d) increased non-uniform snow loads on areas adjacent to roof projections, such as penthouses, large *chimneys* and equipment, and
e) increased snow or ice loads due to snow sliding or meltwater draining from adjacent roofs.

4.1.6.3. Full and Partial Loading

1) A roof or other *building* surface and its structural members subject to loads due to snow accumulation shall be designed for the specified load given in Sentence 4.1.6.2.(1), distributed over the entire loaded area.

2) In addition to the distribution mentioned in Sentence (1), flat roofs and shed roofs, gable roofs of 15° slope or less, and arched or curved roofs shall be designed for the specified uniform snow load indicated in Sentence 4.1.6.2.(1), which shall be calculated using $C_a = 1.0$, distributed on any one portion of the loaded area and half of this load on the remainder of the loaded area, in such a way as to produce the most critical effects on the member concerned. (See Appendix A.)

4.1.6.4. Specified Rain Load

1) Except as provided in Sentence (4), the specified load, **S**, due to the accumulation of rainwater on a surface whose position, shape and deflection under load make such an accumulation possible, is that resulting from the one-day rainfall determined in conformance with Subsection 1.1.3. and applied over the horizontal projection of the surface and all tributary surfaces. (See Appendix A.)

2) The provisions of Sentence (1) apply whether or not the surface is provided with a means of drainage, such as rainwater leaders.

3) Except as provided in Sentence 4.1.6.2.(1), loads due to rain need not be considered to act simultaneously with loads due to snow. (See Appendix A.)

4) Where scuppers are provided and where the position, shape and deflection of the loaded surface make an accumulation of rainwater possible, the loads due to rain shall be the lesser of either the one-day rainfall determined in conformance with Subsection 1.1.3. or a depth of rainwater equal to 30 mm above the level of the scuppers, applied over the horizontal projection of the surface and tributary areas.

4.1.7. Wind Load

4.1.7.1. Specified Wind Load

1) The specified external pressure or suction due to wind on part or all of a surface of a *building* shall be calculated using the formula

$$p = I_W q C_e C_g C_p$$

where

- p = specified external pressure acting statically and in a direction normal to the surface, either as a pressure directed towards the surface or as a suction directed away from the surface,
- I_W = importance factor for wind load, as provided in Table 4.1.7.1.,
- q = reference velocity pressure, as provided in Sentence (4),
- C_e = exposure factor, as provided in Sentence (5),
- C_g = gust effect factor, as provided in Sentence (6), and
- C_p = external pressure coefficient, averaged over the area of the surface considered.

(See Appendix A.)

Table 4.1.7.1.
Importance Factor for Wind Load, I_W
Forming Part of Sentences 4.1.7.1.(1) and (3)

Importance Category	Importance Factor, I_W	
	ULS	SLS
Low	0.8	0.75
Normal	1	0.75
High	1.15	0.75
Post-disaster	1.25	0.75

2) The net wind load for the *building* as a whole shall be the algebraic difference of the loads on the windward and leeward surfaces, and in some cases, may be calculated as the sum of the products of the external pressures or suctions and the areas of the surfaces over which they are averaged as provided in Sentence (1). (See Appendix A.)

3) The net specified pressure due to wind on part or all of a surface of a *building* shall be the algebraic difference of the external pressure or suction as provided in Sentence (1) and the specified internal pressure or suction due to wind calculated using the following formula:

$$p_i = I_W q C_e C_{gi} C_{pi}$$

where
p_i = specified internal pressure acting statically and in a direction normal to the surface, either as a pressure directed towards the surface or as a suction directed away from the surface,
I_W = importance factor for wind load, as provided in Table 4.1.7.1.,
q = reference velocity pressure, as provided in Sentence (4),
C_e = exposure factor, as provided in Sentence (5),
C_{gi} = internal gust effect factor, as provided in Sentence (6), and
C_{pi} = internal pressure coefficient.
(See Appendix A.)

4) The reference velocity pressure, q, shall be the appropriate value determined in conformance with Subsection 1.1.3., based on a probability of being exceeded in any one year of 1 in 50.

5) The exposure factor, C_e, shall be
a) $(h/10)^{0.2}$ but not less than 0.9 for open terrain, where open terrain is level terrain with only scattered *buildings*, trees or other obstructions, open water or shorelines thereof, h being the reference height above *grade* in metres for the surface or part of the surface (see Appendix A),
b) $0.7(h/12)^{0.3}$ but not less than 0.7 for rough terrain, where rough terrain is suburban, urban or wooded terrain extending upwind from the *building* uninterrupted for at least 1 km or 10 times the *building height*, whichever is greater, h being the reference height above *grade* in metres for the surface or part of the surface (see Appendix A),
c) an intermediate value between the two exposures defined in Clauses (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 (see Appendix A), or
d) if a dynamic approach to the action of wind gusts is used, an appropriate value depending on both height and shielding (see Appendix A).

6) The gust effect factor, C_g, shall be one of the following values:
a) for the *building* as a whole and main structural members, $C_g = 2.0$ (see Appendix A),

b) for external pressures and suctions on small elements including cladding, $C_g = 2.5$,
c) for internal pressures, $C_{gi} = 2.0$ or a value determined by detailed calculation that takes into account the sizes of the openings in the *building* envelope, the internal volume and the flexibility of the *building* envelope (see Appendix A), or
d) if a dynamic approach to wind action is used, C_g is a value that is appropriate for the turbulence of the wind and the size and natural frequency of the structure (see Appendix A).

4.1.7.2. Dynamic Effects of Wind

1) *Buildings* whose height is greater than 4 times their minimum effective width, which is defined in Sentence (2), or greater than 120 m and other *buildings* whose light weight, low frequency and low damping properties make them susceptible to vibration shall be designed

a) by experimental methods for the danger of dynamic overloading, vibration and the effects of fatigue, or
b) by using a dynamic approach to the action of wind gusts (see Appendix A).

2) The effective width, w, of a *building* shall be calculated using

$$w = \frac{\sum h_i w_i}{\sum h_i}$$

where the summations are over the height of the *building* for a given wind direction, h_i is the height above *grade* to level i, as defined in Sentence 4.1.7.1.(5), and w_i is the width normal to the wind direction at height h_i; the minimum effective width is the lowest value of the effective width considering all possible wind directions.

4.1.7.3. Full and Partial Loading

1) *Buildings* 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 Clause (a) but with 100% of the load removed from any portion of the area,
c) the wind loads as described in Clause (a) but considered simultaneously at 75% of their full value, and
d) the wind loads as described in Clause (c) but with 50% of these loads removed from any portion of the area.

(See Appendix A.)

4.1.7.4. Interior Walls and Partitions

1) In the design of interior walls and *partitions*, due consideration shall be given to differences in air pressure on opposite sides of the wall or *partition* which may result from

a) pressure differences between the windward and leeward sides of a *building*,
b) stack effects due to a difference in air temperature between the exterior and interior of the *building*, and
c) air pressurization by the mechanical services of the *building*.

4.1.8. Earthquake Load and Effects

4.1.8.1. Analysis

1) The deflections and specified loading due to earthquake motions shall be determined according to the requirements in this Subsection, except that the requirements in this Subsection need not be considered in design if S(0.2), as defined in Sentence 4.1.8.4.(6), is less than or equal to 0.12.

Commentary I

Wind Load and Effects

Summary of Changes from the National Building Code of Canada 1995

Notable changes in the National Building Code of Canada 2005 (NBC):

- Introduction of an importance factor, I_w (see NBC Table 4.1.7.1.), in the expressions for calculating the wind pressures p and p_i
- Replacement of the three return periods of 10, 30 and 100 years by one of 50 years
- Provision of the exposure factor, C_e, for an added category of rough terrain under the Static Procedure
- Modification of the internal gust effect factor, C_{gi}, under the Static Procedure
- Addition of a new definition of the minimum effective width, w, given in NBC Sentence 4.1.7.2.(2)
- Requirement for a higher removal of wind load for partial loading stipulated in NBC Clauses 4.1.7.3.(1)(b) and (d)

Notable changes in this Commentary:

- Complete reorganization of the content to make the Commentary easier to use, as well as the addition of a flow chart (Figure I-1) to guide users to the Paragraphs and Figures that are applicable to the design job at hand
- Introduction of a transition formula given for C_e under Static Procedure for terrains that change from smooth to rough
- Revision of C_{gi} in accordance with NBC Clause 4.1.7.1.(6)(c) given in Paragraph 22, and of C_{pi} given in Paragraphs 30 to 34
- Correction of C_g for speed-up over hills and escarpments given in Paragraph 21 to take into account that only the mean wind speed is increased, not the gust wind speed
- Addition of new Table I-2 in Paragraph 29 to help users find the appropriate Figure in the Commentary for the coefficients C_pC_g, C_p, and C_p^*
- Replacement of the jump in wind load on the building structure from low-rise to high-rise building categories by the transition formulae in Figure I-15
- Change in the localized pressure coefficient, C_p^*, from –1.0 to ±0.9 in areas away from corners of the building, and to –1.2 near the corners
- Replacement of W and D by w and d, respectively, for the calculation of C_g under the Dynamic Procedure and of building vibration

usually take $C_g = 2.0$ and $C_{gi} = 2.0$ – main code 4.1.7.1.(6)

obtain C_pC_g from figure I-7

C_{pi} – from main code [illegible] commentary 31

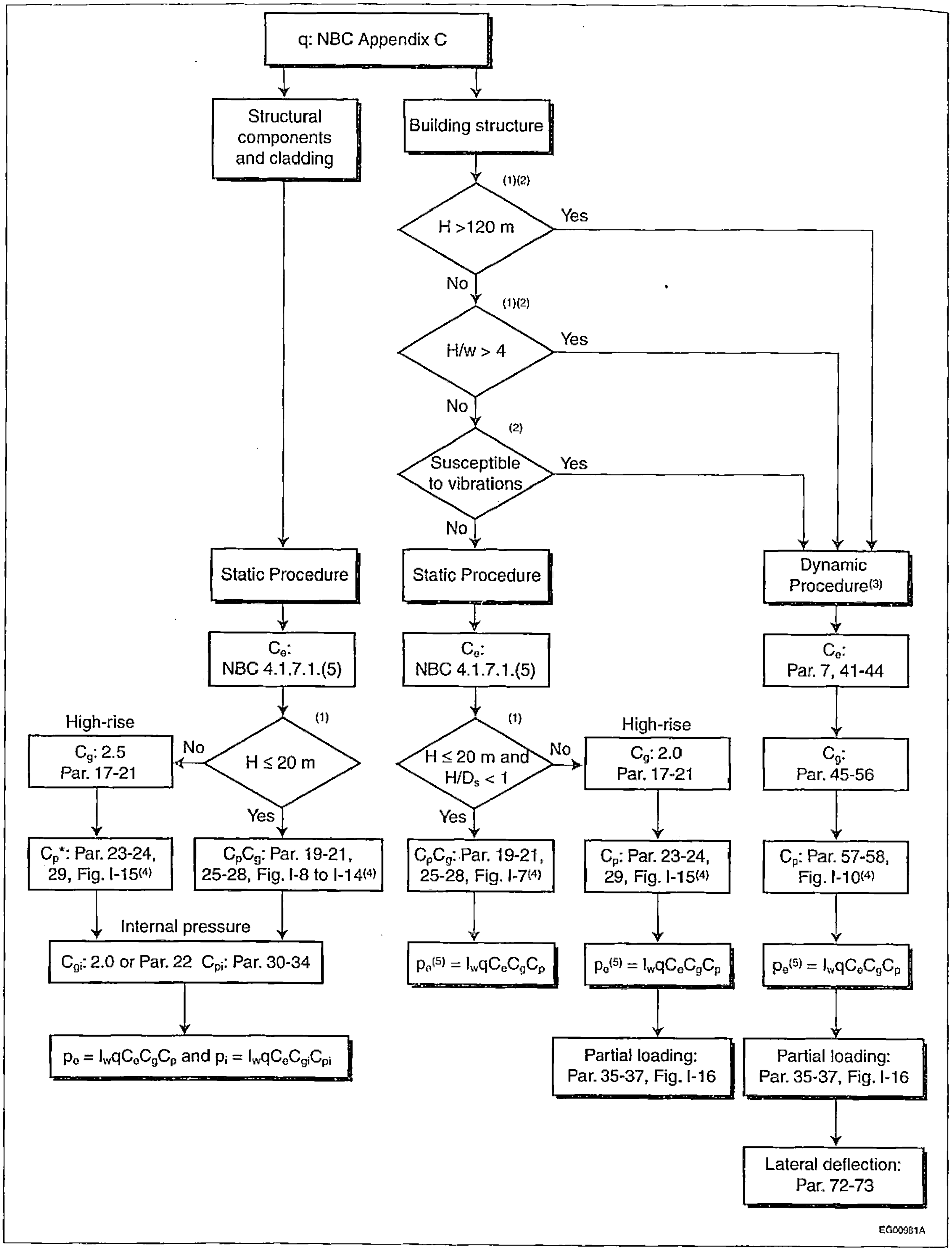


Figure I-1
Flow chart for calculating wind load and effects on buildings

Notes to Figure I-1:

(1) H is the height, D_s the smaller plan dimension, and w the effective width of the building as defined in Sentence 4.1.7.2.(2) of the National Building Code of Canada 2005 (NBC).
(2) See also NBC Sentence 4.1.7.2.(1).
(3) The Experimental Procedure is recommended for some cases—see Paragraph 4.
(4) For round buildings and spherical or curved roofs, see Figures I-24 to I-27.
(5) The internal pressure, p_i, should be considered where it could affect load on the building structure (e.g. roof uplift affecting axial load on columns).

Wind Load Calculation Procedure

1. Three different procedures of determining design wind load on buildings are indicated in NBC Subsection 4.1.7., Wind Load.[1]

2. The first procedure, called the Static Procedure, is appropriate for most cases, including the design of the structure of most low- and medium-rise buildings as well as the cladding of all buildings. The structure or element to be designed in these cases is relatively rigid. Detailed knowledge of the dynamic properties of these structures or elements is not required and dynamic actions of the wind can be represented by equivalent static loads.

3. The second procedure, called the Dynamic Procedure, is intended for determining the overall wind effects, including amplified resonant response, primarily for tall buildings and slender structures but not for cladding and secondary structural members.[5] Its format is the same as that of the Static Procedure except that the gust effect factor, C_g, and the exposure factor, C_e, are determined differently. C_g is derived from a series of calculations involving
   (a) the intensity of wind turbulence for the site as a function of height and of the surface roughness of the surrounding terrain, and
   (b) the properties of the building such as height, width, natural frequency of vibration and damping.
   When multiplied by the reference wind pressure, q, the importance factor, I_w, the exposure factor, C_e, and the pressure coefficient, C_p, this gust effect factor is expected to give a static design pressure that represents the same peak load effect as the dynamic resonant response to the actual turbulent wind. In addition to the calculation of wind load, the calculation of wind-induced lateral deflection and vibration can also be important for some buildings that are required to be treated by the Dynamic Procedure. These topics, as well as vortex shedding of rounded structures, are treated separately in this Commentary.

4. The third procedure, called the Experimental Procedure, consists of wind-tunnel testing or other experimental methods. It can be used as an alternative to the Static and Dynamic Procedures. It is especially recommended for buildings that may be subjected to buffeting or channeling effects caused by upwind obstructions, vortex shedding, or to aerodynamic instability. It is also suitable for determining external pressure coefficients for the design of cladding on buildings whose geometry deviates markedly from common shapes. Information on modern wind-tunnel techniques can be found in References [3], [4], [5] and [6].

5. The applicable exposure factors and some gust effect factors for the Static Procedure are specified in NBC Sentences 4.1.7.1.(5) and (6). The remaining gust effect factors and pressure coefficients for the Static Procedure, and all factors and coefficients for the Dynamic Procedure, are given in this Commentary. Figure I-1 shows the calculation procedure and provides references to applicable provisions in NBC Subsection 4.1.7. and this Commentary to help users determine wind load and effects for buildings.

Reference Wind Pressure

6. NBC Appendix C of Division B contains a description of the procedures followed in obtaining the reference wind pressures, q, based on mean hourly wind speed for the probability of being exceeded per year of 1 in 50, the values commonly referred to as having a return period of 50 years. These values of q are tabulated for many Canadian locations along with other climatic design data. Appendix C of the NBC and Equation (14) (see Paragraph 47) provide information on the conversion of reference wind pressure, q, to reference wind speed, $\bar{V}$, needed in Equation (13) (see Paragraph 47).

Reference Height

7. To calculate external pressure using both the Static and Dynamic Procedures, the reference height for calculating C_e is defined as follows:
   (a) For low-rise buildings, as defined in Paragraph 26, h is the mean height of the roof or 6 m, whichever is greater. The height of the eaves may by substituted for the mean height if the slope of the roof is less than 7°.
   (b) For taller buildings,
      (i) h for the windward face is the actual height of that point above ground,
      (ii) h for the leeward face is half the height of the building, and
      (iii) h for the roof and side walls is the height of the building.
   (c) For any structural element of the building, h is the height of the element above ground.

8. To calculate internal pressure, h for calculating C_e is defined as half the height of the building, exc[illegible] that when a large opening is present, h should be taken as the height of the opening from the groun[illegible]

Static Procedure

Application

9. The Static Procedure can be used to calculate the wind loads on all buildings except those with th[illegible] criteria defined in NBC Sentence 4.1.7.2.(1) and Figure I-1.

Exposure Factor, C_e

10. The exposure factor, C_e, reflects changes in wind speed with height, as well as the effects of variati[illegible] in the surrounding terrain and topography.

11. The value of C_e to be used for the Static Procedure is given in NBC Sentence 4.1.7.1.(5). It is based [illegible] the profile (variation with height) of wind-gust pressure on two types of surrounding terrain, ope[illegible] and rough, which are illustrated in Figures I-2 to I-5. For open terrain, the profile is assumed to [illegible] the 0.2 power law, which is equivalent to the 0.1 power law for wind-gust speeds. For rough ter[illegible] the 0.3 power law is assumed for the wind-gust pressure profile (equivalent to the 0.15 power la[illegible] for wind-gust speed). The wind gust referred to lasts about 3 to 5 s and represents a parcel of win[illegible] which is assumed to have an effect over the whole structure of most ordinary buildings.

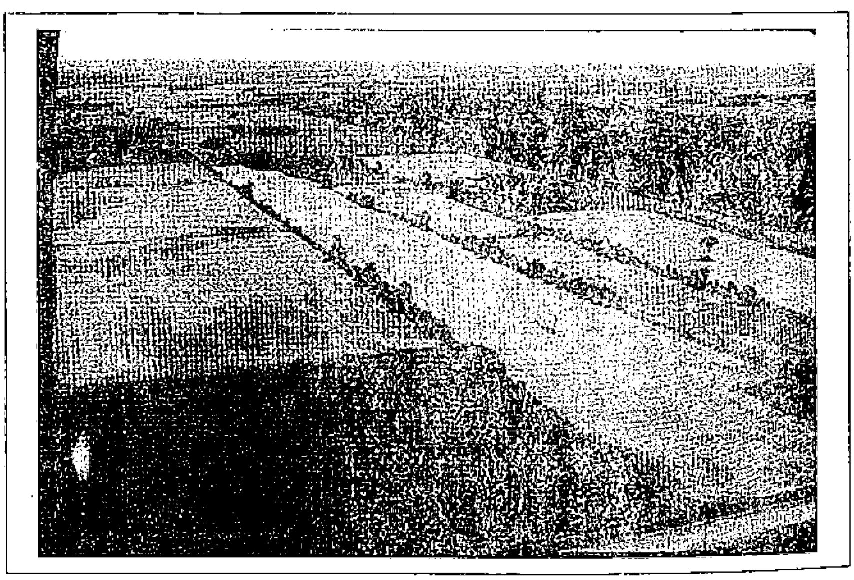


Figure I-2
Example of open terrain under the Static Procedure and of Exposure A under the Dynamic Procedure for determi[illegible] exposure factor, C_e. (See also Figure I-3.)

...3

...of open and rough terrains under the Static Procedure. Buildings located in the foreground near the road should ...ed for open-terrain exposure. Buildings that are located away from the road and deeper into the built-up area ... designed for either an intermediate exposure as given in Paragraph 12, or a rough-terrain exposure as given in ...h 11, depending on the distance from the road. (See also Figure I-4.)

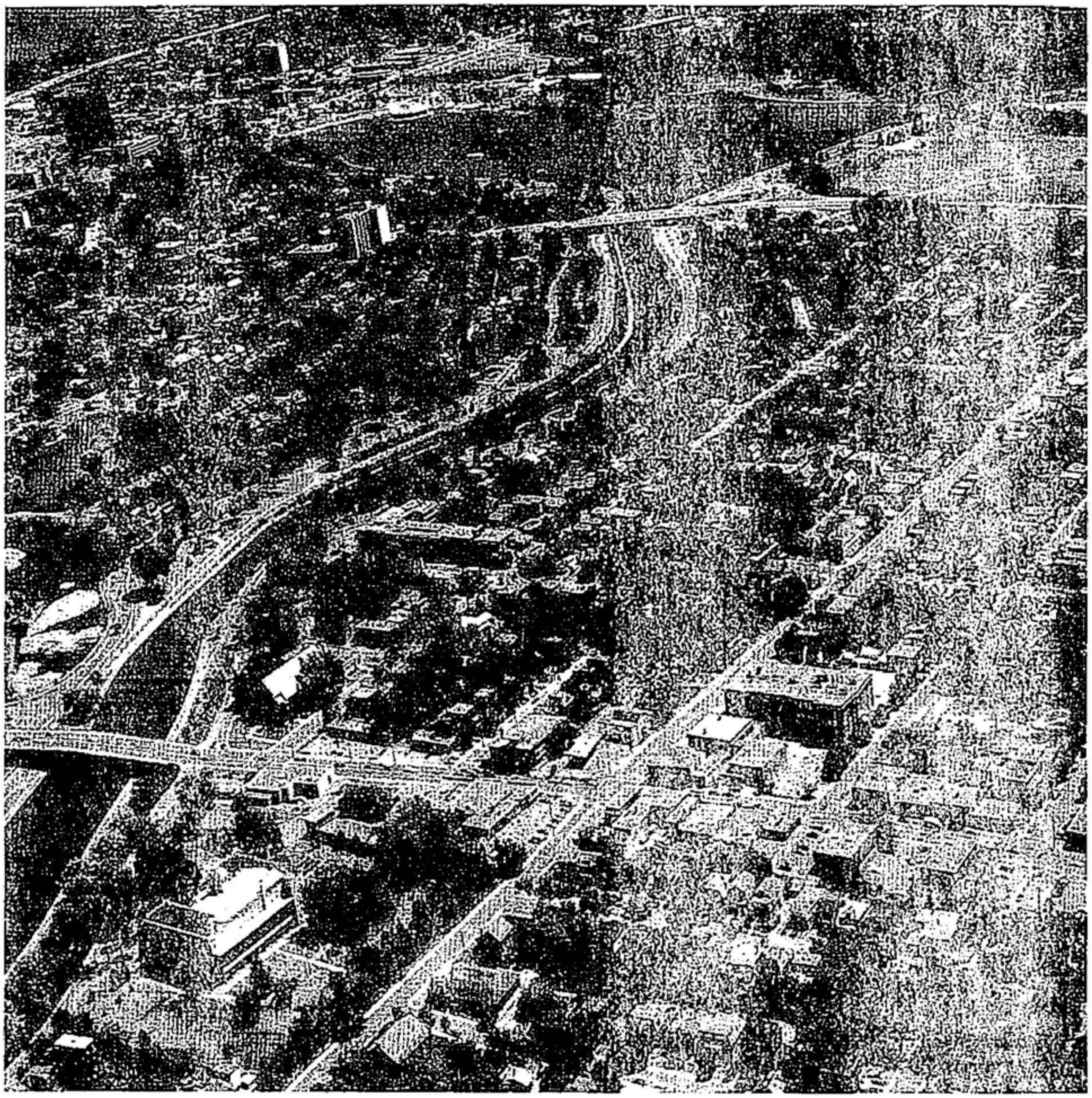


Figure I-4
Example of rough terrain under the Static Procedure and of Exposure B under the Dynamic Procedure. Buildings located on the periphery of the lake and open area in the background may be required to be designed for an open-terrain exposure.

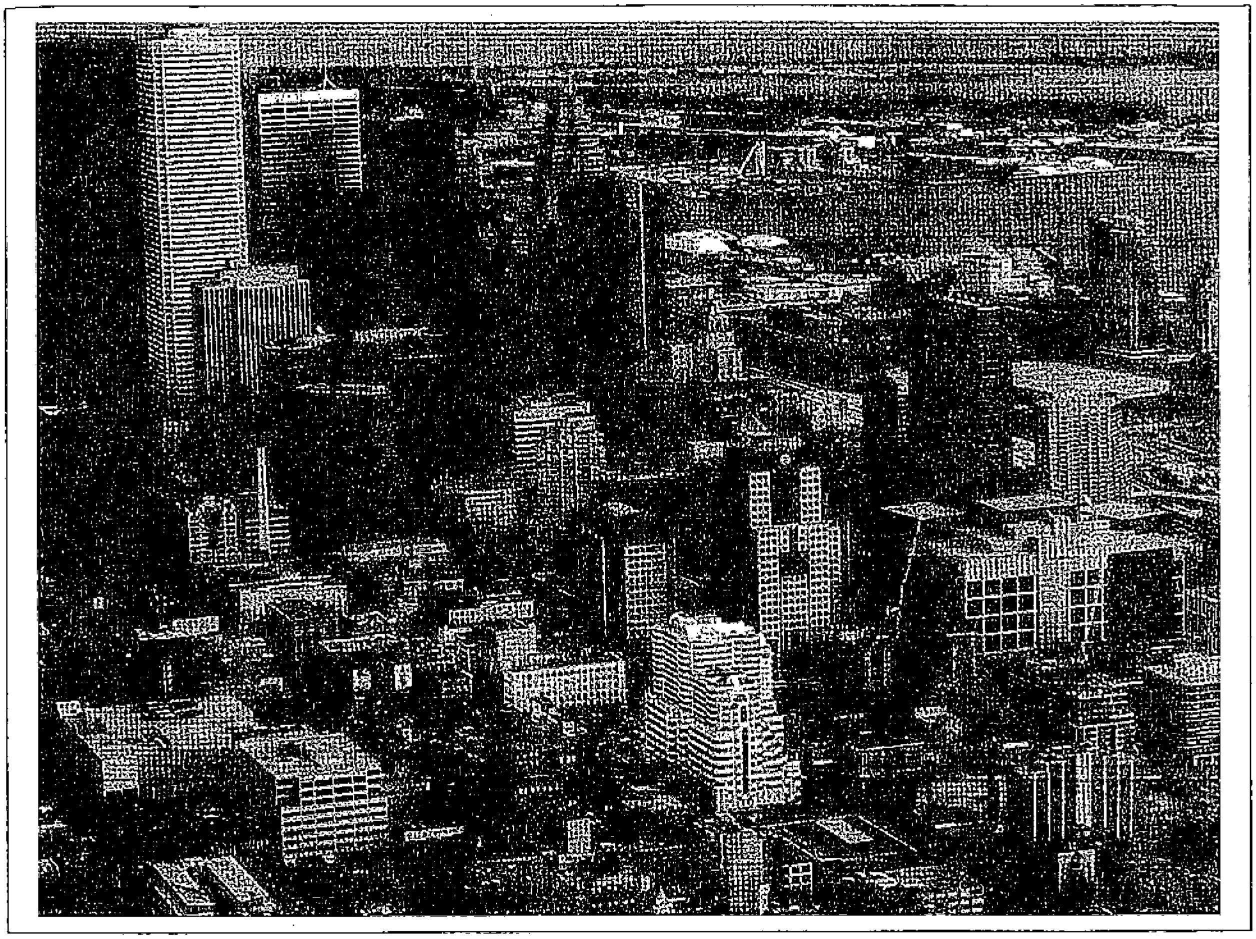


Figure I-5
Example of Exposure C under the Dynamic Procedure. Buildings located on the periphery of the lake in the right background may be required to be designed for Exposure A. In addition, tall buildings in the foreground may be required to be designed by experimental methods to account for channeling, buffeting and vortex-shedding effects.

Changes in Terrain

12. The value of C_e given in Paragraph 11 for rough terrain can be used when the rough terrain extends in the upwind direction for at least 1 km or 10 times the building height, H, whichever is greater. When the rough terrain extends for less than 1 km (i.e. $x < 1$ km) and the building is less than 100 m tall, the value of C_e may be interpolated between those for the open and the rough terrain using the following formulae:

for x_r greater than 0.05 km and less than 1 km,

$$C_e = C_{er}\left(0.816 + 0.184\log_{10}\left(\frac{10}{x_r - 0.05}\right)\right) \leq C_{eo} \qquad (1)$$

and for x_r less than or equal to 0.05 km,

$$C_e = C_{eo} \qquad (2)$$

where x_r is the upwind extent of rough terrain, C_{er} is the C_e for rough terrain, and C_{eo} is the C_e for open terrain.

Speed-up over Hills and Escarpments

13. Hills and escarpments can significantly amplify wind speeds near the ground and this should be reflected in the exposure factor for buildings located on a hill or escarpment. A method that can be used with both the Static and Dynamic Procedures to reflect this amplification is presented below.

14. Buildings on a hill or escarpment with a maximum slope greater than 1 in 10, particularly near a crest, may be subject to significantly higher wind speeds than buildings on level ground. The exposure factor at height z above the surrounding ground elevation is then equal to that over open level terrain multiplied by a factor $(1 + \Delta S(z))^2$, where $\Delta S(z)$ is the "speed-up factor" for the mean wind speed (this effect is illustrated in Figure I-6). Near the crest, and within a distance $|x| < kL$, the exposure factor is modified as follows:

$$C_e^* = C_e \left\{ 1 + \Delta S_{max} \left(1 - \frac{|x|}{kL} \right) e^{(-\alpha z/L)} \right\}^2 \tag{3}$$

where
C_e^* = corresponding modified value for use on the hill or escarpment,
C_e = exposure factor over open level terrain given in Paragraphs 11 and 12 for the Static Procedure, and in Paragraph 41 for the Dynamic Procedure,
ΔS_{max} = relative speed-up factor at the crest near the surface, and
α = decay coefficient for the decrease in speed-up with height.

The values of α and ΔS_{max} depend on the shape and steepness of the hill or escarpment. Representative values for these parameters are given in Table I-1.

Table I-1
Parameters for Maximum Speed-up Over Hills and Escarpments

Shape of Hill or Escarpment	ΔS_{max}[1]	α	k	
			$x < 0$	$x > 0$
2-dimensional ridges (or valleys with negative H)	2.2 H_h/L_h	3	1.5	1.5
2-dimensional escarpments	1.3 H_h/L_h	2.5	1.5	4
3-dimensional axi-symmetrical hills	1.6 H_h/L_h	4	1.5	1.5

(1) For $H_h/L_h > 0.5$, assume that $H_h/L_h = 0.5$ and substitute $2H_h$ for L_h in Equation (3).

15. The definitions of H_h, height, and L_h, length, shown in Figure I-6 are as follows: H_h is the height of the hill or or escarpment, or the difference in elevation between the crest and that of the terrain surrounding the hill or escarpment upwind; L_h is the distance upwind of the crest to where the ground elevation is half of H_h. The maximum slope for rounded hill shapes is roughly $H_h/(2L_h)$. In the expressions above, it is assumed that the wind approaches the hill along the direction of maximum slope, i.e. the direction giving the greatest speed-up near the crest.

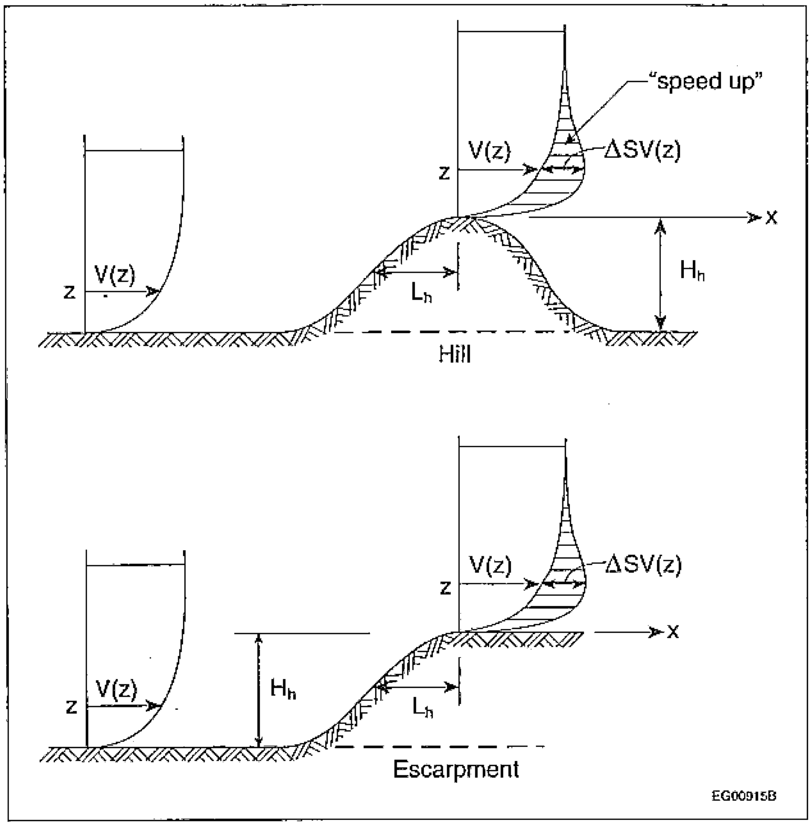


Figure I-6
Definitions for wind speed-up over hills and escarpments

16. Hills and escarpments with slopes less than 1 in 10 are unlikely to produce significant speed-up of the wind. A more detailed discussion of this issue and other simplified models for three-dimensional hills are given in Reference [7]. Background material may be found in References [8] and [9]. Wind tunnel tests and computational methods may be used to obtain design information in other cases.

Gust Effect Factors, C_g and C_{gi}

General

17. In this section, procedures are recommended for determining the external gust effect factor referred to in NBC Sentence 4.1.7.1.(1) and the internal gust effect factor referred to in NBC Sentence 4.1.7.1.(3). These two factors, denoted by C_g and C_{gi} respectively, are defined as the ratio of the maximum effect of the loading to the mean effect of the loading. They take into account:
    (a) random fluctuating wind forces caused by turbulence in the approaching wind and acting for short durations over all or part of the structure,
    (b) fluctuating forces induced by the wake of the structure itself,
    (c) additional inertial forces arising from motion of the structure itself as it responds to the fluctuating wind forces, and
    (d) additional aerodynamic forces due to alterations in the airflow around the structure caused by its motions (aero-elastic effects).

18. All structures are affected to some degree by these forces. The total response may be considered as a superposition of a "background component," which acts quasi-statically, and a "resonant component," which is due to inertial forces arising from excitation close to a natural frequency. For the majority of structures, the resonant component is small and the dynamic effect can be treated by considering only the background component using normal static methods. These structures are amenable to the Static Procedure. For structures that are particularly tall, long, slender, lightweight, flexible or lightly damped, the resonant component may be dominant: these structures should be treated by the Dynamic Procedure.

External Gust Effect Factor, C_g

19. The values of the external gust effect factor, C_g, for small and low-rise structures, or structures and components having a relatively high rigidity, are given in NBC Clauses 4.1.7.1.(6)(a) and (b).

20. The peak pressure coefficients of certain low-rise structures can be determined directly from wind-tunnel tests. These coefficients are composite values of C_pC_g, incorporating the gust effect in addition to aerodynamic shape factors, and are given in Paragraphs 25 to 28 dealing with pressure coefficients. Therefore, a gust effect factor should not be used in conjunction with these coefficients.

Correction of C_g for Speed-up over Hills and Escarpments

21. Speed-up over hills and escarpments principally affects the mean wind speed and not the amplitude of the turbulent fluctuations. This means that a correction should be applied to the gust effect factor for both the Static and Dynamic Procedures to compensate for the associated increase in gust amplitude when the corrected exposure factor, C_e^*, determined with Equation (3) is used. The following expression gives the corrected gust effect factor to be used for designing structures located on hills and escarpments:

$$C_g^* = 1 + (C_g - 1)\sqrt{\frac{C_e}{C_e^*}} \tag{4}$$

where
C_g^* = the corrected factor for hills and escarpments, and
C_g = the gust effect factor for flat terrain.

When a combined C_pC_g value is used, the combined value can be adjusted for hills and escarpments by multiplying it by the ratio C_g^*/C_g, which is calculated using Equation (4) with a value of $C_g = 2.0$ for the building structure and $C_g = 2.5$ for cladding and secondary structural members.

Internal Gust Effect Factor, C_{gi}

22. As stipulated in NBC Clause 4.1.7.1.(6)(c), the default value of the internal gust effect factor, C_{gi}, should be taken as 2.0. However, for large structures enclosing a single unpartitioned volume, the internal pressure takes significant time to respond to changes in external pressure, thus reducing the gust factor. In such cases, the following expression for C_{gi} may be used in lieu of the default value:

$$C_{gi} = 1 + \frac{1}{\sqrt{1+\tau}} \tag{5}$$

where τ is a parameter associated with the time it takes for the internal pressure to respond to changes in external pressure at openings, and τ is given by

$$\tau = \frac{V_0}{6950A}\left[1 + 1.42 \times 10^5 \frac{A_s}{V_0}\delta\right] \tag{6}$$

where
V_0 = internal volume, in m^3,
A = total area of all exterior openings of the volume, in m^2,
A_s = total interior surface area of the volume (excluding slabs on grade), in m^2, and
δ = a measure of the flexibility of the building envelope and is the average outward deflection of the volumes envelope per unit increase in internal pressure, in m^3/N.

A typical value of δ for buildings with sheet metal cladding is about $5 \times 10^{-5}\ m^3/N$. Where δ is difficult to estimate, it may conservatively be taken as zero.

Example: Suppose a building's plan dimensions are 100 m × 50 m and it is 20 m high. It contains a single undivided volume, has a single opening of 5 m², and $\delta = 5\times10^{-5}$ m3/N. Then $V_0 = 100{,}000$ m³, $A = 5$ m², and $A_s = 6000 + 5000 = 11{,}000$ m². Hence

$$\begin{aligned}\tau &= \frac{10^5}{6950 \times 5}\left[1 + 1.42 \times 10^5 \frac{1.1 \times 10^4}{10^5} 5 \times 10^{-5}\right]\\ &= 2.88\,[1 + 0.78]\\ &= 5.1\end{aligned}$$

and

$$\begin{aligned}C_{gi} &= 1 + \frac{1}{\sqrt{1 + 5.1}}\\ &= 1.40\end{aligned}$$

Pressure Coefficients, C_p, C_p^* and C_{pi}

General

23. Pressure coefficients are the non-dimensional ratios of actual wind-induced pressures on a building surface to the velocity pressure of the wind at the reference height. They account for the effects of aerodynamic shape of the building, orientation of the surface with respect to the wind flow, and profile of the wind velocity. Pressure coefficients are usually determined from wind-tunnel experiments on small-scale models, although measurements are occasionally made on full-scale buildings. It is very important to simulate the natural velocity profile and turbulence in the wind tunnel; experiments in uniform flow can be highly misleading.[10][11]

Directionality

24. At any geographical location, winds are the strongest in certain geographical directions. The probability is less than 100% that the direction of the strongest wind will align with the direction that produces the highest pressure on a given surface. Therefore, the actual wind load on a given surface will be less than that computed by combining the reference wind velocity pressure for the location with the peak pressure coefficient for the surface. Directionality effects have been accounted for in the factored loads, and no further reduction should be made to them.

External Pressure Coefficients for Low-Rise Buildings

$H \leq 20$ - low-rise building

25. Recommended external pressure coefficients for designing low-rise buildings are given in Figures I-7 to I-14. They are based on data obtained from systematic boundary-layer wind-tunnel studies. In several instances, these data have been verified against full-scale measurements. The coefficients are based on the maximum gust pressures lasting approximately 1 s and, consequently, include an allowance for the gust effect factor, C_g; they therefore represent the product C_pC_g. These coefficients apply to the tributary area associated with the particular element or member over which the wind pressure is assumed to act.

26. The external pressure-gust coefficients given in Figures I-7 to I-14 are most appropriate for buildings with height-to-width ratios of less than 0.5 and a reference height of less than 20 m, where the width is based on the smaller plan dimension, D_s. In the absence of more case-specific data, these Figures may also be used for buildings with height-to-width ratios of less than 1.0, provided that the reference height is less than 20 m. Beyond these limits, Figure I-15 should be used. These coefficients are based on References [12] and [13].

27. Figure I-7 presents values of C_pC_g for the main wind force resisting system of the building affected by wind pressures on more than one surface, such as in frame buildings. The simplified load distributions in Figure I-7 were developed to represent as closely as possible the structural actions (horizontal thrust, uplift and frame moments) determined directly from experiment. These results make allowance for the partial loading of gusts referred to in NBC Sentence 4.1.7.3.(1).

28. Figures I-8 to I-14 are intended for those effects that are influenced mainly by wind acting over single surfaces, such as the design of cladding and secondary structural members such as purlin[illegible] and girts. They should also be used for the design of structural elements with single surfaces, su[illegible] as roofs for which moment connections are not provided at the roof/wall intersection. In this case the edge region loads need not be included around the entire perimeter of the roof, but only adja[illegible] to the windward edges. For roof slopes exceeding 7° where edge regions are also specified along t[illegible] ridge, these increased loads need only be included on the downstream side. The loads on other e[illegible] regions can be reverted to the values specified for the interior regions.

External Pressure Coefficients for High-Rise Buildings

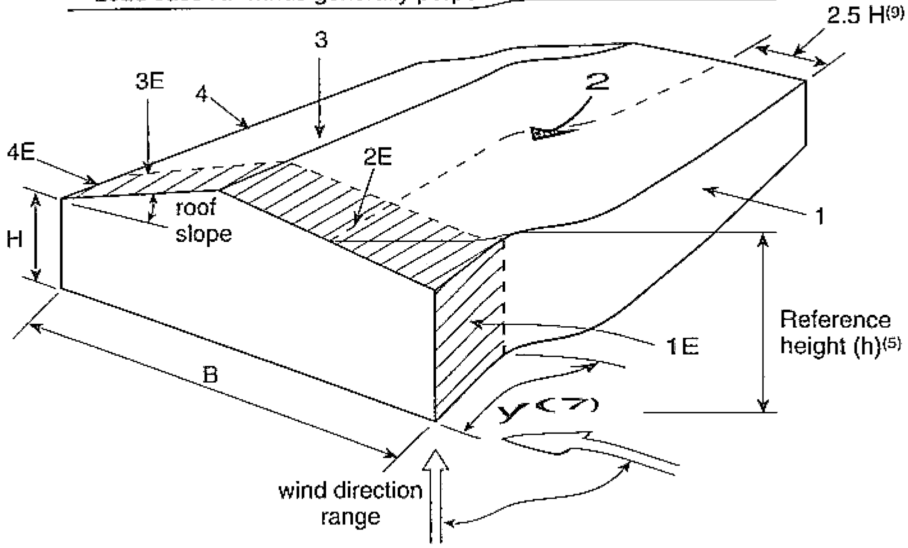
29. Figure I-15 contains the external pressure coefficients to be used for buildings that are rectangula[illegible] plan and whose height, H, is greater than 20 m or their smaller plan dimension, D_s. The coefficie[illegible] are given as either time- and spatially-averaged pressure coefficients, C_p, or simply as time-averag[illegible] local pressure coefficients, C_p^*. A local pressure coefficient, $C_p^* = \pm 0.9$, applicable to the design of small cladding areas (about the size of a window), can occur almost anywhere at any elevation, except near corners where a local C_p^* of 1.2 is appropriate.

Table I-2 indicates which Figure to consult for deriving pressure coefficients.

Table I-2
Index of Figures Containing External Pressure Coefficients

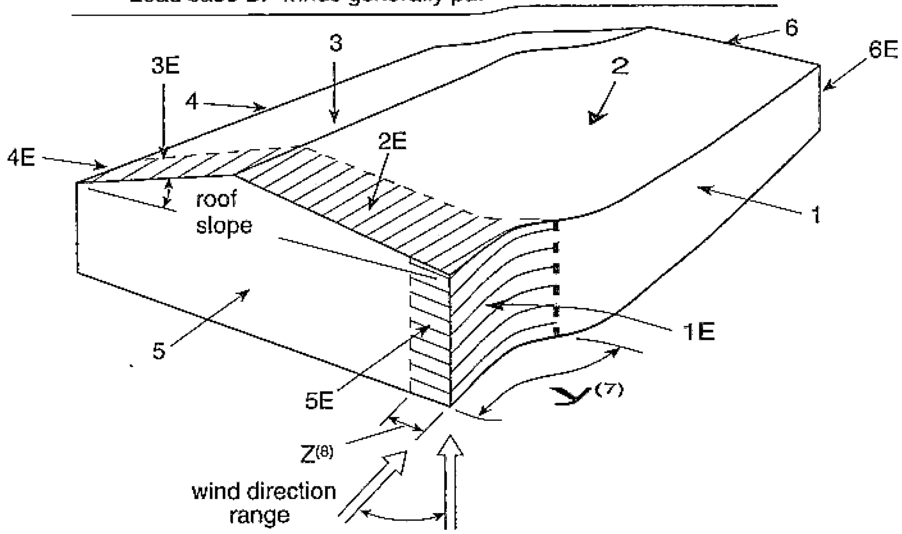
Building Type	Structural Element	Roof slope (α) Limit	Figure Number	[illegible]
Low-rise buildings where $H/D_s < 1$ and $H \leq 20$ m	Primary structural action	—	I-7	
	Walls	—	I-8	
	Roofs			
	(a) general	$\alpha \leq 7°$	I-9	
	(b) stepped flat	$\alpha = 0°$	I-10	
	(c) gabled and hipped, single-ridge	$\alpha \leq 7°$	I-9	
		$\alpha > 7°$	I-11	
	(d) gabled, multiple-ridge	$\alpha \leq 10°$	I-9	
		$\alpha > 10°$	I-12	
	(e) monosloped	$\alpha \leq 3°$	I-9	
		$30° \geq \alpha > 3°$	I-13	
	(f) sawtoothed	$\alpha \leq 10°$	I-9	
		$\alpha > 10°$	I-14	
Buildings where $H/D_s \geq 1$ or $H > 20$ m	—	—	I-15	C_p [illegible]

Load case A: winds generally perpendicular to ridge



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 ridge



of slope	Building surfaces											
	1	1E	2	2E	3	3E	4	4E	5	5E	6	6E
to 90°	−0.85	−0.9	−1.3	−2.0	−0.7	−1.0	−0.85	−0.9	0.75	1.15	−0.55	−0.8

0° to 90°

EG00920A

I-7

al peak composite pressure-gust coefficients, C_pC_g, for primary structural actions arising from wind load acting neously on all surfaces

external

Notes to Figure I-7:

(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, Load Case A and Load Case B are required as two separate loading conditions to generate the wind actions, including torsion, to be resisted by the structural system.

(2) For values of roof slope not shown, the coefficient C_pC_g may be interpolated linearly.

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

(4) For the design of foundations, exclusive of anchorages to the frame, only 70% of the effective load is to be considered.

(5) The reference height, h, for pressures is mid-height of the roof or 6 m, whichever is greater. The eaves height, H, may be substituted for the mean height if the slope of the roof is less than 7°.

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

(7) End-zone width z is the lesser of 10% of the least horizontal dimension or 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.

(8) For B/H > 5 in Load Case A, the listed negative coefficients on surfaces 2 and 2E should only be applied on an area that is 2.5 H wide measured from the windward eaves. The pressures on the remainder of the windward roof should be reduced to the coefficients specified for the leeward roof (i.e. those for 3 and 3E).

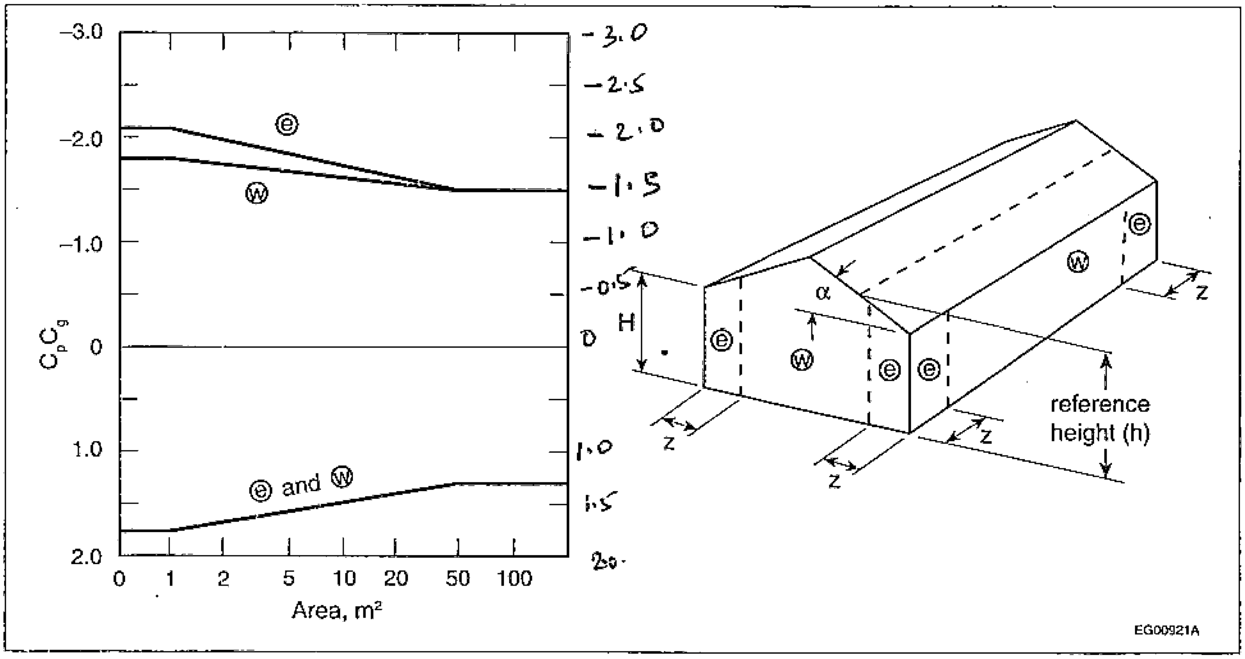


Figure I-8

External peak composite pressure-gust coefficients, C_pC_g, on individual walls for the design of structural components and cladding

Notes to Figure I-8:

(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) End-zone width z is the lesser of 10% of the least horizontal dimension and 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.

(4) Combinations of exterior and interior pressures must be evaluated to obtain the most severe loading.

(5) Positive coefficients denote forces toward the surface, whereas negative coefficients denote forces away from the surface. Each structural element must be designed to withstand the forces of both signs.

(6) Pressure coefficients may generally apply for facades with architectural features; however, when vertical ribs deeper than 1 m are placed on a facade, a local C_pC_g = −2.8 may apply to zone e.[35][36]

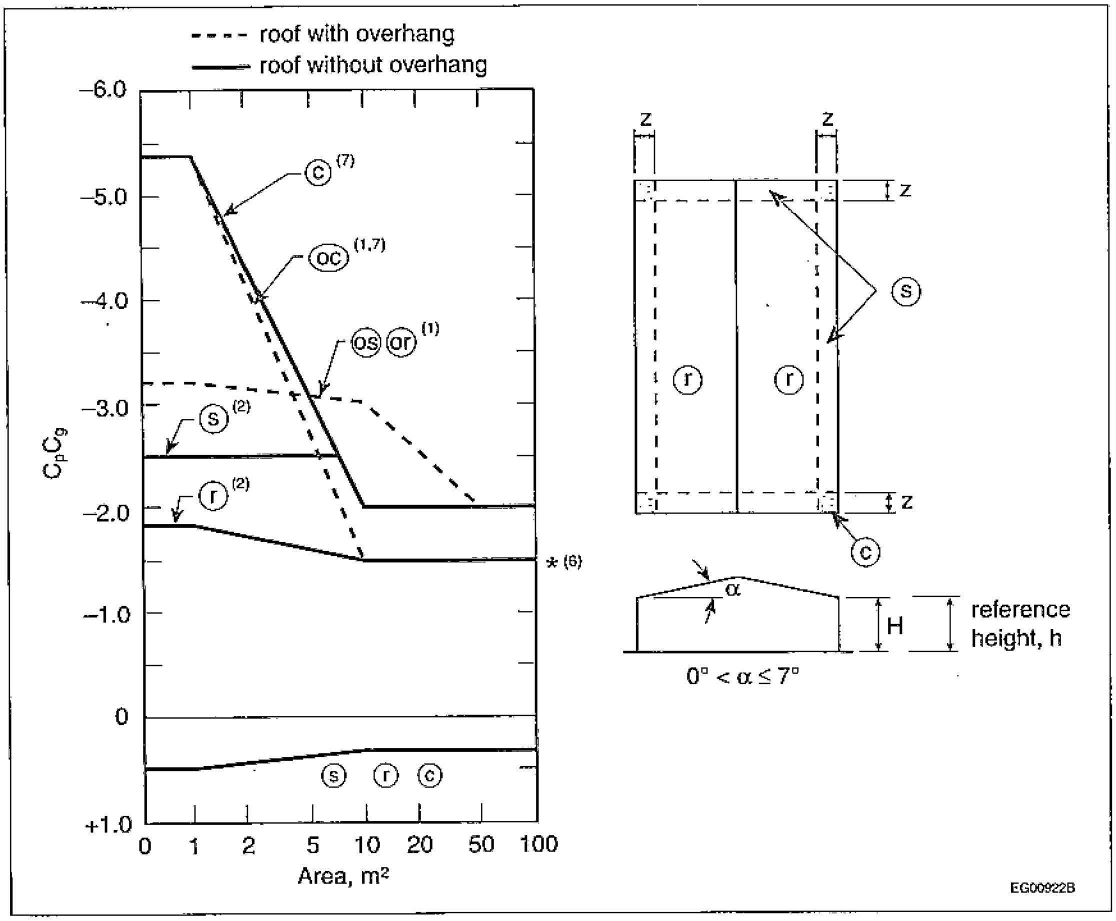


Figure I-9
External peak composite pressure-gust coefficients, C_pC_g, on roofs with a slope of 7° or less for the design of structural components and cladding

Notes to Figure I-9:

(1) Coefficients for overhung roofs have the prefix "o" and refer to the same roof areas as referred to by the corresponding symbol without a prefix. They include contributions from both upper and lower surfaces. In the case of overhangs, the walls are inboard of the roof outline.[37]

(2) s and r apply to both roofs and upper surfaces of canopies.

(3) The abscissa area in the graph is the design tributary area within the specified zone.

(4) End-zone width z is the lesser of 10% of the least horizontal dimension and 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.

(5) Combinations of exterior and interior pressures must be evaluated to obtain the most severe loading.

(6) Positive coefficients denote forces toward the surface, whereas negative coefficients denote forces away from the surface. Each structural element must be designed to withstand the forces of both signs.

(7) For calculating the uplift forces on tributary areas larger than 100 m² on unobstructed nearly-flat roofs with low parapets, and where the centre of the tributary area is at least two building heights from the nearest edge, the value of C_pC_g may be reduced to −1.1 at x/H = 2 and further reduced linearly to −0.6 at x/H= 5, where x is distance to the nearest edge and H is building height.[39]

(8) For roofs having a perimeter parapet that is 1 m high or greater, the corner coefficients C_pC_g for small tributary areas can be reduced from −5.4 to −4.4.[40][41]

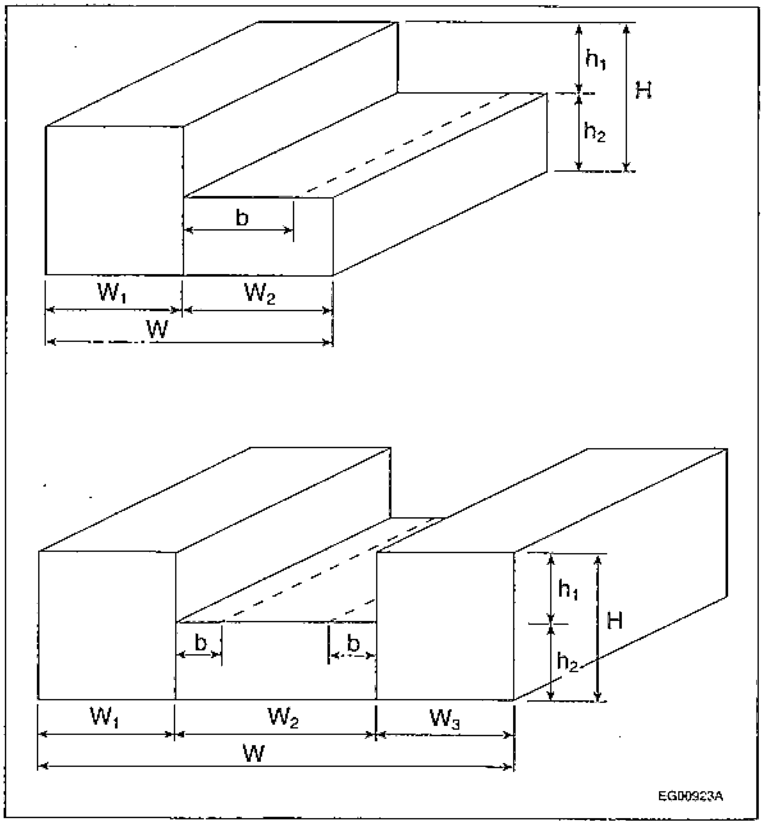


Figure I-10

External peak composite pressure-gust coefficients, C_pC_g, for the design of the structural components and cladding of buildings with stepped roofs

Notes to Figure I-10:

(1) The zone designations, pressure-gust coefficients and notes provided in Figure I-9 apply on both the upper and lower levels of flat stepped roofs, except that on the lower levels, positive pressure-gust coefficients equal to those in Figure I-8 for walls apply for a distance, b, where b is equal to $1.5h_1$ but not greater than 30 m. For all walls in Figure I-10, zone designations and pressure coefficients provided for walls in Figure I-8 apply.[42][43]

(2) Note (1) above applies only when the following conditions are met: $h_1 \geq 0.3H$, $h_1 \geq 3$ m, and W_1, W_2, or W_3 is greater than 0.25W but not greater than 0.75W

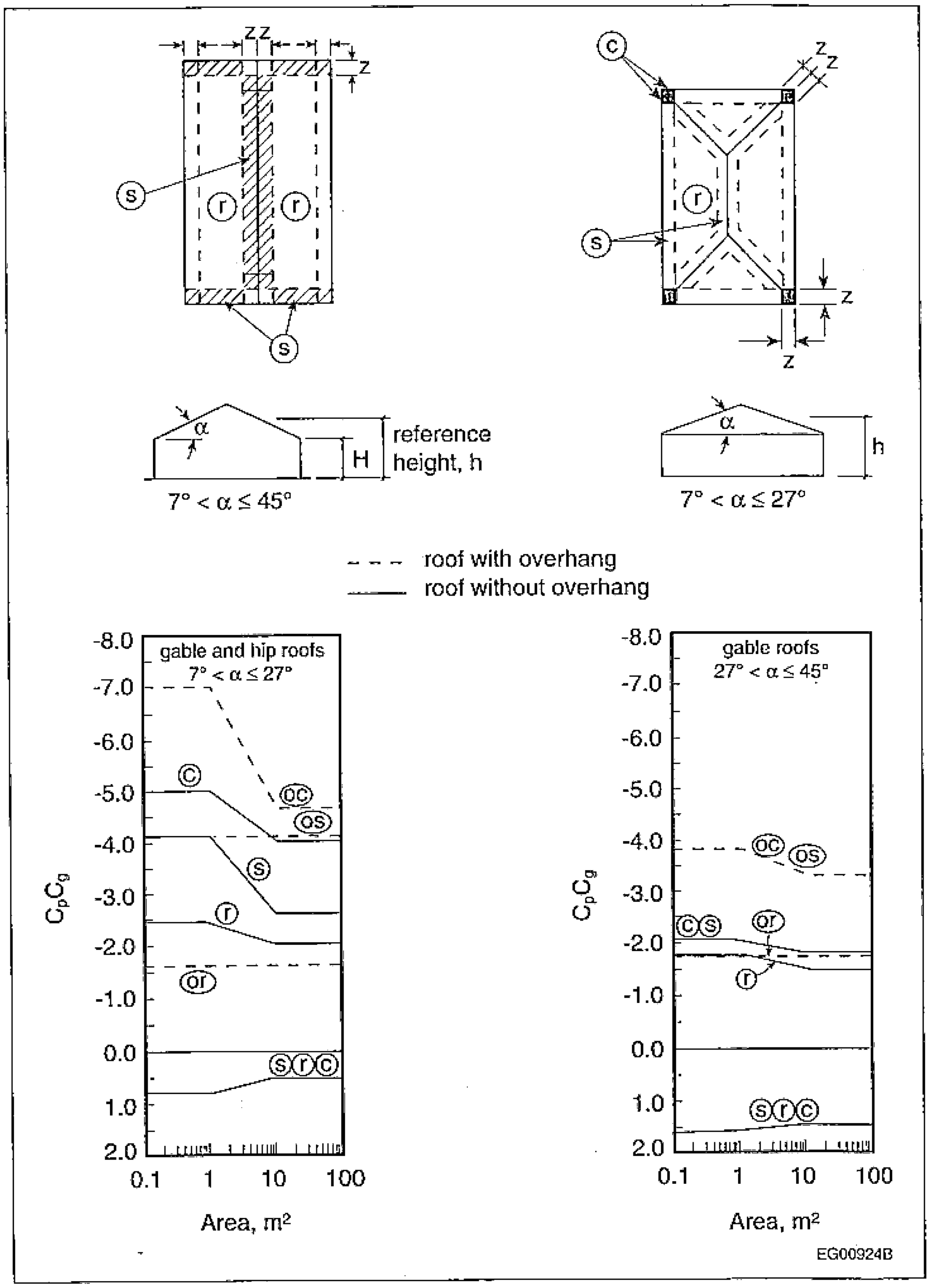


Figure I-11

External peak composite pressure-gust coefficients, C_pC_g, on single-span gabled and hipped roofs with a slope of 7° or greater for the design of structural components and cladding

Notes to Figure I-11:

(1) Coefficients for overhung roofs have the prefix "o" and refer to the same roof areas as referred to by the corresponding symbol without a prefix. They include contributions from both upper and lower surfaces.[24][44]

(2) The abscissa area in the graph is the design tributary area within the specified zone.

(3) End-zone width z is the lesser of 10% of the least horizontal dimension and 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.

(4) Combinations of exterior and interior pressures must be evaluated to obtain the most severe loading.

(5) Positive coefficients denote forces toward the surface, whereas negative coefficients denote forces away from the surface. Each structural element must be designed to withstand the forces of both signs.

(6) For hipped roofs with 7° < α ≤ 27°, edge/ridge strips and pressure-gust coefficients for ridges of gabled roofs apply along each hip.[45]

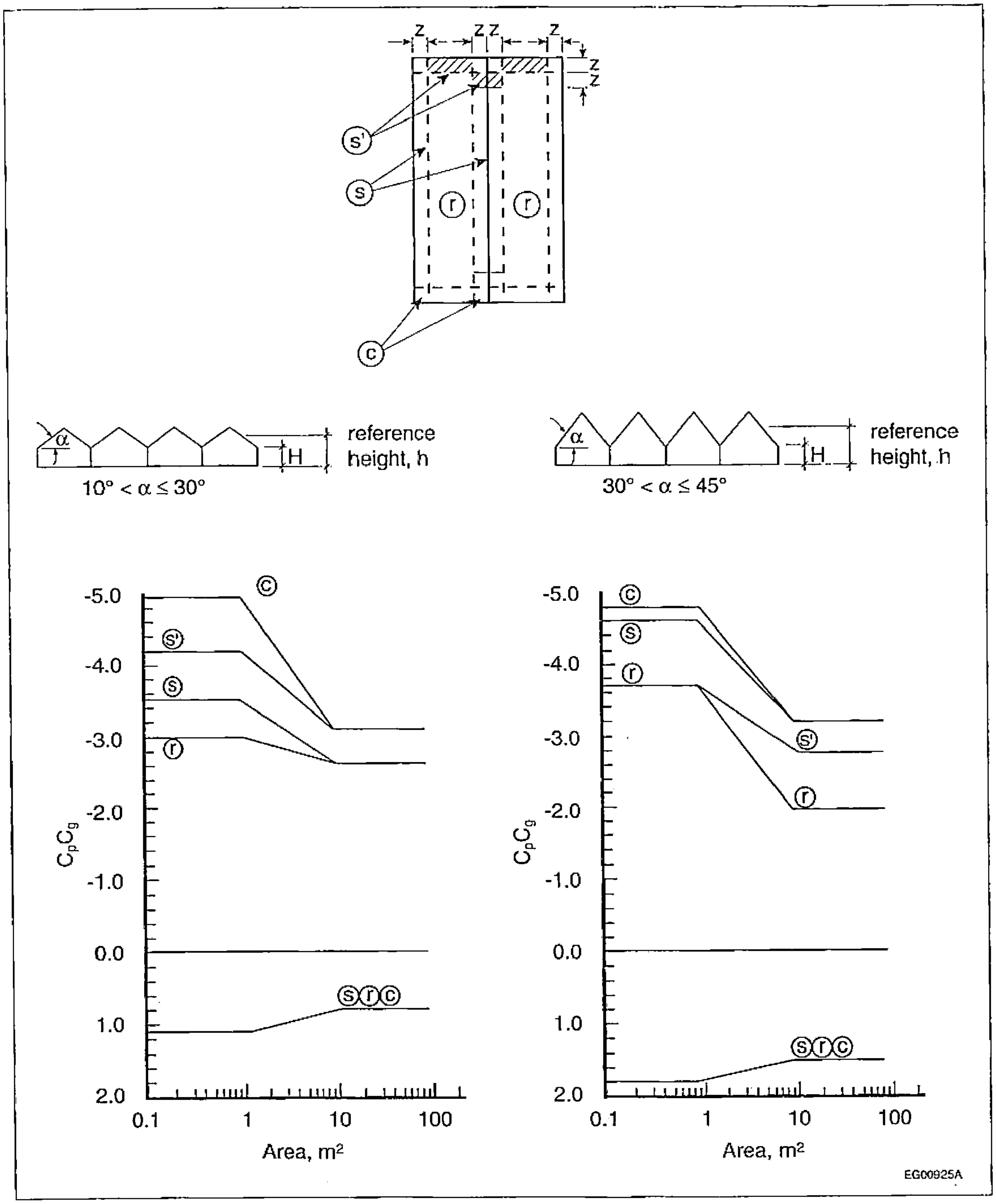


Figure I-12

External peak composite pressure-gust coefficients, C_pC_g, on multi-span gabled (folded) roofs with a slope greater than 10° for the design of structural components and cladding[46][47]

Notes to Figure I-12:

(1) The abscissa area in the graph is the design tributary area within the specified zone.

(2) End-zone width z is the lesser of 10% of the least horizontal dimension and 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.

(3) Combinations of exterior and interior pressures must be evaluated to obtain the most severe loading.

(4) Positive coefficients denote forces toward the surface, whereas negative coefficients denote forces away from the surface. Each structural element must be designed to withstand the forces of both signs.

(5) For $\alpha \le 10°$, the coefficients given in Figure I-9 apply.

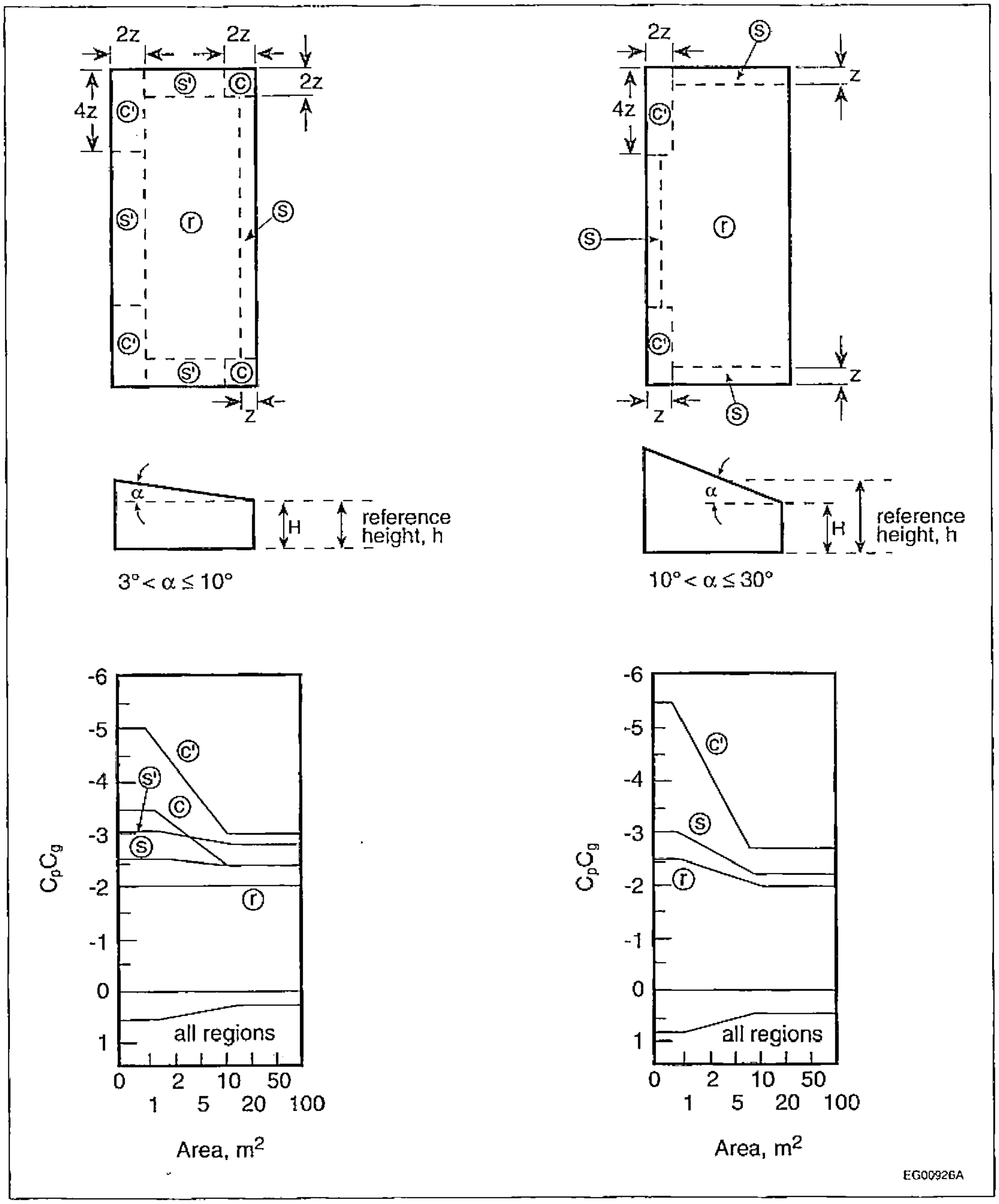


Figure I-13

External peak composite pressure-gust coefficients, C_pC_g, on monosloped roofs for the design of structural components and cladding[48][49]

Notes to Figure I-13:

(1) The abscissa area in the graph is the design tributary area within the specified zone.

(2) End-zone width z is the lesser of 10% of the least horizontal dimension and 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.

(3) Combinations of exterior and interior pressures must be evaluated to obtain the most severe loading.

(4) Positive coefficients denote forces toward the surface, whereas negative coefficients denote forces away from the surface. Each structural element must be designed to withstand the forces of both signs.

(5) For $\alpha \leq 3°$, pressure-gust coefficients given in Figure I-9 apply.

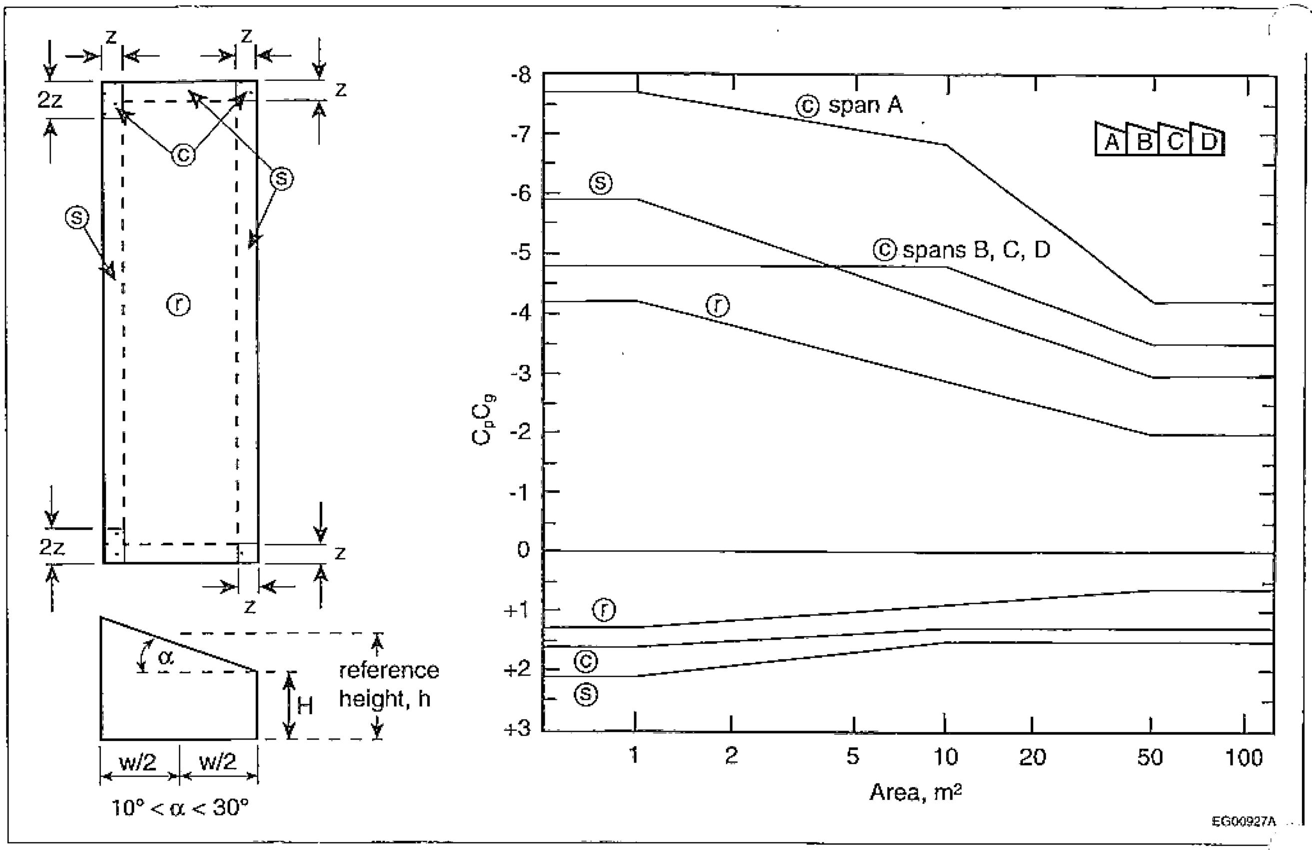


Figure I-14

External peak composite pressure-gust coefficients, C_pC_g, on sawtooth roofs with a slope greater than 10° for the design of roofing and secondary structural members[49]

Notes to Figure I-14:

(1) The abscissa area in the graph is the design tributary area within the specified zone.

(2) End-zone width z is the lesser of 10% of the least horizontal dimension and 40% of height, H, but not less than 4% of the least horizontal dimension or 1 m.

(3) Combinations of exterior and interior pressures must be evaluated to obtain the most severe loading.

(4) Positive coefficients denote forces toward the surface, whereas negative coefficients denote forces away from the surface. Each structural element must be designed to withstand the forces of both signs.

(5) Negative coefficients on the corner zones of Span A differ from those on Spans B, C and D.

(6) For $\alpha \leq 10°$, pressure-gust coefficients given in Figure I-9 apply.

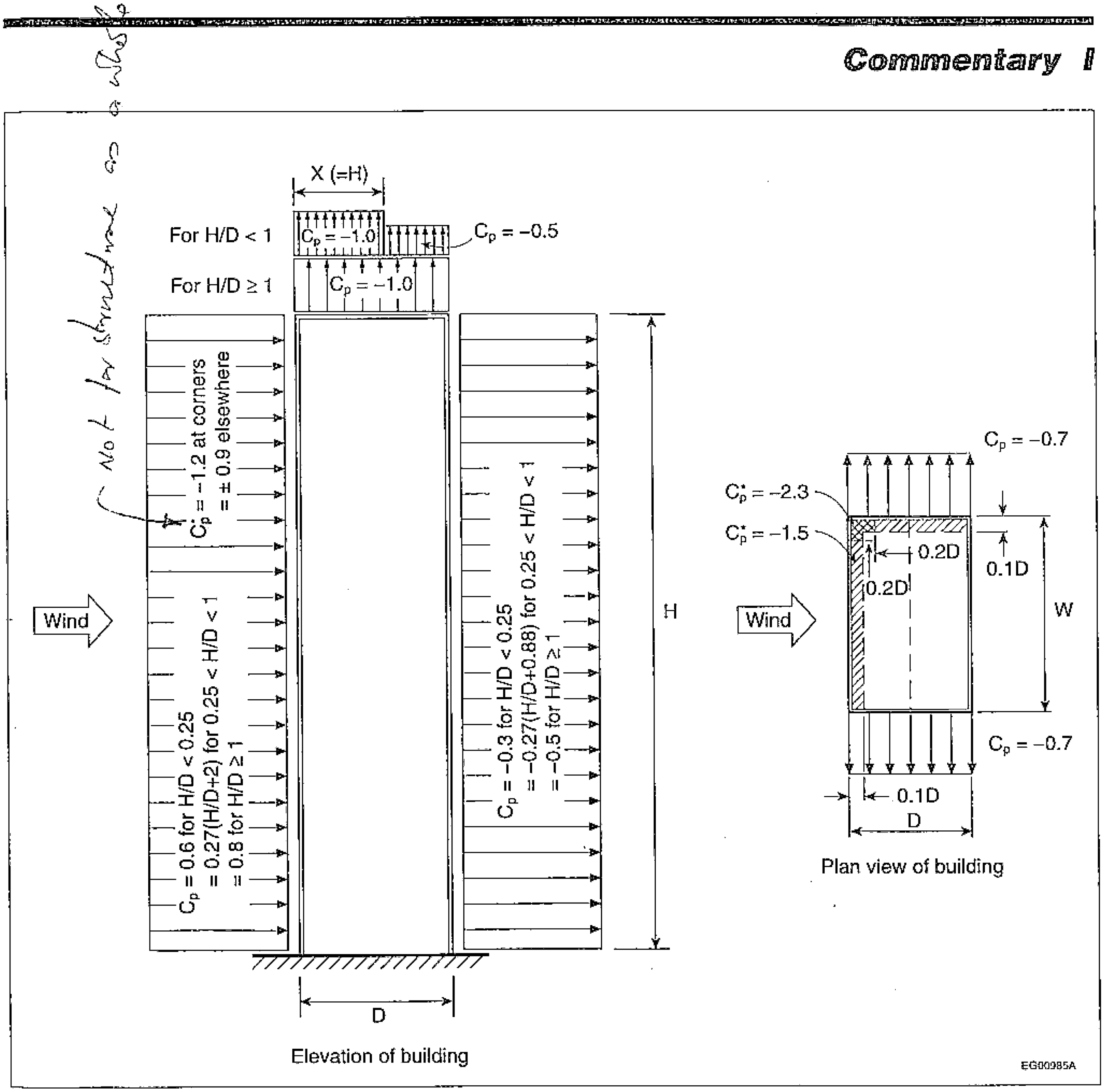


Figure I-15
External pressure coefficients, C_p and C_p^*, for flat-roofed buildings

Notes to Figure I-15:

(1) D and W denote the building plan dimensions at the base in the along-wind and cross-wind directions, respectively.
(2) The coefficients C_p shown on the windward wall are applicable when the wind is perpendicular to the wall.
(3) The coefficients C_p^* account for high local suctions created by wind blowing at a slight angle to the wall. They should be used for designing small cladding and roofing areas, but need not be considered in conjunction with the C_p for overall loading.
(4) Combinations of exterior and interior pressures must be evaluated to obtain the most severe loading. Interior pressure coefficients, C_{pi}, are given in Paragraphs 30 to 34.
(5) Pressure coefficients as shown generally apply for facades that do not contain deep vertical ribs. In such facades, the C_p^* of -1.2 given for corners applies to an edge zone that is 0.1D wide. When vertical ribs deeper than 1 m are placed on a facade, a local $C_p^* = -1.4$ may apply to an edge zone that is 0.2D wide.[35][36]
(6) The value of C_p^* can be reduced from -2.3 to -2.0 for roofs with perimeter parapets that are higher than 1 m.[40][41]
(7) On lower level(s) of flat stepped roofs, positive pressure coefficients equal to those for the walls apply for a distance, b (see Figure I-9 for the definition of b). Segments of the walls above the lower roofs qualify for the same coefficients as the other walls similarly oriented to the wind flow.[42][43]

Internal Pressure Coefficient, C_{pi}

30. The internal pressure coefficient, C_{pi}, defines the effect of wind on the air pressure inside the building and is important in the design of both cladding elements and the primary structure. The

magnitude of this coefficient depends on the distribution and size of the leakage paths and openings that vent the internal air space to the exterior. With very small and uniformly distributed cracks and pores, the leakage is slow. Although the internal pressure will approximately equilibrate to the average external pressure over the exposed surface, the influence of gusts will be attenuated. If the openings are larger and more significant—on the scale of doors or windows—the internal pressure will move closer to that prevailing externally at the largest dominant opening and gust pressures will be felt within the interior.

31. Because of the changeability and uncertainty of the size and distribution of openings, internal pressure coefficients can be wide ranging. In the face of these uncertainties, it is adequate to use the coefficients given below for both the Static and Dynamic Procedures. The coefficient depends on whether there are significant openings and whether small openings producing background leakage are uniformly distributed. In this context, a large or significant opening means a single opening or a combination of openings on any one wall that offers a passage to the wind and whose area exceeds by a factor of 2 or more the leakage area of the remaining building surface, including the roof. Such a significant opening may be provided by main doors, shipping doors, windows and ventilators if they are open during a storm, either through expected usage or through damage.

To handle the range of circumstances that may prevail, three basic design categories are provided below. For each of these three categories, C_{gi} is calculated using the provisions of Paragraph 22:

Category 1: C_{pi} = −0.15 to 0.0

This category deals with buildings without any large or significant openings, but having small uniformly distributed openings amounting to less than 0.1% of total surface area. The value of C_{pi} should be −0.15, except where such openings alleviate an external load, in which case $C_{pi} = 0$ should be used. Internal pressure fluctuates even within buildings having small distributed openings, and the pressure fluctuations occasionally reach $C_{pi} = 0$. Such buildings include high-rise buildings that are nominally sealed, have no operable windows and screen doors, and are mechanically ventilated. Some less common low-rise buildings, such as windowless warehouses with door systems not prone to storm damage, also fall into this category.

Category 2: C_{pi} = −0.45 to 0.3

This category covers buildings in which significant openings, if there are any, can be relied on to be closed during storms but in which background leakage may not be uniformly distributed. Most low-rise buildings fall into this category provided that all elements—especially shipping doors—are designed to be fully wind-resistant. Most high-rise buildings with operable windows or balcony doors also fall into this category.

Category 3: C_{pi} = −0.7 to 0.7

This category covers buildings with large or significant openings through which gusts are transmitted to the interior. Examples of such buildings include sheds with one or more open sides as well as industrial buildings with shipping doors, ventilators or the like, which have a high probability of being open during a storm or not being fully resistant to design wind loads.

32. An ever-present threat in severe storms is the breakage of large unprotected glass areas and other vulnerable components by flying debris. Structures required in post-disaster services should be capable of withstanding all the consequences of failure of glass and conform to the requirements of Category 3. For other structures in which the glass is designed for wind and there is adequate protection against roof uplift, the contingency of glass damage due to debris is covered by normal load factors for wind.

33. In most cases, there is no need to consider non-uniform internal pressures except in the design of internal partitions (see NBC Sentence 4.1.7.4.(1)). Thus, for most structural design, the two limiting values of internal pressure can be considered separately unless interior compartments of the building are well sealed and wind damage or the like could expose one area of the building to Category 3 conditions while the rest of the building remains in Category 1 or 2, resulting in unbalanced internal pressures.

34. Internal pressures are also affected by mechanical ventilation systems and by the stack effect due to different inside and outside air temperatures. Under normal operation, mechanical ventilation

systems create a differential across walls of less than 0.1 kPa, but the stack effect due to differences in temperature of 40°C could amount to a differential of 0.2 kPa per 100 m of building height.[30]

Partial Loading

35. Partial wind loading can, in some cases, cause more severe effects than full loading. Pressure patterns observed in turbulent wind indicate reduced loading on portions of the building faces, which can produce additional torsion due to horizontal shifting of the wind-load vector. Reduced but simultaneous loading along both major axes can be induced by wind blowing diagonally to the building, which can produce higher stresses in some structural members than by wind blowing along any one major axis. Other structures, such as curved roofs, may undergo larger stresses under partial loading. NBC Sentence 4.1.7.3.(1) therefore requires all buildings to be designed for partial loading as well as full loading.

36. Low buildings designed by the Static Procedure to the specifications of Figure I-7 do not need to have further unbalanced loads (see Paragraph 27). Taller buildings, in addition to being designed for the full wind load along each of the principal axes as shown in Figure I-16, Case A, should be checked for maximum additional torsion arising from partial loadings created by applying the wind pressure to only a part of the building face areas as shown in Figure I-16, Case B, for rectangular plan buildings.

37. To account for the potentially more severe effects induced by diagonal wind, and also for the tendency of structures to sway in the across-wind direction, taller structures should be designed to resist 75% of the maximum wind pressures for each of the principal directions applied simultaneously as shown in Figure I-16, Case C. In addition, the influence of removing 50% of the Case C loads from parts of the face areas that maximizes torsion, as shown in Figure I-16, Case D, should be investigated. Further discussion of combined loading effects can be found in References [15] and [16].

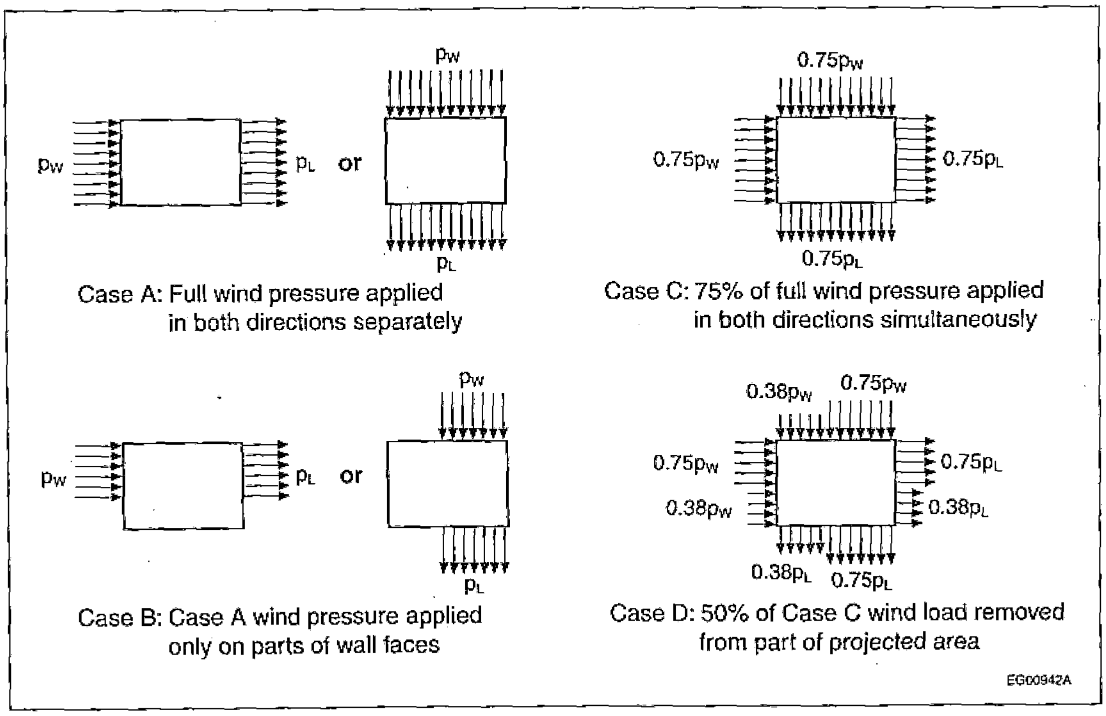


Figure I-16
Full and partial wind loads (see NBC Sentence 4.1.7.3.(1))

Notes to Figure I-16:

(1) p_W and p_L are the windward and leeward wind pressures, respectively, as calculated for the full wind load.
(2) In Case B, the full wind pressure should be applied only to parts of the wall faces so that the wind-induced torsion is maximized.

Commentary I

Dynamic Procedure

Application

38. NBC Sentence 4.1.7.2.(1) requires the use of the Dynamic or Experimental Procedure for buildings whose height is greater than 4 times their minimum effective width, or greater than 120 m, and other buildings whose properties make them susceptible to vibration. Minimum effective width is defined in NBC Sentence 4.1.7.2.(2).

39. In the Dynamic Procedure for calculating wind load on the building structure, the exposure factor, C_e, and external gust effect factor, C_g, are different from the factors used in the Static Procedure, but the pressure coefficient, C_p, is the same. See Figure I-1 for guidance on how the Dynamic Procedure for the structure is carried out in conjunction with the Static Procedure for the cladding.

40. In addition to the calculation of wind load, the calculation of wind-induced lateral deflection, vibration and vortex-shedding effect can also be important for some buildings that are required to be treated by the Dynamic Procedure. These topics are dealt with separately under the sections of this Commentary entitled Lateral Deflection of Tall Buildings, Building Vibration and Vortex Shedding.

Exposure Factor, C_e

41. In the Dynamic Procedure, the exposure factor, C_e, is based on the profile of mean wind speed, which varies considerably with the general roughness of the terrain over which the wind has been blowing before it reaches the building. To determine the exposure factor, three categories of terrain exposure have been established and are illustrated in Figures I-2 to I-5.

    Exposure A (open or standard exposure): open level terrain with only scattered buildings, trees or other obstructions, open water or shorelines thereof. This is the exposure on which the reference wind speeds are based.

$$C_e = \left(\frac{h}{10}\right)^{0.28} \quad \text{for } 1.0 \leq C_e \leq 2.5 \tag{7}$$

    Exposure B (rough exposure): suburban and urban areas, wooded terrain or centres of large towns with very few and scattered tall buildings.

$$C_e = 0.5\left(\frac{h}{12.7}\right)^{0.50} \quad \text{for } 0.5 \leq C_e \leq 2.5 \tag{8}$$

    Exposure C (very rough exposure): centres of large cities with heavy concentrations of tall buildings. At least 50% of the buildings should exceed 4 storeys. This exposure is only applicable to the heavily built-up centres of large cities and should be used with caution because of local channeling and wake buffeting effects that can occur near tall buildings.

$$C_e = 0.4\left(\frac{h}{30}\right)^{0.72} \quad \text{for } 0.4 \leq C_e \leq 2.5 \tag{9}$$

    In Equations (7) to (9), h is the reference height (see Paragraphs 7 and 8) above ground in metres. The exposure factor can be calculated using these Equations or can be obtained directly from Figure I-17.

42. Exposure B or C should not be used unless the applicable terrain roughness persists in the upwind direction for at least 1.0 km or 10 times the height of the building, H, whichever is larger, and the exposure factor should be recalculated if the roughness of the terrain differs from one direction to another.

43. In addition to being used to calculate pressures on building surfaces, the exposure factor is needed for calculating the hourly mean wind speed at the top of the building, V_H, and the gust effect factor, C_g (see Paragraphs 46 and 47).

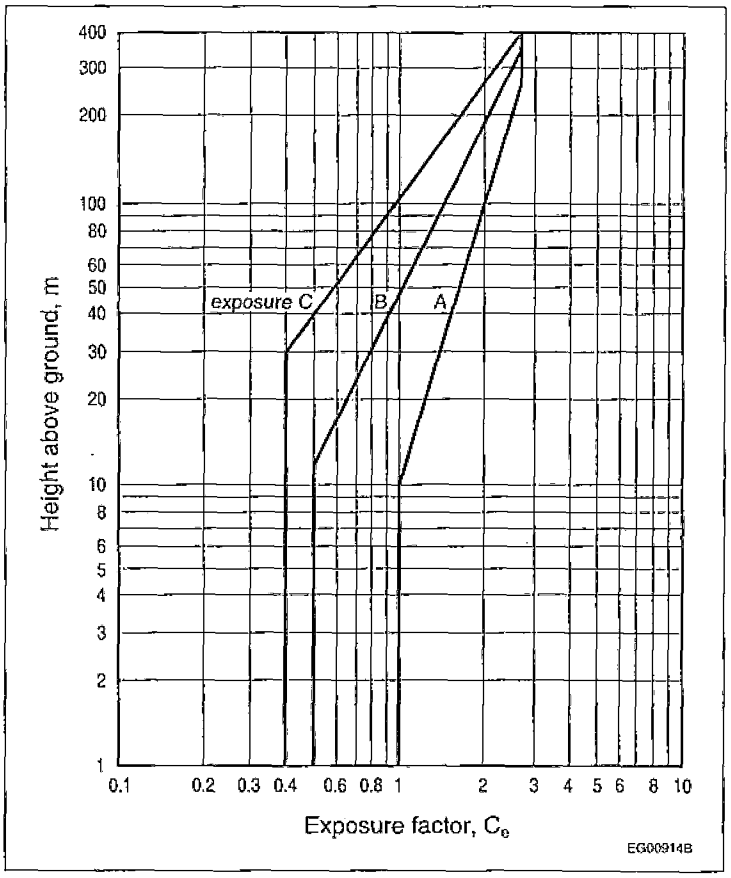


Figure I-17
Exposure factor as a function of terrain roughness and height above ground

Speed-up over Hills and Escarpments

44. The exposure factor must be modified for speed-up over hills and escarpments in both the Static and Dynamic Procedures as covered in Paragraphs 13 to 16. However, as the speed-up principally affects the mean wind speed and not the amplitude of the turbulent fluctuations, a correction also needs to be applied to the gust effect factor as shown in Paragraph 21.

Gust Effect Factor, C_g

General

45. The general discussion on the gust effect factor presented in Paragraphs 17 and 18 under the Static Procedure is also applicable to the Dynamic Procedure.

External Gust Effect Factor, C_g

46. A general expression for the maximum or peak loading effect, denoted by W_p, is as follows:

$$W_p = \mu + g_p \sigma \tag{10}$$

where

μ = mean loading effect,
g_p = statistical peak factor for the loading effect, and
σ = "root-mean square" loading effect.

If this expression is rearranged, the following expression for the gust effect factor, C_g, which is equal to W_p/μ, is obtained:

$$C_g = 1 + g_p\,(\sigma/\mu) \tag{11}$$

The form of the fluctuating wind loading effect, σ, varies with the excitation, whether it is due to gusts, wake pressures or motion-induced forces.

47. The value of σ/μ, the coefficient of variation, can be expressed by

$$\sigma/\mu = \sqrt{\frac{K}{C_{eH}}\left(B + \frac{sF}{\beta}\right)} \tag{12}$$

where

- K = a factor related to the surface roughness coefficient of the terrain (see Paragraph 41 for the definitions of Exposures A, B and C),
  - = 0.08 for Exposure A,
  - = 0.10 for Exposure B,
  - = 0.14 for Exposure C,
- C_{eH} = exposure factor at the top of the building evaluated according to Paragraph 41 or Figure I-17, and modified for speed-up over hill or escarpment if required,
- B = background turbulence factor obtained from Figure I-18 as a function of w/H,
- w = effective width of the windward face of the building, as defined in NBC Sentence 4.1.7.2.(2),
- H = height of the windward face of the building,
- s = size reduction factor obtained from Figure I-19 as a function of w/H and the reduced frequency $f_{nD}H/V_H$,
- f_{nD} = natural frequency of vibration in the along-wind direction, in Hz,
- V_H = mean wind speed, in m/s, at the top of structure, H, evaluated using Equation (13) below,
- F = gust energy ratio at the natural frequency of the structure obtained from Figure I-20 as a function of the wave number, f_{nD}/V_H, and
- β = critical damping ratio in the along-wind direction.

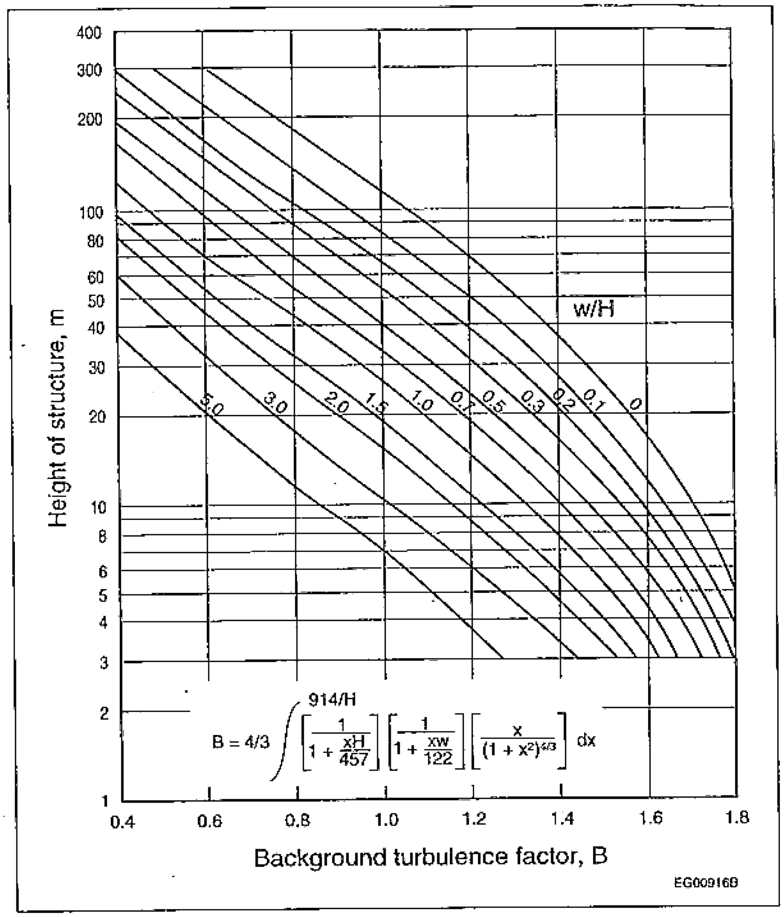


Figure I-18
Background turbulence factor as a function of width and height of structure

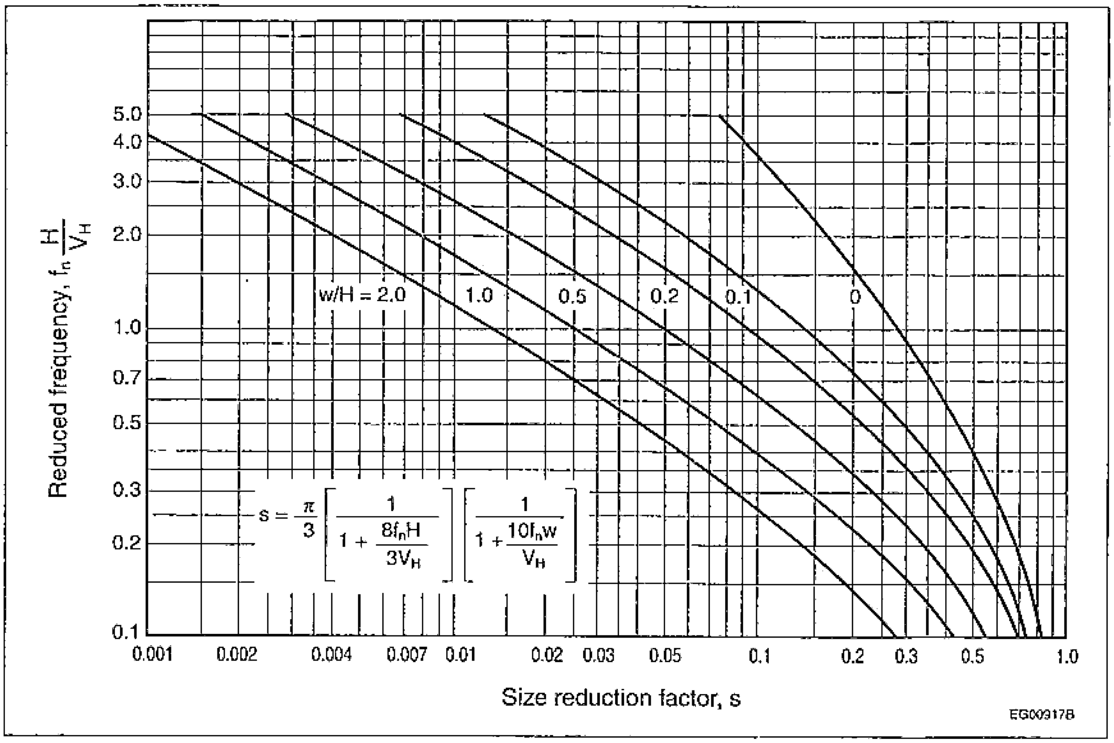


Figure I-19
Size reduction factor as a function of width, height and reduced frequency of structure

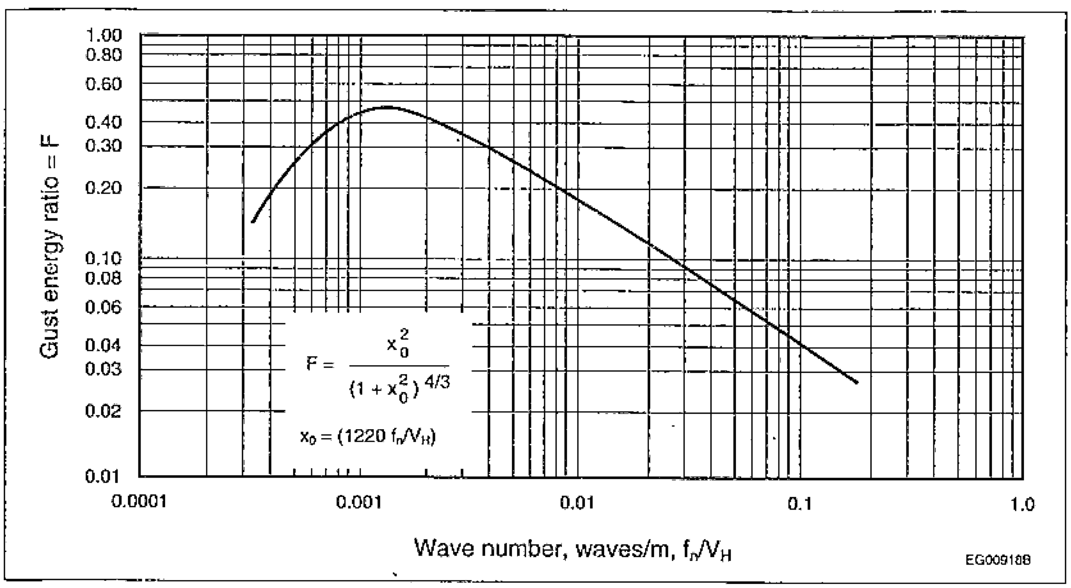


Figure I-20
Gust energy ratio as a function of wave number

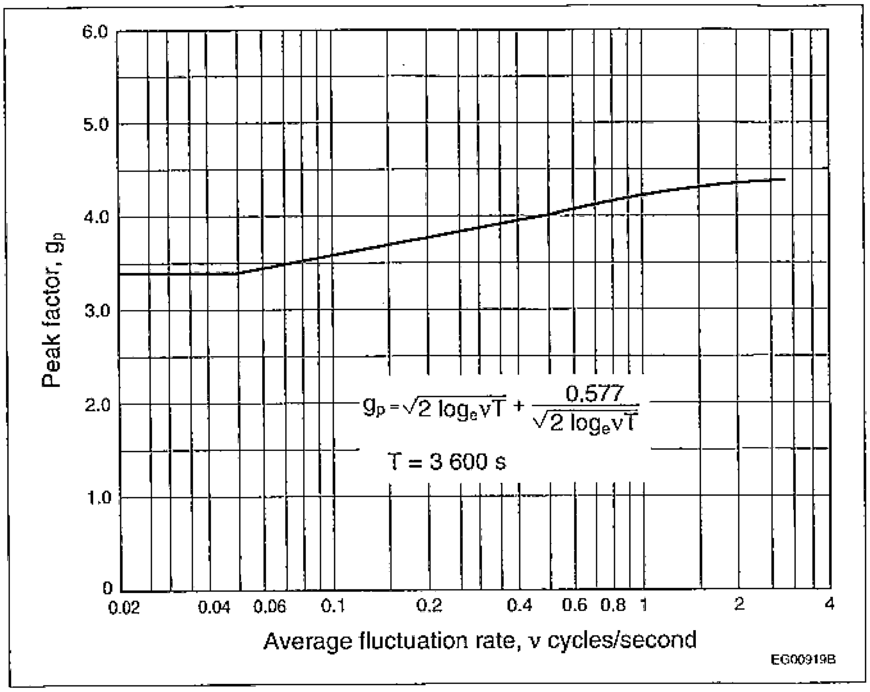


Figure I-21
Peak factor as a function of average fluctuation rate

The mean wind speed at the top of the structure, V_H in m/s, in Figures I-19 and I-20 is given by

$$V_H = \overline{V}\sqrt{C_{eH}} \tag{13}$$

where $\overline{V}$ (m/s), the reference wind speed at the height of 10 m, is determined from the reference velocity pressure, q (kPa), as follows (see NBC Appendix C of Division B):

$$\overline{V} = 39.2\sqrt{q} \tag{14}$$

48. The critical damping ratio, β, is based mainly on experiments on real structures. Expressed as a fraction of critical damping, values commonly used in the design of buildings with steel frames and concrete frames are 1% and 2%, respectively. Masts and stacks, on the other hand, may have much lower inherent or structural damping. Aerodynamic damping in the along-wind direction becomes significant at high wind speeds, but plays no useful role in limiting cross-wind motion induced by vortex shedding. Spread footings on soft or medium-stiff soil provide higher damping compared to pile foundations or spread footings on stiff soil and rock. Damping values measured from more than 20 stacks are tabulated in Reference [17] and the results from 5 more stacks are given in Reference [18]. The logarithmic decrement mentioned therein is 2π times the critical damping ratio. Sachs[17] concludes by stating a range of 0.2% to 0.8% for β for the total damping of closed circular and unlined welded steel stacks, and suggests that the minimum value be used in design. Corresponding ranges for lined welded steel stacks and for unlined reinforced concrete stacks are given as 0.5% to 1% and 1% to 2%, respectively.

49. The peak factor, g_p, in Equation (10) gives the number of standard deviations by which the peak load effect is expected to exceed the mean load effect, and is given in Figure I-21 as a function of the average fluctuation rate. The average fluctuation rate, v, can be estimated as follows using the variables defined for Equation (12):

$$v = f_n\sqrt{\frac{sF}{sF + \beta B}} \tag{15}$$

Explanatory Notes Regarding σ/μ and g_p

50. The response of a tall, slender building to a randomly fluctuating force can be evaluated rather simply by treating it as a rigid, spring-mounted cantilever whose dynamic properties are specified by a single natural frequency and an appropriate damping value. The variance of the output quantity or loading effect is the area under the spectrum of the input quantity (the forcing function) after it has been multiplied by the transfer function. The transfer function is the square of the well-known dynamic load magnification factor for a one-degree-of-freedom oscillating mechanical system.

51. In the case of wind as the randomly fluctuating force, the spectrum of the wind speed must first be multiplied by another transfer function called the aerodynamic admittance function, which in effect describes how the turbulence in the wind is modified by its encounter with the building, at least insofar as its ability to produce a loading effect on the structure is concerned.

52. For the purpose of calculating σ/μ, the spectrum of the wind speed is represented by an algebraic expression derived from observations of real wind. The aerodynamic admittance function is also an algebraic expression, computed on the basis of somewhat simplified assumptions but appearing to be in reasonable agreement with experimental evidence. The spectrum of wind speed is a function of frequency having the shape of a rather broad hump (see Figure I-20). The effect of the aerodynamic admittance is to reduce the ordinates of the curve to the right of the hump more and more as the frequency increases. This is partly a reflection of the reduced effectiveness of small gusts in loading a large area. The effect of the dynamic load magnification factor or mechanical admittance is to create a new peak or hump centred at the natural frequency of the structure—usually well to the right of the broad peak—which represents the maximum density of fluctuating force of the wind.

53. The area under the loading effect spectrum, the square root of which is the coefficient of variation, σ/μ, is taken as the sum of two components: the area under the broad hump, which must be integrated numerically for each structure, and the area under the resonance peak, for which a single analytic expression is available. These components are represented in Equation (12) by B and sF/β, respectively. The factor K/C_{eH} can be thought of as scaling the result for the appropriate input turbulence level. If resonance effects are small, then sF/β will be small compared to the background turbulence B, and vice versa.

54. The peak factor, g_p, depends on the average number of times the mean value of the loading effect is surpassed during the averaging time of 1 hour (3600 s). The functional relationship in Figure I-21 holds when the probability distribution of the mean loading effect is normal (Gaussian).[19]

Correction of C_g for Speed-up over Hills and Escarpments

55. The correction applied under the Static Procedure in Paragraph 21 must also be applied under the Dynamic Procedure.

Sample Calculation of C_g

56. To illustrate the calculation of a gust effect factor, the following sample problem is worked out in detail:

Objective: To obtain the gust effect factor for a building with the following properties:

Height, H	183 m
Across-Wind Effective Width, w	30.5 m
Along-Wind Effective Depth, d	30.5 m
Fundamental natural frequency, f_{nD}	0.2 Hz
Critical damping ratio, β	0.015
Terrain for site	Exposure B
Reference wind speed, $\bar{V}$, at 10 m and in open terrain	27.4 m/s

Step 1: Calculate required parameters.

C_{eH} = 1.90 (from Figure I-17)

$V_H = \bar{V}\sqrt{C_{eH}}$ (Equation (13))

$= 27.4 \times \sqrt{1.90}$

$= 37.8$ m/s

w/H = aspect ratio

$= 30.5/183$

$= 0.17$

f_{nD}/V_H = wave number for calculation of F

$= 0.2/37.8$

$= 0.0053$

$f_{nD}H/V_H$ = reduced frequency for calculation of s

$= 0.2 \times 183/37.8$

$= 0.968$

Step 2: Calculate σ/μ using Equation (12).

K = 0.10 for Exposure B

B = 0.62 (from Figure I-18)

s = 0.11 (from Figure I-19)

F = 0.28 (from Figure I-20)

β = 0.015 (given)

$$\sigma/\mu = \sqrt{\frac{K}{C_{eH}}\left(B + \frac{sF}{\beta}\right)}$$
$$= \sqrt{\frac{0.10}{1.90}\left(0.62 + \frac{0.11 \times 0.28}{0.015}\right)}$$
$$= 0.375$$

Step 3: Calculate v using Equation (15).

f_{nD} = 0.2 Hz (given)

$$v = f_{nD}\sqrt{\frac{sF}{sF + \beta B}}$$
$$= 0.2\sqrt{\frac{0.11 \times 0.28}{0.11 \times 0.28 + 0.015 \times 0.62}}$$
$$= 0.175/s$$

Step 4: Obtain peak factor, g_p.

g_p = 3.75 (from Figure I-21)

Step 5: Calculate C_g using Equation (11).

$C_g = 1 + g_p(\sigma/\mu)$

$= 1 + 3.75(0.375)$

$= 2.41$

Pressure Coefficients, C_p

General

57. The general discussion presented in Paragraphs 23 and 24 under the Static Procedure is also applicable to the Dynamic Procedure.

External Pressure Coefficient, C_p

58. The coefficients given in Figure I-15 under the Static Procedure are also applicable to the Dynamic Procedure (see Paragraph 29).

Partial Loading

Refer to Paragraphs 35 to 37 for partial loading requirements.

Wind Load on Miscellaneous Structures

Interior Walls and Partitions

59. If windows are broken during a storm, considerable pressure differences can result across interior walls and partitions in high-rise buildings, as well as in low-rise buildings in exposed locations. In certain locations, almost the full pressure difference between the windward and leeward sides of the building could be applied across interior walls or partitions. For example, when a large window in a small room on the windward side is broken by flying debris, the full positive pressure is exerted on the walls of that room. Similar conditions could prevail in an apartment building with operable windows or doors. This pressure difference could be aggravated by mechanical ventilation and winter-time stack effects in a tall building. On the other hand, experience does not indicate many failures of interior walls due to pressure differences, and thus interior walls and partitions are not required to be designed for the maximum possible pressure difference. An unfactored pressure difference of at least 0.25 kPa is suggested and a value of 0.5 kPa or higher may be appropriate in cases where the exterior wind pressures are likely to be transmitted to the interior walls and partitions through large openings in the exterior envelope.

Protected Membrane Roofs

60. In the case of a protected-membrane roof in which the insulation is not bonded to the waterproofing membrane, the insulation is not subjected to the same uplift pressure as is applied through the depth of the entire roof assembly, because of air leakage and partial pressure equalization between the top and bottom of the insulation boards. External pressure or uplift due to wind is, therefore, applied to the membrane, which acts as an air barrier between the inside and the outside and prevents pressure equalization. Further information can be found in References [20] and [21].

Unenclosed Parking Structures

61. For multi-level, unenclosed parking structures, the exposed exterior area is reduced compared with enclosed structures. However, internal parts of the structure, and vehicles parked there, are subject to additional wind forces not present in enclosed structures. In lieu of a detailed analysis of the specific structure under consideration, a reasonable and conservative assumption is to treat the unenclosed parking structure as though it were enclosed.

Structural Members and Frames, and Rounded Structures

62. Although the NBC deals primarily with building structures, the present Commentary has a long tradition of providing guidance on determining the wind load on various other structures. Figures I-22 and I-24 to I-33 at the end of the Commentary, which are derived from Standard No. 160 produced by the Swiss Association of Engineers and Architects Standards (SIA),[22] provide such guidance. The Figures are based on wind-tunnel experiments in which the correct velocity profile and wind turbulence were not simulated; they should therefore be regarded with caution. Note that many of these Figures provide formulae for the total wind load rather than the wind pressure as given by the NBC, and hence use a force coefficient rather than a pressure coefficient. The exposure and gust effect factors required in the Figures to calculate the wind load can be determined by using either the Static Procedure, the Dynamic Procedure, or Vortex Shedding of rounded structures described in this Commentary, as deemed appropriate.

63. Wind loads on standalone structural members, and frames, trusses and lattices made of such members can be calculated using Figures I-29 to I-33. The subscript ∞ in these Figures indicates that the coefficients apply to structural members of infinite lengths. The coefficients are multiplied by a reduction factor, k, for structural members of finite lengths. If a structural member cantilevers from a large plate or wall, k should be calculated for a slenderness based on twice the actual length. If a member terminates with both ends in large plates or walls, the reduction factors for infinite length should be used.

64. For framing members that are located behind each other in the direction of the wind, the shielding effect may be taken into account. The shielded parts of the leeward members should be designed with the reduced pressure, q_x, according to Figure I-31. A detailed discussion of the loads on unclad building frameworks is given in Reference [23].

65. As the shape of a structure may change during erection, the wind loads may be temporarily more critical during erection than after completion of the structure.[24] These increased wind loads should be taken into account using the appropriate coefficients from Figures I-7 to I-15 and I-22 to I-33.

66. For constructions made of circular sections with $D\sqrt{qC_e} < 0.167$ and $A_s/A > 0.3$, the shielding factors can be taken by approximation from Figure I-28. If $D\sqrt{qC_e} \geq 0.167$, the shielding effect is small and for a solidity ratio $A_s/A \leq 0.3$, it can be taken into account by a constant shielding factor $k_x = 0.95$.

67. For rounded structures (in contrast to sharp-edged structures), the cross-wind pressures vary with the wind velocity and depend strongly on the Reynolds Number. Pressure coefficients for some rounded structures are given in Figures I-24, I-25, I-28 and I-33, in which the Reynolds Number is expressed differently from the conventional one, by $D\sqrt{qC_e}$, where D is the diameter of the sphere or cylinder in m and q is the velocity pressure in kPa. To convert to the conventional Reynolds Number, multiply $D\sqrt{qC_e}$ by 2.7×10^6.

68. The roughness of rounded structures may be of considerable importance. With reference to Figure I-24, metal, concrete, timber and well-laid masonry without parging can be considered as having a "moderately smooth" surface. Surfaces with ribs projecting more than 2% of the diameter are considered "very rough." In case of doubt, coefficients that result in the greater forces should be used. For cylindrical and spherical objects with substantial stiffening ribs, supports and attached structural members, the pressure coefficients depend on the type, location and relative magnitude of these roughnesses. For vortex shedding of circular cylinders, see Reference [25].

Increased Wind Load due to Icing

69. In locations where the strongest winds and icing may occur simultaneously, forces on structural members, cables and ropes must be calculated assuming an ice covering based on climate and local experience. For the iced condition, values of C_f given in Figure I-28 for thick wire cables for a "rough" surface should be used. Information on icing loads can be obtained from the CSA standard on antennas and towers[26] and the ISO standard on icing load.[27]

Vortex Shedding

70. Slender, free-standing cylindrical structures such as chimneys, observation towers and in some cases, high-rise buildings, should be designed to resist the dynamic effect of vortex shedding. A structure may be considered slender in this context if the ratio of height to width or diameter exceeds 5. When the wind blows across slender prismatic or cylindrical structures, vortices are shed alternately from one side and then the other along the length of the structure, giving rise to a fluctuating force acting at right angles to the wind direction. The wind speed, V_{Hc}, at the top of the structure when the frequency of vortex shedding equals the natural frequency, f_n, is given by:

$$V_{Hc} = \frac{1}{S} f_n D \qquad (16)$$

where
V_{Hc} = critical mean wind speed at the top of the structure, in m/s, when resonance due to vortex shedding occurs,
S = Strouhal Number, which is dependent on the shape of the cross-section,
f_n = frequency, in Hz, and
D = width or diameter, in m.

For circular and near-circular cylinders, the Strouhal Number is approximately 1/6 for small-diameter structures such as chimneys, and 1/5 for large-diameter structures such as observation towers or buildings. For non-circular cylindrical structures, the Strouhal Number is approximately 1/7.

71. The dynamic effects of vortex shedding of circular and near-circular cylindrical structures, including tapered structures, can be estimated in accordance with Reference [25]. Wind-tunnel tests are recommended for non-circular cylindrical structures.

Lateral Deflection of Tall Buildings

72. Lateral deflection of tall buildings under wind loading may require consideration from the standpoints of serviceability or comfort. The general trend is toward more flexible structures, partly because adequate strength can now be achieved by using higher strength materials that may not provide a corresponding increase in stiffness.

73. One symptom of unserviceability may be the cracking of masonry and interior finishes. Unless precautions are taken to permit movement of interior partitions without damage, a maximum lateral deflection limitation of 1/250 to 1/1 000 of the building height should be observed. According to NBC Sentence 4.1.3.5.(3), 1/500 should be used unless other drift limits are specified in the design standards referenced in NBC Section 4.3. or a detailed analysis is made.

Building Vibration

74. While the maximum lateral wind loading and deflection are generally in the direction parallel to the wind (i.e. the along-wind direction), the maximum acceleration of a building leading to possible human perception of motion or even discomfort may occur in the direction perpendicular to the wind (i.e. the across-wind direction). Across-wind accelerations are likely to exceed along-wind accelerations if the building is slender about both axes, that is if $\sqrt{wd}/H$ is less than one-third, where w and d are the across-wind effective width and along-wind effective depth, respectively, and H is the height of the building. The along-wind effective depth, d, is calculated using the formula given in NBC Sentence 4.1.7.2.(2) by replacing w_i by d_i.

75. The accelerations in a building are very dependent on the building's shape, orientation and buffeting from surrounding structures. However, data on the peak across-wind acceleration at the top of the building from a variety of turbulent boundary-layer wind-tunnel studies exhibit much scatter around the following empirical formula:

$$a_W = f_{nW}^2 g_p \sqrt{wd}\left(\frac{a_r}{\rho_B g \sqrt{\beta_W}}\right) \tag{17}$$

76. In less slender structures or for lower wind speeds, the maximum acceleration may be in the along-wind direction and may be estimated from the following expression:

$$a_D = 4\pi^2 f_{nD}^2 g_p \sqrt{\frac{KsF}{C_{eH}\beta_D}\frac{\Delta}{C_g}} \tag{18}$$

The variables in the formulae given in Paragraphs 75 and 76 have the following definitions:

- w, d = across-wind effective width and along-wind effective depth respectively, in m,
- a_W, a_D = peak acceleration in across-wind and along-wind directions, respectively, in m/s²,
- $a_r = 78.5 \times 10^{-3}\left[V_H/\left(f_{nW}\sqrt{wd}\right)\right]^{3.3}$, in N/m³,
- ρ_B = average density of the building, in kg/m³,
- β_W, β_D = fraction of critical damping in across-wind and along-wind directions, respectively,
- f_{nW}, f_{nD} = fundamental natural frequencies in across-wind and along-wind directions, respectively, in Hz,
- Δ = maximum wind-induced lateral deflection at the top of the building in along-wind direction, in m, and
- g = acceleration due to gravity = 9.81 m/s².

The variables g_p, K, s, F, C_{eH}, and C_g are as defined previously in connection with Equations (10) to (12).

77. Although many additional factors such as visual cues, body position and orientation, and state of mind influence human perception of motion, when the amplitude of acceleration is in the range of 0.5% to 1.5% of g, movement of the building becomes perceptible to most people.[28][29][30]

78. Historically, Equations (17) and (18) have been used with one-in-ten-year wind acceleration limits of 1% to 3% of g for the preliminary assessment of tall buildings. In North America in the period 1975 to 2000, many of the tall buildings that underwent detailed wind tunnel studies were designed for a peak one-in-ten-year acceleration in the range of 1.5% to 2.5% of g. The lower end of this range was generally applied to residential buildings and the upper end to office towers; their performance based on these criteria appears to have been generally satisfactory. Other criteria have been published that depend on the building's lowest natural frequency. The ISO criterion[31] can be expressed as a peak acceleration not exceeding 0.928 $f^{-0.412}$ once every 5 years, where f is the lowest natural frequency in Hz. This results in a 5-year criterion of about 1.8% of g when f = 0.2 Hz, and 2% of g when f = 0.1 Hz.

79. Owing to the relative sensitivity of Equations (17) and (18) to the natural frequency of vibration, and of Equation (18) to the corresponding building stiffness, these properties should be determined using fairly rigorous methods, and approximate formulas should be used with caution. For example, the adoption of a natural frequency of 10/N, where N is the number of storeys, may not be consistent with the assumption that the displacement under wind loading is as large as H/500.

Sample Calculation of a_W and a_D

80. A detailed calculation of a_W and a_D using Equations (17) and (18) will be carried out using the sample problem worked out in Paragraph 56 to illustrate the calculation of gust effect factor. Although one-in-ten-year wind data should be used to determine a_W and a_D, the values of C_g and other parameters computed earlier will be used in this example to avoid repeating the computation.

Given that $f_{nW} = f_{nD} = 0.2$ Hz and $q_{10} = 0.49$ kPa

$$\begin{aligned}\beta_W &= \beta_D = 0.015\\ \rho_B &= 176\ \mathrm{kg/m^3}\end{aligned}$$

Step 1: Calculate a_r.

$$\begin{aligned}a_r &= 78.5 \times 10^{-3}\left[V_H/\left(f_{nW}\sqrt{wd}\right)\right]^{3.3}\\ &= 78.5 \times 10^{-3}\,[37.8/(0.2 \times 30.5)]^{3.3}\\ &= 32.3\ \mathrm{N/m^3}\end{aligned}$$

Step 2: Calculate a_W using Equation (17).

$$\begin{aligned}a_W &= 0.2^2 \times 3.75 \times 30.5\left(\frac{32.3}{176 \times 9.81\sqrt{0.015}}\right)\\ &= 0.70\ \mathrm{m/s^2}\end{aligned}$$

Therefore, $a_W/g = 7.1\%$.

Step 3: Calculate a_D/g. a_D is given in Equation (18) as a function of Δ whose value is usually determined from a structural analysis. In this example, Δ_{10}, the value of Δ for one-in-ten-year wind, is assumed equal to 0.35 m.

$$\begin{aligned}a_D &= 4\pi^2 \times 0.2^2 \times 3.75\sqrt{\frac{0.1 \times 0.11 \times 0.28}{1.9 \times 0.015}\,\frac{0.35}{2.41}}\\ &= 0.283\end{aligned}$$

$$a_D/g = 0.283/9.81 = 2.9\%$$

81. In this example, the across-wind accelerations clearly overshadow the along-wind accelerations. A tall building erected in a waterfront location may be exposed to all three terrain conditions for different wind directions.

Tornadoes

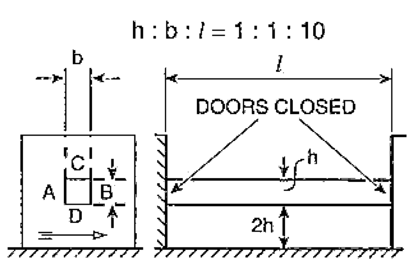
82. Although the probability of any one particular building being hit by a tornado is very small (less than 10^{-5} per year[32]), tornadoes account for the greatest incidence of death and serious injury of building occupants due to structural failure and cause considerable economic loss. With some exceptions, such as nuclear power plants, it is generally not economical to design buildings for tornadoes beyond what is currently required by NBC Subsection 4.1.7. because of the low risk of loss to individual owners (insurance is cheaper). It is, however, important to provide key construction details for the safety of building occupants. Investigations of tornado-damaged areas in Eastern Canada[33][34] have shown that the buildings in which well over 90% of the occupants were killed or seriously injured by tornadoes did not satisfy the following two key details of building construction:
    (a) the anchorage of house floors into the foundation or ground (the floor takes off with the occupants on it), and
    (b) the anchorage of roofs down through concrete block walls (the roof takes off and the unsupported block wall collapses onto the occupants).

83. The first detail—the anchorage of house floors—is essentially covered by NBC Article 9.23.6.1. for typical housing with permanent foundations. CSA Z240.10.1[50] contains anchorage recommendations for protecting mobile homes against the effects of tornadoes. The second detail—roof anchorage in block walls—is essentially covered in CSA S304.1[51] through limit states requirements for wind uplift and, for the empirical method of masonry design, by Clause F.1.4 of the standard. Deficiency of this construction detail is especially serious for open assembly occupancies because there is nothing inside, such as stored goods, to protect the occupants from wall collapse. For such buildings in tornado-prone areas, it is recommended that the block walls contain vertical reinforcing linking the roof to the foundation.

84. For tornado protection, key details such as those indicated above should be designed on the basis of a factored uplift wind suction of 2 kPa on the roof, a factored lateral wind pressure of 1 kPa on the windward wall, and suction of 2 kPa on the leeward wall.

85. Guidance for determining if a given locality is prone to tornadoes may be obtained from Information Services Section, Environment Canada, 4905 Dufferin Street, Toronto, Ontario M3H 5T4; e-mail: climate.services@ec.gc.ca.

Figures



C_p : External pressure coefficients

ϕ	A	B	C	D
0°	+0.8	−1.2	−1.4	−1.5

C_{pi} : Internal pressure coefficients

OPENINGS	ϕ = 0°
Uniformly distributed	−0.5
Predominating on side "A"	+0.7
Predominating on side "B"	−1.1
Predominating on side "C"	−1.3

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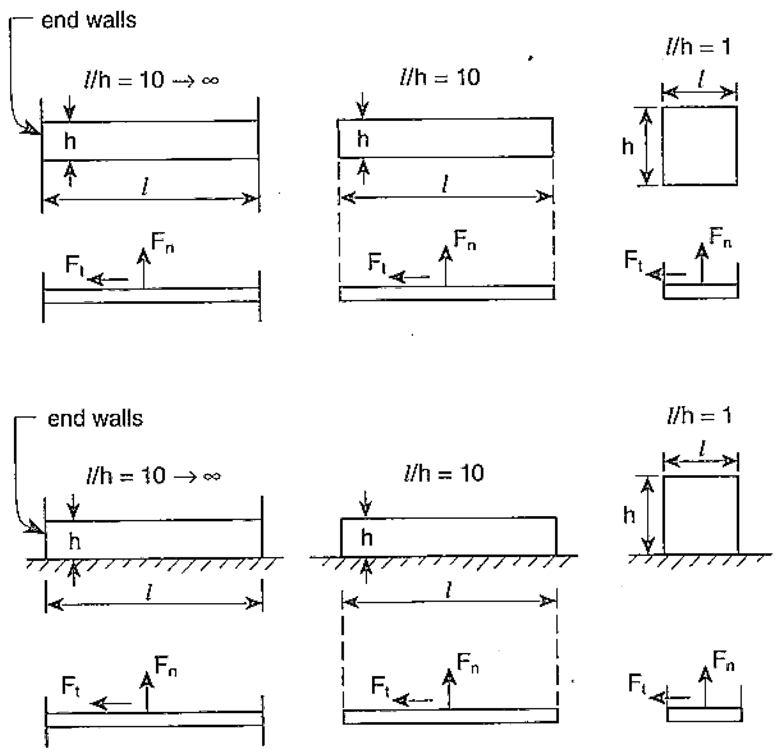
Figure I-22
Closed passage between large walls

Case 1

Case 2



C_f, Force Coefficient for Walls Above Ground

l/h	$10 \rightarrow \infty$ (End Walls)	10	1
C_f	2.0	1.3	1.15

C_f, Force Coefficient for Walls on the Ground

l/h	$10 \rightarrow \infty$ (End Walls)	10	1
C_f	1.3	1.2	1.1

$$F_n = C_f C_n q C_g C_e hl$$
$$F_t = C_f C_t q C_g C_e hl$$

Normal and Transverse Combinations

Case	Normal Component, C_n	Transverse Component, C_t
1	1.0	0.2
2	0.6	0.3

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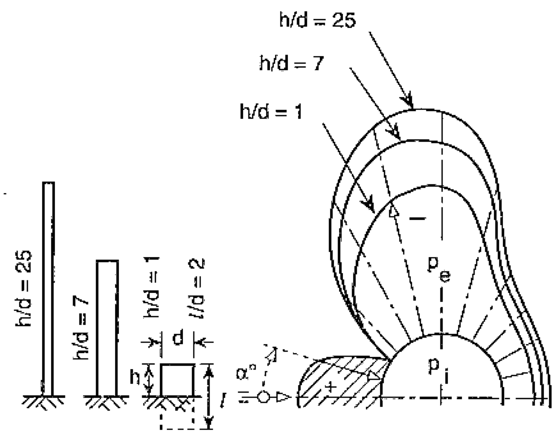
Figure I-23
Free-standing plates, walls and billboards

TOTAL FORCE $F = C_f \cdot q \cdot C_g \cdot C_e \cdot A$, where $A = d \cdot h$



C_f : FORCE COEFFICIENT FOR $d\sqrt{qC_e} > 0.167$

Slenderness h/d = →	25	7	1
Cross section and roughness	C_f	C_f	C_f
Moderately smooth, (metal, timber, concrete)	0.7	0.6	0.5
Rough surface (rounded ribs h = 2%d)	0.9	0.8	0.7
Very rough surface (sharp ribs h = 8%d)	1.2	1.0	0.8
Smooth and rough surface sharp edges	1.4	1.2	1.0

C_p : EXTERNAL PRESS. COEFF. FOR $d\sqrt{qC_e} > 0.167$ and moderately smooth surface

h/d	l/d	α=	0°	15°	30°	45°	60°	75°	90°	105°	120°	135°	150°	165°	180°
25	50	C_p	+1.0	+0.8	+0.1	−0.9	−1.9	−2.5	−2.6	−1.9	−0.9	−0.7	−0.6	−0.6	−0.6
7	14	C_p	+1.0	+0.8	+0.1	−0.8	−1.7	−1.6	−2.2	−1.7	−0.8	−0.6	−0.5	−0.5	−0.5
1	2	C_p	+1.0	+0.8	+0.1	−0.7	−1.2	−1.6	−1.7	−1.2	−0.7	−0.5	−0.4	−0.4	−0.4

$\Delta p = p_i - p_e$ $\quad$ $p_i = C_{pi} \cdot q \cdot C_g \cdot C_e$
$p_e = C_p \cdot q \cdot C_g \cdot C_e$

Stack fully operating $C_{pi} = +0.1$; Stack throttled $C_{pi} = -0.8$

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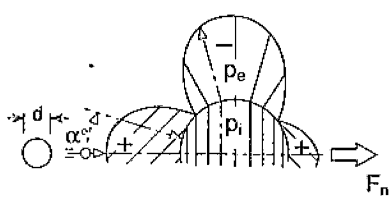
Figure I-24
Cylinders, chimneys and tanks

TOTAL FORCE $F = C_f \cdot q \cdot C_g \cdot C_e \cdot A$; $A = \frac{\pi d^2}{4}$

for $d\sqrt{qC_e} > 0.8$ and moderately smooth surface



C_f : FORCE COEFFICIENT

$C_f = 0.2$

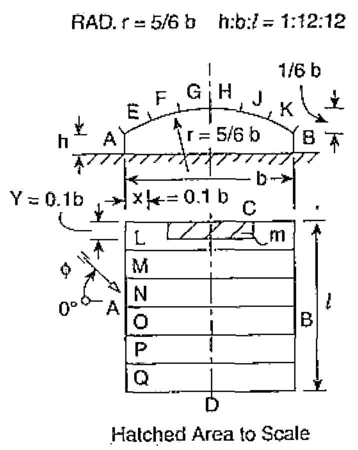
$p = p_i - p_e$ $\quad$ p_i for closed tanks = working pressure
$p_e = C_p \cdot q \cdot C_g \cdot C_e$

C_p : EXTERNAL PRESS. COEFF. FOR $d\sqrt{qC_e} > 0.8$ and moderately smooth surface

α=	0°	15°	30°	45°	60°	75°	90°	105°	120°	135°	150°	165°	180°
C_p	+1.0	+0.9	+0.5	−0.1	−0.7	−1.1	−1.2	−1.0	−0.6	−0.2	+0.1	+0.3	+0.4

EG00933B

Figure I-25
Spheres



C_p : EXTERNAL PRESSURE COEFFICIENTS

φ	A	B	C	D	E	F	G	H	J	K
0°	+0.7	−0.2	−0.3	−0.3	−0.1	−0.5	−0.8	−0.8	−0.4	−0.1
30°	+0.6	−0.3	+0.2	−0.4	−0.1	−0.4	−0.7	−0.9	−0.7	−0.4

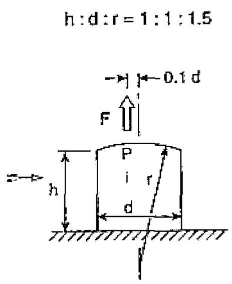
φ	A	B	C	D	L	M	N	O	P	Q
90°	−0.3	−0.3	+0.9	−0.3	−0.8	−0.7	−0.5	−0.3	−0.1	−0.1
30°	Section "m" $C_p^* = -1.8$ with C_p^* min. = −2.5									

C_{pi} : INTERNAL PRESSURE COEFFICIENTS

OPENINGS	φ = 0°	φ = 30°	φ = 90°
Uniformly distributed	±0.2	±0.2	±0.2
Window Y open on side "A"	+0.4	+0.7	−1.0
All doors open on side "C"	−0.1	+0.6	+0.8
Only door X open on side "C"	−1.5	+0.7	+0.4

EG00934B

Figure I-26
Hangar, curved roof with moderately smooth surface



Total force on roof $F = (p_i - p_e)\,A$

p_i = working pressure in kN/m²

$p_e = C_p \cdot q \cdot C_g \cdot C_e$

$A = \frac{\pi}{4} d^2$

External pressure coefficient $C_p = -1.0$

EG00935B

Figure I-27
Roof load on smooth closed tank

C_f = FORCE COEFFICIENTS

$l/d > 100$

Total force $F = C_f \cdot q \cdot C_g \cdot C_e \cdot A$

$A = d \cdot l$

	$d\sqrt{q\,C_e}$ < 0.167	$d\sqrt{q\,C_e}$ > 0.167
Smooth wires, rods, pipes	1.2	0.5
Mod. smooth wires and rods	1.2	0.7
Fine wire cables	1.2	0.9
Thick wire cables	1.3	1.1

EG00936B

Figure I-28
Poles, rods and wires

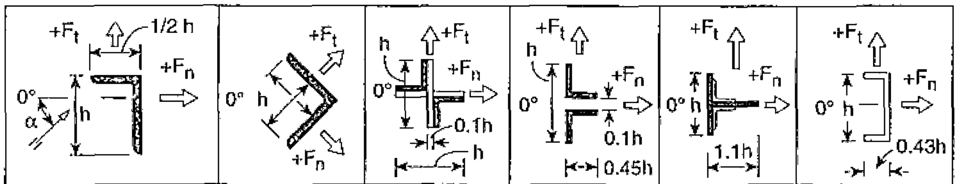
l = Length of member

$A = h \cdot l$ = Area

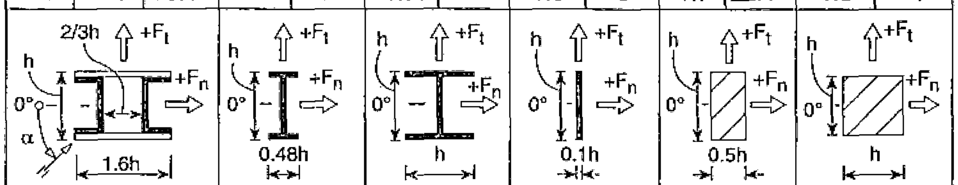
For wind normal to axis of member: Normal force $F_n = k \cdot C_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A$

Tangential force $F_t = k \cdot C_{t\infty} \cdot q \cdot C_g \cdot C_e \cdot A$

$C_{n\infty}$ and $C_{t\infty}$: Force coefficients for an infinitely long member

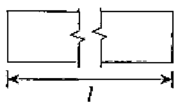


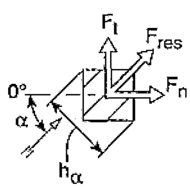
α	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$
0°	+1.9	+0.95	+1.8	+1.8	+1.75	+0.1	+1.6	0	+2.0	0	+2.05	0
45°	+1.8	+0.8	+2.1	+1.8	+0.85	+0.85	+1.5	−0.1	+1.2	+0.9	+1.85	+0.6
90°	+2.0	+1.7	−1.9	−1.0	−0.1	+1.75	−0.95	+0.7	−1.6	+2.15	0	+0.6
135°	−1.8	−0.1	−2.0	+0.3	−0.75	+0.75	−0.5	+1.05	−1.1	+2.4	−1.6	+0.4
180°	−2.0	+0.1	−1.4	−1.4	−1.75	−0.1	−1.5	0	−1.7	±2.1	−1.8	0



α	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$	$C_{n\infty}$	$C_{t\infty}$
0°	+1.4	0	+2.05	0	+1.6	0	+2.0	0	+2.1	0	+2.0	0
45°	+1.2	+1.6	+1.95	+0.6	+1.5	+1.5	+1.8	+0.1	+1.4	+0.7	+1.55	+1.55
90°	0	+2.2	±0.5	+0.9	0	+1.9	0	+0.1	0	+0.75	0	+2.0

For slenderness, h_α is to be used:





k : Reduction factor for members of finite slenderness (in general use full length not panel length)

l/h_α	5	10	20	35	50	100	∞
k	0.60	0.65	0.75	0.85	0.90	0.95	1.0

EG00937B

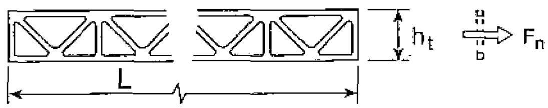
Figure I-29
Structural members, single and assembled sections

A_s = Section area

$A = h_t \cdot L$

A_s/A = Solidity ratio

For wind normal to surface A: Normal force $F_n = k \cdot C_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A_s$



$C_{n\infty}$: Force coeff. for an infinitely long truss, $0 \leq A_s/A \leq 1$

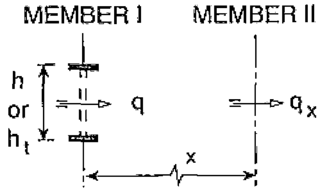
A_s/A	0	0.1	0.15	0.2	0.3 to 0.8	0.95	1.0
$C_{n\infty}$	2.0	1.9	1.8	1.7	1.6	1.8	2.0

k : Reduction factor for trusses of finite length and slenderness

L/h_t \ A_s/A	0.25	0.5	0.9	0.95	1.0
5	0.96	0.91	0.87	0.77	0.60
20	0.98	0.97	0.94	0.89	0.75
50	0.99	0.98	0.97	0.95	0.90
∞	1.0	1.0	1.0	1.0	1.0

EG00938B

Figure I-30
Plane trusses made from sharp-edged sections



$q_x = k_x \cdot q$

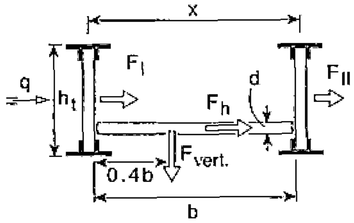
k_x SHIELDING FACTOR

x/h \ A_s/A	0.1	0.2	0.3	0.4	0.5	0.6	0.8	1.0
0.5	0.93	0.75	0.56	0.38	0.19	0	0	0
1	0.99	0.81	0.65	0.48	0.32	0.15	0.15	0.15
2	1.00	0.87	0.73	0.59	0.44	0.30	0.30	0.30
4	1.00	0.90	0.78	0.65	0.52	0.40	0.40	0.40
6	1.00	0.93	0.83	0.72	0.61	0.50	0.50	0.50

EG00939B

Figure I-31
Shielding factors

CASE I WITHOUT VEHICLES



L_B = Length of bridge

k, $C_{n\infty}$, A_s, k_x (1)

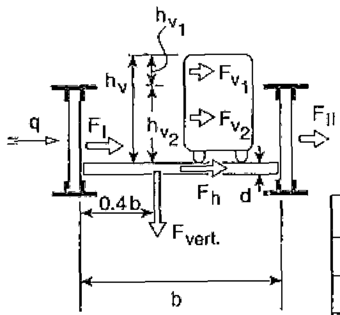
Windward girder $F_I = kC_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A_s$

Leeward girder $F_{II} = kC_{n\infty} \cdot k_x q \cdot C_g \cdot C_e \cdot A_s$

Deck horiz. load $F_h = 1.0 \cdot q \cdot C_g \cdot C_e \cdot L_B$

Deck vert. load $F_{vert.} = 0.6 \cdot q \cdot C_g \cdot C_e \cdot b \cdot L_B$

CASE II WITH VEHICLES



L_v = Length of vehicle; $A_1 = h_{v_1} \cdot L_v$

$A_2 = h_{v_2} \cdot L_v$

Windward girder $F_I = kC_{n\infty} \cdot q \cdot C_g \cdot C_e \cdot A_s$

Leeward girder $F_{II} = kC_{n\infty} \cdot k_x q \cdot C_g \cdot C_e \cdot A_s$

Deck horiz. load $F_h = 1.2 \cdot q \cdot C_g \cdot C_e \cdot d \cdot L_B$

Deck vert. load $F_{vert.} = 0.8 \cdot q \cdot C_g \cdot C_e \cdot b \cdot L_B$

Traffic load $F_{v_1} = C_n q \cdot C_g \cdot C_e \cdot A_1$

$F_{v_2} = C_n \cdot 2/3\, q \cdot C_g \cdot C_e \cdot A_2$

Height and force coefficients	h_v	C_n
Railway vehicle	3.8 m	1.5
Highway vehicle	3.0 m	1.2
Pedestrian	1.7 m	1.0

EG00940B

Figure I-32
Truss and plate girder bridges

Note to Figure I-32:

(1) The values for these coefficients are taken from Figures I-29 and I-30.

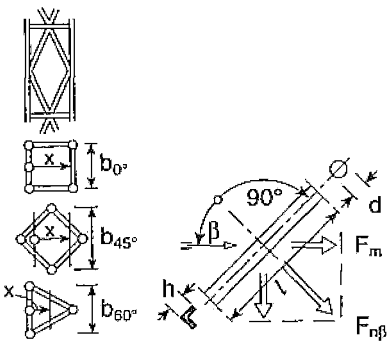
$A_s/A \le 0.3$

$A = d \cdot L$ or $h \cdot L$

L = true length of member

β = angle formed by wind direction and the normal to member axis

k_x = a function of A_s/A and x/b



TOTAL LOAD IN WIND DIRECTION $F = \Sigma F_m$

F_m = FORCE ON MEMBER

$F_m = k \cdot C_{\infty\beta} \cdot q \cdot C_g \cdot C_e \cdot A \cos\beta$

(Shielded member
$F_m = k \cdot C_{\infty\beta} \cdot k_x q \cdot C_g \cdot C_e \cdot A \cos\beta$)

Coeff. $C_{\infty\beta}$: For sharp-edged members $C_{\infty\beta} = k_\beta \cdot C_{n\infty}$ and $k_\beta \cdot C_{t\infty}$

Coeff. $C_{\infty\beta}$, k_β, k, k_x(1)

β	SHARP-EDGED MEMBERS			ROUND MEMBERS, SMOOTH and ROUGH SURFACES, $d\sqrt{qC_e} < 0.167$			ROUND MEMBERS, MODERATELY SMOOTH SURFACES, $d\sqrt{qC_e} < 0.167$		
	k_β	k	k_x	$C_{\infty\beta}$	k	k_x	$C_{\infty\beta}$	k	k_x
0°	1.00	(2)	(3)	1.20	(2)	(3)	0.60	0.9 for $l/d = 25$	0.95 constant
15°	0.98			1.16			0.58		
30°	0.93			1.04			0.53		
45°	0.88			0.85			0.42		
60°	0.80			0.60			0.28		

EG00941B

Figure I-33
Three-dimensional trusses

Notes to Figure I-33:
(1) See Figure I-29 for $C_{n\infty}$ and $C_{t\infty}$ values.
(2) See Figure I-29.
(3) See Figure I-31.

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