BRE Digest

Concise reviews of building technology

Digest 346
November 1989

CI/SfB (J4)

The assessment of wind loads

Part 5: Assessment of wind speed over topography

This Digest is the fifth in a series which is compatible with the proposed British Standard BS 6399:Part 2. It deals with the assessment of wind speeds over topographic features such as hills, ridges, escarpments and cliffs for wind loading calculations. The information here is generally similar to that in Digest 283, but now the new topography factor S_{TOP} is *added* to the wind speed for flat terrain. The proposals for the topography factor are soundly based on theory and agree in value to better than 10 per cent for typical UK topography with the more empirical rules incorporated in BS 8100:Part 1: *Lattice towers and masts.*

This Digest supersedes Digest 283 which is now withdrawn.

Building Research Establishment
DEPARTMENT OF THE ENVIRONMENT

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Definition of terms

G	Gust factor
H	Height above defined topography
L	Horizontal length of the upwind slope measured from foot to crest/summit in the wind direction
L_E	Effective length of upwind slope
q	Dynamic pressure
S_{TOP}	Topography factor
s	Speed increment coefficient
V	Gust wind speed
V	Hourly mean wind speed
V_s	Design wind speed
x	Horizontal position of the site measured in the wind direction from the crest summit (upwind negative; downwind positive)
Z	Vertical height between the foot and the crest 'summit'
ψ	Upwind slope (Z/L)
ψ_E	Effective upwind slope

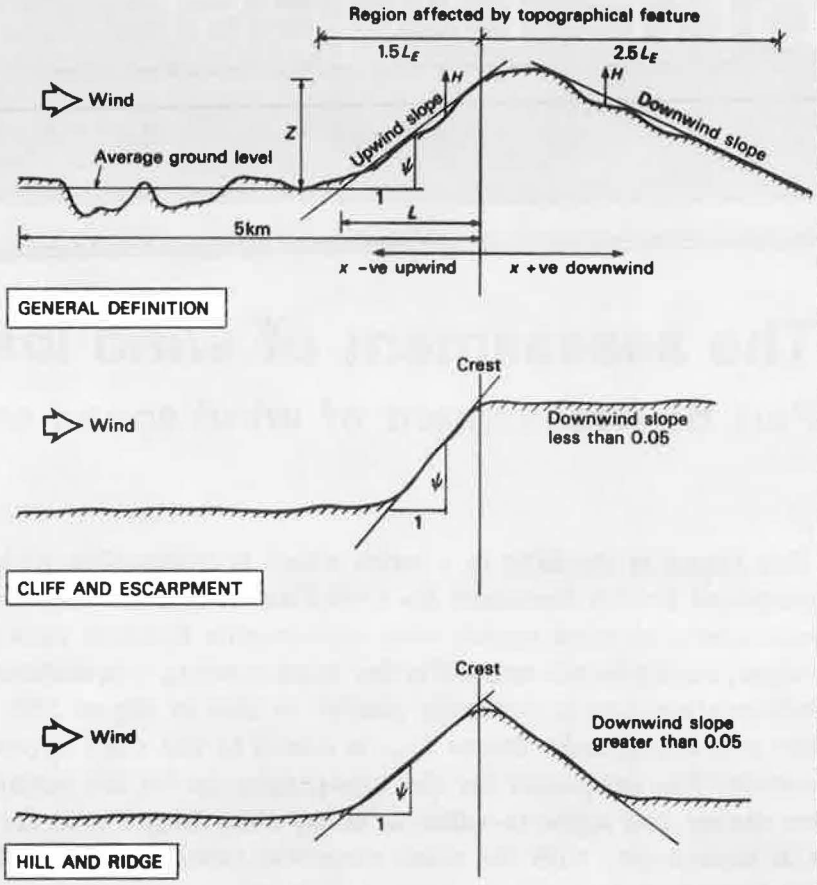


Fig 1 Definitions of topographical dimensions

THE EFFECTS OF TOPOGRAPHY ON THE WIND

Near the summits of hills, or the crests of cliffs, escarpments or ridges, the wind speed is accelerated. In the valleys or near the foot of steep escarpments or ridges the flow may be decelerated. Topography is classified by two parameters: the upwind slope and the form or shape of the topography — see Fig 2; each parameter can be deivided into three categories:

The three categories dependent on the upwind slope are:

Gentle topography: where ψ is less than 0.05;
Shallow topography: where ψ is 0.05 to 0.3;
Steep topography: where ψ is greater than 0.3;

The three categories dependent on the form of the topography are:

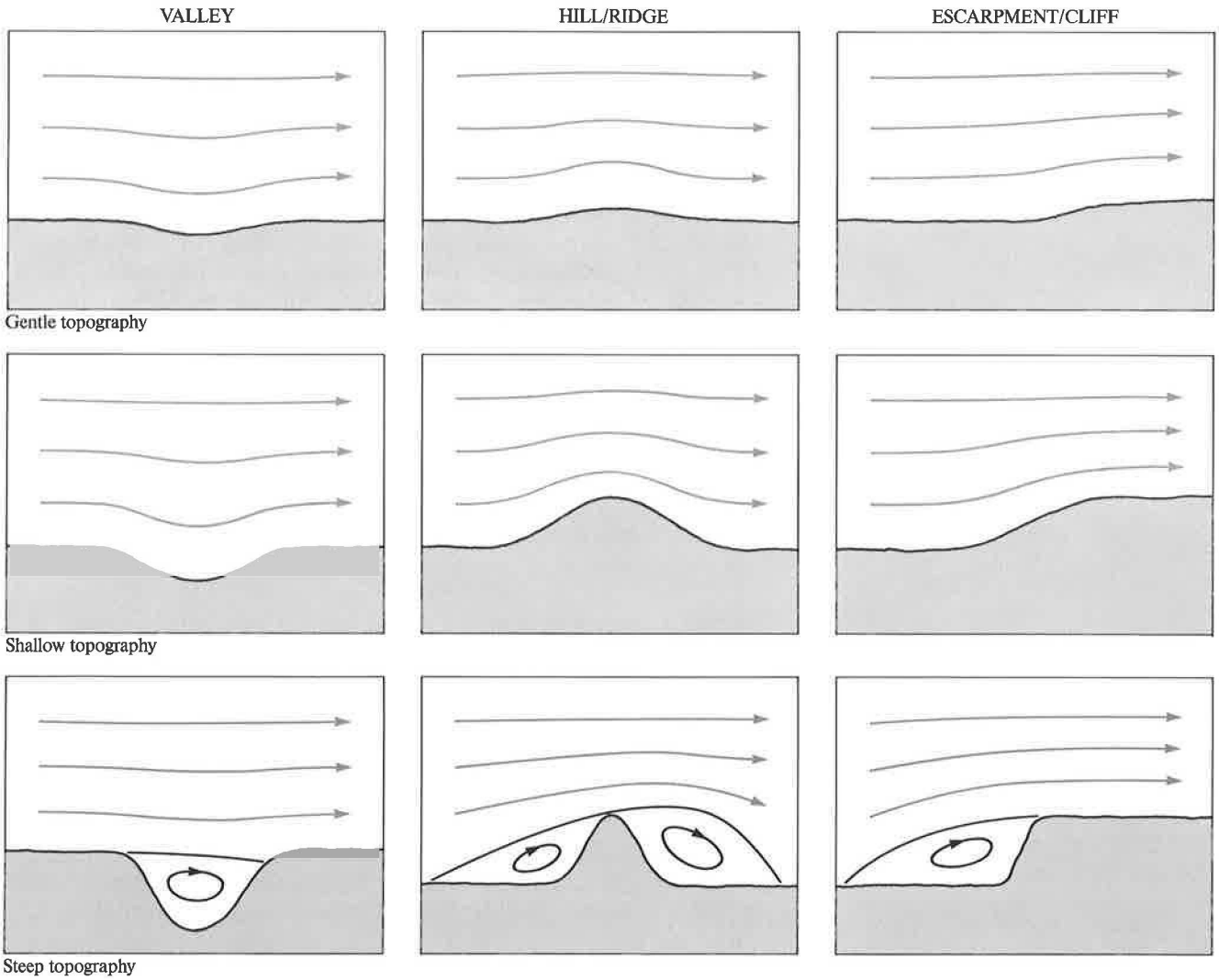
Valleys: where the ground level falls then returns to the original level;
Hills and ridges: where the ground rises then returns to the original level;
Escarpments and cliffs: where the ground rises or falls and then remains at the new level.

If the downwind slope is sensibly level (slope less than 0.05) for a distance exceeding both L and $3.3Z$ the feature should be treated as an escarpment of cliff; otherwise the feature should be treated as a hill, ridge or valley. In undulating terrain it is often difficult to define the base level from which to assess the dimensions of the feature; the average level of the terrain for a distance of 5 km upwind of the site should be taken as the base level in these cases.

Gentle topography
When changes in ground level are gentle (ψ less than 0.05), the balance between the mean wind speed and the turbulence is not significantly disturbed. There is, however, a small but significant dependence on site altitude which affects equally the mean wind speed, the turbulence intensity and the gust speeds. Each of these wind speeds may be taken as increasing by one per cent per 10 metres of site altitude; for the mean speeds this is accounted for by the altitude factor defined in Part 3.

Shallow topography
When changes in ground level are not gentle (ψ greater than 0.05) but the slopes remain below a critical slope of $\psi = 0.3$ (an angle of about 17°), the balance between the mean wind speed and the turbulence is disturbed. The mean wind speed undergoes significant changes in value but the turbulence undergoes small distortions without significantly changing the turbulence intensity. Gust speeds, which are formed by the action of the turbulence superimposed on the mean wind speed, are therefore affected less than the mean speed.

Fig 2 Categories of topography



Wind blowing across a shallow valley in an otherwise flat plain decelerates down the upwind slope to a minimum at the valley bottom, then accelerates up the downwind slope back to the initial speed. As wind blowing along the axis of a valley is not significantly changed, there is no advantage to cross-axis shelter. Accordingly, the effects of this form of valley are not included in these design rules. This form of 'rift' valley is rare, the typical valley in the UK being between hills or ridges.

Wind blowing over a shallow hill, ridge or escarpment accelerates up the upwind slope to a maximum at the summit or crest. The effect varies with height and is greatest near the ground. Downwind of the summit of a hill or ridge the wind decelerates, returning to the initial speed by about $2.5L$ downwind of the summit. Downwind of the crest of an escarpment, wind decelerates more slowly, converging towards a final speed appropriate to the change in altitude for gently topography. These effects have been studied in New Zealand, comparing results from wind tunnel models with full scale[1][2]. These and other data confirm the result from theory[3] that the change in mean wind speed over shallow topography is everywhere proportional to the upwind slope.

The design rules given below were derived from theory and the New Zealand data which apply to shallow topography. When comparisons were made between these rules and BS 8100 it was found that all the major hills in the UK used in the test calculations for the British Standard (and on which towers had been constructed) were within the shallow category (0.05 to 0.3). The majority of UK hills will therefore be in this category.

Steep topography

Where the upwind or downwind slope measured in the wind direction exceeds the critical angle of about 17°, the flow of wind separates from the ground surface leaving regions of separated flow. Such situations are rare in practical sites of construction in the UK. (Topography that is steep when the wind is normal may become shallow when the wind is skewed.)

Wind blowing across a steep valley separates from the upwind edge and jumps across to the downwind edge, leaving a large sheltered region of separated flow. It is not possible to give general design rules to cater for this effect. No shelter occurs when the wind blows directly along the axis of the valley, and it is recommended that no shelter is assumed for any wind direction when making wind loading assessments.

Wind blowing over a steep hill, ridge or escarpment separates from the ground surface ahead of the upstream slope and jumps to a point just below the summit or crest. The boundary of the separated region forms an effective slope equal to the critical value of 17°, so that the flow over the summit or crest is the same as an effective upwind slope $\psi = 0.3$, irrespective of the actual upwind slope ψ: this gives an upper limit to the possible acceleration. The length of the upwind separation region becomes the corresponding effective length of the upwind slope $L_E = 3.3Z$. The flow separates from the ground downwind of the crest or summit if the downwind slope is also steep. Again, it is not possible to give general design rules for the large regions of separated flow, but the design rules given later may be used as upper-bound values.

RANGE OF APPLICABILITY

The design rules for gentle topography are always applicable. The range of applicability of the design rules for shallow and steep topography is shown in Fig 3. The design rules apply accurately only in those cases shown without shading, but these do represent the majority of typical sites affected by topography. The design rules provide upper-bound values in those cases shown by dark shading, where in reality there will be reduced acceleration or even shelter. In the cases shown by light shading the site is either unaffected or sheltered and it is recommended that no topographic corrections be applied.

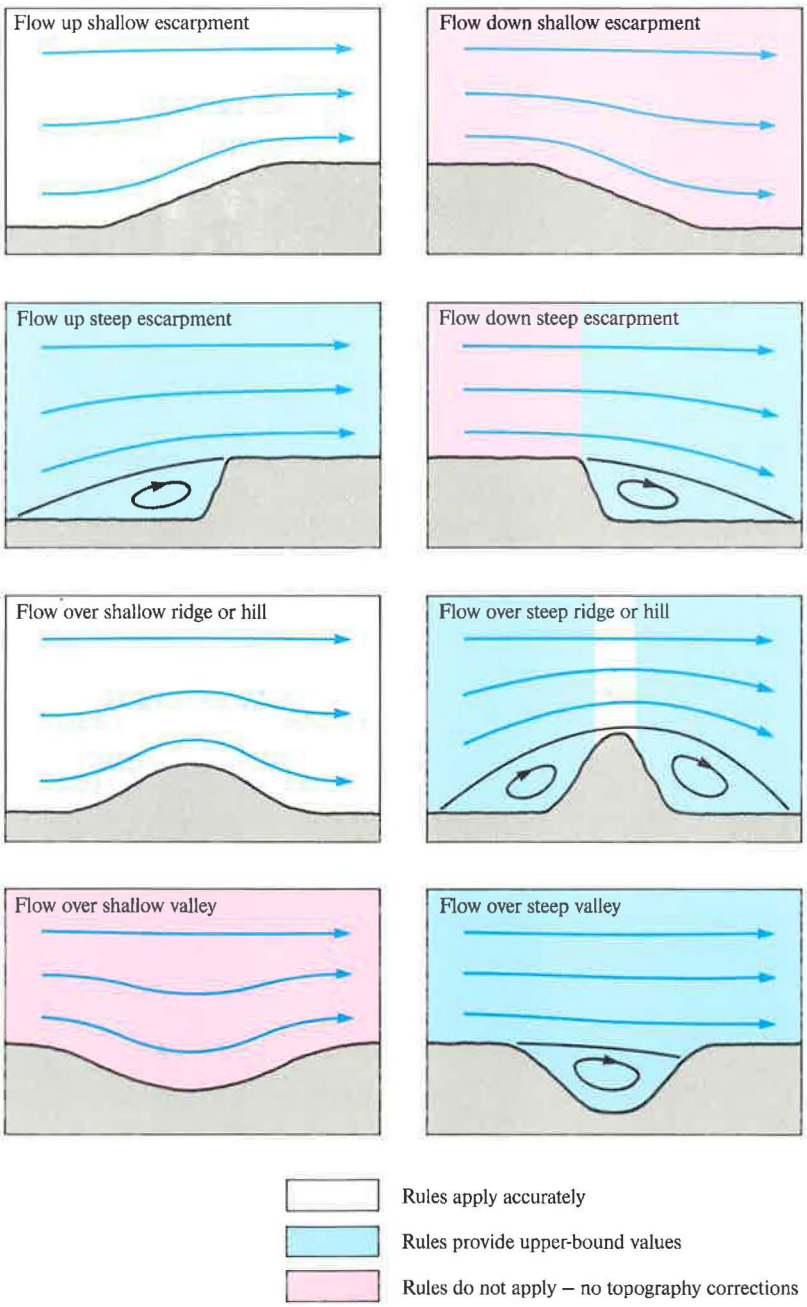


Fig 3 Range of applicability

DESIGN SPEED INCREMENT COEFFICIENT s

An important result of the theory[3] is that a good estimate of the accelerated mean wind speed near the ground at the summit or crest, and $\overline{V}_{crest}$ expressed as a ratio of the incident mean wind speed $\overline{V}$ is given by:

$$\frac{\overline{V}_{crest}}{\overline{V}} = 1 + 2\psi$$

where the topography is shallow and there is no flow separation.

As the increase in mean wind speed is everywhere proportional to the upwind slope ψ, the effect elsewhere can be quantified by a speed increment coefficient $s\{x/L, H/L\}$ (where the brackets denote functional dependence on the position from the crest x and the height above ground H in terms of the upwind slope length L). This coefficient takes values between $s = 0$ where the topography has no effect to $s = 1$ near the ground at the summit or crest. This approach extends to give upper-bound values for steep topography when the effective length L_E of the upwind slope is used in place of L.

Design values of the speed increment coefficient $s\{x/L_E, H/L_E\}$ are plotted against position x/L_E and height above ground H/L_E in terms of the effective upwind slope length in Fig 4.

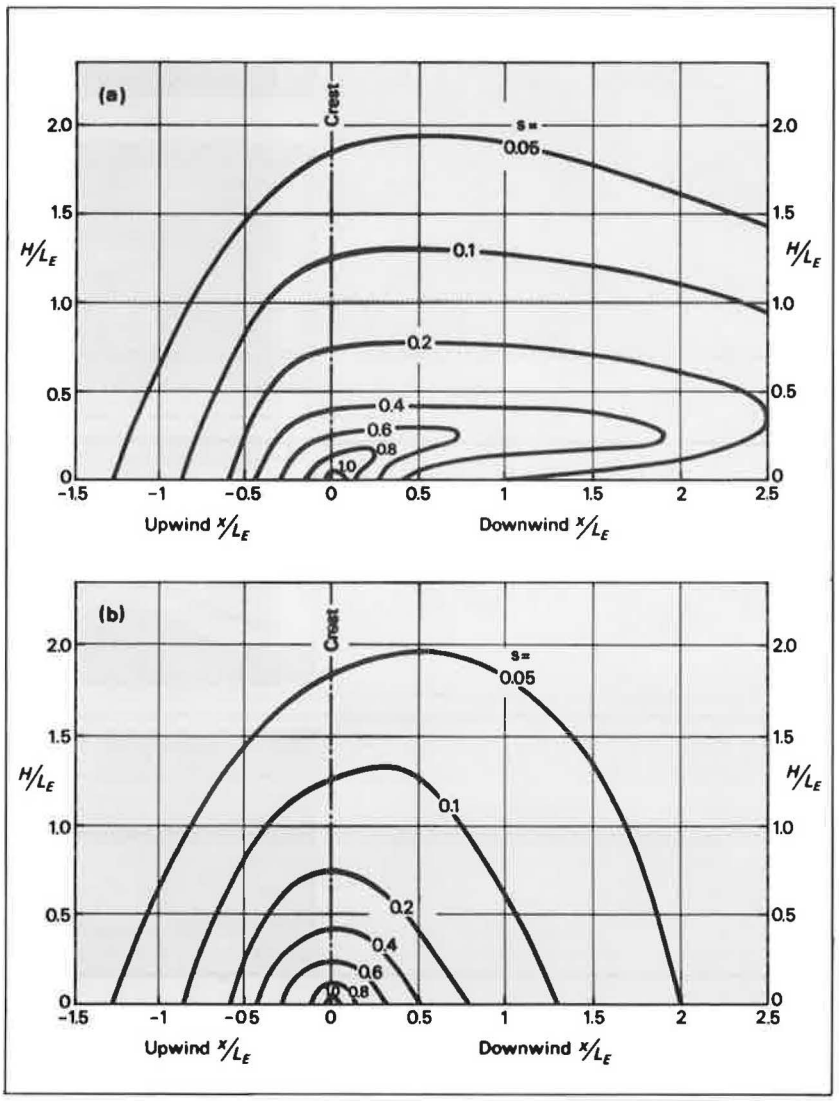


Fig 4 Factors for (a) Cliff and escarpment
(b) Ridge and hill

TOPOGRAPHY FACTOR S_{TOP}

For design purposes the effects of topography can be accounted for by the introduction of a topography factor, S_{TOP}, given by:

$S_{TOP} = 2\,s\,\psi$ for values of ψ up to 0.3

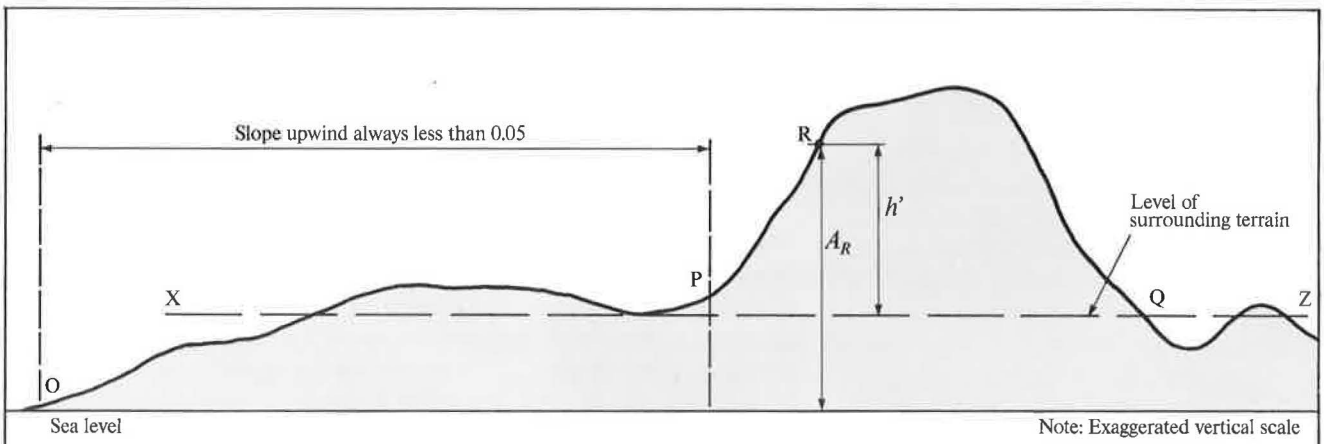
S_{TOP} represents the mean wind speed increment due to the topography and is incorporated directly in the equation of the derivation of the reference wind speed for the site V_{REF}, from the site hourly mean wind speed, $\overline{V}_{SITE}$, as shown in Part 4.

Assessment of topographic dimensions

The first stage in an assessment is to establish the relevant topographic dimensions as defined in Fig 1 for each relevant wind direction (steps of 30° are convenient). Determine from the upwind slope ψ whether the topography is gentle, shallow or steep. If gentle, the wind speeds derived for the site altitude should be used without any further correction. If shallow or steep, the wind speed already determined at the site altitude must be reduced to that at the altitude of the surrounding terrain. This is because the level of the site above mean sea level is already accounted for in the altitude factor. The topography factor, however, is based on wind speeds relative to the altitude of the general surrounding terrain. To ensure that the height correction is not applied twice, the wind speed defined for the site level must be reduced to that at the altitude of the surrounding terrain. This may be done by factoring the site wind speed by:
$(1 - 0.001h')$

where h' is the site height *above* the general level of the surrounding terrain.

This is shown diagrammatically in Fig 5.



For site in zone OP	wind speed derived on basis of altitude factor
For site in zone PQ	wind speed derived on basis of altitude factor and topography factor, as described below.
Example for site R:	$(\overline{V}_{SITE})_R$ derived on basis of altitude of site A_R this must be reduced to $(\overline{V}_{SITE})_{XZ}$ at level of surrounding terrain i.e. to level XZ
	Thus $(\overline{V}_{SITE})_{XZ} = (\overline{V}_{SITE})_R\,(1 - 0.001h')$
	where h' is the altitude of the site above the level of the surrounding terrain.

Fig 5 Adjustment for level of surrounding terrain

It is then necessary to select the appropriate values of upwind slope ψ and slope length L_E from Table 1.

Table 1 Effective parameters for shallow and steep topography

Upwind slope $\psi = (Z/L)$	
Shallow (ψ 0.05 to 0.3)	**Steep (ψ greater than 0.3)**
$L_E = L$	$L_E = Z/0.3$
$S_{TOP} = 2\,s\,\psi$	$S_{TOP} = 0.6$

Calculate the position x of the site relative to the crest or summit and the required height or heights above the ground H as ratios of L_E. If the position is in the range -1.5 to 2.5, the site is influenced by the topography and the assessment should proceed; otherwise the site is not influenced by the topography (and the Topography factor $S_{TOP} = 0.0$).

For each wind direction, select the values of speed increment coefficient $s\{x/L_E, H/L_E\}$ from Fig 4, appropriate to the position, height above ground and shape for that direction.

The topography factor S_{TOP} can then be obtained from Table 1 dependent on the parameters ψ, L_E and s.

ASSESSMENT OF MEAN WIND SPEEDS

Mean wind speeds are required as the basis for the assessment of wind loads on both static and dynamic structures (Parts 1 to 4) and for other purposes such as the assessment of natural ventilation (Digest 210). Equation (1), (2) and (3) of Part 4 may be used for this purpose taking $g_{GUST} = 0.0$ for hourly mean speeds.

For sites in country terrain $\overline{V}_{REF} = S_{SC}\,(1 + S_{TOP})\,\overline{V}_{SITE}$

For sites in town terrain $\overline{V}_{REF} = S_{SC}\,S_{CT}\,(1 + S_{TOP})\,\overline{V}_{SITE}$

where $\overline{V}_{REF}$ is the hourly mean wind speed appropriate to the building height, and the other S factors are defined in Part 4.

DESIGN WIND SPEED AND DYNAMIC PRESSURE

For the assessment of wind loads on static and mildly dynamic structures, the reference wind speed, V_{REF}, and the resulting pressure, p, should be assessed for the site in accordance with the principles given in Part 1. Values of the topography factor S_{TOP} are provided by this Digest. The other S factors given in the formula below are described in Parts 1 to 4.

The reference wind speed V_{REF} for country terrain is then calculated from the formula:

$$\overline{V}_{REF} = V_{SITE}\,S_{SC}\,[1 + (g_{GUST}\,S_{TSC}) + S_{TOP}]$$

and the design dynamic pressure from the formula:

$$q = kV^2_{REF}$$

where $k = 0.613$ kg/m^3 as described in Part 1.

REFERENCES

1 **BOWEN, A J.** Some effects of escarpments on the atmospheric boundary layer. (PhD Thesis) Christchurch, New Zealand, University of Canterbury, Department of Mechanical Engineering, June 1979.

2 **PEARSE, J R.** The influence of two-dimensional hills on simulated atmospheric boundary layers. (PhD Thesis) Christchurch, New Zealand University of Canterbury, Department of Mechanical Engineering, August 1979.

3 **JACKSON, P S and HUNT, J C R.** Turbulent wind flow over a low hill. Quart. J. R. Met. Soc 101, 929-955.

4 **ESDU.** Strong winds in the atmospheric boundary layer: Part 1: mean-hourly wind speeds. Data item B2026. London, ESDU International 1982. 27 Corsham Street, London N1.

FURTHER READING

Building Research Establishment

Digest 210 Principles of natural ventilation.

British Standards Institution

BS 8100: Lattice towers and masts
Part 1: 1986 Code of practice for loading
Part 2: 1986 Guide to the background and use of Part 1 'Code of practice for loading'

See also Further reading *in Part 1*

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The assessment of wind loads

Part 6: Loading coefficients for typical buildings

This is the sixth in a series of Digests which is compatible with the proposed British Standard BS 6399: Part 2. It provides data on pressure coefficients for the walls and roofs of bluff-shaped buildings to enable loads to be derived. The procedure outlined in the Digest is limited to rectangular buildings with flat, gabled or hipped roofs but it is applicable in principle to buildings of more complex shape. The assessment of pressure coefficients for such buildings is provided in Part 2 of *The designer's guide to wind loading of building structures.*

When assessing wind loads on bluff-shaped buildings it is necessary to provide pressure distributions over the various surfaces so that loads can be derived, both for small elements, such as windows and cladding, as well as on whole faces; from this data, overall loads can be determined.

Parts 1 to 4 of this Digest outline a means of deriving dynamic pressures for the appropriate gust duration; these are used in conjunction with the pressure coefficients to determine the loads.

The design gust duration is determined by the size of the loaded area using equation (6) in Part 4.

INFLUENCE OF SLENDERNESS RATIO

The height or, more specifically, the slenderness ratio of a building affects the flow pattern of the wind around or over it.

If the height of the building is greater than about half the crosswind width, the wind tends to flow around the sides rather than over the top, except for the zone very near the top. The critical dimension, is therefore, the width of the building.

If the building is low, that is when the height is less than about half the crosswind width, the wind tends to flow over the top rather than the sides, except for zones close to the ends of the building. The critical dimension is the height of the building.

INFLUENCE OF WIND DIRECTION

The flow conditions are different between wind normal to a face and wind skew to a face. Figures 1 and 3 of Part 1 show that the zones of high suction differ in the two cases. When the wind direction factor S_{DIR} (see Part 3) is taken into account, the overall moments and shears will depend on the orientation of the structure and may not be greatest when the flow is normal.

REFERENCE DYNAMIC PRESSURE

The choice of reference height z_{ref} for the reference dynamic pressure, q, is made to minimise the variation in loading coefficient values over structures of the same shape and form but of different size.

In the case of line-like structures or lattice structures, in which the divergence of the wind flow past the structure is relatively small, the reference pressure should be calculated at the local level. With bluff structures the flow is diverted considerably, so the local dynamic pressure does not provide a set of coefficients which are invariant with the size of the building. Unfortunately, any one fixed reference height will not provide a constant 'universal' coefficient and a compromise has to be sought. In this Digest, the pressure coefficients for bluff structures whose height H is less than $4L$ are related to the reference dynamic pressure as calculated at the top of the building. Such coefficients tend to give vertical zones of constant value which enables local regions to be defined as vertical strips.

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PRESSURE COEFFICIENTS

Vertical walls of rectangular buildings

On the windward face, the slenderness ratio affects the flow pattern as already described. With a slender building, the pressure contours are predominantly vertical and they scale to the crosswind breadth B in Fig 1. As buildings become more squat, the central region of relatively low pressure coefficients expands and the contours move towards the end of the face, and their distance from the edges scales to twice the height $2H$. This enables coefficients to be defined in zones dimensioned in terms of the smaller of the height or face width.

For the side face, the slenderness ratio affects the size of the local high suction region at the upwind end of the face. The width of that region will still depend on the slenderness of the corresponding upwind face, that is still to the crosswind breadth B or twice height $2H$. Loaded regions can, therefore, still be defined as vertical strips for design purposes, dimensioned in terms of the smaller of the height or the upwind face length.

For the rear leeward face the wall experiences a fairly uniform suction throughout.

The peak cladding loads experience similar effects but are modified by atmospheric and building-generated turbulence; the effects of these generally increase the high-load regions.

Pressure coefficients for walls are required for different zones of loaded areas, depending on the proportions of the building and the wind direction. The definitions of dimensions are given in Fig. 1.

The loaded zones, **A** to **D**, are defined in Fig 2 as vertical strips in terms of b where:

$b = B$ or $2H$ whichever is the smaller

Zone **A** is always at the upwind edge of the face (see Fig 1). Depending on the proportion of the face not all the zones will exist or be their full defined size.

For walls, the wind angle θ is defined from normal to the wall being considered. For a rectangular building with the flow normal to the windward wall $\theta = 0°$, for the leeward wall $\theta = 180°$ and for both side walls $\theta = 90°$.

The loaded areas should then be determined as follows:

- Determine the crosswind breadth, B, for the relevant wind direction.
- Determine the height, H, to the top of the wall. The reference dynamic pressure, q, should be determined at this height.
- Calculate b.
- Define zone **A** with width $b/5$ starting from the upwind edge of the face.
- Define zone **B**, extending from $0.2b$ up to b away from the upwind edge of the face. For tall buildings when $H \geq B/2$, zones **A** and **B** occupy the whole face and the procedure is complete. If not:
- Define zone **D** from the downwind edge with width b. If there is insufficient room for the defined size zone **D** occupies the remainder of the face after zones **A** and **B** have been defined.
- Define zone **C** as the remainder of the face having fully defined **A**, **B** and **D**.

In the special case of wind exactly normal to the face, both edges of that face should be treated as upwind edges and zones **A** and **B** are defined away from both these edges. Zone **D** does not appear.

For leeward faces the same convention applies of taking zone **A** from the *upwind* corner of the leeward face (see Fig 1).

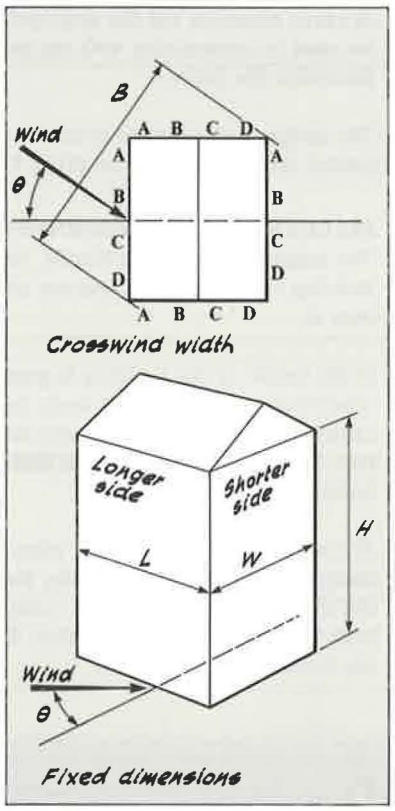
Pressure coefficients for each of these zones are given in Table 1.

Table 1 Pressure coefficients for vertical walls

	$H/B \leq 0.5$				$H/B = 1$		$H/B = 2$		$H/B \geq 4$	
Zone	**A**	**B**	**C**	**D**	**A**	**B**	**A**	**B**	**A**	**B**
PRESSURE COEFFICIENTS										
θ = 0°	0.66	0.83	0.86	0.83	0.78	0.89	0.67	0.80	0.62	0.76
30°	0.80	0.80	0.71	0.49	0.88	0.80	0.83	0.71	0.80	0.68
60°	0.24	0.51	0.40	0.26	−0.85	0.23	0.31	0.31	0.20	0.25
90°	−0.91	−0.68	−0.42	−0.12	−1.21	−0.92	−1.02	−0.82	−1.09	−0.94
120°	−0.63	−0.63	−0.46	−0.29	−0.64	−0.64	−0.54	−0.54	−0.67	−0.67
150°	−0.34	−0.34	−0.26	−0.32	−0.54	−0.54	−0.51	−0.51	−0.51	−0.51
180°	−0.34	−0.34	−0.22	−0.34	−0.24	−0.24	−0.40	−0.40	−0.54	−0.54
PEAK SUCTIONS										
90°	−1.21				−1.27		−1.29		−1.32	

Interpolation may be used. When the result of interpolating between positive and negative values is in the range ± 0.2, the coefficient should be taken as equal to ± 0.2 and both possible values used.

Fig 1 Definitions of dimensions



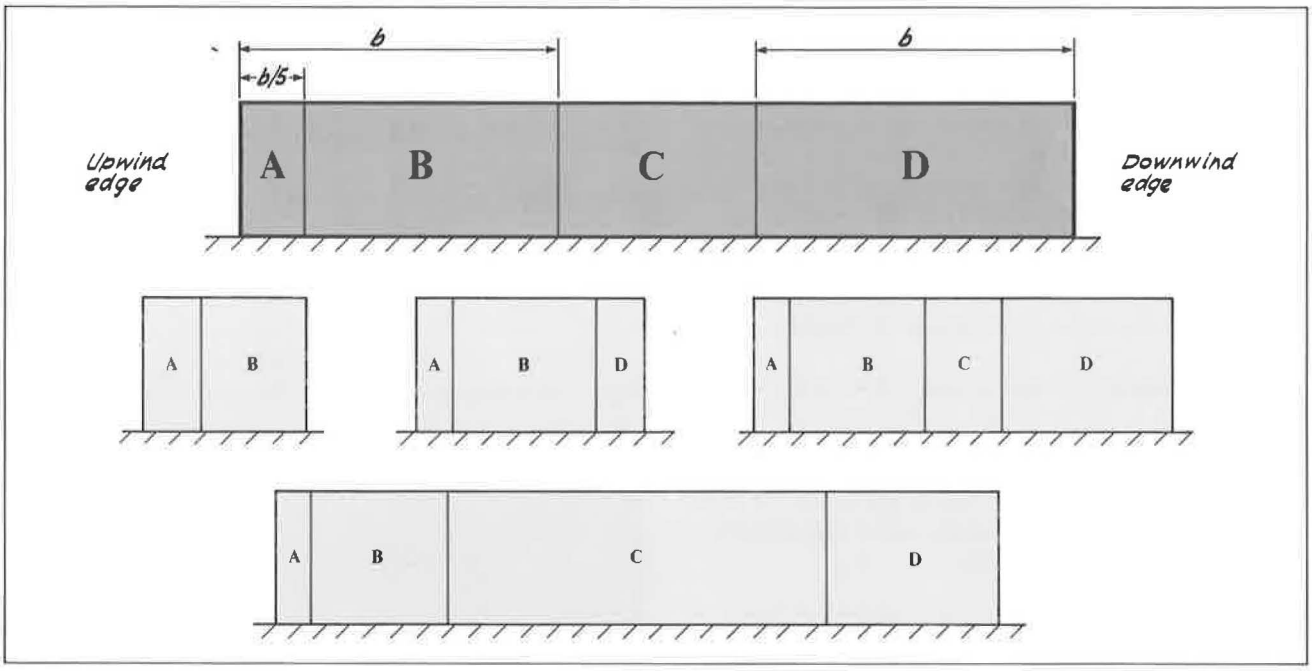


Fig 2 Key to wall pressure data

Roofs

The flow condition over flat roofs with wind normal to a face is similar to flow around the side wall: flow separates at the upwind edge and may re-attach at some distance downwind to form a separation bubble. The size of the separation bubble scales in the same manner to the smaller of the crosswind breadth, B, or twice the height, $2H$, as for walls. However, the corners of vertical walls remain normal to the flow for all wind angles and the separation bubbles form as cylindrical vortices. For roofs this occurs only for wind normal to the eave, at all other angles conical vortices form from the upwind corner.

The flow over monopitch roofs, that is roofs formed by one plane face at a pitch angle, is dependent on that angle. As the angle of pitch, α, increases the vortices formed at the upwind edges decrease in strength and size while the overall pressure rises, such that when $\alpha = 30°$ the vortices have disappeared and the overall pressure becomes positive. At 45° pitch angle, the coefficients exceed unity when the reference pressure is taken at the eave as the wind speed at the high downwind eave is in excess of the chosen reference value. When α is negative, the overall pressure starts to fall; when α reaches $-30°$ the pressure distribution is nearly uniform, with an overall uplift.

Duopitch roofs are formed by two plane faces joined along a common edge to form either a high ridge (positive α) typical of most houses, or a low trough (negative α). The upwind face behaves similarly to a monopitch roof; the downwind face is influenced significantly by the upwind face but is less onerously loaded. Little data are available on unequal pitch duopitch roofs and specialist advice should be sought.

Conventional hipped roofs are formed from duopitch roofs by replacing the gable ends with triangular pitched roofs or 'hips'. Vortices form along each of the ridges which might suggest that loadings on such roofs are more severe. This is not the case because the verge vortices are less severe than for a duopitch roof resulting in the loading on hipped roofs being much less severe.

Definitions

The various forms and parts of roofs are defined as follows — see Fig 3.

Eave — the horizontal edge of the roof — taken as the longer edges for flat roofs and hipped roofs

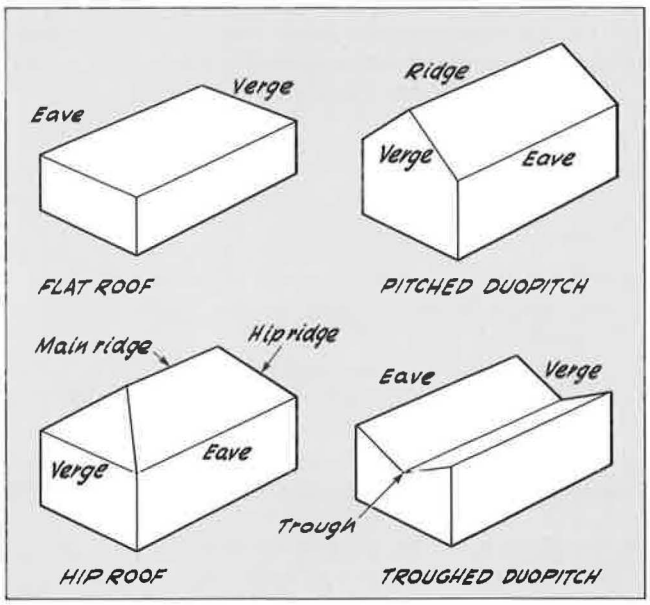
Verge — the non-horizontal edge of the roof, such as the gable edge of monopitch and duopitch roofs — taken as the shorter edges for flat roofs and hipped roofs

Hip — triangular pitched face at each end of the main faces of a hipped roof

Ridge — the highest horizontal line formed where the two faces of a duopitch roof meet. On hipped roofs, the horizontal line is the main ridge; this distinguishes it from the hip ridges which are at the junction of the main face and hip faces

Trough — the lowest horizontal line formed where the two faces of troughed duopitch roofs meet.

Fig 3 Definitions of roof types and parts



FLAT ROOFS — *below 5° pitch*

The roof should be subdivided into zones behind each upwind eave/verge. The general notation and definition of the loaded areas is given in Fig 4; this shows the zones on a rectangular roof when edges 1 and 2 are both upwind.

E_1 and E_2 are the lengths of the upwind eaves/verges measured from upwind external corner to downwind external corners.

H is the height to the eaves. The reference dynamic pressure, q, should be determined at this height.

θ_n is the wind angle expressed as a local wind angle from normal to the eave/verge, n, ie θ_1 and θ_2 in Fig 4.

Loaded zones

Zones of constant pressure coefficients are defined in strips parallel to the eave/verge. These are each further divided downwind from the upwind corner.

The zones at the upwind corner are divided from those along the adjacent eave/verge by the line through the corner in the direction of the wind. This allows zones to be defined for any corner angle.

Once the zones have been defined behind each upwind eave/verge, pressure coefficients for all zones over the whole roof can be obtained.

Pressure coefficients for flat roofs with sharp eaves

Pressure coefficients for each zone for flat roofs with sharp eaves are given in Table 2 in terms of the local wind angle each side from normal to the eaves. These coefficients apply both to pressure for overall loading and for cladding loads.

Table 2 Pressure coefficients for flat roofs with sharp eaves

Local wind direction θ_n	Zone A	B	C	D	E	F	G
0	−1.47	−1.25	−1.15	−1.15	−0.69	−0.71	±0.20
± 30	−2.00	−1.70	−1.38	−1.03	−0.66	−0.67	±0.20
± 60	−1.70	−1.24	−1.10	−0.64	−0.61	−0.42	±0.20
± 90	−1.20	−0.75	−0.52	−0.24	−0.62	±0.20	±0.20

Interpolation may be used. Where both positive and negative values are given both values should be considered.

Curved or chamfered eaves and parapets generally give lower values; see *The designer's guide to wind loading of building structures Part 2.*

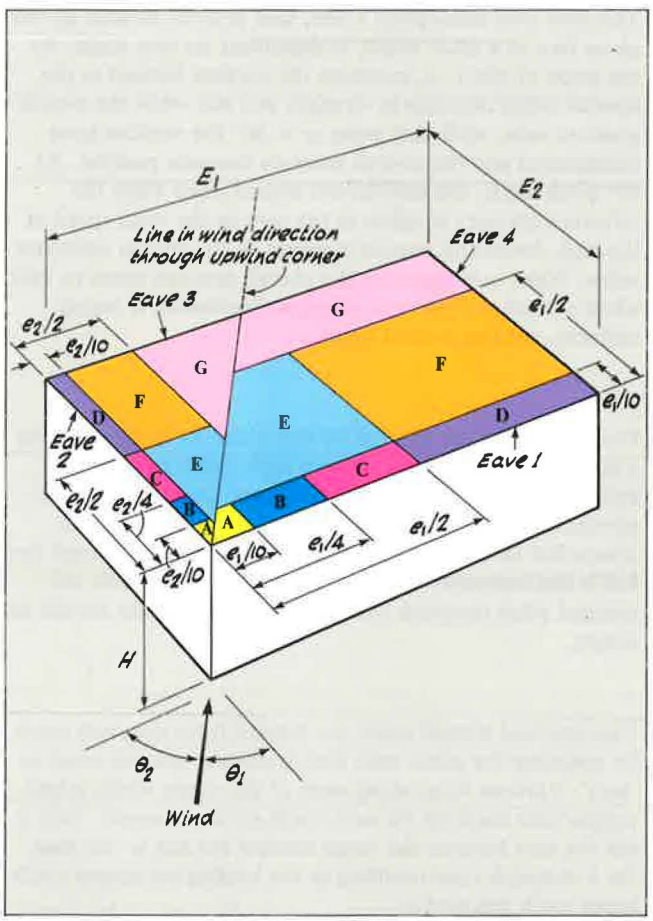
Extent of zones

The extent of the zones and their types should be determined as follows:

- Determine the length E_n of the eave being considered
- Determine the height H_n of the corresponding wall (ie to the ground for simple cuboidal buildings)
- Calculate e as:
  $e = E$ or $2H$ whichever is the smaller
- Draw the boundary line through the upwind corner in the wind direction
- Mark out the depth of the edge zones parallel to the eave $e/10$ behind the eave and define zones **A** to **D** such that:
  Zone **A** extends $e/10$ from upwind corner
  Zone **B** extends from $e/4$ from upwind corner
  Zone **C** extends from $e/4$ to $e/2$ from upwind corner
  Zone **D** extends from $e/2$ to downwind corner

  When the wind is exactly normal to an eave/verge define the zones inwards from both corners.
- Mark out the depth of the central region parallel to the eave from $e/10$ to $e/2$ behind the eave. Define zones **E** and **F** such that:
  Zone **E** extends $e/2$ from upwind corner
  Zone **F** extends from $e/2$ from upwind corner to the downwind corner
- All the remainder of the roof downwind of zones **E** and **F** is zone **G**
- Repeat the above for the adjacent upwind eave (2 for the case considered in Fig 4).

Fig 4 Key for flat roof pressure data



MONOPITCH AND DUOPITCH ROOFS

Figures 5 and 6 show the notation and zones for monopitch and duopitch roofs, in which:

L is the length of the upwind eaves

W is the width of the upwind verge. For duopitch roofs W is the total width of both verges (see Fig 6).

α is the pitch angle of the roof defined from normal to the upwind eave. For monopitch roofs α is taken as positive with the low eave upwind and negative with the high eave upwind.

For duopitch roofs α is taken as positive when the roof has a central ridge and negative when the roof has a central trough.

H is the height to the upwind eave for monopitch roofs and to the upwind eave for each face of duopitch roofs (see Fig 6).

θ is the wind angle from normal to the horizontal eave or ridge.

The loaded zones, **A** to **J**, are defined in Fig 5 as strips parallel to the eave and verge in terms of the widths e and v where:

$e = L$ or $2H$ whichever is the smaller
$v = W$ or $2H$ whichever is the smaller

The reference dynamic pressure, q, is taken at the effective height of each face appropriate to the height of the upwind corner (see Fig 6). Conservatively a single value appropriate to the highest point of the roof could be used.

Fig 5 Key for monopitch and duopitch roofs

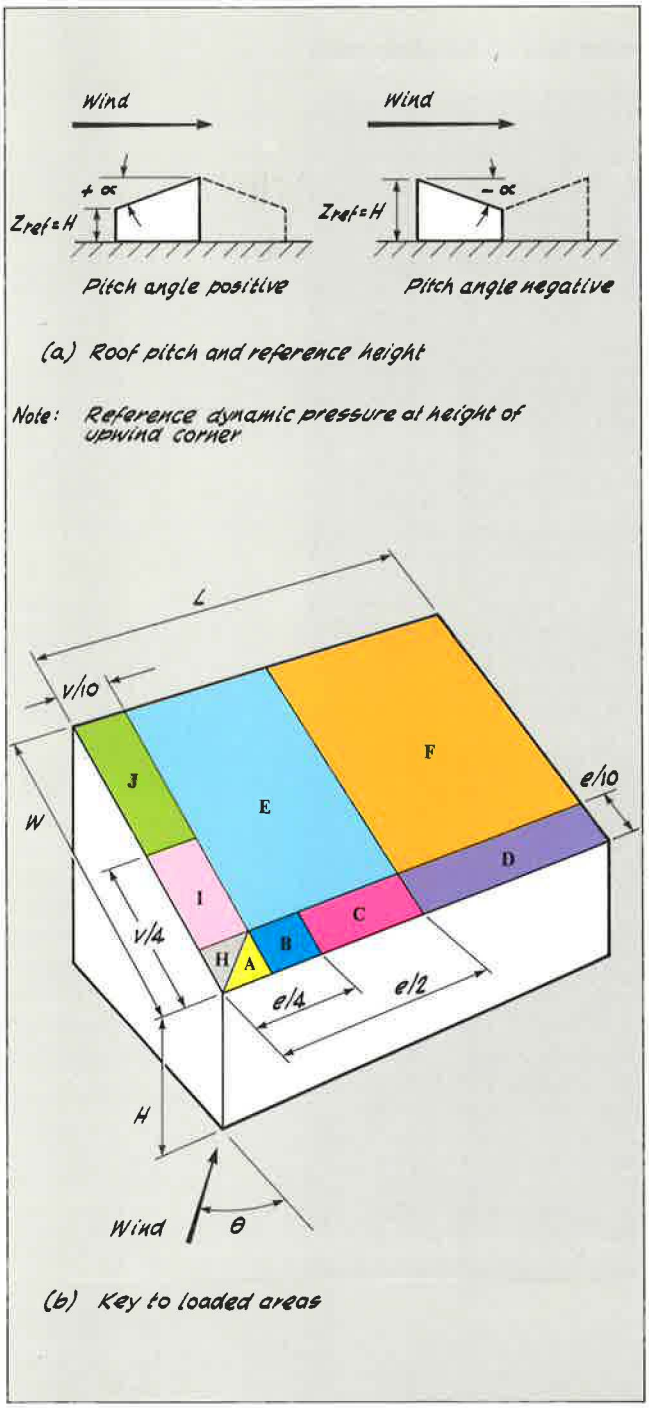
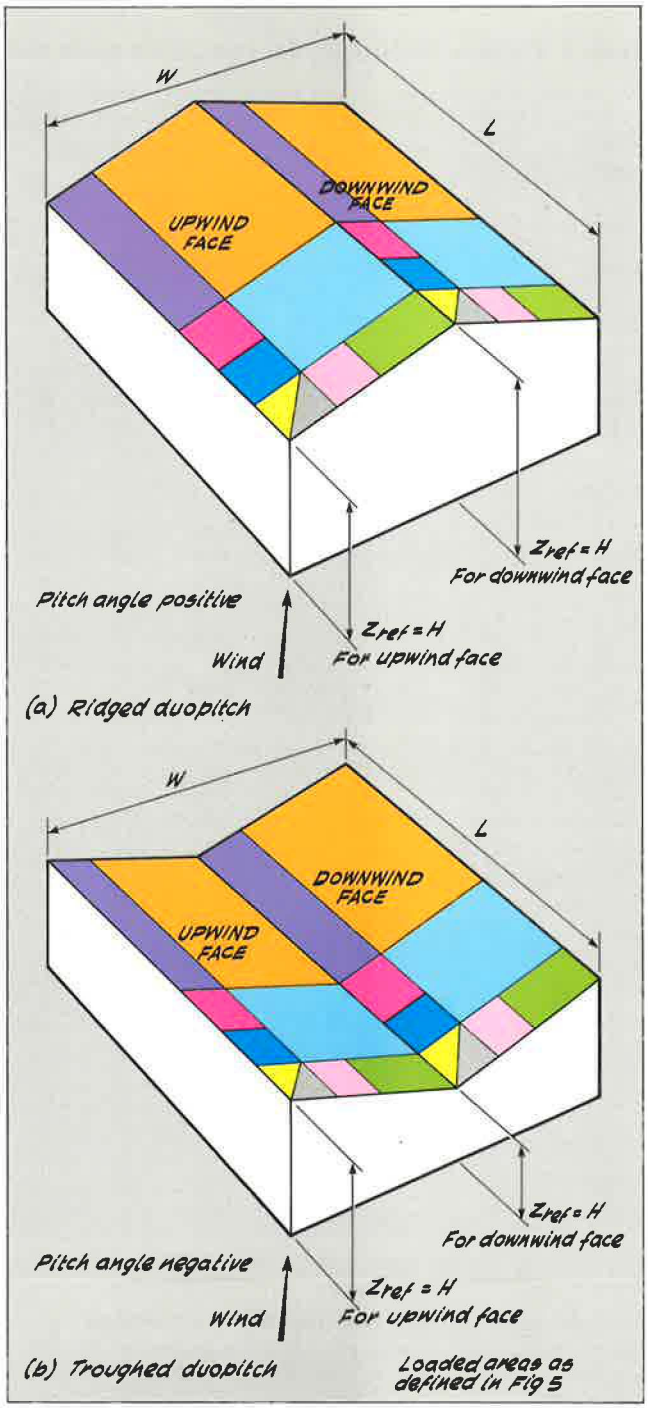


Fig 6 Key for duopitch roofs



Loaded zones

The zones, over which the pressure coefficients are taken as constant, are defined from the upwind corner of each face, and should be determined from Fig 5 appropriate to the values of e and v. These zones are constant for all wind directions for which the corner is upwind.

Monopitch roofs

Pressure coefficients for each of the zones are given in Table 3.

Table 3 Pressure coefficients for monopitch roofs and upwind face of duopitch roofs

Pitch angle α	Wind direction	Zone A	B	C	D	E	F	H	I	J
−45°	0°	−0.61	−0.58	−0.56	−0.41	−0.76	−0.78	−0.62	−0.79	−0.94
	30	−0.53	−0.50	−0.49	−0.55	−0.55	−0.81	−0.52	−0.58	−0.58
	60	−1.11	−1.29	−1.36	−0.96	−0.97	−0.91	−1.05	−0.97	−1.17
	90	−1.25	−0.81	−0.62	−0.42	−0.77	±0.20	−1.48	−1.05	−0.97
−30°	0°	−0.76	−0.68	−0.60	−0.50	−0.76	−0.63	−0.76	−0.85	−0.90
	30	−1.13	−1.02	−1.89	−0.79	−0.84	−0.76	−1.17	−0.87	−0.73
	60	−2.06	−2.33	−2.17	−1.22	−1.03	−0.80	−1.69	−1.18	−1.21
	90	−1.28	−0.94	−0.70	−0.37	−0.70	±0.20	−1.54	−1.10	−1.01
−15°	0°	−1.08	−1.05	−0.97	−0.92	−0.88	−0.82	−1.10	−0.96	−0.83
	30	−2.64	2.37	1.71	1.00	0.93	−0.85	−2.75	−1.66	−1.11
	60	−2.25	−2.15	−1.85	−1.02	−0.76	−0.72	−2.44	−1.60	−1.07
	90	−1.22	−0.79	−0.58	−0.31	−0.60	±0.20	−1.51	−1.12	−1.13
−5°	0°	−1.49	−1.34	−1.19	−1.12	−0.83	−0.82	−1.47	−0.91	−0.67
	30	−2.36	−2.21	−1.63	−1.04	−0.82	−0.77	−2.24	−1.30	−0.91
	60	−1.85	−1.57	−1.28	−0.77	−0.65	−0.54	−2.10	−1.67	−1.09
	90	−1.30	−0.79	−0.58	−0.27	−0.59	±0.20	−1.65	−1.13	−1.20
+5°	0°	−1.39	−1.24	−1.11	−1.19	−0.56	−0.59	−1.39	−0.69	−0.43
	30	−1.78	−1.64	−1.34	−1.09	−0.62	−0.60	−1.75	−1.02	−0.76
	60	−1.67	−1.33	−1.12	−0.71	−0.64	−0.42	−2.05	−1.51	−1.05
	90	−1.21	−0.83	−0.55	−0.25	−0.61	±0.20	−1.48	−1.12	−1.30
+15°	0°	−0.91	−0.83	−0.78	−0.81	−0.21	−0.31	−0.90	−0.36	−0.30
	30	−0.84	−0.88	−0.82	−0.83	−0.21	−0.37	−0.63	±0.20	−0.32
	60	−1.27	−0.86	−0.70	−0.61	−0.54	−0.33	−1.57	−1.21	−0.93
	90	−1.20	−0.84	−0.58	−0.27	−0.64	±0.20	−1.42	−1.10	−1.30
+30°	0°	0.38	0.69	0.79	0.77	0.39	0.40	±0.20	±0.20	±0.20
	30	0.75	0.74	0.22	0.59	0.41	0.26	0.78	0.69	0.47
	60	−0.14	0.43	0.39	0.33	±0.20	±0.20	−0.80	−0.89	−0.83
	90	−1.13	−0.94	−0.77	−0.19	−0.78	±0.20	−1.25	−1.06	−1.36
+45°	0°	0.82	1.02	1.11	1.13	0.75	0.74	0.69	0.56	0.43
	30	1.11	1.09	1.03	0.88	0.77	0.55	1.12	1.00	0.85
	60	0.79	0.69	0.62	0.46	0.38	0.21	0.84	0.82	0.54
	90	−1.17	−0.96	−0.86	−0.33	−0.88	−0.28	−1.25	−1.08	−1.36

Interpolation may be used. When the result of interpolating between positive and negative values is in the range of ± 0.2, the coefficient should be taken as equal to ± 0.2 and both positive values used.

Duopitch roofs
Pressure coefficients for each zone for the upwind face are given in Table 3. Pressure coefficients for each zone of the downwind face can be obtained from Table 4.

These coefficients are appropriate to duopitch faces of equal pitch but may be used without modification provided the upwind and downwind pitch angles are within ± 5° of each other. For duopitch roofs of greater difference in pitch angles, see *The designer's guide to wind loading of building structures Part 2.*

Table 4 Pressure coefficients for the downwind face of duopitch roofs

Pitch angle α	Wind direction	Zone								
		A	B	C	D	E	F	H	I	J
−45°	0°		−0.92		−0.75	−0.75	−0.75		−0.63	
	30		−1.12		−0.52	±0.20	−0.52		−0.32	
	60		−1.04		−0.24	−0.73	−0.24		−1.05	
	90	−1.17	−0.96	−0.86	−0.33	−0.88	−0.28	−1.25	−1.08	−1.36
−30°	0°		−0.78		−0.66	−0.66	−0.47		−0.40	
	30		−0.44		−0.52	−0.52	±0.20		±0.20	
	60		−0.74		−0.27	−0.27	−0.62		−1.01	
	90	−1.13	−0.94	−0.77	−0.19	−0.78	±0.20	−1.25	−1.06	−1.36
−15°	0°		−0.69		−0.52	−0.52	−0.26		−0.21	
	30		±0.20		±0.20	±0.20	±0.20		−0.55	
	60		−0.67		±0.20	±0.20	−0.65		−1.03	
	90	−1.20	−0.84	−0.58	−0.27	−0.64	±0.20	−1.42	−1.10	−1.30
−5°	0°		−0.34		−0.25	−0.25	−0.25		−0.28	
	30		±0.20		±0.20	±0.20	−0.26		−0.48	
	60		−0.69		±0.20	±0.20	−0.66		−0.88	
	90	−1.21	−0.83	−0.55	−0.25	−0.61	±0.20	−1.48	−1.12	−1.30
+5°	0°	−0.32	−0.27	−0.28	−0.28	±0.20	±0.20	−0.36	−0.30	−0.24
	30	−0.70	−0.46	−0.30	−0.23	−0.31	±0.20	−0.71	−0.59	−0.46
	60	−1.04	−0.90	−0.52	±0.20	−0.56	±0.20	−0.97	−0.83	−0.73
	90	−0.90	−0.83	−0.58	±0.20	−0.60	±0.20	−0.89	−0.89	−1.09
+15°	0°	−0.83	−0.81	−0.80	−0.78	−0.39	−0.40	−0.85	−0.55	−0.39
	30	−1.32	−1.14	−1.11	−0.88	−0.46	−0.34	−1.47	1.25	−0.81
	60	−1.31	−0.92	−0.72	−0.58	−0.57	−0.23	−1.45	−1.08	−0.75
	90	−0.81	−0.74	−0.54	±0.20	−0.58	±0.20	−0.83	−0.77	−0.92
+30°	0°	−0.29	−0.26	−0.25	−0.30	−0.30	−0.30	−0.31	−0.32	−0.33
	30	−0.74	−0.63	−0.52	−0.43	−0.39	−0.43	−0.76	−0.51	−0.40
	60	−1.04	−1.05	−0.98	−0.64	−0.58	−0.47	−1.02	−0.67	−0.64
	90	−0.66	−0.61	−0.49	−0.21	−0.49	±0.20	−0.67	−0.58	−0.69
+45°	0°	−0.21	−0.21	−0.21	−0.20	−0.23	−0.23	−0.21	−0.24	−0.26
	30	−0.21	−0.20	−0.20	−0.27	−0.23	−0.26	−0.20	−0.21	−0.22
	60	−0.54	−0.54	−0.51	−0.41	−0.44	−0.38	−0.55	−0.47	−0.50
	90	−0.55	−0.46	−0.38	−0.20	−0.40	±0.20	−0.60	−0.45	−0.47

Interpolation may be used.

Hipped roofs

The following provisions apply to conventional hipped roofs on cuboidal-plan buildings, where all faces of the roof have the same pitch angle in the range $5° < \alpha < 45°$.

The data are valid both for the trapezoidal main faces and the triangular hip faces, with the wind angle, θ_n, expressed as a local wind angle from normal to the eave/verge, ie θ_1 and θ_2 in Fig 7.

Loaded zones

Loaded zones, over which the pressure coefficients may be taken to be constant, are determined from Fig 7 in which:

L is the length of the main face eave

W is the length of the hip face eave

H is the height of the ridge.

For the main face, e should be taken as $2H$ or L whichever is the smaller.

For the hip face, e should be taken as $e = 2H$ or W whichever is the smaller.

θ_n is the local wind direction normal to the eave/verge of the face.

The reference dynamic pressure, q, should be determined at height H, the height of the ridge, or, for those zones immediately adjacent to the eaves, and zones **E** and **F** on the upwind faces, it may be taken as the height of the eaves.

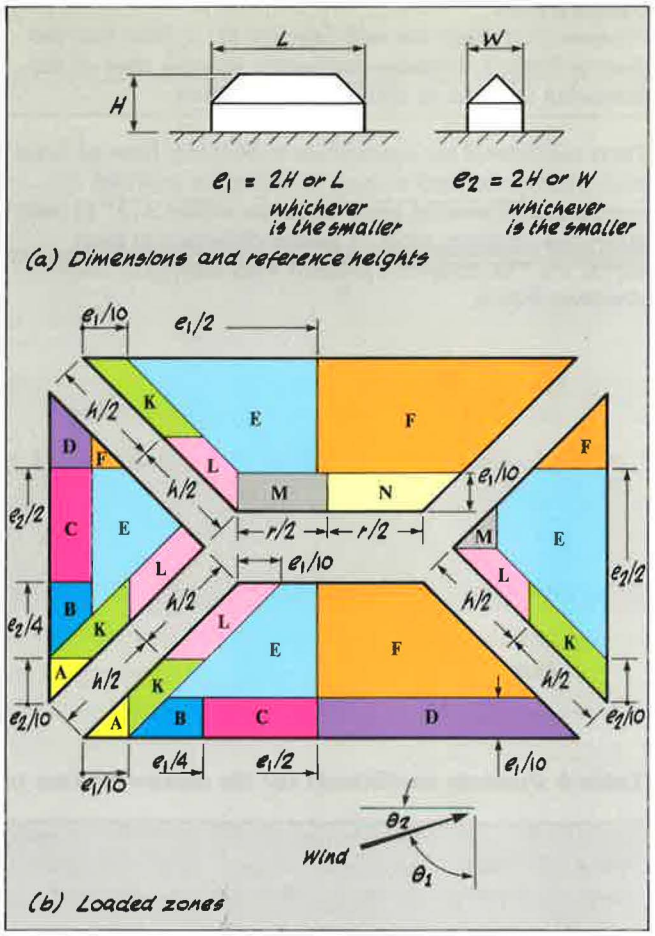


Fig 7 Key for hipped roofs

Pressure coefficients

The pressure coefficients for the upwind faces for zones **A** to **F** may be obtained for the corresponding pitch angle from Table 3 (for positive values of α only). Pressure coefficients for zones **K** and **L** are given in Table 5.

The pressure coefficients for the downwind faces for zones **E** and **F** may be obtained for the corresponding pitch angle from Table 4 (for positive values of α only). Pressure coefficients for zones **K** to **N** are given in Table 5.

Table 5 Pressure coefficients for hipped roofs

Pitch angle α	Wind direction	Upwind face zone K	Upwind face zone L	Downwind face zone K	Downwind face zone L	Downwind face zone M	Downwind face zone N
5°	0°	−0.56	−0.56	−0.31	−0.45	−0.58	−0.58
	30	−0.62	−0.62	−0.60	−0.46	−0.47	−0.54
	60	−1.13	−0.63	−0.76	−0.51	−0.38	−0.36
	90	−1.19	−0.76	−0.89	−0.50	−0.61	±0.20
15°	0°	−0.31	−0.31	−0.44	−0.83	−1.17	−1.17
	30	−0.37	−0.37	−1.00	−0.99	−1.31	−1.13
	60	−0.94	−0.52	−1.43	−0.71	−0.78	−0.80
	90	−1.09	−0.77	−0.97	−0.59	−0.64	±0.20
30°	0°	0.40	0.40	−0.53	−0.33	−0.28	−0.28
	30	0.26	0.26	−0.74	−0.55	−0.51	−0.50
	60	−0.99	−0.47	−1.25	−0.82	−0.77	−0.49
	90	−1.10	−1.01	−1.40	−0.62	−0.78	±0.20
45°	0°	0.74	0.74	−0.65	−0.24	−0.20	−0.20
	30	0.55	0.55	−0.52	−0.22	−0.22	−0.28
	60	−1.11	−0.33	−0.67	−0.35	−0.32	−0.41
	90	−1.22	−0.71	−1.35	−0.43	−0.88	−0.28

Interpolation may be used. When the result of interpolating between positive and negative values is in the range ± 0.20 the coefficient should be taken as equal to ± 0.20 and both possible values used.

For further reading see Part 1

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The assessment of wind loads

Part 7: Wind speeds for serviceability and fatigue assessments

This is the seventh in a series of Digests which is compatible with the proposed new British Standard BS 6399: Part 2. It deals with the assessment of more frequent parent wind speeds in the United Kingdom from the extreme wind speeds given by Part 3. Two procedures are given:

- **for estimating the values of wind speeds occurring for between one and one hundred hours per year, for making serviceability assessments**
- **for estimating the number of occurrences of wind speeds for making fatigue assessments.**

The procedure for serviceability assessments is based on the approach used in BS 8100 *Lattice towers and masts* which has been augmented with new data and has been further refined since publication of BS 8100. The procedure for fatigue assessments is based on analysis of extreme meteorological and loading data in the UK, but gives very similar results to the procedure in *ECCS Recommendations for calculating the effects of wind on constructions.*

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PARENT WIND SPEEDS

The Meteorological Office records the mean wind speed, $\overline{V}$, and maximum gust speed V, in each hour at their 140 anemograph stations across the United Kingdom. The term *parent* is used to describe this complete data record. It is most usefully represented by the cumulative distribution function (CDF), denoted by$P$, which quantifies the probability that the wind speed is *below* any given value. The probability of *exceeding* a value will be denoted here by the symbol Q, and is given for any wind speed, V, by:

$$Q_V = 1 - P_V \qquad (1)$$

As the parent wind is continuous, the proportion of time that the wind speed is below or above the value of V is represented by P_V and Q_V respectively.

As there are 8766 hours in a year, it does not take many years to define the CDF with considerable accuracy. The simplest statistical model assumes the variations of wind speed to be Gaussian or Normally distributed in two orthogonal directions, eg north-south and east-west. This leads to the expectation that the CDF of the parent wind speed *irrespective of direction* will be described by the Rayleigh distribution:

$$P_V = 1 - \exp \frac{-\overline{V}^2}{2V'^2} \qquad (2)$$

where V' is the standard deviation of $\overline{V}$.

Figure 1(a) shows parent data for Lerwick fitted to the Rayleigh distribution.

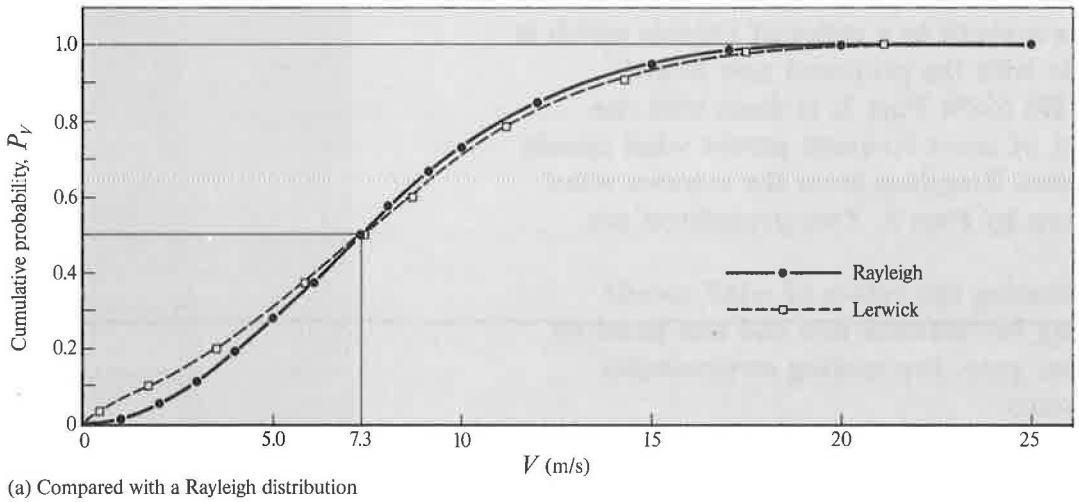
The Rayleigh distribution is a special case of the whole family of distributions called Weibull distributions given by:

$$P_V = 1 - \exp(-(c\overline{V})^k) \qquad (3)$$

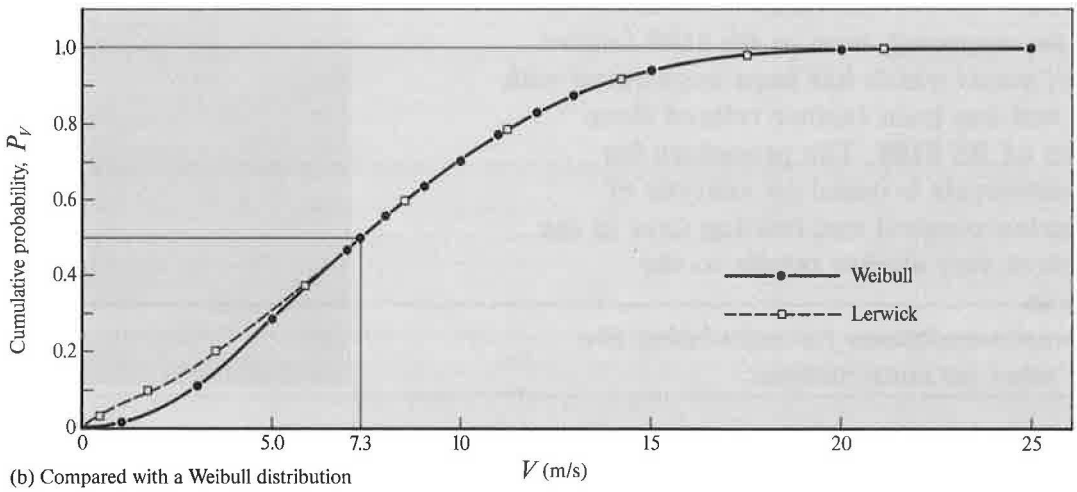
where c and k are values which define the shape of the distribution.

Comparing this equation with (2) shows that the Rayleigh distribution corresponds to a Weibull distribution with $k = 2$. The differences between the Rayleigh model equation (2) and the parent wind data in Fig 1(a) are reduced by adopting the Weibull distribution (3) and allowing the value of k to vary. The same Lerwick data are shown refitted to the Weibull distribution with $k = 1.85$ in Fig 1(b). The fit is excellent except at the lowest wind speeds, where the difference is due to friction in the bearings of the standard cup anemometer.

The parent wind speed distribution, irrespective of direction, is very well represented in the UK by Weibull distributions with k in the range 1.7 to 2.5



(a) Compared with a Rayleigh distribution



(b) Compared with a Weibull distribution

Fig 1 Parent wind speed CDF for Lerwick

EXTREME WIND SPEEDS

The requirement for structures to resist the strongest winds expected in their lifetimes means that the design calculations for the ultimate limit state are made in terms of the extreme wind speeds. An 'extreme' is defined as the maximum value occurring in a set period — usually one year to give annual maxima. If the parent values were statistically independent, the CDF of the maximum wind speed $\overline{V}_{max}$, would be given by:

$$P_{\overline{V}_{max}} = P_V^N \qquad (4)$$

where N is the number of independent values in the set period.

In the case of $N = 1$, there is no change from the parent, but as N increases the CDF shifts to higher values corresponding to the upper 'tail' of the parent. Unfortunately, the parent wind data are not statistically independent because the wind speed at any hour is related by the wind climate to the speed for some hours before and after. The value of N for a one year period is not 8766, but is in the range 100 ~ 300.

Instead of predicting the extreme wind speeds from the parent, the extreme wind climate was conventionally examined directly in terms of the CDF of measured annual maxima. More recently, BRE reanalysed the UK wind climate using the maximum wind speed in every independent storm, resulting also in a direct estimate of the number of storms N. The CDF of annual maxima was then calculated using (4). This procedure is the origin of the map in Part 3 of this Digest. As storm maxima and the annual maxima are related by (4), the possibility exists to reverse the process and estimate parent wind speeds from the extreme wind data given by Parts 3, 4 and 5, at least for the upper tail of the parent CDF in the range $1 \leq N \leq 100$.

HOURS OF WIND

The parent CDFs were formed from the meteorological data for the same sites and over the same period as were used to derive the wind speed map in Part 3. When the parent wind speed was expressed as a fraction of the basic extreme wind speed and the probability of exceedence, Q, was expressed in terms of the hours per year of exceedances, $h = 8766 \times Q$, it was found the CDF for all sites collapsed into a band. The width of this band, which represents the uncertainty of the estimate for any individual site, corresponded to a factor of 3 on hours of exceedence, h. This demonstrated that it is possible to estimate the hours of exceedence of any given serviceability wind speed, V_S, from the reference extreme wind speed, V_{REF}, given by Parts 3, 4 and 5 to an expected accuracy of a factor of 3.

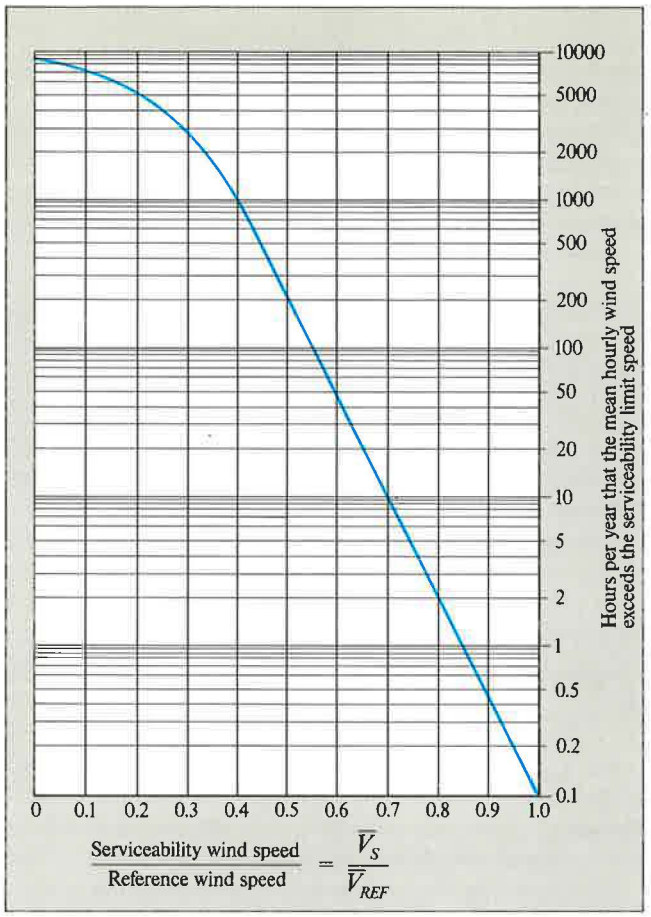


Fig 2 Hours per year that the serviceability wind speed is exceeded

Figure 2 gives a design curve for this purpose. It represents the upper boundary of the band, so gives a safe result when the effect of wind is detrimental to the design. It can be used to determine the serviceability wind speed, $\overline{V}_S$, corresponding to an acceptable number hours of exceedance per year, h, — or the number of hours of exceedence per year corresponding to an acceptable threshold wind speed. In both cases, the reference extreme wind speed, $\overline{V}_{REF}$, must first be determined from Parts 3 & 4 and, if topography is significant, from Part 5.

Should the serviceability wind speed be beneficial to the design assessment (perhaps when calculating the concentration of pollutants) the value for the lower boundary of the band is required and the value of h from Fig 2 should be divided by 3. In making a cost-benefit analysis (perhaps when comparing the cost of providing a wind-power generator against the expected supply of electrical power) a value in the middle of the band is more appropriate and the value of h from Fig 2 should be divided by 1.5.

Figure 2 was derived for all hours of wind, irrespective of direction. As the strongest winds approach the value of $\overline{V}_{REF}$ the directional characteristics converge towards those given by the Direction Factor, S_{DIR} in Part 3. However, in light winds the wind direction is strongly affected by thermal effects, such as sea breezes near coasts. Accordingly, S_{DIR} from Part 3 should be taken to apply only for wind speeds corresponding to the range $h \leq 100$.

FATIGUE LOAD CYCLES

There are two types of fatigue that may need to be considered:

- high-cycle fatigue of dynamic structures caused by oscillations of the structure by resonance at one or more of its natural frequencies, n_i, and
- low-cycle fatigue of static structures by the repeated action of gust loads

High-cycle fatigue occurs when the structure is subjected to very many thousands of load cycles at a small proportion of the ultimate capacity of the structure. The commonest source of regular oscillations of tall and line-like structures is vortex shedding which occurs when the wind speed is close to the critical wind speed (say 10% either side). There are two other sources. The first is buffeting from the turbulence of the wind or wakes of other structures. This occurs at all wind speeds, but increases as the square of wind speed so is much more important in the fewer hours of the strongest winds. The second source is galloping or flutter which occurs only above a given threshold. An estimate of the number of cycles of oscillation, N, is given from the hours for which the condition occurs, $h_1 - h_2$, and the natural frequency of the structure, n (in Hz), by:

$$N = 3600n\,(h_1 - h_2) \qquad (5)$$

where h_1 is the number of hours of exceedence of the *lower* boundary to the condition;
h_2 is the number of hours of exceedence of the *upper* boundary.

If the maximum stress in each cycle can be estimated by calculation or from observations of the motion, the fatigue life of the structure can be determined from the stress-cycle (S-N) curve for the material used. High-cycle fatigue is essentially a serviceability problem, provided the structure is properly inspected and maintained, and that fatigue cracks can be repaired or components replaced before their fatigue life is exhausted. This is common practice, for example, for the holding-down bolts of slender steel chimney stacks. If these precautions are neglected, fatigue damage can accumulate, reducing the strength until a serious structural failure or collapse occurs.

Low-cycle fatigue occurs when the structure is subjected to relatively few load cycles close to the ultimate strength of the structure, that is for a few tens to a few thousand cycles. Such high stresses tend to occur in thin metal claddings around the fixing points.

Owing to the intermittent nature of storms, low-cycle fatigue is accumulated in a few short periods corresponding to the strongest storms. In the case of severe tropical cyclones, such as cyclone Tracy at Darwin in 1974, extensive failures can occur in only a few hours of exposure. A satisfactory inspection of a structure after one severe storm may not guarantee survival of the next.

Since 1974 rules to cope with fatigue in tropical cyclones have been incorporated into Australian regulations, but these are too onerous and inappropriate for the depression-dominated climate of the UK and Europe. A table of cycles at various proportions of the ultimate design load appropriate to a 50-year design life in the UK was proposed by BRE in 1984 and is given in Table 1. This table was derived by counting the number of cycles caused by the highest, second-highest, next-highest ..., *ie* working *downwards* from the extreme. Contemporary to this, the European Convention for Constructional Steelwork (ECCS) recommended a loading sequence derived by counting *upwards* from the parent. These two independent approaches gave almost identical results, giving confidence as to their accuracy.

Table 1 Fatigue test representing typical UK service loads in 50-year exposure period

	Number of cycles	Percentage of design pressure
	1	90
	960	40
Apply sequence	60	60
five times	240	50
	5	80
	14	70
Finish with	1	100

The order of the load cycles in Table 1 is designed to represent the random sequence of loads occurring in nature as a practical loading sequence for applying proof loads to a structure or component. The main sequence of 1280 cycles from zero to between 40% and 90% of design load is repeated five times, giving 6400 cycles in all; then the structure is proof loaded by one cycle of the design load.

More recently, the development of the BRE computer-controlled test rig, BRERWULF, now enables records of real storms to be applied to structural components, obviating the need for a standard load-cycle sequence.

FURTHER READING

REDFEARN D. A test rig for proof-testing building components against wind loads. BRE Information Paper IP19/84. Garston, Building Research Establishment, 1984.

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COOK N J, KEEVIL A P and STOBART R K. BRERWULF — the Big Bad Wolf. Journal of Wind Engineering and Industrial Aerodynamics, 29, (1988), 99 — 107.

See also Further Reading in *Part 1*.

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The assessment of wind loads

Part 8: Internal pressures

This is the eighth in a series of Digests which is compatible with the proposed British Standard BS 6399: Part 2. Internal pressures do not affect overall wind loads on an enclosed structure because their contributions cancel out over all internal faces. However, they do affect the overall loading on external walls or roofs, and on internal walls.

The internal pressure is primarily controlled by:
- the external pressure field around the building;
- the position and size of all openings which connect the inside to the outside of the building.

Minor effects that influence internal pressure are:
- thermal effects;
- pressure rises from air conditioning plant.

This Digest considers the assessment of internal pressures and includes recommendations for design.
Three typical examples are shown in Figure 1.

In Figs 1(a) and 1(b) the openings dominate the internal pressure and are called 'dominant openings'. The effects on design are discussed in this Digest.

POROSITY AND DOMINANT OPENINGS

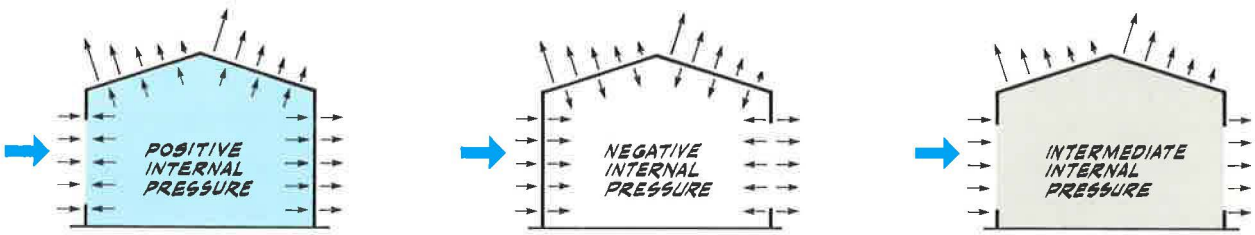
Most buildings have some degree of porosity on each face owing to leakage gaps around doors and windows, through vents and through the cladding itself. The total porosity can amount typically to between 0.01% and 0.1% of the building face area. Chimneys or open doorways can, of course, provide large openings which will affect the porosity. The quantity of air flowing through any aperture is proportional to the cross-sectional area, A, of the aperture and the square root of the pressure drop $(p_e - p_i)$. In Fig 1(a), where there is a large opening in the windward face 1 of area A_1 and external pressure p_1, but normal porosity is assumed in the leeward face 2 of area A_2 and external pressure p_2, flow continuity requires that the inflow and outflow balance so that:

$$A_1\sqrt{p_1 - p_i} = A_2\sqrt{p_i - p_2} \quad \text{.....(1)}$$

$$\frac{p_1 - p_i}{p_i - p_2} = \left(\frac{A_2}{A_1}\right)^2 \quad \text{.....(2)}$$

If the area of the large opening is, say, three times larger than the sum of the distributed porosity of the opposite wall, the internal pressure will rise so that only about 10% of the load is taken on the wall with the large opening: the remaining 90% is taken on the opposite wall. This is effectively treating that wall as solid and hence the large opening is termed 'dominant'. If the large opening is equal in area to the porous area in the opposite wall, the load is shared equally and the opening is not 'dominant'.

An empirical basis has been used to define the limit for 'dominant' openings and this has been taken when the opening is twice as large as the sum of the porosity of the rest of the building, giving a load ratio of 25%:75%. Openings of this size or larger should be treated as dominant with the internal pressure governed by the pressure at this opening. For smaller openings the internal pressure must be estimated from the balance of flow.



Air flows through the opening until the internal pressure equalizes the external pressure on the windward wall. It can be seen that the net loads (suctions) on the roof and leeward wall are increased due to the internal pressure.

(a) Opening in windward wall

Air flows through the opening until the internal pressure equals the external suction on the leeward wall. The loading on the roof is now reduced but the loading on the windward wall is increased.

(b) Opening in leeward wall

A balanced situation. The internal pressure is controlled by the balance of the two flows, which by continuity must be equal and opposite.

(c) Openings in windward and leeward walls

Fig 1 Effects of openings on internal pressures

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BUILDING FLEXIBILITY AND AVERAGING TIME

The pressures that are involved in wind loading problems (of the order of 1000 to 2000 N/m^2) are very small in relation to atmospheric pressure, which is of the order of 100,000 N/m^2. So, if only 1% of the volume of air in an enclosed space is admitted, it will increase the pressure in the space by 1000 N/m^2. The time dependency of the pressure change is extremely complicated but it can be shown that a relatively small opening can be effective in permitting a substantial change of internal pressure in a short time. For conventional buildings with low overall porosity, the response time to equalise the internal and external pressures is long, associated with large damping. However, for a large opening, such as a door or broken window, damping may not be sufficient to prevent resonance (the Helmholtz resonance problem in acoustics) and the response time is short. Such resonance may reflect a small mean change in internal pressure but significant quasi-static fluctuations dependent on the positions and number of dominant openings.

Because of the importance of this, designers should consider the effects of window breakage during a severe storm. Such breakages occur either on the windward face, sometimes as a result of impact from wind-borne debris, or in areas of high local suction near the windward edges of faces. Whether a large access door should be assumed opened or closed depends on the use and control of the building. If control is such that it is extremely unlikely that the door could be open in the full design wind condition, it would be reasonable to design for the closed case at ultimate conditions, but consider the open case for serviceability.

Whether a fast or slow averaging time for internal pressures is beneficial or adverse to a design depends on whether the internal pressure increases or decreases the loading on the most critical face. If a building is rigid, the change in internal pressure requires a certain volume of air to flow in or out of the room to compress or expand the air trapped in the room. If the building is flexible, the volume of the room will change and all the air displaced by this change must also pass through the openings; this has the effect of slowing the response time. Indeed, for buildings such as air-supported fabric structures, the response time is dominated by the volumetric change and can be perhaps 10 times longer than for a rigid building.

CONVENTIONAL BUILDINGS

These are buildings where the porosity of the external skin is distributed in the form of many small openings over several faces. Most buildings come within this category when all large doors and windows are closed.

The load duration, t, for internal pressures may normally be calculated according to the TVL formula:

$$t = 4.5l/\bar{V} \qquad (3)$$

where l = diagonal length of the largest envelope area enclosing the distributed openings on each face

$\bar{V}$ = mean wind speed on the face

With a fully glazed facade, or a building clad in unsealed sheeting, l will be the length taken for the whole face used for external pressures.

It is possible that the response time is larger than t given by the above equation for very flexible structures; in this case recourse to reference literature should be made.

The reference dynamic pressure for calculating the internal pressure should then be taken at the height defined for the external pressure coefficients and for the effective load duration, t.

Internal pressure coefficients should then be determined by quasi-static balance of flow from equation (1), considering all appropriate combinations of open doors and windows in external and internal walls for each limit state. Fortunately, experience shows that typical ranges of internal pressure coefficients can be defined for the commonest forms of building porosity; these are given in Table 1.

LOADS ON INTERNAL WALLS

Differences of internal pressure between rooms generate loads on the internal walls and partitions. These differences can be large enough to cause damage to finishes, difficulty in opening doors or even failure.

In Fig 2, a single partition spans between impermeable walls, dividing the building into two rooms A and B. If the porosity of the internal partition is greater than twice that of the external windward and leeward walls, virtually all the load is taken by the external walls and very little by the partition. Clearly, to minimise loads on internal walls, their porosity should be at least twice that of the external walls.

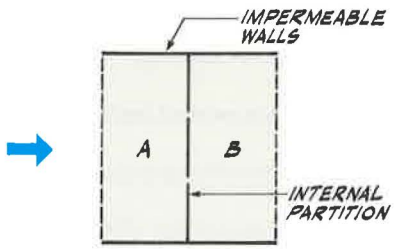


Fig 2 Building with single partition

For multi-room and multi-storey buildings, design procedures were described in the 1974 BRE *Wind loading handbook*. These have been updated using pressure coefficients from Part 6 of this Digest; the results for typical configurations are shown on page 4 under *Typical calculation for multi-room building.* The conclusions drawn are:

- all storeys with the same layout of rooms give closely similar results;
- connections between storeys make no significant difference;
- the largest loads occur on internal walls of corner rooms.

The first two conclusions are useful, since they allow the calculations to be reduced to a single floor when a common floor plan is used.

Table 1 Internal pressures for conventional buildings with and without dominant openings

Condition	Internal pressure coefficient C_{pi}
Two opposite walls equally permeable; other faces impermeable	
Wind normal to permeable face	+ 0.2
Wind normal to impermeable face	− 0.3
Four walls equally permeable; roof impermeable	− 0.3
Dominant openings (see Fig 1)	
area ratio 2	0.75 C_{pe}
area ratio >3	0.90 C_{pe}

MULTI-LAYER CLADDING

As long as a multi-layer skin acts structurally as a single unit (such as a properly tied cavity wall), the wind distribution through the layers is irrelevant. However, there are a number of forms of construction where the way that the wind loads are distributed through the multiple layers is important. Examples are cladding systems assembled from different manufacturers' components, porous cladding systems and loose laid paving/insulation slabs on roofs. Guidance on these is given in Digests 295, 311 and 312.

CONTROL OF INTERNAL PRESSURE

Internal pressure can be controlled by the provision of vents in specified locations; these are normally required for other reasons, such as ventilation or to suppress condensation. Inappropriate positioning of vents may lead to damage to non-structural components, such as ceilings.

Control is effected by incorporating a vent which serves as a dominant opening to an external pressure. In this way, uplift on low-pitch roofs can be reduced, possibly by providing vents in regions of high suction, such as along the line of the ridge. The penalty is to increase the net loading on the windward wall and possibly unacceptably high ventilation rates.

The use of spring-loaded vents, which open only when the pressure difference exceeds a threshold value, may be worth considering, subject to the reliability of the components.

OPEN-SIDED BUILDINGS

For open-sided buildings, such as grandstands, the internal pressure acting on the side opposite the open side contributes to the net load on the building and must be taken into account for stability. The effective load duration, t, can be calculated from equation (3) using the diagonal of the largest open face as the reference length (not the diagonal of the loaded face which is used for the external pressures). The internal pressure varies with angle of incidence, θ, to the normal to the principal open face as shown in Fig 3. It also varies within the building, particularly for Fig 3(b), where the internal pressure increases from Zone B to Zone D for wind inclined at between 30° and 60° to the normal.

One open face

Pressure coefficients for one open face are given in Table 2 for the configurations shown in Figs 3a and 3b.

Two or more open adjacent faces

Pressure coefficients for two or more open faces are given in Table 3 for the configurations shown in Figs 3c and 3d.

Table 2 Coefficients for one open face

Open face	Shorter face	Longer face		
Region	A	B	C	D
θ = 0°	+ 0.85	+ 0.68	+ 0.68	+ 0.68
30°	+ 0.71	+ 0.54	+ 0.70	+ 0.80
60°	+ 0.32	+ 0.38	+ 0.44	+ 0.54
90°	− 0.60	− 0.40	− 0.40	− 0.40
120°	− 0.46	− 0.46	− 0.46	− 0.46
150°	− 0.31	− 0.40	− 0.40	− 0.40
180°	− 0.16	− 0.16	− 0.16	− 0.16

Two opposite open faces

This is the special case where wind can blow through the building, the flow for different angles of incidence being steered by the side walls, creating an additional force on them. This effect is greater when the shorter walls are open.

The maximum overall side force for a building of this configuration will be 1.8 times the force on a closed building of the same shape and will occur with the wind at 45° to the building axis:

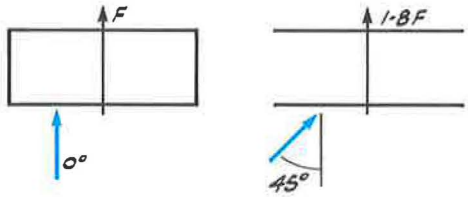
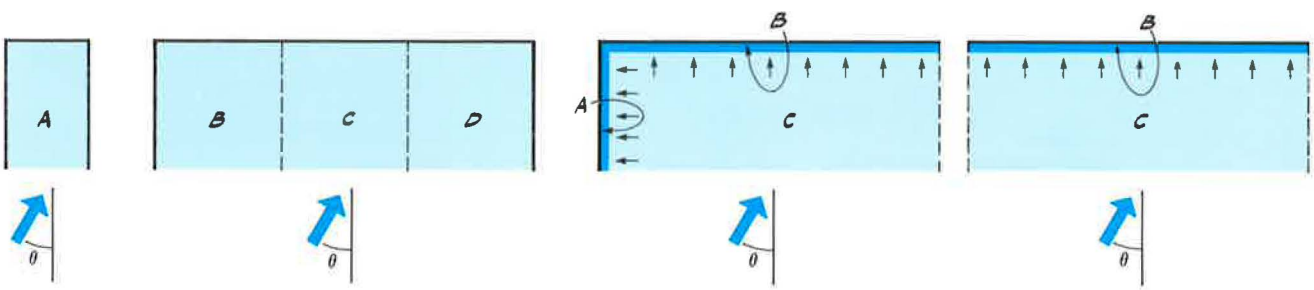


Table 3 Coefficients for two or more open faces

Open face	Longer and one shorter face			Longer and both shorter faces	
Region	A	B	C	B	C
θ = 0°	+ 0.76	+ 0.77	+ 0.63	+ 0.82	+ 0.59
30°	+ 0.51	+ 0.59	+ 0.48	+ 0.68	+ 0.52
60°	− 0.35	− 0.18	− 0.43	+ 0.43	+ 0.33
90°	− 0.26	− 0.34	− 0.38	0	0
120°	− 0.36	− 0.42	− 0.47	− 0.63	− 0.61
150°	− 0.31	− 0.37	− 0.39	− 0.44	− 0.49
180°	− 0.26	− 0.29	− 0.33	− 0.34	− 0.39
210°	− 0.43	− 0.58	− 0.64	− 0.44	− 0.49
240°	− 0.18	− 0.51	− 0.53	− 0.63	− 0.61
270°	+ 0.68	+ 0.77	+ 0.65	0	0
300°	+ 0.74	+ 0.77	+ 0.65	+ 0.43	+ 0.33
330°	+ 0.78	+ 0.78	+ 0.64	+ 0.68	+ 0.52



(a) Shorter face open (b) Longer face open (c) Longer and shorter face open (d) Longer and two shorter faces open

Fig 3 Plan views of typical open-sided buildings

TYPICAL CALCULATION FOR MULTI-ROOM BUILDING

An example of a building with multiple internal divisions is shown in Fig 4. In (a), the wind is shown normal to the longer face; in (b) it is inclined at 60°. All the openings are of equal area,a.

External pressure coefficients are shown in both cases, assuming a height to breadth ratio less than 0.5.

A series of continuity equations can be drawn up as follows, equating flow in to flow out.
For Fig 4(a), $p_1 = p_3$ and $p_4 = p_6$ by symmetry,

	INFLOW	=	OUTFLOW
Rooms 1 & 3:	$a\sqrt{0.83q - p_1}$	=	$a\sqrt{p_1 + 0.68q} + 2a\sqrt{p_1 - p_7}$
Room 2:	$a\sqrt{0.86q - p_2}$	=	$2a\sqrt{p_2 - p_7}$
Rooms 4 & 6:	$2a\sqrt{p_7 - p_4}$	=	$a\sqrt{p_4 + 0.12} + a\sqrt{p_4 + 0.34}$
Room 5:	$2a\sqrt{p_7 - p_5}$	=	$a\sqrt{p_5 + 0.22}$
Room 7: (corridor)	$4a\sqrt{p_1 - p_7} + 2a\sqrt{p_2 - p_7}$	=	$4a\sqrt{p_7 - p_4} + 2a\sqrt{p_7 - p_5}$

In the arrangement of the above equations, each term governed by a square root sign must be positive and each term must be on the correct side of the equation in relation to flow inwards and outwards. It will require an iterative process to arrive at solutions, and this is most easily achieved with a computer program. A suitable algorithm for solving these equations is given in Fig 5. If during iteration, the sign of the square root becomes negative, the term must be moved to the other side of the equation.

Solving these equations gives:
$p_1 = -0.07q$
$p_2 = +0.11q$
$p_4 = -0.12q$
$p_5 = -0.10q$
$p_7 = -0.07q$

If now an additional opening of area $5a$ is made in the windward wall of room 2, to give a dominant opening, the balance for that room will become:

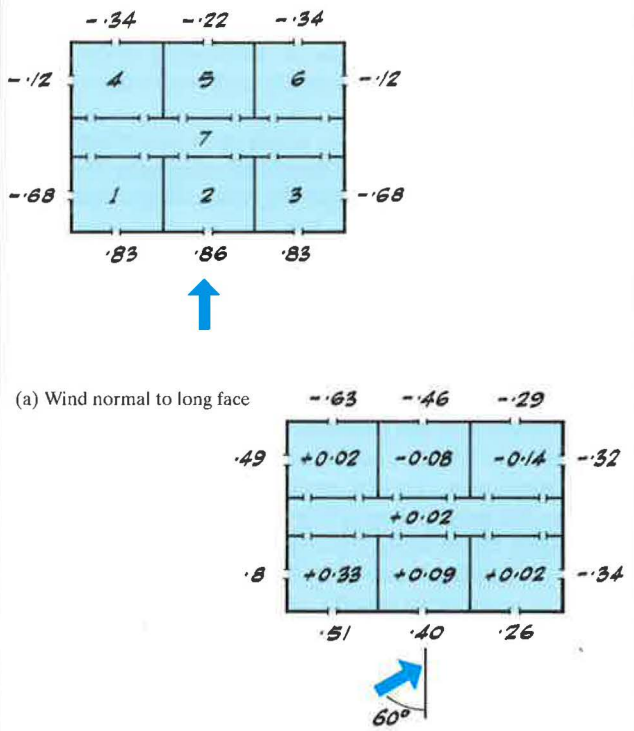
$$6a\sqrt{(0.86q - p_2)} = 2a\sqrt{p_2 - p_7}$$

The other equations remain as before and the solution becomes:
$p_1 = p_3 = +0.03q$
$p_2 = +0.78q$
$p_4 = -0.09q$
$p_5 = p_6 = -0.02q$
$p_7 = +0.03q$

This will give a pressure of $0.75q$ on the wall of the corridor and of $0.81q$ on the dividing partitions between rooms 1 and 2 and rooms 2 and 3.

In Fig 4(b) the resulting internal pressures are shown, using the above procedure, for wind inclined at 60° to the face. Both external walls of the room at the windward corner have positive external pressure, and the internal pressure becomes positive. Similarly, both external walls of the room at the leeward corner have negative external pressure and the internal pressure becomes negative. Again the biggest internal wall loads occur between corner offices and their neighbour and, in this case, on the wall to the corridor.

A similar process to this can be used in cases of multi-layer construction as, for example, to establish the pressure drop across the successive layers of a lined roof. In many practical cases though, there is difficulty in determining the boundary conditions and the relative permeabilities, and only approximate solutions are possible.



(a) Wind normal to long face

(b) Wind at 60° to long face

Fig 4 Building with multiple internal divisions

Fig 5 Simple algorithm for solving internal pressure equations

```
1   Set all internal pressures (PINT) to zero
2   Repeat
3   |       For each room
4   |       |       Set INFLOW to zero
5   |       |       For each opening into room
6   |       |       |       If PEXT > PINT then
7   |       |       |               INFLOW = INFLOW + A x sqrt(PEXT-PINT)
8   |       |       |       Else
9   |       |       |               INFLOW = INFLOW - A x sqrt(PINT-PEXT)
10  |       |       Next opening
11  |       |       PINT = PINT + FACTOR x INFLOW
12  |       Next room
13  |       Display internal pressures (PINT)
14  Until values stabilise
15  Print internal pressures
```

- Lines 5-10 determine whether air flows in or out of each opening and accumulates net inflow into the room.
- Line 11 adjusts the internal pressure for room by increasing it if net air flows in and decreasing it if net air flows out.
- The value of FACTOR determines how quickly the solution converges. If it is too large the solution will oscillate. A good value of FACTOR is one-tenth of the average opening areas into the room.
- Typically the iteration loop Lines 2-14 will need to be repeated about 100 times.

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