STRESS-STRAIN CURVE FOR CONCRETE

- A typical stress-strain curve for concrete, when tested in a compression-testing machine, is shown in Fig. 3.1.
- The stress and strain are approximately proportional to each other up to about half of the ultimate strength.
- At higher stresses, the behavior is considerably inelastic.

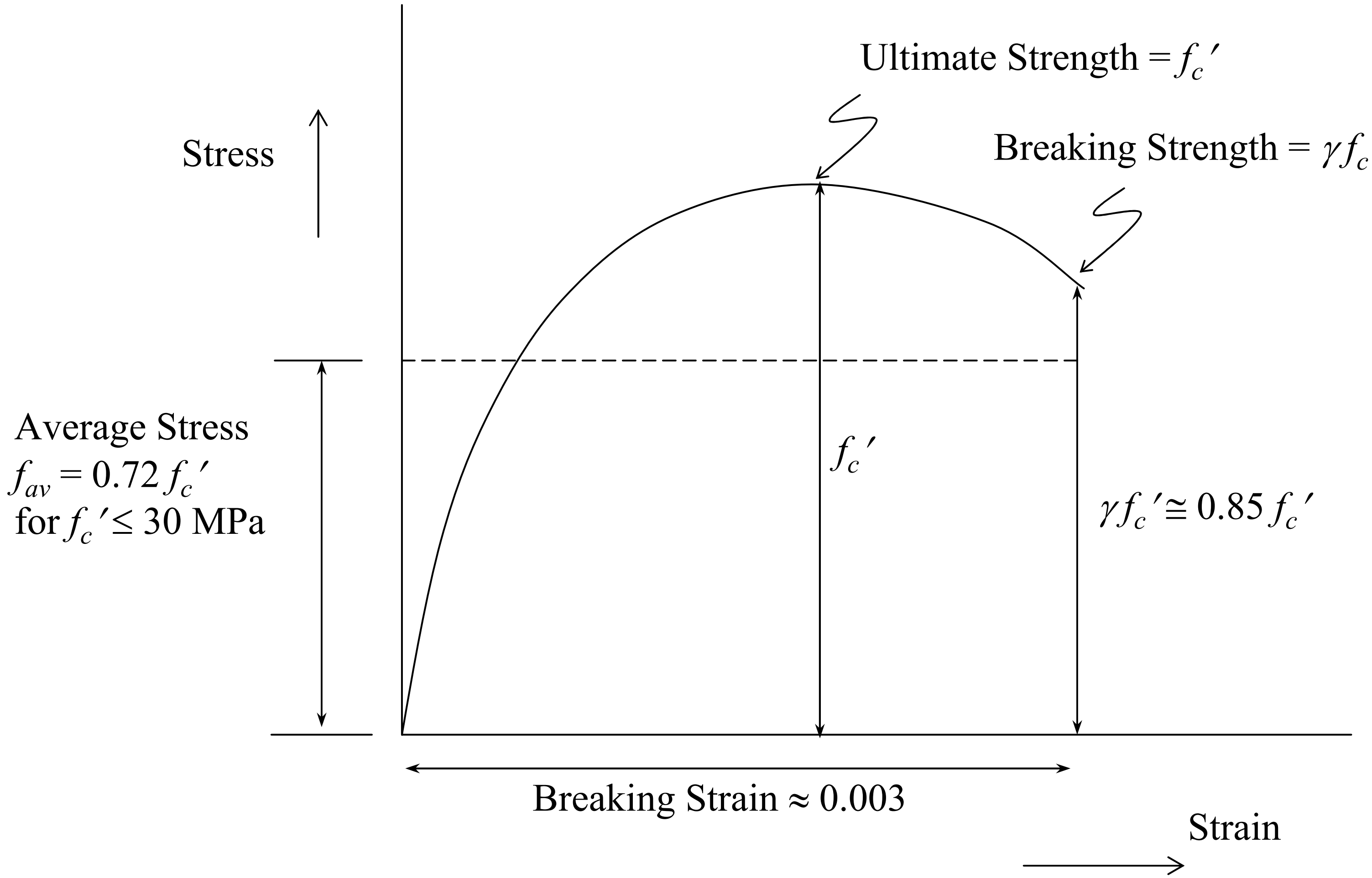


Fig. 3.1. Experimental Stress-Strain Curve For Concrete.

- The final breaking or crushing takes place at a stress level lesser than the maximum/ultimate stress, which is called the breaking stress.
- This value is approximately 15% lesser than the ultimate strength.
- The strain at which crushing of concrete takes place is 0.0025 for high-strength concretes to more than 0.0045 for low-strength concretes.
- For design, a conservative value of 0.003 is considered as the ultimate strain, for all concretes except very high strength ones.

- The area under the curve divided by this ultimate strain gives the average value of the stress.
- The average stress is $0.72\, f_c'$ for concretes up to ultimate strength of 30 MPa (4000 psi).
- The factor γ indicates the ratio of breaking strength to the ultimate strength.

$$\gamma = \frac{\text{Breaking stress}}{\text{Ultimate Stress } (f_c')}$$

$$= 0.85 \text{ (average value)}$$

The ratio of average to crushing stress of concrete is denoted by β_1 and is found for concrete of $f_c' \leq 30$ MPa as under:

$$\beta_1 = \frac{f_{av}}{f_{crush}} = \frac{0.72 f_c'}{0.85 f'} = 0.85 \quad (for\ f_c' \leq 30\ MPa)$$

The concrete on the compression side of a flexural member is also subjected to compressive stresses and strains.

The related stress-strain diagram within the member must be known in order to determine its resistance against the applied loads.

It may be noted in a beam that away from the neutral axis, the strain is linearly increasing just as in the compression-testing machine.

- Hence, the stress-strain behavior in such a member must be identical to that obtained in a separate compression test.
- The stress-strain curve of Fig. 3.1 is developed within the member with the difference that the diagram is rotated counterclockwise through 90° until its strain axis becomes vertical.
- Further, the strain axis is replaced by the depth of section as it is directly proportional to the strain.
- This resulting diagram is shown in Fig. 3.2, considering strain on the vertical axis and stress on the horizontal axis.

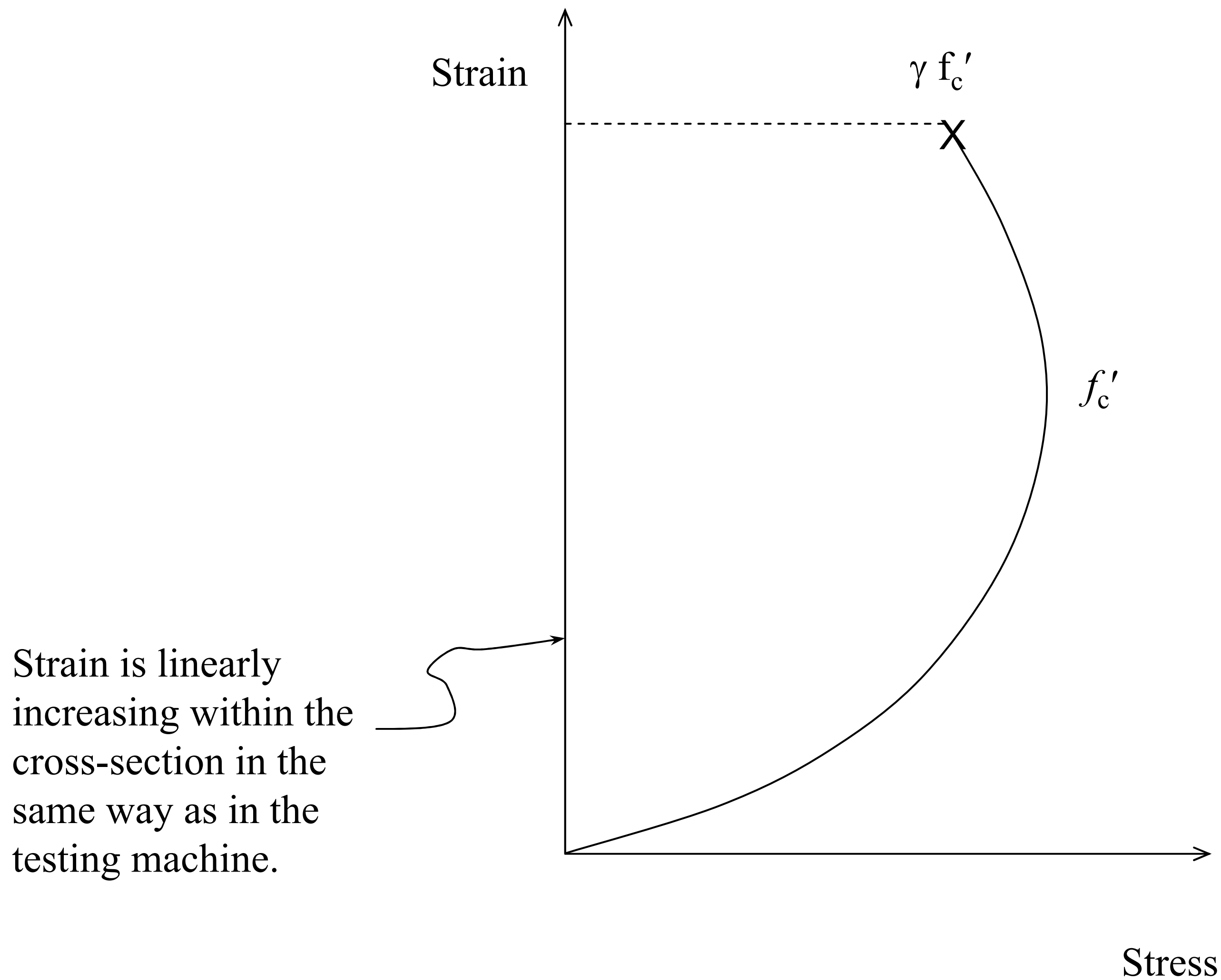


Fig. 3.2. Concrete Stress Block in a Flexural Member.

- The actual parabolic variation of concrete stress is approximated by a rectangular stress distribution in beams to make the calculations simple.
- The uniform stress is taken equal to $0.85f_c'$ and the distance along the depth of beam is taken equal to a, which is lesser than the depth of N.A. denoted by c.
- This is done to approximate the position of the resultant compressive force in the actual stress-strain curve with this idealized curve.
- The value of this dimension, a, is found such that the magnitude of the resultant force remains same in both actual and idealized curves.

$$f_{\mathrm{av}} \times b \times c \text{ (actual curve)} = f_{\mathrm{crush}} \times b \times a \text{ (idealized curve)}$$

$$a = \frac{f_{av}}{f_{crush}} \; c$$

$$= \beta_1 \, c \qquad \textit{(equal to 0.85 c} \text{ for } f_c' \leq \textit{ 30 MPa)}$$

STAGES IN DEVELOPING STRENGTH EXPRESSIONS

- The cross-section is drawn showing the geometric dimensions.
- Strain diagram is drawn at ultimate.
- Stress diagram is obtained as discussed earlier.
- Internal force resultants diagram is drawn.
- All further results are derived from these diagrams.

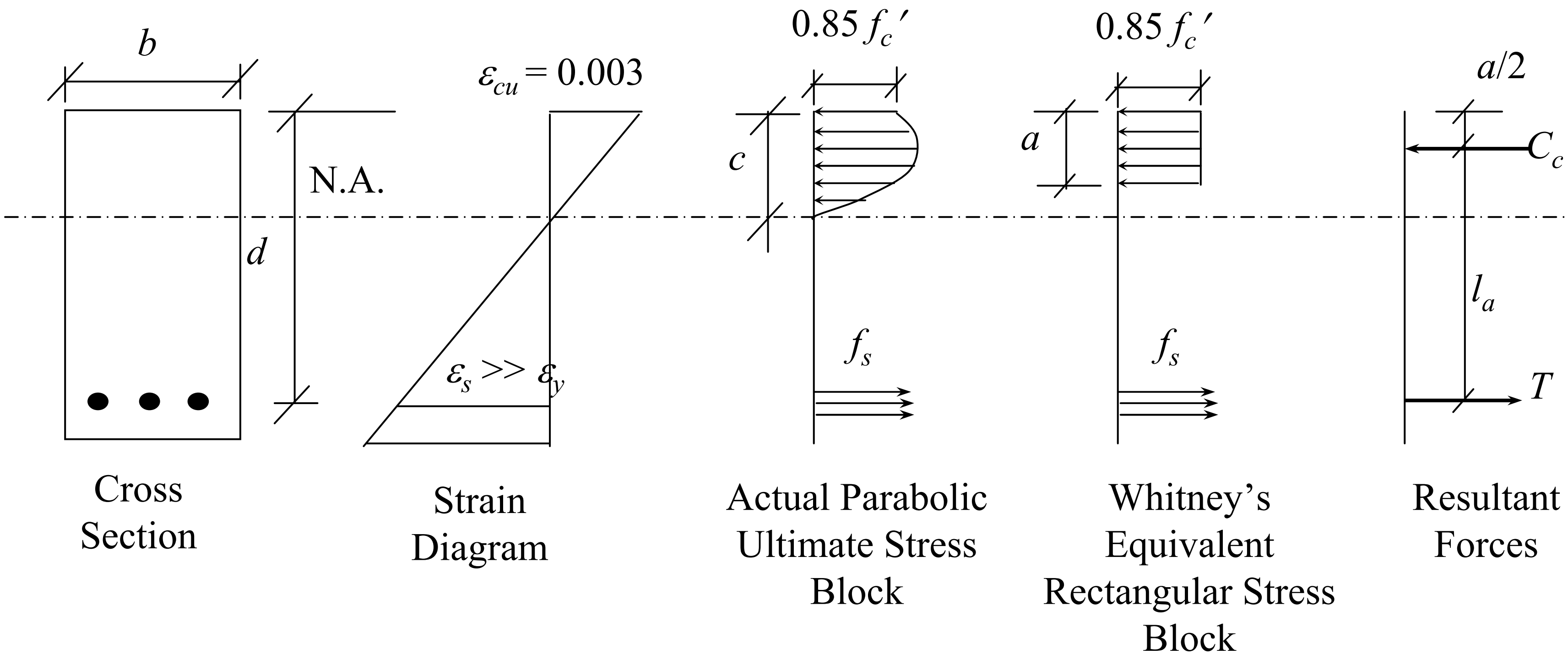


Fig. 3.4. Diagrams at Ultimate Conditions.

For actual parabolic stress block,

$$C_c = f_{av}\, bc$$
$$= 0.72\, f_c{}' bc$$

For equivalent rectangular stress block,

$$C_c = 0.85 f_c{}' ba$$

For both of these to be equal,

$$0.85 f_c{}' bc = 0.72\, f_c{}' ba$$

$$a = \frac{0.72}{0.85}\, c$$

$$a = 0.85\, c$$

$$a = \beta_1\, c \qquad or \qquad \beta_1 = a / c$$

β_1 = ratio of depth of N.A. to the depth of equivalent rectangular stress block

= 0.85 for $f_c' \leq 30$ MPa (4000 psi)
(decreases by 0.05 for every 7 MPa in excess of 30 MPa)

= $1.064 - 0.00714\ f_c' \leq 0.65$
for $f_c' > 30$ MPa (SI Units)

Note: Maximum value is 0.85 and minimum value is 0.65 in the above formulas.

TYPES OF SECTIONS DEPENDING ON FLEXURAL BEHAVIOR

- **Tension-Controlled Sections**

  These are the sections at which the net tensile strain in the extreme tension steel is equal to or greater than 0.005, when the corresponding concrete strain in compression just reaches a strain of 0.003.

$$\frac{c}{d_t} \leq \frac{3}{8}$$

- **Compression-Controlled Sections**

  These are the sections where the net tensile strain in the extreme tension steel is less than or equal to its yield strain (ε_y), when the corresponding concrete strain in compression just reaches a strain of 0.003.

  The value of ε_y may be taken equal to 0.002 for Grade-420 and all prestressing reinforcement.

$$\frac{c}{d_t} \geq \frac{0.003}{\varepsilon_y + 0.003} \qquad \text{(3/5 for Grade 420)}$$

- **Transition Sections**

  When the net tensile strain in the extreme tension steel is between the limiting values for the compression-controlled and the tension-controlled sections, the section behaves as a transition between the two main types of sections.

  The ϕ-factor is usually linearly varied for such sections for a smooth transition from compression-controlled to tension-controlled sections.

STRENGTH REDUCTION FACTORS (ϕ)

The strength reduction factor (also called resistance factor) is reciprocal of the minor part of the overall factor of safety that is applied on the strength of a member to obtain its design strength.

Tension-Controlled Sections

$\phi = 0.90$

Compression-Controlled Sections

a) Members with spiral reinforcement $\phi = 0.70$

b) Other reinforced members $\phi = 0.65$

Transition Sections

The ϕ-factor is linearly increased from compression-controlled ϕ-value to 0.90 as the section changes from compression-controlled to tension-controlled section.

Members with ties, $\phi = 0.65 + \dfrac{0.25}{0.005 - \varepsilon_y}[\varepsilon_t - \varepsilon_y]$

The value must be between 0.65 and 0.90.

For tension controlled section,

$$\rho_{\max} = 0.85\,\beta_1 \times \frac{3}{8} \times \frac{f'_c}{f_y}$$

$$A_{s,\min} = \frac{\sqrt{f'_c}}{4f_y} b_w d \geq \frac{1.4}{f_y} b_w d \quad \text{(SI Units)}$$

The second expression is critical when $f_c' \leq 31.4\ MPa$.

LATERAL LOADS

- Wind Load (Will be discussed here)
- Earthquake Load (details will be covered in another course)
- Water Retaining Load
- Earth Retaining Load
- Granular Material Load on Silos and Bunkers
- Dynamic Load of Retained Materials

WIND LOAD

- Wind load is produced due to change in momentum of an air current striking the surface of a building.
- A building is less likely to experience the other design loads in its life but it is almost certain that the building is likely to be subjected to the design wind loads.
- If the building is very tall, the wind velocity varies along the height and sophisticated codes account for this effect.

Basic wind speed is defined as the fastest wind speed in km/hr having a probability of occurrence of 0.02 and measured at a point 10m high above the ground under exposure category-C conditions, defined later.

3-sec gust wind speed is defined in the new specifications as the fastest wind speed prevailing for 3-sec in km/hr having a probability of occurrence of 0.02 and measured at a point 10m high above the ground under exposure category-C conditions.

Factors Affecting Wind Load

- Basic Wind Speed
- Gradient of wind velocity with height above ground.
- Local variations of pressure due to vortices.

The pressure is the highest at the corners, relatively high at the edges and low at the center of the building, as shown in Fig. 10.2.

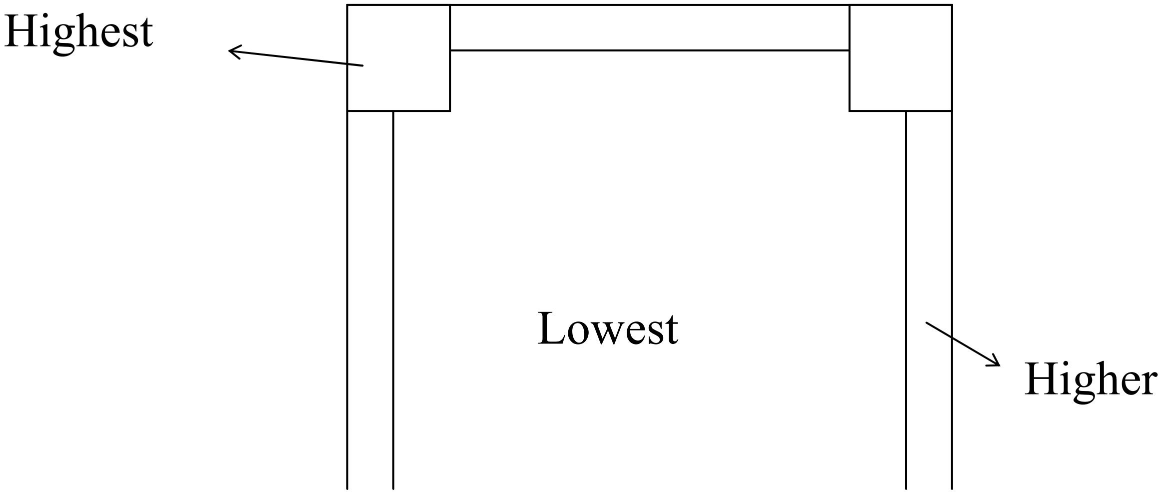


Fig. 10.2. Local Variation of wind Pressure.

- Exposure of the structure.

    For example, the coastal areas will have more wind loads.

    Buildings surrounded in other tall buildings will experience less wind pressures.

    The wind can be just a gust of wind or long wind periods.

There are three ***Exposure Categories*** defined in the UBC-97 Code.

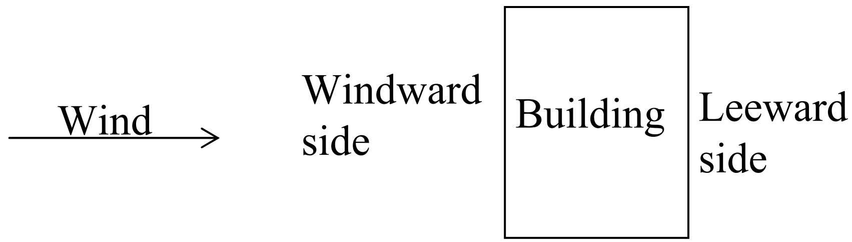
- ***Exposure B*** has terrain with buildings, forest or surface irregularities, covering at least 20 per cent of the ground level area and extending 1.6 km or more from the site.
- ***Exposure C*** has terrain that is flat and generally open, extending 0.8 km or more from the site in any full quadrant.
- ***Exposure D*** is the most severe exposure in areas with basic wind speeds of 129km/hr or greater and has terrain that is flat and unobstructed facing large bodies of water over 1.6 km in width relative to any quadrant of the building site.

  Exposure D extends towards the land from the shoreline 0.4 km or 10 times the building height, whichever is greater.

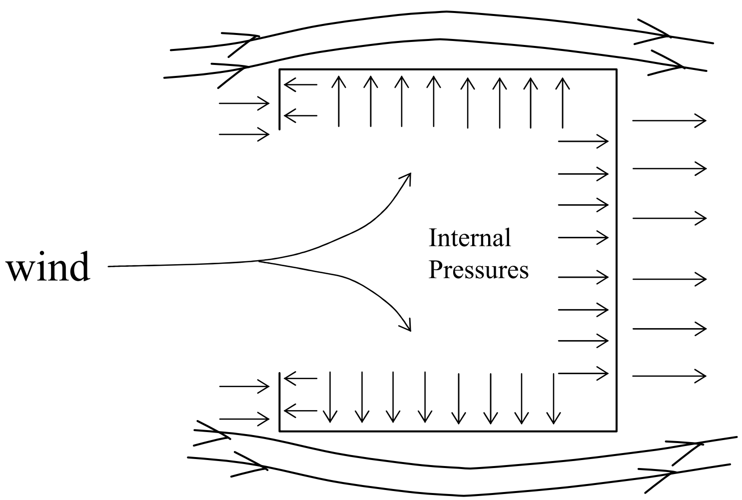
- **Internal pressure**. When the wind enters the building from the windward side and the leeward side is relatively closed, internal pressure is developed.

  This pressure acts like negative pressure or pressure acting away from the structure, as shown in Fig. 10.3.

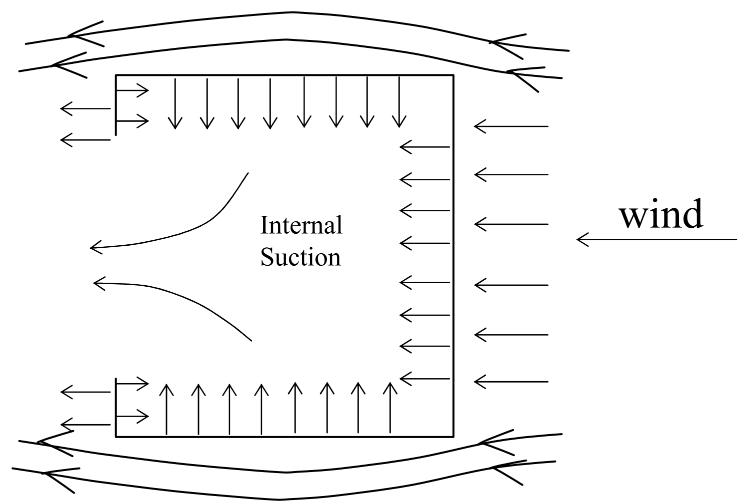
  Similarly, when high-speed wind passes by a building, it produces a vacuum on the leeward side. This vacuum results in internal suction producing negative pressure fro the structure.



(a) Windward and leeward sides.



(b) Internal pressure.



(c) Internal suction.

Fig. 10.3. Various Types of Negative Wind Pressures.

UBC-97 WIND LOADS

Wind load, $P \quad = \quad q_s \, A \, C_D \, I_w$

q_s	$=$	wind stagnation pressure
	$=$	$\frac{1}{2} \rho V^2$
	$\approx$	$0.0475 \, V^2$
ρ	$=$	air density
V	$=$	basic wind speed (km/hr)
A	$=$	effective exposed area

Drag Coefficient, $C_D = C_e \times C_q$

C_e = combined height, exposure and gust factor coefficient (Table 16-G of UBC)

C_q = pressure or shape factor coefficient for the structure or its portion under consideration (Table 16-H of UBC)

I_w = importance factor (Table 16-K of UBC)

= 1.15 for essential and hazardous facilities like hospitals, fire and police stations, disaster centers, water tanks and buildings with occupancy more than 300 people.

= 0.87 for buildings and other structures that represent a low hazard to human life in the even of failure, such as agricultural facilities.

= 1.0 for all other buildings.

Combined Height, Exposure And Gust Factor (C_e)

The values of this coefficient are given in Table 10.1.

Table 10.1. Values of Coefficient (C_e).			
Height Above Average Level of Adjoining Ground (m)		**Exposure C**	**Exposure B**
UBC	Approximate		
0 – 4.57	0 – 4.5	1.06	0.62
6.10	6	1.13	0.67
7.62	7.5	1.19	0.72
9.14	9	1.23	0.76
12.19	12	1.31	0.84
18.29	18	1.43	0.95
24.38	24	1.53	1.04
30.48	30	1.61	1.13
36.58	36	1.67	1.20
48.77	50	1.79	1.31
60.96	60	1.87	1.42
91.44	90	2.05	1.63
121.92	120	2.19	1.80

Pressure Coefficient (C_q)

The values of this coefficient for various parts of the building are given in Table 10.2.

Table 10.2. Pressure Coefficient (C_q).

Part of Structure	Angle	C_q
Windward roof	0° to 9.5° 9.5° to 37.0° 37° to 45° > 45°	0.7 outward 0.9 outward or 0.3 inward, which ever is more critical 0.4 inward 0.7 inward
Leeward or flat roof		0.7 outward
Windward walls		0.8 inward
Leeward walls		0.5 outward
Chimneys, tanks and solid towers square or rectangular hexagonal or octagonal round or elliptical		 1.4 any direction 1.1 any direction 0.8 any direction
Signs, flagpoles, light poles or minor structures		1.4 any direction
Roof eaves without overhangs	< 9.5° 9.5° to 30° 30°	2.3 upward 2.6 upward 1.6 upward
Overhangs at roof eaves and canopies		0.5 added to above values

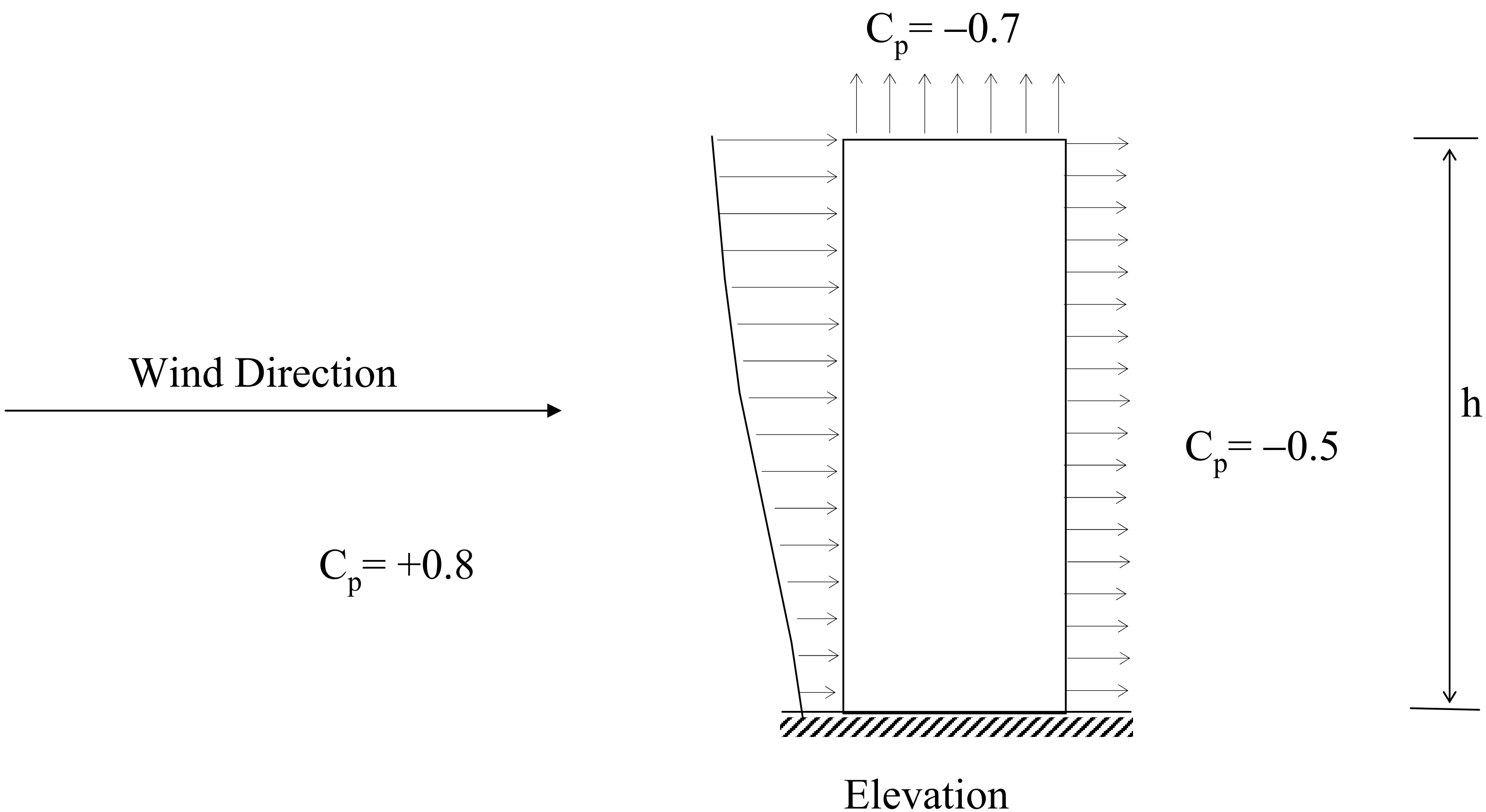


Fig. 10.4. Typical Values Of Pressure Coefficient.

Other UBC Wind Load Provisions

- Wind should be assumed to come from any horizontal direction. No reduction in wind pressure shall be taken for the shielding effect of adjacent structures.
- The base overturning moment for the entire structure, or for any one of its individual primary lateral resisting elements, should not exceed two thirds of the dead load resisting moment.

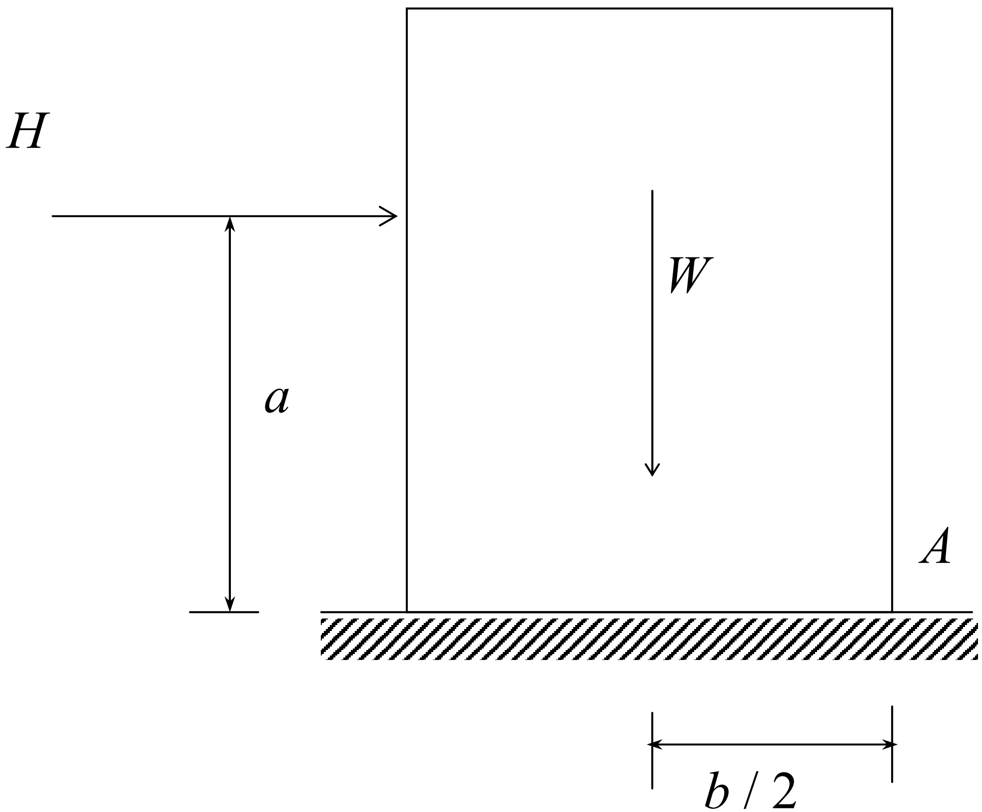


Fig. 10.5. Safety Against Overturning.

Referring to Fig. 10.5, the overturning of structure about point A may be investigated as follows:

Let, H = resultant horizontal wind load

a = height of resultant H from the base

W = resultant of dead loads of the structure

and b = width of the structure

M_w = overturning moment due to wind

$= H \times a$

- For pressures on roofs and leeward walls, C_e shall be evaluated at the mean roof height.
- A building structure or story shall be considered open when 15% or more of the area of the exterior wall on any one side is open (doors & windows, etc.)

Example 10.2: Determine the wind forces for an office building in downtown area with exposure type B. The basic wind speed for the area from charts is 160 km/h. The bay length is 5m and the frames, shown in Fig. 10.7, are at 7.5m at centers perpendicular to the plane of the frames.

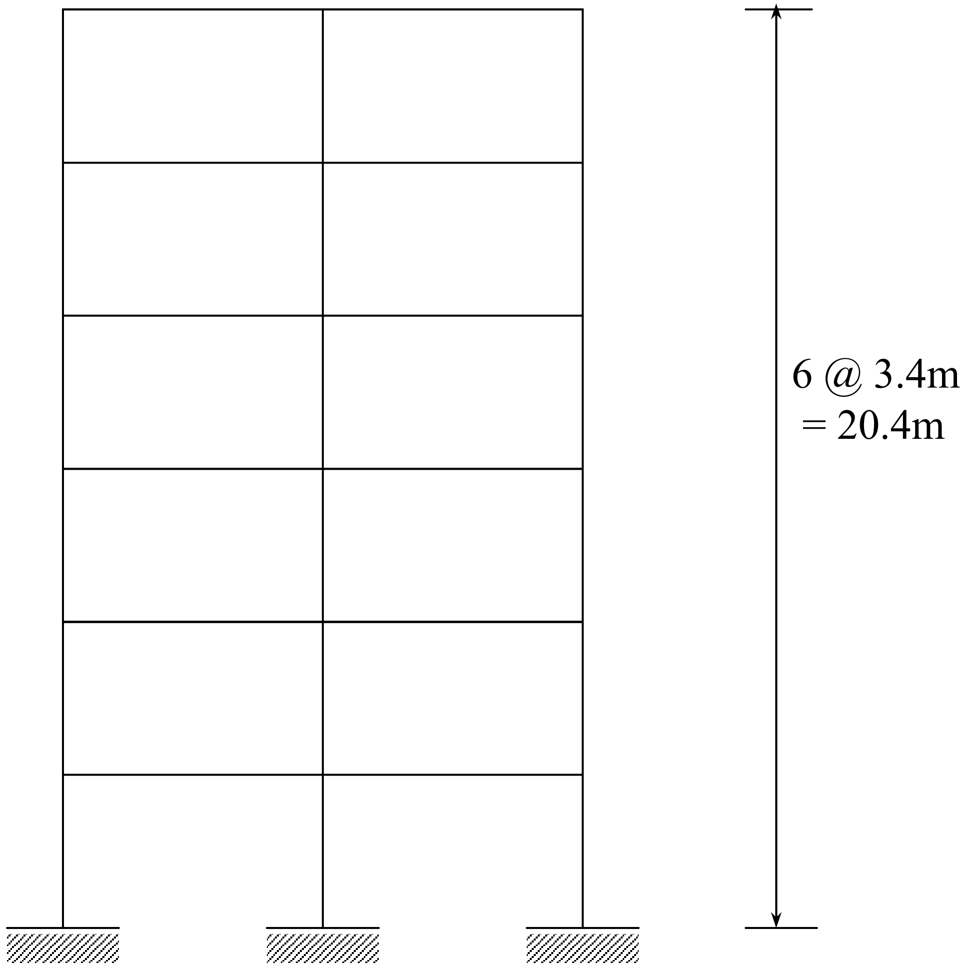


Fig. 10.7. Frame For Example 10.2.

Solution:

For office building: $I_w = 1.0$

Exposure category: B, as the building is downtown

Basic wind speed: 160 km/h

$$q_s = 0.0475\, V^2$$
$$= 0.0475(160)^2 = 1216 \text{ Pa}$$

$C_q = 0.7$ outward for the roof

The values of coefficients C_e and C_q at other locations are given in Table 10.3.

Table 10.3. The Coefficients C_e and C_q at Various Heights.

h **(m)**	**Average** C_e	C_q	
		Windward Wall	**Leeward Wall**
0 – 4.5	0.62	0.80	0.50
4.5 – 6.0	0.65	0.80	0.50
6.0 – 7.5	0.70	0.80	0.50
7.5 – 9.0	0.74	0.80	0.50
9.0 – 12.0	0.80	0.80	0.50
12.0 – 18.0	0.90	0.80	0.50
18.0 – 20.4	0.97	0.80	0.50
20.4	0.99	0.80	0.50

The pressures at various locations are calculated in Table 10.4 and are shown in Fig. 10. 8.

Table 10.4. Calculation of Wind Pressures.		
Pressure	**Calculation**	**Magnitude**
P1	(0.62)(0.8)(1216)(1.0)	603
P2	(0.65)(0.8)(1216)(1.0)	632
P3	(0.70)(0.8)(1216)(1.0)	681
P4	(0.74)(0.8)(1216)(1.0)	720
P5	(0.80)(0.8)(1216)(1.0)	778
P6	(0.90)(0.8)(1216)(1.0)	876
P7	(0.97)(0.8)(1216)(1.0)	944
P8	(0.99)(–0.5)(1216)(1.0)	- 602
P9	(0.99)(–0.7)(1216)(1.0)	- 843

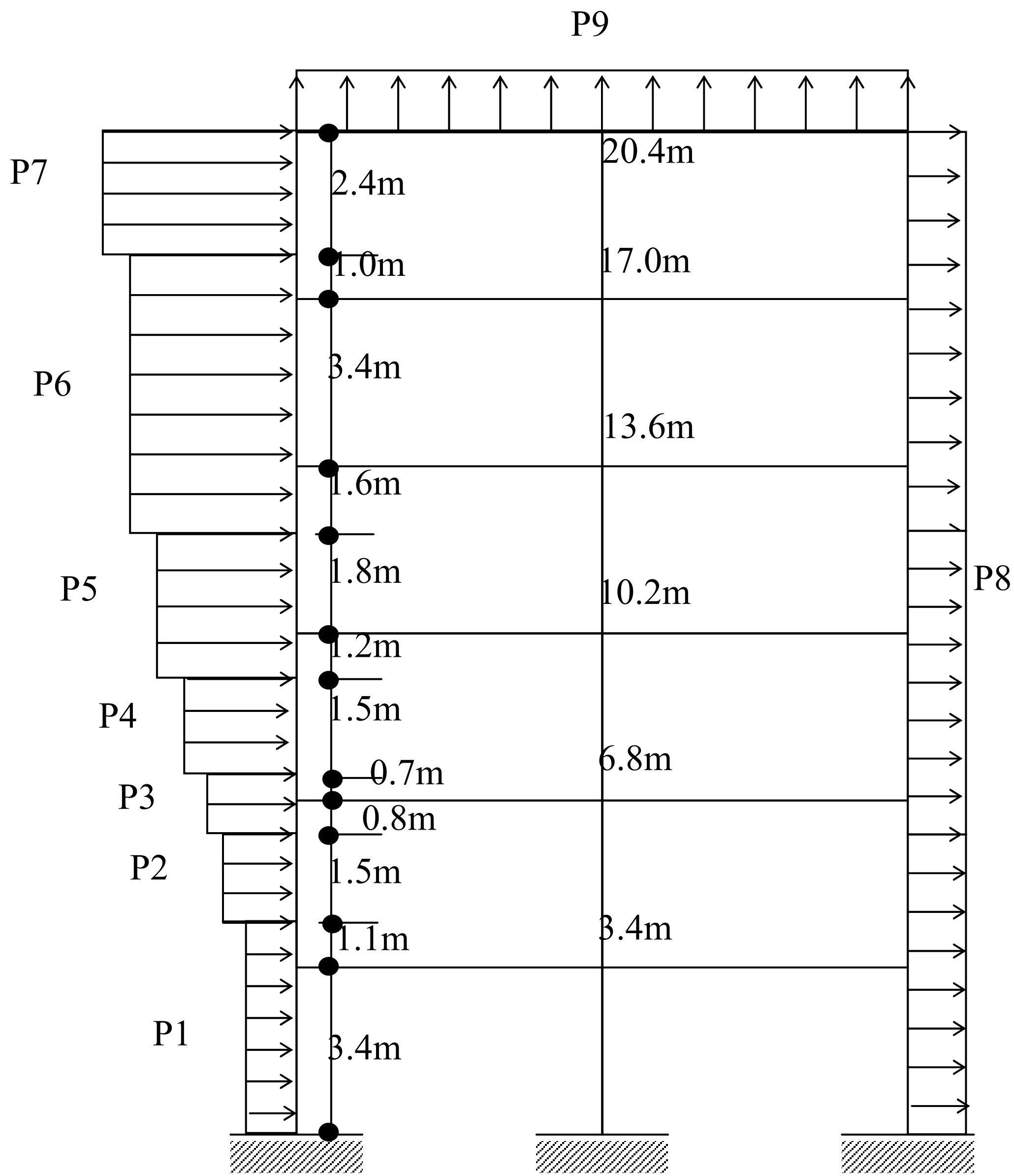


Fig. 10.8. Wind Pressures Acting on Structure of Example 10.2.

The lateral loads acting at the story levels are calculated in Table 10.5 and are shown in Fig. 10.9.

Table 10.5. Calculation of Wind Loads.

Force	Calculation	Magnitude
F1	(2.8)(7.5)(603) + (0.6)(7.5)(632) + (3.4)(7.5)(602)	30.86 kN
F2	(0.9)(7.5)(632) + (1.5)(7.5)(681) + (1.0)(7.5)(720) + (3.4)(7.5)(602)	32.68 kN
F3	(0.5)(7.5)(720) + (2.9)(7.5)(778) + (3.4)(7.5)(602)	34.97 kN
F4	(0.1)(7.5)(778) + (3.3)(7.5)(876) + (3.4)(7.5)(602)	37.62 kN
F5	(2.7)(7.5)(876) + (0.7)(7.5)(944) + (3.4)(7.5)(602)	38.05 kN
F6	(1.7)(7.5)(944) + (1.7)(7.5)(602)	19.71 kN
Roof Load	(7.5)(843)	6.32 kN/m

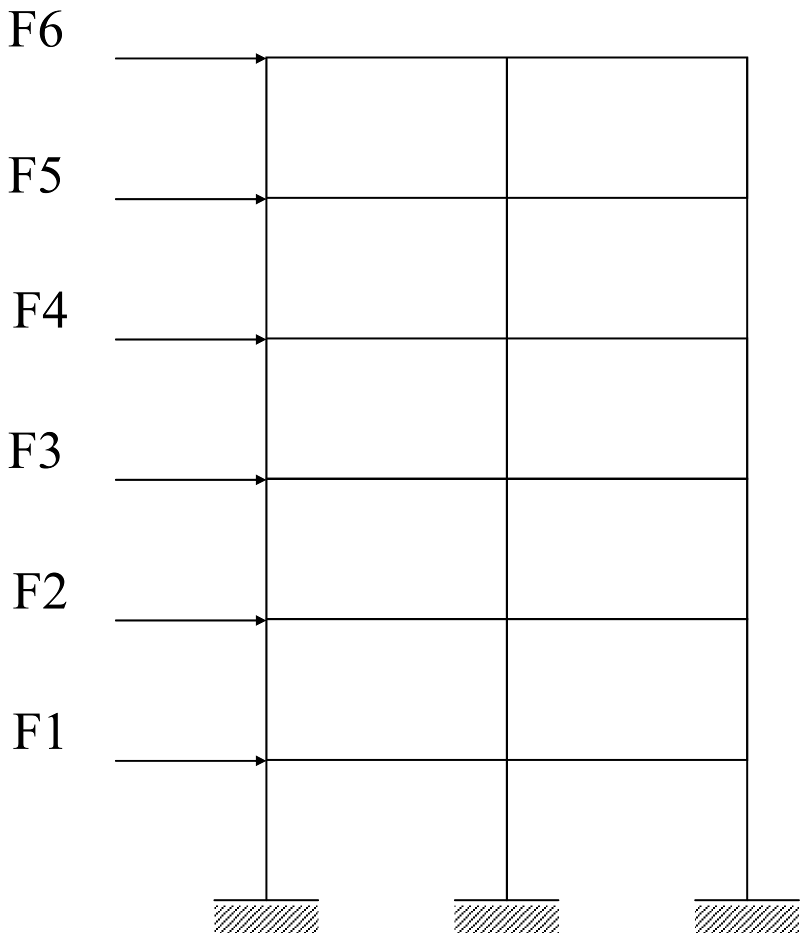


Fig. 10.9. Final Lateral Wind Loads For Example 10.2.

DYNAMIC LOADS

- These are the other main type of lateral loads.
- The values will be discussed in a separate course.
- However, these loads will also be included in this subject.

LOAD COMBINATIONS

- The various loads are combined according to their probability of occurrence together and the required overload factor for design.
- Following load combinations are suggested to be investigated by the ACI Code:

1- $U = 1.4\,(D + F)$

2- $U = 1.2\,(D + F + T) + 1.6\,(L + H)$

$+ 0.5\,(L_r \text{ or } S \text{ or } R)$

3- $U = 1.2\,D + 1.6\,(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or }$

$0.8W)$

4- $U = 1.2\,D + 1.6\,W + 1.0\,L$

$+ 0.5\,(L_r \text{ or } S \text{ or } R)$

5- $U = 1.2\,D + 1.0\,E + 1.0\,L + 0.2\,S$

6- $U = 0.9\,D + 1.6\,W + 1.6\,H$

7- $U = 0.9\,D + 1.0\,E + 1.6\,H$

where,

D = dead loads or related internal moments and forces,

F = loads due to fluid pressure or related internal moments and forces,

T = cumulative effect of temperature, creep, shrinkage and differential settlement,

L = live loads or related internal moments and forces,

H = loads due to soil pressure and pressure of water an in soil or related internal moments and forces,

L_r = roof live load or related internal moments and forces,

S = snow load or related internal moments and forces,

R = rain load or related internal moments and forces,

W = wind load or related internal moments and forces,

and

U = required strength to resist factored loads or related internal moments and forces.

- In general, the lateral loads are more critical if they produce moment reversal at the negative moment sections.
- The uplift acting on the roof may also cause moment reversal.
- ACI Code provisions for development of reinforcement indirectly assure that some strength is reserved for this moment reversal by requiring that at least 25 percent of the maximum positive reinforcement be extended and anchored at the support if the frame is part of the primary lateral load resisting system.

The following ACI provisions can be used to quickly and correctly analyze a frame:

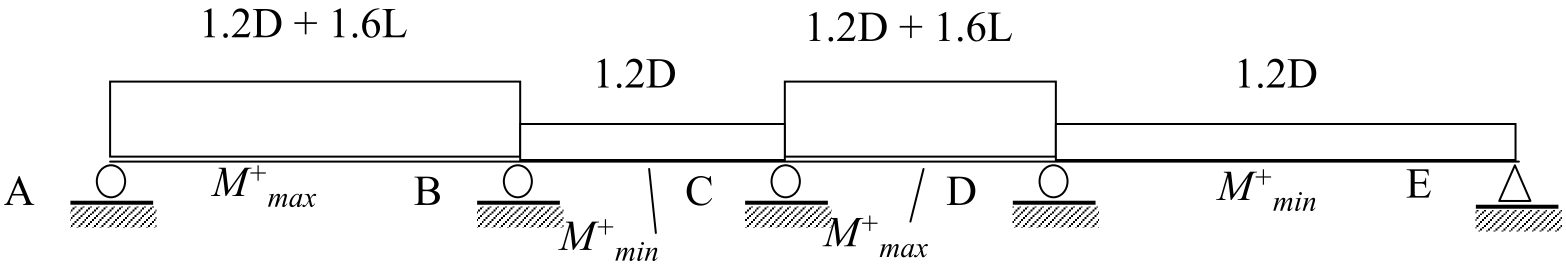
1.	Assume far ends of columns to be fixed.	ACI 8.9.1b
2.	Pattern loads are applied as specified.	ACI 8.9.2
3.	Analyze line representation of structure, and correct moments to face of supports.	ACI 8.7.2 and 8.7.3
4.	Any reasonable assumption for stiffness of members may be used, but must be consistent throughout the analysis.	ACI 8.6.1

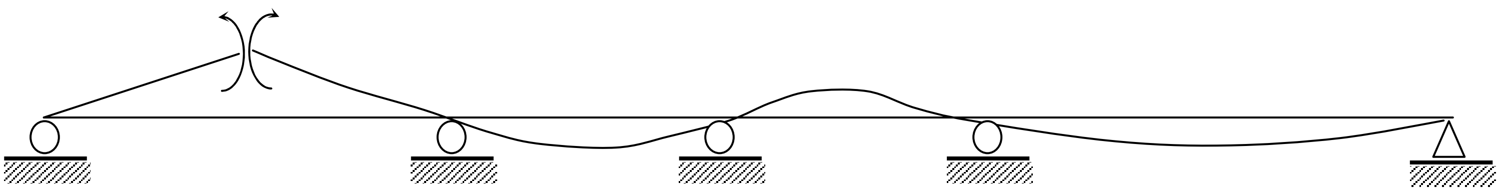
PATTERN LOADING

- The live load changes in magnitude as well as in direction.
- It is possible that some spans out of all the structure are loaded by live load.
- ***Pattern loading*** is that position of live load, which gives maximum force-effect at a particular section.

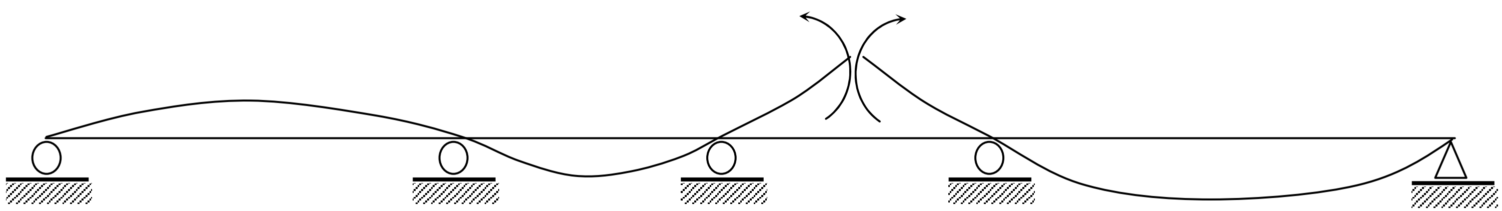
- According to ACI 8.9.2, it is allowed to consider only the following two arrangements of live loads:
- Factored dead load acts on all spans with full factored live load on two adjacent spans. This arrangement gives the maximum negative moment at the central support.
- Factored dead load acts on all spans with full factored live load on alternate spans. This arrangement gives the maximum positive moment within the fully loaded spans.

- The actual number of loading arrangements becomes greater when the adjacent two span loaded condition is applied for each support and when the alternate spans loaded condition is reversed to load those panels which were previously without any live load.
- The loading cases, given in Fig. 10.55, are to be considered for the indicated maximum moments in case of beams.

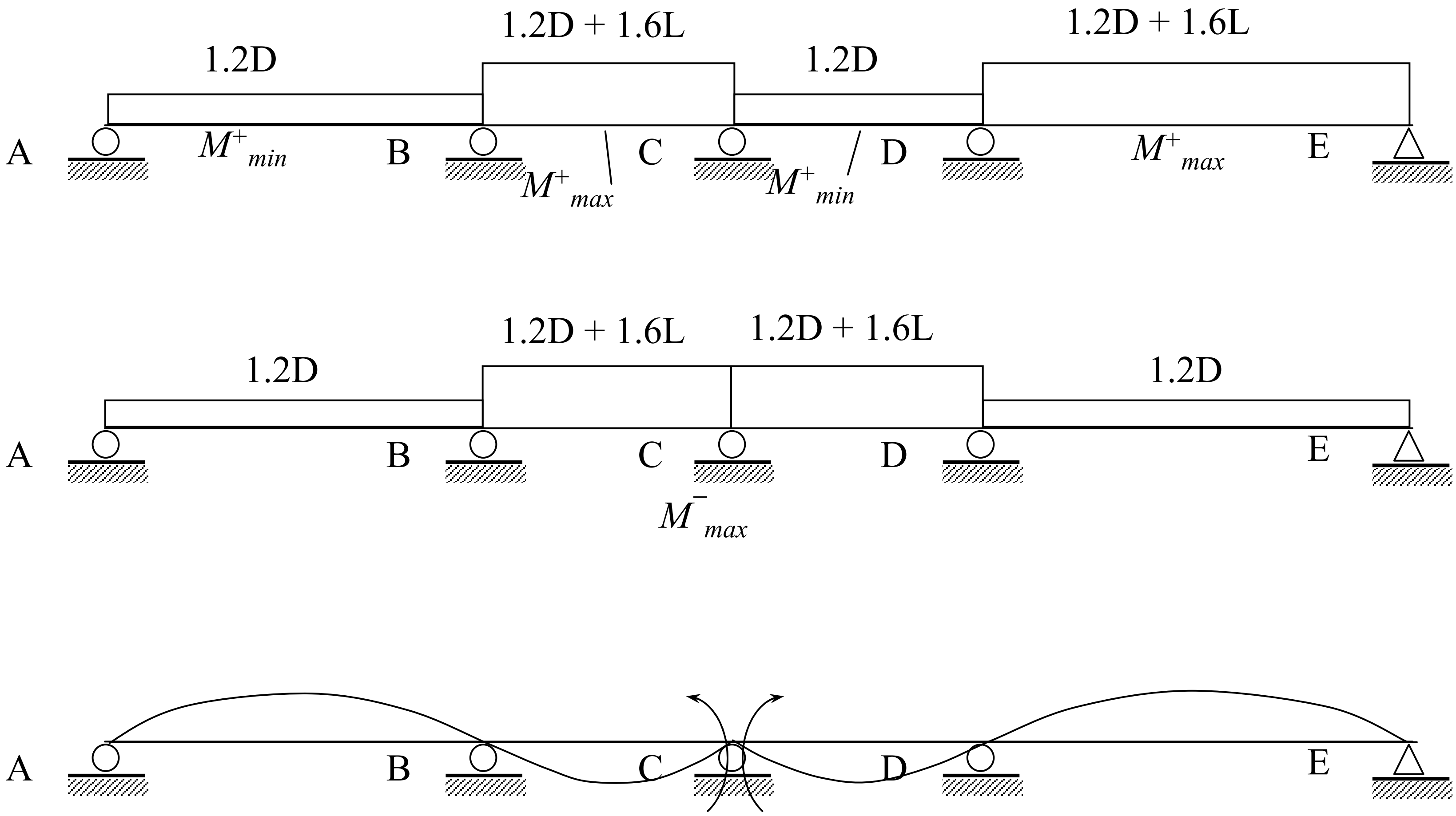




ILD For M^+_{AB}



ILD For M^+_{CD}


1.2D
1.2D + 1.6L
1.2D
1.2D + 1.6L
A
B
C
D
E
M^{+}_{min}
M^{+}_{max}
M^{+}_{min}
M^{+}_{max}
1.2D
1.2D + 1.6L
1.2D + 1.6L
1.2D
A
B
C
D
E
M^{-}_{max}
A
B
C
D
E

ILD For M^{-}_{C}

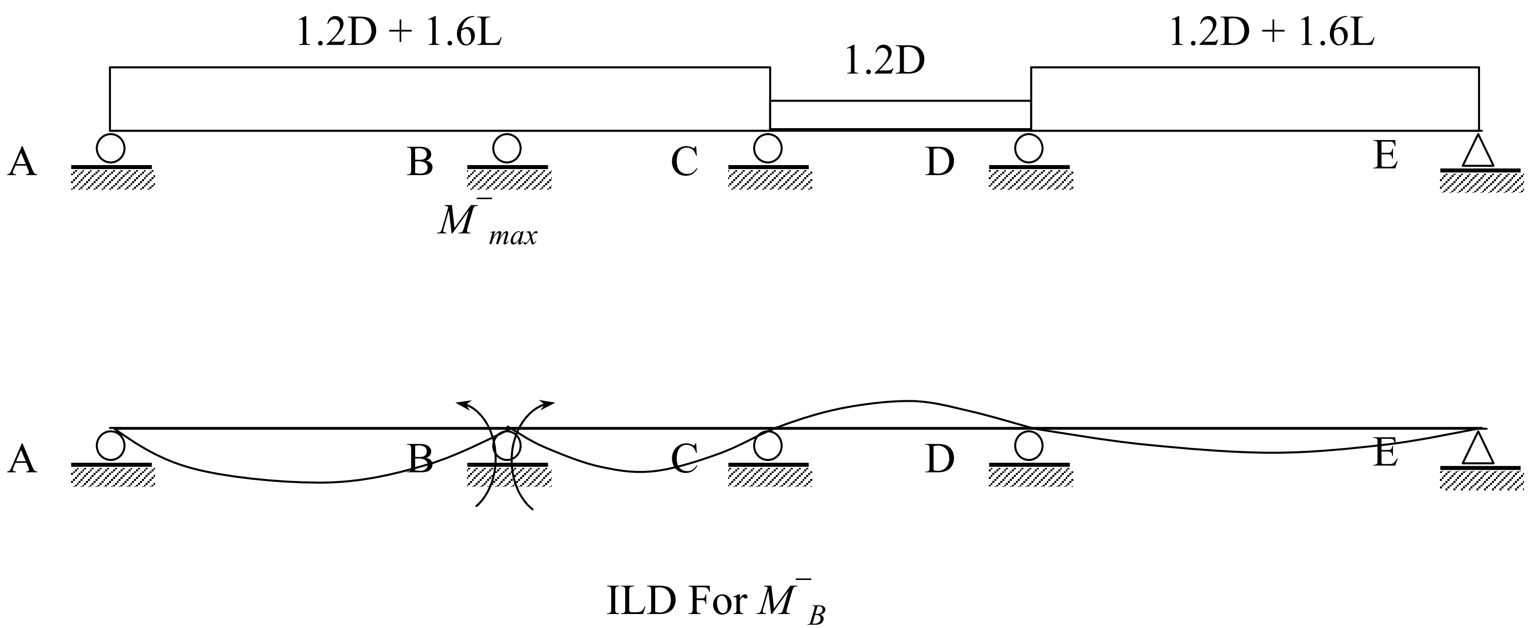
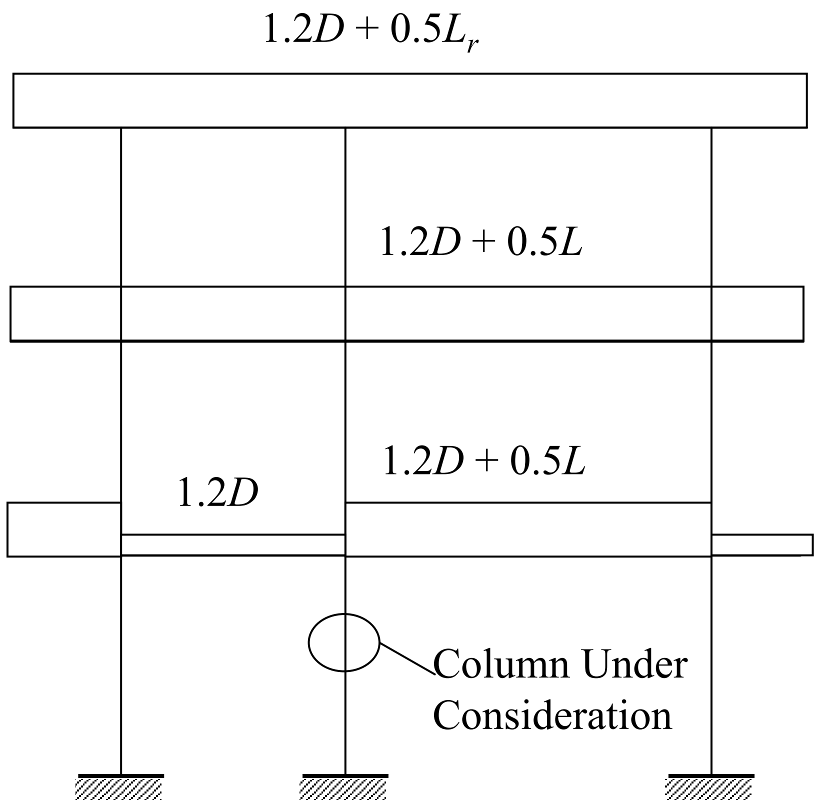
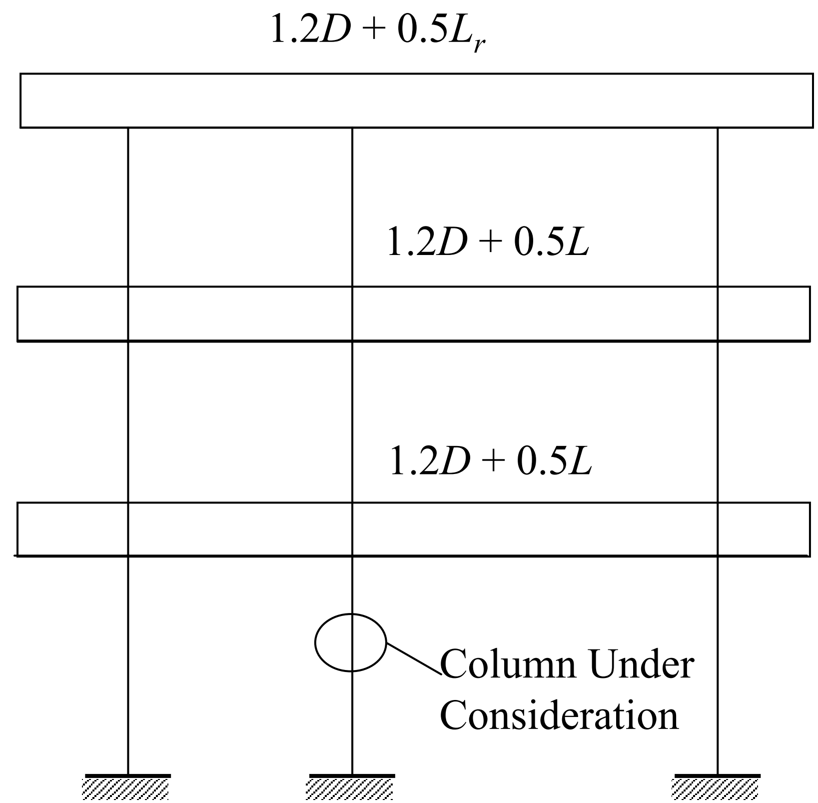


Fig. 10.55. Pattern Loading To Get Maximum Force Effects.

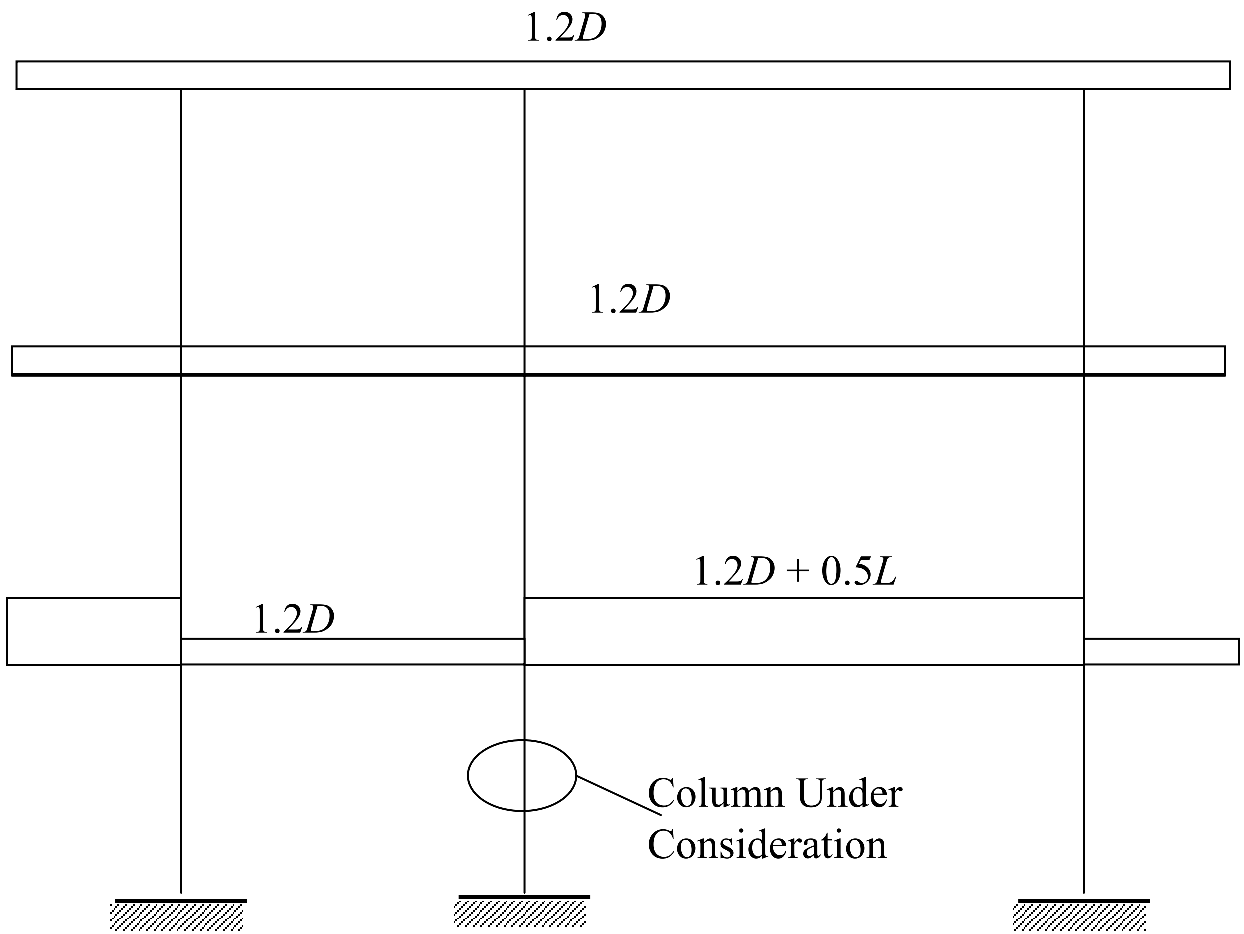
- It is to be noted that to get the influence line diagram for negative (hogging) moment, applied moment and rotation at the section are to be considered in the positive (sagging) direction.
- Three loading arrangements are to be checked for the relative magnitudes of axial load (P_u) and bending moment (M_u) in case of design of columns.
- These pattern loads are shown in Fig. 10.56.



a) Large P_u and Large M_u.



b) Large P_u and Less M_u.



c) Less P_u and Large M_u.

Fig. 10.56. Pattern Loading For Columns.

Continued on next file.