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An Explanatory Handbook on Proposed IS 875 (Part3) Wind Loads on Buildings and Structures

by

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- The views and opinions expressed are those of the authors and not necessarily of the GSDMA, the World Bank, IIT Kanpur, or the Bureau of Indian Standards.
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FOREWORD

This explanatory handbook is meant to provide a supplement to the I.S. 875 (Part 3) – Wind Loads on Buildings and Structure- draft revision.

This work has been supported through a project entitled Review of Building Codes and Preparation of Commentary and Handbooks awarded to IIT Kanpur by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank finances. The views and opinions expressed therein are those of the authors and not necessarily of the GSDMA, the World Bank, IIT Kanpur, or the Bureau of Indian Standards.

Prof. Ashwini Kumar (IIT Kanpur), Dr. N. Lakshmanan (Structural Engineering Research Centre, Chennai), and Prof. P.N. Godbole (VNIT Nagpur) were the reviewers of the document. Prof. L.M. Gupta (VNIT Nagpur) also contributed through review comments.

Section 1 is an introductory note to reflect the background state-of-the-art scenario of wind engineering. This indeed forms the backdrop of the entire exercise of revision in hand.

Section 2 of this volume contains illustrative examples designed to demonstrate the various parts of the “Indian Standard I.S. 875 (Part 3)-Draft Revision” dealing with wind loads on buildings and structures. The examples take the reader to a point whereby the wind load on a particular structure is computed using the code.

Section 3 describes some examples which are unusual from the point of view of determination of wind loads, and for which straightforward answers can not be had from the code. These are dealt with through qualitative discussions. The suggested are only indicative.

CONTENTS

Section - 1

AN INTRODUCTORY NOTE

1.1 General

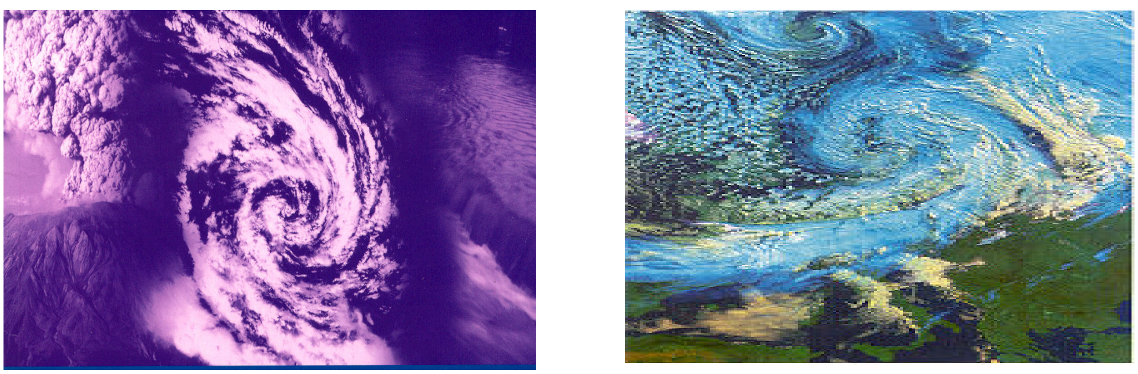
Wind has two aspects. The first – a beneficial one – is that its energy can be utilized to generate power, sail boats and cool down the temperature on a hot day. The other – a parasitic one – is that it loads any and every object that comes in its way. The latter is the aspect an engineer is concerned with, since the load caused has to be sustained by a structure with the specified safety. All civil and industrial structures above ground have thus to be designed to resist wind loads. This introductory note is concerning the aspect of wind engineering dealing with civil engineering structures.

Wind flow generation is on account of atmospheric pressure differentials and manifests itself into various forms, such as,

- Gales and monosonic winds
- Cyclones/Hurricanes/Typhoons
- Tornados
- Thunderstorms
- Localised storms

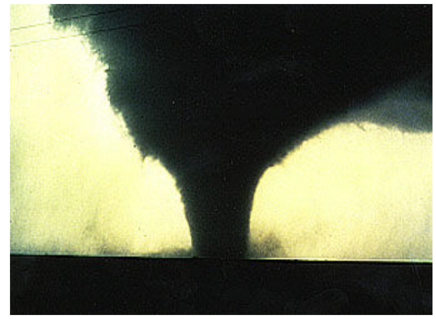
Photographs in figure–1 depict some of these storms.

Friction from the earth's surface leads to 'boundary layer' flow, but characteristics of flow vary depending upon the storm type.



(a) Cyclonic Storms

(b) Thunderstorm



(c) Tornados

Figure 1 : various types of wind storms

Apart from monsoonic winds and gales, of greater interest in India are the cyclonic storms that frequently strike the coasts and the tornados which appear 'freakishly'. Figures 2 and 3 give some data on cyclones.

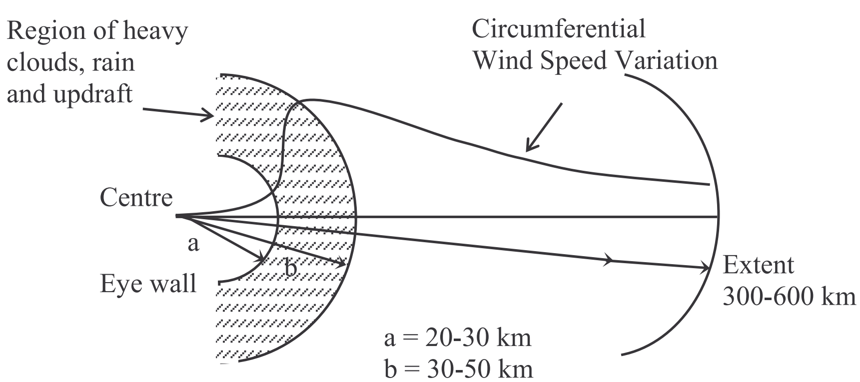
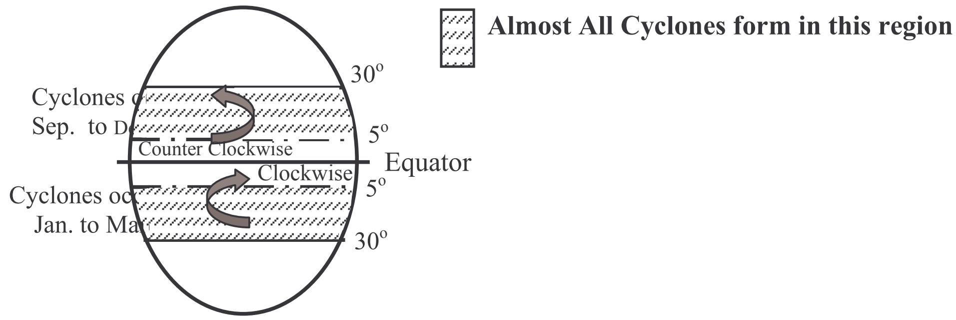


Figure 2: General Structure of a Cyclone



- Total no. of Cyclones/Year (Global) $\approx$ 80
- North-Eastern Hemisphere the worst hit, experiences almost 45% of the Cyclones
- India experiences about 6 cyclones/year, mostly on the East coast

Figure 3 : General Distribution of Cyclones over the Globe

There has been an increasing trend of natural hazard events as well as wind – induced disasters in the recent decades, as also the losses caused in such disasters. The reasons for this increasing trend can be enumerated as below :

- Population density is increasing worldwide, particularly in large cities and agglomerations. In the last twenty years or so the number of cities with more than 1 million population may have doubled from about 200. Likewise mega cities with more than 10 million population may have gone up from about 10 to 25.
- At the same time the standard of living is increasing almost everywhere – a bit faster here, a bit slower there. Together with the growing population density this means an exponential increase in the concentration of values; infrastructure cost sustaining the modern living standard.
- Regions formerly avoided on account of their risk potential are now populated. This applies above all to coastal regions particularly exposed to storms and storm surges, in many cases also to major earthquakes and seismic waves (tsunamis).
- Industry is also moving into extremely dangerous regions, concentrating huge economic values in highly exposed locations.
- In many cases new building materials such as glass and plastics used for facades and roofs are far more susceptible to damage than conventional methods and materials.
- In certain periods, nature also becomes responsible for bigger disasters, with more dramatic trends in unleashing natural hazards.

Tables 1 to 3 contain some relevant information on these events and the losses therefrom.

Table 1* : Natural Disaster Events and Economic Losses Decadewise.

Decade	**1960s**	**1970s**	**1980s**	**1990s**
Numbers	27	47	63	84
Economic losses (US $ billion)	73.1	131.5	204.2	591

Table 2* : Data for All Catastrophes 1985 - 2000

Item	**World**	**Asia-Pacific**	**%**
Loss Events (Nos.)	8,350	3,220	38.6
Economic Losses US$ million	8,95,800	4,26,270	47.6
Insured Losses US$ million	1,69,940	21,970	12.9
Loss of Life (Nos.)	5,36,250	4,33,480	80.8

Table 3* : Data for All Catastrophes Vs Wind for Asia-Pacific 1985 - 2000

Item	**All Catastrophes**	**Wind**	**%**
Loss Events nos.	3,220	1,020	31.7
Economic Losses US$ million	4,26,270	62,120	14.6
Insured Losses US$ million	21,970	12,470	56.8
Loss of Life nos.	4,33,480	60,250	13.9

There are various scales on which windstorms can be measured – these are shown in Table – 4.

* "Topics", Munich Reinsurance Company

Table 4 : Various Scales for Measuring Wind Storms

Windstorm : Scales and Effects										
Beaufort Scale						**Saffir–Simpson Hurricane Scale**				
Bft	Descriptive term	Mean wind speed at 10 m above surface			Wind pressure	SS	Descriptive term	Mean wind speed		
		m/s	Km/h	knots	Kg/m^2			m/s	Km/h	knots
0	Calm	0–0.2	0–1	0–1	0	1	Weak	32.7–42.6	118–153	64–82
1	Light air	0.3–1.5	1–5	1–3	0–0.1	2	Moderate	42.7–49.5	154–177	83–96
2	Light breeze	1.6–3.3	6–11	4–6	2.0–0.6	3	Strong	49.6–58.5	178–209	97–113
3	Gentle breeze	3.4–5.4	12–19	7–10	0.7–1.8	4	Very strong	58.6–69.4	210–249	114–134
4	Moderate breeze	5.5–7.9	20–28	11–15	1.9–3.9	5	Devastating	≥ 69.5	≥ 250	≥ 135
5	Fresh breeze	8.0–10.7	29–38	16–21	4.0–7.2	**Fujita Tornado Scale**				
6	Strong breeze	10.8–13.8	39–49	22–27	7.3–11.9	F	Descriptive term	m/s	Km/h	Knots
7	Near gale	13.9–17.1	50–61	28–33	12.0–18.3	0	Weak	17.2–32.6	62–117	34–63
8	Gale	17.2–20.7	62–74	34–40	18.4–26.8	1	Moderate	32.7–50.1	118–180	64–97
9	Strong gale	20.8–24.4	75–88	41–47	26.9–37.3	2	Strong	50.2–70.2	181–253	98–136
10	Storm	24.5–28.4	89–102	48–55	37.4–50.5	3	Devastating	70.3–92.1	254–332	137–179
11	Violent storm	28.5–32.6	103–117	56–63	50.6–66.5	4	Annihilating	92.2–116.2	333–418	180–226
12	Hurricane	> 32.7	> 118	> 64	> 66.6	5	Disaster	116.3–136.9	419–493	227–266

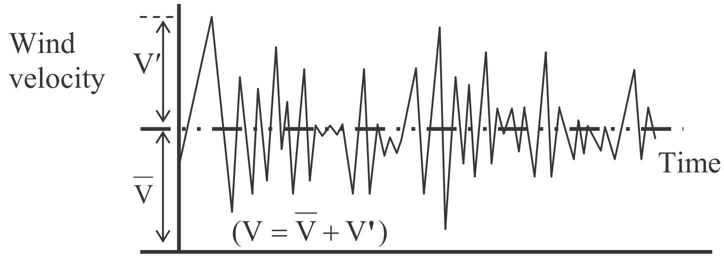
Wind storm disasters cause widespread damage due to high wind speeds, and flooding in case of cyclones. Figure 4 shows typical photographs.

1. Water submergence; 2,3,4,5. Wall damage and roof blown away; 6. Tower collapse, 7. Failure of a tower; 8. Uprooted tree damages nearby building

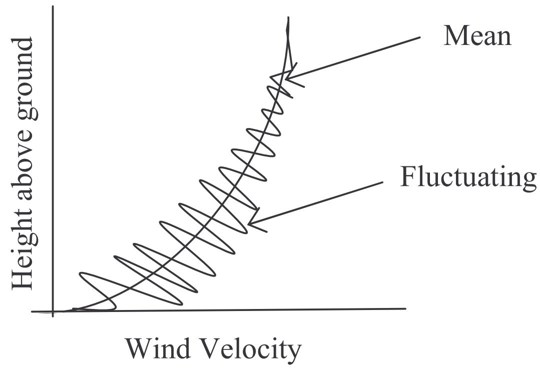
Figure 4: Typical Wind – Storm Damage

1.2 Structure of Wind

Wind is a randomly varying dynamic phenomenon and a trace of velocity versus time for wind will be typically as shown in figure 5. The wind velocity V can be seen as a mean plus a fluctuating component responsible for creating 'gustiness'. Within the earth's boundary layer, both components not only vary with height, but also depend upon the approach terrain and topography, as seen from figure 6. While dealing with rigid structures, the consideration of the 'equivalent static' wind is adequate. However, in dealing with wind-sensitive flexible structures, the consideration of the wind-energy spectrum, integral length scale, averaging time and the frequencies of the structure become important. The determination of wind velocity for a certain geographical location is essentially a matter of statistical reduction of a given measured data. On this depend the various wind zones. Another important decision involved is the averaging time. In as far as averaging time is concerned, it may be anywhere from 2-3 seconds to 10 minutes to an hour. The influence of averaging time on velocity is seen in figure 7.

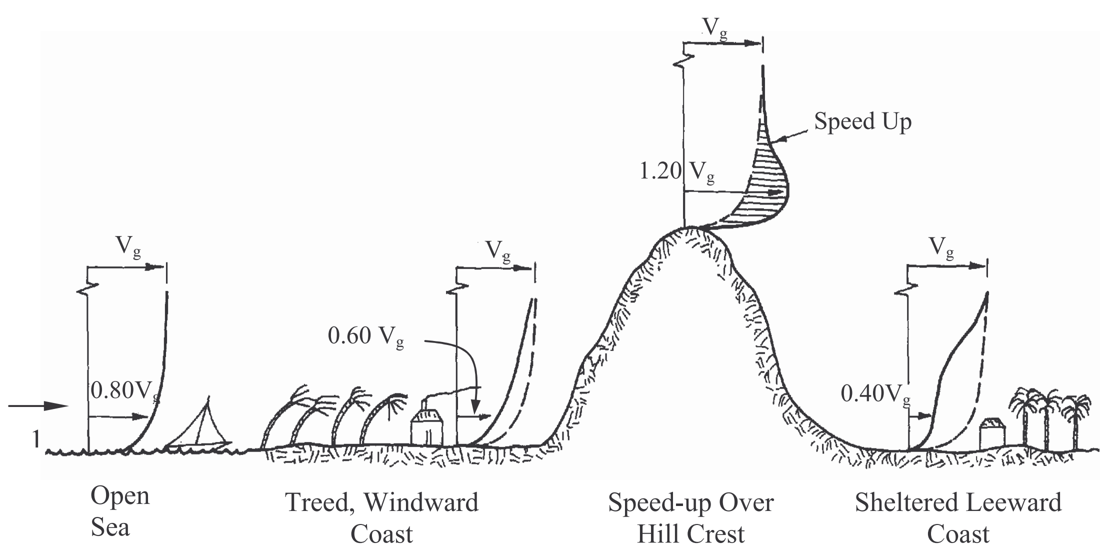


(a)



(b)

Figure 5 : Variation of Wind Velocity with (a) Time (b) Height



V_g = Gradient Wind Velocity

Figure 6 : Influence of Terrain and Topography (Typical)

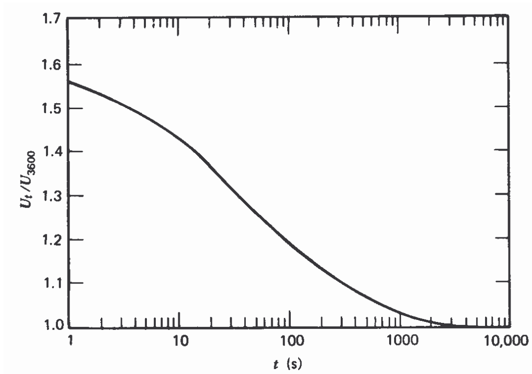


Figure 7: Ratio of probable maximum speed averaged over period 't' to that averaged over one hour.

1.3 Wind Forces on Structures

Most structures present 'bluff forms' to the wind, making it difficult to ascertain the wind forces accurately. Thus the problem of bluff-body aerodynamics remains largely in the empirical, descriptive realm of knowledge. The flow pattern and hence the wind pressures/forces change with the Reynolds number (Re) making the direct application of wind tunnel test results to real structures difficult. Computational methods (CFD/CFE) also pose problems for the high Reynolds Number, Re encountered in practice, leading to their inadequacy.

The oncoming turbulence causes fluctuations in the flow. As a result the wind pressures also change with time, thus affecting the values of design wind pressures. A typical case is that of a cylinder for which pressure distribution changes with Re and surface roughness. The value of the drag coefficient, C_d, for different situations is as given in figure 8. The surface has been roughened for case (c) by using sand, with the grain size K.

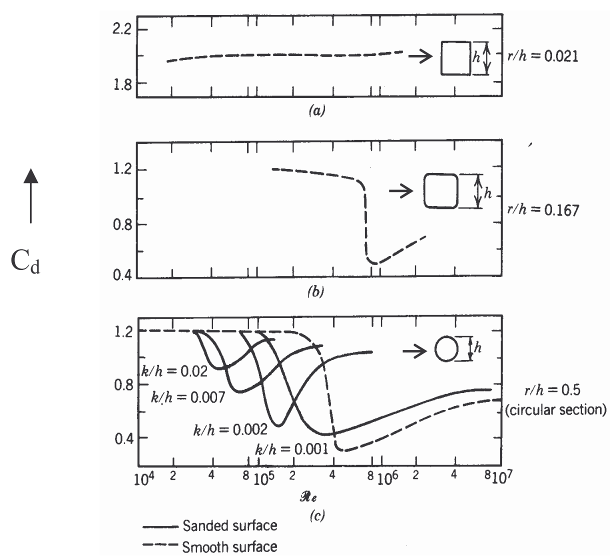


Figure 8: Influence of Reynolds number, corner radius, and surface roughness on drag coefficient, square to circular cylinders (r is the corner radius; k is the grain size of sand)

While the separation of flow in non-bluff sharp edged bodies is well defined, the dimension of the body/building parallel to flow affects the 'reattachment of the separated flow' and hence the suction pressures on leeward side. Turbulence may cause an increase in the suction pressures for thin bodies and a decrease in the thick/long bodies.

The buildings and other civil engineering structures are three-dimensional bodies with a large variety of shapes and have complex flow patterns and therefore varied pressure distributions. As a result, most 3-D studies rely partially or wholly upon experiments. The complexity of wind flow is not only introduced by the geometries of typical structures, but also by the characteristics of the terrain and other structures in the close vicinity. This has necessitated the determination of wind pressures experimentally in wind tunnels using scaled models and simulated winds. Besides, the buildings are never sealed and wind pressures develop inside even in a closed building, with maximum values occurring in open buildings. These generally add to the pressures outside, creating worst possible effect on roofs, as well as walls.

In recent model studies, forces acting over an element of the building have been measured by devices that automatically add pressures occurring simultaneously at several points of the element, weighted by the respective tributary areas (University of Western Ontario). These measurements as well as results of full-scale tests have been used to develop new design load provisions for low-rise buildings that have been incorporated in design standards. It is established that local pressures can have strongly non-Gaussian distributions, especially at corners and edges. Tests on low-rise building models have confirmed that the fluctuating part of the load can in many instances be significantly larger than the mean load and that, for any given storm, peak pressures and the ratio between mean pressures and fluctuating pressures decreases as the terrain roughness increases.

1.3.1 Wind Sensitive Structures

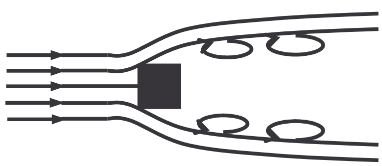
Tall and slender structures are flexible and exhibit a dynamic response to wind. Tall structures vibrate in wind due to the turbulence inherent in the wind as well as that generated by the structure itself due to separation of the flow. Thus there is a mean and a fluctuating response to the wind. Besides, the dynamic forces act not only in the direction of wind flow but also in a direction nearly perpendicular to the flow (lift forces), so that tall structures also exhibit an across-wind response.

Along-wind response has a mean component (time-invariant load obtained from the mean wind speed) and a fluctuating component. The latter is further expressed as a sum of background and resonant components. If the damping is small, which is usually the case, the bulk of the contribution to the dynamic response is due to the resonant portion. Spectral response curves have been developed and are used for predicting the along-wind fluctuating response of a building. The approach is frequency based and uses the theory of random vibrations as well as statistical considerations. In the case of line-like structures, like chimneys, the problem becomes one-dimensional. This simplification is used for analyzing tall buildings as well, which is evidently conservative.

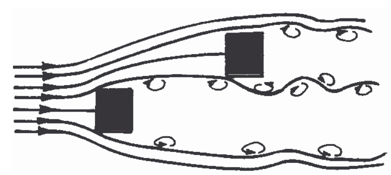
As mentioned already, tall flexible structures, exhibit an across-wind response as well. This is on account of flow separation from the cross section of the structure, which results in vortices being shed at a given frequency. The pattern of this across-wind phenomenon is comparatively more regular for circular sections, such as those for chimneys and towers, which can undergo resonant vibrations when the structural frequency matches the forcing frequency. The response is affected significantly by the turbulence content of the wind, and is larger in smoother flows. When buildings attain slender proportions – which may happen for very tall ones – the across-wind behaviour becomes important.

The theoretical treatment of tall slender structures in the along-wind direction is better developed than for the across-wind direction, and for this reason it may be advisable to undertake model studies in a wind tunnel for such structures.

A body or a structure, such as a building, a tower or a chimney, when placed in a flow of air will experience pressures and forces. When one or more similar or dissimilar bodies are placed downstream or upstream of a structure, the 'stand-alone' values of pressures and forces get altered. This is termed as the Interference Effect. Interference will occur irrespective of whether the bodies involved are rigid or flexible. In the former it is the 'wake' of one body that affects the other, while in the latter the deflections of the body may also affect the wake itself (Figure 9). The phenomenon of interference is experienced extensively in practice but is very difficult to quantify in general because of the variability of situations involved. Systematic wind tunnel studies can nevertheless give some guidance.



Flow around isolated building



Flow around two buildings

Figure 9: Typical wind flow around isolated and two nearby buildings

1.4 Determination of Wind Loads

It is common practice to approach engineering problems for their analysis by employing theoretical or experimental means. Most problems are tackled satisfactorily by using theoretical solutions. Experimentation is carried out only where necessary, often for physical verification of the theory employed. Likewise, for tackling wind engineering problems there are these two approaches. However, in wind engineering, though theoretical solutions have been developed, there is a predominant use of experimental methods. On the theoretical side, on the one hand there is the combination of statistical expression of loads and structural dynamics which has been in common use for the last 40 years, and, computational fluid dynamics which has evolved more recently. A rather useful tool that is developing as part of computational Wind Engineering (CWE) is the use of Artificial Neural Networks (ANN).

The first issue to be looked at is the loading itself. The last 50 years have witnessed remarkable changes in the manner of assessing wind loads in structural design (Davenport, 2001).

During this time the description of wind load has moved from relatively simple, straightforward notions of static drag forces to much more sophisticated models, involving all the manifold questions of climate, meteorology, aerodynamics, structural mechanics and dynamics, and, more recently, reliability. Furthermore, earlier structures were, by present standards, relatively massive. Structural members themselves were heavier due to the relatively weak materials, and dead loads were higher due, for example, to the heavy masonry and stone facades on buildings, and the use of heavy, reinforced concrete deck systems in bridges. These massive structures were frequently much stiffer than they were predicted to be due to the participation of the non-structural components, the contribution of which was difficult to estimate. The massiveness of these structures did little to emphasize the importance of wind forces. Wind loads were considered in a very simple way and only the 'equivalent static' approach was followed. The skyscaper boom did much to change this picture. In this context, changes that have taken place vis-à-vis structural materials and design, whereby masses have dropped to almost half, strengths have more than doubled and damping in structures has decreased.

Pioneering effort to give expression to the complex randomly varying phenomenon of wind loading and its use particularly in slender tall structures came in the 1960s from A.G. Davenport (1961, 1963a, 1963b). There are many other notable contributions - to cite some of them, R.I. Harris (1963), N. Isyumov (1982), B.J. Vickery (1978), and more recently by A. Kareem (1999, 2001) and G. Solari (2001). Methods so developed permit a satisfactory analysis of the along-wind response of tall structures such as towers, chimneys and tall buildings. In as far as the across-wind response is concerned, success is limited, chimneys being more amenable than other tall structures. Similarly, there have been developments in the aerodynamic analysis of long slender bridges, particularly of the cable supported type – pioneering effort coming from R.H. Scanlan, A.G. Davenport, T. Miyata, H.F. Xiang, G. Diana, M. Matsumoto and A. Larsen to name some prominent investigators.

As opposed to the methods of theoretical analysis based on structural dynamics, there is the development of Computational Fluid Dynamics (CFD). Efforts have been made with a significant measure of success to make CFD as part of Computational Wind Engineering (CWE). One of the major issues herein is to be able to model turbulence. The dawn of CFD or CWE may well be considered as the early 1980s. This field of research has since blossomed to an extent that 4-yearly International Conferences on the subject have been held following the first one in Tokyo in 1992. Some of those prominent in their efforts in CWE in recent years are S. Murakami, T. Tamura, W. Rodi, T. Stathopoulos, R. Meroney and Paneer Selvam. It has been possible (Murakami 1999) to obtain wind forces (Stathopolous 1997); determine wind-structure interactions (Murakami et al.1997, Larsen 1998 and Tamura et al. 1995); wind environment around buildings and structures (Song and He 1998); contaminant dispersion (Cowan et al. 1997, Tominaga et al. 1997 and Leitl et al. 1997); wind-driven rain and snowdrift (Choi 1997, Sankaran and Paterson 1997 and Tominaga and Mochida 1998); evaluation of spatial distribution of wind energy (Finarrdi et al. 1998); flow over complex terrain (Montavon 1998), and regional climate (Mochida et al. 1997, Uchida and Ohya 1998). Accuracy achieved as yet is still limited and there is a long-way to go in solving different types of practical problems with reasonable degree of accuracy and effort.

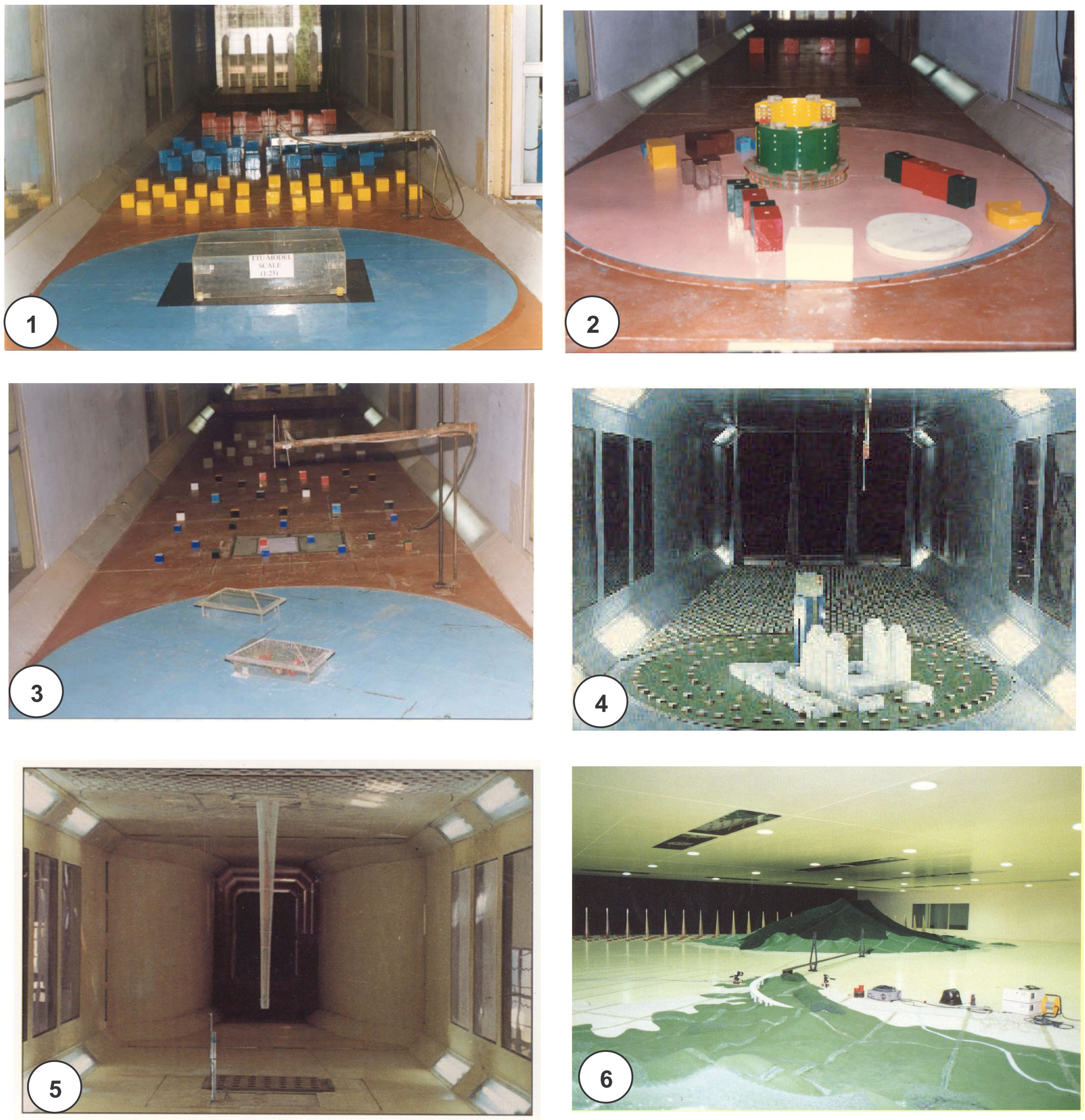
Further, it has to be realised that, a "theory without full-scale observations to back it up is a dangerous commodity in engineering design". It took a Tacoma Narrows Bridge collapse to point out that nature did not always contract to follow an engineering theory. Therefore, despite a great deal of effort being made on development and refinement of theoretical approaches and much success being achieved on this front, experimental methods remain the mainstay of wind engineering research and application.

Much of the early wind tunnel testing was carried out in aeronautical tunnels within uniform flow. There were some cases in which the vertical variation of velocity was modelled. A notable change came when Jensen (1958) carried out experiments on building models, and observed that, "the correct model test for phenomena in the wind must be carried out in a turbulent boundary layer and the model law requires the boundary layer to be scaled as regards the velocity profile." Developments in wind tunnel modelling have since been on the upward trend.

The proliferation of wind tunnel facilities has been quite remarkable over the last 2-3 decades. This has been accompanied by improvements in the design of wind tunnels but even more so in the sophistication and capability of the instrumentation where there has been a dramatic change. A large part of a wind tunnel exercise for model studies can now be remote controlled and there is greater speed and dependability now possible. For the period in focus, it is a fair assessment that those who have made contributions clearly more significant than others in developing the design of wind tunnels and modelling, are J.E. Cermak (1990) and A.G. Davenport.

Interesting literature on low as well as tall buildings has emerged in recent years. To quote some of this : Ahmed (2000), Gupta (1996), Holmes (1983, 1995), Krishna (1995, 1998, 2001), Kwatra (2000), Stathopoulos (1984), Surrey (1999), Tieleman (1997), Yahyai (1990).

There are two moot points that emerge from the current state of the art. One, that the wind simulated in the tunnels is generally synoptic wind, whereas the extreme wind events that govern design wind speeds are often at substantial variance from the kind of wind flow being simulated. Letchford (2001) brings out this point quite clearly in a recent paper. The other point that has been driven home quite effectively in recent years is that the model studies in wind tunnel often fall short of representing the phenomenon in the field. This has pointed to the need for more work on prototype experimentation. Some good examples of prototype studies on low buildings are provided by the Aylesbury experiment (Eaton & Mayne, 1975), Texas Tech building (Levitan & Mehta, 1992), Silsoe structures (Robertson, 1992; Hoxey & Richards, 1995), and a few other structures. Similarly, there are examples where long span cable bridges have been instrumented in Japan. These studies are leading to an improvement in wind tunnel modelling of flow and structure as well as instrumentation. Figure 10 gives photographs of wind tunnels being used for testing models while figure 11 shows the field laboratory for full scale testing at Texas Tech University, USA.



1,2 Building models; 3,4 Study of interference; 5, Chimney model under study; 6, Study of the effect of topography on a bridge model.

Figure 10: Wind Tunnels in use for Model Testing

Figure 11: A Field Laboratory for Full Scale Testing at Texas Tech University

Despite decades of work in wind engineering research, there continue to be 'missing links'. To quote Martin Jensen (Davenport 1999) is apt in this regard :

"It may seem strange that within the vast research field incorrect model laws have been applied, but the explanation is both simple and not very flattering; the model tests have practically never been checked by full scale tests in nature".

To quote Davenport (1999) further, there are many examples of gray areas exemplified below:

- The erroneous assumptions concerning the distribution of wind pressures led to the failure of the Ferrybridge cooling towers.
- Topographic effects can easily give rise to wind action which are significantly different from full scale reality. For example, suspension bridges in mountainous region can have dramatic changes in wind force (> 50%) across the span.
- Wind structure in intense local storms such as tornadoes and "down–bursts" can deviate radically from large scale storms.
- It may be wrong to assume that wind forces on sharp edged bodies are only weakly dependent on scale or Reynolds number. This may be true most of the time but there is evidence that reattachment phenomena can produce a strong sensitivity to Reynolds Number which significantly affects pressures on the circular cylinders, bridge decks and house roofs.
- Wind loads on transmission towers in turbulent wind may deviate from those assumed in static testing. Full scale tests are needed to explore these questions.

1.5 CODES AND STANDARDS

Codes and standards and commentaries / Handbooks thereon provide a key source representing the state of the art information on a given aspect of engineering discipline enabling the extension of knowhow, based on research and experience, into practice.

In view of the increasingly improved database on wind itself, a better understanding of the structure of wind, greater knowledge about the response of structure to wind, the wind loading standards have been in frequent revision. Interpretation of codes often throws up questions, not always easy to resolve. Explanatory or illustrative material is thus of great help to a designer. This volume has been developed for this purpose.

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Section - 2

List of Examples

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.

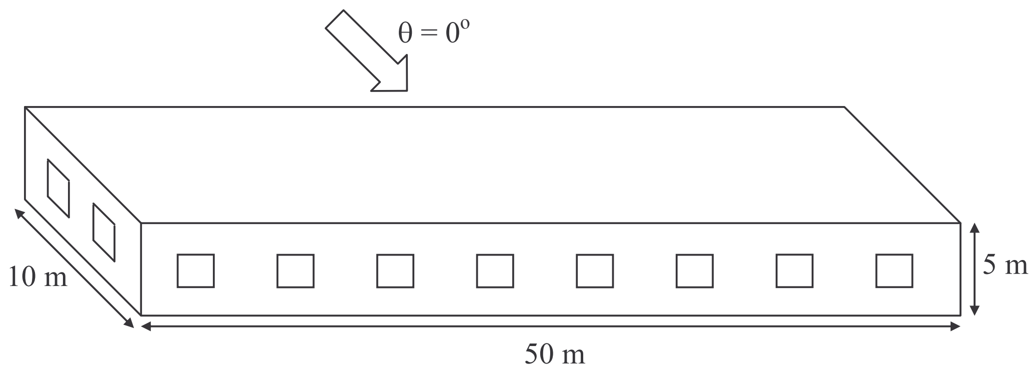


Figure 1.1

<u>Solution:</u>

Wind Data:

1. Wind Zone: Zone IV (V_b= 47m/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 $\boldsymbol{k_1}$ = 1.00
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor $\boldsymbol{k_2}$ = 1.00
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography Factor $\boldsymbol{k_3}$ = 1.00
(IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $\boldsymbol{k_4}$ = 1.00
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor $\boldsymbol{K_d}$ = 0.90
(IS:875-pt.3, Sec 6.1.1)
Area Averaging Factor $\boldsymbol{K_a}$
(IS:875-pt.3, Sec 6.1.2, Table-4)
Tributary area of Short walls = 10 x 5 = 50m^2
===➔ 0.867
Tributary area of Long walls = 50 x 5 = 250m^2
===➔ 0.80
Tributary area of roof = 50 x 10 = 500m^2
===➔ 0.80

<u>Permeability of the Building:</u>

Area of all the walls = 5× (2×10 + 2×50) = 600m^2
Area of all the openings = 20×1.5×1.5 = 45m^2
% opening area = 7.5 %, between 5% and 20%
Hence the building is of Medium permeability.
(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure:

Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$ =
47×1.0×1.0×1.0×1.0 = 47.00 m/s
(IS:875-pt.3, Sec 5.3)
$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m^2
(IS:875-pt.3, Sec 5.4)
$p_d = p_Z \times K_d \times K_a$ = 1.3254×0.9×0.867
=1.034kN/m^2 (short wall)
= 1.3254×0.9×0.8
= 0.954 kN/m^2 (long wall & Roof)
(IS:875-pt.3, Sec 6.1)

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 6.2.1)

Internal Pressure Coefficient

C_{pi} = ± 0.5

(IS:875-pt.3, Sec 6.2.2.2)

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

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

Table 1.1

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.8	-0.8
F	-0.8	-0.4
G	-0.4	-0.8
H	-0.4	-0.4

(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-a-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[1]

Table 1.2

Angle of Incidence	0^o	90^o
Wall – A	+0.7	-0.5
Wall – B	-0.25	-0.5
Wall – C	-0.6	+0.7
Wall – D	-0.6	-0.1

(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 corresponding to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to C_{pi} = ± 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

Design pressures for walls:

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

=1.2 × 1×1 × 0.954 = 1.1448 kN/m^2 Pressure

=-1.0 × 1×1 × 0.954 = -0.954 kN/m^2 Suction

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

=1.2 × 1×1 × 1.034 = 1.2408 kN/m^2 Pressure

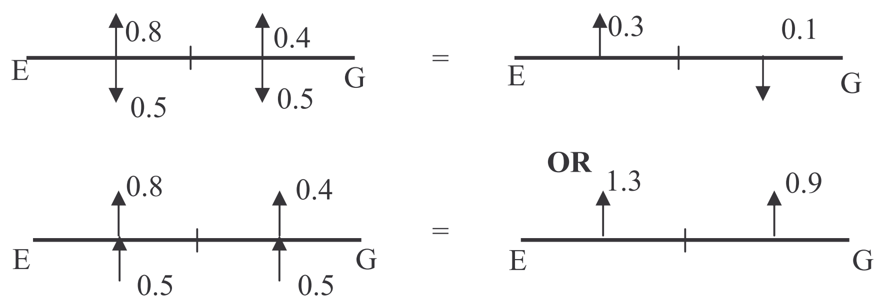
=-1.1 × 1×1 ×1.034 =-1.1374 kN/m^2 Suction

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

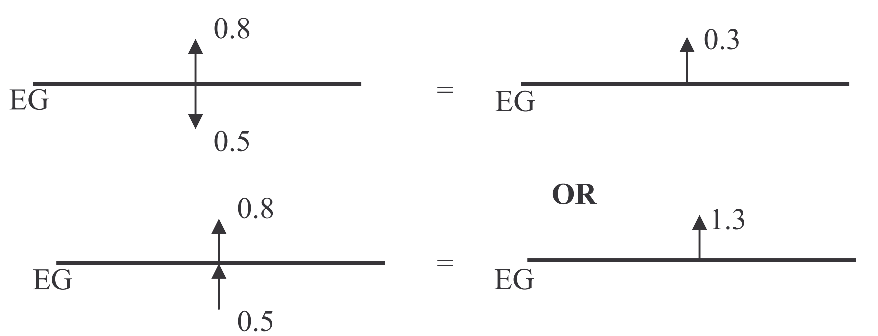
= –1.3 × 1×1 × 0.954 =1.2402 kN/m^2 Suction

= +0.1 × 1×1 ×0.954 =-0.0954 kN/m^2 Pressure

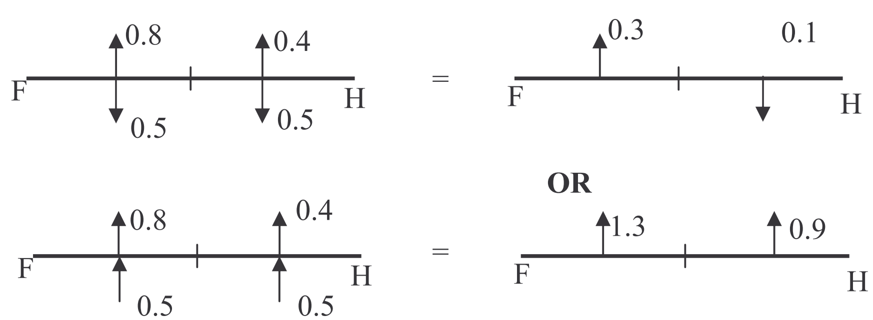
For 0° wind incidence, for E/G (End Zone)



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)

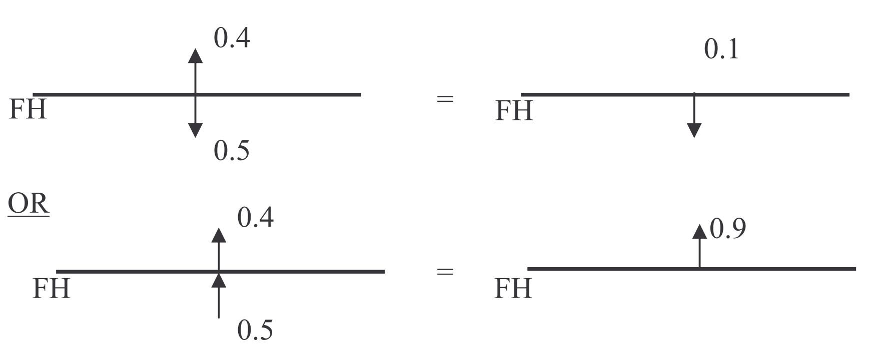


Figure 1.2- Net Roof Pressure Coefficients for different zones and combinations

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?

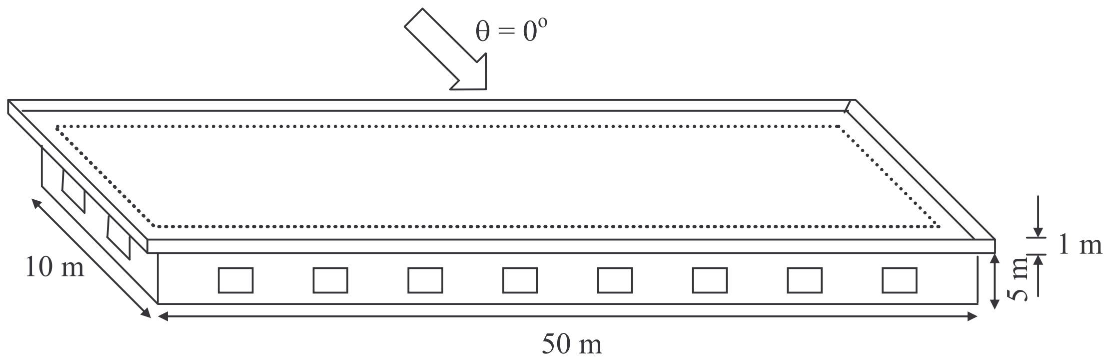


Figure 2.1

Solution:

Wind Data:

1. Wind Zone: Zone IV (V_b= 47m/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 6.1.1)
Area Averaging Factor K_a
(IS:875-pt.3, Sec 6.1.2, Table-4)
Tributary area of Short walls = 10 x 5 = 50m^2
===➔ 0.867
Tributary area of Long walls = 50 x 5 = 250m^2
===➔ 0.80
Tributary area of roof = 53 x 13 = 689m^2
===➔ 0.80

Permeability of the Building:
Area of all the walls = $5\times(2\times10 + 2\times50) = 600$m^2
Area of all the openings = $20\times1.5\times1.5 = 45$m^2
% opening area = 7.5 %, between 5% and 20%
Hence the building is of Medium permeability.
(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure:

Design Wind Speed = $V_Z = V_b\times k_1\times k_2\times k_3\times k_4$ =
$47\times1.0\times1.0\times1.0\times1.0 = 47.00$ m/s
(IS:875-pt.3, Sec 5.3)
$p_Z = 0.6\,(V_Z)^2 = 0.6\times(47.00)^2 = 1325.4$ N/m^2
(IS:875-pt.3, Sec 5.4)
$p_d = p_Z\times K_d\times K_a$ = $1.3254\times0.9\times0.867$
=1.034kN/m^2 (short wall)
= $1.3254\times0.9\times0.8$
= 0.954 kN/m^2 (long wall)
(IS:875-pt.3, Sec 6.1)
$p_d = p_Z\,.\,K_d\,.\,K_a = 1.3254\times0.9\times0.8$
= 0.954 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:

$$F = (C_{pe}-C_{pi})\times A\times p_d$$
(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient
C_{pi} = ± 0.5
(IS:875-pt.3, Sec 6.2.2.2)
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 [**]

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

Table 2.1

Portion of roof	**Wind Incidence Angle** 0^o	90^o
E	-0.8	-0.8
F	-0.8	-0.4
G	-0.4	-0.8
H	-0.4	-0.4

(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-a-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_{pe} for walls*

Table 2.2

Angle of Incidence ➔	0^o	90^o
Wall – A	+0.7	-0.5
Wall – B	-0.25	-0.5
Wall – C	-0.6	+0.7
Wall – D	-0.6	-0.1

(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 corresponding 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

Design pressures for walls:

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

$=1.2 \times 1 \times 1 \times 0.954 = 1.1448$ kN/m^2 Pressure

$=-1.0 \times 1 \times 1 \times 0.954 = -0.954$ kN/m^2 Suction

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

$=1.2 \times 1 \times 1 \times 1.034 = 1.2408$ kN/m^2 Pressure

$=-1.1 \times 1 \times 1 \times 1.034 =-1.1374$ kN/m^2 Suction

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

$= -1.3 \times 1 \times 1 \times 0.954 = 1.24$ kN/m^2 suction

$=+0.1 \times 1 \times 1 \times 0.954 = -0.0954$ kN/m^2 pressure

Max. Design Pressure on parapets shall be same as pressure on the corresponding wall at the top but with $K_a = 1.0$ ➔

$1.2 \times 1.0 \times 1.0 \times 1.3254 \times 0.9 = 1.4314$ kN/m^2 (Pressure)

$-1.1 \times 1.0 \times 1.0 \times 1.3254 \times 0.9 = -1.3121$ kN/m^2 (Suction)

Pressure coefficients on overhanging portion of Roof: (IS:875-pt.3, Sec 6.2.3.5)

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

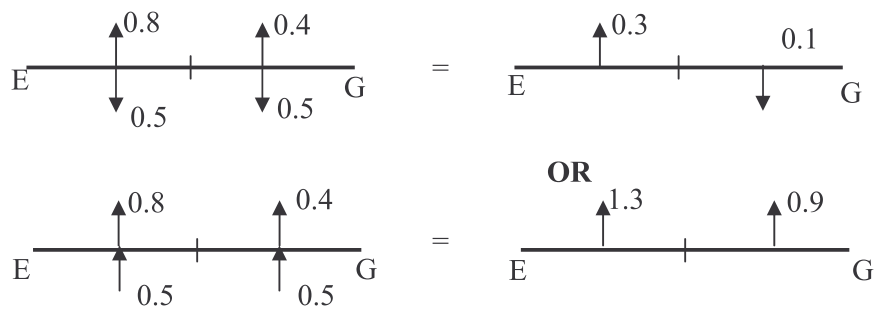
On the underside of overhang: since the overhang is horizontal, the max. Pressure coefficient shall be +1.0 (Section 6.2.3.5)

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 $K_a = 1.0$, i.e. $p_d = 1.3254 \times 0.9 = 1.193$ kN/m^2

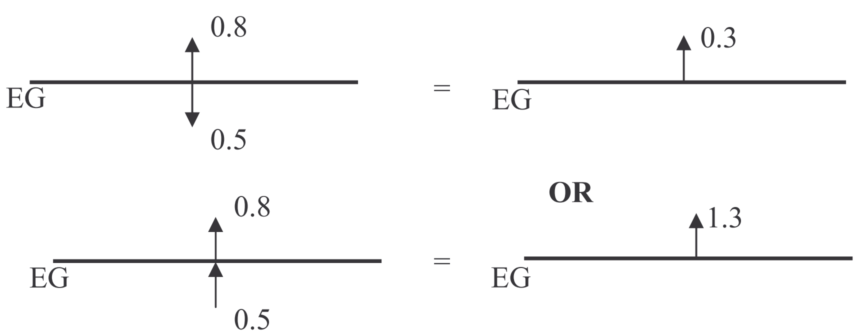
Max. Design Force = $1.8 \times 1.193 = 2.147$ kN/m^2

[**] Roof pressure coefficients for flat roofs without overhangs or parapet are adopted.

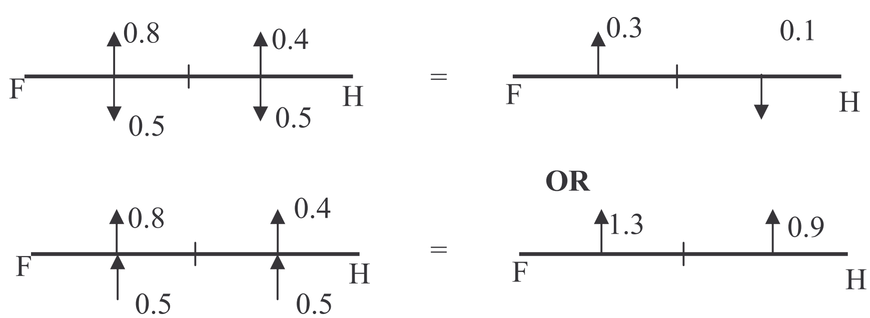
For 0° wind incidence, for E/G (End Zone)



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)

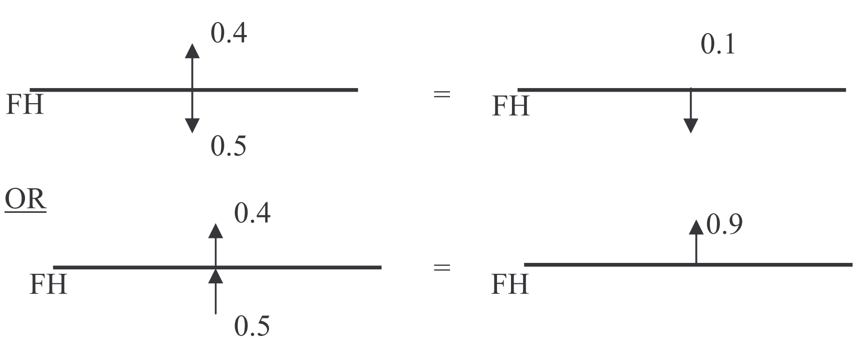


Figure 2.2- Net Roof Pressure Coefficients for different zones and combinations

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 5mc/c horizontally and 3m c/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 × 1.5m.

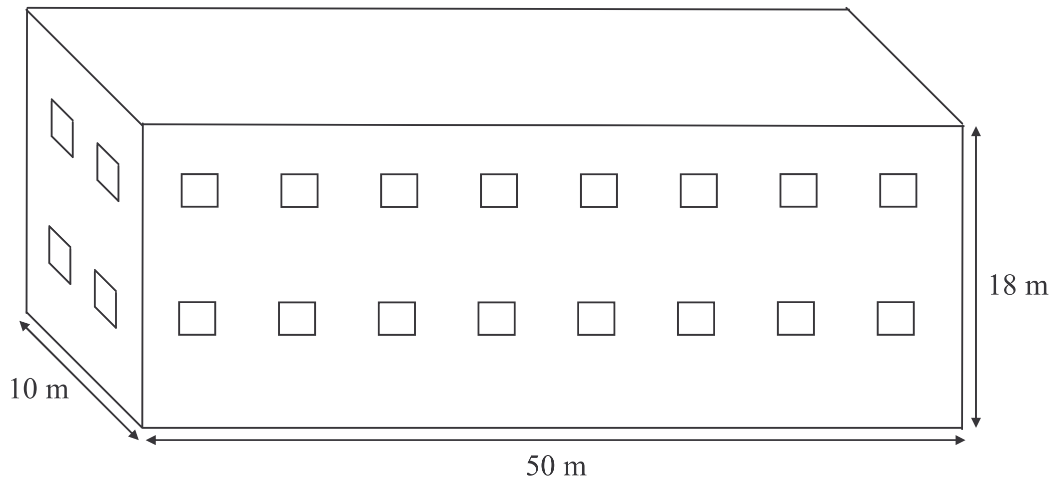


Figure 3.1

Solution:

Wind Data:

1. Wind Zone: Zone IV (V_b= 47m/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 $\boldsymbol{k_1}$ = 1.00
(IS:875-pt.3, Sec 5.3.1, Table-1)
Terrain & Height Factor $\boldsymbol{k_2}$ = varying with height as in Table 3-1.
(IS:875-pt.3, Sec 5.3.2.2, Table-2)
Topography Factor $\boldsymbol{k_3}$ = 1.00
(IS:875-pt.3, Sec 5.3.3.1)
Importance Factor for Cyclonic Region $\boldsymbol{k_4}$ = 1.00
(IS:875-pt.3, Sec 5.3.4)
Wind Directionality Factor $\boldsymbol{K_d}$ = 0.90
(IS:875-pt.3, Sec 6.1.1)
Area Averaging Factor $\boldsymbol{K_a}$
(IS:875-pt.3, Sec 6.1.2, Table-4)

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

Combination factor K_c is to be considered for the design of frames as per Section 6.2.3.13 and Table-20 of IS:875-pt.3.

Permeability of the Building:

Area of all the walls = 18× (2×10+2×50) = 2160 m^2
Area of all the openings = 40×1.5×1.5 = 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 = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$

$= 47 \times 1.0 \times k_2 \times 1.0 \times 1.0 = (47 \times k_2)$ m/s

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

$p_Z = 0.6\,(V_Z)^2$ & $p_d = p_Z \times K_d \times K_a$
(IS:875-pt.3, Sec 5.4 & Sec 6.1)

Table 3.1: Calculation of Variation in Design Wind Speed & Pressure with Height

Height from Ground, m	k_2	V_Z m/s	p_Z kN/m^2	p_d (kN/m^2) Column	Roof beam
Up to 10m	1.0	47	1.3254	0.970	----------
15m	1.05	49.35	1.461	1.069	----------
18m	1.068$^+$	50.196	1.512	1.106	1.180

+ : linearly interpolated

Wind Load Calculations:

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

(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient

$C_{pi} \quad = \quad \pm 0.2$

(IS:875-pt.3, Sec 6.2.2.2)

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^o without local coefficients. For h/w = 1.8, pressure coefficients are tabulated below

Table 3.2

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.7	-0.9
F	-0.7	-0.7
G	-0.6	-0.9
H	-0.6	-0.7

Design Pressure Coefficients for Roof:

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

$= 0.6 \times (50.196)^2 \times 0.9 \times 0.867 = 1.180$ 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-a-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[1]

Table 3.3

Angle of Incidence	0^o	90^o
Wall – A	+0.7	- 0.5
Wall – B	- 0.4	- 0.5
Wall – C	- 0.7	+ 0.8
Wall – D	- 0.7	- 0.1

[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 corresponding to 4 are being used.

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

C_{pnet} for Walls A or B

= 0.7 – (-0.2) = +0.9, pressure

= -0.5 – (+0.2) = -0.7, suction

C_{pnet} for Walls C or D

= 0.8 – (-0.2) = +1.0, pressure

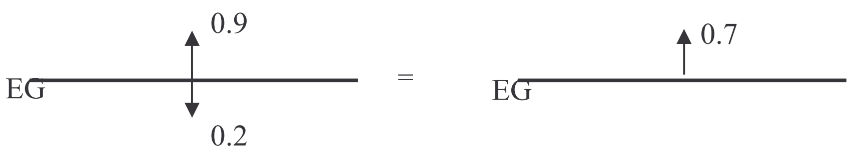
= -0.7 – (+0.2) = -0.9, suction

These C_{pnet} values multiplied by respective design pressure, depending on element & height give the design force per unit area, as in the previous example.

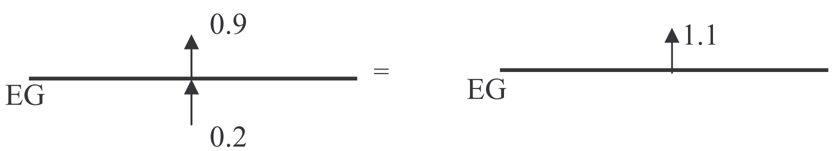
For 0° wind incidence, for E/G (End Zone)

or

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

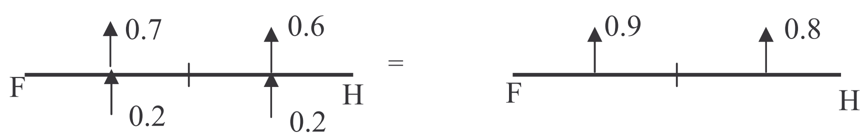


or

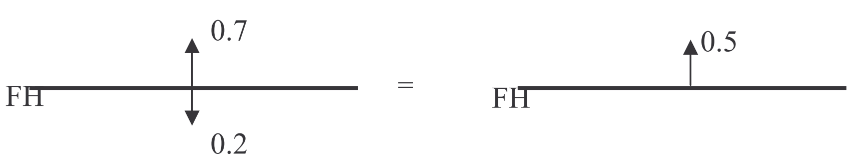


For 0° wind incidence, for F/H (Mid Zone)

or



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



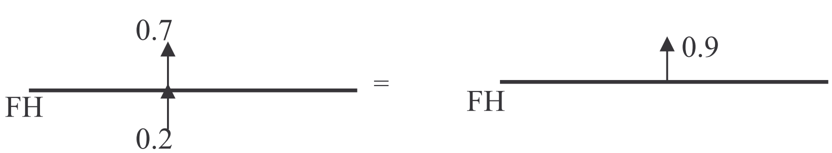


Figure 3.2- Net Roof Pressure Coefficients for different zones and combinations

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 α is 15^0. 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.

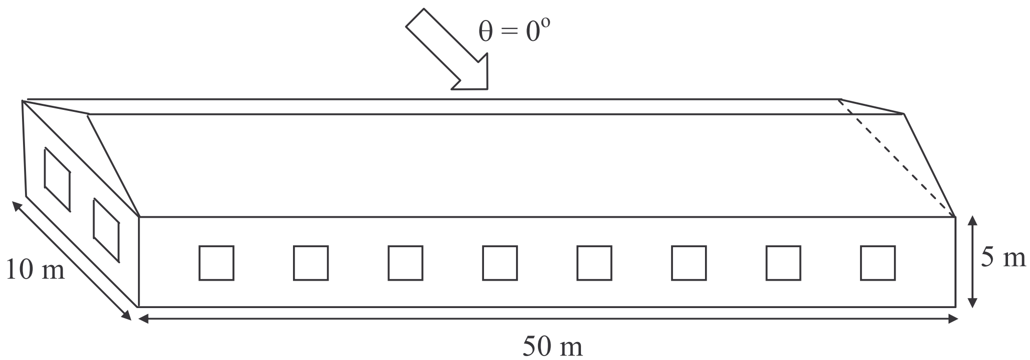


Figure 4.1

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 6.1.1)

Area Averaging factor K_a

(IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for columns = 5 x 5 = 25 m^2

==============================➔ 0.9

Tributary area for Trusses =2x5.176x5=51.76m^2

==============================➔ 0.864

Tributary area for Purlins = 1.4 x 5 = 7.0 m^2

==============================➔ 1.0

Tributary area of short walls for design of wind braces in plan = 10 x 5 + 0.5 x10x1.34 =56.7 m^2

============================➔ 0.858

Permeability of the Building:

Area of all the walls

= 5× (2×10 + 2×50) + 2×0.5×1.34×10 = 613.4 m^2

Area of all the openings

= 20×1.5×1.5 = 45 m^2

% opening area = 7.336 %, between 5% and 20%. Hence the building is of medium permeability.

(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure:

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

$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m^2
(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a = 1.193\ K_a$
(IS:875-pt.3, Sec 6.1)

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.

<u>Wind Load Calculations:</u>

$F = (C_{pe}\text{-}C_{pi}) \times A \times p_d$
(IS:875-pt.3, Sec 6.2.1)

<u>Internal Pressure Coefficient</u> $C_{pi} = \pm 0.5$
(IS:875-pt.3, Sec 6.2.2.2)

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.

<u>External Pressure Coefficients</u>

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

Table 4.1

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-0.6

<u>Design Pressure Coefficients for Roof:</u>

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.

<u>Design Pressure Coefficients for Walls:</u>

Refer Table 5 of the code: $h/w = 0.5$, and $l/w = 5$ therefore C_{pe} for walls[1]

Table 4.2

Angle of Incidence	0^o	90^o
Wall – A	+0.7	- 0.5
Wall – B	- 0.25	- 0.5
Wall – C	- 0.6	+ 0.7
Wall – D	- 0.6	- 0.1

[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 corresponding 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

<u>Local pressure coefficients for the design of claddings and fasteners</u>

Refer Table 6 of IS-875 for Roof Angle = 15^0

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_{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, $p_d = 1.3254 \times 0.9 = 1.193$ kN/m^2

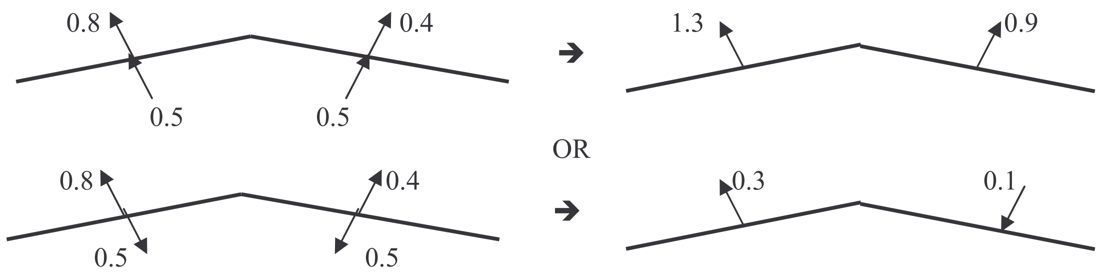
Zone of local coefficients = $0.15 \times 10 = 1.5$m, at ridges, eaves and gable ends & $0.25 \times 10 = 2.5$m for wall corners. In this region the cladding and fasteners shall be checked for increased force.
Refer note below Table 6 of code

<u>Calculations of Force due to Frictional Drag:</u>

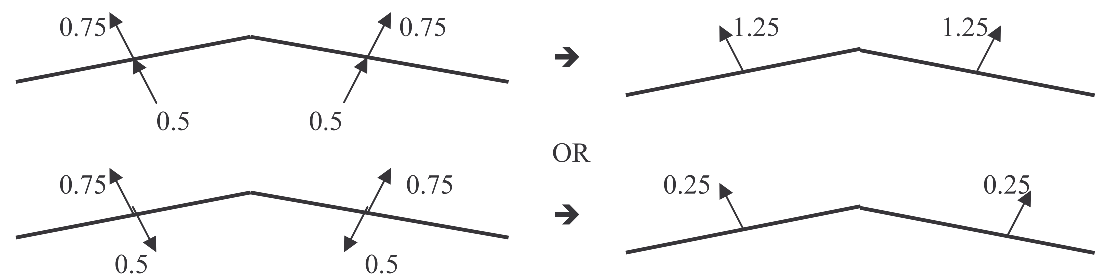
(IS:875-pt.3, Sec 6.3.1)

This will act in the longitudinal direction of the building along the wind. Here $h<b$, therefore, first equation will be used & $C_f' = 0.02$. K_a for roof and walls is 0.8, as area is more than 100m^2.

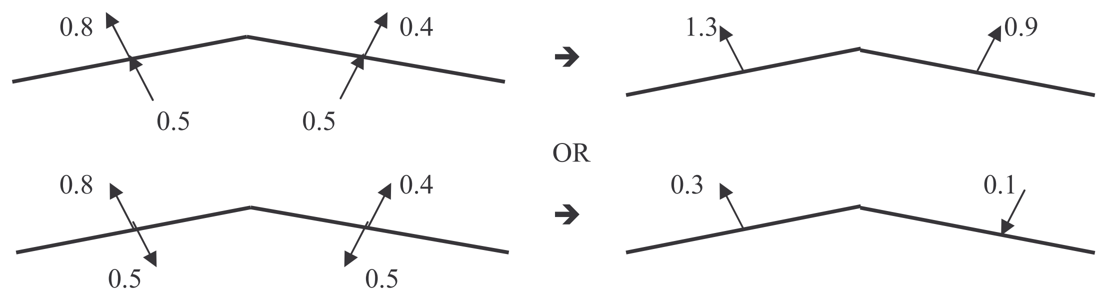
For End Zone E/G; 0° wind incidence



For End Zone E/G; 90° wind incidence



For Mid Zone F/H; 0° wind incidence



For Mid Zone F/H; 90° wind incidence

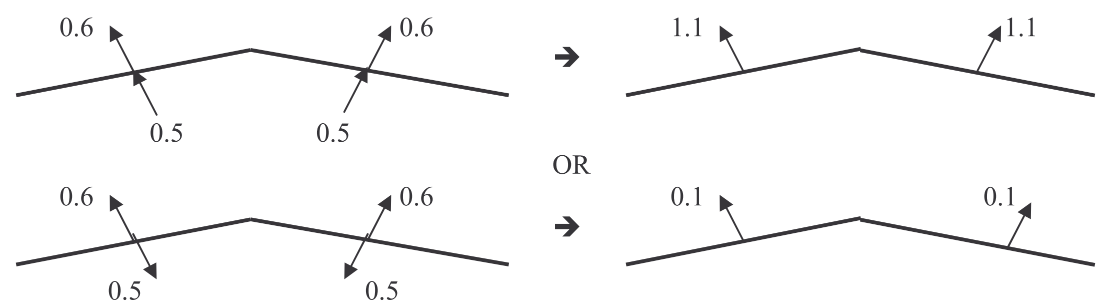


Figure 4.2- Net Roof Pressure Coefficients for different zones and combinations

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?

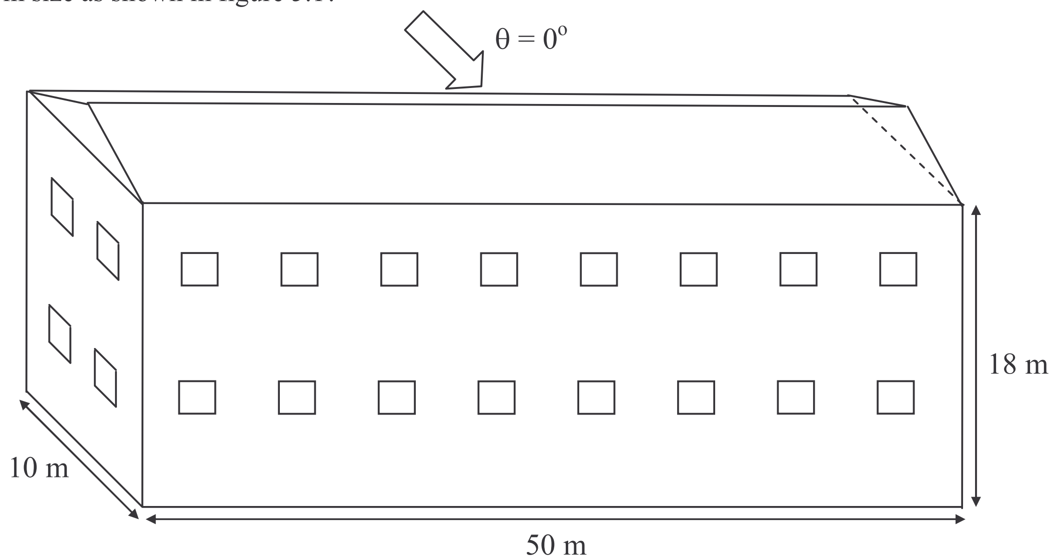


Figure 5.1

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 6.1.1

Area Averaging factor K_a:
(IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for columns = 5 ×18 = 90m^2
==============================➔0.813

Tributary area for Trusses = 2×5.176 × 5= 51.76m^2
=============================➔ 0.864

Tributary area for Purlins = 1.4 × 5 = 7.0m^2
===============================➔ 1.0

Tributary area of short walls for design of wind braces in plan = 10 × 18 + 0.5 × 10 × 1.34 = 186.7m^2 =======================➔ 0.80

Permeability of the Building:

Area of all the walls = 18× (2×10+2×50) = 2160 m^2
Area of all the openings = 40×1.5×1.5 = 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 = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$ = $47 \times 1.0 \times k_2 \times 1.0 \times 1.0 = (47 \times k_2)$ m/s
(IS:875-pt.3, Sec 5.3)

$p_Z = 0.6\,(V_Z)^2$ & $p_d = p_Z \times K_d \times K_a$
(IS:875-pt.3, Sec 5.4 & Sec 6.1)

Table 5.1

Calculations of variation in design wind speed & pressure with height

Height from Ground, m	k_2	V_Z m/s	p_Z kN/m^2	p_d column	p_d truss	p_d purlin
Up to 10m	1.00	47.00	1.325	0.970	---	---
15m	1.05	49.35	1.461	1.069	---	---
18m	1.07$^+$	50.20	1.512	1.106	1.176	1.361

+ : linearly interpolated.

Wind Load Calculations:

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

(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient $C_{pi} = \pm 0.2$

(IS:875-pt.3, Sec 6.2.2.2)

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 Coefficients

On Roof: Using the Table 6 with roof angle 15^o

For $h/w = 1.8$, pressure coefficients are tabulated in Table 5.2 (refer figure below Table 6 of code)

Table 5.2

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.75	-0.8
F	-0.75	-0.8
G	-0.6	-0.8
H	-0.6	-0.8

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 5.2.

Design Pressure Coefficients for Walls:

Refer Table 5 of code: $h/w = 1.8$, and $l/w = 5$ therefore C_{pe} for walls[1]

Table 5.3

Angle of Incidence	0^o	90^o
Wall – A	+ 0.7	- 0.5
Wall – B	- 0.4	- 0.5
Wall – C	- 0.7	+ 0.8
Wall – D	- 0.7	- 0.1

[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 corresponding to 4 are being used.

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

C_{pnet} for Walls A or B

$= 0.7 - (-0.2) = +0.9$, pressure

$= -0.5 - (+0.2) = -0.7$, suction

C_{pnet} for Walls C or D

$= 0.8 - (-0.2) = +1.0$, pressure

$= -0.7 - (+0.2) = -0.9$, suction

Local pressure coefficients for the design of claddings and fasteners

Refer Table 5 of IS-875 for Wall and Table 6 for Roof (Angle = 15^0)

Local C_{pe} for eaves portion in end zone: -1.75
Local C_{pe} for eaves portion in mid zone: -1.5
Local C_{pe} for ridge portion: -1.2
Local C_{pe} for gable edges: -1.75
Local C_{pe} for corners of walls: -1.2

Therefore Max. local C_{pnet} for roof at the edges and the ridge $= -1.75 - (+0.2) = -1.95$

Likewise at the wall edges $= -1.2 - (+0.2) = -1.4$

However, for the use of the local pressure coefficients, the design pressure p_d will be computed with $K_a = 1$. Therefore, $p_d = 1.512 \times 0.9 = 1.3606$ 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.

(Refer note below table 6 of IS:875-pt.3)

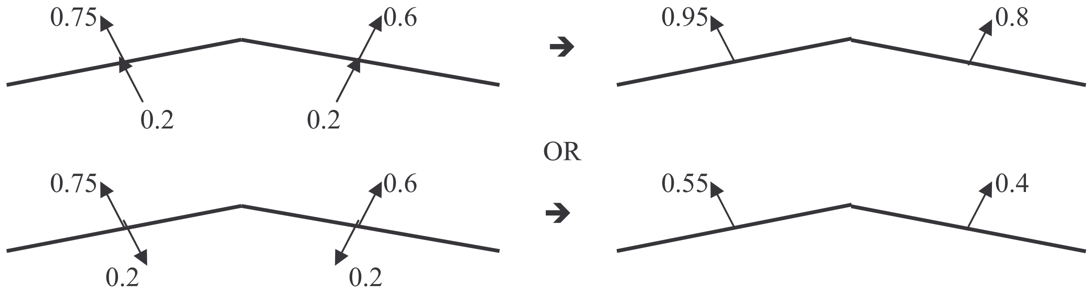
Calculations of Force due to Frictional Drag:

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

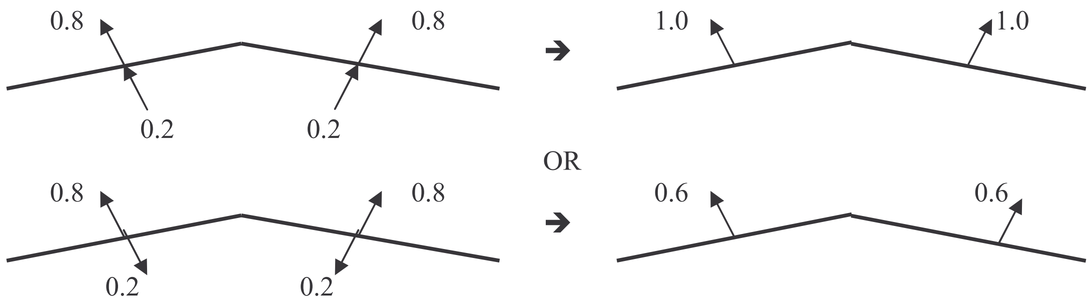
This will act in the longitudinal direction of the building along the wind. Here $h>b$, therefore, second equation will be used & $C_f' = 0.02$. K_a for roof and walls is 0.8, as area is more than 100m^2.

****

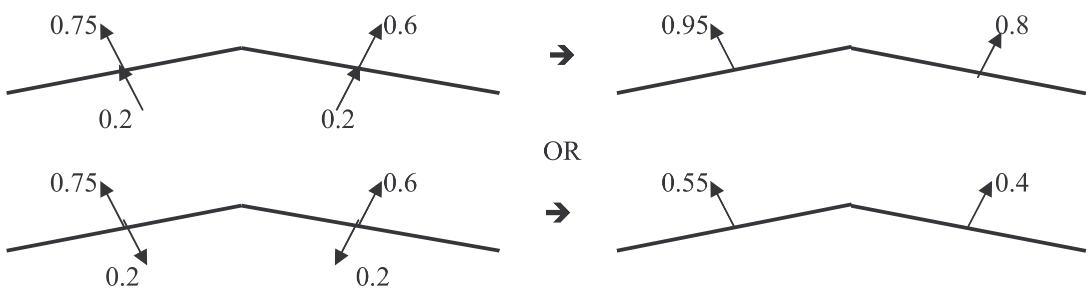
For End Zone E/G; 0° wind incidence



For End Zone E/G; 90° wind incidence



For Mid Zone F/H; 0° wind incidence



For Mid Zone F/H; 90° wind incidence

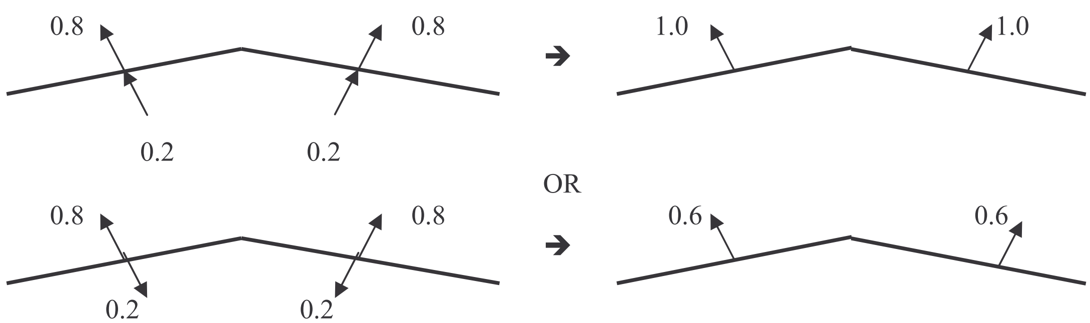


Figure 5.2- Net Roof Pressure Coefficients for different zones and combinations

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 Vishkhapattanam (Andhra Pradesh) near seacoast?

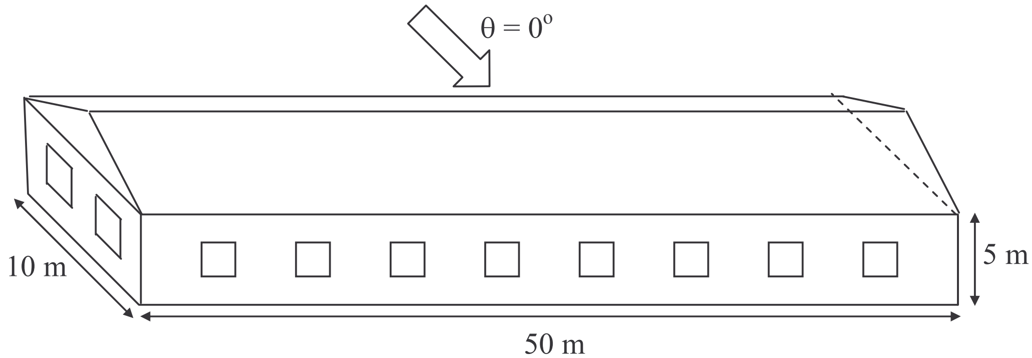


Figure 6.1

Solution:

Wind Data:

1.Wind Zone: Zone V (V_b= 50m/s)---------→
(IS:875-pt. 3, Sec 5.2)

Note: Vishakhapattanam 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----→
(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 6.1.1)

Area Averaging factor K_a
(IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for columns = 5 × 5 = 25 m^2 → 0.9

Tributary area for Trusses =2×5.176×5=51.76m^2
==============================→ 0.864

Tributary area for Purlins = 1.4 × 5 = 7.0 m^2
==============================→ 1.0

Tributary area of short walls for design of wind braces in plan = 10 × 5 + 0.5 ×10×1.34 =56.7 m^2
==============================→ 0.858

* : use 1.15 for Industrial structures

Permeability of the Building:

Area of all the walls
= 5× (2×10 + 2×50) + 2×0.5×1.34×10 = 613.4 m^2

Area of all the openings
= 20×1.5×1.5 = 45 m^2

% opening area = 7.336 %, between 5% and 20%. Hence the building is of medium permeability.

(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$ = 50×1.0×1.05×1.0×1.15 = 60.375 m/s

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

p_Z = 0.6 $(V_Z)^2$ = 0.6× (60.375)^2 = 2187 N/m^2

(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a$
=2.187×0.9×0.858
= 1.689 kN/m^2(short wall)
=2.187×0.9×0.8
=1.574 kN/m^2 (long wall & Roof)
(IS:875-pt.3, Sec 6.1)

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

Wind Load Calculations:

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

(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient $C_{pi} = \pm 0.5$

(IS:875-pt.3, Sec 6.2.2.2)

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.6.2.2.1)

External Pressure Coefficients

Using the Table 6 with roof angle 15^o
For h/w = 0.5, pressure coefficients are tabulated below (IS:875-pt.3, Table 6)

Table 6.1

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-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 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[1]

Table 6.2

Angle of Incidence	0^o	90^o
Wall – A	+0.7	- 0.5
Wall – B	- 0.25	- 0.5
Wall – C	- 0.6	+ 0.7
Wall – D	- 0.6	- 0.1

[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 corresponding 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 = 15^0
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
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/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.

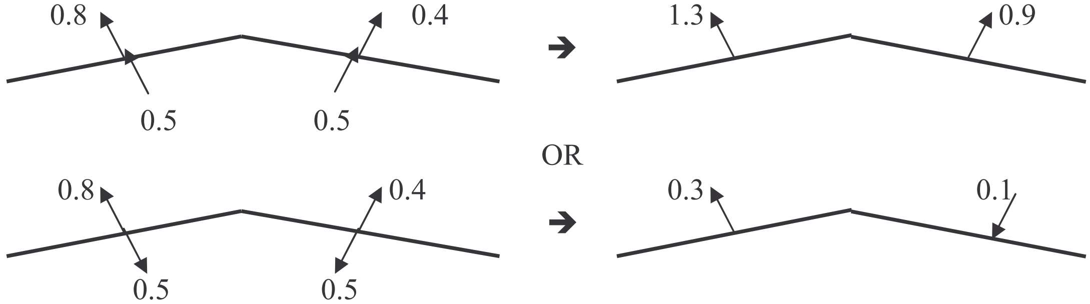
(IS:875-pt. 3, Table 6)

Calculations of Force due to Frictional Drag:

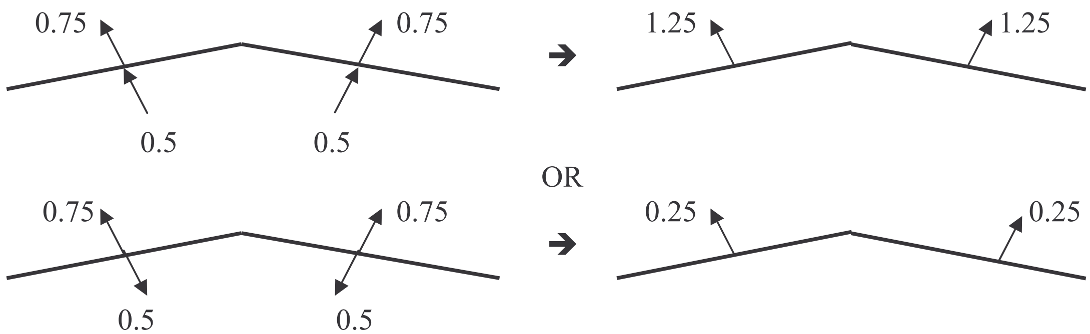
(IS:875-pt.3, Sec 6.3.1)

This will act in the longitudinal direction of the building along the wind. Here $h<b$, therefore, first equation will be used & $C_f' = 0.02$. K_a for roof and walls is 0.8, as area is more than 100m^2.

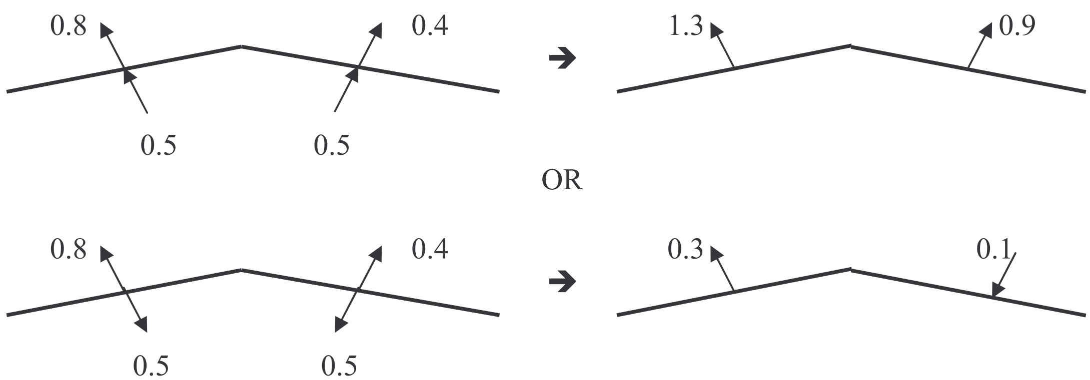
For End Zone E/G; 0^o wind incidence



For End Zone E/G; 90^o wind incidence



For Mid Zone F/H; 0^o wind incidence



For Mid Zone F/H; 90^o wind incidence

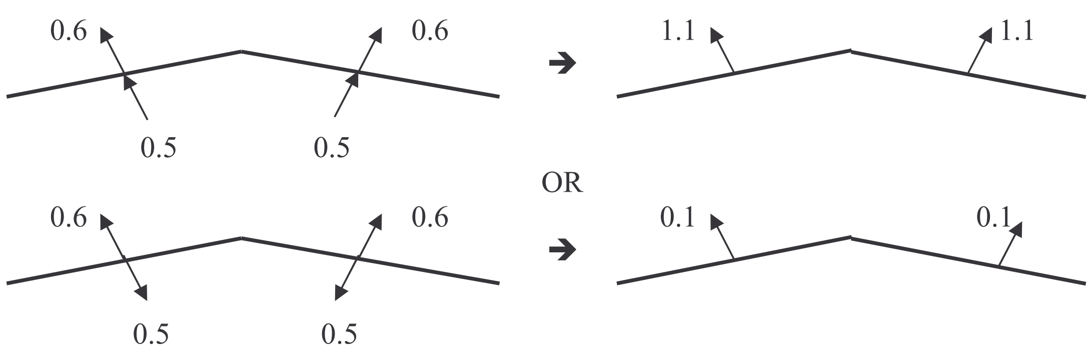


Figure 6.2 - Net Roof Pressure Coefficients for different zones and combinations

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?

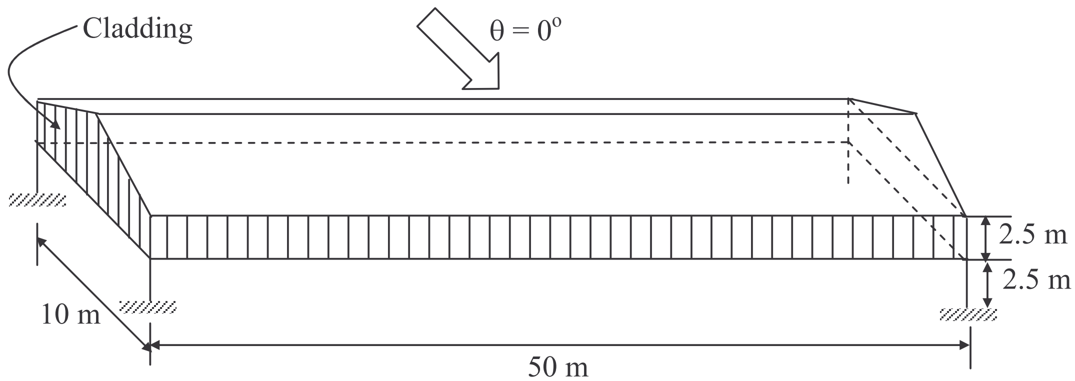


Figure 7.1

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 columns = 5 ×2.5 = 12.5 m^2
==============================➔ 0.983

Tributary area for Trusses =2 × 5.176 × 5 = 51.76m^2
==============================➔ 0.864

Tributary area for Purlins = 1.4 × 5 = 7.0 m^2
==============================➔ 1.0

Tributary area of short walls for design of wind braces in plan= 10 × 2.5 + 0.5 ×10×1.34 =31.7 m^2
==========================➔ 0.891

(IS:875-pt.3, Sec 6.1.2, Table-4)

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 6.2.2.2. As per para 1 of section 6.2.2.2 use more than 20% opening clause and consider ±0.7 internal pressure on walls and roof.

Design Wind Pressure:

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

$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m^2
(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a$
$= 1.193 \times K_a$ kN/m^2
(IS:875-pt.3, Sec 6.1)

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 6.2.1)

Internal Pressure Coefficient C_{pi} = ± 0.7

External Pressure Coefficients:

Using the Table 6 with roof angle 15°

For h/w = 0.5, pressure coefficients are tabulated below (refer figure below Table 6 of code)

Table 7.1

Portion of roof	Wind Incidence Angle 0°	90°
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-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 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 7.2

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 corresponding to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to C_{pi} = ± 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°

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

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 × 0.9 = 1.193 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.

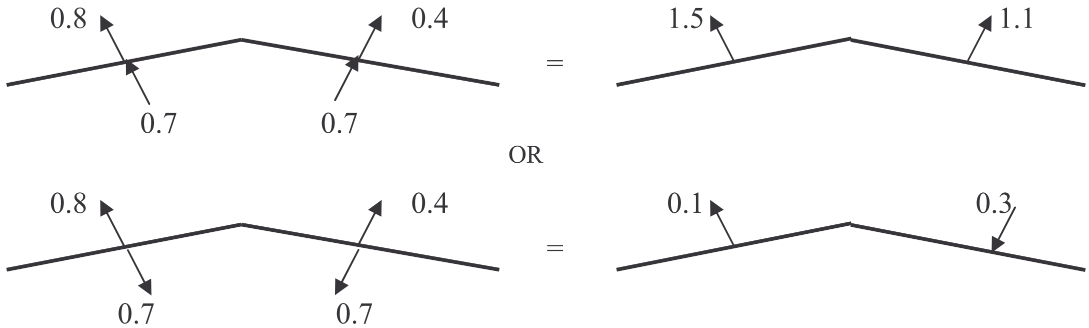
(IS:875-pt.3, Table-5)

Calculations of Force due to Frictional Drag:

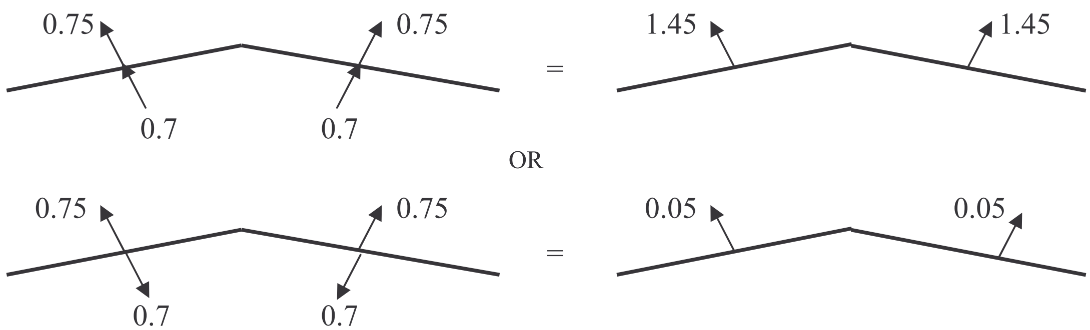
(IS:875-pt.3, Sec 6.3.1)

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.

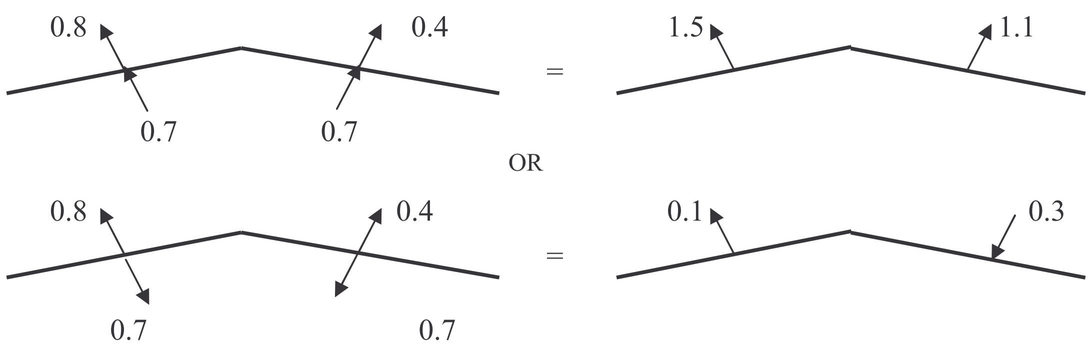
For End Zone E/G; 0° wind incidence



For End Zone E/G; 90° wind incidence



For Mid Zone F/H; 0° wind incidence



For Mid Zone F/H; 90° wind incidence

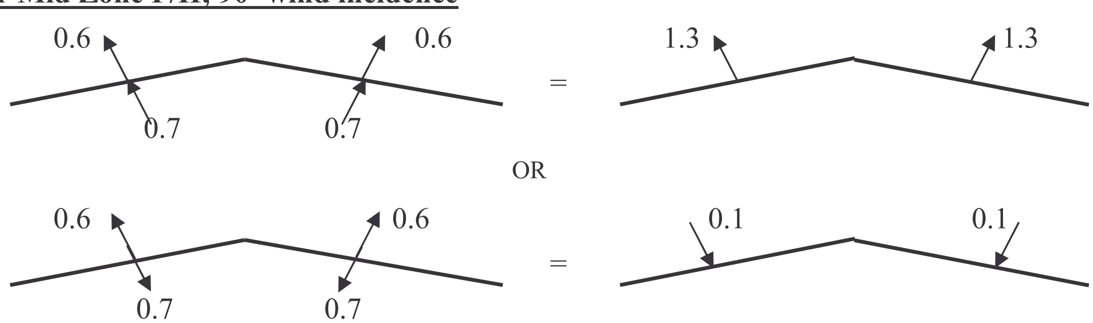


Figure 7.2 - Net Roof Pressure Coefficients for different zones and combinations

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.

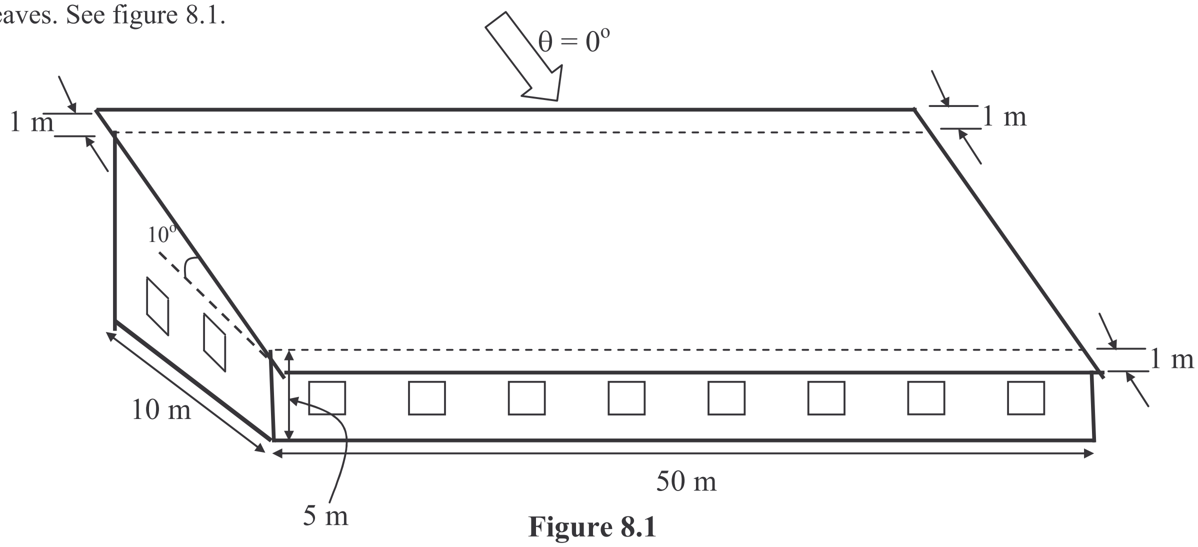


Figure 8.1

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 6.1.1)

Area Averaging factor K_a

(IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for columns = 5 × 6.76 = 33.81 m^2 =➔ 0.888

Tributary area for Trusses =12.19 × 5=60.95m^2 =➔ 0.852

Tributary area for Purlins = 1.4 × 5 = 7.0 m^2 =➔ 1.0

Tributary area of short walls for design of wind braces in plan

= 10 × 5 + 0.5 × 10 × 1.76 = 58.8 m^2 =➔ 0.855

Permeability of the Building:

Area of all the walls = 5× (2×10 + 2×50) + 2 × 0.5×1.76×10 + 1.76×50 = 705.6 m^2

Area of all the openings = 20×1.5×1.5 = 45 m^2

% opening area = 6.378 %, between 5% and 20%

Hence the building is of medium permeability

(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

Design Wind Speed = V_Z = $V_b \times k_1 \times k_2 \times k_3 \times k_4$

= 47×1.0×1.0×1.0×1.0 = 47.00 m/s

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

$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m^2

(IS:875-pt.3, Sec 5.4)

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

$= 1.193 \times K_a$ kN/m^2

(IS:875-pt.3, Sec 6.1)

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 6.2.1)

Table 8.2

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

Table 8.3

Coefficients for underside of overhang portion

Roof side ↓ Wind angle-->	**0°**	**90°**	**180°**	**270°**
Above wall A (higher end)	**+0.75**	-0.50	-0.25	-0.50
Above wall B (Lower end)	-0.25	-0.50	**+1.25**	-0.50
Above wall C (gable end)	-0.60	**+1.00**	-0.60	-0.10
Above wall D (gable end)	-0.60	-0.10	-0.60	**+1.00**

(Refer Clause 6.2.3.5 & Table 5)

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 6.2.3.5)

Design Pressure Coefficients for Roof:

Table 8.1

Portion of Roof	**Wind Incidence Angle**				
	0°	**45°**	**90°**	**135°**	**180°**
Windward (widthwise left half)	-1.0	-1.0	-1.0/ -0.5	-0.8	-0.4
Leeward (widthwise right half)	-0.5	-0.8	-1.0/ -0.5	-1.0	-1.0

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.

Internal Pressure Coefficient C_{pi} = ± 0.5

(IS:875-pt.3, Sec 6.2.2.2)

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 6.2.2.1) along-with external pressure coefficient.

External pressure coefficients will be combined with internal pressure coefficients as earlier, equal to C_{pi} = ± 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:

Local coefficients for roof: Max. value from all the values given in Table 7 of IS:875-pt.3, i.e.{–2.0- (+0.5)} = -2.5, up to 0.15 x w = 0.15 x 12 = 1.8m on all edges of roof.

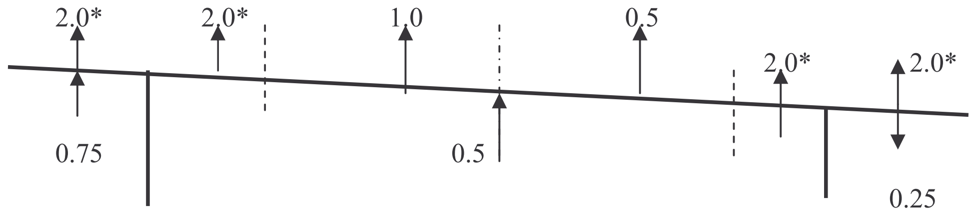
Local coefficients for walls: From Table 5 of the IS:875-pt.3, it is {-1.0 –(0.5)}= -1.5, for a distance of 0.25 x w = .25 x 10 = 2.5m at all corners.

Calculations of Force due to Frictional Drag:

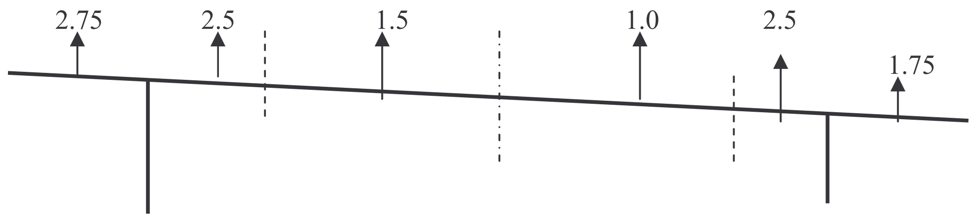
(IS:875-pt.3, Sec 6.3.1)

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. K_a for roof and long walls is 0.8, as area is more than 100m^2.

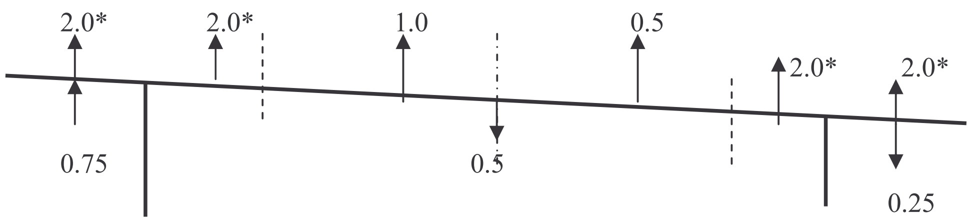
For 0° wind incidence, Cpi = +0.5



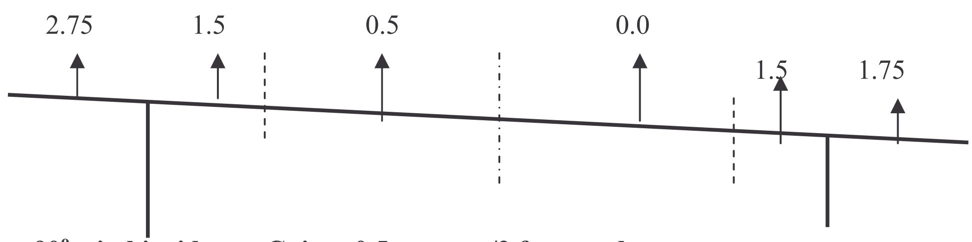
Which is equivalent to ↓



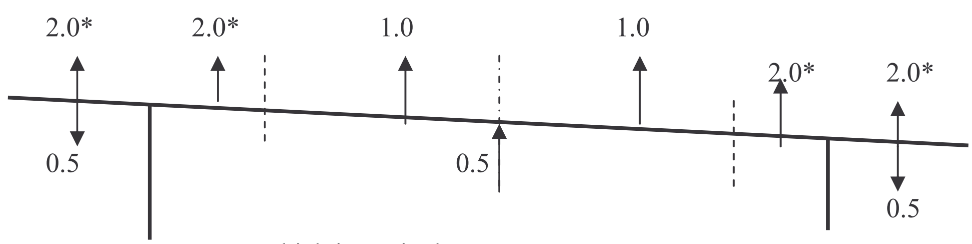
For 0° wind incidence, Cpi = -0.5



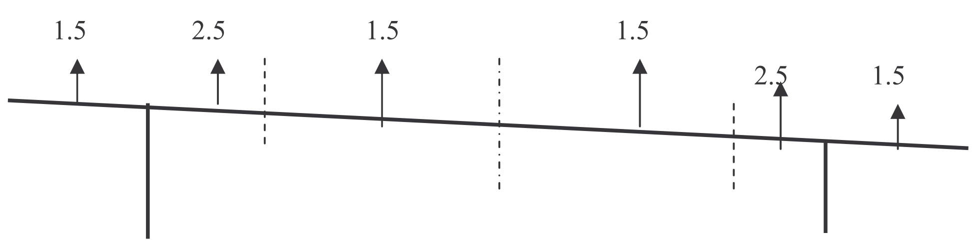
Which is equivalent to ↓



For 90° wind incidence, Cpi = +0.5, up to w/2 from ends



Which is equivalent to ↓



And similarly for other combinations.
* These are local pressure coefficients

Figure 8.2 - Net Roof Pressure Coefficients for different zones and combinations

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?

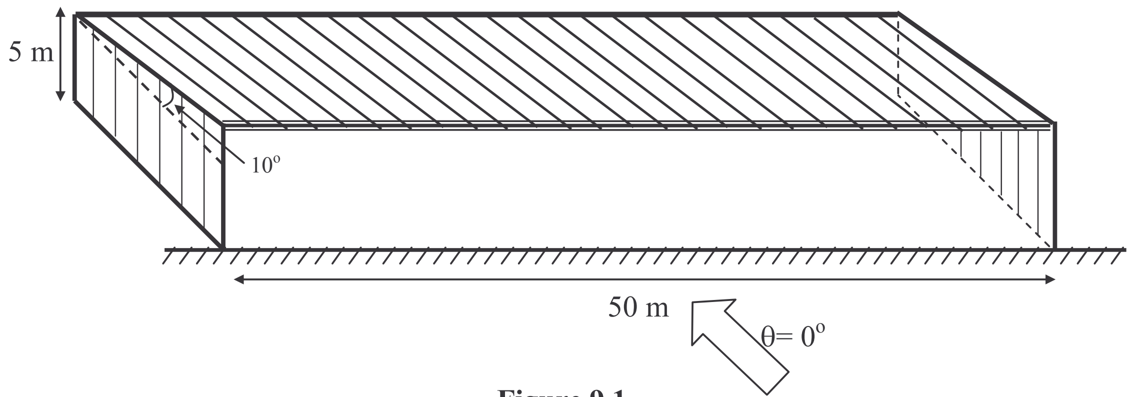


Figure 9.1

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 columns = 5 × 6.76 = 33.81 m^2 =➔ 0.888

Tributary area for Trusses =12.19×5=60.95m^2 =➔ 0.852

Tributary area for Purlins = 1.4 × 5 = 7.0 m^2 =➔ 1.0

Tributary area of short walls for design of wind braces in plan

= 10 x 5 + 0.5 x10x1.76 =58.8 m^2 =➔ 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 6.2.2.2)

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× (47.00)^2 = 1325.4 N/m^2

(IS:875-pt.3, Sec 5.4)

p_d = $p_Z \times K_d \times K_a$ =1.3254×0.9× K_a =1.193× K_a kN/m^2

(IS:875-pt.3, Sec 6.1)

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:

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

(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient: for b/d = 50/10 = 5 > 1

(IS:875-pt.3, Fig. 2, Sec 6.2.2.2)

C_{pi} for 0^o incidence ==➔ +0.8
C_{pi} for 180^o incidence ==➔ -0.4
C_{pi} for 90^o & 270^o incidence ==➔ -0.5

External Pressure Coefficients

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

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

Table 9.1

Portion of Roof	**Wind Incidence Angle**				
	0^o	**45^o**	**90^o**	**135^o**	**180^o**
Windward (widthwise left half)	-1.0	-1.0	-1.0/-0.5	-0.8	-0.4
Leeward (widthwise right half)	-0.5	-0.8	-1.0/-0.5	-1.0	-1.0

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

Angle of Incidence	**0^o**	**90^o**
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 corresponding to 4 are being used.

These will be combined with internal pressure coefficients as given above

C_{pnet} for Walls A or B
= 0.7 – (-0.5) = +1.2, pressure
= -0.5 – (+0.8) = -1.3, suction

C_{pnet} for Walls C or D
= 0.7 – (-0.5) = +1.2, pressure
= -0.6 – (+0.8) = -1.4, 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 × w = 0.15 x 10 = 1.5m 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 × w = .25 × 10 = 2.5m 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 × 0.9 = 1.193 kN/m^2

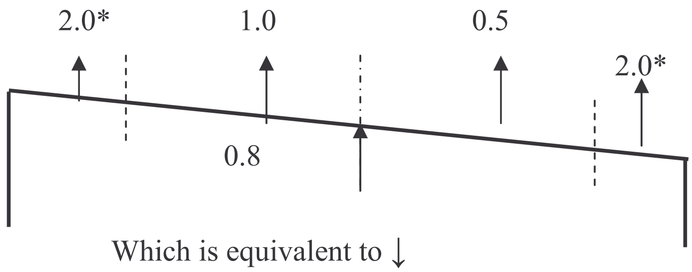
(IS:875-pt.3, Table 7)

Calculations of Force due to Frictional Drag:

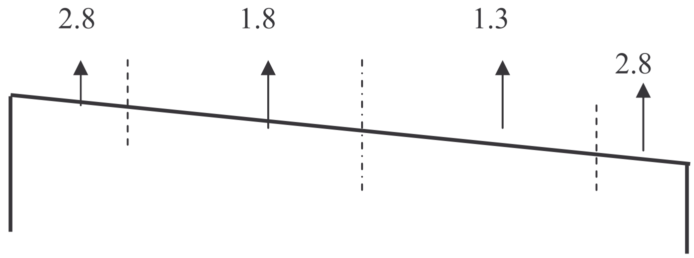
(IS:875-pt.3, Sec 6.3.1)

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. K_a for roof and one long wall is 0.8, as area is more than 100m^2.

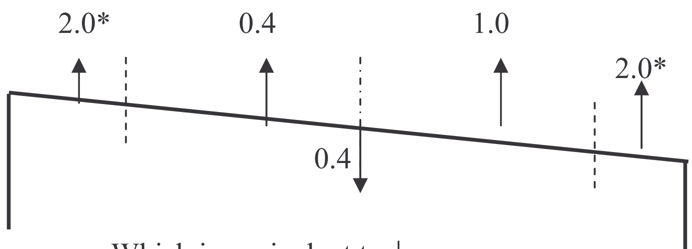
For 0^o wind incidence, *Cpi* = +0.8



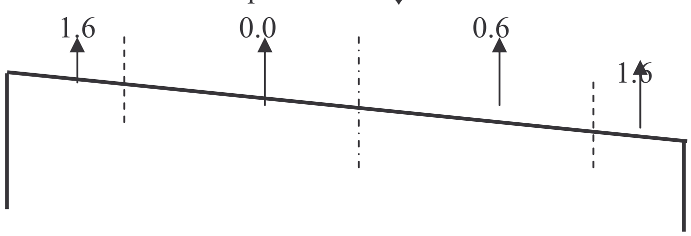
Which is equivalent to ↓



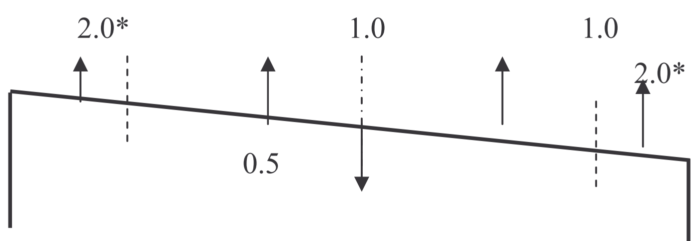
For 180^o wind incidence, Cpi = -0.4



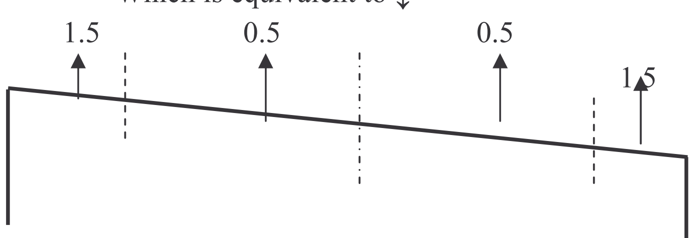
Which is equivalent to ↓



For 90^o / 270^o wind incidence, Cpi = -0.5, up to w/2 from ends



Which is equivalent to ↓



And similarly for other combinations.

*Local Pressure coefficients on roof edges.

Figure 9.2 - Net Roof Pressure Coefficients for different zones and combinations

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.

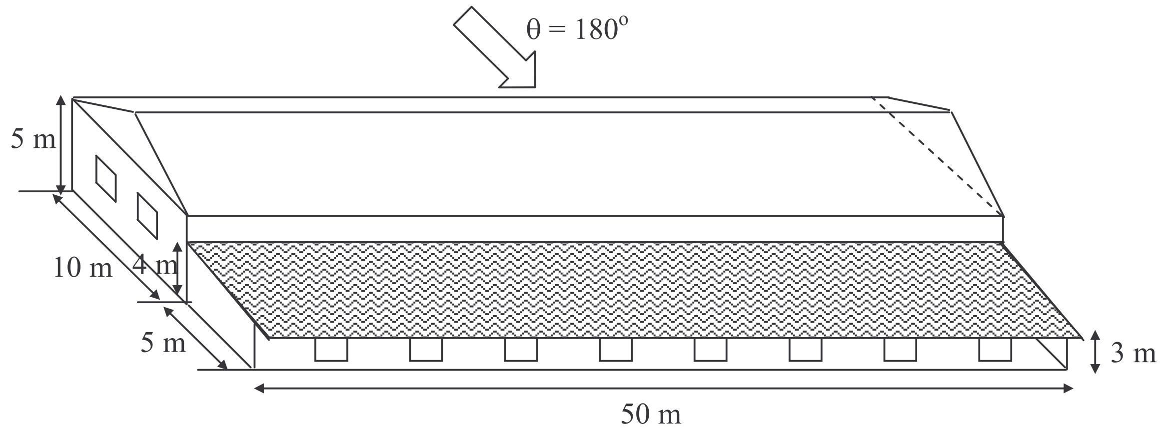


Figure 10.1

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 columns = 5 x 5 = 25 m^2 =→ 0.9

Tributary area for main Trusses = 2 × 5.176 × 5 = 51.76m^2 =→ 0.864

Tributary area for Purlins = 1.4 x 5 = 7.0 m^2=→ 1.0

Tributary area of short walls for design of plan braces,

Main portion = 5 × 10 + 10 × 1.33 × ½ =56.65 m^2 →0.858

Annexe = 5 × 3 + 5 × 1 × ½ = 16.5 m^2 → 0.957

Permeability of the Building: (keeping same as in Ex.4, all openings on the external walls)

Area of all the walls = 5(2×10+50+2×3.5) +1.33 × 10 ×1/2 × 2 +3×50 + 1 × 50 = 598.6 m^2

Area of all the openings =20×1.5×1.5= 45 m^2

% opening area =7.6 %, between 5% and 20%

Hence the building is of Medium permeability.

(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

Design Wind Speed = V_Z = V_b×k_1×k_2×k_3×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× (47.00)^2 = 1325.4 N/m^2

(IS:875:pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a = 1.193\ K_a$

(IS:875-pt.3, Sec 6.1)

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, $F = (C_{pe}\text{-}C_{pi}) * A * p_d$

(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient $C_{pi} = \pm 0.5$

(IS:875-pt.3, Sec 6.2.2.2)

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 6.2.2.1) along-with external pressure coefficient.

External Pressure Coefficients
For Main Building: Using the Table 6 with roof angle 15^o (for '*c*' & '*d*' in table 15)
For *h/w* = 5/10 = 0.5, pressure coefficients are tabulated in Table 10-1.

Table 10.1

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-0.6

For portions '*a*' and '*b*' of the canopy: h_1/h_2 = 5/4 = 1.25 < 1.75

(IS:875-pt.3, Table 15)

For 0^o wind incidence: on '*a*': *Cpe* = -0.45 & on '*b*': *Cpe* = -0.5
For 180^o wind incidence: on '*a*': *Cpe* = -0.4 & on '*b*': *Cpe* = -0.4
For 90^o/ 270^o wind incidence: on '*a*': *Cpe* = -1.0 up to 2.5m from ends and –0.5 thereafter, from IS:875-pt3., Table 7. On '*b*': *Cpe* = - 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 10.2

Angle of Incidence ⟶	0^o	90^o
Wall – A	+0.7	- 0.5
Wall – B	- 0.25	- 0.5
Wall – C	- 0.6	+ 0.7
Wall – D	- 0.6	- 0.1

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_{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

Design Pressure coefficients for Overhangs:
For 0^0 wind incidence, i.e. from overhang side, *Cpi* = +1.25
For other directions, *Cpi* shall be the same as on the adjoining wall, as above, +0.7 or –0.5.
Cpe = -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. 6.2.3.5)

Local pressure coefficients for the design of claddings and fasteners
Refer Table 6 of IS-875 for Roof Angle = 15^0
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_{pnet} = -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 × 0.9 = 1.193 kN/m^2

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.

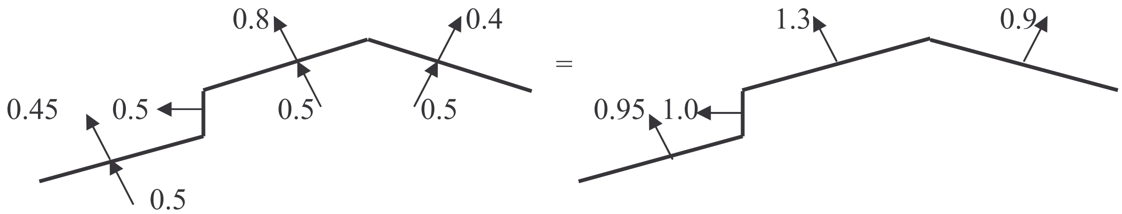
(IS:875-pt.3, Table 6)

Calculations of Force due to Frictional Drag:

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

This will act in the longitudinal direction of the building along the wind. Here $h<b$, therefore, first equation will be used & $C_f' = 0.02$. K_a for roof and walls is 0.8, as area is more than 100m^2.

For End Zone E/G; 0^o wind incidence



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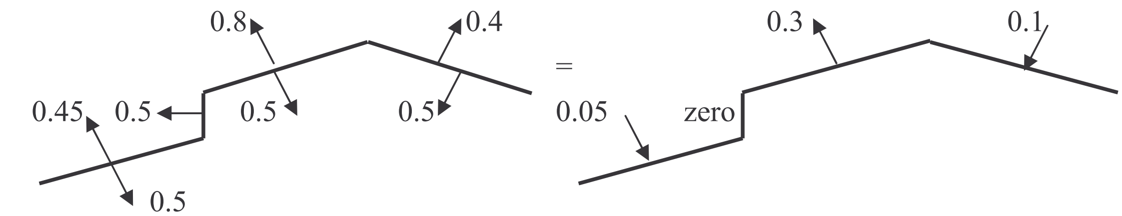
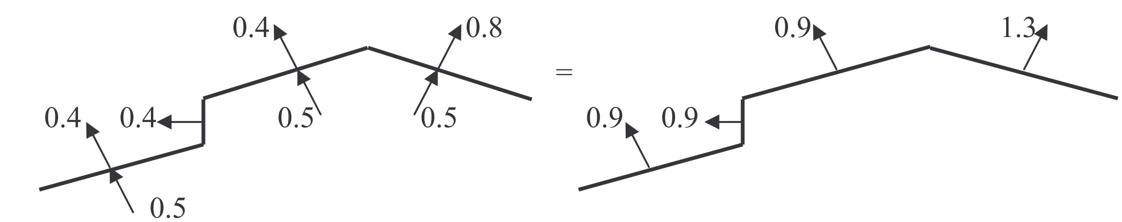


For End Zone E/G; 180^o wind incidence



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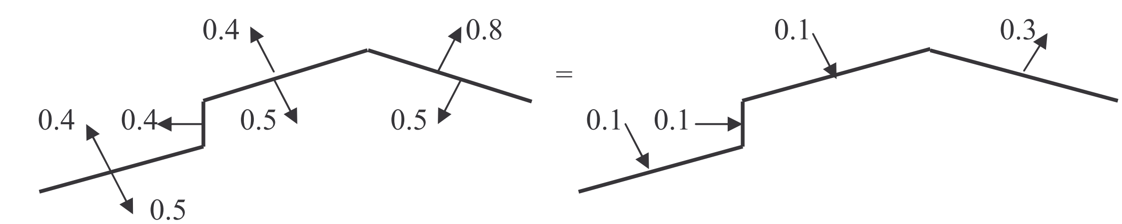
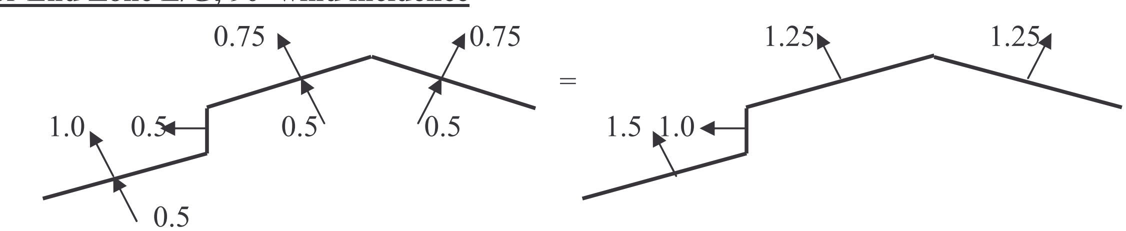
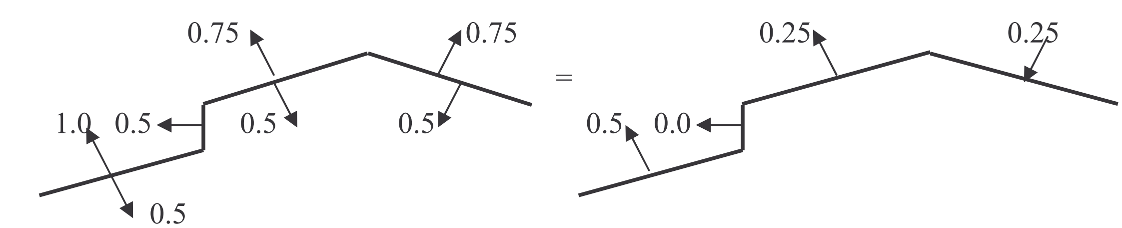


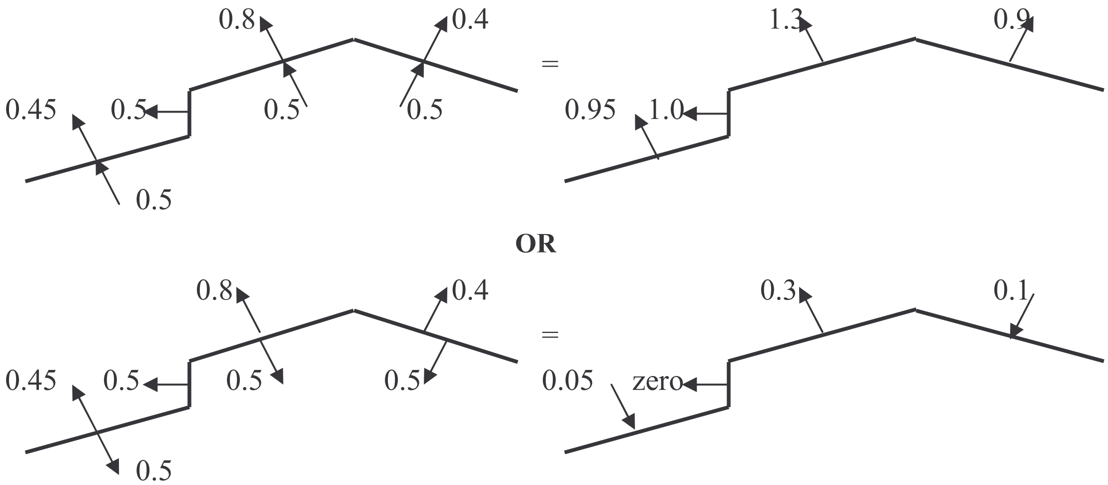
For End Zone E/G; 90^o wind incidence



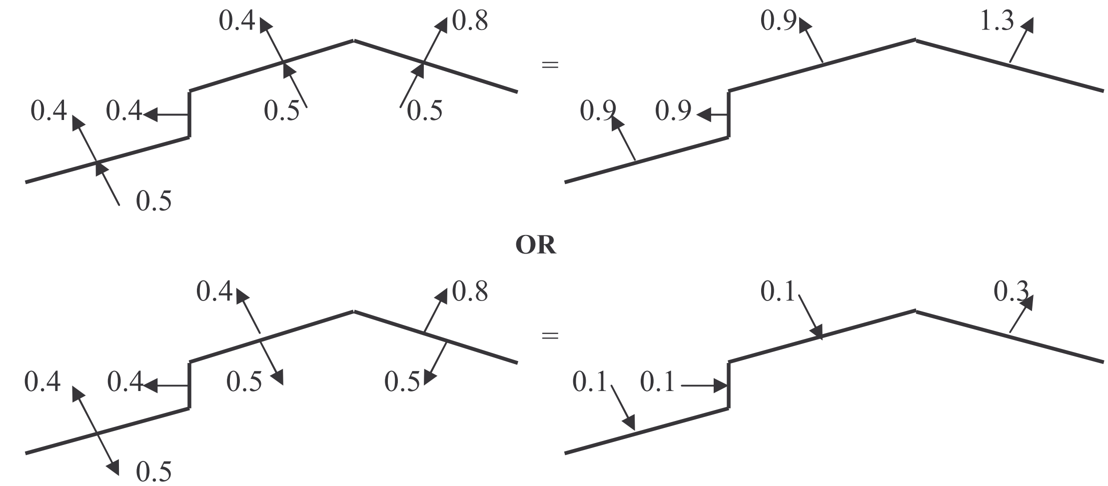
OR



For Mid Zone F/H; 0^o wind incidence



For Mid Zone F/H; 180^o wind incidence



For Mid Zone F/H; 90^o wind incidence

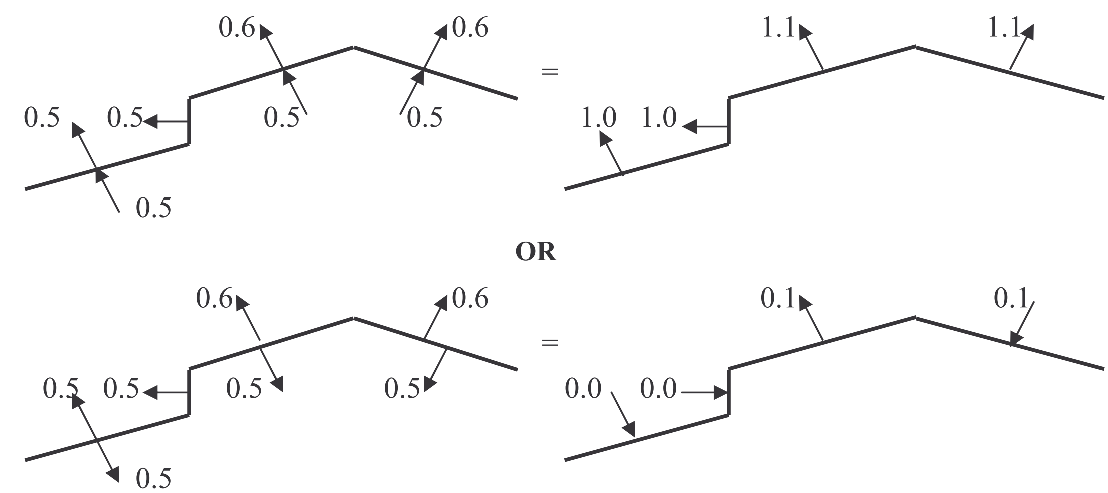


Figure 10.2 - Net Roof Pressure Coefficients for different zones and combinations

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?

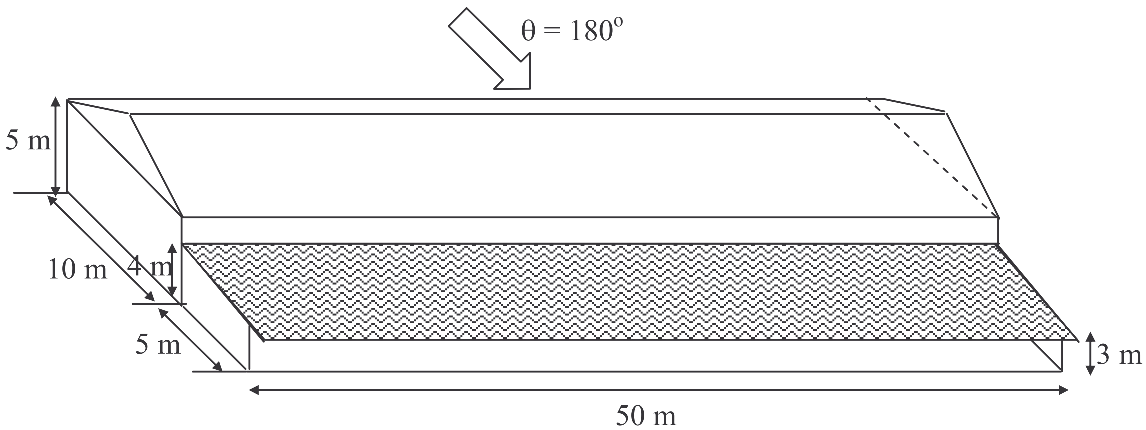


Figure 11.1

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, '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 columns = 5 × 5 = 25 m^2 =➔0.9

Tributary area for main Trusses = 2 × 5.176 × 5 = 51.76m^2 =➔ 0.864

Tributary area for Purlins = 1.4 × 5 = 7.0 m^2 =➔ 1.0

Tributary area of short walls for design of wind braces in plan = 10 × 5 + 6.7 = 56.7m^2 =➔ 0.858

Permeability of the Building: (keeping same as in Ex.4, all openings on the external walls)

Area of all the walls =5× (2×10 + 2×50) + 2 × 6.7 = 613.4 m^2

Area of all the openings = 20×1.5×1.5 = 45 m^2

% opening area =7.3 %, between 5% and 20%

Hence the building is of Medium permeability.

(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$ = 47×1.0×1.0×1.0×1.0 = 47.00 m/s

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

$p_Z = 0.6\ (V_Z)^2 = 0.6\times(47.00)^2 = 1325.4$ N/m^2

(IS:875-pt.3, Sec 5.4)

$p_d = p_Z * K_d * K_a = 1.3254*0.9*K_a = 1.193\ K_a$

(IS:875-pt.3, Sec 6.1)

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:

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

(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient $C_{pi} = \pm 0.5$

(IS:875-pt.3, Sec 6.2.2.2)

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

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

Table 11.1

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-0.6

For portions '*a*', '*b*' and '*e*' of the combined part: $h1/h2$=5/4=1.25<1.75

(IS:875-pt.3, Table 15)

For 0^o wind incidence: on '*a*': Cpe = -0.45 & on '*b*': Cpe = -0.5

For 180^o wind incidence: on '*a*': Cpe = -0.4 & on '*b*': Cpe = -0.4

For 90^o/ 270^o wind incidence: on '*a*': Cpe = -1.0 up to 2.5m from ends and –0.5 thereafter, from IS:875-pt.3, Table 7. On '*b*': Cpe = - 0.5, from IS:875-pt.3, Table 5.

For canopy roof, overhanging from building: Cpi = +1.25, for 0^o wind incidence and –0.5 for other directions which is the max. Pressure on adjoining wall. (refer 6.2.3.5)

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-a-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 11.2

Angle of Incidence	0^o	90^o
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 corresponding 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 = 15^0

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

Therefore Max. $Cpnet$ = -1.2 – (+0.5) = -1.7

For canopy roof, $Cpnet$ = -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_a = 1. Therefore, p_d = 1.3254 × 0.9 = 1.193 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.

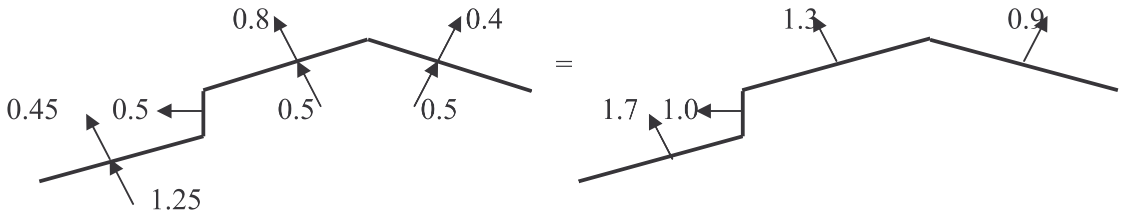
(IS:875-pt.3, Table 6)

Calculations of Force due to Frictional Drag:

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

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 100m^2.

For End Zone E/G; 0° wind incidence



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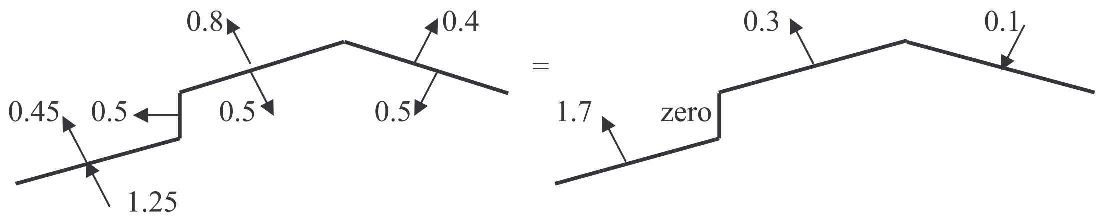
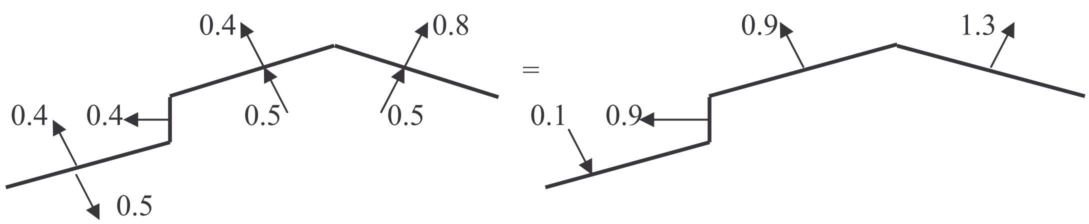
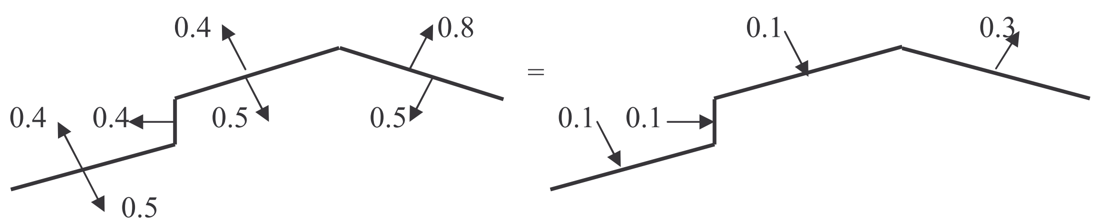


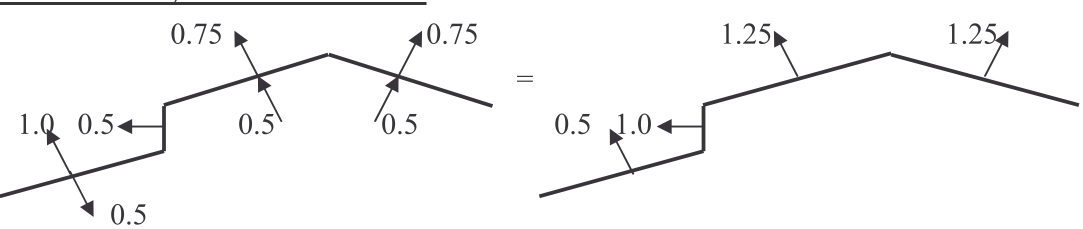
For End Zone E/G; 180° wind incidence



OR



For End Zone E/G; 90° wind incidence



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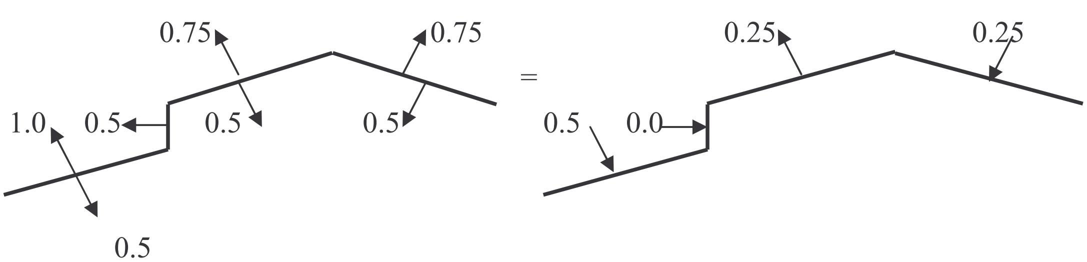
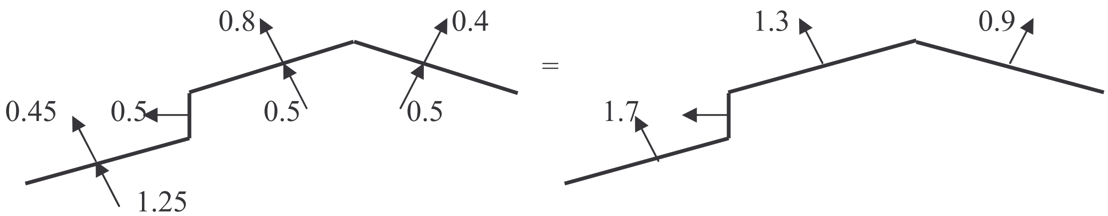


For Mid Zone F/H; 0° wind incidence



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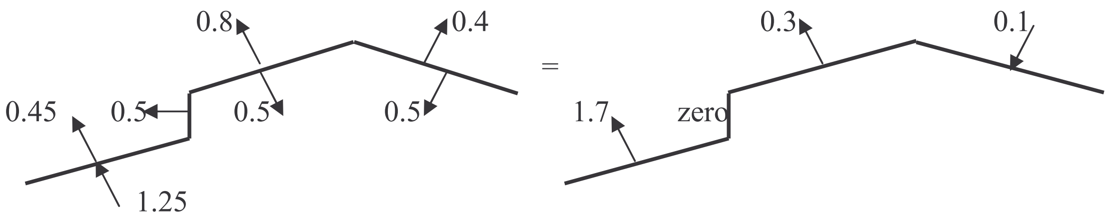
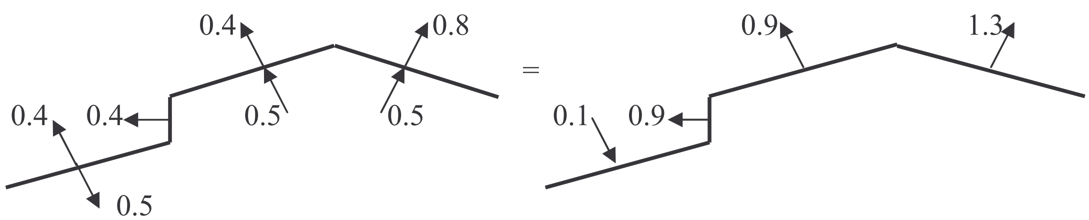
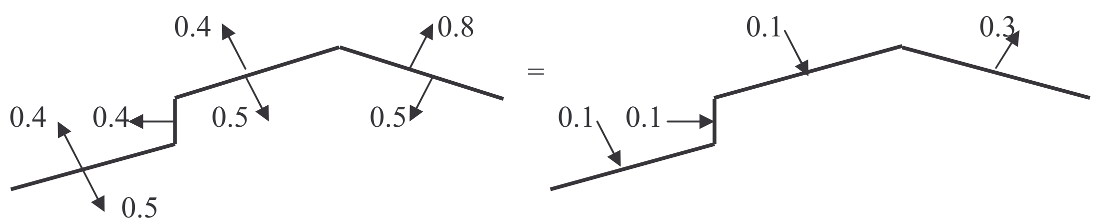


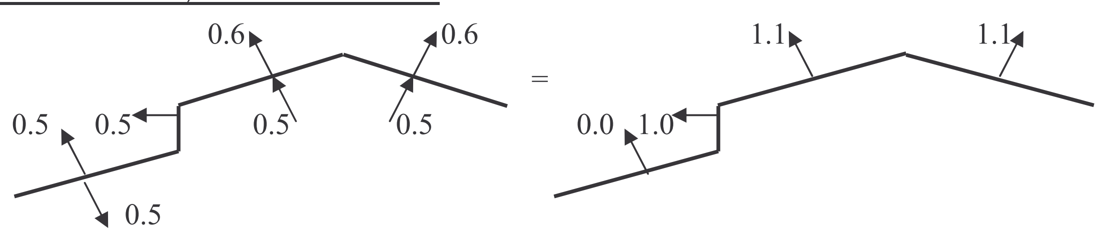
For Mid Zone F/H; 180° wind incidence



OR



For Mid Zone F/H; 90° wind incidence



OR

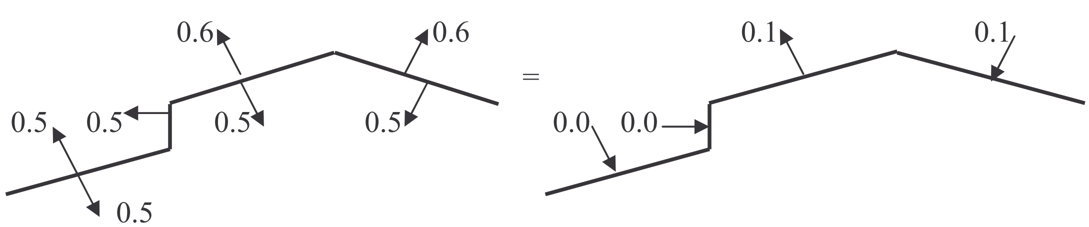


Figure 11.2 - Net Roof Pressure Coefficients for different zones and combinations

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 10m×30m and height 5m, as shown in figure-12.1. The building is situated in outskirts of Jaipur on a hilltop 10m high having upwind and downwind slopes of 18^0 and 10^0, respectively. The building has 16 openings of 1.5m ×1.5m size. The roof is of GC sheeting & the roof angle α is 15^0. Calculate also the local wind pressures on roof & wall cladding. The columns and trusses are at 5m c/c longitudinally, purlins are at 1.4m c/c and columns at Gable ends are at 5m c/c.

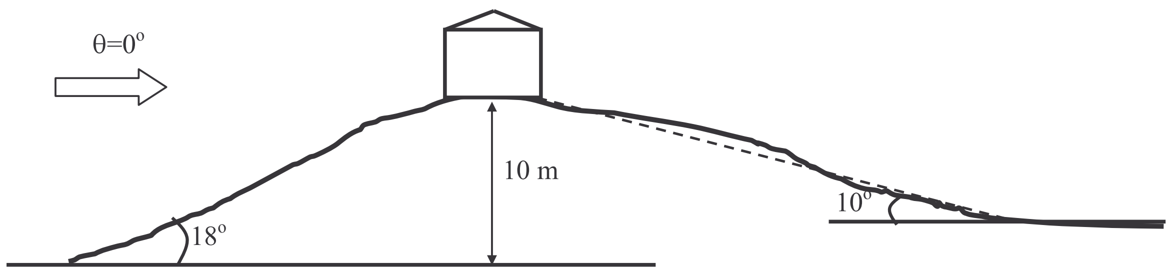


Figure 12.1

Solution:

Wind Data:

1. Wind Zone: Zone IV (V_b= 47m/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 6.1.1)

Area Averaging factor K_a

(IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for columns = 5 ×5 = 25 m^2 =➔ 0.9

Tributary area for Trusses = 2 × 5.176 × 5 = 51.76m^2 =➔ 0.864

Tributary area for Purlins = 1.4 × 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× (2×10 + 2×30) + 2 × 6.7 = 413.4m^2

Area of all the openings = 16×1.5×1.5 =36 m^2

% opening area = 8.71 %, between 5% and 20%

Hence the building is of medium permeability.

(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

Design Wind Speed = V_Z = $V_b \times k_1 \times k_2 \times k_3 \times k_4$ = 47×1.0×1.0×1.198×1.0 = 56.3 m/s

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

$p_Z = 0.6\,(V_Z)^2 = 0.6\times(56.3)^2 = 1902.22$ N/m^2

(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.9022\times0.9\times K_a = 1.712\,K_a$

(IS:875-pt.3, Sec 6.1)

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 6.2.1)

Internal Pressure Coefficient C_{pi} = ± 0.5

(IS:875-pt.3, Sec 6.2.2.2)

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 6.2.2.1) along-with external pressure coefficient

External Pressure Coefficients

Using the Table 6 with roof angle 15^o

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 0^o	90^o
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-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^o	90^o
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 corresponding to 4 are being used.

These will be combined with internal pressure coefficients as earlier, equal to C_{pi} = ± 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 = 15^0

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

Local C_{pe} for corners of walls: -1.0

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 = 1.9022×0.9 = 1.712 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.

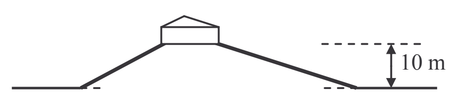
(IS:875-pt.3, Table 6)

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. Here $h<b$, therefore, first equation will be used & C_f' = 0.02. K_a for roof and walls is 0.8, as area is more than 100m^2.

CALCULATIONS FOR TOPOGRAPHY FACTOR k_3



Wind from left:

H = 10 m, z = 10 m, L = 10/tan 18^o = 30.777 k_3 = $1 + C.s$

For θ = 18^o, C = 0.36 (C-2)

Factor 's' is obtained from C-2.1 and figure 17, for crest position

$Le = z/0.3$ = 10/0.3, H/Le = 10/10/0.3 = 0.3 =====➔ s = 0.55

k_3 = 1 + 0.36 × 0.55 = 1.198

Wind from right:

H = 10 m, z = 10 m, L = 10/tan 10^o = 56.7 m

C = 1.2 (z/L) = 1.2 (10/56.7) = 0.21

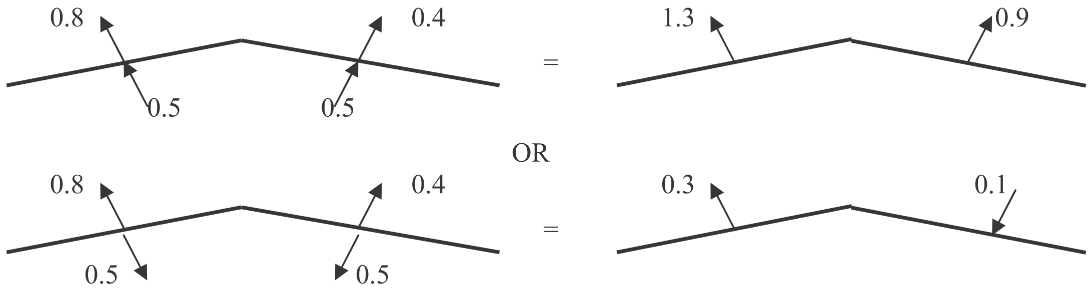
For θ = 10^o, $Le = L$ = 56.7 m

H/Le = 10/56.7 = 0.176 ====➔ s = 0.7

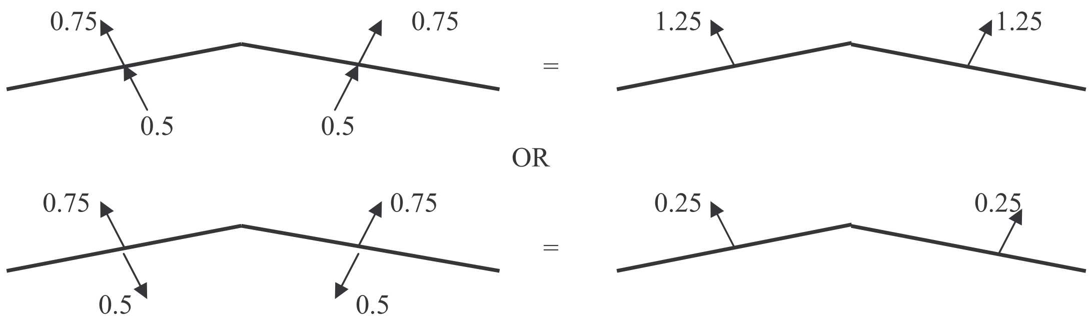
k_3 = 1 + 0.21 × 0.7 = 1.147

Using k_3 = 1.198, being the critical one.

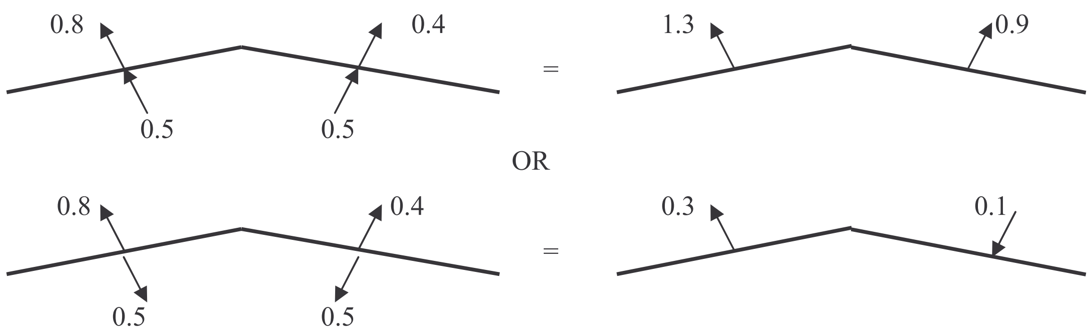
For End Zone E/G; 0° wind incidence



For End Zone E/G; 90° wind incidence



For Mid Zone F/H; 0° wind incidence



For Mid Zone F/H; 90° wind incidence

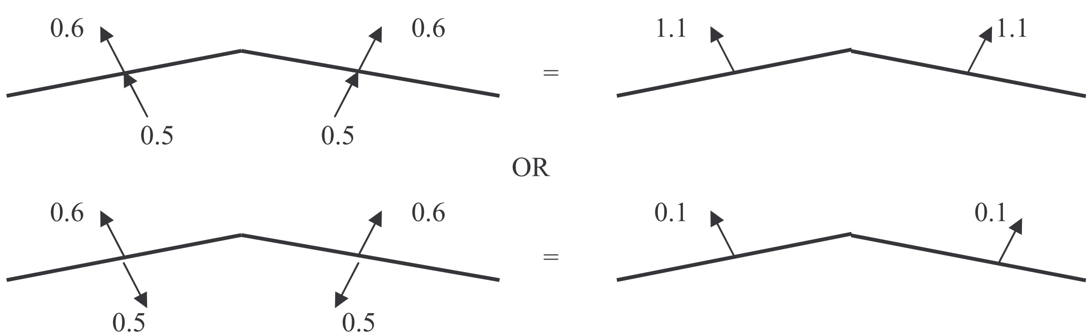


Figure 12.2 - Net Roof Pressure Coefficients for different zones and combinations

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 Example12 is situated on a hill having upwind and downwind slopes of 15^0 and 01^0, respectively as shown in figure 13.1?

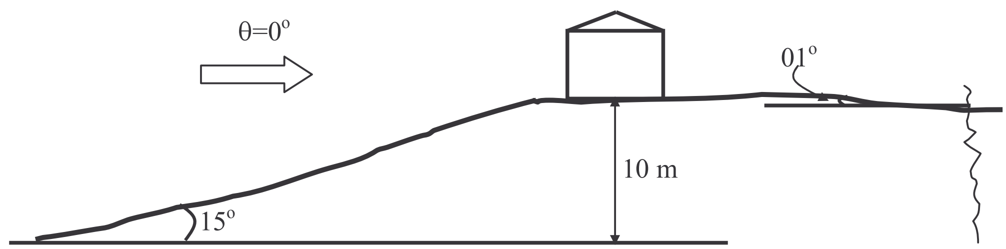


Figure 13.1

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')

* : 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 6.1.1)

Area Averaging factor K_a
(IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for columns = 5 x 5 = 25 m^2 =➔ 0.9

Tributary area for Trusses = 2 x 5.176 x 5 = 51.76m^2 =➔ 0.864

Tributary area for Purlins = 1.4 x 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× (2×10 + 2×30) + 2 × 6.7 = 413.4m^2

Area of all the openings = 16×1.5×1.5 =36 m^2

% opening area =8.71 %, between 5% and 20%

Hence the building is of medium permeability.
(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

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

$p_Z = 0.6\,(V_Z)^2 = 0.6\times(56.071)^2 = 1886.37$ N/m^2
(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.886\times0.9\times K_a = 1.700\ K_a$
(IS:875-pt.3, Sec 6.1)

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 6.2.1)

Internal Pressure Coefficient C_{pi} = ± 0.5
(IS:875-pt.3, Sec 6.2.2.2)

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

Portion of roof	Wind Incidence Angle 0^o	90^o
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-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 13-2.

Table 13.2

Angle of Incidence	0^o	90^o
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 corresponding 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 = 15^0
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
Local C_{pe} for corners of walls: -1.0

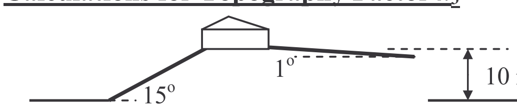
Max. local C_{pnet} for roof at the edges and the ridge = -1.2 – (+0.5) = -1.7
Likewise at the wall edges = -1.0 – (+0.5) = -1.5
However, for the use of the local pressure coefficients, the design pressure p_d will be computed with K_a = 1. Therefore, p_d = 1.886*0.9 = 1.700 kN/m^2

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. Here $h<b$, therefore, first equation will be used & C_f' = 0.02. K_a for roof and walls is 0.8, as area is more than 100m^2.

Calculations for Topography Factor k_3



Wind from left:
Note: Here H = 10m, z = 10m & L = 10/tan 15^0 = 37.32

$k_3 = 1 + C.\ s$

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 16 for crest position

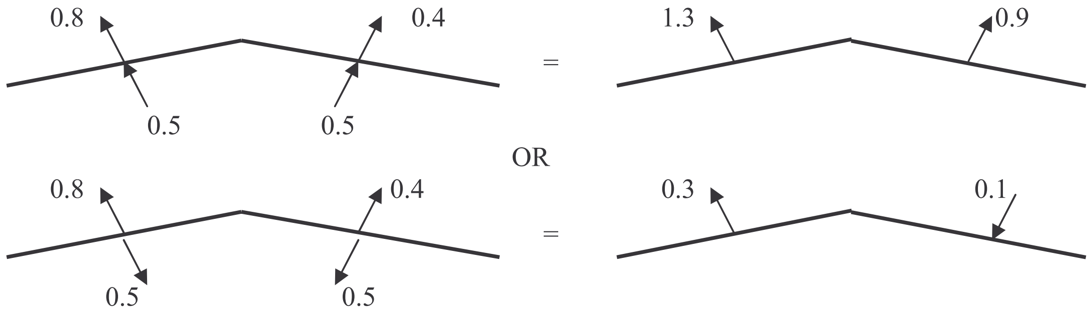
H/Le = (10/37.32) = 0.268 ====➔ s = 0.6

Therefore k_3 = 1 + 0.321 ×0.6 = 1.193, wind from left &

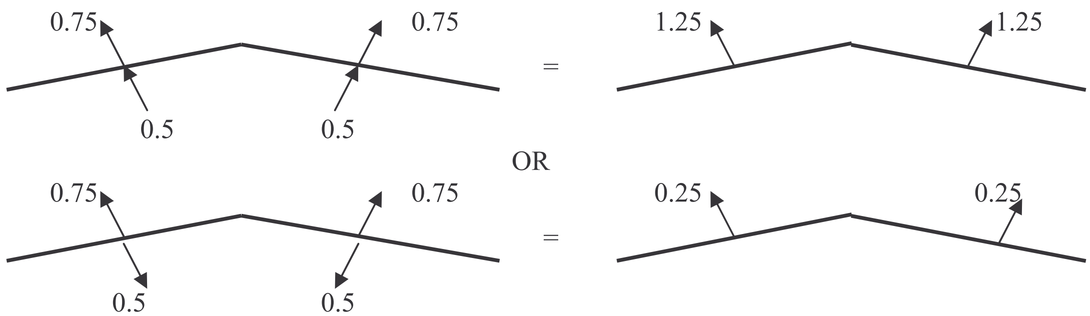
1 + 0.00 × 0.6 = 1.000, wind from right

Using k_3 = 1.193, being the critical one.

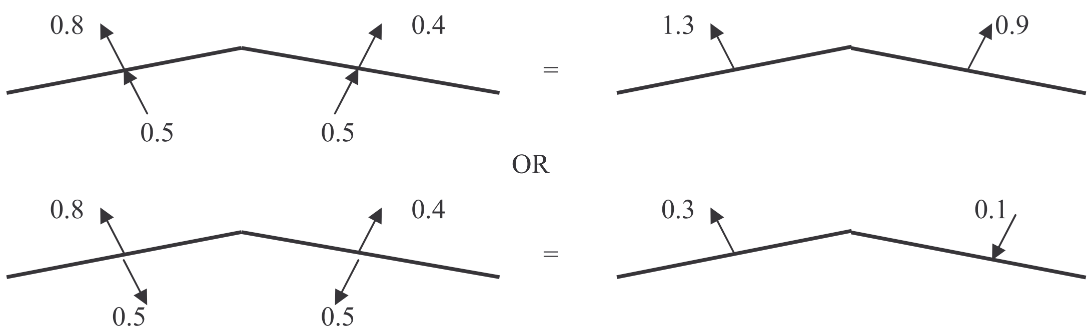
For End Zone E/G; 0^o wind incidence



For End Zone E/G; 90^o wind incidence



For Mid Zone F/H; 0^o wind incidence



For Mid Zone F/H; 90^o wind incidence

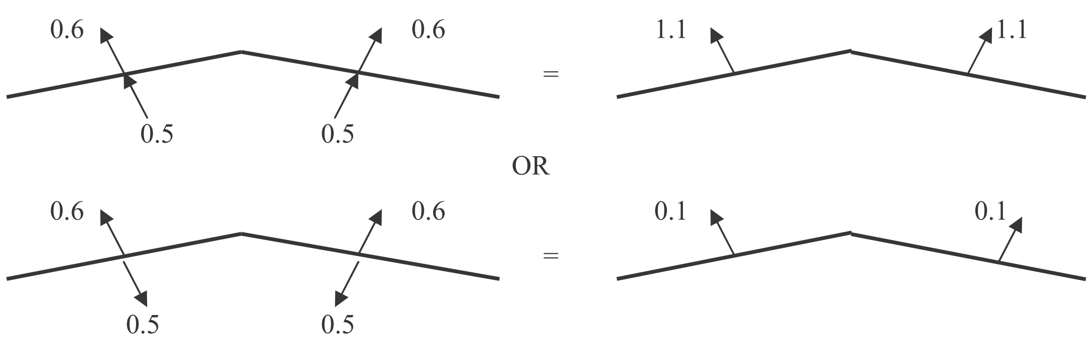


Figure 13.2 - Net Roof Pressure Coefficients for different zones and combinations

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 Example12 is situated in the middle of the upwind slope of a hill 50m high, upwind and downwind slopes being 18^0 and 10^0 respectively, as shown in figure 14.1?

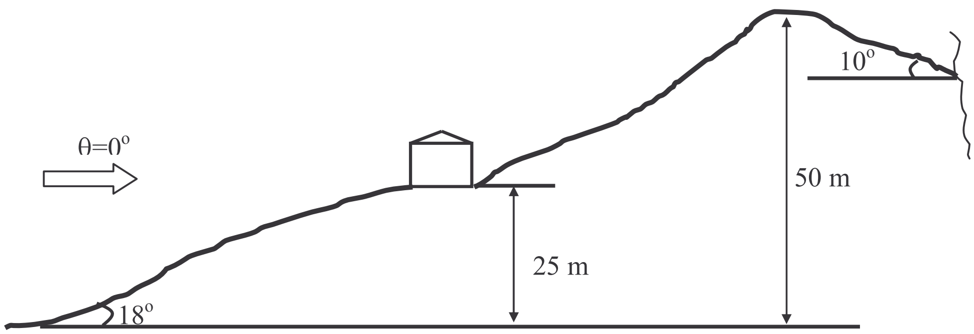


Figure 14.1

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' = 1.127*
(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 6.1.1)
Area Averaging factor K_a= 0.867, for short walls
= 0.80, for long walls & roofs
(IS:875-pt.3, Sec 6.1.2, Table-4)
Tributary area for columns = 5 x 5 = 25 m^2 =➔ 0.9
Tributary area for Trusses = 2 x 5.176 x 5 = 51.76m^2 =➔ 0.864
Tributary area for Purlins = 1.4 x 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× (2×10 + 2×30) + 2 × 6.7 = 413.4m^2
Area of all the openings = 16×1.5×1.5 =36 m^2
% opening area =8.71 %, between 5% and 20%
Hence the building is of medium permeability.
(IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

Design Wind Speed = V_Z = $V_b \times k_1 \times k_2 \times k_3 \times k_4$ = 47×1.0×1.0×1.127×1.0 = 52.97 m/s
(IS:875-pt.3, Sec 5.3)
$p_Z = 0.6\ (V_Z)^2 = 0.6\times(52.97)^2 = 1683.5\ N/m^2$
(IS:875-pt.3, Sec 5.4)
$p_d = p_Z \times K_d \times K_a$ = 1.6835×0.9×0.867 = 1.313 kN/m^2 (short wall)
= 1.6835×0.9×0.8 = 1.212 kN/m^2
(long wall & roof)
(IS:875-pt.3, Sec 6.1)

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 6.2.1)
Internal Pressure Coefficient C_{pi} = ± 0.5
(IS:875-pt.3, Sec 6.2.2.2)
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 6.2.2.1) 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 14-1. (refer figure Table 6 of code)

Table 14.1

Portion of roof	Wind Incidence Angle 0°	90°
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-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 14-2.

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 14-2.

Table 14.2

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 corresponding to 4 are being used.

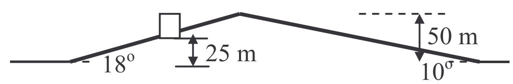
These will be combined with internal pressure coefficients as earlier, equal to $C_{pi} = \pm 0.5$

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 = 15^0
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
Max. local C_{pnet} 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 ×0.9 = 1.515 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 for $\theta = 18^0$, C = 0.36 and {from C-2}

factor 's' is obtained from C-2.1 and IS:875-pt.3, Figure 17 for upwind position

H/Le = (25/166.67) = 0.15 & X/Le = -(77/166.67) = -0.462 ==➔ s = 0.3

$k_3 = 1 + C.\ s$ = 1 + 0.36 ×.3 = 1.108

For wind from right to left: Z = 50m H = 25m

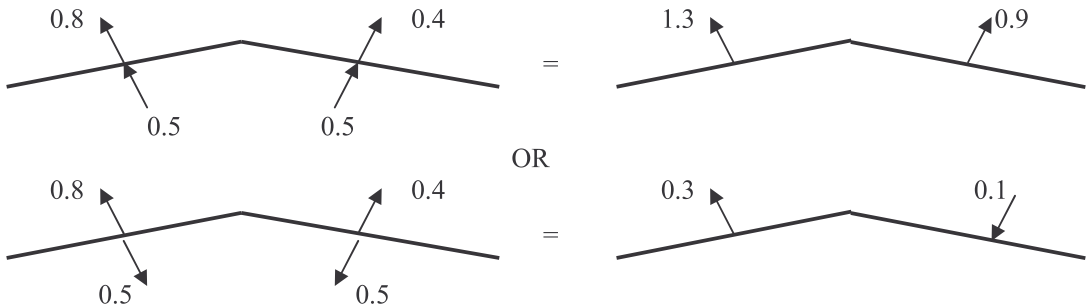
$Le = L$ = 50/tan 10^0 = 283.56m

for $\theta = 10^0$, $C = 1.2(Z/L)$ = 1.2 (50/283.56) = 0.2116

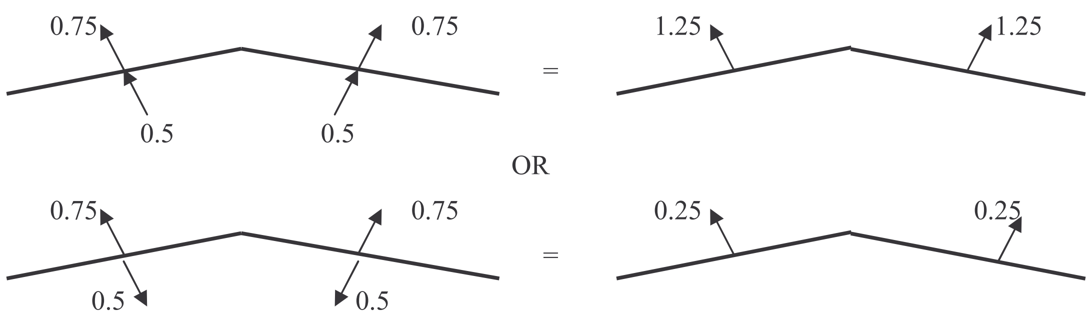
factor 's' is obtained from C-2.1 and IS:875-pt.3, Figure 17 for downwind position
H/Le = (25/283.56) = 0.09 & X/Le = (77/283.56) = 0.271 ==➔ s = 0.6
Therefore k_3 = 1 + 0.2116 ×0.6 = 1.127

Using k_3 = 1.127, being the critical one.

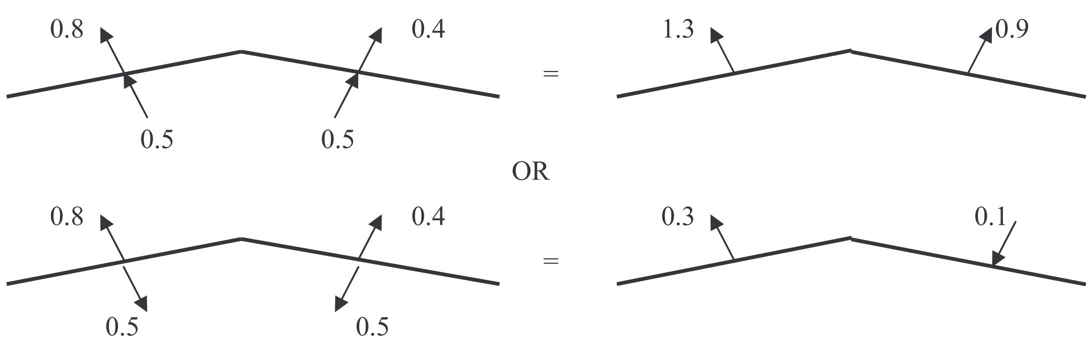
For End Zone E/G; 0° wind incidence



For End Zone E/G; 90° wind incidence



For Mid Zone F/H; 0° wind incidence



For Mid Zone F/H; 90° wind incidence

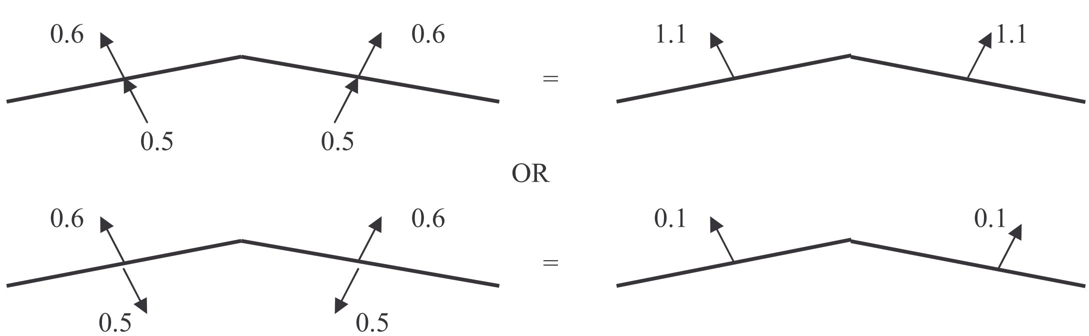


Figure 14.2 -Net Roof Pressure Coefficients for different zones and combinations

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 α is 15^0. 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.

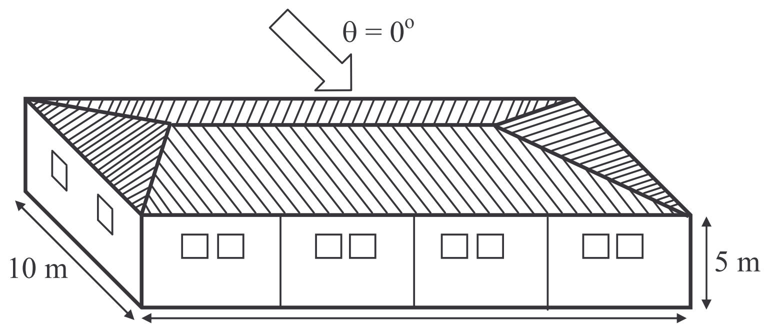


Figure 15.1

Solution:

Wind Data:

1. Wind Zone: Zone IV (V_b= 47m/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 6.1.1)

Area Averaging factor K_a: (IS:875-pt.3, Sec 6.1.2, Table-4)

Tributary area for columns = 5 ×5 = 25 m^2 ====→ 0.9

Tributary area for Trusses = 5.176 ×5 × 2 = 51.76 m^2 =====→ 0.864

Tributary area for Purlins = 1.4 ×5 = 7.0 m^2 ======→ 1.0

Tributary area of short walls = 10 ×5 = 50 m^2 =====→ 0.867 for design of wind braces in plan

Permeability of the Building:

Area of all the walls =5× (2×10 + 2×20) = 300 m^2

Area of all the openings = 20×1.5×1.5 = 45 m^2

% opening area = 15 %, between 5% and 20%

Hence the building is of medium permeability. (IS:875-pt.3, Sec 6.2.2.2)

Design Wind Pressure

Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$

=47×1.0×1.0×1.0×1.0 = 47.00 m/s (IS:875-pt.3, Sec 5.3)

$p_Z = 0.6\,(V_Z)^2 = 0.6\times(47.00)^2 = 1325.4$ N/m^2 (IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.3254\times0.9\times K_a = 1.193\, K_a$ (IS:875-pt.3, Sec 6.1)

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 6.2.1)

Internal Pressure Coefficient C_{pi} = ± 0.5 (IS:875-pt.3, Sec 6.2.2.2)

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 6.2.2.1) along-with external pressure coefficient

External Pressure Coefficients

Using the Table 6 with roof angle 15^o

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

Table 15.1

Portion of roof*	Wind Incidence Angle 0^o	90^o
E	-0.8	-0.75
F	-0.8	-0.6
G	-0.4	-0.75
H	-0.4	-0.6
Hipped slope, M,N	-0.75**	-0.8**(windward) -0.4** (leeward)

* See Figure 15.2.

** These values may be reduced by 20% as per IS:875-pt.3, Sec 6.2.3.2, 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 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

Angle of Incidence	0^o	90^o
Wall – A	+0.7	- 0.5
Wall – B	- 0.25	- 0.5
Wall – C	- 0.6	+ 0.7
Wall – D	- 0.6	- 0.1

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 = 15^0

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 (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 = 1.3254×0.9 = 1.193 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

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. K_a for roof and walls is 0.8, as area is more than 100m^2.

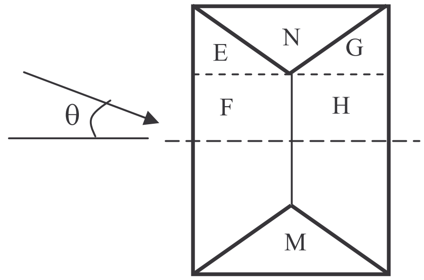
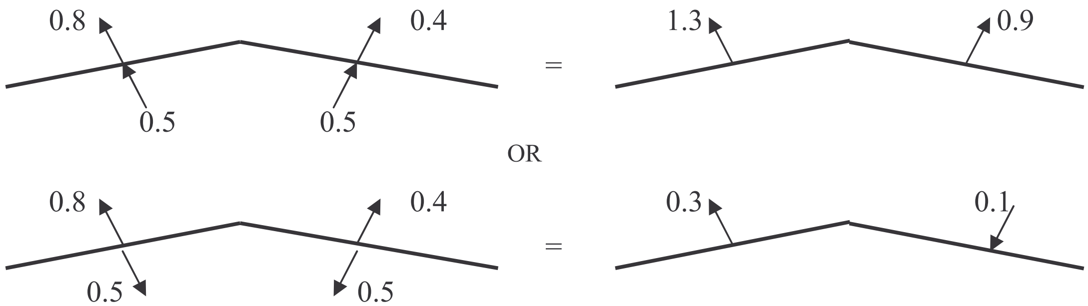
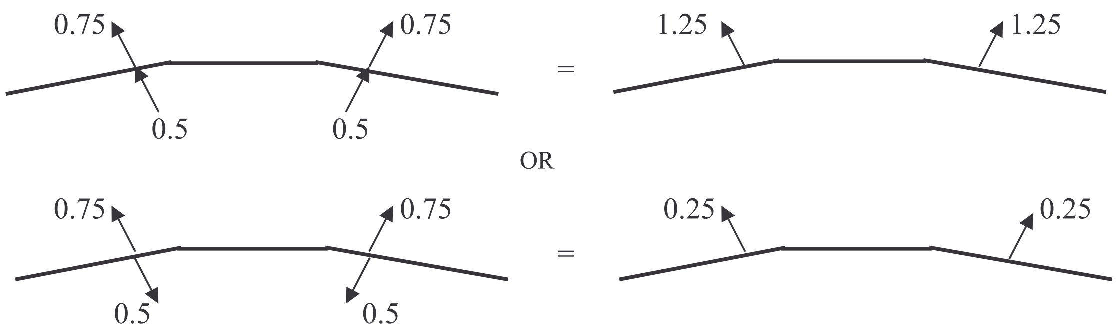


Figure 15.2 - Plan of the building. M,N are hipped slopes

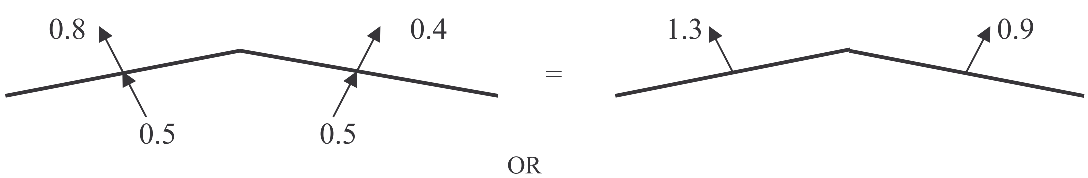
For End Zone E/G; 0^o wind incidence

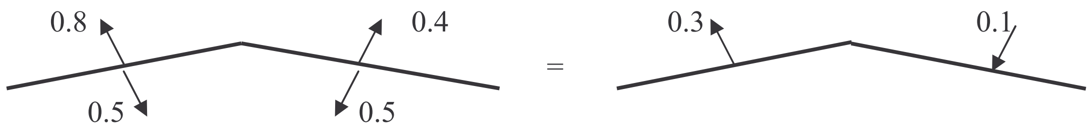


For End Zone E/G; 90^o wind incidence

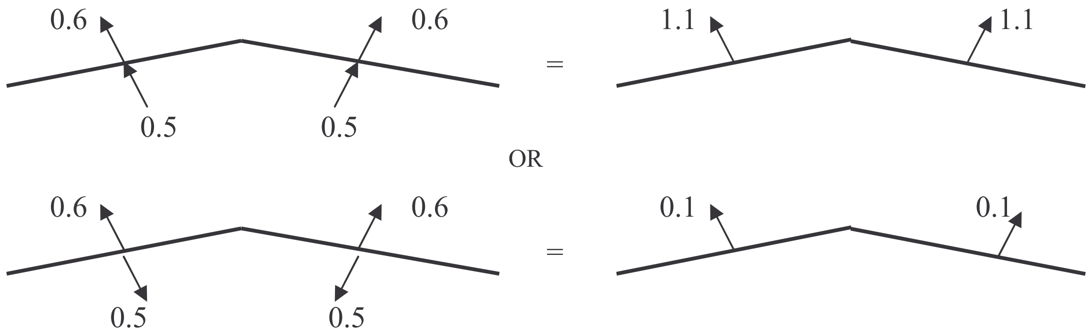


For Mid Zone F/H; 0^o wind incidence

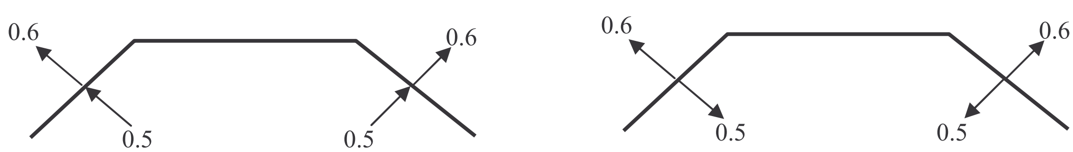




For Mid Zone F/H; 90^o wind incidence



For hipped slopes M,N : 0^o wind incidence



For hipped slopes M,N : 90^o wind incidence

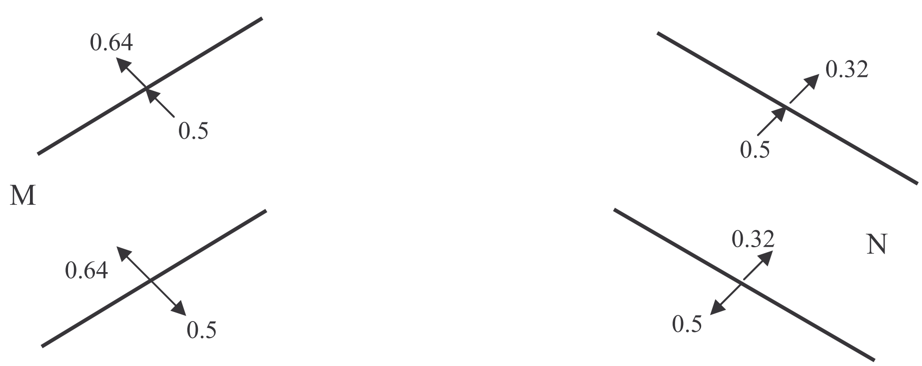


Figure 15.3- Net Roof Pressure Coefficients for different zones and combinations

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 α is 15^0. Assume that full obstruction can occur on one side i.e. the solidity ratio ϕ may vary from 0 to 1.0.

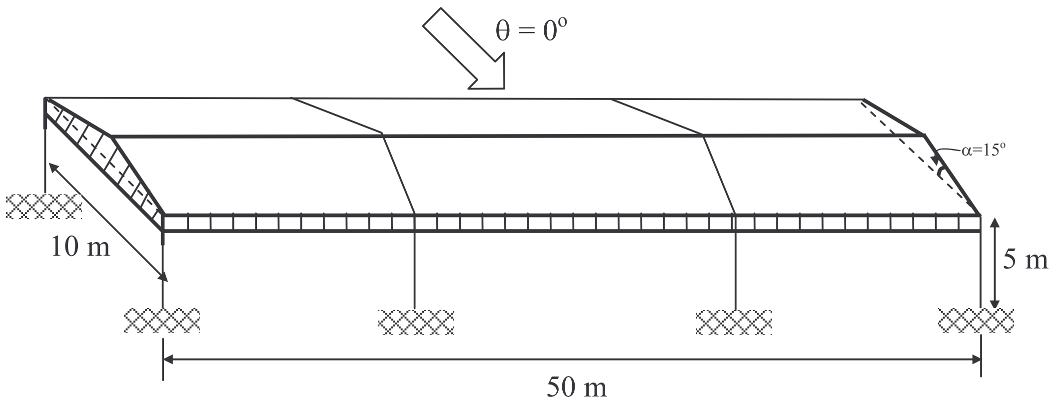


Figure 16.1

Solution:

Wind Data:

1. Wind Zone: Zone IV (V_b= 47m/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_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' = 0.80*, for Roof
(IS:875-pt.3, Sec 6.1.2, 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 $= V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00$ m/s
(IS:875.pt.3, Sec 5.3)

$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m^2
(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times K_a = (1.193 \times K_a)$ kN/m^2
(IS:875.pt.3, Sec 6.1)

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:

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

(IS:875-pt.3, Sec 6.2.1)

Net Pressure Coefficients
(IS:875-pt.3, Table 9)

Roof angle +15^o: $h/w = 0.5$ & $L/w = 5$

Max. +ve roof pressure, for $\phi = 0$ ➔ +0.4

Max. -ve roof pressure, for $\phi = 0$ ➔ -0.8

Max. +ve roof pressure, for $\phi = 1$ ➔ +0.4

Max. -ve roof pressure, for $\phi = 1$ ➔ -1.2

(IS:875-pt.3, Sec. 6.2.3.3)

Force coefficient on facia =→ +1.3

Calculating solidity ratio:

(IS:875-pt.3, Sec 6.2.3.3)

ϕ = area of obstruction perpendicular to wind / min. area under canopy perpendicular to wind

Now depending on position as upwind or downwind, effect is to be considered. Only for downwind obstruction ϕ 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.

Table 16.1 - Local pressure coefficients for the design of claddings and fasteners

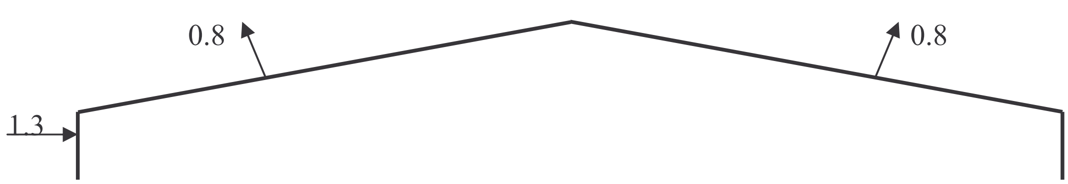
	mid zone	**gable ends**	**eaves zone**	**ridges**
ϕ = 0, +ve	+0.9	+1.9	+1.4	+0.4
ϕ = 0, -ve	-0.9	-1.7	-1.4	-1.8
ϕ = 1, +ve	+0.9	+1.9	+1.4	+0.4
ϕ = 1, -ve	-1.5	-2.2	-1.9	-2.8

Therefore, the fasteners shall be designed for increased force as per *Cpnet* = –2.8 to –1.7, according to ϕ. The spacing in all end zones, extending upto L/10 = 5m at gable ends and w/10 = 1 m at eaves and ridges shall be reduced appropriately.

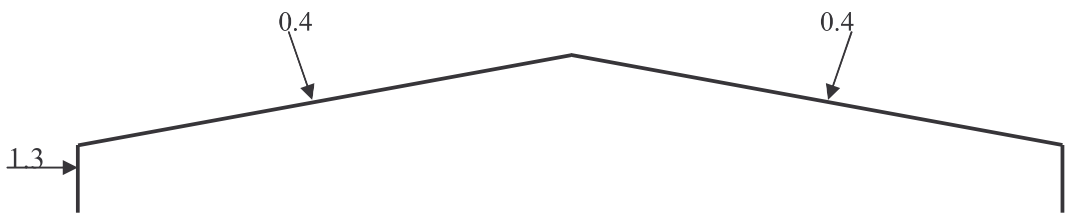
However, for the use of the local pressure coefficients, the design pressure p_d will be computed with K_a = 1. Therefore, $p_d = 1.3254 \times 0.9 = 1.193$ kN/m^2

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

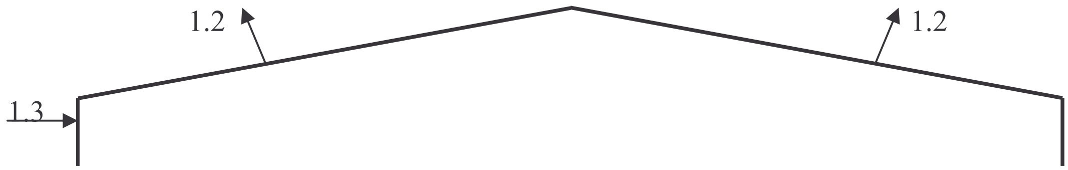
Both slopes at –ve pressure coefficients and ϕ = 0 (case 1)



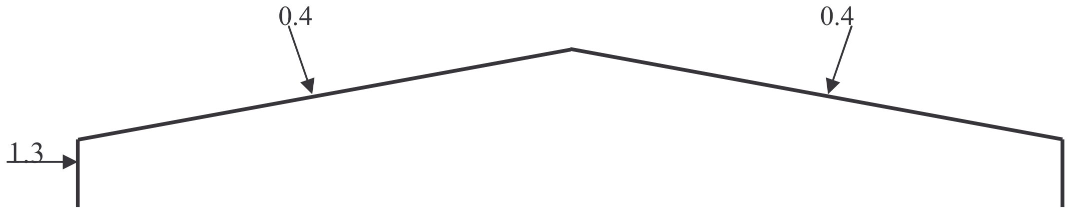
Both slopes at +ve pressure coefficients and ϕ = 0 (case 2)



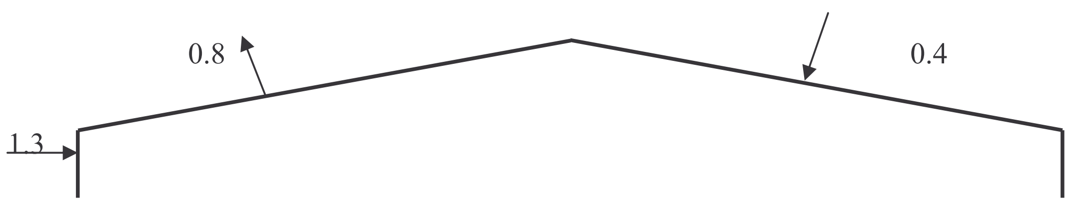
Both slopes at –ve pressure coefficients and ϕ = 1 (case 3)



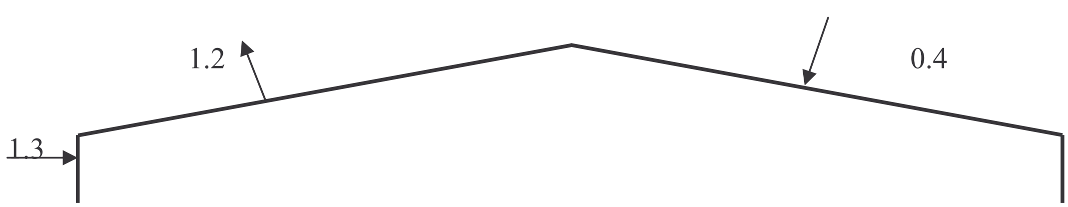
Both slopes at +ve pressure coefficients and ϕ = 1 (case 4)



One slope at –ve and other at +ve pressure coefficient and ϕ = 0 (case 5)



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



Case 1, 2 and 5 need not be analysed.

Figure 16.2-Net Roof Pressure Coefficients for different zones and combinations

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 Example16 is bent up, as in figure 17.1. The roof angle α is 15^o 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.

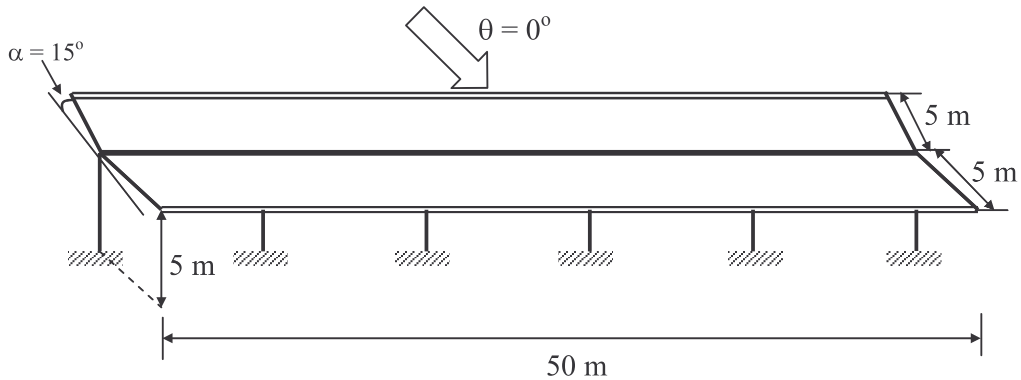


Figure 17.1

Solution:

Wind Data:

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

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' = 0.80*, for Roof (IS:875-pt.3, Sec 6.1.2, 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 $= V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 = 47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00$ m/s (IS:875.pt.3, Sec 5.3)

$p_Z = 0.6\ (V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m^2 (IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times 0.8 = (0.9544)$ kN/m^2, for roof (IS:875.pt.3, Sec 6.1)

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 6.2.1)

Net Pressure Coefficients

With roof angle -15^o : $h/w = 0.5$ & $L/w = 5$ (IS:875-pt.3, Table 9)

Max. +ve roof pressure, for $\phi = 0$ =➔ +0.5

Max. -ve roof pressure, for $\phi = 0$ =➔ -0.6

Max. +ve roof pressure, for $\phi = 0.82$* =➔+0.5

Max. -ve roof pressure, for $\phi = 0.82$* =➔-0.76

*Calculating solidity ratio, ϕ: (IS:875-pt.3, Sec 6.2.3.3)

= area of obstruction perpendicular to wind / min. area under canopy perpendicular to wind

= {3/(5-5 × tan 15^o) = 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 17.1

	mid zone	**gable ends**	**eaves zone**	**ridges**
ϕ = 0, +ve	+0.6	+1.5	+0.7	+1.4
ϕ = 0, -ve	-0.8	-1.3	-1.6	-0.6
ϕ = .82, +ve	+0.6	+1.5	+0.7	+1.4
ϕ = .82, -ve	-1.05	-1.63	-1.85	-1.09

Therefore, the fasteners shall be designed for C_{pnet} = –1.85. The spacing in all end zones, extending up to $L/10$ = 5m at gable ends and $w/10$ = 1m 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_a = 1. Therefore, $p_d = 1.3254 \times 0.9 = 1.193$ kN/m^2

Both slopes at –ve pressure coefficients and ϕ = 0 (case 1)

Both slopes at +ve pressure coefficients and ϕ = 0 (case 2)

Both slopes at –ve pressure coefficients and ϕ = 0.82 (case 3)

Both slopes at +ve pressure coefficients and ϕ = 0.82 (case 4)

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

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

Case 1, 2 and 5 need not be analysed.

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

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×20m and height of 3m up to lower eaves. The canopy is located at Agra (UP) near the city center. The roof angle α is 10^0. See figure 18.1.

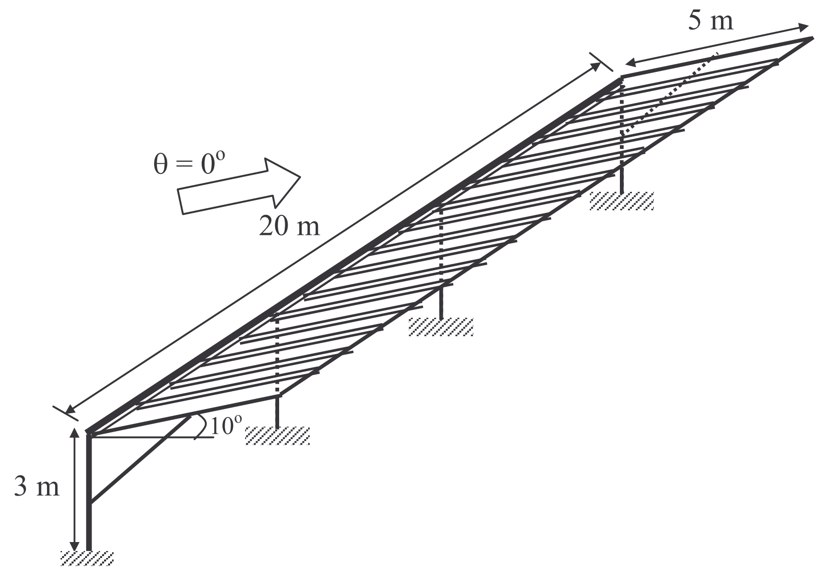


Figure 18.1

Solution:

Wind Data:

1. Wind Zone: Zone IV (V_b= 47m/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 6.1.1)

Area Averaging factor 'K_a' = 0.80*, for Roof
(IS:875-pt.3, Sec 6.1.2, 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)

$p_Z = 0.6\ (V_Z)^2 = 0.6 \times (42.77)^2 = 1097.56$ N/m^2
(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.097 \times 0.9 \times 0.8 = 0.79$ kN/m^2 , for roof
IS:875-pt.3, Sec 6.1)

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:

$$F = (C_{pe} - C_{pi}) \times A \times p_d$$
(IS:875-pt.3, Sec 6.2.1)

Net Pressure Coefficients

Using the IS:875-pt.3, Table 8 with roof angle 10^o 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 × w = 0.10 × 5 = 0.5 m
=➔ +1.6 or –2.1

At ends, up to 0.10 × L = 0.10 × 20 = 2.0 m
=➔ +2.4 or –2.0

In mid zone
=➔ +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.

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 × 0.9 = 0.987 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 α is 15^0. 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.

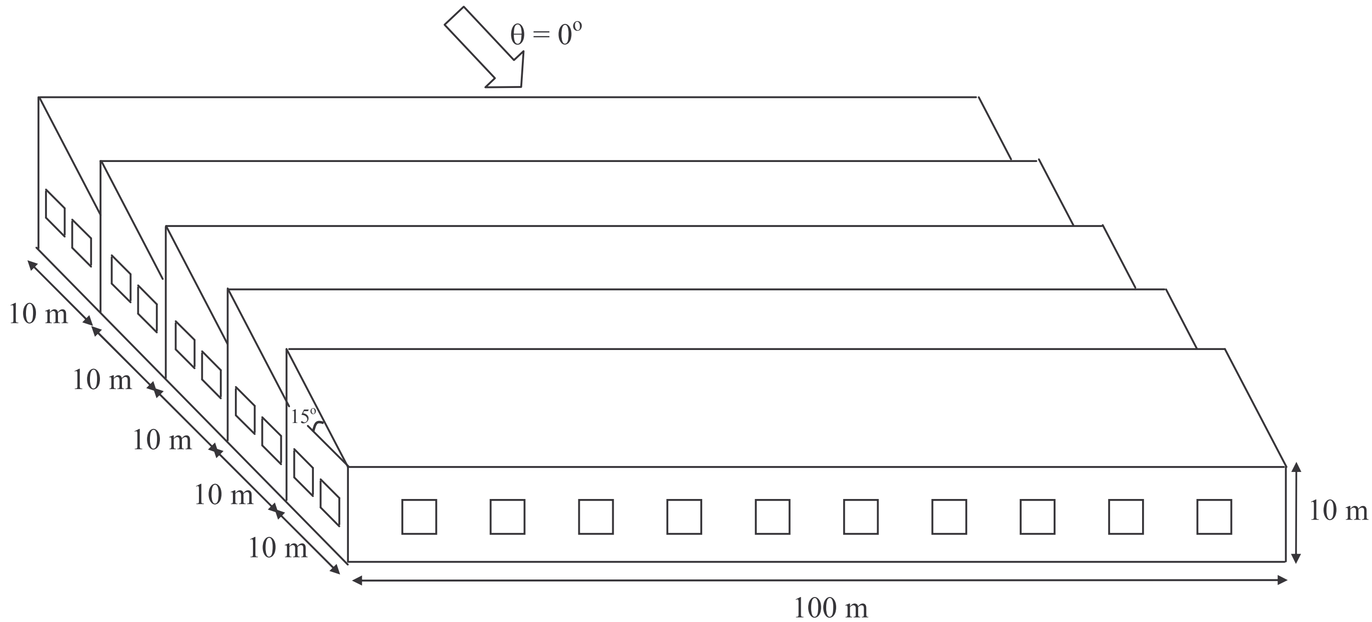


Figure 19.1

Solution:

Wind Data:

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

2. Terrain category: Category 2 (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 *(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-pr.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' = 0.80, Short walls, for Long walls & Roof* (IS:875-pt.3, Sec 6.1.2, Table-4)

Area of short (gable) walls $= 5 \times 2 \times (10+12.68) \times 0.5 \times 10 = 567\ m^2$

Area of long walls $= 100 \times 12.68 + 100 \times 10 = 2268\ m^2$

Area of roof $= 100 \times 10.35 = 1035\ m^2$

* 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.

Permeability of the Building:
Area of all the walls = 567 + 2268 = 2835 m^2
Area of all the openings =20×1.5×1.5 ×2= 90 m^2
% Opening area = 3.17 %, 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 $= V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 =$
$47 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 47.00$ m/s, for walls
$47 \times 1.0 \times 1.03 \times 1.0 \times 1.0 = 48.41$ m/s, for roof
(IS:875-pt.3, Sec 5.3)

$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (47.00)^2 = 1325.4$ N/m^2, for walls
$= 0.6 \times (48.41)^2 = 1406.1$ N/m^2, for roof
(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.3254 \times 0.9 \times 0.8$
$= 0.9543$ kN/m^2 (for walls)
IS:875-pt.3, Sec 6.1)
$= 1.4061 \times 0.9 \times 0.8 = 1.0124$ kN/m^2 (for Roof)

Wind Load Calculations:

$$F = (C_{pe} - C_{pi}) \times A \times p_d$$
(IS:875-pt.3, Sec 6.2.1)

Internal Pressure Coefficient $C_{pi} = \pm\, 0.2$
(IS:875-pt.3, Sec 6.2.2.2)
Note: buildings shall be analysed once for pressure of 0.2 from inside and then for a suction of –0.2 from inside (refer note 2 Sec 6.2.1) along-with external pressure coefficient.

External Pressure Coefficients
Using Tables 5, 6 and 11 (with values in table 11 to take precedence).
$h_{av} / w = 11.34/50 = 0.227$ & $l/w = 100/50 = 2.0$
(IS:875-pt.3, Sec 6.2.3.4)

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

Table 19.1

Portion of roof	Wind Incidence Angle 0°	90°**	180°
A	+0.7	-0.5	-0.2
B	-0.9		-0.2/0.2
C	-0.9		-0.3
D	-0.5/0.2	End Zones -1.0+	-0.2/0.2
M	-0.5/0.5	Mid Zones -0.5+	-0.4
N	-0.5/0.3		-0.4
W	-0.3/0.5		-0.7
X	-0.4		-0.3
Y	-0.2	-0.5	+0.7

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.

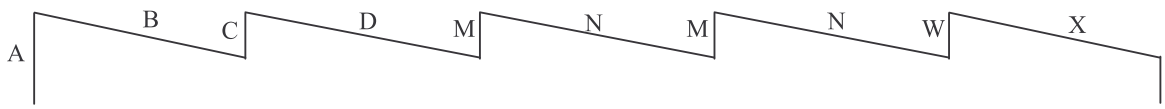
**: Values are from IS:875-pt.3, Table 5 & 7.
+ : Additional values of {-0.05(*n*-1)}, with *n*=4 is applicable in zone *E/G* for a distance equal to *h* (=10m) from Gable ends. However, local pressure coefficients are to be considered above these.
C_{pe} for walls is taken from table 5 for h/w < 0.5 and l/w = 2 and from Table 11. This is as follows:

Table 19.2

Wind angle	Short Wall 50 m wide	Long wall 100m long
0^o	-0.6	+0.7/-0.2
90^o	+0.7/-0.1	-0.5

<u>Design Pressure Coefficients for Roof:</u>

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:



	B	C	D	M	N	W	X
0^o wind, Cpi = +0.2	**-1.1**	-1.1	-0.7	-0.7	-0.7	-0.5	-0.6
0^o wind, Cpi = -0.2	-0.7	-0.7	+0.4	**+0.7**	+0.5	+0.7	-0.2
180^o wind, Cpi = +0.2	-0.4	-0.5	-0.4	-0.6	-0.6	-0.9	-0.5
180^o wind, Cpi = -0.2	+0.4	-0.1	+0.4	-0.2	-0.2	-0.5	-0.1

90^o wind, Cpi = +0.2 (end zones) ←================== -1.2 ======================→

90^o wind, Cpi = -0.2 (end zones) ←================== -0.8 ======================→

90^o wind, Cpi = +0.2 (mid zones) ←================== -0.8 ======================→

90^o wind, Cpi = -0.2 (mid zones) ←================== -0.4 ======================→

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

<u>Design Pressure Coefficients for Walls:</u>

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, pressure

= -0.5 – (+0.2) = -0.7, suction

C_{pnet} for Gable Walls

= +0.7 – (-0.2) = +0.9, pressure

= -0.6 – (+0.2) = -0.8, suction

<u>Local coefficients for roof</u>: 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, $p_d = 1.4061 \times 0.9 = 1.2655$ 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.

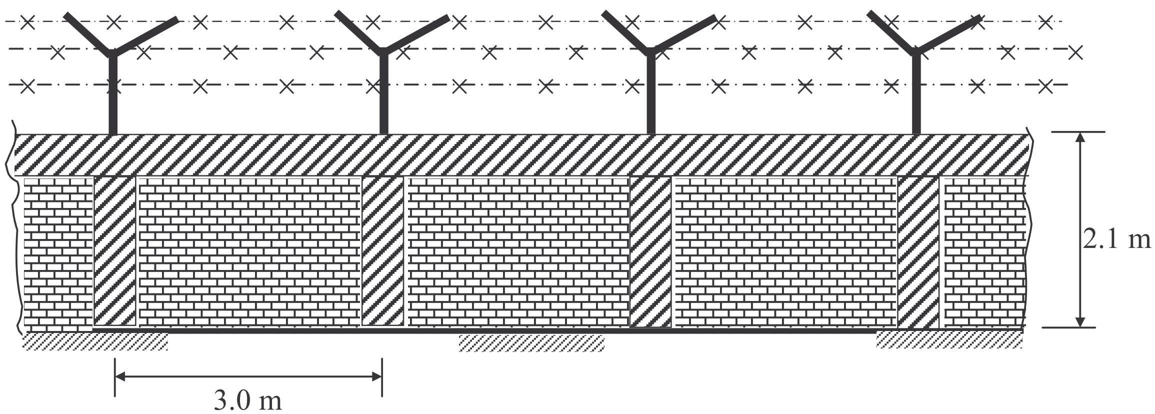


Figure 20.1

Solution:

Wind Data:

1. Wind Zone: Zone II (V_b= 39m/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 6.1.1)

Area Averaging factor 'K_a' = 1.0**
(IS:875-pt.3, Sec 6.1.2, 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 × 2.1 = 6.3m^2, for the design of columns

Design Wind Pressure

Design Wind Speed = V_Z = $V_b \times k_1 \times k_2 \times k_3 \times k_4$ = $47 \times 0.92 \times 1.05 \times 1.0 \times 1.0 = 45.402$ m/s

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

$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (45.402)^2 = 1236.805$ N/m^2
(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 1.2368 \times 0.9 \times 1.00 = 1.113$ kN/m^2
(IS:875-pt.3, Sec 6.1)

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

Wind Load Calculations:

$$F = C_f \times A \times p_d$$
(IS:875-pt.3, Sec 6.3)

Wind Force on Barbed wire fencing:

Assuming the solidity ratio of wire fencing and angles = 0.1

$A_e = 0.1 \times 1.0 \times 0.6 = 0.06$m^2, taking 1m length of wall.

C_f = 1.9, for flat sided single member frames
(IS:875-pt.3, Sec 6.3.3.1, Table 26)

Force on wire fencing per m length = $1.9 \times 0.06 \times 0.8905 = 0.1015$ kN acting at 2.1 + 0.3 = 2.4m 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 21)

Design Wind Force on walls, therefore

F = 1.55 ×1.0 ×0.8905 ×2.1 = 2.90 kN acting at 1.05m from ground.

Oblique wind effects as per 6.3.2.2 and now considered necessary as the wall has l >> 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.

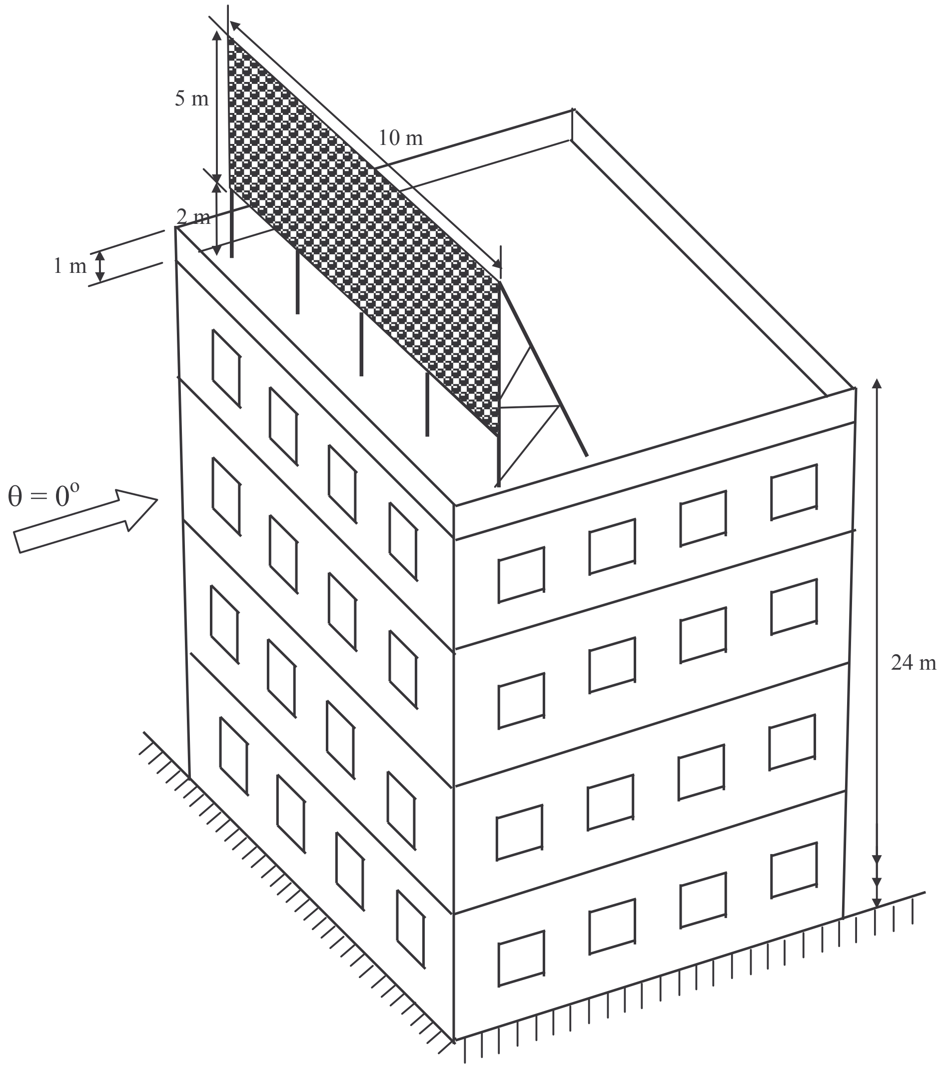


Figure 21.1

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 6.1.1)
Area Averaging factor 'K_a' = 1.0***
(IS:875-pt.3, Sec 6.1.2, Table-4)
* : considering design life of 5 yrs.

**: For average height of the hoarding, 28.5 m

*** : considering tributary area = 5 ×2.0 = 10.0m^2, for the design of Frame supports

Design Wind Pressure

Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4 =$ 47×0.71×1.05×1.0×1.0 = 35.04 m/s

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

$p_Z = 0.6\,(V_Z)^2 = 0.6 \times (35.04)^2 = 736.62$ N/m^2

(IS:875-pt.3, Sec 5.4)

$p_d = p_Z \times K_d \times K_a = 0.737 \times 0.9 \times 1.00 = 0.6633$ kN/m^2

(IS:875-pt.3, Sec 6.1)

<u>Wind Load Calculations:</u>

$$F = C_f \times A \times p_d$$

(IS:875-pt.3, Sec 6.3)

<u>Wind Force on Hoarding:</u>

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 21)

Design Wind Force on hoarding, therefore

F = 1.2 ×1.0 × 0.6633 × 5.0 = 3.98 kN acting at (2 + 2.5) = 4.5m 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 6.3.4.11.

Accordingly, 3.98 × 1.7/1.2 = 5.64 kN at windward edge and

3.98 ×0.44/1.2 = 1.46 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 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.

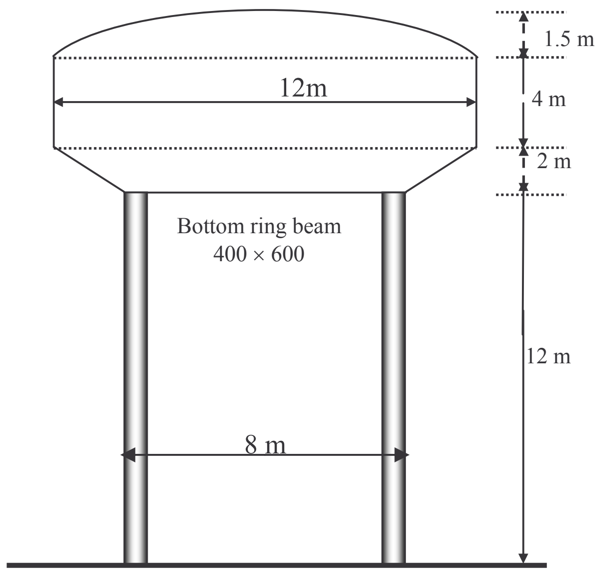


Figure 22.1

Solution:

Wind Data:

1.Wind Zone: Zone IV (V_b= 47m/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 6.1.1)

Area Averaging Factor K_a =1.00, for Staging
= 0.842*, for tank portion
(IS:875-pt.3, Sec 6.1.2, Table-4)

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

Design Wind Pressure

Design Wind Speed` = V_Z= V_b*k_1*k_2*k_3*k_4

= 47*1.07* k_2*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 6.1)

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 6.2.3.8, Table 14)

Therefore, C_{pe} = -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 x D^2 x (p_i - C_{pe} x p_d)
= 0.785 x 12^2 x {0-(-.75) x 1.082}

= 91.732 kN acting upwards at 1.2m from center of dome

(IS:875-pt.3, Sec 6.2.3.8)

``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.

<u>Overall Horizontal Force on the Tank:</u>

$F = C_f \times A_e \times p_d$

(IS:875-pt.3, Sec 6.3, 6.3.3.1(c))

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

$V_z\, x\, b$ = 45.218x12 = 542 > 6,

h/b = 4/12 = 0.333 < 2

Therefore, C_f = 0.7, from table 20(rough) &

p_d = 1.50 x 1.0 x 0.842
= 1.263 kN/m^2 at top &
= 1.393 x 1.0 x 0.842
= 1.173 kN/m^2 at bottom

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

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

Conical bottom:

V_z (avg) = (44.16+42.96)/2 = 43.56 m/s

$V_z\, x\, b$ = 43.56x10 = 435 > 6,

h/b = 2/10 = 0.2 < 2

Therefore, C_f = 0.7, from table 20 &

p_d = 1.393 x 1.0 x 0.842
= 1.173 kN/m^2 at top &
= 1.324 x 1.0 x 0.842
= 0.955 kN/m^2 at bottom

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

$F_{conicaldome}$ = 0.7 x 10 x 1.173
= 8.211 kN/m height

Staging:

p_d = 1.257 x 1 x 1 = 1.257 kN/m^2 up to 10m
= 1.324 x 1 x 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 6.3.3.1c, table 20) and no shielding effect is considered on leeward columns, as the columns are placed far apart on periphery only .

Therefore, for one column:

$V_z\, x\, b$ = 42.96x0.4 = 17.2 > 6

h/b = 11.4/0.4 = 28.5 > 20

Therefore, C_f = 1.2, from table 20 for rough surface finish.

F_{column} = 1.2 x 0.4 x 1.257
= 0.603 kN/m height, up to 10m height

F_{column} = 1.2 x 0.4 x 1.324
= 0.6355 kN/m height, above10m height

$F_{bracings}$ =1.0 x {2 x (8.0-7x.4)} x 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 20 with h/b ratio < 2. (Assuming 0.2x0.4m size braces)

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

Note: C_f values taken from Table 20 are for members of infinite length. Reduction factors for finite length of container, columns and other members can be taken from Table 24, which will further reduce wind forces.

****

Table 22.1: Calculations of Variation in Design Wind Speed & Pressure with Height

Height from Ground, m	k_2 *	V_Z m/s	p_Z kN/m^2	p_d kN/m^2
Up to 10m	0.91	45.773	1.257	1.257
12m	0.854	46.98	1.324	1.324 (for staging
				0.955 (For Tank)
14m	0.878	48.187	1.393	1.173
18m	0.92	50.00	1.50	1.263

* : k_2 values are linearly interpolated.

Table 22.2: Summary of forces and total loads on tank

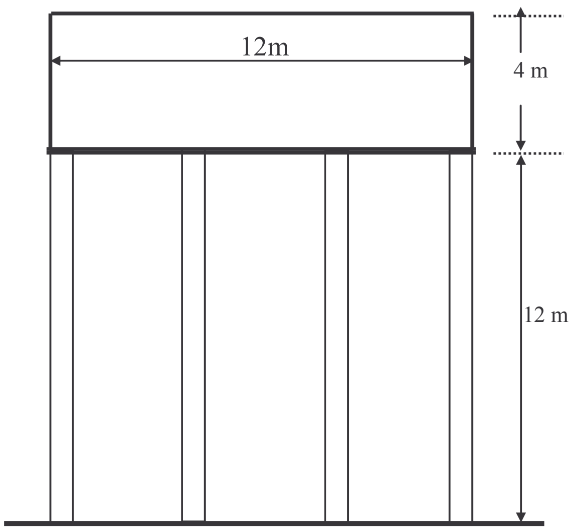
Element	Force per unit height	Height of element	Total horiz-ontal force	CG of force from ground
Cylindrical portion	10.61 kN	4.0m	42.44 kN	16.00m
Conical Dome	8.211 kN	2.0m	16.422 kN	13.067m
Top Ring Beam	10.592 kN	0.6m	6.355 kN	11.7m
All Columns above 10m	0.6355 kN x 12 = 7.626 kN	1.4m	10.676 kN	10.7m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

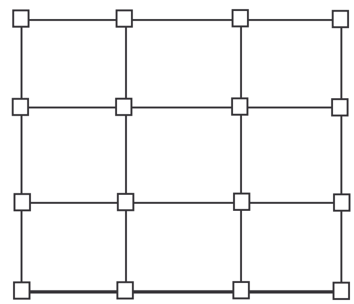
****

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.





Plan of framed staging

Figure 23.1

Solution:

Wind Data:

1.Wind Zone: Zone IV (V_b= 47m/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 6.1.1)

Area Averaging Factor K_a =1.00, for Staging

= 0.87*, for tank portion

(IS:875-pt.3, Sec 6.1.2, Table-4)

*: exposed area of tank container =12 x 4= 48m^2

Design Wind Pressure

Design Wind Speed` = $V_Z = V_b*k_1*k_2*k_3*k_4$

= 47*1.07* k_2*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 6.1)

Wind Load Calculations:

External pressure coefficients for roof and bottom of tank:*

$(z/H) - 1 = (16/4.0) - 1 = 3.0$

H/D = 4/12 = 0.333

(IS:875-pt.3, Sec 6.2.3.8, Table 14)

*: 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 x D

= 0.1 x 12 = 1.2m

The total force acting on the roof of the tank

= A_e x $(p_i - C_{pe}$ x $p_d)$ = 12x12x {0-(-0.65)x1.35}

= 126.36 kN acting upwards at 1.2 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:

$F = C_f \times A_e \times p_d$

(IS:875-pt.3, Sec 6.3, 6.3.3.1(c))

Container portion:

b/d = 1, V_z x b = 49.19 x 12 = 590>10, H/b=0.33

C_f = 0.5, from table 20.

$F_{container}$ =0.5 x 12 x 1.137
= 6.822 kN/m height

Staging:

p_d = 1.131 x 1 x 0.9 = 1.018 kN/m^2 up to 10m
= 1.192 x 1 x 0.9 = 1.073 kN/m^2 above 10m height.

The staging is multiple bay framed type and cl.6.3.3.2 with table 26 and cl.6.3.3.3 with table 27 are therefore applied.

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

=➔ C_f = 1.8, for windward frame members.

Shielding Effect on leeward frame:

Frame- spacing ratio = 4.0/0.4 = 10 =➔ η = 1.0

Hence no shielding occurs.

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

$V_z \times b$ = 42.96x0.4 = 17.2 > 10

h/b = 12/0.4 = 30 > 20

Therefore, C_f = 1.2, from table 20 for rough surface finish. The higher value of C_f i.e. 1.8 is used

F_{column} = 1.8 x 0.4 x 1.131
= 0.814 kN/m height, up to 10m height

F_{column} = 1.8 x 0.4 x 1.192
= 0.858 kN/m height, above10m height

for Bracings:

$V_z \times b$ = 42.96x0.4 = 17.2

h/b = 0.4/3.466 = 0.155 < 2

Therefore, C_f = 1.0, from table 20 for rough surface finish.

$F_{bracings}$ = 1.0 x {3(12.0-4x0.4)} x 1.131
= 35.29 kN/m height, acting at two brace levels, 4m and 8m.

Table 23.1: Calculations of Variation in Design Wind Speed & Pressure with Height

Height from Ground, m	k_2 *	V_Z m/s	p_Z kN/m^2	p_d kN/m^2
Up to 10m	0.91	45.774	1.257	1.131 (for staging)
12m	0.934	46.98	1.324	1.192 (for staging) 1.037 (for tank)
16m	0.978	49.19	1.451	1.137 (for tank)

k_2 values are linearly interpolated.

Table 23.2: Summary of forces and total loads on tank

Element	Force per unit height	Height of element	Total horiz-ontal force	CG of force from ground
Container portion	6.822 kN	4.0m	27.288 kN	14.0m
All Columns above 10m	0.858 kN x 16 = 13.728kN	2.0m	27.456 kN	11.0m
All Columns up to 10m	0.814 kN x 16 = 13.024 kN	10m	130.24 kN	5.0m
Braces, upper level	35.29 kN	0.4m	14.116 kN	8.0m
Braces, lower level	35.29 kN	0.4m	14.116 kN	4.0m

****

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.

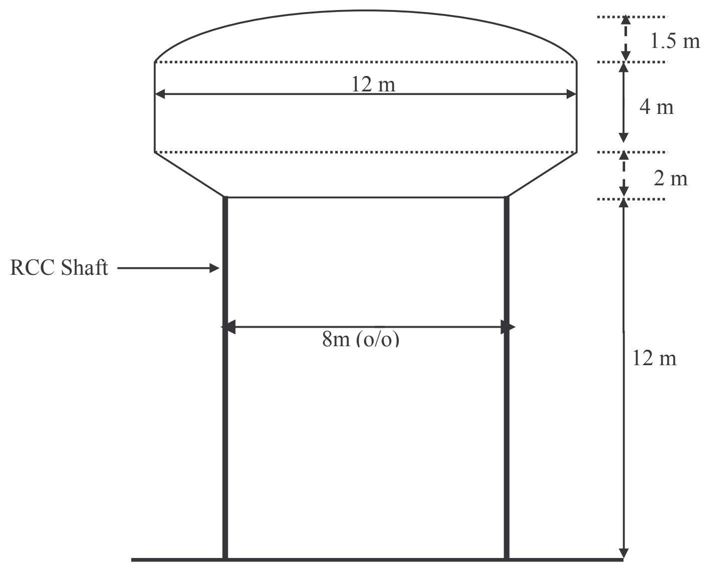


Figure 24.1

Solution:

Wind Data:

1.Wind Zone: Zone IV (V_b= 47m/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 6.1.1)

Area Averaging Factor K_a =0.805*, for Staging
= 0.842**, for tank portion (IS:875-pt.3, Sec 6.1.2, Table-4)

* : area of shaft = 12.0m x 8.0m = 96m^2

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

Design Wind Pressure

Design Wind Speed = $V_Z = V_b*k_1*k_2*k_3*k_4$
= 47*1.07* k_2*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 6.1)

Wind Load Calculations:

External pressure coefficients for roof of tank:
(IS:875-pt.3, Sec 6.2.3.8)

$(z/H) - 1 = (19.5/7.5) - 1 = 1.6$
(IS:875-pt.3, Table 14)

Therefore, C_{pe} = -0.75 for roof

Eccentricity of force at roof
= 0.1 x D = 0.1 x 12 = 1.2m

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 x 12^2 x {0-(-.75) x 1.082}
= 91.732 kN acting upwards at 1.2m from center of dome
(IS:875-pt.3, Sec 6.2.3.8)

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 6.3, 6.3.3.1(c))

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

$V_z x\ b$ = 45.218x12 = 542 > 6,

h/b = 4/12 = 0.333 < 2

Therefore, C_f = 0.7, from table 20, for rough surface &

p_d = 1.50 x 1.0 x 0.842
= 1.263 kN/m^2 at top &
= 1.393 x 1.0 x 0.842
= 1.173 kN/m^2 at bottom

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

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

Conical bottom:

V_z (avg) = (44.16+42.96)/2 = 43.56 m/s

$V_z x\ b$ = 43.56x10 = 435 > 6,

h/b = 2/10 = 0.2 < 2

Therefore, C_f = 0.7, from table 20, for rough surface &

p_d = 1.393 x 1.0 x 0.842
= 1.173 kN/m^2 at top &
= 1.324 x 1.0 x 0.842
= 1.115 kN/m^2 at bottom

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

$F_{conicaldome}$ = 0.7 x 10 x 1.173
= 8.211 kN/m height

Staging:

Shaft is considered as circular member with rough roughness at surface for which

$V_z x\ b$ = 42.96 x 8.0 = 343 > 6,

h/b = 12.0/8.0 = 1.5 < 2

Therefore, C_f = 0.7, from table 20, for rough surface

F_{shaft} = 0.7 x 8.0 x 1.257 x 0.805
= 5.67 kN/m height, up to 10m height

F_{shaft} = 0.7 x 8.0 x 1.324 x 0.805 = 5.97 kN/m height, above10m height

Note: C_f values taken from Table 20 are for members of infinite length. Reduction factors for finite length of container, columns and other members can be taken from Table 24, which will further reduce wind forces.

Table 24.1: Calculations of variation in design wind speed & pressure with height

Height from Ground, m	k_2 *	V_Z m/s	p_Z kN/m^2
Up to 10m	0.91	45.773	1.257
12m	0.934	46.98	1.324
14m	0.958	48.187	1.393
18m	0.994	50.00	1.50

* : k_2 values are linearly interpolated.

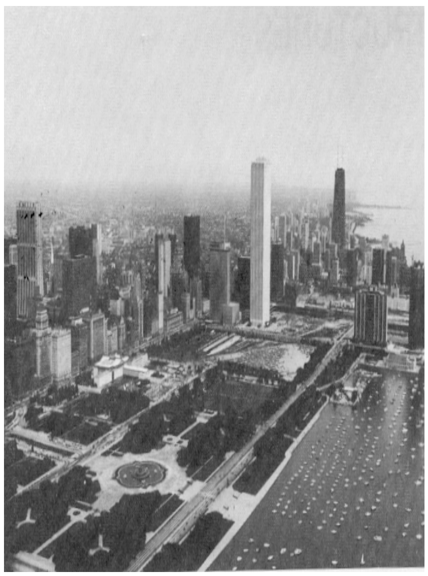
Table 24.2: Summary of Forces and Total Loads on Tank

Element	Force per unit height	Height of element	Total horizontal force	CG of force from ground
Cylindrical portion	10.61 kN	4.0m	42.44 kN	16.00m
Conical Dome	8.211 kN	2.0m	16.422 kN	13.067m
Shaft above 10m	5.97 kN	2.0m	11.94 kN	11.0m
Shaft up to 10m	5.67 kN	10.0m	56.7 kN	5.0m

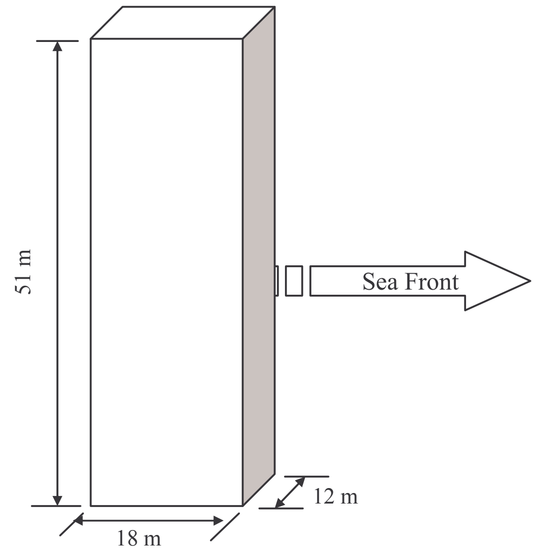
Example 25 - Wind Pressure and Forces on a Multistory Commercial Complex by Force Coefficient Method

Problem Statement:

Calculate design equivalent static wind forces 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.



(a)



(b)

Figure 25.1

Solution:

Wind Data:

1. Wind Zone: Zone III (V_b= 44m/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 = 200m, developed height in TC 3, h_3 = 35m (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. (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)

Importance Factor for Cyclonic Region k_4 = 1.00 (IS:875-pt.3, Sec 5.3.4)

Wind Directionality Factor K_d = 0.90

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 6.1.2, Table-4)

* tributary area for glazing/cladding shall be less than $10m^2$, depends on the supporting system.

(IS:875-pt.3, Sec 6.1.1)

Design Wind Pressure:

Design Wind Speed = $V_Z = V_b \times k_1 \times k_2 \times k_3 \times k_4$

$= 47 \times 1.0 \times k_2 \times 1.0 \times 1.0 = (47 \times k_2)$ 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 6.1)

Table 25.1 : Calculations of Variation in Design Wind Speed with Height

Height from ground, m	**k_2*** **For sea face♠**	**For other faces ♥**	**V_Z (m/s)** **For sea face**	**For other faces**
Up to 9m	0.91	0.80	42.77	37.6
12m	0.934	0.80	43.90	37.6
15m	0.97	0.80	45.59	37.6
18m	0.994	0.80	46.72	37.6
21m	1.015	0.817	47.70	38.40
24m	1.03	0.87	48.41	40.80
27m	1.045	0.92	49.115	43.24
30m	1.06	0.97	49.82	45.59
33m	1.07	0.99	50.29	46.53
36m	1.165+	1.009	54.755	47.423
39m	1.1725	1.0285	55.107	48.34
42m	1.18	1.048	55.46	49.256
45m	1.1875	1.0675	55.81	50.17
48m	1.195	1.087	56.165	51.09
51m	1.2012	1.102	56.456	51.80

* : k_2 values are linearly interpolated.

\+ : Effect of terrain category change from TC3 to TC1 above this height

♠ : For terrain category 1 transiting to category 3

♥ : For terrain category 4

Table 25.2: Calculations of Variation in Design Pressure with Height

Height from	p_Z (kN/m^2)		p_d, for building		p_d, for cladding
ground, m	Sea face	Other face	Sea face	Other faces	All faces*
Up to 9m	1.097	0.848	0.79	0.610	0.987
12m	1.156	0.848	0.832	0.610	1.04
15m	1.247	0.848	0.90	0.610	1.122
18m	1.310	0.848	0.943	0.610	1.178
21m	1.365	0.885	0.983	0.637	1.228
24m	1.406	1.000	1.012	0.720	1.265
27m	1.447	1.122	1.042	0.808	1.302
30m	1.489	1.247	1.072	0.898	1.34
33m	1.517	1.300	1.092	0.936	1.365
36m	1.799	1.349	1.295	0.971	1.619
39m	1.822	1.402	1.312	1.010	1.64
42m	1.845	1.456	1.328	1.048	1.66
45m	1.87	1.510	1.346	1.087	1.683
48m	1.893	1.566	1.363	1.127	1.704
51m	1.912	1.610	1.377	1.159	1.721

Notes: 1. For building faces Ka = 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 6.3)

Force coefficient calculations:
Long-afterbody orientation
a/b = 18/12 = 1.5, h/b = 51/12 = 4.25 =====➔ C_f = 1.2

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

Short-afterbody orientation
a/b = 12/18 = 0.667, h/b = 51/18 = 2.833 =====➔ C_f = 1.35

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

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 6.1.2, Table-4)

Tributary area for calculating wind forces on building frames = 51 x 6 = 306m^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,

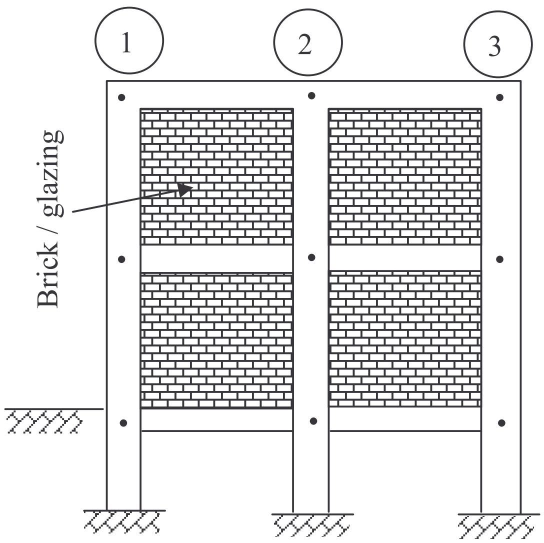


Figure 25.2

For the calculation of wind forces along the height, the area averaging factor for nodes on frame-1 shall depend upon an area

$$= \text{height of the building} \times \left[\frac{\text{c / c dist between frame (1) \& (2)}}{2}\right]$$

for nodes on frame-2, it shall depend upon an area

$$= \text{height of the building} \times \left[\frac{\text{c / c dist between frame (1) \& (2)}}{2} + \frac{\text{c / c dist between frame (2) \& (3)}}{2}\right],$$

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.

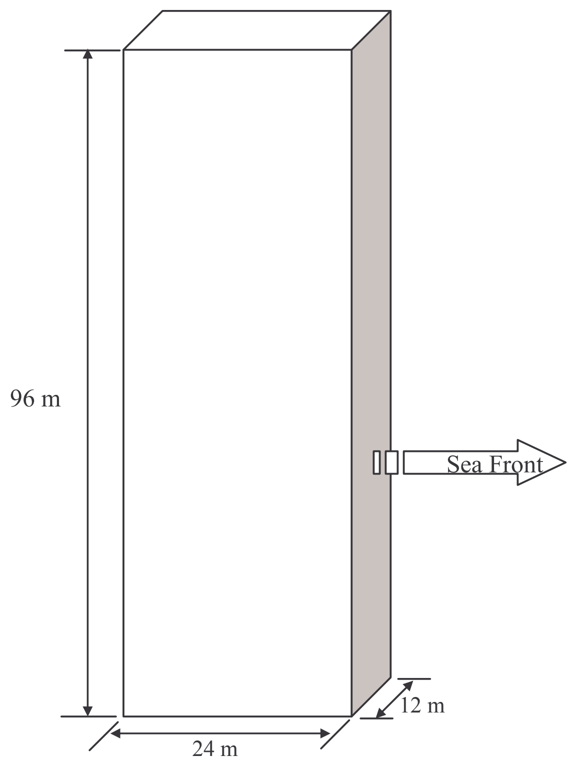


Figure 26.1

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= 200m, developed height in TC3, h_3 = 35m (IS:875-pt.3, Table 3)

Therefore, up to 35m heights, *k2* 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 6.1.1)
Area Averaging factor 'K_a' = 1.00*, for glazing/cladding.

This will be same for the building, since the influence of area is accounted for in the Dynamic Response Factor.

* : *tributary area for glazing/cladding shall be less than 10m², depends on the supporting system.*

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

$p_Z = 0.6\,(V_Z)^2$ & $p_d = p_Z \times K_d \times K_a$
(IS:875-pt.3, Sec 5.4, 6.1)

Table 26.1 : Variation in design wind speed & design pressure with height

Height from Ground, m	k_2* sea Face	k_2* other faces	V_Z (m/s) sea Face	V_Z (m/s) other faces	p_Z (kN/m²) Sea face	p_Z (kN/m²) Other face
Up to 9m	0.91	0.80	42.77	37.6	1.097	0.848
12m	0.934	0.80	43.90	37.6	1.156	0.848
18m	0.994	0.80	46.72	37.6	1.310	0.848
24m	1.03	0.868	48.41	40.796	1.406	1.000
30m	1.06	0.97	49.82	45.59	1.489	1.247
36m	1.165+	1.009	54.755	47.423	1.80	1.349
42m	1.18	1.048	55.46	49.256	1.845	1.456
48m	1.195	1.087	56.165	51.09	1.893	1.566
54m	1.205	1.108	56.635	52.076	1.924	1.627
60m	1.212	1.120	56.964	52.640	1.947	1.662
66m	1.220	1.132	57.340	53.204	1.973	1.698
72m	1.226	1.144	57.622	53.768	1.992	1.734
78m	1.234	1.156	57.980	54.332	2.017	1.771
84m	1.241	1.168	58.327	54.896	2.041	1.808
90m	1.248	1.180	58.656	55.460	2.064	1.845
96m	1.255	1.192	58.985	56.024	2.087	1.883

* : 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)

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. 6)

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. 6)

Effective area 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 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 = 12 m
$L_h = 100\,(96/10)^{0.25} = 176$ m
$f_0 = I/T = \sqrt{12}/(0.09 \times 96) = 0.40$ Hz, $\beta = 0.02$
(IS:875-pt.3, Table 31)

(i) For base floor (s = 0)

$B_s = 1/[1+(36\times96^2+64\times24^2)^{0.5}/(2\times176)] = 0.3670$
$H_s = 1+(0/96)^2 = 1.0$, $V_h = 48.0$ m/s at h = 96 m
$g_R = \sqrt{[2\log_e(3600\times0.40)]} = 3.814$
$I_h = 0.235$ for Terrain Category 4
(IS:875-pt.3, Sec.9.1, Table 30)
$g_v = 3.5$
(IS:875-pt.3, Sec.9.1)

$$S = 1/\left[1+\frac{4\times0.40\times96(1+3.5\times0.235)}{48.0}\right]\left[1+\frac{4\times0.40\times24(1+3.5\times0.235)}{48.0}\right] = 0.0599$$

$N = f_0L_h\,[1+g_v I_h]/v_h = 0.40\times176[1+3.5\times0.235]/48.0 = 2.673$

$E = \pi N/(1+70N^2)^{5/6} = 0.04722$
$C_{dyn} = \{1+2\times0.235[3.5^2\times0.367 + 1.0\times3.814^2\times0.0599\times0.04722/0.02]^{0.5}\}/(1+2\times3.5\times0.235) = 0.833$

(ii) For floor level at mid-height (s=48m)

$H_s = 1+(s/h)^2 = 1+(48/96)^2 = 1.25$
$B_s = 1/[1+36\times48^2+64\times24^2)^{0.5}/(2\times176)] = 0.5042$
$C_{dyn} = 0.903$

(iii) Top floor level (s = 96 m)

$H_s = 2.0$, $B_s = 0.6471$, $C_{dyn} = 0.994$

B. Wind from Sea (on smaller face)

h = 96m, d = 24m, b = 12 m, L_h = 176 m
$f_0 = I/T = \sqrt{24}/(0.09\times96) = 0.567$ Hz,
$\beta = 0.02$
(IS:875-pt.3, Table 39)

(i) For base floor (s = 0)
$B_s = 1/[1+(36\times96^2+64\times12^2)^{0.5}/(2\times176)]$
$= 0.379$
$H_s = 1+(0/96)^2 = 1.0$, $V_h = 60.5$ m/s
$g_R = \sqrt{[2\log_e(3600\times0.567)]} = 3.904$
$I_h = 0.110$ for Terrain Category 1
(IS:875-pt.3, Sec.9.1, Table 40)
$g_v = 3.5$
(IS:875-pt.3, Sec.9.1)

$$S = 1/\left[1+\frac{4\times0.567\times96(1+3.5\times0.110)}{60.5}\right]\times\left[1+\frac{4\times0.567\times12(1+3.5\times0.110)}{60.5}\right] = 0.103$$

$N = f_0L_h\,[1+g_v I_h]/v_h$
$= 0.567\times176[1+3.5\times0.110]/60.5$
$= 2.285$
$E = \pi N/(1+70N^2)^{5/6} = 0.0524$
$C_{dyn} = 0.932$

(ii) Floor level at mid-height (s=48m)
$H_s = 1.25$, $B_s = 0.537$, $C_{dyn} = 0.990$

(iii) Top floor level (s = 96 m)
$H_s = 2.0$, $B_s = 0.786$, $C_{dyn} = 1.090$

For Cross-wind Response

$$C_{dyn} = 1.5 g_R \left(\frac{b}{d}\right) \frac{K_m}{(1+g_v I_h)^2} \left(\frac{z}{h}\right)^k \sqrt{\frac{\pi C_{fs}}{\beta}}$$

$K_m = 0.76 + 0.24\ k = 1.00$ for $k = 1$

A. Wind onto wider face (short afterbody orientation)

For z = h

$$V_n = \frac{V_z}{f_0 b(1+g_v I_h)}$$

$$= \frac{48.0}{0.4 \times 24 (1 + 3.5 \times 0.235)} = 2.75$$

$I_h = 0.261$ (at 2h/3 height)

Using dashed line of IS:875-pt.3, Fig-12

$Log_{10}\ C_{fs} = -3.0$; $C_{fs} = 0.0010$

$$C_{dyn} = 1.5 \times 3.814 \left(\frac{24}{12}\right) \frac{1.00}{(1+3.5\times 0.261)^2} (1)^{1.0} \sqrt{\frac{\pi \times 0.0010}{0.02}}$$

$= 1.24$

However, at the base the 0.261 mode shape deflection is zero and varies linearly with height in this case as k = 1.

Table 26.2 : Calculations of variation in design pressure with height

Height from ground, m	p_Z (kN/m^2) Sea face	Other face	p_d, for building Sea face	Opp. Sea face	Wide faces	p_d, for cladding All faces*
Up to 9m	1.097	0.848	0.987	0.763	0.763	0.987
12m	1.156	0.848	1.040	0. 763	0. 763	1.404
18m	1.310	0.848	0.179	0. 763	0. 763	1.179
24m	1.406	0.998	1.265	0.898	0.897	1.265
30m	1.489	1.247	1.340	1.122	1.122	1.34
36m	1.80	1.349	1.62	1.214	1.214	1.62
42m	1.845	1.456	1.660	1.311	1.311	1.66
48m	1.893	1.566	1.704	1.410	1.410	1.704
54m	1.924	1.627	1.732	1.464	1.464	1.732
60m	1.947	1.662	1.752	1.496	1.496	1.752
66m	1.973	1.698	1.776	1.528	1.528	1.776
72m	1.992	1.734	1.793	1.560	1.560	1.793
78m	2.017	1.771	1.815	1.594	1.594	1.815
84m	2.041	1.808	1.837	1.627	1.627	1.837
90m	2.064	1.845	1.857	1.660	1.660	1.86
96m	2.087	1.883	1.878	1.695	1.695	1.878

Notes: For cladding, only higher wind speed is used for all four faces. However, the designer may choose to vary it face to face.

Section – 3

Some Unusual Cases for the Determination of Wind Forces on Buildings/Structures

Situations may occur where the provisions of the code do not provide direct answers or the answers are incomplete or ambiguous. Some such situations have been identified and discussed in this section. The answers are however only suggestive, as unique situations may rarely arise. Bearing this in mind, the following answers may be considered as advisory opinions. In many cases it would be desirable to carry out wind tunnel studies for getting better answers.

Case 1(a) : A building with two levels of roofs, one considerably higher than the other, is shown in Fig. 1. What shall be the mean roof level for determining wind pressures on the low roof and the walls ?

It is difficult to get straightforward guidance for this type of situation. Pressures on walls and roofs occur due to the flow pattern generated around the structure. It may be appropriate to base the pressures on the windward side on the lower height h_1. However, for the leeward side, since the wake created by the separation from the taller roof may envelope the entire leeward side, the pressures may be based on the height of the taller roof h_2.

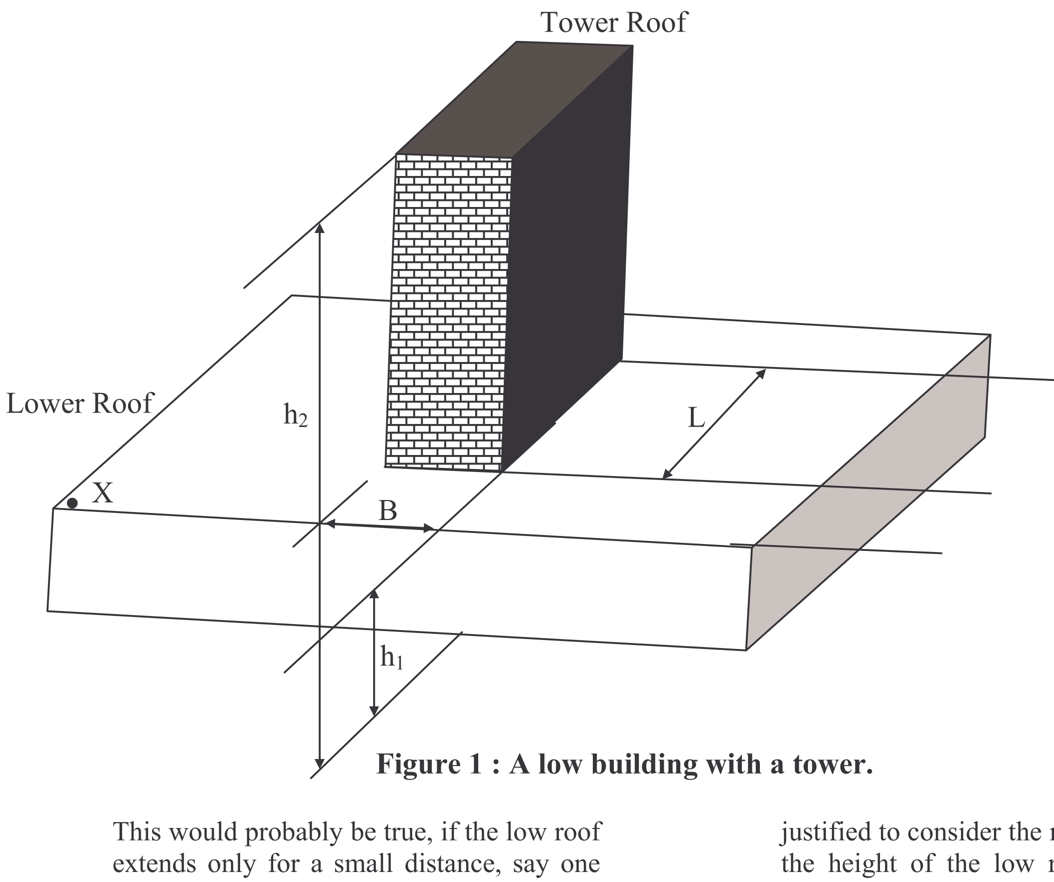


Figure 1 : A low building with a tower.

This would probably be true, if the low roof extends only for a small distance, say one bay, on either side of the higher structure. In structures where the low roof extends much further from the tower, it might be justified to consider the mean roof height as the height of the low roof for roof areas some distance away from the higher roof.

It is however suggested that, in all cases, the height of the taller roof may be taken as the "mean roof height", which would lead to conservative values – but perhaps not too much.

Case 1(b) : What would be the distribution of the wind pressures over the lower roof in the building of Figure-1 ?

The pressures on the lower roof may vary significantly. It will however, be conservative to use the height of the higher roof (h_2) to calculate the leeward pressure. For a point such as X, the actual pressures would more likely be based on the height of the lower roof (h_1), since it is located relatively far from the potential wake of wind around the tower.

As a general rule the wind pressures for points located within a distance of the width of the tower (B or L depending on wind direction) might be based considering h_2 as the mean roof height. For other points design wind pressures might be based on lower roof height h_1 taken as the mean roof height.

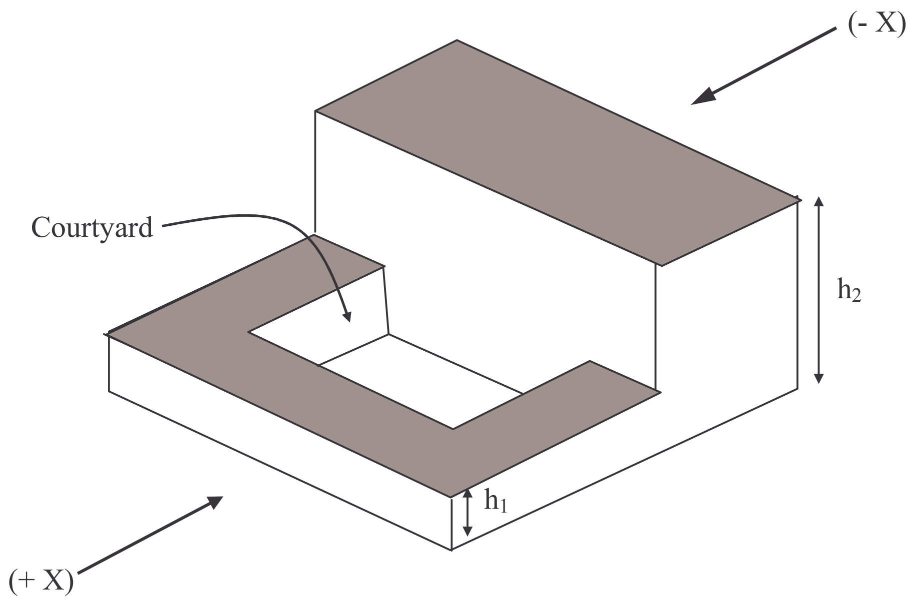


Figure 2 : Building with a courtyard

Case 2 : A building with a courtyard is shown in Figure 2. How would the courtyard affect the wind pressures ? What height shall be used for the pressures on the low building, h_1 or h_2 ? Will the size of the courtyard affect the pressure distribution?

For wind blowing in the direction (+)X, the wall of the low building will experience positive pressures and the roof will have negative pressures (suction). The operative height will be the height h_1. The inner courtyard walls will also experience pressure because of the obstruction from the taller building. For the wind direction (-)X, the lower building roof as well as the outer wall will be subject to suction, with the inner walls being under pressure, because of the wake of the taller building. The applicable height will be h_2. The leeward wall of the taller building will also have suction.

If the courtyard is large, the two buildings will tend to act independently. For small courtyards, the position as described above will prevail, by and large.

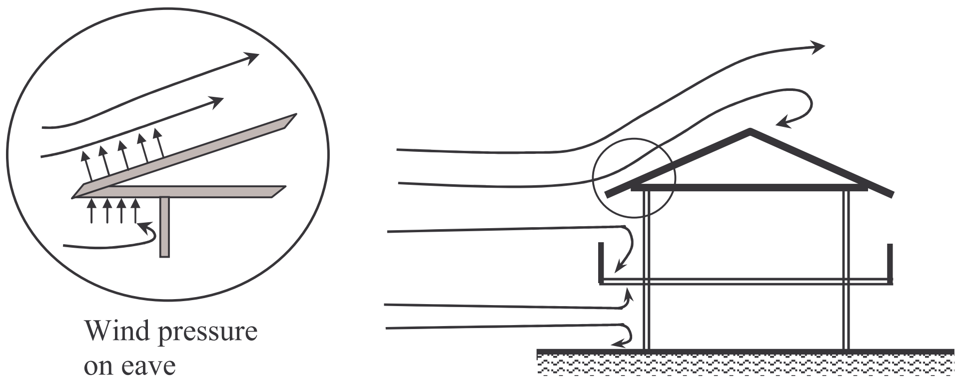


Figure 3 : Building with balconies and overhanging eaves

Case 3 : A structure with balcony and eaves is shown in Figure 3. What pressures will act on the balcony and the eaves?

For the wind direction shown, the pressure on the bottom surface of the balcony would be acting upwards and the pressure on the upper surface would be acting downwards so that, the net pressure would be small.

For wind blowing parallel to the balconies, there would be an upward pressure as in case of an open structure.

The eaves will have a net uplift force due to a combination of the upward pressure on the bottom surface and suction on the roof surface. This is explained by the flow-separation as the wind below the eave surface gets obstructed by the wall in the front.

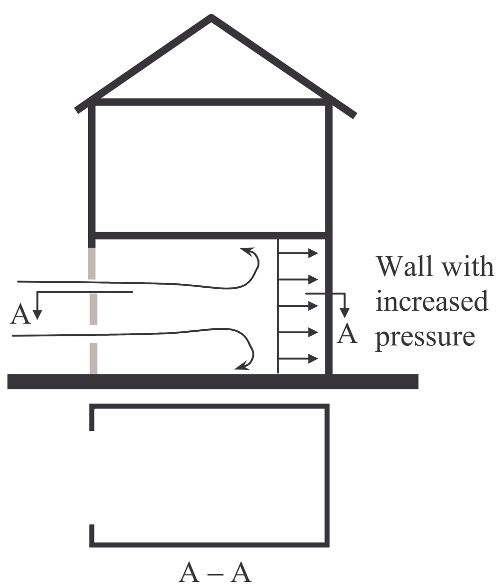


Figure 4 : Partially enclosed garages

Case 4 : ***Figure 4 shows a two-story building which has a partially closed story at the ground floor for parking. What would be the pressure on the walls of the ground floor?***

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It would be prudent to consider an increased C_p for the design of these garage walls as well as the low roof, which is also subjected to the increased internal pressures. The best estimate of this increase can be reached by considering the bottom story as a building with a large opening.

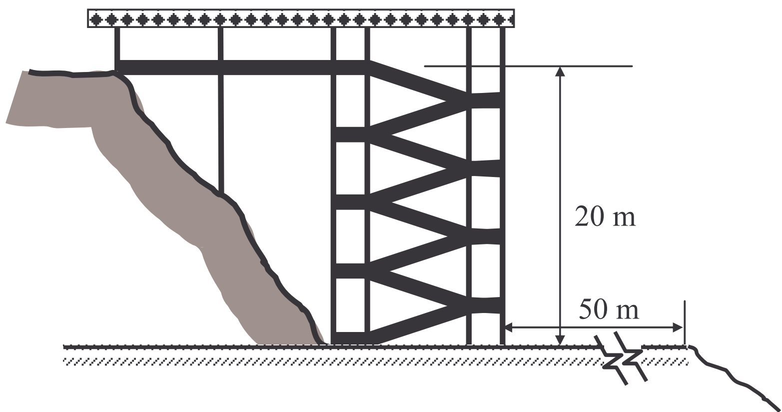

Figure 5 : Stair tower at a water front.

Case 5(a) : ***A stair tower at a waterfront park is within 50m of a large body of water. It is 20 m tall, 10 wide, and open on all sides. How should the wind pressure be evaluated on this structure ?***

IS code provides for a maximum increase of 36% in the wind velocity (given by factor k_3) close to a hill or an escarpment. In all the elements of the stair tower, the increase in wind velocity due to factor k_3 should be considered in order to account for the vertical flow of wind up the face of the hill/escarpment. Since the structure is open type, force coefficients may be used for overall design treating it as an open frame. The vertical pressures may be obtained for the stairs as well as the roof treating them as inclined or flat surfaces. Higher pressures should be taken at the edges and corners.

Case 5(b) : ***The structure being close to sea and thus exposed to salt water spray, its design life may be reduced to say 15 years, which is much less than 50 or 100 years period normally considered in the design. What wind velocity shall be considered for reduced life of the structure ?***

One way of looking at the problem is to consider a reduced life and thus a reduced design wind velocity. The other alternative is to introduce provisions for safeguarding the material from the effects of exposure to salted air, so that the structure has a longer life. Perhaps the latter is preferable.

Case 6 : ***Do structures placed at the crests of hills and escarpments experience increased wind speeds ?***

It is well established that wind velocities increase over hills and escarpments. That is why windmills are placed on tops of hills. There have been more failures of transmission towers located on hills than elsewhere. Results from wind tunnel studies also provide confirmation of this speedup phenomenon. More failures are reported due to wind for buildings on hills than in plains for similar type of buildings.

Case 7 : ***What if a building owner cuts down all the trees that gave the site a lower level of exposure during the process of design and now is transformed to***

higher exposure. Should the structure be upgraded ?

The IS Code does not explicitly require that the building be upgraded to the higher exposure. Perhaps the only exception would be if the trees were cut down before construction began and the designer had the opportunity to examine the design for possible upgradation.

Case 8 : Figure 6 shows parapets on a building. How should the forces on parapets be evaluated for the design of the main frame ?

Forces will be as shown in the figure.

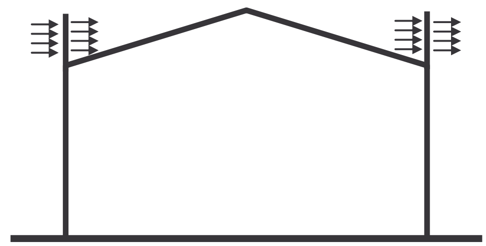

Figure 6: Pressures on parapet