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.
The basic variable is displacement and its derivatives.
Structure Stiffness and Response

- Material Stiffness
- Section Stiffness
- Member Stiffness
- Structure Stiffness

Cross-section Geometry
Member Geometry
Structure Geometry

- Strain
- Rotation
- Deformation
- Force
- Curvature
- Moment
ANALYSIS PROCEDURES

LINEAR PROCEDURE (LP)
- STATIC (LSP)
- DYNAMIC (LDP)

NONLINEAR PROCEDURE (DP)
- STATIC (NLSP)
- DYNAMIC (NLDP, NLTH)
What is Nonlinearity
• 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
• 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

- Brittle fracture
- Local buckling
- Material non-linearity
- Geometric non-linearity
- Fully plastic
- Material and geometric non-linearity

Graph with axes for Load and deformation, showing different stages of behavior: linear, buckling, geometric non-linearity, fully plastic, material non-linearity, and material and geometric non-linearity with local buckling and brittle fracture points.
Structural Behavior

Load vs. deformation graph showing:
- Linear behavior
- Buckling
- Geometric non-linearity
- Material non-linearity
- Material and geometric non-linearity
Main Aspects of F-D Relationship
Complication – Cyclic Degredation

In the first cycle there may be little or no degradation.

After a few cycles, the strength and stiffness may degrade.
Complication – Cyclic Strength Gain

With cyclic deformation, the strength may increase.
Complication – Effect of Strength Loss

Strength in opposite direction may or may not reduced.
From Cyclic to Backbone Curve

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).
Effect of other Nonlinearities

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

**Figure 2-8** – Force-deformation curve with and without the $P-\Delta$ effect (PEER/ATC 2010).
## Linear Vs. Nonlinear

<table>
<thead>
<tr>
<th>Feature</th>
<th>Linear problems</th>
<th>Nonlinear problems</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load displacement relationship</td>
<td>Displacements are linearly dependent on the applied loads.</td>
<td>The load-displacement relationships are usually nonlinear.</td>
</tr>
<tr>
<td>Stress-strain relationship</td>
<td>A linear relationship is assumed between stress and strain.</td>
<td>In problems involving material nonlinearity, the stress-strain relationship is often a nonlinear function of stress, strain and/or time.</td>
</tr>
<tr>
<td>Magnitude of displacement</td>
<td>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.</td>
<td>Displacements may not be small, hence an updated reference state may be needed.</td>
</tr>
</tbody>
</table>
# Linear Vs. Nonlinear

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Linear elastic material properties are usually easy to obtain</th>
<th>Nonlinear material properties may be difficult to obtain and may require additional experimental testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reversibility</td>
<td>The behaviour of the structure is completely reversible upon removal of the external loads</td>
<td>Upon removal of the external loads, the final state may be different from the initial state.</td>
</tr>
<tr>
<td>Boundary Conditions</td>
<td>Boundary conditions remain unchanged throughout the analysis</td>
<td>Boundary conditions may change, e.g. a change in the contact area.</td>
</tr>
<tr>
<td>Loading Sequence</td>
<td>Loading sequence is not important, and the final state is unaffected by the load history</td>
<td>The behaviour of the structure may depend on the load history</td>
</tr>
<tr>
<td>Iterations and increments</td>
<td>The load is applied in one load step with no iterations</td>
<td>The load is often divided into small increment with iterations performed to ensure that equilibrium is satisfied at every load increment</td>
</tr>
<tr>
<td></td>
<td>Computation time</td>
<td>Robustness of solutions</td>
</tr>
<tr>
<td>--------------------------</td>
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</tr>
<tr>
<td>Computation time</td>
<td>Computation time is relatively small in comparison to nonlinear problems</td>
<td>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</td>
</tr>
<tr>
<td>Robustness of solutions</td>
<td>A solution can easily be obtained with no interaction from the user</td>
<td>Factoring and combining of results is not possible</td>
</tr>
<tr>
<td>Use of results</td>
<td>Superposition and scaling allow results to be factored and combined as required</td>
<td></td>
</tr>
<tr>
<td>Initial state of stress/strain</td>
<td>The initial state of stress and/or strain is unimportant</td>
<td></td>
</tr>
</tbody>
</table>
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 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

- Plastic hinge
- Nonlinear spring hinge
- Concentrated plasticity
- Finite length hinge zone
- Distributed plasticity
- Fiber section
- Finite element

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

**Original Chain**

- Ductile Link
- Brittle Links

**Loaded Chain**

- Ductile Link stretches by yielding before breaking
- Brittle Links do not yield

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
• Bilinear material model
Various Models

(a) Modified Hognestad. (From Ref. 3-24.)

(b) Parabola rectangle. (From Ref. 3-25.)

(c) Elastic-plastic.

(d) Todeschini. (From Ref. 3-26.)
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

- **Linear**
- **Whitney**
- **PCA**
- **BS-8110**
- **Parabolic**
- **Unconfined**
- **Mander-1**
- **Mander-2**
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
**The Most Simple Case**

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

**The Most Comprehensive Case**

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

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

\[
M_y = \phi_3 \left[ \frac{1}{\gamma_1} \int_{x} \int_{y} \sigma(x, y) \, dx \, dy \cdot x \ldots + \frac{1}{\gamma_2} \sum_{i=1}^{n} A_i \sigma_i(x, y) \, x_i \ldots \right]
\]
Capacity Interaction Surface

P

My

Mx

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The Moment Curvature Curve

![Moment-Curvature Diagram]

- **Curve Generation Options**
  - Moment Angle: 0
  - Axial Load: 0
  - Maximum strain: 0.025
  - Strain increment: 0.0005

- **Stop Computing When**
  - Maximum strain reached
  - Any part of section fails
  - All parts of section fail
  - The first rebar fails
  - Selected section part fails

- **Curve Points**
  - Solution Log
  - Add To Report

- **Result**
  - At $P_{u} = 0.00$ kip
  - Curvature (1/1000)
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
Confinements

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
Concrete Behavior and Confinement

Kent and Park Model

Idealized Stress-Strain Behavior of Confined Concrete

Confined Area 12” x 16”

Strain, in./in.
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

![Graph showing the definition of ductility](image)

Ductility = $\frac{D_u}{D_y}$
Modeling for Of Members
Modeling of Seismic Resistant Structures

**RC Frame Subjected to EQ Motions**
- location of plastic hinge
- elastic frame member

**Lumped Plasticity Model**
- plastic hinge spring
  - (nonlinear rotational spring)
- elastic frame element
Nonlinear Model for Building Components

\[ M\ddot{u} + C\dot{u} + Ku = -M(r\ddot{u}_g) \]
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
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
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).

**Plastic Hinges**

- reliable predicted behavior
- most ductile plastic deformation
- highest equivalent damping

**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)
- Crushing and disintegrate
- Residual strength
- Cover spall

Cover Concrete (Unconfined Zone)

BEAM CROSS SECTION

MAIN STEEL = energy dissipater

STIRRUP = confinement + shear capacity
Stress, $\sigma$ (MPa)

Strain, $\varepsilon$ (mm / mm)

Slope of Degrading Portion is Delayed due to Transverse Reinforcement

Spalling of cover concrete

Disintegration

Crushing

Core concrete retains some residual strength

$\varepsilon = (\Delta_2 + \Delta_1)/L$

Tensile Strength of Concrete is Assumed to be Zero

Confinement Effect from Transverse Reinforcement

$F'_{cc}$

$R \times F'_{cc}$
Core/Cover Concrete – Hysteretic Response

\[ \sigma = \frac{(\Delta_2 + \Delta_1)}{L} \]

Stress, \( \sigma \) (MPa)

\[ \varepsilon = \frac{F'}{R \times F'_{cc}} \]

Strain, \( \varepsilon \) (mm / mm)

Elastic Loading/Unloading

Inelastic Loading

Loading/Unloading from Backbone Curve

residual strength

crushing

Inelastic Loading

\[ f'_{co} = 29.8 \text{ MPa} \]
\[ \rho_s = 1.14\% \]
Steel Reinforcement – Monotonic Response

\[ \varepsilon = \frac{\Delta_2 + \Delta_1}{L} \]

\[ \sigma = \text{Tension} \triangleleft \rightarrow \text{Compression} \]

\[ \sigma = \sigma_1 + \sigma_2 \]

\[ \varepsilon = \varepsilon_1 + \varepsilon_2 \]

\[ \varepsilon \text{ (onset of strain hardening)} \]

\[ \varepsilon \text{ (fracture)} \]

\[ \varepsilon_{sh} \text{ (onset of strain hardening)} \]

\[ \varepsilon_y \text{ (yielding)} \]

\[ \varepsilon_{sh} \text{ (fracture)} \]

\[ E_p \text{ (fracture)} \]

\[ E_s \text{ (yielding)} \]

\[ F_y \text{ (yielding)} \]

Strain, \( \varepsilon \) (mm / mm)

Stress, \( \sigma \) (MPa)

onset of buckling

onset of strain hardening

yielding

fracture
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
Concept of “Equivalent Plastic Hinge Length, $L_{PH}$”

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

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

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

[Paulay and Priestley, 1992]
• An RC section can be represented by sub-divided layers (fibers). Each layer 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.

\[
N = \int f_s(\varepsilon) b dy \\
N \bigg[ \sum_{i=1}^{n} (f_s(\varepsilon) A_{fiber,i}) \bigg]
\]

\[
M = \int f_s(\varepsilon) y b dy \\
M \bigg[ \sum_{i=1}^{n} (f_s(\varepsilon) y_{cg} A_{fiber,i}) \bigg]
\]

\[
K_{spring} = \frac{EA}{L} = \frac{E_1 A_{fiber}}{L_{PH}}
\]
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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

CASE A – [P = 0 kN]

CASE B – [P = 100 kN]

Tip Displacement (m)

Vertical Force (kN)

SECTION A-A (dimension in mm)

Reversed cyclic displacement

Constant Axial Force (P)

1000 mm

4Φ10

4Φ10

STR 3Φ10@100

300

500

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• 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

Possible failure mechanisms:
A. Flexural
B. Midheight horiz. crack
C. Diagonal crack
D. Horizontal slip
E. Corner crushing

- Plastic hinges
- Crack in frame members
- Crack in infill
- Slip at joints
- Crushing
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.
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

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

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

where: $E_c I_c$ is the bending stiffness of the columns

$E_m$ is the modulus of Elasticity of masonry
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.
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

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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.
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.
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.
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
• **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
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

1. Applied Loads
2. Building Analysis
3. Member Actions
4. Cross-section Actions
5. Material Stress/Strain

From Strains to Response

1. Material Response
2. Section Response
3. Member Response
4. Building Response
5. 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.
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.
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

Original Chain

Ductile Link

Brittle Links

Loaded Chain

Ductile Link stretches by yielding before breaking

Brittle Links do not yield

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

(a) Building performances during earthquakes: two extremes – the ductile and the brittle.  
(b) Brittle failure of a reinforced concrete column

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.
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
Seismic Energy Dissipation Devices – each device is suitable for a certain building.
Example – A Hanging Rope

Mild Steel
$F_y$ ranges from 50-60 ksi

Area = 2 square inch

High Strength Steel
$F_y$ ranges from 100-120 ksi

Choose the correct area for the high strength steel

LOAD = 90 kips
**Example – A Hanging Rope**

**LOAD DESIGN**
Typical design = Area of bar needs to be:

\[ A = \frac{P}{F_y} = \frac{90}{100} = 0.9 \text{ sq. in.} \]

**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 \( 2 \times 60 = 120 \) kips
If we want the mild steel to yield first, we need at least

\[ A = \frac{C_{\text{max}}}{F_y} = \frac{120}{100} = 1.2 \text{ sq. in.} \]
for the high strength steel
Thank You