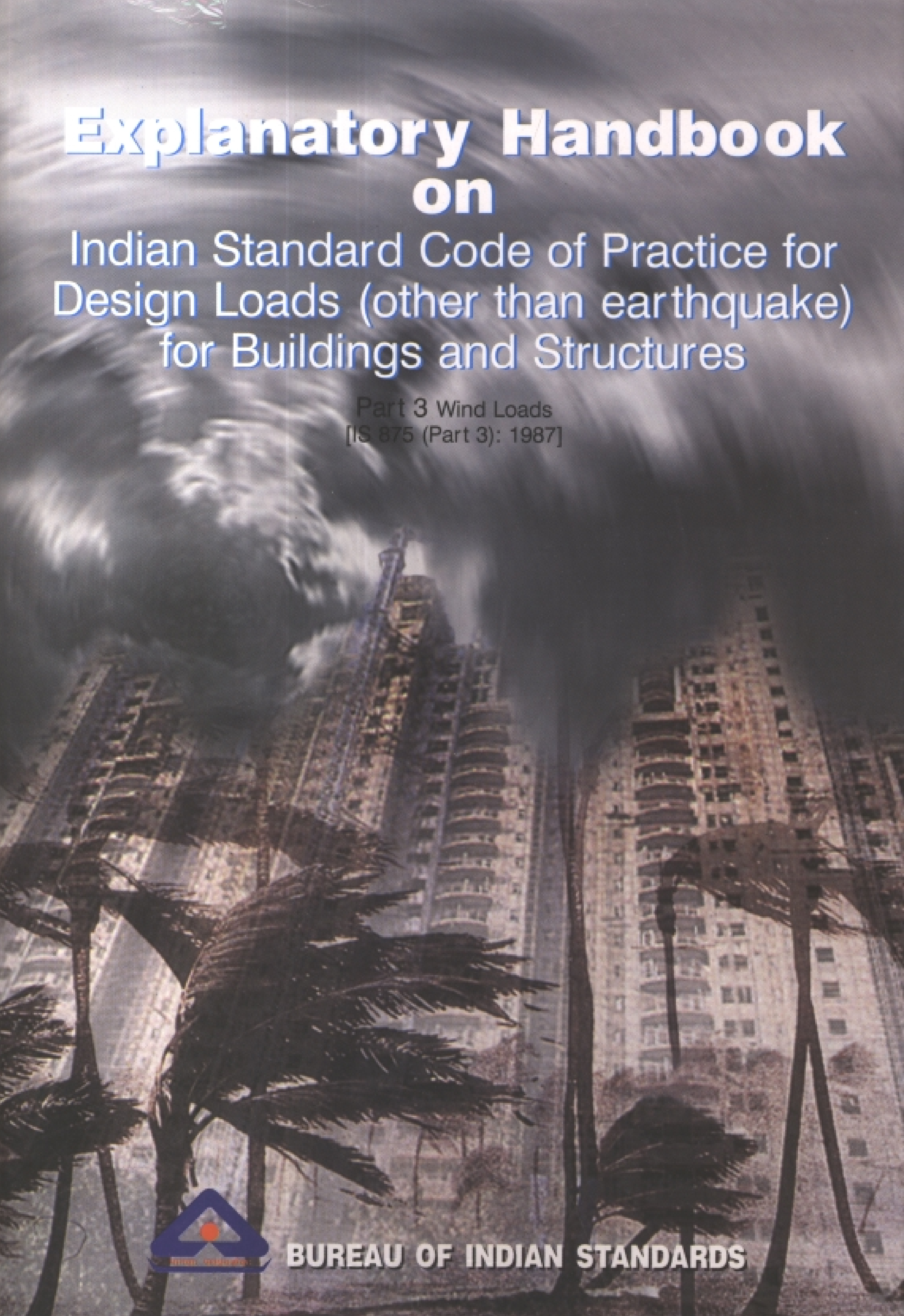
Explanatory Handbook on Indian Standard Code of Practice for Design Loads (other than earthquake) for Buildings and Structures

Part 3 Wind Loads

[IS 875 (Part 3): 1987]



BUREAU OF INDIAN STANDARDS

EXPLANATORY HANDBOOK

ON

INDIAN STANDARD CODE OF PRACTICE FOR DESIGN LOADS (OTHER THAN EARTHQUAKE) FOR BUILDINGS AND STRUCTURES

PART 3 WIND LOADS
[IS 875 (PART 3) : 1987]

BUREAU OF INDIAN STANDARDS
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FOREWORD

Users of various civil engineering codes have been feeling the need for explanatory handbooks and other compilations based on Indian Standards. The need has been further emphasized in view of the publication of the National Building Code of India in 1970 (which has since been revised in 1983) and its implementation. The Expert Group set up in 1972 by the Department of Science and Technology, Government of India carried out in-depth studies in various areas of civil engineering and construction practices. During the preparation of the Fifth Five-Year Plan in 1975, the Group was assigned the task of producing a Science and Technology Plan for research, development and extension work in the sectors of housing and construction technology. One of the items of this plan was the formulation of design handbooks, explanatory handbooks and design aids based on the National Building Code and various Indian Standards and other activities in the promotion of the National Building Code. The Expert Group gave high priority to this item and on the recommendation of the Department of Science and Technology, the Planning Commission approved the following two projects which were assigned to the Bureau of Indian Standards (erstwhile Indian Standards Institution) :

a) Development programme on code implementation for building and civil engineering construction, and

b) Typification for industrial buildings.

A Special Committee for Implementation of Science and Technology Projects (SCIP) consisting of experts connected with different aspects was set up in 1974 to advise the BIS Directorate General in identifying the handbooks and for guiding the development of the work. Under the first project, the Committee has identified several subjects for preparing explanatory handbooks/compilations covering appropriate Indian Standards/Codes/Specifications which include the following :

***Handbooks Published**

1. Design Aids for Reinforced Concrete to IS 456 : 1976 (SP 16 : 1980)
2. Handbook on Masonry Design and Construction (*first revision*) (SP 20 : 1991)
3. Summaries of Indian Standards for Building Materials (SP 21 : 1983)
4. Explanatory Handbook on Codes of Earthquake Engineering (IS 1893 : 1975 and IS 4326 : 1976) (SP 22 : 1982)
5. Handbook on Concrete Mixes (SP 23 : 1982)
6. Explanatory Handbook on Indian Standard Code of Practice for Plain and Reinforced Concrete (IS 456 : 1978) (SP 24 : 1983)
7. Handbook on Causes and Prevention of Cracks in Buildings (SP 25 : 1984)
8. Handbook on Functional Requirements of Industrial Buildings (Lighting and Ventilation) (SP 32 : 1986)
9. Handbook on Timber Engineering (SP 33 : 1986)
10. Hankbook on Concrete Reinforcement and Detailing (SP 34 : 1987)
11. Handbook on Water Supply and Drainage with Special Emphasis on Plumbing (SP 35 : 1987)
12. Handbook on Typified Designs for Structures with Steel Roof Trusses (with and without Cranes) (based on IS Codes) (SP 38 : 1987)
13. Handbook on Structures with Steel Portal Frames (without Cranes) (SP 40 : 1987)

* Handbooks published are available for sale from BIS Headquarters, and from all Branches and Regional Offices of BIS.

14. Handbook on Functional Requirements of Buildings (Other than Industrial Buildings) (SP 41 : 1987)
15. Handbook on structures with Reinforced Concrete Portal Frames (without Cranes) (SP 43 : 1987)
16. Handbook on Structures with Steel Lattice Portal Frames (without Cranes) (SP 47 : 1987)
17. Handbook on Building Construction Practices (Other than Electrical Services) (SP 62 : 1997)
18. Handbook on Construction Safety Practices (SP 70 : 2001)

This Handbook has been written with a view to provide detailed background information on the provision of Indian Standard on Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures : Part 3 Wind Loads [IS 875 (Part 3) : 1987] and also the use of standard for arriving at the wind loads on buildings and structures while evaluating their structural safety. The Handbook will serve as a guide for all those engaged in the structural design of wind sensitive buildings and structures due to their shape, slenderness, flexibility, size and lightness.

It may be noted that the Handbook does not form part of any Indian Standard on the subject and does not have the status of an Indian Standard. Wherever, if there is any dispute about the interpretation or opinion expressed in this Handbook, the provisions of the codes only shall apply; the provisions of this Hankbook should be considered as only supplementary and informative.

The Handbook is based on the first draft prepared by Shri S.K. Agarwal, Head, Wind Engineering, SERC, Ghaziabad. The draft Handbook was circulated for review to STUP Consultants Ltd, Mumbai; National Council for Cement and Building Materials, New Delhi; Central Public Works Department, New Delhi; Indian Institute of Technology, Chennai; Indian Institute of Technology, Kanpur; Structural Engineering Research Centre, Chennai; M.N. Dastur & Co Ltd, Kolkata; Meteorological Office, Pune; Indian Institute of Science, Bangalore; University of Roorkee, Roorkee; Tandon Consultants Pvt Ltd, New Delhi; Central Electricity Authority, New Delhi; Indian Institute of Technology, Kharagpur; Indian Institute of Technology, New Delhi; Engineers India Ltd, New Delhi; N.T.P.C., Noida; Ministry of Railways (RDSO), Lucknow; Department of Space, Bangalore; Nuclear Power Corporation, Mumbai; Moti Lal Nehru Regional Engineering College, Allahabad; Vakil Mehta Seth, Ahmedabad; Anna University, Chennai; UNITECH, New Delhi; Consulting Engineering Services (I) Pvt Ltd, New Delhi; Tata Consulting Engineers, Mumbai; Housing & Urban Development Corporation, New Delhi; National Institute of Construction Management and Research, Mumbai; Howe India (Pvt) Ltd, New Delhi; Army Headquarters, New Delhi; Institution of Engineers, New Delhi; Gammons India Ltd, Mumbai; Central Building Research Institute, Roorkee; Department of Science & Technology, New Delhi; Metallurgical & Engineering Consultants (India) Ltd, Ranchi; Planning Commission, New Delhi; and views expressed by them were taken into consideration while finalizing the Handbook.

Contents

0 INTRODUCTION

0.1 Background

A large number of structures that are being constructed at present tend to be wind-sensitive because of their shapes, slenderness, flexibility, size and lightness. Added to these are the use of a variety of materials which are stressed to much higher percentage of their ultimate strength than in earlier days because of better assurance of the quality of materials. In the social environment that is developing world over, the ancient philosophy of accepting continuing disasters due to wind as ordained by 'fate' and Gods is giving place to demands for economical wind resistant designs. These factors have demanded a more realistic, if not, precise definition of wind loading. Updating of some International Codes of practice, notably the British, Australian, Canadian, American and French has been effected fairly frequently over the last two decades and the present versions incorporate most of the advances made in understanding the wind characteristics and its effect on structures. The new discoveries are such that it is clear that mere issue of amendments to the earlier wind Code IS 875 : 1964 will not be justifiable. The recently issued wind code 'Code of practice for design loads (other than earthquake) for buildings and structures' IS 875 (Part 3) : 1987 differs in many ways from the previous Code first issued in 1964 and attempts not only to rectify the shortfalls of the 1964 code but incorporates recent knowledge of wind effects on structures. The height up to which velocities are given has now been raised to 500 m and the loadings on as many of the commonly encountered buildings and structures, for which there are no other Indian Standards, have been included. Although not explicitly stated, the code recognizes the fact that most of the high winds in India occur due to short duration rotating winds like tropical cyclones along the Coasts or Tornadoes elsewhere, and nearly rectilinear winds of short duration like thunderstorms at many places. In this respect, the high wind loading conditions in India are different from those of temperate zone countries like Europe and Canada. Much of the random response theories, which have been adopted in European/U.S. or Australian Codes are based on these 'fully developed pressure winds' or 'pressure wind' conditions and strictly cannot be applied in most parts of India. But their judicious use, in the absence of proper theories applicable to cyclones, tornadoes and thunderstorms will give adequate safety margins and this is what the present IS 875 (Part 3) : 1987 attempts to do.

In India, success in satisfactory codification of wind loading on structures has remained elusive so far. In most cases, codification has followed, not preceded structural failures or distress. The wind maps given in the 1964 version of the code had been prepared mainly on the basis of extreme value wind data from storms which approached or crossed the Indian coasts during 1890 to 1960, together with the wind data available from about 10-12 continuously recording Dynes Pressure Anemograph (DPA) stations which existed at that time, to get an overall picture for the country. However 3-cup anemometer readings were not much used in the preparation of wind maps because much of such data were synoptic.

The height of DPA instruments varied from 10 m to 30 m at different places. Therefore only one extreme value of wind was given up to 30 m height from ground level without any variation in-between. Further, 1/10th power law had been adopted regardless of terrain conditions, for indicating variation of wind speed with height from 30 m to 150 m, for which there was no supportive evidence. The code gave two wind pressure maps (one giving winds of shorter duration < 5 minutes and another excluding winds of shorter duration) and there was no clear guidance for using either or both of them.

With the publication of the recent revised wind code, IS 875 (Part 3) : 1987, an attempt has been made to remove these deficiencies and provide to the Indian structural engineer adequate guidelines for arriving at more rational wind loading for design purposes.

0.2 Nature of Wind

0.2.1 Wind means the motion of air in the atmosphere. The response of structures to wind depends on the characterstics of the wind. From the point of view of assessing wind load, it is convenient to divide the wind into two categories: 'Rotating and Non-rotating' winds. Rotating winds are caused by tropical cyclones and tornadoes. The wind speeds caused by these may exceed 200 km/h. The duration of such winds at any location varies from 2 to 5 minutes in the case of tornadoes and thunderstorms and about 3 to 4 days in case of tropical cyclones. Non-rotating winds are caused by differential pressures and thus move in preferred direction. These are also called 'Pressure System' winds and when they persist for distances like 50-100 km, are termed as 'Fully developed pressure system winds'. The intensity of such winds is usually given in Beaufort Scale as in Table 1 (other scales for particular types of winds like cyclones have also been proposed). Thunderstorms are rectilinear winds of high speed lasting only a few minutes and start as a strong vertical downdraft, from clouds which spread laterally on reaching the ground. They last for 2 to 5 minutes and wind speed can exceed 200 km/h.

0.2.2 *Gradient Winds*

Winds are primarily caused by the differences in temperature over the earth's surface because the intensity of the sun's radiation received at different latitudes varies, and land areas heat and cool quicker than the sea. Ocean currents also affect the temperature distribution by transporting heat from one part of earth to another. The differences in temperature give rise to the gradients of pressure which set the air in motion. Near the earth's surface, the motion is opposed, and the wind speed reduced, by the surface friction. At the surface, the wind speed reduces to zero and then begins to increase with height, and at some height, known as the gradient height, the motion may be considered to be free of the earth's frictional influence and will attain its 'gradient velocity'. This velocity depends on the pressure variation at that height in the plane parallel to the earth's surface. The gradient height increases with the roughness of the terrain on the earth's surface and is taken to vary from about 300 m for flat ground to about 550 m for very rough terrain, in the Indian and a few other national codes. Although, the gradient height is confused in many codes to be the same as the edge of the earth's (planetary) boundary layer, the two heights are not identical. True gradient height lies well above the edge of the earth's boundary layer height. The Indian Code has adopted a carefully written definition to imply this difference.

0.2.3 *Variation of Mean Wind Speed with Height*

At the earth's surface the wind speed is zero and, in conditions of neutral stability, there is a continuous increase of mean wind speed from the ground to the gradient height. A number of laws for defining this wind speed profile have been suggested, the most widely used of which is the logarithmic profile. The empirical power law profile is also used in many codes.

0.2.3.1 *Logarithmic profile*

The logarithmic profile of the variation of mean wind speed with height is given by

$$V_z/V^* = 1/k.\ \log_e z/z_0 \qquad ...(1)$$

where

V_z = design wind velocity at height z,

k = von Karman's constant with a numerical value of 0.4,

z = height above ground

z_0 = is the surface roughness parameter,

V^* = the friction velocity defined as $\sqrt{\tau_0/\rho}$,

τ_0 = the skin friction force on the wall, and

ρ = the density of air.

0.2.3.2 *Power law*

Another representation for the mean wind speed profile is the simple power law expression:

$$V_z/V_R = (z/z_R)^{\alpha} \qquad ...(2)$$

where

V_R = the speed at a reference height Z_R, usually taken as the standard meteorological height of 10 m, and

α = a constant for a particular site and terrain.

0.2.4 *Nature of Wind Flow Past Bodies*

0.2.4.1 The nature of flow of a fluid past a body resting on a surface depends on the conditions of the surface, the shape of the body, its height, velocity of fluid flow and many other factors. The flow may be

Table 1 Beaufort Scale for Wind Speeds
(*Clause* 0.2.1)

Beaufort No.	Description of Wind	Speed m/s	Description of Wind Effects
0	Calm	≤ 0.4	No noticeable wind
1	Light airs	0.4-1.5	Barely noticeable wind
2	Light breeze	1.6-3.3	Wind felt on face
3	Gentle breeze	3.4-5.4	Wind extends light flag, hair is disturbed; clothing flaps
4	Moderate breeze	5.5-7.9	Wind raises dust, dry soil, loose paper, hair disarranged
5	Fresh breeze	8.0-10.7	Force of wind felt on body, drifting snow becomes air borne; limit of agreeable wind on land
6	Strong breeze	10.8-13.8	Umbrellas used with difficulty; hair blown straight; difficulty in walking steadily; wind noise on ears unpleasant; beginning of blizzards (snow flows in air)
7	Moderate gale	13.9-17.1	Inconvenience felt when walking
8	Fresh gale	17.2-20.7	Progress in walking difficult; maintaining balance in gusts very difficult
9	Strong gale	20.8-24.4	People blown over by gusts
10	Whole gale	24.5-28.5	Small trees uprooted
11	Storm	28.6-34.0	Wide spread damage
12	Hurricane	> 34.0	Failure of ill-designed, structures, like lamp posts, GI sheets to major failures

what is called 'streamlined' or 'laminar' flow or what is called 'turbulent' or 'unsteady flow of random nature'. A streamline is defined as a line drawn through a moving fluid such that the velocity vector is tangential to it. The streamlines coincide with the paths of the particles in steady laminar fluid motion but not when the fluid motion is turbulent. In turbulent fluid motion, the velocity and direction of the particles of small masses of fluid fluctuate in time, mostly in a random and chaotic manner. In uniform flow each streamline has the same constant velocity, but in flows of viscous fluids near a solid boundary, the velocity at the boundary is reduced to zero by the effects of viscosity. Thus the velocity along the streamlines increases with distance of the streamline from the solid boundary until a final unretarded velocity is attained. The region of retarded fluid, which is indicated by the broken lines in Fig. 1 is referred to as the boundary layer. The flow over the surface within the boundary layer may be laminar at an upstream position but transition to turbulent flow may take place at some position as the flow proceeds downstream along the surface. This transition takes place over a distance along the flow direction. If the flow is subject to an adverse gradient, that is a pressure gradient opposing it (such as may be induced by flow over a concave surface, or other circumstance in which it is forced or allowed to expand), its velocity throughout the boundary layer is further retarded and at some position along the surface may be reduced to zero slightly above the surface. Then the opposing pressure will drive the fluid in opposite direction (reversed flow). The boundary layer flow becomes detached from the surface and large eddies are formed and are discharged into the region behind the body called the wake. These eddies are usually discharged either randomly or at fairly regular intervals, depending on the velocity of the fluid and the dimensions and shape of the body. The energy carried downstream by these eddies reduces the pressure in the wake and hence the pressures acting on the body surfaces in contact with it, and this reduction of pressure on the body's leeward side accounts for a large proportion of the drag force. Pressure forces act normal to the surface, and integrating the components of pressure forces over the whole surface of the body in-line and perpendicular to the fluid direction results in alongwind (or drag) and acrosswind (or lift) forces. These are additional to the skin-friction drag, which is caused by the viscosity of the fluid and acts tangentially to the surface. The drag resulting from the pressure distribution is referred to as the pressure drag, and is kept to a minimum in shapes used in aircraft wings called aerofoil sections, which are designed to avoid flow separation.

Except for surfaces with pronounced protuberances that may fix the positions where the flow separates, the positions of transition from laminar to turbulent flow and of flow separation from the surface are dependent on the Reynolds number (R_e) of the flow,

$$R_e = \rho Vl/\mu = Vl/\nu \qquad ...(3)$$

where

μ and ν are, respectively, the dynamic and kinematic viscosities of the fluid and l is a typical length.

The Reynolds number is a measure of the ratio of the inertia forces (that is, force due to acceleration of fluid particles) to viscous forces in a fluid.

0.2.5 *Flow Patterns*

0.2.5.1 *Around circular cylinders*

We will first consider flow past a structure of circular cross-section, since this shape is used in many tall and even short structures. The flow patterns around a circular cylinder are sketched in Fig. 2, and show the marked influence of Reynolds number (R_e). In the subcritical flow regime Fig. 2(a) the boundary layer

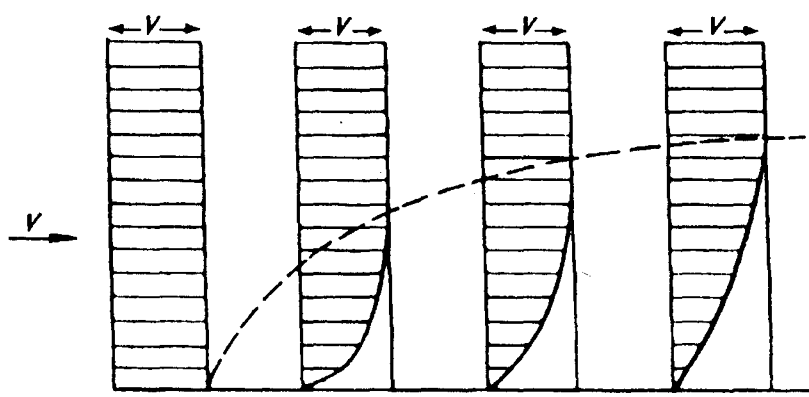
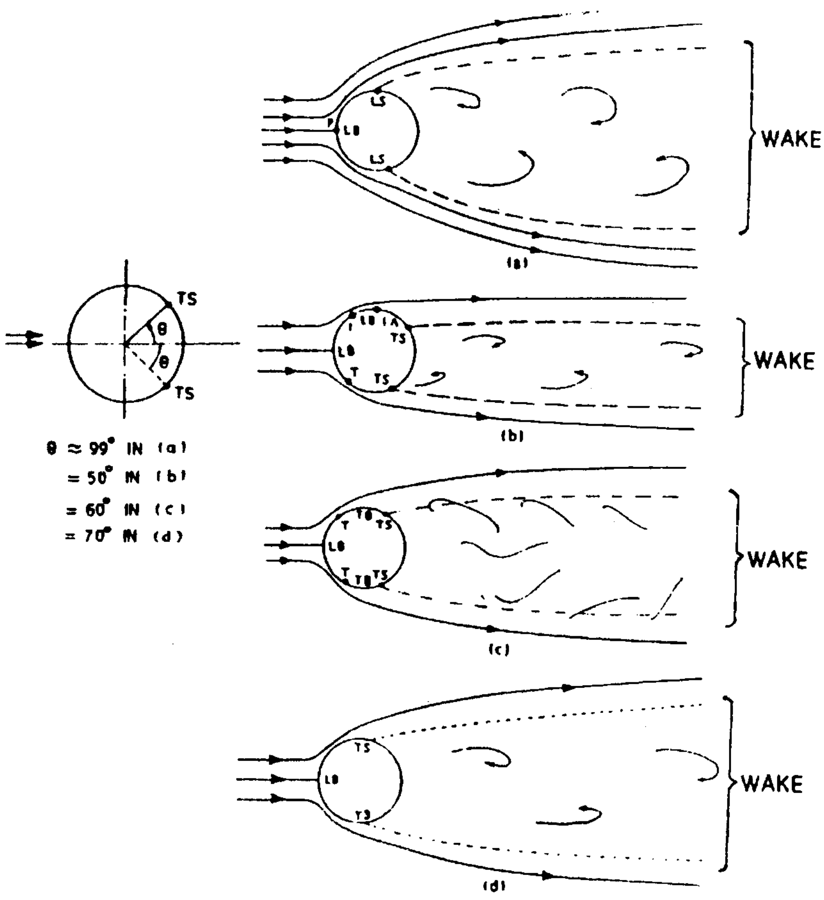


FIG. 1 GROWTH OF A BOUNDARY LAYER IN A FAVOURABLE PRESSURE GRADIENT

remains laminar up to the points of separation, which occur at about 80° from the stagnation point P. Vortices are formed regularly and alternately from each side of the cylinder and are shed into the wake. The width of the wake, and hence the value of the drag coefficient C_d, is greatest in the subcritical flow regime. When the surface is smooth, at about a $R_e \approx 2 \times 10^5$, the laminar boundary layer becomes turbulent at a forward position on the cylinder surface. The separation point shifts more or less suddenly to an angle of about 120° to 130°. In this critical regime Fig. 2(b) C_d falls rapidly as the Reynolds number increases to 5×10^5 when its lowest value is reached. Thereafter, increase of R_e brings the flow into the supercritical regime Fig. 2(c) with an increase in the width of the wake and an increase in C_d. The periodic vortex shedding in this regime is both weak and random. The increase in C_d continues until $R_e \approx 3 \times 10^6$ and the position of the separation point stabilizes at about 109°. In this transcritical (sometimes, perhaps more correctly, referred to as hypercritical) regime Fig. 2(d) the value of C_d decreases slowly with increase of R_e and the wake contains peak energy at frequencies higher than in the subcritical regime vortex shedding frequency. The flow patterns of Fig. 2 are for stationary smooth-surface cylinders in a smooth approach flow; they are modified by surface roughness, by turbulence of the incident wind, and by motion of the cylinder and height to diameter ratio of the cylinder.



LB	=	Laminar Boundary Layer
T	=	Transition from Laminar to Turbulent Boundary Layer
TA	=	Re-attachment of Turbulent Boundary Layer
LS, TS	=	Separation of Laminar and Turbulent Boundaries Respectively
TB	=	Turbulent Boundary Layer

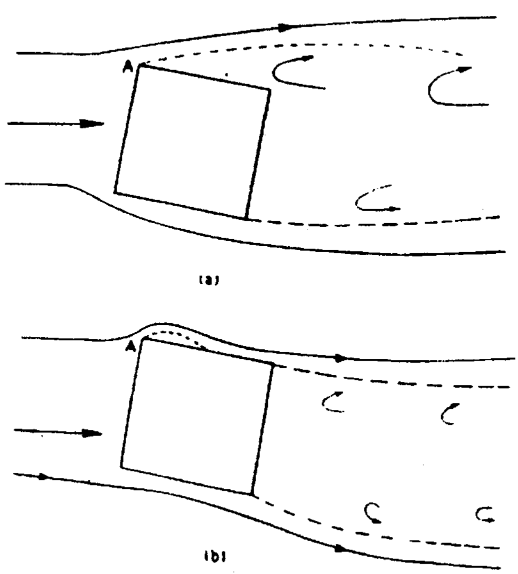
NOTE — Broken line indicates boundary of separated flow.

(a) Subcritical, $300 < R_e < 2 \times 10^5$, $C_d = 1.2$, $S_r = 0.2$; (b) Critical, $2 \times 10^5 < R_e < 5 \times 10^5$, $1.2 > C_d > 0.3$, $0.2 < S_r < 0.5$; (c) Super-critical, $5 \times 10^5 < R_e < 3 \times 10^6$, $0.5 \leq C_d \leq 0.7$, $S_r = 0.5$; (d) Transcritical (Hypercritical), $R_e > 3 \times 10^6$, $C_d = 0.7$, $S_r = 0.27$, $R_e = Vd/v$, where d is the diameter of the cylinder

FIG. 2 SKETCHES OF TWO-DIMENSIONAL AIRFLOW PATTERNS AROUND A CIRCULAR CYLINDER

0.2.5.2 *Around other bluff bodies*

A body is said to be a bluff body if the ratio of its dimension parallel to the flow to its dimension perpendicular to the flow does not differ by more than about 6 and whose drag coefficient based on the smaller dimension exceeds about 0.1. Circular cylinder, square and rectangular prisms, etc, are examples. As an example, the flow patterns around a long square-section prism are sketched in Fig. 3. The patterns, and hence the values of the drag coefficient C_d can be considered independent of R_e if R_e is > about 10^4 because the positions of the points of separation of the flow from the body are fixed by the sharp corners. They are, however, influenced by the turbulence of the approaching stream, the effect of which may be to cause the flow which has become detached at a windward corner (A) to reattach to the side, until finally separated at the leeward corner. Thus in turbulent flows the width of the wake is reduced compared to that in smooth flow, with a corresponding reduction in the value of C_d.



(a) Smooth Incident Flow, C_d = 2.2; (b) Turbulent Incident Flow, C_d = 1.6

FIG. 3 SKETCHES OF AIRFLOW PATTERNS AROUND PRISMS OF SQUARE SECTION WITH SHARP CORNERS

0.2.5.3 *Over buildings*

Figure 4 shows sketches of typical flow patterns around buildings, most of which can be classed as bluff bodies.

0.2.6 *Wind Effects on Structures*

Wind effects on structures can be classified as 'Static' and 'Dynamic'. Strictly speaking, wind effects are only dynamic in nature but in most of the codes these dynamic effects are expressed in terms of an equivalent static load. Static wind effect primarily causes elastic bending and twisting of structure. The dynamic effects of wind are either periodic forces such as due to Vortex Shedding, Flutter, Galloping and Ovalling or non-periodic such as turbulent bufetting (*see* Table 2).

0.3 Shortfalls of the 1964 Code

0.3.1 The Code, which was published more than 35 years ago has provided reasonably good guidance to structural engineers for the design of most of the simple structures. In most cases, its provisions have been safe, if not conservative.

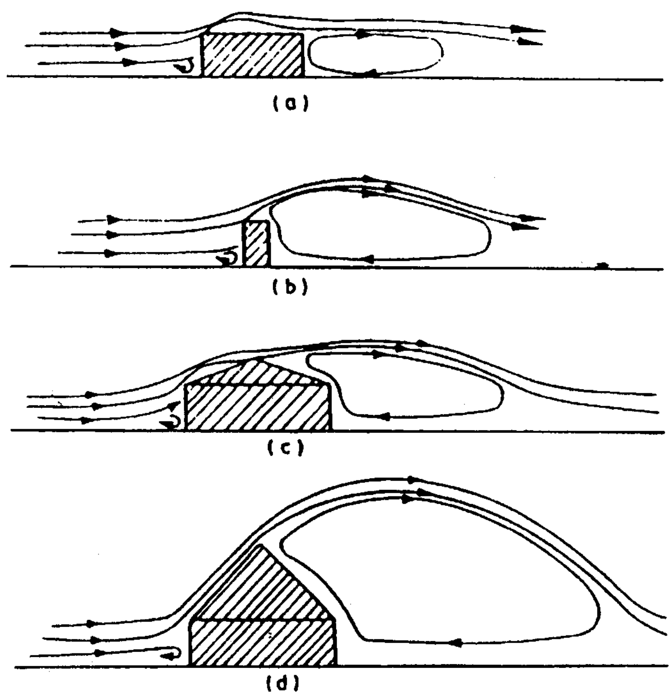
A look at some of the recent International Codes however, clearly indicates the significant advances made in wind engineering in the last few years. Not only is there an improved knowledge of the characteristics of wind, but there have been new trends, innovations and requirements of structural design, which demand a more accurate definition of wind forces. Viewed from this background, the shortfalls of IS 875 : 1964 must be understood in today's context. In the revision of the Code, an attempt has been made to overcome the obvious shortfalls and update the information to present-day knowledge of wind engineering. A serious attempt has been made to ensure that the new phenomenon introduced or/and discussed in the 1987 revision are free from ambiguities and not significantly beyond the existing knowledge base in the country. Some of the more important shortfalls that had surfaced over the years of the Code of 1964, which provided the main impetus to the revision of the 1964 Code are briefly summarized below:

0.3.2 *Wind Zoning*

Of interest to structural designers are wind pressures of short averaging time and gusts with averaging time of say 3 to 15 seconds. The wind map zoning in the 1964 Code represented the state of knowledge of the wind climate and measurements taken earlier over some 30 years. Several studies have since been published based on increasingly available data. It was abundantly clear that the earlier wind zoning either over-estimated or under-estimated the occurrence of extreme wind speeds in different parts of the country. Thus the revision of wind map zoning was long overdue.

0.3.3 *Risk Level*

In the 1964 Code, no indication was available about the return period. Experiences all over the world, by and large, has favoured designing normal structures for a life and return period of 50 years as these criteria, seem to yield safe and economical loads for normal structures. For wind-sensitive structures and for



(a) Flat roof $H/b = 0.5$, Flow Re-attaches; (b) Flat Roof $H/b = 2$, No Flow Re-attachment; (c) Low-pitch Roof, Flow Re-attaches; (d) High Pitch roof, No Flow Re-attachment

FIG. 4 SKETCHES OF FLOW PAST LOW-RISE BUILDINGS SHOWING THE EFFECTS OF ALONG-WIND WIDTH AND OF ROOF PITCH ON THE SEPARATED REGION IN THE WAKE OF THE BUILDING

Table 2 Wind Effects on Structures
(*Clause* 0.2.6)

- Wind Effects
  - Static
    - Pressure & Forces (drag, lift, moment)
    - Static Instability
  - Dynamic
    - Random Response
      - Due to Atmospheric Turbulence
      - Due to Buffetting by Turbulent Wake of other Structures
    - Periodic Response
      - Vortex Excitation (Slender Structures) like Chimneys, Cooling Towers, Stacks, Bridge Decks, etc
      - Buffetting Due to Periodic Wake of other Structures
      - Ovalling (Thin Walled Structures) like Chimneys, Cooling Towers, etc
      - Galloping (Non-circular Chimneys, Conductors, Cable Bridge, Towers, etc)
      - Flutter (Bridge Decks, Antenna Dish)

structures with post-disaster functions, the return period is often increased to 100 years. As a matter of fact, for structures of exceptional importance the return period could logically be even higher to reduce the risk levels still further. These issues are further discussed in **3.3**.

0.3.4 *Wind Velocity Measurement*

The wind instruments used by meteorologists are generally calibrated in terms of wind velocity at standard atmospheric conditions of 15 °C and 760 mm mercury pressure and measure the wind velocity with an averaging time of about 3 seconds . The wind pressure is evaluated by the equation $p = KV^2$, where K is a constant. The Code of 1964 gives wind pressure and these were arrived at from the velocities by using a value of K which is higher by 25 percent as compared to other International Codes.

0.3.5 *Wind Variation with Height and Terrain*

The variation of wind velocity with height and character of surrounding terrain has important implications in structural design. The 1964 Code specified constant pressures up to a height of 30 m and thereafter, a variation corresponding to the following equation was given:

$$P_z = P_{30}\,(z/30)^{\alpha} \qquad \text{...(4)}$$

where

P_{30} = basic wind pressure which is constant up to height of 30 m;

z = height in m above ground of point under consideration; and

α = power index, with a value of 0.2.

Whenever such power laws are used, the generally accepted norm in International Codes is to take a constant velocity up to 10 m height and thereafter a variation given by the following equation to obtain the velocity profile:

$$V_z = V_{10}\,(z/10)^{\alpha} \qquad \text{...(5)}$$

where

V_{10} = velocity at 10 m above ground;

z = height in m of point above ground under consideration; and

α = power law index whose value varies from 0.07 to 0.35 depending upon the terrain and averaging time; larger indices correspond to increasing roughness of terrain and larger averaging times.

In recent times, the logarithmic law is being used more often, mainly due to its better theoretical backing. Considering that the power law index of IS 875 : 1964 was applicable to pressures, the corresponding index for velocity would be 0.1. While structures of low height may not be affected significantly by the low value of the power index, the 1964 Code would yield pressures which are under-estimates in the upper regions of tall structures where more precise values are needed. The Code did not indicate variations with terrain roughness, and was silent about wind pressures above 150 m from ground. With tall structures already reaching heights of 300 m in the country and still taller structures on the drawing board, the short falls of IS 875 : 1964 became particularly evident for tall structures.

0.3.6 *Dynamic Analysis*

For tall, long span and slender structures a 'dynamic analysis' of the structure is essential. Wind gusts cause fluctuating forces on the structure which induce large dynamic motions, including oscillations. The severity of the dynamic motions and/or wind-induced oscillations depends on the natural frequency of vibration and the damping of the structure. The motions are induced in both 'along-wind' direction as well as 'across-wind' direction. The 'along-wind' response of the structure is accounted for by a magnification factor (often called the 'gust factor') applied to the static or direct forces. The 'across-wind' response requires a separate dynamic analysis. The 1964 Code did not give any guidance on these aspects of structural design of tall structures.

0.3.7 *Size of Structure*

The 1964 Code did not indicate any correlation between the structure size and wind loading. The magnitude of the load imparted on a structure by a gust depends, to a significant extent, on the size of the structure. The reason is that the spatial extent of a gust of wind in any directions, having the same velocity as that specified in the Code is small-being about 25 m to 150 m at any instant of time. Thus, a large structure will not experience the highest wind specified at all points on it at the same instant and hence the average total load on the structure will be less than the value obtained by assuming the highest specified wind over its entire extent. Allowance for this needs to be made, although locally the load will be that due to the highest wind.

0.3.8 *Shape of Structure*

Structures can be of many shapes, particularly when we combine the whole gamut of possibilities in concrete and structural steel. The 'direct or static' wind force on a structure is given by the equation $F = SAP$, where S is the shape factor; A is the projected area in the plane perpendicular to the wind direction; and P is the wind pressure. The 1964 Code gave values of the shape factor for the basic shapes (circular, octagon and square), but this information needed to be extended to other shapes for various wind directions. For steel structures of different plan shapes, shielding configurations and solidity ratios, no guidance was available. Therefore it became essential for the designer to either consult other International Codes or take advice from specialists.

0.3.9 *Pressure Coefficients*

All over the world, the unsatisfactory performance of relatively light weight low rise buildings such as the thatched houses in rural India or houses with sheet roofing, led to updating of pressure coefficients (internal and external) for various configurations of buildings, wall openings and wind directions. Additionally, a need was felt to define higher local external negative coefficients at certain locations to cater to concentrations near edges of walls and roofs, where the failures are primarily initiated. It is recognized that considerable research is still needed in this area.

1 SCOPE

The aim of this Handbook is to provide detailed background information on the provisions of IS 875 (Part 3) : 1987 and the use of the standard for estimating wind loads on structures. The emphasis has been on the identification of source material used, philosophical ideas behind the changes in 1987 edition and guidance to the use of the standard. Clauses **0.1** and **0.2** of this Handbook contains background information, philosophical ideas, nature of wind and wind effects on structures. Clause **0.3** brings out the shortfalls of the earlier standard (IS 875 : 1964), *vis-a-vis* wind zoning, risk level, wind velocity and its variation with height, dynamic analysis, effect of structure size and shape. Clause **2** explains the various terminology used in the standard. Clause **3** discusses the main features of the revised Code like new wind zoning map, estimation of design wind speed, concept of return period, effects of terrain roughness and structure size, local topography, correct velocity-pressure relationship. In Clause **4** wind pressure and forces on buildings/structures are discussed in light of pressure coefficient method. Details of force coefficient method are given in **4.3** while dynamic effects like along-wind and across-wind responses are explained in **5** and **6**. Requirements of wind tunnel studies are detailed in **7**. Clause **8** conducts the user to the various provisions of the standard with the help of flow diagrams. Clause **9** contains the methodology for the estimation of wind loads on low rise buildings, with the help of examples. Clause **9.3** is devoted to the estimation of dynamic effects of wind using gust factor method and contains typical solved examples of structures.

NOTE— All Clauses, Figures and Tables in Square Brackets in the Handbook Refer to Clause Numbers, Figure Numbers and Table Numbers of IS 875 (Part 3) : 1987, respectively

2 TERMINOLOGY

2.1 Angle of Attack [3.1.1]

Angle between the direction of wind and the reference axis is known as the angle of attack. As shown in Fig. 5(a), angle 'θ' is angle of attack, between the direction of wind and reference axis (horizontal line).

2.2 Breadth [3.1.2]

Horizontal dimension of the building measured normal to the direction of wind, is termed as the breadth of the structure. As shown in Fig. 5(b), breadth = *AB* for wind along the longitudinal reference axis and *BB'* for wind perpendicular to it. For other wind directions, such as at angle 'θ', it is the maximum width seen along the wind such as the projection of *AB'* on a plane perpendicular to the direction of wind. In Fig. 5(b), this is *AE*.

2.3 Depth [3.1.3]

Horizontal dimension of the building measured in the direction of wind is the depth of the building. As shown in Fig. 5(b), depth = *AA'* for wind along the reference axis and *BA* for wind perpendicular to the reference axis. For other wind directions, such as at angle 'θ', it is the maximum length from the first point on the body which meets the wind to the point where a line from this point along-wind meets a boundary of the body such as *BD*.

2.4 Developed Height [3.1.4]

The velocity increases up to certain height from the ground when wind passes from one terrain category to another terrain category. After covering a certain distance in the new terrain, the velocity profile does not change any more and is said to be fully developed. Before becoming fully developed, the velocity profile will have a mixed character of the fully developed profiles of the upstream and downstream terrains. In this intermediate region, the height up to which the velocity profile changes in the new terrain is known as the Developed Height. [Table 3] gives the values of the Developed Height as a function of the distance for the four terrain categories. Methodology for estimation of the developed height and for using the same has been illustrated in the solved Examples given in **3.5**.

2.5 Effective Frontal Area [3.1.5]

As given in Fig. 5(a) for angle of attack 'θ' = 0, effective frontal area is *ABCD*, which is the projected area of the structure normal to the direction of the wind. For 'θ' not equal to zero, the frontal area is *ACEF*.

2.6 Element Surface Area [3.1.6]

In Fig. 5(a), for angle of attack 'θ' = 0 at height z, area *PQRS* represents the element surface area of height Δz. In the element surface area pressure/force coefficients are to be taken constant.

2.7 Force and Moment Coefficients [3.1.7]

When a structure is immersed in flowing fluid like air, a force is exerted on it which depends on the nature of flow, type of structure and is expressed in terms of non-dimensional coefficient. This non-dimensional coefficient is termed as force-coefficient.

There can be components of force in the directions other than the direction of wind. The force component along the direction of wind is called drag force (now-a-days, more frequently as along-wind force) and the force components in the other normal directions are termed as lift force (or across-wind force) and side force (or transverse force). Likewise, the moment coefficient tending to bend the structure in the direction of wind is called the over turning moment, the coefficient tending to twist it about a vertical axis as torque and the coefficient in the transverse direction as 'sideways' moment coefficient.

2.8 Ground Roughness [3.1.8]

The obstructions like trees, buildings, structures, etc, on ground are termed as ground roughness. The ground roughness tends to retard the flow near ground because of momentum transfer between layers of wind. The velocity profile of wind is dependent on the ground roughness. Although the nature of ground roughness on the surface of the earth varies from flat and very smooth to very rough as in the city centres with tall buildings, it has been found convenient and adequate to categorize them into four fairly distinct categories.

2.9 Gradient Height [3.1.9]

When wind moves over large distances in a particular terrain, the velocity becomes constant above a certain height. The height above which the velocity is found to be constant and is not influenced by surface friction is called the Gradient Height. Implicitly, the Code identifies the gradient height as the height of the planetary boundary layer; the wording of the definition is obviously very carefully done, since the meteorologists' identify the gradient height as one at which there is a balance of Coriolis and pressure forces. The meteorologists' definition of gradient height puts it well above the edge of the atmospheric boundary layer. Thus the definition of gradient height used in the Code is not the same as the gradient height as understood by the meteorologists.

2.10 Pressure Coefficient [3.1.9]

The pressure exerted by wind at a point on the structure will be different from the pressure of wind far upstream of the structure, called 'free stream static pressure'. The ratio of the difference of the pressure at a point on the structure and static pressure of the incident wind to the design wind pressure is termed as Pressure Coefficient.

The concept of pressure coefficient is similar to that of force coefficient. The pressure coefficient at a point multiplied by design wind pressure yields the pressure at that point.

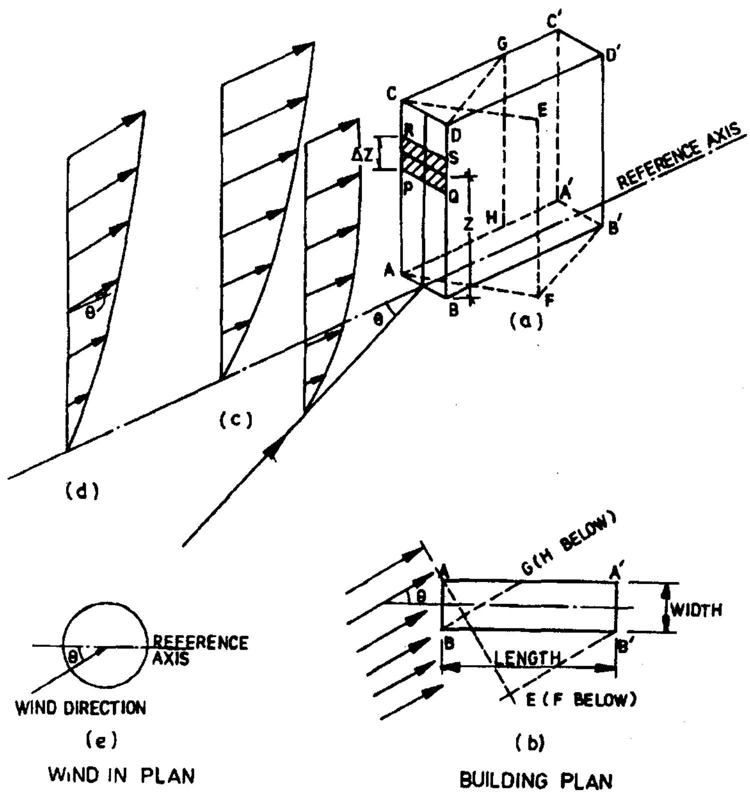


FIG. 5 WIND EFFECTS ON A BUILDING

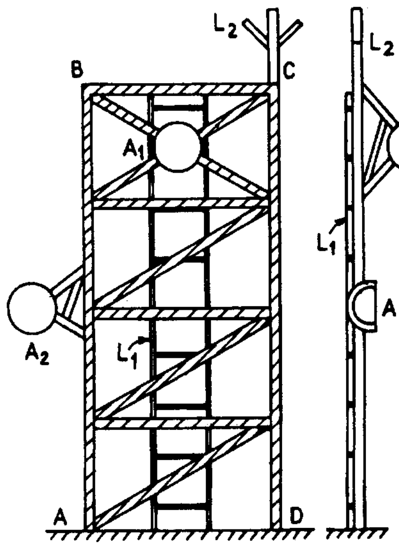
The pressure coefficient may vary with height or width. The pressure acting towards the surface of structure is to be taken as positive and pressure acting away from the surface of structure is to be taken as negative.

2.11 Suction [3.1.9]

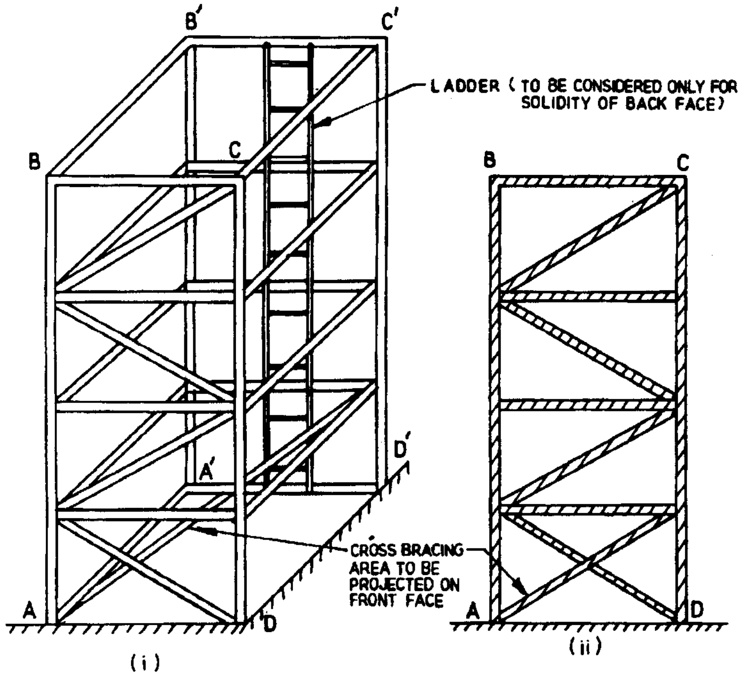
At some points on any body, pressure may fall below atmospheric pressure and is called 'suction'. Suction is the pressure acting away from the surface of structure; negative pressure is the same as suction pressure.

2.12 Solidity Ratio 'ϕ' [3.1.9]

In general, solidity ratio is the ratio of the sum of all the areas of each element of a framed structure normal to the wind to the area of the boundary enclosing the structure. Care must, however, be taken in applying this definition. In a plane frame, the boundary area is to be taken as that of the main frame only and the areas of appurtenances which are within the boundary [as Antenna A_1, or ladder L_1, in Fig. 6(a)] treated as part of the element of the frame. However, the area of the antenna or an appurtenance outside the boundary such as the Antenna A_2 or lightening arrester



6(a) Plane Frame



6(b) Space Frame

FIG. 6 SOLIDITY RATIO — PLANE AND SPACE FRAME

L_2 in Fig. 6(a) should not be taken into account either as part of the enclosed boundary or that of the frame in computing the solidity ratio. The loads on such appurtenances outside the enclosed boundary must be separately estimated and added to the load on the frame at the appropriate point. In Fig. 6(a), the area enclosed by *ABCD* is the boundary area and the area of shaded elements to be considered for estimating the solidity ratio is indicated. Similar principles apply in the case of space frames. In case of cross bracings, the projected area as seen from windward face shall be taken and added to other areas (shown shaded) to arrive at the total solidity ratio. If there are appurtenances within the enclosed volume (*ABCD A'B'C'D'*) [*see* Fig. 6(b)(i)], then their area should be projected on the nearest windward frame for estimating the frame solidity ratio Fig. 6(b)(ii). The solidity ratio of each frame should be found and appropriate shielding factor applied.

2.13 Fetch Length [3.1.9]

The distance over which wind has moved in a particular terrain category before approaching the structure is known as the Fetch Length. The velocity profile of wind changes continuously over the fetch length before stabilizing at the variation given in [Table 2]. The distance required for a stable profile to be formed is given in [Table 3]. Distance *AB* in Fig. 7 is the fetch length.

2.14 Terrain Category [3.1.9]

Buildings, vegetation, walls, trees and waves at sea contribute to the surface roughness and thus influence the local characterstics to which a structure may be exposed. The average ground roughness of large areas is termed as Terrain Category. Based on the average height of the ground roughness four representative terrain categories having fully developed velocity profiles are suggested **[5.3.2]** from the equivalent of a calm sea to inner city area dominated by tall buildings. Terrain category 1 (*TC*-1) applies to exposed open terrain with few or no obstructions and in which the average height of objects surroundings the structure is less than 1.5 m. Category 2 (*TC*-2) refers to open terrain with well scattered obstructions having heights generally between 1.5 to 10 m while Category 3 (*TC*-3) applies to terrain with numerous closely spaced obstructions having size of buildings/structures up to 10 m in height with or without a few isolated tall structures. Terrain Category 4 (*TC*-4) means terrain with numerous large high closely spaced obstructions. However, when fetch lengths are small, the velocity profile is not fully developed and suitable velocity profile with changes in terrain categories be considered **[5.3.2.4]**.

2.15 Velocity Profile [3.1.9]

The variation of wind speed with height is called the Velocity Profile. The velocity profile is dependent on the terrain category, zonal velocity and the fetch length. Figure 8 shows the velocity profile in the four terrain categories.

2.16 Topography [3.1.9]

Geographical features such as mountains, hills, escarpments, etc, of an area is known as the Topography of the area in which the structure is built and this affects the wind speed on and downstream of these features.

3 WIND SPEED AND PRESSURE [5]

3.1 Basic Wind Speed [5.2]

In 1960, there existed about 10-12 continuously recording DPA stations in the country compared to about 50 or so such stations at present. The instruments continuously record wind speed on Y-axis with time on X-axis and in addition, record the direction of wind speed. Most of these stations are located on the coastal belt.

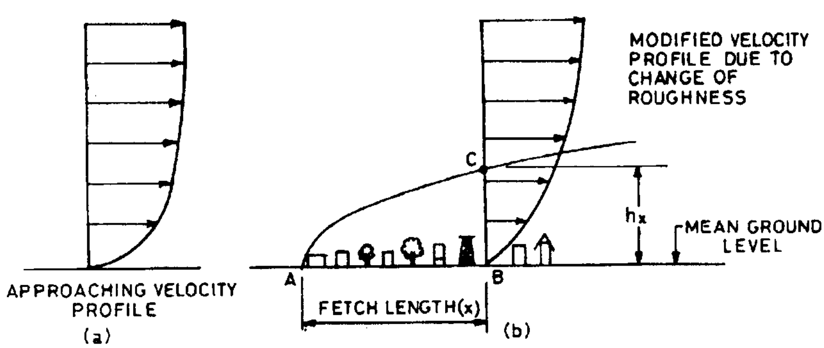


FIG. 7 EFFECT OF CHANGE IN TERRAIN CATEGORIES

The revised wind map is based mainly on the data from the anemograph records of 47 DPA stations for periods from 1948 to 1983. All these wind data have been published in India Meteorological Department publications, 'Indian Weather Review, Annual Summary Part-A' published annually from 1948 onwards. The basic approach followed for preparing the wind map is by extracting wind velocities, since analysis of structures in many cases depends on wind speed directly and also much of the available data on effects of terrain is in terms of wind speed. The use of wind speeds for statistical analysis is also in line with the internationally accepted practice. The peak values of annual maximum wind speeds (wind gustiness) for each of the 47 stations formed the data for statistical analysis to obtain the extreme wind speeds. In the statistical analysis of all the annual peaks at each of the above 47 stations in India, IMD used Fisher-Tippet Type-I (Gumbal) distribution. Before analyzing, recorded values were reduced to values at 10 m height above ground for the normalized terrain Category 2. Since the data for analyses came from DPA which has an averaging time of about 3 seconds, analysis gives gust velocity values averaged over 3 seconds. These values for 50 year return period and 50 year structure life have been given in [Fig. 1] for terrain Category 2. In addition to the use of data available from 47 DPA stations up to 1983, the following effects have been suitably incorporated in the preparation of the basic wind velocity map:

Orography — Orography has been considered in the zoning of wind velocity map since it was observed that continuous hills and mountains spread over a large area affect wind speed, for example regions demarcated by Vindhya mountains. The data spread seems to match the orography of the country.

Palghat Gap — Extreme winds observed during monsoon season over the southern peninsula due to the funneling effect of Palghat gap (reported wind speeds being up to 160 km/h) has been considered.

Cyclonic Storms — Cyclonic storms on east and west coasts are more intense over the sea than on the land. West coast storms are less intense except in Northern Sourashtra compared to east coast storms. Reports of failures of structures at Sriharikota, Gujarat coast, Paradeep and other places have been taken into account for evolving the reference speed in these regions. Storm speeds begin to decrease once they cross the coast and are normally taken to be effective up to a distance of about 60 km inland after striking the coast. However, in Bengal coast its effect has been observed up to 110 km inland. A region along the sea coast extending up to a distance of 60 km (and 110 km along Bengal coast) from the nearest coast line has been identified separately from the main land for wind zoning. In addition, it has also been decided to give the number of cyclones that have struck different sections of east and west coasts in the basic wind map for more sophisticated design load estimation.

Dust Storms — Dust storms associated with high local wind speeds which are short lived and are particularly intense over Rajasthan plains have been considered. Also Norwesters or Kal Baisaki winds over North Eastern plains during summer months have been accounted for. It has been suggested that the influence of wind speed off the coast up to a distance of about 200 km may be taken as 1.15 times the value on the nearest coast in the absence of definite wind data [5.5] as in Australia. The occurrence of a tornado is possible in virtually any part of India. They are particularly more severe in Northern parts of India. The recorded number of these tornadoes is too small to assign any frequency. The devastation caused by a tornado is due to exceptionally high winds about its periphery and the sudden reduction in atmospheric pressure at its centre resulting in a large outward pressure on the elements of the structure. The regional basic wind speeds do not include any allowance for tornadoes directly. It is not the usual practice to allow for the effect of tornadoes unless special requirements are called for as in the case of important structures such as satellite communica-

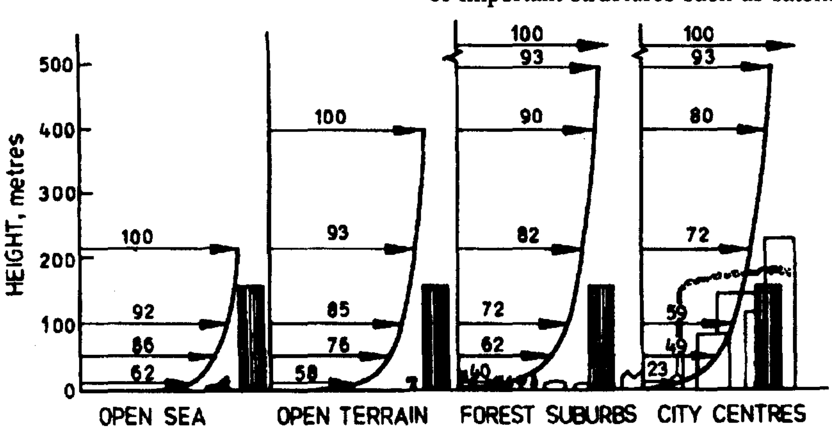


FIG. 8 VELOCITY PROFILE OVER DIFFERENT TERRAIN CATEGORIES

tion towers. The wind speed can now be obtained up to 500 m height above the ground surface. Design wind speed up to 10 m height from mean ground level is taken to be constant. Although not stated in the Code, turbulence level above 500 m is unlikely to be more than 6 percent. Basic wind speed map of India has been presented in the Code. The reference wind speed all over the country has been found to fall into six ranges of basic wind speeds, varying from 33 m/s to 55 m/s for a 50 year return period and a probability of exceedence level of 63 percent. The indicated wind speed characteristics are in line with other International Codes, namely short duration gusts averaged over a time interval of about 3 seconds applicable to 10 m height in 'open' terrain. The Code recognizes that there is not enough meteorological data to draw isotachs (lines joining places of equal velocity) on the wind map of India as seems to be the practice in some other International Codes.

The revised map, when compared to that of the earlier Code shows that in general, in the following areas of the country, the wind velocity was previously underestimated:

— North : North-eastern part of Jammu and Kashmir
— West : Coastal areas of Kutch in Gujarat
— East : Tripura and Mizoram

The wind map indicates updated supplementary information (1877 to 1982) relating to the total number of cyclonic storms. Occasionally, a user comes across a situation where the wind speed at a location on the boundary between two zones has to be obtained. If local wind data is not available, it is recommended that the higher speed of the adjacent zones be used for design.

3.2 Design Wind Speed [5.3]

The design wind speed is dependent on the geographical basic wind speed, return period, height above ground, structure size and local topography. The format of the equation of design wind speed adopted in the new Code is as follows:

$$V_z = V_b\ k_1\ k_2\ k_3 \qquad \text{...(6)}$$

where

V_z = design wind speed at height z in m/s,
V_b = basic wind speed of the wind zone,
k_1 = risk coefficient,
k_2 = terrain, height and structure size factor, and
k_3 = local topography factor.

3.3 Risk Coefficient [5.3.1]

There is sometimes confusion as to the meaning of return period and risk level. Return period (T in years) is the reciprocal of the probability that a given wind speed will be exceeded at least once in any one year. The word, 'period' sometimes causes confusion. 'Risk level', on the other hand means the probability that a given wind speed will be exceeded in a certain number of successive years which is usually the life of the structure. The coefficient k_1 is identified in the Code for return periods of 5, 25, 50 and 100 years alongwith recommendations regarding 'mean probable design life of structures' of various types [Table 1]. For general buildings and structures of permanent nature, the return periods have been recommended to be 50 years and a life of 50 years for the structure. For all temporary structures including temporary boundary walls, it is recommended that a life of 5 years be assumed and the appropriate multiplier k_1 as given in [Table 1] be used. The normal value of risk level recommended is 0.63 for structures whose life expectancy is equal to the return period. The factor k_1 has been used to assess the effect of different risk levels.

At first sight, a risk level of 0.63, which is the probability that the design wind speed is exceeded once in the life of say N years of the structure under the condition that the probability of the exceedence of the design wind in any one year is $1/N$, seems rather high. However experience in many parts of the world has shown that this amount of risk can be taken for normal structures. Although not given in the Code, the risk level can be found from the formula:

$$r = 1 - (1 - 1/T)_N \qquad \text{...(7)}$$

where

N = desired life of the structure in years,
T = return period in years, and
r = risk level.

The designer or owner must decide the amount of risk, he is prepared to take. Let us take one example.

Example. *Only 10% risk acceptable, life 30 years, in zone with wind speed of 39 m/s, Category 2, size of structure 30 m. To find the design wind speed.*

From Eqn (7), we get

$$1/T = 1 - (1 - r)^{1/N}$$
$$= 3.5 \times 10^{-3}$$

That is, this is the acceptable probability that the regional wind speed of 39 m/s will be exceeded in any one year during the 30 year life of the structure for the accepted risk. Using the above values of risk level, life of structure and the quantities A and B corresponding to 39 m/s wind speed in [Table 1], one obtains $k_1 = 1.165$.

Hence, the regional basic wind speed to be used for this structure should be $1.165 \times 39 = 45.44$ m/s. Use of Eqn. (7) is necessary if the return period T only is known from analysis of wind data and risk level has to be assessed.

3.4 Terrain, Height and Structure Size (k_2 factor) [5.3.2]

The coefficient k_2 has been indicated at different heights in a convenient tabular form [Table 2] which identifies four terrain categories (1, 2, 3 or 4) and three classes (that is, sizes) of structures (A, B or C). The k_2 factors are valid only up to the height at which they are specified. For example, the factor 1.17 in *TC*-2, class *A* structure applies only up to 50 m height. As stated in [Note 2 of Table 2], values between 1.12 and 1.17 may be linearly interpolated between heights of 40 m and 50 m or a constant value of 1.17 (the higher one) may be used between 40 m and 50 m. The coefficient k_2 has different values [Table 33] when dealing with wind speeds averaged over 1 hour, which are required for evaluating dynamic effects according to certain theories.

3.5 Fetch and Developed Height Relationship [5.3.2.4]

The revised Code had added the concept of fetch length and developed height for considering the effect of change of terrain category. In the following examples the methodology for considering this effect is explained with the help of illustrations.

Example 1. *A framed tower structure inside a big town at a distance of 0.5 km from the edge of the town; the terrain outside the town is flat open, the structures inside the town are closely spaced and of heights up to 10 m. Height of the tower 55 m. To find the design wind speeds.*

Solution: Here, one can take the terrain outside the town as of category 1 (*TC*-1) and the terrain inside the town to be of category 3 (*TC*-3). *This problem is a case of wind moving from a lower terrain category to a higher terrain category.* Figure 9 gives a sketch of the configuration.

In the above sketch, *AA* is the edge of *TC*-1 profile (say, where the factor as per [Table 2] is 1.35), B_1B_2 is the approach terrain of *TC*-1 and B_2B_3 is the town centre terrain of *TC*-3 [**5.3.2.1**(c)]. The tower is located at *C*, a distance of 0.5 km from the beginning of the town at B_2. In order to apply [**5.3.2.3**], one proceeds as follows:

At *C* draw the profile of multipliers as per [Table 2, Class C] (since the height of the structure is more than 50 m), corresponding to both the terrain categories 1 and 3. Thus *CDH* corresponds to *TC*-1 and *CEFG* to *TC*-3. [**5.3.2.4**] allows one to use two options. As per [**5.3.2.4** (b) (i)], one can use the multipliers of the lowest terrain category, which in this case is *TC*-1, that is, *CDHG*. The second possibility is to use the procedure of [Appendix B]. To use the procedure of [Appendix B], one first notes the height up to which the *TC*-3 profile has penetrated in the vertical direction in the terrain in which the structure is located. In this case, [Table 3] indicates that *TC*-3 has penetrated up to a height of 35 m at a distance of 0.5 km from B_2. Hence one takes the multiplier of *TC*-3 [Class C of Table 2] up to a height of 35 m (by interpolation). Above this height, the multipliers of *TC*-1 are to be used. At a height of 35 m, the multiplier corresponding to *TC*-3 is to be used when computing the design load on the tower, although the multipliers of both *TC*-1 and *TC*-3 are shown at this height in this sketch. The final design multipliers are shown in Table 3.

Table 3 Multipliers for a Structure in *TC*-3 with Upstream *TC*-1
(*Example* 1)

Height (m)	Multipliers as per [5.3.2.4 (b)(i)]	Multipliers as per [Appendix B]
10	0.99	0.82
15	1.03	0.87
20	1.06	0.91
30	1.09	0.96
35	1.10	1.04
50	1.14	1.14
55	1.15	1.15

NOTE— It is observed that the application of [**5.3.2.4** (b)(i)] leads to higher or conservative load.

Example 2. *A chimney of height 150 m in open country at a distance of 10 km from clusters of scattered small villages all round with houses of height less than 10 m. To find the wind speed multiplier profile.*

Solution: Since the terrain up to a distance of 10.0 km from the chimney is open, one may assume that the terrain is of category 1 up to 10 km from the chimney. In view of the location of clusters of small villages beyond 10 km, one can consider the approaching profile to chimney to be *TC*-2 from beyond 10.0 km and *TC*-1 for a distance of 10 km from the chimney. Figure 10 gives the sketch of the configuration. It may be observed that in this problem, the wind flows from a higher terrain category to a lower terrain category. In Fig. 10, *AA* is the edge of the *TC*-2 profile (say the line of multiplier 1.35), B_1B_2 is the approach terrain *TC*-2 and B_2B_3 the *TC*-1 in which the chimney is located. The chimney is located at *C*, which is located at a distance of 10 km from the beginning of *TC*-1, namely B_2. To obtain the appropriate multipliers, one can proceed as follows:

i) Assume that *TC*-1multiplier applies for the entire height, as per [**5.3.2.4** (b) (i)]
ii) Use the procedure of [Appendix B] as follows:

To use the procedure of [Appendix B], draw the variation of the multipliers of *TC*-2 and *TC*-1 at *C* with height. These are respectively *CEFG* and *CDH*. From [Table 3] of the Code, it is observed that the profile of *TC*-1 penetrates up to a height of 80 m, in a distance of 10 km. Thus one takes the *TC*-1 multipliers of Class C (since the maximum chimney dimension exceeds 50 m) up to a height of 80 m. This multiplier has an interpolated value of 1.17 at a height of 80 m. Above this height, one has to take the multipliers as per *TC*-2, Class C. But the multiplier as per *TC*-2, Class C has a value of 1.17 only at a height of 100 m as per [Table 2]. The recommendation of [Appendix B] is to take the same value of the multiplier as per the terrain of location (that is *TC*-1) up to the height of the upstream higher terrain category at which the same multiplier is found as per [Table 2] of the Code. In this case, the recommendation implies that the multiplier 1.17 at 80 m height of *TC*-1 must be continued up to a height of 100 m, above which only the *TC*-2 multipliers of Class C are to be used. Table 4 gives the final recommended multipliers.

Table 4 Multipliers for a Structure in *TC*-1 with Upstream *TC*-2
(*Example* 2)

Height (m)	Multipliers as per [5.3.2.4 (b) (i)]	Multiplier as per [Appendix B]
10	0.99	0.99
15	1.03	1.03
20	1.06	1.06
30	1.09	1.09
50	1.14	1.14
80	1.17	1.17
100	1.20	1.17
150	1.24	1.21

NOTE — As observed earlier [5.3.2.4 (b) (i)] gives higher loads but is simpler to apply.

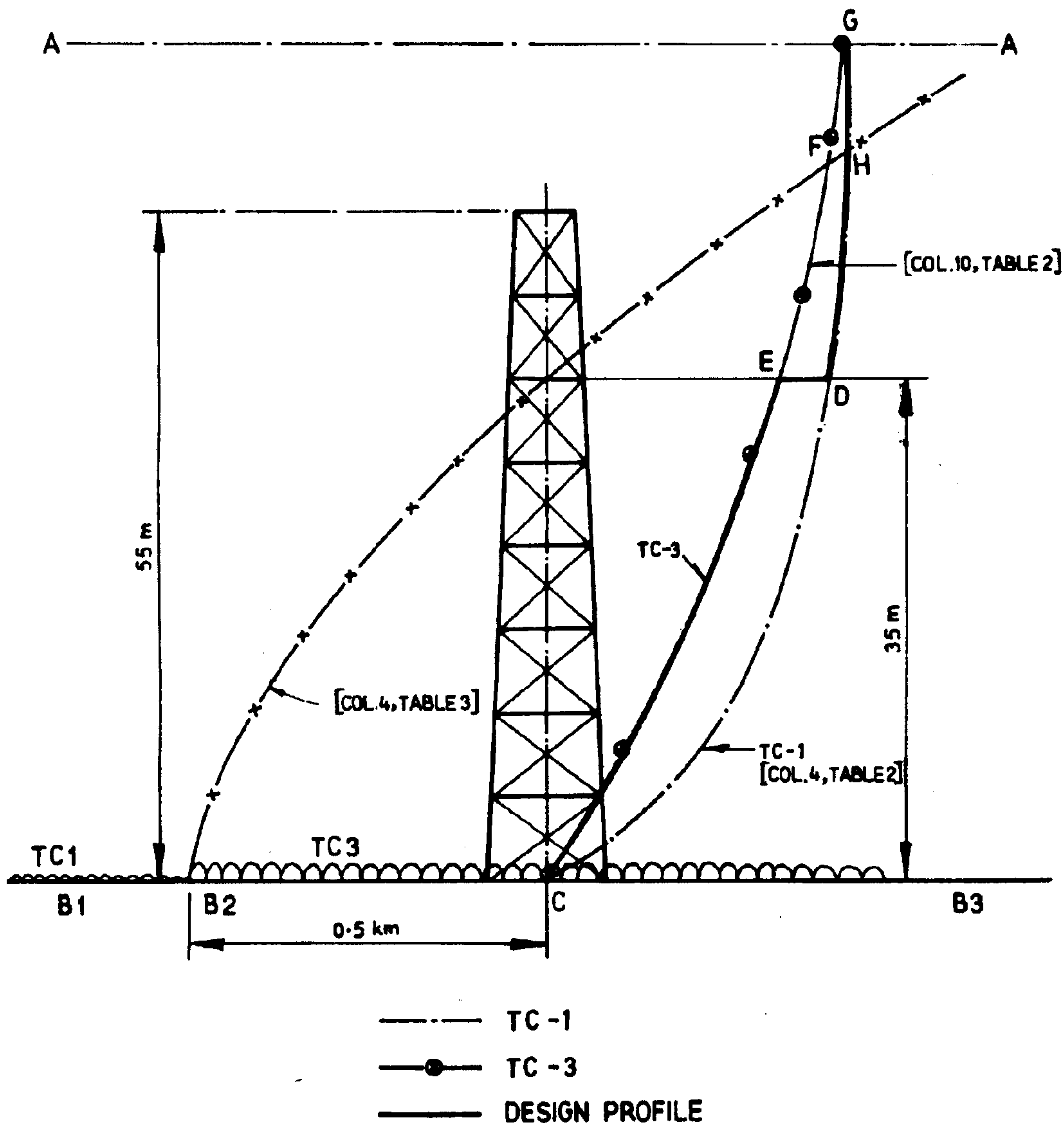


FIG. 9 DETERMINATION OF VELOCITY PROFILE NEAR A CHANGE IN TERRAIN CATEGORY (LESS ROUGH TO MORE ROUGH)

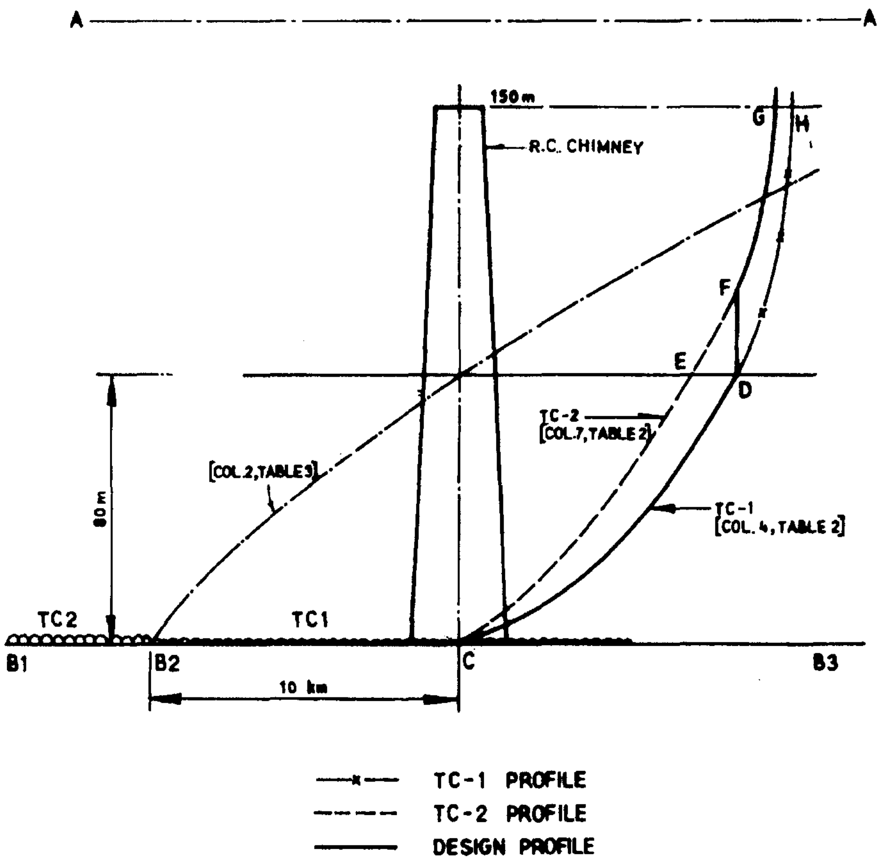


FIG. 10 DETERMINATION OF VELOCITY PROFILE NEAR CHANGE IN TERRAIN CATEGORY (MORE ROUGH TO LESS ROUGH)

Example 3. *A TV tower of height 300 m located in the centre of a city with tall buildings (greater than 50 m in height) up to a height of 3 km from it, closely built houses of up to 10 m height from 3 km to 10 km and flat terrain beyond for 15 km at which sea coast is encountered. Wind is from the sea. To find the wind speed multipliers.*

Solution: This is a problem in which wind passes through four terrains before reaching the structure of interest. At high winds, the sea can be taken as equivalent to *TC*-2 due to the large waves that are generated. In this problem, *TC*-2 is followed by *TC*-1 for 15 km and this is followed by *TC*-3 for 10 km with a final terrain category 4 for 3 km. The configuration is sketched in Fig. 11 where the multiplier profiles of *TC*-1 to *TC*-4 are shown at the location of the TV tower. As before, one can choose the simpler method of [**5.3.2.4** (b) (i)] and use the multipliers of *TC*-1 for the entire height. To use the alternate method given in [Appendix B], one draws first the developing profiles from the beginning of each terrain [Table 3].

a) *AA* is the edge of *TC*-2 (say the line of multiplier 1.35).

b) B_1B_2 is the approach terrain *TC*-2, B_2B_3 is the first intermediate terrain *TC*-1, B_3B_4 is the second intermediate terrain *TC*-3 and B_4B_5 is the final terrain *TC*-4 in which the TV tower is located.

c) From [Table 3], it may be easily determined that the edge of the *TC*-1 profile starting at B_2 meets the TV tower at C_1 at a height of 100 m, the edge of *TC*-3 profile starting at B_3 meets the TV tower C_2 at a height of 250 m and the edge of the *TC*-4 profile starting at B_4 meets the TV tower at C_3 at a height of 220 m and the edge of the upstream *TC*-2 profile is at a height of 490 m.

d) All the four category profiles (of class *C* since the maximum dimension of the structure is more than 50 m) have been drawn with origin at *C*, the base of the TV tower.

e) Proceeding as before, one finds that *TC*-4 swamps *TC*-1 and is effective up to a height of 220 m, from *C* to *D*. At *D*, one encounters *TC*-3 beginning at B_3 up to a height of 250 m. Thus the profile shifts from *D* to *E* on *TC*-3 profile at a height of 220 m and moves along the *TC*-3 profile from *E* to *F*, which is at a height of

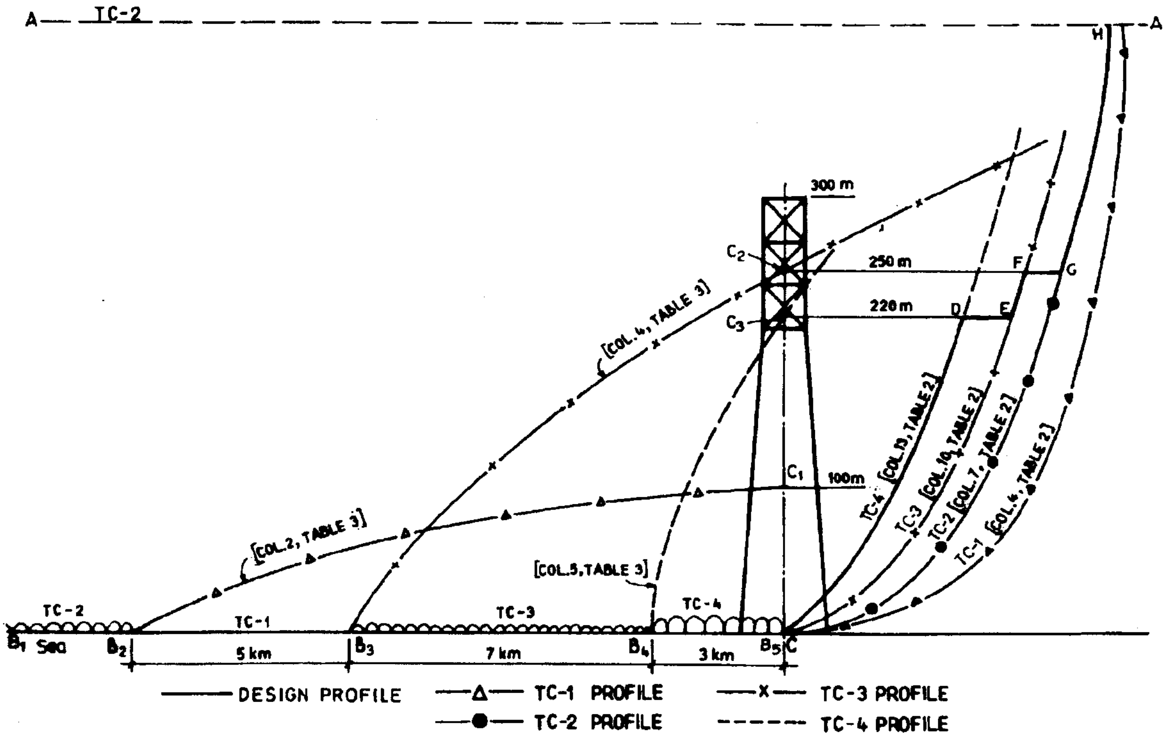


FIG. 11 DETERMINATION OF DESIGN PROFILE INVOLVING MORE THAN ONE CHANGE IN TERRAIN CATEGORY

250 m. Above *F*, the unpenetrated profile is the far upstream *TC*-2 profile and this applies for the rest of the height of the TV tower. Table 5 gives the final multipliers.

Table 5 Multipliers for a Structure in *TC*-4 with Upstream *TC*-3, *TC*-1 and *TC*-2
(*Example* 3)

Height (m)	Multipliers as per [5.3.2.4(b)(i)]	Multipliers as per [Appendix B]
10	0.99	0.67
15	1.03	0.67
20	1.06	0.83
30	1.09	0.83
50	1.14	1.05
100	1.20	1.10
150	1.24	1.10
200	1.26	1.13
220 (interpolated)	1.27	1.19 (*TC*-3 value)
250	1.28	1.26 (*TC*-2 value)
300	1.30	1.31 (*TC*-2 value)

3.6 Topography (k_3) [5.3.3]

The coefficient k_3 allows for undulations in the local terrain in the form of hills, valleys, cliffs, escarpments and caters to both upwind and downwind slopes. Methods for evaluation of k_3 are given in [Appendix C] of the Code.

3.7 Design Wind Pressure [5.4]

One of the important modifications in the Code relates to the velocity-pressure relationship. In the earlier version of the Code, the pressure was over-estimated by about 25 percent for a given wind velocity. The correct equation given now reads as follows:

$$p_z = 0.6\, V_z^2 \qquad ...(8)$$

where

p_z = design wind pressure at height z in N/m^2, and
V_z = design wind speed at height z in m/s.

Strictly speaking, the coefficient 0.6 is only the most probable average for the atmospheric conditions prevailing in India during the whole year. A value of 0.612 5 would be more appropriate in temperate zones above latitude of about 33°. The wind effects in terms of static loading on the structure are determined from design pressures as indicated in **4**. A comparison of wind pressures as per new and 1964 Code is given in the following example.

Example. *Estimation of design wind pressure for a building as per IS 875 : 1964 and IS 875 (Part 3) : 1987.*

Given:
A building of length 35.0 m, width 15.0 m, height 15.0 m
Life of structure: 25 years
Terrain category: 3
Location: Delhi

Topography: Almost flat

Solution:

Previous code [IS 875 : 1964], [Fig. 1A] gives pz=150 kgf/m^2.

Present code [IS 875 (Part 3) : 1987] :

Design wind speed $Vz = V_b.k_1.k_2.k_3$

V_b = 47.0 m/s [Fig. 1]

k_1 = 0.90 [Table 1, N = 25 years]

Since greatest horizontal dimension of 35 m is between 20 m and 50 m

k_2 = 0.88 [Table 2, Class B] at 10.0 m height

k_3 = 1.00

Hence $V_{10} = 47.0 \times 0.90 \times 0.88 \times 1.0 = 37.22$ m/s

Design pressure at 10 m height $= 0.6\ V_z^2$
$= 831.37$ N/m^2
$= 84.83$ kgf/m^2
($g = 9.80$ m/s^2 in most parts of India)

Let the life of the structure be increased to 50 years [k_1=1.0, Table 1], then

$p_{10} = 0.6\ [47.0 \times 1.0 \times 0.88 \times 1.0]^2 = 1\,026.39$ N/m^2
$= 104.73$ kgf/m^2

If the structure is located in a terrain category 2,

then the value of k_2 = 0.98 and
$p_{10} = 0.6\ [(47.0)(1.00)(0.98)(1.00)]^2$
$= 1\,272.91$ N/m^2 $= 129.89$ kgf/m^2

Let us now consider a situation where the same structure is situated at Madras, Calcutta, Roorkee, Bombay, Bangalore and Darbhanga. The design wind pressure will be as given in Table 6, for different possible terrain conditions. From Table 6, it is observed that the design wind pressure is strongly affected by the consideration of the expected life of the structure and terrain category. It is observed that the earlier Code generally over estimated the wind load in city centres. One can also work out that the earlier Code generally underestimated the design wind load, if either the expected life or 'return period' is large (for example, see the case for Bombay in Table 6). This is particularly true for regions where the design pressures, according to the 1964 Code are low.

Table 6 Design Wind Pressure
(Example)

Location	Life (Years)	Terrain	Design Wind Pressure (N/m²)	
			IS 875 : 1987	IS 875 : 1964
Delhi	25	3	831.37	1 470.00
	50	3	1 026.39	
	50	2	1 271.91	
Calcutta	25	3	940.89	1 962.00
Roorkee	25	2	741.84	1 471.50
Bombay	25	3	744.91	981.00
	50	2	1 115.60	
Madras	25	3	940.89	1 962.00
Bangalore	25	3	447.09	981.00
Darbhanga	25	2	1 380.73	1 471.50

4 WIND PRESSURE AND FORCES ON BUILDINGS/STRUCTURES [6]

4.1 General

The Code stipulates requirements for calculation of wind loading from three different points of view:

— The building/structure taken as a whole;

— Individual structural elements such as roofs and walls; and

— Individual cladding units such as sheeting and glazing including their fixtures.

The wind loading is given in terms of pressure coefficients C_p and force coefficients C_f and can be determined as follows:

$$F = C_f\, A_e\, p_d \qquad \text{...(9) [6.3]}$$

$$F = (C_{pe} - C_{pi})\ A\, p_d \qquad \text{...(10) [6.2.1]}$$

where

F = wind load;

C_f = force coefficient;

A_e = effective frontal area obstructing wind, which is identified for each structure;

p_d = design wind pressure;

C_{pe}, C_{pi} = external and internal pressure coefficients; and

A = surface area of structural elements.

4.2 Pressure Coefficients [6.2]

Pressure coefficients are applicable to structural elements like walls and roofs as well as to the design of cladding. The calculation process implies the algebraic addition of C_{pe} and C_{pi} to obtain the final wind loading by the use of Eqn. (10). The Code indicates both these coefficients separately for a wide variety of situations generally encountered in practice.

4.2.1 *External Pressure Coefficient* C_{pe} **[6.2.2]**

External pressure coefficient depends on wind direction, structure configuration in plan, its height *versus* width ratio and, characteristics of roof and its shape. [Tables 4 to 22] are devoted exclusively to the determination of C_{pe}. For the specific design considerations relating to cladding, local coefficients have been separately shown delineating the areas at the edges of walls and roofs where high concentration of negative pressure is often found to exist.

4.2.1.1 *External pressures*

It deals with a large number of cases of gross force coefficients and pressures. Should pressures in [Table 4] be applied at each level in the case of a tall building, with velocity variation as in [Table 2]. The answer, is that the Code implies that the calculation should be carried out for each level as if the air load at that level is independent of that at an adjacent (or lower or higher) level. It is puzzling to note a positive pressure coefficient of 1.25 in **[6.2.2.7]**, when one knows that it cannot exceed +1.0. The reason is that since the pressure coefficients on the small overhanging positions are not given but are known to be more strongly negative on top than the negative pressures on the non-overhanging portions, these numbers are expected to compensate for the projected lower pressures on the top of overhanging portion. Although not stated in the Code, it is desirable to take the same values in **[6.2.2.7]** on the vertical walls above and below the overhanging portion, over a height equal to half the projecting length of the overhang; if no other guideline is available.

4.2.2 *Internal Pressure Coefficient* C_{pi} **[6.2.3]**

Internal pressure coefficients are largely dependent on the percentage of openings in the walls and their location with reference to wind direction. The Code indicates C_{pi} for a range of values with a possible maximum (that is positive pressure) and a possible minimum (that is negative pressure) with the provision that both the extreme values would have to be examined to evaluate critical loading on the concerned member. Three cases have been specifically indicated for arriving at C_{pi}:

- openings up to 5 percent of wall area,
- openings from 5 percent to 20 percent of wall area, and
- openings larger than 20 percent of wall area (including buildings with one side open).

The last case is of particular interest in determining wind loading for structures such as aircraft hangers which have wide full height openable doors forming one side of the enclosure [Fig. 3].

4.2.2.1 *Internal pressures* **[6.2.3]**

The internal pressures given in **[6.2.3]** have been deliberately specified slightly higher (negatively) than the codes of temperate countries to reflect the fact that as a tropical country, the size of windows/doors are larger in India and the normal tendency is to keep them open as much as possible.

4.3 Force Coefficients [6.3]

Force coefficients applicable to the building/structure as a whole as well as to structural frameworks which are temporarily or permanently unclad are also given in **[6]**. For evaluating force coefficients for the clad building/structure as a whole, the Code gives guidance for a variety of plan shapes and height to breadth ratios [Table 23]. It also indicates extra forces occurring due to 'frictional drag' on the walls and roofs of clad buildings **[6.3.1]**. The diameter D to be considered for rough surface when applying **[6.3.2.2]** is the value excluding the height of the roughness, that is excluding the height of flutings or similar 'regular' roughnesses or excluding average height of random roughness like sand particles. This definition of D is valid only if $D/\varepsilon > 100$ (where ε is the average height of the surface roughness). For still larger roughnesses, specialists advice should be sought.

Force coefficients on unclad buildings/structures, frameworks and their individual members, are comprehensively covered in **[6.3.3.3 to 6.3.3.6]**. The frameworks included are those that are single (that is isolated), multiple or in the form of lattice towers. The effects of 'shielding' in parallel multiple frames and the effect of different solidity ratios have been incorporated along with global force coefficients for lattice towers [Tables 28 to 32]. Force coefficients for individual members of various structural shapes and wires/cables have been separately indicated [Tables 26 and 27].

4.3.1 *Individual Members* **[6.3.3.2]**

The force coefficient of rectangular members given in [Fig. 4] reflect the recent discovery that the aspect ratio of the rectangle affects the total force coefficient and peaks at an aspect ratio of about 2/3. Slight discrepancies will be observed between the gross value in [Fig. 4] and the summed up values in [Table 4], for smaller height to width ratios and the value for the square in [Table 26]. Such seeming contradictions will be observed in several international codes also and reflect, mainly the continuation of values taken from earlier codes. The last case in [Table 4], has reconciled the value in [Fig. 4] and that in the last sketch of [Table 4], $h/w \geq 6$.

In the other cases it is recommended that the negative pressure behind the body be further reduced to bring the total force to the value in [Fig. 4]. For example let us consider the case $1/2 < h/w \leq 3/2$, $3/2 \leq l/w < 4$ and $\theta = 0°$ in [Table 4 (item 4)] with $a = 2$, $b = 8$ and $h = 3$. The total force in the direction of wind with pressure coefficient of +0.7 on face *A* and –0.3 on face *B* is 1.0. '*a*' in [Fig. 4] is '*w*' of [Table 4], '*b*' in [Fig. 4] is '*l*' of [Table 4] and '*h*' has the same meaning in both. Hence

$h/w = h/a = [h/b \,.\, b/a]$

Therefore,

$h/b = h/w \,.\, a/b$

$l/w = b/a$

For $h/w = 3/2$ and $l/w = 4$ of [Table 4] we have to look up $h/b = 3/8$, $a/b = 0.25$ in [Fig. 4B] which gives $C_f = 1.18$, while adding the pressures on faces *A* and *B* in [Table 4] yields $C_f = 1.0$. The recommendation is not to change the pressure on the front face *A* (they are usually more reliable and in any case cannot exceed 1.0), but decrease the pressure on the back face from –0.3 to –0.48. The pressures on the sides as well as at the corners need not be changed.

4.3.2 *Force Coefficient for Framed Structures* **[6.3.3.3** and **6.3.3.4]**

The force coefficients for framed structures given in [**6.3.3.3**] and [**6.3.3.4**] reflect the result of data which became available around 1980. A few designers have encountered a condition (in unusual structures), where they found that the sum of the total force on more than three frames was more than that on a solid body of the same outer geometry. This is correct and has to be applied as such if all the frames are finally mounted on a single large structure. In some cases, the designers have found it more economical to have a fully clad structure with an internal braced framework.

5 DYNAMIC EFFECTS [7]

The Code incorporates a new section relating to dynamic effects, recognizing the advent of a large number of tall, flexible and slender structures in the country's sky-line. Structures which require investigation under this section of the Code are the following:

— those with height to minimum lateral dimension ratio of more than 5.0.

— structures with a first mode natural frequency of less than 1.0 Hz.

Guidelines are given in the Code for the approximate determination of natural frequency of multistoreyed buildings and the designer is cautioned regarding certain structural responses such as cross-wind motions, interferences of upwind obstructions, galloping, flutter and ovalling. The Code encourages use of wind tunnel model testing, analytical tools and reference to specialist advice when wind induced oscillations of the structure reach significant proportions. [Clause **7.1**] gives guidelines for the assessment of dynamic effects of wind. It is to be noted that the Strouhal number for non-circular bodies [**7.2**] is usually slightly less than that for circular bodies.

6 GUST FACTOR (*GF*) OR GUST EFFECTIVENESS FACTOR (*GEF*) METHOD [8]

6.1 One doubt that may arise during a comparative study of [Table 33] to find hourly mean wind speed, and peak gust multiplier from [Table 2] is the absence of the class of structure that is its size in [Table 33]. However, further study will show that the variable ϕ takes into account the building size [**8.3** and Fig. 8].

The Code says that the use of random response method for across wind response of structures of non-circular cross-sections have not matured and hence are not given in the Code. Some users may find small differences in the graph for *E* [Fig. 11] and those in some other codes. Because of the local short period intense nature of wind in most parts of our country, the calculated gust factors as per the Indian Code will be found to be generally slightly higher than in other codes which are based on the turbulence characteristics of the 'steady' fully developed pressure system wind.

6.2 Along-Wind Loads [8.3]

The use of gust factor approach has been permitted by the Code for the evaluation of along-wind load.

The calculations involved for the estimation of along-wind load have been reduced to a simple equation given below:

$$F_z = C_f \,.\, A_e \, p_z' \; G \qquad \ldots(11) \; [\mathbf{8.3}]$$

where

F_z = along-wind static loading at height z, applied on effective frontal area A_e of strip under consideration,

p_z' = design pressure at height z corresponding to hourly wind speed, and

G = gust factor.

One of the important departures for along-wind dynamic effects as compared to static effects is to approach the problem of design pressure evaluation from the route of hourly wind instead of short duration gusts as one is required to do for application up to [**6**]. [Table 33] indicates the revised factors k_2 which have to be applied to the basic wind speed (short duration gusts) of [Fig. 1] for arriving at the hourly mean wind speed and consequently the corresponding design pressures, p_z'.

The gust factor *G*, which depends on natural frequency, size and damping of the structure and on the wind characteristics can be evaluated directly from parameters obtained from graphs given in [Fig. 8 to 11].

6.2.1 *Computation of Wind Load by Gust Factor Method*

Step 1. *Hourly Mean Wind* (V_z'): Hourly mean wind speed is maximum wind speeds averaged over one hour.

$$V_z' = V_b.k_1.k_2'.k_3 \qquad ...(12)$$

where

V_b = Regional basic wind speed [Fig. 1],
k_1 = Probability factor [**5.3.1**],
k_2' or k_2 = Terrain and height factor [Table 33], and
k_3 = Topography factor [**5.3.3**].

Step 2. *Design Wind Pressure* (p_z') [**6.2.1**]

$$p_z' = 0.6\,(V_z')^2 \qquad ...(13)$$

Step 3. *Along-wind Load* [**8.3**]

$$F_z = C_f.A_e.p_z'.G \qquad ...(14)$$

where

F_z = along-wind load on structure at any height z;
C_f = force coefficient;
A_e = effective frontal area;
$p_{z'}$ = design wind pressure;
G = the Gust Factor = (peak load/mean load); and is given by

$$G = 1 + g_f \,.\, r\sqrt{[B\,(1+\phi)^2 + SE/\beta]} \qquad ...(15)$$

where

g_f = peak factor defined as the ratio of the expected peak value to the root mean value of a fluctuating load;
r = roughness factor;
$g_f \,.\, r$ is given by [Fig. 8]
B = background factor [Fig. 9];
SE/β = measure of the resonant component of the fluctuating wind load;
S = size reduction factor [Fig. 10];
E = measure of available energy in the wind stream at the natural frequency of the structure [Fig. 11];
β = damping coefficient of the structure [Table 34]; and

$$\phi = \frac{g_f \,.\, r \,.\, \sqrt{B}}{4}$$

and is to be accounted only for buildings less than 75 m high in terrain category 4 and for buildings less than 25 m high in terrain category 3, and is to be taken as zero in all other cases.

6.3 Across-Wind Loads

The Code points out that the method of evaluating across-wind and other components of structural response to wind loading are fairly complex and have not as yet reached a stage where successful codification can be attempted. It recommends use of wind tunnel model studies and specialist advice in cases where significant across-wind loads are anticipated.

7 WIND TUNNEL MODEL STUDIES [1 and 7]

The Code recommends that model studies in wind tunnels be resorted to under several conditions. These are:

i) for structures of unconventional shapes, unusual locations and abnormal environmental conditions, not covered in the Code [**1.1.3**, Note 1];
ii) certain wind induced dynamic response problems [**7.1**, Notes 2 and 9]; and
iii) buildings or shapes for which more accurate data or confirmation of existing data is required.

Unlike in some foreign codes, no guidelines have been prescribed for properly carrying out such studies or limitation of model studies including those due to Reynolds number etc. The reason appears to be that there are very few suitable wind tunnels in our country. However, since the existing ones are being improved and four to five new wind tunnels are likely to come up in the next 3 to 5 years, it is in order to discuss this.

It is now well recognized that wind load on a structure (static or dynamic) depends on:

i) Variations of mean velocity U_0, with height;
ii) Variation of the intensity of turbulence with height, the intensity being defined as $\sqrt{\dfrac{u'^2 + v'^2 + w'^2}{3V_0^2}}$ where u', v' and w' are the departure of the velocity at any time along x,y and z axes, respectively, from V_0, and
iii) the 'scale' of turbulence, which is a measure of the size of coherent blobs of flow, usually swirls of various sizes, called 'eddies'.

The above is not an exhaustive list, but is generally adequate for consideration. Creation of the mean velocity distribution in a wind tunnel is a relatively simple task and about half a dozen methods are available for doing it. However, care has to be exercised with regard to the simulation of (ii) and (iii). Sometimes, it is stated that the intensity of distribution of turbulence and scale must be the same as values scaled down from that in the atmosphere. This may be acceptable for sharp edged bodies whose properties do not depend on the Reynolds number. But the force coefficients on rounded surface bodies depend strongly on Reynolds number and while the spectra must be simulated, the intensities may have to be different, to ensure that force coefficients are not measured in the force bucket (*see* data for R_e between 4×10^5 and 2×10^6 in [Fig. 5] which shows the 'bucket').

The above discussions lead to the following general guidelines for wind tunnel model studies.

i) The variation of the mean velocity with height has to be modelled, to within 2.0 percent to 3.0 percent.
ii) The longitudinal component of turbulence intensity, particularly, has to be modelled suitably to ensure that the non-dimensional spectral energy $n.S_u(n)/\sigma_u$ has the same variation with respect to $n.L_x/V_z$ as in the atmosphere, especially around the natural frequency (or frequencies) of the structure.
iii) The model dimension along the wind should not exceed three times the longitudinal scale of turbulence. This requirement helps in fulfilling (ii) to some extent.
iv) All scaling laws used to extrapolate model results to full scale, incorporate established Reynolds number effect correlations and such corrections as may seem necessary should be applied. It is worth stating that the instrumentation chain used must have adequate response characteristics.

8 APPLICATION OF CODAL PROVISIONS

8.1 Provisions of IS 875 : 1987 can be broadly classified as:

a) Computation of design wind speed based on wind zone, terrain category, topography and wind direction.
b) Computation of design wind pressure.
c) Computation of wind load using pressure co-efficients.
d) Computation of wind load using force coefficients.
e) Computation of along-wind force using gust factor method.

8.2 For easy understanding, the above steps of computation are detailed in the form of flow charts in Fig. 12 to 15. Reference to clauses, tables and charts of IS 875 (Part 3) : 1987 has also been incorporated in these flow diagrams. Further explanation of the above points is given alongwith the solved examples.

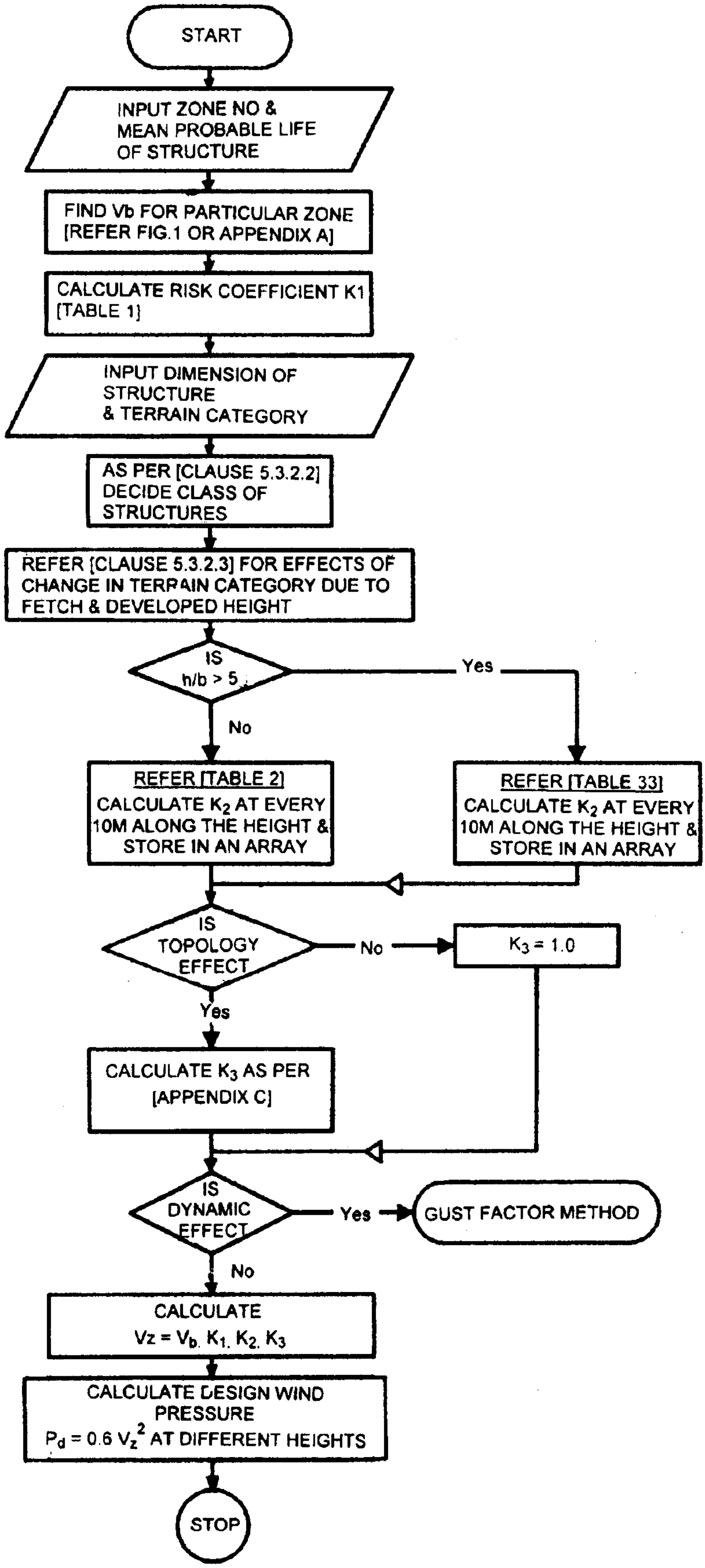


FIG. 12 FLOW DIAGRAM FOR CALCULATING DESIGN WIND PRESSURE

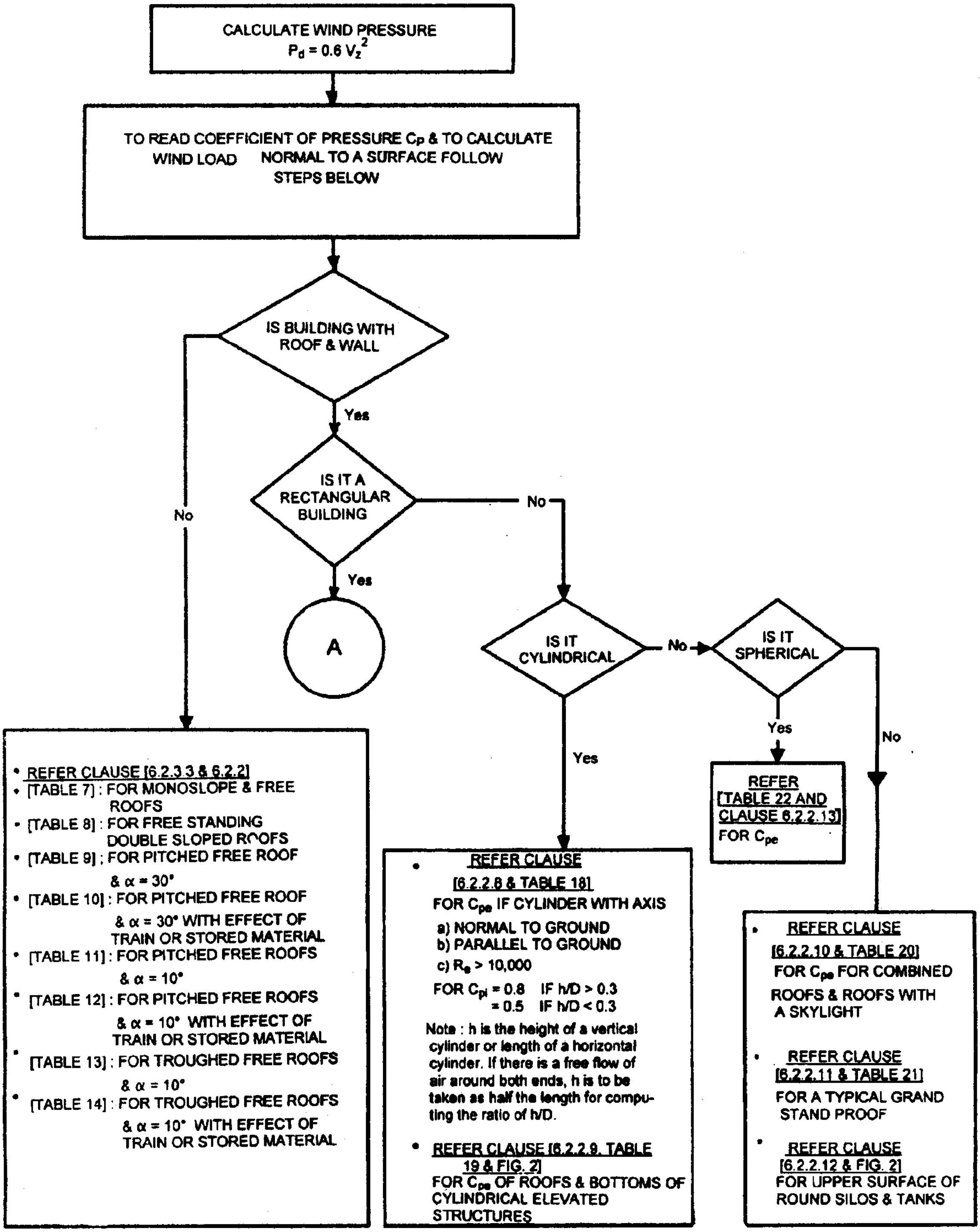


FIG. 13 FLOW DIAGRAM (2) TO READ C_p VALUES OF BUILDING/STRUCTURES (*Continued*)

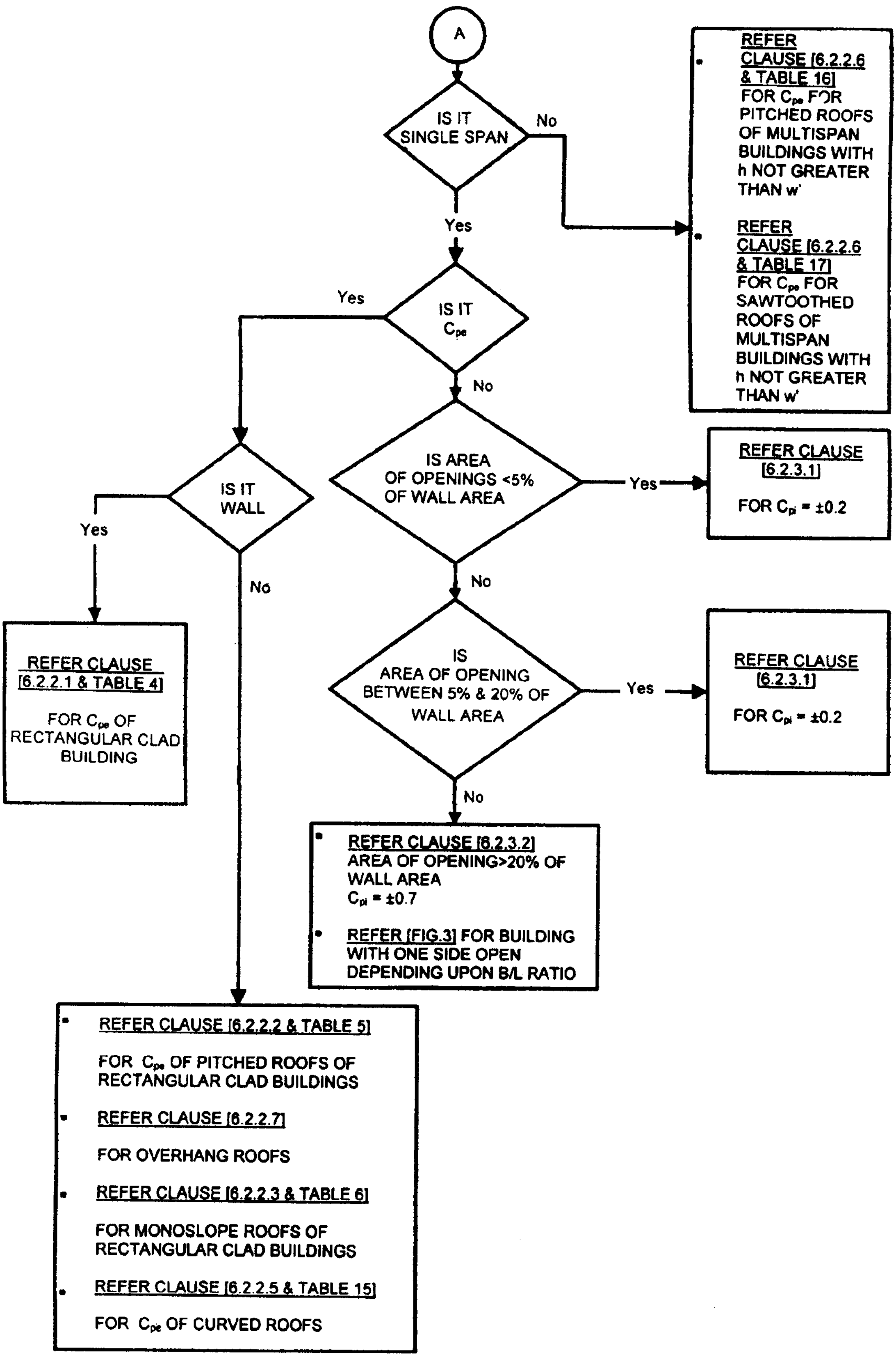


FIG. 13 FLOW DIAGRAM (2) TO READ C_p VALUES OF BUILDINGS/STRUCTURES

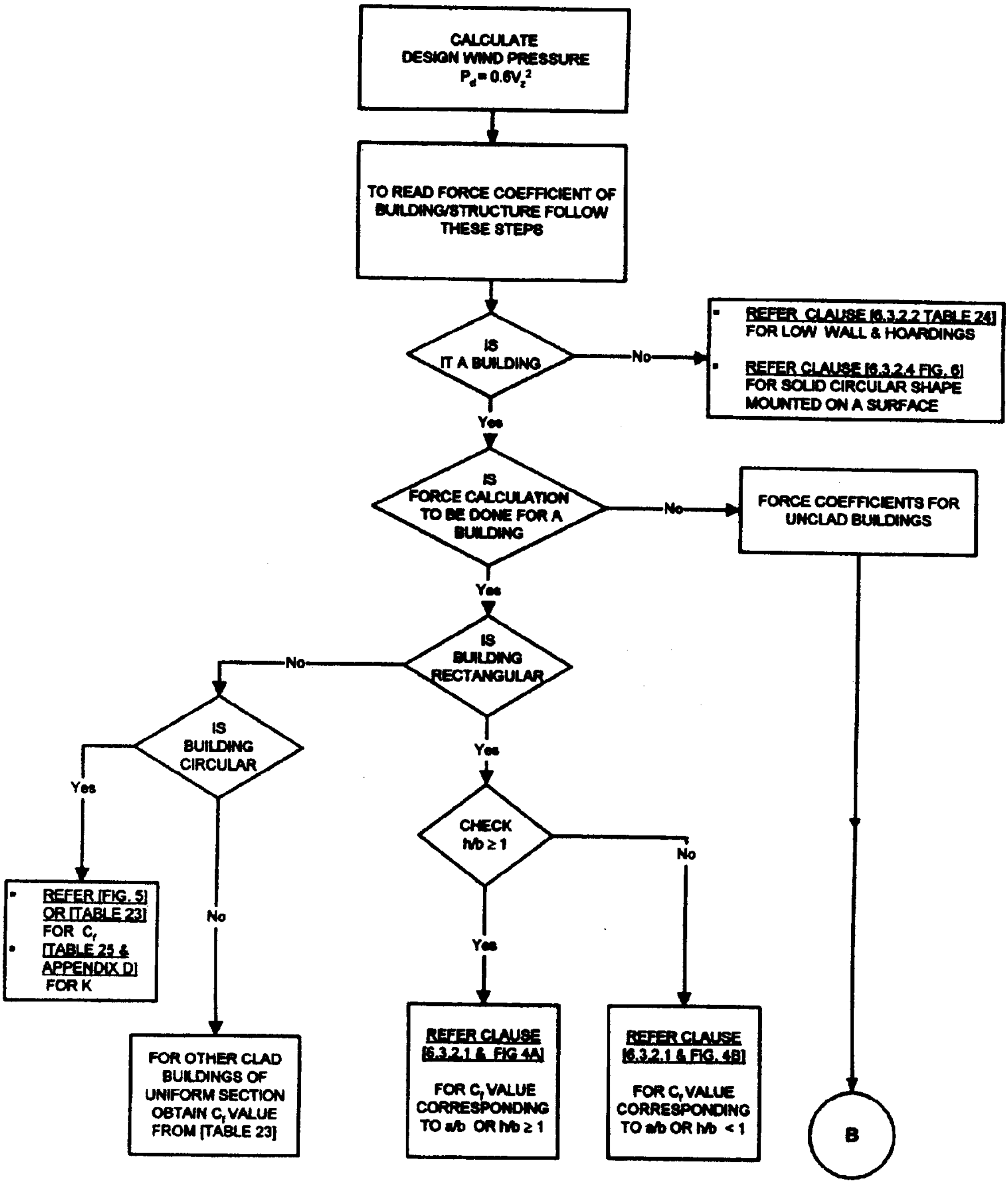


FIG. 14 FLOW DIAGRAM (3) TO READ FORCE COEFFICIENT (*Continued*)

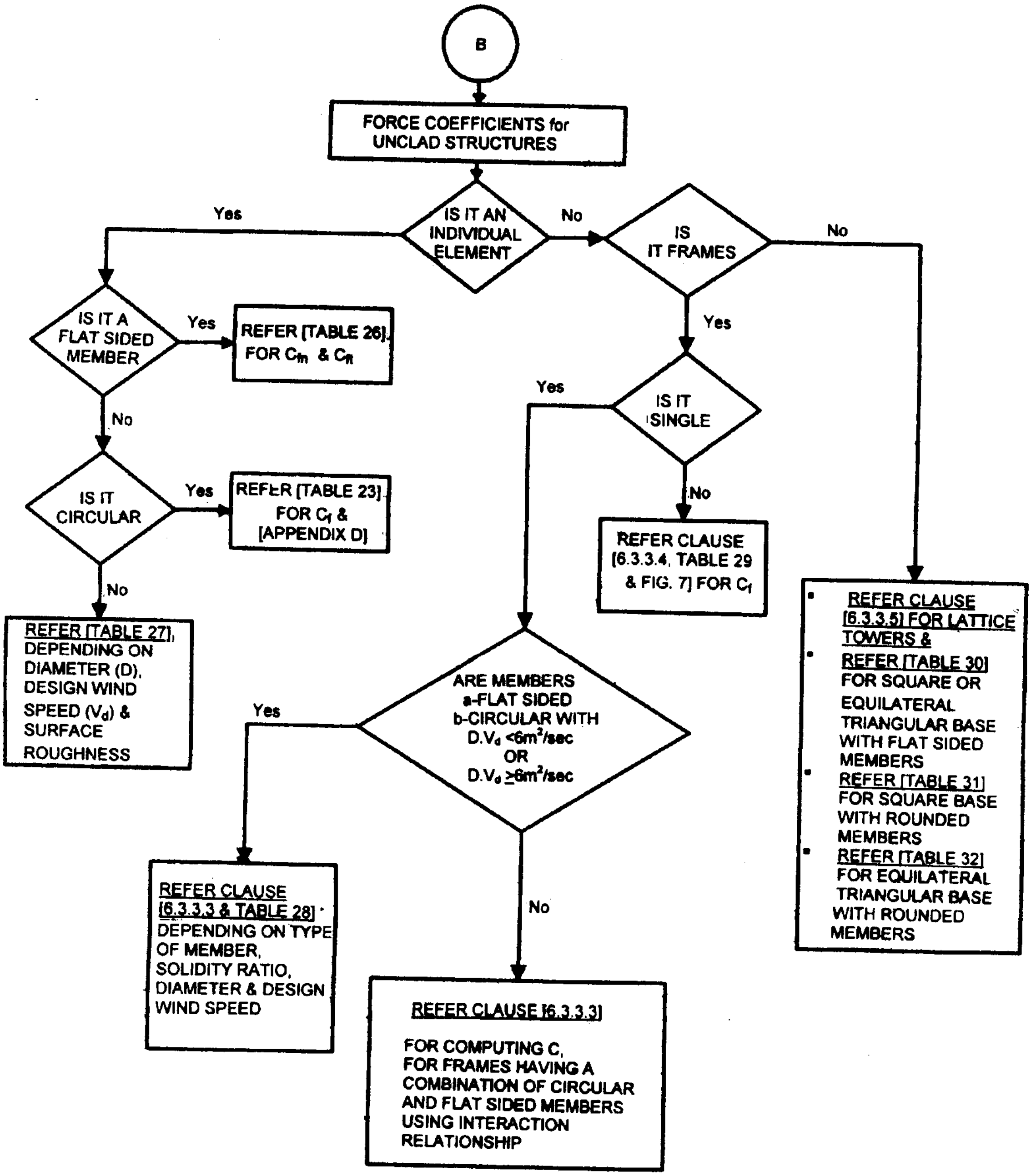


FIG. 14 FLOW DIAGRAM (3) TO READ FORCE COEFFICIENT

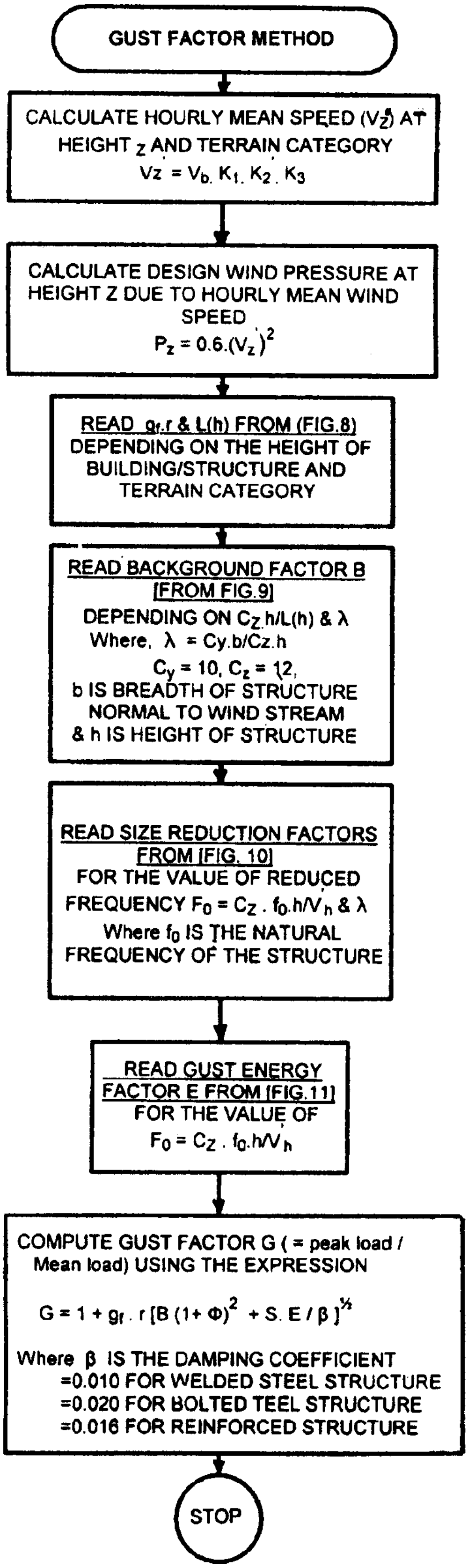


FIG. 15 FLOW DIAGRAM FOR GUST FACTOR

9 ASSESSMENT OF WIND LOADS ON BUILDINGS AND STRUCTURES

9.1 Pressure Coefficient Method

9.1.1 Stepwise computation of wind load using Peak Gust Method is as under

Step 1. *Design Wind Speed* (V_z) **[5.3]**

$$V_z = V_b . k_1 . k_2 . k_3$$

where

V_b = basic wind speed,

V_z = design wind speed,

k_1 = probability factor or risk coefficient [Table 1],

k_2 = terrain, height and structure size factor [Table 2], and

k_3 = topography factor **[5.3.3]**.

Step 2. *Design Wind Pressure* (p_z) **[5.4]**

$$p_z = 1/2 \, \rho \, V_z^2$$

where

ρ = the density of air and its value is 1.2 kg/m^3.

Step 3. *Wind Load Calculation* **[6.2.1]**

The wind load on individual structural elements such as roofs, walls, individual cladding units and their fittings is calculated by accounting the pressure difference between opposite faces of these elements or units and is given by:

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

where

F = wind load,

C_{pe} = external pressure coefficient,

C_{pi} = internal pressure coefficient **[6.2.3.1]**,

A = surface area of structural element or cladding, and

p_d = design wind pressure.

Values of C_{pe} and C_{pi} for different types of roofs, walls of different types of buildings are given in [Tables 4 to 22].

Step 4. *Estimation of Frictional Drag* (F') **[6.3.1]**

Depending upon the d/h or d/b ratio of rectangular clad buildings additional drag force given by Eqn. 16 or 17 is added to the force given by **[6.2]**.

if $h \leq b \; F' = C_f' \, (d - 4h) \, b.p_d + C_f' \, (d - 4h) \, 2h.p_d$ ...(16)

$h > b \; F' = C_f' \, (d - 4b) \, b.p_d + C_f' \, (d - 4b) \, 2h.p_d$ ...(17)

Values of C_f' are given in **[6.3.1]** for different surfaces.

9.1.2 *Solved Examples of Rectangular Clad Buildings*

Example 1. *A rectangular clad building having pitched roof and located in a farm (Fig. 16) [Tables 4 and 5].*

Given:

Physical Parameters:

Height (h)	:	3.5 m
Width (w)	:	10.0 m (excluding the overhangs)
Length (l)	:	18.0 m
Roof angle (α)	:	5°
Overhang	:	0.5 m
Openings on sides	:	10 percent of wall area
External surface of walls	:	Smooth
Flat ground		

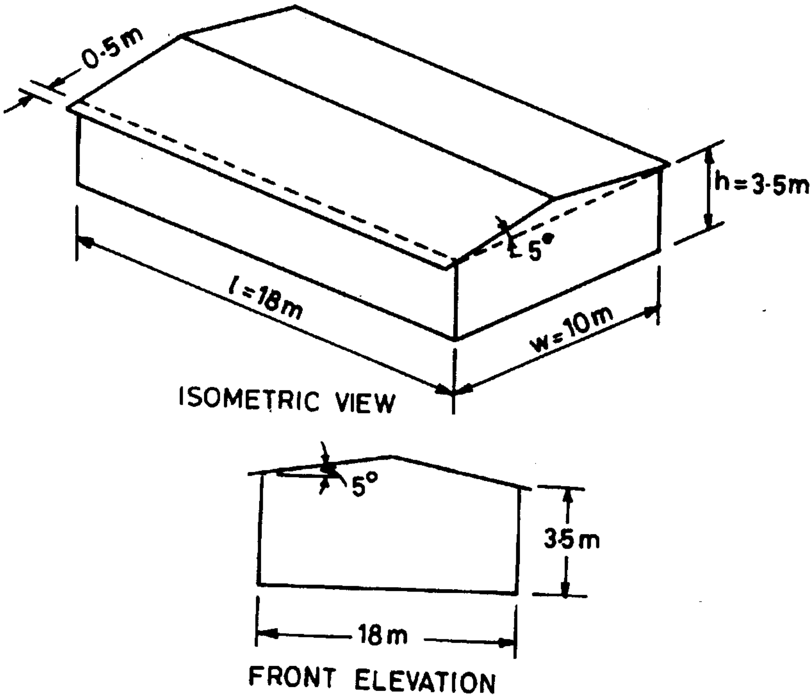


FIG. 16 A RECTANGULAR CLAD BUILDING WITH PITCHED ROOF

Wind Data

Wind zone :	3 (Basic wind speed = 47 m/s)
Terrain category :	1 **[5.3.2.1]**
Class of structure :	A **[5.3.2.2]**

Design Wind Speed (V_{10})

$V_z = V_b . k_1 . k_2 . k_3$

V_b = 47 m/s [**5.2**, Appendix A]

k_1 = 0.90 (Farm Building) [**5.3.1**, Table 1]

k_2 = 1.05 [**5.3.2.2**, Table 2]

k_3 = 1.00 [**5.3.3.1**], for site upwind slope less than 3°

Hence

$V_{10} = 47 \times 0.9 \times 1.05 \times 1.0 = 44.42$ m/s

Design Wind Pressure at 10 m height (p_{10})

$p_z = 0.6\, V_z^2$ [**5.4**]

$p_{10} = 0.6\,(44.42)^2 = 1\ 183.88$ N/m^2

Wind Load Calculation

$F \;=\; (C_{pe} - C_{pi})\, A.p_d$ **[6.2.1**, Tables 4 and 5]

Internal Pressure Coefficients (C_{pi})

Since openings are given as 5-20 percent of the wall areas, the value of C_{pi} = ±0.5 **[6.2.3** and **6.2.3.2]**

The sign will depend on the direction of flow of air relative to the side of openings.

External Pressure Coefficients on Roof (C_{pe})

The C_{pe} for various sectors of the roof excluding the overhangs, are given in Fig. 16(a) and Table 7 as per [Table 5]. For h/w = 3.5/10 = 0.35, which is < 0.5.

Table 7 Overall External Pressure Coefficient

Portion of Roof	Wind Incidence Pressure Coefficient	
	0°	90°
E	–0.9	–0.8
F	–0.9	–0.4
G	–0.4	–0.8
H	–0.4	–0.4

Pressure Coefficients for Overhang [6.2.2.7]

External pressure coefficients are recommended as equivalent to nearby local pressure coefficients

$$\text{Thus } C_{pe} = \begin{bmatrix} -1.4, \text{ corner of width } \; y = 1.5 \text{ m} \\ \\ -1.2, \text{ other than corner} \end{bmatrix}$$

However, for wind ward side of overhang the equivalent C_{pi} (underside of roof surface) is +1.25 while on lee-ward side of overhang the C_{pi} is equal to the C_{pe} of adjoining wall.

Design Pressure Coefficients on Roof (External)

C_p(net) = $C_{pe} - C_{pi}$, are calculated for cladding (local pressure coefficients are given separately). The C_p calculations are summarized as follows:

If C_{pi} = +0.5;

When $\theta = 0°$;

C_p (for region *E* and *F*) = –0.9 – (0.5) = –1.4

C_p (for region *G* and *H*) = –0.4 – (0.5) = –0.9

When $\theta = 90°$;

C_p (for *E* and *G*) = –0.8 – (0.5) = –1.3

C_p (for *F* and *H*) = –0.4 – (0.5) = –0.9

If C_{pi} = –0.5;

When $\theta = 0°$;

C_p (for *E* and *F*) = –0.9 – (–0.5) = –0.4

C_p (for *G* and *H*) = –0.4 – (–0.5) = +0.1

When $\theta = 90°$;

C_p (for *E* and *G*) = –0.8 – (–0.5) = –0.3

C_p (for *F* and *H*) = –0.4 – (–0.5) = +0.1

The C_p values for various sectors of roof are given in Fig. 16(b).

The wind may come from either direction and opening may be on any side, hence the highest C_p value for all zones of cladding is considered as −1.4 outward and + 0.1 inward. Design will have to be checked for both these pressures.

Design Pressure Coefficients on Roof (Local)

C_{pe} local is negative everywhere. Depending on wind direction C_{pi} can be +ve or −ve. Positive value of C_{pi} will increase C_p(net), which is to be considered for calculation of wind loads. The values of local C_p are summarized in Fig. 16(c).

Design Pressure Coefficients on Walls [6.2.2.1 and Table 4]

In this case h/w = 0.35 and l/w = 1.8

— For these values of h/w and l/w [Table 4], the value of C_{pe} as per Codal provisions are given in Fig. 16(d).

Value of C_{pi} is the same as before = ± 0.5

The C_p calculations are summarized below:

If C_{pi} = +0.5;

When $\theta = 0^o$;
C_p (for Wall A) = 0.7 − (0.5) = 0.2
C_p (for Wall B) = −0.25 − (0.5) = −0.75
C_p (for Wall C) = −0.6 − (0.5) = −1.1
C_p (for Wall D) = −0.6 − (0.5) = −1.1

When $\theta = 90°$;
C_p (for Wall A) = −0.5 − (0.5) = −1.0
C_p (for Wall B) = −0.5 − (0.5) = −1.0
C_p (for Wall C) = 0.7 − (0.5) = +0.2
C_p (for Wall D) = −0.1 − (0.5) = −0.6

If C_{pi} = −0.5;

When $\theta = 0^o$;
C_p (for Wall A) = 0.7 − (−0.5) = 1.2
C_p (for Wall B) = −0.25 − (−0.5) = +0.25
C_p (for Wall C) = −0.6 − (−0.5) = −0.1
C_p (for Wall D) = −0.6 − (−0.5) = −0.1

When $\theta = 90°$;
C_p (for Wall A) = −0.5 − (−0.5) = 0.0
C_p (for Wall B) = −0.5 − (−0.5) = 0.0
C_p (for Wall C) = +0.7 − (−0.5) = +1.2
C_p (for Wall D) = −0.1 − (−0.5) = +0.4

The above C_p values for walls are based on wind directions of $\theta = 0°$ and $\theta = 90°$. But with change of angle of attack of wind, position of A–B and C–D get interchanged. Hence, maximum C_p values for walls A or B = +1.2 (and −1.0) and for walls C or D = +1.2 (and −1.1) are to be considered for the design of walls.

The corners of walls experience more force for which local C_p is specified as under C_p (Corner) = −1.0 −(+0.5) = −1.5 or −0.5. Hence, the four corners of the building for width of 0.25w that is 2.50 m are to be designed for C_p = −1.5

Computation of Frictional Drag [6.3 and 6.3.1]

For rectangular clad buildings when d/h or d/b > 4 and $h < b$ frictional drag force F' (acting in the direction of wind, for θ =90° and 270°) is given by

$$F' = C_f' (d - 4h)\, b.p_d + C_f' (d-4h)\, 2\, h.p_d$$

(drag on roof) (drag on walls)

Here, $d = l = 18$ m; $b = w = 10$ m; $h = 3.5$ m

$C_f' = 0.01$ (since external surface of walls is smooth)

Frictional Drag on Roof

$= 0.01\ (18 - 4 \times 3.5)10 \times 1\ 183.88 = 473.55$ N

Frictional Drag on Walls

$= 0.01\ (18 - 4 \times 3.5)\ 2 \times 3.5 \times 1\ 183.88 = 331.50$ N

Total Frictional Drag = 805.05 N

The frictional drag on various structural components is to be distributed such that the first 1/3rd windward portion (6 m) should be able to resist half of frictional drag alongwith other loads and rest 2/3rd lee-ward portion (12 m) should be able to resist other half of frictional drag, on roof as well as on walls separately.

Final design pressure coefficient for cladding, fixtures etc, are given in Fig. 16 (e-g) and force calculations are as follows:

Forces for regions other than local regions (for cladding and fixtures) considering higher value of C_p from Fig. 16(e). Negative values mean that the roof or wall is being pushed outwards or upwards.

Force per unit area for roof when $C_p = -1.4$

$= -1.4 \times 1.0 \times 1\ 183.88 = -1\ 657.43$ N/m^2

Force per unit area for longitudinal walls when $C_p = + 1.2$

$= 1.2 \times 1.0 \times 1\ 183.88 = 1\ 420.66$ N/m^2

Force per unit area for cross walls when $C_p = + 1.2 = 1\ 420.66$ N/m^2

Forces for local regions (for cladding and fixtures) are given in Fig. 16(f).

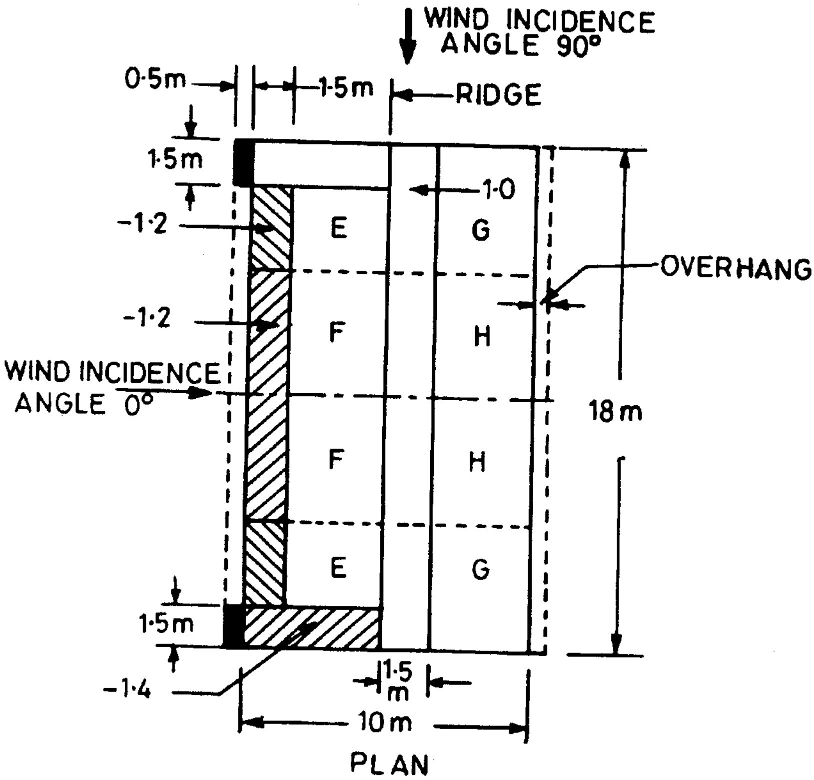
Forces for design of main frames etc.

For the design of main frames various load cases are given in Fig. 16(g). The values given inside the frame are to be used for calculating lateral force transferred from the side walls C or D. The frictional drag is to be taken appropriately alongwith these forces.

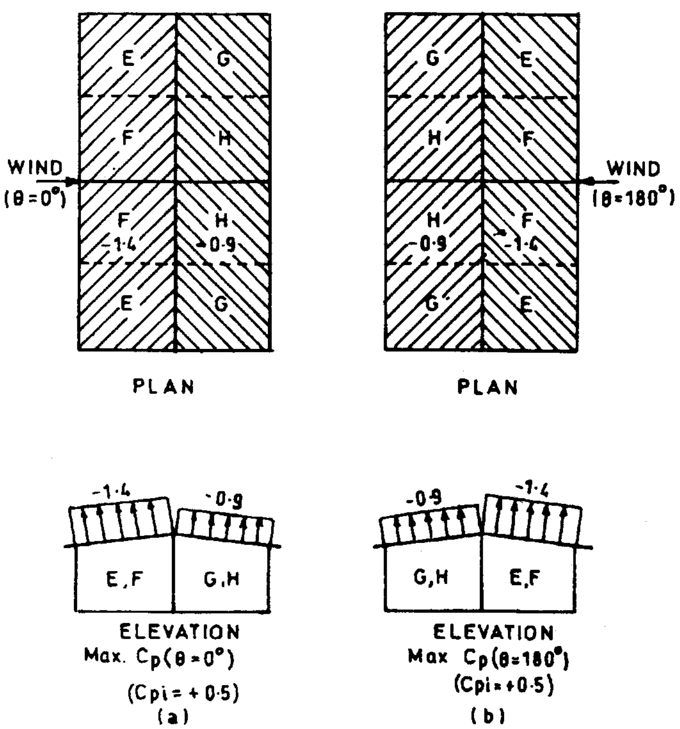
COMMENTS

(i) Each component located in a region is to be designed for the forces acting on it.

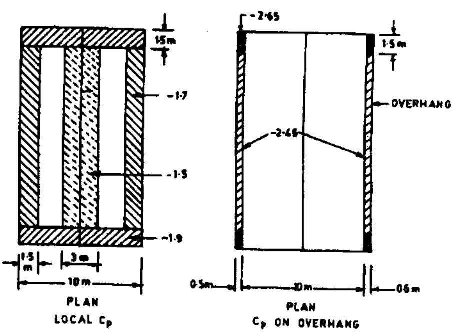
(ii) The fasteners on cladding may be concentrated in the region of high wind pressure as shown in Fig. 16(h).



16(a) C_{pe} for Roof

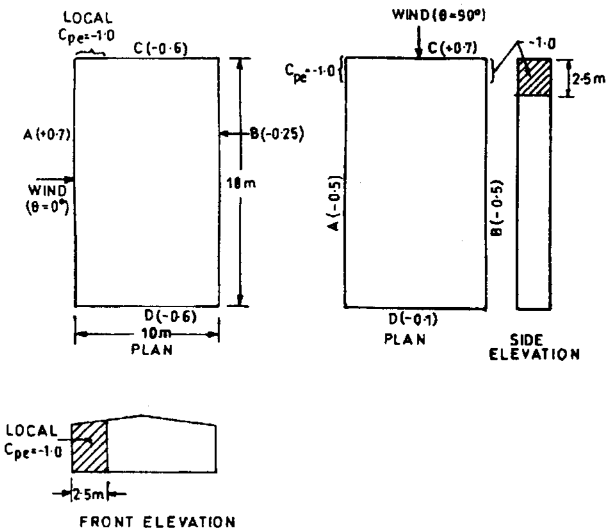


16(b) C_p for Roof

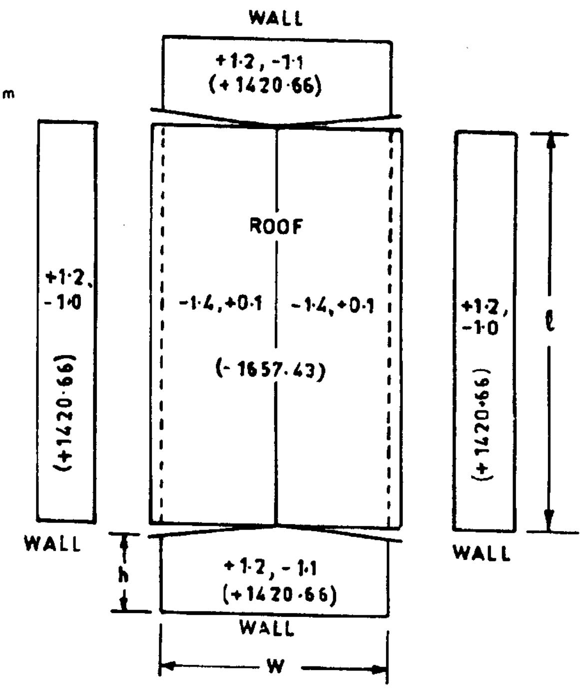


16(c) C_p (Local) for Roof

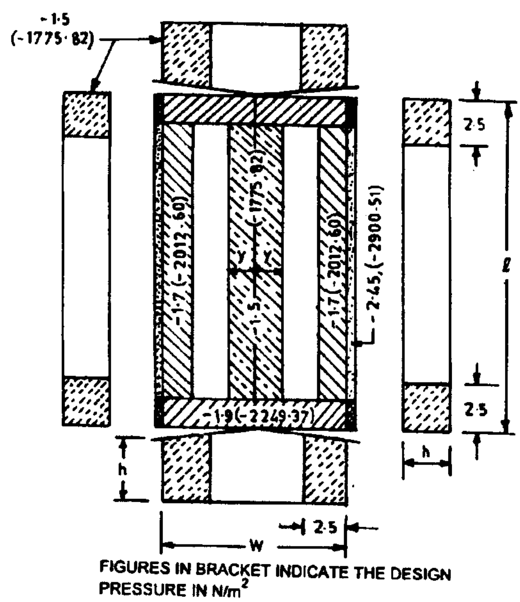
FIG. 16 A RECTANGULAR CLAD BUILDING WITH PITCHED ROOF (*Continued*)



16(d) C_{pe} for Walls

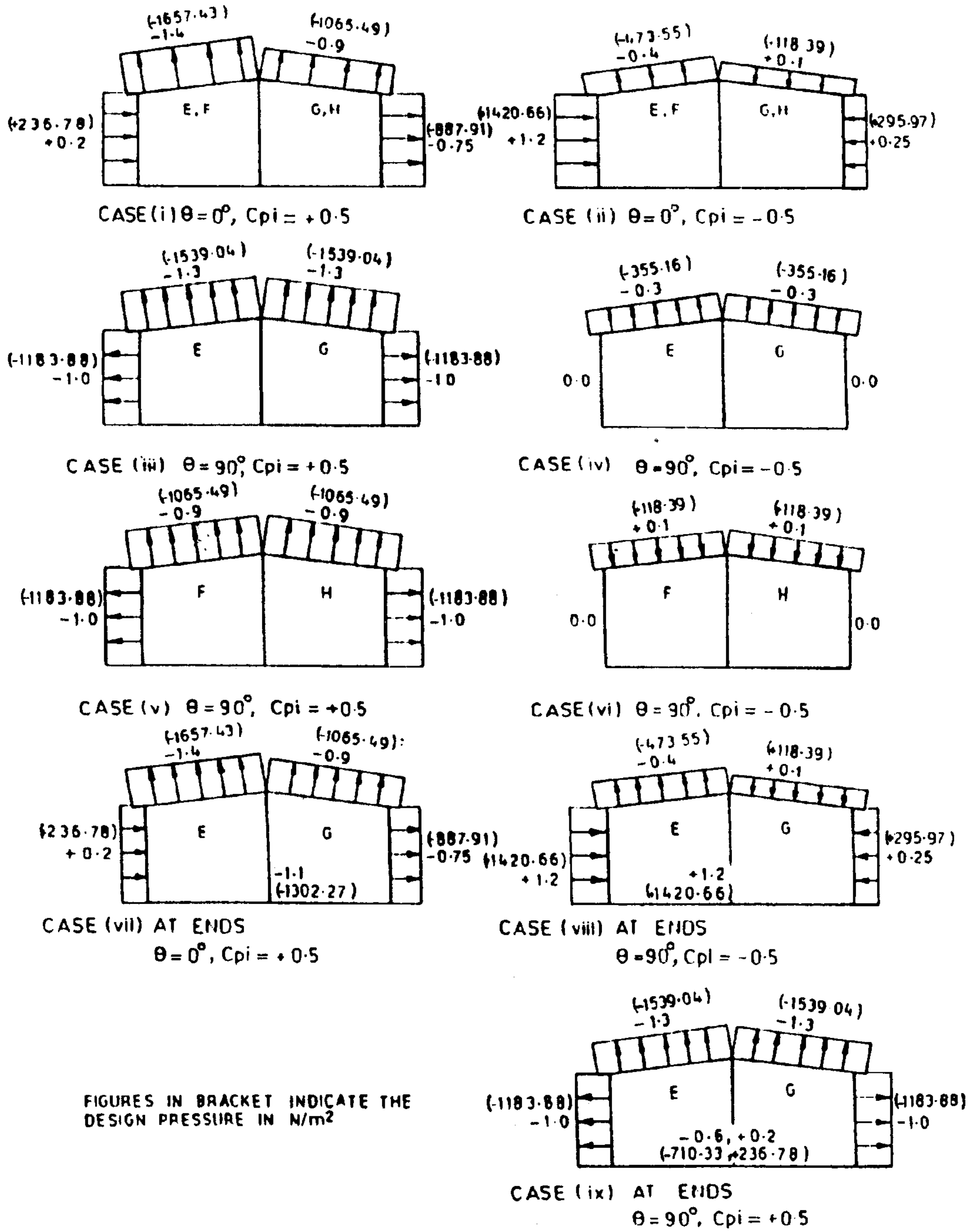


16 (e) C_p (Design) for Cladding



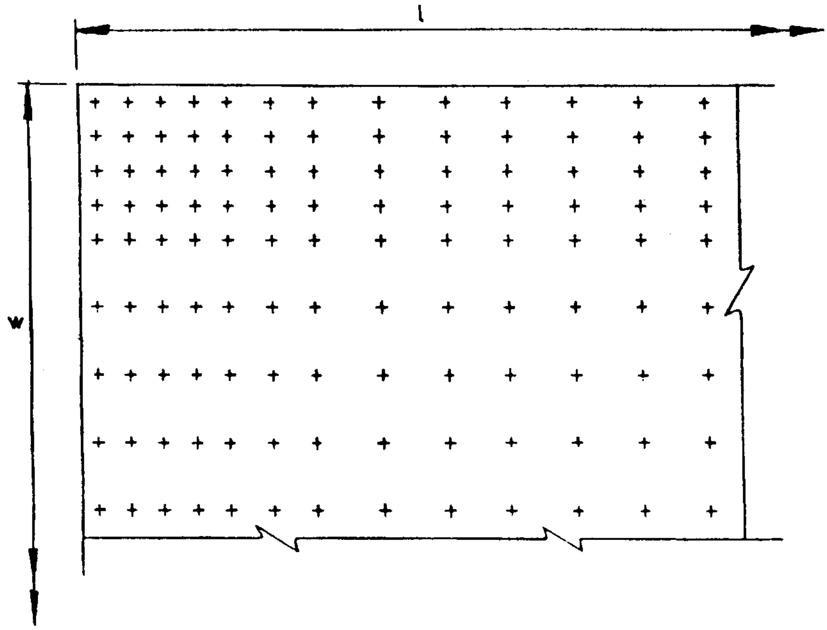
16 (f) C_p (Design-Local) for Cladding

FIG. 16 A RECTANGULAR CLAD BUILDING WITH PITCHED ROOF (*Continued*)



16(g) C_p and Design Forces for Various Load Cases

FIG. 16 A RECTANGULAR CLAD BUILDING WITH PITCHED ROOF (*Continued*)



16(h) Non-Uniform Distribution of Fasteners for Local Regions

FIG. 16 A RECTANGULAR CLAD BUILDING WITH PITCHED ROOF

Example 2. *A rectangular clad building with monoslope roof with overhangs (Fig. 17), [Table 6].*

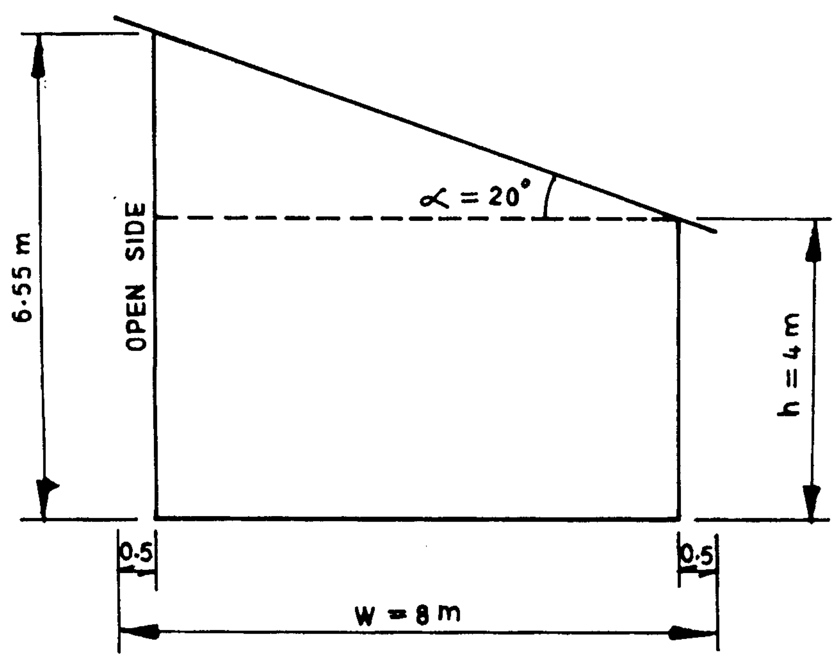


FIG. 17 A RECTANGULAR CLAD BUILDING WITH MONOSLOPE ROOF

Given:

Physical Parameters:

Height (h)	:	4.0 m
Width (w)	:	8.0 m
Length (l)	:	16.0 m
Roof angle (α)	:	20°

Overhang : 0.5 m
Openings : Longer side of the building is open
External surface of wall : Smooth
Ground is almost flat
Life 25 years
Return period 25 years

Wind Data

Wind zone: 2 (Basic wind speed 39 m/s)
Terrain category : 2 [**5.3.2.1**]
Class of structure : A [**5.3.2.2**]

Design Wind Speed (V_z)

$V_z = V_b.k_1.k_2.k_3$
V_b = 39 m/s [**5.2**, Appendix A]
k_1 = 0.92 [**5.3.1**]
k_2 = 1.00 [**5.3.2.2**, Table 2]
k_3 = 1.00 [**5.3.3.1**]
Hence, $V_{10} = 39 \times 0.92 \times 1.0 \times 1.0 = 35.88$ m/s

Design Wind Pressure (p_z)

$p_z = 0.6\ V_z^2$
$p_{10} = 0.6 \times (35.88)^2 = 772.42\ \text{N/m}^2$

Wind Load Calculation

$F = (C_{pe} - C_{pi})\ A.p_d$ [**6.2.1**, Table 6]

Internal Pressure Coefficients (C_{pi}) [6.2.3.2 and Fig. 3]

Since one side of the building is open, and *B/L* or *W/L* < 1, C_{pi} are as follows from [Fig. 3].

$$C_{pi} = \begin{bmatrix} +0.8 & \text{for } \theta = 0^\circ \\ -0.175 & \text{for } \theta = 45^\circ \\ -0.5 & \text{for } \theta = 90^\circ \\ -0.5 & \text{for } \theta = 135^\circ \\ -0.4 & \text{for } \theta = 180^\circ \end{bmatrix}$$

Values of θ for 45° and 135° are obtained by interpolation.

External Pressure Coefficients on Roof (C_{pe})

The C_{pe} for various sectors of the roof and different wind direction angles [**6.2.2.3** and Table 6] are given in Fig. 17(a) and Table 8.

Table 8 External Pressure Coefficient on Roof (C_{pe})
Overall C_{pe} for α = 20°

θ	*H*	*L*
0°	– 0.8	– 0.5
45°	– 1.0	– 0.6
90°	– 0.9 for 4 m from windward – 0.5 for the remaining portion	– 0.9 for 4 m from windward – 0.5 for the remaining portion
135°	– 0.5	– 1.0
180°	– 0.2	– 1.0

Pressure Coefficients for Overhang [6.2.2.7]

Pressure coefficients on top side of overhang (equivalent C_{pe}) are recommended as nearby local external coefficients. However, for windward side of overhang the equivalent C_{pi} (under side of roof surface) are recommended as 0.75 (for overhanging slope upward) and 1.25 (for overhanging slope downward) [**6.2.2.7**]. On lee-ward side of overhang, the C_{pi} is equal to the C_{pe} of adjoining wall. The net maximum pressure coefficients are given below and shown in Fig. 17(b).

Adjacent to Regions

H_l	H_e	L_e	L_l
−1.8 − 0.75 = − 2.55	−2.0 − 1.25 = −3.25	−2.0 − 1.25 = − 3.25	−1.8 − 1.25 = − 3.05

It is to be noted that these values are the maximum coefficients for which the roof in that portion shall be designed, regardless of the direction of wind. It is assumed that the under pressure on the side overhangs H_e and L_e will experience an upward force of 1.25, as also the overhang beyond L_1 and L_2, for some wind direction (for wind from 180° in the latter case). Also that regions H_2 and L_2 are absent in the present example. Since the extent of region (H_1 + H_e) is equal to W, the width of the structure is W = L/2.

Design Pressure Coefficients on Roof

C_p (net) = C_{pe} − C_{pi}, is calculated for different direction of wind as given below :

For $\theta = 0°$;
C_p (region H) = −0.8 − (0.8) = −1.6
C_p (region L) = −0.5 − (0.8) = −1.3

For $\theta = 45°$;
C_p (H) = −1.0 − (0.8) = −1.8
C_p (L) = −0.6 − (0.8) = −1.4

For $\theta = 90°$;
C_p (windward 4 m) = −0.9 − (−0.5) = −0.4
C_p (rest of the portion) = −0.5 − (−0.5) = 0.0

For $\theta = 135°$;
C_p (H) = −0.5 − (−0.4) = −0.1
C_p (L) = −1.0 − (−0.4) = −0.6

For $\theta = 180°$;
C_p (H) = −0.2 − (−0.4) = +0.2
C_p (L) = −1.0 − (−0.4) = −0.6

It is clear from the above calculations that C_p for region H is −1.8 ($\theta = 45°$) and +0.2 ($\theta = 180°$) and for region L is −1.4 ($\theta = 45°$).

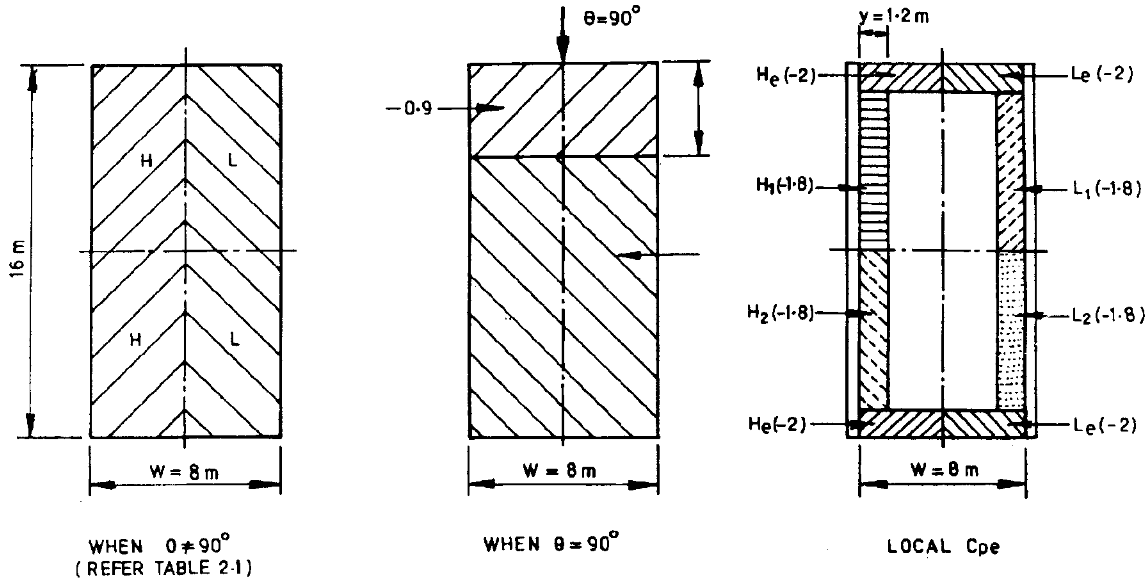
Local Design Pressure Coefficients on Roof

The net pressure coefficients on the edges of the roof and overhangs are shown in Fig. 17(b).

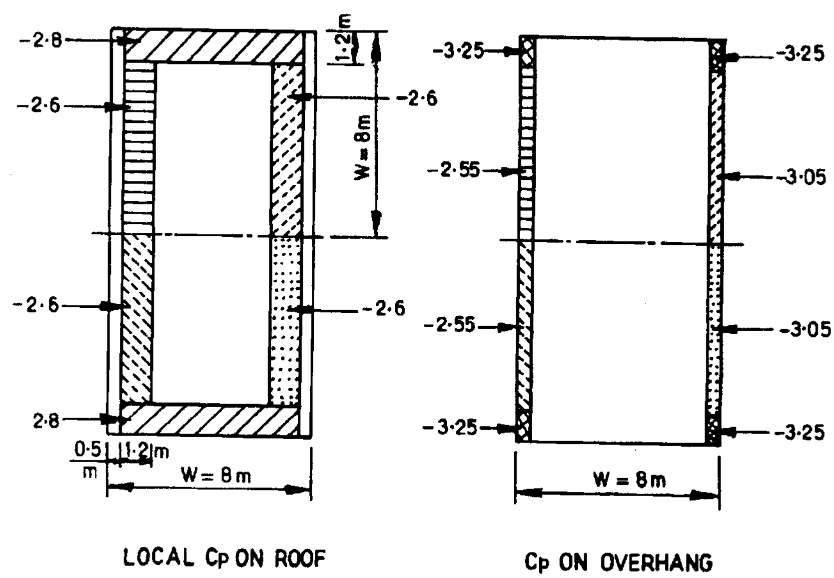
Pressure Coefficients on Walls [**6.2.2.1** and Table 4]

Since one of the walls is open, some care has to be exercised in assessing the internal and external pressure on walls. The internal pressures are to be taken from [Fig. 3], for the case $B/L < 1.0$. The external pressures for the solid walls as well as the pressures in the local region at the corners are to be taken from the appropriate cases of [Table 4]. The external pressures for the case $h/w = 1/2$ and $l/w = 2.0$ are shown in Fig. 17(c). Taking the internal pressure from [Fig. 3], the net pressure on the wall is given in Fig. 17(d). Design must be checked for all the wind loadings conditions (positive and negative) and for each wind direction. C_p for corners of wall are given in Fig. 17(e).

Since d/h or d/b is not greater than 4, the building will not experience frictional drag.

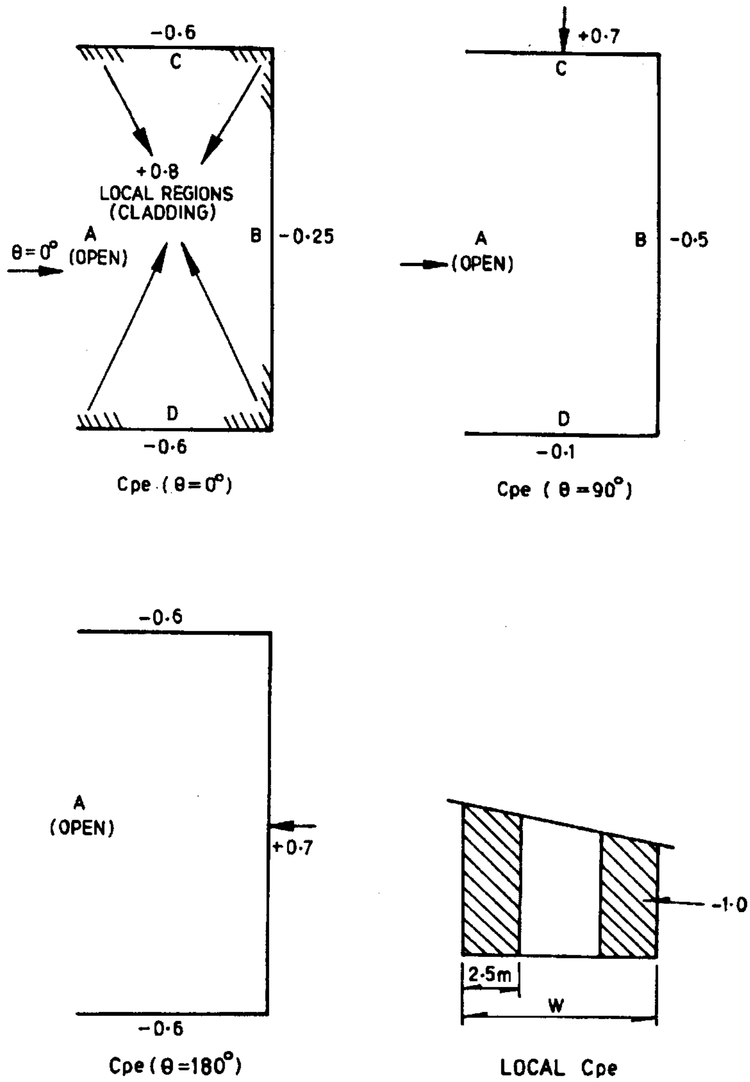


17 (a) C_{pe} for Roof



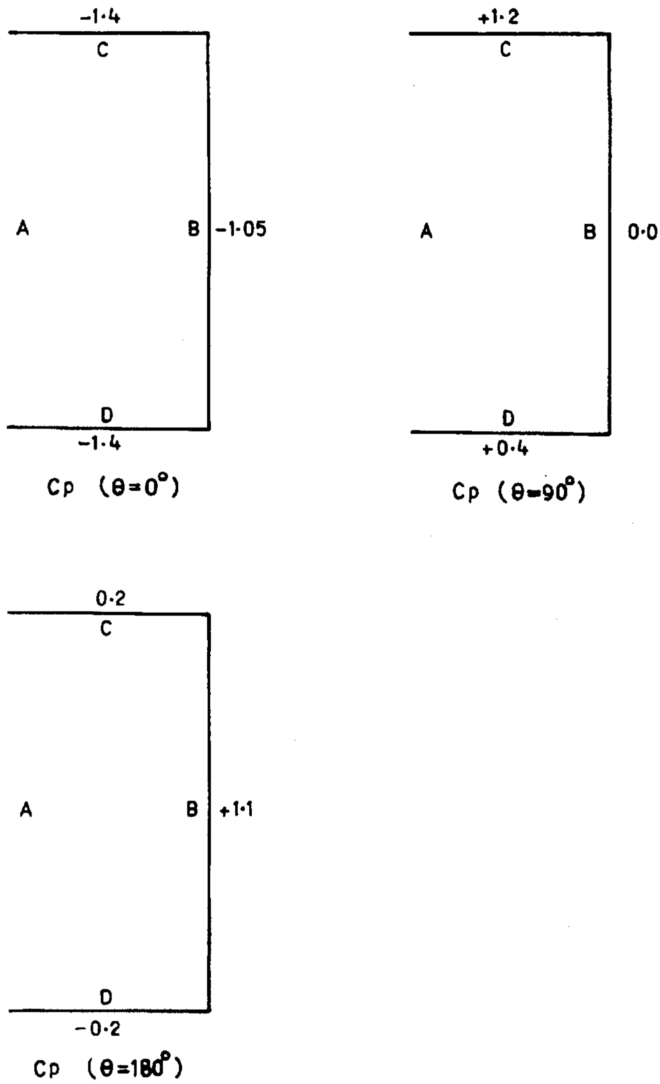
17 (b) C_p (Local) for Roof

FIG. 17 A RECTANGULAR CLAD BUILDING WITH MONOSLOPE ROOF (*Continued*)



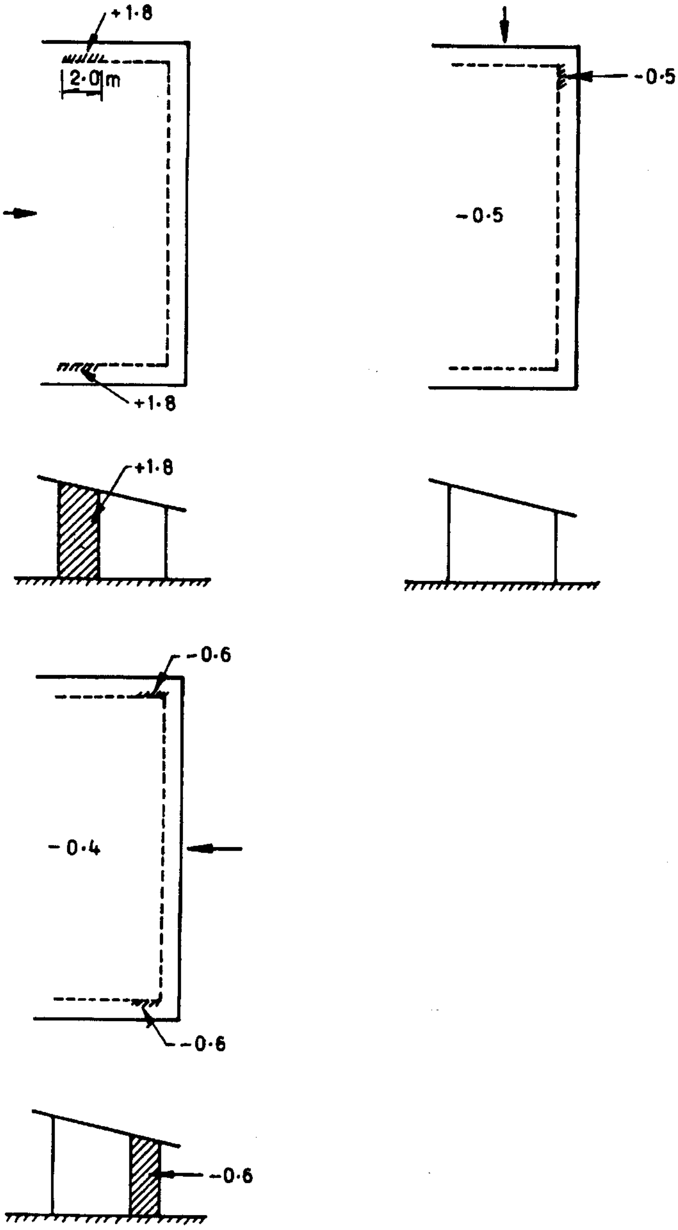
17 (c) C_{pe} for Walls

FIG. 17 A RECTANGULAR CLAD BUILDING WITH MONOSLOPE ROOF *(Continued)*



17 (d) C_p for Walls for Various Angle of Attack of Wind

FIG. 17 A RECTANGULAR CLAD BUILDING WITH MONOSLOPE ROOF (*Continued*)



17 (e) C_p on Corners of Vertical Walls

FIG. 17 A RECTANGULAR CLAD BUILDING WITH MONOSLOPE ROOF

Example 3. *The rectangular clad building of Example 2 has been considered to be situated on a hill having a slope of 9.5° on one side and 19.0° on the other with walls on all sides with opening less than 5 percent [***5.3.3.1** *and Appendix C].*

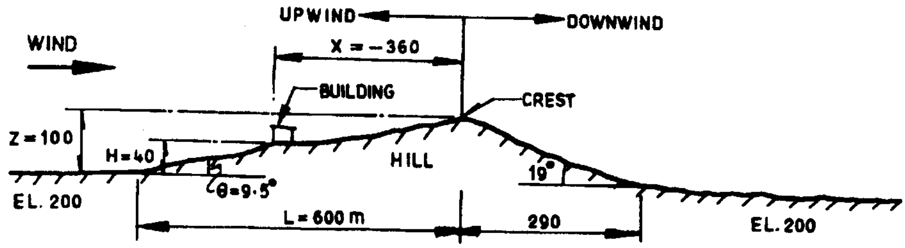
Given:

The approximate features of topography are given in Fig. 18.

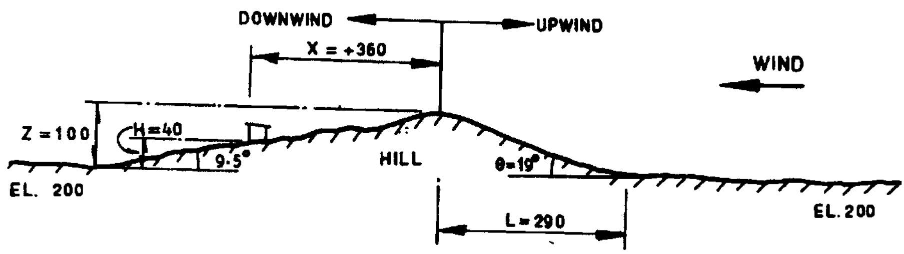
The building is located at 40 m from mean ground level.

The slopes on either side of crest are greater than 3°, hence it is considered as hill.

In such cases, the total methodology of solution remains the same except that the value of k_3 is to be calculated as per [Appendix C].



18(a) Wind Direction from Left to Right



(b) Wind Direction from Right to Left

FIG. 18 A BUILDING WITH MONOSLOPE ROOF ON A HILL SLOPE (*Continued*)

When Wind Blows Left to Right — Fig. 18(a)

Upwind slope $\theta = 9.5°$
Downwind slope $\theta = 19.0°$
Effective horizontal length, $L_e = L = 600$ m
Distance of building (X) from crest $= -(L - H \cot \theta)$
$= -(600 - 40 \cot 9.5°) \approx -360$ m
$H/L_e = 0.067$ and $X/L_e = -0.60$
(Since the building is situated on upwind slope, negative sign is used)

$k_3 = 1 + C_s$
For $\theta = 9.5°$ ($3° < \theta \leq 17°$), $C = 1.2\ (100/600) = 0.2$

$s \approx 0.20$ [Fig. 15]

Hence $k_3 = 1 + 0.2 \times 0.20 = 1.04$

When Wind Blows from Right to Left — Fig. 18(b)

Upwind slope $\theta = 19°$
Downwind slope $\theta = 9.5°$
Effective horizontal length $L_e = z/0.3 = 100/0.3 = 333.33$ m
Distance of building (X) from crest $\approx +360$ m
$H/L_e = 0.12$ and $X/L_e = +1.08$

$k_3 = 1 + C_s$
For $\theta = 19° > 17°$, $C = 0.36$, $s \approx 0.35$ [Fig. 15]
Hence $k_3 = 1 + 0.36 \times 0.35 = 1.126$

NOTE — For design, greater of the two values of k_3 is considered.

Thus,
$V_{10} = 39 \times 0.92 \times 1.0 \times 1.126 = 40.40$ m/s
$p_{10} = 0.6 \times (40.40)^2 = 979.34$ N/m^2

The rest of the procedure is similar to those used for Example 2 except that the pressure on all the walls shall be taken from [Table 4] only (not on three walls only) and the internal pressure shall be taken as recommended in **[6.2.3]**, one observes that the internal pressure coefficient can be either +0.2 or –0.2 and the value which produces greater distress must be considered for design.

External Pressure Coefficient on Roof

These values shall be taken as given in Fig. 18(c) together with the Table 9 given in it.

NOTES

1 Wall pressures indicated when wind is coming at θ = 180°, are the reverse of those given at θ = 0°, that is, +0.7 on *B*, –0.25 on *A*, –0.6 on *C* and *D*, –1.0 on the near edge *F*.

2 Wall pressures indicated when wind is coming at θ = 270° are the reverse of those given at θ = 90°, that is, +0.7 on *D*, –0.1 on *C*, –0.5 on *A* and *B* and –1.0 on edge *H*.

Table 9 External Pressure Coefficient on Roof (C_{pe})

θ	*H*	*L*	Edges Overhang									
			H_1	H_e	L_e	L_1	E_1	E_2	E_3	E_4	E_5	E_6
0°	–0.8	–0.5	–1.8	–2.0	–2.0	–1.8	–1.8	–2.0	–2.0	–2.0	–2.0	–1.8
45°	–1.0	–0.6	–1.8	–2.0	–2.0	–1.8	–1.8	–2.0	–2.0	–2.0	–2.0	–1.8
90° (4 m Windward)	–0.9	–0.9	–1.8	–2.0	–2.0	–1.8	–1.8	–2.0	–2.0	–2.0	–2.0	–1.8
90° (4 m to 16 m)	–0.5	–0.5	–1.8	–2.0	–2.0	–1.8	–1.8	–2.0	–2.0	–2.0	–2.0	–1.8
135°	–0.5	–0.5	–1.8	–2.0	–2.0	–1.8	–1.8	–2.0	–2.0	–2.0	–2.0	–1.8
180°	–0.2	–1.0	–1.8	–2.0	–2.0	–1.8	–1.8	–2.0	–2.0	–2.0	–2.0	–1.8

Fig. 18(d) gives the external pressures on vertical walls.

Design Pressure Coefficient

From the values of the external and internal pressures coefficients, the following design pressures coefficients are worked out :

Walls

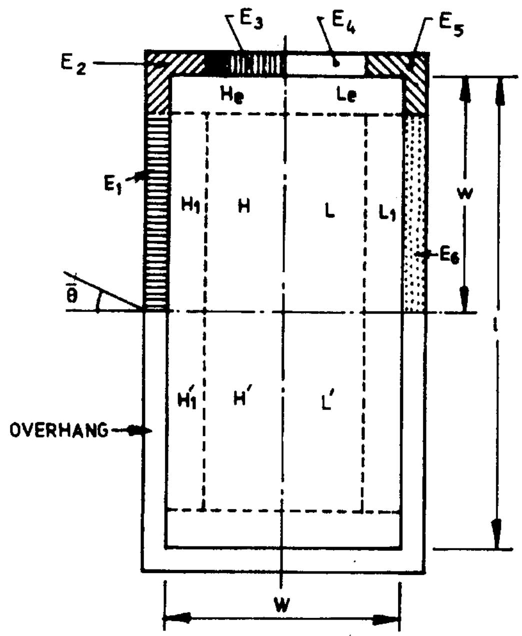
Face *A* {(+0.7 – (–0.2)} = +0.9 and {–0.2 – (0.25)}= –0.45
Face *B* {+0.7 – (–0.2)} = +0.9 and –0.45
Face *C* + 0.9 and {–0.6 – (0.2)} = –0.8
Face *D* + 0.9 and –0.8
Edges *E′*, *E′* and *F*, *F′*, –1.2,
Edges *G*, *G′* and *H*, *H′*, –1.2.

Roof (excluding overhangs)

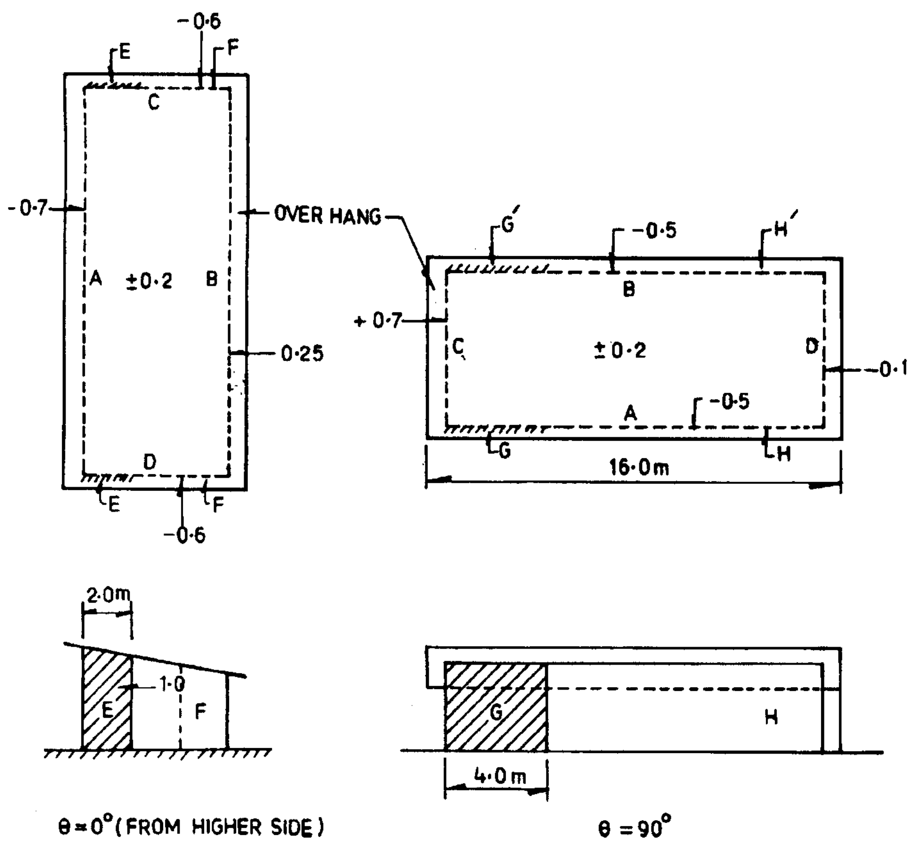
$H, H', L, L' = -1.2$
$H_1, H_1', L_1, L_1' = -2.0$
$H_e, H_e', L_e, L_e' = -2.2$

Overhang

Some care is required in assessing the under pressure on the corner pressure E_2 and E_5. They are taken as equivalent to 'sloping downwards' **[6.2.2.7]** for wind coming from θ = 90°, to be conservative. Portion E_1, definitely slopes upwards for wind at θ = 0° and E_6 slopes downwards for wind at θ = 180°. These lead to the following design pressures for E_1 and E_6 = (– 1.8 – 0.75) = –2.55, E_2 to E_5 = –2.0 – 1.25 = –3.25. The shear on the foundation is observed to be maximum at θ = 0°.



18 (c) C_{pe} on Roof, its Edges and Over Hanging Portion All Round



18 (d) Pressure on Vertical Walls

FIG. 18 A BUILDING WITH MONOSLOPE ROOF ON A HILL SLOPE

Example 4. *A multispan saw tooth shaped roof building (Fig. 19) [Table 17].*

Given:

Physical parameters:

Height (h)	:	6.0 m
Width (w')	:	12.0 m
Length (w)	:	60.0 m
No. of Units	:	4
Roof angle (α)	:	As shown in Fig. 19

Openings on sides : Maximum 5-20 percent of wall area
External surface of walls : Rough (Corrugated)

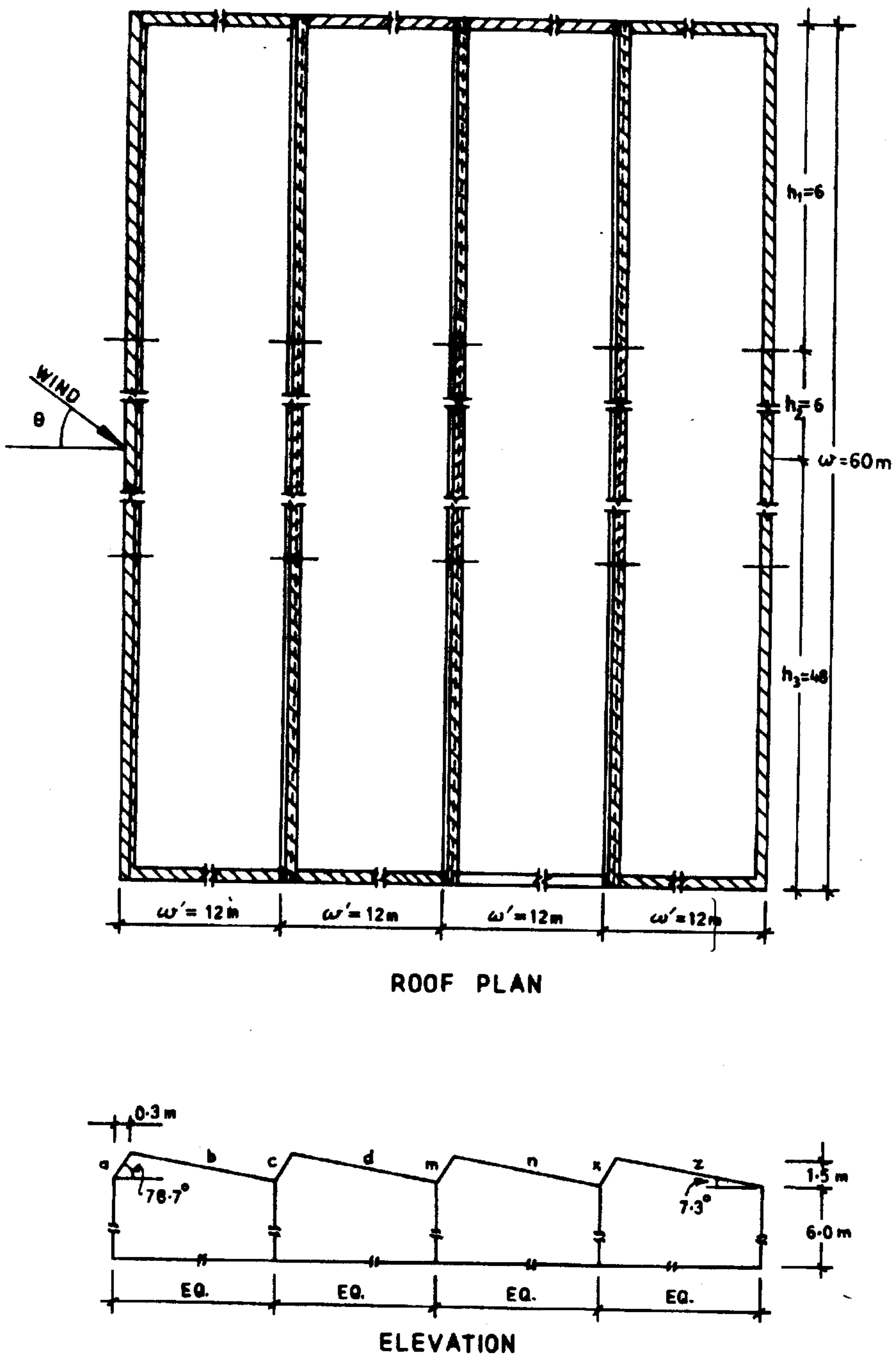


FIG. 19 BUILDING WITH MULTISPAN SAW TOOTH SPEED ROOF (*Continued*)

Wind Data

Wind zone : 2 (Basic wind speed = 39 m/s)
Terrain Category : 2 **[5.3.2.1]**
Class of structure : *C* **[5.3.2.2]**
Topography: Almost flat

Design Wind Speed (V_z)

$V_z = V_b . k_1 . k_2 . k_3$
V_b = 39 m/sec
k_1 = 1.00
k_2 = 0.93
k_3 = 1.00
$V_z = 39 \times 1 \times 0.93 \times 1 = 36.27$ m/s

Design Wind Pressure (p_z)

$p_z = 0.6 \times (V_z)^2 = 0.6\,(36.27)^2 = 789.31\ \text{N/m}^2$

Wind Load Calculation

Internal Pressure Coefficient **[6.2.3** and **6.2.3.2]**
$C_{pi} = \pm 0.5$

External Pressure Coefficient **[6.2.2.6** and Table 17]

Region	*a*	*b*	*c*	*d*	*m*	*n*	*x*	*z*	Local
C_{pe} for									
θ = 0°	+0.6	–0.7	–0.7	–0.4	–0.3	–0.2	–0.1	–0.3	–2.0 and –1.5
θ = 180°	–0.5	–0.3	–0.3	–0.3	–0.4	–0.6	–0.6	–0.1	

Region	h_1	h_2	h_3
C_{pe} for			
θ = 90°	–0.8	–0.6	–0.2
θ = 270°	–0.2	–0.6	–0.8

Design Pressure Coefficients on Roof

C_p (net) = $C_{pe} - C_{pi}$

$h = 6$ m, $h_1 = h_2 = h = 6$ m, $h_3 = 48$ m
Width on either side of ridge = 0.1 w' = 1.2 m

C_p (net) for roof

Region	*a*	*b*	*c*	*d*	*m*	*n*	*x*	*z*	Local
C_p for θ = 0° and C_{pi} = +0.5	+0.1	–1.2	–1.2	–0.9	–0.8	–0.7	–0.6	–0.8	—
C_p for θ = 0° and C_{pi} = –0.5	+1.1	–0.2	–0.2	+0.1	+0.2	+0.3	+0.4	+0.2	—
C_p for θ = 180° and C_{pi}= +0.5	–1.0	–0.8	–0.8	–0.8	–0.9	–1.1	–1.1	–0.6	—
C_p for θ = 180° and C_{pi}= –0.5	0.0	+0.2	+0.2	+0.2	+0.1	–0.1	–0.1	+0.4	—

Design C_p (net) for θ = 0° and 180°

Region	*a*	*b*	*c*	*d*	*m*	*n*	*x*	*z*	Local
Design Pressure Coefficient	+1.1	+0.2	+0.2	+0.2	+0.2	+0.3	+0.4	+0.4	–2.5 and –2.0
	–1.0	–1.2	–1.2	–0.9	–0.9	–1.1	–1.1	–0.8	

These coefficients have been marked in Fig. 19 (a) and (b).

C_p (net) for $\theta = 90°$

Region	h_1	h_2	h_3
when,			
$C_{pi} = +0.5$	−1.3	−1.1	−0.7
$C_{pi} = -0.5$	+0.3	−0.1	−0.3

These coefficients have been indicated in Fig. 19(c).

Final design C_p are:

Region	*a*	*b*	*c*	*d*	*m*	*n*	*x*	*z*
C_p	+1.1	−1.2	−1.2	−0.9	−0.9	−1.1	−1.1	−0.8

The design coefficients for roof cladding are given in Fig. 19(d).

Design Pressure Coefficient for Walls [**6.2.2.1** and Table 4]

As per the definition given in [Table 4].
$w = 48$ m, $l = 60$ m, $h = 6$ m
$h/w = 0.125$ and $l/w = 1.25$

For $h/w \leq 0.5$ and $1 < l/w \leq 1.5$, values of C_p are

Region	*A*	*B*	*C*	*D*	Local
C_{pe} for					
$\theta = 0°$	+0.7	−0.2	−0.5	−0.5	−0.8
C_{pi}=+0.5	−0.5	−0.5	−0.5	−0.5	−0.5
C_{pi}=−0.5	+0.5	+0.5	+0.5	+0.5	+0.5
C_p for $\theta = 0°$	+0.2	−0.7	−1.0	−1.0	−1.3
	+1.2	+0.3	+0.0	+0.0	−0.3
$\theta = 90°$	−0.5	−0.5	+0.7	−0.2	−0.8
C_{pi} =−0.5	+0.5	+0.5	+0.5	+0.5	+0.5
C_{pi} =+0.5	−0.5	−0.5	−0.5	−0.5	−0.5
C_p, $\theta = 90°$	+0.0	+0.0	+1.2	+0.3	−0.3
	−1.0	−1.0	+0.2	−0.7	−1.3

Considering the variability of the wind direction, the C_p (design) for all sides is +1.2 and −1.0 and C_p (Local) −1.3 for corner strips on all four corners of $0.25w = 12$ m width-wise and $0.25\ l = 15$ m length-wise and are indicated in Fig. 19(e). Also extreme values have to be recommended for the range of internal pressures.

Calculation of Frictional Force [6.3.1 and Table 17]

When wind blows parallel to longer wall, that is, $\theta = 90°$ and $270°$, then

$h = 6$ m, $b = 4$, $w' = 48$ m, $d = l = 60$ m
$d/h = 10$ and $h < b$

The total frictional drag force (on both the vertical walls and roofs) is given by

$$F' = \underbrace{C_f' (d - 4h)\, b.p_d}_{\text{drag on roof}} + \underbrace{C_f' (d - 4h)\, 2h.p_d}_{\text{drag on wall}}$$

Here $C_f' = 0.02$ for corrugated sheets as external surface with corrugations across the wind direction.

$F' = 0.02\,(60 - 4 \times 6) \times 48 \times 789.31 + 0.02\,(60 - 4 \times 6) \times 2 \times 6 \times 789.31$
$= 27\,278.55 + 6\,819.64 = 34\,098.19$

The frictional drag force on various structural elements is distributed such that 50 percent of this frictional drag is resisted by windward 1/3rd length (20 m) alongwith other loads and balance 50 percent force by rest 2/3rd length (40 m) in each direction $\theta = 90°$ and $270°$.

Frictional Drag Force for Roof

Frictional drag force = 27 278.55 N, half of this, that is
27 278.55/2 = 13 639.28 N is to be resisted by the area = $20.00 \times 48\ m^2$
Force per unit area = 13 639.28/(20.00 × 48) = 14.21 N/m^2

13 639.28 N is to be resisted by another 2/3rd length that is, 40.00 m
Force per unit area = 13 639.28/(40.00 × 48) = 7.10 N/m^2

The frictional force distribution is indicated in Fig. 19(f).

Frictional Drag Force for Wall

Frictional drag = 6 819.64 N
3 409.82 N is to be resisted by 20.00 m length
Force per unit length = 3 409.82/20.00 = 170.50 N/m

Other half 3 409.82 N is to be resisted by 40.00 m length
Force per unit length = 3 409.82/40.00 = 85.25 N/m

When wind blows parallel to width ($\theta = 0°$ and $180°$), then

$h = 6$ m, $b = l = 60$ m, $d = 4w' = 48$ m, $d/h = 8$ and $h < b$

$F' = 0.02\,(48 - 4 \times 6) \times 60 \times 789.31 + 0.02\,(48 - 4 \times 6) \times 2 \times 6 \times 789.31 = 22.732 + 4.546 = 27.278$

This force is distributed such that half of this frictional drag is to be resisted by windward 1/3rd length (16 m) and balance by rest 2/3rd length (32 m) in each direction $\theta = 0°$ and $180°$.

For Roof

11 366.0 N is to be resisted by the 1/3rd length that is, 48/3 = 16 m
Force per unit area = 11 366.00/(16.00 × 60) = 11.84 N/m^2
11 366.00 N is to be resisted by the 2/3rd length that is, 48 × 2/3 = 32 m
Force per unit area = 11 366.00/(32.00 × 60) = 5.92 N/m^2

The frictional force is indicated in Fig. 19(g)

For Wall

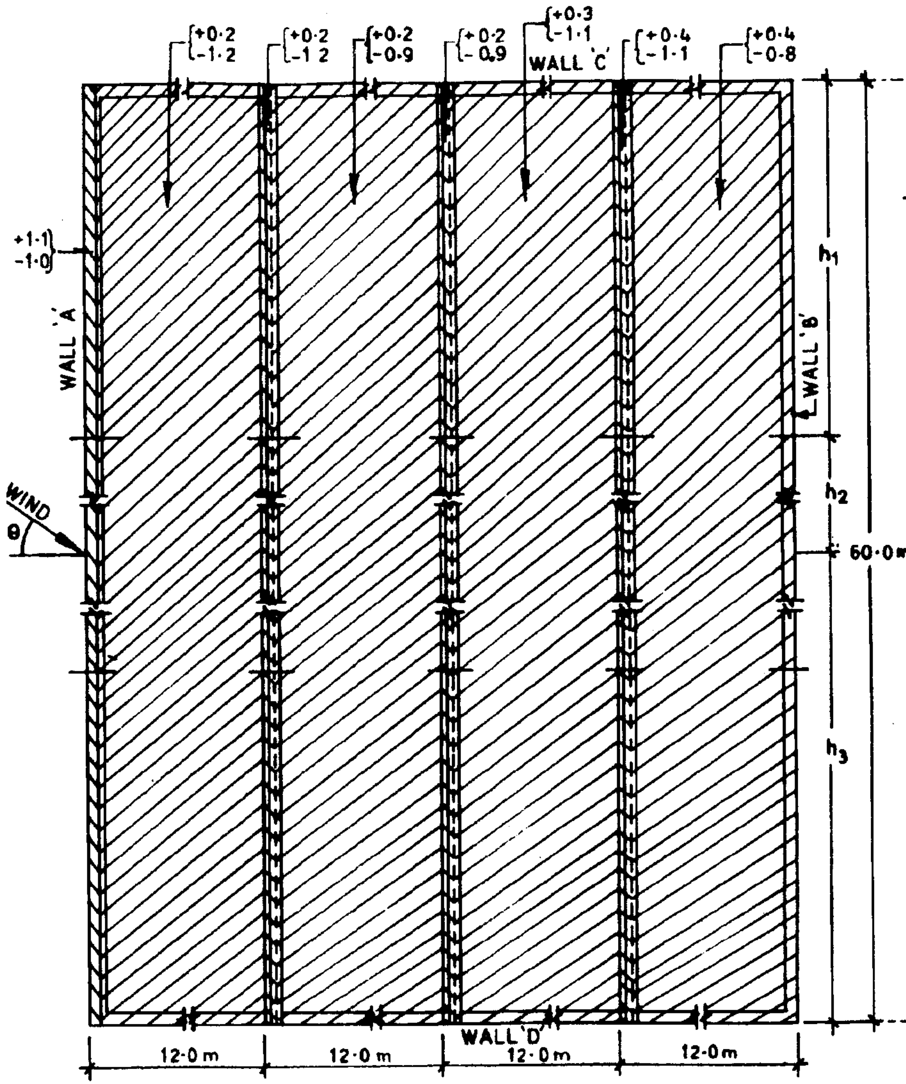
2 273.0 N is to be resisted by the length 20.00 m
Force per unit length = 2 273.0/20.00 = 113.65 N/m

2 273.0 N is to be resisted by the length 40.00 m
Force per unit length = 2 273.0/40.00 = 56.83 N/m

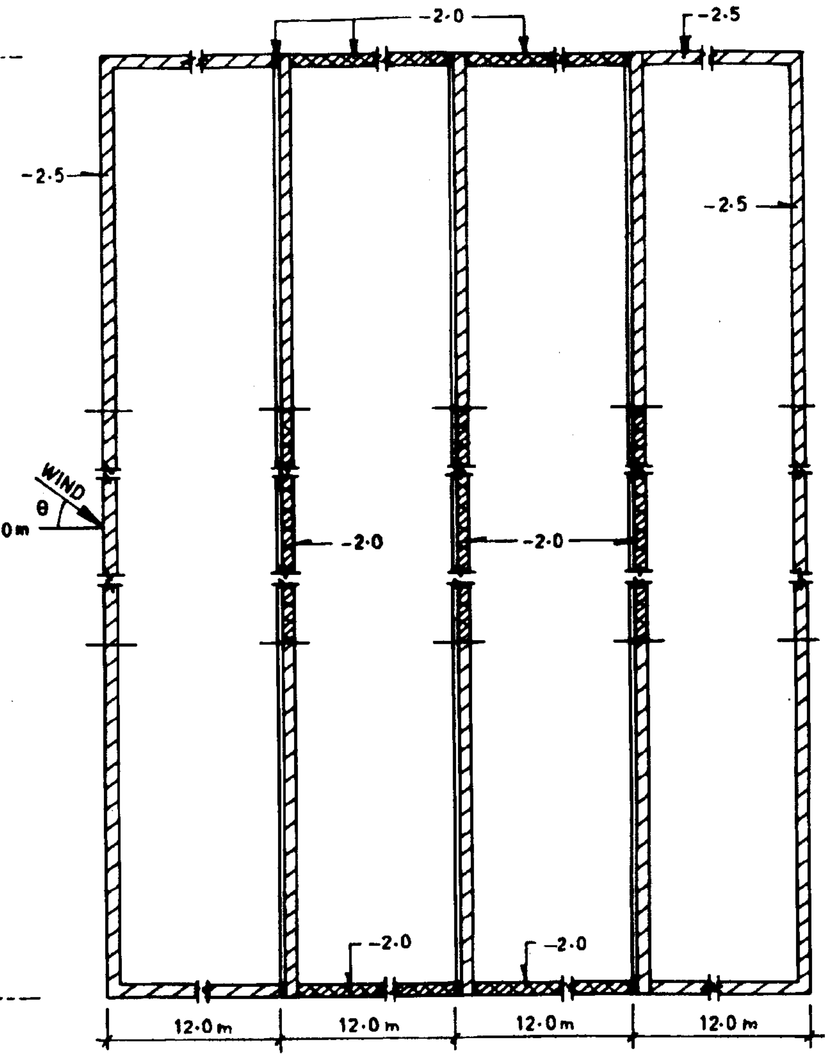
Calculation of Force

$F = C_p . p_d$ per unit area where $p_d = 789.31\ N/m^2$

C_p	*Force*, N/m^2	C_p	*Force*, N/m^2
+1.1	868.24	−1.3	−1 026.10
−1.2	−947.17	−0.7	−552.52
−0.9	−710.38	−2.5	−1 973.28
−1.1	−868.24	−2.0	−1 578.62
−0.8	−631.45	+1.2	+947.17

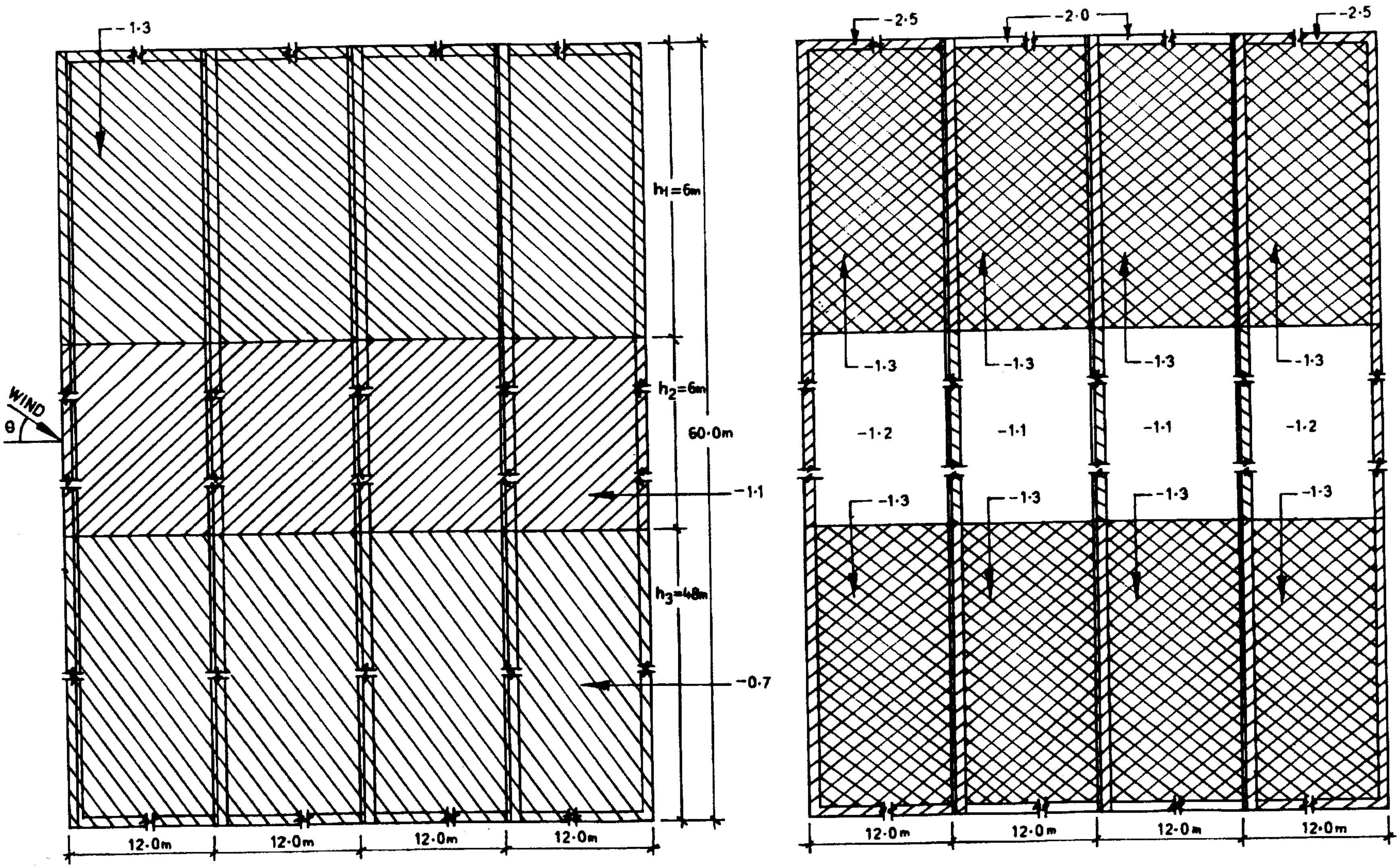


19 (a) C_p (Overall) θ = 0°, θ = 180° Roof Plan



19 (b) C_p (Local) Roof Plan

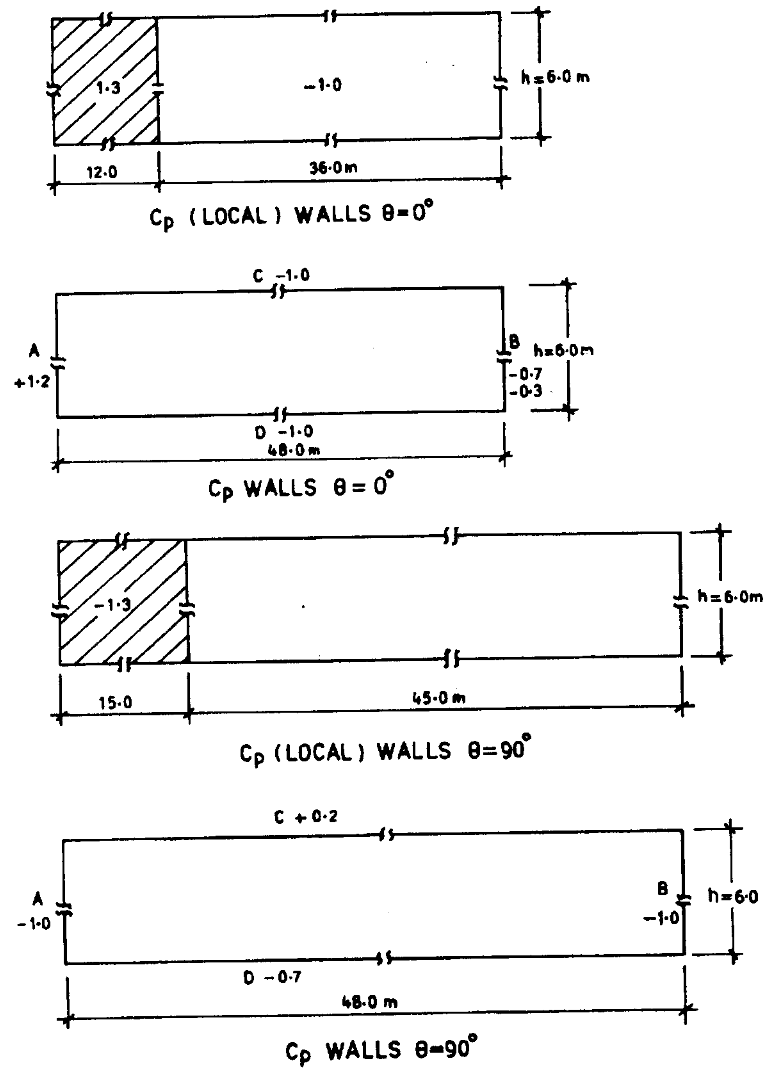
FIG. 19 BUILDING WITH MULTISPAN SAW TOOTH SPEED ROOF (*Continued*)



19 (c) C_p (Overall) θ = 90° Roof Plan

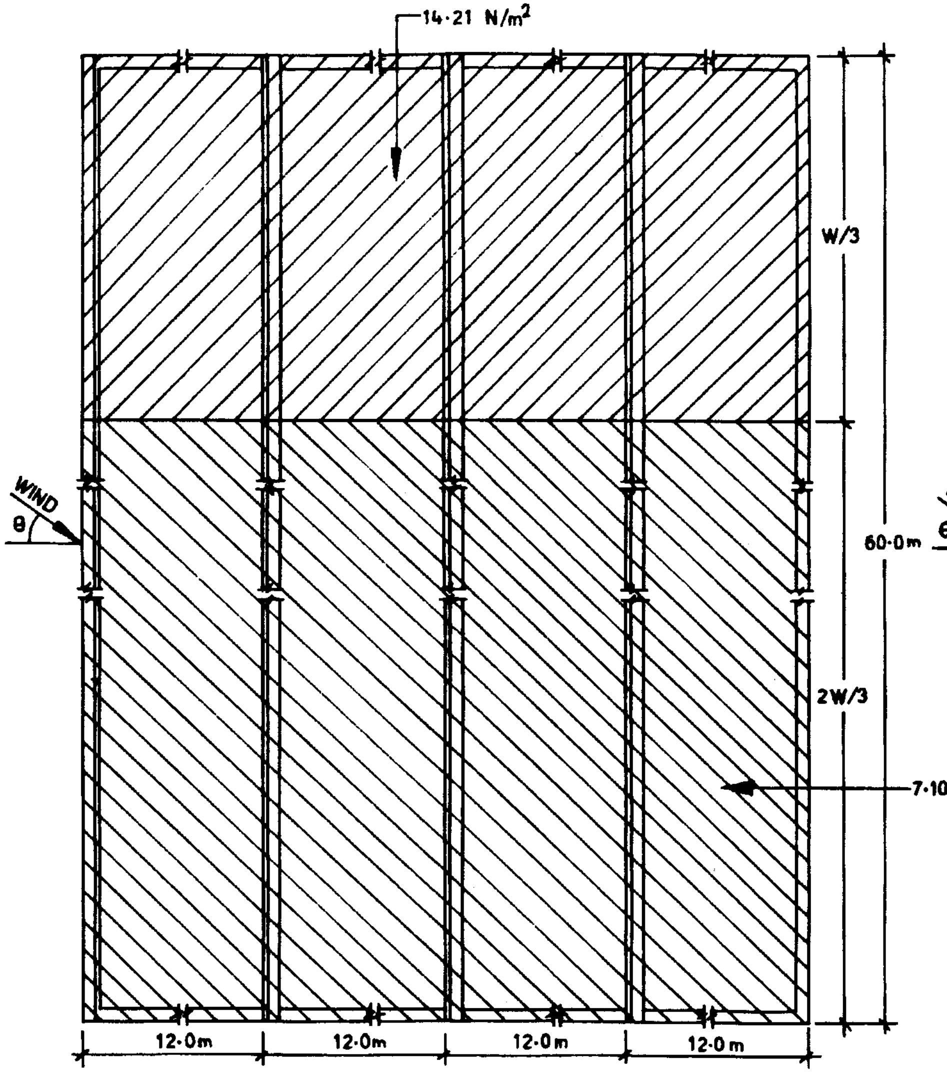
19 (d) C_p Design for Roof Cladding

FIG. 19 BUILDING WITH MULTISPAN SAW TOOTH SPEED ROOF (*Continued*)

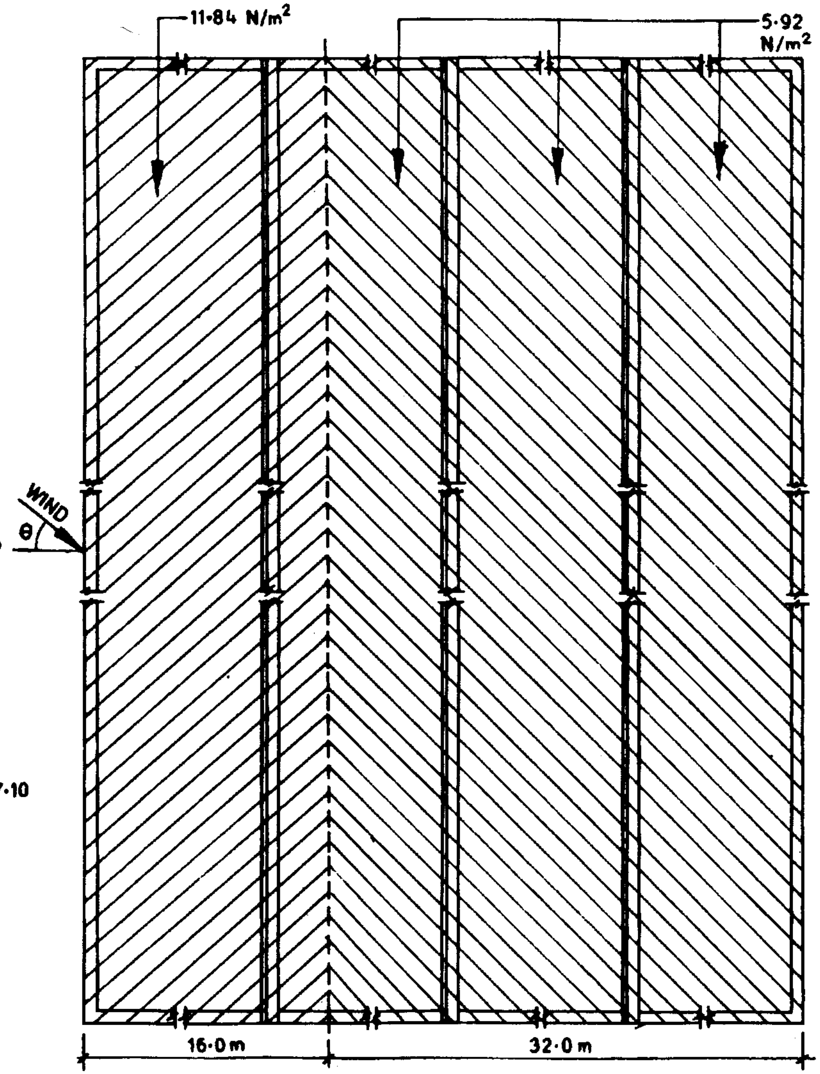


19 (e) C_p for Walls

FIG. 19 BUILDING WITH MULTISPAN SAW TOOTH SPEED ROOF (*Continued*)



19 (f) Frictional Force for θ = 90°



19 (g) Frictional Force for θ = 0°

FIG. 19 BUILDING WITH MULTISPAN SAW TOOTH SPEED ROOF

Example 5 *A rectangular clad building with multispan pitched roof (Fig. 20), [Table 16].*

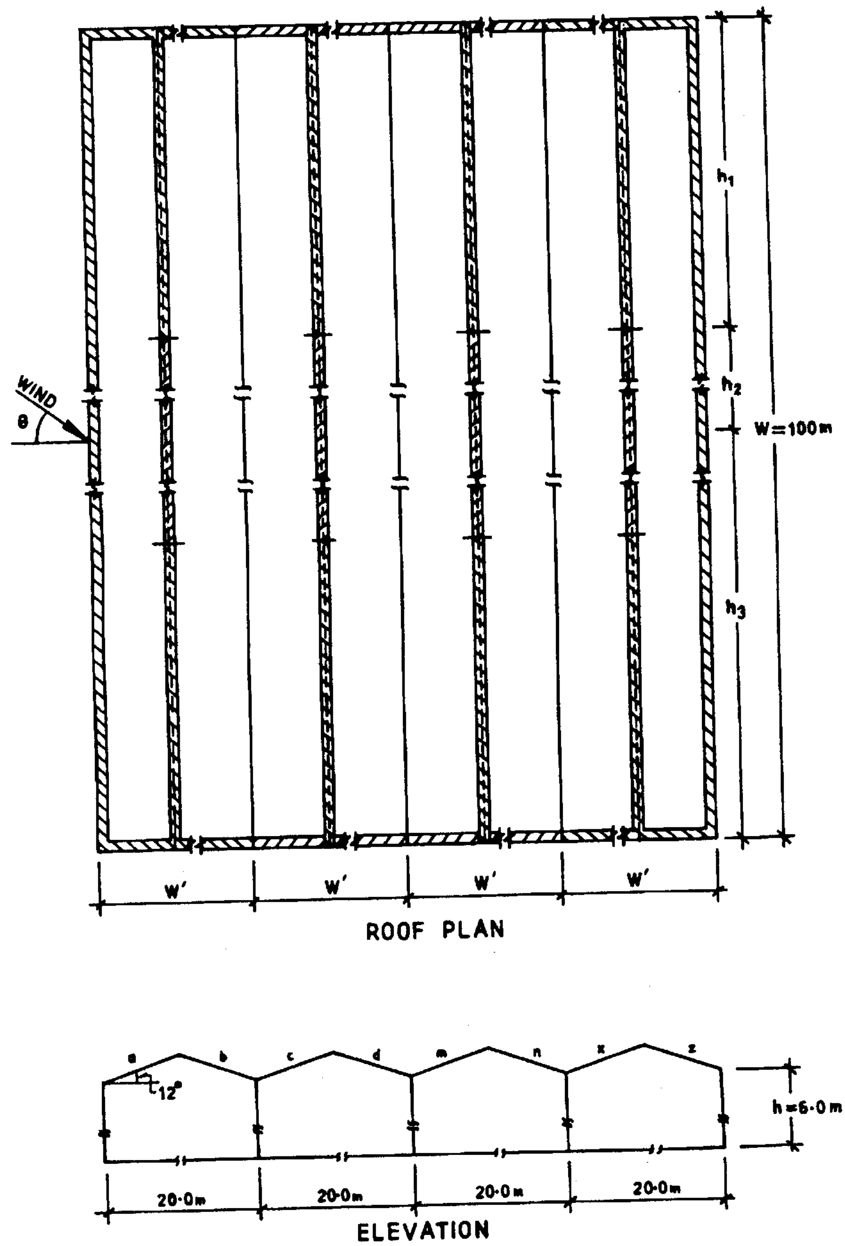


FIG. 20 A BUILDING WITH MULTISPAN PITCHED ROOF (*Continued*)

Given:

Physical Parameters:

Height (h)	:	6.0 m
Width (w')	:	20.0 m
Length (w)	:	100.0 m
No. of Units	:	4
Roof angle (α)	:	12°

Openings on sides : Less than 20 percent of wall area

External surface of walls : Rough (Corrugated)

Number of gable frames in length of 100 m = 21

All spans being equal.

Wind Data

Wind zone : 2 (Basic wind speed = 39 m/s)
Terrain category : 2 **[5.3.2.1]**
Class of structure : C **[5.3.2.2]**
Topography: Almost flat

Design Wind Speed (V_z)

$V_z = V_b . k_1 . k_2 . k_3$
$V_b = 39$ m/s
$k_1 = 1.00$
$k_2 = 0.93$
$k_3 = 1.00$
$V_z = 39 \times 1 \times 0.93 \times 1 = 36.27$ m/s

Design Wind Pressure (p_z)

$p_z = 0.6 \times (V_z)^2 = 0.6\,(36.27)^2 = 789.31\ \text{N/m}^2$

Wind Load Calculation

Internal Pressure Coefficient **[6.2.3** and **6.2.3.2]**

Since openings are less than 20 percent of wall area the value of $C_{pi} = \pm 0.5$

External Pressure Coefficient **[6.2.2.6** and Table 16]

Since the values of pressure coefficients are given in [Table 16] for roof angles of 5°, 10°, 20° ....., the values of C_{pe} for $\alpha = 12°$ are obtained by interpolating between the values at $\alpha = 10°$ and $\alpha = 20°$.

Region	*a*	*b*	*c*	*d*	*m*	*n*	*x*	*z*	Local
C_{pe} for $\theta = 0°$	−1.02	−0.6	−0.4	−0.3	−0.3	−0.3	−0.3	−0.42	−2.0 and −1.5
$\theta = 90°$	−0.42	−0.3	−0.3	−0.3	−0.3	−0.4	−0.6	−1.02	

Region	h_1	h_2	h_3
C_{pe} for $\theta = 90°$	−0.8	−0.6	−0.2
$\theta = 270°$	−0.2	−0.6	−0.8

Design Pressure Coefficients on Roof

C_p (net) $= C_{pe} - C_{pi}$
$h = 6$ m, $h_1 = h_2 = h = 6$ m, $h_3 = 88$ m
Width on either side of ridge, $y = 0.1\,w' = 2.0$ m

Region	*a*	*b*	*c*	*d*	*m*	*n*	*x*	*z*	Local
C_p for $\theta = 0°$ and $C_{pi} = +0.5$	+1.52	−1.1	−0.9	−0.8	−0.8	−0.8	−0.8	−0.92	
C_p for $\theta = 0°$ and $C_{pi} = -0.5$	−0.52	−0.1	+0.1	+0.2	+0.2	+0.2	+0.2	+0.08	
C_p for $\theta = 180°$ and $C_{pi} = +0.5$	−0.92	−0.8	−0.8	−0.8	−0.8	−0.9	−1.1	−1.52	
C_p for $\theta = 180°$ and $C_{pi} = -0.5$	+0.08	+0.2	+0.2	+0.2	+0.2	+0.1	−0.1	−0.52	
Design Pressure Coeff.	−1.52	−1.1	−0.9	−0.8	−0.8	−0.9	−1.1	−1.52	−2.5 and −2.0
	+0.08	+0.2	+0.2	+0.2	+0.2	+0.2	+0.2	+0.08	

These coefficients have been marked in Fig. 20(a) and (b).

Region	h_1	h_2	h_3
C_p for θ = 90° and 270°			
C_p = +0.5	−1.3	−1.1	−0.7
C_p for θ =90° and 270°			
C_p = −0.5	+0.3	−0.1	−0.3

These coefficients have been indicated in Fig. 20(c).

The design coefficients for roof cladding are given in Fig. 20(d) and (e).

Design Pressure Coefficient for Walls **[6.2.2.1** and Table 4]

w = 80 m, l = 100 m, h = 6 m, h/w = 0.075 and l/w = 1.25

For $h/w \leq 0.5$ and $1 < l/w \leq 1.5$, values of C_p are

Region	*A*	*B*	*C*	*D*	Local
C_{pe} for					
θ = 0°	+0.7	−0.2	−0.5	−0.5	−0.8
C_{pi} = +0.5	−0.5	−0.5	−0.5	−0.5	−0.5
C_{pi} = −0.5	+0.5	+0.5	+0.5	+0.5	+0.5
C_p, θ=0°	+0.2	−0.7	−1.0	−1.0	−1.3
	+1.2	+0.3	+0.0	−0.0	−0.3
C_{pe} for					
θ = 90°	−0.5	−0.5	+0.7	−0.2	−0.8
C_{pi} =−0.5	+0.5	+0.5	+0.5	+0.5	+0.5
C_{pi} =+0.5	−0.5	−0.5	−0.5	−0.5	−0.5
C_p, θ = 90°	+0.0	+0.0	+1.2	+0.3	−0.3
	−1.0	−1.0	+0.2	−0.7	−1.3

Considering the variability of the wind direction, the C_p (design) for all for sides is +1.2 and −1.0 and C_p (Local) −1.3 for corner strips on all four corners of $0.25w$ = 20 m width-wise and $0.25 \times l$ = 25 m length-wise and are indicated in Fig. 20 (d) and (e).

Calculation of Frictional Force [6.3.1 and Table 17]

When wind blows parallel to length that is, θ = 90° and 270°, then

h = 6 m, $b = 4w'$ = 80 m, $d = l$ = 100 m, d/h = 16.67 > 4

The frictional drag force is given by

$$F' = \underbrace{C_f' \,(d-4h)\, b.p_d}_{\text{drag on roof}} + \underbrace{C_f' \,(d-4h)\, 2h.p_d}_{\text{drag on wall}}$$

Here C_f' = 0.02 for corrugated sheets as external surface with corrugations across the wind direction .

F' = 0.02 (100 − 24) × 80 × 789.31 + 0.02 (100 − 4 × 6) × 12 × 789.31 = 95 980.09 +14 397.01 = 110 377.1.

The frictional drag force on various structural elements is distributed such that 50 percent of this frictional drag is resist by windward 1/3rd length (33.33 m) alongwith other loads and balance 50 percent force by rest 2/3rd length (66.67m) in each direction θ = 90° and 270°.

For Roof

Frictional drag force = 95 980.09 N
95 980.09/2 = 47 990.05 N is resist by Area = 33.33 × 80 m^2
Force per unit area = 47 990.05/(33.33 × 80) = 17.99 N/m^2
47 990.05 N is resist by another 2/3rd length that is, 66.67 m
Force per unit area = 13 639.28/(66.67 × 80) = 8.99 N/m^2

For Wall

Frictional drag = 14 397.01 N
7 198.50 N is resist by 33.33 m length
that is Force per unit length = 7 198.50/33.33 = 215.97 N/m
Another half that is, 7 189.50 N is resist by 66.66 m length
that is, Force per unit length = 7 198.50/66.66 = 107.97 N/m

When wind blows parallel to width (θ = 0° and 180°), then

$h = 6$ m, $b = l = 100$ m, $d = 4w' = 80$ m, $d/h = 13.33$, $d/b = 0.8$, $d/h > 4$

$F' = 0.02\,(80 - 4 \times 6) \times 100 \times 789.31 + 0.02\,(80 - 4 \times 6) \times 12 \times 789.31 = 88\,402.72 + 10\,608.33 = 99\,011.05$

This force is distributed such that half of this frictional drag is resist by windward 1/3rd length (26.67 m) and balance by rest 2/3rd length (53.33 m) in each direction θ = 0° and 180°.

For Roof

44 201.36 N is resist by the 1/3rd length that is, 80/3 = 26.67 m
Force per unit area = 44 201.36/(26.67 × 100) = 16.57 N/m^2
44 201.36 N is resisted by the 2/3rd length that is, 80 × 2/3 = 53.33 m
Force per unit area = 44 201.36/(53.33 × 100) = 8.29 N/m^2

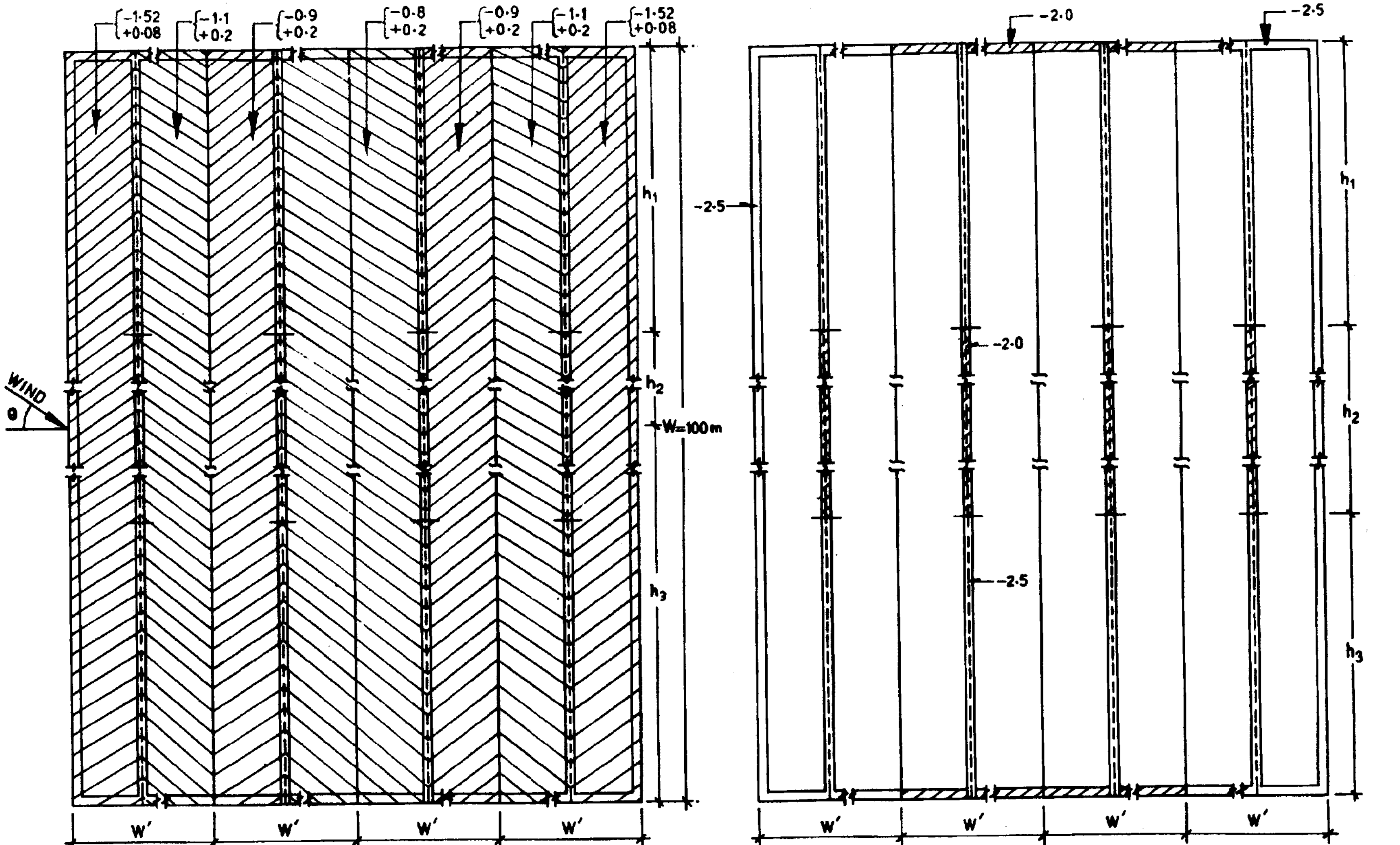
For Wall

5 304.2 N is resist by the length 26.67 m
Force per unit length = 5 304.2/26.67 = 198.88 N/m
5 304.2 N is resist by the length 53.33 m
Force per unit length = 5 304.2/53.33 = 99.46 N/m

Calculation of Force

$F = C_p.p_d$ per unit area, where p_d = 789.31 N/m^2

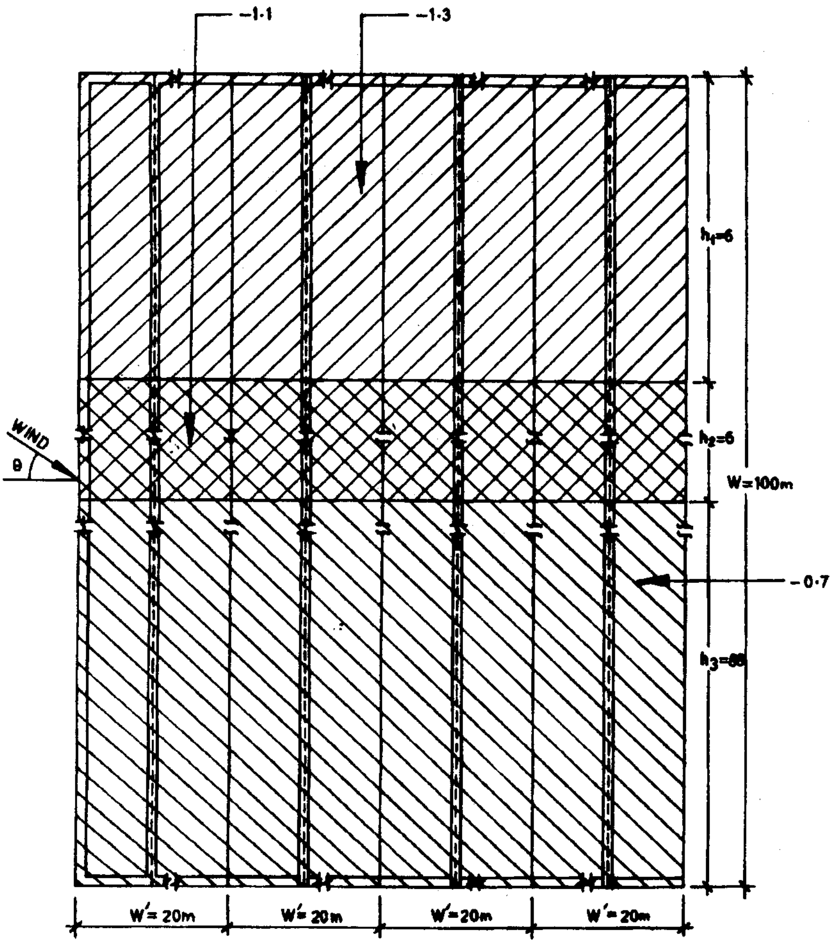
C_p	*Force*, N/m^2	C_p	*Force*, N/m^2
+1.2	947.17	−1.3	−1 026.10
−1.52	−1 199.75	−2.5	−1 973.27
−1.1	−868.24	−2.0	−1 578.62
−0.9	−710.38	−1.0	−789.31
−0.8	−631.45	+1.2	+947.17
−0.7	−552.52		



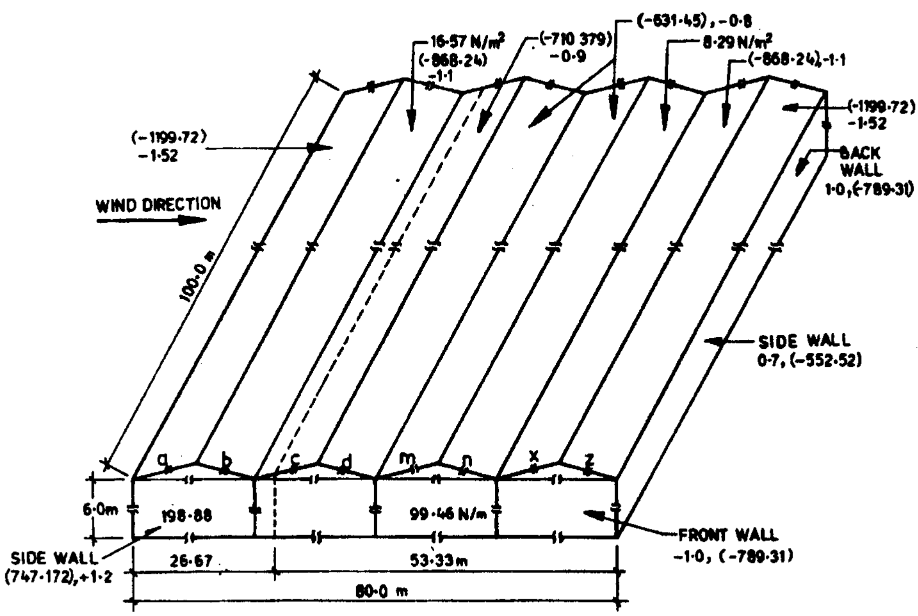
20 (a) Roof Plan C_p (Overall) for θ = 0°, 180°

20 (b) Roof Plan C_p (Local) θ = 0°

FIG. 20 A BUILDING WITH MULTISPAN PITCHED ROOF (*Continued*)

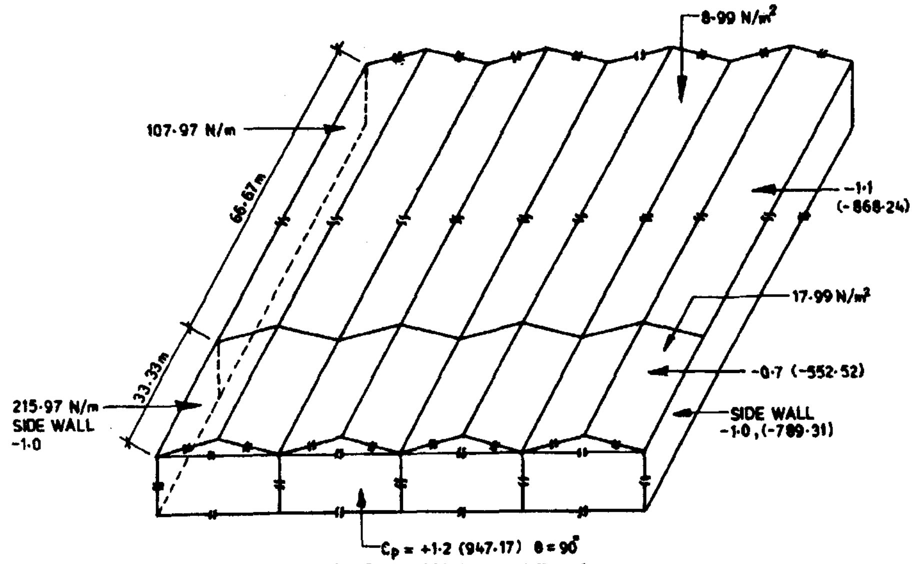


20 (c) Roof Plan C_p (Overall) for θ = 0°



20 (d) Design Wind Presure for Walls and Roof

FIG. 20 A BUILDING WITH MULTISPAN PITCHED ROOF (*Continued*)



20 (e) C_p for Walls and Roofs

FIG. 20 A BUILDING WITH MULTISPAN PITCHED ROOF

Example 6. *A rectangular clad godown building with combined roof (Fig. 21) [Table 20].*
Case 1 — If the Annexe is clad.
Case 2 — If the Annexe is unclad (sides open).

Given:

Physical Parameters:

Height of main building (h_1) :	8.0 m
Height of annexe building (h_2) :	4.0 m
Width of main building (b_2) :	4 m
Width of annexe building (b_1) :	3 m
Slope of roof of main building α :	20°
Slope of roof of annexe building α :	18.4°
Length of building (l) :	18.0 m
Permeability :	> 22 percent of wall area

Wind Data

Wind zone : 2 (basic wind speed = 39m/s)
Terrain category : 2
Class of structure : A
Probable design life of structure : 50 years

Design Wind Speed

V_b = 39 m/sec
k_1 = 1.00
k_2 = 1.00
k_3 = 1.00
$V_z = 39 \times 1 \times 1 \times 1 = 39$ m/s

Design Wind Pressure

$p_z = 0.6(39)^2 = 912.6$ N/m²

Internal Pressure Coefficients

$C_{pi} = \pm 0.7$

External Pressure Coefficients for Roofs [6.2.2.2 and Table 5]

Main Building

$h/w = 8/(4 + 3) = 8/7 = 1.14$, $\alpha = 20°$ and $y = 0.15 \times 7 = 1.05$ m

NOTE — It is not clear from the Code, as to whether the lesser horizontal dimension as seen in the plan including the annexure or of the portion above the annexed should be taken for w. It is recommended that the lesser horizontal dimension at ground level be taken if $h_1 \geq 0.2\, h_2$. If $h_1 < 0.2\, h_2$, w should be the lesser horizontal dimension of the portion above the annexure.

Region	EF	GH	EG	FH
	wind angle $\theta = 0°$		wind angle $\theta = 90°$	
$\alpha = 20°$	–0.7	–0.5	–0.8	–0.6

Figure 21 (a) and (b) give the C_{pe} values for main building including local values.

C_{pe} on roof of annexe building and vertical common wall [**6.2.2.10** and Table 20]

Here $\alpha_1 < 30°$, $b_1 < b_2$ and $h_1/h_2 = 8/4 = 2.0$
$C_{pe} = 0.4\ h_1/h_2 - 0.6 = +0.2$

For portion *a* $C_{pe} = +0.2$ for direction 1
$C_{pe} = -0.4$ for direction 2

For portion *b* $C_{pe} = +0.7$ for direction 1
$C_{pe} = -0.4$ for direction 2

For portions *c* and *d* [Table 5], as given above

C_{pe} for Walls [**6.2.2.1** and Table 4]

$h/w = h_1/(b_2 + b_1) = 7/5 = 1.14$, $0.25w = 0.25 \times 7 = 1.75$ m at $\theta = 0°$ and 4.5 m at $\theta = 90°$ and $l/w = 18/7 = 2.57$

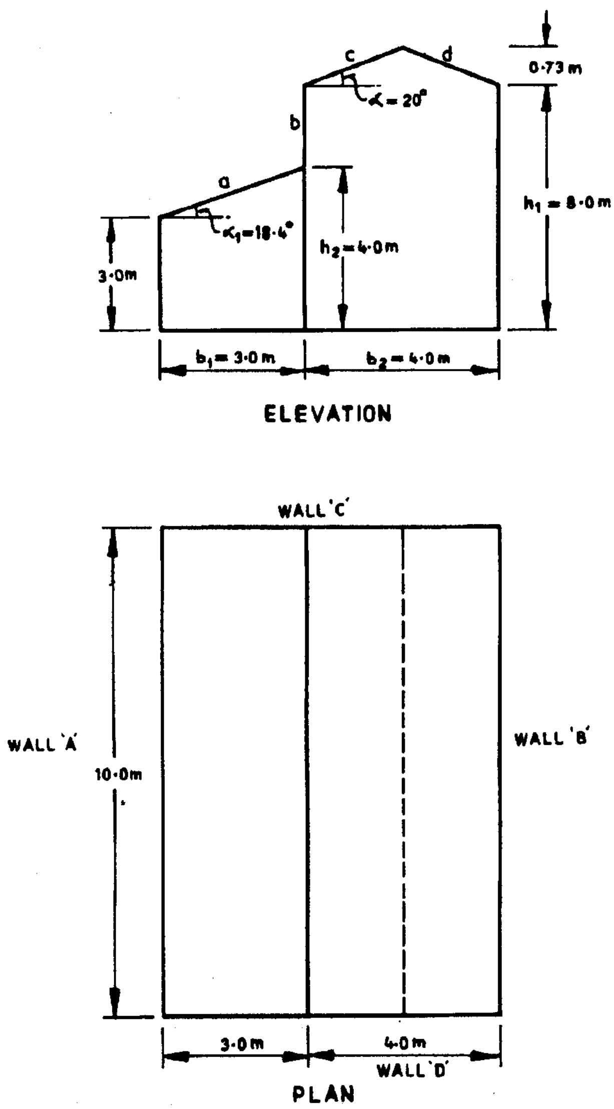


FIG. 21 PITCHED ROOF WITH A VERANDAH (*Continued*)

Region	*A*	*B*	*C*	*D*	Local
when					
$\theta = 0°$	+0.7	−0.3	−0.7	−0.7	−1.1
$\theta = 90°$	−0.5	−0.5	+0.7	−0.1	

C_p(net) for portions c and d [Table 5]

C_p(net) = $C_{pe} - C_{pi}$ and $C_{pi} = \pm 0.7$

For $\theta = 0°$, C_p (net) for region *EF* = − 0.7−0.7 = − 1.4
C_p (net) for region *GH* = − 0.5−0.7 = − 1.2
C_p (net) local = −2.2 and −1.7

For $\theta = 90°$, C_p (net) for region *EG* = −0.8 − 0.7 = −1.5
C_p (net) for region *FH* = −0.6 − 0.7 = −1.3
C_p (net) local = −2.2 and −1.7 for width of 0.6 m

C_p(net) for portions a and b

For portion *a*,

when $\theta = 0°$ $C_p = +0.90$ or $C_p = -0.50$
when $\theta = 180°$ $C_p = -1.1$ or $C_p = +0.3$

For portion *b*,

when $\theta = 0°$ $C_p = +1.4$ or $C_p = 0.0$
when $\theta = 180°$ $C_p = -1.1$ or $C_p = +0.3$

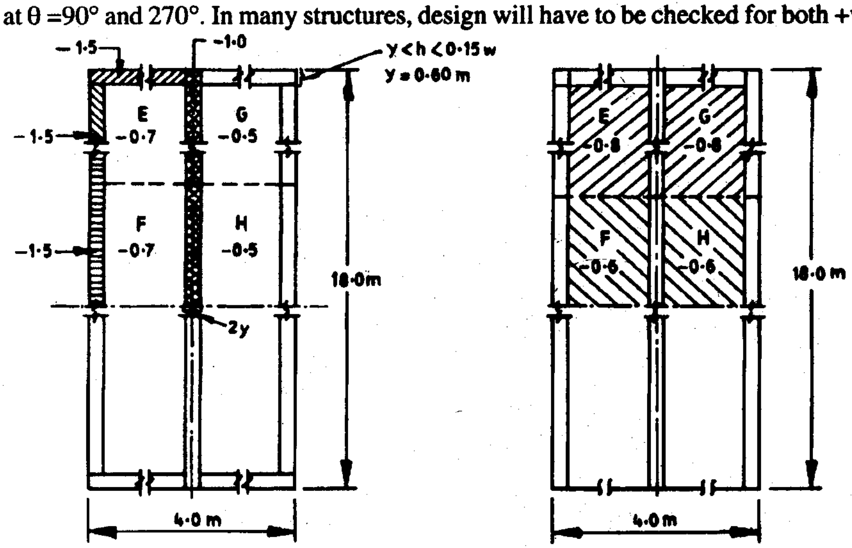
C_p(net) values for roof are shown in Fig. 21 (c) and (d)

C_p(net) for Walls

Region	*A*	*B*	*C*	*D*	Local
When $\theta = 0°$					
	+0.0	−1.0	−1.4	−1.4	
	+1.4	+0.40	+0.0	+0.0	
					−1.8
					−0.4
$\theta = 90°$					
	−1.2	−1.2	+0.0	−0.00	
	+0.2	+0.2	+1.4	+0.60	

These values of C_p(net) on walls are shown in Fig. 21 (e)

It is to be noted that the local pressure of −1.8 must be applied on the vertical wall portion *D* also, as on the front and back walls, at θ =90° and 270°. In many structures, design will have to be checked for both +ve and -ve C_p,



(a) C_{pe} VALUE FOR ROOF (c,d) (θ=0°)

(b) C_{pe} VALUE FOR ROOF (c,d) (θ=90°)

FIG. 21 PITCHED ROOF WITH A VERANDAH (*Continued*)

especially when there are load carrying frames, since some members may go into buckling for one of the loading conditions.

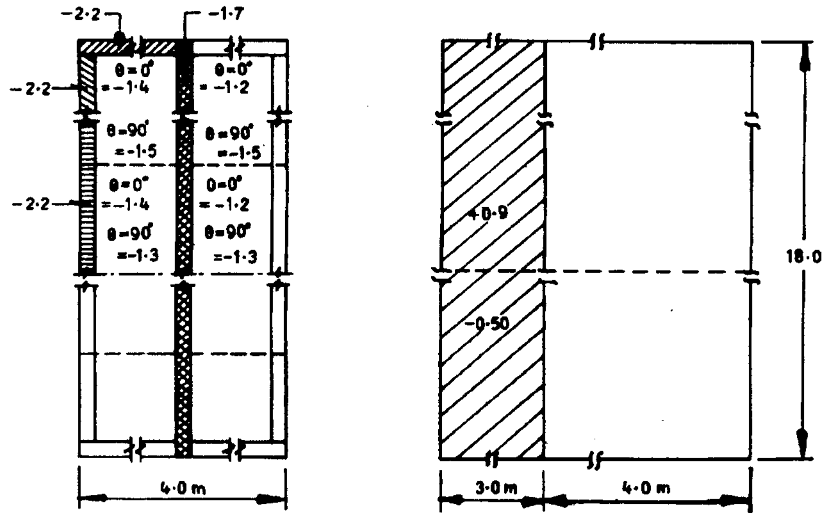
Design pressure coefficients are shown in Fig. 21 (g).

If annexe portion is open except for the overhanging portion, it is treated as overhang **[6.2.2.7]** having $C_{pi} = +1.25$.

Hence $C_p(\text{net}) = 1.45$

The portion of wall *b* below the overhang is to be designed for a pressure coefficient C_{pe} of 0.9 as in [Table 21, portion *H*] for wind from left to right and $C_{pi} = \pm 0.7$ inside pressure and $C_{pe} = -0.4$ and $C_{pi} = \pm 0.7$ for wind from right to left. Although net pressures are generally given at $\theta = 0°$ and 90°, the design shall be checked at $\theta = 180°$ and 270°, unless the structure is built symmetrically in both direction.

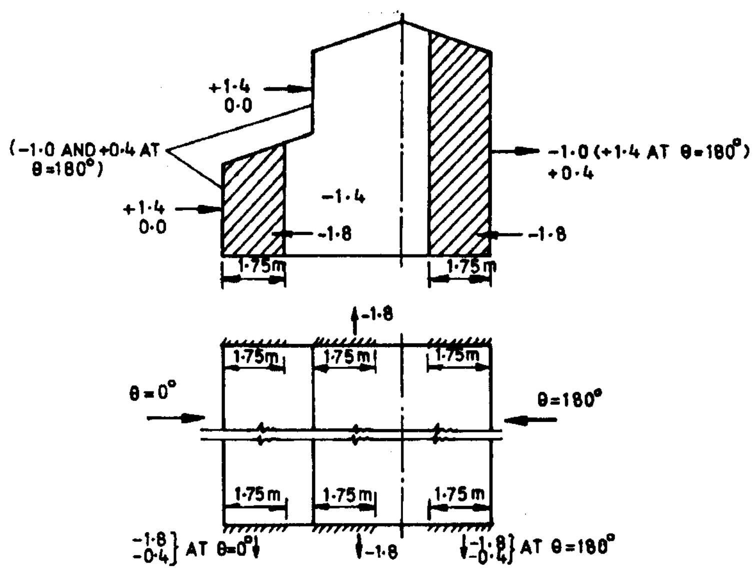
NOTE — That the 'Local' pressures at the corners have also to be assumed to be carried over to the portions *B* and *C* and *D* of the vertical walls of the main building, in addition to the local pressures at the corners of the boundary of the entire building.



(c) C_p DESIGN FOR ROOF(c,d) INCLUDING LOCAL (θ=0°, 90°)

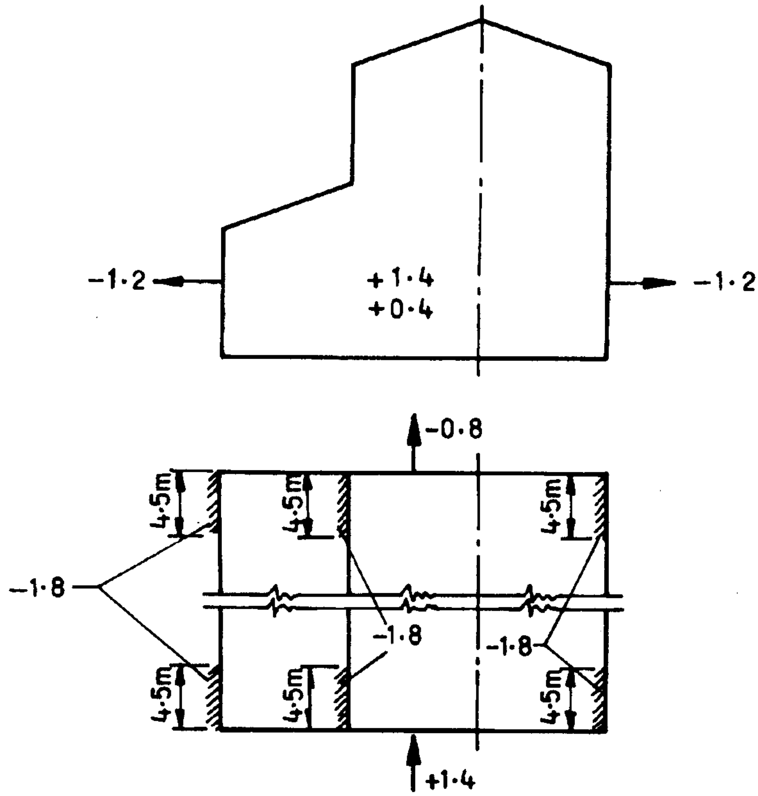
(d) C_p DESIGN FOR PORTION 'a' OF ROOF (θ=0°)

21 (a – d) Main Building Including Local Values



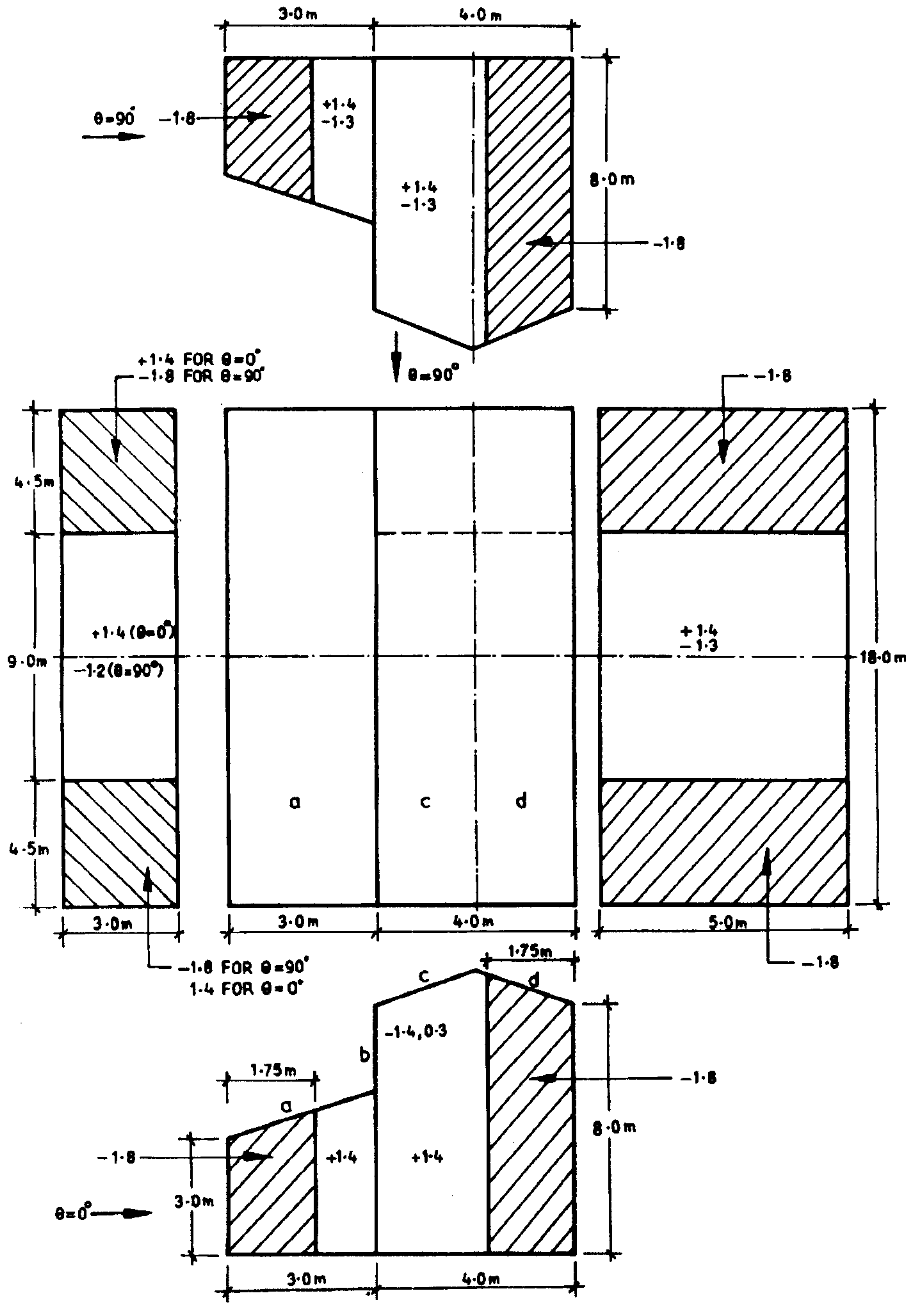
21 (e) Net Pressure Coefficients on Vertical Walls at θ = 0° and 180°

FIG. 21 PITCHED ROOF WITH A VERANDAH (*Continued*)



21 (f) Net Pressure Coefficient on Vertical Walls at θ = 90°

FIG. 21 PITCHED ROOF WITH A VERANDAH (*Continued*)



21(g) Pressure Coefficients on Vertical Walls at θ = 0° and 90°

FIG. 21 PITCHED ROOF WITH A VERANDAH

Example 7. *A rectangular clad building with roof having sky light (Fig. 22), [Table 20].*

Given:

Physical Parameters:

Height (h_1):	6 m
Total height of structure (h_2) :	6.8 m
Width (b_1):	5 m
Width of sky light portion (b_2):	2 m
Total width of structure:	12 m
Length of structure (l):	15 m

Maximum permeability of building: 3 percent of wall area

Wind Data

Wind zone	:	2 (Basic wind speed = 39 m/s)
Terrain category	:	2
Class of structure	:	A
Probable design life of structure	:	50 years
Topography	:	Almost flat

Design Wind Speed

$V_z = V_b.k_1.k_2.k_3$
$V_b = 39$ m/s
$k_1 = 1.00$
$k_2 = 1.00$
$k_3 = 1.00$
$V_z = 39 \times 1 \times 1 \times 1 = 39$ m/s

Design Wind Pressure

$p_z = 0.6(V_z)^2 = 0.6(39)^2 = 912.6$ N/m^2

Internal Pressure Coefficients : $C_{pi} = \pm 0.2$

External Pressure Coefficient on Roof **[6.2.2.10** and Table 20**]**

$b_1 = 5$ m, $b_2 = 2$ m that is, $b_1 > b_2$

For $\theta = 0°$
C_{pe} for portions *a* and *b* is –0.6 and +0.7 respectively.

For $\theta = 180°$
The values of C_{pe} as obtained for $\theta = 0°$ get interchanged.

External Pressure Coefficient on Walls (C_{pe}) **[6.2.2.1** and Table 4**]**

$h/w \leq 0.5$ and $l/w = 1.25$

Region	*A*	*B*	*C*	*D*	Local
When $\theta = 0°$	+0.7	–0.2	–0.5	–0.5	–0.8
$\theta = 90°$	–0.5	–0.5	+0.7	–0.2	

C_p *Local Values for Roof from* [*Table 5*], for $\theta = 0°$

Region	*a*	*d*	*e*	*f*	*g*	*h*
C_{pe} when $C_{pi} = +0.2$	–0.6	–0.5	–0.8	–2.0	–2.0	–2.0
C_p(net) when $C_{pi} = -0.2$	–0.8	–0.7	–1.0	–2.2	–2.2	–2.2
C_p (net)	–0.4	–0.3	–0.6	–1.8	–1.8	–1.8

C_p (net) for roof are shown in Fig. 22 (a) – (d).

C_p (net) for Wall

C_p (net) Local = – 0.8 – 0.2 = –1.0

C_p (net) for main portions of the wall

Region	*A*	*B*	*C*	*D*	Local
$\theta = 0°$ and $C_{pi} = +0.2$	+0.5	–0.4	–0.7	–0.7	–1.0
$\theta = 0°$ and $C_{pi} = -0.2$	+0.9	+0.0	–0.3	–0.3	

Region	A	B	C	D	Local
$\theta = 90°$ and $C_{pi} = +0.2$	–0.7	– 0.7	+0.5	+0.4	– 1.0
$\theta = 90°$ and $C_{pi} = -0.2$	–0.3	–0.3	+0.9	+0.0	
Design Pressure Coefficient	+0.9	–0.7	–0.7	–0.7	–1.0
	–0.7	0.0	–0.9	0.0	

Figure 22 (e-h) give the values of coefficients for the main walls.

Since l/h and l/b are less than 4, there will be no frictional drag on building.

NOTE — That although 'Local' values at the corner of roofs and walls are not specifically indicated in [Table 20, Tables 4 and 5] shall be used in the case of all buildings for local values on the roofs and vertical walls.

Design pressure coefficients for the whole building are given in Fig. 22(i).

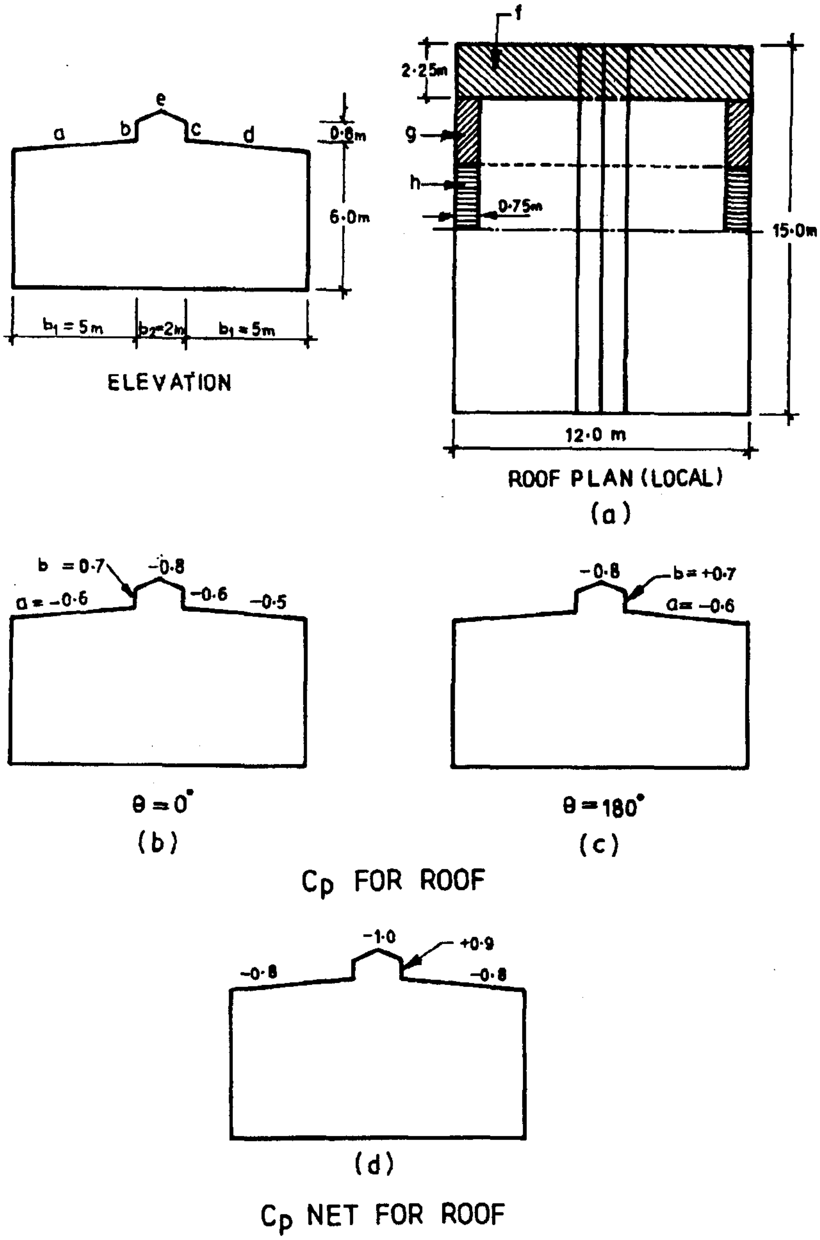
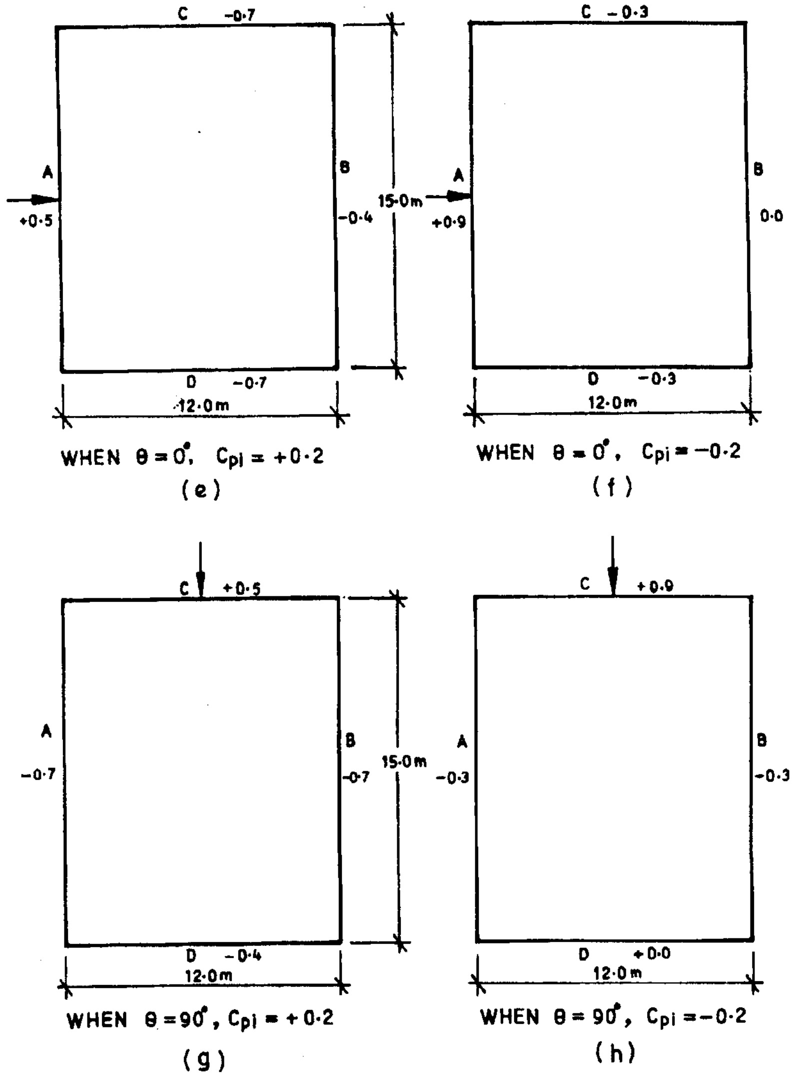
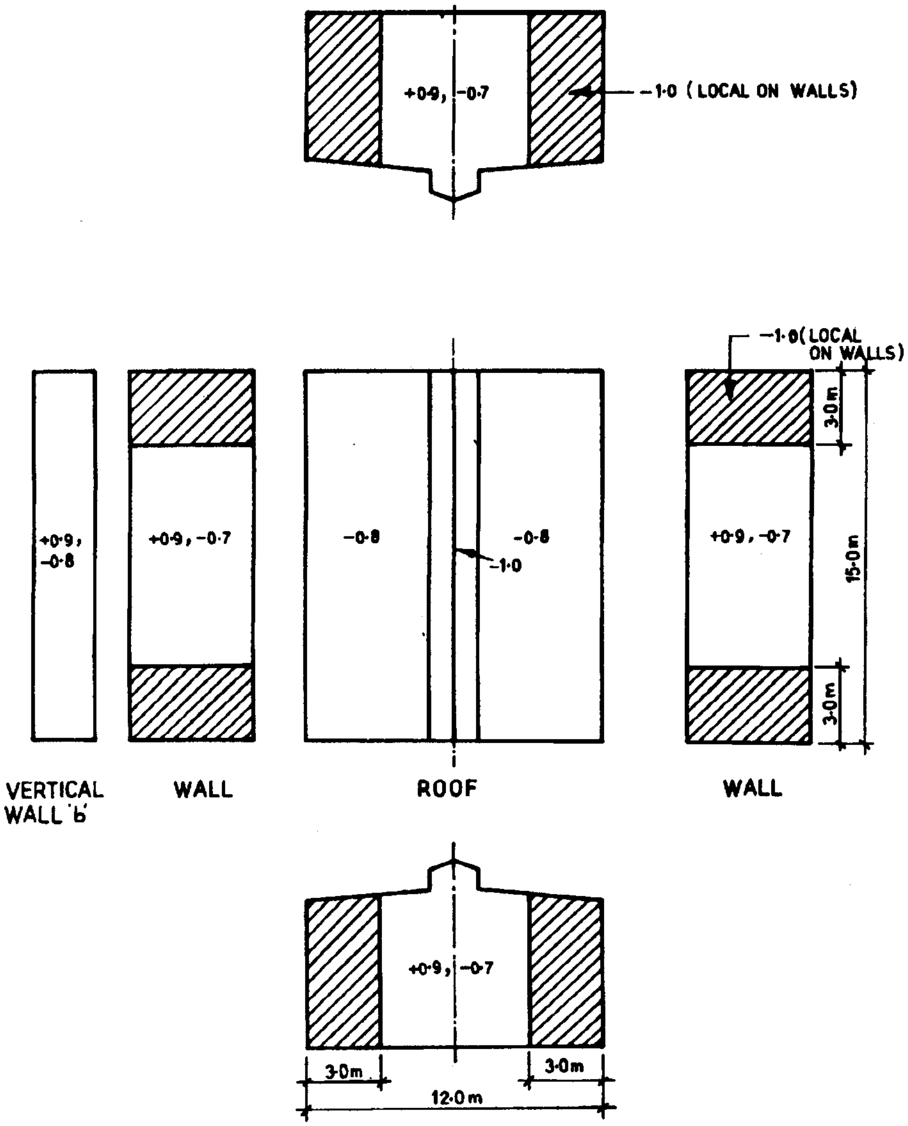


FIG. 22 A RECTANGULAR CLAD BUILDING WITH SKYLIGHT ROOF (*Continued*)



22 (e-h) C_p(net) for Main Walls

FIG. 22 A RECTANGULAR CLAD BUILDING WITH SKYLIGHT ROOF (*Continued*)



22(i) C_p(net) for the Building

FIG. 22 A RECTANGULAR CLAD BUILDING WITH SKYLIGHT ROOF

Example 8. *Consider the rectangular clad building of Example 7 with the same parameters except that $b_1 = b_2$ (Fig. 23) [Table 20].*

In this case $b_1 \leq b_2$, therefore, values of C_{pe} for portion a and b are to be determined from [Table 20].

Here $b_1 = b_2$ and $h_1/h_2 = 6.8/6 = 1.13$

C_{pe} (for portion a) = 2 × 1.13–2.9 = –0.64
and C_{pe} (for portion b) = –0.5 (for direction 1)
C_{pe} (for portion b) = –0.4 (for direction 2)

C_p *Local values for roof from* [Table 5], for $\theta = 0°$

Region	*a*	*b*	*e*	*c*	*d*
C_{pe} when C_{pi} = +0.2	–0.64	–0.5	–0.8	–0.6	–0.5
C_p(net) when C_{pi} = –0.2	–0.84	–0.7	–1.0	–0.8	–0.7
C_p(net)	–0.44	–0.3	–0.6	–0.4	–0.3

C_p *Local values for roof from [Table 5],* for θ =180°

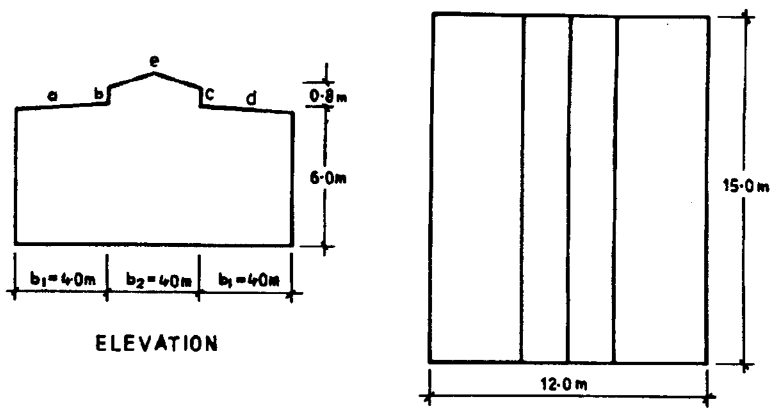
Region	*a*	*b*	*e*	*c*	*d*
C_{pe} when C_{pi} = +0.2	−0.5	−0.6	−0.8	−0.5	−0.64
C_p(net) when C_{pi} = −0.2	−0.7	−0.8	−1.0	−0.7	−0.84
C_p(net)	−0.3	−0.4	−0.6	−0.3	−0.44
C_p design values	−0.84	−0.8	−1.0	−0.8	−0.84

However, on leeward side as per [Table 20, Item (b)], the C_{pe} for pitched portion '*a*' and vertical portion '*b*' are −0.5 and −0.6, respectively Fig. 23 (b) and (c).

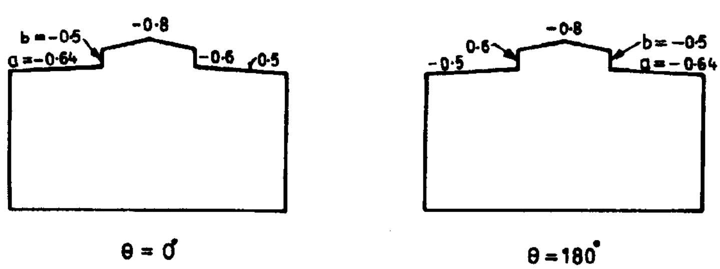
C_p(net) values considering variability of wind direction are given in Fig. 23 (d).

Other values of C_p(net) as well C_p local remain unchanged.

C_p(net) for the roof and the walls are indicated in Fig. 23(e).

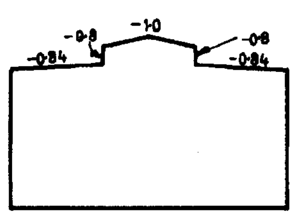


(a)



(b) (c)

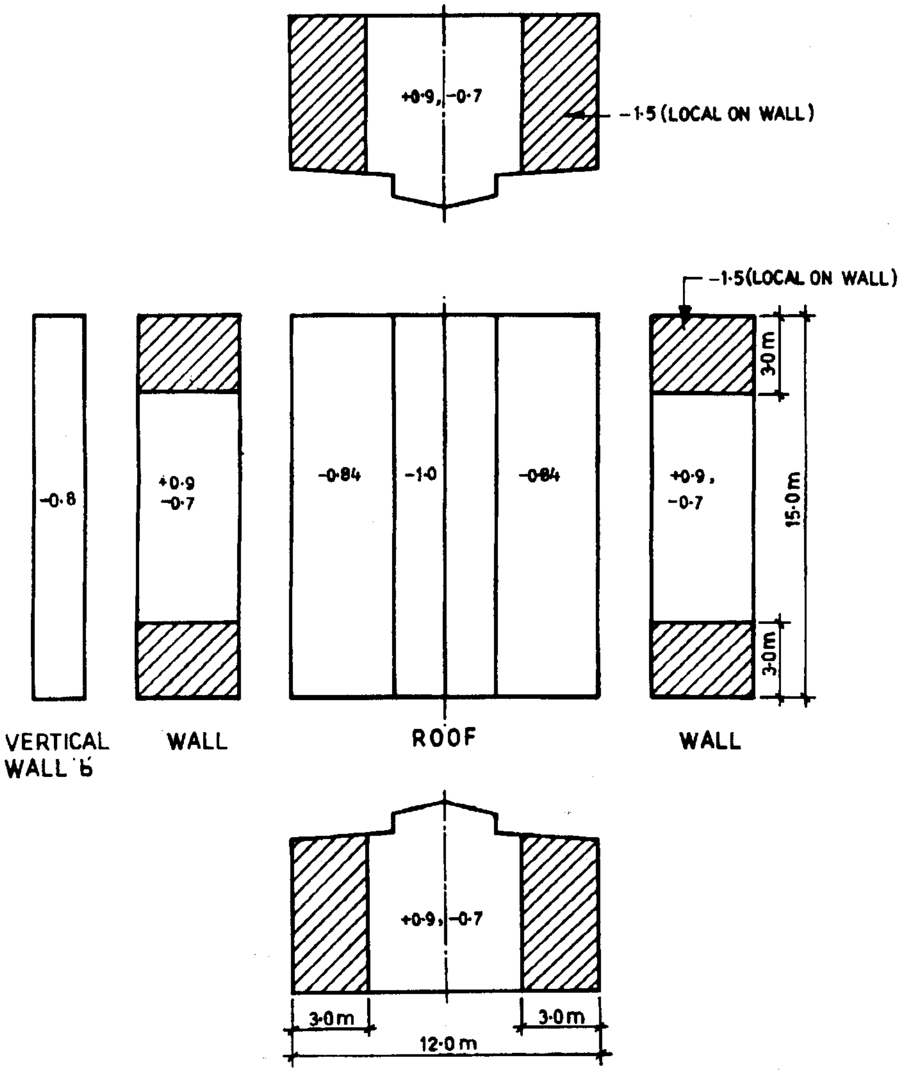
C_{pe} FOR ROOF



(d)

C_p NET FOR ROOF

FIG. 23 A RECTANGULAR CLAD BUILDING WITH SKYLIGHT ROOF SPECIAL CASE (*Continued*)



23 (e) C_p(net) for the Building

FIG. 23 A RECTANGULAR CLAD BUILDING WITH SKYLIGHT ROOF
SPECIAL CASE

Example 9. *Consider the rectangular clad building of Example 9.1.2.7 with the same parameters except that the skylight is of triangular shape (Fig. 24) [Table 20].*

The C_{pe} on roof are given for all roof portions as for pitched roof and skylight portion as –0.6 and +0.4 for windward side and –0.6 and –0.5 for leeward side.

Thus C_p(net) considering the variability of direction of wind is –0.8 and +0.6 for all portions of the roof and C_p(net) as well as C_p local remain unchanged for other portion of building.

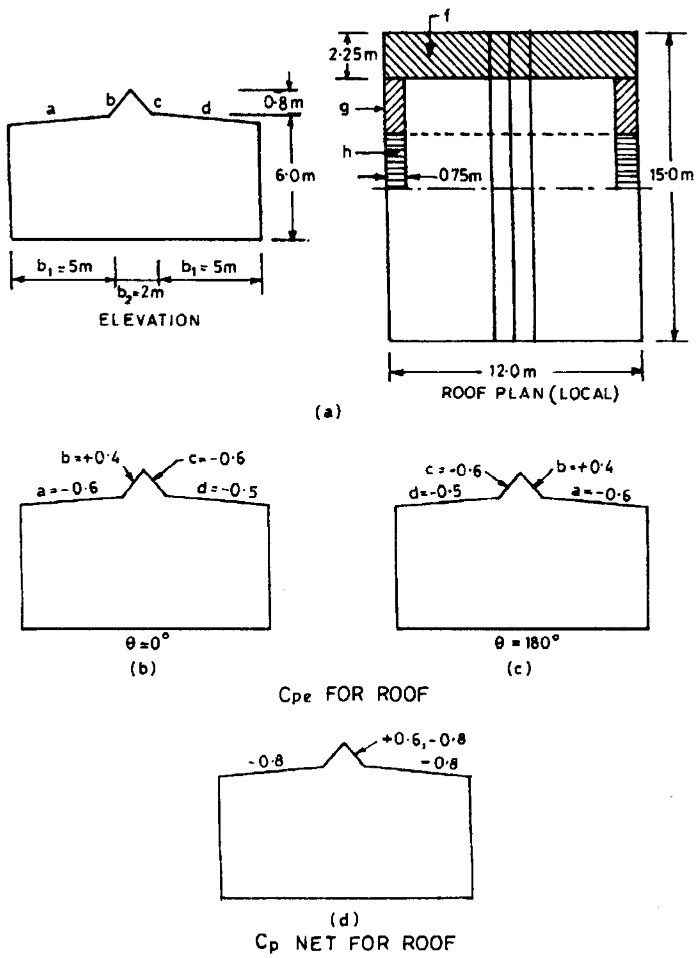


FIG. 24 RETANGULAR CLAD BUILDING WITH SKYLIGHT ROOF

Example 10. *A godown with semi-circular arch springing from ground (Fig. 25) [Table 15].*

Given:

Physical Parameters:

Height (*H*)	:	5.0 m
Width (*l*)	:	10.0 m
Length (*L*)	:	30.0 m
Roof	:	Semi-circular curved roof, springing from ground level
Permeability	:	Openings on sides not more than 5 percent of wall area
External surface	:	Corrugated
Topography	:	Almost flat

Wind Data

Wind zone	:	4 (Basic wind speed = 47 m/s)
Terrain category	:	2
Class of structure	:	B

Design Wind Speed

$V_z = V_b.k_1.k_2.k_3$
$V_b = 47$ m/s
$k_1 = 0.90$ (for mean probable design life of structure as 25 years) [Table 1]
$k_2 = 0.98$ [Table 2]
$k_3 = 1.00$ (for flat ground) **[5.3.3.1]**
$V_z = 47 \times 1.00 \times 0.98 \times 0.90 = 41.45$ m/s

Design Wind Pressure

$p_z = 0.6 \times (V_z)^2 = 0.6\,(41.45)^2 = 1\,031.0$ N/m^2

Internal Pressure Coefficient [6.2.3 and 6.2.3.1]

Since openings are not more than 5 percent of wall area $C_{pi} = \pm 0.2$

External Pressure Coefficient for Roof [6.2.2.5, Table 15 and Table 5]

For $H/l = 5/10 = 0.5$; $C = -1.2$ and $C_1 = +0.7$

The curved roof is converted to pitched roof at 3.5 m height upwards and coefficients for this portion are calculated as per [Table 5] as shown in Fig. 25(a).

For $\theta = 0°$ and $\alpha = 22.6°$

	EF	*GH*	Local 1	Local 2
C_{pe}	–0.3	–0.4	–1.0	–1.2
C_p(net)				
if $C_{pi} = +0.2$	–0.5	–0.6	–1.2	–1.4
if $C_{pi} = -0.2$	–0.1	–0.2	–0.8	–1.0
Design C_p	–0.5	–0.6	–1.2	–1.4

For $\theta = 90°$ and $\alpha = 22.6°$

	EG	*FH*
C_{pe}	–0.7	–0.6
C_p(net)		
if $C_{pi} = +0.2$	–0.9	–0.8
if $C_{pi} = -0.2$	–0.5	–0.4
Design C_p	–0.9	–0.8

C_{pe} (local) are indicated in Figs. 25 (b) and (c)

External Pressure Coefficients for End Walls

$h/w = H/l = 0.5$ and $1.5 < l/w = l/L < 4.0$

Wind angle	*C*	*D*	Local
C_{pe} for $\theta = 0°$	–0.6	–0.6	–1.0
C_p when $C_{pi} = +0.2$	–0.8	–0.8	–1.2
C_p when $C_{pi} = -0.2$	–0.4	–0.4	–0.8
C_{pe} for $\theta = 90°$	+0.7	–0.1	–1.0
C_p when $C_{pi} = +0.2$	–0.5	–0.3	–1.2
C_p when $C_{pi} = -0.2$	+0.9	+0.1	–0.8

Width of local edges = 0.25 w or l = 2.5 m

C_p (net) values for roof are shown in Fig. 25 (d).

Frictional Drag [6.3.1]

$h = H = 5$ m, $b = l = 10$ m, $d = L = 30$ m,
$p_d = 1\ 031.0$ N/m^2

Since $d/h > 4$ and $b/h < 4$, the frictional drag will be experienced by structure for $\theta = 90°$ and $270°$. Also since, the roof is curved there are no walls. The frictional drag is given by:

$F' = C_f'\ (d - 4h)\ b.\ p_d$. and $C_f' = 0.02$ for corrugated surfaces.
$F' = 0.02\ (30 - 4 \times 5) \times 10 \times 1\ 031.0 = 2\ 062.0$ N

Half of the frictional drag is to be resisted by windward 1/3rd (10 m) length and rest half by 2/3rd (20 m) length on leeward side, in addition to other loads.

Forces

$F = C_p.A.p_d$ and $p_d = 1031.0$ N/m^2

C_p	Force, N/m^2
–0.9	–927.9
–0.8	–824.8
–0.6	–618.6
–0.3	–309.3
–1.2	–1 237.2
–1.4	–1 443.4

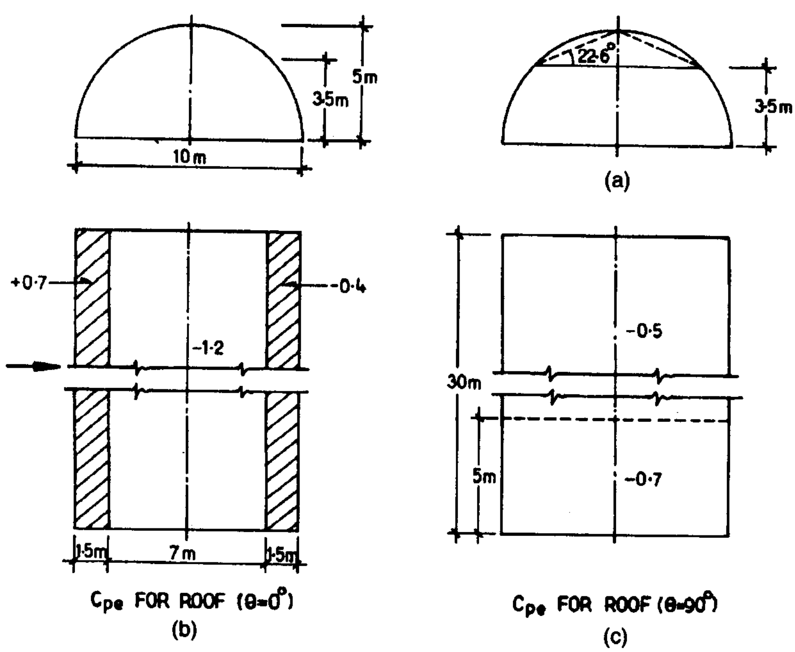


FIG. 25 SEMI-CIRCULAR SHELL ROOF (*Continued*)

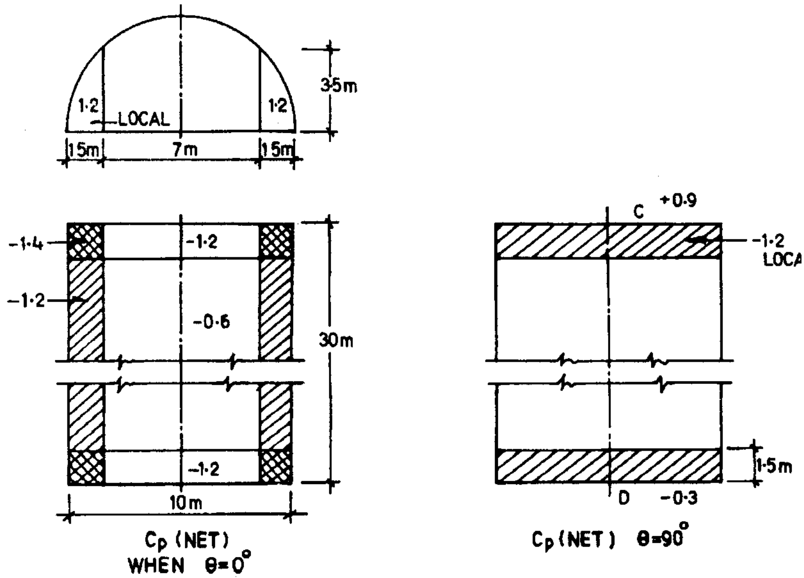


FIG. 25 SEMI-CIRCULAR SHELL ROOF

Example 11. *Rectangular clad structure having circular arch shell roof with overhang (Fig. 26) [Table 15].*

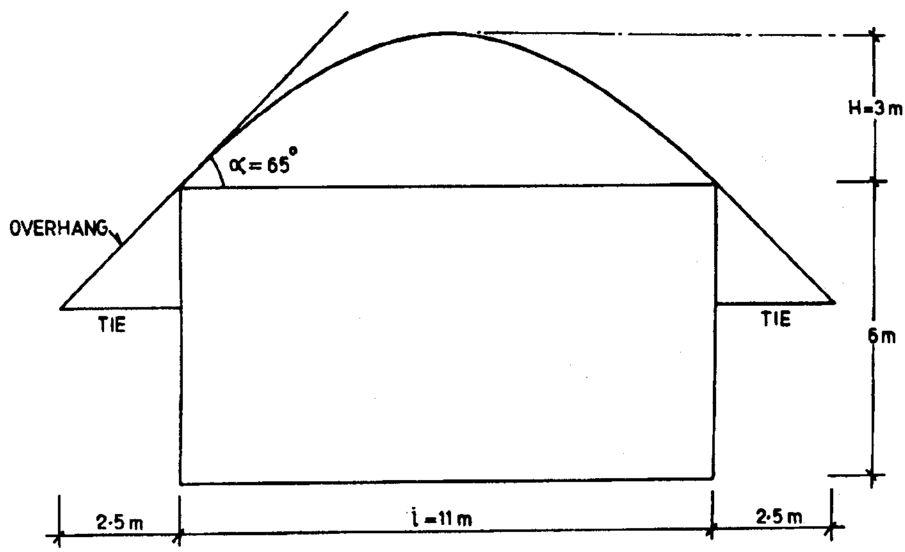


FIG. 26 BUILDING WITH CIRCULAR ARCH SHELL ROOF WITH OVERHANG (*Continued*)

Given:

Physical Parameters:

Height of arch roof (*H*)	:	3 m
Height of structure	:	6 m
Width (*l*)	:	11 m
Length (*L*)	:	30 m
Roof slope	:	65°

Roof	:	Curved circular arch
Openings on sides	:	Openings 5-20 percent of wall area
External surface of cladding	:	Roof made of ferro-cement flat units overlapping each other with dimensions 0.8 m × 2 m; masonry (unplastered) for walls.
Topography	:	Almost flat with less obstructions in surrounding but industrial area far away
Radius of shell	:	6.54 m

Wind Data

Wind zone	:	4 (Basic wind speed = 47 m/s)
Terrain category	:	2
Class of structure	:	B
Mean probable life of structure	:	25 years.

Design Wind Speed [5.3]

$V_z = V_b.k_1.k_2.k_3$
$k_1 = 0.90$
$k_2 = 0.98$
$k_3 = 1.00$
$V_z = 47 \times 0.90 \times 0.98 \times 1.00 = 41.45$ m/s

Design Wind Pressure

$p_z = 0.6\,(V_z)^2 = 0.6\,(41.45)^2 = 1\,031.0$ N/m^2

Internal Pressure Coefficient [6.2.3 and 6.2.3.2]

Since openings are about 5-20 percent of wall area, the value of $C_{pi} = \pm 0.5$. The sign will depend on the direction of flow of air relative to the side of openings.

Pressure Coefficient (on roof) [6.2.2.5, Table 15 and Table 5]

Here $H/l = 3/11 = 0.273$ and $0.7H = 2.1$ m

$\left.\begin{array}{l} C = -0.973 \\ C_2 = -0.408 \end{array}\right\}$ (by interpolation)

For the purpose of local C_{pe} and C_{pi}, the arch roof is considered of equivalent pitched roof having $h/w = 6/11 = 0.545$ and roof angle = 65°

Since the ridge is curved there will be no local coefficients. The C_{pe} values for the roof are shown in Fig. 26(a) to (c)

Pressure Coefficient for Overhangs [6.2.2.7]

C_{pe} at top of overhang portion is the same as that of the nearest top portion of the roof and in this case C_{pe} for $\theta = 270°$ and 90° is −0.7 for 5.5 m width at edges and −0.5 for middle strip of 2.45 m.

For windward side of overhang the equivalent C_{pi} (on under side of roof surface) is 1.25 while on leeward side of overhang the C_{pi} is equal to the C_{pe} of adjoining wall.

C_p(net) values for the roof are indicated in Fig. 26(d).

External Pressure Coefficient for End Walls [6.2.2.1 and Table 4]

$h/w = 6/11 = 0.54$ and $l/w = 30/11 = 2.73$

Wall	*A*	*B*	*C*	*D*	Local
for θ = 0°	+0.7	−0.3	−0.7	−0.7	−1.1
for θ = 90°	−0.5	−0.5	+0.7	−0.1	

Corner width is $0.25w = 2.75$ m
C_p (net) values for the roof are indicated in Fig. 26(e).

Frictional Drag [6.3.1]

$h = H = 3$ m, $B = l = 11$ m, $p_d = 1\,031.0$ N/m^2
$d = L = 30$ m

Since $d/h = 10 > 4$ and $b/h = 3.67 < 4$, frictional drag will be experienced by structure for $\theta = 90°$ and $270°$.

$F' = C_f'(d-4h)b.p_d + C_{f'}(d-4h)2h.p_d$

$C_f' =$ (average of smooth and corrugations surfaces) $= 0.5(0.01 + 0.02) = 0.015$

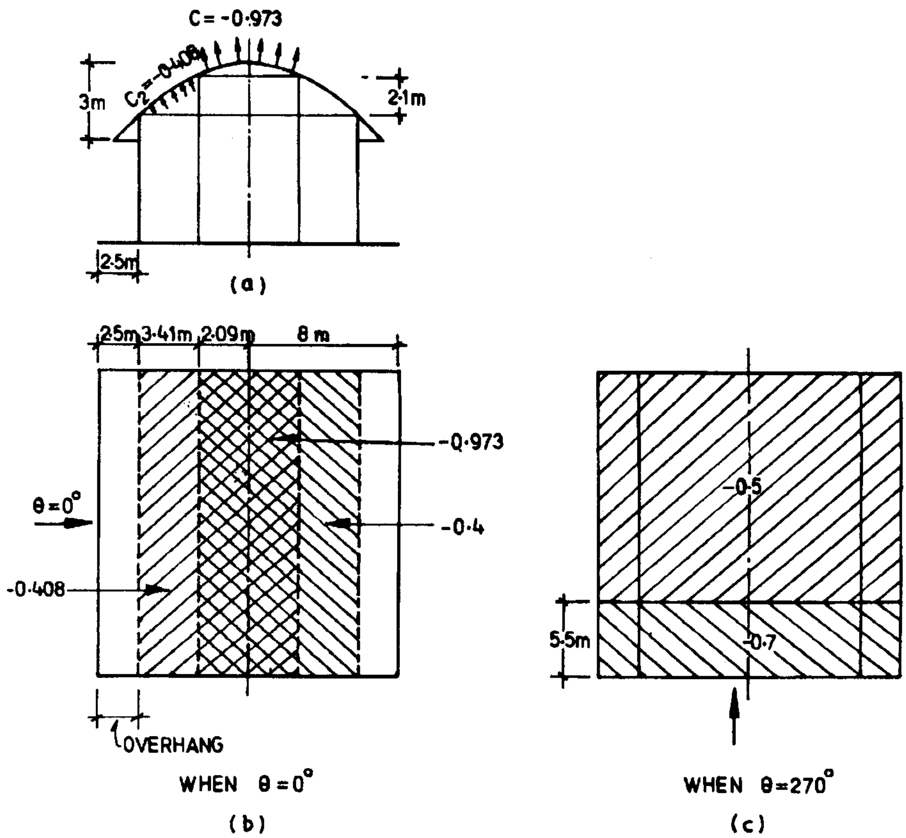
$F' = 0.015\,(30 - 4\times3)\times11\times1\,031 + 0.015\,(30-4\times3)\times2\times3\times1\,031$
$= 3\,062.1$ N $+ 1\,670.2$ N
(roof) (wall)

Half of frictional drag is to be borne by windward 1/3rd length that is, 10 m length and rest half of frictional drag by the 2/3rd length on leeward that is, 20 m, in addition to other loads.

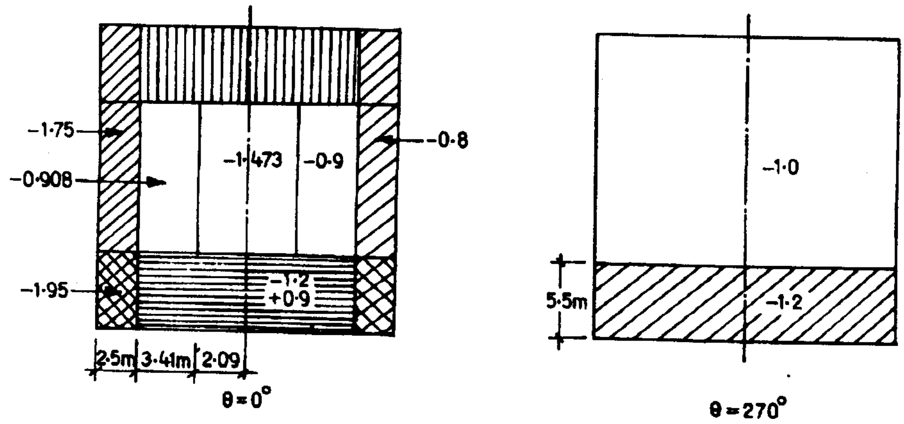
Forces

$p_d = 1\,031.0$ N/m^2

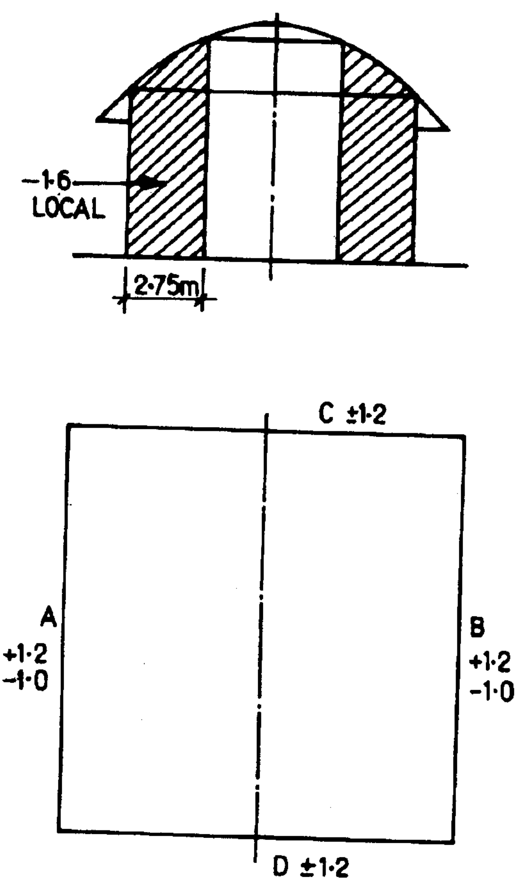
C_p	*Force*, N/m^2
−1.75	−1 804.25
−0.8	−924.8
0.9	927.9
−1.2	−1 237.2
−1.0	−1 031.0
−1.473	−1 518.66
−9.08	−936.15
−1.6	−1 649.6



26 (a) to (c) C_{pe} on Roof
FIG. 26 BUILDING WITH CIRCULAR ARCH SHELL ROOF WITH OVERHANG (*Continued*)



26(d) C_p(net) for Roof



26(e) C_p(net) on Wall

FIG. 26 BUILDING WITH CIRCULAR ARCH SHELL ROOF WITH OVERHANG

Example 12. *Estimation of design wind pressure on glass glazings.*

The determination of equivalent static wind pressure on large glazing has been the subject of considerable study ever since the failure of such glazing on the Kennedy Memorial Building in Boston, USA. It is recognised that such large glazing experience three types of wind loads, all additive, namely:

(i) a static wind load due to a wind speed averaged over a specified time interval,

(ii) a random dynamic wind load due to turbulence in the wind, and

(iii) a resonant wind load due to resonant excitation of the vibration of the panel by the energy of the turbulent fluctuations at the natural frequency of the panel.

Generally speaking, the resonant component can be neglected if the natural frequency of the panels is more than about 250 Hz. For design purposes, the information from experimental and theoretical studies have been codified into equivalent steady design wind pressure, in which all of the three wind load components stated above are already added appropriately.

Although the design wind pressure can be worked out for each level of a building, it is recommended that the pressure corresponding to the top level as well as those at the corners be adopted for all lower levels. There are two main reasons for this recommendation:

First — The assumed need to standardize the thickness of the glass at all places.

Second — The effect of failure of a panel at any height. Such failure may increase the internal pressure coefficient to positive like 0.8 due to free passage of air into the building as against the maximum value of 0.2 for building with no windows. Such increase of internal pressure may bring about the failure of other panels to which the high internal pressure may be communicated by the passages inside the building.

To illustrate, take an example of a square cross-section sealed building just outside New Delhi of height 50.0 m and sides of 15.0 m each, in terrain category 2 and with no windows/openings.

Design Wind Speed [5.3]

$V_z = V_b.k_1.k_2.k_3$

$k_1 = 1.07$, since failure of glass panel is hazardous [Note 2 of **6.2**]

$k_2 = 1.17$, though the size of building is 50 m but the panel size will be less than 20 m, hence the panel is considered as Class A and corresponding value has been picked up from [Table 2]

$k_3 = 1.00$

$V_z = 47 \times 1.07 \times 1.17 \times 1.00 = 58.84$ m/s

Design Wind Pressure

$p_z = 0.6\,(V_z)^2 = 0.6\,(58.84)^2 = 2\,077.24$ N/m^2

Since the building is without windows, as per [**6.2.3.1**], the internal pressure coefficient = $\pm$ 0.2.

Also, $(3/2) < (h/w) < 6$ and $1 < (l/w) < (3/2)$

Considering that glazing is provided at a distance of less than 0.25 times the width of wall from the corner, as per [Table 4] the external local pressure coefficient of −1.20 will govern the design even with minimum internal pressure of + 0.2; resulting in a net pressure coefficient of −1.4. This gives a design pressure for glazing

$p_d = 2\,077.24 \times 1.4 = 2\,908.13$ N/m^2

9.1.3 *Preliminary Explanation with Regard to Structures with Free Roofs [Covered by Tables 8 to 14]*

[Tables 8 to 14] refer to roofs with open bays on all sides. [Table 8] applies to roofs with (h/w) lying between 0.25 and 1.0 and (L/W) lying between 1 and 3. Modifications to the roof pressures due to obstructions are to be applied. When the length of the canopy is large, then [Tables 9 to 14] have to be used, if $(b/d) = 5.0$ or near about. In such cases [Tables 9 and 10] imply that the average pressures will have to be worked out for portions of the roof for wind angles of 0°, 45°, 90° and 0°, 45°, 90°, 135° and 180° when there is obstruction on one side. For example [Tables 9 and 10] give separate pressure on one quarter leeward area of roof, designated as J, at $\theta = 45°$. [Tables 11 and 12] are to be used if the canopy is inclined at 30°. In case (b/d) is substantially more than 5.0 and roof angle is other than those given in the Code, it is better to commission wind tunnel model studies.

Frictional force shall be considered when the roof area is larger in [Tables 9 to 14], compared to the area in [Table 8]. The friction force is to be distributed such that half of the friction force acts on the windward 1/3rd of the surface and the other half on the leeward 2/3rd surface. The forces may be considered to act at the mid plane of these portions.

The definition of the height '*h*' given in [Table 8] for positive and negative roof angles must be carefully noted. It is not the height to the lowest point of the canopy for both positive and negative roof angles.

The definition of the width is that of the main canopy only and will not include the facia if they are inclined and extend beyond the edge of the main canopy.

Case A — No obstructions/stored material below the canopy

[Table 8, read with **6.2.2.4**], gives the overall roof pressures of the canopy. The columns, trusses and foundations are to be checked for these loads. It is to be noted that the Code gives only the force coefficients in the direction of wind as 1.3 for the facia implying that only a facia of angle 90° (that is, a vertical facia) is covered in the Code. If the facia has an angle other than 90°, the loads can be significantly higher and values based on wind tunnel model studies or conservative values based on the loads of roof overhangs of buildings [**6.2.2.2** and Table 5] may be used. If a number of such canopies are being built, it is more economical to get model studies made in wind tunnel to know accurate loadings.

Case B — Obstructions/stored material below the canopy

Here, the solidity ratio as defined in [**6.2.2.4**] has to be found. This is obviously the ratio of the height of the obstruction to the total height of the canopy. As indicated in [**6.2.2.4** and Table 8], the positive pressure is not affected by the obstruction but the negative pressure upstream of the obstruction gets modified. The horizontal force coefficient of 1.3 on the vertical canopy is introduced only on the windward canopy.

The pressures at the corners *A, B, C, D* are shown at the higher value of −3.0 in line with some of the new findings and in the absence of a specific clarification on this point in the Code. It may be noted that as per the Code, the design cannot be checked for opposite pressures on the two arms on the canopies also.

9.1.4 *Solved Examples of Free Roofs*

Example 13. *A rectangular clad building with free pitched roof* [**6.2.2.4** *and Table 9].*

Given:

Physical Parameters:

Height (h)	:	5 m
Length (b)	:	50 m
Span (d)	:	10 m
Roof angle (α)	:	30°

Wind Data

Wind zone	:	4 (Basic wind speed = 47 m/s)
Terrain category	:	1
Class of structure	:	B
Design life of structure	:	25 years.

Design Wind Speed

$V_z = V_b.k_1.k_2.k_3 = 47 \times 0.90 \times 1.03 \times 1.0 = 43.57$ m/s

Design Wind Pressure

$p_z = 0.6\,(V_z)^2 = 0.6 \times (43.57)^2 = 1\,139.00$ N/m^2

CASE I : When there is no stored material underside [*see* **Fig. 27(a)**]

Wind Load Calculation

Pressure Coefficient on Roof and End Surfaces [Table 9]

For Roof

C_p(net) = C_p(top) – C_p(bottom)

For $\theta = 0°$
C_p (for region *D*) = 0.6 – (–1.0) = 1.6
C_p (for region *E*) = –0.5 – (–0.9) = 0.4

For $\theta = 45°$
C_p (for region *D*) = 0.1 – (–0.3) = 0.4
C_p (for region *E*) = –0.6 – (–0.3) = –0.3
C_p (for region *J*) = –1.0 – (–0.2) = –0.8

For $\theta = 90°$
C_p (for region *D*) = –0.3 – (–0.4) = 0.1
C_p (for region *E*) = –0.3 – (–0.4) = 0.1

For End Surfaces

C_p (net) = C_p(front) – C_p(rear)
C_p (windward, end surface) = 0.8 – (0.4) = 0.4
C_p (leeward, end surface) = –0.3 – (0.3) = –0.6

For $\theta = 90°$, there will be force due to frictional drag (Tangentially)
= $0.05\, p_d.b.d = 0.05 \times 1\,139.00 \times 50 \times 10 = 28\,475$ N

Force on portion *D* = 1.6 × 1 139.00 × 1 = 1 822.40 N/m^2

Force on portion *J* = 0.8 × 1 139.00 = 911.20 N/m^2

Force on portion *E* = –0.3 × 1 × 1 139.00 = –341.70 N/m^2

The design pressure coefficients are indicated in Fig. 27(b)

CASE II: If effects of train or stored materials are to be considered [*see* Fig. 27(c)]

Wind Load Calculations

Pressure coefficients on roof and end surfaces are to be taken from [Table 10]

For $\theta = 0°$
C_p (for region *D*) = 0.1 – 0.8 = –0.7 for full length
C_p (for region *E*) = –0.7 – 0.9 = –1.6 for full length

For $\theta = 45°$
C_p (for region *D*) = –0.1 – 0.5 = –0.6
C_p (for region *E*) = –0.8 – 0.5 = –1.3
C_p (for region *J*) = –1.0 – 0.5 = –1.5

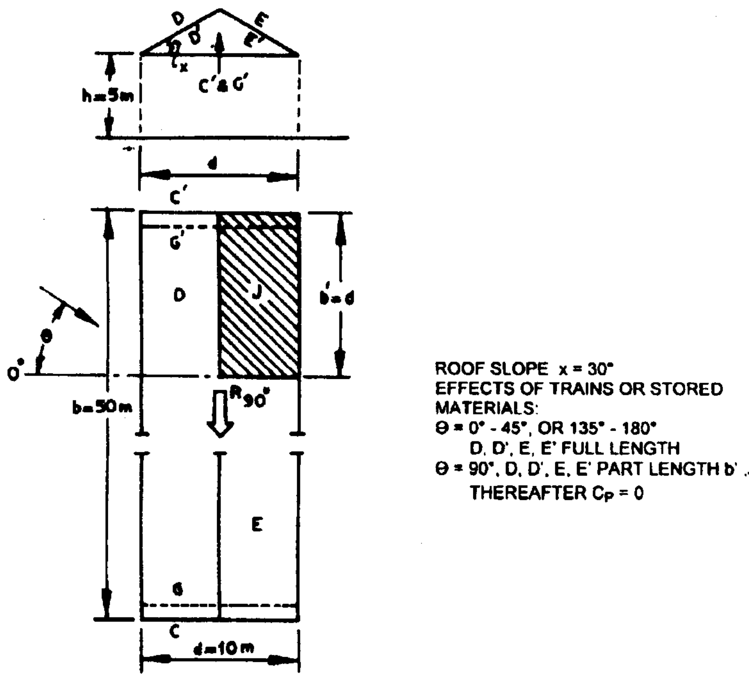
For $\theta = 90°$
C_p (for region *D*) = –0.4 + 0.5 = 0.1
C_p (for region *E*) = –0.4 + 0.5 = 0.1
C_p (windward, end surface) = 0.8 + 0.4 = 1.2
C_p (leeward, end surface) = –0.3 – 0.3 = –0.6

For $\theta = 180°$
C_p (for region *D*) = –0.3 + 0.6 = 0.3
C_p (for region *E*) = 0.4 + 0.6 = 1.0

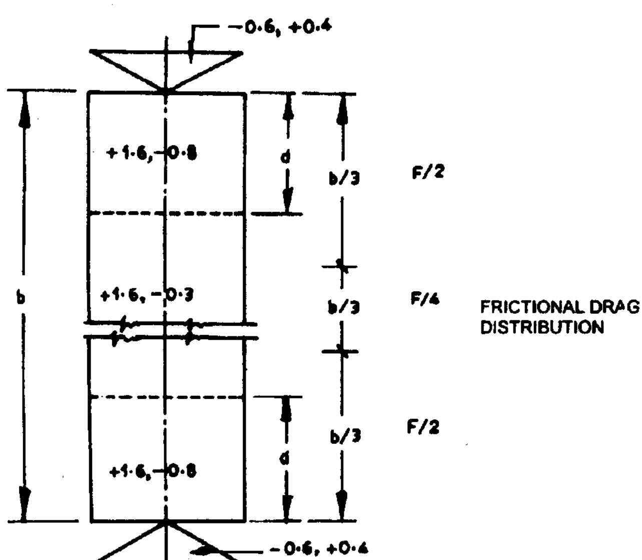
Design pressure coefficients are indicated in Fig. 27 (d).

For $\theta = 90°$, Frictional tangential force = $0.05\, p_d.b.d$

Forces on Different Portions
F_1 = 1.2 × 1 139.00 = 1 366.80 N/m^2
F_2 = –2.0 × 1 139.00 = –2 278.00 N/m^2
F_3 = –1.6 × 1 139.00 = –1 822.40 N/m^2
F_4 = –0.7 × 1 139.00 = – 797.30 N/m^2



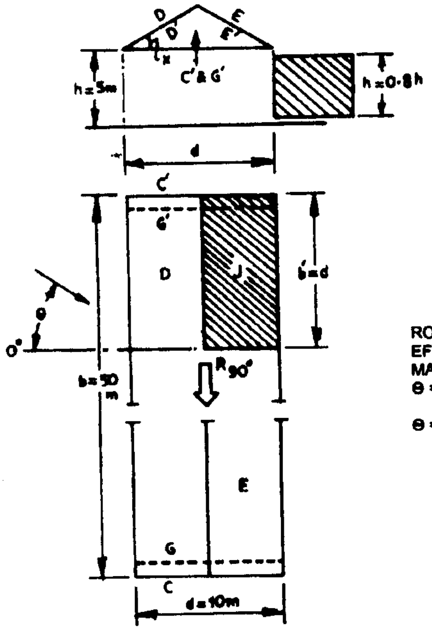
27(a) Plan



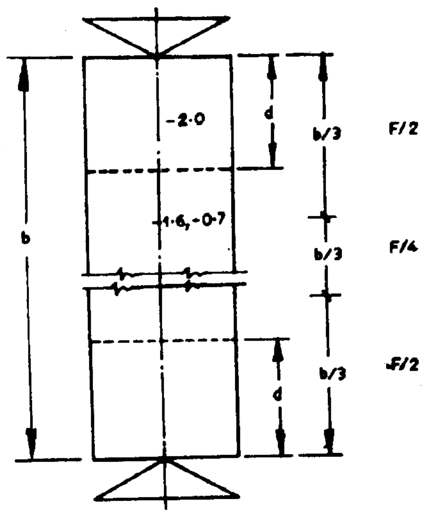
DESIGN PRESSURE COEFFICIENT

27(b)

FIG. 27 EXAMPLE OF PITCHED FREE ROOFS, $\alpha = 30°$ WITH OR WITHOUT EFFECTS OF TRAIN OR STORED MATERIALS (*Continued*)



27 (c)



27 (d) Design Pressure Coefficient

FIG. 27 EXAMPLE OF PITCHED FREE ROOFS, $\alpha = 30°$ WITH OR WITHOUT EFFECTS OF TRAIN OR STORED MATERIALS

Example 14. *A free double sloped temporary roof (Fig. 28) with and without an obstruction underside, and with and without a facia of 0.75 m [***6.2.2.4** *and Table 8].*

Given:

Physical Parameters:

Height of roof (h)	:	3.41 m
Width of roof (w)	:	5.0 m
Length of structure (l)	:	10.0 m
Roof angle (α)	:	20°

The structure is supported on columns at a distance of 1.0 m from the centre. There are 1.0 m high cotton bales along its length, with width of 1.0 m.

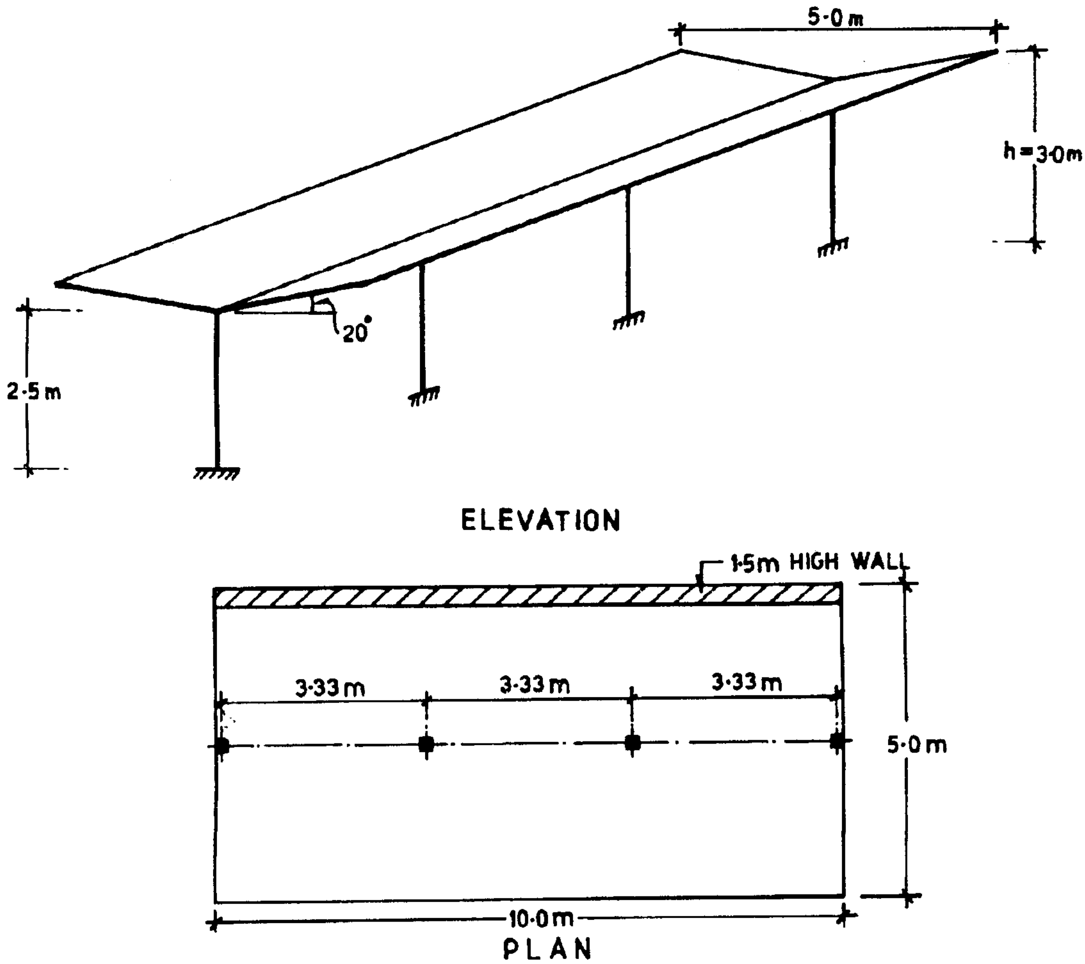


FIG. 28 A FREE DOUBLE-SLOPED ROOF (*Continued*)

Wind Data

Wind zone	:	2 (Basic wind speed = 39 m/s)
Terrain category	:	2
Class of structure	:	A
Life of structure	:	5 years [Table 1]
Topography	:	Flat

Design Wind Speed

$V_z = V_b.k_1.k_2.k_3$

$V_b = 39$ m/s

$k_1 = 0.76$

$k_2 = 1.0$

$k_3 = 1.0$

$V_z = 39 \times 0.76 \times 1 \times 1 = 29.64$ m/s

Design Wind Pressure

$p_z = 0.6\,(V_z)^2 = 0.6\,(29.64)^2 = 878.53$ N/m^2

Pressure Coefficients on Roof

C_p for $\theta = 0°$ [**6.2.2.4** and Table 8]

Since the 1.0 m high cotton bales are normal to wind direction and is on upwind, the solidity ratio(ϕ) = 1.0/2.5 = 0.4

Maximum +ve (thrust)

C_p (overall) = + 0.7
C_p (local) = +0.8, +1.6, +0.6, +1.7 [As shown in Fig. 28 (a)]

Maximum −ve (suction)
C_p (overall) = −0.78
C_p (local) = −1.08, −1.54, −1.78, −0.96 [As shown in Fig. 28 (a)]

C_p for $\theta = 90°$

Since there is no obstruction for air flow $\phi = 0$. All the thrust values of C_p are same as in the case for $\theta = 0°$

Suction values are

C_p (overall) = −0.7
C_p (local) = −0.9, −1.3, −1.6, −0.6

C_p for $\theta = 180°$

Since the obstruction is on downwind side not on upwind side $\phi = 0$. The C_p values are same as in the case of $\theta = 90°$.

The design pressure coefficients C_p are shown in Fig. 28 (a).

Force Calculations

The force on ridge/trough induced by individual slopes is as follows:

$$F = C_p(\text{overall}) \times A \times p_d \text{ (where } A = \text{Area of one side roof)}$$
$$= -0.82 \times 2.5 \times 10 \times 878.53 = -18\,009.86 \text{ N}$$

$F = +0.7\,(2.5 \times 10) \times 878.53 = 15\,374.28$ N acting on centre of roof,
that is (2.5/2 = 1.25 m) away from the trough, acting normal to the surface

Total upward force on support structure
= 2 × (−18 009.86) = − 36 019.72 N = − 36.02 kN (upward)

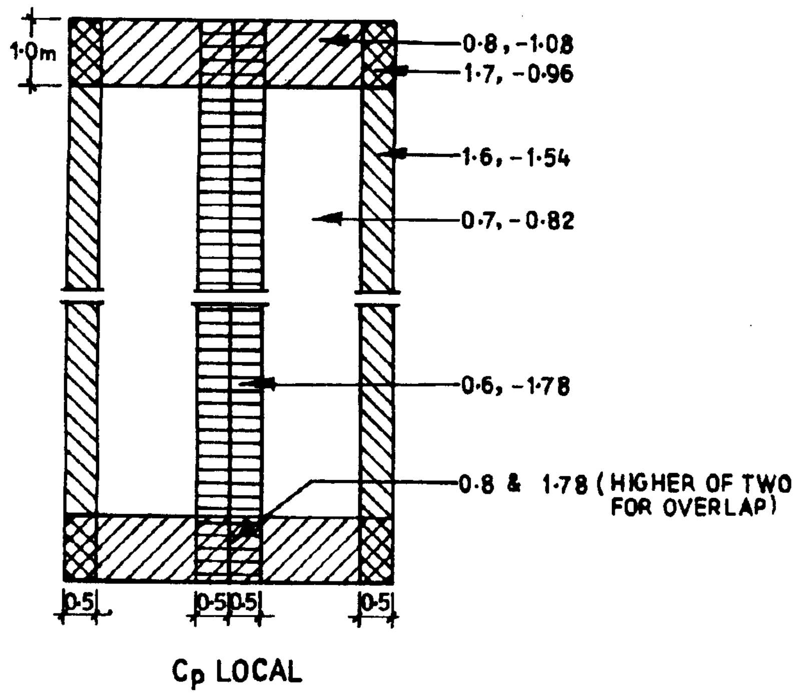
Total downward force

$F = 2 \times (15\,374.28) = 30\,748.56$ N = 30.75 N

Maximum moment induced on support structure is when one side roof experiences thrust and other side roof experiences suction.

Hence the moment at top of support due to vertical component of wind
= (−18 009.86 × 1.25 − 15 374.28 × 1.25) cos 20°
= −41 729.1 cos 20°= 39 212.53N-m

Moment at the base of the support due to horizontal component of wind
(18 009.86 + 15 374.28) × 2.5 sin 20°
= 28 545.12N-m



28(a) Forces on Support Structure

FIG. 28 A FREE DOUBLE-SLOPED ROOF

Example 15. *A rectangular unclad structure with V- troughed free roof* ($\alpha = 10°$) *and underside obstruction (Fig. 29) [***6.2.2.4**, *Tables 13 and 14].*

Given:
Physical Parameters:

Height (h)	:	4 m
Width (d)	:	8 m
Length (b)	:	40 m
Roof angle (α)	:	10°

Since the inclination of the canopy is 10° [Tables 13 and 14] have to be used.

Wind Data

Wind zone	:	2 (Basic wind speed 39 m/s)
Terrain category	:	2 [**5.3.2.1**]
Class of structure	:	B [**5.3.2.2**]
Life of structure	:	50 years.
Topography	:	Flat

Design Wind Speed [5.3]

$V_z = V_b.k_1.k_2.k_3$
$V_b = 39$ m/s
$k_1 = 1.0$
$k_2 = 0.98$
$k_3 = 1.0$ (for flat ground)
$V_z = 39 \times 1 \times 0.98 \times 1 = 38.22$ m/s

Design Wind Pressure

$p_z = 0.6\,(V_z)^2 = 0.6\,(38.22)^2 = 876.46$ N/m^2

Without underside obstruction. [Fig. 29 (a)]

Pressure Coefficients [Table 13]

On roof: $C_p(\text{net}) = C_p(\text{top}) - C_p(\text{bottom})$

Positive forces on a surface are always to be taken as acting towards the surface.

For $\theta = 0°$

C_p(net) (for region D)
$= C_p(D) - C_p(D')$
$= +0.3 - (-0.7) = +1.0$ on full length of roof acting downwards

C_p(net) (for region E)
$= 0.2 - (-0.9) = +1.1$ on full length of roof acting downwards

For $\theta = 45°$ (on full length of roof)

C_p(net) (for region D) $= 0.0 - (-0.2) = +0.2$

C_p(net) (for region E) $= 0.1 - (-0.3) = +0.4$

For $\theta = 90°$

C_p(net) (for region D) $= -0.1 - (0.1) = -0.2$

C_p(net) (for region E) $= -0.1 - (0.1) = -0.2$

Acting on windward width ($b' = d$) and is zero on rest of portion of roof. [**6.2.2.4** and Table 13]

C_p(net) (for region F) $= 0.4 - (-1.5) = +1.9$ [Fig. 29 (a)]

Effect of obstructions below the roof [Table 14]

Pressure Coefficient on Roof

$C_p(\text{net}) = C_p(\text{top}) - C_p(\text{bottom})$

For $\theta = 0°$ on full length

C_p (for region D) $= -0.7 - (0.8) = -1.5$

C_p (for region E) $= -0.6 - (0.6) = -1.2$

C_p (for region F) $= -1.1 - (0.9) = -2.0$

For $\theta = 45°$ on full length

C_p (for region D) $= -0.4 - (0.3) = -0.7$

C_p (for region E) $= -0.2 - (0.2) = -0.4$

For $\theta = 90°$ for windward length of $b' = 8.0$ m

C_p (for region D) $= -0.1 - (0.1) = -0.2$

C_p (for region E) $= -0.1 - (0.1) = -0.2$

For $\theta = 180°$ on full length

C_p (for region D) $= -0.4 - (-0.2) = -0.2$

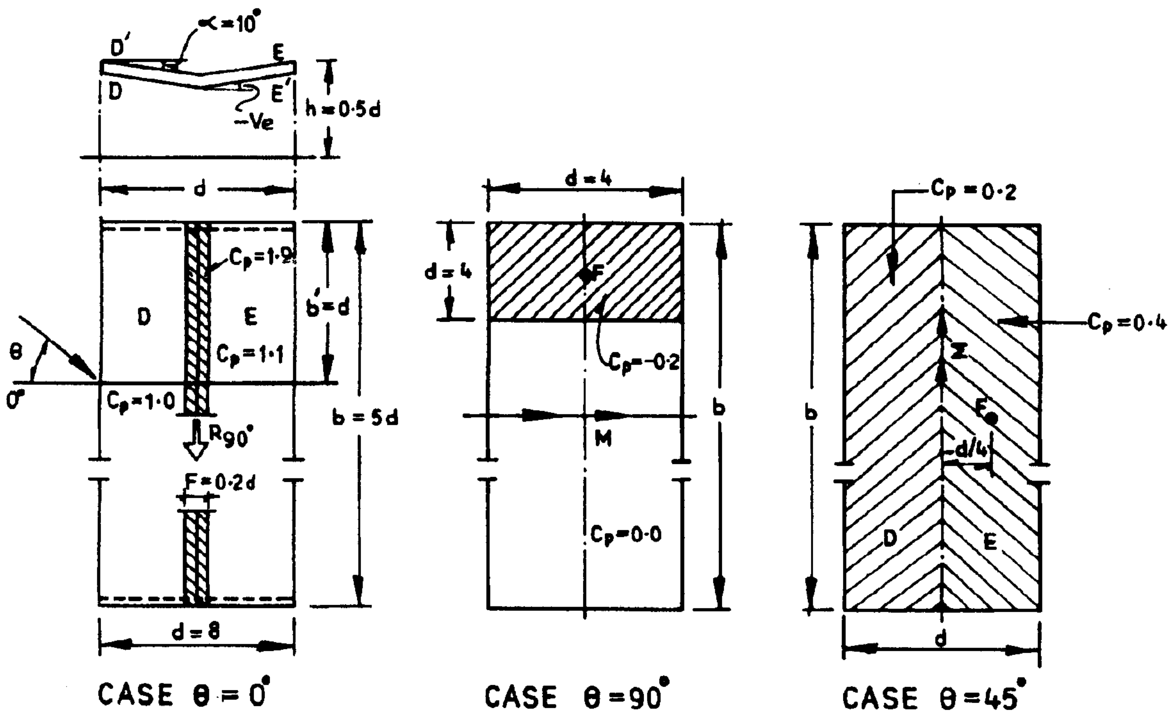
C_p (for region E) $= -0.6 - (-0.3) = -0.3$

Friction force at $\theta = 0°$ and $180°$

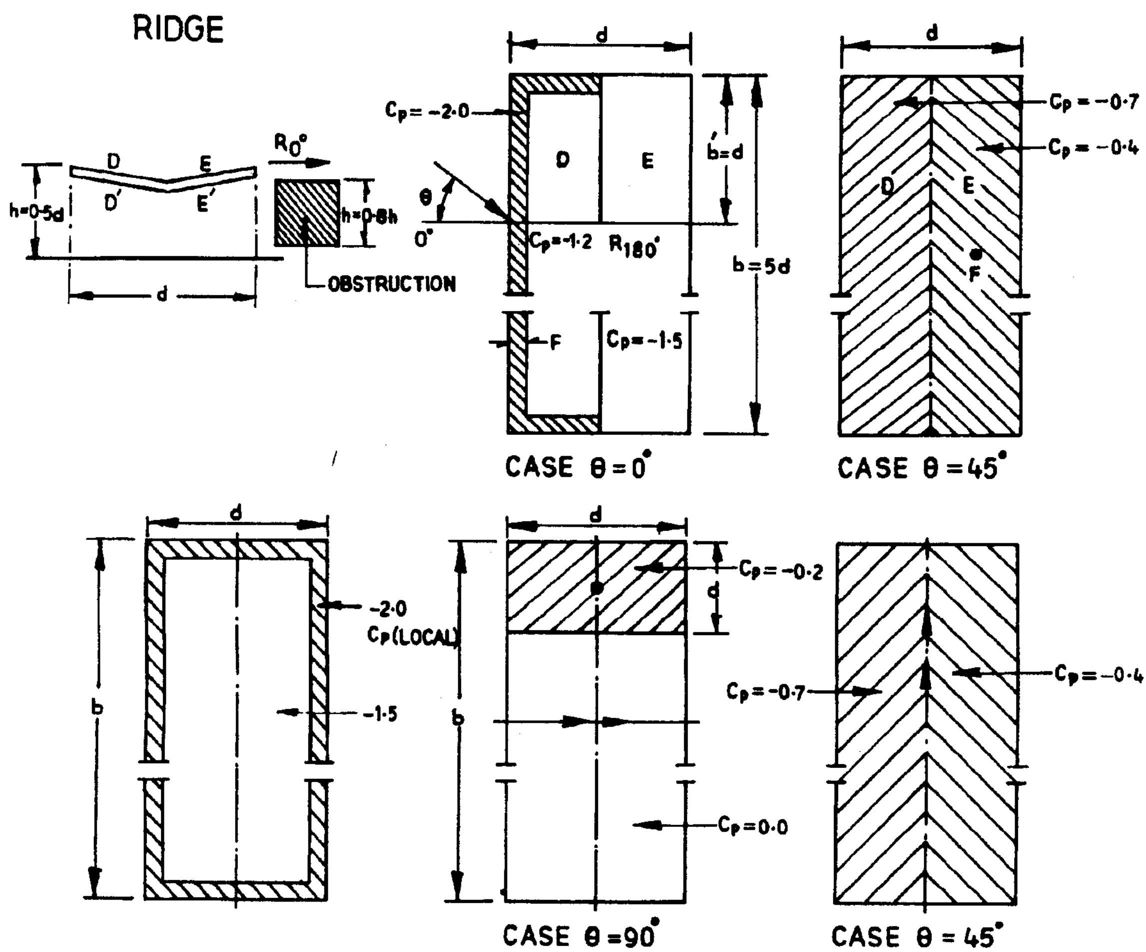
$= 0.1\, p_d.b.d = 0.1 \times 876.46 \times 40 \times 8 = 28.04$ kN

C_p (design) for cladding

Since obstruction may be on either side, hence, maximum C_p is -1.5 over the roof and C_p local on edges is -2.0. Recent measurements suggest that the value should be enhanced to -3.0 at the corners.



29 (a) C_p(net) Design for Cladding with Local Coefficients on Ridge



29 (b) C_p(net) Design for Cladding with Local Coefficients on Ridge with Underside Obstruction

FIG. 29 C_p(net) DESIGN FOR CLADDING WITH OR WITHOUT UNDERSIDE OBSTRUCTION

9.2 Force Coefficient Method [6.3]

9.2.1 In the Force Coefficient method the total wind load F on a particular building or structure as whole is given by

$F = C_f . A_e . p_d$

where

C_f = force coefficient,

A_e = effective frontal area of building or structure, and

p_d = design wind pressure.

The Code specifies different values of force coefficients for different faces of a building. Since the design pressure varies with height, the surface area of building/structure be sub-divided into strips and appropriate pressures are taken over the strips and calculated individually on the area A_e.

9.2.2 *Force Coefficients for Clad Buildings* **[6.3.2]**

[Figure 4] gives the values of the Force Coefficients for rectangular clad buildings of uniform section with flat roofs depending upon the *h/b* and *a/b* values while [Table 23] gives the value for other clad buildings of uniform section.

9.2.2.1 *Force coefficients for circular sections* **[6.3.2.2]**

Though the Code specifies that Force Coefficients for circular cross-section buildings be taken from [Table 23], yet for a precise estimate for infinite length [Fig. 5] may used which accounts for the surface roughness 'ε' and in case the length of the structure is finite the Force Coefficient is modified by a multiplication factor of κ.

9.2.2.2 *Force coefficients for walls and hoardings*

Earlier it was thought that the maximum wind pressure occurs when the wind direction is at right angles to the surface. The wind tunnel tests abroad, however, indicated that the maximum net pressure occurs when the wind is blowing at approximately 45° to the wall /hoarding. Further, as the width to height ratio of wall increases, the net normal pressure across the wall, for wind direction normal to the wall, decreases due to the attenuation of the pressure fluctuations on the leeward side. Near a free end or corner, there is a large increase in load for oblique wind directions blowing on the free end specially at higher width/height ratios. In case, an adjacent wall is running at right angle to the free end the net load near the corner reduces considerably.

In case of hoardings the pressure tends to be a little higher due to the wind flow under the hoarding. When the gap underneath the hoarding. The Code, therefore, recommends in [Table 24] the value of C_f for hoardings/walls of ≤ 15 m with various *b/h* ratios, if resting on the ground or above the ground at a height ≥ 0.25 *h* (*b* and *h* are the width and the height of the wall/hoarding respectively).

9.2.3 *Force Coefficients for Unclad Buildings*

The Code [6.3.3] specifies the values of Force Coefficients for permanently unclad buildings and for frame works of buildings which are temporarily unclad like roof trusses and frame works of multistoreyed buildings/factories during erection.

9.2.4 *Force Coefficients for Individual Members*

For individual members of infinite length the Force Coefficients are to be taken from [Table 23] and for finite length members depending upon the *l/d* or *l/b* values to be modified by a mutiplication factor κ as before. These values are further modified in case the member abuts onto a plate or wall in such a way that the free flow of air around that end of the member is prevented. In such cases the *l/b* value is doubled for determining the value of κ . In case the free flow of air is prevented around both ends of the member the value of *l/b* is = ∞ for calculating the value of κ.

9.2.4.1 *Flat sided members*

[Table 26] specifies the values of the Force Coefficients for two mutually perpendicular directions in case of flat sided members when the wind is normal to the longitudinal axis of the member. These are designated as C_{fn} and C_{ft} respectively, for normal and transverse directions and are given in [Table 26]. The normal and transverse forces are thus given by the following two relations:

Normal Force, $F_n = C_{fn} . p_d . \kappa . l . b$

Transverse Force, $F_t = C_{ft} . p_d . \kappa . l . b$

9.2.4.2 *Circular sections*

The values of Force Coefficients for such members are to be calculated as explained in **9.2.2.1**.

9.2.4.3 *Wires and Cables*

In the case of wires and cables the values of Force Coefficients are dependent on diameter, surface roughness and the design wind speed. The Force Coefficients for such members are to be taken from [Table 27].

9.2.5 *Open Plane Frames*

Single and mutiple open frames are covered in [**6.3.3.3** and **6.3.3.4**]. Depending upon the solidity ratio 'ϕ', Force Coefficients data is divided into three catagories as:

a) All flat sided members;
b) All circular members having either
   i) $DV_d < 6\ m^2/s$
   ii) $DV_d > 6m^2/s$
c) Frames with both flat and circular members.

The values of the Force Coefficients are given in [Table 28]. In case of mutiple parallel frames, the windward frames have a shielding effect on the leeward frames. The wind force on the windward frames or any unshielded part of the leeward frames is computed using [Table 28] but the wind load on fully shielded or partly shielded depends upon the effective solidity ratio and the frame spacing of the windward frame. [Table 29] gives the Shielding Factor 'η' in terms of Effective Solidity Ratio 'β' and the frame spacing ratio.

9.2.6 *Lattice Towers*

The data in [**6.3.3.5**] for overall Force Coefficients for wind blowing into the face of the lattice tower *vis-a-vis* Solidity Ratio 'ϕ' is divided as under:

a) Square towers with flat members,
b) Square towers with rounded members, and
c) Equilateral triangular towers with rounded members.

In case of square towers with flat members, wind is also to be considered blowing into a corner of the tower; for which purpose the Code recommends a multiplying factor of 1.2 for the wind blowing into the face.

9.2.7 *Solved Examples by Force Coefficient Method*

Example 16. *A hoarding of 2.5 m × 1.5 m at height with its centre 2.5 m above ground level is to be erected on National Highway outside Delhi (Fig. 30).*

Given:

Physical Parameters:
Height (h) : 1.5 m
Length (l) : 2.5 m
Height of its centre above the ground : 2.5 m

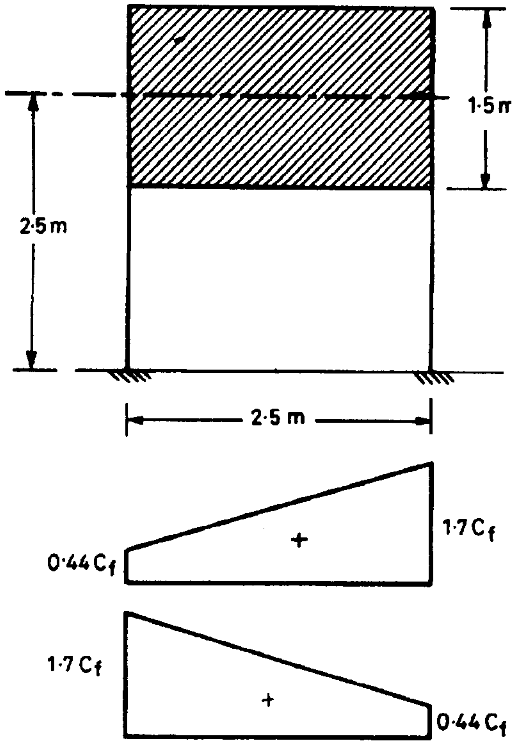
Wind Data

Wind zone : 4 (Basic wind speed = 47 m/s) for Delhi
Terrain category : 1 [**5.3.2.1**]
Class of structures : A [**5.3.2.2**], since all dimensions are less than 20 m.

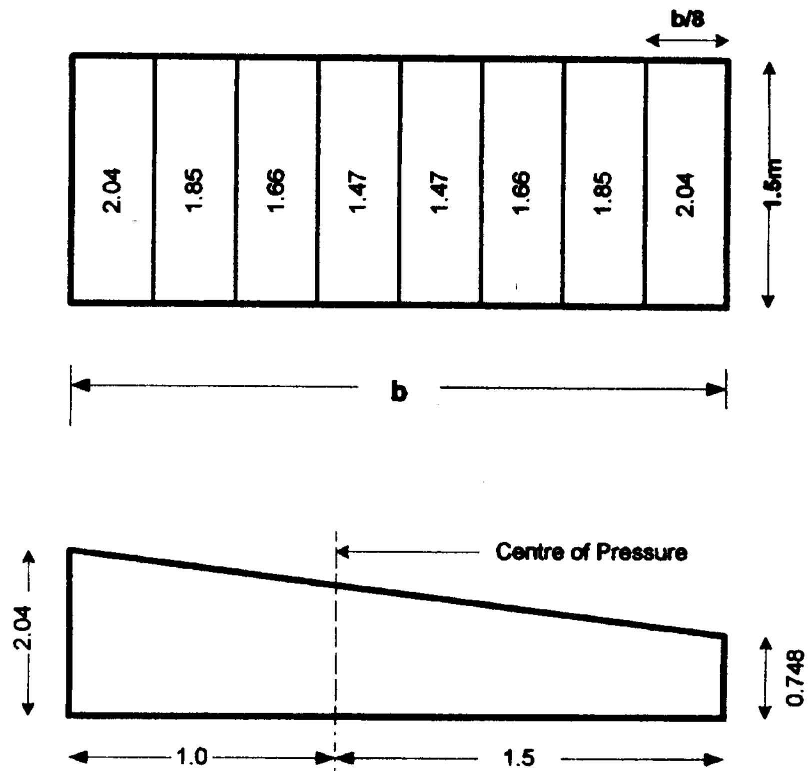
Design Wind Speed (V_z)

V_z = $V_b \cdot k_1 \cdot k_2 \cdot k_3$ [**5.3**]
V_b = 47 m/s
k_1 = Probability factor for V_b = 47 m [**5.3.1** and Table 1]
= 0.71 (considering mean probable design life of structure as 5 years)
k_2 = Terrain, height and structure size factor [**5.3.2** and Table 2] for height $\leq$ 10 m
= 1.05
k_3 = Topography factor [**5.3.3** and Appendix C]
= 1.0, that is, ground is almost flat.

Hence, $V_{10} = 47 \times 0.71 \times 1.05 \times 1.0 = 35.04$ m/s



30 (a) Pressure Normal to Surface Pressure



c_f **Distribution**

30 (b)

FIG. 30 HOARDING

Design Wind Pressure (p_z) [5.4]

$p_z = 0.6 \times (V_z)^2$ and $p_{10} = 0.6 \times (35.04)^2 = 736.68$ N/m^2
C_f = Force coefficient for low wall or hoarding < 15 m

Wind Pressure and Forces [6]

Force Coefficients [6.3 and Table 24]

$b = 2.5$ m
$h = 1.5$ m
$h' = 1.75$ m so $h' > 0.25h$ and $b/h = 1.67$

Hence, $C_f = 1.2$

Effective Frontal Area (A_e) = 2.5 × 1.5 = 3.75 m^2

Wind Force on Hoarding (F)

$F = C_f \cdot A_e \cdot p_d$

$F = 1.2 \times 3.75 \times 736.68 = 3\,315.06$ N or 884.02 N/m^2

Check for Oblique Winds [6.3.2.3]

The value of C_f in the case is 1.7 on one edge and 0.44 on another edge [Fig. 30 (a)].

For overall forces, C_f is to be considered average of these two values, that is, C_f (average) = 1.284

Wind force (average wind force due to oblique wind)

= 1.284 × 3.75 × 736.68 N
= 3 547.11 N or 945.90 N/m^2

However, the pressure distribution on the hoarding is given in [Fig. 30 (b)] alongwith the eccentricity of the force.

NOTE — The static force of 3 547.11 N due to oblique incidence of wind at an eccentricity of 0.25 m from the center of hoarding sheet is to be considered for design of support. The pressure varies on hoarding sheet from maximum of 2 254.24 N/m^2 at edges to 1 085.13 N/m^2 near central portion, with average pressure of 945.90 N/m^2.

Though dynamic wind force may not be experienced by such low structure, the large hoarding may get twisted in strong winds and tearing of sheets may occur near the edges if not designed properly. Hence, the large hoarding may be braced (in its plane) suitably and may be stiffened in normal direction (perpendicular to the plane of hoarding).

Example 17. *A masonry freestanding boundary wall (Fig. 31).*

Given:

Physical Parameters:

Height of wall above ground level	:	3.0 m
Height of fencing (barbed wire)	:	0.6 m
Width of wall	:	0.345 m uniform height
Length of wall	:	1 km
Ground	:	Almost flat
Mass density of brick masonry	:	17.275 kN/m^3

Wind Data

Wind zone	:	2 (Basic wind speed = 39 m/s).
Terrain category	:	2
Class of structures	:	C **[5.3.2.2]**.

Design Wind Speed (V_z)

$V_z = V_b \cdot k_1 \cdot k_2 \cdot k_3$ **[5.3]**
V_b = 39 m/s
k_1 = Probability factor **[5.3.1** and Table 1]
= 0.92 (considering mean probable design life of wall as 25 years)
k_2 = 1.05 for terrain category 2 and class of structure C
k_3 = 1.0, that is, ground is almost flat.

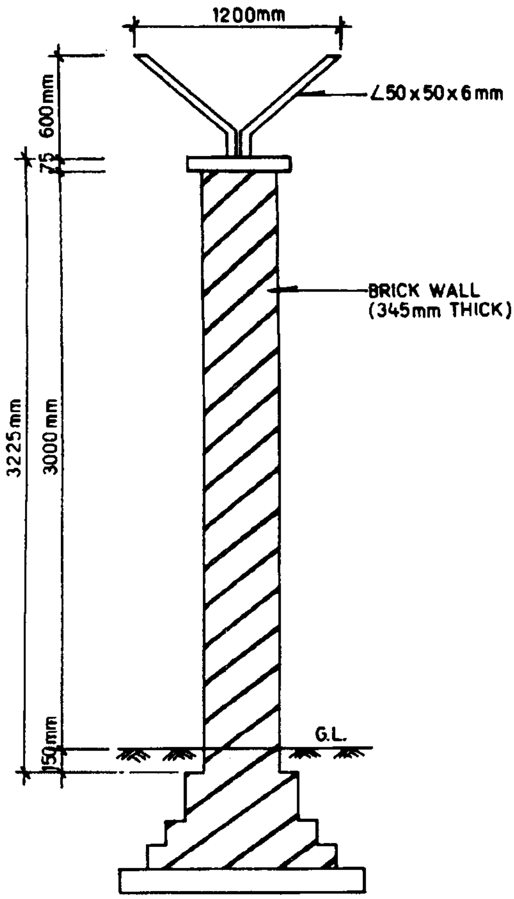


FIG. 31 BOUNDARY WALL (LENGTH ≈ 1km)

Hence

$V_{10} = 39 \times 0.92 \times 0.93 \times 1.0 = 33.37$ m/s

Design Wind Pressure (p_z) [5.4]

$p_z = 0.6 \times (V_z)^2$

$p_{10} = 0.6 \times (33.37)^2 = 668.13$ N/m^2

Wind Load Calculation

Wind force is given by

$F = C_f \cdot A_e \cdot p_z$

Choice of C_f [**6.3.2.3** and Table 24]

Here, b =1 km, h = 3 m and b/h > 160

Since wall is on ground $C_f = 2.0$

Force on Barbed Wire

Solidity ratio = 10 percent, corresponding value of C_f for single frame having flat sided members is 1.9 [Table 28]

$A_e = 0.1$ m^2

$F = 1.9 \times 0.1 \times 668.13 = 127$ N

Force on Wall

$F = 2 \times 1 \times 668.13 = 1\ 336.26$ N/m^2

Forces at Base

Bending moment due to barbed wire and wall

$= 127 \times 3.525 + 0.5 \times 1\ 336.26 \times 3.075 \times 3.225 = 7\ 073.44$ N-m

Total shear at base
$= 127 + 1\ 336.26 \times 3.075 = 4.236$ kN/m length

Calculation of Stresses at Base

Total weight of masonry at base
$= 17.275 \times (3.225 \times 1 \times 0.345) = 19.22$ kN/m width of wall

Bending stress caused by vertical bending produced at base on extreme edge
$= 6\ M/bd^2 = 6 \times 7\ 073.44/1 \times 0.345^2 = 0.357$ N/mm^3

Compressive stresses due to weight at base
$= P/A = 19.22/(1 \times 0.345) = 0.055$ N/mm^2

Maximum tensile stress at extreme fibre for vertical bending
$= (0.055 - 0.357) = 0.3$ N/mm^2

Check for Tensile Stress

According to **5.2.3.2** of IS 1905 : 1980 'Code of practice for structural safety of buildings: Masonry walls (*second revision*)', permissible tensile stress for freestanding vertical walls in vertical bending is 0.07 N/mm^2. However, the tensile stress at base of wall at ground level amounts to 0.3 N/mm^2, therefore, exceeds the permissible value. The base width of wall is to be increased for tensile stress design. The thickness of wall at various heights is given by:

$$t \approx h\ \sqrt{4\ 008.78 \div (7 \times 10^4 + 17\ 275h)}$$

Thus
t at height 1 m from top = 0.214 m
t at height 2 m from top = 0.392 m
t at height 3 m from top = 0.544 m

NOTES

1 The tensile stress criteria may govern the design. For preliminary design the following formula may be used to calculate the wall thickness at a height for prismatic walls not exceeding 10 m in height.

$$t \approx h\ \sqrt{(3 \times C_{fw} \times p_d) \div (7 \times 10^4 + \rho_\beta.h)}$$

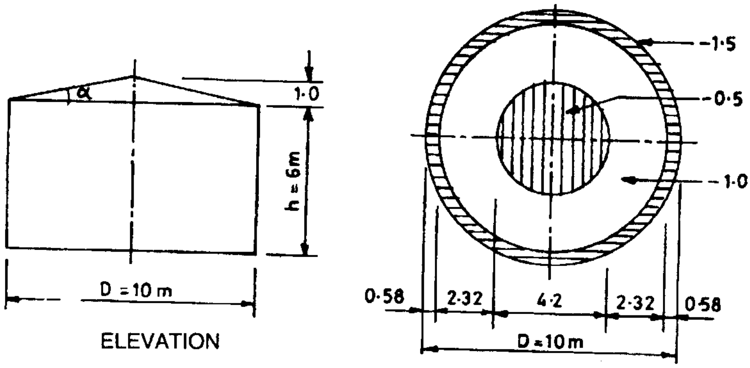
where

t = thickness wall in m,
h = height of section from top of wall (m),
C_{fw} = force coefficient for wall,
p_d = design wind pressure, and
ρ_β = mass density of masonry.

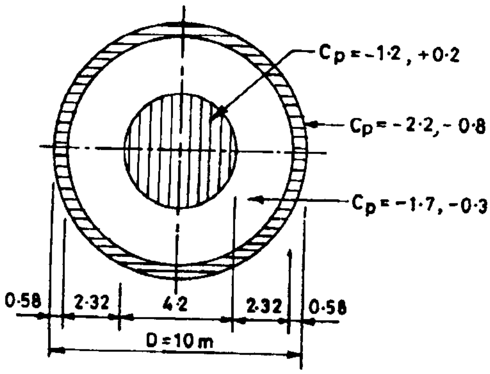
2 The permissible tensile stress in masonry is based on assumption that the mortar used is not leaner than M1 (C/S = 1/5).

3 Though [Table 1] considers mean probable design life of 5 years for boundary walls but the designer may decide higher k_1 factor as given in [Note of Table 1] depending upon the importance given to the structure such as security wall where mean probable design life may be taken as 25 years.

Example 18. *A closed cylindrical petrochemical tank (Fig. 32)* **[6.2.2.8].**



32(a) C_{pe} for Roof



C_p (Design) for Roof

FIG. 32 A CLOSED CYLINDRICAL PETRO-CHEMICAL TANK

Given:

Physical Parameters:

Height (h)	:	6 m
Diameter (d)	:	10 m
Roof angle (α)	:	11.31°
Topography	:	Flat ground
External surface	:	Rough

Wind Data

Wind zone	:	4 (Basic wind speed = 44 m/s)
Terrain category	:	2 **[5.3.2.1]**
Class of structure	:	A **[5.3.2.2]**

Design Wind Speed (V_z)

V_b = 44 m/s
k_1 = 1.07 (considering important structure having a return period of 100 years) **[5.3.1]**
k_2 = 1.0 **[5.3.2.2**, Table 2]
k_3 = 1.0 (site upwind slope < 3°) **[5.3.3.1]**
V_{10} = $44 \times 1.07 \times 1.0 \times 1.0 = 47.08$ m/s

Design Wind Pressure (p_z)

$p_{10} = 0.6 \times (47.08)^2 = 1\,329.92$ N/m^2

Wind Load Calculation

$F = (C_{pe} - C_{pi})\, A.pd$ **[6.2.1]**

Internal Pressure Coefficients on Roof (C_{pi}) [6.2.3.1 and 6.2.2.8]

As per [**6.2.3.1**], the permeability of completed tank may be less than 5 percent and corresponding $C_{pi} = \pm 0.2$ but during fabrication a stage may come when opening on roof etc may be equivalent to open ended cylinder. In such a situation for $h/d = 0.6$ as per [**6.2.2.8**] $C_{pi} = -0.8$. However, there can be an inside positive pressure of +0.7 [**6.2.3.2**] during construction phase. However, if the tank is pressurized for whatever reason, the working pressure must be taken for C_{pi}. The design should be checked with $C_{pi} = -0.8$ as long as there is no roof. Once the placing of the roof begins, the internal pressure, should be changed in accordance with [**6.2.3.2**]. This means positive internal pressures of 0.7 can be expected.

External Pressure Coefficients on Roof (C_{pe}) [Fig. 2]

Here, $h/d = 0.6$ and $\tan \alpha = 0.2$, and [**6.2.2.12** and Fig. 2] are applicable.

Therefore, then $a = 0.15\,h + 0.2\,D$ [since $0.2 < h/d < 2$] = 2.9 m

Since wind may come from any direction, the C_{pe} values are given in [Fig. 32(a)]

Design Pressure Coefficients on Roof (C_p)

$C_p = C_{pe} - C_{pi}$, $C_{pi} = -0.8$

The values of C_p for design of roof cladding when the shell is empty are given in [Fig. 32(b)].

Forces on Shell Portion

For component design, pressure coefficients as described in [Table 18] are to be used.

For overall forces [Table 23] is to be used.

Overall forces on shell (for completed tank)

Here, $V_d.b \geq 6$, $h/D = 0.6$ and external surface is rough

$C_d = 0.7$ [Table 23]

$A_e = 10\,(6+1) = 70\ m^2$

Force $F = C_f.\, A_e.\, p_d = 0.7 \times 70 \times 1\,329.92 = 65\,166.08$ N

When tank is open

When tank is open at top, the external pressure distribution around the tank may be considered as applicable to cylindrical structures described in [Table 18], for $h/D = 1$ and different positions of periphery θ, C_{pe} value are:

θ	0°	15°	30°	45°	60°	75°	90°	105°	120°	135°	150°	165°	180°
C_{pe}	1.0	0.8	0.1	–0.7	–1.2	–1.6	–1.7	–1.2	–0.7	–0.5	–0.4	–0.4	–0.4

$C_{pi} = -0.8$ (when the tank is without roof), and
$C_{pi} = \pm 0.5$ (when the tank is partially open) [**6.2.3.2**]

The maximum net C_p and force are tabulated below

θ	0°	90°	180°
C_{pe}	+1.0	–1.7	–0.4
C_{pi}	–0.8	–0.8	–0.8
C_p	–1.8	–0.9	+0.4
Force N/m^2	–2 393.86	–2 260.86	–1 063.94

Example 19. *A circular water tank resting on shaft (Fig. 33).*

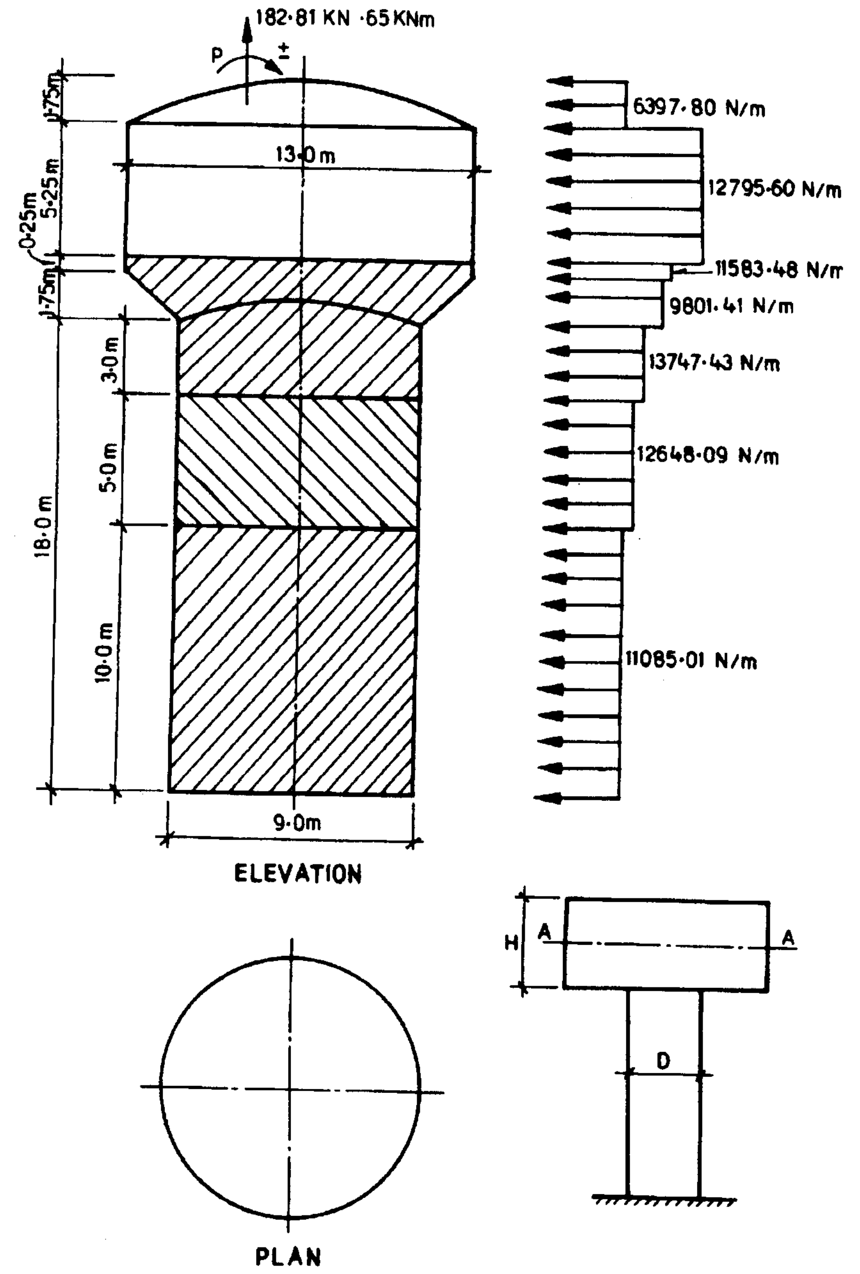


FIG. 33 WATER TANK RESTING ON SHAFT

Given:

Physical Parameters:

Height (h) below top cap	:	25.25 m
Total height (H)	:	27.00 m
Diameter of shaft (outer)	:	9.0 m
Diameter of tank	:	13.0 m
Surface condition	:	Fairly rough on tank and fluted on shaft

Since the height to minimum lateral dimension is less than 5.0, it is not necessary to check the design for dynamic forces [**7.1**].

Wind Data

Wind zone	:	3 (Basic wind speed = 47 m/s)
Terrain category	:	1
Class of structure	:	B
Topography	:	Almost flat
Probable design of structure	:	50 years.

Design Wind Pressure at Various Heights ($p_z = i$)

For calculating design wind pressure at various elevations of the tank only k_2 will change and accordingly p_z at ith elevations have been computed below.

At 27m elevation

V_b = 47 m/s
k_1 = 1.0
k_2 = 1.03 as per [Note 2 of Table 2]
k_3 = 1.0
V_z = $47 \times 1.03 = 48.41$ m/s
p_z = $0.6 \times (48.41)^2 = 1\,406.11$ N/m^2 between 20 m to 27 m

For 20 m to 15 m height

k_2 = 0.98
V_z = $47 \times 0.98 \times 1.0 \times 1.0 = 46.06$ m/s
p_z = $0.6 \times (46.06)^2 = 1\,272.91$ N/m^2

For 10 to 15 m height

k_2 = 0.94
V_z = $47 \times 0.94 \times 1.0 \times 1.0 = 44.18$ m/s
p_z = $0.6 \times (44.18)^2 = 1\,171.12$ N/m^2

For 0 to 10 m height

k_2 = 0.88
V_z = $47 \times 0.88 \times 1.0 \times 1.0 = 41{:}36$ m/s
p_z = $0.6 \times (41.36)^2 = 1\,026.39$ N/m^2

Wind Load Calculation

Uplift wind pressure on roof [Table 19] with H = Total height of the tank portion and D = diameter of the tank portion,

H/D = $9/13 = 0.69$
C_{pe} = -0.78 (by interpolation)
and C_{pi} = ± 0.2 (for less than 5 percent openings)

Vertical load on roof **[6.2.2.9]**

$$P = 0.785\,D^2\,(C_{pi} - C_{pe})\,p_d \text{ with eccentricity } e = 0.1D$$
$$P = 0.785 \times 13^2\,\{0.2 - (-0.78)\,(1\,406.11)\}$$
$$= 182\,810.75 \text{ N}$$

Eccentricity $= 0.1D = 0.1 \times 13 = 1.3$ m
Bending moment $= 1.3 \times 182.81$ kN-m
$= 237.65$ kN-m

Overall Horizontal Force on Tank and Shaft [6.3.2.1 and 6.3.2.2]

In the case of such appurtenances which 'Stand' on another structure, one is confronted with the problem of finding the height to diameter ratio. There is no guideline in the Code as to how such situations are to be handled. A reasonable approach is to take an imaginary plane at half the height and assume that the air flows symmetrically (or bifurcates symmetrically) about this plane so that this plane can be considered as a virtual ground. Then the height to be considered is $H/2$ and diameter is D and the suitable value of height to diameter ratio is $(H/2D)$. In this problem, we have to consider $H/2D = 9/(2 \times 13) = 0.346$.

$F = C_f.A_e.p_d$ [from Table 23 for circular section]
$V_d.b \geq 6$ and rough surface
For tank, $H/D \approx 0.346$ which is less than 0.5 and hence, $C_f = 0.7$

For the shaft, the presence of the wide water tank at the top and the ground at the bottom, may be viewed as making the air flow two dimensional that is H/D for the shaft $\rightarrow \infty$. Hence it is appropriate to take, $C_f = 1.2$ for the rough shaft.

Horizontal Force on Shaft per metre Length

(0.0 to 10.0 m height)	$= 1.20 \times (1 \times 9.0) \times 1\,026.39$ $= 11\,085.01$ N
(10.0 m to 15 m height)	$= 1.20 \times (1 \times 9.0) \times 1\,171.12$ $= 12\,648.09$ N
(15.0 m to 18 m height)	$= 1.20 \times (1 \times 9.0) \times 1\,272.91$ $= 13\,747.43$ N
(18.0 m to 19.75 m height)	$= 0.7 \times (1 \times 11.0) \times 1\,272.91$ $= 9\,801.41$ N
(19.75 m to 20.00 m height)	$= 0.70\,(1 \times 13) \times 1\,272.91$ $= 11\,583.48$ N
(20.00 m to 25.25 m height)	$= 0.7 \times 135 \times 1\,406.11$ $= 12\,795.6$ N
(25.25 m to 27.00 m height)	$= 0.7 \times (13.0/2) \times 1\,406.11$ $= 6\,397.80$ N

[Appendix D] and its note also specify the value of C_f for circular sections and the procedure is as under. It is to be noted that the pressure at the nearest nodal point of [Table 2] above the region is to be used.

The value of the force coefficient given in [Table 23] for circular cross-section may not appear to be consistent with the values read from [Fig. 5], and corrected for height to diameter ratio as per the recommendations in [Table 25]. It will be observed that in general the values in [Table 23] are higher even if one assumes that the values at finite H/D are those corresponding to the use of multipliers in [Table 25] on the values at $H/D \rightarrow \infty$. However, from the wording in [Table 23], it is clear that the roughness envisaged in [Table 23] includes deliberately made small projections, possibly from architectural considerations. This case is not considered in [Fig. 5] The word 'Smooth' in [Table 23] really means in this problem $\varepsilon/D = 0.05 \times 10^{-5}$ of [Fig. 5] (for the force coefficient of 0.6 at $H/D \rightarrow \infty$). For a diameter of 13.0 m, this means that the combined deviations/roughness of surface must be less than 0.65 mm, which is not likely to be achieved in construction.

If the surface is stipulated to be smooth with no fluting or projections, designer may take the values in [Fig. 5] instead of values in [Table 23], taking the multiplier as 0.8 for $H/D < 2.0$. In the present problem the force coefficients then will be 0.64 for tank and 0.8 for shaft.

Example 20. *Consider the circular tank of Example 19 to be supported on 16 columns of 45 cm diameter @ 22.5° radially with a radius of 4.5 m. Other parameters remain unchanged (Fig. 34).*

As in the previous problem, the height to maximum width can be taken as 3.0 and hence no dynamic analysis [**7.1**] is needed.

Force on roof of the dome [**6.2.2.9** and Table 19]

Since tank rests on columns, it is covered under Case (d) of [Table 19]

$H = 9.0$ m, $Z = 27.0$ m, $D = 13.0$ m and $(Z/H - 1) = 2$;
The value of $C_{pe} = -0.75$ (for roof) and $= -0.6$ (for bottom).
Since, openings are less than 5 percent, $C_{pi} = \pm 0.2$

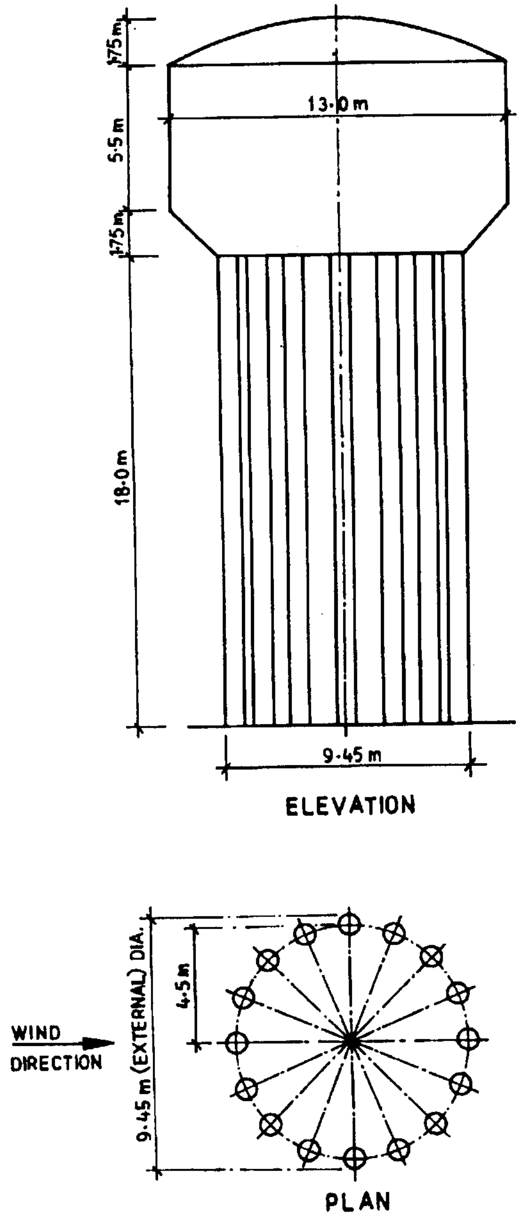


FIG. 34 WATER TANK RESTING ON COLUMNS

Uplift Force on Roof per Square Metre

$= \quad (C_{pe} - C_{pi})\, A.p_z$
$= \quad (-0.75 - 0.2) \times 1 \times 1\,406.11 = +1\,335.80$ N/m^2

Force on Bottom per Square Metre

$= \quad (-0.6 - 0.2) \times 1 \times 1\,272.91 = -1\,018.33$ N/m^2

Uplift Force on Roof [Table 19]

$P \; = \quad 0.785\, D^2\, (C_{pi} - C_{pe}.P_d)$
$= \quad 0.785 \times 13^2\, (0.2 + 0.75 \times 1\,406.11)$
$= \quad 177\,214.50$ N with an eccentricity 'e' = 0.1 D = 1.3 m

Horizontal Force on Tank [6.3.2.1 and 6.3.2.2, Table 23]

For $H/D \approx 0.347$, rough surface, and $V_d\, b \geq 6$;
$C_f = 0.7$

Force on cylinder of tank = 0.7 × (1 × 13) 1406.11 = 12 795.60 N/m^2

Force on taper = 0.7 (1 × 11.1) 1 272.91 = 9 890.51 N/m^2

Force on staging [**6.3, 6.3.3** and **6.3.3.3**, Table 28]

Consider the columns as a frame around a periphery and using [**6.3.3.3**, Table 28]

No. of columns = 16 $D.V_d > 6$ m/s^2

Total gap between cylindrical columns (Fig. 34) is

$g = [r_0 \sin 22.5° - r_1] + \{[r_0 \sin 45° - r_1] - [r_0 \sin 22.5° + r_1]\} + \{[r_0 \sin 67.5° - r_1] - [r_0 \sin 45° + r_1]\} + \{[r_0 \sin 90° - r_1] - [r_0 \sin 67.5° + r_1]\}$

There is no gap for air flow between the columns at $\theta = 90°$ and $\theta = 67.5°$

Hence net gap

$g = r_0 \sin 67.5° - 6r_1 = 4.5 \sin 67.5° - 6 \times 0.225 = 2.807$ m

Solid area = 9.45 – 2.807 = 6.643 m^2

Solidity ratio (ϕ) = 6.643/9.45 = 0.70

Force coefficient (by interpolation) on wind facing columns = 1.28

Overall Forces on Staging

There is no specific guideline in the Code for finding out the force coefficient for such shielded configuration. A conservative approach is to note that gap flow occurs only for θ up to 67.5 and hence the 'frame spacing' as per [**6.3.3.4**] is likely to be closer to unity. The range of values for ϕ and β in [Fig. 7] does not cover the situation here. Hence, it is conservative to take 'effective solidity' as 0.7 and 'frame spacing ratio' as 1 .0 in [Table 29] which yields a shielding factor of 0.6. Hence total force coefficient, C_f = 1.28 + 1.28 × 0.6 = 1.28 (1 + 0.6) = 2.048 for combined effect of windward as well as leeward colums. Here, exposed area, A_e is 6.643 m^2 per m length of staging. The wind induced force per unit length of staging $F_z = C_f \cdot A_e \cdot P_d$, is calculated as follows.

For F_z (0.0 m to 10 m height) = 2.048 × 6.643 × 1 026.39 = 13 963.9 N/m
For F_z (10 m to 15 m height) = 2.048 × 6.643 × 1 171.12 = 15 932.93 N/m
For F_z (15 m to 18 m height) = 2.048 × 6.643 × 1 272.91 = 17 317.77 N/m

NOTE — Most of the time the columns of the staging are braced with cross-beams. The design of the beam should be able to withstand the wind loads imposed on it. Special attention is required in detailing of the junction of the beam and the column. For calculation of forces on beams let us consider cross-beams of size 450 mm (depth) and 300 mm (width) which are placed at a vertical spacing of 3 m. It is more appropriate to adopt a value of C_f from [Fig. 4A.]. The beam can be considered having parameters as a = 300 mm, b = 450 mm, a/b = 0.67 and n = distance between two columns = $r \cdot d\theta$ = 4.5 × 22.5 × 11/180 = 1767 mm. It may be noted that the ends of the cross-beam are obstructed by columns, therefore, from [**6.3.3.2 a** (ii)] the value of $h/b = \infty$ is to be adopted. Correspondingly from [Fig. 4A] C_f for windward area is 2.75 and considering the effect of shielded leeward portion the total C_f = 2.75 × (1 + 0.6) = 4.4 The exposed area of the beam all around the staging at an elevation = area of one beam (that as half perimeter × width) – area covered by columns = 0.45 × 11 × 4.5 – 0.45 × 6.643 = 3.372 m^2. Therefore, the overall forces acting on a cross-beam spanning all around the staging at an elevation 'z' is given by

For F_z (0.0 m to 10 m height) = 4.4 × 3.372 × 1026.39 = 15 228.34 N
For F_z (10 m to 15 m height) = 4.4 × 3.372 × 171.12 = 17 375.67 N

Forces on Each Cross-beam Spanning Two Columns

For calculation of forces on a cross-beam spanning two consequtive columns, the exposed area A_e = 1.0 × 0.45 = 0.45 m^2/m and C_f = 2.75. Correspondingly forces are as follows:
For F_z (0.0 m to 10 m height) = 2.75 × 0.45 × 1 026.39 = 1 270.16 N/m.
For F_z (10 m to 15 m height) = 2.75 × 0.45 × 1 171.12 = 1 449.26 N/m.

Force on Each Column per Unit Length

Force on each column for calculation of design loads are as follows. Here, C_f = 1.2 [Table 23] for an aspect ratio (H/D) of 18 / 0.45 = 40, V_d . b > 6 and for rough r. c. c. column surface, A_e = 1 × 0.45 m^2/m length of column.
For F_z (0.0 m to 10 m height) = 1.2 × 0.45 × 1 026.39 = 554.25 N/m
For F_z (10 m to 15 m height) = 1.2 × 0.45 × 1 171.12 = 632.40 N/m
For F_z (15 m to 18 m height) = 1.2 × 0. 45 × 1 272.91 = 687.37 N/m

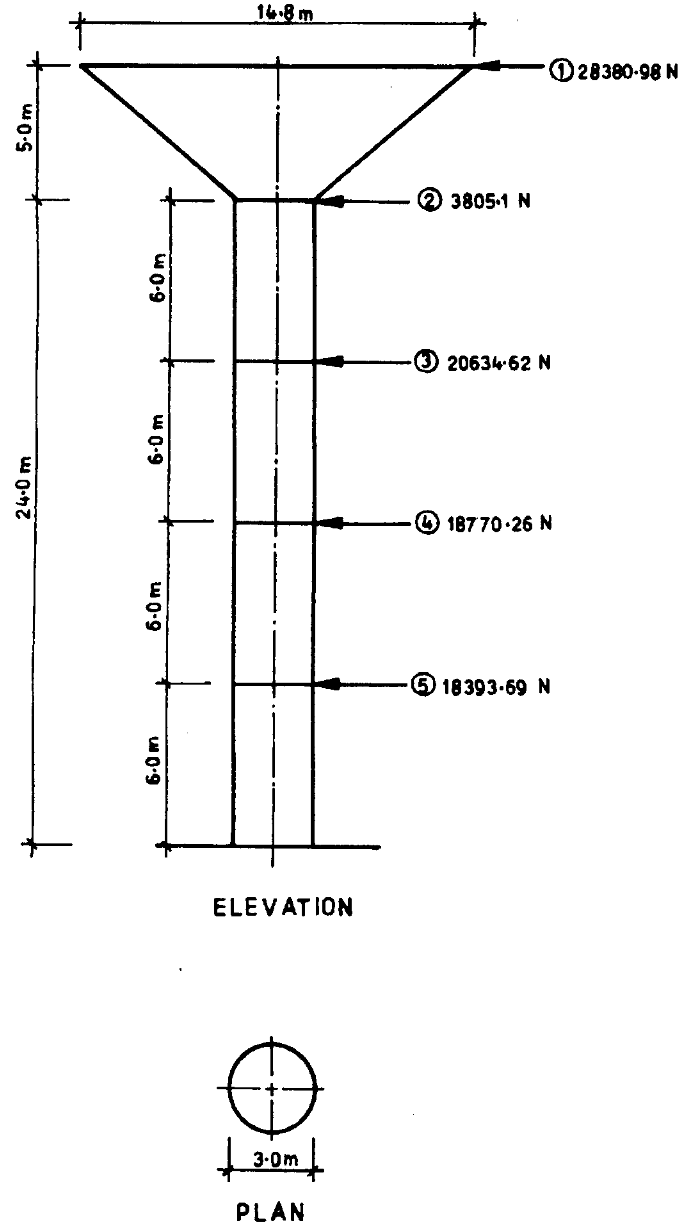


FIG. 35 AN INVERTED TYPE WATER TANK

Example 21. *An inverted type conical water tank (Fig. 35) for a nuclear power plant.*

Given:

Physical Parameters:

Height (h)	: 29.0 m
Top diameter	: 14.8 m
Bottom diameter	: 3.0 m

Wind Data

Wind zone	: 3 (Basic wind speed = 44 m/s)
Terrain category	: 2
Class of structure	: B **[5.3.2.2]**
Topography	: Flat
Probable life of structure	: 100 years.

Design Wind Speed

$V_z = V_b.k_1.k_2.k_3$
V_b = 44 m/s **[5.2]**
k_1 = 1.07 **[5.3.1]**
k_2 = 1.095 **[5.3.2.2** and Table 2] (by interpolation)
k_3 = 1.0 **[5.3.3.1]**
$V_z = 44 \times 1.07 \times 1.095 \times 1.0 = 51.55$ m/s

Design Wind Pressure

For $z = 29$
$p_z = 0.6\,(V_z)^2$
$p_z = 29 = 0.6 \times (V_z)^2 = 0.6 \times (51.55)^2 = 1\,594.44$ N/m^2

The design pressure for other elevations are given in Table 10:

Table 10 Design Pressure at Different Levels

Elevation	k_1	k_2	k_3	V_z (m/s)	p_z (N/m^2)
29	1.07	1.095	1.0	51.55	1 594.44
24	1.07	1.07	1.0	50.37	1 522.28
18	1.07	1.038	1.0	48.87	1 432.96
12	1.07	0.99	1.0	46.61	1 303.49
6	1.07	0.98	1.0	46.14	1 277.34

Force Coefficient

Height/Breadth ratio = 29/5 = 5.8; Considering the surface of the tank as rough, $V_d.b \geq 6$ and for this height/breadth ratio the force coefficient C_f as per [Table 23] is 0.8

Wind Load

Wind force $F_{29} = 0.5.C_f \,.\, pd \,.\, A$
$= 0.5 \times 0.8 \times 1\,594.44 \times A$
$= 0.5 \times 0.8 \times 1\,594.44 \times (14.8 + 3) \times 5 \times 0.5 = 28\,381.03$ N
$F_{24} = 0.8 \times 1\,522.28 \times (3 \times 3 + 22.25) = 38\,057$ N
$F_{18} = 0.8 \times 1\,432.96 \times (6 \times 3) = 20\,634.62$ N
$F_{12} = 0.8 \times 1\,303.49 \times (6 \times 3) = 18\,770.26$ N
$F_6 = 0.8 \times 1\,277.34 \times (6 \times 3) = 18\,393.69$ N

Reaction, shear force and bending moment assuming the water tank as a cantilever beam

Reaction at base = 124 236.55 N

B.M. and S.F. at an elevation of 24 m = –141 904.9 N-m and 28 380.98 N respectively

B.M. and S.F. at an elevation of 18 m = –540 532.78 N-m and 66 437.98 N

B.M. and S.F. at an elevation of 12m = – 1 062 968.3 N-m and 87 072.6 N

B.M. and S.F. at an elevation of 6 m = –1 698 025.5 N-m and 105 842.86 N

B.M. and S.F. at base = –2 443 444.8 N-m and 124 236.55 N

Example 22. *A multistoreyed framed building (Fig. 36).*

Given:
Physical Parameters:

Length	:	50.0 m
Width	:	10 m
Height	:	60 m
Height of each storey	:	4 m
Spacing of frames	:	5 m along the length

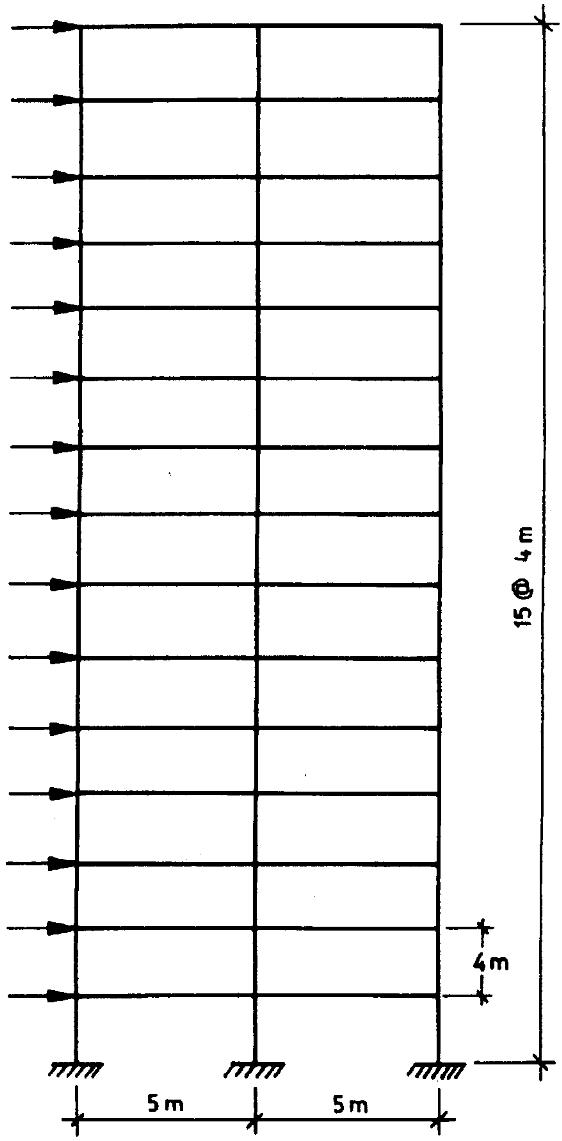


FIG. 36 A MULTISTOREY BUILDING

Wind Data

Wind zone	:	5 (Basic wind speed = 50 m/s)
Terrain category	:	3
Class of structure	:	C (since maximum dimension > 50 m)
Topography	:	Flat that is upwind slope < 3°
Life of structure	:	50 years.

Design Wind Speed

$V_z = V_b.k_1.k_2.k_3$
$k_3 = 1.00$
k_2 = varies with height
$k_1 = 1.00$
$V_z = (50 \times 1.00 \times 1.00)\, k_2 = 50.00\, k_2$

Design Wind Pressure

$p_z = 0.6\, V_z^2 = 1\,500\, k_2^2$ N/m^2

Wind Load Calculations

$F = A_e \times p_d \times C_f$

$a/b = 10/50 = 0.2$

$h/b = 60/50 = 1.2$

C_f for these values from [Fig. 4] = 1.2

After selecting proper values of k_2 from the Code, the values of the design wind pressure are computed and given in Table 11

Table 11 Wind Load at Different Floor Levels

Elevation (m)	k_2	V_z (m/s)	p_z (N/m^2)	Lateral Force (kN)
60	1.04	52.00	1 622.4	19.5
56	1.03	51.5	1 591.5	38.2
52	1.02	51.0	1 560.6	37.4
48	1.01	50.5	1 530.2	36.7
44	1.00	50.0	1 500.0	36.0
40	0.99	49.5	1 470.2	35.3
36	0.975	48.75	1 426.0	34.2
32	0.96	48.0	1 382.4	33.1
28	0.95	47.5	1 353.8	32.6
24	0.93	46.5	1 292.4	31.1
20	0.91	45.5	1 242.0	29.8
16	0.87	43.5	1 135.0	27.3
12	0.84	42.0	1 058.6	25.4
8	0.82	41.0	1 008.6	24.2
4	0.82	41.0	1 008.6	24.2
0	0.00	0.0	0.0	0.0

Example 23. *A 30 m high self supporting square tower (Fig. 37).*

Given:

Physical Parameters:

Height of tower	:	30.0 m
Base width	:	5.0 m
Top width	:	1.50 m
Number of panels	:	8
Panel height	:	Top five 3.0 m and rest three are 5.0 m each
Type of bracings	:	K-type with apex upwards in 5 m panels and X-type in 3 m panel
Member properties	:	Table 12

Wind Data

Wind zone	:	4 (Basic wind speed = 47 m/s)
Life of structure	:	100 years
Topography	:	Flat

Design Wind Speed

$V_z = V_b.k_1.k_2.k_3$

$k_3 = 1.36$

k_2 = varies with height

Table 12 Member Properties

Elevation (m)	Member	Size (mm)	No. & Length (m)
30-27	Horizontal	65 × 65 × 6	1, 1.5
	Vertical	90 × 90 × 6	1, 3.0
	Diagonal	100 × 100 × 6	1, 3.54
	Plan Bracing-3	65 × 65 × 6	1, 2.121
27-24	Horizontal	65 × 65 × 6	1, 1.5
	Vertical	90 × 90 × 6	1, 3.0
	Diagonal	100 × 100 × 6	1, 3.54
	Plan Bracing-3	65 × 65 × 6	1, 2.121
24-21	Horizontal	65 × 65 × 6	1, 1.5
	Vertical	65 × 65 × 6	2, 3.008
	Diagonal	100 × 100 × 6	1, 3.464
	Plan Bracing-3	65 × 65 × 6	1, 2.121
21-18	Horizontal	65 × 65 × 6	1, 1.937
	Vertical	65 × 65 × 6	2, 3.008
	Diagonal	65 × 65 × 6	2, 3.701
	Plan Bracing-3	80 × 80 × 6	1, 2.740
18-15	Horizontal	65 × 65 × 6	2, 2.375
	Vertical	75 × 75 × 6	2, 3.008
	Diagonal	65 × 65 × 6	2, 3.972
	Plan Bracing-3	100 × 100 × 6	1, 3.359
15-10	Horizontal	65 × 65 × 6	2, (1.406 × 2)
	Vertical	75 × 75 × 6	2, 5.027
	Diagonal	90 × 90 × 6	1, 5.317
	Plan Bracing-2	65 × 65 × 6	1, 1.989
	Plan Bracing-1	65 × 65 × 6	1, 2.812
10-5	Horizontal	65 × 65 × 6	2, (1.771 × 2)
	Vertical	90 × 90 × 6	2, 5.027
	Diagonal	90 × 90 × 6	1, 5.449
	Plan Bracing-2	75 × 75 × 6	1, 2.504
	Plan Bracing-1	65 × 65 × 6	1, 3.542
5-GL	Horizontal	65 × 65 × 6	2, (2.135 × 2)
	Vertical	100 × 100 × 6	2, 5.027
	Diagonal	100 × 100 × 6	1, 5.602
	Plan Bracing-2	90 × 90 × 6	1, 3.02
	Plan Bracing-1	65 × 65 × 6	1, 4.271

$k_1 = 1.07$
$V_z = (47 \times 1.07 \times 1.36.)\, k_2 = 68.39\, k_2$

Design Wind Pressure

$p_z = 0.6\,(V_z)^2 = 2\,806.32\, k_2^2$ N/m^2

After selecting proper values of k_2 from the Code, the values of the design wind pressure are computed and given in Table 13.

Computation of Solidity Ratio (ϕ)

Enclosed area of the top half panel that is panel between 15 m and 13.5 m height

$A = 1.50 \times 3.0/2 = 2.254$ m^2

Solid area (A_0)= Area of legs + Area of horizontal members +Area of braces
= 0.702 9 m^2

Add 10 percent for gusset plates etc = 0.702 9 + 0.070 3 = 0.773 2 m^2

Solidity ratio(ϕ) = 0.773 2/2.25 = 0.344

Force coefficient(C_f) for square lattice tower with flat members [**6.3.3.5** and Table 30]

C_f between elevation 30 m and 28.5 m elevations = 2.58 (by interpolation)

Wind Load Calculations

$F = A_e \times p_d \times C_f$

F at elevation 30 m = 0.773 2 × 2 806.32 × 1.13 × 1.13 × 2.58 = 7 150.06 N

Shear Force Calculations

Shear force at elevation 30 m = 7 150.06 N

Bending Moment Calculations

Bending moment at elevation 30 m = 0.00

Bending moment at elevation 27 m = 7 150.06 × 3.0 = 21 450.18 N-m

The panel loads, shear forces and bending moments have been computed for different elevations are given in Table 13.

Table 13 Computation of Velocity, Pressure, Drag Coefficient, Shear Force and Bending Moments — Wind Normal to any Face
k_1 = 1.07, k_3 = 1.36, V_b = 47m/s

No.	Elevation	k_2	V_A (m/s)	p_A (N/m^2)	A_e (m^2)	ϕ	C_f	Panel Load (N)	SF (N)	BM (N-m)
1.	30	1.13	77.28	3 584.25	0.773 2	0.344	2.392	7 150.06	7 150.0	0.0
2.	27	1.21	76.67	3 526.97	1.439 0	0.320	2.462	13 703.34	20 853.4	21 450.18
3.	24	1.112	76.05	3 470.16	1.586 0	0.328 5	2.437	14 623.26	35 476.7	84 010.4
4.	21	1.103	75.44	3 414.72	1.910 0	0.328 7	2.436	17 348.83	52 825.5	190 440.4
5.	18	1.088	74.44	3 322.11	2.193 5	0.308 0	2.497	20 112.25	72 937.8	348 916.8
6.	15	1.07	73.18	3 213.19	3.147 6	0.272 7	2.624	29 694.2	102 632.0	567 730.1
7.	10	1.03	70.45	2 977.92	3.183 0	0.179 7	3.000	32 237.13	134 869.1	1 080 890.0
8.	5	1.03	70.45	2 977.92	5.402 0	0.161 7	3.080	56 158.75	191 027.9	1 755 235.0
9.	0	1.03	70.45	2 977.92	—	—	—	—	191 027.9	2 710 375.0

9.3 Gust Factor Method

9.3.1 Methodology of computation of wind load by Gust Factor Method has been explained in **6**. Therefore, solved examples of structures have been given below to demonstrate the usage of the method:

9.3.2 *Solved Examples Using Gust Factor Method*

Example 24. *To find design wind pressure for a rectangular clad building.*

Given:

Physical Parameters:

Building	:	52 m long, 8 m wide and 40 m high
Life of structure	:	50 years
Terrain category	:	3
Topography	:	Flat
Location	:	Calcutta

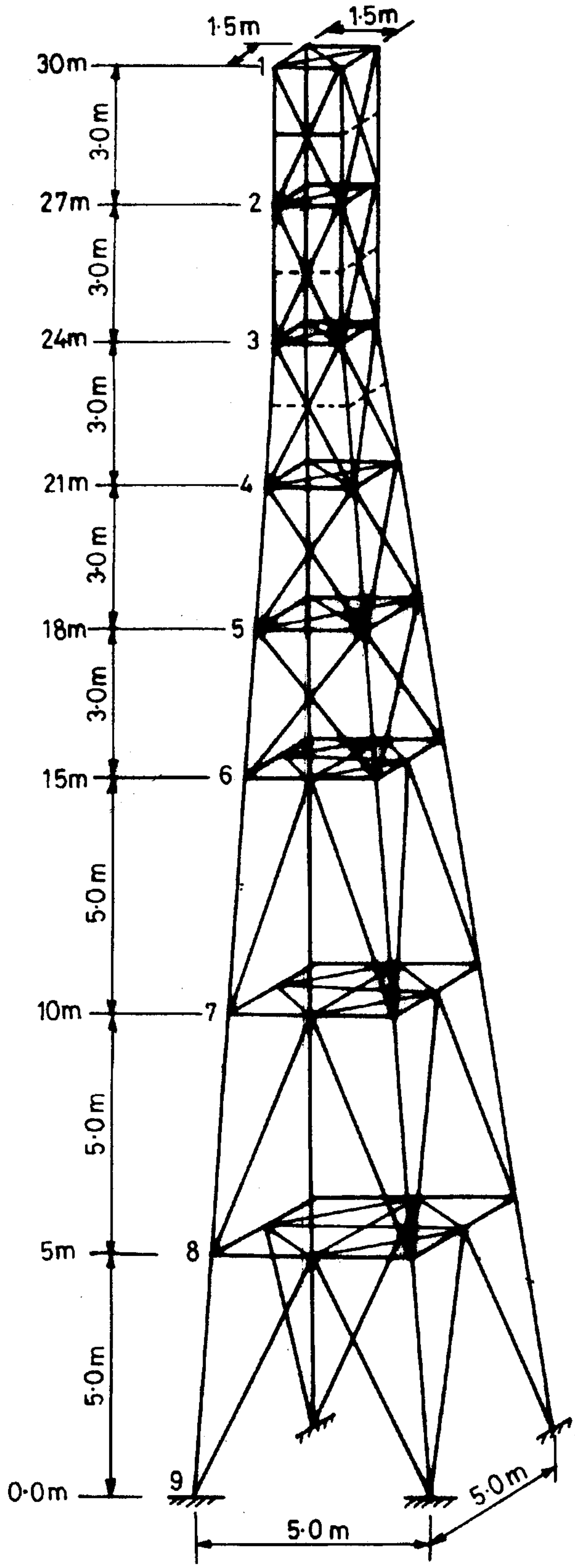


FIG. 37 LATTICE MICROWAVE TOWER

Previous Code [IS 875:1964] [Fig 1A]

$p_z = 210$ kgf/m^2 = 1 960.7 N/m^2 at 30 m height.

Present Code [IS 875 (Part 3) : 1987]

As per [**7.1**] of this Code, since the ratio of height to least lateral dimension (40/8) is 5, dynamic analysis is required.

V_b = 50 m/s
k_1 = 1.0 [Table 1, N = 50]
k_2' = 0.5 [Table 33]
k_3 = 1.0

Design Hourly Mean Wind Speed

$V_z' = 50(1.0)(0.5)(1.0) = 25.0$ m/s

Design Wind Pressure

$p_z' = 0.6\,(25.0)^2 = 375.0$ N/m^2

Computation of Gust Factor (*G*) [8.3]

$$G = 1 + g_f . r \sqrt{[B\,(1+\phi)^2 + SE/\beta]}$$

For the present case, from [Fig. 8]

$g_f \cdot r = 1.7$, $L(h) = 1\,150$ m, $C_y = 10$; $C_z = 12$

Hence, $\lambda = C_y\, b/C_z\, h = 10(8.0)/12(40.0) = 0.166\,7$

Also
$C_z\, h\,/\,L(h) = 12(40.0)/1\,150 = 0.417\,4$

Background turbulence factor B = 0.76 [Fig. 9]

Calculation of reduced frequency

To calculate this parameter, one needs to know the natural frequency (*f*) in the 'first mode'. Using the expression (*b*) of [Note 1 of **7.1**]

$T = 0.09H/\sqrt{d} = 0.09 \times 40.0/\sqrt{8.0} = 1.73$ s

$f = 0.578$ Hz

F_0 [Fig. 10] = $Cz\, f_\phi\, h\,/\,V(h) = 12(0.578)\,(40.0)/33.5 = 11.17$

Size reduction factor (S) from [Fig. 10] is 0.15

The parameter $f_\phi\, L(h)/V(h) = 0.78 \times 1\,150/33.5 = 26.78$

The gust energy factor E from [Fig. 11] is then 0.07

Since the height of the building exceeds 25 m in terrain category 3, $\phi = 0$ **[8.3]**. With the damping coefficient $\beta = 0.016$ [Table 34],

$G = 1 + 1.7 \left\{0.76(1+0)^2 + 0.15(0.07)/0.016\right\} = 1 + 2.519 = 3.519$

Hence the design wind pressure = 3.023 (375.0) = 1 319.7 N/m^2 at 40 m height.

Proceeding similarly, the design wind pressure for larger life and other terrains were considered and the results are given below at 30 m height:

Location	*Life Years*	*Terrain*	*IS 875 (Part-3) : 1987*	*IS 875 : 1964*
Calcutta	50	3	1 689.2	
	100	3	1 970.28	1 960.00
	100	2	3 002.08	

The large increase in the design pressure for Terrain Category 2 which is very common for tall industrial structures outside cities is to be noted.

Example 25. *A mutistoreyed framed building of Example 22.*

Given:

Physical Parameters:

Building	:	50 m long, 10 m wide and 60 m high
Life of structure	:	50 years
Terrain category	:	3
Topography	:	Flat
Location	:	Bhubneshwar

Design Wind Velocity

As per [**7.1**] of this Code, Ratio of height to least lateral dimension (60/10) is 6> 5; dynamic analysis is required.

$T = 0.09H/d = 0.09 \times 60/\sqrt{10} = 1.7$s

Frequency (f) = 1/1.7 = 0.58 Hz < 1.0 Hz; dynamic analysis is required

V_b = 50 m/ s

k_1 = 1.0 [Table 1, N = 50]

k_2' = 0.5 [Table 33]

k_3 = 1.0

Design Hourly Mean Wind Speed

$V_z' = 50\,(1.0)(0.5)(1.0) = 25.0$ m/s

Design Wind Pressure

$p_z' = 0.6\,(25.0)^2 = 375.0$ N/m^2

Computation of Gust Factor (G) [8.3]

$G = 1 + g_f \cdot r \sqrt{[B(1+\phi)^2 + SE/\beta]}$

For the present case, from [Fig. 8]

For 60 m height, $g_f \cdot r = 1.5$, $L(h) = 1\,200$ m, $C_y = 10$; $C_z = 12$

Hence, $\lambda = C_y\, b/C_z\, h = 10(10.0)/12(60.0) = 0.138$

Also

$C_z\, h/L(h) = 12(60.0)/1\,200 = 0.60$

Background Turbulence Factor B = 0.77 [Fig. 9]

Calculation of Reduced Frequency

F_o [Fig. 10] = $C_z f_\phi h / V(h) = 12(0.58)\,(60.0)\,/\,25 = 16.7$

Size reduction factor (S) from [Fig. 10] is 0.28

The parameter $f_\phi\, L(h)/V(h) = 0.58 \times 1\,200/25 = 27.84$

The gust energy factor E from [Fig. 11] is then 0.07

Computaion of Gust Factor

$\phi = 0$ [**8.3**]. With the damping coefficient $\beta = 0.016$ [Table 34] ,

$G = 1 + 1.5\ \sqrt{[0.77\,(1+0)^2 + 0.28\,(0.07)/0.016]} = 3.11$

Computation of wind load at different elevations of the building are given in Table 14.

Table 14 Wind Load at Different Elevations

Level	Elevation (m)	k_g'	V_z' (m/s)	p_z'	Force (kN)
1	60	0.72	36.0	777.6	28.97
2	56	0.71	35.5	756.2	56.40
3	52	0.70	35.0	735.0	54.25
4	48	0.688	34.4	710.0	52.40
5	44	0.676	33.8	685.5	50.80
6	40	0.664	33.2	661.4	48.80
7	36	0.652	32.6	637.6	47.00
8	32	0.640	32.0	614.4	45.30
9	28	0.622	31.1	580.3	42.80
10	24	0.606	30.3	540.0	39.80
11	20	0.590	29.5	522.0	38.50
12	16	0.550	27.5	453.8	33.70
13	12	0.520	26.0	405.6	30.30
14	8	0.500	25.0	375.0	28.00
15	4	0.500	25.0	375.0	28.00
16	0	0.000	00.0	000.0	00.00

Example 26. *The inverted type water tank of Example 21 using Gust Factor Method.*

Given:

Fundamental frequency of the structure: 0.49 Hz

For the 29 m height

$Vz' = V_b.k_1.k_2.k_3$

$k_1 = 1.07$
$k_3 = 1.00$
$k_2 = 0.786$
$V_z' = 44 \times 1.07 \times 0.786 \times 1.00 = 37.00$ m/s

Wind Load

$F_z = C_f.A_e.p_z'.G$
$p_z = 0.6(37)^2 = 821.4$ N/m^2
$A_e = 0.5(14.8 + 3)5 \times 0.5 = 22.25$ m^2
$C_f = 0.8$

Gust factor $G = 1 + g_f \, . \, r\sqrt{[B(1+\phi)^2 + SE/\beta]}$
$g_f.\ r = 1.29$ [Fig. 8]
$\beta = 0.016$
$C_y = 10,\ b = 6,\ C_z = 12,\ h = 29$

$$C_y.b/C_z.h = \frac{10}{12} \times \frac{6}{29} = 0.172\,4$$

$$C_z.h/L(h) = \frac{12 \times 29}{1\,125} = 0.309$$

For these values, $B = 0.8$ from [Fig. 9]

Size reduction factor $(F_0) = C_z.f_0.\ h\ /\ V_h' = 12 \times 0.49 \times 29/37 = 4.61$

For these values, from [Fig. 10] $S = 0.45$

Calculation of E (Gust energy factor) $= 0.49 \times 1\,125/37 = 14.89$

E from [Fig. 11] = 0.09

ϕ = 0 in this case

$G = 1+1.29\sqrt{[0.8 + (0.45 \times 0.09/0.016)]} = 3.35$

Force = $0.8 \times 22.25 \times 821.4 \times 3.35 = 48\,992.00$ N

For the 24 m height

V_b = 44 m/s

k_1 = 1.07

k_2 = 0.766

k_3 = 1.00

$V_z' = 44 \times 1.07 \times 0.766 \times 1.00 = 36.06$ m/s

$p_z' = 0.6\,(36.06)^2 = 780.19$ N/m^2

Force = $780.19 \times 0.8 \times 3.35 \times (22.25 + 9) = 65\,340.91$ N

For the 18 m height

k_2 = 0.738

$V_z' = 44 \times 1.07 \times 0.738 \times 1.00 = 34.74$ m/s

$p_z' = 0.6\,(34.74)^2 = 724.12$ N/m^2

Force = $724.12 \times 0.8 \times 3.35 \times (9 + 9) = 34\,931.55$ N

For the 12 m height

k_2 = 0.69

$V_z' = 44 \times 1.07 \times 0.69 \times 1.00 = 32.48$ m/s

$p_z' = 0.6\,(32.48)^2 = 632.97$ N/m^2

Force = $632.97 \times 0.8 \times 3.32 \times (9 + 9) = 30\,261.03$ N

For the 6 m height

k_2 = 0.67

$V_z' = 44 \times 1.07 \times 0.67 \times 1.00 = 31.54$ m/s

$p_z' = 0.6\,(31.54)^2 = 596.86$ N/m^2

Force = $596.86 \times 0.8 \times 3.35 \times 18 = 28\,792.526$ N

Treating the water tank as a cantilever beam, support reaction

R_A = 208 318.02 N

B.M. and S.F. at a section *x* metres from point *B*

Reaction V_x = 48 992.00 N

Moment $M_x = -FB\,.\,x$

If $x = 0$ $M_x = 0$

If $x = 5$ $M_x = -48\,992 \times 5 = -24\,4\,960$ N-m

B.M. at a section *x* metres from point *F*

$V_x = FB + FF$

$= 48\,992 + 65\,340.91 = 114\,332.91$ N

$M_x = -[FB\,(5 + x) + FF(x)]$

If x = 6 m

$M_x = -[48\,992 \times 11 + 65\,340.91 \times 6]$

$= -[538\,912 + 392\,045/46] = -930\,957.46$ N-m

B.M. and S.F. at a section *x* metres from point *E*

$V_x = 48\,992 + 65\,340.91 + 34\,931.55 = 149\,264.46$ N

$M_x = -[FB\,(11 + x) + FF\,(6 + x) + FE\,(x)]$

If x = 6 m

$M_x = -[48\,992 \times 17 + 65\,340.91 \times 12 + 34\,931.55 \times 6]$

$= -[832\,864 + 784\,090.92 + 209\,589.3] = -1\,826\,544.2$ N-m

B.M. and S.F. at a section x metres from point D

$V_x = FD + FE + FF + FB = 179\,525.49$ N

$M_x = -[48\,992\,(17 + x) + 65\,340.91\,(12 + x) + 34\,931.55\,(6 + x) + 30\,261.03x]$

If $x = 6$ m

$M_x = -[1\,126\,816 + 1\,176\,136.3 + 419\,178.6 + 181\,566.48] = -2\,903\,697.3$ N-m

B.M. and S.F. at a section x metres from point C

$V_x = 208\,318.02$ N

$M_x = -[48\,992\,(23 + x) + 65\,340.91\,(18 + x) + 34\,931.55\,(12 + x) + 30\,261.03\,(6 + x) + 28\,792.53x]$

If $x = 6$ m

$M_x = -[1\,420\,768 + 1\,568\,181.8 + 628\,767.9 + 363\,132.36 + 172\,755.18]$

$= -4\,153\,605.2$ N-m