AIC 48

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ASCE FALL CONVENTION

OCTOBER 22–26, 1979 ATLANTA, GEORGIA

WIND LOADS

ON

LOW–RISE BUILDINGS


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**WIND LOADS ON LOW–RISE BUILDINGS**


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## INTRODUCTION

The wind loads on low-rise structures have been the subject of an intensive research effort at the Boundary Layer Wind Tunnel Laboratory (BLWTL) of the University of Western Ontario. This report briefly outlines the scope and methodology of the research, reviews the primary observations, and discusses a simplified summary of work which is anticipated to be incorporated in the Commentary to the National Building Code of Canada.

### Scope of the Research

Historically, the BLWTL has had a long involvement in determining low-rise building loads as reported in references 1 to 7. Other institutions have also been occupied with this subject as discussed in references 4 and 6. The earlier studies provided useful indications of some of the important features of wind loading on low-rise buildings; however, they were not sufficiently comprehensive to confidently provide the basis for a modern code. Such a code should recognize the importance of the turbulent atmospheric boundary layer and the dominant role of the associated unsteady wind loads.

The immediate aim of the current work has been to significantly extend the data base available by determining the steady and unsteady aerodynamic loads on models representative of a wide variety of engineered low-rise buildings. The final goal has been to develop simplified descriptions of the aerodynamic loads suitable for codification. Much of the data and some preliminary analyses of the work have been presented previously in references 4, 5, 6, and 15. The reader is referred to these sources for details beyond the scope of this brief report.

Some of the features indicated by earlier studies which have guided the current studies are, for example, that the dominant loads are fluctuating and that these are not necessarily organized either spatially over the structure or in time. This has raised the question of the extent to which the effective loads are reduced with increasing tributary area. This may be particularly true in frame structures with a variety of influence lines. They have also indicated

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that terrain roughness can have an important influence on the distribution of pressures and the amplitudes of the fluctuating components. Building shape – in particular, roof pitch – continues to be a primary factor influencing the pressures.

It is worth noting that in principle the National Building Code of Canada (9, 10) reflects these various influences of terrain roughness, fluctuating gust action and the effective pressure coefficient. In this code the effective design pressure $p$ is defined by:

$$p = q \; C_g \; C_e \; C_p$$

where $q$ is the reference velocity pressure, in Canada derived from the hourly mean speed

$C_g$ is the gust effect factor,

$C_e$ is the exposure factor, and

$C_p$ is the effective pressure coefficient based on the hourly mean reference pressure.

For tall buildings, the gust factor is determined in part by the building's dynamic properties; however, for low-rise buildings, resonance of the structure does not normally play a significant role, and the gust factor becomes wholly an aerodynamic factor. As such in this work it has been convenient to work in terms of peak pressure coefficients, which are equivalent to the product $C_g \; C_p$.

The need for a review of low-rise building wind loads has been underlined by recent experience in the U.S.A. where suggested changes in code specifications using available aerodynamic data have, in certain instances, led to significant increases in design loads for low-rise buildings. This appears paradoxical when contrasted with proven performance. In particular, practical experience has shown that engineered low-rise structures – notably those monitored by the Metal Building Manufacturers Association – have had a good record of withstanding wind loads, even though designed using data (11, 12) which are to some extent suspect. The present study has thus been undertaken to resolve this dilemma and to provide a comprehensive data base for future code definition.

**Overall Approach**

Experience with wind loading problems and the early work mentioned above strongly indicate that the dominant wind loads are unsteady and vary markedly from point to point. Since these variations need not be synchronized, their overall effect may be much less than indicated by the commonly assumed simultaneous action of the worst local loads. Therefore, it has been of primary importance in this work to measure time-varying loads averaged over various tributary areas.

The methodology of doing this is discussed in more detail in references 4 to 6. Briefly, both local loads and area loads have been determined. Local loads have been derived using conventional techniques (13). For area loads, pneumatic averaging (14) has been used to provide a set of instantaneous loads associated with tributary areas of structural significance. On the roof these areas are referred to as purlin loads, and on the vertical surfaces as wall loads. An on-line computer program then samples the input load vector and multiplies it by a matrix of influence coefficients to provide a set of structural 'outputs', corresponding typically to shears, bending moments, tensions, deflections, etc. in various structural elements, as illustrated diagrammatically in Figure 1. Since this is done on-line at a high rate, the result is the time-varying behaviour of those wind-induced structural reactions which are of primary design interest. These output loads then inherently take into account the degree of coherence of the wind loads over the roof.

The program of study has required measuring aerodynamic data for a range of building geometries (width/length/height and roof slope) and for two environments (open country and suburban) as summarized in Figure 2. Other factors have also been investigated. These include loads on eaves; loads on corners; changes in building loading due to canopies, parapets and near-by structures of significant size; internal pressure loads; and probability distributions of the most significant loads. All measurements have been carried out in appropriately simulated atmospheric flow, as historically (13) this has been shown to be the only way to obtain results in good agreement with full scale experience. For both the local and distributed wind loads, instantaneous peak (effectively 1 second duration full scale), root mean square and time average (approximately 1 hour full scale) loads were recorded.

As an adjunct to this experiment, wind tunnel tests have also been carried out on a model of the full scale test house at Aylesbury, England (7) and the results are encouraging. In particular, for winds normal to a face, agreement between experiment and full scale is as good as between two similar full scale data sets obtained on different days.

**Highlights of the Study**

The primary observations of the studies detailed in References 4 and 5 are as follows:

1) *Influence of dynamic loads:* The dynamic component of all loading effects is dominant over the mean component. This is particularly true of local pressures, less so for distributed load effects. An example time history of a local pressure is shown in Figure 3.

2) *Influence of terrain roughness:* Marked changes in terrain roughness affect the pressure coefficients but the trends are not always consistent. The dynamic component increases consistently with rougher terrain. The overall peak pressures associated with a particular storm reduce somewhat as terrain roughness increases. This is illustrated in Figure 4, where the means can be seen to be markedly different in the two terrains; however, the peaks are much less affected.

3) *Roof slope, height and length:* The first two influence the loads significantly. The dependence of the load coefficients on height can be reduced considerably by referenceing them to the velocity pressure at eave height. Over the range of lengths studied, for which the length exceeded the width, there is comparatively little change in the measured loads.

4) *Scale:* Buildings differing by up to a factor of two in size may be associated with the same pressure coefficients provided the appropriate eave height velocity pressure is used.

5) *Local roof pressures:* The worst instantaneous suctions generally occur near windward corners and eaves or near the lee of the ridge. Generally for steeper roofs (≥4:12 slope) the ridge dominates, for flatter roofs the corners and eaves. Flow directed at the corners generally is the most severe case. Positive downward pressures occur also, particularly on steeper windward roofs. This, with the leeward suctions, produces unbalanced loads on the frame.

6) *Influence of tributary area on roof loads:* The effective pressures acting on roof panels can be considerably less than the very high instantaneous suctions encountered locally on roof surfaces. This is particularly true near corners for quartering winds, where the loads are not well correlated. This applies if non-simultaneous peaks are averaged and, even more so, if the peak value of pressures averaged simultaneously over the area is taken. A summary of this effect for much of the data examined is shown in Figure 5.

7) *Bay loads:* The integrated horizontal thrust, vertical uplift and frame bending moments show appreciable effects of spatial averaging of the dynamic forces. These loads are significantly less than those determined from the peak local maxima. The end bays and frames are subjected to significantly higher loading effects than are those nearer the centre of the buildings. The effect of wind angle is less on these distributed load effects, particularly for interior bays.

8) *Internal pressures:* Internal pressures are dynamic with gust factors only slightly reduced in comparison to external loads. Furthermore, the internal pressures are well correlated spatially implying their dynamic nature should be considered for structural loads. This dynamic nature and high degree of correlation is dramatically illustrated in Figure 6.

9) *Influence of parapets, eaves and nearby structures:* Low parapets tend to increase the loads near the roof corners but unsteady loads elsewhere on the roof are not markedly increased. Eaves or canopies tend to simply extend the roof area and thus become exposed to the high edge and corner loads whereas the roof over the building itself is somewhat unloaded. The pressures underneath the eaves tend to slightly unload the corners while increasing the loads on eave sections away from corners. Nearby structures generally tend to reduce the wind loads but there are some cases of close proximity and/or large relative heights where loads are increased.

## Implications for Codification

Some general implications are clear with regard to code formulation. First, the basic code form of $C_p = q\ C_e\ C_g\ C_p$ remains a good vehicle for describing the wind action. Secondly, for internal pressures, the emphasis should change so that gust factors are the norm rather than the exception. Thirdly, there is considerable evidence to support some alleviation of the exposure factor, $C_e$, below 30'. Finally there should be some changes to the coefficients recommended for low-rise buildings. It is anticipated that some of these results will be incorporated in the 1980 revisions to the Canadian National Building Code and Commentary. Those changes anticipated are a recognition of the reduction in reference pressure (i.e. $C_e$) down to 6 m, below which $C_e$ remains constant; and revision of the coefficients and associated comments, draft copies of which are included as an Appendix.

It is important to note that coefficients in the Canadian Code are referenced to hourly mean speeds which are somewhat less than the equivalent fastest mile wind speeds as indicated diagrammatically in Figure 7. This follows from the fastest mile speed being a gust speed of duration dependent on the wind speed – at 60 mph it is a one-minute average. Invariably it is averaged over a period considerably shorter than an hour. As a consequence, coefficients designed for the Canadian code are higher. An approximate relationship to correct them to a fastest mile basis is included with the data.

In application, the anticipated requirements are organized into two parts. The first provides loads which act on the 'primary' structure. These are designed to develop such wind actions as frame bending moments, overall uplift and horizontal thrust consistent with the experimental measurements of the peak values of such loads. These 'primary' structural loads include the alleviation for averaging of the smaller gusts over the structure which were implicit in the experimental measurement technique. The second set of requirements provide loads for elements of the structure having smaller tributary areas, which would normally be associated with parts or portions of a single surface of a building. The significant aspect of these loading specifications is their recognition of the reduction in load associated with increasing tributary area – ranging from small areas associated with fasteners to larger areas associated with design of girts and purlins.

An example of the application of these loading requirements is illustrated in Figures 8 and 9 in terms of specified pressures for a fastest mile speed of 80 mph at 30 feet. Figure 8 shows the loadings required for the design of the overall structural system for a building with a 30° roof pitch. This angle was chosen as it illustrates the most complex requirements, having three loading cases rather than the two associated with buildings having roofs of lower pitch. The uppermost diagram of Figure 8 is the loading associated with winds generally perpendicular to the ridge. Because of the positive pressures developed on the windward roof for high roof slopes, the additional loading requirement represented by the bottom sketch is needed arising from quartering winds or winds generally along the ridge line. Finally, the middle sketch represents a loading requirement associated primarily with longitudinal stability. Of special note in Figure 8 are the higher loading requirements for end zones. For lower roof slopes, positive pressure loading of the windward roof is not a major consideration and hence a single loading case replaces the top and bottom sketches of Figure 8. Thus for the very common case of a flat roof or a 1:12 roof slope, essentially only a single loading combination need be considered, with the only additional requirement being that of longitudinal stability.

Figure 9 similarly illustrates the loading requirements for cladding and secondary structural elements. The diagram includes the largest local loads required (associated with the smallest tributary areas considered) and the smallest local loads required shown applied over that tributary area beyond which no further load reduction is suggested. The roof slope considered here is 20° because it again represents the most complex case, having both increased loads in the area of the ridge (which disappear for lower roof slopes) and high corner point loads, (which disappear for higher roof slopes). Both primary and secondary structural loading cases should include consideration of internal pressures, values of which are included in Figures 8 and 9 for the example considered. Details for other geometries, and the influence of canopies are included in the Appendix. An apparent omission worthy of note is that no alleviation for rougher terrain is included. This essentially results from considerations typified by Figure 4, which shows that although mean pressures are dramatically reduced in built-up terrain, peak loads are not affected significantly enough – even for area loads – to warrant a separate set of coefficients – at least for the time being.

### Background to the Codified Model

Space and time do not allow a complete discussion of the method by which the detailed experimental data (representing more than a million numbers) were boiled down to the specifications of Appendix A. This process will be detailed in a later paper. Some comments are noteworthy. Constraints were placed on acceptable forms of the final code model by various considerations – the major one being that the structural loads should be represented by combinations of rectangular load distributions. The essence of the procedure was then to develop envelopes of the worst loads measured for all experimental configurations, broken down into a limited number of classes defined by the strong parameters such as roof slope; and by eliminating other parameters, such as height, by basing coefficients on roof-height reference velocity pressures. Idealized loads, within the constraints of simplicity mentioned above, were then determined which recreated the experimental values of the measured structural and local loads.

Such 'worst case' loads are unduly conservative for a number of reasons. In fact, to arrive at a reasonable load for engineering design purposes, it is most useful to consider the complete loading process from the viewpoint of reliability-based design. Such an analysis considers the uncertainty associated with the definition of the wind climate; with the actual exposure of a structure (its surrounding terrain); and with the magnitude of the actual pressure coefficients (both external and internal), associated with both the variability among experimental configurations and the variability among full scale configurations, and with the nature of the gust action itself. The indications of such analyses, as reported in preliminary form in reference 15, provide a rational basis for reducing the coefficients slightly from the worst case loads, when formulated for use with traditional definitions of wind speeds (which are derived for given return periods, independent of direction). Such allevations have been incorporated in the charts of Appendix A.


### SUMMARY

A major study of wind loads on low-rise buildings has culminated in a relatively simple formulation for the wind loading for such structures. These proposed load requirements reflect many important aspects of the wind action, such as the predominance of unsteady loads, the reduction in effective loading with increased tributary area, and the provision of separate sets of loads, intended to be used together, for design of primary structural members.


### ACKNOWLEDGEMENTS


The work described herein has been financially supported by the Metal Building Manufacturers Association, the American Iron and Steel Institute, the Canadian Steel Industries Construction Council, the National Research Council of Canada and the Boundary Layer Wind Tunnel Laboratory.


### REFERENCES


(Note: This principally lists studies carried out at the BLWTL and forming a basis for the NBCC proposals. Other studies are listed extensively in refs. 4, 6, and 15)

1. Mori, T., "On the Dynamic Wind Forces on a Flat Roof", Transactions of AIJ, No. 206, April 1973.

2. Davenport, A. G. and Surry, D., "Pressures on Low Rise Structures in Turbulent Wind", Proceedings of Canadian Structural Engineering Conference 1974 (Canadian Steel Industries Construction Council).

3. Stathopoulos, T., "Wind Pressure Loads on Flat Roofs", BLWT Report 3–1975, The University of Western Ontario, December 1975.

4. Davenport, A. G., Surry, D. and Stathopoulos, T., "Wind Loads on Low Rise Buildings: Final Report of Phases I and II – Parts 1 and 2", BLWT Report SS8–1977, The University of Western Ontario, second edition, November 1977.

5. Davenport, A. G., Surry, D. and Stathopoulos, T., "Wind Loads on Low Rise Buildings: Final Report of Phase III – Parts 1 and 2", BLWT–SS4–1978, The University of Western Ontario, July 1978.

6. Surry, D., Stathopoulos, T. and Davenport, A. G., "The Wind Loading of Low Rise Buildings", Proceedings of Canadian Structural Engineering Conference 1978 (Canadian Steel Industries Construction Council).

7. Apperley, L., Surry, D., Stathopoulos, T. and Davenport, A. G., "Comparative Measurements of Wind Pressures on a Model of a Full-Scale Experimental House at Aylesbury,

England", Proceedings of 3rd Colloquium on Industrial Aerodynamics, Aachen, Germany, June 1978.

8. Stathopoulos, T., Surry, D. and Davenport, A. G., "Internal Pressure Characteristics of Low–Rise Buildings Due to Wind Action", Proc. Fifth Int. Conf. on Wind Engineering, Colorado State Univ., July 8–14, 1979.

9. National Building Code of Canada 1975, NRCC No. 13982.

10. Commentaries on Part 4 of the National Building Code of Canada 1975, Supplement No. 4, NRCC No. 13989.

11. Blickensderfer, R., "A Primer of Wind Loads on Gable Roof and Flat Roof Buildings", Metal Building Manufacturers Association Publication 4M 7–73.

12. Metal Building Systems Manual, Metal Building Manufacturers Association, Cleveland, Ohio, 1974.

13. Surry, D and Isyumov, N., "Model Studies of Wind Effects – A Perspective on the Problems of Experimental Technique and Instrumentation", Proceedings of the Sixth International Congress on Instrumentation in Aerospace Simulation Facilities, Ottawa, Canada, Sept. 22-24, 1975.

14. Surry, D. and Stathopoulos, T., "An Experimental Approach to the Economical Measurement of Spatially Averaged Wind Loads", J. of Industrial Aerodynamics, Vol. 2, No. 4, Jan. 1978.

15. Stathopoulos, T., "Turbulent Wind Action on Low-Rise Buildings", Ph.D. Thesis, Univ. of Western Ontario, Faculty of Engineering Science, Feb. 1979.

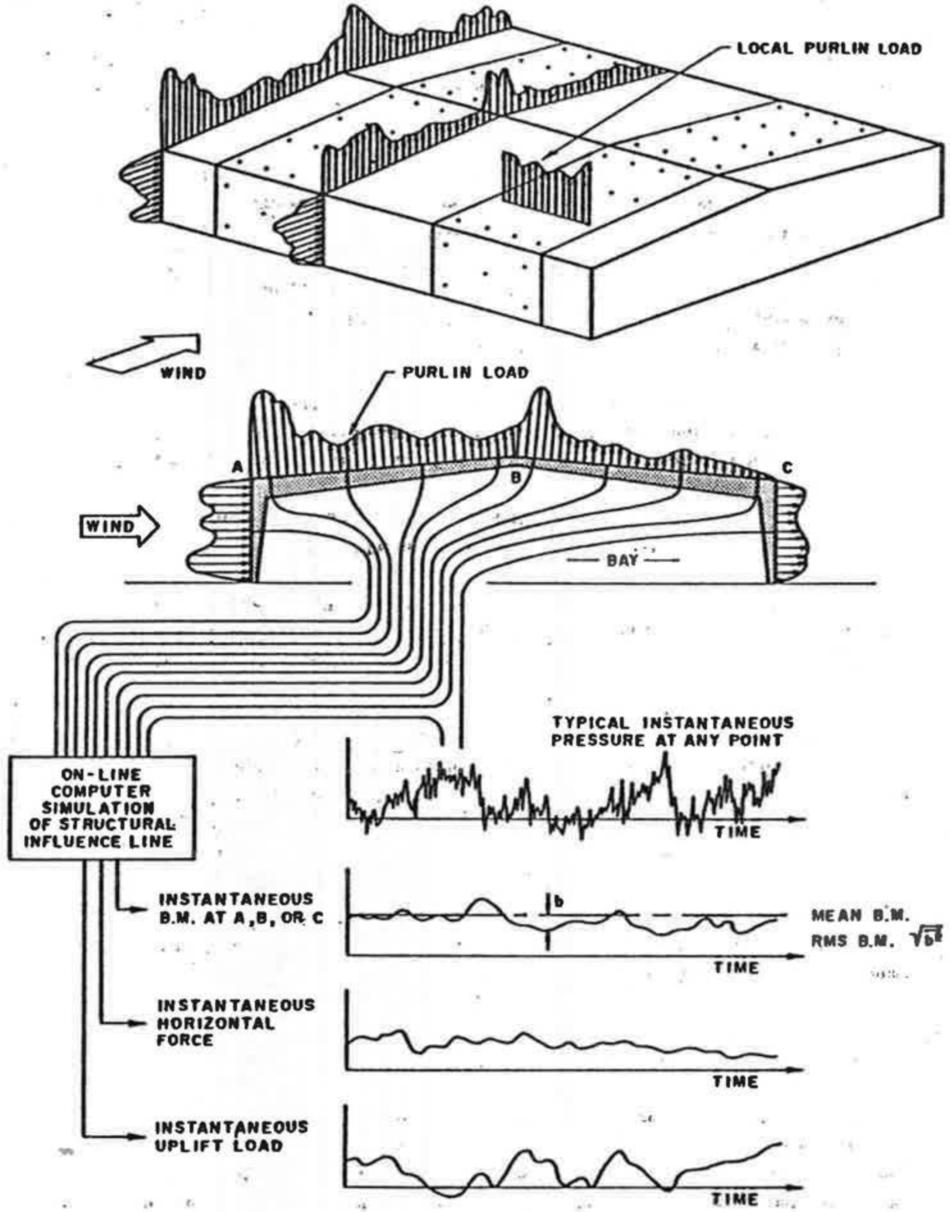



FIG. 1 MEASUREMENT OF UNSTEADY WIND LOADS ON A LOW–RISE STRUCTURE

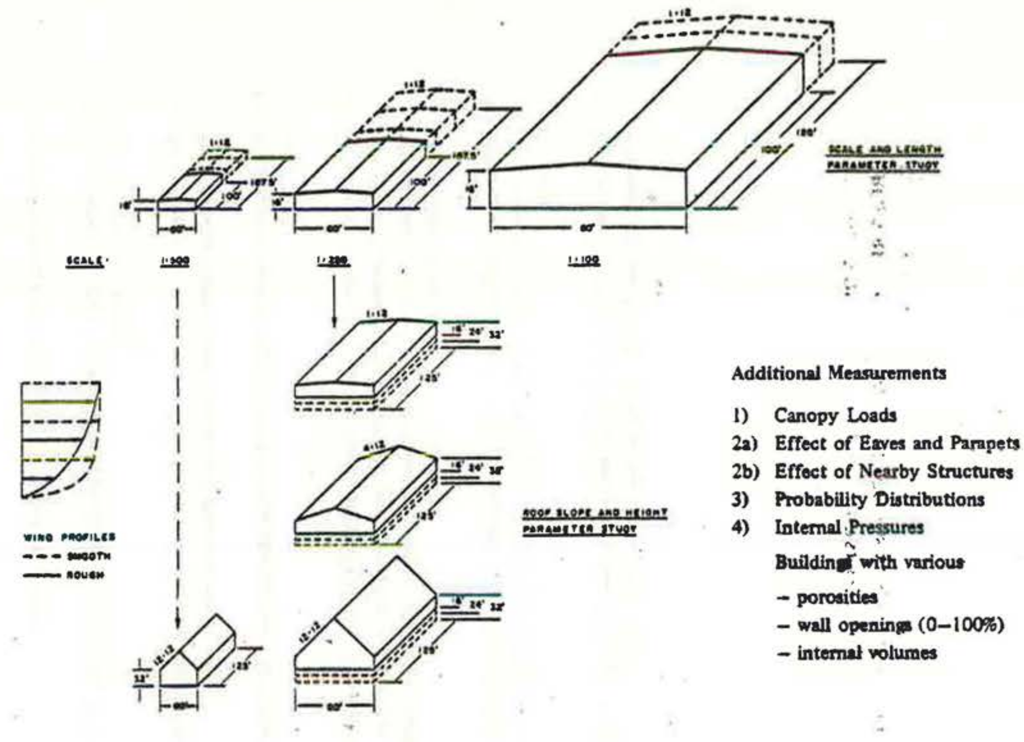



FIG. 2 DIAGRAMMATIC VIEW OF THE CONFIGURATIONS EXAMINED

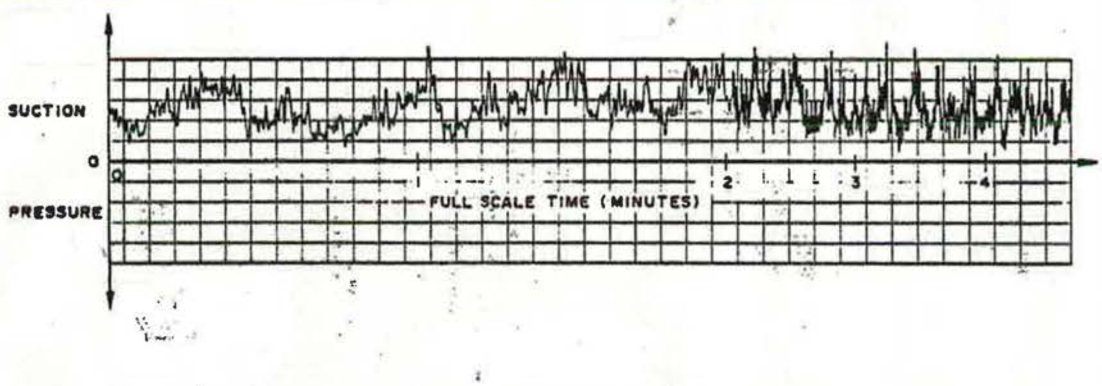



FIG. 3 TYPICAL TRACE OF SUCTION ACTING ON A POINT AT THE EDGE OF A 1:12 ROOF

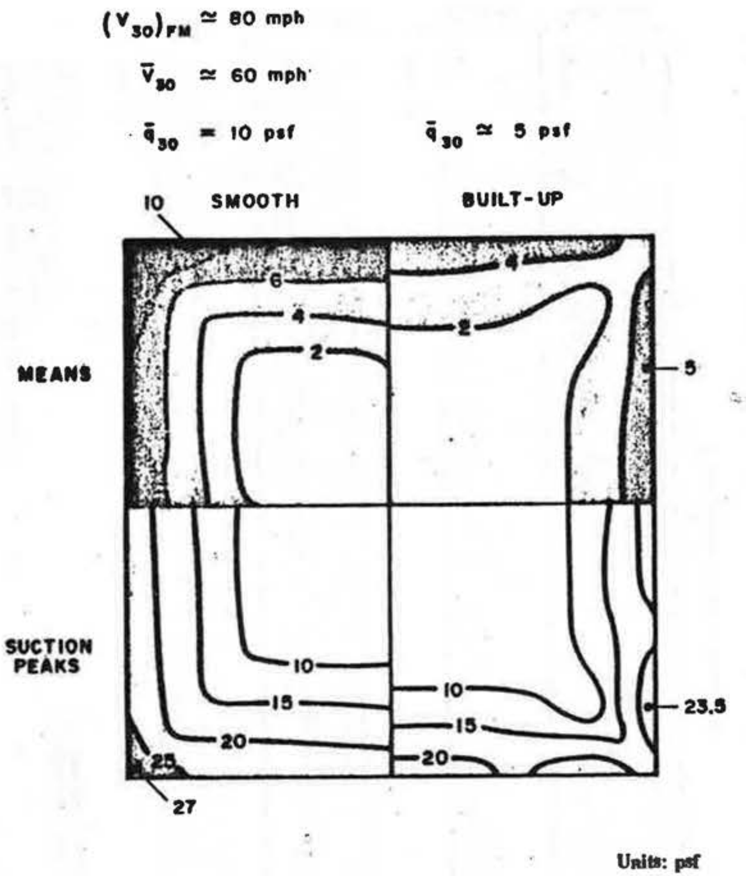



FIG. 4 CONTOURS OF WORST MEAN AND WORST PEAK SUCTION ROOF PRESSURES FOR ALL WIND DIRECTIONS FOR A TYPICAL FLAT ROOF BUILDING

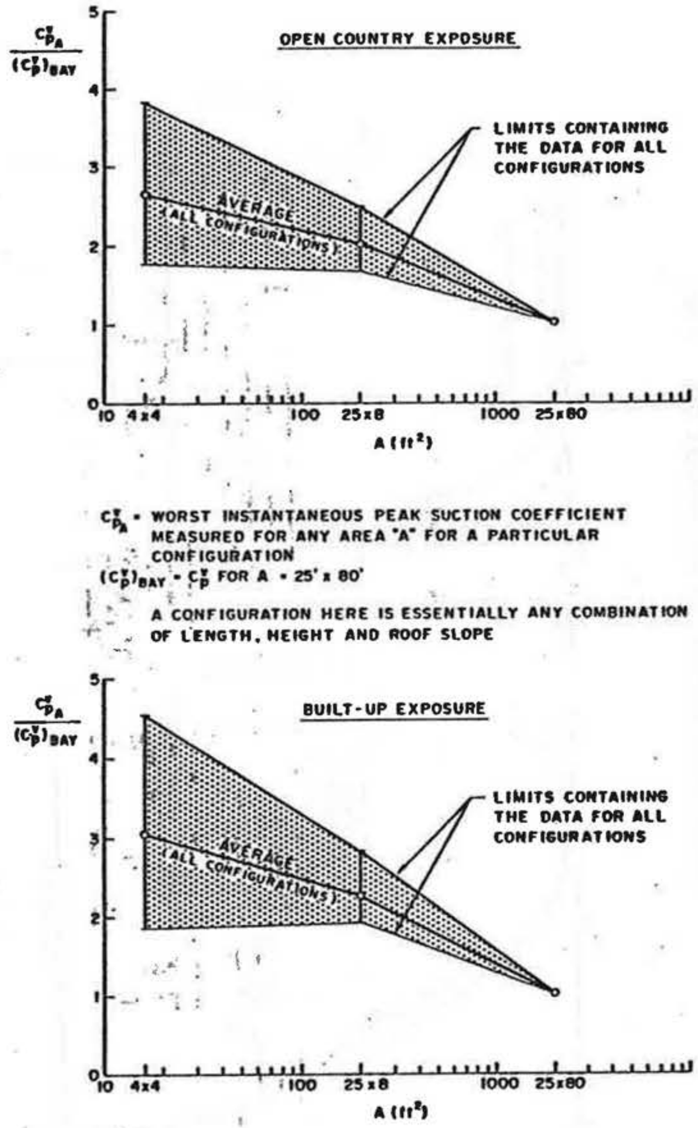



FIG. 5 ILLUSTRATION OF THE REDUCTION OF EFFECTIVE UPLIFT LOADING WITH INCREASING AREA, NORMALIZED FOR MANY BUILDING CONFIGURATIONS

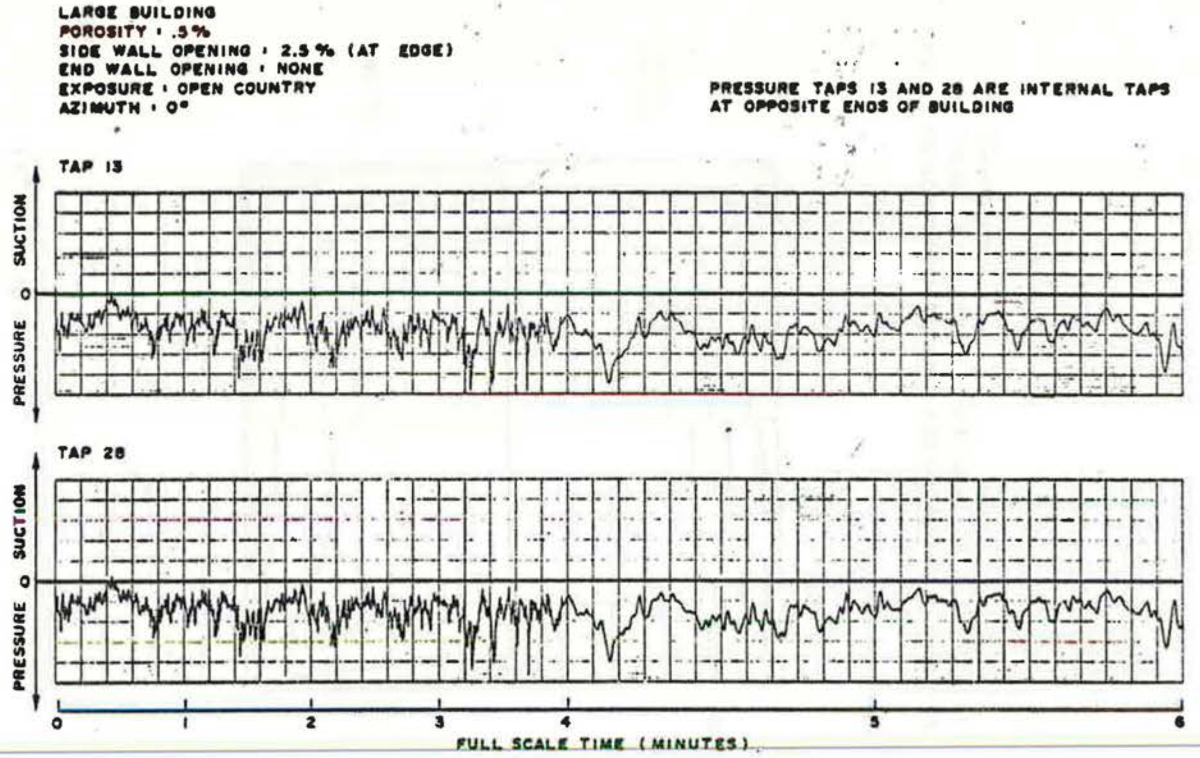



FIG. 6 SIMULTANEOUS TIME TRACES OF INTERNAL PRESSURES FOR TWO DIFFERENT TAPS ILLUSTRATING THE DYNAMIC NATURE OF THE PRESSURES AND THEIR HIGH DEGREE OF CORRELATION

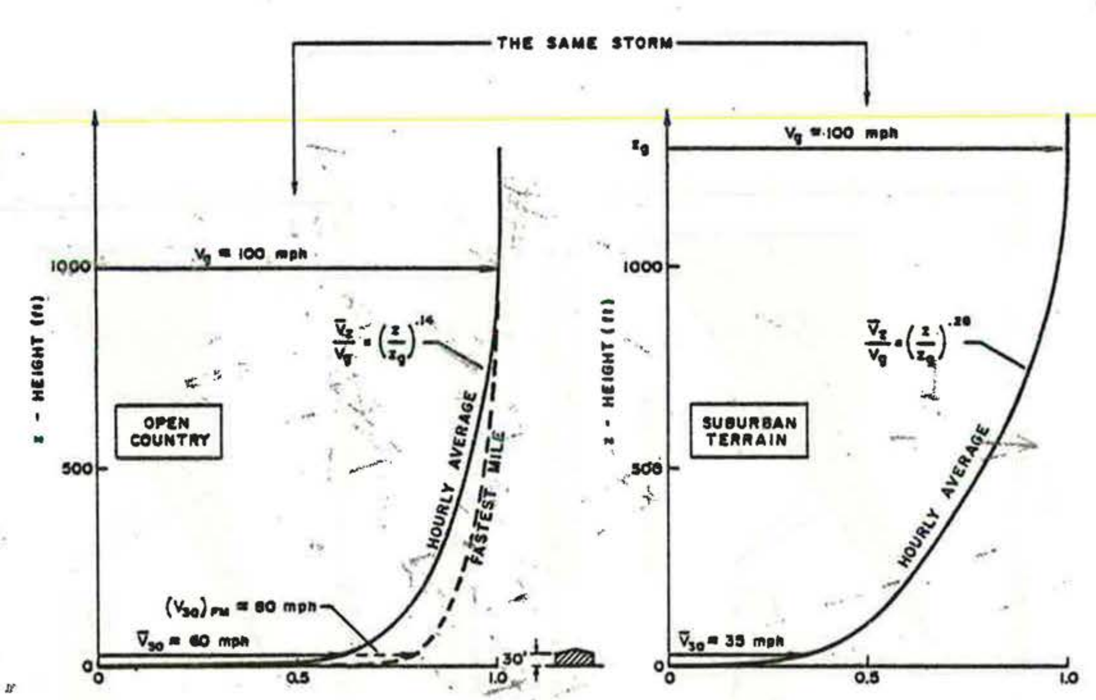
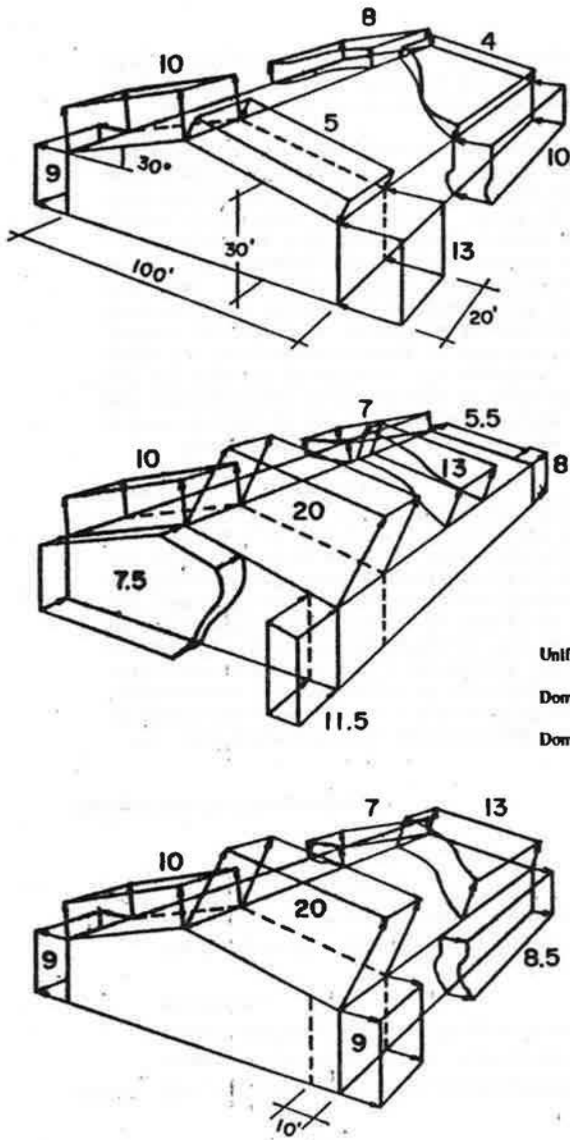



FIG. 7 THE RELATIONSHIP BETWEEN REFERENCE SPEEDS




$(V_{30})_{FM}$ ≈ 80 mph

$\bar{V}_{30}$ ≈ 60 mph

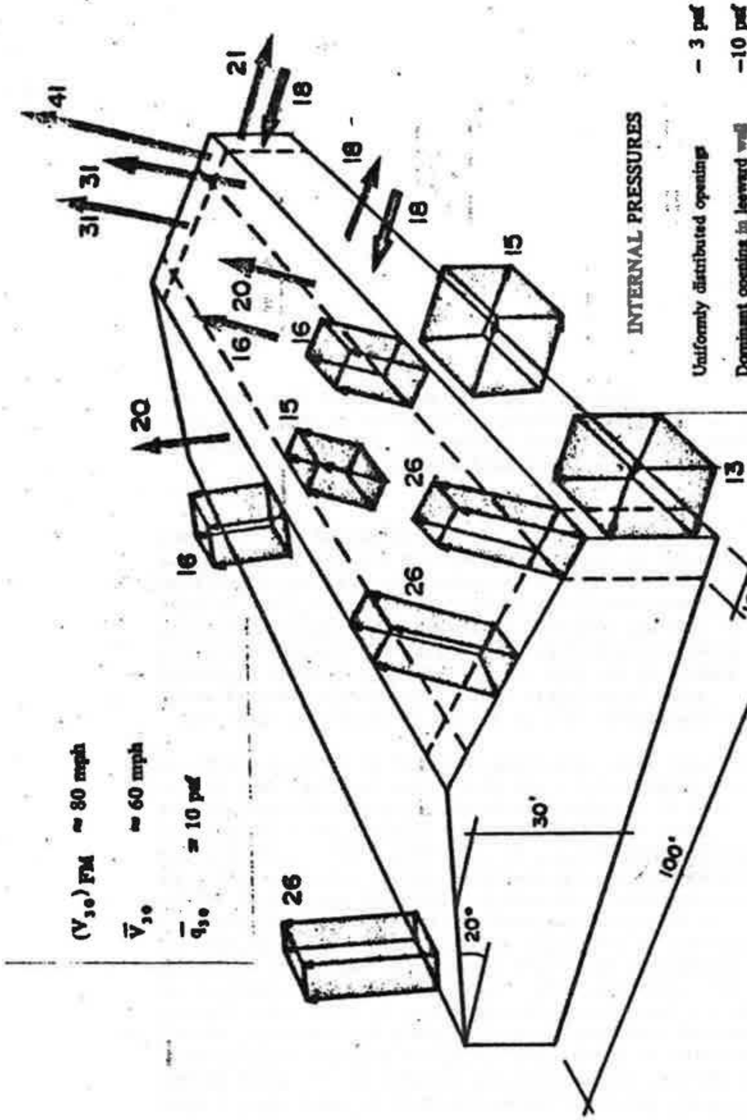
$\bar{q}_{30}$ = 10 psf

**INTERNAL PRESSURES**

| | |
|---|---|
| Uniformly distributed openings | − 3 psf |
| Dominant opening in leeward wall | −10 psf |
| Dominant opening in windward wall | 14 psf |

Units: psf

**FIG. 8 EXAMPLE OF APPLICATION OF PRIMARY STRUCTURAL LOAD REQUIREMENTS FOR A LOW–RISE BUILDING**

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$\bar{q}_{30}$ = 10 psf

Units: psf

**FIG. 9 EXAMPLE OF APPLICATION OF PRESSURES FOR THE DESIGN OF CLADDING AND SECONDARY STRUCTURAL MEMBERS OF VARIOUS TRIBUTARY AREA FOR A LOW–RISE BUILDING**

## APPENDIX A

## DRAFT WIND LOADING SPECIFICATIONS FOR THE COMMENTARY TO THE NATIONAL BUILDING CODE OF CANADA

**Caveat:** The specifications shown below are excerpted from a draft copy of the proposed commentary for illustration only. Minor errors and inconsistencies may still be present. Furthermore these specifications are still subject to change and to final acceptance.

The figures for this Appendix are numbered in order; however, the original titling and draft copy figure numbers have also been retained for reference. It is to these that the notes and text refer.

### Excerpts from Draft Commentary

31. The information on external and internal pressure coefficients given in Figs. B-6 to B-23 covers the requirements for the design of the cladding and the structure as a whole for a variety of simple building geometries. With the exception of B-6 to B-9, the values of the pressure coefficients $C_p$ are given as either time and spatially-averaged pressure coefficients or simply as time-averaged local pressure coefficients, $C_p^*$. In B-6 to B-9, dealing with low rise structures, values of the product $C_p$ $C_g$ are given; this is the form both in which they are used, and of the basic wind tunnel data from which they were derived.

B-6 to B-9 become A-1 to A-4

32. The internal pressure coefficients $C_{pi}$ define the effect of wind on the air pressure inside the building and are important in the design of both cladding elements and the overall structure. The magnitude of these coefficients tends to be uncertain owing to the influence of openings (either intended, or unintended in the case of window breakage) and the normal ventilation of the building envelope. As a consequence, internal pressure coefficients may be wide-ranging. Recent wind tunnel studies on low rise structures (12) have indicated that for the normal ranges of building openings (> .5% of surface area) the internal pressures may have lower mean values than anticipated formerly but they fluctuate significantly and are correlated to external fluctuations.

In the face of these uncertainties an appropriate treatment of internal pressures for both high and low rise structures is to use the coefficients in Fig. B-11.

See Table A-1 excerpted from Fig. B-11

The formula (a) in 4.1.8.1.(2) may be used in most situations, including low rise structures except in those cases where there are dominant openings.

33. Figs. B-6 to B-9 refer to low buildings and present recent data obtained from systematic boundary layer wind tunnel studies. In several instances these data have been verified against available full scale measurements. The coefficients are based on the maximum gust pressures lasting approximately 1 second and consequently include an allowance for the gust factor, $C_g$. The coefficients therefore represent the product $C_pC_g$. An innovative feature of these diagrams is the reference to the tributary area associated with the particular element or member over which the wind pressure is assumed to act. In all cases these should be combined with the appropriate internal pressures. Figs. B-6 to B-9 are most appropriate for buildings with widths greater than twice their heights and for which the reference height does not exceed 20m. Further details of the work on which these results are based is given in References (12) and (13).

34. Fig. B-6 presents values of $C_pC_g$ applicable to those primary structural actions affected by wind pressures on more than one surface such as in framed buildings. These simplified load distributions were developed to yield as closely as possible the structural actions (horizontal thrust, uplift and frame moments) determined directly from experiment. These results include allowance for the partial loading of gusts referred to in the paragraph 4.1.8.3.(1).

35. Figs. B-7 to B-9 are intended for those actions influenced mainly by wind acting over single surfaces such as design of cladding, secondary structural members, and for other actions not described in B-6.

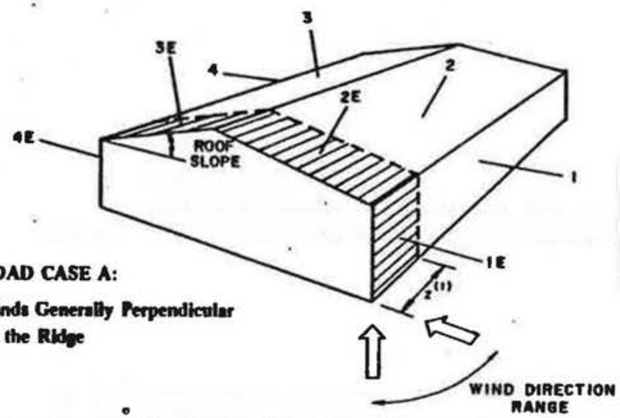
TABLE A-1

| INTERIOR PRESSURES | $C_{pi}$ |
| --- | --- |
| 1. Openings mainly in windward wall. | +0.7 |
| 2. Openings mainly in leeward wall. | -0.5 |
| 3. Openings mainly in walls parallel to wind direction. | -0.7 |
| 4. Openings uniformly distributed in all 4 walls. | -0.3 |

Figure B-11

End wall pressure coefficients, local suction maxima on the roof and interior pressures for use with Figures B-6 to B-9

Reference height for exposure factor: for the calculation of external pressures on end walls use H, the total height of the building. For the calculation of internal pressures, use ½H unless there are dominant openings in the windward wall, in which case use Z, the height to the highest such opening.

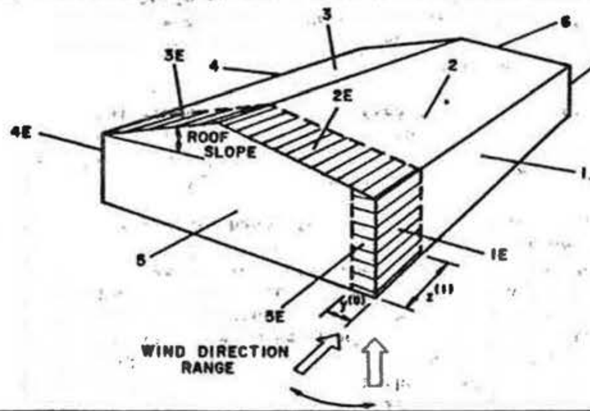



**LOAD CASE A:**

**Winds Generally Perpendicular to the Ridge**

**NOTE: For Fastest Mile Reference Speeds Divide Coefficients by 1.66**

| Roof Slope | Building Surfaces | | | | | | | |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| | 1 | 1E | 2 | 2E | 3 | 3E | 4 | 4E |
| 0 to 5° | 0.75 | 1.15 | −1.3 | −2.0 | −0.7 | −1.0 | −0.55 | −0.8 |
| 20° | 1.0 | 1.5 | −1.3 | −2.0 | −0.9 | −1.3 | −0.8 | −1.2 |
| 30° to 45° | 1.05 | 1.3 | 0.4 | 0.5 | −0.8 | −1.0 | −0.7 | −0.9 |
| 90° | 1.05 | 1.3 | 1.05 | 1.3 | −0.7 | −0.9 | −0.7 | −0.9 |




**LOAD CASE B:**

**Winds Generally Parallel to the Ridge**

| Roof Slope | Building Surfaces | | | | | | | | | | | |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| | 1 | 1E | 2 | 2E | 3 | 3E | 4 | 4E | 5 | 5E | 6 | 6E |
| 0 to 90° | 0 | 0 | −1.3 | −2.0 | −0.7 | −1.0 | 0 | 0 | 0.75 | 1.15 | −0.55 | −0.8 |
| > 20° | −0.85 | −0.9 | −1.3 | −2.0 | −0.7 | −1.0 | −0.85 | −0.9 | 0 | 0 | 0 | 0 |

Figure B-6

COEFFICIENTS ($C_p$.$C_z$) FOR DESIGN OF PRIMARY STRUCTURAL AND WIND BRACING SYSTEMS (1)-(6)

**FIG. A-1a ANTICIPATED FORM OF PRIMARY STRUCTURAL WIND LOAD SPECIFICATIONS FOR LOW-RISE BUILDINGS (See Fig. A-1b for associated notes) – See also Figs. A-2 to A-4.**

**Notes to Figure B-6**

1. The building must be designed for all wind directions. Each corner must be considered in turn as the windward corner shown in the sketches. For all roof slopes, Case A and Case B are required as two separate loading conditions to generate the wind actions, including torsion, to be resisted by the structural system. If the roof slope is 30° or more, a third loading condition is also required as provided for by the second line of Case B.

2. For values of roof slope not shown the coefficient ($C_p C_g$) may be interpolated linearly.

3. Positive coefficients denote forces toward the surface whereas negative coefficients denote forces away from the surface.

4. Interior pressure coefficients $C_{pi}$ are given in Fig B-11.

5. The reference height H for pressures is mid-height of the roof or 6 m whichever is larger. The eave height may be substituted for the mean height, if the slope of the roof is less than 10°.

6. For the design of foundations, but exclusive of anchorages to the frame, only 70% of the effective load is to be considered.

7. End zone width "z" should be the greater of 6 m or 2 y where "y" is the gable wall end zone defined for Case B below. Alternatively, for buildings with frames, the end zone "z" may be the distance between the end and the first interior frame.

8. End zone width "y" is the lesser of 10% of the least horizontal dimension or 40% of height H, except that "y" must be at least 4% of the horizontal dimension, and at least 1 m.

FIG. A–1b NOTES ASSOCIATED WITH Fig. A-1a

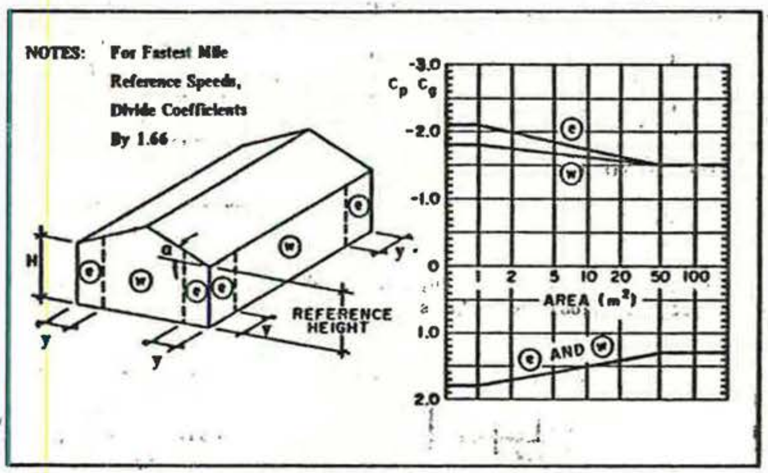



FIGURE B-7

External peak pressure coefficients, $C_pC_g$, on walls for design of cladding, secondary structural members, and surfaces 5 and 6 of Figure B-6.

**Notes to Figure B-7**

1) These coefficients apply for any roof slope, α.
2) The abscissa area in the graph is the design tributary area within the specified zone.
3) y = 10% of least horizontal dimension or 40% of height H, whichever is less. Also y ⩾ 1m, z ⩾ 4% of least horizontal dimension.
4) Interior pressure coefficients $C_{pi}$ are given in Figure B-11.

FIG. A-2 ANTICIPATED FORM OF SECONDARY STRUCTURAL WIND LOAD SPECIFICATIONS FOR WALLS OF LOW-RISE BUILDINGS (See also Figs. A-1 to A-4)

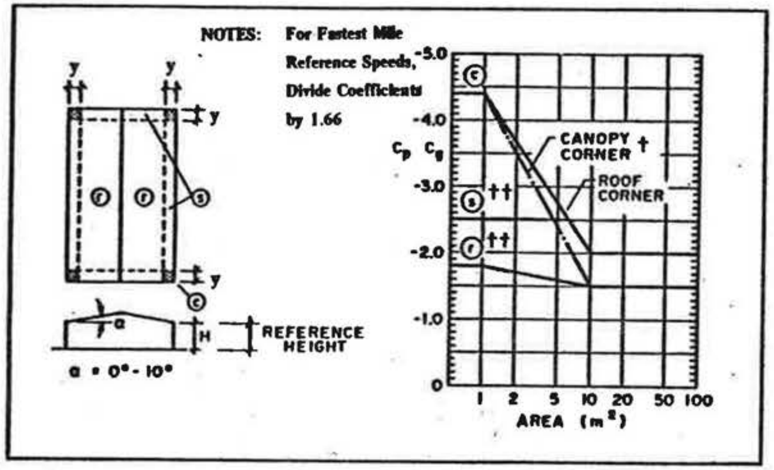



FIGURE B-8

External peak pressure coefficients, $C_pC_g$, on roofs of 10° slope or less for design of cladding and secondary structural members.

Notes to Figure B-8

1) † canopy coefficients include contributions from both upper and lower surfaces
2) †† ⓢ and ⓡ are applicable to both roofs and canopies.
3) The abscissa area in the graph is the design tributary area within the specified zone.
4) y = 10% of least horizontal dimension or 40% of height H, whichever is less. Also y ≥ 1m, z ≥ 4% of least horizontal dimension.
5) Interior pressure coefficients $C_{p_i}$ are given in Figure B-11.

FIG. A–3 ANTICIPATED FORM OF SECONDARY STRUCTURAL WIND LOAD SPECFCATIONS FOR ROOFS OF LOW–RISE BUILDINGS HAVING SLOPES < 10° (See also Figs. A-1 and A-2)

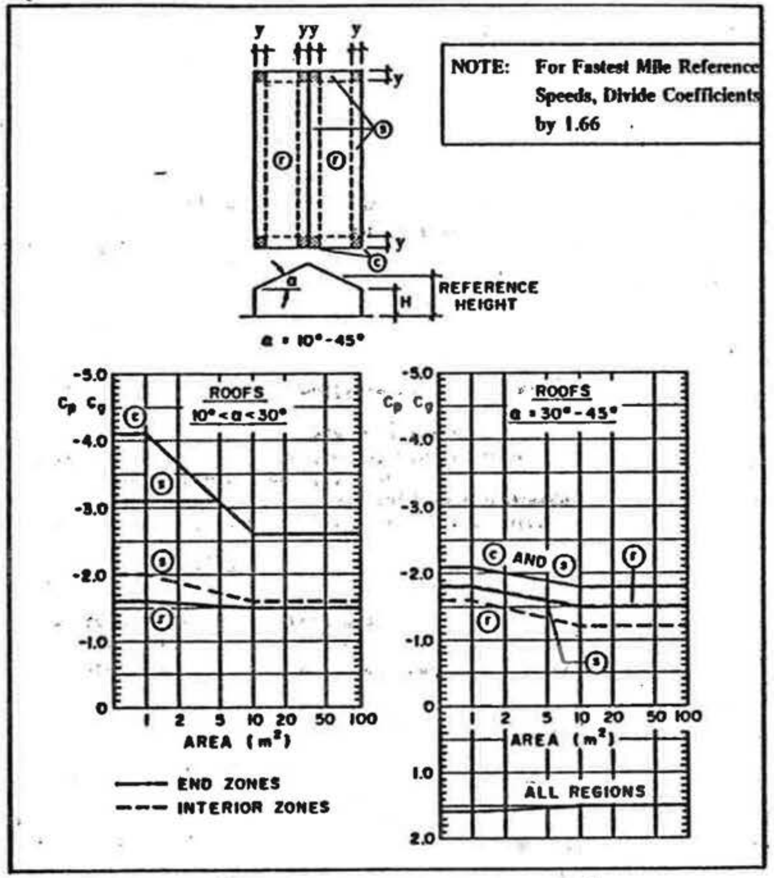



FIGURE B-9

External peak pressure coefficients, $C_pC_g$, on roofs of greater than 10° slope for design of cladding and secondary structural members.

Notes to Figure B-9:

1) The abscissa area in the graph is the design tributary area within the specified zone.
2) y = 10% of least horizontal dimension or 40% of height H, whichever is less. Also y ≥ 1m, y ≥ 4% of least horizontal dimension.
3) Interior pressure coefficients $C_{p_i}$ are given in Fig. B-11.

FIG. A–4 ANTICIPATED FORM OF SECONDARY STRUCTURAL WIND LOAD SPECIFICATIONS FOR ROOFS HAVING SLOPES > 10° (See also Figs. A-1 and A-2)