SOLVED EXAMPLES
To Enhance Understanding about
Use of Revised IS875 Pt.3 (Draft Code)

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Example-1: Wind Pressure and Forces on a Rectangular Clad Building: Flat Roof

Problem Statement:
Calculate wind pressures and design forces on walls and roof of a rectangular building having plan dimensions 10m×50m and height 5m, as shown in figure-1.1. The building is situated in Mohali (Chandigarh) in an upcoming Institutional complex on a fairly level topography. Walls of building have 20 openings of 1.5m×1.5m size. The building has a flat roof supported on load bearing walls.

Solution:
Wind Data:
1. Wind Zone: Zone IV ($V_b = 47$m/s) (IS:875-pt.3, Sec 5.2)
   (Refer Basic Wind Speed Map (Fig. 1)
2. Terrain category: Terrain Category 2 (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient Factor $k_1 = 1.00$ (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor $k_2 = 1.00$ (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography Factor $k_3 = 1.00$ (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $k_4 = 1.00$ (IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor $K_d = 0.90$ (IS:875-pt.3, Sec 6.2.1)
Area Averaging Factor $K_a$ (IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area of Short walls = 10 x 5 = 50$m^2$ => 0.867
Tributary area of Long walls = 50 x 5 = 250$m^2$ => 0.80
Tributary area of roof = 50 x 10 = 500$m^2$ => 0.80

Permeability of the Building:
Area of all the walls = 5 x (2×10 + 2×50) = 600$m^2$
Area of all the openings = 20×1.5×1.5 = 45$m^2$
% opening area = 7.5 %, between 5% and 20%
Hence the building is of Medium permeability. (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure:
Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00$ m/s (IS:875-pt.3, Sec 5.3)
$p_Z = 0.6 \times (47.00)^2 = 1325.4$ N/m$^2$ (IS:875-pt.3, Sec 6.2)
$p_d = p_Z \times K_d \times K_a = 1.3254\times 0.9 \times 0.867$
=1.034kN/m$^2$ (short wall)
= 1.3254\times 0.9 \times 0.8 = 0.954 kN/m$^2$ (long wall & Roof) (IS:875-pt.3, Sec 6.2)

Refer note below Sec. 5.3 for buildings less than 10m height, while making stability calculations & design of the framing.

Wind Load Calculations:
$F = (C_{pe} - C_{pi}) \times A \times p_d$ (IS:875-pt.3, Sec 6.3.1)
Internal Pressure Coefficient

\[ C_{pi} = \pm 0.5 \]  
(IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of –0.5 from inside (IS:875-pt.3, Sec 6.3.2.1) along with external pressure coefficient.

External Pressure Coefficients

On Roof: Using the Table 6 with roof angle 0° without local coefficients. For h/w = 0.5, pressure coefficients are tabulated below

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle 0°</th>
<th>Wind Incidence Angle 90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>-0.8</td>
<td>-0.8</td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
<td>-0.4</td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
<td>-0.8</td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
<td>-0.4</td>
</tr>
</tbody>
</table>

(Refer figure below Table 6 of IS:875-pt.3)

Design Pressure Coefficients for Roof:

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-versa. The combinations will have to be made separately for zones E, F, G, H etc. as shown in Fig. 1.2.

Design Pressure Coefficients for Walls:

For h/w = 0.5 and l/w = 5, \( C_{pe} \) for walls

\[
\text{Table – 1.2}
\]

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>-0.25</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>-0.6</td>
<td>+0.7</td>
</tr>
<tr>
<td>Wall – D</td>
<td>-0.6</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

(Refer Table 5 of IS:875-pt.3)

1. Since the pressure coefficients are given only for buildings with l/w ratio up to 4, for longer buildings i.e. l/w > 4, at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to \( C_{pi} = \pm 0.5 \)

\( C_{pnet} \) for Walls A or B

\[
= 0.7 - (-0.5) = +1.2, \text{ pressure} \\
= -0.5 - (+0.5) = -1.0, \text{ suction}
\]

\( C_{pnet} \) for Walls C or D

\[
= 0.7 - (-0.5) = +1.2, \text{ pressure} \\
= -0.6 - (+0.5) = -1.1, \text{ suction}
\]

Design pressures for walls:

For Long walls: \( F = C_{pnet} \times A_{net} \times p_d \)

\[
= +1.2 \times 1 \times 1 \times 0.954 = 1.1448 \text{ kN/m}^2 \text{ Pressure} \\
= -1.0 \times 1 \times 1 \times 0.954 = -0.954 \text{ kN/m}^2 \text{ Suction}
\]

For Short walls: \( F = C_{pnet} \times A_{net} \times p_d \)

\[
= +1.2 \times 1 \times 1 \times 1.034 = 1.2408 \text{ kN/m}^2 \text{ Pressure} \\
= -1.1 \times 1 \times 1 \times 1.034 = -1.1374 \text{ kN/m}^2 \text{ Suction}
\]

For Roof: \( F = C_{pnet} \times A_{net} \times p_d \)

\[
= +1.3 \times 1 \times 0.954 = 1.2402 \text{ kN/m}^2 \text{ Suction} \\
= -0.1 \times 1 \times 0.954 = -0.0954 \text{ kN/m}^2 \text{ Pressure}
\]

-----------------------------------------------
Figure 1.2- Net Roof Pressure Coefficients for different zones and combinations

For 0° wind incidence, for E/G (End Zone)

\[ E \uparrow 0.8 \downarrow 0.5 \quad OR \quad G \uparrow 0.4 \downarrow 0.5 = \]

\[ E \uparrow 0.3 \downarrow 0.1 \quad OR \quad G \uparrow 1.3 \downarrow 0.9 = \]

For 90° wind incidence, for E/G (End Zone)

\[ E \uparrow 0.8 \downarrow 0.5 \quad OR \quad G \uparrow 0.4 \downarrow 0.5 = \]

\[ E \uparrow 0.3 \downarrow 0.1 \quad OR \quad G \uparrow 1.3 \downarrow 0.9 = \]

For 0° wind incidence, for F/H (Mid Zone)

\[ F \uparrow 0.8 \downarrow 0.5 \quad OR \quad H \uparrow 0.4 \downarrow 0.5 = \]

\[ F \uparrow 0.3 \downarrow 0.1 \quad OR \quad H \uparrow 1.3 \downarrow 0.9 = \]

For 90° wind incidence, for F/H (Mid Zone)

\[ F \uparrow 0.4 \downarrow 0.5 \quad OR \quad H \uparrow 0.4 \downarrow 0.5 = \]

\[ F \uparrow 0.1 \downarrow \quad OR \quad H \uparrow 0.9 \downarrow = \]
Example-2: Wind Pressure and Forces on a Rectangular Clad Building with Parapet & Overhangs: Flat Roof

Problem Statement:
What difference will occur if the building in Ex.1 has 1.5m overhangs and 1m high parapets, as shown in figure 2.1?

Solution:

Wind Data:
1. Wind Zone: Zone IV ($V_b = 47$ m/s) (IS:875-pt.3, Sec 5.2)
   (Refer Basic Wind Speed Map (Fig. 1))
2. Terrain category: Terrain Category 2 (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient Factor $k_1 = 1.00$ (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor $k_2 = 1.00$ (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography Factor $k_3 = 1.00$ (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $k_4 = 1.00$ (IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor $K_d = 0.90$ (IS:875-pt.3, Sec 6.2.1)
Area Averaging Factor $K_a$ (IS:875-pt.3, Sec 6.2.2, Table-4)
Tributary area of Short walls = $10 \times (2 \times 10 + 2 \times 50) = 500\text{m}^2$ 
   $$\Rightarrow 0.867$$
Tributary area of Long walls = $50 \times 5 = 250\text{m}^2$
   $$\Rightarrow 0.80$$
Tributary area of roof = $53 \times 13 = 689\text{m}^2$
   $$\Rightarrow 0.80$$

Permeability of the Building:
Area of all the walls = $5 \times (2 \times 10 + 2 \times 50) = 600\text{m}^2$
Area of all the openings = $20 \times 1.5 \times 1.5 = 45\text{m}^2$

% opening area = 7.5%, between 5% and 20% Hence the building is of Medium permeability. (IS:875-pt.3, Sec.3.2.2)

Design Wind Pressure:
Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00$ m/s (IS:875-pt.3, Sec 5.3)

$p_Z = 0.6 \ (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{N/m}^2$ (IS:875-pt.3, Sec 6.2)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times 0.867$
   = $1.034 \text{kN/m}^2$ (short wall)
   = $1.3254 \times 0.9 \times 0.8$
   = $0.954 \text{kN/m}^2$ (long wall)
   (IS:875-pt.3, Sec 6.2)

$p_d = p_Z \cdot K_a \cdot K_a = 1.3254 \times 0.9 \times 0.8$
   = $0.954 \text{kN/m}^2$ (Roof)

Refer note below Sec. 5.3 for buildings less than 10m height, while making stability calculations & design of the framing.

Wind Load Calculations:
$$F = (C_{pi} \cdot C_{pu}) \times A \times p_d$$ (IS:875-pt.3, Sec 6.3.1)

Internal Pressure Coefficient $C_{pi}$ = ± 0.5 (IS:875-pt.3, Sec 6.3.2.2)
Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of –0.5
from inside (IS:875-pt.3, Sec.3.2.2) along-with external pressure coefficient.

**External Pressure Coefficients**

On Roof: Using the Table 5 with roof angle 0° without local coefficients. For h/w = 0.5, pressure coefficients are tabulated below

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>0°</td>
</tr>
<tr>
<td></td>
<td>-0.8</td>
</tr>
<tr>
<td></td>
<td>90°</td>
</tr>
<tr>
<td></td>
<td>-0.8</td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
</tr>
<tr>
<td></td>
<td>-0.4</td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
</tr>
<tr>
<td></td>
<td>-0.8</td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
</tr>
<tr>
<td></td>
<td>-0.4</td>
</tr>
</tbody>
</table>

*(Refer figure below Table 6 of IS:875-pt.3)*

**Design Pressure Coefficients for Roof:**

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-versa. The combinations will have to be made separately for zones E, F, G, H etc. as given in the fig. 2.2.

**Design Pressure Coefficients for Walls:**

For h/w = 0.5 and l/w = 5, C<sub>pnet</sub> for walls

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>Wall – A</th>
<th>Wall – B</th>
<th>Wall – C</th>
<th>Wall – D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°</td>
<td>+0.7</td>
<td>-0.25</td>
<td>-0.6</td>
<td>-0.6</td>
</tr>
<tr>
<td>90°</td>
<td>-0.5</td>
<td>-0.5</td>
<td>+0.7</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

*(Refer Table 5 of IS:875-pt.3)*

*: Since the pressure coefficients are given only for buildings with l/w ratio up to 4, for longer buildings i.e. l/w > 4, at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to C<sub>p</sub> = ± 0.5

<table>
<thead>
<tr>
<th>C&lt;sub&gt;pnet&lt;/sub&gt; for Walls A or B</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7 – (-0.5) = +1.2, pressure</td>
</tr>
<tr>
<td>-0.5 – (+0.5) = -1.0, suction</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>C&lt;sub&gt;pnet&lt;/sub&gt; for Walls C or D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7 – (-0.5) = +1.2, pressure</td>
</tr>
<tr>
<td>-0.6 – (+0.5) = -1.1, suction</td>
</tr>
</tbody>
</table>

**Pressure coefficients on overhanging portion of Roof:** (IS:875-pt.3, Sec 6.3.3.5)

On the top side of overhang: same as nearest top non-overhanging portion of roof i.e., -0.8 & -0.4

On the underside of overhang: since the overhang is horizontal, the max. Pressure coefficient shall be +1.0 (Section 6.3.3.5)

Therefore overhangs of this building shall be designed for a net upward wind pressure coefficient of -0.8 – (+1.0) = -1.8, i.e. suction, but with K<sub>a</sub> = 1.0, i.e. p<sub>d</sub> = 1.3254×0.9 = 1.193 kN/m<sup>2</sup>

Max. Design Force = 1.8×1.193 = 2.147 kN/m<sup>2</sup>
Figure 2.2- Net Roof Pressure Coefficients for different zones and combinations

For 0° wind incidence, for E/G (End Zone)

\[
\begin{align*}
\text{E} & \hspace{1cm} 0.8 \hspace{1cm} 0.4 \hspace{1cm} \text{G} \\
0.5 & \hspace{1cm} 0.5 \\
\]

= \hspace{1cm}

\[
\begin{align*}
\text{E} & \hspace{1cm} 0.3 \hspace{1cm} 0.1 \hspace{1cm} \text{G} \\
\end{align*}
\]

OR

\[
\begin{align*}
\text{E} & \hspace{1cm} 0.8 \hspace{1cm} 0.4 \hspace{1cm} \text{G} \\
0.5 & \hspace{1cm} 0.5 \\
\]

= \hspace{1cm}

\[
\begin{align*}
\text{E} & \hspace{1cm} 1.3 \hspace{1cm} 0.9 \hspace{1cm} \text{G} \\
\end{align*}
\]

For 90° wind incidence, for E/G (End Zone)

\[
\begin{align*}
\text{EG} & \hspace{1cm} 0.8 \hspace{1cm} 0.5 \\
\end{align*}
\]

= \hspace{1cm}

\[
\begin{align*}
\text{EG} & \hspace{1cm} 0.3 \\
\end{align*}
\]

OR

\[
\begin{align*}
\text{EG} & \hspace{1cm} 0.8 \hspace{1cm} 0.5 \\
\end{align*}
\]

= \hspace{1cm}

\[
\begin{align*}
\text{EG} & \hspace{1cm} 1.3 \\
\end{align*}
\]

For 0° wind incidence, for F/H (Mid Zone)

\[
\begin{align*}
\text{F} & \hspace{1cm} 0.8 \hspace{1cm} 0.4 \hspace{1cm} \text{H} \\
0.5 & \hspace{1cm} 0.5 \\
\]

= \hspace{1cm}

\[
\begin{align*}
\text{F} & \hspace{1cm} 0.3 \hspace{1cm} 0.1 \hspace{1cm} \text{H} \\
\end{align*}
\]

OR

\[
\begin{align*}
\text{F} & \hspace{1cm} 0.8 \hspace{1cm} 0.4 \hspace{1cm} \text{H} \\
0.5 & \hspace{1cm} 0.5 \\
\]

= \hspace{1cm}

\[
\begin{align*}
\text{F} & \hspace{1cm} 1.3 \hspace{1cm} 0.9 \hspace{1cm} \text{H} \\
\end{align*}
\]

For 90° wind incidence, for F/H (Mid Zone)

\[
\begin{align*}
\text{FH} & \hspace{1cm} 0.4 \hspace{1cm} 0.5 \\
\end{align*}
\]

= \hspace{1cm}

\[
\begin{align*}
\text{FH} & \hspace{1cm} 0.1 \\
\end{align*}
\]

OR

\[
\begin{align*}
\text{FH} & \hspace{1cm} 0.4 \hspace{1cm} 0.5 \\
\end{align*}
\]

= \hspace{1cm}

\[
\begin{align*}
\text{FH} & \hspace{1cm} 0.9 \\
\end{align*}
\]
Example 3: Wind Pressure and Forces on a Rectangular Clad Building: Taller with Flat Roof.

Problem Statement:
What difference will occur if the height of building in Ex.1 is 18m and it is to be used for a cold storage? The structure consists of RC column-beam frame at 5m/c horizontally and 3m/c vertically, supporting the wall. The Building has a flat roof with beams at 5m c/c. The building has 40 openings 1.5 m $\times$ 1.5m.

Fig. 3.1

Solution:
Wind Data:
1. Wind Zone: Zone IV ($V_b = 47$ m/s)  
   (IS:875-pt.3, Sec 5.2)  
   (Refer Basic Wind Speed Map (Fig. 1))
2. Terrain category: Terrain Category 2  
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient Factor $k_1 = 1.00$  
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor $k_2 = $ varying with height as in Table 3-1.  
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography Factor $k_3 = 1.00$  
(IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $k_4 = 1.00$  
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor $K_d = 0.90$  
(IS:875-pt.3, Sec 6.2.1)
Area Averaging Factor $K_a$  
(IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns in long as well as short walls $= 5 \times 18 = 90$ m$^2$ $\Rightarrow$ 0.813
Tributary area of roof beam $= 5 \times 10 = 50$ m$^2$  
$\Rightarrow$ 0.867

Combination factor $K_c$ is to be considered for the design of frames as per Section 6.3.3.13 and Table-20 of IS:875-pt.3.

Permeability of the Building:
Area of all the walls $= 18 \times (2 \times 10 + 2 \times 50) = 2160$ m$^2$
Area of all the openings $= 40 \times 1.5 \times 1.5 = 90$ m$^2$
% opening area = 4.166%, less than 5%.
Hence the building is of low permeability.  
(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure
Design Wind Speed $= V_Z = V_b \times k_1 \times k_2 \times k_3 \times K_d$  
$= 47 \times 1.0 \times k_3 \times 1.0 \times 1.0 = (47 \times k_3)$ m/s  
(IS:875-pt.3, Sec 5.3)
$p_Z = 0.6 \times (V_Z)^2$ & $p_{ul} = p_Z \times K_c \times K_d$  
(IS:875-pt.3, Sec 6.2 & Sec 6.2)
Table 3-1: Calculation of Variation in Design Wind Speed & Pressure with Height

<table>
<thead>
<tr>
<th>Height from Ground, m</th>
<th>$k_2$</th>
<th>$V_Z$ m/s</th>
<th>$p_Z$ kN/m$^2$</th>
<th>$p_d$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 10m</td>
<td>1.0</td>
<td>47</td>
<td>1.3254</td>
<td>0.970</td>
</tr>
<tr>
<td>15m</td>
<td>1.05</td>
<td>49.35</td>
<td>1.461</td>
<td>1.069</td>
</tr>
<tr>
<td>18m</td>
<td>1.068+</td>
<td>50.196</td>
<td>1.512</td>
<td>1.106</td>
</tr>
</tbody>
</table>

+ : linearly interpolated

Wind Load Calculations:

$$F = (C_{pe} - C_{pi}) \times A \times p_d$$  
(IS:875-pt.3, Sec 6.3.1)

Internal Pressure Coefficient

$$C_{pi} = \pm 0.2$$  
(IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.2 from inside and then for a suction of –0.2 from inside (refer IS:875-pt.3, Sec 6.2.2.1) along-with external pressure coefficient.

External Pressure Coefficient

On Roof: Using the Table 6 with roof angle 0° without local coefficients. For $h/w = 1.8$, pressure coefficients are tabulated below.

Table 3-2

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0°</td>
</tr>
<tr>
<td></td>
<td>90°</td>
</tr>
<tr>
<td>E</td>
<td>-0.7</td>
</tr>
<tr>
<td>F</td>
<td>-0.7</td>
</tr>
<tr>
<td>G</td>
<td>-0.6</td>
</tr>
<tr>
<td>H</td>
<td>-0.6</td>
</tr>
</tbody>
</table>

Design Pressure Coefficients for Roof:

$$p_d = p_Z \times K_a \times K_u = 0.6 \times (V_Z)^2 \times K_a \times K_u$$

$$= 0.6 \times (50.196)^2 \times 0.9 \times 0.867 = 1.180 \text{ kN/m}^2$$

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-versa. The combinations will have to be made separately for zones E, F, G, H etc. See fig. 3.2.

Design Pressure Coefficients for Walls:

Refer Table 5 of code: $h/w = 1.8$, and $l/w = 5$, therefore $C_{pe}$ for walls

Table 3-3

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>-0.4</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>-0.7</td>
<td>+0.8</td>
</tr>
<tr>
<td>Wall – D</td>
<td>-0.7</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

1: Since the pressure coefficients are given only for buildings with $l/w$ ratio up to 4, for longer buildings i.e. $l/w > 4$, at present values up to 4 are being used. These will be combined with internal pressure coefficients as earlier equal to $C_{pi} = \pm 0.2$

$$C_{pi}$$ for Walls A or B

$$= 0.7 - (-0.2) = +0.9, \text{ pressure}$$

$$= -0.5 - (+0.2) = -0.7, \text{ suction}$$

$$C_{pi}$$ for Walls C or D

$$= 0.8 - (-0.2) = +1.0, \text{ pressure}$$

$$= -0.7 - (+0.2) = -0.9, \text{ suction}$$

These $C_{pi}$ values multiplied by respective design pressure, depending on element & height give the design force per unit area, as in the previous example.
Figure 3.2- Net Roof Pressure Coefficients for different zones and combinations

For 0° wind incidence, for E/G (End Zone)

For 90° wind incidence, for E/G (End Zone)

For 0° wind incidence, for F/H (Mid Zone)

For 90° wind incidence, for F/H (Mid Zone)
Example 4: Wind Pressure and Forces on a Rectangular Clad Building: Pitched Roof

Problem Statement:

Calculate wind pressures and design forces on walls and roof of a rectangular clad building with pitched roof, having plan dimensions 10m×50m and height 5m, as shown in figure-4.1. The building is situated in Dhanbad (Bihar) in an industrial area 500m inside open land on a fairly level topography. Walls of building have 20 openings of 1.5m×1.5m size. The roof is of GC sheeting & the roof angle \( \alpha \) is 15°. Calculate also the local wind pressures on roof & wall cladding. The columns & trusses are at 5m c/c, longitudinally, purlins are at 1.4m c/c and columns at Gable ends are at 5m c/c.

Solution:

Wind Data:
Dhanbad is situated in Zone II at the boundary of Zone II & Zone IV. For such places higher Zone is recommended. Therefore, Wind Zone: Zone IV (\( V_b = 47 \) m/s) (IS:875-pt.3, Sec 5.2)

2. Terrain category: Transition from Category 1 to Category 2. (IS:875-pt.3, Sec 5.3.2.1)

Note: A combined wind speed profile is to be worked out as per Appendix–B (IS:875-pt.3, Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 5m, only effects due to Terrain Category 2 are to be considered. A combined profile would be needed in case of tall structures.

Design Factors:
Risk Coefficient factor \( k_1 = 1.00 \)
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor \( k_2 = 1.00 \)
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor \( k_3 = 1.00 \)
(IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region \( k_4 = 1.00 \)
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor \( K_d = 0.90 \)
(IS:875-pt.3, Sec 6.1.1)

Area averaging factor \( K_a \)
(IS:875-pt.3, Sec6.2.2, Table-4)

Tributary area for columns = 5 x 5 = 25 m²

\[
\text{Tributary area for Trusses} = 2 \times 5.176 \times 5 = 51.76 \text{m}^2
\]

\[
\text{Tributary area for Purlins} = 1.4 \times 5 = 7.0 \text{ m}^2
\]

\[
\text{Tributary area of short walls for design of wind braces in plan} = 10 \times 5 + 0.5 \times 10 \times 1.34 = 56.7 \text{ m}^2
\]

Permeability of the Building:
Area of all the walls = \( 5 \times (2 \times 10 + 2 \times 50) + 2 \times 0.5 \times 1.34 \times 10 = 613.4 \) m²

Area of all the openings = \( 20 \times 1.5 \times 1.5 = 45 \) m²

% opening area = \( 7.336 \) %, between 5% and 20%. Hence the building is of medium permeability. (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure:
Design Wind Speed = \( V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00 \text{ m/s} \)
(IS:875-pt.3, Sec 5.3)

\[
p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{ N/m}^2
\]
(IS:875-pt.3, Sec 5.4)
\[ p_d = p_x \times K_d \times K_a = 1.3254 \times 0.9 \times K_a = 1.193 K_a \]

(IS:875-pt.3, Sec 6.2)

For various members and components, use proper value of \( K_a \), as above. Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

**Wind Load Calculations:**
\[ F = (C_{pe} - C_{pi}) \times A \times p_d \]

(IS:875-pt.3, Sec 6.3.1)

**Internal Pressure Coefficient** \( C_{pi} = \pm 0.5 \)
(IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of \(-0.5\) from inside (refer IS:875-pt.3, Sec 6.2.2.1) along with external pressure coefficient.

**External Pressure Coefficients**
Using the Table 6 with roof angle 15°. For \( h/w = 0.5 \), pressure coefficients are tabulated in Table 4.1 (refer figure below Table 6 of IS:875-pt.3)

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
<th>( 0^\circ )</th>
<th>( 90^\circ )</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>-0.8</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
<td>-0.6</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
<td>-0.6</td>
<td></td>
</tr>
</tbody>
</table>

**Design Pressure Coefficients for Roof:**
Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to -ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for zones E, F, G, H etc., as given in figure 4.2.

**Design Pressure Coefficients for Walls:**
Refer Table 5 of the code: \( h/w = 0.5 \), and \( l/w = 5 \) therefore \( C_{pe} \) for walls

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>( 0^\circ )</th>
<th>( 90^\circ )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>-0.25</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>-0.6</td>
<td>+0.7</td>
</tr>
<tr>
<td>Wall – D</td>
<td>-0.6</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

\(^1\): Since the pressure coefficients are given only for buildings with \( l/w \) ratio up to 4, for longer buildings i.e. \( l/w > 4 \), at present values up to 4 are being used. These will be combined with internal pressure coefficients as earlier, equal to \( C_{pi} = \pm 0.5 \)

\( C_{net} \) for Walls A or B
\[ \begin{align*}
  &= 0.7 - (-0.5) = +1.2, \text{ pressure} \\
  &= -0.5 - (+0.5) = -1.0, \text{ suction}
\end{align*} \]

\( C_{net} \) for Walls C or D
\[ \begin{align*}
  &= 0.7 - (-0.5) = +1.2, \text{ pressure} \\
  &= -0.6 - (+0.5) = -1.1, \text{ suction}
\end{align*} \]

**Local pressure coefficients for the design of claddings and fasteners**
Refer Table 6 of IS-875 for Roof Angle = 15°
Local \( C_{pe} \) for eaves portion in end zone: NA
Local \( C_{pe} \) for eaves portion in mid zone: NA
Local \( C_{pe} \) for ridge portion: -1.2
Local \( C_{pe} \) for gable edges: -1.2
Local \( C_{pe} \) for corners of walls: -1.0

Therefore Max. local \( C_{net} \) for roof at the edges and the ridge = -1.2 - (+0.5) = -1.7
Likewise at the wall edges = -1.0 - (+0.5) = -1.5

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_a = 1 \).
Therefore, \( p_d = 1.3254 \times 0.9 = 1.193 \text{kN/m}^2 \)

Zone of local coefficients = 0.15 \times 10 = 1.5m, at ridges, eaves and gable ends & 0.25 \times 10 = 2.5m for wall corners. In this region the cladding and fasteners shall be checked for increased force.

Refer note below Table 6 of code

**Calculations of Force due to Frictional Drag:**
(IS:875-pt.3, Sec 6.4.1)
This will act in the longitudinal direction of the building along the wind. Here \( h/b \), therefore, first equation will be used & \( C_f' = 0.02 \). This will be added to the wind force on gable walls. \( K_a \) for roof and walls is 0.8, as area is more than 100m².
Figure 4.2 - Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

0.8  0.4
0.5  0.5  1.3  0.9

OR

0.8  0.4
0.5  0.5  0.3  0.1

For End Zone E/G; 90° wind incidence

0.75  0.75
0.5  0.5  1.25  1.25

OR

0.75  0.75
0.5  0.5  0.25  0.25

For Mid Zone F/H; 0° wind incidence

0.8  0.4
0.5  0.5  1.3  0.9

OR

0.8  0.4
0.5  0.5  0.3  0.1

For Mid Zone F/H; 90° wind incidence

0.6  0.6
0.5  0.5  1.1  1.1

OR

0.6  0.6
0.5  0.5  0.1  0.1
Example 5 - Wind Pressure and Forces on a Rectangular Clad Taller Building with Pitched Roof

Problem Statement:
What difference will occur if the height of the building in Example 4 is 18m and it has 40 openings of 1.5m×1.5m size as shown in figure 5.1?

Solution:
Wind Data:
1. Dhanbad is situated in Zone II at the boundary of Zone II & Zone IV. For such places higher Zone is recommended. Therefore, Wind Zone: Zone IV ($V_b = 47$ m/s) (IS:875-pt.3, Sec 5.2)
2. Terrain category:
   Transition from Category 1 (open land) to Category 2 (open land with few structures of low height) (IS:875-pt.3, Sec 5.3.2.1)
Note: A combined wind speed profile is to be worked out as per Appendix – B ((IS:875-pt.3, Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 18m, only effects due to Terrain Category 2 are to be considered. A combined profile would be needed in case of taller structures.

Design Factors:
Risk Coefficient factor $k_1 = 1.00$ (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor $k_2$: Varies with height, as given Table 5.1 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor $k_3 = 1.00$ (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $k_4 = 1.00$ (IS:875-pt.3, Sec 5.3.4)

Wind Directionality factor $K_d = 0.90$ (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor $K_a$: (IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns = $5 \times 18 = 90 \text{m}^2$

Tributary area for Trusses = $2 \times 5.176 \times 5 = 51.76 \text{m}^2$

Tributary area for Purlins = $1.4 \times 5 = 7.0 \text{m}^2$

Tributary area of short walls for design of wind braces in plan = $10 \times 18 + 0.5 \times 10 \times 1.34 = 186.7 \text{m}^2$

Permeability of the Building:
Area of all the walls = $18 \times (2 \times 10+2 \times 50) = 2160 \text{m}^2$
Area of all the openings = $40 \times 1.5 \times 1.5 = 90 \text{m}^2$
% opening area = 4.166 %, less than 5%
Hence the building is of low permeability. (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure
Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 0.9 = (47 \times 0.9) \text{ m/s}$ (IS:875-pt.3, Sec 5.3)
$p_Z = 0.6 \times (V_Z)^2$ & $p_d = p_Z \times K_d \times K_a$ (IS:875-pt.3, Sec 5.4 & Sec 6.2)
### Table 5.1
Calculations of variation in design wind speed & pressure with height

<table>
<thead>
<tr>
<th>Height from Ground, m</th>
<th>$k_v$</th>
<th>$V_z$ m/s</th>
<th>$p_z$ kN/m²</th>
<th>$p_d$ column, truss, purlin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 10m</td>
<td>1.00</td>
<td>47.00</td>
<td>1.325</td>
<td>0.970</td>
</tr>
<tr>
<td>15m</td>
<td>1.05</td>
<td>49.35</td>
<td>1.461</td>
<td>1.069</td>
</tr>
<tr>
<td>18m</td>
<td>1.07</td>
<td>50.20</td>
<td>1.512</td>
<td>1.106</td>
</tr>
</tbody>
</table>

Water Load Calculations:

$F = (C_{pe} - C_{pi}) \times A \times p_d$

(IS:875-pt.3, Sec 6.3.1)

### Table 5.2
Wind incidence angle

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
<th>Angle of Incidence</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>-0.75</td>
<td>0°</td>
</tr>
<tr>
<td>F</td>
<td>-0.75</td>
<td>90°</td>
</tr>
<tr>
<td>G</td>
<td>-0.6</td>
<td>90°</td>
</tr>
<tr>
<td>H</td>
<td>-0.6</td>
<td>90°</td>
</tr>
</tbody>
</table>

### Table 5.3
Angle of Incidence

<table>
<thead>
<tr>
<th>Wall</th>
<th>Angle of Incidence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall A</td>
<td>0°</td>
</tr>
<tr>
<td>Wall B</td>
<td>-0.4</td>
</tr>
<tr>
<td>Wall C</td>
<td>-0.7</td>
</tr>
<tr>
<td>Wall D</td>
<td>-0.7</td>
</tr>
</tbody>
</table>

These will be combined with internal pressure coefficients as earlier equal to $C_{pi} = \pm 0.2$.

<table>
<thead>
<tr>
<th>$C_{pnet}$ for Walls A or B</th>
<th>$C_{pnet}$ for Walls C or D</th>
</tr>
</thead>
<tbody>
<tr>
<td>$= 0.7 - (-0.2) = +0.9, pressure$</td>
<td>$= 0.8 - (-0.2) = +1.0, pressure$</td>
</tr>
<tr>
<td>$= -0.5 - (+0.2) = -0.7, suction$</td>
<td>$= -0.7 - (+0.2) = -0.9, suction$</td>
</tr>
</tbody>
</table>

### Local pressure coefficients for the design of claddings and fasteners

Refer Table 5 of IS-875 for wall and Table 6 for roof (Angle = 15°) For $h/w = 1.8$, pressure coefficients are tabulated in Table 5.2 (refer figure below Table 6 of code)

### Calculations of Force due to Frictional Drag

(IS:875-pt.3, Sec 6.4.1)

This will act in the longitudinal direction of the building along the wind. Here $h > b$, therefore, second equation will be used & $C_p' = 0.02$. This will be added to the wind force on gable walls. $K_a$ for roof and walls is 0.8, as area is more than 100m².
Figure 5-2: Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

For End Zone E/G; 90° wind incidence

For Mid Zone F/H; 0° wind incidence

For Mid Zone F/H; 90° wind incidence
Example 6 - Wind Pressure and Forces on a Rectangular Clad Pitched Roof Short Building in Coastal Region

Problem Statement:
What difference will occur if the building in Example 4 is an industrial building situated in Vishakhapatnam (Andhra Pradesh) near seacoast?

Solution:

Wind Data:

1. Wind Zone: Zone V ($V_b = 50 \text{ m/s}$) \(\rightarrow\) (IS:875-pt. 3, Sec 5.2)

   Note: Vishakhapatnam is situated near seacoast in Zone V. For such places special importance factor for cyclonic region is to be used. (IS:875-pt. 3, Fig. 1)

2. Terrain category: for open seacoast conditions, use Category 1 \(\rightarrow\) (IS:875-pt. 3, Sec 5.3.2.1)

Design Factors:

Risk Coefficient factor ‘\(k_1\)’ = 1.00 (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor ‘\(k_2\)’ = 1.05 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor ‘\(k_3\)’ = 1.00 (IS:875-pt.3,Sec 5.3.3.1)
Importance Factor for Cyclonic Region ‘\(k_4\)’
\[
= 1.15^* 
\]
(IS:875-pt.3, Sec 5.3.4)

Wind Directionality factor ‘\(K_d\)’ = 0.90 (IS:875-pt.3, Sec 6.2.1)

Area Averaging factor ‘\(K_a\)’
(IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns = 5 \times 5 = 25 \text{ m}^2 \Rightarrow 0.9
Tributary area for Trusses = 2 \times 5.176 \times 5 = 51.76 \text{ m}^2
\Rightarrow 0.864
Tributary area for Purlins = 1.4 \times 5 = 7.0 \text{ m}^2
\Rightarrow 1.0

\(\theta = 0^\circ\)

Fig. – 6.1

Tributary area of short walls for design of wind braces in plan = 10 \times 5 + 0.5 \times 10 \times 1.34 = 56.7 \text{ m}^2
\Rightarrow 0.858

*: use 1.15 for Industrial structures

Permeability of the Building:
Area of all the walls
= 5 \times (2 \times 10 + 2 \times 50) + 2 \times 0.5 \times 1.34 \times 10 = 613.4 \text{ m}^2

Area of all the openings
= 20 \times 1.5 \times 1.5 = 45 \text{ m}^2

% opening area = 7.336 %, between 5% and 20%.

Hence the building is of medium permeability. (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure

Design Wind Speed $\left( V_Z \right) = V_b \times k_1 \times k_2 \times k_3 \times k_4$
\[
= 50 \times 1.0 \times 1.05 \times 1.0 \times 1.15 = 60.375 \text{ m/s} 
\]
(IS:875-pt.3, Sec 5.3)

\[
p_Z = 0.6 \left( V_Z \right)^2 = 0.6 \times (60.375)^2 = 2187 \text{ N/m}^2 
\]
(IS:875-pt.3, Sec6.2)

\[
p_d = p_Z \times K_d \times K_a
= 2.187 \times 0.9 \times 0.858
= 1.689 \text{ kN/m}^2 \text{ (short wall)}
= 2.187 \times 0.9 \times 0.8
= 1.574 \text{ kN/m}^2 \text{ (long wall & Roof)} 
\]

(IS:875-pt.3, Sec 6.2)

Refer note below Sec. 5.3 for buildings less than 10m height, while making stability calculations and design of the frame.
Wind Load Calculations:

\[ F = (C_{pe} - C_{pi}) \times A \times p_d \]

(IS:875-pt.3, Sec 6.3.1)

Internal Pressure Coefficient \( C_{pi} = \pm 0.5 \)

(IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of -0.5 from inside along-with external pressure coefficient. (IS:875-pt.3, sec.6.3.2.1)

External Pressure Coefficients

Using the Table 6 with roof angle 15°

For \( h/w = 0.5 \), pressure coefficients are tabulated below (IS:875-pt.3, Table 6)

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>-0.8</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
<td>-0.6</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
<td>-0.6</td>
<td></td>
</tr>
</tbody>
</table>

Design Pressure Coefficients for Roof:

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for zones E, F, G, H etc., as given in figure 6.2

Design Pressure Coefficients for Walls:

Refer Table 5 of IS:875-pt.3 code: \( h/w = 0.5 \), and \( l/w = 5 \) therefore \( C_{pe} \) for walls

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>-0.25</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>-0.6</td>
<td>+0.7</td>
</tr>
<tr>
<td>Wall – D</td>
<td>-0.6</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

1: Since the pressure coefficients are given only for buildings with \( l/w \) ratio up to 4, for longer buildings i.e. \( l/w > 4 \), at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to \( C_{pi} = \pm 0.5 \)

\( C_{pnet} \) for Walls A or B = 0.7 – (-0.5) = +1.2, pressure

\( = -0.5 – (+0.5) = -1.0 \), suction

\( C_{pnet} \) for Walls C or D = 0.7 – (-0.5) = +1.2, pressure

\( = -0.6 – (+0.5) = -1.1 \), suction

Local pressure coefficients for the design of claddings and fasteners

Refer Table 6 of IS-875-pt.3 for Roof Angle = 15°

Local \( C_{pe} \) for eaves portion in end zone: NA

Local \( C_{pe} \) for eaves portion in mid zone: NA

Local \( C_{pe} \) for ridge portion: -1.2

Local \( C_{pe} \) for gable edges: -1.2

Local \( C_{pe} \) for corners of walls: -0.6

Therefore Max. local \( C_{pnet} \) for roof at the edges and the ridge = -1.2 – (+0.5) = -1.7

Likewise at the wall edges = -0.6 – (+0.5) = -1.1

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_a = 1 \). Therefore, \( p_d = 1.753 \times 0.9 = 1.5777 \) kN/m²

Zone of local coefficients = 0.15×10 = 1.5m, at ridges, eaves and gable ends & 0.25*10 = 2.5m for wall corners. In this region the cladding and fasteners shall be checked for increased force.

(IS:875-pt. 3, Table 6)

Calculations of Force due to Frictional Drag:

(IS:875-pt.3, Sec 6.4.1)

This will act in the longitudinal direction of the building along the wind. Here \( h>b \), therefore, first equation will be used & \( C_f' = 0.02 \). This will be added to the wind force on gable walls. \( K_a \) for roof and walls is 0.8, as area is more than 100m².
Figure 6.2 - Net Roof Pressure Coefficients for different zones and combinations
For End Zone E/G; $0^\circ$ wind incidence

For End Zone E/G; $90^\circ$ wind incidence

For Mid Zone F/H; $0^\circ$ wind incidence

For Mid Zone F/H; $90^\circ$ wind incidence
**Example 7 - Wind Pressure and Forces on a Rectangular Partially Clad Building: Pitched Roof**

**Problem Statement:**
What difference will occur if the walls of the building in Example 4 is half clad in upper part and half open as shown in figure 7.1?

**Solution:**

**Wind Data:**
Dhanbad is situated in Zone II at the boundary of Zone II & Zone IV. For such places higher Zone is recommended. Therefore, Wind Zone: Zone IV ($V_b = 47$ m/s) (IS:875-pt.3, Sec 5.2)

2. Terrain category: Transition from Category 1 to Category 2. (IS:875-pt.3, Sec 5.3.2.1)

Note: A combined wind speed profile is to be worked out as per Appendix – B (Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 5m, only effects due to Terrain Category 2 are to be considered. A combined profile would be needed in case of tall structures.

**Design Factors:**
- Risk Coefficient factor ‘$k_1$’ = 1.00 (IS:875-pt.3, Sec 5.3.1, Table-1)
- Terrain & Height factor ‘$k_2$’ = 1.00 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
- Topography factor ‘$k_3$’ = 1.00 (IS:875-pt.3, Sec 5.3.3.1)
- Importance Factor for Cyclonic Region ‘$k_4$’ = 1.00 (IS:875-pt.3, Sec 5.3.4)
- Wind Directionality factor ‘$K_d$’ = 0.90 (IS:875-pt.3, Sec 6.1.1)
- Area Averaging factor $K_a$ (IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for Trusses = $2 \times 5.176 \times 5 = 51.76 \text{m}^2$

---

Tributary area for Purlins = $1.4 \times 5 = 7.0 \text{m}^2$

---

Tributary area for short walls for design of wind braces in plan = $10 \times 2.5 + 0.5 \times 10 \times 1.34 = 31.7 \text{m}^2$

---

Permeability of the Building:
Since the walls are half open, the building comes under the category of large openings and analysis is to be carried out as per Section 6.3.2.2. As per para 1 of section 6.3.2.2 use more than 20% opening clause and consider ±0.7 internal pressure on walls and roof.

**Design Wind Pressure:**

Design Wind Speed $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00 \text{ m/s}$ (IS:875-pt.3, Sec 5.3)

$p_Z = 0.6 (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{ N/m}^2$ (IS:875-pt.3, Sec 6.2)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a$

$= 1.193 \times K_a \text{ kN/m}^2$ (IS:875-pt.3, Sec 6.2)

For various members and components, use proper value of $K_a$, as above. Refer note below Sec. 5.3 for buildings less than 10m height, while making stability calculations and design of the frame.

**Wind Load Calculations:**

$F = (C_{pe} - C_{pi}) \times A \times p_d = C_{pnet} \times A \times p_d$ (IS:875-pt.3, Sec 6.3.1)
**Internal Pressure Coefficient** $C_{pi} = \pm 0.7$

**External Pressure Coefficients:**
Using the Table 6 with roof angle $15^\circ$
For $h/w = 0.5$, pressure coefficients are tabulated below (refer figure below Table 6 of code)

**Table 7-1**

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>-0.8</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
<td>-0.6</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
<td>-0.6</td>
<td></td>
</tr>
</tbody>
</table>

**Design Pressure Coefficients for Roof:**
Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for zones E, F, G, H etc., as in figure 7.2.

**Design Pressure Coefficients for Walls:**
Refer Table 5 of code: $h/w = 0.5$, and $l/w = 5$
therefore $C_{pe}$ for walls*

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>-0.25</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>-0.6</td>
<td>+0.7</td>
</tr>
<tr>
<td>Wall – D</td>
<td>-0.6</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

*: Since the pressure coefficients are given only for buildings with $l/w$ ratio up to 4, for longer buildings i.e. $l/w > 4$, at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to $C_{pi} = \pm 0.7$

$C_{pnet}$ for Walls A or B = $0.7 - (-0.7) = +1.4$, pressure

$= 0.5 - (+0.7) = -1.2$, suction

$C_{pnet}$ for Walls C or D = $0.7 - (-0.7) = +1.4$, pressure

$= -0.6 - (+0.7) = -1.3$, suction

**Local pressure coefficients for the design of claddings and fasteners:**
Refer Table 6 of IS-875 for Roof Angle = $15^\circ$
Local $C_{pe}$ for eaves portion in end zone: NA
Local $C_{pe}$ for eaves portion in mid zone: NA
Local $C_{pe}$ for ridge portion: -1.2
Local $C_{pe}$ for gable edges: -1.2
Local $C_{pe}$ for corners of walls: -0.6

Therefore Max. local $C_{pnet}$ for roof at the edges and the ridge = $-1.2 - (+0.7) = -1.9$
Likewise at the wall edges = $-0.6 - (+0.7) = -1.3$

However, for the use of the local pressure coefficients, the design pressure $p_d$ will be computed with $K_a = 1$. Therefore, $p_d = 1.3254 \times 0.9 = 1.193$ kN/m$^2$

Zone of local coefficients = $0.15 \times 10 = 1.5m$, at ridges, eaves and gable ends & $0.25 \times 10 = 2.5m$ for wall corners. In this region the cladding and fasteners shall be checked for increased force.

(IS:875-pt.3, Table-5)

**Calculations of Force due to Frictional Drag:**
(IS:875-pt.3, Sec 6.4.1)
This will act in the longitudinal direction of the building along the wind. Here $h < b$, therefore, first equation will be used & $C_f = 0.02$. This will be added to the wind force on gable walls. $K_a$ for roof and walls is 0.8, as area is more than 100m$^2$. 
Figure 7.2 - Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

\[ 0.8 \quad 0.4 \]
\[ 0.7 \quad 0.7 \]
\[ = \]
\[ 1.5 \quad 1.1 \]
\[ OR \]
\[ 0.8 \quad 0.4 \]
\[ 0.7 \quad 0.7 \]
\[ = \]
\[ 0.1 \quad 0.3 \]

For End Zone E/G; 90° wind incidence

\[ 0.75 \quad 0.75 \]
\[ 0.7 \quad 0.7 \]
\[ = \]
\[ 1.45 \quad 1.45 \]
\[ OR \]
\[ 0.75 \quad 0.75 \]
\[ 0.7 \quad 0.7 \]
\[ = \]
\[ 0.05 \quad 0.05 \]

For Mid Zone F/H; 0° wind incidence

\[ 0.8 \quad 0.4 \]
\[ 0.7 \quad 0.7 \]
\[ = \]
\[ 1.5 \quad 1.1 \]
\[ OR \]
\[ 0.8 \quad 0.4 \]
\[ 0.7 \quad 0.7 \]
\[ = \]
\[ 0.1 \quad 0.3 \]

For Mid Zone F/H; 90° wind incidence

\[ 0.6 \quad 0.6 \]
\[ 0.7 \quad 0.7 \]
\[ = \]
\[ 1.3 \quad 1.3 \]
OR

0.6

0.7

0.6

0.7

= 

0.1

0.1
Example 8 - Wind Pressure and Forces on a Rectangular Clad Building: Mono-slope Roof

Problem Statement:
What difference will occur in design forces if the building in Example 4 has a mono slope roof with roof angle $\alpha = 10^0$, the eaves height at the lower end being 5m? The building has 1 m wide overhangs at both the eaves. See figure 8.1.

Solution:
Wind Data:
Dhanbad is situated in Zone II at the boundary of Zone II & Zone IV. For such places higher Zone is recommended. Therefore,
Wind Zone: Zone IV ($V_b = 47$ m/s) (IS:875-pt.3, Sec 5.2)
2. Terrain category: Transition from Category 1 to Category 2. (IS:875-pt.3, Sec 5.3.2.1)
Note: A combined wind speed profile is to be worked out as per Appendix – B (IS:875-pt.3, Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 5m, only effects due to Terrain Category 2 are to be considered. A combined profile would be needed in case of tall structures.

Design Factors:
Risk Coefficient factor ‘$k_1$‘ = 1.00 (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor ‘$k_2$‘ = 1.00 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor ‘$k_3$‘ = 1.00 (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region ‘$k_4$‘ = 1.00 (IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor ‘$K_d$‘ = 0.90 (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor $K_a$ = (IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns = $5 \times 6.76 = 33.81 \text{ m}^2$
$\Rightarrow 0.888$
Tributary area for Trusses = $12.19 \times 5 = 60.95 \text{ m}^2$
$\Rightarrow 0.852$
Tributary area for Purlins = $1.4 \times 5 = 7.0 \text{ m}^2$
$\Rightarrow 1.0$

Tributary area of short walls for design of wind braces in plan
= $10 \times 5 + 0.5 \times 10 \times 1.76 = 58.8 \text{ m}^2 \Rightarrow 0.855$

Permeability of the Building:
Area of all the walls = $5(2 \times 10 + 2 \times 50) + 0.5 \times 1.76 \times 10 + 1.76 \times 50 = 696.8 \text{ m}^2$
Area of all the openings = $20 \times 1.5 \times 1.5 = 45 \text{ m}^2$
% opening area = 6.458%, between 5% and 20%
Hence the building is of medium permeability (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure
Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$
= $47 \times 1.0 \times 1.0 \times 1.0 = 47.00 \text{ m/s}$ (IS:875-pt.3, Sec 5.3)

$p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{ N/m}^2$
(IS:875-pt.3, Sec 6.2)

$p_a = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a$
$= 1.193 \times K_a \text{ kN/m}^2$ (IS:875-pt.3, Sec 6.2)
For various members and components, use proper value of $K_a$, as above. Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

Wind Load Calculations:

\[ F = (C_{pe}C_{pi}) \times A \times p_d \]

(IS:875-pt.3, Sec 6.3.1)

### External Pressure Coefficients

Using the Table 7 with roof angle 10°

For $h/w = 5/12 = 0.417$, pressure coefficients are tabulated in Table 8-1 (IS:875-pt.3, Table 7)

Overhang portion: same as local coefficient on nearest non-overhang portion, i.e. –2.0

(IS:875-pt.3, Sec 6.3.3.5)

### Design Pressure Coefficients for Roof:

#### Table 8-1

<table>
<thead>
<tr>
<th>Portion of Roof</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0°</td>
</tr>
<tr>
<td>Windward (widthwise left half)</td>
<td>-1.0</td>
</tr>
<tr>
<td>Leeward (widthwise right half)</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for different zones, as given in Figure 8-2.

### Design Pressure Coefficients for Walls:

Refer Table 5 of code: $h/w = 5/10 = 0.5$, and $l/w = 50/10 = 5$ therefore $C_{pe}$ for walls* are given in Table 8-2.

#### Table 8-2

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>-0.25</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>-0.6</td>
<td>+0.7</td>
</tr>
<tr>
<td>Wall – D</td>
<td>-0.6</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

*: Since the pressure coefficients are given only for buildings with $l/w$ ratio up to 4, for longer buildings i.e. $l/w > 4$, at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to $C_{pi} = \pm 0.5$

$C_{pwa}$ for Walls A or B $= 0.7 - (-0.5) = +1.2$, pressure $= -0.5 - (+0.5) = -1.0$, suction $C_{pwa}$ for Walls C or D $= 0.7 - (-0.5) = +1.2$, pressure $= -0.6 - (+0.5) = -1.1$, suction

#### Local pressure coefficients:

Local coefficients for roof: Max. value from all the values given in Table 7 of IS:875-pt.3, i.e.\{-2.0\} is \{-2.0\}, up to 0.15 x w = 0.15 x 12 = 1.8m on all edges of roof.

Local coefficients for walls: From Table 5 of the IS:875-pt.3, it is \{-1.0\} for a distance of 0.25 x w = 0.25 x 10 = 2.5m at all corners.

### Calculations of Force due to Frictional Drag:

(IS:875-pt.3, Sec 6.4.1)

This will act in the longitudinal direction of the building along the wind. Here $h/b$, therefore, first equation will be used & $C_{f} = 0.02$. This will be added to the wind force on gable walls. $K_a$ for roof and long walls is 0.8, as area is more than 100m².
Figure 8.2 - Net Roof Pressure Coefficients for different zones and combinations

For 0° wind incidence, $C_{pi} = +0.5$

\[
\begin{array}{cccccccc}
2.0^* & 2.0^* & 1.0 & 0.5 & 2.0^* & 2.0^* \\
0.75 & 0.5 & & & & 0.25 \\
\end{array}
\]

Which is equivalent to ↓

\[
\begin{array}{cccccccc}
2.75 & 2.5 & 1.5 & 1.0 & 2.5 & 1.75 \\
\end{array}
\]

For 0° wind incidence, $C_{pi} = -0.5$

\[
\begin{array}{cccccccc}
2.0^* & 2.0^* & 1.0 & 0.5 & & 2.0^* & 2.0^* \\
0.75 & 0.5 & & & & 0.25 \\
\end{array}
\]

Which is equivalent to ↓

\[
\begin{array}{cccccccc}
2.75 & 1.5 & 0.5 & 0.0 & 1.5 & 1.75 \\
\end{array}
\]

For 90° wind incidence, $C_{pi} = +0.5$, up to $w/2$ from ends

\[
\begin{array}{cccccccc}
2.0^* & 2.0^* & 1.0 & 1.0 & 2.0^* & 2.0^* \\
0.5 & 0.5 & & & & 0.5 \\
\end{array}
\]

Which is equivalent to ↓

\[
\begin{array}{cccccccc}
1.5 & 2.5 & 1.5 & 1.5 & 2.5 & 1.5 \\
\end{array}
\]

And similarly for other combinations.

* These are local pressure coefficients
Example 9 - Wind Pressure and Forces on a Rectangular Clad Open Building: Mono-slope Roof

Problem Statement:
What change will occur if the building in Example 8 is open at the higher end as shown in figure 9.1, and is without overhangs?

![Diagram of a building with dimensions 5 m and 50 m]

Solution:
Wind Data:
Dhanbad is situated in Zone II at the boundary of Zone II & Zone IV. For such places higher Zone is recommended. Therefore, Wind Zone: Zone IV ($V_b = 47$ m/s) (IS:875-pt.3, Sec 5.2)

2. Terrain category: Transition from Category 1 to Category 2. (IS:875-pt.3, Sec 5.3.2.1)

Note: A combined wind speed profile is to be worked out as per Appendix – B (Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 5m, only effects due to Terrain Category 2 are to be considered. A combined profile would be needed in case of tall structures.

Design Factors:
Risk Coefficient factor ‘$k_1$’ = 1.00 (IS:875-pt.3, Sec 5.3.1, Table-1)
 Terrain & Height factor ‘$k_2$’ = 1.00 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
 Topography factor ‘$k_3$’ = 1.00 (IS:875-pt.3, Sec 5.3.3.1)
 Importance Factor for Cyclonic Region ‘$k_4$’=1.00 (IS:875-pt.3, Sec 5.3.4)
 Wind Directionality factor ‘$K_d$’ = 0.90 (IS:875-pt.3, Sec 6.2.1)

Area Averaging factor $K_a$ (IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns = $5 \times 6.76 = 33.81$ m² $\Rightarrow 0.888$

Tributary area for Trusses = $10.15 \times 5 = 60.95$ m² $\Rightarrow 0.852$

Tributary area for Purlins = $1.4 \times 5 = 7.0$ m² $\Rightarrow 1.0$

Tributary area of short walls for design of wind braces in plan $= 10 \times 5 + 0.5 \times 10 \times 1.76 = 58.8$ m² $\Rightarrow 0.855$

Permeability of the Building:
Since one of the walls of the structure is open, it comes under the category of large permeability exceeding 20% opening. (IS:875-pt.3, Fig. 2, Sec 6.3.2.2)

Design Wind Pressure
Design Wind Speed $= V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1 \times 1 \times 1 \times 1 = 47.00$ m/s (IS:875-pt.3, Sec 5.3)

$p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m² (IS:875-pt.3, Sec 6.2)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a$

$= 1.193 \times K_a$ kN/m² (IS:875-pt.3, Sec 6.2)

For various members and components, use proper value of $K_a$, as above.
Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

**Wind Load Calculations:**

\[ F = (C_{pe} - C_{pi}) \times A \times p_d \]

(IS:875-pt.3, Sec 6.3.1)

**Internal Pressure Coefficient:** for \( b/d = 50/10 = 5 > 1 \)

(IS:875-pt.3, Fig. 2, Sec 6.3.2.2)

\[ C_{pi} \text{ for } \theta = 0^\circ = +0.8 \]
\[ C_{pi} \text{ for } \theta = 180^\circ = -0.4 \]
\[ C_{pi} \text{ for } \theta = 90^\circ \text{ & } 270^\circ = -0.5 \]

**External Pressure Coefficients**

Using the (IS:875-pt.3, Table 7) with roof angle 10°

For \( h/w = 5/10 = 0.5 \), pressure coefficients are tabulated in table 9.1.

**Table 9-1**

<table>
<thead>
<tr>
<th>Portion of Roof</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward (widthwise left half)</td>
<td>0°</td>
</tr>
<tr>
<td>Windward (widthwise right half)</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

**Design Pressure Coefficients for Roof:**

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for different zones, as given in Figure 8-2.

**Design Pressure Coefficients for Walls:**

Refer Table 5 of code: \( h/w = 5/10 = 0.5 \), and \( l/w = 50/10 = 5 \) therefore \( C_{pe} \) for walls* are given in Table 9-2.

**Table 9-2**

<table>
<thead>
<tr>
<th>Wall</th>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.7</td>
<td>- 0.5</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>0.25</td>
<td>- 0.5</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.6</td>
<td>+ 0.7</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>0.6</td>
<td>- 0.1</td>
<td></td>
</tr>
</tbody>
</table>

*: Since the pressure coefficients are given only for buildings with \( l/w \) ratio up to 4, for longer buildings i.e. \( l/w > 4 \), at present values up to 4 are being used.

These will be combined with internal pressure coefficients as given above

\[ C_{pnet} \text{ for Walls A or B } = 0.7 - (+0.5) = +1.2, \text{ pressure} \]
\[ = -0.5 - (+0.8) = -1.3, \text{ suction} \]
\[ C_{pnet} \text{ for Walls C or D } = 0.7 - (+0.5) = +1.2, \text{ pressure} \]
\[ = -0.6 - (+0.8) = -1.4, \text{ suction} \]

**Local pressure coefficients:**

Local coefficients for roof: Max. value from all the values given in IS:875-pt.3, Table 7, i.e.\{ -2.0-(-0.8) \} = -2.8, up to \( 0.15 \times w = 0.15 \times 10 = 1.5 \)m on all edges of roof.

Local coefficients for walls: From IS:875-pt.3, Table 5, it is \{ -1.0 - (0.8) \} = -1.8, for a distance of \( 0.25 \times w = 0.25 \times 10 = 2.5 \)m at all corners.

In this region the fasteners shall be designed to carry increased force.

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_a = 1 \). Therefore, \( p_d = 1.3254 \times 0.9 = 1.193 \) kN/m²

(IS:875-pt.3, Table 7)

**Calculations of Force due to Frictional Drag:**

(IS:875-pt.3, Sec 6.4.1)

This will act in the longitudinal direction of the building along the wind. Here \( h < b \), therefore, first equation will be used & \( C_f' = 0.02 \). This will be added to the wind force on gable walls. \( K_a \) for roof and one long wall is 0.8, as area is more than 100m².
Figure 9.2 - Net Roof Pressure Coefficients for different zones and combinations

For 0° wind incidence, $C_{pi} = +0.8$

Which is equivalent to ↓

And similarly for other combinations.

*Local Pressure coefficients on roof edges.
Example 10 - Wind Pressure and Forces on a Rectangular Clad Pitched Roof Building with Clad Verandah

Problem Statement:
What difference will occur if the building in Example 4 is attached with a small clad mono-slope building of dimensions 5m width, 3m height on outer wall and 4m on the common wall, as shown in figure-10.1. The monoslope building has 1.0m overhang.

Solution:
Wind Data:
Dhanbad is situated in Zone II at the boundary of Zone II & Zone IV. For such places higher Zone is recommended. Therefore, Wind Zone: Zone IV ($V_b = 47$ m/s) (IS:875-pt.3, Sec 5.2)
2. Terrain category: Transition from Category 1 to Category 2. (IS:875-pt.3, Sec 5.3.2.1)
Note: A combined wind speed profile is to be worked out as per Appendix – B (Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 5m, only effects due to Terrain Category 2 are to be considered. A combined profile would be needed in case of tall structures.

Design Factors:
Risk Coefficient factor ‘$k_1$’ = 1.00 (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor ‘$k_2$’ = 1.00 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor ‘$k_3$’ = 1.00 (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $k_4$=1.00 (IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor ‘$K_d$’ = 0.90 (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor $K_a$ (IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns = $5 \times 5 = 25 \text{ m}^2 \Rightarrow 0.9$
Tributary area for main Trusses = $2 \times 5.176 \times 5 = 51.76 \text{ m}^2 \Rightarrow 0.864$
Tributary area for Purlins = $1.4 \times 5 = 7.0 \text{ m}^2 \Rightarrow 1.0$

Tributary area of short walls for design of plan braces,
Main portion = $5 \times 10 + 10 \times 1.33 \times \frac{1}{2} = 56.65 \text{ m}^2 \Rightarrow 0.858$
Annexe = $5 \times 3 + 5 \times 1 \times \frac{1}{2} = 16.5 \text{ m}^2 \Rightarrow 0.957$

Permeability of the Building: (keeping same as in Ex.4, all openings on the external walls)
Area of all the walls = $5(2\times10+50+2\times3.5) + 1.33 \times 10 \times \frac{1}{2} + 2 + 50 \times 1 + 50 = 598.6 \text{ m}^2$
Area of all the openings = $20\times1.5\times1.5 = 45 \text{ m}^2$
% opening area =7.6 %, between 5% and 20%
Hence the building is of Medium permeability. (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure
Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times K_d$
= $47 \times 1.0 \times 1.0 \times 1.0 \times 0.90 = 47.00 \text{ m/s}$ (IS:875-pt.3, Sec 5.3)
$p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{ N/m}^2$
(IS:875-pt.3, Sec 6.2)
\[ p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a = 1.193 \ K_a \] 
(IS:875-pt.3, Sec 6.2)

For various members and components, use proper value of \( K_a \), as above.

Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

**Wind Load Calculations:**

Total wind force on a joint or member or element, \( F = (C_{pe} - C_{pi}) \times A \times p_d \) 
(IS:875-pt.3, Sec 6.3.1)

**Internal Pressure Coefficient** \( C_{pi} = \pm 0.5 \) 
(IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of –0.5 from inside (refer Sec 6.3.2.1) along-with external pressure coefficient.

**External Pressure Coefficients**

For Main Building: Using the Table 6 with roof angle 15° (for ‘c’ & ‘d’ in Table 21)

For \( h/w = 5/10 = 0.5 \), pressure coefficients are tabulated in Table 10-1.

<table>
<thead>
<tr>
<th>Table 10-1</th>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0°</td>
</tr>
<tr>
<td>E</td>
<td>–0.8</td>
<td>-0.75</td>
</tr>
<tr>
<td>F</td>
<td>–0.8</td>
<td>0.6</td>
</tr>
<tr>
<td>G</td>
<td>–0.4</td>
<td>-0.75</td>
</tr>
<tr>
<td>H</td>
<td>–0.4</td>
<td>0.6</td>
</tr>
</tbody>
</table>

For portions ‘a’ and ‘b’ of the canopy: \( h/h_2 = 5/4 = 1.25 < 1.75 \)
(IS:875-pt.3, Table 21)

For 0° wind incidence: on ‘a’: \( C_{pe} = -0.45 \) & on ‘b’: \( C_{pe} = -0.5 \)

For 180° wind incidence: on ‘a’: \( C_{pe} = -0.4 \) & on ‘b’: \( C_{pe} = -0.4 \)

For 90°/270° wind incidence: on ‘a’: \( C_{pe} = -1.0 \) up to 2.5m from ends and –0.5 thereafter, from IS:875-pt.3, Table 7. On ‘b’: \( C_{pe} = -0.5 \), from IS:875-pt.3, Table 5.

**Design Pressure Coefficients for Roof:**

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for different zones, as given in figure 10-2.

**Design Pressure Coefficients for Walls:**

Refer Table 5 of code: \( h/w = 0.5 \), and \( l/(w1+w2) = 3.33 \) therefore \( C_{pe} \) for walls

<table>
<thead>
<tr>
<th>Table 10-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of Incidence</td>
</tr>
<tr>
<td>Wall – A</td>
</tr>
<tr>
<td>Wall – B</td>
</tr>
<tr>
<td>Wall – C</td>
</tr>
<tr>
<td>Wall – D</td>
</tr>
</tbody>
</table>

Note: Here Walls A, B, C & D refers to the external walls of combined building.

These will be combined with internal pressure coefficients as earlier, equal to \( C_{pi} = \pm 0.5 \)

\( C_{pec} \) for Walls A or B = \( 0.7 – (-0.5) = +1.2 \), pressure = \(-0.5 – (+0.5) = -1.0 \), suction

\( C_{pec} \) for Walls C or D = \( 0.7 – (-0.5) = +1.2 \), pressure = \(-0.6 – (+0.5) = -1.1 \), suction

**Design Pressure coefficients for Overhangs:**

For 0° wind incidence, i.e. from overhang side, \( C_{pi} = +1.25 \)

For other directions, \( C_{pi} \) shall be the same as on the adjoining wall, as above, +0.7 or –0.5.

\( C_{pe} = -2.0 \), being the max. on the nearest non-overhanging portion of canopy roof.

Design pressure coefficient on overhang: \(-2.0\)\( (+1.25) = -3.25 \)
(IS:875-pt.3, Sec. 6.3.3.5)

**Local pressure coefficients for the design of claddings and fasteners**

Refer Table 6 of IS-875 for Roof Angle = 15°

Local \( C_{pe} \) for eaves portion in end zone: NA

Local \( C_{pe} \) for eaves portion in mid zone: NA

Local \( C_{pe} \) for ridge portion: -1.2

Local \( C_{pe} \) for gable edges: -1.2

Local \( C_{pe} \) for canopy roof: -2.0
(IS:875-pt.3, Table 7)

Therefore Max. \( C_{pec} = -1.2 – (+0.5) = -1.7 \), for pitched roof \(-2.0 – (+0.5) = -2.5 \), for canopy roof

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_a = 1 \). Therefore, \( p_d = 1.3254 \times 0.9 = 1.193 \) kN/m².
Zone of local coefficients = 0.15 × 10 = 1.5m, at ridges, eaves and gable ends of pitched roof and 0.15 × 5 = .75m for canopy roof. In this region the fasteners shall be designed to carry increased force.

(IS:875-pt.3, Table 6)

**Calculations of Force due to Frictional Drag:**
(IS:875-pt.3, Sec 6.4.1)
This will act in the longitudinal direction of the building along the wind. Here $h < b$, therefore, first equation will be used & $C_f' = 0.02$. This will be added to the wind force on gable walls. $K_a$ for roof and walls is 0.8, as area is more than 100m$^2$. 
Figure 10-2 - Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

\[ \begin{align*}
0.45 & \quad 0.5 \quad 0.5 \quad 0.5 \quad 0.4 \quad = \quad 1.3 \quad 0.9 \\
0.45 & \quad 0.5 \quad 0.5 \quad 0.5 \quad 0.4 \quad = \quad 0.3 \quad 0.1 \\
\end{align*} \]

OR

\[ \begin{align*}
0.45 & \quad 0.5 \quad 0.5 \quad 0.5 \quad 0.4 \quad = \quad 0.05 \quad \text{zero} \\
0.45 & \quad 0.5 \quad 0.5 \quad 0.5 \quad 0.4 \quad = \quad 1.0 \quad 0.5 \\
\end{align*} \]

For End Zone E/G; 180° wind incidence

\[ \begin{align*}
0.4 & \quad 0.4 \quad 0.5 \quad 0.5 \quad 0.8 \quad = \quad 0.9 \quad 1.3 \\
0.4 & \quad 0.4 \quad 0.5 \quad 0.5 \quad 0.8 \quad = \quad 0.1 \quad 0.3 \\
\end{align*} \]

OR

\[ \begin{align*}
0.4 & \quad 0.4 \quad 0.5 \quad 0.5 \quad 0.8 \quad = \quad 0.5 \quad 1.5 \\
0.4 & \quad 0.4 \quad 0.5 \quad 0.5 \quad 0.8 \quad = \quad 0.5 \quad 0.5 \\
\end{align*} \]

For End Zone E/G; 90° wind incidence

\[ \begin{align*}
1.0 & \quad 0.5 \quad 0.5 \quad 0.5 \quad 0.75 \quad = \quad 1.25 \quad 1.25 \\
1.0 & \quad 0.5 \quad 0.5 \quad 0.5 \quad 0.75 \quad = \quad 0.5 \quad 0.25 \\
\end{align*} \]

OR

\[ \begin{align*}
1.0 & \quad 0.5 \quad 0.5 \quad 0.5 \quad 0.75 \quad = \quad 0.5 \quad 0.25 \\
0.75 & \quad 0.75 \quad 0.5 \quad 0.5 \quad 0.75 \quad = \quad 0.5 \quad 0.0 \\
\end{align*} \]
For Mid Zone F/H; $0^\circ$ wind incidence

For Mid Zone F/H; $180^\circ$ wind incidence

For Mid Zone F/H; $90^\circ$ wind incidence
Example 11 - Wind Pressure and Forces on a Rectangular Clad Pitched Roof Building with Open Verandah

Problem Statement:
What difference will occur if the mono-slope annexe in Example 10 is unclad (open) on all the three sides as shown in figure 11.1?

Solution:

Wind Data:
Dhanbad is situated in Zone II at the boundary of Zone II & Zone IV. For such places higher Zone is recommended. Therefore, Wind Zone: Zone IV ($V_b$ = 47 m/s) (IS:875-pt.3, Sec 5.2)

2. Terrain category: Transition from Category 1 to Category 2. (IS:875-pt.3, Sec 5.3.2.1)

Note: A combined wind speed profile is to be worked out as per Appendix – B (IS:875-pt.3, Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 5m, only effects due to Terrain Category 2 are to be considered.

A combined profile would be needed in case of tall structures.

Design Factors:
Risk Coefficient factor, 'k₁' = 1.00 (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor, 'k₂' = 1.00 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor, 'k₃' = 1.00 (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region, 'k₄' = 1.00 (IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor, 'K_d' = 0.90 (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor $K_a$ (IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns = $5 \times 5 = 25$ m² = 0.9

Tributary area for main Trusses = $2 \times 5.176 \times 5 = 51.76$ m² = 0.864

Tributary area for Purlins = $1.4 \times 5 = 7.0$ m² = 1.0

Tributary area of short walls for design of wind braces in plan = $10 \times 5 + 6.7 = 56.7$ m² = 0.858

Permeability of the Building: (keeping same as in Ex.4, all openings on the external walls)
Area of all the walls = $5 \times (2 \times 10 + 2 \times 50) + 2 \times 6.7 = 613.4$ m²
Area of all the openings = $20 \times 1.5 \times 1.5 = 45$ m²
% opening area = 7.3 %, between 5% and 20%
Hence the building is of Medium permeability. (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure
Design Wind Speed $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_d = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00$ m/s (IS:875-pt.3, Sec 5.3)

$p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m² (IS:875-pt.3, Sec 6.2)

$p_d = p_Z \times K_d \times \kappa_o = 1.3254 \times 0.9 \times \kappa_o = 1.193 \kappa_o$ (IS:875-pt.3, Sec6.2)

For various members and components, use proper value of $\kappa_o$ as above.

Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.
Wind Load Calculations:

\[ F = (C_{pe} - C_{pw}) \times A \times p_d \]  
(IS:875-pt.3, Sec 6.3.1)

Internal Pressure Coefficient \( C_{pi} \) = ± 0.5  
(IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of 0.5 from inside (IS:875-pt.3, Sec 6.3.2.1) along with external pressure coefficient.

External Pressure Coefficients

For Main Building: Using the Table 6 with roof angle 15° (for 'c' & 'd' in IS:875-pt.3, Table 21)  
For \( h/w \leq 0.5 \), pressure coefficients are tabulated in Table 11-1. (Refer figure below IS:875-pt.3, Table 6)

Table 11-1

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle 0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>-0.8</td>
<td>-0.75</td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
<td>-0.6</td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
<td>-0.75</td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
<td>-0.6</td>
</tr>
</tbody>
</table>

For portions ‘a’, ‘b’ and ‘c’ of the combined part: \( h1/h2 = 5/4 = 1.25 \times 1.75 \)  
(IS:875-pt.3, Table 21)

For 0° wind incidence: on ‘a’: \( C_{pe} = -0.45 \) & on ‘b’: \( C_{pe} = -0.5 \)  
For 180° wind incidence: on ‘a’: \( C_{pe} = -0.4 \) & on ‘b’: \( C_{pe} = -0.4 \)  

For 90°/270° wind incidence: on ‘a’: \( C_{pe} = -1.0 \) up to 2.5m from ends & -0.5 thereafter, from IS:875-pt.3, Table 7. On ‘b’: \( C_{pe} = -0.5 \), from IS:875-pt.3, Table 5.

For canopy roof, overhanging from building: \( C_{pi} = +1.25 \), for 0° wind incidence & -0.5 for other directions which is the max. Pressure on adjoining wall. (refer 6.3.3.5)

Design Pressure Coefficients for Roof:

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to -ve external pressure coefficient and vice-versa. The combinations will have to be made separately for zones, as given in figure 11-2.

Design Pressure Coefficients for Walls:

Refer Table 5 of code: \( h/w = 0.5 \), and \( l/w = 5 \) therefore \( C_{pe} \) for walls* are given in Table 11-2.

<table>
<thead>
<tr>
<th>Table 11-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of Incidence</td>
</tr>
<tr>
<td>Wall – A</td>
</tr>
<tr>
<td>Wall – B</td>
</tr>
<tr>
<td>Wall – C</td>
</tr>
<tr>
<td>Wall – D</td>
</tr>
</tbody>
</table>

*: Since the pressure coefficients are given only for buildings with \( h/w\) ratio up to 4, for longer buildings i.e. \( h/w > 4 \), at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to \( C_{pi} = \pm 0.5 \)  
\( C_{pnet} \) for Walls A or B = 0.7 – (-0.5) = +1.2, pressure = -0.5 – (+0.5) = -1.0, suction  
\( C_{pnet} \) for Walls C or D = 0.7 – (-0.5) = +1.2, pressure = -0.6 – (+0.5) = -1.1, suction

Local pressure coefficients for the design of claddings and fasteners

Refer Table 6 of IS-875 for Roof Angle = 15°  
Local \( C_{pe} \) for eaves portion in end zone: NA  
Local \( C_{pe} \) for eaves portion in mid zone: NA  
Local \( C_{pe} \) for ridge portion: -1.2  
Local \( C_{pe} \) for gable edges: -1.2  
Therefore Max. \( C_{pnet} = -1.2 – (+0.5) = -1.7 \)

For canopy roof, \( C_{pnet} = -2.0 – (+0.5) = -2.5 \)  
(IS:875-pt.3, Table 7)

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_f = 1 \). Therefore, \( p_d = 1.3254 \times 0.9 = 1.193 \text{ kN/m}^2 \)

For Pitched roof: Zone of local coefficients = 0.15×10 = 1.5m, at ridges, eaves and gable ends.

For canopy roof: Zone of local coefficients = 0.15×5 = 0.75m, at eaves and gable ends.

In this region the fasteners shall be designed to carry increased force.  
(IS:875-pt.3, Table 6)

Calculations of Force due to Frictional Drag:

(IS:875-pt.3, Sec 6.4.1)

This will act in the longitudinal direction of the building along the wind. Here \( h > b \), therefore, first equation will be used & \( C_f' = 0.02 \). This will be added to the wind force on gable walls. \( K_a \) for roof and walls is 0.8, as area is more than 100m².
Figure 11-2 - Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

For End Zone E/G; 180° wind incidence

For End Zone E/G; 90° wind incidence
For Mid Zone F/H; 0° wind incidence

\[
\begin{align*}
0.45 & \quad 0.8 & \quad 0.4 & \quad 0.5 & \quad 0.5 & \quad 1.25 \\
\text{OR} & & & & & \\
0.45 & \quad 0.8 & \quad 0.4 & \quad 0.5 & \quad 0.5 & \quad 1.25
\end{align*}
\]

For Mid Zone F/H; 180° wind incidence

\[
\begin{align*}
0.4 & \quad 0.4 & \quad 0.8 & \quad 0.5 & \quad 0.5 & \quad 0.5 \\
\text{OR} & & & & & \\
0.4 & \quad 0.4 & \quad 0.8 & \quad 0.5 & \quad 0.5 & \quad 0.5
\end{align*}
\]

For Mid Zone F/H; 90° wind incidence

\[
\begin{align*}
0.5 & \quad 0.5 & \quad 0.6 & \quad 0.6 & \quad 0.5 & \quad 0.5 \\
\text{OR} & & & & & \\
0.5 & \quad 0.5 & \quad 0.6 & \quad 0.6 & \quad 0.5 & \quad 0.5
\end{align*}
\]
Example 12 - Wind Pressure and Forces on a Rectangular Clad Building on A Ridge or Hill: Pitched Roof

Problem Statement:
Calculate wind pressures and design forces on walls and roof of a rectangular clad resort building with pitched roof, having plan dimensions $10\times30$ m and height $5$ m, as shown in figure-12.1. The building is situated in outskirts of Jaipur on a hilltop $10$ m high having upwind and downwind slopes of $18^0$ and $10^0$, respectively. The building has 16 openings of $1.5\times1.5$ m size. The roof is of GC sheeting & the roof angle $\alpha$ is $15^0$. Calculate also the local wind pressures on roof & wall cladding. The columns and trusses are at $5$ m c/c longitudinally, purlins are at $1.4$ m c/c and columns at Gable ends are at $5$ m c/c.

Solution:

Wind Data:
1. Wind Zone: Zone IV ($V_b = 47$ m/s)
   Note: Jaipur is situated in Zone IV.
   (IS:875-pt.3, Sec 5.2)
2. Terrain category: Category 2 for the moderately developed area.
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient factor \( k_1 \) = 1.00
   (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor \( k_2 \) = 1.00
   (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor \( k_3 \) = 1.198*
   (IS:875-pt.3, Sec 5.3.3.1 & App. ‘C’)
* : see calculations of \( k_3 \) at the end.

Importance Factor for Cyclonic Region \( k_4 \) = 1.00
   (IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor \( K_d \) = 0.90
   (IS:875-pt.3, Sec 6.2.1)

Area Averaging factor \( K_a \)
   (IS:875-pt.3, Sec 6.2.2, Table-4)
Tributary area for columns = $5 \times 5 = 25$ m$^2$ = 0.9
Tributary area for Trusses = $2 \times 5.176 \times 5 = 51.76$ m$^2$ = 0.864
Tributary area for Purlins = $1.4 \times 5 = 7.0$ m$^2$ = 1.0
Tributary area of short walls for design of wind braces in plan

= $50 + 6.7 = 56.7$ m$^2$ = 0.858

Permeability of the Building:
Area of all the walls = $5 \times (2 \times 10 + 2 \times 30) + 2 \times 6.7 = 413.4$ m$^2$
Area of all the openings = $16 \times 1.5 \times 1.5 = 36$ m$^2$
% opening area = 8.71 %, between 5% and 20% Hence the building is of medium permeability.
   (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure

Design Wind Speed

\[ V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.198 \times 1.0 = 56.3 \text{ m/s} \]
   (IS:875-pt.3, Sec 5.3)

\[ p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (56.3)^2 = 1902.22 \text{ N/m}^2 \]
   (IS:875-pt.3, Sec 6.2)

\[ p_d = p_Z \times K_d \times K_a = 1.9022 \times 0.9 \times K_a = 1.712 K_a \]
   (IS:875-pt.3, Sec 6.2)

For various members and components, use proper value of \( K_a \), as above.

Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

Wind Load Calculations:

\[ F = (C_{pe} \times C_{pj}) \times A \times p_d \]
   (IS:875-pt.3, Sec 6.3.1)

Internal Pressure Coefficient \( C_{pj} = \pm 0.5 \)
   (IS:875-pt.3, Sec 6.3.2.2)
Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of -0.5 from inside (IS:875-pt.3, 2 Sec 6.3.2.1) along-with external pressure coefficient

External Pressure Coefficients
Using the Table 6 with roof angle 15°
For h/w = 0.5, pressure coefficients are tabulated in Table 12-1. (refer figure of IS:875-pt.3, Table 6)

| Table 12-1 |
|-------------------------------|-----------------|-----------------|
| Portion of roof               | Wind Incidence Angle | Wind Incidence Angle |
| E                             | 0°               | 90°             |
| F                             | -0.8             | -0.75           |
| G                             | -0.8             | -0.6            |
| H                             | -0.4             | -0.75           |
|                               | -0.6             | -0.6            |

Design Pressure Coefficients for Roof:
Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for zones E, F, G, H etc., as given in figure 12-2.

Design Pressure Coefficients for Walls:
Refer Table 5 of code: h/w = 0.5, and l/w = 5 therefore C_pe for walls* are given in Table 12-2.

| Table 12-2 |
|-------------------------------|-----------------|-----------------|
| Angle of Incidence            | 0°               | 90°             |
| Wall – A                      | +0.7             | -0.5            |
| Wall – B                      | -0.25            | -0.5            |
| Wall – C                      | -0.6             | +0.7            |
| Wall – D                      | -0.6             | -0.1            |

*: Since the pressure coefficients are given only for buildings with l/w ratio up to 4, for longer buildings i.e. l/w > 4, at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to C_pe = ±0.5
C_pe for Walls A or B = 0.7 – (-0.5) = +1.2, pressure
= -0.5 – (+0.5) = -1.0, suction
C_pe for Walls C or D = 0.7 – (-0.5) = +1.2, pressure
= -0.6 – (+0.5) = -1.1, suction

Local pressure coefficients for the design of claddings and fasteners
Refer Table 6 of IS-875 for Roof Angle = 15°
Local C_pe for eaves portion in end zone: NA
Local C_pe for eaves portion in mid zone: NA
Local C_pe for ridge portion: -1.2
Local C_pe for gable edges: -1.2
Local C_be for corners of walls: -1.0

Max. local C_pe for roof at the edges and the ridge = -1.2 – (+0.5) = -1.7
Likewise at the wall edges = -1.0 – (+0.5) = -1.5
However, for the use of the local pressure coefficients, the design pressure p_d will be computed with K_a = 1. Therefore, p_d = 1.9022×0.9 = 1.712 kN/m²

Zone of local coefficients = 0.15×10 = 1.5m, at ridges, eaves and gable ends & 0.25×10 = 2.5m for wall corners. In this region the cladding and fasteners shall be checked for increased force.

(IS:875-pt.3, Table 6)

Calculations of Force due to Frictional Drag:
(IS:875-pt.3, Sec 6.3.1)
This will act in the longitudinal direction of the building along the wind. Here h<b, therefore, first equation will be used & C_f = 0.02. This will be added to the wind force on gable walls. K_a for roof and walls is 0.8, as area is more than 100m².

**CALCULATIONS FOR TOPOGRAPHY FACTOR k_z**

Wind from left:
H = 10 m, z = 10 m, L = 10/tan 18° = 30.777 K_z = \frac{H}{L} + C_s
For θ = 18°, C = 0.36 (C-2)
Factor ‘s’ is obtained from C-2.1 and figure15, for crest position
Le = z/0.3 = 10/0.3  \quad H/Le = 10/10.0 = 0.3
\quad \Rightarrow s = 0.55
k_z = 1 + 0.36 × 0.55 = 1.198

Wind from right:
H = 10 m, z = 10 m, L = 10/tan 10° = 56.7 m
C = 1.2 (C/L) = 1.2 (10/56.7) = 0.21
For θ = 10°, Le = L = 56.7 m
H/Le = 10/56.7 = 0.176
\quad \Rightarrow s = 0.7
k_z = 1 + 0.21 × 0.7 = 1.147
Using k_z = 1.198, being the critical one.
Figure 12-2 - Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

\[ \begin{align*}
0.8 & \quad 0.5 \quad 0.5 \\
0.4 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
1.3 & \quad 0.9 \\
0.3 & \quad 0.1 \\
\end{align*} \]

OR

\[ \begin{align*}
0.8 & \quad 0.5 \quad 0.5 \\
0.4 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
0.3 & \quad 0.1 \\
0.3 & \quad 0.1 \\
\end{align*} \]

For End Zone E/G; 90° wind incidence

\[ \begin{align*}
0.75 & \quad 0.5 \quad 0.5 \\
0.75 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
1.25 & \quad 1.25 \\
0.25 & \quad 0.25 \\
\end{align*} \]

OR

\[ \begin{align*}
0.75 & \quad 0.5 \quad 0.5 \\
0.75 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
0.25 & \quad 0.25 \\
0.25 & \quad 0.25 \\
\end{align*} \]

For Mid Zone F/H; 0° wind incidence

\[ \begin{align*}
0.8 & \quad 0.5 \quad 0.5 \\
0.4 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
1.3 & \quad 0.9 \\
0.3 & \quad 0.1 \\
\end{align*} \]

OR

\[ \begin{align*}
0.8 & \quad 0.5 \quad 0.5 \\
0.4 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
0.3 & \quad 0.1 \\
0.3 & \quad 0.1 \\
\end{align*} \]

For Mid Zone F/H; 90° wind incidence

\[ \begin{align*}
0.6 & \quad 0.5 \quad 0.5 \\
0.6 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
1.1 & \quad 1.1 \\
0.1 & \quad 0.1 \\
\end{align*} \]

OR

\[ \begin{align*}
0.6 & \quad 0.5 \quad 0.5 \\
0.6 & \quad 0.5 \quad 0.5 \\
\end{align*} \]

\[ = \]

\[ \begin{align*}
0.1 & \quad 0.1 \\
0.1 & \quad 0.1 \\
\end{align*} \]
Example 13 - Wind Pressure and Forces on a Rectangular Clad Building on A Cliff & Escarpment: Pitched Roof

Problem Statement:
What difference will occur if the building in Example 12 is situated on a hill having upwind and downwind slopes of 15° and 0°, respectively as shown in figure 13.1?

Solution:
Wind Data:
1. Wind Zone: Zone IV (V_b = 47m/s) (IS:875-pt.3, Sec 5.2)
2. Terrain category: Category 2 for the moderately developed area. (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
- Risk Coefficient factor ‘k_1’ = 1.00 (IS:875-pt.3, Sec 5.3.1, Table-1)
- Terrain & Height factor ‘k_2’ = 1.00 (IS:875-pt.3, Sec 5.3.2.2, Table-2)
- Topography factor ‘k_3’ = 1.193* (IS:875-pt.3, Sec 5.3.3.1 & App. ‘C’)
- Importance Factor for Cyclonic Region k_4 =1.00 (IS:875-pt.3, Sec 5.3.4)
- Wind Directionality factor ‘K_d’ = 0.90 (IS:875-pt.3, Sec 6.2.1)

Area Averaging factor K_a (IS:875-pt.3, Sec 6.2.2, Table-4)

Area of all the columns = 5 x 5 = 25 m² = 0.9
Area of all the Trusses = 2 x 5.176 x 5 = 51.76m² = 0.864
Area of all the Purlins = 1.4 x 5 = 7.0 m² = 1.0
Area of all the short walls for design of wind braces in plan = 50 + 6.7 = 56.7 m² = 0.858

Area of all the openings = 16 x 1.5 x 1.5 = 36 m²
% opening area = 8.71 %, between 5% and 20%
Hence the building is of medium permeability. (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure

Design Wind Speed = V_Z = V_b x k_1 x k_2 x k_3 x k_4 = 47 x 1.0 x 1.0 x 1.193 x 1.0 = 56.071 m/s (IS:875-pt.3, Sec 5.3)

p_Z = 0.6 (V_Z)² = 0.6 x (56.071)² = 1886.37 N/m² (IS:875-pt.3, Sec 6.2)

p_d = p_Z x K_d x K_a = 1.886 x 0.9 x K_a = 1.700 K_a (IS:875-pt.3, Sec 6.2)

For various members and components, use proper value of K_a, as above.
Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

Wind Load Calculations:

F = (C_pe - C_pi) x A x p_d (IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient C_pi = ± 0.5 (IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of ~0.5 from inside (IS:875-pt.3, Sec 6.2.2.1) along-with external pressure coefficient

External Pressure Coefficients
Using the IS:875-pt.3, Table 6 with roof angle 15°
For h/w = 0.5, pressure coefficients are tabulated in Table 13-1. (IS:875-pt.3, Table 6)
Table 13-1

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0°</td>
</tr>
<tr>
<td>E</td>
<td>-0.8</td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
</tr>
<tr>
<td></td>
<td>90°</td>
</tr>
<tr>
<td></td>
<td>-0.75</td>
</tr>
<tr>
<td></td>
<td>-0.6</td>
</tr>
<tr>
<td></td>
<td>-0.75</td>
</tr>
<tr>
<td></td>
<td>-0.6</td>
</tr>
</tbody>
</table>

### Design Pressure Coefficients for Roof:

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to -ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for zones E, F, G, H etc., as given in figure 12-2.

### Design Pressure Coefficients for Walls:

Refer Table 5 of code: \( h/w = 0.5 \), and \( l/w = 5 \) therefore \( C_{pe} \) for walls* are given in Table 13-2.

### Table 13-2

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>-0.25</td>
<td>-0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>-0.6</td>
<td>+0.7</td>
</tr>
<tr>
<td>Wall – D</td>
<td>-0.6</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

*: Since the pressure coefficients are given only for buildings with \( l/w \) ratio up to 4, for longer buildings i.e. \( l/w > 4 \), at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to \( C_{pi} = \pm 0.5 \).

\[ C_{net} \] for Walls A or B = 0.7 – (-0.5) = +1.2, pressure
\[ = -0.5 – (+0.5) = -1.0, \text{ suction} \]

\[ C_{net} \] for Walls C or D = 0.7 – (-0.5) = +1.2, pressure
\[ = -0.6 – (+0.5) = -1.1, \text{ suction} \]

### Local pressure coefficients for the design of claddings and fasteners

Refer Table 6 of IS-875 for Roof Angle = 15°

Local \( C_{pe} \) for eaves portion in end zone: NA
Local \( C_{pe} \) for eaves portion in mid zone: NA
Local \( C_{pe} \) for ridge portion: -1.2
Local \( C_{pe} \) for gable edges: -1.2
Local \( C_{pe} \) for corners of walls: -1.0

Max. local \( C_{net} \) for roof at the edges and the ridge = -1.2 – (+0.5) = -1.7
Likewise at the wall edges = -1.0 – (+0.5) = -1.5

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_a = 1 \). Therefore, \( p_d = 1.886*0.9 = 1.700 \text{ kN/m}^2 \)

Zone of local coefficients = 0.15×10 = 1.5m, at ridges, eaves and gable ends & 0.25×10 = 2.5m for wall corners. In this region the cladding and fasteners shall be checked for increased force.

(IS:875-pt.3, Table 6)

### Calculations of Force due to Frictional Drag:

(IS:875-pt.3, Sec 6.3.1)

This will act in the longitudinal direction of the building along the wind. Hence \( h<b \), therefore, first equation will be used & \( C_f' = 0.02 \). This will be added to the wind force on gable walls. \( K_a \) for roof and walls is 0.8, as area is more than 100m².

### Calculations for Topography Factor \( k_3 \)

Wind from left:

**Note:** Here \( H = 10\text{m}, z = 10\text{m} \) & \( L = 10/\tan 15^0 = 37.32 \)

\[ k_3 = 1 + C \cdot s \]

for \( \theta = 15^0 \), \( C = 1.2(z/L) = 1.2 (10/37.32) = 0.321 \)

\{ from C-2 \}

for \( \theta = 01^0 \), \( C = 1.2(z/L) = 1.2 (00/37.32) = 0.00 \)

\{ from C-2 \}

factor ‘s’ is obtained from C-2.1 and figure 14 for crest position

\[ H/Le = (10/37.32) = 0.268 \rightarrow s = 0.6 \]

Therefore \( k_3 = 1 + 0.321 \times 0.6 = 1.193 \), wind from left &

\[ 1 + 0.00 \times 0.6 = 1.000 \], wind from right

Using \( k_3 = 1.193 \), being the critical one.
Figure 13-2 - Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

For End Zone E/G; 90° wind incidence

For Mid Zone F/H; 0° wind incidence

For Mid Zone F/H; 90° wind incidence
Example 14 - Wind Pressure and Forces on a Rectangular Clad Building on Slope of A Ridge or Hill: Pitched Roof

Problem Statement:
What difference will occur if the building in Example 12 is situated in the middle of the upwind slope of a hill 50m high, upwind and downwind slopes being $18^\circ$ and $10^\circ$ respectively, as shown in figure 14.1?

Solution:

Wind Data:
1. Wind Zone: Zone IV ($V_b = 47 m/s$)  
   (IS:875-pt.3, Sec 5.2)
2. Terrain category: Category 2 for the moderately developed area.  
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient factor $'k_1'$ = 1.00  
   (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor $'k_2'$ = 1.00  
   (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor $'k_3'$ = 1.127*  
   (IS:875-pt.3, Sec 5.3.3.1 & App. 'C')
* : see calculations of $k_3$ at the end.
Importance Factor for Cyclonic Region $k_4$ =1.00  
   (IS:875-pt.3, Sec 5.3.4)

Wind Directionality factor $'K_d'$ = 0.90  
   (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor $K_a$= 0.867, for short walls = 0.80, for long walls & roofs  
   (IS:875-pt.3, Sec 6.2.2, Table-4)
Tributary area for columns = 5 x 5 = 25 m$^2$ $\Rightarrow$ 0.9
Tributary area for Trusses = 2 x 5.176 x 5 = 51.76 m$^2$ $\Rightarrow$ 0.864
Tributary area for Purlins = 1.4 x 5 = 7.0 m$^2$ $\Rightarrow$ 1.0
Tributary area of short walls for design of wind braces in plan = 50 + 6.7 = 56.7 m$^2$ $\Rightarrow$ 0.858

Permeability of the Building:
Area of all the walls = $5 \times (2 \times 10 + 2 \times 30) + 2 \times 6.7 = 413.4m^2$
Area of all the openings = $16 \times 1.5 \times 1.5 = 36 m^2$
% opening area = $8.71\%$, between 5% and 20%
Hence the building is of medium permeability.  
   (IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure
Design Wind Speed $V_Z$ = $V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.127 \times 1.0 = 52.97 m/s$  
   (IS:875-pt.3, Sec 5.3)
$p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (52.97)^2 = 1683.5 N/m^2$  
   (IS:875-pt.3, Sec 6.2)
$p_d = p_Z \times K_d \times K_a = 1.6835 \times 0.9 \times 0.867 = 1.313 kN/m^2$ (short wall)  
   = $1.6835 \times 0.9 \times 0.8 = 1.212 kN/m^2$ (long wall & roof)  
   (IS:875-pt.3, Sec 6.2)

Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

Wind Load Calculations:
$F = (C_{pe} \times C_{pi}) \times A \times p_d$  
   (IS:875-pt.3, Sec 6.3.1)
Internal Pressure Coefficient $C_{pi}$ = $\pm 0.5$  
   (IS:875-pt.3, Sec 6.3.2.2)
Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction
of –0.5 from inside (refer Sec 6.3.2.2) along-with external pressure coefficient

External Pressure Coefficients
Using the IS:875-pt.3, Table 6 with roof angle 15°
For h/w = 0.5, pressure coefficients are tabulated in Table 14-1. (refer figure Table 6 of code)

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>-0.8</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>-0.8</td>
<td>-0.6</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
<td>-0.75</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
<td>-0.6</td>
<td></td>
</tr>
</tbody>
</table>

Design Pressure Coefficients for Roof:

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for zones E, F, G, H etc., as given in figure 14-2.

Design Pressure Coefficients for Walls:

Refer Table 5 of code: h/w = 0.5, and l/w = 5 therefore Cpe for walls* are given in Table 14-2.

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>Wall – A</th>
<th>Wall – B</th>
<th>Wall – C</th>
<th>Wall – D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°</td>
<td>+0.7</td>
<td>-0.25</td>
<td>-0.6</td>
<td>-0.6</td>
</tr>
<tr>
<td>90°</td>
<td>-0.5</td>
<td>-0.5</td>
<td>+0.7</td>
<td>-0.1</td>
</tr>
</tbody>
</table>

*: Since the pressure coefficients are given only for buildings with l/w ratio up to 4, for longer buildings i.e. l/w > 4, at present values up to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to Cpi = ± 0.5

\[ C_{net} \text{ for Walls A or B} = 0.7 - (-0.5) = +1.2, \]
\[ \text{pressure} = -0.5 - (+0.5) = -1.0, \text{ suction} \]
\[ C_{net} \text{ for Walls C or D} = 0.7 - (-0.5) = +1.2, \]
\[ \text{pressure} = -0.6 - (+0.5) = -1.1, \text{ suction} \]

Local pressure coefficients for the design of claddings and fasteners
Refer Table 6 of IS-875 for Roof Angle = 15°

Local Cpe for eaves portion in end zone: NA
Local Cpe for eaves portion in mid zone: NA
Local Cpe for ridge portion: -1.2
Local Cpe for gable edges: -1.2
Max. local Cnet for roof at the edges and the ridge = -1.2 - (+0.5) = -1.7

Zone of local coefficients = 0.15 × 10 = 1.5m, at ridges, eaves and gable ends. In this region the cladding and fasteners shall be checked for increased force.

(IS:875-pt.3, Table 6)
However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_a = 1 \). Therefore \( p_d = 1.6835 \times 0.9 = 1.515 \text{ kN/m}^2 \).

Calculations for Topography Factor \( k_3 \)
(Refer IS:875-pt.3, Appendix-C)

For wind from left to right: Z = 50m \( H = 25m \)
\( Le = Z/0.3 = 50/0.3 = 166.67m \) \( \theta = 18^0 \), \( C = 0.36 \) and \{ from C-2 \}
factor ‘s’ is obtained from C-2.1 and IS:875-pt.3, Figure 15 for upwind position
\( H/Le = (25/166.67) = 0.15 \) & \( X/Le = -(77/166.67) = -0.462 \)
\( k_3 = 1 + C \times s = 1 + 0.36 \times 0.3 = 1.108 \)
For wind from right to left: Z = 50m \( H = 25m \)
\( Le = L = 50/\tan 10^0 = 283.56m \)
for \( \theta = 10^0 \), \( C = 1.2(Z/L) = 1.2 (50/283.56) = 0.2116 \)
factor ‘s’ is obtained from C-2.1 and IS:875-pt.3, Figure 15 for downwind position
\( H/Le = (25/283.56) = 0.09 \) & \( X/Le = (77/283.56) = 0.271 \)
\( k_3 = 1 + 0.2116 \times 0.6 = 1.127 \)
Therefore \( k_3 = 1.127 \), being the critical one.
Figure 14-2 - Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

\[
\begin{align*}
0.8 & \quad 0.5 \quad 0.5 \quad 0.4 \\
0.5 & \quad 0.5
\end{align*}
\]

\[
\begin{align*}
0.8 & \quad 0.5 \quad 0.5 \quad 0.4 \\
0.5 & \quad 0.5
\end{align*}
\]

OR

\[
\begin{align*}
0.3 & \quad 0.1 \\
0.1 & \quad 0.1
\end{align*}
\]

For End Zone E/G; 90° wind incidence

\[
\begin{align*}
0.75 & \quad 0.5 \quad 0.5 \quad 0.75 \\
0.5 & \quad 0.5
\end{align*}
\]

\[
\begin{align*}
0.75 & \quad 0.5 \quad 0.5 \quad 0.75 \\
0.5 & \quad 0.5
\end{align*}
\]

OR

\[
\begin{align*}
0.25 & \quad 0.25 \\
0.25 & \quad 0.25
\end{align*}
\]

For Mid Zone F/H; 0° wind incidence

\[
\begin{align*}
0.8 & \quad 0.5 \quad 0.5 \quad 0.4 \\
0.5 & \quad 0.5
\end{align*}
\]

\[
\begin{align*}
0.8 & \quad 0.5 \quad 0.5 \quad 0.4 \\
0.5 & \quad 0.5
\end{align*}
\]

OR

\[
\begin{align*}
0.3 & \quad 0.1 \\
0.1 & \quad 0.1
\end{align*}
\]

For Mid Zone F/H; 90° wind incidence

\[
\begin{align*}
0.6 & \quad 0.5 \quad 0.5 \quad 0.6 \\
0.5 & \quad 0.5
\end{align*}
\]

\[
\begin{align*}
0.6 & \quad 0.5 \quad 0.5 \quad 0.6 \\
0.5 & \quad 0.5
\end{align*}
\]

OR

\[
\begin{align*}
0.1 & \quad 0.1 \\
0.1 & \quad 0.1
\end{align*}
\]
Example 15 - Wind Pressure and Forces on a Rectangular Clad Building: Hipped Roof

Problem Statement:
Calculate wind pressures and design forces on walls and roof of a rectangular clad building with hipped roof, having plan dimensions 10m×20m and height 5m, as shown in figure-15.1. The building is situated in Jaipur on a fairly level topography. Walls of building have 20 openings of 1.5m×1.5m size. The roof is of GC sheeting & the roof angle \( \alpha \) is 15°. Calculate also the local wind pressures on roof & wall cladding. The columns & trusses are at 5m c/c longitudinally, purlins are at 1.4m c/c and columns at Gable ends are at 5m c/c.

Solution:

Wind Data:
1. Wind Zone: Zone IV (\( V_b = 47 \text{m/s} \)) ----> (IS:875-pt.3, Sec 5.2)

2. Terrain category: Terrain Category 2
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient factor \( 'k_1' = 1.00 \)
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor \( 'k_2' = 1.00 \)
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor \( 'k_3' = 1.00 \)
(IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region \( k_4 = 1.00 \)
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor \( 'K_d' = 0.90 \)
(IS:875-pt.3, Sec 6.2.1)
Area Averaging factor \( K_a \):
(IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for columns = 5 ×5 = 25 m²
\( \Rightarrow 0.9 \)

Tributary area for Trusses = 5.176 ×5 ×2 = 51.76 m²
\( \Rightarrow 0.864 \)

Tributary area for Purlins = 1.4 ×5 = 7.0 m²
\( \Rightarrow 1.0 \)

Tributary area of short walls = 10 ×5 = 50 m²
\( \Rightarrow 0.867 \) for design of wind braces in plan

Permeability of the Building:
Area of all the walls = 5× (2×10 + 2×20) + 6.7 = 306.7 m²
Area of all the openings = 20×1.5×1.5 = 45 m²
% opening area = 14.67 %, between 5% and 20%
Hence the building is of medium permeability.
(IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure
Design Wind Speed
\( V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 \)
=47×1.0×1.0×1.0×1.0 = 47.00 m/s
(IS:875-pt.3, Sec 5.3)

\( p_Z = 0.6 (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{N/m}^2 \)
(IS:875-pt.3, Sec 6.2)

\( p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a = 1.193 K_a \)
(IS:875-pt.3, Sec 6.2)

For various members and components use proper value of \( K_a \), as above

Refer note below Sec. 5.3 for buildings less than 10m height, while making stability calculations and design of the frame.

Wind Load Calculations:
\[ F = (C_{pe}C_{pi}) \times A \times p_d \]
(IS:875-pt.3, Sec 6.3.1)
Internal Pressure Coefficient $C_{pi} = \pm 0.5$

(IS:875-pt.3, Sec 6.3.2.2)

Note: buildings shall be analysed once for pressure of 0.5 from inside and then for a suction of -0.5 from inside (refer Sec 6.3.2.1) along-with external pressure coefficient.

External Pressure Coefficients

Using the Table 6 with roof angle 15°

For $h/w = 0.5$, pressure coefficients are tabulated in Table 15-1. (refer figure below Table 6 of code)

Table 15-1

<table>
<thead>
<tr>
<th>Portion of roof*</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>0°</td>
</tr>
<tr>
<td>F</td>
<td>0.8</td>
</tr>
<tr>
<td>G</td>
<td>-0.4</td>
</tr>
<tr>
<td>H</td>
<td>-0.4</td>
</tr>
<tr>
<td>Hipped slope, M,N</td>
<td>-0.75**</td>
</tr>
<tr>
<td></td>
<td>-0.8** (windward)</td>
</tr>
<tr>
<td></td>
<td>-0.4** (leeward)</td>
</tr>
</tbody>
</table>

* See Figure 15.2.

** These values may be reduced by 20% as per IS:875-pt.3, Sec 6.3.3.2, note 3.

Design Pressure Coefficients for Roof:

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to -ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for zones E, F, G, H etc., as given on figure 15.3.

Hipped slopes shall be subjected to a net pressure coefficient of

$-0.8 - (+0.5) = -1.3$  \hspace{1cm} or

$-0.4 - (-0.5) = +0.1$

but all the elements of roof in hipped slope shall be designed for a reduced pressure of 80%.

Design Pressure Coefficients for Walls:

Refer Table 5 of code: $h/w = 0.5$, and $l/w = 2$
therefore $C_{pe}$ for walls* are given in Table 15-2.

Table 15-2

<table>
<thead>
<tr>
<th>Angle of Incidence</th>
<th>0°</th>
<th>90°</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall – A</td>
<td>+0.7</td>
<td>- 0.5</td>
</tr>
<tr>
<td>Wall – B</td>
<td>- 0.25</td>
<td>- 0.5</td>
</tr>
<tr>
<td>Wall – C</td>
<td>- 0.6</td>
<td>+ 0.7</td>
</tr>
<tr>
<td>Wall – D</td>
<td>- 0.6</td>
<td>- 0.1</td>
</tr>
</tbody>
</table>

These will be combined with internal pressure coefficients as earlier, equal to $C_{pi} = \pm 0.5$

$C_{pnet}$ for Walls A or B  $= 0.7 - (-0.5) = +1.2$, pressure

$= -0.5 - (+0.5) = -1.0$, suction

$C_{pnet}$ for Walls C or D  $= 0.7 - (-0.5) = +1.2$, pressure

$= -0.6 - (+0.5) = -1.1$, suction

Local pressure coefficients for the design of claddings and fasteners

Refer Table 6 of IS-875 for Roof Angle = 15°
Local $C_{pe}$ for eaves portion in end zone: NA
Local $C_{pe}$ for eaves portion in mid zone: NA
Local $C_{pe}$ for ridge portion: -1.2
Local $C_{pe}$ for gable edges (hipped part): -1.2x0.8 = -0.96

Local $C_{pe}$ for corners of walls: -1.0
(IS:875-pt.3, Table 5)

Therefore Max. local $C_{pnet}$ for roof at the edges and the ridge = -1.2 – (+0.5) = -1.7
Likewise at the wall edges = -1.0 – (+0.5) = -1.5

However, for the use of the local pressure coefficients, the design pressure $p_d$ will be computed with $K_a = 1$. Therefore, $p_d = 1.3254\times0.9 = 1.193$ kN/m²

Zone of local coefficients = 0.15x10 = 1.5m, at ridges, eaves and gable ends & 0.25x10 = 2.5m for wall corners. In this region the cladding and fasteners shall be checked for increased force

Calculations of Force due to Frictional Drag:

(IS:875-pt.3, Sec6.4.1)

This will act in the longitudinal direction of the building along the wind. Here $h>b$, therefore, first equation will be used & $C_f’ = 0.02$. This will be added to the wind force on gable walls. $K_a$ for roof and walls is 0.8, as area is more than 100m².
Figure-15.2 : Plan of the building. M, N are hipped slopes

Figure 15.3 – Net Roof Pressure Coefficients for different zones and combinations

For End Zone E/G; 0° wind incidence

\[
\begin{align*}
0.8 & \quad 0.4 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

\[
= \\
\begin{align*}
1.3 & \quad 0.9 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

OR

\[
\begin{align*}
0.8 & \quad 0.4 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

\[
= \\
\begin{align*}
0.3 & \quad 0.1 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

For End Zone E/G; 90° wind incidence

\[
\begin{align*}
0.75 & \quad 0.75 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

\[
= \\
\begin{align*}
1.25 & \quad 1.25 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

OR

\[
\begin{align*}
0.75 & \quad 0.75 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

\[
= \\
\begin{align*}
0.25 & \quad 0.25 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

For Mid Zone F/H; 0° wind incidence

\[
\begin{align*}
0.8 & \quad 0.4 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

\[
= \\
\begin{align*}
1.3 & \quad 0.9 \\
0.5 & \quad 0.5 \\
\end{align*}
\]

OR
For Mid Zone F/H; 90° wind incidence

For hipped slopes M,N : 0° wind incidence

For hipped slopes M,N : 90° wind incidence
Example 16 - Wind Pressure and Forces on a free standing duo-pitch roof of an unclad parking shed

Problem Statement:
Calculate wind pressure and design forces on a free standing duo-pitch roof of an unclad parking shed having dimensions 10m×50m and height of 5m up to eaves. The roof of shed is bent down, as in figure 16.1. The shed is located at Bareilly (UP) in the Transport Nagar area. A facia of 1m has been provided at both the longitudinal walls. The roof angle \( \alpha \) is 15°. Assume that full obstruction can occur on one side i.e. the solidity ratio \( \phi \) may vary from 0 to 1.0.

Solution:

Wind Data:
1. Wind Zone: Zone IV \( (V_b = 47 \text{m/s}) \) (IS:875-pt.3, Sec 5.2)
   Note: Bareilly is situated in Zone IV.
2. Terrain category: Category 2 (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient factor \( k_1' = 1.00 \)
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor \( k_2' = 1.00 \)
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor \( k_3' = 1.00 \)
(IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region \( k_d = 1.00 \)
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor \( K_d = 0.90 \)
(IS:875-pt.3, Sec 6.2.1)
Area Averaging factor \( K_a = 0.80^* \), for Roof
(IS:875-pt.3, Sec 6.2.2, Table-4)

\* The value of \( K_a \) is dependent on the tributary area. Thus, \( K_a \) may be computed by working out the tributary area for different elements, and using Table 4 of IS:875-pt.3, as illustrated in some of the previous examples.

Design Wind Pressure
Design Wind Speed \( V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_d = 47 \times 1.0 \times 1.0 \times 1.0 = 47.00 \text{ m/s} \) (IS:875-pt.3, Sec 5.3)
\( p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{ N/m}^2 \) (IS:875-pt.3, Sec 6.2)
\( p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times 1.0 = 1.193 \text{ kN/m}^2 \) (IS:875-pt.3, Sec 6.2)

Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

Wind Load Calculations:
\( F = (C_{pe} \times C_{pi}) \times A \times p_d \) (IS:875-pt.3, Sec 6.3.1)

Net Pressure Coefficients
(Roof angle +15°; \( h/w = 0.5 \) & \( L/w = 5 \))
Max. +ve roof pressure, for \( \phi = 0 \) \( \Rightarrow +0.4 \)
Max. -ve roof pressure, for \( \phi = 0 \) \( \Rightarrow -0.8 \)
Max. +ve roof pressure, for \( \phi = 1 \) \( \Rightarrow +0.4 \)
Max. -ve roof pressure, for \( \phi = 1 \) \( \Rightarrow -1.2 \)
(Forces on facia \( \Rightarrow +1.3 \))
Calculating solidity ratio:

\[ \phi = \frac{\text{area of obstruction perpendicular to wind}}{\text{min. area under canopy perpendicular to wind}} \]

Now depending on position as upwind or downwind, effect is to be considered. Only for downwind obstruction \( \phi \) is to be considered. For upwind blockage \( \phi = 0 \) is to be used.

**Design Pressure Coefficients for Roof:**

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. As per note below Table 9, each slope of the duo pitch canopy should be able to withstand forces using both the max. and min. coefficients, and the whole canopy should be able to support forces using one slope at the max. coefficient with the other slope at the min.

Hence, the design roof pressure combinations would be as given in figure 16.2.

**Local pressure coefficients for the design of claddings and fasteners**

<table>
<thead>
<tr>
<th></th>
<th>mid zone</th>
<th>gable ends</th>
<th>eaves zone</th>
<th>ridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi = 0, +\text{ve} )</td>
<td>+0.9</td>
<td>+1.9</td>
<td>+1.4</td>
<td>+0.4</td>
</tr>
<tr>
<td>( \phi = 0, -\text{ve} )</td>
<td>-0.9</td>
<td>-1.7</td>
<td>-1.4</td>
<td>-1.8</td>
</tr>
<tr>
<td>( \phi = 1, +\text{ve} )</td>
<td>+0.9</td>
<td>+1.9</td>
<td>+1.4</td>
<td>+0.4</td>
</tr>
<tr>
<td>( \phi = 1, -\text{ve} )</td>
<td>-1.5</td>
<td>-2.2</td>
<td>-1.9</td>
<td>-2.8</td>
</tr>
</tbody>
</table>

Therefore, the fasteners shall be designed for increased force as per \( C\eta_{\text{net}} = -2.8 \) to \(-1.7\), according to \( \phi \). The spacing in all end zones, extending upto \( L/10 = 5\text{m} \) at gable ends and \( w/10 = 1\text{m} \) at eaves and ridges shall be reduced appropriately.

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_d = 1 \). Therefore, \( p_d = 1.3254 \times 0.9 = 1.193 \text{ kN/m}^2 \)

Force on facia shall be used for the design of truss and columns.

**Figure 16-2: Net Roof Pressure Coefficients for different zones and combinations**

- **Both slopes at \(-\text{ve}\) pressure coefficients and \( \phi = 0 \) (case 1)**
  
  - 0.8
  
  - 1.3

- **Both slopes at \(+\text{ve}\) pressure coefficients and \( \phi = 0 \) (case 2)**
  
  - 0.4
  
  - 1.3
Both slopes at -ve pressure coefficients and \( \phi = 1 \) (case 3)

Both slopes at +ve pressure coefficients and \( \phi = 1 \) (case 4)

One slope at -ve and other at +ve pressure coefficient and \( \phi = 0 \) (case 5)

One slope at -ve and other at +ve pressure coefficient and \( \phi = 1 \) (case 6)

Case 1, 2 and 5 need not be analysed.
Example 17 - Wind Pressure and Forces on a free-standing duo-pitch roof of an unclad parking shed: Bent up

Problem Statement:
What difference will occur if the roof of Example 16 is bent up, as in figure 17.1. The roof angle $\alpha$ is $15^\circ$ and there is no facia. The roof is used at a railway yard where goods trains 3m high may stand by the side? Height at the eaves is 5m.

Solution:
Wind Data:
1. Wind Zone: Zone IV ($V_b = 47$ m/s)  
   (IS:875-pt.3, Sec 5.2)
2. Terrain category: Category 2

Design Factors:
Risk Coefficient factor $'k_1' = 1.00$  
   (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor $'k_2' = 1.00$  
   (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor $'k_3' = 1.00$  
   (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $k_4 = 1.00$  
   (IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor $'K_d' = 0.90$  
   (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor $'K_a' = 0.80^*$, for Roof  
   (IS:875-pt.3, Sec 6.2.2, Table-4)

* The value of $K_a$ is dependent on the tributary area. Thus, $K_a$ may be computed by working out the tributary area for different elements, and using Table 4 of IS:875-pt.3, as illustrated in some of the previous examples.

Design Wind Speed
Design Wind Speed $= V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00 \text{ m/s}$  
   (IS:875-pt.3, Sec 5.3)
$p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4 \text{ N/m}^2$  
   (IS:875-pt.3, Sec 6.2)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times 0.8 = 0.9544 \text{ kN/m}^2$, for roof  
   (IS:875-pt.3, Sec 6.2)

Refer note below Sec. 5.3 of IS:875-pt.3 for buildings less than 10m height, while making stability calculations and design of the frame.

Wind Load Calculations:
$F = (C_{pe} \times C_{pi}) \times A \times p_d$  
   (IS:875-pt.3, Sec 6.3.1)

Net Pressure Coefficients

With roof angle $-15^\circ$: $h/w = 0.5$ & $L/w = 5$
   (IS:875-pt.3, Table 9)
Max. +ve roof pressure, for $\phi = 0 \Rightarrow +0.5$
Max. -ve roof pressure, for $\phi = 0 \Rightarrow -0.6$
Max. +ve roof pressure, for $\phi = 0.82 \Rightarrow +0.5$
Max. -ve roof pressure, for $\phi = 0.82 \Rightarrow -0.76$

Calculating solidity ratio, $\phi$:  
   (IS:875-pt.3, Sec 6.3.3.3)
$= \text{area of obstruction perpendicular to wind / min. area under canopy perpendicular to wind}$
$= \{3/(5-5 \times \tan 15^\circ)\} = 3/3.66 = 0.82$. Assuming 3m height blockage. For upwind blockage $\phi = 0$ is to be used.
**Design Pressure Coefficients for Roof:**
Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. As per note below Table 9, each slope of the duo pitch canopy should be able to withstand forces using both the max. and min. coefficients, and the whole canopy should be able to support forces using one slope at the max. coefficient with the other slope at the min.

Hence, the design roof pressure combinations would be as shown in Fig. 17.2.

**Local Pressure Coefficients:**
Local pressure coefficients for design of cladding and fasteners obtained from IS:875-pt.3, Table 9 are given below:

Local pressure coefficients for the design of claddings and fasteners

<table>
<thead>
<tr>
<th></th>
<th>mid zone</th>
<th>Table 17-1</th>
<th>eaves zone</th>
<th>ridges</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi = 0$, +ve</td>
<td>+0.6</td>
<td>+1.5</td>
<td>+0.7</td>
<td>+1.4</td>
</tr>
<tr>
<td>$\phi = 0$, -ve</td>
<td>-0.8</td>
<td>-1.3</td>
<td>-1.6</td>
<td>-0.6</td>
</tr>
<tr>
<td>$\phi = .82$, +ve</td>
<td>+0.6</td>
<td>+1.5</td>
<td>+0.7</td>
<td>+1.4</td>
</tr>
<tr>
<td>$\phi = .82$, -ve</td>
<td>-1.05</td>
<td>-1.63</td>
<td>-1.85</td>
<td>-1.09</td>
</tr>
</tbody>
</table>

Therefore, the fasteners shall be designed for $C_{pnet} = -1.85$. The spacing in all end zones, extending up to $L/10 = 5\text{m}$ at gable ends and $w/10 = 1\text{m}$ at eaves and ridges, shall be reduced accordingly.

However, for the use of the local pressure coefficients, the design pressure $p_d$ will be computed with $K_p = 1$. Therefore, $p_d = 1.3254 \times 0.9 = 1.193\ \text{kN/m}^2$. 


Fig. 17.2 Net Roof Pressure Coefficients for different zones and combinations

Both slopes at –ve pressure coefficients and $\phi = 0$ (case 1)

Both slopes at +ve pressure coefficients and $\phi = 0$ (case 2)

Both slopes at –ve pressure coefficients and $\phi = 0.82$ (case 3)

Both slopes at +ve pressure coefficients and $\phi = 0.82$ (case 4)

One slope at –ve and other at +ve pressure coefficients and $\phi = 0$ (case 5)

One slope at -ve and other at +ve pressure coefficients and $\phi = 0.82$ (case 6)

Case 1, 2 and 5 need not be analysed.
Example 18 - Wind Pressure and Forces on a Free Standing Mono-slope Roof

Problem Statement:
Calculate wind pressure and design forces on a freestanding mono-slope roof of a canopy having dimensions 5m x 20m and height of 3m up to lower eaves. The canopy is located at Agra (UP) near the city center. The roof angle $\alpha$ is $10^0$. See figure 18.1.

Solution:

Wind Data:
1. Wind Zone: Zone IV ($V_b = 47$ m/s) 
   (IS:875-pt.3, Sec 5.2)
2. Terrain category: The structure is located near to city center where there will be numerous structures of medium height. This corresponds to the Terrain Category 3. Depending on the type of development, an intermediate condition between category 2 and 3 may also be selected and factor $'k_2'$ may be taken as mean-value. 
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient factor $'k_1'$ = 1.00 
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor $'k_2'$ = 0.91 
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography factor $'k_3'$ = 1.00 
(IS:875-pt.3, Sec 5.3.3.1)
Importance factor for Cyclonic Region $'k_4'$=1.00 
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor $'K_d'$=0.90 
(IS:875-pt.3, Sec 6.2.1)

Area Averaging factor $'K_a'$ = 0.80*, for Roof 
(IS:875-pt.3, Sec 6.2.2, Table-4)
* The value of $K_a$ is dependent on the tributary area. Thus, $K_a$ may be computed by working out the tributary area for different elements, and using Table 4 of IS:875-pt.3, as illustrated in some of the previous examples.

Design Wind Pressure

Design Wind Speed $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 0.91 \times 1.0 \times 1.0 = 42.77$ m/s 
(IS:875-pt.3, Sec 5.3)

$ p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (42.77)^2 = 1097.56$ N/m$^2$ 
(IS:875-pt.3, Sec 6.2)

$p_d = p_Z \times K_d \times K_a = 1.097 \times 0.9 \times 0.8 = 0.79$ kN/m$^2$, for roof 
(IS:875-pt.3, Sec 6.2)

Refer note below IS:875-pt.3, Sec. 5.3 for buildings less than 10m height, while making stability calculations and frame designing.

Wind Load Calculations:

$$ F = (C_{pe} \times C_{pi}) \times A \times p_d $$ 
(IS:875-pt.3, Sec 6.3.1)
Net Pressure Coefficients
Using the IS:875-pt.3, Table 8 with roof angle 10° and solidity ratio $\phi = 0$

For $h/w = 3.9/5 = 0.78$, and $L/w = 20/5 = 4$, pressure coefficients are tabulated below (though values are only given for $L/w$ up to 3).

Max. (largest +ve) overall coefficient = +0.5

Max. (largest -ve) overall coefficient = -0.9

Local coefficients:
At eaves, up to $0.10 \times w = 0.10 \times 5 = 0.5$ m
$\Rightarrow +1.6$ or $-2.1$

At ends, up to $0.10 \times L = 0.10 \times 20 = 2.0$ m
$\Rightarrow +2.4$ or $-2.0$

In mid zone
$\Rightarrow +1.2$ or $-1.5$

**Design Pressure Coefficients for Roof:**
Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Net design pressure coefficient shall be either +0.5 or -0.9. For cladding and fasteners, -1.5 shall be used. In end zones, spacing of fasteners shall be reduced to account for larger local pressures. For cladding design, value of +2.4 should be used.

However, for the use of the local pressure coefficients, the design pressure $p_d$ will be computed with $K_a = 1$. Therefore, $p_d = 1.097 \times 0.9 = 0.987 \text{kN/m}^2$
Example 19 - Wind Pressure and Forces on a Rectangular Clad Building: Multi-span Saw-tooth Roof

Problem Statement:
Calculate wind pressures and design forces on the walls and roof of a multi-span saw tooth (North light) roof building having 5 bays of 10m each. The building is 100m long and height to eaves is 10m, as shown in figure 19.1. The building is situated in Bokaro (WB) in an industrial area 500m inside open land on a fairly level topography. Walls of building have 40 openings of 1.5m×1.5m size. The roof is of GC sheeting & the roof angle $\alpha$ is 15°. Calculate also the local wind pressures on roof & wall cladding. The columns & trusses are at 5m c/c longitudinally, purlins are at 1.4m c/c and columns at Gable ends are at 5m c/c.

Fig. – 19.1

Solution:

Wind Data:
1. Wind Zone: Zone IV ($V_0= 47$ m/s)  
   (IS:875-pt.3, Sec 5.2)

2. Terrain category: Category 2  
   (IS:875-pt.3, Sec 5.3.2.1)

Note: A combined wind speed profile is to be worked out as per Appendix – B (Sec 5.3.2.4) but since height of boundary layer developed after 500m fetch length is more than building height of 12.68m, only effects due to Terrain Category 2 are to be considered. A combined profile would be needed in case of tall structures.

Design Factors:
Risk Coefficient factor ‘$k_1$’ = 1.00  
   (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor ‘$k_2$’ = 1.00 for walls, 1.03 for roofs

Wind Directionality factor ‘$K_d$’= 0.90  
   (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor ‘$K_a$’ = 0.80, Short walls, for Long walls & Roof*  
   (IS:875-pt.3, Sec 6.2.2, Table-4)

Topography factor ‘$k_3$’ = 1.00  
   (IS:875-pt.3, Sec 5.3.3.1)
Importance factor for Cyclonic Region ‘$k_4$’= 1.00  
   (IS:875-pr.3, Sec 5.3.4)

Area of short (gable) walls = 5 × 2 × (10+12.68) ×0.5 × 10 = 1134 m$^2$
Area of long walls = 100 ×12.68 + 100 ×10 = 2268 m$^2$
Area of roof = 100 ×10.35 = 1035 m$^2$

* The value of $K_s$ is dependent on the tributary area. Thus, $K_s$ may be computed by working out the tributary area for different elements, and using Table 4.
Permeability of the Building:
Area of all the walls = 1134 + 2268 = 3402 m²
Area of all the openings = 20 × 1.5 × 1.5 × 2 = 90 m²
% Opening area = 2.65 %, less than 5%
Hence the building is of low permeability.

(IS:875-pt.3, Sec 6.3.2.2)

Design Wind Pressure

Design Wind Speed = \( V_z = V_b \times k_1 \times k_2 \times k_3 \times k_4 \)
47 × 1.0 × 1.0 × 1.0 = 47.00 m/s, for walls
47 × 1.03 × 1.0 × 1.0 = 48.41 m/s, for roof
(IS:875-pt.3, Sec 5.3)

\[ p_z = 0.6 \times (V_z)^2 = 0.6 \times 47.00^2 = 1325.4 \text{ N/m}^2 \text{, for walls} \]
\[ = 0.6 \times 48.41^2 = 1406.1 \text{ N/m}^2 \text{, for roof} \]
(IS:875-pt.3, Sec 6.2)

\[ p_d = p_z \times K_d \times K_a = 1.3254 \times 0.9 \times 0.8 \]
\[ = 0.9543 \text{ kN/m}^2 \text{ (for walls)} \]
(IS:875-pt.3, Sec 6.2)

Wind Load Calculations:

\[ F = (C_{pe} \times C_{pi}) \times A \times p_d \]
(IS:875-pt.3, Sec 6.3.1)

Internal Pressure Coefficient \( C_{pi} = \pm 0.2 \)
(IS:875-pt.3, Sec 6.3.2.1)

Note: buildings shall be analysed once for pressure of 0.2 from inside and then for a suction of –0.2 from inside (refer note 2 Sec 6.3.1) along with external pressure coefficient.

External Pressure Coefficients

Using Tables 5, 6 and 11 (with values in table 11 to take precedence).

\[ h_{av} / w = 11.34 / 50 = 0.227 \text{ & } l / w = 100 / 50 = 2.0 \]
(IS:875-pt.3, Sec 6.3.3.4)

Pressure coefficients for roof are tabulated in Table 19-1.

**Note:** As there is no mention of \( l/w \) ratio & extent of End zones, at ends, these can be considered up to width of one bay i.e. 10 m in this case.

**Table 19-1**

<table>
<thead>
<tr>
<th>Portion of roof</th>
<th>Wind Incidence Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0°</td>
</tr>
<tr>
<td>A</td>
<td>+0.7</td>
</tr>
<tr>
<td>B</td>
<td>-0.9</td>
</tr>
<tr>
<td>C</td>
<td>-0.9</td>
</tr>
<tr>
<td>D</td>
<td>-0.5/0.2</td>
</tr>
<tr>
<td>M</td>
<td>-0.5/0.5</td>
</tr>
<tr>
<td>N</td>
<td>-0.5/0.3</td>
</tr>
<tr>
<td>W</td>
<td>-0.3/0.5</td>
</tr>
<tr>
<td>X</td>
<td>-0.4</td>
</tr>
<tr>
<td>Y</td>
<td>-0.2</td>
</tr>
</tbody>
</table>

**Note:** As there is no mention of \( l/w \) ratio & extent of End zones, at ends, these can be considered up to width of one bay i.e. 10 m in this case.

**Values are from IS:875-pt.3, Table 5 & 7.**

+ : Additional values of \(-0.05(n-1)\), with \( n=4 \) is applicable in zone \( E/G \) for a distance equal to \( h \)

\( C_{pe} \) for walls is taken from table 5 for \( h/w < 0.5 \) and \( l/w = 2 \) and from Table 11. This is as follows:
Design Pressure Coefficients for Roof:

Positive internal pressure will act towards the roof while negative internal pressure will be away from the roof. Hence +ve internal pressure will be added to –ve external pressure coefficient and vice-a-versa. The combinations will have to be made separately for all the surfaces, as under:

<table>
<thead>
<tr>
<th>Wind angle</th>
<th>Short Wall 50 m wide</th>
<th>Long wall 100m long</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°</td>
<td>-0.6</td>
<td>+0.7/-0.2</td>
</tr>
<tr>
<td>90°</td>
<td>+0.7/-0.1</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

Analysis of truss is to be done for all above combinations.

Design Pressure Coefficients for Walls:

\( C_{pe} \) will be combined with internal pressure coefficients as earlier, equal to \( C_{pi} = \pm 0.2 \)

\( C_{pnet} \) for Walls A or Y  
\[ = +0.7 - (-0.2) = +0.9, \text{ pressure} \]
\[ = -0.5 - (+0.2) = -0.7, \text{ suction} \]

\( C_{pnet} \) for Gable Walls  
\[ = +0.7 - (-0.2) = +0.9, \text{ pressure} \]
\[ = -0.6 - (+0.2) = -0.8, \text{ suction} \]

Local coefficients for roof: Maximum of all the values given, in Table 7 & 11, i.e. –2.0 up to 0.1 x 100 = 10.0m on ends & 0.1 x 10m = 1.0m at ridges towards sloping side of roof. In this region the fasteners shall be designed to carry increased force calculated with \( C_{pnet} = -2.0 - (+0.2) = -2.2 \)

However, for the use of the local pressure coefficients, the design pressure \( p_d \) will be computed with \( K_a = 1 \). Therefore, \( p_d = 1.4061 \times 0.9 = 1.2655 \text{ kN/m}^2 \)
Example 20 - Wind Forces on a Free Standing Framed Compound Wall with Barbed Wire Fencing at Top

Problem Statement:
Calculate wind pressure and design forces on a continuous compound wall 2.1m high in RC frame and masonry construction with barbed wire fencing over it, as shown in figure 20.1, and located in Indore (MP) to enclose a land piece near the Airport.

Solution:

Wind Data:
1. Wind Zone: Zone II \( (V_b = 39 \text{m/s}) \)
   (IS:875.pt.3, Fig. 1, Sec 5.2)
2. Terrain category: Category 1 (open land)
   (IS:875- pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient factor \( k_1 = 0.92^* \)
   (IS:875- pt.3, Sec 5.3.1, Table 1)
Terrain & Height factor \( k_2 = 1.05 \)
   (IS:875- pt.3, Sec 5.3.2.2, Table 2)
Topography factor \( k_3 = 1.00 \)
   (IS:875- pt.3, Sec 5.3.3.1)
Importance factor for Cyclonic Region \( k_4 = 1.00 \)
   (IS:875- pt.3, Sec 5.3.4)
Wind Directionality factor \( K_d = 0.90 \)
   (IS:875- pt.3, Sec 6.2.1)
Area Averaging factor \( K_a = 1.00^{**} \)
   (IS:875- pt.3, Sec 6.2.2, Table 4)

* : though table 1 mentions boundary walls to be designed for 5 yrs. life, but considering 25 years of period for framed walls.
** : considering tributary area = 3 \( \times \) 2.1 = 6.3m\(^2\), for the design of columns

Wind Load Calculations:
\[ F = C_f \times A \times p_d \]
   (IS:875- pt.3, Sec 6.4)

Wind Force on Barbed wire fencing:
Assuming the solidity ratio of wire fencing and angles = 0.1
\[ A_r = 0.1 \times 1.0 \times 0.6 = 0.06 \text{m}^2, \text{ taking 1m length of wall.} \]
\[ C_f = 1.9, \text{ for flat sided single member frames} \]
   (IS:875- pt.3, Sec 6.4.3.3, Table 31)
Force on wire fencing per m length = 1.9 \( \times \) 0.06 \( \times \) 0.8905 = 0.1015 kN

Design Wind Pressure
Design Wind Speed \( = V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 0.92 \times 1.05 \times 1.0 \times 1.0 = 45.402 \text{ m/s} \)
   (IS:875- pt.3, Sec 5.3)
\[ p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (45.402)^2 = 1236.805 \text{ N/m}^2 \]
   (IS:875- pt.3, Sec 6.2)
\[ p_d = p_Z \times K_d \times K_a = 1.2368 \times 0.9 \times 1.00 = 1.113 \text{ kN/m}^2 \]
   (IS:875- pt.3, Sec 6.2)
These are to be reduced by 20% as the wall is less than 10m high, as per note below section 5.3.
Hence \( p_d = 1.113 \times 0.8 = 0.8905 \text{ kN/m}^2 \)

These are to be reduced by 20% as the wall is less than 10m high, as per note below section 5.3.
Hence \( p_d = 1.113 \times 0.8 = 0.8905 \text{ kN/m}^2 \)

Wind Load Calculations:
\[ F = C_f \times A \times p_d \]
   (IS:875- pt.3, Sec 6.4)

Wind Force on Barbed wire fencing:
Assuming the solidity ratio of wire fencing and angles = 0.1
\[ A_r = 0.1 \times 1.0 \times 0.6 = 0.06 \text{m}^2, \text{ taking 1m length of wall.} \]
\[ C_f = 1.9, \text{ for flat sided single member frames} \]
   (IS:875- pt.3, Sec 6.4.3.3, Table 31)
Force on wire fencing per m length = 1.9 \( \times \) 0.06 \( \times \) 0.8905 = 0.1015 kN
acting at $2.1 + 0.3 = 2.4\text{m}$ above ground.
Reduction factor $K = 1$ is taken

**Wind Force on wall:**

Since the length of wall is more than 100m, $b/h = 100/2.1 = 47.62$ and the wall is from ground, $C_f = 1.55$, after linear interpolation

(IS:875-pt.3, Table 26)

Design Wind Force on walls, therefore

$F = 1.55 \times 1.0 \times 0.8905 \times 2.1 = 2.90 \text{kN}$ acting at 1.05m from ground.
Oblique wind effects as per 6.4.2.3 and now considered necessary as the wall has $l \gg b$. 
Example 21 - Wind Forces on a Sign Board Hoarding

Problem Statement:
Calculate wind pressure and design forces on a hoarding 10m long and 5m high, to be fixed at the roof of a 24m high building near Cannaught Place area in New Delhi. The base of the hoarding board is 2.0m above the roof level. See figure 21.1.

Solution:
Wind Data:
1. Wind Zone: Zone IV ($V_b= 47m/s$)  
   (IS:875-pt.3, Fig. 1, Sec 5.2)
2. Terrain category: Category 3 (near City Center)  
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient factor ‘$k_1$’ = 0.71*  
   (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height factor ‘$k_2$’ = 1.05**  
   (IS:875-pt.3, Sec 5.3.2.2, Table-2)

Topography factor ‘$k_3$’ = 1.00  
   (IS:875-pt.3, Sec 5.3.3.1)
Importance factor for Cyclonic Region ‘$k_4$’ =1.00  
   (IS:875-pt.3, Sec 5.3.4)
Wind Directionality factor ‘$K_d$’=0.90  
   (IS:875-pt.3, Sec 6.2.1)
Area Averaging factor ‘$K_a$’ = 1.0***  
   (IS:875-pt.3, Sec 6.2.2, Table-4)

*: considering design life of 5 yrs.
**: For average height of the hoarding, 28.5 m
*** : considering tributary area = 5 \times 2.0 = 10.0 m^2, for the design of Frame supports

**Design Wind Pressure**

Design Wind Speed \( V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 0.71 \times 1.05 \times 1.0 \times 1.0 = 35.04 \text{ m/s} \)

(IS:875-pt.3, Sec 5.3)

\[ p_Z = 0.6 \times (V_Z)^2 = 0.6 \times (35.04)^2 = 736.62 \text{ N/m}^2 \]

(IS:875-pt.3, Sec 6.2)

\[ p_d = p_Z \times K_d \times K_a = 0.737 \times 0.9 \times 1.00 = 0.6633 \text{ kN/m}^2 \]

(IS:875-pt.3, Sec 6.2)

**Wind Load Calculations:**

\[ F = C_f \times A \times p_d \]

(IS:875-pt.3, Sec 6.4)

**Wind Force on Hoarding:**

Since the length of hoarding is 10m, \( b/h = 10/5 = 2.0 \) and the hoarding is 2m above roof, \( C_f = 1.2 \)

(IS:875-pt.3, Table 26)

Design Wind Force on hoarding, therefore

\[ F = 1.2 \times 1.0 \times 0.6633 \times 5.0 = 3.98 \text{ kN acting at } (2 + 2.5) = 4.5 \text{ m above roof} \]

To allow for oblique winds, force coefficients of 1.7 and 0.44 are to be taken at two ends, as per section 6.4.2.3.

Accordingly, \( 3.98 \times 1.7/1.2 = 5.64 \text{ kN at windward edge and} \)

\( 3.98 \times 0.44/1.2 = 1.46 \text{ kN at leeward edge shall be considered, per meter width of hoarding.} \)

Pressure distribution

In vertical direction the wind force may be considered constant over the height.

The hoarding sheet will be designed for a force of \( 5.64/5 = 1.128 \text{ kN/m}^2 \)

The frame of hoarding will be designed for average pressure intensity depending on the spacing of vertical frames.
Example 22: Wind Pressure and Forces on an Overhead Intze Type RCC Water Tank on Framed Staging

Problem Statement:
Calculate design wind pressure on a circular overhead water tank of Intze type, supported on a 12-column staging 12m high, as shown in figure-22.1. The columns are 40 cm dia and the braces 20 cm × 40 cm. The tank is proposed to be constructed in a residential locality of New Delhi.

Solution:

Wind Data:
1. Wind Zone: Zone IV (\(V_b = 47\text{m/s}\))
   (IS:875-pt.3, Sec 5.2, Fig. 1)
2. Terrain Category: A residential locality corresponds to Terrain Category 3, as defined in IS-875
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient Factor \(k_1 = 1.07\)
   (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor \(k_2\), varies with height and is given in Table 22.1.
   (IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography Factor \(k_3 = 1.00\)
   (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region \(k_4 = 1.00\)
   (IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor \(K_d = 1.00\)
   (IS:875-pt.3, Sec 6.2.1)
Area Averaging Factor \(K_a = 1.00\), for Staging
   = 0.842*, for tank portion
   (IS:875-pt.3, Sec 6.2.2, Table-4)

\(*\): area of tank (cylindrical and conical part) = 12 x 4 + 10 x 2 = 68 m²

Design Wind Pressure
Design Wind Speed = \(V_Z = V_b * k_1 * k_2 * k_3 * k_4\)
   = 47*1.07*1.0*1.0 = (50.3* \(k_2\)) m/s
   (IS:875-pt.3, Sec 5.3)
\(p_Z = 0.6 \ (V_Z)^2 \ & \ p_d = p_Z * K_d * K_a\)
   (IS:875-pt.3, Sec 5.4 & Sec 6.2)

Wind Load Calculations:
External pressure coefficients for roof and bottom of tank:
\((z/H) - 1 = (19.5/7.5) - 1 = 1.6\)
   (IS:875-pt.3, Sec 6.3.3.8, Table 20)
Therefore, \(C_{pz} = -0.75\) for roof and \(-0.6\) for bottom.
Eccentricity of force at roof
   = 0.1 x D = 0.1 x 12 = 1.2m
Total force acting on the roof of structure
   \(P = 0.785 * D^2 * (p_1 - C_{pz} * p_d)\)
   = 0.785 x 12² x {0-(-0.75) x 1.082}
   = 91.732 kN acting upwards at 1.2m from center of dome
   (IS:875-pt.3, Sec 6.3.3.8)
Note: $p_i$, the internal pressure inside the tank may be due to any liquid stored or for water tanks where there is no pressure due to stored water, internal pressure will be generated due to small permeability which may exist due to openings at roof level e.g. in steel tanks. If no openings exist, as in RCC water tanks, $p_i = 0$.

Roof pressure will be used with Gravity loads for design of dome.

**Overall Horizontal Force on the Tank:**

$$ F = C_f \times A_e \times p_d $$  
(IS:875-pt.3, Sec 6.4)

No horizontal force will act on top dome. The effect of wind pressure on dome has been included in the net vertical force, as above, associated with an eccentricity.

Cylindrical portion:

$V_z (avg) = (46.276+44.16)/2 = 45.218$ m/s  
$V_z \times b = 45.218 \times 12 = 542 > 6$,  
$h/b = 4/12 = 0.333 < 2$

Therefore, $C_f = 0.7$, from table 25(rough) &  
$p_d = 1.50 \times 1.0 \times 0.842$  
$= 1.263$ kN/m$^2$ at top &  
$= 1.393 \times 1.0 \times 0.842$  
$= 1.173$ kN/m$^2$ at bottom

This being a very small difference, higher value may be taken.

$F_{cylinder} = 0.7 \times 12 \times 1.263$  
$= 10.61$ kN/m height

Conical bottom:

$V_z (avg) = (44.16+42.96)/2 = 43.56$ m/s  
$V_z \times b = 43.56 \times 10 = 435 > 6$,  
$h/b = 2/10 = 0.2 < 2$

Therefore, $C_f = 0.7$, from table 20 &  
$p_d = 1.393 \times 1.0 \times 0.842$  
$= 1.173$ kN/m$^2$ at top &  
$= 1.324 \times 1.0 \times 0.842$  
$= 0.955$ kN/m$^2$ at bottom

This being a very small difference, higher value may be taken.

$F_{conical dome} = 0.7 \times 10 \times 1.173$  
$= 8.211$ kN/m height

Staging:

$p_d = 1.257 \times 1 \times 1 = 1.257$ kN/m$^2$ up to 10m  
$= 1.324 \times 1 \times 1 = 1.324$ kN/m$^2$ above 10m height.

In order to calculate the wind force on columns, each column is considered as an individual member (IS:875-pt.3, Sec 6.4.2.2, table 25) and no shielding effect is considered on leeward columns, as the columns are placed far apart on periphery only.

Therefore, for one column:

$V_z \times b = 42.96 \times 0.4 = 17.2 > 6$  
$h/b = 11.4/0.4 = 28.5 > 20$

Therefore, $C_f = 1.2$, from table 25 for rough surface finish.

$F_{column} = 1.2 \times 0.4 \times 1.257$  
$= 0.603$ kN/m height, up to 10m height  
$F_{column} = 1.2 \times 0.4 \times 1.324$  
$= 0.6355$ kN/m height, above 10m height

$F_{bracings} = 1.0 \times \{2 \times (8.0-7\times.4)\} \times 1.257$  
$= 13.072$ kN/m height, acting at two brace levels, 4m and 8m. This is calculated considering it as an individual member and using table 25 with h/b ratio < 2. (Assuming 0.2x0.4m size braces)

$F_{ringbeam} = 1.0 \times 8.0 \times 1.324$  
$= 10.592$ kN/m height, as above.

Note: Cf values taken from Table 25 are for members of infinite length. Reduction factors for finite length of container, columns and other members can be taken from Table 29, which will further reduce wind forces.

**************************************************
Table 22-1: Calculations of Variation in Design Wind Speed & Pressure with Height

<table>
<thead>
<tr>
<th>Height from Ground, m</th>
<th>$k_z$ *</th>
<th>$V_Z$ m/s</th>
<th>$p_z$ kN/m$^2$</th>
<th>$p_y$ kN/m$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 10m</td>
<td>0.91</td>
<td>45.773</td>
<td>1.257</td>
<td>1.257</td>
</tr>
<tr>
<td>12m</td>
<td>0.854</td>
<td>46.98</td>
<td>1.324</td>
<td>1.324</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.955 (for staging)</td>
</tr>
<tr>
<td>14m</td>
<td>0.878</td>
<td>48.187</td>
<td>1.393</td>
<td>1.173</td>
</tr>
<tr>
<td>18m</td>
<td>0.92</td>
<td>50.00</td>
<td>1.50</td>
<td>1.263</td>
</tr>
</tbody>
</table>

* : $k_z$ values are linearly interpolated.

---

Table 22-2: Summary of forces and total loads on tank

<table>
<thead>
<tr>
<th>Element</th>
<th>Force per unit height</th>
<th>Height of element</th>
<th>Total horizontal force</th>
<th>CG of force from ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylindrical portion</td>
<td>10.61 kN</td>
<td>4.0m</td>
<td>42.44 kN</td>
<td>16.00m</td>
</tr>
<tr>
<td>Conical Dome</td>
<td>8.211 kN</td>
<td>2.0m</td>
<td>16.422 kN</td>
<td>13.067m</td>
</tr>
<tr>
<td>Top Ring Beam</td>
<td>10.592 kN</td>
<td>0.6m</td>
<td>6.355 kN</td>
<td>11.7m</td>
</tr>
<tr>
<td>All Columns above 10m</td>
<td>0.6355 kN x 12 = 7.626 kN</td>
<td>1.4m</td>
<td>10.676 kN</td>
<td>10.7m</td>
</tr>
<tr>
<td>All Columns up to 10m</td>
<td>0.603 kN x 12 = 7.236 kN</td>
<td>10m</td>
<td>72.36 kN</td>
<td>5.0m</td>
</tr>
<tr>
<td>Braces, upper level</td>
<td>13.072 kN</td>
<td>0.4m</td>
<td>5.229 kN</td>
<td>8.0m</td>
</tr>
<tr>
<td>Braces, lower level</td>
<td>13.072 kN</td>
<td>0.4m</td>
<td>5.229 kN</td>
<td>4.0m</td>
</tr>
</tbody>
</table>

********************
Example 23: Wind Pressure and Forces on a Square Overhead RCC Water Tank on Framed Staging

Problem Statement:
Calculate design wind pressure on a square overhead RCC water tank of size 12m x 12m supported on a 16-column framed staging 12m high, as shown in figure-23.1. The columns are 400mm square and the braces 20 cm × 40 cm. The tank is proposed to be constructed in a residential locality of New Delhi.

Solution:
Wind Data:
1. Wind Zone: Zone IV (\( V_b = 47 \text{ m/s} \))
   (IS:875-pt.3, Sec 5.2, Fig. 1)
2. Terrain Category: A residential locality corresponds to Terrain Category 3, as defined in IS-875
   (IS:875-pt.3, Sec 5.3.2.1)

Design Factors:
Risk Coefficient Factor \( k_1 = 1.07 \)
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor \( k_2 \), varies with height and is given in Table 23.1.
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography Factor \( k_3 = 1.00 \)
(IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region \( k_4 = 1.00 \)
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor \( K_d = 0.9 \)
(IS:875-pt.3, Sec 6.2.1)
Area Averaging Factor \( K_a = 1.00 \), for Staging
= 0.87*, for tank portion
(IS:875-pt.3, Sec 6.2.2, Table-4)
*: exposed area of tank container = 12 x 4 = 48 m²

Design Wind Pressure
Design Wind Speed\( ^* \) = \( V_Z = V_b \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4 \cdot K_d \)
= \( 47 \cdot 1.07 \cdot k_2 \cdot 1.0 \cdot 1.0 \cdot (50.3 \cdot k_2) \) m/s
(IS:875-pt.3, Sec 5.3)
\[ p_Z = 0.6 \cdot (V_Z)^2 \] \& \[ p_d = p_Z \cdot K_d \cdot K_a \]
(IS:875-pt.3, Sec 6.2 & Sec 6.2)

Wind Load Calculations:
External pressure coefficients for roof and bottom of tank:* \( \frac{z}{H} - 1 = \frac{16}{4.0} - 1 = 3.0 \)
\( H/D = 4/12 = 0.333 \)
(IS:875-pt.3, Sec 6.3.3.8, Table 20)
*: as there is no direct mention about square or rectangular tanks, parameters from different clauses of code are to be taken.
Therefore, $C_{pe} = -0.65$ for roof & $-0.6$ for bottom.

Eccentricity of force at roof = $0.1 \times D$

$$= 0.1 \times 12 = 1.2 \text{m}$$

The total force acting on the roof of the tank

$$= A_x (p_i - C_{pe} \times p_d) = 12 \times 12 \times \{0 - (-0.65) \times 1.35\}$$

$$= 126.36 \text{ kN acting upwards at } 1.2 \text{ m from center.}$$

Note: $p_i$, the internal pressure inside the tank may be due to any liquid stored or for water tanks where there is no pressure due to stored water, internal pressure will be generated due to small permeability which may exist due to openings at roof level e.g. in steel tanks. If no openings exist, as in RCC water tanks, $p_i = 0$.

Roof pressure will be used with Gravity loads for design of slab.

**Overall Horizontal Force on the Tank:**

$$F = C_f \times A_x \times p_d$$

(IS:875-pt.3, Sec 6.4, 6.4.2.2)

Container portion:

$b/d = 1, V_z \times b = 49.19 \times 12 = 590 > 10, H/b=0.33$

$C_f = 0.5$, from table 25.

$$F_{container} = 0.5 \times 12 \times 1.137$$

$$= 6.822 \text{ kN/m height}$$

Staging:

$p_d = 1.131 \times 1 \times 0.9 = 1.018 \text{ kN/m}^2 \text{ up to } 10\text{m}$

$$= 1.192 \times 1 \times 0.9 = 1.073 \text{ kN/m}^2 \text{ above } 10\text{m height.}$$

The staging is multiple bay framed type and cl.6.4.3.3 with table 26 and cl.6.4.3.4 with table 32 are therefore applied.

Solidity ratio of one frame = $(4 \times 0.4 \times 12 + 10.4 \times 0.4 \times 2)/(12 \times 12) = 0.19$

$$\Rightarrow C_f = 1.8, \text{ for windward frame members.}$$

Shielding Effect on leeward frame:

Frame- spacing ratio = $4.0/0.4 = 10 \Rightarrow \eta = 1.0$

Hence no shielding occurs.

Alternatively considering each column/bracing as an individual member, for columns:

$V_z \times b = 42.96 \times 0.4 = 17.2 > 10$

$h/b = 12/0.4 = 30 > 20$

Therefore, $C_f = 1.2$, from table 25 for rough surface finish. The higher value of $C_f$ i.e. 1.8 is used

$$F_{column} = 1.8 \times 0.4 \times 1.131$$

$$= 0.814 \text{ kN/m height, up to } 10\text{m height}$$

$$F_{column} = 1.8 \times 0.4 \times 1.192$$

$$= 0.858 \text{ kN/m height, above } 10\text{m height}$$

for Bracings:

$V_z \times b = 42.96 \times 0.4 = 17.2$

$h/b = 0.4/3.466 = 0.155 < 2$

Therefore, $C_f = 1.0$, from table 25 for rough surface finish.

$$F_{bracings} = 1.0 \times \{3(12.0-4x0.4)\} \times 1.131$$

$$= 35.29 \text{ kN/m height, acting at two brace levels, 4m and 8m.}$$

..............................................................................................................
Table 23-1: Calculations of Variation in Design Wind Speed & Pressure with Height

<table>
<thead>
<tr>
<th>Height from Ground, m</th>
<th>$k_2$ *</th>
<th>$V_Z$ m/s</th>
<th>$p_Z$ kN/m$^2$</th>
<th>$p_d$ kN/m$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 10m</td>
<td>0.91</td>
<td>45.774</td>
<td>1.257</td>
<td>1.131 (for staging)</td>
</tr>
<tr>
<td>12m</td>
<td>0.934</td>
<td>46.98</td>
<td>1.324</td>
<td>1.192 (for staging)</td>
</tr>
<tr>
<td>16m</td>
<td>0.978</td>
<td>49.19</td>
<td>1.451</td>
<td>1.137 (for tank)</td>
</tr>
</tbody>
</table>

$k_2$ values are linearly interpolated.

Table 23-2: Summary of forces and total loads on tank

<table>
<thead>
<tr>
<th>Element</th>
<th>Force per unit height</th>
<th>Height of element</th>
<th>Total horizontal force</th>
<th>CG of force from ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>Container portion</td>
<td>6.822 kN</td>
<td>4.0m</td>
<td>27.288 kN</td>
<td>14.0m</td>
</tr>
<tr>
<td>All Columns above 10m</td>
<td>0.858 kN</td>
<td>2.0m</td>
<td>27.456 kN</td>
<td>11.0m</td>
</tr>
<tr>
<td></td>
<td>x 16 = 13.728kN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>All Columns up to 10m</td>
<td>0.814 kN</td>
<td>10m</td>
<td>130.24 kN</td>
<td>5.0m</td>
</tr>
<tr>
<td></td>
<td>x 16 = 13.024 kN</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Braces, upper level</td>
<td>35.29 kN</td>
<td>0.4m</td>
<td>14.116 kN</td>
<td>8.0m</td>
</tr>
<tr>
<td>Braces, lower level</td>
<td>35.29 kN</td>
<td>0.4m</td>
<td>14.116 kN</td>
<td>4.0m</td>
</tr>
</tbody>
</table>

********************
**Example 24: Wind Pressure and Forces on an Overhead Intze Type RCC Water Tank on Shaft Staging**

**Problem Statement:**
Calculate design wind pressures on a circular overhead water tank of Intze type, supported on an RC shaft staging 12m high, as shown in figure 24.1. The tank is proposed to be constructed in a residential locality of New Delhi.

**Solution:**

**Wind Data:**
1. Wind Zone: Zone IV \( (V_b = 47 \text{m/s}) \)  
   (IS:875-pt.3, Sec 5.2 Fig. 1)
2. Terrain Category: A residential locality corresponds to Terrain Category 3.  
   (IS:875-pt.3, Sec 5.3.2.1)

**Design Factors:**
- Risk Coefficient Factor \( k_1 = 1.07 \)  
  (IS:875-pt.3, Sec 5.3.1, Table-1)
- Terrain & Height Factor \( k_2 \) = Varies with height, and is given in Table 23.1  
  (IS:875-pt.3, Sec 5.3.2.2, Table-2)
- Topography Factor \( k_3 = 1.00 \)  
  (IS:875-pt.3, Sec 5.3.3.1)
- Importance Factor for Cyclonic Region \( k_4 = 1.00 \)  
  (IS:875-pt.3, Sec 5.3.4)
- Wind Directionality Factor \( K_d = 1.00 \)  
  (IS:875-pt.3, Sec 6.2.1)
- Area Averaging Factor \( K_a \) = 0.805*, for Staging  
  (IS:875-pt.3, Sec 6.2.2, Table-4)
- Area of Shaft: \( = 12 \times 0.8 \times 8 \times 0.8 = 96 \text{m}^2 \)
- Area of Tank (cylindrical and conical part) \( = 12 \times 4 + 10 \times 2 = 68 \text{m}^2 \)

**Design Wind Pressure**
Design Wind Speed \( V_Z = \frac{V_b \times k_1 \times k_2 \times k_3 \times k_4}{K_a} \)  
\( = 47 \times 1.07 \times 1.0 \times 1.0 = 50.3 \times k_2 \) m/s  
(IS:875-pt.3, Sec 5.3)

\( p_Z = 0.6 \times (V_Z)^2 \) \& \( p_d = p_Z \times K_a 	imes K_d \)  
(IS:875-pt.3, Sec 6.2 & Sec 6.2)

**Wind Load Calculations:**
External pressure coefficients for roof of tank:  
(IS:875-pt.3, Sec 6.3.3.8)
\( (z/H) - 1 = (19.5/7.5) - 1 = 1.6 \)  
(IS:875-pt.3, Table 20)

Therefore, \( C_{pe} = 0.75 \) for roof

Eccentricity of force at roof \( = 0.1 \times 12 = 1.2 \text{m} \)

Total vertical force acting on the roof of structure \( P = 0.785 \times D^2 \times (p_i - C_{pe} \times p_d) \)  
\( = 0.785 \times 1^2 \times (0.75 \times 1.082) \)  
\( = 91.732 \text{kN acting upwards at 1.2m from center of dome} \)  
(IS:875-pt.3, Sec 6.3.3.8)

Note: \( p_i \), the internal pressure inside the tank may be due to any liquid stored or for water tanks where there is no
pressure due to stored water, internal pressure will be generated due to small permeability which may exist due to openings at roof level e.g. in steel tanks. If no openings exist, as in RCC water tanks, \( p_t = 0 \).

Roof pressure will be used with Gravity loads for design of dome.

**Overall Horizontal Force on the Tank:**

\[
F = C_f \times A_e \times p_d
\]

(IS:875-pt.3, Sec 6.4.6.4.2.2)

No horizontal force will act on top dome. The effect of wind pressure on dome has been included in the net vertical force, as above, associated with an eccentricity.

**Cylindrical portion:**

\[
V_z (\text{avg}) = \frac{(46.276+44.16)}{2} = 45.218 \text{ m/s}
\]

\[
V_z \times b = 45.218 \times 12 = 542 > 6,
\]

\[
h/b = 4/12 = 0.333 < 2
\]

Therefore, \( C_f = 0.7 \), from table 25, for rough surface & \( p_d = 1.50 \times 1.0 \times 0.842 = 1.263 \text{ kN/m}^2 \) at top & 
\[
= 1.393 \times 1.0 \times 0.842 = 1.173 \text{ kN/m}^2 \text{ at bottom}
\]

This being a very small difference, higher value may be taken.

\[
F_{\text{cylinder}} = 0.7 \times 12 \times 1.263 = 10.61 \text{ kN/m height}
\]

Conical bottom:

\[
V_z (\text{avg}) = \frac{(44.16+42.96)}{2} = 43.56 \text{ m/s}
\]

\[
V_z \times b = 43.56 \times 10 = 435 > 6,
\]

\[
h/b = 2/10 = 0.2 < 2
\]

Therefore, \( C_f = 0.7 \), from table 25, for rough surface & \( p_d = 1.393 \times 1.0 \times 0.842 = 1.173 \text{ kN/m}^2 \) at top & 
\[
= 1.324 \times 1.0 \times 0.842 = 1.115 \text{ kN/m}^2 \text{ at bottom}
\]

This being a very small difference, higher value may be taken.

\[
F_{\text{conical dome}} = 0.7 \times 10 \times 1.173 = 8.211 \text{ kN/m height}
\]

**Staging:**

Shaft is considered as circular member with rough roughness at surface for which \( V_z \times b = 42.96 \times 8.0 = 343 > 6, \)

\[
h/b = 12.0/8.0 = 1.5 < 2
\]

Therefore, \( C_f = 0.7 \), from table 25, for rough surface

\[
F_{\text{shaft}} = 0.7 \times 8.0 \times 1.257 \times 0.805 = 5.67 \text{ kN/m height, up to 10m height}
\]

\[
F_{\text{shaft}} = 0.7 \times 8.0 \times 1.324 \times 0.805 = 5.97 \text{ kN/m height, above10m height}
\]

Note: Cf values taken from Table 25 are for members of infinite length. Reduction factors for finite length of container, columns and other members can be taken from Table 25, which will further reduce wind forces.

**Table24-1: Calculations of variation in design wind speed & pressure with height**

<table>
<thead>
<tr>
<th>Height from</th>
<th>( k_2 ) *</th>
<th>( V_Z )</th>
<th>( p_Z )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground, m</td>
<td>0.91</td>
<td>45.773</td>
<td>1.257</td>
</tr>
<tr>
<td>Up to 10m</td>
<td>0.934</td>
<td>46.98</td>
<td>1.324</td>
</tr>
<tr>
<td>12m</td>
<td>0.958</td>
<td>48.187</td>
<td>1.393</td>
</tr>
<tr>
<td>14m</td>
<td>0.994</td>
<td>50.00</td>
<td>1.50</td>
</tr>
</tbody>
</table>

* : \( k_2 \) values are linearly interpolated.

**Table24-2: Summary of Forces and Total Loads on Tank**

<table>
<thead>
<tr>
<th>Element</th>
<th>Force per unit height</th>
<th>Height of element</th>
<th>Total horizontal force</th>
<th>CG of force from ground</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylindrical portion</td>
<td>10.61 kN</td>
<td>4.0m</td>
<td>42.44 kN</td>
<td>16.00m</td>
</tr>
<tr>
<td>Conical Dome</td>
<td>8.211 kN</td>
<td>2.0m</td>
<td>16.422 kN</td>
<td>13.067m</td>
</tr>
<tr>
<td>Shaft above 10m</td>
<td>5.97 kN</td>
<td>2.0m</td>
<td>11.94 kN</td>
<td>11.0m</td>
</tr>
<tr>
<td>Shaft up to 10m</td>
<td>5.67 kN</td>
<td>10.0m</td>
<td>56.7 kN</td>
<td>5.0m</td>
</tr>
</tbody>
</table>
Example 25 - Wind Pressure and Forces on a Multistory Commercial Complex by Force Coefficient Method

Problem Statement:
Calculate design wind forces using force coefficient method on a RCC Multistory commercial complex 12m×18m×51m tall situated in Mumbai. It is proposed to be constructed about 200m inside the sea front. Take average story height as 3.0m and frames spaced 6m c/c in both directions. The building is oriented with its smaller dimension facing the sea, i.e. in long-afterbody orientation.

Solution:
Wind Data:
1. Wind Zone: Zone III \( (V_b = 44 \text{m/s}) \) (IS:875-pt.3, Sec 5.2, Fig. 1)
2. Terrain category: (IS:875-pt.3, Sec 5.3.2.1)
   This building shares special location characteristics. On one face, i.e. sea face, it is exposed to terrain category 1 transiting into terrain category 3 from 200m distance. On the other hand, other faces are exposed to terrain category 4, being located in a commercially developed area with tall structures of height exceeding 25m.

   Therefore, we have to calculate a combined wind profile as per Appendix–B (IS:875-pt.3, Sec 5.3.2.4), transition from terrain category 1 to terrain category 3, for one wind direction and consider terrain category 4 for other three directions.

   Calculating combined wind profile for TC 1 to TC3

   This may be determined using IS:875-pt.3, Sec. 5.3.2.4(b). There are two options but option (ii) will give more rational values and therefore, should be used.

   Fetch Length \( x_3 = 200 \text{m}, \) developed height in TC 3, \( h_3 = 35 \text{m} \) (IS:875-pt.3, Table 3)

   Therefore, up-to 35m height, \( k_2 \) factor shall be as per TC 3 and above 35m it will be as per TC 1.

Design Factors:
Risk Coefficient Factor \( k_1 = 1.00 \) (IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor \( k_2 = \) Varies with height and terrain category, as given in Table 25.1.
Topography Factor \( k_3 = 1.00 \) (IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region \( k_4 = 1.00 \) (IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor \( K_d = 0.90 \) (IS:875-pt.3, Sec 6.2.1)
Area Averaging Factor $K_a$

- $1.00^*$, for glazing/cladding
- $0.8^{**}$, for 12m face
- $0.8^{**}$, for 18m face

(IS:875-pt.3, Sec 6.2.2, Table-4)

* tributary area for glazing/cladding shall be less than 10m$^2$, depends on the supporting system.

Design Wind Pressure:

Design Wind Speed $= V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$

$= 47 \times 1.0 \times k_2 \times 1.0 \times k_4 = (47 \times k_2) \text{ m/s}$

(IS:875-pt.3, Sec 5.3)

$p_Z = 0.6 \times (V_Z)^2$ & $p_d = p_Z \times K_d \times K_a$

(IS:875-pt.3, Sec 6.2 & Sec 6.2)

Table 25.1 : Calculations of Variation in Design Wind Speed with Height

<table>
<thead>
<tr>
<th>Height from ground, m</th>
<th>$k_z^*$</th>
<th>$V_Z$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>for sea face</td>
<td>for other faces</td>
</tr>
<tr>
<td>Up to 9m</td>
<td>0.91</td>
<td>0.80</td>
</tr>
<tr>
<td>12m</td>
<td>0.934</td>
<td>0.80</td>
</tr>
<tr>
<td>15m</td>
<td>0.97</td>
<td>0.80</td>
</tr>
<tr>
<td>18m</td>
<td>0.994</td>
<td>0.80</td>
</tr>
<tr>
<td>21m</td>
<td>1.015</td>
<td>0.817</td>
</tr>
<tr>
<td>24m</td>
<td>1.03</td>
<td>0.87</td>
</tr>
<tr>
<td>27m</td>
<td>1.045</td>
<td>0.92</td>
</tr>
<tr>
<td>30m</td>
<td>1.06</td>
<td>0.97</td>
</tr>
<tr>
<td>33m</td>
<td>1.07</td>
<td>0.99</td>
</tr>
<tr>
<td>36m</td>
<td>1.165$^+$</td>
<td>1.009</td>
</tr>
<tr>
<td>39m</td>
<td>1.1725</td>
<td>1.0285</td>
</tr>
<tr>
<td>42m</td>
<td>1.18</td>
<td>1.048</td>
</tr>
<tr>
<td>45m</td>
<td>1.1875</td>
<td>1.0675</td>
</tr>
<tr>
<td>48m</td>
<td>1.195</td>
<td>1.087</td>
</tr>
<tr>
<td>51m</td>
<td>1.2012</td>
<td>1.102</td>
</tr>
</tbody>
</table>

* : $k_z$ values are linearly interpolated.
+ : value for TC3 is used. Above this the effect of TC changes.
Table 25.2: Calculations of Variation in Design Pressure with Height

<table>
<thead>
<tr>
<th>Height from ground, m</th>
<th>$p_Z$ (kN/m$^2$)</th>
<th>Sea face</th>
<th>Other face</th>
<th>$p_d$, for building</th>
<th>Sea face</th>
<th>Other faces</th>
<th>All faces*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Up to 9m</td>
<td>1.097</td>
<td>0.848</td>
<td></td>
<td>0.79</td>
<td>0.610</td>
<td></td>
<td>0.987</td>
</tr>
<tr>
<td>12m</td>
<td>1.156</td>
<td>0.848</td>
<td></td>
<td>0.832</td>
<td>0.610</td>
<td></td>
<td>1.04</td>
</tr>
<tr>
<td>15m</td>
<td>1.247</td>
<td>0.848</td>
<td></td>
<td>0.90</td>
<td>0.610</td>
<td></td>
<td>1.122</td>
</tr>
<tr>
<td>18m</td>
<td>1.310</td>
<td>0.848</td>
<td></td>
<td>0.943</td>
<td>0.610</td>
<td></td>
<td>1.178</td>
</tr>
<tr>
<td>21m</td>
<td>1.365</td>
<td>0.885</td>
<td></td>
<td>0.983</td>
<td>0.637</td>
<td></td>
<td>1.228</td>
</tr>
<tr>
<td>24m</td>
<td>1.406</td>
<td>1.000</td>
<td></td>
<td>1.012</td>
<td>0.720</td>
<td></td>
<td>1.265</td>
</tr>
<tr>
<td>27m</td>
<td>1.447</td>
<td>1.122</td>
<td></td>
<td>1.042</td>
<td>0.808</td>
<td></td>
<td>1.302</td>
</tr>
<tr>
<td>30m</td>
<td>1.489</td>
<td>1.247</td>
<td></td>
<td>1.072</td>
<td>0.898</td>
<td></td>
<td>1.34</td>
</tr>
<tr>
<td>33m</td>
<td>1.517</td>
<td>1.300</td>
<td></td>
<td>1.092</td>
<td>0.936</td>
<td></td>
<td>1.365</td>
</tr>
<tr>
<td>36m</td>
<td>1.799</td>
<td>1.349</td>
<td></td>
<td>1.295</td>
<td>0.971</td>
<td></td>
<td>1.619</td>
</tr>
<tr>
<td>39m</td>
<td>1.822</td>
<td>1.402</td>
<td></td>
<td>1.312</td>
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<td>1.64</td>
</tr>
<tr>
<td>42m</td>
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<td></td>
<td>1.328</td>
<td>1.048</td>
<td></td>
<td>1.66</td>
</tr>
<tr>
<td>45m</td>
<td>1.87</td>
<td>1.510</td>
<td></td>
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<td>1.087</td>
<td></td>
<td>1.683</td>
</tr>
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<td>48m</td>
<td>1.893</td>
<td>1.566</td>
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</tr>
<tr>
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<td></td>
<td>1.377</td>
<td>1.159</td>
<td></td>
<td>1.721</td>
</tr>
</tbody>
</table>

Notes: 1. For building faces $K_a = 0.8$ is used.
2. For cladding, only higher wind speed is used for all four faces. However, the designer may choose to vary it from face to face.

**Wind Load Calculations:**

**Wind Induced Lateral Force on Structure:**
This will be calculated at every story level and separately for each wind direction, three cases in this problem.

$$F = C_f \times A_e \times p_d$$

(IS:875-pt.3, Sec.6.4)

**Force coefficient calculations:**

Long-afterbody orientation

$a/b = 18/12 = 1.5, h/b = 51/12 = 4.25$

$\Rightarrow C_f = 1.2$

(IS:875-pt.3, Fig. 4)

Short-afterbody orientation

$a/b = 12/18 = 0.667, h/b = 51/18 = 2.833$

$\Rightarrow C_f = 1.35$

(IS:875-pt.3, Fig. 4)

Effective area ($A_e$) calculations:

$6.0 \times 3.0 = 18\text{m}^2$, for intermediate frames
$3.0 \times 3.0 = 9\text{m}^2$, for end frames

For Cladding: depending on the spacing of supporting structure, but the effect of enhanced force at the corners and edges should be considered for fasteners by taking local coefficients from IS:875-pt.3, Table 5.

**Area Averaging Factor**:

(IS:875-pt.3, Sec 6.2.2, Table-4)

Tributary area for calculating wind forces on building frames $= 51 \times 6 = 306\text{m}^2$ in either direction, being the product of height of building & frame spacing in either direction.
As brought out in the commentary also, the area averaging factor has been introduced in this proposed draft, in order to account for loss of correlation between peaks of wind generated force over an area. Since all peaks do not occur simultaneously, the net effect of wind force exerted on the exposed surface is less than the case when whole face is considered to be acted upon by design wind force at a time. Net wind force goes on reducing with increase in the net effective area for the element being analysed. Following example will make things more clear.

Let us consider the face of a framed tall building,

For the calculation of wind forces along the height, the area averaging factor for nodes on frame-1 shall depend upon an area

$$= \text{height of the building} \times \left[ \frac{c/c \text{ dist between frame (1) & (2)}}{2} \right]$$

for nodes on frame-2, it shall depend upon an area

$$= \text{height of the building} \times \left[ \frac{c/c \text{ dist between frame (1) & (2)} + \frac{c/c \text{ dist between frame (2) & (3)}}{2}}{2} \right]$$

and so on

For Calculating the nodal wind force at Beam - Column junction, tributary area should not be considered to determine the Area Averaging Factor because for wind resistance the beams perpendicular to wind direction do not participate & the whole vertical frame (e.g. frame 1, 2, 3 etc.) only resists the along-wind lateral force. Respective nodal forces shall be obtained by using the AAF for the frame by the expression:

(Force coefficients × tributary area × design wind pressure, obtained using AAF of frame).
Example 26 - Wind Pressure and Forces on a Multistory Commercial Complex by Gust Factor Approach

Problem Statement:
Calculate design wind forces using the gust factor approach on a RCC Multistory building 12m × 24m × 96m tall, as in figure 26.1, situated in Mumbai. It is proposed to be constructed about 200m inside the sea front. Take average story height as 3.0m and frames spaced 6m c/c in both directions. The building is oriented with its smaller dimension facing the sea, i.e. in long-afterbody orientation.

Solution:
Wind Data:
Since the ratio of height to least lateral dimension is more than 5, (96/12 = 8) dynamic analysis is needed.
(IS:875-pt.3, Sec 8.1)

1. Wind Zone: Zone III (V_b = 44m/s)
   (IS:875-pt.3, Fig. 1, Sec 5.2)
2. Terrain category:
   (S:875-pt.3, Sec 5.3.2.1)
This building shares special location characteristics. On one face, i.e. sea face it is exposed to terrain category 1 transiting into terrain category 3 from 200m distance. On the other hand, other faces are exposed to terrain category 4, being located in a commercially developed area.
Therefore, we have to calculate a combined wind profile as per Appendix–B (IS:875-pt.3, Sec 5.3.2.4), transition from terrain category 1 to terrain category 3, for one wind direction and consider terrain category 4 for other three directions.
Calculating combined wind profile for TC 1 to TC3
This may be determined using IS:875-pt.3, sec. 5.3.2.4(b). There are two options but option (ii) will give more rational values and therefore, should be used.

Fetch Length \(x_3 = 200\text{m}\), developed height in TC3, \(h_3 = 35\text{m}\) (IS:875-pt.3, Table 3)

Therefore, up to 35m heights, \(k_2\) factor shall be as per TC 3 and above 35m it will be as per TC 1.

**Design Factors:**

- **Risk Coefficient factor** ‘\(k_1\)’ = 1.00  
  (IS:875-pt.3, Sec 5.3.1, Table-1)
- **Terrain & Height factor** ‘\(k_2\)’ = Varies with height and terrain category, as in Table 25.1  
  (IS:875-pt.3, Sec 5.3.2, Table-2)
- **Topography factor** ‘\(k_3\)’ = 1.00  
  (IS:875-pt.3, Sec 5.3.3.1)
- **Cyclonic Region factor** ‘\(k_4\)’ = 1.00  
  (IS:875-pr.3, Sec 5.3.4)

**Wind Directionality factor** ‘\(K_d\)’ = 0.90  
(IS:875-pt.3, Sec 6.2.1)

**Area Averaging factor** ‘\(K_a\)’ = 1.00* for glazing/cladding.

*: *tributary area for glazing/cladding shall be less than 10\(m^2\), depends on the supporting system.*

**Design Wind Pressure**

Design Wind Speed = \(V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4\)  
\(= 47 \times 1.0 \times k_2 \times 1.0 \times 1.0 \times (47 \times k_2) \text{ m/s}\)  
(IS:875-pt.3, Sec 5.3)

\[p_Z = 0.6 (V_Z)^2 \quad \text{&} \quad p_d = p_Z \times K_d \times K_a\]  
(IS:875-pt.3, Sec 6.2, 6.2)

**Table 26.1 : Variation in design wind speed & design pressure with height**

<table>
<thead>
<tr>
<th>Height from Ground, m</th>
<th>(k_2)</th>
<th>(V_Z) (m/s)</th>
<th>(p_Z) (kN/m(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sea Face</td>
<td>Other faces</td>
<td>Sea Face</td>
</tr>
<tr>
<td>Up to 9m</td>
<td>0.91</td>
<td>0.80</td>
<td>42.77</td>
</tr>
<tr>
<td>12m</td>
<td>0.934</td>
<td>0.80</td>
<td>43.90</td>
</tr>
<tr>
<td>18m</td>
<td>0.994</td>
<td>0.80</td>
<td>46.72</td>
</tr>
<tr>
<td>24m</td>
<td>1.03</td>
<td>0.868</td>
<td>48.41</td>
</tr>
<tr>
<td>30m</td>
<td>1.06</td>
<td>0.97</td>
<td>49.82</td>
</tr>
<tr>
<td>36m</td>
<td>1.165+</td>
<td>1.009</td>
<td>54.755</td>
</tr>
<tr>
<td>42m</td>
<td>1.18</td>
<td>1.048</td>
<td>55.46</td>
</tr>
<tr>
<td>48m</td>
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<td>1.087</td>
<td>56.165</td>
</tr>
<tr>
<td>54m</td>
<td>1.205</td>
<td>1.108</td>
<td>56.635</td>
</tr>
<tr>
<td>60m</td>
<td>1.212</td>
<td>1.120</td>
<td>56.964</td>
</tr>
<tr>
<td>66m</td>
<td>1.220</td>
<td>1.132</td>
<td>57.340</td>
</tr>
<tr>
<td>72m</td>
<td>1.226</td>
<td>1.144</td>
<td>57.622</td>
</tr>
<tr>
<td>78m</td>
<td>1.234</td>
<td>1.156</td>
<td>57.980</td>
</tr>
<tr>
<td>84m</td>
<td>1.241</td>
<td>1.168</td>
<td>58.327</td>
</tr>
<tr>
<td>90m</td>
<td>1.248</td>
<td>1.180</td>
<td>58.656</td>
</tr>
<tr>
<td>96m</td>
<td>1.255</td>
<td>1.192</td>
<td>58.985</td>
</tr>
</tbody>
</table>

*: \(k_2\) values are linearly interpolated.

+: Effect of terrain category changes from TC-3 to TC-1.
Wind Induced Lateral Forces on Structure:

This will be calculated at every story level and separately for each wind direction, for the three cases in this problem.

\[ F = C_f \times A_e \times p_d \times C_{dyn} \]  
(IS:875-pt.3, Sec 6.3.9.1 also)

**Force coefficient calculations:**

Long-afterbody orientation  
\[ a/b = 24/12 = 2.0, \ h/b = 96/12 = 8.0 \Rightarrow C_f = 1.25 \]  
(IS:875-pt.3, Fig. 4)

Short-afterbody orientation  
\[ a/b = 12/24 = 0.5, \ h/b = 96/24 = 4.0 \Rightarrow C_f = 1.40 \]  
(IS:875-pt.3, Fig. 4)

**Effective area calculations:**

\[ 6.0 \times 3.0 = 18m^2, \text{ for intermediate frames} \]  
\[ 3.0 \times 3.0 = 9m^2, \text{ for end frames} \]

For Cladding: depending on the spacing of supporting structure, but the effect of enhanced force at the corners and edges should be considered by taking local coefficients from IS:875-pt.3, Table 5.

**Dynamic Response Factor Calculations:**

(Along-wind)

A. **Wind onto wider face**

\[ h = 96m, \ b_{oh} = 24m, \ d = 12m \]  
\[ L_b = 70 (96/10)^{0.25} = 123.22m \]  
\[ f_o = 1/T = \sqrt{d/(0.09 \times 96)} = 0.40 \]  
Hz, \( \beta = 0.016 \) (IS:875-pt.3, Table 36)

(i) For base floor (s = 0)

\[ B_s = 1/[1+(.26 \times 96^2+.46 \times 24^2)^{0.5}/(123.22)] = 0.71 \]  
\[ H_s = 1+(0/96)^2 = 1.0, \ V_h = 48.0m/s \text{ at } h = 96m \]  
\[ g_r = \sqrt{[2 \log_e(3600 \times 0.40)]= 3.814} \]  
\[ I_h = .1018 \text{ for Terrain Category 4} \]  
(IS:875-pt.3, Sec. 5.5)

\[ g_r = 4 \]  
(IS:875-pt.3, Sec.9.2)

\[ S = 1/[1+(3.5x.4x96)/34.5][1+(4x.4x24)/34.5] = 0.143 \]  
\[ N = n_sL_s/V_h = .4x123.22/34.5 = 1.43 \]  
\[ E = \pi N / (1+70.8N^2)^{5/6} = 0.0706 \]

\[ \bar{\Omega} = g_vI_h \sqrt{B_s/2} = 4x.1018x\sqrt{0.71/2} = 0.172 \]  
\[ G = 1+r\sqrt{[g_vB_s(1+ \bar{\Omega})^2+(H_sX_g2xSxEx\beta)]} = 1+2x.1018x\sqrt{[4^2x.71x(1+0.172)^2]+(1x.3.814^2x.143x.0706)/.016} \]

(ii) For floor level at mid-height (s=48m)

\[ H_s = 1+(s/h)^2 = 1+(48/96)^2 = 1.25 \]  
\[ B_s = 1/[1+0.26x48^2+0.46x24^2]^{0.5}/(123.22)] = .807 \]

\[ \bar{\Omega} = g_vI_h \sqrt{B_s/2} = 0.183 \]  
\[ G = 1+2x.1018x\sqrt{[4^2x.807x(1+0.183)^2]+(1.25x3.814^2x.143x.0706)/.016} \]

(iii) Top floor level (s = 96 m)

\[ H_s = 2.0, \ B_s = .883, \]  
\[ \bar{\Omega} = g_vI_h \sqrt{B_s/2} = 4x.1018x\sqrt{0.883/2} = 0.191 \]  
\[ G = 1+r\sqrt{[g_vB_s(1+ \bar{\Omega})^2+(H_sX_g2xSxEx\beta)]} = 1+2x.1018x\sqrt{[4^2x.883x(1+0.191)^2+(2x3.814^2x.143x.0706)/.016}] \]

B. **Wind from Sea (on smaller face)**

\[ h = 96m, \ d = 24m, \ b = 12m, \ L_h =149.62m \]  
\[ n_s = I/T = \sqrt{24/(0.09 \times 96)} = 0.567 \text{ Hz}, \]  
\[ \beta = 0.016 \]  
(IS:875-pt.3, Table 36)

(i) For base floor (s = 0)

\[ B_h = 1/[1+(.26 \times 96^2+.46 \times 12^2)^{0.5}/(149.62)] = 1.33 \]
\[ H_s = 1 + (0/96)^2 = 1.0, \quad V_h = 60.5 \text{ m/s} \]
\[ g_r = \sqrt{2 \log_e (3600 \times 0.567)} = 3.904 \]
\[ I_h = 0.110 \text{ for Terrain Category 1} \]
\[ (\text{IS:875-pt.3, Sec. 5.5}) \]
\[ g_v = 3 \]
\[ S = 1/[1 + (3.5 \times 0.567 \times 96)/34.5][1 + (4 \times 0.567 \times 12)/34.5] = 0.086 \]
\[ N = n_a \frac{L_h}{V_h} = 0.567 \times 149.62/34.5 = 2.46 \]
\[ E = \frac{\pi N}{(1 + 70.8 N^2)^{5/6}} = 0.05 \]
\[ \Phi = g_v I_h \sqrt{B_s/2} = 3x.1018 \times \sqrt{0.85/2} = 0.173 \]
\[ G = 1 + r \sqrt{[g_v B_s (1 + \Phi)^2 + (H_s g_r^2 x S x E)]/\beta} = 1 + 2x.1018x[3^2x1.33x(1+0.173)^2 + (1x3.9^2x0.086x0.05)/0.016] = 1.91 \]
\[ (\text{ii}) \text{ Floor level at mid-height (s=48m)} \]
\[ H_s = 1.25, \quad B_s = 0.85, \]
\[ \Phi = 3x.1018 \times \sqrt{0.85/2} = 0.138 \]
\[ G = 1 + r \sqrt{[g_v B_s (1 + \Phi)^2 + (H_s g_r^2 x S x E)]/\beta} = 1 + 2x.1018x[3^2x0.85x(1+0.138)^2 + (1.25x3.814^2x0.086x0.05)/0.016] = 1.78 \]
\[ (\text{iii}) \text{ Top floor level (s=96 m)} \]
\[ H_s = 2.0, \quad B_s = 0.95, \]
\[ \Phi = 3x.1018 \times \sqrt{0.95/2} = 0.146 \]
\[ G = 1 + r \sqrt{[g_v B_s (1 + \Phi)^2 + (H_s g_r^2 x S x E)]/\beta} = 1 + 2x.1018x[3^2x0.95x(1+0.146)^2 + (2x3.814^2x0.086x0.05)/0.016] = 1.88 \]

**A. Wind onto wider face (short afterbody orientation)**

For \( z = h \)

\[ V_n = \frac{V_z}{f_0 b (1 + g_v I_h)} = \frac{48.0}{0.4 \times 24(1 + 3.5 \times 0.235)} = 2.75 \]
\[ I_h = 0.261 \text{ (at } 2h/3 \text{ height)} \]

Using dashed line of IS:875-pt.3, Fig-12

\[ \log_{10} C_{fs} = -3.0 ; \quad C_{fs} = 0.0010 \]

\[ C_{dyn} = 1.5 \times 3.814 \left( \frac{24}{12} \right) \left( \frac{1.00}{(1 + 3.5 \times 0.261)^2} \right) \left( 1 + 0 \right) \sqrt{\pi \times 0.0010 \times 0.02} = 1.24 \]

However, at the base the 0.261 mode shape deflection is zero and varies linearly with height in this case as \( k = 1 \).
<table>
<thead>
<tr>
<th>Height from ground, m</th>
<th>( p_z ) (kN/m(^2))</th>
<th>( p_d ) for building</th>
<th>( p_d ) for cladding</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sea face</td>
<td>Other face</td>
<td>Sea face</td>
</tr>
<tr>
<td>Up to 9m</td>
<td>1.097</td>
<td>0.848</td>
<td>0.987</td>
</tr>
<tr>
<td>12m</td>
<td>1.156</td>
<td>0.848</td>
<td>1.040</td>
</tr>
<tr>
<td>18m</td>
<td>1.310</td>
<td>0.848</td>
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</tr>
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<td>1.704</td>
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<td>1.808</td>
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<tr>
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</tr>
<tr>
<td>96m</td>
<td>2.087</td>
<td>1.883</td>
<td>1.878</td>
</tr>
</tbody>
</table>

Notes: For cladding, only higher wind speed is used for all four faces. However, the designer may choose to vary it face to face.