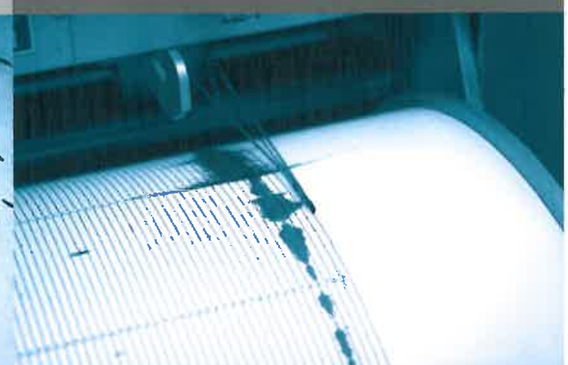
NBC 2005

Snow, Wind and Earthquake Load Design Criteria For Steel Building Systems



Canadian Sheet Steel Building Institute

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NBC 2005 Snow, Wind and Earthquake Load Design Criteria For Steel Building Systems

Preface

One of the objects of the CSSBI and its members is the development of standards that promote safety, performance and good practice. This bulletin is published as a guide for designers, specifiers and users of Steel Building Systems (SBS) and as a reference for building code officials and other authorities.

The material presented herein has been prepared for the general information of the reader and care has been taken to ensure that this Bulletin is a reasonable interpretation of the applicable code requirements. While the material is believed to be technically correct and in accordance with recognized practice at the time of publication, it does not obviate the need to determine its suitability for a given situation. Neither the CANADIAN SHEET STEEL BUILDING INSTITUTE nor its Members warrant or assume liability for the suitability of this bulletin for any general or particular application.

Much of the content of this Bulletin is taken directly from the *User's Guide - NBC 2005 Structural Commentaries (Part 4 of Division B)*, and has been expanded upon where additional explanation was considered helpful for understanding the application of the NBC 2005 load provisions. The earthquake loads for which a Steel Building System should be designed are intended to satisfy the provisions of both the *National Building Code of Canada 2005* and *CAN/CSA-S16-04, Limit States Design of Steel Structures*.

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CSSBI
Canadian Sheet Steel Building Institute

Introduction

The objective of this bulletin is to present the NBC 2005 loading requirements in a format that is easy to understand. Illustrative examples are included to assist. The focus is on Steel Building Systems, however, the loading criteria are applicable to most low rise building construction.

The *National Building Code of Canada* is a model document used by the provinces and territories of Canada in the preparation of their own building codes, which are the governing legislation for building construction. For specific designs, the loading criteria presented in this Bulletin should be checked against the requirements of the applicable building code.

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Importance Categories

For the purposes of determining the wind, snow and earthquake loading requirements in accordance with NBC 2005, the building must be assigned an Importance Category based on the intended use and occupancy. These categories are presented in the Table I1 (reproduced from NBC Table 4.1.2.1).

Table I1: Importance Categories for Buildings

Use and Occupancy	Importance Category
Buildings that represent a low direct or indirect hazard to human life in the event of failure, including: • low human-occupancy buildings, where it can be shown that collapse is not likely to cause injury or other serious consequences • minor storage buildings	Low(1)
All buildings except those listed in Importance Categories Low, High and Post-disaster	Normal
Buildings that are likely to be used as post-disaster shelters, including buildings whose primary use is: • as an elementary, middle or secondary school • as a community centre Manufacturing and storage facilities containing toxic, explosive or other hazardous substances in sufficient quantities to be dangerous to the public if released	High
Post-disaster buildings and buildings that are essential to the provision of services in the event of a disaster, and include: • hospitals, emergency treatment facilities and blood banks • telephone exchanges • power generating stations and electrical substations • control centres for air, land and marine transportation • public water treatment facilities and buildings having critical national defense functions • Buildings of the following type, unless exempted from this designation by the authority having jurisdiction: - emergency response facilities - fire, rescue and police stations, and housing for vehicles, aircraft or boats used for such purposes - communications facilities, including radio and television studios	Post-disaster

(1) Low human-occupancy farm buildings are defined in the National Farm Building Code of Canada 1995 as having an occupancy load of 1 person or less per 40 m^2 of floor area. Minor storage buildings include only those storage buildings that represent a low direct or indirect hazard to human life in the event of structural failure, either because people are unlikely to be affected by structural failure, or because structural failure causing damage to materials or equipment does not present a direct threat to human life.

A. Snow Load Design Criteria

A1 Minimum Design Snow Load

Manufacturers of Steel Building Systems who are members of the CANADIAN SHEET STEEL BUILDING INSTITUTE use the following criteria to establish minimum design snow loads, unless otherwise required by the governing building regulations or the design authority:

a) for all occupancy classifications and building sizes covered by Part 4 or Part 9 of NBC 2005, the requirements of Division B, Subsection 4.1.6, "Loads Due to Snow and Rain", are followed;

b) the ground snow loads and rain loads to be used in conjunction with (a) above are the values given in NBC 2005, Division B, Appendix C; and,

c) the snow load shape factors to be used in conjunction with (a) above are the values given in Commentary G of the *User's Guide - NBC 2005 Structural Commentaries (Part 4 of Division B)*.

A2 Determination of Snow Loads

A2.1 Specified Snow Loads:

Snow loads on roofs vary according to geographic location (climate), site exposure, shape and type of roof, and also from one winter to another. To account for these varying conditions, the specified snow load, S, on a roof or other surface is expressed as follows:

$$S = I_s [S_s (C_b C_w C_s C_a) + S_r]$$

where,

I_s = importance factor for snow load as provided in Table A1 (NBC 2005, Table 4.1.6.2)

S_s = 1-in-50 year ground snow load by geographic location as specified by the authority having jurisdiction, or in the absence of such data, as listed in NBC 2005, Division B, Appendix C, for selected locations in Canada, (kPa).

C_b = basic roof snow load factor (see A2.2)

C_w = wind exposure factor (see A2.3)

C_s = slope factor (see A2.4)

C_a = shape factor (see A2.5)

S_r = 1-in-50 year associated rain load by geographic location as specified by the authority having jurisdiction, or in the absence of such data, as listed in NBC 2005, Division B, Appendix C, for selected locations in Canada, but not greater than $S_s(C_bC_wC_sC_a)$ (kPa).

Table A1: Importance Factor for Snow Load (I_s)
(See Table I1 for the Importance Category definitions)

Importance Category	Importance Factor, I_s	
	ULS	SLS
Low	0.8	0.9
Normal	1.0	0.9
High	1.15	0.9
Post-disaster	1.25	0.9

Note: ULS = Ultimate Limit State, SLS = Serviceability Limit State

A2.2 Basic Roof Snow Load Factor, C_b

The basic roof snow load factor, C_b, shall be 0.8, except that for large roofs where the wind is less effective in removing the snow, it shall be:
a) $1.0 - (30/l_c)^2$ for roofs with $C_w = 1.0$ and l_c greater than or equal to 70 m; or
b) $1.3 - (140/l_c)^2$ for roofs with $C_w = 0.75$ or 0.5 and l_c greater than or equal to 200 m.

where

l_c = characteristic length of the upper or lower roof, defined as $2w - w^2/l$ (metres)
w = smaller plan dimension of the roof (metres)
l = larger plan dimension of the roof (metres)

A2.3 Wind Exposure Factor, C_w

The wind exposure factor, C_w, is taken equal to 1.0 except under certain circumstances. Observations in many areas of Canada have shown that where a roof or a part of it is fully exposed to wind, some of the snow is blown off or prevented from accumulating and the average snow load is reduced.

For buildings in the Low and Normal Importance Categories, with roofs fully exposed to the wind, though not for very large roofs where it may be inappropriate, the wind exposure factor, C_w, may be reduced to 0.75 rather than 1.0 (or 0.5 rather than 1.0 for exposed sites north of the tree line). This substitution applies under the following conditions:

a) the building is located in an open location containing only scattered buildings, trees or other obstructions, open water and shorelines thereof, so that the roof is exposed to the wind on all sides and is expected to remain so during its lifetime; not shielded within a distance from the building equal to 10 times the height of the obstruction above the roof level;
b) the roof does not have any significant obstructions, such as parapet walls, within a distance of at least 10 times the difference between the height of the obstruction, and $C_bC_wS_s/\gamma$ (in metres); (See A2.7 for γ)
c) the load case under consideration does not involve accumulation of snow due to drifting from adjacent surfaces (e.g. the other side of a gable roof).

Sample Calculation:

C_b = 0.8, $C_w = 0.75$, $S_s = 2.1$ kPa, $\gamma = 3.0$ kN/m^3
Height of obstruction above roof level = 2m
Minimum distance between obstruction and roof area
where the $C_w = 0.75$ is applied
$= 10[(2.0) - (0.8)(0.75)(2.1)/3.0)] = 15.8$ m

A2.4 Slope Factor, C_s

Under most conditions, less snow accumulates on steep than on flat and moderately sloped roofs, because of sliding, better drainage, and saltation. The slope factor, C_s, accounts for these effects by reducing the snow load linearly from full snow load at a 30° slope to zero at a 70° slope. To be able to use the full slope reduction, the snow should be able to slide completely off the roof surface under consideration.

$C_s = 1.0$ for $\alpha \leq 30°$
$C_s = (70° - \alpha)/40°$ for $30° < \alpha \leq 70°$
$C_s = 0$ for $\alpha > 70°$

A lesser value of C_s is permitted for unobstructed, smooth, slippery roofs such as metal or glass where snow and ice can slide completely off the roof. In this case, the load may be reduced linearly from full load at 15° to zero at 60°.

C_s = 1.0 for $\alpha \leq 15°$
C_s = $(60° - \alpha)/45°$ for $15° < \alpha \leq 60°$
C_s = 0 for $\alpha > 60°$

The slope factor shall be 1.0 when used in conjunction with shape factors for increased snow load in valleys and from accumulations of snow sliding off an upper roof.

A2.5 Shape Factor, C_a

Due to the effects of wind encountering obstructions, uneven roof surfaces, and snow sliding off one surface onto another, there are many areas of a roof that can accumulate significantly higher snow loads. Figures A1 through A4 illustrate the basic accumulation of snow on various roof types, while Figure A5 through A8 depict the localized accumulations of snow in roof valleys, adjacent to projections, and resulting from snow sliding.

A2.6 Full and Partial Loading

Article 4.1.6.3 of the NBC 2005 stipulates that in addition to the distribution of uniform snow loads, the building shall be designed for a 50% partial load on part of the area in such a way as to produce the greatest effect on the member considered. This requirement is reflected in load Case 2 in Figures A1, A2, A3, and A4.

A2.7 Unit Weight of Snow

In the calculation of loads due to snow on roofs, a measurement or good estimate of the unit weight of the snow is necessary. The unit weight of snow on roofs, γ, obtained from measurements at a number of stations across Canada varied from about 1.0 to 4.5 kN/m^3. An average value for use in design in lieu of better local data is γ = 3.0 kN/m^3. In some places where the maximum roof load is reached only after contributions from many snowstorms, a unit weight as high as 4.0 kN/m^3 may be appropriate.

A3 Notes to the Figures

The 2005 NBC requires, as did previous editions, that the snow load distributions (Figures A1 through A4) be considered plus the effects of any special conditions of snow load accumulations (Figures A5 through A8) resulting from shielding or sliding snow. The likelihood that non-uniformity of snow load will be the prevailing mode increases as the tributary roof area under consideration is increased. Such non-uniformity may create an imbalance effect that is more critical to the supporting structure that a heavier mass of uniform snow.

A minimum of two snow load distributions are considered in the design of structural members supporting larger roof areas (e.g. rigid frames, continuous beams, continuous purlins). For roof cladding, a uniform snow load is generally assumed for design purposes. Additional snow load accumulations are superimposed on the appropriate snow load distributions, where applicable. No special provision is made for the effects of full or partial snow removal, since removal is not necessary where design loads and safety margins are adequate, and may, in fact, cause damage to the roof surface.

A4 Illustrative Example #1

A4.1 Introduction

The building under consideration is illustrated in Figure A9. There is a main building that has a small mechanical room in the centre of the roof, as well as a canopy over the front entrance. In addition to the main building, there is an adjacent building that has a lower roof elevation. The building location is Kitchener, ON where there is a ground snow load of $S_s = 2.0$ kPa and a rain load of $S_r = 0.4$ kPa. For this type of building the Importance Category is "Normal" and so the Importance Factor for Snow is $I_s = 1.0$. A snow density of $\gamma = 3.0$ kN/m^3 is assumed for this location.

A4.2 Main Roof

The main roof (ignoring the area around the mechanical room which is addressed in section A4.4) may have an accumulation around the parapet that needs to be checked. Since this is the highest roof in the area, the wind exposure factor is $C_w = 0.75$. The slope factor $C_s = 1.0$ since this is a flat roof.

Step 1: Determine the basic roof snow load factor, C_b

The upper level roof dimensions are: $l = 60$ m, w = 40 m

$l_c = 2w - w^2/l = (2)(40) - (40)^2/(60) = 53.3$

Since $l_c = 53.3 < 200$, with $C_w = 0.75$, there is no increase in the basic roof snow load factor: $\therefore$ $\mathbf{C_b = 0.8}$

Step 2: Determine whether drifting occurs at the parapet

$h_p = 0.5$ m

$0.8S_s/\gamma = (0.8)(2.0)/(3.0) = 0.53 \text{ m} > h_p$,

Since the depth of the snow over the roof area (ignoring any reductions due to wind) is greater than the parapet height, drifting is not a concern: $\therefore$ $\mathbf{C_a = 1.0}$

Step 3: Determine the specified snow load, S

$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(2.0)(0.8)(0.75)(1.0)(1.0) + (0.4)]$

S = 1.6 kPa

Note that the unbalanced loads illustrated in Figures A2 or A3 may also apply.

A4.3 Canopy Loading (use Figure A6)

$C_b = 0.8$, $C_w = 1.0$, $C_s = 1.0$, l_c (upper roof) = 53.3 (from section A4.2)

Step 1: Determine F

$F = 0.35[\gamma l_c/S_s - 6(\gamma h_p/S_s)^2]^{0.5} + C_b$

$= (0.35)[(3.0)(53.3)/(2.0) - 6((3.0)(0.5)/(2.0))^2]^{0.5} + 0.8$

$= 3.86 > 2.0$, $\therefore$ $\mathbf{F = 3.86}$

Step 2: Determine the maximum shape factor, $C_a(0)$

$C_a(0)$ is the minimum of the following:

$C_a(0) = F/C_b = 3.86/0.8 = 4.83$, or

$C_a(0) = \gamma h/C_b S_s = (3.0)(5.0)/(0.8)(2.0) = 9.38$, $\therefore$ $\mathbf{C_a(0) = 4.83}$

Step 3: Determine the maximum snow load, S_{max}

$S_{max} = I_s [S_s C_b C_w C_s C_a(0) + S_r] = (1.0)[(2.0)(0.8)(1.0)(1.0)(4.83) + (0.4)]$

$\mathbf{S_{max} = 8.13}$ **kPa**

Step 4: Determine the length of the snowdrift, x_d

x_d is the minimum of the following:

$x_d = 5(h - C_b S_s/\gamma) = 5[(5 - (0.8)(2.0)/(3.0)] = 22.3$ m, or

$x_d = 5(S_s/\gamma)(F - C_b) = (5)[(2.0)/(3.0)](3.86 - 0.8) = 10.2$ m

$\mathbf{x_d = 10.2}$ **m**

Step 5: Determine the snow load at the canopy edge, S_{edge}

$S_{edge} = S_{xd} + (S_{max} - S_{xd})(x_d - L)/x_d$

where,

S_{xd} = (1.0)[(2.0)(0.8)(1.0)(1.0)(1.0) + (0.4)] = 2.0 kPa
L = canopy width = 2.5 m

S_{edge} = 2 + (8.13 – 2)(10.2 – 2.5)/(10.2)
S_{edge} = 6.63 kPa

A4.4 Roof Area Adjacent to Mechanical Room

The mechanical room will create snow accumulations around it that need to be considered. The height h = 2 m and the width b = 3 m.

Step 1: Determine whether snow accumulation needs to be considered

$3S_s/\gamma$ = (3)(2.0)/(3.0) = 2.0 m < b = 3.0 m, therefore, snow accumulation does need to be considered.

Step 2: Determine the maximum shape factor, $C_a(0)$

$C_a(0) = F/C_b$ = 3.86/0.8 = 4.83, or
$C_a(0) = 0.67\gamma h/C_bS_s$ = (0.67)(3.0)(2.0)/(0.8)(2.0) = 2.51, but
Minimum $C_a(0) = 0.8/C_b$ = (0.8)/(0.8) = 1.0
Maximum $C_a(0) = 2/C_b$ = (2)/(0.8) = 2.5
$\therefore C_a(0) = 2.5$

Step 3: Determine the maximum snow load, S_{max}

$S_{max} = I_s [S_s C_b C_w C_s C_a(0) + S_r]$ = (1.0)[(2.0)(0.8)(1.0)(1.0)(2.5) + (0.4)]
S_{max} = 4.4 kPa

Step 4: Determine the length of the snowdrift, x_d

x_d = 2h = 2(2) = 4 m, but 3m ≤ x_d ≤ 9 m
x_d = 4.0 m

Step 5: Determine the length of the affected zone

$h' = h - C_bC_wS_s/\gamma$ = 2 – (0.8)(0.75)(2.0)/(3.0) = 1.6
10h' = 16 m
At 16 m away from the mechanical room, C_w can be reduced to 0.75.

Step 6: Determine the specified snow load, S

Within a distance of 16 m from the mechanical room:
$S = I_s [S_s C_b C_w C_s C_a + S_r]$ = (1.0)[(2.0)(0.8)(1.0)(1.0)(1.0) + (0.4)]
S = 2.0 kPa
Beyond 16 m from the mechanical room:
$S = I_s [S_s C_b C_w C_s C_a + S_r]$ = (1.0)[(2.0)(0.8)(0.75)(1.0)(1.0) + (0.4)]
S = 1.6 kPa

A4.5 Lower Roof

The adjacent roof is within 5 m of the higher roof and 3.5 m lower, therefore, the influence of additional snow accumulation must be considered. This is a gable roof, but since the roof slope is only 5°, there is no sliding and C_s = 1.0. In addition, the provisions of unbalanced loading must be considered as described in Figure A2.

C_w = 0.75 (where allowed)
C_s = 1.0
l_c (upper roof) = 53.3 (from section A4.2)

Step 1: Determine the basic roof snow load factor, C_b

The lower roof dimensions are: l = 36 m, w = 22 m

$l_c = 2w - w^2/l = (2)(22) - (22)^2/(36) = 30.6$

Since $l_c = 30.6 < 70$, with $C_w = 1.0$, there is no increase in the basic roof snow load factor: $\therefore$ **$C_b = 0.8$**

Step 2: Determine F

$F = 0.35[\gamma l_c/S_s - 6(\gamma h_p/S_s)^2]^{0.5} + C_b$

$= (0.35)[(3.0)(53.3)/(2.0)-6((3.0)(0.5)/(2.0))^2]^{0.5} + 0.8$

$= 3.86 > 2.0$, $\therefore$ **$F = 3.86$**

Step 3: Determine the maximum shape factor, $C_a(0)$

$C_a(0)$ is the minimum of the following:

$C_a(0) = F/C_b = 3.86/0.8 = 4.83$, or

$C_a(0) = (\gamma h/C_b S_s) = (3.0)(3.5)/(0.8)(2.0) = 6.56$ $\therefore$ **$C_a(0) = 4.83$**

Step 4: Determine the maximum snow load, S_{max}

$S_{max} = I_s [S_s C_b C_w C_s C_a(0) + S_r] = (1.0)[(2.0)(0.8)(1.0)(1.0)(4.83) + (0.4)]$

$S_{max} = 8.13$ kPa

Step 5: Determine the length of the snowdrift, x_d

x_d is the minimum of the following:

$x_d = 5(h-C_bS_s/\gamma) = 5[(3.5 - (0.8)(2.0)/(3.0)] = 14.8$ m, or

$x_d = 5(S_s/\gamma)(F - C_b) = (5)[(2.0)/(3.0)](3.86 - 0.8) = 10.2$ m

$x_d = 10.2$ m

Step 6: Determine the snow load at the roof eave, S_{eave}

$S_{eave} = S_{xd} + (S_{max} - S_{xd})(x_d - L)/x_d$

where,

$S_{xd} = (1.0)[(2.0)(0.8)(1.0)(1.0)(1.0) + (0.4)] = 2.0$ kPa

$L = gap = 3.0$ m

$S_{eave} = 2 + (8.13 - 2)(10.2 - 3)/(10.2)$

$S_{eave} = 6.33$ kPa

Step 7: Determine the length of the affected zone

$h' = h - C_bC_wS_s/\gamma = 3.5 - (0.8)(0.75)(2.0)/(3.0) = 3.1$

$10h' = 31$ m

Since the building is only 22 m wide, there will be no reduction in the snow load due to the wind exposure factor ($C_w = 1.0$).

Step 8: Determine the specified snow load, S

$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(2.0)(0.8)(1.0)(1.0)(1.0) + (0.4)]$

$S = 2.0$ kPa

A4.6 Summary of Specified Roof Snow Loads

Figure A10 summarizes the roof snow load distributions for Illustrative Example #1.

A5 Illustrative Example #2

A5.1 Introduction

Consider an SBS in Chilliwack, BC where there is a ground snow load of $S_s = 2.2$ kPa and a rain load of $S_r = 0.3$ kPa. Importance Category is "Normal" and so the Importance Factor for Snow is $I_s = 1.0$. A snow density of $\gamma = 3.0$ kN/m³ is assumed for this location. The roof is metal and has sloping multi-level areas as illustrated in Figure A11.

A5.2 Upper Roof

$C_w = 0.75$

Step 1: Determine the basic roof snow load factor, C_b

The upper level roof dimensions are: $l = 30$ m, w = 32 m

$l_c = 2w - w^2/l = (2)(30) - (30)^2/(32) = 31.9$

Since $l_c = 31.9 < 200$, with $C_w = 0.75$, there is no increase in the basic roof snow load factor. $\therefore$ **$C_b = 0.8$**

Step 2: Determine the sliding factor, C_s

Since the upper roof is metal, and sloping more than 15°, a reduction in the snow load is allowed.

$C_s = (60 - \alpha)/45 = (60 - 20)/45$

$C_s = 0.89$

Step 3: Design Case 1, Figure A1(B)

$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(2.2)(0.8)(0.75)(0.89)(1.0) + (0.3)]$

S = 1.47 kPa

Step 4: Design Case 3, Figure A1(B)

$C_a(\alpha = 20°) = 1.25$

$C_w = 1.0$

$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(2.2)(0.8)(1.0)(0.89)(1.25) + (0.3)]$

S = 2.26 kPa

A5.3 Lower Roof

$C_b = 0.8$

$C_w = 1.0$

$C_s = 1.0$ since the roof slope is less than 15°

l_c (upper roof) = 31.9 (from section A5.2)

Step 1: Determine F

$F = 0.35[\gamma l_c/S_s - 6(\gamma h_p/S_s)^2]^{0.5} + C_b$, where $h_p = 0$

$= (0.35)[(3.0)(31.9)/(2.2)]^{0.5} + 0.8$

$= 3.11 > 2.0$, $\therefore$ **F = 3.11**

Step 2: Determine the maximum shape factor, $C_a(0)$

$C_a(0)$ is the minimum of the following:

$C_a(0) = F/C_b = 3.11/0.8 = 3.89$, or

$C_a(0) = \gamma h/C_b S_s = (3.0)(4.0)/(0.8)(2.2) = 6.82$, $\therefore$ **$C_a(0) = 3.89$**

Step 3: Determine the maximum snow load, S_{max}

$S_{max} = I_s [S_s C_b C_w C_s C_a(0) + S_r] = (1.0)[(2.2)(0.8)(1.0)(1.0)(3.89) + (0.3)]$

$S_{max} = 7.15$ kPa

Step 4: Determine the length of the snowdrift, x_d

x_d is the minimum of the following:

$x_d = 5(h - C_b S_s/\gamma) = 5[(4 - (0.8)(2.2)/(3.0)] = 17.1$ m, or

$x_d = 5(S_s/\gamma)(F - C_b) = (5)[(2.2)/(3.0)](3.11 - 0.8) = 8.47$ m

$x_d = 8.47$ m

Step 5: Determine the length of the affected zone

$h' = h - C_b C_w S_s/\gamma = 4.0 - (0.8)(0.75)(2.2)/(3.0) = 3.56$

$10h' = 36$ m

Since the lower roof is only 16 m wide, there will be no reduction in the snow load due to the wind exposure factor ($C_w = 1.0$).

Step 6: Determine the specified snow load, S

$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(2.2)(0.8)(1.0)(1.0)(1.0) + (0.3)]$

S = 2.06 kPa

Step 7: Sliding snow accumulation

Since the upper roof slopes towards the lower roof at an angle greater than 15°, the possibility of snow accumulation on the lower roof due to sliding must be considered.

S(upper roof) = 1.47 kPa

Snow sliding onto lower roof = $1/2$(1.47 kPa)(16 m) = 11.8 kN/m

The snow accumulates on the lower roof in a triangular shape over a width equal to x_d.

$S_{max} = 7.15 + 2(11.8)/(8.47)$

S_{max} = 9.94 kPa

A5.4 Summary of Roof Snow Loads

Figure A11 summarizes the roof snow load distributions for Illustrative Example #2.

A6 Illustrative Example #3

A6.1 Introduction

Consider a SBS in Moncton, NB where there is a ground snow load of S_s = 3.0 kPa and a rain load of S_r = 0.6 kPa. The building length is 50 m and has a saw-tooth configuration as illustrated in Figure A12. For this type of building the Importance Category is "Normal" and so the Importance Factor for Snow is I_s = 1.0. A snow density of γ = 3.0 kN/m^3 is assumed for this location.

A6.2 Load Case I (use Figure A5)

$C_w = 0.75$

C_s = 1.0 since the roof slope is not greater than 15°

Step 1: Determine the basic roof snow load factor, C_b

The upper level roof dimensions are: l = 50 m, w = 20 m

$l_c = 2w - w^2/l = (2)(20) - (20)^2/(50) = 32$

Since $l_c = 32 < 200$, with $C_w = 0.75$, there is no increase in the basic roof snow load factor.

$\therefore C_b = 0.8$

$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(3.0)(0.8)(0.75)(1.0)(1.0) + (0.6)]$

S = 2.40 kPa

A6.3 Load Case 2 (use Figure A5)

$C_w = 1.0$

For $0 < x \leq 20/4 = 5$ m, $C_a = 1/C_b = 1/0.8 = 1.25$

$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(3.0)(0.8)(1.0)(1.0)(1.25) + (0.6)]$

S = 3.60 kPa

For $5 < x \leq 20/2 = 10$ m, $C_a = 0.5/C_b = 0.5/0.8 = 0.625$
$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(3.0)(0.8)(1.0)(1.0)(0.625) + (0.6)]$
S = 2.10 kPa

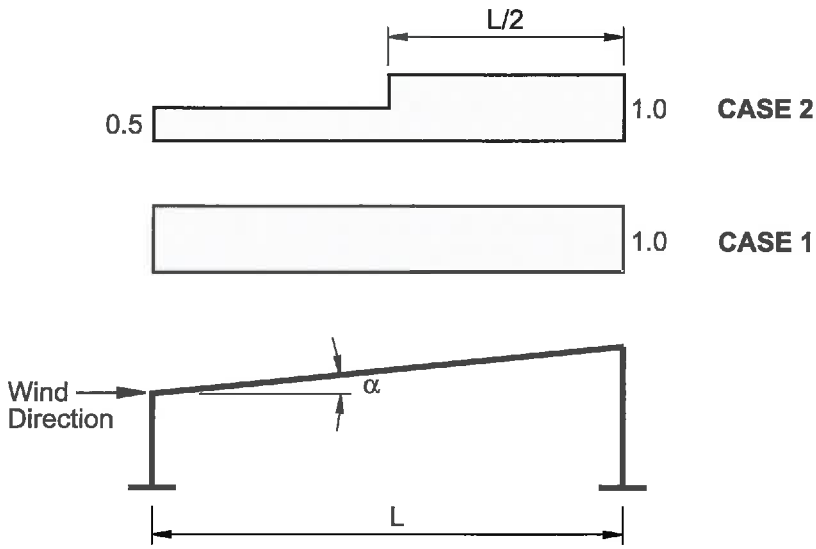
A6.4 Load Case 3 (use Figure A5)
$C_w = 1.0$
For $0 < x \leq 20/8 = 2.5$ m, $C_a = 1.5/C_b = 1.5/0.8 = 1.875$
$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(3.0)(0.8)(1.0)(1.0)(1.875) + (0.6)]$
S = 5.10 kPa

For $2.5 < x \leq 20/2 = 10$ m, $C_a = 0.5/C_b = 0.5/0.8 = 0.625$
$S = I_s [S_s C_b C_w C_s C_a + S_r] = (1.0)[(3.0)(0.8)(1.0)(1.0)(0.625) + (0.6)]$
S = 2.10 kPa

A6.5 Other Load Cases
Multispan structures must also be designed for the load patterns described in Figures A1(B) and A4. For the load cases shown in Figure A12, those portions of the span may be taken as uniformly loaded.

Figure A1(A): Shape Factor, C_a, for Single Slope Roofs

Flat or Single Slope



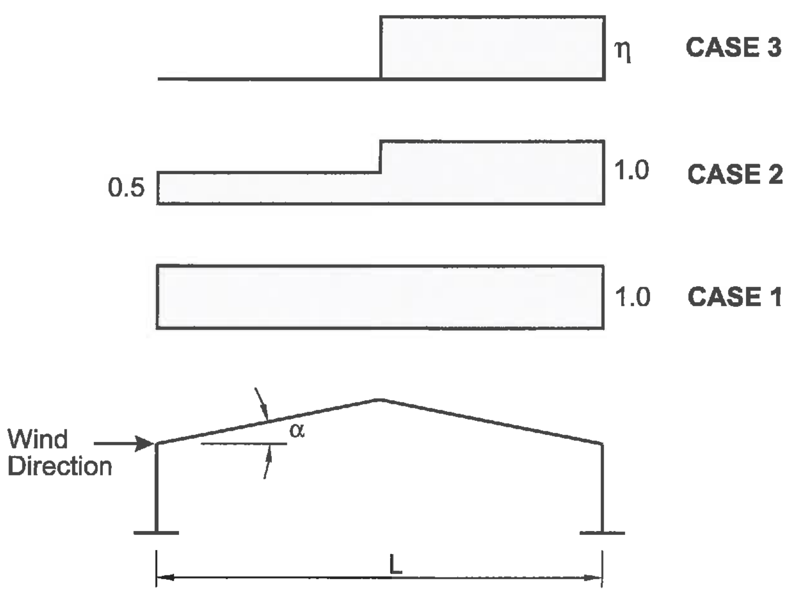
Where $\alpha \leq 15^{\circ}$ design for CASE 1 and 2

Where $\alpha > 15^{\circ}$ design for CASE 1 only

CASE 2 includes "Opposite Hand" distribution mode

CASE 2 may be critical for some truss-type roof members

Figure A1(B): Shape Factor, C_a, for Single Gable Roofs



Where $\alpha \leq 15^{\circ}$, design for CASE 1 and 2

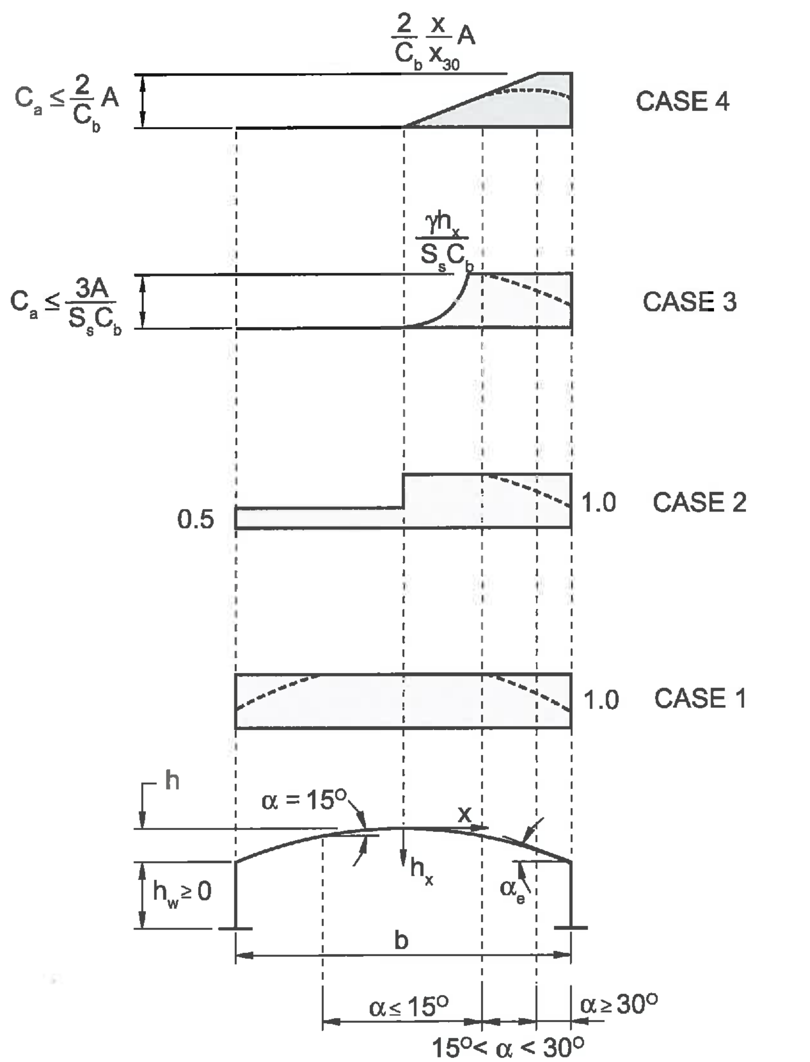
Where $\alpha > 15^{\circ}$, design for CASE 1 and 3

CASE 2 and 3 include "Opposite Hand" distribution mode

$\eta = 0.25 + \alpha / 20 \leq 1.25$

For CASE 3, $C_w = 1.0$

Figure A1(C): Shape Factor, C_a, for Simple Arch Slippery Roofs



Design for CASE 1 and 2 for all building shapes. C_w = 1.0, 0.75 or 0.5, and $C_s = f(\alpha)$

Where h/b > 0.05 design for CASE 3 or 4 whichever produces the lower load per unit length of building perpendicular to span b. C_w = 1.0 and $C_s = f(\alpha)$.

CASE 2, 3 and 4 include "Opposite Hand" distribution mode

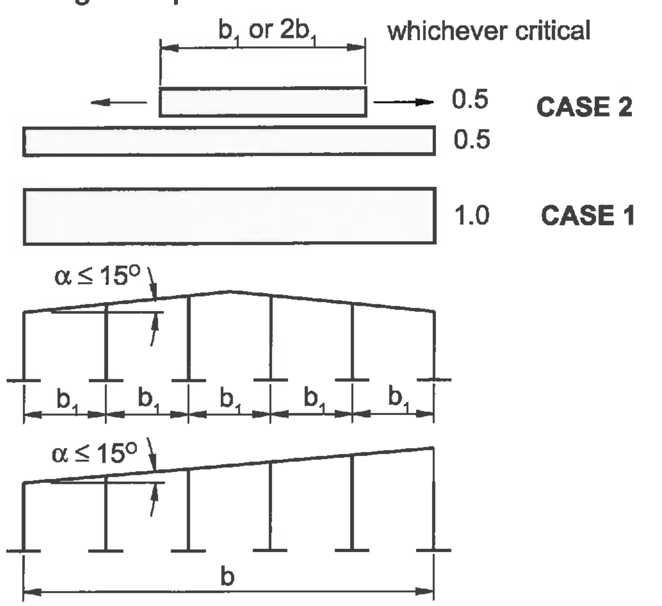
These distributions are for unobstructed slippery roofs. For other roofs, refer to Figure G-2 in the 2005 NBC Structural Commentaries (Part 4)

The dashed lines in the distributions show the effects of C_a. These distributions can be used if a more detailed analysis is required

$$A = \frac{h/b}{0.12} \le 1.0$$

Figure A2: Shape Factor, C_a, for Continuous Beams

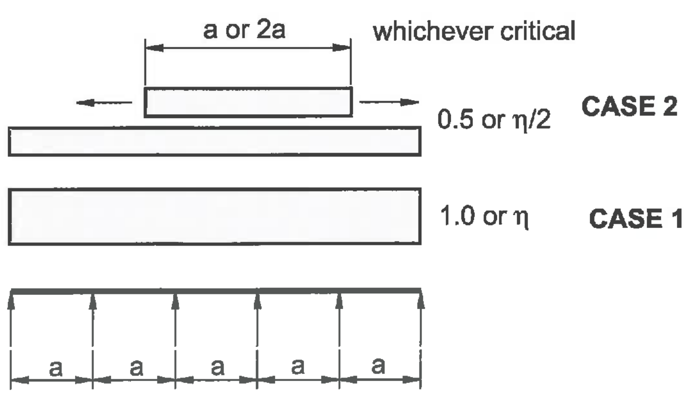
Gable or Single Slope



Design for CASE 1 and 2

Unbalanced load CASE 2 starts and ends at a column

Figure A3: Shape Factor, C_a, for Continuous Purlins



Design for CASE 1 and 2

For gable roof slopes ≤ 15° and arch roofs with h/b ≤ 0.05 use 1.0

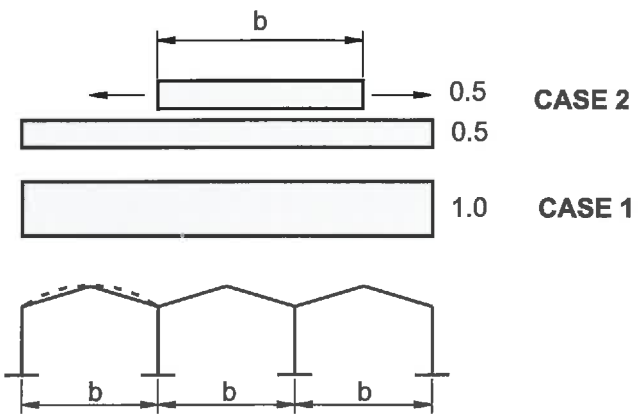
For gable roof slopes > 15° use η as noted in Fig.A1(B)

For arch roofs with h/b > 0.05 see Fig.A1(C)

Unbalanced load CASE 2 starts and ends at a support

Figure A4: Shape Factor, C_a, for Multiple Spans

Gable or Arch



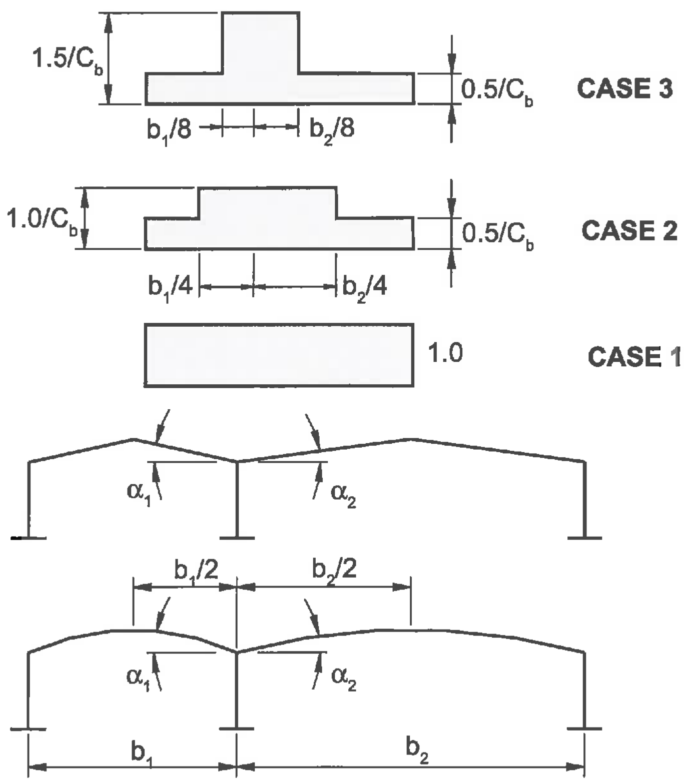
Design for CASE 1 and 2

Also design each span for loading as per Fig.A1(A), (B) or (C)

Also design for snow accumulation in valleys as per Fig.A5

Unbalanced load CASE 2 starts and ends at a column

Figure A5: Shape Factor, C_a, for Valleys



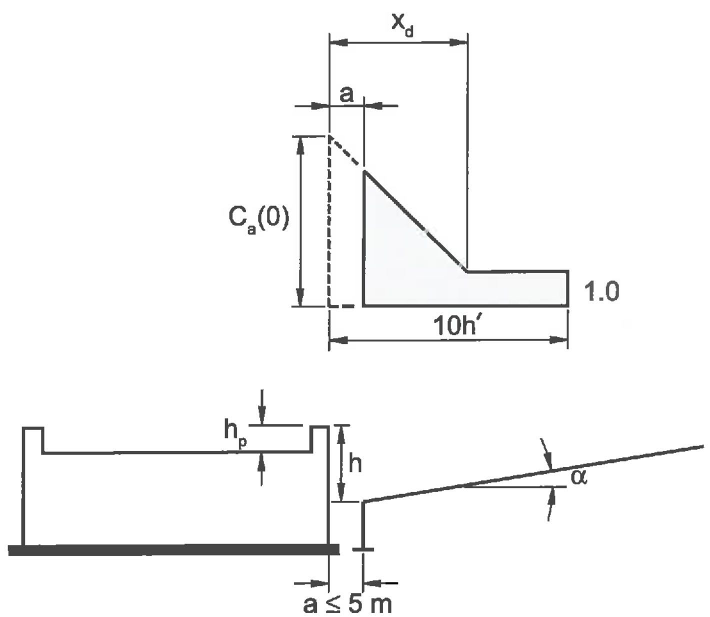
Where both α_1 and $\alpha_2 \leq 10°$, design for CASE 1 only. C_w = 1.0. 0.75 or 0.5 and $C_s = f(\alpha)$

Where α_1 and/or $\alpha_2 > 10°$, design for CASE 1. 2 and 3

For CASE 2 and 3, C_w = 1.0 and C_s = 1.0

NOTE:
Portions of spans where loading is not indicated may be taken as uniformly loaded when determining the effects of snow accumulations in roof valleys.

Figure A6: Shape Factor, C_a, for Lower of Multi-Level Roofs



$C_a(0)$ is the lesser of:

$C_a(0) = (\gamma h)/(C_b S_s)$ or

$C_a(0) = F/C_b$

where F is the greater of:

$F = 2$ or

$F = 0.35(\gamma l_c/S_s - 6(\gamma h_p/S_s)^2)^{0.5} + C_b$

l_c is for the upper roof

x_d is the lesser of:

$x_d = 5(h - C_b S_s/\gamma)$ or

$x_d = 5(S_s/\gamma)(F - C_b)$

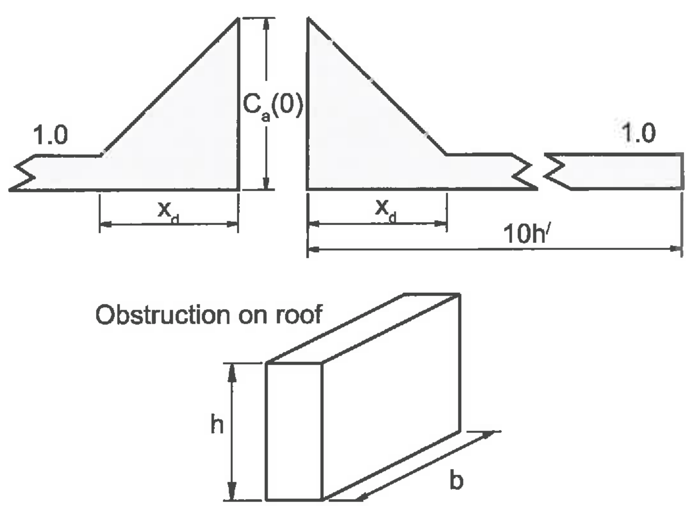
$C_w = 1.0$ for $x_d \leq 10h'$

$C_s = f(\alpha)$

$h' = h - C_b C_w S_s/\gamma$

NOTE:
Additional load from sliding snow should be considered. See Fig.A8

Figure A7: Shape Factor, C_a, Adjacent to Obstructions



For $b \leq 3S_s/\gamma$, $C_a = 1.0$

For $b > 3S_s/\gamma$, C_a is shown in the Figure with x_d and $C_a(0)$ determined as follows:

$x_d = 2h$, but $3m \leq x_d \leq 9m$

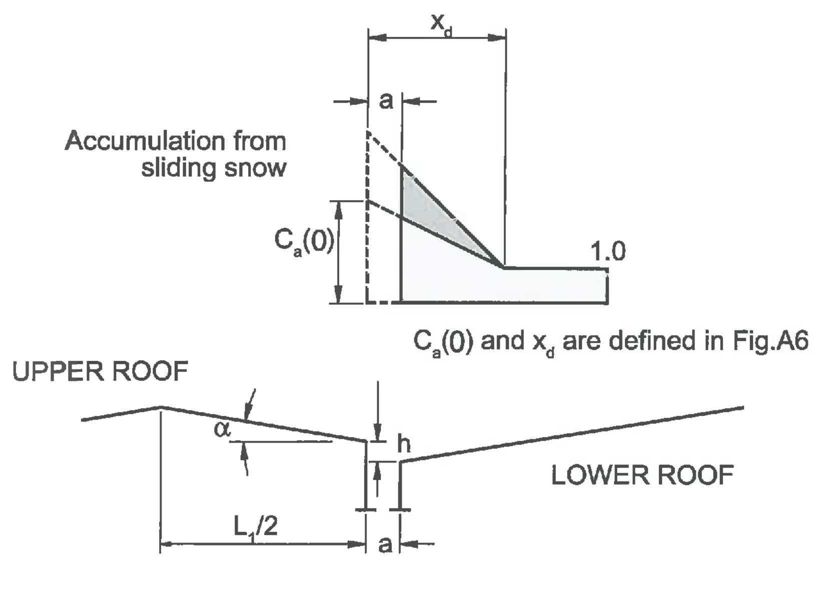
$C_a(0) = 0.67\gamma h/C_b S_s$, but
$0.8/C_b \leq C_a(0) \leq 2.0/C_b$

$C_w = 1.0$ for $x_d \leq 10h'$

$C_s = f(\alpha)$

$h' = h - C_b C_w S_s/\gamma$

Figure A8: Shape Factor, C_a, for Sliding Snow



In the calculation of C_b and $C_a(0)$, l_c and w are taken as the plan dimensions of the lower and upper roofs respectively.

Design lower roof for loading according to Fig.A6, plus 50% of the snow on the portion of the upper roof that slopes towards the lower roof (i.e. $0.5SL_1/2$ per unit of building length)

Distribute additional snow as indicated

$C_w = 1.0$

NOTE:
Under certain conditions snow may slide at very low slopes. Check where critical. Where snow build-up inhibits sliding, a reduced percentage may be taken.

Figure A9: Example #1 Building Geometry

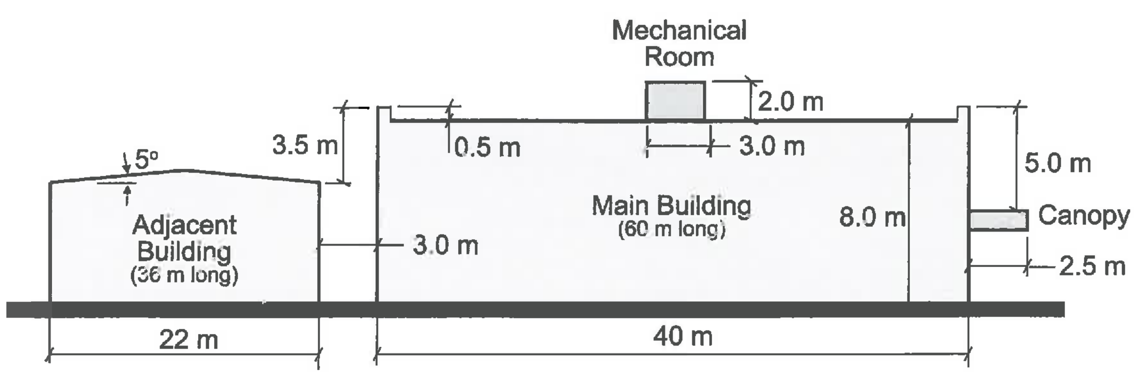


Figure A10: Snow Load Distribution for Example #1

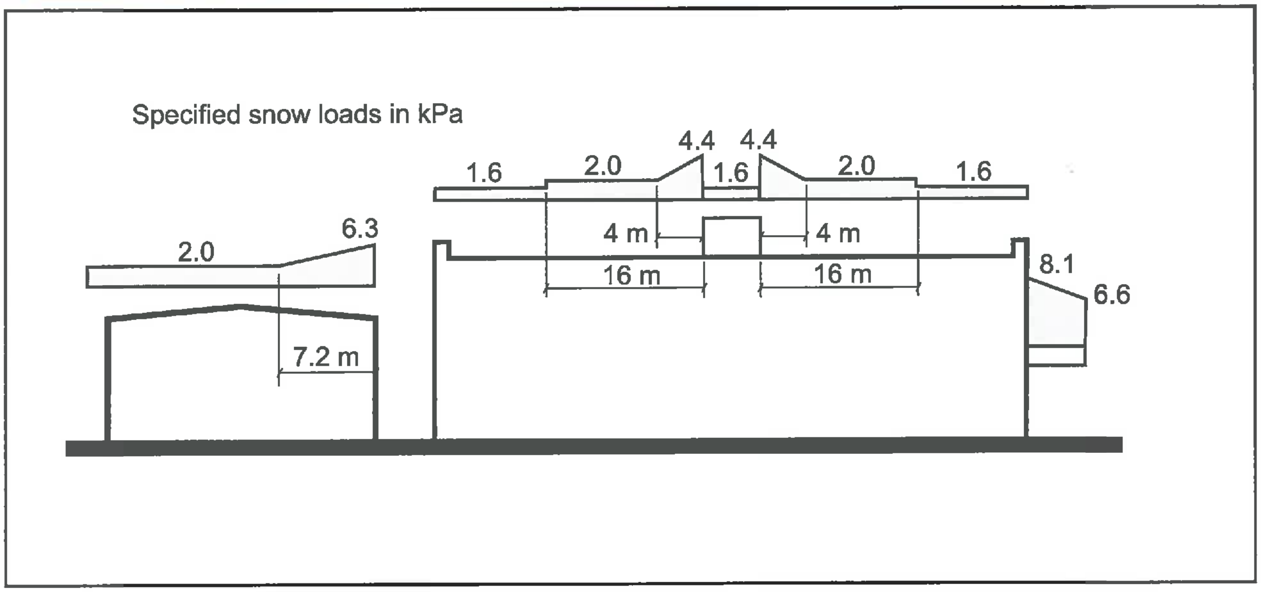


Figure A11: Snow Load Distribution for Example #2

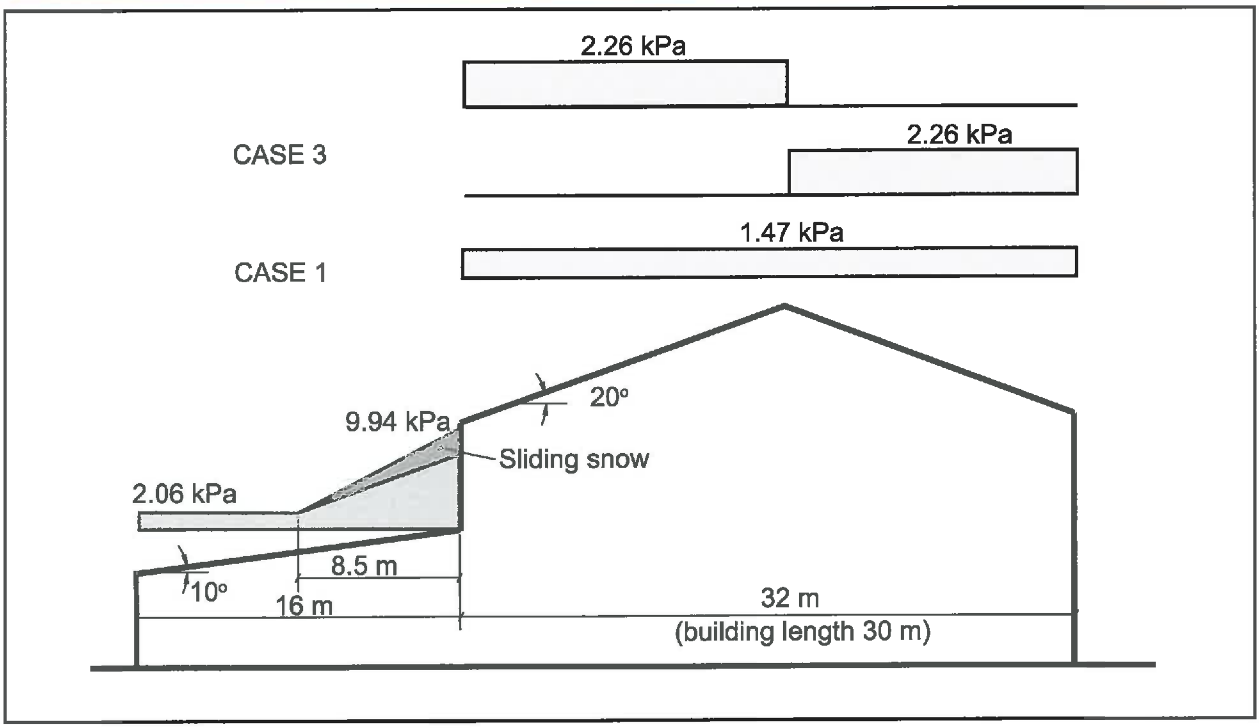
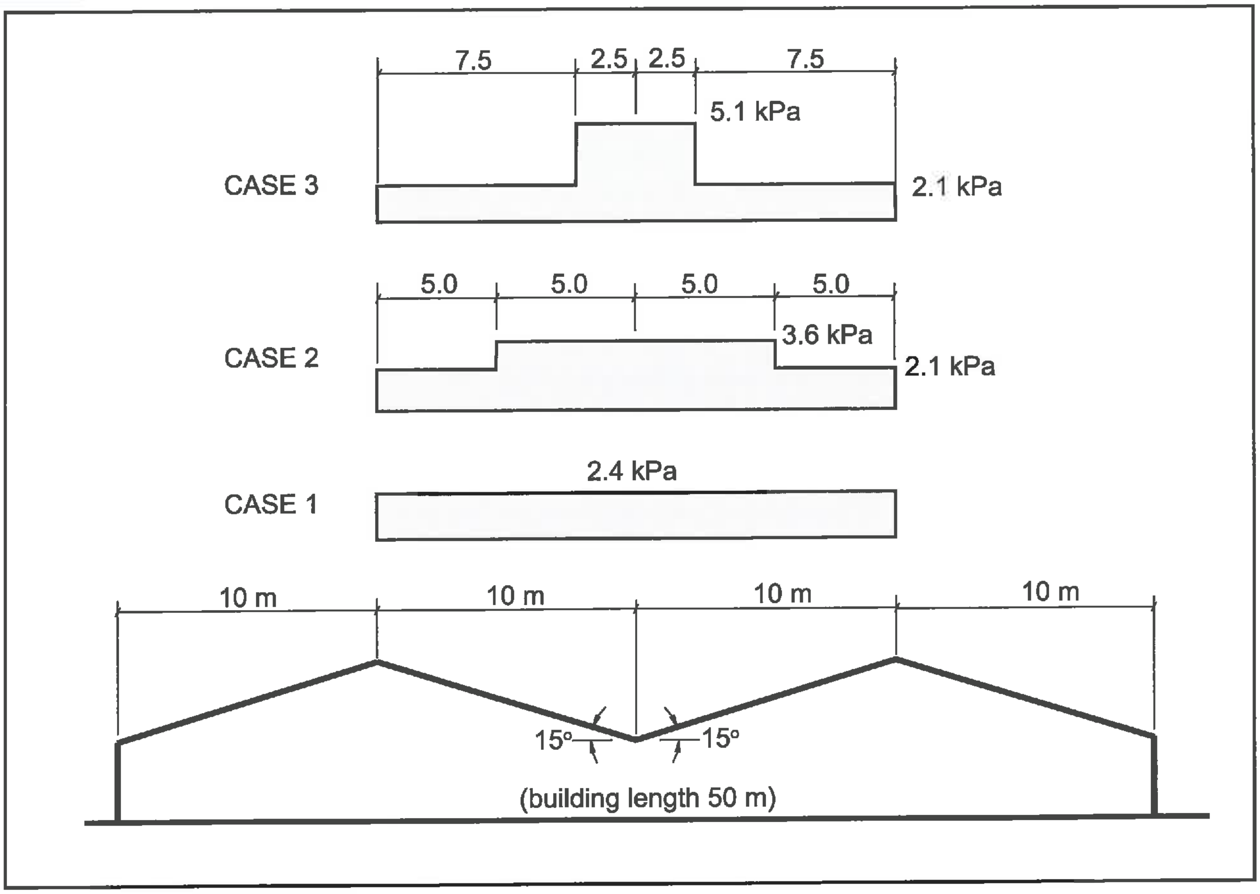


Figure A12: Snow Load Distribution for Example #3



B. Wind Load Design Criteria

B1 Application

The design criteria provided in this section are directly applicable to low rise buildings having height-to-width ratios (H/D_s) not greater than 0.5 and for which the reference height does not exceed 20m. In the absence of more case-specific data, the same criteria may also be used for building where H/D_s is not more than 1.0 and where the reference height does not exceed 20m. Beyond these latter limits, reference should be made to the *User's Guide – NBC 2005 Structural Commentaries (Part 4 of Division B).*

B2 Minimum Design Wind Loads

Manufacturers of Steel Building Systems who are members of the CANADIAN SHEET STEEL BUILDING INSTITUTE use the following criteria to establish minimum design wind loads, unless otherwise required by the governing building regulations or the design authority:

a) For all occupancy classifications and building sizes covered by Part 4 or Part 9 of the NBC 2005, the requirements of Subsection 4.1.7, "Wind Load", are followed.

b) The reference velocity pressures to be used in conjunction with (a) above are given in the *NBC 2005, Division B, Appendix C.*

c) The wind pressure coefficients and factors to be used in conjunction with (a) above are values given in Commentary I of the *User's Guide – NBC 2005 Structural Commentaries (Part 4 of Division B).* The designated "*Static Procedure*" as applicable to low and medium height buildings is used to select the "*exposure*" and "*gust effect*" factors.

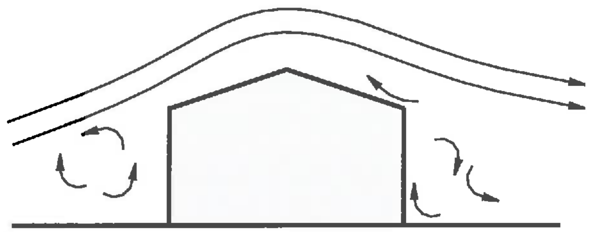
B3 Determination of Wind Pressures

B3.1 General

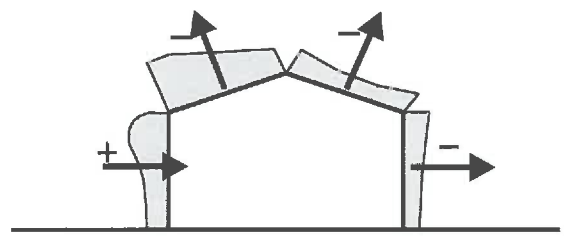
The magnitude of the wind pressures exerted on a building depends primarily on the speed of the wind, the air density, and the interaction between the airflow and the building. Figure B1 illustrates a representative building shape and the airflow lines around it. Pressures are exerted on all surfaces both internally and externally. These pressures are non-uniform, fluctuate widely, and can be much higher than average in certain localized areas. For buildings with flat and low-sloped roofs, the windward wall is the only surface subject to a positive external pressure; all other surfaces experience a negative external pressure. Internal pressure as a result of wind may be either positive or negative, depending on the location and size of openings penetrating the building enclosure.

Denoting the external pressure as "p" and the internal pressure as "p_i", then the net wind pressure on the enclosure of a building is the algebraic difference between the two, i.e. ($p_n = p - p_i$) since p and p_i are considered to act concurrently. Figure B2 illustrates the four cases of external and internal pressures acting.

Figure B1: Air Flow Lines and Resulting External Pressures

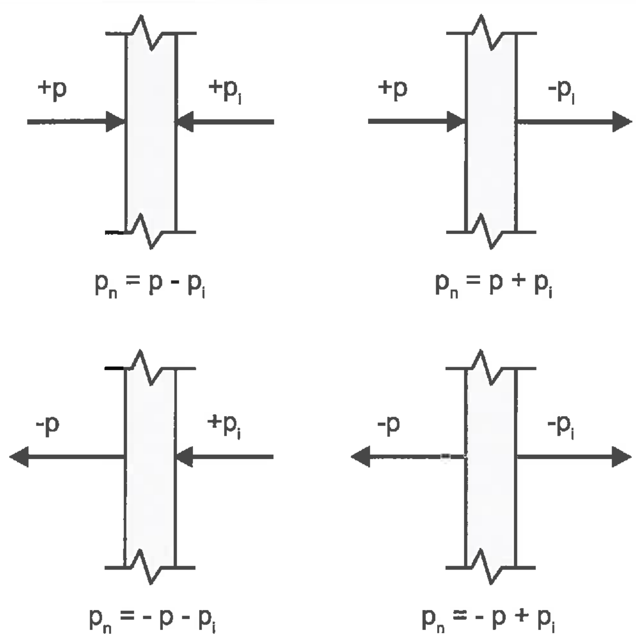


(a) Air Flow Lines



(b) Wind Induced External Pressures

Figure B2: Net Pressures



The net, or resultant, pressure is taken with respect to the external surface. The sign convention used throughout is to consider a pressure as positive when it is directed towards a surface and negative when it is directed away from a surface.

The specified external pressure, p, is calculated from the expression:

$$p = I_w \cdot q \cdot C_e \cdot C_p C_g$$

The specified internal pressure, p_i is calculated from the expression:

$$p_i = I_w \cdot q \cdot C_e \cdot C_{pi} C_{gi}$$

Where

- p = specified external pressure acting statically and in a direction normal to the surface either as a pressure directed towards the surface (positive pressure) or as a suction directed away from the surface (negative pressure)
- p_i = specified internal pressure acting statically and in a direction normal to the surface either as a pressure directed towards the surface (positive pressure) or as a suction directed away from the surface (negative pressure)
- q = reference velocity pressure by geographic location for levels of probability having return periods of 1-in-50 years as listed in *NBC 2005, Division B, Appendix C* for selected locations in Canada
- C_e = exposure factor (see B3.3)
- C_g = gust effect factor (see B3.4)
- C_{gi} = internal gust effect factor (see B3.4)
- C_p = external pressure coefficient averaged over the area of the surface considered (see B3.5)
- C_{pi} = internal pressure coefficient (see B3.5)
- I_w = importance factor for wind load as provided in Table B1 (NBC 2005, Table 4.1.7.1)

Table B1: Importance Factor for Wind Load (I_w)
(See Table I.1 for the Importance Category definitions)

Importance Category	Importance Factor, I_w	
	ULS	SLS
Low	0.8	0.75
Normal	1.0	0.75
High	1.15	0.75
Post-disaster	1.25	0.75

ULS = Ultimate Limit State (e.g. strength), SLS = Serviceability Limit State (e.g. deflection)

B3.2 Conversion from Wind Speeds to Wind Pressure

Since wind is air in motion, the pressure it can exert is related to its kinetic energy (a function of wind speed) expressed as:

$$p = \tfrac{1}{2}\rho V^2$$

Where

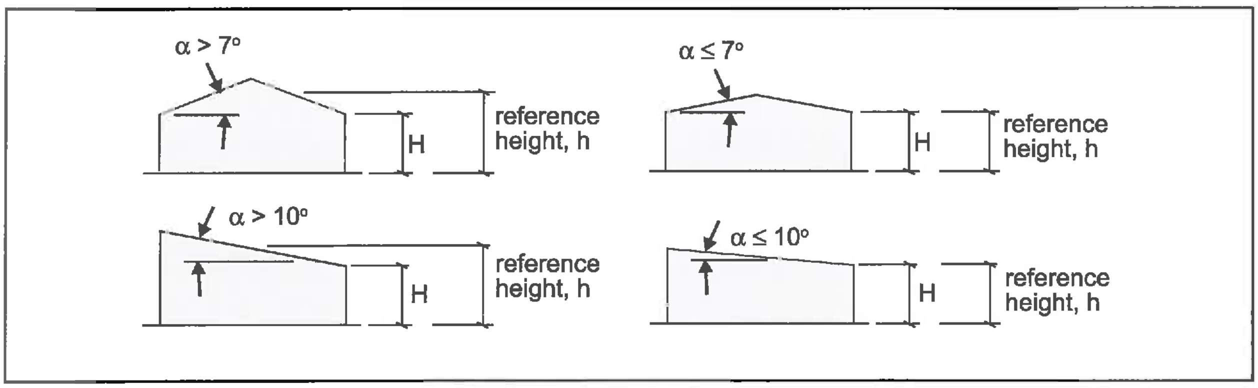
- p = velocity pressure occurring if the full kinetic energy is transformed into pressure, and equal to the reference velocity pressure as given in *NBC 2005, Division B, Appendix C* (kPa)
- ρ = mass density of air (1.2929 kg/m^3)
- V = hourly wind velocity (m/s)

B3.3 Exposure Factor, C_e

The exposure factor reflects the increase in wind speed with height and the effects of variation in the surrounding terrain.

a) For buildings in open level terrain with only scattered buildings, trees or other obstructions, open water or shorelines, the exposure factor maybe taken as $(h/10)^{0.2}$ but not less than 0.9. Where the reference height, h in metres, is defined in Figure B3.
b) For buildings in rough terrain, where rough terrain is suburban, urban or wooded terrain extending upwind from the building uninterrupted for a least 1 km or 10 times the building height, whichever is greater, the exposure factor maybe taken as $0.7(h/12)^{0.3}$ but not less than 0.7. Where the reference height, h in metres, is defined in Figure B3.
c) The exposure factors maybe taken as a value intermediate between the two exposures defined in (a) and (b) in cases where the site is less than 1 km or 10 times the building height from a change in terrain conditions, whichever is greater, provided an appropriate interpolation method is used.
d) An appropriate value depending on both height and shielding may be selected for the exposure factor if a dynamic approach to the action of wind gusts is used.

Figure B3: Reference Heights



The calculations of C_e using the reference height, h, as shown in Figure B3, determines the exposure factor used to calculate external pressures, p. However the exposure factor used to calculate the internal pressure, p_i, is also determined using these methods but using only one half of the reference height.

B3.4 Gust Effect Factors, C_g and C_{gi}

Wind rarely blows at a constant velocity (implying a constant pressure) but comes in gusts, causing intermittently higher pressures. The gust effect factor, C_g, is used to account for this effect. Gusts not only affect the external pressures, but can also affect the internal pressures, depending on the size and distribution of openings in the building enclosure. The gust effect factor, C_g, is assumed to equal 2.0 for the building as a whole and main structural members; 2.5 for external pressures and suction on small elements including cladding; for internal pressure C_{gi} is taken as 2.0 or as determined by detailed calculation which accounts for the sizes of the openings in the building envelope, the internal volume and the flexibility of the building envelope; if a dynamic approach to wind action is used, an appropriate value for C_{gi} can be selected depending on the turbulence of the wind and the size and natural frequency of the structure.

B3.5 Pressure Coefficients, C_p and C_{pi}

The pressure coefficients C_p and C_{pi} are the non-dimensional ratios of wind-induced pressures on a building surface to the reference velocity pressure, q. Wind blowing around a building will create areas of localized pressures that are substantially higher or lower than the reference velocity pressure, typically at wall corners and roof eaves. The external pressure coefficient, C_p, is used to modify the reference velocity pressure to reflect the pressure difference in specific areas. Internal pressures can also fluctuate significantly and are correlated to external pressure fluctuation, but C_{pi} is considered to be constant over the entire interior of the building surface.

B3.6 Full and Partial Loading

Building and structural members shall be capable of withstanding the effects of

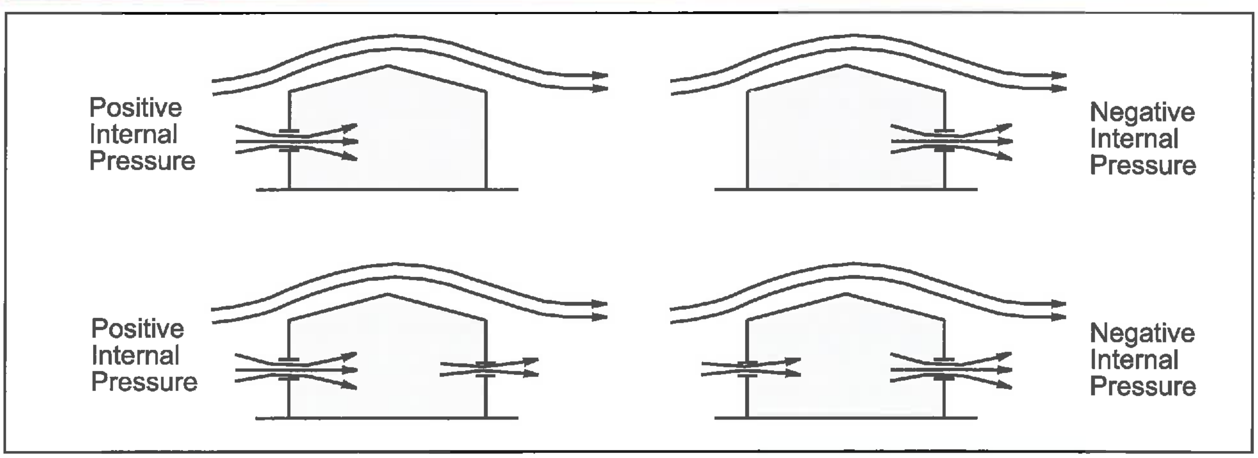
a) the full wind loads acting along each of the 2 principal horizontal axes considered separately;
b) the wind loads as described in (a) but with 100% of the load removed from any portion of the area;
c) the wind loads as in (a) but considered simultaneously at 75% of their value; and,
d) the wind loads as described in (c) but with 50% of these loads removed from any portion of the area.

B3.7 Building Categories for Determination of Internal Pressures

The internal pressure due to wind ($I_w \cdot q \cdot C_e \cdot C_{pi} C_{gi}$) is dependent upon the size and distribution of openings in the building enclosure. Figure B4 illustrates the basic principle of wind-induced internal air pressures.

For design purposes, a large or significant opening is a single or combination of openings on any one wall that offers an opening to the wind exceeding by a factor of 2 or more that of the estimated total leakage area of the remaining building surface including the roof. A large opening may result either from deliberate intent or failure to design an element, such as a window or door, for the appropriate wind pressure. If a door or window is designed to resist the relevant factored wind load, and is not required to be open during periods of high wind, then the area of the opening would be equivalent to the leakage area around the door or window assembly. For typical buildings, whose background leakage area is 0.1 percent of the total surface area, including the roof, an opening greater than 1 percent of the equivalent wall would constitute a "large" opening.

Figure B4: Effect of Openings on Internal Pressure



Three categories of building can be delineated as follows:

Category 1: Building with no "large" openings but having small ones uniformly distributed amounting to less than 0.1% of the total surface area. The value of C_{pi} should be -0.15 except where that alleviates an external load, then $C_{pi} = 0$ should be used. Buildings in this category would include most buildings that are nominally sealed and ventilated mechanically, and other buildings such as windowless warehouses with door systems not prone to storm damage.

Category 2: Buildings in this category have large openings that can be relied on to be closed during storms and small openings uniformly distributed. The value of C_{pi} should be -0.45 to +0.3. Examples would include buildings which, although nominally sealed, may experience a significant imbalance due to the air leakage around doors and windows or through other small openings.

Category 3: Buildings with "large" openings through which gusts are transmitted to the interior. The value of C_{pi} should be -0.7 to +0.7. Such buildings would include, for example, sheds with one open side, and buildings with shipping doors, ventilators or the like which cannot be considered to be closed during a period of high wind.

Notes:

1. It is the responsibility of the party responsible for the design of the building to specify the appropriate building category for purposes of determining internal pressures; otherwise the SBS manufacture has no choice but to make a judgement decision based on the information available.
2. Windows and doors of an SBS supplied by CSSBI members are designed according to the same criteria as the SBS as a whole.

B4 Building Deflection Under Wind Load

Typically, the horizontal deflection (drift) of an SBS is calculated based on the deflection of a single structural frame loaded by its portion of the specified wind load acting statically on the building. However, load sharing, diaphragm action, bracing and other factors contribute significantly to the effective stiffness of a complete SBS. Therefore, the actual drift due to dynamic wind is often but a fraction of the theoretical deflection of a single frame. If the stiffness of the building will be of particular importance to the intended occupancy, the SBS manufacturer should be consulted.

B5 Explanation of Tables B2 through B7

Tables B2 through B7 in this Bulletin give the combined wind pressure coefficients C_pC_g and $C_{pi}C_{gi}$ for the principal structural elements of an SBS. To arrive at the specified internal and external pressures, the coefficients C_p , C_{pi}, C_g and C_{gi} are to be combined with the exposure factor, C_e, (described previously) and the reference velocity pressure, q (given in *NBC 2005 Appendix C of Division B*). Allowances for partial loading are included, without the need for further calculations.

In the tables, z = 10% of the least horizontal dimension or 40% of the height, H, whichever is less. Also, z ≥ 1m and z ≥ 4% of the least horizontal dimension.

The sign convention used for the combined $C_pC_g + C_{pi}C_{gi}$ coefficient is taken with respect to the exterior surface (i.e. positive or negative pressures). In the tables, the number given in brackets after $C_{pi}C_{gi}$ (i.e. (1), (2), & (3)) indicates the building category for which the $C_{pi}C_{gi}$ coefficients apply.

For building configurations not covered by these tables refer to the *User's Guide – NBC 2005 Structural Commentaries (Part 4 of Division B)*.

The coefficients given in the tables are based on specific tributary areas (e.g. no more than 1 m^2 for cladding, not less than 10 m^2 for purlins and equal to 10 m^2 for girts) and are conservative if applied to larger areas.

For convenience, the exposure factor, C_e, is assumed to be the same for both external and internal pressure calculations. If the designer elects otherwise, the coefficients "$C_pC_g + C_{pi}C_{gi}$" given in the following tables will have to be revised to become "$C_eC_pC_g + C_{ei}C_{pi}C_{gi}$".

B6 Illustrative Example

B6.1 Introduction

Consider an SBS building in Toronto, ON, 20m wide x 50m long with an 8m eave height and a roof slope of 1:12 or 5°. The building has various openings, making it a "Category 2" type building. The external cladding and the internal liner sheet are assumed to act independently, i.e. non-composite wall construction. For this type of building the Importance Category is "Normal" and so the Importance Factor for Wind is $I_w = 1.0$.

B6.2 Hourly Wind Pressure, q

Refer to *NBC 2005, Appendix C of Division B* for the tabulated values of hourly wind pressures for selected locations in Canada. Toronto has the following hourly wind pressure:

$$q(1/50) = 0.52 \text{ kPa}$$

B6.3 Exposure Factor, C_e

The exposure factor can be calculated using the *"1/5 power law"*.

$$Ce = \left(\frac{H'}{10}\right)^{0.2} = \left(\frac{8}{10}\right)^{0.2} = 0.96\text{, use } 1.0$$

B6.4 Determine the 'z' Factor

The magnitude of 'z' must be determined to define the areas of localized high pressures and is dependent on the building size.

$$z = (0.10)\cdot(20\text{m}) = 2 \text{ m}$$

but

$$z \le (0.40)\cdot(8\text{m}) = 2.4 \text{ m}$$

and

$$z \ge 1 \text{ m}$$

and

$$z \ge (0.04)\cdot(20\text{m}) = 0.8 \text{ m}$$

Therefore

$$z = 2 \text{ m}$$

B6.5 Internal Pressure on Liner Sheet

Since the building walls and roof are constructed non-compositely (i.e. the cladding and liner sheet act independently) the liner sheet must be capable of resisting the internal air pressure.

$p_i = I_w \cdot q \cdot C_e \cdot C_{pi}C_{gi}$

where

$I_w = 1.0$
$q = 0.52$
$C_e = 1.0$
$C_{pi}C_{gi} = -0.9, +0.6$ (Category 2)

Therefore

$p_i = (1.0)(0.52)(1.0)(-0.90) = -0.47$ kPa, or
$p_i = (1.0)(0.52)(1.0)(+0.60) = +0.31$ kPa

B6.6 Wind Induced Pressures on Roof Cladding

For a building with a roof slope of less than 7°, the wind will cause a negative pressure (uplift) on the roof cladding, with localized higher pressures, and a uniform positive pressure.

$p = I_w \cdot q \cdot C_e \cdot C_pC_g$

where

$I_w = 1.0$
$q = 0.52$
$C_e = 1.0$
C_pC_g are taken from Table B2

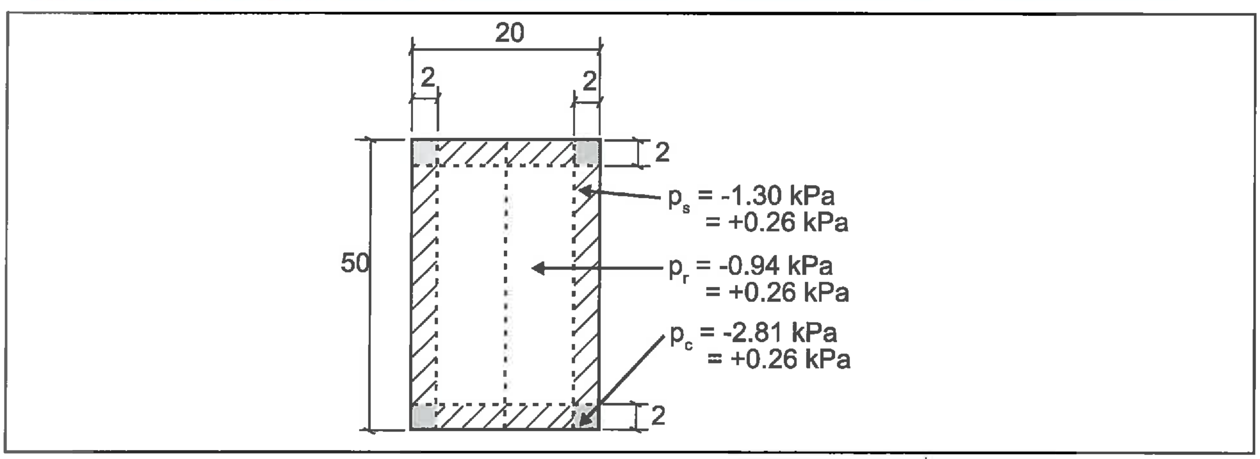
For area 'r', $C_pC_g = +0.5, -1.8$
$p = (1)(0.52)(1)(+0.5) = +0.26$ kPa
$p = (1)(0.52)(1)(-1.8) = -0.94$ kPa

For area 's', $C_pC_g = +0.5, -2.5$
$p = (1)(0.52)(1)(+0.5) = +0.26$ kPa
$p = (1)(0.52)(1)(-2.5) = -1.30$ kPa

For area 'c', $C_pC_g = +0.5, -5.4$
$p = (1)(0.52)(1)(+0.5) = +0.26$ kPa
$p = (1)(0.52)(1)(-5.4) = -2.81$ kPa

Considering a tributary area of 1.0 m², the resulting wind pressures are shown in Figure B5.

Figure B5: Pressures on Roof Cladding



B6.7 Wind Induced Loads on Roof Purlins

The wind-induced pressure on the roof cladding will be carried in turn by the roof purlins. In addition, the internal pressure on the roof liner sheet will also be carried by the roof purlins. The wind pressure loading on the roof purlins can be calculated from the wind pressure distribution given below. To calculate the load on the purlin, multiply the pressure by the tributary area.

$$p = I_w \cdot q \cdot C_e \cdot (C_pC_g + C_{pi}C_{gi})$$

where

I_w = 1.0
q = 0.52
C_e = 1.0
$C_pC_g + C_{pi}C_{gi}$ are taken from Table B3(A)

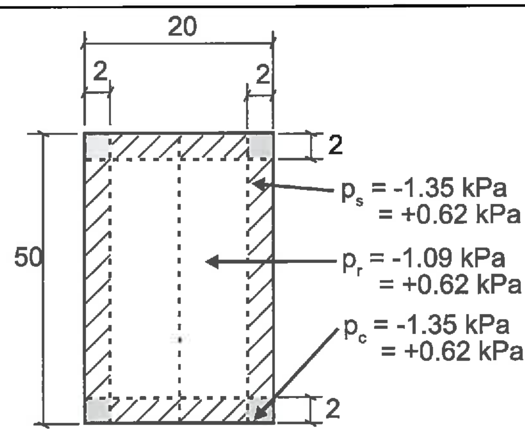
For area 'r', $(C_pC_g + C_{pi}C_{gi}) = +1.2, -2.1$
$p = (1)(0.52)(1)(+1.2) = +0.62$ kPa
$p = (1)(0.52)(1)(-2.1) = -1.09$ kPa

For area 's', $(C_pC_g + C_{pi}C_{gi}) = +1.2, -2.6$
$p = (1)(0.52)(1)(+1.2) = +0.62$ kPa
$p = (1)(0.52)(1)(-2.6) = -1.35$ kPa

For area 'c', $(C_pC_g + C_{pi}C_{gi}) = +1.2, -2.6$
$p = (1)(0.52)(1)(+1.2) = +0.62$ kPa
$p = (1)(0.52)(1)(-2.6) = -1.35$ kPa

The resulting wind pressures are shown in Figure B6.

Figure B6: Pressures Transferred to Roof Purlins



B6.8 Wind Induced Pressure on Wall Cladding

Wind acting on a building can come from any direction and, therefore, can cause either positive or negative pressures on the wall cladding with higher localized pressures at the corners.

$$p = I_w \cdot q \cdot C_e \cdot C_pC_g$$

where

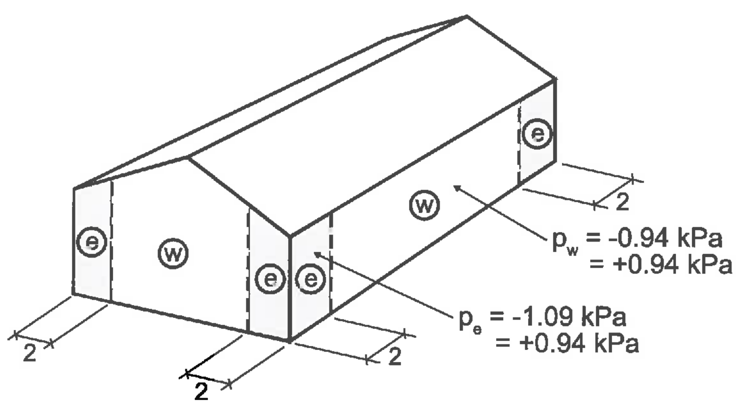
I_w = 1.0
q = 0.52
C_e = 1.0
C_pC_g are taken from Table B4

For area 'w', C_pC_g = +1.8, -1.8
p = (1)(0.52)(1)(+1.8) = +0.94 kPa
p = (1)(0.52)(1)(-1.8) = -0.94 kPa

For area 'e', C_pC_g = +1.8, -2.1
p = (1)(0.52)(1)(+1.8) = +0.94 kPa
p = (1)(0.52)(1)(-2.1) = -1.09 kPa

The resulting wind pressures are shown in Figure B7.

Figure B7: Pressures on Wall Cladding



B6.9 Wind Induced Loads on Wall Girts

The wind induced pressure on the wall cladding and the interior wall liner sheet both act on the wall girts. The wind pressure loading on the wall girts can be calculated from the wind pressure distribution given below.

To calculate the load on a girts, multiply the pressure by the tributary area of the girt.

$$p = I_w \cdot q \cdot C_e \cdot (C_pC_g + C_{pi}C_{gi})$$

where

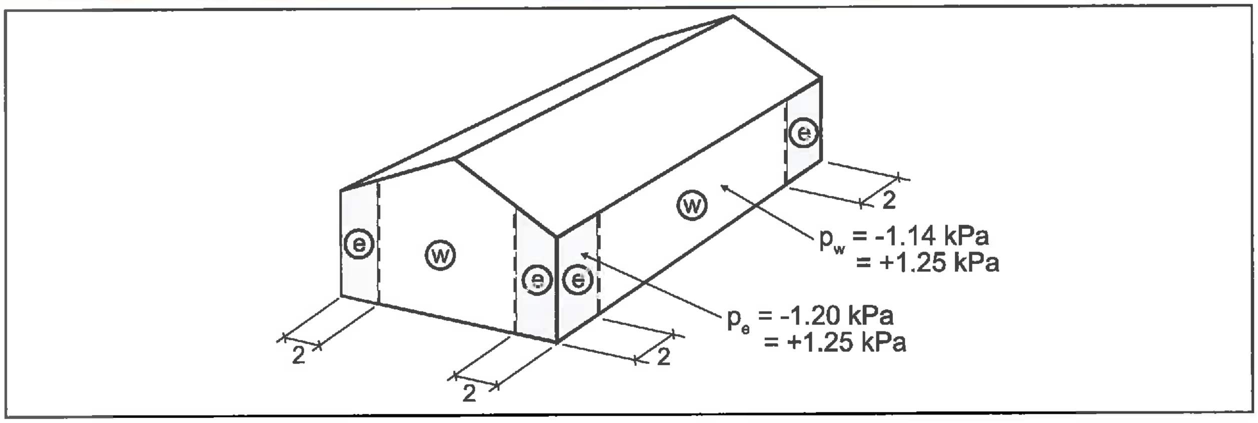
I_w = 1.0
q = 0.52
C_e = 1.0
$C_pC_g + C_{pi}C_{gi}$ are taken from Table B5

For area 'w', $(C_pC_g + C_{pi}C_{gi})$ = +2.4, -2.2
p = (1)(0.52)(1)(+2.4) = +1.25 kPa
p = (1)(0.52)(1)(-2.2) = -1.14 kPa

For area 'e', $(C_pC_g + C_{pi}C_{gi})$ = +2.4, -2.3
p = (1)(0.52)(1)(+2.4) = +1.25 kPa
p = (1)(0.52)(1)(-2.3) = -1.20 kPa

The resulting wind pressures are shown in Figure B8.

Figure B8: Pressures Transferred to Wall Girts



B6.10 Wind Induced Loads on Interior Frames

The wind induced pressure on the wall and roof cladding is transferred through the girts and purlins and acts on the structural frames. To calculate the wind induced load on an interior frame, multiply the pressure shown below by the tributary areas of the purlins and girts to obtain reactions which will act on the structural frames as a series of point loads. Note that the wind can blow from either direction.

Design Case 1: External wind pressures acting on all external surfaces simultaneously.

$$p = I_w \cdot q \cdot C_e \cdot C_p C_g$$

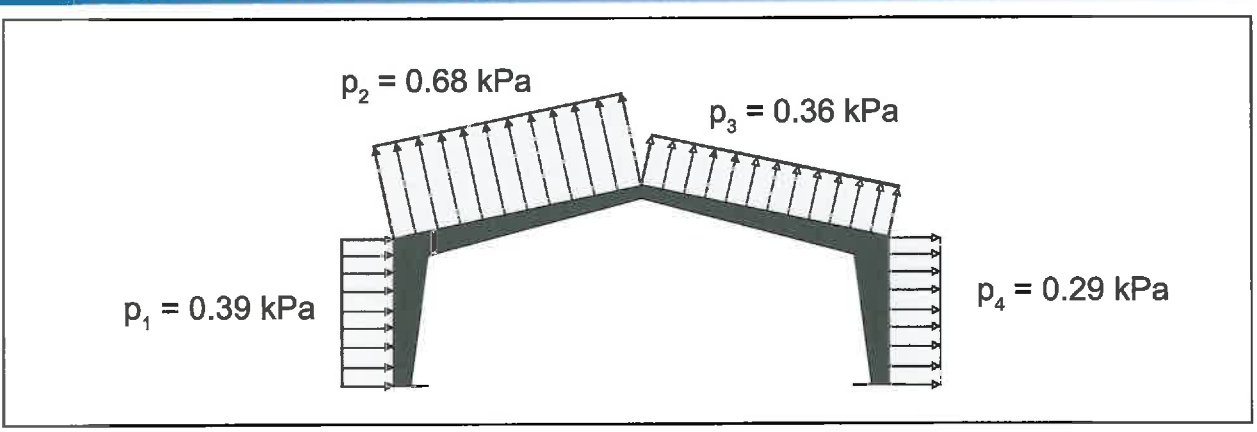
where

I_w = 1.0
q = 0.52
C_e = 1.0
C_pC_g are taken from Table B6 for an interior frame

For area (1), C_pC_g = +0.75, p = (1)(0.52)(1)(+0.75) = +0.39 kPa
For area (2), C_pC_g = -1.30, p = (1)(0.52)(1)(-1.30) = -0.68 kPa
For area (3), C_pC_g = -0.70, p = (1)(0.52)(1)(-0.70) = -0.36 kPa
For area (4), C_pC_g = -0.55, p = (1)(0.52)(1)(-0.55) = -0.29 kPa

The resulting wind pressures are shown in Figure B9.

Figure B9: Design Case 1 Frame Loading



Design Case 2: External wind pressures plus internal wind pressures acting together.

$$p = I_w \cdot q \cdot C_e \cdot (C_pC_g + C_{pi}C_{gi})$$

where,

I_w = 1.0
q = 0.52
C_e = 1.0
C_pC_g are taken from Table B6 for an interior frame
$C_{pi}C_{gi}$ = -0.90 or +0.60

The sign convention is relative to the external cladding surface. Therefore, the internal pressure coefficients change sign.

For a positive internal pressure ($C_{pi}C_{gi}$ = +0.60).
For area (1), p = (1)(0.52)(1)(+0.75 –0.60) = +0.08 kPa
For area (2), p = (1)(0.52)(1)(-1.30 –0.60) = -0.99 kPa
For area (3), p = (1)(0.52)(1)(-0.70 –0.60) = -0.68 kPa
For area (4), p = (1)(0.52)(1)(-0.55 –0.60) = -0.60 kPa

For a negative internal pressure ($C_{pi}C_{gi}$ = -0.90)
For area (1), p = (1)(0.52)(1)(+0.75 +0.90) = +0.86 kPa
For area (2), p = (1)(0.52)(1)(-1.30 +0.90) = -0.21 kPa
For area (3), p = (1)(0.52)(1)(-0.70 +0.90) = +0.10 kPa
For area (4), p = (1)(0.52)(1)(-0.55 +0.90) = +0.18 kPa

The resulting wind pressures are shown in Figure B10.

Figure B10: Design Case 2 Frame Loading

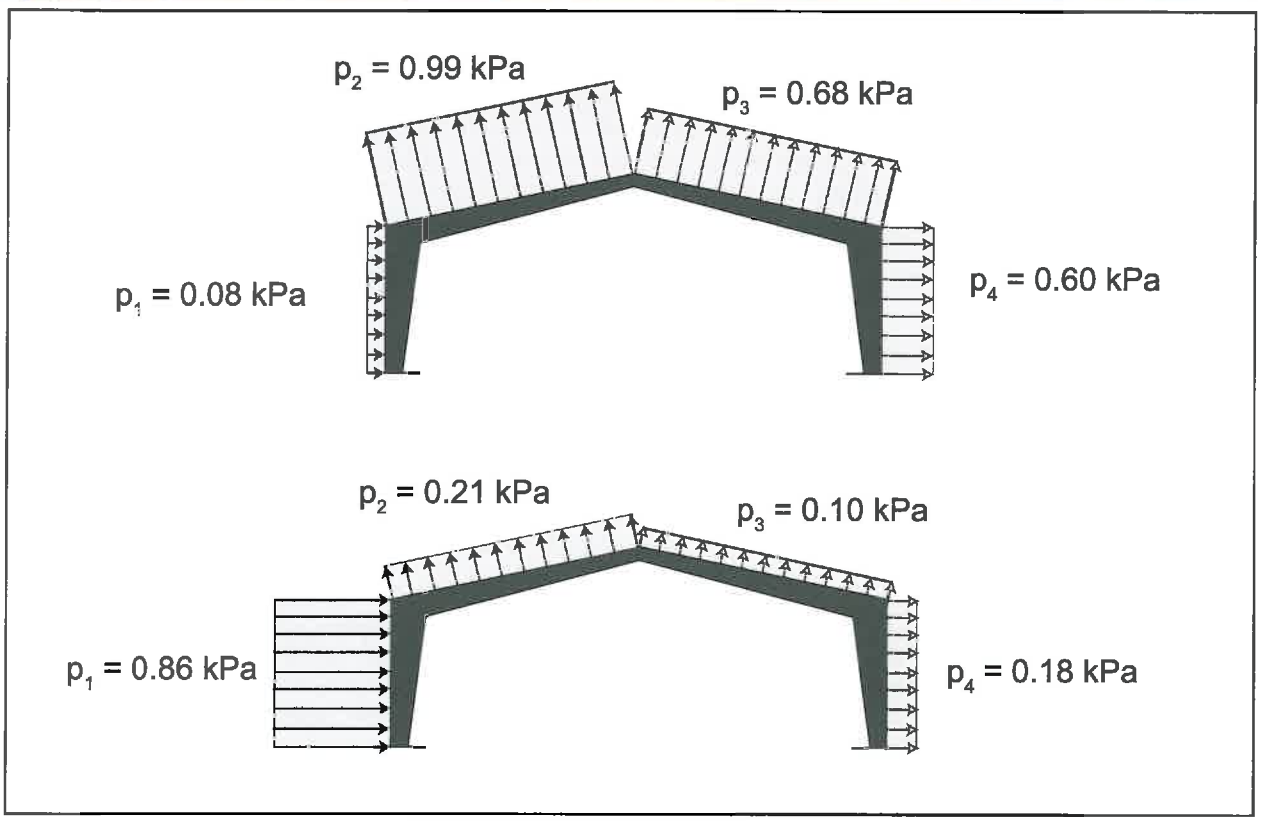
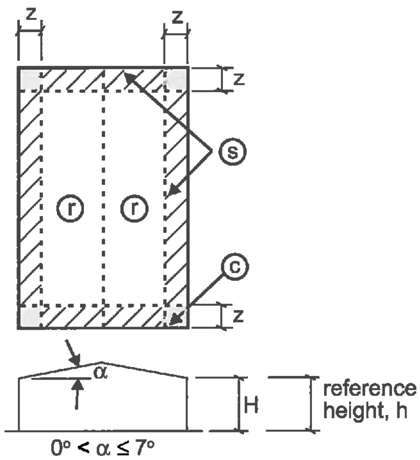


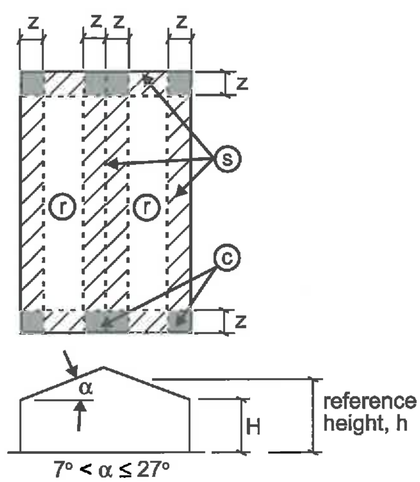
TABLE B2: ROOF CLADDING

B2(A) FOR ROOF SLOPES OF 7° OR LESS



Building Category		r	s	c		
	C_pC_g	+0.5 -1.8	+0.5 -2.5	+0.5 -5.4		
(3)	$C_{pi}C_{gi}$	±1.4	±1.4	±1.4		Non-composite interior and exterior sheets
(2)	$C_{pi}C_{gi}$	+0.6 -0.9	+0.6 -0.9	+0.6 -0.9		Non-composite interior and exterior sheets
(1)	$C_{pi}C_{gi}$	+0.0 -0.3	+0.0 -0.3	+0.0 -0.3		Non-composite interior and exterior sheets
(3)	$C_pC_g + C_{pi}C_{gi}$	+1.9 -3.2	+1.9 -3.9	+1.9 -6.8	Single sheet or composite interior plus exterior sheet	Single sheet or composite interior plus exterior sheet
(2)	$C_pC_g + C_{pi}C_{gi}$	+1.4 -2.4	+1.4 -3.1	+1.4 -6.0	Single sheet or composite interior plus exterior sheet	Single sheet or composite interior plus exterior sheet
(1)	$C_pC_g + C_{pi}C_{gi}$	+0.8 -1.8	+0.8 -2.5	+0.8 -5.4	Single sheet or composite interior plus exterior sheet	Single sheet or composite interior plus exterior sheet

B2(B) FOR ROOF SLOPES GREATER THAN 7°, BUT LESS THAN 27°



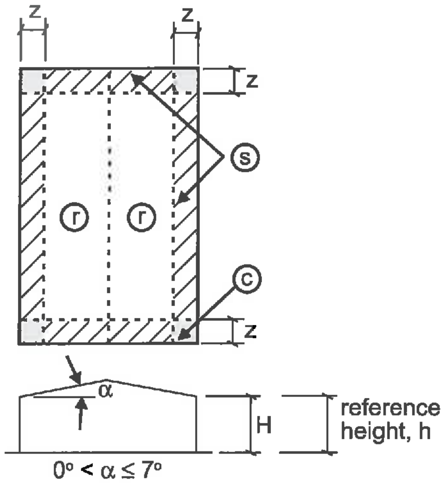
Building Category		r	s	c	
	C_pC_g	+0.8 -2.4	+0.8 -4.1	+0.8 -5.0	
(3)	$C_{pi}C_{gi}$	±1.4	±1.4	±1.4	Non-composite interior and exterior sheets
(2)	$C_{pi}C_{gi}$	+0.6 -0.9	+0.6 -0.9	+0.6 -0.9	Non-composite interior and exterior sheets
(1)	$C_{pi}C_{gi}$	+0.0 -0.3	+0.0 -0.3	+0.0 -0.3	Non-composite interior and exterior sheets
(3)	$C_pC_g + C_{pi}C_{gi}$	+2.2 -3.8	+2.2 -5.5	+2.2 -6.4	Single sheet or composite interior plus exterior sheet
(2)	$C_pC_g + C_{pi}C_{gi}$	+1.7 -3.0	+1.7 -4.7	+1.7 -5.6	Single sheet or composite interior plus exterior sheet
(1)	$C_pC_g + C_{pi}C_{gi}$	+1.1 -2.4	+1.1 -4.1	+1.1 -5.0	Single sheet or composite interior plus exterior sheet

NOTES:

- Coefficients are based on a tributary area ≤ 1 m² and are conservative for larger areas
- For larger tributary areas, refer to Figures I-9 and I-11 in the Users Guide.
- q should be selected on a 1 in 50 year return period.
- z is defined in section B5 of the preceding text
- Coefficients are appropriate in all wind directions
- For roofs with parapets, refer to Note 8, Figure I-9 in the Users Guide

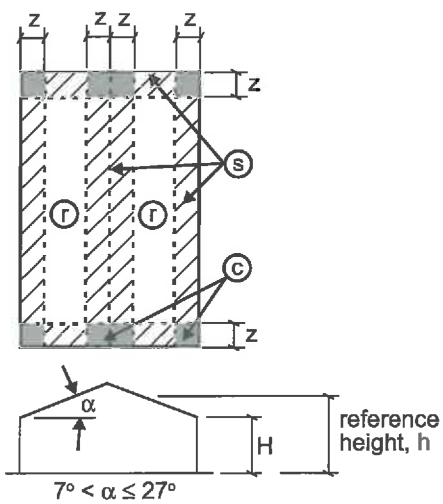
TABLE B3: PURLINS

B3(A) FOR ROOF SLOPES OF 7° OR LESS



Building Category		r	s	c
	C_pC_g	+0.3 -1.5	+0.3 -2.0	+0.3 -2.0
(3)	$C_pC_g + C_{pi}C_{gi}$	+1.7 -2.9	+1.7 -3.4	+1.7 -3.4
(2)	$C_pC_g + C_{pi}C_{gi}$	+1.2 -2.1	+1.2 -2.6	+1.2 -2.6
(1)	$C_pC_g + C_{pi}C_{gi}$	+0.6 -1.5	+0.6 -2.0	+0.6 -2.0

B3(B) FOR ROOF SLOPES GREATER THAN 7°, BUT LESS THAN 27°

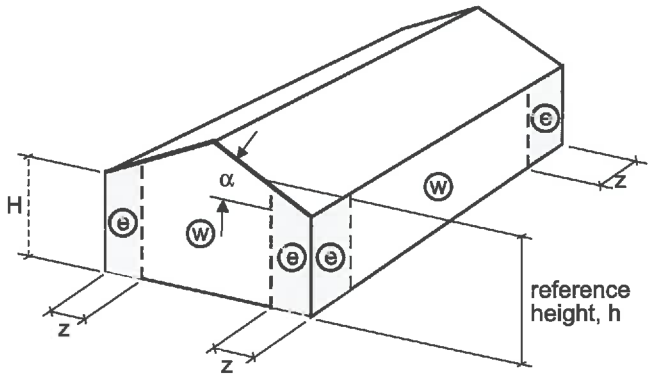


Building Category		r	s	c
	C_pC_g	+0.5 -2.0	+0.5 -2.6	+0.5 -4.0
(3)	$C_pC_g + C_{pi}C_{gi}$	+1.9 -3.4	+1.9 -4.0	+1.9 -5.4
(2)	$C_pC_g + C_{pi}C_{gi}$	+1.4 -2.6	+1.4 -3.2	+1.4 -4.6
(1)	$C_pC_g + C_{pi}C_{gi}$	+0.8 -2.0	+0.8 -2.6	+0.8 -4.0

NOTES:

- Coefficients are based on a tributary area ≥ 10 m^2
- For smaller tributary areas, refer to Figures I-9 and I-11 in the Users Guide.
- q should be selected on a 1 in 50 year return period.
- z is defined in section B5 of the preceding text
- Coefficients are appropriate in all wind directions

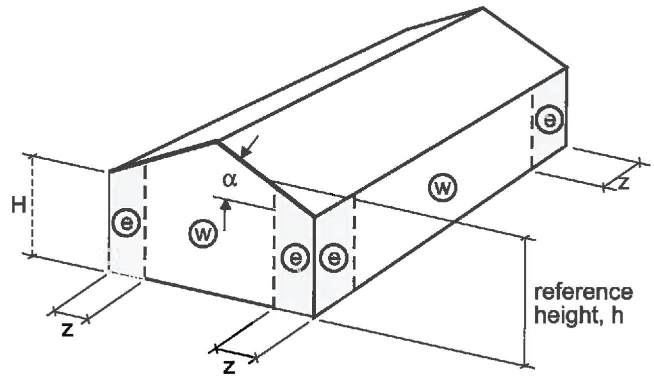
TABLE B4: WALL CLADDING



Building Category		w		e		
	C_pC_g	+1.8	-1.8	+1.8	-2.1	
(3)	$C_{pi}C_{gi}$	-1.4	+1.4	-1.4	+1.4	Non-composite interior and exterior sheets
(2)	$C_{pi}C_{gi}$	-0.9	+0.6	-0.9	+0.6	
(1)	$C_{pi}C_{gi}$	-0.3	+0.0	-0.3	+0.0	
(3)	$C_pC_g + C_{pi}C_{gi}$	+3.2	-3.2	+3.2	-3.5	Single sheet or composite interior plus exterior sheet
(2)	$C_pC_g + C_{pi}C_{gi}$	+2.7	-2.4	+2.7	-2.7	
(1)	$C_pC_g + C_{pi}C_{gi}$	+2.1	-1.8	+2.1	-2.1	

NOTES:
- Coefficients are based on a tributary area ≤ 1 m^2 and are conservative for larger areas
- For larger tributary areas, refer to Figure I-8 in the Users Guide.
- q should be selected on a 1 in 50 year return period.
- z is defined in section B5 of the preceding text
- Coefficients are appropriate in all wind directions
- This table applies to buildings with any roof slope, α

TABLE B5: GIRTS AND ENDWALL COLUMNS



B5(A) GIRTS

Building Category		w		e	
	C_pC_g	+1.5	-1.6	+1.5	-1.7
(3)	$C_{pi}C_{gi}$	-1.4	+1.4	-1.4	+1.4
(2)	$C_{pi}C_{gi}$	-0.9	+0.6	-0.9	+0.6
(1)	$C_{pi}C_{gi}$	-0.3	+0.0	-0.3	+0.0
(3)	$C_pC_g + C_{pi}C_{gi}$	+2.9	-3.0	+2.9	-3.1
(2)	$C_pC_g + C_{pi}C_{gi}$	+2.4	-2.2	+2.4	-2.3
(1)	$C_pC_g + C_{pi}C_{gi}$	+1.8	-1.6	+1.8	-1.7

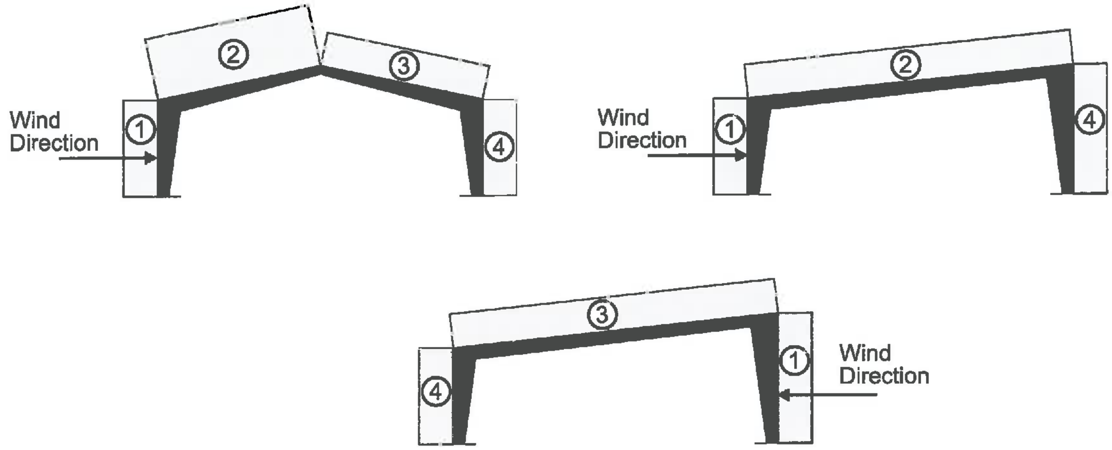
B5(B) ENDWALL COLUMNS

Building Category		w		e	
	C_pC_g	+0.75	-0.55	+1.15	-0.80
(3)	$C_pC_g + C_{pi}C_{gi}$	+2.15	-1.95	+2.55	-2.20
(2)	$C_pC_g + C_{pi}C_{gi}$	+1.65	-1.15	+2.05	-1.40
(1)	$C_pC_g + C_{pi}C_{gi}$	+1.05	-0.55	+1.45	-0.80

NOTES:

- Coefficients in Table B5(A) are based on a tributary area = 10 m^2 and are conservative for larger areas
- For other tributary areas, refer to Figure I-8 in the Users Guide.
- Coefficients in Table B5(B) are independent of tributary area and are based on Figure I-7 in the Users Guide.
- q should be selected on a 1 in 50 year return period.
- z is defined in section B5 of the preceding text
- Coefficients are appropriate in all wind directions
- This table applies to buildings with any roof slope, α

TABLE B6: RIGID FRAMES PERPENDICULAR TO SIDEWALL



B6(A) FOR ROOF SLOPES OF 5° OR LESS

	1	2	3	4	
C_pC_g	+1.15	-2.00	-1.00	-0.80	Endwall
C_pC_g	+0.95	-1.65	-0.85	-0.68	1st Interior Frame
C_pC_g	+0.75	-1.30	-0.70	-0.55	Interior Frame

DESIGN CASE 1 - External wind loads on all surfaces simultaneously
DESIGN CASE 2 – External wind loads plus internal pressures

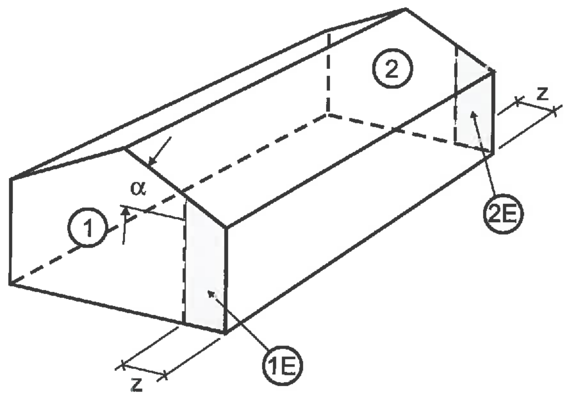
B6(B) FOR ROOF SLOPES OF 20°

	1	2	3	4	
C_pC_g	+1.50	-2.00	-1.30	-1.20	Endwall
C_pC_g	+1.25	-1.65	-1.10	-1.00	1st Interior Frame
C_pC_g	+1.00	-1.30	-0.90	-0.80	Interior Frame

NOTES:

- For roof slopes between 5° and 20°, linear interpolation between the two tables is appropriate
- Frames shall be designed for wind blowing from either direction
- q should be selected on a 1 in 50 year return period
- Coefficients are from Load Case A in Figure I-7 in the Users Guide
- Values for the 1st Interior Frame are interpolations between the values for the Endwall and Interior Frame

TABLE B7: LONGITUDINAL BRACING



	1	1E	2	2E
C_pC_g	+0.75	+1.15	-0.55	-0.80

NOTES:

- Bracing is to resist the wind loads on the endwalls
- Braces shall be designed to resist wind loads from either direction
- This table applies to any roof slope
- z is defined in section B5 of the preceding text
- Coefficients are from Load Case B in Figure I-7 of the Users Guide

C. Earthquake Load Design Criteria

C1 Major Changes from NBC 1995 to 2005[1]

(a) Updated Hazard in Spectral Format

The seismic hazard inherent in the NBC 1995 was described in terms of peak ground velocity, v, and acceleration, a, determined at a probability of exceedance of 10% in 50 years. The period-dependent variation of seismic forces was obtained by multiplying v by a seismic response factor dependent on the ratio of v to a. NBC 2005 is based on uniform hazard spectra – spectral acceleration ordinates at different periods calculated at the same probability of exceedance. The Geological Survey of Canada is providing spectral acceleration values for specific geographical locations in Canada, incorporating the differences in the spectral shape across the country in the determination of the design forces rather than being approximated by amplifying zonal values of peak ground velocity.

(b) Change in Return Period (Probability of Exceedance)

The NBC 1995 was based on a seismic hazard of 10% in 50-year probability of exceedance, corresponding to a return period of 475 years (ignoring the contributions of various sources of conservatism in this estimation). However, this return period did not provide for a uniform margin of collapse in all parts of Canada. In order to address this inconsistent treatment across the county, the NBC 2005 has adopted the use of "maximum considered earthquake ground motion" defined as earthquake ground motion having a 2% in 50-year probability of exceedance, corresponding to a return period of approximately 2500 years. The detailed explanations of why this change was necessary can be found in the *NBC 2005 User's Guide, Commentary J.*

(c) Period-Dependent Site Factors

The amplification of seismic motions from rock to soil can be significant, especially at sites with soft soil conditions. The NBC 1995 included a foundation factor, but it did not vary with period or with the intensity of the underlying rock motion. New research has allowed the NBC 2005 to categorize soil profiles using quantitative measures of soil properties, while recognizing the period-dependence of ground motions and the effects of the intensity of underlying rock motion. This means that a more in-depth investigation of site conditions is necessary to determine the design loads appropriate to the building.

(d) Delineation of Effects of Overstrength and Ductility

The NBC has recognized that the seismic forces are reduced when structural response goes into the inelastic range, enabling structures to resist strong earthquake shaking, provided they have the capacity to deform inelastically through several load reversals without a significant loss of strength. NBC 1995 recognized this by including the force modification factor, R, used to reduce the lateral seismic force, V. NBC 2005 replaces the R and U factors with two components: an overstrength factor, R_o, and a ductility factor, R_d. These factors are dependent on the specific Seismic Force Resisting System (SFRS) and reflect the minimum level of overstrength and the ductility of the system. These factors are often very specific to pre-defined structural systems such as those described in CSA-S16 for steel structures.

(e) Period Calculations

The calculation of the fundamental lateral period, T_a, is significant because its value determines the spectral acceleration, $S(T_a)$, which in turn determines the seismic design

[1] Taken from User's Guide – NBC 2005 Structural Commentaries (Part 4 of Division B) National Research Council of Canada, 2006

force. The formulae for calculating the periods of moment-resisting frames are the same in NBC 2005 as in NBC 1995, although upper limits are now applied to the periods of the SFRS rather than placing a lower limit on the seismic force.

(f) Higher Mode Effects in the Equivalent Static Force Procedure

The static equivalent lateral seismic force calculated in NBC 2005 is based on the assumption that the main features of the dynamic response of the structure can be represented by a single mode response at the fundamental period, T_a. Many structures with longer periods have significantly higher mode effects, taken into account by modifications to both the value of the seismic design force and the distribution of the shears and moments along the height of the structure. The simulation of higher mode effects in an equivalent static procedure is not valid for structures with long periods because their response is dominated by the second or even third mode. Consequently, NBC 2005 allows the Equivalent Static Force Procedure for regular structures with periods less than 2 s, whose height is less than 60 m, located in regions of low seismicity. For all other structures, dynamic analyses must be used.

(g) Irregularities

The NBC 1995 included some general statements regarding discontinuous vertical resisting elements, but there were no specific requirements concerning structures with irregularities. NBC 2005 includes eight types of irregularities and specifications regarding the analysis and design of each. Irregularities can be caused by variations in vertical stiffness, dead load, vertical geometry, in-plane discontinuity in the vertical lateral-force resisting element, out-of-plane offsets, weak storey, torsional sensitivity, and non-orthogonal systems. With some exceptions, irregular structures must be designed using a dynamic analysis.

(h) Dynamic Analysis Requirements

Dynamic analysis was not required in NBC 1995 although designers were given the option of using it to determine the distribution of seismic forces within the structure or for determining torsional moments. In NBC 2005 a dynamic analysis is required except for limited applications where the Equivalent Static Force Procedure is allowed. The rational for this radical change is that Linear Dynamic Analysis may stimulate the effects on a structure more specifically than the static method but both rely on the quality of the model for satisfactory results. Conducting a dynamic analysis is facilitated by the fact that seismic hazard is now specified in terms of spectral acceleration.

(i) Special Provisions

NBC 1995 included a number of special provisions that imposed restrictions when the velocity or acceleration related seismic zone was at a certain level, limiting the kind of structural system that could be used. NBC 2005 continues to include restrictions, but the limits are included directly in the table defining the force modification factors for the structural system. Other restrictions are included throughout the various clauses.

C2 Minimum Live Loads Due to Earthquake

C2.1 General

The design for live loads due to earthquakes assumes that the force generated from a ground motion is applied as a horizontal shear force, V, located at the base of the structure. The base shear force is translated into lateral forces at each storey level in proportion to the distribution of mass within the building. The base shear and lateral forces are calculated for both principal directions of the building, and it is typical to consider these separately for steel building systems.

With the increased complexities inherent in the NBC 2005, it is beyond the scope of this bulletin to present all of the details for determining the earthquake loads: expert guidance is recommended.

C2.2 The Design Process (Equivalent Static Force Procedure)

NBC 2005 stipulates one of two approaches for determining the design earthquake loads: the *Dynamic Analysis Procedure* or the *Equivalent Static Force Procedure*. The Equivalent Static Force Procedure is a simplified method applicable to structures that satisfy very specific requirements. The steps listed in Table C1 provide a flowchart of the design process using the Equivalent Static Force Procedure. Many steel building systems would qualify to be designed using this simplified method, and so the requirements are presented here. Other references are necessary for applying the Dynamic Analysis Procedure, which is beyond the scope of this bulletin.

Table C1: Flowchart for the Equivalent Static Force Procedure

Step	Task	NBC 2005 Reference
1	Determine $S_a(T)$ for site location from Table C-2, Appendix C of Division B, or as provided by geotechnical analysis	
2	If $S(0.2) \leq 0.12$ then seismic design is not required	4.1.8.1.1)
3	Establish Seismic Force Resisting System(s)	4.1.8.3.3)
4	Confirm that there are no stiff elements affecting the SFRS response by adding stiffness or unintended load paths	4.1.8.3.6) 4.1.8.3.7)
5	Confirm a 2-D model to be valid for the structure (e.g. no rigid diaphragm or eccentric loads/stiffness) in each orthogonal direction	4.1.8.3.8)
6	Obtain site properties: $S_a(T)$, obtained from Step 1 Site Class (A to F) from Table 4.1.8.4.A per geotechnical engineer Determine acceleration and velocity based coefficients: F_a from Table 4.1.8.4.B F_v from Table 4.1.8.4.C Calculate spectral acceleration values $S(T)$ for $T=0.2, 0.5, 1.0, 2.0$ and ≥ 4 s	 4.1.8.4.1) 4.1.8.4.2) & 3) 4.1.8.4.4) 4.1.8.4.5) 4.1.8.4.6)
7	Determine Earthquake Importance Factor, I_E, from Table 4.1.8.5	4.1.8.5.1)
8	Determine structural configuration (irregularities) per Table 4.1.8.6	4.1.8.6
9	Static analysis may be permitted even if structure is irregular within certain limits provided in Table 4.1.8.6	4.1.8.7
10	Determine SFRS Ductility and Overstrength Factors from Table 4.1.8.9, with consideration of multiple congruent bracing systems and height restrictions SFRS Ductility Factor, R_d System Overstrength Factor, R_o	4.1.8.9
11	Determine lateral force variables for Equivalent Static Force Procedure Fundamental lateral period, T_a, for each orthogonal direction Dead Load, W Higher Mode Factor, M_v Base Overturning Moment Reduction Factor, J Minimum Lateral Earthquake Force, V Force distribution, F_x Overturning moments, M_x Torsional effects, if appropriate	 4.1.8.11.4) 4.1.8.11.5) 4.1.8.11.4) 4.1.8.11.5) 4.1.8.11.2) 4.1.8.11.6) 4.1.8.11.7) 4.1.8.11.9) & 10)
12	Apply loads in static analysis model with consideration of force distribution	

C2.3 Notations

F_a = acceleration-based site coefficient
F_v = velocity-based site coefficient
I_E = earthquake importance factor of the structure
J = numerical reduction coefficient for base overturning moment
M_v = factor to account for higher mode effect on base shear
R_d = ductility-related force modification factor reflecting the capability of a structure to dissipate energy through inelastic behaviour
R_o = overstrength-related force modification factor accounting for the dependable portion of reserve strength in a structure designed according to the NBC 2005
$S(T)$ = design spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T
$S_a(T)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T
SFRS = Seismic Force Resisting System(s) is that part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects
T = period in seconds
T_a = fundamental lateral period of vibration of the building, in seconds, in the direction under consideration
V = lateral earthquake design force at the base of the structure
W = dead load, except that the minimum partition load need not exceed 0.5 kPa, plus 25% of the design snow load plus 60% of the storage load for areas used for storage, except that storage garages need not be considered storage areas, and the full contents of any tanks

Notes

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