University of California, San Diego
Faculty of Engineering

COMPRESSION MEMBERS

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INTRODUCTION

<u>Compression Members with No Bending Moment</u>

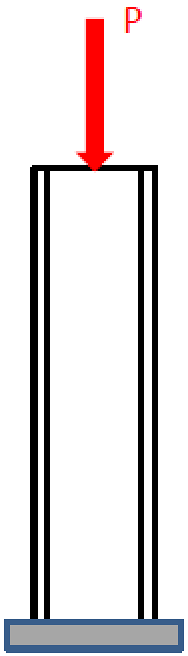
‘There are few situations where structural steel elements are subjected to concentric compressive axial forces without any accompanying bending moment. Examples include truss web members, compression chords of some trusses, and some columns in buildings.

In this section, the analysis and design of structural members subject to axial compression with no accompanying bending moment. In Chapter 8, beam–columns will be discussed, that is, structural steel elements subjected to combined axial loads and bending moments, which may occur due to eccentrically applied axial load or to bending loads acting within the length of the member.

INTRODUCTION

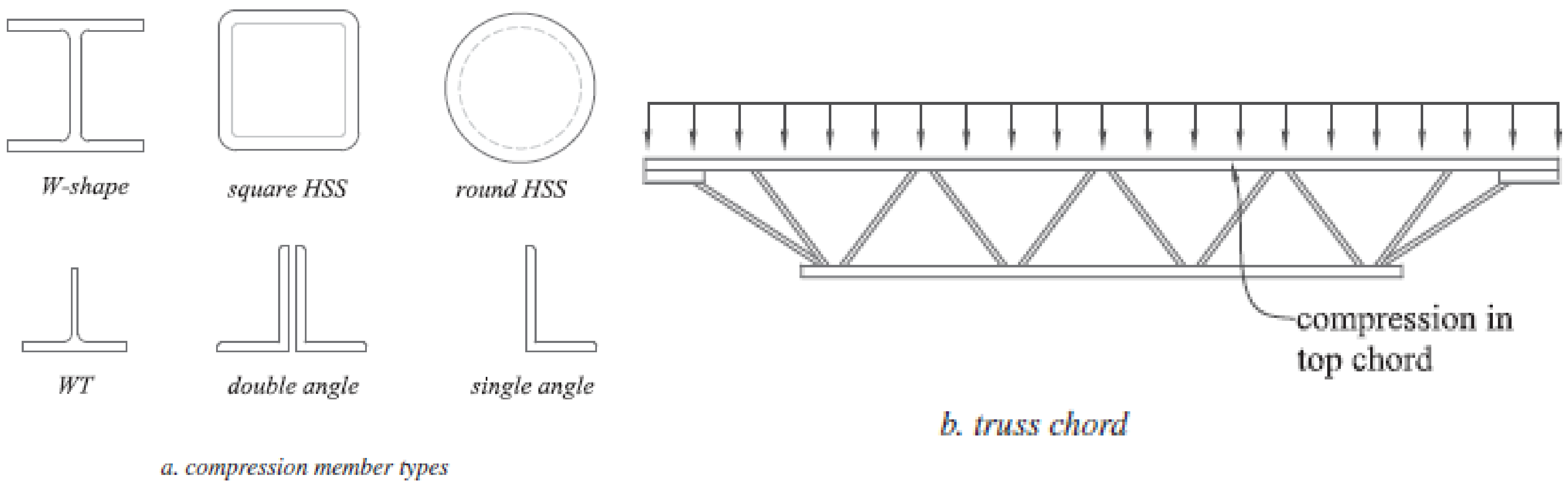
Compression Members with No Bending Moment

Columns are defined as the structural members subject to primarily axial compression force (no bending moment). General load on compression members include axial force, bending and torsional moments. In this part of the course, we will only consider “axially loaded” column member. In other words, only axial (compressive) force exists.



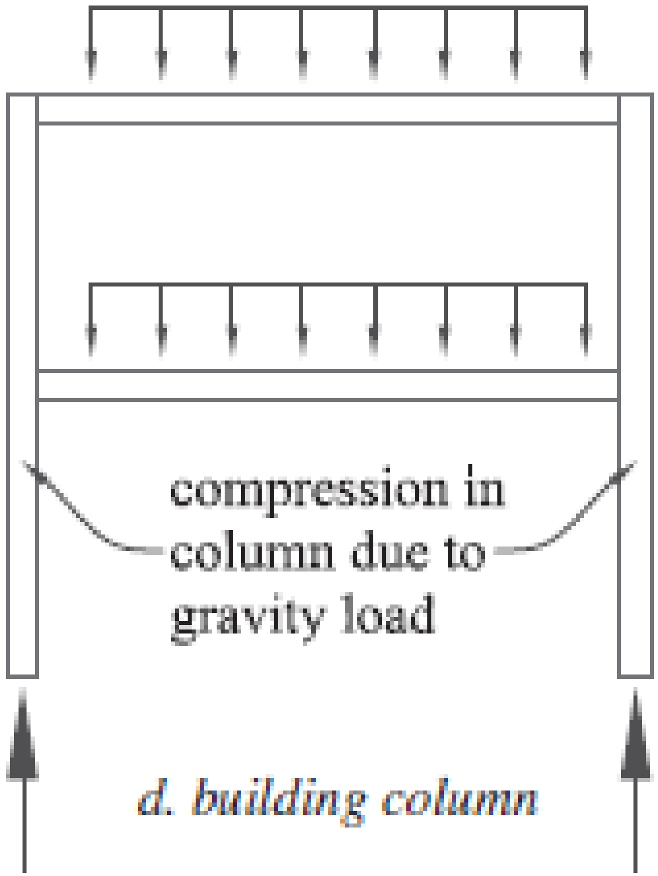
Compression Members with No Bending Moment

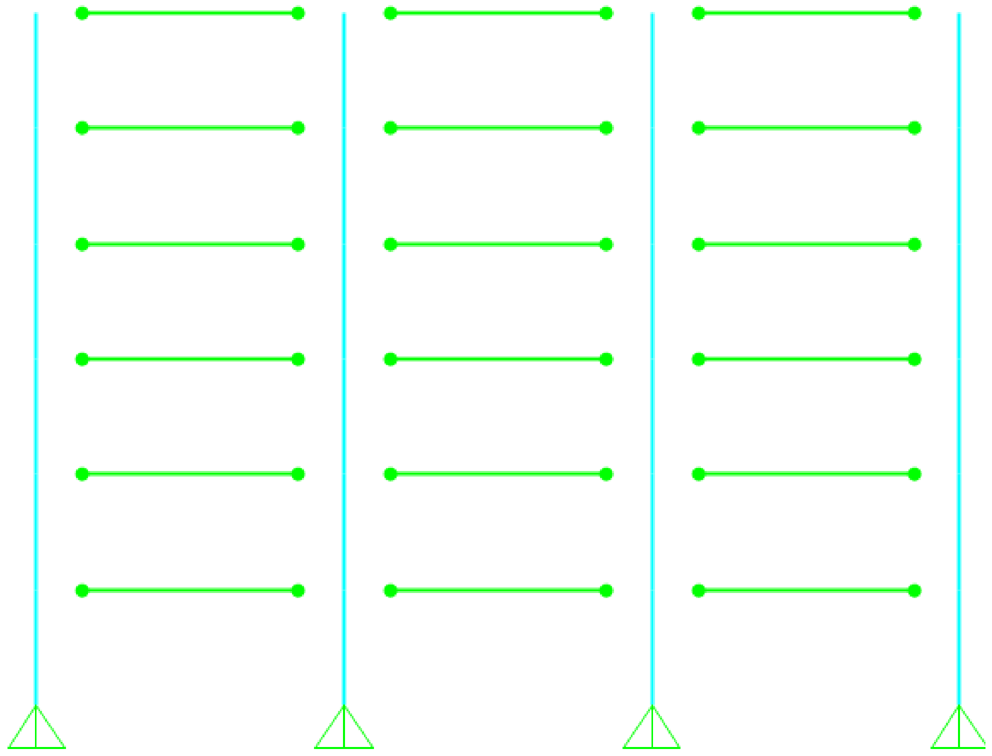
'In structural steel, the common shapes used for columns are wide flange shapes, round and square hollow structural sections (HSS), and built-up sections. For truss members, double- or single-angle shapes are used, as well as round and square HSS and WT-shapes.'



(Abi Aghayere and Jason Vigil (2015))

INTRODUCTION



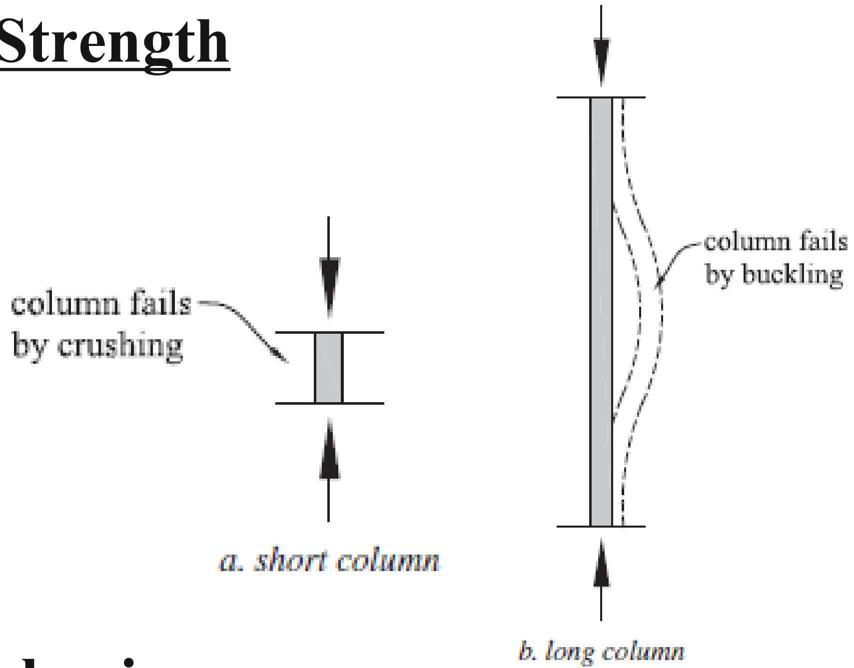


Compression Members with No Bending Moment

COLUMN CRITICAL BUCKLING LOAD

<u>Factors Effecting Compressive Strength</u>

- Cross-section
- Support conditions
- Yield strength of steel
- Residual stresses
- Bending axis
- Initial imperfections
- Width-thickness ratios



<u>Types of Columns in Terms of Behavior</u>

- Short Column: Failure mode is by crushing compression.
- Long Columns: Failure mode is buckling at the midspan of the member. This is called a slender, or long, column.
- Intermediate Columns: Fail by a combination of buckling and compression.

COLUMN CRITICAL BUCKLING LOAD

Euler Critical Buckling Load

For a **pure long compression member**, the axial load at which the column begins to bow outward (flexural buckling) is called the **Euler critical buckling load**.

Assuming a perfectly straight member without any initial crookedness and no residual stresses, the Euler critical buckling load for a column with pinned ends:

$$P_e = \frac{\pi^2 EI}{L^2} \quad (5.1)$$

P_e = Euler critical buckling load

E = modulus of elasticity = 200,000 MPa

I = moment of inertia of the cross section (mm^4)

L = length of the column between brace points

COLUMN CRITICAL BUCKLING LOAD

Euler Elastic Critical Buckling Stress

Knowing that $I = Ai^2$ and that the compression stress on any member is f_c = P/A, we can express the Euler critical buckling load in terms of stress as:

$$F_e = \frac{\pi^2 E}{\left(\frac{L}{i}\right)^2} \quad (5.2)$$

F_e = Euler critical buckling stress (MPa)
E = modulus of elasticity = 200,000 MPa
I = moment of inertia of the cross section (mm^4)
L = length of the column between brace points

A = cross sectional area of the compression member (mm^2)
i = radius of gyration of the compression member (mm)

$$i = \sqrt{\frac{I}{A}}$$

COLUMN CRITICAL BUCKLING LOAD

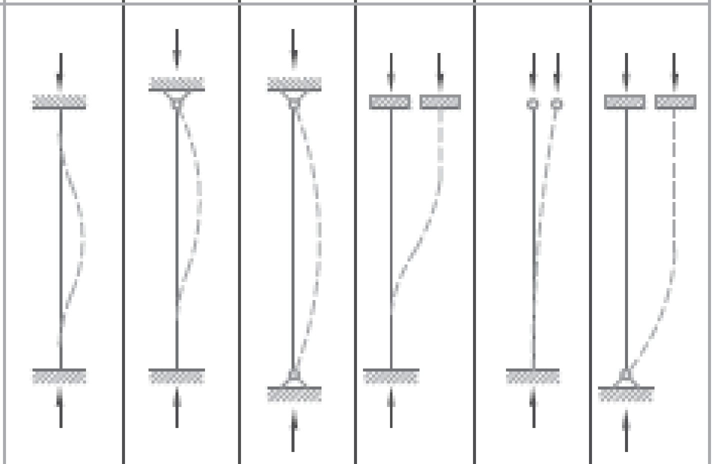
<u>**Elastic Critical Buckling Stress**</u>

Equations (5-1) and (5-2) assume that the ends of the column are pinned. For other end conditions, an adjustment or **effective length factor, *K***, is applied to the column length. The effective length of a column is defined as ***KL***, where ***K*** is usually determined by one of two methods:

1. AISCM, Table C-C2.2 — The recommended design values from this table are commonly used in design practice to determine the effective lengths of columns because the theoretical values assume idealized end support conditions.

COLUMN CRITICAL BUCKLING LOAD

AISCM C2-2 Table C-C2: Buckling Length Coefficients

	a.	b.	c.	d.	e.	f.
Buckling modes						
Theoretical K-value	0.50	0.70	1.0	1.0	2.0	2.0
Recommended design K when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.4
End-condition legend	Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed, translation free Rotation free, translation free					

Adapted from Table C-C2.2[1]

This table is especially useful for preliminary design when the size of the beams, girders, and columns are still unknown.

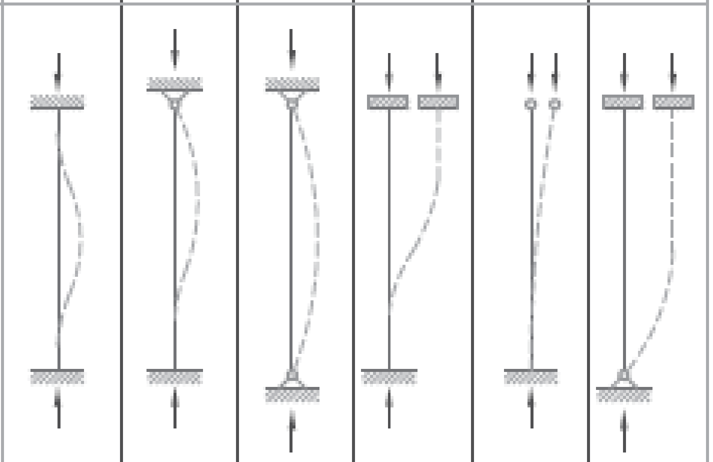
a through c represent columns in braced frames.

d through f represent columns in unbraced frames.

It should be noted that for building columns supported at the top and bottom ends,

it is common design practice to assume that the column is pinned at both ends,

AISCM C2-2 Table C-C2: Buckling Length Coefficients

	a.	*b.*	*c.*	*d.*	*e.*	*f.*
Buckling modes						
Theoretical K-value	0.50	0.70	1.0	1.0	2.0	2.0
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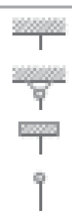
End-condition legend		Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed, translation free Rotation free, translation free				

Adapted from Table C-C2.2[1]

It should be noted that for preliminary design of building columns supported at the top and bottom ends, it is common design practice to assume that the column is pinned at both ends, resulting in a practical effective length factor, ***K***, of 1.0. For columns fixed at both ends, the recommended design value is ***K*** = 0.65.

COLUMN CRITICAL BUCKLING LOAD

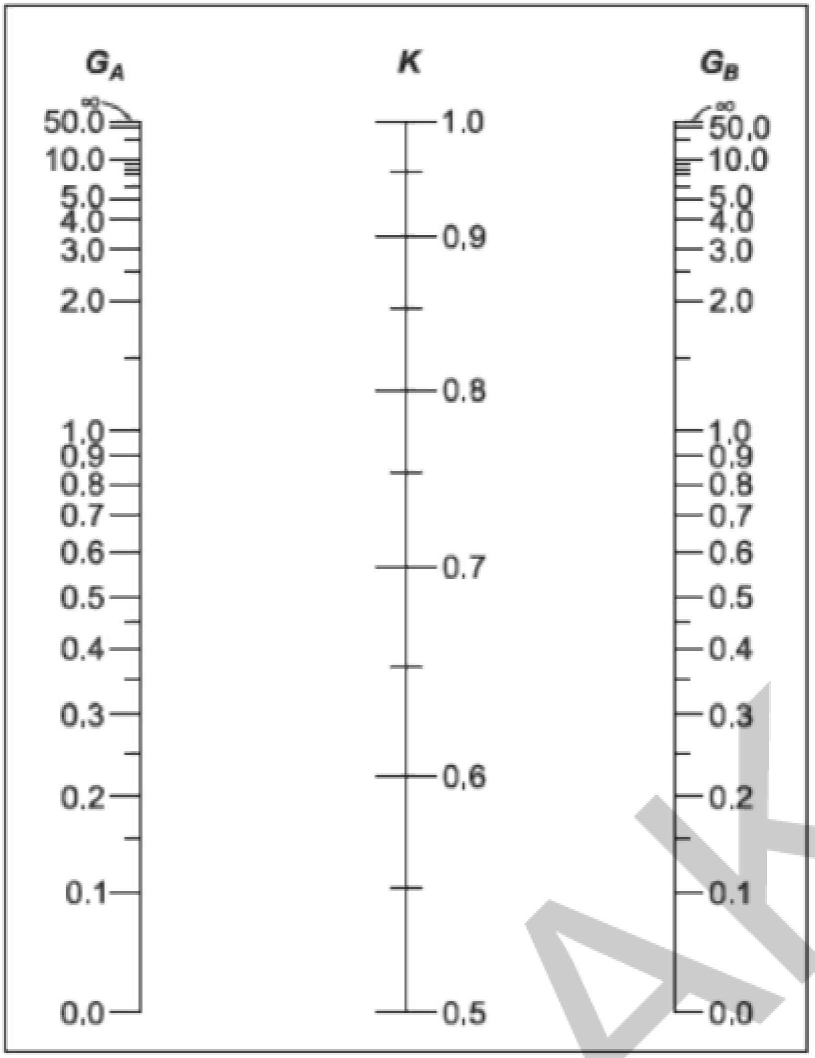
Elastic Critical Buckling Stress

Equations (5-1) and (5-2) assume that the ends of the column are pinned. For other end conditions, an adjustment or **effective length factor, *K***, is applied to the column length. The effective length of a column is defined as ***KL***, where ***K*** is usually determined by one of two methods:

2. Nomographs or alignment charts (Yönetmelik Şekil 6.1 ve 6.2) — The alignment charts use the actual restraints at the girder-to-column connections to determine the effective length factor, ***K***. They provide more accurate values for the effective length factor than AISCM, Table C-C2.2, but the process of obtaining these values is more tedious than the first method, and the alignment charts can only be used if the initial sizes of the columns and girders are known. This method will be discussed later in this section.

COLUMN CRITICAL BUCKLING LOAD

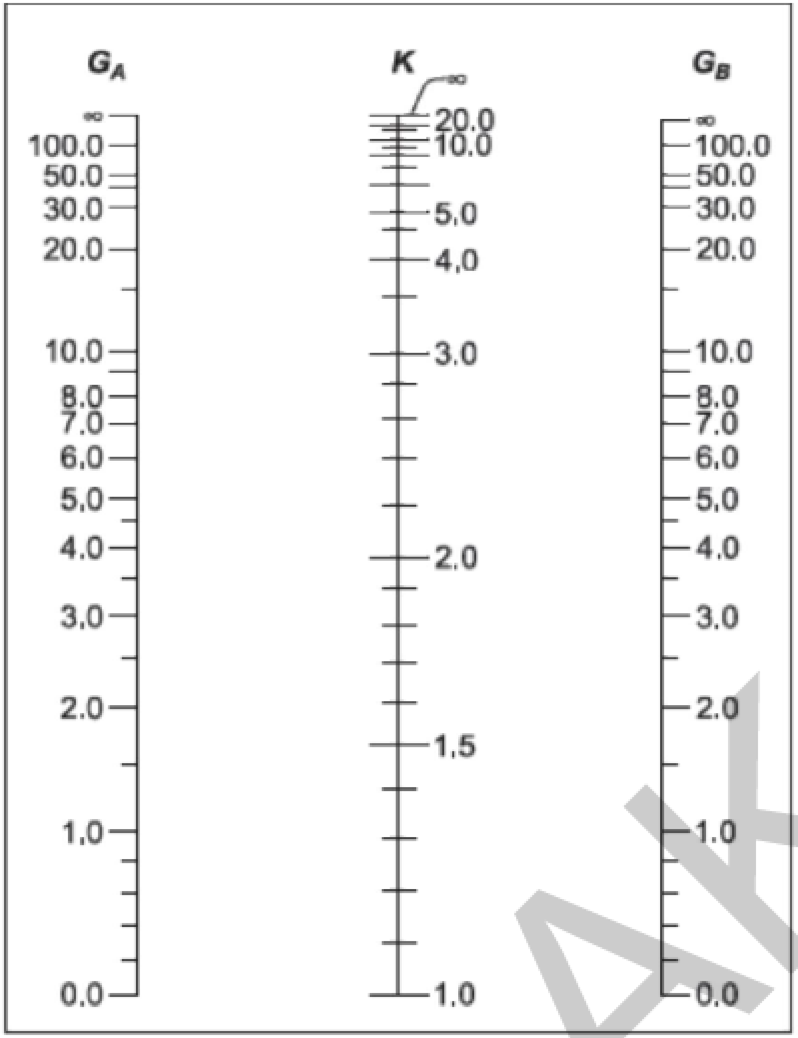
Nomograms or Alignment Charts



Şekil 6.1 – Yanal yerdeğiştirmesi önlenmiş sistemlerde burkulma boyu katsayısı, *K*

COLUMN CRITICAL BUCKLING LOAD

Nomograms or Alignment Charts



Şekil 6.2 – Yanal yerdeğiştirmesi önlenmemiş sistemlerde burkulma boyu katsayısı, *K*

COLUMN CRITICAL BUCKLING LOAD

Elastic Critical Buckling Stress

2. Nomographs or alignment charts (Yönetmelik Şekil 6.1 ve 6.2)

When the column end conditions are other than pinned, equations (5-1) and (5-2) are modified as follows:

$$P_e = \frac{\pi^2 EI}{(KL)^2} \quad (5.3)$$

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{i}\right)^2} \quad (5.4)$$

KL/i = slenderness ratio

Both the AISC and Turkish Specification recommends limiting the slenderness ratio to:

$$\frac{KL}{i} \leq 200 \quad (5.5)$$

COLUMN CRITICAL BUCKLING LOAD

Braced versus Unbraced Frames

In using the alignment charts or Table C-C2.2 of the AISCM, it is necessary to distinguish between braced and unbraced frames.

Braced frames exist in buildings where the lateral loads are resisted by diagonal bracing or shearwalls as shown in the following slides.

The beams and girders in braced frames are usually connected to the columns with simple shear connections, and thus there is very little moment restraint at these connections.

The ends of columns in braced frames are assumed to have no appreciable relative lateral sway; therefore, the term **nonsway** or **sidesway-inhibited (not-permitted)** is used to describe these frames.

COLUMN CRITICAL BUCKLING LOAD

Braced versus Unbraced Frames

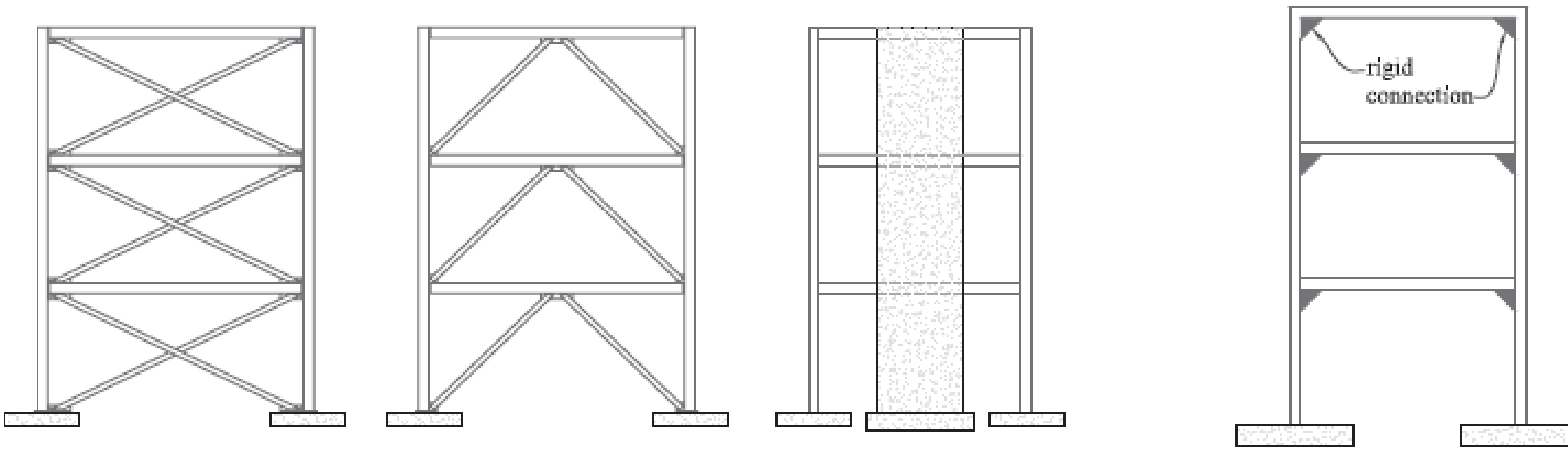
The effective length factor for columns in braced frames is taken as 1.0.

In unbraced or moment frames, the lateral loads are resisted through bending of the beams, girders, and columns, and thus the girder-to-column and beam-to-column connections are designed as moment connections.

The ends of columns in unbraced frames undergo relatively appreciable sidesway; therefore, the term **sway** or **sway-uninhibited (permitted)** is used to describe these frames. The effective length of columns in moment frames is usually greater than 1.0.

COLUMN CRITICAL BUCKLING LOAD

Braced versus Unbraced Frames



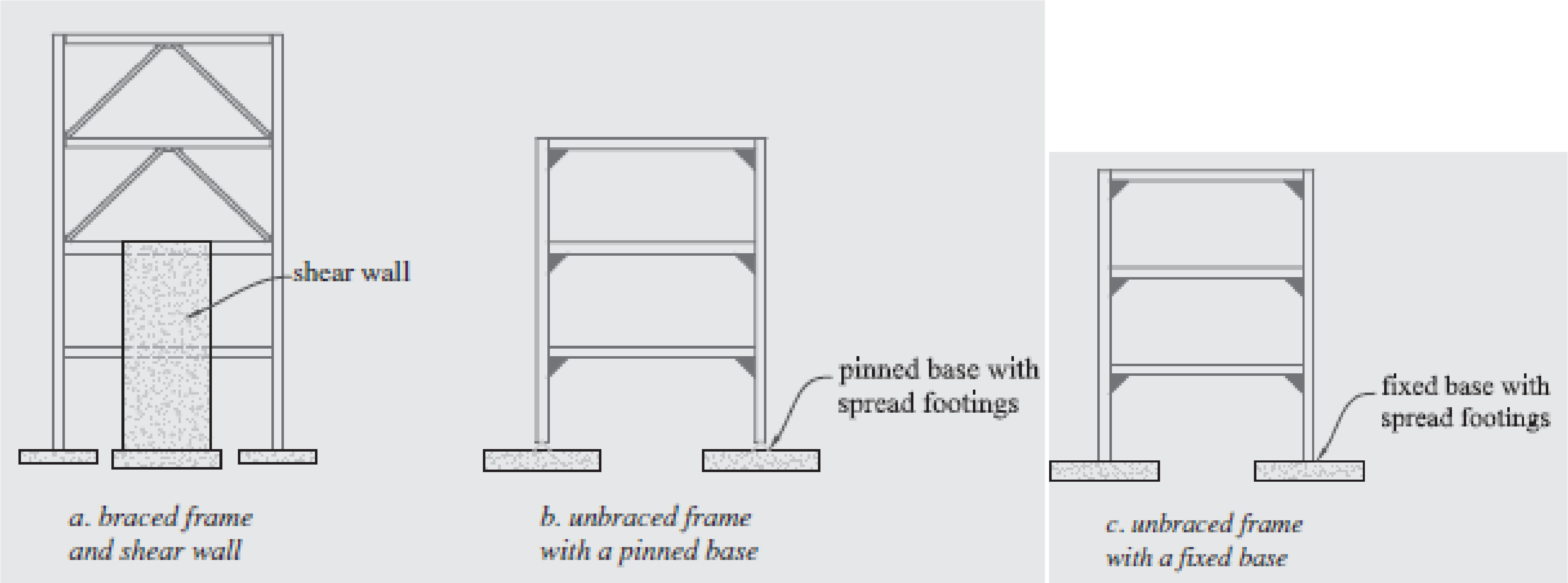
X-brace chevron or *K*-brace shear wall

a. braced frames

b. unbraced frames

Braced versus Unbraced Frames: Example 5.1

Determine the effective length factor for the ground floor columns in the following frames:



COLUMN CRITICAL BUCKLING LOAD

Braced versus Unbraced Frames: Example 5.1

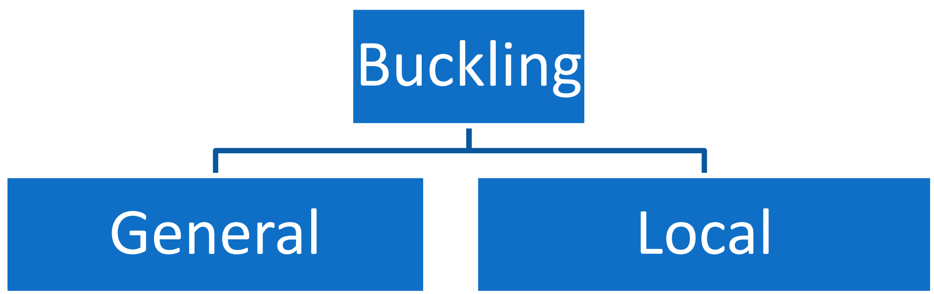
a. Braced Frame: Since the building is braced by diagonal braces and shear walls, the K-value for all columns in the building is assumed to be 1.0.

b. Unbraced Frames (Moment Frame with Pinned Column Bases): Since the bottom ends of the ground floor columns are pinned, the effective length factor, K, for each column at this level in the moment frame is 2.4.

c. Unbraced Frames (Moment Frame with Fixed Column Bases) Since the bottom ends of the ground floor columns are fixed, the effective length factor, K, for each column at this level in the moment frame is 1.2.

Design Process

Characteristic axial compressive strength, Pn, of the element under axial pressure bending buckling, torsional buckling around any of the main cross-sectional axes and/or bending torsion buckling limit conditions to be calculated according to the maximum strengths taken as the smallest.



Goal:

$$\underbrace{\phi_{cp} P_n}_{\text{Capacity}} \geq \underbrace{P_u}_{\text{Demand}}$$

FLEXURAL BUCKLING

Goal:

Capacity ≥ Demand

$$P_n = F_{cr} A_g$$

Here, the critical buckling stress, F_{cr}, will be obtained by the equations below

$$\frac{L_c}{i} \leq 4.71\sqrt{\frac{E}{F_y}} \quad \left(veya\ \frac{F_y}{F_e} \leq 2.25\right) \quad \Rightarrow \quad F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y$$

$$\frac{L_c}{i} > 4.71\sqrt{\frac{E}{F_y}} \quad \left(veya\ \frac{F_y}{F_e} > 2.25\right) \quad \Rightarrow \quad F_{cr} = 0.877 F_e$$

A_g : Area
F_e : Elastik buckling stress
F_y : Characteristic ultimate stress
F_{cr} : Critical buckling stress

The elastic buckling stress, Fe, for the bending buckling limit state around any of the cross-sectional principal axes of the element under the axial compression force, shall be determined according to the following principles.

FLEXURAL BUCKLING

The bending buckling limit state shall be taken into account in all pressure elements regardless of their cross-sectional properties. Accordingly, the elastic buckling stress, Fe, for the bending buckling limit state of the elements under the pressure force, will be calculated by the following equation

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{i}\right)^2}$$

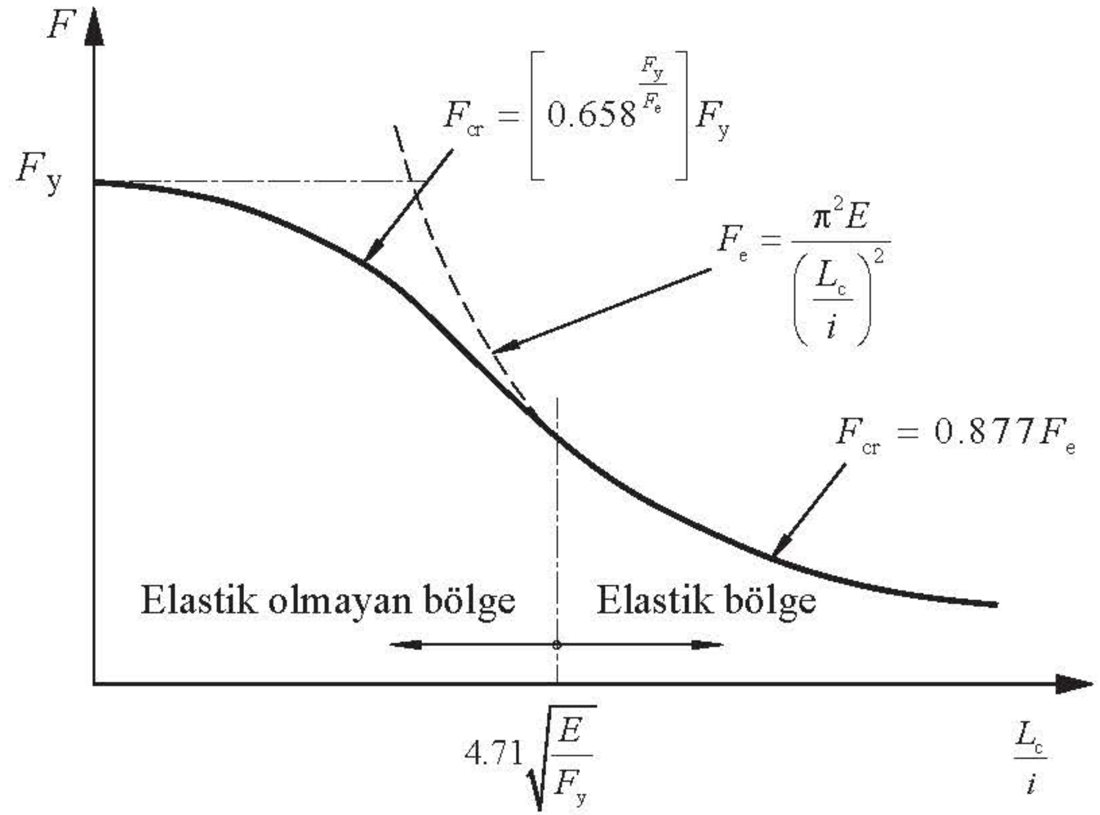


Figure 5.1 Strength curve for steel element under axial compressive force

COLUMN STRENGTH

Equation (8-2) accounts for the case where **inelastic buckling** dominates the column behavior because of the presence of residual stresses in the member.

Equation (8-3) accounts for **elastic buckling** in long or slender columns.

COLUMN STRENGTH

Failure Modes for Compression Members

There are 4 failure modes (limit states) to consider for compression members. Three of them involve global buckling and one involves local buckling.

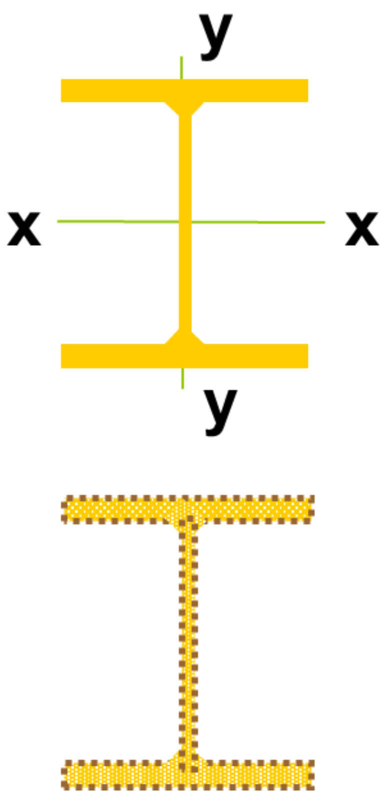
1- Flexural Buckling (Eğilmeli Burkulma)
2- Torsional Buckling (Burulmalı Burkulma)
3- Flexural-Torsional Buckling (Eğilmeli Burulmalı Burkulma)
4- Local Buckling (Yerel Burkulma)

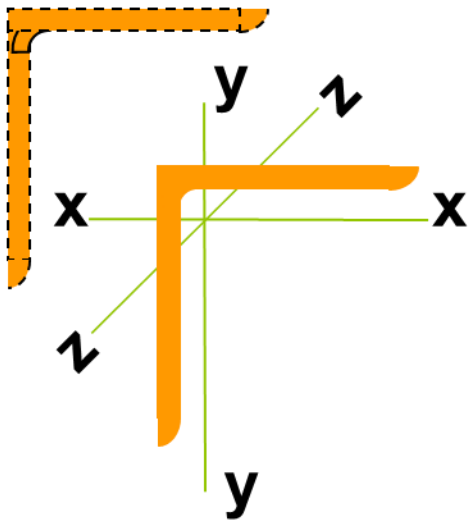
For each of the global failure modes, a separate elastic buckling stress, F_e, is calculated to determine the critical buckling stress, F_{cr}, of the compression member in question. The smallest of them all, determines the nominal capacity of the member.

COLUMN STRENGTH

Failure Modes for Compression Members

1- Flexural Buckling (Eğilmeli Burkulma)



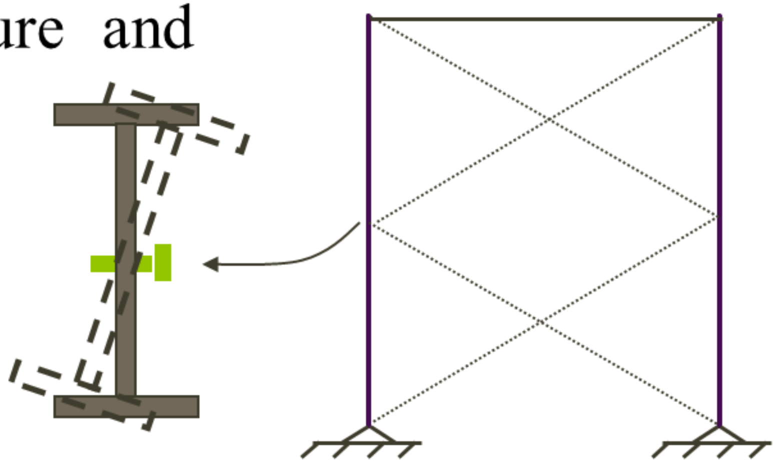


COLUMN STRENGTH

Failure Modes for Compression Members

1- Flexural Buckling (Eğilmeli Burkulma)

Usually governs doubly-symmetric shapes, provided that the unbraced lengths are the same for flexure and torsion.

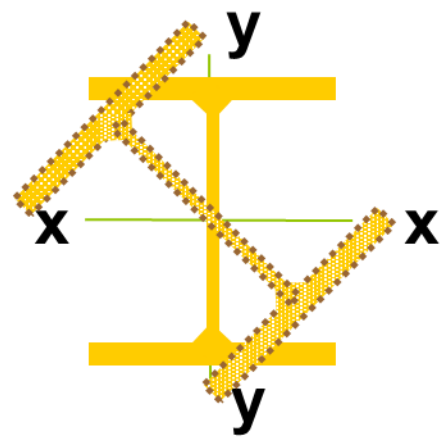


Laterally braced. However, bolts wont prevent torsion.

Failure Modes for Compression Members

2- Torsional Buckling (Burulmalı Burkulma)

Rotation around the shear center.
Generally for doubly-symmetric sections.

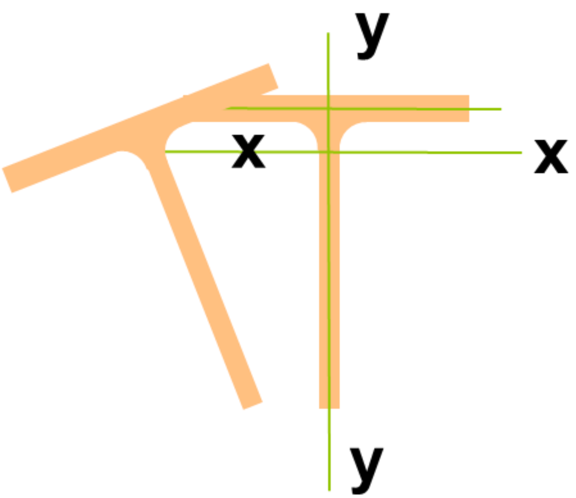


COLUMN STRENGTH

Failure Modes for Compression Members

3- Flexural Torsional Buckling (Eğilmeli Burulmalı Burkulma)

Rotation plus lateral displacement.



COLUMN STRENGTH

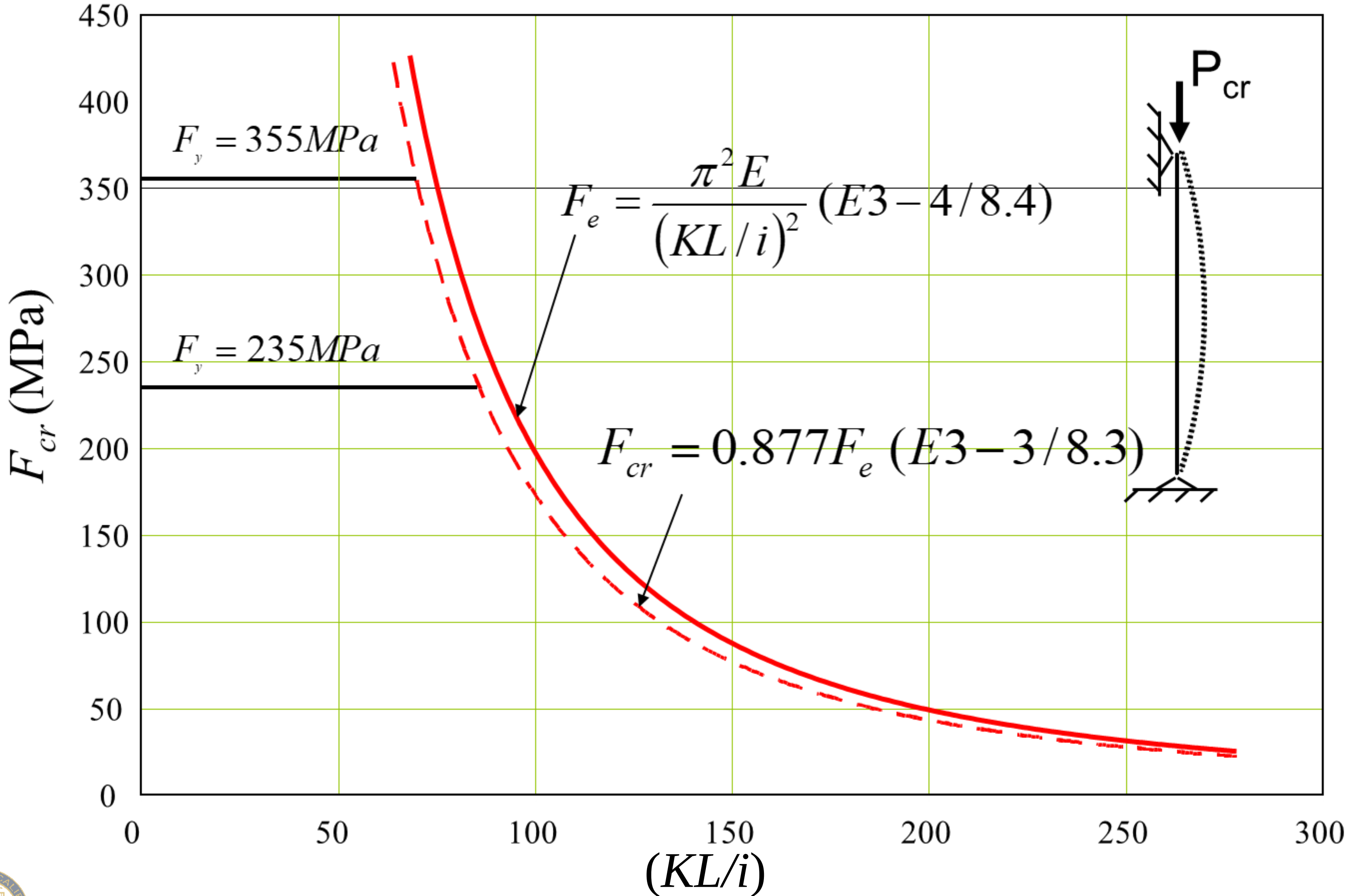
Failure Modes for Compression Members

-Flexural buckling needs to be checked in all sections.

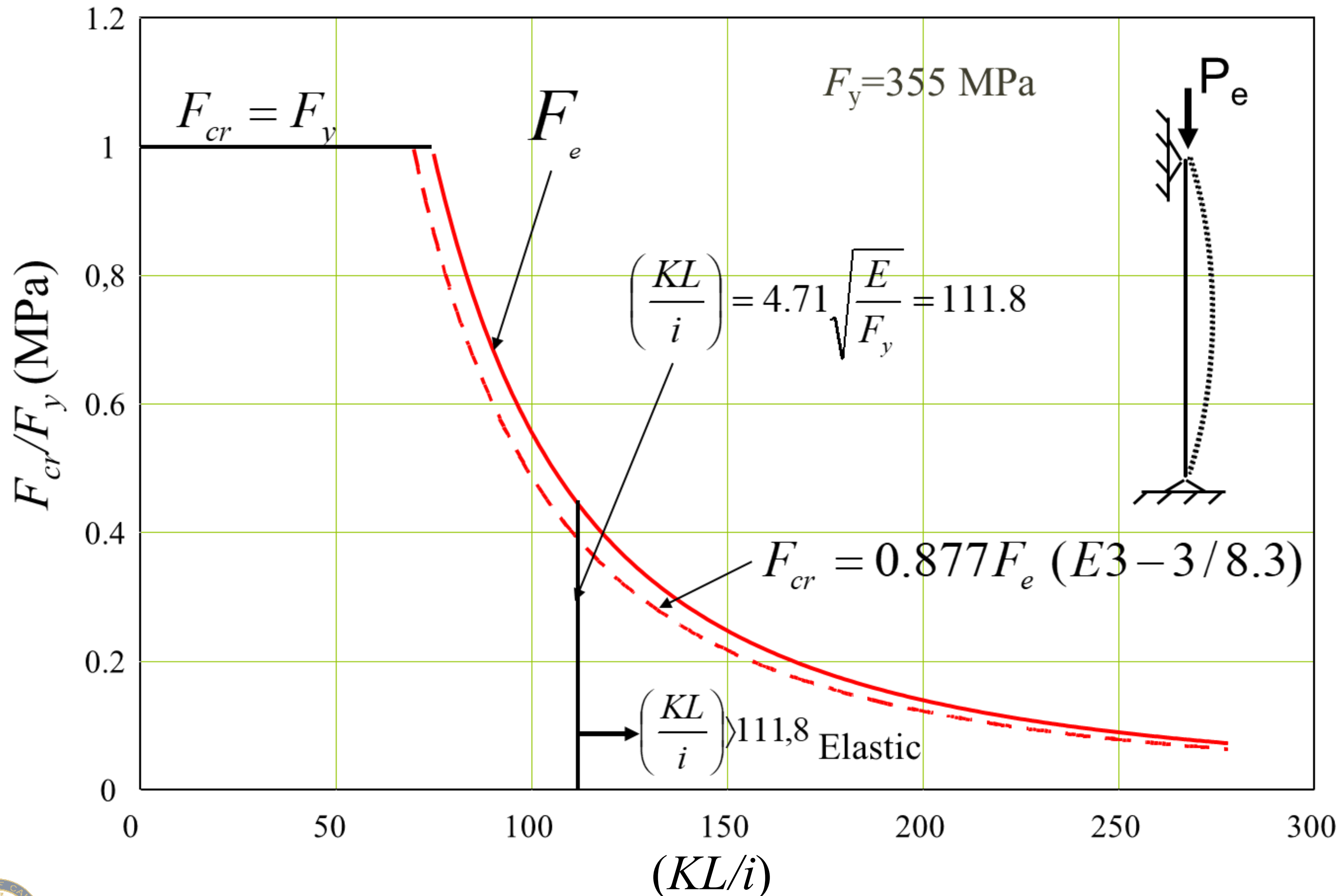
-Torsional buckling should be checked for doubly-symmetric sections.

-Flexural torsional buckling should be checked for single symmetric sections or sections with no symmetry at all.

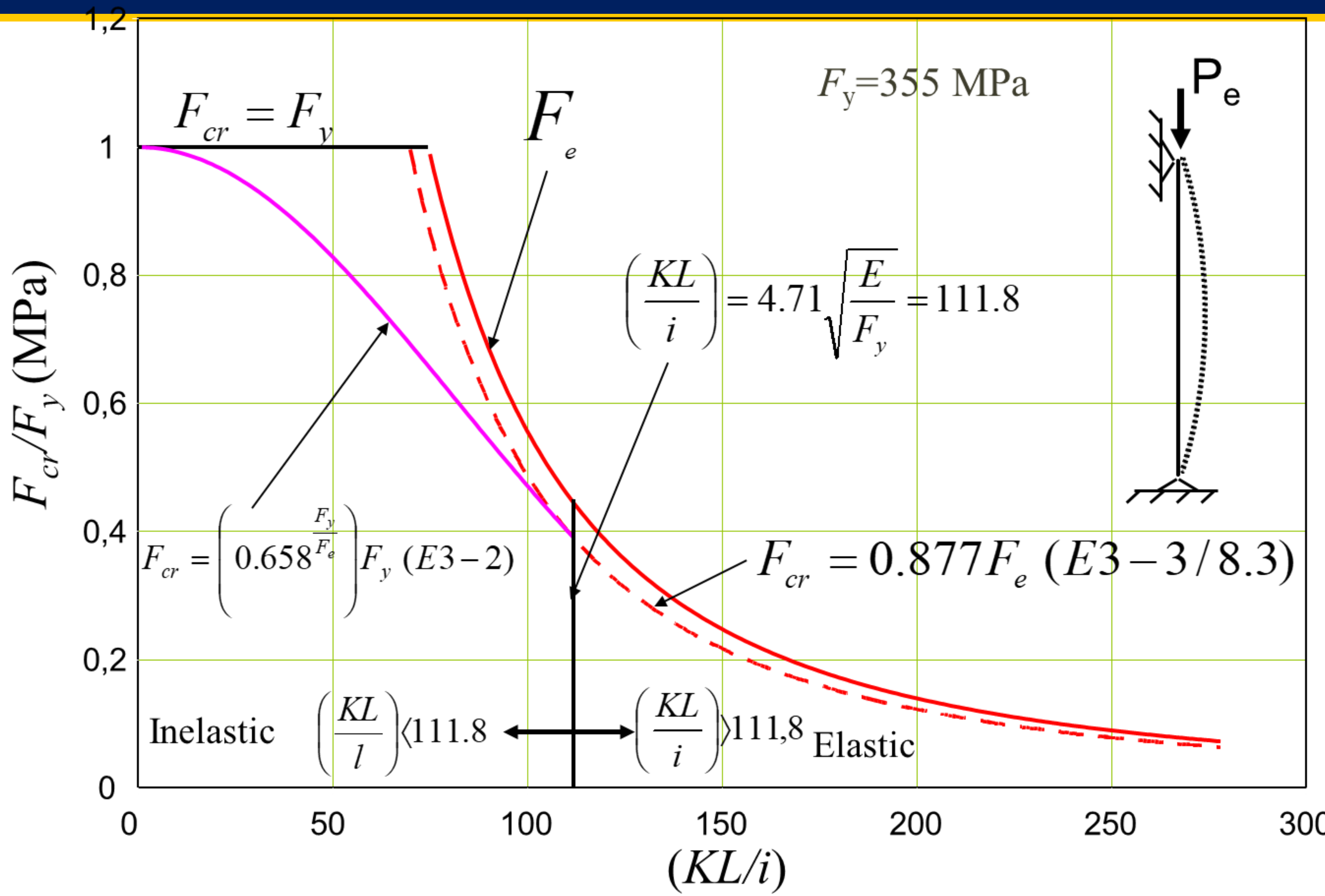
COLUMN STRENGTH



COLUMN STRENGTH



COLUMN STRENGTH



COLUMN STRENGTH

Yönetmelik 8. Bölüm

$$P_u \leq \phi P_n \qquad P_a \leq P_n / \Omega \qquad P_n = A_g F_{cr}$$

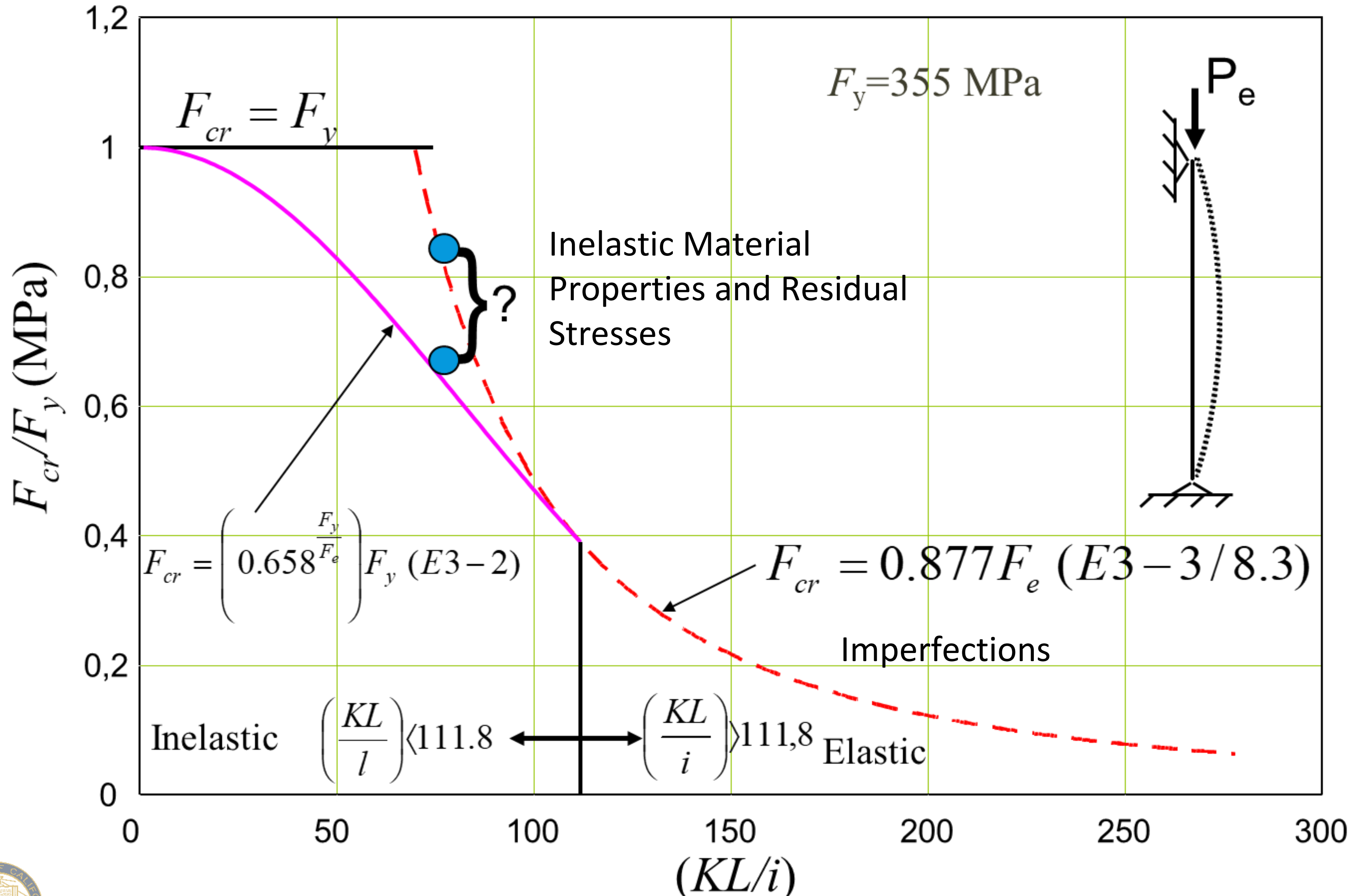
$$(a)\ \ KL/i \leq 4.71\sqrt{E/F_y} \qquad \left(or\ \frac{F_y}{F_e} \leq 2.25 \right)$$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}} \right) F_y \qquad (Yönetmelik\ 8.2)$$ (Inelastic Buckling)

$$(b)\ \ KL/i > 4.71\sqrt{E/F_y} \qquad \left(or\ \frac{F_y}{F_e} > 2.25 \right)$$ (Elastic Buckling)

$$F_{cr} = 0.877\,F_e \quad (Yönetmelik\ 8.3)$$

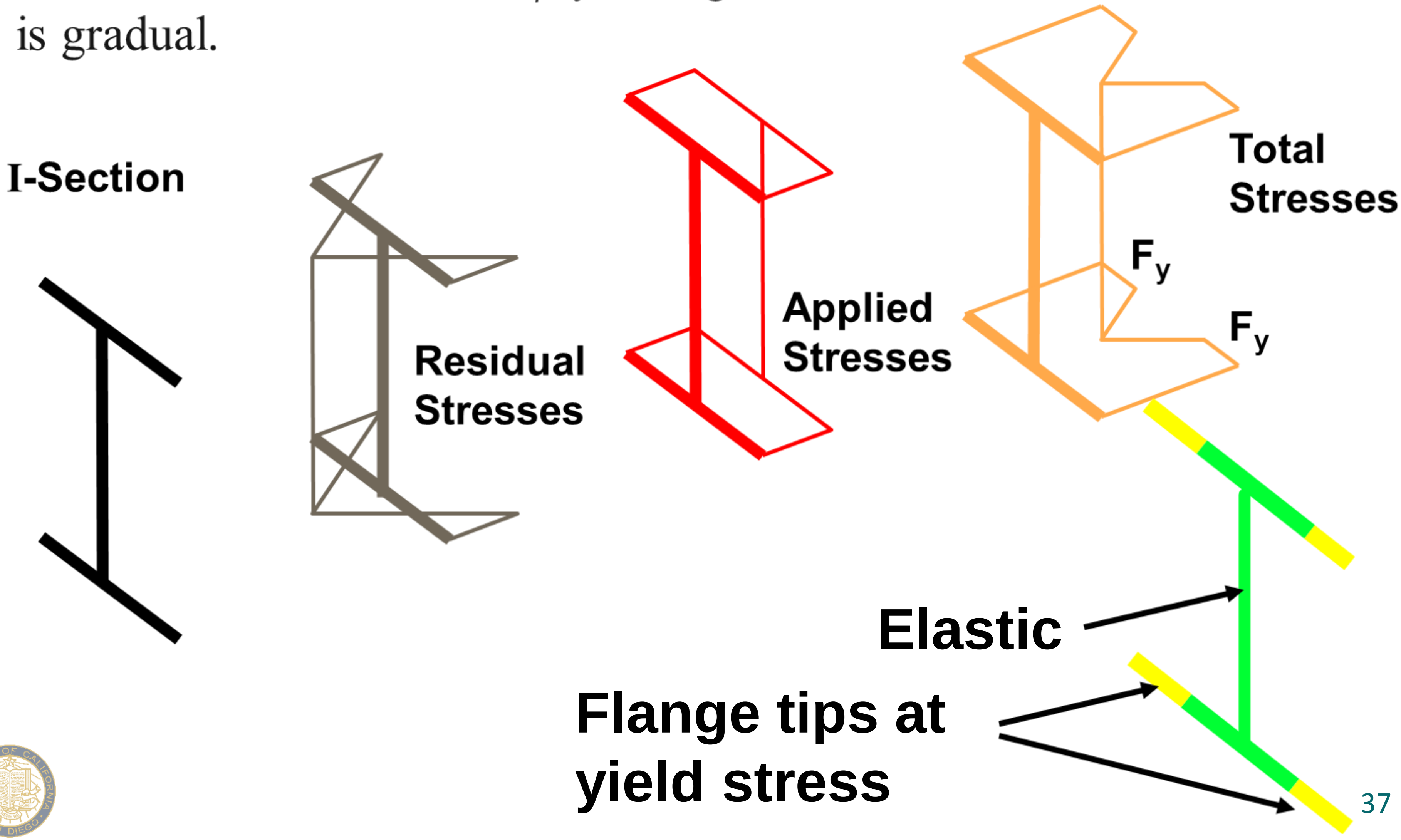
COLUMN STRENGTH



COLUMN STRENGTH

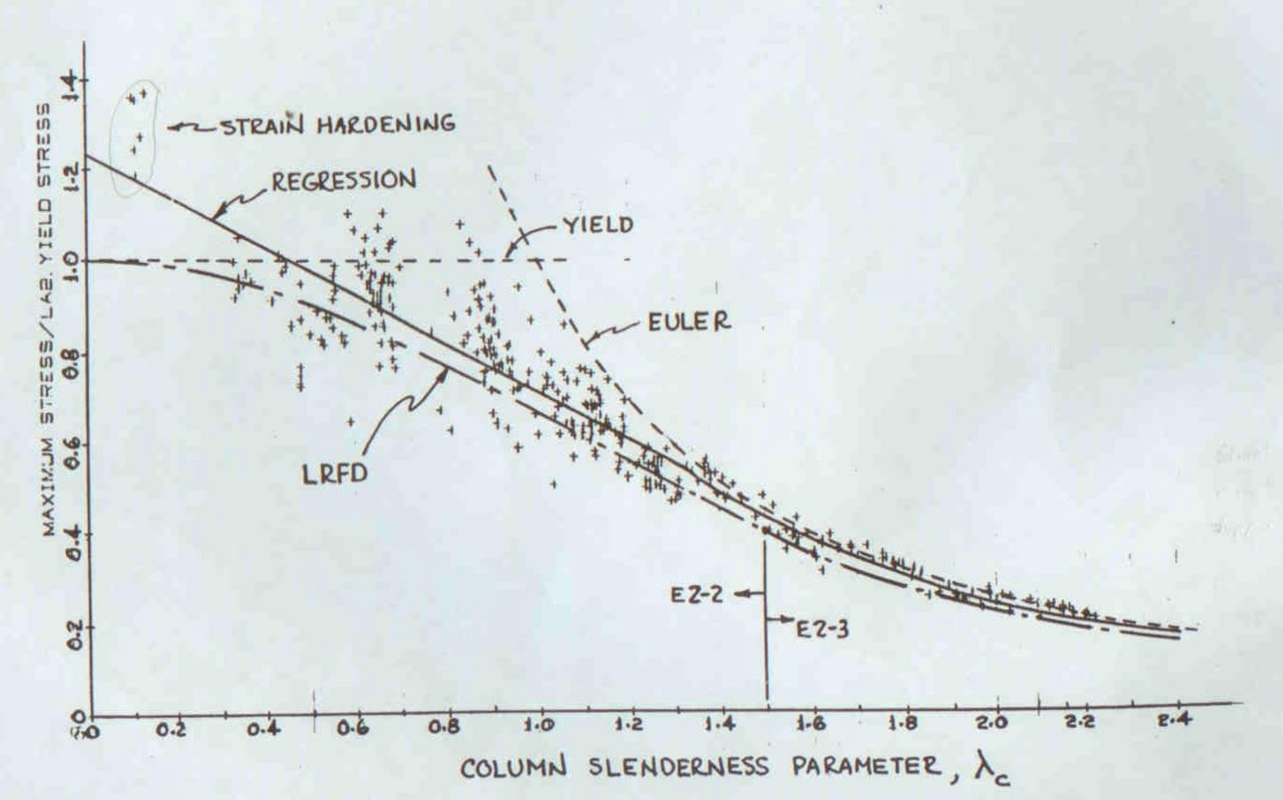
Inelastic Buckling for Compression Buckling

Due to residual stresses, F_r, yielding of a section is gradual.



COLUMN STRENGTH

Data from 389 tests are shown below, along with the Euler stress, a statistically developed mean curve through the data given by the solid line, and the LRFD column curve which is more of a lower bound to the data.

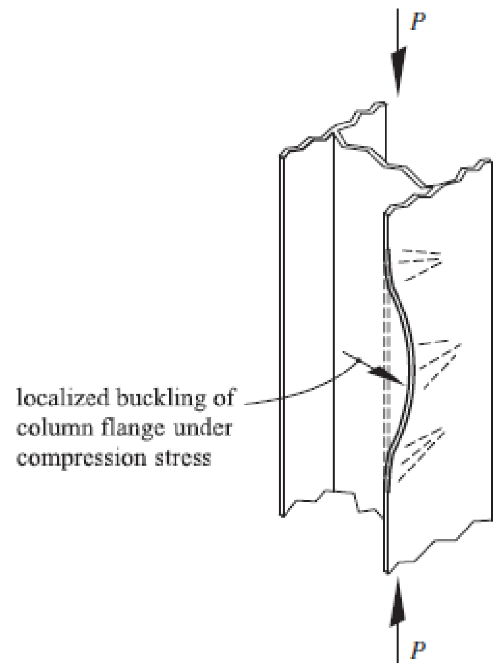


LOCAL STABILITY OF COLUMNS

Local Buckling

The preceding section was based on the global strength and buckling of the column member as a whole. In this section, we will look at the local stability of the individual elements that make up the column section.

Local buckling leads to a reduction in the strength of a compression member and prevents the member from reaching its overall compression capacity.



LOCAL STABILITY OF COLUMNS

Local Buckling

To avoid or prevent local buckling, the AISC and Turkish specifications prescribes limits to the width-to-thickness ratios (slenderness ratio) 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.

Noncompact 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.

LOCAL STABILITY OF COLUMNS

Local Buckling

The use of slender sections as compression members is not efficient or economical.

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

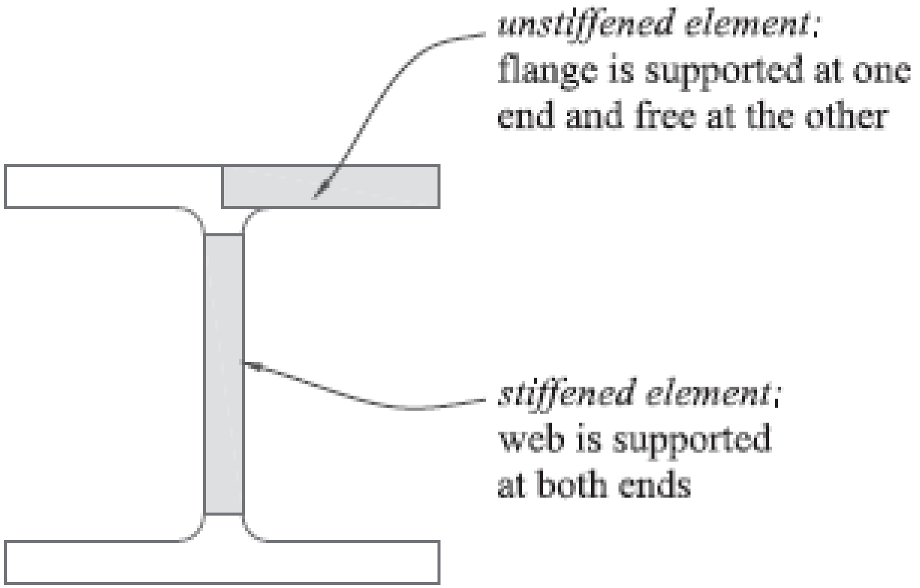
Stiffened elements are supported along both edges parallel to the applied axial load. An example of this is the web of an I-shaped column where the flanges are connected on either end of the web. An **unstiffened element** has only one unsupported edge parallel to the axial load—for example, the outstanding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

LOCAL STABILITY OF COLUMNS

Local Buckling

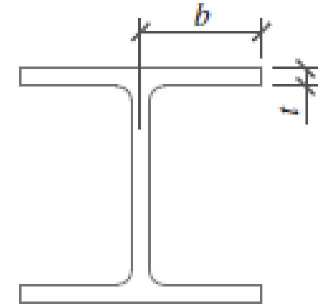
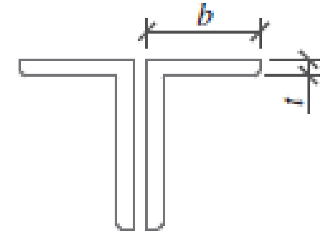
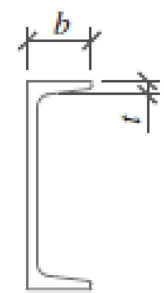
The use of slender sections as compression members is not efficient or economical.

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

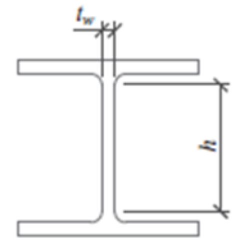
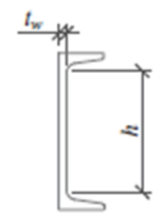
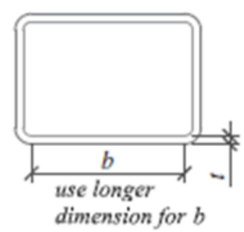
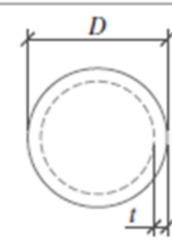


shapes, such an element is said to be slender and a reduction is applied to the gross area, A_g, in equation (5-6). For elements that are compact or noncompact, equation (5-6) can be used directly.

Yönetmelik Table 5-1A

	Description	Limiting Width-Thickness Ratio λ_p (compact)	Limiting Width-Thickness Ratio λ_r (noncompact)	Details
	Flanges of I-shaped sections			
Unstiffened	Outstanding legs of double angles in continuous contact	N/A	$\frac{b}{t} \leq 0.56\sqrt{\frac{E}{F_y}}$	
	Flanges of C-shapes			

Yönetmelik Table 5-1A

	Description	Limiting Width–Thickness Ratio: λ_p (compact)	Limiting Width–Thickness Ratio: λ_r (noncompact)	Details
Stiffened	Webs of I-shaped sections	N/A	$\frac{h}{t_w} \leq 1.49\sqrt{\frac{E}{F_y}}$	
Stiffened	Webs of C-shapes	N/A	$\frac{h}{t_w} \leq 1.49\sqrt{\frac{E}{F_y}}$	
Stiffened	Square or rectangular HSS	$\frac{b}{t} \leq 1.12\sqrt{\frac{E}{F_y}}$	$\frac{b}{t} \leq 1.40\sqrt{\frac{E}{F_y}}$	
Stiffened	Round HSS or pipes	N/A	$\frac{D}{t} \leq 0.11\left(\frac{E}{F_y}\right)$	

Note: N/A = not applicable.

Local Buckling

For column shapes with slender elements, the following reduction factors apply to the gross area, A_g:

$$P_n = F_{cr} A_e \qquad (8.23)$$

P_n = nominal axial compressive strength

F_{cr} = critical buckling stress

A_e = effective area = $(b\text{-}b_e)t$

8.5.1 – Effective area for slender members **(**Narin Enkesit Parçalarında Etkin Alan

$$A_e = \left(b - b_e\right)t$$

LOCAL STABILITY OF COLUMNS

$$\lambda \le \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \Rightarrow b_e = b$$

$$\lambda \rangle \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \Rightarrow b_e = b\left(1 - c_1 \sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad F_{el} = \left(c_2 \frac{\lambda_r}{\lambda}\right) F_y$$

b = width of the element (for tees this is d; for webs this is h) (mm)

c_1, c_2 = effective width imperfection adjustment factor determined from **Table 8.2**

λ = width-to-thickness ratio for the element as defined in Section 5.4.1

λ_r = limiting width-to-thickness ratio as defined in **Table 5-1A**

F_{el} = elastic local buckling stress determined according to Eq. 8.25

LOCAL STABILITY OF COLUMNS

TABLO 8.2 NARİN ENKESİT PARÇALARINDA c_1 ve c_2 KATSAYILARI

Narin enkesit parçası	c_1	c_2
Rijitleştirilmiş enkesit parçaları (Kutu enkesitlerin cidarları hariç)	0.18	1.31
Kutu enkesitlerin cidarları	0.20	1.38
Diğer tüm elemanlar	0.22	1.49

Most wide flange shapes do not have slender elements; therefore, no reduction is necessary for such sections. There are, in fact, very few sections listed in the AISCM and Archelor Table that have slender elements and these are usually indicated by a footnote. However, some double-angle shapes, and WT-shapes are made up of slender elements.

LOCAL STABILITY OF COLUMNS

Analysis Methods

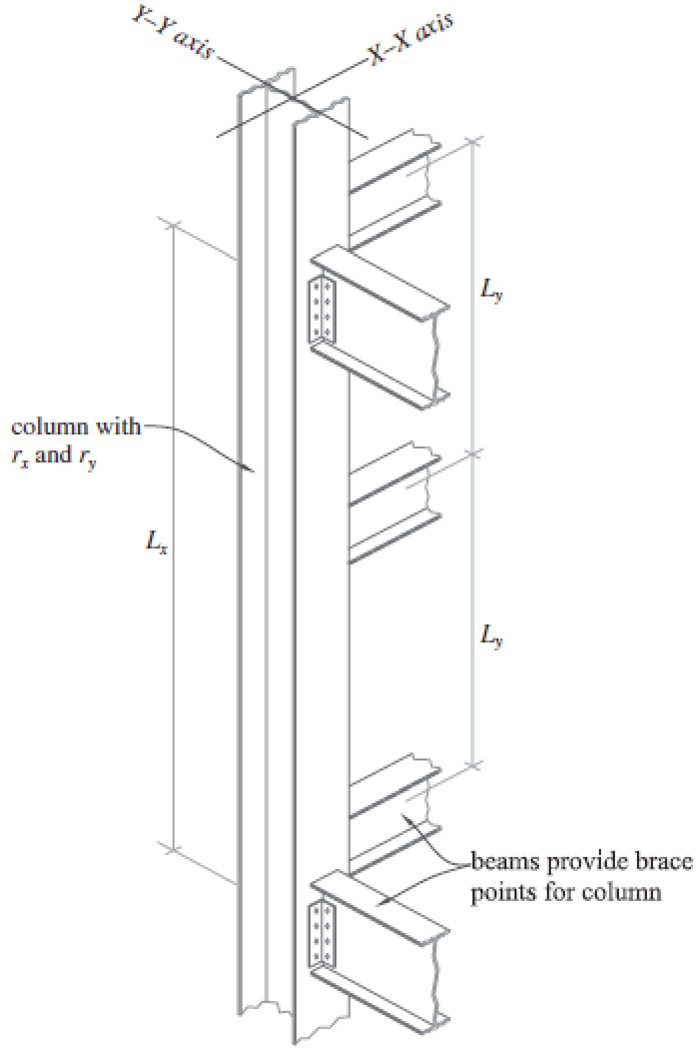
It is sometimes necessary to determine the strength of an existing structural member for which the size is known; this process is called analysis, as opposed to design, where the size of the member is unknown and has to be determined.

There are several methods available for the analysis of compression members:

1- Determine the effective length, ***KL***, and the slenderness ratio, ***KL/i***, for each axis of the column. For many shapes, both ***KL*** and ***i*** are different for each axis.

Method 1: Use equations (8-1) through (8-4), using the larger of $(KL/i)_x$ and $(KL/i)_y$.

Effective Length and Slenderness Ratio



ANALYSIS PROCEDURE FOR COMP. MEMBERS

Analysis Methods

Method 2: Use the AISCM Critical Stress Tables

This table gives the critical buckling stress, ϕF_{cr}, as a function of (KL/i) for various values of F_y.

For a given (KL/i), determine F_{cr} from the table using the larger of $(KL/i)_x$ and $(KL/i)_y$.

Knowing the critical buckling stress, the axial design capacity can be calculated from the equation:

where A_g is the gross cross-sectional area of the compression member.

Analysis Methods

Method 3: Use the AISCM Compression Strength Tables.

These tables give the design strength, $\phi_c P_n$ Pn, of selected shapes for various effective lengths, ***KL***, and for selected values of F_y. Go to the appropriate table with ***KL***, using the larger of:

$$\frac{K_x L_x}{\left(\dfrac{i_x}{i_y}\right)}$$ and $K_y L_y$.

ANALYSIS PROCEDURE FOR COMP. MEMBERS

Analysis Methods

Notes:

1- Ensure that the slenderness ratio for the member is not greater than 200, that is:

$$\frac{KL}{i} \leq 200$$

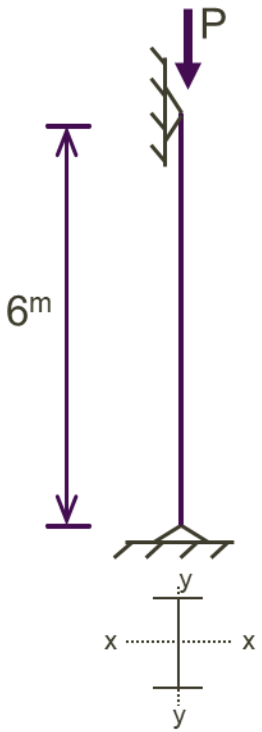
2- Check that local buckling will not occur, and if local buckling limits are not satisfied, modify the critical buckling stress, F_{cr}, using equations (8-23) through (8-25).

3- Use column load tables whenever possible.

4- Equations (8-1) through (8-4) can be used in all cases for column shapes that have no slender elements.

Example 5.1

Calculate the design compressive strength of a HE 320A column, 6 m long, and pinned at both ends. Use EN10025-2 S275 grade steel.



HE 320A
i_x=135.8 mm
i_y=74.9 mm
I_x=22930 × 10^4 mm^4
I_y=6985 × 10^4 mm^4

Weak axis controls since $(KL)_x = (KL)_y$

$$\frac{KL}{i_y} = \frac{1.0x6000^{mm}}{74.9^{mm}} = 80.1 \leq 200$$

$$\frac{KL}{i_y} \langle 4.71\sqrt{\frac{E}{F_y}} = 127.0$$

Check the slenderness criteria for compression elements:
b_f = 300 mm
t_f = 15.5 mm
t_w = 9 mm
h = 225 mm

ANALYSIS PROCEDURE FOR COMP. MEMBERS

$$\frac{b}{t} \leq 0.56\sqrt{\frac{E}{F_y}}; \quad \frac{300^{mm}/2}{15.5^{mm}} = 9.68 < 0.56\sqrt{\frac{200000^{MPa}}{275^{MPa}}} = 15.1\, OK$$

$$\frac{h}{t_w} \leq 1.49\sqrt{\frac{E}{F_y}}; \quad \frac{225^{mm}}{9^{mm}} = 25 < 1.49\sqrt{\frac{200000^{MPa}}{275^{MPa}}} = 40.2\, OK$$

Determine the flexural buckling stress, F_{cr}:

$$\frac{KL}{i_y} = \frac{1.0x6000^{mm}}{74.9^{mm}} = 80.1 \qquad \frac{KL}{i_y} \langle 4.71\sqrt{\frac{E}{F_y}} = 127.0 \leq 200$$

$$F_e = \frac{\pi^2 E}{(kL/i)^2} = \frac{3.14^2 x200000^{N/mm^2}}{(80.1)^2} = 307.7\, MPa$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\frac{275^{MPa}}{307.7^{MPa}}}\right] 275^{MPa} = 189.2\, MPa$$

ANALYSIS PROCEDURE FOR COMP. MEMBERS

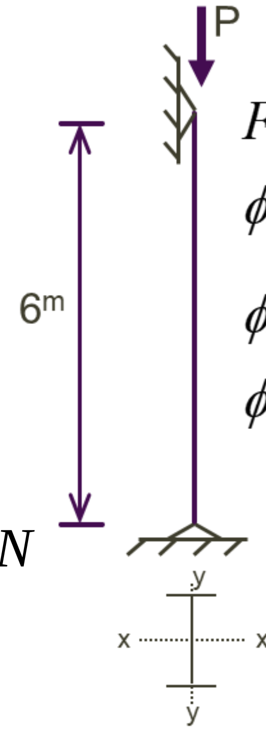
Determine the flexural buckling stress, F_{cr}:

$$F_{cr} = 189.2\ MPa$$

$$\phi_c F_{cr} = 0.90 x 189.2^{MPa} = 170.3 MPa$$

$$\phi_c P_n = \phi_c F_{cr} A_g$$

$$\phi_c P_n = 0.90 x 189.2^{MPa} x 12440^{mm^2} = 2118.3 kN$$



$$F_{cr} = 189.2\ MPa$$

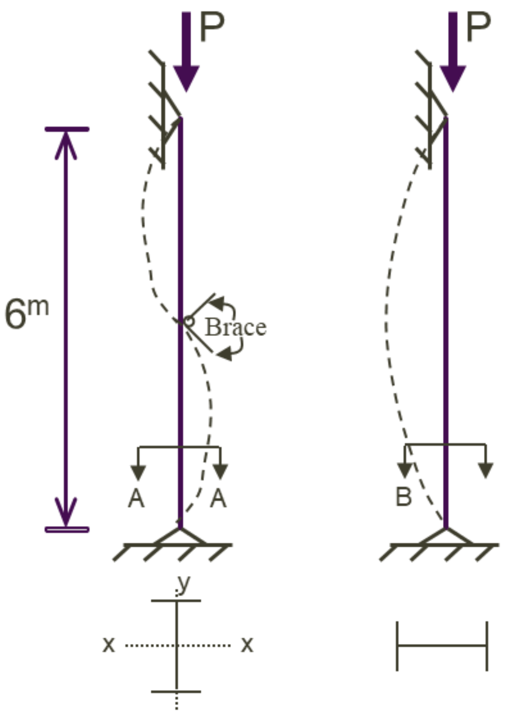
$$\phi P_n = \phi A_g F_{cr}$$

$$\phi P_n = 0.90 x 12440^{mm^2} x 189.2^{MPa}$$

$$\phi P_n = 2118.3\ kN$$

Example 5.2

Calculate the design compressive strength of a HE 320A column, 6 m long, pinned at both ends and braced at midheight in the weak axis. Use EN10025-2 S275 grade steel.



A-A Kesiti
Zayıf Eksen

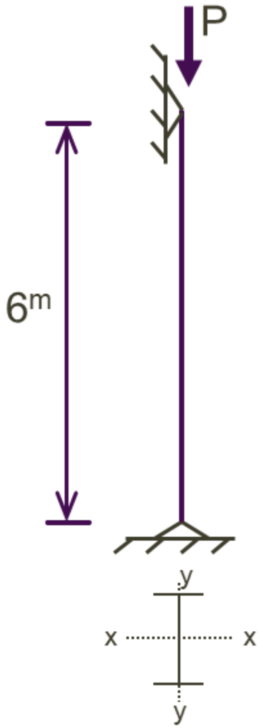
B-B Kesiti
Kuvvetli Eksen

$$\frac{KL}{i_y} = \frac{1.0x3000^{mm}}{74.9^{mm}} = 40.1$$

$$\frac{KL}{i_x} = \frac{1.0x6000^{mm}}{135.8^{mm}} = 44.2 \leq 200$$ Strong axis controls.

$$\frac{KL}{i_x} = 44.2 \langle 4.71\sqrt{\frac{E}{F_y}} = 127.0$$ Inelastic buckling.

ANALYSIS PROCEDURE FOR COMP. MEMBERS



HE 320A
i_x=135.8 mm
i_y=74.9 mm
I_x=22930 × 10^4 mm^4
I_y=6985 × 10^4 mm^4

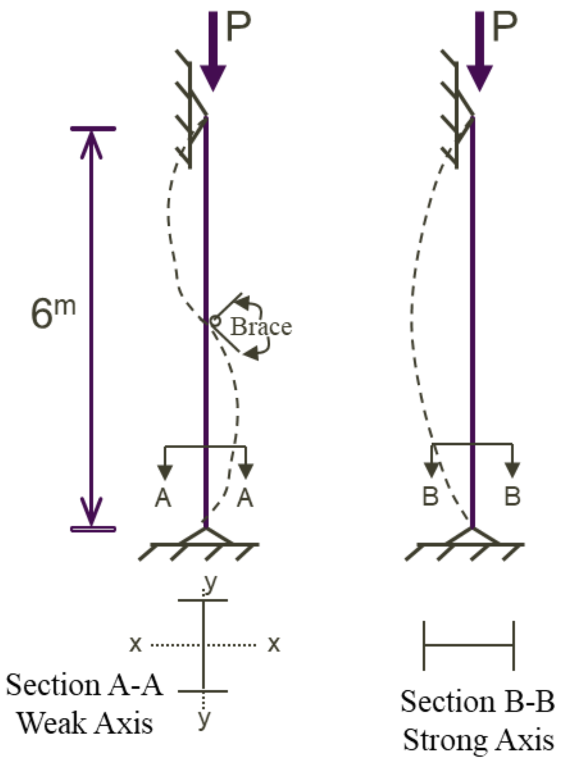
Check the slenderness criteria for compression elements:
b_f = 300 mm
t_f = 15.5 mm
t_w = 9 mm
h = 225 mm

$$\frac{b}{t} \leq 0.56\sqrt{\frac{E}{F_y}}; \quad \frac{300^{mm}/2}{15.5^{mm}} = 9.68 < 0.56\sqrt{\frac{200000^{MPa}}{275^{MPa}}} = 15.1\,OK$$

$$\frac{h}{t_w} \leq 1.49\sqrt{\frac{E}{F_y}}; \quad \frac{225^{mm}}{9^{mm}} = 25 < 1.49\sqrt{\frac{200000^{MPa}}{275^{MPa}}} = 40.2\;OK$$

No need to reduce the strength due to local buckling.

ANALYSIS PROCEDURE FOR COMP. MEMBERS



$$\frac{KL}{i_x} = 44.2 \langle 4.71\sqrt{\frac{E}{F_y}} = 127.0$$ Inelastic buckling.

$$F_e = \frac{\pi^2 E}{(kL/i)_x^{\ 2}} = \frac{3.14^2\, x200000^{N/mm^2}}{(44.2)^2} = 1010.4MPa$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = \left[0.658^{\frac{275^{MPa}}{1010.4^{MPa}}}\right] 275^{MPa} = 245.4MPa$$

$$\phi P_n = \phi F_{cr} A_g = 0.9x245.4^{MPa}\, x12440^{mm^2} = 2747.5\ kN$$

Tablo: 6-1 How to use Column Load Tables

Column Load Tables are prepared for weak axis KL/i_y. For cases where strong axis comtrols the design, an equivalent KL_{eq} needs to be utilized.

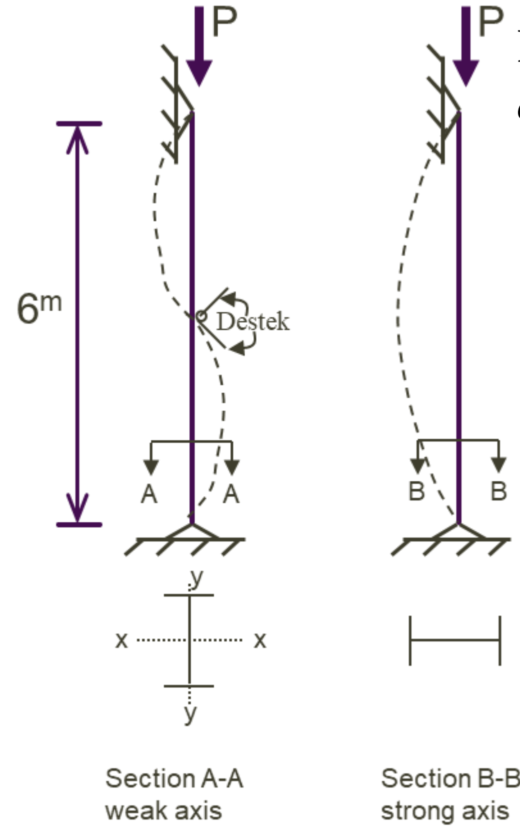
ANALYSIS PROCEDURE FOR COMP. MEMBERS

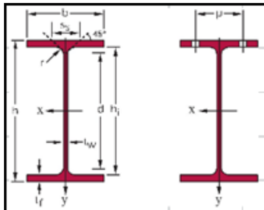
$$\frac{KL}{i_x} = \frac{KL_{eş}}{i_y} \Rightarrow KL_{eq} = \frac{KL}{i_x} i_y = \frac{KL}{i_x / i_y}$$

$$KL_{eq} = \frac{KL}{i_x} i_y = 44.2x74.9^{mm} = 3310\ mm$$

Tablo 6-1
For KL = 3.3 m, ϕP_n = 2758.8 kN
Calculated: ϕP_n = 2747.5 kN

Tablo 6-1
For KL = 3.3
ϕP_n = 2758.8 kN





F_y=275 MPa

Tablo 6-1
İZİN VERİLEBİLİR
EKSENEL YÜK (kN)
HE Kesitleri

Kesit	HE 320					
	A		B		M	
Tasarım	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
0.0	2048.5	3078.9	2656.1	3992.2	5137.7	7722.0
2.3	1943.5	2921.1	2522.8	3791.7	4903.3	7369.6
2.5	1919.7	2885.2	2492.5	3746.2	4849.8	7289.3
2.8	1893.6	2846.1	2459.4	3696.5	4791.5	7201.6
3.0	1865.6	2803.9	2423.7	3642.8	4728.3	7106.7
3.3	1835.5	2758.8	2385.5	3585.4	4660.7	7005.0
3.5	1803.6	2710.8	2344.9	3524.3	4588.7	6896.8
3.8	1769.9	2660.2	2302.0	3459.9	4512.6	6782.4
4.0	1734.6	2607.2	2257.1	3392.4	4432.6	6662.2
4.3	1697.9	2551.9	2210.2	3321.9	4349.1	6536.7
4.5	1659.7	2494.5	2161.5	3248.8	4262.2	6406.1
4.8	1620.3	2435.3	2111.3	3173.2	4172.2	6270.8

...uk efektif uzunluk KL (m)

DESIGN PROCEDURE FOR COMP. MEMBERS

In designing columns using the column load tables use the following steps

a. Calculate P_u (i.e., the factored load on the column).

b. Obtain the recommended effective length factor, K, and calculate the

effective length, KL, for each axis.

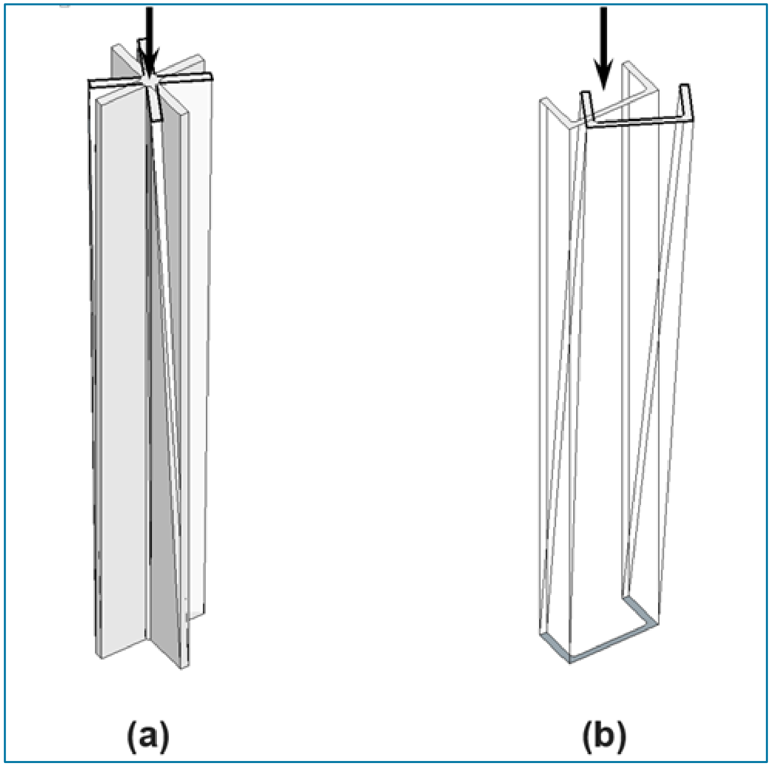
c. Enter the column load tables with a KL value that is the larger of $(K_x L_x)/(i_x/i_y)$ and $K_y L_y$, and move horizontally until the lightest column section is found with a design strength, $\phi P_n >$ the factored load, P_u.

d. Check the strength of the column by hand.

It is recommended to use the column load tables whenever possible because they are the easiest to use.

FLEXURAL TORSIONAL BUCKLING

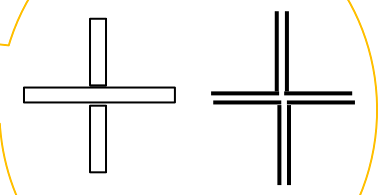
It is seen in double symmetry members such as double angle and "T", double symmetry and false sections like "+" and some pressure bars with cross section without symmetry axis.



FLEXURAL TORSIONAL BUCKLING

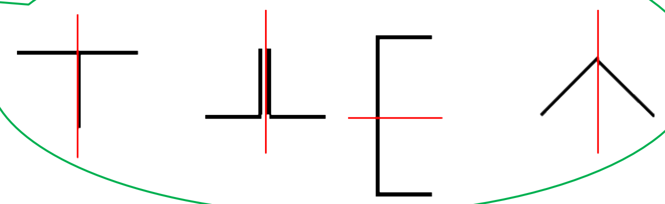
For doubly symmetric sections, elastic stress F_e ;

$$F_e = \left(\frac{\pi E C_w}{(K_z L)^2} + GJ \right) \frac{1}{I_x + I_y} \quad \text{(Eqn. 8.5)}$$



For singly symmetric sections, elastic stress F_e ;

$$F_e = \frac{F_{ey} + F_{ez}}{2H} \left(1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{(F_{ey} + F_{ez})^2}} \right) \quad \text{(Eqn. 8.6)}$$



FLEXURAL TORSIONAL BUCKLING

For cross sections without an axis of symmetry, Fe is the small root of the equation below.

$$\left(F_e - F_{ex}\right)\left(F_e - F_{ey}\right)\left(F_e - F_{ez}\right) - F_e^{2}\left(F_e - F_{ey}\right)\left(\frac{x_0}{\bar{i}_0}\right)^2 - F_e^{2}\left(F_e - F_{ex}\right)\left(\frac{y_0}{\bar{i}_0}\right)^2 = 0 \quad \text{(Eqn. 8.7)}$$

In this eqn.;

C_w : Warping Constant

K_z : Effective length factor in z-direction

G : Shear Modulus (7720 kN/ cm2)

J : Torsional Constant (approximately $J = \frac{1}{3}\sum b_i t_i^3$

FLEXURAL TORSIONAL BUCKLING

The elastic buckling stress (Fe) in bending buckling for both axes is as follows;

$$F_{ex} = \frac{\pi^2 E}{\left(L_{cx} / i_x\right)^2} \quad \text{(Denk. 8.8)} \qquad F_{ey} = \frac{\pi^2 E}{\left(L_{cy} / i_y\right)^2} \quad \text{(Eqn. 8.9)}$$

Here "y" is the symmetry axis in single symmetrical sections.

Elastic buckling stress in torsional stress;

$$F_{ez} = \left(\frac{\pi^2 E C_w}{\left(L_{cz}\right)^2} + GJ\right)\frac{1}{A_g \bar{i}_0^{\,2}} \quad \text{(Eqn. 8.10)}$$

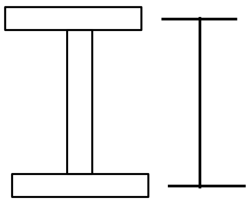
Bending Constant $H = 1 - \frac{x_0^2 + y_0^2}{\bar{i}_0^{\,2}}$ (Eqn. 8.11)

FLEXURAL TORSIONAL BUCKLING

Here z is the longitudinal axis. x_0 and y_0 are the coordinate axes of the shear centers of the cross-section and $\bar{i}_0^{\,2}$ are the polar radius of inertia calculated with respect to the shear center.

$$\bar{i}_0^{\,2} = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} \quad \text{(Eqn. 8.12)}$$

The distortion constant (C_w) in double symmetry axis I sections where h_0 is the distance between the centroids of the heads;



$$C_w = \frac{I_y h_0^2}{4} \qquad \text{(Eqn. 8.13)}$$

In most L and T sections the Cw value is very small and practically zero can be taken.

FLEXURAL TORSIONAL BUCKLING

$$F_{ez} = \frac{GJ}{A_g \bar{i}_0^2}$$

It is calculated by the expression. For sections consisting of double L and T profiles, the first term in Fez expression is omitted; obtained. For pairs L and T's, besides the second term in the same expression, the first term is negligible and this approach is acceptable. In this case, elastic buckling stress in bending-torsion stress;

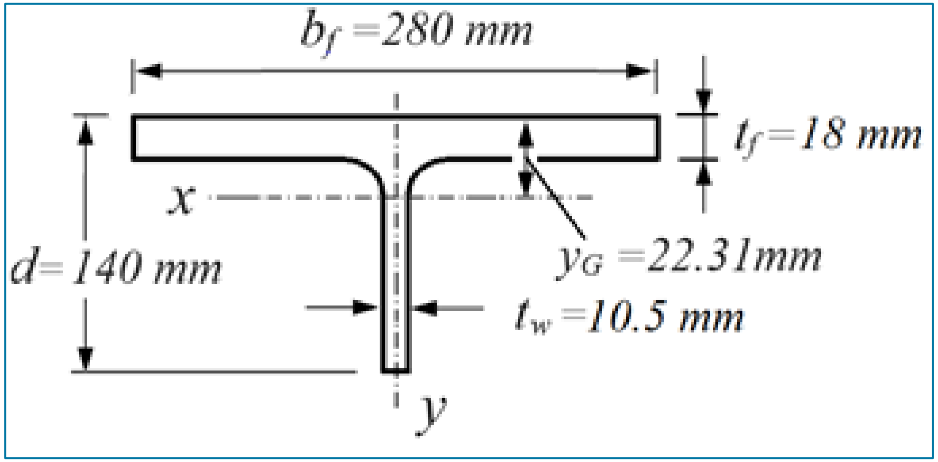
$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left(1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right)$$

The characteristic compressive strength (P_n) is calculated as follows:

$$P_n = F_e A_g \longrightarrow P_n = F_{cr} A_g$$

EXAMPLE

Calculate the characteristic compressive strength of a column with a ½ HE280 B cross section. Buckling length of 5.0 m in both axes (Use steel S355).



$$1/2\ HE280\ B:$$

$$A_g = 65.70\ \text{cm}^2,\ \ I_x = 673\ \text{cm}^4,\ I_y = 3297\ \text{cm}^4$$

$$J = 71.85 cm^4$$

$$y_G = 22.31 mm$$

$$i_x = 3.2\ \text{cm}$$

$$i_y = 7.1\ \text{cm}$$

Local buckling control:

- Flange (Tablo 5.1A, Case 1):

$$\frac{b_f/2}{t_f} = \frac{280/2}{18} = 7.78 < 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{20000}{35.5}} = 13.29$$ **Non-slender**

- Web (Tablo 5.1A, Case 5):

$$\frac{h}{t_w} = \frac{140}{10.5} = 13.33 < 0.75\sqrt{\frac{E}{F_y}} = 0.75\sqrt{\frac{20000}{35.5}} = 17.80$$ **Non-slender**

$$\frac{L_{cx}}{i_x} = \frac{500}{3.2} = 156 > \frac{L_{cy}}{i_y} = \frac{500}{7.1} = 70$$ Kolon x eksenine dik burkulur.

EXAMPLE

Characteristic compressive strength for flexural buckling limit state

$$156 \; > \; 4.71\sqrt{\frac{20000}{35.5}} = 112$$ **Inelastic buckling**

$$F_{cr} = 0.877 F_e = 0.877 \times \frac{\pi^2 \times 20000}{156^{\,2}} = 7.11 \text{ kN/cm}^2$$

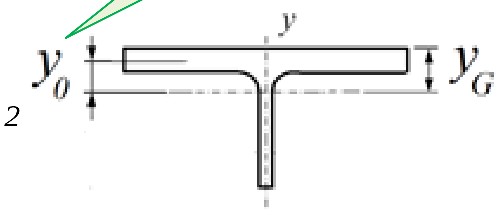
$$P_n = F_{cr} A_g = 7.1087 \times 65.70 = 467 \text{ kN}$$

EXAMPLE

Flexural-torsional buckling limit state;

$$1/2HE280\ B: \quad x_0 = 0\ , \quad y_0 = y_G - \frac{t_f}{2} = 22.31 - 18/2 = 13.31mm$$

y_0 kayma merkezi koordinat ekseni



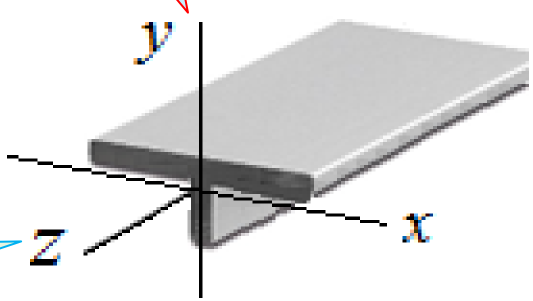
$$\bar{i}_0^{\,2} = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} = 0 + (13.31)^2 + \frac{673 + 3297}{65.7} = 62.19\,cm^2$$

Elastic buckling limit state

y simetri ekseni

$$F_{ey} = \frac{\pi^2 E}{\left(L_{cy}/i_y\right)^2} = \frac{\pi^2 \times 20000}{70^{\ 2}} = 40.24\ \text{kN/cm}^2$$

$$\frac{L_{cy}}{i_y} = \frac{500}{7.1} = 70\ <\ 4.71\sqrt{\frac{20000}{35.5}} = 112$$



z boyuna ekseni

EXAMPLE

Since the buckling is in the inelastic region, the F_{ey} value to be substituted in the formula is once again;

$$F_{ey} = 0.658^{Fy/Fe} F_y = 0.658^{35.5/40.24} 35.5 = 24.54 \text{ kN/cm}^2$$

$$F_{ez} = \frac{G\,J}{A_g\,\bar{i}_0^{\,2}} = \frac{7720 \times 71.85}{65.70 \times 62.19} = 135.76 \text{ kN/cm}^2$$

$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{i}_0^{\,2}} = 1 - \frac{0 + 1.331^2}{62.19^2} = 0.971$$

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left[1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{\left(F_{ey} + F_{ez}\right)^2}}\right] = \frac{24.54 + 135.76}{2 \times 0.971}\left[1 - \sqrt{1 - \frac{4 \times 24.54 \times 135.76 \times 0.971}{\left(24.54 + 135.76\right)^2}}\right] = 24.4 \text{ kN/cm}^2$$

$$F_e \rightarrow F_{cr}$$

$$\frac{L_{cy}}{i_y} = \frac{500}{7.1} = 70 < 4.71\sqrt{\frac{20000}{35.5}} = 112$$

veya

$$\frac{F_y}{F_e} = \frac{35.5}{24.39} = 1.45 < 2.25$$

$$F_{cr} = \left(0.658^{35.5/24.4}\right) x 35.5 = 19.3\ kN/cm^2$$

EXAMPLE

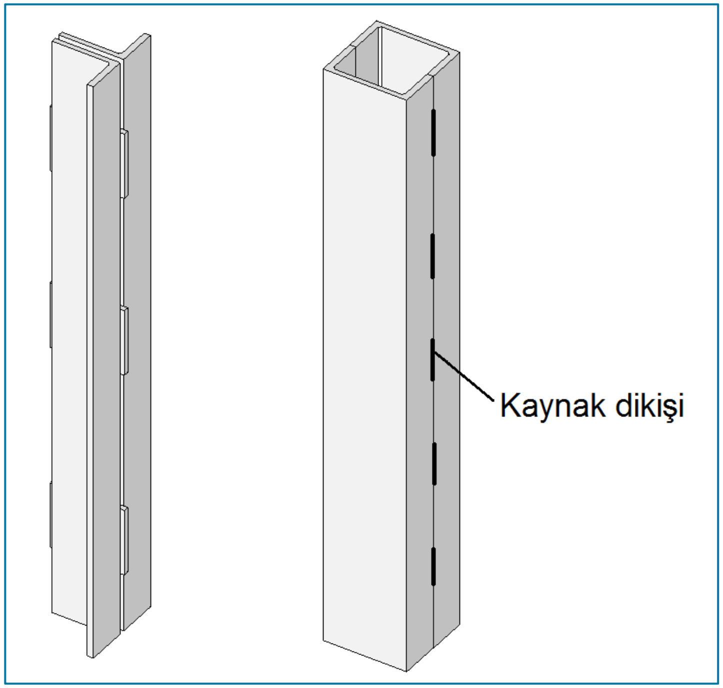
$$P_n = F_{cr}A_g = 19.3x65.7 = 1268\ kN$$

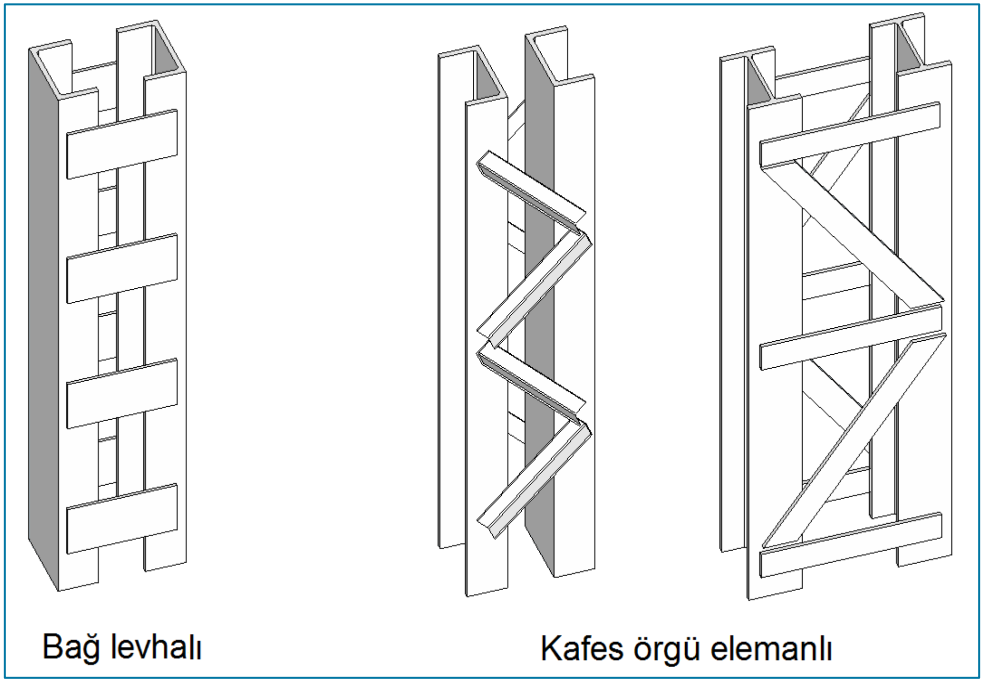
Thus the characteristic compressive strength of the column is:

$$P_n = min(467; 1268) \longrightarrow P_n = 467\ kN$$

BUILD-UP SECTIONS

These elements are "multi-piece" pressure bars in which individual rods that are in contact with each other or positioned at a certain distance are connected to each other by connecting elements.





BUILD-UP SECTIONS

BUILD-UP SECTIONS

The effect of shear strain on the characteristic compressive strength strength in the buckling limit state around the symmetry axis (y-axis) in pressure elements with built cross-section is considered with the effective slenderness ratio (L_c / i) m. Apart from that, they are designed like one-piece pressure bars.

The effective slenderness ratios (L_c / i) m of the elements are defined as follows depending on the characteristics of the joining tools used in the bond plate and lattice lattice elements;

1) In combinations where simple tightening method is applied with bolts;

$$\left(\frac{L_c}{i}\right)_m = \sqrt{\left(\frac{L_c}{i}\right)_o^2 + \left(\frac{a}{i_i}\right)^2}$$

BUILD-UP SECTIONS

2) In joints where welded joints and bolts are tightened (prestressed bolts) with the A, B or C class surface preparation defined in Regulation-Table 13.11;

$$\frac{a}{i_i} \leq 40 \quad ise; \quad \left(\frac{L_c}{i}\right)_m = \left(\frac{L_c}{i}\right)_o$$

TABLE 8.1

$$\frac{a}{i_i} > 40 \quad ise; \quad \left(\frac{L_c}{i}\right)_m = \sqrt{\left(\frac{L_c}{i}\right)_o^2 + \left(\frac{K_i a}{i_i}\right)^2}$$

$(L_c/i)_m$: Modified slenderness ratio

$(L_c/i)_o$: Effective slenderness ratio for one member

i : The radius of gyration of the element cross-section with respect to the buckling axis

i_i : Minimum radius of gyration of a single part a

a : Distance between the connecters

K_i : 0.50 (for angles back-to-back)

K_i : 0.75 (for channels back-to-back)

K_i : 1.00 (for other cases)

BUILD-UP SECTIONS

Some of the conditions to be considered in the design of multi-piece pressure bars are given below (Regulation - 8.4):

- In case of split multi-part pressure rods, the ratio of the distance (a) between the fasteners to the minimum radius of inertia (a / ii) should not exceed of the maximum slenderness of the pressure bar:

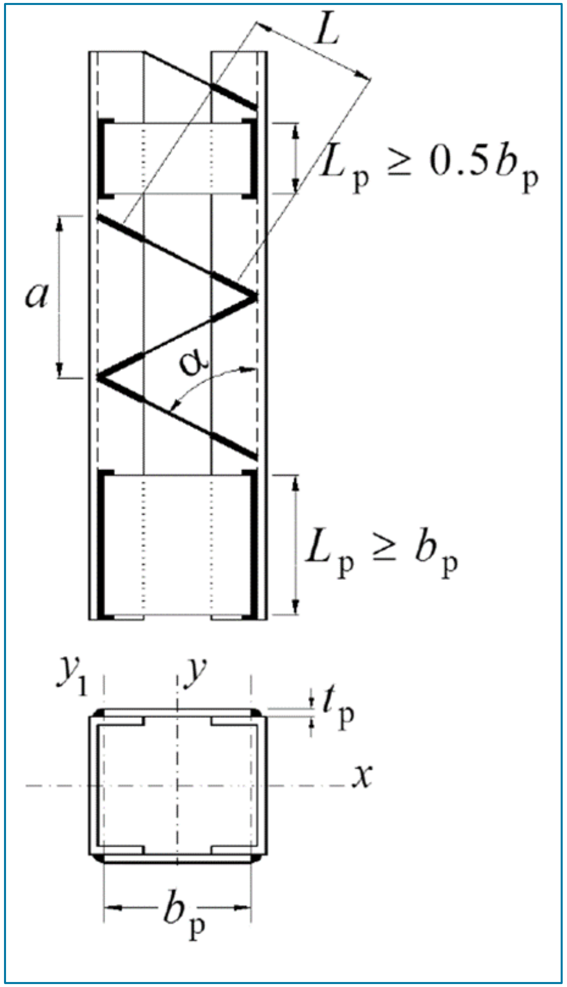
$$\frac{a}{i_i} \le \frac{3}{4}\left(\frac{L_c}{i}\right)_{maks}$$

- The application conditions specified in the Regulation-Table 8.1 for end and intermediate connections in multi-part pressure elements must be fulfilled.

- The angle made by the mesh elements with the element longitudinal axis should not be less than 600 in single diagonal lattice joints and 450 in cross-diagonal lattice joints.

BUILD-UP SECTIONS

- The slenderness of the single diagonal knit elements should meet the condition L / i <140, and the slenderness of the cross knit members should satisfy 0.7L / i <200.
- In multi-part pressure rods with all parts in contact or very close to each other, the distance a is allowed to be considered as a one-piece pressure element with parts continuously joined, if the conditions in Table 8.1e and Table 8.1.f are met.
- Due to the bending torsional deformation of the multi-piece pressure bar, the additional shear force generated in knitting elements, tie plates and joints is taken as 2% of the axial force strength of the pressure bar:



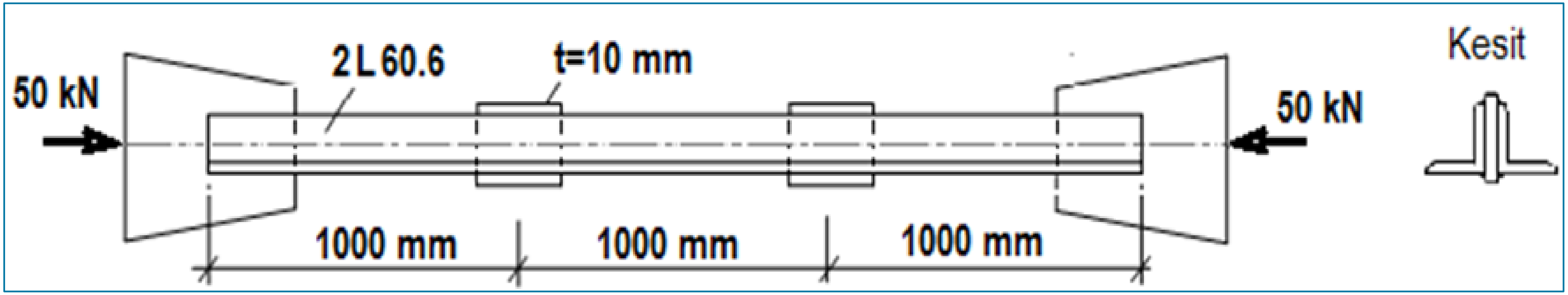
$$V_u=0.02P_d$$

EXAMPLE

Consisting of 2-L60.6 profile, the 3.0 m long truss system upper head bar is under the effect of 50 kN axial pressure force. (S 275)

a) Determine the characteristic compressive force strength of the element.

b) Check the design compressive strength of the element (YDKT)



EXAMPLE

S275 $F_y = 27.5 \text{ kN/cm}^2 \quad F_u = 43.0 \text{ kN/cm}^2$

L60.6 $A_g = 6.91 \text{ cm}^2 \quad I_x = 22.79 \text{ cm}^4 \quad I_y = 22.79 \text{cm}^4 \quad J = 0.789 \text{ cm}^4$

$i_x = i_y = 1.82 \text{ cm} \quad i_{min} = 1.17 \text{ cm} \quad x_g = y_g = 1.69 \text{ cm}$

For the arm of corner (Table 5.1A, Case 3):

$$\lambda \le \lambda_r \rightarrow \frac{b}{t} \le 0.45\sqrt{\frac{E}{F_y}} \rightarrow \frac{60}{6} = 10 < 0.45 \times \sqrt{\frac{20000}{27.5}} = 12.13$$

Non-slender section

Column buckling length $L_{cx} = L_{cy} = K \times L = 1 \times 3000 = 3000 \text{ mm} = 300\text{cm}$

EXAMPLE

$$i_x = 1.82 \text{ cm}$$

$$I_y = 2\left(22.79 + 6.91 \times \left(\frac{1.0}{2} + 1.69\right)^2\right) = 111.86 \text{ cm}^4$$

$$i_y = \sqrt{\frac{111.86}{2 \times 6.91}} = 2.85 \text{ cm}$$

$$\left(\frac{L_{cx}}{i_x}\right) = \frac{300}{1.82} = 165$$

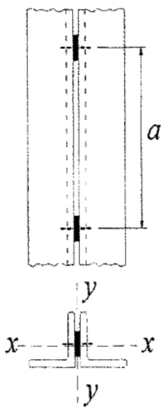
$$\left(\frac{L_{cy}}{i_y}\right) = \frac{300}{2.85} = 105$$

$$\left(\frac{L_c}{i}\right)_{maks} = 165 < 200$$

EXAMPLE

Check the distance between connectors: *Yönetmenlik Sayfa 85*

$$\frac{a}{i_1} \leq \frac{3}{4}\left(\frac{L_c}{i}\right)_{maks} \rightarrow \quad \frac{100}{1.17} = 85.45 < \frac{3}{4} \times 165 = 123.75 \quad \textbf{OK}$$



For angles back to back K_i =0.5 and modified slenderness ratio; *Yönetmenlik Sayfa 84*

$$\frac{a}{i_1} = 85.45 > 40 \rightarrow \left(\frac{L_c}{i}\right)_m = \sqrt{\left(\frac{L_c}{i}\right)_0^2 + \left(\frac{K_i\, a}{i_1}\right)^2} = \sqrt{105^2 + \left(\frac{0.5 \times 100}{1.17}\right)^2} = 113.4$$

$$\frac{a}{i_i} \leq 40 \quad ise; \quad \left(\frac{L_c}{i}\right)_m = \left(\frac{L_c}{i}\right)_o$$

$$\frac{a}{i_i} > 40 \quad ise; \quad \left(\frac{L_c}{i}\right)_m = \sqrt{\left(\frac{L_c}{i}\right)_o^2 + \left(\frac{K_i a}{i_i}\right)^2}$$

EXAMPLE

Flexural Buckling

$$\left(\frac{L_{cy}}{i_y}\right)_m = 113.4 < \left(\frac{L_{cx}}{i_x}\right) = 165$$

So buckling in x-axis

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{L_{cx}}{i_x}\right)^2} = \frac{\pi^2 \times 20000}{(165)^2} = 7.25 \text{ kN/cm}^2$$

$$\frac{F_y}{F_e} = \frac{27.5}{7.25} = 3.79 > 2.25$$

So critical buckling stress

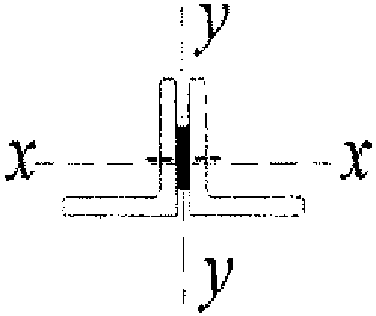
$$F_{cr} = 0.877 F_e = 0.877 \times 7.25 = 6.36 \text{ kN/cm}^2$$

EXAMPLE

Flexural-torsional buckling

Coordinates of the center of gravity of the center of gravity for the double corner cross section

$$x_o = 0, \qquad y_o = 16.9 - 6/2 = 13.9 \text{ mm}$$



The square of the polar radius of gyration calculated with respect to the slip center;

$$\bar{i}_o^{\,2} = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} = 0^2 + 1.39^2 + \frac{2\times 22.79 + 111.86}{2\times 6.91} = 13.32 \text{ cm}^2$$

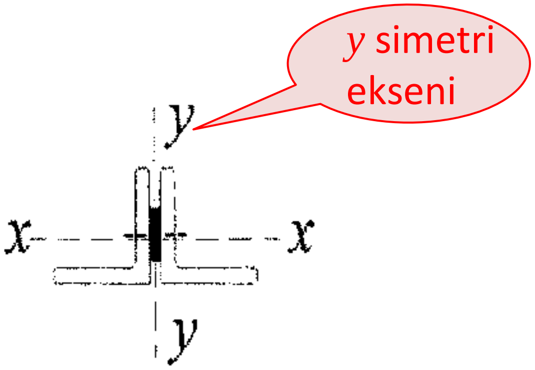
$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{i}_o^{\,2}} = 1 - \frac{0 + 1.39^2}{13.32} = 0.855$$

EXAMPLE

Elastic buckling stress for torsional stress:
z longitudinal axis

$$F_{ez} = \frac{G\,J}{A_g\,\bar{i}_o^{\,2}} = \frac{7720\times\left(2\times 0.789\right)}{2\times 6.91\times 13.32} = 66.18\ \text{kN/cm}^2$$

Elastic buckling stress for y axis of symmetry:



$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{i_y}\right)^2} = \frac{\pi^2 \times 20000}{(113.4)^2} = 15.35\ \text{kN/cm}^2$$

Elastic buckling stress in the flexural - torsional buckling limit state:

$$F_e = \left(\frac{F_{ey}+F_{ez}}{2H}\right)\left[1-\sqrt{1-\frac{4F_{ey}F_{ez}H}{\left(F_{ey}+F_{ez}\right)^2}}\,\right]$$

EXAMPLE

$$F_e = \left(\frac{15.35+66.18}{2\times 0.855}\right)\left[1-\sqrt{1-\frac{4\times 15.35\times 66.18\times 0.855}{\left(15.35+66.18\right)^2}}\right] = 14.74 \text{ kN/cm}^2$$

$$\frac{F_y}{F_e} = \frac{27.5}{14.74} = 1.87 < 2.25$$

So critical buckling stress for bending - torsional stress;

$$F_{cr} = \left[0.658^{F_y/F_e}\right]F_y = \left[0.658^{1.87}\right]\times 27.5 = 12.60 \text{ kN/cm}^2$$

Since 6.36 kN/cm² < 12.60 kN/cm² characteristic buckling strength can be defined based on the flexural buckling limit state

$$P_n = F_{cr}A_g = 6.36\times\left(2\times 6.91\right) = 87.9 \text{ kN}$$

EXAMPLE

For LRFD $P_u = 1.4P_G = 1.4 \times 50 = 70 \text{ kN}$

Design compressive strength,

$$P_d = \phi_c P_n = 0.9 \times 87.9 = 79 \text{ kN} > P_u = 70\,kN$$ **OK**