2. LOADS AND ALLOWABLE STRESSES

2.1 General

The structural safety shall be established by computing the actual stresses in the structural member and being sure that these stresses do not exceed the allowable (working) stresses specified below. The actual stresses are calculated used different load combinations mentioned in the "Egyptian Code of Practice for Loads and Forces for Structural Elements". Deflections shall also be computed and shall in no case exceed the limits herein after specified.

2.2 primary and secondary stresses

The stresses shall be calculated according to the following two cases of loading:

2.1 Case I: Primary Stresses

This case of loading is the case of main loads affecting the structure. Any of the following loading types or the sum of them is considered a main "primary load". Dead Loads, Live Loads or superimposed Loads, Dynamic Effects, Centrifugal Forces,…..etc.

2.2 Case II: Primary and Additional Stresses

In case of adding one or more of the loads which are classified as secondary loads to the primary loads to the previously mentioned "Case I", the case of loading is then denoted by (case "II"). Examples of the secondary loads are (Wind Loads or Earthquake Loads, Braking Forces, Lateral Shock, Settlement of Supports in addition to the Shrinkage and Creep of concrete).

Important notes:

1. Stresses due to wind loads shall be considered as primary stresses (Case I) for such structures as transmission and microwave towers, transmission pole structures and wind bracing systems.
2. In the design of steel sections, they shall be designed according to “case I”, then they should be checked fir (case II) and the actual stresses shall in no case exceed the allowable stresses by more than 20%.

2.3 Loads

Loads are taken from the Egyptian code for claculating the loads and forces in buildings. There are some special loads for steel structures. In the following sections, some of these loads are summarized.

2.3.1 DEAD LOAD

Dead loads are taken as the loads of the covering material in addition to the own weight of steel structure itself. In case of lack of information about the exact weight of the covering material (in case of single storey buildings), the following weights may be used in the structural analysis;

Single Layer Steel Sheets	5kg/m^2 to 8 kg/m^2
Double Layered Steel Sheets with Isolation	10kg/m^2 to 15 kg/m^2

In case of multi-storey buidings and using chequred steel plates as a floor, the wight of the cheaqured steel plate may be considered about 50 kg/m^2.

In case of multi-storey buidings and using reinforced concrete slab as a floor, the wight of the slab may be considered considering the reinforced concrete specific weight equal to 2500 kg/m^3.

2.3.2 Live Load

Live load is a gravity load acting when the structure is in service. This

load varies in magnitude and location. Human occupants, furniture, movable equipment, stored goods, vehicles and cranes are considered live loads. Live loads are usually prescribed by the local building codes. For building with flat roofs, live loads are considered according to table (4-1) in the Egyptian Code for Calculating Loads and Forces on buildings.

In case of inclined roofs, live loads are considered taking into consideration the angle of roof slope "α" according to the curves presented in Figure (2-1) [Egyptian Code for Calculating Loads and Forces on buildings]. In this figure, two curves are plooted for both accessible and inaccessible roofs. The value of the live load can be calculated using equations 2-1 and 2-2 instead of the curve.

L.L. = 60 – 66 2/3 $\tan\alpha > 20$ kg/m^2 (for Inaccessible roofs) (2-1)
L.L. = 100 – 83 1/3 $\tan\alpha > 50$ kg/m^2 (for Accessible roofs) (2-2)
L.L. = 200 – 166 2/3 $\tan\alpha > 50$ kg/m^2 (for Accessible roofs) (2-3)

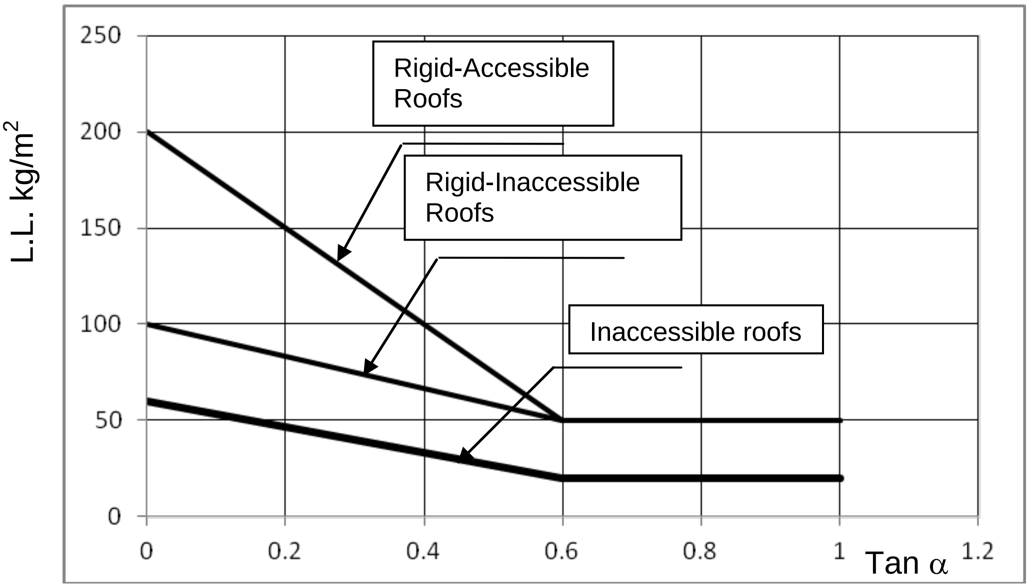


Figure (2-1) Live Loads on Inclined Roofs

2.3.3 WIND LOAD

All steel structures are subject to wind load. On any typical building of rectangular plan wind causes pressure on the winward side and suction on the leeward side, as well as either uplift or downward pressure on the roof. Wind loads affect the building in direction perpendicular to the surface. This force is considered positive if it is in surface direction (pressure) and negative if it is outside the surface, away from the surface direction (suction).
The external pressure or suction of wind force affecting the building surfaces is calculated using the following equation:

$$P_e = C_e.k.q \tag{2-4}$$

Where;
P_e is the external design static wind pressure affecting the external unit area, t/m^2
C_e is the coefficient of wind effect (pressure or suction) and it depends on the building geometric shape and is taken as shown in figures (2-2) to (2-5) for different types of frames.
k is a coefficient changes with the change of building height and taken according to table (2-1),
q is the basic wind pressure calculated from equation (2-5).

$$q = 0.5\,\rho\,V^2\,C_t\,C_s \tag{2-5}$$

Where:
V is the wind speed for a 3 seconds period at +10m from the ground level according to table (2-2)
ρ is the density of air (taken 1.25 kg/m^3),
C_t is the earth surface topographic coefficient calculated from the Egyptian Code for Calculating Loads and Forces in Structures and Buildings (Code No. 210), table 7-2.
C_s is the structure coefficient and taken as follows:
= 1 in case of; a- buildings of height < 60 m
b- trussed buildings,

c- buildings of height/minimum breadth < 4

>1 in other cases and calculated according appendix 7-A in the Egyptian Code for Calculating Loads and Forces in Structures and Buildings (Code No. 210).

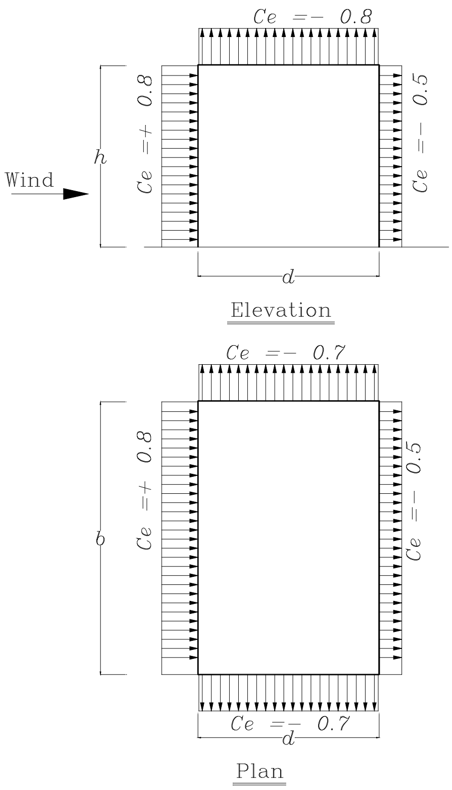


Figure (2-2) Coefficient of Wind Effect on Rectangular Frames

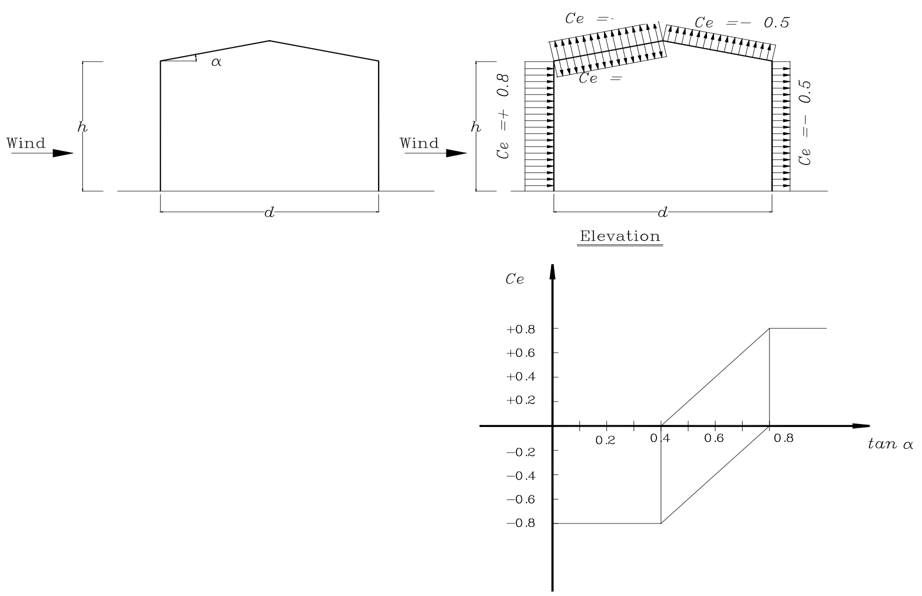


Figure (2-3) Coefficient of Wind Effect on Pitched Roof Frames

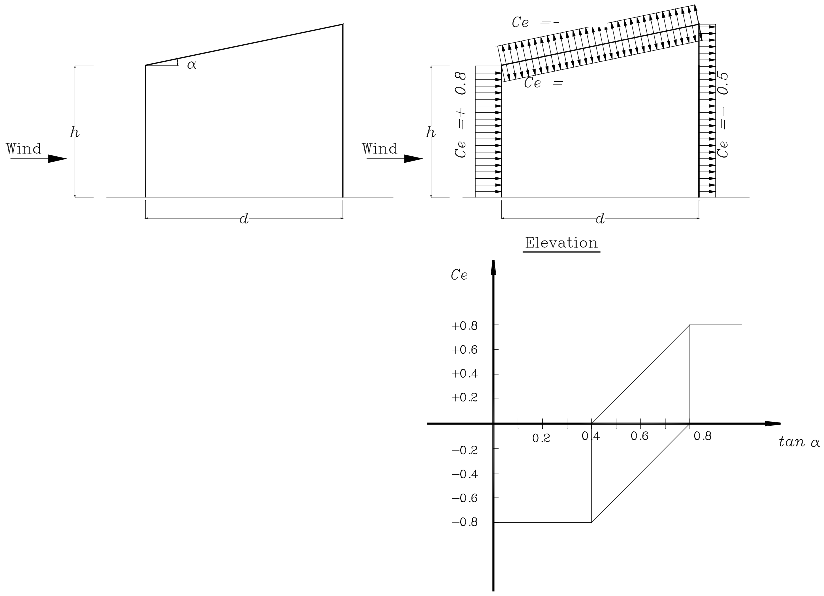


Figure (2-4) Coefficient of Wind Effect on Inclined Roof Frames

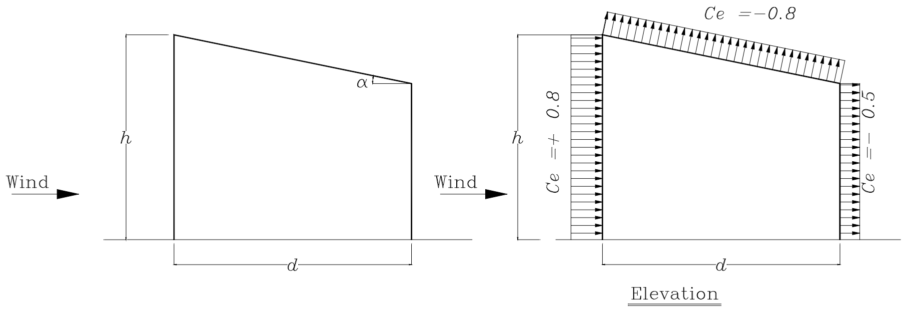


Figure (2-5) Coefficient of Wind Effect on Inclined Roof Frames

Table (2-1) Values of Coefficient *“k”*

Height “m”	Coefficient *“k”*
0 to 10	1.0
10 to 20	1.15
20 to 30	1.40
30 to 50	1.60
50 to 80	1.85
80 to 120	2.10
120 to 160	2.30
More than 160	2.50

The height of building taken to calculate the coefficient *“k”* is the height of location the wind pressure is calculated at it.

Table (2-2) Basic wind Speed at different Locations in Egypt (m/sec)

Location	*V “m/sec”*
Marsa Matrouh, El-Dabaa, Zaafarana	42
El-Saloum	39
Alexandria, Abu Sower, Hurghada and all coastal areas	36
Cairo, Asyot, El-Dakhla, Aswan, Siwa, Luxor	33
Fayoum, Menya, Tanta, Modereyat El-Tahrir, Mansoura, Damanhour	30

2.3 CLASSIFICATION OF CROSS-SECTION

In the Egyptian code of practice, the sections are classified according to local buckling of componenet plate elements into three section classes; compact sections, non-compact sections and slender sections as follows:

2.3.1 Class 1 (Compact Sections)

Compact sections are the sections that can achieve plastic moment capacity without the occurrence of local buckling of any of its component plate elements subjected to compressive stress.

2.3.2 Class 2 (Non-Compact Sections)

Non-compact sections are the sections that can achieve the yield moment capacity without the occurrence of local buckling of any of its component plate elements subjected to compressive stress.

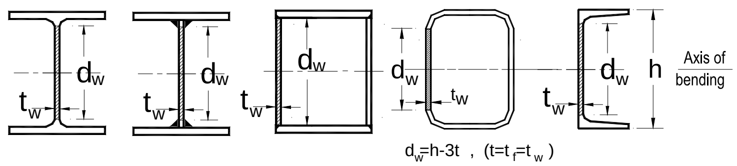
2.3.3 Class 3 (Slender Sections)

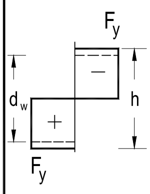
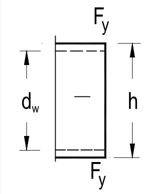
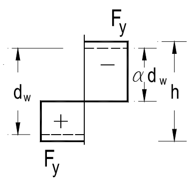
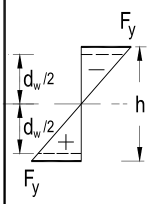
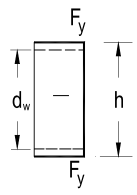
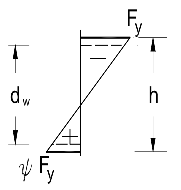
Slender sections are the sections in which local buckling of component plate elements subjected to compressive stress takes place before failure takes place. When any of the compression component elements of a cross-section is classified as class 3, the whole section shall be designed as class 3 cross-section.

The limiting plate width-to-thickness ratio for class 1 and class 2 are given in the following tables:

Table (2.3a) Maximum Width to Thickness Ratios for Stiffened Compression Elements

(a) Webs: (Internal elements perpendicular to axis of bending)

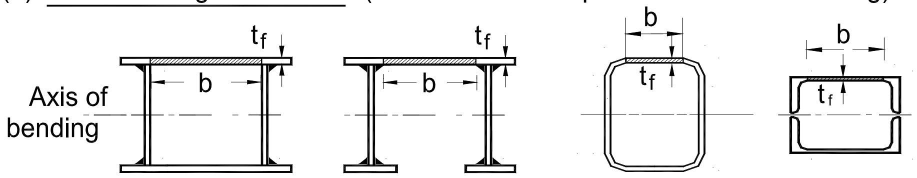


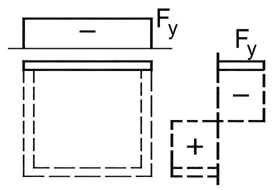
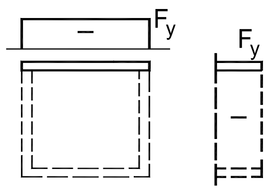
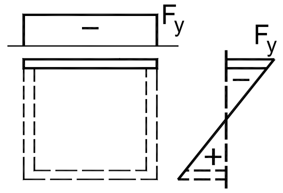
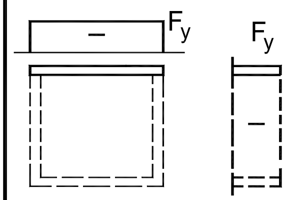
Class / Type	Web Subject to Bending	Web Subject to Compression	Web Subject to Bending and Compression	
1. Compact Stress distribution in element. Not for single channel				
	$\alpha = 0.5$	$\alpha = 1.0$	$\alpha > 0.5$	$\alpha \leqslant 0.5$
	$\frac{d_w}{t_w} \leqslant \frac{127}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{58}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{699/\sqrt{F_y}}{13\alpha - 1}$	$\frac{d_w}{t_w} \leqslant \frac{63.6/\alpha}{\sqrt{F_y}}$
2.Non-Compact Stress distribution in element.				
	$\psi = -1$	$\psi = 1$	$\psi > -1$	$\psi \leqslant -1$
	$\frac{d_w}{t_w} \leqslant \frac{190}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{64}{\sqrt{F_y}}$	$\frac{d_w}{t_w} \leqslant \frac{190/\sqrt{F_y}}{2+\psi}$	$\frac{d_w}{t_w} \leqslant \frac{95(1-\psi)\sqrt{-\psi}}{\sqrt{F_y}}$

F_y in t/cm^2

Table (2.3b)Maximum Width to Thickness Ratios for Stiffened Compression Elements

(b) Internal Flange Elements: (Internal elements parallel to axis of bending)

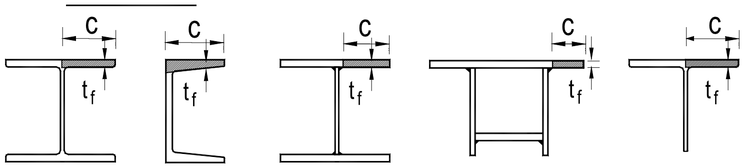


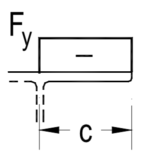
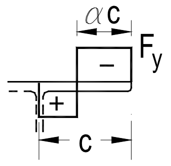
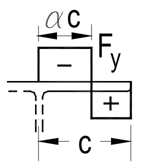
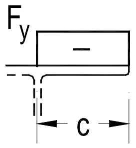
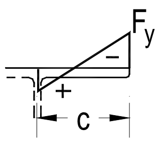
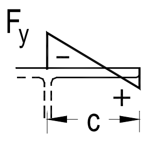
Class / Type	Section in Bending	Section in Compression
1. Compact Stess distribution in element and across section		
	$\frac{b}{t_f} \leqslant \frac{58}{\sqrt{F_y}}$	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$
2. Non-Compact Stess distribution in element and across section		
	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$	$\frac{b}{t_f} \leqslant \frac{64}{\sqrt{F_y}}$

F_y in t/cm^2

Table (2.3c) Maximum Width to Thickness Ratios for Unstiffened Compression Elements

(c) Outstand Flanges

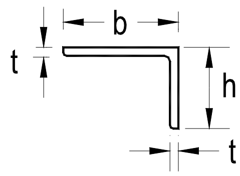
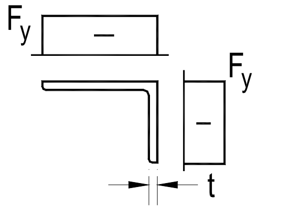
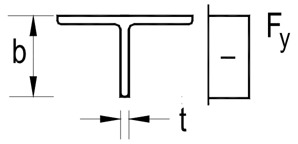
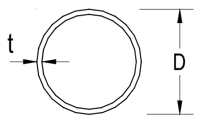


Class / Type	Flange Subject to Compression Due to M_x	Flange Subject to Compression and Bending: Tip in Comprssion	Flange Subject to Compression and Bending: Tip in Tension
1. Compact Stress distribution in element			
Rolled	$\frac{c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 16.9/\sqrt{\alpha F_y}$
Welded	$\frac{c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{F_y}$	$\frac{\alpha c}{t_f} \leqslant 15.3/\sqrt{\alpha F_y}$
2. Non-Compact Stress distribution in element			
Rolled	$\frac{c}{t_f} \leqslant 23/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 35\sqrt{K_\sigma/F_y}$	
Welded	$\frac{c}{t_f} \leqslant 21/\sqrt{F_y}$	$\frac{c}{t_f} \leqslant 32\sqrt{K_\sigma/F_y}$	

F_y in t/cm^2 For K_σ see Tables 2.3 & 2.4

Table (2.3d) Maximum Width to Thickness Ratios for Compression

Elements

(d) Angles: Refer also to (Table 2.1c) "Outstand flanges"	 (Does not apply to angles in continuous contact with other components)
Class	Section In Compression
Stress distribution across section	
Non-compact	$b/t \le 23/\sqrt{F_y}$; $(b+h)/2t \le 17/\sqrt{F_y}$ (*)
Class	Section In Compression
(e) T-section:	
Non-compact	$b/t \le 30/\sqrt{F_y}$
(f) Tubular section:	
Class	Section In Bending and/or Compression
1. Compact	$D/t \le 165/F_y$
2. Non-Compact	$D/t \le 211/F_y$

F_y in t/cm^2 (*) For unequal angles

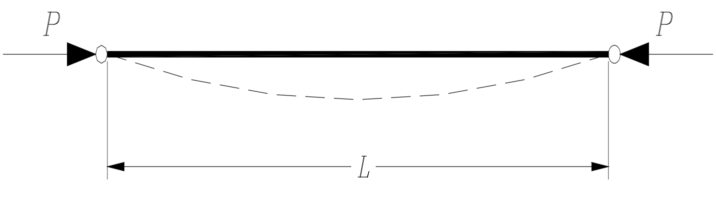
2.4 critical load / stress

The maximum strength of an axially loaded member is equal to the allowable compressive stress multiplied by the cross-sectional area of the member. However for long axially loaded compression members failure may occur first by elastic buckling. The elastic critical load corresponding to this failure load is given by Euler (1757) and known as Euler load. For ideal pinned compression member shown in figure(2-6):

Euler Load = $P_e = \dfrac{\pi^2 EI}{L^2}$ ton (2-6)

Critical stress = $\dfrac{P_e}{A} = \dfrac{\pi^2 EI}{L^2 A} = \dfrac{\pi^2 E}{L^2}\left(\dfrac{I}{A}\right) = \dfrac{\pi^2 E}{L^2} r^2 = \dfrac{\pi^2 E}{\lambda^2}$ t/cm^2(2-7)

Where *L* is the buckling length of the member, *A* is the cross-sectional area, and *I* is the moment of inertia in the buckling direction, *r* is the radius of gyration and *E* is the modulus of elasticity *(for steel = 2100 t/cm^2)*. The factor λ introduced in the above equation is equal to *L/r* and is known as the slenderness ratio of the compression member.



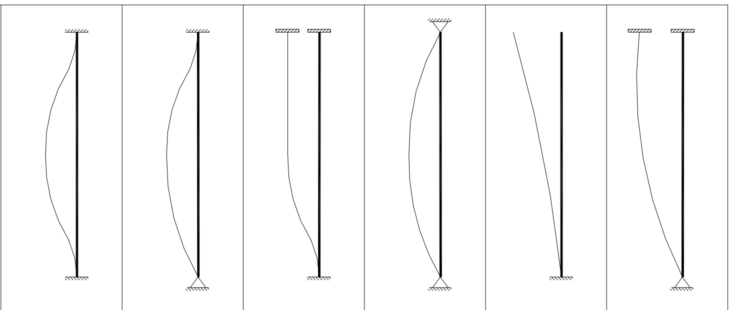
Figure(2-6) Buckling Mode of Pin-Ended Column (Euler, 1757)

The above equation is applicable only for pin-ended columns. For different end conditions, the effective buckling length concept is introduced. The slenderness ratio is redefined as follows:

$$\lambda = \frac{L_b}{r} = \frac{KL}{r} \qquad (2\text{-}8)$$

Where *“K”* is the buckling factor.
For ideal columns with well-defined boundary conditions the *K* is given in the shown table:

Table (2-4) Buckling Length Factor “K” for Members with Well Defined End-Conditions.



BUCKJLING MODE						
K	0.65	0.80	1.20	1.00	2.10	2.00
END CONDITIONS	ROTATION PREVENTED & TRANSLATION PREVENTED					
	ROTATION PERMITTED & TRANSLATION PREVENTED					
	ROTATION PREVENTED & TRANSLATION PERMITTED					
	ROTATION PERMITTED & TRANSLATION PERMITTED					

Tables (2-5) to (2-8) shows the buckling lengths in most of the familiar compression members in trusses of buildings and bridges.

Table 2-5: Buckling Length of Compression Members in Buildings and Bridge Bracing Systems

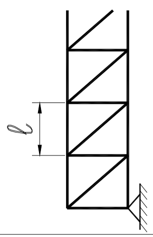
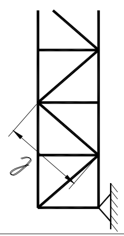
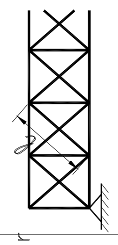
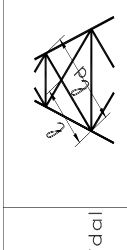
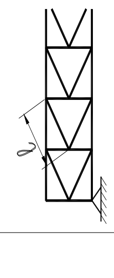
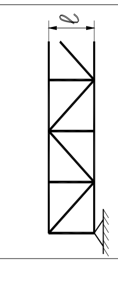
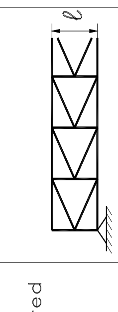
Member		In-Plane	Out-of-Plane: Compression Chord Effectively Braced	Out-of-Plane: Compression Chord Unbraced
Chords		ℓ	ℓ	0.75 span (Clause 4.3.2.2)
Diagonals -Single Triangulated web system		ℓ	ℓ	$1.2\,\ell$
-Multiple Intersected web rectangular system adequately connected		$0.5\,\ell$	$0.75\,\ell$	ℓ

Table 2-6: Buckling Length of Compression Members in Buildings and Bridge Bracing Systems (Cont.)

Member		In-Plane	Out-of-Plane: Compression Chord Effectively Braced	Out-of-Plane: Compression Chord Unbraced
Diagonals –Multiple Intersected web trapezoidal system adequately connected		ℓ	$0.8\,\ell_d$	——
– K–system		ℓ	$1.2\,\ell$	$1.5\,\ell$
Vertical members –Single triangulated web system		ℓ	ℓ	$1.2\,\ell$
–K–intersected web system		$0.5\,\ell$	$(0.75+0.25\frac{N_S}{N_L})\ell$	$(0.90+0.30\frac{N_S}{N_L})\ell$

N_S = Smaller value of compression force

N_L = Larger value of compression force

Table 2-7: Buckling Length of Compression Members in Bridges

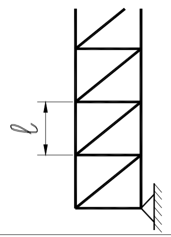
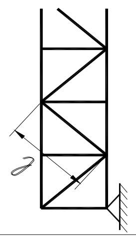
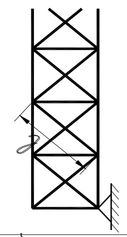
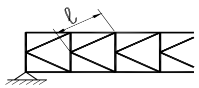
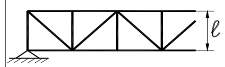
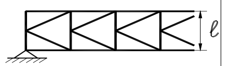
Member		In-Plane	Out-of-Plane: Compression Chord Effectively Braced	Out-of-Plane: Compression Chord Unbraced
Chords		$0.85\ \ell$	$0.85\ \ell$	0.75 Span (Clause 4.3.2.2) or Equation 4.2 if using U-Frames
Diagonals -Single Triangulated web system		$0.70\ \ell$	$0.85\ \ell$	$1.2\ \ell$
-Multiple Intersected web rectangular system adequately connected		$0.85\ \ell/2$	$0.75\ \ell$	ℓ

Table 2-8: Buckling Length of Compression Members in Bridges

Member		In-Plane	Out-of-Plane	
			Compression Chord Effectively Braced	Compression Chord Unbraced
Diagonals – K-system		$0.9\ \ell$	$1.2\ \ell$	$1.5\ \ell$
Vertical members –Single triangulated web system		$0.7\ \ell$	$0.85\ \ell$	$1.2\ \ell$
–K-intersected web system		0.35ℓ	$(0.75+0.25\frac{N_S}{N_L})\ell$	$(0.90+0.30\frac{N_S}{N_L})\ell$

N_S = Smaller value of compression force
N_L = Larger value of compression force

2.5 maximum stiffness Limits

2.5.1 MAXIMUM SLENDERNESS RATIOS FOR COMPRESSION MEMBERS

The slenderness ratio of a compression member shall not exceed λ_{max} Given in the next table:

Maximum Allowable Slenderness Ratios of Compression Members

Member	λ_{max}
Buildings:	
Compression members	180
Bracing systems and secondary members	200
Bridges:	
Compression members in railway bridges	90
Compression members in roadway bridges	110
Bracing systems	140

2.5.2 MAXIMUM SLENDERNESS RATIOS FOR TENSION MEMBERS

The maximum slenderness ratio of a tension member (excluding wires) :

Tension member in		$\lambda = l_{eff} / r$
Buildings		300
Bridges:	Roadway Bridges	180
	Railway Bridges	160
	Vertical hungers	300
	Bracing system	200

2.5.3 The length/depth ratio

Another stiffness requirements for tension members (excluding wires) is:

Tension member in	***l / d***
Buildings	60
Roadway Bridges	35
Railway Bridges	30