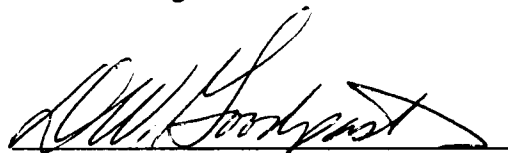
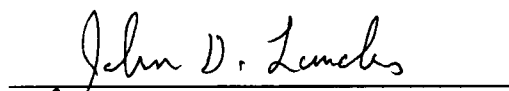


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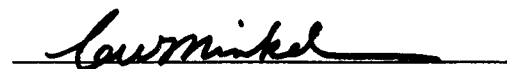
I am submitting herewith a thesis written by Ronald Albert Saklem entitled "Boundary Layer Wind Flow Characteristics and Wind Effects on Low-Rise Buildings." I have examined the final copy of this thesis for form and content and recommend that it be accepted in the partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.


David Goodpasture, Major Professor

We have read this thesis
and recommend its acceptance:




Accepted for the Council:


Associate Vice Chancellor and
Dean of The Graduate School

BOUNDARY LAYER WIND FLOW CHARACTERISTICS AND WIND EFFECTS
ON LOW-RISE BUILDINGS

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Ronald Albert Saklem
August 1998

DEDICATION

This thesis is dedicated to my parents

Mr. and Mrs. Albert Armand Saklem

and

to ethics and professionalism in the practice of engineering.

ACKNOWLEDGEMENTS

Foremost, I wish to acknowledge the steadfast love of my Lord and Savior Jesus Christ, whereby I was able to persevere in meeting all requirements for completing this thesis. Professor David W. Goodpasture was instrumental in my coming to better understand the physical phenomena for which the structural engineer must design. He encouraged me to study advanced mechanics of materials, and wind effects on structures, which lead to the writing of this thesis. Professor Goodpasture also instructed me in the area of technical writing. Dr. Mancil W. Milligan provided tips, which were instrumental in my coming to better understand the characteristics of wind flow, and the language of wind engineering. Professor John D. Landes encouraged me to persevere in the study of advanced mechanics of materials, and he provided friendly support for my completing this thesis. Professor Richard M. Bennett assisted in my literature search, and provided spirited support for my completing this thesis. Mr. and Mrs Richard Eckert shared their home with me for a time, as did Professor and Mrs. William Bass.

ABSTRACT

Procedures for evaluating design wind loads in terms of the roughness of terrain, average wind velocity, building geometry and desired building safety are available in standard and model building codes. Basic understanding of wind flow characteristics, knowledge of bluff-body aerodynamics and general knowledge of wind climatology are helpful in implementing the provisions of these codes in design practice. Initially, information about the wind climatology of the United States is presented in this paper. Subsequently, the subject of micrometeorology is opened, as it relates to wind engineering. General information about wind tunnel modeling is then presented as a means of more descriptively illustrating the relationships of wind flow and terrain characteristics to the dynamic pressure responses of low buildings. Finally, data used in the development of the ASCE standard's provisions for the calculation of wind loads affecting mono-slope and multi-span gable roofs of low-rise buildings are presented. Additionally, a comparative study of the performance of hip and gable roofs in severe wind events is presented. Based on the descriptions of wind flow characteristics presented in the body of the paper, an interpretation of the gust effect factor model provided in the commentary of the ASCE standard is rendered.

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NOMENCLATURE

α	power law exponent
$C_{\bar{F}}$	mean total force coefficient
C_F^+	total force coefficient, positive peak
C_F^-	total force coefficient, suction peak
C_F	total force coefficient, root mean square value
C_p	pressure coefficient, mean value
C_p^-	pressure coefficient, suction peak
C_p^+	pressure coefficient, positive peak
C_p	pressure coefficient, root mean square value
δ	atmospheric boundary layer thickness
GC_p	gust pressure coefficient
H	denotes a characteristic building dimension, e.g., eave height, building height (flat roof), average roof height.
$\frac{H}{z_o}$	Jenson number
$I(z)$	relative turbulence intensity, longitudinal component
k	Von Karman constant
L_u^x	along wind integral length scale of turbulence (AWILST)
L_u^y	longitudinal integral length scale of turbulence, lateral component
Q	background response

q_h	mean dynamic pressure at eave height
q_z	mean dynamic pressure at height z
ρ	density of air
$R_{u_1 u_2}(x)$	Cross correlation of longitudinal velocity fluctuations
σ_u	root mean square (rms) of longitudinal wind velocity fluctuations
τ_o	surface shear
$\bar{U}$	mean wind velocity, longitudinal component
U_g	gradient wind velocity
$\frac{\partial U}{\partial z}$	wind velocity gradient, longitudinal component
u	longitudinal velocity fluctuations (LVF)
u_*	shear velocity
u, v, w	longitudinal, transverse and vertical wind velocity fluctuations, respectively
$v_{f, gable}$	failure wind speed, gable roof
$v_{f, hip}$	failure wind speed, hip roof
W	transverse building dimension
x, y, z	longitudinal, transverse and vertical building directions, respectively
z_d	logarithmic wind velocity profile displacement height
z_o	roughness length of the terrain

LIST OF ABBREVIATIONS

AA.....	Arithmetic averaging of dynamic pressure response
ABL.....	Atmospheric boundary layer
ANSI.....	American National Specifications Institute
ASCE.....	American Society of Civil Engineers
ASCE 7-95	Specification of Minimum Design Loads for Buildings and Other Structures
AWILST...	along wind Integral length scale of turbulence
BLWT.....	boundary layer wind tunnel
CBS.....	Center for Building Studies, Concordia University, Canada
DMSGGR....	multiple span gable roof
FAA.....	Federal Aviation Association
HZ.....	hertz, frequency response of dynamic pressure fluctuations
LILST.....	longitudinal integral length scales of turbulence
LVF.....	longitudinal velocity fluctuations
LTI.....	longitudinal turbulence intensity
SSGR.....	single span gable roof
MRI.....	mean recurrence interval
MWFRS...	main wind force resisting system
NBCC.....	National Building Code of Canada
NOAA.....	National Oceanic and Atmospheric Administration
NWS.....	National Weather Service
PA.....	pneumatic averaging of dynamic pressure response
rms.....	root mean square

RSL..... roughness sub-layer
SL..... surface layer of atmospheric boundary layer
TTU..... Texas Technological University
UBC..... Unified Building Code
UFWT..... uniform flow wind tunnel
UWO..... University of Western Ontario

CHAPTER 1

OBJECTIVE AND SCOPE

1.1 Objective:

An interpretive presentation of the characteristics of wind flow in various exposures, as they relate to the specification of wind loads affecting structures, is the objective of this study. The need for a paper of this type arises, because 1) wind engineering terminology is not well known to a great many design professionals; and 2) a great deal of summarization is involved in the codification of research data. More specifically, this paper is intended to provide necessary information for better understanding wind flow phenomena, so as to facilitate interpreting wind codes consistently with their precepts when their specifications do not explicitly address some aspect(s) of any given project.

1.2 Scope:

This study focuses upon boundary layer wind flow characteristics, and the aerodynamic behavior of quasi-static structures. More specifically, the areas investigated in this study include, primarily, the relationship of the roughness characteristics of terrain to boundary layer wind flow characteristics, and associated dynamic pressure responses of low-rise structures.

CHAPTER 2

WIND CLIMATOLOGY OF THE UNITED STATES

2.1 General Information:

Panel on the Assessment of Wind Engineering Issues in the United States, et al., 1993

The average annual wind speeds in the Continental United States range between 8-12 mph (4-6 m/s) (1). Certain topographical features produce higher average wind speeds. The mechanism responsible for higher wind speeds in mountain passes, and over hills, ridges and escarpments is essentially a funneling effect, in which the crest of such topographic features acts as a nozzle. Wind speeds along coastlines are also higher on average, because less friction is developed in the approaching flow at its interface with seawater than at its interface with inland terrain. It is noted, however, that shear stress develops rapidly in the surface layer after winds move inland. Conversely, average wind speeds are lower in sheltered regions, such as cities and forests. The obstruction to flow that is presented by many bluff-bodies situated in close proximity is the primary reason for reduced wind speeds in sheltered areas. Wind speeds of 50 mph (22m/s) are common; and nearly every part of the country will experience gust speeds of over 85 mph (38 m/s) occasionally (8). The Atlantic and Gulf coasts can expect gust speeds of over 140 mph (63 m/s).

Thunderstorm activity produces most of the high winds in the central one-half of the country (1). In the western mountain states, high winds are usually attributable to

down-slope winds, which can produce gusts to 140 mph (60m/s). Severe winds along the East and Gulf coasts are generally the result of thunderstorms and hurricanes. Although they are not responsible for most of the extreme wind activity on the northern part of the Atlantic coast, extratropical cyclones produce very high wind speeds locally. In some instances winds associated with extratropical cyclones have reaped more havoc than hurricane winds. More damage was caused throughout Pennsylvania, New Jersey and the interior of southeastern New York by the passage of an extratropical cyclone in November 1950 than by the hurricane of 1938. The record gust speeds in this country, set in April 1934 on Mt. Washington, New Hampshire were associated with the passage of an extratropical cyclone.

2.2 Measurement of Wind Speeds:

Peak wind speeds are considered in the estimation of wind loads on building structures. Wind speed measurements are taken at the National Weather Service (NWS) facilities of the National Oceanic and Atmospheric Administration (NOAA), facilities of the Federal Aviation Administration (FAA), military bases and at other publicly and privately owned stations. Measured wind speeds depend upon instrument height, averaging time and the terrain that is upwind of the measurement site. During hurricanes, anemometers are often disabled at less than maximum wind speeds. Therefore, adequate data from which to predict maximum gust speeds during hurricanes is not widely available. Accordingly, mathematical simulations of hurricane winds are used to predict design wind speeds for hurricane prone regions.

2.3 Exposure:

Random variations in terrain, and the attendant mechanically induced turbulence, influence the wind speed near the ground so as to make it difficult to obtain wind speed measurements that are representative of large areas. Therefore, wind speed measurements are made in open terrain, i.e., airport exposure, at heights that are well above ground. Wind speed measurements are usually made at a height of 33ft (10m), although this may vary with the distribution of roughness elements at a measurement site.

2.4 Averaging Time:

Variations in measured wind speeds and directions are smoothed by any of several averaging techniques. The peak wind speeds used to calculate design wind forces on structures are averaged over periods of time ranging from 2 to 5 seconds. Because of variations in the response characteristics of measuring equipment, this averaging time actually ranges from about 1 to 5 seconds, for a median value of 3 seconds. In editions of ASCE 7 earlier than the 1995 revision, fastest-mile wind speeds were specified as basic design speeds. Part of a program to improve the forecasting capability of the NWS has involved phasing out fastest-mile wind speed measurements. Currently, three-second gust speeds are being archived (8) by the NWS. Figures 2.1 and 2.2 show the wind climatology of the United States, as defined by fastest mile wind speeds and 3 second gust speeds, respectively.

The contours of the fifty-year fastest-mile wind speed map for the eastern $\frac{3}{4}$ of the lower 48 states, figure 2.1, were unnecessary for the fifty-year 3-second gust speed

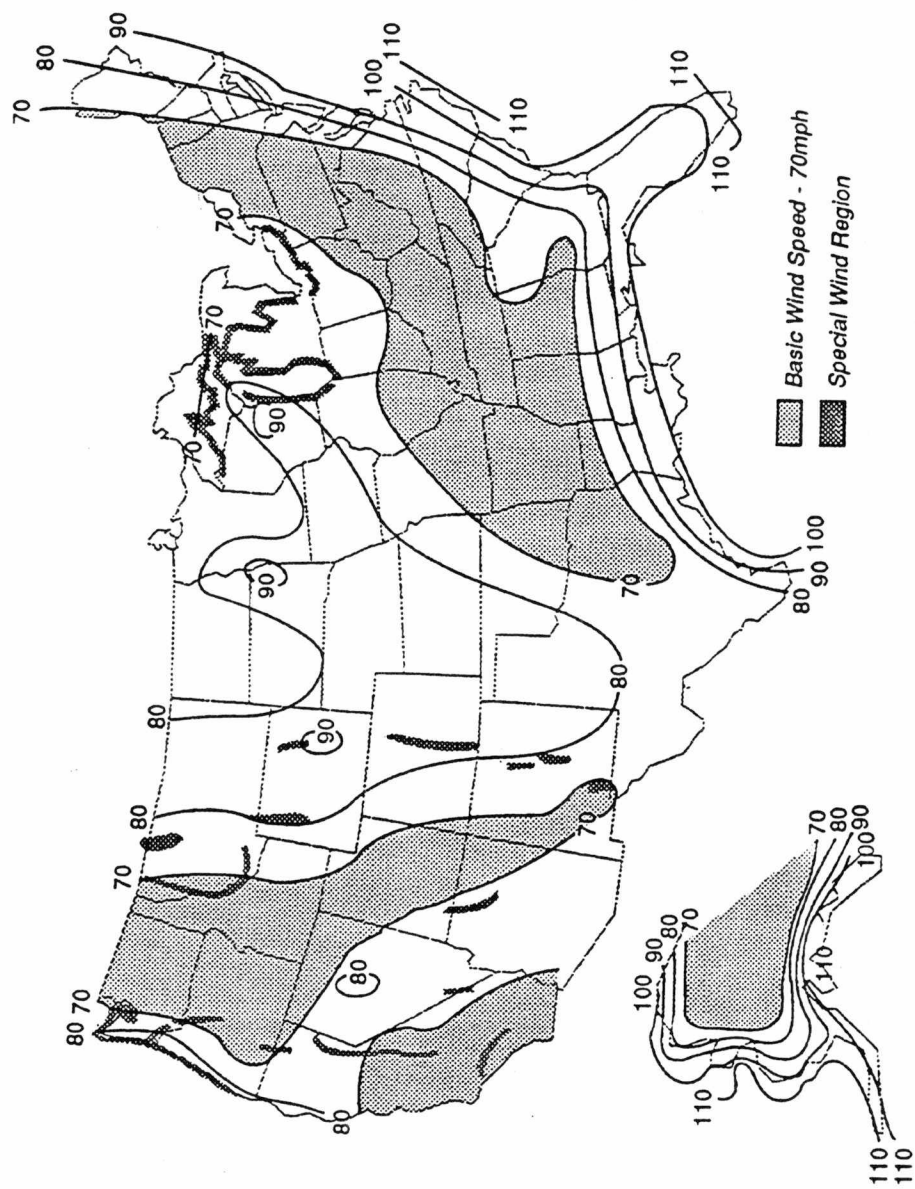


Figure 2.1: Fifty-year fastest mile wind speeds (mph) at 33-ft (10 m) above ground. Broadly hatched regions have 77-mph winds at mean recurrence intervals of less than fifty years. Special wind regions are subject to extremely hazardous wind conditions.
Source: ANSI (1982)

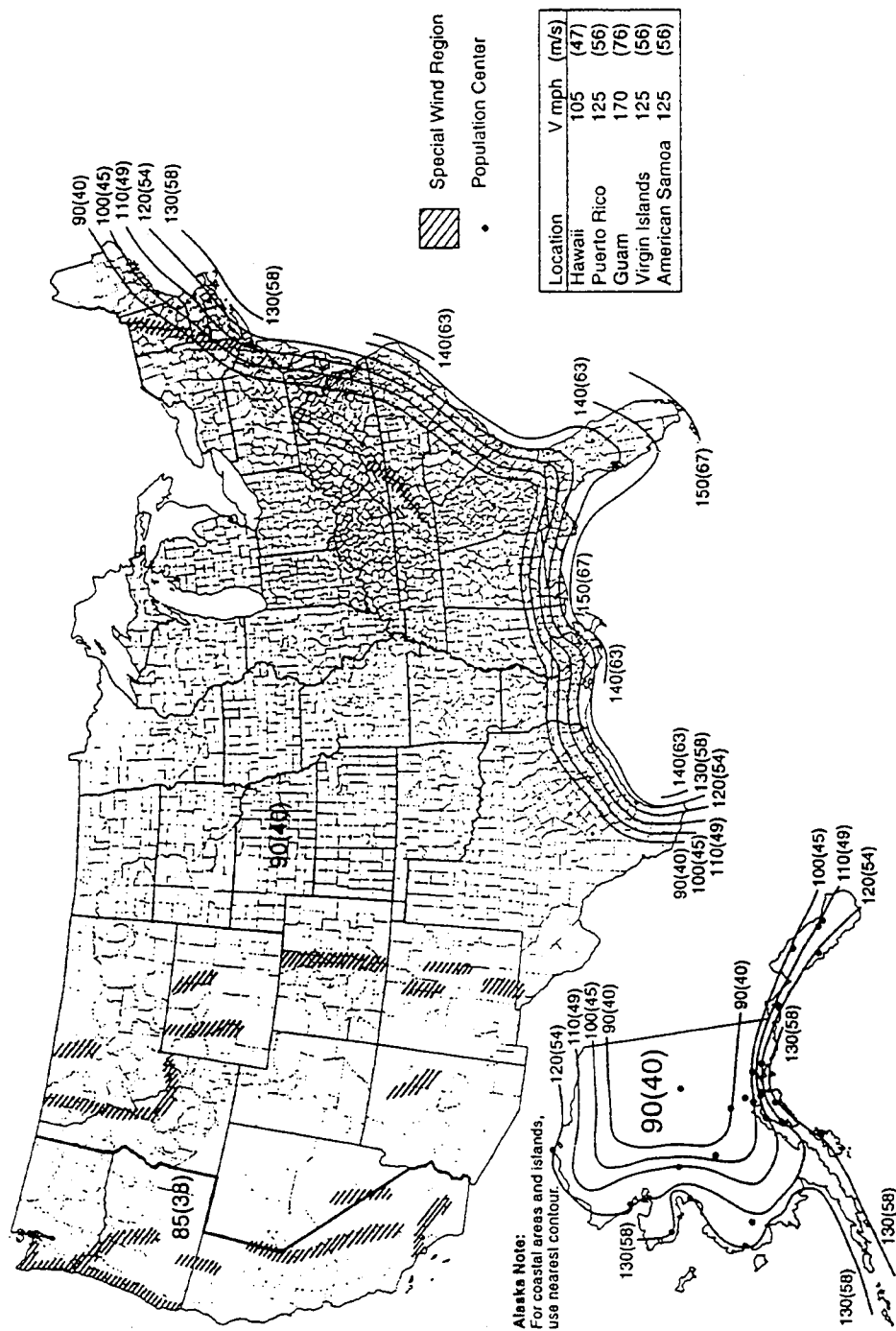


Figure 2.2: Three second gust speeds (mph) at 33-ft (10 m) above ground. Mean recurrence intervals of fifty years. Special wind regions are subject to extremely hazardous wind conditions (from 8).

map, figure 2.2. These contours were removed because of insufficient variation in fifty-year 3-second gust speeds in this region. Also, the 85 mph contour was near enough the state lines of California, Washington and Oregon to permit using them in lieu of a contour line as the division between 85 and 90 mph regions of the 3-second gust speed map. The increases in the wind speeds indicated for the interior of Alaska and the Aleutian Islands, figure 2.1 vs. figure 2.2, reflect differences between fastest-mile, and short duration gust speeds. The margin of difference between these maps for the Atlantic and Gulf coast regions reflects recent predictions of hurricane speeds using Monte Carlo numerical simulations of hurricane winds in these areas (9,10,11), and conversion to gust speeds using factors obtained from (12, 13). For the Pacific Northwest, where sufficient gust speed data was unavailable, three-second gust speeds were estimated by applying open country gust factors to the existing fastest mile wind speeds (8).

2.5 Comparability of Readings:

The comparability of readings taken in different locations is ensured by either (1) using standard averaging procedures at each station so that the 50-year speeds from each of them may be area averaged; or (2) taking samples from a very large area and fitting the data by an appropriate probabilistic model (1). The basic design wind speed is then specified according to a mean recurrence interval. Fifty-year winds are the basic wind speeds in the ASCE standard (8).

The second process may be described as follows. A series of the largest annual wind speeds (extreme values) of a well behaved climate, such as the non-hurricane regions of the country, may be analyzed statistically to obtain a reasonable prediction of

long term wind events throughout the entire region from which the data series was collected (2). Sampling errors are minimized by assembling data from several NWS stations in state sized areas (8). These data are fit using a type I probability distribution. Among the available probabilistic models, this distribution has been shown by Simiu, and Filliben in (14, 15, 16) to yield about the best representation of the probabilistic behavior of the largest annual wind speeds in a well behaved climate. The basic wind speeds obtained by this process are comparable to those obtained by area averaging the 50-year speeds from each station (process 1).

The basic design speeds published in ASCE 7-95 for inland regions were determined using a Fisher Tippet Type I probability distribution (61). A Weibull distribution, however, was used in determining the corresponding values for the hurricane prone Atlantic and Gulf coastlines. Although the associated risk factors are higher for the hurricane prone than inland wind zones, the importance factors published in ASCE 7-95 are the same for each of them, because their difference was incorporated in the contours of the basic wind speed map.

CHAPTER 3

MICROMETEOROLOGY

3.1 Exposure:

Wind flow is a complex phenomenon, any functional representation of which is necessarily one of many variables. Among these variables are the horizontal pressure gradient, the angular velocity of the earth, temperature gradients, and the roughness of terrain (2). To implement the provisions of the ASCE standard for evaluating design wind loads, only the last of these must be treated as a variable. The roughness of terrain is defined in the ASCE standard in terms of exposure categories. Representative exposure categories include downtown; suburban and wooded; open and coastal terrain (8). The greatest frictional resistance to wind flow, and hence the most gradual transition from the wind velocity near the earth's surface to that of the free stream outside of the earth's atmospheric boundary layer (ABL) is characteristic of the most sheltered terrain. Profiles of the longitudinal component mean wind velocity gradient, and rms wind speeds for several exposure categories are presented in figure 3.1.

3.2 Velocity Profile:

Shear stress in boundary layer wind flows is conventionally evaluated in terms of wind velocity, the roughness parameters of the terrain and the mass density of air. Csanady (18) defines a classical method of evaluating surface shear. This method is based on boundary layer theory, which involves finding a mathematical function that

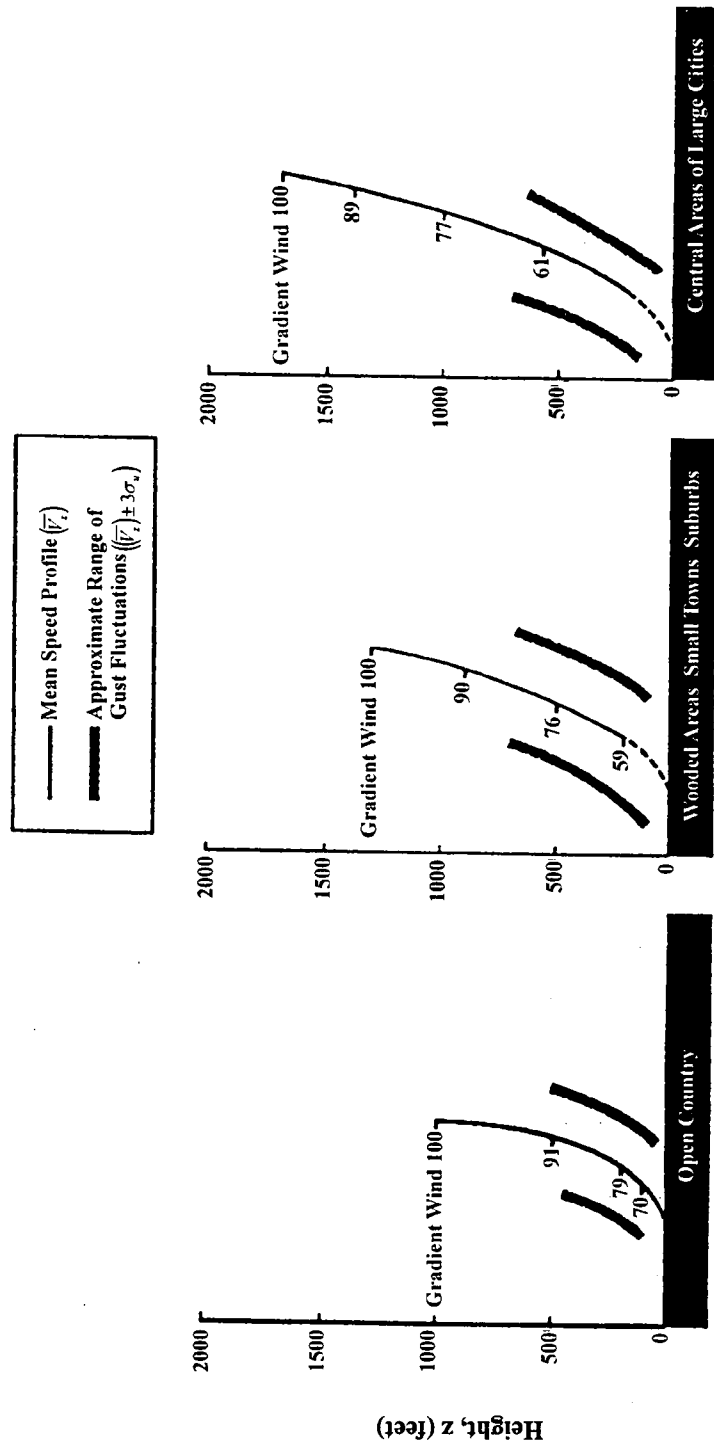


Figure 3.1: Mean longitudinal component wind velocity (mph) and turbulent fluctuations based on rms wind speeds (σ_u) (mph) for various terrain (from 1).

represents the motion in the viscous flow region near contact surfaces, and patching it to a functional representation of in-viscid outer-layer flow.

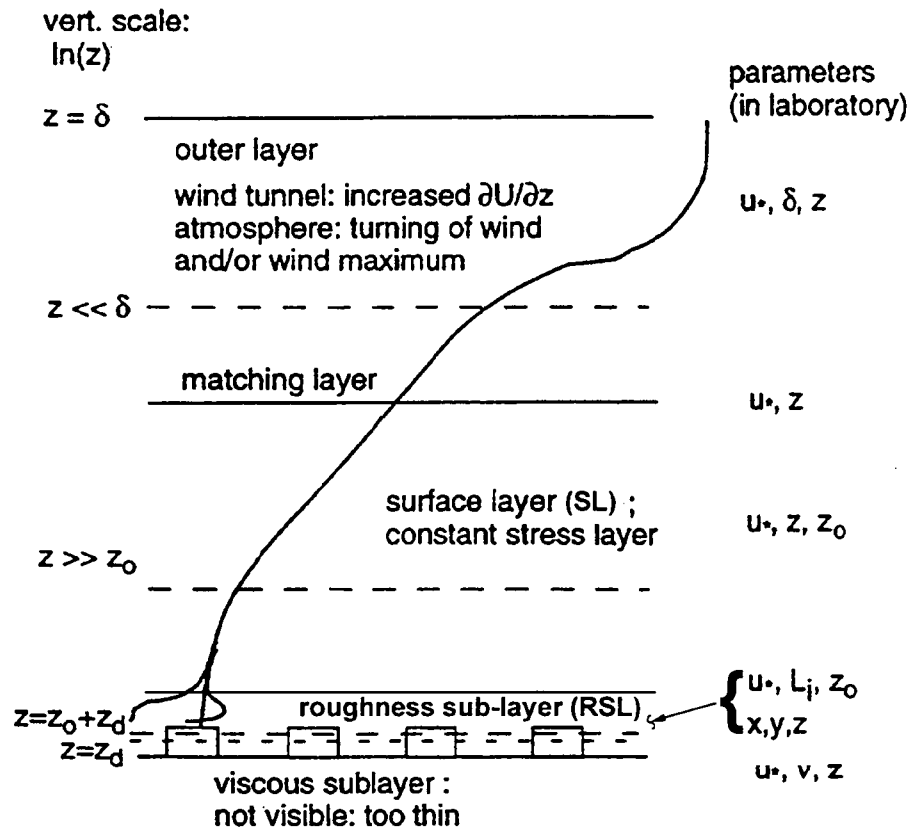
Actually, the flow conditions necessary for the application of boundary layer theory are not found in boundary layer wind flows. Foremost, a gradual transition between the flow characteristics of the viscous and inertial sub-layers is necessary for boundary layer theory to be successful. In the bluff-body flows typical of the ABL, viscous drag is dominated by pressure drag (17). This results in flow separation at bluff edges, and the attendant formation of turbulent wakes, which have no mean direction or velocity. Accordingly, functional representations of viscous and higher strata flow cannot be spliced. Also, although shear stress generally decreases with increasing altitude, the viscosity of the ABL is not entirely predictable (2). By approximating the viscosity of wind flow, and defining a profile displacement height, functional representations of the shear stress in the surface and outer regions of the ABL may be obtained. Boundary layer theory may be used to splice them.

3.3 Layer Structure:

Equipe Dynamique de l' Atmosphere Habitee, Laboratoire de Mecanique des Fluides

3.3.1 General Information:

The ABL may be broken into sub-layers as indicated in figure 3.2. This figure features a wind tunnel simulated mean velocity profile of an urban area. The horizontal in-homogeneity of the flow in the roughness sub-layer (RSL) (indicated by a "frayed" velocity profile) is characteristic of the flow separations that



Note: The "horizontal in-homogeneity" of the flow, indicated by "frayed ends" of the wind speed profile, actually describes the flow below the level where the mean flow begins to develop, i.e., between the zero plane displacement height (z_d), and the average elevation of the roughness elements (generally). This chaotic flow region is shown above its actual level to avoid cluttering the sketch.

Figure 3.2: Layer structure of wind tunnel boundary layer with parameters influencing $\partial U / \partial z$ in each layer. Notice the lack of spatial homogeneity in the lower part of the roughness sub-layer (from 17).

limit the applicability of boundary layer theory to ABL wind flows (2). The profile displacement height (z_d), also referred to as the zero plane displacement, demarcates the height above which the mean flow begins to develop. In general, it is slightly below the general elevation of the obstructions to wind flow in a given locality. Boundary layer theory may be applied to splice the functional representations of the flow in the surface and outer layers of the ABL (2, 17).

To the right of figure 3.2 the parameters are listed which influence the flow in each sublayer (17). Because viscous effects are dominated by pressure drag, the viscous sublayer is not discussed. In the RSL, each of the length scales (x, y, z) of the surface geometry can be important. The influence of these dimensions is apparent in the horizontal inhomogeneity of the flow in the lower region of the RSL. The length scales of the surface geometry are also related to systematic differences between the flow in RSL and the surface layer (SL). These systematic differences are related to longitudinal velocity fluctuations. One such systematic difference between these layers is that turbulent fluctuations are greater in the RSL than in the SL. Considerations of experimental data (19) suggest an estimate of the height of the RSL of about two times the roughness element height (17).

3.3.2 Surface Layer (SL):

In the SL the only remaining influential length scale is the roughness length of the terrain (z_o). In figure 3.2, the upper bound of the SL is indicated by $z \ll \delta$, where δ is the gradient height of the boundary layer. Throughout the SL, shear stress in the flow is

approximately constant, and the well known "law of the wall" is representative of the mean longitudinal component wind velocity profile.

$$\frac{\bar{U}}{u_*} \propto \left(\frac{z - z_d}{z_o} \right)$$

In this expression, $\bar{U}$ is the mean longitudinal component wind velocity; u_* is the shear "velocity"; z is the height above ground; z_d is the profile displacement height and z_o is the roughness length of the terrain. In the SL the constant of proportionality is a function of

$$\frac{z - z_d}{z_o} \quad (17). \text{ The shear velocity is written as } u_* = \left(\frac{\tau_o}{\rho} \right)^{\frac{1}{2}}, \text{ where } \tau_o \text{ is called the surface}$$

shear, and ρ is the mass density of air.

3.3.3 Outer Layer:

In the outer layer, the ratio $\frac{z}{\delta}$, where δ is the boundary layer thickness, is significant, and $\frac{z}{z_o} \rightarrow \infty$. Turbulent fluctuations in the longitudinal wind velocity are generally decreased, and shear stress in the flow decreases with height. In this part of the boundary layer "the velocity defect law" is representative of the mean longitudinal component wind velocity profile.

$$\frac{\bar{U} - U_g}{u_*} \propto \left(\frac{z - z_d}{\delta} \right)$$

The gradient wind velocity is represented by U_g . In the outer layer, the constant of proportionality is a function of $\frac{z - z_d}{\delta}$, and it may also depend on external factors such as

the horizontal pressure gradient (17). A slightly different expression of the velocity defect law applies to the ABL.

3.3.4 Overlap Layer:

The layer in which the "law of the wall" and the "velocity defect law" both apply is called the "matching," or "overlap" layer. In figure 3.2, it is delimited by

$z_o \ll z \ll \delta$. The existence of the matching layer is conditional of there being a gradual transition from the flow conditions of the surface layer to those of outer layer (17). The logarithmic overlap law applies to about 10 % of the ABL (2).

$$\overline{U}_z = \frac{1}{k} u_* \ln \left(\frac{z - z_d}{z_o} \right)$$

The Von Karman constant is denoted k . It is pointed out that z_o and z_d are parameters of the logarithmic velocity profile (2, 17). Therefore, if in an attempt to evaluate these roughness parameters, wind speed data taken from outside of the matching layer are fitted to the logarithmic law, then z_o and z_d will have no physical significance, i.e., they will only be curve fitting parameters. The reason for this, as figure 3.2 shows, is that length scales other than z_o influence the gradient $\frac{\partial U}{\partial z}$ outside of the matching layer.

The wind velocity profiles presented in ASCE 7-95 are power law profiles.

$$\frac{\overline{U}}{\overline{U}_g} = \left(\frac{z}{\delta} \right)^\alpha$$

However, the power law exponents specified by the ASCE are consistent with logarithmic law profiles in the lower 500 feet of the atmosphere (8).

3.4 Turbulence:

For the determination of wind loads, the ABL of a given locality is defined by the mean longitudinal wind velocity, and the turbulence characteristics; which include the spectrum of longitudinal velocity fluctuations, the longitudinal integral length scales of turbulence (LILST) and the longitudinal turbulence intensity (LTI) (29).

3.4.1 Energy Spectrum of Turbulent Motion:

The turbulence spectrum describes the distribution of energy within the ABL (2). This distribution of energy has been described as an “energy cascade.” Energy is transferred from relatively large, low frequency eddies, that have a high level of potential energy to smaller eddies having relatively high frequencies. Energy is dissipated by the smallest eddies of the flow at surfaces of contact.

3.4.2 Longitudinal Integral Length Scales of Turbulence (LILST):

The LILST have three components, including longitudinal, transverse and vertical (x, y, z) measurements of the magnitudes of gusts associated with longitudinal wind velocity fluctuations (u) (2, 5, 7). There are nine integral length scales of turbulence in all (2). These include longitudinal, transverse and vertical (x, y, z) magnitudes of gusts associated with wind velocity fluctuations acting in each of these directions (u, v, w). The LILST are of interest in the wind engineering of building structures. The along wind component (x) of the LILST (AWILST) produced by longitudinal wind velocity fluctuations u_1 and u_2 , occurring at points x_1 and $x_1 + x$, has been defined as:

$$L_u^x = \frac{1}{\sigma_u} \int_0^{\infty} R_{u_1 u_2}(x) dx$$

In this expression $R_{u_1 u_2}(x)$ is the cross covariance function of the longitudinal velocity fluctuations $u_1 \equiv u(x_1, y_1, z_1, t)$, and $u_2 \equiv u(x_1 + x, y_1, z_1, t)$; and σ_u is the rms value of u_1 and u_2 . It is noted that in a horizontally homogeneous flow, such as that of wind flow above the zero plane displacement (z_d), u_1 and u_2 are independent of x_1 and y_1 (2).

An expression of L_u^x as a function of height and constants relating to terrain type is available in the commentary of ASCE 7-95. The AWILST is used in ASCE 7-95 in conjunction with the calculation of gust effect factors. More specifically, the AWILST is used in the code to estimate the correlation of longitudinal velocity fluctuations (LVF) with respect to transverse and vertical building dimensions. This measure of the effect of LVF on the overall wind loading of buildings is referred to as the background response (Q) (8). The spatial correlation of LVF relative to large buildings is reduced, as compared to that of smaller buildings in a given wind event. Hence, the background response of larger buildings is reduced relative to that of smaller buildings. Therefore, calculated gust effect factors may be lower than the values permitted by the simplified method of ASCE 7-95.

A depiction of the extent to which longitudinal wind velocity fluctuations are correlated relative to a structural panel is shown in figure 3.3. These fluctuations are represented as vortices (whirlpools). The degree to which the LVF are correlated relative to the panel is indicated by the sizes of the vortices and their positions relative to one another. A large scale of turbulence is represented by the superposition of eddies to form a gust that envelops the panel shown in the figure. In this case, LVF significantly increase the overall loading of the panel. Similarly, holding the correlation of the LVF constant, their importance decreases as the panel size increases.

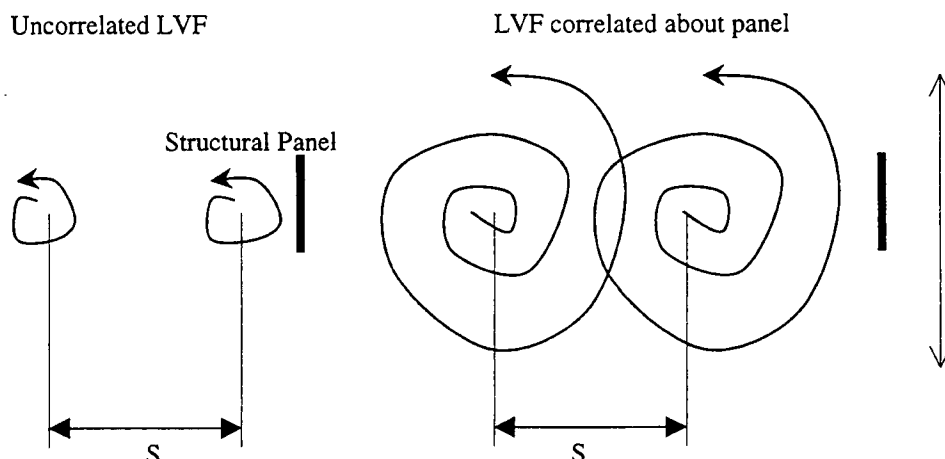


Figure 3.3: Illustration of small (left) and large (right) scales of turbulence relative to the breadth of a structural panel; background response.

3.4.3 Relative Longitudinal Turbulence Intensity (LTI):

LTI, denoted $I(z)$, is the ratio of the longitudinal component mean and rms wind velocities.

$$I(z) = \frac{\sigma_{u_z}}{\overline{U_z}}$$

LTI is greatest in sheltered terrain, where rms wind speeds (σ_{u_z}) are characteristically high and mean ($\overline{U_z}$) wind speeds are reduced (8, 20, 56). The balance of mass and energy in the flow requires these conditions. Although increased roughness reduces mean wind speeds, flow channeling, such as occurs in corridors between buildings, produces high rms wind speeds. These high rms winds increase the loading of smaller structures located opposite these corridors.

Depending upon the amount of shelter present, the effects of high turbulence intensity, such as channeling, are more or less significant. In suburban exposure, reduced

mean wind speeds as a result of increased roughness, as compared to open terrain, dominate channeling effects, and loading is reduced by about 15% (8). In metropolitan exposure, however, channeling effects can increase loading considerably, particularly where smaller structures are concerned. A conservative design might involve specifying loads for a relatively small structure cited in metropolitan exposure as though it was cited in open exposure. Such a design would be consistent with the low building provisions of ASCE 7-95.

3.5 Case Study:

Division of Building Technology, CSIR, Pretoria, S. Africa

A study was conducted to determine the influence of an approximately 155 m high ridge on the boundary layer characteristics of a city located 2.4 km downstream. The experiments were performed in a wind tunnel boundary layer using a geometric scale of 1:2000. The first series of tests was conducted without the city in the wind tunnel. These tests were performed to investigate the influence of the ridge upon the development to the boundary layer in the open terrain below it.

The spectra of longitudinal velocity fluctuations upwind of the ridge were evaluated at heights of 5 and 25 mm. In figure 3.4, these spectra are superimposed on the Harris-von-Karman reference spectra. A good simulation of the higher frequency velocity fluctuations in the approach flow was attained, as indicated by figure 3.4.

The ridge's obstruction to wind flow resulted in its acceleration at the ridge crest. This phenomenon is similar to channeling, described in section 3.4.3. The mass of a given volume of air approaching the ridge is constant. However, in the proximity of the

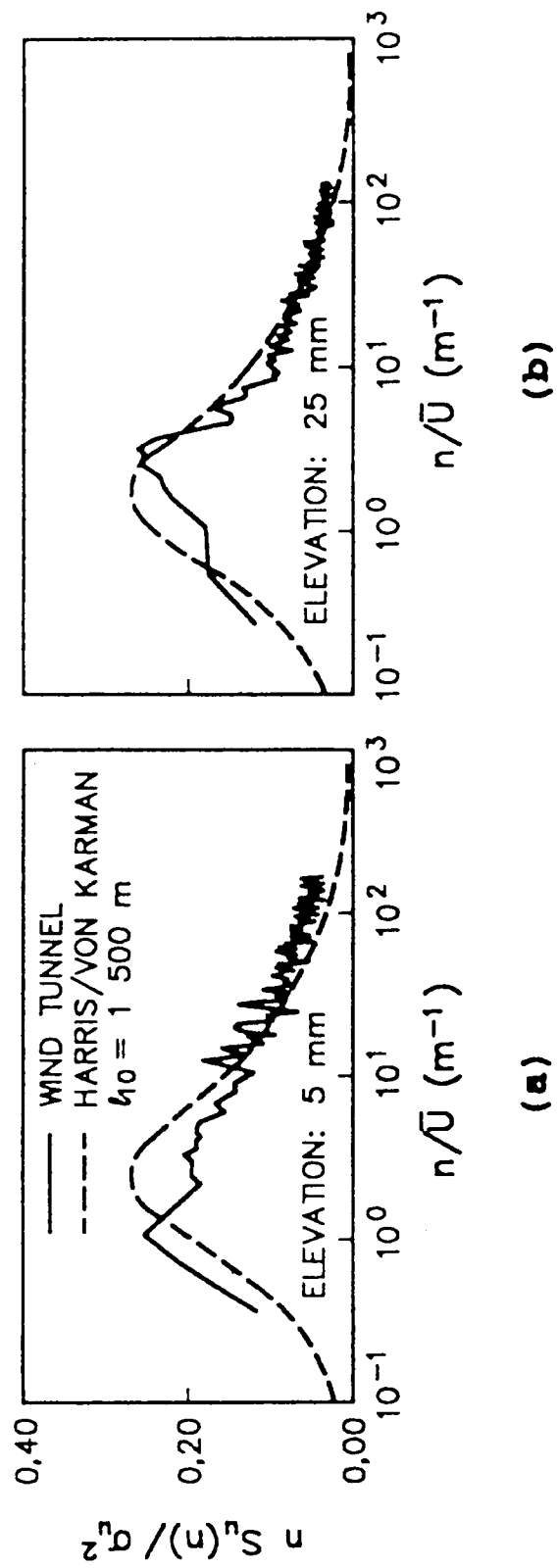
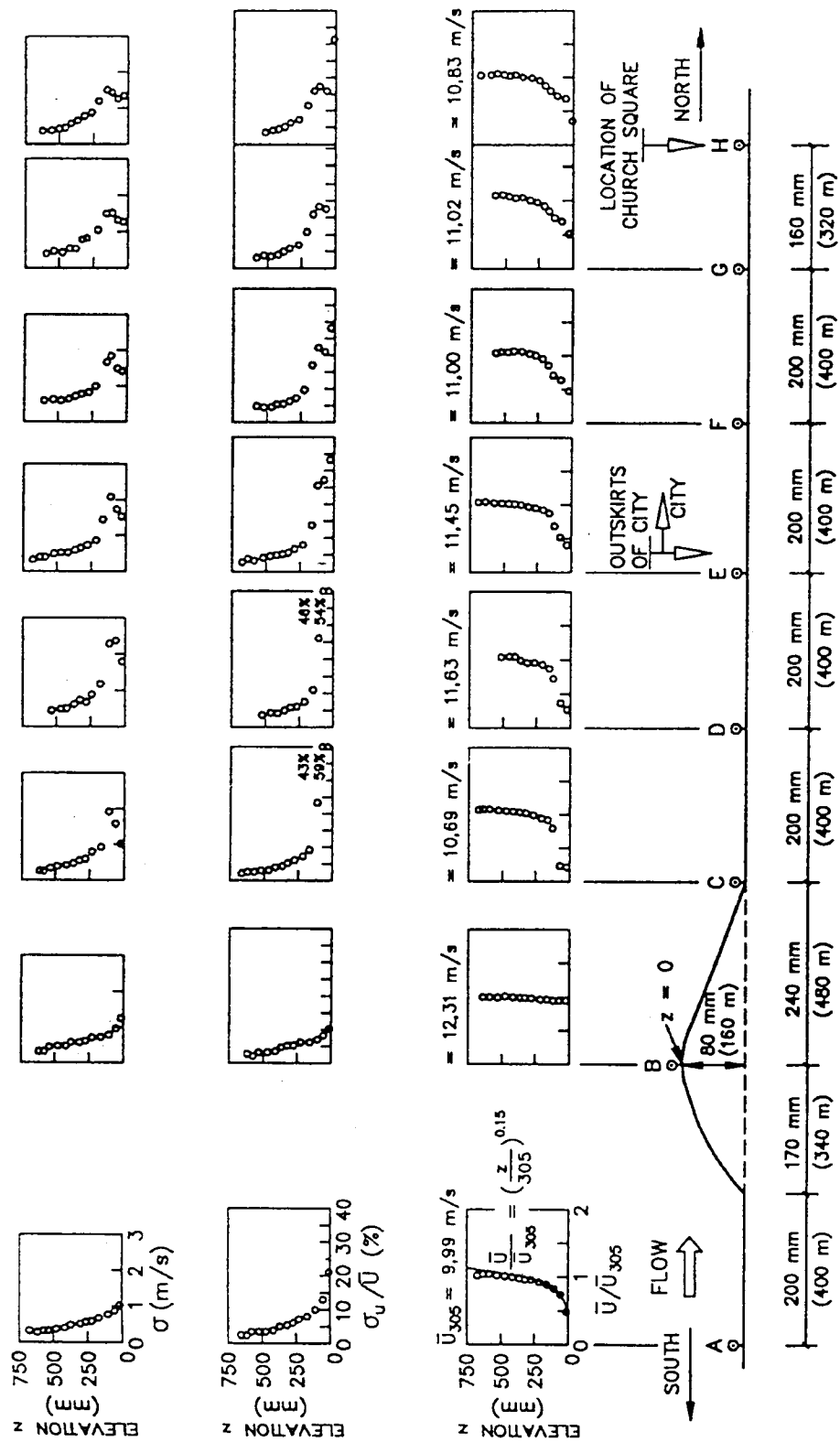


Figure 3.4: Longitudinal turbulence spectra in open terrain at elevations of (a) 5 mm and (b) 25 mm (form 20).

ridge, the ABL is confined to a smaller spatial volume. Therefore the wind's velocity must increase as it passes over the ridge. This effect was most pronounced at the ridge crest. The height at which the velocity profile became linear was about 305 mm. The mean longitudinal component wind velocity was referenced to this height. At point A the reference mean wind velocity $\left(\overline{U}_{305}^A\right)$ was about 10 m/s , as shown in figure 3.4. The corresponding value at the ridge crest, $\overline{U}_{305}^B$, was about 12.3 m/s . However, the velocity profile was vertical at this location. This indicated that the balance of mass and energy required the wind to approach its gradient velocity at the ridge crest.

Vortices induced in the flow at the ridge crest did not significantly increase the rms wind speed, denoted σ in figure 3.5, at their point of entry (point B). Accordingly, the LTI, denoted $\sigma_u/\overline{U}$ in figure 3.5, was markedly reduced at the ridge crest, because of the much-increased mean wind velocity at that location. The wind decelerated in the open terrain below the ridge. This phenomenon was most apparent at the base of the ridge, point C, where the LTI reached 59%. It was evident, however, as far downstream as station H, corresponding to the location of the church square. Within the velocity deficit region, the maximum rms wind speed was recorded at higher elevations at each successive station further downstream, stations C-H. The wafting of vortices from the wake of the ridge into the boundary layer downstream produced this trend (20).

A refined study of the ridge's influence on the downstream flow was undertaken using data taken in the terrain below the ridge at intervals of just 50 mm. Horizontally, the first of these measurements were taken 50 mm upstream of point C, and the last of



NOTE:
VERTICAL SCALE EXAGGERATED

Figure 3.5: Influence of upwind hill on mean wind speed and turbulence profiles developing in open terrain (from 20).

them at point *E*. At each station, data were collected at heights of 10 and 50 mm, or 20 and 100 m, full-scale.

Foremost, the spectrum of longitudinal velocity fluctuations was evaluated in the flow approaching the ridge, as well as in the flow downstream of the ridge. The frequency ranges in each of these spectra were equally wide. However, the spectrum of the downstream flow was shifted toward the higher frequencies. This was most apparent within about 200 mm (400 m full-scale) of the base of the hill (point D); where the profiles of the length scales of turbulence, $L_{u_{10}}/L_{u_{10}}^A$ and $L_{u_{50}}/L_{u_{50}}^A$, had values of less than 0.5, as shown in figure 3.6. In the wake of the ridge, the rms wind speed increased sharply. Maximums were recorded at about 100 mm (200 m, full-scale) below the base of the ridge. However the mean wind speed of the downstream flow was significantly reduced relative to the upstream flow as far downstream as the station corresponding to the maximum rms wind speed. This is indicated by the profiles $\bar{U}_{10}/\bar{U}_{750}^A$ and $\bar{U}_{50}/\bar{U}_{750}^A$, and their reference values $(\bar{U}_{10}/\bar{U}_{750})^A$ and $(\bar{U}_{50}/\bar{U}_{750})^A$, shown in figure 3.6.

Accordingly, the intensity of turbulence in the flow downstream of the ridge relative to that of the upstream flow was increasingly high to the 100 mm mark. This is indicated by the profiles $\sigma_{u_{10}}/\bar{U}_{750}^A$ and $\sigma_{u_{50}}/\bar{U}_{750}^A$, together with their reference values, $(\sigma_{u_{10}}/\bar{U}_{750})^A$ and $(\sigma_{u_{50}}/\bar{U}_{750})^A$, also shown in figure 3.6.

A comprehensive indication of the ridge's influence on the downstream flow was provided by the longitudinal turbulence spectra of the boundary layer, with and without the presence of the ridge in the wind tunnel. These spectra were measured at elevations

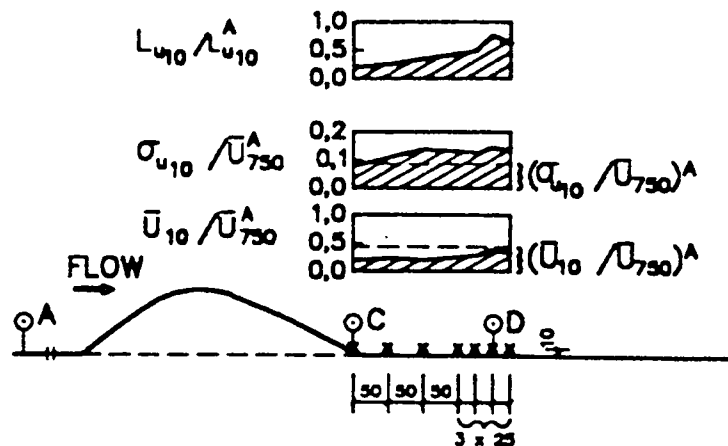
ELEVATION 10 mm

$$\bar{U}_{750}^A = 12,5 \text{ m/s}$$

$$\bar{U}_{10}^A = 5,5 \text{ m/s}$$

$$\sigma_{u_{10}}^A = 1,1 \text{ m/s}$$

$$L_{u_{10}}^A = 0,8 \text{ m}$$



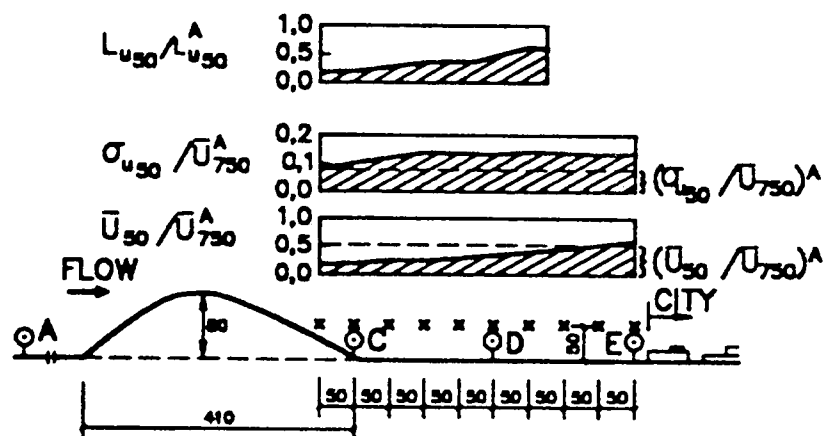
ELEVATION 50 mm

$$\bar{U}_{750}^A = 12,5 \text{ m/s}$$

$$\bar{U}_{50}^A = 6,3 \text{ m/s}$$

$$\sigma_{u_{50}}^A = 1,02 \text{ m/s}$$

$$L_{u_{50}}^A = 1,1 \text{ m}$$



NOTE: DIMENSIONS IN mm

Figure 3.6: Variations in flow parameters behind the hill at elevations of 10 mm and 50 mm (from 20).

of 10 and 25 mm, or 20 and 50 m, respectively, full-scale. The spectrum at 25 mm, with the ridge present, was shifted toward the higher frequency range, as shown in figure 3.7.

In the next series of tests, the city was situated in the open terrain below the ridge. The roughness length of the city (z_o) was 0.3 m. This is about 30 times that of open terrain (0.01 m) (8). The influence of increased roughness was evident in the resulting boundary layer profiles, shown in figure 3.8. Typical of a city, the wind structure was characterized by high rms wind velocities. The greatest of the rms wind speed data were grouped toward higher elevations in figure 3.8. This was primarily the result of the increased roughness length of the terrain, although the influence of the ridge on the turbulence spectrum contributed to this nominally (figure 3.7 (b)). Also, the mean wind velocity was much reduced in RSL. This was particularly evident close to the church square (stations 0 and 1), as shown in figure 3.8. Accordingly, the turbulence intensity data were grouped above and to the right of that of the open terrain, particularly near these stations.

Figure 3.9 shows simultaneous plots of the boundary layer characteristics of the city, with and without the ridge present in the wind tunnel. The parameters were measured at elevations of 10 mm, 20 mm and 50 mm, (20 m, 40 m and 100 m full-scale). Because very high intensities of turbulence develop in the lower regions of a city boundary layer, the data taken at 10 mm (20 m full-scale) were subject to error. This plot was useful, however, for identification and comparison of trends.

Beginning in the outskirts of the city, at an elevation of 50 mm, a systematic trend was apparent. Without the ridge, the mean wind speed was higher, and the rms wind velocity was lower. Therefore, the intensity of turbulence was lower. At the 50 mm

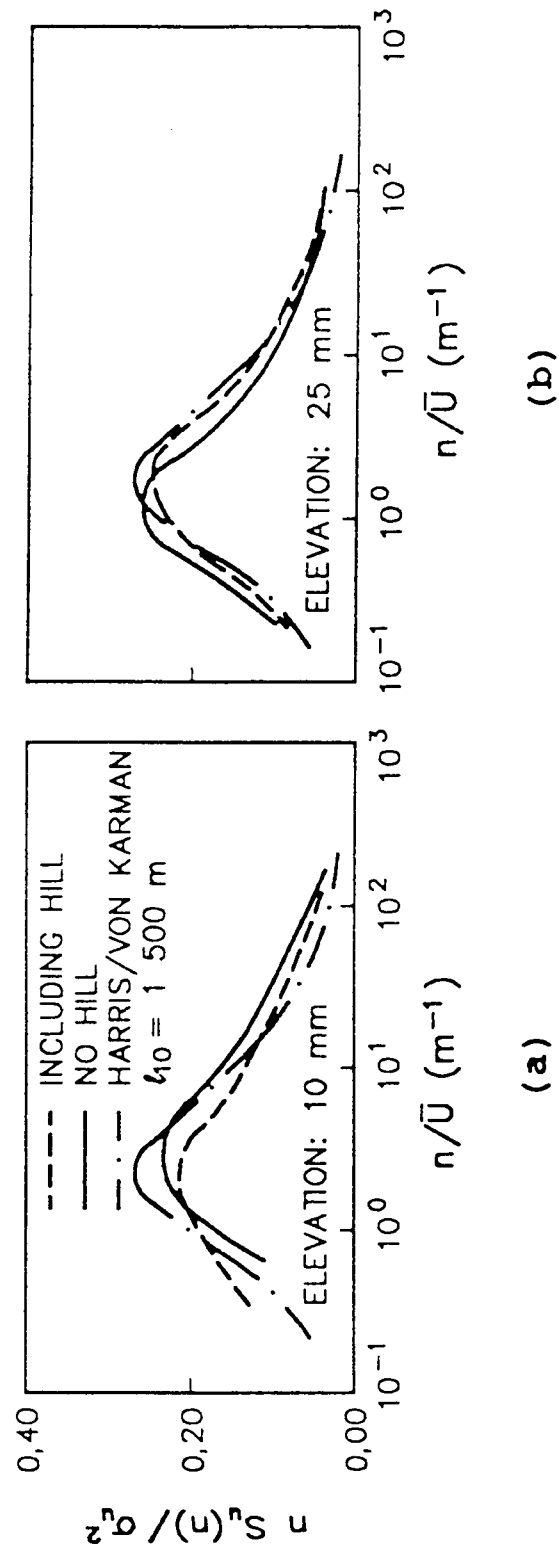


Figure 3.7: Comparison of longitudinal turbulence spectra in open terrain with and without the presence of the ridge in the wind tunnel at elevations of (a) 10 mm and (b) 25 mm (from 20).

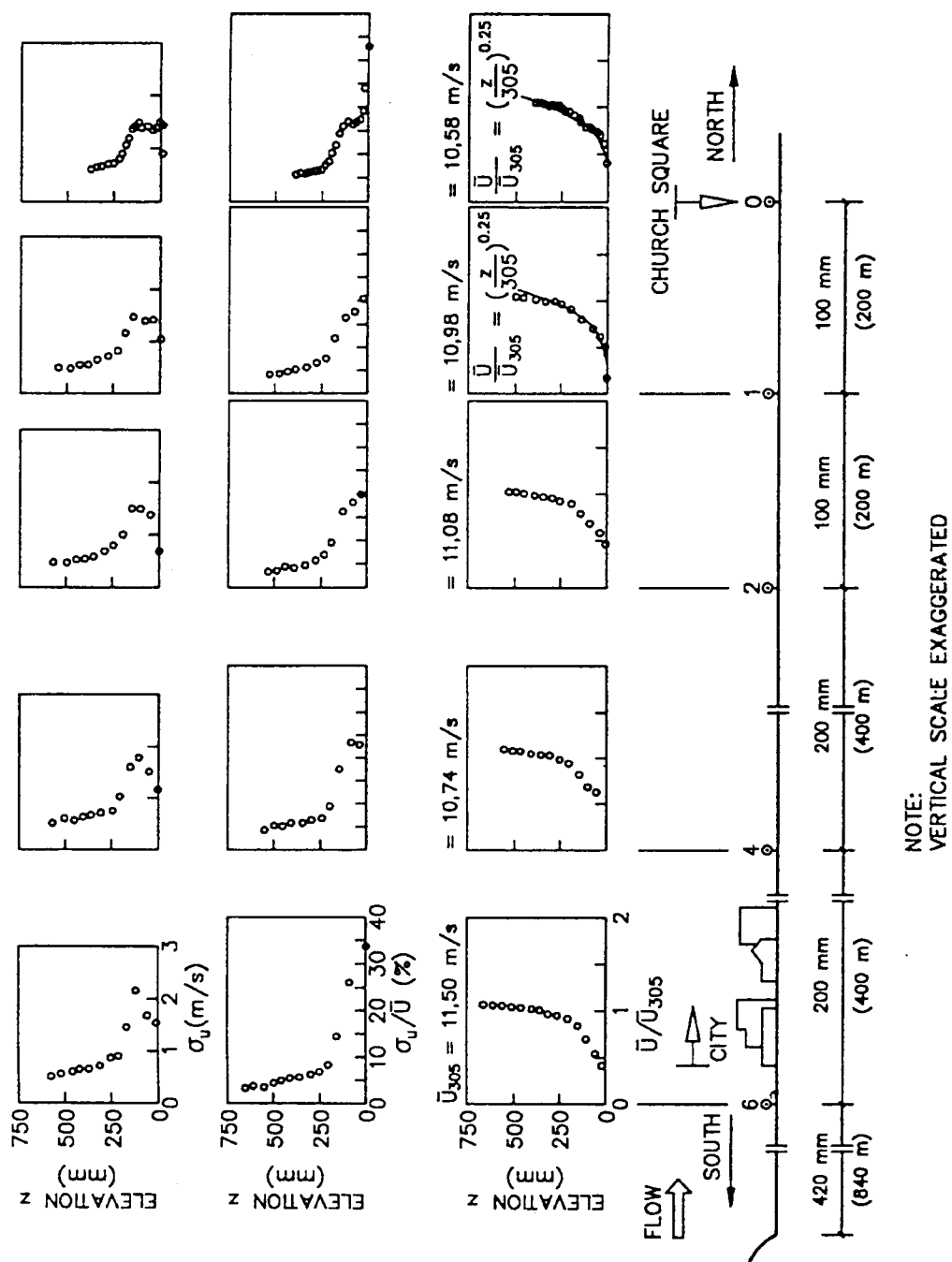


Figure 3.8: Development of mean wind speed and turbulence profiles in city terrain influenced by ridge (from 20).

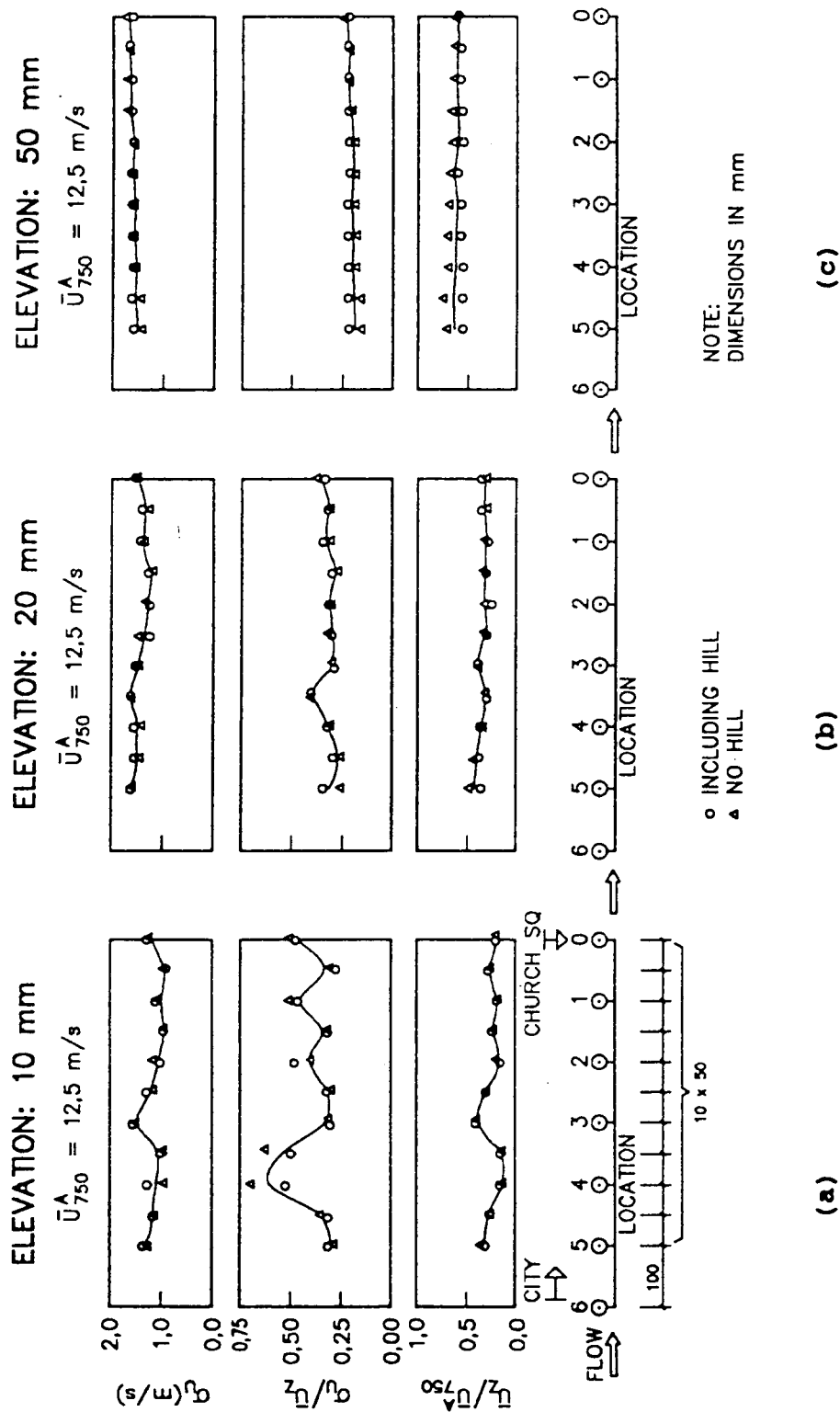


Figure 3.9: Influence of upwind ridge on mean wind and turbulence within city at elevations of (a) 10 mm, (b) 20 mm and (c) 50 mm (1:2000 scale) (from 20).

elevation, this trend extends from the outskirt, 300 mm into the city (600 m full-scale), i.e., between locations 5 and 2 in figure 3.9. At the 20 mm elevation, this trend was apparent only in the outskirts of the city. This systematic trend was not apparent anywhere in the lowest region of the boundary layer (20).

It was concluded that although the ridge had a noticeable influence on the wind structure developing below it in open country, it had no significant effect on the boundary layer characteristics of the city (20). More specifically, turbulence in the RSL of the city was not significantly increased by the presence of the ridge in the upstream flow. Accordingly, the boundary layer that developed in the city did so in accordance with local roughness characteristics.

CHAPTER 4

WIND TUNNEL MODELING OF ABL WIND FLOWS AND WIND EFFECTS ON LOW-RISE BUILDINGS

4.1 General Information:

The practice of modeling ABL wind flows has evolved over the last 36 years (1). The first boundary layer wind tunnel (BLWT) was built in 1961 at Colorado State University. A schematic drawing of this boundary layer wind tunnel is shown in figure 4.1. Boundary layer wind tunnels are especially designed for modeling the wind velocity gradients and turbulence characteristics of ABL wind flows. For this purpose BLWT's are equipped with long working sections, generally between 10 and 30 meters in length. Aeronautical wind tunnels and BLWT's are distinguished by this feature; i.e., a working section of only about 5 meters in length is necessary to model smooth aeronautical flows, whereas longer working sections are required to model the velocity gradients and turbulence characteristics of boundary layer wind flows.

In the United States small commercial and industrial buildings, as well as single and multiple family dwellings make up the vast majority of the construction industry. The cost of these structures is low relative to the cost of wind tunnel modeling. Therefore, building codes are used to define design wind loads for these structures. Because full-scale tests are very costly, wind tunnel data obtained on a research basis is

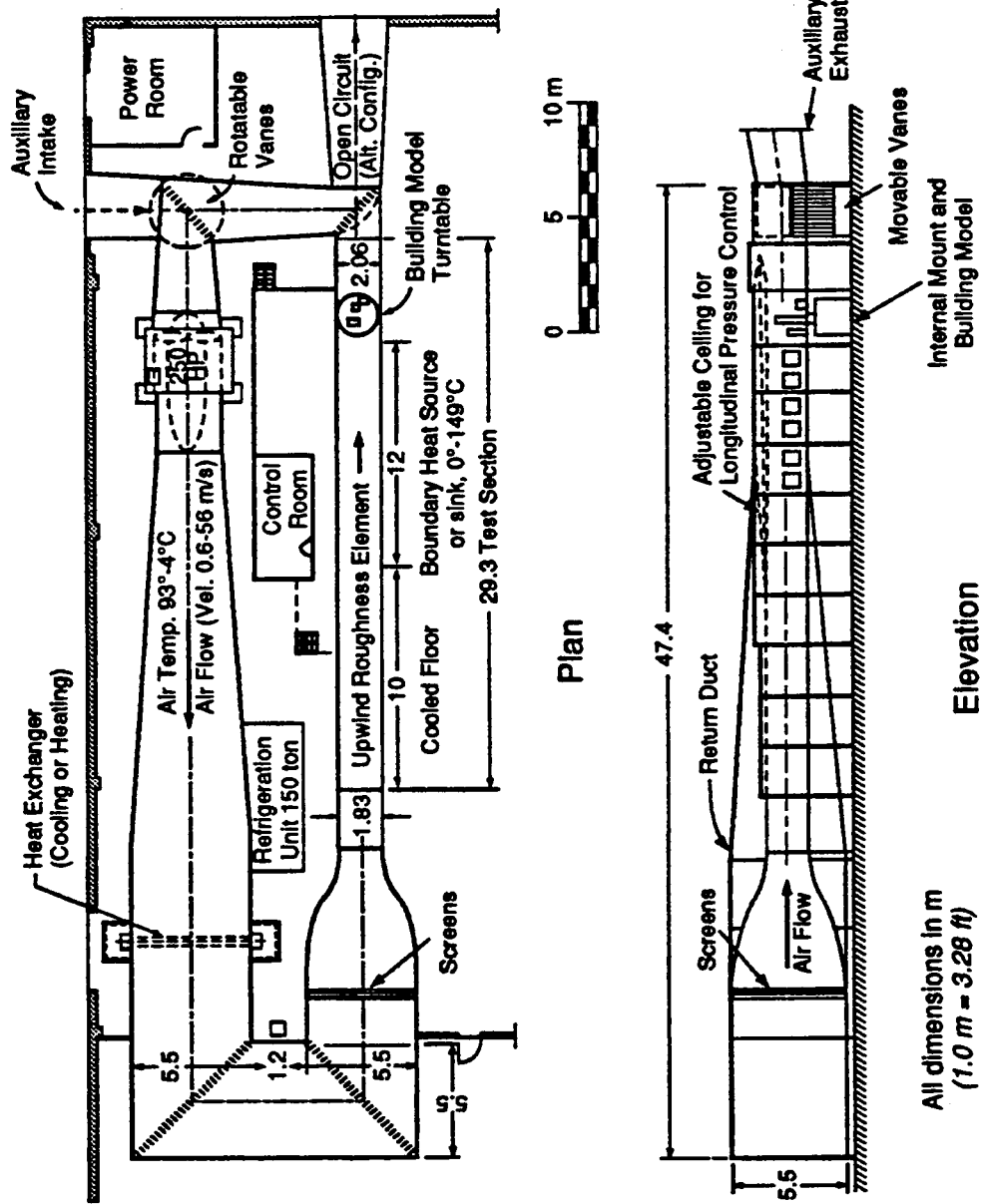


Figure 4.1: Schematic of a boundary layer wind tunnel (from 1).

used to develop the methodology of determining design wind loads that are presented in building codes (1). Furthermore, codification of research data necessarily involves generalizations that impute to the finished building code a degree of arbitrariness, concerning particular loading cases. Therefore, BLWT studies are frequently undertaken to substantiate values of design wind loads that have been determined in accordance with the statutes of building codes (28).

4.2 History:

Early research (1930's) indicated that wind effects on building structures cannot be accurately simulated in uniform flow wind tunnels (UFWT's) (3). In 1936 Bailey, Irminger and Nokkentved compared field measurements of pressure distributions on buildings to pressure distributions on model buildings in UFWT's. They found that the full and model-scale data were disparate. In 1943 Bailey and Vincenta compared the full and model-scale data of the 1936 comparison to the pressure distribution on a model building in a turbulent boundary layer. These data were in considerable agreement.

Jensen's research of the 1950's and early 1960's indicated conclusively that pressure distributions on buildings depend upon, for one thing, the size of the building relative to the roughness length of the terrain (21, 22). The ratio $\frac{H}{z_o}$ is the descriptor of the relationship that Jensen observed, in which H is a characteristic building dimension, and z_o is the roughness length of the terrain. This parameter is often called the Jensen number. The roughness length of the terrain is proportional to the size of mechanically

induced eddies; i.e., eddies induced in the flow as a result of contact between wind flow and earth's surface structure. Just as increases in wind loading that are associated with turbulence in the approaching wind, increased loading resulting from mechanically induced fluctuations in wind speed are also proportional to the sizes of eddies (refer to figure 3.8). Accordingly, decreasing Jensen numbers, as well as increasing LILST, are associated with increasing peak and rms pressure coefficients (5). Other than Jensen, early proponents of modeling wind as a turbulent flow included Strom at New York University and Cermak at Colorado State University (3).

Despite the evidence that wind should be modeled as a turbulent flow, researchers were pessimistic about the potential for accurately simulating ABL flows. Furthermore, mean pressure coefficients obtained from data taken in UFWT's were excessive, and therefore provided conservative predictions of wind loads on buildings. Therefore, researchers continued to predict wind loads on buildings from data obtained in studies that involved uniform flow fields until the middle of the 1970's.

4.3 Current Practice:

The importance of capturing the mechanical as well as natural turbulence structure of wind flow to simulate pressure distributions on buildings is now well recognized among researchers. The perception that mean pressure coefficients obtained from UFWT studies were excessive, and therefore provide conservative predictions of wind loads on buildings was superseded by the results of BLWT studies. It has been shown that significant increases in wind loading result from peak pressures produced by turbulent

fluctuations in wind velocity, and that these increases in loading are not indicated by associated mean pressure coefficients (5, 31, 32, 33, 34, 35, 38, 39).

4.4 Simulation of Wind Flow:

Wind tunnel operating speeds vary between approximately 10 and 50 m/s (22 and 112 mph) (28). Model wind speeds are a function of the speed of the wind tunnel fan.

Changes in the wind direction are achieved by rotating the model in the wind tunnel.

Accurate simulation of wind flow requires that the model wind must represent the actual wind with respect to its velocity gradient, and turbulence characteristics, including the scales and spectra of turbulence, and the longitudinal turbulence intensity (29).

Development of requisite turbulence and velocity profiles is facilitated by the provision of a long test section containing roughness elements that are characteristic of the terrain surrounding the test structure.

The length of the working section of a wind tunnel needed to develop the turbulence characteristics of a full-scale wind can be minimized by situating special roughness elements, and or barriers upwind of the model to accelerate the growth of the boundary layer. For example, a large-scale (1:150) open country boundary layer may be developed over just 10 meters by placing large blocks at the entrance of the wind tunnel's working section, and in its collector, and providing carpeting along its length (29). Such an arrangement is shown in figure 4.2. The blocks shed vortices that enrich the low frequency range of the turbulence spectrum. Accordingly, the scales of turbulence required to accurately simulate the turbulence characteristics within the surface layer of

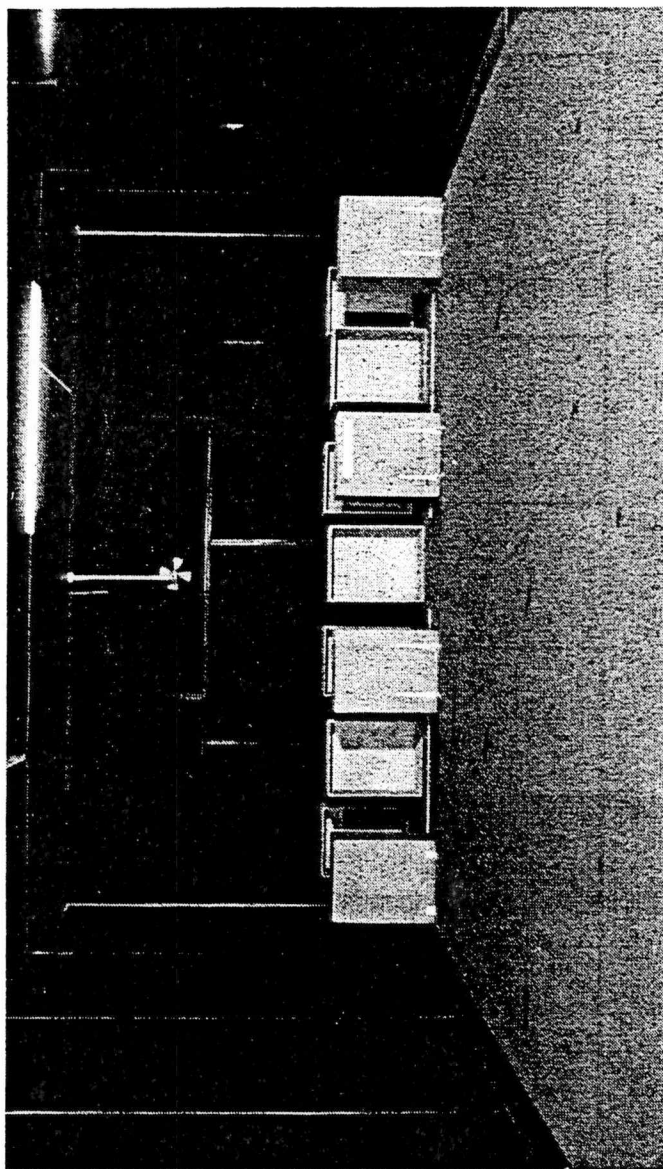


Figure 4.2: The large blocks are placed to generate vortices that enrich the lower frequency range of the turbulence spectrum in the wind tunnel test section (from 29).

the full-scale wind may be developed over a short fetch. A relatively long fetch would be required to develop the requisite scales of turbulence if the boundary layer were naturally developed. (A naturally developed boundary layer is one that has been developed without the use of special roughness elements, such as those shown in figure 4.2, to accelerate its growth.)

4.5 Proximity Models:

Vortices shed from neighboring structures, i.e., signature turbulence, can produce critical loading effects (5, 8, 28). Proximity models are used to capture the influence of buildings and other structures within the near field of a test building upon its wind loading (28). Structures within 1500 to 2500 feet of a test building are considered to occupy its near field. Prominent structures and topographic features outside the near field of the test building may also be influential in determining its wind loading. Future construction, as well, may influence the wind loading of the test building, and should therefore be included in proximity models. Furthermore, the effects of the test building on the wind loading of existing structures must be assessed as well.

Generally, the degree of accuracy required in proximity models decreases with increasing distance from the test structure. More specifically, the structures closest to the test building should be modeled in considerable detail. However, structures that are not in the immediate proximity of the test structure may be represented by blocks (28). The geometrical characteristics of the exteriors of structures in the immediate proximity of the test building are important, because they influence the channeling and waking effects of

those structures; which are important in determining the turbulent fluctuations in the wind loading of the test building.

4.6 Geometric Scaling Laws:

The accuracy obtained in wind tunnel simulations of pressure distributions on buildings depends upon certain scaling laws. These scaling laws are defined by the parameters $\frac{H}{z_o}$, $\frac{H}{\delta}$, $\frac{H}{L_u}$ and $\frac{\rho \bar{U} H}{\mu}$; where H is a characteristic building dimension, and z_o , δ , L_u , ρ and μ are the roughness length of the terrain, the gradient height of the ABL, the LILST, and the fluid mass density and dynamic viscosity, respectively. The scaling laws are satisfied if these parameters are consistent between full and model scales. A study of the influence on wind tunnel test results of inequality between the full and model scale values of the scaling parameters was conducted at James Cook University (5).

4.7 Case Study:

James Cook University of North Queensland, Australia (JCU)

4.7.1 Description of Distortion Types:

Tests were performed on model low-rise buildings in the BLWT at JCU to determine the effects of three types of distortion upon the total wind forces acting on the building's surfaces. Each of these types of distortion is common in wind tunnel modeling. They include (1) distortion in the Jensen number; (2) disproportionately small simulated LILST, although geometric scaling is consistent between full and model scales;

and (3) distortion resulting from the placement of a disproportionately large model building into an otherwise correctly simulated boundary layer. These three types of distortion are characterized by violation of the scaling laws $\frac{H}{z_o}$, $\frac{H}{L_u}$, and $\frac{H}{\delta}$, respectively.

In the interest of obtaining the greatest accuracy in peak pressure measurements, some of the pressure taps are placed as near as possible to such locations as corners, ridges and along the edges of roofs. These locations are particularly important in the determination of peak wind loads, because the shedding of vortices from them exerts intense suction on components (secondary framing members) and cladding. To facilitate placing pressure taps in the proximity of these critical locations, large-scale models must be employed. This may result in the first, and or third case scenarios, in which the model is disproportionately large in comparison to the roughness length of the terrain, and or the height of the model boundary layer, respectively.

The second type of distortion results if the scales of turbulence in the full-scale wind are not accurately simulated. While the model and the boundary layer thickness may be commensurately scaled with respect to geometry, and the Jensen number also correct, the LILST of the model wind are disproportionately small relative to the full-scale LILST. Consequently, gusts of the magnitude necessary to produce peak wind pressures that are representative of the full-scale wind are not developed in the wind tunnel. It is noted that enriching the low frequency range of the turbulence spectrum by a means such as that illustrated in figure 4.2 is one way of correcting this situation.

The first type of distortion, inconsistent Jensen scaling, would result if, for example, a poor proximity model were used; i.e., the near field of the test structure were not replicated to a reasonable degree of accuracy. Failing to accurately model the roughness characteristics of the terrain, or improper scaling of the test structure are other potential causes of inconsistency between full and model-scale Jensen numbers.

4.7.2 Experimental Details

A sketch of the model featuring its dimensions, the roof angle and the azimuth of the model wind is shown in figure 4.3. Although scaling of the flow in tests of quasi-static structures is usually limited to the turbulence and velocity profiles, and the longitudinal component of the LILST (L_u^x), Surry has suggested in (30) that the lateral component of the LILST (L_u^y) is the most important of the LILST in determining total loads. Therefore L_u^y was used in scaling the turbulence in the model boundary layers of these tests.

As a preliminary step, the effects on wind loading of varying the Jensen number, as well as the ratio $\frac{L_u^y}{W}$ were investigated. Assessments of these variations were made according to the mean, peak and rms total force coefficients, which are $C_{\bar{F}}$, $C_{\hat{F}}$ and $C_{\check{F}}$, and $C_{\bar{M}_F}$, respectively. The force coefficients were computed by normalizing the force data according to the product of the hatched areas in figure 4.3 and the mean dynamic pressure at eave height (q_h).

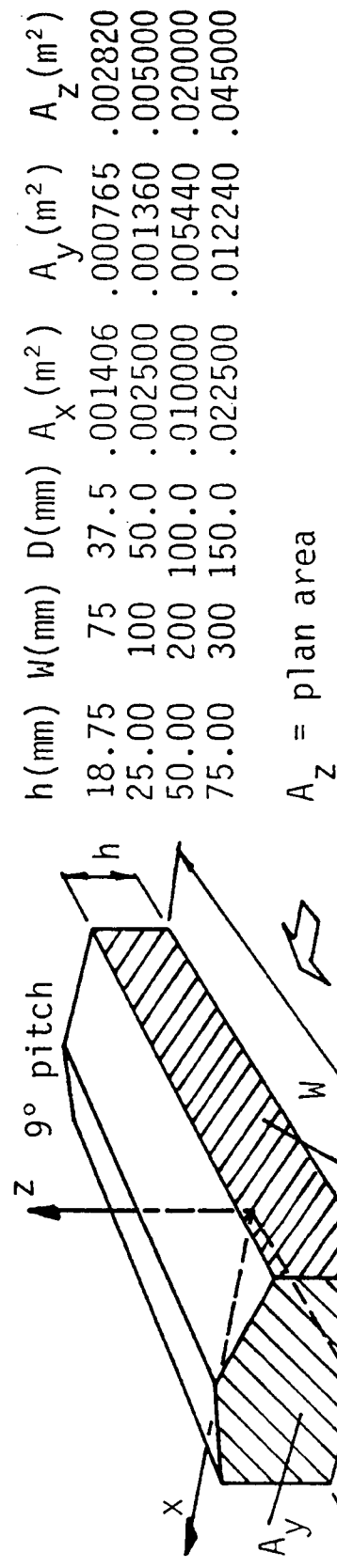


Figure 4.3: Model low-rise building used at JCU in the research of the effects of geometric and boundary layer scale distortion upon total wind loads (from 5).

These force coefficients are:

$$C_{\hat{F}_x} = \frac{\max \int_{A_x} (p_x(t) - \overline{p_x}) dA_x}{q_h A_x}, \quad C_{\overline{F}_x} = \frac{\frac{1}{T} \int_0^T \int_{A_x} p_x(t) dt dA_x}{q_h A_x}, \quad C_{F_x} = \frac{\left[\frac{1}{T} \int_0^T \int_{A_x} (p_x(t) - \overline{p_x}) dt dA_x \right]^{\frac{1}{2}}}{q_h A_x}$$

$$C_{\hat{F}_z} = \frac{\max \int_{A_z} (p_z(t) - \overline{p_z}) dA_z}{q_h A_z}, \quad C_{\overline{F}_z} = \frac{\frac{1}{T} \int_0^T \int_{A_z} p_z(t) dt dA_z}{q_h A_z}, \quad C_{F_z} = \frac{\left[\frac{1}{T} \int_0^T \int_{A_z} (p_z(t) - \overline{p_z}) dt dA_z \right]^{\frac{1}{2}}}{q_h A_z},$$

where p is the temporal mean surface pressure with respect to the static reference

pressure, and q_h is the mean dynamic pressure at eave height $\left(\frac{1}{2} \rho \overline{U}_h^2 \right)$.

4.7.3 Results:

The mean force coefficients, as shown in figure 4.4, varied little with natural and mechanically induced LVF, which are represented by the ratios $\frac{L_u^y}{W}$ and $\frac{h}{z_o}$, respectively.

However, the effects of the turbulent fluctuations indicated by these parameters were apparent in rms and peak force coefficients. Accordingly, inconsistency between full and model scale values of the Jensen number, or the ratio $\frac{L_u^y}{W}$ may result in significant underestimation of critical wind loads. Best and Holmes found in (46) that turbulent fluctuations in the approaching wind accounted for 71% of the total pressure energy on the central bay of a low-rise building.

The partial derivatives of the force coefficients, $\frac{\partial C_F}{\partial \left(\frac{h}{z_o} \right)}$ and $\frac{\partial C_F}{\partial \left(\frac{L_u^y}{W} \right)}$, were

estimated using figure 4.4 (5).

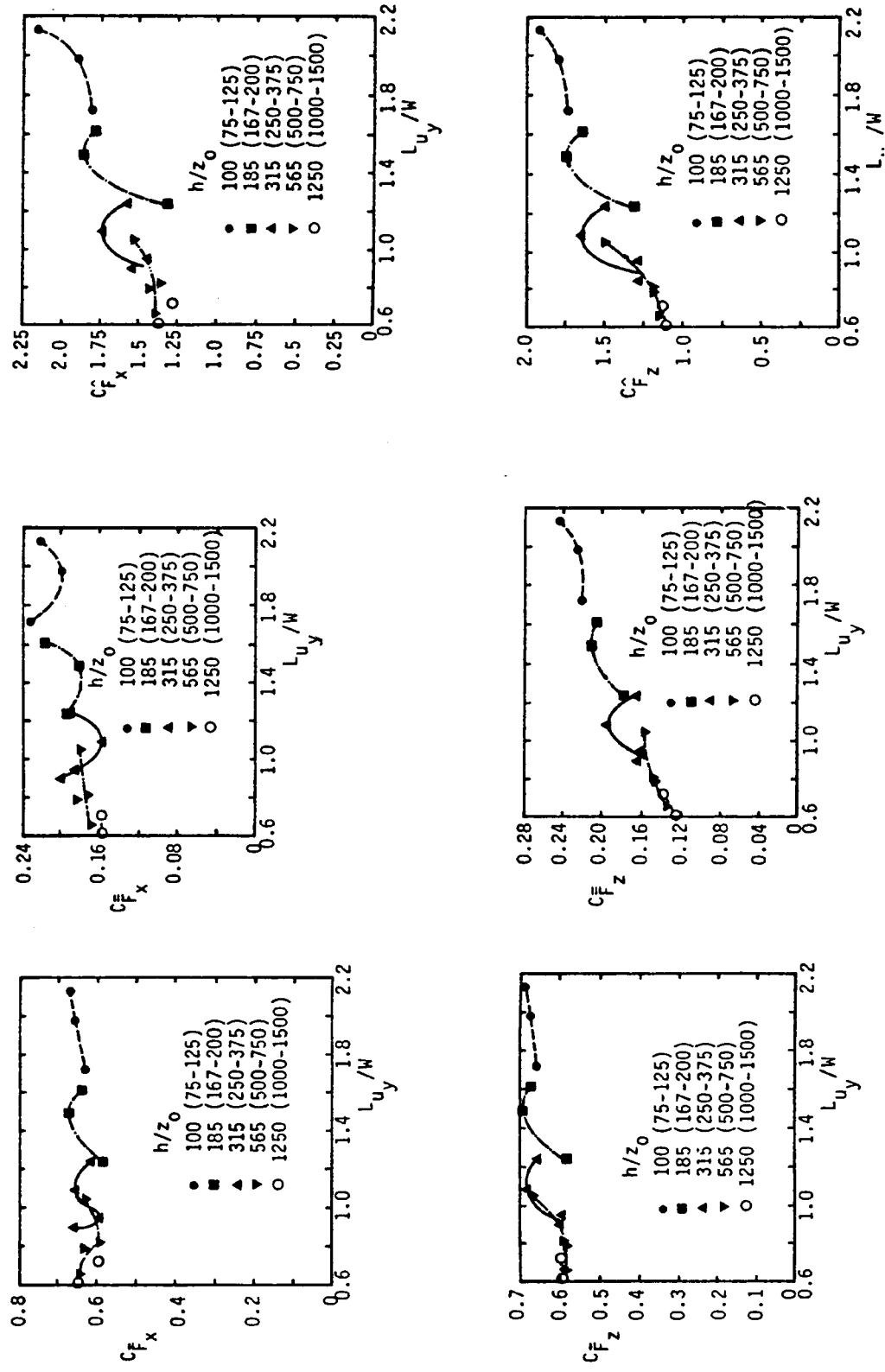


Figure 4.4: Variation of total force coefficients with lateral component integral length scale of turbulence and Jensen number. Order: left to right: mean, rms, and peak coefficients; top and bottom: horizontal and vertical coefficients, respectively (from 5).

The data of table 4.1 reflect an increase in the Jensen number by a factor of 2, while holding the parameter $\frac{L_u^y}{W}$ constant. Physically, this situation represents an approximately fifty percent reduction in the scale of mechanically induced turbulence, while the scale of turbulence in the approaching wind is held constant. (Decreasing the roughness length of the terrain by a factor of two effectively doubles the Jensen number.) This increase in the Jensen number resulted in a decrease of roughly 5% in the mean force coefficients, and about 12% in the rms, as well as peak force coefficients. However, decreasing L_u^y by a factor of two relative to its full-scale value, while holding the Jensen number constant between full and model scales, produced a decrease of over 40% in the peak vertical force coefficient, while the peak horizontal force coefficient decreased by only about 14%. Only an approximately 20% reduction in the value of the mean vertical force coefficient concurred with the 40% decrease in the value of the peak vertical force coefficient. These results are indicated by the data of table 4.2.

The sensitivity of the peak and rms force coefficients to the lateral component of the LILST (L_u^y) was indicative of the importance of accurately simulating the spectrum of longitudinal velocity fluctuations in the approaching wind. More specifically, it is generally insufficient to satisfy only the scaling law represented by the Jensen number to obtain an accurate representation of peak loading effects. It is noted, however, that as long as a test structure is fully immersed in a turbulent boundary layer, as in very sheltered terrain, then satisfying the Jensen number is generally sufficient for accurate representation of the local boundary layer.

Table 4.1: Estimated percent change in each force coefficient when the Jensen number $\frac{h}{z_o}$ is increased by a factor of two, while $\frac{L_u^y}{W}$ is held constant (from 5).

$\frac{L_u^y}{W}$	$C_{\bar{F}_x}$	$C_{\bar{F}_z}$	$C_{F_x}^-$	$C_{F_z}^-$	$C_{F_x}^+$	$C_{F_z}^+$
0.84	-6	-2	-8	-16	-6	-13
1.90	-4	-3	-12	-12	-15	-11

Table 4.2: Estimated percent change in each force coefficient when $\frac{L_u^y}{W}$ is decreased by a factor of two, while the Jensen number $\frac{h}{z_o}$ is held constant (from 5).

$\frac{h}{z_o}$	$C_{\bar{F}_x}$	$C_{\bar{F}_z}$	$C_{F_x}^-$	$C_{F_z}^-$	$C_{F_x}^+$	$C_{F_z}^+$	$\frac{L_u^y}{W}$ range
315	-	-23	-	-8	-15	-42	1.4 to 0.7
565	-	-20	-7	-20	-13	-41	1.1 to 0.55

The reason for this is that the Jensen number is reasonably representative of the mean longitudinal component wind velocity and turbulence intensity profiles (5, 28); which are the boundary layer characteristics that determine the loading of low-rise (quasi-static) structures in very sheltered terrain.

Table 4.3 indicates the errors incurred in the force coefficients when a model of twice the correct size is placed into a correctly scaled boundary layer (5). Errors in the force coefficients associated with this type of scaling error ranged between -6 and -23 percent, with the greatest discrepancy being observed, again, in the peak vertical pressure coefficient.

Table 4.3: Estimated percent change in each force coefficient when the model size is doubled relative to the height of the model boundary layer (from 5).

$C_{\bar{F}_x}$	$C_{\bar{F}_z}$	C_{F_x}	C_{F_z}	$C_{\wedge_{F_x}}$	$C_{\wedge_{F_z}}$
-6	-11	-13	-15	-16	-23

CHAPTER 5

WIND EFFECTS ON THE ROOFS OF LOW-RISE BUILDINGS

5.1 Introduction:

Damage resulting from recent hurricanes along the Atlantic coast of the United States has caused the level of interest in roofing practice to reach an all time high. Leakage through accidental cracks in the roofs of low buildings is a major cause of damage to them (31, 47, 48, 49). Amirkhanian reports in (47) that in 95% of the cases of wind damage to low-buildings caused by Hurricane Hugo, over 95% of the damage was to roofs. However, only 6.5% of the insured value of these buildings could be attributed to wind damage. Actually, the greatest contribution of the wind to damage by Hurricane Hugo was in opening the roofs of buildings to damage by flooding and rain. Similar observations concerning the contribution of wind to the damage of low buildings in Hurricane Andrew were reported by Vognild in (48).

5.2 Recent Research:

Recent research efforts have focused upon the relationship between the type and slope of roofing, and its capacity to withstand heavy wind loads. Forensic studies conducted by Amirkhanian, Vognild, and Morse-Fortier, reported in 47, 48, and 49, respectively, provided results which indicated that hip roofs are most likely to survive severe wind events. Specifically, Amirkhanian's findings concerning damage caused by Hurricane Hugo indicated that the relative capacities of hipped, gabled and flat roofs to

withstand severe wind events are from greatest to least, respectively (47). The corners of flat roofs are prone to developing extremely intense suction in strong winds (31, 47, 48, 49, 50, 51). Gable ends are susceptible to becoming damaged by uplift, especially at ridge corners (33). Gable roofs are also subject to developing intense lift along their ridges and eaves (32, 33, 47, 48, 49).

5.2.1 Flat Roofs:

Concerning the occurrence of intense uplift near the corners of flat roofs, the responsible mechanism has been identified as roof vortex (50). This phenomenon involves the generation of vortices at roof corner locations by adjacent straight, sharp edges, as shown in figure 5.1. These vortices behave very much like delta wing vortices, which are known for the uplift they exert on airplane wings. Lin and Surry report in (50) that the intensity of the suction associated with roof vortex increases approaching the vertex of a corner. Their data yielded pressure coefficients of -12 near edges, and close to corners under oblique winds (azimuths other than 90°). Very near corners, suctions that yielded pressure coefficients of -16 to -18 resulted. Along rays between 10 and 22° with respect to an edge where a corner vortex exists, extremely high suction peaks resulted.

5.2.1.1 Reducing Critical Loads on Flat Roofs:

Various mitigation strategies for reducing the intensity of suction exerted at corners of flat roofs were investigated by Lin and Surry in (50). These strategies included (1) the use of partial parapets of width $0.12 H$, and height $0.06 H$ (H denotes roof height); (2) the application of rooftop cylinders; and (3) rounding the edges of roofs

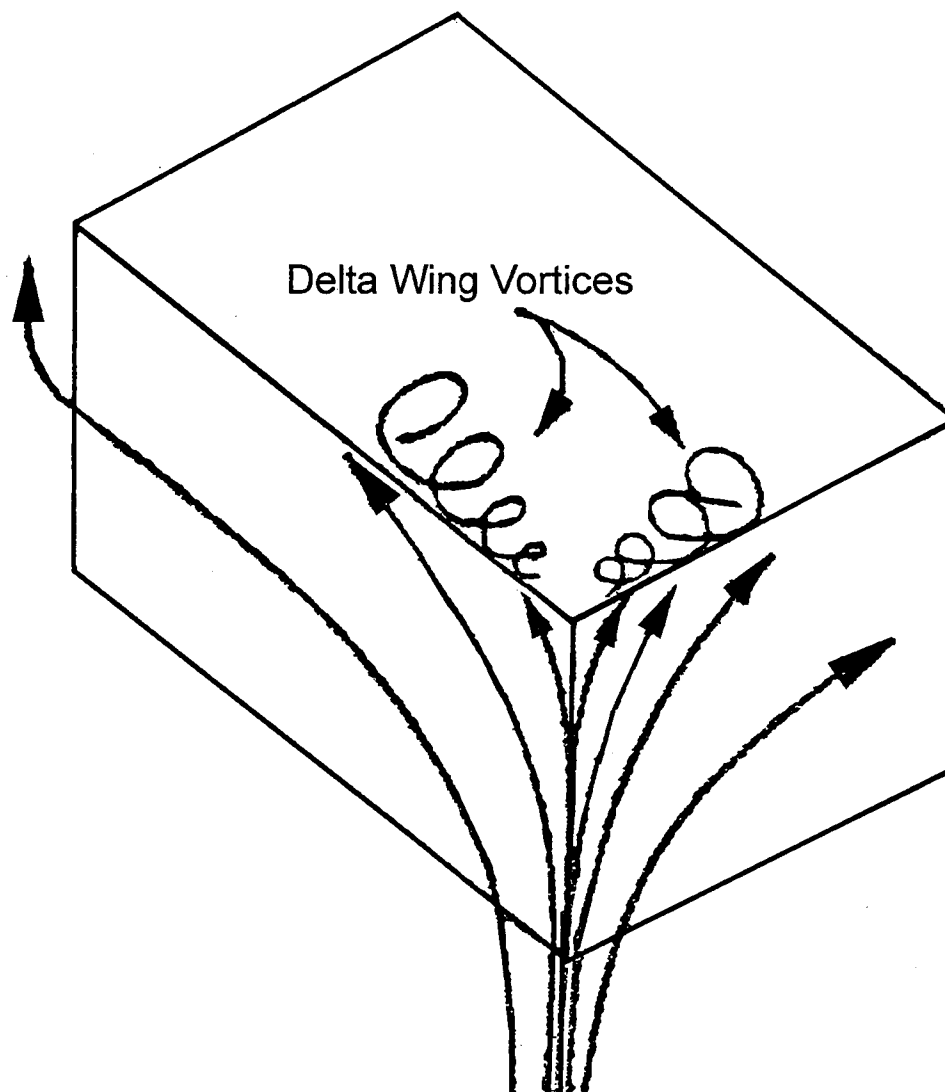


Figure 5.1: Roof vortex caused by oblique wind (from Newberry, Eaton 1974).

by attaching plates having radii of $0.10 H$. The greatest effectiveness of partial parapets was in reducing fluctuating pressures, providing decreases of 24-40% in the values of peak suction coefficients. Corresponding reductions in the values of rms, and mean pressure coefficients were just 20-30%, and 10%, respectively. The application of rooftop cylinders was effective in reducing peak suction coefficients by 60%, and rms and mean pressure coefficients by 55, and 50%, respectively. However, decreased effectiveness with increasing distance from their location was identified as a deficiency in the performance of rooftop cylinders. Rounding the edges of roofs was the best of any of the mitigation strategies investigated, effectively reducing peak, mean and rms pressure coefficients by over 60%.

5.2.2 Sloping Roofs:

Concerning the relative aerodynamic behavior of roofs with nonzero slopes, reviews of studies involving hip and gable roofs (32), single and multiple span gable roofs (33) and mono-slope roofs (34), follow.

5.2.2.1 Mono-slope Roofs:

Center for Building Studies (CBS), Concordia University, Montreal, Quebec H3G 1M8, Canada:

5.2.2.1.1 Experimental Details:

The working section of the wind tunnel is 12.20 m long and 1.80 m wide. It has an adjustable roof height averaging 1.60 m. A detailed description of the design and fabrication of the wind tunnel at CBS is presented by Stathopoulos in (52). A geometric scale of 1:400 has been suggested for accurate modeling of the most important characteristics of the atmospheric boundary layer (ABL) in the strong wind conditions of

open terrain (33, 34). These characteristics include turbulence intensity and velocity profiles, turbulence scales and turbulence spectra (29). However, a relaxed 1:200 geometric scale was used. Relaxing the geometric scale facilitates placing pressure taps very close to critical locations, such as corners and edges, so that the most critical suction peaks are included in the data. Placing pressure taps very close to bluff edges also improves the frequency response (34). The sensitivity of the pressure measuring system to fluctuations in the dynamic pressure; i.e., the system's capacity to measure the amplitudes of fluctuations in the dynamic pressure response without attenuation, is described in terms of a percentage by the "frequency response." Very close to bluff edges, turbulent effects are strongest. Therefore, by placing pressure sensors (taps) very close to these locations, the best frequency response is attainable. A frequency response of almost 100% was attained for dynamic pressure fluctuations of up to 100 Hz in the experiments of this study.

Concerning the accuracy of results, ensuring that the most critical suction peaks were included in the data was considered to be more important, than slight disparities between full and model scale boundary layer characteristics that might result from using a larger geometric scale. A proposal by B.E. Lee in (53), and experimental results reported by D. Surry and T. Stathopoulos in (54) suggested this prioritization concerning the choice of geometric scales. More recently, however, much larger geometric scales than 1:200 have been employed for the same reason that a "relaxed" 1:200 scale was used in this experiment. One example of this is Texas Technological University's (TTU) experimental building. Lin and Surry used a 1:50 scale model of the TTU building in their recent study of extreme corner suctions affecting flat roofs (50). Much greater

suctions were discovered in this study than in previous studies of the same phenomena using smaller scale models. (Refer to section 5.2.1.) Nonetheless, the present study, in which relatively small-scale models were used, provided much information about the aerodynamic behavior of mono-slope roofs. These results, and the results of a later study, presented in (59) by Surry and Stathopoulos, form the basis of the design load specifications of the ASCE standard for the mono-slope roof (8).

The effects of scaling were taken into account by comparing results obtained in tests involving two different geometric scales. The models included a large, 1:200 scale Plexiglas model constructed at CBS, and a small, 1:500 scale model provided by UWO. The roof angle of the CBS model was 4.8° (1:12 pitch), whereas the UWO model had an adjustable roof angle of between 0 and 18.4° (4:12 pitch). The full-scale dimensions of the UWO model were $51.0 \times 20.5 \times 7.6$ m (L $\times$ W $\times$ H), whereas the CBS model had a full-scale length of 61 m, and adjustable dimensions of width and height, ranging between 12.20 and 24.40 m, and 3.66 and 12.20 m, respectively, as shown in figure 5.2. An exploded plan view of the CBS model is shown in figure 5.2. The dashed lines in the figure indicate the variable width (W), and eave height (H) dimensions. The reference wind direction (a) is also indicated in the figure. This model was equipped with 153 pressure taps, situated as shown in figure (5.2). The closest of these taps to the critical edge and corner locations were placed 1.30 mm from them (0.26m full-scale). The pressure taps of the smaller-scale (UWO) model were fewer in number, and the nearest of them to critical corner and edge locations were 2.4 mm from them (1.2 m full-scale). Because pressure tap locations on the two models involved did not correspond, the results obtained provided only qualitative information.

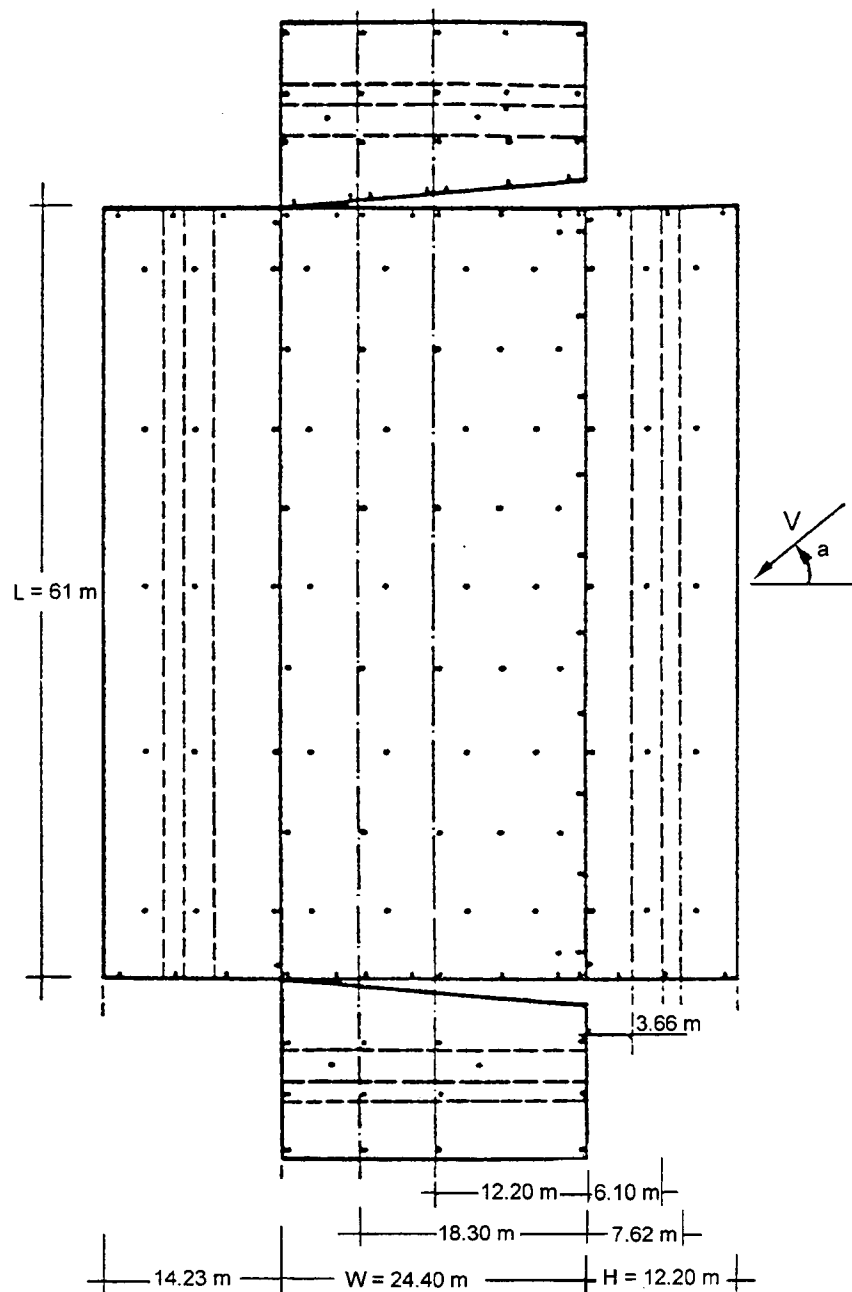


Figure 5.2: Exploded plan view of the CBS model (from 34). The dots in the figure indicate pressure tap locations.

5.2.2.1.2 Results:

5.2.2.1.2.1 Local Pressure Measurements:

Local pressure distributions resulting on the wall and roof surfaces of the wide (24.4 m, full-scale) model are indicated by contour lines in figure 5.3. Azimuths ranging from zero to 90° were enveloped in the tests to determine these local, mean (C_p) and instantaneous peak $(C_{p, \text{peak}})$ suction coefficients. These results were similar to those of tests involving buildings having flat and gable roofs, in that suction occurred over the entire roof, and coefficient magnitudes varied inversely with increasing distance from the windward edge(s) of the roof (32, 34, 51). The occurrence of large mean suction coefficients near the eave corner (figure 5.3) was unusual. Previous experiments that involved mono-slope roofs yielded much lower mean suctions in this area (34). Pressure readings taken at the 30° azimuth contributed the most to producing these large mean values. Conversely, the values of the peak suctions were comparable to values obtained in earlier studies of flat and gable roofs.

Local peak, and mean pressure distributions that resulted on the narrow model (12.20 m, full-scale) are shown in figure 5.4. For the narrow building configuration, the critical azimuths were greater than 90°. (Wind directed toward the ridge produced the greatest dynamic pressure response.) Hence, the additional azimuths indicated in the figure. Building width and local suction coefficient magnitudes were inversely related. This is apparent in figures 5.3, and 5.4. Local wall suctions, however, were nearly independent of building width.

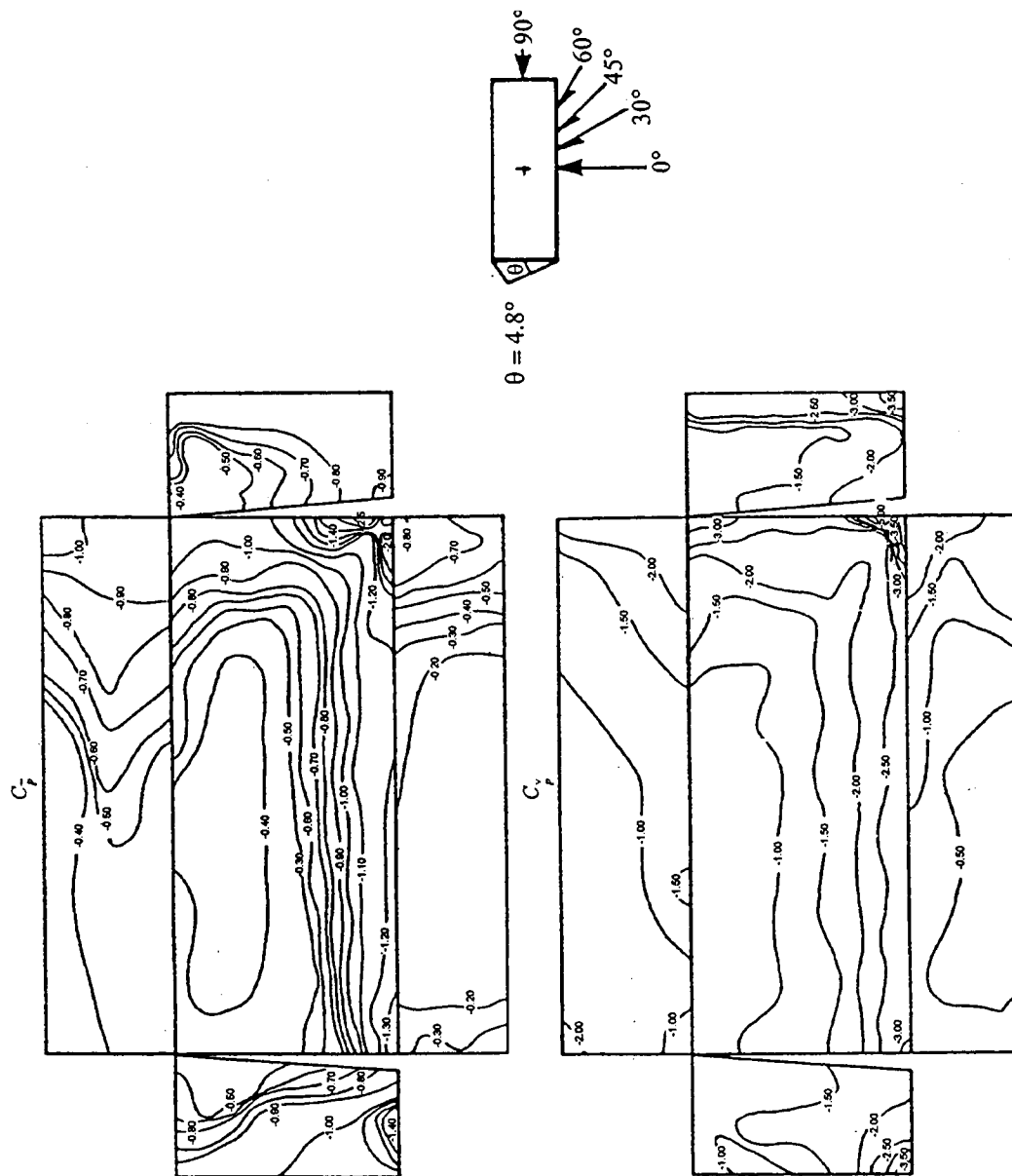


Figure 5.3: Most critical mean and instantaneous peak pressure coefficients for the wide building ($W=24.40$ m) (from 34).

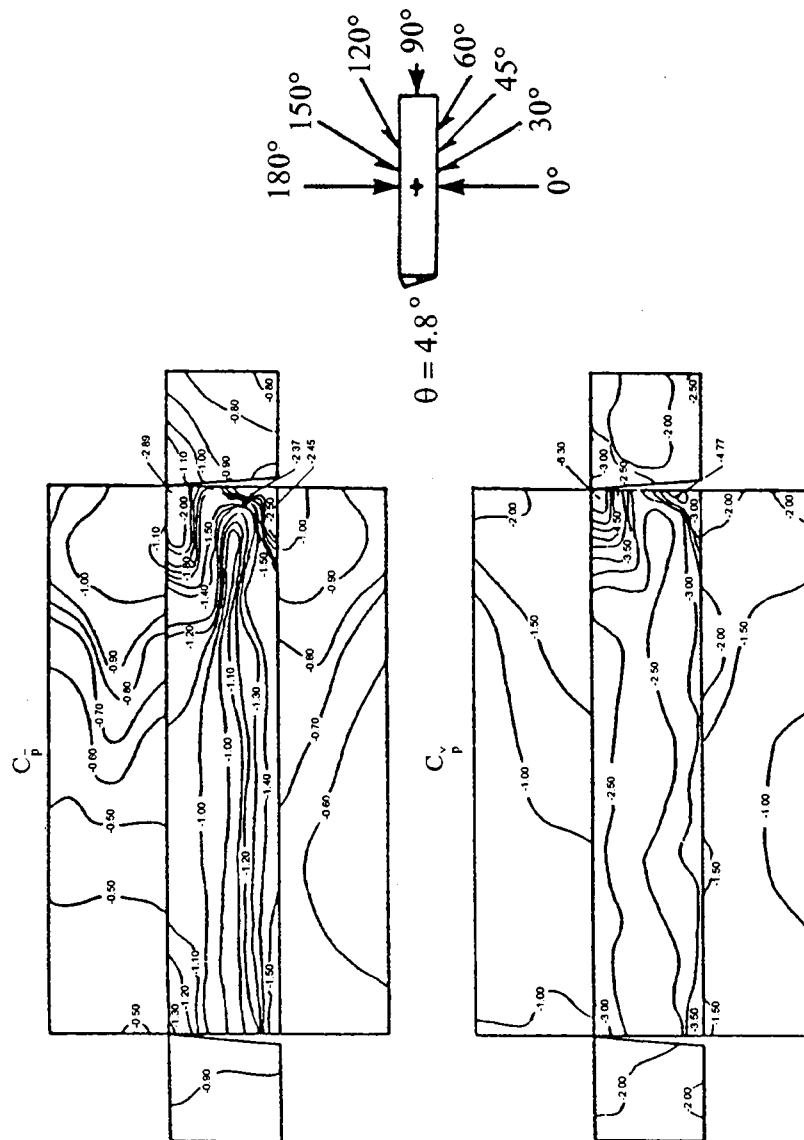


Figure 5.4: Most critical mean and instantaneous peak pressure coefficients for the narrow building ($W=12.20$ m) (from 34).

Findings of D. Surry et al. (55), indicated that referencing pressure data to mean dynamic pressure at building height might eliminate the influence of building height upon pressure coefficient values. However, during testing, pressure data are usually referenced to mean dynamic pressure at the gradient height of the model boundary layer (z_g).

Pressure coefficients may be subsequently referenced to building height. This is done by multiplying them by the ratio of the mean dynamic pressure at gradient height to the mean dynamic pressure at building height. This ratio reduces to the dividend of the mean

longitudinal wind velocities at the gradient and building heights, squared $\left(\frac{\overline{U}_g}{\overline{U}_H} \right)^2$. The

dynamic pressure at building height was determined using the velocity profile of the model boundary layer, shown in figure 5.5. The gradient wind velocity was 13 m/s, and the gradient height was approximately 1.6 m. This method permits the specification of gust pressure coefficients (GC_p) regardless of building height. It has been implemented in the ASCE standard, and other standard building codes such as the NBCC.

However, the dynamic pressure response of low buildings with mono-slope roofs is not entirely independent of building height. While being applicable to non-critical azimuths, the height independence of pressure coefficients on low buildings with mono-slope roofs breaks down within a range of critical azimuths. As shown in figure 5.6, mean, as well as, peak local suction coefficients, which were referenced to mean dynamic pressure at eave height, increased markedly at the ridge corner tap at an azimuth of 150° , as the eave height (H) was increased.

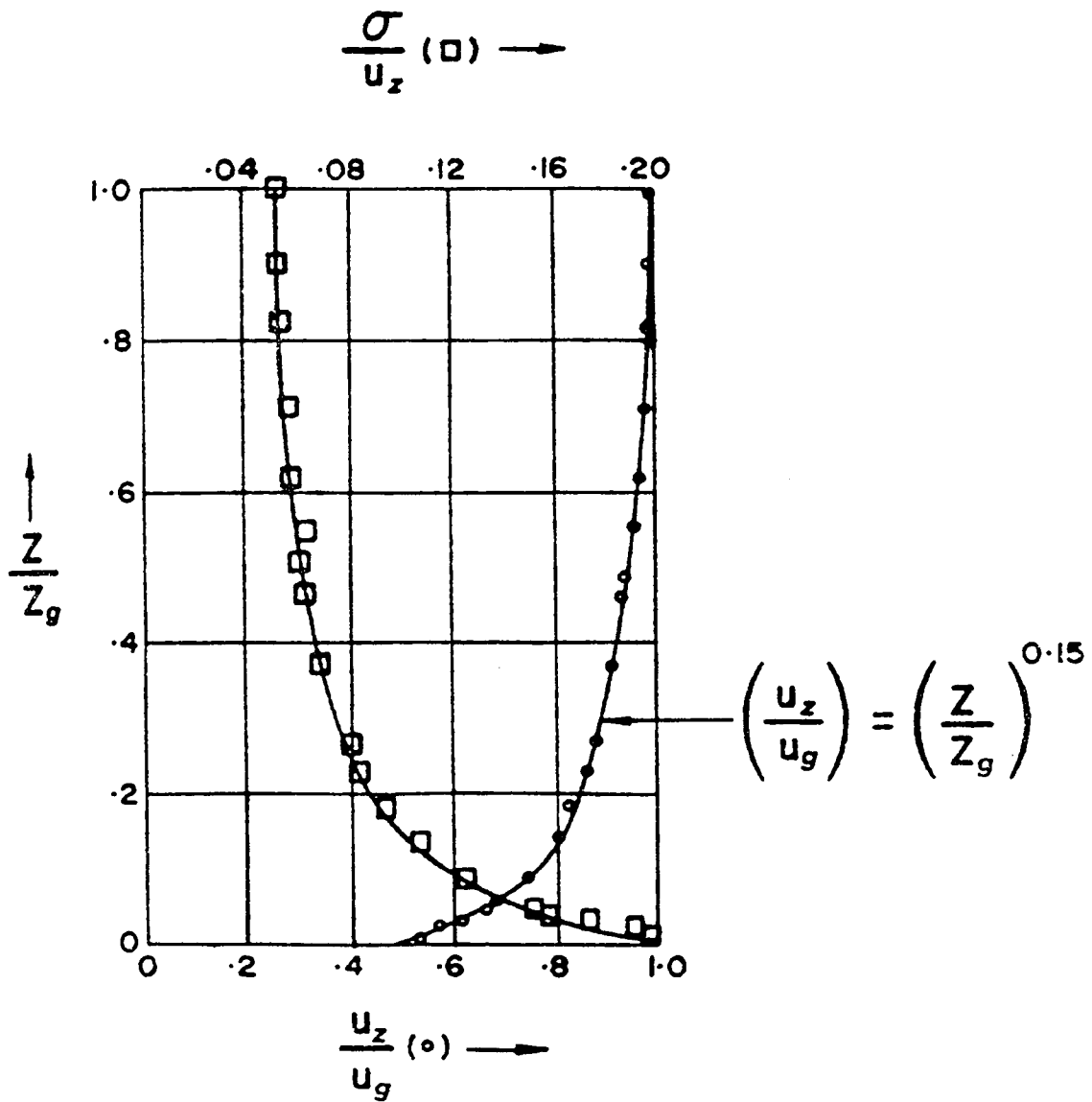


Figure 5.5: Mean velocity and turbulence intensity profiles, open country exposure (from 34).

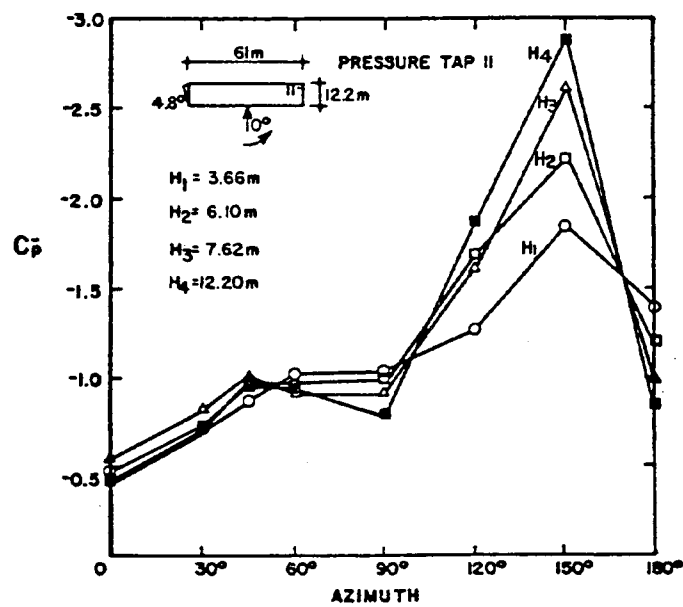
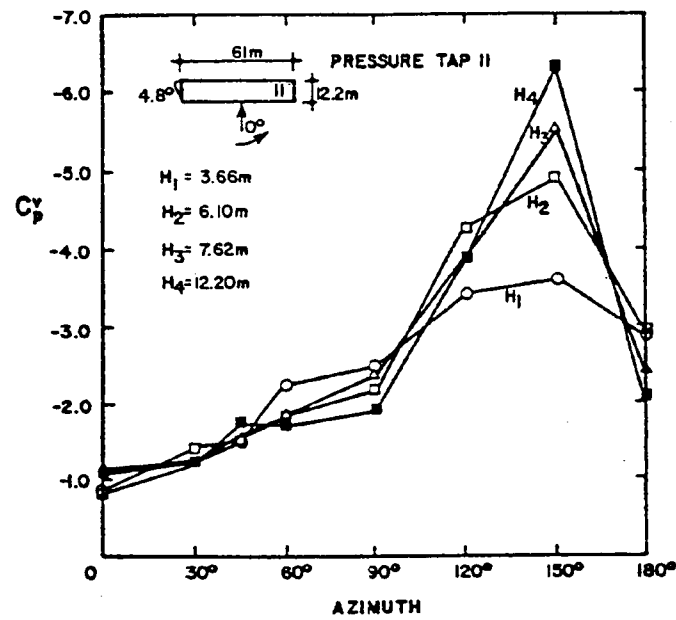


Figure 5.6: Corner suctions for buildings of various heights (from 34).

Figure 5.7 shows test results pertaining to the relationship between building height, and the value of local pressure coefficients. The pressure data included readings taken throughout the envelopes of the tallest (H_4), and the least tall (H_1) models. The widest (24.40 m) and narrowest (12.20 m) models were included in the tests, and only the most critical wind azimuths were used. Observing in figure 5.7 that the vast majority of the data points fall on the side of the 45° line associated with the tallest building, the magnitudes of local pressure coefficients was clearly an increasing function of building height. Furthermore, the data points are grouped further from the 45° line in the plots associated with the wider building. This indicated that the rate of increase in suction coefficient magnitudes with building height, considering coefficients on all building surfaces, varied directly with building width. Specifically, linear regression of the data showed that the mean pressure coefficients were, on average, greater for the taller building by 0.12, and 0.20 for narrower, and wider buildings, respectively. The peak pressure data, however, were more scattered.

Special notice was taken of the fact that the three data points in each plot that had much greater magnitudes for the higher building were recorded at the roof corner taps. Furthermore, these data were more distant from the majority of the data on the narrower buildings. This indicated that the influence of building height on local suction coefficient magnitudes was most important near roof corners; and, as indicated by figures 5.3 and 5.4, an inverse relationship existed between building width and roof suctions. Separate analyses of the roofs and walls showed that the increase in suction coefficient magnitudes with building height, indicated in figure 5.7, was mostly attributable to roof pressure data. Wall suctions were nearly independent of building height.

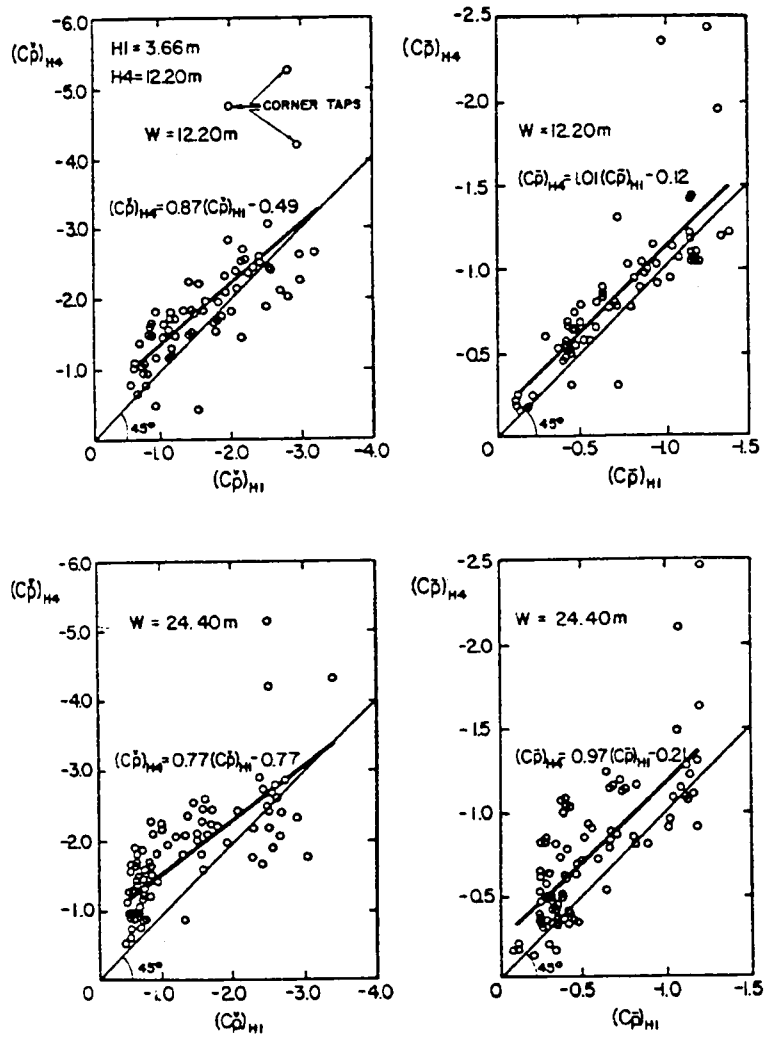


Figure 5.7: The effect of building height on local pressure coefficients (from 34).

Similar plots, shown in figure 5.8, indicated the effect of building width on roof pressure coefficients. The data in this figure include the most critical mean and peak local pressure coefficients. They were computed from data measured at correspondent pressure taps on the roofs of the widest and narrowest buildings. Wind direction was enveloped in process of developing this figure; i.e., the plotted coefficients were computed from the most critical pressure data recorded at all azimuths. As the plots show, the vast majority of the data points are on the side of the 45° line associated with the narrowest building. Clearly, therefore, roof suction was inversely related to building width. The peak pressure data were less scattered for variable building width than for variable building height. Linear regression of the data indicated that peak, as well as, mean pressure coefficients were about 20% higher on the narrower building.

The effects of scaling were investigated by comparing mean and peak local pressure coefficients computed from data recorded at taps located throughout the building envelopes of the 1:500 (UWO), and 1:200 (CBS) scale models. Each of these models had a full-scale height of 7.62 m. Plots of the results are shown in figure 5.9. Virtually all of the data points are on the side of the 45° line associated with the larger scale (1:200) model. Therefore, pressure coefficients were an increasing function of geometric scale. At the time of these tests, this result was considered to be tentative. Currently, the facility of larger scale models to permit the placement of pressure taps relatively closely to critical locations, such as roof corners, and the attendant ability to capture critical pressure data resulting at these locations is well known (50, 51, 53, 54). Even the "relaxed" 1:200 scale (CBS) model was one-half the scale of most wind tunnel models in use today.

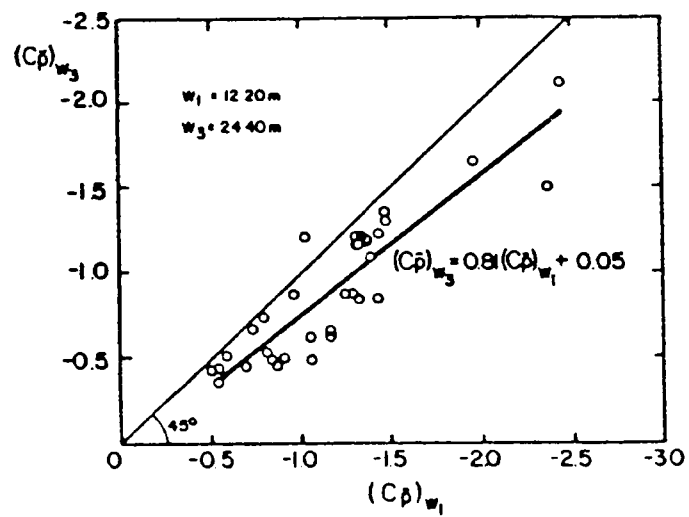
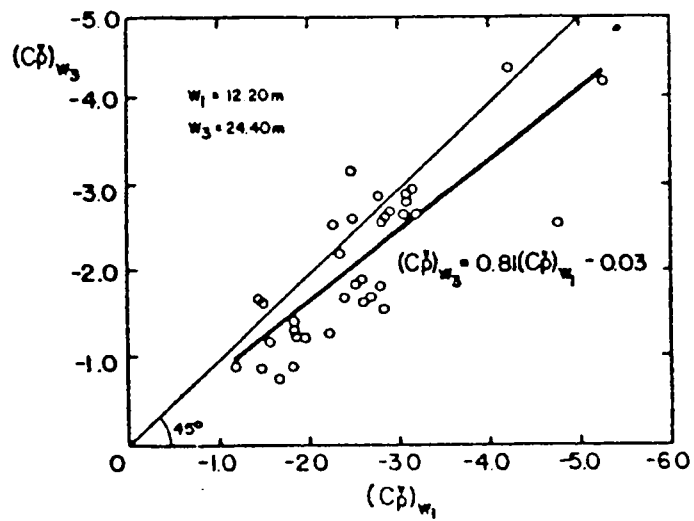


Figure 5.8: The effect of building width on local pressure coefficients (from 34).

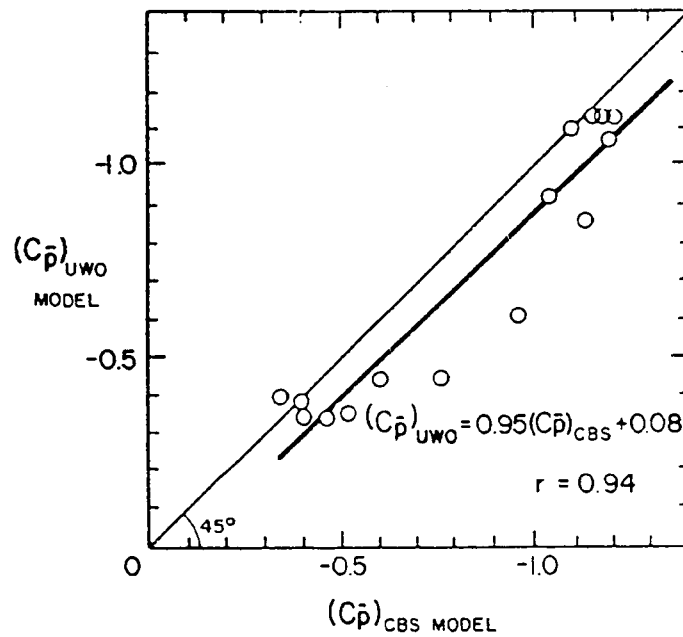
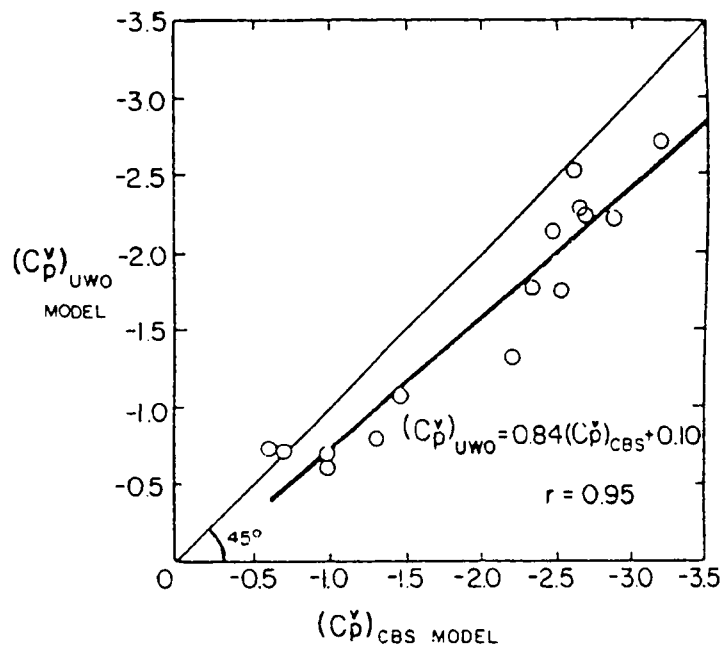


Figure 5.9: Comparison of pressure coefficients affecting large (CBS) and small (UWO) scale models in the same wind event (from 34).

Concerning the instrumentation of flat roofs, Kind (51) suggests that unless there are pressure taps located no more than $0.07 H$ from roof edges, suction peaks will be missed. To satisfy this criterion in the present experiment, pressure taps would have had to be located no further than $0.07 \times 7.6 = 0.5$ m from the roof edge. Actually, pressure taps were located at distances of 0.26, and 1.2 m from the roof edges of the CBS (1:200), and UWO (1:500) models, respectively. Noting that flat roofs are the worst case scenario regarding corner loads (50, 51), it might reasonably be concluded that the most critical suction peaks were captured on the CBS models of this experiment. Similarly, it might reasonably be suspected that the most critical peaks were missed on the UWO model.

The influence of roof slope upon the dynamic pressure response of the entire building envelope was investigated by experimenting with the 1:500 scale (UWO) model. Suction peaks were probably missed on this (small-scale) model. However, a qualitative indication of how local pressure responses varied with roof slope was obtained. Tests were conducted on 1:12, 2:12 and 4:12 roof-sloped models. The models were rotated between azimuths of 0 and 180° in the wind tunnel. The vast majority of the critical suction data were recorded at azimuths of 30, or 150° . Only the most critical data were used to compute the mean, and peak local suction coefficients. The distributions of these data were as indicated by the coefficients shown in figure 5.10. As the figure shows, peak suction coefficients near the windward ridge corner varied directly with roof slope. Although peak suction coefficients at the eave were unaffected by variations in roof slope, mean suction coefficients at the eave decreased as roof angles were increased. Wall suction coefficients were nearly independent of the roof angle.

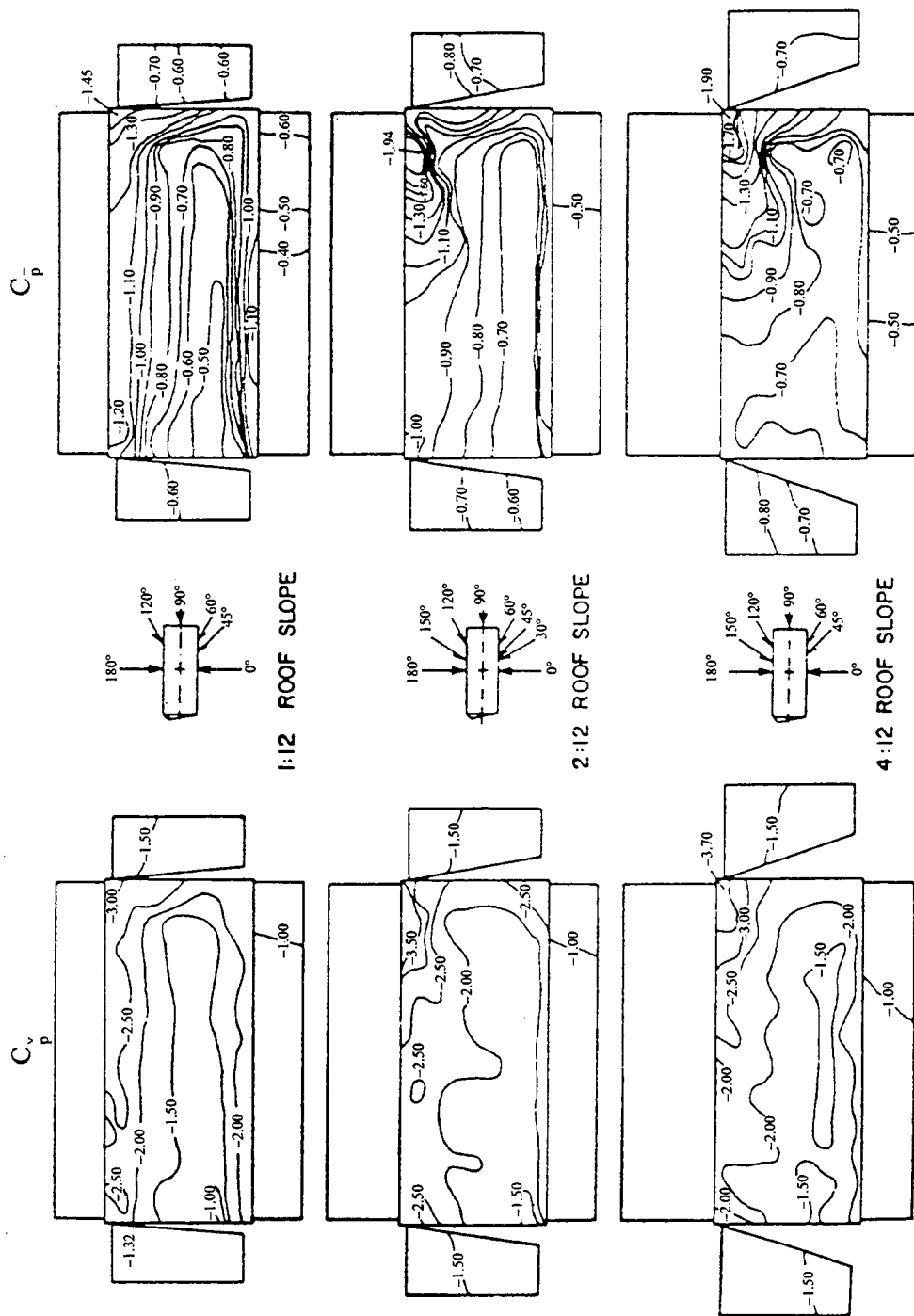


Figure 5.10: The effect of roof slope on pressure coefficients (from 34).

5.2.2.1.2.2 Area Averaged Loads:

5.2.2.1.2.2.1 Basis:

Because suction peaks are localized effects, gust pressure coefficients are decreasing functions of tributary area (8). In the ASCE standard, gust pressure coefficients (GC_p) are presented as functions of "effective wind area." The definition of the "effective wind area" acknowledges the localized nature of pressure peaks, allowing that the tributary width of components and cladding need not be taken as less than 1/3 of the member span (8).

5.2.2.1.2.2.2 Comparison of Arithmetic and Pneumatic Averaging:

There are two common techniques of spatially averaging pressures. One of these is arithmetic averaging (AA), and the other is pneumatic averaging (PA). In the PA process, pressure data taken throughout an area of arbitrary size are processed in such a way as to render them as a single input. This is accomplished by attaching leads from the pressure taps within the area of interest to a manifold (57). In effect, the pressure readings from each pressure tap are received by the data acquisition system as a single, average reading. Accordingly, PA admits the actual distribution of pressures occurring at any instant of the sampling period to the averaging process. Therefore, the randomness of the duration, and spatial organization of the dynamic (turbulent) component of the loading are accounted for by PA. Conversely, arithmetic averages are not computed sequentially. Therefore, actual (instantaneous) pressure distributions are not represented by an arithmetic average. Consequently, the lack of spatial correlation, and the chronology of turbulent wind effects are not accounted for, as they are by PA.

PA and AA were compared using the CBS (1:200 scale) model. Its variable width and height dimensions were set to 24.40 and 7.62 m, respectively. Mean and peak area suction coefficients were computed by each averaging technique for each roof panel. The extreme coefficient values, considering all panels, are plotted in figure 5.11. As the distribution of the data indicates, mean suction coefficients were approximately equal, regardless of the averaging technique used to compute them. However, as the data distribution also indicates, peak pressure coefficients were considerably lower when obtained by PA. This is reasonable because PA, unlike AA, accounted for the lack of spatial correlation and the chronology of the turbulent effects that produce the most critical suctions (57).

5.2.2.1.2.2.3 Area Pressure Coefficients, Areas of Various Sizes:

Area averages of suction data taken on the CBS (1:200 scale) model, considering all azimuths, were computed. The most critical peak, and mean suction coefficients computed from these averages are listed in table 5.1. These listings include panel loads for buildings of various geometries. Each table entry corresponds spatially to the roof panel layout described as follows. Two rows of five 12.20×6.10 m panels covered the roof of the building that had a full-scale width of 12.20 m. Similarly, 4 rows of five 12.20×6.10 m panels covered the roof that had a width of 24.40 m.

As indicated by the values of the entries in table 5.1, the greatest mean, as well as, peak area suctions occurred in the ridge corner panels. Figures 5.3 and 5.4, like table 5.1, feature buildings of different widths; however, the figures show local pressure distributions. Comparing local peak, as well as mean suction coefficients near the windward ridge corner of the wide and narrow buildings, figures 5.3 and 5.4,

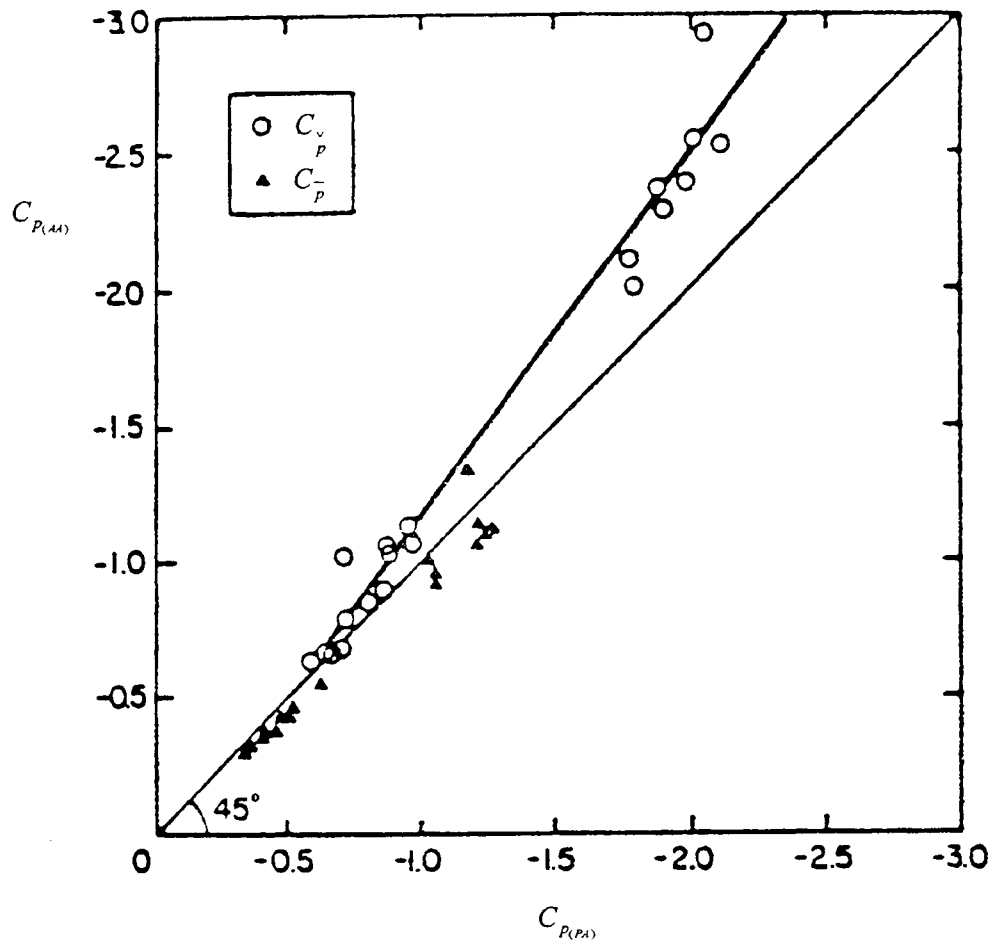


Figure 5.11: Comparison of pneumatically averaged (PA) and arithmetically averaged (AA) area pressure coefficients for roof panels (from 34).

Table 5.1: Most critical mean and peak panel pressure coefficients for buildings of various geometries (from 34).

Height (m)	Width = 12.20 m		Width = 24.40 m									
3.66	-1.90(-1.12)	-1.93(-1.08)	-1.71(-1.09)	-1.96(-1.27)	-3.11(-1.56)	-2.42(-1.44)	-2.55(-1.38)	-2.22(-1.37)	-2.38(-1.61)	-3.67(-2.06)		
	-1.74(-1.06)	-1.77(-1.07)	-1.71(-1.03)	-1.76(-1.04)	-1.70(-1.02)	-1.56(-0.79)	-1.78(-1.11)	-1.83(-1.16)	-1.90(-1.11)	-1.79(-1.07)		
7.62	-1.93(-1.03)	-2.11(-1.01)	-1.65(-1.09)	-2.06(-1.46)	-3.92(-2.29)	-1.05(-0.52)	-1.13(-0.68)	-1.06(-0.63)	-1.16(-0.58)	-1.82(-1.04)		
	-2.76(-1.43)	-2.46(-1.42)	-2.35(-1.41)	-2.18(-1.39)	-2.33(-1.46)	-1.86(-0.99)	-1.66(-1.03)	-1.69(-0.98)	-1.64(-0.99)	-1.85(-1.00)		
12.20	-2.17(-0.94)	-1.82(-0.92)	-1.70(-1.10)	-1.94(-1.40)	-3.60(-2.37)	-2.33(-1.27)	-2.00(-1.14)	-2.06(-1.16)	-2.13(-1.68)	-4.19(-2.38)		
	-2.31(-1.36)	-2.30(-1.26)	-2.28(-1.24)	-2.10(-1.39)	-2.73(-1.66)	-1.64(-1.03)	-1.83(-1.14)	-2.08(-1.17)	-2.34(-1.16)	-1.92(-1.24)		
						-1.26(-0.67)	-1.33(-0.79)	-1.38(-0.84)	-1.37(-0.79)	-1.77(-1.18)		
						-2.11(-1.21)	-2.01(-1.24)	-1.98(-1.26)	-1.90(-1.21)	-2.05(-1.18)		

respectively; then comparing the respective coefficients in table 5.1, clearly, building width had a greater influence on local, than area loads.

Furthermore, the influence of building width on area loads diminished as building height was increased. Specifically, mean, as well as peak suction coefficients in the right ridge corner panel, table 5.1, differed by more with respect to building width for the 3.66, than 7.62 m building height. In fact, the area suction coefficients of the building that had a height of 7.62 m were almost independent of building width; and panel suction coefficients of the tallest (12.20 m) building were omitted for the wider model in table 5.1. It is noted that the relationship between roof suction magnitudes and building width, observed to be inverse for local loads, especially near corners (figures 5.3, 5.4, and 5.7), was direct for area averaged loads (table 5.1).

Figure 5.12 indicates that the increase in area loads with building height, like that of local loads, (figure 5.6), is important within a certain range of azimuths (approx. 120-150°). The difference between area averaged peak suction coefficients within the range of critical azimuths on buildings of heights H_1 and H_3 (figure 5.12) was smaller than that of local suction peaks (figure 5.6). This corroborates the major indication of figure 5.7, which was that the influence of building height is most important very near corners. As in the case of local suction coefficients, figure 5.4, greater area loads resulted on the roof of the narrow building when the wind was directed toward the ridge, (figure 5.12).

Figure 5.13 shows that in the most critically loaded areas of the roof, ridge corner panels, increasing roof slopes were associated with increasing mean, as well as, peak area averaged suctions. Conversely, area averaged peak suctions decreased in magnitude within the eave corner panel as roof angles were increased; and the magnitude of mean

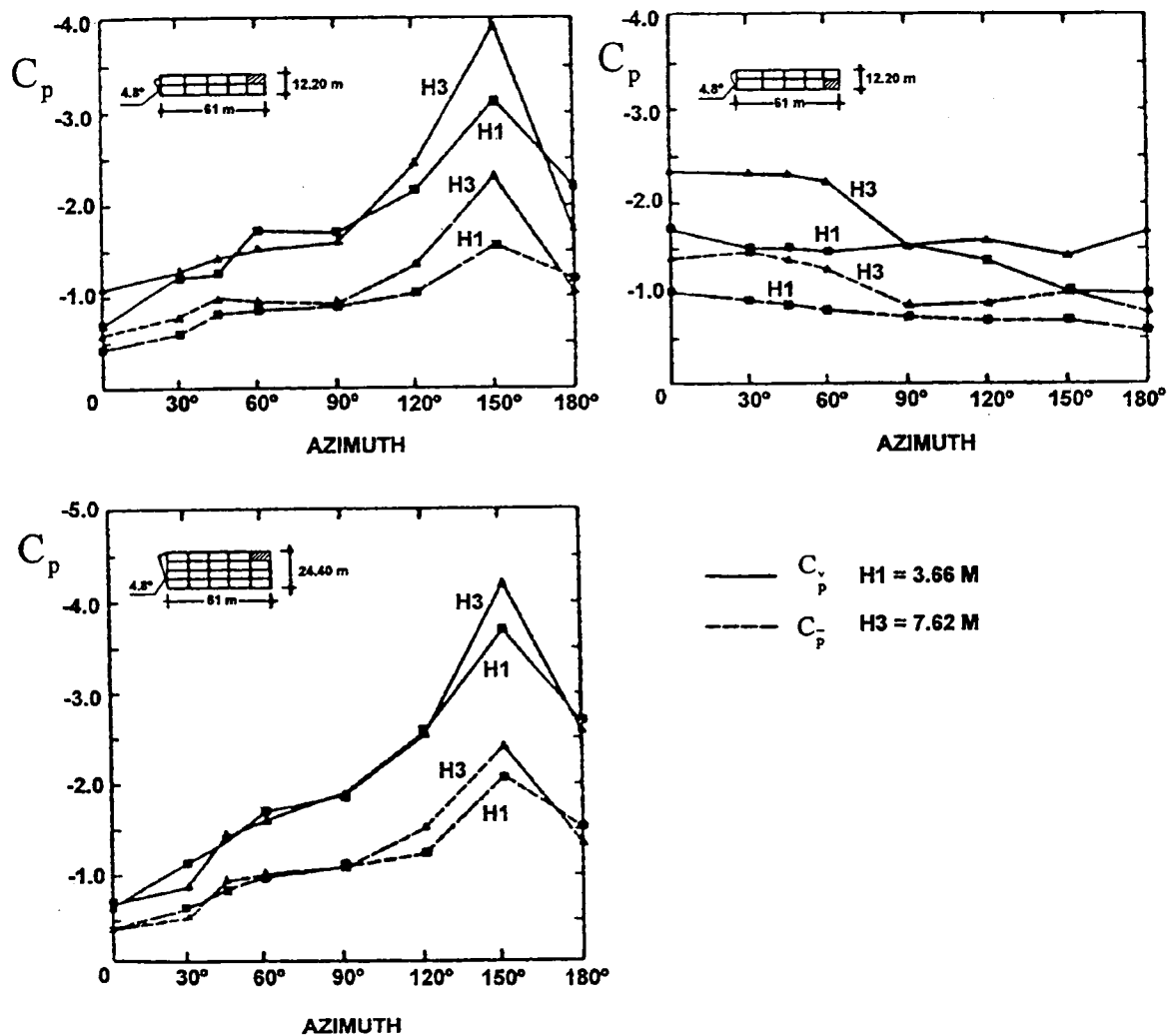


Figure 5.12: Spatially averaged pressure coefficients affecting roof corner panels (from 34).

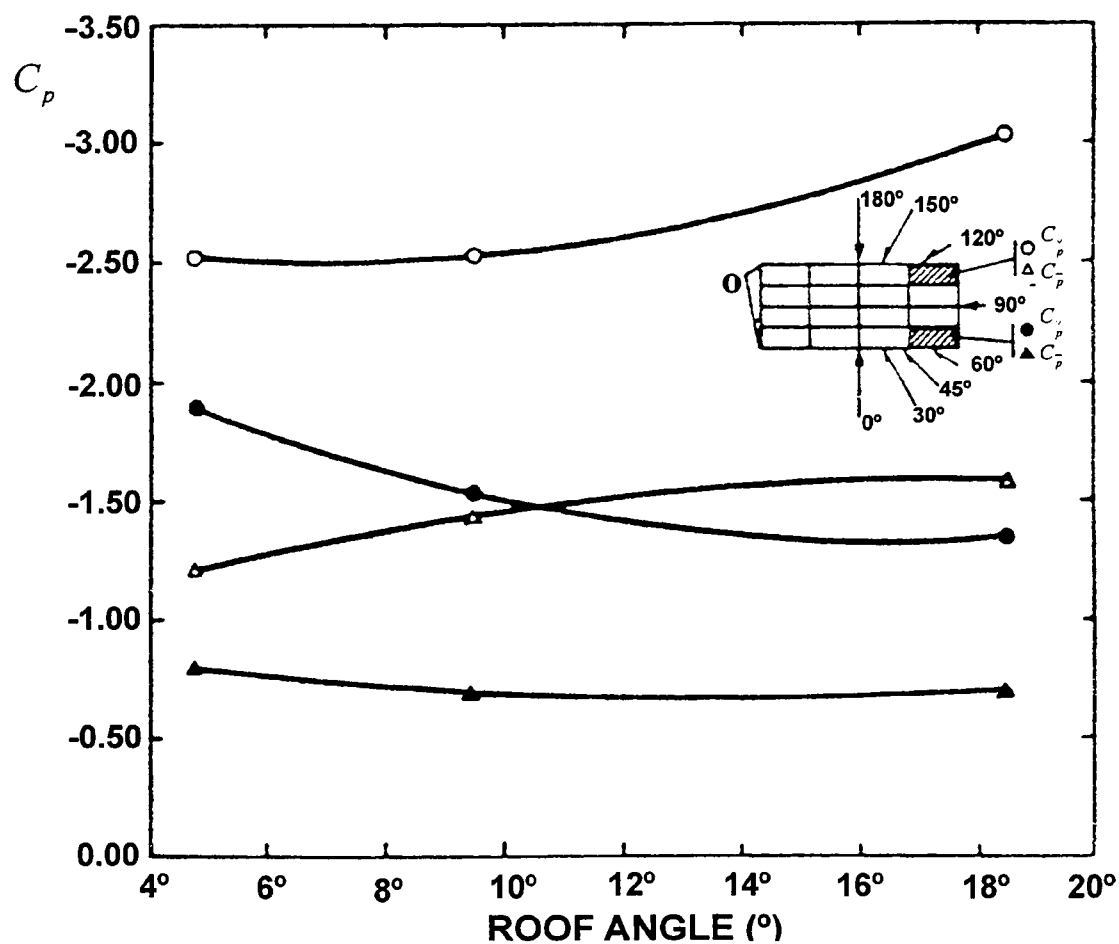


Figure 5.13: The effect of roof slope on spatially averaged pressure coefficients (from 34).

suctions decreased slightly in this panel, as roof angles were increased. It was noted that area averaged pressures on the UWO (1:500) model were lower than those found on the CBS model (1:200 scale). This was likely a consequence of scaling. Again, small-scale models do not accommodate the placement of pressure taps closely enough to critical edge and corner locations for the most extreme suction peaks to be captured.

In figure 5.14, pressure coefficients obtained by pneumatic averaging for the roof corner area at the eave are presented in terms of building height, width and wind azimuth. These coefficients correspond to the area of the four corner taps. These taps are indicated in figure 5.2. Foremost, the coefficient magnitudes in this figure indicate that the most critical corner area loads are virtually independent of building width. Like highly localized corner loads (figure 5.6), and corner panel loads (table 5.1), area corner loads increase with building height, particularly within a range demarcated by the critical azimuths. Generally, the critical azimuths were 0 and 90°, as shown in figure 5.14. Unusually large mean suctions resulted near the eave corner of the wide building, primarily from pressure readings taken at 30° (discussion of figure 5.3). Conversely, area corner loads were decreasing in a range of azimuths containing 30° (figure 5.14).

5.2.2.2 Hip and Gable Roofs:

**Boundary Layer Wind Tunnel Laboratory University of Western Ontario, London,
Ontario N6A 5B9, Canada**

5.2.2.2.1 Experimental Details:

A geometric scale of 1:100 was selected so that pressure taps could be placed in the proximity of critical locations. This is twice the geometric scale of the CBS model used to study mono-slope roofs (section 5.2.2.1). Interest in capturing the dynamic

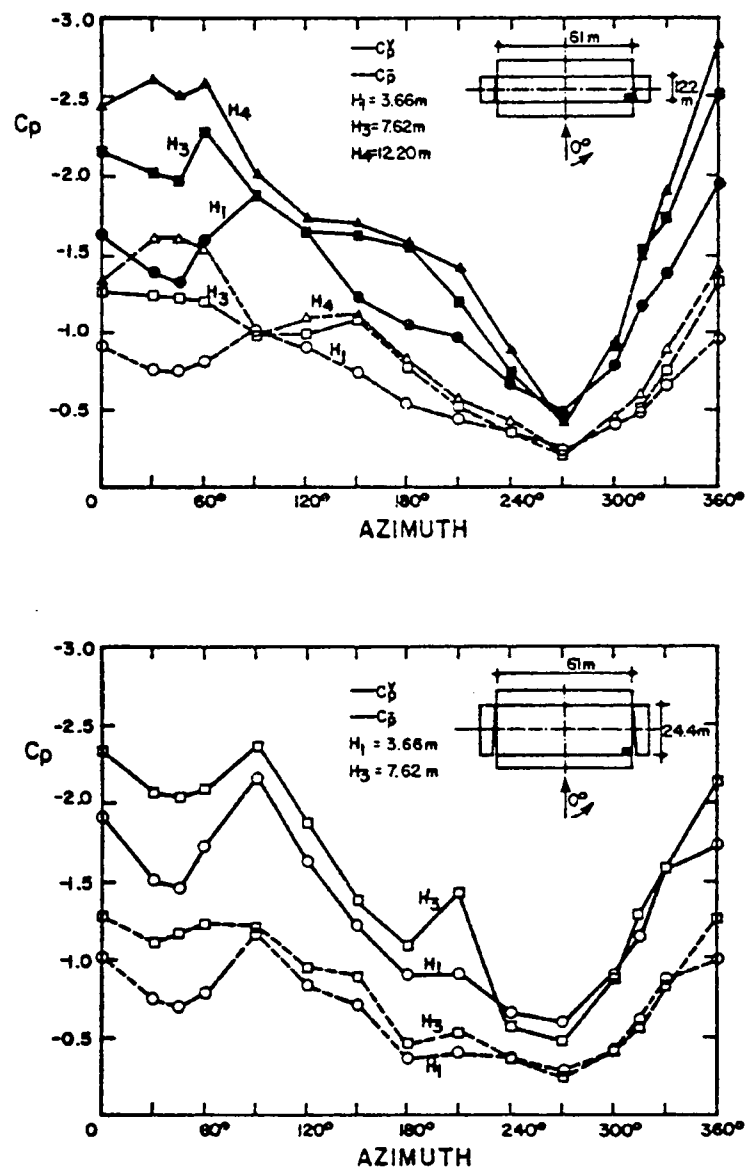


Figure 5.14: Corner area loads for buildings of various heights (from 34).

(turbulent) component of pressure responses, which is noted for activity at bluff-edges, has motivated the usage of increasingly larger geometric scales in the recent history of wind tunnel modeling (34, 50, 51, 54). Tests were conducted in simulated open, as well as, suburban exposure. The associated turbulence intensity, and velocity profiles are shown in figure 5.15.

The aspect ratio of the models was 2:1 (Length $\times$ width), and the full-scale eave height was 3 meters. Concerning the hip roof models, hip planes extended over $\frac{1}{4}$ of the building length. Because the intention of the study was to understand an aspect of hip roof behavior, namely the superior performance of hip roofs in severe wind events, a roof angle 18.4° was selected. This is a median value of the 10 to 30° range of roof angles that are common to hip roofs. It is noted that for roof angles of less than 10° , the aerodynamic behavior of hip and gable roofs is much the same (32).

Pressure coefficients presented in figures contained herein were referenced to mean dynamic pressure at building height. Again, this serves to eliminate the effect of building height upon pressure coefficient values (55). The reference wind direction was perpendicular to the ridge. Azimuths ranging from 0 to 90° were enveloped in the tests. Pressure data were taken at 10° intervals, and at 45° . The pneumatic averaging technique was used to compute area loads.

5.2.2.2.2 Results:

Figure 5.16 shows how hip and gable roof framing differ. The gable roof has a constant cross section throughout its length. Gable frames span between load bearing sidewalls. The end wall is usually only nominally attached, "toe nailed," to the roof structure, and therefore does not contribute significantly to the resistance of uplift (32).

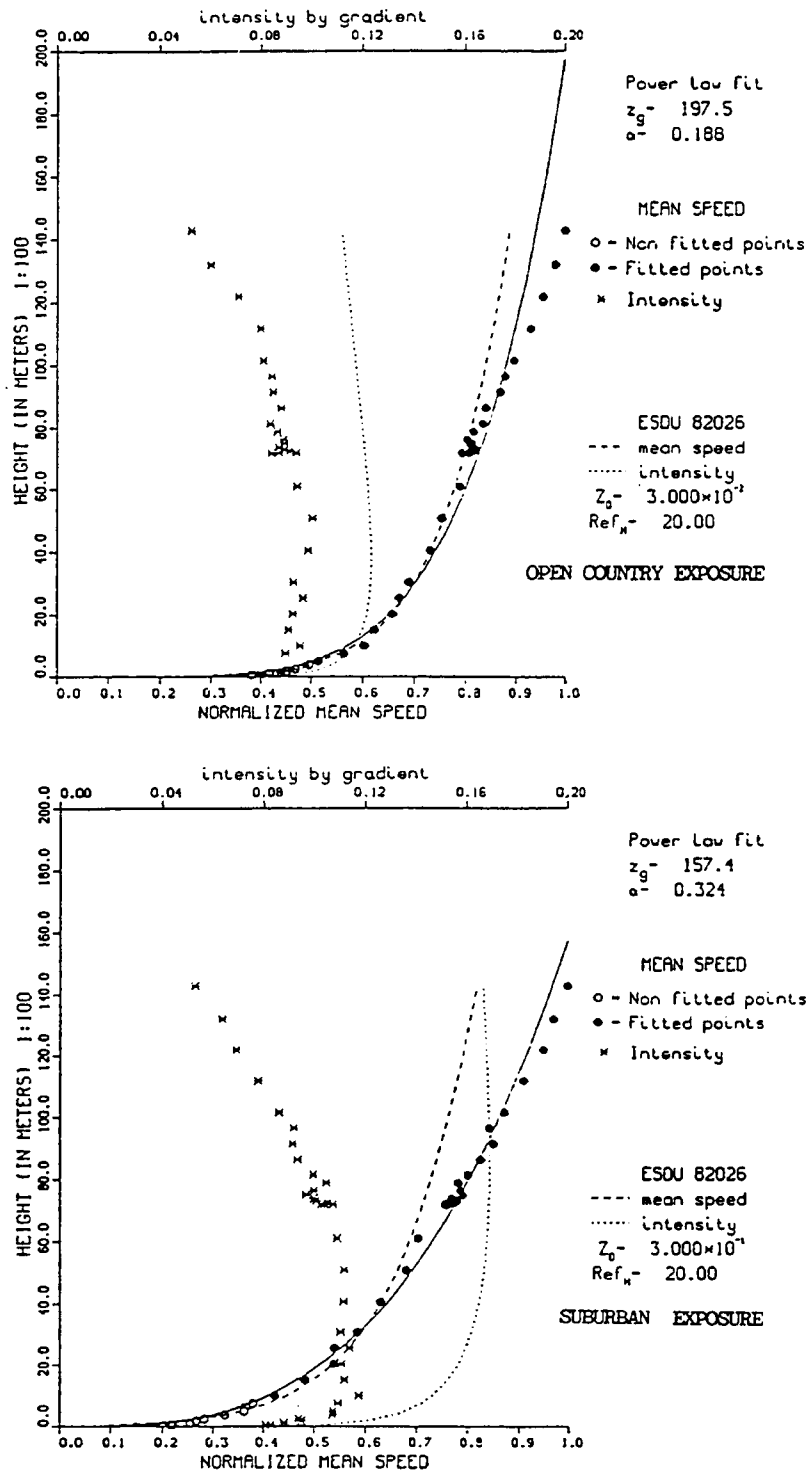


Figure 5.15: Mean wind velocity and turbulence intensity profiles, open and suburban exposure (from 32).

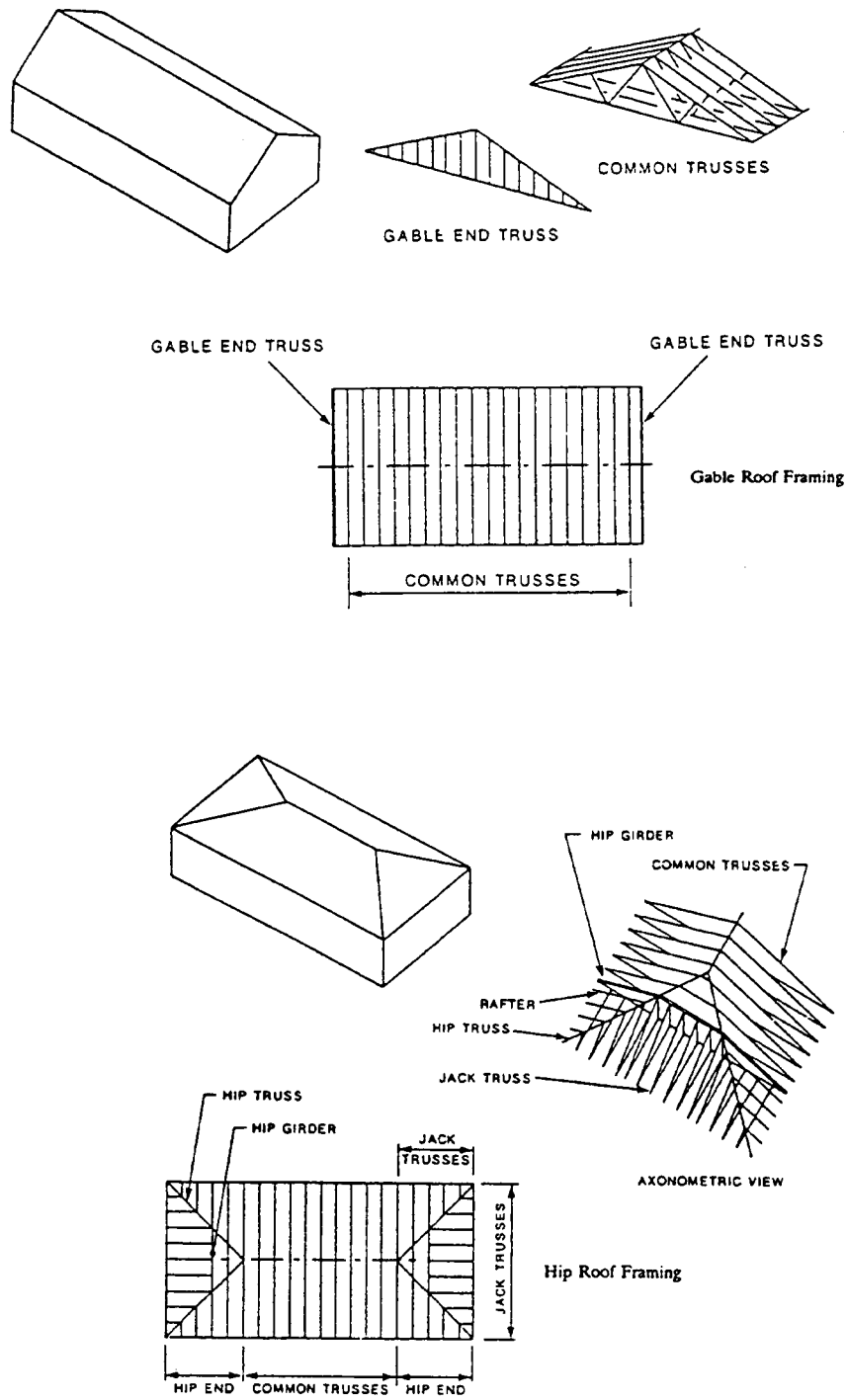


Figure 5.16: Typical Structural framing of hip and gable roofs (from 32).

The framing of the hip roof differs, such that all external walls are load bearing. Hip trusses span between building corners and hip girders. Jack trusses, in turn, span between hip trusses and external walls. The jack trusses are secured to the end, as well as, sidewalls. Therefore, all exterior walls are load bearing.

Aerodynamically, hip and gable roofs are quite different. Generally, hip construction tends to break up the spatial correlation of suction peaks observed on gable roofs. For example, wind acting in the ridge parallel direction of a gable roof produces intense span-wise correlated suction peaks acting upon the windward end truss. The first several common trusses are also affected. Conversely, wind acting in the ridge parallel direction of a hip roof will produce spatially disorganized suction peaks of considerably lower magnitudes acting upon the jack trusses. Although the aerodynamic behavior of the hip roof, itself, is seemingly very advantageous, the overall instantaneous uplift and the global overturning moments are much the same for each roof type, as shown in figures 5.17 and 5.18, respectively. This indicates that spatial disorganization of pressure distributions relative to hip roof framing accounts, primarily, for the superior performance of the hip roof in severe wind events (32).

The degree to which suction peaks are span-wise correlated relative to the first full-span truss of each roof type is indicated in figure 5.19. Uplift coefficients obtained by pneumatic averaging are plotted as a function of wind direction. As shown in figure 5.19, suction peaks, labeled "MIN," were much higher at most azimuths on the gable end truss than on the first full-span truss of the hip roof. Also, the span length of the longest jack truss was $\frac{1}{2}$ that of the gable trusses. This is indicated by the 2:1 aspect ratio of the roof plans, and the fact that the hip plane extended over $\frac{1}{4}$ of the roof length. (See figure

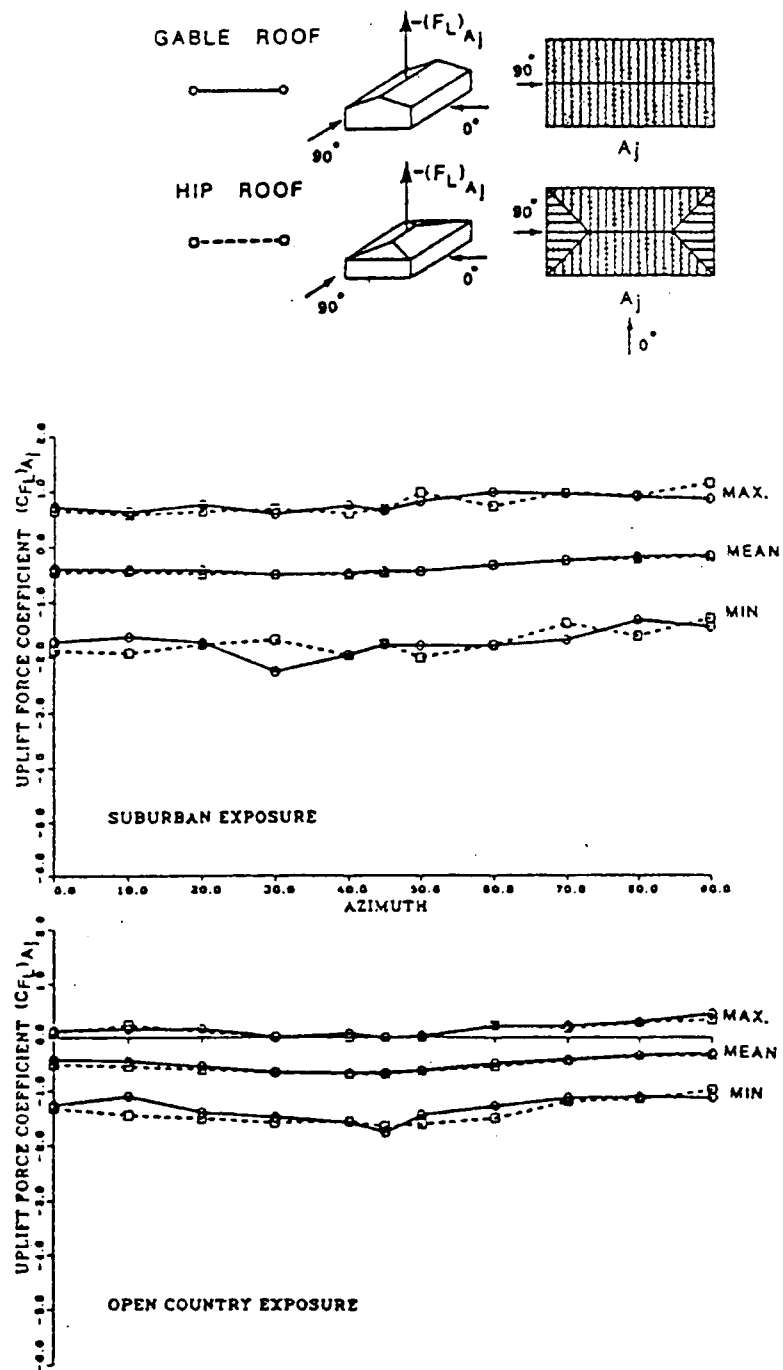


Figure 5.17: Uplift coefficients for total roof area (from 32).

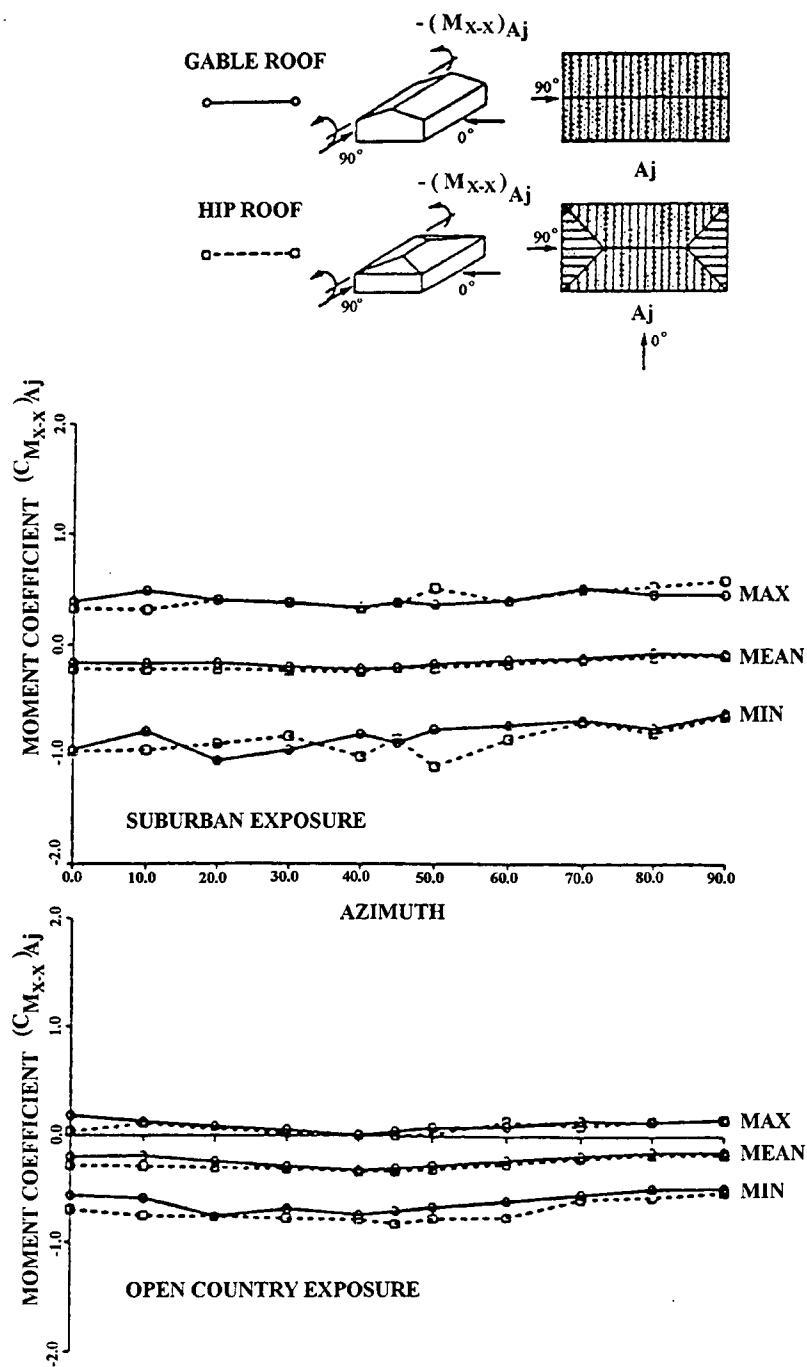


Figure 5.18: Moment coefficients for total roof area (from 32).

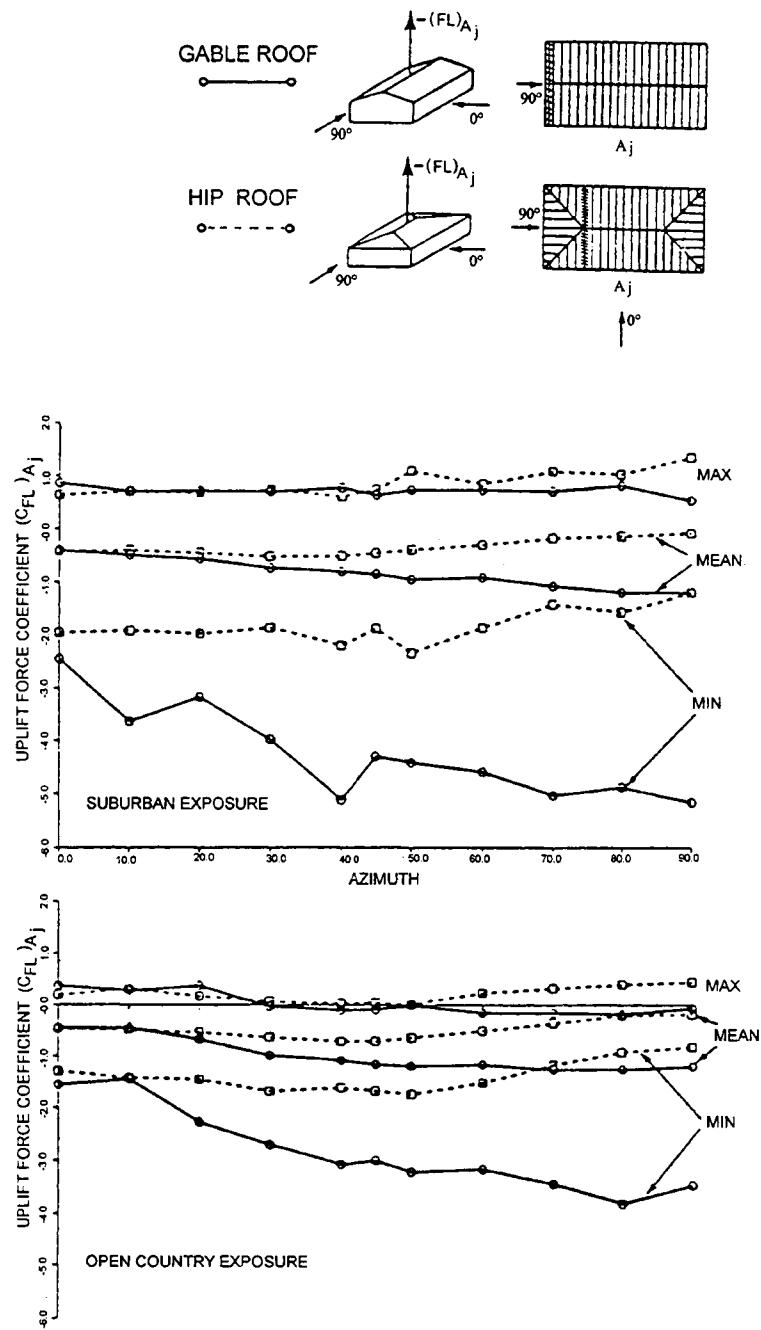


Figure 5.19: Uplift coefficients for the first full-span truss, each roof type (from 32).

5.16.) Accordingly, the end reactions of the jack trusses at the end walls were small compared to those of gable end trusses.

It is noted that the appearance in figure 5.19 of higher suction peaks in suburban exposure is a consequence of referencing pressure data within each respective exposure. Relative to open terrain, the mean component of the loading is reduced in suburban terrain, whereas the rms component of the loading ($C_{p,-}$) is increased. The net effect is that the peak loads in suburban exposure, $C_{p,+}$ and $C_{p,-}$, are about 15% lower than those of open country exposure, as reported by Ho (56).

Whereas hip roofs are framed such that pressure distributions are spatially disorganized relative to the spans of the jack trusses, bluff-edged gable end trusses cause flow separation, with the attendant formation of vortices at their leading edges (2, 21, 32, 35). Therefore, intense span-wise correlated suctions occur on gable end trusses. Therefore, large end reactions result. Because gable end trusses are often only nominally attached to end walls, failure frequently results. This type of failure is shown in figure 5.20. In severe wind events, surface elements such as shingles also perform better on hip roofs than on gable roofs. Superior aerodynamic performance of hip roofs, and not necessarily the favorable relationship between pressure distributions and hip roof framing, was likely responsible for this disparity in performance, which is featured in figure 5.21.

As indicated by figure 5.22, the suction peaks that affected the gable roof were, on average, 50% greater than those of the hip roof in the same wind event. As indicated by figure 5.19, the distributed load that affected the tributary area of the first full-span

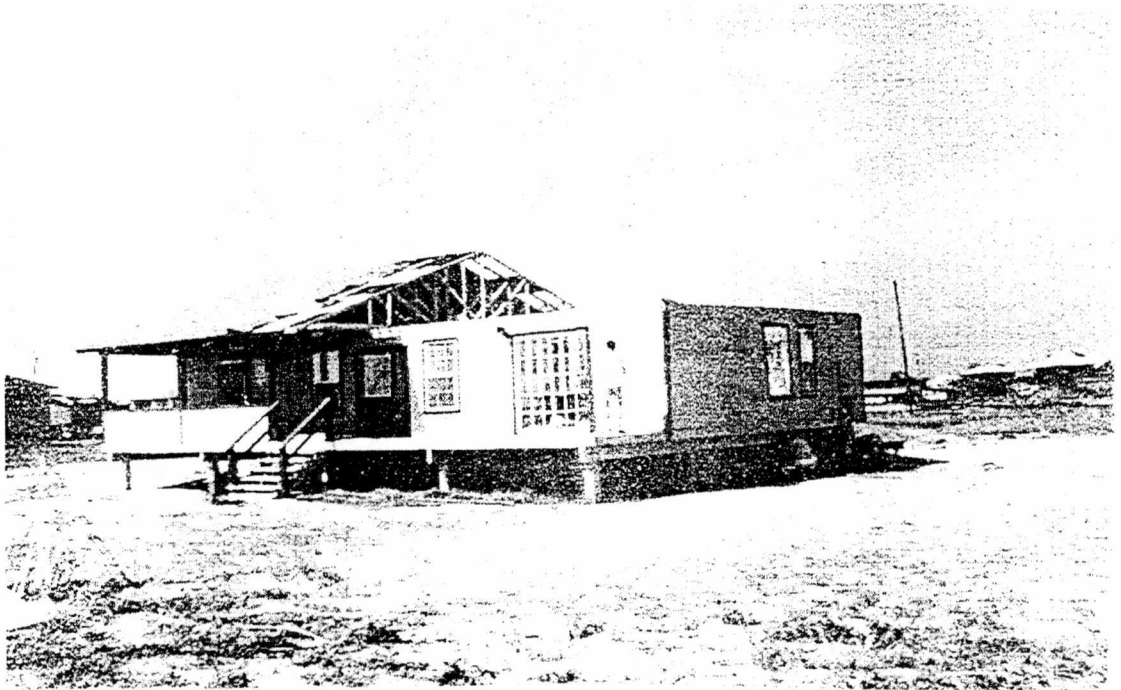
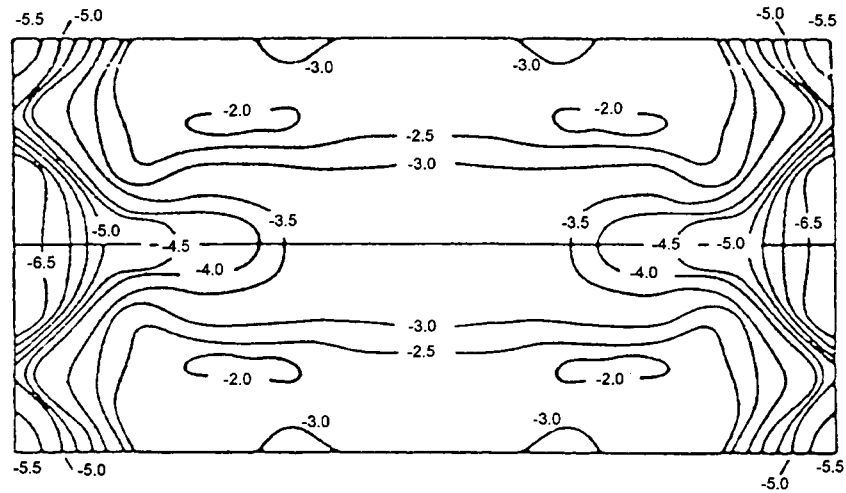


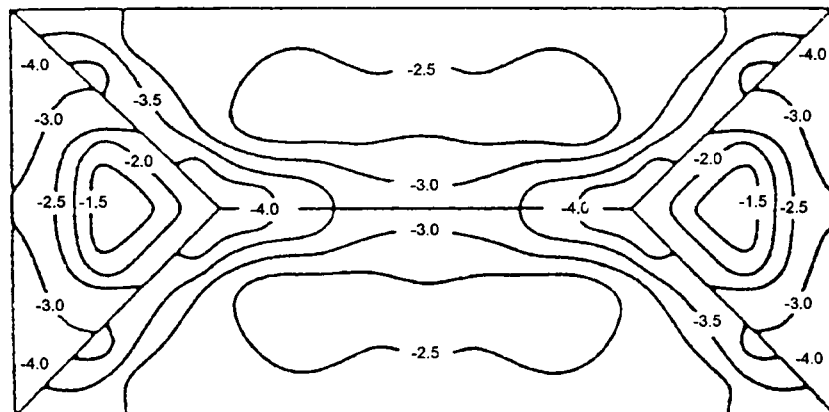
Figure 5.20: Failure of gable end trusses (from 32).



Figure 5.21: Gable end shingle damage (from 32).



GABLE ROOF



HIP ROOF

Figure 5.22: Most critical peak suction coefficients, all azimuths (from 32).

truss of the gable roof was 100% greater, on average, than that of the hip roof.

Accordingly, the failure speed of a gable roof is about 70% that of a hip roof.

$$\left(\left(C_v \right)_p \right)_{f, hip} \approx 2 \left(C_v \right)_p \bigg|_{f, gable}, C_v \propto v^2, \text{ where } v \equiv \text{velocity}, \therefore v_{f, gable} \approx 0.7 v_{f, hip} \right)$$

Using a centimeter scale, and figure 5.23, the velocity of the fifty-year wind is proportional to 27 mm. Therefore, the velocity of 70% of the fifty-year wind is proportional to $0.70 \times 27 \approx 19$ mm. This measurement corresponds in figure 5.23 to approximately the 20 and 25 year winds in inland and hurricane regions, respectively.

The annual risks of exceeding these wind speeds are, 0.04 and $20^{-1} \approx 0.05$, respectively.

Therefore, the increased risks of the structural failure of gable roofs, as compared to hip

roofs, in hurricane and inland regions are $\frac{0.04 - 0.02}{0.02} \times 100 \approx 100\%$, and

$\frac{0.05 - 0.02}{0.02} \times 100 = 150\%$, respectively (60).

5.2.2.3 Double and Multiple Span Gable Roofs:

Center for Building Studies (CBS), Concordia University, Montreal, Quebec H3G 1M8, Canada

5.2.2.3.1 Experimental Details:

A geometric scale of 1:400 was used to facilitate accurately modeling the characteristics of the atmospheric boundary layer in open country during strong wind conditions. The normalized mean velocity and turbulence profiles of the model boundary layer are denoted $\bar{v}/\bar{v}_g$ and $\bar{v}_{rms}/\bar{v}$, respectively, in figure 5.24. Generally, the difficulty in wind tunnel modeling concerning turbulence scales, is in making them large enough relative to the model to provide a realistic simulation of the turbulent component

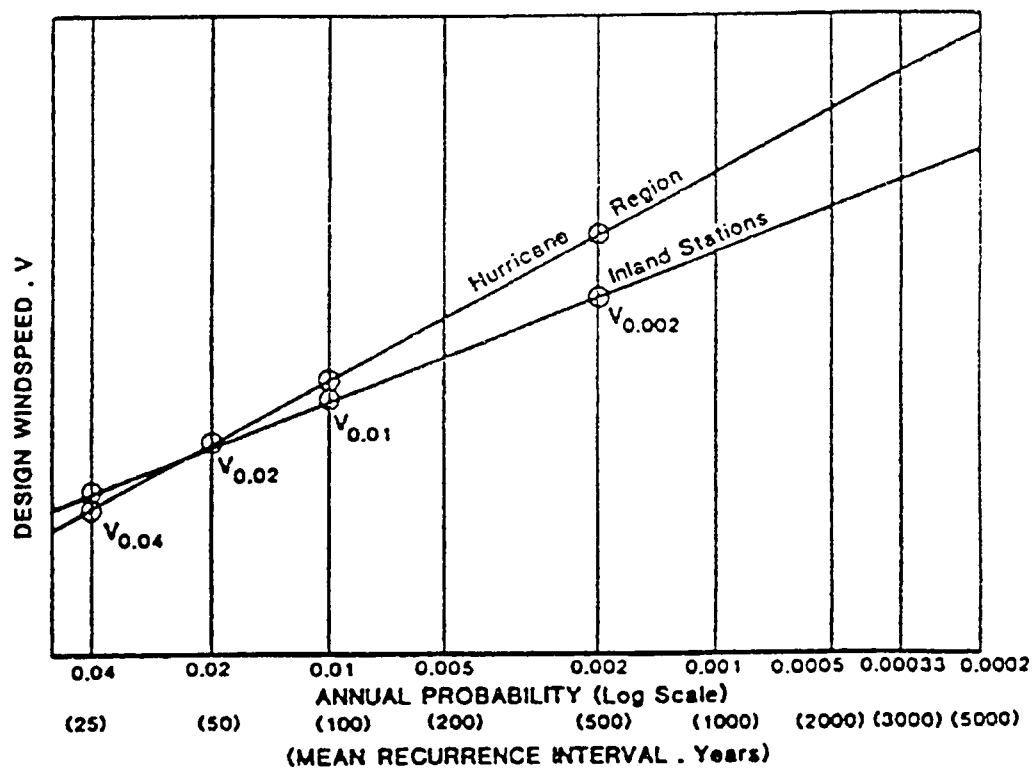


Figure 5.23: Probability distributions of wind speeds (from 60).

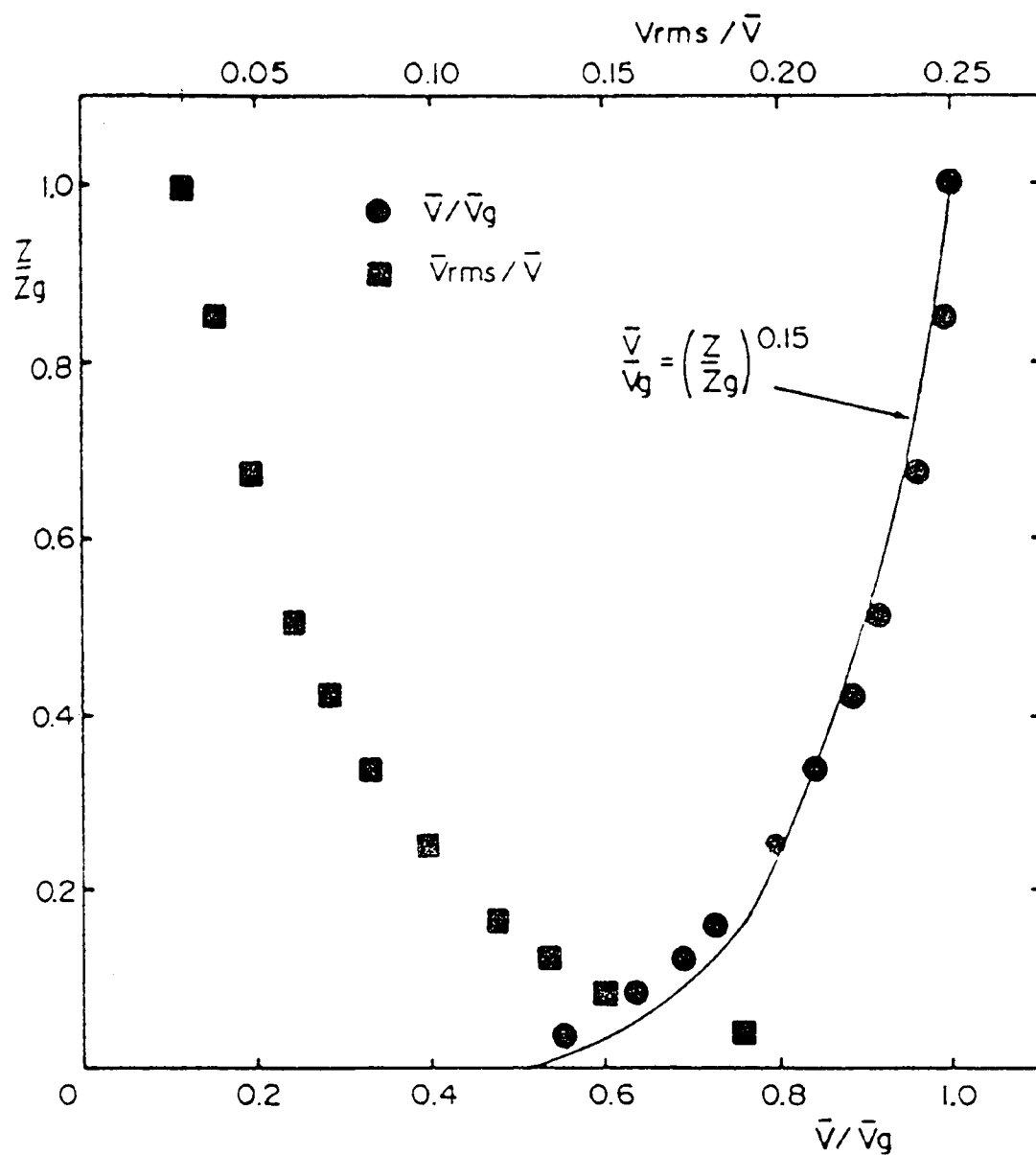


Figure 5.24: Mean velocity and turbulence profiles, open exposure (from 33).

of the dynamic pressure response (29). Therefore, reducing the geometric scale is an expedient to attaining more realistic scales of turbulence, and hence a better simulation of strong wind conditions in open terrain.

The dimensions of the model were $152.5 \times 97.5 \times 29$ mm, or $63.4 \times 40.6 \times 12.2$ m, full-scale. 4:12 and 12:12 roof-slopes were tested. The model was instrumented with approximately 100 roof pressure taps. To determine the effect of additional roof spans, dummy models were attached to the instrumented model. One, two and four span roofs were tested. Plan and elevation sketches of the models, including the full-scale dimensions, reference wind direction, roof slope, and pressure tap layout are shown in figure 5.25.

A time scale of approximately 1:100 between full and model scale, respectively, was used. The sampling rate was 500 samples per second (sps), or 5 sps full-scale. The lead tubes between the pressure taps and the manifold were equipped with damping structures so that a good frequency response would be obtained. This system of damping facilitated measuring turbulent fluctuations in the dynamic pressure response having frequencies of up to 100 HZ (one HZ full-scale) without significant attenuation of their amplitudes (33, 34,).

The sampling period and measuring scheme were as follows. Samples were taken during real time intervals of 16 seconds. During these intervals, maximum and mean pressure data were recorded. Using these data, mean and rms pressure coefficients were computed. The sixteen second sampling periods were used to ensure that (1) the pressure data included peaks that were representative of those occurring throughout full-scale 30 minute intervals; and (2) statistically stable estimates of mean and rms pressures were

Full - scale Dimensions

$$b = 40.6 \text{ m}$$

$$d = 63.4 \text{ m}$$

$$h = 12.2 \text{ m}$$

$$a/c = 4/12, 12/12$$

Wind
Direction

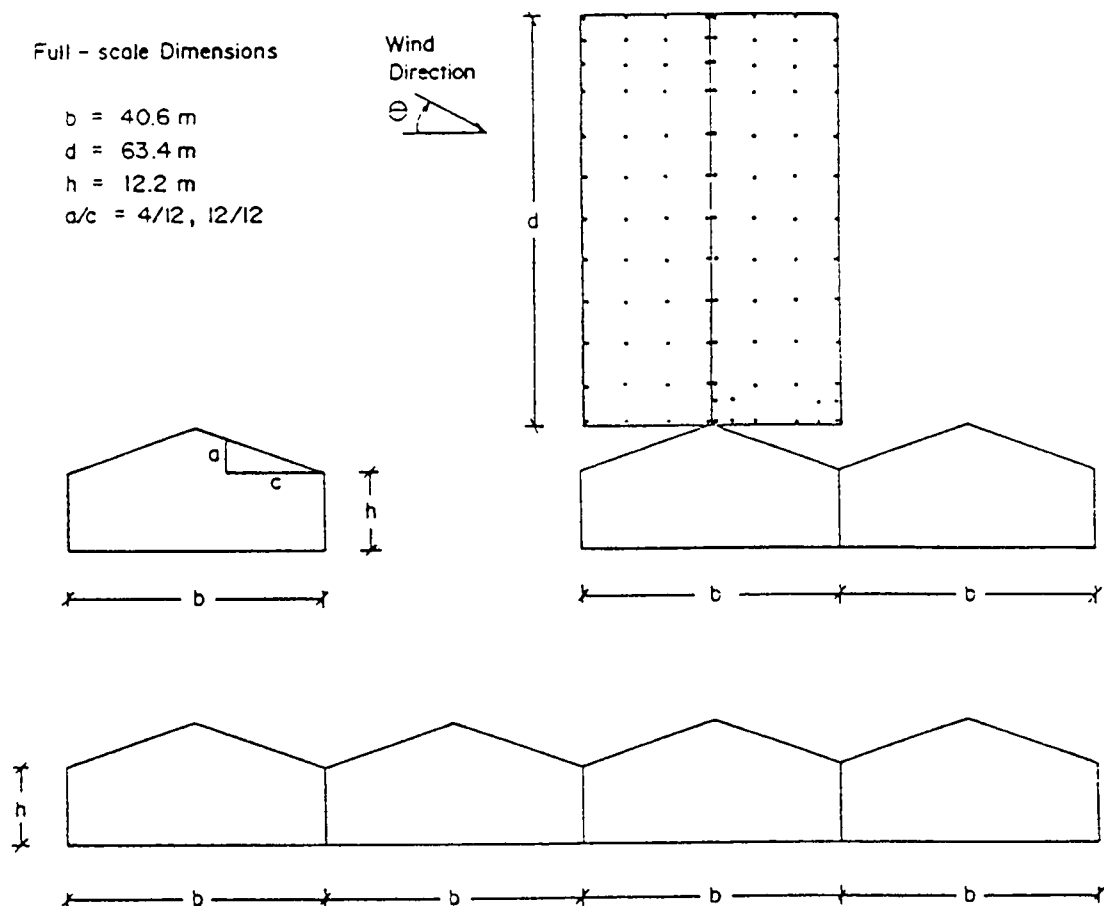


Figure 5.25: Model dimensions and pressure tap layout (from 33).

obtained. At locations where extremely large pressure coefficients were determined, pressure readings taken during five 16 second intervals were averaged to ensure the statistical stability of the pressure coefficients specified for these locations. The pressure coefficients appearing in the figures provided with the experimental results were referenced to the mean dynamic pressure at eave height.

5.2.2.3.2 Results:

As indicated in figures 5.26 and 5.27, suction peaks at ridge and trough corners are generally higher for double and multiple span roofs than for single span roofs. Notable exceptions occur at ridge corners of the 4:12 roof-sloped models between azimuths of about 30 to 45°, and 135 to 150° (figures 5.26). Within these ranges of azimuths, suction peak magnitudes generally varied inversely with the number of spans. Suction peaks affecting the 12:12 roof-sloped models occurred at an azimuth of about 90° at ridge, as well as trough corners. Additional peaks affected the trough corners of the double span 45° roof-sloped model at azimuths of about 25 and 165°. Suction peaks affecting the ridge corners of the 4:12 roof-sloped models resulted at an azimuth of about 30°. At the trough corners of the 4:12 models, suction peaks occurred between azimuths of 90 and 120°.

Figures 5.28, 5.29, and 5.30 depict the relationship between the magnitudes of the most critical peak suction coefficients, and the number of roof spans. The values of the coefficients in these figures have been factored by 0.8, for comparison to NBCC coefficients. For single span roofs, except at ridge and trough corners (figure 5.28), the NBCC and experimental coefficients agree reasonably well. For double and multiple

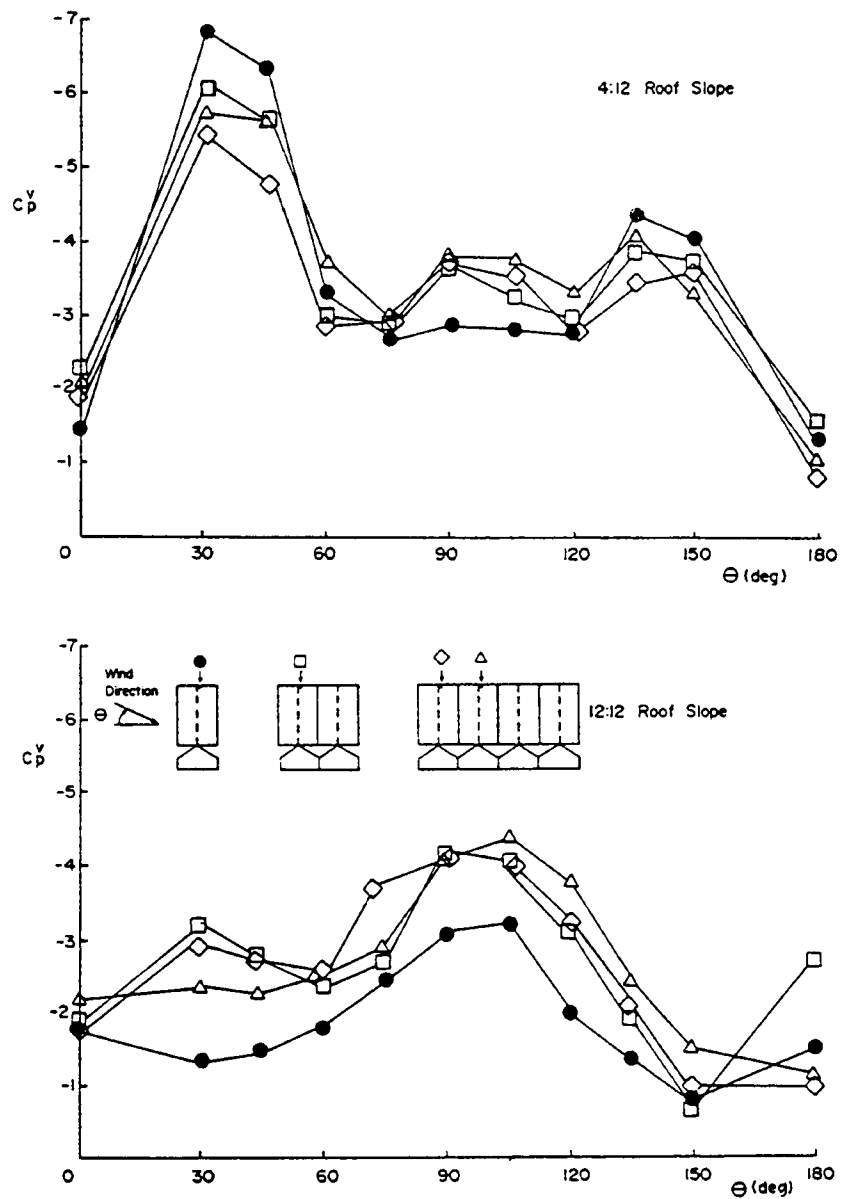


Figure 5.26: Variation in peak suction coefficients with wind direction at ridge corners (from 33).

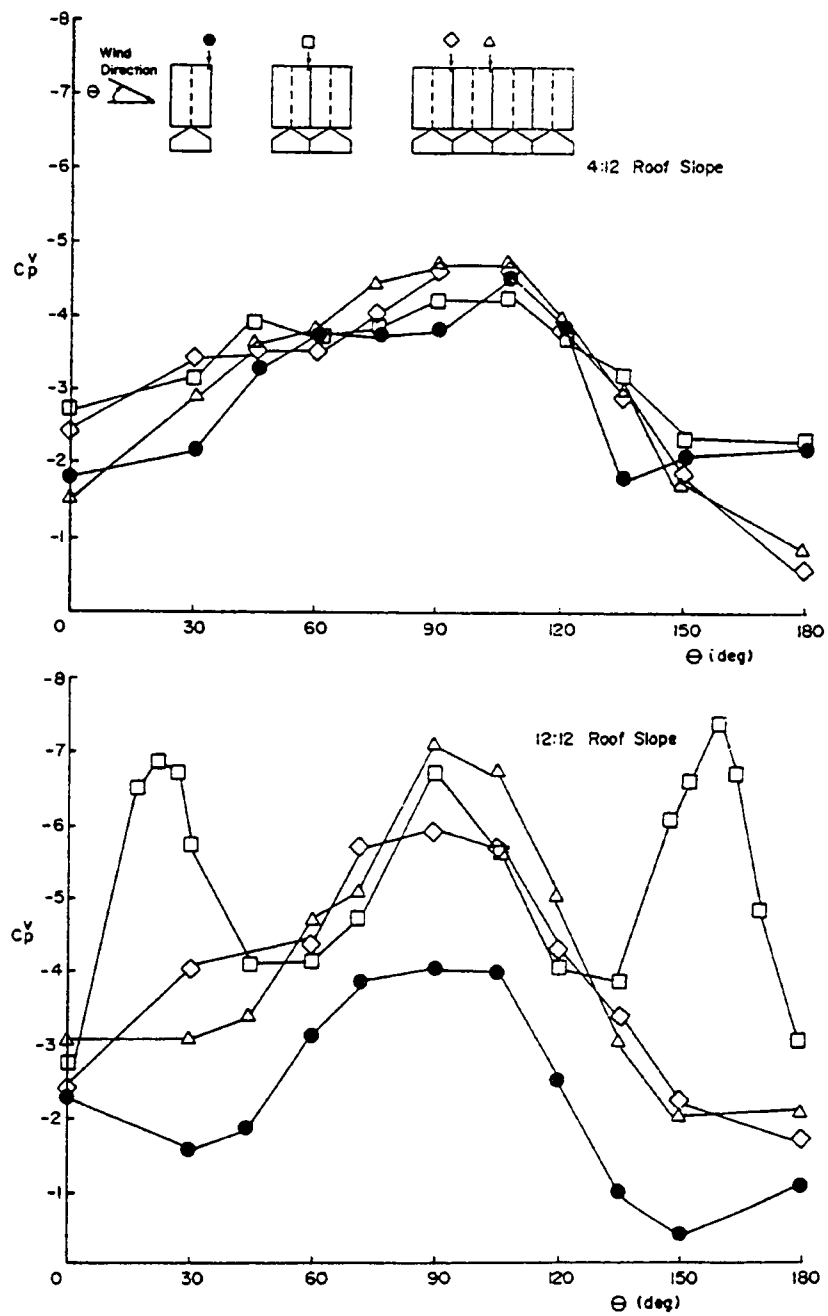


Figure 5.27: Variation in peak suction coefficients with wind direction at trough corners (from 33).

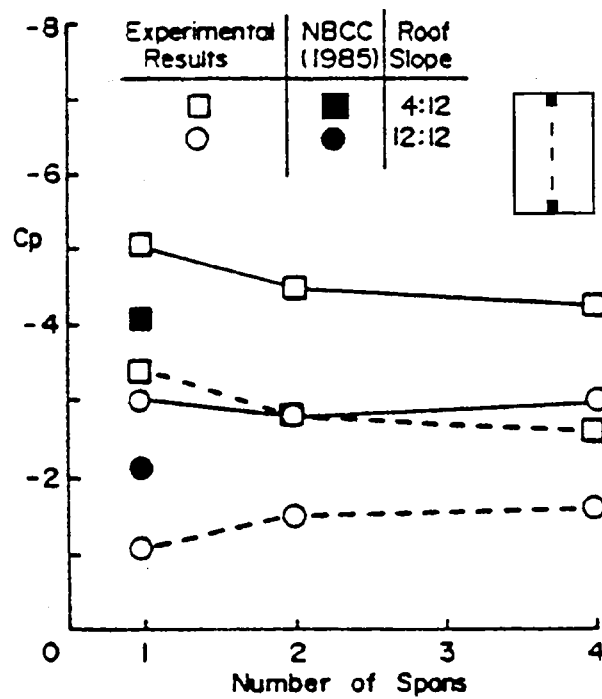
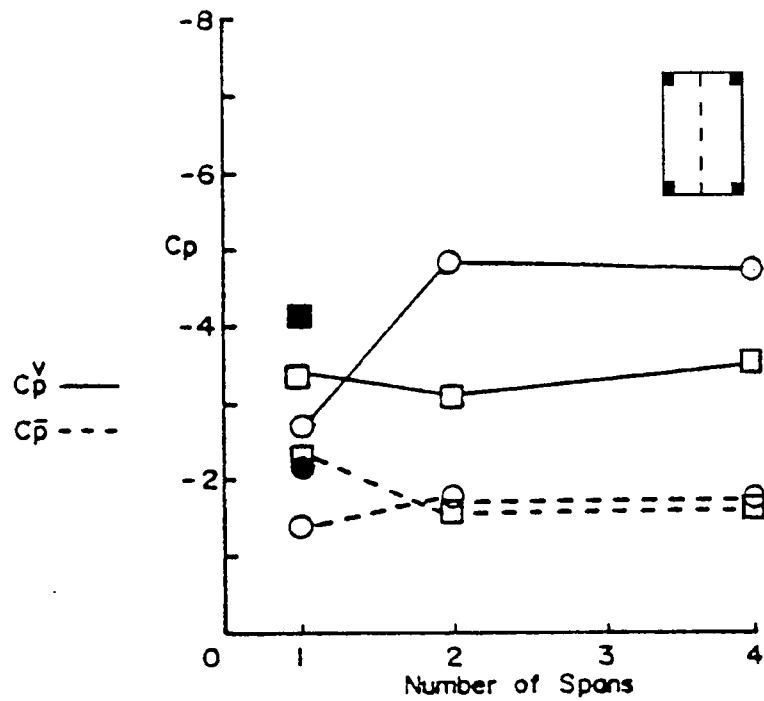


Figure 5.28: Most critical peak suction coefficients measured at ridge and trough corners, and NBCC values (from 33).

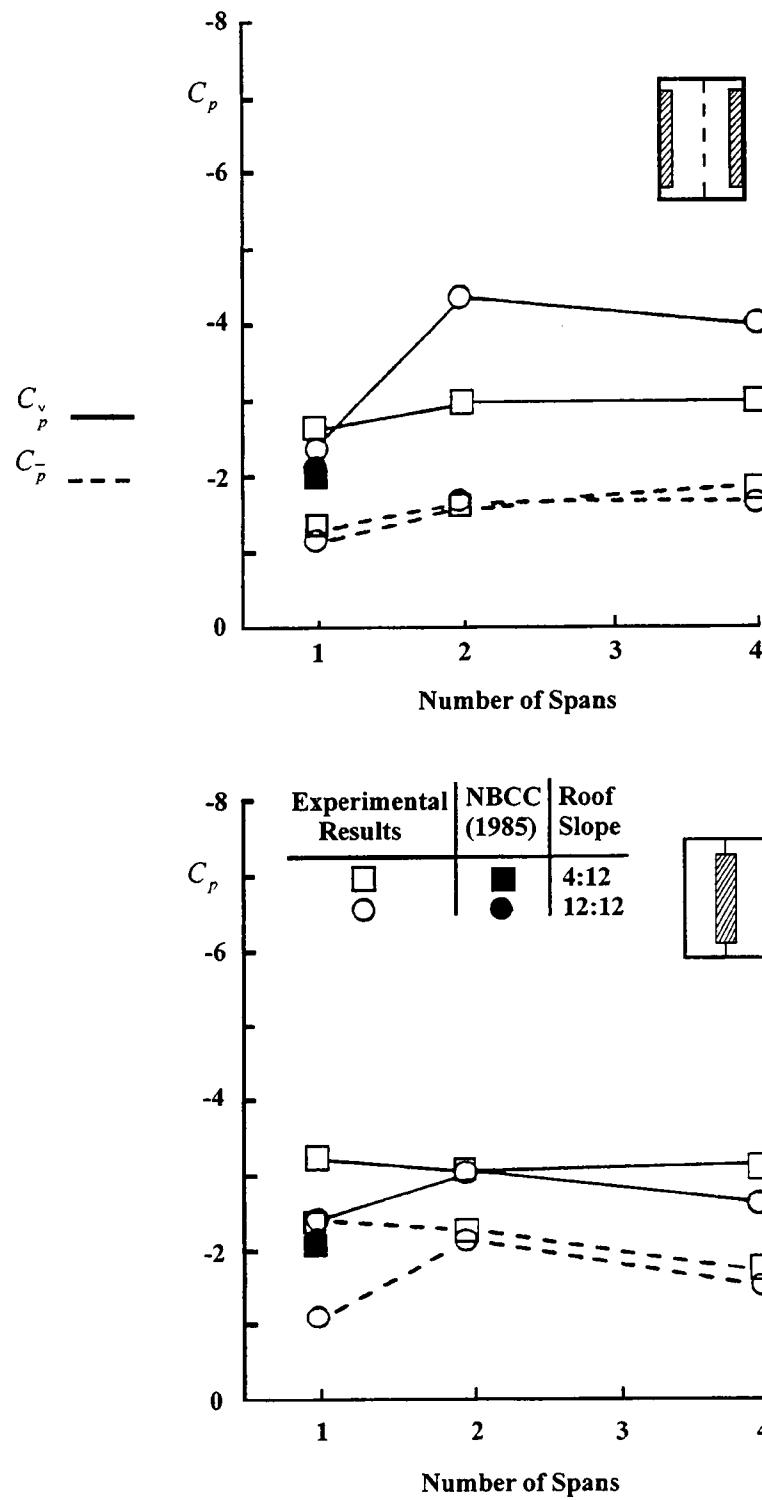


Figure 5.29: Most critical peak suction coefficients measured along ridges and in troughs, and NBCC values (from 33).

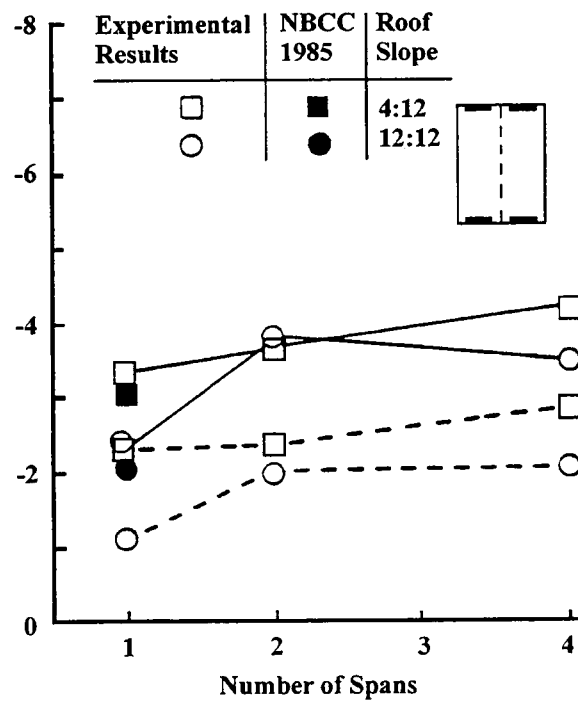
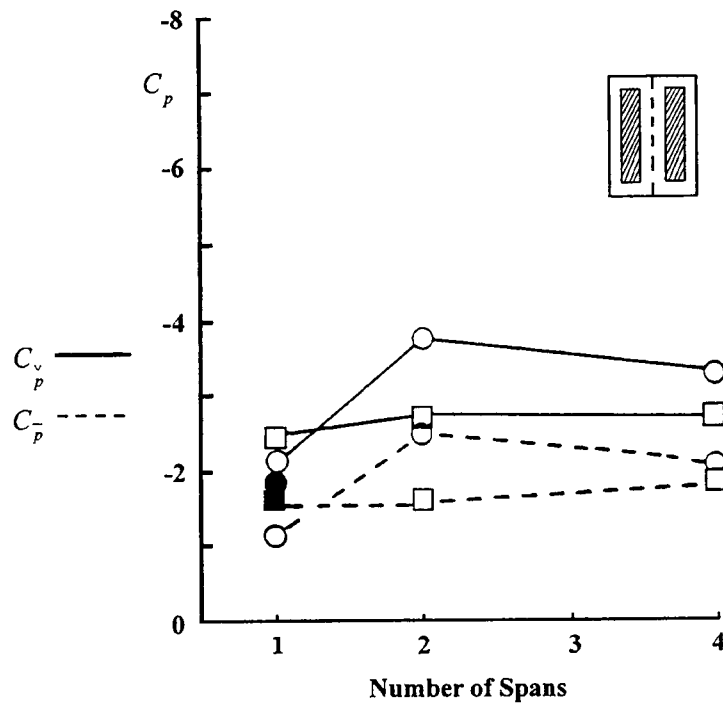


Figure 5.30: Most critical peak suction coefficients along rakes and within interior roof planes, and NBCC values (from 33).

span roofs, however, the NBCC coefficients are generally low relative to experimental values. The results of this study, however, were implemented in ASCE 7-95 (8).

The results indicated in figures 5.28, 5.29 and 5.30 agree with the results indicated in figures 5.26 and 5.27. Both sets of figures show that the magnitudes of extreme suction peaks generally varied directly with the number of roof spans. This trend was most pronounced on 12:12 roofs along the length of the troughs, and especially near trough corners (figures 5.28 and 5.29). Marked increases in magnitudes of extreme suction peaks also occurred in the bodies of the 12:12 roof planes, and along the rake, as the number of spans was increased from one to two (figure 5.30). With additional spans, these peaks remained nearly constant, or else decreased slightly. The magnitudes of extreme suction peaks affecting the rakes of the 4:12 roofs increased linearly as the number of spans was increased from one to four (figure 5.30). Generally, the effect of additional spans was to increase the most critical suction peaks by about 50% (33).

The occurrence of strong spatially correlated uplift along gable ends is typical of the aerodynamic behavior of gable roofs (figures 5.20, 5.21). This effect was markedly increased along the rake, and especially at the trough corners, of 12:12 double and multiple span roofs. Extreme peaks affecting the rakes of 4:12 roofs increased with the number of spans (figure 5.30); however, extreme peaks affecting trough corners did not vary significantly with the number of spans (figure 5.28), and extreme peaks at ridge corners decreased with the number of spans (figure 5.28). Accordingly, double and multiple span gable roofs, particularly steeper ones, may be more susceptible than single span gable roofs to end truss failure, and shingle damage near gable ends. Also, increased suction peaks affecting the body of roof planes, and the troughs of double and

multiple span roofs (figure 5.29), may result in greater susceptibility to shingle damage in these areas, as compared to single span roofs.

CHAPTER 6

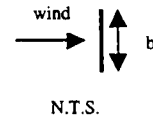
INTERPRETATION OF RESULTS AND CONCLUSIONS

6.1 Gust Factor Model (ASCE 7-95):

Essentially, by increasing the gust factor toward values associated with relatively open terrain, the gust factor model effectively accounts for channeling effects upon smaller structures embedded in highly sheltered terrain. By specifying the along wind integral length scale of turbulence (AWILST), and the longitudinal turbulence intensity (LTI) according to a minimum center of pressure height ($z_{\min}$), the effects of channeling are effectively likened to increased scales of turbulence in the approaching wind, such as are common to open terrain. Furthermore, the model specifies the amount by which channeling effects increase the overall loading of individual buildings according to a factor Q . This factor incorporates building height, width relative to wind direction and the AWILST as a single parameter, namely, the background response of the structure to along wind loading.

To obtain greater accuracy, or possibly reduced design pressures, as compared to those obtained when using the simplified method of ASCE 7-95, i.e., $G = 0.80$ for exposures A and B , and $G = 0.85$ for exposures C and D , gust effect factors may be calculated. Reduced gust effect factors are warranted when a structure is large relative to the scales of turbulence in the approaching wind (refer to sections 3.4.2, 3.4.3 and figure 3.3). Table 6.1 indicates the effects upon the gust effect factor (G) of varying least horizontal building dimensions (b) and building heights (h) in various terrain; although

Table 6.1: Values of gust effect factor (G) (ASCE 7-95), various exposures, building widths and heights (h, b). Shows influence of building width and height on background response (Q), and gust effect factor.



Exposure A																	
l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c	l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c	l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c
180	0.5	60	40	200	0.45	180	0.5	60	40	150	0.45	180	0.5	60	40	100	0.45
180	0.5	60	35	190	0.45	180	0.5	60	35	140	0.45	180	0.5	60	35	90	0.45
180	0.5	60	30	180	0.45	180	0.5	60	30	130	0.45	180	0.5	60	30	80	0.45
180	0.5	60	25	170	0.45	180	0.5	60	25	120	0.45	180	0.5	60	25	70	0.45
180	0.5	60	20	160	0.45	180	0.5	60	20	110	0.45	180	0.5	60	20	60	0.45
180	0.5	60	15	150	0.45	180	0.5	60	15	100	0.45	180	0.5	60	15	50	0.45
L_z (ft)	Q	I_z	G					L_z (ft)	Q	I_z	G	L_z (ft)	Q	I_z	G		
243	0.86	0.41	0.81					243	0.88	0.41	0.82	243	0.91	0.41	0.84		
243	0.87	0.41	0.81					243	0.89	0.41	0.83	243	0.92	0.41	0.84		
243	0.87	0.41	0.81					243	0.90	0.41	0.83	243	0.92	0.41	0.85		
243	0.88	0.41	0.82					243	0.90	0.41	0.84	243	0.93	0.41	0.86		
243	0.89	0.41	0.82					243	0.91	0.41	0.84	243	0.94	0.41	0.86		
243	0.89	0.41	0.83					243	0.92	0.41	0.85	243	0.95	0.41	0.87		
Exposure B																	
l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c	l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c	l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c
320	0.33	30	40	200	0.3	320	0.33	30	40	150	0.3	320	0.33	30	40	100	0.3
320	0.33	30	35	190	0.3	320	0.33	30	35	140	0.3	320	0.33	30	35	90	0.3
320	0.33	30	30	180	0.3	320	0.33	30	30	130	0.3	320	0.33	30	30	80	0.3
320	0.33	30	25	170	0.3	320	0.33	30	25	120	0.3	320	0.33	30	25	70	0.3
320	0.33	30	20	160	0.3	320	0.33	30	20	110	0.3	320	0.33	30	20	60	0.3
320	0.33	30	15	150	0.3	320	0.33	30	15	100	0.3	320	0.33	30	15	50	0.3
L_z (ft)	Q	I_z	G					L_z (ft)	Q	I_z	G	L_z (ft)	Q	I_z	G		
310	0.88	0.30	0.83					310	0.90	0.30	0.84	310	0.92	0.30	0.85		
310	0.89	0.30	0.83					310	0.91	0.30	0.84	310	0.93	0.30	0.86		
310	0.89	0.30	0.84					310	0.92	0.30	0.85	310	0.94	0.30	0.86		
310	0.90	0.30	0.84					310	0.92	0.30	0.85	310	0.95	0.30	0.87		
310	0.91	0.30	0.84					310	0.93	0.30	0.86	310	0.95	0.30	0.87		
310	0.91	0.30	0.85					310	0.94	0.30	0.86	310	0.96	0.30	0.88		
Exposure C																	
l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c	l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c	l (ft)	$\mathcal{E}$	z_{mn} (ft)	h (ft)	b (ft)	c
500	0.2	15	40	200	0.2	500	0.2	15	40	150	0.2	500	0.2	15	40	100	0.2
500	0.2	15	35	190	0.2	500	0.2	15	35	140	0.2	500	0.2	15	35	90	0.2
500	0.2	15	30	180	0.2	500	0.2	15	30	130	0.2	500	0.2	15	30	80	0.2
500	0.2	15	25	170	0.2	500	0.2	15	25	120	0.2	500	0.2	15	25	70	0.2
500	0.2	15	20	160	0.2	500	0.2	15	20	110	0.2	500	0.2	15	20	60	0.2
500	0.2	15	15	150	0.2	500	0.2	15	15	100	0.2	500	0.2	15	15	50	0.2
L_z (ft)	Q	I_z	G					L_z (ft)	Q	I_z	G	L_z (ft)	Q	I_z	G		
469	0.92	0.21	0.85					469	0.93	0.21	0.86	469	0.95	0.21	0.87		
457	0.92	0.22	0.86					457	0.93	0.22	0.86	457	0.95	0.22	0.87		
443	0.92	0.22	0.86					443	0.94	0.22	0.87	443	0.96	0.22	0.88		
427	0.92	0.23	0.86					427	0.94	0.23	0.87	427	0.96	0.23	0.88		
427	0.93	0.23	0.86					427	0.95	0.23	0.87	427	0.97	0.23	0.88		
427	0.93	0.23	0.86					427	0.95	0.23	0.87	427	0.97	0.23	0.88		

variations with h are only effective when $z_{\min}$ is less than $0.6h$. When selecting gust effect factors from table 6.1, and marginal building heights or widths are involved, selecting the factor corresponding to the next lower height or width is a conservative choice.

6.2 Mono-slope Roofs:

Generally, the most critical suction peaks affecting mono-slope roofs varied inversely with building width (figures 5.3 vs. 5.4, and 5.8). These peaks generally resulted from wind directed toward the ridge (figures 5.6, 5.12). Peak, as well as mean local and area suctions near ridge corners varied directly with roof slopes (figure 5.10, 5.13). Conversely, the suctions measured near eave corners varied inversely with roof angles (5.13).

Referencing pressure data to mean dynamic pressure at building height has been found to eliminate the influence of building height on pressure coefficients determined for low buildings. It was discovered, however, that the mean, as well as peak local and area averaged dynamic pressure responses of low buildings with mono-slope roofs are not independent of building height (figures 5.6, 5.7, 5.12, table 5.1). The dependence of pressure coefficients upon building height was most important near building corners (figures 5.6, 5.7), where it was most pronounced on the narrowest models tested (figure 5.7). Separate analyses of roofs and walls indicated that the influence of building height upon the dynamic pressure response was almost completely attributable to data recorded on roofs.

While these results indicated a height dependency of pressure coefficients affecting mono-slope roofs that was not apparent in wind tunnel tests of other types of roofs frequently used in low-rise construction, they did not provide enough information to zone mono-slope roof pressure distributions as a function of height. The GC_p specified for mono-slope roofs in ASCE 7-95 were developed from this study, and a subsequent study (59). Building height was not specified as a variable in ASCE 7-95 for the determination of design wind pressures affecting components and cladding of low-rise buildings. However, it was reportedly one of several factors enveloped in the wind tunnel tests from which the GC_p of ASCE 7-95 were determined (8). Accordingly, the GC_p specified for mono-slope roofs should yield design wind pressures that conservatively account for the effect of building height.

6.3 Hip and Gable Roofs:

The performance of hip and gable roofs in severe wind events differs according to their framing, as well as their aerodynamic behavior. Because end walls are usually only nominally attached to gable end trusses, they do not contribute significantly to the resistance of uplift. Hip roof framing differs, such that all walls are load bearing. Typically, hip trusses span between building corners and hip girders (figure 5.16). Jack trusses, in turn, span between hip trusses and external walls. Jack trusses are secured to end, as well as sidewalls. Therefore, all exterior walls of a house with a hip roof are load bearing.

Aerodynamically, hip and gable roofs differ in that suction peaks affecting hip roofs are about half the magnitude of those affecting gable roofs (figure 5.22).

Furthermore, hip roof framing is arranged such that suction peaks are disorganized relative to main framing members. Conversely, ridge parallel winds affecting gable roofs produce spatially correlated suctions relative to main framing members (figures 5.19, 5.22), that are stronger than those affecting hip roofs. The difference in the magnitudes of suction peaks affecting each roof type cites a distinct advantage of the hip roof regarding surface elements. However, total uplift and global overturning moments were nearly the same for each type of roof (figures 5.17, 5.18). Therefore, the spatial disorganization of suction peaks relative to main roof framing members is the primary reason for the superior performance of the hip roof in severe wind events.

Because, the distributed load affecting a gable end truss is about twice that of the first full span truss of a hip roof (figure 5.19), corresponding failure pressures should

differ by a factor of about two. Symbolically stated,
$$\left(C_{\substack{v \\ p}} \right)_{f, \text{ gable}} = \frac{1}{2} \left(C_{\substack{v \\ p}} \right)_{f, \text{ hip}} .$$

Accordingly, the wind speed producing gable roof failure should be about 70 % that of a hip roof. Considering 70% of the fifty-year wind in hurricane and inland terrain, the increased risks of structural failure of gable roofs relative to that of hip roofs are about 100 and 150% in each of these regions, respectively (figure 5.23).

Figures 6-5 to 6-7 of ASCE 7-95 for the evaluation of design wind loads for components and cladding of low-rise buildings were developed from data taken in wind tunnel tests in which factors such as geometric differences between hip and gable roofs were enveloped (8). However, substantially reduced risks of hip roof failure in severe wind events, as compared to that of gable roofs, would seem to warrant some reduction in design loads for hip roofs. The following allowances are suggested.

The one hundred and fifty percent increased annual risk assessed to gable roof failure due to severe wind events in inland terrain corresponds to a 3% increase in the annual probability of exceeding the 50-year wind $\left(\left(1.50 \times \frac{1}{50} \right) \times 100 = 3\% \right)$. Adding this 3% increase to the annual risk of exceeding the 50-year gust speeds specified as design winds in ASCE 7-95, a relative annual risk of 5% is obtained, corresponding to the 20-year wind. Accordingly, it is suggested that the inland design wind for hip roofs may be taken as the 20-year wind when the basic design wind is the 50-year wind. Using table C6-5, commentary ASCE 7-95, and conservatively selecting the next higher mean recurrence interval (MRI) nearest twenty years (25 years), a 0.93 factor is selected for application to the basic wind speed when determining design pressures for hip roofs in inland terrain.

Similarly, on the Atlantic and Gulf coasts, where the annual risk of gable roof failure due to severe wind events is only 100% greater than that of hip roof failure, a relative annual risk of 4% is calculated. Accordingly, the twenty-five-year wind is suggested as the design wind for hip roofs in the Atlantic and Gulf coast regions. Referring again to table C6-5, and considering the factors listed for hurricane regions, a 0.89 factor is selected for application to the basic wind speed when determining design pressures for hip roofs in the Atlantic and Gulf coast regions.

6.4 Double and Multiple Span Gable Roofs:

The occurrence of strong spatially correlated suction peaks in various areas of gable roofs is typical of their aerodynamic behavior (figures 5.19, 5.22). This effect was

considerably increased along gable ends of double and multiple span gable roofs (DMSGR's) (figure 5.30). Accordingly, DMSGR's may be more susceptible than single span gable roofs (SSGR's) to end truss failure, and shingle damage near gable ends. Suction peaks affecting the lengths and corners of troughs, and roof plane interiors of steeper DMSGR's were also markedly increased (figures 5.28, 5.29, 5.30) relative to those of SSGR's. Therefore, DMSGR's may be more susceptible than SSGR's to shingle damage in these areas as well.

Table 6.2 shows a comparison of experimental pressure coefficients (figures 5.28, 5.29 and 5.30) and gust pressure coefficients specified in ASCE 7-95 for DMSGR's. Except for the inconsistencies noted in zones 1 and 2 (see table 6.2), the relative sizes of GC_p specified for various zones of DMSGR's in ASCE 7-95 seem to be conservatively consistent with Stathopoulos and Saathoff's experimental results (33). However, inconsistent with the data, these GC_p , are generally *lower* than those specified for SSGR's.

Accordingly, the following procedure is suggested for determining design wind pressures for gable roofs having two or more spans. Determine the basic wind speed from figure 6-1, ASCE 7-95. Apply the factors listed in table 6.3 for each roof zone to the basic wind speed, and calculate velocity pressures (q_z) for *each* zone accordingly. Calculate the wind effective areas of each of *these* zones, and take the corresponding GC_p from the specifications for SSGR's (ASCE 7-95). Using these GC_p , and the velocity pressures calculated for each roof zone, calculate design wind pressures (p) for each of the six zones listed in table 6.3.

Table 6.2: Comparison: Experimental C_p vs. GC_p , figure 6-6, ASCE 7-95, corresponding roof zones.

Roof Zone	Code	Experiment	Comment
3 (ridge and trough corners)	Higher GC_p specified for flatter roofs	The largest C_p affected trough corners of steeper roofs. The next largest C_p affected ridge corners of flatter roofs.	To envelop the largest C_p that affected ridge, as well as trough corners, the code's specification of GC_p should be according to C_p affecting trough corners of steeper roofs. Actually, the greatest GC_p are specified for flatter roofs. According to the difference in C_p affecting steeper and flatter roofs, an approximately 5% difference between GC_p specified for steeper and flatter roofs would be expected (fig. 5.28). The difference between the highest GC_p specified for steeper and flatter roofs is actually about 4%, would be conservative if higher GC_p were specified for steeper roofs.
2 (ridges, troughs and gable ends)	Highest GC_p specified for steeper roofs. 12% reduction for flatter roofs.	Highest C_p affected troughs of steeper roofs. Next higher peak affected rake of flatter, 4 span roof. Its magnitude was about 2% lower than that of the highest peak.	The code's zone 2 includes gable ends. The C_p affecting gable ends of flatter roofs were about 2% lower than the greatest C_p affecting any zone 2 region (troughs of steeper roofs) (figs. 5.29, 5.30). Therefore, the 12 % difference between GC_p specified for steeper and flatter roofs in zone 2 seems excessive.
1 (roof plane interiors)	Zone of lowest GC_p . Highest GC_p specified for steeper roofs. 20% reduction for flatter roofs. Percent difference: zone 1 and 3 GC_p : Steeper roofs: 23% Flatter roofs: 41%	Experimental C_p generally lowest, this zone. Percent difference: zone 1 and 3 C_p : Steeper roofs: 21% Flatter roofs: 41%	The C_p affecting steeper roofs in zone 1 were 21% lower than corresponding zone 3 C_p . For steeper roofs, the greatest GC_p in zone 1 are about 23% lower than corresponding zone 3 GC_p , consistent with the 21% difference in corresponding C_p . The corresponding percent differences for flatter roofs were 41% for C_p , as well as GC_p , also indicating consistency between GC_p and C_p . The difference between C_p affecting flatter and steeper roofs in zone 1 was about 29%, and the greatest C_p affected steeper roofs. Zone 1 GC_p are about 20% lower for flatter roofs than for steeper roofs (conservative).
Summary: Except for the inconsistencies noted in zones 1 and 2, the relative sizes of GC_p specified for various zones of DMSGR's in ASCE 7-95 seem to be conservatively consistent with Stathopoulos and Saathoff's experimental results (33). However, inconsistent with the data, these GC_p are generally lower than those specified for SSGR's.			

Nomenclature: C_p : Most critical peak suction coefficients GC_p : Gust pressure coefficients, ASCE 7-95 DMSGR's: Double & multiple span gable roofs SSGR's: Single span gable roofs

Table 6.3: Basic wind speed factors for gable roofs having two or more spans.

Zone	Percent increase in critical pressure coefficients, single to multiple span gables.		Percent difference in failure wind speed (V_f), single and MSGR's.		$(V_f)_{SSGR} = X(V_f)_{MSGR}$		Factors for application to $(V_{33})_{50}$ in determination of p for MSGR's			
	$\frac{\left(C_{p, MSGR}\right) - \left(C_{p, SSGR}\right)}{\left(C_{p, SSGR}\right)} \times 100$		$\frac{\sqrt{\left(C_{p, MSGR}\right)} - \sqrt{\left(C_{p, SSGR}\right)}}{\sqrt{\left(C_{p, SSGR}\right)}} \times 100$				f_{MSGR}			
	Inland	Hurricane	Inland	Hurricane						
Roof Pitch	12:12	4:12	12:12	4:12	12:12	4:12	$30^{\circ} < \theta < 45^{\circ}$		$10^{\circ} < \theta < 30^{\circ}$	
Trough Corners (figure 5.28)	73%	—	31%	—	X=1.31	—	1.092	1.105	—	—
Ridge Corners	—	—	—	—	—	—	—	—	—	—
Troughs (figure 5.29)	79%	15%	34%	7.4%	X=1.34	X=1.074	1.104	1.105	1.023	—
Ridge	25%	—	12%	—	X=1.12	—	1.036	1.011	—	—
Interior of roof planes (figure 5.30)	76%	8%	33%	4%	X=1.33	X=1.04	1.104	1.105	1.010	—
Rake	60%	25%	26%	12%	X=1.26	X=1.12	1.083	1.063	1.037	1.015

Notes: The fifty-year wind corresponds to 27 mm, figure 5.23. Accordingly, 27 multiplied the values "X," and corresponding MRI's were scaled from figure 5.23. Factors (f_{MSGR}) for application to $(V_{33})_{50}$, were then taken from table C6-5, ASCE 7-95 commentary.

For MRI's other than those listed in table C6-5, f_{MSGR} were calculated using the following equations, which are nearest fits to the factors and MRI's listed in table C6-5. Dashes (-) indicates negligible values.

$$\text{Inland: } f_{MSGR} = 0.6160 + (9.8600E - 2)\ln(R)$$

$$\text{Hurricane: } f_{MSGR} = 0.3094 + (0.1701)\ln(R)$$

Nomenclature:

MRI:	Mean Recurrence Interval	p:	Design pressure for determination of wind loads on buildings
$(V_{33})_{50}$:	Basic wind speed, figure 6-1, ASCE 7-95		
MSGR:	Multiple span gable roof (gable roof having two or more spans)	$C_{p, p}$:	Most critical experimental peak pressure coefficients (from 33)
SSGR:	Single span gable roof	f_{MSGR}:	Factor for application to $(V_{33})_{50}$ in calculation of p for MSGR's (from 8).
V_f:	Failure wind speed	θ:	Roof angle

Use of the factored basic wind speed and observance of the zones indicated in table 6.3 would provide higher design pressures, and more discriminate zoning for DMSGR's than SSGR's, consistent with experimental results. When applied to basic wind speeds, the factors listed in table 6.3 should have the effect of increasing design pressures commensurately with the increased probability of failure indicated by the experimentally determined differences between critical suctions affecting single and DMSGR's.

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