Manual for the design of building structures to Eurocode 1 and Basis of Structural Design

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Constitution of the Task Group

Dr J Tubman PhD BSc(Eng) CEng FIStructE MICE (Scott Wilson) *Chairman*
B C Ayres BSc CEng MIStructE DMS (London Borough of Bromley)
M G Baker CEng MIStructE (HSP Consulting LLP)
Professor I W Burgess BA PhD CEng MIStructE (University of Sheffield)
D Dibb-Fuller CEng FIStructE FIHeem (Gifford)
R Shipman BSc CEng MICE PGCE (Communities and Local Government)

Consultants

Dr P A Blackmore PhD BSc CEng MIStructE MICE FRMetS (BRE Group)
Professor H Gulvanessian CBE BSc(Eng) MSc CEng FICE (Associate, BRE Group)
T Lennon BEng BA (BRE Group)
R Rupasinghe BSc(Eng) (BRE Group)

Secretary to the Task Group

Dr J D Littler PhD (The Institution of Structural Engineers)

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Notation

Note The notation used is based on ISO 3898: 1997[1].

Latin upper case letters

A	Accidental action
A	Loaded area (Chapter 3 only)
A	Site altitude above sea level (Chapter 5 only)
A	Area (Chapter 6 only)
A_0	Basic area
A_d	Design value of an accidental action
A_{fr}	Area swept by the wind
A_{ref}	Reference area
A_t	Total area of enclosure
A_v	Area of vertical openings
C_e	Exposure coefficient
C_t	Thermal coefficient
E	Effect of actions
E_d	Design value of effect of actions
$E_{d,dst}$	Design value of effect of destabilising actions
$E_{d,stb}$	Design value of effect of stabilising actions
$E_{fi,d,t}$	Design value of the relevant effects of actions in the fire situation at time t
F	Action
F	Collision force (Chapter 9 only)
F_d	Design value of an action
F_{dx}	Horizontal static equivalent or dynamic design frontal force
F_{dy}	Horizontal static equivalent or dynamic design lateral force
F_{fr}	Resultant friction force
F_{hn}	Nominal horizontal forces
F_k	Characteristic value of an action
F_{rep}	Representative value of an action
F_s	Force per metre length exerted by a sliding mass of snow
F_w	Resultant wind force
G	Permanent action
G_d	Design value of a permanent action
$G_{d,inf}$	Lower design value of a permanent action
$G_{d,sup}$	Upper design value of a permanent action
G_k	Characteristic value of a permanent action
$G_{k,j}$	Characteristic value of permanent action j
$G_{k,j,sup}/G_{k,j,inf}$	Upper/lower characteristic value of permanent action j
H	Height of a topographic feature
H	Height of the compartment (Appendix E only)
I_v	Turbulence intensity

K_{rd}	Reduction factor for parapets
L_d	Actual length of a downwind slope
L_e	Effective length of an upwind slope
L_u	Actual length of an upwind slope
O	Opening factor $\left(A_v \sqrt{h}/A_t\right)$
P	Relevant representative value of a prestressing action (see EC2 to EC6 plus EC8 and EC9)
Q	Variable action
Q_c	Construction loads (general symbol)
Q_{ca}	Construction loads due to working personnel, staff and visitors, possibly with handtools or other small site equipment
Q_{cb}	Construction loads due to storage of movable items (e.g. building and construction materials, precast elements, and equipment)
Q_{cc}	Construction loads due to non permanent equipment in position for use during execution, either static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or during movement (e.g. travelling forms, launching girders and nose, counterweights)
Q_{cd}	Construction loads due to movable heavy machinery and equipment, usually wheeled or tracked (e.g. cranes, lifts, vehicles, lift trucks, power installations, jacks, heavy control devices)
Q_{ce}	Construction loads from accumulation of waste materials (e.g. surplus construction materials, excavated soil or demolition materials)
Q_{cf}	Construction loads from parts of a structure in temporary states (under execution) before the final design actions take effect
Q_d	Design value of a variable action
Q_k	Characteristic value of a variable concentrated load (Chapter 3 only)
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*
Q_W	Wind actions
Q_{wa}	Actions caused by water
R	Resistance
R_d	Design value of the resistance
$R_{fi,d,t}$	Design value of the resistance of the member in the fire situation at time *t*
R_k	Characteristic value of the resistance
S_e	Snow load per metre length due to overhang
T_{max}	Maximum shade air temperature with an annual probability of being exceeded of 0.02 (equivalent to a mean return period of 50 years)
T_{min}	Minimum shade air temperature with a 0.02 annual probability of falling below this value (equivalent to a mean return period of 50 years)
T_0	Initial temperature when structural element is restrained
T_{in}	Air temperature of the inner environment
T_{out}	Temperature of the outer environment
ΔT_1, ΔT_2, ΔT_3, ΔT_4	Values of heating (cooling) temperature differences
ΔT	Sum of linear temperature difference component and non-linear part of the temperature difference component
ΔT_E	Non-linear part of the temperature difference component
ΔT_M	Linear temperature difference component
ΔT_N	Overall range of uniform temperature component

ΔT_p	Temperature difference between different parts of a structure given by the difference of average temperatures of these parts
ΔT_U	Uniform temperature component
X	Material property
X_d	Design value of a material property
X_k	Characteristic value of a material property

Latin lower case letters

a	Height of the application area of a collision force
a_d	Design values of geometrical data
a_k	Characteristic values of geometrical data
a_{nom}	Nominal value of geometrical data
b	Width of construction work (Chapter 5 only)
b	Width of the structure (the length of the surface perpendicular to the wind direction if not otherwise specified) (Chapter 6 only)
b	Width of an obstacle (e.g. building column) (Chapter 9 only)
b	$\sqrt{(\rho c \lambda)}$ and should lie between 100 and 2200 (Appendix E only)
c	Specific heat of boundary of enclosure
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_{lat}	Aerodynamic exciting coefficient
c_p	Pressure coefficient
c_{pe}	External wind pressure coefficients for free-standing walls
c_{prob}	Probability factor
c_r	Roughness factor
c_o	Orography 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) (Chapter 6 only)
e	Eccentricity of a force or edge distance
g_k	Weight per unit area, or weight per unit length
h	Height of construction work (Chapter 5 only)
h	Height of the structure (Chapter 6 only)
h	Clearance height from roadway surfacing to underside of a building (Chapter 9 only)
h	Height of vertical openings (Appendix E only)
h_{ave}	Obstruction height
h_{dis}	Displacement height
k	Coefficient to take account of the irregular shape of snow
k_b	Conversion factor for the compartment thermal properties
k_c	A correction factor dependent on the structural material
k_p	Peak factor

k_r	Terrain factor
l	Length of a horizontal structure
l_s	Length of snow drift or snow loaded area
m	Mass
n	Number of storeys
n_1	Fundamental frequency
p	Annual probability of exceedence
q_b	Reference mean (basic) velocity pressure
$q_{ca,k}$	Characteristic values of the uniformly distributed loads of construction loads Q_{ca}
$q_{cb,k}$	Characteristic values of the uniformly distributed loads of construction loads Q_{cb}
$q_{cc,k}$	Characteristic values of the uniformly distributed loads representing construction loads Q_{cc}
$q_{f,d}$	Design value of the fire load density related to the surface area of the floor
q_k	Characteristic value of a uniformly distributed load, or line load
q_p	Peak velocity pressure
$q_{t,d}$	Design value of the fire load density related to the total surface area of the enclosure
r	Radius
r_F	Reduction factor
s	Snow load on the roof (Chapter 5 only)
s	Distance from structural element to centre-line of road or track (Chapter 9 only)
s_k	Characteristic value of snow on the ground at the relevant site
t	Time
$t_{fi,d}$	Design value of the fire resistance for a given load
$t_{fi,req}$	Required fire resistance time
t_{lim}	A minimum value for the duration of the fire based on slow, medium or fast fire growth rates
t^*	Parametric time = $\mathrm{t}.\Gamma$
u	Horizontal displacement of a structure or structural member
v_m	Mean wind velocity
$v_{b,0}$	Fundamental value of the basic wind velocity
v_b	Basic wind velocity
v_v	Velocity
w	Vertical deflection of a structural member
w	Wind pressure (Chapter 6 only)
w_f	Ventilation factor
x	Horizontal distance of the site from the top of a crest
z	Height above ground
z_{ave}	Average height
z_0	Roughness length
z_e, z_i	Reference height for external wind action, internal pressure
z_g	Distance from the ground to the considered component
z_{max}	Maximum height
z_{min}	Minimum height
z_s	Reference height for determining the structural factor

Greek upper case letters

Γ	$[O/b]^2/(0.04/1160)^2$
Δa	Change made to nominal geometrical data for particular design purposes, e.g. assessment of effects of imperfections
Φ	Upwind slope

Greek lower case letters

α	Pitch of roof, measured from horizontal
α_A	Reduction factor
α_n	Reduction factor
α_T	Coefficient of linear expansion
α_v	A_v / A_f
γ	Partial factor (safety or serviceability)
γ	Bulk weight density (Chapter 3 only)
γ	Weight density of snow (Chapter 5 only)
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*
$\gamma_{G,j,sup}$ / $\gamma_{G,j,inf}$	Partial factor for permanent action *j* in calculating upper/lower design values
γ_m	Partial factor for a material property
γ_M	Partial factor for a material property, also accounting for model uncertainties and dimensional variations
γ_P	Partial factor for prestressing actions (see EC2 to EC6 plus EC8 and EC9)
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*
γ_{Rd}	Partial factor associated with the uncertainty of the resistance model
γ_{Sd}	Partial factor associated with the uncertainty of the action and/or action effect model
θ	Wind direction
θ_d	Design value of material temperature for a given load
$\theta_{cr,d}$	Design value of the critical material temperature for a given load
θ_g	Gas temperature in the fire compartment
λ	Slenderness ratio
λ	Thermal conductivity of boundary (Appendix E only)
μ	Snow load shape coefficient (Chapter 5 only)

μ	Opening ratio, permeability of a skin (Chapter 6 only)
ξ	Reduction factor
ρ	Air density
ρ	Density of boundary enclosure (Appendix E only)
ρ_{wa}	Density of water
σ_v	Standard deviation of the turbulence
φ	Dynamic magnification factor (Chapter 3 only)
φ	Solidity ratio, blockage of canopy (Chapter 6 only)
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
ψ_{mc}	Reduction factor for multibay canopies
ψ_s	Shelter factor for walls and fences

Subscript indices (used only in Chapter 6 and appendices F, G and I)

e	External; exposure
fr	Friction
i	Internal
m	Mean
p	Peak; parapet
ref	Reference
v	Wind velocity
z	Vertical direction

Foreword

BS EN 1990: 2002 Eurocode: Basis of Structural Design and BS EN 1991: 2002 – 2006: Eurocode 1: Actions on Structures (in various parts) and their respective UK National Annexes provide the basis for all structural design carried out to the Eurocodes for projects in the UK.

This *Manual* is one volume in a set of Eurocode Manuals published by the Institution of Structural Engineers and is unique in that it covers two Eurocodes, namely BS EN 1990 (referred to as EC0 in this *Manual*) and BS EN 1991 (referred to as EC1). It is hoped that the set of Manuals will form an invaluable tool for all practising engineers in the UK and further afield and will assist in the transition towards adoption of the Eurocode suite in the UK.

The aim of the Task Group in preparing this *Manual* was to produce a working version of EC0 and EC1 that would be of use to practising engineers for the design of the majority of "straightforward" buildings in the UK. The complexity of the source documents has made this an extremely difficult task and there have been long debates as to which material should be included in the main body of the *Manual*, which should be consigned to the Appendices, and which should be omitted altogether. We have strived to achieve the necessary balance but, inevitably, this is a subjective issue. There will undoubtedly be occasions when designers require access to the much wider scope of the source documents EC0 and EC1.

Institution guidelines require that the *Manual* is not a commentary document. In general, therefore, background to clauses in EC0 and EC1 cannot be given; however, it is hoped that the form of presentation adopted will reveal at least some of the underlying philosophy in a way that is accessible and useful to readers.

The Eurocodes introduce some terminology and conventions that will not be familiar to those raised on a diet of BS 6399, BS 5950, BS 8110 and the other so-called "current" design standards in the UK. There have been calls for Eurocode terminology to be re-cast in traditional, recognisable UK terms in this *Manual*. Those calls have been rejected as such a course would serve only to postpone the inevitable. The only major exception is the continued use of a decimal point rather than a comma. The reader may derive some consolation from the fact that the source documents EC0 and EC1 are themselves not consistent in their use of terminology; for example, the Eurocodes call for snow and imposed *loads* but wind and accidental *actions*.

I would like to convey my personal thanks to all the members of the Task Group and to the four consultants who assisted to such a great extent with the drafting, the consultants largely funded by DCLG to whom the Institution extends its thanks. The *Manual* would not have been completed without the sterling efforts of Dr John Littler, Secretary to the Task Group and I would like to extend my particular thanks to him for his patience, perseverance and attention to detail. Finally, I must thank the members of the Institution's Technical Publications Panel for providing very constructive criticism which has undoubtedly helped shape and improve the *Manual*.

I hope you will find the content of the following pages of use to you as you address the new era of structural design in the UK – the Eurocodes.

Dr John Tubman
Chairman

Chapter 1: Introduction

1.1 Aims of the *Manual*

This *Manual* provides guidance on the design of building structures. Structures designed in accordance with this *Manual* will normally comply with BS EN 1990: 2002[2], BS EN 1991-1-1: 2002[3], BS EN 1991-1-2: 2002[4], BS EN 1991-1-3: 2003[5], BS EN 1991-1-4: 2005[6], BS EN 1991-1-5: 2003[7] BS EN 1991-1-6: 2005[8] and BS EN 1991-1-7: 2006[9].

Given the complexity of loadings (actions) and load combinations in the Eurocodes, it is likely that computer analysis for load effects will be carried out in most cases. The intention of this *Manual*, however, is to facilitate the generation of load cases and load combinations for simpler structures and those amenable to hand calculation. In any case, it is intended that such hand analysis methods are used to verify the output of more sophisticated methods.

1.2 The Eurocode system

1.2.1 The rationale behind the Eurocode system

The structural Eurocodes were initiated by the European Commission but are now produced by the Comité Européen de Normalisation (CEN) which is the European standards organisation, its members being the national standards bodies of the EU and EFTA countries, e.g. British Standards Institution (BSI).

CEN makes available the design standards as full European Standards EN (Euronorms). These are then published as National Standards, e.g. by BSI:

BS EN 1990:	Eurocode –	Basis of structural design (EC0)
BS EN 1991:	Eurocode 1:	Actions on structures (EC1)
	Part 1-1:	General actions – densities, self-weight and imposed loads (EC1 Part 1-1)
	Part 1-2:	General actions – actions on structures exposed to fire (EC1 Part 1-2)
	Part 1-3:	General actions – snow loads (EC1 Part 1-3)
	Part 1-4:	General actions – wind loads (EC1 Part 1-4)
	Part 1-5:	General actions – thermal actions (EC1 Part 1-5)
	Part 1-6:	General actions – actions during execution (EC1 Part 1-6)

	Part 1-7:	General actions – accidental actions (EC1 Part 1-7)
	Part 2:	Traffic loads on bridges (EC1 Part 2)
	Part 3:	Actions induced by cranes and machinery (EC1 Part 3)
	Part 4:	Actions on structures – silos and tanks (EC1 Part 4)
BS EN 1992:	Eurocode 2:	Design of concrete structures (EC2)
BS EN 1993:	Eurocode 3:	Design of steel structures (EC3)
BS EN 1994:	Eurocode 4:	Design of composite steel and concrete structures (EC4)
BS EN 1995:	Eurocode 5:	Design of timber structures (EC5)
BS EN 1996:	Eurocode 6:	Design of masonry structures (EC6)
BS EN 1997:	Eurocode 7:	Geotechnical design (EC7)
BS EN 1998:	Eurocode 8:	Design of structures for earthquake resistance (EC8)
BS EN 1999:	Eurocode 9:	Design of aluminium structures (EC9)

For brevity and simplicity, for the rest of this *Manual* each Eurocode will be referred to by the abbreviation in brackets above.

Each of the ten Eurocodes is made up of separate parts, which cover the technical aspects of the structural and fire design of buildings and civil engineering structures. The Eurocodes are a harmonised set of documents that have to be used together. Figure 1.1 shows the structure of the Eurocode family and the links between the Eurocodes.

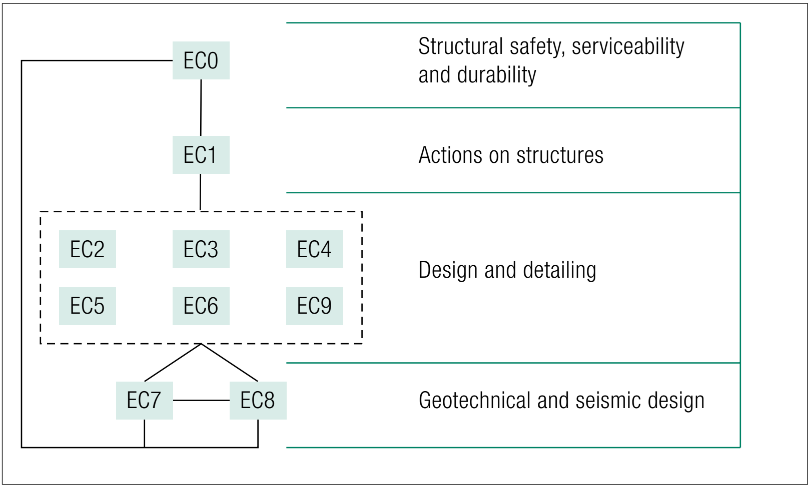


Fig 1.1 Links between the Eurocodes

All Eurocodes follow a common editorial style. They contain 'Principles' and 'Application rules'. Principles are identified by the letter P following the paragraph number. Principles are general statements and definitions for which there is no alternative, as well as requirements and analytical models for which no alternative is permitted unless specifically stated.

Application rules are generally recognised rules which comply with the Principles and satisfy their requirements. Alternative rules may be used provided that compliance with the Principles can be demonstrated, however the resulting design cannot be claimed to be wholly in accordance with the Eurocode although it will remain in accordance with the Principles.

1.2.2 Role of EC0

For the design of buildings and civil engineering works every Eurocode part from EC1 to EC9 has to be used together with EC0.

EC0 is therefore the key head code in the Eurocode system and provides the material independent information required for the design of buildings and civil engineering works within that system.

1.2.3 Assumptions made in EC0

A design which employs the rules in this *Manual* is deemed to meet the requirements in the Eurocodes provided the assumptions given below are satisfied.

The general assumptions made in EC0 (which apply to all the Eurocodes) are:
– the choice of the structural system and the design of the structure are made by appropriately qualified and experienced personnel
– execution is carried out by personnel having the appropriate skill and experience
– adequate supervision and quality control is provided in design offices and during execution of the work, i.e. factories, plants, and on site
– the structure will be adequately maintained
– the structure will be used in accordance with the design assumptions
– the construction materials and products are used as specified in the Eurocodes or in the relevant execution standards, or reference material or product specifications.

1.2.4 Using Eurocodes at a National level

1.2.4.1 Use of Eurocodes as National Standards
The European Commission recognises the responsibility of regulatory authorities (e.g. the Sustainable Buildings Division at Communities and Local Government) or national competent authorities (e.g. the Highways Agency) in each EU Member State. Their right to determine values related to safety matters at national level through a National Annex has been safeguarded.

It is the responsibility of each National Standards Body (e.g. BSI in the UK) to use Eurocodes as National Standards.

The National Standard will comprise, without any alterations, the full text of the Eurocode and its annexes as made available by CEN. This may be preceded by a national title page and national foreword, and may be followed by a National Annex (see Figure 1.2).

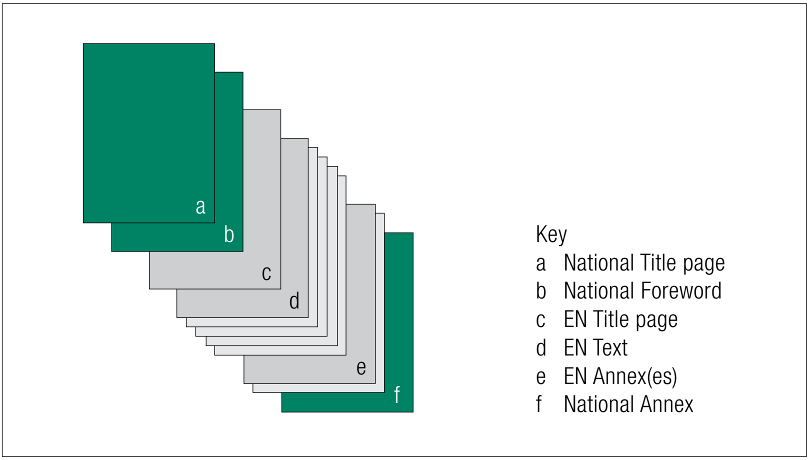


Fig 1.2 Format of the national publication of Eurocodes

1.2.4.2 Rules and Contents of National Annexes for Eurocodes

A National Annex may only contain:

– decisions on the application of informative annexes
– references to non-contradictory complementary information to assist the user in applying the Eurocode
– Nationally Determined Parameters (NDPs).

NDPs are those parameters to be used for the design of buildings and civil engineering works to be constructed in the country concerned, which are left open in the Eurocode for national choice.

In the UK, BSI has published the NDPs in a National Annex for each Eurocode part with the agreement of the national competent authorities.

A National Annex cannot change or modify the content of the Eurocode text in any way other than where the Eurocode indicates that national choices may be made by means of NDPs.

1.2.4.3 Using Eurocodes to design structures in a different country
Each EU Member State will have a different National Annex – the Annex used must be the one applicable where the building or civil engineering work is being constructed.

For example, a UK designer will have to use the appropriate Eurocode with the UK National Annex when designing a building in the UK. The same designer, designing a building in Italy, will have to use the Eurocode with the Italian National Annex.

1.2.4.4 Nationally Determined Parameters
The values, classes or methods to be chosen or determined at national level, called 'Nationally Determined Parameters' (NDPs), will allow the EU Member States to choose the level of safety, aspects of durability and economy applicable to works in their territory. They include:
– values and/or classes where alternatives are given in the Eurocodes
– values to be used where only a symbol is given in the Eurocodes
– country-specific data (geographical, climatic, etc.), e.g. snow maps
– procedures to be used where alternative procedures are given in the Eurocodes.

Within this *Manual* the values of NDPs adopted by the UK are shown in bold. This system has been adopted to warn the reader that a different value may apply if the design is to be constructed outside the UK. The NDP values (and those from the Eurocode) are those current at the date of publication of the *Manual*.

Where a whole paragraph or section is in bold this indicates that it is informative (i.e. for information, not a requirement) in the main body of the Eurocode, but that the UK National Annex states that it is normative (i.e. it should have the same force as the main body of the Eurocode) in the UK. This system has been adopted to warn the reader that different procedures may be applicable if the design is to be constructed outside the UK.

1.3 Scope of the *Manual*

This *Manual* has been written using the UK parameters and should therefore only be used for buildings that are intended to be constructed in the UK.

The range of structures and structural elements covered by the *Manual* is limited to building structures (other than agricultural buildings) that are in consequence classes CC1, CC2a and CC2b as listed in EC1 Part 1-7 on Accidental actions (see Table 1.1).

In using the *Manual* the following should be noted:

– The *Manual* has been drafted to comply with EC0, EC1 Part 1-1, EC1 Part 1-2, EC1 Part 1-3, EC1 Part 1-4, EC1 Part 1-5, EC1 Part 1-6 and EC1 Part 1-7 together with the relevant UK National Annexes as published to March 2010.
– For elements of foundation and substructure the *Manual* assumes that appropriate section sizes and loads have been obtained from EC7[10].
– The *Manual* assumes that the design is being undertaken with the appropriate material Eurocode together with the relevant UK National Annexes.
– Vibrations are not generally dealt with in this *Manual* and dynamic structures are not covered by it. For such structures, expert advice should be sought.
– Seismic actions are not covered by this *Manual*. Refer to EC8[11] and the Instititution Manual for the seismic design of steel and concrete buildings to Eurocode 8[12].
– Fatigue verifications are not covered by the *Manual.*
– External explosions are only covered to a limited extent by EC1 Part 1-7 (and therefore this *Manual*) – see Chapter 9.
– The *Manual* assumes a design life of the permanent structure of 50 years (except where specifically indicated otherwise). Designers should refer to their client to check that a design life of 50 years is acceptable. However, departing from a 50 year design life would strictly mean that the values of partial factors would need to change and therefore this would be a dangerous course of action unless the correct partial factors for the revised design life were obtained and used. Climatic actions for design lives other than 50 years should be obtained from EC1 Part 1-3, Part 1-4 and Part 1-5. Section 8.3.1 of this *Manual* should be used to determine climatic actions for design lives less than 50 years.
– The issue of sustainability is not included in this *Manual* as it is not addressed in the Eurocodes.

For structures or elements outside this scope or further information EC1 Parts 1-1 to 1-7 should be used.

The guidance given in the *Manual* may be applicable for determining actions and their effects for structures in consequence class CC3 (see Table 1.1). However, there will often be more complex actions (dynamics, other accidental actions, etc.) that apply to CC3 structures and users of this *Manual* should be aware of the need to look outside the *Manual* where necessary for more information.

For simplicity, reference to ‘clauses’ e.g. ‘EC0, Clause 4.1.2(3)’ will be restricted to clauses in the Eurocodes or other Codes. References to other parts of this *Manual* will be by Section number, e.g. ‘Section 7.2.3’ or ‘Chapter 4’.

Table 1.1 Scope of the *Manual* in terms of consequence classes

	Included in the *Manual*	Excluded from the *Manual*
Consequence class	Example of categorisation of building type and occupancy	Example of categorisation of building type and occupancy
CC1	– Single occupancy houses not exceeding 4 storeys – Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height	– Agricultural buildings
CC2a Lower Risk Group	– 5 storey single occupancy houses – Hotels not exceeding 4 storeys – Flats, apartments and other residential buildings not exceeding 4 storeys – Offices not exceeding 4 storeys – Industrial buildings not exceeding 3 storeys – Retailing premises not exceeding 3 storeys of less than 1000 m^2 floor area in each storey – Single storey educational buildings – All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000m^2 at each storey	– Advice on loads due to industrial activities
CC2b Upper Risk Group	– Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys – Educational buildings greater than single storey but not exceeding 15 storeys – Retailing premises greater than 3 storeys but not exceeding 15 storeys – Hospitals not exceeding 3 storeys – Offices greater than 4 storeys but not exceeding 15 storeys – All buildings to which the public are admitted and which contain floor areas exceeding 2000m^2 but not exceeding 5000m^2 at each storey – Car parking not exceeding 6 storeys	

Table 1.1 Continued

	Included in the *Manual*	Excluded from the *Manual*
Consequence class	Example of categorisation of building type and occupancy	Example of categorisation of building type and occupancy
CC3		– All buildings exceeding 15 storeys – All buildings to which the public are admitted and which contain floor areas exceeding 5000m^2 at each storey – Hospitals exceeding 3 storeys – Car parking exceeding 6 storeys – All buildings to which members of the public are admitted in significant numbers – Stadia accommodating more than 5000 spectators – Buildings containing hazardous substances and / or processes

1.4 General principles

One engineer should be responsible for the overall design of a structure, including stability, and should ensure the compatibility of the design and details of parts and components even where some or all of the design and details of those parts and components are not made by the same engineer.

The engineer should provide a front sheet of Design Information listing all the relevant design data (see Appendix A).

In addition to the adequacy of the structure in its permanent condition, the engineer should consider site constraints, buildability, maintainability and decommissioning.

Engineers should take account of their responsibilities as ‘Designer’ under the Construction (Design & Management) Regulations[13].

1.5 Notation and terminology

The notation and terminology in the *Manual* follow the Eurocode system except that a full stop rather than a comma is used as a decimal separator.

The definitions given in ISO 2394[14], ISO 3898[1], ISO 8930[15] apply to this *Manual*.

The definitions of the axes are as shown in Figure 1.3. Note that this local axis convention is different from that traditionally used in the UK prior to the introduction of the Eurocodes.

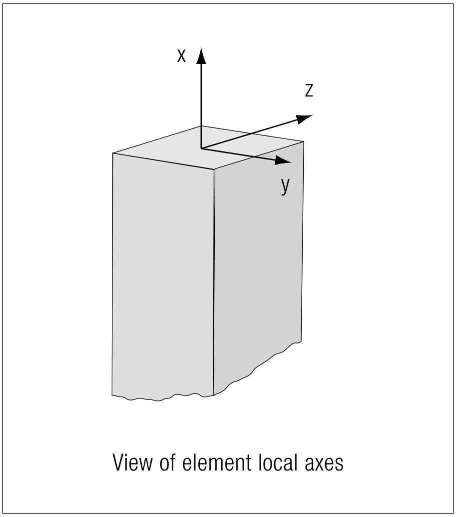


Fig 1.3 Notation for geometric axes

Chapter 2: Basis of Structural Design

2.1 Scope

This Chapter gives guidance contained in EN 1990: Eurocode – Basis of Structural Design[2] (EC0) for buildings within the scope of this *Manual*.

EC0 establishes the principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability.

EC0 is intended to be used in conjunction with EC1 to EC9 for the structural design of buildings.

EC0 provides a comprehensive list of terms and definitions. Those used in this Chapter are given in Sections 2.2 to 2.5.

2.2 Common terms and definitions used in EC0 and EC2 to EC9

Construction material – Material used in construction work, e.g. concrete, steel, timber, masonry.

Construction works – Everything that is constructed or results from construction operations.
Note this definition accords with ISO 6707-1[16]. The term covers both building and civil engineering works. It refers to the complete construction works comprising structural, non-structural and geotechnical elements.

Execution – All activities carried out for the physical completion of the work including procurement, the inspection and documentation thereof.
Note The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

Form of structure – Arrangement of structural members.
Note Forms of structure are, for example, frames, trusses.

Method of construction – Manner in which the execution will be carried out, e.g. cast in place, prefabricated, cantilevered.

Structural member – Physically distinguishable part of a structure, e.g. a column, a beam, a slab, a foundation pile.
Note These are parts of a structure which are intended to carry load or are intended by the designer to be capable of doing so in certain circumstances.

Structural model – Idealisation of the structural system used for the purposes of analysis, design and verification.

Structural system – Load-bearing members of a building and the way in which these members function together.

Structure – Organised combination of connected parts designed to carry loads (safely) and provide adequate rigidity.

Type of building or civil engineering works – Type of construction works designating its intended purpose, e.g. dwelling house, retaining wall, industrial building, road bridge.

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.

2.3 Special terms relating to design in general

Accidental design situation – Design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure.

Basic variable – Part of a specified set of variables representing physical quantities which characterise actions and environmental influences, geometrical quantities, and material properties including soil properties.

Design criteria – Quantitative formulations that describe for each limit state the conditions to be fulfilled.
Note for example, expression 6.10 in EC0 is a design criterion. See Section 2.10.3.3.

Design situations – Sets of physical conditions representing the real conditions occurring during a certain time interval for which the design will demonstrate that relevant limit states are not exceeded.

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. The UK National Annex for EC0 gives an indicative design working life of 50 years for buildings.

Fire design – Design of a structure to fulfil the required performance in case of fire.

Hazard – An unusual and severe event, e.g. an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation from intended dimensions.

Irreversible serviceability limit states – Serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed.

Limit states – States beyond which the structure no longer fulfils the relevant design criteria.

Load arrangement – Identification of the position, magnitude and direction of a free action.

Load case – Compatible load arrangements, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for a particular verification.
Note The words 'fixed variable actions and permanent actions' should be interpreted to mean 'particular values of variable actions and permanent actions'.

Maintenance – Set of pre-planned activities performed during the working life of the structure in order to enable it to fulfil the requirements for reliability.
Note Activities to restore the structure after an accidental or seismic event are normally outside the scope of maintenance.

Nominal value – Value fixed on non-statistical basis, for instance on acquired experience or on physical conditions.

Persistent design situation – Design situation that is relevant during a period of the same order as the design working life of the structure. Generally it refers to conditions of normal use.

Reliability – Ability of a structure or a structural member to fulfil the specified requirements, including the design working life, for which it has been designed. Reliability is usually expressed in probabilistic terms.
Note Reliability normally covers safety, serviceability and durability of a structure.

Repair – Activities performed to preserve or to restore the function of a structure that fall outside the definition of maintenance.

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.

Reversible serviceability limit states – Serviceability limit states where no consequences of actions exceeding the specified service requirements will remain when the actions are removed.

Seismic design situation – Design situation involving exceptional conditions of the structure when subjected to a seismic event.

Serviceability criterion – Design criterion for a serviceability limit state.

Serviceability limit states – States that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met e.g. deflection, dynamic movement.

Strength – Mechanical property of a material indicating its ability to resist actions, usually given in units of stress.

Transient design situation – Design situation that 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 temporary conditions of the structure, of use, or exposure, e.g. during construction or repair.

Ultimate limit states – States associated with collapse or with other similar forms of structural failure.
Note They generally correspond to the maximum load-carrying resistance of a structure or structural member.

2.4 Terms relating to actions

Accidental action (*A*) – Action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the 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 variable or accidental actions, depending on the available information on statistical distributions.

Accompanying value of a variable action (ψQ_k) – Value of a variable action that accompanies the leading action in a combination
Note The accompanying value of a variable action may be the combination value, the frequent value or the quasi-permanent value.

Action (*F*)
(a) Set of forces (loads) applied to the structure (direct action)
(b) Set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).

Characteristic value of an action (F_k) – Principal representative value of an action.
Note In so far as a characteristic value can be fixed on statistical bases, it is chosen so as to correspond to 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.

Combination of actions – Set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions.

Combination value of a variable action ($\psi_0 Q_k$) – Value chosen – in so far as it can be fixed on a statistical basis – so that the probability that the effects caused by the combination will be exceeded is approximately the same as by the characteristic value of an individual action.

Design value of an action (F_d) – Value obtained by multiplying the representative value by the partial factor γ_f.
Note The product of the representative value multiplied by the partial factor $\gamma_F = \gamma_{Sd} \times \gamma_f$ may also be designated as the design value of the action.

Dynamic action – Action that causes significant acceleration of the structure or structural members.

Effect of action (*E*) – Internal force, moment, stress, strain, deflection, rotation etc on structural members, or on the whole structure.

Fixed action – Action that has a fixed distribution and position over the structure or structural member.

Free action – Action that may have various spatial distributions over the structure.

Frequent value of a variable action ($\psi_1 Q_k$) – Value determined – in so far as it can be fixed on a statistical basis – so that either the total time, within the reference period, during which it is exceeded is only a small given part of the reference period, or the frequency of it being exceeded is limited to a given value.

Geotechnical action – Action transmitted to the structure by the ground, fill or groundwater.

Permanent action (G) – Action that 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 limit value.
Note Permanent actions normally refer to what are termed dead loads in the UK e.g. self-weight of structures and fixed equipment.

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.

Quasi-static action – Dynamic action represented by an equivalent static action in a static model.

Reference period – Chosen period of time that is used as a basis for assessing statistically variable actions, and possibly for accidental actions.

Representative value of an action (F_{rep}) – Value used for the verification of a limit state. A representative value may be the characteristic value (F_k) or an accompanying value (ψF_k).

Seismic action (A_E) – Action that arises due to earthquake ground motions.

Single action – Action that can be assumed to be statistically independent in time and space of any other action acting on the structure.

Static action – Action that does not cause significant acceleration of the structure or structural members.

Variable action (Q) – Action for which the variation in magnitude with time is neither negligible nor monotonic.
Note Variable actions normally refer to imposed loads in the UK e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads. Such actions are not monotonic in that they may increase or decrease with time.

2.5 Terms relating to material and product properties and geometrical data

Characteristic value (X_k or R_k) – Value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances.

Characteristic value of a geometrical property (a_k) – Value usually corresponding to the dimensions specified in the design. Where relevant, values of geometrical quantities may correspond to some prescribed fractiles of the statistical distribution.

Design value of a geometrical property (a_d) – Generally a nominal value. Where relevant, values of geometrical quantities may correspond to some prescribed fractile of the statistical distribution.
Note The design value of a geometrical property is generally equal to the characteristic value. However, it may be treated differently in cases where the limit state under consideration is very sensitive to the value of the geometrical property, for example when considering the effect of geometrical imperfections on buckling. In such cases, the design value will normally be established as a value specified directly, for example in an appropriate European Standard. Alternatively, it can be established from a statistical basis, with a value corresponding to a more appropriate fractile (e.g. a rarer value) than applies to the characteristic value.

Design value of a material or product property (X_d or R_d) – Value obtained by dividing the characteristic value by a partial factor γ_m or γ_M, or, in special circumstances, by direct determination.

2.6 Requirements

2.6.1 Basic requirements

The basic requirements are for safety, serviceability, fire resistance and robustness.

This Section should be read with Section 1.2.3 and the first paragraph of Section 2.6.4. Attention is particularly drawn to the final paragraph of this Section.

EC0 requires a structure to be designed and executed in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economical way:

- sustain all actions and influences likely to occur during execution and use. (This is an EC0 ultimate limit state requirement. 'Actions and influences' are those defined by or inferred from the design assumptions for the structure)
- remain fit for the use for which it is required. (This is an EC0 serviceability requirement and should be interpreted to mean 'the structure and its elements should meet the serviceability requirements defined by or inferred from the design assumptions').

The structure will need to be designed to have adequate:

- structural resistance
- serviceability performance
- durability.

In addition, the designer should address constructability.

In the case of fire, the structural resistance should be adequate for the required period of time. See Chapter 4 for actions on structures exposed to fire.

Regarding robustness, EC0 requires a structure to be designed and executed in such a way that it will not be damaged by events such as:

- explosion
- impact
- the consequences of human errors

to an extent disproportionate to the original cause. The events to be taken into account to meet this requirement should be agreed for an individual project with the client and the relevant authority. See Chapter 9 for accidental actions.

The basic requirements given above should be met by the following (all with the particular project in mind):

- the choice of suitable materials
- appropriate design and detailing
- specifying control procedures for design, production, execution, and use.

In accordance with EC0 the provisions of Section 2.6.1 of this *Manual* should be interpreted on the basis that due skill and care appropriate to the circumstances are exercised in the design, based on such knowledge and good practice as are generally available at the time the design of the structure is carried out.

2.6.2 Reliability management

The reliability required for structures within the scope of this *Manual* should be achieved:
– by design in accordance with EC0 and EC1 to EC9, or their corresponding Institution Manuals[17-20], and
– by appropriate execution and quality management measures.

For the range of structures for which this *Manual* is intended, it is recommended that the following quality control measures are observed so that errors in design and execution, and gross human errors, are reduced:
– The design should be checked by different persons than the originators and in accordance with the relevant quality assurance/checking procedures of the organisation. Those persons should have experience of the design, implementation and operation of similar types of construction work.
– The design should be communicated to the contractor in a clear and unambiguous way.
– During construction, adequate inspection should be made to see that the design intent is achieved.

2.6.3 Design working life

For the structures within the scope of this *Manual* a design working life of 50 years should generally be assumed, unless otherwise required by the client.

2.6.4 Durability

To achieve durability, EC0 requires the structure to be designed such that deterioration over its design working life does not impair its performance, having due regard to its environment and the anticipated level of maintenance. The following inter-related factors should be considered in the design:
– the intended or foreseeable use of the structure
– the required design criteria
– the expected environmental conditions
– the composition, properties and performance of the materials and products
– the properties of the soil
– the choice of the structural system
– the shape of members and the structural detailing
– the quality of workmanship, and the level of control
– the particular protective measures
– the intended maintenance during the design working life.

2.6.5 Quality management

In order to provide a structure that corresponds to the requirements and design assumptions, appropriate quality management measures should be in place. These measures include:
– defined reliability requirements
– organisational measures, including defined responsibilities for the various parties and individuals
– controls at the stages of design, execution, use and maintenance.

2.7 Principles of limit states design

2.7.1 Limit state principles and design situations

When using the Eurocodes the design needs to be verified using limit state principles. A distinction between ultimate limit states (see Section 2.7.2) and serviceability limit states (see Section 2.7.3) is made.

Verification of one of the two limit states may be omitted provided that sufficient information is available to prove that it is satisfied by the other.

Design should be based on the use of structural and load models for relevant limit states and it should be verified that no limit state is exceeded when relevant design values for:
– actions
– material properties, or
– product properties, and
– geometrical data
are used in these models. This requirement should be achieved by the partial factor method, described in Sections 2.9 to 2.11.

Limit states should be verified for the relevant design situations (see Sections 2.7.2 and 2.7.3) taking into account the circumstances under which the structure is required to fulfil its function and that can reasonably be foreseen to occur during execution and use.

2.7.2 Ultimate limit states

Ultimate limit states concern:
– the safety of people, and/or
– the safety of the structure.

In some circumstances, to be agreed for a particular project with the client and the relevant authority, the limit states that concern the protection of contents should also be classified as ultimate limit states.

The following design situations should be taken into account for ultimate limit state verification:
– persistent design situations, which refer to the conditions of normal use
– transient design situations, which refer to temporary conditions applicable to the structure, e.g. during execution or repair
– accidental design situations, which refer to exceptional conditions applicable to the structure or to its exposure, e.g. to fire, explosion, impact or the consequences of localised failure.

The following ultimate limit states need to be verified where they are relevant:
– loss of equilibrium of the structure or any part of it, considered as a rigid body
– failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of stability of the structure or any part of it, including supports and foundations.

2.7.3 Serviceability limit states

Serviceability limit states concern:
– the functioning of the structure or structural members under normal use
– the comfort of people
– the appearance of the construction works.

Normally the serviceability requirements are those agreed for each individual project. A distinction should be made between reversible and irreversible serviceability limit states (see Section 2.3).

The verification of serviceability limit states should be based on:
(a) deformations that:
  – affect the appearance
  – affect the comfort of users
  – affect the functioning of the structure (including the functioning of machines or services), or
  – cause damage to finishes or non-structural members.
(b) vibrations that:
  – cause discomfort to people
  – limit the functional effectiveness of the structure, or
  – cause damage to finishes or non-structural members.
(c) damage that is likely to adversely affect:
  – the appearance
  – the durability, or
  – the functioning
  – of the structure.

2.8 Actions

2.8.1 General and characteristic values of actions

The actions to be used in the design are:
– permanent actions *(G)*, e.g. self-weight of structures and fixed equipment
– variable actions *(Q)*, e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads
– accidental actions *(A),* e.g. explosions and impact.

The characteristic value F_k of an action is specified as a mean value, an upper or lower value, or a nominal value when it does not refer to a known statistical distribution.

2.8.2 Characteristic value of a permanent action (G_k)

The characteristic value of a permanent action (G_k) should be assessed as follows:
– if the variability of G (e.g. for quality controlled factory produced members or for construction works with a reasonable level of quality control) can be considered as small, one single value G_k may be used
– if the variability of G cannot be considered as small, two values may need to be used where the results of a verification are very sensitive to variations in the magnitude of a permanent action from place to place in the structure: an upper value $G_{k,sup}$ and a lower value $G_{k,inf}$. Supplementary advice is given in Section B.1 of Appendix B for the determination of $G_{k,sup}$ and $G_{k,inf}$.

Where a single value of G_k is used, the self-weight of the structure should be calculated on the basis of the nominal dimensions and mean unit masses.

For the majority of structures designed in accordance with this *Manual*, it is envisaged that the single G_k value approach will be valid. Therefore, further consideration of $G_{k,sup}$ and $G_{k,inf}$ is confined to Appendix B of this *Manual.*

2.8.3 Values of variable actions (Q_k)

A variable action has four representative values to be used for the appropriate design situations for the ultimate or serviceability limit state verifications. These are:
– its characteristic value Q_k
– the combination value $\psi_0 Q_k$
– the frequent value $\psi_1 Q_k$
– the quasi permanent value $\psi_2 Q_k$.

These values are comprehensively described in Section B.2 of Appendix B.

2.8.4 Accidental actions (A_d)

The design value A_d of an accidental action is normally a nominal value and should be specified for individual projects. See Chapter 9.

2.8.5 Geotechnical actions

For geotechnical actions see EC7.

2.9 Verification by the partial factor method

It should be verified that, for all the relevant design situations, no limit state is exceeded when design values for actions or effects of actions and resistances are used in the design models.

For the selected design situations and the relevant limit states the individual actions (i.e. the characteristic, or other representative values) for the critical load cases should be combined as detailed in this Chapter. Actions that cannot occur simultaneously, for example due to physical reasons, do not need to be considered together in combination.

2.10 Ultimate limit states

2.10.1 General

For the scope of structures covered by this *Manual* the following ultimate limit states need to be verified as appropriate:

EQU – Loss of static equilibrium of the structure (see Note below) or any part of it considered as a rigid body, where the strength of construction materials or the ground is generally not governing. The EQU case should consider situations where minor variations in the value or the spatial distribution of actions might be significant in the context of loss of equilibrium;

STR – Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs;

GEO – Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance.

Note EQU is used to verify the stability of the structure or parts of it e.g. against overturning, sliding or uplift.

The design values of actions for each of the limit states EQU, STR and GEO should be in accordance with Tables 2.1 to 2.4. Table 2.1 also includes reference to the accidental design situation. This is dealt with in Section 2.10.3.5 and Table 2.5.

Table 2.1 Sources of partial factors for Ultimate Limit State combinations

Design situation	Expression in EC0	Simplified Combination Expression in this *Manual*	Table in this *Manual* for obtaining γ factors
Persistent/transient EQU	6.10	2.1	Table 2.2
Persistent/transient STR Superstructure elements only i.e. foundations not considered	6.10[a]	2.2	Table 2.3
Persistent/transient GEO component of STR/GEO for determining external dimensions of foundation elements e.g. footings, piles	6.10	2.3	Table 2.4
Persistent/transient STR component of STR/GEO for determining strength of substructure or foundation elements	6.10	2.2	Table 2.3
Accidental	6.11b	2.4	Table 2.5[b]

Notes

a Expressions 6.10a/6.10b given in Table C.1 may be used as an alternative (see Section 2.10.3.1).

b All γ factors are unity for the accidental situation.

Table 2.2 Partial factors and combinations for EQU

Persistent and transient design situations	Permanent actions[a,b]		Leading variable action[c]	Accompanying variable actions[c]
	Unfavourable	Favourable		
Expression 2.1 in this *Manual*	**1.10** $G_{k,j}$	**0.90** $G_{k,j}$	**1.5** $Q_{k,1}$ (0 when favourable)	**1.5** $\psi_{0,i}\, Q_{k,i}$ (0 when favourable)

Notes

a This table assumes low variability in G_k and therefore that $G_{k,sup}$ and $G_{k,inf}$ need not be used (see Section 2.8.2).

b The single source principle for permanent loads does not apply for EQU. Each combination should therefore allow for favourable and unfavourable dispositions of permanent actions (see Section 2.10.3.1).

c Variable actions are those listed in Table 2.6.

Table 2.3 Partial factors and combinations for STR and the STR component of STR/GEO

Persistent and transient design situations	Permanent actions[a, b]		Leading variable action[c]	Accompanying variable actions[c]
	Unfavourable	Favourable		
Expression 2.2 in this *Manual*	**1.35** $G_{k,j}$	**1.00** $G_{k,j}$	**1.5** $Q_{k,1}$ (0 when favourable)	**1.5** $\psi_{0,i}$ $Q_{k,i}$ (0 when favourable)

Notes

a This table assumes low variability in G_k and therefore that $G_{k,sup}$ and $G_{k,inf}$ need not be used (see Section 2.8.2).

b The single source principle (see Section 2.10.3.1) applies in this case. Therefore, the characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,j} = 1.35$ if the total resulting action effect is unfavourable; $\gamma_{G,j} = 1.00$ 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. e.g. several concrete spans in a floor interrupted by a steel span.

c Variable actions are those listed in Table 2.6.

Table 2.4 Partial factors and combinations for the GEO component of STR/GEO

Persistent and transient design situations	Permanent actions[a]		Leading variable action[b]	Accompanying variable actions[b]
	Unfavourable	Favourable		
Expression 2.3 in this *Manual*	**1.0** $G_{k,j}$	**1.00** $G_{k,j}$	**1.3** $Q_{k,1}$ (0 when favourable)	**1.3** $\psi_{0,i}$ $Q_{k,i}$ (0 when favourable)

Notes

a This table assumes low variability in G_k and therefore that $G_{k,sup}$ and $G_{k,inf}$ need not be used (see Section 2.8.2).

b Variable actions are those listed in Table 2.6.

Table 2.5 Partial factors and combinations for the accidental design situation

Accidental Design situation	Permanent actions[a]		Leading accidental action	Accompanying variable actions[b]	
	Unfavourable	Favourable		Main (if any)	Others
Expression 2.4 in this *Manual*	$G_{k,j}$	$G_{k,j}$	A_d	$\psi_{1,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$

Notes

a This table assumes low variability in G_k and therefore that $G_{k,sup}$ and $G_{k,inf}$ need not be used (see Section 2.8.2).

b Variable actions are those listed in Table 2.6

Figure 2.1 illustrates the use of EQU, STR and GEO limit states in relation to a simple structure.

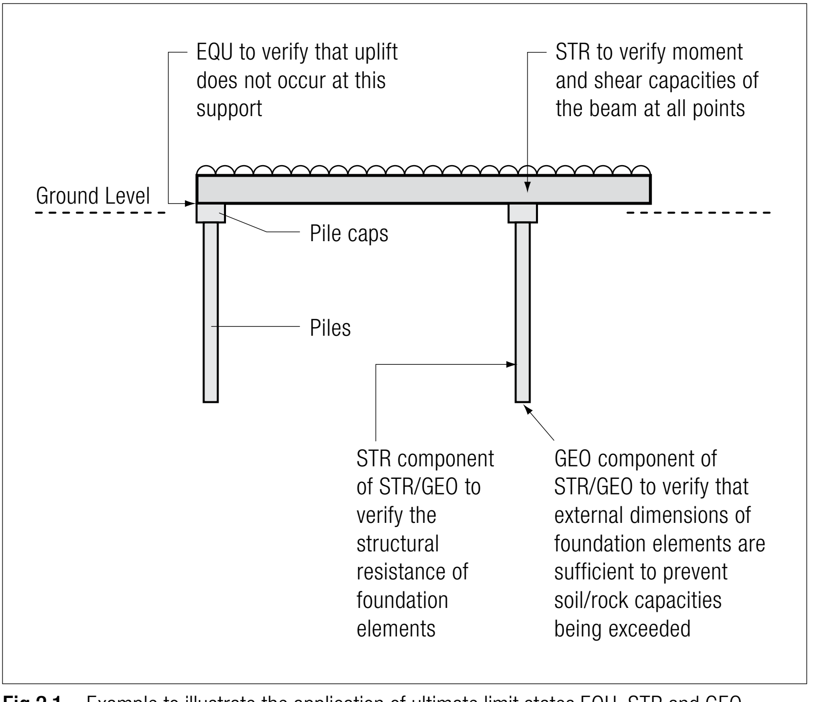


Fig 2.1 Example to illustrate the application of ultimate limit states EQU, STR and GEO

2.10.2 Verification of static equilibrium and resistance

2.10.2.1 Static equilibrium

When considering a limit state of static equilibrium of the structure (EQU), the verification will need to ensure that:

$$E_{d,dst} \leqslant E_{d,stb} \qquad \text{(6.7 in EC0)}$$

where:

$E_{d,dst}$ is the design value of the effect of destabilising actions
$E_{d,stb}$ is the design value of the effect of stabilising actions.

2.10.2.2 Resistance

When considering a limit state of rupture or excessive deformation of a section, member or connection (STR for the superstructure and the STR component of STR/GEO for the structural design of the foundations), the verification will need to ensure that:

$$E_d \leqslant R_d \qquad \text{(6.8 in EC0)}$$

where:

E_d is the design value of the effect of actions such as moment or a vector representing several internal forces or moments
R_d is the design value of the corresponding resistance.

2.10.3 Combination of actions

2.10.3.1 General

This section explains the expressions used in the Eurocode system for combining actions and their effects.

It is assumed that expression 6.10 from EC0 will be used for the majority of structures designed in accordance with this *Manual*. Expression 6.10 is given in Section 2.10.3.3. An explanation of it and simplified forms of it are also given in Section 2.10.3.3. These simplified forms should prove sufficient for most of the structures falling within the scope of this *Manual*.

Guidance is given for ultimate limit state combinations of actions.

Partial safety factors (γ factors) are summarised in Section 2.10.4. Values of these factors for combinations of favourable and unfavourable actions are presented there.

The Eurocode system introduces additional factors called combination factors (ψ factors). Values of these factors are given in Table 2.6 and their use is illustrated in this Section and in Section 2.10.4.

EC0 also presents expressions 6.10a and 6.10b, which, as a pair, may be used instead of expression 6.10 for the STR limit state. Consideration of expressions 6.10a and 6.10b is confined to Appendix C of this *Manual*. However, it should be noted that the use of these expression will, in most circumstances, lead to a more economic design.

The concept of $G_{k,sup}$ and $G_{k,inf}$ was introduced in Section 2.8.2 and it was noted that all further consideration of that concept is confined to Appendix B in this *Manual.*

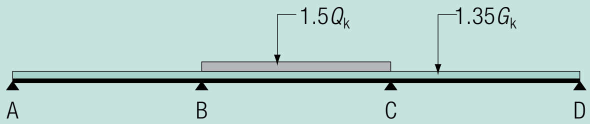
A different, though related issue is the 'single source principle' for permanent actions in EC0. It must be stressed that this principle applies only to STR and GEO verifications and not to EQU verifications. The principle states that all permanent actions from one source are assigned the same value of partial factor γ in any one load combination.

Example 2.1 illustrates the use of the single source principle.

Example 2.1: Single Source Principle

Consider a three span beam ABCD as shown and STR verification:

For maximum midspan sagging moment on span BC, the single source principle demands the following:



i.e. G_k attracts γ = 1.35 throughout, in spite of the fact that a more adverse midspan sagging moment would be obtained in span BC if reduced values of γ were to be applied on spans AB and CD.

Note, however, that in the case of EQU verification, different γ factors would be used in favourable and unfavourable areas i.e. the single source principle would not apply.

For each critical load case, the design values of the effects of actions (E_d) will need to be determined by combining the values of actions. All actions that can exist simultaneously should be considered together in combination. Actions that cannot exist simultaneously due to physical or functional reasons should not be considered together in combination.

Each combination of actions should include:
– a leading variable action, for the verification of persistent and transient design situations, or
– an accidental action, for the verification of the accidental design situation.

In each of these cases, other variable actions might be present. Self-weight actions will generally be present in all combinations.

The combinations of actions for verifying ultimate limit states should be in accordance with Sections 2.10.3.3 and 2.10.4.

2.10.3.2 Sensitivity to the magnitude of a permanent action

Where the results of a verification are very sensitive to variations in the magnitude of a permanent action from place to place in the structure (e.g. for the verification of static equilibrium), the unfavourable and favourable parts of this action need to be considered as individual actions.

2.10.3.3 Combinations of actions for persistent or transient design situations (fundamental combinations)

The combination of actions for the Eurocodes is given in expression 6.10 of EC0:

$$\sum_{j \geqslant 1} \gamma_{G,j} G_{k,j} \text{ " + " } \gamma_p P \text{ " + " } \gamma_{Q,1} Q_{k,1} \text{ " + " } \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(6.10 in EC0)}$$

where:
" + " means 'to be combined with'
Σ means 'the combined effect of'

Tables 2.2 to 2.4 give the above expressions in tabular form.

With regard to the characteristic live loads (variable actions) Q_k, where only one variable action is being considered in the combination, this is chosen to be $Q_{k,1}$. When a number of variable actions act simultaneously, the leading action is chosen as $Q_{k,1}$ and the other actions (accompanying) are chosen as $Q_{k,i}$ and are reduced by an appropriate combination factor. Where it is not obvious which should be the leading variable action, each action should be checked in turn and the worst case taken. Characteristic values are used for $Q_{k,1}$ and $Q_{k,i}$.

Figure 2.2 explains the constituent elements of Expression 6.10 for combining the effects of actions in the Eurocode system.

As an alternative to the normal approach given in EC0 for the combination of actions, simplified representations are given below. These are likely to be applicable to the majority of structures designed in accordance with this *Manual*. The designer must ensure in applying the simplified expressions below that all actions that can exist simultaneously are considered together in combination.

Design effect | Permanent actions | Prestress (where present) | Leading variable action | Accompanying variable actions

$$E_d = E\left[\sum_{j\geqslant 1}\gamma_{G,j}\,G_{k,j}\text{ " + " }\gamma_P P\text{ " + " }\gamma_{Q,1}\,Q_{k,1}\text{ " + " }\sum_{i\geqslant 1}\gamma_{Q,i}\,\psi_{0,i}\,Q_{k,i}\right]$$

Effect of | Combined with

Fig 2.2 The constituent elements of expression 6.10

(a) **Limit state EQU**

For *destabilising effects* where both G_k and Q_k are destabilising and assuming *two* variable actions, the combination of actions E_d becomes

$$E_d = 1.1G_k \text{ " + " } 1.5Q_{k,1} \text{ " + " } 1.5\psi_0 Q_{k,2} \qquad (2.1)$$

where:

G_k is the self-weight

$Q_{k,1}$ is the leading variable action, and

$Q_{k,2}$ is the accompanying variable action.

If G_k is a stabilising action its accompanying γ value in expression 2.1 reduces to 0.9 (see Table 2.2); if Q_k is a stabilising action, it is taken as zero (see Table 2.2).

In accordance with Table 2.6, $\psi_0 = 0.7$ for most imposed loads and 0.5 for snow loads (site altitude < 1000m above sea level) and 0.5 for wind actions.

Table 2.6 Values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see Chapter 3)			
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, vehicle weight ⩽ 30kN	**0.7**	**0.7**	**0.6**
Category G: traffic area, 30kN< vehicle weight ⩽ 160kN	**0.7**	**0.5**	**0.3**
Category H: roofs	**0.7**	**0**	**0**
Snow loads on buildings (see Chapter 5)			
– for sites located at altitude H > 1000m a.s.l.	**0.70**	**0.50**	**0.20**
– for sites located at altitude H ⩽ 1000m a.s.l.	**0.50**	**0.20**	**0**
Wind loads on buildings (see Chapter 6)	**0.5**	**0.2**	**0**
Temperature (non-fire) in buildings (see Chapter 7)	**0.6**	**0.5**	**0**

Note
Accidental actions (see Chapter 9) are not variable actions and are always considered as lead actions (see Section 2.10.3.5) and hence should never be used with any reduction coefficients (e.g. ψ).

(b) Limit state STR for superstructure elements

For *unfavourable effects* and assuming *two* variable actions, the combination of actions E_d becomes

$$E_d = 1.35G_k \text{ " + " } 1.5Q_{k,1} \text{ " + " } 1.5\psi_0 Q_{k,2} \tag{2.2}$$

where:
G_k is the self-weight
$Q_{k,1}$ is the leading variable action, and
$Q_{k,2}$ is the accompanying variable action.

If G_k is favourable its accompanying γ value in expression 2.2 reduces to 1.0 (see Table 2.3); if Q_k is favourable, it is taken as zero (see Table 2.3). However, the single source principle applies to G_k (see Section 2.10.3) and therefore in any one combination G_k should be assigned a single γ value.

In accordance with Table 2.6, $\psi_0 = 0.7$ for most imposed loads and 0.5 for snow loads (site altitude < 1000m above sea level) and 0.5 for wind actions.

Example 2.2 illustrates the use of leading and accompanying actions in STR verification.

Example 2.2: Single Source Principle
Consider a structural element where the variable actions that can exist simultaneously are the imposed load and the wind action. Verify the limit state STR for the superstructure for the unfavourable case.

Expression 2.2 is used:
$E_d = 1.35G_k$ " + " $1.5Q_{k,1}$ " + " $1.5\psi_0 Q_{k,2}$

When the imposed load is the leading action $Q_{k,1}$ and the wind action is the accompanying action $Q_{k,2}$

$E_d = 1.35G_k$ " + " $1.5Q_{k,1}$ " + " $(1.5 \times 0.5)Q_{k,2}$

where $\psi_0 = 0.5$ for wind actions

When the wind load is the leading action $Q_{k,1}$ and the imposed load is the accompanying action $Q_{k,2}$

$E_d = 1.35G_k$ " + " $1.5Q_{k,1}$ " + " $(1.5 \times 0.7)Q_{k,2}$

where $\psi_0 = 0.7$ for imposed loads

This differs from current UK practice where partial safety factors would be 1.2 for self-weight and each variable action.

(c) Limit state STR/GEO for substructure and foundation elements

(i) For sizing the element (the GEO component of STR/GEO)

For un*favourable effects* and assuming *two* variable actions, the combination of actions E_d becomes

$$E_d = 1.0G_k \text{ " + " } 1.3Q_{k,1} \text{ " + " } 1.3\psi_0 Q_{k,2} \qquad (2.3)$$

where:
G_k is the self-weight
$Q_{k,1}$ is the leading variable action, and
$Q_{k,2}$ is the accompanying variable action.

In the favourable situation, G_k retains its 1.0 partial factor (see Table 2.4); if Q_k is favourable, it is taken as zero (see Table 2.4).

In accordance with Table 2.6, $\psi_0 = 0.7$ for most imposed loads and 0.5 for snow loads (site altitude < 1000m above sea level) and 0.5 for wind actions.

(ii) For determining structural resistance of substructure or foundation elements (the STR component of STR/GEO)

This situation is dealt with under Section 2.10.3.3 (b) above i.e. expression 2.2 is used.

2.10.3.4 Examples illustrating combinations of actions for the EQU, STR and GEO ultimate limit states

Example 2.3 shows EQU, STR and GEO combinations for a cantilever beam.

Example 2.3: Ultimate limit state (EQU, STR and GEO) verification for a cantilever beam

A cantilevered beam comprises a reinforced concrete tee beam with cross-sectional area 1m^2 and flange width 3m. Assume low variability in self-weight, giving a single value of G_k = 25kN/m. Office loading (including partitions – see Table 3.1 and Section 3.7.3.3) for which Q_k = 10kN/m. Prestress is not present and geotechnical actions are not present.

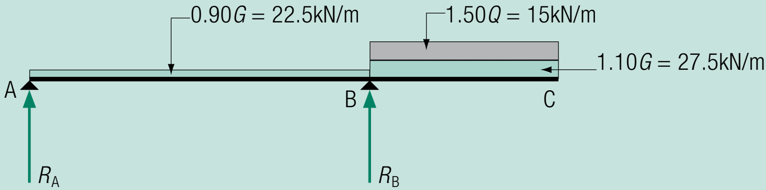
The objective of this example, which is illustrative, is to show the load arrangements on spans AB and BC for ultimate limit states EQU, STR and GEO using expressions 2.1 to 2.3.

The single source principle explained in Section 2.10.3.1 and in Table 2.3 Note b has been used for self-weight for the STR and GEO limit states.

Static equilibrium verification (limit state EQU)
This case investigates the possibility of loss of static equilibrium of the beam, i.e. uplift at A (see 2.10.3.3 (a)).

EQU Load Case

EQU: The most onerous case is with:
Span BC loaded with imposed load Qk
G_k Unfavourable situation on span BC (destabilising effect)
G_k Favourable situation on span AB (stabilising effect)



From statics,

$R_A = 51.4\text{kN} > 0$

Therefore, no uplift at A.

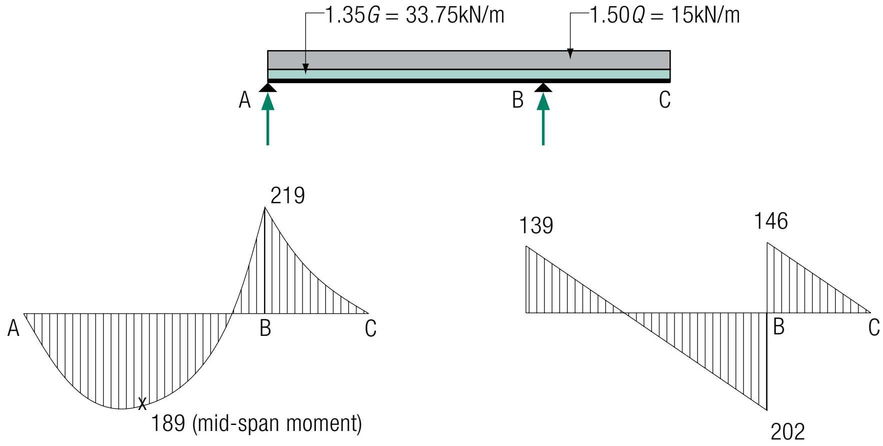
Notes
51.4kN is not the force for which support at A would be designed. In this example the value of R_A is not critical. However, for EQU to be satisfied, it should be shown that R_A is greater than zero.
The single source principle does not apply to EQU, hence different values of factored self-weight have been used on the two spans.

Forces for member strength verification (limit state STR)
This case will be used to determine:
- shear forces and bending moments in the beam (STR for superstructure elements, see 2.10.3.3 (b), and
- support reactions (STR component of STR/GEO for structural resistance of substructure and foundation elements, see 2.10.3.3 (c)(ii)).

Different loading cases need to be considered to determine peak values at different points on the beam. These cases are shown below.

STR Load Cases (1) and (2)

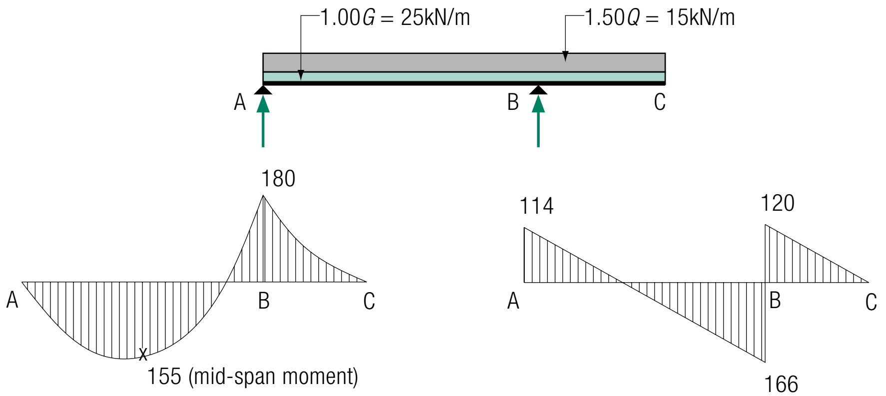
STR(1): Both spans AB and BC loaded with imposed load Q_k
G_k Unfavourable situation



Bending moments (kNm)

Shear forces (kN)

STR(2): Both spans AB and BC loaded with imposed load Q_k
G_k Favourable situation



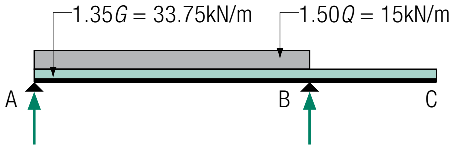
Bending moments (kNm)

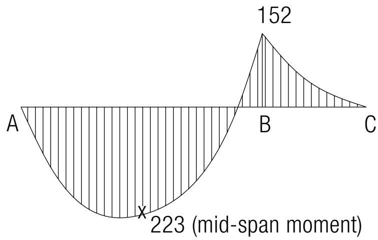
Shear forces (kN)

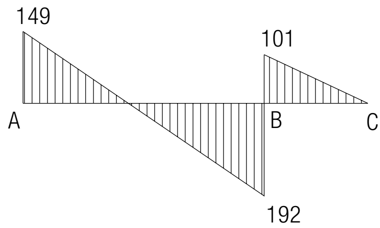
Note
By inspection, Case STR(2) is clearly less critical than Case STR(1) for any part of the beam. It has been included here for the sake of clarity and completeness.

STR Load Cases (3) and (4)

STR(3): Only span AB loaded with imposed load Q_k
G_k Unfavourable situation



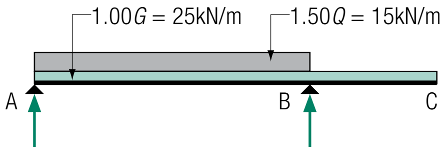


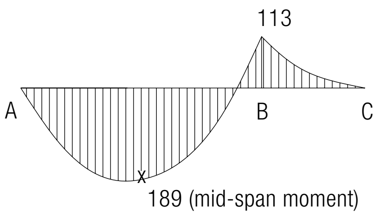


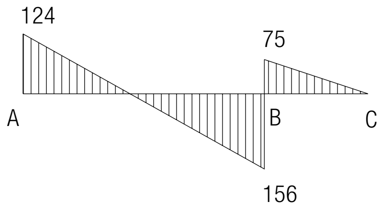
Bending moments (kNm)

Shear forces (kN)

STR(4): Only span AB loaded with imposed load Q_k
G_k Favourable situation







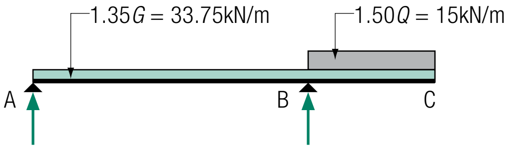
Bending moments (kNm)

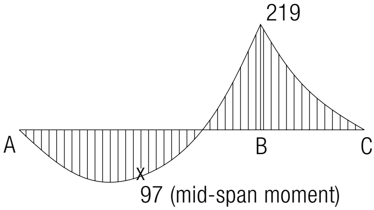
Shear forces (kN)

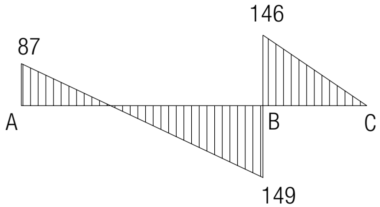
Note
Case STR(4) is less critical everywhere than Case STR(3).

STR Load Cases (5) and (6)

STR(5): Only span BC loaded with imposed load Q_k
G_k Unfavourable situation



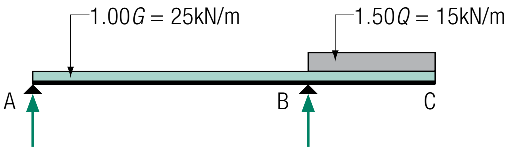


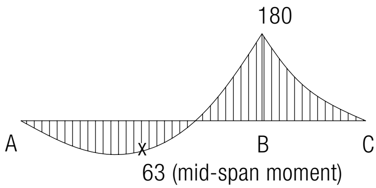


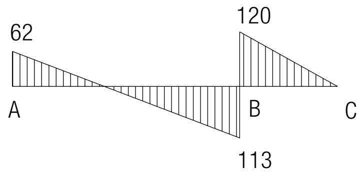
Bending moments (kNm)

Shear forces (kN)

STR(6): Only span BC loaded with imposed load Q_k
G_k Favourable situation







Bending moments (kNm)

Shear forces (kN)

Note
Case STR(6) is less critical everywhere than Case STR(5).

The maximum effects resulting from STR checks:

Shear force and bending moment on BC at B	– from Case STR(1)
Bending moment at mid-span AB	– from Case STR(3)
Support reaction at A	– from Case STR(3)
Support reaction at B	– from Case STR(1)

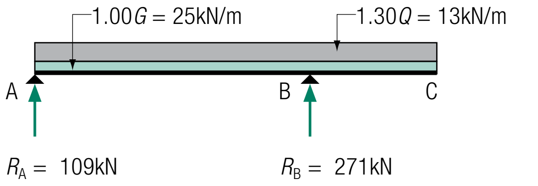
(Note that the above support reactions at A and B would be used for the structural design of piles in those locations.)

Forces for determination of external dimensions of foundations as controlled by soil/rock characteristics (limit state GEO)

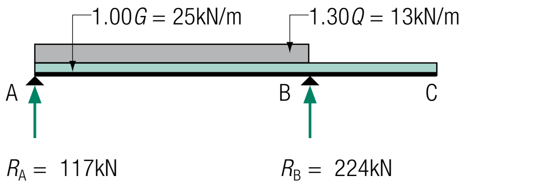
Different loading cases need to be considered to determine the maximum support reactions that should be used to determine the external dimensions of the foundations (GEO component of STR/GEO for sizing the substructure and foundation elements, see 2.10.3.3 (c)(i)) are given below.

GEO Load Cases (1), (2) and (3)

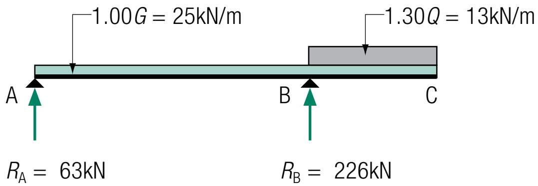
GEO(1): Both spans AB and BC loaded with imposed load Q_k
G_k Unfavourable and favourable situations



GEO(2): Only span AB loaded with imposed load Q_k
G_k Unfavourable and favourable situations



GEO(3): Only span BC loaded with imposed load Q_k
G_k Unfavourable and favourable situations



The maximum effects resulting from GEO checks for sizing foundation elements:
Support reaction at A – from Case GEO(2)
Support reaction at B – from Case GEO(1)

Example 2.4 shows STR combinations for a three span beam.

Example 2.4: STR limit state verification for a three span beam

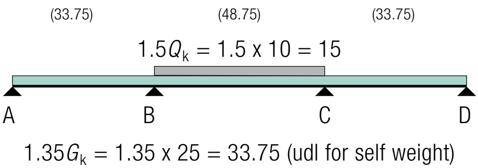
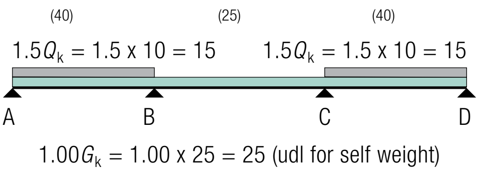
A reinforced concrete tee beam has cross-sectional area 1m^2 and flange width 3m. Assume low variability in self-weight, giving a single value of G_k = 25kN/m. Office loading (including partitions – see Table 3.1 and Section 3.7.3.3) for which Q_k =10kN/m. Prestress is not present and geotechnical actions are not present.

The single source principle explained in Section 2.10.3.1 and in Table 2.3 Note b is used for self-weight. Imposed loads are applied in appropriate locations in the usual way to generate maximum adverse effects.

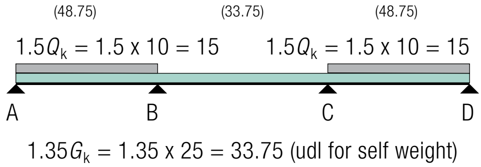
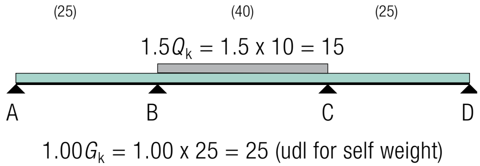
The objective of this example, which is illustrative, is to show the load arrangements for a 3 span beam for determining the effects of actions for the mid-span points for spans AB and BC in accordance with the STR limit state (i.e. expression 2.2 and Table 2.3).

Note All the factored loads in the diagrams below are in kN/m.

Span BC – midspan sagging and hogging

Expression/condition	Application of actions and partial factors	Comments
Expression 2.2 (Unfavourable effect – span BC)		Q_k = 0 for spans AB and CD This gives the maximum sagging moment in span BC
Expression 2.2 (Favourable effect – span BC)		Q_k = 0 for span BC This gives, if any, the maximum hogging moment in span BC

Span AB – midspan sagging and hogging

Expression/condition	Application of actions and partial factors	Comments
Expression 2.2 (Unfavourable effect – span AB)		$Q_k = 0$ for span BC This gives the maximum sagging moment in span AB/CD
Expression 2.2 (Favourable effect – span AB)		$Q_k = 0$ for spans AB and CD This gives, if any, the maximum hogging moment in span AB/CD

Example C.1 in Appendix C presents this same example but using expressions 6.10a/ 6.10b from EC0. Comparison of the above results with those in Example C.1 gives an indication of the differences in values of combined loadings that arise between expressions 6.10 and 6.10a/6.10b.

2.10.3.5 Combinations of actions for accidental design situations

The combination of actions for the accidental design situation is given in expression 6.11b of EC0:

$$\sum_{j \geqslant 1} G_{k,j} \text{ "+" } P \text{ "+" } A_d \text{ "+" } \psi_{1,1} Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad \text{(6.11b in EC0)}$$

Combinations of actions for accidental design situations should either
– involve an explicit accidental action A (fire or impact), or
– refer to a situation after an accidental event ($A = 0$).

Note the Eurocode recommends the use of 6.11b for structures after the accidental event. However, the designer may wish to consider the use of 6.10 instead. (6.11b uses the frequent value of the load, whereas 6.10 uses the characteristic value).

For fire situations, apart from the temperature effect on the material properties, A_d should represent the design value of the indirect thermal action due to fire.

As an alternative to the use of expression 6.11b in EC0, a simplified representation is given in expression 2.4 below. This is likely to be applicable to the majority of structures designed in accordance with this *Manual*. The designer must ensure in applying the simplified expression below that all actions that can exist simultaneously are considered together in combination.

Assuming two variable actions accompany the accidental action A_d, the combination of actions E_d becomes:

$$E_d = G_k \text{ " + " } A_d \text{ " + " } \psi_1 Q_{k,1} \text{ " + " } \psi_2 Q_{k,2} \tag{2.4}$$

where:
G_k is the self-weight
A_d is the accidental action
$Q_{k,1}$ is the leading variable action, and
$Q_{k,2}$ is the accompanying variable action

γ factors are 1.0 for self-weight, accidental actions and variable actions. ψ_1 and ψ_2 can be obtained from Table 2.6 for the leading and accompanying variable actions.

Note that A_d never has ψ factors applied.

Table 2.5 summarises partial factors and combinations for the accidental design situation.

2.10.4 Partial factors for actions and combinations of actions

Values of the γ factors to be used at the ultimate limit state are given in the tables referred to in Table 2.1. Table 2.1 also shows the relevant combination expressions in this *Manual* and in EC0.

The ψ factors are given in Table 2.6.

2.11 Serviceability limit states

2.11.1 General

Serviceability limit states in buildings should take into account criteria related, for example, to floor stiffness, differential floor levels, storey sway and/or building sway and roof stiffness. Stiffness criteria may, for example, be expressed in terms of limits for vertical deflections and sway criteria may be expressed in terms of limits for horizontal displacements.

Serviceability criteria should be specified for each project and agreed with the client.

In essence, EC0 identifies three different serviceability combinations. Each addresses a different aspect of serviceability behaviour and each is assigned its own load combination expression in the Eurocode. The text below and

Appendix D of this *Manual* contain more information on the difference between these three combinations.

In the absence of specific requirements in the Manuals for EC2 to EC6 it is recommended that the following combinations of actions be used to verify particular serviceability requirements:

– The 'characteristic combination' should be used to verify function and damage to structural and non-structural elements (e.g. partition walls). If the functioning of the structure or damage to finishes or non-structural members (e.g. partition walls, claddings) is being considered, account should be taken of those effects of permanent and variable actions that occur after the execution of the member or finish concerned.
– The 'frequent combination' should be used to verify comfort criteria for users, criteria associated with the use of machinery, criteria for the avoidance of water ponding, etc.
– The 'quasi-permanent combination' should be used for criteria associated with the appearance of the structure. Long term deformations due to shrinkage, relaxation or creep should be considered where relevant, and calculated by using the effects of the permanent actions and quasi-permanent values of the variable actions.

2.11.2 Verification

When considering the serviceability limit states of the structure, the verification will need to ensure that:

$$E_{\mathrm{d}} \leqslant C_{\mathrm{d}}$$

where:

E_{d} is the design value of the effect of actions specified in the serviceability criterion, determined on the basis of the relevant combination in Section 2.11.3.

C_{d} is the limiting design value for the relevant serviceability criterion. See Appendix D for a list of typical limiting values for certain serviceability criteria.

2.11.3 Combination of actions

The combinations of actions to be taken into account in the relevant design situations should be appropriate for the serviceability requirements and performance criteria being verified.

This section presents the relevant load combination expressions from EC0. It also presents simplified versions of the EC0 expressions. It is believed that these simplified expressions will prove sufficient for the majority of structures designed in accordance with this *Manual*. Users of this document are, however, reminded of the need to combine all relevant actions.

With regard to the characteristic live loads (variable actions) Q_k, where only one variable action is being considered in the combination, this is chosen to be $Q_{k,1}$. When a number of variable actions act simultaneously, the leading action is chosen as $Q_{k,1}$ and the other actions (accompanying) are chosen as $Q_{k,i}$ and are reduced by an appropriate combination factor. Where it is not obvious which should be the leading variable action, each action should be checked in turn and the worst case taken. Characteristic values are used for $Q_{k,1}$ and $Q_{k,i}$.

For serviceability limit states the partial factors (γ factors) for actions should be taken as 1.0 unless specified otherwise in the Manuals for EC2 to EC6 or in the source Eurocodes EC2 to EC6.

The following notation applies to combinations (a) to (c) below:
G_k is the self-weight
$Q_{k,1}$ is the leading variable action, and
$Q_{k,2}$ is the accompanying variable action

ψ_0, ψ_1 and ψ_2 (the combination factors) can be obtained from Table 2.6 for the leading and accompanying variable actions.

(a) Characteristic combination

The combination expression in EC0 for this case is:

$$\sum_{j \geqslant 1} G_{k,j} \text{ "+" } P \text{ "+" } Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad \text{(6.14b in EC0)}$$

A simplified version of this expression, likely to be applicable to many of the structures designed in accordance with this *Manual*, is:

$$E_d = G_k \text{ "+" } Q_{k,1} \text{ "+" } \psi_0 Q_{k,2} \qquad (2.5)$$

(b) Frequent combination

The combination expression in EC0 for this case is:

$$\sum_{j \geqslant 1} G_{k,j} \text{ "+" } P \text{ "+" } \psi_{1,1} Q_{k,1} \text{ "+" } \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad \text{(6.15b in EC0)}$$

A simplified version of this expression, likely to be applicable to many of the structures designed in accordance with this *Manual*, is:

$$E_d = G_k \text{ "+" } \psi_1 Q_{k,1} \text{ "+" } \psi_2 Q_{k,2} \qquad (2.6)$$

(c) Quasi-permanent combination

The combination expression in EC0 for this case is:

$$\sum_{j \geqslant 1} G_{k,j} \text{ "+" } P \text{ "+" } \sum_{i \geqslant 1} \psi_{2,i} Q_{k,i} \qquad \text{(6.16b in EC0)}$$

A simplified version of this expression, likely to be applicable to many of the structures designed in accordance with this *Manual*, is:

$$E_{\mathrm{d}} = G_{\mathrm{k}} \text{ " + " } \psi_2 Q_{\mathrm{k,1}} \qquad (2.7)$$

Table 2.7 gives the above expressions in tabular form.

Table 2.7 Combinations of actions for the Serviceability Limit State

Combination	Permanent actions G_d	Variable actions Q_d	
		Leading	Others
Characteristic	$G_{k,j}$	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{k,j}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{k,j}$	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

2.11.4 Vertical and horizontal deformations

Vertical and horizontal deformations should be calculated in accordance with the Manuals for EC2 to EC6, and by using the appropriate combinations of actions according to the expressions in Section 2.11.3 and Table 2.7.

In the absence of specific requirements in the Manuals for EC2 to EC6, it is recommended that the indicative values for limiting deflection in Appendix D should be used.

2.11.5 Vibrations

To achieve satisfactory vibration behaviour of buildings and their structural members under serviceability conditions, the following aspects, amongst others, should be considered:
– the comfort of the user
– the functioning of the structure or its structural members (*e.g.* cracks in partitions, damage to cladding, sensitivity of building contents to vibrations)
– the functioning of equipment sensitive to movement.

For the serviceability limit state of a structure or a structural member not to be exceeded when subjected to vibrations, the natural frequency of vibrations of the structure or structural member should be kept above appropriate values which depend upon the function of the building and the source of the vibration, and agreed with the client and/or the relevant authority.

If the natural frequency of vibrations of the structure is lower than the appropriate value specialist advice should be sought.

Possible sources of vibration that should be investigated include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, and wind actions. These and other sources should be specified for each project and agreed with the client.

Chapter 3: Densities, self-weight and imposed loads in buildings

3.1 Scope

This Chapter gives design guidance contained in EN1991-1-1[3] (EC1 Part 1-1) for actions for the structural design of buildings within the scope of this *Manual* for the following subjects:

– Densities of construction materials and stored materials.
– Self-weight of construction works and methods for the assessment of the characteristic values of self-weight of construction works.
– Imposed loads for buildings including characteristic values of imposed loads for floors and roofs according to category of use in the following areas in buildings:
  – residential, social, commercial and administration areas
  – garage and vehicle traffic areas
  – areas for storage and industrial activities
  – roofs
  – helicopter landing areas.
– Barriers or walls having the function of barriers, horizontal forces and additional guidance for vehicle barriers in car parks.

3.2 Terms and definitions

A basic list of definitions is given in Chapter 2: Basis of Structural Design. Specific additional definitions for densities, self-weight and imposed loads in buildings are:

Bulk weight density – The bulk weight density is the overall weight per unit volume of a material, including a normal distribution of micro-voids, voids and pores.
Note In everyday usage this term is frequently abbreviated to 'density' (which is strictly mass per unit volume).

Angle of repose – The angle of repose is the angle which the natural slope of the sides of a heaped pile of loose material makes to the horizontal.

Gross weight of vehicle – The gross weight of a vehicle includes the self-weight of the vehicle together with the maximum weight of the goods it is permitted to carry.

Structural elements – Structural elements comprise the primary structural frame and supporting structures.

Non-structural elements – Non-structural elements are those that 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.

Partitions – Non load bearing walls.

Movable partitions – Movable partitions are those which can be moved on the floor, be added or removed or re-built at another place.

3.3 Classification of actions

3.3.1 General

Classification of actions considers the variation of actions in time and space.

3.3.2 Self-weight

The self-weight of construction works is generally classified as a permanent fixed action. However,
- where this self-weight can vary in time, it should be taken into account by the upper and lower characteristic values (see Section 2.8.2)
- in some cases the self-weight is classified as a free action (e.g. for movable partitions).

Note movable partitions (see Section 3.7.3.3) should be treated as an additional imposed load.

Landscaping (e.g. trees, shrubs, hard/paved surfaces) and earth loads on roofs and terraces have to be considered as permanent actions. In this case the design 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. See also EC7.

3.3.3 Imposed loads

For the design of structures within the scope of this *Manual*, imposed loads are classified as variable free actions, unless otherwise specified.

In the Eurocodes imposed loads are taken into account as quasi-static actions (see Section 2.4). The load models may include dynamic effects if there is no risk of resonance or other significant dynamic response of the structure (see EC2 to EC9).

For situations
– where actions cause significant acceleration of the structure or structural members, these actions are classified as dynamic actions and should be considered using a dynamic analysis; and
– where resonance effects from syncronised rhythmical movement of people or dancing or jumping may be expected, the appropriate load model should be determined and used in dynamic analysis.

Normally these situations will not apply for the design of structures within the scope of this *Manual*, but if required the procedure used in the National Annex for EC1 Part 1-1 and BRE Digest 426[21] may be used.

For forklifts and helicopters, the additional loadings due to masses and inertial forces caused by fluctuating effects need to be taken into account by a dynamic magnification factor φ which is applied to the static load values, as described in Expression 3.4.

3.4 Design situations

3.4.1 General

The relevant permanent and imposed loads to be used for each identified design situation (see Chapter 2) should be determined considering Section 3.4.2 for permanent loads and Section 3.4.3 for imposed loads.

3.4.2 Permanent loads

The total self-weight of structural and non-structural members should be taken into account in combinations of actions as a single action.

For areas where the removal or addition of structural or non-structural elements is intended, the critical load cases will need to be taken into account in the design.

The full range and fluctuations of water levels must be taken into account as appropriate to the relevant design situations. See EC7.

The moisture content of bulk materials should be considered in design situations of buildings used for storage purposes. The values for the densities provided in Annex A of EC1 Part 1-1 are for materials in the dry state.

3.4.3 Imposed loads

3.4.3.1 For ultimate limit state verifications

For areas which are intended to be subjected to different categories of loadings the most critical load case must be used in the design.

In design situations when imposed loads act simultaneously with other variable actions (e.g. actions induced by wind or snow) the total imposed loads considered in the load case can be considered as a single action.

On roofs, imposed loads need not be applied together simultaneously with snow loads or wind actions.

For an imposed load considered as an accompanying action in the relevant design situation only one of the two factors ψ (Table 2.6) and α_n (Section 3.7.3.4(b) should be applied.

3.4.3.2 For serviceability limit state verifications

The imposed loads to be considered for serviceability limit state verifications should be specified in accordance with the service conditions and the requirements concerning the performance of the structure.

3.5 Specific weights of construction and stored materials

Suitable characteristic values of specific weights (in kN/m³) for the more commonly used construction materials are given below.

Plain concrete		**24**	
Reinforced concrete		**25**	
Steel		**78.5**	
Timber	**(oak)**	**7.2 – 9.6**	
	(pitch pine)	**6.7**	
	(Douglas fir)	**5.3**	
Granite		**26.0 – 29.3**	
Sandstone		**22.0 – 24.0**	
Slate		**28.3**	
Aluminium		**27.7**	
Asphalt		**20.8**	
Clay brickwork		**15.0 – 17.0**	**(gross i.e. including voids)**
Facing brickwork		**13.0 – 22.5**	**(gross)**
Engineering brickwork		**21.0 – 24.0**	**(gross)**
Aircrete masonry		**4.5 – 8.0**	**(solid)**

Aggregate blocks (lightweight)		**8.5 – 11.0**	**(solid)**
Aggregate blocks (medium)		**14.0**	**(solid)**
Aggregate blocks (dense/architectural)		**19.0 – 21.0**	**(solid)**
Glass	**(plate)**	**27.9**	
Lead		**113.3**	
Plaster	**(gypsum)**	**19.2**	
Water		**10**	

The above specific weights have been calculated from densities by using a value for gravity of 10m/s^2.

Specific weights of roof and floor coverings can be obtained from proprietary sources.

A comprehensive list for ranges of characteristic values of densities is given in Annex A of EC1 Part 1-1 and BS648[22]. Annex A of EC1 Part 1-1 gives:
– mean values for densities of construction materials, and
– mean values for densities and angles of repose for stored materials.
These mean values can be used as characteristic values.

Where materials are used with a significant scatter of densities e.g. due to their source, water content etc, the characteristic value of these densities should be assessed in accordance with Section 2.8.2 and EC0 clause 4.1.2.

3.6 Self-weight of construction works

3.6.1 Representation of actions

The self-weight of the construction works is represented by a single characteristic value.

The self-weight of the construction works includes:
– the structure
– non-structural elements (e.g. roofing, partitions, wall claddings, suspended ceilings, thermal insulation)
– fixed services (e.g. equipment for lifts, heating, ventilating and air conditioning equipment, electrical equipment)
– the weight of earth.

Loads due to movable partitions (see Section 3.2) have to be treated as imposed loads in the design. For determining the effect of the self-weight due to movable partitions, an equivalent uniformly distributed load given in Section 3.7.3.3 has to be added to the imposed load given in Table 3.1.

3.6.2 Characteristic values of self-weight

The characteristic value for self-weight can be calculated on the basis of the nominal dimensions, and the characteristic values of the densities given in Section 3.5 or in Annex A of EC1 Part 1-1.

3.7 Imposed loads on buildings

3.7.1 Representation of actions

Imposed loads on buildings are due to occupancy. The values for loads given in this Chapter account for:
– normal use by persons
– furniture and movable objects (e.g. movable partitions, storage, the contents of containers)
– vehicles
– anticipating rare events, such as concentrations of persons or of furniture, or the moving or stacking of objects which may occur during reorganisation or redecoration.

The imposed loads specified in this Chapter are modelled by uniformly distributed loads, line loads or concentrated loads or combinations of these loads.

The imposed loads for floors and roof areas in buildings given in this Chapter are appropriate to the type of activity/occupancy for which the area will be used.

Loads due to heavy equipment (e.g. in communal kitchens, radiology rooms, boiler rooms etc) are not included in this Chapter. These can be obtained from the manufacturer and agreed between the client and/or the relevant Authority.

3.7.2 Load arrangements

3.7.2.1 Floors, beams and roofs

For the design of a floor structure within one storey or the roof, the imposed load should be taken into account as a *free action* applied at the most unfavourable part of the influence area of the action effects considered. 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 3.1.

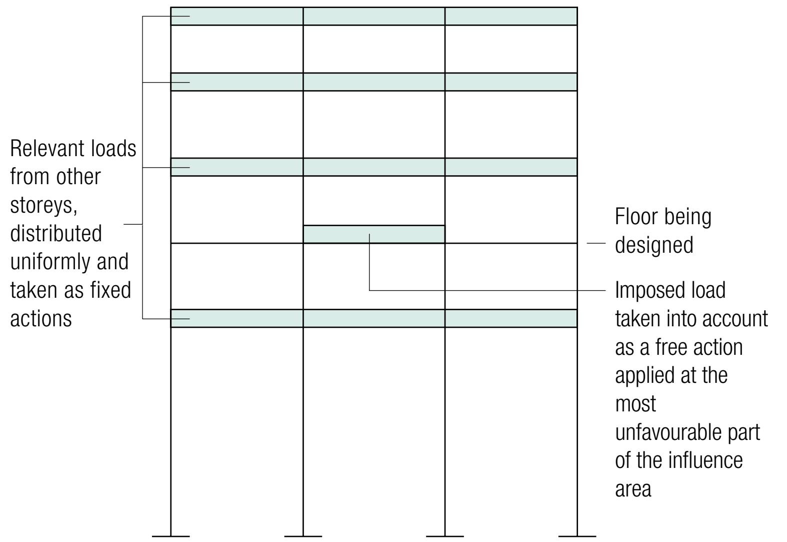


Fig 3.1 Load arrangement for floors, beams and roof

To check that there is a minimum local resistance of the floor structure (e.g. crushing, punching shear) a separate verification should be performed with a concentrated load acting alone.

Imposed loads from a single category may be reduced according to the areas supported by the appropriate member, by a reduction factor α_A according to Section 3.7.3.4.

3.7.2.2 Columns and walls

For the design of columns and walls, the imposed load should be placed at all unfavourable locations. Where imposed loads from several storeys act on columns and walls, the total imposed loads may be reduced by a factor α_n according to 3.7.3.4 and 3.4.3.1.

3.7.3 Characteristic values of imposed loads – residential, social, commercial and administration areas

3.7.3.1 Categories

The categories adopted for this type of activity are

A Areas for domestic and residential activities
B Office areas
C Areas where people may congregate (with the exception of areas defined under category A, B, and D in Table 3.1)
D Shopping areas

See Table 3.1, in which the above categories of loaded areas are further sub-categorised.

3.7.3.2 Values of actions

Table 3.1 gives the characteristic values q_k (uniformly distributed load) and Q_k (concentrated load) for each sub-category that should be used in the design.

For local verifications a concentrated load Q_k acting alone should be taken into account.

The concentrated load has 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 as a square with sides of 50mm. See also Section 3.7.6.2.

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

3.7.3.3 Partitions

Provided that a floor allows a lateral distribution of loads, the self-weight of movable partitions may be taken into account by a uniformly distributed load q_k which should be added to the imposed loads of floors given in Table 3.1. The value for q_k is dependent on the self-weight of partitions as follows:
– for movable partitions with a self-weight $\leqslant$ 1.0kN/m wall length: q_k = 0.5kN/m^2
– for movable partitions with a self-weight $\leqslant$ 2.0kN/m wall length: q_k = 0.8kN/m^2
– for movable partitions with a self-weight $\leqslant$ 3.0kN/m wall length: q_k =1.2kN/m^2.

When movable partitions are not included in the design, the designer may nevertheless decide to make some allowance for future flexibility.

For heavier partitions the design should take into account:
– the locations and directions of the partitions
– the structural form of the floors.

The possibility of removal of partitions should be considered.

Table 3.1 Imposed loads on floors, balconies and stairs for residential, social, commercial and administration areas

Category of loaded area	Specific use	Sub-category	Interpretation	q_k kN/m^2	Q_k kN
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[a]. For communal areas in other blocks of flats, see A5, A6 and C3	1.5	2.0
		A2	Bedrooms and dormitories except those in self-contained single family dwelling units and in hotels and motels	1.5	2.0
		A3	Bedrooms in hotels and motels; hospital wards; toilet areas	2.0	2.0
		A4	Billiard/snooker rooms	2.0	2.7
		A5	Balconies in single family dwelling units and communal areas in blocks of flats with limited use[a]	2.5	2.0
		A6	Balconies in hostels, guest houses, residential clubs and communal areas in blocks of flats except those covered by Note a	See Note f	2.0[g]
		A7	Balconies in hotels and motels	See Note h	2.0[g]
B	Office areas	B1	General use other than in B2	2.5	2.7
		B2	At or below ground floor level	3.0	2.7

Category of loaded area	Specific use	Sub-category		Interpretation	q_k kN/m²	Q_k kN
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[b]	2.0	3.0
			C12	Reading rooms with no book storage	2.5	4.0
			C13	Classrooms	3.0	3.0
		C2: Areas with fixed seats	C21	Assembly areas with fixed seating[c]	4.0	3.6
			C22	Places of worship	3.0	2.7
		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 a	3.0	4.5
			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 a	3.0	4.0
			C33	Corridors, hallways, aisles in all buildings not covered by C31 and C32, including hotels and motels and institutional buildings subjected to crowds	4.0	4.5
			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	5.0	4.5
			C35	Stairs, landings in all buildings not covered by C31 and C32, including hotels and motels and institutional buildings subjected to crowds	4.0	4.0
			C36	Walkways – Light duty (access suitable for one person, walkway width approx 600mm)	3.0	2.0

Category of loaded area	Specific use	Sub-category		Interpretation	q_k kN/m²	Q_k kN
C	Areas where people may congregate (with the exception of areas defined under category A, B and D)	C3: Areas without obstacles for moving people	C37	Walkways – General duty (regular two-way pedestrian traffic)	5.0	3.6
			C38	Walkways – Heavy duty (high density pedestrian traffic including escape routes)	7.5	4.5
			C39	Museum floors and art galleries for exhibition purposes	4.0	4.5
		C4: Areas with possible physical activities	C41	Dance halls and studios, gymnasia, stages[d]	5.0	3.6
			C42	Drill halls and drill rooms[d]	5.0	7.0
		C5: Areas susceptible to large crowds	C51	Assembly areas without fixed seating, concert halls, bars and places of worship[d e]	5.0	3.6
			C52	Stages in public assembly areas[d]	7.5	4.5
D	Shopping areas	D1		Areas in general retail shops	4.0	3.6
		D2		Areas in department stores	4.0	3.6

Notes

a 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.

b 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 d.

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

d For structures that might be susceptible to resonance effects, reference should be made to Sections 2.11.5 and 3.3.3.

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

f Same as the rooms to which they give access but with a minimum of 3.0kN/m².

g The load should be concentrated at the outer edge.

h Same as the rooms to which they give access but with a minimum of 4.0kN/m².

3.7.3.4 Reduction factors

(a) Reduction factor for imposed loads for floors and accessible roofs

In accordance with Section 3.7.2.1, a reduction factor α_A may be applied to the q_k values for imposed loads given in Table 3.1 for floors and accessible roofs Category I. Loads that have been specifically determined from knowledge of the proposed use of the structure do not qualify for reduction.

The reduction factor α_A should be determined using Expression 3.1.

$$\alpha_A = 1.0 - A/1{,}000 \geqslant 0.75 \tag{3.1}$$

where:
A is the area (m^2) supported.

(b) Reduction factors for imposed loads from several storeys

In accordance with Section 3.7.2.2 and provided that the area is classified according to Table 3.1 into the 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 that have been specifically determined from knowledge of the proposed use of the structure do not qualify for reduction.

The reduction factor α_n should be determined using Expression 3.2.

$$\left.\begin{array}{ll} \alpha_n = 1.1 - n/10 & \text{for } 1 \leqslant n \leqslant 5 \\ \alpha_n = 0.6 & \text{for } 5 < n \leqslant 10 \\ \alpha_n = 0.5 & \text{for } n > 10 \end{array}\right\} \tag{3.2}$$

where:
n is the number of storeys with loads qualifying for reduction.

(c) Restrictions on applying reduction factors

Load reductions based on area in Section 3.7.3.4(a) may be applied if $\alpha_A < \alpha_n$.

The reductions given by Expression 3.1 cannot be used in combination with those determined from Expression 3.2.

3.7.4 Characteristic values of imposed loads – areas for storage and industrial activities

3.7.4.1 Categories

The categories adopted for type of activity are
E1 Areas susceptible to accumulation of goods, including access areas
E2 Industrial use

See Table 3.2, in which the above categories of loaded areas are further sub-categorised.

3.7.4.2 Values of actions

Table 3.2 gives the characteristic values q_k (uniformly distributed load) and Q_k (concentrated load) for category E1 that have to be used in the design. The values of densities for storage materials should be obtained from Annex A of EC1 Part 1-1.

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

Loads in industrial areas should be assessed considering the intended use and the equipment which is to be installed.

3.7.4.3 Load arrangements

The loading arrangement should be chosen to ensure that it produces the most unfavourable conditions allowed in use.

Any effects resulting from filling and emptying should be taken into account.

3.7.4.4 Actions induced by forklifts

EC1 Part 1-1 classifies 6 classes of forklifts, FL 1 to FL 6, depending on net weight, dimensions and hoisting loads, see Table 3.3 and Figure 3.2.

The static vertical axle load Q_k of a forklift for classes FL 1 to FL 6 should be obtained from Table 3.3.

It is recommended that the details of the specific forklifts are checked with the client/manufacturer in case they differ from the information in Table 3.3.

Actions due to forklifts having a net weight greater than 110kN should be considered as concentrated loads acting together with the appropriate imposed distributed loads.

Table 3.2 Imposed floor loads due to storage for the UK

Category of loaded area	Specific use	Sub-category	Interpretation	q_k kN/m^2	Q_k kN
E1	Areas susceptible to accumulation of goods, including access areas	E11	General areas for static equipment not specified elsewhere (institutional and public buildings)	2.0	1.8
		E12	Reading rooms with book storage, e.g. libraries	4.0	4.5
		E13[a]	General storage other than those specified (see Note a)	2.4 per metre of storage height	7.0
		E14	File rooms, filing and storage space (offices)	5.0	4.5
		E15	Stack rooms (books)	2.4 per metre of storage height but with a minimum of 6.5	7.0
		E16	Paper storage for printing plants and stationery stores	4.0 per metre of storage height	9.0
		E17	Dense mobile stacking (books) on mobile trolleys, in public and institutional buildings	4.8 per metre of storage height but with a minimum of 9.6	7.0
		E18	Dense mobile stacking (books) on mobile trucks, in warehouses	4.8 per metre of storage height but with a minimum of 15.0	7.0
		E19	Cold storage	5.0 per metre of storage height but with a minimum of 15.0	9.0

Table 3.2 Imposed floor loads due to storage for the UK

Category of loaded area	Specific use	Interpretation	q_k kN/m^2	Q_k kN
E2	Industrial use	Communal kitchens[b]	3.0	4.5
		Operating theatres, X-ray rooms, utility rooms	2.0	4.5
		Work rooms (light industrial) without storage	2.5	1.8
		Kitchens, laundries, laboratories	3.0	4.5
		Rooms with mainframe computers or similar equipment	3.5	4.5
		Machinery halls, circulation spaces therein	4.0	4.5
		Cinematographic projection rooms	5.0	See Note c
		Factories, workshops and similar buildings (general industrial)	5.0	4.5
		Foundries	20.0	See Note c
		Catwalks	-	1.0 at 1m centres
		Fly galleries (i.e. access structures used in theatres to hang scenery, curtains, etc.)	4.5kN/m run distributed uniformly over width	-
		Ladders	-	1.5 rung load

Notes

a E13 is a general sub-category, however, designers are encouraged to liaise with clients to determine more specific load values than the lower bound value given in this Table.

b Except those covered by category A in Table 3.1.

c To be determined for specific use.

Table 3.3 Dimensions of forklift according to classes FL

Class of Forklift	Net weight kN	Hoisting load kN	Width of axle a m	Overall width b m	Overall length l m	Axle load Q_k kN
FL 1	21	10	0.85	1.00	2.60	26
FL 2	31	15	0.95	1.10	3.00	40
FL 3	44	25	1.00	1.20	3.30	63
FL 4	60	40	1.20	1.40	4.00	90
FL 5	90	60	1.50	1.90	4.60	140
FL 6	110	80	1.80	2.30	5.10	170

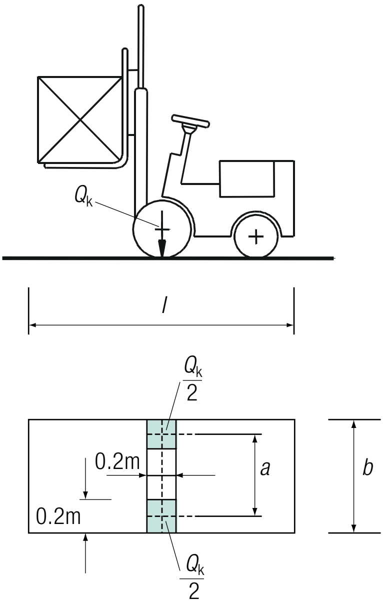


Fig 3.2 Dimensions of forklifts

The static vertical axle load Q_k should be increased by the dynamic factor φ using expression 3.3. The dynamic factor φ takes into account the inertial effects caused by acceleration and deceleration of the hoisting load.

$$Q_{k,dyn} = \varphi \, Q_k \tag{3.3}$$

where:
$Q_{k,dyn}$ is the dynamic characteristic value of the action
φ is the dynamic magnification factor
Q_k is the static characteristic value of the action.

with $\varphi = 1.40$ for pneumatic tyres
$\varphi = 2.00$ for solid tyres.

The vertical axle loads Q_k and $Q_{k,dyn}$ of a forklift should be arranged according to Figure 3.2.

Horizontal loads due to acceleration or deceleration of forklifts may be taken as 30% of the vertical axle loads Q_k.

3.7.4.5 Actions induced by transport vehicles and by special devices for maintenance

This Chapter does not give guidance on actions induced by transport vehicles and by special devices for maintenance. Specialist advice (e.g. from the manufacturers) should be obtained for:
– modelling actions from transport vehicles that move on floors freely or are guided by rails and
– special devices for maintenance.

Generally, however:
– The modelling of these actions should consider the pattern of wheel loads.
– The static values of the vertical wheel loads should be given in terms of net weights and payloads.
– The vertical and horizontal wheel loads should be determined for the specific case.
– The load arrangement including the dimensions relevant for the design should be determined for the specific case.
– Contact areas for wheel loading should be obtained from vehicle manufacturers/hirers.

3.7.5 Characteristic values of imposed loads – garages and vehicle traffic areas (excluding bridges)

3.7.5.1 Categories

The categories adopted for traffic and parking areas in buildings are:

F Traffic and parking areas for light vehicles (⩽30kN gross vehicle weight and ⩽ 8 seats not including driver). Examples include garages and parking areas.

G Traffic and parking areas for medium vehicles (>30kN, ⩽ 160kN gross vehicle weight, on 2 axles). Examples include access routes, delivery zones, zones accessible to fire engines (⩽ 160kN gross vehicle weight).

Access to areas designed to category F should be limited by physical means built into the structure.

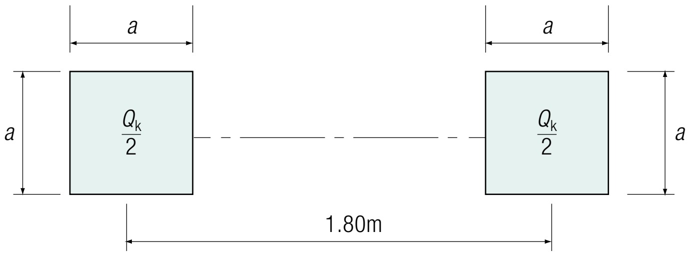
Consideration should be given to posting appropriate warning signs in areas designed as categories F or G.

3.7.5.2 Values of actions

The load model to be used is a single axle with a load Q_k with dimensions according to Figure 3.3 and a uniformly distributed load q_k. Table 3.4 gives the characteristic values of q_k (uniformly distributed load) and Q_k (concentrated load) for each category.

q_k should be used for determination of general effects and Q_k for local effects.

The axle load should be applied in accordance with Figure 3.3 in the positions which will produce the most adverse effects of the action.



For category F, a = 100mm
For category G, a = 200mm

Fig 3.3 Dimensions of axle load

Table 3.4 Imposed floor loads on garages and vehicle traffic areas

Categories of traffic areas	q_k kN/m²	Q_k kN
Category F (gross vehicle weight ≤30kN)	2.5	10.0
Category G (30kN < gross vehicle weight ≤160kN)	5.0	To be determined for specific use
Note q_k and Q_k should not be applied simultaneously.		

3.7.6 Characteristic values of imposed loads – roofs

3.7.6.1 Categories

For roofs the three categories of loaded area H, I and K, are adopted according to their accessibility, as follows:

H Roofs not accessible except for normal maintenance and repair

I Roofs accessible with occupancy according to categories A to G, see Tables 3.1, 3.2 and 3.4

K Roofs accessible for special services, such as helicopter landing areas.

This section gives characteristic values of imposed loads on roofs. Characteristic values of snow loads on roofs are given in Chapter 5.

3.7.6.2 Values of actions

Minimum imposed loads for roofs of category H are given in Table 3.5. Imposed loads for roofs of category I are given in Tables 3.1, 3.2 and 3.4.

The loads for roofs of category K which provide areas for helicopter landing areas are given in Table 3.6.

Table 3.5 gives the minimum characteristic values q_k (uniformly distributed load) and Q_k (concentrated load) for Category H roofs. They are related to the projected area of the roof under consideration.

The values given in Table 3.5 do not take into account uncontrolled accumulations of construction materials that may occur during maintenance. Further guidance for this is given in Chapter 8. It should be noted that $q_k = 0.6$ and $Q_k = 0.9$ do not take into account possible accumulation of construction materials. Although the values in Table 3.5 are from the UK National Annex to EC1 Part 1-4, it is recommended that $q_k = 0.6$ is used for all roof slopes to allow for access equipment used during maintenance.

For roofs separate verifications will need to be performed for the concentrated load Q_k and the uniformly distributed load q_k, acting independently.

Table 3.5 Minimum imposed loads on roofs (Category H) not accessible except for normal maintenance and repair

Roof slope, α degrees	q_k kN/m²	Q_k kN
$\alpha < 30^o$	0.6	0.9
$30^o \leqslant \alpha < 60^o$	$0.6[(60 - \alpha)/30]$	
$\alpha \geqslant 60^o$	0	

Notes
a All roof slopes *q* are measured from the horizontal and all loads should be applied vertically.
b 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.

Table 3.6 Imposed loads on roofs of category K for helicopters

Class of Helicopter	Take-off load Q of helicopter	Take-off load Q_k	Dimension of the loaded area (m x m)
HC1	$Q \leqslant 20$kN	$Q_k = 20$kN	0.2 x 0.2
HC2	$20\text{kN} < Q \leqslant 60$kN	$Q_k = 60$kN	0.3 x 0.3

Roofs, other than those with roof sheeting, should be designed to resist 1.5kN based on a square area of side 50mm. Roof elements with a profiled or discontinuously laid surface, should be designed so that the concentrated load Q_k acts over the effective area provided by load spreading arrangements.

Table 3.6 gives the Imposed loads on roofs of category K for helicopters.

The actions from helicopters on landing areas should be determined in accordance with Table 3.6, and using expression 3.4 and the appropriate dynamic factor.

$$Q_{k,dyn} = \varphi \; Q_k \qquad (3.4)$$

where:
$Q_{k,dyn}$ is the dynamic characteristic value of the action
φ is the dynamic magnification factor
Q_k is the take-off load

with $\varphi = 1.40$ applied to the take off load Q_k to take account of impact effects.

Access ladders and walkways should be assumed to be loaded according to Table 3.5 for a roof slope < 20°. For walkways which are part of a designated escape route, q_k should be according to Table 3.1. For walkways for service a minimum characteristic value Q_k of 1.5kN should be taken.

The following loads should be used for the design of frames and coverings of access hatches (other than glazing), the supports of ceilings and similar structures:
– without access: no imposed load
– with access: 0.25kN/m² distributed over the whole area or the area supported by the frames, and a concentrated load of 0.9kN placed so as to produce maximum effects of actions and stresses in the affected member.

3.7.7 Characteristic values of imposed loads – horizontal loads on parapets and partition walls acting as barriers

The characteristic values of the line load q_k acting at the height of the top of the partition wall or parapets but not higher than 1.20m are given in Table 3.7.

3.8 Vehicle barriers and parapets for car parks

3.8.1 General

Barriers and parapets in car parking areas need to be designed to resist the horizontal loads given in Section 3.8.2.

It should be noted that the 2500kg vehicle mass quoted in Section 3.8.2 is different to delineate between categories F and G in imposed loading in Section 3.7.5.1 and Tables 2.6 and 3.4. In addition there is an apparent discrepancy in height between the height of application given in Section 3.8.3 and that in Section 9.4.3.1.

3.8.2 Determination of horizontal characteristic force

The horizontal characteristic force F (in kN), normal to and uniformly distributed over any length of 1.5m of a barrier for a car park, is required to withstand the impact of a vehicle is given by:

$$F = 0.5mv^2 / (\delta_c + \delta_b)$$

where:
m **is the gross mass of the vehicle (in kg)**
v **is the velocity of the vehicle (in m/s) normal to the barrier**
δ_c **is the deformation of the vehicle (in mm)**
δ_b **is the deformation of the barrier (in mm)**

Table 3.7 Horizontal loads on partition walls and parapets

Category of loaded area	Sub-category	Interpretation	q_k kN/m
A (including sub-categories in Table 3.1)	(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 3.1)	(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 3.1)[a]	(v)	Areas having fixed seating within 530mm 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
C5 (including sub-categories in Table 3.1)	(ix)	Footways or pavements less than 3m wide adjacent to sunken areas Footways or pavements greater than 3m 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 3.2)	(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 600mm 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 Section 3.8

Note

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

(a) Car parks designed for vehicles not exceeding 2500kg

Where the car park has been designed on the basis that the gross mass of the vehicles using it will not exceed 2500kg the following values are used to determine the force *F*:

m = 1500kg
v = 4.5m/s
δ_c = 100mm unless better evidence is available.

For a rigid barrier, for which δ_b may be given as zero, the characteristic force *F* appropriate to vehicles up to 2500kg gross mass should be taken as 150kN.

(b) Car parks designed for vehicles exceeding 2500kg

Where the car park has been designed for vehicles whose gross mass exceeds 2500kg the following values are used to determine the characteristic force *F*:

m = the actual mass of the vehicle for which the car park is designed (in kg)
v = 4.5m/s
δ_c = 100mm unless better evidence is available.

3.8.3 Heights where forces are considered to act

The forces determined as in Section 3.8.2(a) or 3.8.2(b) may be considered to act at bumper height. In the case of car parks intended for vehicles whose gross mass does not exceed 2500kg this height may be taken as 375mm above the floor level.

Barriers to access ramps of car parks have to withstand one half of the force determined in Section 3.8.2(a) or 3.8.2(b) acting at a height of 610mm above the ramp.

Opposite the ends of straight ramps intended for downward travel which exceed 20m in length the barrier has to withstand twice the force determined in Section 3.8.2(a) acting at a height of 610mm above the ramp.

Chapter 4: Actions on structures exposed to fire

4.1 Scope

This Chapter provides guidance on the appropriate thermal actions to be used in subsequent structural design. Unlike other forms of loading such as dead and imposed, wind or snow there is no corresponding National code with which structural engineers will be familiar. For this reason background information is presented on the structural fire engineering design process and guidance given on the appropriate procedures for specific circumstances. Important new concepts are illustrated with reference to simplified worked examples in Appendix E.

This Chapter is only intended to be an introduction to the process of structural fire engineering. It is not intended to be a manual for the design of structures for fire resistance. Consequently, specialist structural fire engineering advice should be sought where structural engineers are unfamiliar with the process.

Given the inter-disciplinary nature of fire engineering it is particularly important to identify roles and responsibilities within the project team.

4.2 Terms and definitions

A basic list of definitions is provided in EC1 Part 1-2[4]. Specific additional definitions for the fire actions part are:

Design fire – Specified fire development assumed for design purposes.

Design fire load density $q_{f,d}$ (MJ/m^2) – Fire load density considered for determining thermal actions in fire design; its value makes allowance for uncertainties.

Design fire scenario – Specific fire scenario for which an analysis will be carried out.

Equivalent time of fire exposure $t_{e,d}$ (min) – Time of exposure to the standard time-temperature curve deemed to have the same heating effect as a real fire in a real compartment.

External fire curve – Nominal time-temperature curve intended for the outside of separating external walls which can be exposed to fire from different parts of the façade, i.e. directly from the inside of the respective fire compartment or from a compartment situated below or adjacent to the external wall. This covers the situation of flames emerging through openings.

Fire compartment – Space within a building, extending over one or several floors, which is enclosed by separating elements such that fire spread beyond the compartment is prevented for a specified fire exposure and for a specified period of time.

Fire load Q_{fi} (MJ) – Sum of thermal energies which are released by combustion of all combustible materials in a space.

Fire resistance (min) – Ability of a structure or a member to fulfil its required functions (load-bearing function and/or fire-separating function) for a specified load level, for a specified fire exposure and for a specified period of time.

Flashover – Simultaneous ignition of all fire loads in a compartment.

Fully developed fire – State of full involvement of all combustible materials in a fire within a specified space.

Hydrocarbon fire curve – Nominal time-temperature curve representing the effects of a hydrocarbon fire.

Opening factor *O* ($m^{½}$) – Factor representing the amount of ventilation depending on the area of openings in the compartment walls, on the height of these openings and on the total area of the enclosure surfaces.

Standard fire resistance rating – Fire resistance when exposed to the standard time-temperature curve.

Standard time-temperature curve – Nominal curve defined in EN 13501[23] for representing a model of a fully developed fire in a compartment.

Natural fire model – Design fire based on the physical parameters of the specific compartment. Natural fire models include simplified fire models, localised fires and advanced fire models.

Simplified fire model – Design fire based on specific physical parameters within a limited field of application. The parametric temperature-time curves as detailed in Annex A of EC1 Part 1-2 are examples of simplified fire models. An example of the use of this technique is included in Appendix E.

4.3 Structural fire design procedure

4.3.1 Introduction

Traditional UK fire design is concerned only with nominal fire exposures. The designer must demonstrate that the structural elements (beams, columns, walls, floors) have at least the mandatory fire resistance required by Building Regulations. For traditional building materials this is generally achieved with respect to standard details that have achieved the required performance in a standard fire test as set out in National and International test standards. Such an approach will be appropriate for the vast majority of 'simple' buildings covered by this *Manual*. The fire part of EC1 contains a number of calculation methods which can be used where the traditional prescriptive approach is either inappropriate or uneconomic. Examples of where alternative calculation methods may be used include:

– complex structures such as sports stadia, airport terminals or exhibition halls where the requirements of the regulations may be inappropriate
– design for complete burn-out rather than prescriptive fire resistance rating requirements
– renovation of existing structures where a change of use results in a new fire resistance category being applied to the building
– resolving situations where the fire resistance is less than that required by the regulations as a result of problems with workmanship or inaccurate specification of materials.

EC1 Part 1-2 sets out the thermal and mechanical actions to be used in the structural design of buildings exposed to fire. The designer may follow either a prescriptive approach based on a nominal (standard) fire exposure or a performance based approach which takes into account the physical characteristics of the individual fire compartment (fire load, compartment geometry, ventilation conditions and thermal properties of the compartment boundaries) or some combination of the two. These alternative design procedures are illustrated schematically in Figure 4.1 (a-d).

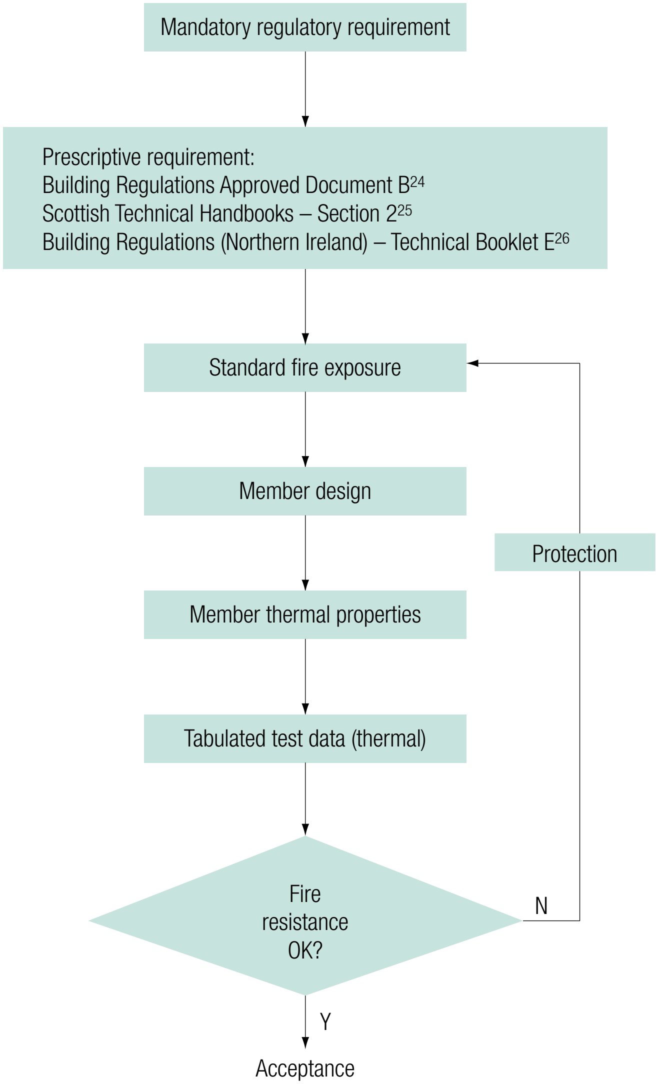


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Fig 4.1(a) Level 1 design procedure – Combination of prescriptive and tabulated (e.g. 'Yellow Book'[27], BS8110[28])

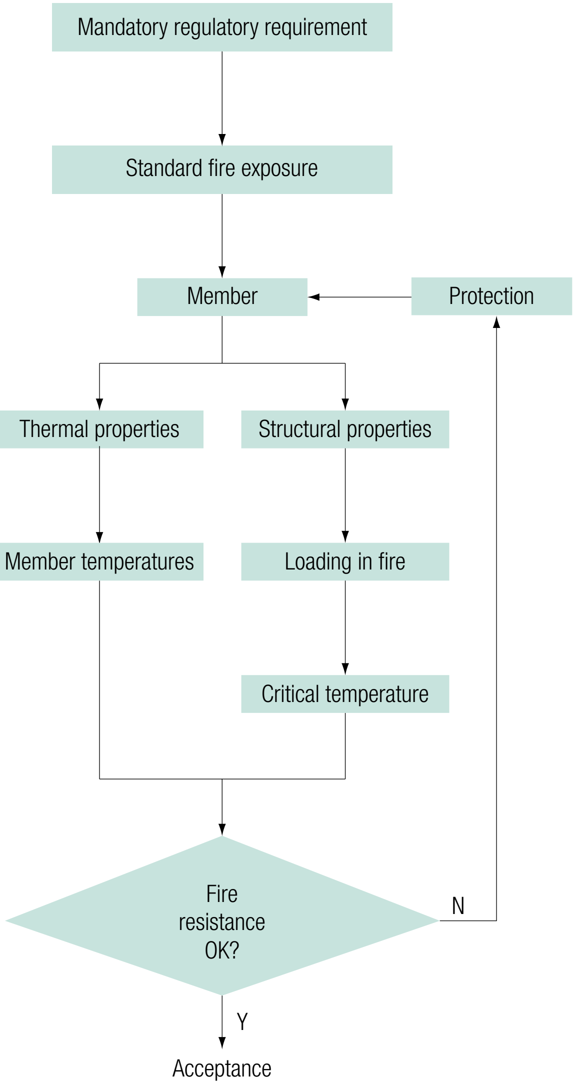

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Fig 4.1(b) Level 2 design procedure – Combination of prescriptive and performance based solution (e.g. simple calculation methods from EC2 Part 1-2[29] and EC3 Part 1-2[30])

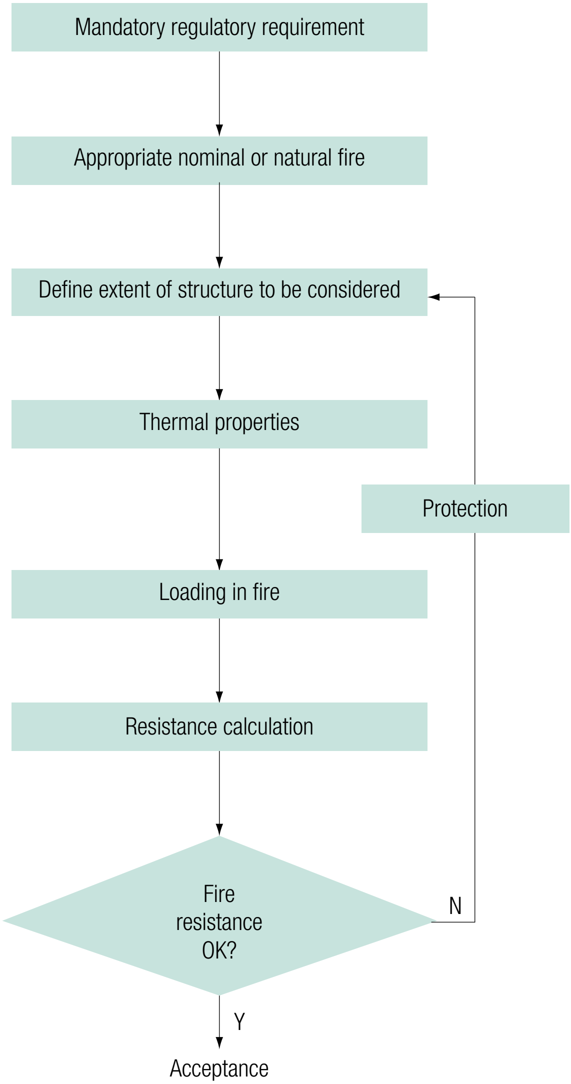

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Fig 4.1(c) Level 3 design procedure – Performance based advanced manual method (e.g. SCI Publication P288[31])

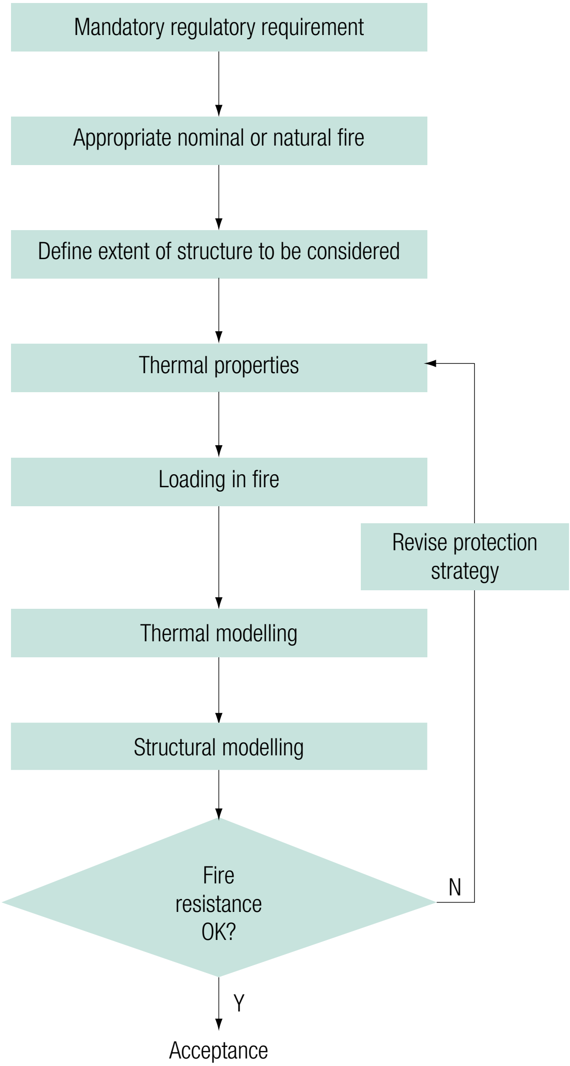

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Fig 4.1(d) Level 4 design guidance – Performance based advanced calculation (e.g. Software based thermo-structural analysis tools)

4.3.2 Design fire scenario(s)

The first step in a structural fire engineering design is to identify suitable design fire scenarios appropriate to the specific building. These should be determined on the basis of a fire risk assessment and on the level of design process being used taking into account the likely ignition sources and any fire detection/suppression systems available. The choice of design fire scenario will dictate the choice of the design fire to be used in subsequent analysis.

The design fire scenarios selected will identify specific compartment geometries with associated fire loads and ventilation conditions. The specific areas to be used for design should be based on a 'reasonable worst case scenario'.

4.3.3 Design fire

For each design fire scenario, a design fire will be chosen that represents the likely risk within that area. Normally the design fire is only applied to one compartment at a time.

Post-flashover conditions are generally of significance for the assessment of the fire resistance of structures.

Flashover is defined as simultaneous ignition of all fire loads within the compartment and is characterised by uniform temperatures, external flaming through openings and temperatures above 550ºC. For fully developed post-flashover building fires the usual choice is between nominal (standard) and natural fire exposures. Nominal fires are representative fires for the purpose of classification and comparison but bear no relationship to the specific characteristics (fire load, thermal properties of the compartment lining, ventilation conditions) of the building considered.

Natural fires are calculation techniques based on a consideration of the physical parameters specific to a particular building or compartment.

4.3.4 Temperature analysis

Temperature analysis of structural members is usually undertaken based on procedures in the fire parts of the material codes (EC2 Part 1-2, EC3 Part 1-2, EC4 Part 1-2, EC5 Part 1-2, EC6 Part 1-2 and EC9 Part 1-2). The location of the design fire relative to the member should be taken into account. In many cases this will dictate whether a localised or post-flashover fire model is appropriate for the given design scenario. Fire through openings should be considered where appropriate and for separating members fire from one or both sides may need to be considered depending on the specific circumstances.

Where a nominal fire is used as the design fire the temperature analysis for the structural member is only carried out for the heating phase corresponding to the required fire resistance period. No account is taken of temperatures in the cooling phase.

Where a natural fire model is used for the design fire the temperature analysis of the structural members should be continued for the entire duration of the fire including the cooling phase.

4.3.5 Mechanical analysis

The mechanical (structural) analysis should be performed for the same duration as the temperature analysis. Therefore where a natural fire model is used the mechanical behaviour should be determined for the full duration including the cooling phase.

Mechanical resistance is determined in accordance with the requirements of the fire parts of the material codes. Verification of adequate resistance may be undertaken with respect to time:

$$t_{\mathrm{fi,d}} \geqslant t_{\mathrm{fi,req}}$$

or strength:

$$R_{\mathrm{fi,d,t}} \geqslant E_{\mathrm{fi,d,t}}$$

or temperature:

$$\theta_{\mathrm{d}} \leqslant \theta_{\mathrm{cr,d}}$$

where:

$t_{\mathrm{fi,d}}$ is the design value of the fire resistance for a given load
$t_{\mathrm{fi,req}}$ is the required fire resistance time
$R_{\mathrm{fi,d,t}}$ is the design value of the resistance of the member in the fire situation at time t
$E_{\mathrm{fi,d,t}}$ is the design value of the relevant effects of actions in the fire situation at time t
θ_{d} is the design value of material temperature for a given load
$\theta_{\mathrm{cr,d}}$ is the design value of the critical material temperature for a given load.

In many cases the assessment will lead to the same final point in terms of the member design.

4.4 Thermal actions for temperature analysis

4.4.1 Introduction

This section of the Code gives general rules governing the basic principles of heat transfer from the fire to the structural member covering convective and radiative heat flux.

This Section sets out the procedures to be used for nominal time-temperature curves, and natural fire models including simplified and advanced models.

4.4.2 Nominal temperature-time curves

4.4.2.1 General

Nominal temperature-time curves represent heating conditions associated with standard furnace conditions as given in BS EN 1363[32] dealing with cellulosic conditions (fuel load characterised by materials principally composed of wood or wood-based materials) external exposure and hydrocarbon exposure.

4.4.2.2 Standard temperature-time curve

The standard temperature-time curve is given by:

$$\theta_g = 20 + 345 \log_{10}(8t + 1) \quad (^\circ\text{C})$$

where:

θ_g is the gas temperature in the fire compartment (°C)

t is the time (min).

This is a post-flashover fire model and the temperature is assumed to be uniform within the compartment. The temperature-time response is illustrated in Figure 4.2.

4.4.2.3 External fire curve

The external fire curve is given by:

$$\theta_g = 660\,(1 - 0.687\,e^{-0.32t} - 0.313\,e^{-3.8t}) + 20 \quad (^\circ\text{C})$$

where θ_g and t are as described above. This curve is very rarely used in practice. The temperature-time curve is illustrated in Figure 4.2.

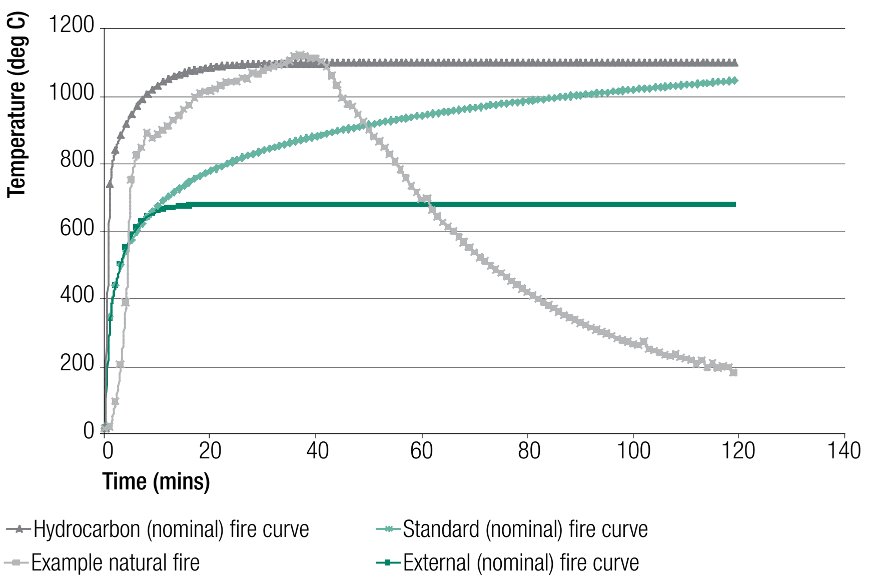


Fig 4.2 Comparison between nominal and natural fire curves

4.4.2.4 Hydrocarbon curve

The hydrocarbon curve is used where the calorific value of the fire load is significantly higher than the standard cellulosic curve such as in the petro-chemical industry. The hydrocarbon temperature-time curve is given by:

$$\theta_g = 1080(1 - 0.325\, e^{-0.167t} - 0.675\, e^{-2.5t}) + 20 \qquad (°C)$$

where θ_g and *t* are as described above. The temperature-time curve is illustrated in Figure 4.2.

4.4.2.5 Other nominal curves

The code presents specific examples of nominal temperature-time curves. However, a number of other standard relationships are available for specific purposes. These include a smouldering fire curve where the available oxygen for combustion is limited and a series of tunnel and jet fire curves varying in severity depending on the specific fire design scenario.

4.4.3 Natural fire models

4.4.3.1 General

A number of natural fire models are available ranging from simple calculation procedures through to advanced methods based on computational fluid dynamics.

4.4.3.2 Simplified fire models

Simplified fire models are based on specific physical parameters with a limited field of application generally related to the conditions for which validation has been undertaken. They can be used for either compartment fires where a uniform temperature distribution is assumed or localised fires where a non-uniform temperature distribution is assumed. A number of simple models are presented in the informative annexes within the code. Please note that, for use in the UK, these sections are replaced by non-contradictory, complementary information referenced in the National Annex. Parametric temperature-time curves are one example of simplified fire models. Further information may be found in Appendix E of this *Manual*.

4.4.3.3 Localised fires

Where, for whatever reason, flashover is unlikely to occur, the effects of a localised fire on structural performance should be assessed. Although a method for calculating the thermal exposure due to a localised fire is presented in Annex C of EC1 Part 1-2, the work to develop the UK National Annex found that this was inadequate. The UK National Annex therefore recommends the use of the method in PD 6688-1-2: 2007[33].

4.4.3.4 Advanced fire models

Only general guidance is presented in the code on the use of advanced models for calculating gas temperatures within the fire compartment. Annex D describes briefly the use of one zone, two zone and field (CFD) models. Such models should only be used by experienced personnel who understand the assumptions and limitations of the model. Further guidance on the use of advanced methods is available through the Institution of Structural Engineers[34].

4.5 Mechanical actions for structural analysis

4.5.1 General

The most important aspect under this heading is that dealing with combinations of actions for the fire limit state. An accurate assessment of the performance of a structural member during a fire requires a knowledge of both the reduction in material properties at increasing temperature and an accurate assessment of the loads acting on the structure at the time of the fire. Load effects have a significant impact during a fire and this is reflected in the requirement for realistic load levels to be in place during standard fire tests.

4.5.2 Actions from normal temperature design

Actions should be considered as for normal temperature design if they are likely to be present in the fire situation. Fire is an accidental loading case. Therefore representative values of variable actions accounting for the accidental situation of fire exposure should be introduced in accordance with the provisions of EC0.

4.5.3 Additional actions

Simultaneous occurrence with other accidental actions need not be considered. Additional actions induced by the fire such as localised collapse of walls or machinery may need to be considered. This should form part of the overall fire risk assessment. There is no requirement for fire walls designed to the standard fire exposure to resist a horizontal impact load.

4.5.4 Combination rules for actions

The calculation of load effects at the fire limit state is different from the procedure adopted in current National Standards. The designer must be familiar with both EC0 which provides the required load combinations (as for ambient temperature design) and with EC1 Part 1-2 which, in addition to specifying the available options for thermal actions for temperature analysis (see above) also specifies the mechanical actions for structural analysis. In particular EC1 Part 1-2 specifies the partial factor for imposed (assuming leading variable action) loading for the fire limit state. Fire loading is an ultimate limit state accidental design situation utilising the combination of actions given as equation 6.11b of EC0. This equation is reproduced in Chapter 2 of this *Manual* (Section 2.10.3.5).

In the fire situation A_d is the effect of the fire itself on the structure i.e. the effects of restrained thermal expansion, thermal gradients etc. However, EC1 Part 1-2 states that

"Indirect actions from adjacent members need not be considered when fire safety requirements refer to members under standard fire conditions"

and

"Imposed and constrained expansions and deformations caused by temperature changes due to fire exposure result in effects of actions e.g. forces and moments which should be considered with the exception of those where they:

– May be recognised *a priori* to be negligible or favourable
– Are accounted for by conservatively chosen support models and boundary conditions and/or implicitly considered by conservatively specified fire safety requirements."

EC0 allows the use of either ψ_1 or ψ_2 with the main variable action (generally the imposed load). EC1 Part 1-2 recommends the use of ψ_2, however the UK National Annex specifies the use of ψ_1 as detailed in Table 4.1 (see also Chapter 2, Table 2.6).

Table 4.1 ψ_1 ψ_2 values

Action	ψ_1	ψ_2
Imposed loads in buildings		
Category A: domestic, residential	**0.5**	**0.3**
Category B: office areas	**0.5**	**0.3**
Category C: congregation areas	**0.7**	**0.6**
Category D: shopping areas	**0.7**	**0.6**
Category E: storage areas	**0.9**	**0.8**
Category F: traffic area, $\leqslant$ 30kN	**0.7**	**0.6**
Category G: traffic area, 30-160kN	**0.5**	**0.3**
Category H: roofs	**0.0**	**0.0**
Snow load: H $\leqslant$ 1000m a.s.l.	**0.2**	**0.0**
Snow load: H $>$ 1000m a.s.l.	**0.5**	**0.2**
Wind loads on buildings	**0.2**	**0.0**

Chapter 5: Snow loads

5.1 Scope

This Chapter gives design guidance contained in EN1991-1-3[5] (EC1 Part 1-3) to determine the values of loads due to snow to be used for the structural design of buildings within the scope of this *Manual*.

This Chapter does not give guidance on specialist aspects of snow loading, for example:
- impact snow loads resulting from snow sliding off or falling from a higher roof
- the additional wind loads which could result from changes in shape or size of the construction works due to the presence of snow or the accretion 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).

For a large flat roof of length greater than 40m see Section 5.6.4.2.

5.2 Terms and definitions

A basic list of definitions is given in Chapter 2: Basis of Structural Design. Additional definitions for snow loads on buildings are:

Characteristic value of snow load on the ground – snow load on the ground based on an annual probability of being exceeded of 0.02, excluding exceptional snow loads.

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.

Characteristic value of snow load on the roof – product of the characteristic snow load on the ground and 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.

Undrifted snow load on the 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.

Drifted snow load on the roof – load arrangement which describes the snow load distribution resulting from snow having been moved from one location to another location on a roof, e.g. by the action of the wind.

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.

Thermal coefficient – coefficient defining the reduction of snow load on roofs as a function of the heat flux through the roof, causing snow melting.

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 of the snow layer on the ground resulting from a snow fall which has an exceptionally infrequent likelihood of occurring.

Exceptional drifted snow load on the roof – load arrangement which describes the load of the snow layer on the roof resulting from a snow deposition pattern which has an exceptionally infrequent likelihood of occurring.

5.3 Classification of actions

5.3.1 General

Classification of actions considers the variation of actions in time and space.

5.3.2 Snow loads

Within the scope of this *Manual* snow loads are generally classified as variable, fixed actions. However certain drift loads (see Sections 5.4 and 5.6.3(b)) are treated as accidental actions. See also Chapter 2.

Snow loads covered in this *Manual* are classified as static actions, see Chapter 2.

5.4 Design situations

The relevant snow loads are to be used for each identified design situation in accordance with Section 2.8.3 and/or Appendix B.

The guidance given in EC1 Part 1-3 assumes that in the UK, exceptional snow falls are unlikely to occur but exceptional snow drifts may occur (see Section 5.2 for definitions). Based on this assumption, the following design situations apply:

- **the transient/persistent design situation should be used for both the undrifted snow load arrangements determined using Sections 5.6 and 5.7, and the drifted snow load arrangements determined using**
  - **Section 5.6.4.2 for mono-pitch roofs**
  - **Section 5.6.4.3 for duo-pitch roofs.**
- **the accidental design situation should be used for snow load arrangements determined using**
  - **Section 5.6.4.4 for multi-span roofs**
  - **Section 5.6.4.5 for roofs abutting taller construction works; and**
  - **Section 5.7.2 for projections and obstructions.**

5.5 Snow load on the ground

5.5.1 Characteristic values

The characteristic value of snow load on the ground (s_k) is defined in accordance with Section 2.8.3 and/or Appendix B, and the definition for characteristic snow load on the ground is given in Section 5.2.

The characteristic ground snow loads s_k to be used in the UK should be obtained from the map shown in Figure 5.1 and Expression 5.1.

$$s_k = [0.15 + (0.1Z + 0.05)] + [(A - 100)/525] \qquad (5.1)$$

where:
s_k is the characteristic ground snow load (kN/m^2)
Z is the zone number obtained from the map in Figure 5.1
A is the site altitude (m).

Unusual local effects may not have been accounted for in the analysis undertaken to produce the ground snow load map given in Figure 5.1. These include local shelter from the wind, which can result in increased local snow loads and local configurations in mountainous areas, which may funnel the snow and give increased local loading. If the designer suspects that there are unusual local conditions that need to be taken into account, then the Meteorological Office should be consulted.

For coastal sites below 100m the map value should be used without the altitude modification.

Where a more refined characteristic ground snow load value s_k is required, the Meteorological Office should be consulted.

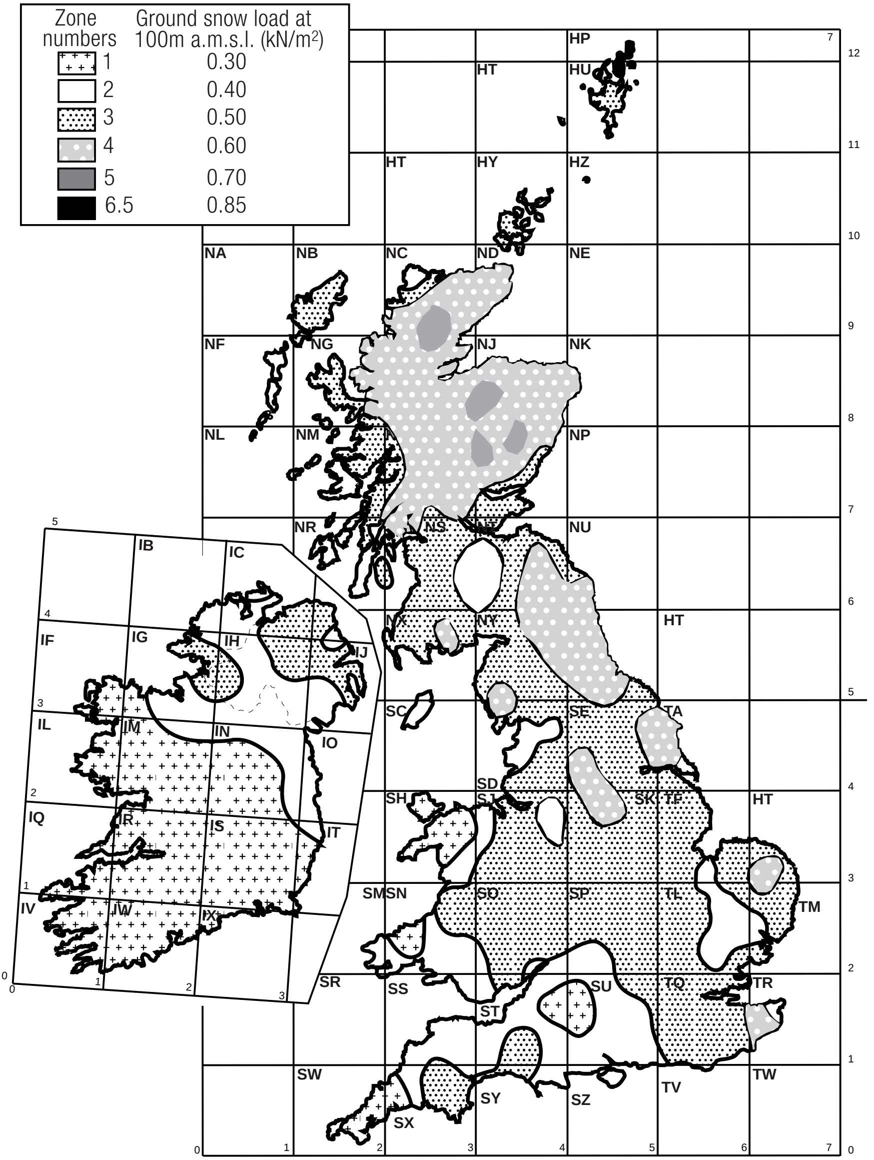
5.5.2 Other representative values for snow load

The other representative values for snow load on the roof (see Chapter 2 and/or Appendix B) are as follows:

– Combination value $\psi_0\, s$
– Frequent value $\psi_1\, s$
– Quasi-permanent value $\psi_2\, s$

where:
***s* is the snow load over the roof.**

Table 2.6 gives values that should be used in the UK for the coefficients ψ_0, ψ_1 and ψ_2.



Jersey – Zone 3, 0.50kN/m^2 at 100m a.m.s.l. Guernsey – Zone 2, 0.40kN/m^2 at 100m a.m.s.l.

Note Although the above map includes the Republic of Ireland, the *Manual* does not necessarily cover the design of buildings to be constructed there.

Fig 5.1 Characteristic ground snow load map

5.6 Snow load on roofs

5.6.1 Nature of the load

Snow can be deposited on a roof in many different patterns. The properties of a roof or other factors causing different patterns include:
– the shape of the roof
– its thermal properties
– the roughness of its surface
– the amount of heat generated under the roof
– the proximity of nearby buildings
– the surrounding terrain
– the local meteorological climate, in particular its windiness, temperature variations, and likelihood of precipitation (either as rain or as snow).

5.6.2 Load arrangements

The two primary load arrangements to be taken into account are:
– undrifted snow load on roofs (see Section 5.2)
– drifted snow load on roofs (see Section 5.2).

The loads should be assumed to act vertically and refer to a horizontal projection of the roof area.

Specialist advice should be sought where the consecutive melting and freezing of snow together with possible rainfall is likely to occur and block roof drainage.

5.6.3 Snow load on the roof

The snow loads on roofs, s, for the different design situations are determined as follows:

(a) for persistent / transient design situations:

$$s = \mu_i C_e C_t s_k \tag{5.2}$$

(b) for accidental design situations which should be used to determine the imposed roof loads due to drifted snow for the following cases:
(i) multi-span roofs (see Section 5.6.4.4)
(ii) roofs abutting and close to taller structures (see Section 5.6.4.5)
(iii) drifting at projections and obstructions (see Section 5.7.2)

the exceptional snow drift load is the accidental action given by:

$$s = \mu_i s_k \tag{5.3}$$

where:
μ_i is the snow load shape coefficient
s_k is the characteristic value of snow load on the ground
C_e is the exposure coefficient
C_t is the thermal coefficient.

The recommended value for exposure coefficient C_e is 1.0 for all topographies.

The recommended value for thermal coefficient C_t is 1.0 for all roofing materials.

5.6.4 Snow load shape coefficients

5.6.4.1 General

This Section gives snow load shape coefficients for undrifted and drifted snow load arrangements for the types of roofs identified in this *Manual*.

Special consideration should be given to the snow load shape coefficients to be used where the roof has an external geometry which may lead to increases in snow load that are considered significant in comparison with that of a roof with linear profile.

Shape coefficients, for the undrifted case only, for the roof shapes in Sections 5.6.4.2, 5.6.4.3 and 5.6.4.4, are given in Figure 5.2 and Table 5.1. The shape coefficients for the drifted cases are given in
(a) Table 5.2 for duo-pitch roofs and
(b) Section 5.6.4.4(b) and Figure 5.6 for multi-span roofs.

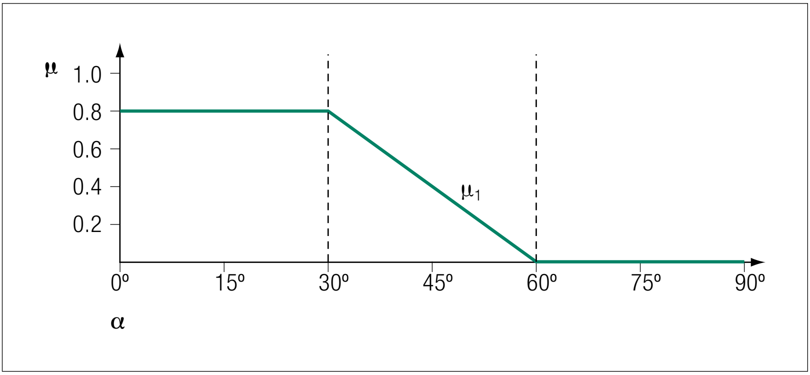


Fig 5.2 Snow load shape coefficient μ_1

Table 5.1 Snow load shape coefficient μ_1

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

5.6.4.2 Monopitch roofs

The snow load shape coefficient μ_1 that should be used for monopitch roofs is given in Table 5.1 and shown in Figure 5.2 and Figure 5.3.

The values given in Table 5.1 apply when the snow is not prevented from sliding off the roof. Where snow fences or other obstructions exist or where the lower edge of the roof is terminated with a parapet, then the snow load shape coefficient should not be reduced below 0.8.

The load arrangement of Figure 5.3 is applicable for both the undrifted and drifted load arrangements.

The designer should give special consideration to large flat roofs which are treated as a monopitch roof with $\alpha = 0°$, where the value of μ_1 can be = 1.0.

Note There is research evidence that for larger roofs (e.g. square or almost square roofs with length about 40m) the snow layer may be non uniform and the maximum value of the ratio between the roof and the ground snow loads reaches unity.

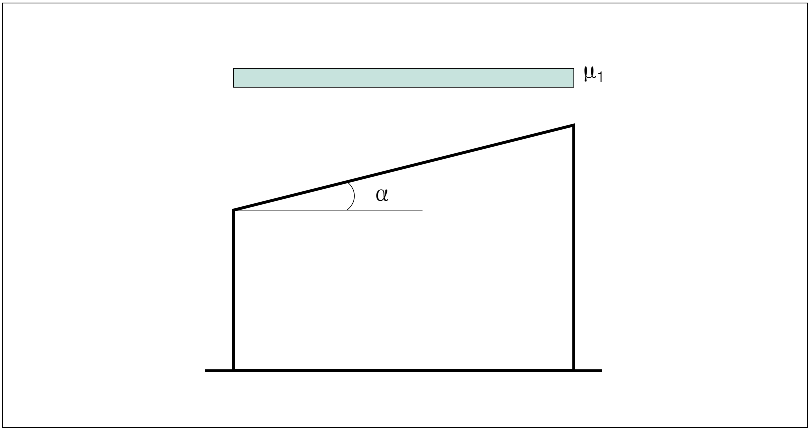


Fig 5.3 Snow load shape coefficient – monopitch roof

5.6.4.3 Pitched roofs

The snow load shape coefficient μ_1 that should be used for pitched roofs is given in
– Table 5.1 and shown in Figure 5.4 for the undrifted case (i)
– Table 5.2 and shown in Figure 5.4 for the drifted cases (ii) and (iii).

The values given in Tables 5.1 and 5.2 apply when the snow is not prevented from sliding off the roof. Where snow fences or other obstructions exist or where the lower edge of the roof is terminated with a parapet, then the snow load shape coefficient should not be reduced below 0.8.

The undrifted load arrangement which should be used is shown in Figure 5.4, case (i).

The drifted load arrangements which should be used are shown in Figure 5.4, cases (ii) and (iii), unless specified for local conditions.

Table 5.2 Drifted snow load shape coefficient for a duo-pitched roof

Angle of pitch of roof ($\alpha_{i,\, i=1,2}$)	$0° \leqslant \alpha_i \leqslant 15°$	$15° < \alpha_i \leqslant 30°$	$30° < \alpha_i < 60°$	$\alpha_i \geqslant 60°$
Snow load shape coefficient, μ_1	0.8	$0.8 + 0.4\,(\alpha_i - 15)/15$	$1.2(60 - \alpha_i)/30$	0.0

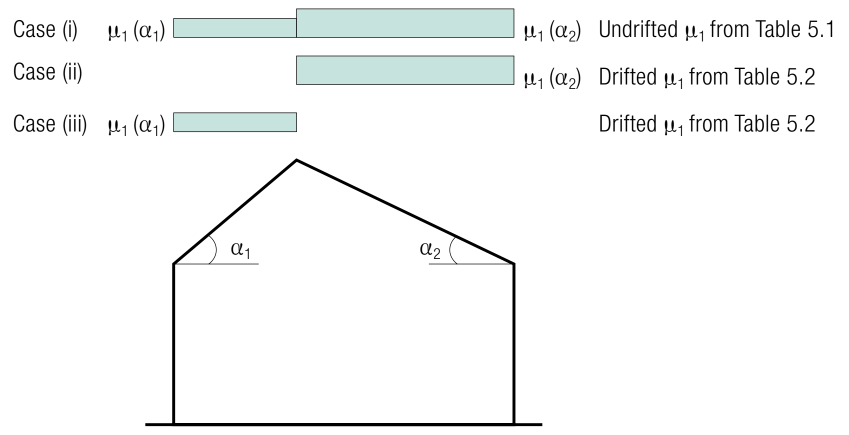


Fig 5.4 Snow load shape coefficient – pitched roofs

5.6.4.4 Multi-span roofs

The values of snow load shape coefficient μ_1 that should be used for multi-pitched roofs is given in
– Table 5.1 and shown in Figure 5.5 for the undrifted case
– Section 5.6.4.4(b) and shown in Figure 5.6 for drifted cases.

(a) Undrifted load arrangement for multi-span roofs

The undrifted load arrangement which should be used is shown in Figure 5.5.

Figure 5.5 does not apply for the design of multi-span roofs where one or both sides of the valley have a slope greater than 60º.

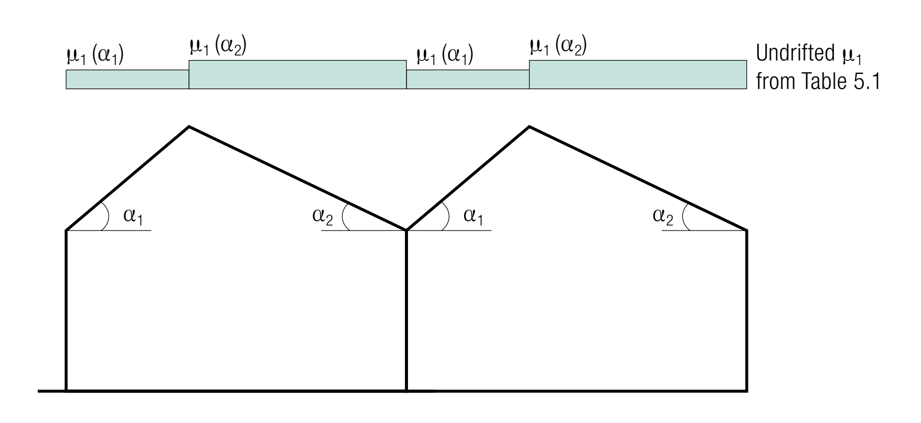


Fig 5.5 Snow load shape coefficients for multi-span roofs

(b) Drifted load arrangement for multi-span roofs

When considering the load cases using snow load shape coefficients for drifting for multi-span roofs obtained from this Section it should be assumed that they are exceptional snow drift loads and that there is no snow elsewhere on the roof.

The drifted load arrangement which should be used is shown in Figure 5.6 and the value for the snow load shape coefficient for an exceptional snow drift that should be used for valleys of multi-span roofs is given below.

The shape coefficient μ_1 in Figure 5.6 is determined as the least value of:
$\mu_1 = 2h/s_k$
$\mu_1 = 2b_3/(l_{s1} + l_{s2})$
$\mu_1 = 5$

μ_1 is regarded as dimensionless.

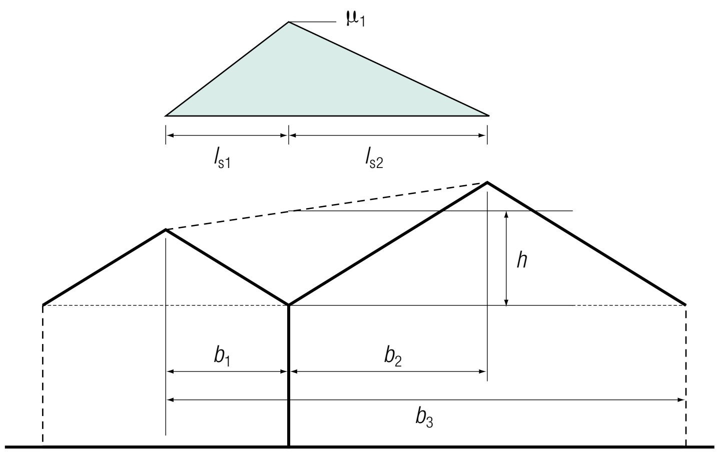


Fig 5.6 Shape coefficient and drift lengths for exceptional snow drifts – valleys of multi-span roofs

The drift lengths are determined as:

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

The units to be used for the above calculations are kN/m^2 and m as appropriate.

For roofs of more than two spans with approximately symmetrical and uniform geometry, b_3 (see Figure 5.6) should be taken as the horizontal dimension of three slopes (i.e. span x 1.5) and this snow load distribution should be considered applicable to every valley, although not necessarily simultaneously.

Care should be taken when selecting b_3 for roofs with non-uniform geometry. Significant differences in ridge height and/or span may act as obstructions to the free movement of snow across the roof and influence the amount of snow theoretically available to form the drift.

Where simultaneous drifts in several valleys of a multi-span roof are being considered in the design of a structure as a whole, a maximum limit on the amount of drifted snow on the roof should be applied. The total snow load per metre width in all the simultaneous drifts should not exceed the product of the ground snow load and the length of the building perpendicular to the valley ridges.

If the structure is susceptible to asymmetric loading, the design should also consider the possibility of drifts of differing severity in the valleys.

5.6.4.5 Roof abutting and close to taller construction works

The values of snow load shape coefficients μ_n (n = 1, 2 or 3) that should be used for roofs abutting and close to taller construction works are given in
– Section 5.6.4.5(a) for the undrifted case and shown in Figure 5.7
– Section 5.6.4.5(b) with Table 5.3 and shown in Figure 5.8 for the drifted cases.

(a) Undrifted load arrangement for roofs abutting and close to taller construction works

The snow load shape coefficient that should be used for roofs abutting and close to taller construction works is μ_1 = 0.8 (assuming the lower roof is flat).

The undrifted load arrangement which should be used is shown in Figure 5.7.

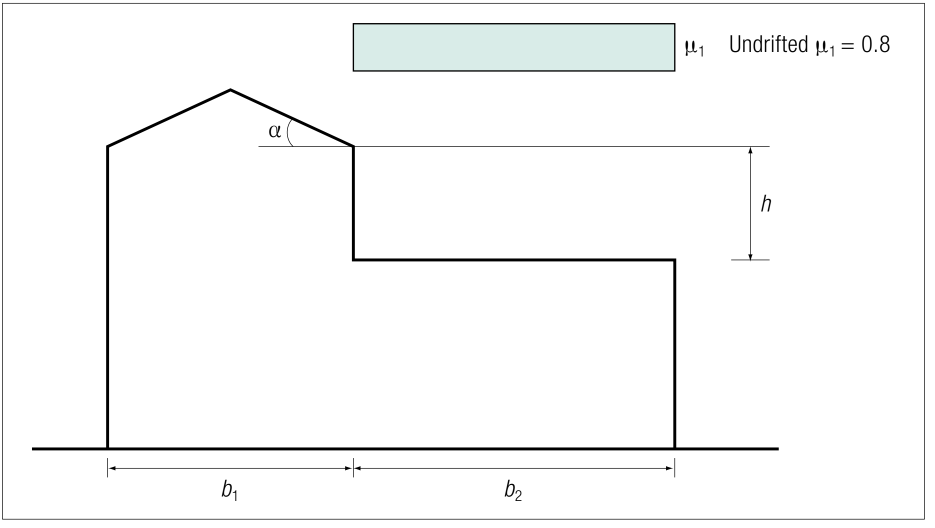


Fig 5.7 Snow load shape coefficient for roofs abutting taller construction works

(b) Drifted load arrangement for roofs abutting and close to taller construction works

When considering the load cases using snow load shape coefficients for drifting for roofs abutting and close to taller construction works obtained from this Section it should be assumed that they are exceptional snow drift loads and that there is no snow elsewhere on the roof.

The snow load shape coefficient for exceptional snow drifts that should be used for roofs abutting a taller construction work are given in Figure 5.8 and Table 5.3.

The snow load case given in Figure 5.8 is also applicable for roofs close to, but not abutting, taller buildings, with the proviso that it is only necessary to consider the load actually on the lower roof, i.e. the load implied between the two buildings can be ignored.

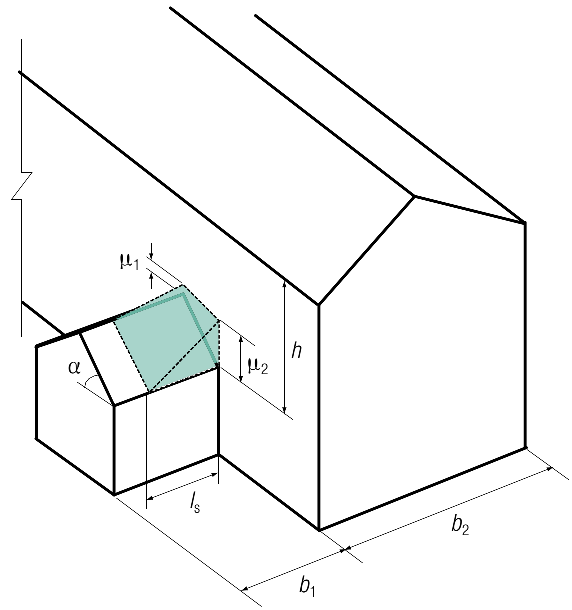


Fig 5.8 Shape coefficients and drift lengths for exceptional snow drifts – roofs abutting and close to taller structures

The effect of structures close to, but not abutting the lower roof will depend on the roof areas available from which snow can be blown into the drift and the difference in levels. However, as an approximate rule, it is only necessary to consider nearby structures when they are less than 1.5m away.

The drift length l_s is the least value of $5h$, b_1 or 15m.

Table 5.3 Shape coefficients for exceptional snow drifts for roofs abutting and close to taller structures

Shape coefficient	Angle of roof pitch α			
	$0° \leqslant \alpha \leqslant 15°$	$15° < \alpha \leqslant 30°$	$30° < \alpha < 60°$	$60° \leqslant \alpha$
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 15m.

5.7 Local effects

5.7.1 General

This Section gives forces to be applied for the local verifications of:
– drifting, where the drifting occurs at projections, obstructions and parapets (see Section 5.7.2)
– the edge of the roof (see Section 5.7.3)
– snow fences (see Section 5.7.4).

The design situations to be considered in accordance with Section 2.7.1 are the persistent/transient design situations, except for drifting against projections and obstructions and parapets where the accidental design situation is considered.

5.7.2 Roofs where drifting occurs at projections, obstructions and parapets

In windy conditions drifting of snow can occur on any roof which has obstructions as these cause areas of aerodynamic shade in which snow accumulates.

(a) Projections and obstructions
The snow load shape coefficients for exceptional snow drifts that should be used for roofs where drifting occurs at projections and obstructions are given below and in Figure 5.9. The effect of drifting can be ignored for vertical elevations not greater than $1m^2$ against which a drift could form.

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

The shape coefficients given in Figure 5.9 are determined as the lesser value of:
$\mu_1 = 2h_1/s_k$ or 5
$\mu_2 = 2h_2/s_k$ or 5.

μ_1 and μ_2 are taken as dimensionless. The units to be used for the above calculations are kN/m^2 and m as appropriate.

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

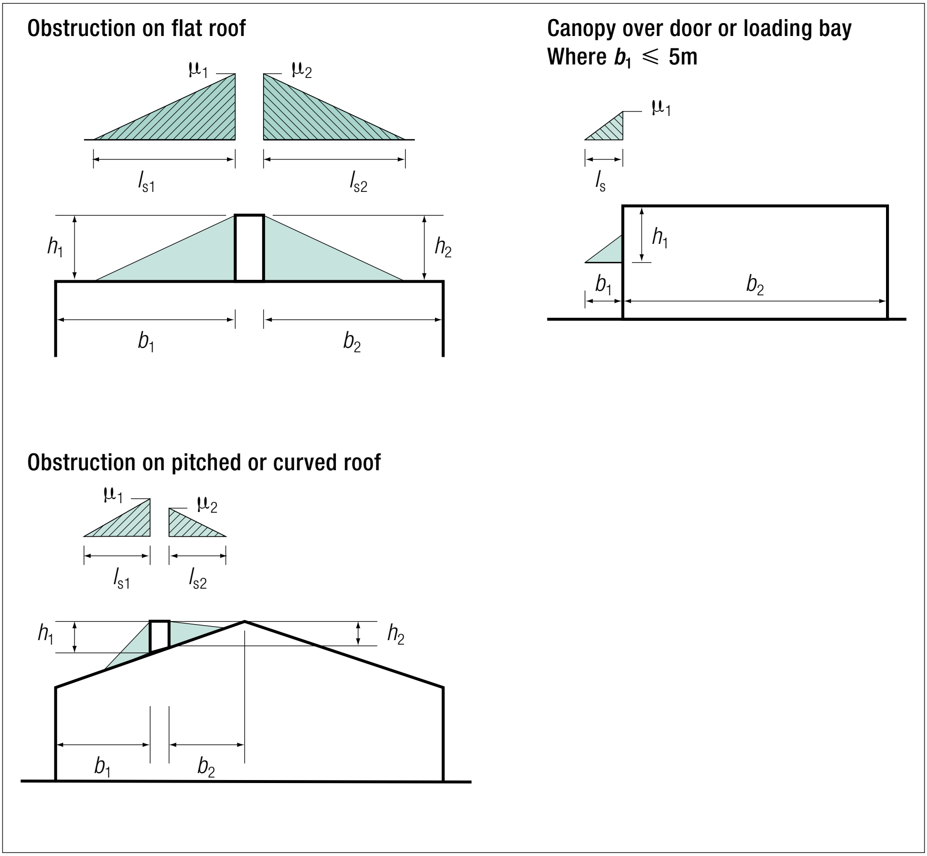


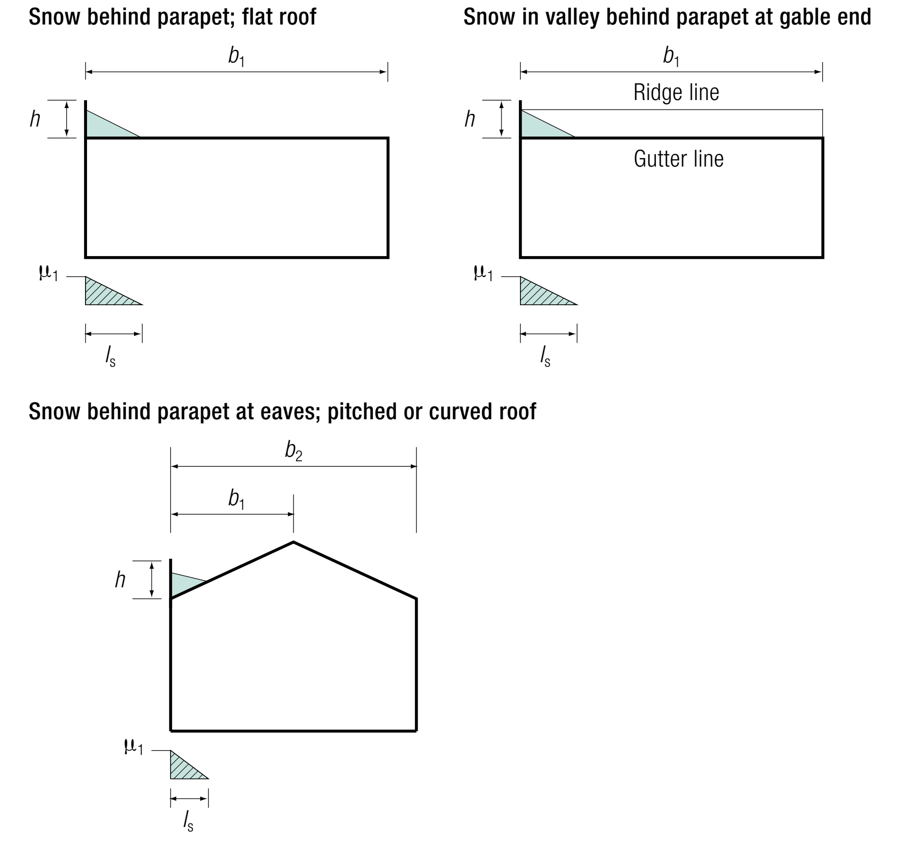
Fig 5.9 Shape coefficients for exceptional snow drifts for roofs where drifting occurs at projections and obstructions

The drift length (l_{si}) is taken as the lesser value of $5h$ or b_i, where $i = 1$ or 2. For the purposes of evaluating l_{si}, the value of h should be $h \leqslant 1$m.

(b) Parapets

Shape coefficients for drifting behind parapets are given below. The snow load shape coefficients for exceptional snow drifts that should be used for roofs where drifting occurs at parapets are given in Figure 5.10.

The shape coefficient given in Figure 5.10 is determined as the least value of:
$\mu_1 = 2h/s_k$
$\mu_1 = 2b/l_s$ where b is the larger of b_1 or b_2
$\mu_1 = 8$.



Note b_2 should be used in the calculation of shape coefficient

Fig 5.10 Shape coefficients for exceptional snow drifts for roofs where drifting occurs at parapets

The drift length l_s should be taken as the least value of $5h$, b_1 or 15m.

μ_1 is taken as dimensionless. The units to be used for the above calculations are kN/m^2 and m as appropriate.

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 300mm above the ridge.

5.7.3 Snow overhanging the edge of a roof

Snow overhanging the edge of a roof should be considered, but the guidance given in this Section applies only for sites at altitudes greater than 800m above sea level.

The design of those parts of a roof 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 as follows:

$$s_e = k\, s^2/3$$

where:

s_e is snow load per metre length due to the overhang (see Figure 5.11)

s is the most onerous undrifted snow load appropriate to the roof under consideration (see Section 5.6.3).

k is a coefficient to take account of the irregular shape of the snow, and $k = 9/s$, but $k \leqslant s$. ***Note*** In practice, this means that $k = s$ will almost always be the design situation, in which case, $s_e = s^3/3$.

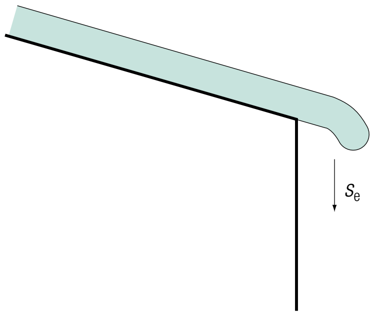


Fig 5.11 Snow overhanging the edge of a roof

5.7.4 Snow loads on snow-guards and other obstacles

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 this calculation the force F_s exerted by a sliding mass of snow, in the direction of slide, per unit length of the building should be taken as:

$F_s = s\, b \sin\alpha$

where:

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 Section 5.6.3)

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

α pitch of the roof, measured from the horizontal.

5.8 Bulk weight density of snow

The bulk weight density of snow varies. In general it increases with the duration of the snow cover and depends on the site location, climate and altitude.

Except where specified in Sections 5.1 to 5.7 indicative values for the mean bulk weight density of snow on the ground given in Table 5.4 may be used.

Table 5.4 Mean bulk weight density of snow

Type of snow	Bulk weight density kN/m^3
Fresh	1.0
Settled (several hours or days after its fall)	2.0
Old (several weeks or months after its fall)	2.5 – 3.5
Wet	4.0

Chapter 6: Wind actions

6.1 Introduction

This Chapter provides guidance on the quasi-static wind actions to be applied to common building types. Guidance for bridges, domes, vaulted roofs, lattice structures and elements is not included. This *Manual* excludes circular buildings and slender buildings whose height/inwind depth >5.0 and dynamic and aeroelastic building response, because these are not applicable to common building types; guidance on dynamic response is given in Chapter 6 of EN 1991-1-4[6] (EC1 Part 1-4).

EC1 Part 1-4 allows National choice (NDPs) in more than 50 clauses; in many cases the recommended values have not been used and different equations and values have been adopted for use in the UK. Consequently NDPs are included within this Chapter but, unlike other parts of this *Manual,* these NDPs have not been shown in bold because it would detract from the readability of the Chapter.

6.2 General

6.2.1 Scope

EC1 Part 1-4 gives guidance for the determination of wind action on buildings and civil engineering works and is applicable to land-based structures up to 200m tall, and their components and appendages. The scope of this standard is much wider than existing individual British Standards, such as BS 6399-2[35].

6.2.2 Design assisted by testing and measurement

EC1 Part 1-4 allows properly conducted and validated full scale measurements and wind tunnel testing to be used to determine the wind loads and structural response of a particular building. Wind tunnel studies might be required when the form of the building is not covered by EC1 Part 1-4 or where loading data are required in more detail than given in EC1 Part 1-4. The UK National Annex gives guidance on the requirements for wind tunnel testing.

6.2.3 Definitions

A basic list of definitions is provided in EC0 Section 1.5. Specific additional definitions for the wind actions part are:

Fundamental basic wind velocity $v_{b,0}$ – This is the mean wind velocity measured over a 10 minute averaging period at a height of 10m above flat open country terrain with an annual risk of exceedance of 0.02.

Basic wind velocity v_b – This is the fundamental basic wind velocity ($v_{b,0}$) modified to account for seasonal and directional effects.

Mean wind velocity $v_m(z)$ – This is the basic wind velocity (v_b) modified to account for the effects of terrain roughness and orography at height z above the ground.

Peak velocity pressure $q_p(z)$ – This is the potential pressure available from the kinetic energy of the basic wind velocity combined with the turbulent fluctuations of the wind.

Pressure coefficient c_p – A non dimensional ratio giving the effect of the wind on external or internal surfaces.

Orography factor c_o – This factor accounts for the effects of wind speed-up over hills, ridges and escarpments. This factor is also known as the topography factor in some codes and standards, e.g. BS 6399-2.

Characteristic values – The characteristic value of an action is its main representative value. For wind action the characteristic values have an annual probability of exceedance of 0.02, which is equivalent to a mean return period of 50 years.

6.3 Design situations

This Section describes the design situations that need to be considered. These are the standard design situations identified in accordance with EC0, such as Persistent, Transient and Accidental design situations. The following additional design situations should be taken into account:
– other actions such as snow, traffic or ice which modify the wind loads (see EC1 Part 1-3, BS EN 1991-2[36] and ISO FDIS 12494[37])
– changes to the structure during construction which could modify the wind loads (see EC1 Part 1-6)
– where windows or doors are assumed in the design to be shut under storm conditions, the effect of these being open or breaking should be treated as an accidental design situation
– the effects of fatigue. These only need to be considered for susceptible structures and are not covered in this *Manual*.

6.4 Wind velocity and peak velocity pressure

6.4.1 General

The wind is assumed to comprise a fluctuating wind speed component superimposed on the mean wind speed, the combined effect of which is used to determine the peak velocity pressure which is the main parameter used to determine the wind pressures and forces. The peak velocity pressure, $q_p(z)$, includes the effects of the National wind climate and local factors related to terrain roughness, orography, site altitude and directional and seasonal effects.

The UK National Annex has simplified the calculation of the peak velocity pressure $q_\mathrm{p}(z)$ for the majority of common building structures. The appropriate calculation route depends on the height of the building and whether orography is significant. Where the height z >50m *and* where orography is significant then the detailed procedure for calculating $q_\mathrm{p}(z)$ given in Appendix F should be used. In all other cases the simplified procedure in Section 6.4.5 may be used. Figure 6.1 defines the regions on a hill, ridge or escarpment where orography is significant. The effects of orography can be ignored when the average slope of the upwind terrain (considered over a distance of up to 10 times the height of the orographic feature) is less than 3°, a slope of 0.05 or 1:20. Where orography is significant the procedure given in Appendix G should be used.

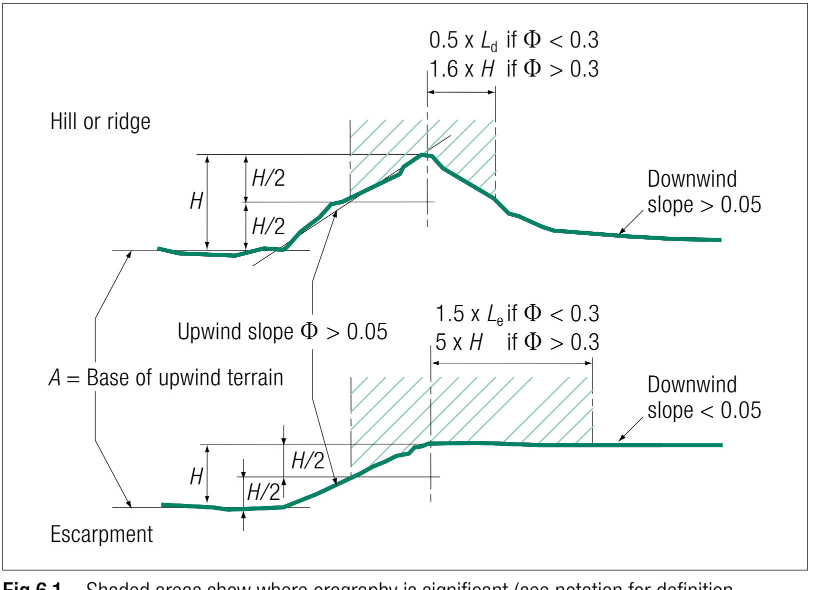


Fig 6.1 Shaded areas show where orography is significant (see notation for definition of symbols)

The flow chart in Figure 6.2 shows the simplified procedure for determining $q_p(z)$.

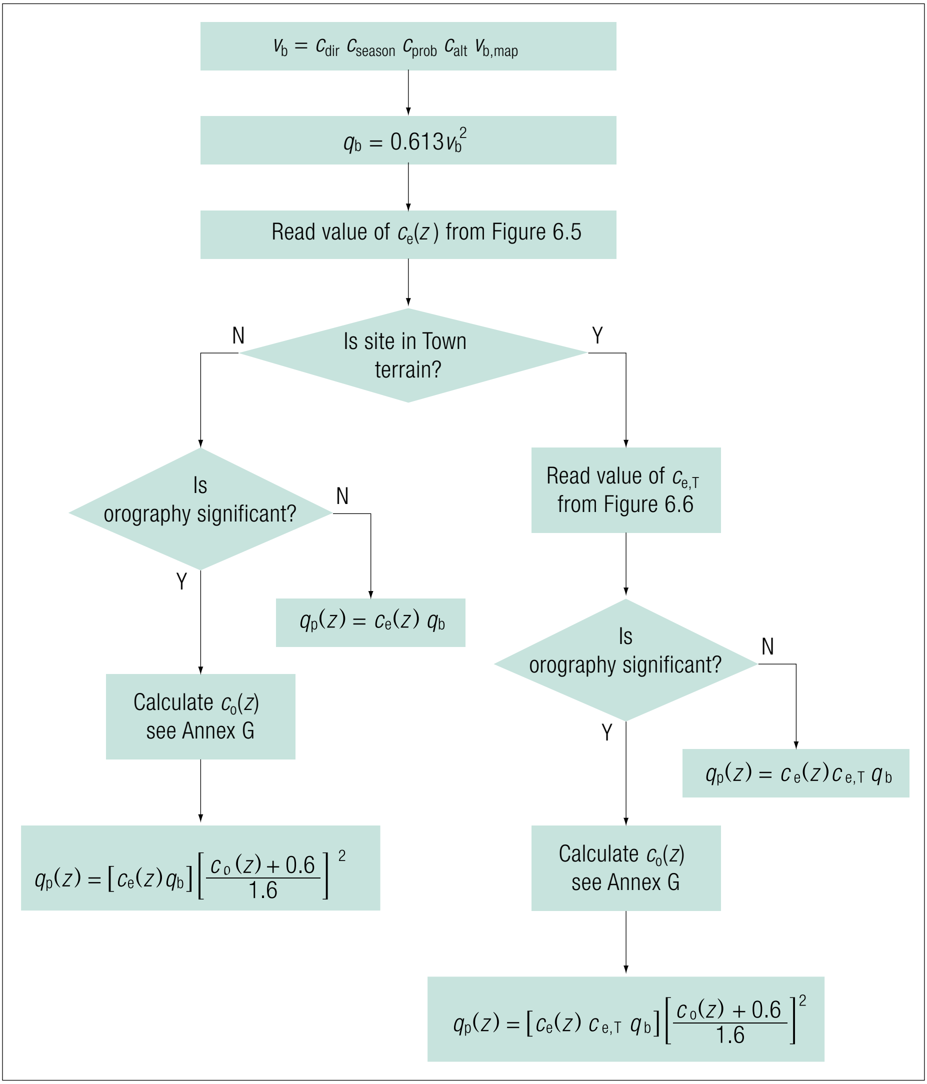


Fig 6.2 Flow chart for calculating peak velocity pressure for buildings on flat terrain or for buildings where orography is significant and $z \leqslant$ 50m

6.4.2 Basic wind velocity

The basic wind velocity, v_b, defined as a function of wind direction and time of year at 10m above ground for terrain category II, *called Country terrain in the UK National Annex* is given by:

$$v_b = c_{dir}\, c_{season}\, c_{prob}\, v_{b,0}$$

where:

c_{dir} is the directional factor given in Table 6.1

c_{season} is the season factor given in Table 6.2**.** c_{season} should only be used for temporary buildings erected for less than one year, or during the construction phase for temporary works

c_{prob} is the probability factor = 1.0 for the general case of an annual probability of exceedance of 0.02, (equivalent to a mean return period of 50 years). The value of c_{prob} for other probabilities of exceedance is given by:

$$c_{prob} = \left(\frac{1 - 0.2\ln(-\ln(1-p))}{1 - 0.2\ln(-\ln(0.98))}\right)^{1/2}$$

where p is the required *annual* risk of exceedance.

Note A probability factor <1.0 should be used with caution for temporary works. Safeguards should be in place in case of project overrun.

$v_{b,0}$ is the fundamental value of the basic wind velocity given by:

$$v_{b,0} = v_{b,map}\, c_{alt}$$

where:

$v_{b,map}$ is given in Figure 6.3

c_{alt} is the altitude factor given by:

$c_{alt} = 1 + 0.001\, A$ for $z \leqslant 10$m

$c_{alt} = 1 + 0.001\, A\, (10/z)^{1/5}$ for $z > 10$m

where:

A is the altitude of the site in metres above mean sea level

z is either z_s as defined in Figure 6.10 or z_e the height of the part above ground as defined in Figure 6.12.

Note 1 The equation for c_{alt} for $z \leqslant 10$m can be used for any building height and is always conservative.

Note 2 On hilly sites where orography is significant, as defined by the shaded zones in Figure 6.1, altitude A should be measured from the upwind base of the orographic feature for each wind direction considered.

The basic velocity pressure, $q_{b,}$ is given by:

$$q_b = \tfrac{1}{2}\,\rho\, v_b^2$$

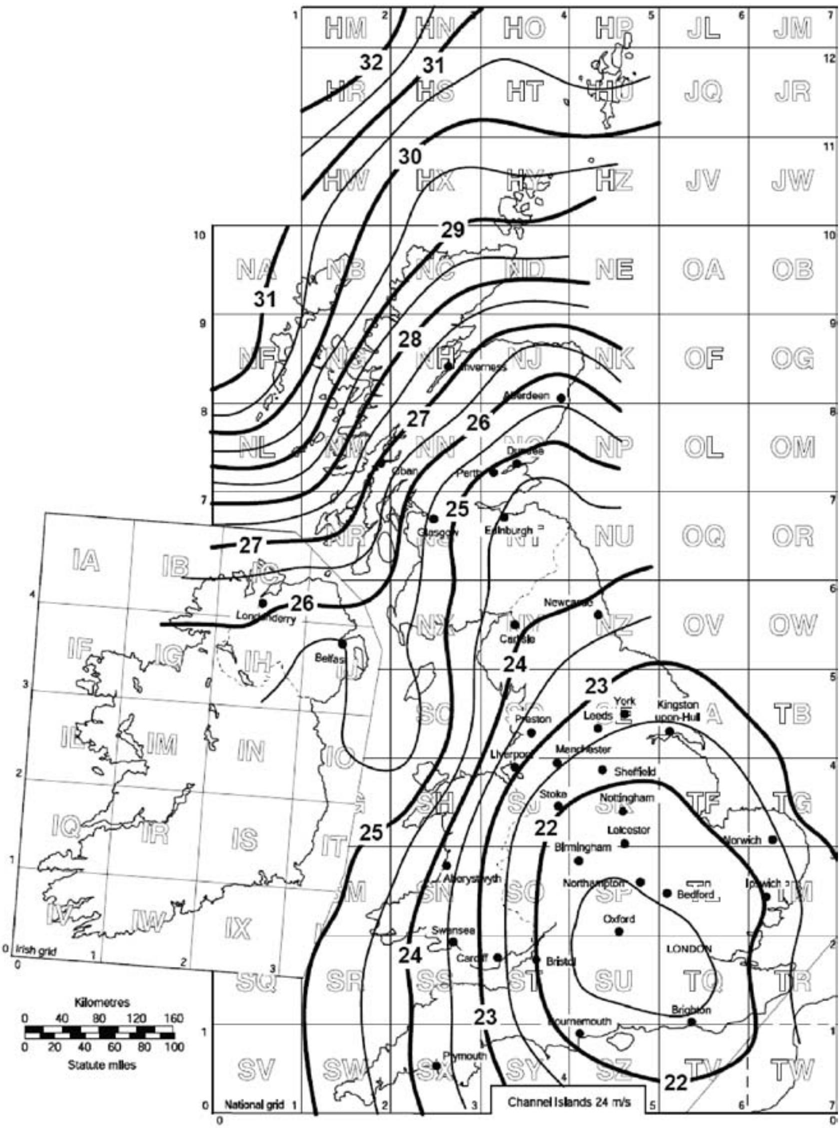
where ρ is the air density, which in the UK is taken as 1.226kg/m^3

Table 6.1 Directional factor c_{dir}

Direction	0º	30º	60º	90º	120º	150º	180º	210º	240º	270º	300º	330º
c_{dir}	0.78	0.73	0.73	0.74	0.73	0.80	0.85	0.93	1.00	0.99	0.91	0.82

Table 6.2 Seasonal factor c_{season}

<table>
<tr><th>Months</th><th>1 month</th><th colspan="2">2 months</th><th colspan="4">4 months</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">0.90</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 The factor for the 6 month winter period October to March inclusive is 1.00 and for the 6 month summer period April to September inclusive is 0.84.</td></tr>
</table>



Note Although the above map includes the Republic of Ireland, the *Manual* does not necessarily cover the design of buildings to be constructed there.

Fig 6.3 Value of $V_{b,map}$ (m/s)

6.4.3 Terrain roughness

EC1 Part 1-4 has five terrain categories which are described in Table 6.3. Appendix H has photographs showing these terrain categories.

The UK National Annex has simplified the EC1 Part 1-4 classification of terrain categories into three, Sea, Country and Town, defined as follows:
– Sea terrain in the UK is terrain category 0.
– Country terrain in the UK combines terrain categories I and II which have been considered together to give a single category.
– Town terrain in the UK combines terrain categories III and IV which have been considered together to give a single category. Note that areas of permanent forest should be treated as Town terrain.

The upwind terrain roughness should be determined for each 30° wind sector using the descriptions in Table 6.3 and the photographs in Appendix H. Variations in terrain roughness of less than 10% by area may be ignored. If in doubt then take the smoothest (most onerous) roughness category. An Ordnance Survey map or road atlas can be a useful aid to give an indication of the terrain category. Areas of Town terrain are usually coloured pink or brown (depending on the map). It is generally safe to take these coloured areas as representing the extent of areas of Town terrain.

Table 6.3 EC1 Part 1-4 terrain categories

Terrain category	
0	Sea or coastal area exposed to the open sea
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)
IV	Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15m

6.4.4 Closely spaced buildings and obstacles

In Town terrain, closely spaced surrounding buildings may provide shelter which can cause the wind to behave as if the ground level was raised to a displacement height, h_{dis}, see Figure 6.4, lifting the profile of the peak velocity pressure*.* The displacement height should be subtracted from the actual height of the structure to give a reduced effective height ($z - h_{dis}$).

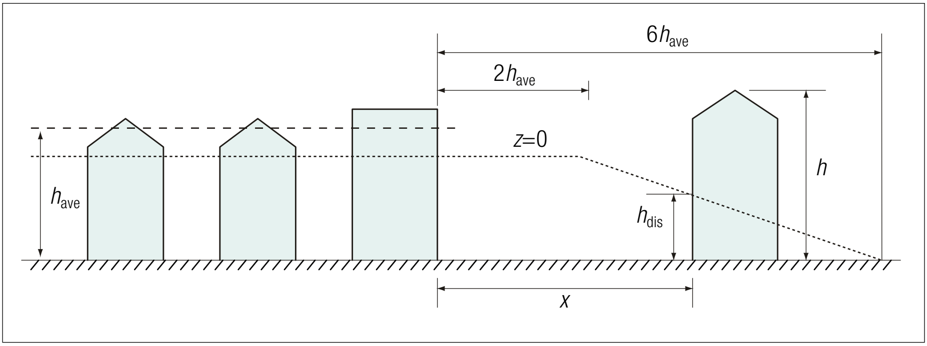


Fig 6.4 Obstruction height and upwind spacing

h_{dis} may be determined as follows:

$x \leqslant 2h_{ave}$	h_{dis} is the lesser of $0.8h_{ave}$ or $0.6h$
$2h_{ave} < x < 6h_{ave}$	h_{dis} is the lesser of $1.2h_{ave} - 0.2x$ or $0.6h$
$x \geqslant 6h_{ave}$	$h_{dis} = 0$.

In the absence of more accurate information, for sites in Town terrain estimate the value of h_{dis} assuming a typical storey height of 3m. For sites in Country terrain $h_{dis} = 0$.

6.4.5 Simplified procedure for peak velocity pressure

The simplified procedure for determining peak velocity pressure may be used for all buildings where orography is not significant and for buildings ⩽50m tall where orography is significant.

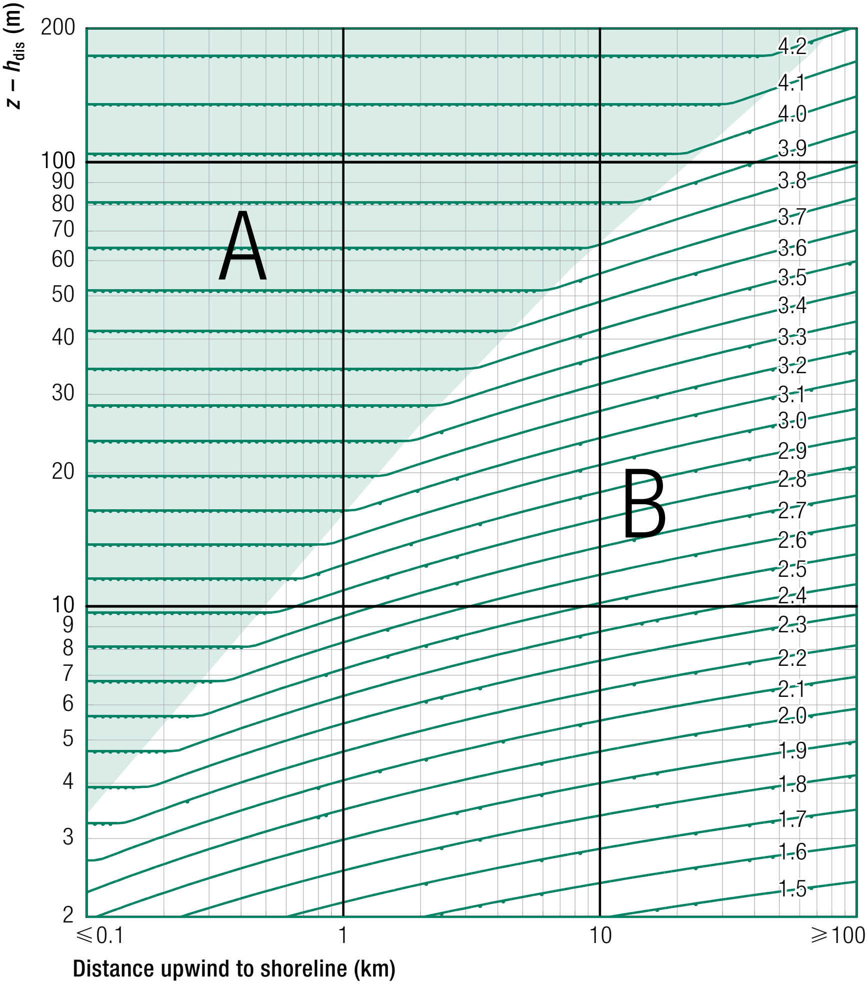
When orography is not significant the peak velocity pressure, $q_p(z)$, is given by:
$q_p(z) = c_e(z)\ q_b$ for sites in Country terrain.
$q_p(z) = c_e(z)\ c_{e,T}\ q_b$ for sites in Town terrain.

When orography is significant *and* z ⩽ 50m the peak velocity pressure, $q_p(z)$, is given by:
$q_p(z) = [q_b\ c_e(z)]\ [(c_o(z) + 0.6)/1.6]^2$ for sites in Country terrain
$q_p(z) = [q_b\ c_e(z)\ c_{e,T}]\ [(c_o(z) + 0.6)/1.6]^2$ for sites in Town terrain

where:
$c_e(z)$ is the exposure factor, given in Figure 6.5
$c_{e,T}$ is the exposure correction factor for sites in Town terrain, given in Figure 6.6
$c_o(z)$ is the orography factor, The procedure given in Appendix G should be used for determining the orography factor. Only sites that lie within the shaded area of Figure 6.1 need to be considered. Outside these zones the orography factor may be taken as 1.0.

Note The appropriate value of h_{dis} should be used in Figures 6.5 and 6.6. For sites in Country terrain $h_{dis} = 0$. For sites in Town terrain h_{dis} is given in Section 6.4.4.

When using Figures 6.5 and 6.6 the relevant zone, A, B or C, should be noted. These zones are used with Table 6.4 to determine the appropriate size effect factor.

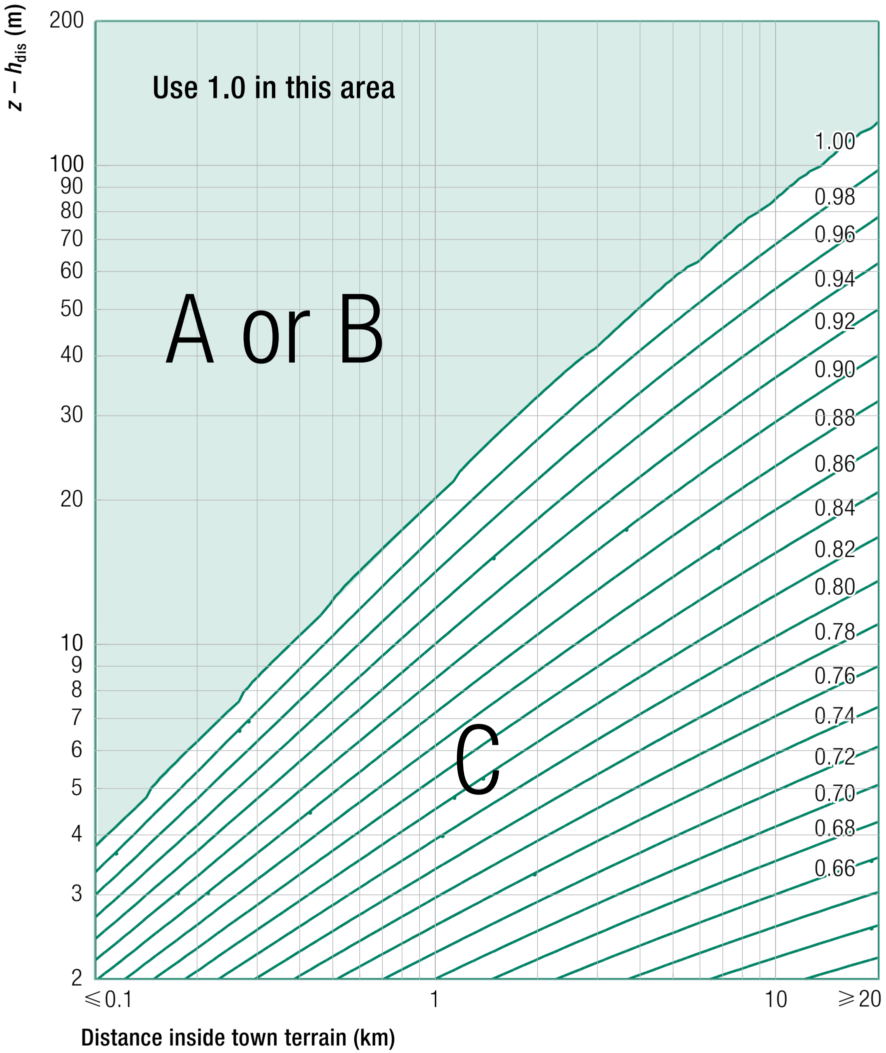


Notes

a z is the height at which q_p is determined.

b Zones A and B are indicated for use in Table 6.4.

Fig 6.5 Exposure factor $c_e(z)$



Notes

a *z* is the height at which q_p is determined.

b Zones A,B and C are indicated for use in Table 6.4.

Fig 6.6 Exposure correction factor $c_{e,T}$ for Town terrain

The peak velocity pressure should be determined for each 30° wind sector for the most precise determination of wind loads. However, there are alternative, more conservative approaches which might be more appropriate in some cases. These are described below:

Single worst case irrespective of direction. In this approach c_{dir} is taken as 1.0 for all directions and single values of $c_e(z)$ and $c_{e,T}$ are determined using the closest distance to sea and the closest distance to the edge of town in Figures 6.5 and 6.6. This approach will give the most conservative result and is not recommended when orography is significant.

Four orthogonal cases. In this approach the four worst case values of c_{dir} are determined for each 90° wind sector (±45° about orthogonal to each building face). Values of $c_e(z)$ and $c_{e,T}$ are determined for each 90° sector taking the closest distance to sea and closest distance to the edge of town in each sector. The orography factor c_o(z) is determined by taking the most onerous value for each 90° sector of the orographic feature, relative to the orthogonal building faces. This approach will give a good compromise between conservatism and calculation effort.

Twelve wind directions. This is the full directional approach; c_{dir}, $c_e(z)$, $c_{e,T}$ and c_o(z) are determined for each 30° sector. This approach gives the lowest design loads.

6.4.6 Large and considerably higher neighbouring structures

Where the building is close to a neighbouring building that is at least twice the average height of surrounding structures then increased wind velocities could occur.

As a first approximation, the peak velocity pressure at height z_n ($z_e = z_n$) may be used for the design of the nearby surrounding buildings, where z_n is the notional height of the building under consideration, see Figure 6.7.

$$x \leqslant r \qquad z_n = \tfrac{1}{2} r$$

$$r < x < 2r \qquad z_n = \tfrac{1}{2}\left(r - \left(1 - \frac{2h_{low}}{r}\right)(x - r)\right)$$

$$x \geqslant 2r \qquad z_n = h_{low}$$

in which the radius r is:

$$r = h_{high} \qquad \text{if } h_{high} \leqslant 2\,d_{large}$$

$$r = 2\,d_{large} \qquad \text{if } h_{high} > 2\,d_{large}$$

The building height h_{low}, the radius r, the distance x and the dimensions d_{small} and d_{large} are defined in Figure 6.7. Increased wind velocities can be disregarded when h_{low} is more than half the height h_{high} of the high building.

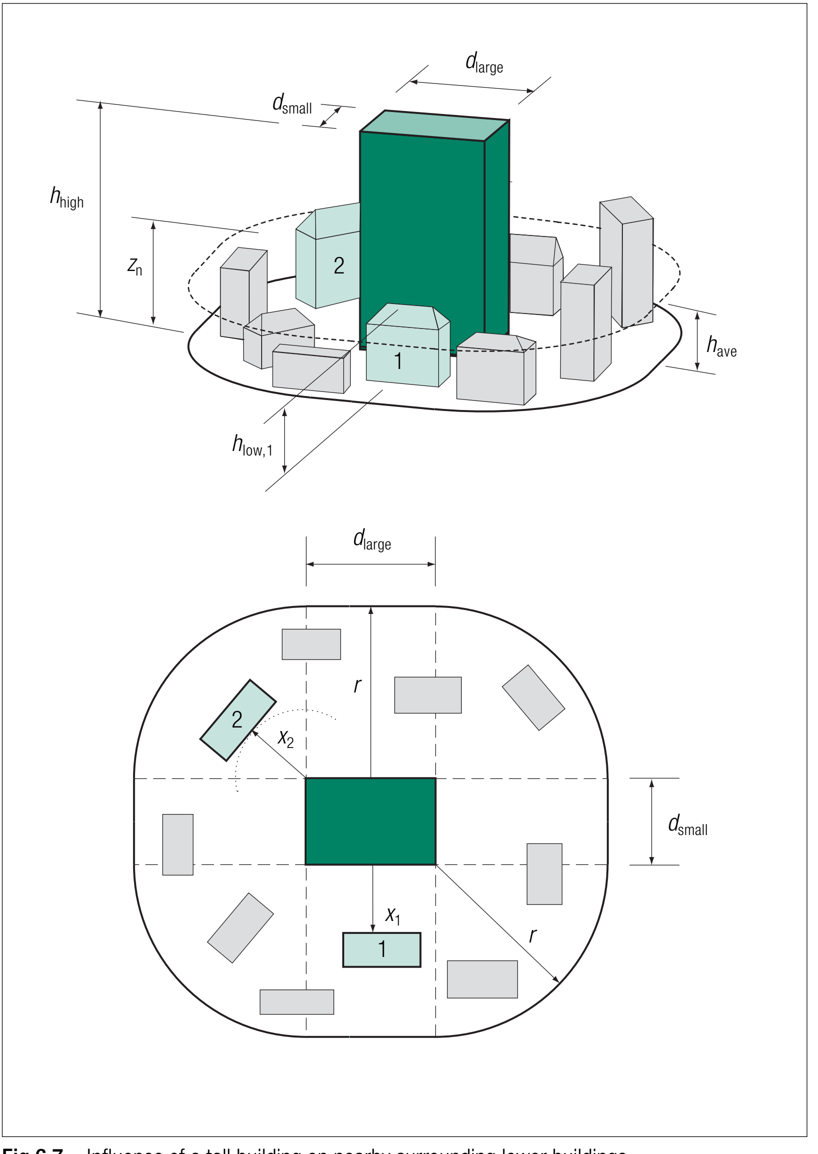


Fig 6.7 Influence of a tall building on nearby surrounding lower buildings

6.5 Wind pressures and forces

6.5.1 General

This Section gives procedures for calculating the wind actions (wind pressures and forces) on a structure or structural element.

6.5.2 Wind pressures on a surface

The wind can act on both the external and internal surfaces of a structure or element and where appropriate both should be considered. External pressures are generated by the direct action of the wind blowing on an external surface. Internal pressures are generated by the overall effect of wind blowing in and out of all the openings and the general porosity of the building envelope.

The wind pressure w_e acting on an external surface should be calculated using:

$$w_e = q_p(z_e)\, c_{pe}$$

where:
$q_p(z_e)$ is the peak velocity pressure at height z_e
z_e is the reference height for the external pressure
c_{pe} is the external pressure coefficient.

The wind pressure w_i acting on an internal surface should be calculated using:

$$w_i = q_p(z_i)\, c_{pi}$$

where:
$q_p(z_i)$ is the peak velocity pressure at height z_i
z_i is the reference height for the internal pressure
c_{pi} is the internal pressure coefficient.

The net pressure acting on a surface such as a wall or roof is the difference between the pressures on either side of that element, i.e. $w_{net} = w_e - w_i$. The sign of the pressures should be taken into account when calculating the net pressure. Pressures acting towards the surface are taken as positive and pressures acting away from the surface are taken as negative. Figure 6.8 gives examples of external and internal pressures on surfaces.

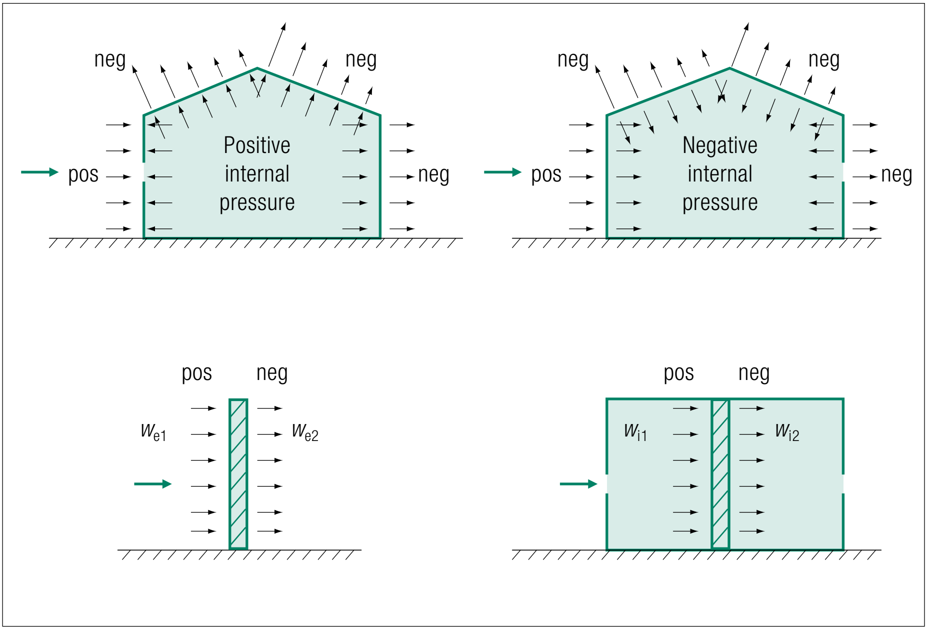


Fig 6.8 Examples of wind pressure acting on internal and external surfaces

6.5.3 Wind forces

Wind forces on a complete structure, structural element or component can be determined from pressure coefficients by vectorial summation of the external and internal surface pressures (including friction effects where appropriate) using:

External forces $$F_{w,e} = c_s c_d \sum_{\text{surfaces}} w_e A_{ref}$$

Internal forces $$F_{w,i} = \sum_{\text{surfaces}} w_i A_{ref}$$

Friction forces $$F_{fr} = c_{fr} q_p (z_e) A_{fr}$$

where:

c_s is the size factor as defined in Section 6.6

c_d is the dynamic factor as defined in Section 6.6

A_{ref} is the reference area of the individual surface

c_{fr} is the friction coefficient derived from Section 6.7.5

A_{fr} is the area of external surface parallel to the wind, given in Section 6.7.5.

The overall force is given by summing $F_{w,e} + F_{w,i} + F_{fr}$ taking account of the signs of the forces.

The friction force F_{fr} can act on all surfaces (roofs and walls) parallel to the wind. The effects of wind friction on the surface can be disregarded when the total area of all surfaces parallel to (or at a small angle to) the wind is $\leqslant$ 4 times the total area of the windward and leeward external surfaces. Small is not defined in EC1 Part 1-4, however angles within ±5° should satisfy this criterion.

Obtaining the wind forces by summation of the pressures on the windward and leeward faces can be conservative because this calculation assumes that the maximum forces on the windward and leeward faces are fully correlated. A reduction factor given in Section 6.7.2.2 may be applied to the summation of the pressures acting on the windward and leeward surfaces, i.e. both walls and to roofs, to account for this non-simultaneous wind action. This factor is given in the Note to Table 6.5

6.6 Structural factor $c_s c_d$

6.6.1 General

This Section gives guidance on the determination of the structural factor $c_s c_d$ which has two parts. The size effect factor c_s accounts for the lack of correlation of the wind gusts over the surfaces of a structure and the dynamic factor c_d accounts for the dynamic response of the structure in its fundamental mode of vibration. The c_s factor has a value $\leqslant$ 1.0. The larger the size of the structure or element the smaller the value of c_s. The c_d factor has a value $\geqslant$ 1.0. The more dynamically sensitive the structure the larger the value of c_d. The EC1 Part 1-4 recommended procedure combines these two factors but allows them to be separated in National Annexes. The UK National Annex allows these factors to be separated.

6.6.2 Determination of $c_s c_d$

In the UK the $c_s c_d$ factor may be considered as a single factor or c_s and c_d may be considered separately. A less conservative result will generally be obtained by considering $c_s c_d$ as separate factors. However, there are occasions where for speed or simplicity it is more convenient to treat $c_s c_d$ as a single factor. In the following cases the combined factor $c_s c_d$ can safely (but generally conservatively) be taken as 1.0.

(a) Buildings with a height less than 15m
(b) Cladding panels and elements
(c) Framed buildings less than 100m tall which have structural walls and whose height is less than 4 times the in-wind depth *d*, see Figure 6.10.

Table 6.4 and Figure 6.9 can be used to determine separate values of c_s and c_d. The values in Table 6.4 and Figure 6.9 have been derived from expressions 6.2 and 6.3 in EC1 Part 1-4.

Table 6.4 gives values of c_s based on the sum of the width (b) and height (h) of the building, building part or element. This Table may be used to determine c_s values for the complete building or part of a building or for individual cladding and roofing elements.

Figure 6.9 gives values of c_d for a range of typical building types in terms of the logarithmic decrement of structural damping, δ_s. This Figure has been derived assuming v_b = 26m/s, n_1h = 46, $z_e = 0.6h$. Figure 6.10 shows the structural dimensions and reference heights to be used with Figure 6.9. For buildings outside the ranges of Table 6.4 and Figure 6.9 the procedures in Annex B of EC1 Part 1-4 should be used.

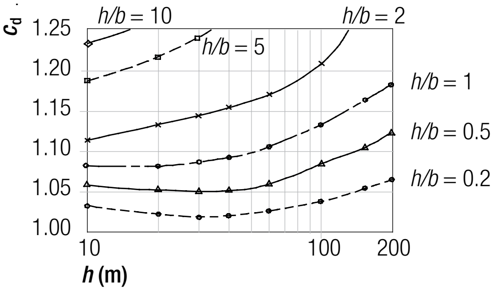
Figure 6.9 can only be used when the building corresponds to one of the general forms shown in Figure 6.10 and when only the along-wind response in the fundamental mode of vibration is significant.

Table 6.4 Size effect factor c_s for zones A, B and C

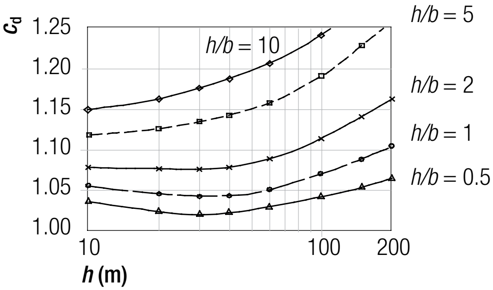
$b+h$ (m)	$(z-h_{dis})$ = 5m			$(z-h_{dis})$ = 10m			$(z-h_{dis})$= 30m			$(z-h_{dis})$ = 50m			$(z-h_{dis})$ = 200m		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
1	0.99	0.98	0.97	0.99	0.99	0.97	0.99	0.99	0.98	0.99	0.99	0.99	0.99	0.99	0.99
5	0.96	0.96	0.92	0.97	0.96	0.93	0.98	0.97	0.95	0.98	0.98	0.96	0.98	0.98	0.98
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

Notes
b = cross wind width of the building or building part or the width of an individual element.
h = height of the building or building part or the length of an individual element.

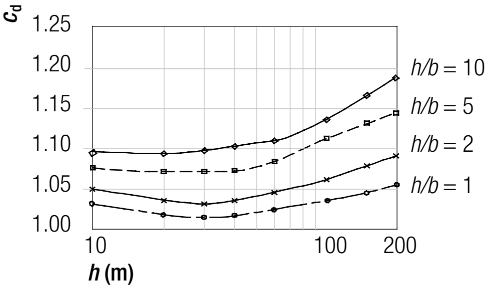
Welded steel unclad frames,
δ_s = 0.025



Bolted steel and concrete frames,
δ_s = 0.05



Portal sheds and similar light structures with few internal walls,
δ_s = 0.1



Framed buildings with structural walls around lifts and stairs,
δ_s = 0.2

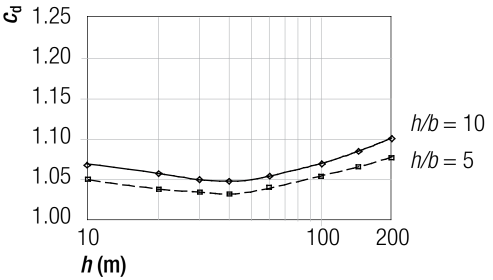


Fig 6.9 Dynamic factor c_d (for $v_b = 26$m/s, $n_1 h = 46$, $z_e = 0.6h$)

The following buildings may be assumed to respond statically and the dynamic factor c_d may be taken as 1.0:
– framed buildings that have structural walls around lifts and stairwells with other internal masonry walls (e.g. apartment buildings)
– buildings of masonry construction
– timber-framed housing
– cladding panels and elements.

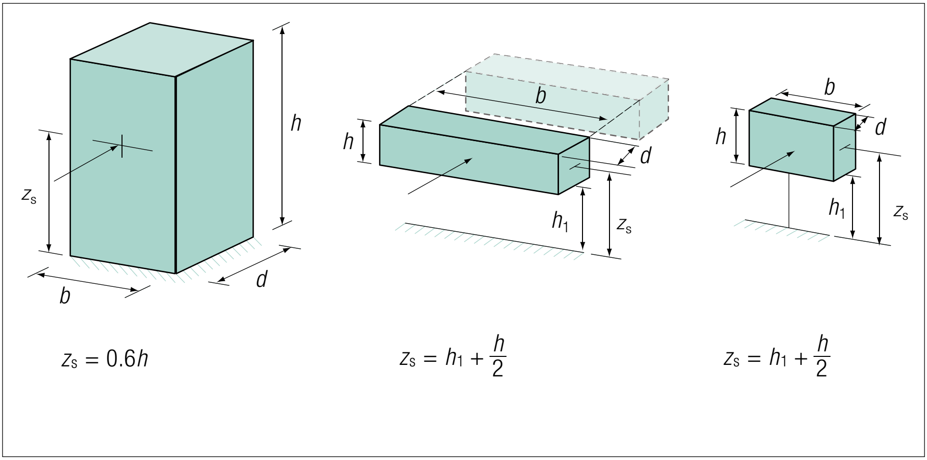


Fig 6.10 General shapes of structures for use with Figure 6.9

6.7 Pressure and force coefficients

6.7.1 General

6.7.1.1 Introduction

This Section gives values of the appropriate aerodynamic coefficients for buildings and elements of buildings.

The aerodynamic coefficients included in this Section are:
– internal and external pressure coefficients
– overall force coefficients
– net pressure coefficients
– friction coefficients.

6.7.1.2 Choice of aerodynamic coefficient

Pressure coefficients are used for buildings and cladding elements. Both internal and external pressure coefficients should be considered.

Net pressure coefficients and overall force coefficients are used for canopy roofs. Net pressure coefficients are used for free-standing walls, parapets and fences. Net pressure coefficients give the combined effect of wind pressures acting on both sides of the roof or wall element considered.

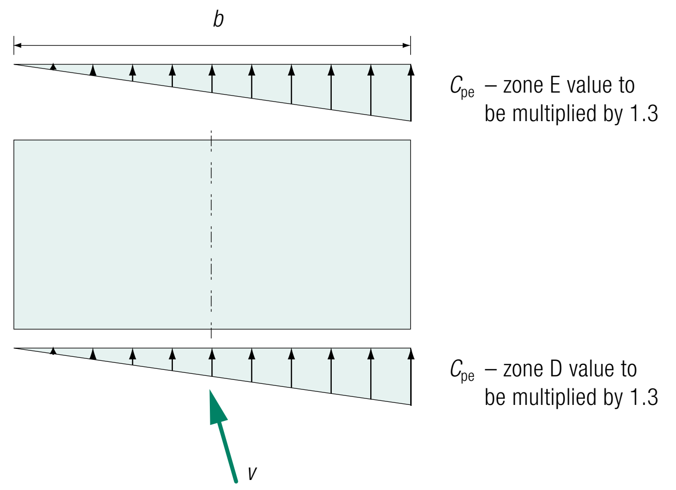
Friction coefficients are used for wall and roof surfaces.

6.7.1.3 Asymmetric and counteracting pressures and forces

The short term fluctuations of wind gusts over the surface of a structure may give rise to asymmetric loading. The effects of asymmetric loading, for example torsion in buildings with a single central core, only need to be taken into account for those structures susceptible to this form of loading.

The recommended procedure in EC1 Part 1-4 to account for torsion should not be used in the UK, Figure 6.11 should be used instead.

Rules for asymmetric loading of free-standing canopies are given in Section 6.7.3. For other structural forms an allowance for asymmetric loading can be made by completely removing the wind load from those parts of the structure where it will have a beneficial effect.



The zones and C_{pe} values are given in Table 6.5 and Figure 6.13

Fig 6.11 Pressure distribution used to take torsional effects into account

6.7.1.4 Effects of ice and snow

If the shape of the structure or element is changed by an accumulation of ice or snow then its effect on the wind loads should be taken into account. EC1 Part 1-3 provides guidance on snow drift shapes.

6.7.2 Pressure coefficients for buildings

6.7.2.1 General

The values of external pressure coefficients, c_{pe}, depend on the size of the loaded area. $c_{pe,1}$ values are given for loaded areas of $\leqslant$ 1m^2 and $c_{pe,10}$ values for loaded areas of > 10m^2. EC1 Part 1-4 gives a recommended procedure for determining the c_{pe} value for loaded areas between 1m^2 and 10m^2. This recommended procedure should not be used in the UK. In the UK the $c_{pe,1}$ values should be used for all areas $\leqslant$ 1m^2 and the $c_{pe,10}$ values should be used for all areas >1m^2. The $c_{pe,1}$ values should be used for small cladding elements and fixings and the $c_{pe,10}$ values for larger cladding elements and for overall structural loads.

c_{pe} values are given for orthogonal wind directions 0°, 90° and where appropriate 180°. These represent the worst case values over the range of wind directions of ±45° about each orthogonal axis.

No specific coefficients are given for overhanging or protruding roofs. In these cases the pressure on the top surface of the overhanging roof is determined from the appropriate roof zone and the pressure on the underside is taken as that on the adjacent part of the vertical wall.

6.7.2.2 Vertical walls of rectangular plan buildings

The reference height z_e for a wall or wall element is always taken at the top of the wall or element considered. The pressures on vertical walls may be considered as a number of discrete parts depending on the *h/b* ratio of the building as shown in Figure 6.12. The velocity pressure should be assumed to be uniform over each horizontal strip considered. The following three cases should be considered:

- A building, whose height *h* is less than *b* should be considered to be one part.
- A building, whose height *h* is greater than *b*, but less than 2*b*, may be considered as two parts, comprising: a lower part extending upwards from the ground with a height equal to *b* and an upper part consisting of the remainder.
- A building, whose height *h* is greater than 2*b* may be considered as multiple parts, comprising: a lower part extending upwards from the ground by a height equal to *b*; an upper part extending downwards from the top by a height equal to *b* and a middle region, between the upper and lower parts, which may be divided into horizontal strips with a height h_{strip}.

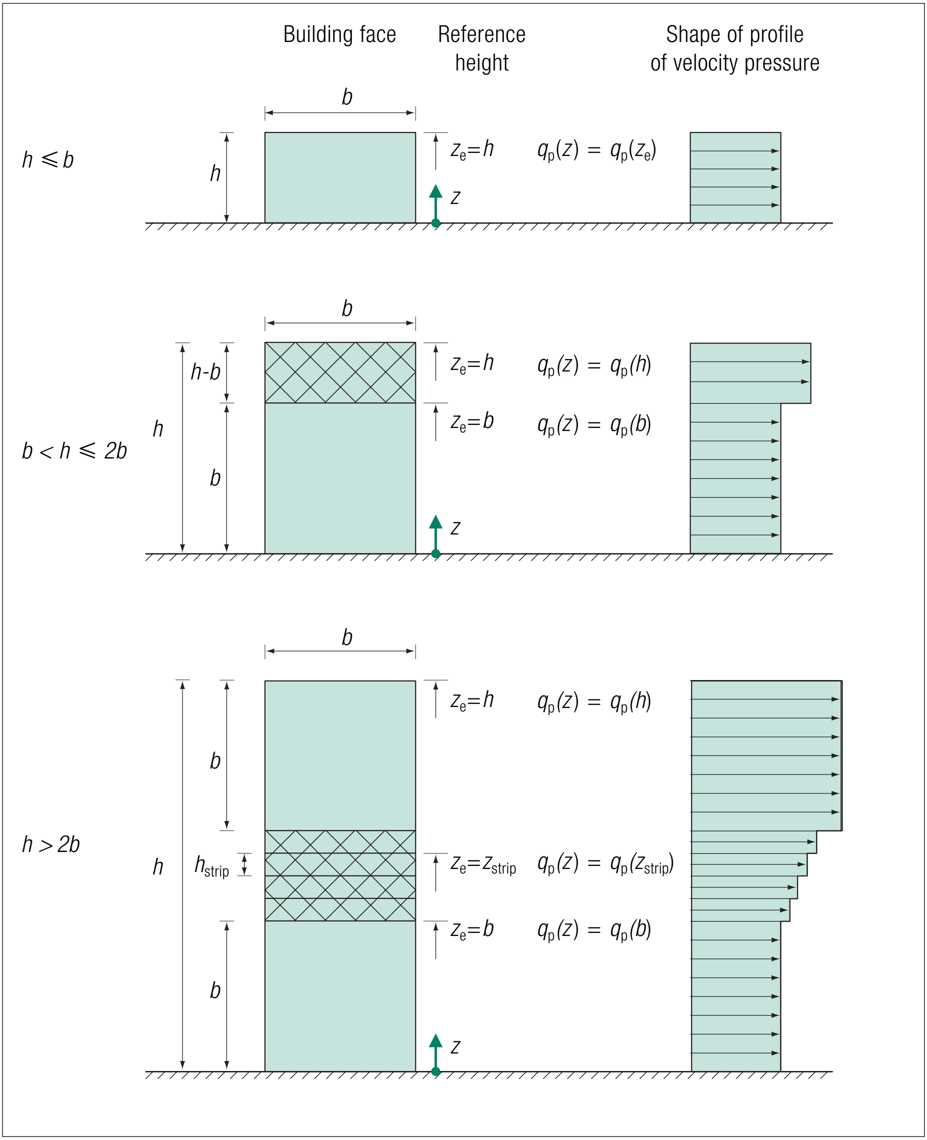
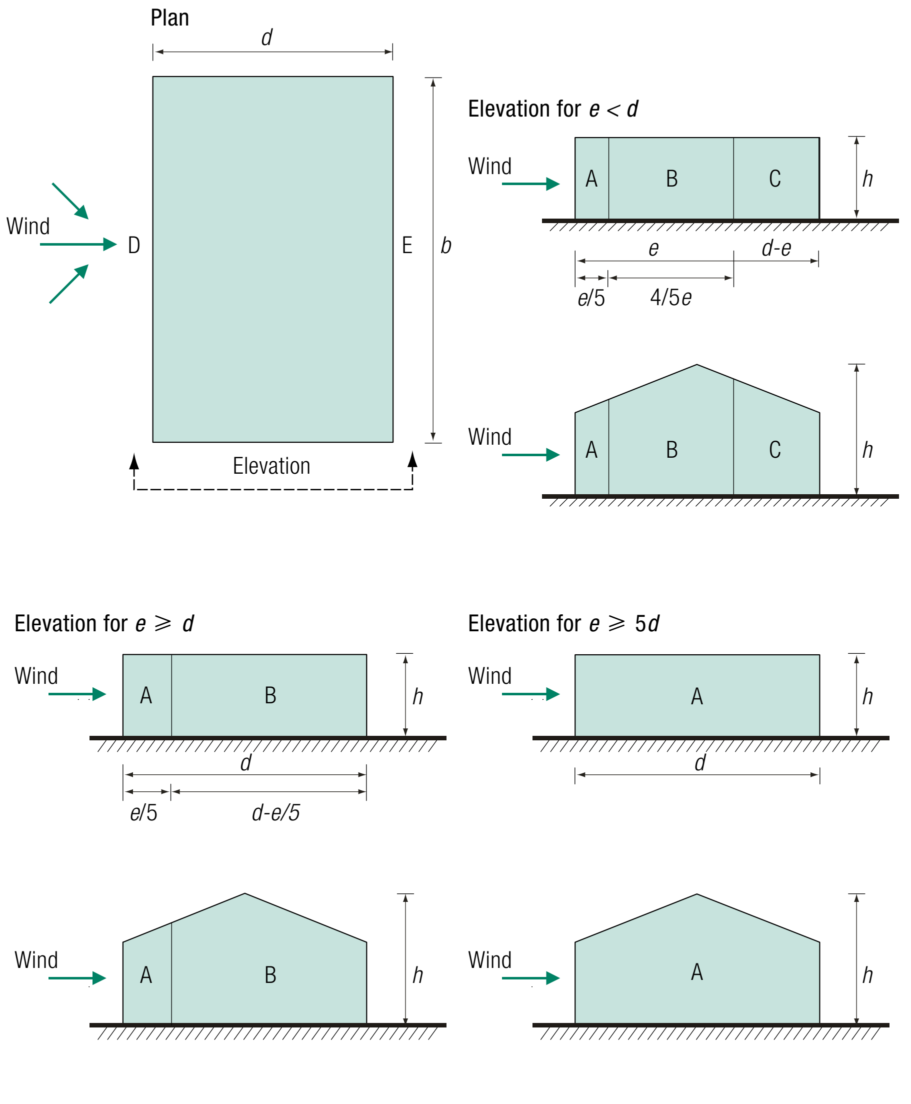


Fig 6.12 Key for division by parts of vertical walls showing reference height, z_e, dependence on h and b and corresponding velocity pressure profile

The external pressure coefficients $c_{pe,10}$ and $c_{pe,1}$ for the walls of rectangular plan buildings are defined in Figure 6.13 and the values are given in Tables 6.5 and 6.6.



$e = b$ or $2h$, whichever is smaller
b: crosswind dimension

Fig 6.13 Key to loaded zones on vertical walls

Table 6.5 External coefficients ($c_{pe,10}$) 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	
$\leqslant$ 0.25	-1.2	-1.4	-0.8	-1.1	-0.5		+0.7	+1.0	-0.3	

Note The lack of correlation of wind pressures between the windward and leeward side may be considered as follows. For buildings with $h/d \geqslant 5$ the resulting force is multiplied by 1. For buildings with $h/d \leqslant 1$, the resulting force is multiplied by 0.85. For intermediate values of h/d, linear interpolation may be applied.

For the determination of overall loads on buildings, the net pressure coefficients in Table 6.6 may be used instead of the sum of the pressure coefficients for zones D and E. The factor accounting for lack of correlation between the front and rear faces may also be applied to the net pressure coefficients.

Table 6.6 Net pressure coefficients for vertical walls of rectangular buildings

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

Notes

a The coefficients may be applied to non-vertical walls within ±15º of vertical, walls with a slope >15° should be treated as roofs.

b Where the walls of two buildings face each other and the gap between them is less than e, wind funnelling may accelerate the flow and make the pressure coefficients in zones A, B and C more negative. The following rules should be used to determine when funnelling will occur:

(1) where the gap between the buildings is $<e/4$ or $>e$, the isolated coefficient values should be used

(2) where the gap between the buildings is $>e/4$ and $<e$, either:

(i) use the funnelling values, conservatively, or

(ii) take 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, and interpolate linearly according to the actual gap

(3) where the two buildings are sheltered by upwind buildings, such that $(z_e - h_{dis}) < 0.4 z_e$ for the lower of the two buildings, then funneling may be disregarded

(4) The external pressure coefficients for side faces affected by funnelling should be taken as: -1.6 for Zone A, -0.9 for Zone B and -0.9 for Zone C.

6.7.2.3 Flat roofs

Flat roofs are defined as having a roof slope (α) of $-5° < \alpha < 5°$.

The reference height for flat roofs and roofs with curved or mansard eaves should be taken as h. The reference height for flat roofs with parapets should be taken as $z_e = h + h_p$, see Figure 6.14. The wind loads on the parapet may be determined by treating it as a free-standing wall of height h_p. The reference height for the parapet should be taken as $z_e = h + h_p$, see Section 6.7.4.

The flat roof should be divided into zones as shown in Figure 6.14. The pressure coefficients for each zone are given in Table 6.7.

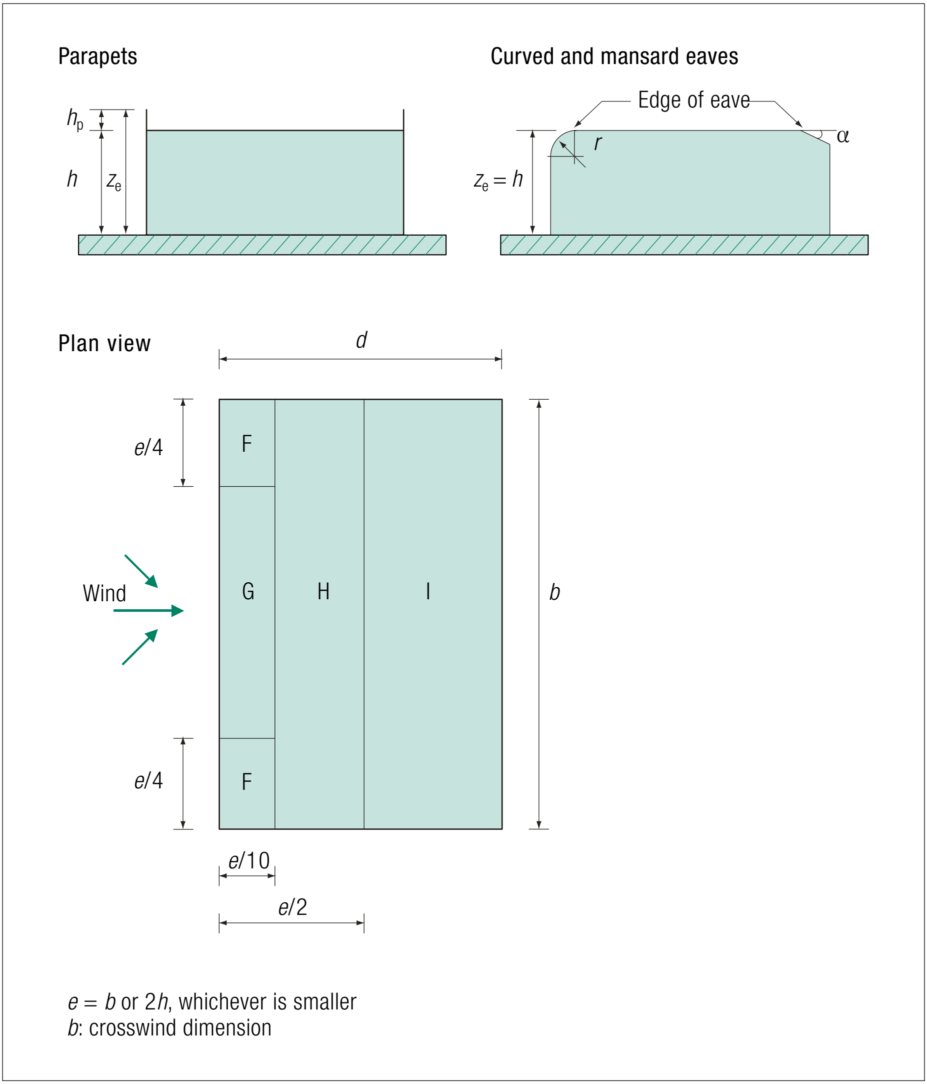


Fig 6.14 Key to loaded zones on flat roofs

Table 6.7 External pressure coefficients for flat roofs

Roof type		Zone F $c_{pe,10}$	F $c_{pe,1}$	G $c_{pe,10}$	G $c_{pe,1}$	H $c_{pe,10}$	H $c_{pe,1}$	I $c_{pe,10}$	I $c_{pe,1}$
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	
	$h_p/h \geqslant 0.10$	-1.2	-1.8	-0.8	-1.4	-0.7	-1.2	+0.2	
								-0.2	
Curved Eaves	r/h = 0.05	-1.0	-1.5	-1.2	-1.8	-0.4		+0.2	
								-0.2	
	r/h = 0.10	-0.7	-1.2	-0.8	-1.4	-0.3		+0.2	
								-0.2	
	r/h = 0.20	-0.5	-0.8	-0.5	-0.8	-0.3		+0.2	
								-0.2	
Mansard Eaves	α = 30°	-1.0	-1.5	-1.0	-1.5	-0.3		+0.2	
								-0.2	
	α = 45°	-1.2	-1.8	-1.3	-1.9	-0.4		+0.2	
								-0.2	
	α = 60°	-1.3	-1.9	-1.3	-1.9	-0.5		+0.2	
								-0.2	

Notes

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

b For roofs with mansard eaves, linear interpolation between α = 30°, 45° and α = 60° may be used. For α > 60° linear interpolation between the values for α = 60° and the values for flat roofs with sharp eaves may be used.

c In Zone I, where positive and negative values are given, both values should be considered.

d For the mansard eave itself, the external pressure coefficients are given in Table 6.9 depending on the pitch angle of the mansard eave.

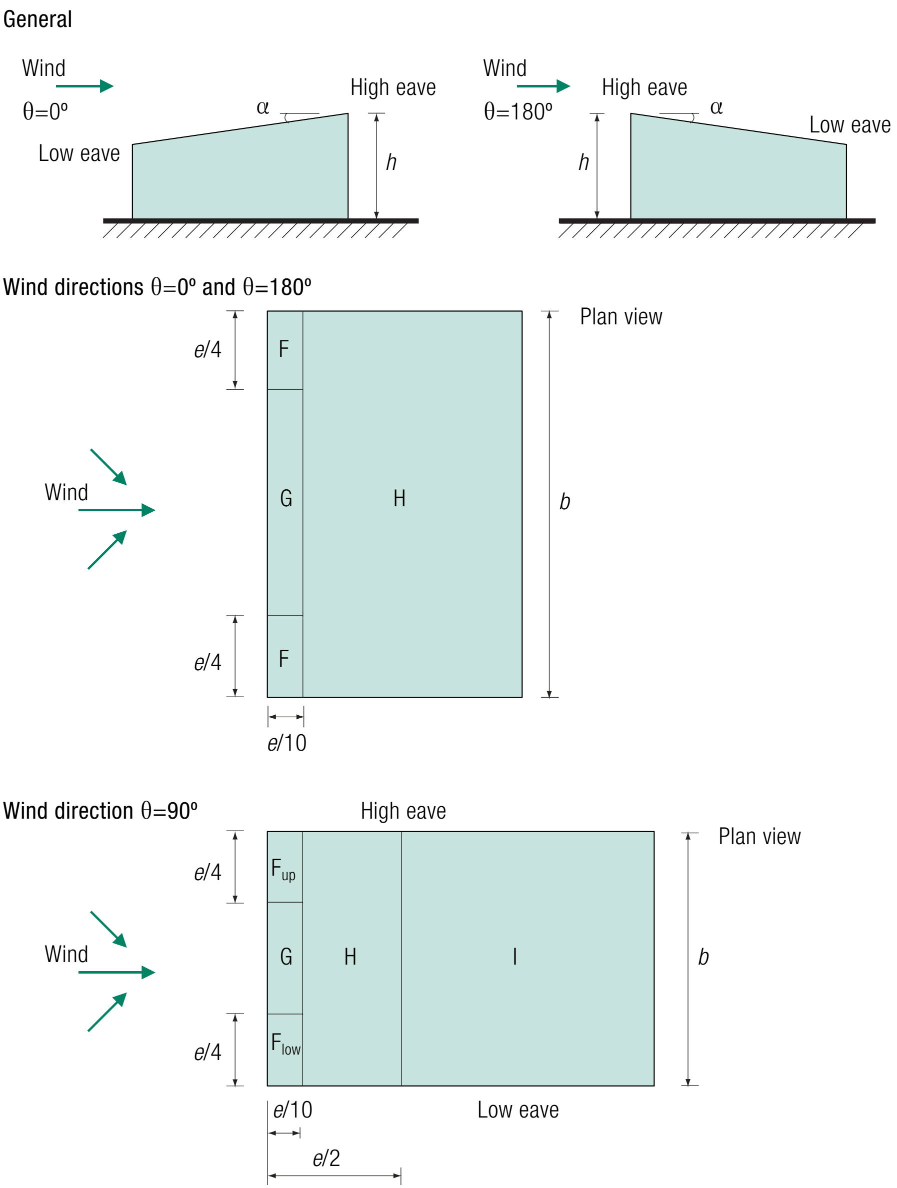
e For the curved eave itself, the external pressure coefficients are given by linear interpolation along the curve, between values on the wall and on the roof.

f For parapet heights h_p/h<0.025 interpolation may be used between these values and the values for a sharp eaves roof (where h_p/h=0.0).

Note It is likely that this table from EC1 Part 1-4 will be amended in the near future. Appendix I gives the expected amended table.

6.7.2.4 Monopitch roofs

The roof, including protruding parts, should be divided into zones as shown in Figure 6.15. The reference height z_e should be taken equal to h. The pressure coefficients for each monopitch roof zone are given in Table 6.8.



$e = b$ or $2h$, whichever is smaller

b: crosswind dimension

Fig 6.15 Key to loaded zones on monopitch roofs

Table 6.8 External pressure coefficients for monopitch roofs

Pitch angle α	Zone for wind direction $\theta = 0°$						Zone for wind direction $\theta = 180°$					
	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 $\theta = 90°$									
	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	

Notes

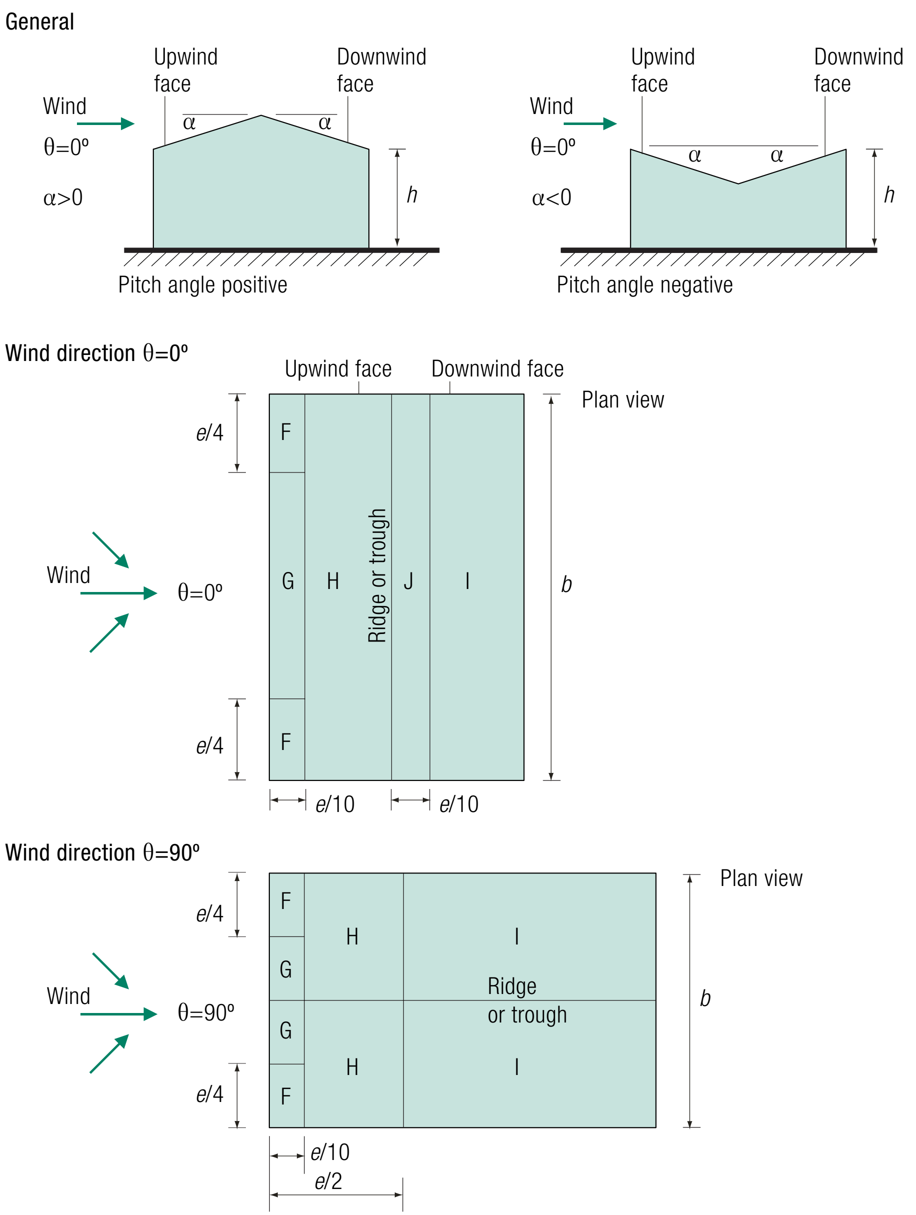
a At $\theta = 0°$ the pressure changes rapidly between positive and negative values around a pitch angle of $\alpha = +5°$ to $+45°$, so both positive and negative values are given. Two cases should be considered: one with all positive values, and one with all negative values. No mixing of positive and negative values is allowed on the same face.

b Linear interpolation for intermediate pitch angles may be used between values of the same sign. The values equal to +0.0 and -0.0 are given for interpolation purposes.

Note It is likely that this table from EC1 Part 1-4 will be amended in the near future. Appendix I gives the expected amended table.

6.7.2.5 Duopitch roofs

The roof, including protruding parts, should be divided in zones as shown in Figure 6.16. The reference height z_e should be taken as h. The pressure coefficients for each duopitch roof zone are given in Table 6.9.



$e = b$ or $2h$, whichever is smaller
b: crosswind dimension

Fig 6.16 Key to loaded areas on duopitch roofs

Table 6.9 External pressure coefficients for duopitch 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	

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	

Notes

a 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. 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.

b 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 6.7.2.3). The values equal to +0.0 and -0.0 are given for interpolation purposes.

Note It is likely that this table from EC1 Part 1-4 will be amended in the near future. Appendix I gives the expected amended table.

6.7.2.6 Hipped roofs

The roof, including protruding parts, should be divided into zones as shown in Figure 6.17. The reference height z_e should be taken as h. The pressure coefficients that should be used for each hipped roof zone are given in Table 6.10.

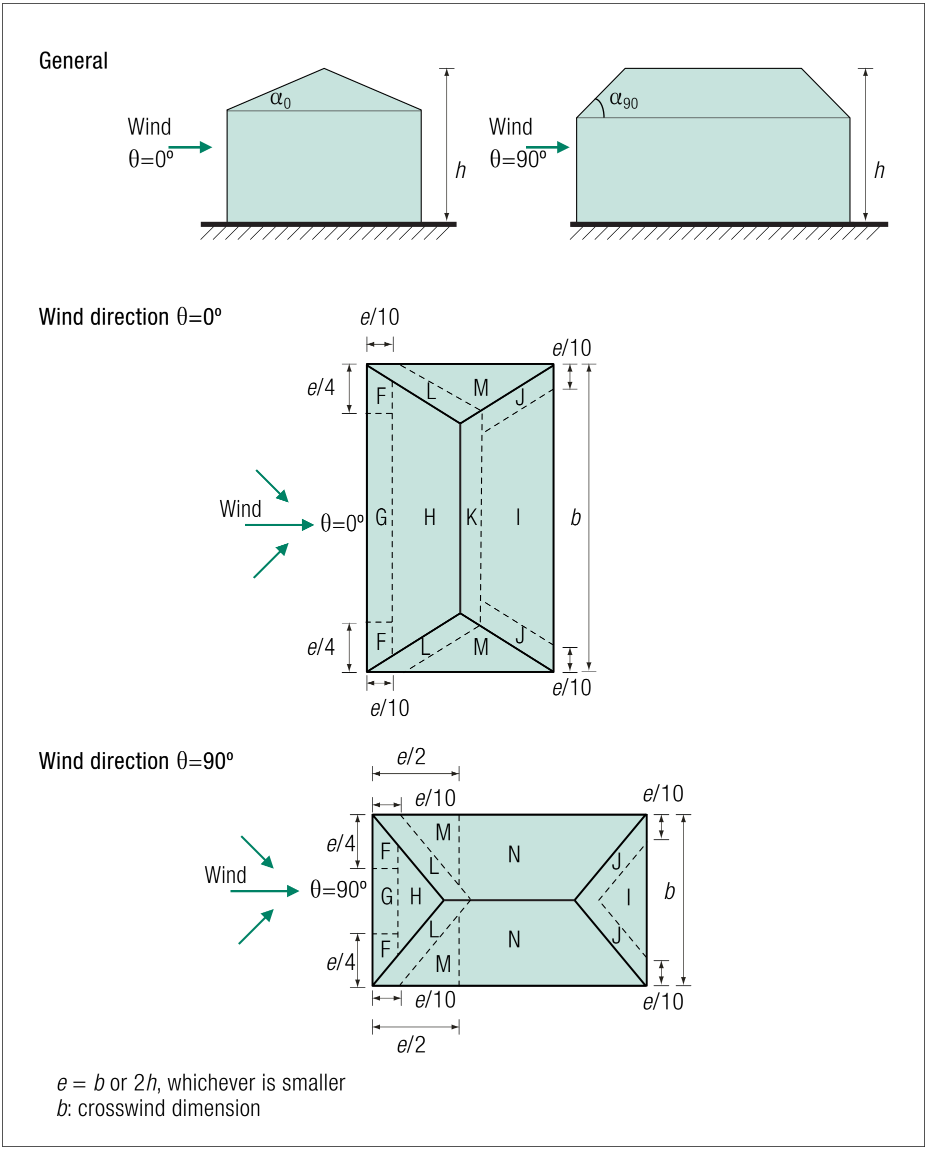


Fig 6.17 Key to loaded zones on hipped roofs

Table 6.10 External pressure coefficients for hipped roofs

Pitch angle	Zone for wind direction $\theta = 0°$ and $\theta = 90°$																	
	F		G		H		I		J		K		L		M		N	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$
5°	-1.7	-2.5	-1.2	-2.0	-0.6	-1.2	-0.3		-0.6		-0.6		-1.2	-2.0	-0.6	-1.2	-0.4	
	+0.0		+0.0		+0.0													
15°	-0.9	-2.0	-0.8	-1.5	-0.3		-0.5		-1.0	-1.5	-1.2	-2.0	-1.4	-2.0	-0.6	-1.2	-0.3	
	+0.2		+0.2		+0.2													
30°	-0.5	-1.5	-0.5	-1.5	-0.2		-0.4		-0.7	-1.2	-0.5		-1.4	-2.0	-0.8	-1.2	-0.2	
	+0.5		+0.7		+0.4													
45°	-0.0		-0.0		-0.0		-0.3		-0.6		-0.3		-1.3	-2.0	-0.8	-1.2	-0.2	
	+0.7		+0.7		+0.6													
60°	+0.7		+0.7		+0.7		-0.3		-0.6		-0.3		-1.2	-2.0	-0.4		-0.2	
75°	+0.8		+0.8		+0.8		-0.3		-0.6		-0.3		-1.2	-2.0	-0.4		-0.2	

Notes

a At $\theta = 0°$ the pressures change rapidly between positive and negative values on the windward face at pitch angle of $\alpha = +5°$ to $+45°$, so both positive and negative values are given. 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.

b 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 and -0.0 are given for interpolation purposes.

c The pitch angle of the windward face will always govern the pressure coefficients even where the roof has different pitch angles on the hip slopes.

Note It is likely that this table from EC1 Part 1-4 will be amended in the near future. Appendix I gives the expected amended table.

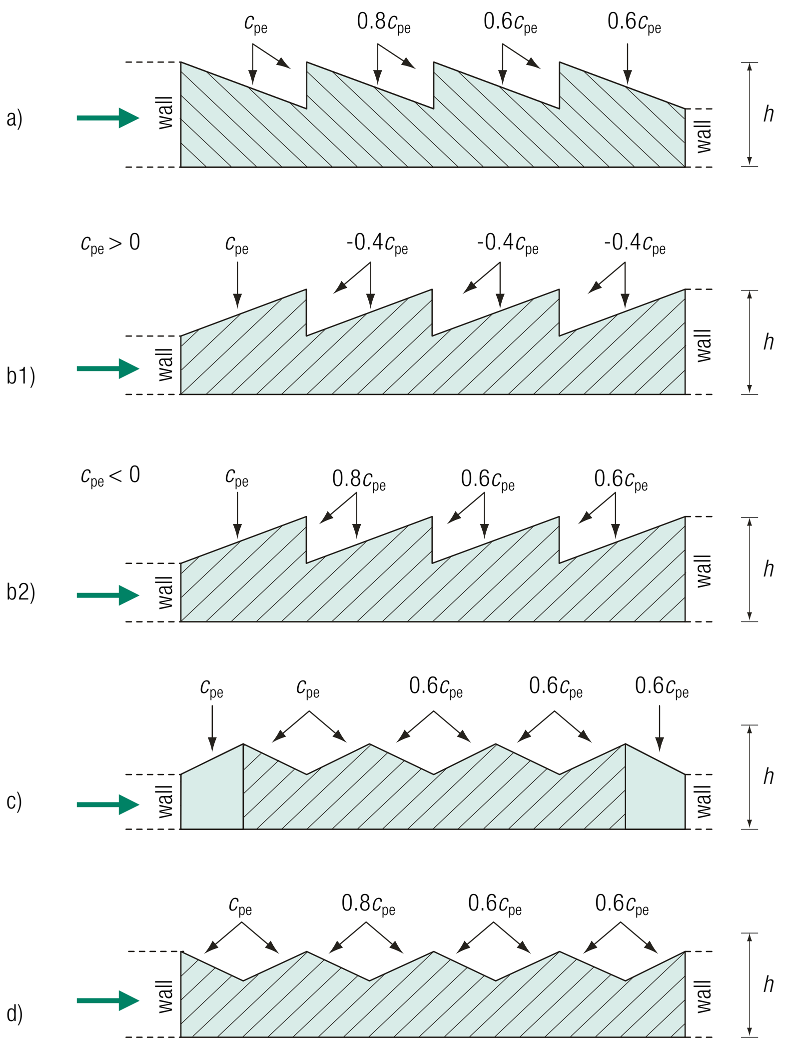
6.7.2.7 Multispan roofs

Pressure coefficients for multispan roofs are derived from the pressure coefficients on individual monopitch roofs (see Section 6.7.2.4) or duopitch roofs (see Section 6.7.2.5), but with reduction factors applied where appropriate to account for the shelter effect of upwind spans. Figure 6.18 gives the reduction factors to apply.

The roof zones F, G and H should be considered only for the upwind faces. The zones J and I should be considered for each span of the multispan roof. The reference height z_e should be taken as h.

6.7.2.8 Internal pressure

Internal and external pressures are considered to act at the same time. The worst combination of external and internal pressures should be considered for every combination of possible openings and other leakage paths.



Notes

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

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.

Fig 6.18 Key to loaded areas in multispan roofs

The internal pressure coefficient, c_{pi}, depends on the size and distribution of the openings in the building envelope which includes open windows, ventilators, chimneys, etc. as well as background permeability such as air leakage around doors, windows, services and through the building envelope. When at least two sides of the building (either the facades or roof) have openings which are >30% of the area of those sides then the internal pressure rules do not apply and the rules for canopy roofs (Section 6.7.3) or free-standing walls (Section 6.7.4) should be used. Table 6.11 gives some general values of background permeability. Where more specific information is available it should be used. Modern construction methods are likely to lead to lower values than those in Table 6.11.

Table 6.11 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	10.5×10^{-4}
Energy efficient housing	4.0×10^{-4}

Where a specific external opening, such as a door or a window, would create a dominant face when open but is considered to be closed in the ultimate limit state, during severe windstorms, the condition with the door or window open should be considered as an accidental design situation in accordance with EC0. A face of a building should be regarded as dominant when the area of openings in that face is at least twice the total area of openings and leakages in the remaining faces of the building considered. This can also be applied to individual internal volumes within the building.

For a building with a dominant face the internal pressure should be taken as a fraction of the external pressure at the openings of the dominant face as follows:

When the area of the openings in the dominant face is twice the total area of the openings in the remaining faces.

$$c_{pi} = 0.75c_{pe}$$

When the area of the openings in the dominant face is $\geqslant$ 3 times the total area of the openings in the remaining faces.

$$c_{pi} = 0.9c_{pe}$$

where c_{pe} is the value for the external pressure coefficient at the openings in the dominant face.

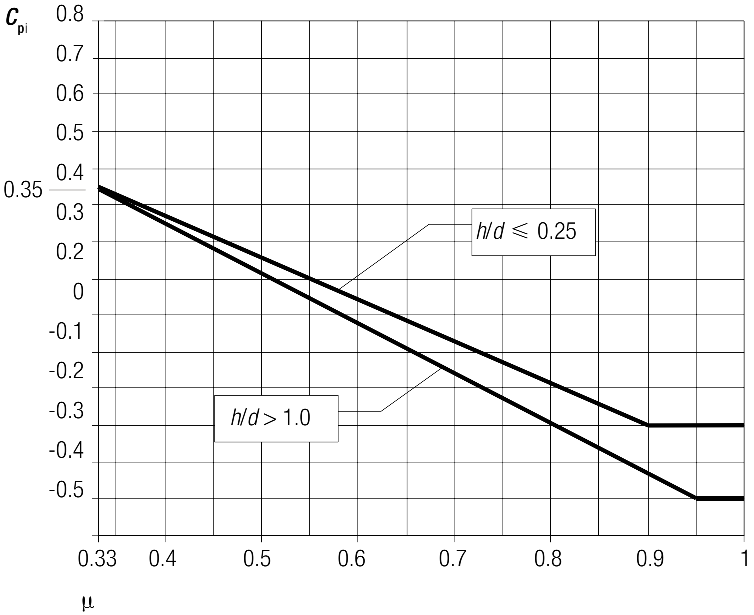
When the area of the openings in the dominant face is between 2 and 3 times the area of the openings in the remaining faces then linear interpolation may be used.

When the dominant openings span zones with different values of external pressures, an area weighted average value of c_{pe} may be used.

For buildings without a dominant face, the internal pressure coefficient c_{pi} should be determined from Figure 6.19, and is a function of the ratio of the height and the depth of the building, h/d, and the opening ratio, μ, which depends on the area of openings in the side faces and all parts of the roof, and is given by:

$$\mu = \frac{\sum \text{area of openings where } c_{pe} \text{ is negative or - 0.0}}{\sum \text{area of all openings}}$$

μ should be determined for each wind direction θ. Values of -0.0 are given in Tables 6.8, 6.9 and 6.10.



Note
For values between $h/d = 0.25$ and $h/d = 1.0$ linear interpolation may be used.

Fig 6.19 Internal pressure coefficients for uniformly distributed openings

Where it is not possible, or not considered justified, to estimate μ for a particular case then c_{pi} may be taken as the more onerous of +0.2 and -0.3.

The reference height z_i for the internal pressures should be taken as the height z_e used for the external pressures on the faces which contribute through their openings to the internal pressure. If there are several openings the largest value of z_e should be used to determine z_i.

6.7.2.9 Pressure on walls or roofs with more than one skin

For multiskin walls and roofs the wind force acting on each skin should be determined. The permeability of a skin is defined as the ratio of the total area of the opening to the total area of the skin. A skin can be considered as impermeable if the value is less than 0.1%.

If only one skin is permeable, then the wind force on the impermeable skin should be determined from the difference between the internal and the external wind pressure.

If more than one skin is permeable then 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.

As a first approximation the following guidance will apply in cases where the extremities of the layer between the skins are air tight (see Figure 6.20(a)) and where the free distance between the skins is less than 100mm.
– For walls and roofs with an impermeable inside skin and a permeable outside skin with approximately uniformly distributed openings, the net pressure coefficient on the outside skin is $c_{p,net}$ = two-thirds c_{pe} for positive external pressures and $c_{p,net}$ = one-third c_{pe} for negative external pressures. The net pressure coefficient on the inside skin is given by $c_{p,net} = c_{pe} - c_{pi}$.
– For walls and roofs with an impermeable inside skin and an impermeable more rigid, outside skin, the net pressure coefficient on the outside skin is $c_{p,net} = c_{pe} - c_{pi}$.
– For walls and roofs with a permeable inside skin with approximately uniformly distributed openings and an impermeable outside skin, the net pressure coefficient on the outside skin is $c_{p,net} = c_{pe} - c_{pi}$, and the net pressure coefficient on the inside skin is $c_{p,net}$ = one-third c_{pi}.
– For walls and roofs with an impermeable outside skin and an impermeable, more rigid inside skin, the net pressure coefficient on the outside skin is $c_{p,net} = c_{pe}$ and the net pressure coefficient on the inside skin is $c_{p,net} = c_{pe} - c_{pi}$.

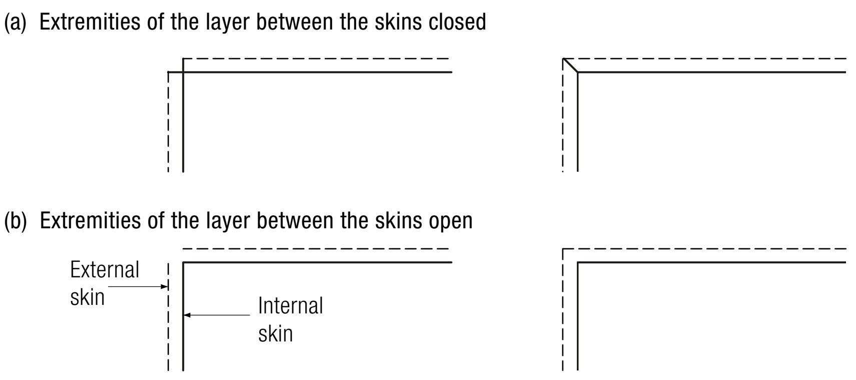


Fig 6.20 Corner details for walls or roofs with more than one skin

If the air layer between the skins is not sealed at its extremities (see Figure 6.20(b), then these rules do not apply and specialist advice should be sought.

These rules do not apply to tiled or slated roofs, which should be designed to BS5534[38] or to masonry cavity walls. EN 1996-1[39] allows the design lateral load on a masonry wall to be apportioned between the leaves of the wall in proportion to their strength (design moment of resistance) or to the stiffness of each leaf. The wall ties provided must be capable of transmitting the loads involved.

6.7.3 Canopy roofs

A canopy roof is the roof of a structure that does not have permanent walls, such as a petrol station, Dutch barn, etc.

The degree of blockage under a canopy roof will affect the air flow and the resulting wind loads. The blockage, φ is defined as the ratio of the area of obstructions under the canopy divided by the cross-sectional area under the canopy, both areas being normal to the wind direction. $\varphi = 0$ represents an empty canopy, and $\varphi = 1$ represents the canopy fully blocked with contents to the down wind eaves, which is the worst case. Figure 6.21 shows examples of different blockages.

The overall force coefficients, c_f, and net pressure coefficients $c_{p,net}$, given in Tables 6.12 and 6.13 for $\varphi = 0$ and $\varphi = 1$ take account of the combined effect of wind acting on both the upper and lower surfaces of the canopies for all wind directions. Intermediate values may be obtained by linear interpolation.

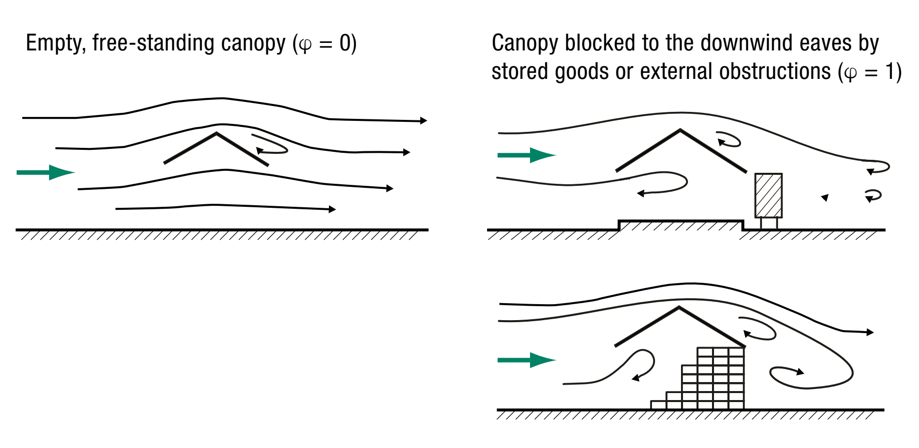


Fig 6.21 Airflow over canopy roofs

Downwind of the position of maximum blockage, $c_{p,net}$ values for $\varphi = 0$ should be used.

The overall force coefficient should be used in the design of the overall structure. The net pressure coefficients should be used in the design of the roofing elements and fixings.

Each canopy must be able to support the following load cases:
– for a monopitch canopy (Table 6.12) the centre of pressure should be taken at $d/4$ from the windward edge (d = along wind dimension, see Figure 6.22)
– for a duopitch canopy (Table 6.13) the centre of pressure should be taken at the centre of each slope (Figure 6.23). In addition, a duopitch canopy should be able to support one pitch with the maximum or minimum load and the other pitch unloaded
– for a multibay duopitch canopy (see Figure 6.24) the load on each bay may be determined by applying the reduction factors ψ_{mc} given in Table 6.14 to the overall force and net pressure coefficients for isolated duo-pitch canopies given in Table 6.13.

For canopies with double skins, the impermeable skin and its fixings should be designed using the $c_{p,net}$ values given in Tables 6.12 and 6.13. The permeable skin and its fixings should be designed using one-third $c_{p,net}$.

Friction forces should be considered.

The reference height z_e should be taken as h as shown in Figures 6.22 and 6.23.

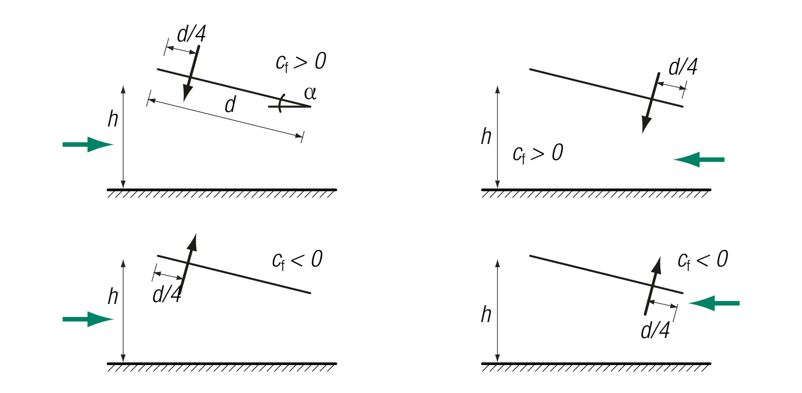


Fig 6.22 Location of the centre of force for monopitch canopies

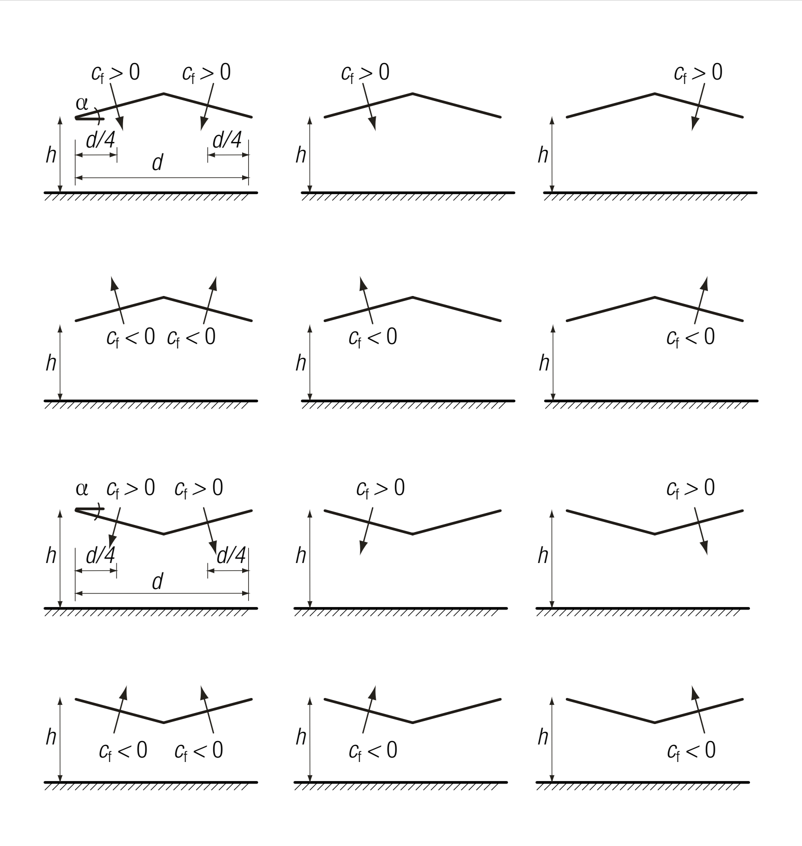
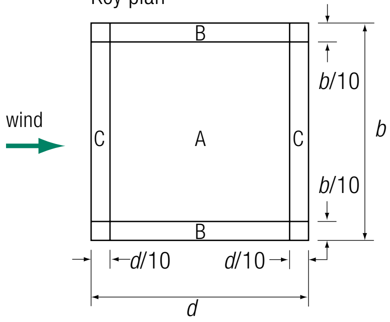


Fig 6.23 Load cases and position of centre of pressure on duopitch canopies

Table 6.12 $c_{p,net}$ and c_f values for monopitch canopies

Net pressure coefficients $c_{p,net}$

Key plan



Roof angle α	Blockage φ	Overall force coefficients c_f	Zone A	Zone B	Zone C
0°	Maximum all φ	+ 0.2	+ 0.5	+ 1.8	+ 1.1
	Minimum φ = 0	-0.5	-0.6	-1.3	-1.4
	Minimum φ = 1	-1.3	-1.5	-1.8	-2.2
5°	Maximum all φ	+ 0.4	+ 0.8	+ 2.1	+ 1.3
	Minimum φ = 0	-0.7	-1.1	-1.7	-1.8
	Minimum φ = 1	-1.4	-1.6	-2.2	-2.5
10°	Maximum all φ	+ 0.5	+ 1.2	+ 2.4	+ 1.6
	Minimum φ = 0	-0.9	-1.5	-2.0	-2.1
	Minimum φ = 1	-1.4	-2.1	-2.6	-2.7
15°	Maximum all φ	+ 0.7	+ 1.4	+ 2.7	+ 1.8
	Minimum φ = 0	-1.1	-1.8	-2.4	-2.5
	Minimum φ = 1	-1.4	-1.6	-2.9	-3.0
20°	Maximum all φ	+ 0.8	+ 1.7	+ 2.9	+ 2.1
	Minimum φ = 0	-1.3	-2.2	-2.8	-2.9
	Minimum φ = 1	-1.4	-1.6	-2.9	-3.0
25°	Maximum all φ	+ 1.0	+ 2.0	+ 3.1	+ 2.3
	Minimum φ = 0	-1.6	-2.6	-3.2	-3.2
	Minimum φ = 1	-1.4	-1.5	-2.5	-2.8
30°	Maximum all φ	+ 1.2	+ 2.2	+ 3.2	+ 2.4
	Minimum φ = 0	-1.8	-3.0	-3.8	-3.6
	Minimum φ = 1	-1.4	-1.5	-2.2	-2.7

Note

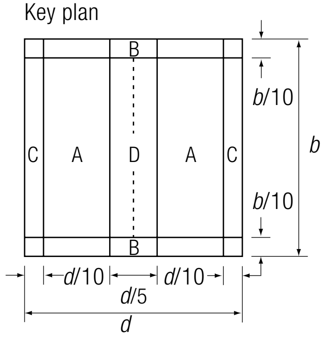
+ values indicate a net downward acting wind action.

– values represent a net upward acting wind action.

Table 6.13 $c_{p,net}$ and c_f values for duopitch canopies

Net pressure coefficients $c_{p,net}$

Key plan



Roof angle α	Blockage φ	Overall force coefficients c_f	Zone A	Zone B	Zone C	Zone D
– 20º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.7 -0.7 -1.3	+0.8 -0.9 -1.5	+1.6 -1.3 -2.4	+0.6 -1.6 -2.4	+1.7 -0.6 -0.6
– 15º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.5 -0.6 -1.4	+0.6 -0.8 -1.6	+1.5 -1.3 -2.7	+0.7 -1.6 -2.6	+1.4 -0.6 -0.6
– 10º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.4 -0.6 -1.4	+0.6 -0.8 -1.6	+1.4 -1.3 -2.7	+0.8 -1.5 -2.6	+1.1 -0.6 -0.6
– 5º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.3 -0.5 -1.3	+0.5 -0.7 -1.5	+1.5 -1.3 -2.4	+0.8 -1.6 -2.4	+0.8 -0.6 -0.6
+ 5º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.3 -0.6 -1.3	+0.6 -0.6 -1.3	+1.8 -1.4 -2.0	+1.3 -1.4 -1.8	+0.4 -1.1 -1.5
+ 10º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.4 -0.7 -1.3	+0.7 -0.7 -1.3	+1.8 -1.5 -2.0	+1.4 -1.4 -1.8	+0.4 -1.4 -1.8
+ 15º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.4 -0.8 -1.3	+0.9 -0.9 -1.3	+1.9 -1.7 -2.2	+1.4 -1.4 -1.6	+0.4 -1.8 -2.1
+ 20º	Maximum all φ Minimum φ = 0 Minimum φ = 1	+0.6 -0.9 -1.3	+1.1 -1.2 -1.4	+1.9 -1.8 -2.2	+1.5 -1.4 -1.6	+0.4 -2.0 -2.1

Table 6.13 continued

Roof angle α	Blockage φ	Overall force coefficients c_f	Zone A	Zone B	Zone C	Zone D
+ 25º	Maximum all φ Minimum $\varphi = 0$ Minimum $\varphi = 1$	+0.7 -1.0 -1.3	+1.2 -1.4 -1.4	+1.9 -1.9 -2.0	+1.6 -1.4 -1.5	+0.5 -2.0 -2.0
+ 30º	Maximum all φ Minimum $\varphi = 0$ Minimum $\varphi = 1$	+ 0.9 -1.0 -1.3	+ 1.3 -1.4 -1.4	+ 1.9 -1.9 -1.8	+ 1.6 -1.4 -1.4	+ 0.7 -2.0 -2.0
Note + values indicate a net downward acting wind action. – values represent a net upward acting wind action.						

Table 6.14 Reduction factors ψ_{mc} for multibay canopies

Bay	Location	ψ_{mc} factors for all φ	
		On maximum (downward) force and pressure coefficients	On minimum (upward) force and pressure coefficients
1 2 3	End bay Second bay Third and subsequent bays	1.0 0.9 0.7	0.8 0.7 0.7

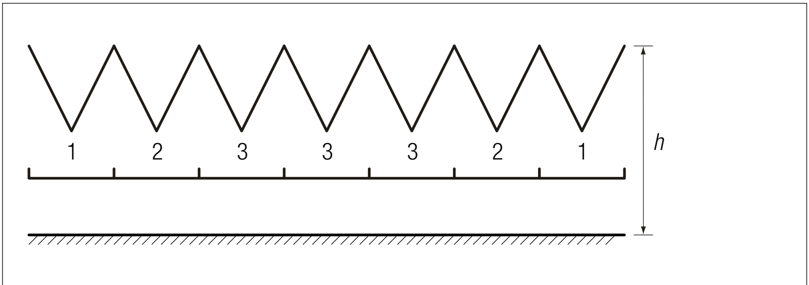


Fig 6.24 Key to bay numbers on multibay canopies

6.7.4 Free-standing walls, parapets and fences

6.7.4.1 General

Net pressure coefficients $c_{p,net}$ for free-standing walls, parapets and fences are given in Table 6.15. Figure 6.25 shows the pressure zones. Net pressure coefficients are given for solid walls ($\varphi = 1.0$) and for walls which have 20% openings ($\varphi = 0.8$). The reference area in both cases is the gross wall area. Linear interpolation may be used for solidity ratios between 0.8 and 1.0. Porous walls and fences with a solidity ratio $\varphi < 0.8$ should be treated as plane lattices in accordance with EC1 Part 1-4, Clause 7.11.

The reference height for free standing walls and fences should be taken as $z_e = h$, see Figure 6.25. The reference height for parapets on buildings should be taken as $z_e = (h + h_p)$, see Figure 6.14.

Table 6.15 Net pressure coefficients $c_{p,net}$ for free-standing walls and parapets

Solidity	Zone		A	B	C	D
$\varphi = 1.0$	Without return corners	$l/h \leqslant 3$	2.3	1.4	1.2	N/A
		$l/h = 5$	2.9	1.8	1.4	1.2
		$l/h \geqslant 10$	3.4	2.1	1.7	1.2
	with return corners of length $\geqslant h$ [a]		2.1	1.8	1.4	1.2
$\varphi = 0.8$			1.2	1.2	1.2	1.2

Note

a Linear interpolation may be used for return corner lengths between 0.0 and *h*.

6.7.4.2 Shelter factors for walls and fences

If there are other walls or fences upwind that are equal in height or taller than the wall or fence under consideration, then advantage may be taken of shelter. The value of the shelter factor ψ_s depends on the spacing between the walls or fences x, and the solidity φ, of the upwind (sheltering) wall or fence. Values of ψ_s are given in Figure 6.26.

The resulting net pressure coefficient on the sheltered wall, $c_{p,net,s}$, is given by

$$c_{p,net,s} = \psi_s \, c_{p,net}$$

The shelter factor should not be applied in the end zones within a distance of h measured from the free end of the wall. In addition, no advantage from shelter should be taken on parts of the downwind wall which extend beyond the projected ends of the upwind wall.

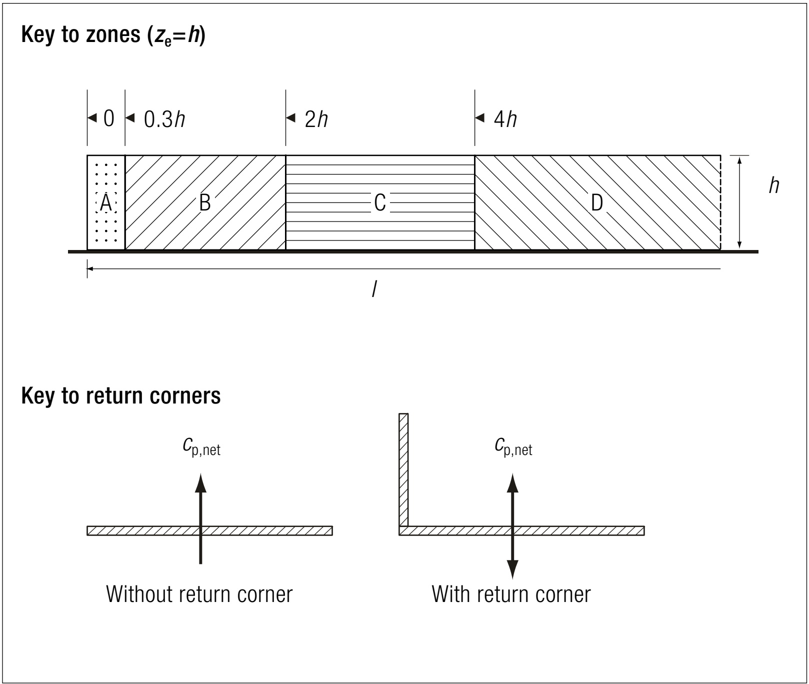


Fig 6.25 Key to loaded zones of free-standing walls, fences and parapets

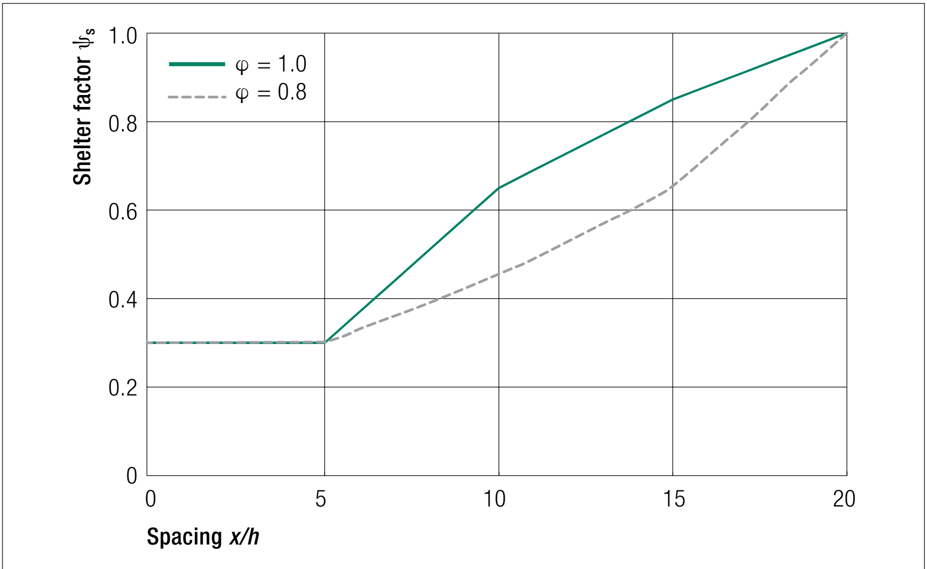


Fig 6.26 Shelter factor ψ_s for walls and fences for φ – values of 0.8 and 1.0

6.7.5 Friction coefficients

Friction forces can arise when the wind blows parallel to external surfaces such as walls or roofs. Friction coefficients c_{fr}, for walls and roof surfaces are given in Table 6.16.

The reference area A_{fr} is given in Figure 6.27. For enclosed (i.e. fully clad) buildings, friction forces should only be applied to those parts of the roof or wall surfaces parallel to the wind which extend beyond a distance of $2b$ or $4h$, whichever is the smallest, from the upwind eaves or corner.

The reference height z_e should be taken as the building height h, see Figure 6.27.

Table 6.16 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

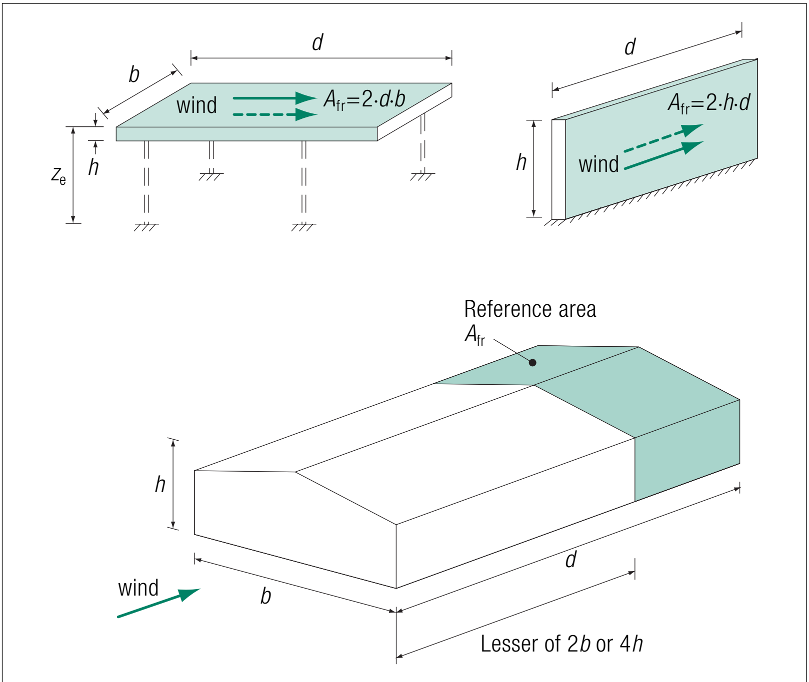


Fig 6.27 Reference area for friction

Chapter 7: Thermal actions

7.1 General

7.1.1 Scope

This Chapter gives design guidance contained in EN1991-1-5[7] (EC1 Part 1-5) for calculating thermal actions on buildings.

This Chapter also describes the changes in the temperature of structural elements. Characteristic values of thermal actions are presented for use in the design of structures which are exposed to daily and seasonal climatic changes.

7.1.2 Introductory advice for using this Chapter for the design of buildings

There are buildings which can be characterised as sensitive to the effects of thermal actions which need to be designed to resist their effects. The designer should use experience together with the guidance contained in this *Manual* to determine whether a building is sensitive to the effects of thermal actions.

The five criteria which are important and need to be considered by the designer are:
– material
– geometry
– restraint
– movement joints
– temperature range and frequency.

These criteria are described below:

Materials (coefficient of linear expansion) – Coefficients of linear expansion to determine temperature induced strains are given in Table 7.1 of this *Manual* for a selection of common building materials.

Geometry – Buildings which have abrupt changes in geometry such as wings, re-entrant corners and courtyards will be prone to differential thermal movement at junctions of architectural massing. This thermal movement may be accompanied by movement from other actions such as the differential settlement of foundations.

Restraints – Most buildings rely on bracing or framing systems to provide overall structural stability. These stability systems have the potential to create points of restraint against temperature movement which in turn lead to temperature induced stresses. The combination of changes in plan geometry and the proximity of such systems can cause particular problems.

Buildings with stability systems located at the ends of the building may experience significant temperature induced stresses in both the stability system and connected elements of structure. It will be preferable to locate stability systems in the middle of the external face of the building or distributed evenly along it provided that each stability system only works in one direction.

Multi-storey buildings may exhibit significant differential temperature movements between the frame and its cladding. Cladding support systems should be designed to accommodate differential movement between the structural frame and the cladding whilst still catering for normal vertical and horizontal loads from the cladding.

Consideration should be given to the construction conditions which may expose elements of structure to temporary temperature effects.

Movement joints – Thermal effects are ameliorated either by provision of movement joints or allowing for effects of restraint in the design. EC1 Part 1-5 does not provide guidance on movement joints and the reader is referred to relevant guidance elsewhere.

Temperature range and frequency – Methods for determining an appropriate temperature range are given in EC1 Part 1-5. Frequency should be determined for the project under consideration and the following information may be helpful:

- Most occupied buildings are either heated and/or cooled (e.g. air conditioned) and therefore tend to have a daily temperature cycle which, in the majority of cases, will not be significant.
- Unheated buildings and car parks may have large daily and seasonal temperature cycles. Buildings like these should be designed for thermal action.
- Temperatures can be increased by solar gain so roofs and top decks of car parks should be checked. Car parks with black surfacing or thin surfacing of any colour are particularly susceptible to solar gain.
- Temperature effects in cold stores and buildings with other extreme internal temperatures should be considered.

Extreme temperature changes due to equipment failures should be taken into account in the design.

At the construction stage normal thermal action may need to be added to other strain related effects such as:
– concrete curing temperatures (for large or thick pours)
– shrinkage.

Due account needs to be taken of the actual properties of concrete during construction.

7.1.3 Terms and definitions

A basic list of definitions is given in Chapter 2: Basis of Structural Design. Specific additional definitions for thermal actions in buildings are:

Thermal actions – Thermal actions on a structure or a structural element are those actions that arise from the changes of temperature fields within a specified time interval.

Note Temperature fields means temperatures and distribution of temperature affecting the structure.

Shade air temperature – The shade air temperature is the temperature measured by thermometers placed in a white painted louvred wooden box known as a “Stevenson screen”.

Maximum shade air temperature T_{max} – Value of maximum shade air temperature with an annual probability of being exceeded of 0.02 (equivalent to a mean return period of 50 years), based on the maximum hourly values recorded.

Minimum shade air temperature T_{min} – Value of minimum shade air temperature with a 0.02 annual probability of falling below this value (equivalent to a mean return period of 50 years), based on the minimum hourly values recorded.

Initial temperature T_0 – The temperature of a structural element at the relevant stage of its restraint.

Cladding – The part of the building which provides a weatherproof membrane. Generally cladding will only carry self-weight and/or wind actions.

Uniform temperature component ΔT_u – The temperature, constant over the cross-section, which governs the expansion or contraction of an element or structure.

Temperature difference component ΔT_E, ΔT_{My}, ΔT_{Mz} – The part of a temperature profile in a structural element representing the temperature difference between the outer face (i.e. surface) of the element and any point within it.

Note See also Section 7.5.2(b)

Average temperature of a structural element T – Average temperature of a structural element due to climatic temperatures in the winter or summer season and also due to operational temperatures.

7.2 Classification of actions

7.2.1. General

Classification of actions considers the variation of actions in time and space.

7.2.2 Thermal actions

For the scope of structures covered by this *Manual*, thermal actions are generally classified as variable and indirect actions, as defined in Chapter 2.

All values of thermal actions given in this Chapter are characteristic values unless stated otherwise.

7.3 Design situations

Thermal actions are to be determined for each identified design situation in accordance with Section 2.7.1.

The elements of loadbearing structures need to be checked to ensure that thermal effects will not cause overstressing of the structure. Overstressing may be ameliorated either by:
– the provision of movement joints or
– allowing for the effects in the design.

7.4 Representation of actions

Daily and seasonal changes in shade air temperature, solar radiation, re-radiation, etc., give rise to variations of the temperature distribution within individual elements of a structure.

To determine the effects on a building due to thermal actions the following have to be considered:
– local climatic conditions
– the orientation of the structure
– finishes (e.g. cladding in buildings)
– heating and ventilation regimes and thermal insulation, and
– the structural form and the nature of connections and movement joints.

The temperature distribution within an individual structural element is made up of the constituent components shown in Figure 7.1 (which are described below). The designer should consider which of these components are appropriate. See also Section 7.5.2 (a) to (c).

(a) A uniform temperature component, ΔT_u (see Figure 7.1(a)).
(b) A linearly varying temperature difference component about the *z-z* axis, ΔT_{My}(see Figure 7.1(b)).
(c) A linearly varying temperature difference component about the *y-y* axis, ΔT_{Mz} (see Figure 7.1(c)).
(d) A non-linear temperature difference component, ΔT_E. This results in a system of self-equilibrated stresses which produce no net load effect on the element (see Figure 7.1(d)). Figure 7.1(d) is not normally applicable to the design of buildings.

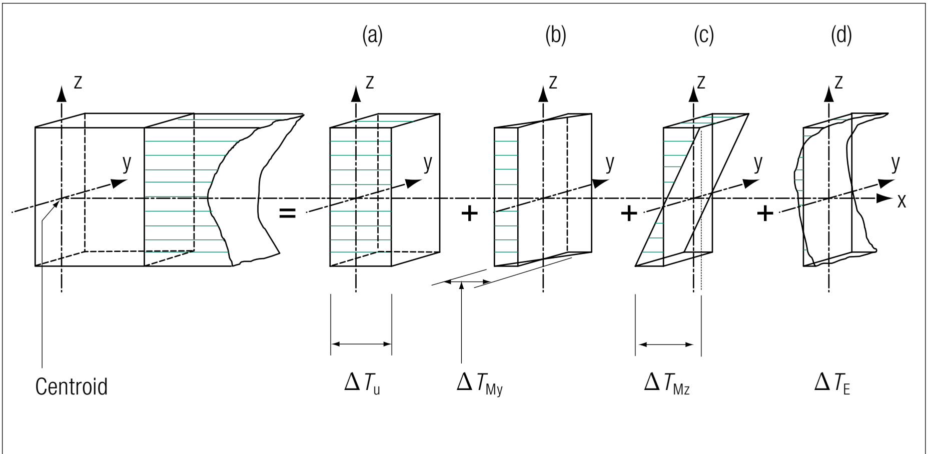


Fig 7.1 Diagrammatic representation of constituent components of a temperature profile

For deriving the effects due to thermal actions Table 7.1 gives values for the coefficient of linear expansion for a selection of commonly used materials.

Table 7.1 Coefficients of linear expansion

Material	α_T ($\times 10^{-6}$/°C)
Aluminium, aluminium alloy	24
Stainless steel	16
Structural steel, wrought or cast iron	12 [a]
Concrete, normal density	12 [b]
Concrete, lightweight aggregate	7
Masonry with concrete units	10 [c, d]
Masonry with clay units	6 [c, d]
Glass	([e])
Timber, along grain	5 [d]
Timber, across grain	30-70 [d, f]

Notes

a For composite structures the coefficient of linear expansion of the steel component may be taken as equal to 10×10^{-6}/°C to neglect restraining effects from different α_T values.

b α_T for concrete varies with age, moisture content and aggregate types. EC2 Part 1-1 states that a value of 10×10^{-6} may be used unless there is better information available. When limestone aggregates are used in concrete, α_T may be as low as 9×10^{-6}/°C. However, for concrete with flint and quartzite aggregate, α_T may be taken as 12×10^{-6}/°C.

c Values for masonry will vary depending on the type.

d For some materials other parameters (e.g. moisture content) also need to be considered. See EC5 Part 1-1[40] and EC6 Part 1-1[39].

e For glass see references 41 to 44.

f Values for timber across the grain can vary considerably according to the type of timber.

7.5 Temperature changes in buildings

7.5.1 General

Thermal actions on buildings due to:
– climatic temperature changes, and
– operational temperature changes inside the building
need to be considered in the design of buildings where there is a possibility of the ultimate or serviceability limit states being exceeded due to thermal movement and/or stresses.

Thermal movement and/or stresses due to temperature changes may also be influenced by:

– shading or changes in shading provided by adjacent buildings
– use of different materials with different thermal expansion coefficients and heat transfer properties
– use of different shapes of cross-section with different uniform temperature.

7.5.2 Determination of temperatures

The determination of thermal actions on buildings due to climatic and operational temperature changes is explained in this Section. In addition, regional data and experience should be taken into account.

Climatic effects need to be determined by considering the variation of shade air temperature and solar radiation.

Operational effects, as a result of heating, cooling, technological or industrial processes etc, also need to be considered for the particular project if appropriate.

In accordance with the temperature components given in Section 7.4, climatic and operational thermal actions on a structural element should be determined using the following basic quantities:

(a) A uniform temperature component ΔT_u given by the difference between the average temperature T (see also Section 7.5.3) of an element and its initial temperature T_0 (see Figure 7.1(a)).
ΔT_u is defined as:

$$\Delta T_u = T - T_0 \qquad (7.1)$$

(b) A linearly varying temperature component given by the difference ΔT_M between the temperatures on the opposite surfaces of a cross-section, or on the surfaces of individual layers (see Figures 7.1(b) and 7.1(c)).
(c) A temperature difference ΔT_p between different parts of a structure given by the difference of average temperatures of these parts.

The values of ΔT_M and ΔT_p should be determined for the particular project as appropriate.

In addition, local effects of thermal actions should be considered where relevant (e.g. at supports or fixings of structural and cladding elements). Adequate representation of thermal actions should be defined taking into account the location of the building and structural detailing.

7.5.3 Determination of temperature profiles

The temperature T in Expression 7.1 should be determined as the average temperature of a structural element in winter or summer, using the appropriate temperature profile determined from this Section. When elements of one layer are considered and when the environmental conditions on both sides are similar, T may be approximately determined as the average of inner and outer environment temperature T_{in} and T_{out}.

In the case of a sandwich element T is the average temperature of a particular layer. Specialist guidance is given in Annex D of EC1 Part 1-6.

a) Determination of the temperature of the inner environment, T_{in}

The temperature of the inner environment, T_{in}, should be determined in accordance with Table 7.2 and Figure 7.2.

Table 7.2 Indicative temperatures of inner environment T_{in}

Season	Temperature T_{in}	Temperature T_{in} for the UK
Summer	$T1$	**20°C**
Winter	$T2$	**25°C**

With reference to Table 7.2, the values for T_1 and T_2 for the UK are in accordance with the National Annex to EC1 Part 1-6, and should be used where no other data are available.

Note The designer may agree the values for $T1$ and $T2$ with the client and the competent authority.

The CIBSE Guide A[45] sets internal building environment comfort criteria for some typical building uses in its Table 1.5 as follows:

– Education buildings	21-23ºC
– Dwellings	23-25ºC
– Hospitals	21-25ºC
– Hotels	21-25ºC
– Libraries	21-25ºC
– Museums, Art galleries	21-23ºC
– Offices	22-24ºC
– Retail	21-25ºC
– Sports halls	14-16ºC
– Swimming pools	23-26ºC

Further and more detailed information may be obtained in CIBSE Guide A[45].

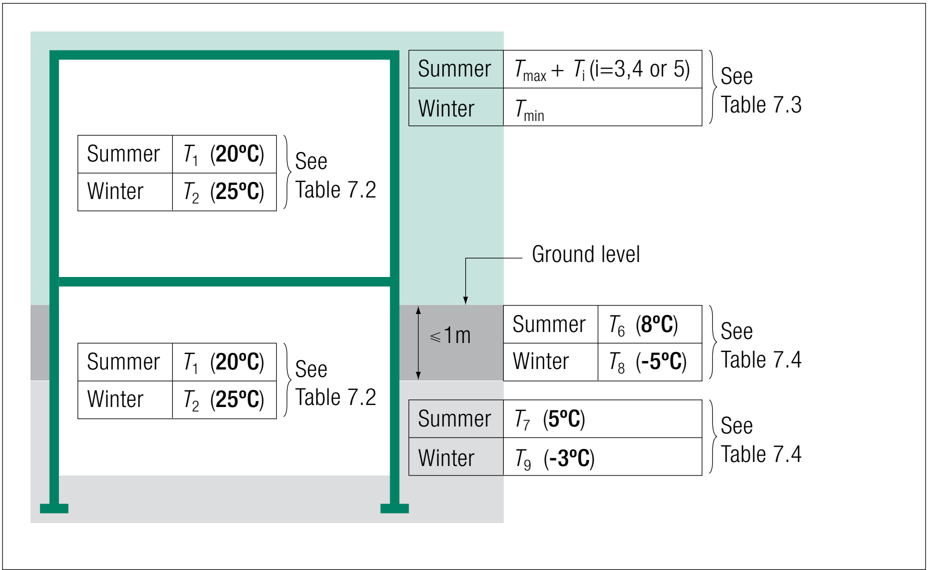


Fig 7.2 Diagrammatic representations of internal and external temperatures (with UK values in brackets)

b) Determination of the temperature of the outer environment, T_{out}

The temperature of the outer environment, T_{out}, should be determined in accordance with:
- Table 7.3 for parts located above ground level
- Table 7.4 for parts located below ground level.

The temperatures T_{out} for the summer season as given in Table 7.3 (for the parts of buildings above ground level) and shown diagrammatically in Figure 7.2 are dependent on the surface absorptivity and the orientation of the building:
- the maximum temperature T_{out} is usually reached in the UK for surfaces facing south-west or for horizontal surfaces
- the minimum T_{min} is usually reached in the UK for surfaces facing north-east.

With reference to Table 7.3 the values of the
- **maximum shade air temperature T_{max}, are given in Figure 7.3** and
- **minimum shade air temperature T_{min}, are given in Figure 7.4**.

Table 7.3 Indicative temperatures T_{out} for parts of buildings above ground level

Season	Significant factor		Temperature T_{out} in ºC
Summer	Relative absorptivity depending on surface colour	0.5 bright light surface (e.g. Steel, light coloured bricks, plaster, whitewash, light coloured paints)	$T_{max} + T_3$
		0.7 reflective light coloured surface (e.g. yellow or buff bricks, red bricks, stone, concrete, dark paints)	$T_{max} + T_4$
		0.9 dark surface (e.g. black, non-metallic surfaces)	$T_{max} + T_5$
Winter			T_{min}
Note Absoptivity values depend on colour and finish.			

Solar radiation effects T_3, T_4, and T_5 in the UK are given by the following values:

– **For north-east facing elements *T*3 = 0°C, *T*4 = 2°C, and *T*5 = 4°C.** For other orientations of a building see Figure 7.5.

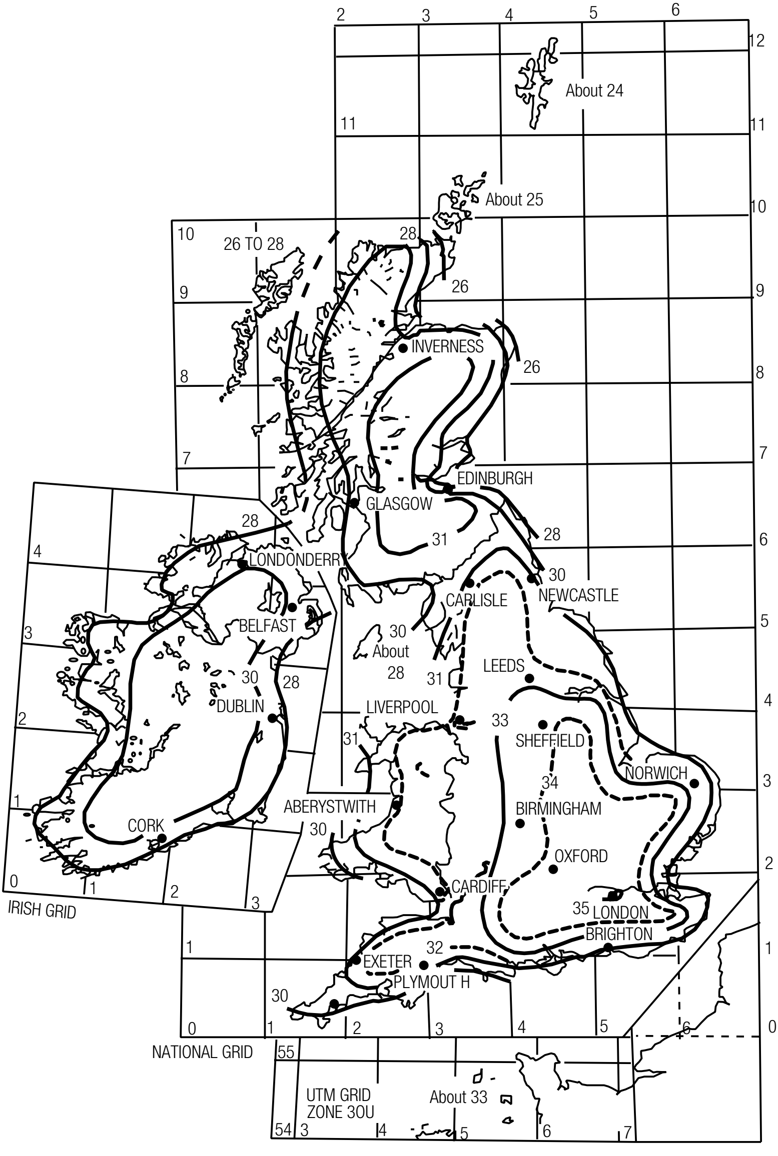
– **For south, south-west or horizontal facing elements *T*3 = 18°C, *T*4 = 30°C, and *T*5 = 42°C**. For other orientations of a building see Figure 7.5.

The temperatures T_{out} for the summer and winter seasons for the parts of buildings below ground level are given in Table 7.4 and shown diagrammatically in Figure 7.2.

Table 7.4 Indicative temperatures T_{out} for underground parts of buildings

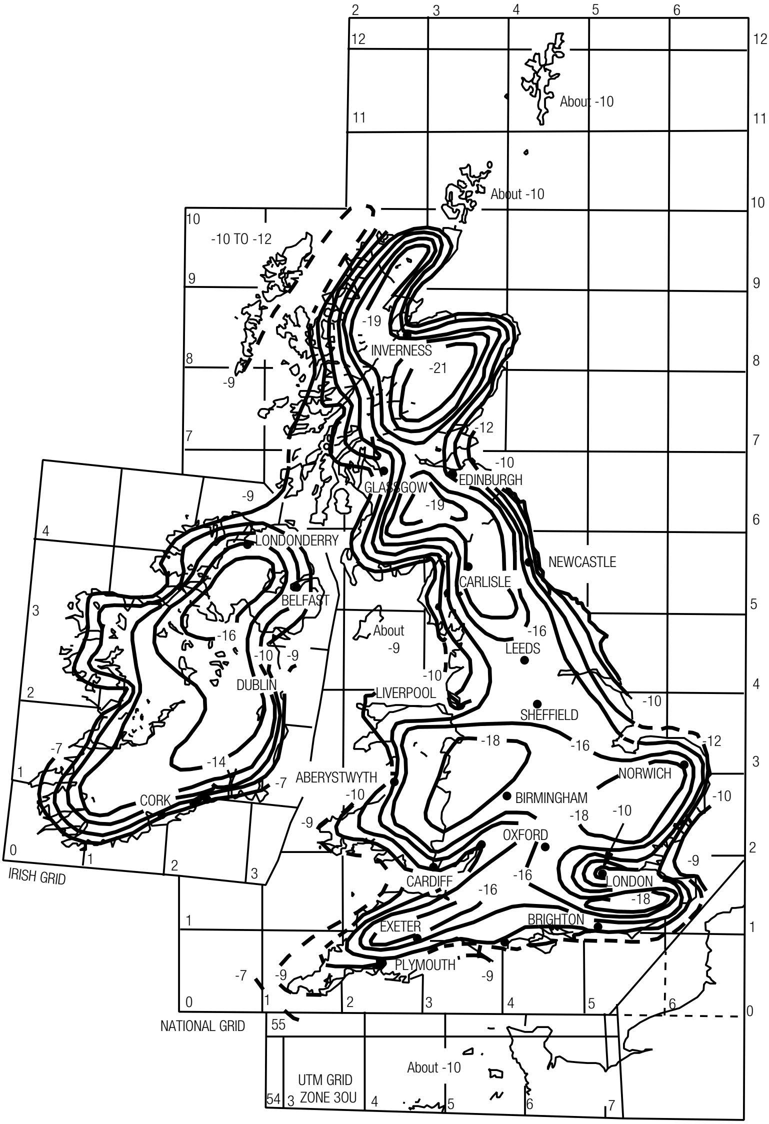
Season	Depth below the ground level	Temperature T_{out}	Temperature T_{out} for the UK
Summer	Less than 1m	T_6	**8°C**
	More than 1m	T_7	**5°C**
Winter	Less than 1m	T_8	**-5°C**
	More than 1m	T_9	**-3°C**

With reference to Table 7.4 the values for T_6, T_7, T_8 and T_9 for the UK are in accordance with the National Annex to EC1 Part 1-6, and should be used where no other data are available.



Note The isotherms are derived from Meteorological Office Data. Although the above map includes the Republic of Ireland, the *Manual* does not necessarily cover the design of buildings to be constructed there.

Fig 7.3 Isotherms of maximum shade air temperature (°C)



Note The isotherms are derived from Meteorological Office Data. Although the above map includes the Republic of Ireland, the *Manual* does not necessarily cover the design of buildings to be constructed there.

Fig 7.4 Isotherms of minimum shade air temperature (°C)

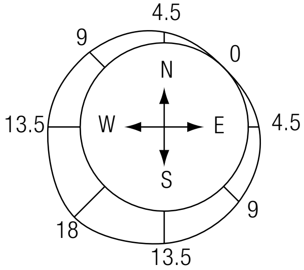
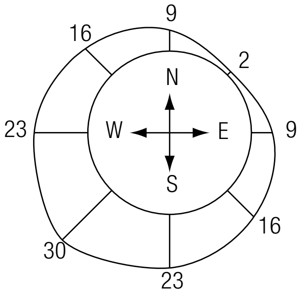
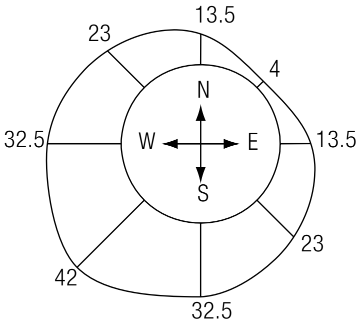
Building surface	Solar radiation effect	Building temperatures
Bright light surface	T_3	
Reflective light coloured surface	T_4	
Dark surface	T_5	

Fig 7.5 Solar radiation effects T_3, T_4, and T_5 for different building orientations

Chapter 8: Actions during execution

8.1 General

8.1.1 Scope

This Chapter gives design guidance contained in EN1991-1-6[8] (EC1 Part 1-6) for the determination of actions which may occur during the execution of buildings. The scope of this Chapter includes structural alterations such as refurbishment and/or partial or full demolition.

This Chapter describes rules for the determination of the actions that need to be considered during execution of buildings, including the following aspects (using Eurocode terminology):
- actions on structural and non-structural members during handling
- geotechnical actions
- actions due to prestressing effects
- predeformations (e.g. precamber or pre-set)
- temperature, shrinkage, hydration effects
- wind actions
- snow loads
- actions caused by water
- construction loads
- accidental actions.

Actions due to atmospheric icing, seismic actions and effects due to vandalism are outside the scope of this Chapter.

Guidance is given on the following:
- Classification of actions
- Design situations and limit states
- Representation of actions for both general actions covered by the other Chapters of this *Manual* and construction loads.

This Chapter also gives rules for the determination of actions which may be used for the design of auxiliary construction works (defined in Section 8.1.3), needed for the execution of buildings.

Normally design rules for auxiliary construction works are defined for the individual project. Guidance may be found in the relevant European standards. For example, the recommended design rules for formwork and falsework in BS EN 12810[46], BS EN 12811[47], BS EN 12812[48] or TG20:(05 [1])[49] may be used and defined for the individual project.

8.1.2 Introductory advice for using this Chapter for the design of buildings

The guidance given in Chapter 8 is based on EC1 Part 1-6 which gives guidance for the determination of actions for the design or verification of structures during their execution stages. It gives rules also for the determination of actions to be used for the design of auxiliary construction works, as well as for the design of structures in transient design situations, as defined in Chapter 2, such as refurbishment, reconstruction and partial or total demolition. Auxiliary construction works include those which are sometimes known as temporary structures. The scope of EC1 Part 1-6 is wide and it is the first such code in Europe and will benefit many, including clients, designers and contractors of varying types. Amongst all the many defined actions, particular attention is paid to the introduction of a number of different types of construction loads, which may typically be present during execution stages but which are unlikely to be present after completion of the structure. Execution involves many parties, including building owners, regulatory bodies, designers and contractors. The close cooperation of these parties is essential to the success of a project.

8.1.3 Terms and definitions

A basic list of definitions is given in Chapter 2: Basis of Structural Design. Specific additional definitions for actions during execution are:

Auxiliary construction works – Any works associated with the construction processes that are not required after use when the related execution activities are completed and they can be removed (e.g. falsework, scaffolding, propping systems, cofferdam, bracing, launching nose).

Note Commonly referred to as temporary works. Completed structures for temporary use are not regarded as auxiliary construction works (e.g. demountable offices).

Construction load – Load that can be present due to execution activities, but is not present when the execution activities are completed.

8.2 Classification of actions

8.2.1 General

Classification of actions considers the variation of actions in time and space.

8.2.2 Actions during execution

Actions during execution are grouped under two headings: construction loads (see Section 8.2.4) and actions other than construction loads (see Section 8.2.3).

8.2.3 Actions (other than construction loads) during execution stages

Table 8.1 gives the classifications of the actions that need to be considered (other than construction loads).

8.2.4 Construction loads

Construction loads (see also Section 8.4.10) are classified as variable actions (Q_c). Table 8.2 gives the full description and classification of construction loads.

Construction loads, which are caused by cranes, equipment, auxiliary construction works/structures may be classified as fixed or free actions depending on the possible position(s) for use.

Where construction loads are classified as fixed, then tolerances for possible deviations from the theoretical position need to be defined. Normally the deviations may be defined for the individual project.

Where construction loads are classified as free, then the limits of the area where they may be moved or positioned need to be determined. The limits need to be defined for the individual project.

In accordance with EC0 clause 1.3(2), control measures may have to be adopted to verify the conformity of the position and moving of construction loads with the design assumptions.

Table 8.1 Classification of actions (other than construction loads) during execution stages

Related Section in this *Manual*	Action	Classification				Remarks	Source
		Variation in time	Classification / Origin	Spatial variation	Nature (static / dynamic)		
8.4.2	Self-weight	Permanent	Direct	Fixed with tolerance / free	Static	Free during transportation / storage. Dynamic if dropped	EC1 Part 1-1
8.4.3	Soil movement	Permanent	Indirect	Free	Static		EC7
8.4.3	Earth pressure	Permanent / variable	Direct	Free	Static		EC7
8.4.4	Prestressing	Permanent / variable	Direct	Fixed	Static	Variable for local design (anchorage)	EC0, EC2 to EC9
8.4.5	Pre-deformations	Permanent / variable	Indirect	Free	Static		EC0
8.4.6	Temperature	Variable	Indirect	Free	Static		EC1 Part 1-5
8.4.6	Shrinkage / hydration effects	Permanent / variable	Indirect	Free	Static		EC2, EC3, EC4
8.4.7	Wind actions	Variable / accidental	Direct	Fixed / free	Static / dynamic	[a]	EC1 Part 1-4
8.4.8	Snow loads	Variable / accidental	Direct	Fixed / free	Static / dynamic	[a]	EC1 Part 1-3
8.4.9	Actions due to water	Permanent / variable / accidental	Direct	Fixed / free	Static / dynamic	Permanent / variable according to project specifications. Dynamic for water currents if relevant	EC0
8.4.11	Accidental	Accidental	Direct / indirect	Free	Static / dynamic	[a]	EC0, EC1 Part 1-7

Note

a The source documents need to be examined with the National Annexes in which additional relevant information may be provided.

Table 8.2 Classification of construction loads

Related Section in this *Manual*	Action (short description)	Classification				Remarks	Source
		Variation in time	Classification / Origin	Spatial variation	Nature (static / dynamic)		
8.4.10	Personnel and handtools	Variable	Direct	Free	Static		
8.4.10	Storage movable items	Variable	Direct	Free	Static / dynamic	Dynamic in case of dropped loads	EC1 Part 1-1
8.4.10	Non-permanent equipment	Variable	Direct	Fixed / free	Static / dynamic		EC1 Part 3[50]
8.4.10	Movable heavy machinery and equipment	Variable	Direct	Free	Static / dynamic		EC1 Part 2[36], EC1 Part 3[50]
8.4.10	Accumulation of waste materials	Variable	Direct	Free	Static / dynamic	Can also impose loads on, for example, vertical surfaces	EC1 Part 1-1
8.4.10	Loads from parts of structure in temporary states	Variable	Direct	Free	Static	Dynamic effects are excluded	EC1 Part 1-1

8.3 Design situations and limit states

8.3.1 General – identification of design situations

Transient and accidental design situations, in accordance with Chapter 2 need to be taken into account as appropriate in designs for execution.

For wind actions during storm conditions the BCSA Publication No 39/05 *Guide to steel erection in windy conditions*[51] can be used.

The design situations need to:
– be selected as appropriate for the structure as a whole, the structural members, the partially completed structure, and also for auxiliary construction works and equipment
– take into account the conditions that apply from stage to stage during execution in accordance with Section 2.7.1
– be in accordance with the execution processes anticipated in the design. Design situations should take account of any revisions to the execution processes.

A selected transient design situation is associated with a nominal duration. The chosen nominal duration needs to be equal to or greater than the anticipated duration of the stage of execution under consideration. The design situations should take into account the likelihood for any corresponding return periods of variable actions (e.g. climatic actions).

Return periods for the determination of the characteristic values of variable actions during execution

The return periods for the determination of characteristic values of variable actions during execution are defined for the individual project, but using the values given in Table 8.3 as a minimum. For return periods of $\leqslant$ 3 days and $\leqslant$ 3 months (but > 3 days) the advice given in notes [a] and [b] of Table 8.3 may be followed.

Where it is intended that an execution stage will prescribe limiting climatic conditions or a weather window, the characteristic climatic actions need to be determined taking into account:
– the anticipated duration of the execution stage
– the reliability of meteorological predictions
– time to organise protection measures.

Table 8.3 Recommended return periods for the determination of the characteristic values of climatic actions

Duration	Return period (years)
$\leqslant$ 3 days	2 [a]
$\leqslant$ 3 months (but > 3 days)	5 [b]
$\leqslant$ 1 year (but > 3 months)	10
> 1 year	50

Notes

a A nominal duration of three days, to be chosen for short execution phases, corresponds to the extent in time of reliable meteorological predictions for the location of the site. This choice may be kept for a slightly longer execution phase if appropriate organisational measures are taken. The concept of mean return period is generally not appropriate for short term duration.

b For a nominal duration of up to three months actions may be determined taking into account appropriate seasonal and shorter term meteorological climatic variations.

A realistic minimum wind velocity during execution should be used for design. EC1 Part 1-6 specifies a minimum wind velocity of 20m/s but this is an NDP. The UK National Annex states that the minimum wind velocity should be defined for the individual project.

The characteristic value $Q_{k,R}$ of a variable action for the return period of *R* years may be determined on the basis of the characteristic value $Q_{k,50}$ for a variable action for a 50 year return period. This may be determined from the general relationship:

$$Q_{k,R} = k\, Q_{k,50}$$

where:
***k* is the reduction coefficient of a variable action based on the extreme-value distribution as explained in EC1 Part 1-3 (for snow loads), EC1 Part 1-4 (for wind loads) and EC1 Part 1-5 (for thermal actions). Relationships for determining *k* for thermal, snow and wind actions respectively are given in appropriate Parts of EC1. Values of *k* for different return periods *R* and different climatic actions are given in Table 8.4.**

Table 8.4 Reduction coefficient k of actions $Q_{k,R}$ for different return periods R

	Reduction coefficient k			
	Thermal actions		Snow loads	Wind actions
	for $T_{max,R}$	for $T_{min,R}$	for $s_{n,R}$	for $v_{b,R}$
2 years	0.8	0.45	0.64	0.77
5 years	0.86	0.63	0.75	0.85
10 years	0.91	0.74	0.83	0.90
50 years	1	1	1	1

The conditions for the combination of snow loads and wind actions with construction loads Q_c (see Section 8.4.10.1) should be defined for the individual project.

Other considerations

Imperfections in the geometry of the structure and of structural members should be defined for the individual project considering relevant execution standards.

Where the structure or parts of it are subjected to accelerations that may give rise to dynamic or inertia effects, these effects should be taken into account. Examples of situations that may cause accelerations are impact from a lorry, a dropped object, reciprocating machinery left on or beside a partially completed structure, etc. Significant accelerations may be excluded where possible movements are strictly controlled by appropriate devices.

Actions caused by water, including for example flooding or uplift due to groundwater, should be considered for the individual project.

Actions due to creep and shrinkage and elastic shortening in concrete construction works should be determined on the basis of the expected dates and duration associated with the design situations, where appropriate. Actions due to post-tensioning operations should be determined for short term effects on the structure (elastic shortening) and secondary (e.g. parasitic) effects.

8.3.2 Ultimate limit states

Ultimate limit states need to be verified for all selected transient and accidental design situations as appropriate during execution in accordance with Chapter 2.

The combinations of actions for accidental design situations (Chapter 2 expression 6.11b) can either include the accidental action explicitly or refer to a situation after an accidental event.

Generally, accidental design situations refer to exceptional conditions applicable to the structure or its exposure, such as:
– impact
– local failure and subsequent progressive collapse
– fall of structural or non-structural parts
– in the case of buildings, abnormal concentrations of building equipment and/or building materials
– water accumulation on steel roofs
– fire, etc.

Fire during construction should be treated as an accidental action. The verifications of the structure may need to take into account the appropriate geometry and resistance of the partially completed structure corresponding to the selected design situations. Patterns of temporary restraint should be taken into account.

For transient and accidental design situations the ultimate limit state verifications should be based on combinations of actions applied with the partial factors for actions γ_F and the appropriate ψ factors. For values of γ_G, γ_Q and ψ factors see Chapter 2.

To determine the representative values of the variable action due to construction loads ψ_0 is 1.0. The minimum recommended value of ψ_2 is 0.2.

Note ψ_1 does not apply to construction loads during execution.

8.3.3 Serviceability limit states

The serviceability limit states for the selected design situations during execution need to be verified, as appropriate, in accordance with Chapter 2. The combinations of actions should be established in accordance with Chapter 2. In general, for the verification of serviceability limit states for transient design situations during execution, the combinations of actions to be taken into account are the characteristic and the quasi-permanent combinations as defined in Chapter 2. To determine the representative values of the variable action due to construction loads ψ_0 is 1.0. The minimum recommended value of ψ_2 is 0.2.

Note Where frequent values of particular actions need to be considered, these values may be defined for the individual project.

The criteria associated with the serviceability limit states during execution need to take into account the requirements for the completed structure. The criteria associated with serviceability limit

states are defined for the individual project. See also EC2 to EC9. Any operations during execution which can cause excessive cracking and/or early deflections and which may adversely affect the durability, fitness for use and/or aesthetic appearance in the permanent condition should be avoided.

The Serviceability requirements for auxiliary construction works should be defined for the individual project. See also BS EN 12810[46], BS EN 12811[47], BS EN 12812[48], BS EN 12813[52] and related codes such as TG 20 (05[1])[49].

Load effects due to shrinkage, elastic shortening (post-tensioned concrete) and temperature need to be taken into account in the design and should be minimised by appropriate detailing.

8.4 Representation of actions

8.4.1 General

Characteristic and other representative values of actions need to be determined in accordance with EC0, EC1, EC7 and the associated Institution Manuals.

The representative values of actions during execution may be different from those used in the design of the completed structure. Common actions during execution, specific construction loads and methods for establishing their values are given in this Section.

See also Section 8.2 for classification of actions and Section 8.3 for nominal duration of transient design situations.

Action effects during execution may be minimised or eliminated by appropriate detailing, providing auxiliary construction works or by protecting/safety devices.

Representative values of construction loads (Q_c) should be determined taking into account their variations in time. See Section 8.4.10 for construction loads.

Interaction effects between structures and parts of structures should be taken into account during execution. Such structures should include structures that form part of the auxiliary construction works.

When parts of a structure are braced or supported by other parts of a structure (e.g. by propping floor beams for concreting) the actions on these parts resulting from bracing or supporting should be taken into account. Depending on the construction procedures, the supporting parts of the structure may be subjected to loads greater than the imposed loads for which they are designed for the persistent design situation. Additionally, any concrete elements (e.g. supporting slabs) may not have developed their full strength capacities.

8.4.2 Actions on structural and non-structural members during handling

The self-weight of structural and non-structural members during handling should be determined in accordance with Chapter 3.

Actions on structural and non-structural members due to support positions and conditions during transporting or storage should be taken into account, where appropriate.

8.4.3 Geotechnical actions

The characteristic values of geotechnical parameters, soil and earth pressures, and limiting values for movements of foundations have to be determined according to EC7.

The soil movements of the foundations of the structure and of auxiliary construction works, for example temporary supports during execution, need to be assessed from the results of geotechnical investigations. Such investigations should be carried out to give information on both absolute and relative values of movements, their time dependency and possible scatter. Movements of auxiliary construction works may cause displacements and additional stresses.

The characteristic values of soil movements estimated using statistical methods from geotechnical investigation data should be used as nominal values for imposed deformations of the structure.

8.4.4 Actions due to prestressing

Actions due to prestressing have to be taken into account, including the effects of interactions between the structure and auxiliary construction works (e.g. falsework) where relevant. Prestressing forces during execution may be determined according to the requirements of EC2 to EC6 and possible specific requirements defined for the individual project.

Prestressing forces during the execution stage may be taken into account as permanent actions.

8.4.5 Predeformations

The treatment of the effects of predeformations need to be in conformity with the relevant design Eurocode (from EC2 to EC6), and their associated Institution Manuals.

Action effects from execution processes should be taken into account, especially where predeformations are applied to a structure in order to generate action effects for improving its final behaviour, particularly for structural safety and serviceability requirements.

The action effects from predeformations should be checked against design criteria.

8.4.6 Temperature, shrinkage and hydration effects

The effects of temperature, shrinkage and hydration may have to be taken into account in each construction phase, as appropriate. For buildings, the actions due to temperature and shrinkage are not generally significant if appropriate detailing has been provided for the persistent design situation. Climatic thermal actions should be determined according to Chapter 7.

If considered necessary by the designer thermal actions due to hydration, and shrinkage effects in building materials should be determined according to EC2 to EC6 as appropriate.

8.4.7 Wind actions

The characteristic values of static wind forces Q_W should be determined according to Chapter 6 for the appropriate return period.

For lifting and moving operations or other construction phases that are of short duration, the maximum acceptable wind speed for the operations should be specified.

Wind actions on parts of the structure that are intended to be internal parts after its completion (e.g. internal walls) should be taken into account for execution processes. In such cases, the external pressure coefficients c_{pe} may have to be applied (e.g. for free-standing walls).

When determining wind forces, the areas of equipment, falsework and other auxiliary construction works that are loaded should be taken into account.

The need for a dynamic response design procedure for wind actions should be determined for the execution stages, taking into account the degree of completeness and stability of the structure and its various elements. Criteria and procedures may be defined for the individual project.

8.4.8 Snow loads

Snow loads should be determined according to Chapter 5 for the conditions of site and the required return period.

8.4.9 Actions caused by water

In general, actions due to water, including ground water, (Q_{wa}) should be represented as static pressures.

Actions caused by water may be taken into account in combinations as permanent or variable actions. The classification of actions caused by water as permanent or variable may be defined for the individual project, taking account of the specific environmental conditions. Consideration should be given to hydrostatic loading in part-built structures.

Actions from rainwater need to be taken into account for conditions where there may be collection of water such as ponding effects from, for example, inadequate drainage, imperfections of surfaces, deflections and/or failure of dewatering devices.

8.4.10 Construction loads

8.4.10.1 General

Construction loads (Q_c) may be represented in the appropriate design situations (see Chapter 2) either as one single variable action, or where appropriate different types of construction loads may be grouped and applied as a single variable action. Single and/or a grouping of construction loads should be considered to act simultaneously with non-construction loads as appropriate.

Construction loads to be considered are given in Table 8.5.

The characteristic values of construction loads, including vertical and horizontal components where relevant, need to be determined according to the technical requirements for the execution of the works and the requirements of EC0. Values of ψ factors for construction loads are given in Section 8.3.2.

Other types of construction loads may need to be taken into account. These loads may be defined for the individual project.

Horizontal actions resulting from the effects of construction loads need to be determined and taken into account in the structural design of a partly completed structure as well as the completed structure. When construction loads cause dynamic effects, these effects need to be taken into account.

Table 8.5 Representation of construction loads Q_c

Construction Loads Q_c				
Actions			Representation	Notes and remarks
Type	Symbol	Description		
Personnel and handtools	Q_{ca}	Working personnel, staff and visitors, possibly with handtools or other small site equipment	Modelled as a uniformly distributed load q_{ca} and applied to obtain the most unfavourable effects	The characteristic value $q_{ca,k}$ of the uniformly distributed load may be defined for the individual project and a minimum value of **0.75kN/m²** is recommended
Storage of movable items	Q_{cb}	Storage of movable items, e.g.: – building and construction materials, precast elements – equipment	Modelled as free actions and should be represented as appropriate by: – a uniformly distributed load q_{cb} – a concentrated load F_{cb}	The characteristic values of the uniformly distributed load and the concentrated load values may be defined for the individual project. For densities of construction materials, see EC1 Part 1-1
Non-permanent equipment	Q_{cc}	Non-permanent equipment in position for use during execution, either: – static (e.g. formwork panels, scaffolding, falsework, machinery, containers) or – during movement (e.g. travelling forms, launching girders and nose, counterweights)	Modelled as free actions and should be represented as appropriate by: – a uniformly distributed load q_{cc} ***Note*** A concentrated load F_{cc} may occur for the individual project. No advice is given in EC1 Part 1-6	These loads may be defined for the individual project using information given by the supplier. Unless more accurate information is available, the loads may be modelled by a uniformly distributed load with a recommended minimum characteristic value of $\mathbf{q_{cc,k} = 0.5kN/m^2}$. A range of CEN design codes are available, for example, see EN 12811[47] and for formwork and falsework design see EN 12812[48]
Movable heavy machinery and equipment	Q_{cd}	Movable heavy machinery and equipment, usually wheeled or tracked, (e.g. cranes, lifts, vehicles, lift trucks, power installations, jacks, heavy lifting devices)	Unless specified should be modelled on information given in the relevant parts of EC1	Information for the determination of actions due to vehicles when not defined in the project specification may be found in EC1 Part 2[36]. Information for the determination of actions due to cranes is given in EC1 Part 3[50]

Table 8.5 Continued

Construction Loads Q_c				
Actions			Representation	Notes and remarks
Type	Symbol	Description		
Accumulation of waste materials	Q_{ce}	Accumulation of waste materials (e.g. surplus construction materials, excavated soil, or demolition materials)	Taken into account by considering possible mass effects on horizontal, inclined and vertical elements (such as walls)	These loads may vary significantly, and over short time periods, depending on types of materials, climatic conditions, build-up rates and clearance rates, for example
Loads from parts of a structure in a temporary state	Q_{cf}	Loads from parts of a structure in a temporary state (under execution) before the final design actions take effect (e.g. loads from lifting operations)	Taken into account and modelled according to the planned execution sequences, including the consequences of those sequences (e.g. loads and reverse load effects due to particular processes of construction, such as sub-assemblage)	See also Section 8.4.10.2 for loads due to fresh concrete

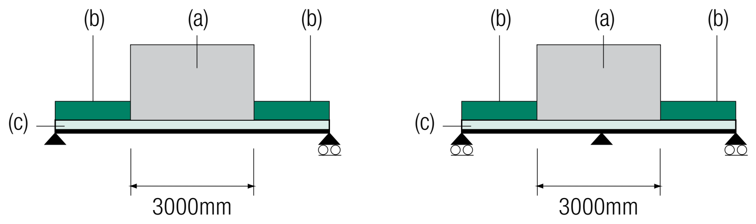
8.4.10.2 Construction loads during the casting of concrete

Actions to be taken into account simultaneously during the casting of concrete may include working personnel with small site equipment (Q_{ca}), formwork and loadbearing members (Q_{cc}) and the weight of fresh concrete, as appropriate. The weight of fresh concrete comprises two components: the weight for the design thickness ($Q_{cf,1}$) and the weight of additional concrete which exists in small temporary local areas of overfilling awaiting final placement and compaction ($Q_{cf,2}$).

Loads (a), (b) and (c) in Table 8.6 should be positioned to cause the maximum effects. The loads may or may not be symmetrical.

Table 8.6 Recommended characteristic values of actions due to construction loads during casting of concrete

Action	Loaded area	Load in kN/m2
(a)	Inside the working area 3m × 3m (or the span length if less)	10% of the self-weight of the weight of the fresh concrete for the design thickness but not less than 0.75kN/m2 and not more than 1.5kN/m2. Includes Q_{ca} and $Q_{cf,2}$
(b)	Outside the working area	A minimum of 0.75kN/m2 covering Q_{ca}
(c)	Actual area	Self-weight of the formwork, and load-bearing element (Q_{cc}) (minimum of 0.5kN/m2) and the weight of the fresh concrete for the design thickness ($Q_{cf,1}$)



The density of fresh concrete is 25kN/m2.

Alternative values for Q_{ca} and Q_{cc} may be determined for the individual project if a specific assessment is undertaken.

Values for Q_{cf} should be assessed and determined for the individual project taking account of the information provided in Table 8.5 and Section 8.4.10.2.

Other values may have to be defined, for example, when using self-levelling concrete or precast products.

Horizontal actions of fresh concrete may need to be taken into account where appropriate.

8.4.11 Accidental actions

Accidental actions such as impact from construction vehicles, cranes, building equipment or materials in transit (e.g. skip of fresh concrete), and/or local failure of final or temporary supports, including dynamic effects, that may result in collapse of load-bearing structural members, need to be taken into account where relevant. For considering dynamic effects due to accidental actions a value for the dynamic amplification factor of 2 is appropriate, subject to assessment or as specified for the individual project.

Abnormal concentrations of building equipment and/or building materials on load-bearing structural members are not regarded as accidental actions.

Actions due to falls of equipment onto or from a structure, including the dynamic effects, need to be defined and taken into account where relevant. The dynamic effects due to such falls of equipment need to be defined for the individual project.

Where relevant, a human impact load may need to be taken into account as an accidental action, represented by a quasi-static vertical force. The design values of the human impact force to be used are:

- 2.5kN applied over an area 200mm × 200mm, to account for stumbling effects
- 6.0kN applied over an area 300mm × 300mm, to account for falling effects.

The effects of the accidental actions described in this Section should be assessed to determine the potential for inducing movement in the structure. The extent and effect of any such movement should be determined, and the potential for progressive collapse assessed. See also Chapter 9.

Accidental actions used for design situations should be taken into account for any changes of the execution process. To ensure that the appropriate design criteria are applied at all times, corrective measures need to be taken as work proceeds.

Fire actions need to be taken into account where appropriate.

8.4.12 Horizontal actions

Horizontal actions resulting from, for example, wind forces and the effects of sway imperfections and sway deformations should be taken into account.

Nominal horizontal sway forces (F_{hn}) may be applied only when such a method can be justified as appropriate and can be demonstrated to be reasonable for a particular case. In such cases, the determined nominal horizontal forces should be applied at locations to give the worst effects, and may not always correspond to those of the vertical loads. The characteristic value of these equivalent horizontal forces is 3% of the vertical loads from the most unfavourable combination of actions.

Chapter 9: Accidental actions

9.1 General

9.1.1 Scope

This Chapter gives design guidance contained in EN1991-1-7[9] (EC1 Part 1-7) for Accidental Actions for the structural design of buildings within the scope of this *Manual* for the following subjects:

Strategies and rules for safeguarding buildings against identifiable and unidentifiable accidental actions (see also Section 9.3.1). The Chapter defines:
– strategies based on identified accidental actions, and
– strategies based on limiting the extent of failure (for unidentified and identified accidental actions).

Guidance is also given for the following:
– classification of actions (Section 9.2)
– design situations (Section 9.3)
– impact (Section 9.4)
– internal explosions (Section 9.5)
– design for consequences of localised failure in buildings from an unspecified cause (Section 9.6).

This Chapter does not specifically deal with accidental actions caused by external explosions, warfare and terrorist activities and these types of actions are outside the scope of this *Manual*.

9.1.2 Introductory advice for using this Chapter for the design of buildings

The guidance given in Chapter 9 concerns the basic requirement (see Section 2.6.1) for the robustness of building structures and the avoidance of progressive collapse resulting from an internal explosion or an impact.

9.1.3 Terms and definitions

A basic list of definitions is given in Chapter 2: Basis of Structural Design. Specific additional definitions for accidental actions are:

Consequence class – classification of the consequences of failure of the structure or part of it.

Deflagration – Propagation of a combustion zone at a velocity that is less than the speed of sound in the unreacted medium.

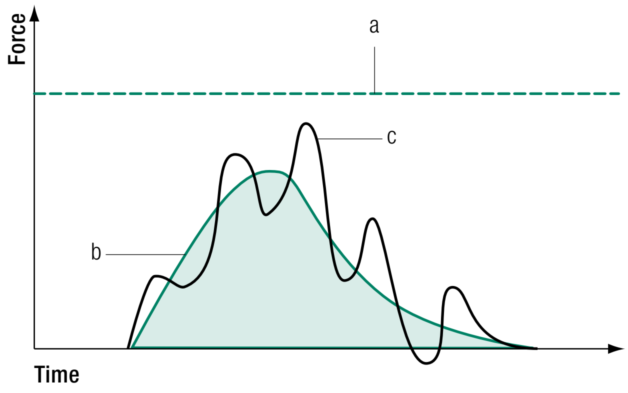
Dynamic force – Force that varies in time and which may cause significant dynamic effects on the structure; in the case of impact, the dynamic force represents the force with an associated contact area at the point of impact (see Figure 9.1).

Equivalent static force – An alternative representation for a dynamic force including the dynamic response of the structure (see Figure 9.1).

Impacting object – The object impacting upon the structure (i.e. vehicle, ship, etc.).

Key element – A structural member upon which the stability of the remainder or significant part of the structure depends.

Load-bearing wall construction – Non-framed masonry construction mainly supporting vertical loading. Also includes lightweight panel construction comprising timber or steel vertical studs at close centres with particle board, expanded metal or alternative sheathing.



a Equivalent static force
b Dynamic force
c Structural response

Fig 9.1 Representation of dynamic force

Localised failure – That part of a structure that is assumed to have collapsed, or been severely disabled, by an accidental event.

Risk – A measure of the combination (usually the product) of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence.

Robustness – The ability of a structure to withstand events like fire, internal explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.

Substructure – That part of a building structure that supports the superstructure. In the case of buildings this usually relates to the foundations and other construction work below ground level.

Superstructure – That part of a building structure that is supported by the substructure. In the case of buildings this usually relates to the above ground construction.

Venting panel – non-structural part of the enclosure (wall, floor, ceiling) with limited resistance that is intended to relieve the developing pressure from deflagration in order to reduce pressure on structural parts of the building.

9.2 Classification of actions

9.2.1 General

Classification of actions considers the variation of actions in time and space.

9.2.2 Accidental actions

Actions within the scope of this Chapter are classified as accidental actions in accordance with Chapter 2 and with EC0, clause 4.1.1.

Accidental actions due to impact should be considered as free actions (see Section 2.10.3.5). Accidental actions due to explosions should be treated as fixed actions in the design.

9.3 Design situations

9.3.1 General

Structures to withstand accidental actions have to be designed to the accidental design situations in accordance with Chapter 2. The strategies that can be considered for accidental design situations are illustrated in Figure 9.2.

The two strategies shown in Figure 9.2 recognise that an accidental action can be an *identified* (e.g. a delivery lorry impacting a large supermarket) or an *unidentified* (e.g. an internal gas explosion in a block of flats) accidental action.

For a design which is based on catering for *identified accidental actions* see Section 9.3.2. Notional values for *identified accidental actions* (e.g. in the case of some impacts and some internal explosions) are given in Section 9.4 for impact and Section 9.5 for internal explosions.

For a design which is based on catering for *unidentified accidental actions* see Section 9.3.3. This strategy covers a wide range of possible events and is related to strategies based on *limiting the extent of localised failure*. Guidance for buildings is given in Section 9.6. This strategy can also be used for identified accidental actions.

The strategies and rules to be taken into account for the individual project should be agreed with the client and the relevant authority.

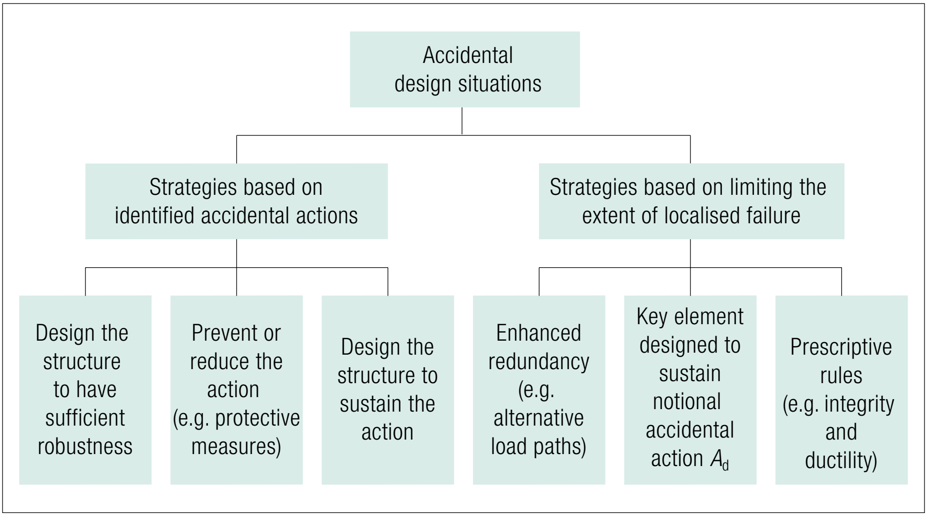


Fig 9.2 Strategies for accidental design situations

9.3.2 Accidental design situations – strategies for identified accidental actions

The different accidental actions that should be taken into account in the design (see Section 2.8.4) depend upon:
– the measures taken for preventing or reducing the severity of an accidental action
– the probability of occurrence of the identified accidental action
– the consequences of failure due to the identified accidental action
– public perception
– the level of acceptable risk.

Note The probability of occurrence and consequences of accidental actions can be associated with a certain risk level. If this level cannot be accepted, additional measures are necessary. A zero risk level, however, is unattainable and in most cases it will be necessary to accept a certain level of risk. Such a risk level can be determined by considering various factors, such as the potential number of casualties, the economic consequences and the cost of safety measures, etc. For specialist advice see Annex B of EC1 Part 1-7.

A localised failure due to an identified accidental action may be acceptable, provided:
– the stability of the whole structure is not endangered and
– that the overall load-bearing capacity of the structure is maintained to permit necessary emergency measures to be taken. For building structures such emergency measures may involve the safe evacuation of persons from the premises and its surroundings, inspection and the installation of temporary supports.

Measures should be taken to mitigate the risk of accidental actions and these measures should include, as appropriate, one or more of the following strategies:
– preventing the action from occurring (e.g. in the case of impact on the building from traffic, by providing adequate clearances between the trafficked lanes and the structure) or reducing the probability and/or magnitude of the action to an acceptable level through the structural design process (e.g. in the case of an explosion in a building by providing sacrificial venting panels with a low mass and strength to reduce the effect of the explosion)
– protecting the structure against the effects of an accidental action by reducing the effects of the action on the structure (e.g. by protective bollards or safety barriers).

The safety of the structure immediately following the occurrence of the identified accidental action has to be taken into account. This includes consideration of progressive collapse for building structures. See Section 9.6.

Within this strategy the structure has to have sufficient robustness and this can be achieved by adopting one or more of the following approaches:
– by designing certain components of the structure upon which stability depends as key elements (see Section 9.1.3) to increase the likelihood of the structure's survival following an accidental event
– designing structural members, and selecting materials, to have sufficient ductility capable of absorbing significant strain energy without rupture
– incorporating sufficient redundancy in the structure to facilitate the transfer of actions to alternative load paths following an accidental event.

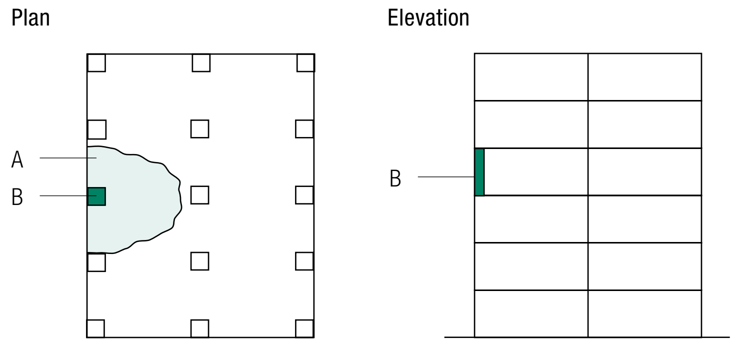
Accidental actions have to, where appropriate, be applied simultaneously in combination with permanent and other variable actions in accordance with Chapter 2 expression 6.11b/EC0 and Table 2.5.

9.3.3 Accidental design situations – strategies for limiting the extent of localised failure

In the design, the potential failure of the structure arising from an unspecified cause has to be mitigated.

The mitigation should be achieved by adopting one or more of the following approaches:
(a) designing key elements, on which the stability of the structure depends, to sustain the effects of a notional model of accidental action A_d; the model to be used for buildings is a uniformly distributed notional load applicable in any direction to the key element and any attached components (e.g. claddings, etc). The value for the uniformly distributed load is 34kN/m^2 for building structures. A_d is described in Section 9.6.7.
(b) designing the structure so that in the event of a localised failure (e.g. failure of a single member) the stability of the whole structure or of a significant part of it would not be endangered. The acceptable limit of 'localised failure' for building structures in the UK is 100m^2 *(Note:* at the time of writing, Building Regulations say 70m^2*)* or 15% of the floor area, whichever is less, on two adjacent storeys caused by the removal of any supporting column, pier, wall or beam supporting a wall or columns. See Figure 9.3. This is likely to provide the structure with sufficient robustness regardless of whether an identified accidental action has been taken into account.
(c) applying prescriptive design/detailing rules that provide acceptable robustness for the structure (e.g. three-dimensional tying for additional integrity). Rules and examples for this approach for buildings are given in Section 9.6.

***Note* According to UK practice it is necessary to provide horizontal ties as given in approach (c), even when the design involves approaches (a) or (b).**



A is 15% of the floor area or 100m² whichever is the smaller, in each of two adjacent storeys.
Note that at the time of writing Building Regulations say 70m², not 100m².

B is the column to be removed notionally.

Fig 9.3 Acceptable localised damage

9.3.4 Accidental design situations – use of consequence classes

In EC1 Part 1-7, the strategies for accidental design situations are normally based on the following consequence classes as defined in EC0 and shown in Table 9.1.

Table 9.1 Definition of consequence classes

Consequence Class	Description
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great.**
CC2	**Medium** consequence for loss of human life, *and* economic, social or environmental consequences **considerable.**
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible.**

Structures within consequence classes CC1 and CC2 generally fall within the scope of this *Manual,* but see also Table 1.1.

A classification of consequence classes relating to buildings is provided in Table 9.6.

Accidental design situations for CC1 and CC2 are considered in the following manner:

CC1: no specific consideration is necessary for accidental actions except to ensure that the robustness and stability rules given in EC0 to EC9, as applicable, are met;

CC2: depending upon the specific circumstances of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design/detailing rules may be applied;

For consideration of CC3 (high consequences of failure) see specialist advice in EC1 Part 1-7.

9.4 Impact

9.4.1 Field of application

Section 9.4 defines accidental actions due to the following events:
– impact from road vehicles (see Section 9.4.3)
– impact from forklift trucks (see Section 9.4.4)
– impact from trains (see Section 9.4.5)
– the hard landing of helicopters on roofs (see Section 9.4.6).

This *Manual* gives guidance for buildings for which actions due to impact need to be taken into account including:
– buildings used for car parking
– buildings in which vehicles or forklift trucks are permitted, and
– buildings that are located adjacent to either road or railway traffic.

9.4.2 Representation of actions

Actions due to impact should be determined by a dynamic analysis or represented by an equivalent static force.

The actions due to impact given in Section 9.4 assume that the impacting body absorbs all the energy.

For structural design the actions due to impact may be represented by an equivalent static force giving the equivalent action effects in the structure. This simplified model may be used for the verification of static equilibrium, for strength verifications and for the determination of deformations of the impacted structure.

9.4.3 Accidental actions caused by road vehicles

9.4.3.1 Impact on columns and walls of buildings

Vehicle collision need not be considered for most buildings in CC1 and CC2 provided that Sections 9.5 and 9.6 are used appropriately.

For situations where the client or the competent authority requires the consideration of impacts on particular buildings (e.g. a large supermarket or a high profile office building etc.) caused by road vehicles the guidance given below should be used.

Design values for actions (equivalent static design force) due to impact on columns and walls of buildings adjacent to various types of roads are given in Table 9.2. For buildings where the distance of the centre-line of the nearest trafficked lane to the structural member is ⩾ 10m, the equivalent static design force due to vehicle impact need not be considered.

The choice of the values may take account of the consequences of the impact, the expected volume, type and speed of traffic, and any mitigating measures provided.

Table 9.2 Indicative equivalent static design forces due to vehicular impact on supporting structures (e.g. columns and walls) adjacent to roadways

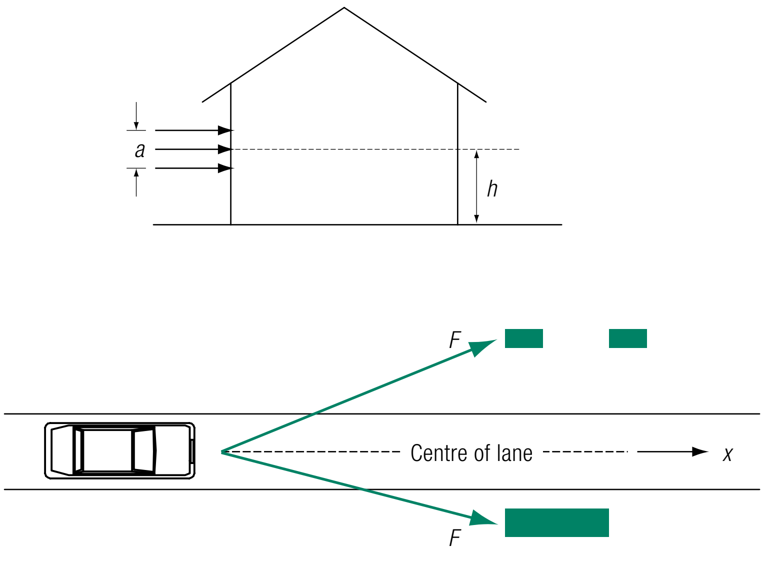
Category of traffic	Force F_{dx} [a] kN	Force F_{dy} [a] kN
Motorways and country national and main roads	1000	500
Country roads in rural area	750	375
Roads in urban area	500	250
Courtyards and parking garages with access to: – Cars – Lorries[b]	 50 150	 25 75

Notes
a x = direction of normal travel, y = perpendicular to the direction of normal travel.
b The term 'lorry' refers to vehicles with maximum gross weight greater than 3.5 tonnes.

The design may assume that F_{dx} does not act simultaneously with F_{dy}.

For impact on columns and walls of buildings the applicable area of resulting collision force F should be taken as follows (see Figure 9.4).

- For impact from lorries the collision force F may be applied at a height h of 1.5m above the level of the carriageway or higher where certain types of protective barriers are provided. The application area is a = 0.5m (height) by 1.50m (width) or the member width, whichever is the smaller.
- For impact from cars the collision force F may be applied at h = 0.50m above the level of the carriageway. The recommended application area is a = 0.25m (height) by 1.50m (width) or the member width, whichever is the smaller.



a is the height of the force application area. Ranges from 0.25m (cars) to 0.50m (lorries).

h is the location of the resulting collision force *F*, i.e. the height above the level of the carriageway. Ranges from 0.50m (cars) to 1.50m (lorries).

x is the direction of travel.

Fig 9.4 Collision force from road traffic on columns and walls of buildings

9.4.3.2 Impact on superstructures

Design values for actions due to impact from lorries and/or loads carried by lorries on members of the superstructure (e.g. the underside or the side of a building over a roadway) should be taken into account unless adequate clearances or suitable protection measures to avoid impact are provided.

The equivalent static design forces are given in Table 9.3. These values should be used when the clearance between the roadway and the underside of the building is less than or equal to **5.7m**. The equivalent static design force does not need to be taken into account when the clearance between the roadway and the underside of the building is more than **5.7m**.

Table 9.3 Indicative equivalent static design forces due to impact on superstructures

Category of traffic	Equivalent static design force F_{dx} [a] kN
Motorways and country national and main roads	500
Country roads in rural area	375
Roads in urban area	250
Courtyards and parking garages	75
Note **a** x = direction of normal travel.	

The choice of the values may take account of the consequences of the impact, the expected volume and type of traffic, and any mitigating (protective and preventative) measures provided.

Where appropriate, forces perpendicular to the direction of normal travel, F_{dy}, should also be taken into account. F_{dy} need not act simultaneously with F_{dx}. F_{dy} may be taken as 50% of F_{dx}.

The applicable area of the impact force F on the members of the superstructure may be taken as a square of side 0.25m.

9.4.4 Accidental actions caused by forklift trucks

For the design of buildings an equivalent static design force F due to accidental impact from forklift trucks may be taken as follows.

$$F = 5W$$

where W is the sum of the net weight and hoisting load of a loaded forklift truck (given in Chapter 3, Table 3.3) applied at a height of 0.75m above floor level.

9.4.5 Accidental actions caused by derailed rail traffic under or adjacent to building structures

9.4.5.1 Classification of structures subject to impact from derailed railway traffic

The design values for actions due to impact on supporting members (e.g. columns and walls) caused by derailed trains passing under or adjacent to building structures are dependent upon the class of the impacted structure as defined in Table 9.4. The strategy for design can also include other appropriate measures (both preventative and protective) to reduce, as far as is reasonably practicable, the effects of an accidental impact from a derailed train against supports of structures located above or adjacent to the tracks.

For more extensive guidance on accidental actions related to rail traffic, reference may be made to the UIC-code 777-2[53].

Table 9.4 Classes of structures subject to impact from derailed railway traffic

Class A	Structures that span across or near to the operational railway that are either permanently occupied or serve as a temporary gathering place for people or consist of more than one storey.
Class B ***Note*** Class B structures are outside the scope of this Manual	Massive structures that span across or near the operational railway such as bridges carrying vehicular traffic or single storey buildings that are not permanently occupied or do not serve as a temporary gathering place for people.

9.4.5.2 Accidental design situations in relation to the class of structure

Situations involving the derailment of rail traffic under or on the approach to a structure classified as Class A or B should be taken into account as an accidental design situation, in accordance with Section 2.7.1.

(a) Class A structures

For class A structures, where the maximum speed of rail traffic at the location is less than or equal to 120km/h, design values for the static equivalent forces due to impact on supporting structural members (e.g. columns, walls) are given in Table 9.5.

Table 9.5 Indicative horizontal static equivalent design forces due to impact for class A structures over or alongside railways

Distance 'd' (in metres) from structural elements to the centreline of the nearest track	Force F_{dx} [a] kN	Force F_{dy} [a] kN
Structural elements: d < 3m	To be specified for the individual project. See also Annex B of EC1 Part 1-7	To be specified for the individual project. See also Annex B of EC1 Part 1-7
For continuous walls and wall type structures: 3m ⩽ d ⩽ 5m	4000	1500
d > 5 m	0	0
Note **a** x = track direction, y = perpendicular to track direction.		

Further information on impact forces on structures alongside railways may be found in UIC 777-2[53].

The forces F_{dx} and F_{dy} (see Table 9.5) should be applied at a height of 1.8m above track level. The design should take into account F_{dx} and F_{dy} separately. If the maximum speed of rail traffic at the location is 50km/h or less, the values of the forces in Table 9.4 may be reduced by 50%. Further information may be found in UIC 777-2[53].

Locations where the maximum permitted speed of rail traffic is greater than 120km/h are outside the scope of this *Manual*.

(b) Class B structures

Class B structures are outside the scope of this *Manual*.

9.4.5.3 Structures located in areas beyond track ends

Actions caused by overrunning of rail traffic beyond the end of a track or tracks (for example at a terminal station) should be taken into account as an accidental design situation in accordance with EC0 when the structure or its supports are located in the area immediately beyond the track ends, but their treatment is outside the scope of this *Manual*.

9.4.6 Accidental actions caused by helicopters

For buildings with roofs designated as a landing pad for helicopters, an emergency landing force should be taken into account. The vertical equivalent static design force F_d should be determined from the following expression:

$$F_d = C\sqrt{m}$$

where:

F_d is the emergency landing force in kN
C is 3kN(kg$^{-1/2}$)
m is the mass of the helicopter (kg).

The force due to impact should be considered as acting on any part of the landing pad as well as on the roof structure within a distance of 7m from the edge of the landing pad. The area of impact should be taken as 2m x 2m.

9.5 Internal explosions in buildings

9.5.1 Field of application

Explosions need to be taken into account in the design of a building where gas is burned or regulated. Information on explosive materials such as dust, explosive gases, or liquids forming explosive vapour or gas which are stored or transported (e.g. chemical facilities, vessels, bunkers, sewage constructions, dwellings with gas installations, energy ducts, road and rail tunnels), which are outside the scope of this *Manual*, can be obtained from EC1 Part 1-7.

This Chapter defines actions due to internal explosions only.

9.5.2 Representation of action

For construction works classified as CC1 (see Section 9.3.4) no specific consideration of the effects of an explosion should be necessary other than complying with the rules for connections and interaction between components provided in EC2 to EC9. See Section 9.6 and Table 9.7.

For construction works classified as CC2, key elements of the structure should be designed to resist actions by either using an analysis based upon equivalent static load models, or by applying prescriptive design/detailing rules. See Section 9.6 and Table 9.7.

9.5.3 Principles for design

Structures need to be designed to resist progressive collapse resulting from an internal explosion, in accordance with the requirements given in Section 2.6.

9.6 Design for consequences of localised failure in buildings from an unspecified cause

9.6.1 Scope

This Section gives rules and methods for designing buildings to sustain an extent of localised failure from an unspecified cause without disproportionate collapse. Whilst other approaches may be equally valid, adoption of this strategy is likely to ensure that a building, depending upon its consequence class (see Section 9.3.4), is sufficiently robust to sustain a limited extent of damage or failure without collapse.

9.6.2 Introduction

Designing a building such that neither the whole building nor a significant part of it will collapse if localised failure were sustained is an acceptable strategy, in accordance with Section 9.3. Adopting this strategy should provide a building with sufficient robustness to survive a reasonable range of unidentified accidental actions.

9.6.3 Consequence classes of buildings

Table 9.6 provides a categorisation of building types/occupancies to consequence classes. This categorisation relates to the low (CC1), medium (CC2), and high (CC3) consequence classes given in Section 9.3.4.

The recommended strategies for providing a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse for consequence classes CC1, CC2a and CC2b are shown in Table 9.7.

Table 9.6 Categorisation of consequence classes

Consequence class	Example of categorisation of building type and occupancy
CC1	Single occupancy houses not exceeding 4 storeys. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height.
CC2a Lower Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1000m^2 floor area in each storey. Single storey educational buildings. All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000m^2 at each storey.
CC2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000m^2 but not exceeding 5000m^2 at each storey. Car parking not exceeding 6 storeys.
CC3	All buildings defined above as consequence class CC2 (Lower and Upper Risk Groups) that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5000 spectators. Buildings containing hazardous substances and/or processes.

Notes

a For buildings intended for more than one type of use the 'consequence class' should be that relating to the most onerous type.
b In determining the number of storeys, basement storeys may be excluded provided such basement storeys fulfil the requirements of 'Consequence class CC2b Upper Risk Group'.
c Table 9.6 is not exhaustive.
d Consequence class CC3 is outside the scope of this *Manual*.

Table 9.7 Strategies for consequence classes

Class	Recommended Strategy
CC1	Provided the building has been designed and constructed in accordance with the rules given in EC2 to EC9 for satisfying stability in normal use, no further specific consideration with regard to accidental actions from unidentified causes is necessary.
CC2a Lower Risk Group	In addition to the recommended strategies for consequence class CC1, effective horizontal ties, or effective anchorage of suspended floors to walls, as defined in Sections 9.6.4.1 and 9.6.4.2 respectively for framed and load-bearing wall construction should be provided.
CC2b Higher Risk Group	In addition to the recommended strategies for consequence class CC1: – horizontal ties, as defined in Sections 9.6.4.1 and 9.6.4.2 respectively for framed and load-bearing wall construction (see Section 9.1.3), together with vertical ties, as defined in Section 9.6.5, in all supporting columns and walls should be provided, or alternatively, – the building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall as defined in Section 9.6.6 (one at a time in each storey of the building) the building remains stable and that any local damage does not exceed a certain limit (see *Note*). Where the notional removal of such columns and sections of walls would result in an extent of damage in excess of the agreed limit, or other such limit specified, then such elements should be designed as 'key elements' (see *Note* and Section 9.6.7). In the case of buildings of load-bearing wall construction, the notional removal of sections of wall, one at a time, is likely to be the most practical strategy to adopt.
Note According to UK practice, buildings checked by notional removal of load bearing columns, walls, etc., or where key elements have been designed, should also have horizontal ties as described above.	

The limit of admissible local damage is given in Section 9.3.3b) and shown in Figure 9.3.

9.6.4 Horizontal ties

9.6.4.1 Framed structures

Horizontal ties should be provided around the perimeter of each floor and roof level and internally in two orthogonal directions to tie the column and wall elements securely to the structure of the building. The ties should be continuous and be arranged as closely as practicable to the edges of floors and lines of columns and walls. At least 30% of the ties should be located within the close vicinity of the centre lines of the columns and the walls.

Horizontal ties may comprise rolled steel sections, steel bar reinforcement in concrete slabs, or steel mesh reinforcement and profiled steel sheeting in composite steel/concrete floors (if directly connected to the steel beams with shear connectors). The ties may consist of a combination of the above types.

Each continuous tie, including its end connections, should be capable of sustaining a design tensile load of T_i for the accidental limit state in the case of internal ties, and T_p, in the case of perimeter ties, (see Figure 9.5) equal to the following values:

For internal ties
$T_i = 0.8\,(g_k + \psi q_k)sL$ or 75kN, whichever is the greater (9.1)

For perimeter ties
$T_p = 0.4\,(g_k + \psi q_k)sL$ or 75kN, whichever is the greater (9.2)

where:
s is the spacing of ties
L is the span of the tie (e.g. see Figure 9.5)
ψ is the relevant factor in the expression for combination of action effects for the accidental design situation (i.e. ψ_1 in accordance with expression 6.11b of EC0 given in Chapter 2, and is normally taken as 0.5).

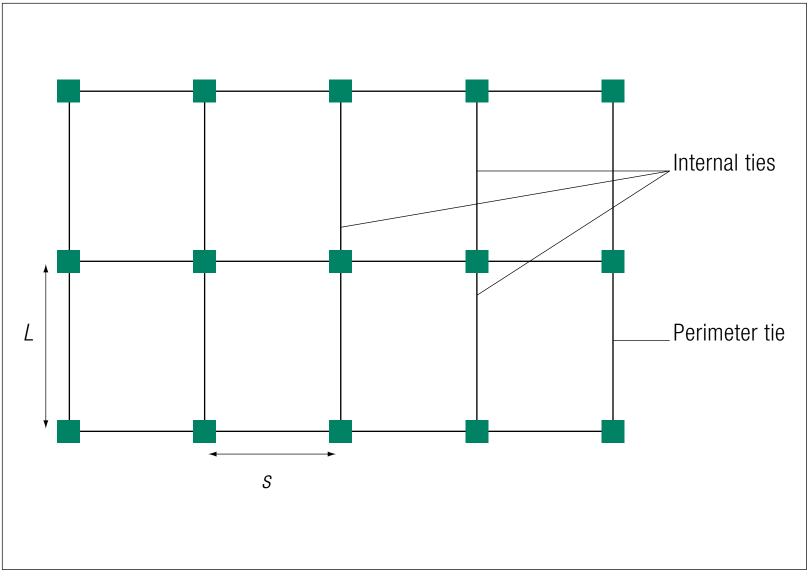


Fig 9.5 Description of spacing and span of ties for a discrete system (plan view)

In the case of lightweight building structures (e.g. those whose primary structure is timber or cold formed thin gauge steel) the values for minimum horizontal tie forces in expressions 9.1 and 9.2 should be taken as 15kN and 7.5kN respectively.

Members used for sustaining actions other than accidental actions may be utilised for the above ties.

9.6.4.2 Load-bearing wall construction

For class CC2a buildings (Lower Risk Group) (see Table 9.6):

Appropriate robustness should be provided by adopting a cellular form of construction designed to facilitate interaction of all components including an appropriate means of anchoring the floor to the walls.

For class CC2b buildings (Upper Risk Group) (see Table 9.6):
Continuous horizontal ties should be provided in the floors. Internal ties should be distributed throughout the floors in both orthogonal directions. Perimeter ties should extend around the perimeter of the floor slabs within a 1.2m width of the slab. The design tensile load in the ties should be determined as follows:

For internal ties:
T_i = the greater of F_t kN/m width of slab or

$$\frac{F_t(g_k + \psi q_k)}{7.5}\frac{z}{5} \text{ kN/m width of slab} \qquad (9.3)$$

For perimeter ties:

$$T_p = F_t \text{ kN} \qquad (9.4)$$

where:

F_t is 60 or 20 + 4n_s, whichever is less (note the F_t value has no units)
n_s is the number of storeys
z (in metres) is the lesser of:
- 5 times the clear storey height H (in metres), or
- the greatest distance in metres, in the direction of the tie, between the centres of the columns or other vertical load-bearing members whether this distance is spanned by:
  - a single slab or
  - by a system of beams and slabs.

Dimensions H and z are illustrated in Figure 9.6.

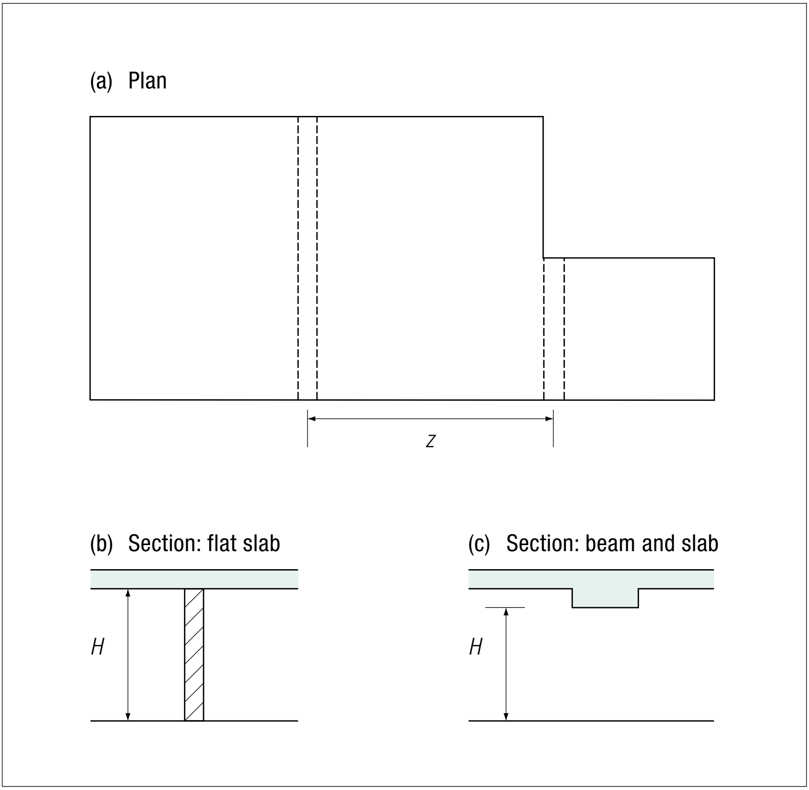


Fig 9.6 Illustration of dimensions *H* and *z*

9.6.5 Vertical ties

Each column and wall should be tied continuously from the foundations to the roof level.

In the case of framed buildings (e.g. steel or reinforced concrete structures) the columns and walls carrying vertical actions should be capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. Such accidental design loading should not be assumed to act simultaneously with permanent and variable actions that may be acting on the structure.

For load-bearing wall construction (see definition in Section 9.1.3) the vertical ties may be considered effective if all the following conditions are satisfied:

- the clear height of the wall, H, measured in metres between faces of floors or roof does not exceed $20t$, where t is the thickness of the wall in metres
- the ties are designed to sustain the following vertical tie force T:

$$T = \frac{34A}{8000}\left(\frac{H}{t}\right)^2 \text{ N, or 100kN/m of wall, whichever is the greater} \quad (9.5)$$

where:

A is the cross-sectional area in mm^2 of the wall measured on plan, excluding the non loadbearing leaf of a cavity wall

- the vertical ties are grouped at 5m maximum centres along the wall and occur no greater than 2.5m from an unrestrained end of the wall
- for masonry walls their thickness is at least 150mm and they are made with units having a minimum compressive strength of $5N/mm^2$ in accordance with EC6 Part 1-1[39].

9.6.6 Nominal section of load-bearing wall

The nominal section (length) of load-bearing wall construction referred to in Table 9.7 should be taken as follows:

- for a reinforced concrete wall, a length not exceeding $2.25H$
- for an external masonry wall, or a timber or steel stud wall, the length measured between lateral supports provided by other vertical building components (e.g. columns or transverse partition walls)
- for an internal masonry wall, or a timber or steel stud wall, a length not exceeding $2.25H$.

where:

H is the clear storey height in metres.

9.6.7 Key elements

In accordance with Section 9.3.3(a), for building structures a 'key element', as referred to in Table 9.6, should be capable of sustaining an accidental design action of $A_d = 34kN/m^2$ applied in the horizontal and vertical directions (in one direction at a time) to the member and any attached components having regard to the design ultimate strength of such components and their connections. Such accidental design loading should be applied in accordance with expression 6.11b of EC0 given in Chapter 2 of this *Manual.*

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Appendix A: Design data

Design data should include some or all of the following:
- company, contract, job number and date
- client, architect, engineer and checker responsible
- checking category (if applicable)
- project organisation – details of any design subcontracts
- Building Regulation authority or other and date of submission
- general description of building, intended use, location, and any unusual environmental conditions
- consequence class
- site constraints
- principal design codes and other important reference documents used
- performance criteria
- design methodology including computer programs used
- materials and proprietary systems
- design assumptions
- structural form, stability and robustness provisions
- general loading conditions including environmental and exposure conditions
- fire resistance requirements
- durability
- soil and groundwater conditions (including contamination)
- foundation type and design
- drainage
- movement joints
- quality plan
- site supervisor's check list or critical structural information to be included in such a list
- health and safety issues including risk management and CDM
- maintenance assumptions and recommendations
- other relevant data or information.

Appendix B: Supplementary advice on the characteristic value of a permanent action (G_k) and values of variable actions (Q_k)

B.1 Supplementary advice on Section 2.8.2 (Characteristic value of a permanent action – G_k)

Assuming a reasonable level of quality control it may be assumed that $G_k = G_{k,sup} = G_{k,inf}$.

It is recognised in this *Manual* that, whilst the Eurocode provisions are written around quantified variability, it will generally be very difficult to quantify this variability. First, the design is done in advance of execution and site works and therefore the designer will have to make assumptions about the level of control that will be achieved on site and the variability of as-constructed works. Second, even if a retrospective check of a design is to be done, the quantification of variability from site measurements is likely to be time-consuming and difficult.

The values to be used in the design for $G_{k,inf}$ and $G_{k,sup}$ should be chosen using engineering judgement and experience, and as explained below.

The following relationships[54] can be used to determine the lower value $G_{k,inf}$ and the upper value $G_{k,sup}$.

$$G_{k,inf} = \mu_G - 1.64\sigma_G = \mu_G (1 - 1.64V_G)$$
$$G_{k,sup} = \mu_G + 1.64\sigma_G = \mu_G (1 + 1.64V_G)$$

where:

V_G is the coefficient of variation of G
μ_G is the mean value of G
σ_G is the standard deviation of G.

Generally when $V_G \geqslant 0.10$, it is considered that the variability of G should be considered. For structures susceptible to overturning, the variability of G should be considered when $V_G \geqslant 0.05$.

From the above

- for a coefficient of variation $V_G = 0.10$, $G_{k,inf}$ and $G_{k,sup}$ will be 16.4% less than and greater than the mean value for G respectively
- for a coefficient of variation $V_G = 0.05$, $G_{k,inf}$ and $G_{k,sup}$ will be 8.2% less than and greater than the mean value for G respectively.

Note Further guidance may be obtained from reference 54 (in particular from Clauses 4.1.2 and 7.3.2).

B.2 Supplementary advice on Section 2.8.3 (Values of variable actions – Q_k)

A variable action has four representative values. These are used for the appropriate design situations in ultimate and serviceability limit state verifications. They are:
– the characteristic value Q_k
– the combination value $\psi_0 Q_k$
– the frequent value $\psi_1 Q_k$
– the quasi-permanent value $\psi_2 Q_k$.

The combination value $\psi_0 Q_k$, the frequent value $\psi_1 Q_k$, and the quasi-permanent value $\psi_2 Q_k$ are shown in diagrammatic form in Figure B.1.

(a) The characteristic value Q_k

For variable actions, the characteristic value (Q_k) (its main representative value) corresponds to either:
– an upper value with an intended probability of not being exceeded during some specific reference period; or
– a lower value with an intended probability of being achieved during some specific reference period; or
– a nominal value, which may be specified in cases where a statistical distribution is not known.

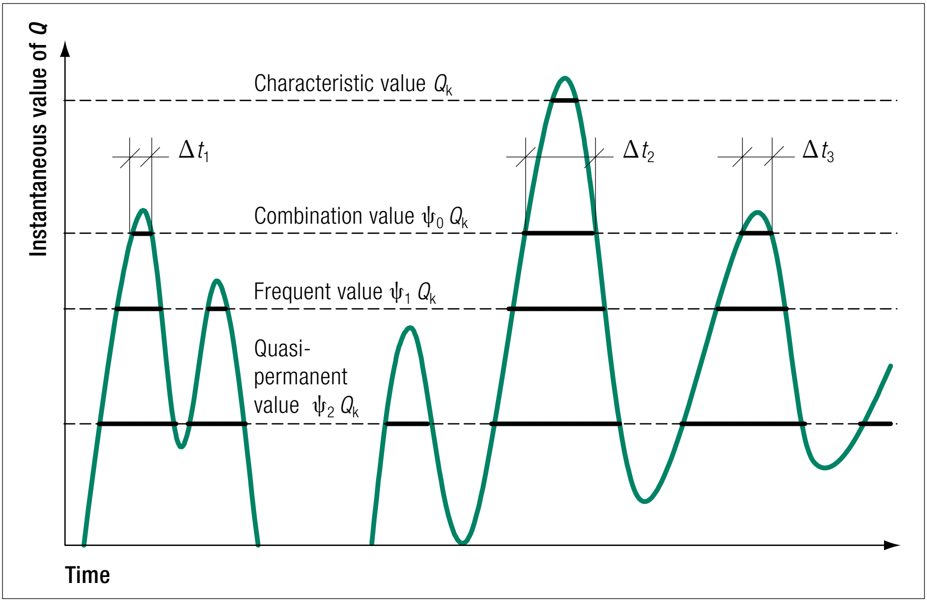


Fig B.1 Diagrammatic representation of representative values for actions

(b) The combination value $\psi_0 Q_k$

The combination value, represented as the product $\psi_0 Q_k$, is used for the:
- verification of ultimate limit states as described in Section 2.10.3.3
- verification of irreversible serviceability limit states (e.g. functionality of fittings with brittle behaviour) in order to take account of the reduced probability of simultaneous occurrence of the most unfavourable values of several independent actions.

ψ_0 is applied to the characteristic value of all accompanying actions for the EQU, STR and GEO ultimate limit state verifications in Section 2.10.3.3.

(c) The frequent value $\psi_1 Q_k$

The frequent value, represented as the product $\psi_1 Q_k$, is used for the:
- frequent combination in the serviceability limit states as described in Section 2.11.3; and
- verification of the accidental design situation of the ultimate limit state as described in Section 2.10.3.5.

In both cases, the reduction factor ψ_1 is applied as a multiplier of the leading variable action. An example of its use is normal office activity.

(d) The quasi-permanent value $\psi_2 Q_k$

The quasi-permanent value, represented as the product $\psi_2 Q_k$, is used for the:
- verification of ultimate limit states involving accidental actions as described in Section 2.10.3.5; and
- for the verification of frequent and quasi-permanent combinations in the serviceability limit state as described in Section 2.11.3.

Quasi-permanent values are also used for the calculation of long-term effects (e.g. cracking in slabs).

Appendix C: Combination of actions by means of expressions 6.10, 6.10a and 6.10b in EC0

In Section 2.10.3.1 it was noted that EC0 contains expressions 6.10, 6.10a and 6.10b and that more economical designs might result from the use of expressions 6.10a/6.10b. In such cases, verification of structural members can be done using expression 6.10 and/or the less favourable of the two expressions 6.10a and 6.10b. Expressions 6.10a/6.10b cannot be used in all circumstances; they can only be used as an alternative to expression 6.10 for the STR limit state. Expressions 6.10a/6.10b cannot be used for EQU, GEO or the STR component of STR/ GEO.

The full versions of the three expressions as shown in EC0 are shown below:

$$\sum_{j\geqslant 1} \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_p P \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(6.10 in EC0)}$$

$$\begin{cases} \sum_{j\geqslant 1} \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_p P \text{ "+" } \gamma_{Q,1} \psi_{0,1} Q_{k,1} \text{ "+" } \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} & \text{(6.10a in EC0)} \\ \sum_{j\geqslant 1} \xi_j \gamma_{G,j} G_{k,j} \text{ "+" } \gamma_p P \text{ "+" } \gamma_{Q,1} Q_{k,1} \text{ "+" } \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} & \text{(6.10b in EC0)} \end{cases}$$

where:

" + "	means "to be combined with"
Σ	means "the combined effect of"
ξ	is a reduction factor for *unfavourable* permanent actions *G*.

Expression 6.10 was discussed in Section 2.10.3.3 and simplified forms of it were also given there. Readers of this *Manual* are reminded of the need to use $G_{k,sup}$ and $G_{k,inf}$ in cases where variability in the values of permanent actions might be significant. See Section 2.8.2 and Appendix B in this regard. The same considerations relating to $G_{k,sup}$ and $G_{k,inf}$ apply when using expressions 6.10a/6.10b.

Verification for STR using expressions 6.10a/6.10b should be done in accordance with Table C.1.

Table C.1 Design values of actions (for STR)

Persistent and transient design situations	Permanent actions [b, d, e, f, g]		Leading variable action [c]	Accompanying variable actions [c, e, h]	
	Unfavourable [a]	Favourable		Main	Others
(Exp. 6.10a)	$1.35G_{k,j}$	$1.00G_{k,j}$		$1.5\psi_{0,1}Q_{k,1}$	$1.5\psi_{0,i}Q_{k,i}$
(Exp. 6.10b)	$0.925 \times 1.35G_{k,j}$	$1.00G_{k,j}$	$1.5Q_{k,1}$		$1.5y\psi_{0,i}Q_{k,i}$

Notes

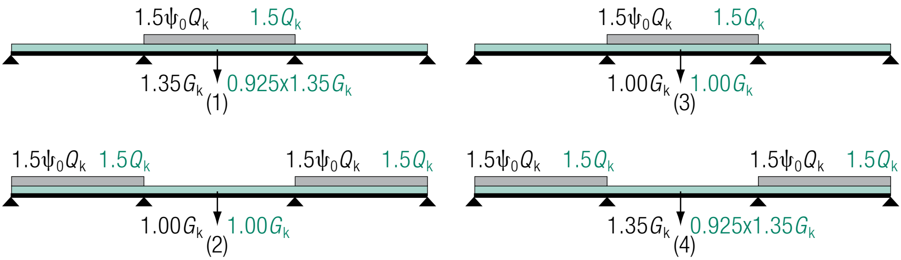
a When using expression 6.10b, $\gamma_{G,j}$ is multiplied by the reduction factor $\xi = 0.925$ becoming 1.25; this reduction factor $\xi = 0.925$ applies to unfavourable actions only.

b This note relates to the single source principle (see Section 2.10.3) The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,j} = 1.35$ ($0.925 \times 1.35G_{k,j}$ when using 6.10b) if the total resulting action effect is unfavourable; $\gamma_{G,j} = 1.00$ 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, e.g. several concrete spans in a floor interrupted by a steel span.

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

d Explanation of the use of $1.35G_{k,j}$ ($0.925 \times 1.35G_{k,j}$ when using 6.10b) and $1.00G_{k,j}$: EC0 differentiates between unfavourable and favourable effects of an action. In the diagrams below the factors to be used with expression 6.10a are shown in black and those used in 6.10b are shown in green.

Treatment of alternate spans by EC0



e Treatment of loading for alternate spans: Considering verification using expression 6.10a and 6.10b, the use of $1.35G_{k,j}$ ($0.925 \times 1.35G_{k,j}$ when using 6.10b) or $1.00G_{k,j}$ and the load arrangements for permanent and variable actions are illustrated in the above Figure. In diagrams (1) and (4) all three spans are loaded with $1.35G_{k,j}$ ($0.925 \times 1.35G_{k,j}$ when using 6.10b), and in diagrams (2) and (3) all three spans are loaded with $1.00G_{k,j}$. Diagrams (1) and (2), respectively, will give the maximum sagging and, if any, hogging moments in the mid-span region of the central span. Diagrams (3) and (4), respectively, will give the maximum hogging moment, if any, and the maximum sagging moment in the mid-span region of the end spans.

f If the variability of $G_{k,j}$ can be considered as small, the above table (i.e. showing one single value $G_{k,j}$) may be used.

g If the variability of $G_{k,j}$ cannot be considered as small, two values may need to be used where the results of a verification are very sensitive to variations in the magnitude of a permanent action from place to place in the structure: an upper value $G_{k,j,sup}$ for an unfavourable permanent action and a lower value $G_{k,j,inf}$ for a favourable permanent action. See Section 2.8.2 and Appendix B for selection of $G_{k,j,sup}$ and $G_{k,j,inf}$.

h Variable actions are those considered in Table 2.6.

Example C1: Combination of actions using expressions 6.10a/6.10b

Example 2.4 in Section 2.10.3.4 is reconsidered and the combinations of actions that result from the use of expressions 6.10a/6.10b are presented here.

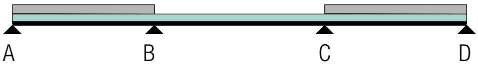
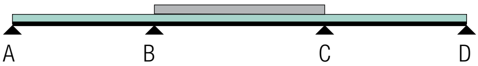
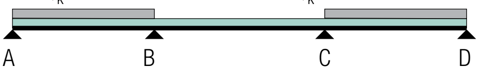
The objective in this example is restricted to action effects at the midspan points of spans AB and BC. Assume low variability in self-weight, giving a single value of $G_k = 25$kN/m. Office loading, for which $Q_k = 10$kN/m. Prestress is not present and geotechnical actions are not present.

The single source principle is used for self-weight. The 'chess-board' principle has been used for live load.

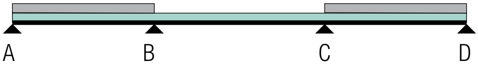
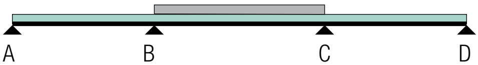
Note

All the factored loads in the diagrams below are in kN/m.

Example C1.1 Expressions 6.10a and 6.10b, span BC

Expression/condition	Application of actions and partial factors	Comments
6.10a (Unfavourable effect – span BC)	(33.75) (44.25) (33.75) $1.5\psi_0 Q_k$ = 1.5 x 0.7x10 = 10.5  $1.35G_k$ = 1.35 x 25 = 33.75 (udl for self weight)	ψ_0 = 0.7 for office areas (Table 2.6) This gives the maximum sagging moment in span BC for expression 6.10a
6.10a (Favourable effect – span BC)	(35.5) (25) (35.5) $1.5\psi_0 Q_k$ = 1.5 x 0.7 x 10 = 10.5 $1.5\psi_0 Q_k$ = 1.5 x 0.7 x 10 = 10.5  $1.00G_k$ = 1.00 x 25 = 25 (udl for self weight)	ψ_0 = 0.7 for office areas (Table 2.6) This gives the maximum hogging moment (if any) in span BC for expression 6.10a
6.10b (Unfavourable effect – span BC)	(31.22) (46.22) (31.22) $1.5Q_k$ = 1.5 x 10 = 15  0.925 x $1.35G_k$ = 0.925 x 1.35 x 25 = 31.22 (udl for self weight)	See Note **a** of Table C.1 This gives the maximum sagging moment in span BC for expression 6.10b
6.10b (Favourable effect – span BC)	(40) (25) (40) $1.5Q_k$ = 1.5 x 10 = 15 $1.5Q_k$ = 1.5 x 10 = 15  $1.00G_k$ = 1.00 x 25 = 25 (udl for self weight)	This gives the maximum hogging moment (if any) in span BC for expression 6.10b

Example C1.2 Expressions 6.10a and 6.10b, span AB

Expression/condition	Application of actions and partial factors	Comments
6.10a (Unfavourable effect – span AB)	(44.25) (33.75) (44.25) $1.5\psi_0 Q_k$ = 1.5 x 0.7 x 10 = 10.5 $1.5\psi_0 Q_k$ = 1.5 x 0.7 x 10 = 10.5  $1.35G_k$ = 1.35 x 25 = 33.75 (udl for self weight)	ψ_0 = 0.7 for office areas (Table 2.6) This gives the maximum sagging moment in span AB for expression 6.10a
6.10a (Favourable effect – span AB)	(25) (35.5) (25) $1.5\psi_0 Q_k$ = 1.5 x 0.7 x 10 = 10.5  $1.00G_k$ = 1.00 x 25 = 25 (udl for self weight)	ψ_0 = 0.7 for office areas (Table 2.6) This gives the maximum hogging moment (if any) in span AB for expression 6.10a
6.10b (Unfavourable effect – span AB)	(46.22) (31.22) (46.22) $1.5Q_k$ = 1.5 x 10 = 15 $1.5Q_k$ = 1.5 x 10 = 15  0.925 x $1.35G_k$ = 0.925 x 1.35 x 25 = 31.22 (udl for self weight)	See Note **a** of Table C.1 This gives the maximum sagging moment in span AB for expression 6.10b
6.10b (Favourable effect – span AB)	(25) (40) (25) $1.5Q_k$ = 1.5 x 10 = 15  $1.00G_k$ = 1.00 x 25 = 25 (udl for self weight)	This gives the maximum hogging moment (if any) in span AB for expression 6.10b

Concluding note on Example C1

The different cases considered in the above Example are the most critical for moments in spans AB and BC and would form the basis of verifications for the STR ultimate limit state or the STR component of the STR/GEO ultimate limit state. Moments, shears, reactions, etc. would be calculated from the combined actions shown and would depend on span lengths, etc.

Appendix D: Serviceability limit state verifications: Vertical and horizontal deformations

D.1 General

Vertical and horizontal deformations should be calculated in accordance with EC2 to EC6, and by using the appropriate combinations of actions according to Table 2.7.

D.2 Vertical deflections

Vertical deflections are represented schematically in Figure D.1.

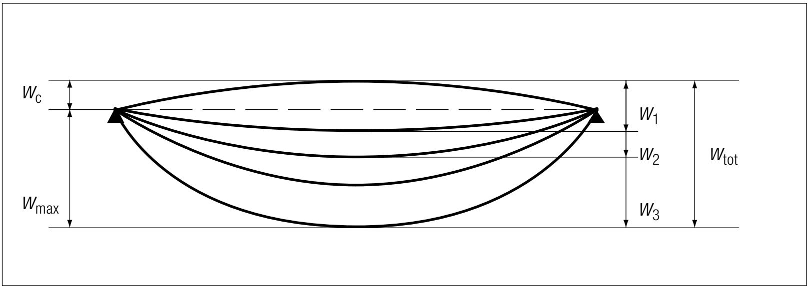


Fig D.1 Definitions of vertical deflections

Key

w_c Precamber in the unloaded structural member

w_1 Initial part of the deflection under permanent loads of the relevant combination of actions according to expressions 2.5 to 2.7 (expressions 6.14b to 6.16b in EC0)

w_2 Long-term part of the deflection under permanent loads

w_3 Additional part of the deflection due to the variable actions of the relevant combination of actions according to expressions 2.5 to 2.7 (expressions 6.14b to 6.16b in EC0)

w_{tot} Total deflection as sum of w_1, w_2, w_3

w_{max} Total deflection other than that due to precamber (i.e. w_{tot} - w_c).

If the functioning of the structure or damage to finishes or to non-structural members (e.g. partition walls, claddings) is being considered, the verification for deflection should take account of those effects of permanent and variable actions that occur after the execution of the member or finish concerned.

In the absence of specific requirements in the Manuals for EC2 to EC6, it is recommended that the indicative values for limiting deflection in Table D.1 are used together with the appropriate combination of action expressions.

Table D.1 Indicative limiting values for vertical deflections

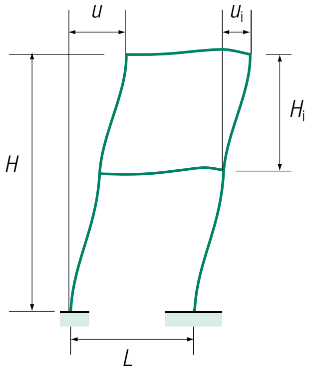
	Serviceability Limit States Vertical deflections – See Figure D.1		
	Irreversible effects of Actions	Reversible effects of Actions	
Serviceability Requirement	Characteristic Combination (Expression 6.14b in EC0) w_{tot} [a] or w_{max}	Frequent Combination (Expression 6.15b in EC0) w_{max}	Quasi-permanent Combination (Expression 6.16b in EC0) w_{max}
Function and damage to non-structural elements (e.g. partition walls claddings etc)[b] – Brittle – Non-brittle **Function** and damage to structural elements	 **⩽ *L*/500 to *L*/360** **⩽ *L*/300 to *L*/200** **⩽ *L*/300 to *L*/200**		
To avoid ponding of water		**⩽ *L*/250** [c]	
Comfort of user or functioning of machinery		**⩽ *L*/300**	
Crane gantry girders [d], deflection due to static wheel loads		**⩽ *L*/600**	
Appearance			**⩽ *L*/250**

Notes

a The benefits of any pre-camber may be considered if appropriate.
b These figures assume that partitions, cladding and finishes have not been specifically detailed to allow for anticipated deflections.
c The deflection limit of *L*/250 is appropriate for flat roofs of 2.5% slope or greater. A more restrictive limit would apply for roofs of less than this slope.
d Advice should be sought from the crane manufacturer.

D.3 Horizontal deflections

Horizontal displacements are represented schematically in Figure D.2.



u overall horizontal displacement over the building height H
u_i horizontal displacement over a storey height H_i

Fig D.2 Definition of horizontal displacements

In the absence of specific requirements in the Manuals for EC2 to EC6 it is recommended that the indicative values for limiting deflection given in Table D.2 are used together with the appropriate combination of action expressions.

Table D.2 Indicative limiting values for horizontal deflections due to variable actions

	Serviceability Limit States Horizontal deflections – See Figure D.2		
	Irreversible effects of Actions	Reversible effects of Actions	
Serviceability Requirement	Characteristic Combination (Expression 6.14b in EC0)	Frequent Combination (Expression 6.15b in EC0)	Quasi-permanent Combination (Expression 6.16b in EC0)
Function and damage to structural and non-structural elements – Single storey buildings – top of column – Each storey in a multi-storey building – The structure as a whole for a multi-storey building	 $\boldsymbol{u \leqslant H/300}$ $\boldsymbol{u_i \leqslant H_i/500}$ **to** $\boldsymbol{H_i/300}$ $\boldsymbol{u \leqslant H/500}$		
For crane gantry girders – horizontal deflections due to crane surge[a]		$\boldsymbol{\leqslant L/500}$	
Appearance			$\boldsymbol{u_i \leqslant H_i/250}$
Note **a** Advice should be sought from the crane manufacturer.			

Appendix E: Detailed information and worked examples for fire actions

E.1 Parametric fire exposure

For internal members of fire compartments, a method for the calculation of the gas temperature in the compartment is given in informative Annex A of EC1 Part 1-2. The temperature-time curves in the heating phase are given by:

$$\theta_g = 1325(1\text{-}0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}) + 20 \quad (^{\circ}C)$$

where:

θ_g	=	temperature in the fire compartment	(°C)
t^*	=	$t.\Gamma$	(h)
t	=	time	(h)
Γ	=	$[O/b]^2/(0.04/1160)^2$	(-)
b	=	thermal absorpivity: $\sqrt{(\rho c \lambda)}$ and should lie between 100 and 2200	($J/m^2s^{0.5}K$)
O	=	opening factor ($A_v\sqrt{h}/A_t$)	($m^{0.5}$)
A_v	=	area of vertical openings	(m^2)
h	=	height of vertical openings	(m)
A_t	=	total area of enclosure	(m^2)
ρ	=	density of boundary enclosure	(kg/m^3)
c	=	specific heat of boundary of enclosure	(J/kgK)
λ	=	thermal conductivity of boundary	(W/mK)

The concept of parametric time (t^*) is used to modify the predicted time-temperature relationship.

The values 0.04 and 1160 relate to the opening factor and the thermal inertia of the standard fire compartment as used in the original test programme.

The temperature within the compartment is then assumed to vary as a simple exponential function of modified time dependent on the variation in the ventilation area and the properties of the compartment linings from this "standard" compartment. The theory assumes that temperature rise is independent of fire load. In order to account for the depletion of the fuel or for the active intervention of the fire brigade or sprinkler systems the duration of the fire must be considered. This is a complex process and depends on the rate of burning of the material which itself is dependent on the ventilation and the physical characteristics and distribution of the fuel.

The parametric approach is a relatively straightforward calculation ideally suited for modern spreadsheets. It provides a reasonable estimate of the average time-temperature response for a wide range of compartments and is a major improvement on the nominal fires which bear little or no relation to a realistic fire scenario. The parametric fire curves comprise a heating phase represented by an exponential curve up to a maximum temperature θ_{max} occurring at a corresponding time of t_{max}, followed by a linearly deceasing cooling phase.

The maximum temperature in the heating phase occurs at a time t_{max} given by:

t_{max} = greater of $0.2 \times 10^{-3} \times q_{t,d}/O$ or t_{lim} (h)

where:

$q_{t,d}$ is the design value of the fire load density related to the total surface area of the enclosure. Values of $q_{t,d}$ should be in the range from 50 to $1000 MJ/m^2$.

t_{lim} is a minimum value for the duration of the fire based on slow, medium or fast fire growth rates. For office accommodation a medium fire growth rate should be assumed corresponding to a value of t_{lim} equal to 0.33 hours (20 minutes). For other occupancies the reader should refer to Table E.5 of EC1 Part 1-2.

For most practical combinations of fire load, compartment geometry and opening factor t_{max} will be in excess of the 20 minute limit. The temperature-time curves for the cooling phase are then given by:

$\theta_g = \theta_{max} - 625(t^* - t^*_{max})$ for $t^*_{max} \leqslant 0.5$ (h)

$\theta_g = \theta_{max} - 250(3 - t^*_{max})(t^* - t^*_{max})$ for $0.5 < t^*_{max} < 2$ (h)

$\theta_g = \theta_{max} - 250(t^* - t^*_{max})$ for $t^*_{max} \geqslant 2$ (h)

Although in EC1 Part 1-2 a number of restrictions are imposed on the use of the parametric approach such as maximum floor area of compartment and maximum height of compartment, the UK National Annex allows the approach to be used outside this limited scope using the non-contradictory, complementary information published in PD 6688-1-2[33]. The relevant input parameters are illustrated schematically in Figure E.1.

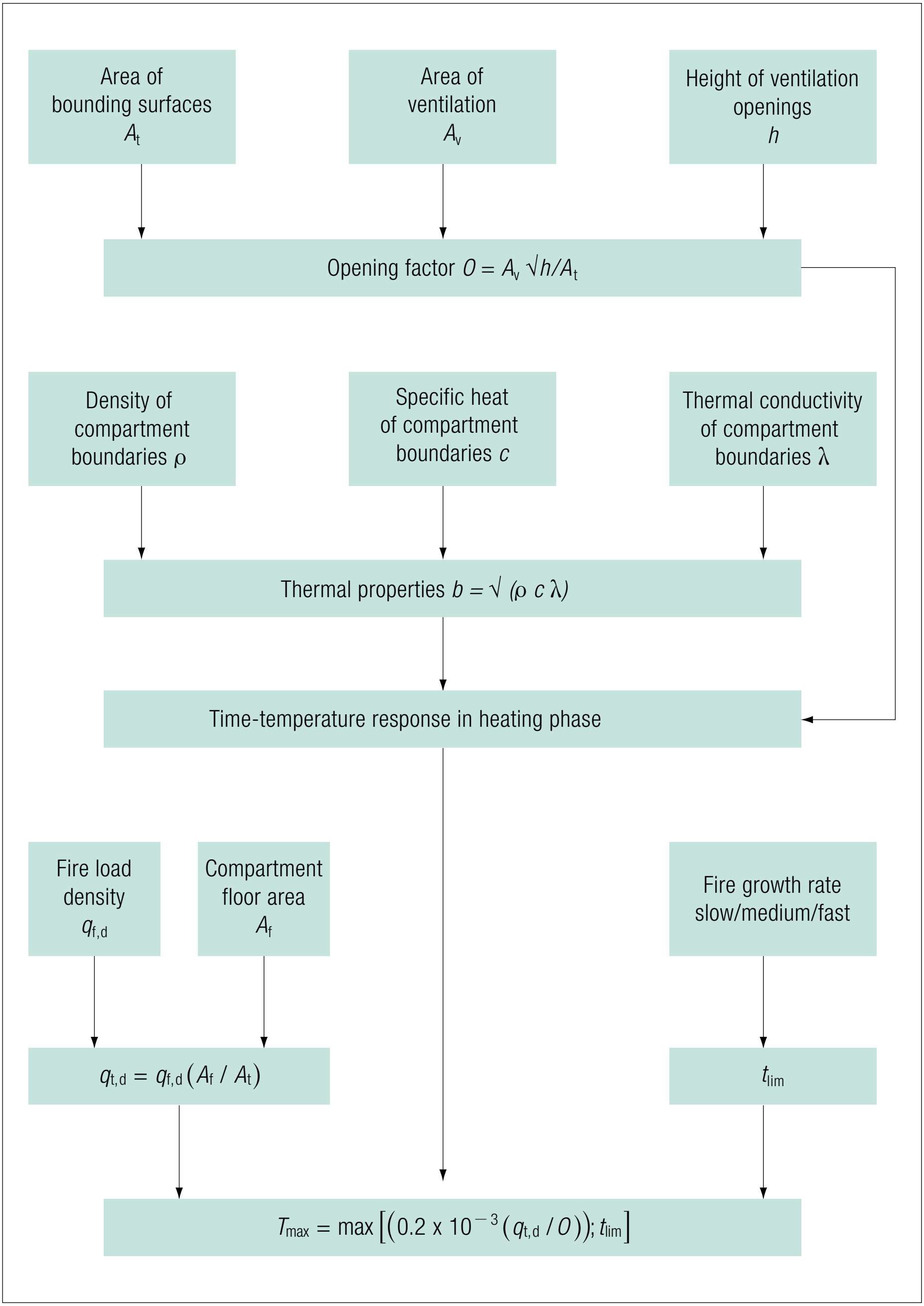


Fig E.1 Input values for parametric calculation

The parametric calculation is illustrated with a simple worked example.

Example: Parametric temperature-time curves

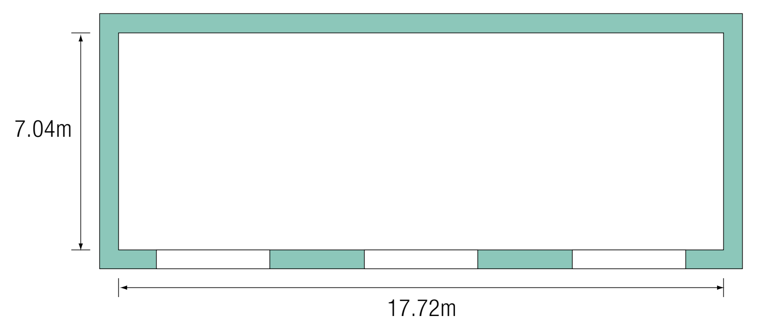


Fig E.2 Plan of fire compartment

Design data:
Dimensions of the compartment
width = 7.04m
length = 17.72m
height = 3.6m

Dimensions of windows
width = 2.2m
height = 1.6m
$n = 3$

Table E.1 Thermal properties of enclosure surfaces

	Density, ρ kg/m³	Specific heat, C J/kg K	Thermal conductivity, λ W/m² K	Thermal absorptivity $b_i = \sqrt{(\rho c \lambda)}$ J/m² s$^{0.5}$ K	Area of enclosure surface, A_j m²
Walls – plasterboard	900	1250	0.24	520	167.7
Floor – plasterboard	900	1250	0.24	520	124.7
Ceiling – concrete	2400	1506	1.5	2328	124.7

$$b = \sum \frac{(b_j A_j)}{A_t - A_v}$$

b = ((167.7+124.7)520+2328x124.7)/(167.7+124.7+124.7) = 1060 J/m^{2}s$^{0.5}$K
100 < 1060J/m^{2}s$^{0.5}$K < 2200 is OK

Total area of vertical openings on all walls
$A_v = 3 \times 2.2 \times 1.6 = 10.56\text{m}^2$

Opening factor of the fire compartment
$O = A_v \sqrt{h} / A_t = 10.56 \times \sqrt{1.6} / 427.6 = 0.0312\text{m}^{0.5}$

$0.02 < 0.0312\text{m}^{0.5} < 0.2$ is OK

Time factor function of the opening factor O and the thermal absorptivity b
$\Gamma = (O / b)^2 / (0.04/1160)^2 = (0.0312/1060)^2 / (0.04/1160)^2 = 0.728$

Fire load
$q_{f,d} = 570$ MJ/m^2
$q_{t,d} = q_{f,d} (A_f / A_t) = 570 (124.7/427.6) = 166.2$ MJ/m^2
$t_{lim} = 20$ min = 0.33 hour

Maximum temperature will be at time
$t_{max} = \max [t_{lim}; (0.2 \times 10^{-3}\, q_{t,d} / O)] =$
$\max [20; (0.33 \times 10^{-3} \times 166.2 / 0.0312)] = 1.065$ hour
$t^*_{max} = t_{max}\, \Gamma = 1.065 \times 0.728 = 0.776$ hour

The temperature-time curves in the heating phase are given by
$\theta_g = 1325 (1 - 0.324\, e^{-0.2\, t^*} - 0.204\, e^{-1.7\, t^*} - 0.472\, e^{-19\, t^*}) + 20$

Maximum temperature is
$\theta_{g,max} = 1325 (1 - 0.324\, e^{-0.2\, t^*} - 0.204\, e^{-1.7\, t^*} - 0.472\, e^{-19\, t^*}) + 20$
$= 1325 (1 - 0.324\, e^{-0.2 * 0.776} - 0.204\, e^{-1.7 * 0.776} - 0.472\, e^{-19 * 0.776}) + 20$
$= 905.1$ deg C

Cooling phase
For $t^*_{max} < 2$ hour the temperature in the cooling phase is given by
$\theta_g = \theta_{max} - 250(3 - t^*_{max})(t^* - t^*_{max})$

Table E.2 Parametric temperature-time curve

		t	t^*	Θ_g
second	minute	hour	hour	deg C
0	0.000	0	0.00000	20.0
5	0.083	0.00139	0.00101	32.5
10	0.167	0.00278	0.00202	44.7
15	0.250	0.00417	0.00303	56.7
....	....	....	....	....
....	....	....	....	....
3810	63.500	1.0583	0.77047	904.1
3815	63.583	1.0597	0.77148	904.3
3820	63.667	1.0611	0.77249	904.5
3825	63.750	1.0625	0.77350	904.7
3830	63.833	1.0638	0.77451	904.9
3835	**63.917**	**1.0652**	**0.77552**	**905.1**
3840	64.000	1.0666	0.77653	904.5
3845	64.083	1.0680	0.77754	904.0
....	....	....	....	....

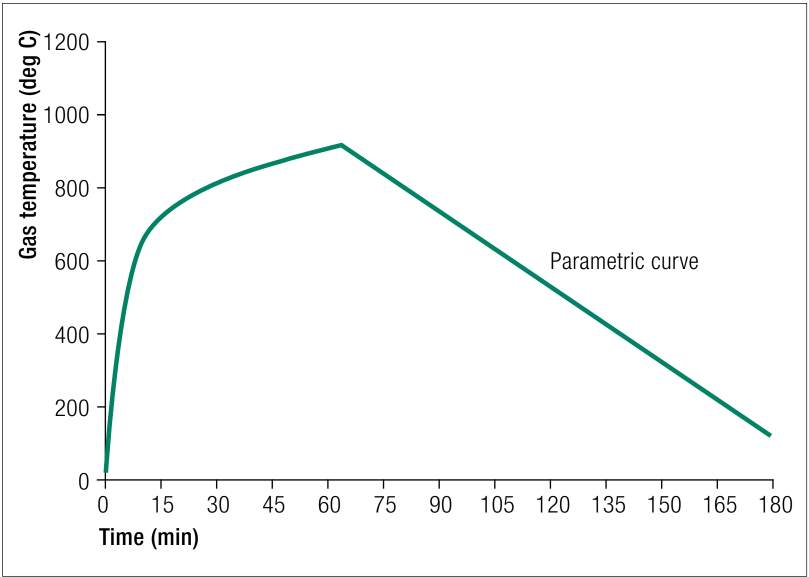


Fig E.3 Parametric curve

E.2 External atmosphere temperatures

For external members, the radiative heat flux component should be calculated as the sum of the contributions of the fire compartment and of the flames emerging from the openings. For external members exposed to fire through openings in the façade, a method for the calculation of the heating conditions is given in Annex B of EC1 Part 1-2. This procedure is based on work by Law[55] and was assessed during the development of the UK National Annex. It was found that certain combinations of window size and compartment geometry gave rise to excessively high temperatures. Where calculated temperatures of the fire or flames within a building using this method exceed 1750°K and 1850°K respectively these values should be used as upper limits.

E.3 Equivalent time of fire exposure

The concept of an equivalent period of fire exposure has been used for many years to quantify a real fire exposure (based on physical parameters) with an equivalent period of heating in the standard furnace test. The basic concept is illustrated in Figure E.4 and relates the maximum temperature in a natural fire to the time taken to reach the same temperature in a standard furnace test. The concept has been derived largely from tests on protected steel members although it has been extended to incorporate reinforced concrete members. In this way it operates as a translation of fire severity into a language which can be understood by building professionals and regulators.

The Eurocode equation is given below:

$$t_{e,d} = (q_{f,d} \times w_f \times k_b)\, k_c$$

where:

$q_{f,d}$ is the design fire load (MJ/m²)
k_b is the conversion factor for the compartment thermal properties (min.m²/MJ)
w_f is the ventilation factor
k_c is a correction factor dependent on the structural material.

Work undertaken in developing the UK National Annex showed that the method is material independent and therefore the correction factor could not be supported. In terms of heat transfer to structural members the method has been validated using k_c = 1.0 for protected steel, reinforced concrete and unprotected steel (for fire resistance periods of up to 30 minutes).

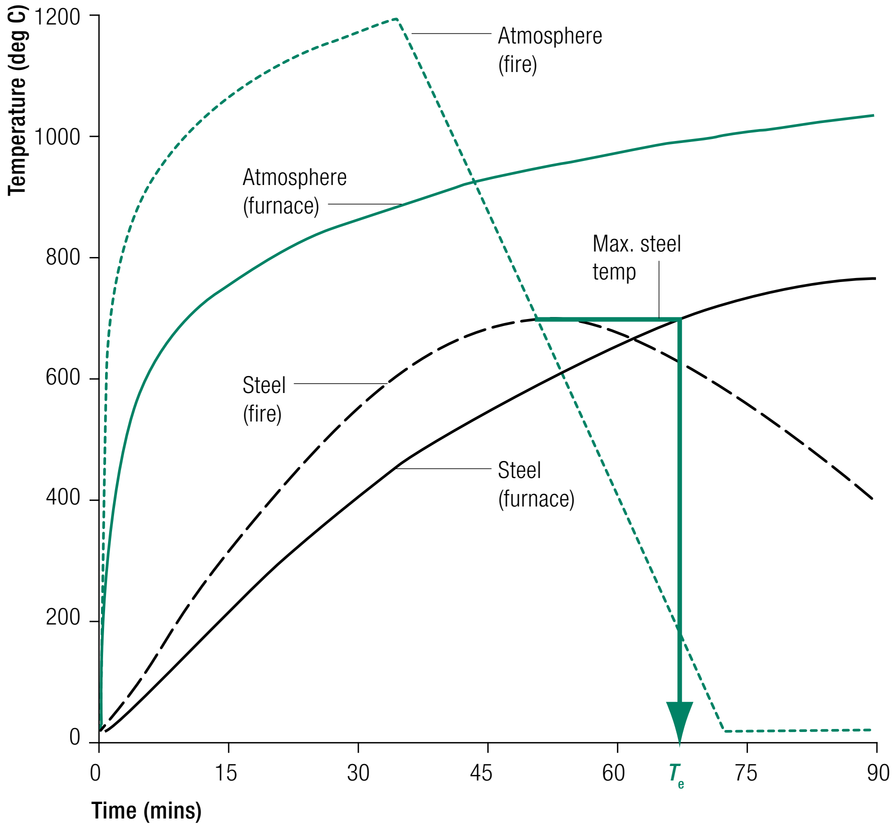


Fig E.4 Concept of time equivalence (based on protected steel member)

Fire load density refers to the material available for combustion and tabulated data based on the results from surveys are available related to specific occupancies. For design purposes the 80% fractile value is usually adopted. This is the value that is not exceeded in 80% of the sample occupancies. Table E.3 shows the values from the published guidance[33] referenced in the UK National Annex.

Table E.3 Fire load densities for different occupancies

Occupancy	Fire load density			
	Average (MJ/m²)	Fractile (MJ/m²)		
		80%	**90%**	**95%**
Dwelling	780	870	920	970
Hospital	230	350	440	520
Hospital storage	2000	3000	3700	4400
Hotel bedroom	310	400	460	510
Offices	420	570	670	760
Shops	600	900	1100	1300
Manufacturing	300	470	590	720
Manufacturing and storage	1180	1800	2240	2690
Libraries	1500	2250	2550	-
Schools	285	360	410	450

The ventilation factor w_f is derived from a consideration of the height of the compartment and the ratio of the openings to the floor area such that:
$w_f = (6/H)^{0.3} [0.62 + 90 (0.4 - \alpha_v)^4] \geqslant 0.5$ (in the absence of horizontal openings)

where H is the height of the compartment and $\alpha_v = A_v / A_f$. Alternatively, for small fire compartments where the floor area is less than 100m² the ventilation factor may be calculated from:

$w_f = O^{-0.5} \times A_f / A_t$

where O is the opening factor $A_v \sqrt{h} / A_t$, with h the (weighted) mean height of the ventilation openings. In carrying out a time equivalent analysis consideration should be given to the changes in ventilation which occur during the course of a fire, which have a significant bearing on temperatures attained and overall fire duration.

The National Annex sets out the appropriate values for k_b which differ from those in the informative annex. The default value for use in the UK is $k_b = 0.09$.

The concept of time equivalence has been used to develop an alternative classification system for building structures. However, the concept does not take into account implicit safety levels built in to existing National regulations to account for occupant mobility, ease of fire fighting and evacuation strategies. The output from time equivalent calculations therefore should not be used in isolation but should be part of an overall fire strategy for the building. The procedure in terms of input parameters is summarised in Figure E.5. For more detailed information the reader should consult the UK National Annex.

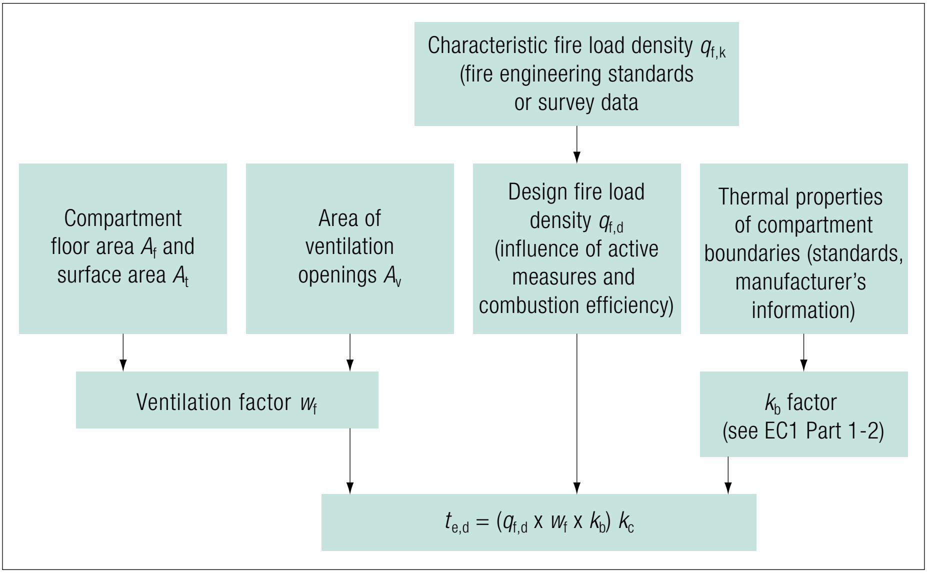


Fig E.5 Input values for equivalent time of fire exposure

The concept of time equivalence is illustrated with reference to a simple worked example. This is the same structure used for the worked example on parametric time-temperature curves.

Design data

Design fire load density
$q_{f,d} = 570$ MJ/m^2

Thermal absorptivity
$b = 1060$ J/m^{2}s$^{0.5}$K

Total area of vertical openings on all walls
$A_v = 10.56$m^2

Total area of horizontal openings on roof
$A_h = 0.0$m^2

Floor area
$A_f = 124.66$m^2

Total area of enclosure
$A_t = 427.56$m^2

Height of the fire compartment
$H = 3.6\text{m}$

Opening factor
$O = 0.0312\text{m}^{0.5}$

Calculation

Correction factor for reinforced concrete or protected steel is
$k_c = 1.0$ (EC1 Part 1-2, table F1, page 53)

Conversion factor (EC1 Part 1-2, table F2, page 54)
For $b \geqslant 720$ J/m^{2}s$^{0.5}$K and $b \leqslant 2500$ J/m^{2}s$^{0.5}$K is
$k_b = 0.055$ min m^2/MJ

However, the UK National Annex gives a conversion factor $k_b = 0.07$
(***Note*** If no detailed information is available on the thermal properties of the compartment linings or if there are uncertainties about the final construction or changes may be made over the course of the building's design life then the default value of $k_b = 0.09$ should be used.)

Area of vertical openings related to the floor area
$\alpha_v = A_v / A_f = 10.56/124.66 = 0.08$
With limit
$0.025 \leqslant \alpha_v \leqslant 0.25$, therefore 0.08 is OK

Area of horizontal openings related to the floor area
$\alpha_h = A_h / A_f = 0.0/124.66 = 0$
$b_v = 12.5(1+10 \times \alpha_v - \alpha_v^{\ 2}) = 12.5(1+10 \times 0.08 - 0.08^2) = 23.00$
With limit
$b_v \geqslant 10.0$, therefore 23.00 is OK

Ventilation factor is calculated as:
$w_f = (6.0/\text{H})^{0.3} \times [\ 0.62+90(0.4 - \alpha_v)^4 /(1+b_v\alpha_h)]$
$w_f = (6.0/3.6)^{0.3} \times [\ 0.62+90(0.4 - 0.08)^4 /(1+23.00 \times 0)] = 1.76$
With limit
$w_f = \geqslant 0.5$, therefore 1.76 is OK

The equivalent time of standard fire exposure is
$\boldsymbol{t_{e,d}} = (q_{f,d} \times k_b \times w_f)k_c = (570 \times 0.07 \times 1.76)1.0 = \boldsymbol{70.22\ min}$

(***Note*** The calculation has assumed a single opening factor corresponding to removal of all glazing within the fire compartment at the onset of flashover. In reality the breaking of glass is a time dependent variable and sensitivity studies should be undertaken to ascertain the effect of only part of the glazing failing during the fire. In the example if it is assumed that only half of the openings are available to contribute to the combustion process then the value of time equivalent increases to 97.32 min).

Appendix F: Detailed procedure for wind peak velocity pressure

F.1 Introduction

This detailed procedure for determining peak velocity pressure is from EC1 Part 1-4 and the UK National Annex and may be used for any building whether or not orography is significant. However, it is generally only necessary to use this procedure when orography is significant and the height z is >50m. (For the reasons given in Section 6.1, NDPs are not shown in bold in this appendix).

Figure F.1 gives a flow chart for using the detailed method.

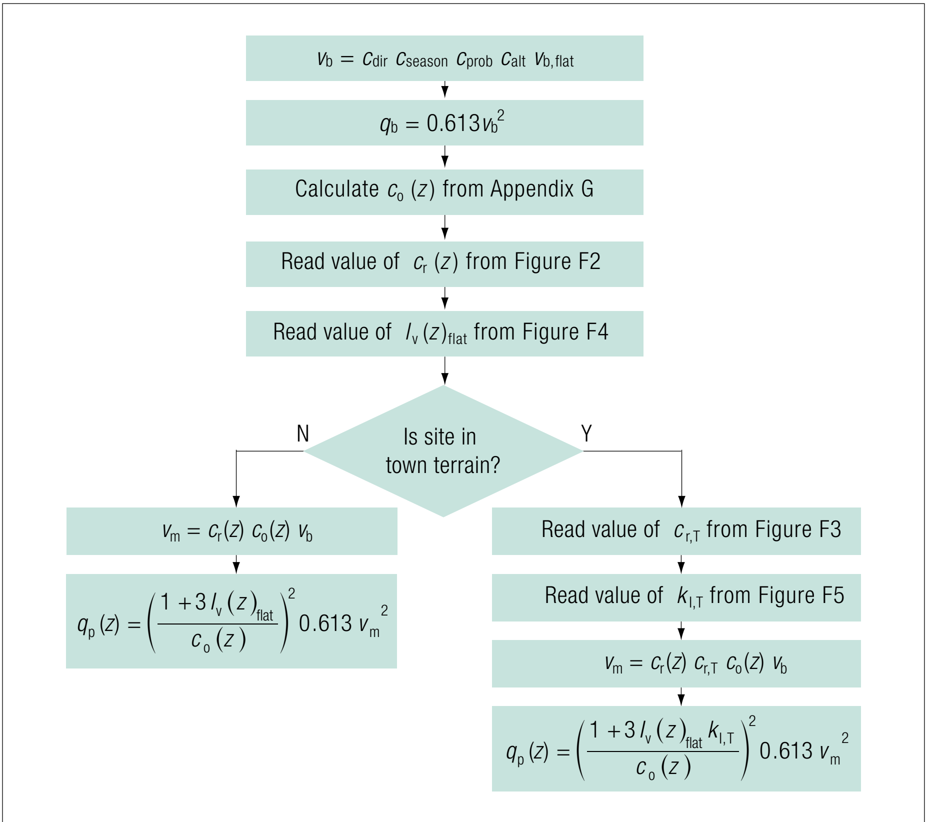


Fig F.1 Flow chart for determining q_p when orography is significant

F.2 Mean wind velocity

The mean wind velocity v_m depends on the effects of terrain roughness and orography and is only required in the calculation of the peak velocity pressure using the detail procedure (and for dynamic response calculations).

The mean wind velocity $v_m(z)$ at height z above ground level is given by:
$v_m(z) = c_r(z)\, c_o(z)\, v_b$ for sites in Country terrain
$v_m(z) = c_r(z)\, c_{r,T}\, c_o(z)\, v_b$ for sites in Town terrain

where:
$c_r(z)$ is the roughness factor, see Section F.3
$c_{r,T}$ is the roughness correction factor for sites in Town terrain, see Section F.3
$c_o(z)$ is the orography factor, see Appendix G.

F.3 Roughness factor

The roughness factor $c_r(z)$ accounts for the variability of the mean wind velocity due to the height above ground and the roughness of the terrain upwind of the site in the wind direction considered. The roughness factor $c_r(z)$ is obtained directly from Figure F.2. For sites adjacent to Sea terrain (sea or large inland lakes and estuaries), the distance upwind from the shoreline should be taken as 0.1km. For sites in Town terrain, the value of the roughness correction factor $c_{r,T}$ is obtained from Figure F.3.

Note The appropriate value of h_{dis} should be used in Figures F.2 and F.3. For sites in Country terrain $h_{dis} = 0$. For sites in Town terrain h_{dis} is given by Section 6.4.4.

The terrain roughness to be used for a given wind direction depends on the ground roughness and the uniformity of the roughness in each upwind wind sector considered. Variations of less than 10% by area in terrain roughness may be ignored. If in doubt then take the smoothest (most onerous) roughness category.

When a pressure or force coefficient is defined for a nominal angular sector, the smoothest roughness category within any 30° angular wind sector should be used.

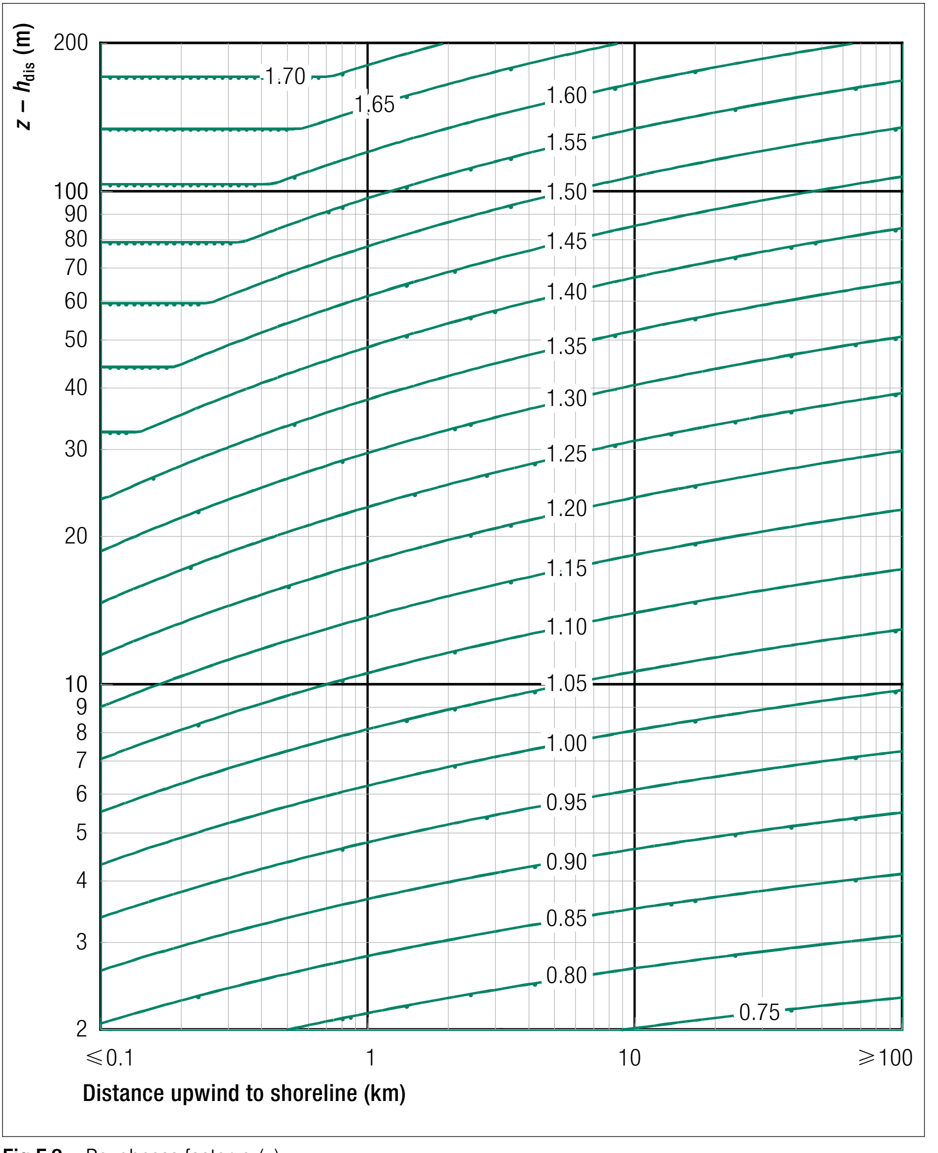


Fig F.2 Roughness factor $c_r\,(z)$

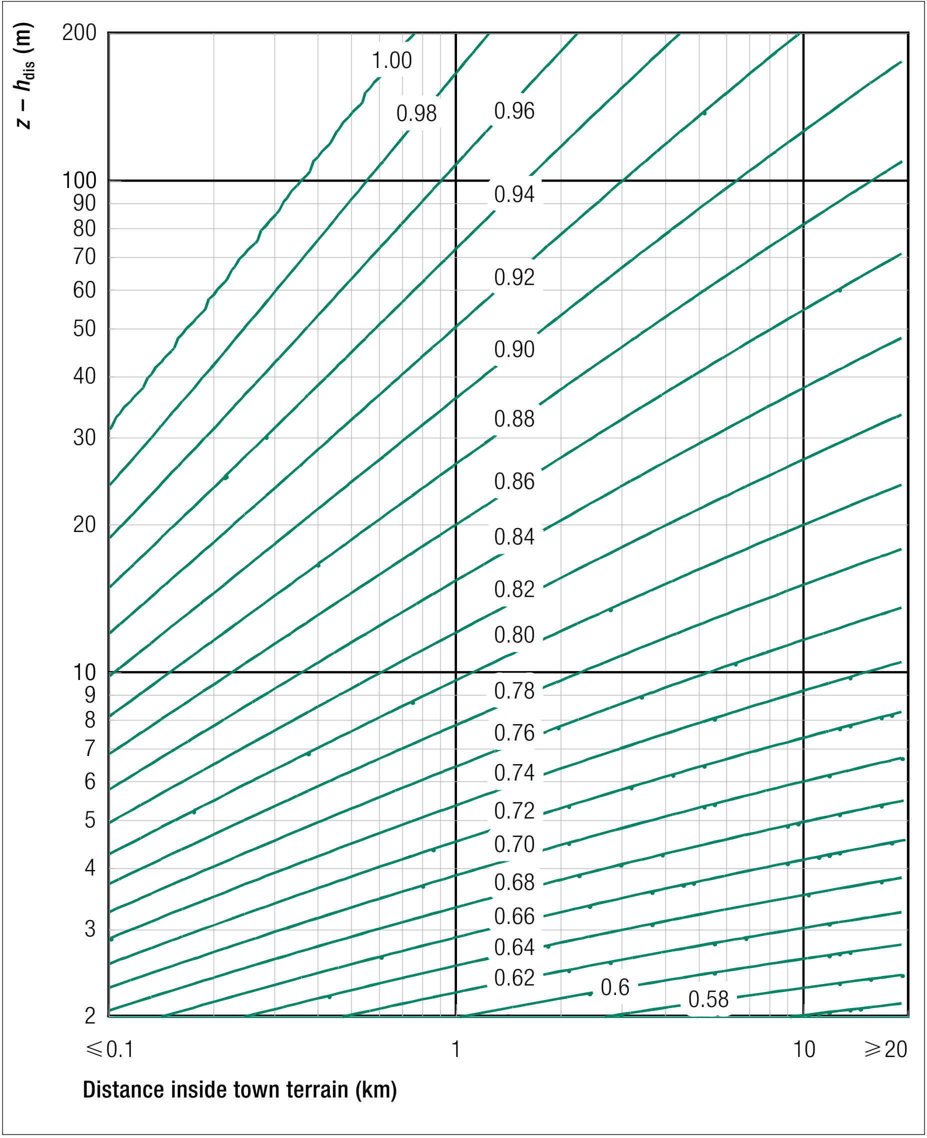


Fig F.3 Correction factor $c_{r,T}$ for sites in Town terrain

F.4 Wind turbulence

The wind turbulence is given in terms of the turbulence intensity $I_v(z)$, which is the standard deviation of the turbulence divided by the mean wind velocity.

In terrain where orography is significant, $I_v(z)$ is given by:

$$I_v(z) = \frac{I_v(z)_{flat}}{c_o(z)}$$ for sites in Country terrain and

$$I_v(z) = \frac{I_v(z)_{flat} k_{I,T}}{c_o(z)}$$ for sites in Town terrain

where:

$I_v(z)_{flat}$ is the turbulence intensity over flat terrain
$k_{I,T}$ is the turbulence correction factor for sites in Town terrain
c_o is the orography factor from Appendix G

The value of $I_v(z)_{flat}$ is given in Figure F.4. The value of $k_{I,T}$ is given in Figure F.5.

F.5 Peak velocity pressure

Where orography is significant *and* the building height is >50m tall, the peak velocity pressure $q_p(z)$ is given by:

$$q_p(z) = \left(\frac{1 + 3I_v(z)_{flat}}{c_o(z)}\right)^2 0.613 v_m^2$$ for sites in Country terrain and

$$q_p(z) = \left(\frac{1 + 3I_v(z)_{flat} k_{I,T}}{c_o(z)}\right)^2 0.613 v_m^2$$ for sites in Town terrain.

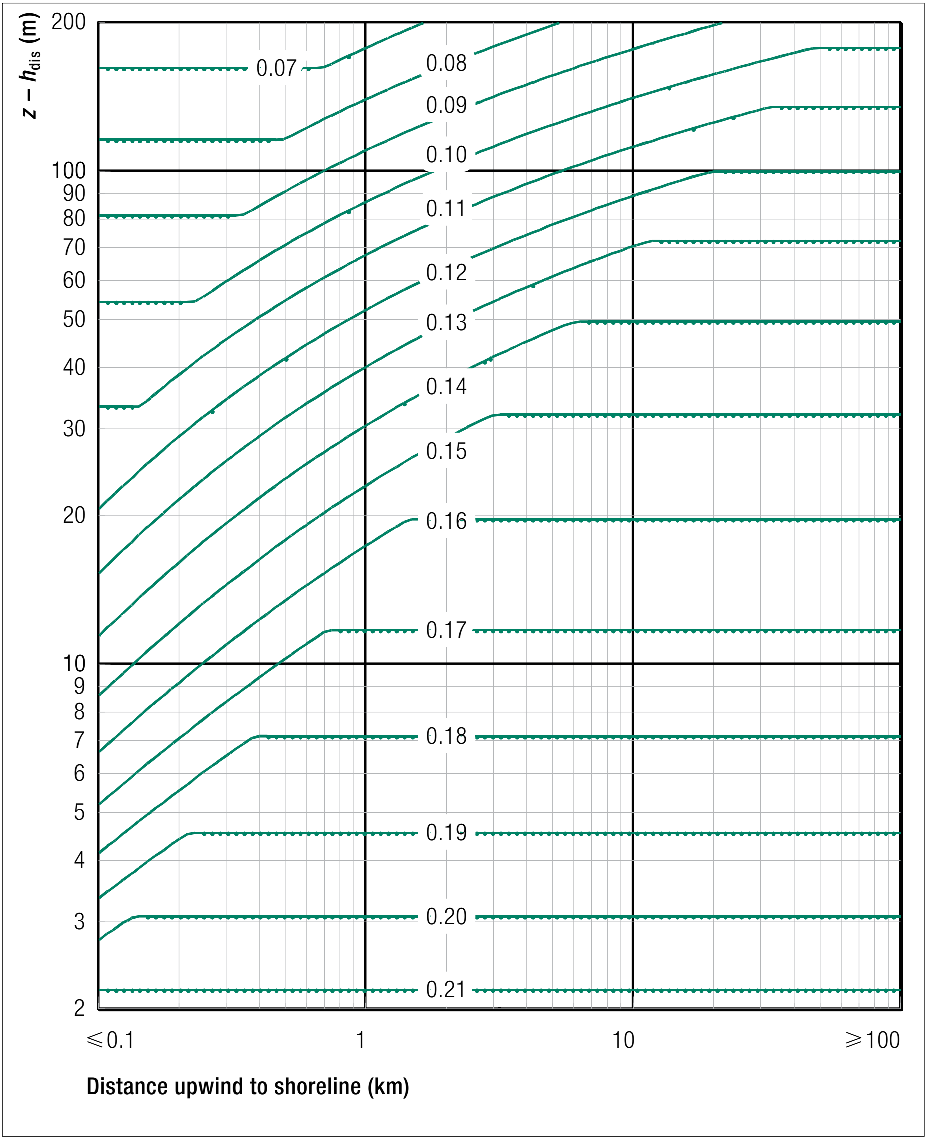


Fig F.4 Turbulence intensity for flat terrain $I_v(z)_{flat}$

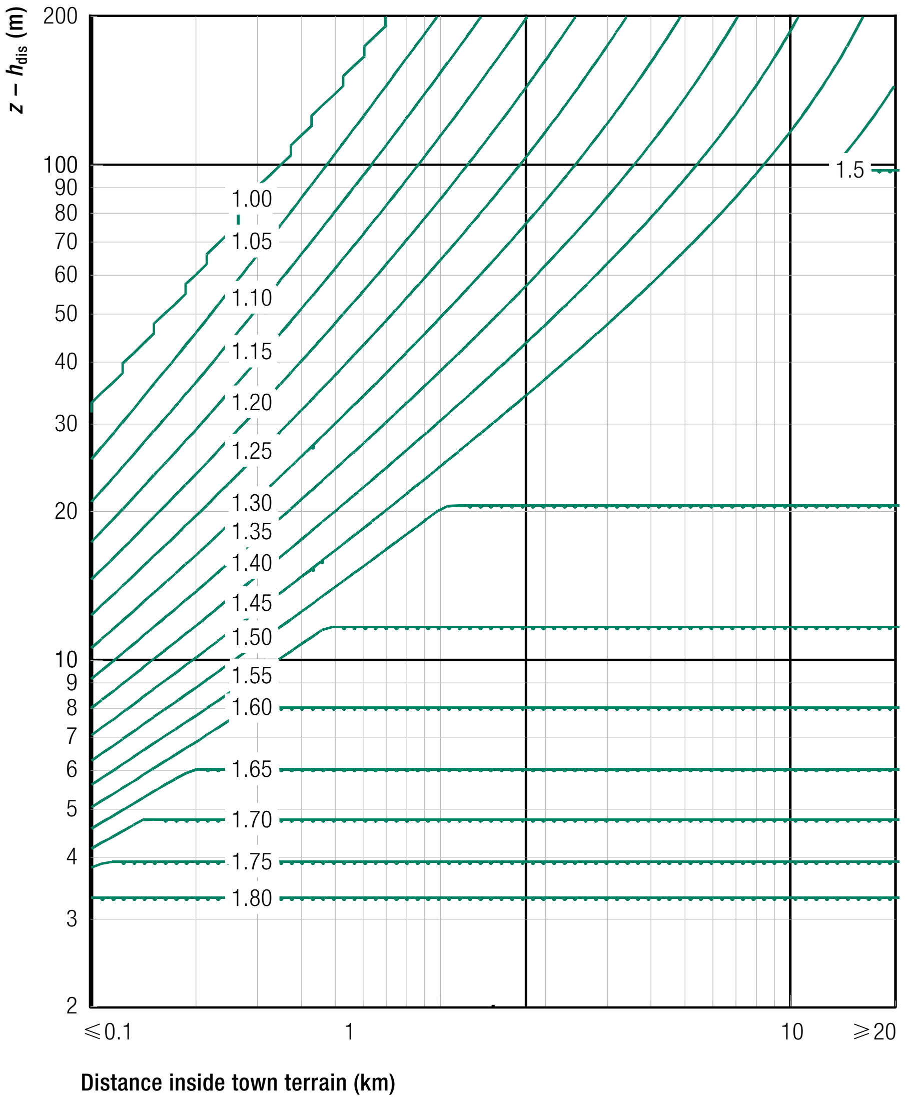
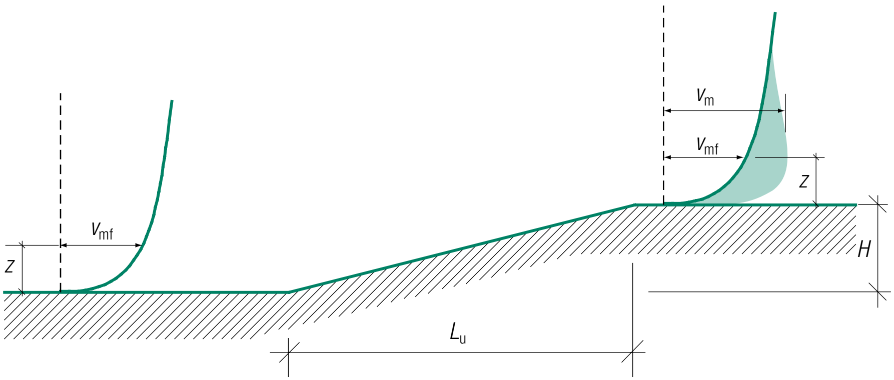


Fig F.5 Turbulence correction factor $k_{I,T}$ for Town terrain

Appendix G: Numerical calculation of orography factor

On isolated hills and ridges or cliffs and escarpments different wind velocities occur dependent on the upstream slope $\Phi = H/L_u$ in the wind direction, where the height H and the length L_u are defined in Figure G1.



v_m: mean wind velocity at height z above terrain
v_{mf}: mean wind velocity above flat terrain
$c_o = v_m / v_{mf}$

Fig G.1 Illustration of increase of wind velocities over orography

The largest increase in the wind velocities occurs near the top of the slope and is determined from the orography factor $c_o(z)$. The slope has no significant effect on the standard deviation of the turbulence.

The orography factor, $c_o(z)=v_m/v_{mf}$ accounts for the increase of mean wind speed over isolated hills and escarpments (not undulating and mountainous regions). It is related to the wind velocity at the base of the hill or escarpment. The effects of orography should be taken into account in the following situations:

For sites on upwind slopes of hills and ridges:
– where $0.05 < \Phi \leqslant 0.3$ and $|x| \leqslant L_u/2$

For sites on downwind slopes of hills and ridges:
– where $\Phi < 0.3$ and $x < L_d / 2$
– where $\Phi \geqslant 0.3$ and $x < 1.6\,H$

For sites on upwind slopes of cliffs and escarpments:
– where $0.05 < \Phi \leqslant 0.3$ and $|x| \leqslant L_u/2$

For sites on downwind slopes of cliffs and escarpments:
– where $\Phi < 0.3$ and $x < 1.5\,L_e$
– where $\Phi \geqslant 0.3$ and $x < 5\,H$

$c_o(z)$ is defined by:
$c_o = 1$ for $\Phi < 0.05$
$c_o = 1 + 2s\Phi$ for $0.05 < \Phi < 0.3$
$c_o = 1 + 0.6s$ for $\Phi > 0.3$

where:

s	is the orographic location factor, to be obtained from Figure G.2 or Figure G.3 scaled to the length of the effective upwind slope length, L_e
Φ	is the upwind slope H/L_u in the wind direction (see Figure G.2 and Figure G.3)
L_e	is the effective length of the upwind slope, defined in Table G.1
L_u	is the actual length of the upwind slope in the wind direction
L_d	is the actual length of the downwind slope in the wind direction
H	is the effective height of the feature
x	is the horizontal distance of the site from the top of the crest
$\|x\|$	is the modulus of x – for upwind slopes this is the horizontal distance of the site from the top of the crest ignoring whether it is upwind or downwind of the crest
z	is the vertical distance from the ground level of the site.

Table G.1 Values of the effective length L_e

Type of slope ($\Phi = H/L_u$)	
Shallow ($0.05 < \Phi < 0.3$)	Steep ($\Phi > 0.3$)
$L_e = L_u$	$L_e = H/0.3$

Note Values of *s* given in Figures G.2 and G.3 extend beyond the ranges defined in this Section. These values are given to allow interpolation to the edge of the ranges. It is optional whether the orographic effects beyond these boundaries are considered.

In valleys, $c_o(z)$ may be taken as 1.0 if no speed up due to funnelling effects is expected. For structures situated within steep-sided valleys care should be taken to account for any increase of wind speed caused by funnelling; if in doubt then specialist advice should be sought.

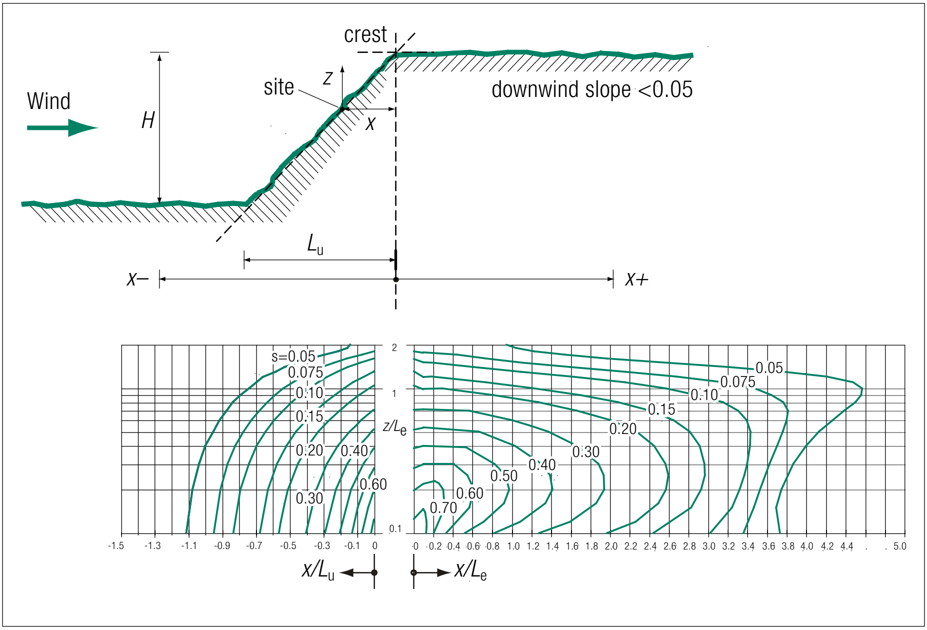


Fig G.2 Factor *s* for cliffs and escarpments

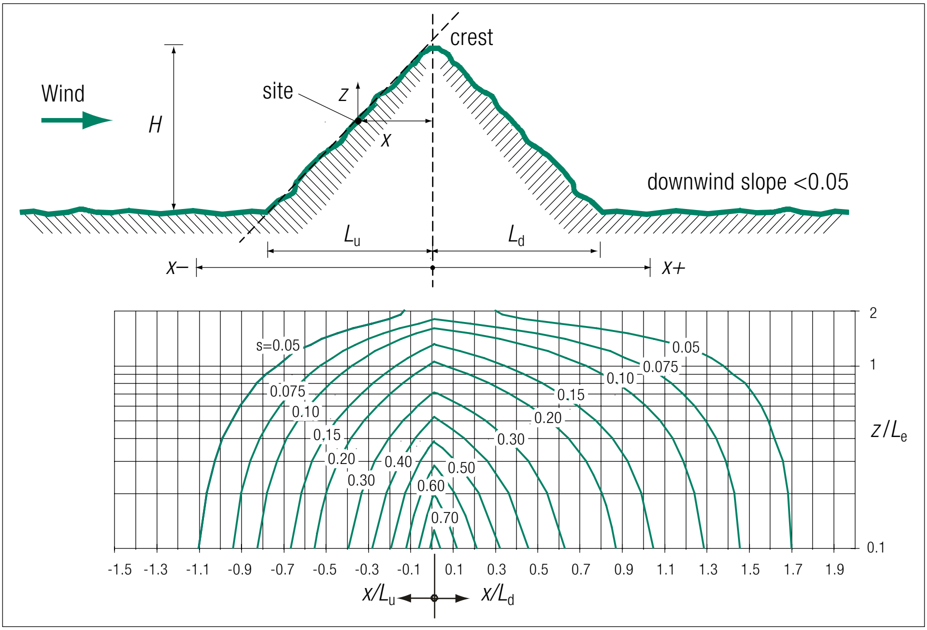


Fig G.3 Factor *s* for hills and ridges

The following equations may be used to determine the value of orographic location factor, s. As these expressions are empirical, it is important that values of the parameters used are restricted to the stated ranges, otherwise invalid values will be generated.

(a) upwind section for all orography (Figures G.2 and G.3):

For the ranges:

$$-1.5 \leqslant \frac{X}{L_u} \leqslant 0 \quad \text{and} \quad 0 \leqslant \frac{X}{L_e} \leqslant 2.0 \quad \text{take } s = Ae^{B}$$

where:

$$A = 0.1552\left(\frac{z}{L_e}\right)^4 - 0.8575\left(\frac{z}{L_e}\right)^3 + 1.8133\left(\frac{z}{L_e}\right)^2 - 1.9115\left(\frac{z}{L_e}\right) + 1.0124$$

and

$$B = \frac{Cx}{L_u}$$

and

$$C = 0.3542\left(\frac{z}{L_e}\right)^2 - 1.0577\left(\frac{z}{L_e}\right) + 2.6456$$

when $\frac{X}{L_u} < -1.5$ or $\frac{z}{L_e} > 2$ take $s = 0$.

(b) downwind section for cliffs and escarpments (Figure G.2):

For the ranges:

$$0.1 \leqslant \frac{X}{L_e} \leqslant 3.5 \text{ and } 0.1 \leqslant \frac{Z}{L_e} \leqslant 2.0 \text{ take } s = A\left(\log\left[\frac{X}{L_e}\right]\right)^2 + B\left(\log\left[\frac{X}{L_e}\right]\right) + C$$

where:

$$A = -1.3420\left(\log\left[\frac{Z}{L_e}\right]\right)^3 - 0.8222\left(\log\left[\frac{Z}{L_e}\right]\right)^2 + 0.4609 \log\left[\frac{Z}{L_e}\right] - 0.0791$$

and

$$B = -1.0196\left(\log\left[\frac{Z}{L_e}\right]\right)^3 - 0.8910\left(\log\left[\frac{Z}{L_e}\right]\right)^2 + 0.5343 \log\left[\frac{Z}{L_e}\right] - 0.1156$$

and

$$C = 0.8030\left(\log\left[\frac{Z}{L_e}\right]\right)^3 + 0.4236\left(\log\left[\frac{Z}{L_e}\right]\right)^2 - 0.5738 \log\left[\frac{Z}{L_e}\right] + 0.1606$$

For the range:

$0 \leqslant \frac{X}{L_e} \leqslant 0.1$ interpolate between values for $\frac{X}{L_e} = 0$ and $\frac{X}{L_e} = 0.1$

when $\frac{Z}{L_e} < 0.1$ use the values for $\frac{Z}{L_e} = 0.1$.

when $\frac{Z}{L_d} > 3.5$ or $\frac{Z}{L_e} > 2.0$ take $s = 0$.

(c) downwind section for hills and ridges (Figure G.3):

For the ranges:

$$0 \leqslant \frac{x}{L_d} \leqslant 2.0 \text{ and } 0 \leqslant \frac{z}{L_e} \leqslant 2.0 \text{ take } s = Ae^B$$

where:

$$A = 0.1552\left(\frac{z}{L_e}\right)^4 - 0.8575\left(\frac{z}{L_e}\right)^3 + 1.8133\left(\frac{z}{L_e}\right)^2 - 1.9115\left(\frac{z}{L_e}\right) + 1.0124$$

and

$$B = \frac{Cx}{L_u}$$

and

$$C = -0.3056\left(\frac{z}{L_e}\right)^2 + 1.0212\left(\frac{z}{L_e}\right) - 1.7637$$

when $\frac{x}{L_u} > 2.0$ or $\frac{z}{L_e} > 2.0$ take $s = 0$.

Appendix H: Photographs of terrain types as defined in EC1 Part 1-4

Fig H.1 Terrain category I (Country terrain)

Fig H.2 Terrain category II (Country terrain)

Fig H.3 Terrain category III (Town terrain)

Fig H.4 Terrain category IV (Town terrain)

Appendix I: Amended tables of external pressure coefficients for roofs

I.1 Status of tables in Appendix I

It is expected that Table 6.7 (Flat roofs), Table 6.8 (Monopitch roofs) Table 6.9 (Duo-pitch roofs) and Table 6.10 (Hip roofs) will be replaced in a future amendment to EC1 Part 1-4. The following tables are the expected replacement tables. However, the reader should confirm that these tables are correct before using them.

Table I.1 Replacement Table 6.7 External pressure coefficients for flat roofs

<table>
<tr><th rowspan="3" colspan="2">Roof type</th><th colspan="8">Zone</th></tr>
<tr><th colspan="2">F</th><th colspan="2">G</th><th colspan="2">H</th><th colspan="2">I</th></tr>
<tr><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" colspan="2">Sharp eaves</td><td rowspan="2">-2.0</td><td rowspan="2">-2.5</td><td rowspan="2">-1.4</td><td rowspan="2">-2.0</td><td rowspan="2">-0.7</td><td rowspan="2">-1.2</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="6">With Parapets</td><td rowspan="2">h/e=0.05</td><td rowspan="2">-1.9</td><td rowspan="2">-2.2</td><td rowspan="2">-1.3</td><td rowspan="2">-1.8</td><td rowspan="2">-0.7</td><td rowspan="2">-1.2</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="2">h/e=0.1</td><td rowspan="2">-1.85</td><td rowspan="2">-2.0</td><td rowspan="2">-1.3</td><td rowspan="2">-1.6</td><td rowspan="2">-0.7</td><td rowspan="2">-1.2</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="2">$h/e \geqslant 0.20$</td><td rowspan="2">-1.4</td><td rowspan="2">-1.8</td><td rowspan="2">-1.0</td><td rowspan="2">-1.4</td><td rowspan="2">-0.7</td><td rowspan="2">-1.2</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="6">Curved Eaves</td><td rowspan="2">$r/e = 0.05$</td><td rowspan="2">-1.05</td><td rowspan="2">-1.5</td><td rowspan="2">-1.2</td><td rowspan="2">-1.8</td><td rowspan="2" colspan="2">-0.4</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="2">$r/e = 0.10$</td><td rowspan="2">-0.75</td><td rowspan="2">-1.2</td><td rowspan="2">-0.8</td><td rowspan="2">-1.4</td><td rowspan="2" colspan="2">-0.3</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="2">$r/e = 0.20$</td><td rowspan="2">-0.55</td><td rowspan="2">-0.8</td><td rowspan="2">-0.55</td><td rowspan="2">-0.8</td><td rowspan="2" colspan="2">-0.3</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="6">Mansard Eaves</td><td rowspan="2">$\alpha = 30°$</td><td rowspan="2">-0.95</td><td rowspan="2">-1.5</td><td rowspan="2">-1.0</td><td rowspan="2">-1.5</td><td rowspan="2" colspan="2">-0.3</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="2">$\alpha = 45°$</td><td rowspan="2">-1.2</td><td rowspan="2">-1.8</td><td rowspan="2">-1.3</td><td rowspan="2">-1.9</td><td rowspan="2" colspan="2">-0.4</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
<tr><td rowspan="2">$\alpha = 60°$</td><td rowspan="2">-1.35</td><td rowspan="2">-1.9</td><td rowspan="2">-1.25</td><td rowspan="2">-1.9</td><td rowspan="2" colspan="2">-0.5</td><td colspan="2">+0.2</td></tr>
<tr><td colspan="2">-0.2</td></tr>
</table>

Notes

a *e* is defined in Figure 6.14.

b For roofs with parapets or curved eaves, linear interpolation may be used for intermediate values of *h/e* and *r/e*.

c 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.

d In Zone I, where positive and negative values are given, both values should be considered.

e For the mansard eave itself, the external pressure coefficients are given in Table I.3 "External pressure coefficients for duopitch roofs: wind direction 0°", Zone F and G, depending on the pitch angle of the mansard eave.

f 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.

Table I.2 Replacement Table 6.8 External pressure coefficients for monopitch roofs

<table>
<tr><th rowspan="3">Pitch Angle α</th><th colspan="6">Zone for wind direction θ = 0°</th><th colspan="6">Zone for wind direction θ = 180°</th></tr>
<tr><th colspan="2">F</th><th colspan="2">G</th><th colspan="2">H</th><th colspan="2">F</th><th colspan="2">G</th><th colspan="2">H</th></tr>
<tr><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>-1.8</td><td>-2.5</td><td>-1.2</td><td>-2.0</td><td>-0.6</td><td>-1.2</td><td rowspan="2">-2.4</td><td rowspan="2">-2.5</td><td rowspan="2">-1.1</td><td rowspan="2">-2.0</td><td rowspan="2">-0.8</td><td rowspan="2">-1.2</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>-1.1</td><td>-2.0</td><td>-0.8</td><td>-1.5</td><td colspan="2">-0.4</td><td rowspan="2">-2.6</td><td rowspan="2">-2.8</td><td rowspan="2">-1.0</td><td rowspan="2">-2.0</td><td rowspan="2">-0.9</td><td rowspan="2">-1.2</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">-1.7</td><td rowspan="2">-2.3</td><td rowspan="2">-1.0</td><td rowspan="2">-1.5</td><td rowspan="2" colspan="2">-0.9</td></tr>
<tr><td colspan="2">+0.8</td><td colspan="2">+0.5</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">-0.9</td><td rowspan="2">-1.3</td><td rowspan="2" colspan="2">-0.8</td><td rowspan="2" colspan="2">-0.9</td></tr>
<tr><td colspan="2">+0.8</td><td colspan="2">+0.6</td><td colspan="2">+0.7</td></tr>
<tr><td>60°</td><td colspan="2">+0.8</td><td colspan="2">+0.8</td><td colspan="2">+0.8</td><td>-1.0</td><td>-1.0</td><td colspan="2">-0.7</td><td colspan="2">-0.7</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>-1.1</td><td>-1.0</td><td colspan="2">-0.7</td><td colspan="2">-0.7</td></tr>
</table>

<table>
<tr><th rowspan="3">Pitch Angle α</th><th colspan="10">Zone for wind direction θ = 90°</th></tr>
<tr><th colspan="2">F_{up}</th><th colspan="2">F_{low}</th><th colspan="2">G</th><th colspan="2">H</th><th colspan="2">I</th></tr>
<tr><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>5°</td><td>-2.2
+0.0</td><td>-2.6</td><td>-2.1
+0.0</td><td>-2.4</td><td>-1.1
+0.0</td><td>-2.0</td><td>-0.7
+0.0</td><td>-1.2</td><td colspan="2">-0.7
+0.0</td></tr>
<tr><td>15°</td><td>-2.6
+0.2</td><td>-2.9</td><td>-1.6
+0.2</td><td>-2.4</td><td>-1.1
+0.2</td><td>-2.5</td><td>-0.8
+0.2</td><td>-1.2</td><td>-0.8
+0.2</td><td>-1.2</td></tr>
<tr><td>30°</td><td>-1.7
+0.5</td><td>-2.9</td><td>-1.3
+0.5</td><td>-2.0</td><td>-1.2
+0.4</td><td>-2.0</td><td>-1.0
+0.3</td><td>-1.3</td><td>-0.8
+0.2</td><td>-1.2</td></tr>
<tr><td>45°</td><td>-1.5
+0.6</td><td>-2.4</td><td>-1.3
+0.6</td><td>-2.0</td><td>-1.2
+0.5</td><td>-2.0</td><td>-1.0
+0.4</td><td>-1.3</td><td>-0.9
+0.3</td><td>-1.2</td></tr>
<tr><td>60°</td><td>-1.2
+0.7</td><td>-2.0</td><td>-1.2
+0.7</td><td>-2.0</td><td>-1.2
+0.7</td><td>-2.0</td><td>-0.4
+0.5</td><td>-1.3</td><td>-0.2
+0.5</td><td>-1.2</td></tr>
<tr><td>75°</td><td>-1.2
+0.8</td><td>-2.0</td><td>-1.2
0.8</td><td>-2.0</td><td>-1.2
+0.8</td><td>-2.0</td><td>-0.4
+0.7</td><td>-1.3</td><td colspan="2">-0.2
+0.6</td></tr>
</table>

Notes

a At θ = 0° 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.

b 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.

Table I.3 Replacement Table 6.9 External pressure coefficients for duopitch roofs

Pitch Angle α	Zone for wind direction $\theta = 0°$									
	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°	-0.9		-0.8		-0.9		-0.7		-1.1	-1.5
-30°	-1.7	-2.0	-1.0	-1.5	-0.9		-0.7		-0.8	-1.4
-15°	-2.6	-2.8	-1.0	-2.0	-0.9	-1.2	-0.5		-0.7	-1.2
-5°	-2.4	-2.5	-1.2	-2.0	-0.8	-1.2	-0.5		-0.5	
5°	-1.8	-2.5	-1.2	-2.0	-0.6	-1.2	-0.6		-0.9	
	+0.0		+0.0		+0.0					
15°	-1.1	-2.0	-0.8	-1.5	-0.4		-0.5		-1.3	-1.5
	+0.2		+0.2		+0.2					
30°	-0.5	-1.5	-0.5	-1.5	-0.2		-0.5		-0.9	
	+0.8		+0.5		+0.4					
45°	-0.0		-0.0		-0.0		-0.3		-0.4	
	+0.8		+0.6		+0.7					
60°	+0.8		+0.8		+0.8		-0.6		-0.8	
75°	+0.8		+0.8		+0.8		-0.8		-0.9	

Pitch Angle α	Zone for wind direction $\theta = 90°$							
	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°	-1.5	-2.0	-1.3	-2.0	-1.0	-1.3	-0.9	-1.2
-30°	-1.7	-2.1	-1.3	-2.0	-1.0	-1.3	-0.8	-1.2
-15°	-2.6	-2.5	-1.4	-2.0	-0.8	-1.2	-0.8	-1.2
-5°	-2.2	-2.5	-1.5	-2.0	-0.7	-1.2	-0.7	-1.2
5°	-2.0	-2.2	-1.1	-2.0	-0.6	-1.2	-0.5	
15°	-1.6	-2.0	-1.5	-2.0	-0.6	-1.2	-0.4	
30°	-1.2	-1.5	-1.1	-2.0	-0.6	-1.2	-0.5	
45°	-1.2	-1.5	-1.2	-2.0	-0.6	-1.2	-0.4	
60°	-1.2	-1.5	-1.2	-2.0	-0.7	-1.0	-0.6	
75°	-1.2	-1.5	-1.2	-2.0	-1.15	-1.0	-0.6	

Notes

a At $\theta = 0°$ the pressure changes rapidly between positive and negative values on the windward face around a pitch angle of $\alpha = -5°$ 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**.**

b Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. (Do not interpolate between $\alpha = +5°$ and $\alpha = -5°$, but use the data for flat roofs in Table I.1). The values equal to 0.0 are given for interpolation purposes.

Table I.4 Replacement Table 6.10 External pressure coefficients for hipped roofs

<table>
<tr><th rowspan="3">Pitch angle α_0 for θ=0° α_{90} for θ=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>$c_{pe,10}$</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>-1.8</td><td>-2.5</td><td>-1.2</td><td>-2.0</td><td>-0.6</td><td>-1.2</td><td colspan="2" rowspan="2">-0.6</td><td colspan="2" rowspan="2">-0.8</td><td colspan="2" rowspan="2">-0.6</td><td rowspan="2">-1.1</td><td rowspan="2">-2.0</td><td rowspan="2">-0.6</td><td rowspan="2">-1.2</td><td colspan="2" rowspan="2">-0.6</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>-1.3</td><td>-2.0</td><td>-0.8</td><td>-1.5</td><td colspan="2">-0.5</td><td colspan="2" rowspan="2">-0.6</td><td rowspan="2">-1.4</td><td rowspan="2">-1.5</td><td rowspan="2">-1.3</td><td rowspan="2">-2.0</td><td rowspan="2">-0.9</td><td rowspan="2">-2.0</td><td rowspan="2">-0.6</td><td rowspan="2">-1.2</td><td colspan="2" rowspan="2">-0.4</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 colspan="2" rowspan="2">-0.6</td><td rowspan="2">-1.3</td><td rowspan="2">-1.2</td><td colspan="2" rowspan="2">-0.8</td><td rowspan="2">-1.0</td><td rowspan="2">-2.0</td><td rowspan="2">-0.6</td><td rowspan="2">-1.2</td><td colspan="2" rowspan="2">-0.5</td></tr>
<tr><td colspan="2">+0.8</td><td colspan="2">+0.5</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 colspan="2" rowspan="2">-0.4</td><td colspan="2" rowspan="2">-0.7</td><td colspan="2" rowspan="2">-0.4</td><td rowspan="2">-1.1</td><td rowspan="2">-2.0</td><td rowspan="2">-1.15</td><td rowspan="2">-1.2</td><td colspan="2" rowspan="2">-0.4</td></tr>
<tr><td colspan="2">+0.8</td><td colspan="2">+0.6</td><td colspan="2">+0.7</td></tr>
<tr><td>60°</td><td colspan="2">+0.8</td><td colspan="2">+0.8</td><td colspan="2">+0.8</td><td colspan="2">-0.7</td><td colspan="2">-0.6</td><td colspan="2">-0.3</td><td>-1.2</td><td>-2.0</td><td colspan="2">-0.7</td><td colspan="2">-0.6</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">-1.2</td><td colspan="2">-0.6</td><td colspan="2">-0.3</td><td>-1.2</td><td>-2.0</td><td colspan="2">-0.5</td><td colspan="2">-0.6</td></tr>
</table>

Notes

a At $\theta = 0°$ the pressures changes rapidly between positive and negative values on the windward face at pitch angle of $\alpha = +5°$ 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 are allowed.

b 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.

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