Credits: 3 + 0
PG 2022
Spring 2020 Semester

Performance-based Seismic Design of Structures

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Inelastic Behaviour of Structural Components

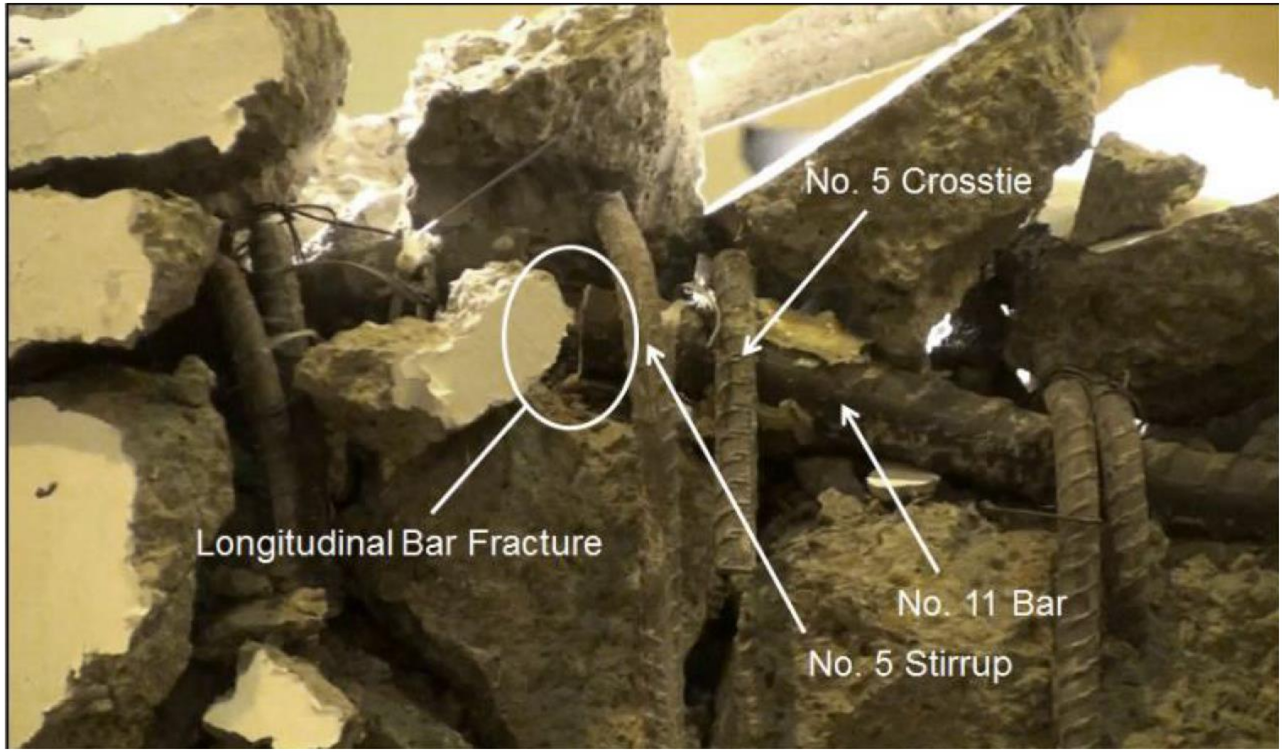
Source:

Dr. Pramin Norachan
AIT Solutions
Thailand

Failure Modes of RC Moment-resisting Frame Elements

- Flexural softening due to longitudinal bar yielding
- Spalling of concrete cover
- Crushing of the concrete core
- Shear failure either prior or subsequent to flexural softening
- Loss of confinement from fracture of transverse reinforcement
- Longitudinal bar buckling and fracture
- Lap splice failure
- Bond failure across longitudinal bars
- Instability due to slenderness effects or excessive loss of strength

Longitudinal Rebar Failure



PEER Report 2013/16

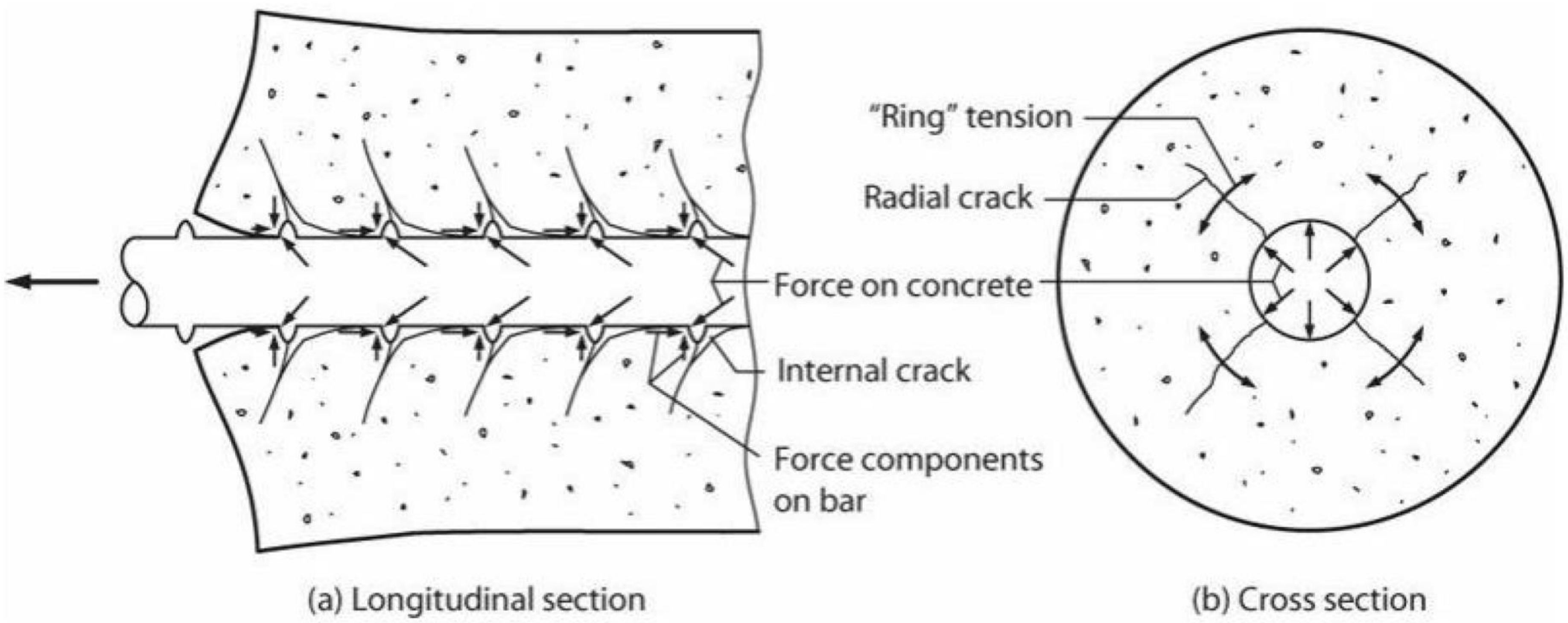
Column Shear and Axial Failures

Rebar Buckling and Lap Splice Failure

Severe spalling of column cover concrete exposing longitudinal steel. Source: FEMA P-58/BD-3.8.6

Column with inadequate ties
Source: FEMA 451

Anchorage Failure



Reinforced Concrete Moment Frames

- **Seismically detailed moment frames**
  - Ensure ductile flexure yielding
  - Prevent less desirable failure modes (shear, anchorage failures)
  - Initiate loss of lateral strength at relatively large deformations
- **Non-seismically detailed frames**
  - Any failure mode can be expected whether ductile or not
  - Premature failures cause significant strength losses and structural instability
  - Anchorage failures of longitudinal bars, lap-splice failures, shear failures, interface shear failures, premature buckling of longitudinal bars, and crushing of the concrete core due to inadequate confinement.

Failure Modes of Columns

- Nonstructural components can shift the location of hinging, as well as increase shear and local deformation demands on columns significantly.
  - Those effects should be eliminated by altering the details of nonstructural components, if such effects are anticipated, their effects on member behavior should be accounted for in simulation.

- Shear failure can occur prior or subsequent to flexural yielding in frame members.
  - If the estimated shear strength of a member is lower than its probable flexural strength, shear failure is expected prior to flexural yielding at relatively low deformation levels. However, if shear strength is slightly larger than flexural strength (less than 30% larger), then shear failure could occur after inelastic deformations in the hinge regions where the strength of shear transfer mechanisms degrade with inelastic deformations.

Joint Failures

Joint Failures

- If beam and column are adequately reinforced to resist the seismic forces then **joint may become a weak link**. Thus, unconfined joint may fail in pure shear with no yielding of beam or column reinforcement.

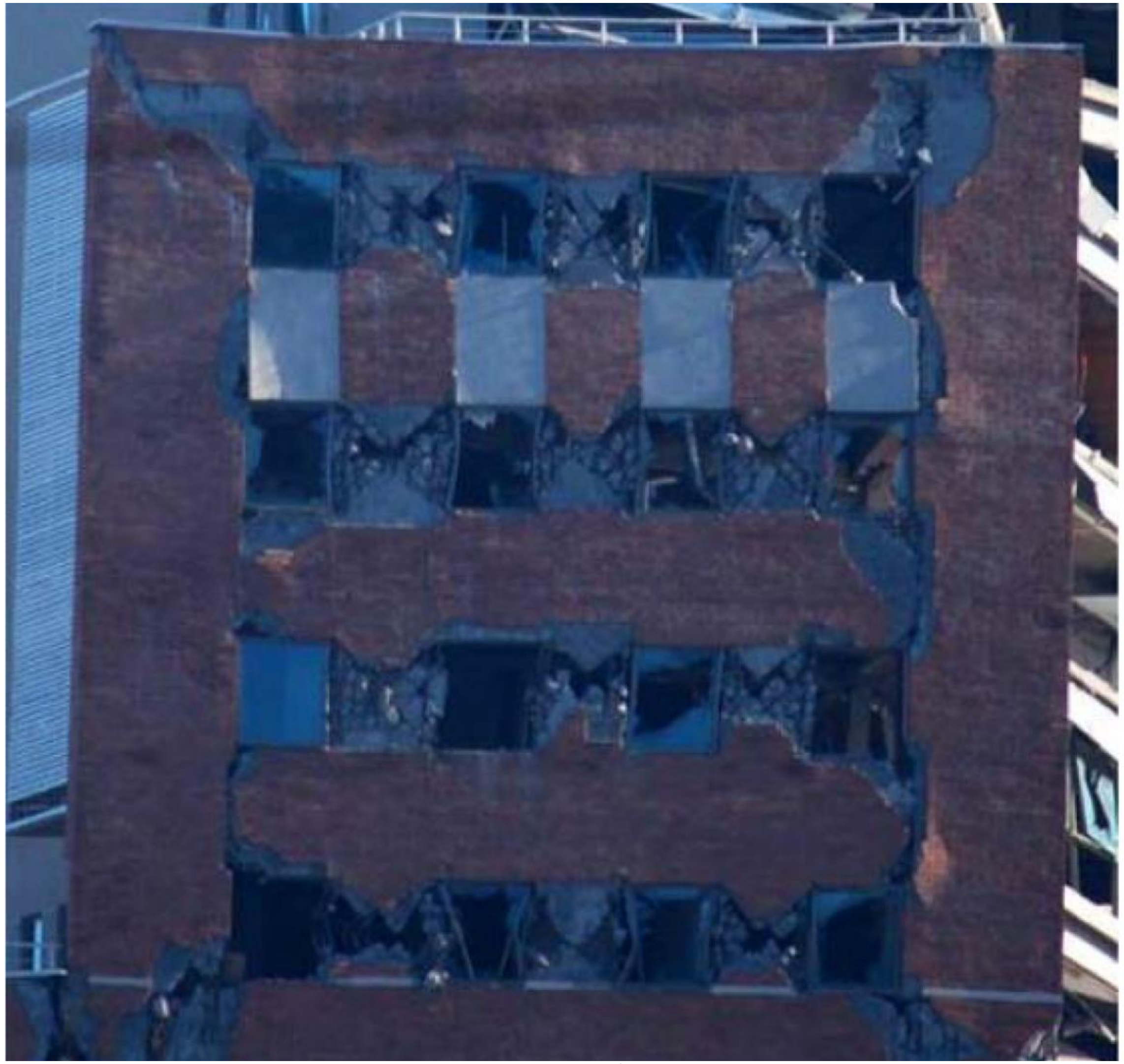
- Joints may also fail after yielding of top and bottom reinforcements of beams, the joint experiences severe shear cracking and subsequently joint shear failure. Compared to pure joint failure, this failure is more ductile since it involves beam yielding.

- To prevent joint failures it is important to make sure that joint strength should be greater than the strength of them elements framing into it.

Factors Effecting Joint Strength

- Joint aspect ratio has significant impact on shear strength of joints, the shear strength reduces if we have high aspect ratio.
- Confinement of the joint either through transverse frame members or reinforcement increases the joint shear strength.
- High axial load ratios reduce the ductility of the joints and more rapid strength degradation after initiation of failure.
- Beam Reinforcement ratios effect the joint shear strength and determine what type of failure occurs in joints.

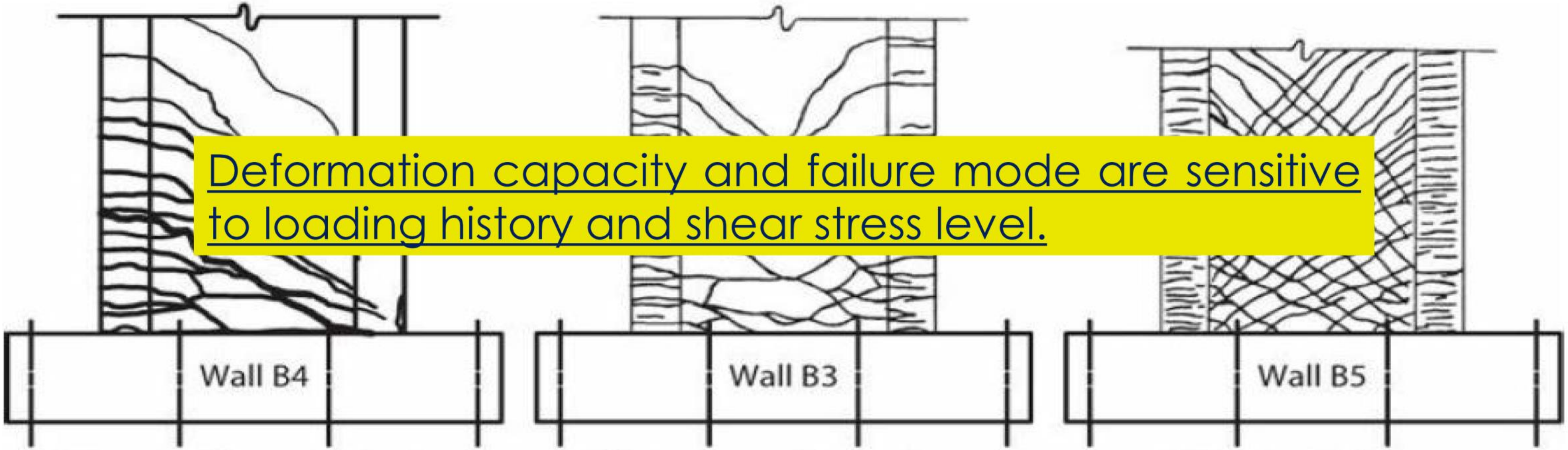
Failure Modes of RC Shear Wall Elements



Shear failures in vertical and horizontal wall segments

Sources:
1) AIT Solutions
2) “Seismic Design of Reinforced Concrete Buildings” by Jack Moehle

Behavior of Slender Structural Walls



(a) Monotonic, moderate shear (b) Cyclic, moderate shear (c) Cyclic, high shear

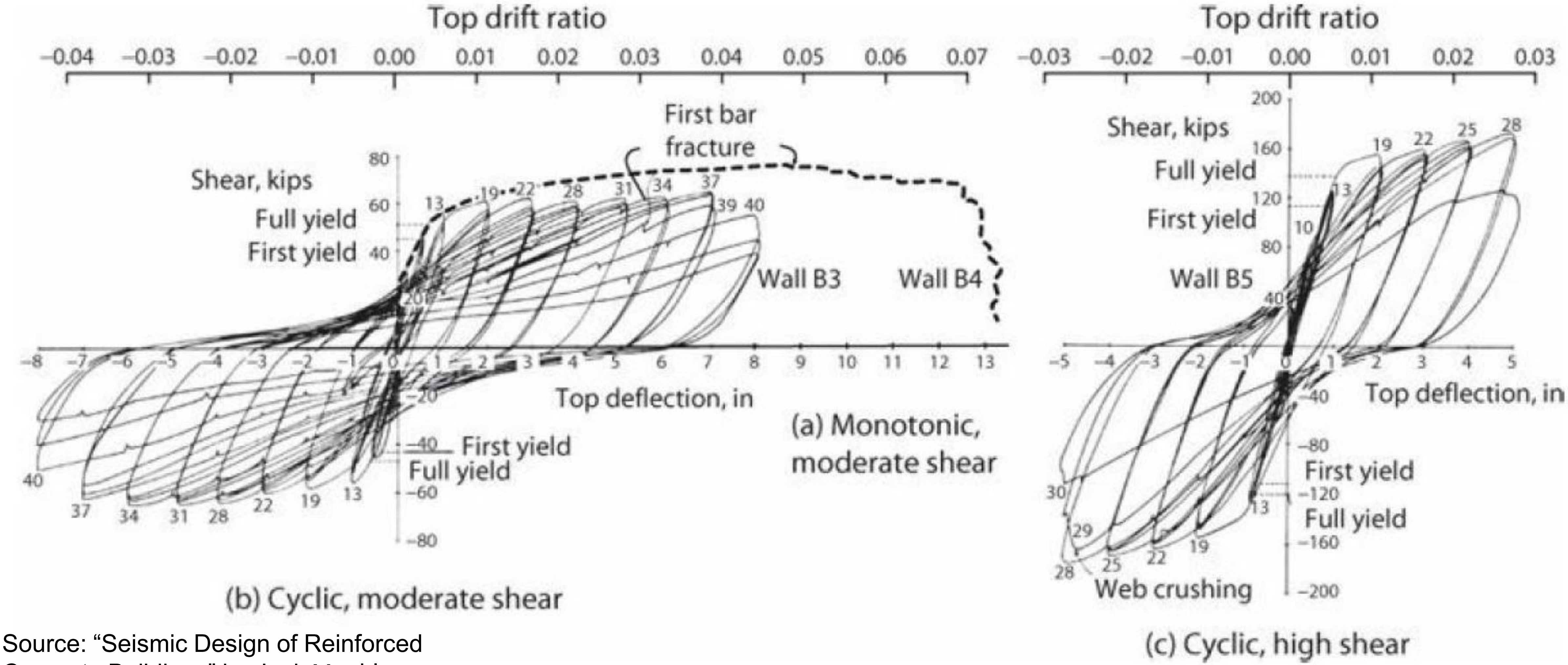
Deformation capacity and failure mode are sensitive to loading history and shear stress level.

Monotonic loading
Maximum shear = 0.33 $A_{cv} \sqrt{f'_c}$ (MPa)
Cracking = Fan pattern in yielding region
Drift ratio = 0.07
Failure = Fracture of longitudinal reinforcement

Cyclic loading, Moderate shear
Maximum shear = 0.26 $A_{cv} \sqrt{f'_c}$ (MPa)
Cracking = Crisscrossed pattern
Drift ratio = 0.045

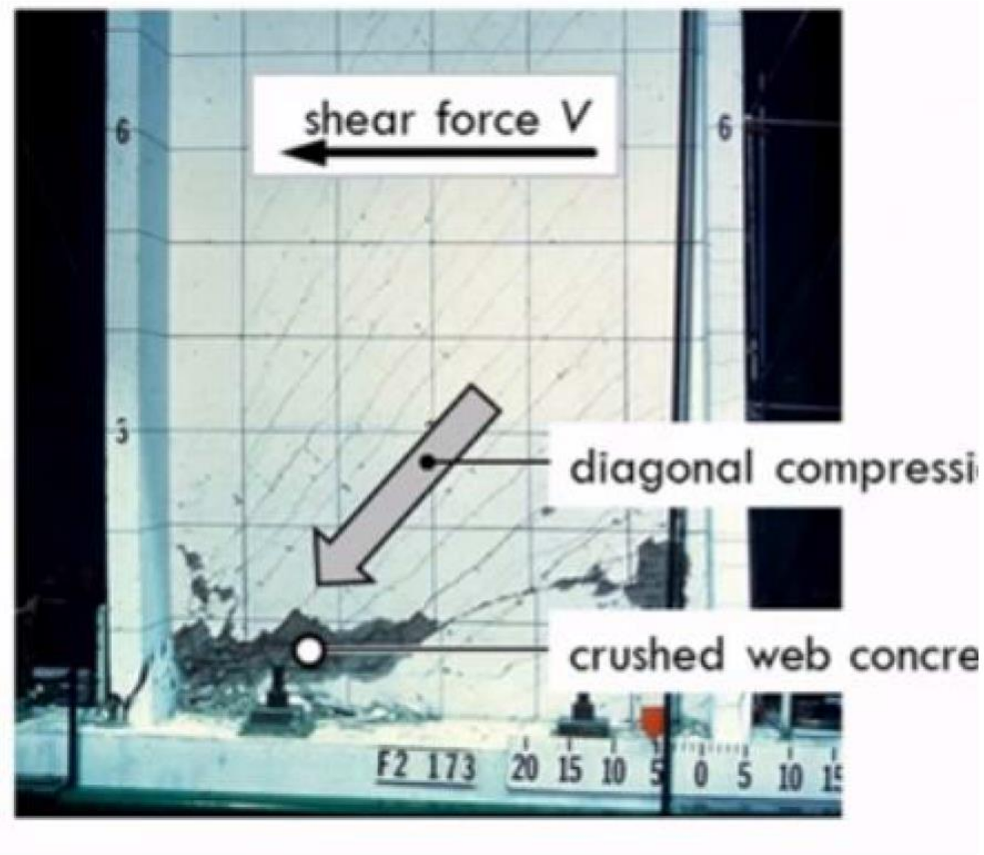
Cyclic loading, High shear
Maximum shear = 0.73 $A_{cv} \sqrt{f'_c}$ (MPa)
Cracking = Extensive diagonal cracking
Drift ratio = 0.028
Failure = Developed design moment and failed by diagonal compression strut

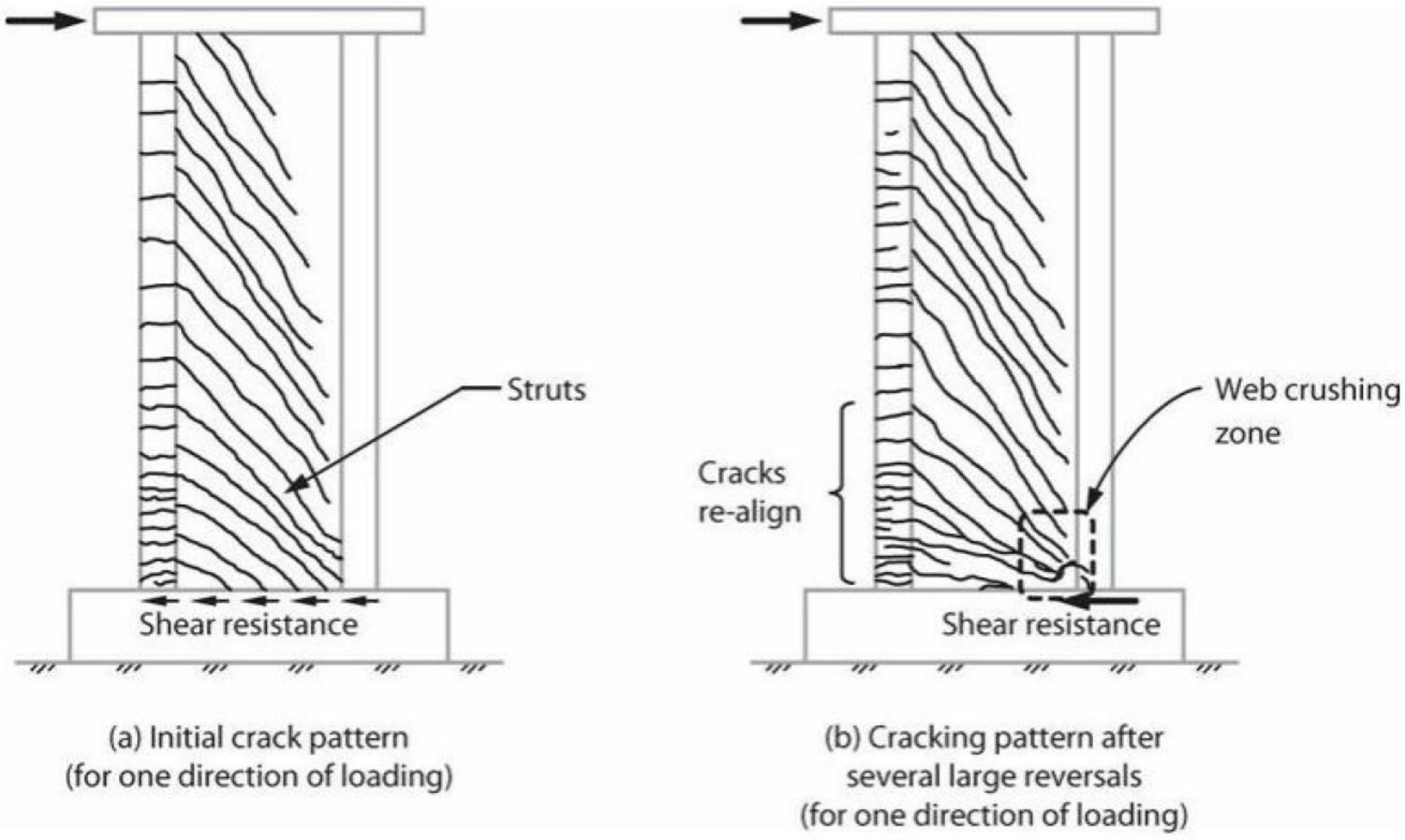
Load-displacement relationship of shear walls



Source: “Seismic Design of Reinforced Concrete Buildings” by Jack Moehle

Damage Patterns and Hysteretic Response of Shear Walls

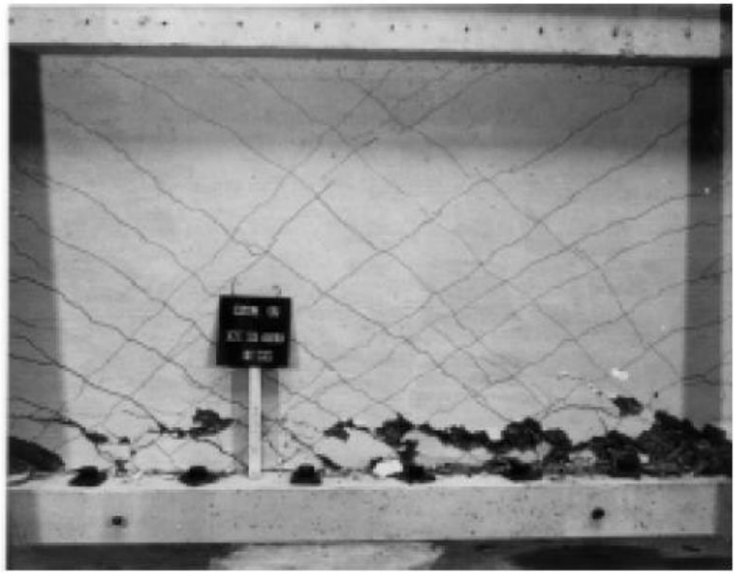




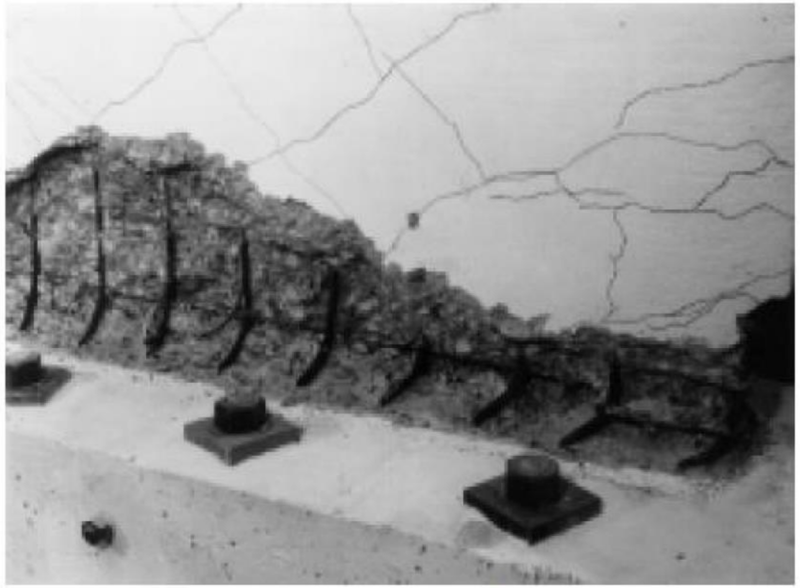
Source: "Seismic Design of Reinforced Concrete Buildings" by Jack Moehle

Flexure / Web crushing

Damage Patterns and Hysteretic Response of Shear Walls



Splitting and Crushing of Concrete at Base of Wall

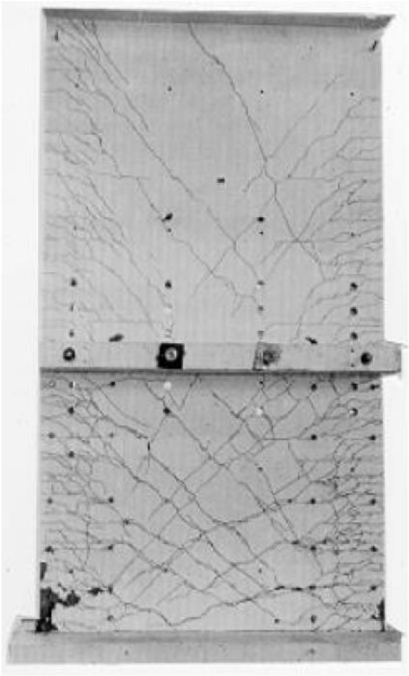


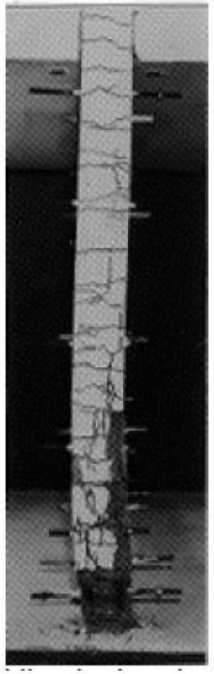
Compression Toe

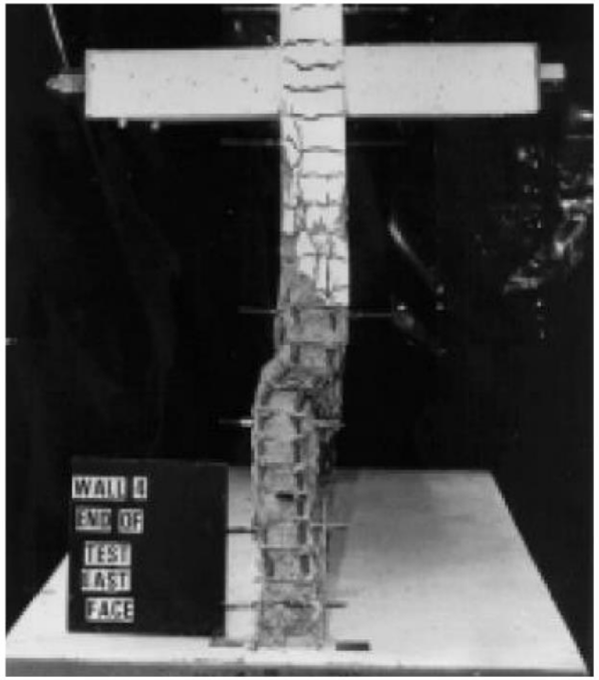
Flexure / Sliding shear

Source: FEMA 307

Damage Patterns and Hysteretic Response of Shear Walls



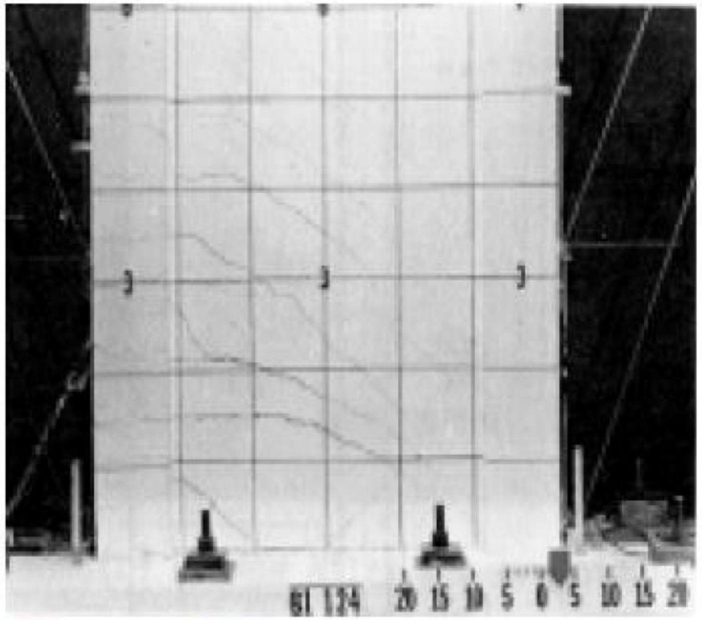




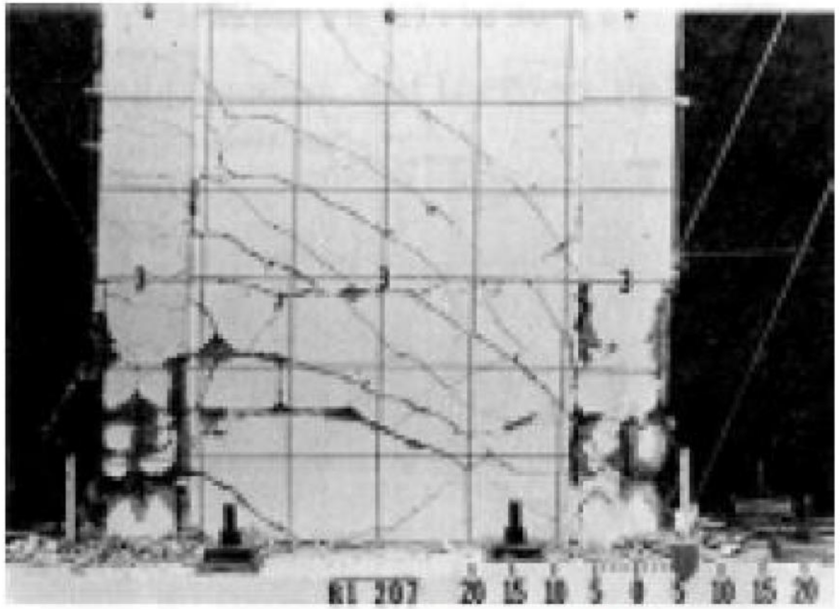
Flexure / Out-of-plane wall buckling

Source: FEMA 307

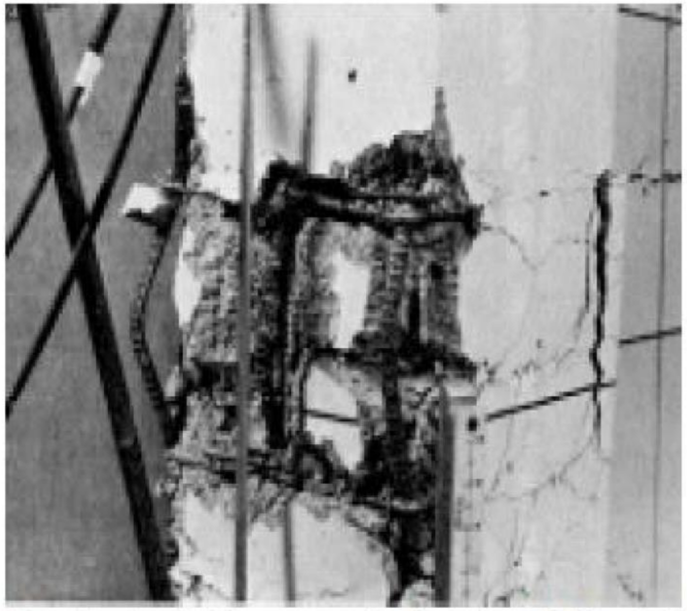
Damage Patterns and Hysteretic Response of Shear Walls



Damage at +3-in. deflection
$\Delta = 3$ in $\Delta/h_w = 0.017$ $\lambda_Q = 1.0$



Damage during Load Cycle 34
$\Delta = 6$ in $\Delta/h_w = 0.033$ $\lambda_Q = 0.6$

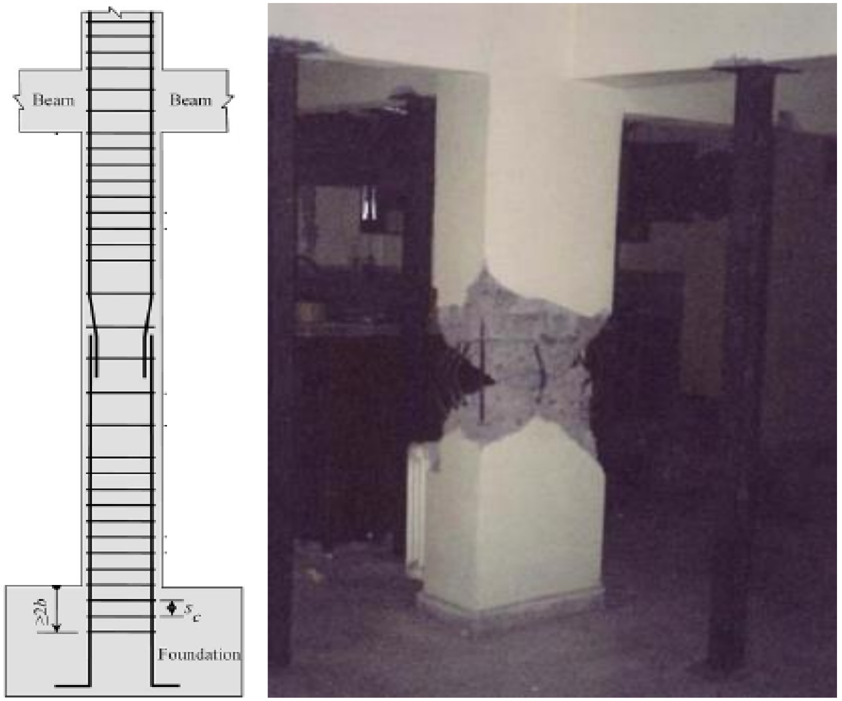


Buckled reinforcement after Load Cycle 30
$\Delta = 4$ in $\Delta/h_w = 0.022$ $\lambda_Q = 0.9$

Flexure / Boundary compression

Source: FEMA 307

Seismic Performance of RC Buildings

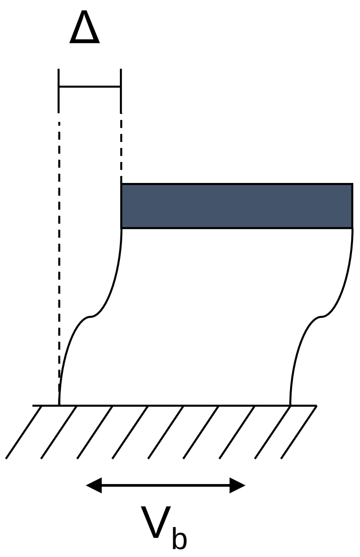


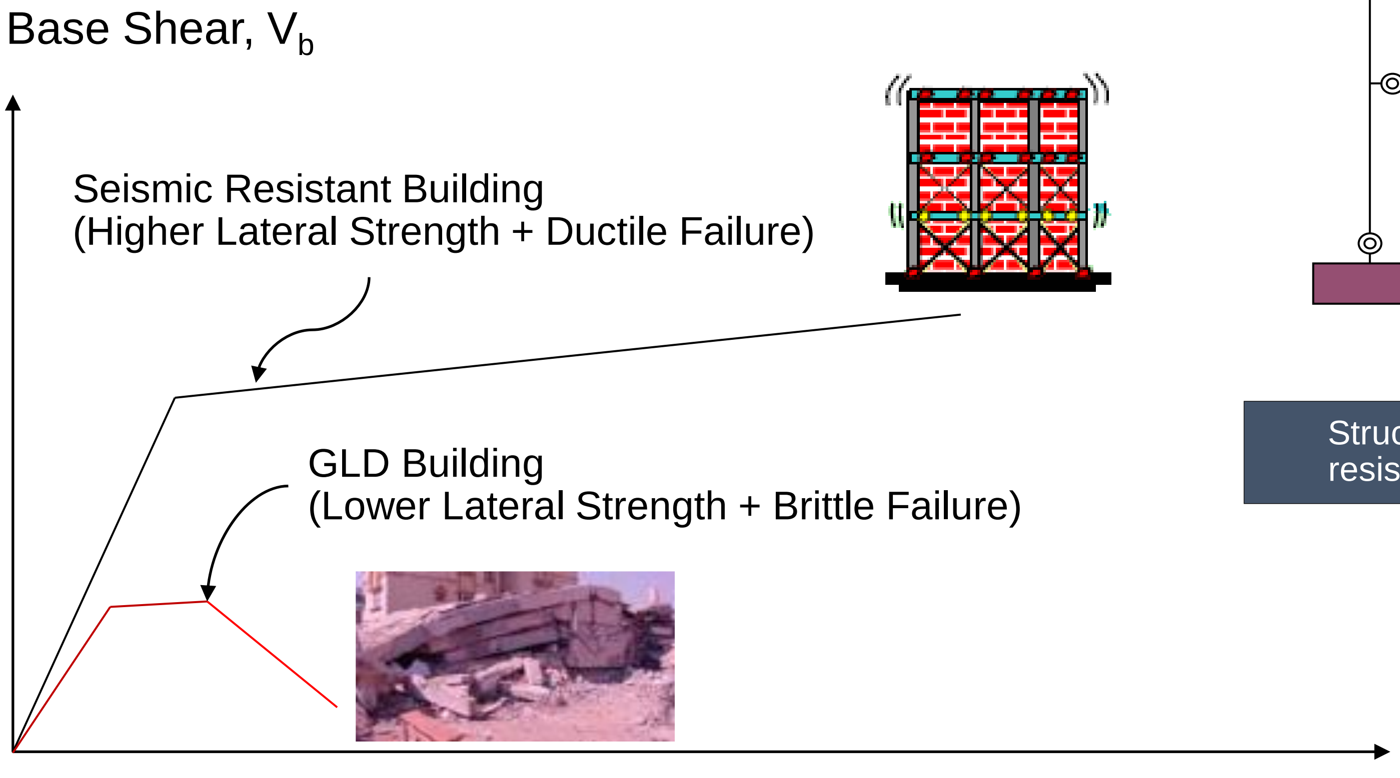
Lap-Splice Failure in Columns

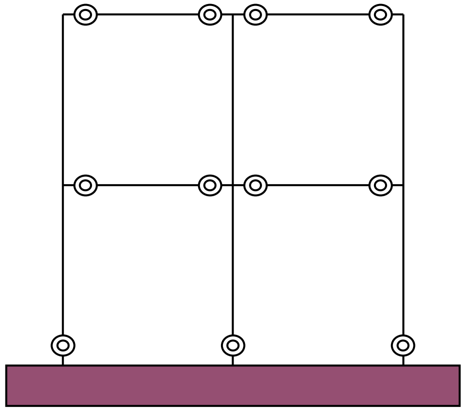
Infill-Failure Induced Soft/Weak Story

Soil Bearing Capacity Failure

Seismic Resistance of RC Buildings







Structural model for seismic resistant designed buildings

Introduction to Nonlinear Analysis

Why Nonlinear Analysis?

- Buildings do not respond as linearly elastic systems during strong ground shaking

- Improve Understanding of Building Behavior
  - More accurate prediction of global displacement
  - More realistic prediction of earthquake demand on individual components and elements
  - More reliable identification of “bad actors”

- Reduce Impact and Cost of Seismic Retrofit
  - Less conservative acceptance criteria
  - Less extensive construction

- Advance the State of the Practice

Why Use Nonlinear Analysis?

- In some cases, the moments predicted by an elastic analysis may be reasonably accurate, but the **deflections** predicted by an elastic analysis may **probably be too small**.

- On the other hand, in some cases, the elastic analysis may overestimate the moments, e.g. if the central support of the structure settles by 4 in (100mm) for instance, a traditional elastic analysis of this case would overestimate the moments caused by the support settlement.

- …Hence, the decision to use nonlinear analysis should be based from a **need that cannot be satisfied by merely linear approximation**.

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 $\sin(\theta) = \theta$.

- 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.

- 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.

Linear vs. Nonlinear Problems

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 Problems

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.

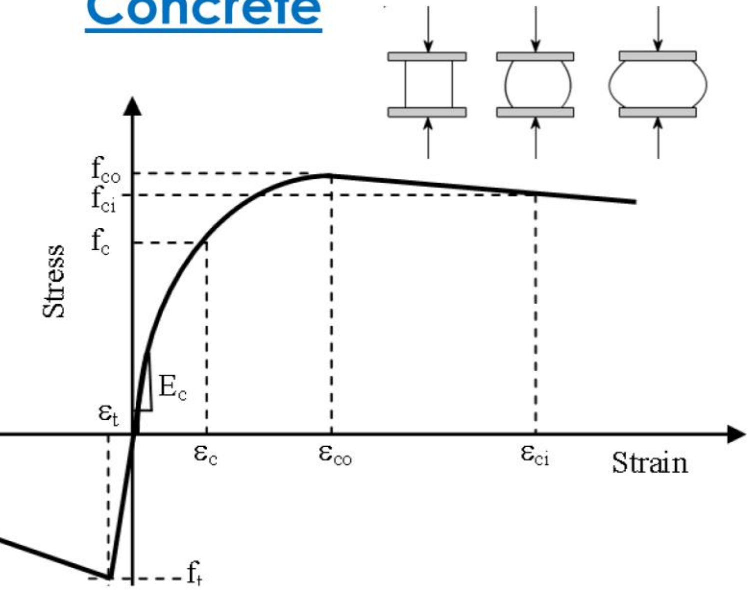
Linear vs. Nonlinear

Computation time	Computation time is relatively small in comparison to nonlinear problems.	Due to the many solution steps required for load increments 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.

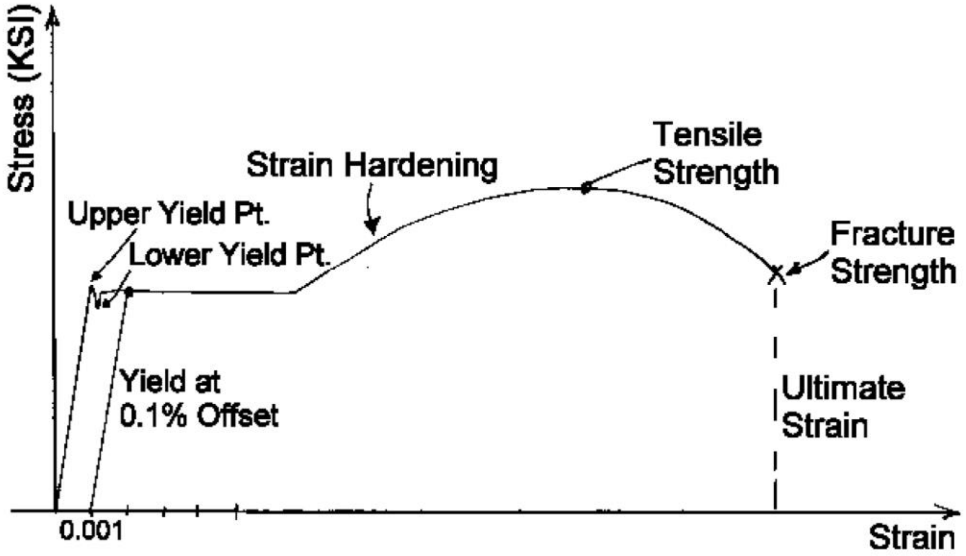
Three Types of Nonlinearity

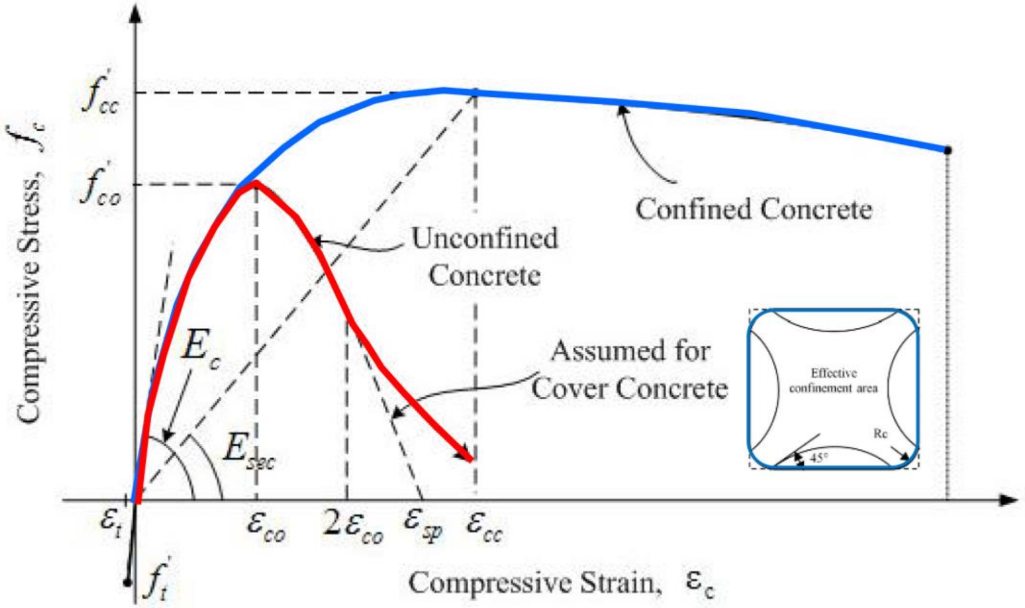
- **Material Nonlinearity**
  - Due to inelastic behavior of constituent materials such as concrete and steel when strained beyond proportional limit resulting to cracking, crushing, sliding, yielding, fracture, etc.
- **Geometric Nonlinearity**
  - Due to Due to change in shape of the structure.
  - 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.

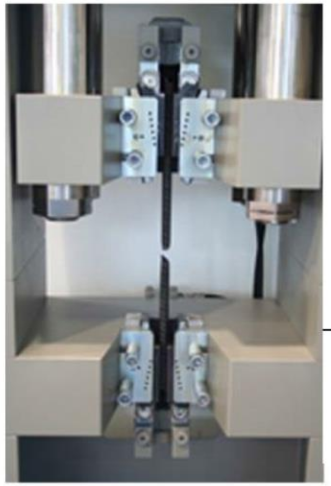
Concrete

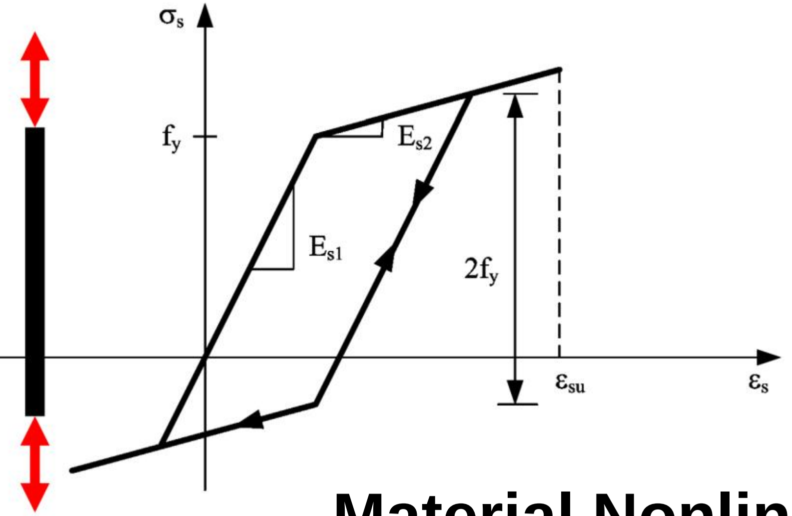


Steel









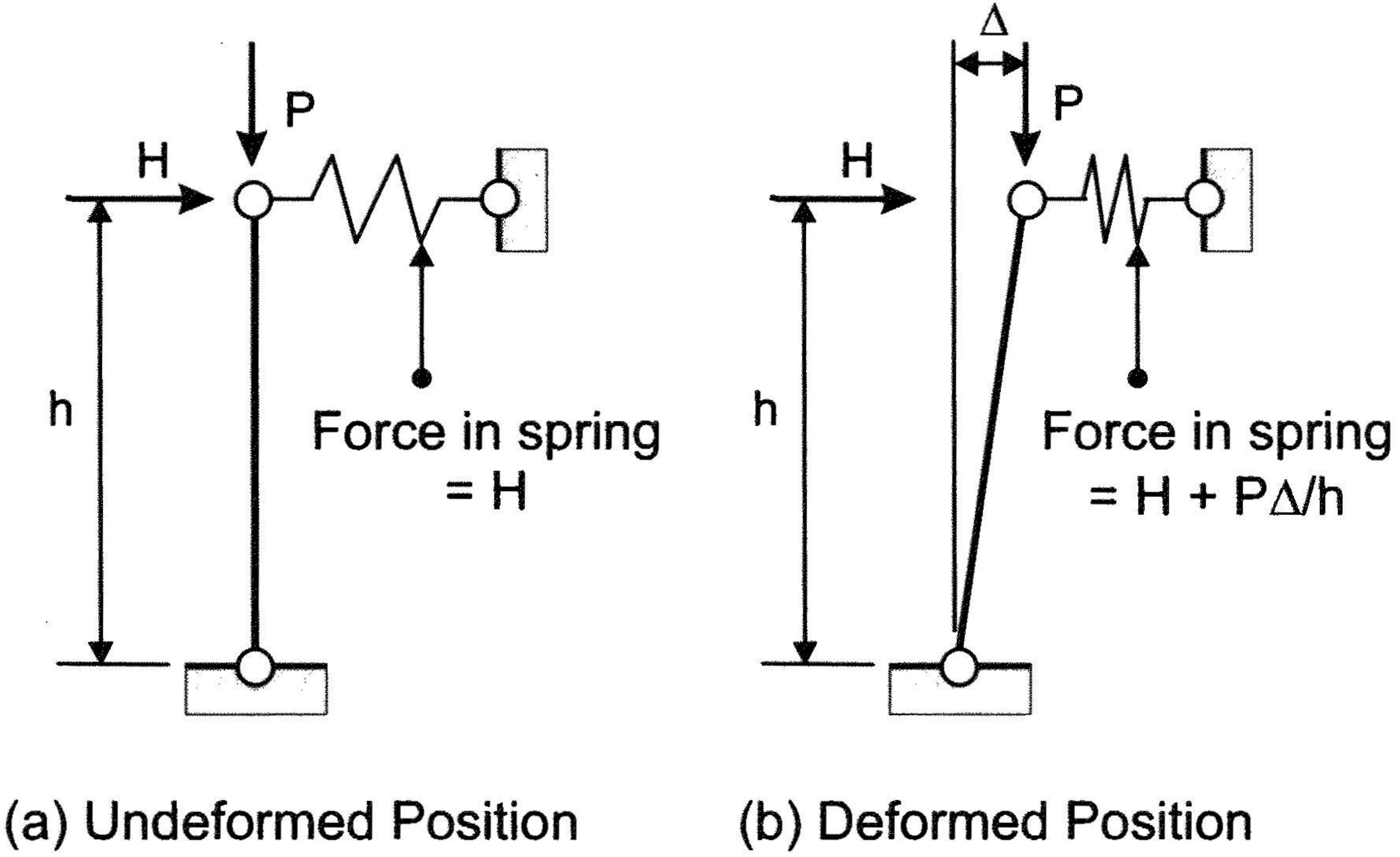
Material Nonlinearity

Causes of Geometric Nonlinearity

- There are two causes of geometric nonlinearity, the first based on equilibrium and the second on compatibility (continuity).
- Geometric nonlinearity occurs when the displacements of a structure are large enough to affect one or both of the following.
  - **(1) The equilibrium relationships.** Equilibrium in the deformed position of the structure may be significantly different from that in the undeformed position.
  - **(2) The compatibility relationships.** The relationships between element deformations and element end displacements may be significantly nonlinear.

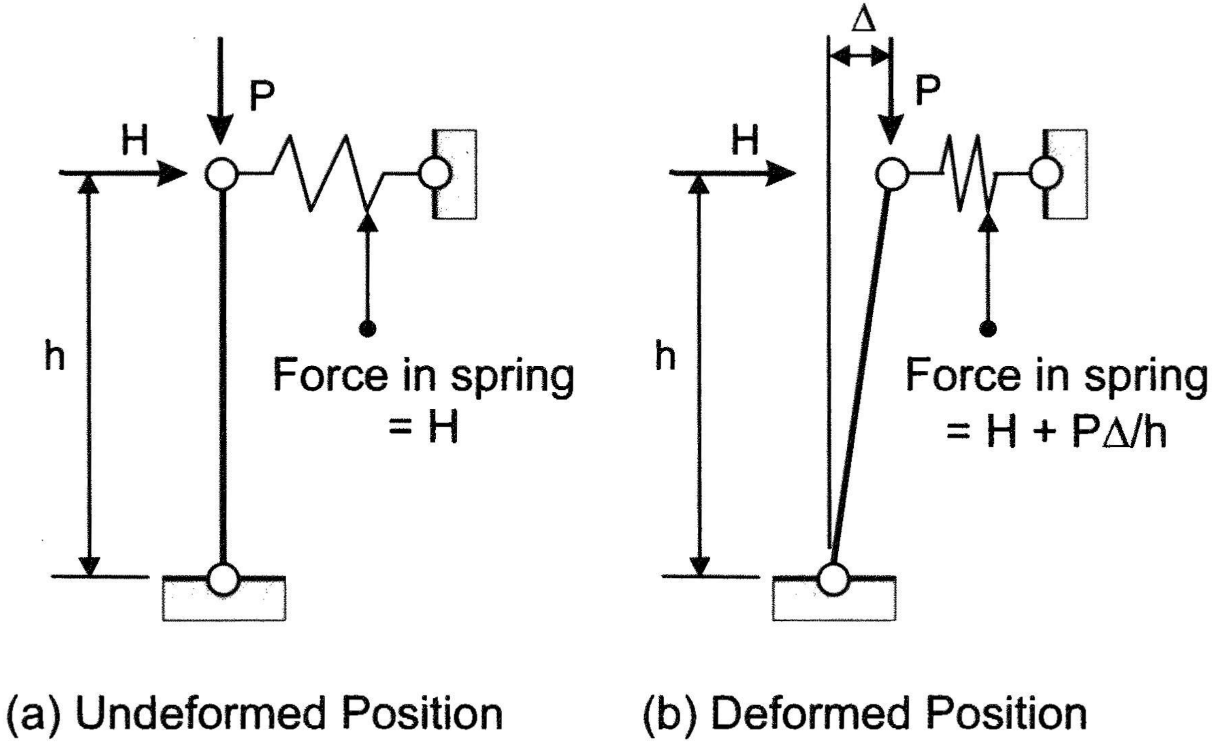
Causes of Geometric Nonlinearity: Equilibrium

- Strictly speaking, equilibrium between external loads and internal forces must be satisfied in the **deformed position of the structure**.
- However, if the displacements are small, it can be a reasonable approximation to consider equilibrium in the initial, undeformed position.
- Since this position is fixed, the equilibrium relationships are linear. For example, doubling the external loads exactly doubles the internal forces (assuming no material nonlinearity).



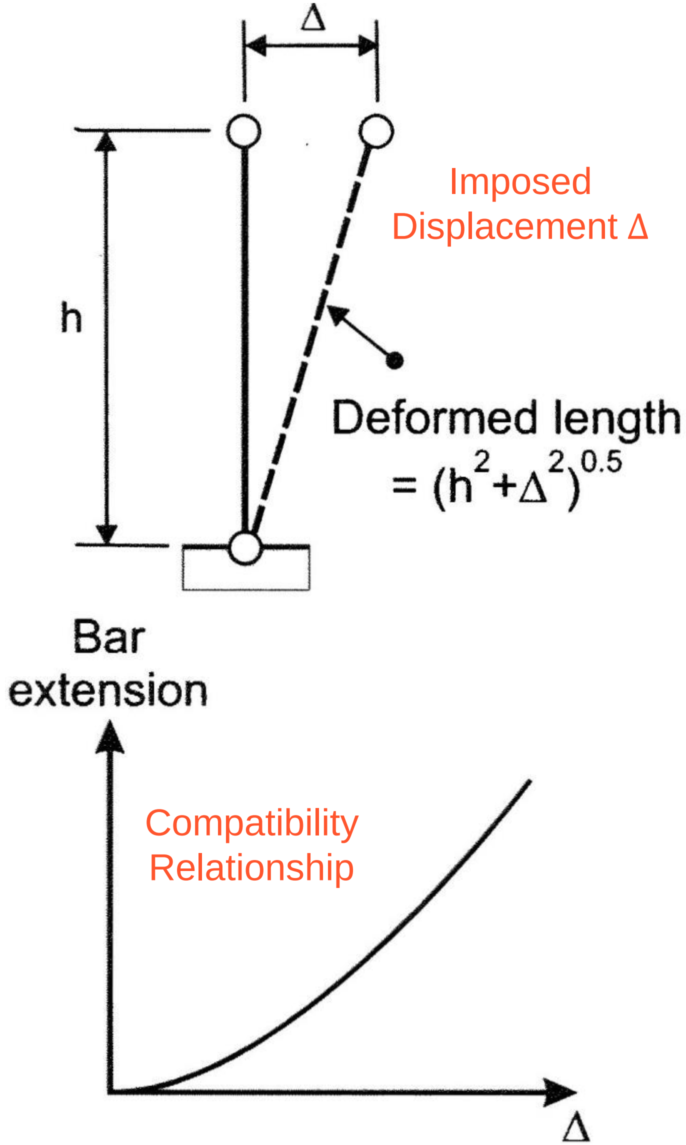
Causes of Geometric Nonlinearity: Equilibrium

- Figure (a) shows the undeformed position. The bending moment at the pinned base must be zero, so by simple equilibrium the force in the spring is equal to the horizontal load.
- Figure (b) shows the deformed position, assuming that the spring compresses and the top of the bar moves horizontally by an amount Δ. Again, the bending moment at the base is zero, so to satisfy equilibrium the force in the spring must be larger than the applied load. Also, the spring force is not proportional to the load. For example, if P and H are doubled, the force in the spring more than doubles.



Causes of Geometric Nonlinearity: Compatibility (Continuity)

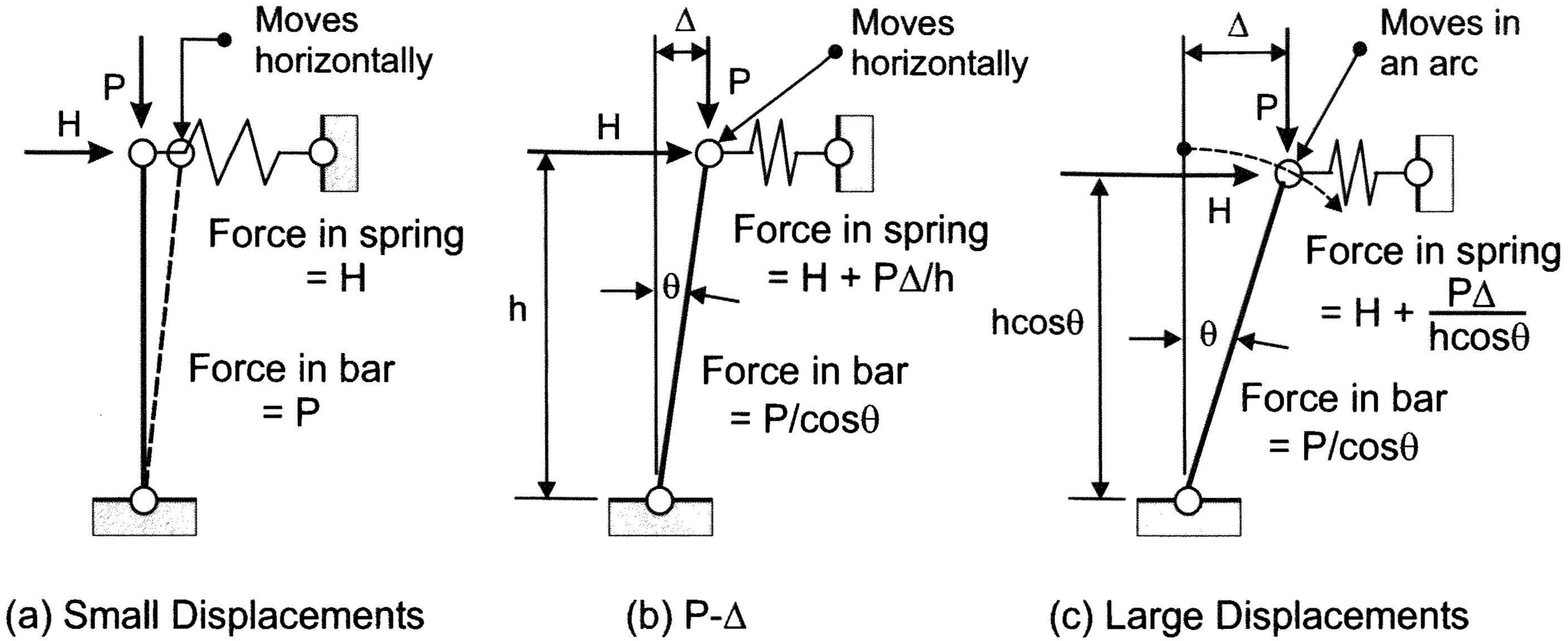
- There is a geometrical relationship between the displacements of a structure and the deformations of its components. Figure shows such a relationship.
- In Figure, the top of the bar moves horizontally. Hence, the bar must extend to maintain continuity. Figure (b) shows the relationship between displacement and bar extension. The bar extension is the deformed length minus the undeformed length, h.
- For a very small horizontal displacement the bar extension is close to zero (in the limit, for a vanishingly small displacement, the bar extension is exactly zero). For larger displacements the bar extends, with a nonlinear relationship between displacement and extension.



Analysis Types to Account for Geometric Nonlinearity

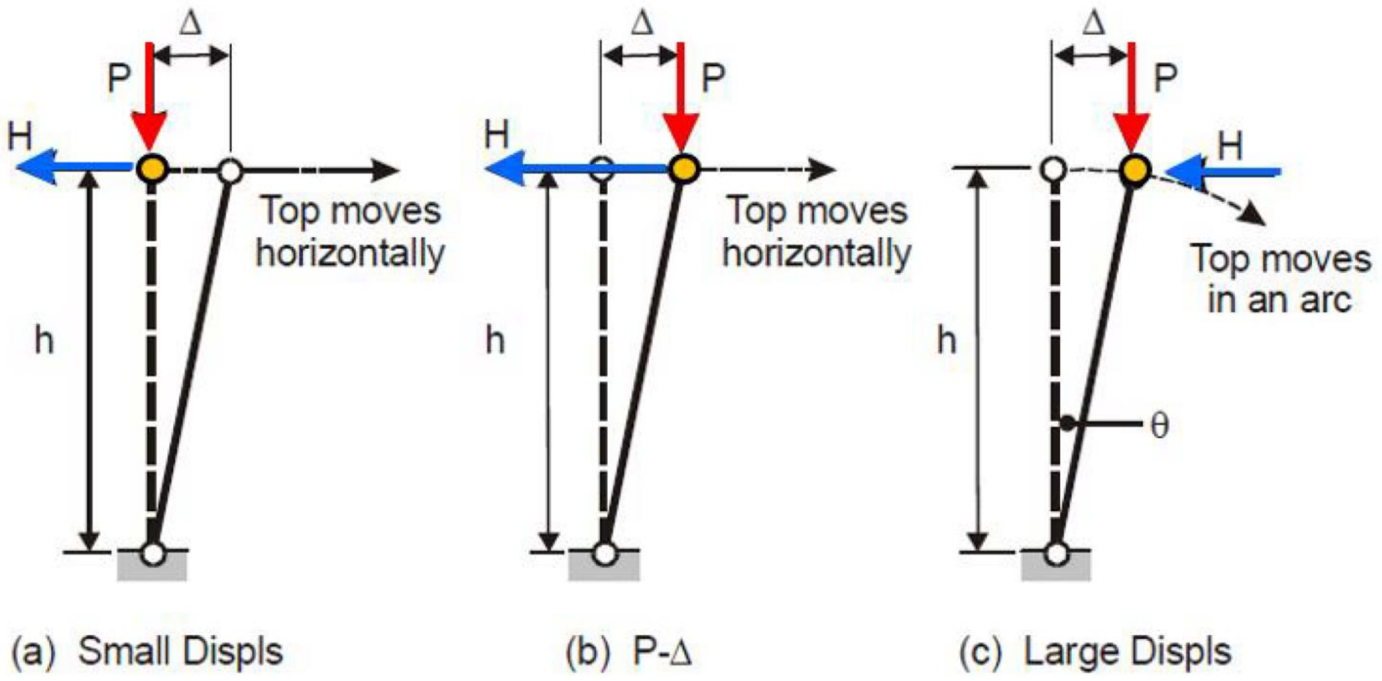
- For analysis, the effects of large displacements on the equilibrium and compatibility relationships can be treated separately.
- Consequently, there are three different types of analysis that can be carried out, as follows.
  - **(1) Small displacements analysis**. This is one extreme. Equilibrium is considered in the undeformed position, and for compatibility the displacements are assumed to be vanishingly small.
  - **(2) True large displacements analysis**. This is the other extreme. Equilibrium is considered in the deformed position, and for compatibility the displacements are assumed to be finite. The compatibility relationships are nonlinear. In this case, geometric nonlinearity is considered with no approximations.
  - **(3) P-Δ analysis**. This is in the middle. Equilibrium is considered in the deformed position (with some minor approximations), and the compatibility relationships are assumed to be linear. In this case, geometric nonlinearity is considered approximately.
- There is a fourth type (deformed position for equilibrium, small displacements for compatibility), but this is never used.

Analysis Types to Account for Geometric Nonlinearity



Assume that the bar is stiff axially, so that it has negligible axial deformation.

Geometric Nonlinearity



Equilibrium	Undeformed position	Deformed position (minor approximations)	Deformed position
Compatibility relationships	Linear	Linear	Nonlinear
Geometric nonlinearity	Ignored	Considered approximately	Considered

Source: Pramin Norachan (AIT Solutions)

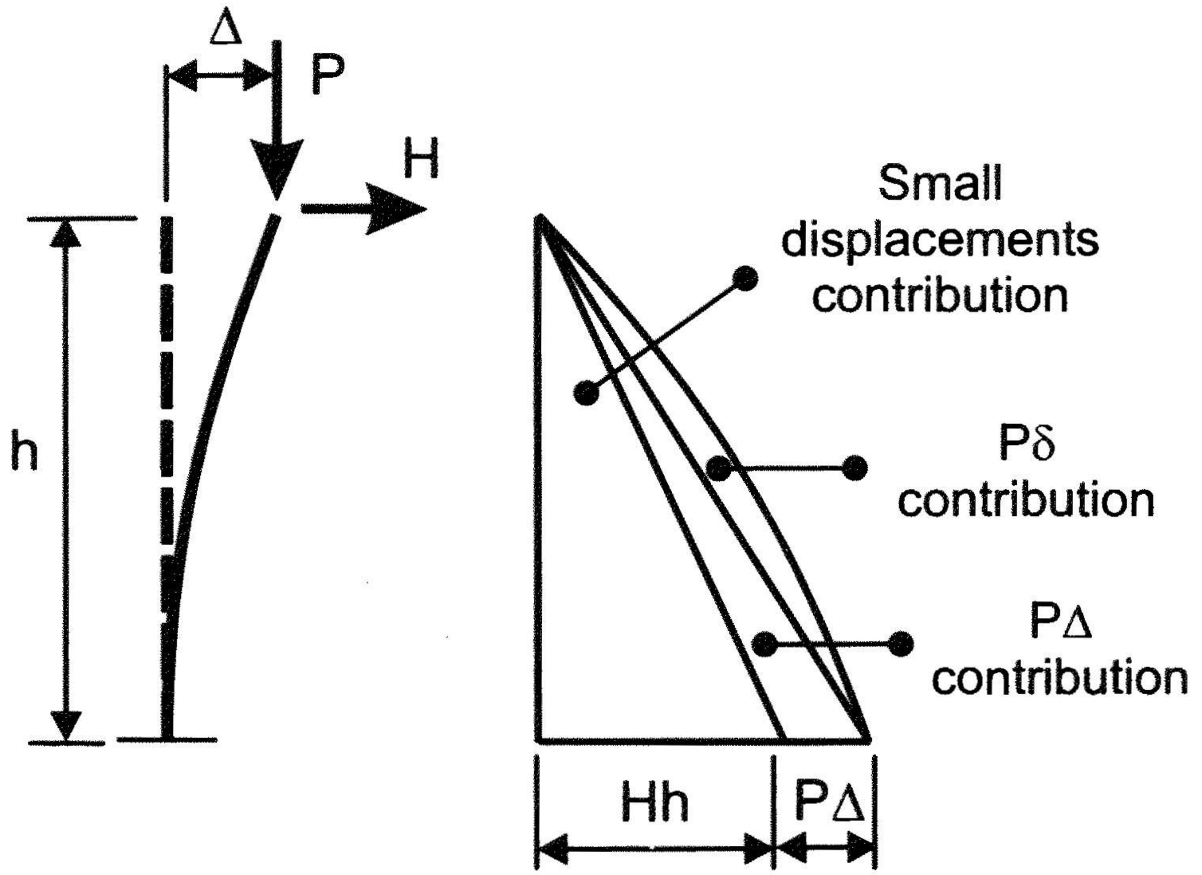
Analysis Types to Account for Geometric Nonlinearity

- The differences among the three methods depend on the relative values of the loads P and H, and on the displacement Δ. Consider two example cases as follows.
    - (1) P = 0, and Δ/h = 0.1 (i.e., 10% drift ratio, which is a very large drift for most structures). For all three methods the force in the spring is H and the force in the bar is zero. The only difference is that the vertical displacement is negligible for small displacements and P-Δ. analysis, and equal to a small value (0.005h) for large displacements analysis.
    - 2) P/H = 5, Δ/h = 0.10. For the small displacements case the forces in the spring and bar are respectively H and P. For the P-Δ case the forces are 1.5H and 0.995P. For the large displacements case the forces are 1.503H and 0.995P. The vertical displacements are essentially the same as for P = 0.
- These examples show that small displacements analysis can be in error when there are substantial gravity loads and large drifts, but only in the force in the spring (in the second example above there is an error of 50% in the spring force).

Practical Guideline to Account for Geometric Nonlinearity

- If geometric nonlinearity must be considered, **it is almost always accurate enough to use P-Δ analysis**.
- Only for very flexible structures, such as cable structures, is it necessary to use large displacements analysis.
- P-Δ analysis is more efficient computationally than large displacements analysis.
- For most structures, it is a waste of computer time to account for true large displacements.

P-Δ vs. P-δ



(a) Column (b) Bending Moments

Geometric Nonlinearity

- The large-stress and large-displacement effects are both termed geometric (or **kinematic**) nonlinearity, as distinguished from material nonlinearity. Kinematic nonlinearity may also be referred to as **second-order geometric effects**.
- For each nonlinear static and nonlinear direct integration time-history analysis, you may choose to consider:
  - No geometric nonlinearity
  - P-delta effects only
  - Large displacement and P-delta effects

Material Nonlinearity

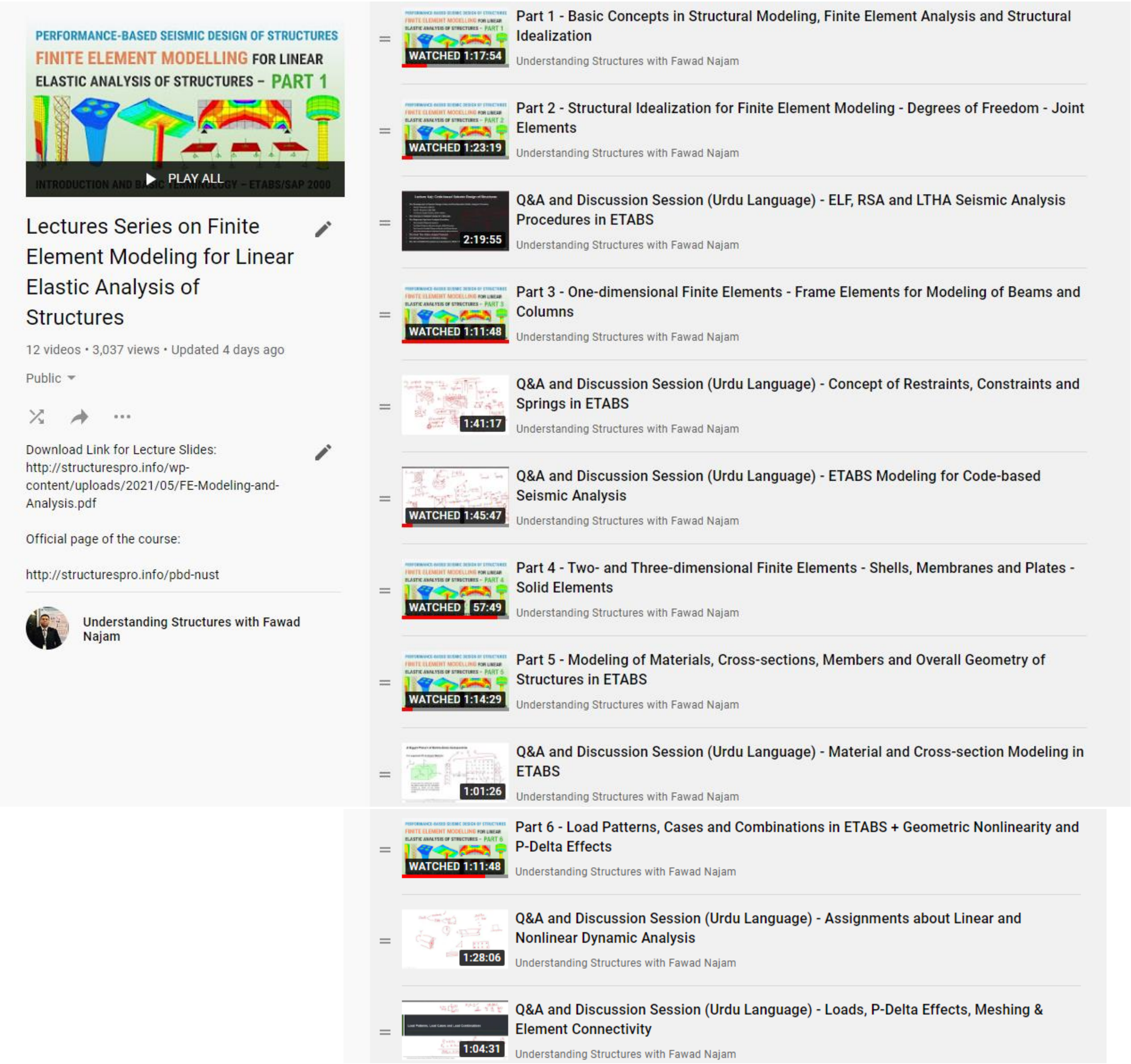
- Material nonlinearity has a wide range of causes, many of which are poorly understood, and it is not governed by any single theory.
- Our knowledge of material nonlinearity depends almost entirely on what we observe in experiments on actual structures and structural components.
- Material nonlinearity is subject to judgment and interpretation.

Geometric Nonlinearity

- Geometrical nonlinearity has clear causes and is governed by a well-defined mathematical theory.
- Geometrical nonlinearity has two well-defined causes (equilibrium and compatibility), both of which are governed by clear mathematical rules
- Geometrical nonlinearity is not subject to judgment and interpretation.
- This does not mean, however, that geometrical nonlinearity is easy to account for in an analysis. Its effects can be complex and subtle, and they can be difficult to capture in an analysis model.

Lectures Series on Finite Element Modeling for Linear Elastic Analysis of Structures

Link: https://www.youtube.com/playlist?list=PL48SKuieCUq9WzNWSgbv44KoAASXukGXe



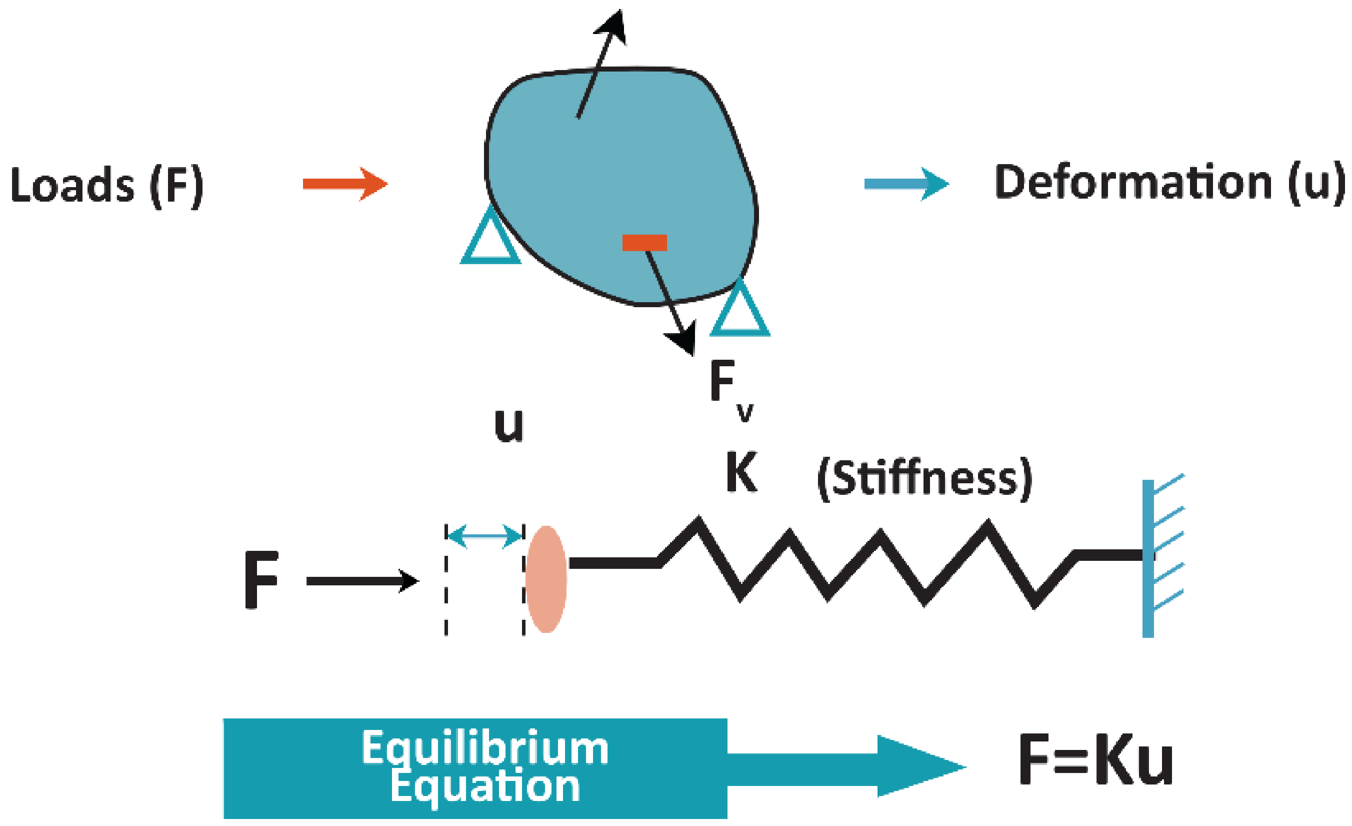
Boundary Nonlinearity

- **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

Important Considerations

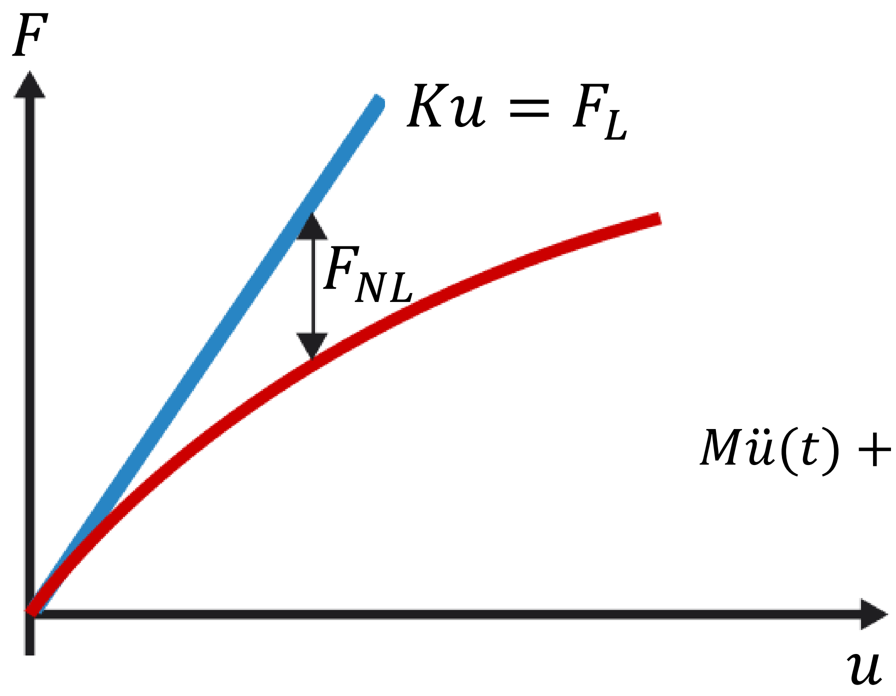
- Nonlinear analysis takes time and patience.
- Each nonlinear problem is different.
- Start simple and build up gradually.
- Run linear static loads and modal analysis first.
- Add hinges gradually beginning with the areas where you expect the most non-linearity.
- Perform initial analyses without geometric non-linearity. Add P-delta effects, and large deformations, much later.

Structure as a Linear Spring



Static Equilibrium

Source of Nonlinear Force

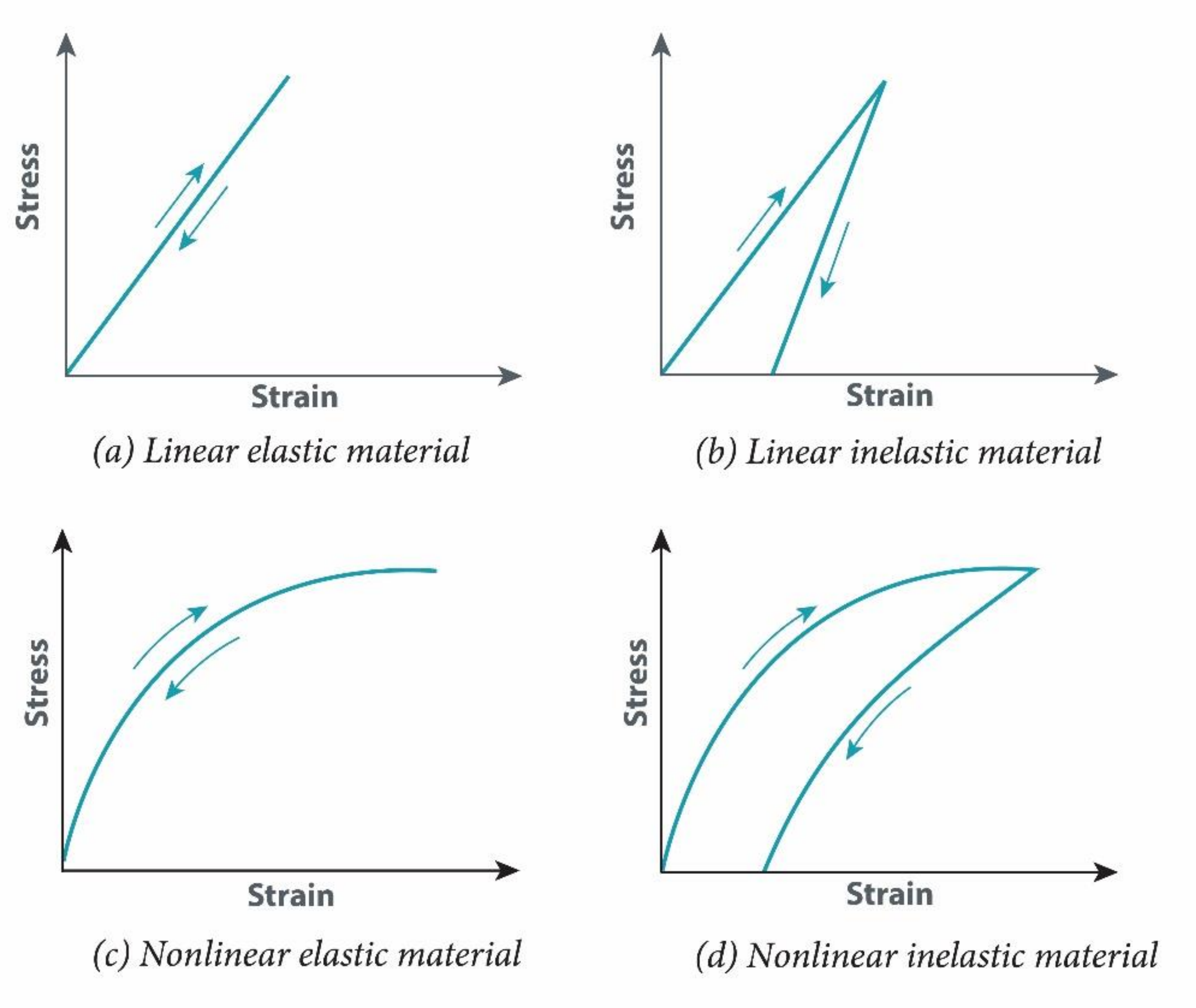


Non-linear Equilibrium

$$Ku - F_{NL} = F$$

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

Linear vs. Non-linear Response



Basic Analysis Types

Excitation	Structure	Response	Basic Analysis Type
Static	Elastic	Linear	Linear-Elastic-Static Analysis
Static	Elastic	Nonlinear	Nonlinear-Elastic-Static Analysis
Static	Inelastic	Linear	Linear-Inelastic-Static Analysis
Static	Inelastic	Nonlinear	Nonlinear-Inelastic-Static Analysis
Dynamic	Elastic	Linear	Linear-Elastic-Dynamic Analysis
Dynamic	Elastic	Nonlinear	Nonlinear-Elastic-Dynamic Analysis
Dynamic	Inelastic	Linear	Linear-Inelastic-Dynamic Analysis
Dynamic	Inelastic	Nonlinear	Nonlinear-Inelastic-Dynamic Analysis

The Basic Equilibrium Equations

- Linear-Static

$$\boldsymbol{Ku} = \boldsymbol{F} \qquad 1$$

- Linear-Dynamic

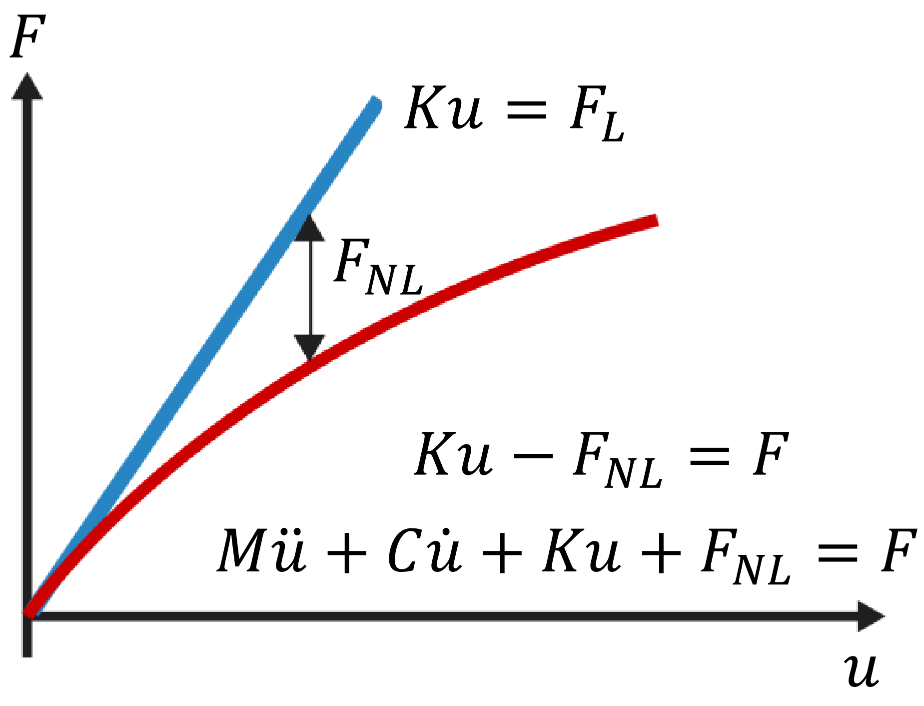
$$\boldsymbol{M\ddot{u}}(t) + \boldsymbol{C\dot{u}}(t) + \boldsymbol{Ku}(t) = \boldsymbol{F}(t) \qquad 2$$

- Nonlinear - Static

$$\boldsymbol{Ku} - \boldsymbol{F}_{NL} = \boldsymbol{F} \qquad 3$$

- Nonlinear-Dynamic

$$\boldsymbol{M\ddot{u}}(t) + \boldsymbol{C\dot{u}}(t) + \boldsymbol{Ku}(t) + \boldsymbol{F}(t)_{NL} = \boldsymbol{F}(t) \qquad 4$$



Comprehensive Equilibrium Equation

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + \boldsymbol{C}\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) + \boldsymbol{F}(t)_{NL} = \boldsymbol{F}(t)$$

- Cover all Static, Dynamic, Elastic, Non-elastic, Damped, Un-damped, Linear, Non-Linear cases and their combinations
- Handles response for
  - Basic Dead and Live Loads
  - Seismic, Wind, Vibration and Fire analysis

Dynamic Equilibrium

Mass-Acceleration

Stiffness-Displacement

External Force

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + \boldsymbol{C}\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) + \boldsymbol{F}(t)_{NL} = \boldsymbol{F}(t)$$

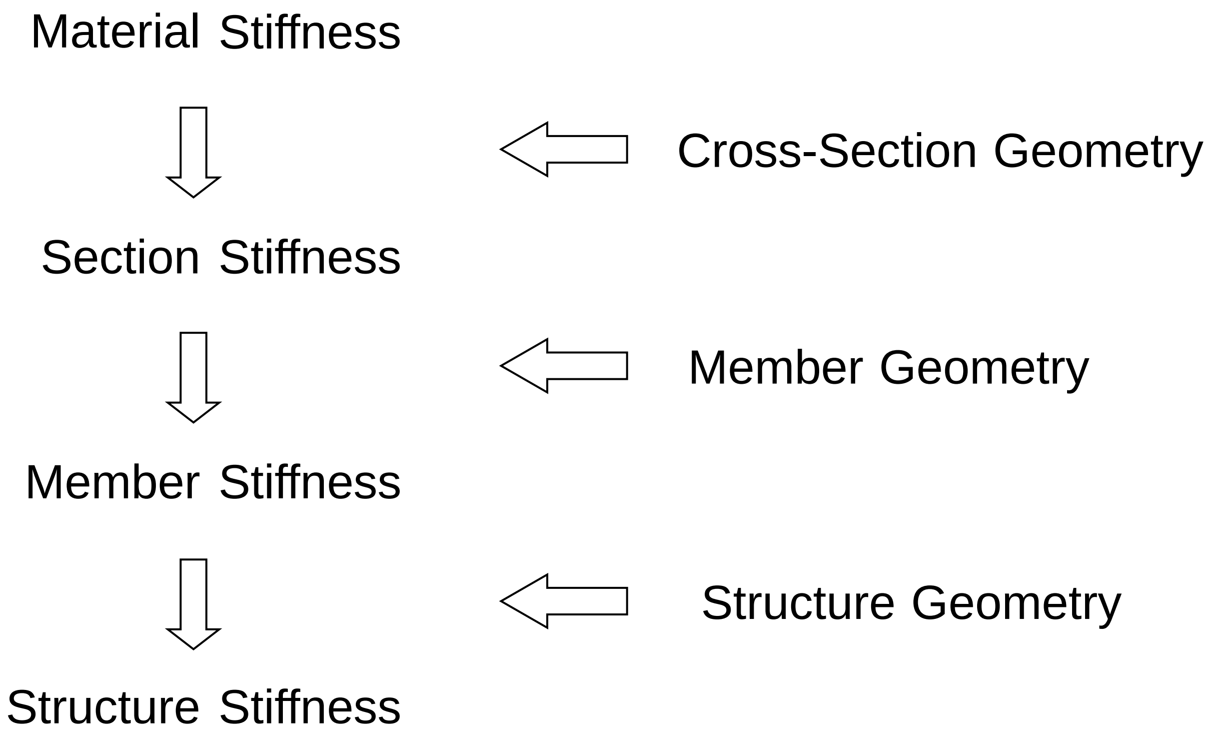
Damping-Velocity

Nonlinearity

The basic variable is displacement and its derivatives

What is Stiffness “made off”?

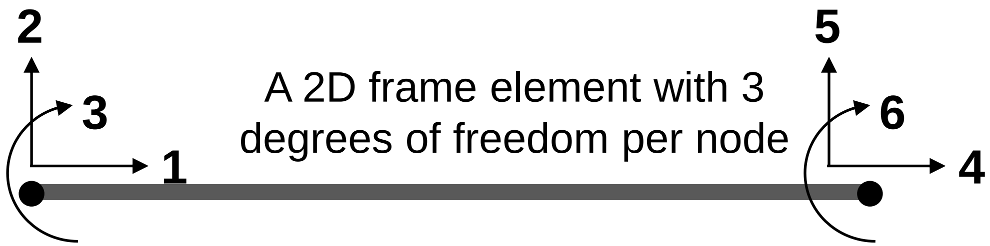
- The overall stiffness of the structure is derived from the overall geometry and connectivity of the members and their stiffness.
- The member stiffness is derived from the cross-section stiffness, and member geometry.
- The cross-section stiffness is derived from the material stiffness and the cross-section geometry.
- All of these stiffness relationships may be linear or nonlinear.



What is Stiffness “made off”?

- The overall resistance of the structures to overall loads, called the **Global Structure Stiffness**.
- This is derived from the sum of stiffness of its members, their connectivity and the boundary or the restraining conditions.
  - The resistance of each member to local actions called the **Member Stiffness** is derived from the cross-section stiffness and the geometry of the member.
  - The resistance of the cross-section to overall strains (**Cross-section Stiffness**). This is derived from the cross-section geometry and the stiffness of the materials from which it is made.
  - The resistance of the material to strain derived from the stiffness of the material particles (**Material Stiffness**).

What is Stiffness (K) “made off”?



$$K = f(E, A, I, L)$$

Material Property (E)

Cross-section Properties (A, I)

Member Property (L)

$$K = \begin{pmatrix} EA/L & 0 & 0 & -EA/L & 0 & 0 \\ 0 & 12EI/L^3 & 6EI/L^2 & 0 & -12EI/L^3 & 6EI/L^2 \\ 0 & 6EI/L^2 & 4EI/L & 0 & -6EI/L^2 & 2EI/L \\ -EA/L & 0 & 0 & EA/L & 0 & 0 \\ & -12EI/L^3 & -6EI/L^2 & 0 & 12EI/L^3 & -6EI/L^2 \\ 0 & 6EI/L^2 & 2EI/L & 0 & -6EI/L^2 & 4EI/L \end{pmatrix}$$

“Actual” Stiffness Estimation Influenced by

- The state of the structure at any given time
  - Damage
  - Deformation
  - Cracking
  - Creep/Shrinkage
  - Stress-state

Material Stiffness

↓ ← Cross-Section Geometry

Section Stiffness

↓ ← Member Geometry

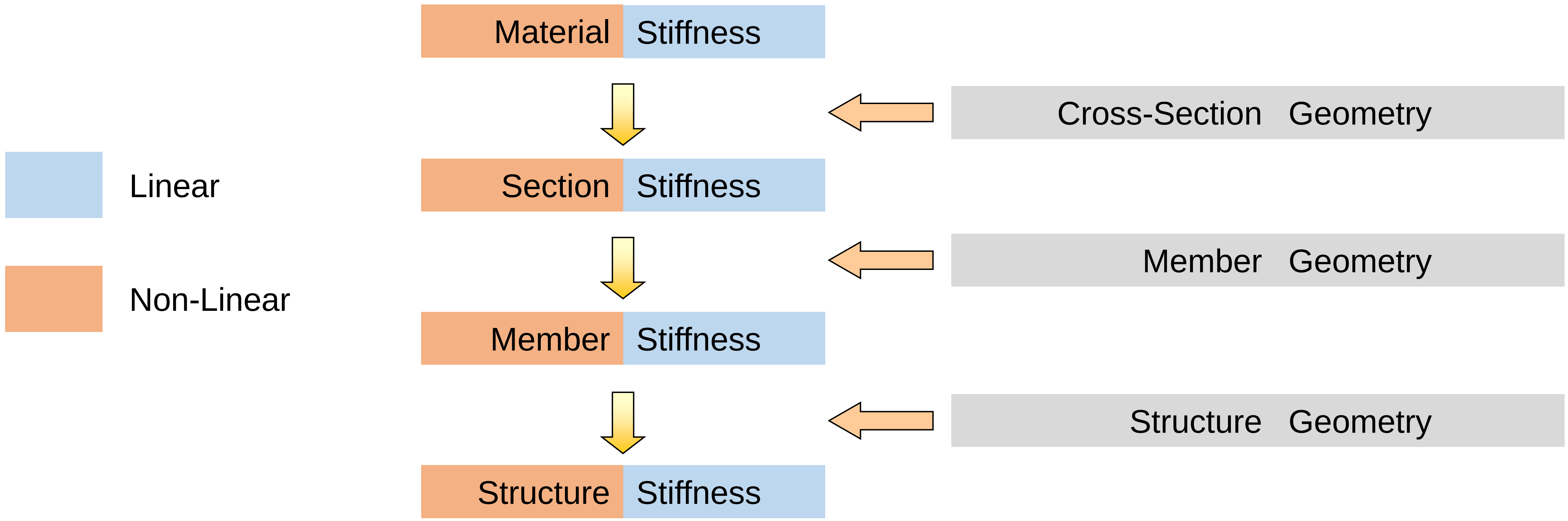
Member Stiffness

↓ ← Structure Geometry

Structure Stiffness

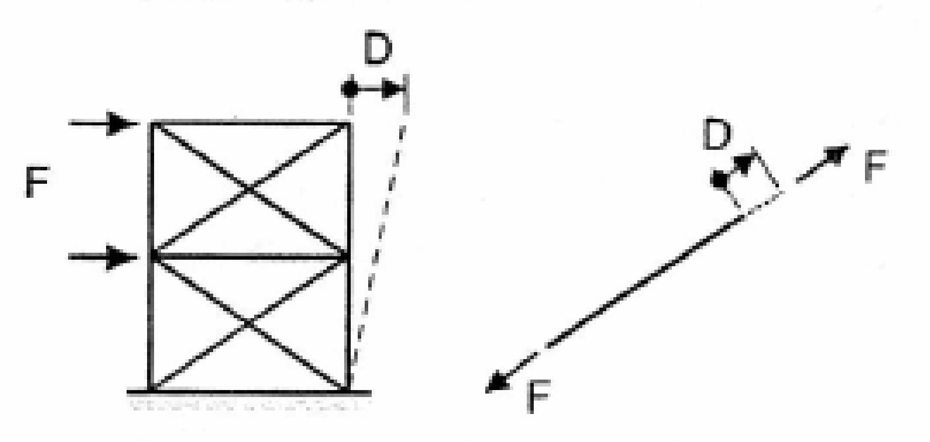
The Structure Stiffness - K

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

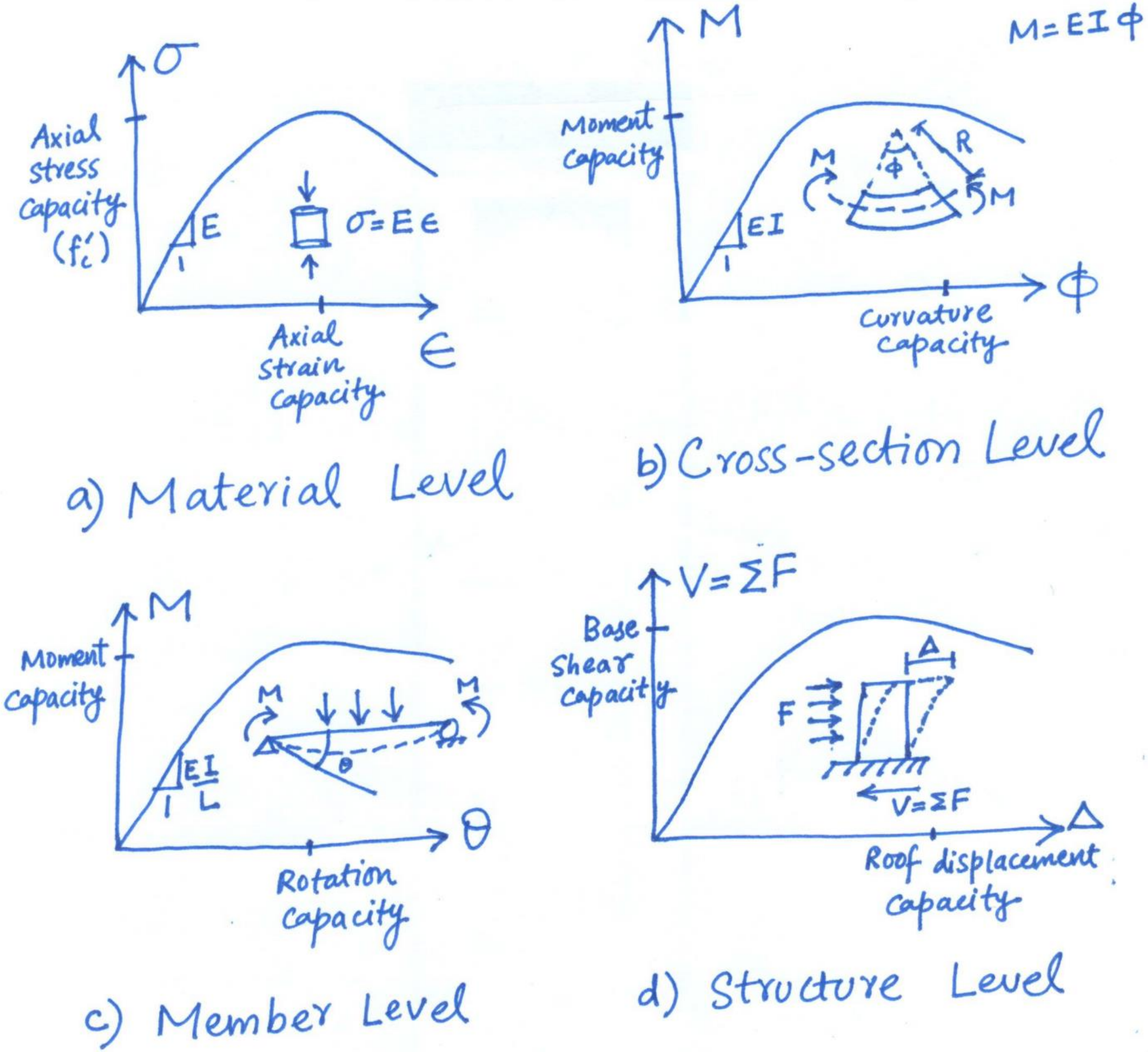


Nonlinear Analysis and Action-Deformation Curves

- 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 (Action-Deformation Curves).
- The Structure F-D relationship is obtained by structural analysis.



Action-Deformation Curves



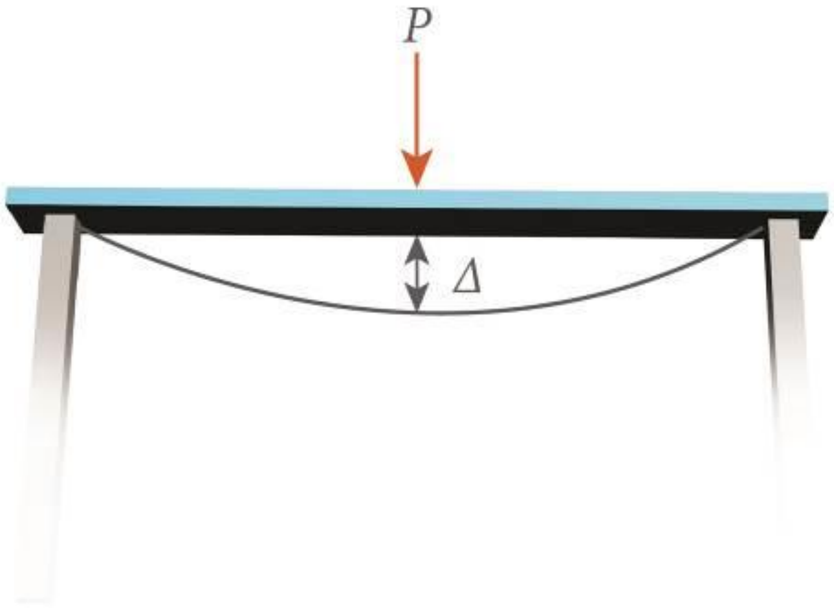
Action-Deformation Curves

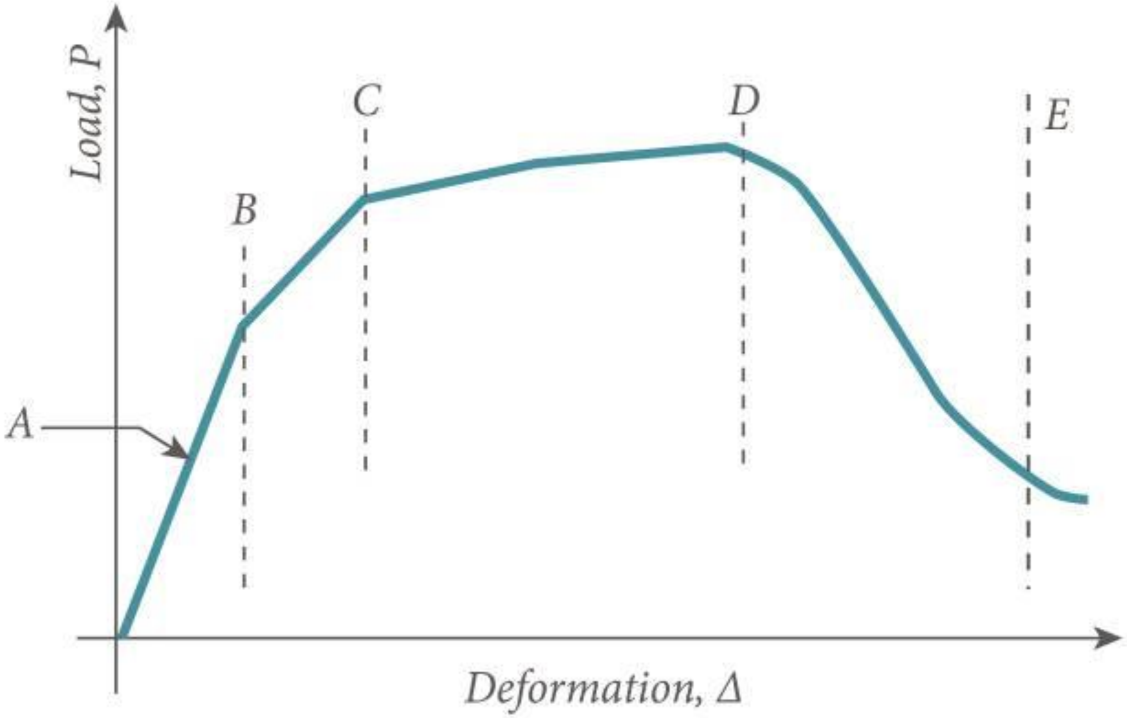
- Relationship between action and corresponding deformation

- These relationships can be obtained at several levels
  1. The Structural Level: Load - Deflection relationship
  2. The Member Level: Moment - Rotation relationship
  3. The Cross-section Level: Moment - Curvature relationship
  4. The Material Level : Stress-Strain relationship

- The Action-Deformation curves show the entire response of the structure, member, cross-section or material

General Force-Displacement Relationship





General Force-Displacement Relationship

Point 'A' corresponds to the serviceability design considerations and working strength or allowable strength design concepts.

Point 'B' is the point up to which the relationship between load and deformation can be considered nearly linear and the deformations are relatively small.

Point 'C' roughly corresponds to the ultimate strength considerations or the design capacity consideration.

Point 'D' is the point at which the load value starts to drop with increasing deformations

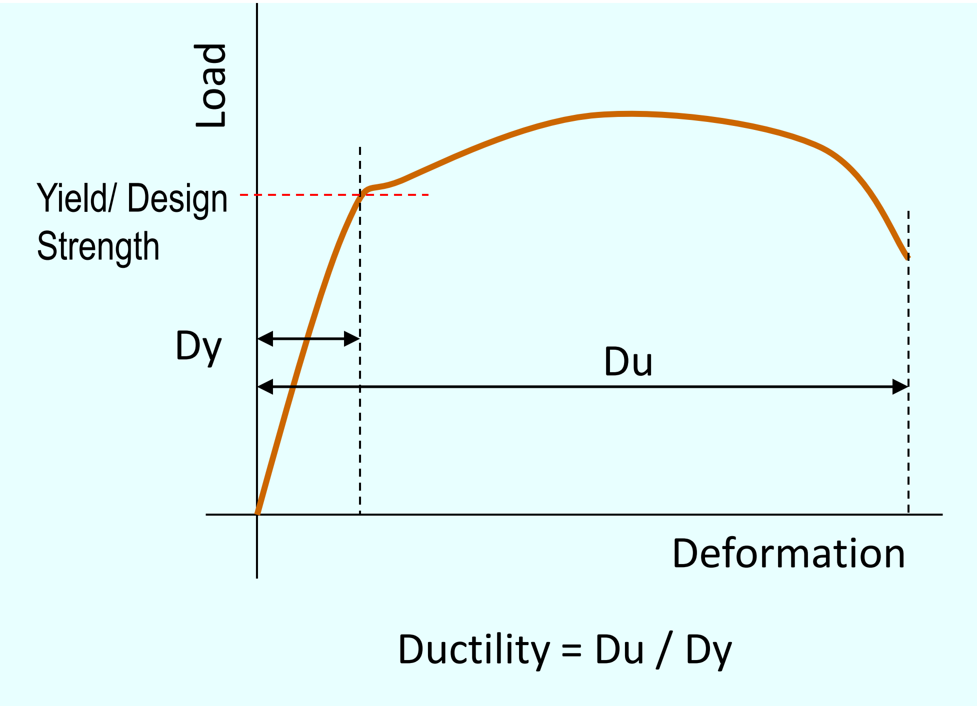
Point 'E' is the point at which the load value is reduced to just a fraction of ultimate load (residual strength)

The Moment-Curvature Curve – An Example of Action-Deformation 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

Ductility – Definition and Usage

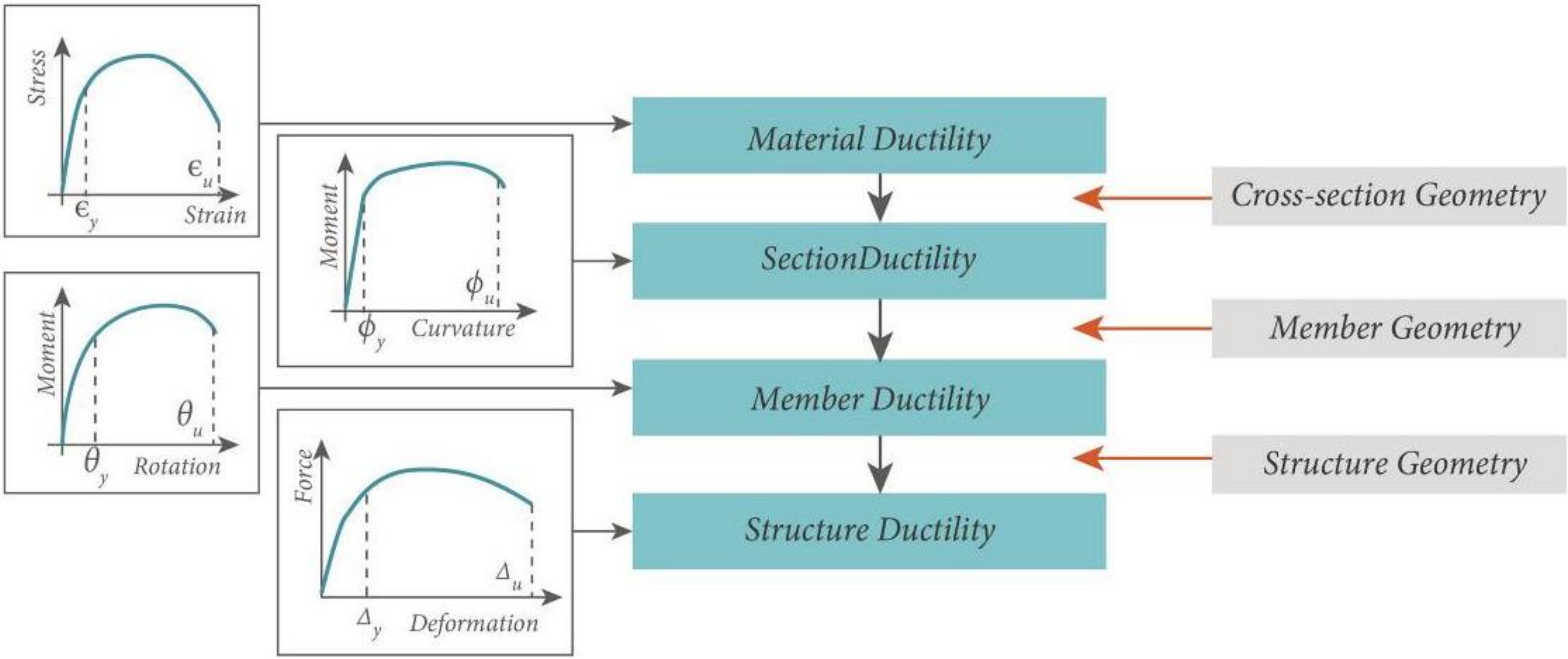
- Ductility can be defined as the “ratio of deformation and a given stage to the maximum deformation capacity”
- Normally ductility is measured from the deformation at design strength to the maximum deformation at failure



What Effects Ductility!

- The most important factor effecting ductility of reinforced concrete cross-section is the **confinement** of concrete.
  - Amount of confinement steel
  - Shape of confinement steel
- Other factors include:
  - Presence of Axial Load
  - Stress-strain curve of rebars
  - Amount of rebars in tension
  - Amount of rebars in compression
  - The shape of cross-section

Ductility Levels



Estimating Stiffness through “Cracking Factors”

- Code specified cracking factors
  - Typical applied to all members
  - At all locations
  - For all load cases

- Not realistic, and subject to considerable variation and debate

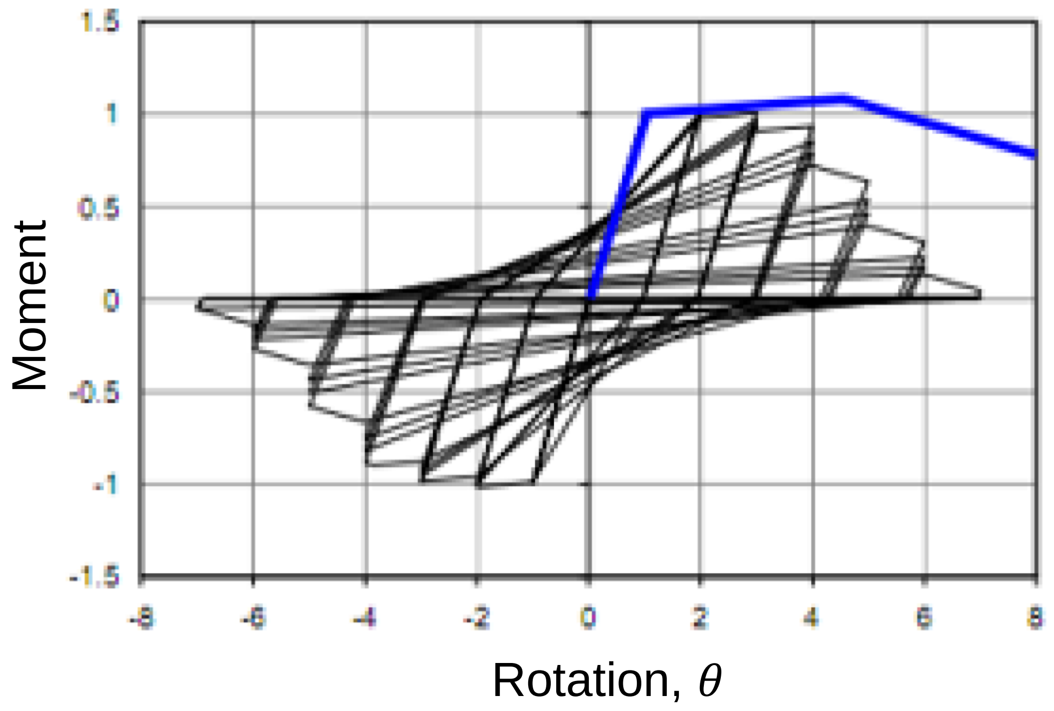
How to Get Action-Deformation Curves

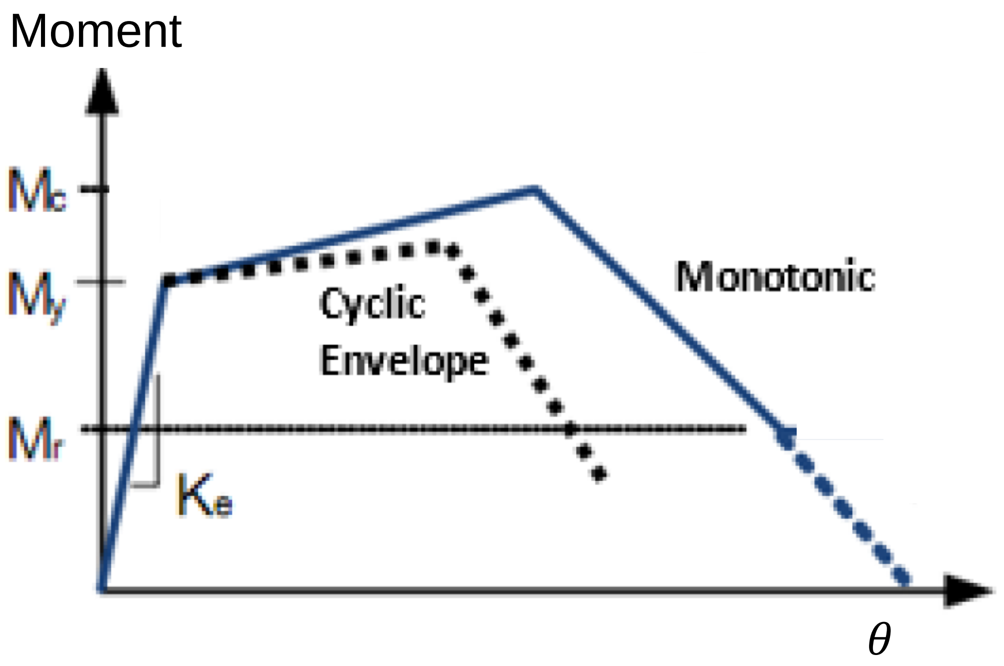
1. **By actual measurements**
   - Apply load, measure deflection
   - Apply load, measure stress and strain

2. **By computations**
   - Use material models, cross-section dimensions to get Moment-Curvature Curves

3. **By combination of measurement and computations**
   - Calibrate computation models with actual measurements
   - Some parameters obtained by measurement and some by computations

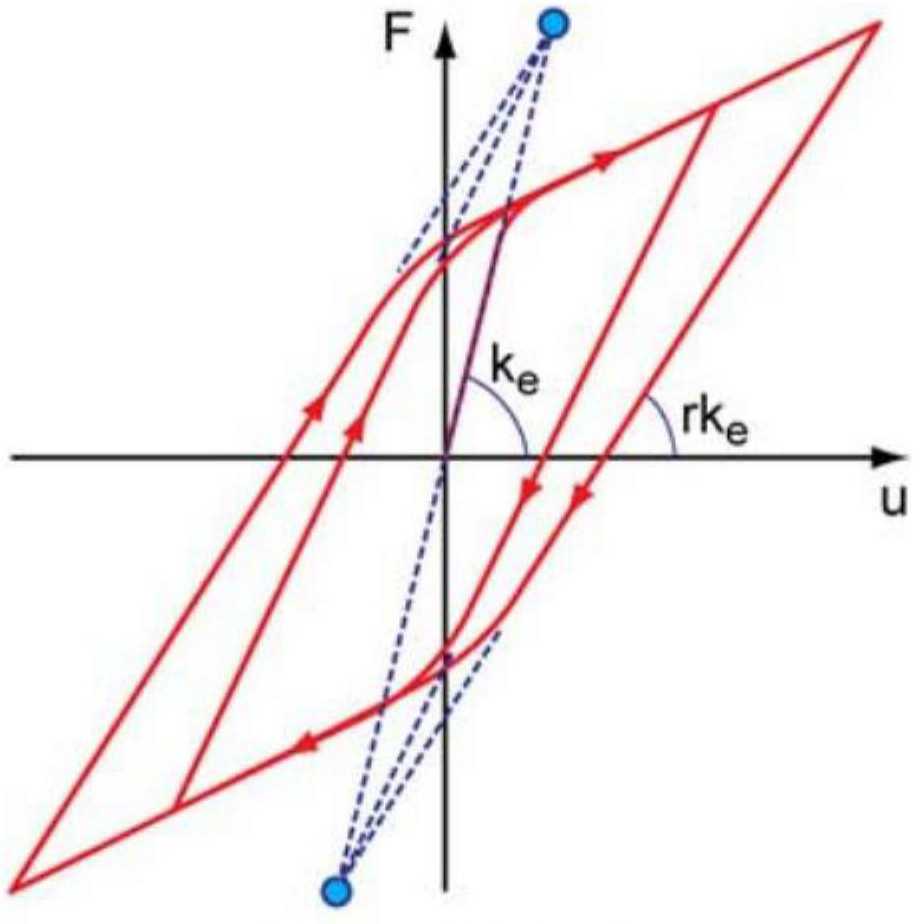
Monotonic vs. Cyclic Action-Deformation Curves





Phenomena Associated with Cyclic Action-Deformation Curves

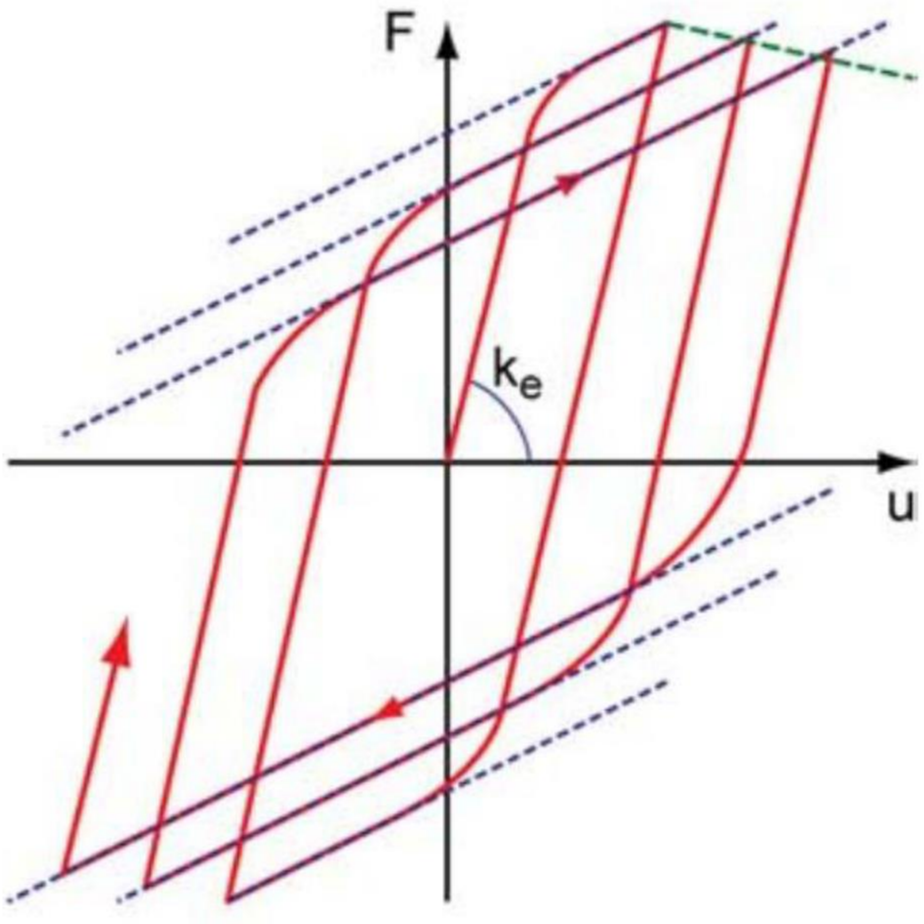
- ***Stiffness degradation*** occurs when the unloading stiffness of the hysteresis loop is smaller than the initial stiffness.
- Stiffness degradation reduces the area of the loop which is a measure of the amount of inelastic energy that is dissipated under cyclic deformation
- So, stiffness degradation also leads to energy degradation.



Model with stiffness degradation

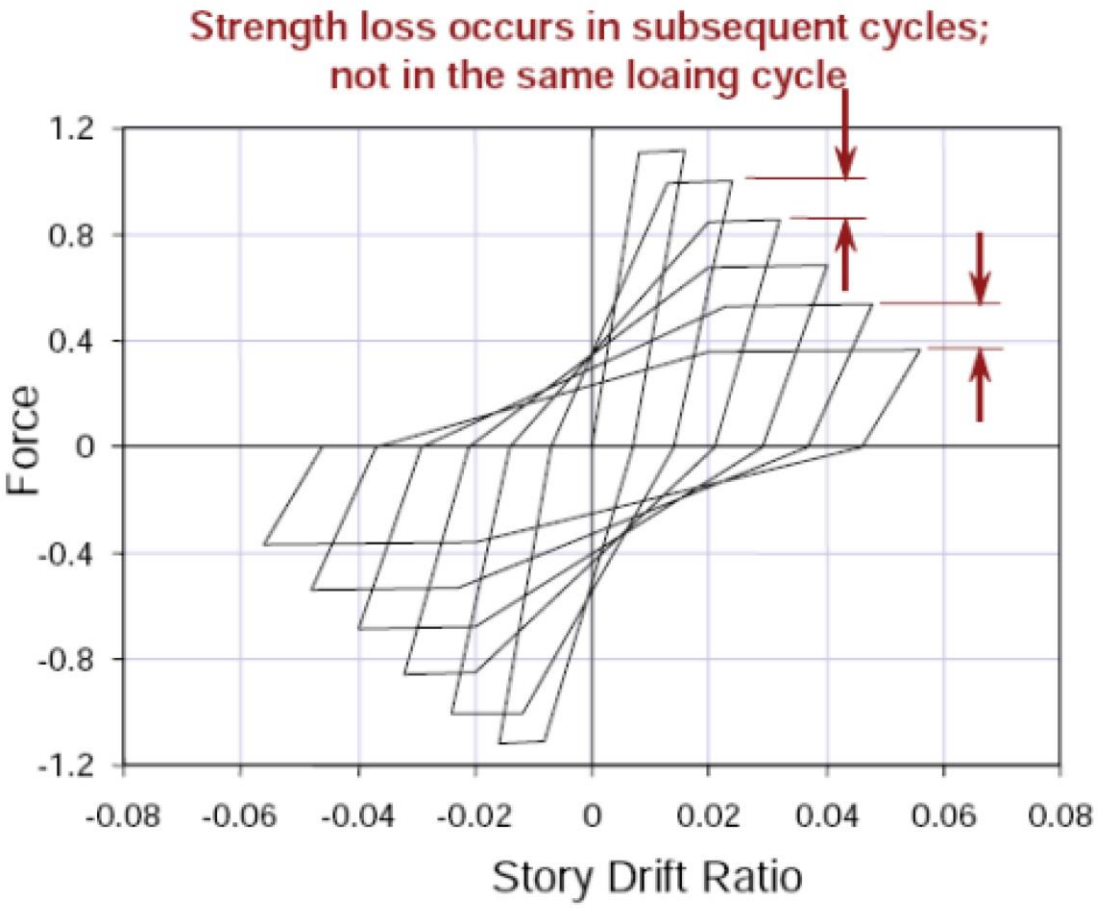
Phenomena Associated with Cyclic Action-Deformation Curves

- ***Strength degradation*** is when there is a reduction in strength when a member is loaded cyclically.
- The strength deterioration generally depends on the number and amplitude of cycles.
- Strength degradation can be "***In-Cycle***" or "***Cyclic***".

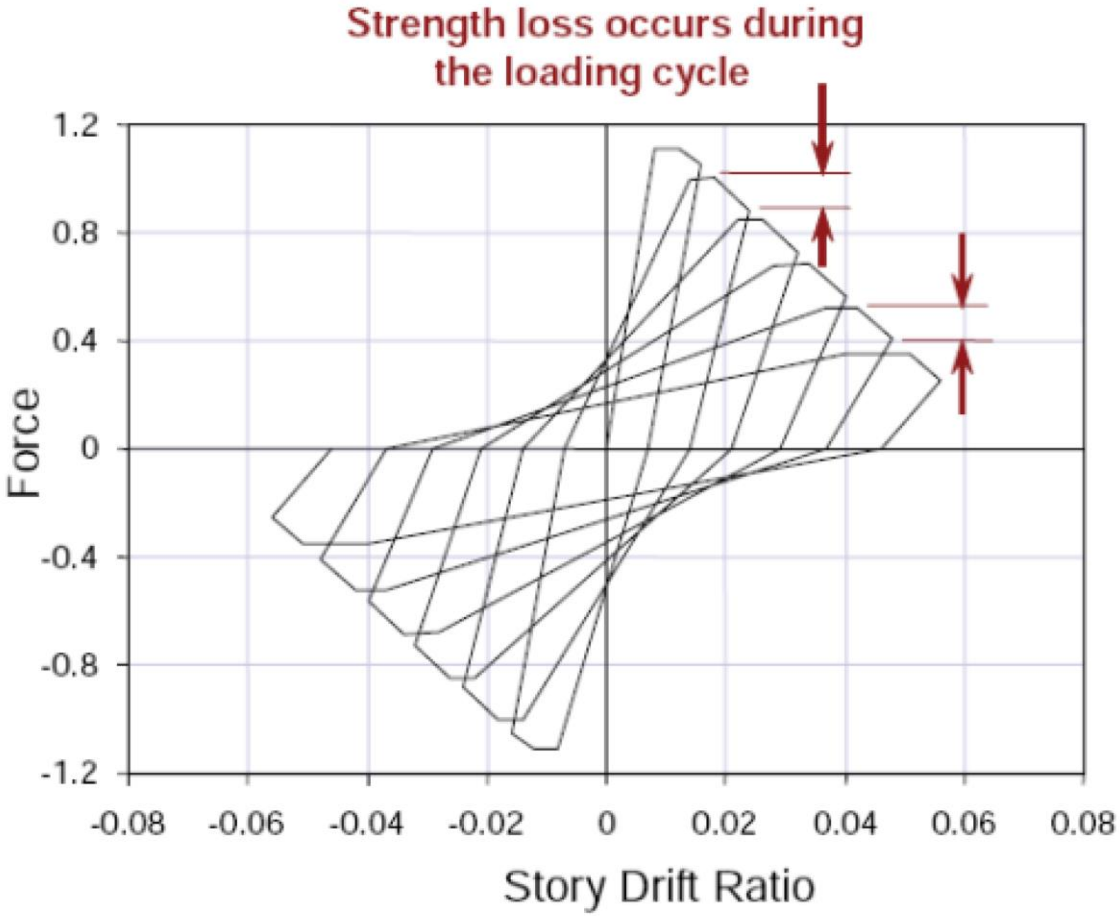


Model with cyclic strength degradation

Phenomena Associated with Cyclic Action-Deformation Curves



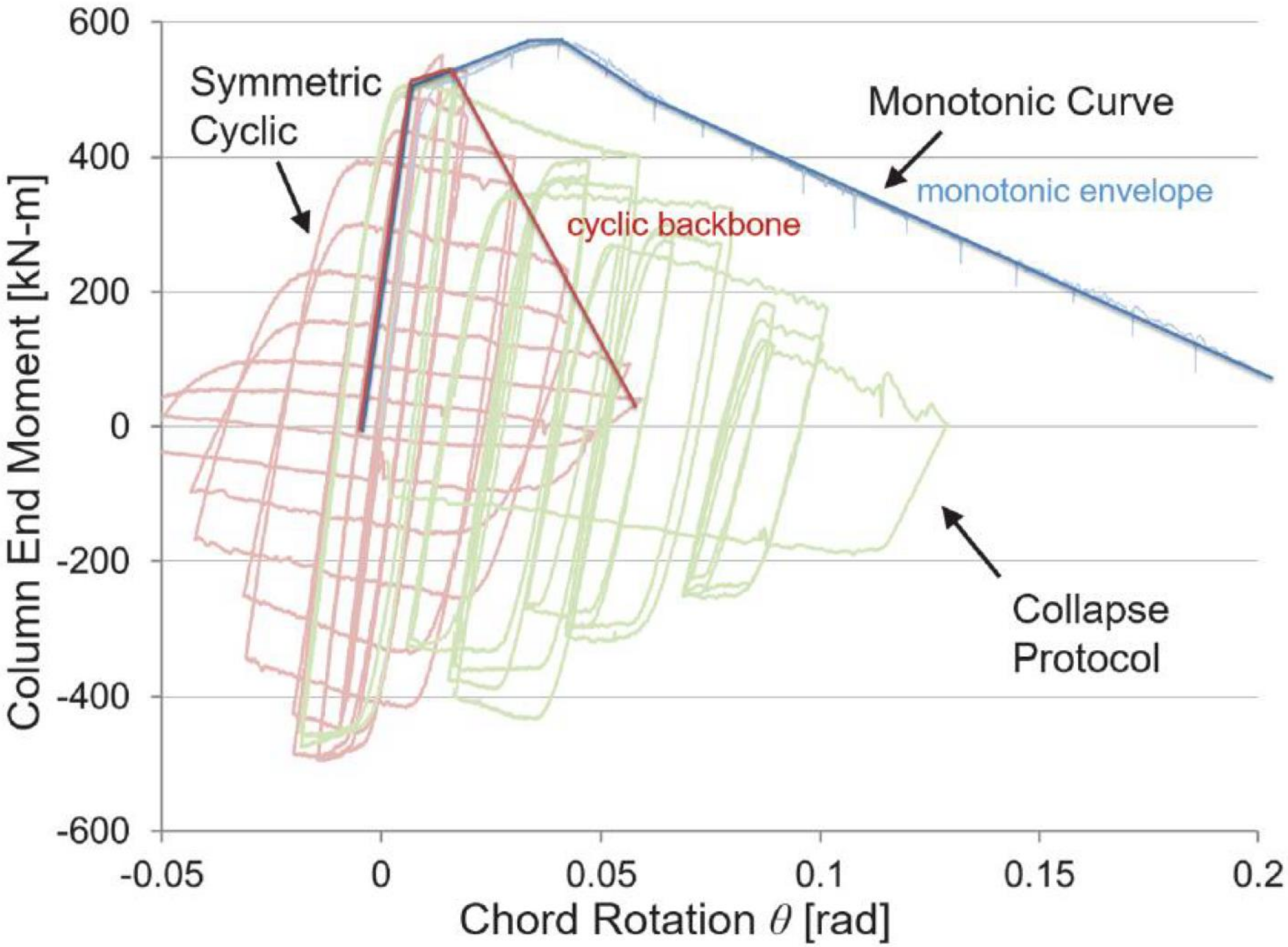
Cyclic strength degradation



In-cycle strength degradation

Phenomena Associated with Cyclic Action-Deformation Curves

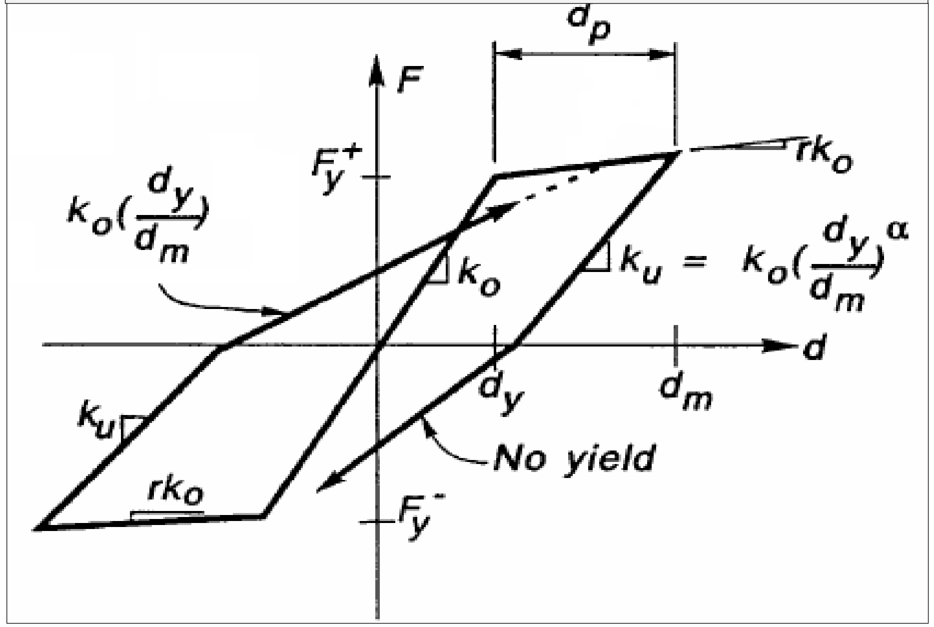
Ductility Degradation

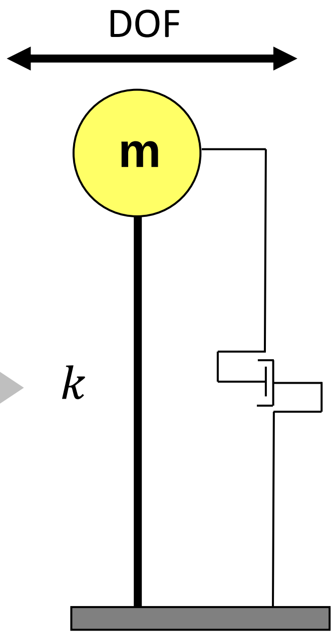


Source: NIST GCR 17-917-46v1

Nonlinear Analysis of an SDF System

Assumed Nonlinear Force-Deformation Behavior

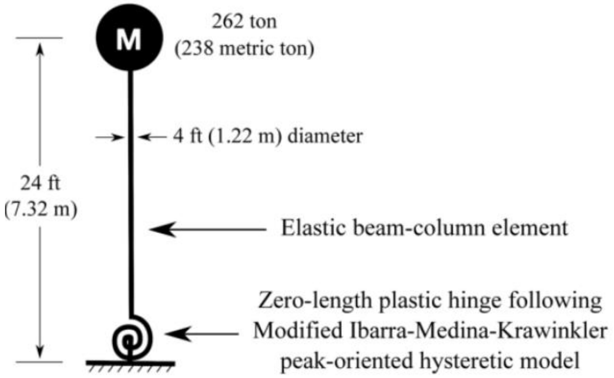


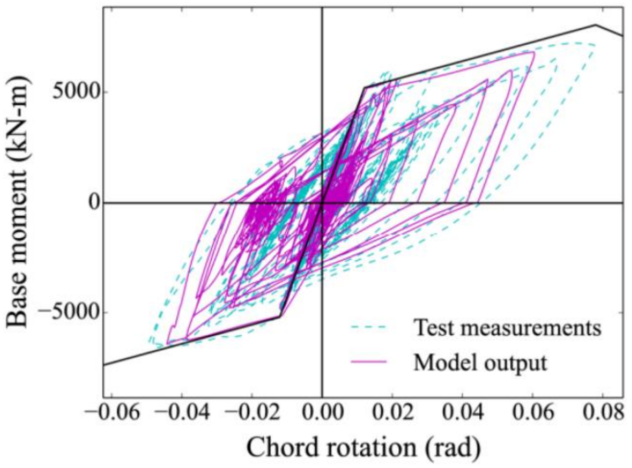


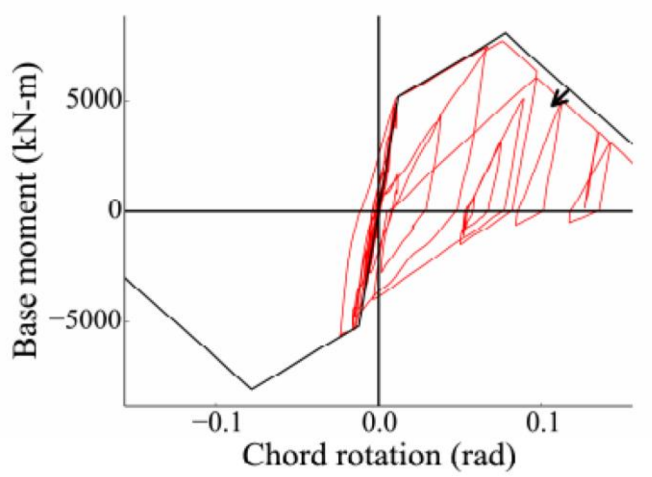
Damping 5% Assumed

Effect of Cyclic Degradation on Structural Response

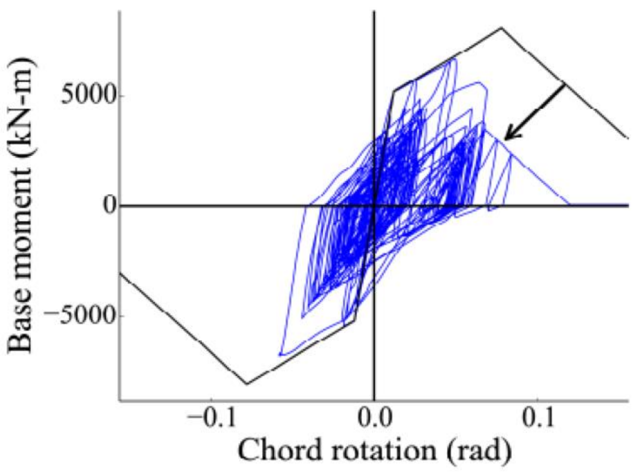
- To investigate the interaction of structural characteristics with the effect of ground motion duration, a bridge pier structure was employed.
- The parameters of the model were calibrated to experimental measurements, the results of which are compared.
- The spectrally equivalent, long and short duration record sets were used in this study.
- The structure subjected to the long duration ground motion experiences a larger number of hysteretic cycles, it deteriorates more, and thus, collapses at a lower ground motion intensity when compared to the short duration ground motion.







Short Duration



Long Duration

Source: Pramin Norachan (AIT Solutions)

Seismic Analysis Procedures

Linear Static Procedures

- Equivalent Static Analysis Procedure

Nonlinear Static Procedures

- Capacity Spectrum Method
- Displacement Coefficient Method
- Various Other Pushover Analysis Methods

Linear Dynamic Procedures

- Response Spectrum Analysis
- Linear Response History Analysis (Direct Integration)
- Linear Modal Response History Analysis

Nonlinear Dynamic Procedures

- Nonlinear Response History Analysis

Results from Nonlinear Analysis

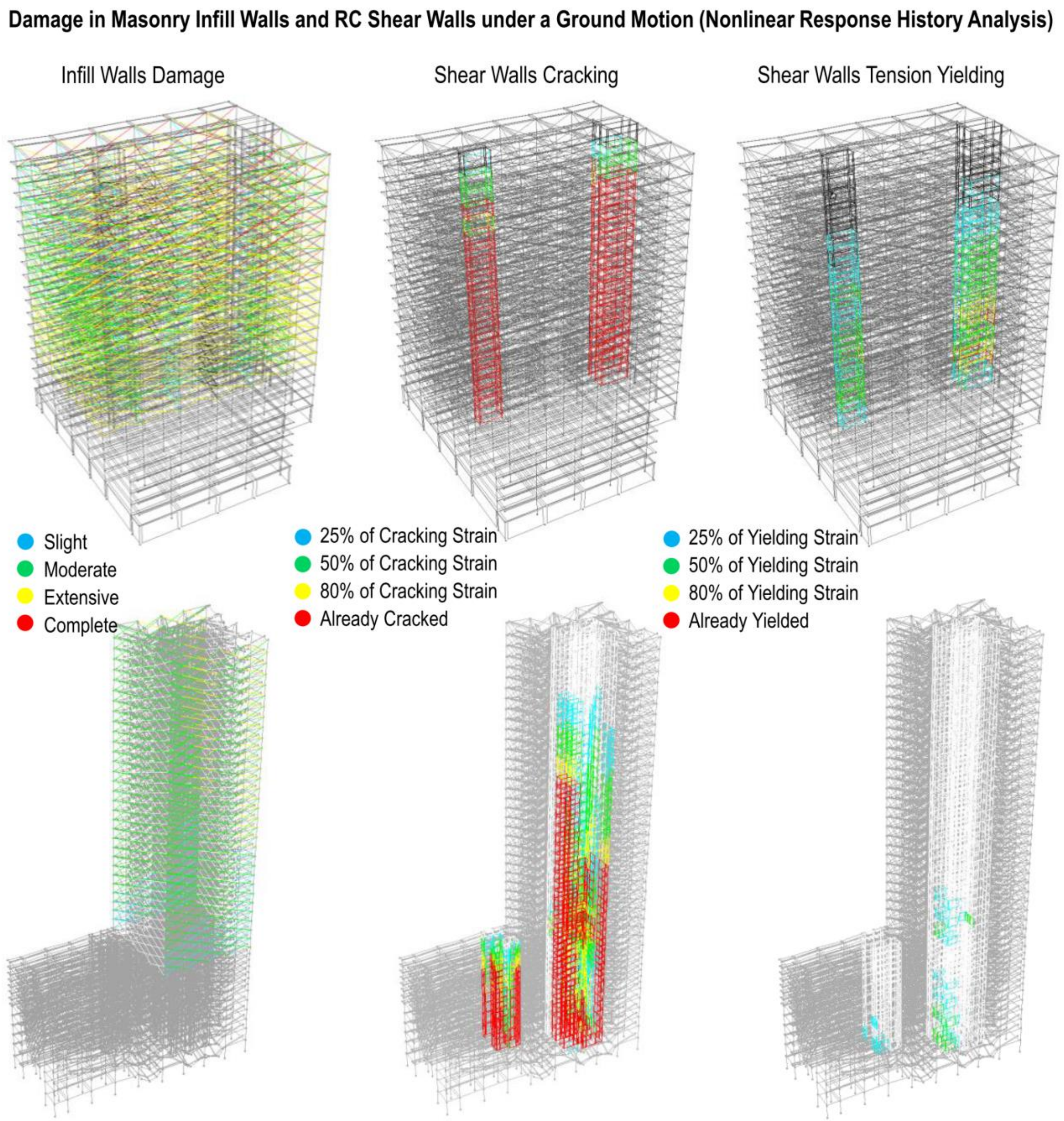


Figure 1-1: An example of structural damage characterized by the strain demand-to-capacity ratios as obtained from the nonlinear response history analysis procedure. The damage in masonry infill walls and RC shear walls under an example ground motion is shown.

Results from Nonlinear Analysis

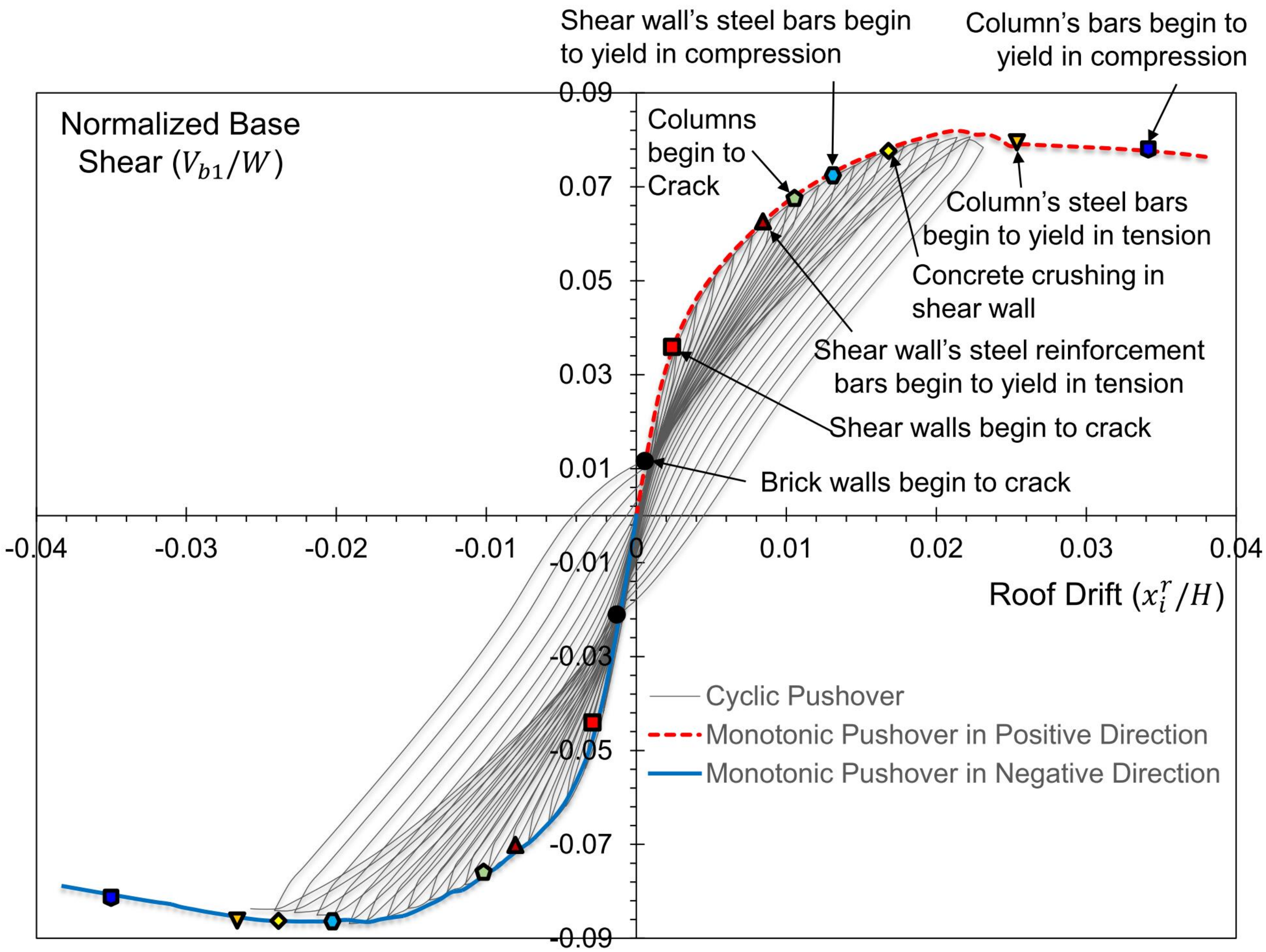


Figure 1-2: An example of the progression of structural damage as obtained from the monotonic and reversed-cyclic pushover analysis procedures.

Nonlinear Static and Dynamic Analysis Results

Software Demonstration

PERFORM 3D

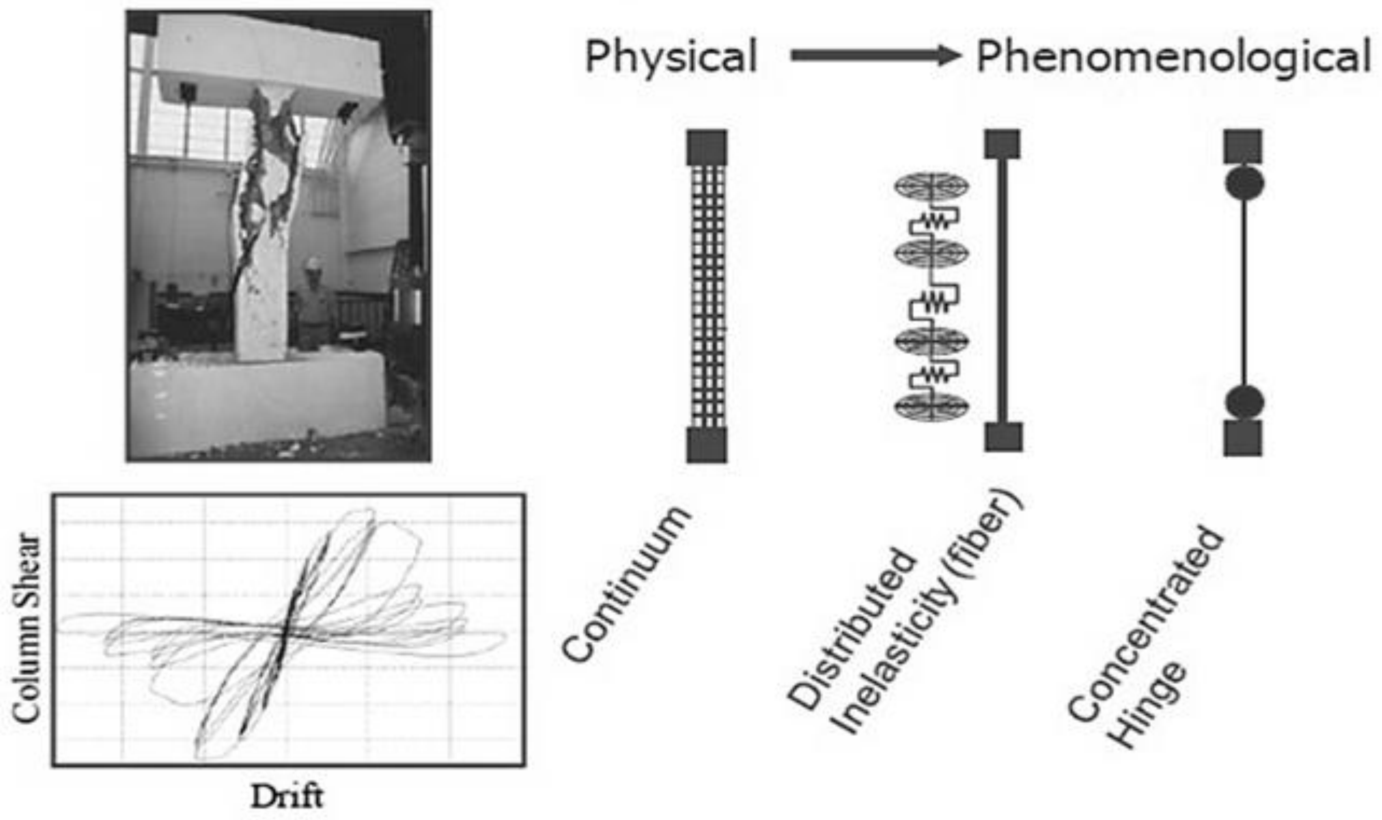
Basic Approaches of Nonlinear Modeling of Structures

The Nonlinear Analysis

- The nonlinear analysis aims to **simulate all significant modes of deformation** and deterioration in the structure from the onset of damage to complete collapse.
- Therefore, unlike a linear elastic model, the nonlinear model of an RC structure **should be able to capture** all local inelastic phenomenon including **concrete cracking, crushing, steel yielding, buckling, fracture and bond slip between steel and concrete, etc**.
- The nonlinear models can generally be classified based on the degree of idealization used in the model.

Continuum Models, Distributed Inelasticity Models and Lumped Inelasticity Models

A comparison of three idealized model types for simulating the nonlinear response of a reinforced concrete beam-column is shown in Figure (taken from ATC 72 [4]).



Continuum Models

- Continuum Models **explicitly** model the nonlinear behavior of the materials and elements that comprise the component.
- A continuum model might include finite elements representing the concrete, longitudinal reinforcement, and shear reinforcement, in which associated constitutive models (e.g. the nonlinear stress-strain curves of concrete and steel) would represent various nonlinear phenomenon.
- Continuum models generally do not enforce any predefined behavioral modes and, instead, seek to model the **underlying physics** of the materials and elements.
- They do not require definitions of member stiffness, strength or deformation capacity, as these effects are **inherently captured** in the model through the material properties.

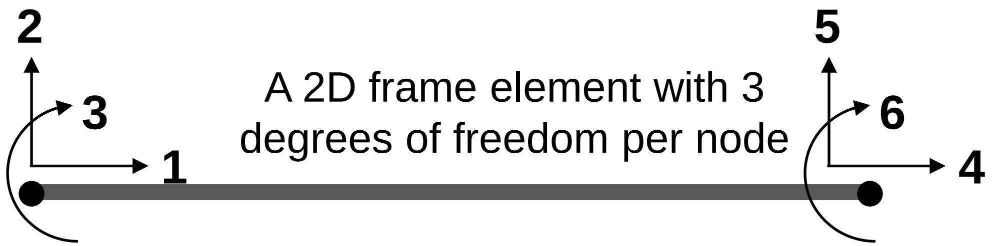
Lumped Inelasticity Models

- At the other extreme are lumped plasticity (concentrated hinge) models in which the nonlinear action is **lumped at certain points** of the structure and the nonlinear functions between various actions and corresponding deformations are assigned at those points. In this way, these models are defined entirely by the phenomenological description of the overall force-deformation response of the component.
- For example, a “concentrated hinge element” assigned at both ends of a beam or column might represent a lumped nonlinear flexural behavior (cracking, yielding and post-yielding behavior) defined by a nonlinear function between end moment and resulting curvature (or rotation) of the member. This nonlinear function should correspond to the observed force-deformation behavior and hysteretic test data of similar beam or column components.

Distributed Inelasticity Models

- In between the two extremes are distributed inelasticity (fiber) models, which can explicitly capture some aspects of nonlinear behavior while some effects are captured implicitly.
- For example, the complete nonlinear stress-strain curves of materials can be defined to capture important aspects of material nonlinearity. However, the integration of flexural stresses and strains through the cross section and along the member is considered implicitly.
- These models typically enforce some behavior assumptions (e.g., plane sections remain plane) in combination with explicit modeling of uniaxial material response.

What is Stiffness (K) “made off”?



$$K = f(E, A, I, L)$$

- E: Material Property
- A, I: Cross-section Properties
- L: Member Property

$$K = \begin{pmatrix} EA/L & 0 & 0 & -EA/L & 0 & 0 \\ 0 & 12EI/L^3 & 6EI/L^2 & 0 & -12EI/L^3 & 6EI/L^2 \\ 0 & 6EI/L^2 & 4EI/L & 0 & -6EI/L^2 & 2EI/L \\ -EA/L & 0 & 0 & EA/L & 0 & 0 \\ & -12EI/L^3 & -6EI/L^2 & 0 & 12EI/L^3 & -6EI/L^2 \\ 0 & 6EI/L^2 & 2EI/L & 0 & -6EI/L^2 & 4EI/L \end{pmatrix}$$

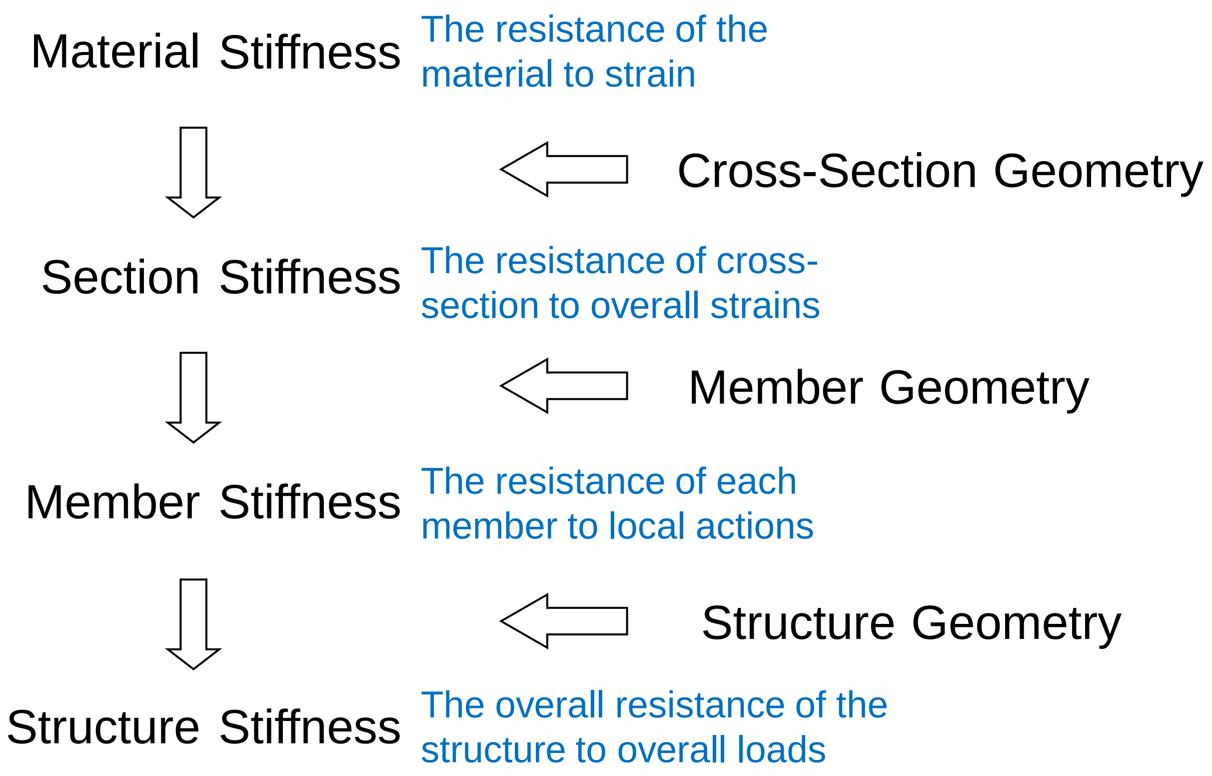
What is Stiffness (K) “made off”?

- The **overall stiffness** of the structure is derived from the overall geometry and connectivity of the members, their stiffnesses, and the boundary conditions.

- The **member stiffness** is derived from the **cross-section stiffness**, and member geometry.

- The **cross-section stiffness** is derived from the **material stiffness** and the cross-section geometry.

- All of these stiffness relationships may be linear or nonlinear.

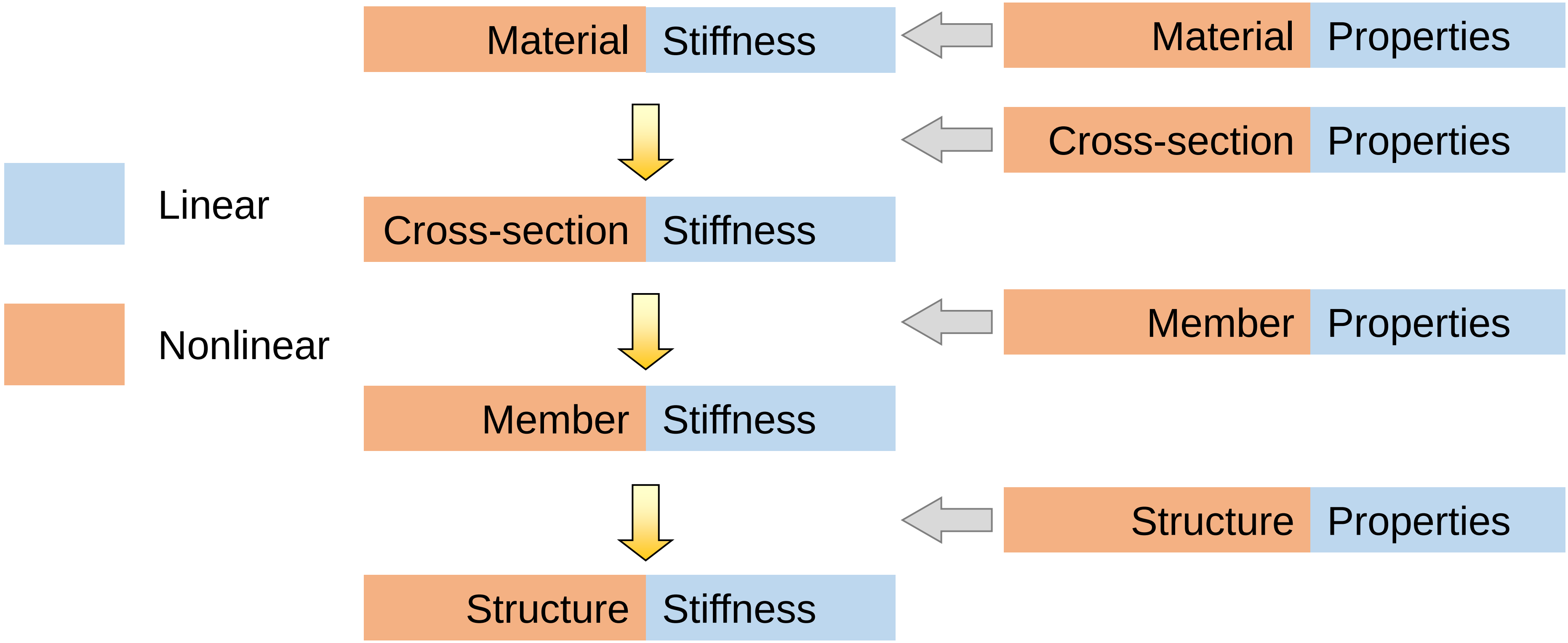


The Global Structure Stiffness - K

$$Ku = F$$

or

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



Practical Approaches for Nonlinear Modeling of Structures

Defining Inelastic Behavior at

- **Nonlinear Modeling of Materials** → **Material Level** → **Fiber Modeling Approach**
- **Nonlinear Modeling of Cross-sections** → **Cross-section Level**
- **Nonlinear Modeling of Members** → **Member Level**

Cross-section Level and **Member Level** → **Plastic Hinge Modeling Approach**

Nonlinear Modeling Approaches – NIST, 2010

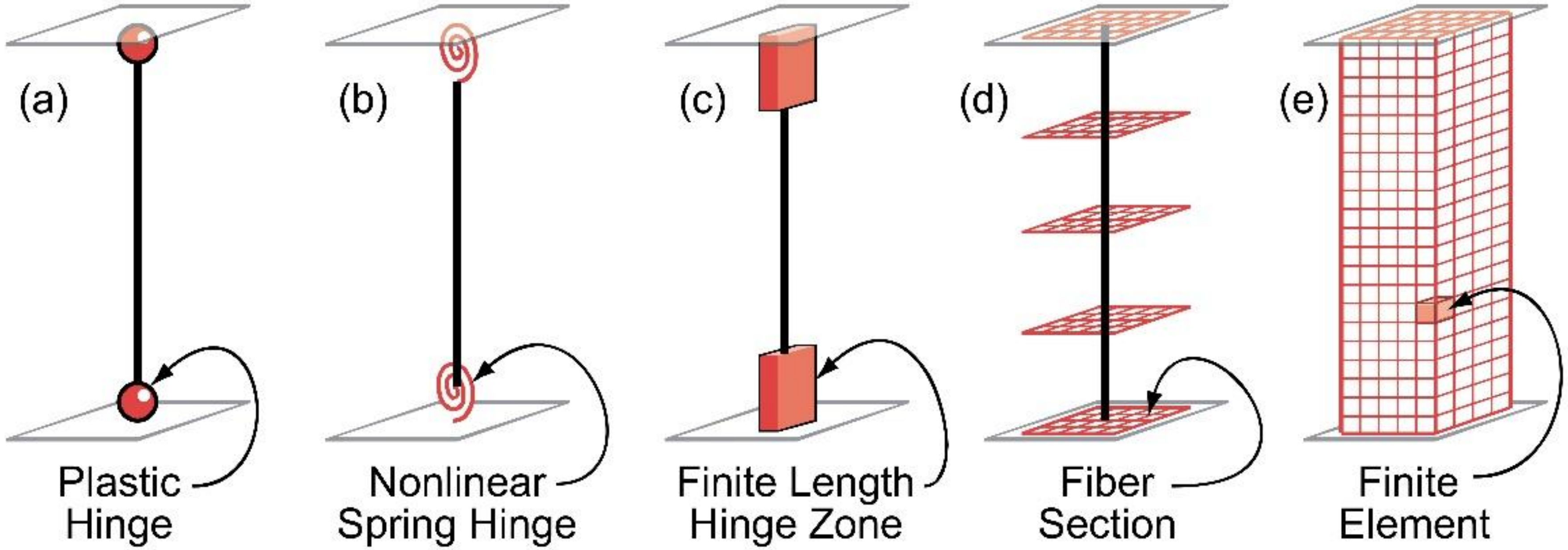


Figure 2-7 Range of structural model types (NIST, 2010).

Nonlinear Modeling Approaches – NIST, 2010

- For beam-column elements, models for nonlinear analysis can range from uniaxial spring or hinge models, to more fundamental fiber-type models, to detailed continuum finite element models.
- In general, all models are phenomenological in that they rely on empirical calibration to observed behavior at some level of idealization.
- The choice of model type for a given application involves a balance between reliability, practicality, and computational efficiency, subject to the capabilities of available software and computational resources. The optimal model type depends on many factors, including the structural system and materials, governing modes of behavior, the expected amount of nonlinearity, and the level of detail available for the input and output data. The reliability of the model comes from its ability to capture the critical types of deformation that are of interest to the modeler and control the response.

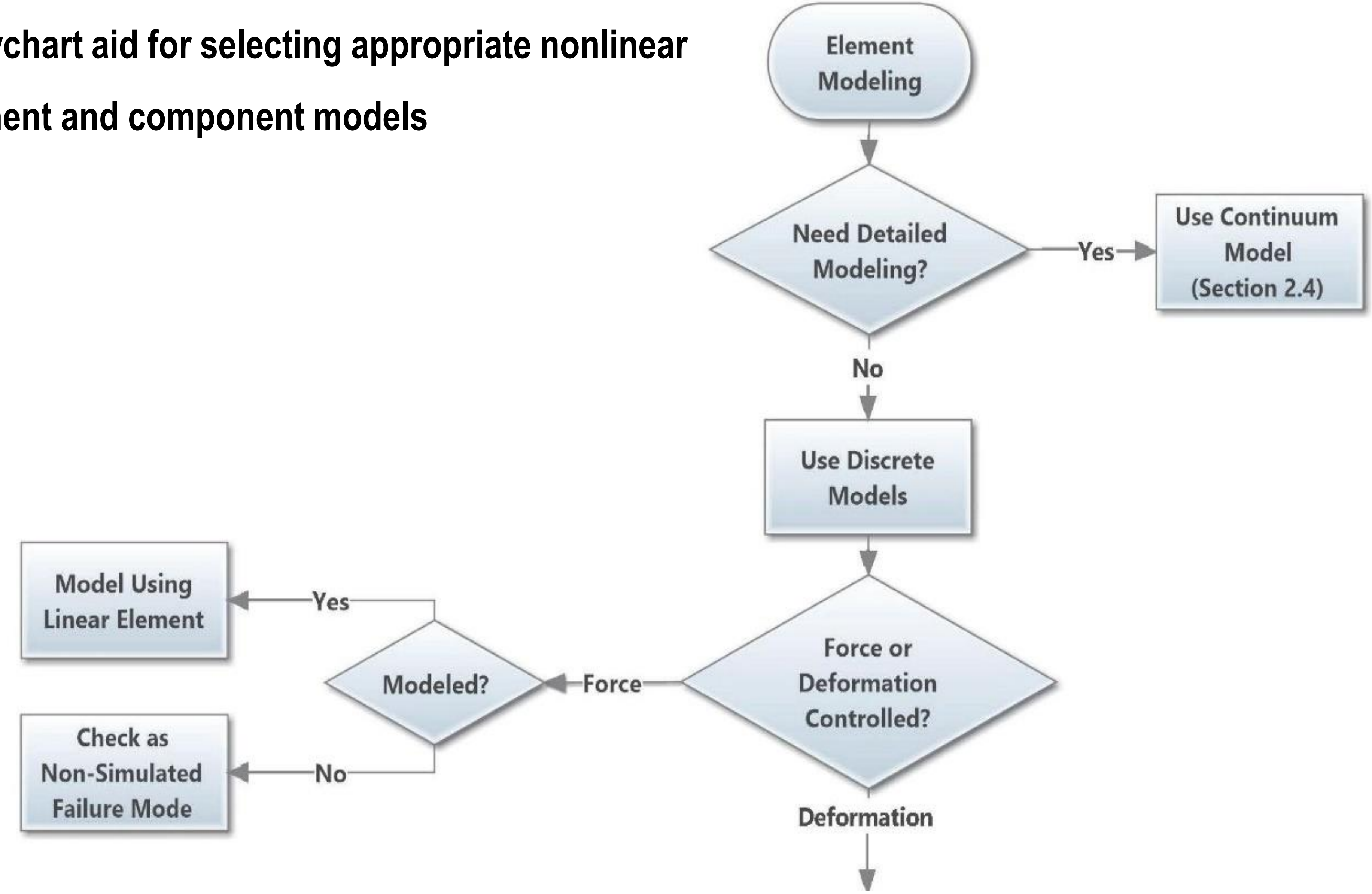
Nonlinear Modeling Approaches – NIST, 2010

- **Hinge and spring models** have the advantage of being computationally efficient by modeling highly nonlinear effects in localized regions of the structure with few degrees of freedom. The models generally employ pre-defined functions to define the nonlinear response of the component. Concentrated spring models are typically implemented to capture single degree of freedom response (e.g., M-θ), but they may include multiaxial response through yield surfaces (e.g., Px -My -Mz interaction) or other means. By capturing complicated behavior with highly idealized models, concentrated hinge models are very versatile, but they are also empirical and limited to modeling phenomena over the range of components and behavior modes for which they have been calibrated.
- For beam-columns, **fiber-type models** provide the capability to numerically integrate material response through the member cross sections at a more fundamental material level. The fiber-type integration through the cross section can be used either in conjunction with a finite length hinge zone or with model formulations that simulate distributed inelasticity along the member length. Fiber-type models for beams and beam-columns generally invoke kinematic assumptions, such as the Euler-Bernoulli (plane sections remain plane) assumption, to relate uniaxial stresses and strains through the member cross section to stress resultants and generalized strains for the cross section.

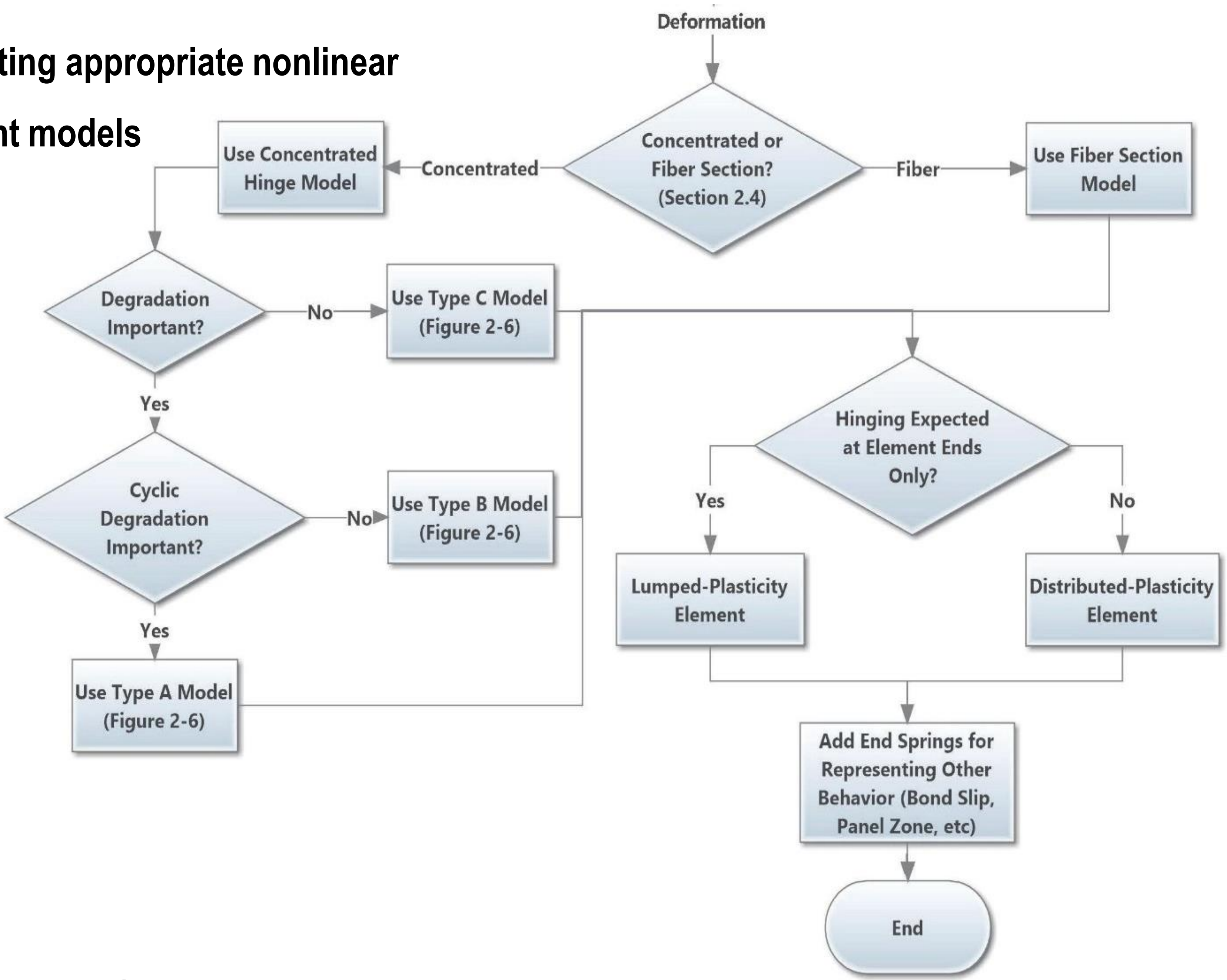
Nonlinear Modeling Approaches – NIST, 2010

- **Continuum finite element models** represent the behavior at the most fundamental level and provide the ability to model the complete interaction of three-dimensional behavior, including complex geometries and multi-axial stress and strain states.
- However, three-dimensional (3D) continuum models are the most computationally intensive, particularly where the numerical mesh refinement is controlled by the smallest dimension of the member (e.g., where finite element meshing through the thickness or depth of a member will dictate the mesh size required along the member length).
- For this reason, 3D continuum models are typically only used to simulate portions of overall systems. In addition, while continuum models offer the potential for capturing response at very fundamental levels, their practical application is limited by both computational resources and data to calibrate certain localized behavioral effects. For example, 2D shell or 3D continuum finite element models can capture the response of isotropic steel materials fairly well, whereas many unresolved challenges remain for simulating the detailed behavior of reinforced concrete members, considering concrete cracking/dilation and interactions between steel reinforcing bars and concrete (e.g., bond and anchorage).

Flowchart aid for selecting appropriate nonlinear element and component models



Flowchart aid for selecting appropriate nonlinear element and component models

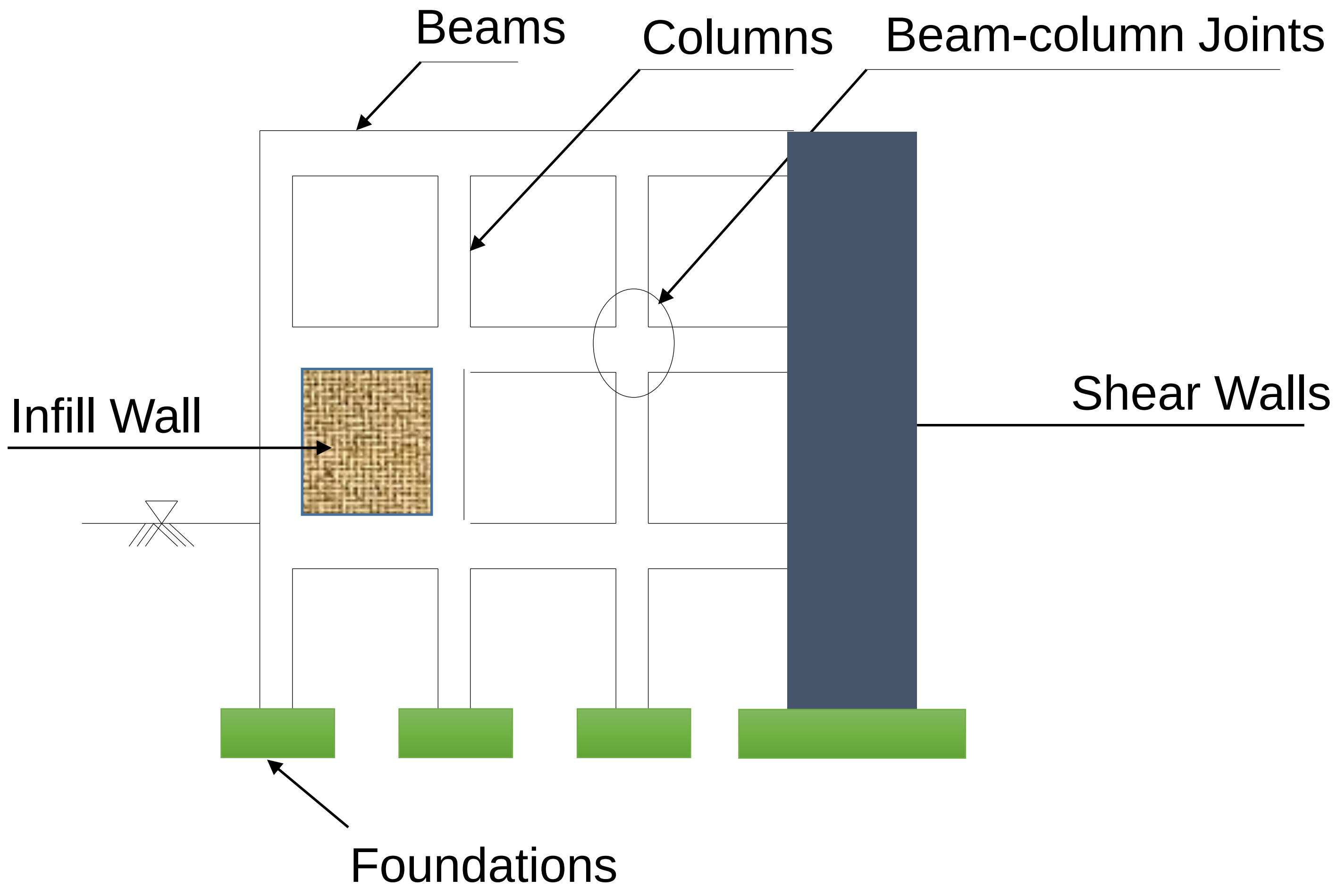


Flowchart aid for selecting appropriate nonlinear element and component models

- The first decision point outlined in Figure 2-15 is the level of detail of the model, with the primary choice being between a continuum model and a discrete model (i.e., fiber or concentrated hinge). For discrete models (which is typical for design), the next decision is how each component in the model is classified and modeled, either as a force- or deformation-controlled component, as well as which components to model (i.e., decision of if the gravity framing components are included in the model).

- After this, the model types for each component and the level of detail of the modeling can be decided. All of these modeling decisions are centered on how simplified versus complex the structural model should be and the needed level of complexity of the model depends on aspects such as the expected failure modes and their consequences, the anticipated locations of damage, and the anticipated level of nonlinearity.

Nonlinear Models for Building Components



Nonlinear Models for Building Components

- 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
- Springs – Foundation and Soil Modeling

Thank you