WIND LOADS ON LOW BUILDINGS

by

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INTRODUCTION AND BACKGROUND

Major changes affecting low buildings have been made in Commentary B, Chapter 4 of the Supplement[1] to the National Building Code of Canada, 1980.[2] These changes are the latest in a series of steps taken to rationalize the calculation of wind loads. They are based on experiments in which a boundary layer tunnel was used to simulate wind interacting with a building and its immediate environment. In the 1970 Commentary on Wind Loads, detailed tables of pressure coefficients for low buildings modelled in smooth-flow tunnels were replaced by one simple table based on early boundary layer wind tunnel simulations. At the same time, two separate methods for dealing with tall, slender structures were introduced. The first method involved calculation (called the "detailed procedure") and the other wind tunnel testing.

Until recently, however, advanced methods of wind load analysis were applied mainly to tall buildings because wind has a considerable influence on their design. A low-rise building normally does not represent a large enough concentration of capital to justify a model test. As a result, research leading to the improvement of design guidelines was slow to develop.

What changed the picture for low-rise buildings was the reaction of the Metal Building Manufacturers Association in the United States to a tendency toward increased design wind loads in proposed code revisions. From their experience with buildings sold and observed to function adequately over periods of ten to twenty years, they concluded that there was little justification for any increase in design loads. This led them to support a comprehensive testing program by the Boundary Layer Wind Tunnel at the University of Western Ontario. Final reports[3,4] of different phases of the program were finished in 1977 and 1978. Technical papers[5,6] presenting the results for critical consideration by designers and engineers appeared in 1978 and 1979.

The next step was to reduce the information to a set of concise design guidelines. A key element in the data reduction was to create a small number of loading patterns that would reproduce all the critical design wind effects, such as frame bending moments, purlin loads and so on. As a result, critical effects found for several different wind directions were represented in a single load pattern that did not necessarily correspond to any one direction. However, the advantage of dealing with only a limited number of load patterns (instead of many) had to be balanced against the difficulty of finding patterns that "looked right" for certain specified wind directions.

In 1979, draft recommendations were submitted to the appropriate Revision Committee of the National Building Code of Canada. Unfortunately, there was insufficient time to resolve all problems of presentation before the final decision to accept the new approach had to be made. The committee members, therefore, asked that explanatory and background material be provided to help designers become familiar with the new procedures. This Note is a response to their request and is intended to be read in conjunction with Commentary B on Wind Loads.[1]

NEW FEATURES

A tremendous amount of data compression and selection has gone into the construction of the four figures (B-6 through B-9 of the Commentary) relating to external peak pressure coefficients for low-rise buildings. The importance of the gusty component of the wind for small buildings requires an elaborate procedure of simultaneous recording of pressures over various tributary areas. These areas represent design concerns ranging from cladding elements, purlins and bay areas to the sliding and overturning forces on the building as a whole. An appreciation of the many factors to be considered and the way in which they were handled can be gained by reading Reference 5.

The new recommendations deal separately with questions of over-all structural effects on primary members and the effects on secondary members and cladding. The dominant gusty component and the steady component of the wind are represented together by "peak coefficients." This is equivalent to combining the gust effect factor and the external pressure coefficient. As an added convenience for the designer, the simplified loading patterns already include allowances for partial loading. These patterns fulfil the requirement of Clause 4.1.8.3.(1) of the National Building Code without the need for further calculation.

Significant load reductions result from area averaging of uncorrelated small gusts. These reductions are already incorporated in the peak coefficients for primary members. For cladding and secondary members, however, the peak pressure coefficients are given in graphs as functions of the tributary areas.

IMPORTANT VARIABLES

Reference Height

Since wind effects are expressed in non-dimensional terms as peak pressure coefficients, they must be multiplied by the basic design pressure for the site, taken at some appropriate height above ground. The mid-height of the roof was found to be the most useful height for summarizing the data for different sizes and shapes of building. It was therefore chosen as the reference height for calculating the exposure factor C_e.

Roof Angle

Roof angle strongly affects the flow around a low-rise building. As well, for steep angles, the slope of the roof adds considerably to the reference height. The eave height may be used if the roof angle is 10 degrees or less. Three roof slopes (1:12, 4:12 and 12:12) form the experimental basis for three categories of roof cladding coefficients (<10, 10-30 and 30-45 degrees). In interpolating for steeper roofs on the primary structure, coefficients labelled "90 degrees" are also provided. No roof, however, could be quite that steep!

End Zones

An important finding of the tests concerns the extra load exerted on the ends of the building compared to the middle portion. Typically, only one end will be loaded at one time; although each end must be designed in its turn for this unbalanced condition. There are two different end zones: one on the roof and long walls and one on the gable-end wall (see Figure B-6 of the Commentary). The end zone dimensions, labelled "y" and "z", are governed by one of the following: reference height, least horizontal dimension of the building, a minimum or maximum dimension, or spacing of interior frames. The footnotes to the appropriate figure (B-6 through B-9) provide the details.

Notice that the length of the building does not influence the width of the end zones. One unexpected result of the testing was the discovery that the length of the building did not appear to be an important variable.

Internal Pressures and Openings

The internal pressure coefficient is needed to complete the analysis of loads acting on various parts of the structure, particularly in the case of cladding. Several experiments were made on models with openings in various walls, as well as varying percentages of background porosity evenly distributed among all walls. The internal pressure appeared to be as dynamic as the external pressure, but its intensity was significantly lower. For wall openings of more than 20% of the wall area, the internal pressure coefficients were independent of the background porosity, which was varied in these experiments from 0 to 3%. The most critical condition occurred with openings in the windward wall, generally causing positive internal pressure coefficients.[7]

Additional research is required to determine the extent and effect of dominant openings and normal leakage areas in full-scale buildings. It was decided, therefore, not to recommend changes at this time in the internal pressure coefficients. The internal pressure coefficients are given for both low and high buildings in Figure B-11 of the Commentary.

The following suggestions, although not spelled out in the Commentary, do not conflict with a conservative interpretation of its recommendations. The uniform distribution of leakage required to qualify for an internal pressure coefficient of -0.3 for all wind directions may be difficult to ensure in the case of low buildings with many operable openings (doors and windows). They are also more susceptible to window breakage from wind-borne missiles during storms which immediately alters the internal pressure, usually for the worse.

As a general rule, therefore, it is prudent to design either for a zero internal pressure coefficient or for -0.3, whichever is worse in any situation. However, if there are large doors unable to withstand the full design load, it may be necessary to use a positive internal pressure multiplied by the gust factor, i.e., 0.7 × 2.0. If windows are likely to be broken by debris, a total internal peak pressure coefficient of ±0.3 may be appropriate.

Surrounding Terrain and Buildings

The characteristics of the surrounding terrain and the influence of nearby structures also play a role in (but do not enter into) the new procedure. Two different terrains were investigated: open country and suburban. A moderate reduction of loads can generally be expected for the latter, but it was decided to introduce only the results for the open countr case. Nearby structures have a tendency to reduce rather than to increase wind effects; although exceptions do occur. Where strong interactions are suspected, special wind tunnel tests may be warranted. An example might be the presence of a much taller building which deflects a strong flow sideways and downward onto a lower building located near the edge of its wake.

SAMPLE LOADING PATTERN FOR PRIMARY STRUCTURAL DESIGN

The loading patterns in Figure B-6 of the Commentary are composites of critical effects from a number of wind directions. Even so, it is convenie to think in terms of two basic patterns: winds perpendicular to the ridge (Case A) and winds parallel to the ridge (Cases B, 1 and 2). In general, Case A supplies the forces in the plane of the frames, whereas the main concern in supplying Case B1 is to provide for sliding and overturning in t longitudinal direction. For roof slopes of 20 degrees and over, however, it was found necessary to devise a third loading pattern to supplement Case Even though its purpose is not to supply longitudinal loads, Case B2 (like is associated with "winds generally parallel to the ridge."

To illustrate, a building with a roof angle of 20 degrees will be discussed for the case of a zero internal pressure coefficient. The net pe pressure coefficients will, therefore, be taken directly from Figure B-6 of the Commentary. The least horizontal dimension is 40 m and the eave height is 10 m, making the reference height (mid-height of the roof) 14 m. The procedure for low-rise buildings normally applies only to a height of 20 m, but the Commentary allows an extension to greater heights provided the height:width ratio is not greater than two. The end zone width on the end wall is 4 m (lesser of 10% of 40 m or 40% of 14 m). The end zone width parallel to the ridge is 8 m.

The peak coefficients and their applicable areas are shown in Figures 1, 2 and 3 of this paper. In an actual design situation, the loads would be calculated by multiplying the coefficients of Figure 1 by the exposure factor for the reference height of 14 m (C_e = 1.1 from Table 4.1.8A of the Code) and by the reference velocity pressure with annual probability of exceedance of 1/30. The resulting pressures over the given areas would then be applied to calculate forces:

$$F = p\ A = q\ C_e\ (C_g\ C_{pe} - C_{pi})\ A \tag{1}$$

The product of the gust effect factor and the external pressure coefficient, $C_g\ C_{pe}$, in Equation (1) is the peak coefficient obtained from Figure B-6 of the Commentary.

CLADDING LOAD PATTERNS

The new external peak coefficients depend on the tributary areas. Several regions are marked out for separate coefficients (four on the roof, two on the walls, both inward and outward coefficients). As well, within each region, the value of the coefficient decreases as it is calculated for larger and larger tributary areas: from 1 m^2 to 50 m^2 for walls and from 1, 5, or 6.4 m^2 to 10 m^2 for roofs. End and edge regions, including the ridge, are all subject to large outward pressures. Negative internal pressures tend to alleviate the net outward pressure. If there are dominant openings through the walls or roof connecting the interior with regions of positive external pressure, however, the opposite is true.

With a zero internal pressure coefficient in the building discussed above, the net outward coefficient for a wall area 1 m^2 or less would be -2.1. This applies to the wall area labelled "e" in any of the end areas of Figure B-7 of the Commentary. In the case of cladding situated on a windward wall, however, it is more conservative to assume an internal peak pressure coefficient of -0.3. The net inward coefficient would therefore be 1.8 - (-0.3) = 2.1.

For a 1 m^2 area located on the roof (corner shaded regions "c" in Figure B-9 of the Commentary), $C_p\ C_g$ = -4.1; for those marked "s' ", $C_p\ C_g$ = -3.1. In the interior regions labelled "r", $C_p\ C_g$ = -1.6.

If a roof area appropriate to purlin design is considered, for example 10 m^2, the external peak coefficients are somewhat smaller in magnitude. The net outward coefficient for regions "c" and "s' " is -2.5. For region "s" it is -1.6 and for the interior region "r", -1.5. The area effect is even more pronounced for the lower roof angles in Figure B-8 of the Commentary.

CONCLUDING REMARKS

The new material on low buildings may seem unduly complicated to someone who has not had prior exposure to the multitude of variables affecting the interaction between building and wind. However, as this Note may suggest, the opposite reaction may be equally justified: how is it possible to do justice to the problem in so compact a form? The references are strongly recommended to anyone who wishes more than just an operational knowledge of the design method in its present form. Further improvements are needed and will be forthcoming as research progresses, particularly in the area of internal pressures.

Two points should be stressed so that users of the new information will be aware of its impact on the design and construction of low buildings. The first point is favourable. Substantial economies should result both because in general loads will be less than formerly required and because there will be a redistribution of wind-resistive elements toward the ends of the buildings where structural strength will do the most good. The second point is that the use of lower design loads implies a greater probability that they will be reached during the life of the building. Thus, the handiwork of both designer and builder will stand a greater chance of being put to the test.

REFERENCES

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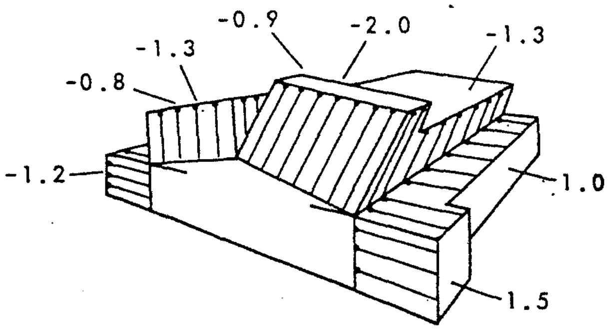

FIGURE 1

PEAK COEFFICIENTS FOR PRIMARY MEMBERS FOR CASE A

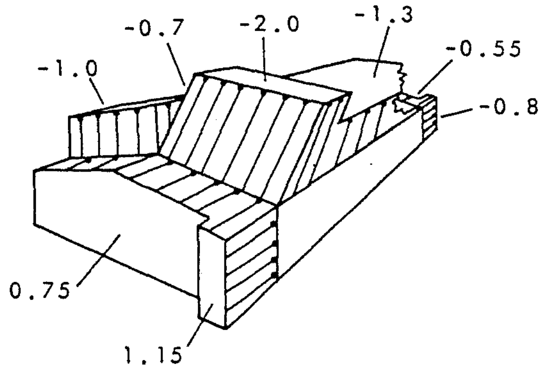

FIGURE 2

PEAK COEFFICIENTS FOR PRIMARY MEMBERS FOR CASE B1 (REQUIRED FOR ALL BUILDINGS TO PROVIDE FOR LONGITUDINAL FORCES)

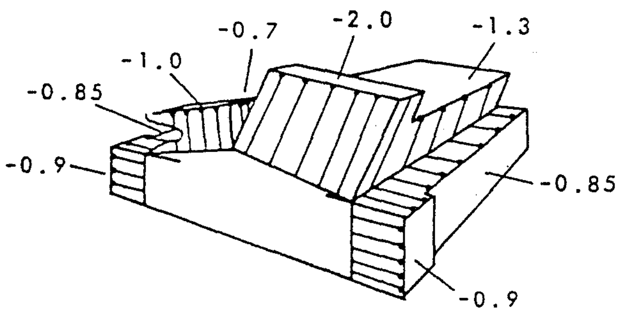

FIGURE 3

PEAK COEFFICIENTS FOR PRIMARY MEMBERS FOR CASE B2 (SUPPLEMENTARY TO CASE A FOR BUILDINGS WITH ROOF SLOPES ≧ 20°)