EBCS-1

Ethiopian Building Code Standard

BASIS OF DESIGN AND ACTIONS ON STRUCTURES

Ministry of Works & Urban Development
Addis Ababa, Ethiopia
1995

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EBCS - 1
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FOREWORD

The Proclamation to define the powers and duties of the Central and Regional Executive Organs of the Transitional Government of Ethiopia No. 41/1993 empowers the Ministry of Works and Urban Development to prepare the Country's Building Code, issue Standards for design and construction works, and follow up and supervise the implementation of same.

In exercise of these powers and in discharge of its responsibility, the Ministry is issuing a series of **Building Code Standards** of general application.

The purpose of these standards is to serve as nationally recognized documents, the application of which is deemed to ensure compliance of buildings with the minimum requirements for design, construction and quality of materials set down by the **National Building Code.**

The major benefits to be gained in applying these standards are the harmonization of professional practice and the ensuring of appropriate levels of safety, health and economy with due consideration of the objective conditions and needs of the country.

As these standards are technical documents which, by their very nature, require periodic updating, revised editions will be issued by the Ministry from time to time as appropriate.

The Ministry welcomes comments and suggestions on all aspect of the **Ethiopian Building** Code Standards. All feedback received will be carefully reviewed by professional experts in the field of building construction with a view to possible incorporation of amendments in future editions.

Haile Assegidie
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TABLE OF CONTENTS

CHAPTER 1 - BASIS OF DESIGN 1

1.1 INTRODUCTION 1
1.1.1 Scope 1
1.1.2 Assumptions 1
1.1.3 Definitions 2
1.1.4 Symbols 6

1.2 REQUIREMENTS 9
1.2.1 Fundamental Requirements 9
1.2.2 Reliability Differentiation 9
1.2.3 Design Situations 10
1.2.4 Design Working Life 11
1.2.5 Durability 11
1.2.6 Quality Assurance 12

1.3 LIMIT STATES 12
1.3.1 General 12
1.3.2 Ultimate Limit States 12
1.3.3 Serviceability Limit States 12
1.3.4 Limit State Design 13

1.4 ACTIONS AND ENVIRONMENTAL INFLUENCES 13
1.4.2 Characteristic Values of Actions 14
1.4.3 Other Representative Values of Variable and Accidental Actions 15
1.4.4 Environmental Influences 16

1.5 MATERIAL PROPERTIES 16
1.6 GEOMETRICAL DATA 17
1.7 MODELLING FOR STRUCTURAL ANALYSIS AND RESISTANCE 17
1.7.1 General 17
1.7.2 Modelling in the Case of Static Actions 17
1.7.3 Modelling in the Case of Dynamic Actions 18

1.8 DESIGN ASSISTED BY TESTING 18
1.8.1 General 18
1.8.2 Types of Tests 18
1.8.3 Derivation of Design Values 19

1.9 VERIFICATION BY THE PARTIAL SAFETY FACTOR METHOD 20
1.9.1 General 20
1.9.2 Limitations and Simplifications 20
1.9.3 Design Values 21
1.9.3.1 *Design Values of Actions* 21
1.9.3.2 *Design Values of the Effects of Actions* 21
1.9.3.3 *Design Values of Material Properties* 22
1.9.3.4 *Design Values of Geometrical Data* 22

1.9.3.5 *Design Resistance* 22

1.9.4 Ultimate Limit States 23
1.9.4.1 *Verifications of Static Equilibrium and Strength* 23
1.9.4.2 *Combination of Actions* 23
1.9.4.3 *Partial Safety Factors* 25
1.9.4.4 *Ψ Factors* 27
1.9.4.5 *Simplified Verification for Building Structures* 27
1.9.4.6 *Partial Safety Factors for Materials* 27
1.9.5 Serviceability Limit States 28
1.9.5.1 *Verification of Serviceability* 28
1.9.5.3 *Partial Safety Factors* 29
1.9.5.4 *Ψ Factors* 29
1.9.5.5 *Simplified Verification for Building Structures* 29
1.9.5.6 *Partial Safety Factors for Materials* 29

CHAPTER 2 - ACTION ON STRUCTURES-DENSITIES, SELF-WEIGHT AND IMPOSED LOADS **31**

2.1 GENERAL 31
2.1.1 Scope 31

2.2 CLASSIFICATION OF ACTIONS 31
2.2.1 Self-Weight 31
2.2.2 Imposed Loads 32

2.3 DESIGN SITUATIONS 32
2.3.1 General 32
2.3.2 Self-Weight 32
2.3.3 Imposed Loads 32

2.4 DENSITIES OF BUILDING MATERIALS AND STORED MATERIALS 32
2.4.1 Definitions 32
2.4.2 Tables 33

2.5 SELF-WEIGHT OF CONSTRUCTION ELEMENTS 41
2.5.1 Representation of Actions 41
2.5.2 Load Arrangements 42
2.5.3 Self-Weight - Characteristic Values 42
2.5.3.1 *Assessment of Self-Weight* 42

2.6 IMPOSED LOADS ON BUILDINGS 43
2.6.1 Representation of Actions 43
2.6.2 Load Arrangements 44
2.6.2.1 *Horizontal Members* 44
2.6.2.2 *Vertical Members* 44
2.6.3 Imposed Loads - Characteristic Values 44
2.6.3.1 *Residential, Social, Commercial and Administration Area* 44
2.6.3.2 *Garage and Vehicle Traffic Areas* 46
2.6.3.3 *Areas for Storage and Industrial Activities* 48
2.6.3.4 *Roofs* 48

2.6.4 Horizontal Loads on Partition Walls and Barriers due to Persons 49

CHAPTER 3 - WIND ACTIONS **51**

3.2 CLASSIFICATION OF ACTIONS 51
3.3 DESIGN SITUATIONS 51
3.4 REPRESENTATION OF ACTIONS 51
3.4.1 Expansion of the Wind Actions and the Response of the Structures 51
3.4.2 Modelling of Wind Actions 52

3.5 WIND PRESSURE ON SURFACES 52
3.5.1 Field of Application 52
3.5.2 External Pressure 53
3.5.3 Internal Pressure 53
3.5.4 Net Pressure 53

3.6 WIND FORCES 53
3.6.1 Wind Forces from Pressures 53
3.6.2 Friction Force 54

3.7 REFERENCE WIND 55
3.7.1 Reference Wind Pressure 55
3.7.2 Reference Wind Velocity 55
3.7.3 Annual Probabilities of Exceedence other than 0.02 56

3.8 WIND PARAMETER 57
3.8.1 Mean Wind Velocity 57
3.8.2 Roughness Coefficient 57
3.8.3 Terrain Categories 57
3.8.4 Topography Coefficient 58
3.8.5 Exposure Coefficient 60

3.9 CHOICE OF PROCEDURES 61
3.9.1 General 61
3.9.2 Criteria for the Choice 61
3.9.3 Dynamic Coefficient for Gust Wind Response 61
3.9.4 Vortex Shedding, Aeroelastic Instability and Dynamic Interference Effects 67
3.9.4.1 *General* 67
3.9.4.2 *Field of Application* 67

Appendix A - Aerodynamic Coefficients **69**

.A.1 GENERAL 69

A.2 BUILDINGS 69
A.2.1 General 69
A.2.2 Vertical Walls of Rectangular Plan Buildings 70
A.2.3 Flat roofs 71
A.2.4 Monopitch Roofs 73
A.2.5 Duopitch Roofs 75
A.2.6 Hipped Roofs 78

	A.2.7 Multispan Roofs	78
	A.2.8 Vaulted Roofs and Domes	80
A.3	CANOPY ROOFS	84
A.4	FREE-STANDING BOUNDARY WALLS, FENCES AND SIGNBOARDS	88
	A.4.1 Solid Boundary Walls	88
	A.4.2 Pressure Coefficients for Porous Fences	90
	A.4.3 Signboards	90
A.5	STRUCTURAL ELEMENTS WITH RECTANGULAR SECTIONS	91
A.6	STRUCTURAL ELEMENTS WITH SHARP EDGED SECTION	91
A.7	STRUCTURAL ELEMENTS WITH REGULAR POLYGONAL SECTION	94
A.8	CIRCULAR CYLINDERS	95
	A.8.1 External Pressure Coefficients	95
	A.8.2 Force Coefficients	97
A.9	SPHERES	98
A.10	LATTICE STRUCTURES AND SCAFFOLDINGS	99
A.11	FRICTION COEFFICIENTS C_{FR}	100
A.12	EFFECTIVE SLENDERNESS λ AND SLENDERNESS REDUCTION FACTOR ψ_λ	102

CHAPTER 1
BASIS OF DESIGN

1.1 INTRODUCTION

1.1.1 Scope

(1) This Chapter establishes the principles and requirements for safety and serviceability of structures, describes the basis for design and verification and gives guidelines for related aspects of structural reliability.

(2) This Chapter provides the basis and general principles for the structural design of buildings and civil engineering works including geotechnical aspects and shall be used in conjunction with the other parts of EBCS 1. This Chapter relates to all circumstances in which a structure is required to give adequate performance, including fire and seismic events.

(3) This Chapter may also be used as a basis for the design of structures not covered in EBCS 2 to 8 and where other materials or other actions outside the scope of EBCS 1 are involved.

(4) This Chapter is also applicable to structural design for the execution stage and structural design for temporary structures, provided that appropriate adjustments outside the scope of ENV 1991 are made.

(5) This Chapter also gives some simplified methods of verification which are applicable to buildings and other common construction works.

(6) Design procedures and data relevant to the design of bridges and other construction works which are not completely covered in this Chapter may be obtained from other Chapters of EBCS 1 and other relevant Eurocodes.

(7) This Chapter is not directly intended for the structural appraisal of existing construction in developing the design of repairs and alterations or assessing changes of use but may be so used where applicable.

(8) This Chapter does not completely cover the design of special construction works which require unusual reliability considerations, such as nuclear structures, for which specific design procedures should be used.

(9) This Chapter does not completely cover the design of structures where deformations modify direct actions.

1.1.2 Assumptions

The following assumptions apply:

(a) The choice of the structural system and the design of a structure is made by appropriately qualified and experienced personnel.
(b) Execution is carried out by personnel having the appropriate skill and experience.

(c) Adequate supervision and quality control is provided during execution of the work, i.e. in design offices, factories, plants and on site.
(d) The construction materials and products are used as specified in this Code or in ENVs EBCS 2 to 8 or in the relevant supporting material or product specifications.
(e) The structure will be adequately maintained.
(f) The structure will be used in accordance with the design assumptions.
(g) Design procedures are valid only when the requirements for the materials, execution and workmanship given in EBCS 2 to 8 are also complied with.

1.1.3 Definitions

(1) Unless otherwise stated in the following, the terminology used in the International Standard ISO 8930:1987 is adopted.

Note: Most definitions are reproduced from ISO 8930:1987.

(2) The following terms are used in common for EBCS 1 to 8 with the following meaning:

(a) **Construction Works:** Everything that is constructed or results from construction operations. This definition accords with ISO 6707: Part 1. The term covers both building and civil engineering works. It refers to the complete construction works comprising structural, non-structural and geotechnical elements.

(b) **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.

(c) **Type of construction:** Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction, composite steel and concrete construction.

(d) **Method of construction:** Manner in which the execution will be carried out, e.g. cast in place, prefabricated, cantilevered.

(e) **Construction material:** Material used in construction work, e.g. concrete, steel, timber, masonry.

(f) **Structure:** Organized combination of connected parts designed to provide some measure of rigidity. ISO 6707: Part 1 gives the same definition but adds " or a construction works having such an arrangement".

(g) **Form of structure:** The arrangement of structural elements, such as beam, column, arch, foundation piles. Forms of structure are, for example, frames, suspension bridges.

(h) **Structural system:** The load-bearing elements of a building or civil engineering works and the way in which these elements function together.

(i) **Structural model:** The idealization of the structural system used for the purposes of analysis and design.

(j) **Execution:** The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

(3) Special terms relating to design in general are:

(a) **Design criteria:** The quantitative formulations which describe for each limit state the conditions to be fulfilled.

(b) **Design situations:** Those sets of physical conditions representing a certain time interval for which the design will demonstrate that relevant limit states are not exceeded.

(c) **Transient design situation:** Design situation which is relevant during a period much shorter that the design working life of the structure and which as a high probability of occurrence. It refers to temporary conditions of the structure, of use, or exposure, e.g. during construction or repair.

(d) **Persistent design situation:** Design situation which 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.

(e) **Accidental design situation:** Design situation involving exceptional conditions of the structure or its exposure, e.g. fire, explosion, impact or local failure.

(f) **Design working life:** The assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without substantial repair being necessary.

(g) **Hazard:** Exceptionally unusual and severe event, e.g. an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation form intended dimensions.

(h) **Load arrangement:** Identification of the position, magnitude and direction of a free action.

(i) **Load case:** Compatible load arrangements, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for a particular verification.

(j) **Limit states:** States associated with collapse, or with other similar forms of structural failure. They generally correspond to the maximum load-carrying resistance of a structure or structural part.

(k) **Ultimate limit states:** States associated with collapse, or with other similar forms of structural failure. They generally correspond to the maximum load-carrying resistance of a structure or structural part.

(l) **Serviceability limit states:** States which correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met.

(m) **Irreversible serviceability limit states:** Limit states which will remain permanently exceeded when the responsible actions are removed.

(n) **Reversible serviceability limit states:** Limit states which will not be exceeded when the responsible actions are removed.

(o) **Resistance:** Mechanical property of a component, a cross-section, or a member of a structure, e.g. bending resistance, buckling resistance.

(p) **Maintenance:** The total set of activities performed during the working life of the structure to preserve its function.

(q) **Strength:** Mechanical property of a material, usually given in units of stress.

(r) **Reliability:** Reliability covers safety, serviceability and durability of a structure.

(4) Terms relating to actions are

(a) **Action:**

(i) Force (load) applied to the structure (direct action)

(ii) An imposed or constrained deformation or an imposed acceleration caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).

(b) **Action effect:** The effect of actions on structural members, e.g.. internal force, moment, stress, strain.

(c) **Permanent action *(G)*:** Action which is likely to act throughout a given design situation and for which the variation in magnitude with time is negligible in relation to the mean value, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value.

(d) **Variable action *(Q)*:** Action, which is unlikely to act throughout a given design situation or for which the variation in magnitude with time is neither negligible in relation to the mean value nor monotonic.

(e) **Accidental action *(A)*** Action, usually of short duration, which is unlikely to occur with a significant magnitude over the period of time under consideration during the design working life. An accidental action can be expected in many cases to cause severe consequences unless special measures are taken.

(f) **Seismic action (A_E):** Action which arises due to earthquake ground motions.

(g) **Fixed action:** Action which may has a fixed distribution over the structure such that the magnitude and direction of the action are determined unambiguously for the whole structure if this magnitude and direction are determined at one point on the structure.

(h) **Free action:** Action which may have any spatial distribution over the structure within given limits.

(i) **Single action:** Action that can be assumed to be statistically independent in time and space of any other action acting on the structure.

(j) **Static action:** Action which does not cause significant acceleration of the structure or structural members.

(k) **Dynamic action:** Action which causes significant acceleration of the structure or structural members.

(l) **Quasi-static action:** Dynamic action that can be described by static models in which the dynamic effects are included.

(m) **Representative value of an action:** Value used for the verification of a limit state.

(n) **Characteristic value of an action:** The principal representative value of an action. In so far as this 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.

(o) **Reference period:** See (n) above.

(p) **Combination values:** Values associated with the use of combinations of actions (see (t) below to take account of a reduced probability of simultaneous occurrence of the most unfavourable values of several independent actions.

(q) **Frequent value of a variable action:** The value determined such that:

(i) the total time, within a chosen period of time, during which it is exceeded for a specified part, or
(ii) the frequency with which it is exceeded,

is limited to a given value.

(r) **Quasi-permanent value of a variable action:** The value determined such that the total time, within a chosen period of time, during which it is exceeded is a considerable part of the chosen period of time.

(s) **Design value of an action F_d:** The value obtained by multiplying the representative value by the partial safety factor γ_F.

(t) **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.

(5) Terms relating to material properties

(a) **Characteristic value X_k:** The value of a material 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. A nominal value is used as the characteristic value in some circumstances.

(b) **Design value of a material property X_d:** Value obtained by dividing the characteristic value by a partial factor γ_M or, in special circumstances, by direct determination.

(6) Terms relating to geometrical data are:

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

(b) **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.

1.1.4 Symbols

(1) For the purposes of this Code, the following symbols are used. The notation used is based on ISO 3898:1987.

A	accidental action
A	area
A	loaded area
A_d	design value of an accidental action
A_{Ed}	design value of seismic action
A_{EK}	characteristic seismic action
A_{fr}	area swept by the wind
A_K	characteristic value of an accidental action
A_{ref}	reference area
C_d	nominal value, or a function of certain design properties of materials
E	effect of an action
E_d	design value of effects of actions
$E_{d,dst}$	design effect of destabilizing action
$E_{d,stb}$	design effect of stabilizing actions
F	action
F_d	design value of an action
F_{fr}	resultant friction force
F_k	characteristic value of an action
F_{rep}	representative value of an action
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_{ind}	indirect permanent action
G_{kj}	characteristic value of permanent action j
$G_{k,inf}$	lower characteristic value of a permanent action
G_k	characteristic value of a permanent action
$G_{k,sup}$	upper characteristic value of a permanent action
H	height of a topographic feature
I	importance factor
I_v	turbulence intensity
K_l	shape parameter
L_e	effective length of an upwind slope
L_u	actual length of an upwind slope
P	prestressing action
P_d	design value of a prestressing action
P_k	characteristic value of a prestressing action
Q	variable, action
Q_d	design value of a variable action
Q_{ind}	indirect variable action

Q_{ki}	characteristic value of the non-dominant variable action i
Q_k	characteristic value of a single variable action
Q_k	concentrated load
Q_{k1}	characteristic value of the dominant variable action
R	resistance
R_d	design value of the resistance
Re	reynolds number
R_k	characteristic resistance
X	material property
X_d	design value of a material property
X_k	characteristic value of a material property
a_d	design value of geometrical data
a_{nom}	nominal value of geometrical data
a_k	characteristic dimension
b	width of the structure
c_{ALT}	altitude factor
c_d	dynamic coefficient
c_{DIR}	direction factor
c_e	exposure coefficient
c_f	force coefficient
c_{f0}	force coefficient of structures or structural elements with infinite slenderness ratio
c_p	pressure coefficient
c_r	roughness coefficient
c_t	topography coefficient
c_{TEM}	temporary factor
d	depth of the structure, diameter
e	eccentricity of a force or edge distance
g	peak factor
g_k	weight per unit area, or weight per unit length
h	height of the structure
k	equivalent roughness
k_T	terrain factor
l	length of a horizontal structure
n	exponent
p	annual probability of exceedence
q_k	uniformly distributed load, or line load
q_{ref}	reference mean velocity pressure
r	radius
s	factor
t	plate thickness
v_m	mean wind velocity
v_{ref}	reference wind velocity
w	wind pressure
x	horizontal distance of the site from the top of a crest
z	height above ground
z_e, z_i	reference height for local and internal pressure
z_o	roughness length
z_{min}	minimum height

Indices

e	external, exposure
fr	friction
i	internal, mode number
j	current number of incremental area or point of a structure
m	mean
ref	reference
z	vertical direction

Upper case Greek letters

θ	torsional angle
Φ	upwind slope
Φ_B	obstruction factor

Lower case Greek letters

$\alpha_{\alpha n}$	reduction coefficients
γ	bulk weight desity
γ	partial safety factor (safety or serviceability)
γ_A	partial safety factor for accidental actions
γ_F	partial safety factor for actions, also accounting for model uncertainties and dimensional variations
γ_{GA}	as γ_G but for accidental design situations
γ_G	partial safety factor for permanent action
γ_{GAj}	as γ_{Gj} but for accidental design situations
$\gamma_{G,inf}$	partial safety factor for permanent actions in calculating lower design values
γ_{Gj}	partial safety factor for permanent action j
$\gamma_{G,sup}$	partial safety factor for permanent actions in calculating upper design values
γ_M	partial safety factor for a material property, also accounting for model uncertainties and dimensional variations
γ_m	partial safety factor for a material property
γ_P	partial safety factor for prestressing actions
γ_{PA}	as γ_p but for accidental design situations
γ_Q	partial safety factor for variable actions
γ_{Qi}	partial safety factor for variable action i
γ_{rd}	partial safety factor associated with the uncertainty of the resistance model and the dimensional variations
γ_R	partial safety factor for the resistance, including uncertainties in material properties, model uncertainties and dimensional variations
γ_{Rd}	partial safety factor associated with the uncertainty of the resistance model
γ_{sd}	partial safety factor associated with the uncertainty of the action and/or action effect model
δ_a	change made to nominal geometrical data for particular design purpose, e.g. assessment of effective imperfection
η	conversion factor
λ	slenderness ratio
ν	expected frequency, Poisson ratio, kinematic viscosity
ξ	reduction factor
ρ	air density
ϕ	angle of repose

Ψ_{λ}	reduction factor of force coefficient for structural
ψ_o	coefficient for combination value of a variable action
ψ_2	coefficient for quasi-permanent value of a variable action
ψ_r	reduction factor of force coefficient for square sections with rounded corners
ψ_1	coefficient for frequent value of a variable action
φ	solidity ratio

1.2 REQUIREMENTS

1.2.1 Fundamental Requirements

(1) A structure shall be designed and executed in such a way that it will, during its intended life with appropriate degrees of reliability and in an economic way:

(a) remain fit for the use for which it is required; and
(b) sustain all actions and influences likely to occur during execution and use.

(2) Design according to (1) above implies that due regard is given to structural safety and serviceability, including durability, in both cases.

(3) A structure shall also be designed and executed in such a way that it will not be damaged by events like fire, explosion, impact or consequences of human errors, to an extent disproportionate to the original cause.

(4) The potential damage shall be avoided or limited by appropriate choice of one or more of the following:

(a) avoiding, eliminating or reducing the hazards which the structure may sustain;
(b) selecting a structural form which has low sensitivity to the hazards considered;
(c) selecting a structural form and design that can survive adequately the accidental removal of an individual element or a limited part of the structure, or the occurrence of acceptable localized damage;
(d) avoiding as far as possible structural systems which may collapse without warning;
(e) tying the structure together.

(5) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing, and by specifying control procedures for design, production, execution and use relevant to the particular project.

1.2.2 Reliability Differentiation

(1) The reliability required for the majority of structures shall be obtained by design and execution according to EBCS 1 to 8, and appropriate quality assurance measures.

(2) A different level of reliability may be generally adopted:

(a) for structural safety;
(b) for serviceability;

(3) A different level of reliability may depend on:

(a) the cause and mode of failure;
(b) the possible consequences of failure in terms or risk to life, injury, potential economic losses and the level of social inconvenience;
(c) the expense and procedures necessary to reduce the risk of failure;
(d) different degrees of reliability required at national, regional or local level.

(4) Differentiation of the required levels of reliability in relation to structural safety and serviceability may be obtained by the classification of whole structures or by the classification of structural components.

(5) The required reliability relating to structural safety or serviceability may be achieved by suitable combinations of the following measures:

(a) Measures relating to design:

- Serviceability requirements;
- representative values of actions;
- the choice of partial factors or appropriate quantities in design calculations;
- consideration of durability;
- consideration of the degree of robustness (structural integrity);
- the amount and quality of preliminary investigations of soils and possible environmental influences;
- the accuracy of the mechanical models used;
- the stringency of the detailing rules.

(b) Measures relating to quality assurance to reduce the risk of hazards in:

- gross human errors;
- design;
- execution.

(6) Within individual reliability levels, the procedures to reduce risks associated with various potential causes of failure may, in certain circumstances, be interchanged to a limited extent. An increase of effort within one type of measure may be considered to compensate for a reduction of effort within another type.

1.2.3 Design Situations

(1) The circumstances in which the structure may be required to fulfil its function shall be considered and the relevant design situations selected. The selected design situations shall be sufficiently severe and so varied as to encompass all conditions which can reasonably be foreseen to occur during the execution and use of the structure.

(2) Design situations are classified as follows:

(a) persistent situations, which refer to the conditions of normal use;
(b) transient situations, which refer to temporary conditions applicable to the structure, e.g. during execution or repair;
(c) accidental situations, which refer to exceptional conditions applicable to the structure or to its exposure, e.g. to fire, explosion, impact;
(d) seismic situations, which refer to exceptional conditions applicable to the structure when subjected to seismic events.

(3) Information for specific situations for each class is given in other Parts of EBCS 1 to 8.

1.2.4 Design Working Life

(1) The design working life is the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.

(2) An indication of the required design working life is given in Table 1.1.

Table 1.1 Design Working Life Classification

Class	Required Design Working life (years)	Examples
1	1-5	Temporary structure
2	25	Replaceable structural parts, e.g gantry girders, bearings
3	50	Building structures and other common structures
4	100	Monumental building structures, bridges, and other civil engineering structures

1.2.5 Durability

(1) It is an assumption in design that the durability of a structure or part of it in its environment is such that it remains fit for use during the design working life given appropriate maintenance.

(2) The structure should be designed in such a way that deterioration should not impair the durability and performance of the structure having due regard to the anticipated level of maintenance.

(3) The following interrelated factors shall be considered to ensure an adequately durable structure:

(a) the intended and possible future use of the structure;
(b) the required performance criteria;
(c) the expected environmental influences;
(d) the composition, properties and performance of the materials;
(e) the choice of the structural system;
(f) the shape of members and the structural detailing;
(g) the quality of workmanship, and level of control;
(h) the particular protective measures;
(i) the maintenance during the intended life.

(4) The relevant EBCS 2 to 8 specify the appropriate measures.

(5) The environmental conditions shall be appraised at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials and products.

(6) The degree of deterioration may be estimated on the basis of calculations, experimental investigation, experience from earlier constructions, or a combination of these considerations.

1.2.6 Quality Assurance

(1) It is assumed that appropriate quality assurance measures are taken in order to provide a structure which corresponds to the requirements and to the assumptions made in the design. These measures comprise definition of the reliability requirements, organizational measures and controls at the stages of design, execution, use and maintenance.

1.3 LIMIT STATES

1.3.1 General

(1) Limit states are states beyond which the structure no longer satisfies the design performance requirements.

(2) In general, a distinction is made between ultimate limit states and serviceability limit states. Verification of one of the two limit states may be omitted if sufficient information is available to prove that the requirements of one limit state are met by the other.

(3) Limit states may relate to persistent, transient or accidental design situations.

1.3.2 Ultimate Limit States

(1) Ultimate limit states are those associated with collapse or with other similar forms of structural failure.

(2) States prior to structural collapse, which, for simplicity, are considered in place of the collapse itself are also treated as ultimate limit states.

(3) Ultimate limit states concern:

- (a) the safety of the structure and it contents;
- (b) the safety of people.

(4) Ultimate limit states which may require consideration include:

- (a) loss of equilibrium of the structure or any part of it, considered as a rigid body;
- (b) 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;
- (c) failure caused by fatigue or other time-dependent effects.

1.3.3 Serviceability Limit States

(1) Serviceability limit states correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met.

(2) The serviceability requirements concern:

(a) the functioning of the construction works or parts of them;
(b) the comfort of people;
(c) the appearance.

(3) A distinction shall be made, if relevant, between reversible and irreversible serviceability limit states.

(4) Unless specified otherwise, the serviceability requirements should be determined in contracts and/or in the design.

(5) Serviceability limit states which may require consideration include:

(a) deformations and displacements which affect the appearance or effective use of the structure (including the functioning of machines or services) or cause damage to finishes or non-structural elements;
(b) vibrations which cause discomfort to people, damage to the structure or to the materials it supports, or which limit its functional effectiveness;.
(c) damage (including cracking) which is likely to affect appearance, durability or the function of the structure adversely;
(d) Observable damage caused by fatigue and other time-dependent effects.

1.3.4 Limit State Design

(1) Limit state design shall be carried out by:

(a) setting up structural and load models for relevant ultimate and serviceability limit states to be considered in the various design situations and load cases;
(b) verifying that the limit states are not exceeded when design values for actions, material properties and geometrical data are used in the models.

(2) Design values are generally obtained by using the characteristic or representative values in combination with partial and other factors as defined in EBCS 1 to 8.

(3) In exceptional cases, it may be appropriate to determine design values directly. The values should be chosen cautiously and should correspond to at least the same degree of reliability for the various limit states as implied in the partial factors in this Code.

1.4 ACTIONS AND ENVIRONMENTAL INFLUENCES

1.4.1 Principal Classifications

(1) An action (F) is:

(a) a direct action, i.e. force (load) applied to the structure; or
(b) an indirect action, i.e. an imposed or constrained deformation or an imposed acceleration caused, for example by temperature changes, moisture variation, uneven settlement or earthquakes.

(2) Actions are classified:

(a) by their variation in time:

(i) permanent actions (*G*), e.g. self-weight of structures, fixed equipment and road surfacings;
(ii) variable actions (*Q*), e.g. imposed loads, wind loads or snow loads;
(iii) accidental actions (*A*), e.g. explosions, or impact from vehicles.

(b) by their spatial variation:

(i) fixed actions, e.g. self-weight;
(ii) free actions, e.g. movable imposed loads, wind loads, snow loads.

(c) by their nature and/or the structural response:

(i) static actions, which do not cause significant acceleration of the structure or structural member;
(ii) dynamic actions, which cause significant acceleration of the structure or structural member.

(3) In many cases, dynamic effects of actions may be calculated from quasi-static actions by increasing the magnitude of the static actions or by the introduction of an equivalent static action.

(4) Some actions, for example seismic actions and snow loads, can be considered as either accidental and/or variable actions, depending on the site location (see other Parts of ENV 1991).

(5) Prestressing (*P*) is a permanent action. Detailed information is given in EBCS 2,3 and 4.

(6) Indirect actions are either permanent $G_{ind.}$ (e.g. settlement of support), or variable $Q_{ind.}$ (e.g. temperature effect). and should be treated accordingly.

(7) An action is described by a model, its magnitude being represented in the most common cases by one scalar which may take on several representative values. For some actions (multi-component actions) and some verifications (e.g. for static equilibrium) the magnitude is represented by several values. For fatigue verifications and dynamic analysis a more complex representation of the magnitudes of some actions may be necessary.

1.4.2 Characteristic Values of Actions

(1) The characteristic value of an action is its main representative value.

(2) Characteristic value of actions F_K shall be specified:

(a) in the relevant parts of ENV 1991, as a mean value, an upper or lower value, or a nominal value (which does not refer to a known statististical distribution);
(b) in the design, provided that the provisions, specified in EBCS 1 are observed.

Note: The provisions may be specified by the relevant competent authority.

(3) The characteristic value of a permanent action shall be determined as follows:

(a) if the variability of *G* is small, one single value G_K may be used;

(b) if the variability of G is not small, two values have to be used; an upper value $G_{K,sup}$ and a lower value $G_{K,inf}$.

(4) In most cases the variability of G can be assumed to be small if G does not vary significantly during the design working life of the structure and its coefficient of variation is not greater than 0.1. However in such cases when the structure is very sensitive to variations in G (e.g. some types of prestressed concrete structures), two values have to be used even if the coefficient of variation is small.

(5) The following may be assumed in most cases:

(a) G_k is the mean value
(b) G_{kinf} is the 0.5 fractile, and G_{ksup} is the 0.95 fractile of the statistical; distribution for G which may be assumed to be Gaussian.

(6) The self-weight of the structure can, in most cases, be represented by a single characteristic value and be calculated on the basis of the nominal dimensions and mean unit masses. The values are given in Chapter 2.

(7) For variable actions the characteristic value (Q_k) corresponds to either.

(a) an upper value with an intended probability of not being exceeded or a lower value with an intended probability of not falling below, during some reference period;
(b) a nominal value which may be specified in cases where a statistical distribution is not known.

Values are given in Chapter 2 and 3

(8) The following may be assumed for the time-varying part for most cases of characteristic values of variable actions:

(a) the intended probability is 0.98;
(b) the reference period is one year.

However in some cases the character of the action makes another reference period more appropriate. In addition, design values for other variables within the action model may have to be chosen, which may influence the probability of being exceeded for the resulting total action.

(9) Actions caused by water should normally be based on water levels and include a geometrical parameter to allow for fluctuation of water level. Tides, currents and waves should be taken into account where relevant.

(10) For accidental actions the representative value is generally a characteristic value A_k corresponding to a specified value.

(11) Values of A_{Ed} for seismic actions are given in EBCS 8.

(12) For multi-component actions (see Section 1.4.1 (7) the characteristic action is represented by groups of values, to be considered alternatively in design calculations.

1.4.3 Other Representative Values of Variable and Accidental Actions

(1) In the most common cases the other representative values of a variable action are:

(a) the combination value generally represented as a product: $\Psi_o\ Q_k$;
(b) the frequent value generally represented as a product: $\Psi_1\ Q_k$;
(c) the quasi-permanent value generally represented as a product: $\Psi_2\ Q_k$.

(2) Combination values are associated with the use of combinations of actions, to take account of a reduced probability of simultaneous occurrence of the most unfavourable values of several independent actions.

(3) The frequent value is determined such that:

(a) the total time, within a chosen period of time, during which it is exceeded for a specified part, or
(b) the frequency with which it is exceeded,

is limited to a given value.

(4) The part of the chosen period of time or the frequency, mentioned in (3) above should be chosen with due regard to the type of construction works considered and the purpose of the calculations. Unless other values are specified the part may be chosen to be 0.05 or the frequency to be 300 per year for ordinary buildings.

(5) The quasi-permanent value is so determined that the total time, within a chosen period of time, during which it is exceeded is a considerable part of the chosen period of time.

(6) The part of the chosen period of time, mentioned in (5) above, may be chosen to be 0.5. The quasi-permanent value may also be determined as the value averaged over the chosen period of time.

(7) These representative values and the characteristic value are used to define the design values of the actions and the combinations of actions as explained in section 9. The combination values are used for the verification of ultimate limit states and irreversible serviceability limit states. The frequent values and quasi-permanent values are used for the verification of ultimate limit states involving accidental actions and for the verification of reversible serviceability limit states. The quasi-permanent values are also used for the calculation of long term effects of serviceability limit states. More detailed rules concerning the use of representative values are given, for example, in EBCS 2 to 8

(8) For some structures or some actions other representative values or other types of description of actions may be required, e.g. the fatigue load and the number of cycles when fatigue is considered.

1.4.4 Environmental Influences

The environmental influences which may affect the durability of the structure shall be considered in the choice of structural materials, their specification, the structural concept and detailed design. The EBCS 2 to 8 specify the relevant measures.

1.5 MATERIAL PROPERTIES

(1) Properties of materials (including soil and rock) or products are represented by characteristic values which correspond to the value of the property having a prescribed probability of not being attained in a hypothetical unlimited test series. They generally correspond for a particular property to a specified fractile of the assumed statistical distribution of the property of the material in the structure.

(2) Unless otherwise stated in EBCS 2 to 8, the characteristic values should be defined as the 5% fractile for strength parameters and as the mean value for stiffness parameters.

(3) Material property values shall normally be determined from standardized tests performed under specified conditions. A conversion factor shall be applied where it is necessary to convert the test results into values which can be assumed to represent the behaviour of the material in the structure or the ground (see also EBCS 2 to 8).

(4) A material strength may have two characteristic values, an upper and a lower. In most cases only the lower value will need to be considered. In some cases, different values may be adopted depending on the type of problem considered. Where an upper estimate of strength is required (e.g. for the tensile strength of concrete for the calculation of the effects of indirect actions) a nominal upper value of the strength should normally be taken into account.

(5) Where there is a lack of information on the statistical distribution of the property a nominal value may be used; where the limit state equation is not significantly sensitive to its variability a mean value may be considered as the characteristic value.

(6) Values of material properties are given in EBCS 2 to 8.

1.6 GEOMETRICAL DATA

(1) Geometrical data are represented by their characteristic values, or in the case of imperfections directly by their design values.

(2) The characteristic values usually correspond to dimensions specified in the design.

(3) Where relevant, values of geometrical quantities may correspond to some prescribed fractile of the statistical distribution.

(4) Tolerances for connected parts which are made from different materials shall be mutually compatible. Imperfections which have to be taken into account in the design of structural members are given in EBCS 2 to 8

1.7 MODELLING FOR STRUCTURAL ANALYSIS AND RESISTANCE

1.7.1 General

(1) Calculations shall be performed using appropriate design models involving relevant variables. The models shall be appropriate for predicting the structural behaviour and the limit states considered.

(2) Design models should normally be based on established engineering theory and practice, verified experimentally if necessary.

1.7.2 Modelling in the Case of Static Actions

(1) The modelling for static actions should normally be based on an appropriate choice of the force - deformation relationships of the members and their connections.

(2) Effects of displacements and deformations should be considered in the context of ultimate limit state verifications (including static equilibrium) if they result in an increase of the effects of actions by more than 10%.

(3) In general the structural analysis models for serviceability limit states and fatigue may be linear.

1.7.3 Modelling in the Case of Dynamic Actions

(1) When dynamic actions may be considered as quasi-static, the dynamic parts are considered either by including them in the static values or by applying equivalent dynamic amplification factors to the static actions. For some equivalent dynamic amplification factors, the natural frequencies have to be determined.

(2) In some cases (e.g. for cross wind vibrations or seismic actions) the actions may be defined by provisions for a modal analysis based on a linear material and geometric behaviour. For regular structures, where only the fundamental mode is relevant, an explicit modal analysis may be substituted by an analysis with equivalent static actions, depending on mode shape, natural frequency and damping.

(3) In some cases the dynamic actions may be expressed in terms of time histories or in the frequency domain, for which the structural response may be determined by appropriate methods. When dynamic actions may cause vibrations that may infringe serviceability limit states guidance for assessing these limit states is given in annex C, together with the models of some actions.

1.8 DESIGN ASSISTED BY TESTING

1.8.1 General

(1) Where calculation rules or material properties given in EBCS 2 TO 8 are not sufficient or where economy may result from tests on prototypes, part of the design procedure may be performed on the basis of tests. Some of the clauses in this section may also be helpful in cases where the performance of an existing structure is to be investigated.

(2) Tests shall be set up and evaluated in such a way that the structure has the same level of reliability with respect to all possible limit states and design situations as achieved by design based on calculation procedures specified in EBCS 2 to 8.

(3) Sampling of test specimens and conditions during testing should be representative.

(4) Where EBCS 2 to 8 include implicit safety provisions related to comparable situations, these provisions shall be taken into account in assessing the test results and may give rise to corrections. An example is the effect of tensile strength in the bending resistance of concrete beams, which is normally neglected during design.

1.8.2 Types of Tests

(1) The following test types are distinguished:

- (a) tests to establish directly the ultimate resistance or serviceability properties of structural parts e.g. fire tests;
- (b) tests to obtain specific material properties, e.g. ground testing in situ or in the laboratory, testing of new materials;
- (c) tests to reduce uncertainties in parameters in load or resistance models, e.g. wind tunnel testing, testing of full size prototypes, testing of scale models;
- (d) control tests to check the quality of the delivered products or the consistency of the production characteristics, e.g. concrete cube testing;

(e) tests during execution in order to take account of actual conditions experienced e.g. post-tensioning, soil conditions;
(f) control tests to check the behaviour of the actual structure or structural elements after completion, e.g. proof loading for the ultimate or serviceability limit states.

(2) For test types (a), (b) and (c), the test results may be available at the time of design; in those cases the design values can be derived from the tests. For test types (d),(e) and (f) the test results may not be available at the time of design; in these cases the design values correspond to that part of the production that is expected to meet the acceptance criteria at a later stage.

1.8.3 Derivation of Design Values

(1) The derivation of the design values for a material property, a model parameter or a resistance value from tests can be performed in either of the following two ways:

(a) by assessing a characteristic value, which is divided by a partial safety factor and possibly multiplied by an explicit conversion factor;
(b) by direct determination of the design value, implicitly or explicitly accounting for the conversion aspects and the total reliability required..

(2) In general method (a) above should be used. The derivation of a characteristic value from tests should be performed taking account of:

(a) the scatter of test data;
(b) statistical uncertainty resulting from a limited number of tests;
(c) implicit or explicit conversion factors resulting from influences not sufficiently covered by the tests such as:

  (i) time and duration effects, not taken care of in the tests;
  (ii) scale, volumes and length effects;
  (iii) deviating environmental, loading and boundary conditions;
  (iv) the way that safety factors as partial factors or additive elements are applied to get design values.

The partial safety factor used in method (a) above should be chosen in such a way that there is sufficient similarity between the tests under consideration and the usual application field of the partial safety factor used in numerical verifications. (see also Section 1.3.4).

(3) When for special cases method (b) above is used, the determination of the design values should be carried out by considering:

(a) the relevant limit states;
(b) the required level of reliability;
(c) the statistical and model uncertainties;
(d) the compatibility with the assumptions for the action side;
(e) the classification of design working life of the relevant structure according to Section 2;
(f) prior knowledge from similar cases or calculations.

(4) Further information may be found in EBCS 2 to 8.

1.9 VERIFICATION BY THE PARTIAL SAFETY FACTOR METHOD

1.9.1 General

(1) In EBCS 2 to 8 the reliability according to the limit state concept is achieved by application of the partial factor safety method. In the partial safety factor method, it is verified that, in all relevant design situations, the limit states are not exceeded when design values for actions, material properties and geometrical data are used in the design models.

(2) In particular, it shall be verified that:

(a) the effects of design actions do not exceed the design resistance of the structure at the ultimate limit state; and
(b) the effects of design actions do not exceed the performance criteria for the serviceability limit state.

Other verifications may also need to be considered for particular structures e.g. fatigue. Details are presented in EBCS 2 to 8.

(3) The selected design situations shall be considered and critical load cases identified. For each critical load case, the design values of the effects of actions in combination shall be determined.

(4) A load case identifies compatible load arrangements, sets of deformations and imperfections which should be considered simultaneously for a particular verification.

(5) Rules for the combination of independent actions in design situations are given in this section. Actions which cannot occur simultaneously, for example, due to physical reasons, should not be considered together in combination.

(6) A load arrangement identifies the position, magnitude and direction of a free action. Rules for different arrangements within a single action are given in Chapters 2 and 3.

(7) Possible deviations from the assumed directions or positions of actions should be considered.

(8) The design values used for different limit states may be different and are specified in this section.

1.9.2 Limitations and Simplifications

(1) Application rules in Chapter 1 are limited to ultimate and serviceability limit states for structures subject to static loading. This includes cases where the dynamic effects are assessed using equivalent quasi-static loads and dynamic amplification factors, e.g. wind.

(2) Simplified verification based on the limit state concept may be used:

(a) by considering only limit states and load combinations which from experience or special criteria are known to be potentially critical for the design;
(b) by using the simplified verification for ultimate limit states and/or serviceability limit states as specified for buildings in Sections 1.9.4.5 and 1.9.5.5;
(c) by specifying particular detailing rules and/or provisions to meet the safety and serviceability requirements without calculation.

1.9.3 Design Values

1.9.3.1 *Design Values of Actions*

(1) The design value F_d of an action is expressed in general terms as:

$$F_d = \gamma_F F_{rep} \tag{1.1}$$

where γ_F is the partial safety factor for the action considered taking account of:

(a) the possibility of unfavourable deviations of the actions;
(b) the possibility of inaccurate modelling of the actions;
(c) uncertainties in the assessment of effects of actions.

F_{rep} is the representative value of the action.

(2) Depending on the type of verification and combination procedures, design values for particular actions are expressed as follows:

$$\begin{aligned} G_d &= \gamma_G G_k \text{ or } G_k \\ Q_d &= \gamma Q_k,\ \gamma Q \Psi_o Q_k,\ \Psi_1 Q_k \text{ or } Q_k \\ A_d &= \gamma_A A_k \text{ or } A_d \\ P_d &= \gamma_p P_k \text{ or } P_k \\ A_{Ed} &= A_{Ed} \end{aligned} \tag{1.2}$$

(3) Where distinction has to be made between favourable and unfavourable effects of permanent actions, two different partial saftey factors shall be used.

(4) For seismic actions the design value may depend on the structural behaviour characteristics (see EBCS 8).

1.9.3.2 *Design Values of the Effects of Actions*

(1) The effects of actions (E) are responses (for example internal forces and moments, stresses, strains and displacements) of the structure to the actions. For a specific load case the design value of the effect of actions (E_d) is determined from the design values of the actions, geometrical data and material properties when relevant:

$$E_d = E(F_{d1}, F_{d2}, \ldots a_{d1}, a_{d2}, \ldots X_{d1}, X_{d2}, \ldots) \tag{1.3}$$

where $F_{d1}, \ldots, a_{d1}, \ldots$ and $X_{d1}, \ldots$ are chosen according to Sections 1.9.3.1, 1.9.3.3 and 1.9.3.4, respectively.

(2) In some cases, in particular for non-linear analysis, the effect of the uncertainties in the models used in the calculations should be considered explicitly. This may lead to the application of a coefficient of model uncertainty γ_{sd} applied either to the actions or to the action effects, whichever is the more conservative. The factor γ_{sd} may refer to uncertainties in the action model and/or the action effect model.

(3) For non-linear analysis, i.e. when the effect is not proportional to the action, the following simplified rules may be considered in the case of a single predominant action.

(a) When the effect increases more than the action, the partial safety factor is applied to the representative value of the action.
(b) When the effect increases less than the action, the partial safety factor is applied to the action effect of the representative value of the action.

In other cases more refined methods are necessary which are defined in the relevant Codes (e.g. for prestressed structures).

1.9.3.3 *Design Values of Material Properties*

(1) The design value X_d of a material or product property is generally defined as:

$$X_d = \eta X_k/\gamma_M \text{ or } X_d = X_k/\gamma_M \tag{1.4}$$

where γ_M is the partial saftey factor for the material or product property, given in EBCS 2 to 8 which covers
(a) unfavourable deviations from the characteristic values;
(b) inaccuracies in the conversion factors; and
(c) uncertainties in the geometric properties and the resistance model.

η is the conversion factor taking into account the effect of the duration of the load, volume and scale effects, effects of moisture and temperature and so on.

In some cases the conversion is implicitly taken into account by the characteristic value itself, as indicated by the definition of η, or by η_M.

1.9.3.4 *Design Values of Geometrical Data*

(1) Design values of geometrical data are generally represented by the nominal values:

$$a_d = a_{nom} \tag{1.5}$$

Where necessary EBCS 2 to 8 may give further specifications.

(2) In some cases when deviations in the geometrical data have a significant effect on the reliability of a structure, the geometrical design values are defined by:

$$a_d = a_{nom} + \Delta_a \tag{1.6}$$

where Δ_a takes account of the possibility of unfavourable deviations from the characteristic values
Δ_a is only introduced where the influence of deviations is critical, e.g. imperfections in buckling analysis. Values of Δ_a are given in EBCS 2 to 8.

1.9.3.5 *Design Resistance*

(1) Design values for the material properties, geometrical data and effects of actions, when relevant, shall be used to determine the design resistance R_d from:

$$R_d = R(a_{d1}, a_{d2}, \ldots X_{d1}, X_{d2}, \ldots) \tag{1.7}$$

where a_{d1}, ... is defined in Section 1.9.3.4 and X_{d1}, ... in Section 1.9.3.3.

(2) Operational verification formulae, based on the principle of expression (1.7), may have one of the following forms:

$$R_d = R\{X_k / \gamma_M, a_{nom}\} \tag{1.7a}$$
$$R_d = R\{X_k, a_{nom}\} / \gamma_R \tag{1.7b}$$
$$R_d = R\{X_k / \gamma_m, a_{nom}\} / \gamma_{rd} \tag{1.7c}$$

where γ_R is a partial safety factor for the resistance;
γ_m is a material safety factor;
γ_{rd} covers uncertainties in the resistance model and in the geometrical properties.

(3) The design resistance may also be obtained directly from the characteristic value of a product resistance, without explicit determination of design values for individual basic variables, from:

$$R_d = R_k / \gamma_R \tag{1.7d}$$

This is applicable for steel members, piles, etc. and is often used in connection with design by testing.

1.9.4 Ultimate Limit States

1.9.4.1 *Verifications of Static Equilibrium and Strength*

(1) When considering a limit state of static equilibrium or of gross displacement of the structure as a rigid body, it shall be verified that:

$$E_{d,dst} \leq E_{d,stb} \tag{1.8}$$

where $E_{d,dst}$ is the design value of the effect of destabilizing actions;
$E_{d,stb}$ is the design value of the effect of stabilizing actions.

In some cases if may be necessary to replace eq. (1.8) by an interaction formula.

(2) When considering a limit state of rupture or excessive deformation of a section, member or connection it shall be verified that:

$$E_d \leq R_d \tag{1.9}$$

where E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments;
R_d is the corresponding design resistance, associating all structural properties with the respective design values.

In some cases it may be necessary to replace eq. (1.9) by an interaction formula. The required load cases are identified as described in Section 1.9.1.

1.9.4.2 *Combination of Actions*

(1) For each critical load case, the design values of the effects of actions (E_d) should be determined by combining the values of actions which occur simultaneously, as follows:

(a) **Persistent and transient situations:** Design values of the dominant variable actions and the combination design values of other actions.

(b) **Accidental situations:** Design values of permanent actions together with the frequent value of the dominant variable action and the quasi-permanent values of other variable actions and the design value of one accidental action.

(c) **Seismic situations:** Characteristic values of the permanent actions together with the quasi-permanent values of the other variable actions and the design value of the seismic actions.

(2) When the dominant action is not obvious, each variable action should be considered in turn as the dominant action.

(3) The above combination process is represented in Table 1.1.

Table 1.1: Design Values of Actions for use in the Combination of Actions

Design situation	Permanent actions G_d	Single variable actions Q_d		Accidental actions or seismic actions A_d
		Dominant	Others	
Persistent and transient	$\gamma_G G_k$ $(\gamma_p P_k)$	$\eta_{Q1} Q_{k1}$	$\eta_{Q1} \Psi_{Qi} Q_{ki}$	
Accidental	$\gamma_{GA} G_k$ $(\gamma_{PA} P_k)$	$\Psi_{11} Q_{k1}$	$\Psi_{2i} Q_{ki}$	$\gamma_A A_k$ or A_d
Seismic	G_k		$\Psi_{2i} Q_{ki}$	A_{Ed}

Symbolically the combinations may be represented as follows

(a) persistent and transient design situations for ultimate limit states verification other than those relating to fatigue

$$\sum_{j \geq 1} \gamma G_{kj} + \gamma_{p_k} + \gamma_{qi} + \sum_{i > l} \gamma_{Qi} \psi_{oi} Q_{ki} \qquad (1.10)$$

Note: This combination rule is an amalgamation of two separate load combinations:

$$\sum_{j \geq 1} \gamma G_{kj} + \gamma_{p_k} + \gamma_{qi} + \sum \psi_{01} Q_{kl} + \sum_{i > l} \gamma_{Qi} \psi_{01} Q_{ki} \qquad (1.10a)$$

$$\sum_{j \geq 1} \xi \gamma G_{kj} + \gamma_{p_k} + \gamma_{qi} + \sum \psi_{01} Q_{kl} + \sum_{i > l} \gamma_{Qi} \psi_{01} Q_{ki} \qquad (1.10b)$$

ξ is a reduction factor for γ_{Gj} within the range 0.85 and 1.

(b) Combinations for accidental design situations

$$\sum \gamma_{GAj} G_{kj} + \gamma_{PA} P_k + A_d + \psi_{11} Q_{kl} + \sum \psi_{2i} Q_{ki} \qquad (1.11)$$

(c) Combination for the seismic design situation

$$\sum G_{kj} \ + \ P_k \ + \ A_{Ed} \ + \ \sum \psi_{2i} \, Q_{ki} \tag{1.12}$$

"+" "implies" to be combined with"
Σ implies "the combined effect of";
G_{kj} is the characteristic value of permanent actions;
P_k is the characteristic value of a prestressing action;
Q_{k1} is the characteristic value of the variable action;
Q_{kj} is the characteristic value of the variable actions;
A_d is the design value of the accidental action;
A_{Ed} is the design value of seismic action;
γ_{Gj} is the partial factor for permanent action j;
γ_{GAj} is the same as γ_{Gj}, but for accidental design situations;
γ_{PA} is the same as γ_p, but for accidental actions;
γ_p is the partial factor for prestressing actions;
γ_{Qi} is the partial factor for variable action i;
Ψ are combination coefficients (see 1.4.3).

(4) Combinations for accidental design situations either involve and explicit accidental action (A (e.g. fire or impact)or refer to a situation after an accidental event (A=0). For fire situations, apart from the temperature effect on the material properties, A_d refers to the design value of the indirect thermal action.

(5) Equations (1.10) and (1.11) may refer to either actions or action effects; for non-linear analysis, see Section 1.9.3.2 (3).

(6) Where components of a vectorial force are partially correlated, the factors to any favourable component may be reduced by 20%.

(7) Imposed deformations should be considered where relevant.

(8) In some cases eqs. (9.10) to (9.12) need modification; detailed rules are given in the relevant parts of EBCS 1 to 8.

1.9.4.3 *Partial Safety Factors*

(1) In the relevant load cases, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values, those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values.

(2) Where the result of a verification may be very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and the favourable parts of this action shall be considered as individual actions. This applies in particular to the verification of static equilibrium.

(3) For building structures, the partial safety factors for ultimate limit states in the persistent, transient and accidental design situations are given in Table 1.2. The values have been based on theoretical considerations, experience and back calculations on existing designs.

Table 1.2 Partial Safety Factors: Ultimate Limit States for Buildings

Case[1])	Action	Symbol	Situations	
			P/T	A
Case A Loss of static equilibrium;strength of structural material or ground insignificant (see Section 1.9.4.1	Permanent actions:self weight of structural and non-structural components, permanent actions caused by ground, ground-water and free water - unfavourable - favourable	 γ_{Gsup} [4)] γ_{Ginf} [4)]	 1.10[2)] 0.90[2)]	 1.00 1.00
	Variable actions - unfavourable	 γ_Q	 1.60	 1.00
	Accidental actions	γ_A		1.00
Case B[5)] Failure of structure or structural elements, including those of the footing, piles, basement walls etc., governed by strength of structural material (see Section 1.9.4.1)	Permanent actions [6)] (see above) - unfavourable - favourable	 γ_{Gsup} [4)] γ_{Ginf} [4)]	 1.30[3)] 1.00[3)]	 1.00 1.00
	Variable actions - unfavourable	 γ_Q	 1.60	 1.00
	Accidental actions	γ_A		1.00
Case C[5)] Failure in the ground	Permanent actions (see above) - unfavourable - favourable	 γ_{Gsup} [4)] γ_{Ginf} [4)]	 1.00 1.00	 1.00 1.00
	Variable actions - unfavourable	 γ_Q	 1.30	 1.00
	Accidental actions	γ_A		1.00

P: Persistent situation *T*: Transient situation *A*: Accidental situation

(1) The design should be verified for each A, B and C separately as relevant.
(2) In this verification the characteristic value of the unfavourable part of the permanent action is multiplied by the factor 1.1 and the favourable part by the factor 0.9. More refined rules are given in EBCS 3 and EBCS 4.
(3) In this verification the characteristic values of all permanent actions from one source are multiplied by 1.3 if the total resulting action effect is unfavourable and by 1.0 if the total resulting action effect is favourable.
(4) In cases when the limit state is very sensitive to variation of permanent actions, the upper and lower characteristic values of these actions should be taken according to Section 1.4.2 (3).
(5) For cases B and C the design ground properties may be different, see EBCS 7.
(6) Instead of using γ_G (1.30 and γ_Q = (1.60) for lateral earth pressure actions the design ground properties may be introduced in accordance with EBCS 7 and a model factor γ_{sd} is applied.

1.9.4.4 Ψ *Factors*

(1) Ψ factors for buildings are given in Table 1.3. For other applications see relevant Chapter of this code.

Table 1.3 ΨFactors for Buildings

Action	Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings [1)			
category A: domestic, residential	0.7	0.5	0.3
category B: offices	0.7	0.5	0.3
category C: congregation areas	0.7	0.7	0.6
category D: shopping	0.7	0.7	0.6
category E: storage	1.0	0.9	0.8
Traffic loads in buildings			
category F: vehicle weight ≤30kN	0.7	0.7	0.6
category G: 30kN < vehicle weight ≤160kN	0.7	0.5	0.3
category H: roofs	0	0	0
Wind loads on buildings	0.6	0.5	0
Temperature (non-fire) in buildings	0.6	0.5	0
(1) For combination of imposed loads in multistorey buildings, see Chapter 2.			

1.9.4.5 *Simplified Verification for Building Structures*

(1) The process for the persistent and transient situations described in Section 1.9.4.2 may be simplified by considering the most unfavourable for the following combinations:

(a) Design situations with only one variable action Q_{kl}

$$\sum_{j\geq 1} \gamma_{Gi}\, G_{kj} + 1.6\, Q_{kl} \tag{1.13}$$

(b) Design situations with two or more variable actions $Q_{k,i}$

$$\sum_{j\geq l} \gamma_{Gj} + 1.35 \sum Q_{ki} \tag{1.14}$$

In this case the effect of actions should also be verified for the dominant variable actions using Eq. (1.13).

(2) The γ_G values are given in Table 1.2.

1.9.4.6 *Partial Safety Factors for Materials*

Partial safety factors for properties of materials and products are given in EBCS 2 to 8.

1.9.5 Serviceability Limit States

1.9.5.1 *Verification of Serviceability*

(1) It shall be verified that:

$$E_d \leq C_d \tag{1.15}$$

where C_d is a nominal value or a function of certain design properties of materials related to the design effects of actions considered; and

E_d is the design value of the action effect (e.g. displacement, acceleration), determined on the basis of one of the combinations defined in Section 1.9.5.2.

1.9.5.2 Combination of Actions

(1) The combination of actions to be considered for serviceability limit states depends on the nature of the effect of actions being checked, e.g. irreversible, reversible or long term. Three combinations designated by the representative value of the dominant action are given in Table 1.4.

Table 1.4 Design Values of Actions for use in the Combination of Actions

Combination	Permanent actions G_d	Variable actions Q_d	
		Dominant	Others
Characteristic (rare)	G_k (P_k)	Q_{kl}	$\psi_{0i}Q_{ki}$
Frequent	G_k (P_k)	$\Psi_{1l}Q_{kl}$	$\Psi\Psi_{2i}Q_{ki}$
Quasi-permanent	G_k (P_k)	$\Psi_{2l}Q_{kl}$	$\Psi_{2i}Q_{ki}$
Note: For serviceability limit states, the partial factors (serviceability) γ_G γ_Q are taken as 1.0 except where specified otherwise.			

(2) Three combinations of actions for serviceability limit states are defined symbolically by the following expressions:

(a) Characteristic (rare) combination

$$\Sigma\, G_{ki} \text{ "+" } P_k \text{ "+" } Q_{k1} \text{ "+" } \Sigma\Psi_{o1}Q_{ki} \tag{1.16}$$

(b) Frequent combination

$$\Sigma\, G_{kj} \text{ "+" } P_k \text{ "+" } \Psi_{11}Q_{k1} \text{ "+" } \Sigma\Psi_{2i}Q_{ki} \tag{1.17}$$

(c) Quasi-permanent combination

$$\Sigma\, G_{kj} \text{ "+" } P_k \text{ "+" } \Sigma\, \Psi_{2i}Q_{ki} \tag{1.18}$$

Where the notation is as given in Sections 1.1.6 and 1.9.4.2

(3) Loads due to imposed deformations should be considered where relevant.

(4) In some cases Eqs. (1.16) to (1.18) may require a modification; detailed rules are given in EBCS 1 to 8.

1.9.5.3 *Partial Safety Factors*

The partial safety factors for serviceability limit states are equal to 1.0 except where specified otherwise, e.g. in EBCS 2 to 8.

1.9.5.4 Ψ *Factors*

Values of Ψ factors are given in Table 1.3

1.9.5.5 *Simplified Verification for Building Structures*

(1) For building structures the characteristic (rare) combination may be simplified to the following expressions, which may also be used as a substitute for the frequent combination.

(a) Design situations with only one variable action Q_{k1}

$$\sum_{j \geq 1} G_{kj} + Q_{k1} \tag{1.19}$$

(b) Design situations with two or more variable actions, Q_{k1}

$$\sum_{j \geq 1} G_{kj} + 0.9 \sum_{i \geq 1} Q_{ki} \tag{1.20}$$

In this case the effect of actions should also be verified for the dominant variable action using Eq. (1.19).

(2) Where simplified prescriptive rules are given for serviceability limit states, detailed calculations using combinations of actions are not required.

1.9.5.6 *Partial Safety Factors for Materials*

Partial safety factors for the properties of materials and products are given in EBCS 2 to 8.

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CHAPTER 2
ACTION ON STRUCTURES - DENSITIES, SELF-WEIGHT AND IMPOSED LOADS

2.1 GENERAL

2.1.1 Scope

(1) Design guidance and actions are provided for the structural design of buildings and civil engineering works including some geotechnical aspects for the following subjects:

(a) Densities of construction materials and stored materials;
(b) Self-weight of construction elements;
(c) Imposed loads

(2) Section 2.4 gives characteristic values for densities of specific building materials, and stored materials. In addition for specific materials, the angle of repose is provided.

(3) Section 2.5 provides methods for the assessment of the characteristic values of self-weight of construction elements.

(4) Section 2.6 gives characteristic values of imposed loads on floors and roofs in building structures.

(5) These characteristic values are defined according to category of use as follows:

(a) areas in dwellings, offices etc;
(b) garage and vehicle traffic areas;
(c) areas for storage and industrial activities;
(d) roofs

(6) The loads on traffic areas given in Section 2.6 refers to vehicles up to a gross weight of 160kN.

(7) For barriers or partition walls having the function of barriers, horizontal forces due to persons are given.

(8) Section 2.6 does not specify fatigue loads and dynamic loads causing vibrations or dynamic effects.

2.2 CLASSIFICATION OF ACTIONS

2.2.1 Self-Weight

(1) Self-weights of construction elements are classified as permanent actions and generally also are fixed actions (see Chapter 1).

(2) Earth loads on roofs and terraces shall be considered as variable actions. Pressure on basement walls induced by earth loads shall however be considered a permanent action. Pore water pressure shall also be considered a permanent action.

(3) Loads due to ballast shall be considered as variable actions.

2.2.2 Imposed Loads

(1) Imposed loads are classified as variable and free actions (see Chapter 1).

(2) Imposed loads should be considered as static loads, non resonant dynamic effects being considered.

2.3 DESIGN SITUATIONS

2.3.1 General

(1) The relevant self-weights and imposed loads shall be determined for each design situation identified in accordance with Chapter 1.

2.3.2 Self-Weight

(1) Post-execution additional new coatings and/or distribution conduits should be considered in design situations.

(2) The source and moisture content of bulk materials should be considered in design situations of buildings used for storage purposes.

2.3.3 Imposed Loads

(1) For cases involving interaction with other types of load (e.g. wind), the total imposed load on a building shall be considered as a single action.

(2) Where the characteristic value of the imposed load is reduced by ψ factors in combination with other actions, the loads shall be assumed in all storeys without reduction by the factor α_n.

(3) This Chapter does not specify fatigue loads.

(4) In the case of production areas where the number of load variations or the effects of vibrations may cause fatigue, a fatigue load model shall be established for the particular case.

2.4 DENSITIES OF BUILDING MATERIALS AND STORED MATERIALS

2.4.1 Definitions

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

(2) 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.

2.4.2 Tables

(1) The densities and angles of repose provided in Tables 2.1 to 2.8 of some materials may vary from those indicated depending on moisture content, settlement and depth of storage.

Table 2.1 Construction Materials

Materials	Density γ [kN/m³]
concrete	
lightweight	9-20
normal weight	24*
heavyweight	>28
reinforced and prestressed concrete	+ 1
unhardened concrete	+ 1
mortar	
cement mortar	23
gypsum mortar	17
lime mortar	19
masonry units	
basalt	27
limestone	25
granite	27
sandstone	23
Trachyte	26
metals	
aluminium	27
brass	83
bronze	83
copper	87
iron, cast	71
iron, wrought	76
lead	112
steel	77
zinc	71

*density may be in the range 20-28 depending on local material

Table 2.1 Construction Materials (cont'd)

Material	Density γ [kN/m³]
wood	
Bahir Zaf (Eucalyptus Globulus Labill)	8.5
Kerero (Pouteria Ferrginea)	6.5
Sombo (Ekebergia Rueppeliana)	6.5
Tid (Juniperus Procera)	7.5
Zegba (Podacargus Gracilior)	6.0
plywood:	
raw plywood (softwood and birch)	6
laminboard and blockboard	4
particleboards:	
chipboard	8
cement-bonded particleboard	12
flakeboard, oriented strand board, waterboard	7
fibre building board:	
hardboard, standard and tempered	10
medium density fibreboard	8
softboard	4
other materials	
glass, in sheets	25
plastics:	
acrylic sheet	12
polystyrene, expanded,granules	0.25
slate	29

Table 2.2 Stored Materials - Building and Construction

Materials	Density γ [kN/m³]	Angle of repose ϕ [°]
aggregates		
lightweight (pumice)	7	30
lightweight (scoria)	12	30
normal	14	30
gravel	14	35
sand	14	30
brick sand, crushed brick, broken bricks	15	-
vermiculite		
exfoliated, aggregate for concrete	1	-
crude	6 - 9	-
bentonite		
loose	8	40
shaken down	11	-
cement		
in bulk	16	28
in bag	15	-
fly ash	10 - 14	25
glass, in sheets	25	-
gypsum, ground	15	25
lignite filter ash	15	20
lime	13	25
limestone, powder	13	27
magnesite, ground	12	-
plastics,		
polyethylene, polystyrol granulated	6.4	-
polyvinylchloride, powder	5.9	-
polyester resin	11.8	-
glue resins	13	-
water, fresh	10	-

Table 2.3 Stored Materials - Agricultural

Materials	Density γ [kN/m^3]	Angle of repose ϕ [°]
farmyard		
manure (minimum 60% solids)	7.8	-
manure (with dry straw)	9.3	45
dry chicken manure	6.9	45
slurry (maximum 20% solids)	10.8	-
fertiliser, artificial		
NPK, granulated	8 - 12	25
basic slag, crushed	13.7	35
phosphates, granulated	10 - 16	30
potassium sulphate	12 - 16	28
urea	7 - 8	24
fodder, green, loosely stacked	3.5 - 4.5	-
grain		
whole ($\leq$ 14% moisture content unless indicated otherwise)		
general	7.8	30
barley	8.0	30
beans	7.0	30
brewer's grain (wet)	7.0	30
coffee	8.8	30
herbage seeds	8.0	30
corn	3.4	30
maize in bulk	7.4	30
maize in bags	8.0	30
legumes	5.0	-
oats	5.0	30
oilseed rape	8.0	30
sorghum	6.4	25
rye	9.0	25
tef	7.0	30
wheat in bulk	7.8	30
wheat in bags	7.5	-

Table 2.3 Stored Materials - Agricultural (contn'd)

Material	Density γ [kN/m^3]	Angle of repose ϕ [°]
grass cubes	7.8	40
hay		
(baled)	1 - 3	-
(rolled bales)	6 - 7	-
hides and skins	8 - 9	25
hops	1 - 2	
malt	4 - 6	
meal	7	45
ground	7	40
cubes	9.5	-
silage	5 - 10	-
straw		-
in bulk (dry)	0.7	-
baled	1.5	-
tobacco in bales	3.5 - 5	-
wool		-
in bulk	3	-
baled	7 - 13	-

Table 2.4 Stored Materials - Foodstuffs

Materials	Density γ [kN/m^3]	Angle of Repose ϕ [°]
butter	9.5	-
eggs, in stands	4 - 5	-
flour		
bulk	6	25
bagged	5	-
fruit		
loose	8.3	30
boxed	6.5	-
honey	13	-
milk	10.5	-
sugar		-
bulk (loose)	9.5	35
sacks (compact)	16.0	-
vegetables, green		
cabbages	4	-
lettuce	5	-
vegetables, legumes		
beans		
general	8.1	35
soya	7.4	-
peas	7.8	-
vegetables, root		
general	8.8	-
beetroot	7.4	40
carrots	7.8	35
onions	7	35
potatoes		
in bulk	7	35
in boxes	4.4	-

Table 2.5 Stored Materials - Liquids

Materials	Density γ [kN/m^3]
beverages	
beer	10.3
milk	10.1
water, fresh	9.8
wine	10
natural oils	
castor oil	
linseed oil	9.3
	9.2
organic liquids and acids	
alcohol	7.8
ether	7.4
hydrochloric acid (40% by weight)	11.8
methylated spirit	7.8
nitric acid (91% by weight)	14.7
sulphuric acid (30% by weight)	13.7
sulphuric acid (37% by weight)	17.7
turpentine, white spirit	8.3
hydrocarbons	
aniline	9.8
benzene (benzole)	8.8
coal tar	10.8 - 12.8
creosote	10.8
naphtha	7.8
paraffin (kerosene)	8.3
benzine (benzoline)	6.9
oil, crude (petroleum)	9.8 - 12.8
diesel	8.3
fuel	7.8 - 9.8
heavy	12.3
lubricating	8.8
petrol (gasolene,gasoline)	7.4
liquid gas	
butane	5.7
propane	5.0
other liquids	
mercury	133
red lead paint	59
white lead, in oil	38
sludge, over 50% by volume water	10.8

Table 2.6 Stored Materials - Solid Fuels

Materials	Density γ [kN/m^3]	Angle of Repose ϕ [°]
charcoal		
air-filled	4	-
air-free	15	-
firewood	5.4	45

Table 2.7 Stored Materials - Industrial and General

Material	Density γ [kN/m^3]	Angle of Repose ϕ [°]
books and documents		
books and documents	6	-
densely stored	8.5	-
filing racks and cabinets	6	-
garments and rags, bundled	11	-
ice, lumps	8.5	-
leather, piled	10	-
paper		-
layers	11.0	-
rolls	15.0	-
rubber	10 - 17	45
rock salt	22	40
salt	12	
sawdust		
dry, bagged	3	-
dry, loose	2.5	45
wet, loose	5	45
tar, bitumen	14	-

Table 2.8 Flooring and Walling

Materials	Density kN/m³
Flooring	
clay tiling	21
Marble tiling	27
Parquet, timber board	9
PVC covering	16
Rubber covering	17
Granulithic, terrazzo paving	23
Walling	
Solid brick	22
Perfurated brick	19
Concrete hollow-block	
Stone aggregate	14-20*
Lightweight (pumice) aggregate	10-14*
Asoestos cement sheet	17
Fibrous plaster board	10

*Lower values for smaller size thicknesses (100mm to 200mm)

2.5 SELF-WEIGHT OF CONSTRUCTION ELEMENTS

2.5.1 Representation of Actions

(1) Construction elements include structural and non-structural elements.

(2) For the purpose of this section, the self-weight of non-structural elements shall include the weight of fixed machinery.

(3) Non-structural elements include:

(a) roofing
(b) surfacing and coverings
(c) non-structural partition walls and linings
(d) hand rails, safety barriers, parapets and kerbs
(e) wall cladding
(f) suspended ceilings
(g) insulation
(h) fixed machinery
(i) earth and balast

(4) Fixed machinery includes:

(a) lifts and moving stairways
(b) heating, ventilating and air conditioning equipment
(c) electrical equipment
(d) pipes without their contents
(e) cable trunking and conduits.

(5) Loads due to movable partitions shall be treated as imposed loads (see Section 2.6).

(6) The self-weight of industrial equipment should be considered as an imposed load. Only the self-weights of equipment incorporated into the construction shall be classified as permanent actions.

(7) Where there is a reasonable likelihood that services will at some time be relocated within the building, loads due to these services shall be considered as imposed loads.

2.5.2 Load Arrangements

(1) In the case where the self-weight is classified as a fixed action it may be assumed that the variations of densities as well as the differences between nominal and actual dimensions of construction elements do not change within a given structure.

2.5.3 Self-Weight - Characteristic Values

2.5.3.1 *Assessment of Self-Weight*

2.5.3.1.1 *Characteristic Value*

(1) The weights of parts of structures and of non-structural elements shall be determined from the weights of the elements of which they are composed.

(2) Unless more reliable data are available (i.e. from product standards, the supplier or by direct weighing), the characteristics value of the weight of individual elements shall be estimated from nominal dimensions and the nominal densities of their constituent materials.

(3) However, in accordance with Chapter 1, it may be necessary to consider both upper and lower characteristic values for the self-weight. This may apply to thin concrete members or in cases of uncertainty about the precise value of self-weight, or where dimensional alternatives and the type of materials to be used remain open at the design stage.

2.5.3.1.2 *Dimensions*

(1) In general nominal dimensions should be those as shown on the drawings.

(2) In general, where the weight of thin finishes is small in comparison in comparison with the weight of the elements to which they are applied, it is not necessary to consider variation in finish thickness. However variation in thickness may need to be considered when the thickness depends on the deflection of the structural component to which the finish is applied or when the maintenance of the finish may include the addition of further layers of material.

2.5.3.1.3 *Densities*

(1) For the assessment of nominal densities Section 2.4 should be used.

(2) If the density of the material is likely to deviate significantly from the specified value, such deviation shall be considered.

(3) For structures where more accurate values are required, for example, where a design is likely to be particularly sensitive to variations in permanent load, a representative sample of the materials to

be used should be tested at representative moisture contents. Characteristic and representative values should then be determined according to Chapter 1.

(4) For some materials the bulk weight density has significant variability and may be dependent on the source and moisture content.

2.5.3.2 *Self-Weight for Buildings*

2.5.3.2.1 *Floors and Walls and Partitions*

(1) For determining the effect of the self-weight due to partitions, an equivalent uniformly distributed load may be used.

(2) Account should be taken of voids made for the purpose of thermal insulation or for the reduction of weight.

(3) For suspended beam and block floors and beam and hollow-pot floors, data may be provided by the manufacturer. When the dimensions of thin concrete slabs are unlikely to be controlled to within $\pm$ 5% of their nominal values a range of values for the permanent load shall be taken into account and treated as indicated in Chapter 1.

(4) For determining the weight of unrendered masonry walls the weight of mortar shall be taken into account.

2.5.3.2.2 *Roofs*

(1) The weights shall be calculated from the weight of the component materials and the geometry (e.g. pitch tiles/sq.metre etc).

(2) Information may be taken from documents provided by the manufacturer.

2.5.3.2.3. *Claddings and Finishes*

(1) For the purpose of this section claddings shall be considered to include curtain walling (and fixings), overcladding (and fixings) and roof coverings.

(2) When designing individual structural elements the estimation of weight shall include the weight of claddings and finishes, unless alternative provision has been made (see Section 2.5.3.1). Finishes include in-situ finishes (such as plaster and screeds), prefabricated wall-panel finishes, and timber and other floor finishes.

2.6 IMPOSED LOADS ON BUILDINGS

2.6.1 Representation of Actions

(1) Imposed loads on buildings are those arising from occupancy. They may be caused by:

(a) normal use by persons;
(b) furniture and moveable objects (e.g.lightweight moveable partitions, storage, the contents of containers);
(c) machines and vehicles;

(d) exceptional use, such as exceptional concentrations of persons or of furniture, or the moving or stacking of commodities which may occur during reorganization or redecoration.

(2) The self-weight of structural and non-structural components and of fixed equipment shall be taken into account according to Section 2.5.

(3) Imposed loads are modelled by uniformly distributed loads or concentrated loads or combination of these loads.

(4) The characteristic values of the loads are determined for a reference period, (See Chapter 1).

(5) The characteristic values of the loads are composed of long-term, medium-term and short-term components that, according to their duration, may have different effects on materials sensitive to time-dependent actions.

2.6.2 Load Arrangements

2.6.2.1 *Horizontal Members*

(1) For the design of the elements of a floor structure within one storey the action shall be assumed as a free action on the most unfavourable tributary zone of the influence area. Where the loads on other floors are relevant, they may be assumed to be distributed uniformly (fixed actions).

(2) Imposed loads from a single occupancy may be reduced according to the tributary area by a reduction factor α_A according to Sections 2.6.3.1.2(3) and 2.6.2.2(4).

(3) To ensure a minimum local resistance of the floor structure a separate verification shall be performed with a concentrated load that, unless stated otherwise, shall not be combined with the uniformly distributed loads or other variable loads.

2.6.2.2 *Vertical Members*

(1) For the design of columns or walls acting as vertical members, loaded from several storeys, the loads on the floor of each storey shall be assumed to distributed uniformly (fixed actions).

(2) Where the imposed loads from several storeys are relevant, the loads may be reduced by a reduction factor α_n according to Sections 2.6.1.2(4)

2.6.3 Imposed Loads - Characteristic Values

2.6.3.1 *Residential, Social, Commercial and Administration Area*

2.6.3.1.1 *Categories*

(1) Areas in residential, social, commercial and administration buildings are divided into five categories according to their specific uses shown in Table 2.9.

Table 2.9 Categories of Building Areas

Category	Specific Use	Example
A	Area for domestic and residential activities	Rooms in residential buildings and houses; rooms and wards in hospitals; bedrooms in hotels and hostels; kitchens and toilets
B	Areas where people may congregate (with the exception of areas defined under category A,B,D, and E)	C1: Areas with tables, etc. e.g. areas in schools, cafes, restaurants, dining halls, reading rooms, receptions etc. C2: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, etc. C3: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc and access areas in public and administration buildings, hotels, etc. C4: Areas susceptible to overcrowding, e.g. dance halls, gymnastic rooms, stages, etc. C5: Areas susceptible to overcrowding, e.g. in buildings for public-events like concert halls, sports halls including stands, terraces and access areas, etc.
D	Shopping areas	D1: Areas in general retail shops, e.g. areas in warehouses, stationery and office stores, etc.
E	Areas susceptible to accumulation of goods, including access areas	Areas for storage use including libraries. The loads defined in Table 2.10 shall be taken as minimum loads unless more appropriate loads are defined for the specific case. Further guidance is given in Table 2.7.

2.6.3.1.2 *Values of Actions*

(1) The characteristic values q_k and Q_k are given in Table 2.10.

(2) For local verifications a concentrated load Q_k acting alone shall be taken into account. The characteristic values Q_k are given in Table 2.9. Where concentrated loads from storage racks or from lifting equipment may be expected Q_k shall be determined for the individual case (see Section 2.6.3.3).

The local concentrated load shall be considered to act at any point on the floor, balcony or stairs and to have an application area comprising a square with a 50mm side.

(3) The reduction factor α_A for categories A to E should be determined as follows:
With the restriction for categories C and D: $\alpha_A \geq 0.6$

$$\alpha_A = 5/7 \cdot \psi_o + \frac{A_o}{A} \leq 1.0 \qquad (2.1)$$

where ψ_o is the factor according to Chapter 1.
A_o 10.0m²
A is the loaded area

(4) The reduction factor α_n for categories A to E should be determined as follows:

$$\alpha_n = \frac{2+(n-2)\ \psi_0}{n} \qquad (2.2)$$

where n is the number of storeys (>2) above the loaded structural elements

Table 2.10 Imposed Loads on Floors in Buildings

Loaded area	q_k (kN/m2)	Q_k (kN)
Category A - general	2.0	2.0
- stairs	3.0	2.0
- balconies	4.0	2.0
Category B	3.0	2.0
Category C - C1	3.0	4.0
- C2	4.0	4.0
- C3	5.0	4.9
- C4	5.0	7.0
- C5	5.0	4.0
Category D - D1	5.0	4.0
- D2	5.0	7.0
Category E	6.0	7.0

2.6.3.2 *Garage and Vehicle Traffic Areas*

2.6.3.2.1 *Categories*

(1) Traffic areas in buildings are divided into two categories according to their accessibility for vehicles as shown in Table 2.11.

Table 2.11 Traffic Areas in Buildings

Category	Specific Use	Example
F	Traffic and parking areas for light vehicles ($\leq$ 30 kN total weight and $\leq$ 8 seats not including driver)	e.g. garages; parking areas, parking halls
G	Traffic and parking areas for medium vehicles (>30 kN, $\leq$ 160 kN total weight, on 2 axles)	e.g. access routes; delivery zones; zones accessible to fire engines ($\leq$ 160 kN total weight)

(2) Access to areas designed to category F shall be limited by physical means built into the structure.

(3) Areas designed to categories F and G should be posted with the appropriate warning signs.

2.6.3.2.2 *Values of Actions*

(1) The characteristic values for the concentrated loads Q_k representing a single axle with dimensions according to Fig. 2.1 and the distributed load q_k are given in Table 2.12.

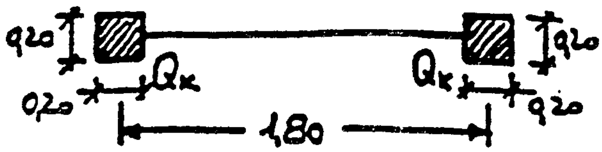

Figure 2.1 Dimensions of Axle Load

Table 2.12 Imposed Loads on Garages and Vehicle Traffic Areas

Traffic areas	q_k (kN/m2)	Q_k (kN)
Category F vehicle weight: $\leq$30, $\leq$160 kN	2.0	10
Category G vehicle weight: >30, $\leq$160 kN	5.0	45

(2) Both the concentrated load Q_k and the uniformly distributed load q_k shall be considered to act together.

(3) Each concentrated load shall be applied on a square surface with a 200mm side in the positions which will produce the most adverse effects.

(4) The reduction coefficient α_A for categories F and G shall be considered as follows:

$$\alpha_A = 1.0 \tag{2.3}$$

(5) The reduction coefficient α_n for categories F and G shall be considered as follows:

$$\alpha_n = 1.0 \tag{2.4}$$

2.6.3.3 *Areas for Storage and Industrial Activities*

(1) The characteristic value of the imposed load and also the loading arrangement (free or fixed actions) shall be defined, respectively, by the maximum value taking account of dynamic effects if appropriate and the most unfavourable condition allowed in use.

(2) The maximum permitted loads should be indicated by sings in the rooms concerned.

(3) The characteristic values of vertical loads in storage areas may be derived by taking the values given in Section 2.4 and upper design values for stacking heights. When stored material exerts horizontal forces on walls etc., the horizontal for may be determined from specialist literature. Any effects of filling and emptying shall be taken into account.

(4) Loads for storage areas for books and files shall be determined from the loaded area and the height of the book cases using the density values in Section 2.4.

(5) Loads on industrial areas may comprise machines, production units, heavy rolling engines that can have a defined lane, suspended cranes, etc. that cannot be modelled by uniformly distributed loads but need more detailed modelling.

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

2.6.3.4 *Roofs*

2.6.3.4.1 *Categories*

(1) Roofs are divided according to their accessibility into three categories as shown in Table 2.13.

Table 2.13 Categorization of Roofs

Category	Specific Use
H	Roofs not accessible except for normal maintenance, repair, painting and minor repairs
I	Roofs accessible with occupancy according to categories A - G
K	Roofs accessible for special services, such as helicopter landings

(2) Loads for roofs of category H are given in Table 2.13 Loads for roofs of category I are given in Table 2.10 and Table 2.12 according to the specific use. For roofs of category K the loads should be established for the particular case.

2.6.3.4.2 *Values of Actions*

(1) The characteristic values Q_k and q_k are given in Table 2.14. They are related to the projected area of the roof under consideration.

Table 2.14 Imposed Loads on Roofs

Roofs		q_k (kN/m^2)	Q_k (kN)
Category H	Flat roof	0.5	1.0
	Sloping roof	0.25	1.0

(2) Separate verification shall be performed for the concentrated load Q_k and the uniformly distributed load q_k, acting independently.

(3) For local checks the concentrated load Q_k is given in Table 2.14. The application area of Q_k comprises a square with a 50mm side.

(4) The reduction coefficient α_A for category H shall be considered as follows:

$$\alpha_A = 1.0 \qquad (2.5)$$

(5) Access ladders and walkways shall be assumed to be loaded according to Table 2.2 for a roof slope < 20°. For walkways which are part of a designated escape route q_k shall be assumed to be:

$$q_k = 3.0 \text{ kN/m}^2 \qquad (2.6)$$

(6) The effects of water ponding on roofs should be considered.

2.6.4 Horizontal Loads on Partition Walls and Barriers due to Persons

(1) The characteristic values of the line load q_k acting at the height of the hand rail but not higher than 1.20m are given in Table 2.15.

Table 2.15 Horizontal Loads on Partition Walls and Barriers due to Persons

Loaded areas	q_k (kN/m)
Category A	0.5
Category B and C1	1.0
Categories C2 - C4 and D	1.5
Category C5	3.0

(2) For areas susceptible to significant overcrowding associated with public events e.g. for sports stadia, stands,stands, stages, assembly halls or conference rooms, the line load shall be taken according to category C5.

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CHAPTER 3
WIND ACTIONS

3.1 SCOPE

(1) This Chapter gives rules and methods for calculating wind loads on building structures up to a height of 200m, their components and appendages.

(2) Wind loads shall be calculated for each of the loaded areas under consideration. These may be:

(a) the whole structure
(b) parts of the structure, i.e components, cladding units and their fixings

(3) This Chapter also gives rules for chimneys and other cantilevered structures. Special requirements for lattice towers are not given.

3.2 CLASSIFICATION OF ACTIONS

(1) Wind action are classified as free actions, see Chapter 1.

3.3 DESIGN SITUATIONS

(1) The relevant wind actions shall be determined for each design situation identified in accordance with Chapter 1.

(2) The effect of changes of the form of the construction works which may modify the external and internal wind pressure (such as doors normally closed but left open under storm conditions) shall be considered.

(3) Structures susceptible to dynamic effect shall be designed for fatigue loading.

3.4 REPRESENTATION OF ACTIONS

3.4.1 Expansion of the Wind Actions and the Response of the Structures

(1) Wind actions are fluctuating with time. They act directly on the external surfaces of enclosed structures and, through porosity of the external surface, also act indirectly on the internal surfaces. They may also directly affect the internal surface of open structures. Pressures act on areas of the surface producing forces normal to the surface for the structure or for individual cladding components. Additionally, when large areas of structures are swept by the wind, frictional forces acting tangentially to the surface, may be significant.

To achieve the design aims account shall be taken of:

(a) turbulent wind acting over part or all of the structure (see Section 3.5 and 3.6 respectively)
(b) fluctuating pressures induced by the wake behind the structure (see Section 3.9.4)
(c) fluctuating forces induced by the motion of the structure (see Section 3.9.4).

(2) The total response of structures and their elements may be considered as the superposition of a "background" component, which acts quasi-statically and "resonant" components due to excitation close to natural frequencies. For the majority of structures the resonant components are small and the wind load can be simplified by considering the background component only. Such structures can be calculated by a simplified method. The limits to such structures are set down in Section 3.9.

(3) The dynamic effects are divided into different types according to the physical effect of the wind:

(a) stochastic and resonant response (alongwind, crosswind and torsional direction) due to turbulence and wake effects
(b) response due to vortex shedding
(c) galloping
(d) interference
(e) divergence and flutter.

(4) In this Chapter, the wind action is represented by a set of quasi-static pressures or forces whose effects are equivalent to the extreme effects of the wind. Slender structures such as chimneys, observation towers, component elements of open frames and trusses, and in some cases high rise buildings shall be designed to resist the dynamic effect of vortex shedding. General rules for evaluating such situations are provided in Section 3.9.4. Criteria are also given for aeroelastic instability.

3.4.2 Modelling of Wind Actions

(1) The wind action is represented either as a wind pressure or a wind force. The action on the structure caused by the wind pressure is assumed to act normal to the surface except where otherwise specified; e.g. for tangential friction forces.

(2) The following parameters are used several times and are defined below:

- q_{ref} reference mean wind velocity pressure derived from reference wind velocity as defined in Section 3.7.1. It is used as the characteristic value
- $c_e(z)$ exposure coefficient accounting for the terrain and height above ground z given in Section 3.8.5. The coefficient also modifies the mean pressure to a peak pressure allowing for turbulence
- z reference height defined in Appendix A appropriate to the relevant pressure coefficient ($z = z_e$) for external pressure and force coefficient, $z = z_i$ for internal pressure coefficient)
- c_d dynamic coefficient accounting for both correlation and dynamic magnification given in Section 3.9.

3.5 WIND PRESSURE ON SURFACES

3.5.1 Field of Application

(1) The representation of the wind pressure given in this Section is valid for surfaces which are sufficiently rigid to neglect their resonant vibrations caused by the wind, as is normally the case.

(2) If a natural frequency of vibration of the surface is low (i.e. less than 5 Hz), these vibrations may become significant, and they shall be taken into account. These effects are not covered by this Chapter.

3.5.2 External Pressure

(1) The wind pressure acting on the external surfaces of a structure W_e shall be obtained from:

$$W_e = q_{ref}\, c_e(z_e)\, c_{pe} \tag{3.1}$$

where c_{pe} is the external pressure coefficient derived from Appendix A.

3.5.3 Internal Pressure

(1) The wind pressure acting on the internal surfaces of a structure W_i shall be obtained from:

$$W_i = q_{ref}\; c_e(z_i)\; c_{pi} \tag{3.2}$$

where C_{pi} is the internal pressure coefficient obtained from Appendix A.

3.5.4 Net Pressure

(1) The net wind pressure across a wall or an element is the difference of the pressures on each surface taking due account of their signs. (Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative). Examples are given in Figure 3.1.

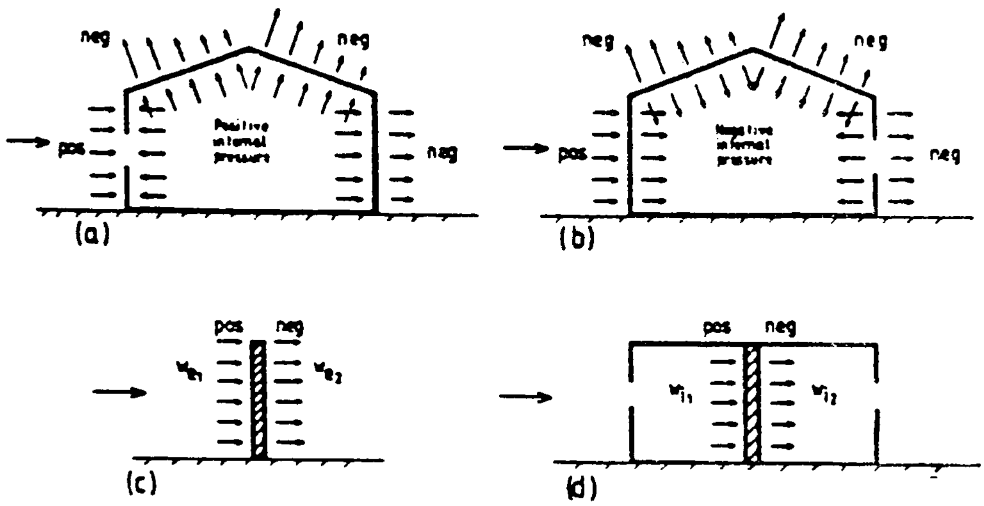

Figure 3.1 Pressure on Surfaces

3.6 WIND FORCES

3.6.1 Wind Forces from Pressures

(1) The wind forces acting on a structure or a structural component may be determined in two ways:

(a) by means of global forces
(b) as a summation of pressures acting on surfaces provided that the structure or the structural component is not sensitive to dynamic response ($C_d < 1.2$ see Section 3.9).

(2) The global force F_w shall be obtained from the following expression:

$$F_w = q_{ref}\ c_e(z_e)\ c_d\ c_f\ A_{ref} \tag{3.3}$$

where c_f is the force coefficient derived from Section 3.10
A_{ref} is the reference area for c_f (generally the projected area of the structure normal to the wind) as defined in Section 3.10

(3) For lattice structures and for vertical cantilevered structures with a slenderness ratio height/width > 2 and with nearly constant cross-section (e.g. tall buildings, chimneys, towers) the force F_{wj} on the incremental area A_j at the height z_j is:

$$F_{wj} = q_{ref}\ c_e(z_j)\ c_d\ c_{fj}\ A_j \tag{3.4}$$

where z_j is the height of the centre of gravity of incremental area A_j
c_{fj} is the force coefficient for incremental area A_j as defined in Section 3.10
A_j is the incremental area

(4) Torsional effects due to inclined or non correlated wind may be represented on non circular nearly symmetric structures by the force F_w acting with the eccentricity e:

$$e = \frac{b}{10} \tag{3.5}$$

where b is the dimension of the cross section transverse to the main axis considered (see Fig.3.2).

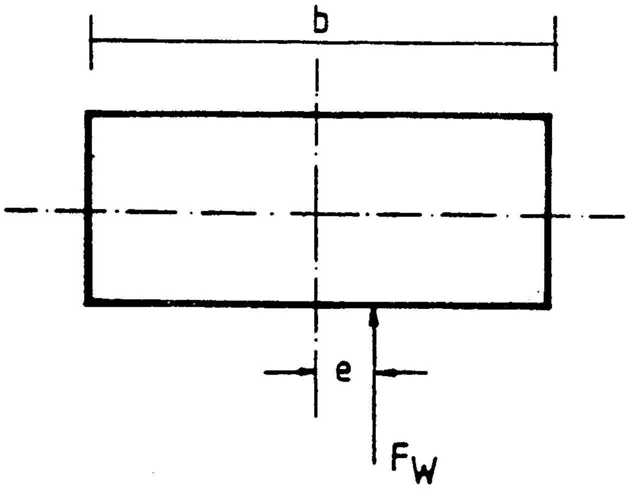

Figure 3.2 Wind Force Acting on Cross Section

(5) More detailed values of the eccentricity for special cross sections are presented in Section 3.10.

3.6.2 Friction Force

(1) For structures with large area swept by the wind (e.g. large free standing roofs), friction forces, F_{fr} may be significant. They shall be obtained from:

$$F_{fr} = q_{ref}\ c_e(z_e)\ c_{fr}\ A_{fr} \tag{3.6}$$

where c_{fr} is the friction coefficient derived from Section 3.10.13
A_{fr} is the area swept by the wind

3.7 REFERENCE WIND

3.7.1 Reference Wind Pressure

(1) The reference mean wind velocity pressure q_{ref} shall be determined from:

$$q_{ref} = \frac{\rho}{2} V_{ref}^2$$

where V_{ref} is the reference wind velocity as defined in Section 3.7.2
ρ is the air density

The air density is affected by altitude and depends on the temperature and pressure to be expected in the region during wind storms. A temperature of 20°C has been selected as appropriate for Ethiopia and the variation of mean atmospheric pressure with altitude is given in Table 3.1.

Table 3.1 Values of Air Density ρ

Site Altitude(m) Above sea level	ρ(kg/m^3
0	1.20
500	1.12
1000	1.06
1500	1.00
2000	0.94

3.7.2 Reference Wind Velocity

(1) The reference wind velocity v_{ref} is defined as the 10 minute mean wind velocity at 10m above ground of terrain category II (see Table 3.2) having an annual probability of exceedence of 0.02 (commonly referred to as having a mean return period of 50 years).

(2) It shall be determined from:

$$v_{ref} = c_{DIR}\ c_{TEM}\ c_{ALT}\ v_{ref,o} \tag{3.7}$$

where $v_{ref,o}$ is the basic value of the reference wind velocity to be taken as 22m/sec
v_{DIR} is the direction factor to be taken as 1.0.
c_{TEM} is the temporary (seasonal) factor to be taken as 1.0.
c_{ALT} is the altitude factor to be taken as 1.0.

(3) For temporary structures, which are:

(a) structures during construction (which may require temporary bracing supports
(b) structures whose life time is known and is less than one year

a reduction of the reference wind velocity may be allowed depending upon:

(a) the duration of the situation
(b) the possibilities of protecting or strengthening of the structure during wind storms
(c) the time needed to protect or strengthen the structure
(d) the probability of occurrence of wind storms
(e) the possibilities of forecasting wind storms

Based on Section 3.7.3 and/or on special local climate situation the temporary factor c_{TEM} according to Eq. (3.7) describes this reduction.

(4) Transportable structures which may be dismantled and rebuilt at any time in the year are not considered to be temporary structures.

3.7.3 Annual Probabilities of Exceedence other than 0.02

(1) The reference wind velocity $v_{ref}(p)$ for annual probabilities of exceedence p other than the value of 0.02 (see Section 3.7.2 (1) can be found using the following expression:

$$v_{ref}(p) = v_{ref}\left[\frac{1-K_1\ln[-\ln(1-p)]}{1-K_1\ln(-\ln 0.98)}\right]^n \tag{3.8}$$

where v_{ref} is the reference velocity with an annual probability of exceedence of 0.02 (see Section 3.7.2)

k_1 shape parameter. The representative value $K_1 = 0.2$ can be used.

n exponent. The representative value $n = 0.5$ can be used.

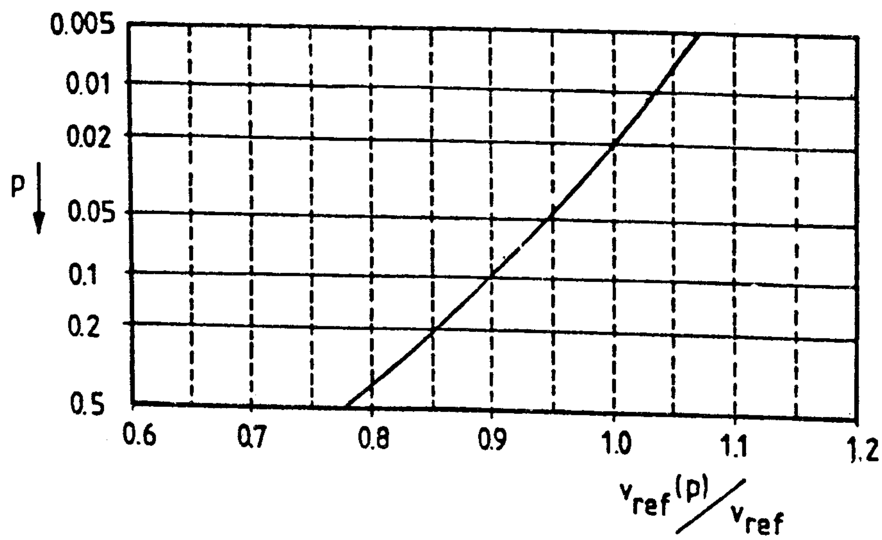

Figure 3.3 Ratio $v_{ref}(p)$ v_{ref} for k_1 = 0.2 and n = 0.5

3.8 WIND PARAMETER

3.8.1 Mean Wind Velocity

(1) In order to define the Reynolds number in A.8 and the wind coefficients and other parameters of this Chapter, the mean wind velocity $v_m(z)$ is required. It is defined by:

$$v_m(z) = c_r(z)\ c_t(z)\ v_{ref} \tag{3.9}$$

where v_{ref} is the reference wind velocity (Section 3.7.2)
$c_r(z)$ is the roughness coefficient (Section 3.8.2)
$c_t(z)$ is the topography coefficient (Section 3.8.4)

3.8.2 Roughness Coefficient

(1) The roughness coefficient $c_r(z)$ accounts for the variability of mean wind velocity at the site of the structure due to:

(a) the height above ground level
(b) the roughness of the terrain depending on the wind direction.

(2) The roughness coefficient at height z is defined by the logarithmic profile:

$$c_r(z) = k_T\ \ln(z/z_o) \qquad \text{for} \qquad z_{min} \leq z \leq 200\text{m} \tag{3.10}$$

$$c_r(z) = c_r(z_{min}) \qquad \text{for} \qquad z < z_{min} \tag{3.11}$$

where k_T is the terrain factor
z_o is the roughness length
z_{min} is the minimum height

These parameters depend on the terrain category as given in Table 3.2.

(3) At heights more than 200m above ground level specialist advice is recommended.

3.8.3 Terrain Categories

(1) The terrain categories are defined in Table 3.2.

Table 3.2 Terrain Categories and Related Parameters

Terrain Category	k_r	z_o(m)	z_{min}(m)
I Lakes with at least 5km fetch upwind and smooth flat country without obstacles	0.17	0.01	2
II Farmland with boundary hedges, occasional small farm structure, houses or trees	0.19	0.05	4
III Suburban or industrial areas and permanent forests	0.22	0.3	8
IV Urban areas in which at least 15% of the surface is covered with buildings and their average height exceeds 15m	0.24	1	16

(2) If the structure is situated near a change of terrain roughness at a distance:

(a) less than 2km from the smoother category I
(b) less than 1km from the smoother categories II and II
the smoother terrain category in the upwind direction should be used.

(3) In the above transition zones small areas of different roughness should be ignored (less than 10% of the area under consideration).

(4) When there is any doubt about the choice between two categories in the definition of a given area, the worse case should be taken.

(5) Table 3.3 gives roughness coefficient $c_r(z)$ for selected values of height z

Table 3.3 Roughness Coefficient c_r

Terrain Category	z(m)							
	2	4	8	16	30	50	100	200
I	0.90	1.02	1.14	1.25	1.36	1.45	1.57	1.68
II	0.83	0.83	0.96	1.10	1.22	1.31	1.44	1.58
III	0.72	0.72	0.72	0.87	1.01	1.13	1.28	1.43
IV	0.67	0.67	0.67	0.67	0.82	0.94	1.11	1.27

3.8.4 Topography Coefficient

(1) The topography coefficient $c_t(z)$ 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. it shall be considered for locations within the topography affected zone (see Fig. 3.5 and 3.6). It is defined by:

$$\begin{aligned} c_t &= 1 && \text{for} \quad \Phi < 0.05 \\ c_t &= 1 + 2\,s\,\Phi && \text{for} \quad 0.05 < \Phi < 0.3 \\ c_t &= 1 + 0.6\,s && \text{for} \quad \Phi > 0.3 \end{aligned} \tag{3.12}$$

Where s is the factor to be obtained by interpolation from the value of $s = 1.0$ at the crest of a hill, ridge or escarpment and the value $s = 0$ at boundary of the topography affected zone. (see figs. 3.5 and 3.6). Interpolation shall be linear with horizontal distance from crest and with height above the local ground level.

- Φ is the upwind slope H/L_u in the wind direction (see figs. 3.6 and 3.7)
- L_e is the effective length of the upwind slope, defined in Table 3.4
- L_u is the actual length of the upwind slope in the wind direction
- L_d is the actual length of 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
- z is the vertical distance from the ground level of the site

Table 3.4 Values of L_e

Slope ($\Phi = H/L_u$)	
Shallow ($0.05 < \Phi < 0.3$):	Steep ($\Phi > 0.3$)
$L_e = L_u$	$L_e = H/0.3$

(2) In valleys, $c_t(z)$ may be set to 1.0 if no speed up due to funnelling effects is to be expected. For structures situated within steep-sided valleys care should be taken to account for any increase of wind speed caused by funnelling.

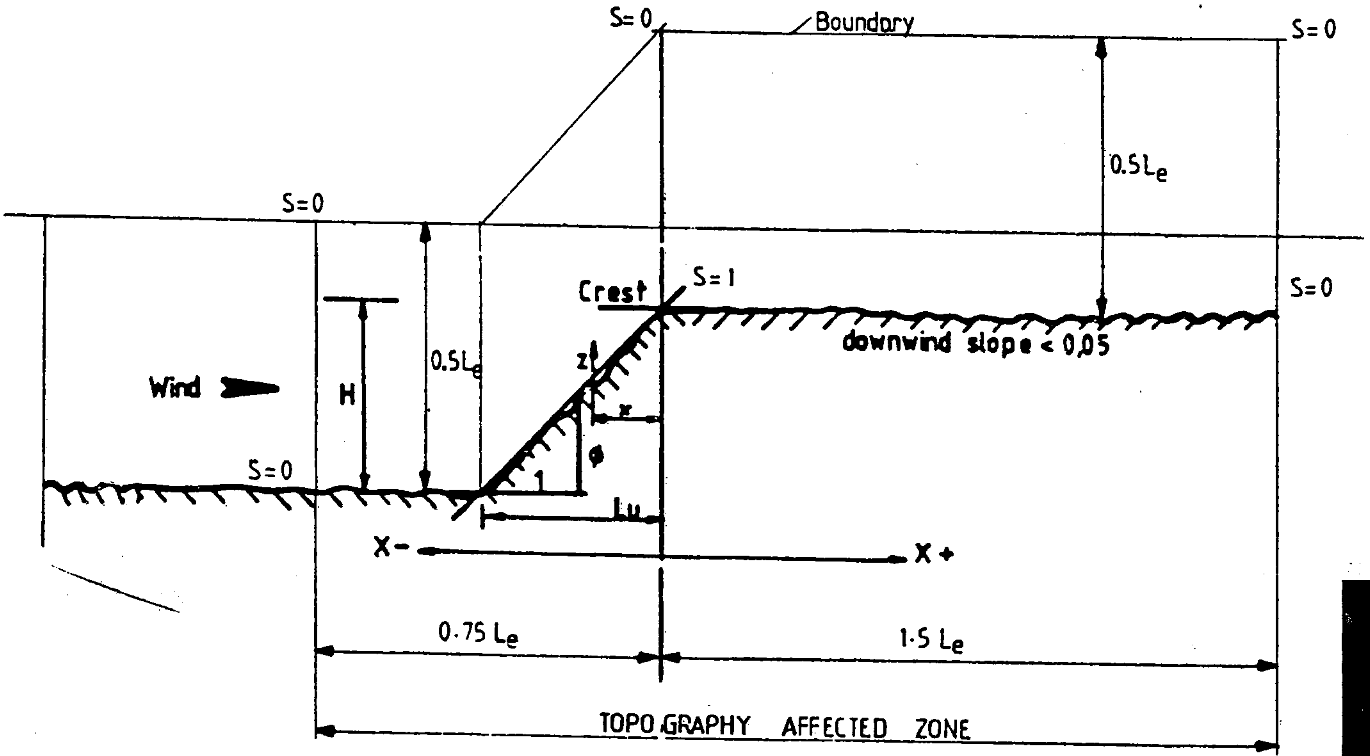

Figure 3.5 Factor s: Cliffs and Escarpments

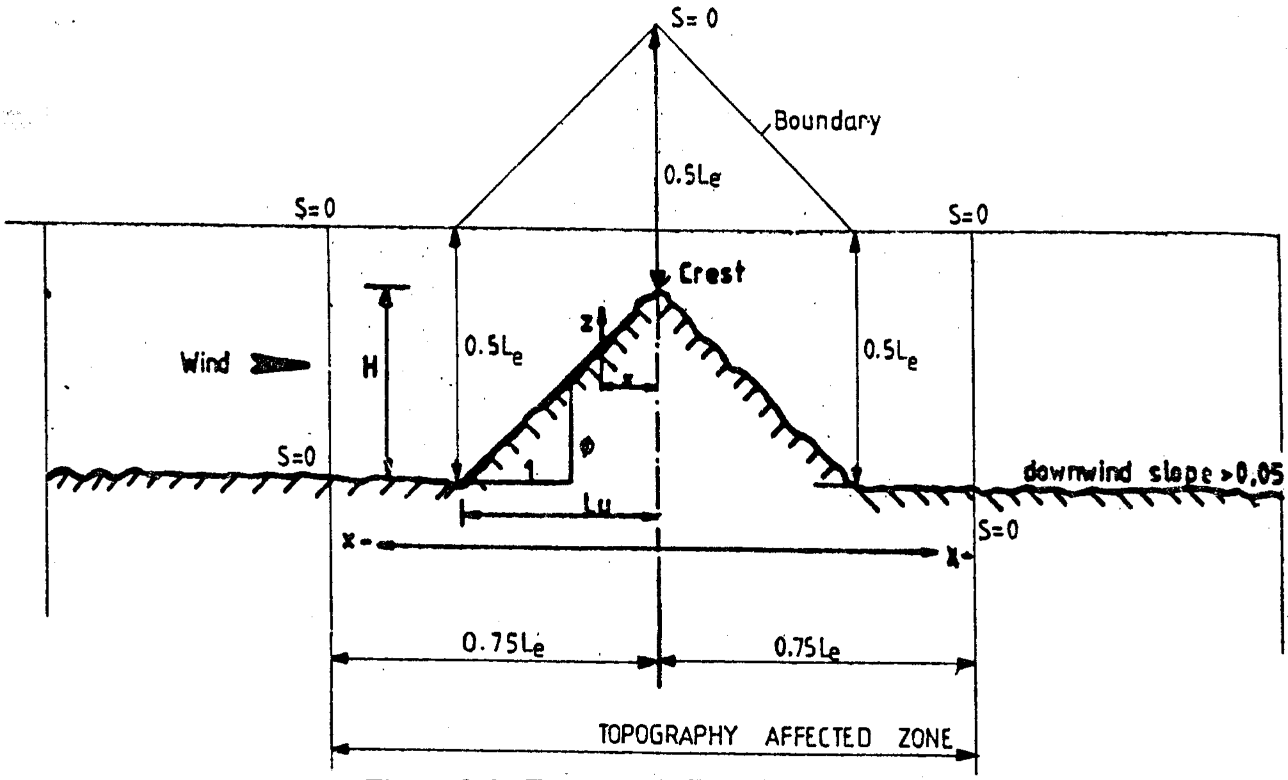

Figure 3.6 Factor *s*: Hills and Ridges

3.8.5 Exposure Coefficient

(1) The exposure coefficient, $c_e(z)$ takes into account the effects of terrain roughness, topography and height above ground on the mean wind speed and turbulence. It is defined by:

$$c_e(z) = c^2_r(z)\ c^2_t\ (z)[1 + 2g\ I_V\ (z)] \tag{3.13}$$

where g is the peak factor
$I_V\ (z)$ is the turbulence intensity, given by:

$$I_v\ (z) = \frac{k_T}{c_r\ (z)\cdot c_t(z)} \tag{3.14}$$

(2) For codification purposes it has been assumed that the quasi-static gust load is determined from:

$$c_e(z) = c_r^2\ (z)\ c_t^2\ (z)\ \left[1 + \frac{7\ k_T}{c_r(z)\ c_t(z)}\right] \tag{3.15}$$

where k_T is the terrain factor as defined in Section 3.8.2
$c_r(z)$ is the roughness coefficient as defined in Section 3.8.3
$c_t(z)$ is the topography coefficient as defined in Section 3.8.4

(3) The exposure coefficient $c_e(z)$is given in Table 3.5 for each terrain category defined in Section 3.8.2.

(4) For structures which need to be designed by a detailed dynamic analysis method, the simplification in (2) above is not used.

3.9 CHOICE OF PROCEDURES

3.9.1 General

(1) Two procedures for calculating wind loads are required.

(a) the simple procedure of this Code applies to those structures whose structural properties do not make them susceptible to dynamic excitation. This procedure can also be used for the design of mildly dynamic structures by the use of the dynamic coefficient C_d. The value of this coefficient depends upon the type of structure (concrete, steel, composite), the height of the structure and its breadth.

(b) a detailed dynamic analysis procedure is required for those structures which are likely to be susceptible to dynamic excitation and for which the value of the dynamic coefficient C_d is greater than 1.2.

(2) The dynamic coefficient c_d takes into account the reduction effects due to the lack of correlation of pressures over surfaces as well as the magnification effects due to the frequency content of turbulence close to the fundamental frequency of the structure.

(3) Section 3.9.2 defines the field of application of this section, and the criteria for choosing between simple and detailed procedures.

(4) Section 3.9.3 sets down the values of c_d for use with the simple procedure (in-wind response).

(5) Section 3.9.4 gives criteria for vortex shedding and galloping.

3.9.2 Criteria for the Choice

(1) The simple procedure may be used for buildings and chimneys less than 200m tall provided the value of c_d (see Section 3.9.3) is less than 1.2 (in-wind response). In all other cases a detailed dynamic analysis is required in accordance with specialist literature.

3.9.3 Dynamic Coefficient for Gust Wind Response

(1) Values of c_d set out in Figs. 3.7 to 3.13 are based on typical values of the relevant parameters and simplified equations for natural frequencies of structures.

(2) Values of c_d for buildings are set out in Figs.3. 7 to 3.9 depending on the material of construction.

(3) Values of c_d for chimneys are set out in Figs. 3.10 to 3.13 depending on the form of construction.

(4) For values of $1.0 \leq c_d \leq 1.2$ it is recommended that a detailed procedure should be used.

Table 3.5 Exposure Cofficient c_e.

CATEGORY I								
	z(m)							
c_t	2	4	8	16	30	50	100	200
1.0	1.88	2.25	2.64	3.07	3.47	3.82	4.31	4.84
1.1	2.16	2.59	3.05	3.55	4.02	4.43	5.02	5.63
1.2	2.45	2.95	3.48	4.06	4.61	5.09	5.77	6.49
1.3	2.76	3.33	3.94	4.60	5.24	5.78	6.57	7.39
1.4	3.09	3.73	4.42	5.17	5.90	6.52	7.41	8.36
1.5	3.43	4.15	4.93	5.78	6.60	7.30	8.31	9.38
CATEGORY II								
	z(m)							
c_t	2	4	8	16	30	50	100	200
1.0	1.80	1.80	2.21	2.66	3.09	3.47	4.01	4.58
1.1	2.06	2.06	2.54	3.06	3.57	4.00	4.64	5.31
1.2	2.33	2.33	2.88	3.48	4.07	4.58	5.31	6.09
1.3	2.61	2.61	3.24	3.92	4.60	5.18	6.02	6.92
1.4	2.91	2.91	3.62	4.40	5.16	5.82	6.78	7.80
1.5	3.22	3.22	4.02	4.89	5.75	6.49	7.57	8.73
CATEGORY III								
	z(m)							
c_t	2	4	8	16	30	50	100	200
1.0	1.63	1.63	1.63	2.11	2.59	3.00	3.60	4.25
1.1	1.86	1.86	1.86	2.41	2.96	3.44	4.14	4.90
1.2	2.09	2.09	2.09	2.72	3.35	3.90	4.71	5.59
1.3	2.33	2.33	2.33	3.04	3.76	4.39	5.32	6.32
1.4	2.58	2.58	2.58	3.39	4.20	4.91	5.96	7.09
1.5	2.84	2.84	2.84	3.74	4.65	5.45	6.63	7.91
CATEGORY IV								
	z(m)							
c_t	2	4	8	16	30	50	100	200
1.0	1.56	1.56	1.56	1.56	2.04	2.46	3.08	3.75
1.1	1.77	1.77	1.77	1.77	2.31	2.80	3.52	4.31
1.2	1.98	1.98	1.98	1.98	2.61	3.16	3.99	4.89
1.3	2.20	2.20	2.20	2.20	2.91	3.54	4.48	5.51
1.4	2.43	2.43	2.43	2.43	3.23	3.94	4.99	6.16
1.5	2.67	2.67	2.67	2.67	3.56	4.35	5.53	6.84

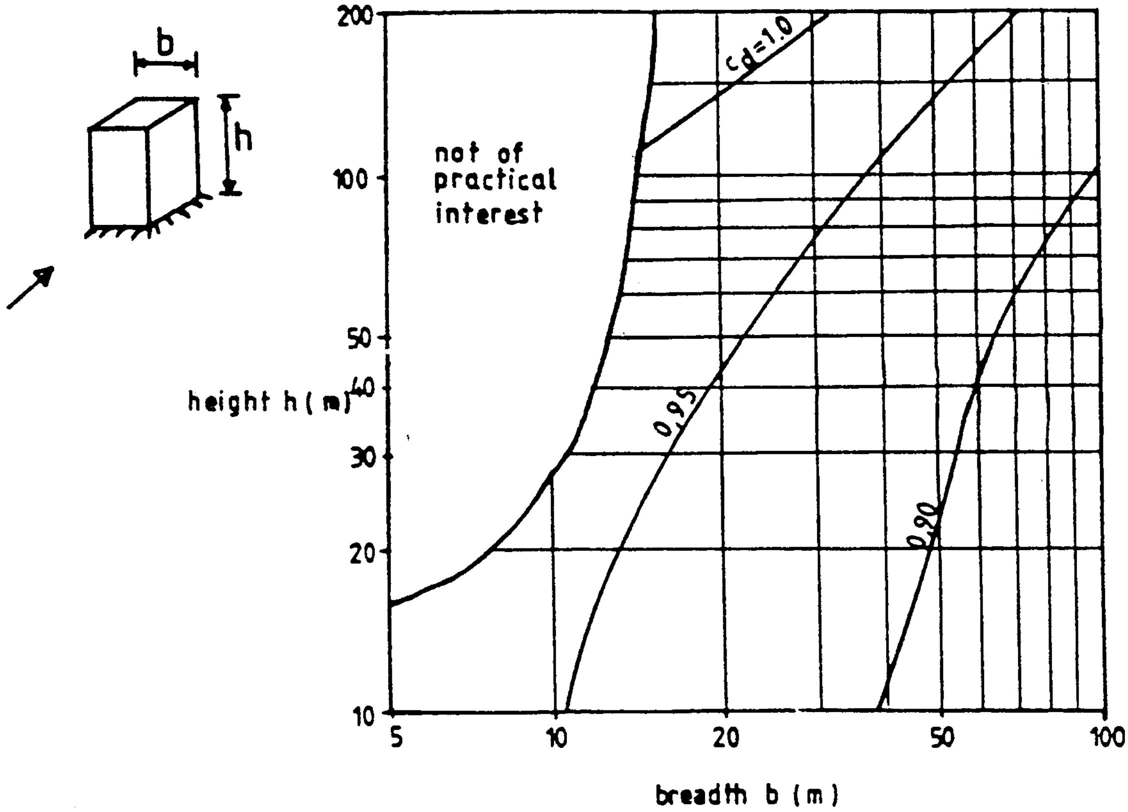

Figure 3.7 C_d Values for Concrete and Masonry Buildings

Note: The criteria set down in this figure do not address comfort conditions at serviceability. If this is likely to be of concern, more detailed procedures should be used.

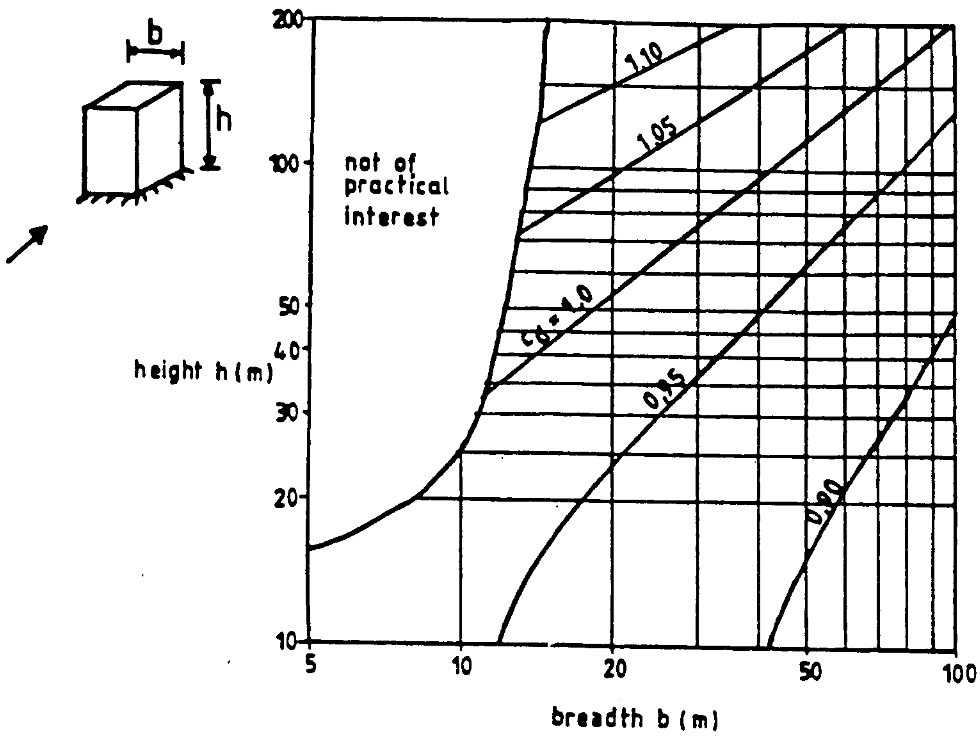

Figure 3.8 C_d Values for Steel Buildings

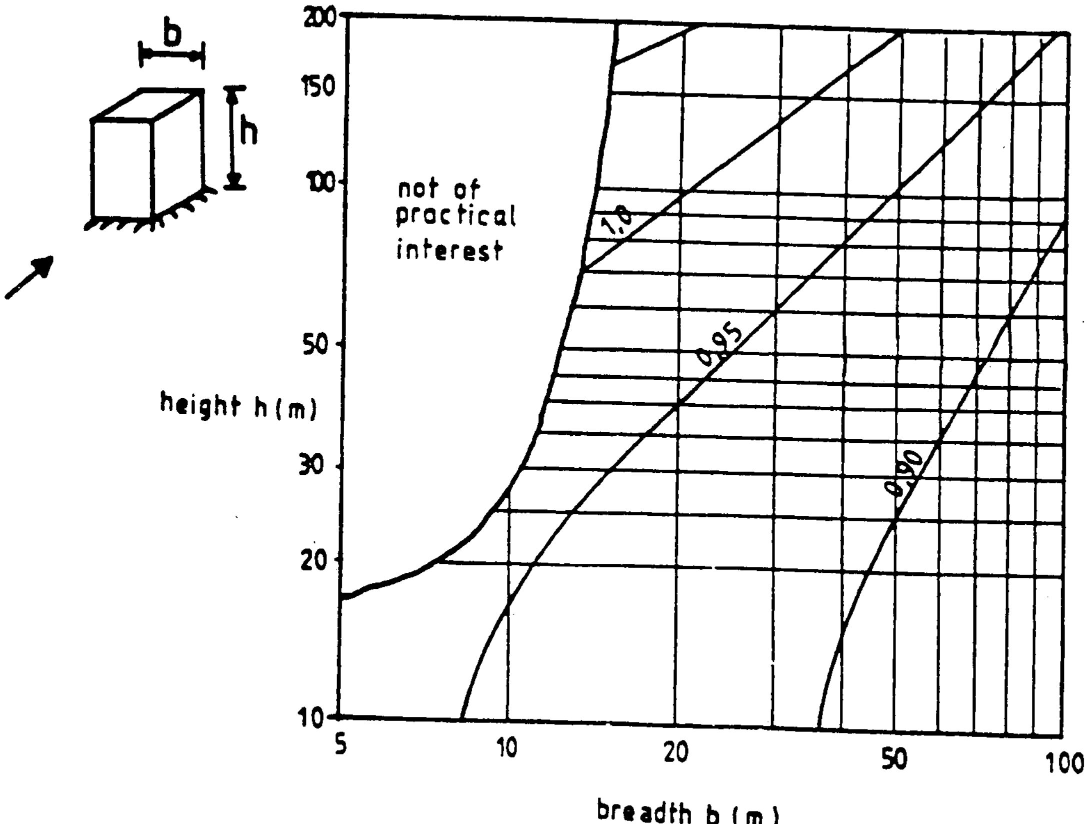

Figure 3.9 C_d Values for Composite (Steel/Concrete) Buildings

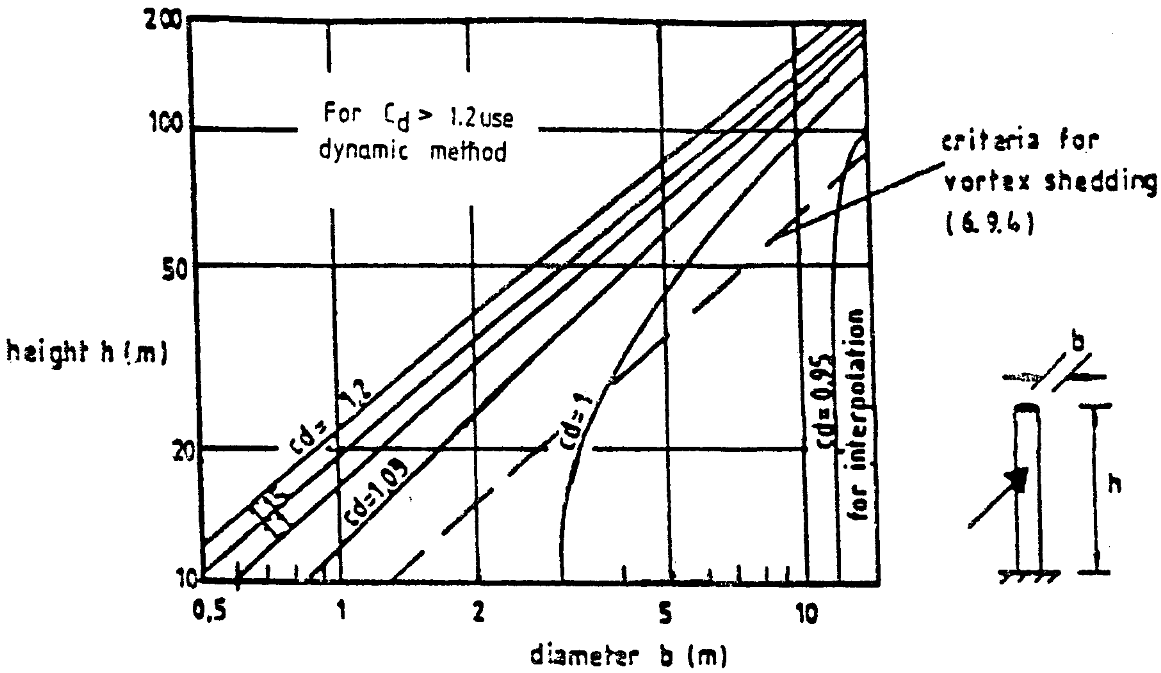

Figure 3.10 C_d Values for Unlined Welded Steel Chimneys

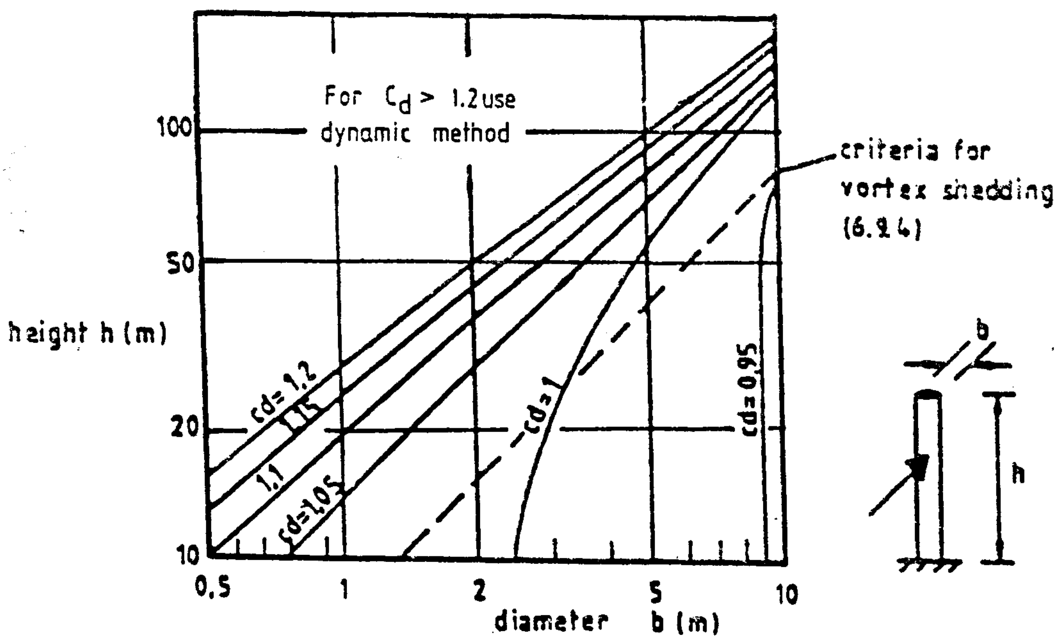

Figure 3.11 C_d Values for Lined Steel Chimneys

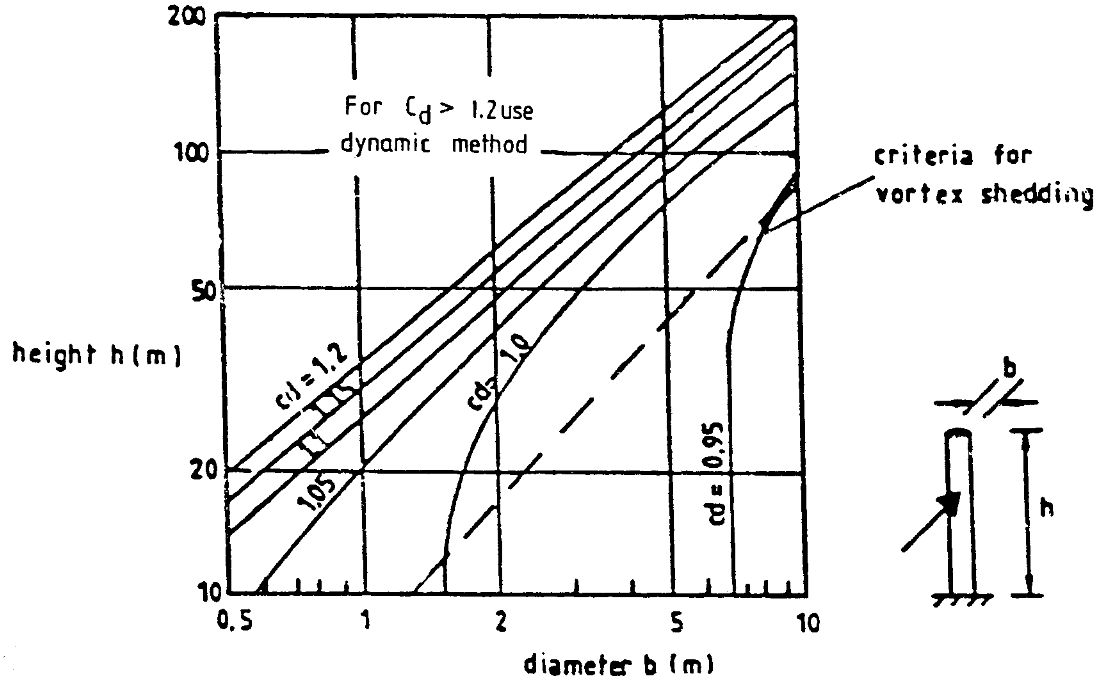

Figure 3.12 C_d Values for Brick Lined Steel Chimney

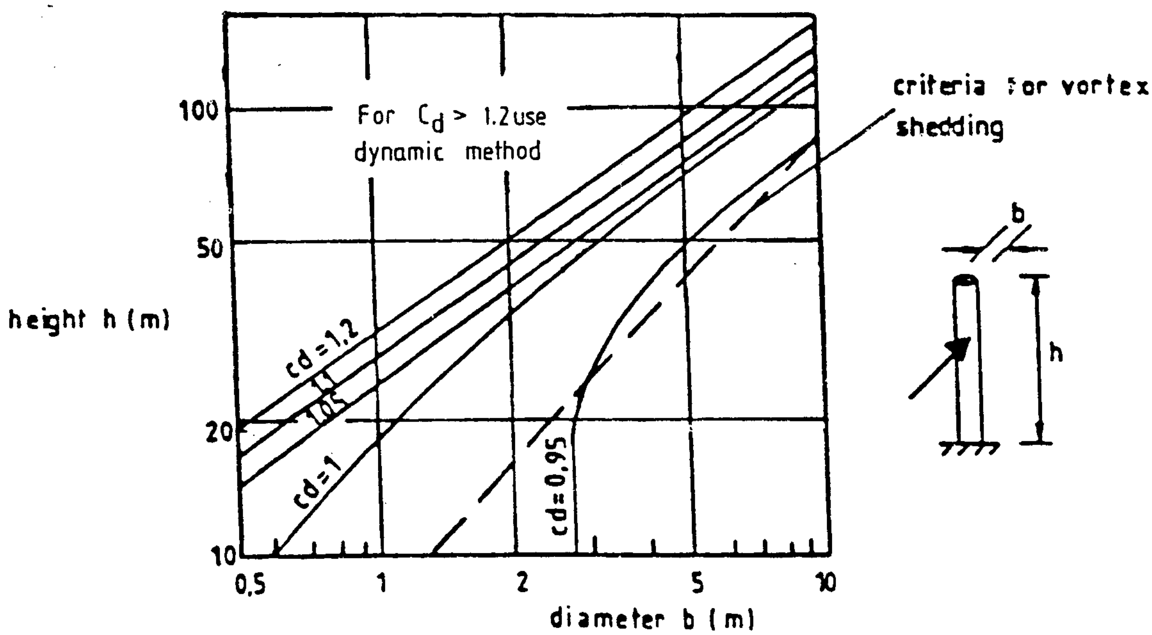

Figure 3.13 C_d Vlaues for Reinforced Concrete Chimney

3.9.4 Vortex Shedding, Aeroelastic Instability and Dynamic Interference Effects

3.9.4.1 *General*

(1) For slender structures the following phenomena of dynamics and instability effects have to be considered:

(a) vortex shedding
(b) galloping
(c) flutter
(d) divergence
(e) interference galloping

(2) Rules for analyzing such phenomena may be obtained from specialist literature.

(3) Criteria for the field of application of vortex shedding and galloping are given in Section 3.9.4.2.

3.9.4.2 *Field of Application*

(1) Buildings whose geometric dimensions satisfy the criteria given in fig. 3.14 need not be checked for vortex shedding and galloping. Buildings which do not satisfy these criteria shall be checked for vortex shedding and galloping.

(2) Elongated structures, such as chimneys, whose geometric dimensions satisfy the criteria given in figs. 3.10 to 3.13 and need not to be checked for vortex shedding, galloping, flutter and interference galloping. Such structures which do not satisfy these criteria shall be checked for these phenomena.

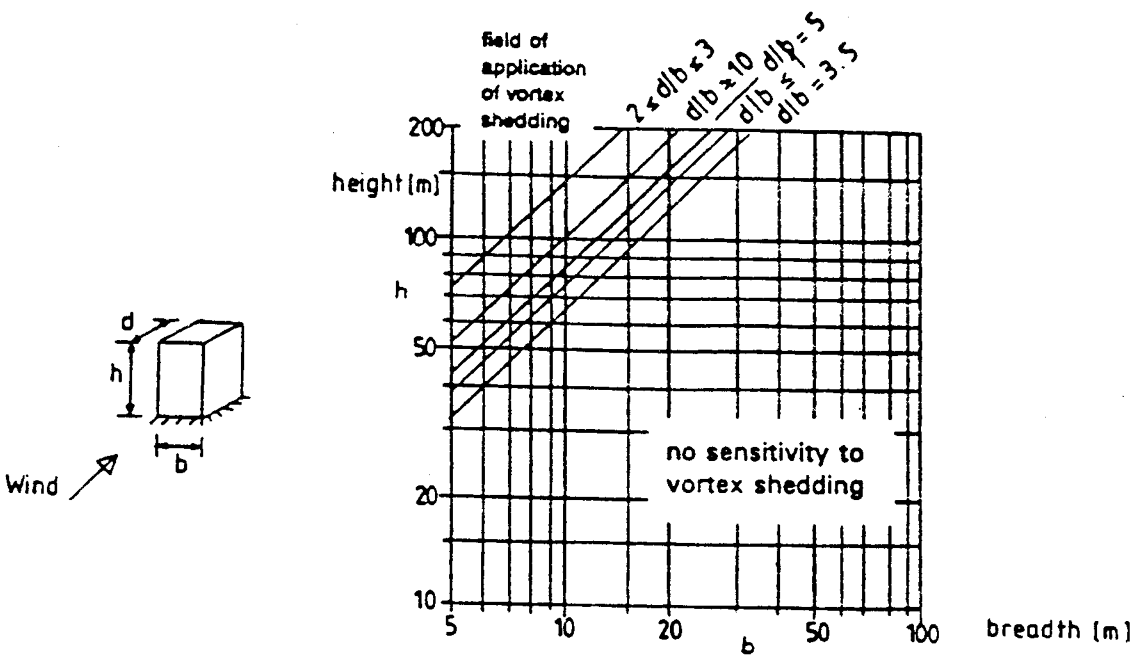

Figure 3.14 Ciriteria for Buildings for Vortex Shedding and Galloping

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Appendix A
Aerodynamic Coefficients

A.1 GENERAL

(1) This section presents the aerodynamic coefficients of the following structures, structural elements and components:

- Buildings (Section A.2)
- Canopy roofs (Section A.3)
- Free-standing boundary walls, fences and signboards (Section A.4)
- Structural elements with rectangular Section (Section A.5)
- Structural elements with sharp edged Section (Section A.6)
- Structural elements with regular polygonal Section (Secion A.7)
- Circular cylinders (Section A.8)
- Spheres (Section A.9)
- Lattice structures and scaffoldings (Section A.10)
- Friction coefficients (Section A.11)
- Effective slenderness and slenderness reduction factor (Section A.12)

A.2 BUILDINGS

A.2.1 General

(1) The external pressure coefficients c_{pe} for buildings and individual parts of buildings depend on the size of the loaded area A. They are given for loaded areas A of $1m^2$ and $10m^2$ in the relevant tables for the appropriate building configurations as $c_{pe.1}$ and $c_{pe.10}$ respectively. For other loaded areas the variation of the values may be obtained from fig. A.1.

Note: The loaded area is the area of the structure, which produces the wind action in the section to be calculated.

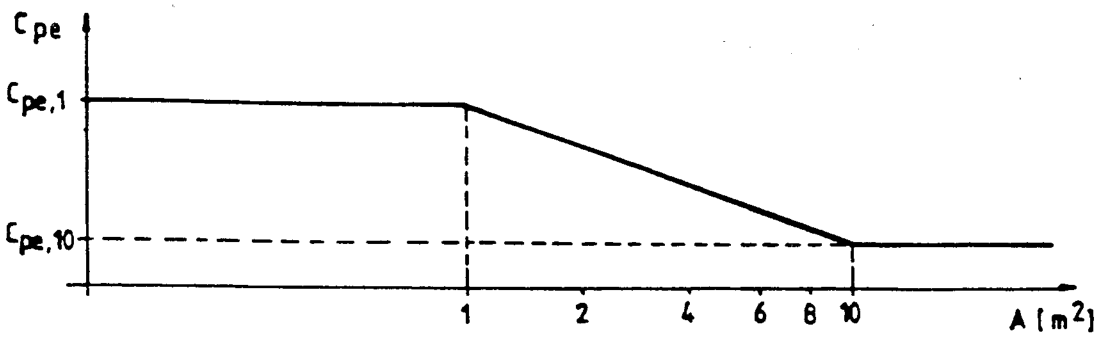

Figure A.1 Variation of External Pressure Coefficient for Buildings with Size of the Loaded Area *A*.

Note: The Figure is based on the following:

$c_{pe} = c_{pe,1}$ $\qquad A \leq 1m^2$

$c_{pe} = c_{pe,1} + (c_{pe,10} - c_{pe,1})\log_{10}A$ $\qquad 1m^2 < A < 10m^2$

$A \geq 10m^2$ $\qquad c_{pe} = c_{pe,10}$

Appendix A
Aerodynamic Coefficients

A.1 GENERAL

(1) This section presents the aerodynamic coefficients of the following structures, structural elements and components:

- Buildings (Section A.2)
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- Structural elements with rectangular Section (Section A.5)
- Structural elements with sharp edged Section (Section A.6)
- Structural elements with regular polygonal Section (Secion A.7)
- Circular cylinders (Section A.8)
- Spheres (Section A.9)
- Lattice structures and scaffoldings (Section A.10)
- Friction coefficients (Section A.11)
- Effective slenderness and slenderness reduction factor (Section A.12)

A.2 BUILDINGS

A.2.1 General

(1) The external pressure coefficients c_{pe} for buildings and individual parts of buildings depend on the size of the loaded area A. They are given for loaded areas A of $1m^2$ and $10m^2$ in the relevant tables for the appropriate building configurations as $c_{pe.1}$ and $c_{pe.10}$ respectively. For other loaded areas the variation of the values may be obtained from fig. A.1.

Note: The loaded area is the area of the structure, which produces the wind action in the section to be calculated.

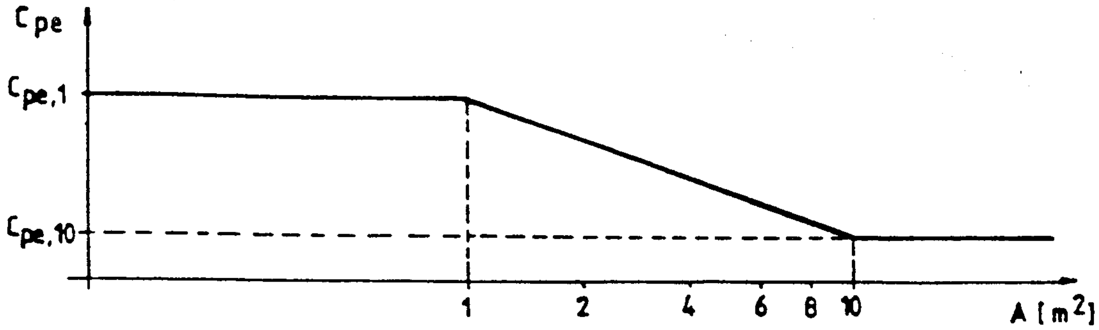


Figure A.1 Variation of External Pressure Coefficient for Buildings with Size of the Loaded Area A.

Note: The Figure is based on the following:

$c_{pe} = c_{pe,1}$	$A \leq 1m^2$
$c_{pe} = c_{pe,1} + (c_{pe,10} - c_{pe,1})\log_{10}A$	$1m^2 < A < 10m^2$
$A \geq 10m^2$	$c_{pe} = c_{pe,10}$

(2) The values $c_{pe,10}$ and $c_{pe,1}$ in Tables A.1 to A.5 are given for orthogonal wind directions 0°, 90°, 180° but represent highest values obtained in a range of wind direction $\theta = \pm 45°$ either side of the relevant orthogonal direction.

(3) These values are only applicable to buildings.

A.2.2 Vertical Walls of Rectangular Plan Buildings

(1) The reference height, z_e, for walls of rectangular plan buildings depends on the aspect ratio h/b and is given in Fig. A.2 for the following three cases.

(a) Buildings, whose height h is less than b, shall be considered to be one part.
(b) Buildings, whose height h is greater than b, but less than $2b$, shall be considered to be two parts, comprising: a lower part extending upwards from the ground by a height equal to b and an upper part.
(c) Buildings, whose height h is greater than $2b$, shall be considered to be in 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, divided into as many horizontal strips with a maximum height of b as desired.

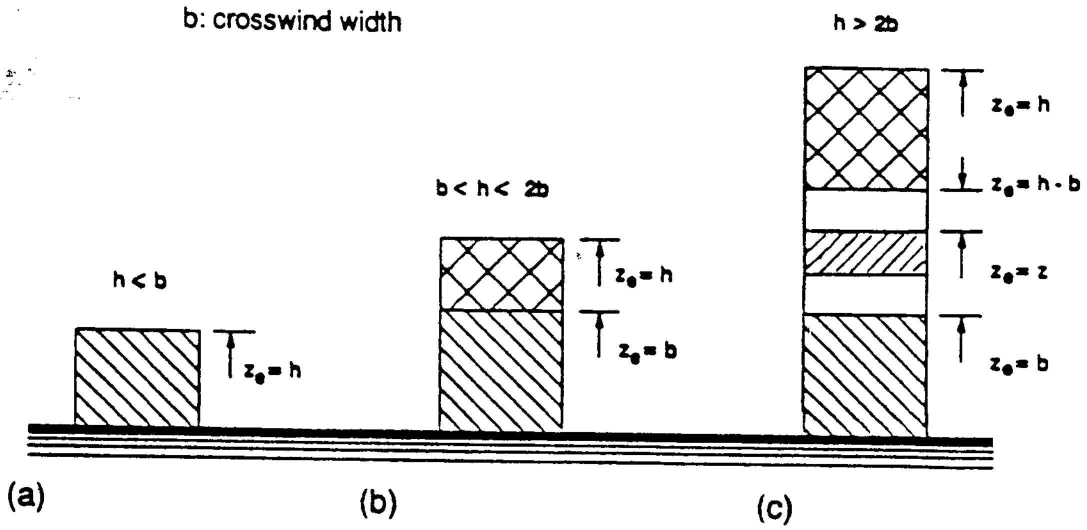


Figure A.2 Reference Height Z_e Depending on h and b.

(2) The external pressure coefficients $c_{pe,10}$ and $c_{pe,1}$ for zone A, B, C, D, and E defined in Fig. A.3 are given in Table A.1 depending on the ratio d/h. Intermediate values may be interpolated linearly.

(3) Friction forces should be considered only for long buildings (see Section 3.6.2).

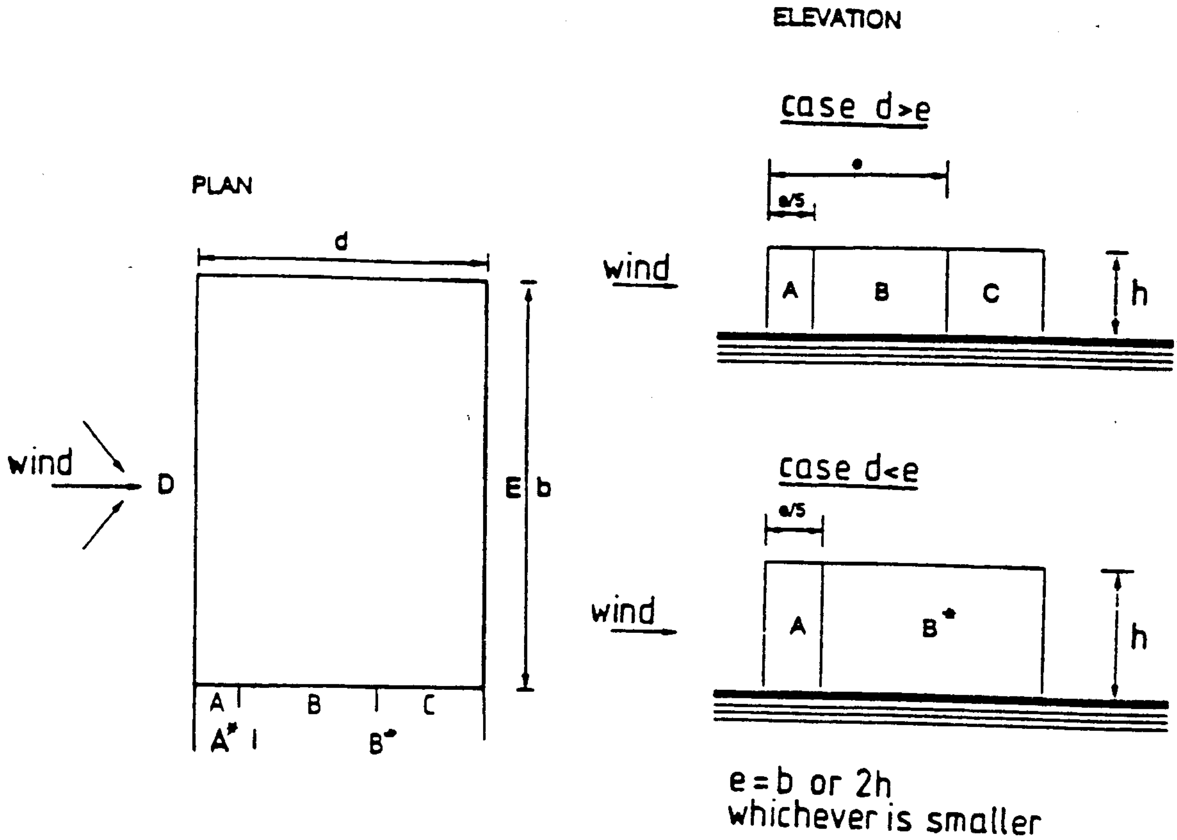


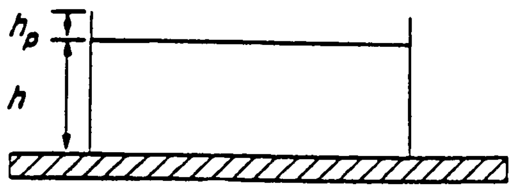
Figure A.3 Key for Vertical Walls

Table A.1 External Pressure Coefficients for Vertical Walls of Rectangular Plan Buildings

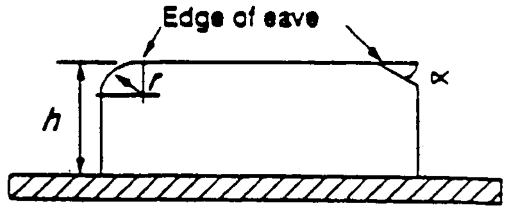
Zone	A		B,B*		C		D		E	
d/h	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
≤ 1	- 1.0	- 1.3	- 0.8	- 0.1	- 0.5		+0.8	+1.0	- 0.3	
≥ 4	- 1.0	- 1.3	- 0.8	- 0.1	- 0.5		+0.6	+1.0	- 0.3	

A.2.3 *Flat roofs*

(1) Flat roofs are defined within a slope of $\pm 4°$.
(2) The roof should be divided into zones as shown in Fig. A.4
(3) The reference height z_e should be taken as h.
(4) Pressure coefficients for each zone are given in Table A.2.
(5) For long roofs friction forces should be considered (see Section 3.6.2).



Parapets



Curved and mansard eaves

reference height :
$z_e = h$

$e = b$ or $2h$
whichever is smaller

b : crosswind dimension

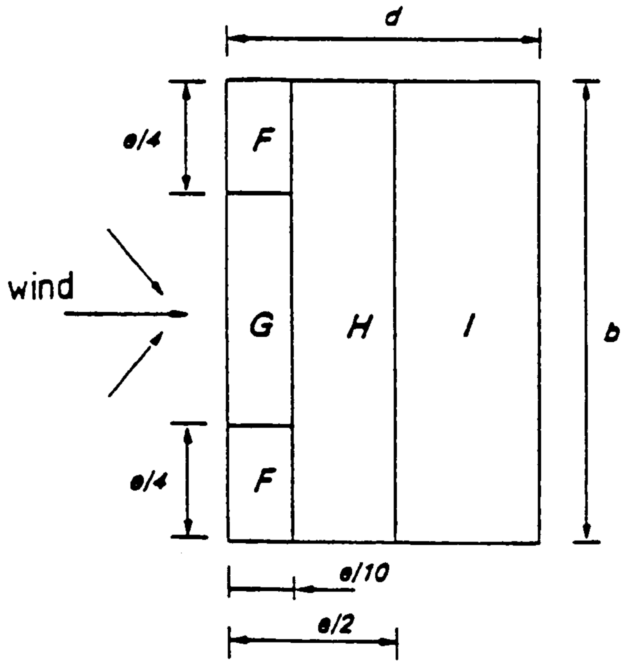


Figure A.4 Key for Flat Roofs

Table A.2 External Pressure Coefficients for flat roofs

		Zone							
		F		G		H			I
		$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$
Sharp eaves		- 1.8	- 2.5	- 1.2	- 2.0	- 0.7	- 1.2	± 0.2	
with parapets	$H_p/h = 0.025$	- 1.6	- 2.2	- 1.1	- 1.8	- 0.7	- 1.2	± 0.2	
	$H_p/h = 0.05$	- 1.4	- 2.0	- 0.9	- 1.6	- 0.7	- 1.2	± 0.2	
	$H_p/h = 0.10$	- 1.2	- 1.8	- 0.8	- 1.4	- 0.7	- 1.2	± 0.2	
Curved eaves	$r/h = 0.05$	- 1.0	- 1.5	- 1.2	- 1.8	- 0.4		± 0.2	
	$r/h = 0.10$	- 0.7	- 1.2	- 0.8	- 1.4	- 0.3		± 0.2	
	$r/h = 0.20$	- 0.5	- 0.8	- 0.5	- 0.8	- 0.3		± 0.2	
mansard eaves	$\alpha = 30°$	- 1.0	- 1.5	- 1.0	- 1.5	- 0.3		± 0.2	
	$\alpha = 45°$	- 1.2	- 1.8	- 1.3	- 1.9	- 0.4		± 0.2	
	$\alpha = 60°$	- 1.3	- 1.9	- 1.3	- 1.9	- 0.5		± 0.2	

Notes:

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

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

(iii) In Zone I, where positive and negative values are given, both values shall be considered.

(iv) For the mansard eave itself, the external pressure coefficients are given in Table 3.2.4 "External pressure coefficients for duopitch roofs: wind direction 0" Zone F and G, depending on the pitch angle of the mansard eave.

(V) 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.

A.2.4 Monopitch Roofs

(1) The roof should be divided into zones as shown in Fig. A.5

(2) The reference height z_e should be taken as h.

(3) Pressure coefficients for each zone are given in Table A.3

(4) For long roofs friction forces should be considered (see Section 3.6.2).

(5) For elongated roof corners (see Fig. A.5) the zone R is under the same pressure as the corresponding vertical wall. This rule is also applicable for roofs of other types.

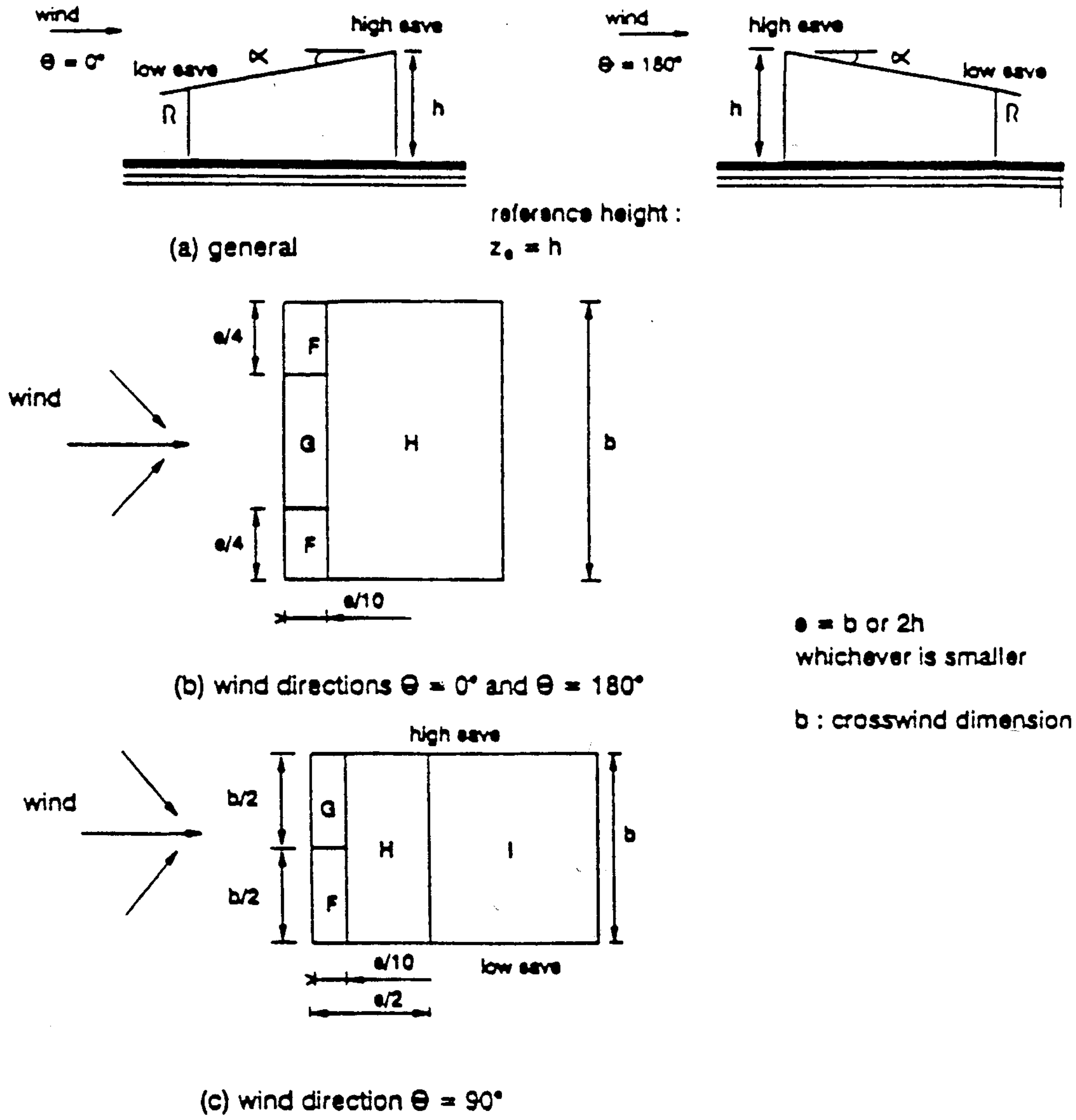


Figure A.5 Key for Monopitch Roofs

Table A.3 External Pressure Coefficients for Monopitch Roofs

Pitch angle a	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
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°	+ 0.7		+ 0.7		+ 0.6		- 0.6	- 1.3	- 0.5		- 0.7	
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 a	Zone for wind direction $\theta = 90°$							
	F		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	- 1.6	- 2.2	- 1.8	- 2.0	- 0.6	- 1.2	0.5	
15°	- 1.3	- 2.0	- 1.9	- 2.5	- 0.8	- 1.2	- 0,7	- 1.2
30°	- 1.2	- 2.0	- 1.5	- 2.0	- 1.0	- 1.3	- 0,8	- 1.2
45°	- 1.2	- 2.0	- 1.4	- 2.0	- 1.0	- 1.3	- 0,9	- 1.2
60°	- 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	- 1.0	- 1.3	- 0.5	

Note: (i) At $\theta = 0°$ the pressure changes rapidly between positive and negative values around a pitch angle of $1 = +15°$ to $+ 30°$, so both positive and negative values are given.
(ii) Linear interpolation for intermediate pitch angles may be used between values of same sign.

A.2.5 Duopitch Roofs

(1) The roof should be divided into zones as shown in Fig. A.6

(2) The reference height z_e should be taken as h.

(3) The pressure coefficients for each zone are given in Table A.4

(4) For long roofs friction forces should be considered (see Section 3.6.2)

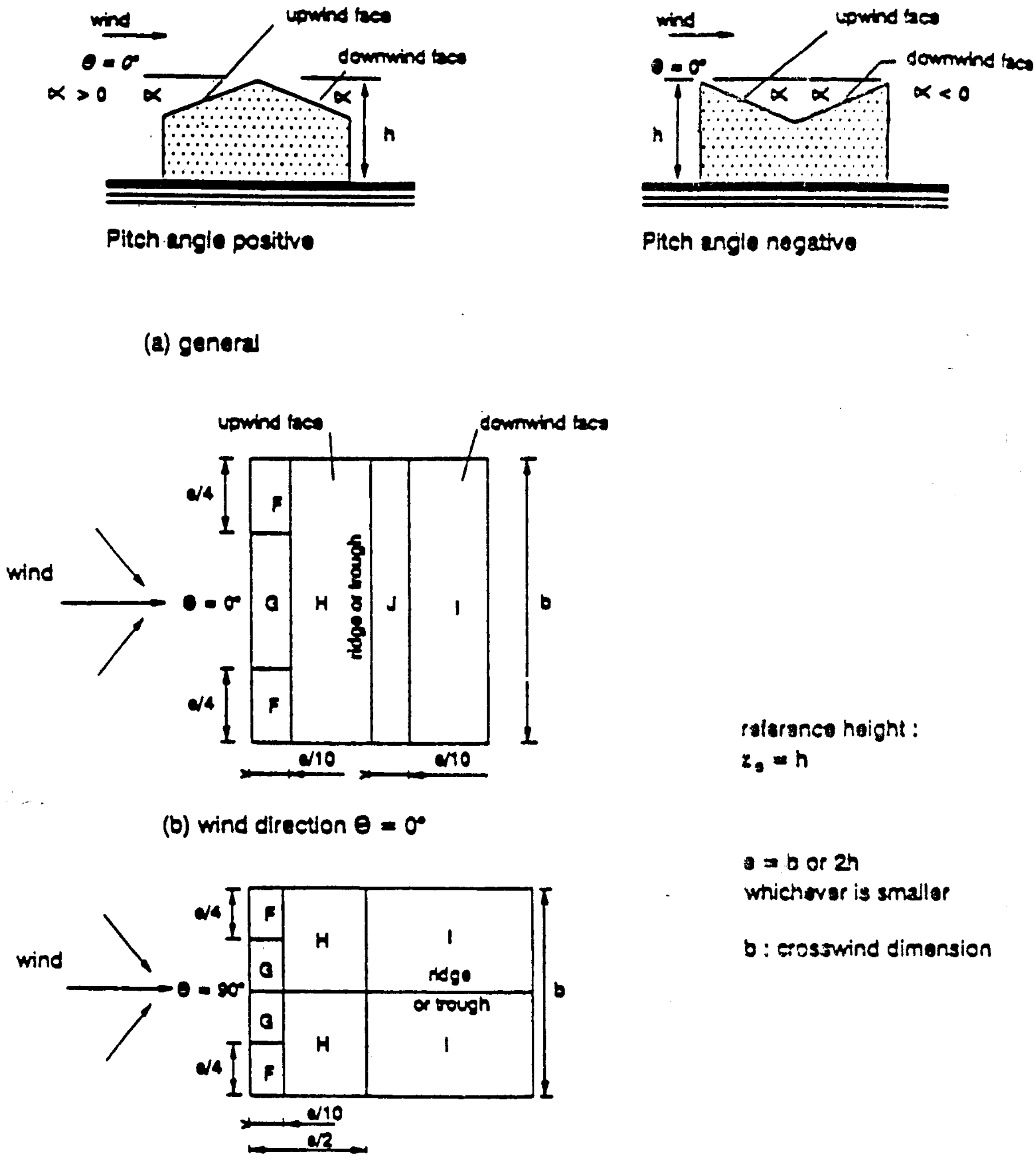


Figure A.6 Key for Duopitch Roofs

Table A.4 External Pressure Coefficients for Duopitch Roofs

	Zone for wind direction $\theta = 0^\circ$									
Pitch	F		G		H		I		J	
angle α	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$
- 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	- 0.3		- 0.3	
5°	- 1.7	- 2.5	- 1.2	- 2.0	- 0.6	- 1,2	- 0.3		- 0.3	
15°	- 0.9	- 2.0	- 0.8	- 1.5	- 0.3		- 0.4		- 1.0	- 1.5
	+ 0.2		+ 0.2		+ 0.2					
30°	- 0.5	- 1.5	- 0.5	- 1.5	- 0.2		- 0.4		- 0.5	
	+ 0.7		+ 0.7		+ 0.4					
45°	+ 0.7		+ 0.7		+ 0.6		- 0.2		- 0.3	
60°	+ 0.7		+ 0.7		+ 0.7		- 0.2		- 0.3	
75°	+ 0.8		+ 0.8		+ 0.8		- 0.2		- 0.3	

	Zone for wind direction $\theta = 90^\circ$							
	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	- 0.5	
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: (i) At $\theta = 0^\circ$ the pressure changes rapidly between positive and negative values on the windward face around a pitch angle of $1 = +15^\circ$ to $+ 30^\circ$, so both positive and negative values are given.

(ii) Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. (Do not interpolate between $\alpha = +5^\circ$ and $\alpha = -5^\circ$, but use the data for flat roofs in Section A.6.

A.2.6 Hipped Roofs

(1) The roof should be divided into zones as shown in Fig. A.7.

(2) The reference height z_e should be taken as h.

(3) The pressure coefficients are given in Table A.5.

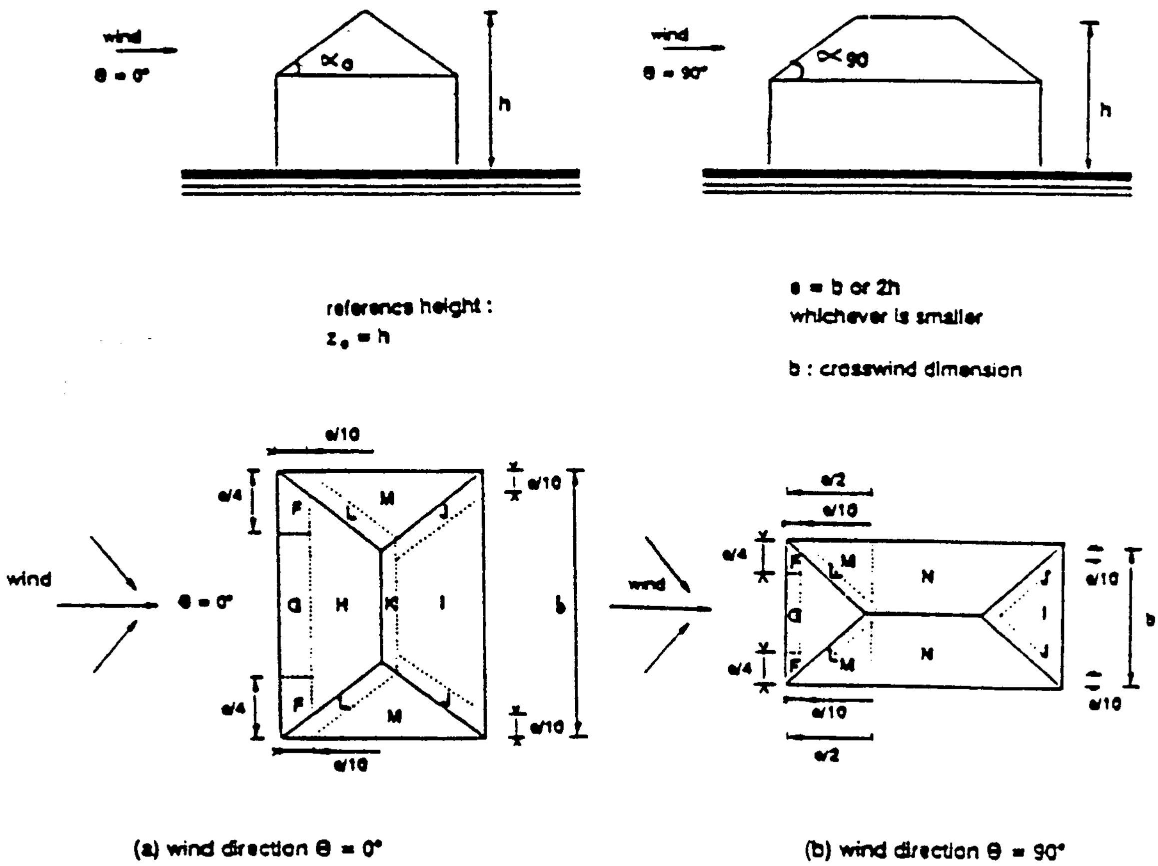


Figure A.7 Key for Hipped Roofs

A.2.7 Multispan Roofs

(1) Pressure coefficients for each span of multispan roofs should be derived from Section A.2.4 for monopitch roofs modified for their position according to Fig. A.8.

(2) The reference height z_e should be taken as h.

(3) For long roofs friction forces should be considered (see Section 3.6.2).

Table A.5 External Pressure Coefficients for Hipped Roofs of Buildings

Pitch angle α_o = for $\theta = 0$ α_{90} for $\theta = 90^\circ$	Zone for wind direction $\theta = 90^\circ$																	
			G		*H*		*I*		*J*		*K*		*L*		*M*		*N*	
	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$	$C_{pe,10}$	$C_{pe,1}$
5°	-1.7	-2.5	-1.2	-2.0	-0.6	-1.2	-0.3		-0.6		0.6		-1.2	-2.0	-0.6	-1.2	-0.4	
+ 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°	+0.7		+0.7		+0.6		-0.3		0.6		-0.3		-1.3	-2.0	-0.8	-1.2	-0.2	
+ 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:

(i) At $\theta = 0^\circ$ the pressure changes rapidly between positive and negative values on the windward face at pitch angle of 1 = 15° to +30°, so both positive and negative values are given.

(ii) Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. (do not interpolate between $\alpha = +5^\circ$ and $\alpha = -5^\circ$, but use the data for flat roofs in A.3.

(iii) The pitch angle of the windward face always will govern the pressure coefficients.

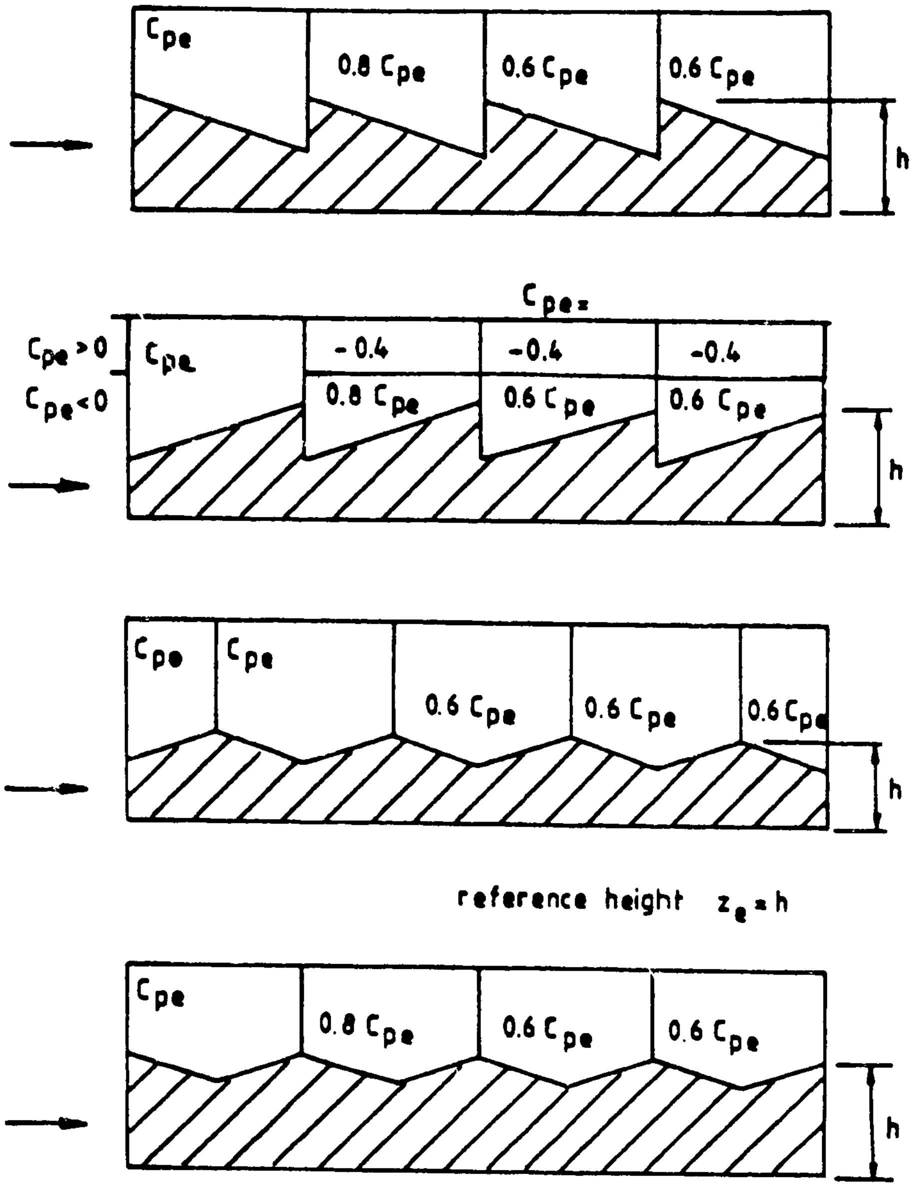


Figure A.8 Key to Multispan Roofs

A.2.8 Vaulted Roofs and Domes

(1) This section applies to circular cylindrical roofs and domes.

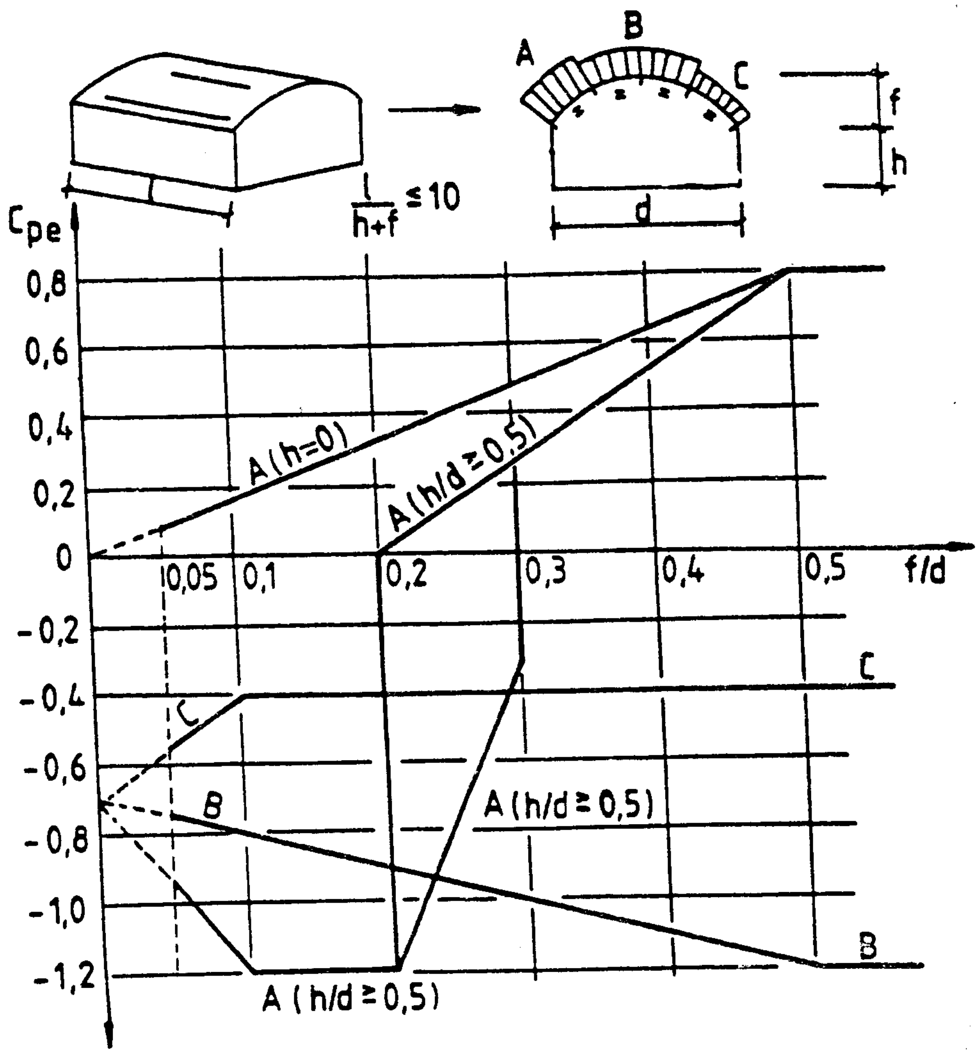
(2) The roof should be divided into zones as shown in Figs. A.9 and A.10.

(3) The reference height should be taken as:

$$z_e = h + f/2 \tag{A.1}$$

(4) The pressure coefficients are given in Fig. A.9 and Fig. A.10.

(5) Pressure coefficients for the walls should be taken from A.2.2.



Note (i) for $0 \leq h/d \leq 0.5$, $c_{pe,10}$ is obtained by linear interpolation
(ii) for $0.2 \leq f/d \leq 0.3$ and $h/d \geq 0.5$, two values of $c_{pe,10}$ have be considered
(iii) the diagram is not applicable for flat roofs

Figure A.9 External Pressure Coefficients for Vaulted Roofs with Rectangular Base and $l/(h + f) \leq 10$

Note: $c_{pe,10}$ is constant along arcs of circles, intersections of the sphere and of planes perpendicular to the wind: it can be determined as a first approximation by linear interpolation between the values in A, B and C along the arcs of circles parallel to the wind. In the same way the values of $c_{pe,10}$ in A if $0 < h/d < 1$ and in B or C if $0 < h/d < 0.5$ can be obtained by linear interpolation in the figure above.

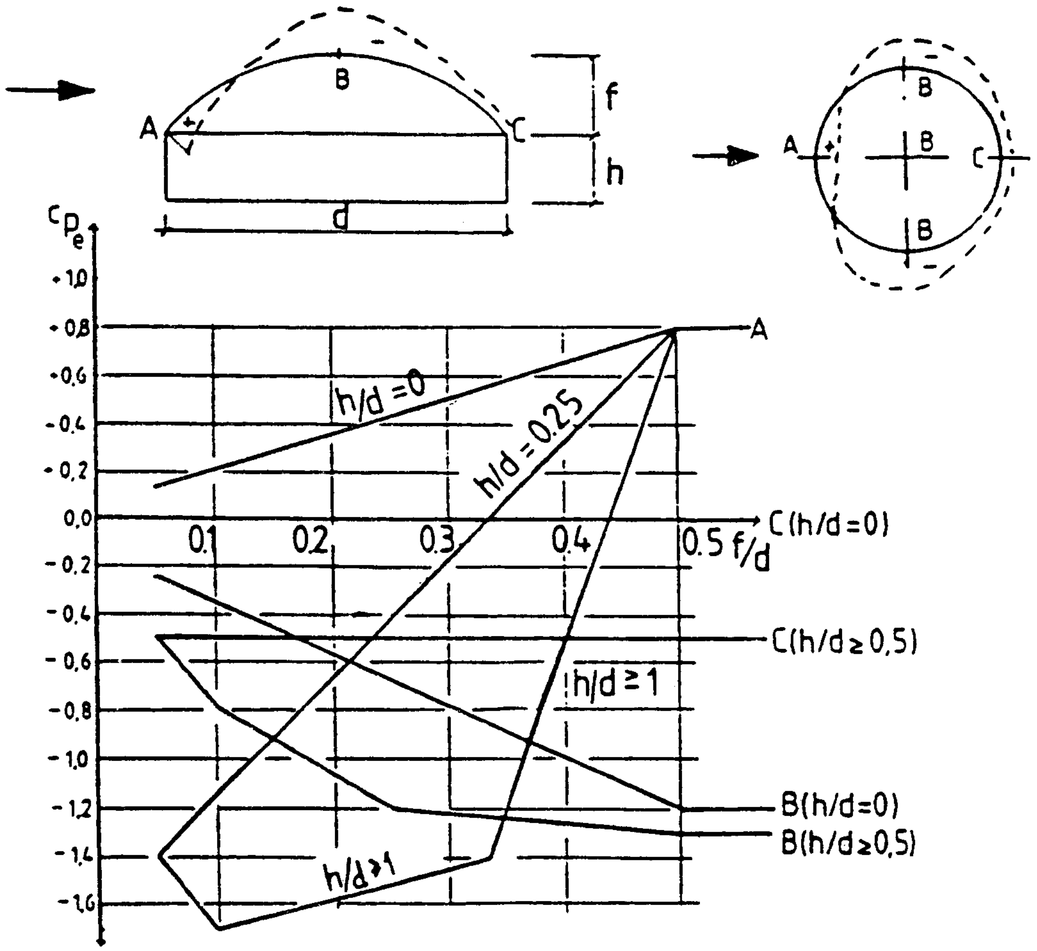


Figure A.10 External Pressure Coefficients $c_{pe,10}$ for Domes with Circular Base

A.2.9 Internal Pressure

(1) The internal pressure coefficient c_{pi} for buildings without internal partitions is given in Fig. A.11 and is a function of the opening ratio μ, which is defined as

$$\mu = \frac{\sum \text{ area of openings at the leeward and wind parallel sides}}{\sum \text{ area of openings at the winward, leeward and wind parallel sides}} \quad \text{(A.2)}$$

(2) The reference height z_i without internal partition and floors is the mean height of the openings with homogeneous distribution of height of the dominant opening. an opening is regarded as dominant, if the ratio of its area to that of the remaining openings is larger than 10.

(3) the reference height z_i for buildings without internal partitions but with compartmentation by internal floors is the mean height of the level considered.

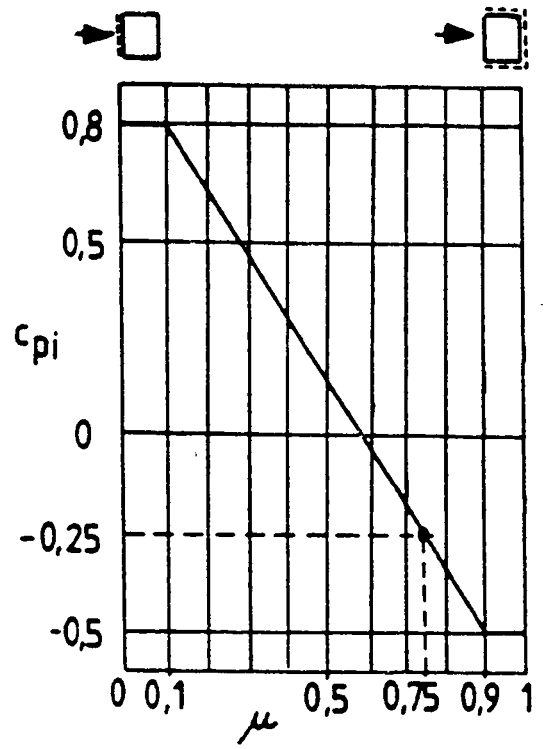


Figure A.11 Internal Pressure Coefficient c_{pi} for Buildings with Openings in the Walls

(4) For a homogeneous distribution of openings for a nearly square building the value c_{pi} = -0.25 shall be used.

(5) The worst values have to be considered for any combination of possible openings.

(6) For closed buildings with internal partitions and opening windows the extreme values:

$$c_{pi} = 0.8 \text{ or } c_{pi} = -0.5 \qquad \text{(A.3)}$$

(7) In figure the most intensive suction is assumed to be c_{pi} = -0.5 (lowest point of the curve). If one or more dominant openings exist in areas with more intensive suction than -0.5, then the curve continues down to the lower value.

(8) Internal and external pressures are considered to act at the same time.

(9) the internal pressure coefficient of open silos is:

$$c_{pi} = -0.8 \qquad \text{(A.4)}$$

The reference height z_j is equal to the height of the silos.

A.3 Canopy Roofs

(1) Canopy roofs are roofs of buildings, which do not have permanent walls, such as petrol station canopies, dutch barns, etc.

(2) The degree of blockage under the canopy is shown in Fig. A.12 It depends on the solidity ratio φ, which is the ratio of the area of possible obstructions under the canopy divided by the cross area under the canopy, being both areas normal to the wind direction. $\varphi = 0$ represents an empty canopy, $\varphi = 1$ represents the canopy fully blocked with contents to the down wind eaves only (this is not a closed building).

(3) The net pressure coefficients $c_{p,net}$ are given in Table A.7 to A.9 for $\varphi = 0$ and $\varphi = 1$. Intermediate values may be linearly interpolated.

(4) Downwind of the position of maximum blockage, $c_{p,net}$ are given in Table 10.3.1 to 10.3.3 for $\varphi = 0$ and $\varphi = 1$. Intermediate values may be linearly interpolated.

(5) The overall coefficient represents the resulting force. The local coefficient represents the maximum local force for different wind directions.

(6) Each canopy must be able to support the maximum (upward) loads as defined below:

(i) for monopitch canopy (Table A.7) the centre of pressure shall be taken at w/4 from the windward edge (w = alongwind dimension, Fig. A.13)

(ii) for duopitch canopy (Table A.8) the center of pressure shall be taken at the center of each slope (Fig. A.14.)
In addition, a duopitch canopy must be able to support one pitch with the maximum or minimum load, the other pitch being unloaded.

(iii) for multibay duopitch canopy each bay can be calculated by applying the reduction factors given in Table A.9 to the $c_{p.net}$ values given in Table A.8.

In case of double skin, the impermeable skin and its fixings shall be calculated with $c_{p.net}$ and the permeable skin and its fixings with 1/3 $c_{p.net}$.

(7) Friction forces should be considered (see Section 3.6.2).

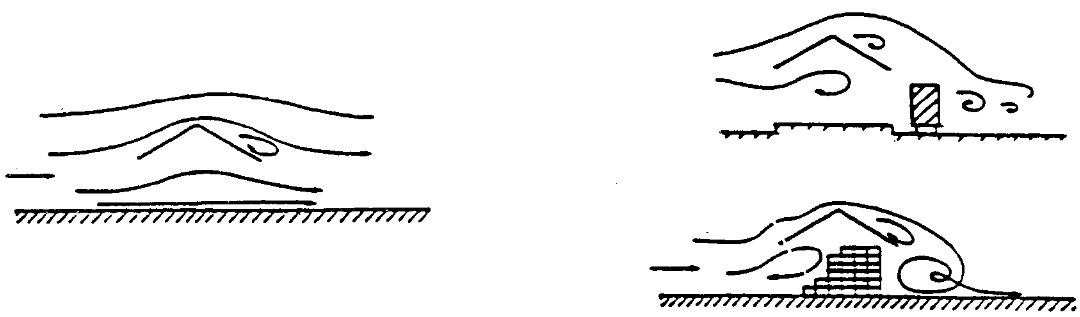
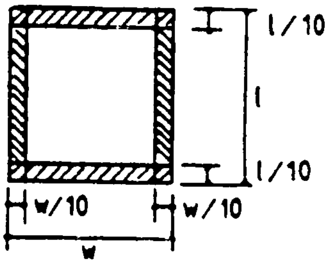
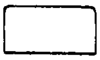
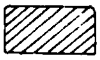
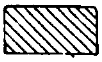


Figure A.12 Airflow over Canopy Roofs

(8) Loads on each slope of multibay canopies shown in Fig. A.15 are determined by applying the factors given in Table A.9 to the overall coefficients for isolated duo-pitch canopies.

Table A.7 $C_{p,net}$ Values for Monopitch Canopies

Roof angle α [deg.]	Blockage φ	Overall coefficients	Local coefficients 		
					
0	Minimum all φ Minimum $\varphi=0$ Minimum $\varphi=1$	+ 0.2 - 0.5 - 1.3	+ 0.5 - 0.6 - 1.5	+ 1.8 - 1.3 - 1.8	+ 1.1 - 1.4 - 2.2
5	Minimum all φ Minimum $\varphi=0$ Minimum $\varphi=1$	+ 0.4 - 0.7 - 1.4	+ 0.8 - 1.1 - 1.6	+ 2.1 - 1.7 - 2.2	+ 1.3 - 1.8 - 2.5
10	Minimum all φ Minimum $\varphi=0$ Minimum $\varphi=1$	+ 0.5 - 0.9 - 1.4	+ 1.2 - 1.5 - 2.1	+ 2.4 - 2.0 - 2.6	+ 1.6 - 2.1 - 2.7
15	Minimum all φ Minimum $\varphi=0$ Minimum $\varphi=1$	+ 0.7 - 1.1 - 1.4	+ 1.4 - 1.8 - 1.6	+ 2.7 - 2.4 - 2.9	+ 1.8 - 2.5 - 3.0
20	Minimum all φ Minimum $\varphi=0$ Minimum $\varphi=1$	+ 0.8 - 1.3 - 1.4	+ 1.7 - 2.2 - 1.6	+ 2.9 - 2.8 - 2.9	+ 2.1 - 2.9 - 3.0
25	Minimum all φ Minimum $\varphi=0$ Minimum $\varphi=1$	+ 1.0 - 1.6 - 1.4	+ 2.0 - 2.6 - 1.5	+ 3.1 - 3.2 - 2.5	+ 2.3 - 3.2 - 2.8
30	Minimum all φ Minimum $\varphi=0$ Minimum $\varphi=1$	+ 1.2 - 1.8 - 1.4	+ 2.2 - 3.0 - 1.5	+ 3.2 - 3.8 - 2.2	+ 2.4 - 3.6 - 2.7

Note (i) + down
- up

(ii) $z_{ref} = h$

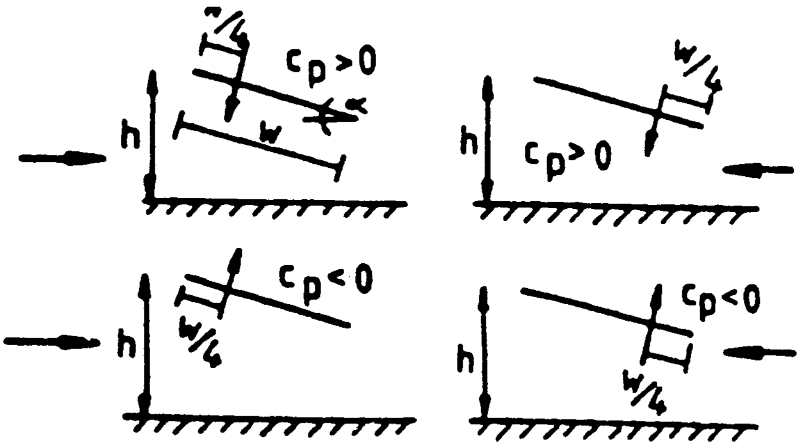
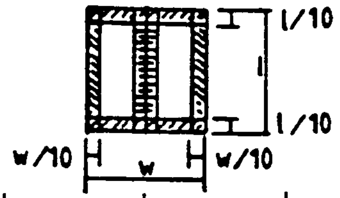
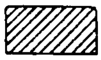


Figure A.13 Load Arrangements for Monopitch Canopies

Table A.8 $C_{p.net}$ Values for Duopitch Canopies

Roof angle α [deg.]	Blockage φ	Overall coefficients	Local coefficients  			
						
- 20	Minimum 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
- 15	Minimum 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
- 10	Minimum 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
- 5	Minimum 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
+ 5	Minimum all φ Minimum φ=0 Minimum φ=1	+ 0.3 - 0.6 - 1.4	+ 0.6 - 0.6 - 1.3	+ 1.8 - 1.4 - 2.0	+ 1.3 - 1.4 - 1.8	+ 0.4 - 1.1 - 1.5
+ 10	Minimum 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
+ 15	Minimum 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
+ 20	Minimum 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
+ 25	Minimum all φ Minimum φ=0 Minimum φ=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
+ 30	Minimum all φ Minimum φ=0 Minimum φ=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 (i) + down
- up

(ii) $z_{ref} = h$

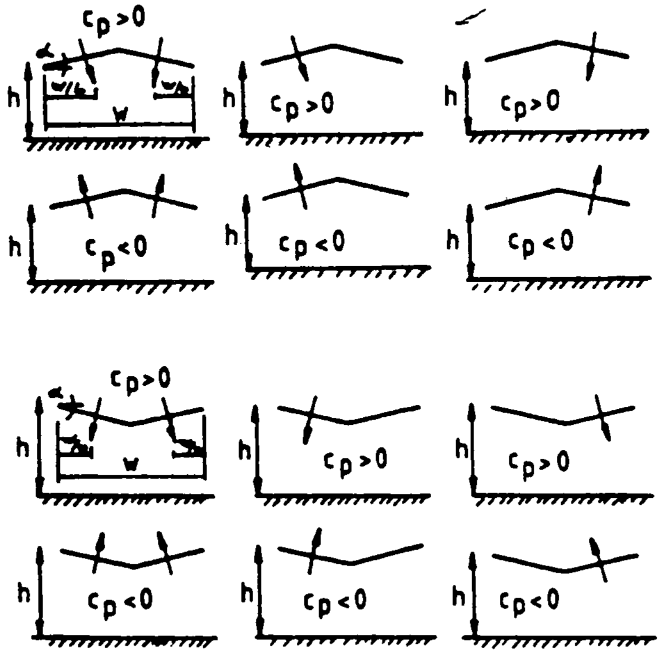


Figure A.14 Load Arrangement for Doupitch Canopies

Table A.9 $c_{p,net}$ Values for Multibay Canopies

Bay	Location	Factors for all φ	
		on maximum (downward) overall coefficient	on minimum (upward) overall coefficient
1	end bay	1.00	0.81
2	second bay	0.87	0.64
3	third and subsequent bays	0.68	0.63

A.4 Free-Standing Boundary Walls, Fences and Signboards

A.4.1 Solid Boundary Walls

(1) The wall should be divided into zones as shown in Fig. A.16

(2) Values of net pressure coefficients $c_{p,net}$ for free-standing walls and parapets, with or without return corners, are given in Table A.10 for two values of solidity. Solidity $\varphi = 1$ refers to solid walls, while
$\varphi = 0.8$, refers to walls which are 80% solid and 20% open. The reference area in both cases is the gross area.

(3) Linear interpolation for solidity ratio may be used in the range $0.8 < \varphi < 1$. For porous walls with solidity less than 0.8, coefficients should be derived as for plane lattice frames (see Section A.10).

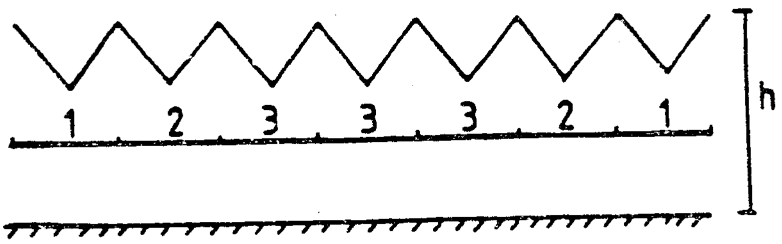


Figure A.15 Multibay Canopies

(4) The slenderness factor ψ_s (see Section A.12) may be applied.

(5) The reference height z_e should be taken as h.

Table A.10 Net Pressure Coefficients for Free-Standing Walls

Solidity	Zone	A	B	C	D
$\varphi = 1$	without return corners	3.4	2.1	1.7	1.2
	With return corners	2.1	1.8	1.4	1.2
$\varphi = 0.8$		1.2	1.2	1.2	1.2

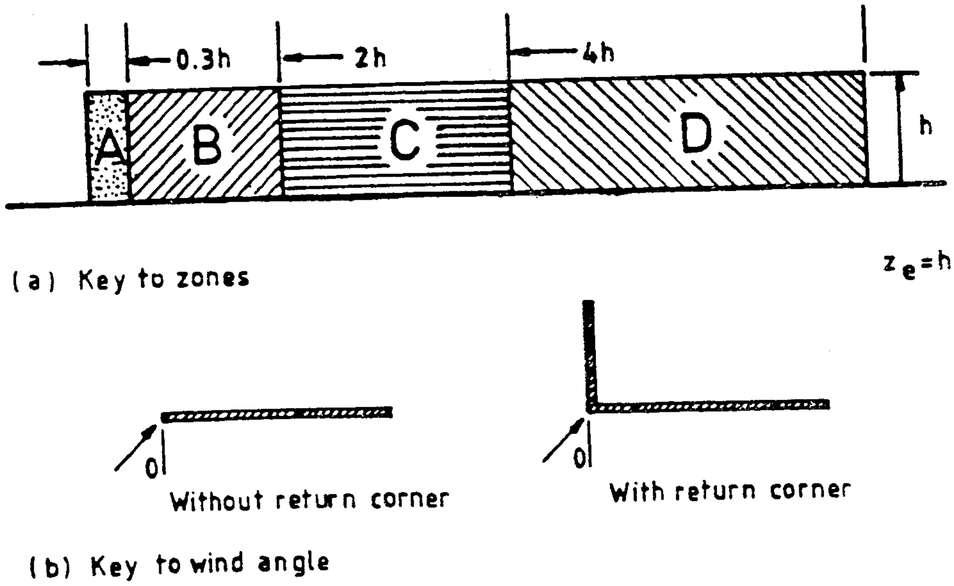


Figure A.16 Key to Boundary Walls

A.4.2 Pressure Coefficients for Porous Fences

(1) Porous fences with solidity ratio $\psi \leq 0.8$ should be treated as a plane lattice using the provisions of Section A.10.

A.4.3 Signboards

(1) The force coefficients for signboards, separated from the ground by at least $d/4$ height (see Fig. A.17), is given by:

$$c_f = 2.5\ \psi\lambda \qquad \text{(A.5)}$$

where ψ_λ slenderness reduction factor (see Section 10.12)

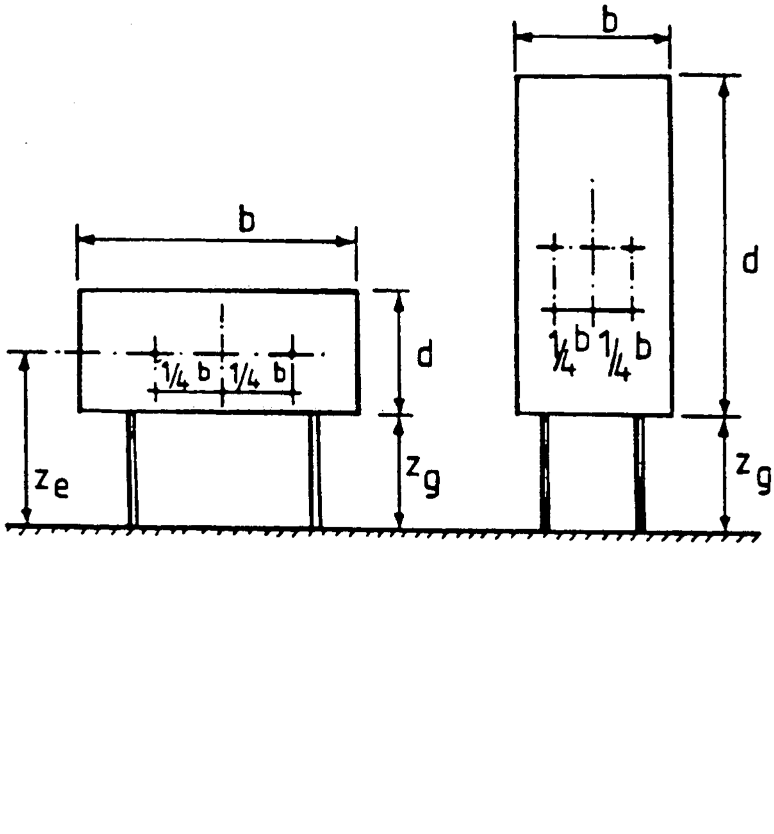


Figure A.17 Key for Signboards

Note: (i) reference : $z_e = z_g + d/2$
(ii) reference area: $A_{ref} = b\ d$
(iii) $z_g \geq d/4$ if not assumed as boundary wall

(2) The resultant force normal to the signboard should be taken to act at the height of the center of the board, with an horizontal eccentricity of:

$$e = \pm 0.25\, b \tag{A.6}$$

A.5 Structural Elements with Rectangular Sections

(1) The force coefficient c_f of structural elements of rectangular section and with wind blowing normally to a face is given by:

$$c_f = c_{f.o}\, \psi_r\, \psi_\lambda \tag{A.7}$$

where $c_{f.o}$ force coefficient of rectangular sections with sharp corners and infinite slenderness ratio λ ($\lambda = l/b$, l = length, b = width of element) as given in Fig. A.18.
ψ_r reduction factor for square sections with rounded corners, ψ_r are given in Fig. A.19.
ψ_λ reduction factor for elements with finite slenderness ratio as defined in Section 10.12.

(2) The reference area A_{ref} is:

$$A_{ref} = l\, b \tag{A.8}$$

The reference height z_e is equal to the height above ground of the section being considered.

(3) For plate-like sections ($d/b < 0.2$) lift forces at certain wind angles of attack may give rise to higher values of c_f up to an increase of 25% (for example, see Section A.4 signboards).

A.6 Structural Elements with Sharp Edged Section

(1) The force coefficient c_f of structural elements with sharp edged section (e.g. elements with cross sections such as those shown in Fig. A.20) is given by:

$$c_f = c_{f.o}\, \psi_\lambda \tag{A.9}$$

where $c_{f.o}$ force coefficient of structural elements with infinite slenderness ratio λ ($\lambda = l/b$, l = length, b = width), as defined in Fig. A.18. It is given for all sections and for both wind directions as: $c_{f.o} = 2.0$
ψ_λ slenderness reduction factor (see Section A.12)

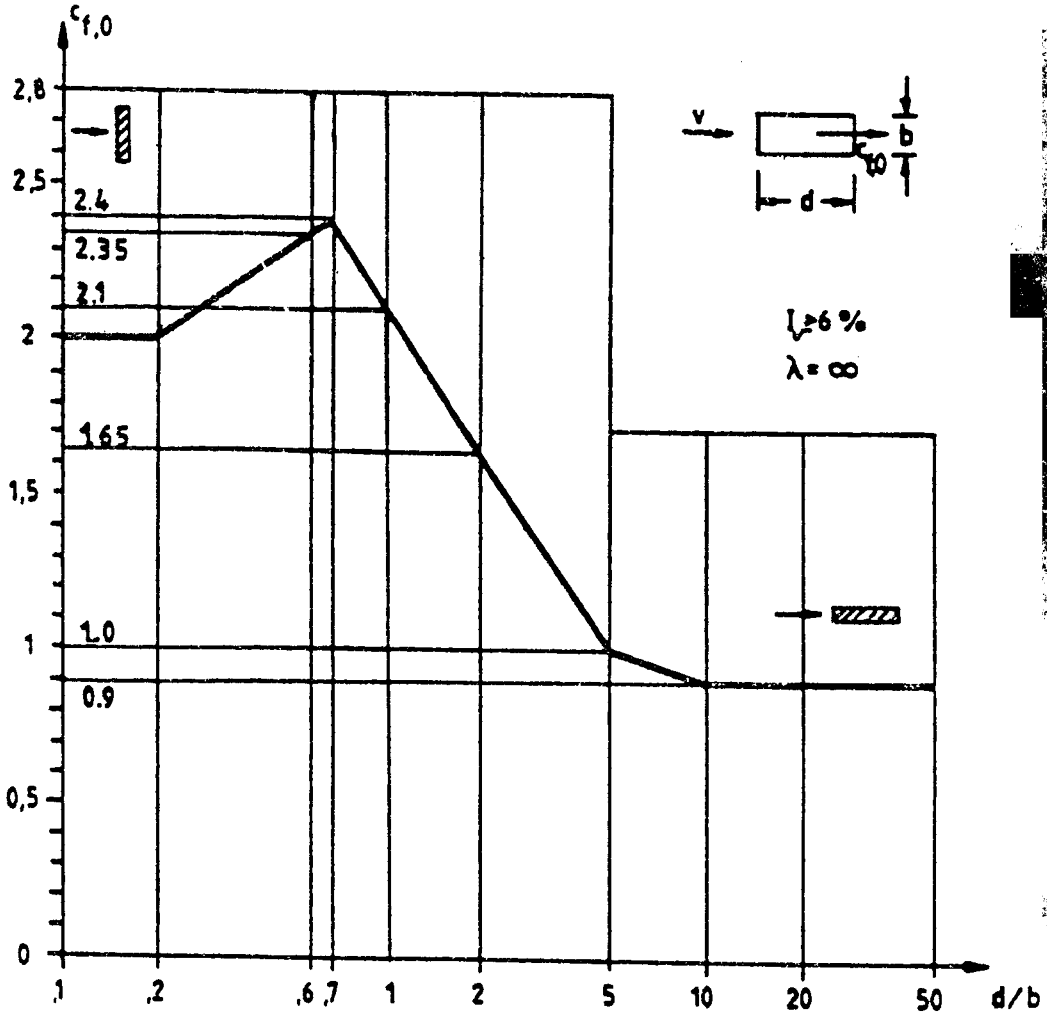


Figure A.18 Force Coefficients $C_{f,o}$ of Rectangular Section with Sharp Corners and Slenderness $\lambda = l/b = \infty$ and Trubulence Intensity of $l_v \geq 6\%$

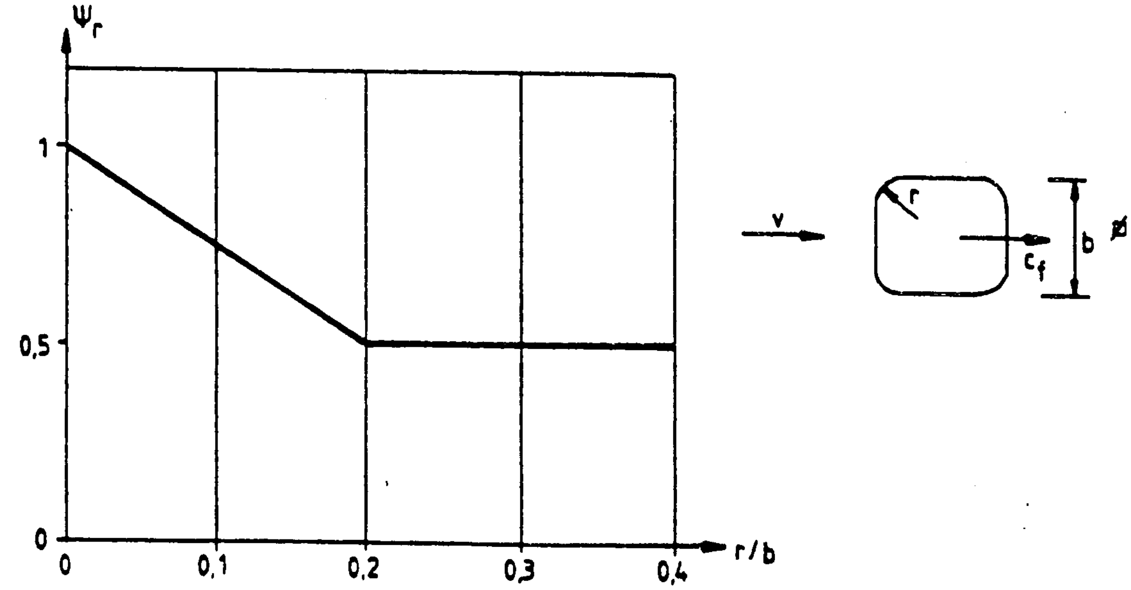
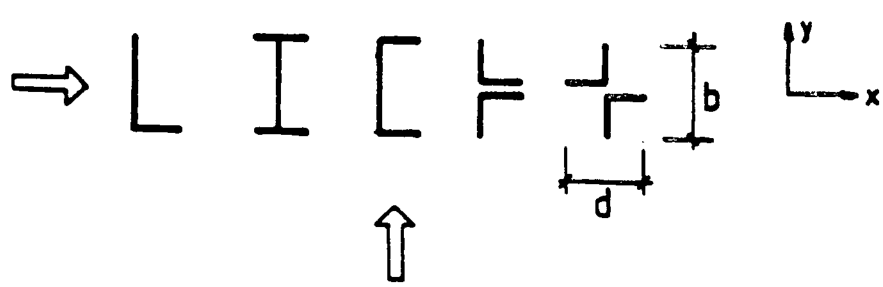


Figure A.19 Reduction Factor ψ_r for a Square Cross-Section with Rounded Corners



Note: L = length

Figure A.20 Sharp Edged Structural Section

(2) The reference areas:

In x-direction: $A_{ref.x} = lb$
In y-direction: $A_{ref.y} = lb$

(3) In all cases the reference height z_e is equal to the height above ground of the section being considered.

A.7 Structural Elements with Regular Polygonal Section

(1) The force coefficient c_f of structural elements with regular polygonal section with 5 or more sides is given by:

$$c_f = c_{f.o}\,\psi_\lambda \tag{A.10}$$

where $c_{f.o}$ force coefficient of structural elements with infinite slenderness ratio λ ($\lambda = \ell/b$, l = length, b = diameter of circumscribed circumference, see Fig. A.21)as defined in Table A.11.

ψ_λ slenderness reduction factor as defined in A.14

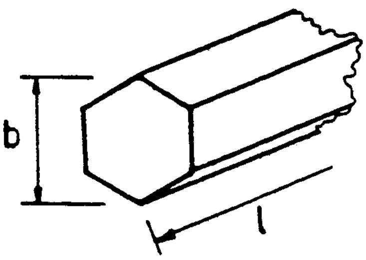


Figure A.21 Regular Polygonal Section

(2) The reference area A_{ref} is:

$$A_{ref} = lb \tag{A.11}$$

(3) The reference height z_e is equal to the height above ground of the section being considered.

Table A.11 Force Coefficient $c_{f.o1}$ for Regular Polygonal Sections

Number of sides	Sections	Finish of Surface and of Corners	Reynolds Number Re	$C_{f.o}$
5	pentagon	all	all	1.8
6	hexagon	all	all	1.6
8	octagon	surface smooth $r/b < 0.75(2)$	$Re \leq 2.4 \,.\, 10^5$ $Re \geq 3 \,.\, 10^5$	1.45 1.3
		surface smooth $r/b < 0.75(2)$	$Re \leq 2 \,.\, 10^5$ $Re \geq 7 \,.\, 10^5$	1.3 1.1
10	decagon	all	all	1.3
12	dodecagon	surface smooth (3) corners rounded	$2 \,.\, 10^5 < Re < 1.2 \,.10^6$	0.9
		all other	$Re < 2 \,.\, 10^5$ $Re \leq 4 \,.\, 10^5$	1.3 1.1
16		surface smooth (3) corners rounded	$Re < 2 \,.\, 10^5$	like circular cylinders
			$2 \,.\, 10^5 \leq Re < 1.2 \,.10^6$	0.7
18		surface smooth (3) corners rounded	$Re < 2 \,.\, 10^5$	like circular cylinders
			$2 \,.\, 10^5 \leq Re < 1.2 \,.10^5$	0.7

Note: (1) Reynold number, Re, is defined in Section A.8
(2) r = corner radius, b = diameter
(3) from tests in wind tunnel with galvanised steel surface and a section with b = 0.3m and corner radius of 0.06 b

A.8 CIRCULAR CYLINDERS

A.8.1 External Pressure Coefficients

(1) Pressure coefficients of circular sections depends upon the Reynolds numbers Re defined as:

$$Re = \frac{b v_m(z_e)}{v} \qquad \text{(A.12)}$$

where b diameter
v kinematic viscocity of the air ($v = 15.10^{-6}$ m²/S)
$v_m(z_e)$ mean wind velocity as defined in Section 3.8.1

(2) The external pressure coefficients c_{pe} of circular cylinders is given by:

$$c_{pe} = c_{p,o}\, \psi \lambda \alpha \qquad \text{(A.13)}$$

where $c_{p,o}$ external pressure coefficient for infinite slenderness ratio λ (see (3) below)

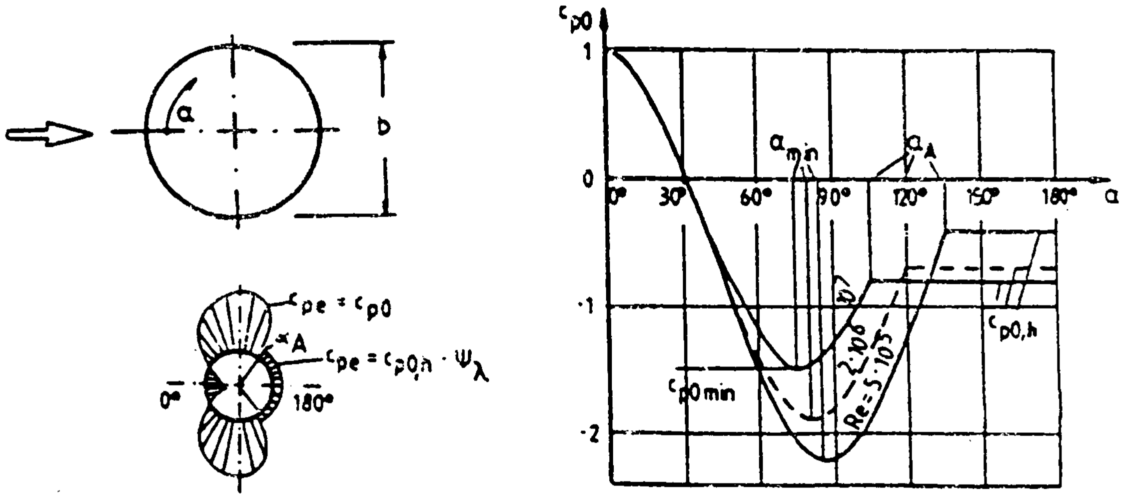
$\psi_{\lambda\alpha}$ slenderness reduction factor (see (4) below)

(3) The external pressure coefficient $C_{p,o}$ is given in Fig.A.22 for various Reynolds numbers as a function of angle α.

(4) The slenderness reduction factor $\psi_{\lambda\alpha}$ is given by:

$$\begin{array}{llll} \psi_{\lambda\alpha} & = 1 & \text{for} & 0^\circ \leq \alpha \leq \alpha_A \\ & & & 360^\circ - \alpha_A \leq \alpha \leq 360^\circ \\ \psi_{\lambda\alpha} & = \psi_\lambda & \text{for} & \alpha_A \leq \alpha \leq 360^\circ \ \alpha_A \end{array} \qquad \text{(A.14)}$$

where α_A position of the flow separation (see Fig. A.23)
ψ_λ slenderness reduction factor (see Section A.2)



Note: (i) Intermediate values may be interpolated linearly
(ii) typical values in the above Fig. are shown in the Table below

Re	α_{min}	$c_{po,min}$	α_A	$c_{po,h}$
5 x 10^5	85	-2.2	135	-0.4
2 x 10^6	80	-1.9	120	-0.7
107	75	-1.5	105	-0.8

where α_{min} position of the minimum pressure
$c_{po,min}$ value of the minimum pressure coefficient
α_A position of the flow separation
$c_{po,h}$ base pressure coefficient

(iii) The above Fig.is based on an equivalent roughness *K/B* less than 5.10^{-4}. Typical values of roughness height *k* are given in Table A.12.

Figure A.22 **Pressure Distribution for Circular Cylinders for Different Reynolds Number Ranges and Infinite Slenderness Ratio**

(5) The reference area A_{ref} is:

$$A_{ref} = lb \tag{A.15}$$

(6) The reference height z_e is equal to the height above ground of the section being considered

A.8.2 Force Coefficients

(1) The force coefficient c_f, for a finite circular cylinder is given by:

$$c_f = c_{f,o}\ \psi_\lambda \tag{A.16}$$

where $c_{f,o}$ force coefficient of cylinder with infinite slenderness (see Fig. A.23)
c_λ slenderness reduction factor (see A.12).

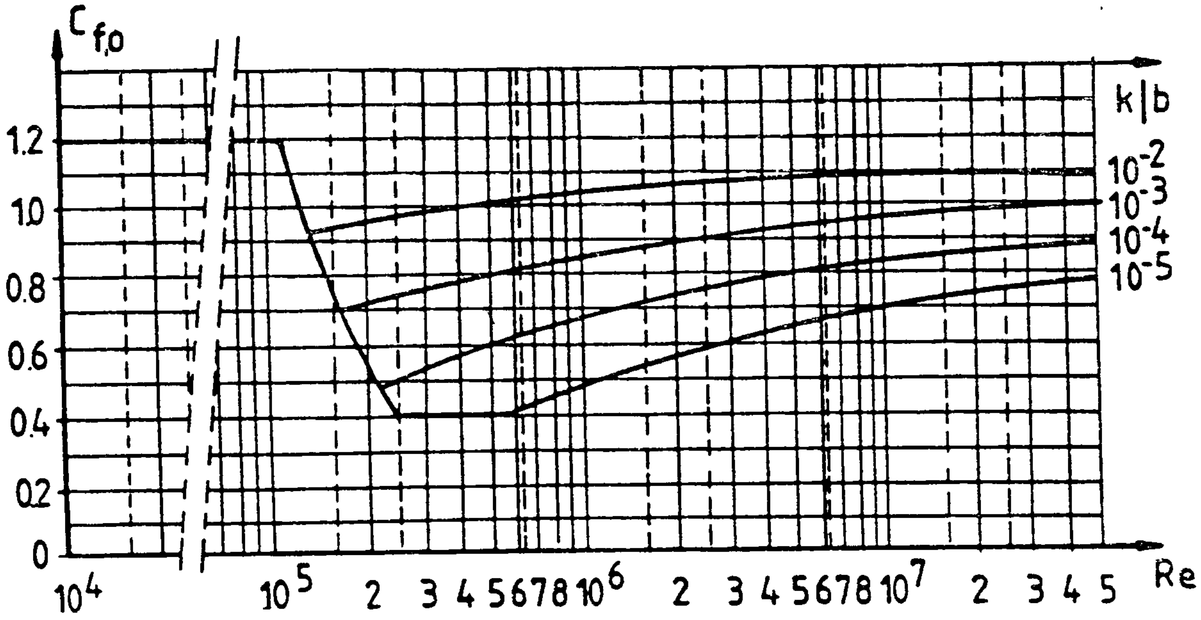


Figure A.23 Force Coefficient $C_{1,}$ *0* for Circular Cylinders with Infinite Slenderness Ratio and for Different Equivalent Roughness *K/B*

(2) Values of equivalent surface roughness k are given in Table A.12

(3) For stranded cables $c_{l,o}$ is equal to 1.2 for all values of the Reynolds number R_e.

Table A.12 Equivalent Surface Roughness k

Type of surface	Equivalent roughness k(mm)	Equivalent roughness k(mm)	Equivalent roughness k(mm)
glass	0.0015	galvanised steel	0.2
polished metal	0.002	smooth concrete	0.2
finè paint	0.006	rough concrete	1.0
spray paint	0.02	rust	2.0
bright steel	0.05	brickwork	3.0
cast iron	0.2		

(3) The reference area A_{ef} is:

$$A_{ref} = lb \qquad \text{(A.17)}$$

(4) The reference height z_e is equal to the height above ground of the section being considered.

(5) For cylinders near a plane surface with a distance ration $z_g/b < 1.5$ (see Fig. A.24) special advice is necessary.

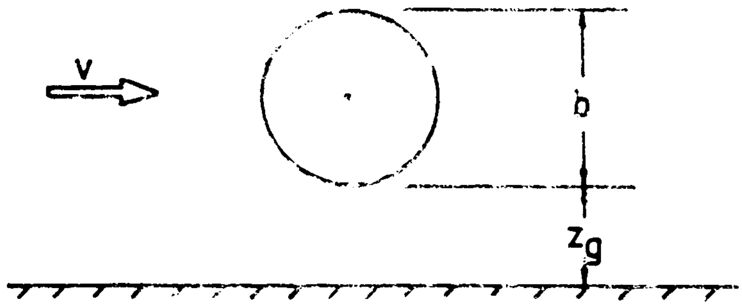


Figure A.24 Cylinder near a Plane Surface

A.9 SPHERES

(1) The alongwind force coefficient $c_{f,x}$ of spheres is given in Fig. A.25 as a function of the Reynolds number Re (see A.8.1) and the equivalent roughness K/b (see Table A.12)

(2) The values in Fig. A.25 are limited to values $z_g > b/2$, where z_g is the distance of sphere from a plain surface, b is the diameter, Fig. A.26 For $z_g < b/2$ the force coefficient $C_{f,x}$ shall be multiplied by a factor 1.6.

) The vertical force coefficient $c_{f,z}$ of spheres shall be assumed to be:

$$c_{f,z} = 0 \quad \text{for } z_g > b/2 \tag{A.18}$$

$$c_{f,z} = +0.6 \quad \text{for } z_g > b/2$$

) In both cases the reference area A_{ref} is:

$$A_{ref} = \frac{\pi b^2}{4} \tag{A.19}$$

5) The reference height should be taken as: (A.20)

$$z_e = z_g + b/2$$

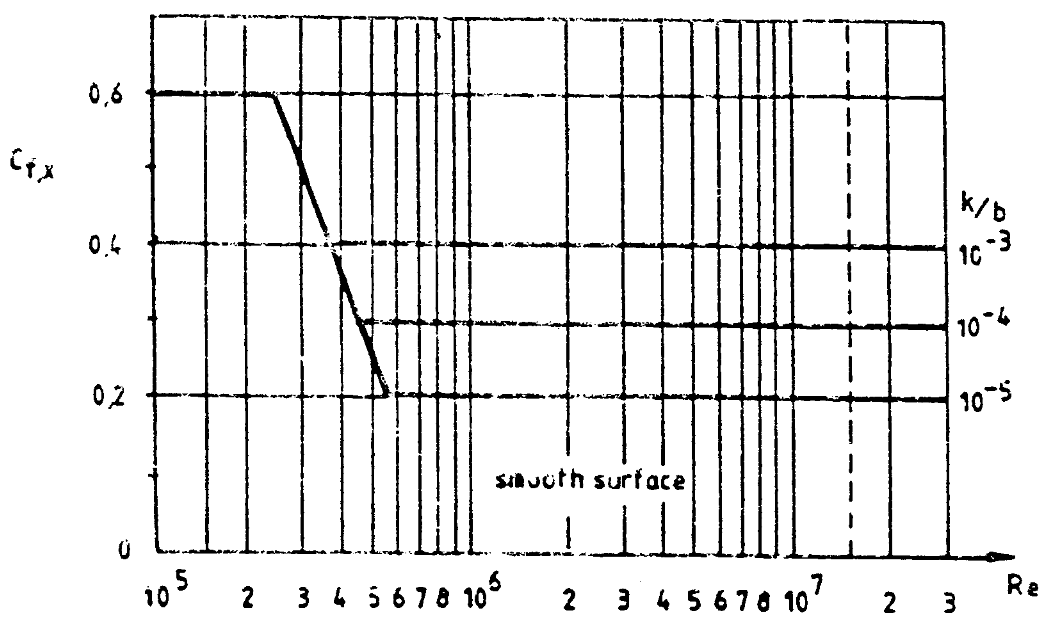


Figure A.25 Alongwind Force Coefficient of a Sphere

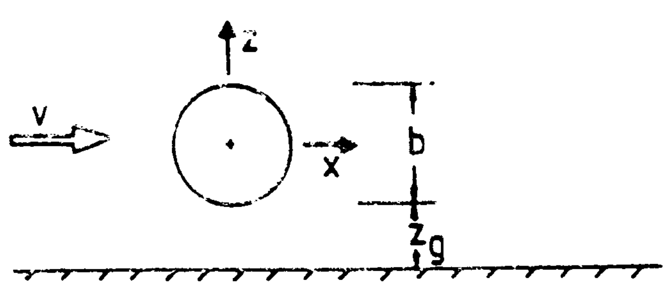


Figure A.26 Sphere near a Plain Surface

A.10 Lattice Structures and Scaffoldings

(1) The force coefficient c_f, of lattice structures and scaffoldings is given by:

$$c_f = c_{f,o}\ \psi_\lambda\ \psi_{sc} \tag{A.21}$$

where $c_{f,o}$ force coefficient of lattice structures and scaffoldings with infinite slenderness λ ($\lambda = lb$, l =length, b = width, Fig. A.27). It is given by Figs. A.28 to A.30 as a function of solidity ratio φ (2) and Reynolds number Re

re Reynolds number given by Eq. A.12 and calculated using the member diameter b_i

ψ_λ slenderness reduction factor (see A.12)

ψ_{sc} reduction factor for scaffolding without-air tightness devices and affected by solid building faces (see Fig. A.31) plotted as a function of the obstruction factor Φ_B.

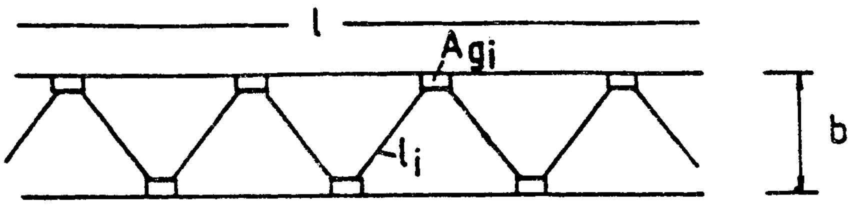


Figure A.27 Lattice Structures or Scaffolding

(2) The obstruction factor is given by:

$$\phi_B = \frac{A_{B,n}}{A_{B,g}} \tag{A.22}$$

where $A_{b,n}$ net area of the face

$A_{B,g}$ gross area of the face

(3) Solidity ratio, is defined by:

$$\varphi = A/A_c \tag{A.23}$$

where A Sums of the projected area of the members and gusset plates of the face $= \Sigma_j b_i l_i + \Sigma A_{gi}$

A_c the area enclosed by the boundaries of the face projected normal to the face $= b$

l length of the lattice

b width of the lattice

$b_i l_i$ width and length of the individual member i

A_{gi} area of the gusset plate i

(4) The reference area A_{ref} is defined by:

$$A_{ref} = A \tag{A.24}$$

(5) The reference height z_e is equal to the height of the element above ground.

A.11 Friction Coefficients c_{fr}

(1) Friction coefficients c_{fr}, for long walls and roof surfaces are given in Table A.13

(2) The reference areas swept by the wind A_{ref} are given Fig. A.32

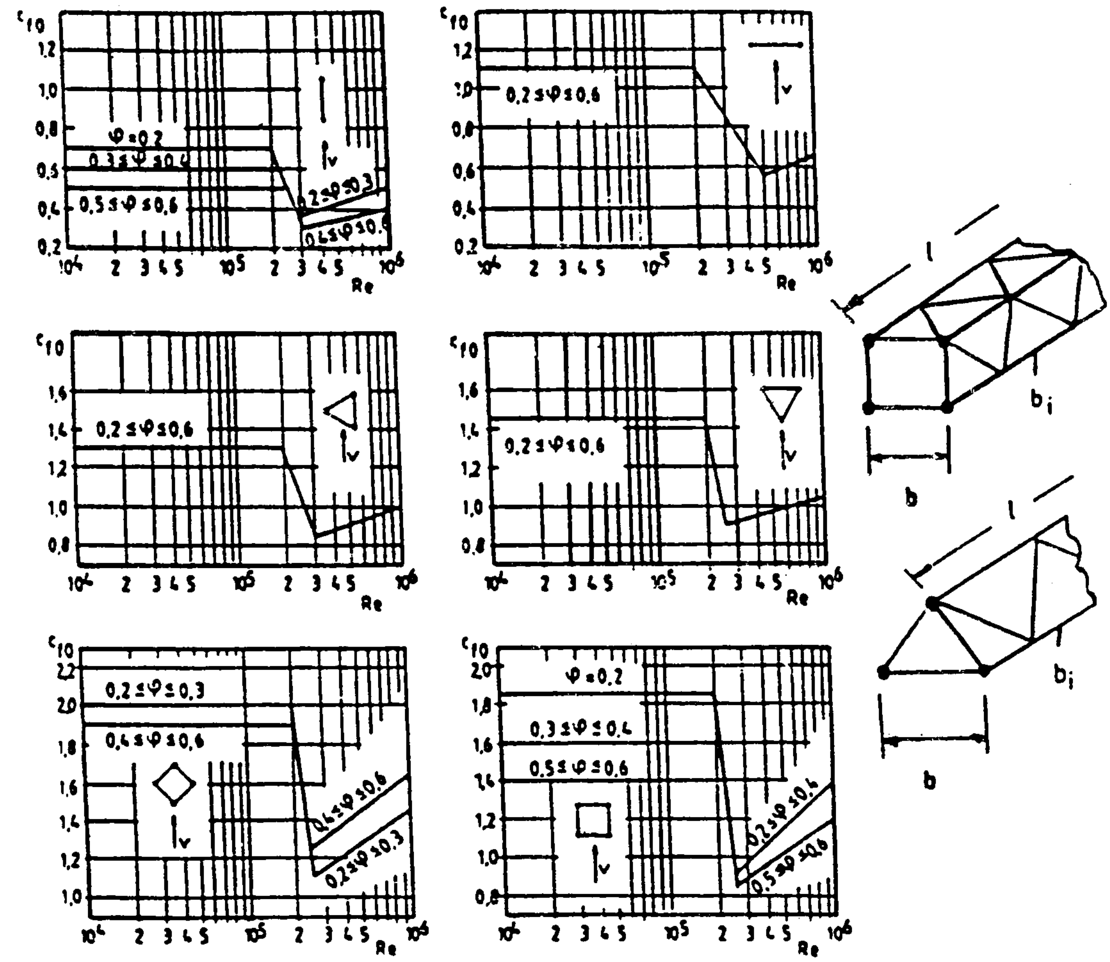
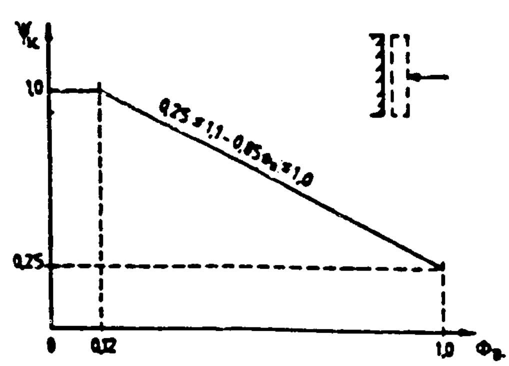


Figure A.30 Force Coefficient $c_{f,0}$ for Plane and Spatial Lattice Structure with Members of Circular Cross-Section



	Ψ_{sc}
with protecting walls	0.03
with tilts	0.1
with nets	0.2

Figure A.31 Reduction Factors for the Force Coefficients of Scaffoldings without Air-Tightness Devices, Affected by Solid Building-Face Versus Obstruction Factor Φ_B

(3) The reference height z_e should be taken into account according to Fig. A.32

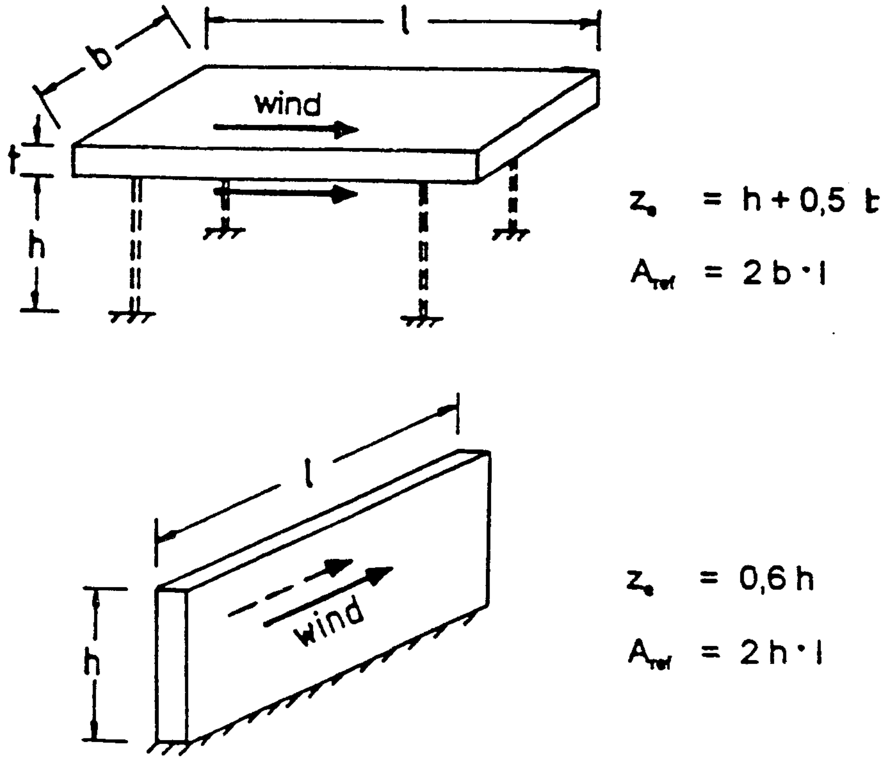


Figure A.32 Key to Reference Area A_{ref} for Walls and Roof Surfaces

Table A.13 Frictional Coefficients c_{fr} for Walls and Roof Surfaces

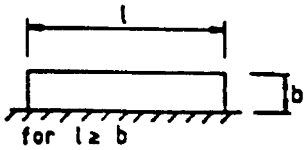
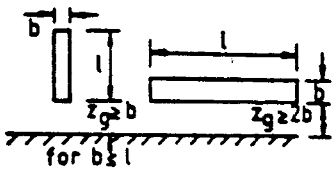
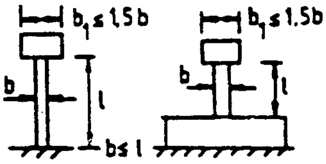
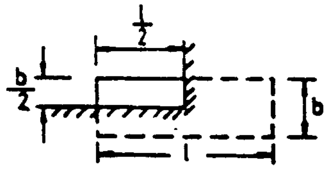
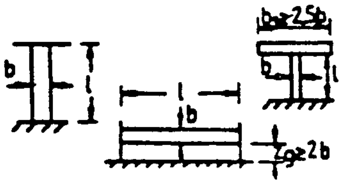
Surface	Friction coefficient C_{fr}
smooth (i.e. steel, smooth concrete	0.01
rough (i.e rough concrete, tar boards)	0.02
very rough (i.e. ripples, ribs, folds)	0.04

A.12 EFFECTIVE SLENDERNESS λ AND SLENDERNESS REDUCTION FACTOR ψ_λ

(1) The effective slenderness λ is defined in Table A.14

(2) The slenderness reduction factor ψ_λ, versus the effective slenderness λ and for different solidity ratios φ is given in Fig. A.33.

Table A.14 Effective Slenderness λ for Cylinders, Polygonal Sections, Rectangular Sections, Sign Boards, Sharp Edged Structural Sections and Lattice Structures

No	Position of the structure, wind normal to the plane of the page	Effective slenderness λ
1		l/b
2		
3		$l/b \leq 70$
4		
5		$l/b \geq 70$

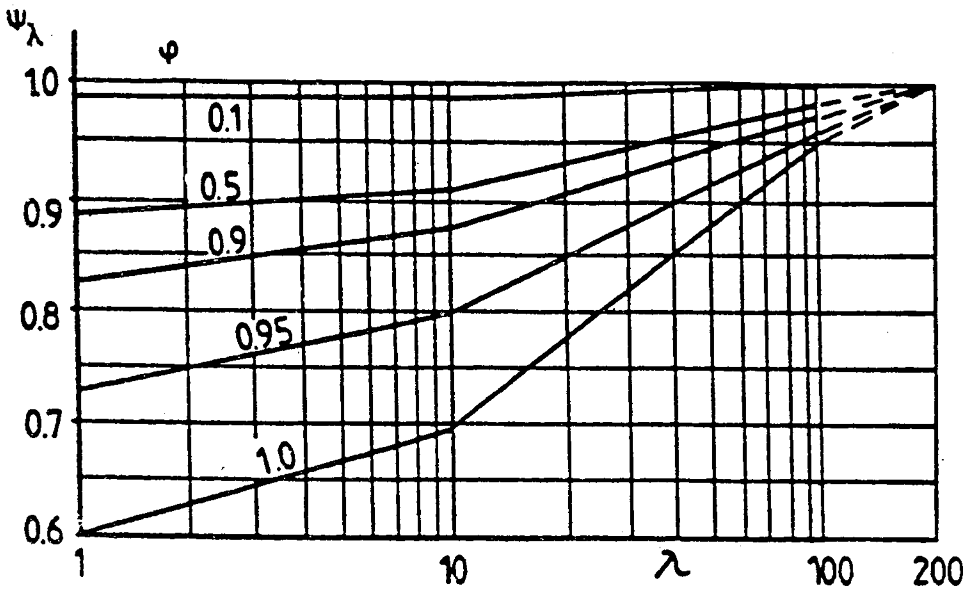


Figure A.33 Slenderness Reduction Factory $\psi\lambda$ as a Function of Solidity Ratio φ Versus Slenderness λ

(3) The solidity ratio φ is given by (see Fig. A.34):

$$\varphi = A/A_c \tag{A.24}$$

where A sum of the projected areas of the members
A_c enclosed area $A_c = lb$

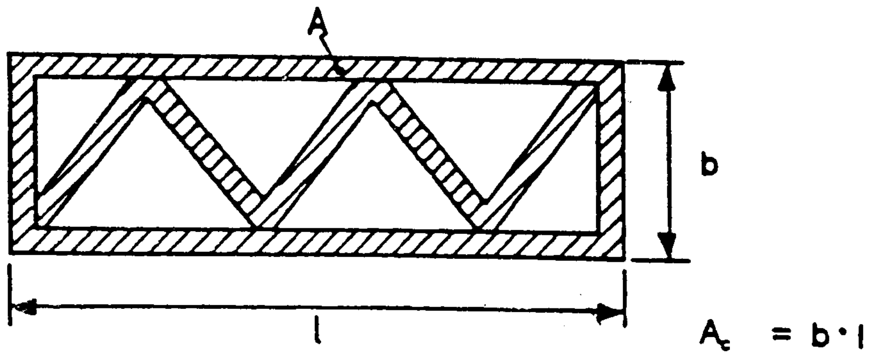


Figure A.34 Definition of Solidity Ratio φ

INDEX

Accidental 24
Accidental action 4'
Accidental actions 14, 15, 24
Accidental situation 26
Accidental situations 10, 24
Action 4
Action effect 4
Action on structures 31
Actions 13, 21, 24
Administration Area 44
Aerodynamic coefficients 69
Aeroelastic Instability 67
Aggregates 35
Agricultural 36
Air density 55
Angles of repose 33
Annual Probabilities of Exceedence 56
Assumptions 1

Barriers 49
Basis of design 1
Beverages 39
Brick lined steel chimney 66
Building materials 32
Buildings 69, 70

Canopy roofs 69, 84
Categories of building areas 45
Cement 35
Characteristic 28
Characteristic value 5, 23, 42
Characteristic value of a geometrical property 5
Characteristic value of an action 5
Characteristic values 24, 42, 44
Characteristic values of actions 14
Circular cylinders 69, 95, 96
Civil engineering 2
Civil engineering works 2
Claddings 43
Classification of actions 31, 51
Cliffs 59
Combination of actions 5, 23, 24, 28
Combination value 16
Combination values 5
Commercial 44
Composite (steel/concrete) buildings 64
Concrete 33
Concrete and masonry buildings 63
Construction material 2
Construction materials 31, 33, 34
Construction Works 2
Control tests 19
Cylinders 103

Definitions 2
Densities 31, 33, 42
Derivation of the design 19
Design assisted by testing 18
Design criteria 3
Design models 17
Design resistance 22
Design situation 24
Design situations 3, 10, 27, 51
Design value 23
Design value of a geometrical property 6
Design value of a material property 5
Design value of an action 5
Design values 19, 21, 22, 24, 28
Design values of actions 21
Design working life 3, 11
Destabilizing actions 23
Dimensions 42
Direct action 13
Divergence 52, 67
Domes 80
Dominant action 24
Duopitch Canopies 87
Duopitch canopy 84
Duopitch Roofs 75, 77
Durability 11
Dynamic Interference Effects 67
Dynamic action 4
Dynamic actions 14, 18
Dynamic Coefficient 61
Dynamic effects 52

Effective slenderness 69, 103
Effects of Actions 21
Effects of displacements 17
Environmental influences 11, 13, 16
Equivalent Surface Roughness 98
Escarpments 59
Execution 2
Explosion 3, 9
Exposure Coefficient 60
Exposure Cofficient 62
External Pressure 53
External Pressure Coefficient 69
External pressure coefficients 69-71, 73, 75, 77, 79, 95

Factors 59
Failure in the ground 26
Failure of structure 26
Farmyard 36
Fatigue 16, 18, 20
Fences 69, 88
Fertiliser 36
Finishes 43
Fire 9
Fixed action 4
Fixed actions 14, 31
Flat roof 49
Flat roofs 71, 72
Flooring 41
Floors 43

Fluctuating forces 51
Fluctuating pressures 51
Flutter 52, 67
Foodstuffs 38
Force Coefficients 97
Form of structure 2
Free action 4
Free actions 14
Free-standing boundary walls 69
Free-Standing Walls 89
Frequency of vibration 52
Frequent 28
Frequent value 16
Frequent value of a variable action 5
Friction coefficients 69, 100
Friction Force 54
Friction forces 84
Frictional Coefficients 102
Fundamental Requirements 9

Galloping 52, 67
Garage 46
Garages 47
Geometrical data 17, 21, 22
Global force 54
Grain 36
Gust Wind Response 61

Hazard 3
Hills 60
Hipped Roofs 78, 79
Horizontal Loads 49
Horizontal Members 44
Hydrocarbons 39

Impact 3, 9
Imposed loads 27, 31, 32, 48
IMPOSED LOADS ON BUILDINGS 43
Imposed Loads on Floors 46
Indirect action 13
Industrial Activities 48
Industrial areas 48
Intended probability 15
Interference 52
Interference galloping 67
Internal Pressure 53, 82
Irreversible serviceability limit states 3

Lattice structures 69, 99, 103
Limit State Design 13
Limit states 3, 12
Limitations 20
Lined Steel Chimneys 65
Liquids 39
Load arrangement 3
Load Arrangements 42, 44
Load case 3
Local failure 3
Loss of static equilibrium 26

Maintenance 4
Masonry units 33
MATERIAL PROPERTIES 16, 22
Materials 29
Mean Wind Velocity 57
Metals 33
Method of construction 2
Modelling 17
Modelling of Wind Actions 52
Monopitch Canopies 85
Monopitch canopy 84
Monopitch Roofs 73-75
Monumental building 11
Mortar 33
Multibay Canopies 88
Multibay duopitch canopy 84
Multispan Roofs 78

Net Pressure 53
Net pressure coefficients 84
Non-linear analysis 21

Parking areas 47
Partial Safety Factors for Materials 29
Partial safety factor 19, 21-23
PARTIAL SAFETY FACTOR METHOD 20
Partial Safety Factors 25, 26, 29
Partial Safety Factors for Materials 27
Partition Walls 49
Partitions 43
Performance criteria 11
Permanent action 4, 14, 32
Permanent actions 14, 24, 28
Persistent and transient 24
Persistent and transient situations 23
Persistent design situation 3
Persistent situation 26
Persistent situations 10
Plastics 35
Polygonal Sections 103
Porous Fences 90
Potential damage 9
Pressure on Surfaces 53
Pressures 53
Prestressed structures 22
Prestressing 14
Properties of materials 16

Quality assurance 10, 12
Quasi-permanent 16, 24, 28
Quasi-permanent value of a variable action 5
Quasi-static action 5

Rectangular sections 103
Reduction coefficient 49

Reduction factor 46
Reference period 5, 15
Reference wind pressure 55
Reference wind velocity 55, 57
Regular polygonal sections 95
Reinforced concrete chimney 66
Reliability 4, 9
Reliability differentiation 9
Representation of actions 41, 43, 51
Representative value of an action 5
Representative value of the action 22
Representative values of variable 15
Requirements 9
Residential 44
Resistance 3
Resonant response 52
Reversible serviceability limit states 3
Ridges 60
Robustness 10
Roofs 43, 48
Roughness coefficient 57, 58

Scaffoldings 69, 99
Seismic 24
Seismic action 4
Seismic actions 15, 24
Seismic situations 10, 24
Self-weight 31, 32, 42
Self-Weight of construction elements 41
Serviceability 9, 10
Serviceability limit states 3, 12, 13, 28, 29
Sharp edged structural sections 103
Shopping areas 45
Sign boards 103
Signboards 69, 88, 90
Simplifications 20
Simplified verification 20, 27, 29
Single action 4
Single variable actions 24
Slenderness reduction factor 69
Sloping roof 49
Social 44
Solid boundary walls 88
Solid fuels 40
Spatial variation 14
Spheres 69, 98
Stabilizing actions 23
Static action 4
Static actions 14, 17
Static equilibrium 23
Steel Buildings 64
Storage areas 48
Stored materials 31, 32, 35, 36, 40
Strength 4, 23
Structural elements 69
Structural elements with rectangular sections 91
Structural elements with regular polygonal section 94
Structural Elements with Sharp Edged Section 91
Structural model 2
Structural safety 9
Structural system 2
Structure 2
Symbols 6

Temperature 27
Temporary structure 11
Temporary structures 55
Terrain Categories 57
Tolerances 17
Topography coefficient 57, 58
Torsional effects 54
Traffic loads 27
Transient design situation 3
Transient situation 26
Transient situations 10
Turbulent 51
Type of building 2
Type of construction 2
Types of Tests 18

Ultimate limit states 3, 12, 16, 23, 26
Unfavourable deviations 22

Values of Actions 47, 49
Variable action 4
Variable actions 14, 15, 32
Vaulted Roofs 80
Vegetables 38
Vehicle traffic area 47
Vehicle traffic areas 46
Verification 20, 21
Verification of serviceability 28
Verifications 23
Vertical members 44
Vertical walls 70, 71
Vortex shedding 52, 67
Vortex shedding and galloping 67

Walkways 49
Walling 41
Walls 43, 88
Water ponding 49
Welded steel chimneys 65
Wind actions 51
Wind forces 53
Wind loads 27, 51
Wind pressure on surfaces 52
Wood 34
Workmanship 11
Ψ Factors 27, 29