University of California, San Diego
Faculty of Engineering

DESIGN OF FLEXURAL MEMBERS

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CONTENTS

INTRODUCTION

Beams

‘Beams are the most common members found in a typical steel structure. Beams are primarily loaded in bending about a primary axis of the member.

Beams with axial loads are called beam-columns, and these will be covered in Section 7.’

INTRODUCTION

Beams

‘Common types of beam are classified by the function that they serve:

- A girder is a member that is generally larger in section and supports other beams or framing members.
- A joist is typically a lighter section than a beam—such as an open-web steel joist.
- A stringer is a diagonal member that is the main support beam for a stair.
- A lintel (or loose lintel) is usually a smaller section that frames over a wall opening.
- A girt is a horizontal member that supports exterior cladding or siding for lateral wind loads.

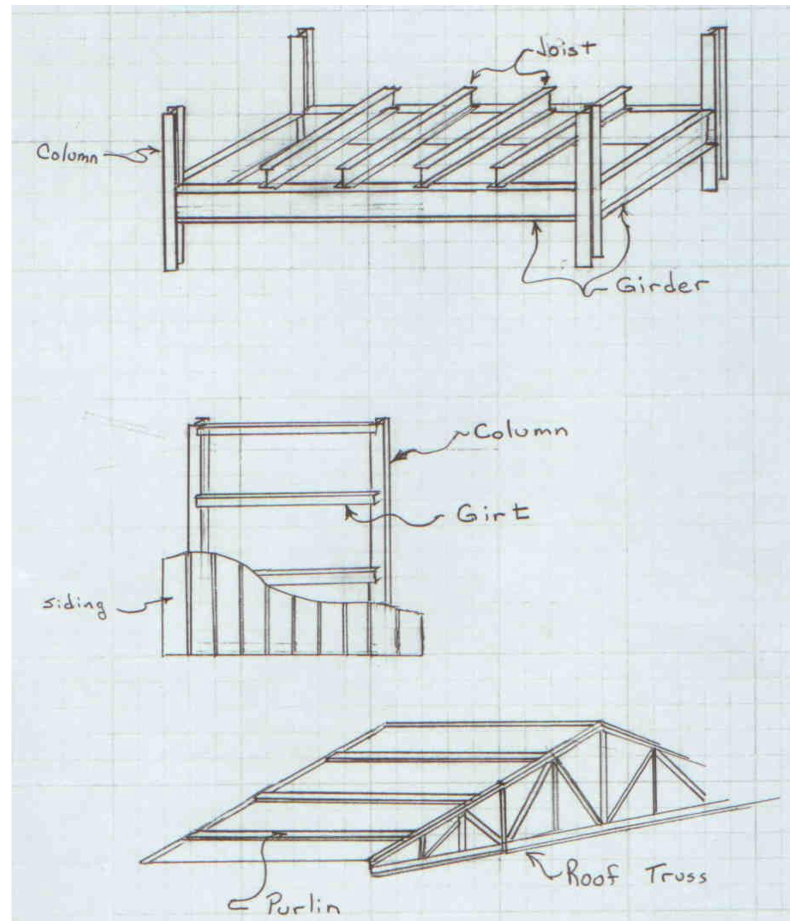
Beams exist in structures in several types of members.

Girders: Usually the most important beams at a wide spacing.

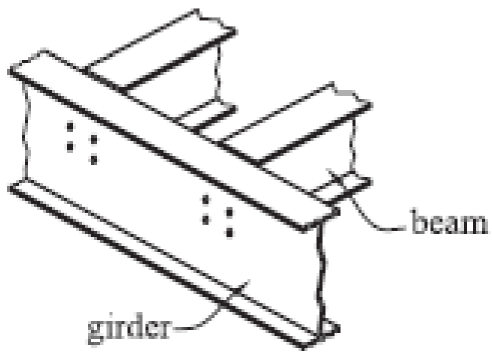
Joists: Less important beams that are closely spaced. Maybe W-shapes or often bar joists.

Girt: Horizontal wall beam. Metal siding is often connected to the girts.

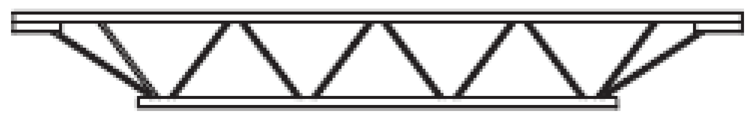
Purlin: Roof beams spanning between trusses.



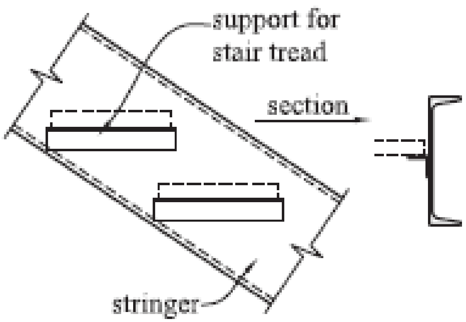
Beams exist in structures in several types of members.



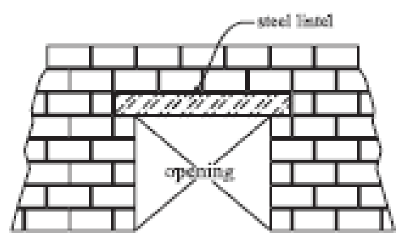
a. floor beams and girders



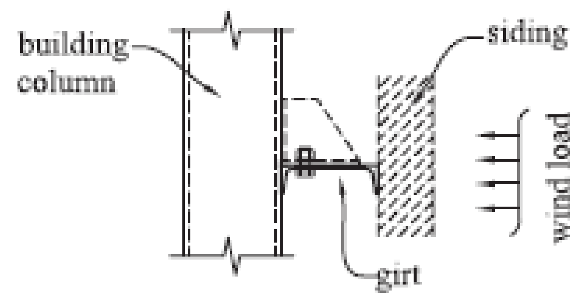
b. open-web steel joist



c. stringer



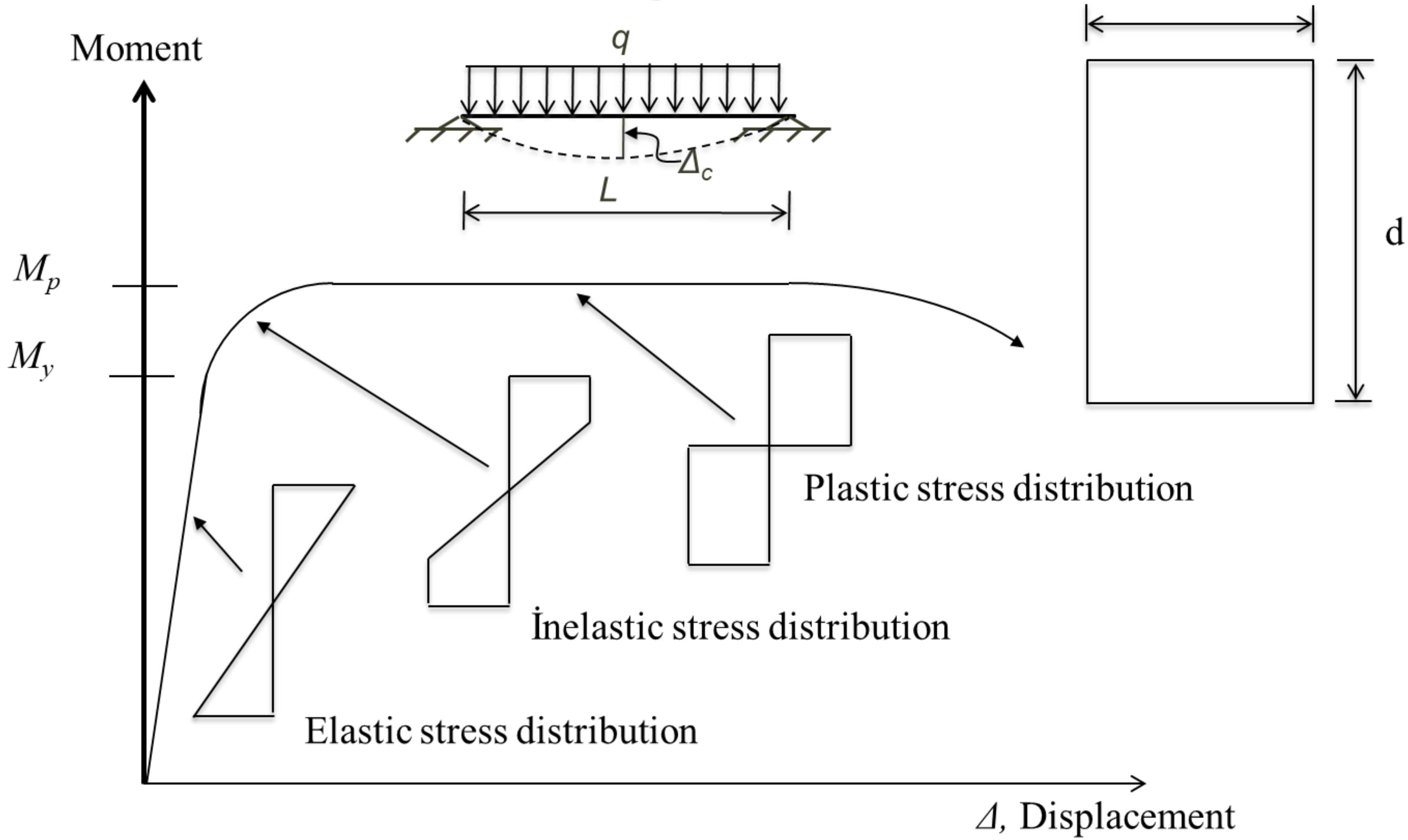
d. loose lintel



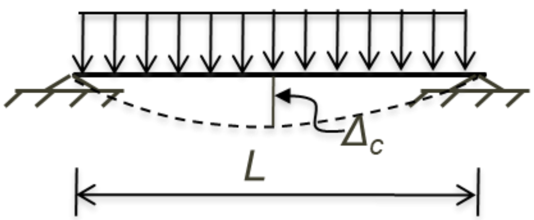
e. girt

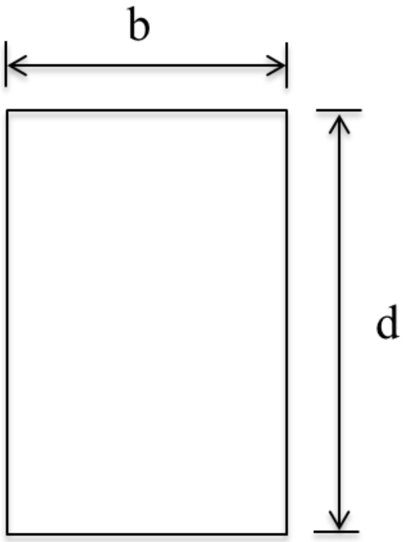
INTRODUCTION

Flexural Behavior of a Rectangular Cross Section



Flexural Behavior of a Rectangular Cross Section





Elastic Stage:

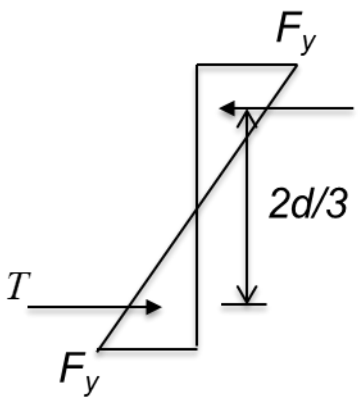
$$\sigma_x = \frac{M_x}{W_{ex}}$$

$$W_{ex} = \frac{I_x}{c}$$: elastic section modulus with respect to the x-axis

$$W_{ex} = \frac{bd^3/12}{d/2} = \frac{bd^2}{6}$$

Flexural Behavior of a Rectangular Cross Section

Yield Strength: M_y



$$C = \left(\frac{d}{2} x b\right)\frac{F_y}{2} = F_y \frac{bd}{4}$$

Equilibrium equation in the horizontal direction:

C=T

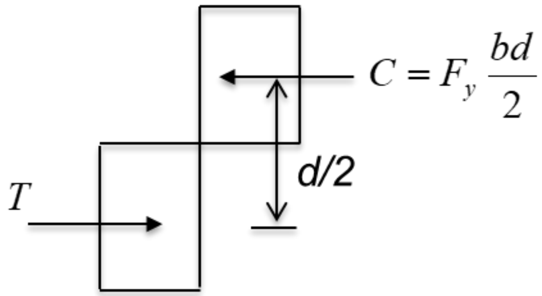
$$M_y = Cx\frac{2d}{3} = F_y \frac{bd}{4} x \frac{2d}{3} = F_y \frac{bd^2}{6}$$

$M_y = F_y W_{ex}$: Yield moment in strong axis

INTRODUCTION

Flexural Behavior of a Rectangular Cross Section

Plastik moment: M_p



Equilibrium equation in the horizontal direction: C=T

$$M_p = Cx\frac{d}{2} = F_y\frac{bd}{2}x\frac{d}{2} = F_y\frac{bd^2}{4}$$

$M_p = F_y W_{px}$: Plastic moment in strong axis

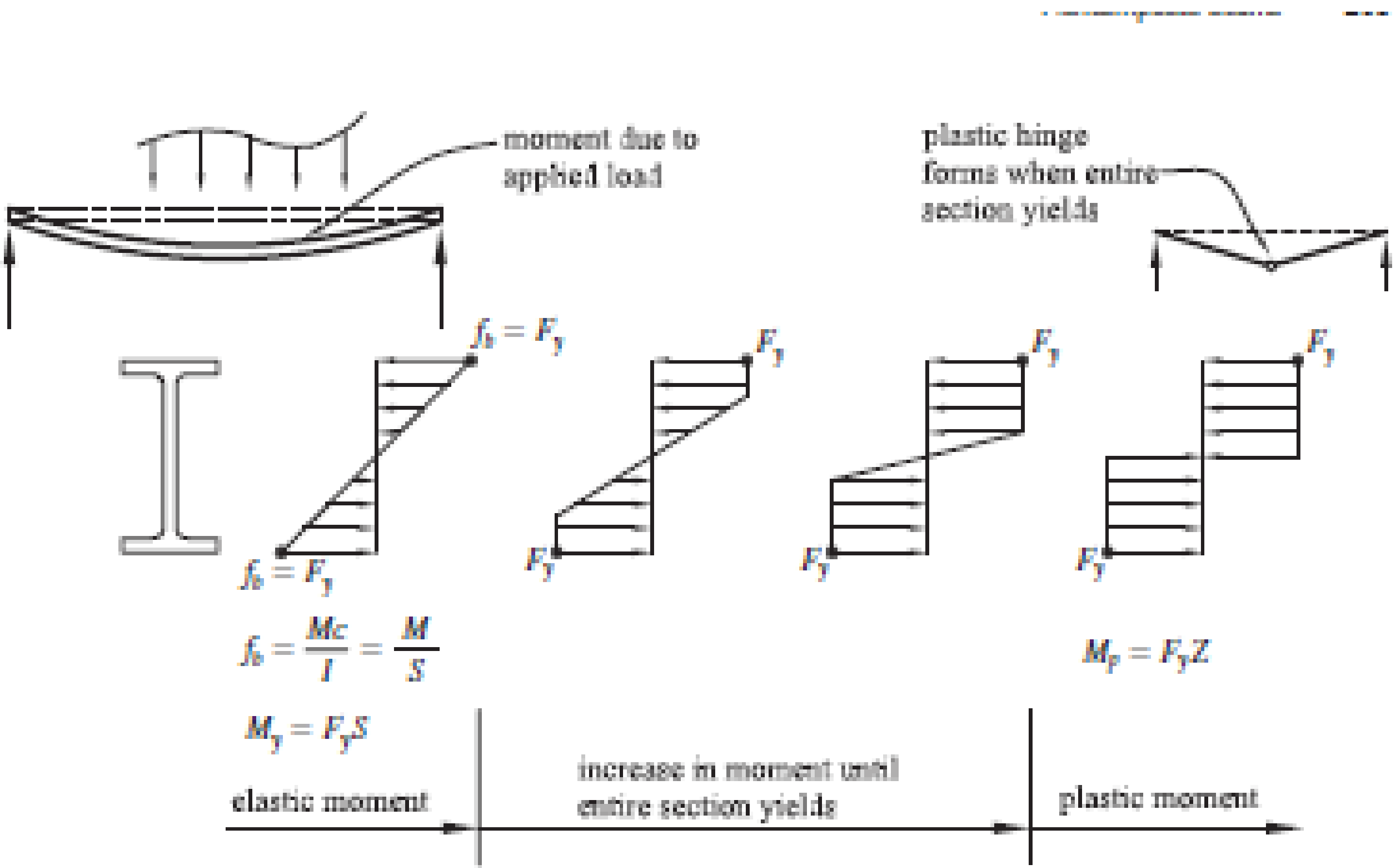
W_{px}: Plastic section modulus with respect to x-axis

W_{px} = Moment of the area with respect to the plastic centroid.

To calculate the plastic centroid: C=T

For a homogeneous section: A_t=A_c

Flexural Behavior of a Rectangular Cross Section



Flexural Behavior of a Rectangular Cross Section

Shape Factor of the cross section = $\xi = M_p/M_y$

For a rectangular cross section:

$$\xi = \frac{M_p}{M_y} = \frac{F_y bd^2/4}{F_y bd^2/6}$$

$$\xi = 1.50$$

The plastic strength of a rectangular section is %50 higher than its yield strength.

Importance of Shape Factor

$$M_u \leq \phi M_p$$

$$M_u = M_{service} L.F.$$

$$M_p = \xi M_y$$

$$M_{service} \leq \frac{\phi M_y}{L.F.} \xi$$

$$\frac{M_{service}}{M_y} \leq \frac{\phi}{L.F.} \xi$$

L.F. = Load Factors

$$\left.\begin{array}{l} L.F.\ is\ small \\ \xi\ is\ high \end{array}\right\} \frac{\phi}{L.F.} \xi \rangle 1.0 \Rightarrow$$ Yielding under service loads

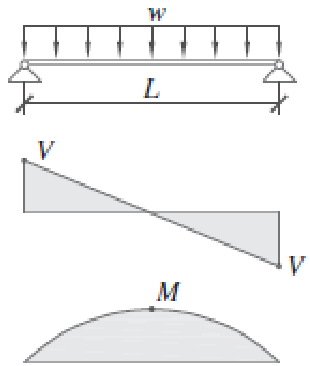
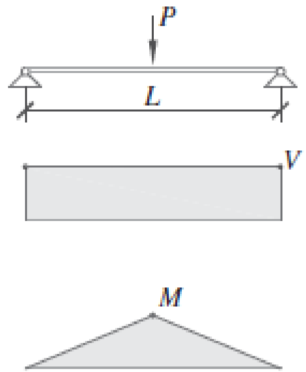
Consequences of yielding under service loads:

<u>Serviceability Problems</u>

High displacement

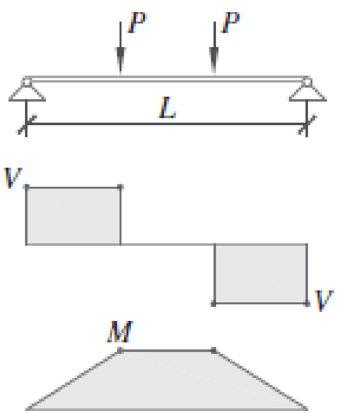
Permanent displacements

Summary of Shear, Moment and Deflection Formulas

Loading	Loading Diagram	Maximum Shear	Maximum Moment	Maximum Deflection
Uniformly loaded simple span	 *a. uniformly loaded*	$V = \frac{wL}{2}$	$M = \frac{wL^2}{8}$	$\Delta = \frac{5wL^4}{384EI}$
Concentrated load at midspan	 *b. concentrated load at midspan*	$V = \frac{P}{2}$	$M = \frac{PL}{4}$	$\Delta = \frac{PL^3}{48EI}$

Summary of Shear, Moment and Deflection Formulas

Concentrated loads at ⅓ points



c. concentrated loads at ⅓ points

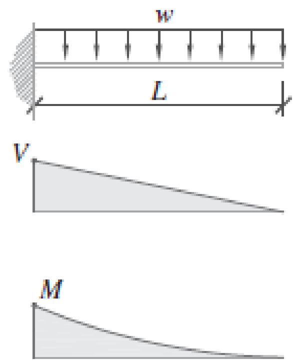
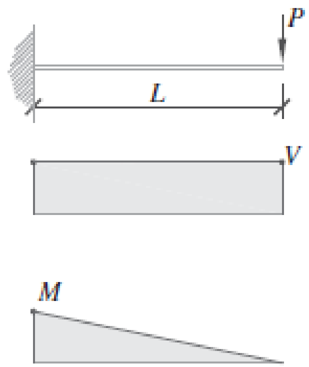
$V = P$

$M = \frac{PL}{3}$

$\Delta = \frac{PL^3}{28EI}$

INTRODUCTION

Summary of Shear, Moment and Deflection Formulas

Loading	Loading Diagram	Maximum Shear	Maximum Moment	Maximum Deflection
Uniformly loaded, cantilever	 *d. uniformly loaded, cantilever*	$V = wL$	$M = \frac{wL^2}{2}$	$\Delta = \frac{wL^4}{4EI}$
Concentrated load at end of cantilever	 *e. concentrated load at end of cantilever*	$V = P$	$M = PL$	$\Delta = \frac{PL^3}{3EI}$

CLASSIFICATION OF BEAMS

Beams

All flexural members are classified as either:

- Compact,
- Noncompact, or
- Slender,

depending on the width-to-thickness ratios of the individual elements that form the beam section.

There are also two type of elements that are defined in the AISC specification:

- Stiffened
- Unstiffened

CLASSIFICATION OF BEAMS

Beams

Stiffened elements are supported along both edges parallel to the load. An example of this is the web of an I-shaped beam because it is connected to flanges on either end of the web.

An unstiffened element has only one unsupported edge parallel to the load; an example of this is the outstanding flange of an I-shaped beam that is connected to the web on one side and free on the other end.

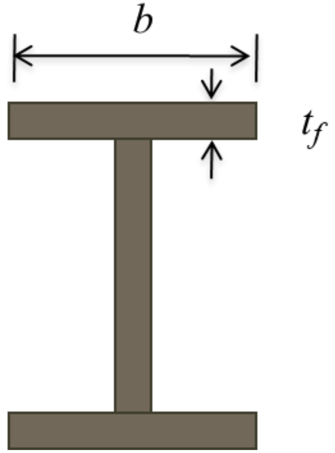
Table 6-2 gives the upper limits for the width-to-thickness ratios for the individual elements of a beam section. These ratios provide the basis for the beam section. When the width-to-thickness ratio is less than λ_p, then the section is compact. When the ratio is greater than λ_p but less than λ_r, then the shape is noncompact. When the ratio is greater than λ_r, the section is classified as slender.

CLASSIFICATION OF BEAMS

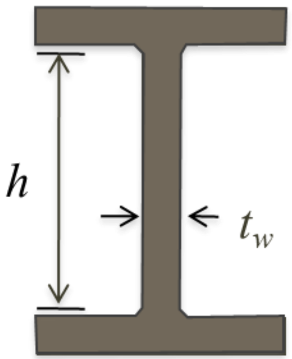
Beams.

Compact, Noncompact and Slender Sections: **Table 5.1B**

Flange: $b/2t_f$



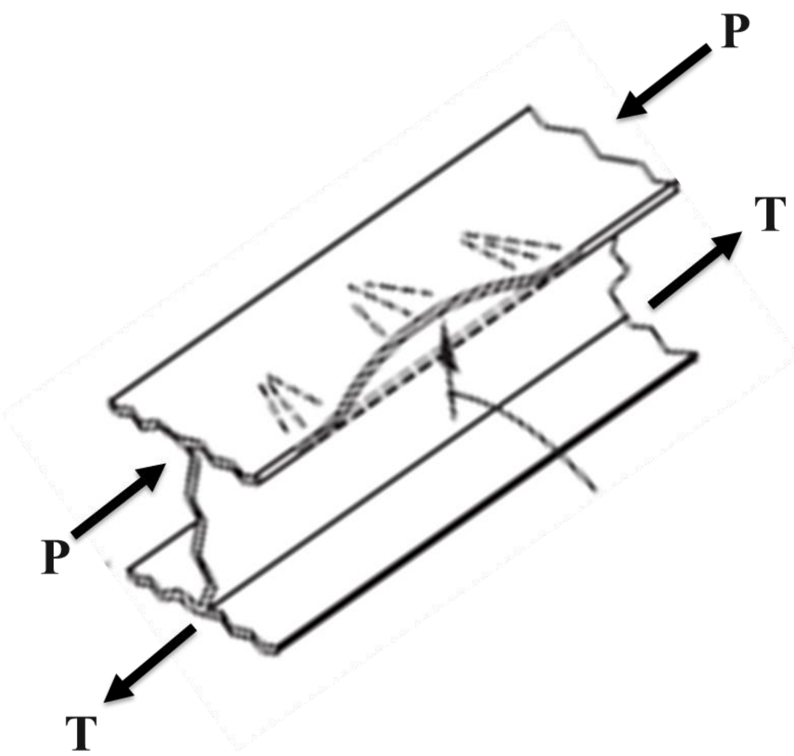
Web: h/t_w



CLASSIFICATION OF BEAMS

Beams: Local Buckling

Local buckling leads to a reduction in the flexural strength of a beam member and prevents the member from reaching its overall flexural capacity, plastic moment, M_p.



CLASSIFICATION OF BEAMS

Beams: Local Buckling

To avoid or prevent local buckling, the AISC and Turkish specifications prescribes limits to the width-to-thickness ratios of the plate components that make up the structural member. These limits are given in section B4 of the AISCM and section 5.4. In Section B4 or 5.4 three possible local stability parameters are defined: **compact, noncompact, or slender**.

Compact section: reaches its cross-sectional material strength, or capacity, before local buckling occurs.

N**oncompact section**: only a portion of the cross-section reaches its yield strength before local buckling occurs.

Slender section: the cross-section does not yield and the strength of the member is governed by local buckling.

CLASSIFICATION OF BEAMS

Beams: Local Buckling

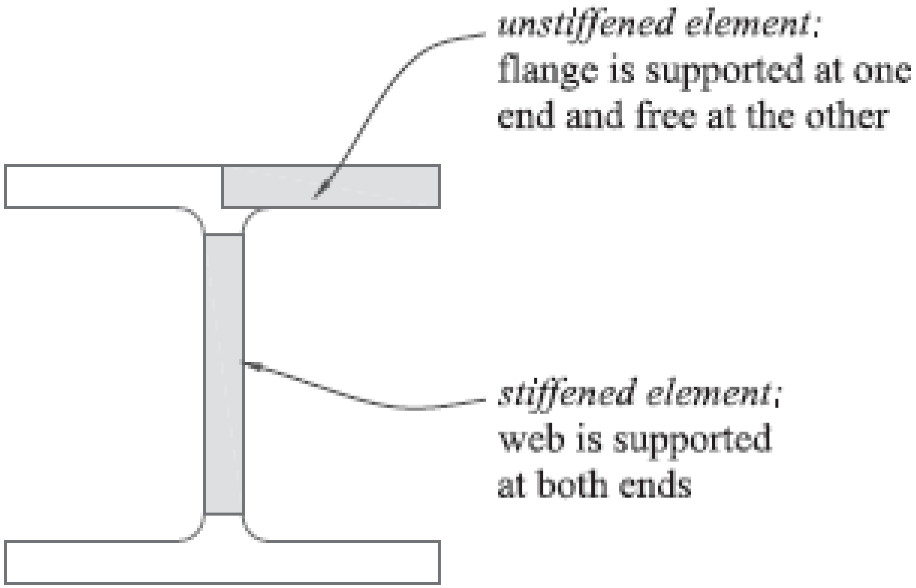
There are two type of elements of a column section that are defined in the YÖNETMELİK and AISC: stiffened and unstiffened.

Stiffened elements are supported along both edges parallel to the applied axial load. An example of this is the web of an I-shaped column where the flanges are connected on either end of the web. An **unstiffened element** has only one unsupported edge parallel to the axial load—for example, the outstanding flange of an I-shaped column that is connected to the web on one edge and free along the other edge.

CLASSIFICATION OF BEAMS

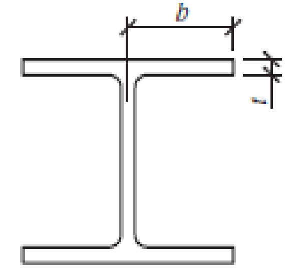
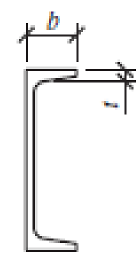
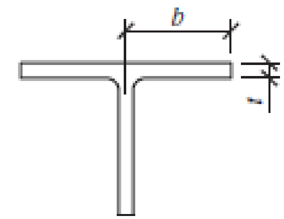
Beams: Local Buckling

The limiting criteria for compact, noncompact, and slender elements as a function of the width-to-thickness ratio is shown in Yönetmelik Table 5-1B.



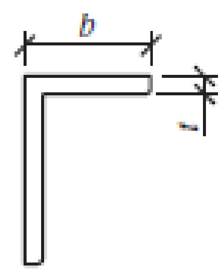
CLASSIFICATION OF BEAMS

Limiting Width-Thickness Ratios ForFlexural Elements

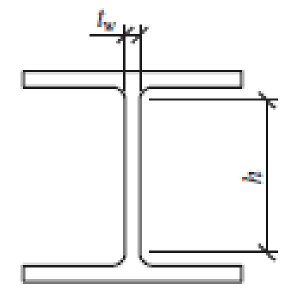
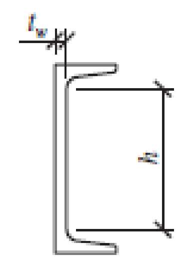
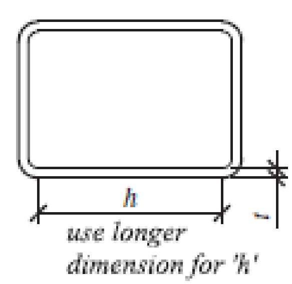
	Decsription	Limiting Width–Thickness Ratio: λ_p, λ_{pf} (flange), λ_{pw} (web) (compact)	Limiting Width–Thickness Ratio: λ_r, λ_{rf} (flange), λ_{rw} (web) (noncompact)	Details
Unstiffened	Flanges of I-shaped sections			
	Flanges of C-shapes	$\frac{b}{t} \leq 0.38\sqrt{\frac{E}{F_y}}$	$\frac{b}{t} \leq 1.0\sqrt{\frac{E}{F_y}}$	
	Flanges of WT-shapes			

CLASSIFICATION OF BEAMS

Limiting Width-Thickness Ratios ForFlexural Elements

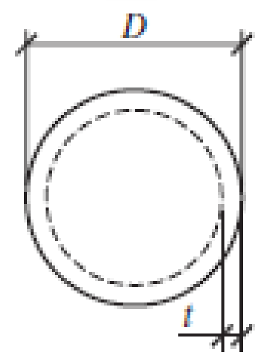
	Decsription	Limiting Width–Thickness Ratio λ_p λ_{pf} (flange) λ_{pw} (web) (compact)	Limiting Width–Thickness Ratio λ_r λ_{rf} (flange) λ_{rw} (web) (noncompact)	Details
Unstiffened	Outstanding legs of single angles	$\frac{b}{t} \leq 0.54\sqrt{\frac{E}{F_y}}$	$\frac{b}{t} \leq 0.91\sqrt{\frac{E}{F_y}}$	

CLASSIFICATION OF BEAMS

	Decsription	Limiting Width-Thickness Ratio: λ_p λ_{pf} (flange) λ_{pw} (web) (compact)	Limiting Width-Thickness Ratio: λ_r λ_{rf} (flange) λ_{rw} (web) (noncompact)	Details
Stiffened	Webs of I-shaped sections	$\frac{h}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}$	$\frac{h}{t_w} \leq 5.70\sqrt{\frac{E}{F_y}}$	
	Webs of C-shapes			
	Square or rectangular HSS	$\frac{h}{t} \leq 2.42\sqrt{\frac{E}{F_y}}$	$\frac{h}{t} \leq 5.70\sqrt{\frac{E}{F_y}}$	

CLASSIFICATION OF BEAMS

Limiting Width-Thickness Ratios ForFlexural Elements

	Decsription	Limiting Width-Thickness Ratio λ_p λ_{pf} (flange) λ_{pw} (web) (compact)	Limiting Width-Thickness Ratio λ_r λ_{rf} (flange) λ_{rw} (web) (noncompact)	Details
Stiffened		$\frac{\ }{t} \leq 0.07 \frac{\ }{F_y}$	$\frac{\ }{t} \leq 0.31 \frac{\ }{F_y}$	

CLASSIFICATION OF BEAMS

Beams

The classification of a beam is necessary since the design strength of the beam is a function of its classification for flange and web local buckling.

CLASSIFICATION OF BEAMS

Example 6.1:

Determine the classification of a HE 300 A and a HE 500 A for Fy = 355 MPa. Check both the flange and the web.

Flanges

HE 300 A

$$\frac{b_f}{2t_f} = \frac{300^{mm}}{2x14^{mm}} = 10.7$$

HE 400 A

$$\frac{b_f}{2t_f} = \frac{300^{mm}}{2x19^{mm}} = 7.9$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 9.02 > 7.9$$

HE 400 A flange is compact

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}} = 1.0\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 23.7 > 10.7 > 9.02$$

HE 300 A flange is noncompact

CLASSIFICATION OF BEAMS

Example 6.1:

Determine the classification of a HE 300 A and a HE 500 A for Fy = 355 MPa. Check both the flange and the web.

Webs

HE 300 A

$$\frac{h}{t_w} = \frac{208^{mm}}{8.5^{mm}} = 24.5$$

HE 400 A

$$\frac{h}{t_w} = \frac{298^{mm}}{11^{mm}} = 27.1$$

$$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 89.2 > 24.5 \quad and \quad 27.1$$

$$\lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 135.3$$

HE 400 A and 300 A webs are compact

INTRODUCTION

Design Checks for Beams

‘The basic design checks for beams includes checking:

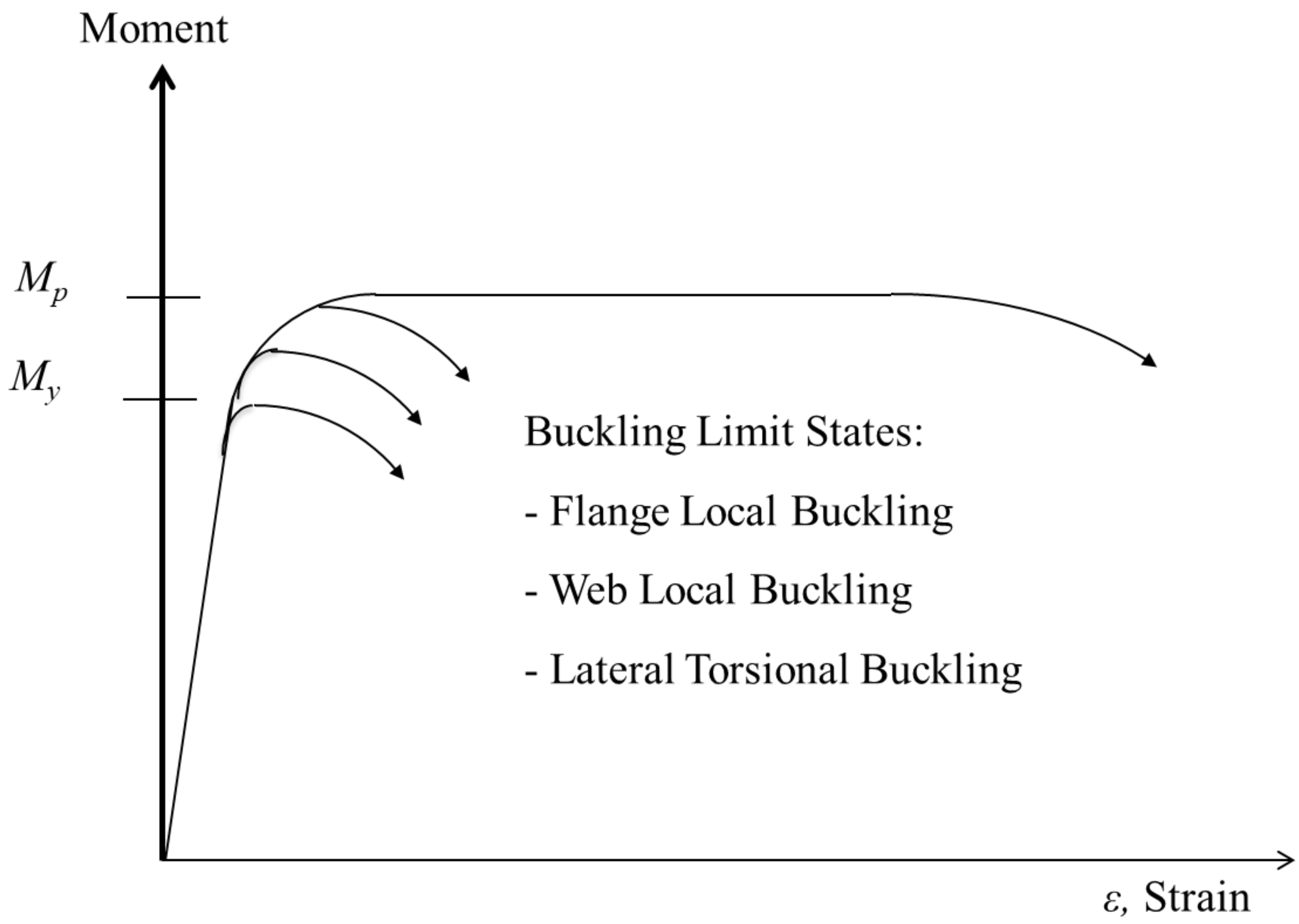
- Bending (Flexure),
- Shear,
- Deflection.

The loading conditions and beam configuration will dictate which of the preceding design parameters controls the size of the beam.’

Limit States For Flexure:

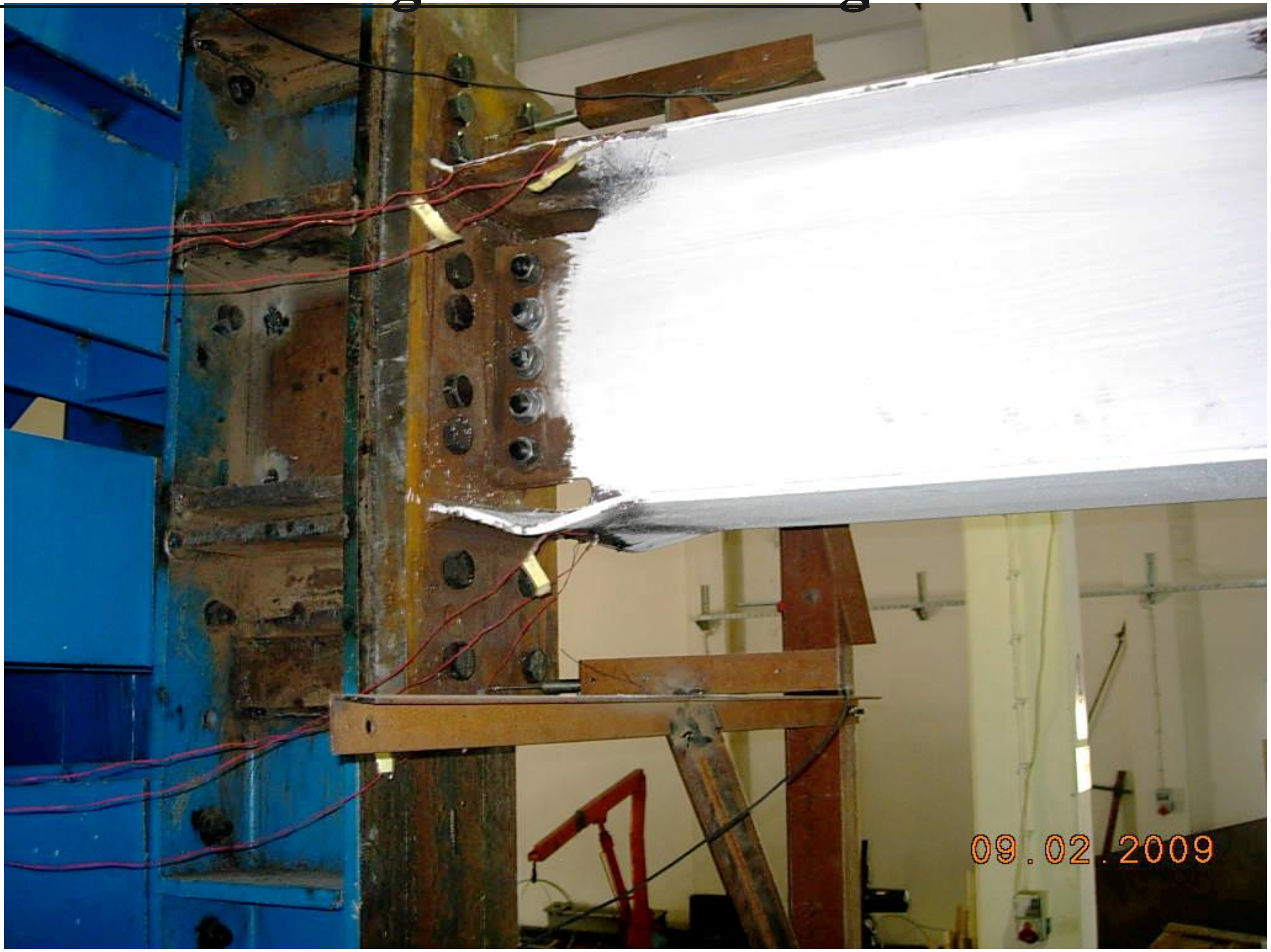
- Flange Local Buckling
- Web Local Buckling
- Lateral Torsional Buckling
- Yielding of Tension or Compression Flange

DESIGN CHECK FOR BEAMS



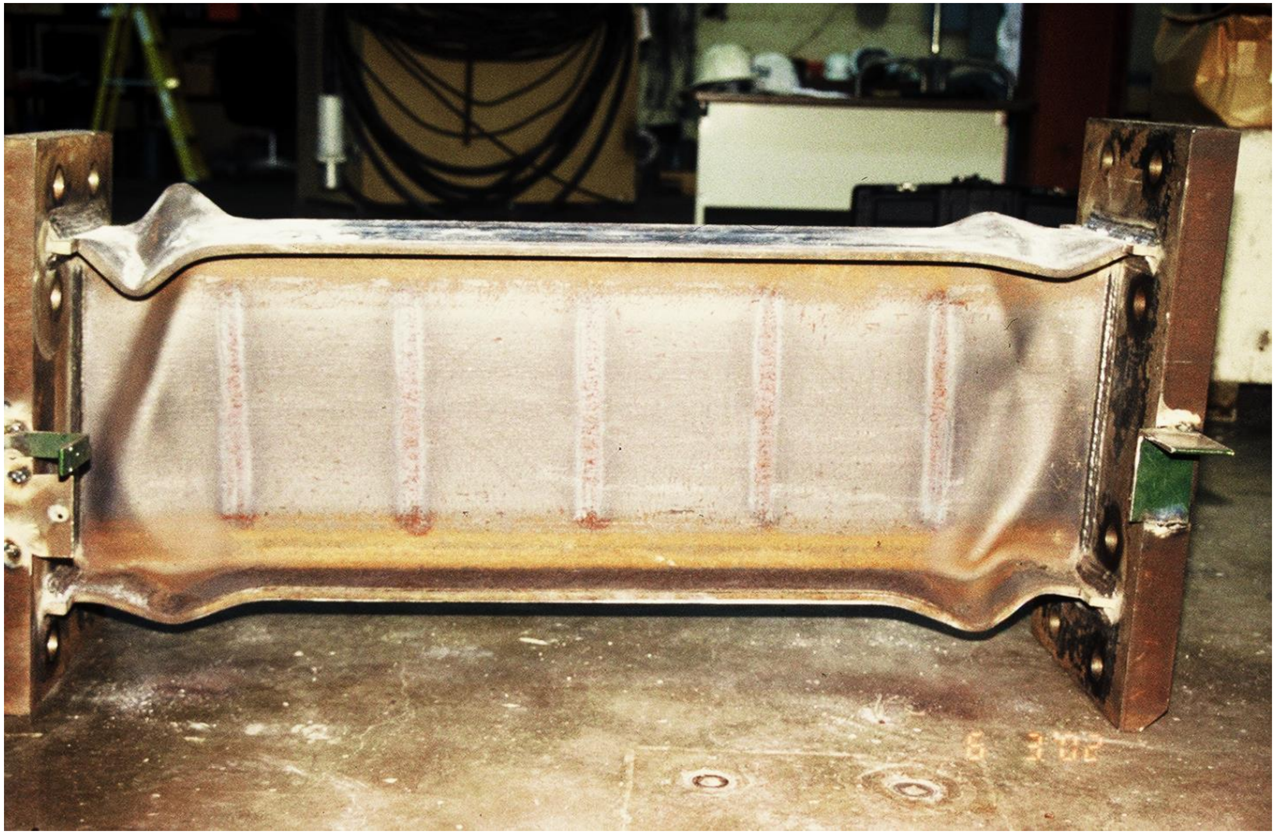
DESIGN CHECK FOR BEAMS

Flexural Behavior: Flange Local Buckling



DESIGN CHECK FOR BEAMS

Flexural Behavior: Flange and Web Local Buckling



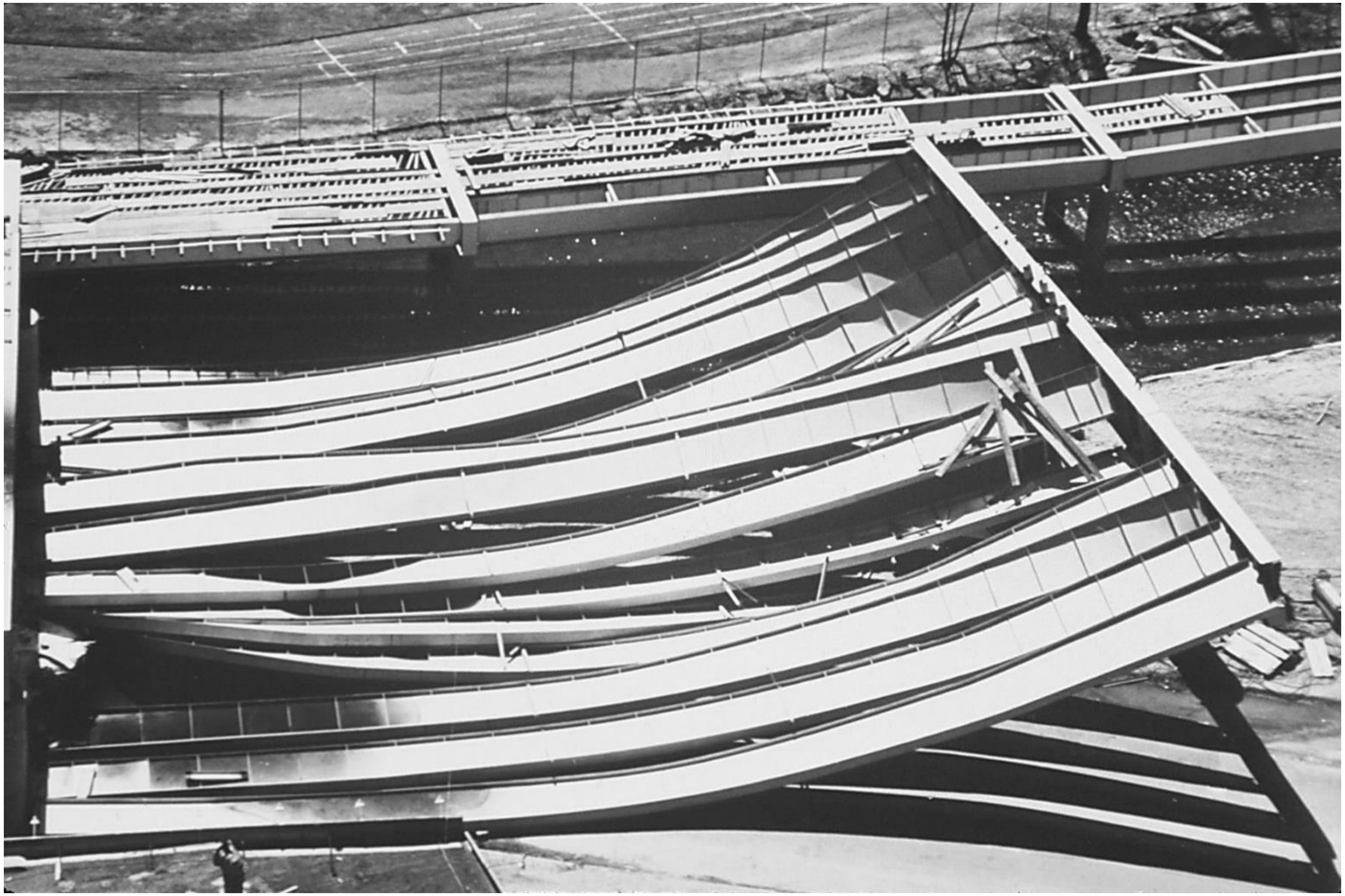
Photograph: Courtesy of Prof. Engelhardt

Flexural Behavior: Flexural Torsional Buckling

Photograph: Courtesy of Prof. Engelhardt

DESIGN CHECK FOR BEAMS

Flexural Behavior: Flexural Torsional Buckling



Photograph: Courtesy of Prof. Engelhardt

DESIGN CHECK FOR BEAMS

Flexural Behavior: Flexural Torsional Buckling

Photograph: Courtesy of Prof. Engelhardt

DESIGN CHECK FOR BEAMS

9.1 General (Turkish Steel Specification)

For every flexural element:

$$\phi_t = 0.90\,(YDKT) \qquad \Omega_t = 1.67\,(GKT)$$

$$M_u \le \phi_t M_n \quad (YDKT) \qquad M_a \le \frac{M_n}{\Omega} \quad (GKT)$$

M_n = Nominal flexural strength

ϕM_n = Design flexural strength

M_n/Ω = Allowable flexural strength

M_u = Required flexural strength (LRFD)

M_a = Required flexural strength (ASD)

DESIGN CHECK FOR BEAMS

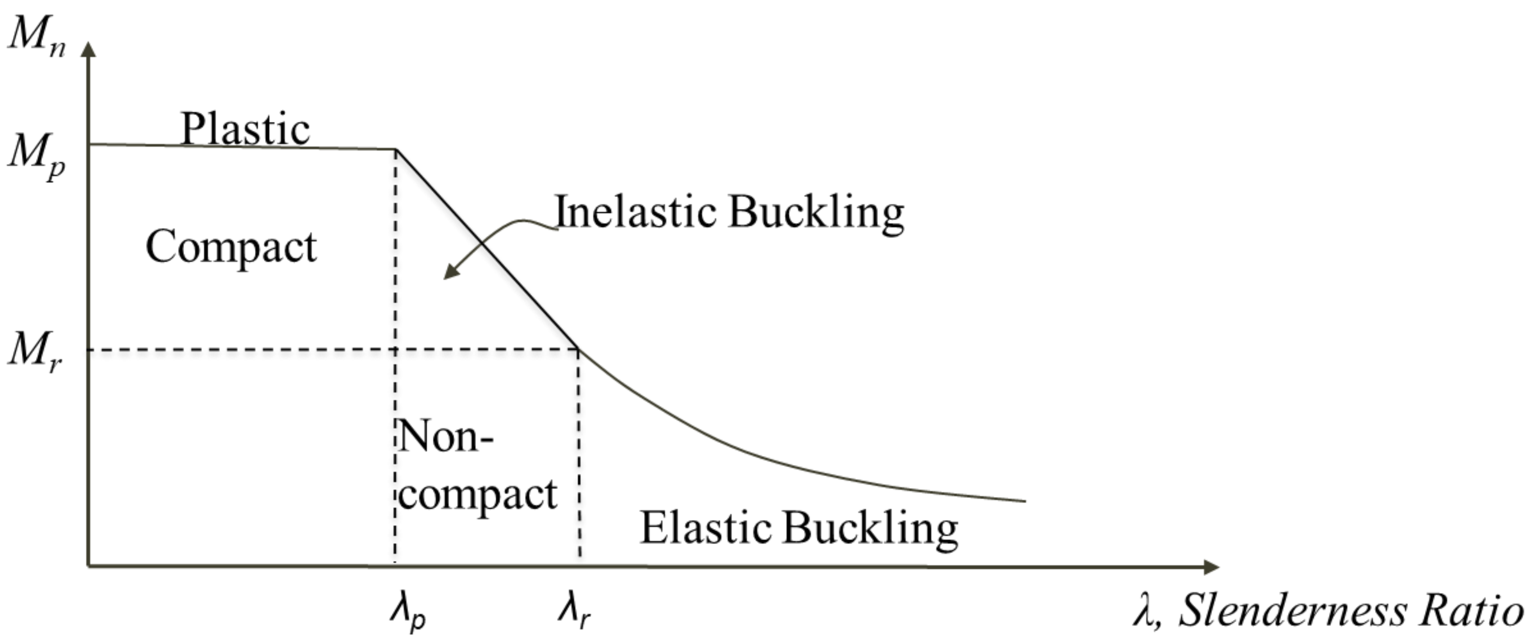
Factors that affect buckling:

- Support conditions
- Initial imperfections
- Residual stresses
- Load height effects

DESIGN CHECK FOR BEAMS

<u>Flexural Behavior: Flange and Web Local Buckling</u>

Compact, Noncompact and Slender Sections: **Table 5.1B**



M_p = Plastic moment of the section = $F_y W_{plx}$

M_r = Limit for elastic behavior = $0.7 F_y W_{ex}$ (LRFD 1999: $= S_x(F_y - F_R)$)

W_{elx} = Elastic section modulus with respect to the x-axis

F_R = Residual stresses = 70 MPa (rolled sections); 110 MPa (Built up sections)

DESIGN CHECK FOR BEAMS

Flexural Behavior: Flange and Web Local Buckling

Compact, Noncompact and Slender Sections: **Flanges**

Rolled I-Sections

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}}$$

$$\lambda_r = 1.00\sqrt{\frac{E}{F_y}}$$

Built Up I-Sections

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}}$$

$$\lambda_r = 0.95\sqrt{k_c \frac{E}{F_L}}$$

$$0.35 \leq k_c = \frac{4}{\sqrt{\frac{h}{t_w}}} \langle 0.76$$

$$F_L = 0.7F_y$$

DESIGN CHECK FOR BEAMS

Flexural Behavior: Flange and Web Local Buckling

Compact, Noncompact and Slender Sections: **Webs**

Doubly Symmetric I-Sections

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}}$$

$$\lambda_r = 5.70\sqrt{\frac{E}{F_y}}$$

Singly Symmetric I-Sections

$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} \le \lambda_r$$

$$\lambda_r = 5.70\sqrt{\frac{E}{F_y}}$$

DESIGN CHECK FOR BEAMS

Flexural Behavior: Flange and Web Local Buckling

Compact, Noncompact and Slender Sections: **S235**

Flange: Rolled I-Sections

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 10.25$$

$$\lambda_r = 1.00\sqrt{\frac{E}{F_y}} = 26.97$$

Web: Doubly Symmetric I-Sections

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 101.40$$

$$\lambda_r = 5.70\sqrt{\frac{E}{F_y}} = 153.72$$

DESIGN CHECK FOR BEAMS

Flexural Behavior: Flange and Web Local Buckling

Compact, Noncompact and Slender Sections: **S355**

Flange: Rolled I-Sections

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 9.02$$

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 89.25$$

$$\lambda_r = 1.00\sqrt{\frac{E}{F_y}} = 23.74$$

$$\lambda_r = 5.70\sqrt{\frac{E}{F_y}} = 135.29$$

DESIGN CHECK FOR BEAMS

Flexural Design: Flange and Web Local Buckling for Doubly Symmetric I-Sections

$$\frac{b_f}{2t_f} \le 0.38\sqrt{\frac{E}{F_y}}$$

$$\frac{h}{t_w} \le 3.76\sqrt{\frac{E}{F_y}}$$

→ Compact

Sufficient Lateral Support

Compact + Sufficient Lateral Support → $\phi M_n = \phi M_p$

DESIGN CHECK FOR BEAMS

Flexural Design: Flange and Web Local Buckling for Doubly Symmetric I-Sections

Look at the compact limit for webs: $\lambda_p=3.76\sqrt{(E/F_y)}$
Most slender HE-Section: HE 1000A, λ=57.6
Most Slender W-Section: W760×134, λ=57.6
Most slender IPE-Section: IPE 750×137, λ=59.6

$$59.6 = 3.76\sqrt{\frac{200000^{MPa}}{F_y}} \rightarrow F_y = 796MPa$$

For the sections mentioned above WLB is not a problem: $F_y \leq 796$ MPa
Most of these sections are also compact for FLB

Flexural Design: Flange and Web Local Buckling for Doubly Symmetric I-Sections: Example 6.1

Determine the classification of a HE 300 A and a HE 400 A for Fy = 355 MPa. Check both the flange and the web.

HE 300 A (Flange)

$$\frac{b_f}{2t_f} = \frac{300^{mm}}{2x14^{mm}} = 10.7$$

HE 400 A (Flange)

$$\frac{b_f}{2t_f} = \frac{300^{mm}}{2x19^{mm}} = 7.9$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 9.02 < 7.9$$

HE 400 A flange is compact

$$\lambda_{pr} = 1.00\sqrt{\frac{E}{F_y}} = 1.00\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 23.7 < 10.7 < 9.02$$

HE 300 A flange is noncompact

DESIGN CHECK FOR BEAMS

Flexural Design: Flange and Web Local Buckling for Doubly Symmetric I-Sections: Example 6.1

Determine the classification of a HE 300 A and a HE 400 A for F_y = 355 MPa. Check both the flange and the web.

HE 300 A (Web)

$$\frac{h}{t_w} = \frac{208^{mm}}{8.5^{mm}} = 24.5$$

HE 400 A (Web)

$$\frac{h}{t_w} = \frac{298^{mm}}{11^{mm}} = 27.1$$

$$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 89.3 > 24.5 \quad and \quad 27.1$$

$$\lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{200000^{MPa}}{355^{MPa}}} = 135.3$$

HE 300 A and 400 a webs are compact

DESIGN CHECK FOR BEAMS

Flexural Design: Flange and Web Local Buckling

$$\lambda \leq \lambda_p \Rightarrow M_n = M_p$$

$\lambda_p \langle \lambda \langle \lambda_r$

- Flanges $\Rightarrow M_n = M_p - (M_p - M_r)\left(\dfrac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)$
- Web — Taken into account with R_{pc} factor

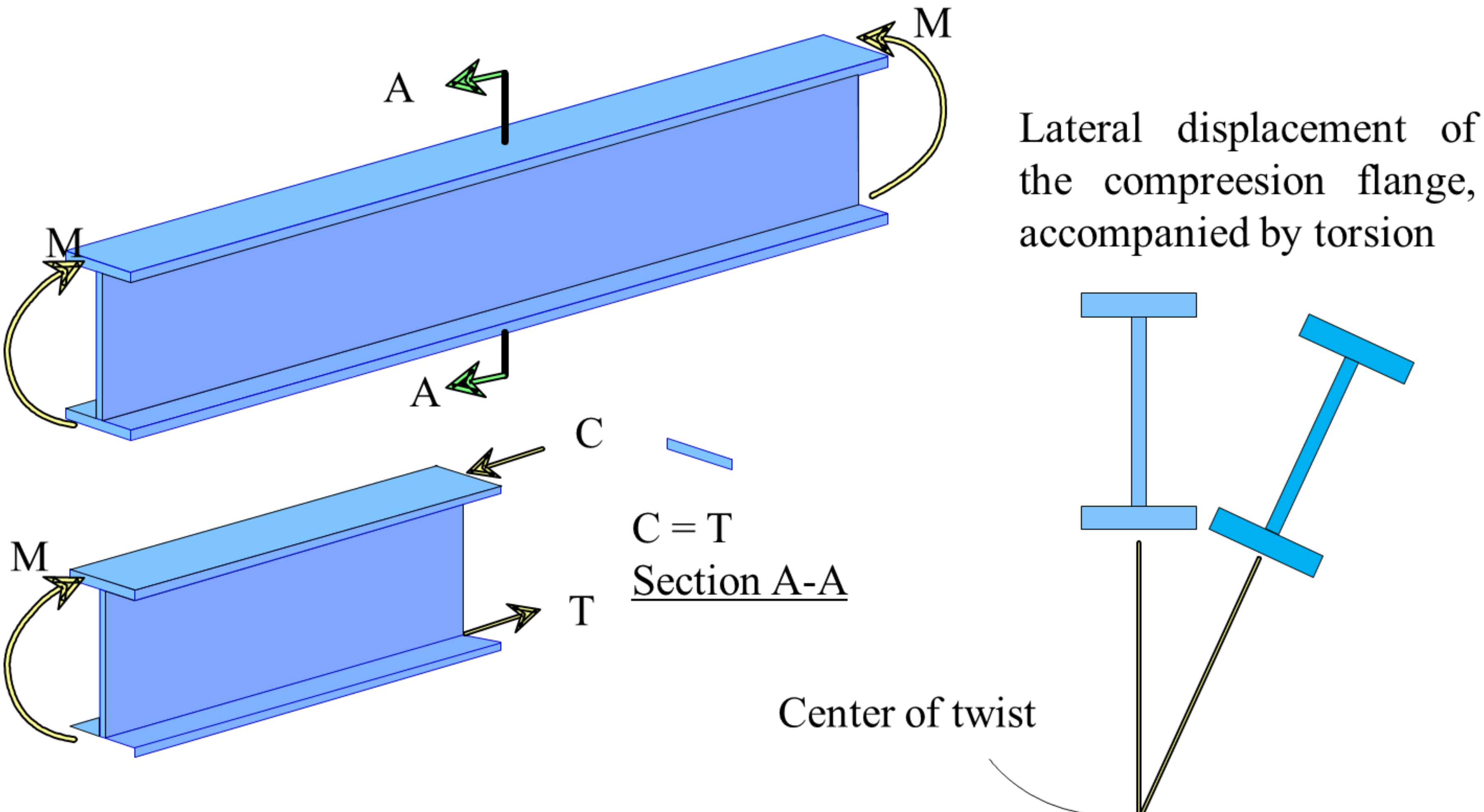
For $\lambda \rangle \lambda_r$

- Flanges $\Rightarrow M_n = \dfrac{0.9 E k_c W_{ex}}{\lambda^2}$
- Web — Taken into account with R_{pc} factor

Follow Sections 9.2-9.12 (Turkish Steel Specification)

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)



Drawing: Courtesy of Prof. Helwig

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)

Lateral–torsional buckling occurs when the distance between lateral brace points is large enough that the beam fails by lateral, outward movement in combination with a twisting action (Δ and θ, respectively.

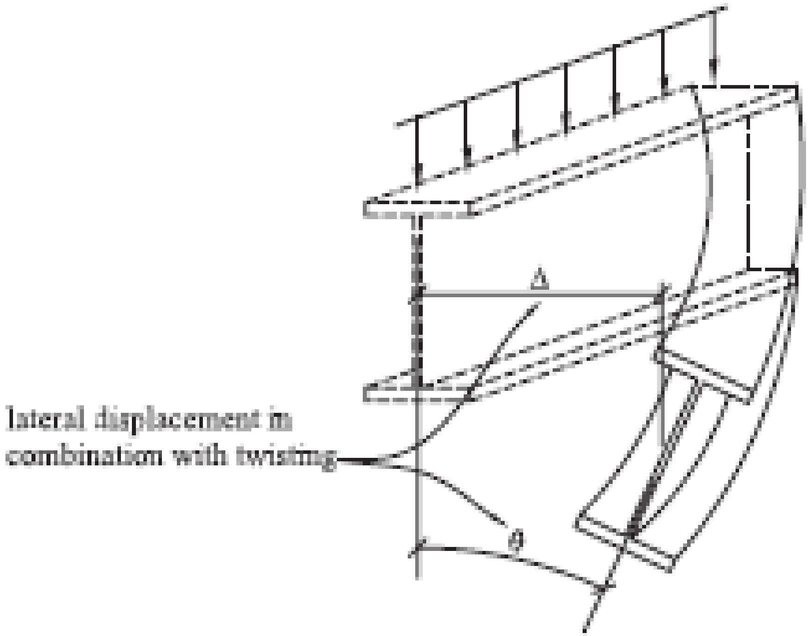
Beams with wider flanges are less susceptible to lateral–torsional buckling because the wider flanges provide more resistance to lateral displacement.

In general, adequate restraint against lateral–torsional buckling is accomplished by the addition of a brace or similar restraint somewhere between the centroid of the member and the compression flange.

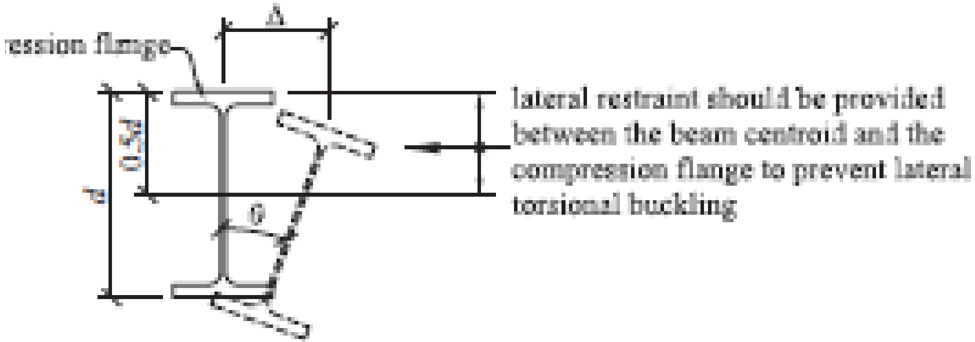
For simple-span beams supporting normal gravity loads, the top flange is the compression flange, but the bottom flange could be in compression for continuous beams or beams in moment frames.

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)



a. lateral torsional buckling behavior

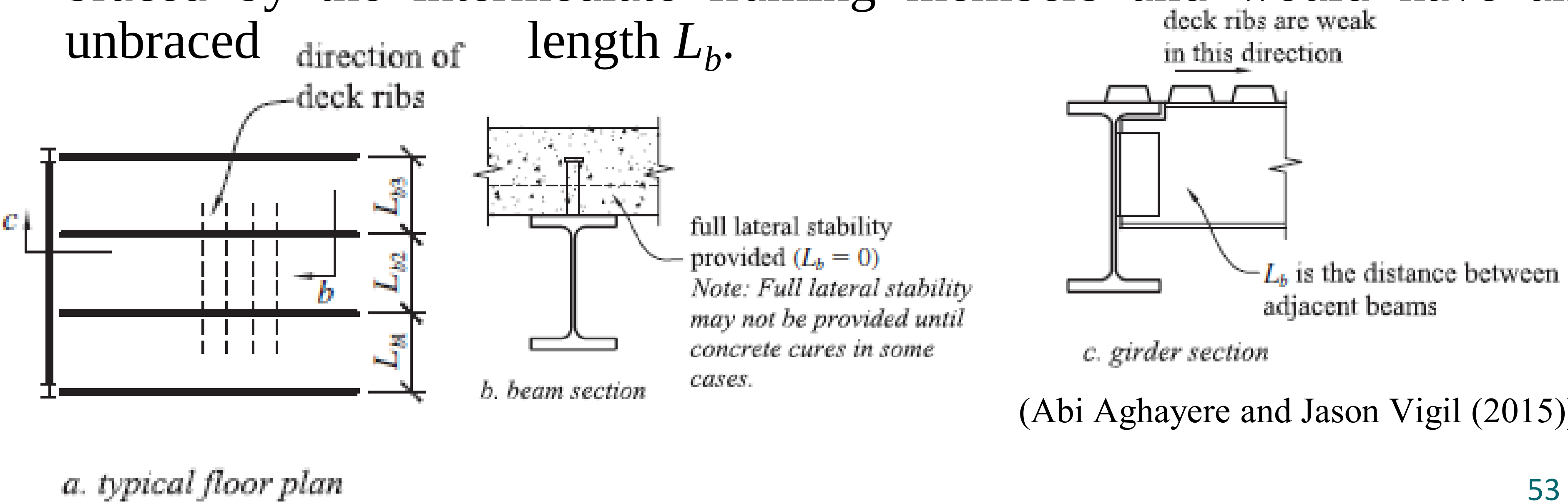


b. lateral torsional buckling restraint

(Abi Aghayere and Jason Vigil (2015))

Flexural Design: Lateral Torsional Buckling (LTB)

Lateral–torsional buckling can be controlled in several ways, but it is usually dependent on the actual construction details used. Beams with a metal deck oriented perpendicular to the beam span are considered fully braced, whereas the girder in the right figure is not considered braced by the deck because the deck has very little stiffness to prevent lateral displacement of the girder. This girder would be considered braced by the intermediate framing members and would have an unbraced length L_b.



(Abi Aghayere and Jason Vigil (2015))

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)

When full lateral stability is provided for a beam, the nominal moment strength is the plastic moment capacity of the beam ($M_p = F_y W_x$).

Once the unbraced length reaches a certain upper limit, lateral–torsional buckling will occur and therefore the nominal bending strength will likewise decrease. The failure mode for lateral–torsional buckling can be either inelastic or elastic. The AISC specification defines the unbraced length at which inelastic lateral–torsional buckling occurs as:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}.$$

(Abi Aghayere and Jason Vigil (2015))

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)

L_p is also the maximum unbraced length at which the nominal bending strength equals the plastic moment capacity. The unbraced length at which elastic lateral–torsional buckling occurs is:

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{Jc}\right)^2}},$$

$$r_{ts} = \left(\frac{\sqrt{I_y C_w}}{S_x}\right)^{1/2},$$

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \text{ (for channel shapes)},$$

$c = 1.0$ (for I-shapes),

F_y = Yield strength,

E = Modulus of elasticity,

J = Torsional constant,

S_x = Section modulus (x-axis),

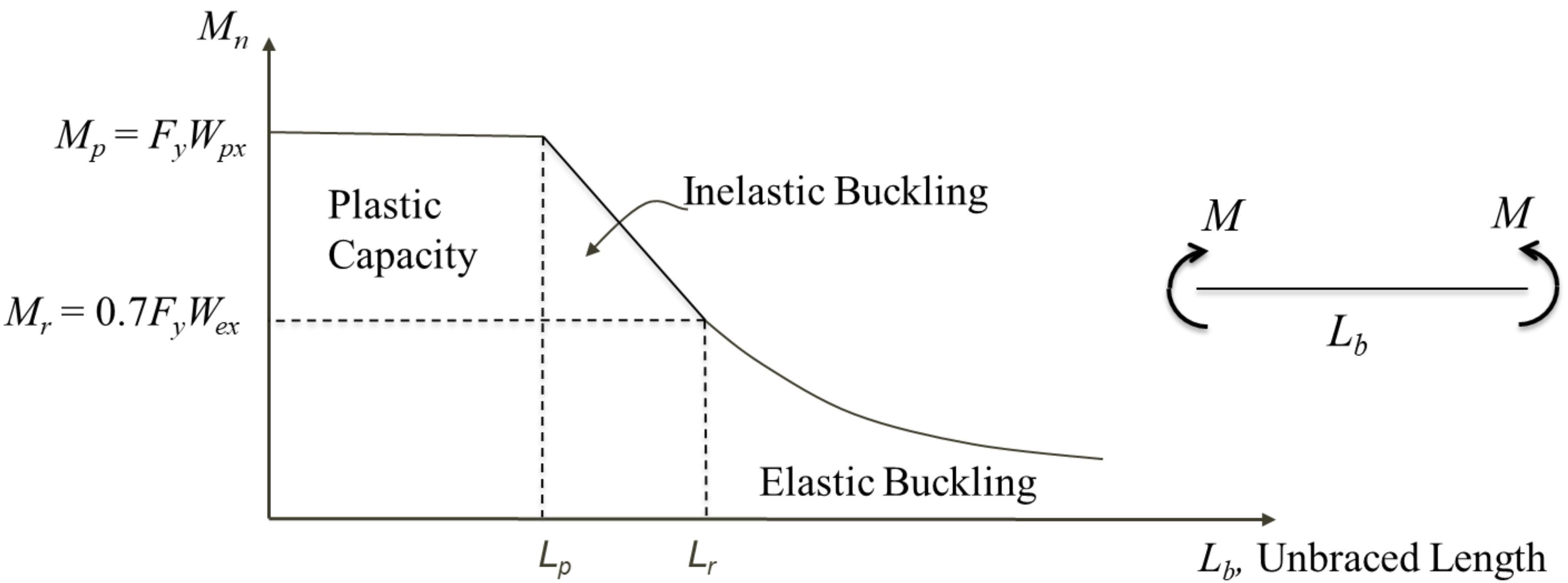
I_y = Moment of inertia (y-axis),

C_w = Warping constant, and

h_o = Distance between flange centroids.

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling Beam Curve for Uniform Moment



DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)

When lateral–torsional buckling is not a concern (i.e., when the unbraced length, $L_b < L_p$), the failure mode is flexural yielding. The nominal bending strength for flexural yielding is:

$$M_n = M_p = F_y W_{plx} \left(when\ L_b \leq L_p \right)$$

M_n = Nominal flexural stength of the section
M_p = Plastic moment of the section = $F_y W_{px}$
F_y = Yield strength of the section
W_{plx} = Plastic section modulus of the section with respect to the x-axis

(Abi Aghayere and Jason Vigil (2015))

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)

For compact I-shapes and C-shapes when L_p < L_b <L_r, the nominal flexural strength is:

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yW_{elx}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \left(when\ L_p < L_b < L_r\right)$$

In the above equation, the term $0.7F_yW_{elx}$ is also referred to as M_r,which corresponds to the limiting buckling moment when $L_b = L_r$ and is the transition point between inelastic and elastic lateral–torsional buckling.

M_n = Nominal flexural stength of the section, C_b = Moment gradient factor

M_p = Plastic moment of the section = F_yW_{px}

F_y = Yield strength of the section

W_{elx} = Elastic section modulus of the section with respect to the x-axis

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)

For compact I-shapes and C-shapes when $L_b > L_r$, the nominal flexural strength is:

$$M_n = F_{cr} W_{elx} \le M_p$$

$$F_{cr} = C_b \frac{\pi^2 E}{\left(\frac{L_b}{i_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{W_{ex} h_o} \left(\frac{L_b}{i_{ts}}\right)^2}$$

DESIGN CHECK FOR BEAMS

Flexural Design: Lateral Torsional Buckling (LTB)

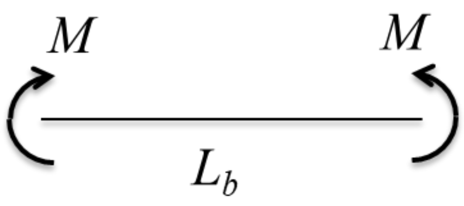
Laterl Torsional Buckling Moment (Elastic) for Doubly Symmetric Sections

(For uniform mooment)

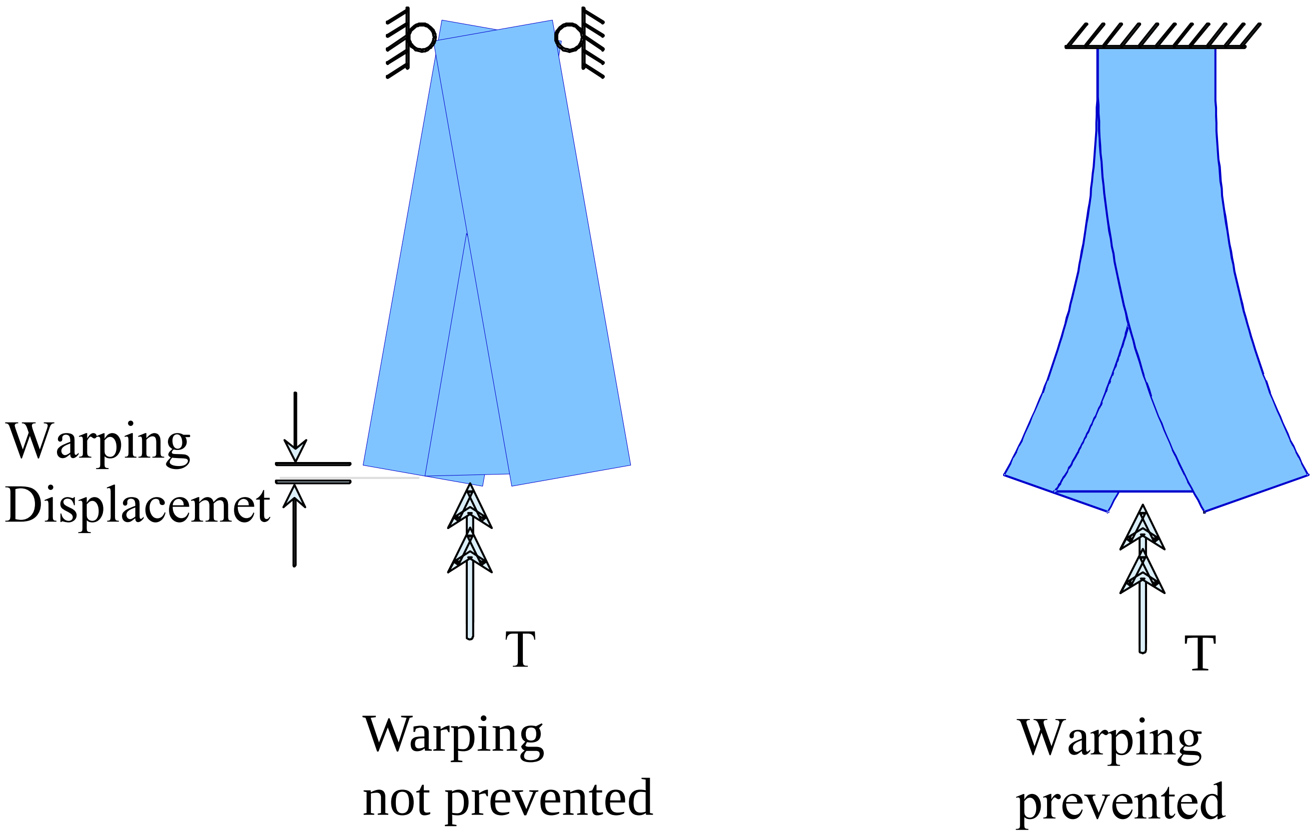
$$M_e = \frac{\pi}{L_b}\sqrt{EI_yGJ + E^2 I_y C_w \frac{\pi^2}{L_b^2}}$$

St. Venant Torsion | Warping Torsion

L_b: unbraced length
E: elastic modulus (200000 MPa)
I_y: weak axis moment of inertia
J: torsional constant = $\sum(bt^3)/3$
G: shear modulus of elasticity of steel (77200 MPa)
C_w: warping constant = $I_y d^2/4$
d = distance between centroids of flanges



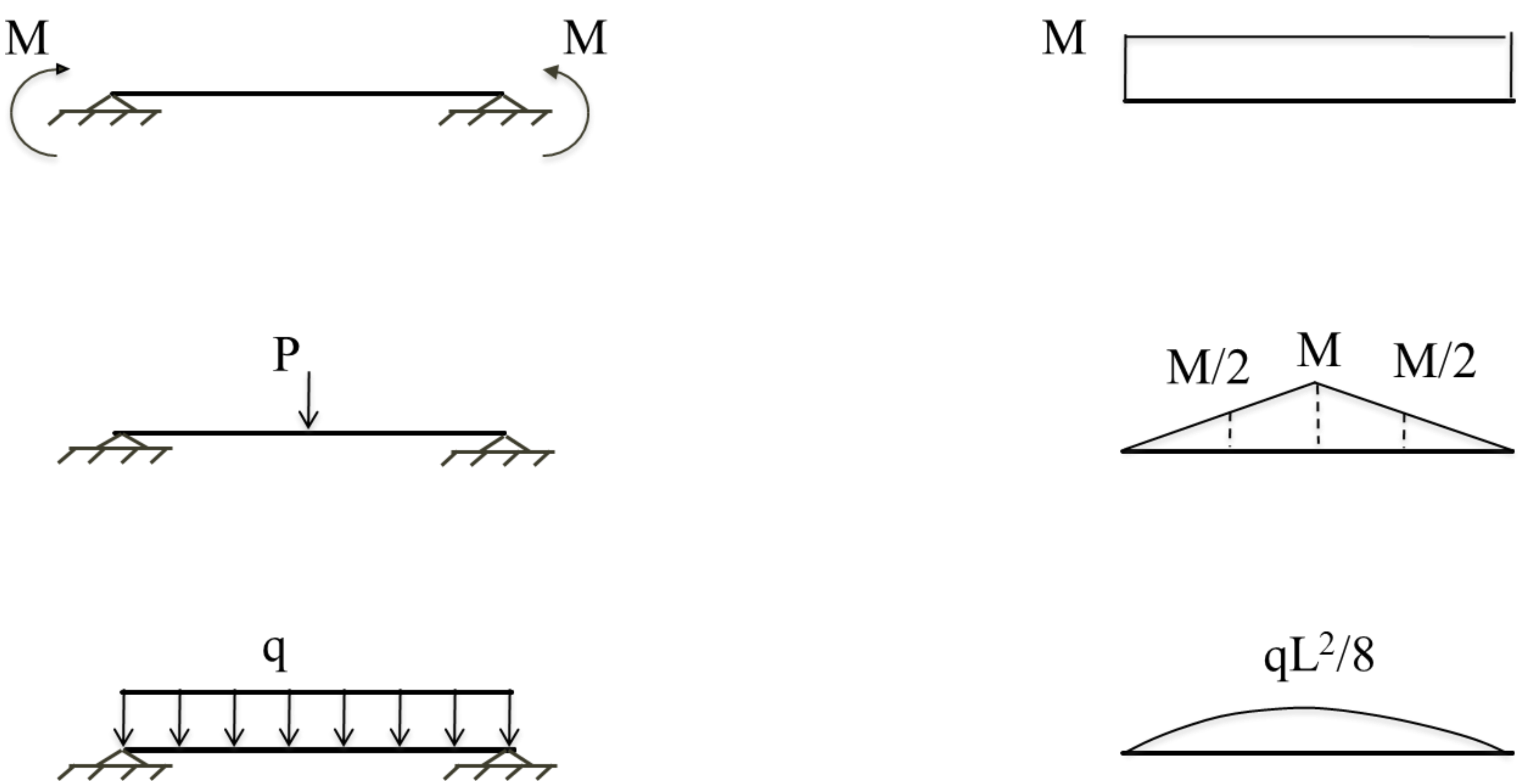
Flexural Design: Warping Behavior



Drawing: Courtesy of Prof. Helwig

DESIGN CHECK FOR BEAMS

Flexural Design: C_b, Moment Gradient Factor



Flexural Design: C_b, Moment Gradient Factor

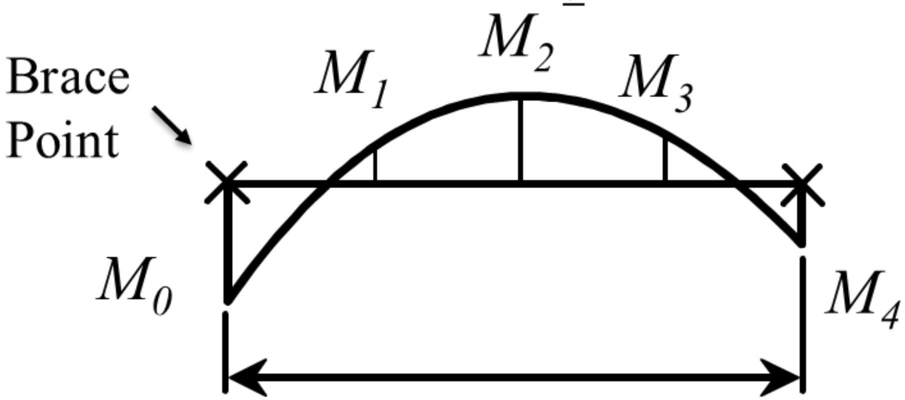
$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_1 + 4M_2 + 3M_3} \qquad \textbf{(9.1)}$$

M_{max} = absolute value of maximum moment in the unbraced segment, (N-mm)
M_1 = absolute value of moment at quarter point of the unbraced segment, (N-mm)
M_2 = absolute value of moment at centerline of the unbraced segment, (N-mm)
M_3 = absolute value of moment at three-quarter point of the unbraced segment, (N-mm)

Eq.(9.1) is valid for all doubly symmetric sections and singly symmetric sections under single curvature. For singly symmetric sections under reverse curvature, C_b can be obtained by analysis. Conservatively, C_b=1.0 can be used.

DESIGN CHECK FOR BEAMS

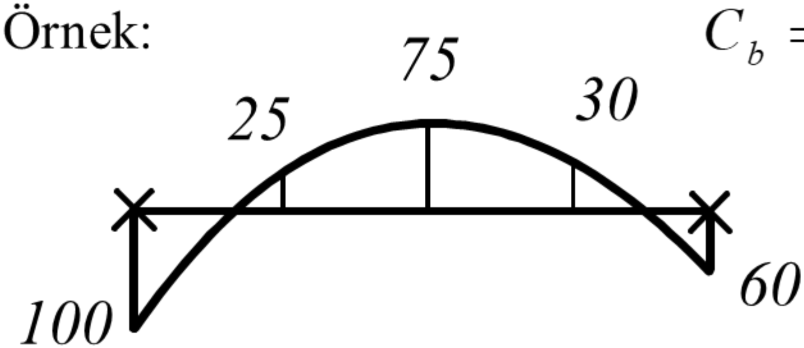
Flexural Design: C_b, Moment Gradient Factor



$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_1 + 4M_2 + 3M_3}$$

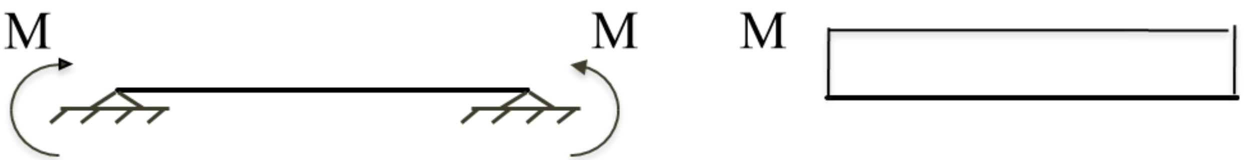
Note: All moment values are absolute.

Örnek:



$$C_b = \frac{12.5x100}{2.5x100 + 3x25 + 4x75 + 3x30} = 1.75$$

Flexural Design: C_b, Moment Gradient Factor

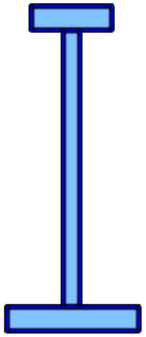


$$C_b = 1.00$$

$$C_b = 1.32$$

$$C_b = 1.14$$

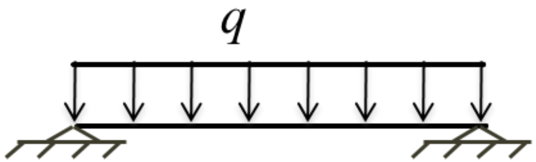
Flexural Design: C_b, Moment Gradient Factor for SSS

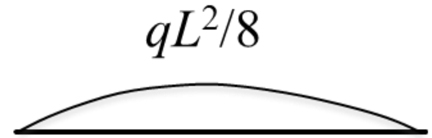


SSRC Guide to Stability Design (2010)

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3M_1 + 4M_2 + 3M_3} R \leq 3.0$$

$$R = 1.0$$





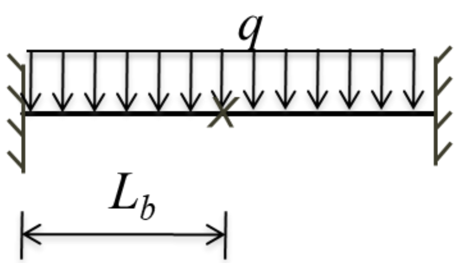
Single Curvature

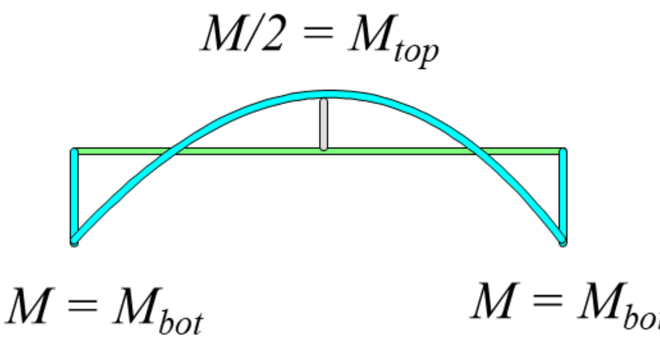
$$R = (0.5 + 2\rho_{top}{}^2)$$

valid for:

$$0.1 \leq \rho \leq 0.9$$

$$\rho_{top} = \frac{I_{ytop}}{I_y}$$





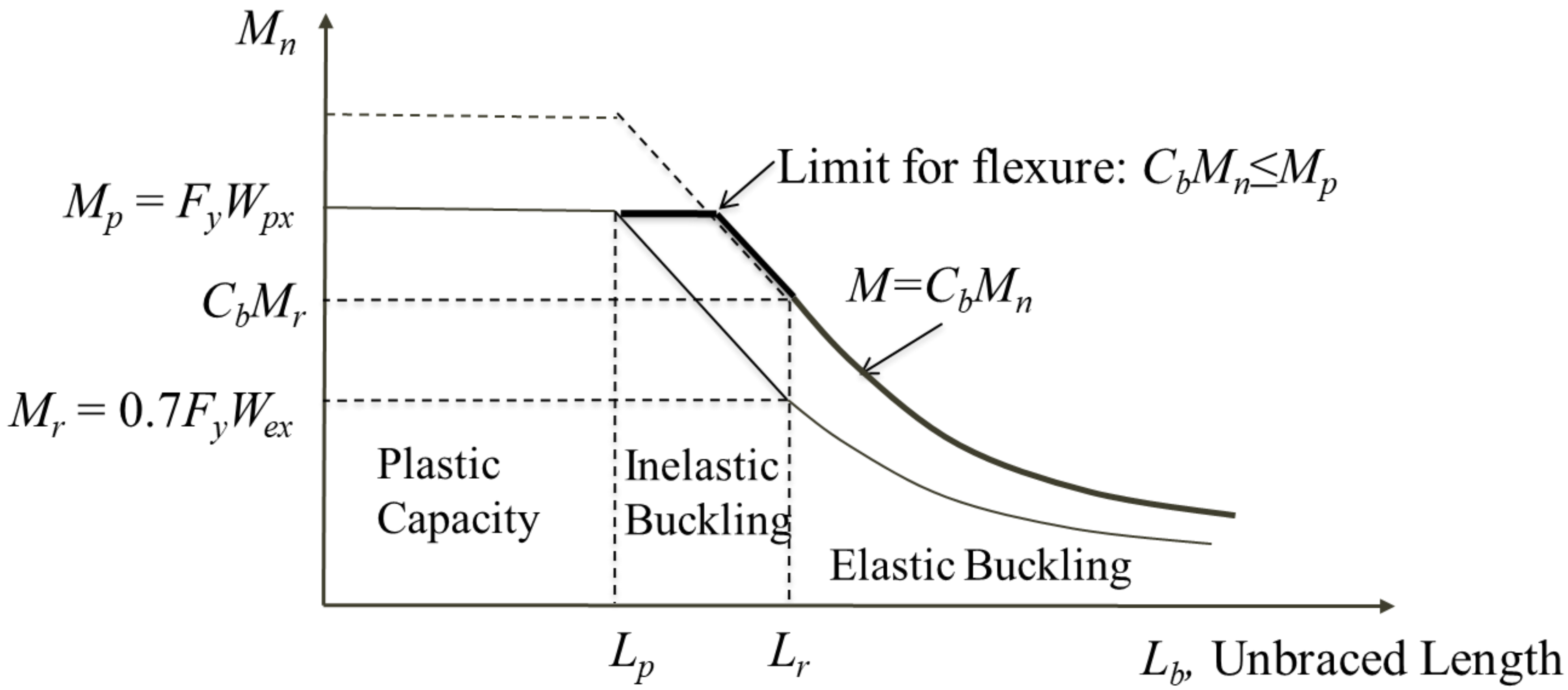
ρ = Mono-symmetry constant

I_{ytop} = moment of inertia of the top flange with respect to the y-y axis

I_y = momet of inertia of the entire section with respect to y-y axis

DESIGN CHECK FOR BEAMS

Flexural Design: Beam curve with C_b, Moment Gradient Factor



DESIGN CHECK FOR BEAMS

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of:

9.2.1 Yielding (plastic moment)

9.2.2 Lateral-torsional buckling. Akma Sınır Durumu

DESIGN CHECK FOR BEAMS

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

9.2.1 Yielding

$$M_n = M_p = F_y W_{px}$$

M_n = nominal flexural strength

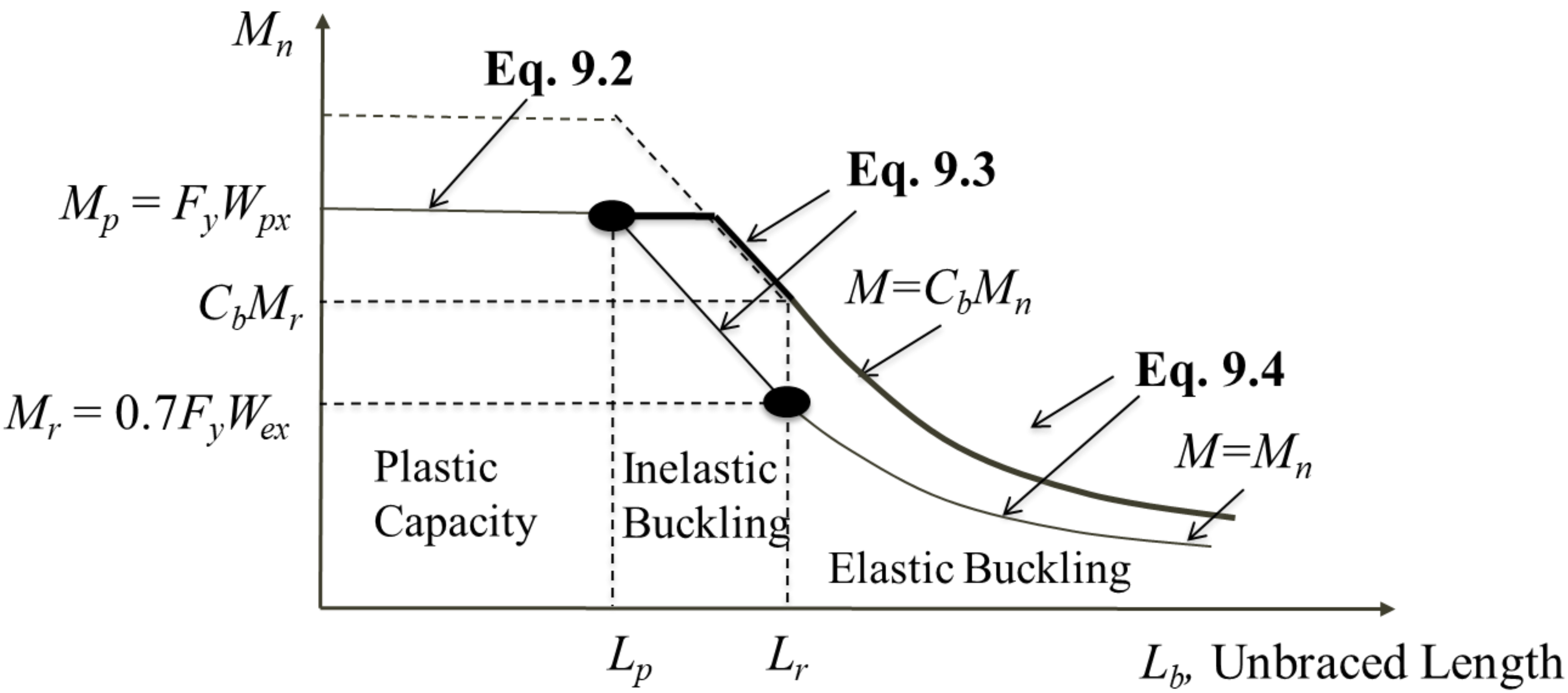
M_p = plastic moment

F_y = specified minimum yield stress of the type of steel being used, (MPa)

W_{plx} = plastic section modulus about the x-axis, (mm^3)

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

9.2.2 Lateral Torsional Buckling Limit State



DESIGN CHECK FOR BEAMS

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

9.2.2 Lateral Torsional Buckling Limit State

$$L_b \leq L_p \Rightarrow M_n = M_p$$ **Eq. 9.2**

$$L_p \langle L_b \leq L_r \Rightarrow M_n = C_b \left[M_p - (M_p - 0.7 F_y W_{ex}) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$ **Eq. 9.3**

$$L_b \rangle L_r \Rightarrow M_n = F_{cr} W_{ex} \leq M_p$$ **Eq. 9.4**

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{i_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{W_{ex} h_o} \left(\frac{L_b}{i_{ts}} \right)^2}$$ **Eq. 9.5**

DESIGN CHECK FOR BEAMS

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

9.2.2 Lateral Torsional Buckling Limit State

$$L_p = 1.76\, i_y \sqrt{\frac{E}{F_y}}$$ **Eq. 9.6a**

$$L_r = 1.95\, i_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{W_{ex}h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{W_{ex}h_o}{Jc}\right)^2}}$$ **Eq. 9.6b**

For doubly symmetric I-shapes: c = 1.0 **Eq. 9.7a**

For singly symmetric U-channels: $c = \frac{h_0}{2}\sqrt{\frac{IyE}{C_w}}$ **Eq. 9.7b**

Effective Radius of gyration: $i_{ts}^{\;2} = \frac{\sqrt{I_y C_w}}{W_{ex}}$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

9.2.2 Lateral Torsional Buckling Limit State

M_n : nominal flexural strength (N-mm)

M_p : plastic moment (N-mm)

F_y : specified minimum yield stress of the type of steel being used, (MPa)

W_{elx} : elastic section modulus taken about the x-axis, (mm^3)

E : elastic modulus (200000 MPa)

C_b : moment gradient factor defined by Eq.(9.1)

L_b : length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, (mm)

L_p : limiting laterally unbraced length for the limit state of yielding, (mm)

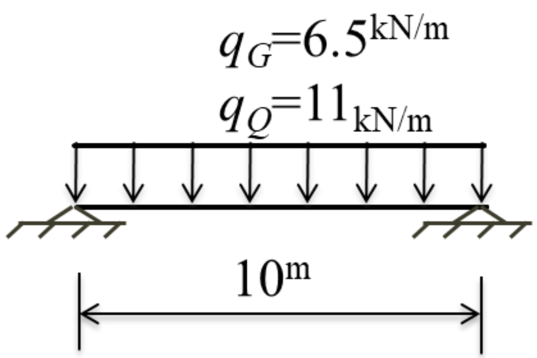
L_r : limiting unbraced length for the limit state of inelastic lateral-torsional buckling, (mm)

i_y : radius of gyration with respect to the y-axis

i_{ts} : effective Radius of gyration. J : torsional constant (mm^4) C_w : Çarpılma sabiti.

h_o : distance between the flange centroids, (mm) (= d-t_f).

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

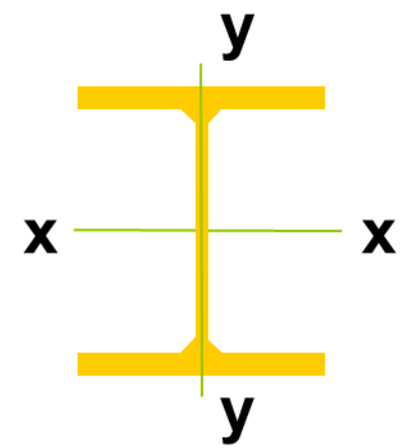


Continuously braced against LTB

S275

$F_y = 275$ MPa

$F_u = 430$ MPa



Choose an HE-section for the above beam.

Depth limit: 45 cm

Serviceability limit state: L/360

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Required Strength:
LRFD

$$q_u = 1.2x6.5^{kN/m} + 1.6x11^{kN/m}$$

$$q_u = 25.4\ kN/m$$

$$M_u = \frac{25.4^{kN/m}(10^m)^2}{8}$$

$$M_u = 317.5\ kN-m$$

Required Strength:
ASD

$$q_a = 6.5^{kN/m} + 11^{kN/m}$$

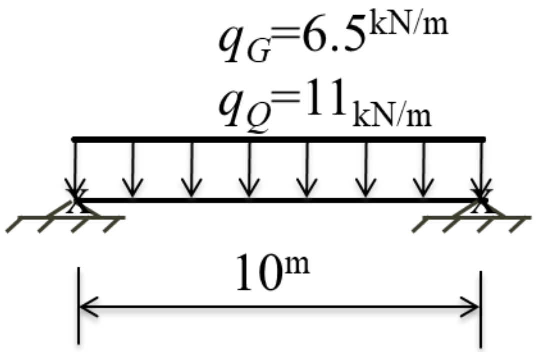
$$q_a = 17.5\ kN/m$$

$$M_a = \frac{17.5^{kN/m}(10^m)^2}{8}$$

$$M_a = 218.8\ kN-m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-1: AISC Design Examples V14.2 F1-1A



S275

$F_y = 275$ MPa

$F_u = 430$ MPa

Continuously braced against LTB

Select a compact section

Limit States:

9.2.1 Yielding

~~**9.2.2** Lateral Torsional Buckling~~

15.2 Serviceability

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-1: AISC Design Examples V14.2 F1-1A

Required moment of inertia to satisfy L/360 serviceability limit state **(15.2)**:

$$\Delta_{\max} = \frac{L}{360} = \frac{10000^{\ mm}}{360} = 27\ mm$$

$$I_{x(req)} = \frac{5 q_Q L^4}{384\, E \Delta_{\max}}$$

$$I_{x(req)} = \frac{5(11000^{\ N/m})(1^{m}/1000^{\ mm})(10000^{\ mm})^4}{384\,(200000^{\ N/mm^2})27^{\ mm}}$$

$$I_{x(req)} = 26524\ x10^4\ mm^4$$

DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-1: AISC Design Examples V14.2 F1-1A

$$I_{x(req)} = 26524 \ x10^{4} \ mm^{4}$$

Bu tablolar tasarımda kullanılmaz.

Tabloların kullanımıyla ilgili tüm sorumluluk kullanıcıya aittir.

Tablo 6-1
HE ve HL Kesitleri

Bu tablolar tasarımda kullanılmaz.

Tabloların kullanımıyla ilgili tüm sorumluluk kullanıcıya aittir.

Tablo 6-1

HE ve HL Kesitleri

Kesit		Alan	Derinlik	Gövde	Başlık		Ölçüler			Kompakt Kesit Kriterleri		x-x Ekseni (Güçlü Eksen)				y-y Ekseni (Zayıf Eksen)				Burulma Katsayıları	
				Kalınlık	Genişlik	Kalınlık															
İsim	G	A	h	t_w	b_f	t_f	k	$k_1(r)$	$T(d)$	$b_f/2t_f$	d/t_w	I_x	$W_{el.x}$	$W_{pl.x}$	i_x	I_y	$W_{el.y}$	$W_{pl.y}$	i_y	J	C_w
	kg/m	mm^2	mm	mm	mm	mm	mm	mm	mm			mm^4	mm^3	mm^3	mm	mm^4	mm^3	mm^3	mm	mm^4	mm^6
		x 10^2										x 10^4	x 10^3	x 10^3	x 10	x 10^4	x 10^3	x 10^3	x 10	x 10^4	x 10^9
HE 340 A	105,0	133,5	330,0	9,5	300,0	16,5	43,5	27,0	243,0	9,1	25,6	27690,0	1678,0	1850,0	14,40	7436,0	495,7	755,9	7,46	127,2	1824,0
HE 340 B	134,0	170,9	340,0	12,0	300,0	21,5	48,5	27,0	243,0	7,0	20,3	36660,0	2156,0	2408,0	14,65	9690,0	646,0	985,7	7,53	257,2	2454,0
HE 340 M	248,0	315,8	377,0	21,0	309,0	40,0	67,0	27,0	243,0	3,9	11,6	76370,0	4052,0	4718,0	15,55	19710,0	####	1953,0	7,90	1506,0	5584,0

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-1: AISC Design Examples V14.2 F1-1A

HE 340 A: 105 kg/m, $h = 330$ mm, $t_w = 9.5^{mm}$, $t_f = 16.5^{mm}$, $b_f = 300_{mm}$, $d = 243^{mm}$
$I_x = 27690 \times 10^4$ mm^4, $\phi M_p = 457.9$ kN-m, $M_p/\Omega_b = 304.6$ kN-m

HE 320 B: 127 kg/m, $h = 320$ mm,
$I_x = 30820 \times 10^4$ mm^4, $\phi M_p = 531.9$ KN-m, $M_p/\Omega_b = 353.9$ kN-m

Beam is continuously braced.Check the compactness of HE 340 A (lighter).

Flange

$$\lambda_f = \frac{b}{2t_f} = \frac{300^{mm}}{2(16^{mm})} = 9.09$$

$$\lambda_f \langle \lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 10.25$$

Web

$$\lambda_w = \frac{d}{t_w} = \frac{243^{mm}}{9.5^{mm}} = 25.58$$

$$\lambda_w \langle \lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 101.40$$

DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-1: AISC Design Examples V14.2 F1-1A

HE 340 A: 105 kg/m, h = 330 mm,

$I_x = 27690 \text{ x } 10^4 \text{ mm}^4$, $\phi M_p = 457.9$ kN-m, $M_p/\Omega_b = 304.6$ kN-m

9.2.1 Yielding Limit State:

$$M_n = M_p = F_y W_{px} = 275^{MPa} x1850000^{mm^3} = 508.75 \; kN - m$$

$$I_x = 27690 \; x10^4 \; mm^4 \rangle I_{x(gerekli)} = 26524 \; x10^4 \; mm^4$$ **(15.2) OK**

$$\phi M_p = 457.9 \; kN - m \rangle M_u = 317.5 \; kN - m$$ **(9.2.1) OK**

$$M_p / \Omega = 304.6 \; kN - m \rangle M_a = 218.8 \; kN - m$$ **(9.2.1) OK**

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-1: AISC Design Examples V14.2 F1-1A

$M_u = 317.5\ kN - m$ $M_a = 218.8\ kN - m$

F_y=275 MPa

Tablo 6-4

İZİN VERİLEBİLİR MOMENT KAPASİTESİ (kNm)

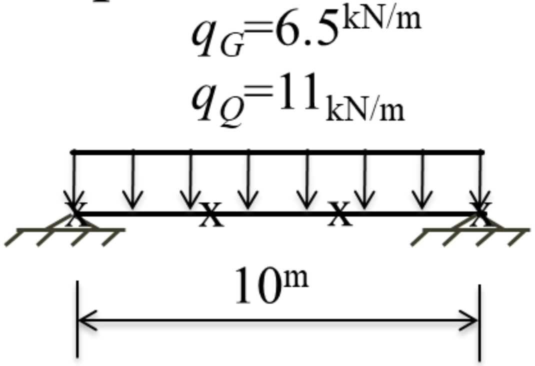
HE Kesitleri

Ω_b=1.67 φ_b=0.9, Ω_v=1.50 φ_v=1.0

Tasarım		M_{px}/Ω_b	$\varphi_b M_{px}$	M_{rx}/Ω_b	$\varphi_b M_{rx}$	L_p	L_r	I_x	V_{nx}/Ω_v	$\varphi_v V_{nx}$
Kesit	Z_x	kNm	kNm	kNm	kNm				kN	kN
	mm³ x10³	ASD	LRFD	ASD	LRFD	mm	mm	mm⁴ x10⁴	ASD	LRFD
HE 360M	4989	821,5	1234,8	495,3	744,5	3716,4	23726,1	84870	727,7	1091,5
HE 360B	2683	441,8	664,0	276,6	415,8	3555,0	15142,0	43190	433,1	649,7
HE 360A	2088	343,8	516,8	218,0	327,6	3526,6	13199,4	33090	346,5	519,8
HE 340M	4718	776,9	1167,7	467,1	702,0	3749,6	25111,0	76370	686,1	1029,1
HE 340B	2408	396,5	596,0	248,5	373,5	3574,0	15386,9	36660	392,0	588,1
HE 340A	1850	304,6	457,9	193,4	290,7	3540,8	13317,1	27690	310,4	465,5
HE 320M	4435	730,3	1097,7	437,6	657,7	3773,4	26582,0	68130	644,5	966,7
HE 320B	2149	353,9	531,9	222,0	333,7	3593,0	15665,9	30820	352,9	529,4
HE 320A	1628	268,1	402,9	170,5	256,2	3555,0	13452,0	22930	276,2	414,3

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-2: AISC Design Examples V14.2 F1-2A



S275

$F_y = 275$ MPa

$F_u = 430$ MPa

x- Braced at every L/3

Calculate the design and allowable strength of HE 340 A beam.

Required Strength:

LRFD

$$M_u = 317.5\ kN - m$$

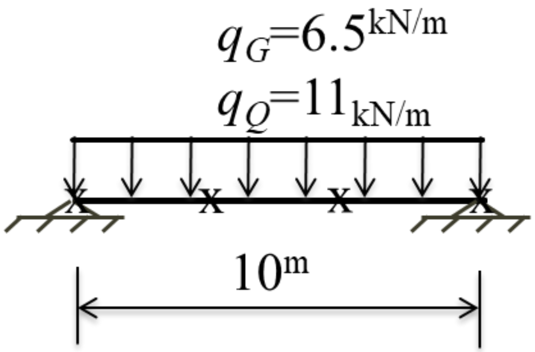
Allowable Strength:

ASD

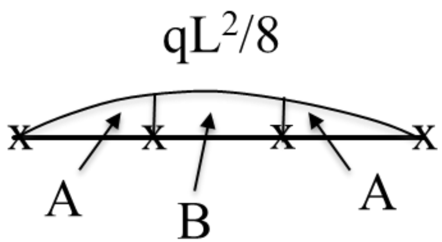
$$M_a = 218.8\ kN - m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-2: AISC Design Examples V14.2 F1-2A



x- Braced at every L/3



Limit States:

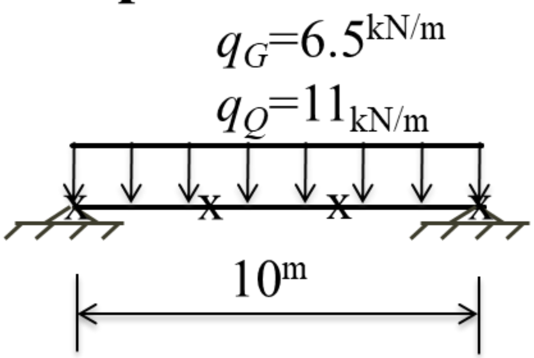
9.2.1 Yielding

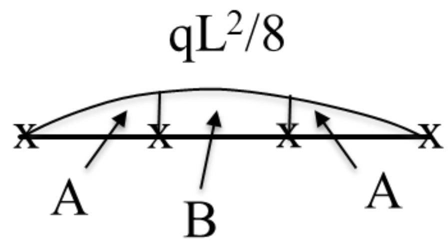
9.2.2 Lateral Torsional Buckling

15.2 Serviceability

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-2: AISC Design Examples V14.2 F1-2A





L_b = 10m/3 = 3.33 m

9.2.2 Lateral torsional buckling limit state:

$$L_p = 1.76\, i_y \sqrt{\frac{E}{F_y}} = 3.54\, m$$

$$L_r = 1.95\, i_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{W_{ex} h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{W_{ex} h_o}{Jc}\right)^2}} = 13.3m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-2: AISC Design Examples V14.2 F1-2A

$$L_b \le L_p \Rightarrow M_n = M_p$$

$$L_b = 3.33\,m \le L_p = 3.54\,m \Rightarrow M_n = M_p$$

$$M_p = F_y W_{plx} = 275^{N/mm^2} x1850000^{mm^3}$$

$$M_p = 508.75\ kN - m$$

$$\phi M_p = 0.9x508.75^{kN-m} = 457.9\ kN - m \rangle M_u = 317.5_{kN-m}$$

$$M_p / \Omega = 508.75^{kN-m} / 1.67 = 304.6\ kN - m \rangle M_a = 218.8\ kN - m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-2: AISC Design Examples V14.2 F1-2A

Let us show that section B is more critical in calculating C_b

Section B: (Moments have been shown as % of maximum moment)

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3M_1 + 4M_2 + 3M_3}$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.972) + 4(1.00) + 3(0.972)}$$

$$C_b = 1.01$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-2: AISC Design Examples V14.2 F1-2A

Section A: (Moments have been shown as % of maximum moment)

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3 M_1 + 4 M_2 + 3 M_3}$$

$$C_b = \frac{12.5(0.889)}{2.5(0.889) + 3(0.306) + 4(0.556) + 3(0.750)}$$

$$C_b = 1.46$$

Section B has a higher moment and a lower C_b and therefore more critical.

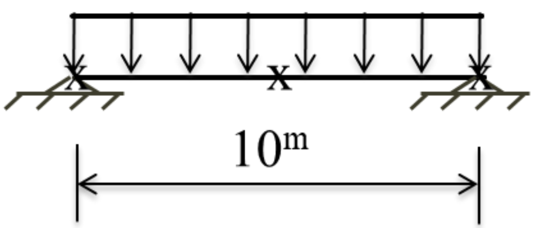
DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-3: AISC Design Examples V14.2 F1-3A

q_G=6.5 kN/m

q_Q=11 kN/m



x- Braced at L/2

S275

$F_y = 275$ MPa

$F_u = 430$ MPa

Calculate the design and allowable strength of HE 340 A beam.

Required Strength:

<u>LRFD</u>

$$M_u = 317.5\ kN - m$$

Allowable Strength:

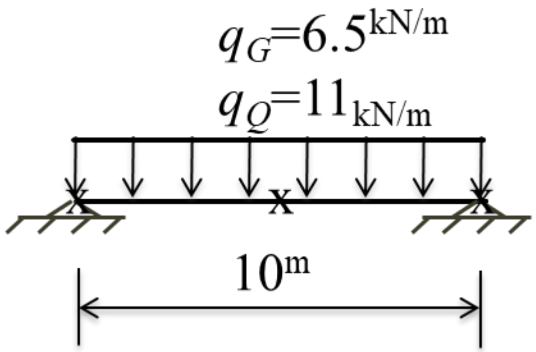
<u>ASD</u>

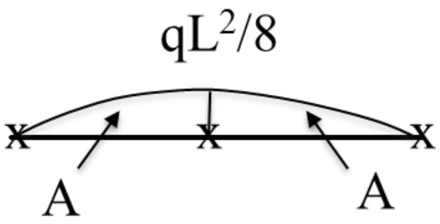
$$M_a = 218.8\ kN - m$$

DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-3: AISC Design Examples V14.2 F1-3A





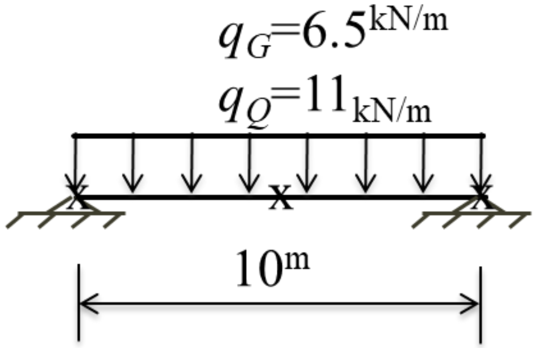
x- Braced at L/2

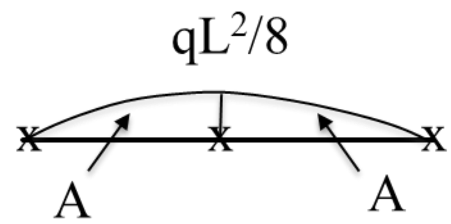
Limit States:
9.2.1 Yielding
9.2.2 Lateral Torsional Buckling
15.2 Serviceability

DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-3: AISC Design Examples V14.2 F1-3A





$L_b = 10^m/2 = 5.0$ m

9.2.2 Lateral Torsional Buckling Limit State:

Two Section A. $C_b = ?$

$$L_p = 1.76\, i_y \sqrt{\frac{E}{F_y}} = 3.54\, m$$

$$L_r = 1.95\, i_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{W_{ex} h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{W_{ex} h_o}{Jc}\right)^2}} = 13.3m$$

DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-3: AISC Design Examples V14.2 F1-3A

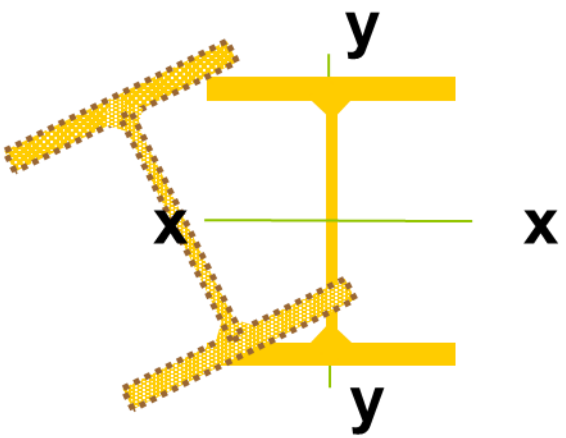
$$L_p = 3.54\,m \langle L_b = 5.0m \leq L_r = 13.3m$$

$$M_r = 0.7F_yW_{ex} = \frac{0.7x275^{N/mm^2}x1678000^{mm^3}}{1000^{N/k}x1000^{mm/m}}$$

$$M_r = 323.0\;kN - m$$

$$M_p = F_yW_{px} = \frac{275^{N/mm^2}x1850000^{mm^3}}{1000^{N/k}x1000^{mm/m}}$$

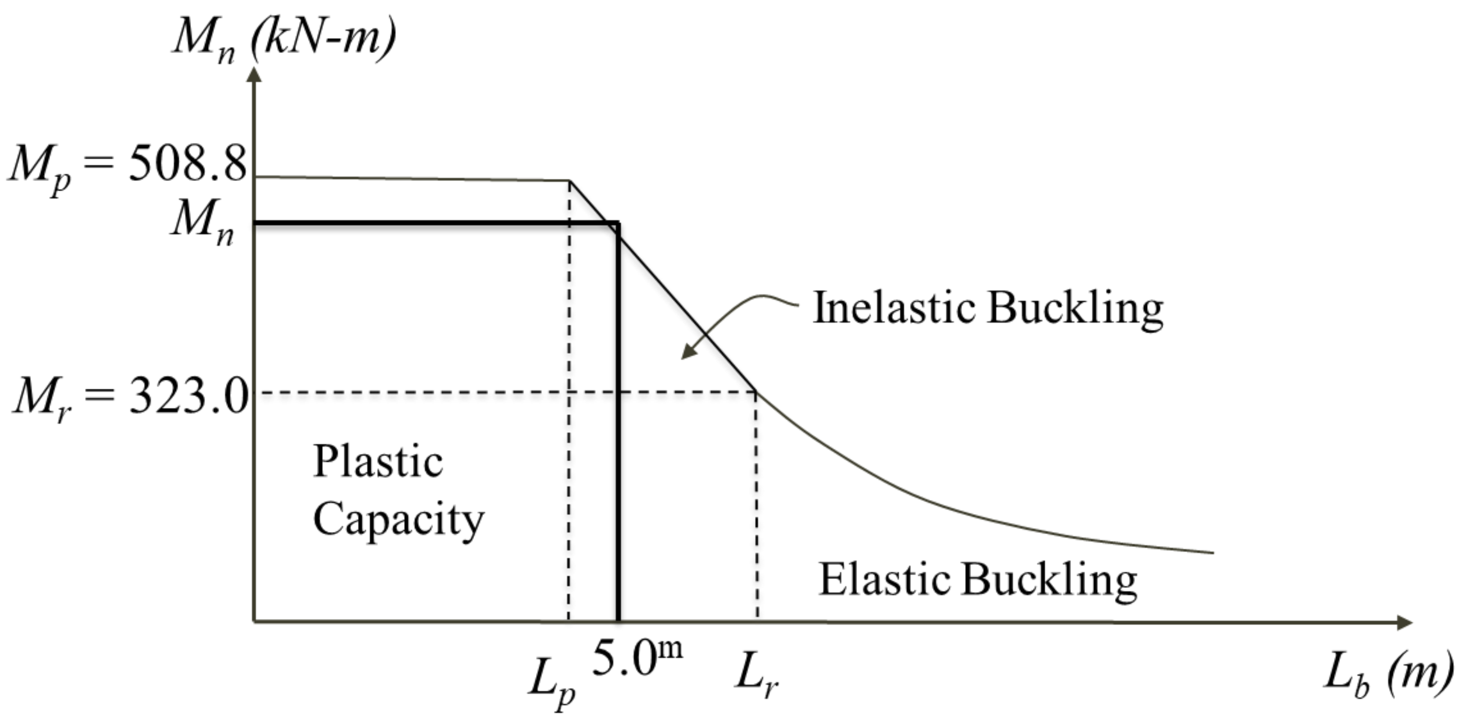
$$M_p = 508.75\;kN - m$$



9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-3: AISC Design Examples V14.2 F1-3A

$$L_p = 3.54\,m \langle L_b = 5.0m \le L_r = 13.3m$$



9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-3: AISC Design Examples V14.2 F1-3A

$$L_p = 3.54\,m \langle L_b = 5.00\,m \le L_r = 13.3\,m$$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yW_{ex})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

Section A: (Moments are % of maximum moment)

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_1 + 4M_2 + 3M_3}$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.438) + 4(0.751) + 3(0.938)}$$

$$C_b = 1.30$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-3: AISC Design Examples V14.2 F1-3A

$$L_p = 3.54\,m \langle L_b = 5.00\,m \le L_r = 13.3$$

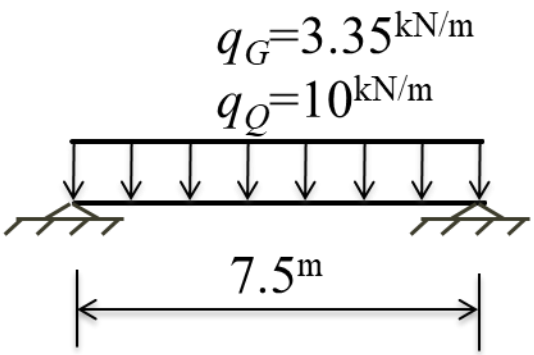
$$M_n = C_b\left[M_p - (M_p - 0.7F_y W_{ex})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

$$M_n = 1.30\left[\begin{array}{l} 508.75^{kN-m} - \left(\begin{array}{l} 508.75^{kN-m} - \\ 0.7x275^{N/mm^2}x1678000^{mm^3}x\dfrac{1}{1E06} \end{array}\right) \\ \left(\dfrac{5.00^m - 3.54^m}{12.96^m - 3.54^m}\right) \end{array}\right] \le M_p$$

$$M_n = 624\ kN - m \rangle M_p = 508.75\ kN/m \Rightarrow M_n = M_p = 508.75\ kN - m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-4: AISC Design Examples V14.2 F1-4A

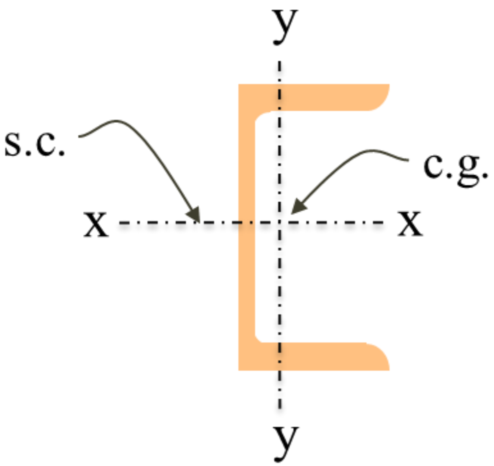


Continuously braced against LTB

S235

$F_y = 235$ MPa

$F_u = 360$ MPa



The beam is a roof beam. Choose a UPE-section.

Serviceability limit state: L/240

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-4: AISC Design Examples V14.2 F1-4A

Required Strength:
LRFD

$$q_u = 1.2x3.35^{kN/m} + 1.6x10^{kN/m}$$

$$q_u = 20.0\ kN/m$$

$$M_u = \frac{20.2^{kN/m}(7,5^m)^2}{8}$$

$$M_u = 140.8\ kN-m$$

Required Strength:
ASD

$$q_a = 3.35^{kN/m} + 10^{kN/m}$$

$$q_a = 13.35\ kN/m$$

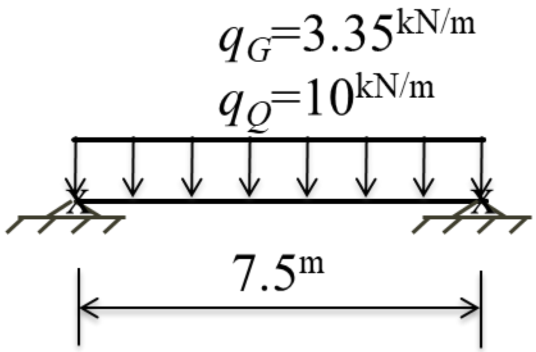
$$M_a = \frac{13.35^{kN/m}(7.5^m)^2}{8}$$

$$M_a = 93.9\ kN-m$$

DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-4: AISC Design Examples V14.2 F1-4A



Continuously braced against LTB

Limit States:

9.2.1 Yielding

~~**9.2.2** Lateral Torsional Buckling~~

15.2 Serviceability

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-4: AISC Design Examples V14.2 F1-4A

Required moment of inertia to satisfy L/240 serviceability limit state **(15.2)**:

$$\Delta_{\max} = \frac{L}{240} = \frac{7500^{mm}}{240} = 31.25\ mm$$

$$I_{x(req)} = \frac{5 q_Q L^4}{384 E \Delta_{\max}}$$

$$I_{x(req)} = \frac{5(10000^{N/m})(1^m/1000^{mm})(7500^{mm})^4}{384(200000^{N/mm^2})31.25^{mm}}$$

$$I_{x(req)} = 6591.8x10^4\ mm^4$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-4: AISC Design Examples V14.2 F1-4A

UPE 300: 44.4 kg/m, h = 300 mm,

$I_x = 7823 \text{ x } 10^4 \text{ mm}^4$, $\phi M_p = 129.7$ kN-m, $M_p/\Omega_b = 86.3$ kN-m

UPE 330: 53.2 kg/m, h = 330 mm, t_w = 11 mm, t_f = 16 mm, b_f = 105 mm, $d = 262^{mm}$

$I_x = 11010 \text{ x } 10^4 \text{ mm}^4$, $M_p = F_y W_{px} = 235^{MPa} \times 791900^{mm3} = 186.1$ kN-m

$\phi M_p = 0.9 \times 186.1^{kN\text{-}m} = 167.5$ kN-m, $M_p/\Omega_b = 111.4$ kN-m

The plastic capacity of UPE 300 is smaller than the required strength.
Choose UPE 330 and control compactness.

Flange

$$\lambda_f = \frac{b}{t_f} = \frac{105^{mm}}{16^{mm}} = 6.56$$

$$\lambda_f \langle \lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 11.08$$

Web

$$\lambda_w = \frac{d}{t_w} = \frac{262^{mm}}{11^{mm}} = 23.82$$

$$\lambda_w \langle \lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 109.6$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-4: AISC Design Examples V14.2 F1-4A

UPE 330: 53.2 kg/m, h = 330 mm,

I_x = 11010 x 10^4 mm^4, ϕM_p = 167.5 KN-m, M_p/Ω_b = 111.4 kN-m

9.2.1 Yielding Limit State:

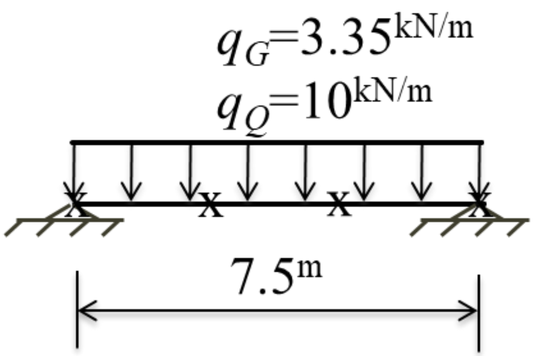
$$M_n = M_p = F_y W_{px} = 235^{MPa} x 791900^{mm^3} = 186.1\ kN - m$$

$$I_x = 11010\ x10^4\ mm^4 \rangle I_{x(gerekli)} = 6591.8x10^4\ mm^4$$ **(15.2) OK**

$$\phi M_n = \phi M_p = 167.5\ kN - m \rangle M_u = 140.8\ kN - m$$ **(9.2.1) OK**

$$M_n / \Omega = M_p / \Omega = 111.4\ kN - m \rangle M_a = 93.9\ kN - m$$ **(9.2.1) OK**

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

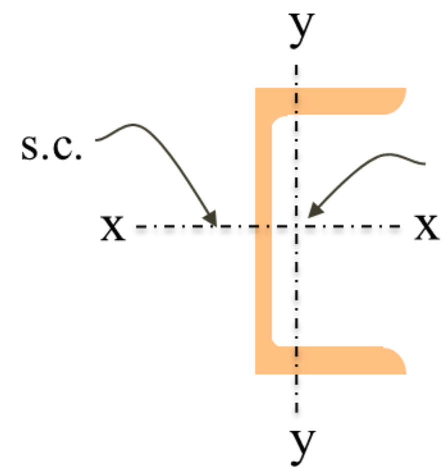


x- Braced at every L/3

S235

$F_y = 235$ MPa

$F_u = 360$ MPa



Calculate the design and allowable strength of UPE 330

Required Strength:

LRFD

$$M_u = 140.8\ kN-m$$

Required Strength:

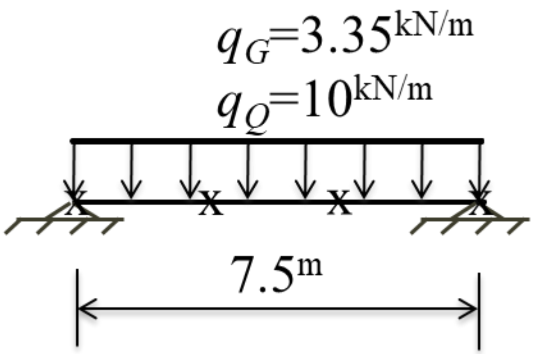
ASD

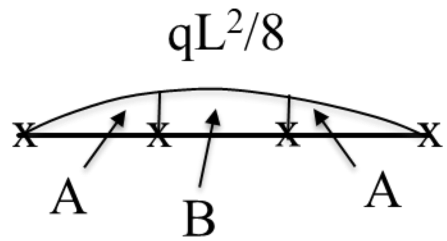
$$M_a = 93.9\ kN-m$$

DESIGN FOR FLEXURE

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-5: AISC Design Examples V14.2 F1-5A





$L_b = 7.5^m/3 = 2.50$ m

Limit States:

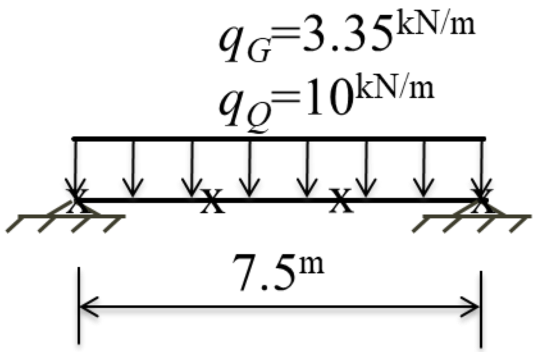
9.2.1 Yielding

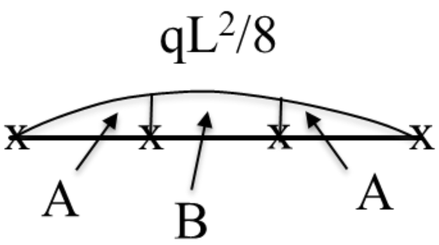
9.2.2 Lateral Torsional Buckling

15.2 Serviceability

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-5: AISC Design Examples V14.2 F1-5A





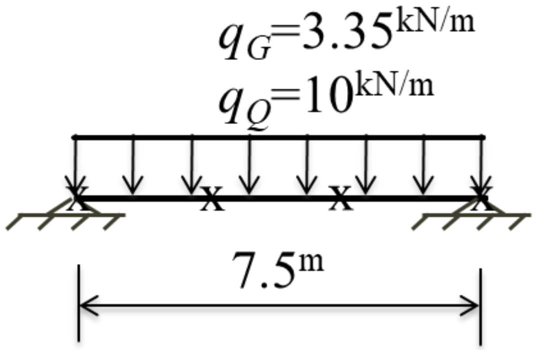
$L_b = 7.5^m/3 = 2.50$ m

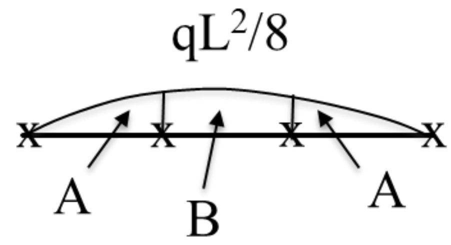
9.2.1 Yielding Limit State:

$$M_n = M_p = F_y W_{px} = 235^{MPa} x 791900^{mm^3} = 186.1\ kN - m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-5: AISC Design Examples V14.2 F1-5A





$L_b = 7.5^m/3 = 2.50$ m

9.2.2 Lateral Torsional Buckling Limit State: Section B. $C_b \approx 1.0$

$$L_p = 1.76\, i_y \sqrt{\frac{E}{F_y}} = 1.63\, m$$

$$L_r = 1.95\, i_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{W_{ex} h_o}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{W_{ex} h_o}{Jc}\right)^2}} = 6.67\, m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-5: AISC Design Examples V14.2 F1-5A

$$L_p = 1.63\,m \langle L_b = 2.50\,m \leq L_r = 6.67\,m$$

$$M_r = 0.7 F_y W_{ex} = \frac{0.7 x 235^{N/mm^2} x 667100^{mm^3}}{1000^{N/k} x 1000^{mm/m}}$$

$$M_r = 156.8\ kN - m$$

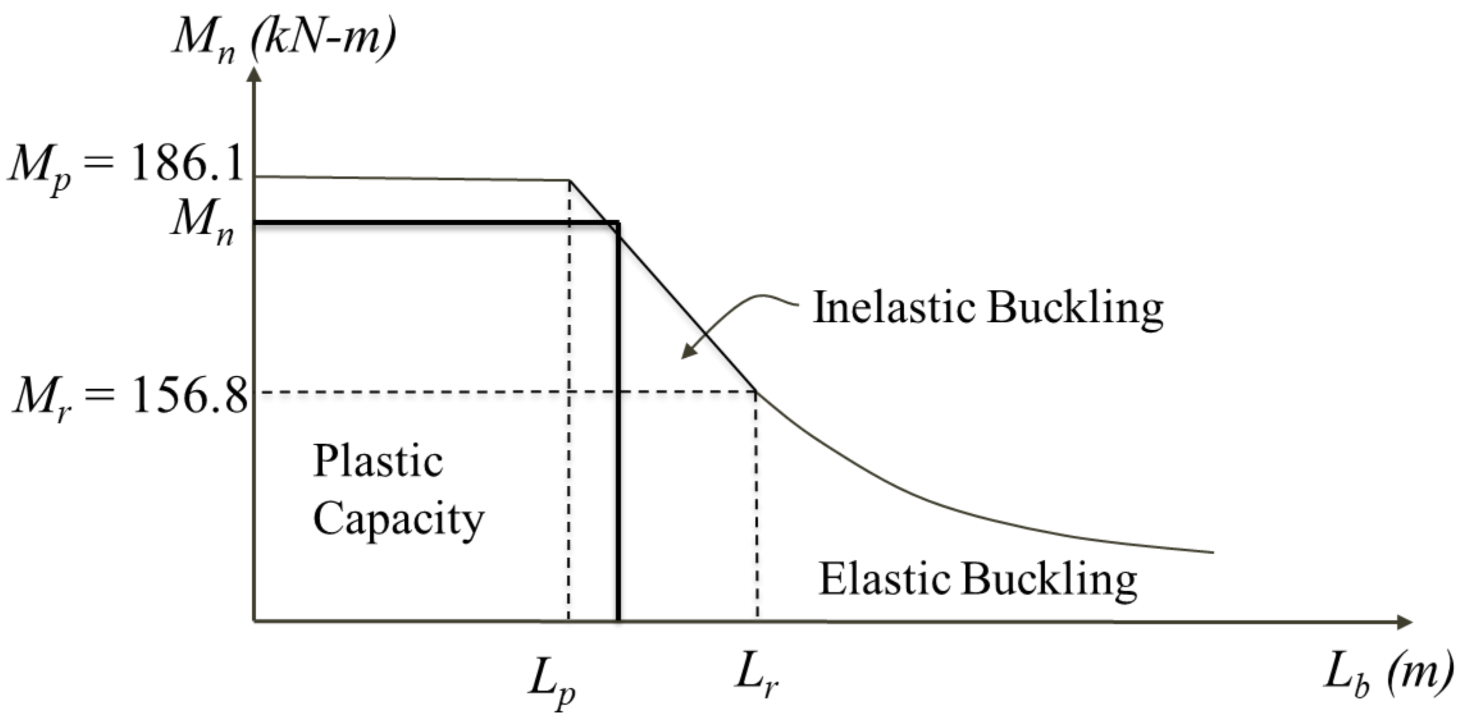
$$M_p = F_y W_{px} = \frac{235^{N/mm^2} x 791900^{mm^3}}{1000^{N/k} x 1000^{mm/m}}$$

$$M_p = 186.1\ kN - m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-5: AISC Design Examples V14.2 F1-5A

$$L_p = 1.63\,m \langle L_b = 2.50\,m \le L_r = 6.67\,m$$



9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-5: AISC Design Examples V14.2 F1-5A

$$L_p = 1.63\,m \langle L_b = 2.50\,m \le L_r = 6.67\,m$$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yW_{ex})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

$$M_n = 1.0\left[186.1^{kN-m} - \begin{pmatrix} 186.1^{kN-m} - \\ 0.7x235^{N/mm^2}x667100^{mm^3}x\dfrac{1}{1E06} \end{pmatrix} \left(\frac{2.5^m - 1.63^m}{6.67^m - 1.63^m}\right)\right] \le M_p$$

$$M_n = 172.9\,kN - m \le M_p = 186.1\,kN/m \Rightarrow M_n = 172.9\,kN - m$$

9.2 Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis

Example 9.2-5: AISC Design Examples V14.2 F1-5A

9.2.2 Lateral Torsional Buckling Limit State is more critical:

$$I_x = 11010\ x10^4\ mm^4 \rangle I_{x(gerekli)} = 6591.8x10^4\ mm^4$$ **OK**

$$\phi M_n = 0.9x172.9^{kN-m} = 155.6\ kN-m \rangle M_u = 140.8\ kN-m$$ **OK**

$$M_n / \Omega = 172.9^{kN-m} / 1.67 = 103.5\ kN-m \rangle M_a = 93.9\ kN-m$$ **OK**

DESIGN FOR FLEXURE

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

The nominal flexural strength, M_n, shall be the lower value obtained according to:

9.3.1 Lateral Torsional Buckling

For lateral-torsional buckling, the provisions of Section 9.2.2 shall apply.

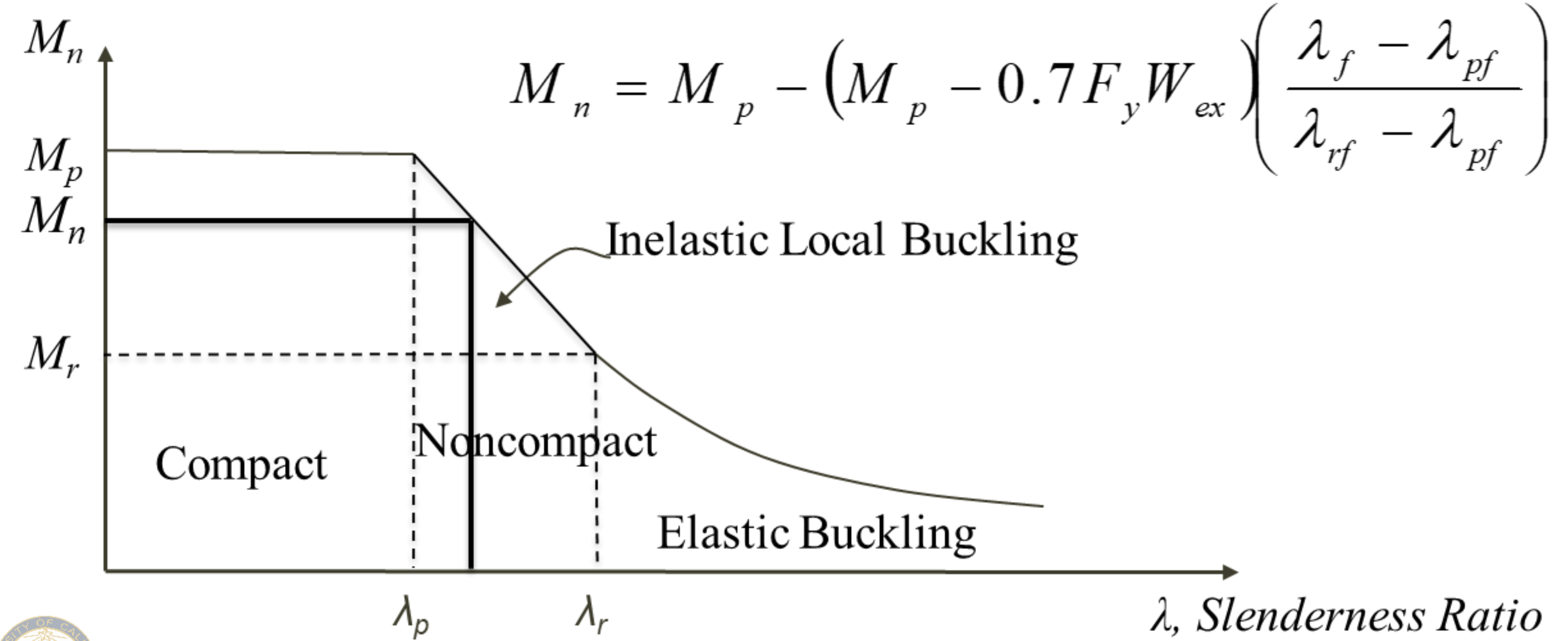
9.3.2 Compression Flange Local Buckling

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

9.3.2 Local Buckling Limit State

(a) For sections with noncompact flanges, M_n, shall be calculated by **Eq. (9.9)**.

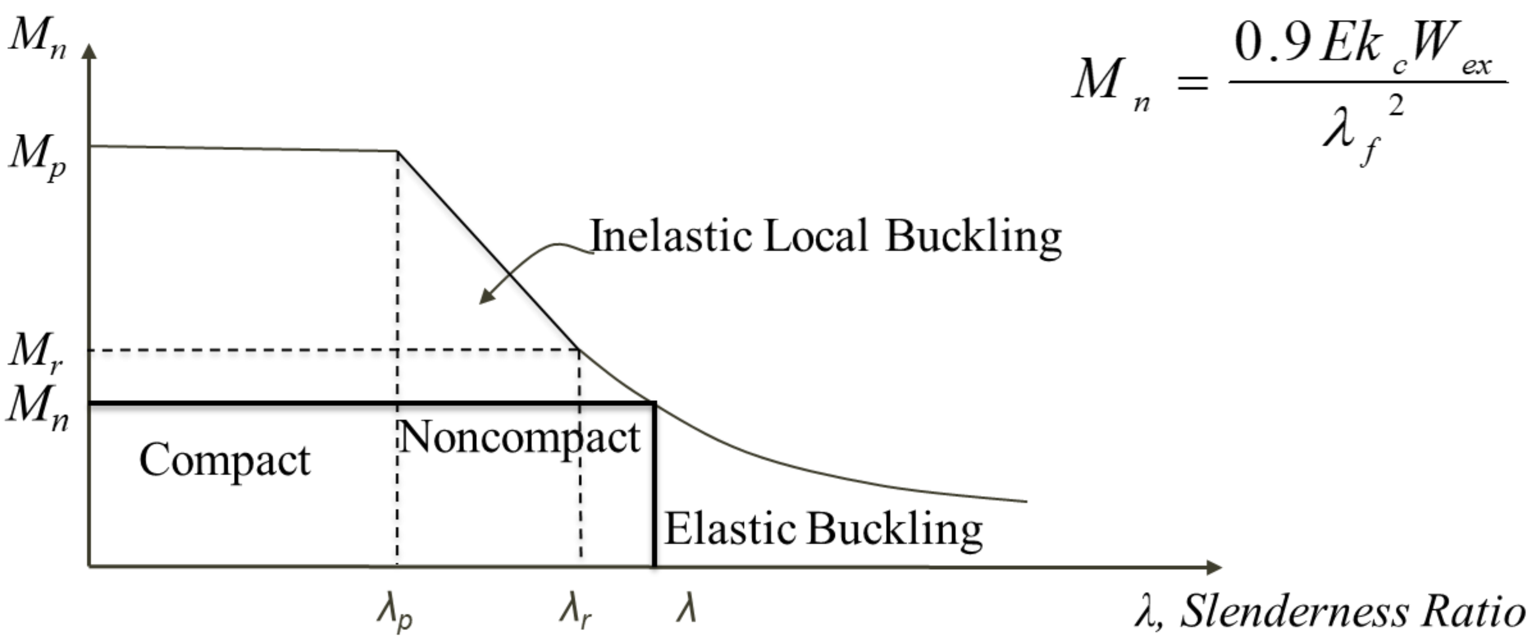
$$M_n = M_p - \left(M_p - 0.7F_yW_{ex}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \textbf{(9.9)}$$



9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

9.3.2 Local Buckling Limit State

(b) For sections with slender flanges, M_n, shall be calculated by **Eq. (9.10).**



$$M_n = \frac{0.9 E k_c W_{ex}}{\lambda_f^2} \quad \textbf{(9.10)}$$

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

9.3 Notations

M_n = nominal flexural strength

M_p = plastic moment

F_y = specified minimum yield stress of the type of steel being used, (MPa)

W_{ex} = elastic section modulus taken about the x-axis, (mm^3)

E = elastic modulus (200000 MPa)

λ_f = flange slenderness ratio, ($\lambda = b_f/2t_f$) **(Table 5.1B)**

λ_{pf} = limiting slenderness parameter for compact flange **(Tablo 5.1B)**

λ_{rf} = limiting slenderness parameter for noncompact flange **(Tablo 5.1B)**

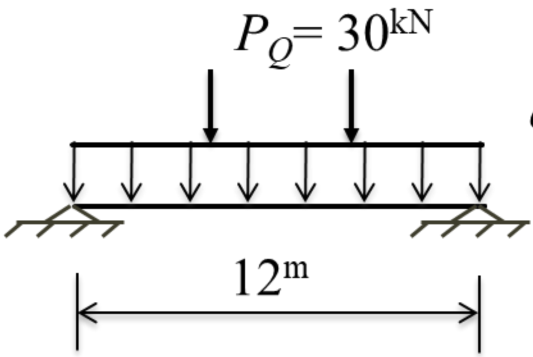
k_c = coefficient for slender unstiffened elements $0.35 \leq k_c = 4\sqrt{h/t_w} \leq 0.76$

h = defined in Section **5.4.1**

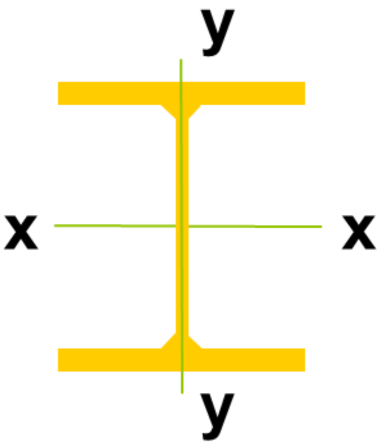
t_w = thickness of web

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

Example 9.3-1: AISC Design Examples V14.2 F3-A



S275
$F_y = 275$ MPa
$F_u = 430$ MPa



Continuously braced against lateral torsional buckling.
Concentrated loads are spaced at L/3 from the supports.
Select a HE-section.
Choose a section with a noncompact flange.
Serviceability limit state: L/360

Limit States:
~~**9.3.1** Lateral Torsional Buckling~~
9.3.2 Local Buckling
15.2 Serviceability Limit State

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

Example 9.3-1: AISC Design Examples V14.2 F3-A

Required Strength:
LRFD

$$q_u = 1.2x0.75^{kN/m} = 0.9\ kN/m$$

$$P_u = 1.6x30^{kN} = 48\ kN$$

$$M_u = \frac{0.9^{kN/m}(12^{m})^2}{8} + 48^{kN}\ x\ \frac{12^{m}}{3}$$

$$M_u = 208.2\ kN-m$$

Required Strength:
ASD

$$q_a = 0.75^{kN/m}$$

$$P_a = 30^{kN}$$

$$M_a = \frac{0.75^{kN/m}(12^{m})^2}{8} + 30^{kN}\ x\ \frac{12^{m}}{3}$$

$$M_u = 133.5\ kN-m$$

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

Example 9.3-1: AISC Design Examples V14.2 F3-A

Choose a HE section with a noncompact flange, which is rare: HE 280 A (76.4 kg/m)

HE 280 A: 76.4 kg/m, h = 270 mm, , $t_w = 8^{mm}$, $t_f = 13^{mm}$, $b_f = 280^{mm}$, $d = 196^{mm}$

$I_x = 13670 \times 10^4$ mm^4, $\phi M_p = 275.2$ kN-m, $M_p/\Omega_b = 183.1$ kN-m

Noncompact Flange

$$\lambda_f = \frac{b}{2t_f} = \frac{280^{mm}}{2(13^{mm})} = 10.8$$

$$\lambda_f \rangle \lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 10.25$$

$$\lambda_r = 1.00\sqrt{\frac{E}{F_y}} = 27.0 \rangle \lambda_f$$

Compact Web

$$\lambda_w = \frac{d}{t_w} = \frac{196^{mm}}{8^{mm}} = 24.5$$

$$\lambda_w \langle \lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 101.40$$

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

Example 9.3-1: AISC Design Examples V14.2 F3-A

9.3.2 Local Buckling Limit State

$$\lambda_p = 10.25 \langle \lambda = 10.8 \le \lambda_r = 27.0 m$$

$$M_r = 0.7 F_y W_{ex} = \frac{0.7 x 275^{N/mm^2} x 1013000^{mm^3}}{1000^{N/k} x 1000^{mm/m}}$$

$$M_r = 195.0\ kN - m$$

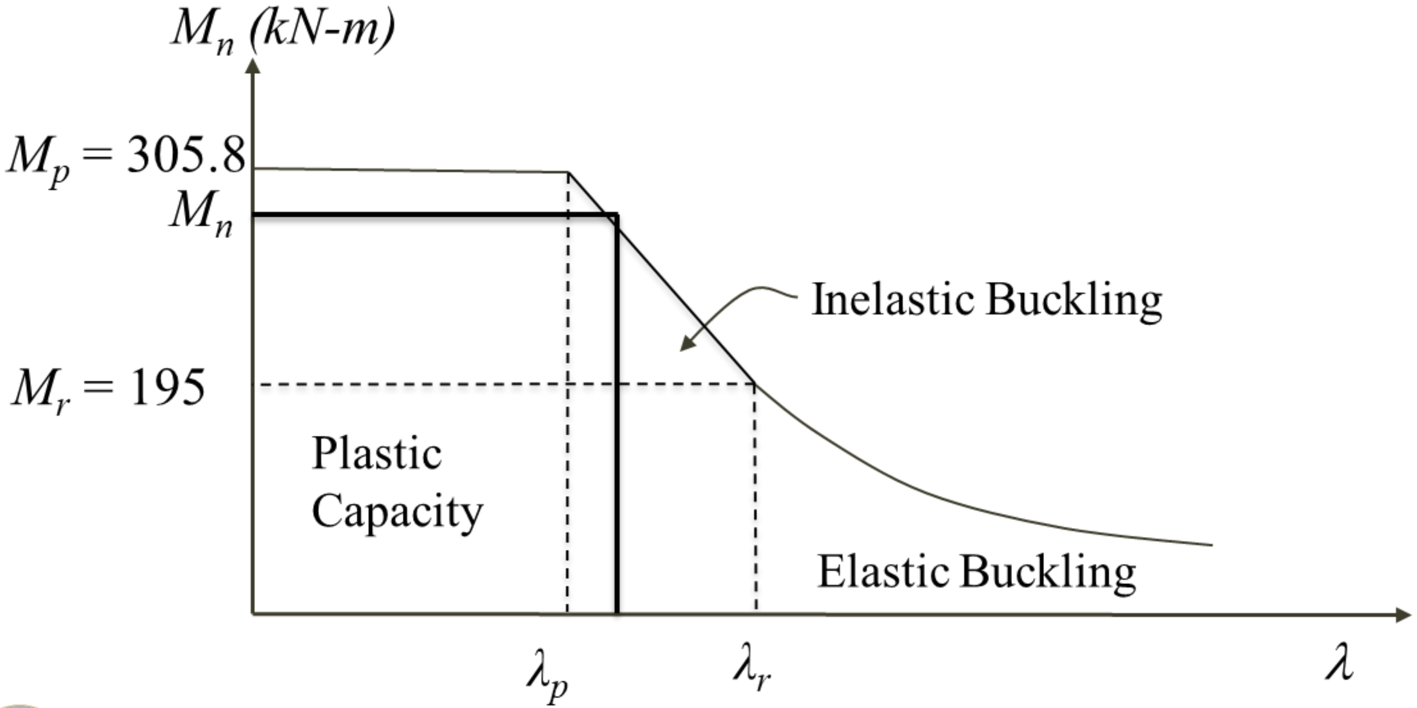
$$M_p = F_y W_{px} = \frac{275^{N/mm^2} x 11120000^{mm^3}}{1000^{N/k} x 1000^{mm/m}}$$

$$M_p = 305.8\ kN - m$$

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

Example 9.3-1: AISC Design Examples V14.2 F3-A

$$\lambda_p = 10.25 \langle \lambda = 10.8 \leq \lambda_r = 27.0$$



9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

Example 9.3-1: AISC Design Examples V14.2 F3-A

$$\lambda_p = 10.25 \langle \lambda = 10.8 \le \lambda_r = 27.0$$

$$M_n = \left[M_p - (M_p - 0.7 F_y W_{ex}) \left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p} \right) \right]$$

$$M_n = \left[308.5^{kN-m} - \left(308.5^{kN-m} - 0.7x275^{N/mm^2} x1013000^{mm^3} x \frac{1}{1E06} \right) \left(\frac{10.8 - 10.25}{27.0 - 10.25} \right) \right]$$

(9.3.2) OK

$$M_n = 0.9x304.8\ kN - m = 274.32\ kN - m \rangle M_u = 208.2\ kN - m$$

9.3 Doubly Symmetric Compact I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis

Example 9.3-1: AISC Design Examples V14.2 F3-A

Required moment of inertia to satisfy the serviceability limit state: L/360 **(15.2)**

$$\Delta_{\max} = \frac{L}{360} = \frac{12000^{mm}}{360} = 33.3\ mm$$

$$I_{x(required)} = \frac{P_Q L^3}{28 E \Delta_{\max}}$$

$$I_{x(required)} = \frac{(30000^{N})(12000^{mm})^3}{28(200000^{N/mm^2})33{,}3^{mm}}$$

$$I_{x(required)} = 27774\ x10^4\ mm^4 \rangle I_{HE\,280\,A} = 13670\ x10^4\ mm^4$$

(15.2): NOT OK. Need a heavier section

Shear Strength of Steel:

Shear Yield Strength = $0.6F_y$

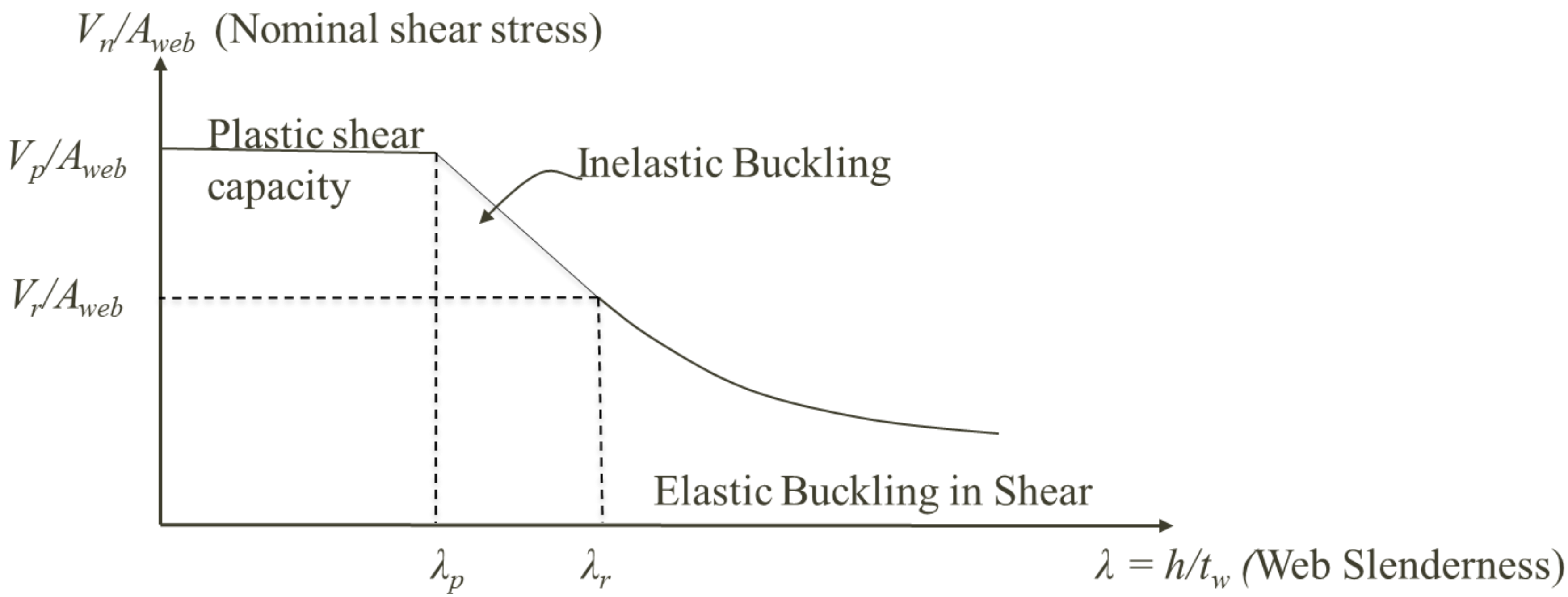
Shear Fracture Strength = $0.6F_u$ (not really used in beams. This will later be used in connections.)

Web carries most of the shear force and the web area is determined as follows:

Web Area = $d \times t_w$, where d is the overall depth of the section.

Shear Stress = $\tau = V/A_{web}$

Shear Curve:



V_p: plastic shear strength of the section
V_r: limit for elastic shear strength of the section

Shear Design:

Chapter 10 in the Turkish Steel Specification addresses shear design.

The design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v, shall be determined as follows:

$$\phi_v = 0.90(LRFD) \qquad \Omega_v = 1.67(ASD)$$

$$V_u \leq \phi_b V_n \quad (LRFD) \qquad V_a \leq \frac{V_n}{\Omega_b} \quad (ASD)$$

For rolled I-shaped members with $\lambda \leq 2.24\sqrt{(E/F_y)}$:

Shear Design:

10.2 I-Sections and U-Sections

10.2.1 Shear Strength

$$V_n = 0.6F_y A_w C_{v1}$$ **Eq. (10.1)**

(a) For rolled I-shaped members with **$\lambda \leq 2.24\sqrt{(E/F_y)}$**:

$$\frac{h}{t_w} \leq 2.24\sqrt{\frac{E}{F_y}} \Rightarrow \quad \phi_b = 1.0\,(LRFD) \quad \Omega_b = 1.50\,(ASD) \quad C_{v1} = 1.0$$

SHEAR DESIGN

10.2 I-Sections and U-Sections

10.2.1 Shear Strength

$$V_n = 0.6 F_y A_w C_{v1}$$ **Eq. (10.1)**

(b) For webs of all other doubly symmetric shapes and singly symmetric I-shapes, and channels, except round HSS, C_{v1} is determined as follows:

$$\phi_v = 0.90 (LRFD) \qquad \frac{h}{t_w} \le 1.10\sqrt{\frac{k_v E}{F_y}} \Rightarrow C_{v1} = 1.0$$ **Denk. (10.2a)**

$$\Omega_v = 1.67 (ASD) \qquad \frac{h}{t_w} > 1.10\sqrt{\frac{k_v E}{F_y}} \Rightarrow C_{v1} = \frac{1.1\sqrt{k_v E / F_y}}{h / t_w}$$ **Denk. (10.2b)**

Shear Design:

10.2 I-Sections and U-Sections

10.2.1 Shear Strength

The web shear post buckling strength coefficient, k_v, is determined as follows:

(a) For webs without transverse stiffeners and with $h/t_w < 260$:

$$\frac{h}{t_w} \leq 260 \Rightarrow k_v = 5.34 \qquad \textbf{Denk. (10.3a)}$$

Shear Design:

10.2 I-Sections and U-Sections

10.2.1 Shear Strength

The web shear post buckling strength coefficient, k_v, is determined as follows:

(b) For webs with transverse stiffeners:

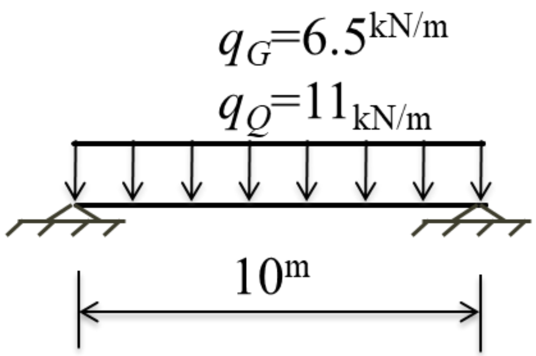
$$\frac{a}{h} \leq 3.0 \Rightarrow k_v = 5 + \frac{5}{(a/h)^2}$$ **Denk. (10.3b)**

$$\frac{a}{h} > 3.0 \Rightarrow k_v = 5.34$$ **Denk. (10.3c)**

a = clear distance between transverse stiffeners, (mm)

BEAM DESIGN PROCEDURE

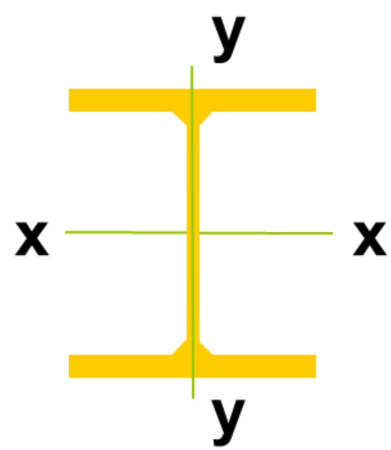
Example 6.1: AISC Design Examples V14.2 F1-1A



S275

$F_y = 275$ MPa

$F_u = 430$ MPa



Continuously braced against lateral torsional buckling

Choose a HE shape for the beam shown.

Depth of the beam is limited to: 45 cm

Serviceability limit state: L/360

BEAM DESIGN PROCEDURE

Example 6.1: AISC Design Examples V14.2 F1-1A

Required Strength:
<u>LRFD</u>

$$q_u = 1.2x6.5^{kN/m} + 1.6x11^{kN/m}$$

$$q_u = 25.4\ kN/m$$

$$M_u = \frac{25.4^{kN/m}(10^m)^2}{8}$$

$$M_u = 317.5\ kN - m$$

$$V_u = \frac{25.4^{kN/m}(10^m)}{2}$$

$$V_u = 127\ kN$$

Required Strength:
<u>ASD</u>

$$q_a = 6.5^{kN/m} + 11^{kN/m}$$

$$q_a = 17.5\ kN/m$$

$$M_a = \frac{17.5^{kN/m}(10^m)^2}{8}$$

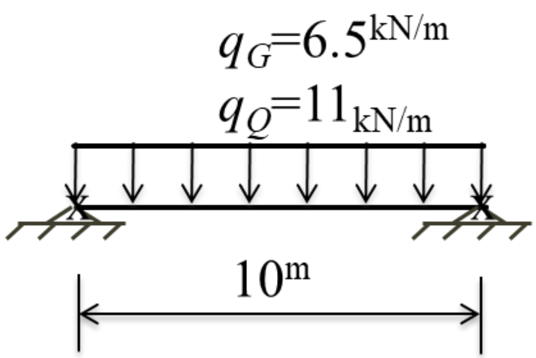
$$M_a = 218.8\ kN - m$$

$$V_a = \frac{17.5^{kN/m}(10^m)}{2}$$

$$V_a = 87.5\ kN$$

BEAM DESIGN PROCEDURE

Example 6.1: AISC Design Examples V14.2 F1-1A



S275
$F_y = 275$ MPa
$F_u = 430$ MPa

Continuously braced against lateral torsional buckling
Choose a doubly symmetric compact I-shape (Section 9.2).

Limit States:
9.2.1 Yielding
~~**9.2.2** Lateral Torsional Buckling~~
15.2 Serviceability
10.2 Shear Yielding

Example 6.1: AISC Design Examples V14.2 F1-1A

Required moment of inertia to scomply **Section 15.2** serviceability limit state:

$$\Delta_{\max} = \frac{L}{360} = \frac{10000^{mm}}{360} = 27\ mm$$

$$I_{x(required)} = \frac{5 q_Q L^4}{384 E \Delta_{\max}}$$

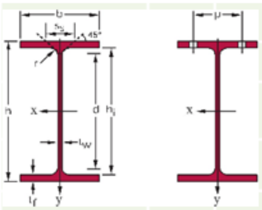
$$I_{x(required)} = \frac{5(11000^{N/m})(1^m/1000^{mm})(10000^{mm})^4}{384(200000^{N/mm^2})27^{mm}}$$

$$I_{x(required)} = 26524\ x10^4\ mm^4$$

BEAM DESIGN PROCEDURE

Example 6.1: AISC Design Examples V14.2 F1-1A

$$I_{x(required)} = 26524\ x10^4\ mm^4$$

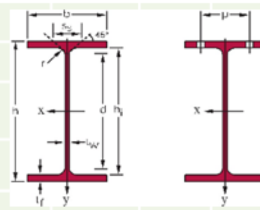


Bu tablolar tasarımda kullanılmaz.

Tabloların kullanımıyla ilgili tüm sorumluluk kullanıcıya aittir.

Tablo 6-1

HE ve HL Kesitleri



Bu tablolar tasarımda kullanılmaz.

oların kullanımıyla ilgili tüm sorumluluk kullanıcıya aittir.

Tablo 6-1

HE ve HL Kesitleri

Kesit		Alan	Derinlik	Gövde Kalınlık	Başlık Genişlik	Başlık Kalınlık	Ölçüler			Kompakt Kesit Kriterleri		x-x Ekseni (Güçlü Eksen)				y-y Ekseni (Zayıf Eksen)				Burulma Katsayıları	
İsim	G	A	h	t_w	b_f	t_f	k	k_1(r)	T(d)	$b_f/2t_f$	d/t_w	I_x	$W_{el.x}$	$W_{pl.x}$	i_x	I_y	$W_{el.y}$	$W_{pl.y}$	i_y	J	C_w
	kg/m	mm^2 x 10^2	mm	mm	mm	mm	mm	mm	mm			mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	mm^4 x 10^4	mm^3 x 10^3	mm^3 x 10^3	mm x 10	mm^4 x 10^4	mm^6 x 10^9
HE 340 A	105,0	133,5	330,0	9,5	300,0	16,5	43,5	27,0	243,0	9,1	25,6	27690,0	1678,0	1850,0	14,40	7436,0	495,7	755,9	7,46	127,2	1824,0
HE 340 B	134,0	170,9	340,0	12,0	300,0	21,5	48,5	27,0	243,0	7,0	20,3	36660,0	2156,0	2408,0	14,65	9690,0	646,0	985,7	7,53	257,2	2454,0
HE 340 M	248,0	315,8	377,0	21,0	309,0	40,0	67,0	27,0	243,0	3,9	11,6	76370,0	4052,0	4718,0	15,55	19710,0	####	1953,0	7,90	1506,0	5584,0

BEAM DESIGN PROCEDURE

Example 6.1: AISC Design Examples V14.2 F1-1A

HE 340 A: 105 kg/m, h = 330 mm, $t_w = 9.5^{mm}$, $t_f = 16.5^{mm}$, $b_f = 300_{mm}$, $d = 243^{mm}$
$I_x = 27690 \times 10^4$ mm^4, ϕM_p = 457.9 kN-m, M_p/Ω_b = 304.6 kN-m

HE 320 B: 127 kg/m, h = 320 mm,
$I_x = 30820 \times 10^4$ mm^4, ϕM_p = 531.9 KN-m, M_p/Ω_b = 353.9 kN-m

The beam is continuously braced against LTB.
Check the compactness of HE 340 A (lighter).

Flange

$$\lambda_f = \frac{b}{2t_f} = \frac{300^{mm}}{2(16^{mm})} = 9.09$$

$$\lambda_f \langle \lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 10.25$$

Web

$$\lambda_w = \frac{d}{t_w} = \frac{243^{mm}}{9.5^{mm}} = 25.58$$

$$\lambda_w \langle \lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 101.40$$

Example 6.1: AISC Design Examples V14.2 F1-1A

HE 340 A: 105 kg/m, h = 330 mm,

$I_x = 27690 \text{ x } 10^4 \text{ mm}^4$, $\phi M_p = 457.9$ kN-m, $M_p/\Omega_b = 304.6$ kN-m

9.2.1 Yielding Limit State:

$$M_n = M_p = F_y W_{px} = 275^{MPa} x1850000^{mm^3} = 508.75\ kN - m$$

$$I_x = 27690\ x10^4\ mm^4 \rangle I_{x(required)} = 26524\ x10^4\ mm^4 \quad \textbf{(15.2) OK}$$

$$\phi M_p = 457.9\ kN - m \rangle M_u = 317.5\ kN - m \quad \textbf{(9.2.1) OK}$$

$$M_p / \Omega = 304.6\ kN - m \rangle M_a = 218.8\ kN - m \quad \textbf{(9.2.1) OK}$$

BEAM DESIGN PROCEDURE

Example 6.1: AISC Design Examples V14.2 F1-1A

$$M_u = 317.5\ kN - m \qquad M_a = 218.8\ kN - m$$

F_y=275 MPa

Tablo 6-4

İZİN VERİLEBİLİR MOMENT KAPASİTESİ (kNm)

HE Kesitleri

Ω_b=1.67 φ_b=0.9, Ω_v=1.50 φ_v=1.0

Tasarım		M_{px}/Ω_b	$\varphi_b M_{px}$	M_{rx}/Ω_b	$\varphi_b M_{rx}$	L_p	L_r	I_x	V_{nx}/Ω_v	$\varphi_v V_{nx}$
Kesit	Z_x	kNm	kNm	kNm	kNm				kN	kN
	mm³ x10³	ASD	LRFD	ASD	LRFD	mm	mm	mm⁴ x10⁴	ASD	LRFD
HE 360M	4989	821,5	1234,8	495,3	744,5	3716,4	23726,1	84870	727,7	1091,5
HE 360B	2683	441,8	664,0	276,6	415,8	3555,0	15142,0	43190	433,1	649,7
HE 360A	2088	343,8	516,8	218,0	327,6	3526,6	13199,4	33090	346,5	519,8
HE 340M	4718	776,9	1167,7	467,1	702,0	3749,6	25111,0	76370	686,1	1029,1
HE 340B	2408	396,5	596,0	248,5	373,5	3574,0	15386,9	36660	392,0	588,1
HE 340A	1850	304,6	457,9	193,4	290,7	3540,8	13317,1	27690	310,4	465,5
HE 320M	4435	730,3	1097,7	437,6	657,7	3773,4	26582,0	68130	644,5	966,7
HE 320B	2149	353,9	531,9	222,0	333,7	3593,0	15665,9	30820	352,9	529,4
HE 320A	1628	268,1	402,9	170,5	256,2	3555,0	13452,0	22930	276,2	414,3

BEAM DESIGN PROCEDURE

Example 6.1: AISC Design Examples V14.2 F1-1A

HE 340 A: 105 kg/m, $h = 330$ mm, $t_w = 9.5^{mm}$, $t_f = 16.5^{mm}$, $b_f = 300_{mm}$, $d = 243^{mm}$

$I_x = 27690 \times 10^4$ mm^4, $\phi M_p = 457.9$ kN-m, $M_p/\Omega_b = 304.6$ kN-m

10.2 Shear Yielding Limit State:

$$\frac{h}{t_w} = \frac{330^{mm}}{9.5^{mm}} = 34.7 \le 2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{200000^{MPa}}{275^{MPa}}} = 60.40 \Rightarrow$$

$$\phi_b = 1.0\,(YDKT) \qquad \Omega_b = 1.50\,(GKT)$$

$$C_{v1} = 1.0$$

BEAM DESIGN PROCEDURE

Example 6.1: AISC Design Examples V14.2 F1-1A

HE 340 A: 105 kg/m, $h = 330$ mm, $t_w = 9.5^{mm}$, $t_f = 16.5^{mm}$, $b_f = 300_{mm}$, $d = 243^{mm}$
$I_x = 27690 \times 10^4$ mm^4, $\phi M_p = 457.9$ kN-m, $M_p/\Omega_b = 304.6$ kN-m

(a) For webs of rolled I-shaped members with **$\lambda \leq 2.24\sqrt{(E/F_y)}$: (10.2.1)**

$$V_n = 0.6 F_y A_w C_{v1}$$

$$V_n = 0.6x275^{MPa} x330^{mm} x9.5^{mm}$$

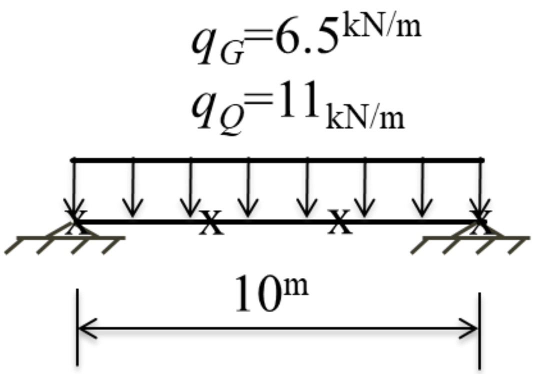
$$V_n = 517.3\ kN$$

$$\phi V_n = 1.0x517.3^{kN} = 517.3\ kN > 127 kN$$

(10.2.1) OK

$$\frac{V_n}{\Omega} = \frac{517.3^{kN}}{1.5} = 344.9 kN > 87.5 kN$$

Example 6.2: AISC Design Examples V14.2 F1-2A



S275

$F_y = 275$ MPa

$F_u = 430$ MPa

x- Brace Points: Spaced at L/3

Calculate the required moment for HE 340 A.

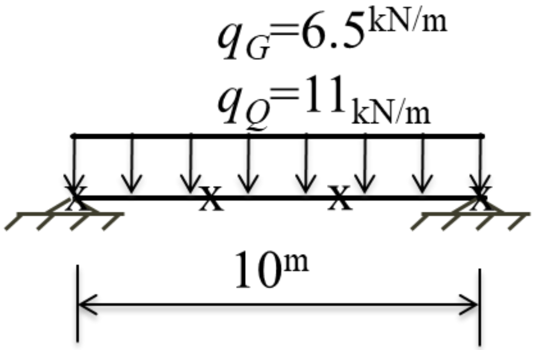
Required Strength:
LRFD

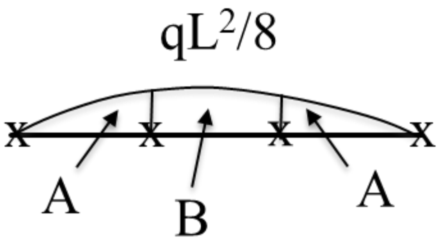
$$M_u = 317.5\ kN - m$$

Required Strength:
ASD

$$M_a = 218.8\ kN - m$$

Example 6.2: AISC Design Examples V14.2 F1-2A

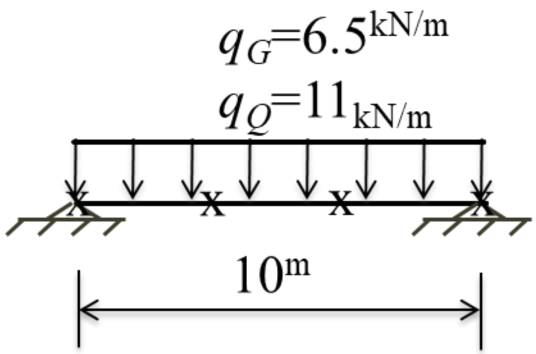


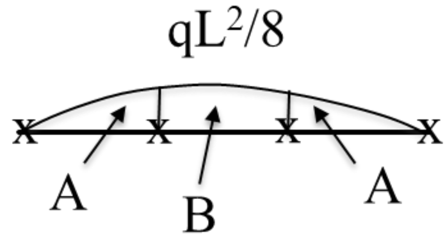


x- Brace Points: Spaced at L/3

Limit States:
9.2.1 Yielding
9.2.2 Lateral Torsional Buckling
15.2 Serviceability
10.2 Shear Yielding

Example 6.2: AISC Design Examples V14.2 F1-2A





L_b = 10m/3 = 3.33 m

9.2.2 Lateral Torsional Buckling Limit State:

Section B is more critical. C_b can be taken as = 1.0

$$L_p = 1.76\, i_y \sqrt{\frac{E}{F_y}} = 3.54\, m$$

$$L_r = 1.95\, i_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{W_{ex} h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y}{E} \frac{W_{ex} h_o}{Jc} \right)^2}} = 13.3\, m$$

Example 6.2: AISC Design Examples V14.2 F1-2A

$$L_b \le L_p \Rightarrow M_n = M_p$$

$$L_b = 3.33\,m \le L_p = 3.54\,m \Rightarrow M_n = M_p$$

$$M_p = F_y W_{px} = 275^{N/mm^2} x1850000^{\ mm^3}$$

$$M_p = 508.75\ kN - m$$

$$\phi M_p = 0.9x508.75^{kN-m} = 457.9\ kN - m \rangle M_u = 317.5_{kN-m}$$

$$M_p / \Omega = 508.75^{kN-m} / 1.67 = 304.6\ kN - m \rangle M_a = 218.8\ kN - m$$

Example 6.2: AISC Design Examples V14.2 F1-2A

Show that C_b of section B is more critical than C_b of section A.

Section B: (Moments are given as percentages of maximum moment)

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_1 + 4M_2 + 3M_3}$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.972) + 4(1.00) + 3(0.972)}$$

$$C_b = 1.01$$

BEAM DESIGN PROCEDURE

Example 6.2: AISC Design Examples V14.2 F1-2A

Show that C_b of section B is more critical than C_b of section A.

Section A: (Moments are given as percentages of maximum moment)

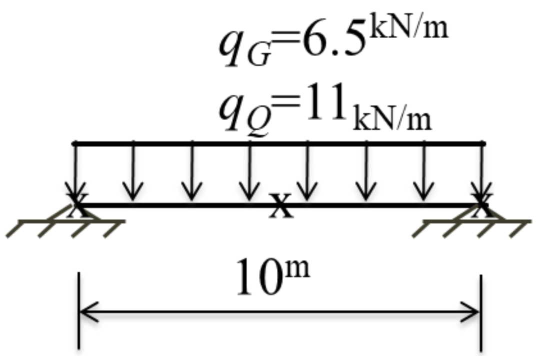
$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_1 + 4M_2 + 3M_3}$$

$$C_b = \frac{12.5(0.889)}{2.5(0.889) + 3(0.306) + 4(0.556) + 3(0.750)}$$

$$C_b = 1.46$$

Section B, which has a higher moment and a lower C_b is more critical

Example 6.3: AISC Design Examples V14.2 F1-3A



x- Brace Locations: Spaced at L/2

S275

$F_y = 275$ MPa

$F_u = 430$ MPa

Calculate the required moment for HE 340 A.

Required Strength:
LRFD

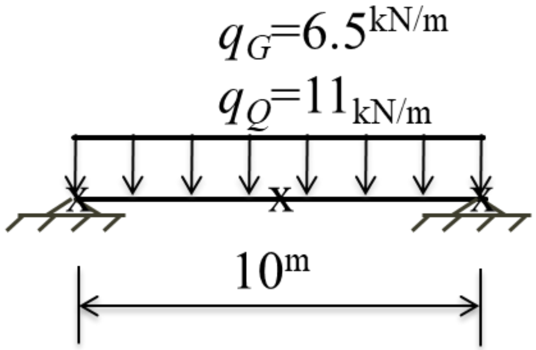
$$M_u = 317.5\ kN - m$$

Required Strength:
ASD

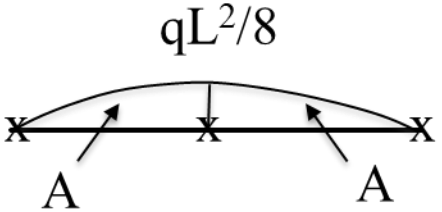
$$M_a = 218.8\ kN - m$$

BEAM DESIGN PROCEDURE

Example 6.3: AISC Design Examples V14.2 F1-3A



x- Brace Locations: Spaced at L/2



Limit States:

9.2.1 Yielding

9.2.2 Lateral Torsional Buckling

15.2 Serviceability

10.2 Shear Yielding

BEAM DESIGN PROCEDURE

Example 6.3: AISC Design Examples V14.2 F1-3A

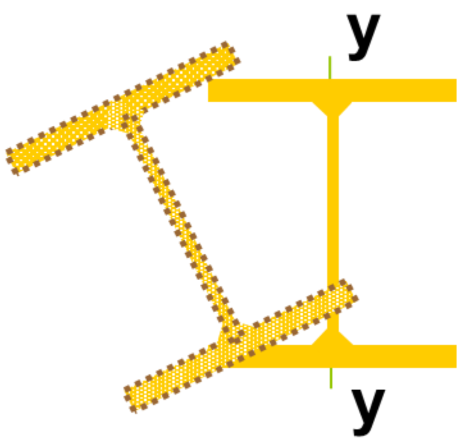
$$L_p = 3.54m \langle L_b = 5.0m \le L_r = 13.3m$$

$$M_r = 0.7F_y W_{ex} = \frac{0.7x275^{N/mm^2} x1678000^{mm^3}}{1000^{N/k} x1000^{mm/m}}$$

$$M_r = 323.0\ kN - m$$

$$M_p = F_y W_{px} = \frac{275^{N/mm^2} x1850000^{mm^3}}{1000^{N/k} x1000^{mm/m}}$$

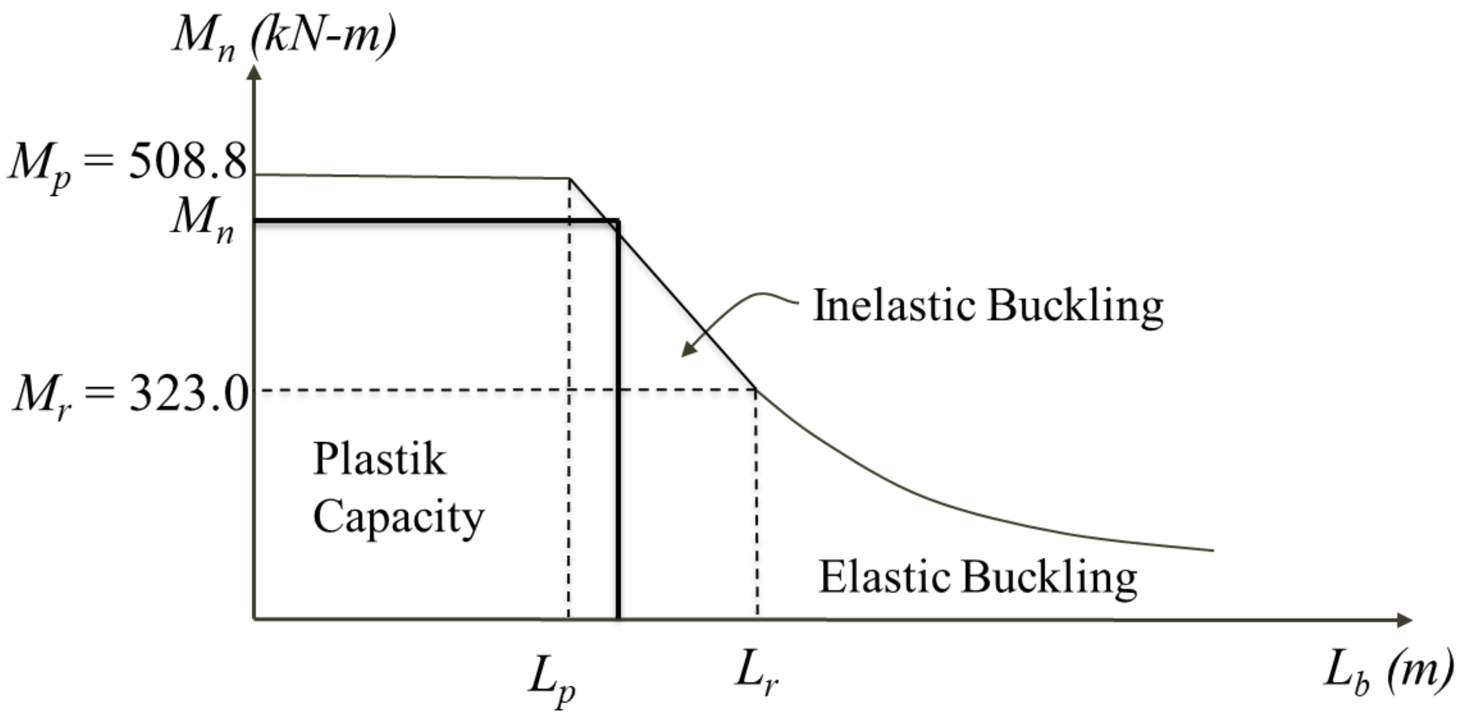
$$M_p = 508.75\ kN - m$$



BEAM DESIGN PROCEDURE

Example 6.3: AISC Design Examples V14.2 F1-3A

$$L_p = 3.54\,m \langle L_b = 5.0\,m \le L_r = 13.3\,m$$



BEAM DESIGN PROCEDURE

Example 6.3: AISC Design Examples V14.2 F1-3A

$$L_p = 3.54\,m \langle L_b = 5.00\,m \le L_r = 13.3\,m$$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yW_{ex})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

Section A: (Momens are shown as percentages of maximum moments)

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_1 + 4M_2 + 3M_3}$$

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.438) + 4(0.751) + 3(0.938)}$$

$$C_b = 1.30$$

BEAM DESIGN PROCEDURE

Example 6.3: AISC Design Examples V14.2 F1-3A

$$L_p = 3.54\,m \langle L_b = 5.00\,m \le L_r = 13.3$$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yW_{ex})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

$$M_n = 1.30\left[508.75^{kN-m} - \left(508.75^{kN-m} - 0.7x275^{N/mm^2}x1678000^{mm^3}x\frac{1}{1E06}\right)\left(\frac{5.00^m - 3.54^m}{12.96^m - 3.54^m}\right)\right] \le M_p$$

$$M_n = 624\ kN - m \rangle M_p = 508.75\ kN/m \Rightarrow M_n = M_p = 508.75\ kN - m$$

BEAM DESIGN PROCEDURE

9.3 Kuvvetli Eksenleri Etrafında Eğilme Etkisindeki Kompakt Gövdeli ve Kompakt Olmayan veya Narin Başlıklı Çift Simetri Eksenli I-Enkesitli Elemanlar

Bu tür elemanlar için *karakteristik eğilme momenti dayanımı, M_n,* aşağıda verilen sınır durumları için hesaplanan değerlerin en küçüğü olarak alınacaktır.

9.3.1 Yanal Burulmalı Burkulma Sınır Durumu

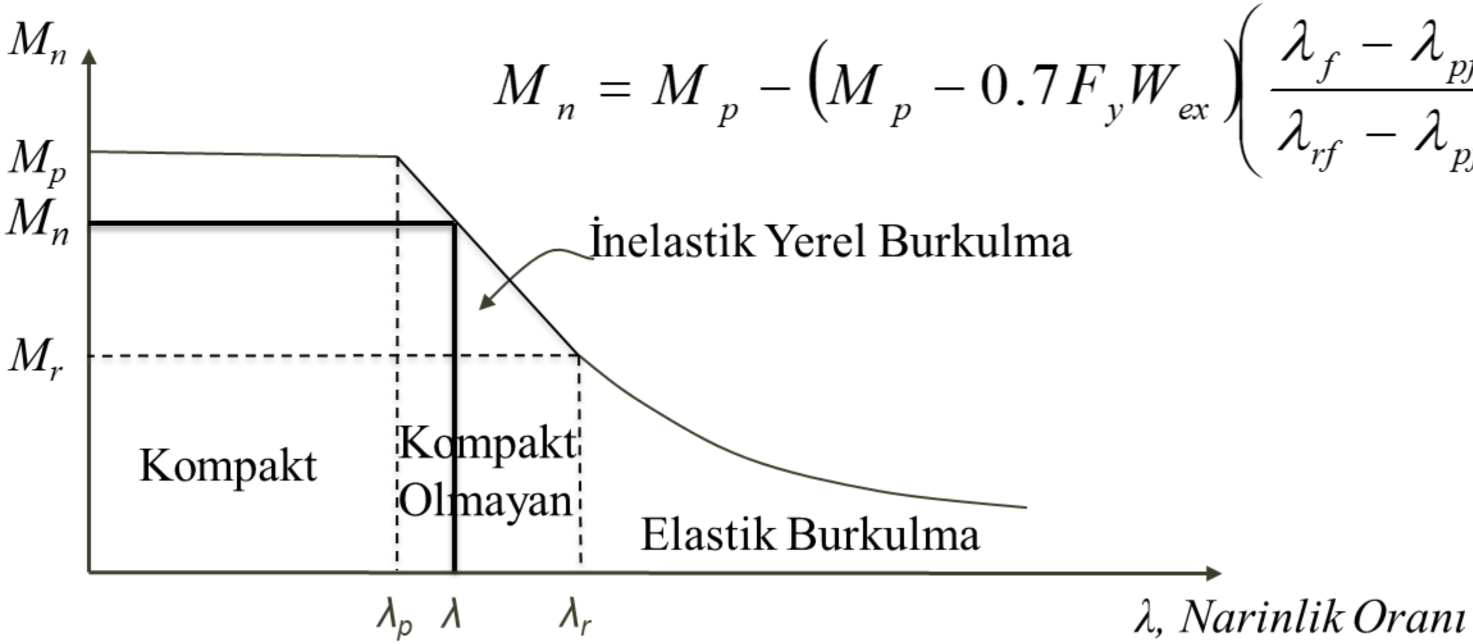
Karakteristik eğilme momenti dayanımı, M_n, Bölüm 9.2.2'ye göre belirlenecektir.

9.3.2 Yerel Burkulma Sınır Durumu

9.3.2 Yerel Burkulma Sınır Durumu

(a) I-enkesitin gövde parçasının kompakt ve başlık parçalarının kompakt olmayan koşulunu sağlaması durumunda, karakteristik eğilme momenti dayanımı, M_n, **Denk.(9.9)** ile hesaplanacaktır.

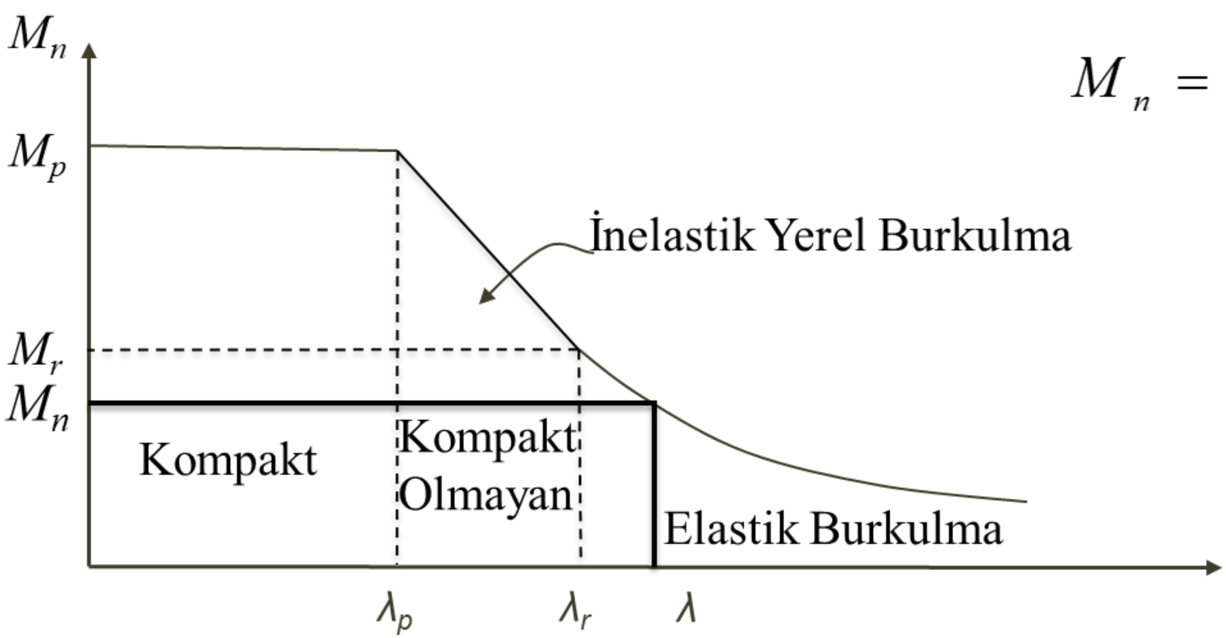
$$M_n = M_p - \left(M_p - 0.7 F_y W_{ex}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \textbf{(9.9)}$$



BEAM DESIGN PROCEDURE

9.3.2 Yerel Burkulma Sınır Durumu

(b) I-enkesitin gövde parçasının kompakt koşulunu sağlaması ve başlık parçalarının narin olması durumunda, karakteristik eğilme momenti dayanımı, M_n, **Denk.(9.10)** ile belirlenecektir.



$$M_n = \frac{0.9 E k_c W_{ex}}{\lambda_f^2} \qquad \textbf{(9.10)}$$

BEAM DESIGN PROCEDURE

M_n = Karakteristik eğilme momenti

M_p = Plastik eğilme momenti

F_y = Yapısal çelik karakteristik akma dayanımı

W_{ex} = x-ekseni etrafında elastik mukavemet momenti

E = Yapısal çelik elastisite modülü (200000 MPa)

λ_f = Enkesitin başlık parçası narinliği, ($\lambda = b_f / 2t_f$) **(Tablo 5.1B)**

λ_{pf} = Kompakt başlık parçası için narinlik sınırı **(Tablo 5.1B)**

λ_{rf} = Kompakt olmayan başlık parçası için narinlik sınırı **(Tablo 5.1B)**

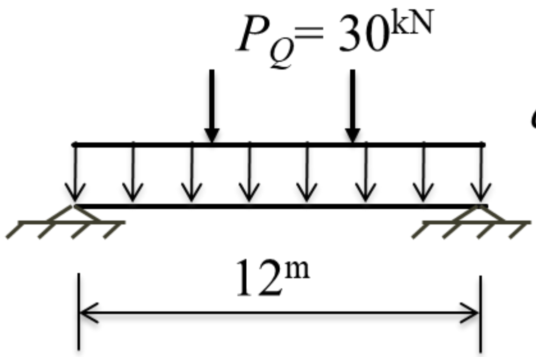
k_c = Levha burkulma katsayısı,

h = **Bölüm 5.4.1'**de tanımlanan enkesit ölçüsü.;

t_w = Gövde kalınlığı.

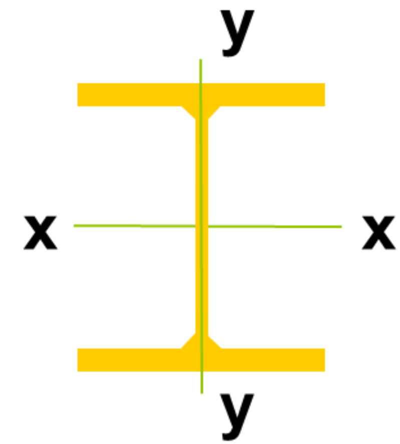
BEAM DESIGN PROCEDURE

Örnek 9.3-1: AISC Design Examples V14.2 F3-A



$q_G = 0.75^{kN/m}$

S275
$F_y = 275$ MPa
$F_u = 430$ MPa



Yanal olarak sürekli desteklenmiş.
Tekil yükler mesnetlerden L/3 mesafede.
Gösterilen kiriş için HE-enkesit seçin.
Başlık yerel burkulmasının etkilerini göstermek için başlığı kompakt olmayan bir kesit seçilecektir.
Kullanılabilirlik sınır durumu: L/360

Sınır Durumları:
~~**9.3.1** Yanal Burulmalı Burkulma~~
9.3.2 Yerel Burkulma
15.2 Kullanılabilirlik

Örnek 9.3-1:

Gerekli Dayanım:
<u>YDKT</u>

$$q_u = 1.2x0.75^{kN/m}$$

$$q_u = 0.9\ kN/m$$

$$P_u = 1.6x30^{kN}$$

$$P_u = 48\ kN$$

$$M_u = \frac{0.9^{kN/m}(12^m)^2}{8} + 48^{kN}x\frac{12^m}{3}$$

$$M_u = 208.2\ kN - m$$

Gerekli Dayanım:
<u>GKT</u>

$$q_a = 0.75^{kN/m}$$

$$P_a = 30^{kN}$$

$$M_a = \frac{0.75^{kN/m}(12^m)^2}{8} + 30^{kN}x\frac{12^m}{3}$$

$$M_u = 133.5\ kN - m$$

Örnek 9.3-1:

HE 280 A: 76.4 kg/m, h = 270 mm, , $t_w = 8^{mm}$, $t_f = 13^{mm}$, $b_f = 280^{mm}$, $d = 196^{mm}$
$I_x = 13670 \times 10^4$ mm^4, ϕM_p = 275.2 kN-m, M_p/Ω_b = 183.1 kN-m

Kompakt Olmayan Başlık

$$\lambda_f = \frac{b}{2t_f} = \frac{280^{mm}}{2(13^{mm})} = 10.8$$

$$\lambda_f \rangle \lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 10.25$$

$$\lambda_r = 1.00\sqrt{\frac{E}{F_y}} = 27.0 \rangle \lambda_f$$

Kompakt Gövde

$$\lambda_w = \frac{d}{t_w} = \frac{196^{mm}}{8^{mm}} = 24.5$$

$$\lambda_w \langle \lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 101.40$$

Örnek 9.3-1:

$$\lambda_p = 10.25 \langle \lambda = 10.8 \leq \lambda_r = 27.0m$$

$$M_r = 0.7F_yW_{ex} = \frac{0.7x275^{N/mm^2}x1013000^{mm^3}}{1000^{N/k}x1000^{mm/m}}$$

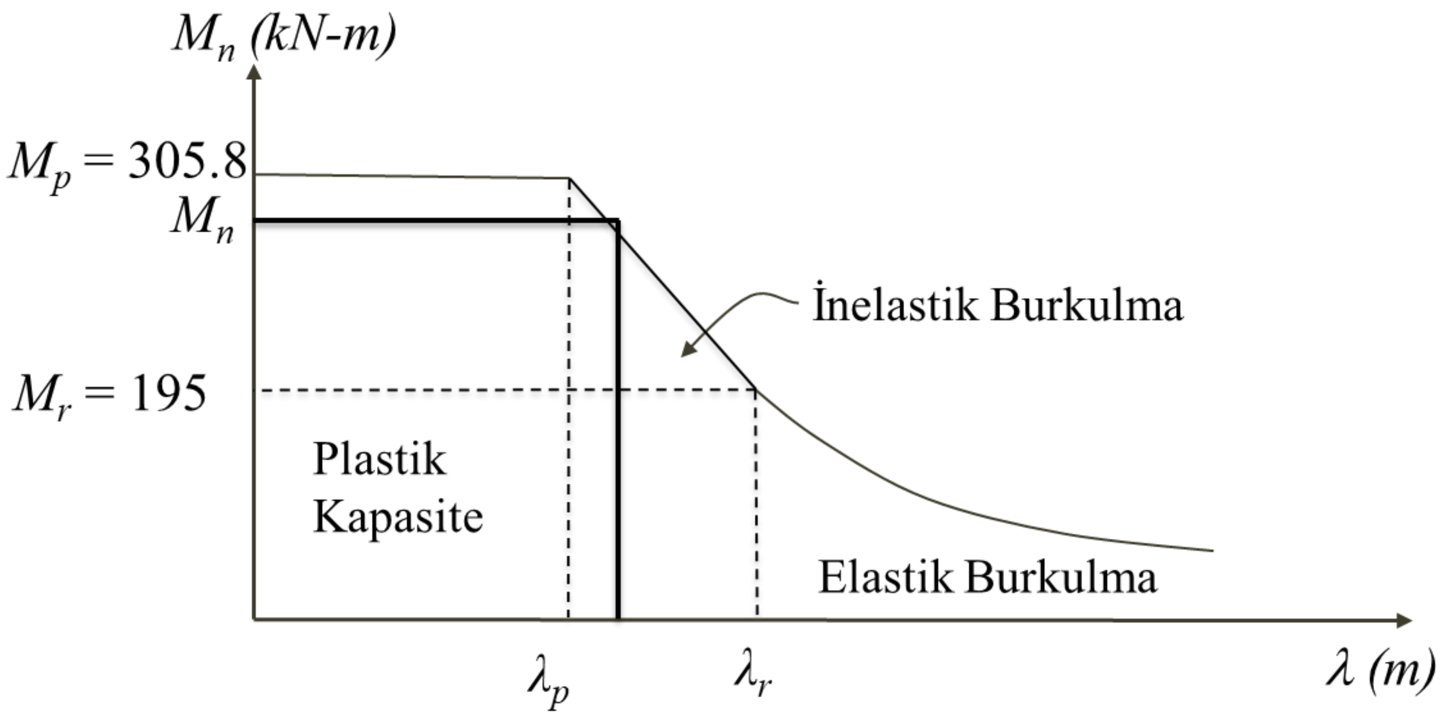
$$M_r = 195.0\ kN - m$$

$$M_p = F_yW_{px} = \frac{275^{N/mm^2}x11120000^{mm^3}}{1000^{N/k}x1000^{mm/m}}$$

$$M_p = 305.8\ kN - m$$

Örnek 9.3-1:

$$\lambda_p = 10.25 \langle \lambda = 10.8 \leq \lambda_r = 27.0$$



Örnek 9.3-1:

$$M_n = \left[M_p - (M_p - 0.7 F_y W_{ex}) \left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p} \right) \right]$$

$$M_n = \left[308.5^{kN-m} - \left(308.5^{kN-m} - 0.7 x 275^{N/mm^2} x1013000^{mm^3} x \frac{1}{1E06} \right) \left(\frac{10.8 - 10.25}{27.0 - 10.25} \right) \right]$$

(9.3.2) Tamam

$$M_n = 0.9 x 304.8\ kN - m = 274.32\ kN - m \rangle M_u = 208.2\ kN - m$$

Örnek 9.3-1:

$$\Delta_{\max} = \frac{L}{360} = \frac{12000^{mm}}{360} = 33.3\ mm$$

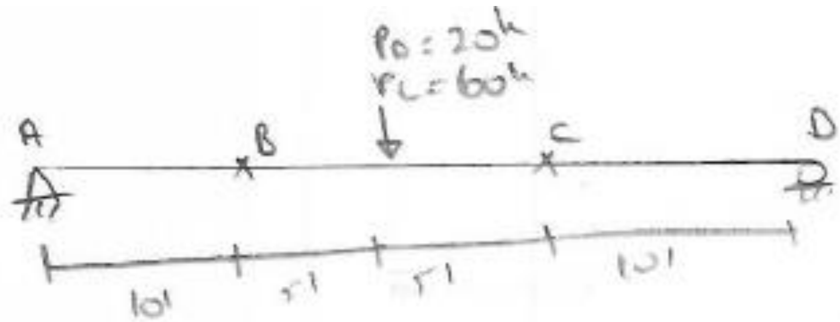
$$I_{x(gerekli)} = \frac{P_Q L^3}{28 E \Delta_{\max}}$$

$$I_{x(gerekli)} = \frac{(30000^{N})(12000^{mm})^3}{28(200000^{N/mm^2})33{,}3^{mm}}$$

$$I_{x(gerekli)} = 27774\ x10^4\ mm^4 \rangle I_{HE\,280\,A} = 13670\ x10^4\ mm^4$$

(15.2) Daha büyük bir kesit gerekli

Example:

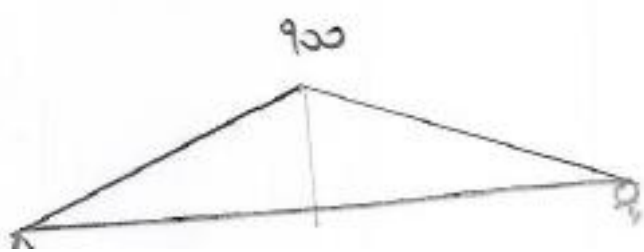


A992 steel. Ignore self weight. $\Delta_{LL} \le L/360$

Solution: Step 1 $P_u = 1.2P_D + 1.6P_L = 1.2 \times 20 + 1.6 \times 60 = 120^k$

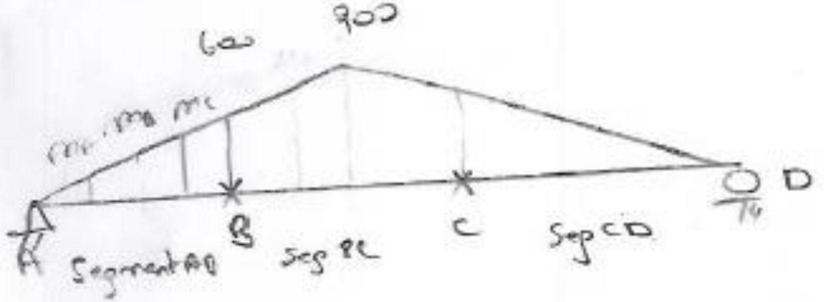
Step 2 Draw moment diagram to find max demand

$M_{max} = \frac{PL}{4} = 900$ k-ft



BEAM DESIGN PROCEDURE

Step 3 Determine Cb factors for each unbraced seg



$$C_b = \frac{12.5\,M_{max}}{2.5\,M_{max} + 3M_A + 4M_B + 3M_C}$$

Segment AB

$$C_b = \frac{12.5 \times 600}{2.5 \times 600 + 3 \times 150 + 4 \times 300 + 3 \times 450} = 1.67$$

BEAM DESIGN PROCEDURE

Segment BC

Cb = 1.087

Segment CD

Cb = 1.67

Step 4 Trial Design.

	Lb	Mu	Cb	Meff = Mu/Cb
AB	10'	600	1.67	359.3
BC	10'	900	1.087	828
CD	10'	600	1.67	359.3

LTB key points

* Cb factor applied to supply (capacity side)
* Lb (unbraced length)

BEAM DESIGN PROCEDURE

Assume segment BC governs the design:

Enter "Beam Chart" with $L_b = 10'$ and $M_{eff} = 828$ k-ft

From AISCM pg 3-118, try W27x84 $\phi_b M_n = 843$ k-ft $> M_{eff}$

$\downarrow \times 1.083$

① $\phi_b M_n = 916 > M_u$

& check if $\phi_b M_p > M_u$

From Zx Table; $\phi_b M_p = 915$ k-ft $>$ ② $M_u = 900$ k-ft

Step 5 Check if AB and CD segments are okay.

From AISCM Table 3-2

BEAM DESIGN PROCEDURE

W27x84 $\begin{cases} \phi_b M_p = 915 \text{ k-ft} \\ \phi_b BF = 26.4 \text{ kips} \end{cases}$

$L_p = 7.31'$

$L_r = 20.8'$

Since $L_p < L_b < L_r \rightarrow$ Zone 2

($L_b = 10'$)

$$\phi_b M_n = C_b [\phi_b M_p - \phi_b BF (L_b - L_p)]$$

$$= 1.67 [915 - 26.4 (10 - 7.31)]$$

$$= 1409 \text{ k-ft} \not< \phi_b M_p = 915 \text{ k-ft}$$

$$\phi_b M_n = 915 \text{ k-ft} > M_u = 600 \text{ k-ft (OK)}$$

BEAM DESIGN PROCEDURE