Concise Eurocodes: Loadings on Structures

BS EN 1991: Eurocode 1

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BS EN 1991-1-1:2002. Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings. British Standards Institution, 2004

BS EN 1991-1-3:2003. Eurocode 1 – Actions on structures. Part 1-3: General actions – Snow loads. British Standards Institution, 2004

BS EN 1991-1-4:2005. Eurocode 1: Actions on structures. Part 1-4: General actions – Wind actions. London, British Standards Institution, 2005

NA to BS EN 1990:2002+A1:2005 UK National Annex for Eurocode – Basis of structural design. British Standards Institution, 2009

NA to BS EN 1991-1-1:2002. UK National Annex to Eurocode 1: Actions on structures – Part 1.1: General actions – Densities, self-weight, imposed loads for buildings. British Standards Institution, 2002

NA to BS EN 1991-1-3:2003. UK National Annex to Eurocode 1: Actions on structures. Part 1-3: General actions – Snow loads. British Standards Institution, 2007

UK NA to BS EN 1991-1-4:2005. National Annex to Eurocode 1: Actions on structures. Part 1-4: General actions – Wind actions. London, British Standards Institution, 2008

Other sources:

Department for Communities and Local Government. *Guide to the Use of EN 1990 Basis of Structural Design*. DCLG Publications, 2006

The Institution of Structural Engineers. *Manual for the design of building structures to Eurocode 1*. Institution of Structural Engineers, 2008

Cook, N. *Designers' Guide to EN 1991-1-4: 2005 Eurocode 1: Actions on structures. Part 1-4: General actions – Wind actions*. London, Thomas Telford, 2007

A note on references

NOTE The source references in the left-hand margins relate to tables, figures or equations from the aforementioned documents.

The prefix	**'EC1-1-x'**	relates to **BS EN 1991-1-x: Eurocode 1: Actions on structures. Part 1-x**
and	**'EC0'**	relates to **BS EN 1990: Eurocode 0: Basis of structural design**

When a reference is prefixed by **'NA'** it relates to the appropriate **UK National Annex to Eurocode 1.**

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Introduction

This guide is designed to help both practising engineers and undergraduate students of civil and structural engineering to assess the main loadings on buildings in design assignments, on the basis of *BS EN 1991: Eurocode 1: Actions on Structures*. Eurocode 1 is a lengthy, and often involved, document which can appear daunting both to students and to practising engineers wishing to upgrade their expertise to take in Eurocode-based design processes.

Eurocode 1, in its seven major parts, covers all the categories of loading ('actions' in Eurocode terminology) with which structures in all parts of the world may need to cope within their lifespan, including the predictable loadings caused by gravity, wind and, in many locations, snow; it also deals with the extreme cases of hazard loading, such as fire. Only the main loadings routinely used in building design in the UK are covered here, and there is no intention to make the treatment encyclopaedic. It is assumed that the guide is to be used in the UK, and where references are made to National Annexes the UK National Application documents have been used. Students can trace the source of particular information by following the source references in the left-hand margins. Eurocode 1 is abbreviated to EC1 in the left margin.

A brief sequence of the steps involved in determining the various design loadings is provided in each section in the form of a flowchart, and the subsequent text then addresses the processes which make up the appropriate flowchart.

The Eurocodes, although separate, are designed to be used together and should satisfy the principles specified in *BS EN 1990: Eurocode 0: Basis of Structural Design*, abbreviated to EC0 in the left margin. This guide begins with a brief summary of the requirements of Eurocode 0. It should be understood that this is not intended to be an explanation of its philosophy, or of the risk analysis principles on which much of it is based. Although this is valuable material for those who wish to go deeply into the principles as well as the practice of the Eurocodes, the idea here is simply to make it possible to allow students and engineers to familiarize themselves with the usual load assessment process by practising it in the simple and practical design of structural elements. A key objective has been to keep the document as slim and manageable as possible in order to minimize the mass of very rarely used material which is included.

Source reference

Terms and definitions

General

EC0 1.5.1.1 **construction works**
everything that is constructed or results from construction operations

NOTE This definition is in accordance with ISO 6707-1. The term covers both building and civil engineering works. It refers to the complete construction works comprising structural, non-structural and geotechnical elements.

EC0 1.5.1.2 **type of building or civil engineering works**
designation of the intended purpose of construction works (e.g. dwelling house, retaining wall, industrial building, road bridge)

EC0 1.5.1.3 **type of construction**
indication of the principal structural material (e.g. reinforced concrete construction, steel construction, timber construction, masonry construction, steel and concrete composite construction)

EC0 1.5.1.4 **method of construction**
manner in which the execution will be carried out (e.g. cast in place, prefabricated, cantilevered)

EC0 1.5.1.5 **construction material**
material used in construction work (e.g. concrete, steel, timber, masonry)

EC0 1.5.1.6 **structure**
combination of connected parts designed to carry loads and provide adequate rigidity

EC0 1.5.1.7 **structural member**
physically distinguishable part of a structure (e.g. a column, a beam, a slab, a foundation pile)

EC0 1.5.1.8 **form of structure**
arrangement of structural members (e.g. frames, suspension bridges)

EC0 1.5.1.9 **structural system**
load-bearing members of a building or civil engineering works and the way in which these members function together

EC0 1.5.1.10 **structural model**
idealization of the structural system used for the purpose of analysis, design and checking

Source reference	
EC0 1.5.1.11	**execution** all activities carried out for the physical completion of the work, including procurement, inspection and documentation **NOTE** The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.
EC0 1.5.2.2	**design situation** representation of the real conditions occurring during the time period for which the design will conform to the conditions of relevant limit states
EC0 1.5.2.3	**transient design situation** design situation which is relevant during a period much shorter than the design working life of the structure, and which has a high probability of occurrence **NOTE** A transient design situation refers to a temporary condition of the structure, of use, or of exposure (e.g. during construction or repair).
EC0 1.5.2.4	**persistent design situation** design situation which is relevant during a period of the same order as the design working life of the structure **NOTE** This generally refers to conditions of normal use.
EC0 1.5.2.5	**accidental design situation** design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure
EC0 1.5.2.6	**fire design** design of a structure to fulfil the performance requirements for the case of fire
EC0 1.5.2.7	**seismic design situation** design situation involving exceptional conditions of the structure when subjected to a seismic event (earthquake)
EC0 1.5.2.8	**design working life** assumed period for which a structure, or part of it, is to be used for its intended purpose, with anticipated maintenance but without major repair being necessary
EC0 1.5.2.9	**hazard** for the purposes of EN 1990 to EN 1999, this is defined as an unusual and severe event (e.g. an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation from intended dimensions)
EC0 1.5.2.10	**load arrangement** identification of the position, magnitude and direction of a free action

Source reference	
EC0 1.5.2.11	**load case** compatible load arrangements, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for a particular design check
EC0 1.5.2.12	**limit states** states beyond which the structure no longer fulfils its relevant design criteria
EC0 1.5.2.13	**ultimate limit states** states associated with collapse, or with other similar forms of structural failure **NOTE** These generally correspond to the maximum load-carrying resistance of a structure or structural member.
EC0 1.5.2.14	**serviceability limit states** states which correspond to conditions beyond which the specified service requirements for a structure or structural member are no longer met
EC0 1.5.2.14.1	**irreversible serviceability limit states** serviceability limit states for which some consequences of actions exceeding the specified service requirements will remain when the actions are removed
EC0 1.5.2.14.2	**reversible serviceability limit states** serviceability limit states for which no consequences of actions exceeding the specified service requirements will remain when the actions are removed
EC0 1.5.2.15	**resistance** capacity of a member, or component, or a cross-section of a member, or component of a structure, to withstand actions without mechanical failure (e.g. bending resistance, buckling resistance, tension resistance)
EC0 1.5.2.17	**reliability** ability of a structure or structural member to fulfil the specified requirements, including its design working life, for which it has been designed; reliability is usually expressed in probabilistic terms **NOTE** Reliability covers safety, serviceability and durability of a structure.
EC0 1.5.2.20	**maintenance** activities performed during the working life of the structure in order to enable it to fulfil its requirements for reliability **NOTE** Activities to restore the structure after an accidental or seismic event are normally considered to be outside the scope of maintenance.

Source reference	
EC0 1.5.2.21	**repair** activities that fall outside the definition of maintenance, performed to preserve or to restore the function of a structure
EC0 1.5.2.22	**nominal value** value fixed on a non-statistical basis, for instance on the basis of acquired experience or physical conditions
EC0 1.5.3.1	**action *(F)*** a) **direct action:** a set of forces (loads) applied to the structure b) **indirect action:** a set of imposed deformations or accelerations caused, for example, by temperature changes, moisture variation, uneven settlement or earthquakes
EC0 1.5.3.2	**effect *(E)* of an action** effect of actions on structural members (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation)
EC0 1.5.3.3	**permanent action *(G)*** action which is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limiting value
EC0 1.5.3.4	**variable action *(Q)*** action whose variation in magnitude with time is neither negligible nor monotonic
EC0 1.5.3.5	**accidental action *(A)*** action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during its design working life **NOTE 1** An accidental action can be expected, in many cases, to cause severe consequences unless appropriate measures are taken. **NOTE 2** Impact, snow, wind and seismic actions may be either variable or accidental actions, depending on the available information on statistical distributions.
EC0 1.5.3.6	**seismic action (A_E)** action that arises due to earthquake ground motions
EC0 1.5.3.7	**geotechnical action** action transmitted to the structure by the ground, fill or groundwater
EC0 1.5.3.8	**fixed action** action which has a fixed distribution and position across the structure or structural member, so that its magnitude and direction can be determined unambiguously for the whole structure or structural member if they are defined at one point on the structure or structural member

Source reference	
EC0 1.5.3.9	**free action** action which may have different spatial distributions over the structure
EC0 1.5.3.11	**static action** action which does not cause any significant acceleration of the structure or structural members
EC0 1.5.3.12	**dynamic action** action which causes significant acceleration of the structure or structural members
EC0 1.5.3.13	**quasi-static action** dynamic action which can be represented by an equivalent static action in a static model
EC0 1.5.3.14	**characteristic value (F_k) of an action** principal representative value of an action **NOTE** Insofar as a characteristic value can be fixed on a statistical basis, it is chosen so as to have a prescribed probability of not being exceeded on the unfavourable side during a 'reference period', taking into account the design working life of the structure and the duration of the design situation.
EC0 1.5.3.15	**reference period** period of time that is chosen as a basis for assessing statistically variable actions, and sometimes for accidental actions
EC0 1.5.3.16	**combination value of a variable action ($\psi_0 Q_k$)** value chosen – if it can be determined on a statistical basis – so that the probability of the effects caused by the combination being exceeded is approximately the same as that by the characteristic value of the individual action. It may be expressed as a particular part of the characteristic value by setting the factor $\psi_0 \leq 1$
EC0 1.5.3.17	**frequent value of a variable action ($\psi_1 Q_k$)** value determined – if it can be determined on a statistical basis – either so that the total time within the reference period during which it is exceeded is only a small given part of the reference period, or so that the frequency of its being exceeded is limited to a given value. It may be expressed as a particular part of the characteristic value by setting the factor $\psi_1 \leq 1$
EC0 1.5.3.18	**quasi-permanent value of a variable action ($\psi_2 Q_k$)** value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. It may be expressed as a particular part of the characteristic value by setting the factor $\psi_2 \leq 1$
EC0 1.5.3.19	**accompanying value of a variable action ($\psi\ Qk$)** value of a variable action that accompanies the leading action in a combination

Source reference	
	NOTE The accompanying value of a variable action may be the combination value, the frequent value or the quasi-permanent value.
EC0 1.5.3.20	**representative value of an action *(F_{rep})*** value used for checking a limit state. A representative value may be the characteristic value (F_k) or an accompanying value (ψF_k)
EC0 1.5.3.22	**combination of actions** set of design values used for checking a limit state under the simultaneous influence of different actions

Gravity loading: specific terms and definitions

EC1-1-1 1.4.2	**angle of repose** angle which the natural slope of any side of a heaped pile of loose material makes to the horizontal
EC1-1-1 1.4.3	**gross weight of a vehicle** this includes the self-weight of the vehicle, together with the maximum weight of the goods it is permitted to carry
EC1-1-1 1.4.4	**structural elements** those which comprise the primary structural frame and supporting structures. For bridges, structural elements comprise girders, structural slabs and elements providing support such as cable stays
EC1-1-1 1.4.5	**non-structural elements** those which include completion and finishing elements connected with the structure, including road surfacing and non-structural parapets. They also include services and machinery fixed permanently to, or within, the structure
EC1-1-1 1.4.6	**partitions** non-load-bearing walls
EC1-1-1 1.4.7	**movable partitions** those which can be moved on the floor, be added to, removed or rebuilt in another place

Snow loading: specific terms and definitions

EC1-1-3 1.6.1	**characteristic value of snow load on the ground** Snow load on the ground based on a probability of being exceeded once in 50 years, excluding exceptional snow loads
EC1-1-3 1.6.2	**altitude of the site** height above mean sea level of the site where the structure is to be located, or is already located for an existing structure

Source reference	
EC1-1-3 1.6.4	**characteristic value of snow load on a roof** product of the characteristic snow load on the ground and the appropriate coefficients **NOTE** These coefficients are chosen so that the probability of the calculated snow load on the roof does not exceed the probability of the characteristic value of the snow load on the ground.
EC1-1-3 1.6.5	**undrifted snow load on a roof** load arrangement which describes the uniformly distributed snow load on the roof, affected only by the shape of the roof, before any redistribution of snow due to other climatic actions
EC1-1-3 1.6.6	**drifted snow load on a roof** load arrangement which describes the snow load distribution resulting from snow having been moved (e.g. by the action of the wind) from one location to another location on a roof
EC1-1-3 1.6.7	**roof snow load shape coefficient** ratio of the snow load on the roof to the undrifted snow load on the ground, without the influence of exposure and thermal effects
EC1-1-3 1.6.8	**thermal coefficient** coefficient defining the reduction of snow load on roofs as a function of the conduction of heat flux through the roof, causing snow melting
EC1-1-3 1.6.9	**exposure coefficient** coefficient defining the reduction or increase of load on a roof of an unheated building, as a fraction of the characteristic snow load on the ground
	exceptional snow load on the ground load caused by the snow layer on the ground, resulting from a snowfall which has an exceptionally infrequent likelihood of occurring
EC1-1-3 1.6.10	**load due to exceptional snow drift** load arrangement which describes the load caused by the snow layer on the roof, resulting from a snow deposition pattern which has an exceptionally infrequent likelihood of occurring

Notation

General

NOTE For ease of comprehension, the symbols in this guide correspond to how they are presented in BS EN 1990 and BS EN 1991.

A	accidental action
A_d	design value of an accidental action
E	effect of actions
E_d	design value of effect of actions
F	action
F_d	design value of an action
F_k	characteristic value of an action j
F_{rep}	representative value of an action
G	permanent action
G_k	characteristic value of a permanent action
$G_{k,j}$	characteristic value of permanent action j
$G_{kj,sup}$, $G_{kj,inf}$	upper/lower characteristic value of permanent action
P	relevant representative value of a pre-stressing action (see BS EN 1992 to BS EN 1996 and BS EN 1998 to BS EN 1999)
Q	variable action
Q_k	characteristic value of a single variable action
$Q_{k,1}$	characteristic value of the leading variable action 1
$Q_{k,i}$	characteristic value of the accompanying variable action i
γ	partial factor (safety or serviceability)
γ_f	partial factor for actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values
γ_F	partial factor for actions, also accounting for model uncertainties and dimensional variations
γ_g	partial factor for permanent actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values
γ_G	partial factor for permanent actions, also accounting for model uncertainties and dimensional variations
$\gamma_{G,j}$	partial factor for permanent action j
$\gamma_{Gj,sup}$ / $\gamma_{Gj,inf}$	partial factor for permanent action j in calculating upper/lower design values
γ_P	partial factor for pre-stressing actions (see BS EN 1992 to BS EN 1996 and BS EN 1998 to BS EN 1999)
γ_q	partial factor for variable actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values

γ_Q	partial factor for variable actions, also accounting for model uncertainties and dimensional variations
$\gamma_{Q,I}$	partial factor for variable action i
ξ	reduction factor
ψ_0	factor for combination value of a variable action
ψ_1	factor for frequent value of a variable action
ψ_2	factor for quasi-permanent value of a variable action

Gravity loading: specific notation

γ	bulk weight density
φ	dynamic magnification factor
ϕ	angle of repose (degrees)
α_A	reduction factor
α_n	reduction factor
A	loaded area
A_0	basic area
g_k	weight per unit area, or weight per unit length
N	number of storeys
q_k	characteristic value of a uniformly distributed load, or line load

Snow loading: specific notation

α	pitch of roof, measured from horizontal [°o]
γ	weight density of snow [kN/m^3]
A	site altitude above sea level [m]
B	width of construction work [m]
C_e	exposure coefficient
C_t	thermal coefficient
D	depth of the snow layer [m]
F_s	force per metre length exerted by a sliding mass of snow [kN/m]
H	height of construction work [m]
K	coefficient to take account of the irregular shape of snow
l_s	length of snow drift or snow loaded area [m]
S	snow load on the roof [kN/m^2]
S_e	snow load per metre length due to overhang [kN/m]
s_k	characteristic value of snow on the ground at the relevant site [kN/m^2]
μ_i	snow load shape coefficients (various subscripts i)

Wind loading: specific notation

A	area
A_{fr}	area swept by the wind
A_{ref}	reference area
B	width of the structure (the length of the surface perpendicular to the wind direction if not otherwise specified)
c_{alt}	altitude factor
c_d	dynamic factor
c_{dir}	directional factor
$c_e(z)$	exposure factor
c_f	force coefficient
c_{fr}	friction coefficient
c_o	orography factor
c_p	pressure coefficient
$c_{pe,1}$, $c_{pe,10}$	external pressure coefficients
c_{prob}	probability factor
c_s	size factor
c_{season}	seasonal factor
D	depth of the structure (the length of the surface parallel to the wind direction if not otherwise specified)
E	eccentricity of a force or edge distance
F_{fr}	resultant friction force
F_w	resultant wind force
H	height of a topographic feature
H	height of the structure
h_{ave}	obstruction height
h_{dis}	displacement height
L_d	actual length of a downwind slope
L_e	effective length of an upwind slope
L_u	actual length of an upwind slope
P	annual probability of exceedence
q_b	reference mean (basic) velocity pressure
q_p	peak velocity pressure
v_b	basic wind velocity
$v_{b,0}$	fundamental value of the basic wind velocity
W	wind pressure
X	horizontal distance of the site from the top of a crest
Z	height above ground
z_{ave}	average height
z_e, z_i	reference height for external wind action, internal pressure
z_{max}	maximum height
z_{min}	minimum height
z_s	reference height for determining the structural factor
δ_s	structural logarithmic decrement of damping
μ	opening ratio, permeability of a skin

Indices

Crit	critical
E	external; exposure
Fr	friction
I	internal
P	peak; parapet
Ref	reference
V	wind velocity
X	along-wind direction
Y	cross-wind direction
Z	vertical direction

Creating load cases

according to the principles of

BS EN 1990: Eurocode 0 — Basis of structural design

Source reference

BS EN 1990: Eurocode 0 – Basis of structural design

This is not a précis of Eurocode 0. For some engineers it will become important to understand its particular philosophy of limit-state design, and the statistical principles which underpin it. However, the sole objective of this brief section is to provide just enough Eurocode 0 information to enable realistic load cases to be created using the loading information contained in the sections on gravity, snow and wind loading on buildings.

EN 1990 (BS EN 1990 in the UK) is the head code for the Structural Eurocodes. It is used alongside Eurocodes 1 to 9 (BS EN 1991 to BS EN 1999) for the structural design of buildings and civil engineering works. It provides the principles and requirements for safety and serviceability, and gives general principles for the structural design and checking of buildings and civil engineering structures. It also provides guidelines for the related aspects of structural reliability, durability and quality control.

In previous British Standards, the rules for satisfying safety, serviceability and durability criteria are identified in the individual design codes, which are largely based on the specific materials used in particular forms of construction. BS EN 1990, however, provides material-independent guidelines, and therefore presents a clearer general basis for meeting particular design criteria.

Requirements

The fundamental requirements concern safety, serviceability, fire exposure and robustness of the structure. They require that a structure is designed and built such that, during its intended life:

EC0 2.1(1)P — It should sustain all actions and influences likely to occur during its construction and use, with an appropriate degree of reliability, and remain fit for the use for which it is required, while being constructed in an economical way.

EC0 2.1(4) — It will not be damaged by events such as explosion, impact or the consequences of human error, to an extent which is disproportionate to the original cause.

EC0 2.1(5)P — The code also defines ways by which potential damage can be avoided or limited. These include:

- avoiding, eliminating or reducing the hazards to which the structure may be subjected;

Source reference

- selecting a structural form which has low sensitivity to the hazards identified;
- designing the structure to survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localized damage;
- avoiding as far as possible structural systems that can collapse without warning;
- tying the structural members together.

EC0 2.1(6)

In addition, the basic requirements are met by:

- choosing suitable materials;
- appropriate design and detailing;
- specifying control procedures for design, production, construction and use that are relevant to the particular project.

EC0 2.2 (1)P

The design of the structure should be in accordance with the structural Eurocodes BS EN 1990 to BS EN 1999, and by using appropriate controls on execution and quality management.

These requirements should be satisfied for the entire design working life of the building.

EC0 Table NA.2.1

Table BSD. 1: Indicative design working lives for various structure types

Design working life category	Indicative design working life (years)	Examples
[a]1	10	Temporary structures
2	10 to 30	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 25	Agricultural and similar structures
4	50	Building structures and other common structures, not limited elsewhere in this table
5	120	Monumental building structures, highway and railway bridges, and other civil engineering structures

[a] Structures or parts of structures that can be dismantled with a view to being reused should not be considered as temporary

EC0 2.4(1)P

The durability of the structure should be maintained throughout its design working life.

Source reference

Design situations

Design according to the structural Eurocodes follows 'Limit State' principles. The two principal limit states for which the structure needs to be designed are the **ultimate limit state** (ULS) and the **serviceability limit state** (SLS).

Ultimate limit state (ULS)

EC0 3.3(1)P

ULS concerns:

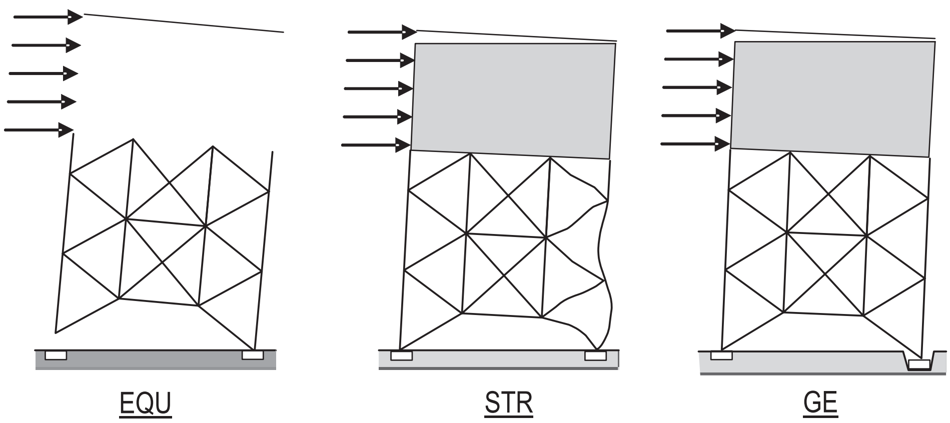
- safety of people; and
- safety of structures.

EC0 6.4.1(1)P

The conditions which should be checked under ULS are each given an acronym, as follows:

- **EQU:** **Loss of static equilibrium of the structure** (or any part of it) when considered as a rigid body. This can be thought of as 'overturning', as illustrated in Figure BSD. 1.
- **STR:** **Internal failure or excessive deformation of the structure** or its structural members, including footings, piles, basement walls, etc. This generally concerns the strength of the materials of a structure, or the stability of its members.
- **GEO:** **Failure or excessive deformation of the ground** or foundations on which the structure sits

Figure BSD. 1: The EQU, STR and GEO conditions



Source reference

The other condition covered in general terms (although it will not be covered at all in this guide) is:

- **FAT:** **Fatigue failure** of the structure or its structural members

EC0 3.2(2)P The design situations under which ULS design checks may be performed are:

- **persistent** design situations, which concern the conditions of normal use;
- **transient** design situations, which concern temporary conditions applicable to the structure (e.g. during construction or repair);
- **accidental** design situations, which concern exceptional conditions applicable to the structure or to its exposure (e.g. fire, explosion, impact or the consequences of localized failure);
- **seismic** design situations, which concern conditions applicable to the structure when subjected to earthquake and other seismic events.

Serviceability limit state (SLS)

EC0 3.4(1)P SLS concerns:

- whether deformation of the structure and its structural members allows the building to function properly in its normal intended use;
- the comfort of people;
- the appearance of the finished building.

EC0 3.4(3) Checking of SLS should be based on criteria concerning:

- **deformations** (deflections) which affect the appearance of the building, the comfort of users, or the way the structure (including machines or services within it) functions;
- **deformations** which cause damage to finishes (e.g. cracking of plaster) or non-structural members;
- **vibrations** which cause discomfort to people, or which limit the functional effectiveness of the building.

Actions

The term **action** is used in the Eurocodes in order to group together generically all external influences on a structure's performance. It encompasses loading by gravity and wind, but includes also vibration, thermal effects, fire and seismic loading.

Separate combinations of actions are used to check the structure for the design situation being considered,. For each of the particular design situations an appropriate representative value for each action is used.

Source reference

Representative values of actions

The main actions to be used in load cases used for design are:

- **permanent actions *G*:** e.g. self-weight of structures and fixed equipment;
- **variable actions *Q*:** e.g. imposed loads on building floors and beams; snow loads on roofs; wind loading on walls and roofs.
- **accidental actions *A*:** e.g. fire, explosions and impact.

Characteristic values G_k and Q_k

EC0 4.1.2(7) The **characteristic** value of an action can be considered as either:

- a maximum value which should never be exceeded, or a minimum value which should definitely be achieved during some specified reference period; or
- a nominal value, which may be specified in cases where the statistical distribution is not known.

Permanent actions

The characteristic value of a permanent action (G_k) may be a single value if its variability is known to be low (e.g. the self-weight of quality-controlled factory-produced members). If the variability of G cannot be considered as small, and its magnitude may vary from place to place in the structure, then an upper value $G_{k,sup}$ and a lower value $G_{k,inf}$ may occasionally be used.

Variable actions

Up to four types of representative value may be needed for the variable and accidental actions. The types most commonly used for variable actions are:

- the **characteristic value Q_k**

and combinations of the characteristic value with other variable actions, multiplied by different combination factors:

- the **combination value $\psi_0 Q_k$**
- the **frequent value $\psi_1 Q_k$**
- the **quasi-permanent value $\psi_2 Q_k$**

Explanations of the representative values and the design situations in which they arise are given below. The 'ψ_x' factors generally reduce the value of a variable action present in an accidental situation compared with the characteristic value.

Source reference

Combination value $\psi_0 Q_k$

EC0 4.1.3(1)(a) The combination value is used for checking:

1. Ultimate limit states;
2. *Irreversible* serviceability limit states (e.g. deflections which fracture brittle fittings or finishes).

It is associated with combinations of actions. The combination factor ψ_0 reduces Q_k because of the low probability of the most unfavourable values of several independent actions occurring simultaneously.

Frequent value $\psi_1 Q_k$

EC0 4.1.3(1)(b) The frequent value is used for checking:

1. Ultimate limit states involving accidental actions;
2. *Reversible* serviceability limit states, primarily associated with frequent combinations.

In both cases the reduction factor ψ_1 multiplies the *leading* variable action. The frequent value $\psi_1 Q_k$ of a variable action Q is determined so that the total proportion of a chosen period of time during which Q exceeds $\psi_1 Q_k$ is less than a specified small part of the period.

Quasi-permanent value $\psi_2 Q_k$

EC0 4.1.3(1)(c) The quasi-permanent value is used for checking:

1. Ultimate limit states involving accidental actions;
2. *Reversible* serviceability limit states.

Quasi-permanent values are also used for the calculation of long-term effects (e.g. cosmetic cracking of a slab) and to represent combinations of variable seismic actions. The quasi-permanent value $\psi_2 Q_k$ is defined so that the total proportion of a chosen period of time during which Q exceeds $\psi_2 Q_k$ is a considerable part (more than half) of the chosen period.

Load combinations for design

EC0 6.4.3.1(1)P Design values (E_d) of the effects of actions are determined by combining the values of actions (e.g. self-weight and imposed occupancy loads) which are considered by the designer to be capable of occurring simultaneously.

Source reference

The simplest, and most common, design case for ULS of an isolated structural member, such as a beam or slab, is a factored combination of the primary permanent and variable loadings:

'Dead + Imposed' $\gamma_G G_k + \gamma_{Q,1} Q_{k,1}$ (BSD.1)

in which

γ_G and $\gamma_{Q,1}$ are the partial safety factors for the permanent and principal variable actions G_k and Q_k respectively

If wind force is combined with dead and imposed loadings, then the effect of wind acting simultaneously with these gravity-induced loads is reduced because of the relatively low probability of extreme values of all the elements of the combination.

'Dead + Imposed + Wind' $\gamma_G G_k + \gamma_{Q,1} Q_{k,1} + \gamma_{Q,2} \psi_{0,2} Q_{k,2}$ (BSD. 2)

The 'combination factor' $\psi_{0,2}$ is an example from a range of such factors specified in Eurocode 0 (see Tables BSD. 2 and BSD. 3). In Equations BSD.1 and BSD.2 the imposed load $Q_{k,1}$ is the **leading variable action**; wind loading is classified as an **accompanying action**, and thus acquires a combination factor.

For pre-stressed members the variability of pre-stressing forces is taken into account by specifying a separate partial safety factor for pre-stress force in cases where this is needed:

'Dead + Imposed + Pre-stress' $\gamma_G G_k + \gamma_P P + \gamma_{Q,1} Q_{k,1}$ (BSD. 3)

EC0 6.4.3.1(2) In creating design load combinations, each combination of actions should include:

- the permanent action; and
- a leading variable action

General combinations of simultaneous actions for STR and GEO limit states

In Eurocode 0 the various possibilities of design load combinations for persistent or transient design situations for the STR and GEO limit states are presented in the following very general form:

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} G_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} G_{k,i} \qquad \text{(BSD. 4)}$$

in which

"+" implies 'to be combined with'
Σ implies 'the combined effect of'

Source reference

EC0 6.4.3.1(4)P

Where the results are sensitive to variations of the magnitudes of permanent actions from place to place in the structure, the unfavourable and favourable parts of this action are considered individually.

EC0 6.4.3.2(3)

In cases where permanent loadings may cause locally favourable or unfavourable effects, the load case should be the less favourable of:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}\psi_{0,1}Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}G_{k,i} \qquad \text{(BSD. 5)}$$

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \qquad \text{(BSD. 6)}$$

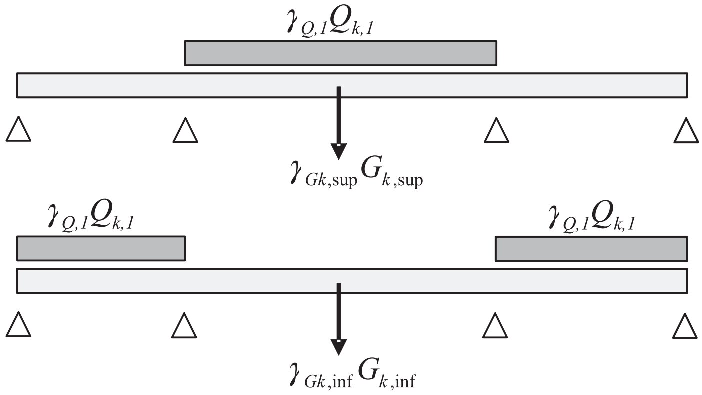
in which

ξ is a reduction factor for **unfavourable** permanent actions G

Consider the mid-span bending moment in the central span of the 3-span beam shown in Figure BSD.2, for the two possible patterns of loading shown.

IStructE Manual

Figure BSD. 2: Treatment of alternate spans



In the first continuous beam case above, permanent loads are unfavourable to the bending moment in the middle beam span. This situation can be reversed for hogging moment in the same location, as shown in the second case. Design of the beams in this structure may therefore consider both conditions, where permanent loads may be either favourable or unfavourable.

Thus, for the two models shown in Figure BSD. 2, the 'simplified' expressions for loads on the middle span might be:

For **sagging moment** in the middle beam

$$\gamma_{Gk,sup}G_k + \gamma_{Q,1}\psi_{0,1}Q_{k,1} \qquad \text{(BSD. 7)}$$

For **hogging moment** in the middle beam $\xi\gamma_{Gk,inf}G_k$ (BSD. 8)

Depending on the actual magnitudes of loads and the beam dimensions, there may or may not be a need for use of BSD. 8. Similar criteria can be used for the other design situations for both ULS and SLS.

Source reference

Partial safety factors for different design situations

As discussed above, each design situation requires selection of appropriate partial safety factors for its analysis. The subsequent tables present various partial safety factors for analysis at ULS. Refer to BS EN 1990 and the UK National Annex to BS EN 1990 for values for other design situations.

EC0 Table A1.2(A)

Table BSD. 2: Design values of actions (EQU) (Set A)

Note that this table is used for overall equilibrium (EQU) of structures rather than structural capacity of members or parts of the structure.

Persistent and transient design situations	**Permanent actions**		**Leading variable action**	**Accompanying variable actions**	
	Unfavourable	Favourable		Main (if any)	Others
(Eqn BSD. 4)	1.10 $G_{kj,sup}$	0.90 $G_{kj,inf}$	1.5 $Q_{k,1}$ (0 if favourable)		1.5 $\psi_{0,i}\, Q_{k,i}$ (0 if favourable)

Source reference

EC0 Table A1.2(B)

Table BSD. 3: Design values of actions (STR/GEO) (Set B)

These values of partial safety factors are used for the main cases of strength (STR) of **structures and structural members**, such as beams and columns, including those which are affected by geotechnical effects, but not for the design of foundations themselves.

Persistent and transient design situations	**Permanent actions**		**Leading variable action**	**Accompanying variable actions**	
	Unfavour-able	Favour-able		Main (if any)	Others
(Eqn BSD. 4)	1.35 $G_{kj,sup}$	1.00 $G_{kj,inf}$	1.5 $Q_{k,1}$		1.5 $\psi_{0,i}$ $Q_{k,i}$

Persistent and transient design situations	**Permanent actions**		**Leading variable action**	**Accompanying variable actions**	
	Unfavour-able	Favour-able	Action	Main (if any)	Others
(Eqn BSD. 5)	1.35 $G_{kj,sup}$	1.00 $G_{kj,inf}$		1.5 $\psi_{0,1}$ $Q_{k,1}$	1.5 $\psi_{0,i}$ $Q_{k,i}$
(Eqn BSD. 6)	0.925 x 1.35 $G_{kj,sup}$	1.00 $G_{kj,inf}$	1.5 $Q_{k,1}$		1.5 $\psi_{0,i}$ $Q_{k,i}$

NOTE 1 Either Equation BSD. 4, or Equation BSD. 5, together with Equation BSD. 6, may be used, as appropriate.

NOTE 2 When using expression BSD. 6, $\gamma_{G,sup}$ is multiplied by the reduction factor ξ = 0.925, becoming 1.25.

NOTE 3 The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self-weight of the structure may be considered as coming from one source; this also applies if different materials are involved.

NOTE 4 When variable actions are favourable Q_k should be taken as 0.

Source reference

EC0 Table A1.2(C)

Table BSD. 4: Design values of actions (STR/GEO) (Set C)

Foundation members (footings, piles, basement walls, etc.) involving geotechnical actions and the resistance of the ground (GEO) should be analysed using values taken from this table, in conjunction with BS EN 1997.

Persistent and transient design situations	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eqn BSD. 4)	1.0 $G_{kj,sup}$	1.0 $G_{kj,inf}$	1.3 $Q_{k,1}$ (0 if favourable)		1.3 $\gamma_{0,I}\, Q_{k,I}$ (0 if favourable)

Combination factors for appropriate design situations

A summary of design situations and the respective representative values which can be used is given in Table BSD. 5. Values of Ψ factors for buildings are shown in Table BSD. 6.

Table BSD. 5: Application of Ψ_0, Ψ_1 and Ψ_2 coefficients for variable actions at ULS and SLS

Limit state	Design situation	Combination value Ψ_0	Frequent value Ψ_1	Quasi-permanent value Ψ_2
Ultimate (ULS)	Persistent	Non-leading	✘	✘
	Transient	Non-leading	✘	✘
	Accidental	✘	Leading	Leading and non-leading
	Seismic	✘	✘	All variable actions
Serviceability (SLS)	Characteristic	Non-leading	✘	✘
	Frequent	✘	Leading	Non-leading
	Quasi-permanent	✘	✘	All variable actions
✘ means not applied				

Source reference

EC0 Table NA.A1.1

Table BSD. 6: Values of combination factors Ψ for buildings

Action	Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings category (BS EN 1991-1.1)			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area, weight ≤ 30 kN	0.7	0.7	0.6
Category G: traffic area, 30kN < weight ≤ 160 kN	0.7	0.5	0.3
Category H: roofs[a]	0.7	0.0	0.0
Snow loads on buildings (BS EN 1991-1.3)			
• for sites located at altitude H > 1,000m above sea level	0.7	0.5	0.2
• for sites located at altitude H ≤ 1,000m above sea level	0.5	0.2	0.0
Wind loads on buildings (BS EN 1991-1.4)	0.5	0.2	0.0
Temperature (non-fire) in buildings (BS EN 1991-1.5)	0.6	0.5	0.0
[a] See also BS EN 1991-1.1, 3.3.2(1)			

BS EN 1991: Eurocode 1-1 – Gravity loading

Source reference

Gravity loading

Introduction

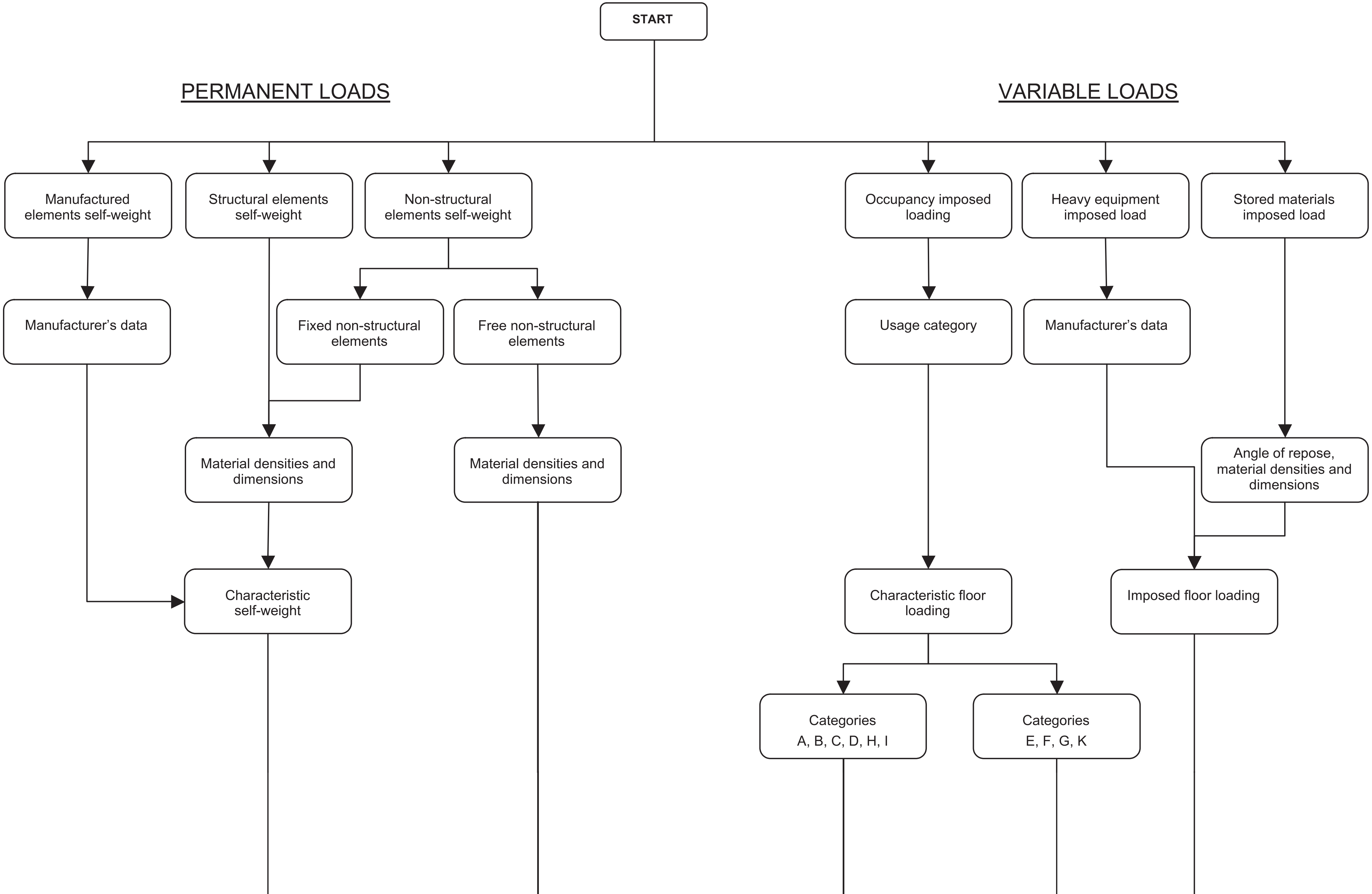
Gravity loads are classified as either permanent or variable actions. Loads which occur due to the self-weight of building components are **permanent actions**, or dead loads, while loads due to the type of occupancy, furnishings and fixings, storage, etc. are **variable actions**, or imposed loads.

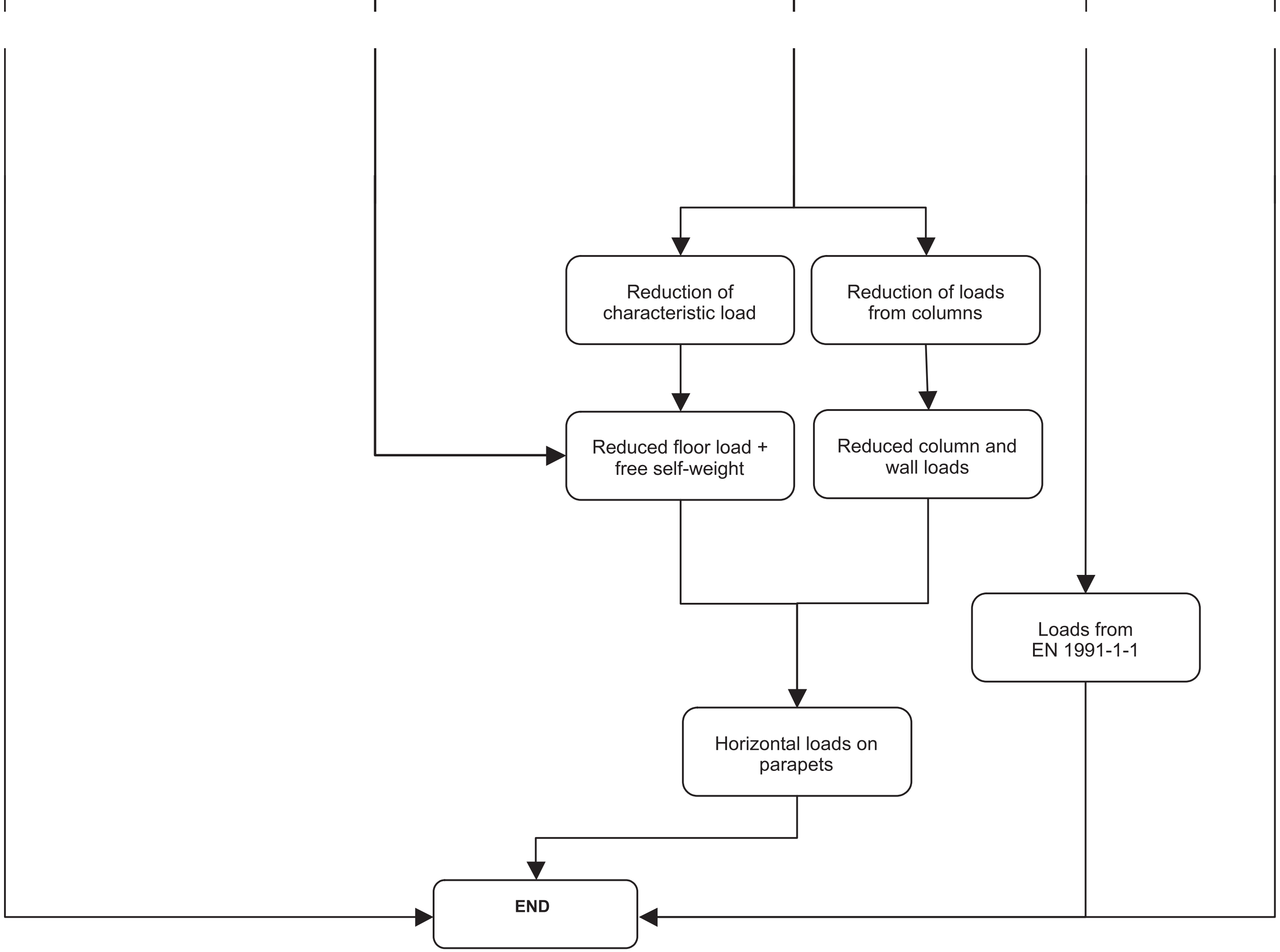
BS EN 1991-1-1 is the Eurocode that is used to determine the gravity loads to be used in building design; this section outlines the process of calculating the characteristic dead and imposed loads on structures to be built in the UK.

The section does not cover the rather specialized loads due to vehicles, fork-lift trucks and helicopters. Interested readers should consult BS EN 1991-1.1 for details of the imposed loads in respect of these cases.

A flowchart of the calculation process for characteristic self-weights and imposed loads is presented, followed by explanations of the various steps in calculating individual characteristic self-weight and imposed loads.

Flowchart of gravity loading calculation




Reduction of characteristic load
Reduction of loads from columns
Reduced floor load + free self-weight
Reduced column and wall loads
Loads from EN 1991-1-1
Horizontal loads on parapets
END

Source reference

Self-weight

EC1-1-1 5.1(1)

The components that make up the self-weight of a building are derived from its structural elements, the non-structural elements and the installed services in the building. Examples of each category are given in Table G. 1.

EC1-1-1 5.1(2 & 3)

Table G. 1: Typical elements of construction works

Structural elements	Non-structural elements	Fixed services
Slabs Columns Beams Walls Foundations	Roofing Surfacing and coverings Partitions and linings Hand rails, safety barriers, parapets and kerbs Wall cladding Suspended ceilings Thermal insulation Bridge furniture Fixed services	Equipment for lifts and moving stairways Heating, ventilating and air conditioning equipment Electrical equipment Empty pipes Cable trunking and conduits

Non-structural elements are further divided into fixed and free components. Free components are those loads which are free to be located at any location on a given floor at any given time. A typical example of a free non-structural element of construction is a movable partition.

EC1-1-1 5.2.2(2)P

Free non-structural elements are therefore treated as *variable* (or imposed) loads. They are usually represented by equivalent uniformly distributed floor loads. Values of equivalent floor loads for movable partitions are therefore shown in Table G. 4.

EC1-1-1 5.1(1)

Weights of structural and fixed non-structural elements are determined based on their dimensions and material densities.

EC1-1-1 5.2.1(2)

The nominal dimensions of the construction elements should be those shown on the drawings.

EC1-1-1 4.1(1)

Mean values of material densities of the various construction materials should be used as characteristic values.

EC0 4.1.2(1)P

The characteristic value F_k of an action can sometimes be specified in different ways (as a mean value, an upper or lower value, or a purely nominal value).

EC1-1-1 5.2.2(1)

For manufactured elements, such as plant for lifts and moving stairways, flooring systems, facades and ceilings, data from the manufacturer should be used to calculate self-weight.

Source reference	
EC1-1-1 2.1(4)P	Earth loads on roofs and terraces should be considered as permanent actions. The design also needs to consider variations in moisture content and variation in depth, that may be caused by uncontrolled accumulation during the design life of the structure.
EC1-1-1 3.2(5)	The source and moisture content of bulk materials should be considered in design situations of buildings used for storage purposes. It should be noted that the values of densities provided in Annex A of BS EN 1991-1-1 are for materials in the dry state.

Imposed loads

EC1-1-1 2.2

Imposed loads in buildings are classified as *variable free* actions and are considered as quasi-static for design purposes. They are usually modelled by uniformly distributed floor loads, line loads, concentrated loads or combinations of these.

They occur as a consequence of:

- loads due to occupancy;
- loads due to heavy equipment;
- loads due to stored materials.

EC1-1-1 6.1(4)

Loads that are due to heavy equipment, such as machinery, should be agreed between the client and/or the relevant authority. These loads are usually obtained from the manufacturer's data.

EC1-1-1 4.1(1)

Loads due to stored materials can be determined by obtaining their angles of repose (from Annex A of BS EN 1991-1-1), material densities and dimensions.

EC1-1-1 6.1(1)

Occupancy loading is what is generally termed the characteristic imposed load of a structure. It is made up of loads generated by:

- normal use by people;
- furniture and movable objects (e.g. movable partitions, storage, the contents of containers);
- vehicles;.
- anticipating occasional events, such as concentrations of people or furniture, or the moving or stacking of objects which may occur during reorganization or redecoration.

EC1-1-1 6.2.1(1)P

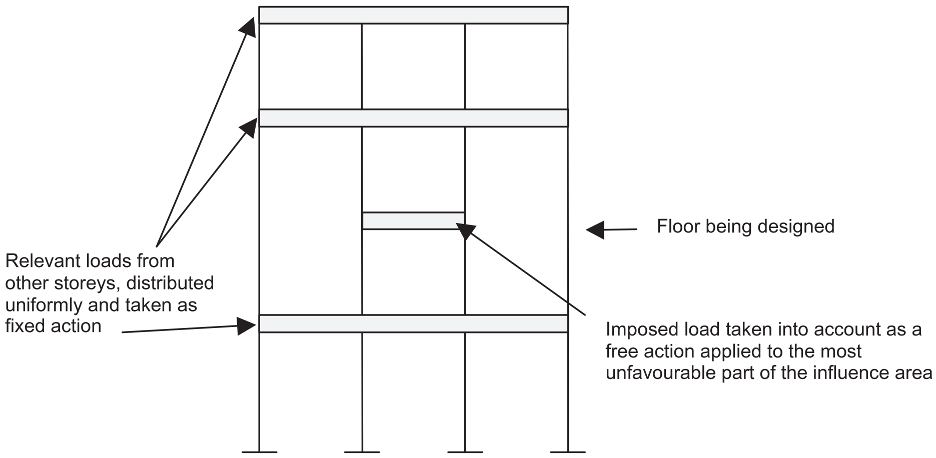
For the design of a floor structure within a single storey or a roof, the imposed load should be considered as a free action applied to the most unfavourable part of the area influenced by the action effects considered.

EC1-1-1 6.2.1(2)

Where the loads on other storeys are relevant, they may be assumed to be distributed uniformly and taken into account as fixed actions. See Figure G. 1.

Source reference

Figure G. 1: Load arrangement for floors, beams and roof



EC1-1-1 6.1(3)

Occupancy loads are subdivided into categories, according to their usage, as follows:

- A Areas for domestic and residential activities
- B Office areas
- C Areas where people may congregate (with the exception of areas defined under categories A, B, and D)
- D Shopping areas
- E Access areas, areas susceptible to accumulation of goods and areas for industrial use
- F Traffic and parking areas for light vehicles
- G Traffic and parking areas for medium vehicles
- H Roofs which are not accessible except for normal maintenance and repair
- I Roofs which are accessible with occupancy according to categories A to D
- K Roofs which are accessible for special services, such as helicopter-landing areas

As a guide to designing structures to the Eurocodes, this document only examines the occupancy loads in categories, A, B, C, D, E, H and I. For designs which require the other categories, refer to BS EN 1991-1-1 and the National Annex to BS EN 1991-1-1.

EN 1991 and its corresponding UK National Annex further divide the occupancy categories mentioned above into sub-categories, and provide characteristic imposed loads for floors, balconies and stairs. Table G. 2 shows the sub-categories that are used in the UK for categories A to D.

Source reference

EC1-1-1 Table NA.2

Table G. 2: Categories of residential, social, commercial and administration areas including additional sub-categories (UK)

Category of loaded area	Specific use	Sub-category	Example
A	Areas for domestic and residential activities	A1	All usages within self-contained dwelling units (a unit occupied by a single family or a modular student accommodation unit with a secure door and comprising not more than six single bedrooms and an internal corridor) Communal areas (including kitchens) in blocks of flats with limited use (see Note 1). For communal areas in other blocks of flats, see A5, A6 and C3
		A2	Bedrooms and dormitories except those in self-contained single family dwelling units, hotels and motels
		A3	Bedrooms in hotels and motels; hospital wards; toilet areas
		A4	Billiard/snooker rooms
		A5	Balconies in single family dwelling units and communal areas in blocks of flats with limited use (see Note 1)
		A6	Balconies in hostels, guest houses, residential clubs and communal areas in blocks of flats except those covered by Note 1
		A7	Balconies in hotels and motels
B	Office areas	B1	General use other than in B2
		B2	At or below ground floor level

Source reference

Category of loaded area	Specific use	Sub-category		Example
C	Areas where people may congregate (with the exception of areas defined under category A, B and D)	C1: Areas with tables	C11	Public, institutional and communal dining rooms and lounges, cafes and restaurants (see Note 2)
			C12	Reading rooms with no book storage
			C13	Classrooms
		C2: Areas with fixed seats	C21	Assembly areas with fixed seating (see Note 3)
			C22	Places of worship
		C3: Areas without obstacles for moving people	C31	Corridors, hallways, aisles in institutional-type buildings not subjected to crowds or wheeled vehicles, hostels, guest houses, residential clubs, and communal areas in blocks of flats not covered by Note 1
			C32	Stairs, landings in institutional-type buildings not subjected to crowds or wheeled vehicles, hostels, guest houses, residential clubs, and communal areas in blocks of flats not covered by Note 1
			C33	Corridors, hallways, aisles in all buildings not covered by C31 and C32, including hotels and motels and institutional buildings subjected to crowds
			C34	Corridors, hallways, aisles in all buildings not covered by C31 and C32, including hotels and motels and institutional buildings subjected to wheeled vehicles, including trolleys
			C35	Stairs, landings in all buildings not covered by C31 and C32, including hotels and motels and institutional buildings subjected to crowds

Category of loaded area	Specific use	Sub-category		Example
			C36	Walkways — Light duty (access suitable for one person, walkway width approx. 600 mm)
			C37	Walkways — General duty (regular two-way pedestrian traffic)
			C38	Walkways — Heavy duty (high-density pedestrian traffic including escape routes)
			C39	Museum floors and art galleries for exhibition purposes
		C4: Areas with possible physical activities	C41	Dance halls and studios, gymnasia, stages (see Note 5)
			C42	Drill halls and drill rooms (Note 5)
		C5: Areas susceptible to large crowds	C51	Assembly areas without fixed seating, concert halls, bars and places of worship (see Note 4 and Note 5)
			C52	Stages in public assembly areas (see Note 5)
D	Shopping areas	D1		Areas in general retail shops
		D2		Areas in department stores

NOTE 1 Communal areas in blocks of flats with limited use are blocks of flats not more than three storeys in height and with not more than four self-contained dwelling units per floor accessible from one staircase.

NOTE 2 Where the areas described by C11 might be subjected to loads due to physical activities or overcrowding, e.g. a hotel dining room used as a dance floor, imposed loads should be based on C4 or C5 as appropriate. Reference should also be made to Note 5.

NOTE 3 Fixed seating is seating where its removal and the use of the space for other purposes is improbable.

NOTE 4 For grandstands and stadia, reference should be made to the requirements of the appropriate certifying authority.

NOTE 5 For structures that might be susceptible to resonance effects, reference should be made to **NA.2.1 in NA to BS EN 1991-1-1**.

Source reference

The minimum characteristic imposed loads to be used for the floors, balconies and stairs in buildings in the various sub-categories listed in Table G. 2 are shown in Table G. 3. Uniform floor loads are shown under q_k for checking general effects, while concentrated loads are shown under Q_k for checking local effects, such as crushing and punching shear.

EC1-1-1 6.3.1.2(5)

Concentrated loads have to be considered to act at any point on the floor, balcony or stairs over an area with a shape which is appropriate to the use and form of the floor. The shape may normally be assumed to be a square with a width of 50 mm.

EC1-1-1 Table NA.3

Table G. 3: Imposed loads on floors, balconies and stairs in buildings

Category of loaded area			q_k (kN/m^2)	Q_k (kN)
Category A	A1		1.5	2.0
	A2		1.5	2.0
	A3		2.0	2.0
	A4		2.0	2.7
	A5		2.5	2.0
	A6		Same as the rooms to which they give access but with a minimum of 3.0	2.0 (concentrated at the outer edge)
	A7		Same as the rooms to which they give access but with a minimum of 4.0	2.0 (concentrated at the outer edge)
Category B	B1		2.5	2.7
	B2		3.0	2.7
Category C	C1	C11	2.0	3.0
		C12	2.5	4.0
		C13	3.0	3.0
	C2	C21	4.0	3.6
		C22	3.0	2.7
	C3	C31	3.0	4.5
		C32	3.0	4.0
		C33	4.0	4.5
		C34	5.0	4.5
		C35	4.0	4.0

Source reference

Category of loaded area			q_k (kN/m^2)	Q_k (kN)
		C36	3.0	2.0
		C37	5.0	3.6
		C38	7.5	4.5
		C39	4.0	4.5
	C4	C41	5.0	3.6
		C42	5.0	7.0
	C5	C51	5.0	3.6
		C52	7.5	4.5
Category D	D1/D2		4.0	3.6

EC1-1-1 6.3.1.2(2)

Where necessary q_k and Q_k should be increased in the design (e.g. for stairs and balconies, depending on the occupancy and dimensions).

EC1-1-1 6.3.1.2(7)P

Where floors are subjected to multiple use, they should be designed for the most unfavourable category of loading which produces the highest effects of actions (e.g. force or deflection) in the member under consideration.

Partitions

EC1-1-1 6.3.1.2(8)

Provided that a floor allows a lateral distribution of loads, the self-weight of movable partitions may be taken into account using a uniformly distributed load q_k which should be added to the imposed loads of floors given in Table G. 3. The value of q_k is dependent on the self-weight of the partitions as shown in Table G. 4.

EC1-1-1 6.3.1.2

Table G. 4: Equivalent floor loads for movable partitions

Movable partition classification	Equivalent floor load
With self-weight ≤ 1.0kN/m wall length	0.5 kN/m^2
With self-weight ≤ 2.0kN/m wall length*	0.8 kN/m^2
With self-weight ≤ 3.0kN/m wall length	1.2 kN/m^2
* Surveys indicate that most office partitioning falls into the second category of 0.8 kN/m^2.	

EC1-1-1 6.3.1.2(2)

For heavier partitions the design should take into account:

- the locations and directions of the partitions;
- the structural form of the floors.

Source reference

Areas for storage and industrial activities

Areas for storage and industrial activities are divided into two main categories:

E1 Areas susceptible to accumulation of goods, including access areas;
E2 Industrial uses, which are not covered in this guide.

In the UK National Application Document, the categories are further divided to aid easier classification for design purposes. The sub-categories of storage area are listed in Table G. 5.

EC1-1-1 Table NA.4

Table G. 5: Categories of storage areas for the UK

Category of loaded area	Specific use	Sub-category	Examples
E1	Areas susceptible to accumulation of goods, including access areas	E11	General areas for static equipment not specified elsewhere (institutional and public buildings)
		E12	Reading rooms with book storage, e.g. libraries
		E13	General storage other than those specified
		E14	File rooms, filing and storage space (offices)
		E15	Stack rooms (books)
		E16	Paper storage for printing plants and stationery stores
		E17	Dense mobile stacking (books) on mobile trolleys, in public and institutional buildings
		E18	Dense mobile stacking (books) on mobile trucks, in warehouses
		E19	Cold storage

Category E1 areas shown in Table G. 5 are designed using characteristic loads from Table G. 6.

Source reference

EC1-1-1 Table NA.5

Table G. 6: Imposed floor loads in storage areas (Category E1)

Sub-category of loaded area	q_k (kN/m^2)	Q_k (kN)
E11	2.0	1.8
E12	4.0	4.5
E13*	2.4 per metre of storage height	7.0
E14	5.0	4.5
E15	2.4 per metre of storage height but with a minimum of 6.5	7.0
E16	4.0 per metre of storage height	9.0
E17	4.8 per metre of storage height but with a minimum of 9.6	7.0
E18	4.8 per metre of storage height but with a minimum of 15.0	7.0
E19	5.0 per metre of storage height but with a minimum of 15.0	9.0

* **NOTE** E13 is a general category. However, designers are encouraged to liaise with clients to determine more specific load values than the lower bound value given in this table.

EC1-1-1 6.3.2.2(2)P

The characteristic value of the imposed load is the maximum value taking account of dynamic effects if appropriate. Loading conditions should be defined so that they produce the most unfavourable conditions allowed in use.

EC1-1-1 6.3.2.2(3)

The characteristic values of vertical loads in storage areas should be derived by taking into account the density and the upper design values for stacking heights. If these stored materials exert horizontal loads, the forces should be determined with reference to BS EN 1991-4.

NOTE Densities of storage materials should be obtained from Annex A of BS EN 1991-1-1.

EC1-1-1 6.3.2.2(4)

Any effects of filling and emptying should be taken into account.

EC1-1-1 6.3.2.2(5)

Loads for storage areas for books and other documents should be determined from the loaded area and the height of the bookcases, using the appropriate values for density.

Source reference

Roofs

The three categories of loaded areas adopted for roofs, categorized according to their accessibility, are as follows:

H Roofs not accessible except for normal maintenance and repair
I Roofs accessible with occupancy according to categories A to D
K Roofs accessible for special services, such as helicopter-landing areas

Imposed loads for roofs of Category H are given in Table G. 7. Imposed loads for roofs of Category I are the same as those for Categories A to D, and are given in Table G. 3. Loads for Category K, which provides areas for helicopter landing, are not discussed in this document.

EC1-1-1 Table NA.7

The minimum characteristic values q_k (uniformly distributed load) and Q_k (concentrated load) for Category H roofs are shown in Table G. 7. They are related to the horizontal projection of the area of the roof under consideration.

EC1-1-1 Table NA.7

Table G. 7: Imposed loads on roofs (Category H)

Roof slope α in degrees	q_k (kN/m^2)	Q_k (kN)
$\alpha < 30°$	0.6	0.9
$30° \leq \alpha < 60°$	$0.6\,[(60 - \alpha)/30]$	
$\alpha \geq 60°$	0	

NOTE 1 All roof slopes α [°] are measured from the horizontal and all loads should be applied vertically.

NOTE 2 q_k may be assumed to act on an area A. It is recommended that the value of A should be the whole area of the roof.

EC1-1-1 6.3.4.2(2)

The minimum characteristic values given in Table G. 7 do not take into account uncontrolled accumulations of construction materials that may occur during maintenance. Further guidance for this is given in BS EN 1991-1-6: *Actions during execution*.

EC1-1-1 6.3.4(3)P

As with floor loads, separate checks will need to be performed for concentrated loads Q_k and uniformly distributed loads q_k, acting independently.

EC1-1-1 6.3.4(4)

Roofs which do not have roof sheeting should be designed to resist a concentrated load Q_k of 1.5 kN on an area based on a 50 mm-sided square. Roof elements with a profiled or discontinuously laid surface should be designed so that the concentrated load acts over the effective area provided by a load-spreading arrangement.

Source reference

Reduction of loads

Floors, beams and roofs

EC1-1-1 6.2.1(4) Imposed loads in any single category may be reduced, according to the areas supported by the appropriate member, by a reduction factor α_A.

EC1-1-1 6.3.1.2(10) The reduction factor α_A may be applied to the q_k values for imposed loads in Table G.3 for floors, as well as for roofs of categories H (Table G. 7) and I. Loads which have been specifically determined on the basis of specific knowledge of the proposed use of a structure do not qualify for this reduction.

EC1-1-1 Eqn NA.1 The reduction factor α_A is calculated using Equation G. 4

$$\alpha_A = 1.0 - A/1{,}000 \geq 0.75 \qquad \text{(G. 4)}$$

Where A is the area supported in m^2.

Columns and walls

EC1-1-1 6.2.2(2) Where imposed loads from several storeys act on columns and walls, the total imposed loads may be reduced by a factor α_n.

EC1-1-1 6.3.1.2(11) Provided that the area is classified according to Table G. 3 into categories A to D, for columns and walls, the total imposed loads from several storeys may be multiplied by the reduction factor α_n. Loads which have been specifically determined on the basis of specific knowledge of the proposed use of a structure do not qualify for this reduction.

EC1-1-1 Eqn NA.2 If n is the number of storeys with loads qualifying for reduction the reduction factor α_n should be calculated from Equation G. 5:

$$\begin{aligned} \alpha_n &= 1.1 - n/10 && \text{for } 1 \leq n \leq 5 \\ \alpha_n &= 0.6 && \text{for } 5 < n \leq 10 \qquad \text{(G. 5)} \\ \alpha_n &= 0.5 && \text{for } n > 10 \end{aligned}$$

Load reductions based on floor area above may be applied if $\alpha_A < \alpha_n$.

NOTE The reductions given by Equation (G. 4) cannot be used in combination with those given by Equation (G. 5). Hence, column loads which are reduced by the factor α_n must be based on unreduced floor loads and vice versa.

Source reference

Horizontal loads on walls, balustrades, handrails and parapets

EC1-1-1 Table NA.8

The characteristic values of the line load q_k acting at the height of the partition wall or parapet, but not higher than 1.20 m, are given in Table G. 8.

Table G. 8: Horizontal loads on partition walls and parapets

Category of loaded area	**Sub-category**	**Examples**	q_k (kN/m)
A (including sub-categories in Table G. 3)	(i)	All areas within or serving exclusively one dwelling, including stairs, landings, etc. but excluding external balconies and edges of roofs [see (vii)]	0.36
	(ii)	Residential areas not covered by (i)	0.74
B and C1 (including sub-categories in Table G. 3)	(iii)	Areas not susceptible to overcrowding in office and institutional buildings, reading rooms and classrooms including stairs	0.74
	(iv)	Restaurants and cafes	1.5
C2, C3, C4 and D (including sub-categories in Table G. 3)[a]	(v)	Areas having fixed seating within 530 mm of the barrier, balustrade or parapet	1.5
	(vi)	Stairs, landings, balustrades, corridors and ramps	0.74
	(vii)	External balconies and edges of roofs Footways within building curtilage and adjacent to basement/sunken areas	0.74
	(viii)	All retail areas	1.5

Source reference

Category of loaded area	Sub-category	Examples	q_k (kN/m)
C5 (including sub-categories in Table G. 3)	(ix)	Footways or pavements less than 3 m wide adjacent to sunken areas Footways or pavements greater than 3 m wide adjacent to sunken areas	1.5
	(x)	Theatres, cinemas, discotheques, bars, auditoria, shopping malls, assembly areas, studios	3.0
	(xi)	Grandstands and stadia	See requirements of the appropriate certifying authority
E (including sub-categories in Table G. 5)	(xii)	Industrial; and storage buildings except as given by (xiii) and (xiv).	0.74
	(xiii)	Light pedestrian traffic routes in industrial and storage buildings except designated escape routes	0.36
	(xiv)	Light access stairs and gangways not more than 600 mm wide	0,22
F and G	(xv)	Pedestrian areas in car parks including stairs, landings, ramps, edges or internal floors, footways, edges of roofs	1.5
	(xvi)	Horizontal loads imposed by vehicles	See subclause 3.7

[a] For areas where large crowds might occur, see C5.

BS EN 1991-1-3: Eurocode 1 – Snow loading

Source reference

Snow loading

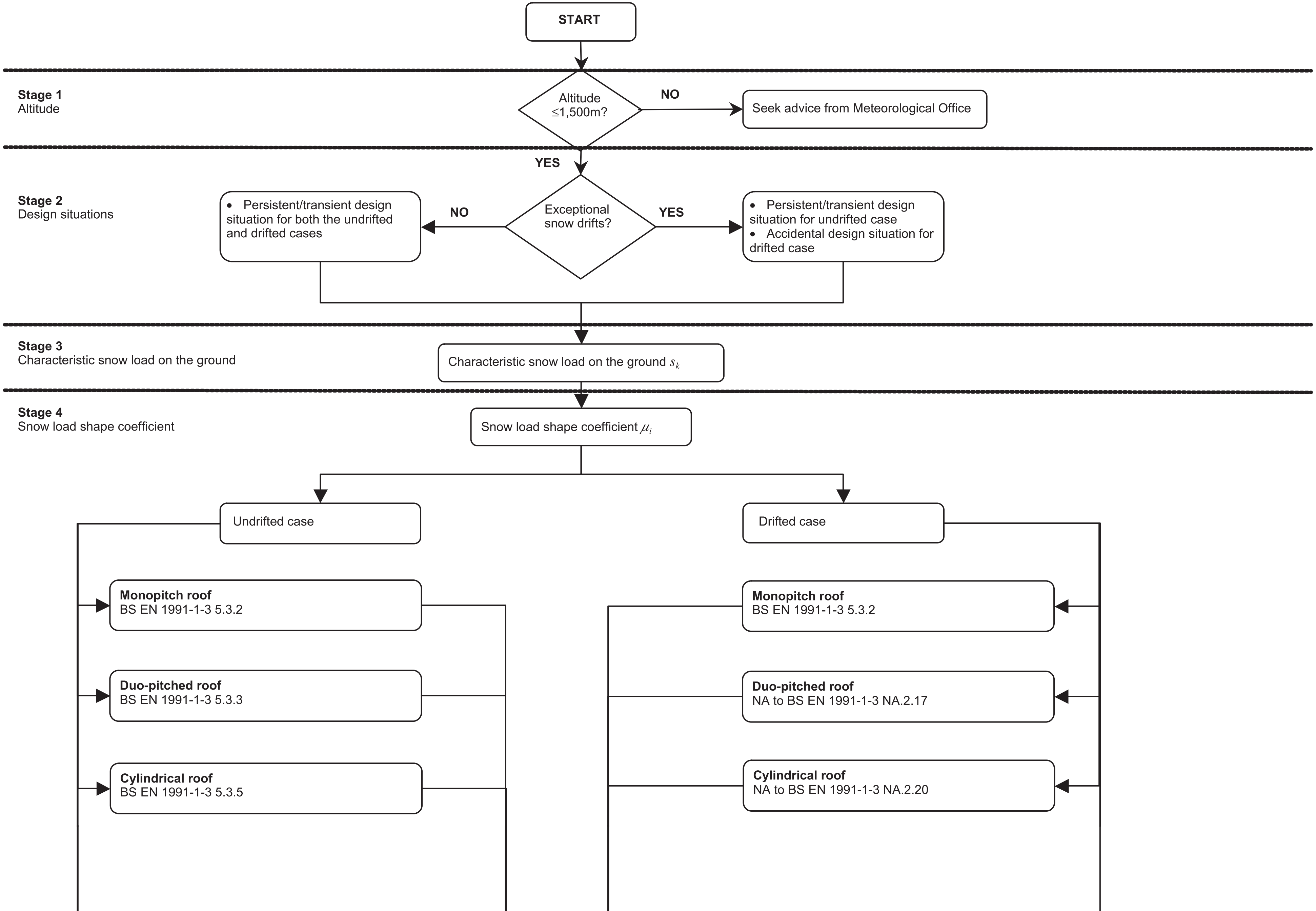
Snow loads on a roof depend on a number of factors. These include the altitude of the building site, the meteorological climate of the area, the geometry of the roof, its thermal properties and the proximity of neighbouring buildings. This section outlines the process of calculating the snow load on a given structure according to BS EN 1991-1-3, with particular reference to buildings in the UK.

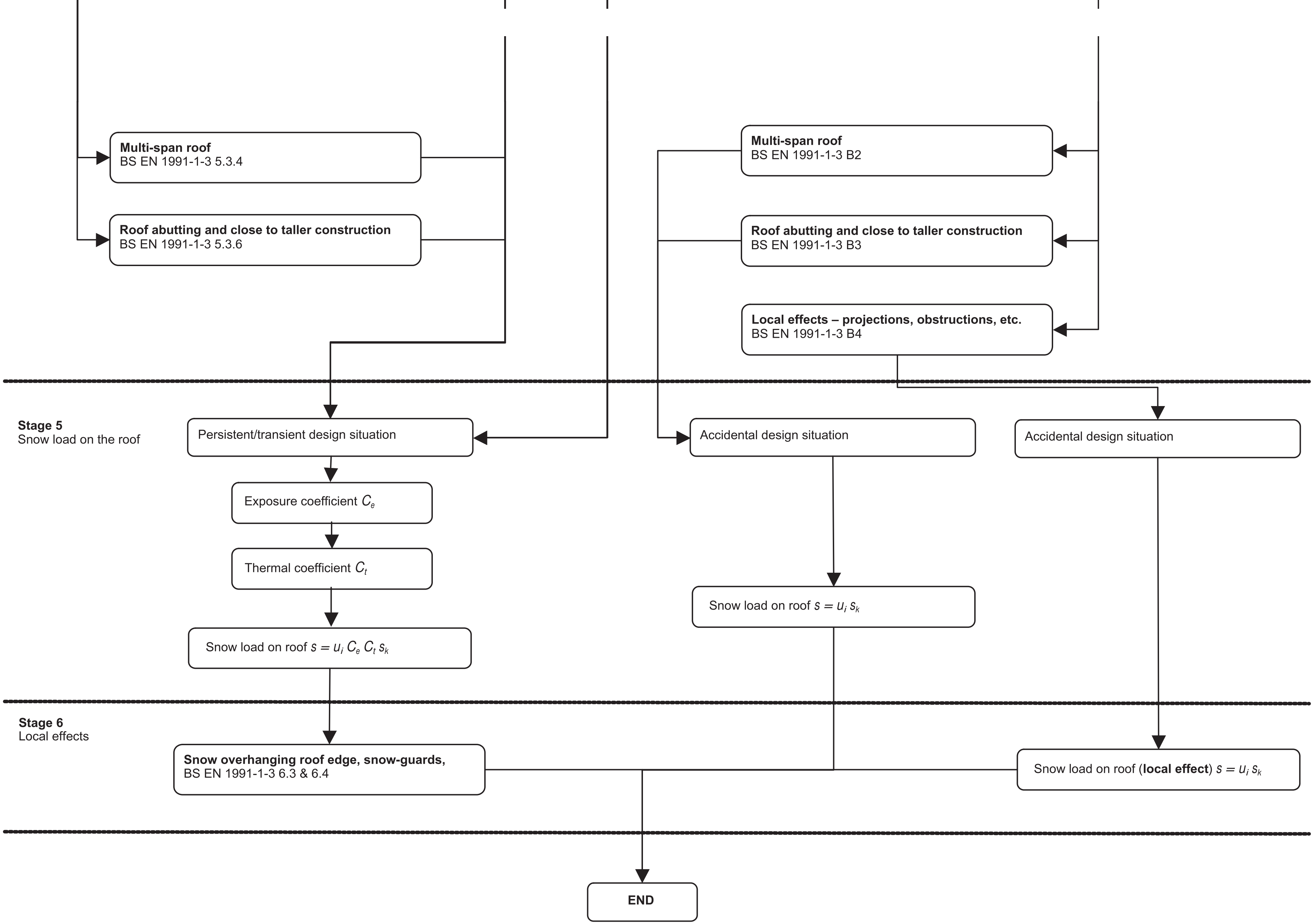
It does not cover specialist aspects of snow loading, such as:

- impact loads resulting from snow sliding off or falling from a higher roof;
- the additional wind loads that could result from changes in shape or size of the construction works as a result of the presence of snow or a build-up of ice;
- loads in areas where snow is present all year round;
- ice loading;
- lateral loading due to snow (e.g. lateral loads exerted by drifts).

A flowchart of the snow action calculation process is shown on the next page, followed by explanations of the various stages in the subsequent sub-sections.

Flowchart of snow action calculation




Multi-span roof
BS EN 1991-1-3 5.3.4
Roof abutting and close to taller construction
BS EN 1991-1-3 5.3.6
Multi-span roof
BS EN 1991-1-3 B2
Roof abutting and close to taller construction
BS EN 1991-1-3 B3
Local effects – projections, obstructions, etc.
BS EN 1991-1-3 B4
Stage 5
Snow load on the roof
Persistent/transient design situation
Accidental design situation
Accidental design situation
Exposure coefficient C_e
Thermal coefficient C_t
Snow load on roof $s = u_i\ s_k$
Snow load on roof $s = u_i\ C_e\ C_t\ s_k$
Stage 6
Local effects
Snow overhanging roof edge, snow-guards,
BS EN 1991-1-3 6.3 & 6.4
Snow load on roof (local effect) $s = u_i\ s_k$
END

Source reference	
	Stage 1: Altitude
EC1-1-3 1.1(1)	BS EN 1991-1-3 can only be used for new and existing building locations at a maximum altitude of 1,500 m above sea level.
EC1-1-3 NA.2.1	Seek advice from the Meteorological Office for sites above 1,500 m.
	Stage 2: Design situations
EC1-1-3 3	Two main conditions are assessed: 1. Normal conditions 2. Exceptional conditions
EC1-1-3 3.2	**Normal conditions** are those that do not include exceptional snow loads or snow drifts. The **transient** or **persistent** design situations are used for normal conditions.
EC1-1-3 5.2(3)	For **normal conditions**, the snow load on a roof is calculated as: $s = \mu_i C_e C_t s_k$ in which μ_i is the snow load shape coefficient C_e is the exposure coefficient C_t is the thermal coefficient s_k is the characteristic snow load on the ground
EC1-1-3 3.3	**Exceptional conditions** are those that involve exceptional snow loads or snow drifts. The **accidental** design situation is used for exceptional conditions.
EC1-1-3 NA.2.2	In the UK exceptional snow drifts can occur, but exceptional snow loads are unlikely to occur. Therefore, for situations where exceptional drifts have to be anticipated, the accidental design situation is used for calculation of the drifted snow load. See Table S. 2.1. The conditions relating to snow loading in the UK are classified under Case B2 (below). The conditions which are classified (in the UK) as exceptional snow drifts are:
EC1-1-3 NA.2.18	• snow drifts on multi-span roofs;
EC1-1-3 NA.2.21	• snow drifts on roofs abutting and close to taller construction;
EC1-1-3 NA.2.23	• drifting (local effects) at projections, obstructions and parapets.
EC1-1-3 5.2(3)	For **exceptional conditions**, the snow load on a roof is calculated as: $s = \mu_i s_k$ in which μ_i is the snow load shape coefficient s_k is the characteristic snow load on the ground

Source reference

EC1-1-3 Table A.1

Table S. 2.1: Design situations and load arrangements for different locations

Normal		Exceptional conditions	
Case A	**Case B1**	**Case B2**	**Case B3**
No exceptional falls No exceptional drift	Exceptional falls No exceptional drift	No exceptional falls Exceptional drift	Exceptional falls Exceptional drift
Persistent/transient design situation [1] Undrifted $\mu_i\ C_e\ C_t\ s_k$ [2] Drifted $\mu_i\ C_e\ C_t\ s_k$	Persistent/transient design situation [1] Undrifted $\mu_i\ C_e\ C_t\ s_k$ [2] Drifted $\mu_i\ C_e\ C_t\ s_k$ Accidental design situation (where snow is the accidental action) [3] Undrifted $\mu_i\ C_e\ C_t\ C_{esl}\ s_k$ [4] Drifted $\mu_i\ C_e\ C_t\ C_{esl}\ s_k$	Persistent/transient design situation [1] Undrifted $\mu_i\ C_e\ C_t\ s_k$ [2] Drifted $\mu_i\ C_e\ C_t\ s_k$ (except for roof shapes in Annex B) Accidental design situation (where snow is the accidental action) [3] Drifted $\mu_i\ s_k$ (for roof shapes in Annex B)	Persistent/transient design situation [1] Undrifted $\mu_i\ C_e\ C_t\ s_k$ [2] Drifted $\mu_i\ C_e\ C_t\ s_k$ (except for roof shapes in Annex B) Accidental design situation (where snow is the accidental action) [3] Undrifted $\mu_i\ C_e\ C_t\ C_{esl}\ s_k$ [4] Drifted $\mu_i\ s_k$ (for roof shapes in Annex B)

NOTE 1 Exceptional conditions are defined according to National Annex.
NOTE 2 For cases B1 and B3 the National Annex may define design situations which apply for the particular local effects described in Section 6.

Source reference	

Stage 3: Characteristic snow load on the ground

To calculate the snow load on the roof, the characteristic snow load on the ground s_k and the snow load shape coefficient μ_i have to be determined.

EC1-1-3 NA.2.8

The characteristic snow load on the ground is obtained from Equation S. 3.1 and Figure S. 3.1.

EC1-1-3 Eqn NA.1

$$s_k = \left[0.15 + \left(0.1Z + 0.05\right)\right] + \left(\frac{A - 100}{525}\right) \qquad \text{(S. 3.1)}$$

in which:

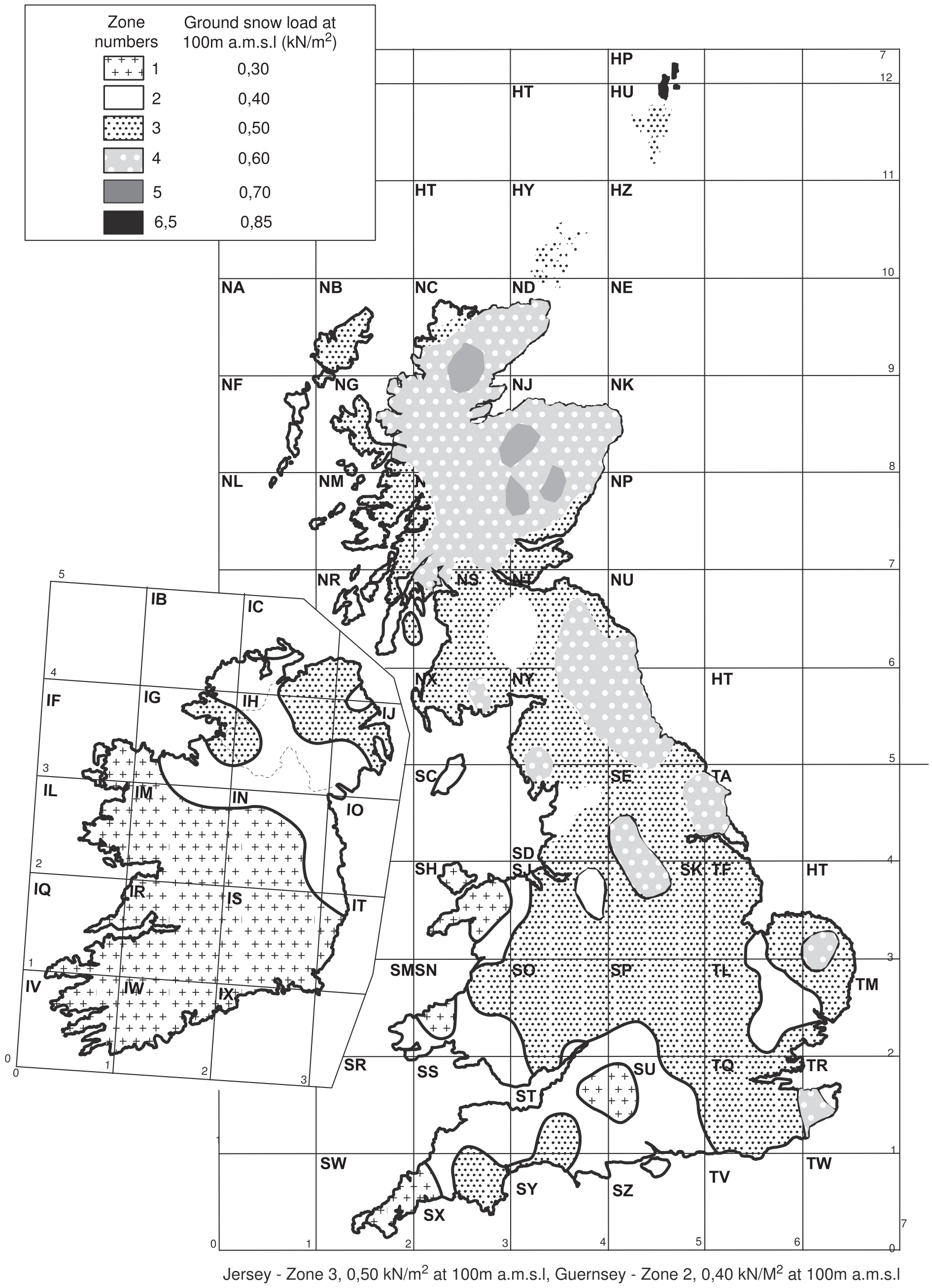
s_k is the characteristic ground snow load (kN/m^2)
Z is the zone number, obtained from Figure S 3.1
A is the site altitude (m)

EC1-1-3 NA.2.8

In Figure S. 3.1, unusual **local effects** may not have been considered. These include local shelter from the wind and local configurations in mountainous areas. These may contribute to higher characteristic loads.

If local effects need to be accounted for in design, then for coastal sites below 100 m the map in Figure S. 3.1 can be used without an altitude modification. In all other cases consult the Meteorological Office.

Source reference	
EC1-1-3 Figure NA.1	**Figure S. 3.1: Characteristic ground snow load map for the UK and Ireland**



Source reference

Stage 4: Snow load shape coefficients

EC1-1-3 5.1 (1)P

Snow loads can be deposited on a roof in many different patterns. To account for these, two primary load arrangements are considered in design. They are:

- undrifted snow loads;
- drifted snow loads.

EC1-1-3 5.2 (4)

The load is assumed to act vertically on a horizontal projection of the roof area.

In general, undrifted load arrangements are considered as a uniform snow load on the entire roof, while drifted load arrangements are considered as individual uniform snow loads acting alone on different parts of the roof. Several drifted load arrangements may need to be examined to determine the worst condition.

The shape of the drifted load depends on the shape of the roof. This in turn determines the snow load shape coefficient for subsequent calculation of the snow load on the roof.

The five roof types considered in EN1991-1-3 are:

1. Monopitch roofs
2. Duo-pitch roofs
3. Multi-span roofs
4. Cylindrical roofs
5. Roofs abutting and close to taller constructions

EC1-1-3 NA.2.12

In the UK, drifted snow load conditions on multi-span roofs and roofs abutting and close to taller constructions are classified as **exceptional snow drifts**, and are therefore designed under the **accidental** design situation. In addition, drifts at projections, obstructions and parapets are treated as accidental loads.

EC1-1-3 6.2 & 6.3

However, drifts due to snow overhanging the edge of a roof, and snow loads on snow-guards, are treated under **normal** conditions.

The following sub-sections outline how to calculate the snow load shape coefficients due to the various roof shapes and drift conditions.

4.1 Monopitch roof

EC1-1-3 5.3.2(1)

The snow load shape coefficient values to be used for monopitch roofs are given in Table S. 4.1 and illustrated in Figure S. 4.1.

EC1-1-3 5.3.2(3)

The load arrangement shown in Figure S. 4.1 is used for both the **undrifted** and **drifted** cases.

Source reference

EC1-1-3 5.3.2(2)

Where there are obstructions, snow fences, or the lower edge of the roof is terminated with a parapet, $\mu_1 = 0.8$.

IStructE EC1 Manual

Large flat roofs, treated as monopitch roofs with $\alpha = 0°$, should have $\mu_1 = 1.0$.

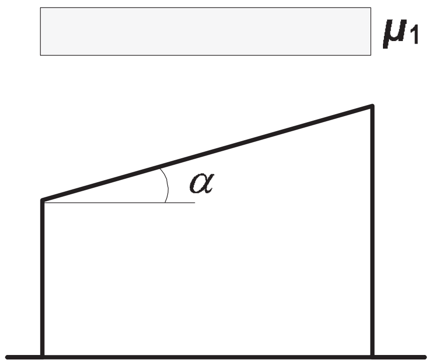
EC1-1-3 Table 5.2

Table S. 4.1: Snow load shape coefficients for monopitch roofs

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0.8	$0.8(60 - \alpha)/30$	0.0

EC1-1-3 Figure 5.2

Figure S. 4.1: Snow load shape coefficients for monopitch roofs



4.2 Duo-pitch roofs

Undrifted case

EC1-1-3 5.3.3(1)

The snow load shape coefficients to be used for the **undrifted** case are treated as if the roofs were two monopitch roofs. The values are given in Table S. 4.1 and shown as Case (i) in Figure S. 4.2.

Drifted case

EC1-1-3 NA.2.17

The snow load shape coefficients to be used for the drifted case are given in Table S. 4.2 and shown as Cases (ii) and (iii) in Figure S. 4.2. Both cases should be examined.

Where there are obstructions, snow fences, or the lower edge of the roof is terminated with a parapet, $\mu_1 = 0.8$.

Source reference

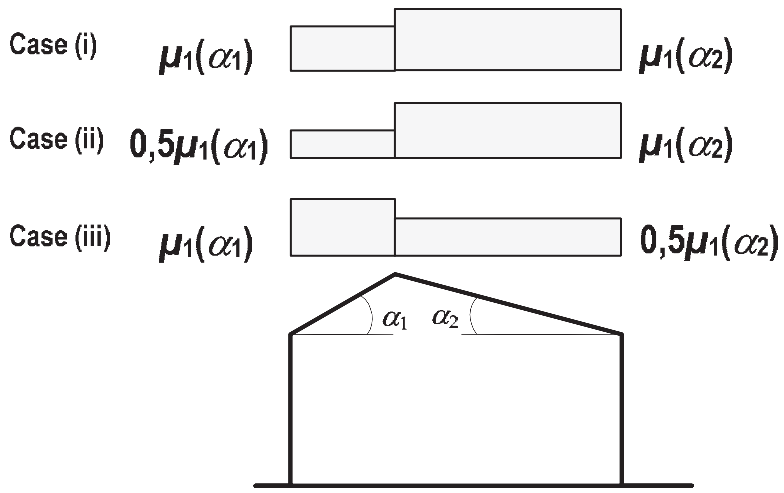
EC1-1-3 Table NA.1

Table S. 4.2: Drifted snow load shape coefficients for duo-pitch roofs

Snow load shape coefficient	Angle of pitch of roof (α_i for i =1,2)			
	0° ≤ α_i ≤ 15 °	15 ° < α_i ≤ 30°	30° < α_i ≤ 60°	α_i ≥ 60°
μ_i	0.8	0.8 + 0.4(α_i – 15)/15	1.2(60 – α_i)/30	0.0

IStructE EC1 Manual

Figure S. 4.2: Snow load shape coefficients for duo-pitch roofs



4.3 Multi-span roofs

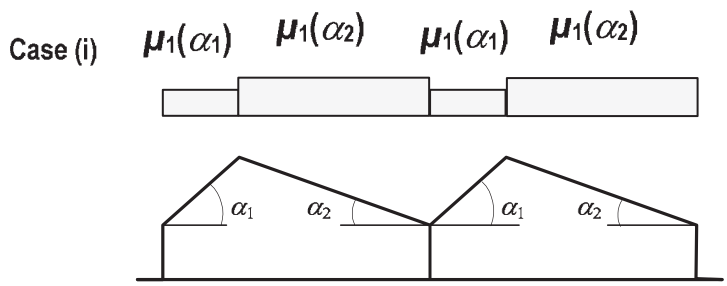
Undrifted case

EC1-1-3 5.3.4(1)

The snow load shape coefficients to be used for the **undrifted** load arrangement are given in Table S. 4.1 and shown in Figure S. 4.3.

EC1-1-3 Fig 5.4(i)

Figure S. 4.3: Snow load shape coefficients for undrifted snow loading for multi-span roofs



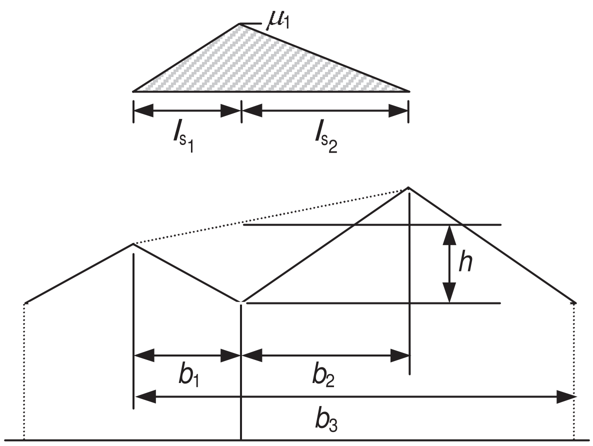
Source reference

Drifted case

EC1-1-3 NA.2.18 The drifted load arrangement is treated as an exceptional snow drift, and has to be designed as an accidental design situation.

EC1-1-3 B2(1) The snow load shape coefficients to be used for drifted load arrangements of multi-span roofs are shown in Figure S. 4.4 and are given by the least of the μ_1 values in Equation S. 4.1.

EC1-1-3 Fig B1 **Figure S. 4.4: Shape coefficient and drift lengths for exceptional snow drifts for multi-span roofs**



EC1-1-3 B2(2)

$$\mu_1 = 2h/s_k$$

$$\mu_1 = 2b_3/(l_{s1} + l_{s2}) \qquad \text{(S. 4.1)}$$

$$\mu_1 = 5$$

where

$l_{s1} = b_1$, $l_{s2} = b_2$

EC1-1-3 B2(3) For roofs with more than two spans which are approximately symmetrical and have uniform geometry:

$b_3 = span \times 1.5$ (or the horizontal dimension of the three slopes)

The snow load distribution should be applied to every valley, but not necessarily simultaneously.

EC1-1-3 B2(5) Where simultaneous drifts in several valleys of a multi-span roof are being considered in the design of a structure as a whole, the total snow load per metre within all the simultaneously considered drifts should not exceed the product of the ground snow load and the length of the building perpendicular to the valley ridges.

To account for asymmetric loading, the design should consider the possibility of patterns of drifts of differing severity in the valleys.

Source reference

4.4 Cylindrical roofs

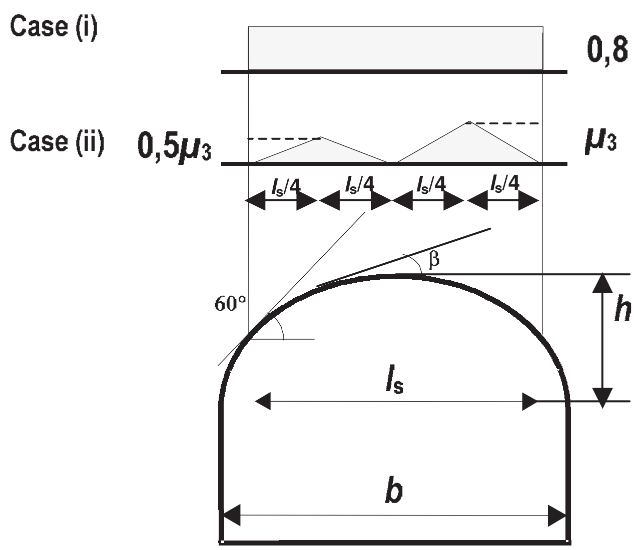
Undrifted case

EC1-1-3 5.3.5(2)

The snow load shape coefficient that should be used for the undrifted load arrangement is μ_1 = 0.8, as shown in Figure S. 4.5.

EC1-1-3 Figure 5.6

Figure S. 4.5: Undrifted snow load shape coefficient for cylindrical roofs



Drifted case

EC1-1-3 NA.2.20

The load arrangement shown in Figure S. 4.6 and Table S. 4.3 should be used to determine the snow load shape coefficient for the drifted case in cylindrical roofs.

The angles shown in Figure S. 4.6 are defined as follows:

- δ is the angle between the horizontal and the tangent to the roof at the eaves.
- α for ($\delta \leq 60$ °) is the angle between the horizontal and a line drawn from the crown to the eaves.
- α for ($\delta > 60$ °) is the angle between the horizontal and a line drawn from the crown to the point of the roof where the tangent to the surface makes an angle of 60° with the horizontal.

The load arrangements in Figure S. 4.6 need only be considered for roofs where α is greater than 15°. The value for the snow load shape coefficient for one side of the roof should be zero, while the values for the other side should be obtained from Figure S. 4.6 and Table S. 4.3. The values for the snow load shape coefficients are assumed to be constant in the direction parallel to the eaves.

Source reference

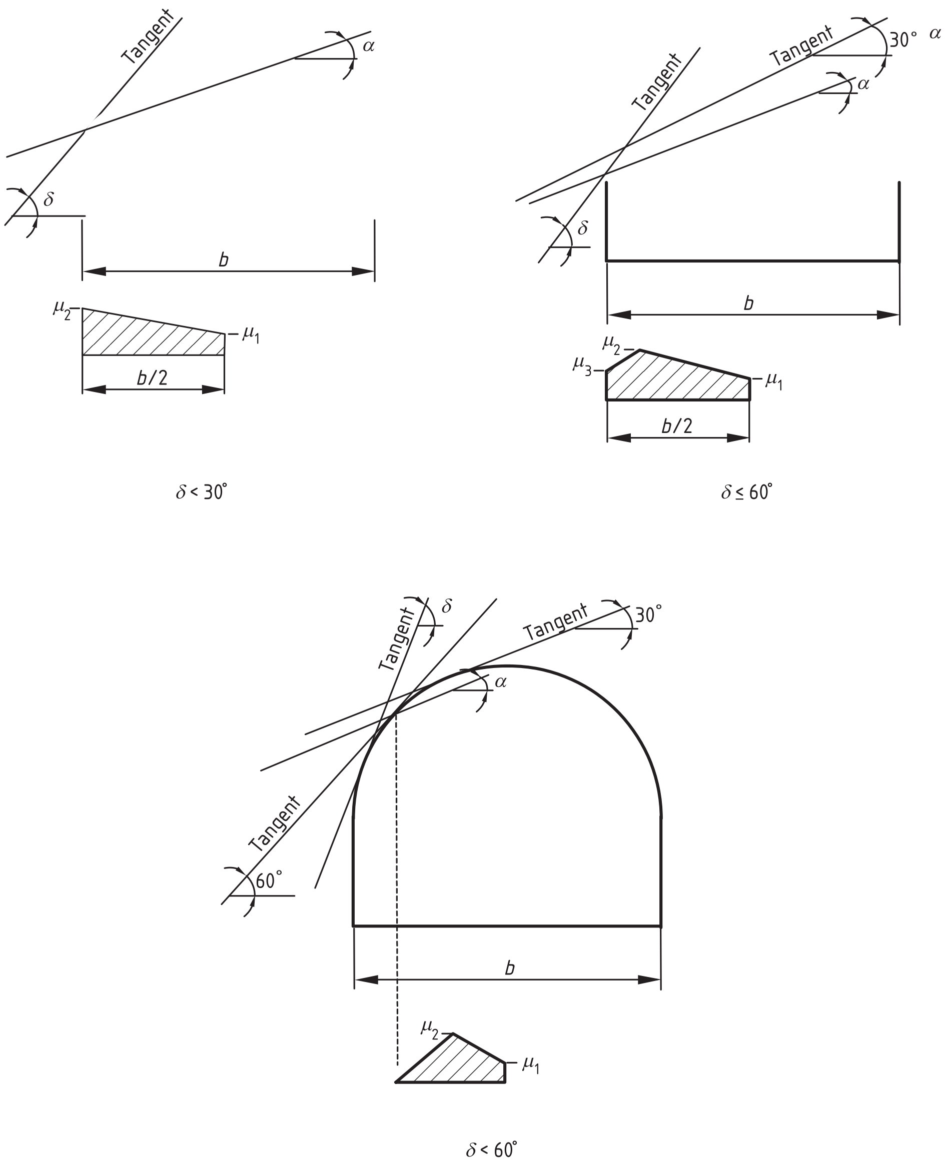
EC1-1-3 Table NA.2

Table S. 4.3: Drifted snow load shape coefficients for cylindrical roofs in the UK

Snow load shape coefficient	Equivalent slope for a curved roof α			
	$0° \leq \alpha_i \leq 15°$	$15° < \alpha_i \leq 30°$	$30° < \alpha_i < 60°$	$\alpha_i \geq 60°$
μ_1	0	0.4		0
μ_2	0	$0.8 + 0.4[(\alpha - 15)/15]$	$1.2\ (60 - \alpha)/30$	0
μ_3	0	$\mu_2\ (60 - \delta)/30$	$\mu_2\ (60 - \delta)/30$	0

EC1-1-3 Figure NA.3

Figure S. 4.6: Drifted snow load arrangements for a cylindrical roof in the UK



Source reference

4.5 Roof abutting and close to taller construction

Undrifted case

EC1-1-3 5.3.6(1)

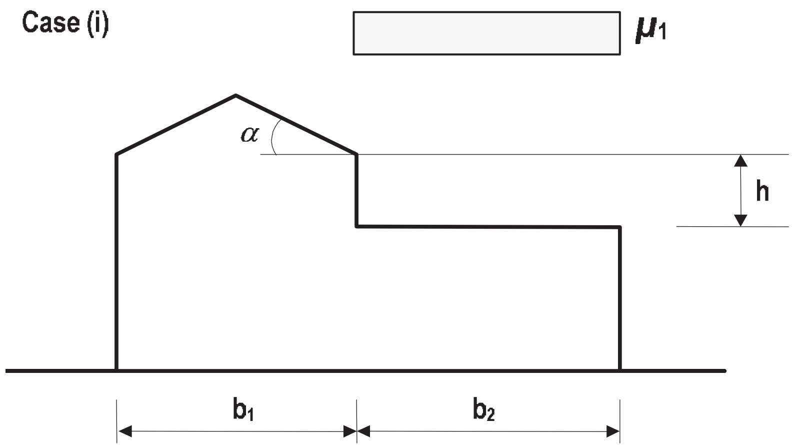
The snow load shape coefficient that should be used for the undrifted load arrangement for roofs abutting and close to taller construction is $\mu_1 = 0.8$.

It is assumed that the lower roof is flat.

The load arrangement to be used is shown in Figure S. 4.7.

EC1-1-3 Figure 5.7(i)

Figure S. 4.7: Snow load shape coefficients for roofs abutting taller construction



Drifted case

EC1-1-3 NA.2.22

The drifted load arrangement is considered an **exceptional** snow drift in the UK. The design therefore counts as an **accidental** design situation.

EC1-1-3 B3(1)

The snow load shape coefficients for the drifted load arrangement are given in Figure S. 4.8 and Table S. 4.4.

EC1-1-3 Table B1

Table S. 4.4: Shape coefficients for drifts for roofs abutting and close to taller construction

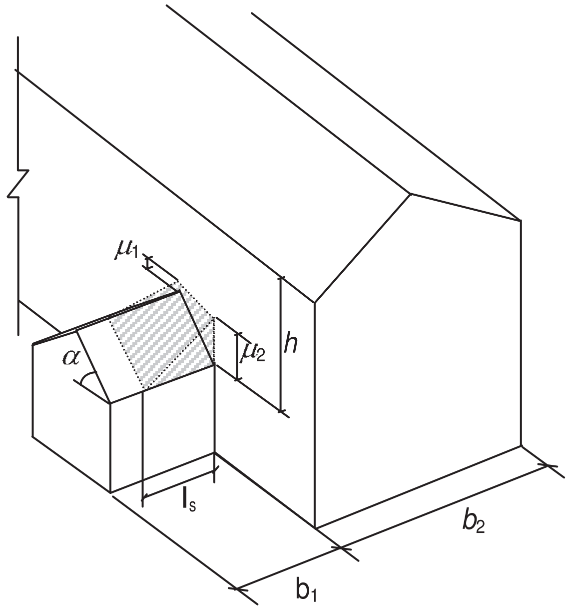
Snow load shape coefficient	Angle of roof pitch α_1			
	$0° \le \alpha_i \le 15°$	$15° < \alpha_i \le 30°$	$30° < \alpha_i < 60°$	$\alpha_i \ge 60°$
μ_1	μ_3	$\mu_3\,(30 - \alpha)/15$	0	0
μ_2	μ_3	μ_3	$\mu_3(60 - \alpha)/30$	0

NOTE μ_3 is the least value of $2h/s_k$, $2b/l_s$ or 8. where b is the larger of b_1 or b_2 and l_s is the least value of $5h$, b_1 or 15 m.

Source reference

EC1-1-3 Figure B2

Figure S. 4.8: Shape coefficients and drift lengths for drifts on roofs abutting and close to taller structures



EC1-1-3 B3(2)

The snow load arrangement shown in Figure S. 4.8 applies also for roofs close to, but not abutting, taller buildings. However, it is necessary to consider the actual load on the lower roof.

4.6 Local effects – roofs where drifting occurs at projections, obstructions and parapets

EC1-1-3 6.1(1)

This section outlines how to calculate forces applied locally by drifting at:

1. Projections, obstructions and parapets
2. Edges of the roof
3. Snow fences

EC1-1-3 NA. 2.23

Drifting at projections, obstructions and parapets are considered **exceptional** snow drifts, and therefore follow the **accidental** design principle.

Drifting at projections and obstructions

EC1-1-3 B4(1)

The snow load shape coefficients for drifting at projections and obstructions are μ_1 and μ_2, defined in Figure S. 4.9.

Source reference

EC1-1-3 B4(2) — If the vertical elevation against which a drift could occur is not greater than 1 m, the effect of drifting can be ignored. This applies to:

- drifting on canopies projecting not more than 5 m from the face of the building over doors and loading bays, irrespective of the height of the obstruction;
- slender obstructions over 1 m high but not more than 2 m wide may be considered as local projections. For this case h may be taken as the lesser of the projection height or width perpendicular to the direction of the wind.

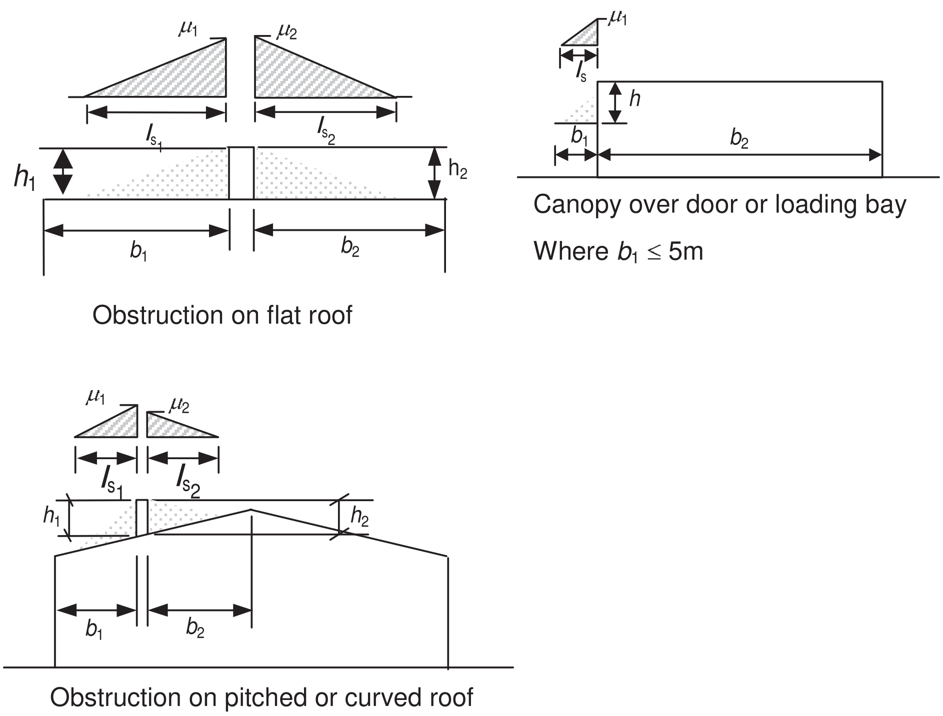
EC1-1-3 B4(2)c) — The shape coefficients defined in Figure S. 4.9 are the lower of:

$\mu_1 = 2h_1/s_k$ or 5
$\mu_2 = 2h_2/s_k$ or 5

For door canopies projecting not more than 5 m from the building, μ_1 should not exceed $2b/l_{s1}$, where b is the larger of b_1 and b_2.

EC1-1-3 B4(2)d) — The drift length (l_{si}) is taken as the least value of $5h$ or b_i, where i = 1 or 2 and $h \leq 1$m.

EC1-1-3 Fig B3 — **Figure S. 4.9: Snow load shape coefficients for exceptional snow drifts for roofs where drifting occurs at projections and obstructions**



Source reference

Drifting at parapets

EC1-1-3 B4(3)

The snow load shape coefficient that should be used for a roof where drifting occurs at parapets is given by the least value of Equations S. 4.2, with variables defined in Figure S. 4.10.

$$\mu_1 = 2h/s_k$$
$$\mu_1 = 2b/l_s \qquad \text{(S. 4.2)}$$
$$\mu_1 = 8$$

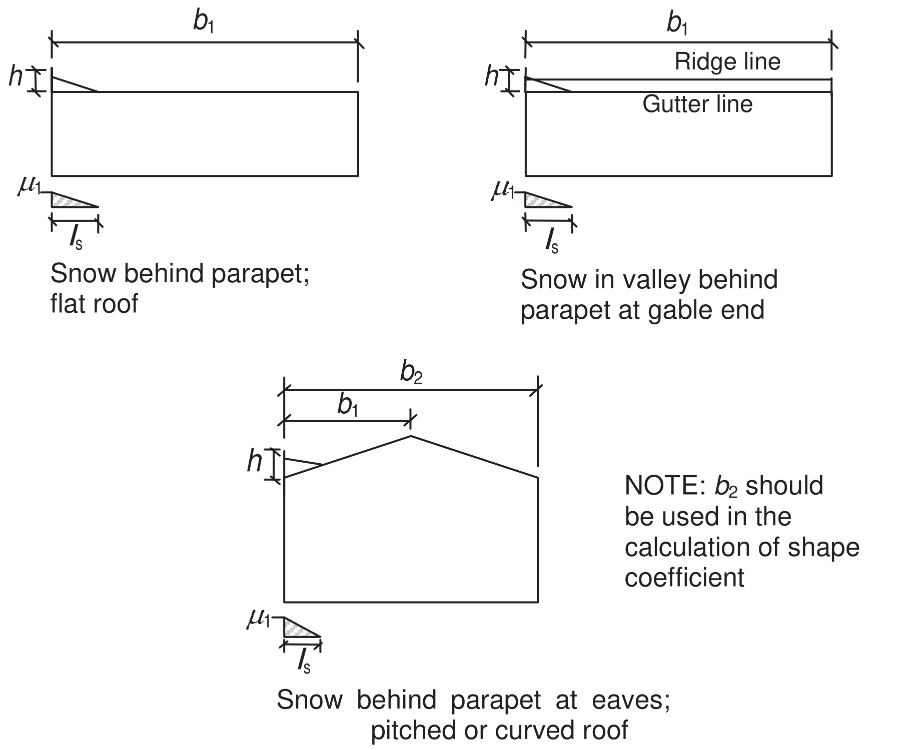
where b is the larger of b_1 or b_2 and the drift length (l_s) should be taken as the least value of $5h$, b_1 or 15 m.

EC1-1-3 B4(5)

For drifting in a valley behind a parapet at a gable end, the snow load at the face of the parapet should be assumed to decrease linearly from its maximum value in the valley to zero at the adjacent ridges, providing the parapet does not project more than 300 mm above the ridge.

EC1-1-3 Fig B4

Figure S. 4.10: Shape coefficients for local effects – roofs where drifting occurs at parapets



Source reference	
	Stage 5: Snow load on roofs
	Persistent or transient design situation
EC1-1-3 5.2(3)P a)	$s = \mu_i C_e C_t s_k$ for normal conditions
	in which:
	μ_i is the snow load shape coefficient C_e is the exposure coefficient C_t is the thermal coefficient s_k is the characteristic snow load on the ground
	Stages 3 and 4 have shown how the characteristic snow load on the ground and the snow load shape coefficient can be obtained.
	Exposure coefficient (UK)
EC1-1-3 5.2(7) & EC1-1-3 NA.2.15	$C_e = 1.0$
	Thermal coefficient (UK)
EC1-1-3 5.2(8) & EC1-1-3 NA.2.16	$C_t = 1.0$
	Accidental design situation
EC1-1-3 5.2(3)P c)	$s = \mu_i s_k$ for exceptional snow drifts
	Stage 6: Local effects
	Drifting at the edges of a roof and on snow-guards
	Snow overhanging the edge of a roof
EC1-1-3 NA2.24	The effects of snow overhanging the edge of a roof should be considered for sites at altitudes greater than 800 m above sea level.
EC1-1-3 6.3(2)	The design of roofs cantilevered out beyond the walls should take account of snow overhanging the edge of the roof, in addition to the load on that part of the roof.
	The loads due to the overhang may be assumed to act at the edge of the roof and may be calculated using:
	$s_e = k\, s^2/\gamma$ (S. 6.1)

Source reference

in which:

s_e is snow load per metre length due to the overhang (see Figure S. 6.1)

s is the most onerous undrifted load case appropriate for the roof under consideration

γ is the weight density of snow which may be taken as 3 kN/m^3

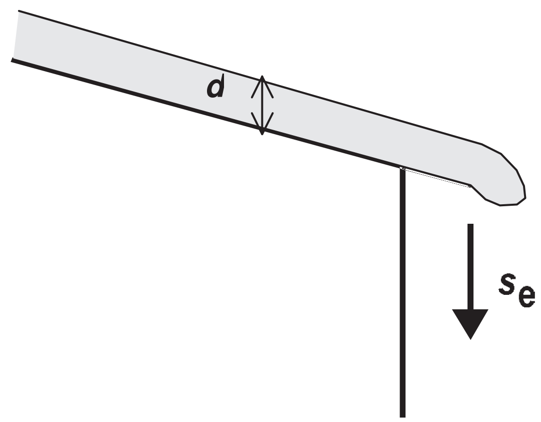
k is a coefficient to take account of the irregular shape of the snow, and

$k = 3/d$ but $k \leq d\,\gamma$

where d is the depth of the snow layer on the roof in metres (see Figure S. 6.1)

EC1-1-3 Figure 6.2

Figure S. 6.1: Snow overhanging the edge of a roof



Snow loads on snow-guards and other obstacles

EC1-1-3 6.4(1)

Under certain conditions snow may slide down a pitched roof. The coefficient of friction between the snow and the roof may be assumed to be zero. For snow sliding down a pitched or curved roof, the force F_s exerted by the sliding mass per unit length of the building is:

$$F_s = s.b.\sin\alpha \quad \text{(S. 6.2)}$$

in which:

s is the snow load on the roof relative to the most onerous **undrifted** load case appropriate for roof area from which snow could slide (see 4.1.1)

b is the width on plan (horizontal) from the guard or obstacle to the next guard or to the ridge

α is the pitch of the roof, measured from the horizontal

BS EN 1991-1-4: Eurocode 1 – Wind loading

Source reference

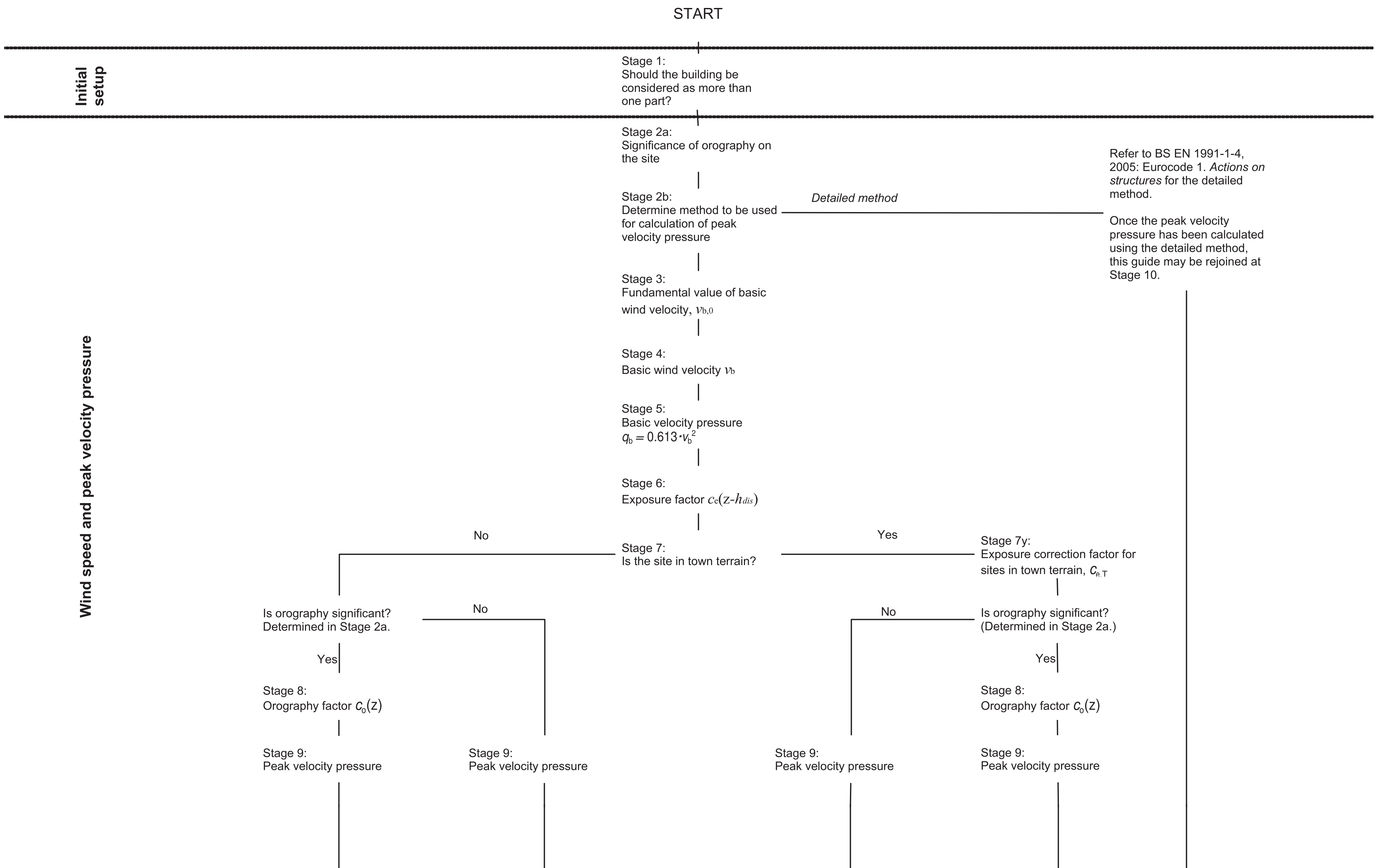
Wind loading

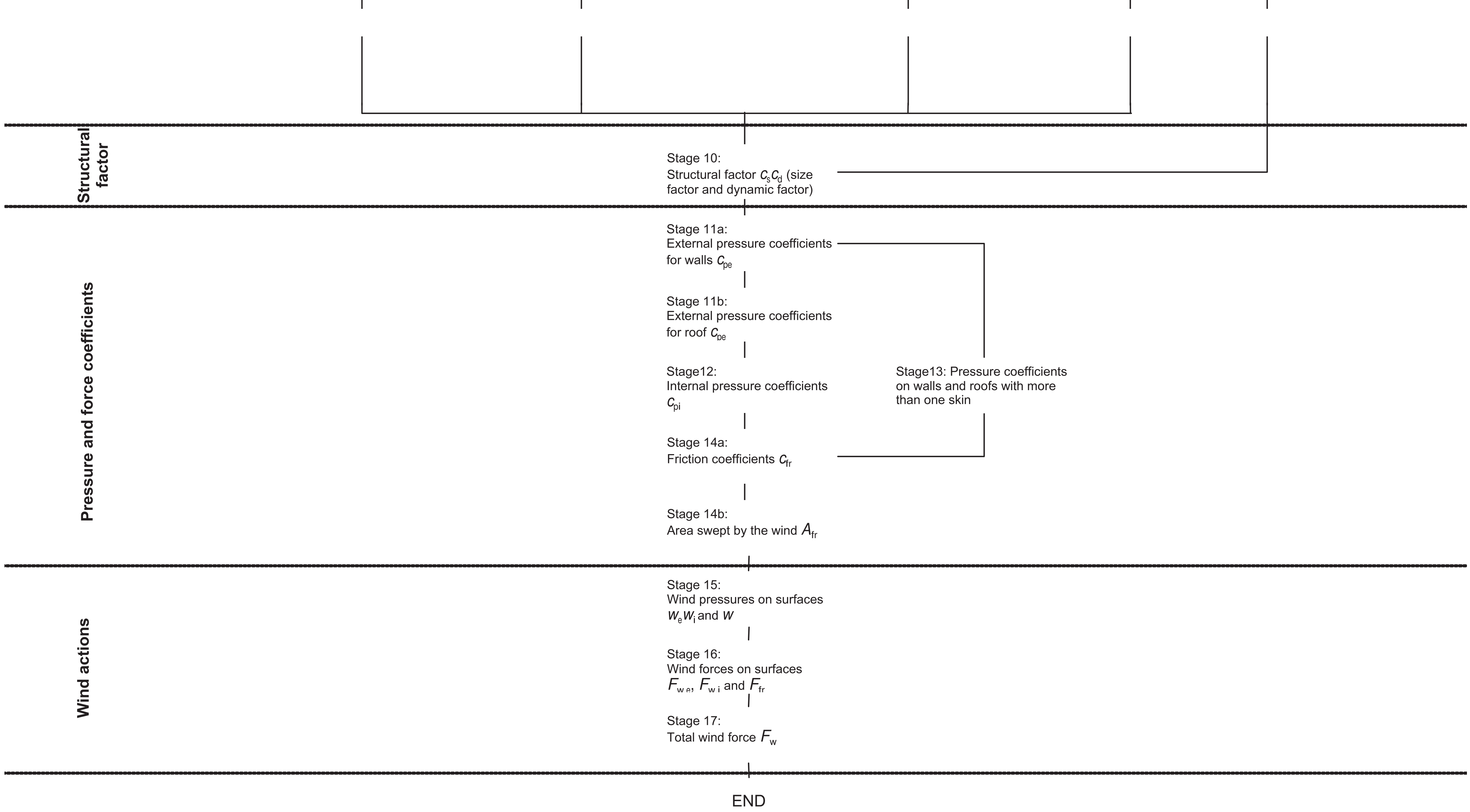
Introduction and outline of procedure

The effect of wind on a building depends on many factors, from the general wind climate and the exposure of the building to the shape and dimensions of the structure. This section aims to explain the main issues affecting wind loading, so that they can be readily understood.

The procedure for calculating wind loads on a building is explained in a step-by-step manner, in accordance with Eurocode 1 and the UK National Annex. A flowchart showing the 17 stages of the process is also provided.

Flowchart of wind action calculation





Structural factor
Stage 10:
Structural factor $c_s c_d$ (size factor and dynamic factor)
Pressure and force coefficients
Stage 11a:
External pressure coefficients for walls c_{pe}
Stage 11b:
External pressure coefficients for roof c_{pe}
Stage12:
Internal pressure coefficients c_{pi}
Stage13: Pressure coefficients on walls and roofs with more than one skin
Stage 14a:
Friction coefficients c_{fr}
Stage 14b:
Area swept by the wind A_{fr}
Wind actions
Stage 15:
Wind pressures on surfaces $w_e w_i$ and w
Stage 16:
Wind forces on surfaces $F_{w\,e}$, $F_{w\,i}$ and F_{fr}
Stage 17:
Total wind force F_w
END

Source reference

Initial consideration of the building

Stage 1: Should the windward face be considered as more than one part?

Ratio of h/b:

For windward walls of rectangular plan buildings:

Depending on the ratio of height to breadth, the windward wall may be considered as one part or a number of parts. The following three cases explain how the windward wall should be considered:

a) $h < b$
This should be considered as one part of height h.

b) $b < h < 2b$

This may be considered as two parts: a lower part equal in height to b and an upper part consisting of the remainder of the height (h-b).

c) $h > 2b$

This may be considered as multiple parts: a lower part equal in height to b, an upper part extending downwards from the top equal in height to b and a middle region which may be divided into horizontal strips, each with a height h_{strip}.

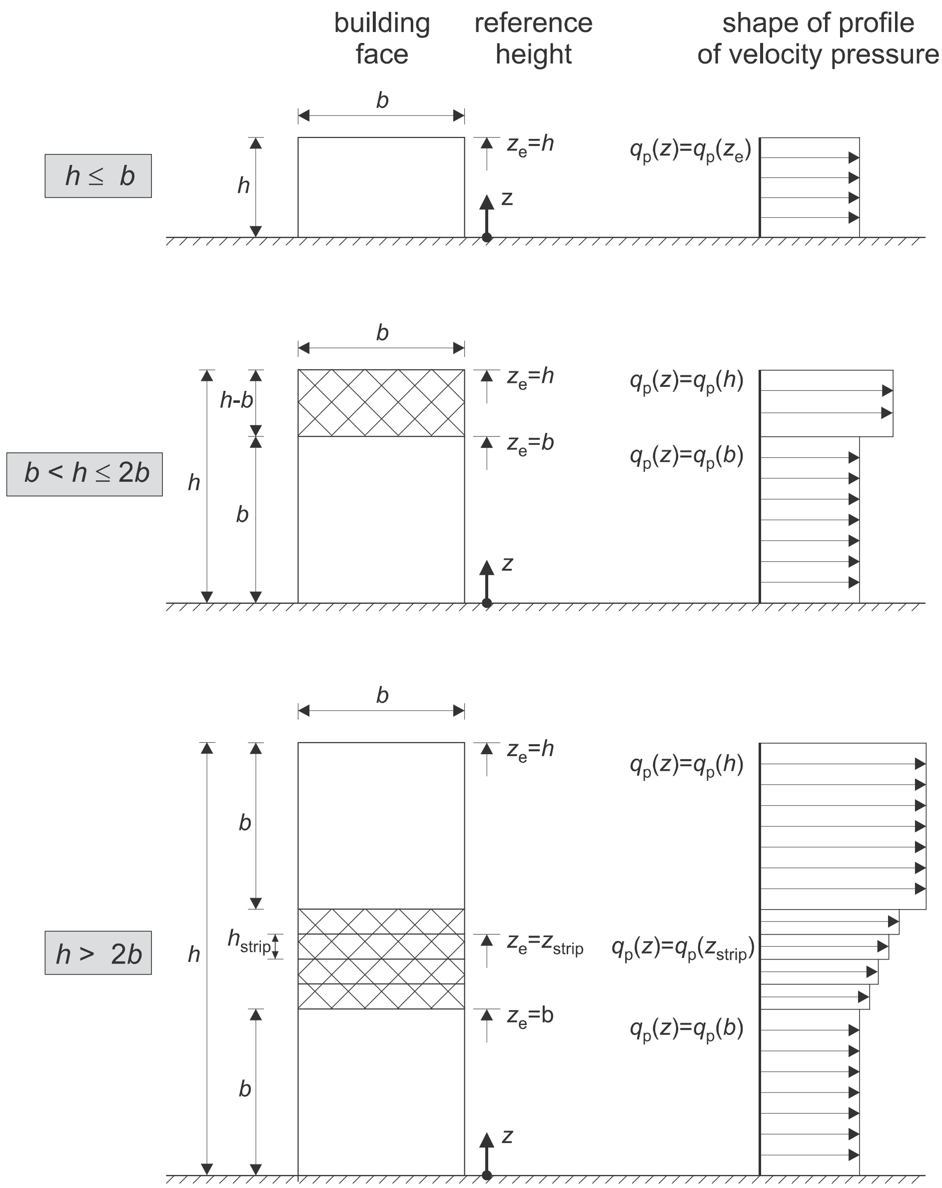
Figure W. 1.1 shows the reference heights z_e for these three cases. The reference height is always taken from the top of the element or wall being considered.

If the building needs to be considered as more than one part, each part needs to be considered separately. For example, if there are two parts, the following steps need to be carried out twice, once for each part.

Source reference

EC1-1-4 Figure 7.4

Figure W. 1.1: Reference height z_e depending on *h* and *b* of the building, and corresponding velocity pressure profiles



NOTE The velocity pressure should be assumed to be uniform over each horizontal strip considered.

Wind speed *v* and peak velocity pressure

This section outlines the procedure for calculating the site wind speed and corresponding peak velocity pressure for any given location and site exposure.

Wind speed is assumed to be a combination of a fluctuating wind speed and a mean wind speed. This combination of wind speeds is used to determine peak velocity pressure. The peak velocity pressure is later required to calculate wind pressures and forces.

Source reference

Stage 2a: Significance of orography on the site

Definition Orography is the section of physical geography that deals with mountains and hills. The consideration of orography is important in calculating wind pressure, as the terrain of the land can greatly alter the local wind characteristics. Greater wind speeds can be created at the top of a slope than at the base, by the upwind slope ϕ.

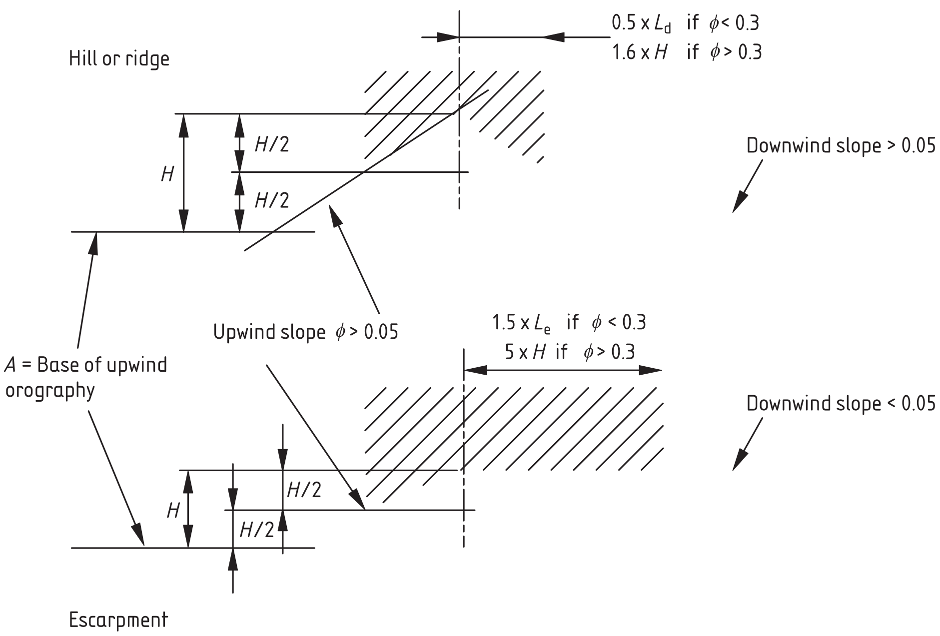
Is orography significant?

If the upwind slope ϕ is greater than 0.05 (an angle of 2.865°) and the building site lies within a certain distance from the crest of the slope *X*, then orography is significant.

Orography should be taken into consideration for the situations shown in Figure W. 2.1 and Tables W. 2.1, W. 2.2 and W. 2.3.

EC1-1-4 Figure NA.2

Figure W. 2.1: Areas where orography is significant



NOTE Hatched areas show where orography is significant.

EC1-1-4 Annex A.3 (3)

Table W. 2.1: Hills or ridges (downwind slope > 0.05)

Upwind slope ϕ	Site location *X*
$0.05 < \phi$	$X \leq 0.5\ L_u$
$\phi < 0.3$	$X < 0.5\ L_d$
$\phi \geq 0.3$	$X < 1.6\ H$

Source reference

EC1-1-4 Annex A.3 (3)

Table W. 2.2: Escarpments and cliffs (downwind slope < 0.05)

Slope ϕ	Horizontal distance of the site from the top of the crest X
$0.05 < \phi$	$X \leq 0.5\ L_u$
$\phi < 0.3$	$X < 1.5\ L_e$
$\phi \geq 0.3$	$X < 5\ H$

ϕ upwind slope H/L_u
L_u horizontal length of the upwind slope
L_e effective length of the upwind slope, defined in Table W. 3.3
L_d horizontal length of the downwind slope
H height of the feature from base of upwind orography to the crest of the slope
X horizontal distance of the site from the top of the crest

EC1-1-4 Table A.2

Table W. 2.3: Value of the effective length L_e

Shallow ($0.05 < \phi < 0.3$)	Steep ($\phi > 0.3$)
$L_e = L_u$	$L_e = H / 0.3$

Stage 2b: Determine method to be used for calculation of peak velocity pressure

The method adopted for determining peak velocity pressure depends on the height of the building and whether orography is significant.

- Is the building >50 m tall?
- Is orography significant? (answered in Stage 2a)

If the answer to **both** of the above questions is **yes**, the **detailed method** should be used.

For all other cases the **simplified method,** explained in this guide, may be used.

Source reference

The simplified method for determining peak velocity pressure

Stage 3: Fundamental value of basic wind velocity $v_{b,0}$

Definition The mean wind velocity over a 10-minute period at 10 m above ground level in flat open country terrain. The value gained is irrespective of wind direction and time of year and takes into account a 0.02 annual risk of being exceeded.

EC1-1-4 Eqn.(NA.1)

$$v_{b,0} = v_{b,map} c_{alt} \quad \text{(W. 3.1)}$$

$v_{b,map}$ the value of the fundamental basic wind velocity before altitude correction is applied, given in **Figure W. 3.1**

c_{alt} the altitude factor, which takes into account the height of the site above sea level

EC1-1-4 Eq.(NA.2a)

$$c_{alt} = 1 + 0.001A \qquad \text{for } z \leq 10 \text{ m} \qquad \text{(W. 3.2a)}$$

EC1-1-4 Eq.(NA.2a)

$$c_{alt} = 1 + 0.001A(10/(z - h_{dis}))^{0.2} \qquad \text{for } z > 10 \text{ m} \qquad \text{(W. 3.2b)}$$

NOTE Equation W. 3.2a can be used conservatively for any building height.

A where orography is not significant: the altitude of the site above mean sea level in metres

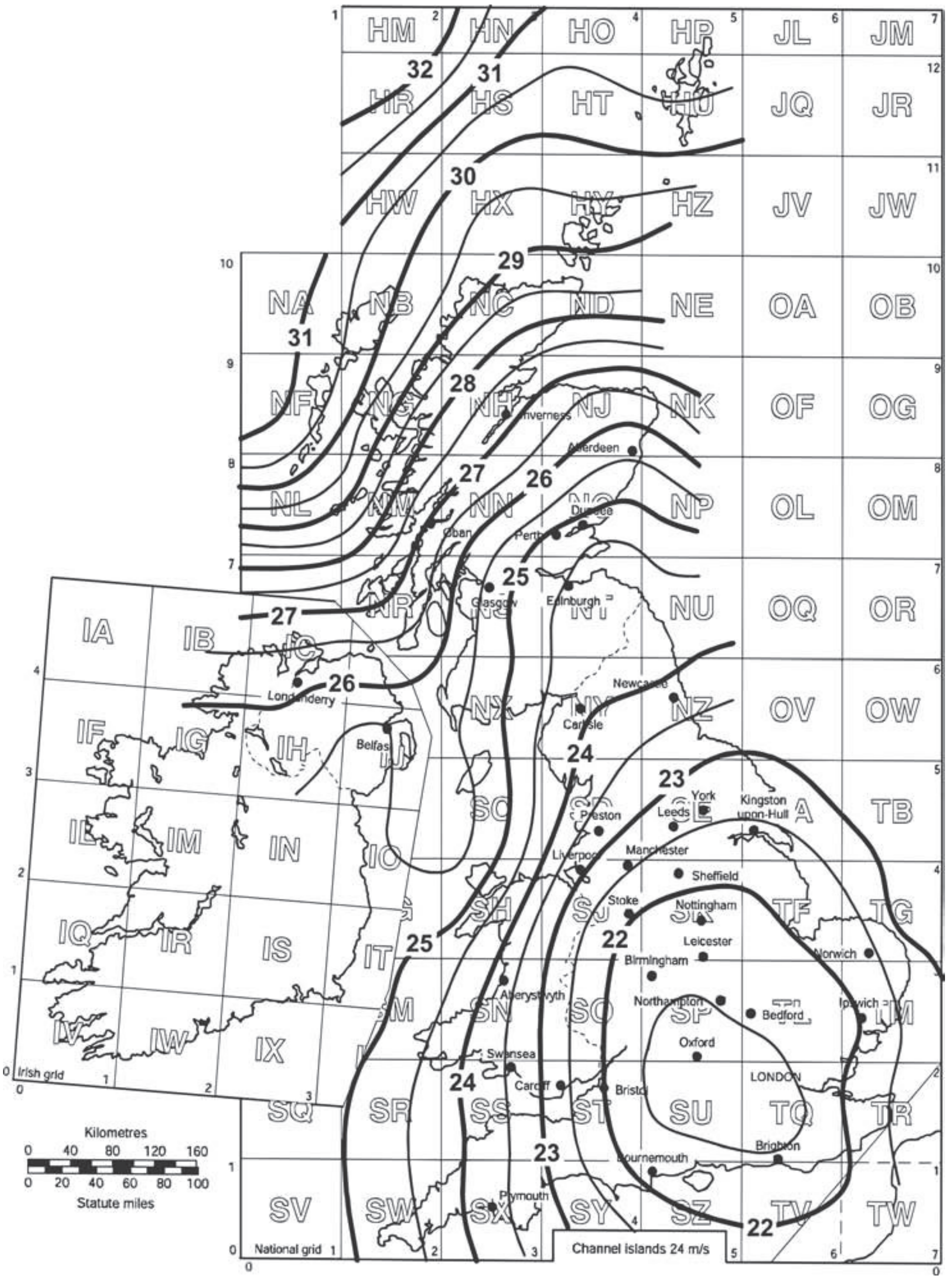
where orography is significant: the altitude of the upwind base of the site above mean sea level in metres

z either z_s as defined in Figure W. 3.2, if the windward face is considered as one part; or z_e, as defined in Figure W. 1.1, if the windward face is considered as more than one part

H_{dis} the displacement height

In areas where buildings with average height exceeding 15 m cover at least 15% of the ground, wind can be made by closely spaced buildings to behave as if the ground level were raised to a displacement height. See Figure W. 3.3 and Equations 3.3 a–c.

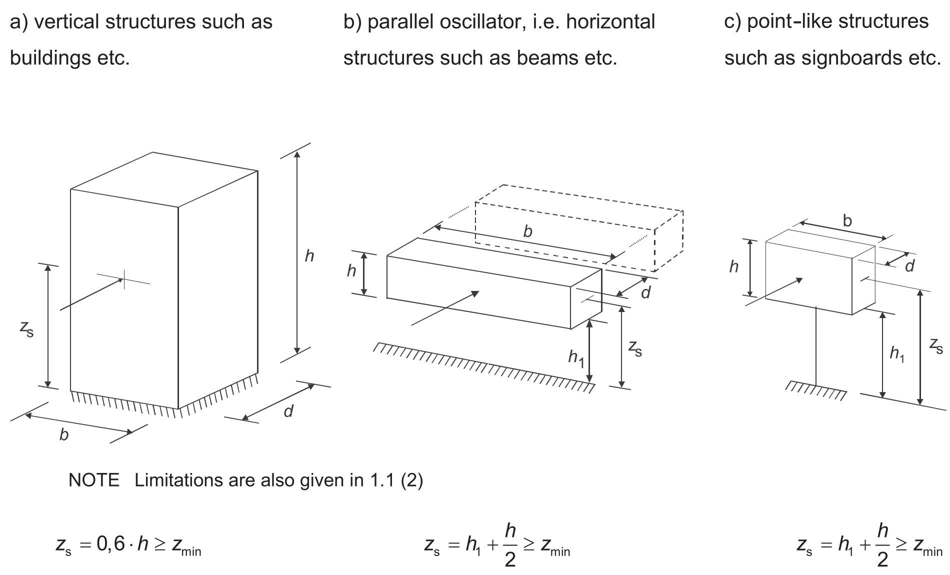
Source reference	
EC1-1-4 Figure NA.1	**Figure W. 3.1: Basic wind speed map**



Source reference

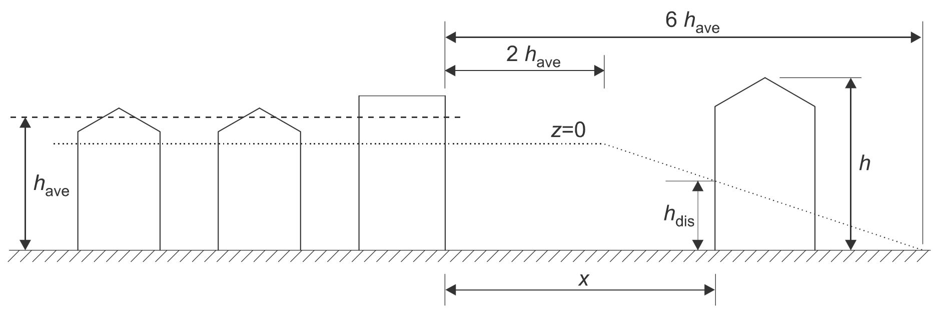
EC1-1-4 Figure 6.1

Figure W. 3.2: General shapes of structures covered by the design procedure, showing the structural dimensions and reference heights z_e



EC1-1-4 Figure A.5

Figure W. 3.3: Obstructing height and upwind spacing



For areas in country terrain

$h_{dis} = 0$ (W. 4.4a)

For areas in dense town terrain

$x \leq 2h_{ave}$ — h_{dis} is the lesser of $0.8h_{ave}$ or $0.6\ h$ (W. 4.4b)

EC1-1-4 Eqn. A.15

$2h_{ave} \leq x \leq 6h_{ave}$ — h_{dis} is the lesser of $1.2h_{ave} - 0.2x$ or $0.6\ h$ (W. 4.4c)

$x \geq 6h_{ave}$ — $h_{dis} = 0$ (W. 4.4d)

in which

h_{ave} is the average height of the obstructing buildings (if detailed information is not available the obstruction height h_{ave} may be taken as 15 m)

x is the horizontal distance to neighbouring building

h is the height of the building

Source reference

Stage 4: Basic wind velocity v_b

Definition The fundamental basic wind velocity, altered to take into account the season and the direction of the wind being considered.

EC1-1-4 Eqn.(4.1)

$$v_b = c_{dir}\, c_{season}\, v_{b,0}\, c_{prob} \quad \text{(W. 4.1)}$$

c_{dir} Directional factor – Table W. 4.1
This factor is used to consider the direction of the wind being considered.

c_{season} Seasonal factor – Table W. 4.2
This factor is used to assess the wind loads on temporary structures or structures under construction, for one-, two-, four- or six-month periods during the year. For non-temporary and finished structures this factor may be ignored.

$v_{b,0}$ Fundamental value of basic wind velocity, calculated in Stage 3.

c_{prob} Probability factor
This allows the annual probability of exceeding the basic wind velocity to be altered from 0.02.

$c_{prob} = 1$ for the general case of p = 0.02.

EC1-1-4 Eqn.(4.2) & EC1-1-4 NA.2.8

$$c_{prob} = \left[\frac{1-0.2\ln(-\ln(1-p))}{1-0.2\ln(-\ln(0.98))}\right]^{0.5} \quad \text{for all other cases.} \quad \text{(W. 4.6)}$$

p annual probability of values being exceeded

EC1-1-4 Table NA.1

Table W. 4.1: Directional factor c_{dir}

Direction	0°	30°	60°	90°	120°	150°	180°	210°	240°	270°	300°	330°
c_{dir}	0.7	0.73	0.73	0.74	0.73	0.8	0.85	0.93	1.00	0.99	0.91	0.82

NOTE 1 Interpolation may be used within Table W. 4.1.

NOTE 2 The directions are defined by angles from due North in a clockwise direction.

NOTE 3 Where the wind loading on a building is assessed only for orthogonal load cases, the maximum value of the factor for the directions that lie ± 45° either side of the normal to the face of the building is to be used.

NOTE 4 Conservatively, c_{dir} may be taken as 1.0 for all directions.

Source reference

EC1-1-4 Table NA.2

Table W. 4.2: Seasonal factor c_{season}

<table>
<tr><th>Months</th><th>1 month</th><th colspan="2">2 month</th><th colspan="4">4 month</th></tr>
<tr><td>January</td><td>0.98</td><td rowspan="2">0.98</td><td></td><td rowspan="4">0.98</td><td></td><td rowspan="2"></td><td rowspan="3"></td></tr>
<tr><td>February</td><td>0.83</td><td rowspan="2">0.86</td><td rowspan="4">0.87</td></tr>
<tr><td>March</td><td>0.82</td><td rowspan="2">0.83</td><td rowspan="4">0.83</td></tr>
<tr><td>April</td><td>0.75</td><td rowspan="2">0.75</td><td rowspan="4">0.76</td></tr>
<tr><td>May</td><td>0.69</td><td rowspan="2">0.71</td><td rowspan="4">0.73</td></tr>
<tr><td>June</td><td>0.66</td><td rowspan="2">0.67</td><td rowspan="4">0.83</td></tr>
<tr><td>July</td><td>0.62</td><td rowspan="2">0.71</td><td rowspan="4">0.86</td></tr>
<tr><td>August</td><td>0.71</td><td rowspan="2">0.82</td><td rowspan="4">0.9</td></tr>
<tr><td>September</td><td>0.82</td><td rowspan="2">0.85</td><td rowspan="4">0.96</td></tr>
<tr><td>October</td><td>0.82</td><td rowspan="2">0.89</td><td rowspan="4">1.00</td></tr>
<tr><td>November</td><td>0.88</td><td rowspan="2">0.95</td><td rowspan="4">1.00</td></tr>
<tr><td>December</td><td>0.94</td><td rowspan="2">1.00</td><td rowspan="4">1.00</td></tr>
<tr><td>January</td><td>0.98</td><td rowspan="2">0.98</td><td rowspan="3"></td></tr>
<tr><td>February</td><td>0.83</td><td rowspan="2">0.86</td><td rowspan="2"></td></tr>
<tr><td>March</td><td>0.82</td><td></td><td></td></tr>
<tr><td colspan="8">NOTE 1 The factor for the six-month winter period October to March inclusive is 1.00 and for the six-month summer period April to September inclusive is 0.84.
NOTE 2 These factors provide the 0.02 probability of exceeding the period given.
NOTE 3 For transportable structures which may be used at any time in the year c_{season} should be taken as 1.0.</td></tr>
</table>

Stage 5: Basic velocity pressure q_b

Definition The dynamic pressure caused by the basic wind velocity.

EC1-1-4 Eqn. (4.10)

$$q_b = 0.613 v_b^2 \quad \text{(W. 5.1)}$$

v_b basic wind velocity, as calculated in Stage 4

Source reference

Stage 6: Exposure factor $c_e(z - h_{dis})$

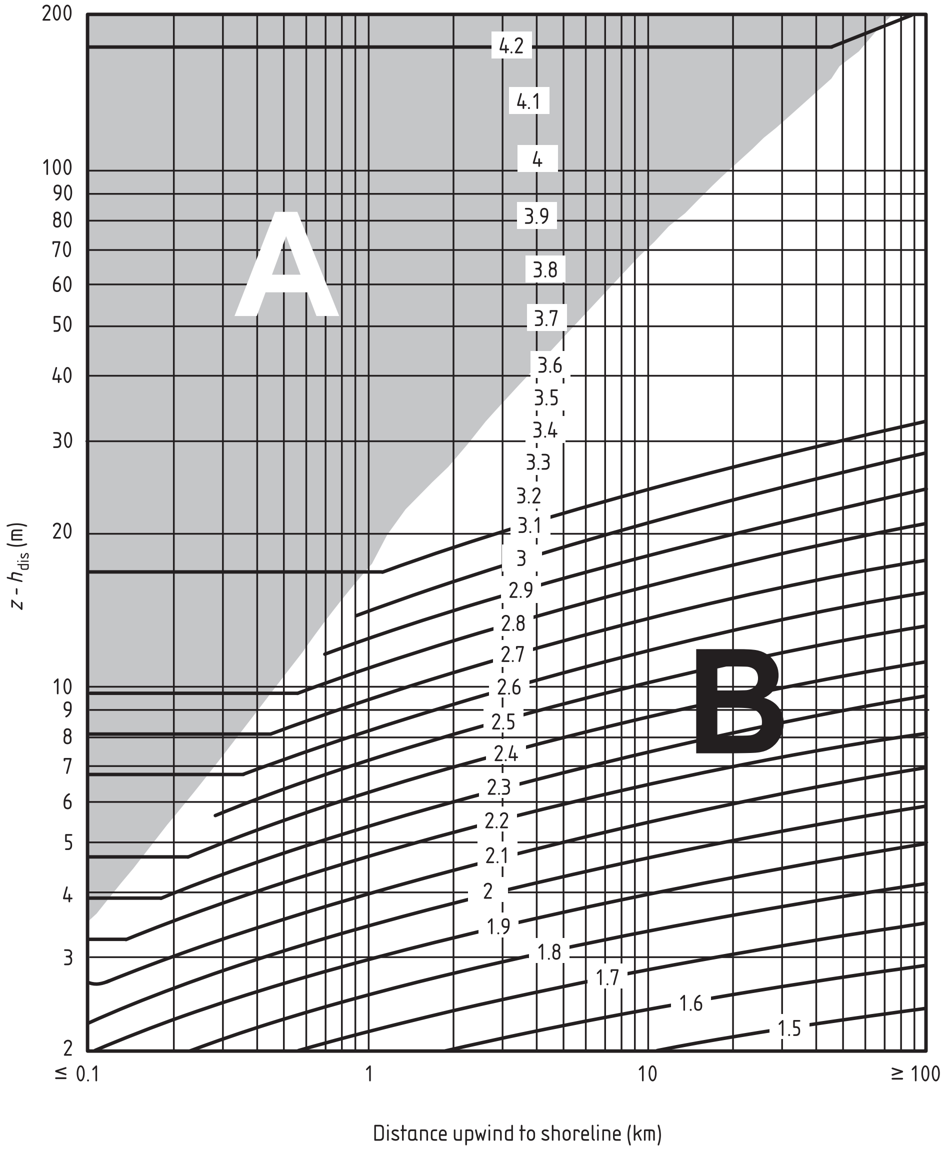
Definition The factor which accounts for the exposure of the building in relation to the distance upwind to the sea.

Using Figure W. 6.1 determine the exposure factor and note in which area (A or B) the result lies. For urban terrain you will later use Figure W. 7.2 to decide additionally whether it fits a further category, C.

Required values z-h_{dis} height above ground
Distance upwind to shore line (km)

EC1-1-4 Figure NA.7

Figure W. 6.1: Values of $c_e(z - h_{dis})$



Source reference

Stage 7: Is the site in town terrain?

Figure W. 7.1: Terrain types

Town terrain

(a) (b)

Country terrain

(c) (d)

Source reference

Stage 7y: Exposure correction factor $c_{e,T}$ for sites in town terrain

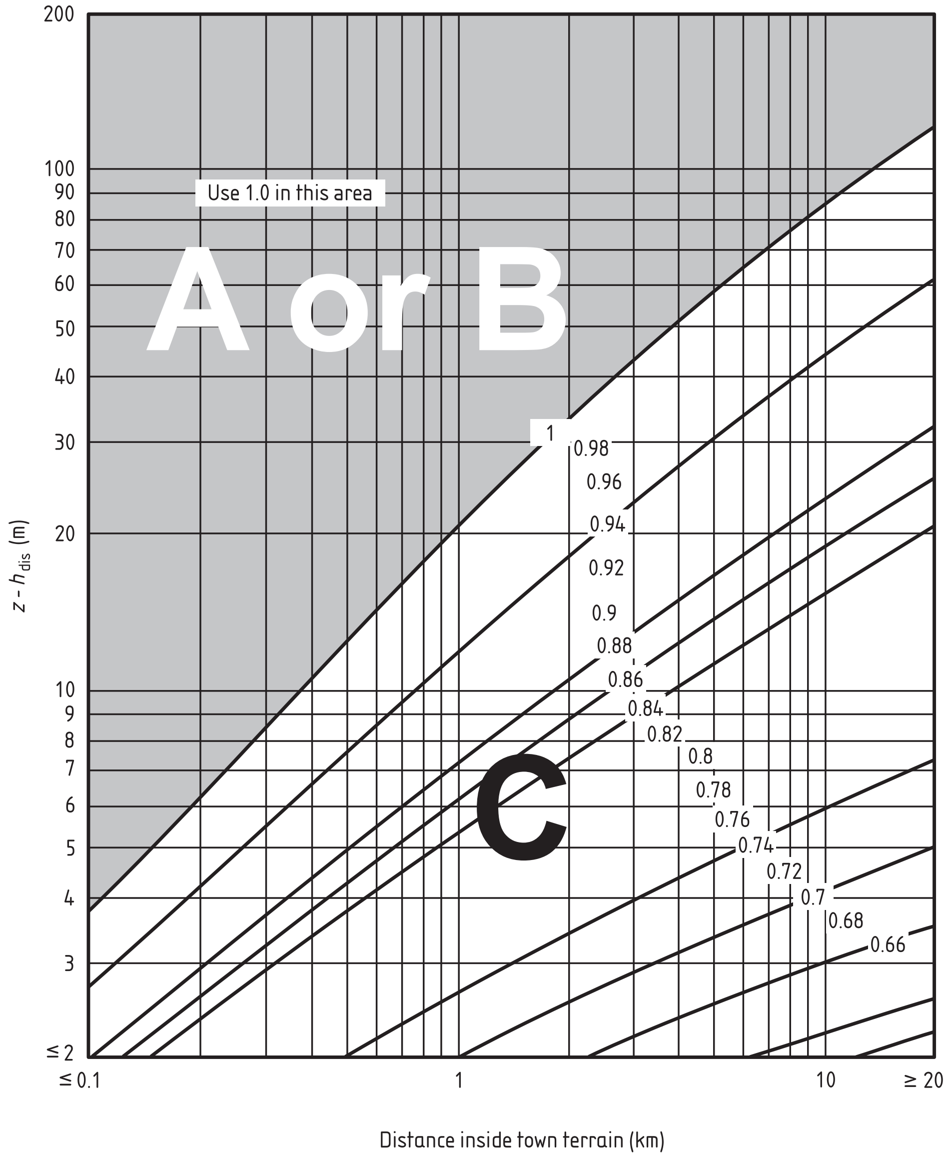
Using Figure W. 7.2, determine the exposure correction factor.

Required values: z-h_{dis} height above ground
Distance inside town terrain (km)

Also note in which area the result lies – A, B or C.

EC1-1-4 Figure NA.8

Figure W. 7.2: Values of exposure correction factor $c_{e,T}$ for sites in town terrain



Source reference

Stage 8: Orography factor $c_o(z - h_{dis})$

Definition The factor which takes into account the increased wind speed that occurs at the top of a slope (although this method cannot be used for mountainous regions).

Different expressions for $c_o(z)$ are given as:

EC1-1-4 (A.1) $c_o(z - h_{dis}) = 1$ for $\phi < 0.05$ (i.e. orography not significant) (W. 8.1a)

EC1-1-4 (A.2) $c_o(z - h_{dis}) = 1 + 2s$ for $0.05 < \phi < 0.3$ (W. 8.1b)

EC1-1-4 (A.3) $c_o(z - h_{dis}) = 1 + 0.6s\phi$ for $\phi > 0.3$ (W. 8.1c)

z vertical distance from the ground level of the site

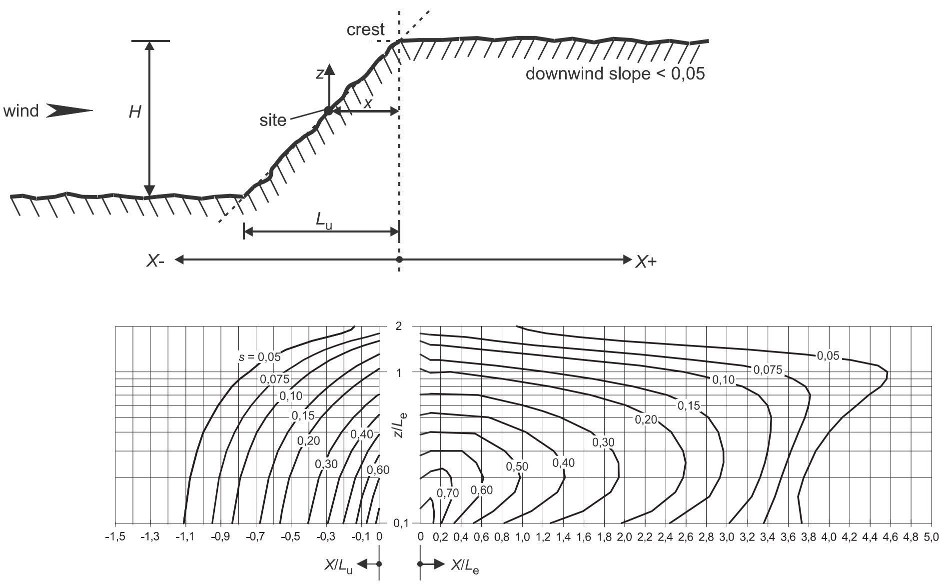
s orographic location factor scaled to the effective length of the upwind slope (see Figures W. 8.1 or W. 8.2).

ϕ the upwind slope H/L_u in the wind direction (see Figures W. 8.1 or W. 8.2)

h_{dis} the displacement height, calculated in Stage 3.

EC1-1-4 Figure A.2

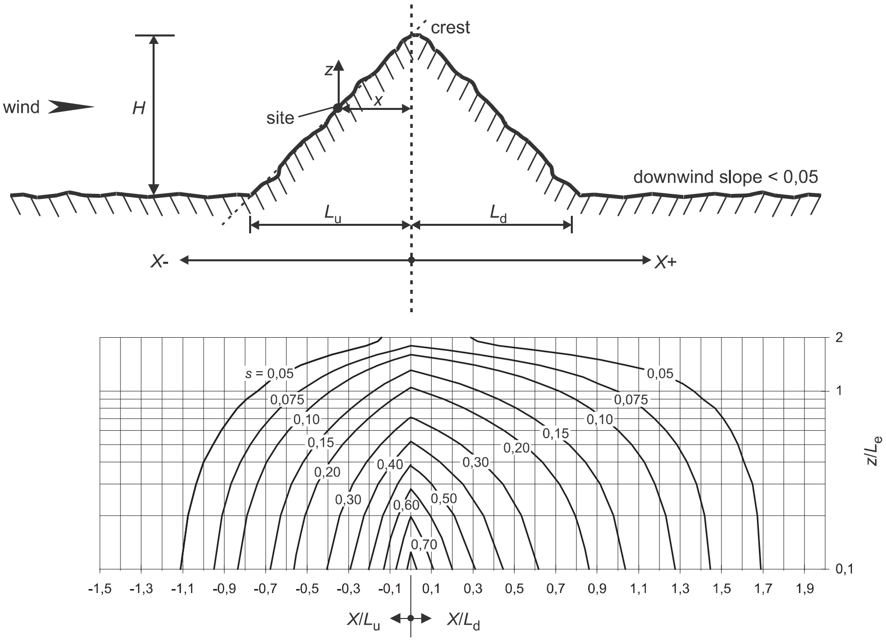
Figure W. 8.1: Factor *s* for cliffs and escarpments



Source reference

EC1-1-4 Figure A.3

Figure W. 8.2: Factor s for hills and ridges



Stage 9: Peak velocity pressure $q_p(z - h_{dis})$

Definition This is the main factor used to determine the wind pressures and forces. It is the value which specifies the wind which is being considered and accounts for the mean wind velocity and also a fluctuating wind speed.

When a site is in country terrain

Orography is significant (and z<50m)

EC1-1-4 Eqn(NA.4a)

$$q_p(z - h_{dis}) = c_e(z - h_{dis}).q_b.((c_o(z - h_{dis}) + 0.6) / 1.6)^2 \qquad \text{(W. 9.1a)}$$

Orography is not significant

EC1-1-4 Eqn(NA.3a)

$$q_p(z - h_{dis}) = c_e(z - h_{dis}).q_b \qquad \text{(W. 9.1b)}$$

When a site is in town terrain

Orography is significant (and z<50m)

EC1-1-4 Eqn(NA.4a)

$$q_p(z - h_{dis}) = c_e(z - h_{dis}).c_{e,T}.q_b.((c_o(z - h_{dis}) + 0.6) / 1.6)^2 \qquad \text{(W. 9.1c)}$$

Orography is not significant

EC1-1-4 Eqn(NA.3b)

$$q_p(z - h_{dis}) = c_e(z - h_{dis}).c_{e,T}.q_b \qquad \text{(W. 9.1d)}$$

Source reference

For the most accurate result the peak velocity pressure should be calculated for each of 12 wind directions at 30° intervals. It may, however, be obvious that there is a single worst case which can then be used for all directions. Alternatively, four orthogonal directions normal to the main axes of the building can be used.

Considerably higher neighbouring structures

If a neighbouring building is more than twice the average height of the surrounding structures, as a first approximation the design of any of those nearby structures may be based on the peak velocity pressure at height z_n ($z_e = z_n$) above ground.

EC1-1-4 Eqn(A.14)

$x \le r$	$z_n = 0.5r$	(W. 9.2a)
$r < x < 2r$	$z_n = 0.5(r - (1 - 2h_{low}/r)\,(x - r)$	(W. 9.2b)
$x \ge r$	$z_n = h_{low}$	(W. 9.2c)

where the radius r is:

$r = h_{high}$	if	$h_{high} \le 2d_{large}$
$r = 2d_{large}$	if	$h_{high} > 2d_{large}$

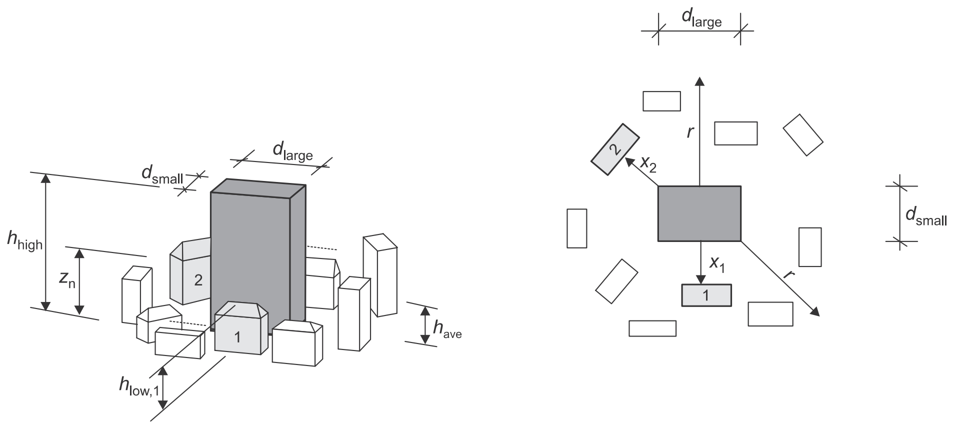
Refer to Figure W. 9.1:

h_{low}	structural height
r	radius
x	distance
d_{small}	dimension
d_{large}	dimension

Increased wind velocities can be disregarded when h_{low} is more than half the height h_{high} of the high building (i.e. $z_n = h_{low}$).

EC1-1-4 Figure A.4

Figure W. 9.1: Influence of a high-rise building on two different nearby structures (1 and 2)



Source reference

Stage 10: Structural factor $c_s c_d$

Definition The structural factor has two parts: the **size factor** c_s, and the **dynamic factor** c_d.

c_s accounts for the lack of correlation of the wind gusts over the surfaces of the structure and is always ≤ 1. The larger the structure or element, the smaller the c_s value.

c_d accounts for the dynamic response of the structure in its fundamental mode of vibration and is always ≥ 1. The more dynamically sensitive the structure, the larger the c_d value.

Determination of separate values of c_s and c_d

EC1-1-4 6.2

In the following cases the structural factor $c_s c_d$ may be taken as 1.0:

- buildings less than 15 m in height;
- cladding panels and elements;
- framed buildings less than 100 m tall which have structural walls and the height of which is less than four times the in-wind depth;
- for facade and roof elements having a natural frequency greater than 5 Hz.

For buildings which fall outside these cases the following *method* for calculating c_s and c_d separately should be used.

The size factor c_s

First determine whether structure is in zone A, B or C.

Sites in country terrain

Zones determined and using Figure W. 6.1 with respect to distance from shore (z-h_{dis}).

Sites in town terrain

It is first determined whether zone C applies using Figure W. 7.2 with respect to distance into town and (z-h_{dis}). If not, zone A or B will apply depending on the distance of the site from shore and (z-h_{dis}) as shown in Figure W. 6.1.

The size factor c_s can then be obtained from Table W. 10.1.

This method can be used for part of a building as well as for the whole structure; it can also be used for individual roofing or cladding elements.

Source reference

EC1-1-4 Table NA.3

Table W. 10.1: Size factor c_s for zones A, B and C

h+b m	z-h_{dis} = 6m			z-h_{dis} = 10m			z-h_{dis} = 30m			z-h_{dis} = 50m			z-h_{dis} = 200m		
	A	**B**	**C**	**A**	**B**	**C**	**A**	**B**	**C**	**A**	**B**	**C**	**A**	**B**	**C**
1	0.99	0.98	0.97	0.99	0.99	0.97	0.99	0.99	0.98	0.99	0.99	0.99	0.99	0.99	0.99
5	0.96	0.96	0.92	0.97	0.96	0.93	0.98	0.97	0.95	0.98	0.98	0.96	0.98	0.98	0.98
10	0.95	0.94	0.88	0.95	0.95	0.90	0.96	0.96	0.93	0.97	0.96	0.94	0.98	0.97	0.97
20	0.93	0.91	0.84	0.93	0.92	0.87	0.95	0.94	0.90	0.95	0.95	0.92	0.96	0.96	0.95
30	0.91	0.89	0.81	0.92	0.91	0.84	0.94	0.93	0.88	0.94	0.93	0.90	0.96	0.95	0.93
40	0.90	0.88	0.79	0.91	0.89	0.82	0.93	0.91	0.86	0.93	0.92	0.88	0.95	0.94	0.92
50	0.89	0.86	0.77	0.90	0.88	0.80	0.92	0.90	0.85	0.92	0.91	0.87	0.94	0.94	0.91
70	0.87	0.84	0.74	0.88	0.86	0.77	0.90	0.89	0.83	0.91	0.90	0.85	0.93	0.92	0.90
100	0.85	0.82	0.71	0.86	0.84	0.74	0.89	0.87	0.80	0.90	0.88	0.82	0.92	0.91	0.88
150	0.83	0.80	0.67	0.84	0.82	0.71	0.87	0.85	0.77	0.88	0.86	0.79	0.90	0.89	0.85
200	0.81	0.78	0.65	0.83	0.80	0.69	0.85	0.83	0.74	0.86	0.84	0.77	0.89	0.88	0.83
300	0.79	0.75	0.62	0.80	0.77	0.65	0.83	0.80	0.71	0.84	0.82	0.73	0.87	0.85	0.80

Interpolation may be used.

b cross wind breadth of building or building part or width of element

h height of building or building part or length of element

z height of building or height to top of element

Source reference

The dynamic factor c_d

In the following cases the dynamic factor c_d may be taken as 1.0, as the buildings are assumed to respond statically:

- framed buildings with internal masonry walls as well as structural walls around lifts and stairwells;
- masonry constructed buildings;
- timber-framed housing;
- cladding panels and elements.

For buildings which fall outside these cases the following method for calculating c_d should be used.

EC1-1-4 NA.2.20

The dynamic factor c_d accounts for the effect of fluctuating wind loads in combination with the resonance of the structure. The simplified approach given in Figure W. 10.1 has been derived for typical buildings with typical damping and natural frequency characteristics. More accurate values are given using the procedure in BS EN 1991-1-4:2005 6.3.

First use Table W. 10.2 to find the value of logarithmic decrement of structural damping δ_s.

The dynamic factor c_d can then be obtained from Figure W. 10.1.

EC1-1-4 Table F.2

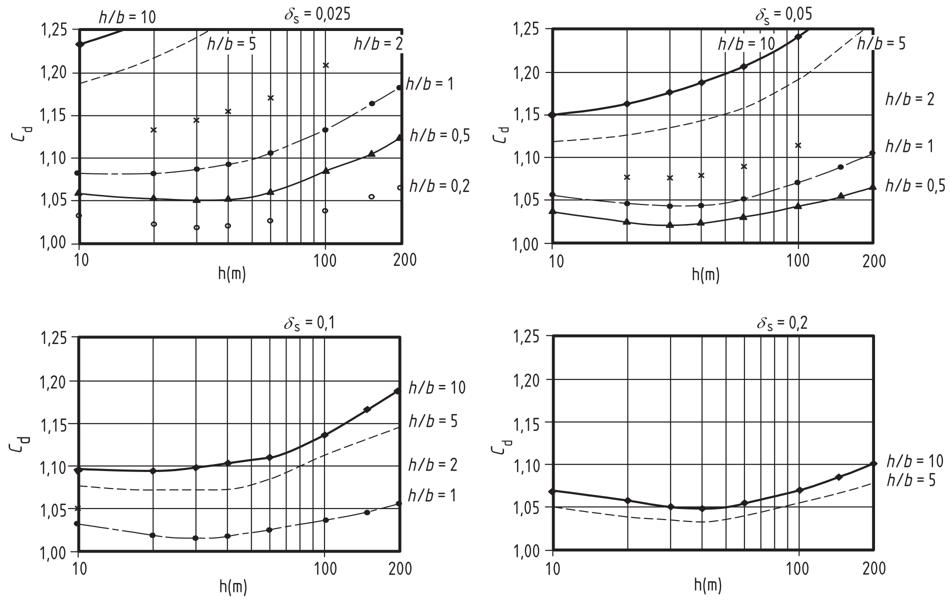
Table W. 10.2: Approximate values of logarithmic decrement δ_s of structural damping in the fundamental mode

Structural type	Structural damping δ_s
Reinforced concrete buildings	0.10
Steel buildings	0.05
Mixed structures concrete + steel	0.08

Source reference

EC1-1-4 Figure NA.9

Figure W. 10.1: Dynamic factor c_d for various values of logarithmic decrement of structural damping δ_s



NOTE Values are based on basic wind velocity, v_b = 26 m/s, the reference height for the external pressure $z_e = 0.6h$ and natural frequency of the structure of the mode 1 multiplied by height of structure $n_1h = 46$. Values of c_d do not alter significantly with other wind speeds.

Stage 11: Pressure coefficients for buildings

Introduction to pressure and force coefficients

General

Pressure and force coefficients define the aerodynamic effects of wind loads on a structure.

Pressure coefficients deal with the effect which the form of the structure has on the internal and external pressure distribution.

Force coefficients integrate the effect of the surface pressure distribution on the overall loads.

Source reference

The different coefficients and when to use them

Depending on the form of the structure, the following coefficients should be used:

1) Internal and external pressure coefficients

These coefficients give the normal wind pressures over both external and internal surfaces. They are used when the distribution of loads on surfaces is needed (e.g. for cladding loads).

The external pressure coefficients are divided into overall coefficients and local coefficients. Local coefficients give the pressure coefficients for loaded areas of 1 m^2. They may be used for the design of small elements and fixings. Overall coefficients give the pressure coefficients for loaded areas of 10 m^2. They may be used for loaded areas larger than 10 m^2.

- buildings
- cladding elements

EC1-1-3 NA 1.27

2) Net pressure coefficients

EC1-1-3 Table NA.4

These coefficients relate to the net normal wind pressure across a surface and are used when a structure has no 'inside' (e.g. for canopies and boundary walls). They give the joint result of pressures acting on both sides of the element being considered.

- canopy roofs
- free-standing walls, parapets and fences
- rectangular plan buildings

3) Friction coefficients

These coefficients relate to tangential wind pressures in the direction of the wind caused by friction or by the accumulated normal pressures on protrusions on the surface (e.g. corrugations or ribs).

- wall surfaces
- roof surfaces

Source reference

4) Force coefficients

These coefficients relate to loads on individual elements or overall loads on structures, where the spatial distribution need not be defined. They include both normal and friction pressures.

- structural elements with rectangular cross section
- structural elements with sharp edged section
- structural elements with regular polygonal section
- lattice structures and scaffoldings

Asymmetric and counteracting pressures and forces

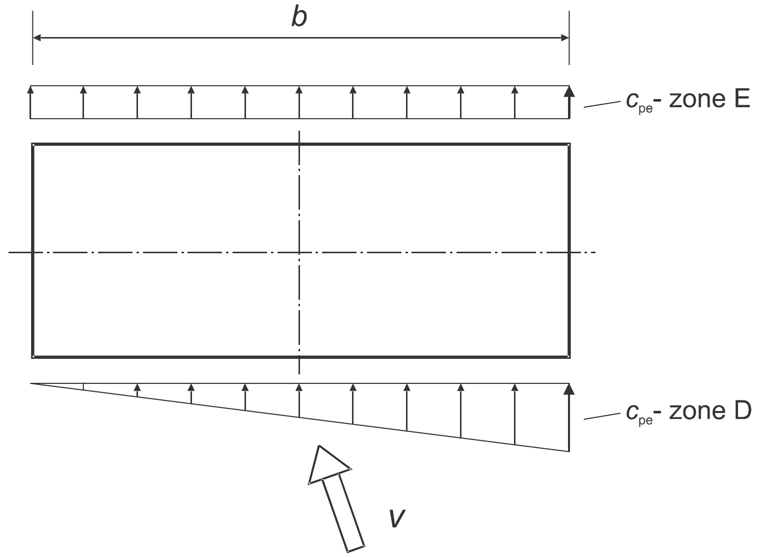
Wind loading patterns are simplified in the design codes and therefore the effect of asymmetric loading is not really considered. To account for this, if instantaneous fluctuations of wind over surfaces can give rise to significant asymmetry of loading and the structural form is likely to be sensitive to such loading (e.g. torsion in nominally symmetric single core buildings) then their effect should be taken into account.

For rectangular buildings Figure W.11.1 should be used to account for torsion.

For signboards and free-standing canopies, BS EN 1991-1-4 7.3 and 7.4 should be used.

EC1-1-4 Figure NA10
EC1-1-4 Figure 7.5
EC1-1-4 Table 7.1

Figure W. 11.1: Pressure distribution used to take torsional effects into account



Source reference

Effects of ice and snow

Pressure and force coefficients are highly dependent on the shape of the structure. If the accumulation of snow on a building or element could cause a change in shape of the structure (e.g. a lattice structure could effectively become solid by the lattice becoming blocked by snow). For guidance on lattice structures see EC1 Part 3; for other susceptible structures, seek specialist guidance.

Pressure coefficients for buildings

If the walls and/or roof of the building have **more than one skin**, see Stage 13.

External pressure coefficients c_{pe}

There are two values for external pressure: $c_{pe,1}$ and $c_{pe,10}$.

$c_{pe,1}$ values should be used for all loaded areas 1m^2 or less (e.g. for small cladding elements and fixings).

$c_{pe,10}$ should be used for all loaded areas >1m^2 (e.g. larger cladding elements and overall structural loads).

Calculate loaded area *A* in m^2

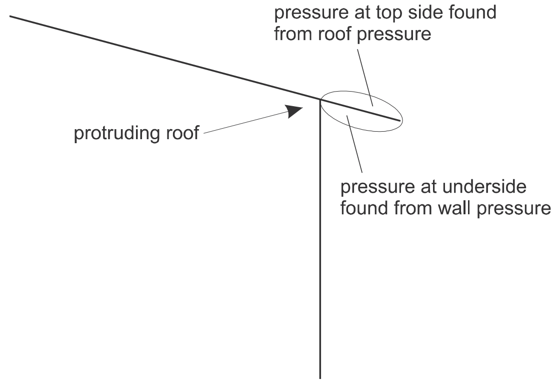
$A > 10 \quad c_{pe} = c_{pe,10}$

$A < 1 \quad c_{pe} = c_{pe,1}$

c_{pe} values are given for orthogonal wind directions 0°, 90° and 180°, which cover the worst case wind directions at ± 45° about each orthogonal axis.

EC1-1-4 7.2.1(3) For protruding roof corners the pressure on the underside of the roof overhang is equal to the pressure for the zone of the vertical wall directly connected to the protruding roof; the pressure at the top side of the roof overhang is equal to the pressure of the zone defined for the roof.

EC1-1-4 Figure 7.3 **Figure W. 11.2: Illustration of relevant pressures for protruding roofs**



Source reference

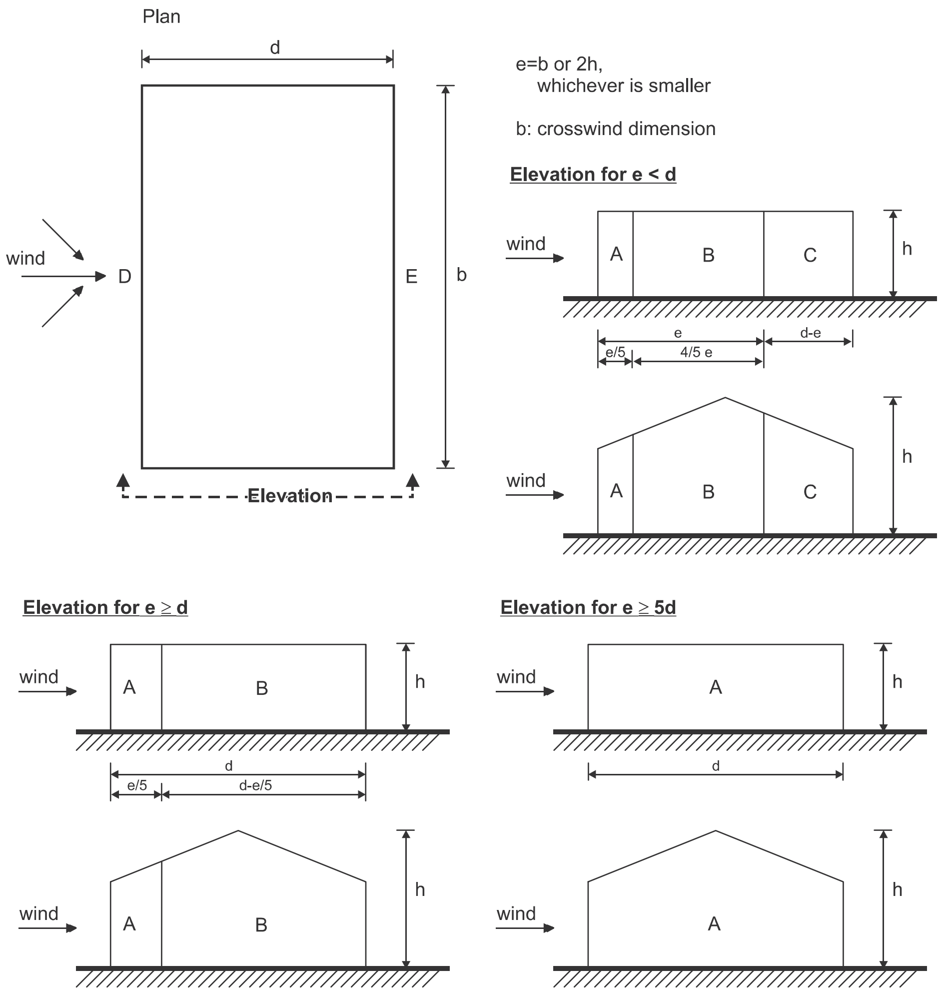
Stage 11a: External pressure coefficients for walls

Vertical walls of rectangular plan buildings

1. The reference height z_e is always taken from the top of the wall or wall part; see Figure W. 11.2.
2. The wall or wall part should be divided into areas A, B, C, D and E, as shown in Figure W. 11.3.
3. The external pressure coefficients $c_{pe,10}$ and $c_{pe,1}$ for each area are given in Table W. 11.1.
4. For the determination of overall loads on buildings, the net pressure coefficients in Table W. 11.2 may be used instead of the sum of the pressure coefficients for zones D and E. (The factor for accounting for lack of correlation between the front and rear faces may also be applied to the net pressure coefficients.)

EC1-1-4 Figure 7.5

Figure W. 11.3: Key for vertical walls



Source reference

EC1-1-4 Table 7.1

Table W. 11.1: Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

Zone	A		B		C		D		E	
h/*d*	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	–1.2	–1.4	–0.8	–1.1	–0.5		+0.8	+1.0	–0.7	
1	–1.2	–1.4	–0.8	–1.1	–0.5		+0.8	+1.0	–0.5	
≤ 0.25	–1.2	–1.4	–0.8	–1.1	–0.5		+0.7	+1.0	–0.3	
NOTE Linear interpolation may be used for intermediate values of *h*/*d*.										

EC1-1-4 Table NA.4

Table W. 11.2: Net pressure coefficients for vertical walls of rectangular buildings

h/*d*	Net pressure coefficient $c_{pe,10}$
5	1.3
1.0	1.1
≥ 0.25	0.8

NOTE 1 The coefficients may also be applied to non-vertical walls within ±15° of vertical.

NOTE 2 Where the walls of two buildings face each other and the gap between them is less than e (smaller value of e in case of buildings with different e values), 'funnelling' will accelerate the flow and make the pressure coefficients in zones A, B and C more negative than in the case where the building is 'isolated', according to the following:

Where the gap between the buildings is < *e* /4 or > *e*, the coefficient for isolated case should be used.

Where the gap between the buildings is > *e* /4 and < *e*, either use the funnelling values conservatively, or interpolate linearly according to the actual gap between the following values: the funnelling values to apply for a gap of *e* /2 and the isolated values to apply for a gap of *e* /4 and a gap of *e*;

Where the two buildings are sheltered by upwind buildings such that ($hr - h_{dis}$) < 0.4*hr* for the lower of the two buildings, then funnelling can be disregarded.

NOTE 3 The external pressure coefficients for side faces affected by funnelling should be taken as *p* 1.6 for zone A, *p* 0.9 for zone B and *p* 0.9 for zone C.

Stage 11b: External pressure coefficients for roofs

What category of roof does the building have?

Flat roof	Section 11b.1
Mono-pitch	Section 11b.2
Duo-pitch	Section 11b.3
Hipped	Section 11b.4
Multi-span	Section 11b.5
Vaulted or domed	Section 11b.6

NOTE If the roof has **more than one skin**, see Stage 13.

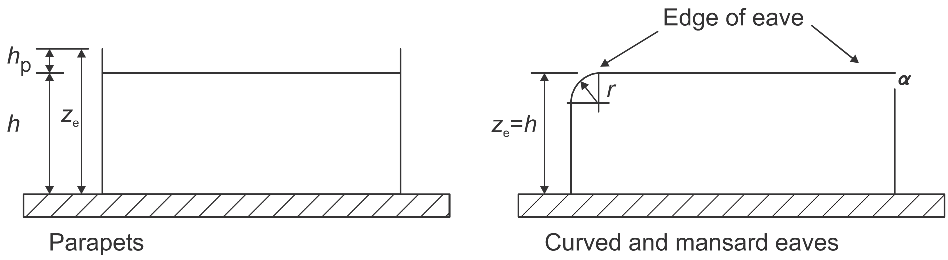
Source reference

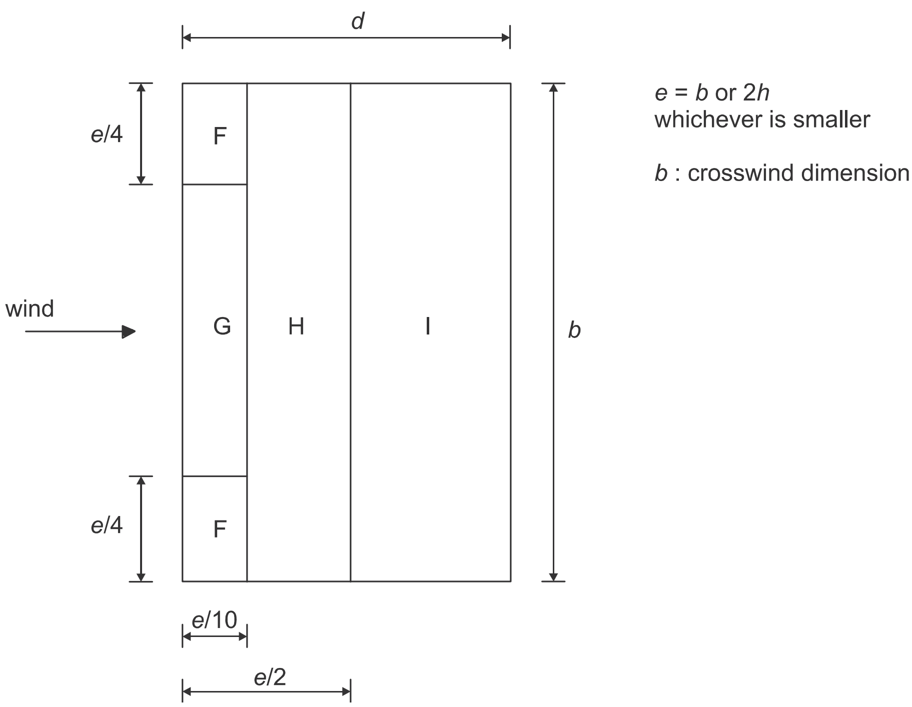
11b.1 Flat roofs

1. Flat roofs are defined by having a slope (α) of $-5° < \alpha < 5°$
2. The reference height z_e should be taken as h for flat roofs and roofs with curved or mansard eaves.
   The reference height z_e should be taken as $(h + h_p)$ for flat roofs with parapets, as shown in Figure W. 11.4.
3. The roof should be divided into areas, as shown in Figure W. 11.4. The pressure coefficients for each area are shown in Table W. 11.3.
4. The resulting pressure on a parapet should be determined using BS EN 1991-1-4 Section 7.4.

EC1-1-4 Figure 7.6

Figure W. 11.4: Key for flat roofs





Source reference

EC1-1-4 Table 7.2

Table W. 11.3: External pressure coefficients for flat roofs

Roof type		Zone F		Zone G		Zone H		Zone I	
		$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
Sharp eaves		–1.8	–2.5	–1.2	–2.0	–0.7	–1.2	+0.2	
								–0.2	
With parapets	$h_p/h = 0.025$	–1.6	–2.2	–1.1	–1.8	–0.7	–1.2	+0.2	
								–0.2	
	$h_p/h = 0.05$	–1.4	–2.0	–0.9	–1.6	–0.7	–1.2	+0.2	
								–0.2	
	$h_p/h = 0.10$	–1.2	–1.8	–0.8	–1.4	–0.7	–1.2	+0.2	
								–0.2	
Curved eaves	$h_p/h = 0.05$	–1.0	–1.5	–1.2	–1.8	–0.4		+0.2	
								–0.2	
	$h_p/h = 0.10$	–0.7	–1.2	–0.8	–1.4	–0.3		+0.2	
								–0.2	
	$h_p/h = 0.20$	–0.5	–0.8	–0.5	–0.8	–0.3		+0.2	
								–0.2	
Mansard eaves	$\alpha = 30°$	–1.0	–1.5	–1.0	–1.5	–0.3		+0.2	
								–0.2	
	$\alpha = 45°$	–1.2	–1.8	–1.3	–1.9	–0.4		+0.2	
								–0.2	
	$\alpha = 60°$	–1.3	–1.9	–1.3	–1.9	–0.5		+0.2	
								–0.2	

NOTE 1 For roofs with parapets or curved eaves, linear interpolation may be used for intermediate values of h_p/h and r/h.

NOTE 2 For roofs with mansard eaves, linear interpolation between $\alpha = 30°$, 45° and $\alpha = 60°$ may be used. For $\alpha > 60°$ linear interpolation between the values for $\alpha = 60°$ and the values for flat roofs with sharp eaves may be used.

NOTE 3 In zone I, where positive and negative values are given, both values shall be considered, and the more onerous sign should be used.

NOTE 4 For the mansard eave itself, the external pressure coefficients are given in BS EN 1991-1-4 Table 7.4 'External pressure coefficients for duopitch roofs: wind direction 0°, zone F and G, depending on the pitch angle of the mansard eave.

NOTE 5 For the curved eave itself, the external pressure coefficients are given by linear interpolation along the curve, between values on the wall and on the roof.

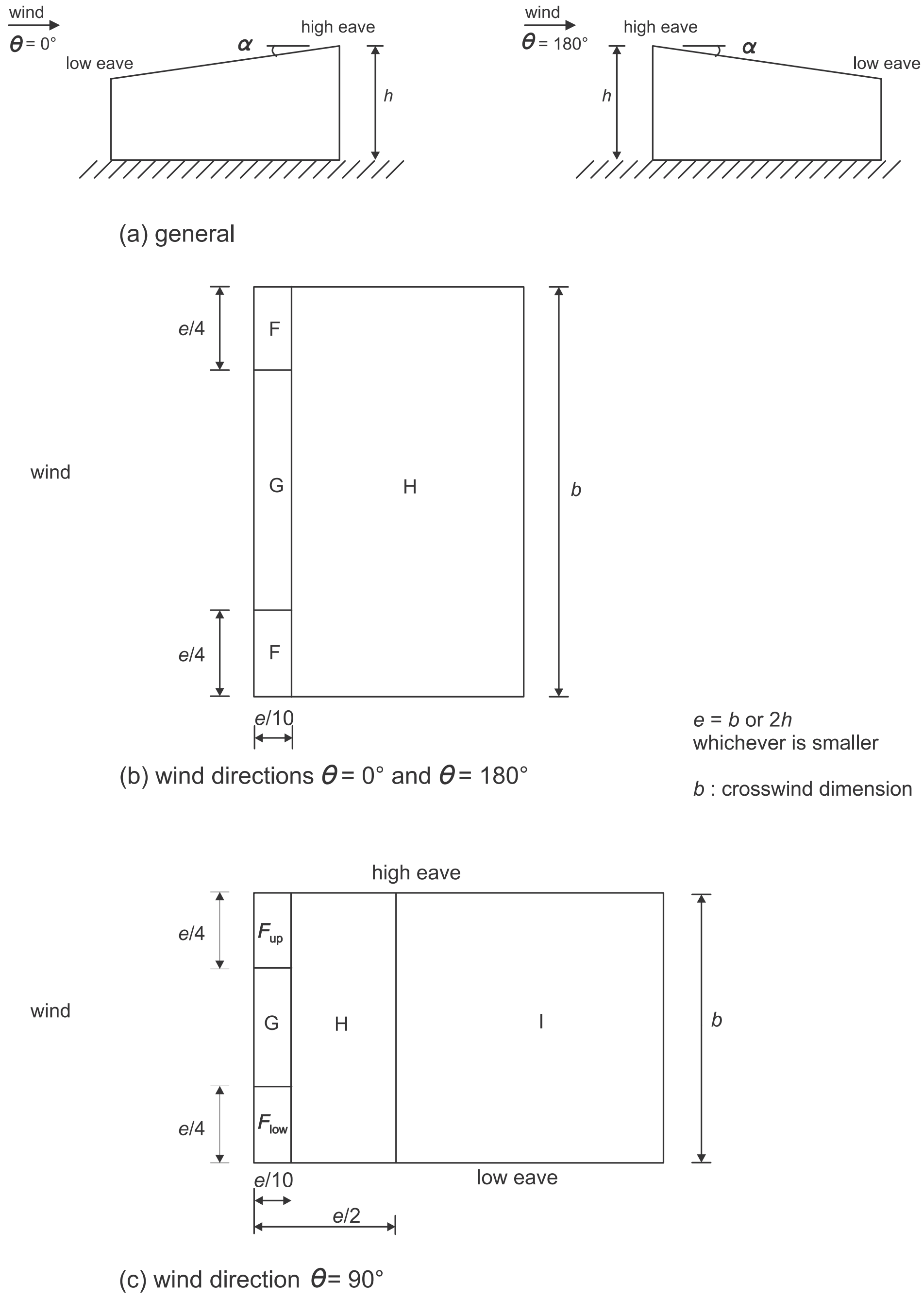
Source reference

11b.2 Mono-pitch roofs

1. Mono-pitch roofs are defined by a single pitch greater than 5°.
2. The reference height z_e should be taken as h.
3. The roof should be divided into areas, as shown in Figure W. 11.5. The pressure coefficients for each area are shown in Table W. 11.4.

EC1-1-4 Figure 7.7

Figure W. 11.5: Key for mono-pitch roofs



Source reference

EC1-1-4 Table 7.3

Table W. 11.4: External pressure coefficients for mono-pitch roofs

Pitch angle α	Zone for wind direction θ = 0°						Zone for wind direction θ = 180°					
	F		**G**		**H**		**F**		**G**		**H**	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	–1.7	–2.5	–1.2	–2.0	–0.6	–1.2	–2.3	–2.5	–1.3	–2.0	–0.8	–1.2
	+0.0		+0.0		+0.0							
15°	–0.9	–2.0	–0.8	–1.5	–0.3		–2.5	–2.8	–1.3	–2.0	–0.9	–1.2
	+0.2		+0.2		+0.2							
30°	–0.5	–1.5	–0.5	–1.5	–0.2		–1.1	–2.3	–0.8	–1.5	–0.8	
	+0.7		+0.7		+0.4							
45°	–0.0		–0.0		–0.0		–0.6	–1.3	–0.5		–0.7	
	+0.7		+0.7		+0.6							
60°	+0.7		+0.7		+0.7		–0.5	–1.0	–0.5		–0.5	
75°	+0.8		+0.8		+0.8		0.5	–1.0	–0.5		–0.5	

Pitch angle α	Zone for wind direction θ = 90°									
	F_{up}		**F_{low}**		**G**		**H**		**I**	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	–2.1	–2.6	–2.1	–2.4	–1.8	–2.0	–0.6	–1.2	–0.5	
15°	–2.4	–2.9	–1.6	–2.4	–1.9	–2.5	–0.8	–1.2	–0.7	–1.2
30°	–2.1	–2.9	–1.3	–2.0	–1.5	–2.0	–1.0	–1.3	–0.8	–1.2
45°	–1.5	–2.4	–1.3	–2.0	–1.4	–2.0	–1.0	–1.3	–0.9	–1.2
60°	–1.2	–2.0	–1.2	–2.0	–1.2	–2.0	–1.0	–1.3	–0.7	–1.2
75°	–1.2	–2.0	–1.2	–2.0	–1.2	–2.0	–1.0	–1.3	–0.5	

NOTE 1 At θ = 0° (see Table a) the pressure changes rapidly between positive and negative values around a pitch angle of α = +5° to +45°, so both positive and negative values are given. For those roofs, two cases should be considered: one with all positive values, and one with all negative values. No mixing of positive and negative values is allowed on the same face.

NOTE 2 Linear interpolation for intermediate pitch angles may be used between values of the same sign. The values equal to 0.0 are given for interpolation purposes.

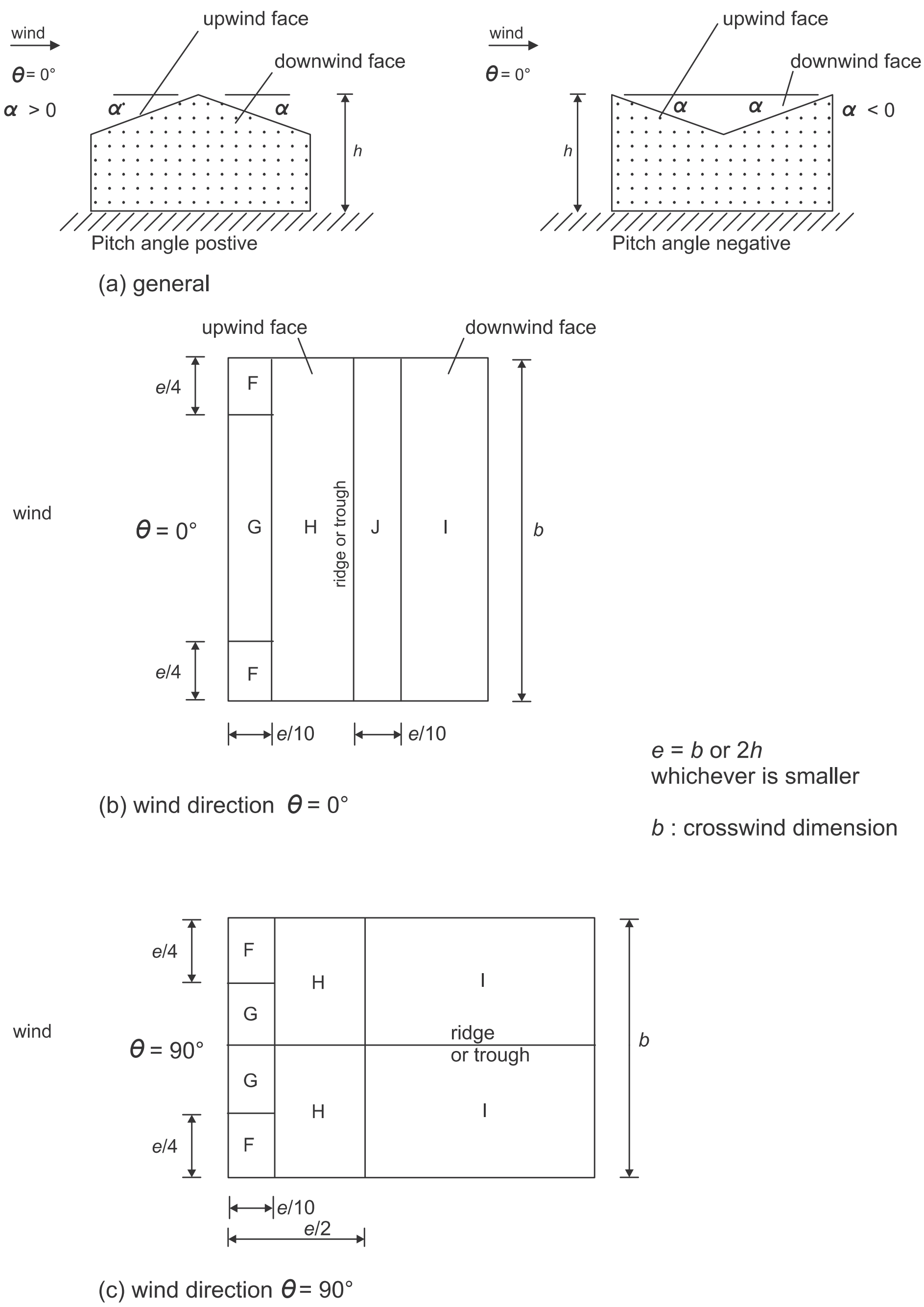
Source reference

11b.3 Duo-pitch roofs

1. Duo-pitch roofs are defined by a double pitch, whether peaked or troughed, with a pitch angle greater than 5°.
2. Where the pitches are not of equal slope, but are reasonably similar, the pitch of the upwind slope should be used.
3. The reference height z_e should be taken as h.
4. The roof should be divided into areas, as shown in Figure W. 11.6. The pressure coefficients for each area are shown in Table W. 11.5.

EC1-1-4 Figure 7.8

Figure W. 11.6: Key for duo-pitch roofs



Source reference

EC1-1-4 Table 7.4a

Table W. 11.5a: External pressure coefficients for duo-pitch roofs

Pitch angle α	Zone for wind direction θ = 0°									
	F		G		H		I		J	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
–45°	–0.6		–0.6		–0.8		–0.7		–1.0	–1.5
–30°	–1.1	–2.0	–0.8	–1.5	–0.8		–0.6		–0.8	–1.4
–15°	–2.5	–2.8	–1.3	–2.0	–0.9	–1.2	–0.5		–0.7	–1.2
–5°	–2.3	–2.5	–1.2	–2.0	–0.8	–1.2	+0.2		+0.2	
							–0.6		–0.6	
5°	–1.7	–2.5	–1.2	–2.0	–0.6	–1.2	–0.6		+0.2	
	+0.0		+0.0		+0.0				–0.6	
15°	–0.9	–2.0	–0.8	–1.5	–0.3		–0.4		–1.0	–1.5
	+0.2		+0.2		+0.2		+0.0		+0.0	+0.0
30°	–0.5	–1.5	–0.5	–1.5	–0.2		–0.4		–0.5	
	+0.7		+0.7		+0.4		+0.0		+0.0	
45°	–0.0		–0.0		–0.0		–0.2		–0.3	
	+0.7		+0.7		+0.6		+0.0		+0.0	
60°	+0.7		+0.7		+0.7		–0.2		–0.3	
75°	+0.8		+0.8		+0.8		–0.2		–0.3	

NOTE 1 At θ = 0° the pressure changes rapidly between positive and negative values on the windward face around a pitch angle of α = –5° to +45°, so both positive and negative values are given. For those roofs, four cases should be considered where the largest or smallest values of all areas F, G and H are combined with the largest or smallest values in areas I and J. No mixing of positive and negative values is allowed on the same face.

NOTE 2 Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. (Do not interpolate between α = +5° and α = –5°, but use the data for flat roofs in BS EN1991-1-4 Section 7.2.3). The values equal to 0.0 are given for interpolation purposes.

Source reference

EC1-1-4 Table 7.4b

Table W. 11.5b: External pressure coefficients for duo-pitch roofs

Pitch angle α	Zone for wind direction θ = 90°							
	F		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
–45°	–1.4	–2.0	–1.2	–2.0	–1.0	–1.3	–0.9	–1.2
–30°	–1.5	–2.1	–1.2	–2.0	–1.0	–1.3	–0.9	–1.2
–15°	–1.9	–2.5	–1.2	–2.0	–0.8	–1.2	–0.8	–1.2
–5°	–1.8	–2.5	–1.2	–2.0	–0.7	–1.2	–0.6	–1.2
5°	–1.6	–2.2	–1.3	–2.0	–0.7	–1.2	–0.6	
15°	–1.3	–2.0	–1.3	–2.0	–0.6	–1.2	–0.5	
30°	–1.1	–1.5	–1.4	–2.0	–0.8	–1.2	–0.5	
45°	–1.1	–1.5	–1.4	–2.0	–0.9	–1.2	–0.5	
60°	–1.1	–1.5	–1.2	–2.0	–0.8	–1.0	–0.5	
75°	–1.1	–1.5	–1.2	–2.0	–0.8	–1.0	–0.5	

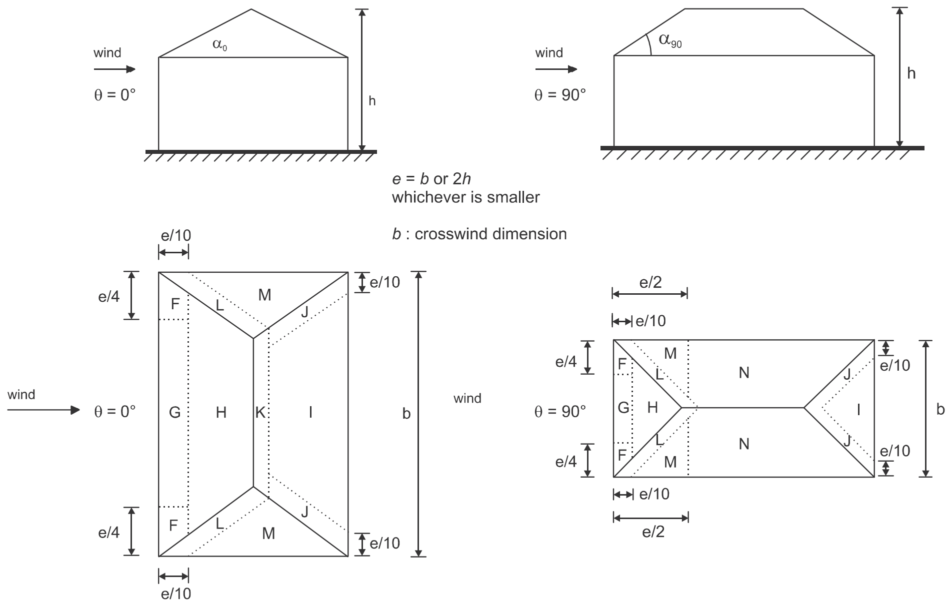
Source reference

11b.4 Hipped roofs

1. All hipped roofs with a pitch angle greater than 5° are covered by this clause, even in cases where the pitches differ between the main and the hip slopes.
2. The pitch of the windward slope should be taken as the datum slope.
3. The reference height z_e should be taken as h.
4. The roof should be divided into areas, as shown in Figure W. 11.7. The pressure coefficients for each area are shown in Table W. 11.6.

EC1-1-4 Figure 7.9

Figure W. 11.7: Key for hipped roofs



(a) wind direction θ = 0°

(b) wind direction θ = 90°

Source reference

EC1-1-4 Table 7.5

Table W. 11.6: External pressure coefficients for hipped roofs

<table>
<tr><th rowspan="2">Pitch angle α_0 for $\theta = 0°$
α_{90} for $\theta = 90°$</th><th colspan="18">Zone for wind direction $\theta = 0°$ and $\theta = 90°$</th></tr>
<tr><th colspan="2">F</th><th colspan="2">G</th><th colspan="2">H</th><th colspan="2">I</th><th colspan="2">J</th><th colspan="2">K</th><th colspan="2">L</th><th colspan="2">M</th><th colspan="2">N</th></tr>
<tr><th></th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th><th>$c_{pe,10}$</th><th>$c_{pe,1}$</th></tr>
<tr><td rowspan="2">5°</td><td>–1.7</td><td>–2.5</td><td>–1.2</td><td>–2.0</td><td>–0.6</td><td>–1.2</td><td rowspan="2" colspan="2">–0.3</td><td rowspan="2" colspan="2">–0.6</td><td rowspan="2" colspan="2">–0.6</td><td rowspan="2">–1.2</td><td rowspan="2">–2.0</td><td rowspan="2">–0.6</td><td rowspan="2">–1.2</td><td rowspan="2" colspan="2">–0.4</td></tr>
<tr><td colspan="2">+0.0</td><td colspan="2">+0.0</td><td colspan="2">+0.0</td></tr>
<tr><td rowspan="2">15°</td><td>–0.9</td><td>–2.0</td><td>–0.8</td><td>–1.5</td><td colspan="2">–0.3</td><td rowspan="2" colspan="2">–0.5</td><td rowspan="2">–1.0</td><td rowspan="2">–1.5</td><td rowspan="2">–1.2</td><td rowspan="2">–2.0</td><td rowspan="2">–1,4</td><td rowspan="2">–2.0</td><td rowspan="2">–0.6</td><td rowspan="2">–1.2</td><td rowspan="2" colspan="2">–0.3</td></tr>
<tr><td colspan="2">+0.2</td><td colspan="2">+0.2</td><td colspan="2">+0.2</td></tr>
<tr><td rowspan="2">30°</td><td>–0.5</td><td>–1.5</td><td>–0.5</td><td>–1.5</td><td colspan="2">–0.2</td><td rowspan="2" colspan="2">–0.4</td><td rowspan="2">–0.7</td><td rowspan="2">–1.2</td><td rowspan="2" colspan="2">–0.5</td><td rowspan="2">–1.4</td><td rowspan="2">–2.0</td><td rowspan="2">–0.8</td><td rowspan="2">–1.2</td><td rowspan="2" colspan="2">–0.2</td></tr>
<tr><td colspan="2">+0.5</td><td colspan="2">+0.7</td><td colspan="2">+0.4</td></tr>
<tr><td rowspan="2">45°</td><td colspan="2">–0.0</td><td colspan="2">–0.0</td><td colspan="2">–0.0</td><td rowspan="2" colspan="2">–0.3</td><td rowspan="2" colspan="2">–0.6</td><td rowspan="2" colspan="2">–0.3</td><td rowspan="2">–1.3</td><td rowspan="2">–2.0</td><td rowspan="2">–0.8</td><td rowspan="2">–1.2</td><td rowspan="2" colspan="2">–0.2</td></tr>
<tr><td colspan="2">+0.7</td><td colspan="2">+0.7</td><td colspan="2">+0.6</td></tr>
<tr><td>60°</td><td colspan="2">+0.7</td><td colspan="2">+0.7</td><td colspan="2">+0.7</td><td colspan="2">–0.3</td><td colspan="2">–0.6</td><td colspan="2">–0.3</td><td>–1.2</td><td>–2.0</td><td colspan="2">–0.4</td><td colspan="2">–0.2</td></tr>
<tr><td>75°</td><td colspan="2">+0.8</td><td colspan="2">+0.8</td><td colspan="2">+0.8</td><td colspan="2">–0.3</td><td colspan="2">–0.6</td><td colspan="2">–0.3</td><td>–1.2</td><td>–2.0</td><td colspan="2">–0.4</td><td colspan="2">–0.2</td></tr>
</table>

NOTE 1 At $\theta = 0°$ the pressure changes rapidly between positive and negative values on the windward face at pitch angle of $\theta = +5°$ to +45° so both positive and negative values are given. For those roofs, two cases should be considered: one with all positive values, and one with all negative values. No mixing of positive and negative values is allowed.

NOTE 2 Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. The values equal to 0.0 are given for interpolation purposes.

NOTE 3 The pitch angle of the windward face will always govern the pressure coefficients.

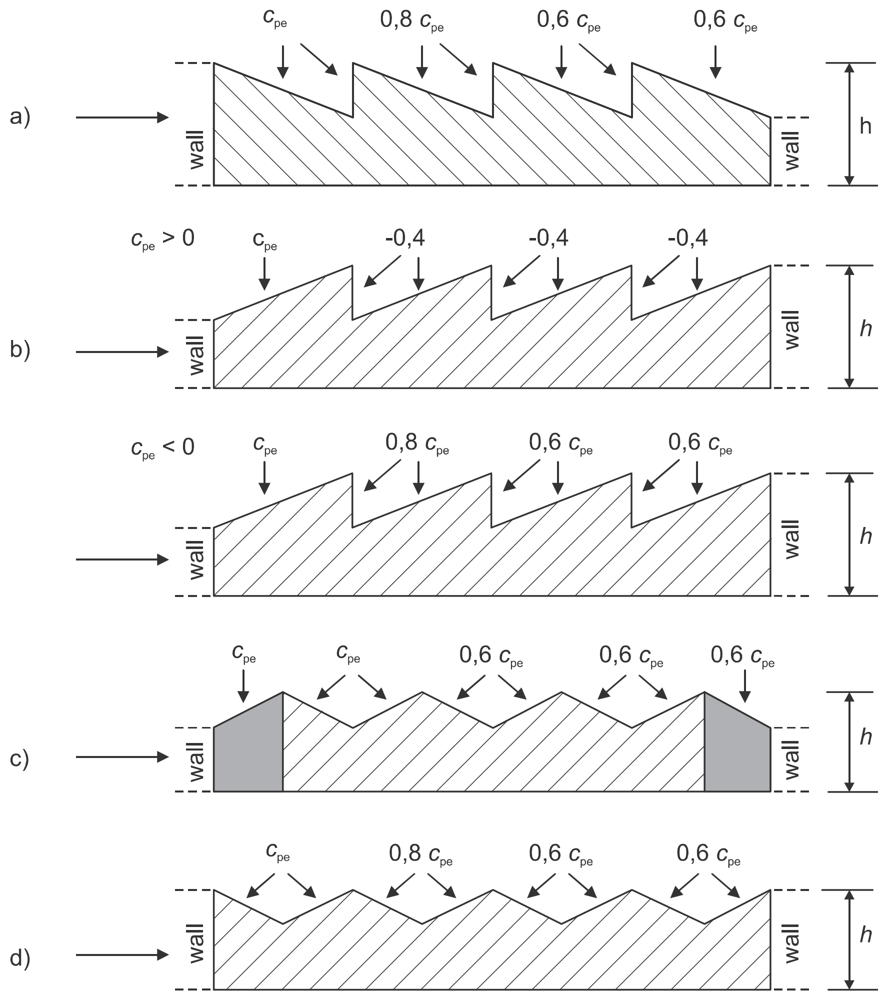
Source reference

11b.5 Multi-span roofs

1. The reference height z_e should be taken as h.
2. Pressure coefficients for multi-span roofs are derived from individual pressure coefficients of each individual span (i.e. from the pressure coefficients for mono or duo-pitch roofs). These coefficients are then modified by reduction factors to account for the effect of sheltered areas on upwind slopes.
3. Figures W. 11.8 a) and b) show the reduction factors for mono-pitch roofs. Figures W. 11.8 c) and d) show the reduction factors for duo-pitch roofs.
4. Zones F, G and J should be considered only for the upwind faces. Zones H and I should be considered for each span of the multi-span roof.

EC1-1-4 Figure 7.10

Figure W. 11.8: Key to multi-span roofs



NOTE 1 In configuration b) two cases should be considered depending on the sign of pressure coefficient c_{pe} on the first roof.

NOTE 2 In configuration c) the first c_{pe} is the c_{pe} of the monopitch roof; the second and all following c_{pe} are the c_{pe} of the troughed duopitch roof.

Source reference

11b.6 Vaulted roofs and domes

The reference height z_e should be taken as (h+f) as shown in Figure W. 11.7.

Vaulted roofs

1. The roof should be divided into areas, as shown Figure W. 11.9.
2. The recommended values of $c_{pe,10}$ are given in Figures W. 11.10 and W. 11.11 for zones A and B respectively.
3. For zone C the external pressure coefficients should be taken as 0.5 for $h/d \leq 5$ and $f/d \leq 0.5$

Domed roofs

1. The roof should be divided into areas, as shown in Figure W. 11.12.
2. The recommended values of $c_{pe,10}$ are given in Figure W. 11.12 for the different zones.

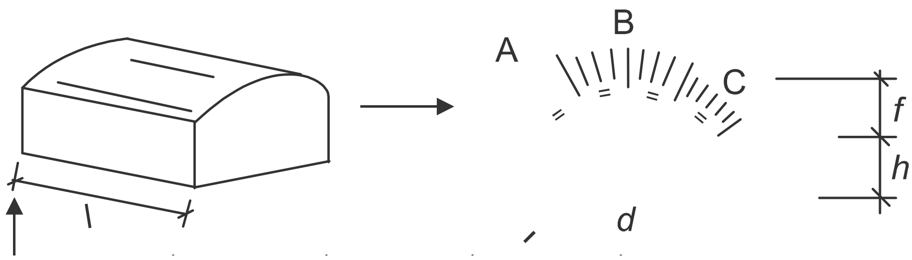
NOTE 1 The $c_{pe,1}$ values from Table W. 11.5a are to be used for local ridge and edge areas in zones G, F and J. The equivalent roof pitch is to be taken as the tangent of the part of the barrel vault under consideration.

NOTE 2 The values of $c_{pe,10}$ and $c_{pe,1}$ for duo-pitch roofs given Table W. 11.5b may be used for wind blowing on to the gable end of cylindrical roofs (θ = 90°).

NOTE 3 For h/d 0.5 there will be alternative cases of positive and negative pressures on zone A; both need to be considered.

EC1-1-4 Figure 7.11

Figure W. 11.9: Key to zones A, B and C for vaulted roofs

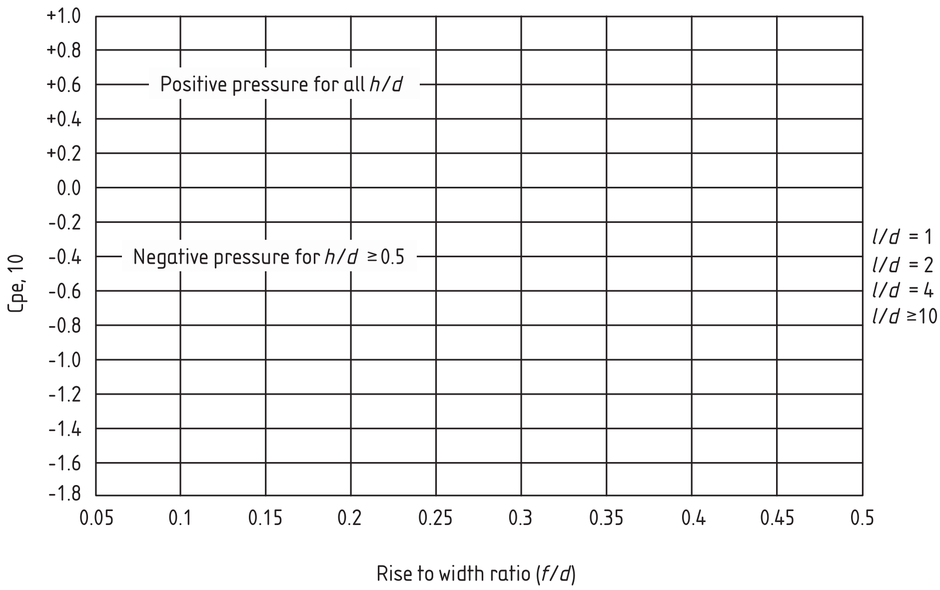


Zone A Windward
Zone B Central
Zone C Leeward

Source reference

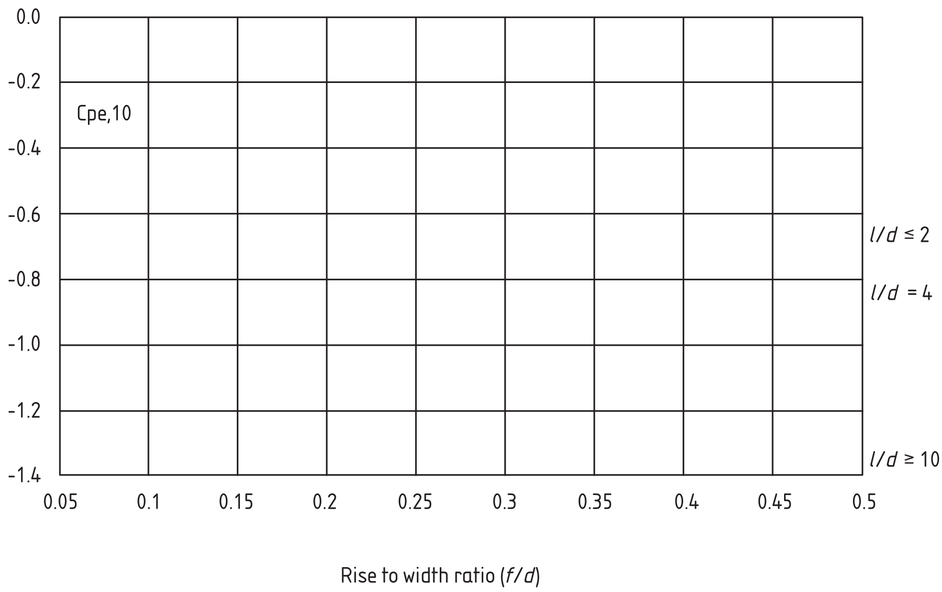
EC1-1-4 Figure NA.11

Figure W. 11.10: External pressure coefficients $c_{pe,10}$ for zone A of vaulted roofs for $\theta = 0°$



EC1-1-4 Figure NA.12

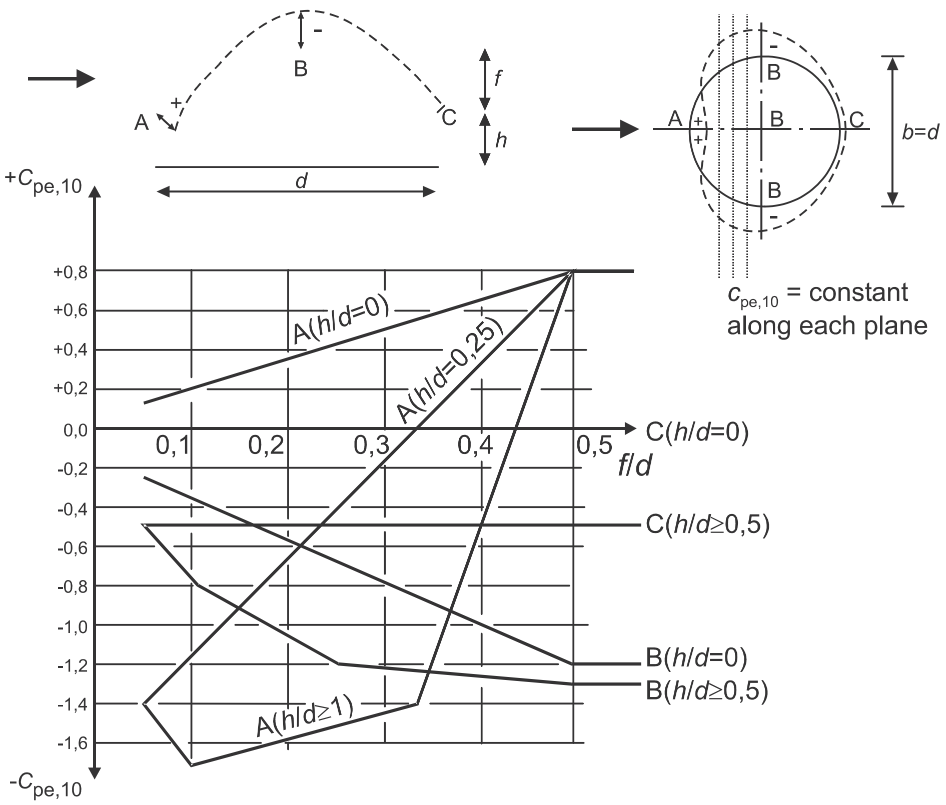
Figure W. 11.11: External pressure coefficients $c_{pe,10}$ for zone B of vaulted roofs



Source reference

EC1-1-4 Figure 7.12

Figure W. 11.12: Recommended values of external pressure coefficients $c_{pe,10}$ for domes with circular base



NOTE $c_{pe,10}$ is constant along arcs of circles, intersections of the sphere and of planes perpendicular to the wind; it can be determined as a first approximation by linear interpolation between the values in A, B and C along the arcs of circles parallel to the wind. In the same way the values of $c_{pe,10}$ in A if $0 < h/d < 1$ and in B or C if $0 < h/d < 0,5$ can be obtained by linear interpolation in the Figure above.

Stage 12: Internal pressure c_{pi}

Internal and external pressures are taken to act at the same time. The method for determining internal pressure coefficients requires the worst combination of external and internal pressures to be considered for every combination of possible openings and other leakage paths.

The internal pressure of a building is dependent on two things: the **external pressure** on the building and the **flow through openings** in the building. The internal pressure coefficient c_{pi} is therefore dependent on the size and location openings in the building envelope. Openings in the envelope include open windows, ventilators and chimneys, as well as background permeability such as air leakage around doors, windows, services and through the building envelope.

Source reference

Table W. 12.1 states typical permeability of types of construction. Modern construction methods, however, may lead to lower values than those stated. If more specific information is available, those values should be used instead of Table W. 12.1.

EC1-1-4 Table NA.5

Table W. 12.1: Typical permeability of construction in the UK

Form of construction	Permeability (open area/total area)
Office curtain walling	3.5×10^{-4}
Office internal partition walling	7.0×10^{-4}
Housing generally	10.5×10^{-4}
Energy efficient housing	4.0×10^{-4}

If the permeability of the walls is ***not known*** *the more onerous of* $c_{pi} = -0.3$ *and* $c_{pi} = +0.2$ *should be taken*.

If the permeability of the walls is ***known:***

1. Do at least two sides of the building (walls or roof) have openings greater than 30 % of each side's area?

EC1-1-4 7.3 and 7.4

If **yes**, the **rules relating to internal pressures do not apply**. Instead, the building should either be considered as a canopy roof, if there are at least two 'open' walls, or a set of free-standing walls if there is an 'open' roof.

2. Is there a dominant face to the building?

The answer is **yes** if:

Area of openings in one face ≥ 2x (area of openings in the remaining faces)

NOTE In some cases an opening, such as a door or window, would be dominant when open but is assumed closed in the ultimate limit state (e.g. during severe wind storms). The instance with the opening open should be considered as an accidental design situation in accordance with BS EN 1990. This is particularly important for tall internal walls when the wall has to carry the external wind load due to openings in the building envelope.

Source reference	

Dominant face

Area of the openings at the dominant face = 2x (area of the openings in the remaining faces)

EC1-1-4 Eqn. (7.1)

$$c_{pi} = 0.75c_{pe} \quad \text{(W. 12.1)}$$

Area of the openings at the dominant face ≥ 3x (area of the openings in the remaining faces)

EC1-1.4 Eqn. (7.2)

$$c_{pi} = 0.90c_{pe} \quad \text{(W. 12.2)}$$

When the area of the openings at the dominant face is between 2 and 3 times the area of the openings in the remaining faces, linear interpolation for calculating c_{pi} may be used.

c_{pe} value for the external pressure coefficient at the openings in the dominant face.
When these openings are located in zones with different values of external pressure, an area weighted average value of c_{pe} should be used.

No dominant face

Internal pressure coefficients are a function of the ratio of the height and the depth of the building, *h/d*, and the opening ratio μ for each wind direction θ.

The opening ratio μ should first be calculated from W. 12.3 for each wind direction. The Internal pressure coefficients c_{pi} can then be determined from Figure W. 12.1.

EC1-1-4 Eqn. (7.3)

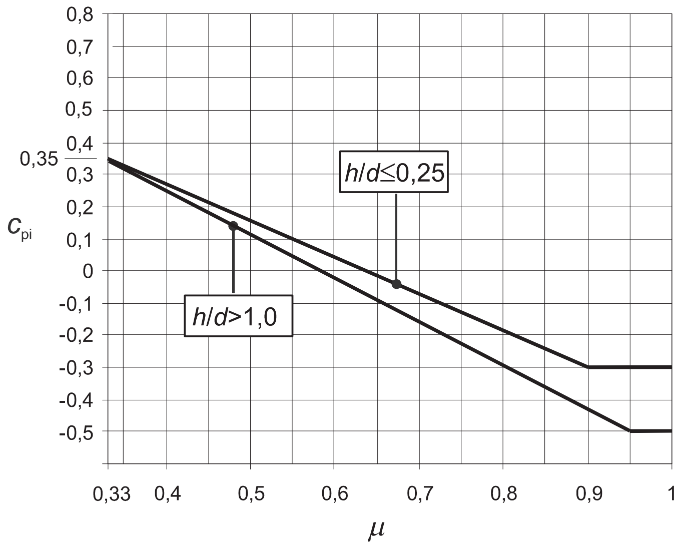
$$\mu = \frac{\sum \text{area of openings, where } c_{pe} \text{ is negative or } -0.0}{\sum \text{area of all openings}} \quad \text{(W. 12.3)}$$

Where it is not reasonable or possible to estimate μ then c_{pi} should be taken as the more onerous of +0.2 or –0.3.

Source reference

EC1-1-4 Figure 7.13

Figure W. 12.1: Internal pressure coefficients for uniformly distributed openings



NOTE 1 For values between *h*/*d* = 0.25 and *h*/*d* = 1.0 linear interpolation may be used.

NOTE 2 The reference height z_i for internal pressures should be taken as the reference height z_e for the external pressures on the faces which contribute through their openings to the internal pressure. If there are several openings in one face the highest value of z_e should be taken.

Stage 13: Pressures on walls and roofs with more than one skin

Exceptions

When a cavity wall has one or more leaves which are effectively tied together and constructed of small masonry units (i.e. block, brick, random rubble masonry or square dressed natural stone) the cavity wall should be treated as a single-skin element.

For small-format overlapping roofing elements with unsealed laps, such as tiles or slates, these rules do not apply. These should be designed according to BS 5534.

Air-tight layer between the skins

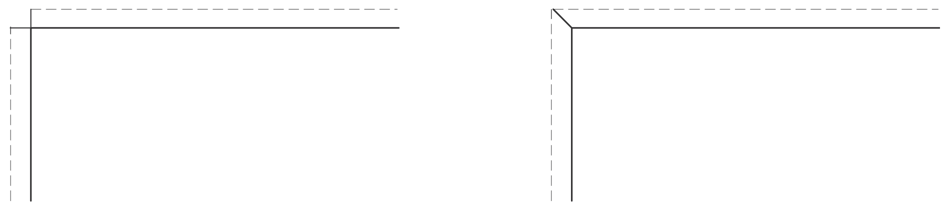
Figure W. 13.1(a) shows examples where the extremities of the space between the skins are closed.

Source reference

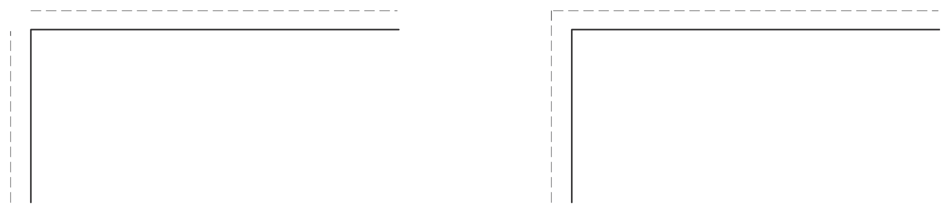
If the air between skins is not sealed and totally contained between the skins being considered, as in Figure W. 13.1(b), these rules do not apply and specialist advice should be sought.

EC1-1-4 Figure 7.14

Figure W. 13.1: Corner details for external walls with more than one skin



(a) extremities of the layer between skins closed



(b) extremities of the layer between skins open

For multi-skin walls the wind force on each skin is to be calculated separately. The permeability μ of a skin is expressed by the ratio of the total area of the opening to the total area of the skin. When the value μ is less than 0.1%, the skin is defined as impermeable.

One skin permeable

When only one skin is permeable, the wind force on the impermeable skin should be determined from the difference between the internal and the external wind pressure as described in Stage 15c.

More than one skin permeable

When more than one skin is permeable, the wind force on each skin depends on:

- the relative rigidity of the skins;
- the external and internal pressures;
- the distance between the skins;
- the permeability of the skins;
- the openings at the extremities of the layer between the skins.

Source reference

NA Section 7.2.10

First approximations

For situations where the space between the skins is **air tight** and is **less than 100 mm** – not including the insulation material when there is no air flow within the insulation – the following first approximations may be used:

- Where more than one skin is permeable, for a first approximation it can be taken that the wind pressure on the most rigid skin is the difference between the internal and the external pressures.
- Where the inside skin is impermeable and the outside skin is permeable with approximately uniformly distributed openings, the wind force on the *outside skin* may be calculated as:

  $c_{p,net} = 2/3 \cdot c_{pe}$ for over-pressure
  $c_{p,net} = 1/3 \cdot c_{pe}$ for under-pressure

  and the wind force on the *inside skin* may be calculated as:

  $c_{p,net} = c_{pe} - c_{pi}$

- Where the inside skin is permeable with approximately uniformly distributed openings and the *outside skin* is impermeable, the wind force on the outside skin may be calculated as:

  $c_{p,net} = c_{pe} - c_{pi}$

  and the wind force on the *inside skin* as:

  $c_{p,net} = 1/3 \cdot c_{pi}$

- When both the inside and outside skins are impermeable but the outside skin is more rigid, the wind force on the *outside skin* may be calculated as:

  $c_{p,net} = c_{pe} - c_{pi}$

- When both the inside and outside skins are impermeable but the inside skin is more rigid, the wind force on the *outside skin* may be calculated as:

  $c_{p,net} = c_{pe}$

  and the wind force on the *inside skin* as:

  $c_{p,net} = c_{pe} - c_{pi}$

Source reference

Stage 14a: Friction coefficients c_{fr}

The friction coefficients c_{fr} for walls and roof surfaces are given in **Table W. 13.1**.

EC1-1-4 Table 7.10

Table W. 13.1: Friction coefficients c_{fr} for walls, parapets and roof surfaces

Surface	Friction coefficient c_{fr}
Smooth (i.e. steel, smooth concrete)	0.01
Rough (i.e. rough concrete, tar-boards)	0.02
Very rough (i.e. ripples, ribs, folds)	0.04

Stage 14b: Area swept by the wind A_{fr}

Figure W. 13.1 shows **three simplified cases** for which friction needs to be considered:

1. A **horizontal plate-like canopy**, supported by legs. In this case friction is assumed to act over the whole bottom and top surface of the plate in all wind directions. For this type of structure:

EC1-1-4 Figure 7.22

$A_{fr} = 2.b.c$ (W. 14.1)

2. A **vertical plate-like structure**, such as a wall. In this case friction is assumed to act over both sides, but only when wind acts parallel to the wall. For this type of structure:

EC1-1-4 Figure 7.22

$A_{fr} = 2.h.d$ (W. 14.2)

3. An **enclosed building with a pitch**. In this case friction acts only on an area of the external surface parallel to the wind at a certain distance from the upwind eaves or corners. For this type of building:

   A_{fr} = the area of external surfaces (parallel to the wind direction) located beyond a distance of either 2·b or 4·h, whichever is less, from the upwind eaves or corners.

   It may be assumed that there is no friction acting on pitched roofs when the **wind is normal to the pitch** or eaves.

Source reference

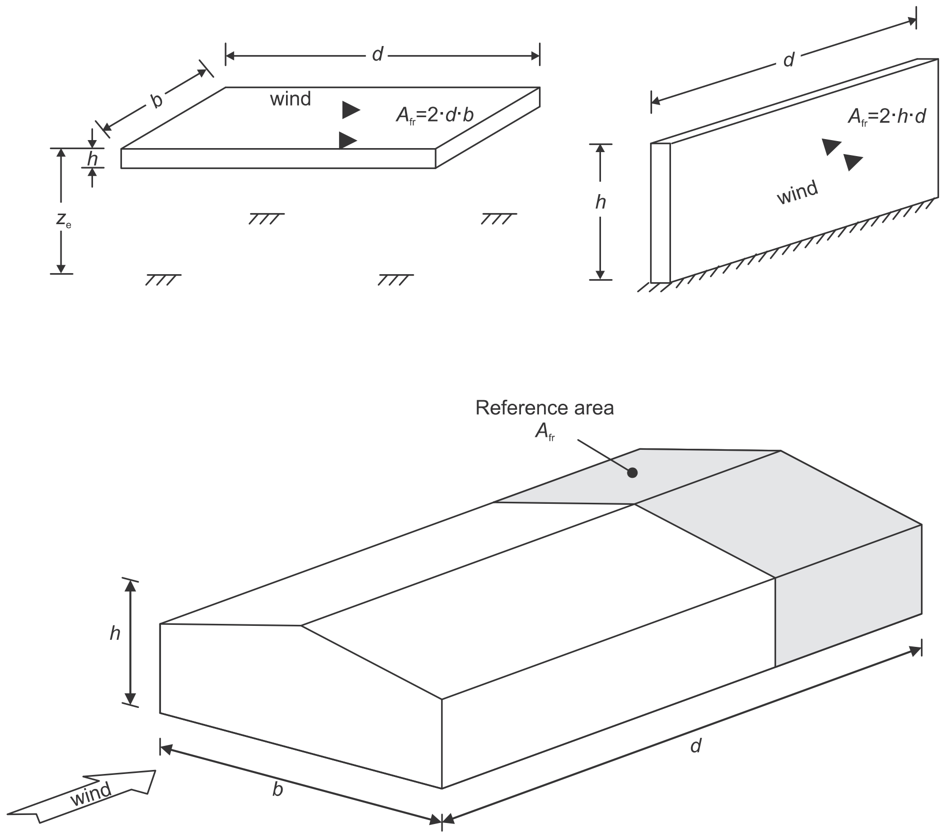
These are not the only cases for which friction needs to be considered, for example:

- A **pitched canopy** supported by legs. In this case friction is assumed to act over the whole bottom and top surface, as (1), but only when the wind acts parallel to the pitch.
- An **enclosed building with a flat roof**. In this case friction will be experienced for all wind directions, as (1), on the defined downwind external area as there is no ridge.

The reference height z_e should be taken as the structure height above ground or building height h, as shown in Figure W. 13.1.

EC1-1-4 Figure 7.22

Figure W. 14.1 Reference areas for friction



Source reference

Wind actions

Wind actions include wind pressures and wind forces which act on a structural element or surface.

Stage 15: Wind pressures on surfaces

Stage 15a: External pressures w_e

The external wind pressure w_e can be found using:

EC1-1-4 Eq. (5.1)

$$w_e = q_p(z_e - h_{dis}).c_{pe} \quad \text{(W. 15.1)}$$

$q_p(z_e - h_{dis})$	peak velocity pressure, as calculated in Stage 9
z_e	the reference height for the external pressure
c_{pe}	the external pressure coefficient, as calculated in Stages 11a or 11b

Stage 15b: Internal pressures w_i

The internal wind pressure w_i can be found using:

EC1-1-4 Eq. (5.2)

$$w_i = q_p(z_i - h_{dis}).c_{pi} \quad \text{(W. 15.2)}$$

$q_p(z_i - h_{dis})$	peak velocity pressure, as calculated in Stage 9
z_i	the reference height for the internal pressure
c_{pi}	the internal pressure coefficient, as calculated in Stages 11a or 11b

Stage 15c: Net pressures w

The net pressure is the difference between the external pressure and the internal pressure, taking account of their signs. Examples are shown in Figure W. 15.1.

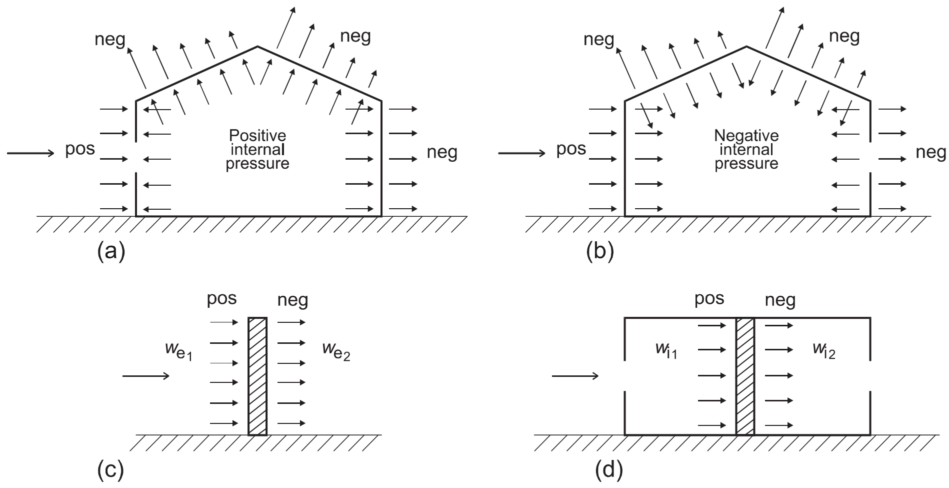
$$w = w_e - w_i \quad \text{(W. 15.3)}$$

Pressure:	towards the surface	+ve
Suction:	away from the surface	–ve

Source reference

EC1-1-4 Figure 5.1

Figure W. 15.1: Pressures on surfaces



Stage 16: Wind forces on surfaces

The wind force on a structure or structural element can be determined by calculating forces either directly, using force coefficients, or indirectly, using surface pressures and friction effects, the latter of which are used here.

The forces obtained from summing the pressures on the windward and leeward faces represent the maximum possible force. It is, however, very unlikely for these two maximum forces to act simultaneously. Therefore, a reduction factor may be applied to the summation of the pressures acting on both walls and roofs of the windward and leeward surfaces to account for the lack of correlation.

The following factors should be applied to the horizontal force component from all windward and leeward surfaces:

$h/d \geq 5$ the resulting force is multiplied by 1
$h/d \leq 1$ the resulting force is multiplied by 0.85

For **intermediate values of** h/d, linear interpolation may be used.

Wind forces should be **calculated for each surface** of the building. Forces on windward and leeward faces should then be **factored** to allow for lack of correlation.

Source reference	

Stage 16a: External forces $F_{w,e}$

EC1-1-4 (5.5)

$$F_{w,e} = c_s.c_d \sum w_e.A_{ref} \quad \text{(W. 16.1)}$$

Stage 16b: Internal forces $F_{w,i}$

EC1-1-4 (5.6)

$$F_{w,i} = c_s.c_d \sum w_i.A_{ref} \quad \text{(W. 16.2)}$$

Stage 16c: Friction forces F_{fr}

Friction forces F_{fr} act in the direction of the wind, parallel to external surfaces, and can arise when wind blows parallel to a surface. Areas where friction is applicable are explained in Stage 14.

EC1-1-4 (5.7)

$$F_{fr} = c_{fr}.q_p(z_e - h_{dis}).A_{fr} \quad \text{(W. 16.3)}$$

This effect can be **ignored when**:

Total area of all surfaces parallel with (or at a small angle to) the wind	<	4x [the total area of all external surfaces perpendicular to the wind] (windward and leeward)

c_sc_d	structural factor, as calculated in Stage 10
c_s	size factor
c_d	dynamic factor
A_{ref}	reference area of the individual surface
w_e	external wind pressure, as calculated in Stage 15a
w_i	internal wind pressure, as calculated in Stage 15b
c_{fr}	friction coefficient, as determined in Stage 14
$q_p(z_e - h_{dis})$	peak velocity pressure, as calculated in Stage 9
A_{fr}	area of external surface parallel to the wind, as determined in Stage 14

Stage 17: Total wind force F_w

EC1-1-4 Section 5.3(3)

$$F_w = \sum (F_{w,e} + F_{w,i} + F_{fr}) \quad (17.1)$$

$F_{w,e}$	external forces, as calculated in Stage 16a
$F_{w,I}$	internal forces, as calculated in Stage 16b
F_{fr}	friction forces, as calculated in Stage 16c

Image and text references

Figure W. 7.1 (a) Satellite Map, centred on SW1. Available from www.maps.google.co.uk. [accessed 28 April 2009]

Figure W. 7.2 (b) Satellite Map, centred on YO12. Available from www.maps.google.co.uk. [accessed 28 April 2009]

Figure W. 7.3 (c) Satellite Map, centred on YO11. Available from www.maps.google.co.uk. [accessed 28 April 2009]

Figure W. 7.4 (d) Satellite Map, centred on YO13. Available from www.maps.google.co.uk. [accessed 28 April 2009]

Cook, N (2007) *Designers' Guide to EN 1991-1-4: 2005 Eurocode 1: Actions on structures. Part 1-4: General Actions – Wind Actions*, London: Thomas Telford, p. 45.

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