).
- The magnitude of the P-delta effect is related to the:-
  - Magnitude of axial load P
  - Stiffness/slenderness of the structure as a whole.
  - Slenderness of individual elements

P-Delta Effect – Basic Concept

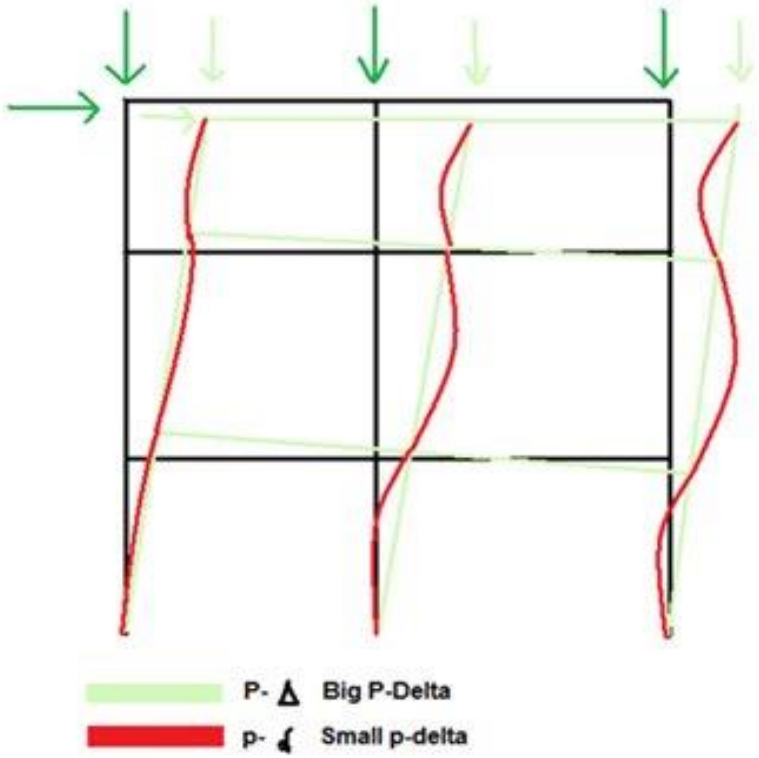


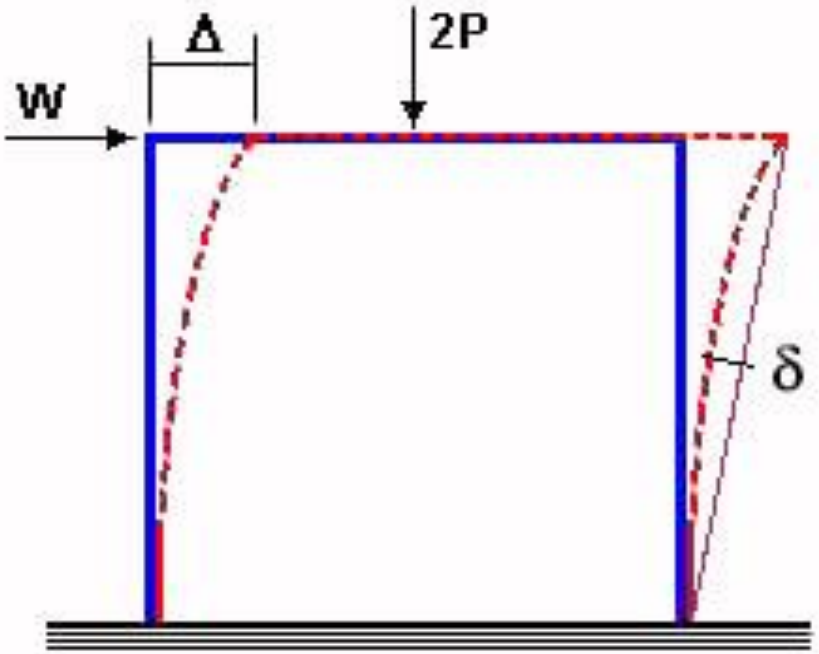
P-Δ and P-δ

- When P-Delta results are presented, two effects are incorporated into the results,
  - P-big-Delta (P-Δ), and
  - P-little-delta (P-δ).
- The P-Δ effect applies to members only and involves a modified bending moment acting along the length of the member. The columns experiences not only an axial loads (1st order) but also a moment equal to the vertical force, P, multiplied by the displacement, Δ.
- The P- δ effect is generally less important and results from the bent shape of a member that is carrying an axial force.

Difference Between P-Δ and P-δ

- Most people understand P-Delta as -
  - Frame deflects; Delta,
  - Load P is then eccentric to the base, this introduces further moments or 'second order effects'
- However, this only illustrates the P-"BIG" delta effect (P-Δ) which is only part of the second order effect.
- P Delta effect occurs at both an overall structural level and an element level.


P- Δ Big P-Delta
p- δ Small p-delta


Δ
2P
W
δ

Difference Between P-Δ and P-δ

- To obtain true design forces and moments, which accommodate all the P-Delta effects, then the analysis method used should account for both P-Δ and P-δ; both the deltas (Δ and δ) are inextricably linked – an increase in one brings about an increase in the other.
- The columns no longer remain straight after the initial analysis, resulting in not just the P-Δ moment, but also a P- δ moment that varies over the length of the member.
- In all cases when δ or Δ are "large", the results should be carefully examined.

P-Δ and P-δ

- Analysis does not calculate p-δ directly, but this is not a serious problem because the largest moments in a frame under lateral force sway generally occur at the column ends where p-δ is zero.
- The Analysis Procedure for P-Delta Analysis
  - First, a standard first-order analysis is performed to calculate the deflection, D. Then, additional passes of "deflected" analyses are made with the deflected shape of the structure, resulting in potentially more moments.
  - The iterative process is continued until either results converge, some maximum iteration limit is reached, or results start to diverge (some element's stiffness goes 'negative' indicating buckling).
  - The results are based on the final "deflected" run.

P-Delta Analysis in SAP2000

- The program can include the P-Delta effects in almost all Non-linear analysis types
- Specific P-Delta analysis can also be carried out
- The P-Delta analysis basically considers the geometric nonlinear effects directly
- The material nonlinear effects can be handled by modification of cross-section properties
- The Buckling Analysis is not the same as P-Delta Analysis
- No magnification of moments is needed if P-Delta Analysis has been carried out

P-Delta Analysis in ETABS

- Specific P-Delta analysis is available
- The P-Delta analysis basically considers the geometric nonlinear effects directly
- The material nonlinear effects can be handled by modification of cross-section properties
- The Buckling Analysis is not the same as P-Delta Analysis
- No magnification of moments is needed if P-Delta Analysis has been carried out

Occurrences of Boundary Nonlinearity

- Contact
  - contact conditions such as constraints and restraints which allow parts or portions of the same part to touch or lift off each other.
  - model the interactions of certain systems.
- Forces
  - represent loads that can be defined as displacement or velocity based such as earthquakes and soil conditions

Capacity based Design of Structural Components

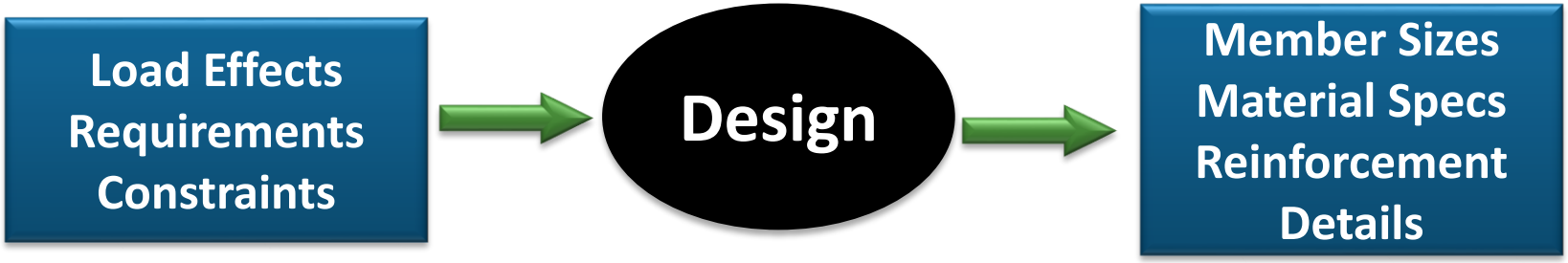
Seminar on Performance Based Design of RC Buildings

Asian Institute of Technology

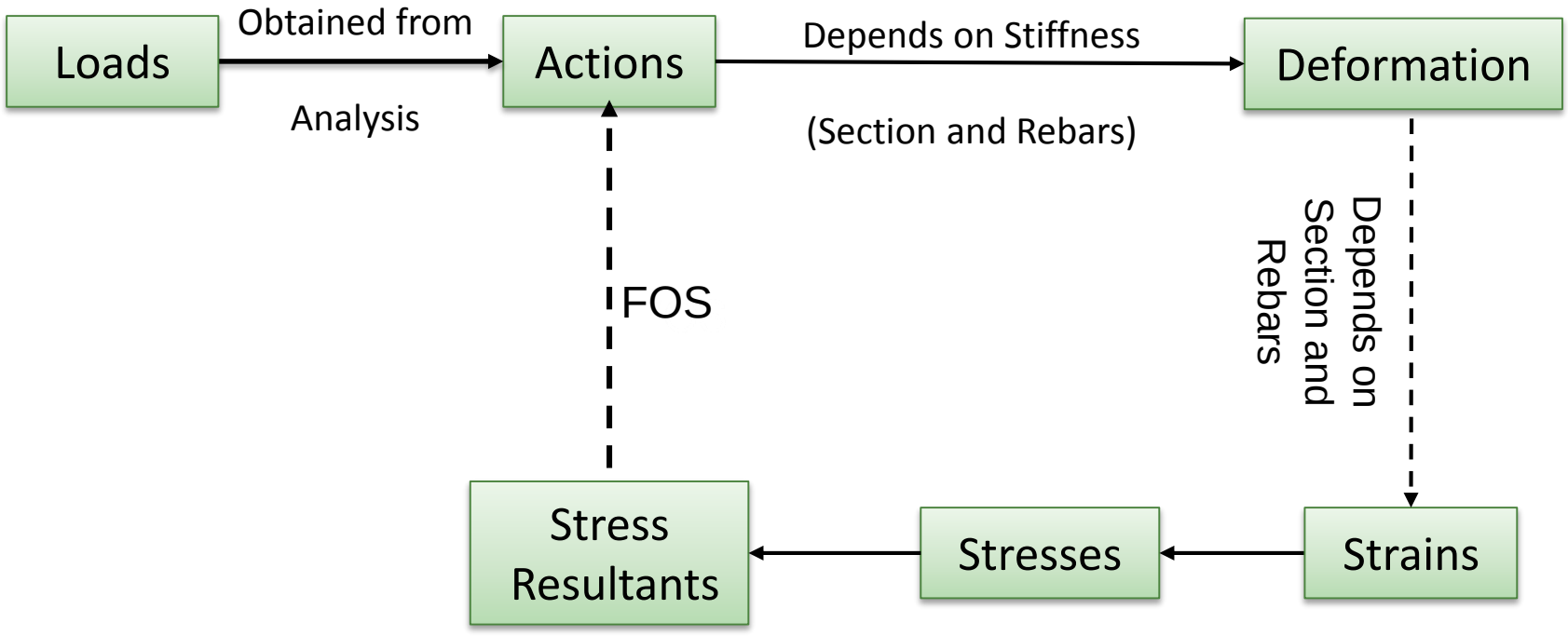
Dr. Naveed Anwar
Executive Director, AIT Consulting
Affiliated Faculty, Structural Engineering
Director, ACECOMS

Design Process

- "Structural Design is the process of proportioning the structure to safely resist the applied forces in the most cost effective and friendly manner"



Loads and Stress Resultants



Section Capacity/Section Design Process

The Response and Design

From Loads to Stresses

Applied Loads
↓
Building Analysis
↓
Member Actions
↓
Cross-section Actions
↓
Material Stress/Strain

From Strains to Response

Material Response
↓
Section Response
↓
Member Response
↓
Building Response
↓
Load Capacity

From Serviceability to Performance

- **Satisfying one design level does not ensure that other design levels will be satisfied**
  - Serviceability design only ensures that deflections and vibrations, etc., for service loads are within limits but says nothing about strength.
  - Strength design ensures that a certain factor of safety against overload is available within a member or a cross-section but says nothing about what happens if the load exceeds the design level.
  - Performance design ensures that the structure as a whole reaches a specified demand level. Performance design can include both service and strength design levels.

Lateral Strength Based Design

- This is most common seismic design approach adopted nowadays.
- It is based on providing the structure with the minimum lateral strength to resist seismic loads, assuming that the structure will behave adequately in the non-linear range.
- For this reason only some simple construction detail rules are needed to be satisfied.

Displacement Based Design

- In this method the structure is designed to possess adequate ductility so that it can dissipate energy by yielding and survive the shock.
- This method operates directly with deformation quantities hence gives better insight on the expected performance of the structures.
- The displacement based design approach has been adopted by the seismic codes of many countries.

Capacity Based Design

- In this design approach the structures are designed in such a way so that plastic hinges can form only in predetermined positions and in predetermined sequences.
- The concept of this method is to avoid brittle mode of failure.
- This is achieved by designing the brittle modes of failure to have higher strength than ductile modes.

Energy Based Design

- This is the most promising and futuristic approach of earthquake resistant design.
- In this approach it is assume that the total energy input is collectively resisted by kinetic energy, the elastic strain energy and energy dissipated through plastic deformations and damping.

Capacity Design Approach – General Concepts

What is Capacity Design?

- **Capacity Design** is a design process in which it is decided which objects within a structural system will be permitted to yield (ductile components) and which objects will remain elastic (brittle components).

- Once ductile and brittle systems are decided upon, design proceeds according to the following guidelines:
  - **Ductile components** are designed with sufficient deformation capacity such that they may satisfy displacement-based demand-capacity ratio.
  - **Brittle components** are designed to achieve sufficient strength levels such that they may satisfy strength-based demand-capacity ratio.

Why Capacity Design?

- It is best to implement Capacity Design because structural performance is then a deliberate intention of the designer, and not revealed in a secondary manner by computational tools.
- Further, because of the many sources of uncertainty inherent to structural modeling and analysis, unless ductile systems are predetermined, a computational tool may not accurately indicate which systems will achieve inelastic response.
- In summary, Capacity Design enables the creation of a more reliable computational model, which should lead to better structural design.

Why Capacity Design?

- Capacity Design also comes to the relief of computational time. When an engineer knows which objects will behave elastically, and which will be permitted to yield, material nonlinearity need only be modeled for ductile components, while components which will not yield need only consider elastic stiffness properties. These relationships are linear, which provides for a more simple formulation of less computational demand.
- Brittle components are redesigned such that strength capacity exceeds that demanded. A level of complexity comes with the redesign of ductile components, however, in that ductile components may satisfy nonlinear demand-capacity criteria through a balance of both strength and deformation capacity.
- While Capacity Design should lead to more reliable modeling and more accurate results, engineers should note that computational models only represent a mathematical simulation of physical phenomena, and cannot exactly predict structural behavior.
- Too many sources of uncertainty exist, and it is up to the designer to best characterize as many behavioral parameters as is practical.

Capacity Design Philosophy

- Structures are designed for many limit states

- **Load Design** = all limit states must occur beyond a minimum load level
- **Capacity Design** = same as above, except now we choose that one limit state is to occur before any other

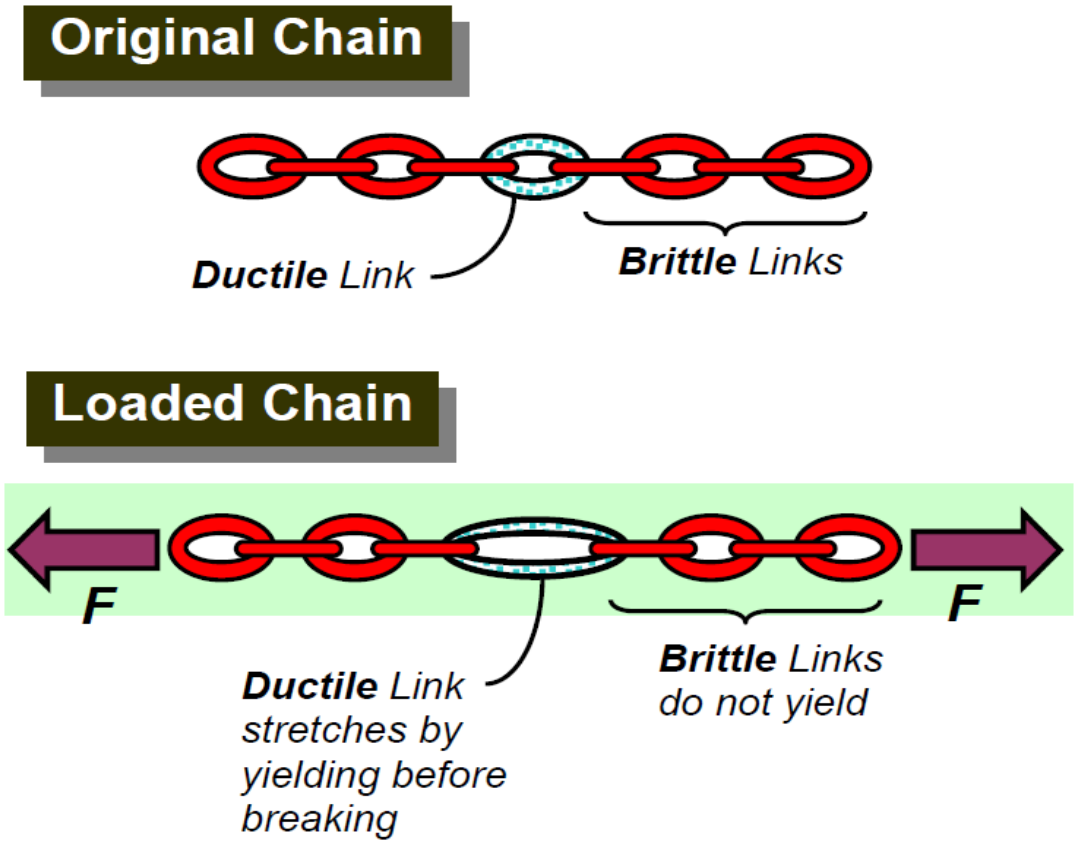
- **Difficult part is choosing the Limit State that should govern**

Where is Capacity Design Used?

- Seismic Design Guidelines
  - Seismic design guidelines (UBC) are written with a specific intent of capacity design

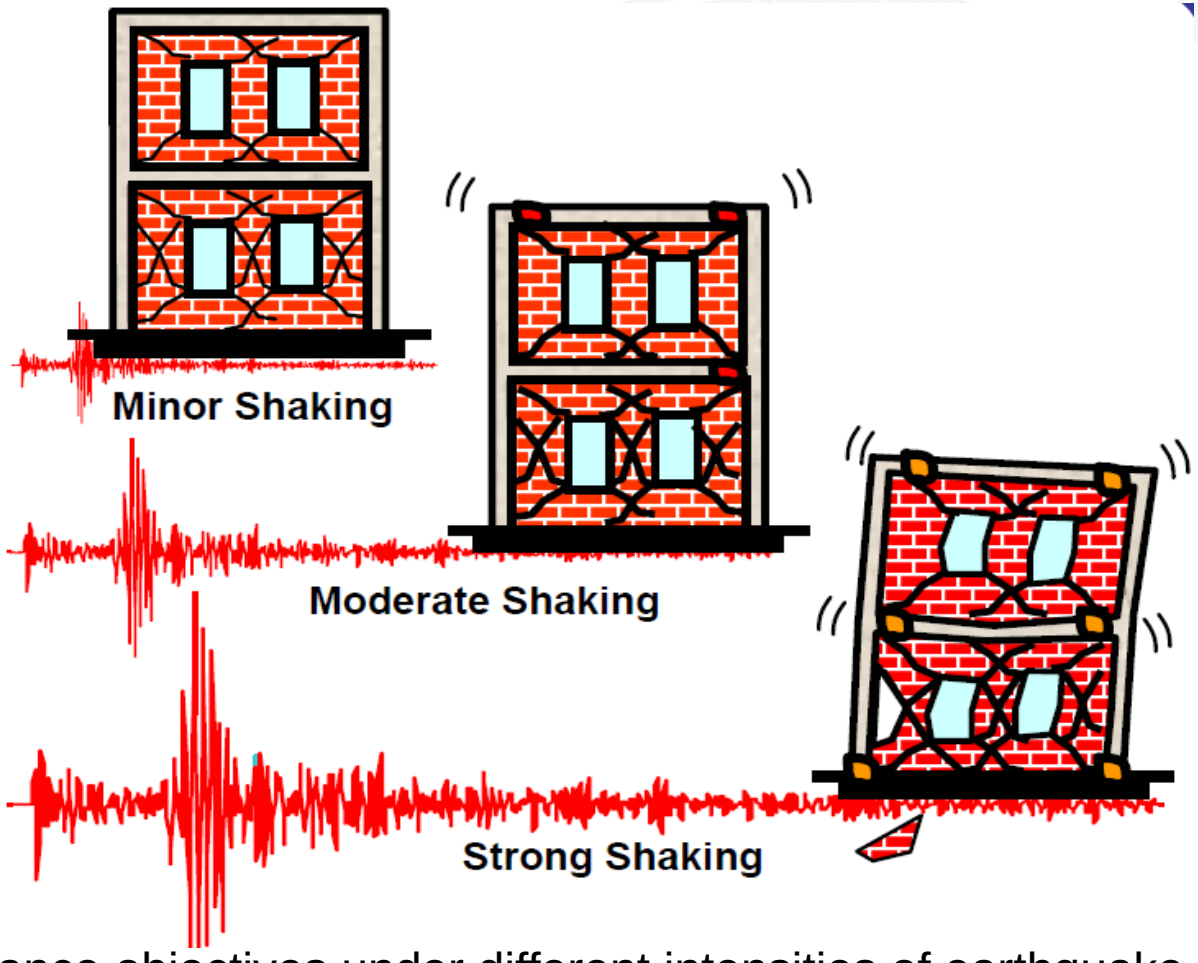
- Special Moment Resisting Frame = hinges should form in beam
- Special Concentric Braced Frame = braces should yield in tension
- Eccentric Braced Frame = link region of beam to yield in shear

Ductile Link Analogy



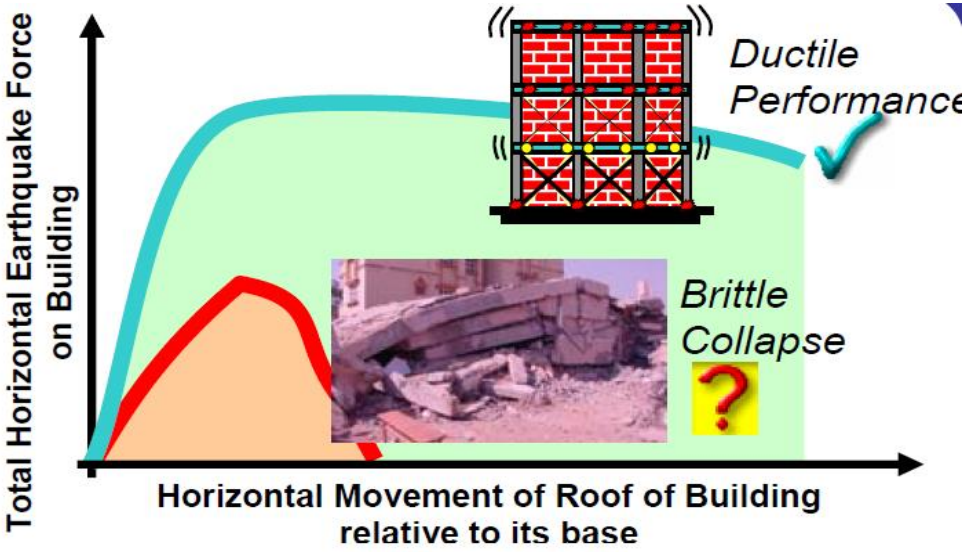
Ductile chain design

C.V.R.Murty, 2002



Performance objectives under different intensities of earthquake shaking – seeking low repairable damage under minor shaking and collapse-prevention under strong shaking.

C.V.R.Murty, 2002



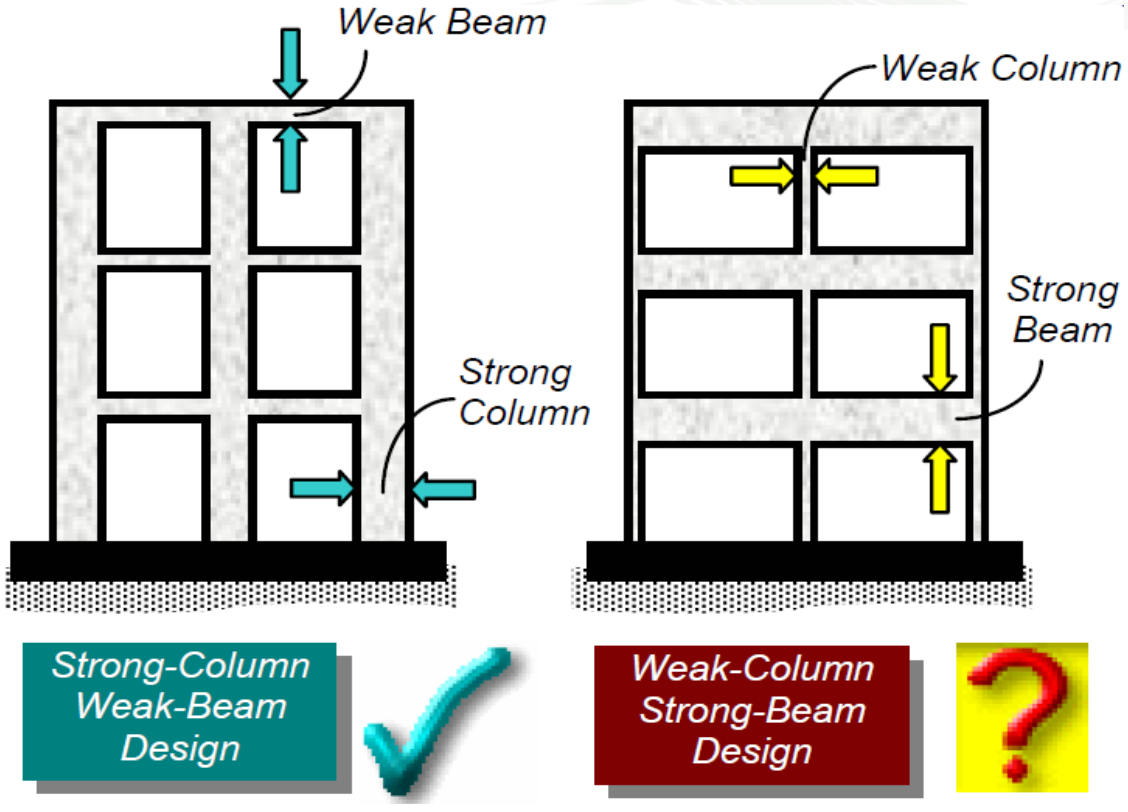
(a) Building performances during earthquakes: two extremes – *the ductile and the brittle.*

Photo from: Housner & Jennings, *Earthquake Design Criteria*, EERI, USA

(b) Brittle failure of a reinforced concrete column

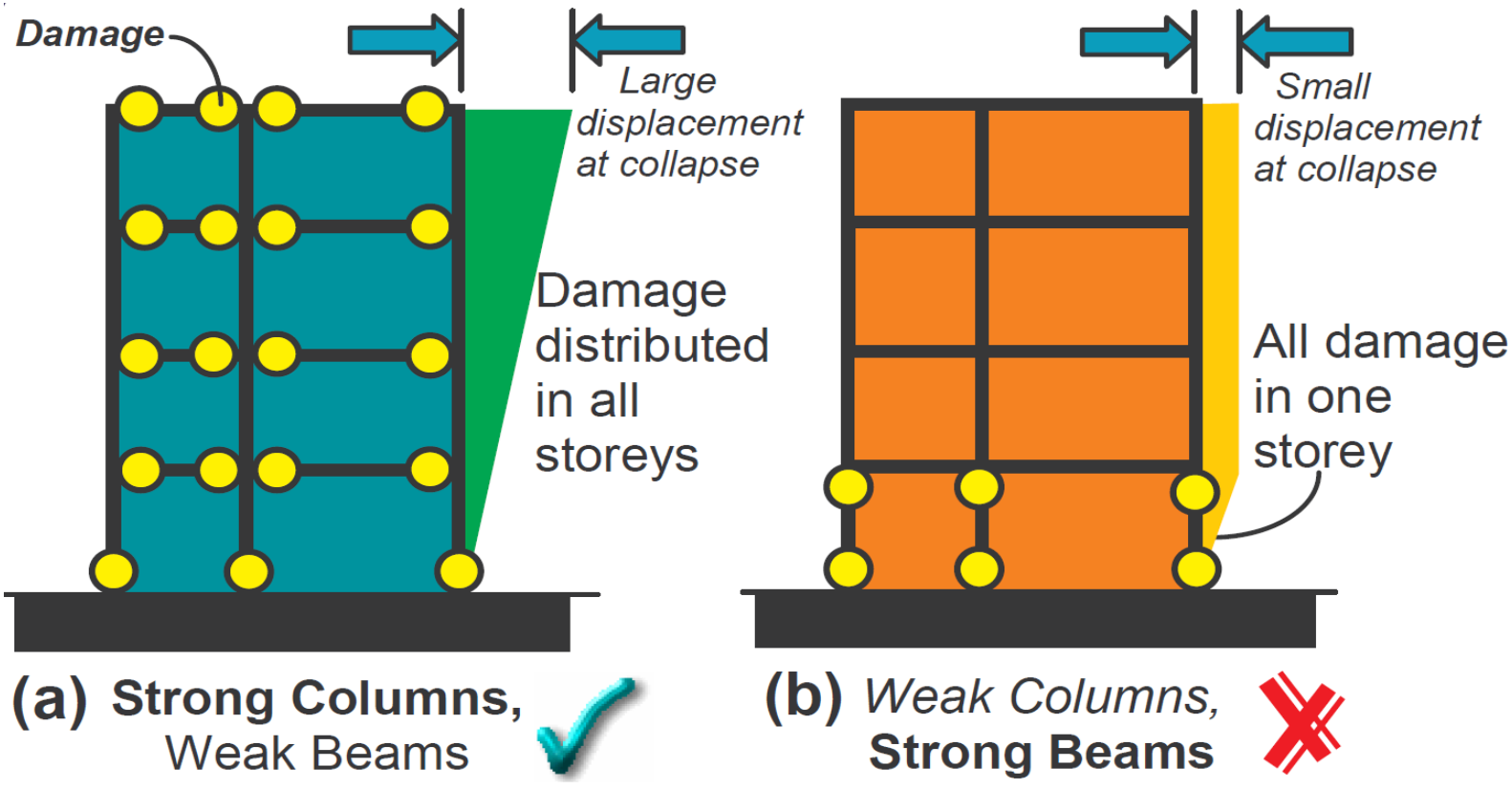
Ductile and brittle structures – seismic design attempts to avoid structures of the latter kind.

C.V.R.Murty, 2002



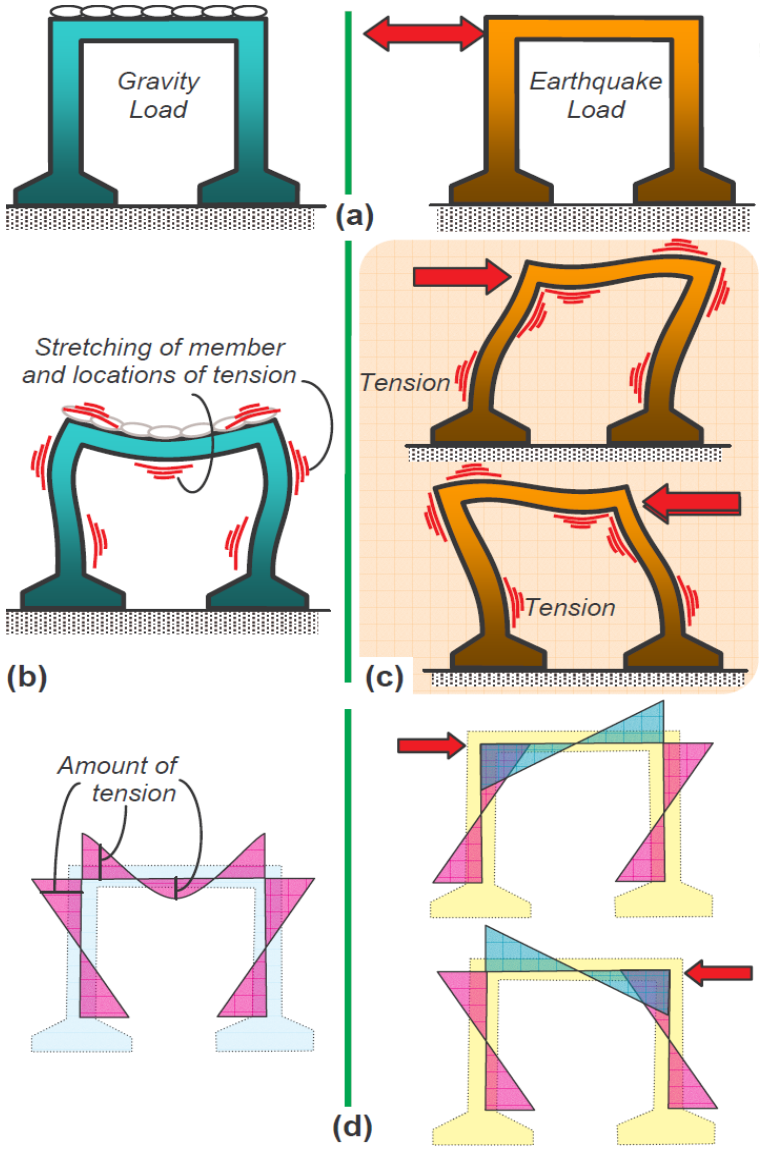
The beams must be the weakest links and not the columns – this can be achieved by appropriately sizing the members and providing correct amount of steel reinforcement in them.

C.V.R.Murty, 2002



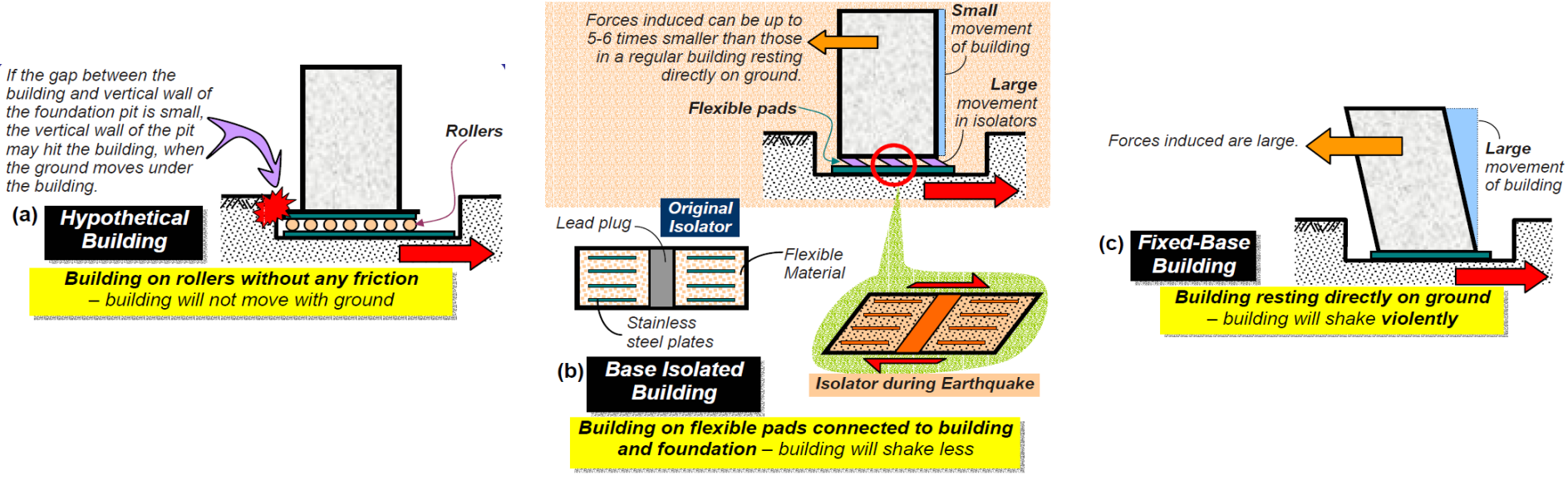
Two Distinct design of buildings that result in different earthquake performances – Columns should be stronger than beams

C.V.R.Murty, 2002



Earthquake shaking reverses tension and compression in members – Reinforcement is required on both faces of the members.

C.V.R.Murty, 2002



Building on flexible supports shakes lesser – this technique is called **Base Isolation**.

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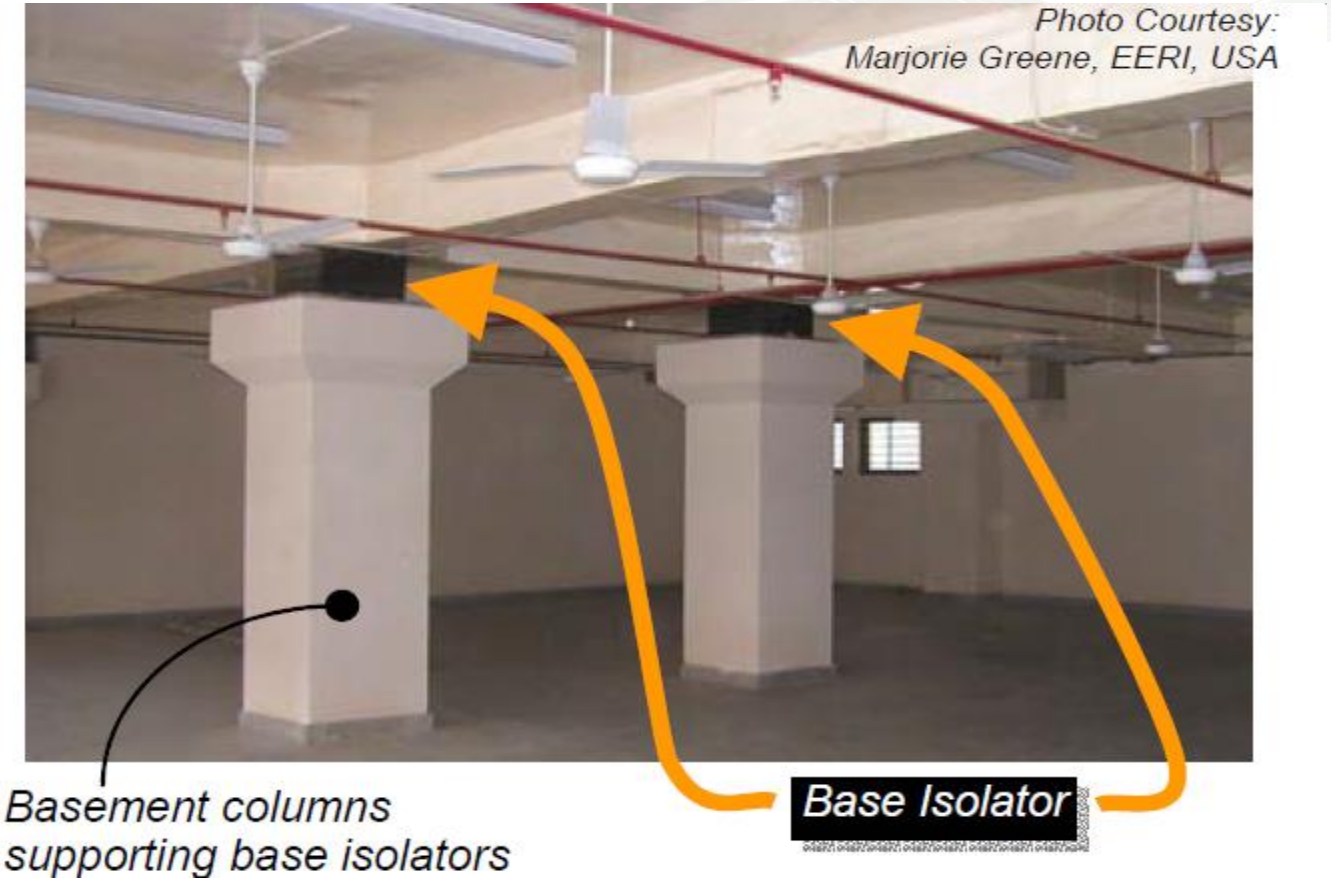
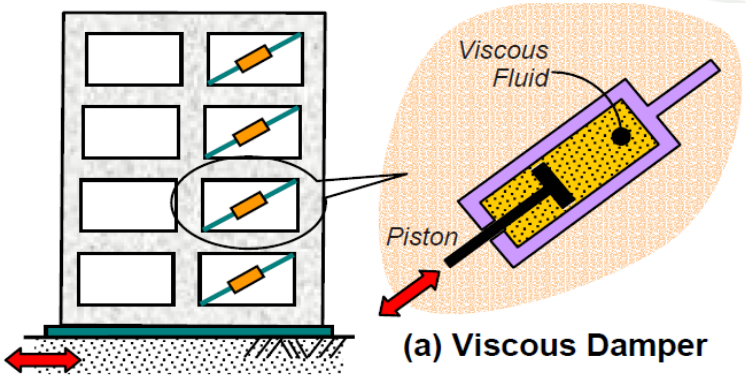
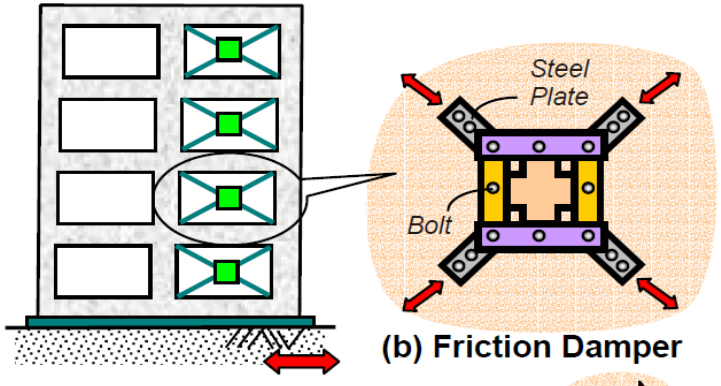


View of Basement in a Hospital building – built with base isolators after the original building collapsed during the 2001 Bhuj earthquake.

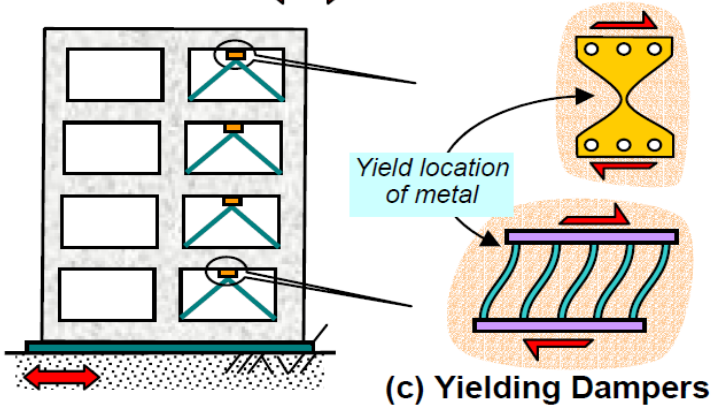
C.V.R.Murty, 2002



(a) Viscous Damper



(b) Friction Damper

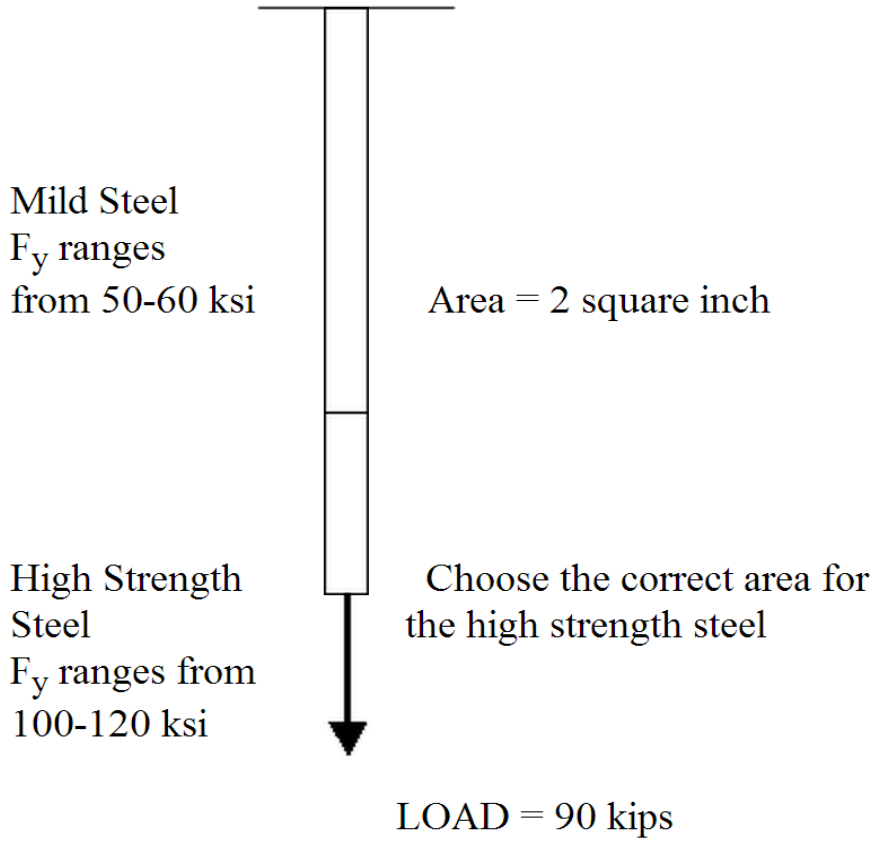


(c) Yielding Dampers

Seismic Energy Dissipation Devices – each device is suitable for a certain building**.**

C.V.R.Murty, 2002

Example – A Hanging Rope



Example – A Hanging Rope

LOAD DESIGN

Typical design = Area of bar needs to be:

$$A = P/F_y = 90 / 100 = 0.9 \text{ sq. in.}$$

CAPACITY DESIGN

Now we add another criteria

The mild steel bar must yield before the high strength steel

The high strength steel has been selected to carry the load of 90 kips.
But the mild steel has a maximum strength of 2 x 60 = 120 kips
If we want the mild steel to yield first, we need at least

$$A = C_{max} / F_y = 120 / 100 = 1.2 \text{ sq. in.}$$

for the high strength steel

Thank You