Basic Concepts in Ductile Detailing of Steel Structures

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Overview of Presentation

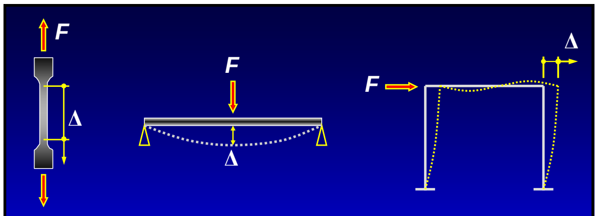
- **What is Ductility ?**
- **Why is Ductility Important ?**
- **How Do We Achieve Ductility in Steel Structures ?**

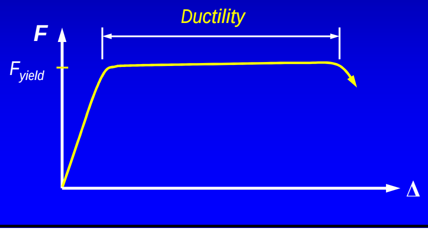
What is Ductility ?

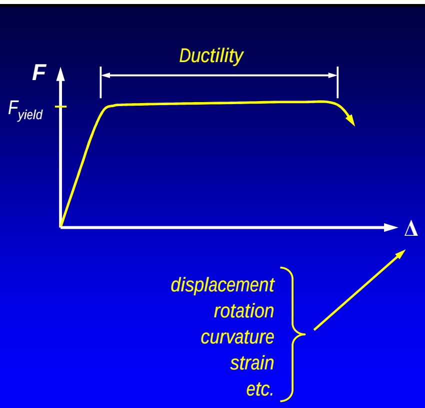
Ductility: **The ability to sustain large inelastic deformations without significant loss in strength.**

Ductility = inelastic deformation capacity

Ductility:
- **material response**
- **structural component response (members and connections)**
- **global frame response**

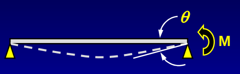


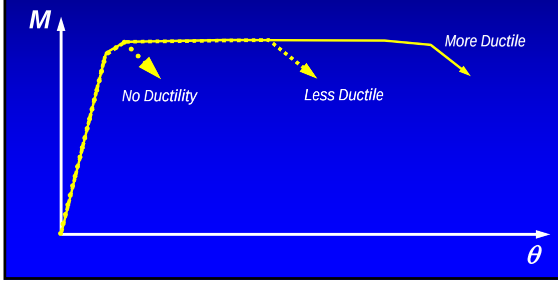


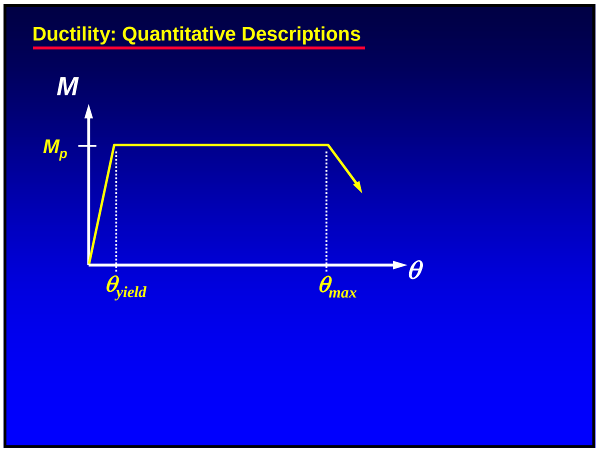


displacement
rotation
curvature
strain
etc.

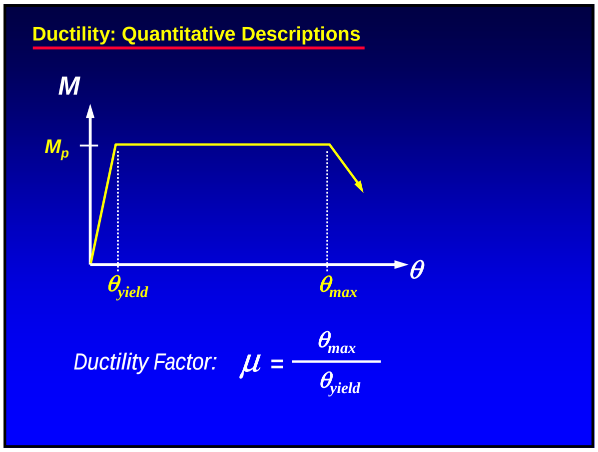
Ductility: Qualitative Description





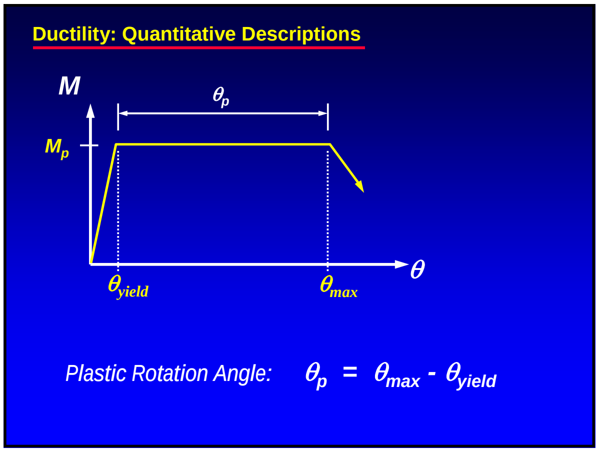


Ductility: Quantitative Descriptions



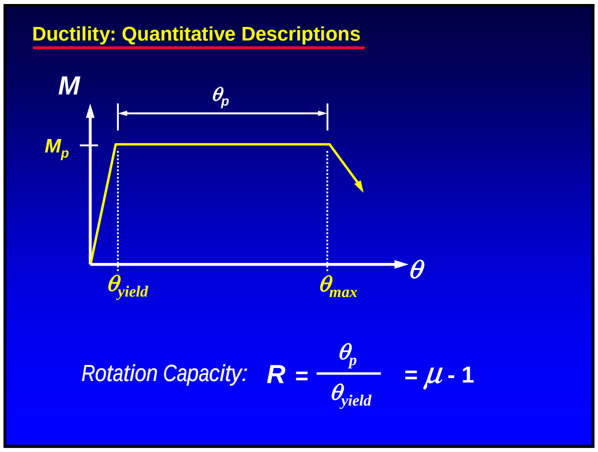
Ductility: Quantitative Descriptions

Ductility Factor: $\mu = \frac{\theta_{max}}{\theta_{yield}}$



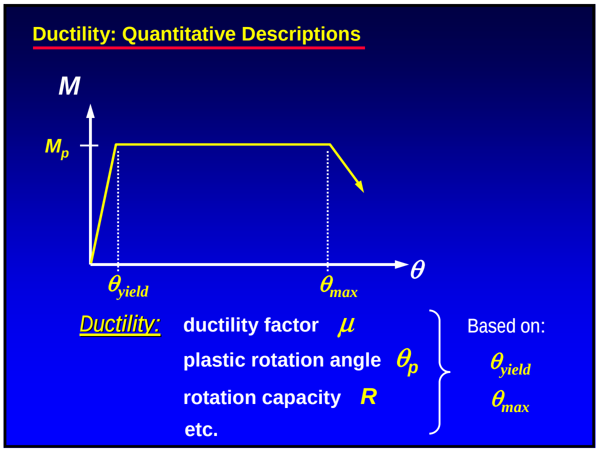
Ductility: Quantitative Descriptions

Plastic Rotation Angle: $\theta_p = \theta_{max} - \theta_{yield}$



Ductility: Quantitative Descriptions

Rotation Capacity: $R = \frac{\theta_p}{\theta_{yield}} = \mu - 1$

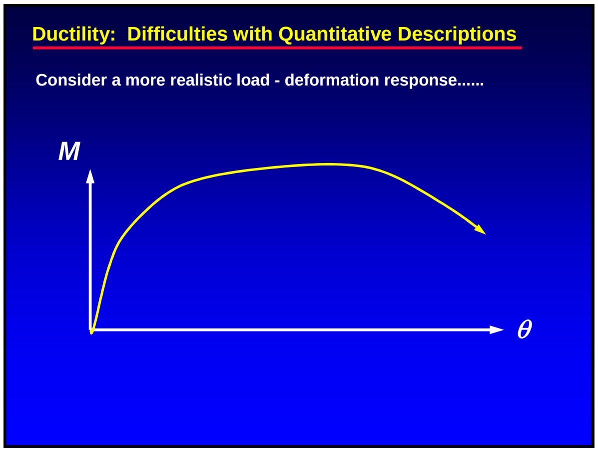


Ductility: Quantitative Descriptions

Ductility:

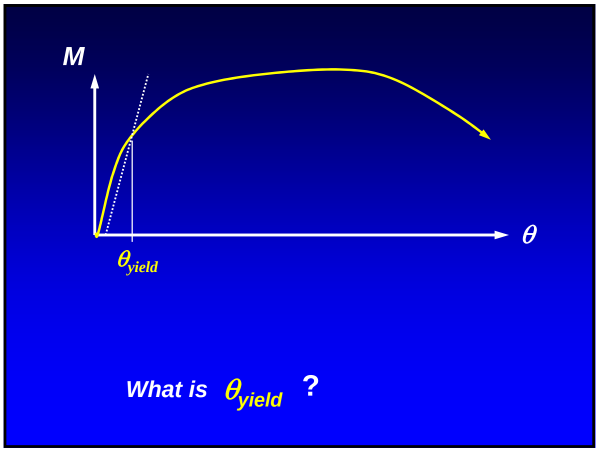
- **ductility factor** μ
- **plastic rotation angle** θ_p
- **rotation capacity** R
- **etc.**

Based on: θ_{yield}, θ_{max}

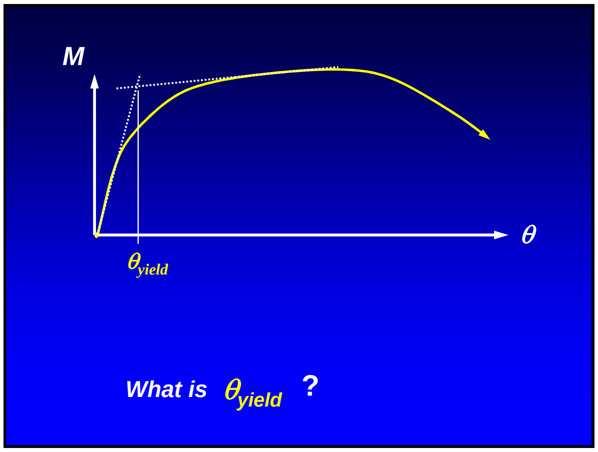


Ductility: Difficulties with Quantitative Descriptions

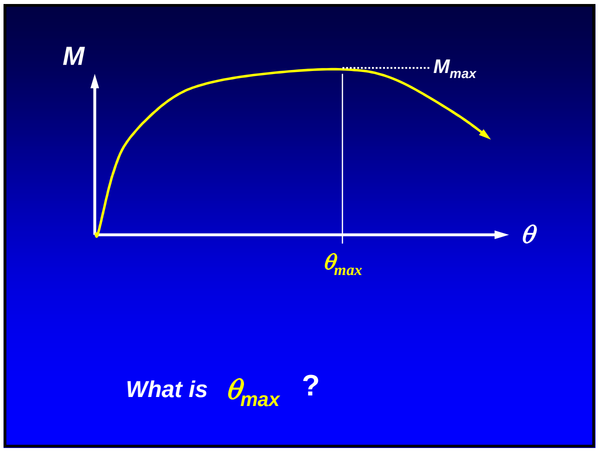
Consider a more realistic load - deformation response......



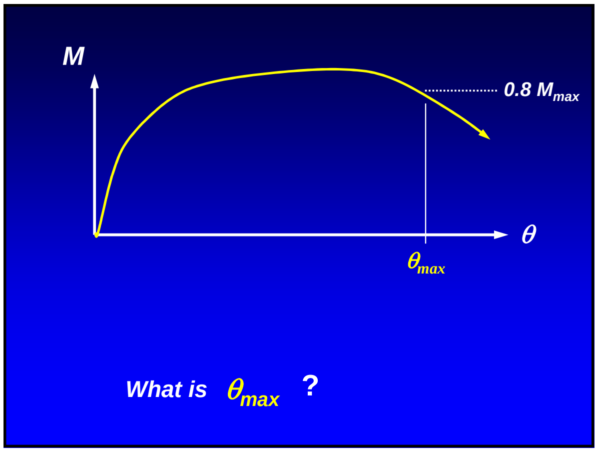
What is θ_{yield} ?



What is θ_{yield} ?



What is θ_{max} ?



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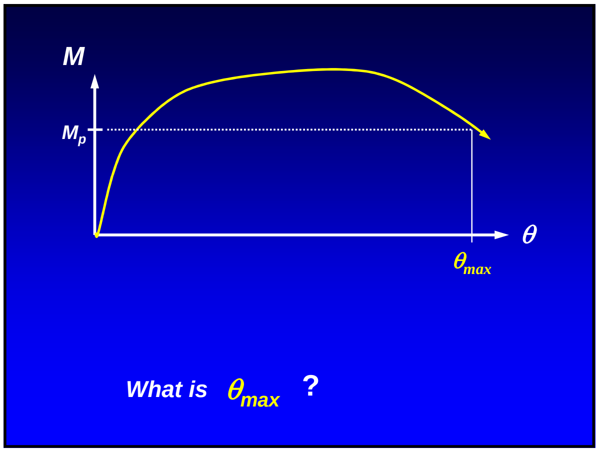
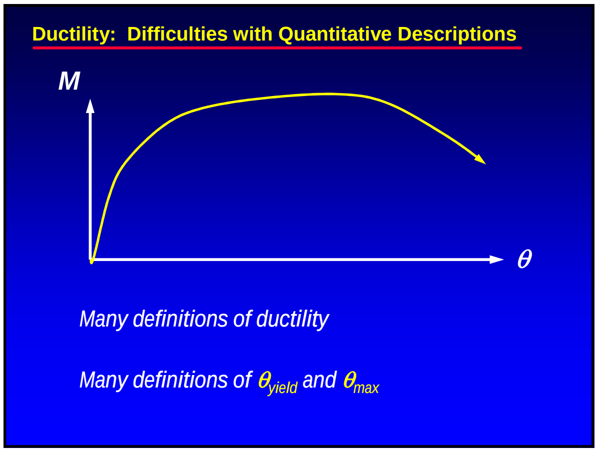


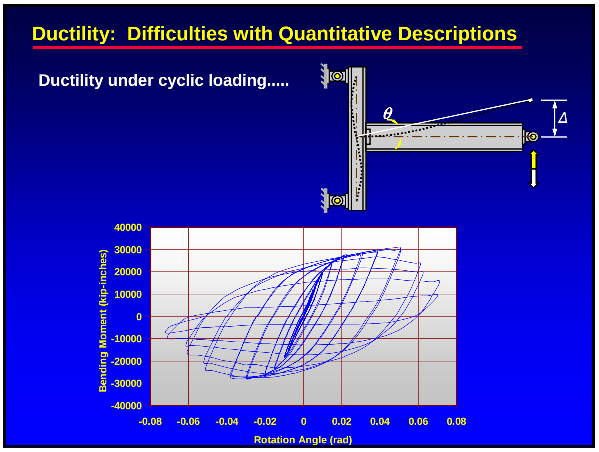
What is θ_{max} ?



Ductility: Difficulties with Quantitative Descriptions

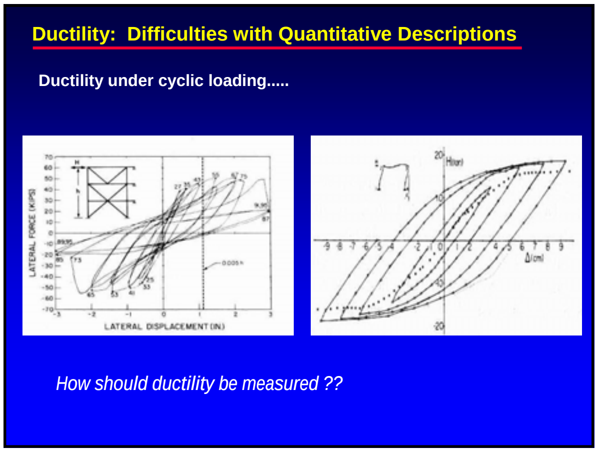
Many definitions of ductility

Many definitions of θ_{yield} and θ_{max}



Ductility: Difficulties with Quantitative Descriptions

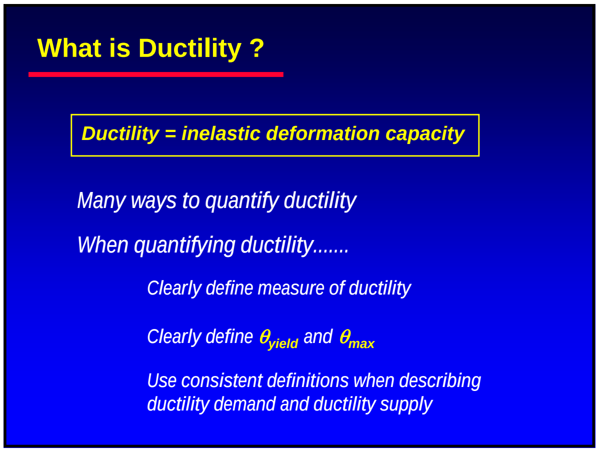
Ductility under cyclic loading.....



Ductility: Difficulties with Quantitative Descriptions

Ductility under cyclic loading.....

How should ductility be measured ??



What is Ductility ?

Ductility = inelastic deformation capacity

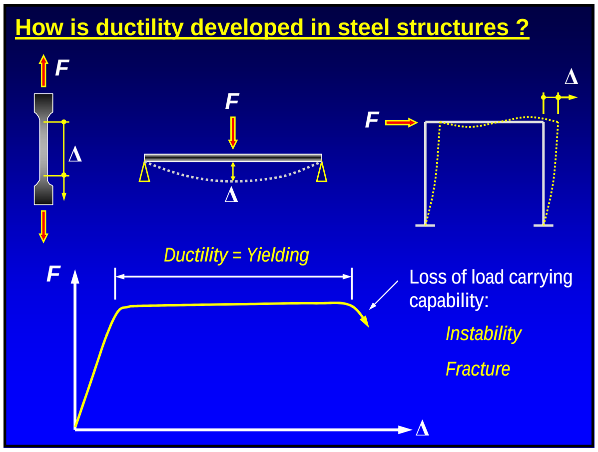
Many ways to quantify ductility

When quantifying ductility.......

Clearly define measure of ductility

Clearly define θ_{yield} and θ_{max}

Use consistent definitions when describing ductility demand and ductility supply



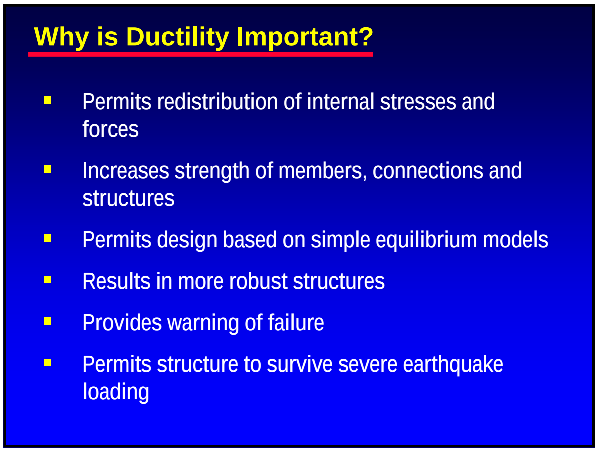
How is ductility developed in steel structures ?

Ductility = Yielding

Loss of load carrying capability:

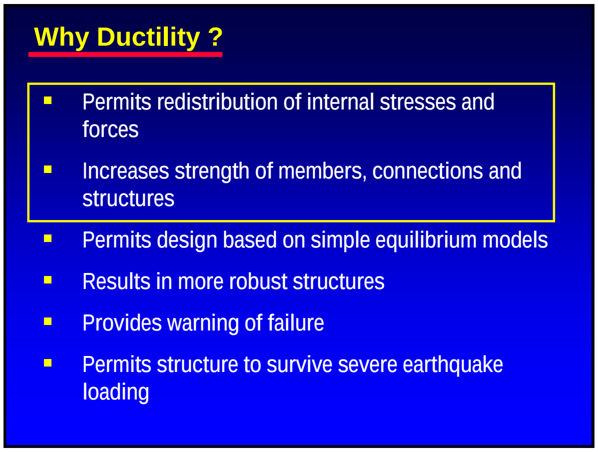
Instability

Fracture



Why is Ductility Important?

- Permits redistribution of internal stresses and forces
- Increases strength of members, connections and structures
- Permits design based on simple equilibrium models
- Results in more robust structures
- Provides warning of failure
- Permits structure to survive severe earthquake loading



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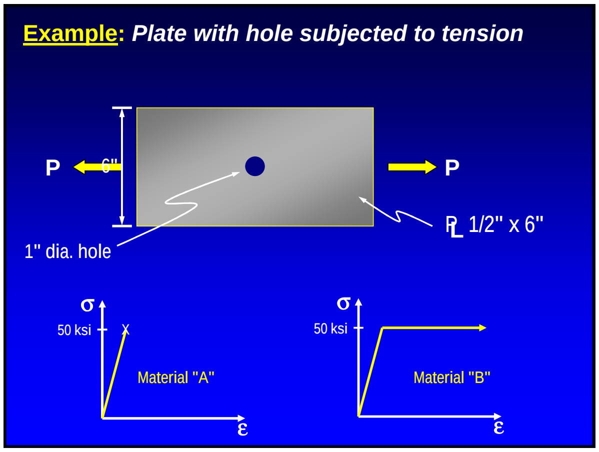
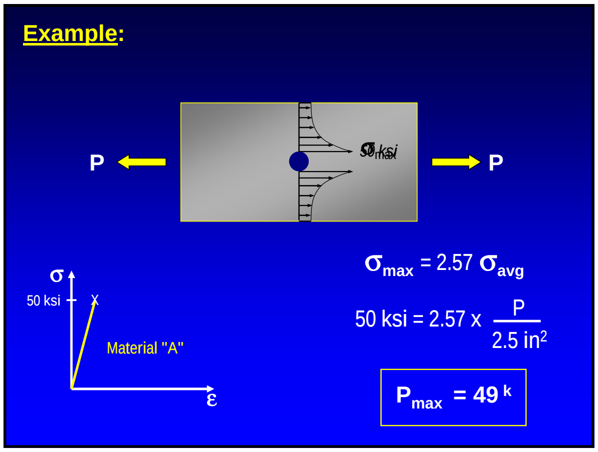


Example: *Plate with hole subjected to tension*

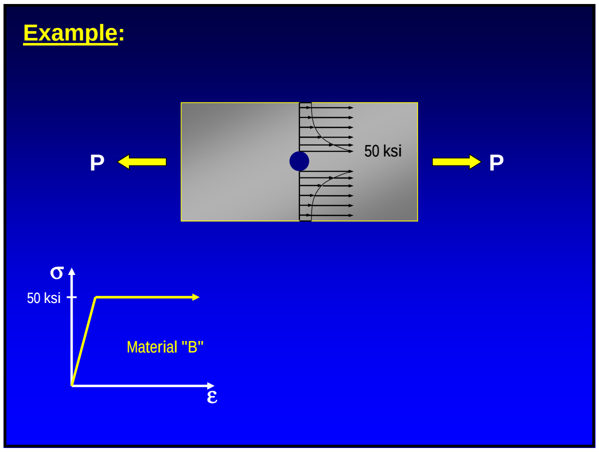


Example:

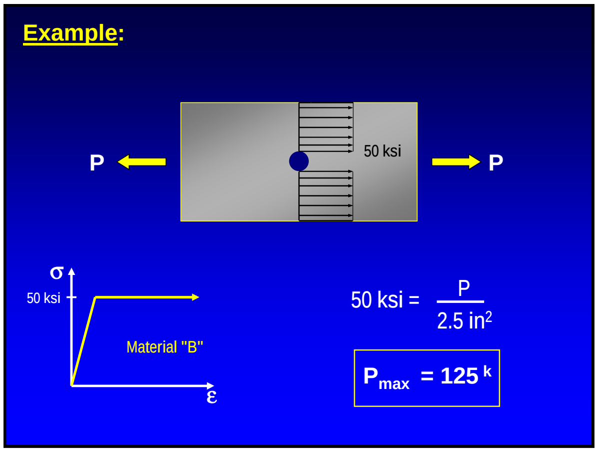
$$\sigma_{max} = 2.57\ \sigma_{avg}$$

$$50\text{ ksi} = 2.57 \times \frac{P}{2.5\text{ in}^2}$$

$$\boxed{P_{max}\ = 49^{\,k}}$$



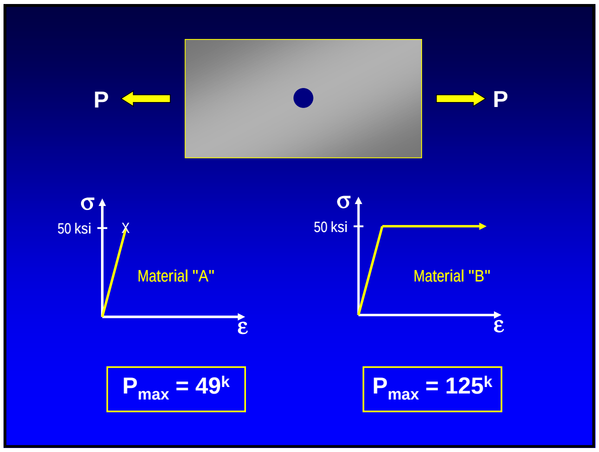
Example:



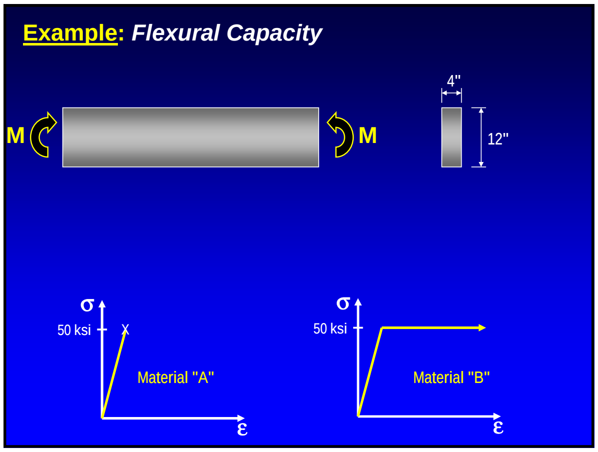
Example:

$$50\text{ ksi} = \frac{P}{2.5\text{ in}^2}$$

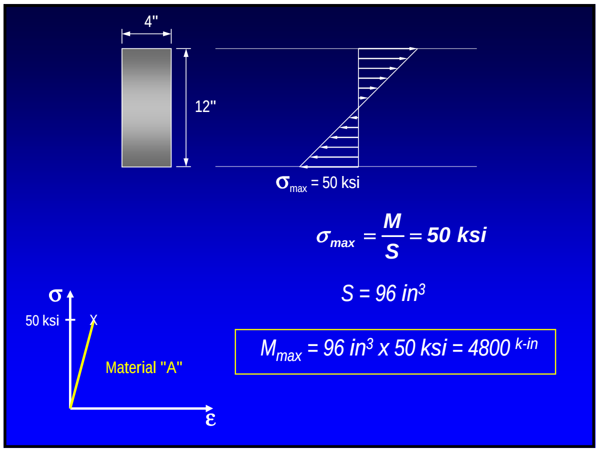
$$\boxed{P_{max}\ = 125^{\,k}}$$



$\boxed{P_{max} = 49^k}$ $\boxed{P_{max} = 125^k}$



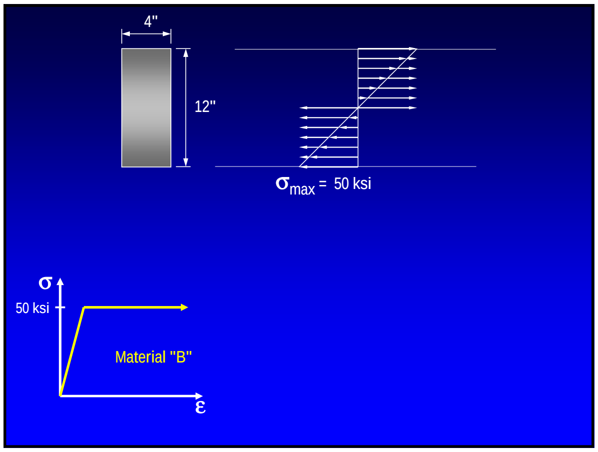
Example: *Flexural Capacity*

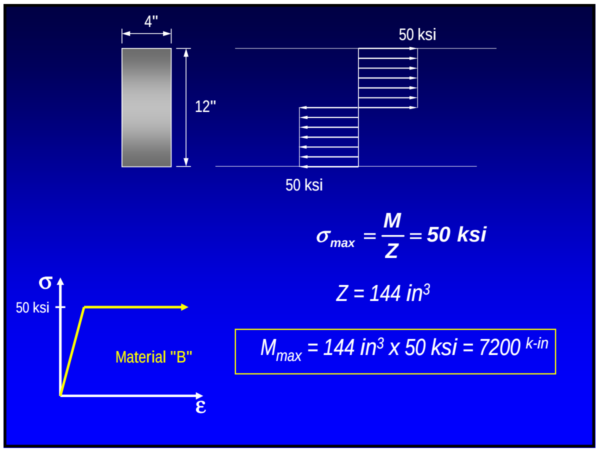


$$\sigma_{max} = \frac{M}{S} = 50\ ksi$$

$$S = 96\ in^3$$

$$\boxed{M_{max} = 96\ in^3 \times 50\ ksi = 4800\ ^{k\text{-}in}}$$

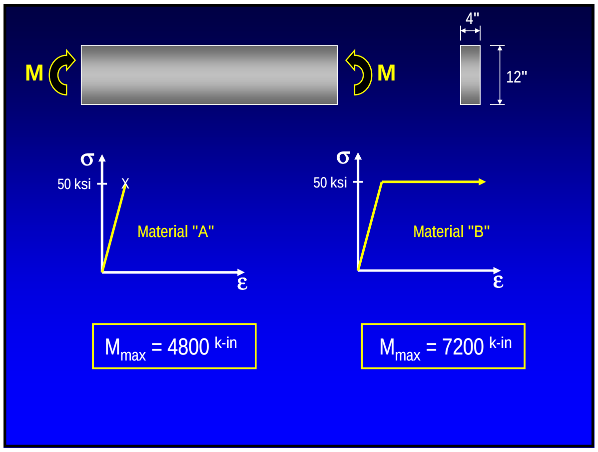




$$\sigma_{max} = \frac{M}{Z} = 50\ ksi$$

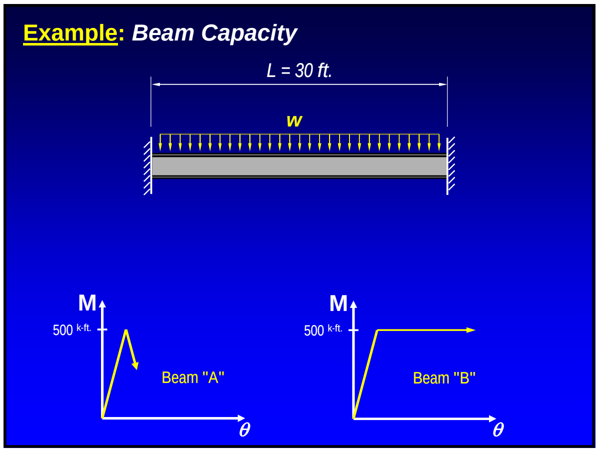
$$Z = 144\ in^3$$

$$\boxed{M_{max} = 144\ in^3 \times 50\ ksi = 7200\ ^{k\text{-}in}}$$

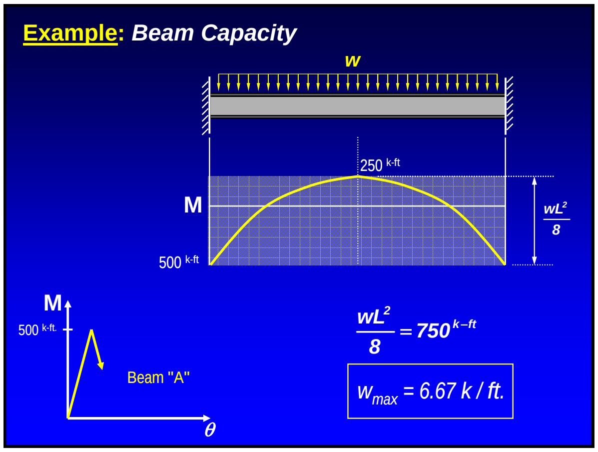


$$\boxed{M_{max} = 4800\ ^{k\text{-}in}}$$

$$\boxed{M_{max} = 7200\ ^{k\text{-}in}}$$



Example: ***Beam Capacity***

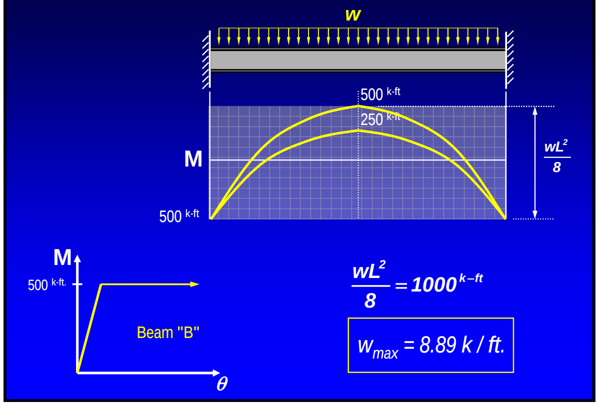


Example: ***Beam Capacity***

$$\frac{wL^2}{8} = 750^{k\text{-}ft}$$

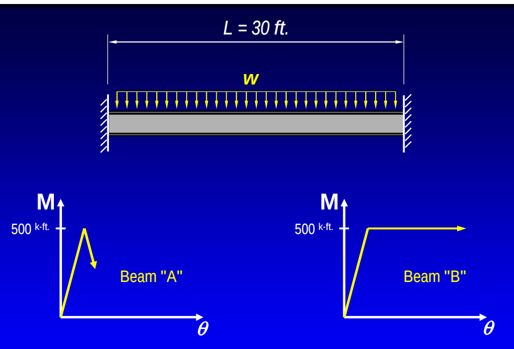
$$\boxed{w_{max} = 6.67\ k/ft.}$$

Example: ***Beam Capacity***



$$\frac{wL^2}{8} = 1000^{k-ft}$$

$$\boxed{w_{max} = 8.89\ k / ft.}$$



$$\boxed{w_{max} = 6.67\ k / ft.}$$

$$\boxed{w_{max} = 8.89\ k / ft.}$$

Why Ductility ?

- Permits redistribution of internal stresses and forces
- Increases strength of members, connections and structures
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- Results in more robust structures
- Provides warning of failure
- Permits structure to survive severe earthquake loading

Lower Bound Theorem of Plastic Analysis

A limit load based on an internal stress or force distribution that satisfies:

1. Equilibrium
2. Material Strength Limits for Ductile Response ($\sigma \leq F_y$, $M \leq M_p$, $P \leq P_y$, etc)

is less than or equal to the true limit load.

Lower bound theorem only applicable for ductile structures

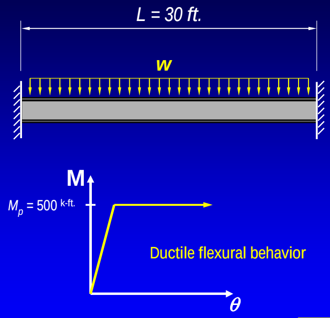
Implications of the lower bound theorem ....

For a structure made of ductile materials and components:

Designs satisfying equilibrium and material strength limits are safe.

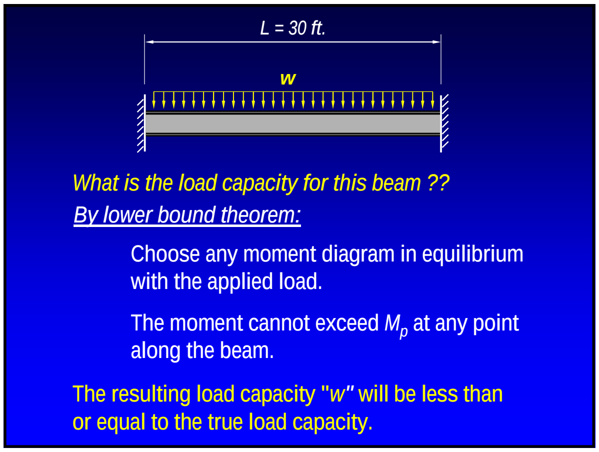
As a designer, as long as we satisfy equilibrium (i.e. provide a load path), a ductile structure will redistribute internal stresses and forces so as to find the available load path.

Example of lower bound theorem: ***Beam Capacity***



What is the load capacity for this beam ??

$$\boxed{w_{max} = 8.89\ k / ft.}$$



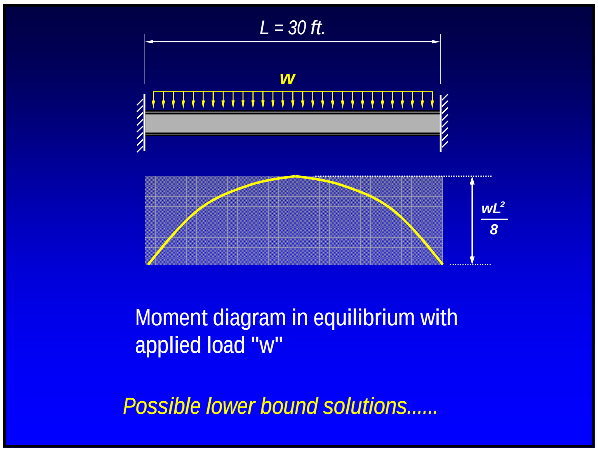
What is the load capacity for this beam ??

By lower bound theorem:

Choose any moment diagram in equilibrium with the applied load.

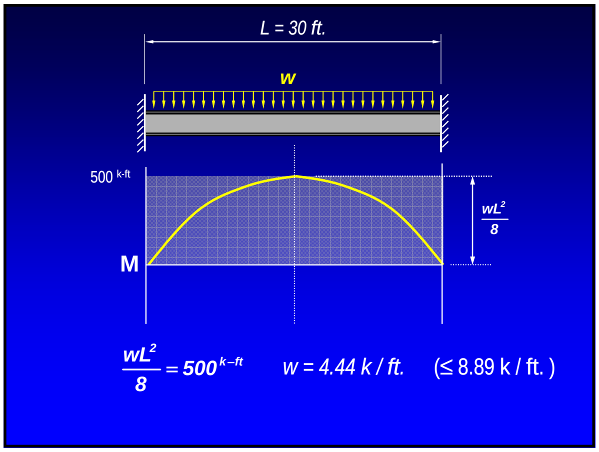
The moment cannot exceed M_p at any point along the beam.

The resulting load capacity "*w*" will be less than or equal to the true load capacity.

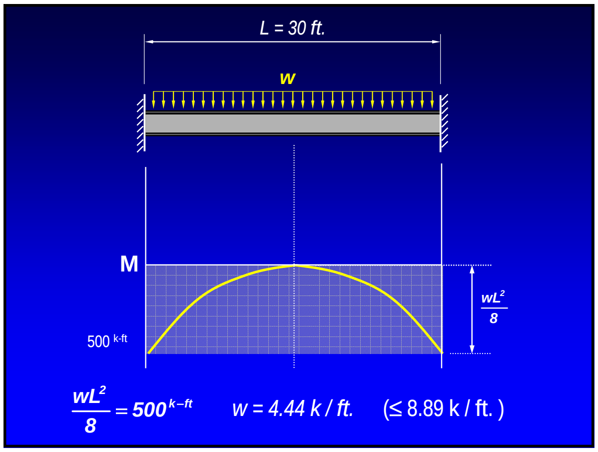


Moment diagram in equilibrium with applied load "w"

Possible lower bound solutions......



$\frac{wL^2}{8} = 500^{k-ft}$ *w = 4.44 k / ft.* (≤ 8.89 k / ft.)



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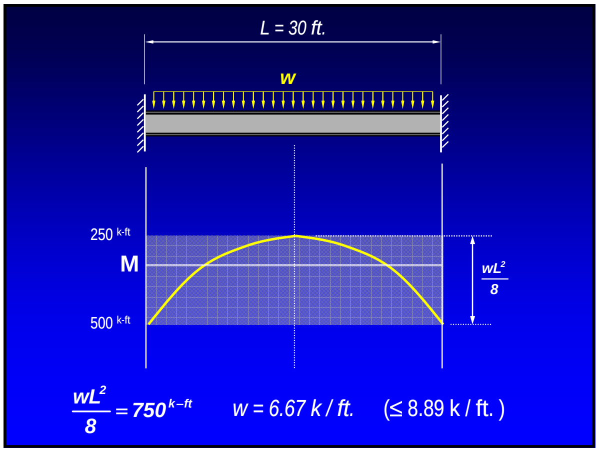

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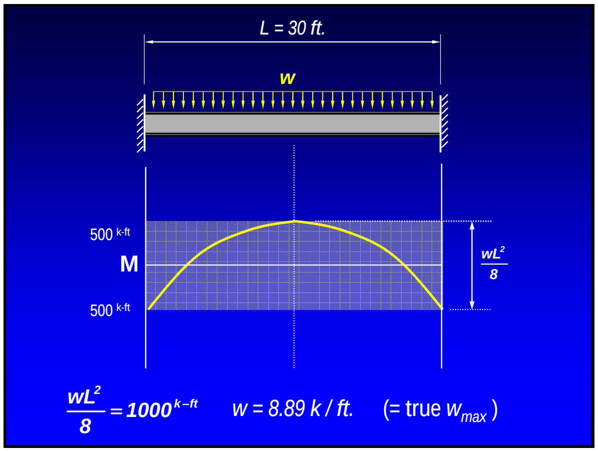
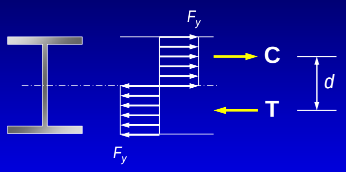


$\frac{wL^2}{8} = 1000^{k-ft}$ *w = 8.89 k / ft.* (= true w_{max})

Examples of lower bound theorem

Flexural capacity of steel section:



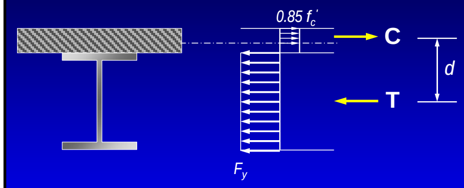
$\sigma \le F_y$

Equilibrium: $C = T$

$M_n = C * d = Z F_y$

Examples of lower bound theorem

Flexural capacity of a composite section:



$\sigma_{steel} \le F_y$ $\sigma_{conc} \le 0.85 f_c'$

Equilibrium: $C = T$

$M_n = C * d$

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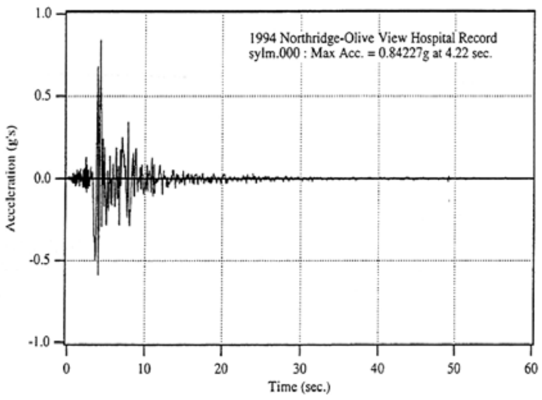
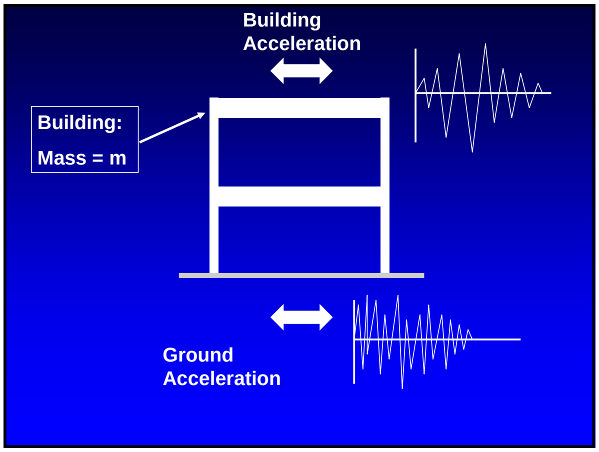
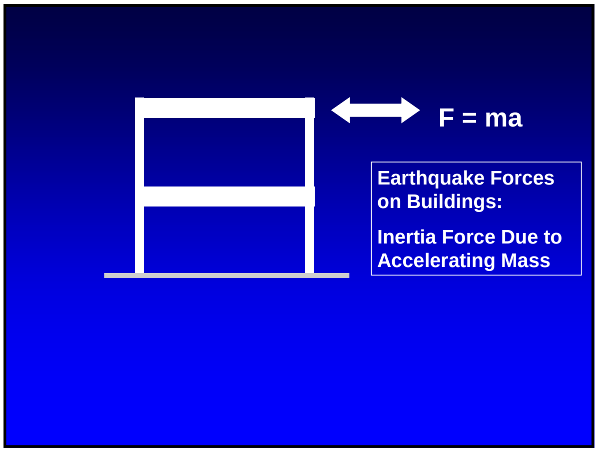






Earthquake Forces on Buildings:

Inertia Force Due to Accelerating Mass

Conventional Building Code Philosophy for Earthquake-Resistant Design

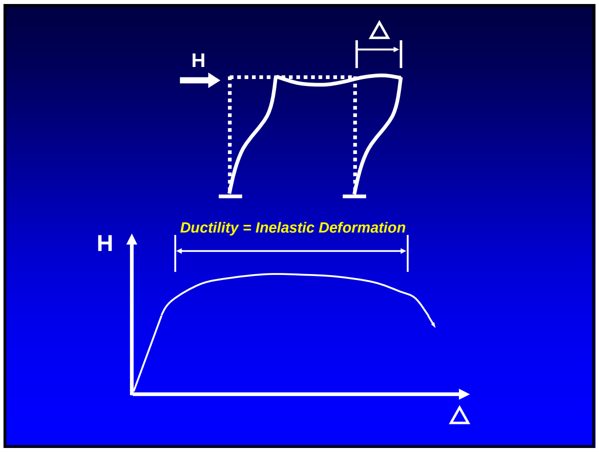
Objective: Prevent collapse in the extreme earthquake likely to occur at a building site.

Objectives are not to:

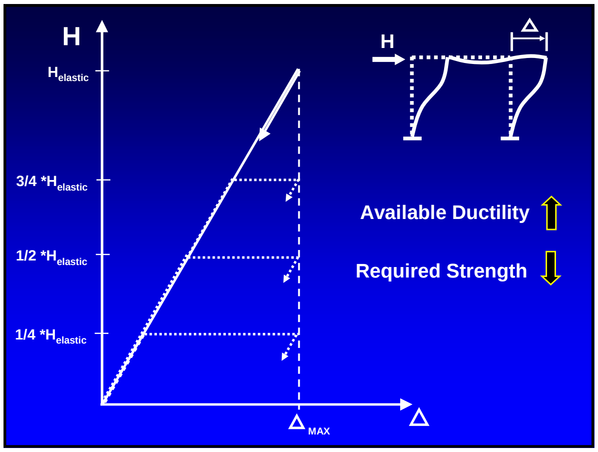
- limit damage
- maintain function
- provide for easy repair

To Survive Strong Earthquake without Collapse:

Design for Ductile Behavior



Ductility = Inelastic Deformation



Available Ductility ⇧

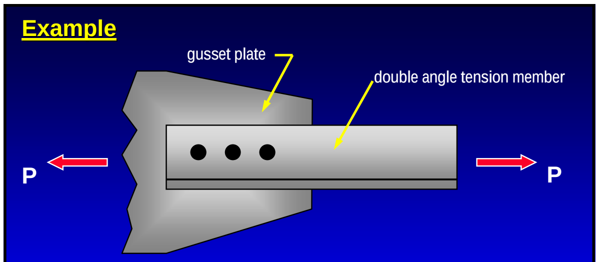
Required Strength ⇩

How Do We Achieve Ductility in Steel Structures ?

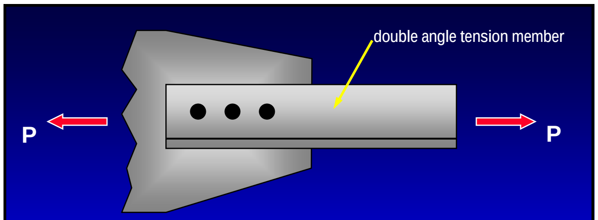
Achieving Ductile Response....

Ductile Limit States Must Precede Brittle Limit States

Example



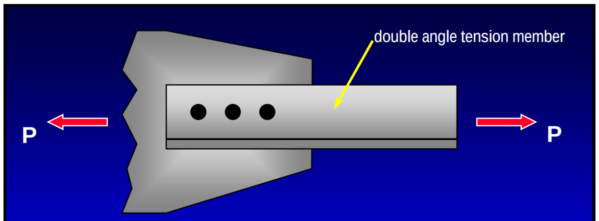
Ductile Limit State:	Gross-section yielding of tension member
Brittle Limit States:	Net-section fracture of tension member
	Block-shear fracture of tension member
	Net-section fracture of gusset plate
	Block-shear fracture of gusset plate
	Bolt shear fracture
	Plate bearing failure in double angles or gusset



Example: Gross-section yielding of tension member must precede net section fracture of tension member

Gross-section yield: $P_{yield} = A_g F_y$

Net-section fracture: $P_{fracture} = A_e F_u$



$P_{yield} \leq P_{fracture}$

The *required strength* for brittle limit states is defined by the capacity of the ductile element

$A_g F_y \leq A_e F_u$

$\frac{A_e}{A_g} \geq \frac{F_y}{F_u}$

$\frac{F_y}{F_u}$ = yield ratio

Steels with a low yield ratio are preferable for ductile behavior

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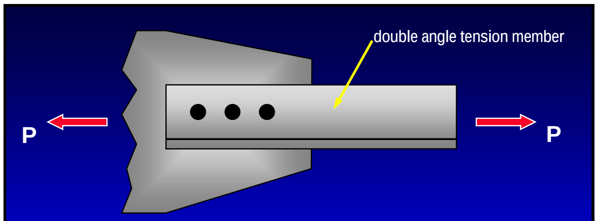
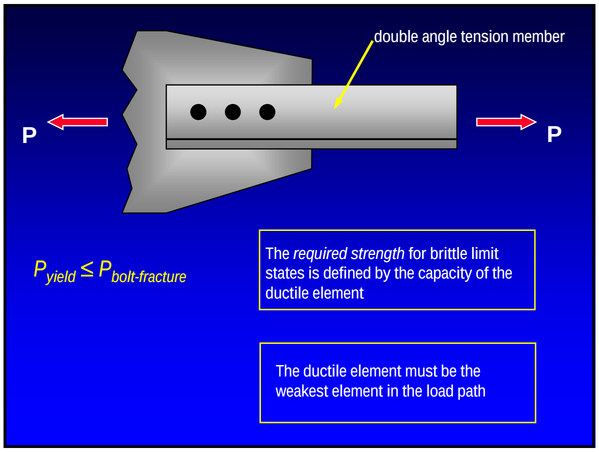


Example: Gross-section yielding of tension member must precede bolt shear fracture

Gross-section yield: $P_{yield} = A_g F_y$

Bolt shear fracture: $P_{bolt\text{-}fracture} = n_b n_s A_b F_v$

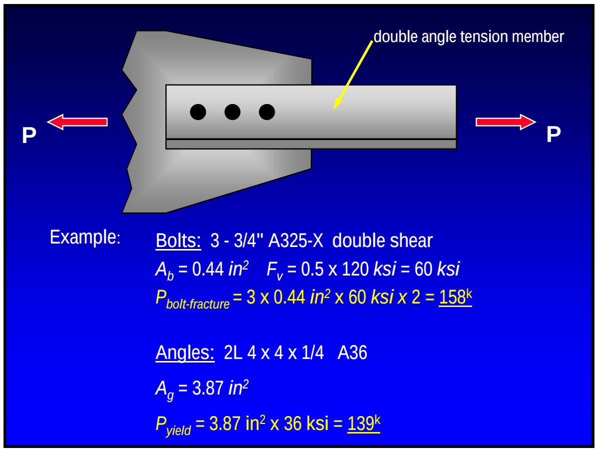
$$F_v = \begin{cases} 0.4 F_{u\text{-}bolt} & \text{-N} \\ 0.5 F_{u\text{-}bolt} & \text{-X} \end{cases}$$



$P_{yield} \leq P_{bolt\text{-}fracture}$

The *required strength* for brittle limit states is defined by the capacity of the ductile element

The ductile element must be the weakest element in the load path



Example:

Bolts: 3 - 3/4" A325-X double shear

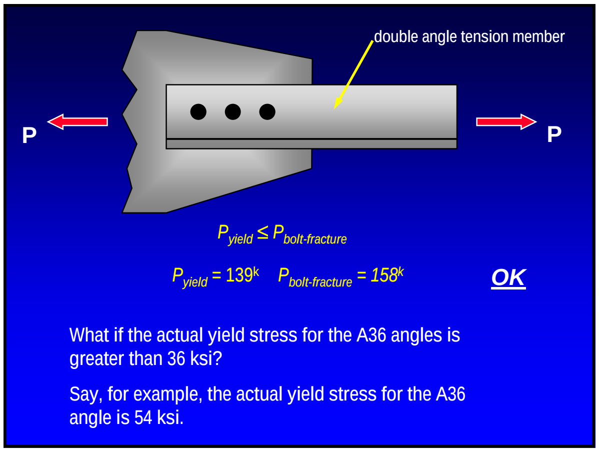
$A_b = 0.44\ in^2 \quad F_v = 0.5 \times 120\ ksi = 60\ ksi$

$P_{bolt\text{-}fracture} = 3 \times 0.44\ in^2 \times 60\ ksi \times 2 = \underline{158^k}$

Angles: 2L 4 x 4 x 1/4 A36

$A_g = 3.87\ in^2$

$P_{yield} = 3.87\ \text{in}^2 \times 36\ \text{ksi} = \underline{139^k}$



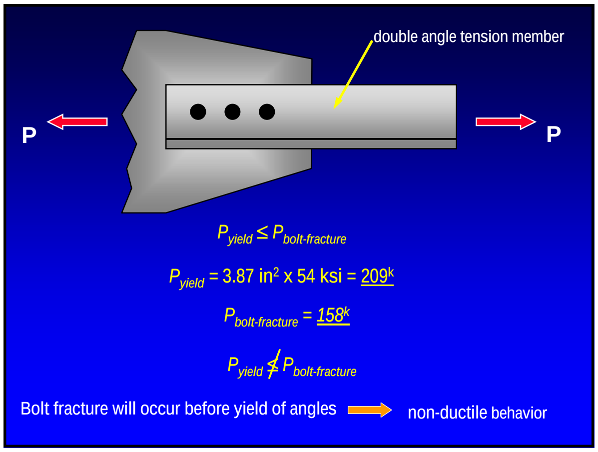
$P_{yield} \leq P_{bolt\text{-}fracture}$

$P_{yield} = 139^k \quad P_{bolt\text{-}fracture} = 158^k$

OK

What if the actual yield stress for the A36 angles is greater than 36 ksi?

Say, for example, the actual yield stress for the A36 angle is 54 ksi.



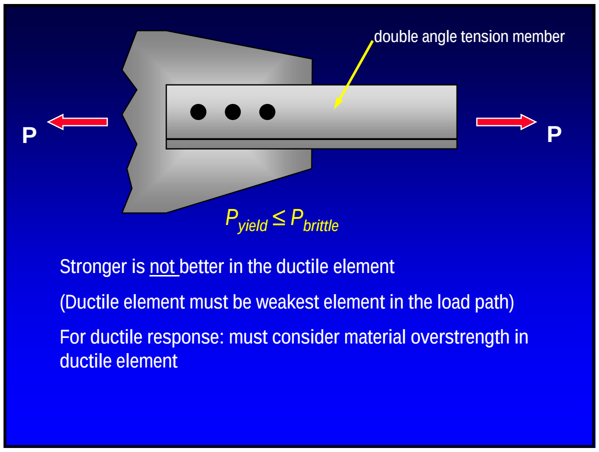
$P_{yield} \leq P_{bolt\text{-}fracture}$

$P_{yield} = 3.87\ \text{in}^2 \times 54\ \text{ksi} = \underline{209^k}$

$P_{bolt\text{-}fracture} = \underline{158^k}$

$P_{yield} \nleq P_{bolt\text{-}fracture}$

Bolt fracture will occur before yield of angles → non-ductile behavior

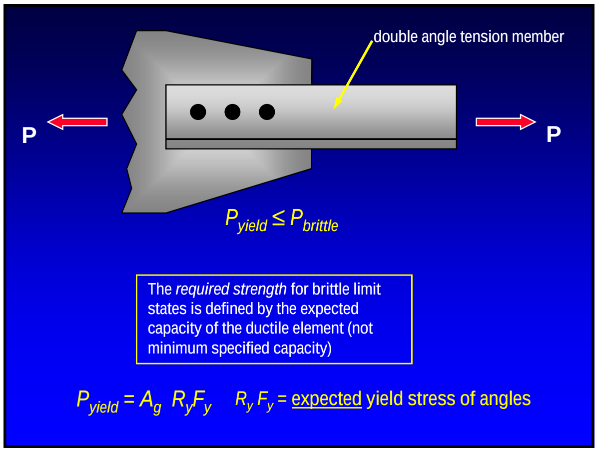


$P_{yield} \leq P_{brittle}$

Stronger is not better in the ductile element

(Ductile element must be weakest element in the load path)

For ductile response: must consider material overstrength in ductile element



$P_{yield} \leq P_{brittle}$

The *required strength* for brittle limit states is defined by the expected capacity of the ductile element (not minimum specified capacity)

$P_{yield} = A_g\ R_y F_y \qquad R_y\,F_y =$ expected yield stress of angles

Achieving Ductile Response....

Ductile Limit States Must Precede Brittle Limit States

Define the *required strength* for brittle limit states based on the expected yield capacity for ductile element

The ductile element must be the weakest in the load path

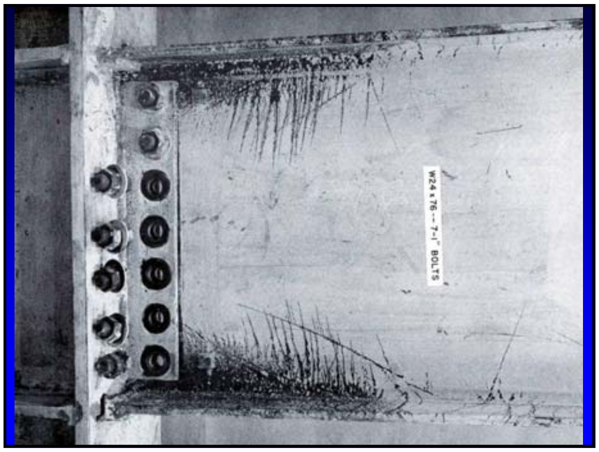
Unanticipated over strength in the ductile element can lead to non-ductile behavior.

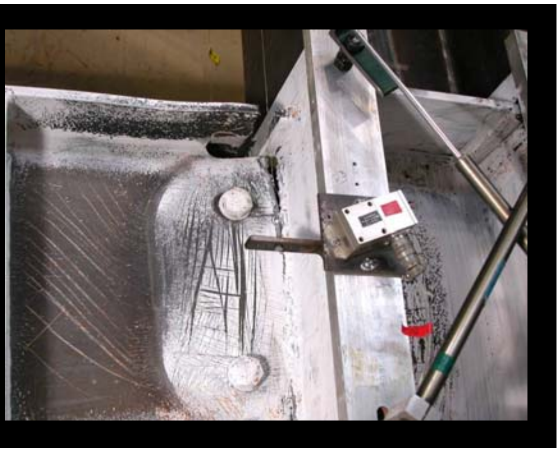
Steels with a low value of yield ratio, F_y / F_u are preferable for ductile elements

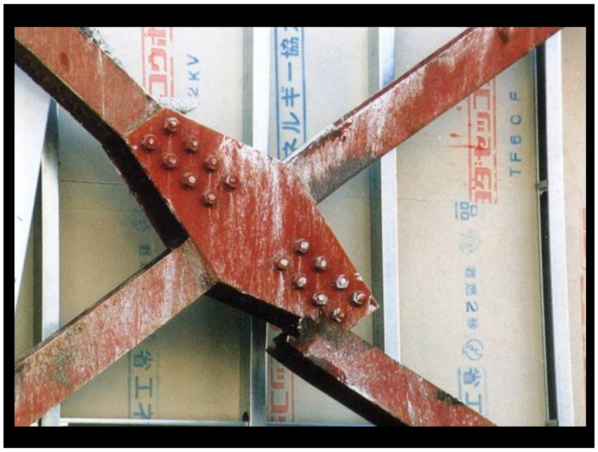
Achieving Ductile Response....

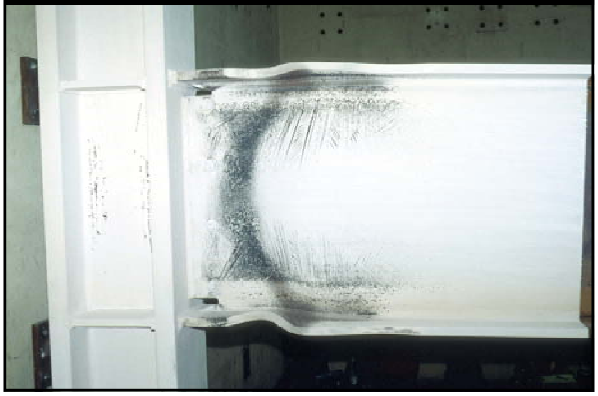
Connection response is generally non-ductile.....

Connections should be stronger than connected members



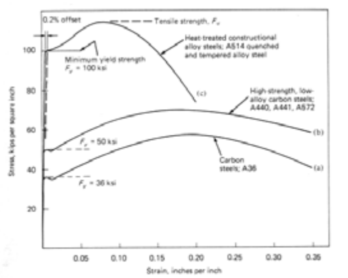






Achieving Ductile Response....

Be cautious of high-strength steels



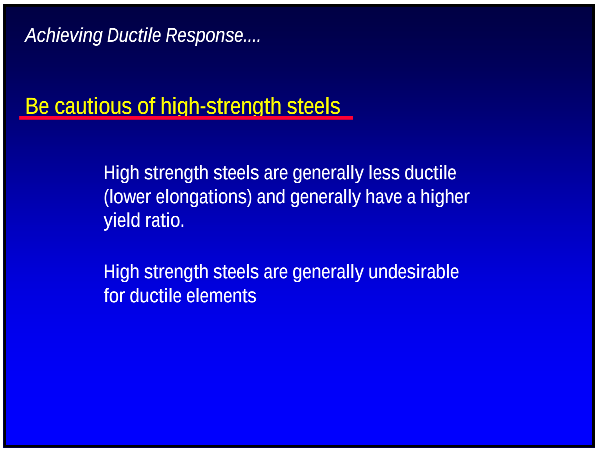
Ref: Salmon and Johnson - Steel Structures: Design and Behavior

General Trends:

As F_y ↑

Elongation (material ductility) ↓

F_y / F_u ↑

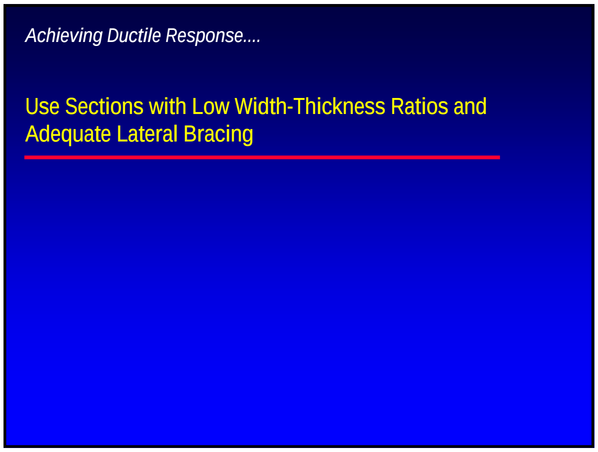


Achieving Ductile Response....

Be cautious of high-strength steels

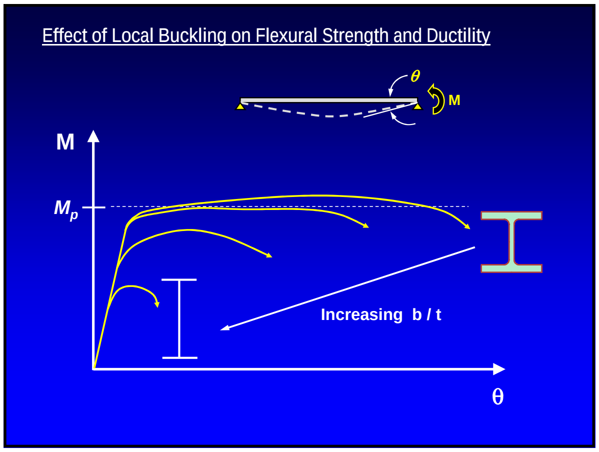
High strength steels are generally less ductile (lower elongations) and generally have a higher yield ratio.

High strength steels are generally undesirable for ductile elements

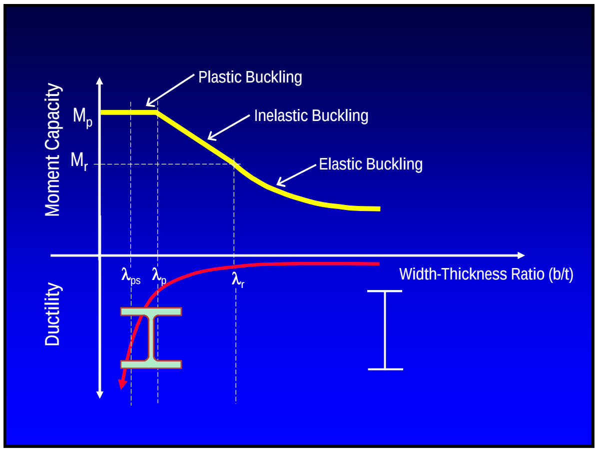


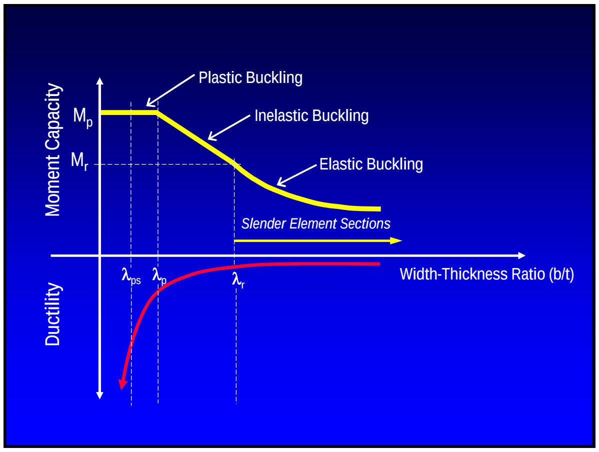
Achieving Ductile Response....

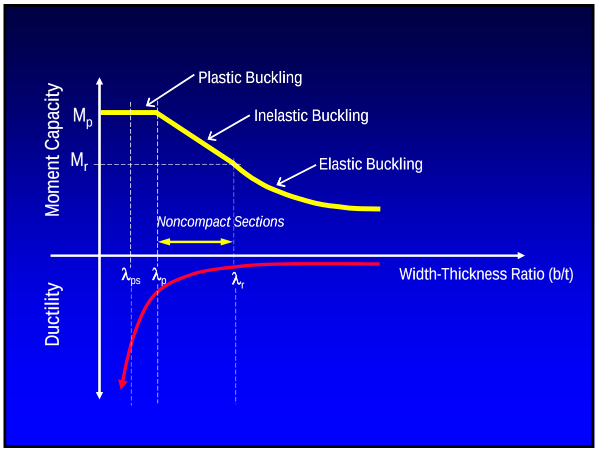
Use Sections with Low Width-Thickness Ratios and Adequate Lateral Bracing

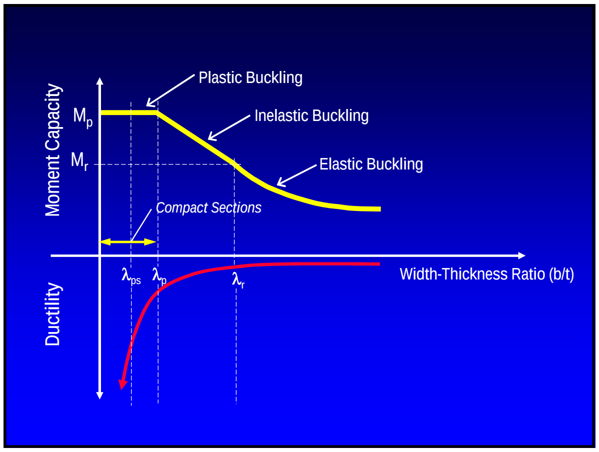


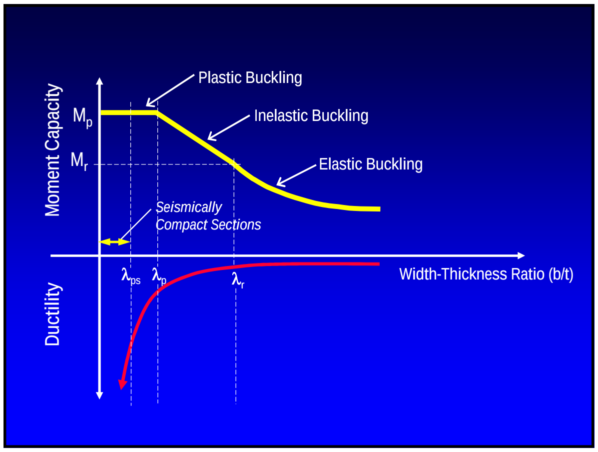
Effect of Local Buckling on Flexural Strength and Ductility

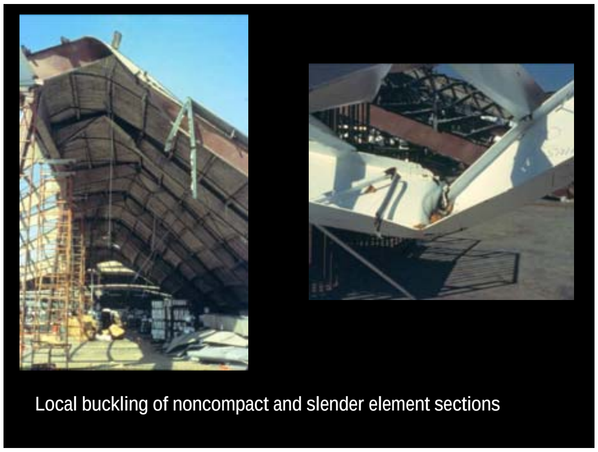


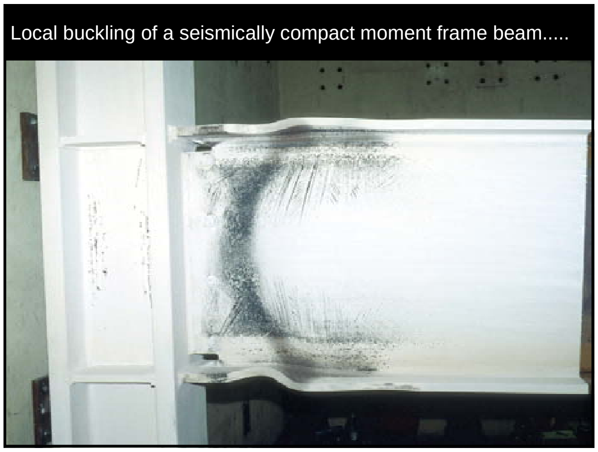


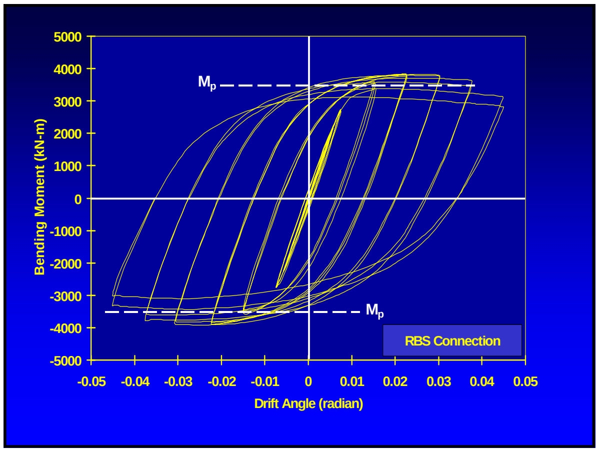




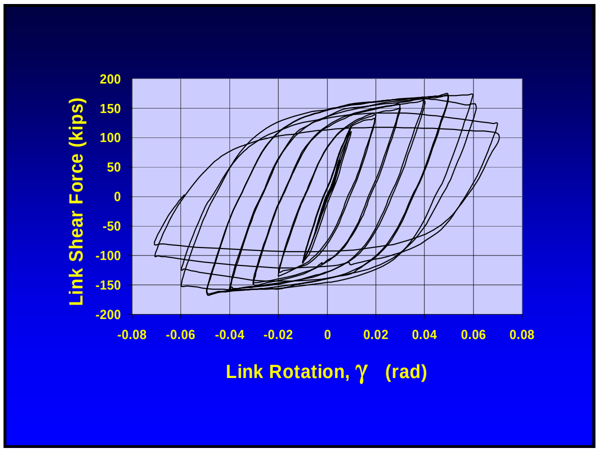


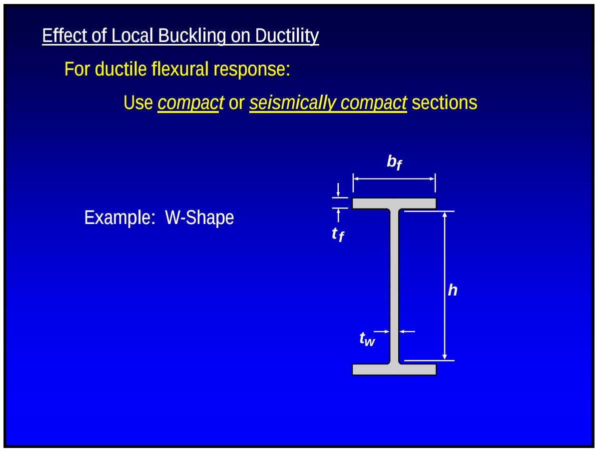

Local buckling of noncompact and slender element sections


Local buckling of a seismically compact moment frame beam.....




Local buckling of a seismically compact EBF link.....





Effect of Local Buckling on Ductility

For ductile flexural response:

Use *compact* or *seismically compact* sections

Example: W-Shape

Beam Flanges

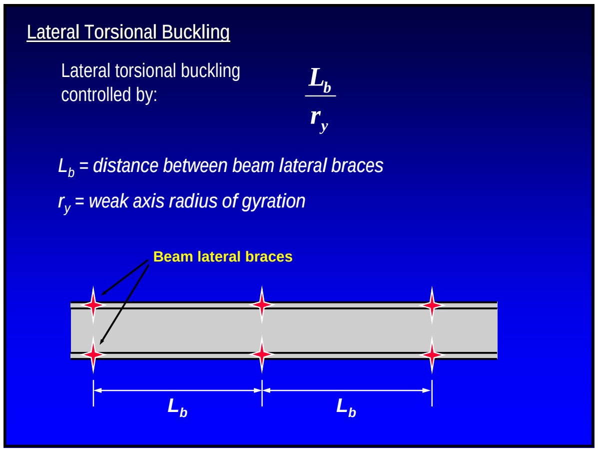
Compact: $\frac{b_f}{2t_f} \leq 0.38\sqrt{\frac{E_s}{F_y}}$

Seismically Compact: $\frac{b_f}{2t_f} \leq 0.30\sqrt{\frac{E_s}{F_y}}$

Beam Web

Compact: $\frac{h}{t_w} \leq 3.76\sqrt{\frac{E_s}{F_y}}$

Seismically Compact: $\frac{h}{t_w} \leq 2.45\sqrt{\frac{E_s}{F_y}}$

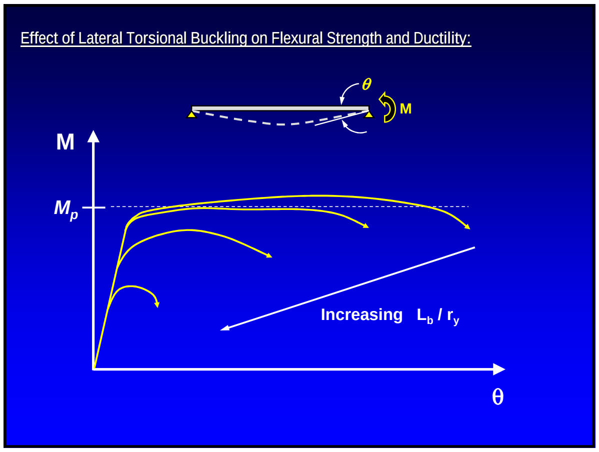


Lateral Torsional Buckling

Lateral torsional buckling controlled by: $\frac{L_b}{r_y}$

L_b = *distance between beam lateral braces*

r_y = *weak axis radius of gyration*



Effect of Lateral Torsional Buckling on Flexural Strength and Ductility:

Effect of Lateral Buckling on Ductility

For ductile flexural response:

Use lateral bracing based on *plastic design* requirements or *seismic design requirements*

Plastic Design: $L_b \le \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right)r_y$

Seismic Design: $L_b \le 0.086\left(\frac{E}{F_y}\right)r_y$

Achieving Ductile Response....

Recognize that buckling of a compression member is non-ductile

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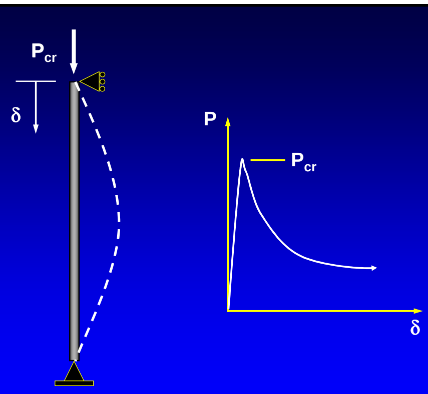
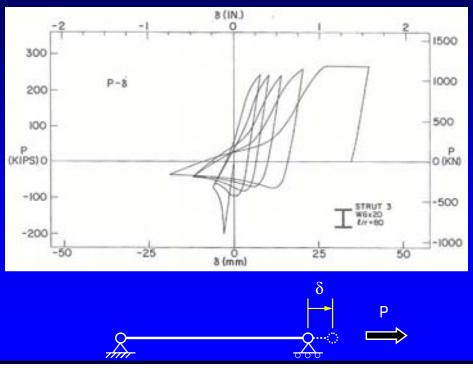


Experimental Behavior of Brace Under Cyclic Axial Loading

W6x20 Kl/r = 80



How Do We Achieve Ductile Response in Steel Structures ?

- Ductile limit states must precede brittle limit states
  - Ductile elements must be the weakest in the load path
  - Stronger is not better in ductile elements
  - Define *Required Strength* for brittle limit states based on expected yield capacity of ductile element
- Provide connections that are stronger than members
- Avoid high strength steels in ductile elements
- Use cross-sections with low b/t ratios
- Provide adequate lateral bracing
- Recognize that compression member buckling is non-ductile

How Do We Achieve Ductile Response in Steel Structures ?