8 Low-rise buildings

8.1 Introduction

For the purposes of this chapter, *low-rise buildings* are defined as roofed low-rise structures less than 15 m in height. Large roofs on major structures such as sports stadia, including arched roofs, are discussed in Chapter 10; free-standing roofs and canopies are covered in Chapter 14.

The following factors make the assessment of wind loads for low-rise buildings as difficult as for taller buildings and other larger structures:

- They are usually immersed within the layer of aerodynamic roughness on the earth's surface, where the turbulence intensities are high, and interference and shelter effects are important, but difficult to quantify
- Roof loadings, with all the variations due to changes in geometry, are of critical importance for low-rise buildings. The highest wind loadings on the surface of a low-rise structure are generally the suctions on the roof, and many structural failures are initiated there
- Low-rise buildings often have a single internal space, and internal pressures can be very significant, especially when a dominant opening occurs in a windward wall. The magnitude of internal pressure peaks, and their correlation with peaks in external pressure, must be assessed.

However, resonant dynamic effects can normally be neglected for smaller buildings. The majority of structural damage in windstorms is incurred by low-rise buildings, especially family dwellings, which are often non-engineered and lacking in maintenance.

The following sections will discuss the history of research on wind loads on low-rise buildings, the general characteristics of wind pressures and model scaling criteria, and a summary of the results of the many studies that were carried out in the 1970s, 1980s and 1990s.

Several comprehensive reviews of wind loads on low-rise buildings have been made by Holmes (1983), Stathopoulos (1984, 1995), Krishna (1995) and Surry (1999).

8.2 Historical

8.2.1 Early wind tunnel studies

Some of the earliest applications of wind tunnels were in the study of wind pressures on low-rise buildings. The two earliest investigations were by Irminger (1894) in Copenhagen, Denmark, and Kernot (1893) in Melbourne, Australia. Irminger used a small tunnel driven by the suction of a factory chimney, and measured pressures on a variety of models, including one of a house. He demonstrated the importance of roof suction, a poorly understood concept at the time. Kernot used what would now be called an open-jet wind tunnel (see Section 7.2.1), as well as a whirling arm apparatus, and measured forces on a variety of building shapes. The effects of roof pitch, parapets and adjacent buildings were all examined.

Over the following thirty years, isolated studies were carried out in aeronautical wind tunnels at the National Physical Laboratory (N.P.L.) in the United Kingdom, the D.L.R. laboratories at Göttingen, Germany, the National Bureau of Standards in the United States, and the Central Aero-Hydrodynamical Institute of the U.S.S.R. These early measurements showed some disagreement with each other, although they were all measurements of steady wind pressures in nominally steady flow conditions. This was probably due to small but different levels of turbulence in the various wind tunnels, (Chapter 4 discusses the effect of turbulence on the mean flow and pressures on bluff bodies), and other effects, such as blockage.

In Denmark, Irminger, with Nokkentved (1930), carried out further wind tunnel studies on low-rise buildings. These tests were again carried out in steady, uniform flow conditions, but included some innovative work on models with porous walls, and the measurement of internal as well as external pressures. Similar but less extensive, measurements were carried out by Richardson and Miller (1932) in Australia.

In 1936 the American Society of Civil Engineers (1936) surveyed the data available at that time on wind loads on steel buildings. This survey included consideration of 'rounded and sloping roofs'. These data consisted of a variety of early wind tunnel measurements presumably carried out in smooth flow.

Flachsbart, at the Göttingen Laboratories in Germany, is well known for his extensive wind-tunnel measurements on lattice frames and bridges trusses, in the 1930s. Less well-known, however, is the work he did in comparing wind pressures on a low-rise building in smooth and boundary-layer flow. Unfortunately this work – probably the first boundary-layer wind tunnel study – was not published at the time; however, it has been rediscovered, and reported by Simiu and Scanlan (1996).

Recognition of the importance of boundary-layer flow was also made by Bailey and Vincent (1943) at the National Physical Laboratories in the United Kingdom. In doing so they were able to make some progress in explaining differences, between wind tunnel and full-scale measurements of pressures, on a low-rise shed.

However, it was not until the 1950s that Jensen (1958), at the Technical University of Denmark, explained satisfactorily the differences between full-scale and wind tunnel model measurements of wind pressures. Figure 8.1 reproduces some of his measurements, which fully established the importance of using a turbulent boundary-layer flow to obtain pressure coefficients in agreement with full-scale values. The non-dimensional ratio of building height to roughness length, h/z_o was later named the *Jensen number* (see Section 4.4.5), in recognition of this work. Jensen and Franck (1965) later carried out extensive wind tunnel measurements on a range of building shapes in a small boundary-layer wind tunnel.

The work of Jensen and Franck was the precursor to a series of generic, wind tunnel studies of wind loads on low-rise buildings in the 1970s and 1980s, including those on industrial buildings by Davenport *et al.* (1977), and on houses by Holmes (1983, 1994). Results from these studies are discussed in later sections.

Important contributions to the understanding to the effect of large groupings of bluff bodies in turbulent boundary layers, representative of large groups of low-rise buildings, were made by Lee and Soliman (1977) and Hussain and Lee (1980). Three types of flow

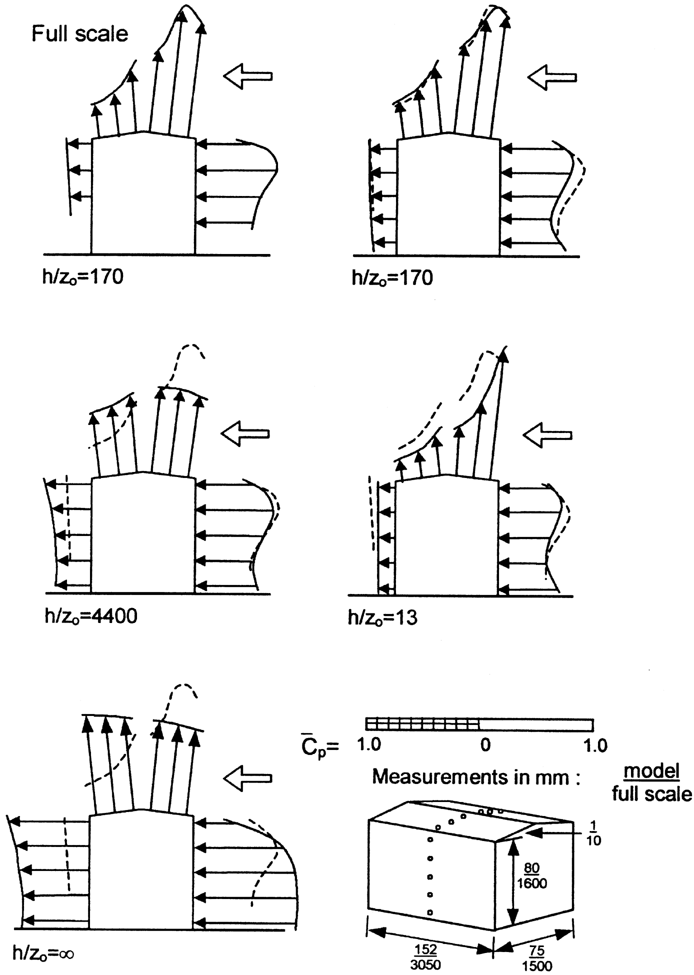

Figure 8.1 Pioneer boundary-layer measurements of Jensen (1958).

were established, depending on the building spacing: *skimming* flow (close spacing), *wake-interference* flow (medium spacing), and *isolated-roughness* flow (far spacing).

8.2.2 Full-scale studies

The last three decades of the twentieth century were notable for a number of full-scale studies of wind loads on low-rise buildings. In these studies, advantage was taken of the considerable developments that had taken place in electronic instrumentation, and computer-based statistical analysis techniques, and provided a vast body of data which challenged wind tunnel modelling techniques.

In the early 1970s, the Building Research Establishment in the United Kingdom commenced a programme of full-scale measurements on a specially constructed experimental building, representative of a two-storey low-rise building at Aylesbury, England. The building had the unique feature of a roof pitch which was adjustable between five and forty-five degrees (Figure 8.2).

The results obtained in the Aylesbury Experiment emphasized the highly fluctuating nature of the wind pressures, and the high pressure peaks in separated flow regions near the roof eaves and ridge, and near the wall corners (Eaton and Mayne, 1975; Eaton *et al*. 1975). Unfortunately the experiment was discontinued, and the experimental building dismantled after only two years at the Aylesbury site. However, interest from wind tunnel researchers in the Aylesbury data continued through the 1980s, when the International Aylesbury Comparative Experiment was established. Seventeen wind tunnel laboratories around the world tested identical 1/100 scale models of the Aylesbury building, using various techniques for modelling the upwind terrain and approaching flow conditions. This unique experiment showed significant differences in the measured pressure coefficients – attributed mainly to different techniques used to obtain the reference static and dynamic pressures, and in modelling the hedges in the upwind terrain at the full-scale site (Sill *et al.*, 1989, 1992).

In the late 1980s, two new full-scale experiments on low-rise buildings were set up in Lubbock, Texas, U.S.A., and Silsoe, U.K. The Lubbock experiment, known as the Texas Tech Field Experiment, comprised a small steel shed of height, 4.0 m, and plan dimensions, 9.1 and 13.7 m; the building had a near-flat roof (Figure 8.3). The building had the unique capability of being mounted on a turntable, thus enabling control of the building orientation relative to the mean wind direction. Pressures were measured with high response pressure transducers mounted close to the pressure tappings on the roof and walls; the transducers were moved around to different positions at different times during the course of the experiments. A 50-m high mast upwind of the building, in the prevailing wind direction, had several levels of anemometers, enabling the approaching wind properties to be well defined. The upwind terrain was quite flat and open. The reference static

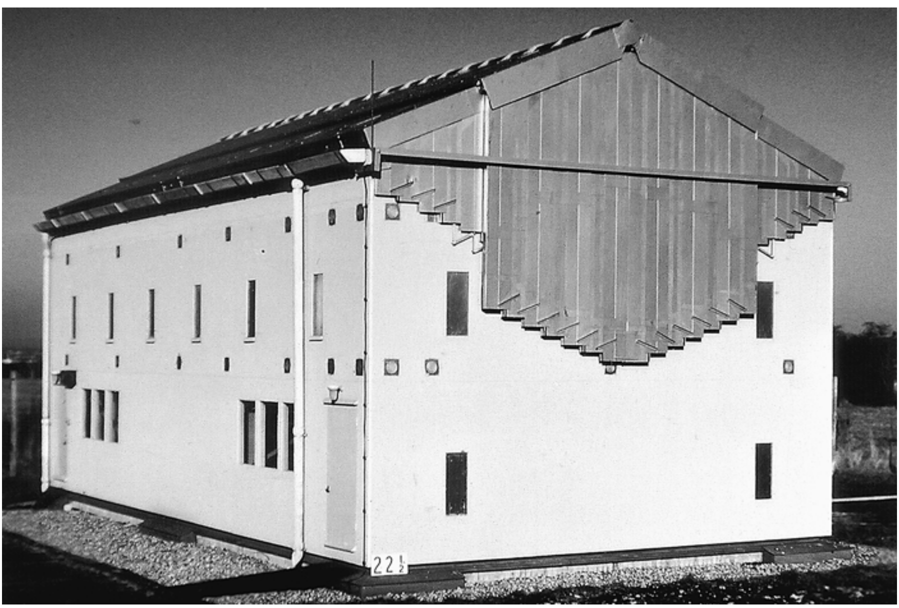

Figure 8.2 Aylesbury Experimental Building (United Kingdom 1970–75).

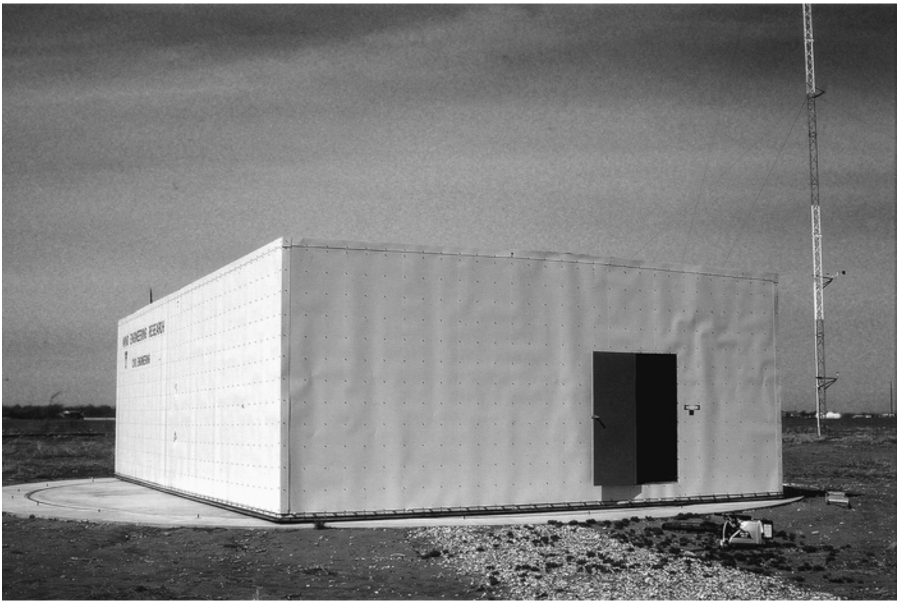

Figure 8.3 Texas Tech Field Experiment (United States 1987–99).

pressure was obtained from an underground box, 23 m away from the centre of the test building (Levitan and Mehta, 1992a and 1992b).

The Texas Tech experiment produced a large amount of wind pressure data for a variety of wind directions. External and internal pressures, with and without dominant openings in the walls, were recorded. Very high extreme pressures at the windward corner of the roof for ‘quartering’ winds blowing directly on to the corner, at about 45 degrees to the walls, were measured; these were considerably greater than those measured at equivalent positions on small (1/100) scale wind tunnel models. The internal pressures, however, showed similar characteristics to those measured on wind tunnel models, and predicted by theoretical models.

The Silsoe Structures Building was a larger steel portal-framed structure, 24 m long, 12.9 m span, and 4 m to the eaves, with a 10 degrees roof pitch, located in open country. As well as seventy pressure tapping points on the building roof and walls, the building was equipped with twelve strain gauge positions on the central portal frame to enable measurements of structural response to be made (Robertson, 1992).

The building could be fitted with both curved and sharp eaves. The curved eaves were found to give lower mean negative pressures immediately downwind of the windward wall, than those produced by the sharp eaves. Measurements of strain in the portal frame were found to be predicted quite well by a structural analysis computer program when the correct column fixity was applied. Spectral densities of the strains were also measured – these showed the effects of Helmholtz resonance (Section 6.2.3) on the internal pressures, when there was an opening in the end wall of the building. Generally these measurements justified a *quasi-steady* approach to wind loads on low-rise buildings (Section 4.6.2).

8.3 General characteristics of wind loads on low-rise buildings

Full-scale measurements of wind pressures on low-rise buildings, such as those described in Section 8.2.2, show the highly fluctuating nature of wind pressures, area-averaged wind loads, and load effects, or responses, on these structures. The fluctuations with time can be attributed to two sources (see also Section 4.6.1):

- Pressure fluctuations induced by upwind turbulent velocity fluctuations (see Chapter 3). In an urban situation, the turbulence may arise from the wakes of upwind buildings.
- Unsteady pressures produced by local vortex shedding, and other unsteady flow phenomena, in the separated flow regions near sharp corners, roof eaves and ridges (see Chapter 4).

These two phenomena may interact with each other to further complicate the situation.

It should be noted that, as well as a variation with time, as shown for a single point on a building in Figure 8.4, there is a variation with space, i.e. the same pressure or response variation with time, may not occur simultaneously at different points separated from each other, on a building.

8.3.1 Pressure coefficients

The basic definition of a pressure coefficient for a bluff body was given in Section 4.2.1, and the root-mean-square fluctuating (standard deviation) pressure coefficient was defined in Section 4.6.4. A general time-varying pressure coefficient, $C_p(t)$, for buildings in stationary, or synoptic, windstorms is as follows:

$$C_p(t) = \frac{p(t) - p_0}{\frac{1}{2}\rho_a \bar{U}^2} \tag{8.1}$$

where p_0 is a static reference pressure, (normally atmospheric pressure measured at a convenient location near the building, but not affected by the flow around the building), ρ_a is the density of air, and $\bar{U}$ is the mean (time-averaged) velocity measured at an appro-

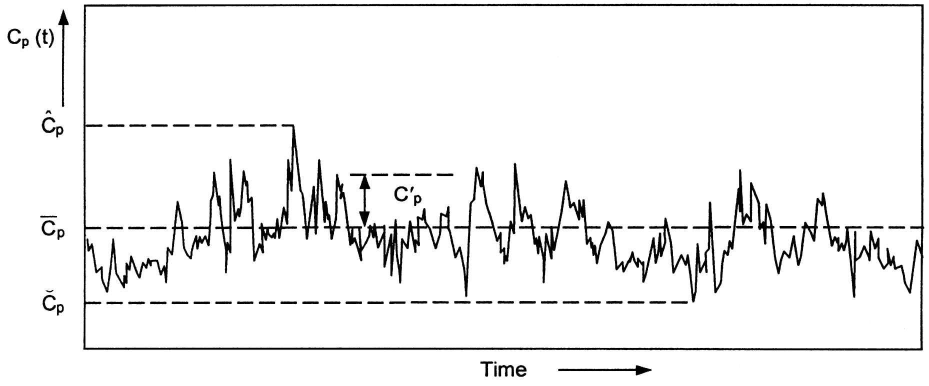

Figure 8.4 Typical variation of wind pressure and definition of pressure coefficients.

priate defined reference height, as in the atmospheric boundary layer, there is a variation of mean wind speed with height (Section 3.2). In the case of a low-rise building, this is usually taken to be at roof height, either at eaves level, mid-height of the roof, or at the highest level of the roof; as for the static pressure, this must be away from the direct influence of the building.

Figure 8.4 shows a typical variation of $C_p(t)$ on a low-rise building, and four significant values of the pressure coefficient: $\bar{C}_p$, the mean or time-averaged pressure coefficient; C_p' (= σ_{Cp}), the root-mean-squared (r.m.s.) fluctuating value, or standard deviation, representing the average departure from the mean; $\hat{C}_p$ (or $C_{\hat{p}}$), the maximum value of the pressure coefficient in a given time period; $\breve{C}_p$ (or $C_{\breve{p}}$), the minimum value of the pressure coefficient in a given time period.

8.3.2 Dependence of pressure coefficients

The dependence of pressure coefficients on other non-dimensional quantities, such as Reynolds number and Jensen number, in the general context of bluff-body aerodynamics, was discussed in Section 4.2.3. This dependence is applicable to wind loads on low-rise buildings.

For bodies which are sharp-edged, and on which points of flow separation are generally fixed, the flow patterns and pressure coefficients are *relatively* insensitive to viscous effects and hence Reynolds number. This means that, provided an adequate reproduction of the turbulent flow characteristics in atmospheric boundary-layer flow is achieved, and the model is geometrically correct, wind tunnel tests can be used to predict pressure and force coefficients on full-scale buildings. However, the full-scale studies from the Texas Tech Field Experiment have indicated that for certain wind directions, pressure peaks in some separated flow regions are not reproduced in wind tunnel tests with small scale models, and some Reynolds number dependency *is* indicated.

As discussed in Section 8.2.1, Martin Jensen identified the Jensen number, h/z_o, the ratio of building height to the aerodynamic roughness length in the logarithmic law (Sections 3.2.1 and 4.4.5), as the most critical parameter in determining mean pressure coefficients on low-rise buildings. The Jensen number clearly directly influences the mean pressure distributions on a building through the effect of the mean velocity profile with height. However, in a fully developed boundary-layer over a rough ground surface, the turbulence quantities such as intensities (Section 3.3.1) and spectra (Section 3.3.4) should also scale with the ratio z/z_o near the ground. There is an indirect influence of the turbulence properties on the mean pressure coefficients (Section 4.4.2), which would have been responsible for some of the differences observed by Jensen (1958), and seen in Figure 8.1. In wind tunnel tests, the turbulence intensity similarity will only be achieved with h/z_o equality, if the turbulent inner surface layer in the atmospheric boundary layer has been correctly simulated in the boundary-layer in the wind tunnel. Many researchers prefer to treat parameters such as turbulence intensities and ratios of turbulence length scale to building dimension as independent non-dimensional quantities (see Section 4.2.3), but unfortunately it is difficult to independently vary these parameters in wind tunnel tests.

Fluctuating and peak external pressures on low-rise buildings which are most relevant to structural design, are highly dependent on the turbulence properties in the approach flow, especially turbulence intensities. Consequently peak load effects, such as bending moments in framing members, are also dependent on the upwind turbulence. For ‘correctly’ simulated boundary layers, in which turbulence quantities near the ground scale as

z/z_o, as discussed in the previous paragraph, peak load effects can be reduced to a variation with Jensen number (e.g. Holmes and Carpenter, 1990).

Finally, the question of the dependency of pressures and load effects on low-rise buildings in windstorms of the downdraft type (Section 1.3.5) arises. As discussed in Section 3.2.5, these winds have boundary layers which are not strongly dependent on the surface roughness on the ground – hence the Jensen number may not be such an important parameter. Further research is required to identify non-dimensional parameters in the downdraft flow which are relevant to wind pressures on buildings in these types of storms.

8.3.3 Flow patterns and mean pressure distributions

Figure 8.5 shows the main features of flow over a building with a low-pitched roof, which has many of the features of flow around a two-dimensional bluff body described in Section 4.1. The flow separates at the top of the windward wall and *re-attaches* at a region further downwind on the roof, forming a separation zone or 'bubble'. However, this bubble exists only as a time average. The separation zone is bounded by a free shear layer a region of high velocity gradients, and high turbulence. This layer rolls up intermittently to form vortices; as these are shed downwind, they may produce high negative pressure peaks on the roof surface. The effect of turbulence in the approaching flow is to cause the vortices to roll up closer to the leading edge, and a shorter distance to the re-attachment zone results.

The longitudinal intensities of turbulence at typical roof heights of low-rise buildings, are 20% or greater, and separation zone lengths are shorter, compared to those in smooth, or low turbulence, flow. Small separation zones with high shear layer curvatures, are associated with low pressures, that is high initial negative pressures, but rapid pressure recovery downwind.

Roof pitches up to about 10 degrees, for wind normal to a ridge or gable end, are *aerodynamically flat*. When the mean wind direction is parallel to a ridge line, the roof is also seen as aerodynamically flat, for any roof pitch. For winds normal to the ridge line, and roof pitches between 10 and 20 degrees, a second flow separation occurs at the ridge, producing regions of high negative pressures on both sides of the ridge. Downwind of the ridge, a second re-attachment of the flow occurs with an accompanying recovery in pressure. At roof pitches greater than about 20 degrees, positive mean pressures occur

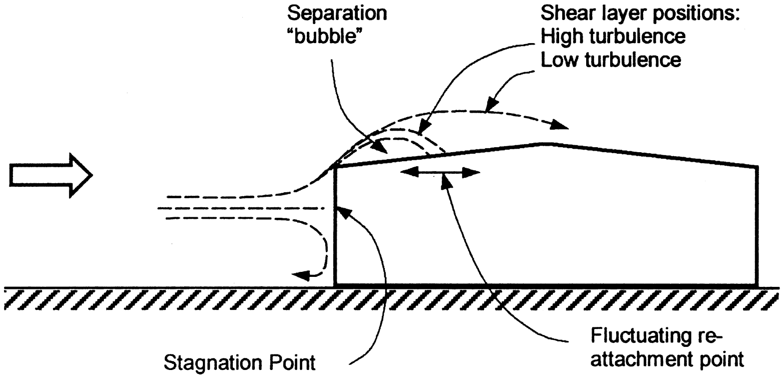

Figure 8.5 Wind flow around a low-rise building.

on the upwind roof face, and fully separated flows without re-attachment occur downwind of the ridge giving relatively uniform negative mean pressures on the downwind roof slope.

It should be noted that the above comments are applicable only to low-rise buildings with height/downwind depth (*h*/*d*) ratios less than about 0.5. As this ratio increases, roof pressures generally become more negative. This influence can be seen in Figure 8.6 which shows the mean pressure distribution along the centre line of low-rise buildings for various roof pitches and *h*/*d* ratios; the horizontal dimension across the wind (into the paper in Figure 8.7), is about twice the along-wind dimension. For higher buildings with *h*/*d* ratios of 3 or greater, the roof pressure will be negative on both faces, even for roof slopes greater than 20 degrees.

Similar flow separation and re-attachment, as described for roofs, occurs on the side walls of low-rise buildings, although the magnitude of the mean pressure coefficients is generally lower. The mean pressures on windward walls are positive with respect to the freestream static pressure. Leeward walls are influenced by the re-circulating wake, and generally experience negative pressures of lower magnitude; however, the values depend on the building dimensions, including the roof pitch angle.

When the wind blows obliquely on to the corner of a roof, a more complex flow pattern emerges as shown in Figure 8.7. *Conical* vortices similar to those found on delta-wings of aircraft occur. Figure 8.8 shows these vortices visualised by smoke – their axes are inclined slightly to the adjacent walls forming the corner. The pressures underneath these are the largest to occur on the low-pitched roofs, square or rectangular in planform, although the areas over which they act are usually quite small, and are more significant for pressures on small areas of cladding than for the loads in major structural members.

In the following sections, the effects of building geometries on design loads will be discussed in more detail.

8.3.4 Fluctuating pressures

The root-mean-squared fluctuating, or standard deviation, pressure coefficient, defined in Sections 4.6.4 and 8.3.1, is a measure of the general level of pressure fluctuations at a point on a building. As discussed in Section 8.3.2, the values obtained on a particular building are generally dependent on the turbulence intensities in the approaching flow, which in turn are dependent on the Jensen number. In boundary-layer winds over open country terrain, for which longitudinal turbulence intensities are typically around 20%, at heights typical of eaves heights on low-rise buildings, the values of r.m.s. pressure coefficients (based on a dynamic pressure calculated from the mean wind speed at eaves height) on windward walls, are typically in the range 0.3 to 0.4. In separated–re-attaching flow regions on side walls, values of C_p' of 0.6 or greater can occur. Even higher values can occur at critical points on roofs, with values greater than 1.0 being not uncommon.

High instantaneous peak pressures tend to occur at the same locations as high r.m.s. fluctuating pressures. The highest negative peak pressures are associated with the conical vortices generated at the roof corners of low-pitch buildings, for quartering winds blowing on to the corner in question (Figures 8.7 and 8.8). Figure 8.9 shows a short sample of pressure–time history, from a pressure measurement position near the formation point of one of these vortices, on the Texas Tech building (Mehta *et al.*, 1992). This shows that high peak pressure peaks occur as 'spikes' over very short time periods. Values of negative peak pressure coefficients as high as –10 often occur, and magnitudes of –20 have occasionally been measured.

The probability density function (p.d.f.) and cumulative distribution function (c.d.f.) are

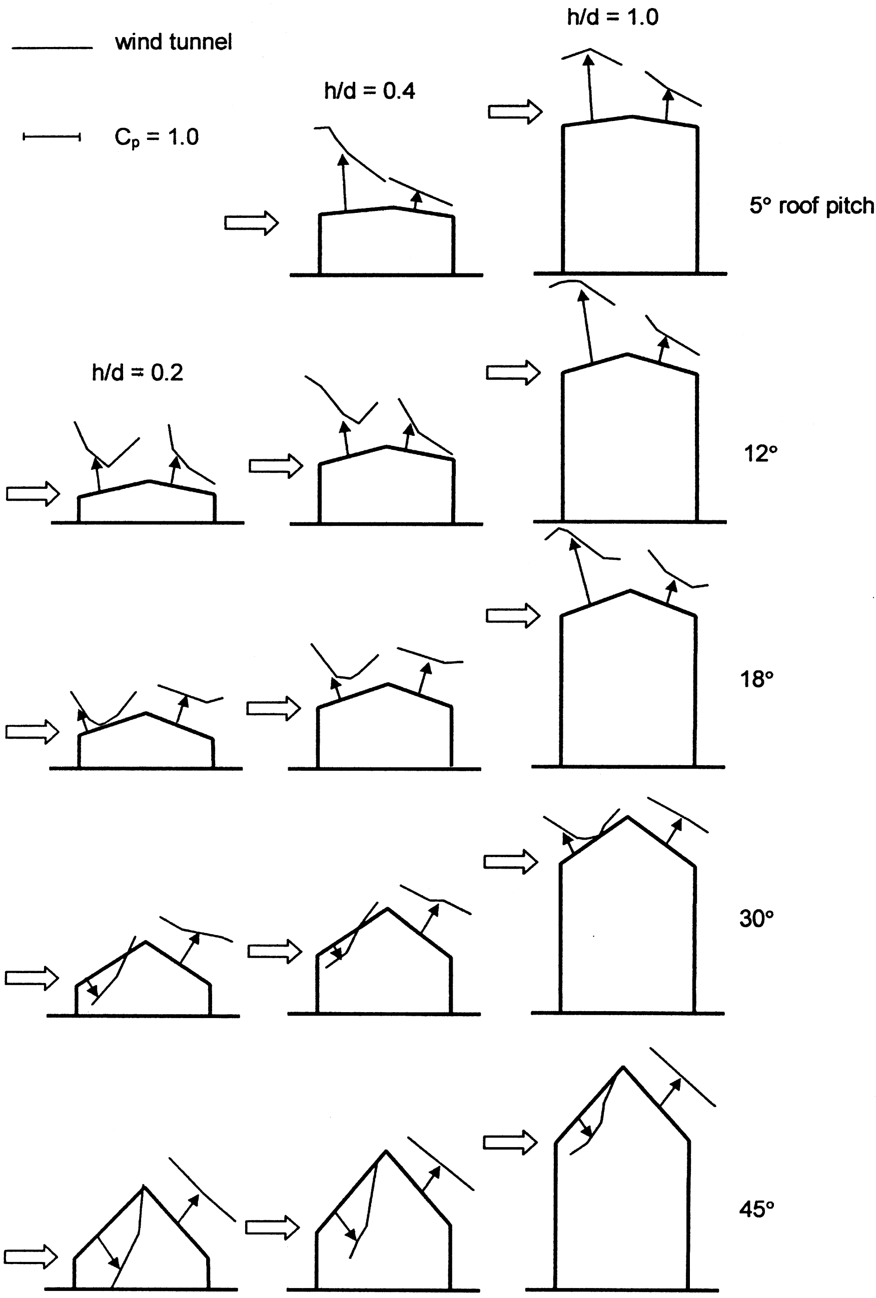

Figure 8.6 Mean pressure distributions on pitched roofs.

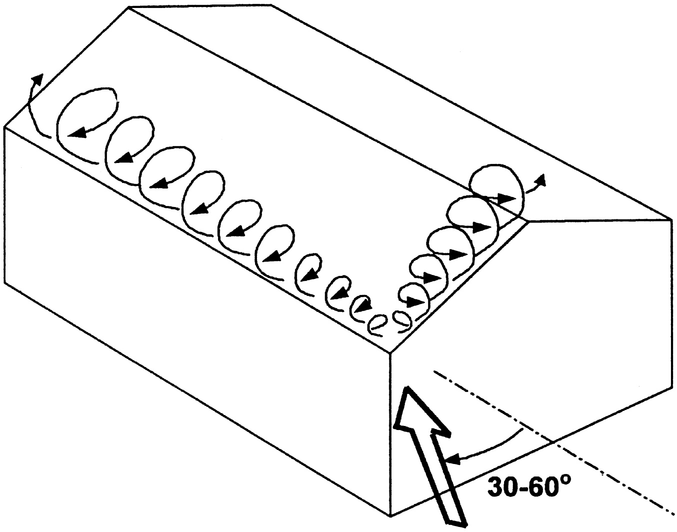

Figure 8.7 Conical vortices for oblique wind directions.

Figure 8.8 Corner vortices generated by quartering winds (from the Texas Tech Field Experiment).

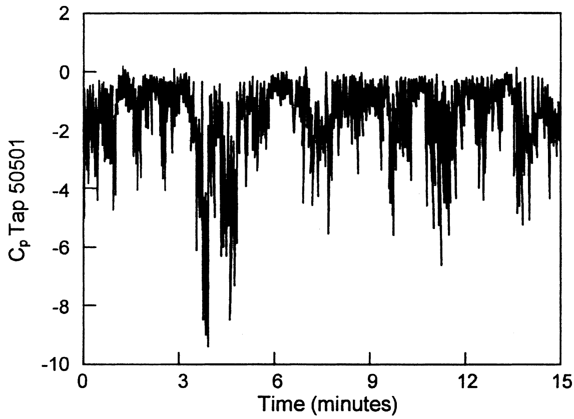

Figure 8.9 Pressure coefficient versus time from a corner pressure tap (Mehta *et al*., 1992).

measures of the amplitude variations in pressure fluctuations at a point. Even though the upwind velocity fluctuations in boundary-layer winds are nearly Gaussian (Section 3.3.2 and Appendix C, Section C3.1), this is not the case for pressure fluctuations on buildings. Figure 8.10 shows a wind tunnel measurement of the c.d.f. for pressure fluctuations on the windward wall of a low-rise building model (Holmes, 1981, 1983). On this graph, a straight line indicates a Gaussian distribution. Clearly the measurements showed upward curvature, or positive skewness (Appendix C, Figure C3). This can, in part, be explained by the square-law relationship between pressure and velocity (see equation (4.12)), (Holmes, 1981, and Appendix C3.3). Negative skewness occurs for pressure fluctuations in separated flow regions of a building.

The spatial structure of fluctuating pressures on low-rise buildings has been investigated in detail by a number of researchers, using a technique known as *Proper Orthogonal*

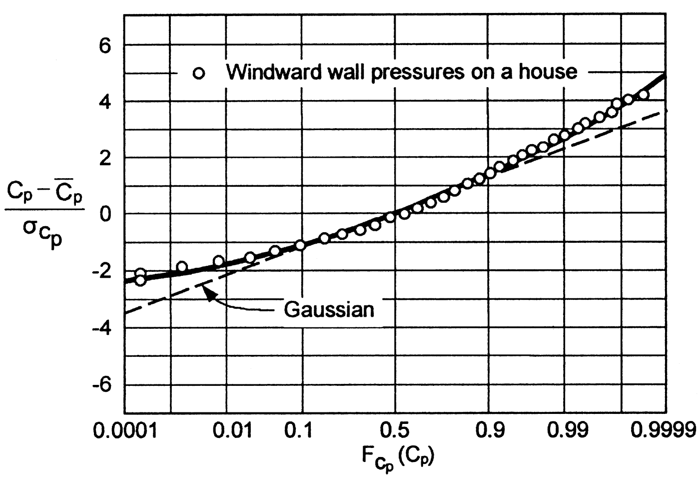

Figure 8.10 Cumulative probability distribution for pressure fluctuations on the windward wall of a house.

Decomposition (e.g. Best and Holmes, 1983; Holmes, 1990a; Letchford and Mehta, 1993; Bienkiewicz *et al.*, 1993; Ho *et al.*, 1995; Holmes *et al.*, 1997; Baker, 1999). The mathematics of this technique is beyond the scope of this book, but the method allows the complexity of the space–time structure of the pressure fluctuations on a complete roof, building, or tributary area, to be simplified into a series of 'modes', each with its own spatial form. Surprisingly few of these modes are required to describe the complexity of the variations. Invariably, for low-rise buildings, the first, and strongest, mode is 'driven' by the quasi-steady mechanism associated with upwind turbulence fluctuations.

8.4 Buildings with pitched roofs

8.4.1 Cladding loads

Figures 8.11 and 8.12 show contours of the worst minimum pressure coefficients, for any wind direction, measured in wind tunnel tests on models of single storey houses with gable roofs of various pitches (Holmes, 1994). The simulated approach terrain in the approach boundary-layer flow, was representative of open country, and the wind direction was varied at 10 degree intervals during the tests. The coefficients are all defined with respect to the *eaves* height mean wind speed.

The highest magnitude coefficients occur on the roof. At the lowest pitch (10 degrees) the contours of highest negative pressures converge towards the corner of the roof; the effect of increasing the roof pitch is to emphasize the gable end as the worst loaded region. The worst local negative peak pressures occur on the 20 degree pitch roof in this area. The highest magnitude minima on the walls occur near a corner.

Similar plots for shapes representative of industrial buildings with roof pitches of 5, 18 and 45 degrees pitch, are shown in Figures 8.13, 8.14, and 8.15 (Davenport *et al.*, 1977). In these figures, contours of maximum pressure coefficients, as well as minimum pressure coefficients, are plotted. Plots are given for three different eaves heights, for each roof pitch. Results from building models located in simulated urban terrain are shown.

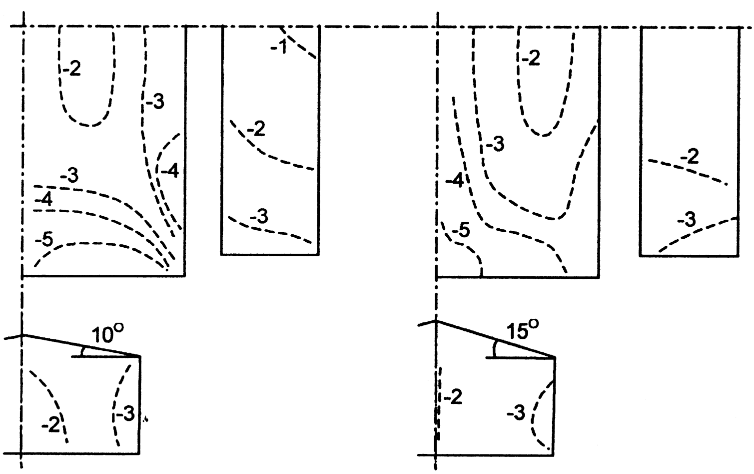

Figure 8.11 Largest minimum pressure coefficients, $\breve{C}_p$, for houses with roofs of 10 and 15 degree pitch (for any wind direction) (Holmes, 1994).

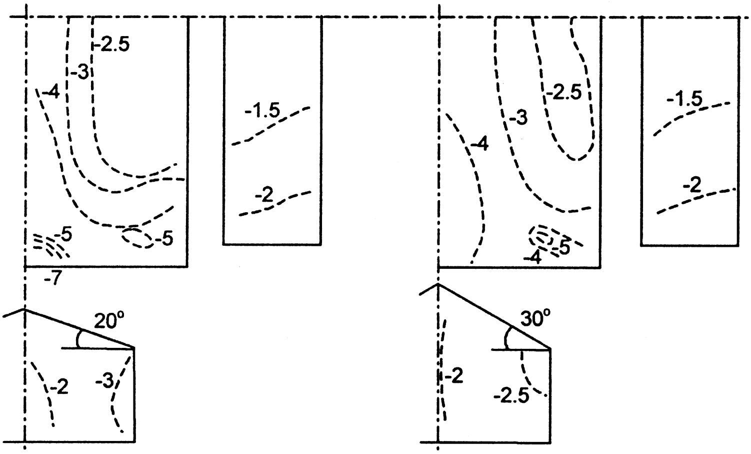

Figure 8.12 Largest minimum pressure coefficients, $\check{C}_p$, for houses with roofs of 20 and 30 degree pitch (for any wind direction) (Holmes, 1994).

For any given roof pitch, there is not a large variation in the magnitudes of the minimum and maximum pressure coefficients with eaves height – however, the pressure coefficients are defined with respect to the mean dynamic pressure at eaves height in each case. Since the mean velocity, and hence dynamic pressure, in a boundary layer increases with increasing height, the pressures themselves will generally increase with the height of the building. Since the fluctuating pressure coefficients are closely related to the turbulence intensities in the approach flow, lower magnitudes might be expected at greater eaves heights, where the turbulence intensities are lower, and this can be seen in Figures 8.13–8.15. However, the local pressure peaks are also influenced by local flow separations, and hence by the relative building dimensions.

The worst minimum pressure coefficients for the 18-degree pitch roofs (Figure 8.14), occur near the ridge at the gable end (compare also the house with the 20-degree pitch roof in Figure 8.12). For the 5-degree pitch case (Figure 8.13), there is a more even distribution of the largest minimum (negative) pressure coefficients around the edge of the roof. For the 45-degree pitch, the corner regions of the roof generally experience the

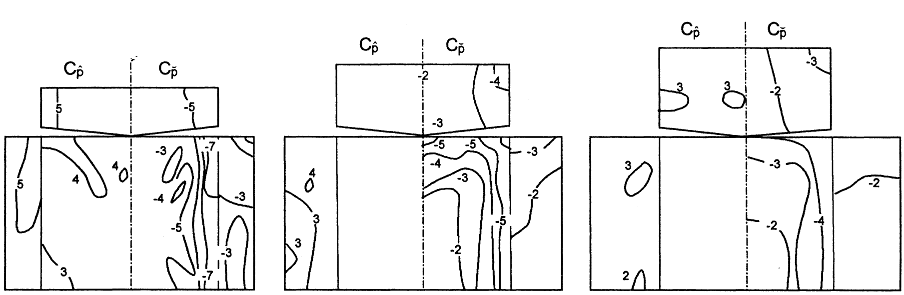

Figure 8.13 Largest maximum and minimum pressure coefficients, $\hat{C}_p$ and $\check{C}_p$, for industrial buildings with roofs of 5 degree pitch (for any wind direction) (Davenport *et al.*, 1977).

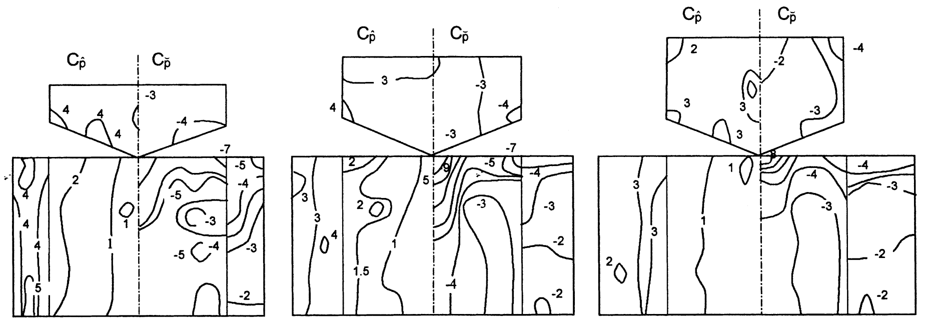

Figure 8.14 Largest maximum and minimum pressure coefficients, $\hat{C}_p$ and $\breve{C}_p$, for industrial buildings with roofs of 18 degree pitch (for any wind direction) (Davenport *et al.*, 1977).

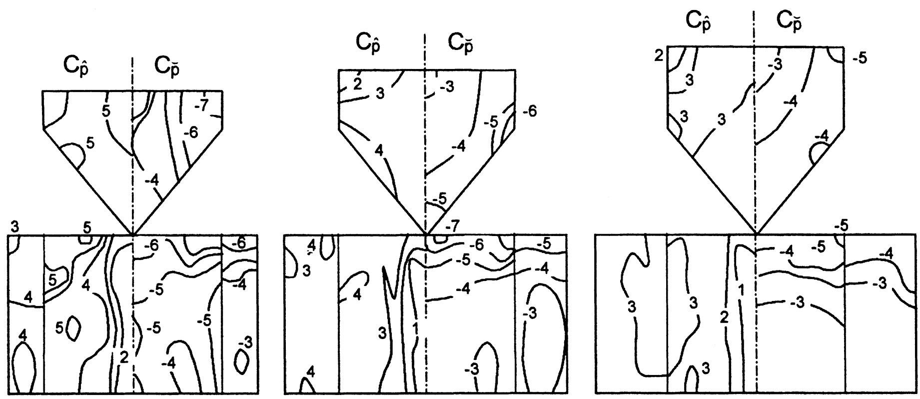

Figure 8.15 Largest maximum and minimum pressure coefficients, $\hat{C}_p$ and $\breve{C}_p$, for industrial buildings with roofs of 45 degree pitch (for any wind direction) (Davenport, *et al.*, 1977).

largest minima; the maximum pressure coefficients are also significant in magnitude on the 45-degree pitch roof.

Plots such as those in Figures 8.11 to 8.15 can be used as a guide to the specification of wind loads for the design of cladding. However, it should be noted that if the design wind speeds are non-uniform with direction, as they normally will be, the contours of maximum and minimum *pressures* (as opposed to pressure coefficients) will be different, and will depend on the site and the building orientation.

8.4.2 Structural loads and equivalent static load distributions

The effective peak wind loads acting on a major structural element such as the portal frame of a low-rise building are dependent on two factors:

- the correlation or statistical relationship between the fluctuating pressures on different

parts of the tributary surface area 'seen' by the frame; this can be regarded as an area-averaging effect
- the influence coefficients which relates pressures at points or panels on the surface to particular load effects, such as bending moments or reactions.

Chapter 5 described methods for determining effective static loading distributions, which represent the wind loads which are equivalent in their structural effect to fluctuating (background) wind pressures, and to the resonant (inertial) loads when they are significant. For the low-rise buildings under discussion in this chapter, resonant effects can be ignored, but the fluctuating, or background, loading is quite significant because of the high turbulence intensities near the ground. Some examples of the application of the methods discussed in Chapter 5, will be given in this chapter.

To illustrate the problem consider Figure 8.16. This shows instantaneous external pressure distributions occurring at three different times during a windstorm around a portal frame supporting a low-rise building. These pressure distributions are clearly different from each other in both shape and magnitude. The value of a load effect such as the bending moment at the knee of the frame, will respond to these pressures in a way that might produce the time history of bending moment versus time given in Figure 8.17. Over a given time period, a maximum bending moment will occur. A minimum bending moment will also occur. Depending on the sign of the bending moment produced by the dead loads acting on the structure, one of these extremes will be the critical one for design of the structure. Methods for determination of the *expected* pressure distribution which corresponds to the maximum or minimum wind-induced bending moment were discussed in Chapter 5. The effective static pressure distribution so determined, must lie between the extreme point pressure limits of the pressures around the frame, as shown in Figure 8.18.

It is of interest to consider the distributions of pressure coefficients given in wind codes and standards. Usually an 'envelope' loading is specified with pressures uniformly distrib-

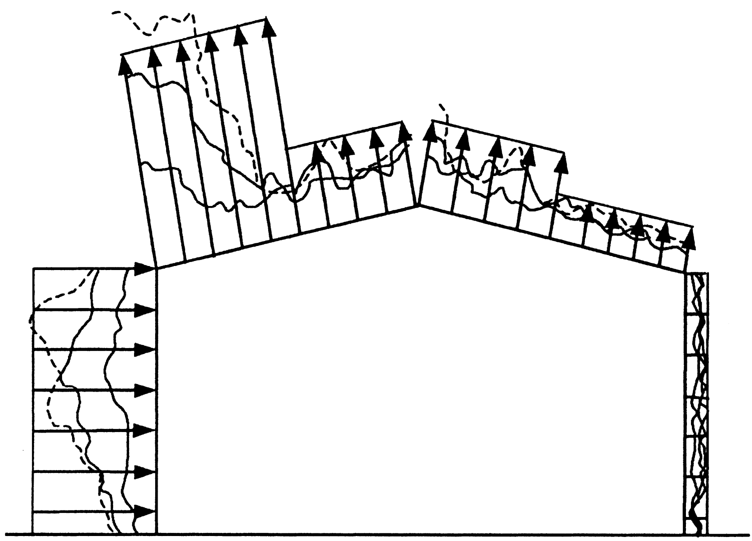

Figure 8.16 Instantaneous external pressure distributions on the frame of a low-rise building, and simplified code distributions (Holmes and Syme, 1994).

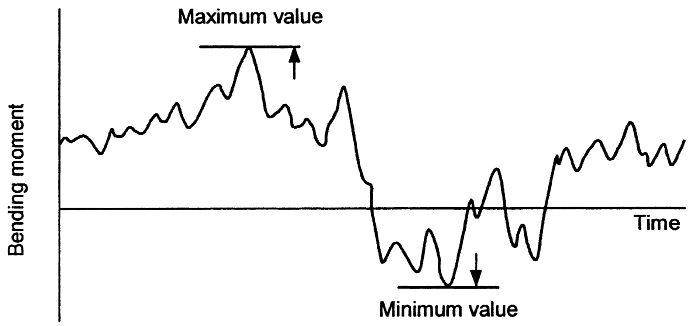

Figure 8.17 Time history of a bending moment (Holmes and Syme, 1994).

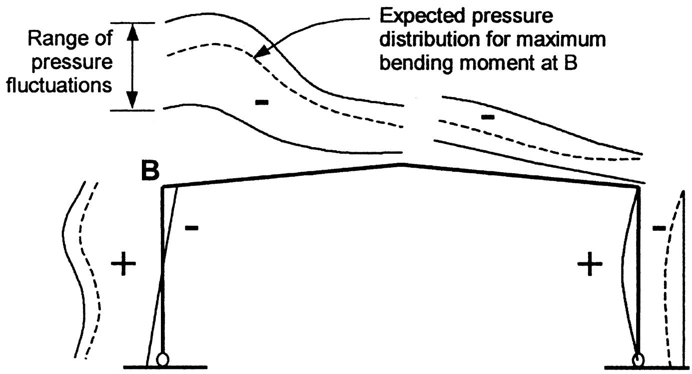

Figure 8.18 Effective static load distribution for a corner bending moment (Holmes and Syme, 1994).

uted in length along the columns and rafters, as shown in Figure 8.16. These are usually, but not always, conservative loadings which will give over-estimates of load effects such as bending moments.

8.4.3 Effect of surrounding buildings – shelter and interference

Most low-rise buildings are in an urban situation, and are often surrounded by buildings of similar size. The shelter and aerodynamic interference effect of upstream buildings can be very significant on the wind loads. This aspect was the motivation for the studies by Lee and Soliman (1977) and Hussain and Lee (1980) on grouped buildings, as discussed in Section 8.2.1. Three flow regimes were identified depending upon the building spacing. The study on tropical houses, described by Holmes (1994), included a large number of grouped building situations for buildings with roofs of 10 degree pitch. This study showed that upstream buildings of the same height reduced the wall pressures and the pressures at the leading edge of the roof significantly, but had less effect on pressures on other parts of the roof. The building height/spacing ratio was the major parameter, with the number of shielding rows being of lesser importance.

A series of wind tunnel pressure measurements, for both structural loads and local cladding loads, on a flat-roofed building, situated in a variety of 'random city' environments was carried out by Ho *et al.* (1990, 1991). It was found that the mean component of the wind loads decreased, and the fluctuating component increased, resulting in a less-distinct variation in peak wind load with direction. The expected peak loads in the urban environment were much lower than those on the isolated building. It was also found that a high coefficient of variation (60 to 80%) of wind loads occurred on the building in the urban environment due to the variation in *location* of the building. For the isolated building, similar coefficients of variation occurred, but in this case, they resulted from variation due to *wind direction*.

8.5 Multi-span buildings

The arrangement of industrial low-rise buildings as a series of connected spans is common practice for reasons of structural efficiency, lighting and ventilation. Such configurations also allow for expansion in stages of a factory or warehouse.

Wind tunnel studies of wind pressures on multi-span buildings of the 'saw-tooth' type with 20 degree pitch were reported by Holmes (1990b), and by Saathoff and Stathopoulos (1992) on 15 degree pitch buildings of this type. Multi-span gable roof buildings were studied by Holmes (1990b) (5 degree pitch), and by Stathopoulos and Saathoff (1994) (18 and 45 degree pitch). The main interest in these studies was to determine the difference in wind loads for multi-span buildings, and the corresponding single span monoslope and gable roof buildings, respectively.

As for single-span buildings, the aerodynamic behaviour of multi-span buildings is quite dependent on the roof pitch. Multi-span buildings of low pitch (say less than ten degrees) are aerodynamically flat, as discussed in Section 8.3.3. Consequently, quite low mean and fluctuating pressures are obtained on the downwind spans, as illustrated in Figure 8.19. The pressures on the first windward span are generally similar to those on a single span building of the same geometry.

For the gable roof buildings, and for the saw-tooth roof with the roofs sloping downwards away from the wind, the downwind spans experience much lower magnitude negative mean pressures than the windward spans. For the opposite wind direction on the saw-tooth configuration, the highest magnitude mean pressure coefficients occur on the second span downwind, due to the separation bubble formed in the valley.

8.6 Internal pressures

In Chapter 6, the prediction of internal pressures in buildings in general are discussed. For low-rise buildings in particular, the internal pressure loading may form a high proportion of the total wind loading for both major structural elements and cladding. In severe windstorms, such as hurricanes or typhoons, failures of roofs often occur following window failure on the windward wall, which generates high positive internal pressures acting together with negative external pressures.

8.7 Summary

This chapter has discussed various aspects of the design of low buildings for wind loads. The long history of investigation into wind loads has been discussed, and the use of the modern boundary-layer wind tunnel for determination of design loading coefficients is

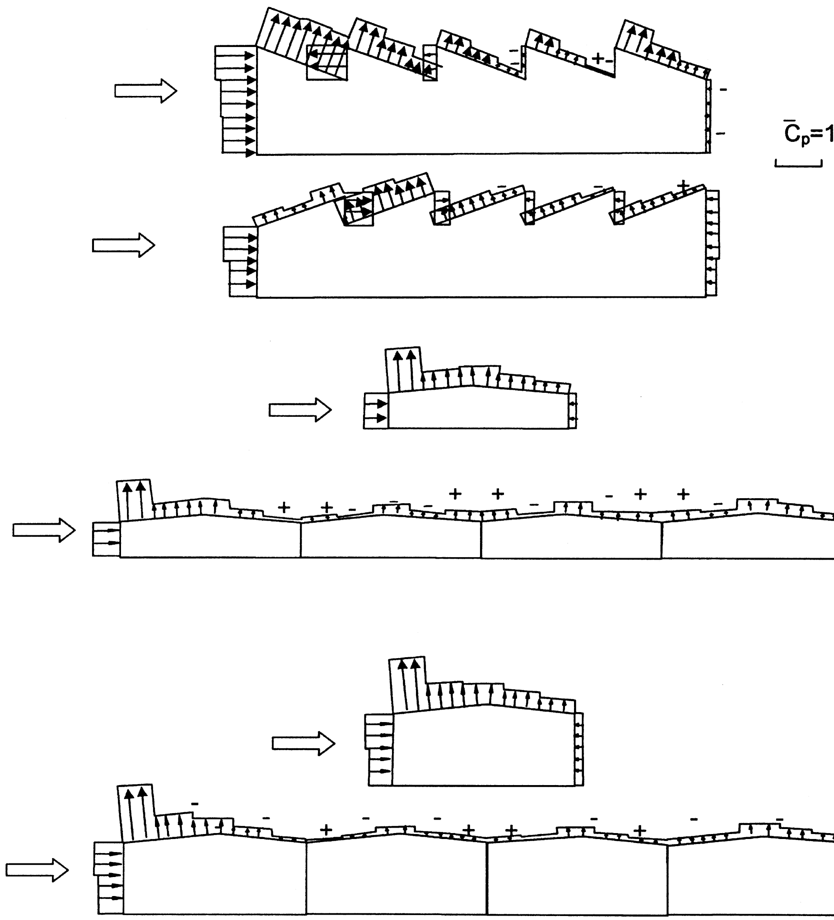

Figure 8.19 Mean pressure distributions on multi-span buildings and comparison with a single span (Holmes, 1990).

covered. The characteristics of loads for major structural members and foundations, and for local cladding have been considered for buildings with flat, and pitched roofs. The effect of shelter and interference from surrounding buildings has been considered. Multi-span building configurations have also been discussed.

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