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Wind actions on structures

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

International Standards are drafted in accordance with the rules given in the ISO/IEC Directives, Part 2.

The main task of technical committees is to prepare International Standards. Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights.

ISO 4354 was prepared by Technical Committee ISO/TC 98, *Bases for design of structures*, Subcommittee SC 3, *Loads, forces and other actions*.

This second edition cancels and replaces the first edition (ISO 4354:1997), which been technically revised.

Introduction

This International Standard is intended for use by countries without an adequate wind loading standard and as a bridge between existing International Standards. The data in the annexes, with the exception of Annex A, whilst formally only informative, and limited to the most common usage, are intended for use within the definitions in this International Standard. Additional data will be provided from time to time in ISO Technical Reports for use on the same basis.

Wind actions on structures

1 Scope

This International Standard describes the actions of wind on structures and specifies methods of calculating characteristic values of wind loads for use in designing buildings, towers, chimneys, bridges and other structures, as well as their components and appendages. The loads are suitable for use in conjunction with ISO 2394 and other International Standards concerned with wind loads. In particular, this International Standard facilitates the conversion between peak and mean wind speed methodologies and covers the three main storm types, synoptic winds, thunderstorms and tropical cyclones (hurricanes and typhoons).

This International Standard provides the basic methods from which to determine wind loading analytically through the determination of design pressures or orthogonal along-wind and cross-wind forces and moments for structures of simple shape and wind directionality effects, and through wind tunnel or computational determinations of pressure, forces and moments for structures with complex shapes and wind directionality effects resulting in complex combinations of forces and moments.

Structures of unusual nature, size or complexity (e.g. tall buildings, long span bridges, large span roofs, guyed masts, offshore and moving structures) typically require a special engineering study; some guidance is given on the limitations of this International Standard in these cases.

Two methods of analytical determination of design wind loads are given in this International Standard, one based on a peak velocity and the other on a mean velocity. Both methods can be used when dynamic response effects are important, and where they are not important only the peak-velocity method is used in this International Standard by taking the peak dynamic response factor to be unity. To simplify presentation, the method based on the peak velocity is given in the main body of this International Standard and the method based on the mean velocity is given in a normative Annex A.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 2394, *General principles on reliability for structures*

3 Symbols

Symbol	Term	Unit
A	Tributary or local area (area of application of pressure coefficient C_p)	m^2
A_{ref}	Reference area for force on overall structure or part of structure	m^2
C_{dyn}	Peak dynamic response factor	1
$C_{dyn, m}$	Mean dynamic response factor	1
C_{exp}	Peak exposure factor	1
$C_{exp, m}$	Mean exposure factor	1
C_F	Force coefficient	1
C_{Fm}	Mean force coefficient	1
C_p	Pressure coefficient (time and spatially averaged)	1
$C_{\sigma F}$	Standard deviation force coefficient	1
F	Peak force	N
F_{loc}	Peak force on a tributary or local area	N
F_m	Mean force	N
g	Peak factor	1
g_v	Wind speed peak factor	1
h	Height	m
I_v	Wind speed turbulence intensity	1
p	Pressure	Nm^{-2}
$q_{ref, m}$	Regional reference mean dynamic pressure	Nm^{-2}
q_{site}	Site peak dynamic pressure	Nm^{-2}
$q_{site, m}$	Site mean dynamic pressure	Nm^{-2}
V	Peak wind speed	ms^{-1}
V_{hcr}	Critical wind speed at the top of the structure	ms^{-1}
V_m	Mean wind speed	ms^{-1}
V_{ref}	Regional peak reference wind speed (with return period)	ms^{-1}
$V_{ref, m}$	Regional mean reference wind speed	ms^{-1}
V_{site}	Site peak velocity	ms^{-1}
$V_{site, m}$	Site mean velocity	ms^{-1}
σ_F	Standard deviation of force	N

4 Wind actions

Wind actions that shall be considered in the design of the structure can produce the following:

a) excessive forces or instability in the structure or its structural members or elements;

b) excessive deflection or distortion of the structure or its elements;

c) repeated dynamic forces causing fatigue of structural elements;

d) aeroelastic instability, in which motion of the structure in wind produces aerodynamic forces augmenting the motion;

e) excessive dynamic movements causing concern or discomfort to occupants or onlookers;

f) effects of interference from existing and potential future buildings.

NOTE Wind pressure and force given in this International Standard are equivalent static wind loads and not pure external excitations. As the equivalent static wind loads are essentially based on linear elastic building and structure behaviour, it is necessary to give careful attention if they are applied to design in the plastic region.

5 Wind pressure

For the actions referred to in Clause 4 a), b), c) and e), the effective wind pressure, p, shall be determined from a relationship incorporating the site dynamic pressure, q_{site}, defined in Clause 7 and Clause 8, a pressure coefficient, C_{p}, and a dynamic response factor, C_{dyn}, of the general form of Equation (1):

$$p = q_{\text{site}} \times C_{\text{p}} \times C_{\text{dyn}} \tag{1}$$

The wind pressure is assumed to act statically in a direction normal to the surface of the structure or element, except where tangential frictional forces are specifically identified. Both internal and external pressures shall be considered. Integration of pressures shall be undertaken to obtain global forces or forces for defined tributary areas.

The effects of wind from all directions shall be considered.

6 Wind force

For some structures, it may be appropriate to represent the wind forces, F, by their resultants. These resultants shall include along-wind (drag), cross-wind (lift), torsional and overturning actions. Different magnitudes and distributions of the wind force can be necessary to evaluate the actions described in Clause 4 a), b), c) and e).

The derivation of effective wind forces on an element, or resultant forces and moments, shall be determined by using either the peak reference dynamic pressure given here, or the mean reference dynamic pressure given in Annex A.

The peak reference pressure method assumes that the dynamic effects can be represented by a maximum or peak loading effect based on a peak reference pressure combined with a mean pressure coefficient, (or mean pressure coefficient modified for local effects relating to area of application and statistical characteristics), and a peak dynamic response factor, C_{dyn}, from the general relationships given in Equations (2) and (3):

$$F_{\text{loc}} = q_{\text{site}} \times C_{\text{p}} \times C_{\text{dyn}} \times A \tag{2}$$

$$F = q_{\text{site}} \times C_{\text{F}} \times C_{\text{dyn}} \times A_{\text{ref}} \tag{3}$$

Equation (2) is used for the force on a tributary or local area, A. Equation (3) is used for the total force on the whole structure or part of the structure. The value of C_{dyn} can be taken as 1,0 except where the structure is dynamically wind-sensitive, as described in Annex E.

In many cases total loads on whole structures will be determined from loads determined for various components, or facades, or from along-wind and cross-wind components. These forces contribute simultaneously but are not usually well correlated. Methods for determining load combinations are given in Annex G.

It is important when using the dynamic method that the ways in which the equivalent static wind forces are distributed include not only the mean and fluctuating (background) wind forces acting on the structure's exterior, but also the inertial forces due to motions of the structure's mass.

7 Site peak dynamic pressure

The site peak dynamic pressure, q_{site}, shall be determined from the regionally derived reference wind speed, V_{ref}, along with appropriate exposure factors, C_{exp}, relating to wind speed for the site defined by the expression:

$$q_{site} = 0{,}5 \times \rho (V_{site})^2 \quad (4)$$

In Equation (4) the peak design wind speed locally at the site, adjusted for local exposure conditions, is given by Equation (5):

$$V_{site} = V_{ref} C_{exp} \quad (5)$$

where the exposure factor is determined as described in Clause 8.

The reference wind speed is normally the specified value of the wind speed for the geographical area in which the structure is located. It refers to a standard exposure (i.e. roughness, height and topography), averaging time and probability of excedence in one year (which can be approximated by an average return period for design application as required from serviceability to ultimate limit state determinations). In some situations, the reference wind speed can be specified as varying with direction. In the annexes of this International Standard the standard exposure is at 10 m height in open country terrain and q_{site} is based on a maximum 3 s mean gust wind speed, V.

Analysis procedures and values are given in Annex B and Annex C.

In certain cases, critical loading can occur at wind speeds differing from, and perhaps lower than, that specified above (e.g. due to vortex shedding). These critical wind speeds (with reference to height h) are denoted V_{hcr} and substituted for V_{site}. These cases are discussed in Annex E.

8 Exposure factor

The exposure factor, C_{exp}, relating to wind speed, accounts for the variability of the wind speed at the site of the structure for each storm type, due to

a) the height above ground level,

b) the roughness of the terrain (including change of roughness), and

c) the topography.

Values of the exposure factor are given in Annex C, and can vary with wind direction. Further guidance on the application of directional design wind speeds is given in Annex C.

9 Pressure and force coefficients

A pressure coefficient, C_p, is an aerodynamic wind-induced pressure expressed as a fraction of the reference pressure. A force coefficient, C_F, is an aerodynamic wind-induced force expressed as a ratio of the aerodynamic force exerted on a structure or its parts to a reference pressure multiplied by a reference area.

Pressure coefficients are specified as appropriate fractile values of the respective extreme actions. The fractile value to be used is defined in Annex D.

Pressure and force coefficients are influenced by the shape of the structure, the exposure, the relative wind direction, the Reynolds number and the averaging time. Values of pressure and force coefficients are presented in Annex D in tables as non-simultaneous values for the design of cladding or parts of the structure and in figures as simultaneous distributions for the design of the load-bearing structure.

Enclosed structures are subject to internal pressures determined by the size and distribution of the openings in the building envelope and by any pressurization, mechanical or otherwise. Allowance shall be made for these by combining pressure coefficients for the external pressures with those for the internal pressures.

Pressure and force coefficients shall be determined from one of the following sources:

a) Annex D;

b) appropriate wind tunnel tests, as described in Annex H;

c) appropriate computationally based data, as described in Annex I;

d) other codes or standards, provided that appropriate adjustment is made for any discrepancies, e.g. in averaging time and exposure from those used in this International Standard, and provided that adequate provision is made for a dynamic response factor.

10 Dynamic response factor

The dynamic response factor, C_{dyn}, accounts for the following actions of the wind:

a) fluctuating pressures due to random wind gusts acting over all or a part of the surface area of the structure;

b) fluctuating pressures in the wake of the structure (vortex shedding forces), producing resultant forces acting cross-wind, as well as torsionally and along-wind;

c) fluctuating pressures induced by the motion of the structure due to wind.

Information on these effects and appropriate values of the dynamic response factor is given in Annex E.

11 Criterion for aeroelastic instability

For structures affected by the wind actions specified in Clause 4 d) that cause aeroelastic instability, it is necessary to show that the performance of the structure, without further application of a load factor, is acceptable up to a wind speed somewhat higher than the design peak wind speed for the structure. Unless alternative rational procedures are available, this wind speed shall be taken as no less than the ultimate limit state site peak velocity, V_{site}, or 1,5 times the ultimate limit state 10-min mean design wind speed for the site location and height of the structure.

12 Methods of determination of wind loads

Two methods of determining design wind loads are given in this International Standard, one based on a peak velocity pressure and the other on a mean velocity pressure. Both methods are intended for use in a detailed way when dynamic response effects are important with an appropriate value of either a peak or mean dynamic response factor, guidance for which is given in Annex E. When dynamic response effects are not important, such as for the design of cladding for most typical structures and the design of the main structural systems of small- to medium-sized structures with little dynamic response effect, the peak velocity pressure method is intended for use in a simplified way by taking the peak dynamic response factor, C_{dyn}, to be unity.

For certain wind-sensitive structures specialist supplementary studies are recommended. Structures sensitive to wind include those which are particularly flexible, slender, tall or of light weight and those within complex surroundings. Unusual geometry is able to also give rise to an unexpectedly large response to wind. In these instances, supplementary studies by an expert in the field are recommended and these typically include wind tunnel tests. These tests can be used to establish details of the overall structural loads and the distribution of external local pressures. Details of suitable testing procedures are given in Annex H.

Both methods of analysis are intended for use with performance-based and limit state design methods, the requirements for which are introduced at the choice of an appropriate probability of excedance for the regional reference wind speed, defined in Clause 7.

This International Standard may be used to interpret between national and regional wind loading standards by using the relationships given in Annex B and Annex C. In using this International Standard or interpreting between national standards, it is essential that account be taken of the storm type most applicable to both the ultimate limit state and the serviceability design.

Alternative methods of analysis to those recommended in this International Standard may be permitted provided it can be demonstrated that the level of safety achieved is generally equivalent to that achieved in this International Standard. Guidance on the level of safety is given in Annex J.

Annex A
(normative)

Mean velocity method

A.1 General

Two methods for analytical determination of design wind loads are given in this International Standard, one based on a peak velocity that has been presented in the main body of this International Standard, and the other on a mean velocity that is to be presented in this normative Annex A. The mean velocity method has its main application in the determination of dynamic response effects because it transparently represents the response of the structure in terms of mean and fluctuating components, which is a more accurate description of the response process and more appropriate when considering complex structures requiring load distributions with height and load combinations as used in determinations based on wind tunnel model measurements.

A.2 Wind force analysis procedures

The mean velocity method assumes that the dynamic effects can be represented by a general expression for the peak loading effect, F, made up from the addition of a mean force component, F_{m}, and a fluctuating force component made up from the product of a peak factor, g, and the standard deviation of the force, σ_{F}, as given in Equation (A.1):

$$F = F_{\mathrm{m}} + g \times \sigma_{\mathrm{F}} \tag{A.1}$$

where the mean and standard deviation forces can be obtained using mean and standard deviation force coefficients, C_{Fm} and $C_{\sigma_{\mathrm{F}}}$, as given in Equations (A.2) and (A.3):

$$F_{\mathrm{m}} = q_{\mathrm{site,\,m}} \times C_{F\mathrm{m}} \times A_{\mathrm{ref}} \tag{A.2}$$

and

$$\sigma_{\mathrm{F}} = q_{\mathrm{site,\,m}} \times C_{\sigma\mathrm{m}} \times A_{\mathrm{ref}} \tag{A.3}$$

To facilitate the conversion to the peak dynamic response factor, C_{dyn}, a mean dynamic response factor, $C_{\mathrm{dyn,\,m}}$, (often referred to as a gust effect factor for along-wind forces) can be given as Equation (A4):

$$C_{\mathrm{dyn,\,m}} = F/F_{\mathrm{m}} = 1 + g \times \sigma_{\mathrm{F}}/F_{\mathrm{m}} \tag{A.4}$$

And for local force, F_{loc}, on tributary or local area, A, as Equation (A.5):

$$F_{\mathrm{loc}} = F_{\mathrm{m}} \times C_{\mathrm{dyn,\,m}} = q_{\mathrm{site,\,m}} \times C_{\mathrm{p}} \times C_{\mathrm{dyn,\,m}} \times A \tag{A.5}$$

NOTE C_{p} in this application is the actual mean value and not a modified mean value.

For overall force, F, on a structure with reference area, A_{ref}, Equation (A.6) is used as follows:

$$F = F_{\mathrm{m}} \times C_{\mathrm{dyn,\,m}} = q_{\mathrm{site,\,m}}\ C_{\mathrm{Fm}} \times C_{\mathrm{dyn,\,m}} \times A_{\mathrm{ref}} \tag{A.6}$$

The peak and mean dynamic response factors can be linked through the relationship between the peak and mean wind speeds with the introduction of a wind speed peak factor, g_v, and turbulence intensity, I_v, as given in Equations (A.7) and (A.8):

$$V = V_m (1 + g_v I_v) \quad \text{(A.7)}$$

$$C_{dyn} = C_{dyn,\,m} / (1 + g_v I_v)^2 \quad \text{(A.8)}$$

[and as an approximation, for low turbulence intensity, $C_{dyn} = C_{dyn,\,m} / (1 + 2g_v I_v)$]

A.3 Site mean dynamic pressure

The site mean dynamic pressure, $q_{site,\,m}$, shall be determined from regionally derived mean reference wind speed, $V_{ref,\,m}$, along with appropriate exposure factors, $C_{exp,\,m}$, relating to wind speed, for the site defined using Equation (A.9):

$$q_{site,\,m} = 0,5 \times \rho\,(V_{site,\,m})^2 \quad \text{(A.9)}$$

where

$$V_{site,\,m} = V_{ref,\,m} \times C_{exp,\,m} \quad \text{(A.10)}$$

In Equation (A.10), $V_{site,\,m}$ is the mean design wind speed relating to the local exposure conditions and height of the structure as described in A.4.

The reference wind speed is normally the specified value of the wind speed for the geographical area in which the structure is located. It refers to a standard exposure (i.e. roughness, height and topography), averaging time and probability of excedence in one year (which can be approximated by an average return period for design application as required from serviceability to ultimate limit state determinations). In some situations, the reference wind speed can be specified as varying with direction. In the Annexes of this International Standard the standard exposure is at 10 m height in open country terrain and $q_{ref,\,m}$ is based on a maximum 10-min mean wind speed, $V_{ref,\,m}$.

Analysis procedures and values are given in Annex B and Annex C.

In certain cases, critical loading can occur at wind speeds differing from that specified above (e.g. due to vortex shedding). These critical wind speeds (with reference to height, h) are denoted V_{hcr} and are substituted for V_{ref}. These cases are discussed in Annex E.

A.4 Mean exposure factor

The mean exposure factors, $C_{exp,\,m}$, relating to mean wind speed account for the variability of the mean wind speed at the site of the structure, for each storm type, due to

a) the height above ground level,

b) the roughness of the terrain (including change of roughness), and

c) the topography.

Values of the mean exposure factor are given in Annex C and can vary with wind direction. Further guidance on the application of directional design wind speeds is given in Annex C.

Annex B
(informative)

Determination of reference wind speed

B.1 General

Clause 7 defines site peak and mean dynamic pressures in terms of reference wind speeds and exposure factors. The reference wind speeds for any probability of excedance in one year (average return period) shall be determined from regionally derived reference wind speeds, and for use in this International Standard the two reference wind speeds will be referenced to standard averaging times and exposure, defined as follows:

— V_{ref} is the maximum wind speed averaged over 3 s referenced to a height of 10 m over flat open country terrain;

— $V_{\text{ref, m}}$ is the maximum mean wind speed averaged over 10 min referenced to a height of 10 m over flat open country terrain.

Many national standards are based on annual extremes which can be appropriate where a single storm type causes the extremes. However, generally, yearly extremes do not form an appropriate basis for the extreme value analysis of wind speeds. This is especially true if the respective storm phenomenon tends to occur in families or clusters. Then, in a specific (calendar) year more than one event corresponding to the analysed storm type might be obtained, while in another year no event of the analysed storm type has occurred. In such cases, the ensemble of yearly extremes contains irrelevant data and neglects other relevant data. The ensemble therefore should consist of independent extremes above an appropriate threshold for each storm type.

B.2 Analysis procedures

The defined reference wind speeds shall be obtained from regional wind speeds, which, if not referenced as given in the definitions in Clause B.1, shall be converted by the following procedures.

a) Conversion for wind speeds referenced to different terrain roughness or height, $V_{\text{tr, z}}$, shall use the exposure factors, C_{exp}, $C_{\text{exp, m}}$, given in Annex C for the appropriate storm type as given in Equations (B.1) and (B.2):

$$V_{\text{ref}} = \frac{V_{\text{tr, z}}}{C_{\text{exp}}} \tag{B.1}$$

$$V_{\text{ref,m}} = \frac{V_{\text{tr, z, m}}}{C_{\text{exp, m}}} \tag{B.2}$$

b) Conversion to wind speeds, V_T, referenced for different averaging times, T, shall use an averaging time factor, k_T, which is based on a peak factor, g_{v}, and the turbulence intensity, I_{v}, as given in Equations (B.3) and (B.4):

$$V_T = k_T V_{T=3\,600\text{ s}} \tag{B.3}$$

$$k_T = 1 + g_{\text{v}} I_{\text{v}} \tag{B.4}$$

k_T relates to the maximum wind speed averaged over a given period of T seconds within the hourly mean wind speed for a given hour.

g_v is a peak factor that depends primarily on the time over which the maximum wind speed is averaged, T, and weakly dependent on height. A full derivation methodology is given in References [2], [3] and [4], as summarized in Reference [5]. For this International Standard, average values of g_v for heights between 3 m and 300 m are given in Table B.1 along with an evaluation of k_T for a range of values of T for the specific reference conditions of z = 10 m and z_0 = 0,03 m.

I_v is the turbulence intensity defined as the standard deviation of wind speed divided by the hourly mean wind speed, using Equation (B.5):

$$I_v = \frac{\sigma_v}{V_{m,\,T=3\,600\text{ s}}} \tag{B.5}$$

Table B.1 — Averaging peak factors, g_v, and evaluation of reference averaging time factor, $k_{T,\,z}$

Averaging time T s	Average peak factor g_v	Reference averaging time factor $k_{T,}$ z = 10 m, z_0 = 0,03 m
1	3,90	1,62
3	3,00	1,53
10	2,40	1,42
30	1,65	1,27
100	0,90	1,15
600	0,28	1,05
3 600	0	1,00

The most common conversions needed for reference conditions in relation to averaging time, T, are:

— $V_{\text{ref},\,T=3\text{ s}} = 1{,}53\ V_{\text{ref},\,T=3\,600\text{ s}}$

— $V_{\text{ref},\,T=600\text{ s}} = 1{,}05\ V_{\text{ref},\,T=3\,600\text{ s}}$

— $V_{\text{ref},\,T=3\text{ s}} = 1{,}46\ V_{\text{ref},\,T=600\text{ s}}$

Anemometer measurements might not have been made in the open country reference conditions of z_0 = 0,03 m and more detailed correction might be needed to obtain the appropriate peak factor as well as the conversions given by Equations (B.1) and (B.2), such as using Equation (C.3) with Figure C.1.

The hourly mean wind speed is meaningful for synoptic storms, but for tropical cyclone storms and thunderstorms there are no statistically stationary values of the hourly mean wind speed. In these latter cases an equivalent 10 min mean wind speed can be used in this International Standard to facilitate the determination of the dynamic response, although the use of the 3 s wind speed is recommended in thunderstorm climates. In addition, historical wind speed data in thunderstorm climates need to be referenced to averaging times of 3 s or less for the meaningful determination of extreme value design wind speeds.

Annex C
(informative)

Determination of exposure factors

C.1 General

Clause 8 defines the exposure factors in terms of the variation of the reference wind speeds and consequently, the site peak and mean velocity pressures, with height, terrain roughness, change in terrain roughness and topography for the various storm types. The exposure factors will be made up of factors for each of these phenomena as shown in Equations (C.1) and (C.2):

$$C_{\text{exp}} = k_{\text{tr},\, z} \times k_{\text{trchange}} \times k_{\text{topog}} \tag{C.1}$$

$$C_{\text{exp, m}} = k_{\text{tr},\, z,\, \text{m}} \times k_{\text{trchange, m}} \times k_{\text{topog, m}} \tag{C.2}$$

where

$k_{\text{tr},\, z}$ is the peak terrain roughness and height exposure factors;

$k_{\text{tr},\, z,\, \text{m}}$ is the mean terrain roughness and height exposure factors;

k_{trchange} is the peak terrain roughness change exposure factors;

$k_{\text{trchange, m}}$ is the mean terrain roughness change exposure factors;

k_{topog} is the peak topography exposure factors;

$k_{\text{topog, m}}$ is the mean topography exposure factors;

z is the height above ground level.

C.2 Wind profiles over flat terrain

Values of the terrain roughness and height exposure factors will be given for three storm types, along with values of turbulence intensity.

C.2.1 Synoptic storm profiles

Synoptic storm hourly mean wind speed and turbulence intensity profiles for four terrain roughness categories will be defined by the following logarithmic law relations (after Reference [2]) to fit roughness length values of $z_0 = 0{,}003$, 0,03, 0,3 and 3,0 m, defined as terrain roughness categories 1, 2, 3 and 4 respectively, and a gradient height (z_{G}) hourly mean wind speed of 50 ms^{-1}. See Equations (C.3) to (C.10):

$$V_{\text{tr},\, z,\, T = 3\,600\ \text{s}} = \frac{u_*}{0{,}4}\left\{\ln\left(\frac{z}{z_0}\right) + 5{,}75\left(\frac{z}{z_{\text{G}}}\right) - 1{,}88\left(\frac{z}{z_{\text{G}}}\right)^2 - 1{,}33\left(\frac{z}{z_{\text{G}}}\right)^3 + 0{,}25\left(\frac{z}{z_{\text{G}}}\right)^4\right\} \tag{C.3}$$

$$z_{\text{G}} = \frac{u_*}{6f} \tag{C.4}$$

$$\sigma_{\mathrm{v}} = \frac{7{,}5\eta u_{*}\left[0{,}538 + 0{,}09\ln\left(\frac{z}{z_0}\right)\right]^{\eta^{16}}}{1 + 0{,}156\ln\left(\frac{u_{*}}{fz_0}\right)} \tag{C.5}$$

$$\eta = 1 - \frac{6fz}{u_{*}} \tag{C.6}$$

$$f = 2\Omega\sin\phi \tag{C.7}$$

where

ϕ is the latitude,

Ω is angular velocity of the Earth's rotation (= $72{,}9 \times 10^{-6}$ radians/s),

u_{*} is the frictional velocity $\left(\sqrt{\frac{\text{surface friction shear stress}}{\text{atmospheric air density}}}\right)$.

$$I_{\mathrm{v},z} = \frac{\sigma_{\mathrm{v},z}}{V_{z,\,T=3\,600\ \mathrm{s}}} \tag{C.8}$$

(i.e., turbulence intensity based on hourly mean wind speed at a specific location of height and terrain roughness)

$$V_{\mathrm{tr},z,\,T=3\ \mathrm{s}} = V_{z,\,T=3\,600\ \mathrm{s}}\left(1 + 3{,}0 I_{\mathrm{v},z}\right) \tag{C.9}$$

$$V_{\mathrm{tr},z,\,T=600\ \mathrm{s}} = V_{z,\,T=3\,600\ \mathrm{s}}\left(1 + 0{,}28 I_{\mathrm{v},z}\right) \tag{C.10}$$

Values of z_0 for various terrain roughness conditions are given in Figure C.1.

Some evaluations of the terrain roughness and height exposure factors and turbulence intensities for synoptic storm profiles are given in Table C.1 for a latitude of 40°. The Deaves and Harris equations are not valid for latitudes approaching zero degrees and would not normally be used for values of $\phi < 20$ °. The evaluations in Table C.1 differ slightly from those defined in C.1 because the Deaves and Harris equations are based on a mean averaging time of T = 3 600 s, as used in some countries. Hence the evaluation of the mean wind speed profiles have been carried out for T = 3 600 s, then converted to the peak and mean reference times used in this International Standard of T = 3 s and T = 600 s, respectively. Further, because of the calculation method used, the terrain roughness and height exposure factors in Table C.1 are all given as a ratio with the peak reference wind speed, V_{ref}, which is for $T = 3$ s at $z = 10$ m in open country terrain, as given in Equations (C.11), (C.12) and (C.13):

$$k_{\mathrm{tr},z} = \frac{V_{\mathrm{tr},z}}{V_{\mathrm{ref}}} \tag{C.11}$$

$$k_{\mathrm{tr},z,\mathrm{m}} = \frac{V_{\mathrm{tr},z,\mathrm{m}}}{V_{\mathrm{ref}}} \tag{C.12}$$

$$k_{\mathrm{tr},z,\,T=3\,600\ \mathrm{s}} = \frac{V_{\mathrm{tr},z,\,T=3\,600\ \mathrm{s}}}{V_{\mathrm{ref}}} \tag{C.13}$$

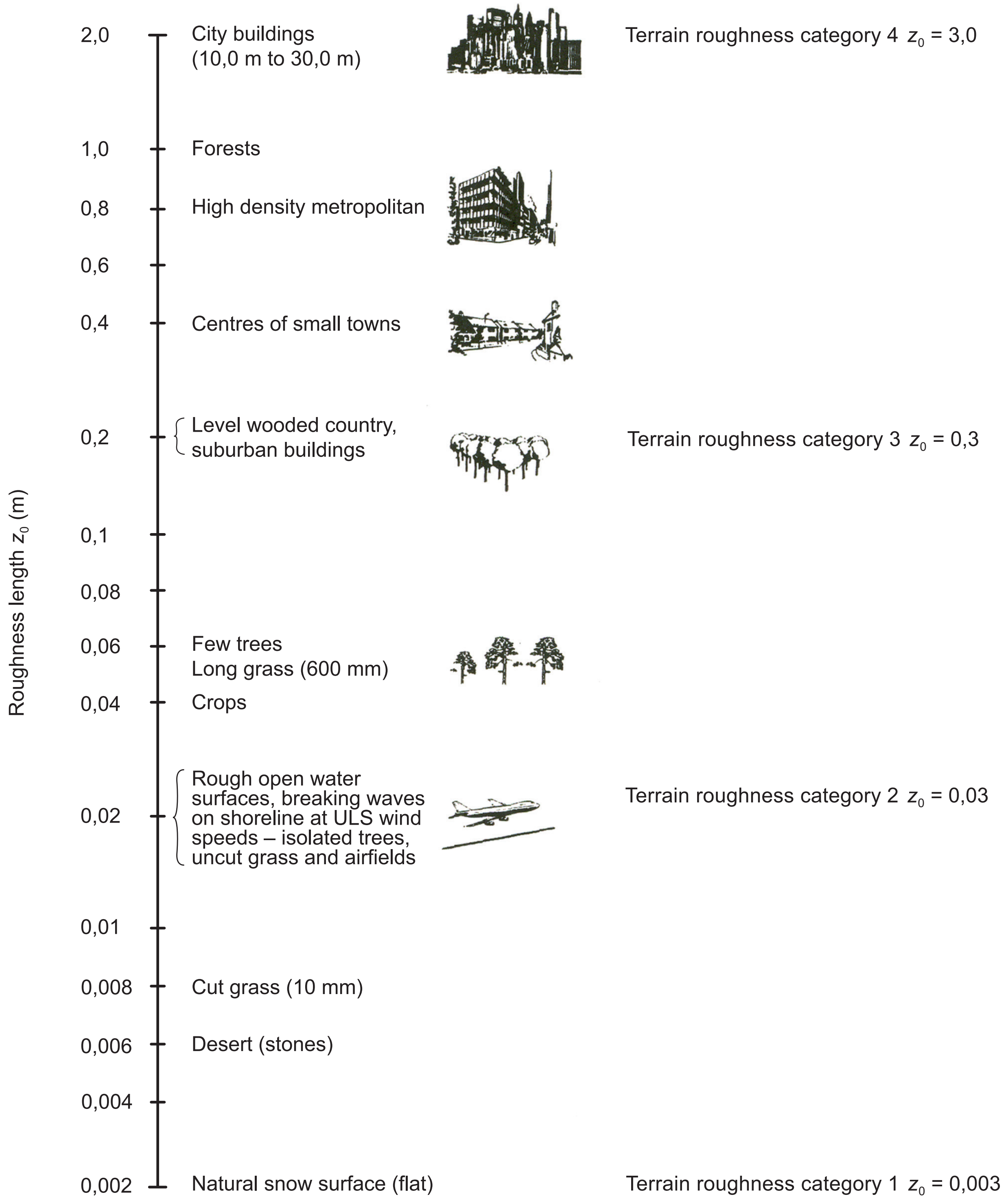
Where, as specified in this International Standard any parameter without a definition for averaging time, T, is a peak value for $T = 3$ s.

If the mean terrain roughness and height exposure factors are required, they can be obtained from the ratio of the factors in Table C.1, with either the $T = 600$ s ou $T = 3\ 600$ s factor at $z = 10$ m in open country terrain, as appropriate.

The logarithmic law profiles can be approximated by power law profiles with only the one variable for terrain roughness category being the power law exponent, β, as given in Equation (C.14):

$$V_{\text{tr},\,z} = V_{\text{tr},\,z=10\ \text{m}} \left(\frac{z}{10}\right)^{\beta} \text{ and } V_{\text{tr},\,z,\,\text{m}} = V_{\text{tr},\,z=10\ \text{m},\,\text{m}} \left(\frac{z}{10}\right)^{\beta_\text{m}} \quad \text{(C.14)}$$

Values for β and β_m have been fitted between 10 m and 200 m and are given in Table C.1.



Key

z_0 roughness length, expressed in metres

Figure C.1 — Description of terrain roughness lengths, z_0

Table C.1 — Terrain roughness and height exposure factors, $k_{tr,z}$ and $k_{tr,z,m}$, and turbulence intensity profiles for four terrain roughness categories over flat terrain for synoptic storms for latitude $\phi = 40°$

Terrain roughness category	Characteristics				
	Height z m	$k_{tr,z}$ (T = 3 s)	$k_{tr,z,m}$ (T = 600 s)	$k_{tr,z}$ (T = 3 600 s)	Turbulence intensity $I_{v,z,T=3\,600\,s}$
1. Open sea flat surface z_0 = 0,003 m	3	0,97	0,70	0,67	0,148
	5	1,03	0,75	0,72	0,142
	10	1,11	0,82	0,79	0,135
	20	1,19	0,89	0,86	0,127
	50	1,28	0,99	0,96	0,112
	100	1,33	1,07	1,04	0,095
	200	1,39	1,15	1,13	0,076
	500	1,49	1,31	1,29	0,052
	1 000	1,58	1,46	1,44	0,032
β	—	β = 0,074	β_m = 0,113	$\beta_{T=3\,600}$ = 0,120	—
2. Open country/ open sea in ultimate limit state conditions z_0 = 0,03 m	3	0,83	0,55	0,52	0,203
	5	0,90	0,61	0,58	0,191
	10	1,00	0,69	0,655	0,178
	20	1,10	0,77	0,73	0,165
	50	1,21	0,88	0,85	0,147
	100	1,29	0,97	0,94	0,128
	200	1,36	1,07	1,04	0,106
	500	1,48	1,23	1,21	0,074
	1 000	1,58	1,40	1,38	0,048
β	—	β = 0,103	β_m = 0,147	$\beta_{T=3\,600}$ = 0,154	—
3. Suburban z_0 = 0,3 m	3	0,84	—	—	—
	5	0,84	0,40	0,37	0,311
	10	0,84	0,50	0,47	0,269
	20	0,96	0,60	0,56	0,239
	50	1,12	0,73	0,69	0,208
	100	1,23	0,83	0,79	0,184
	200	1,33	0,95	0,91	0,156
	500	1,47	1,13	1,10	0,111
	1 000	1,58	1,32	1,29	0,075
β	—	β = 0,152	β_m = 0,214	$\beta_{T=3\,600}$ = 0,220	—
4. Urban z_0 = 3,0 m	3	0,59	—	—	—
	5	0,59	—	—	—
	10	0,59	0,23	0,20	0,677
	20	0,74	0,35	0,31	0,473
	50	0,95	0,51	0,46	0,355
	100	1,12	0,64	0,59	0,302
	200	1,27	0,77	0,72	0,254
	500	1,46	0,99	0,94	0,184
	1 000	1,59	1,20	1,16	0,126
β	—	β = 0,256	β_m = 0,403	$\beta_{T=3\,600}$ = 0,428	—

NOTE Values in terrain roughness categories 3 and 4 below 5 m and 10 m have been left conservatively constant or blank as these heights are close to or below the actual roughness elements in these categories. It is necessary to determine the effects of shielding from wind tunnel measurements of relevant references.

C.2.2 Tropical cyclone storm profiles

Research into wind speed profiles for tropical cyclones (typhoons or hurricanes) has produced a wide scatter of results over the last few decades. A recent review, in Reference [9], has recommended that there is not enough evidence to use other than logarithmic-law (or power-law) profiles near the ground (up to 500 to 1 000 m). Hence, it is recommended that terrain roughness category 2 given in Table C.1 be used for tropical cyclones.

Measurements by References [6] and [7] and studies by Reference [8] have shown episodes of relatively high values of turbulence intensity embedded in tropical cyclone records. In particular, the vertical components relative to the longitudinal components are significantly higher than for a neutral boundary layer flow. During these episodes there is evidence that the increased energy occurs in the inertial subrange, hence the averaging time factor given in Table B.1 is not applicable to these episodes.

C.2.3 Thunderstorm profiles

Whilst the general characteristics of thunderstorms and downbursts are well known, the measurements of profile data are very sparse. Measurements and some unpublished data have enabled a preliminary envelope of peak wind speeds to be developed for this International Standard, but more information, such as that currently being collected at Texas Tech University[1)], will be important to continuing validation. An enveloping profile of peak wind speeds for thunderstorms is given using Equation (C.15):

$$k_{\mathrm{tr},\,z,\,\mathrm{peak}} = 0{,}821 + 7{,}55 \times 10^{-4} z - 6{,}75 \times 10^{-6} z^2 + 1{,}06 \times 10^{-8} z^3 - 4{,}97 \times 10^{-12} z^4 + 0{,}079 \ln(z - 1{,}4) \qquad \text{(C.15)}$$

Information on effective turbulence intensity in severe thunderstorm downdrafts is given in Reference [10].

Some evaluation of this enveloping profile is given in Table C.2. No guidance can be given with respect to turbulence intensities in thunderstorms at this stage; only the synoptic storm turbulence intensities for the appropriate terrain are available at present.

Table C.2 — Terrain roughness and height exposure factors for peak wind speeds, $k_{\mathrm{tr},\,z}$, for three terrain roughness categories for thunderstorms

Terrain roughness category	Height Z m	$k_{\mathrm{tr},\,z}$
1, 2 and 3 Open sea, country and suburban	3	0,86
	5	0,93
	10	1,00
	20	1,06
	50	1,15
	100	1,20
	200	1,20
	500	1,02
	1 000	1,00

1) Wind Science and Engineering Research Centre, Texas Tech University, Lubbock, Texas, USA.

For thunderstorm environments only the peak wind speeds are relevant for the determination of design wind speeds. Use of peak wind speeds for the determination of design loads is recommended, although an artificial mean wind speed approach could be used for dynamic analysis. It is noted that in many cases for tall structures the thunderstorm data might only control the ultimate limit state design loads and at the serviceability levels the synoptic storm winds are likely to control. In all mixed storm environments it is important that the extreme wind speed analyses be done for data separated into each storm type.

C.3 Turbulence spectrum and length scale

C.3.1 Turbulence spectrum

The most commonly used expression for the longitudinal spectrum of turbulence is due to von Karman, which is also generally used as an approximation in terms of wind speed and is given in Equation (C.16):

$$\frac{f\,S_{\mathrm{v}}}{\sigma_{\mathrm{v}}{}^2}=\frac{4\left(\dfrac{f\,L_{\mathrm{v}}}{V_{\mathrm{m}}}\right)}{\left\{1+70{,}8\left(\dfrac{f\,L_{\mathrm{v}}}{V_{\mathrm{m}}}\right)^2\right\}^{\frac{5}{6}}} \tag{C.16}$$

where

f is the frequency;

S_{v} is the longitudinal spectrum of turbulence in terms of wind speed;

L_{v} is the longitudinal integral length scale;

V_{m} is mean wind speed;

σ_{v} is the standard deviation of wind speed.

C.3.2 Integral length scale of turbulence

The determination of integral length scale of turbulence is very complex, but for wind engineering calculations it is possible to conveniently approximate it using Equation (C.17):

$$L_{\mathrm{v}}=100\left(\frac{z}{30}\right)^{0{,}5} \tag{C.17}$$

where

L_{v} is the integral length scale, in metres;

z is the height above ground, in metres.

C.4 Change in terrain roughness effects

Adjustments to height exposure factors and terrain roughness due to changes in terrain roughness in immediate approach fetches can be approximated by a linear weighting of the height exposure factors and terrain roughness over a defined averaging distance for a given height, as given in Table C.3 and Table C.4

and illustrated in Figure C.2. For heights over 10 m a lag distance as a function of height needs to be applied using Equation (C.18):

$$x_{\text{lag}} = z_{0,\,\text{tr max}} \left(\frac{z}{0{,}3 z_{0,\,\text{tr max}}} \right)^{1{,}25} \qquad \text{(C.18)}$$

where

x_{lag} is the distance downwind from the start of a new terrain roughness to the position where the developed height of the inner layer (as shown in Figure C.2) equals z, expressed in metres;

$z_{0,\,\text{tr max}}$ is the larger of the two roughness lengths at a boundary between roughnesses;

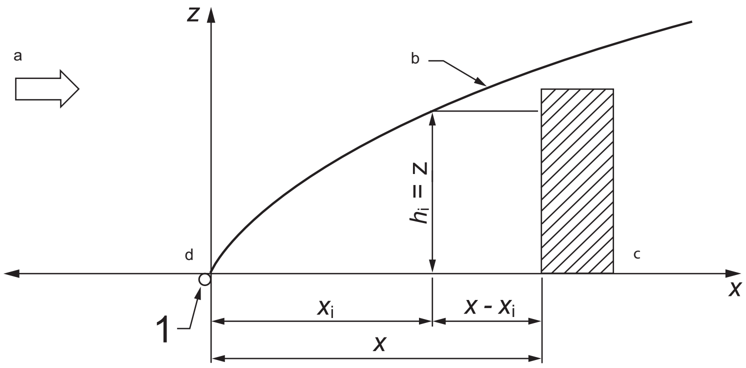
z is the reference height of the structure above the average local ground level.

Table C.3 — Averaging distance for structure height

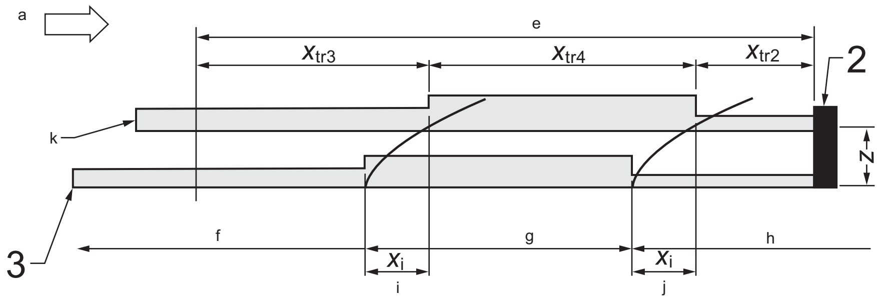
Structure height m	**Averaging distance upwind of structure** m
$h < 50$	1 000
$50 \leqslant h < 100$	2 000
$100 \leqslant h < 200$	3 000

Table C.4 — Roughness lengths for terrain roughness categories

Terrain roughness category	**Roughness length** m
1	0,003
2	0,03
3	0,3
4	3,0



a) Notation for changes in terrain roughness category



$$k_{\mathrm{tr}\,z} = \frac{k_{\mathrm{tr}\,2} x_{\mathrm{tr}\,2} + k_{\mathrm{tr}\,4} x_{\mathrm{tr}\,4} + k_{\mathrm{tr}\,3} x_{\mathrm{tr}\,3}}{\text{Averaging distance}} \quad \text{for the case illustrated}$$

b) Example of changes in terrain roughness category

Key

1 start of new terrain roughness

2 structure

3 actual surface

a Wind direction.

b Developed height of inner layer.

c Newterrain roughness category.

d Upstream terrain roughness category.

e Averaging distance.

f Terrain roughness category 3.

g Terrain roughness category 4.

h Terrain roughness category 2.

i Lag distance (tr3 to tr4).

j Lag distance (tr4 to tr2).

k Lagged response et height, z.

Figure C.2 — Changes in terrain category

C.5 Topographic and orographic effects

For structures situated in hilly, undulating terrain, the speed up of the wind velocity over hills and escarpments is an important consideration. Two “topographical multipliers” can be defined as follows for the mean and peak velocity speed up over small-scale features.

k_{topog} = (peak wind speed at height z above the feature) /

(peak wind speed at height z above the upwind flat ground)

$k_{\text{topog, m}}$ = (mean wind speed at height z above the feature) /

(mean wind speed at height z above the upwind flat ground)

For a particular site, values of these topographic multipliers can be obtained from properly conducted wind tunnel model tests or numerical calculations. For situations where this is not possible, they can be obtained using Equations (C.19) and (C.20):

$$k_{\text{topog}} = 1 + \frac{V_{\text{ref, m}}}{V_{\text{ref}}} k_1 \psi s(x, z) \tag{C.19}$$

$$k_{\text{topog, m}} = 1 + k_1 \psi s(x, z) \tag{C.20}$$

In Equations (C.19) and (C.20), ψ is the slope of the hill ($H/2L_{\text{H}}$) where H is the slope height of the feature and L_{H} is the slope length (see Figure C.3); k_1 is given in Table C.5 and the function $s(x,z)$ is found using Equation (C.21):

$$s(x, z) = 1 - \frac{|x|}{k_2 L_{\text{H}}} e^{-k_3 \frac{z}{L_{\text{H}}}} \tag{C.21}$$

k_2 and k_3 are also given in Table C.5; and

L_{H} is the horizontal distance upwind from the crest of the hill or escarpment to a level half the height below the crest as shown in Figure C.3.

Table C.5 — Parameters for the calculation of topographic multipliers

	k_1	k_2		k_3
		$x < 0$	$x > 0$	
Two-dimensional ridge	4,4	0,75	0,75	1,5
Two-dimensional escarpment	3,6	0,75	2	1,25
Three-dimensional axisymmetric hills	3,2	0,75	0,75	0,8

NOTE 1 For slopes of less than 0,05 the effects of topography can be ignored and the topographic multipliers can be taken as 1,0.

NOTE 2 Once the upwind slope of a hill or escarpment reaches a value of about 0,3 (about 17°), separations occur on the upwind face and the above values no longer strictly apply. However, for slopes between about 0,3 and 1 (17° to 45°), the separation bubble on the upwind slope causes an effective slope to the wind that is relatively constant. The topographic multipliers, at or near the crest, are therefore also fairly constant with upwind slope in this range. Thus for this range of slopes the actual slope can be replaced by an effective slope equal to about 0,3.

Note that the multiplier for the peak velocity is lower than that for the mean, as the effect of topography is primarily to accelerate the mean flow component of velocity.

In addition to the wind speed up over small-scale topographic features, variations in wind speed can be caused by the effects of larger scale “orographic” features. Orographic multipliers can be defined in the same way as the topographic multipliers as follows.

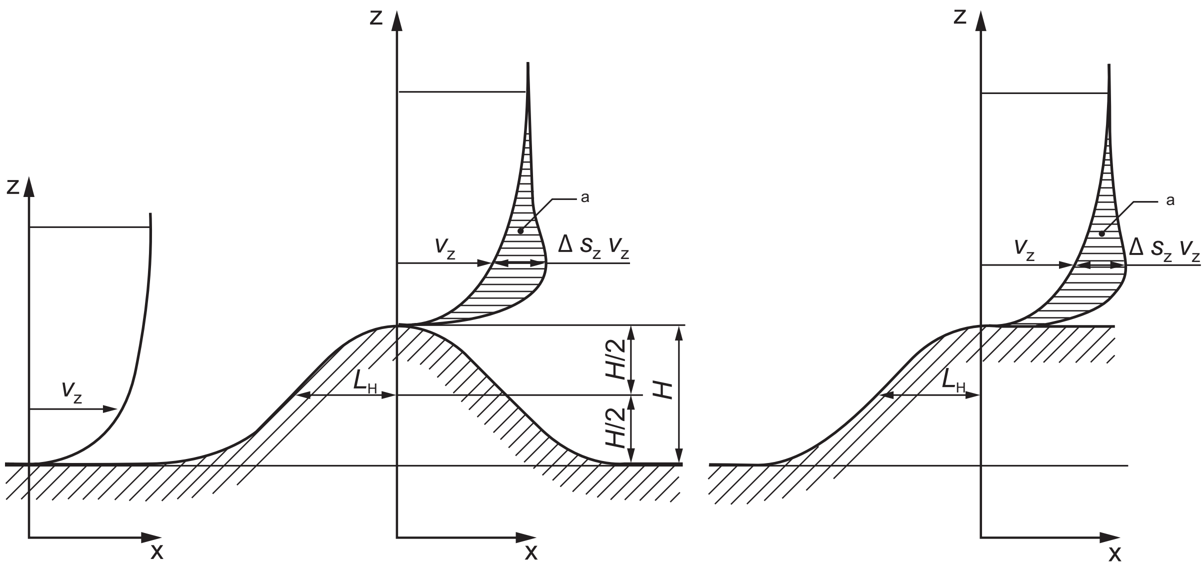
k_{orog} = (peak wind speed at height, z, above the orography) /

(peak wind speed at height, z, in the absence of the orography)

$k_{\text{orog, m}}$ = (mean wind speed at height, z, above the orography) /

(mean wind speed at height, z, in the absence of the orography)

These orographic multipliers are usually dominated by local conditions, and are best obtained from full-scale measurements or from numerical meso-scale meteorological calculations. If such information is not available, these multipliers should be taken to have the value of 1,0.



a) Flat **b) Hill** **c) Escarpment**

[a] "speed-up".

Figure C.3 — Definitions for wind "speed-up" over low hills

Annex D
(informative)

Aerodynamic pressure and force coefficients

D.1 General

The aerodynamic pressure and force coefficients, C_p, and C_F, are dimensionless aerodynamic coefficients that refer to wind-induced pressures or forces. They express the wind-induced action on the structure and its elements as a ratio to a reference velocity pressure in the oncoming flow.

Normally, the aerodynamic force coefficients refer to a mean (time-averaged) wind action, while the aerodynamic pressure coefficients refer to an appropriate fractile value of the respective extremes.

The extremes of the aerodynamic pressure coefficients have to be sampled for a reference period appropriate to the duration of the storm type. In this annex, a full-scale 1 h reference sampling time will be used (i.e. for 1 h in synoptic storms).

An appropriate design fractile value for the aerodynamic pressure coefficients is the 80 % fractile of the respective extremes. This value can be obtained from the mean and rms value of the extremes as follows in Equation (D.1), assuming that the extremes follow a Type I extreme value distribution:

$$C_{\mathrm{p,\,80\%}} = C_{\mathrm{p,\,mean}} \pm 0{,}7 \times C_{\mathrm{p,\,rms}} \qquad \text{(D.1)}$$

where

$C_{\mathrm{p,\,80\%}}$ is the 80 %-fractile of the extreme pressure coefficients corresponding to a reference period of 1 h;

$C_{\mathrm{p,\,mean}}$ is the mean value of the extreme pressure coefficients;

$C_{\mathrm{p,\,rms}}$ is the rms value of the extreme pressure coefficients from 1 h samples;

The minus sign in Equation (D.1) is used in case of negative mean pressure coefficients.

If for the estimation of the 80 %-fractile value extremes sampled from shorter subsets are used, the appropriate design value of the aerodynamic pressure coefficient can be estimated based on Equation (D.2) assuming that the subsets are independent:

$$p_{\mathrm{target,\,subset}} = 0{,}80^{\frac{T_{\mathrm{subset}}}{T_{\mathrm{target}}}} \qquad \text{(D.2)}$$

where

$p_{\mathrm{target,\,subset}}$ is the target non-excedance probability in the subset;

T_{target} is the appropriate reference period, e.g. 1 h for synoptic storms;

T_{subset} is the duration of the subset.

For example, with extremes sampled from 10-min subsets, the target non-excedance probability in Equation (D.2) becomes 0,96. The design value of the aerodynamic pressure coefficients in Equation (D.1) can be estimated with the respective statistical parameters from the subsets as given in Equation (D.3):

$$C_{\mathrm{p,\,80\%}} = C_{\mathrm{p,\,mean,\,subset}} \pm 2{,}1 C_{\mathrm{p,\,rms,\,subset}} \qquad \text{(D.3)}$$

where

$C_{\mathrm{p,\,80\%}}$ is the design value of the aerodynamic pressure coefficients, 80 %-fractile of the extreme pressure coefficients corresponding to the reference period of 1 h;

$C_{\mathrm{p,\,mean,\,subset}}$ is the mean value of the extreme pressure coefficients from 10-min subsets;

$C_{\mathrm{p,\,rms,\,subset}}$ is the rms value of the extreme pressure coefficients from 10-min subsets.

D.2 Reference exposure factors

The pressure and force coefficients are normally defined in conjunction with a reference dynamic pressure either at the same level, z, in the flow or at some fixed reference height, h, e.g. mean roof height. In all cases the reference dynamic pressure should be noted. In addition, any force coefficients should have reference areas clearly defined.

Net pressure across surface elements should be considered as the combination of pressures on external and internal surfaces.

D.3 Wind tunnel testing procedures

Pressure and force coefficients should normally be determined from wind tunnel testing on models. The dynamic factor, C_{dyn}, should, where possible, also be obtained in these tests. These wind tunnel tests must adequately simulate the atmospheric boundary layer and where possible the surroundings at the site in question. This involves modelling the mean velocity and turbulence intensity profiles and the spectrum of the longitudinal component of turbulence as a minimum requirement. Pressure and force coefficients for sharp-edged bodies are usually insensitive to wind velocity, however, curved surfaces, such as cylinders and domes, are sensitive to wind speed, surface roughness and turbulence characteristics, collectively known as Reynolds number effects, and special care should be taken with extrapolating from model tests of such structures.

D.4 Forms of presentation

Pressure and force coefficients are presented in this annex in tables as non-simultaneous values for the design of the cladding or parts of the structure and in figures as simultaneous distributions for the design of the load bearing structure. Simultaneous action of external and internal pressures must be taken into account when appropriate factors for area load and local load effects are incorporated to estimate these simultaneous load distributions. Consistent simultaneous distributions are best obtained using the LRC method (See Clause D.10 and Clause D.11).

Pressure and force coefficients in various national standards and codes, and in the literature, can be defined as mean values or mean extreme values, using different reference averaging times for wind speed (gust or hourly mean), different reference times, different flow conditions (open/rural, or urban), different reference areas (projected frontal or plan area), and reference heights (mean, eaves or ridge). Special care is needed in adopting other values to ensure consistency with the method of this International Standard.

Pressure and force coefficients will be given in this annex for simple rectangular shapes, in open terrain, referenced to the peak velocity pressure (i.e. based on a 3 s mean gust wind speed) at mean roof height, or at height, z, $q_{\mathrm{site,\,h}}$, or $q_{\mathrm{site,\,z}}$.

D.5 Building height definition

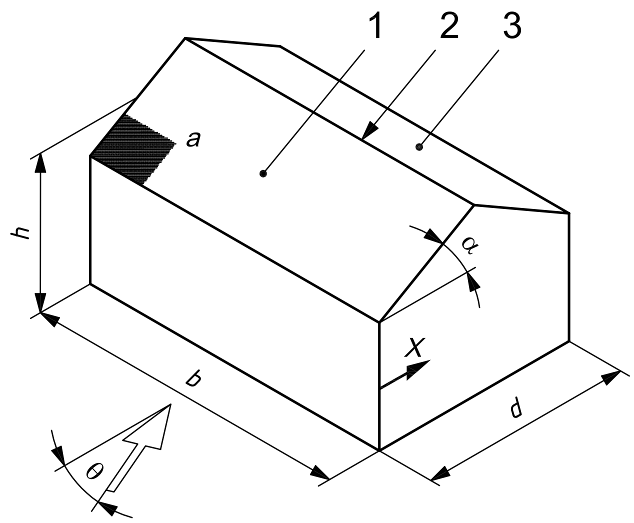
This International Standard defines two types of buildings based on their actual height and height aspect ratio:

a) low buildings are those of height $\leqslant$ 25 m and with height aspect ratio $h/b \leqslant 1$;

b) high buildings have height > 25 m or $h/b > 1$.

For low buildings, the effects of resonance are negligible and C_{dyn} then incorporates the effects of wind turbulence only, and due to a lack of correlation of pressures on different surfaces combinations of loading from different surfaces generally require a reduction in maximum loading over the whole structure, for which $C_{\text{dyn}} = 0,85$.

For high and/or slender buildings resonance of the structure becomes significant and separate calculation of these effects through C_{dyn} is required. See Figure D.1.



Key

1 upwind roof

2 ridge line

3 downwind roof

α Roof pitch angle.

θ Wind direction.

a a is the least of $0,2b$, $0,2d$ or h.

b The dimension in across-wind direction.

d The dimension in along-wind direction.

h The ridge height.

Figure D.1 — Building geometry definitions

D.6 External pressure coefficients

Pressure coefficients for external surfaces, $C_{\text{p, e}}$, of rectangular planform buildings with pitched roofs are presented in this clause. They are a function of the building geometry, including aspect ratios, h/b and d/b, roof pitch, α, and wind direction, θ. Windward, leeward and side walls, and upwind and downwind roofs will change designation according to the wind direction under consideration.

Where two values of pressure coefficient are specified, each must be used in design and combined with internal pressure coefficients for the most severe condition.

For intermediate values of parameters, h/b, α, linear interpolation between values of the same sign shall be carried out.

Table D.1 — Windward walls

h m	**External pressure coefficient** $C_{\text{p, e}}$	**Reference dynamic pressure**
> 25	0,8	$q_{\text{site}, z}$
⩽ 25	0,8	$q_{\text{site}, z}$
	0,7	$q_{\text{site, h}}$

Table D.2 — Leeward walls

α deg (°)	d/b	$C_{\text{p, e}}$	**Reference dynamic pressure**
< 10	⩽ 1 2 ⩾ 4	–0,5 –0,3 –0,2	$q_{\text{site, h}}$
10	All values	–0,3	$q_{\text{site, h}}$
15		–0,3	$q_{\text{site, h}}$
20		–0,4	$q_{\text{site, h}}$
⩾ 25	⩽ 0,1 ⩾ 0,3	–0,75 –0,5	$q_{\text{site, h}}$

Table D.3 — Side walls

Horizontal distance from windward edge	$C_{\text{p, e}}$	**Reference dynamic pressure**
< 1h	–0,65	$q_{\text{site, h}}$
1h to 2h	–0,5	$q_{\text{site, h}}$
2h to 3h	–0,3	$q_{\text{site, h}}$
> 3h	–0,2	$q_{\text{site, h}}$

Table D.4 — Roofs, α<10

Horizontal distance from windward edge of roof	$C_{p, e}$		Reference dynamic pressure
	$h/d \leqslant 0{,}5$	$h/d \geqslant 1{,}0$	$q_{site, h}$
< 0,5h	–0,9, –0,4	–1,3, –0,6	$q_{site, h}$
0,5h to 1h	–0,9, –0,4	–0,7, –0,3	$q_{site, h}$
1h to 2h	–0,5, 0	(–0,7, –0,3) For interpolation only	$q_{site, h}$
2h to 3h	–0,3, 0,1		$q_{site, h}$
> 3h	–0,2, 0,2		$q_{site, h}$

Table D.5 — Upwind roofs, $\alpha \geqslant$10

h/d	$C_{p, e}$							Reference dynamic pressure
	Roof pitch α (deg)							
	10	**15**	**20**	**25**	**30**	**35**	$\geqslant$ **45**	
$\leqslant$ 0,25	–0,7, –0,3	–0,5, 0,0	–0,3, 0,2	–0,2, 0,3	–0,2, 0,4	0,0, 0,5	0, 0,8sinα	$q_{site, h}$
0,5	–0,9, –0,4	–0,7, –0,3	–0,4, 0,0	–0,3, 0,2	–0,2, 0,3	–0,2, 0,4		$q_{site, h}$
$\geqslant$ 1,0	–1,3, –0,6	–1,0, –0,5	–0,7, –0,3	–0,5, 0,0	–0,3, 0,2	–0,2, 0,3		$q_{site, h}$

Table D.6 — Downwind roofs, $\alpha \geqslant$ 10

h/d	$C_{p, e}$					Reference dynamic pressure
	Roof pitch α (deg)					
	10	**15**	**20**	$\geqslant$ **25**		
$\leqslant$ 0,25	–0,3	–0,5	–0,6	b/d<3	–0,6	$q_{site, h}$
0,5	–0,5	–0,5	–0,6	3 $\leqslant b/d \leqslant$ 8	–0,06(7 + b/d)	$q_{site, h}$
$\geqslant$ 1,0	–0,7	–0,6	–0,6	b/d > 8	–0,9	$q_{site, h}$

D.7 Internal pressure coefficients

Aerodynamic shape factors for internal pressure, $C_{p,i}$, are presented in this clause. Internal pressure is a function of building permeability and external openings. Building permeability is defined as any area where leakage of flow into or out of the building can take place (e.g. gaps and vents). Openings can be permanent or failure induced, e.g., window breakage or door blowout. For most structures, C_{dyn} = 0,85 can be assumed with these internal shape factors.

Dominant openings in a surface occur when the sum of all openings in that surface exceeds the sum of all openings in any one other surface.

Where two values of shape factor are specified, each must be used in design and combined with external shape factors for the most severe condition. See Table D.7 and Table D.8.

Table D.7 — Internal shape factor, without dominant opening

Condition	$C_{p,i}$	Reference dynamic pressure
One wall permeable, other walls impermeable 1. Windward wall permeable 2. Windward wall impermeable	 0,6 –0,3	$q_{site,\ h}$
Two or three walls equally permeable, other walls impermeable 1. Windward wall permeable 2. Windward wall impermeable	 –0,1, 0,2 –0,3	$q_{site,\ h}$
All walls equally permeable	–0,3, 0,0	$q_{site,\ h}$
All walls impermeable (sealed building)	–0,2, 0,0	$q_{site,\ h}$

Table D.8 — Internal shape factor, with dominant opening

Dominant opening location	Ratio of dominant opening area in a surface to total open area (including permeability) of any one other surface					Reference dynamic pressure
	$\leqslant$ 0,5	1	2	3	$\geqslant$ 6	
Windward wall	–0,3, 0,0	–0,1, 0,2	0,7 ($C_{p,\ e}$)	0,85 ($C_{p,\ e}$)	($C_{p,\ e}$)	$q_{site,\ z}$
Leeward wall	–0,3, 0,0	–0,3, 0,0	1,3 ($C_{p,\ e}$)	1,1 ($C_{p,\ e}$)	($C_{p,\ e}$)	$q_{site,\ h}$
Side wall	–0,3, 0,0	–0,3, 0,0	($C_{p,\ e}$)	($C_{p,\ e}$)	($C_{p,\ e}$)	$q_{site,\ h}$
Roof	–0,3, 0,0	–0,3, 0,15 ($C_{p,\ e}$)	($C_{p,\ e}$)	($C_{p,\ e}$)	($C_{p,\ e}$)	$q_{site,\ h}$
NOTE $C_{p,\ e}$ is the relevant external pressure coefficient at the dominant opening location for the relevant wind direction.						

D.8 Factor for area load effects

The lack of correlation of fluctuating wind pressures over large areas on a single surface leads to an overestimation of loads on that surface and a factor for area load effects, K_a, is introduced. K_a is a function of the actual size of area contributing to the load effect under consideration. Table D.9 gives these factors. For intermediate values of area, linear interpolation is allowed.

Table D.9 — Area load effect factor

Tributary area for load effect calculation m^2	Factor for area load effects K_a
$\leqslant$ 10	1,0
25	0,9
$\geqslant$ 100	0,8

D.9 Factor for local load effects

The averaging of shape factors over the entire faces shown in Figure D.1 leads to an underestimation of pressures adjacent to edges and over smaller areas due to the fluctuating nature of the wind and a factor for local load effects, K_l, is introduced. K_l depends on the size of the area under consideration and its proximity to an edge. The characteristic dimension for local load effects, a, is defined as the least of 0,2b, 0,2d or h. See Table D.10.

Table D.10 — Local load effect factor

Case	h m	Area	Proximity to edge	K_l
Positive pressures Windward wall	all	$\leqslant 0{,}25a^2$	anywhere	1,5
Negative pressures Roof edges and downwind roof surfaces near ridges, $\alpha \geqslant 10$ °	all	$\leqslant 0{,}25\ a^2$ $\leqslant a^2$	$< 0{,}5\ a$ $< a$	2,0 1,5
Negative pressures Side walls near windward edges	$\leqslant 25$	$\leqslant 0{,}25\ a^2$ $\leqslant a^2$	$< 0{,}5\ a$ $< a$	2,0 1,5
	> 25	$\leqslant 0{,}25\ a^2$ $\leqslant a^2$ $\leqslant a^2$	$< 0{,}5\ a$ $< a$ $> a$	3,0 2,0 1,5
All other areas	All			1,0

D.10 The LRC method

Due to the turbulent nature of the oncoming flow, wind-induced actions on structures are highly fluctuating, varying in amplitude and frequency. The pressure coefficients and load effect factors presented in Clause D.6 to Clause D.9 generally produce load distributions for design that are conservative and represent only approximate worst-case pressure distributions. From the almost infinite number of possible distributions of pressures on a structure, the load-response correlation (LRC) method allows for the identification of effective pressure distributions that lead to extreme actions (drag or lift) or action effects (internal forces, e.g. bending moment). This clause presents the LRC method followed by an example in Clause D.11.

The first step is to specify for each contributing pressure, p_i, a weighting factor or influence factor, a_i, which is in the case of a global force like drag and lift simply the contributing area of the pressure, p_i, in the direction of that force, or in the case of an internal force is the respective structural response to a unit load at the position of p_i.

The effective distribution is obtained as the mean pressure distribution plus a weighted rms contribution using Equation (D.4):

$$C_{p_i,\text{eff}} = C_{p_i,\text{mean}} + g_r \times \rho_{rp_i} \times C_{p_i,\text{rms}} \qquad \text{(D.4)}$$

where

$C_{p_i,\text{mean}}$ is the mean pressure coefficient;

$C_{p_i,\text{rms}}$ is the rms-pressure coefficient;

$\rho_{\mathrm{rp_i}}$ is the correlation between the pressure at i and the response under consideration;

g_{r} is the peak factor for the response under consideration.

The peak factor for the response under consideration is obtained from an appropriate fractile value, e.g. the 80 % fractile, of the extreme responses using Equation (D.5):

$$g_{\mathrm{r}} = \frac{R_{\mathrm{mean}} \pm 0{,}7 R_{\mathrm{rms}} - r_{\mathrm{mean}}}{r_{\mathrm{rms}}} \tag{D.5}$$

where

R_{mean} is the mean value of extreme response;

R_{rms} is the rms value of extreme response;

r_{mean} is the mean value of response;

r_{rms} is the rms value of response.

The negative sign in Equation (D.5) is used in case of negative pressure coefficients.

The mean and the rms value of the response are obtained using Equation (D.6):

$$r_{\mathrm{mean}} = \sum a_{\mathrm{i}} \times C_{\mathrm{p_i,mean}} \tag{D.6}$$

$$r_{\mathrm{rms}} = \left[\sum_{\mathrm{i}} \sum_{\mathrm{j}} a_{\mathrm{i}} \times a_{\mathrm{j}} \times \rho_{\mathrm{p_i p_j}} \times C_{\mathrm{p_i,rms}} \times C_{\mathrm{p_j,rms}} \right]^{1/2}$$

r_{mean} is the mean value of the response;

r_{rms} is the rms value of the response;

a_{i} is the weighting factor;

$C_{\mathrm{p_i,mean}}$ is the mean value of the pressure coefficient;

$C_{\mathrm{p_i,rms}}$ is the rms value of the pressure coefficient;

$\rho_{\mathrm{p_i p_j}}$ is the correlation between pressure, i, and pressure, j.

The correlation between the pressure at i and the response is obtained using Equation (D.7):

$$\rho_{\mathrm{rp_i}} = \frac{\sum_{\mathrm{j}} a_{\mathrm{j}} \times \rho_{\mathrm{p_i p_j}} \times C_{\mathrm{p_j,rms}}}{\left[\sum_{\mathrm{i}} \sum_{\mathrm{j}} a_{\mathrm{i}} \times a_{\mathrm{j}} \times \rho_{\mathrm{p_i p_j}} \times C_{\mathrm{p_i,rms}} \times C_{\mathrm{p_j,rms}} \right]^{1/2}} \tag{D.7}$$

D.11 External pressure coefficient, $C_{p, e}$, for flat and gable roof buildings for wind normal to the ridge line

In this clause, simultaneous pressure coefficient distributions and non-simultaneous pressure coefficients derived from the LRC method are presented for the centre bay of flat and gable roof rectangular buildings with the following geometry:

$b / d / h$ = 1,33 / 1 / 0,5

where

b is the dimension in across wind direction;

d is the dimension in along wind direction;

h is the ridge height.

Simultaneous pressure distributions are given in Figure D.2. They have been derived to reproduce the 80 %-fractile of the extreme drag on the building and lift on the roof. They can be used as approximations for calculating the extreme forces in the load-bearing structure (main frame).

For intermediate roof slope, α, linear interpolation can be applied. As reference pressure, the 3-s gust velocity pressure at mean roof height is used.

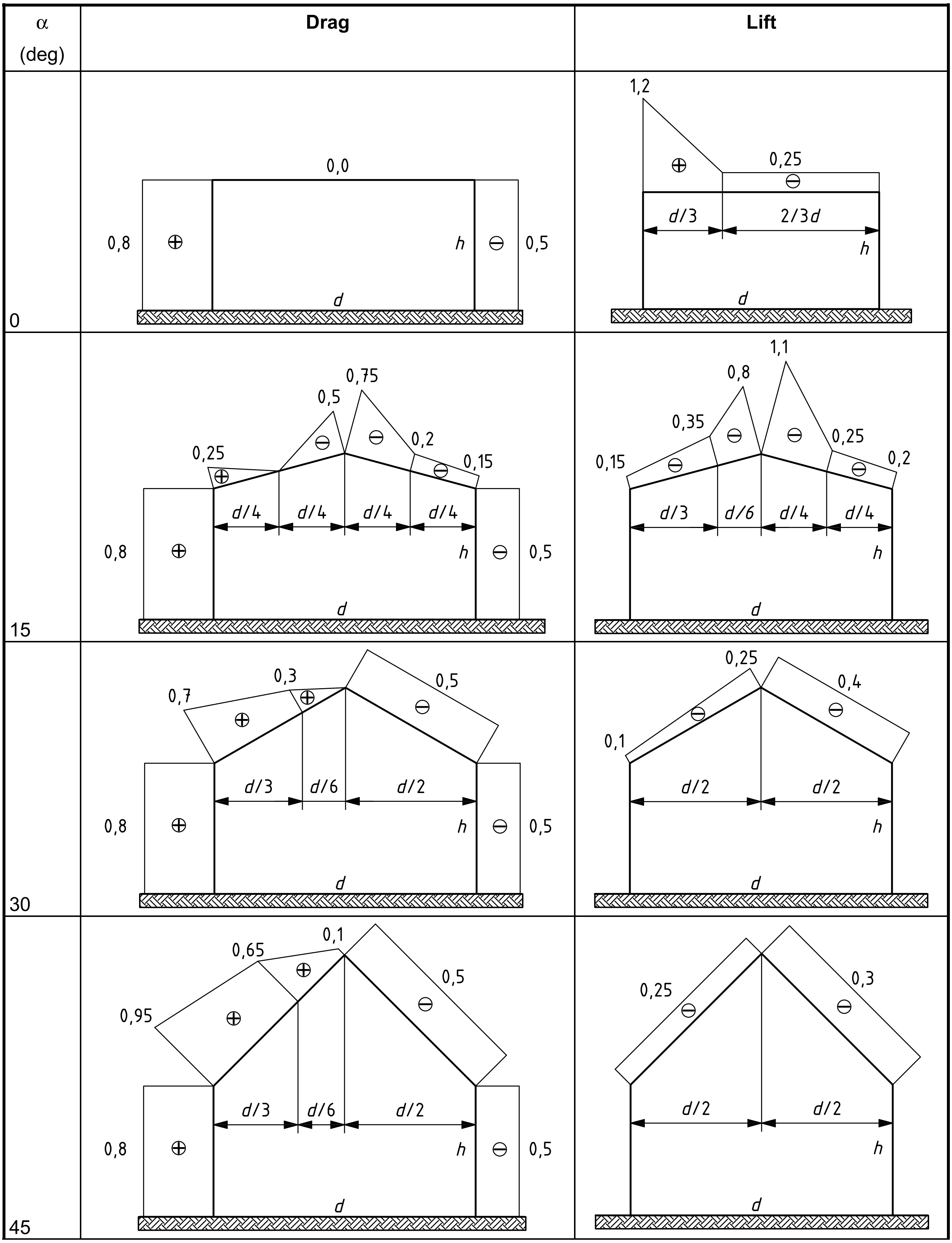


Figure D.2 (*continued*)

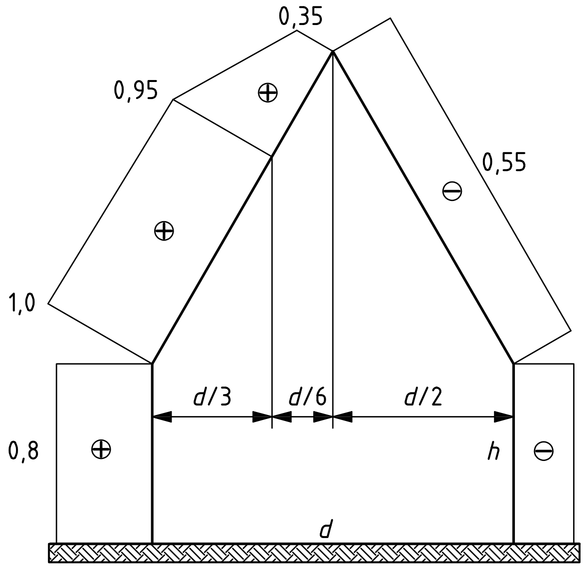
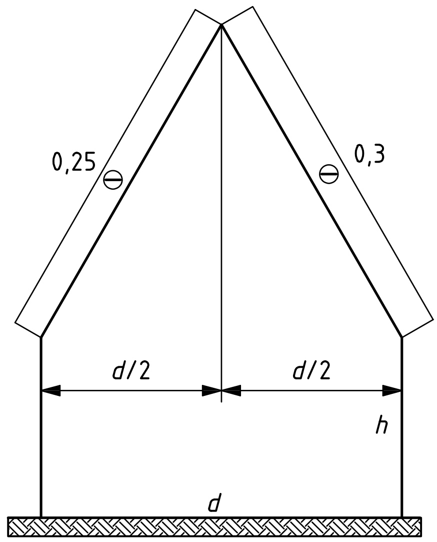
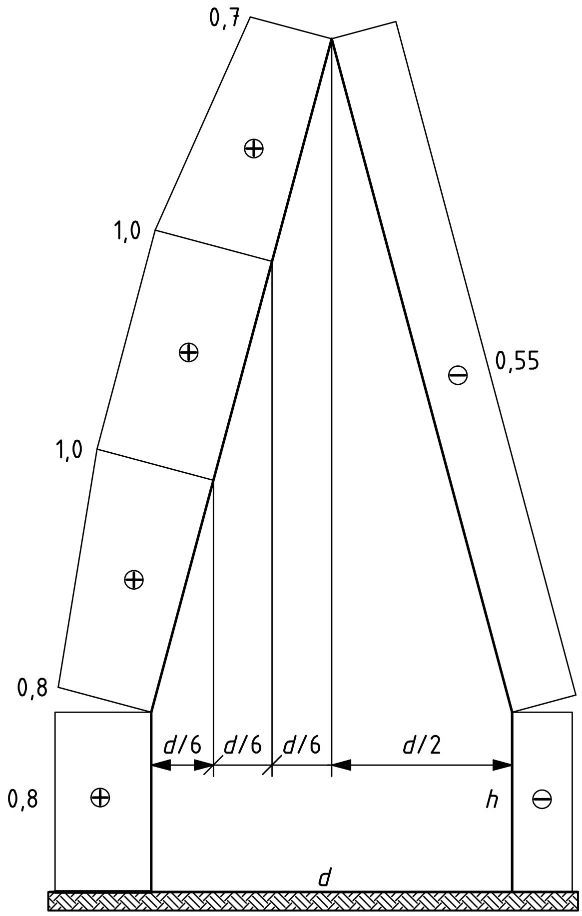
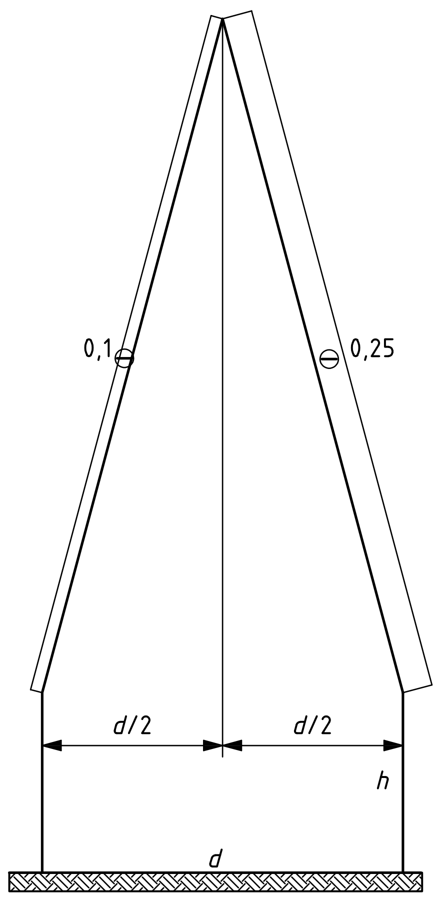
α (deg)	Drag	Lift
60		
75		

Figure D.2 — Simultaneous pressure coefficient distributions for flat roofs and gable roofs for overall loads

Table D.11 summarizes the pressure coefficients for the local wind-induced actions. For different positions along the centre bay of the roof, two extremes (max. and min.) in terms of the 80 % fractile are specified.

These extremes will not occur simultaneously. For intermediate position, x/d, and intermediate roof slope, α, linear interpolation can be applied. As reference pressure, the peak velocity pressure at ridge height is used.

Table D.11 — Non-simultaneous pressure coefficients on flat and gable roofs for local load effects

region	α	0 °		15 °		30 °		45 °		60 °		75 °	
	x/d	max	min	max	min	max	min	max	min	max	min	max	min
upwind	0	0,20	–1,80	0,30	–0,90	0,70	–0,25	1,00	–0,10	1,15	–0,05	1,10	0
	0,33	0,20	–0,80	0,05	–0,40	0,40	–0,25	0,75	–0,15	1,05	–0,10	1,15	0
	0,5	0,20	–0,60	0	–1,10	0,10	–0,40	0,30	–0,35	0,55	–0,25	0,90	–0,15
downwind	0,5	0,20	–0,55	0	–1,60	0	–0,55	0	–0,60	0	–0,65	0	–0,65
	0,75	0,20	–0,55	0	–0,50	0	–0,65	0	–0,70	0	–0,80	0	–0,80
	1	0,20	–0,45	0	–0,35	0	–0,75	0	–0,85	0	–0,95	0	–1,00
NOTE The reference value is peak velocity pressure at ridge height.													

D.12 Aerodynamic force coefficient

The aerodynamic force coefficient, C_{Fm}, is an aerodynamic wind-induced force expressed as a ratio of the aerodynamic force exerted on a structure, or its parts, to a reference dynamic pressure multiplied by a reference area, and in this clause refers only to a mean (time-averaged) wind action.

i.e. $$C_{Fm} = \frac{F_m}{q_{site,m} A_{ref}}$$

The aerodynamic force coefficient can be obtained from wind tunnel measurements or suitable references to such measurements. Some illustrative values are given in Figure D.3 for a building with a circular section, Figure D.4 for a free roof, Figure D.5 for lattice structures, Figure D.6 for wind normal to the centre section of a fence and Figure D.7 for sectional force coefficients on structural components.

These are limited examples and shall not be applied outside the scopes noted.

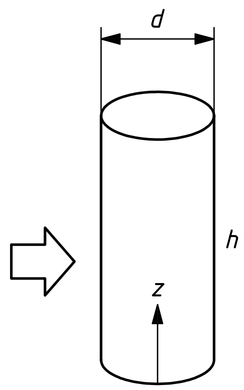
$C_{Fm} = 1{,}2\ k_1 k_2 k_z$

where

k_1 is the factor for aspect ratio;

k_2 is the factor for surface roughness;

k_z is the factor for vertical profile.



Factor for aspect ratio k_1

$h/d < 1$	$1 \leqslant h/d \leqslant 8$
0,6	$0{,}6\ (h/d)^{0{,}14}$

Factor for surface roughness k_2

smooth surface (metal, concrete, flat curtain walls, etc.)	0,75
rough surface (1 % relative roughness, rough curtain walls, etc.)	0,9
very rough surface (5 % relative roughness)	1

Factor for vertical profile k_z

$z < 0{,}8h$	$z \geqslant 0{,}8h$
$(z / h)^{2\beta}$	$0{,}8^{2\beta}$

Key

d building diameter, expressed in metres

h reference height, expressed in metres

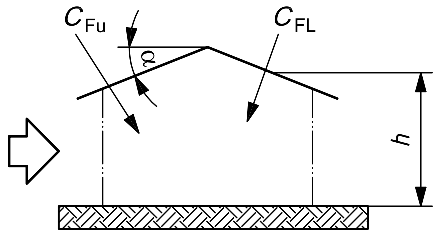
β power-law index of mean wind speed profile

$q_{site,\ m}$ is taken at the top of the building

A_{ref} is $h\ d$

Figure D.3 — Wind force coefficients for buildings with circular sections for $h/d \leqslant 8$

roof pitch α (°)	windward roof C_{Fu}		leeward roof C_{FL}	
	positive	negative	positive	negative
$-30° \leqslant \alpha \leqslant -10°$	$0{,}7 + 0{,}01\alpha$	$-0{,}6 + 0{,}03\alpha$	$0{,}05 - 0{,}025\alpha$	$-1{,}2 - 0{,}03\alpha$
$-10° < \alpha < 10°$	0,6	–0,9	0,3	–0,9
$10° \leqslant \alpha \leqslant 30°$	$0{,}3 + 0{,}03\alpha$	$-1{,}15 + 0{,}025\alpha$	0,3	$-0{,}6 - 0{,}03\alpha$



Key

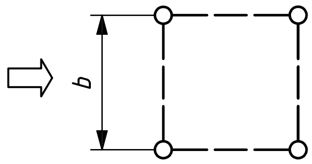
h mean roof height, expressed in metres

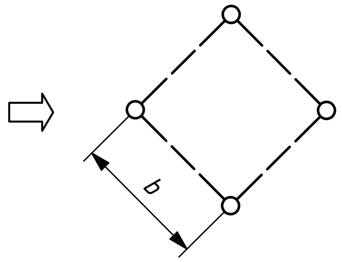
α roof pitch, expressed in degrees

A_{ref} is the relevant roof area. $q_{site,\ m}$ is taken at the top of the roof. Positive indicates downwards.

Figure D.4 — Force coefficient for free roofs with no blockage underneath

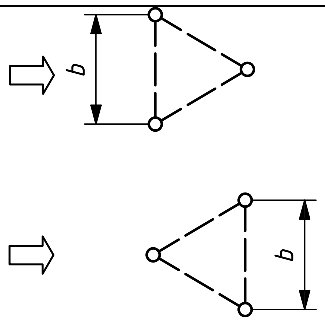
i) Square plan



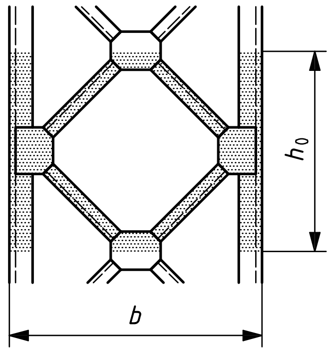


solidity ratio φ	angle	circular pipe	angle	circular pipe
0	3,8	2,3	4,4	2,5
0,5	1,9	1,4	2,3	1,7
0,6	1,9	1,4	2,3	1,7

ii) Triangle plan



solidity ratio φ	angle	circular pipe
0	3,8	2,3
0,5	1,9	1,4
0,6	1,9	1,4



The solidity ratio, φ, is defined by $\varphi = A_F / A_0$

where

A_F is the projection area per panel ( ) (m^2);

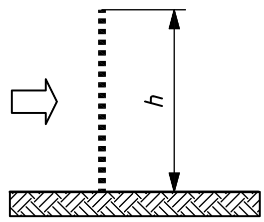
A_0 is the whole plane area ($=bh_0$) (m^2).

A_{ref} is the average diameter or breadth of a section of tower members multiplied by their individual lengths on one face projected normal to that face. Linear interpolation is permitted for values of solidity ratio, φ, other than shown.

The force coefficients given are for circular cylinders in sub-critical Reynolds number flow conditions, defined by member diameter (m) multiplied by wind speed (m/s) being less than 3,0. Force coefficients at super-critical Reynolds number flow conditions shall be established from contemporary literature, or measured, as they are dependent on surface roughness and turbulence as well as Reynolds number.

Figure D.5 — Sectional wind force coefficients for lattice structures

Solidity ratio φ	C_{Fm}
0	1,2
0,2	1,5
0,6	1,7
$\geqslant$ 0,9 (solid fences included)	1,2

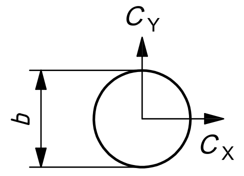


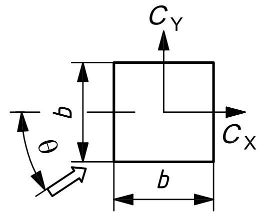
The definition of solidity ratio, φ, is the same as that in Figure D.5.

Linear interpolation is permitted for values of solidity ratio, φ, other than shown.

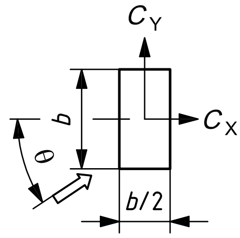
A_{ref} is fence height, h, multiplied by the solidity ratio, φ, and by the length of the fence.

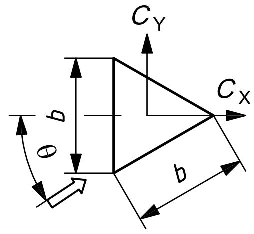
Figure D.6 — Wind force coefficients on the centre section of a fence on the ground





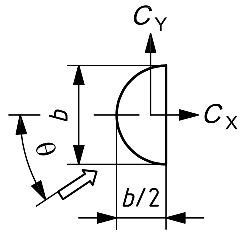
(1)

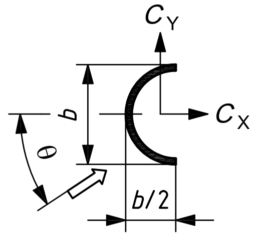


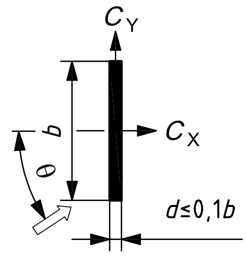


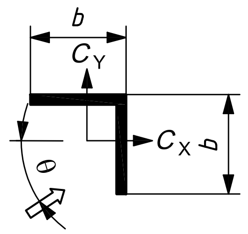
x	θ	C_X	C_Y	θ	C_X	C_Y	θ	C_X	C_Y
1,2	0°	2,1	0	0°	2,4	0	0°	2,1	0
	45°	1,6	1,6	45°	1,6	0,7	30°	2,1	–0,2
				90°	0	0,8	60°	0,7	1,1

(1) Do not interpolate, seek further reference for other rectangular width to depth ratios





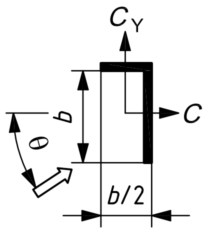
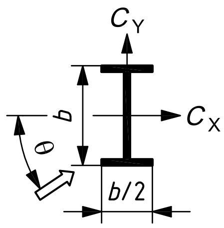
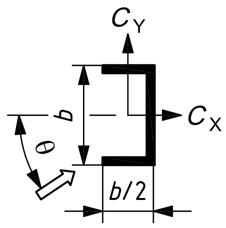


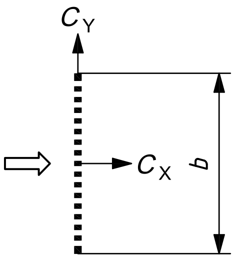


θ	C_X	C_Y	θ	C_X	C_Y	θ	C_X	C_Y	θ	C_X	C_Y
0°	1,2	0	0°	1,1	0	0°	2,0	0	0°	1,9	2,2
45°	0,8	0,8	45°	0,8	0,7	45°	1,8	0,1	45°	2,3	2,3
90°	0,6	0,5	90°	0,9	0,5	90°	0	0,1	90°	2,2	1,9
135°	–1,7	0,6	135°	–2,3	0,6				135°	–1,9	–0,6
180°	–2,3	0	180°	–2,5	0				180°	–2,0	0,3
									225°	–1,4	–1,4

A_{ref} is width, b, multiplied by unit length.

Figure D.7 — Sectional wind force coefficients for structural components

 			 			 		
θ	C_X	C_Y	θ	C_X	C_Y	θ	C_X	C_Y
0°	2,0	1,1	0°	2,1	0	0°	2,6	0
45°	2,3	1,1	45°	2,1	0,6	45°	2,0	0,8
90°	1,8	0,8	90°	± 0,6	0,7	90°	± 0,6	0,8
135°	–1,7	0				135°	–1,6	0,6
180°	–2,0	0,1				180°	–2,0	0
270°	0,6	–0,8						
315°	1,2	–0,2						



Solidity ratio φ	C_X
0	2
0,2	2
0,6	2,7
$\geqslant$ 0,9 (solid plate included)	2

The area for calculating the wind loads is $bl\varphi$ (l = net length).

The definition of solidity ratio, φ, is the same as that in Figure D.5.

Linear interpolation is permitted for values of φ other than shown.

NOTE C_x and C_y correspond to C_{Fm}.

Figure D.7 (*continued*)

Annex E
(informative)

Dynamic response factors

E.1 General

This Annex presents procedures recommended for determining the dynamic response factors, C_{dyn}, and $C_{dyn,\,m}$, referred to in Clauses 6, 10 and 12 of this International Standard. The dynamic response factor, C_{dyn}, is the ratio of the maximum loading effect to the dynamic loading effect obtained by assuming the quasi-steady condition based on the site peak velocity pressure. The mean dynamic response factor, $C_{dyn,\,m}$, is the ratio of the maximum loading effect to the mean loading effect incorporating the site mean velocity pressure, which is identical to the original definition of the gust loading factor by Reference [11].

They take into account the dynamic (i.e. fluctuating with time) action of:

— random wind gusts acting for short durations over all or part of the structure;

— fluctuating pressures induced by the wake of the structure, including vortex shedding forces;

— fluctuating forces induced by the motion of the structure due to wind.

These forces act on the external surfaces of the structure as a whole or on cladding components and can also affect internal surfaces. They act longitudinally, laterally or torsionally and can be amplified by resonance of the structure at one of its natural frequencies.

All structures are affected to some degree by these forces. The total response can be considered as a superposition of a “mean” component due to the mean wind force, a “background” component, which acts quasi-statically without any structural dynamic magnification, and a “resonant” component due to excitation close to a natural frequency. For the majority of structures, the resonant component is small and the dynamic factor can be simplified by considering the background component only by using normal static methods. For structures that are particularly tall or long, slender, lightweight, flexible or lightly damped, the resonant component can be dominant. Criteria are suggested in Clause E.2 to determine when the dynamic effects of the resonant component can be significant.

This annex first discusses the procedures and methodologies recommended for estimating the mean dynamic response factor, $C_{dyn,\,m}$, based on the site mean velocity pressure, $q_{site,\,m}$. Then, the peak dynamic response factor, C_{dyn}, based on the site peak velocity pressure, q_{site}, is discussed in Clause E.4.

A general expression for the maximum or peak loading effect, W, is Equation (E.1):

$$W = W_m + g_w \sigma_w \tag{E.1}$$

where

W_m is the mean load effect;

σ_w is the standard deviation of the load effect;

g_w is a statistical peak factor for the load effect.

According to this expression, the mean dynamic response factor can be identified as Equation (E.2):

$$C_{\text{dyn, m}} = \frac{W}{W_{\text{m}}} = 1 + \frac{g_{\text{w}}\sigma_{\text{w}}}{W_{\text{m}}}, \text{ if } W_{\text{m}} \neq 0 \qquad \text{(E.2)}$$

The form of the fluctuating wind loading effect, σ_{w}, varies with the excitation, whether due to gusts, wake pressures or motion-induced forces.

The general expression for the equivalent static wind load is given in Equations (E.3) and (E.4):

$$F_{\text{loc}} = q_{\text{site, m}} C_{\text{p}} C_{\text{dyn, m}} A \qquad \text{(E.3)}$$

$$F = q_{\text{site, m}} C_{\text{Fm}} C_{\text{dyn, m}} A_{\text{ref}} \qquad \text{(E.4)}$$

where

$q_{\text{site, m}}$ is the site mean velocity pressure;

C_{p} and C_{Fm} are the pressure coefficient and mean force coefficient, respectively;

$C_{\text{dyn, m}}$ is the mean dynamic response factor based on the site mean velocity pressure;

A and A_{ref} are the area of application and projected area normal to the wind direction of the building at height, z.

E.2 Classification of structures

E.2.1 Rigid structures

The fundamental natural frequency of a rigid building is high enough that the resonant component of the dynamic response is negligible. Therefore, the equivalent static wind load consists of the mean component, F_{m}, and the background component, F_{B}.

Wind pressures acting on distant positions on the surface of a general building lose correlation, and the size reduction effect should be taken into account in the estimation of the background component, F_{B}.

E.2.2 Dynamically sensitive structures

A flexible structure such as a tall building or a long-span roof is generally vulnerable to wind, and the resonant component of its dynamic response is not negligible. Therefore, the equivalent static wind load consists of the mean component, F_{m}, the background component, F_{B}, and the resonant component, F_{R}.

For the along-wind excitation of buildings, a criterion for assessment of sensitivity to resonance is

$$\left(\frac{R_{\text{D}}}{B_{\text{D}}}\right)^2 < \text{constant} \qquad \text{(E.5)}$$

$R_{\text{D}}/B_{\text{D}}$ is evaluated by Equations (E.25) and (E.26). A suitable value for the constant is 0,3.

Flexible and slender buildings vibrate not only in the along-wind direction but also in the crosswind direction, mainly due to vortices shed from the structure, and the torsional response cannot be ignored either.

The condition in Equations (E.6) and (E.7) is an example of the criteria of structures with a rectangular cross-section where the crosswind and torsional responses need to be examined.

$$\frac{h}{\sqrt{bd}} \geqslant \text{constant} \tag{E.6}$$

and

$$\frac{V_{\text{site, m}}}{f_{\text{LT}}\sqrt{bd}} \geqslant \text{constant} \tag{E.7}$$

where

h is the height of the structure;

b is the breadth of the structure (horizontal length normal to the wind direction);

d is the depth of the structure (horizontal length parallel to the wind direction);

f_{LT} is the lowest natural frequency of the crosswind or torsional vibration;

$V_{\text{site, m}}$ is the site mean wind speed.

A suitable value for the constant in Equation (E.6) is 3, and in Equation (E.7) is 0,4.

Roofs are subject to separated flows from the windward eaves, and the characteristics of wind loads on roofs are different from those of horizontal wind loads. The roof wind load can govern the entire behaviour of the structural frames of a building with a long-span roof. The resonant component of the roof wind load becomes predominant under the following condition:

$$\frac{V_{\text{site, m}}}{f_{\text{R}}h} \geqslant \text{constant} \tag{E.8}$$

where

f_{R} is the first mode natural frequency of the roof.

An appropriate value of the constant in (E.8) is 0,7. It is recommended that the roof wind load be estimated using a formula similar to Equation (E.4) reflecting the dynamic behaviour of the roof structures in C_{dyn}. Reference [17] shows an example of a closed form expression of the roof wind load.

E.2.3 Particularly wind-sensitive structures

Particularly wind-sensitive structures with light weight and low damping, e.g. chimneys, masts, monumental towers, long-span bridges and members of space structures, can produce a significant vibration due to the periodic vortex shedding enhanced by the structure's motion or aeroelastic instability.

E.2.3.1 Rectangular cross-sections

For tall slender buildings with a rectangular cross-section satisfying both of the following conditions, special treatment for vortex resonances and aeroelastic instabilities such as consultation with experts and performance of wind tunnel tests is necessary:

$$\frac{h}{\sqrt{bd}} \geqslant \text{constant} \tag{E.9}$$

and

$$V_{\text{hcr}} \leqslant 1{,}5 V_{\text{site, m}} \tag{E.10}$$

where

V_{hcr} is the critical wind speed for crosswind or torsional aeroelastic instability;

$V_{\text{site, m}}$ is the site mean wind speed.

The critical wind speed in Equation (E.10) is given as Equations (E.11) and (E.12).

For crosswind aeroelastic instability:

$$V_{\text{hcr}} = V^{*}{}_{\text{Lcr}} f_{\text{L}} \sqrt{bd} \tag{E.11}$$

For torsional aeroelastic instability:

$$V_{\text{hcr}} = V^{*}{}_{\text{Tcr}} f_{\text{T}} \sqrt{bd} \tag{E.12}$$

where

$V^{*}{}_{\text{Lcr}}$ and $V^{*}{}_{\text{Tcr}}$ are the reduced critical velocities for crosswind and torsional aeroelastic instabilities, respectively;

f_{L} and f_{T} are the first mode natural frequencies of crosswind and torsional vibration.

A suitable value for the constant in Equation (E.9) is 4. Examples of appropriate reduced critical velocities in Equations (E.11) and (E.12) are shown in Table E.1 and Table E.2.

Table E.1 — Reduced critical wind speed, V^*_{Lcr}, for crosswind aeroelastic instability of structures with a rectangular cross-section (Reference [17])

Terrain category	Side ratio d/b	Scruton number S_C [a]	Reduced critical wind speed V^*_{Lcr}
Open country	$d/b \leqslant 0{,}8$	$S_C \leqslant 26$	$0{,}42 S_C$
		$S_C > 26$	11
	$0{,}8 < d/b \leqslant 1{,}5$	Any S_C	$0{,}032 S_C + 7{,}3$
	$1{,}5 < d/b \leqslant 2{,}5$	$S_C \leqslant 7{,}5$	2,3
		$7{,}5 < S_C \leqslant 30$	12
		$S_C > 30$	$0{,}4 S_C$
	$d/b > 2{,}5$	$S_C \leqslant 15$	3,7
		$S_C > 15$	Stable
Urban area	$d/b \leqslant 0{,}8$	Any S_C	$0{,}12 S_C + 6{,}7$
	$0{,}8 < d/b \leqslant 1{,}2$		$0{,}019 S_C + 8{,}8$
	$d/b > 1{,}2$		11

[a] The Scruton number is defined as $S_C = 4\pi \zeta_{\mathrm{str},L} m_{\mathrm{h}} / (\rho b d)$. Here, $\zeta_{\mathrm{str},L}$ is the damping ratio for the first mode crosswind vibration; m_{h} is the mass per unit height of the structure; and ρ is the air density.

Table E.2 — Reduced critical wind speed, V^*_{Tcr}, for torsional aeroelastic instability of structures with a rectangular cross-section (Reference [17])

Side ratio d/b	Inertial moment-damping parameter, ξ_{T} [a]	Reduced critical wind speed, V^*_{Tcr}
$d/b \leqslant 1{,}5$	$\xi_{\mathrm{T}} \leqslant 0{,}05$	2
	$0{,}05 < \xi_{\mathrm{T}} \leqslant 0{,}1$	11
	$\xi_{\mathrm{T}} > 0{,}1$	Stable
$1{,}5 < d/b \leqslant 2{,}5$	$\xi_{\mathrm{T}} \leqslant 0{,}05$	2
	$0{,}05 < \xi_{\mathrm{T}} \leqslant 0{,}15$	$4 + 8\,\xi_{\mathrm{T}}$
	$\xi_{\mathrm{T}} > 0{,}15$	$8{,}6 + 7{,}4\,\xi_{\mathrm{T}}$
$2{,}5 < d/b \leqslant 5$	$\xi_{\mathrm{T}} \leqslant 0{,}05$	2
	$\xi_{\mathrm{T}} > 0{,}05$	$5 + 10{,}5\,\xi_{\mathrm{T}}$

[a] The inertial moment-damping parameter is defined as $\xi_{\mathrm{T}} = \zeta_{\mathrm{str,T}} \left(b^2 + d^2\right) m_{\mathrm{h}} / \left(36 \rho b^2 d^2\right)$. Here, $\zeta_{\mathrm{str,T}}$ is the damping ratio for the first mode torsional vibration; m_{h} is the mass per unit height of the structure; and ρ is the air density.

NOTE Table E.2 is not applicable to buildings with a side ratio of $d/b > 5$.

E.2.3.2 Cylindrical structures

For slender, free-standing cylindrical structures with a circular cross-section satisfying the following conditions, the vortex resonance shall be checked as given in Annex F:

$$\frac{h}{D} \geqslant \text{constant} \tag{E.13}$$

and

$$V_{\mathrm{hcr}} \leqslant 1{,}5 V_{\mathrm{site,\,m}} \tag{E.14}$$

where

D is the average diameter over the top third of the free-standing structure;

V_{hcr} is the critical wind speed for vortex resonance;

$V_{\mathrm{site,\,m}}$ is the site mean wind speed.

The critical wind speed for vortex resonance in Equation (E.14) is given as Equation (E.15):

$$V_{\mathrm{hcr}} = \frac{f_{\mathrm{L}} D}{Sr} \tag{E.15}$$

where

Sr is the Strouhal number, and can be used as 0,2 for circular cylinders.

A suitable value for the constant in Equation (E.13) can be between 6 and 7.

E.2.3.3 Long-span bridges

For long-span bridges satisfying the conditions in Equation (E.16), special treatment is required either by consultation with experts or by wind tunnel studies.

$$V_{\mathrm{hcr}} \leqslant 1{,}5 V_{\mathrm{site,\,m}} \tag{E.16}$$

where

V_{hcr} is the critical wind speed for aeroelastic instability;

$V_{\mathrm{site,\,m}}$ is the site mean wind speed.

Unless alternative rational procedures are available, the critical wind speed, V_{hcr}, in Equation (E.16) can be calculated from Equations (E.17) to (E.20).

For flutter (Reference [15]):

$$V_{\mathrm{hcr}} = 2{,}5 f_{\theta} b_{\mathrm{B}} \tag{E.17}$$

For galloping (Reference [15]):

$$V_{\mathrm{hcr}} = 4{,}0 f_{\mathrm{h}} b_{\mathrm{B}} \tag{E.18}$$

For vortex-induced response of bending mode (References [16] and [12]):

$$V_{\mathrm{hcr}} = 2{,}0 f_{\mathrm{h}} b_{\mathrm{B}} \tag{E.19}$$

For vortex-induced response of torsional mode (References [16] and [12]):

$$V_{\mathrm{hcr}} = 1{,}3 f_{\theta} b_{\mathrm{B}} \tag{E.20}$$

where,

f_{θ} is the natural frequency of the first torsional mode of the bridge;

f_{h} is the natural frequency of the first bending mode of the bridge;

b_{B} is the width of the bridge section (horizontal length of the section).

Galloping can take place for solid steel girders whose width, b_{B}, to depth (vertical length of the section), d_{b}, ratio, $b_{\mathrm{B}} / d_{\mathrm{b}}$, is less than five when turbulence intensity is small enough, such as for sea winds. Vortex-induced vibrations are possible for solid girders when turbulence intensity is small. The formulae are based on many wind tunnel test results for long-span bridges (see References [15], [16] and [12]).

E.3 Dynamically sensitive structures

E.3.1 Along-wind response

The equivalent static wind load distribution given by Equation (E.4) is proportional to the wind force coefficient, $C_{F\mathrm{m}}$, which represents the mean wind force distribution. Regarding the spatial distribution of the equivalent static wind load, different distributions are recommended for the mean component, the background component, and the resonant component (References [13] and [19]). The load-response correlation (LRC) formula proposed by Kasperski (1992) is a very useful tool for estimating the quasi-static component, but it is not easy to develop the formula into a generic form available in codes or regulations. It is recommended that the wind load distribution for the resonant component be proportional to the inertial load.

Reference [19] proposes the use of the same wind load distribution as the mean component for the background component as an expedient way, although the resonant component should be proportional to the inertial load. Accordingly, for dynamically sensitive structures, the mean dynamic response factor expressed by Equation (E.2) can be written as Equation (21):

$$\begin{aligned} C_{\mathrm{dyn,\,m}} &= 1 + 2 I_{\mathrm{vh}} \sqrt{g_{\mathrm{DB}}^2 B_{\mathrm{D}}^2 + g_{\mathrm{DR}}^2 R_{\mathrm{D}}^2} \\ &= 1 + \sqrt{G_{\mathrm{B}}^2 + G_{\mathrm{R}}^2} \end{aligned} \tag{E.21}$$

where

I_{vh} is the turbulence intensity of wind speed at the reference height given in Annex C;

B_{D} is the background response factor for the along-wind base bending moment;

R_{D} is the resonant response factor for the along-wind base bending moment.

G_B and G_R in Equation (E.21) are the dynamic response factors for background and resonant components, and given as Equations (E.22) and (E.23):

$$\begin{aligned} G_B &= 2I_{vh}g_{DB}B_D \\ &\approx 2I_{vh}g_v B_D \end{aligned} \quad \text{(E.22)}$$

$$G_R = 2I_{vh}g_{DR}R_D \quad \text{(E.23)}$$

where

g_{DB} is the peak factor for the background component, and can be equated to the peak factor, g_v, for the fluctuating wind speed, with 3,4 as an appropriate value;

g_{DR} is the peak factor for the resonant component, which can be calculated using Equation (E.24):

$$g_{DR} = \sqrt{2\ln(\nu T)} + \frac{0{,}577\,2}{\sqrt{2\ln(\nu T)}} \quad \text{(E.24)}$$

where

ν is the cycling rate of the vibration and approximated by the first mode natural frequency of the along-wind vibration, f_D;

T is the averaging time for the site mean velocity pressure.

For tall structures cantilevered at the base, such as tall buildings, chimneys and towers, the quantities B_D and R_D are to be determined by an appropriate method based on the maximum base bending moment.

An example of the closed form expressions of B_D and R_D is shown as follows:

$$B_D = \frac{1+0{,}2\beta}{1+\dfrac{0{,}63\left(\dfrac{\sqrt{bh}}{L_{vh}}\right)^{0{,}56}}{\left(\dfrac{h}{b}\right)^{\gamma}}} \quad \text{(E.25)}$$

and

$$R_D = (1+0{,}6\beta)\frac{3}{2+k}K\sqrt{\frac{\pi}{4\zeta_D}E_D S\left(0{,}57-0{,}35\beta+r\right)} \quad \text{(E.26)}$$

where

β is the power law exponent of mean wind speed profile given in Annex C;

b is the breadth of the structure;

h is the reference height (mean roof height);

L_{vh} is the turbulence scale at the reference height given in Annex B;

γ is a constant, of which appropriate values are 0,15 for $h/b < 1$; and 0,07 for $h/b \geqslant 1$;

K is the mode correction factor given in Equation (E.27);

ζ_D is the damping ratio for the along-wind vibration;

E_D is the spectral energy factor given in Equation (E.28);

S is the size reduction factor given in Equation (E.29);

r is the factor representing correlation effects of the windward pressures and the leeward pressures and is defined in Equation (E.30).

The mode correction factor and other parameters in Equation (E.26) can be given using Equations (E.27) to (E.30):

$$K = 0{,}27k + 0{,}73 \tag{E.27}$$

$$E_D = \frac{4\left(\dfrac{f_D L_{vh}}{V_{site,m}}\right)}{\left\{1 + 70{,}8\left(\dfrac{f_D L_{vh}}{V_{site,m}}\right)^2\right\}^{5/6}} \tag{E.28}$$

$$S = \frac{0{,}9}{\sqrt{1 + 6\left(\dfrac{f_D h}{V_{site,m}}\right)^2}\left(1 + 3\dfrac{f_D b}{V_{site,m}}\right)} \tag{E.29}$$

$$r = \frac{2\sqrt{0{,}053 - 0{,}042\beta}}{1 + 20\left(\dfrac{f_D b}{V_{site,m}}\right)} \tag{E.30}$$

where

k is the mode shape power exponent for the first mode of the vibration expressed as $\phi_{1z} = \left(\frac{z}{h}\right)^k$;

f_D is the first mode natural frequency of the along-wind vibration;

$V_{site,m}$ is the site mean wind speed.

NOTE The spectral energy factor expressed by Equation (E.28) is based on the Karman type wind spectrum.

The above closed form expressions are given as an example of the mean dynamic response factor based on the maximum base bending moment (Reference [19]). A more sophisticated method is given in Reference [13] for different load effects such as shear force and bending moment at any height.

The background component, F_B, of the equivalent static wind load per unit height at height, z, is given using Equation (E.31):

$$\begin{aligned} F_B &= G_B F_m \\ &= G_B q_{site,m} C_{Fm} b \end{aligned} \tag{E.31}$$

F_m in Equation (E.31) is the mean wind load per unit height at height, z, which can be calculated from Equation (E.4) by setting $C_{\text{dyn,m}} = 1$ and $A_{\text{ref}} = b$ as follows in Equation (E.32):

$$F_m = q_{\text{site,m}} C_{Fm} b \tag{E.32}$$

The resonant component, F_R, of the equivalent static wind load per unit height at height, z, is given using Equation (E.33):

$$F_R = (2+k) q_{\text{site,m}} C_M \left(\frac{z}{h}\right)^k G_R b \tag{E.33}$$

where

C_M is the mean along-wind base bending moment coefficient defined by $C_M = \dfrac{M_m}{q_{\text{site,m}} h^2 b}$, and can be determined from the pressure coefficients given in Annex D;

M_m is the mean base bending moment due to the mean wind load distribution, F_m.

An example of an empirical formula for the mean along-wind base moment coefficient for rectangular prisms is given in Equation (E.34):

$$C_M = 0{,}7\left(1 - 0{,}1 d/b\right)\left(1 - 0{,}4\beta\right) \tag{E.34}$$

for $d / b > 3$ use $d / b = 3$

where

b is the windward face depth;

d is the streamwide face depth;

β is the power law index of mean wind speed profile.

Combining the load effects due to the mean, the background and the resonant wind load components, the maximum or peak along-wind loading effect, W_D, is given as Equation (E.35):

$$W_D = W_m + \sqrt{W_B^2 + W_R^2} \tag{E.35}$$

where

W_m is the mean load effect due to the mean wind load component, F_m, given using Equation (E.32);

W_B is the load effect due to the background wind load component, F_B, given using Equation (E.31);

W_R is the load effect due to the resonant wind load component, F_R, given using Equation (E.33).

E.3.2 Crosswind response

The crosswind equivalent static load per unit height at height, z, of a tall building with a rectangular cross-section, F_L, should be estimated by a similar formula to Equation (E.4) based on the site mean velocity pressure using Equation (E.36):

$$F_L = q_{\text{site,m}} \left(C_{Fm} C_{\text{dyn,m}}\right)_L b \tag{E.36}$$

Equation (E.37) can be used to give the product of the aerodynamic shape factor and the mean dynamic response factor $\left(C_{\mathrm{Fm}}C_{\mathrm{dyn,\,m}}\right)_{\mathrm{L}}$ (Reference [17]):

$$\left(C_{\mathrm{Fm}}C_{\mathrm{dyn,\,m}}\right)_{\mathrm{L}} = 3C'_{\mathrm{L}}\left(\frac{z}{h}\right)^{k} g_{\mathrm{L}}\sqrt{1+R_{\mathrm{L}}^{2}} \tag{E.37}$$

where

$q_{\mathrm{site,\,m}}$ is the site mean velocity pressure;

C'_{L} is the standard deviation of the crosswind base bending moment coefficient given by $C'_{\mathrm{L}} = \dfrac{\sigma_{\mathrm{ML}}}{q_{\mathrm{site,\,m}}h^{2}b}$, and an empirical formula is given in Equation (E.38);

σ_{ML} is the standard deviation of the crosswind base bending moment;

g_{L} is the peak factor of the crosswind base bending moment, and is given as $g_{\mathrm{L}} = \sqrt{2\ln(\nu T)} + \dfrac{0{,}577\,2}{\sqrt{2\ln(\nu T)}}$;

ν is the cycling rate of the vibration and is approximated by the first mode natural frequency of the crosswind vibration, f_{L};

T is the averaging time for the site mean velocity pressure;

R_{L} is the resonance factor for the crosswind base bending moment, and is given as $R_{\mathrm{L}} = K\sqrt{\dfrac{\pi E_{\mathrm{L}}}{4\zeta_{\mathrm{L}}}}$;

K is the mode shape correction factor given as Equation (E.27);

E_{L} is the crosswind force spectrum coefficient given as Equation (E.39);

ζ_{L} is the damping ratio for the first mode of crosswind vibration.

Equation (E.38) gives an example of an empirical formula of the standard deviation of the crosswind base bending moment coefficient, C'_{L} (Reference [17]):

$$C'_{\mathrm{L}} = 0{,}0082\left(\frac{d}{b}\right)^{3} - 0{,}07\left(\frac{d}{b}\right)^{2} + 0{,}22\frac{d}{b} \tag{E.38}$$

where

b is the breadth of the structure;

d is the depth of the structure.

The crosswind force spectrum coefficient, E_L, generalized for a linear mode shape can be obtained by the high-frequency force balance test in a wind tunnel. Its empirical formula is given as a function of the side ratio, d/b, the aspect ratio, h/b, turbulence intensity, I_vh, and so on. Equation (E.39) gives an example of the empirical formula of E_L (Reference [17]):

$$E_\mathrm{L} = \sum_{j=1}^{n} \left[\frac{4\kappa j\left(1+0{,}6\beta_j\right)\beta j}{\pi} \frac{\left(\dfrac{f_\mathrm{L}}{f_\mathrm{sj}}\right)^2}{\left\{1-\left(\dfrac{f_\mathrm{L}}{f_\mathrm{sj}}\right)^2\right\}^2 + 4\beta_j^2\left(\dfrac{f_\mathrm{L}}{f_\mathrm{sj}}\right)^2} \right] \tag{E.39}$$

where

$$n = \begin{cases} 1, & \text{for } d/b < 3 \\ 2, & \text{for } d/b \geqslant 3 \end{cases}$$

$\kappa_1 = 0{,}85$ and $\kappa_2 = 0{,}02$

$$f_\mathrm{s1} = \frac{0{,}12}{\left\{1+0{,}38\left(\dfrac{d}{b}\right)^2\right\}^{0{,}89}} \frac{V_\mathrm{site,\,m}}{b}$$

$$f_\mathrm{s2} = \frac{0{,}56}{\left(\dfrac{d}{b}\right)^{0{,}85}} \frac{V_\mathrm{site,\,m}}{b}$$

$$\beta_1 = \frac{\left(\dfrac{d}{b}\right)^4 + 2{,}3\left(\dfrac{d}{b}\right)^2}{2{,}4\left(\dfrac{d}{b}\right)^4 - 9{,}2\left(\dfrac{d}{b}\right)^3 + 18\left(\dfrac{d}{b}\right)^2 + 9{,}5\dfrac{d}{b} - 0{,}15} + \frac{0{,}12}{\dfrac{d}{b}}$$

$$\beta_2 = \frac{0{,}28}{\left(\dfrac{d}{b}\right)^{0{,}34}}$$

These expressions for estimating crosswind loads are valid for tall buildings having a uniform rectangular section from the bottom to the top with a side ratio, d/b, ranging from 0,2 to 5, constructed in urban area, when wind is normal to the front face. These expressions are also valid only for the building with an aspect ratio, $h/\sqrt{bd}$, ranging from 3 to 6. If the aspect ratio or the side ratio is outside the above ranges, or if the building is constructed in an open flat terrain or a seaside, a special investigation such as a wind tunnel test is recommended to obtain more suitable information.

E.3.3 Torsional response

The torsional equivalent static wind load, i.e. torsional moment, per unit height at height, z, of a building with a rectangular cross-section, M_T, should be estimated based on the mean site velocity pressure as follows:

$$M_\mathrm{T} = q_\mathrm{site,\,m}\left(C_{F\mathrm{m}} C_\mathrm{dyn,\,m}\right)_\mathrm{T} b^2 \tag{E.40}$$

where Equation (E41) gives the product of the aerodynamic shape factor and the mean dynamic response factor $\left(C_{F\mathrm{m}}C_{\mathrm{dyn,\,m}}\right)_{\mathrm{T}}$ (Reference [17]):

$$\left(C_{F\mathrm{m}}C_{\mathrm{dyn,\,m}}\right)_{\mathrm{T}} = 3C'_{\mathrm{T}}\left(\frac{z}{h}\right)^{k} g_{\mathrm{T}}\sqrt{1+R_{\mathrm{T}}^{2}} \quad \text{(E.41)}$$

where

C'_{T} is the standard deviation of the torsional moment coefficient at the base of the structure given by $C'_{\mathrm{T}} = \dfrac{\sigma_{\mathrm{MT}}}{q_{\mathrm{site,\,m}}hb^{2}}$, and an empirical formula is given in Equation (E.42);

σ_{MT} is the standard deviation of the torsional moment at the base of the structure;

g_{T} is the peak factor of torsional moment at the base of the structure, and is given as $g_{\mathrm{T}} = \sqrt{2\ln(\nu T)} + \dfrac{0{,}577}{\sqrt{2\ln(\nu T)}}$;

ν is the cycling rate of the vibration and is approximated by the lowest natural frequency of the torsional vibration, f_{T};

T is the averaging time for the site mean velocity pressure;

R_{T} is the resonance factor for torsional base moment of the structure, and is given by $R_{\mathrm{T}} = K\sqrt{\dfrac{\pi E_{\mathrm{T}}}{4\zeta_{\mathrm{T}}}}$;

K is the mode shape correction factor given by Equation (E.27);

E_{T} is the torsional moment spectral energy factor;

ζ_{T} is the damping ratio for the first mode for torsional vibration.

Equation (E.42) gives an example of an empirical formula of the standard deviation of the torsional moment coefficient, C'_{T} (Reference [17]):

$$C'_{\mathrm{T}} = \left\{0{,}003\,4 + 0{,}007\,8\left(\frac{d}{b}\right)^{2}\right\}^{0{,}78} \quad \text{(E.42)}$$

Equation (E.43) gives an example of an empirical formula of the torsional moment spectral energy factor, E_{T}, generalized for a uniform mode shape (Reference [17]):

$$E_{\mathrm{T}} = \begin{cases} \dfrac{0{,}14\,J_{\mathrm{T}}^{2}\left(V_{\mathrm{T}}^{*}\right)^{2\beta_{\mathrm{T}}}}{\pi}\dfrac{d(b^{2}+d^{2})^{2}}{l^{2}b^{3}} & V_{\mathrm{T}}^{*} \leqslant 4{,}5 \text{ and } 6 \leqslant V_{\mathrm{T}}^{*} \leqslant 10 \\ E_{4.5}\exp\left[3{,}5\ln\left(\dfrac{E_{6}}{E_{4.5}}\right)\ln\left(\dfrac{V_{\mathrm{T}}^{*}}{4{,}5}\right)\right] & 4{,}5 < V_{\mathrm{T}}^{*} < 6 \end{cases} \quad \text{(E.43)}$$

where

$V_T^* = \dfrac{V_{\text{site, m}}}{f_T\sqrt{(bd)}}$ is the reduced mean wind speed;

l is the larger of b and d;

$E_{4,5}$ and E_6 are the values of E_T at V^*_T = 4,5 and 6, respectively, using the first equation for E_T;

$$J_T = \begin{cases} \dfrac{-1{,}1\dfrac{d}{b}+0{,}97}{\left(\dfrac{d}{b}\right)^2+0{,}85\dfrac{d}{b}+3{,}3}+0{,}17 & V_T^* \leqslant 4{,}5 \\[2ex] \dfrac{0{,}077\dfrac{d}{b}-0{,}16}{\left(\dfrac{d}{b}\right)^2-0{,}96\dfrac{d}{b}+0{,}42}+\dfrac{0{,}35}{\dfrac{d}{b}}+0{,}095 & 6 \leqslant V_T^* \leqslant 10 \end{cases}$$

$$\beta_T = \begin{cases} \dfrac{\dfrac{d}{b}+3{,}6}{\left(\dfrac{d}{b}\right)^2-5{,}1\dfrac{d}{b}+9{,}1}+\dfrac{0{,}14}{\dfrac{d}{b}}+0{,}14 & V_T^* \leqslant 4{,}5 \\[2ex] \dfrac{0{,}44\left(\dfrac{d}{b}\right)^2-0{,}006\,4}{\left(\dfrac{d}{b}\right)^4-0{,}26\left(\dfrac{d}{b}\right)^2+0{,}1}+0{,}2 & 6 \leqslant V_T^* \leqslant 10 \end{cases}$$

The above expressions are valid for tall buildings with an aspect ratio, $h/\sqrt{(bd)}$ ranging from 3 to 6 and a side ratio, d/b, ranging from 0,2 to 5 constructed in an urban area. These expressions are also valid only for a reduced wind speed $V_T^* \leqslant 10$. The mass distribution along the vertical axis is assumed as almost uniform. If the aspect ratio or the side ratio is outside the above ranges, or if the structure is constructed in an open flat terrain or a seaside, a special investigation such as a wind tunnel test is recommended to obtain more suitable information. Special attention is paid to torsional vibrations for structures with a large eccentricity, an asymmetric plane, or a small torsional rigidity.

A more accurate estimation can be made by using wind tunnel data directly. An example is the 3-D Gust Loading Factor for estimating dynamic load components in three directions based on an aerodynamic loading database (Reference [20]). The aerodynamic loading database can be obtained from high-frequency force balance tests or simultaneously monitored surface pressure measurements on scaled building models. The internet provides the opportunity to pool and archive the international stores of wind tunnel data. An "e-database" of aerodynamic wind loads based on high-frequency base-balance (HFBB) measurements on a host of isolated tall building models is accessible to the worldwide internet community at some authorized web sites (Reference [20]). Users can select the geometry and dimensions of a model building from the available choices and specify an urban or suburban condition.

E.4 Methodology based on peak velocity pressure

The methodology given in this clause should be used for response calculations based on the site peak velocity pressure, q_{site}, calculated according to Equation (4) in Clause 7.

E.4.1 General expression of equivalent static wind load

The equivalent static wind load based on the site peak velocity pressure is given as Equations (E.44) and (E.45):

$$F_{\text{loc}} = q_{\text{site}} C_{\text{p}} C_{\text{dyn}} A \tag{E.44}$$

$$F = q_{\text{site}} C_{\text{F}} C_{\text{dyn}} A_{\text{ref}} \tag{E.45}$$

where

q_{site} is the site peak velocity pressure;

C_{p} and C_{F} are the pressure coefficient and force coefficient, respectively;

C_{dyn} is the peak dynamic response factor based on the site peak velocity pressure;

A and A_{ref} are the area of application and projected area normal to the wind direction of the building at height, z.

The peak dynamic response factor, C_{dyn}, can be expressed by the mean dynamic response factor, $C_{\text{dyn, m}}$, using Equation (E.46):

$$C_{\text{dyn}} = \frac{C_{\text{dyn, m}}}{1 + 2 g_{\text{v}} I_{\text{vh}}} \tag{E.46}$$

where

g_{v} is the peak factor for the fluctuating wind speed, with 3,4 as an appropriate value (see Annex B);

I_{vh} is the turbulence intensity of wind speed at the reference height, given in Annex B.

For the rigid structures discussed in E.2.1, the value of C_{dyn} is generally less than unity due to the size reduction effect. However, the peak dynamic response factor, C_{dyn}, can simply be taken as unity for a small rigid structure, as stated in Clause 12.

E.4.2 Along-wind response

The peak dynamic response factor, C_{dyn}, is given as Equation (E.46) which can be written as Equation (E.47):

$$C_{\text{dyn}} = \frac{1 + 2 I_{\text{vh}} \sqrt{g_{\text{DB}}{}^2 B_{\text{D}}{}^2 + g_{\text{DR}}{}^2 R_{\text{D}}{}^2}}{1 + 2 g_{\text{v}} I_{\text{vh}}} \tag{E.47}$$

where g_{DB}, B_{D}, g_{DR} and R_{D} are defined in E.3.1 [Equations (E.24), (E.25) and (E.26)].

The peak along-wind base bending moment is given as Equation (E.48):

$$M = C_{\text{dyn}} \int_0^h q_{\text{site}} C_{\text{F}} bz \, dz \tag{E.48}$$

The resonant component of C_{dyn} can be written as Equation (E.49):

$$C_{dyn,R} = \frac{2I_{vh} g_{DR} R_D}{1 + 2g_v I_{vh}} \tag{E.49}$$

The resonant along-wind base bending moment is then given as Equation (E.50):

$$M_R = \frac{C_{dyn,R}}{C_{dyn}} M = \frac{2I_{vh} g_{DR} R_D}{1 + 2I_{vh}\sqrt{g_{DB}{}^2 B_D{}^2 + g_{DR}{}^2 R_D{}^2}} M \tag{E.50}$$

Then, the resonant component of the equivalent static wind load per unit height is obtained by distributing the resonant base moment, M_R, in proportion to the mode shape, as given in Equation (E.51):

$$F_R = \frac{(2+k)}{h^2} M_R \left(\frac{z}{h}\right)^k \tag{E.51}$$

The mean component of the equivalent static wind load per unit height is given as Equation (E.52):

$$F_m = \frac{1}{1 + 2g_v I_{vh}} q_{site} C_F b \tag{E.52}$$

NOTE This is not the same distribution as that given by Equation (E.32).

The background component of the equivalent static wind load per unit height is given as Equation (E.53):

$$F_B = C_{dyn,B} q_{site} C_F b \tag{E.53}$$

where

$$C_{dyn,B} = \frac{2I_{vh} g_{DB} B_D}{1 + 2g_v I_{vh}} \tag{E.54}$$

The separate load distributions due to the mean [Equation (E.52)], background [Equation (E.53)] and resonant [Equation (E.51)] contributions can then be applied to obtain load effects, W_m, W_B and W_R. These are then combined to give a peak along-wind load effect according to Equation (E.34).

E.4.3 Crosswind response

The crosswind equivalent static load based on the site peak velocity pressure should be estimated by a formula similar to Equation (E.36) as follows:

$$F_L = q_{site} \left(C_F C_{dyn}\right)_L b \tag{E.55}$$

where $\left(C_F C_{dyn}\right)_L$ can be expressed by $\left(C_F C_{dyn,m}\right)_L$, given as Equation (E.37) in E.3.2, as Equation (E.56):

$$\left(C_F C_{dyn}\right)_L = \frac{\left(C_F C_{dyn,m}\right)_L}{1 + 2g_v I_{vh}} \tag{E.56}$$

E.4.4 Torsional response

The torsional equivalent static load based on the site peak velocity pressure should be estimated by a formula similar to Equation (E.40), as follows in Equation (E.57):

$$M_{\mathrm{T}} = q_{\mathrm{site}} \left(C_{\mathrm{F}} C_{\mathrm{dyn}} \right)_{\mathrm{T}} b^{2} \tag{E.57}$$

where $\left(C_{\mathrm{F}} C_{\mathrm{dyn}} \right)_{\mathrm{T}}$ can be expressed by $\left(C_{\mathrm{F}} C_{\mathrm{dyn,\,m}} \right)_{\mathrm{T}}$ given by Equation (E.41) in E.3.3 as follows:

$$\left(C_{\mathrm{F}} C_{\mathrm{dyn}} \right)_{\mathrm{T}} = \frac{\left(C_{\mathrm{F}} C_{\mathrm{dyn,\,m}} \right)_{\mathrm{T}}}{1 + 2 g_{\mathrm{v}} I_{\mathrm{vh}}} \tag{E.58}$$

E.5 Peak accelerations

The along-wind, crosswind, and torsional peak accelerations can be found from Equations (E.59), (E.60) and (E.61):

$$\ddot{x}_{\mathrm{D}} = \left(2\pi f_{\mathrm{D}} \right)^{2} \frac{2 g_{\mathrm{D}} I_{\mathrm{vh}} R_{\mathrm{D}}}{1 + 2 g_{\mathrm{D}} I_{\mathrm{vh}} \sqrt{B_{\mathrm{D}}^{2} + R_{\mathrm{D}}^{2}}} x_{\mathrm{D}} \tag{E.59}$$

$$\ddot{x}_{\mathrm{L}} = \left(2\pi f_{\mathrm{L}} \right)^{2} \frac{R_{\mathrm{L}}}{\sqrt{1 + R_{\mathrm{L}}^{2}}} x_{\mathrm{L}} \tag{E.60}$$

$$\ddot{\theta}_{\mathrm{T}} = \left(2\pi f_{\mathrm{T}} \right)^{2} \frac{R_{\mathrm{T}}}{\sqrt{1 + R_{\mathrm{T}}^{2}}} \theta_{\mathrm{T}} \tag{E.61}$$

where

$\ddot{x}_{\mathrm{D}}$ is the peak along-wind acceleration;

$\ddot{x}_{\mathrm{L}}$ is the peak crosswind acceleration;

θ_{T} is the peak angular acceleration;

x_{D} is the peak along-wind deflection including the mean component obtained by the along-wind equivalent static load specified in E.3.1 and E.4.2;

x_{L} is the peak crosswind deflection obtained by the crosswind equivalent static load specified in E.3.2 and E.4.3;

θ_{T} is the peak angular deflection obtained by the torsional equivalent static wind load specified in E.3.3 and E.4.4.

The habitability of buildings with respect to wind-induced vibrations is assessed by ISO 10137, based on peak accelerations.

E.6 Damping

The damping ratios, ζ, in the formulae for resonance components appearing in E.3 are the total damping ratio expressed as a ratio to the critical damping. It consists of the structural damping ratio, ζ_{str}, the aerodynamic damping ratio, ζ_{aer}, and the additional damping ratio, ζ_{aux}, provided by auxiliary damping devices or damping materials. Then, the damping ratio is expressed using Equation (E.62):

$$\zeta = \zeta_{\text{str}} + \zeta_{\text{aer}} + \zeta_{\text{aux}} \tag{E.62}$$

E.6.1 Structural damping

The damping ratio, ζ, can be equated to the structural damping, ζ_{str}, in many cases. The structural damping comes from various sources such as material damping, friction damping at contact surfaces of connections, main frames, secondary members and cladding, and soil-structure interactions.

Representative values of structural damping are given in Table E.3. Different values are given for different structural types, for different materials, for different heights, and for different amplitude levels (Reference [18]).

NOTE There is high variability in values of damping found in structures.

Table E.3 — Representative values of structural damping ratio, ζ_{str}

Type of structure		Material	
		Steel	**Concrete**
Building	$h = 40$ m	0,018	0,02
	$h = 50$ m	0,015	0,02
	$h = 60$ m	0,015	0,015
	$h = 70$ m	0,015	0,015
	$h > 80$ m	0,01	0,012
Chimney		0,002 [a]	0,01
		0,005 [b]	
Lattice tower		0,001	—

NOTE 1 The damping ratio is equal to the logarithmic decrement of damping divided by 2π.

NOTE 2 Lower values (75 % of the above-mentioned values) are recommended for evaluation of habitability to horizontal vibrations of structures.

[a] Unlined, all welded.

[b] Lined.

It is well-known that the damping ratio increases with amplitude, but the natural frequency decreases. These facts can be represented by so-called "stick-slip" phenomena generating friction damping at contact surfaces, so its contribution is significant. The contribution of the soil-structure interactions is also significant, especially for low-rise structures. The soil-structure interaction appears in damping predictors as a term proportional to the natural frequency. The contribution of the material damping is not significant, especially in steel structures.

E.6.2 Aerodynamic damping

Flexible structures vibrating in the wind experience the effects of aerodynamic damping as well as structural damping. For slender structures, the aerodynamic damping ratio can be expressed using Equation (E.63):

$$\zeta_{\text{aer}} = \left(\frac{\rho_{\text{air}} d^2}{m_l}\right)\left(\frac{V_{\text{site,m}}}{f_0 d}\right) C_{\text{aer}} \tag{E.63}$$

In Equation (E.63) d and m_l are representative length and mass per unit length of the cross-section, and f_0 is the natural frequency. The aerodynamic damping coefficient, C_{aer}, is a function of the geometry and the reduced wind speed, $V_{site,m}/(f_0 d)$. At large enough values of the reduced wind speed (say greater than 10), the value of C_{aer} usually approaches a quasi-steady value in the approximate range of ±0,5.

As a guide it is useful to determine the magnitude of the product of the first two terms on the right-hand side of Equation (E.63); if this yields a value greater than approximately 0,01, the aerodynamic damping is likely to be of comparable magnitude to the structural damping.

It should be noted that the aerodynamic damping can be negative for the particularly wind-sensitive structures treated in Annex F. If it is, and if it is large enough that the sum of the aerodynamic damping and the structural damping is less than zero, instability will result and lead to steadily increasing amplitude. Negative aerodynamic damping is encountered with many shapes including rectangular building shapes and some bridge cross-sections.

Galloping is one of the most commonly known crosswind vibrations due to motion-induced negative aerodynamic damping. The value of C_{aer} in Equation (E.63) is approximately $\{(\partial C_L/\partial\theta)+C_D\}/(4\pi)$, where C_L is the lift coefficient (shape factor for the crosswind direction), C_D is the drag coefficient (shape factor for the along-wind direction), and θ is the angle of attack. Galloping can arise for a structure with a sectional shape that has a negative slope, $\partial C_L/\partial\theta$, with a magnitude larger than C_D.

It is also encountered at vortex shedding frequencies and is responsible for the large amplitude motion occasionally associated with this.

Equation (E.63) is useful for determining only approximate magnitudes.

For along-wind aerodynamic damping, the value of C_{aer} in Equation (E.63) is approximately $C_D/(4\pi)$. The along-wind aerodynamic damping ratio for general buildings is positive and generally negligible.

E.6.3 Auxiliary damping

A variety of devices or materials for auxiliary damping, such as dampers based on friction, material yield, visco-elasticity and viscous fluid; tuned mass and tuned liquid dampers; as well as active control systems, are widely used in particular for slender and flexible tall buildings. Irrespective of the different ways of implementation, i.e. passive, active or hybrid, the two major mechanisms of vibration control are energy dissipation and vibration isolation. The effectiveness of these devices should be appropriately evaluated, based on experimental and theoretical investigations with support by experts, and the verified auxiliary damping ratio, ζ_{aux}, should be used.

Annex F
(informative)

Structures subject to critical excitation vortex resonance and aeroelastic instability

F.1 General

This annex describes vortex resonance and aeroelastic instability for tall and slender buildings and structures and their components referred to in Clause 11. Particularly wind-sensitive structures satisfying the conditions specified in E.2.3 can produce a significant vibration due to the vortex shedding or aeroelastic instability.

F.2 Vortex resonance

Slender, free-standing cylindrical structures, such as chimneys, masts, observation towers and members of space frames should be designed to resist the dynamic effect of vortex shedding. When the wind blows across a slender prismatic or cylindrical body, vortices are shed alternately from one side and then the other giving rise to fluctuating forces acting along the length of the body at right angles to the wind direction and the axis of the body. There is, in addition, the tendency for the aerodynamic damping to become negative.

There have been several studies on methods of estimating the vortex resonance of a structure with a circular section, e.g. Reference [21]. Almost all are rather hypothetical, and there is nothing commonly accepted.

This clause presents an empirical method of estimating the equivalent static wind load for vortex resonance of a circular cylinder, but it should be noted that it is approximate and should be used as diagnostic advice.

The equivalent static wind load per unit length can be calculated from Equation (F.1):

$$F_\mathrm{L} = \left(2\pi f_\mathrm{L}\right)^2 m_l \phi_\mathrm{i1} x_\mathrm{L} \tag{F.1}$$

where

f_L is the first mode natural frequency of the crosswind vibration;

m_l is the mass per unit length;

ϕ_i1 is the first mode shape of the crosswind vibration normalized to unity at the top of the circular free-standing structure, or at the centre of the circular member supported at both ends;

x_L is the peak displacement at the top of the free-standing structure, or at the centre of the member.

The peak displacement, x_L, in Equation (F.1) is approximately given as Equation (F.2):

$$x_\mathrm{L} = \frac{Kd}{S_\mathrm{C}{}^{\lambda}} \tag{F.2}$$

where

K and λ are parameters depending on Reynolds number, $R_e \approx 6{,}7 V_{hcr} d \times 10^4$, and the design target;

d is the average diameter over the top third of the free-standing structure or the diameter of members;

S_C is the Scruton Number defined as $S_C = 4\pi\zeta_{str} m_l \big/ \left(\rho_{air} d^2\right)$;

V_{hcr} is the critical wind speed specified in E.2.3 in Annex E.

The mass per unit length, m_l, and the diameter, d, should be averaged over the top third for the free-standing structure.

Appropriate values of parameters K and λ for circular free-standing structures are given as:

$K = 0{,}65$ and $\lambda = 1$ for $R_e < 3 \times 10^5$;

$K = 0{,}35$ and $\lambda = 1{,}2$ for $R_e \geqslant 3 \times 10^5$.

Those for circular members supported at both ends are given as:

$K = 1{,}3$ and $\lambda = 1$ for $R_e < 3 \times 10^5$.

The recommendations of this clause apply to free-standing structures or members vibrating in the fundamental mode. For other mode shapes, a dynamic analysis would be appropriate although Equation (E.15) can be used to determine the critical wind speed, and the application of Equation (F.2) will yield a rough estimate of the effects of vortex shedding. Slender structures with cross-sections other than circular can also give rise to vortex shedding, but data are limited. In such cases wind-tunnel tests provide the most satisfactory method of estimating the likely response.

The peak displacement can be changed due to the surface roughness of the cylinder, the turbulence intensity of the approaching wind, interference effects of more than two chimneys, and so on. The above-mentioned estimation of the equivalent static wind load is only for an isolated chimney in a flow with a relatively low turbulence. Further investigations or wind tunnel tests with support by experts are recommended for more accurate estimation for individual cases.

F.3 Aeroelastic instability

The motion (or distortion) of a structure due to wind loads can cause aeroelastic forces that depend, in part, on the velocity of the structure itself. These forces are called aerodynamic damping forces and can either oppose or assist the motion. If they oppose the motion, they constitute positive aerodynamic damping; if they assist the motion, negative aerodynamic damping. The sum of the aerodynamic damping and the structural damping constitutes the total effective damping. The total resonant response of the structure is inversely related to the total damping.

As noted in E.6.2, if the aerodynamic damping is negative the total damping available is reduced, in some cases to zero. Under these circumstances the amplitudes of vibration become extremely large and the motion is described as unstable. The limitation of the amplitudes, if any, is defined by the “non-linear effects” of the system and flow. Generally, these amplitudes are prohibitively large; under certain circumstances, however, with suitably flexible structures, it is possible to accept the limiting amplitudes provided that fatigue is taken into account.

Aerodynamic damping forces will be generated whenever a structure is in motion through the flow. Negative aerodynamic damping forces can arise under a variety of circumstances. One of these, described as galloping, involves the transverse vibration of a structure and is associated with a “negative slope” to the side force/angle of attack relationship. The necessary characteristics are exhibited by both common structural shapes and iced cables. A particularly important situation is produced by the negative aerodynamic damping forces set up at the critical wind speed at which the vortex-shedding frequency coincides with the natural frequency of the structure. The forces are sometimes referred to as “locked-in” forces rather than negative aerodynamic damping forces.

In addition to galloping, negative aerodynamic characteristics are also found at certain wind speeds in torsional motion of bridge decks. This form of instability is called “torsional flutter”. Other forms of instability can occur involving the coupling of several vibration modes. These are described as “coupled flutter”. Torsional flutter and coupled flutter are only likely to affect light and flexible structures such as suspension bridges. In all these instances, the problem should be given special treatment.

Annex G
(informative)

Mode combinations

G.1 General

Wind pressures acting on a building randomly fluctuate in space and are always distributed asymmetrically, even if the wind direction is normal to a face of a building having a rectangular cross-section. A symmetric pressure pattern can only be obtained for a temporally averaged pressure distribution and never appears in a realistic situation. Even at the moment when the along-wind force component becomes a maximum, it is obvious that the crosswind and torsional components always accompany it. Thus, a structure should be designed considering a wind load combination of along-wind, crosswind and torsional wind loads.

If the design wind speed is estimated regardless of wind directionality and applied to any wind direction, consideration of the wind load combination can provide overestimation. However, if wind directionality is taken into account, more precise wind load estimation is required and the wind load combination must be considered.

G.2 Low-rise and medium-rise buildings

For buildings not satisfying the conditions of Equations (E.6) and (E.7), where crosswind and torsional responses are not significant, an increasing factor of $1+K_{\mathrm{C}}$ is recommended to be considered in estimating the peak normal stresses in columns.

The combination factor, K_{C}, is given by the following equation for buildings with a rectangular plan (Reference [23]).

$$K_{\mathrm{C}} = 0{,}35\frac{d}{b} \quad (\geqslant 0{,}2) \tag{G.1}$$

where

b is the breadth of the structure;

d is the depth of the structure.

G.3 Tall and slender buildings

For tall and slender buildings satisfying the conditions of Equations (E.6) and (E.7), the combination of along-wind, crosswind and torsional components must be considered. The wind load combination varies with the shape of the structure, natural frequencies of along-wind, crosswind and torsional vibrations, and so on. Table G.1 shows an example of expedient load combination rules (Reference [22]).

Table G.1 — Combinations of wind load effects

Combination	Along-wind force	Crosswind force	Torsional moment
1	W_D	$0{,}4W_\mathrm{L}$	$0{,}4W_\mathrm{T}$
2	$W_\mathrm{D}\left(0{,}4+\dfrac{0{,}6}{C_\mathrm{dyn,\,m}}\right)$	W_L	κW_T
3	$W_\mathrm{D}\left(0{,}4+\dfrac{0{,}6}{C_\mathrm{dyn,\,m}}\right)$	κW_L	W_T
NOTE W_D, W_L and W_T are the load effects due to the along-wind load, the crosswind load and the torsional load, specified in E.3.1, E.3.2 and E.3.3, respectively.			

The typical values of κ in Table G.1 are given in Table G.2. The values in Table G.2 are appropriate only for the general cases, where the first mode natural frequency of the torsional vibration is 30 % to 40 % higher than that of the crosswind vibration. If both the crosswind and torsional natural frequencies are closer than that, or the building has some eccentricity, more detailed examination shall be made for the wind load combinations.

Table G.2 — Typical values of κ

d/b	$\dfrac{f_1 b}{V_\mathrm{site,\,m}}$ 1)	κ
0,5	0,1	0,55
	0,2	0,65
	0,6	0,8
1,0	0,1	0,55
	0,3	0,55
	0,6	0,65
2,0	—	0,55

1) f_1 is the smaller of the first mode natural frequencies of crosswind and torsional vibrations, f_L and f_T.

2) For intermediate values of $f_1 b/V_\mathrm{site,\,m}$, use linear interpolation.

Annex H
(informative)

Wind tunnel testing

Pressure and force coefficients can be obtained from wind tunnel tests for use with the site peak and mean velocity pressures obtained from Clause 7 to give pressure and force loading data. Any wind tunnel tests should conform to the following principles.

a) For synoptic and tropical cyclone wind climates, the natural wind should be modelled to take account of the variation in mean wind speed and turbulence intensity above the ground, and the variation of wind speed over local topography and the variation in wind speed due to localized changes in surface roughness as defined in Annex C. In addition, the correct turbulence spectral characteristics should be modelled. Special care should be taken to ensure that the spectral density at frequencies corresponding to model scale are accurately reproduced, and the length scale of the model wind simulation shall be determined using either spectral fitting techniques or the integral scale.

NOTE It is not necessary for the full depth of the atmospheric boundary layer to be simulated.

b) For thunderstorm and tornado winds, methods should be used that adequately generate the spatial and temporal variation of wind speeds and turbulence characteristics within such systems.

c) The geometric scale of the model should be such that it is within a factor of two of the geometric scale of the wind simulation.

d) Measurements should be made to determine the pressure and force coefficients that are consistent with the methods outlined in Annex D.

e) Where specific load effects are required, the wind tunnel data that should be obtained should be that required by the LRC method in Annex D, i.e. the mean and rms values of pressure coefficients, the peak factor for the load response under consideration and the correlations between fluctuating pressure coefficients over the surface of the structure.

f) The response characteristics of the wind tunnel experimentation should be consistent with the above-mentioned measurements.

For dynamic structures, the mass, stiffness and damping characteristics of the full-scale structure should be either incorporated into the model according to established scaling laws, or appropriately accounted for in analysing the data obtained from a rigid model.

Annex I
(informative)

Computation-based methods

Pressure and force coefficients can in principle be obtained using suitable computational fluid dynamics (CFD) techniques and this methodology will improve with time and could become a promising tool. Requirements are the same as those outlined in Annex H for wind tunnel measurements, but it should be noted that with the current state of development of CFD techniques, such methods are not able to fully reproduce the fluctuating flow characteristics required to obtain the appropriate fractile of the extreme value distribution of pressure coefficients, or the correct correlations between fluctuating pressure coefficients over the surface to give large area (or global) force or moment coefficients. Until this can be done, the use of such methods for force and pressure coefficient determination is not recommended.

Annex J
(informative)

Reliability considerations

The required degree of reliability of a structure under wind action should be determined taking into account:

a) the cause and mode of failure;

b) the consequence of failure in terms of risk to life, economy and social fabric;

c) the cost of reducing the risk of failure;

d) the social and environmental conditions of a particular location.

The differentiation of the required degrees of reliability can be obtained by the classification of the whole structure or by the classification of the structural elements or by a combination of both. Reliability differentiation by classification of structural elements is best done by adjustment of the resistance factors for the elements. Reliability differentiation by classification of the whole building is best done by adjustment to the load factor or the selection of the design event. Table J.1 is an example of the classification system for whole buildings.

Table J.1 — Importance levels of buildings and structures

Importance level	Building or structure type
1	Buildings or structures presenting a low degree of hazard to life and other property in the case of failure.
2	Buildings or structures not included in importance levels 1, 3 and 4.
3	Buildings or structures that are designed to contain a large number of people.
4	Buildings or structures that are essential to post-disaster recovery or associated with hazardous facilities.

The choice of the values of wind action in conjunction with the choice of the degree of reliability for the design calculation is the usual method for establishing the required degree of reliability for structures under wind. Table J.2 is an example of such a system.

Table J.2 — Examples of relationship between classification system, design value for wind action and degree of reliability of structure

Importance level	Required degree of reliability for design under wind in terms of life-time target reliability indices (probability of failure)	Values of design wind action in terms of annual probability of excedance
1	2,3 (10^{-2})	1:200
2	3,1 (10^{-3})	1:500
3	3,7 (10^{-4})	1:1 000
4	4,2 (10^{-5})	1:2 000

Reliability assessment should be carried out in accordance with ISO 2394. Target reliability indices are usually established by calibration. Once chosen, the design value of wind action is determined by selecting the appropriate wind action factor together with a representative value for the wind action (the European method) or by selecting the appropriate design event in terms of annual probability of excedance of that event.

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